

November 7, 2018
Project No. 1012-325-19-01

PUBLIC BUILDING COMMISSION OF CHICAGO

GEOTECHNICAL EXPLORATION McDADE ELEMENTARY CLASSICAL SCHOOL

**8801 S. INDIANA AVENUE
CHICAGO, ILLINOIS**

**Prepared For:
Public Building Commission of Chicago
50 West Washington Street
Room 200
Chicago, IL 60602**

PREPARED BY





November 7, 2018
Project No. 1012-325-19-01

Attn: Mike Powell
Public Building Commission of Chicago
50 West Washington Street, Room 200
Chicago, IL 60602

**Re: Geotechnical Exploration
McDade Classical Elementary School
8801 S. Indiana Avenue
Chicago, Illinois**

Dear Mr. Powell:

In compliance with your request, **Weaver Consultants Group (WCG)** has completed the geotechnical exploration at the site of the above-referenced project. Our work was completed in general accordance with the scope of services detailed in the notice-to-proceed letter dated October 12, 2018. The purpose of this study was to explore the stratification and engineering properties of the subsurface soils and to provide recommendations for foundations of the proposed building and site improvements.

In the body of this report, we present a summary of our findings, an interpretation of the subsurface conditions, our design recommendations, and construction considerations. The property location map, boring location plan, and soil profiles are presented as figures. The soil boring logs and methods for field and laboratory operations are presented in **Appendix A**. Select calculations are provided in the **Appendix B**. General Qualifications and Contractual Considerations are presented in **Appendix C**.

Thank you for selecting our firm to assist with this phase of the project. Please call us if there are any questions concerning this report.

Sincerely,

Weaver Consultants Group

A handwritten signature in black ink, appearing to read 'Steve Schubert'.

Steve Schubert, PE
Geotechnical Engineering Manager

A handwritten signature in black ink, appearing to read 'John J. Talbot'.

John Talbot, PE
Project Director

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Figure 2: Boring Location Plan

Figure 3: Soil Cross-Section A-A'

Figure 4: Soil Cross-Section B-B'

Boring Logs

Boring Logs: B-1 through B-5

APPENDICES**Appendix A - Field Exploration**

Log of Soil Boring General Notes

Unified Soil Classification System

Field Exploration Procedures

Boring Log Preparation & Laboratory Testing Procedures

Appendix B – Laboratory Test Results**Appendix C - Calculations****Appendix D - Qualifications**

General Qualifications and Contractual Considerations

1 EXECUTIVE SUMMARY

The Public Building Commission of Chicago is proposing to construct a single-story annex onto the eastern portion of James E. McDade Elementary Classical School in Chicago, Illinois (see **Figure 1**). For the design and construction of the annex and associated improvements, a geotechnical study was performed consisting of five (5) soil borings in the project area.

In summary, the soil borings performed for the proposed design and construction of the development indicate that the site soils generally consist of surficial fill, underlain by medium stiff to stiff clay, underlain by very stiff to hard hardpan to the terminal depths of the borings. In our opinion, the proposed building should be supported on a deep foundation system extending into the hardpan layer. We recommend the deep foundation system be designed for a maximum allowable end bearing pressure of 10,000 psf bearing on native soils at least 20 feet below surface.

To model stress-deformation characteristics of the subgrade under floor slabs, a subgrade modulus of 150 pounds per cubic inch is recommended, given the subgrade is prepared to the recommendations included in this report. We also recommend using Site Class C for seismic design at this site.

For the light-duty pavements anticipated at the site, we recommend 4 inches of asphalt over at least 6 inches of base course.

Infiltration tests were performed near the improved playlot areas. Based on the results of the tests, we recommend that the designer assume no infiltration will occur in these areas.

A detailed discussion of design parameters and construction considerations is included in subsequent sections of this report.

2 PROJECT INFORMATION

2.1 Project Description and Location

The Public Building Commission of Chicago (PBC) is proposing to improve the property at James E. McDade Elementary Classical School (McDade), located at 8801 S. Indiana Avenue in Chicago, Illinois. The site improvements will primarily consist of a 9,000 ft² annex, which will include a gymnasium, classrooms, and storage spaces. Minor pavement and storm water improvements will be made in conjunction with construction activities.

A geotechnical exploration program, consisting of soil borings, was performed at the site to facilitate the design and construction of the proposed development. WCG and PBC agreed upon the boring locations and depths prior to commencing the field activities. A total of five (5) soil borings were drilled for the project, each of which were in the proximity of the annex. The borings extended to depths of 30 feet below ground surface (bgs).

The location of each soil boring is presented in **Figure 2**.

2.2 Site Description

The proposed annex area is generally flat and primarily covered with asphalt. The eastern edge of the proposed annex area includes some landscaped grassy cover. Some underground utilities are present in the area and will need to be relocated prior to constructing the annex.

3 FIELD EXPLORATION

Field exploration activities were performed at the site on October 23 and 24, 2018. All borings were advanced with a truck-mounted CME-75 drill rig equipped with 4-inch outside diameter (O.D.) solid stem augers, mud rotary tooling, and an automatic Standard Penetration Test (SPT) hammer. SPT samples were collected at 2.5-foot intervals to a depth of 15 feet bgs and at 5-foot intervals thereafter. A representative sample from each SPT was retained in jars and sent to the WCG geotechnical laboratory for further evaluation. Select undisturbed cohesive samples were obtained by pushing Shelby tubes in accordance with ASTM D1587.

A WCG geotechnical engineer provided oversight for all field exploration activities, logged soil samples, performed field tests, and retained representative samples prior to sending to the WCG geotechnical laboratory. Pocket penetrometer tests were performed in the field to estimate unconfined compressive strength of cohesive samples.

Selected samples from the borings were tested in the WCG geotechnical laboratory to verify field soil classifications and to determine pertinent engineering properties. Moisture content determinations (ASTM D2216), grain size determinations (ASTM D422), Atterberg Limits classifications tests (ASTM D4318), and unconfined compressive strength tests (ASTM D2166) were performed on select samples in accordance with current ASTM test methods.

Two infiltration tests were performed at the site by using a single-ring infiltrometer in accordance with the guidelines in the Chicago Stormwater Ordinance Manual. Surficial materials were augered to a depth of about 2 feet below surface and the infiltrometer ring was set in the borehole. A seal was created around ring and the interior of the ring was filled with water. The water level decline was measured against time. The measurements were used to calculate the design infiltration rate.

Further information on the field exploration activities and laboratory testing is provided in the **Appendix A**.

4 SITE AND SUBSURFACE CONDITIONS

Our interpretation of the subsurface conditions is based on five (5) soil borings, spaced across the proposed annex area. The following discussion is general; for more specific information, refer to the boring logs presented in **Appendix A**.

4.1 Surface Conditions

Borings B-1, B-3, B-4, and B-5 were located in the asphalt area immediately east of McDade and encountered 2 to 4.5 inches of asphalt underlain by 2-3 inches of aggregate base course. Boring B-2 was located in a grassy area and encountered approximately 6 inches of topsoil at the surface.

4.2 Subsurface Conditions

In general, below the surficial topsoil and pavement sections, the subsurface soil profile consists of a thin layer of clayey fill, underlain by native granular soil, underlain by native medium stiff to stiff clayey soil, underlain by very stiff to stiff hardpan. These subsurface soil layers in the borings are described in more detail below.

- **Fill Material** – Fill material was encountered in each boring beneath the surficial material in each boring. The fill was a mixture of clay, gravel, and sand, occasionally with a significant amount of organic matter. The thickness of the fill layer ranged from 1 to 4 feet. Clayey portion of the fill is typically described as stiff to very stiff based on estimated unconfined compressive strength (Q_p) values between 1.25 and 3.0 tons per square foot (tsf). The relative density of the granular fill is described as loose to medium dense based on Standard Penetration Test (N) values between 8 and 10 blows per foot (bpf).
- **Sand with Silt** – Native sand was encountered in each boring beneath the fill. The sandy layer had varying amounts of silt as classified as either sand with silt (SP-SM) or silty sand (SM) in accordance with the Unified Soil Classification System (USCS). The thickness of this layer ranged from about 1 to 2.5 feet. The relative density of the sand was described as loose to medium dense based on N-values ranging from 5 to 10 bpf.
- **Medium Stiff to Stiff Clay** – A clay layer was encountered beneath the native sand with silt layer. This native clay layer extended to depths of about 13 to 16.5 feet bgs. The clay classified as lean clay (CL) in accordance with the USCS. The consistency of the native clay layer was described as medium stiff to stiff based on estimated unconfined compressive strength values between 0.5 and 2.0 tsf.

- **Very Stiff to Hard Hardpan** – A silty and clayey hardpan layer was encountered beneath the medium stiff to stiff clays. This hardpan layer extended beyond the termination depths of each boring. The hardpan soil classified as either lean clay (CL), silt (ML), or silty clay (CL-ML) in accordance with the USCS. The consistency of soils was described as very stiff to hard based on field-estimated unconfined compressive strength (Qp) values from 3.25 tsf to over 4.5 tsf.

4.3 Groundwater Conditions

Groundwater level observations noted during drilling and after completion of the drilling operations are recorded on the boring logs. Groundwater was generally observed at depths between 4 and 6 feet bgs. Due to the low permeability nature of much of the soil profile, long term groundwater levels may differ.

Fluctuations in the water table should be anticipated throughout the year with variations in precipitation and other environmental or physical factors. Seasonal fluctuations in the groundwater level should be expected due to variations in precipitation, evaporation, and surface water runoff.

5 DESIGN RECOMMENDATIONS

5.1 Basis

Our recommendations for the proposed development are based on data presented in this report which included five (5) soil borings spaced across the entire project area. Subsurface variations can exist at a site which may not be indicated by such a dispersed and limited boring program. If such variations or unexpected conditions are encountered during construction, or if the project information is incorrect or changed, we should be informed immediately since the validity of our recommendations may be affected.

5.2 Building Foundations

Based on the anticipated loading conditions for the proposed school annex and the soil conditions, the proposed building should be supported on deep foundations extending to the very stiff to hard clay layer with Q_p values of at least 3.75 tsf that were encountered at depths below 17 feet bgs. If the building is supported on shallow spread footings, unacceptable long-term settlements may develop due to consolidation of the upper clay layer. Alternative for deep foundation support are provided in the following sections.

5.2.1 Drilled Piers

In general, drilled piers consisting of shaft or belled piers can be dimensioned to exert a net allowable bearing pressure up to 10,000 pounds per square foot (psf) on the very stiff to hard clay stratum observed at about 17 feet bgs. We recommend the deep foundations extend at least 3 feet into the bearing layer and have a minimum pier length of 20 feet. Skin friction should be neglected for the entire shaft length.

Drilled pier foundations should be designed with a minimum shaft diameter of 30 inches to facilitate clean out and possible dewatering of the pier excavations. The squeeze analysis performed for driller piers at the site indicates that casing will not be required. However, we recommend the contractor be prepared with temporary casing to extend through any zones observed to be susceptible to squeezing or caving, and to control possible groundwater seepage.

Care should be taken so that the side and bottom of the pier excavations are not disturbed during construction. The bottom of the piers should be free of loose soil or debris prior to reinforcing steel and concrete placement.

5.2.2 Helical Piers

A helical pier foundation system could also be considered as a deep foundation system to support the building loads. Helical piers are a proprietary manufactured foundation system. A number of companies have developed and market suitable systems. A helical pier specialty contractor would design and install the piers.

Helical piers are installed using a rotary machine. The leading section of the helical pier is manufactured the helices which are drilled into the ground. Extensions can be added until the helices reach the appropriate depth at the predetermined depth/torque. This system transfers the load to the bearing plates (helices) that are located in firm soil below the compressible soils. To aid in lateral support of piers, a grouted pull-down pier may be constructed by installing a series of soil displacement disks to create an open annulus in which grout may be placed around the pier shaft.

The helical pier foundation option offers the possible advantage of a shallower pier depth, along with a warrantee by the specialty contractor. Other advantages include: no heavy equipment necessary for installation, equipment used in relatively small and can be easily mobilized in limited access areas, quick installation, limited excavation required, and causes minimal vibration that could adversely impact existing adjacent structures.

Based on the soil boring data, it is anticipated that an approved installer of the proprietary helical piers could design and install a system with working capabilities of 10-20 tons per pier when installed to an adequate depth into firm competent bearing soils located below the compressible clays. After the piers are installed, a grade beam would need to be cast over the top of the helical piers to provide anchorage and support for the structure.

5.3 Adjacent Existing Building

To help preclude possible undermining of existing McDade building foundations, excavations for proposed deep foundations should not extend below imaginary lines extending at a 45-degree angle downward and outward from the edges of the existing building footings. Using the helical pier alternative can reduce the risk of undermining existing foundations as minimal soil is excavated during the helical pier installation process. If any of these criteria cannot be met, the Geotechnical Engineer should be consulted for further evaluations.

Additionally, any significant stress from the existing foundations transmitted to soils beneath the proposed foundations should be accounted for in new foundation settlement estimates.

5.4 Floor Slab

If the risk of minor settlement of slabs can be tolerated, non-structural floor slabs may be supported on suitably prepared (compacted) subgrade independent of the deep foundation system. Support of floor slabs on the existing near-surface fill and buried topsoil should not be considered because of the risk of differential settlement of slabs. The existing fill and organic soil should be excavated beneath floor slabs and replaced with compacted structural fill, in accordance with **Section 6.2**.

A structural slab system should be considered if the risk of slab differential settlement cannot be tolerated.

Non-structural floor slabs should be structurally independent of the building columns and walls, and liberally jointed in accordance with ACI recommendations to reduce distress due to differential movement. A vertical modulus of subgrade reaction (k_{30}) of 150 pounds per cubic inch (pci) is recommended for design of slabs-on-grade supported on structural fill. We recommend that a plastic vapor barrier be placed under the floor slab where moisture-sensitive floor coverings will be used or where moisture-sensitive product or equipment will be stored.

The building floor slabs should be supported on a minimum 4-inch thick, relatively clean, free-draining granular base course bearing on a suitably prepared subgrade, including the removal and replacement of near-surface fill and organic soils. In our opinion, relatively clean, free-draining granular soil should contain no more than 5 percent fines, by dry weight, passing a No. 200 U.S. Standard sieve.

5.5 Seismic

A seismic site classification is required for the estimation of minimum earthquake design forces. The coefficient is a function of soil type (i.e., depth of soil and strata types) and depth to bedrock. Although the depth to bedrock was not confirmed by the boring program, published geological information indicates that it is likely to be on the order of 30 to 80 feet below the existing ground surface. Based on the average property descriptions in the 2015 International Building Code (IBC) and our general knowledge of geological conditions in the locale, in our opinion, the soil conditions at this site most closely resemble the site classification C.

5.6 Pavement Recommendations

Our recommendations are based on the assumption that the paved areas will be constructed on a proof-rolled (or stabilized) subgrade (see **Section 6.1**), or on structural fill overlying the same.

Serviceable pavements can be achieved by different combinations of materials and thicknesses, varied to provide roughly equivalent strengths. Local practice for existing pavement construction could be reviewed for other blends or combinations of materials that have been found satisfactory and for applicable minimum standards. For new pavements at the site, we provide the following guidelines that have been developed from the results of our geotechnical exploration assuming minimal truck traffic, moderate relatively low levels of vehicle traffic, and an assumed California Bearing Ratio (CBR) value of 4.

- 4 inches of compacted asphalt (combined surface and binder course);
- 6 inches of compacted granular base course.

We recommend that the base course consist of a dense-graded, crushed aggregate material, such as IDOT CA-6 stone. The gradation of this material is described in the Illinois Department of Transportation (IDOT) specifications. In our opinion, crushed aggregate material, such as gravel, slag, limestone or crushed concrete are acceptable base course materials as long as they approximate the recommended IDOT gradations and are approved by the design engineer/architect. The base course should be compacted to no less than 98 percent of its maximum standard Proctor density, or its equivalent relative density. Further, suitable primer and tack coats should be placed between the base course and between the overlying asphalt layers. In addition, all asphalt material and paving operations should meet applicable specifications of the Asphalt Institute and the IDOT specifications.

Structural fill and aggregate base course materials should be compacted to at least 98 percent of the maximum standard Proctor dry density (ASTM D698). Additionally, structural fill placed in the top 3.5 feet should not be frost susceptible.

We do not anticipate any problems due to the high groundwater table underlying the Site since the proposed asphalt areas are expected to be located several feet above the current groundwater levels. However, we recommend that the pavement and aggregate base course be properly graded and sufficiently high above any adjacent drainage ponds or swales to provide for positive pavement surface and base drainage.

The procedures we have used to develop our pavement guidelines are consistent with generally accepted engineering practice and are intended to provide a 20-year life span. However, based upon our past experience, we have found that proper construction techniques, quality of drainage, pavement maintenance and actual traffic loads are the major factors in determining pavement life and performance. It is important that experienced technical personnel observe construction activities to check that the pavement layers are constructed as designed.

5.7 Estimated Infiltration Rate

Two single ring infiltrometer tests were conducted at the site near the proposed playlot improvements to determine design infiltration rates. The tests were located near borings B-1 and B-5. The site soils were tested at a depth of approximately 2 feet below the existing surface. Infiltration testing was conducted in accordance with the test methods and procedures described in the Chicago Stormwater Ordinance Manual.

Based on the results of the infiltration test and the guidelines in the Chicago Stormwater Ordinance Manual, we do not recommend incorporating infiltration into the site best management practices (BMPs). The test results yielded design infiltration rates of 0.03 in/hour and 0.0 in/hour near borings B-1 and B-5, respectively. The infiltration test calculations are provided in **Appendix B**.

6 CONSTRUCTION CONSIDERATIONS

6.1 Site Preparation

All structural areas plus, where feasible, a minimum lateral margin of 5 feet beyond the perimeter of the proposed construction should be initially prepared by stripping/removing and grubbing the vegetation, topsoil, near-surface fill, and unsuitable materials.

Following the stripping/removal activities, the slab and pavement areas should be proofrolled to detect any localized soft or loose materials. Proofrolling consists of repeated passes of a loaded, pneumatic-tired vehicle, such as a tandem-axle dump-truck or front end loader (minimum 20 ton weight). The proofrolling activities should be observed by the Geotechnical Engineer or his representative. Any areas judged by the engineer or his representative that need improvement should be densified further or otherwise improved at the engineer's discretion.

After successful preparation of the subgrade, placement of the structural fill may then proceed as necessary to establish design grades. Where fill is required in the proposed building area, we recommend that it consist of granular structural fill. Where structural fill is required under slabs, it should extend laterally beyond all edges of the footings at least 6 inches for every 12 inches of undercut or fill depth required below the base of the slab. The structural fill should meet the requirements of **Section 6.2** and be placed in accordance with **Section 6.3**.

6.2 Structural Fill

Structural fill, defined as any fill which will support structural loads, should be free of organic material, have a plasticity index of less than 25 percent, a maximum particle size of no more than 3 in., and a maximum dry density in excess of 100 pcf, as determined by the standard Proctor compaction test (ASTM D698). In addition, structural fill should not be frost susceptible if placed within 3½ feet from the surface. The structural fill should be compacted to at least 98 percent of its maximum standard Proctor dry density (ASTM D698) under the foundations or floor slabs.

Except for the topsoil and fill encountered near the surface of the site, most site soils encountered in the borings appear suitable for use as structural fill. Some wetting or drying of these soils may be necessary to achieve proper compaction.

6.3 Fill Placement Control

To achieve the recommended compaction of structural fill, we suggest that the fill be placed and compacted in layers not exceeding 8 inches in loose lift thickness. To observe compliance

with the recommended density standards, we recommend that in-place density tests be performed at a frequency of at least one test for every 2,500 ft² of fill area per each lift of compacted fill placed in the proposed construction areas.

6.4 Construction Observations

We recommend that all floor slabs, footing subgrades, and utility trenches be observed by a qualified Geotechnical Engineer or his representative prior to placement of any reinforcing steel, concrete materials, or trench backfill materials. These observations are to confirm that the exposed soil layers are consistent with those encountered in the borings and to check that the exposed soils are of uniform consistency and adequate density.

6.5 Groundwater Concerns

Groundwater was found during subsurface investigation of this site as described in **Section 4.3** of this report. The design of a deep foundation system should consider the possibility that groundwater may be encountered during construction. We do not expect foundation excavations or utility excavations at this site to experience serious dewatering issues. However, if groundwater inflow, or surface water runoff (from a precipitation event) occurs, it should be removed by sumps and filtered pumps. Should these measures be inadequate or should groundwater conditions different than those described in this report be encountered, we recommend that WCG be contacted immediately to make appropriate recommendations.

6.6 Excavation Slope Stability

Our exploration did not include a detailed analysis of slope stability for any temporary excavation condition, including utility trenches. Based on the soil conditions encountered at the boring locations, temporary shallow construction excavations could expose sandy and clayey soils. For such conditions, it is our opinion that shallow temporary excavations can be cut with side slopes of 2H: 1V. However, current OSHA standards must be met and may be more restrictive. Hence, if safe side slopes cannot be maintained due to loose granular soil conditions, then the excavation sides should be flattened, shielded or shored in accordance with current OSHA standards.

7 GEOTECHNICAL RISK

The concept of risk is an important aspect of any geotechnical evaluation. The primary reason for this is that the analytical methods used by geotechnical engineers are generally empirical and must be tempered by engineering judgment and experience. Therefore, the solutions or recommendations presented in any geotechnical evaluation should not be considered risk free, and more importantly, are not a guarantee that the interaction between the soils and the proposed structure will perform as predicted, desired, or intended. The engineering recommendations presented in the preceding sections constitute our best estimate of those measures that are necessary to help the structure perform in a satisfactory manner based on the information generated during this and previous evaluations and our experience in working with these conditions.

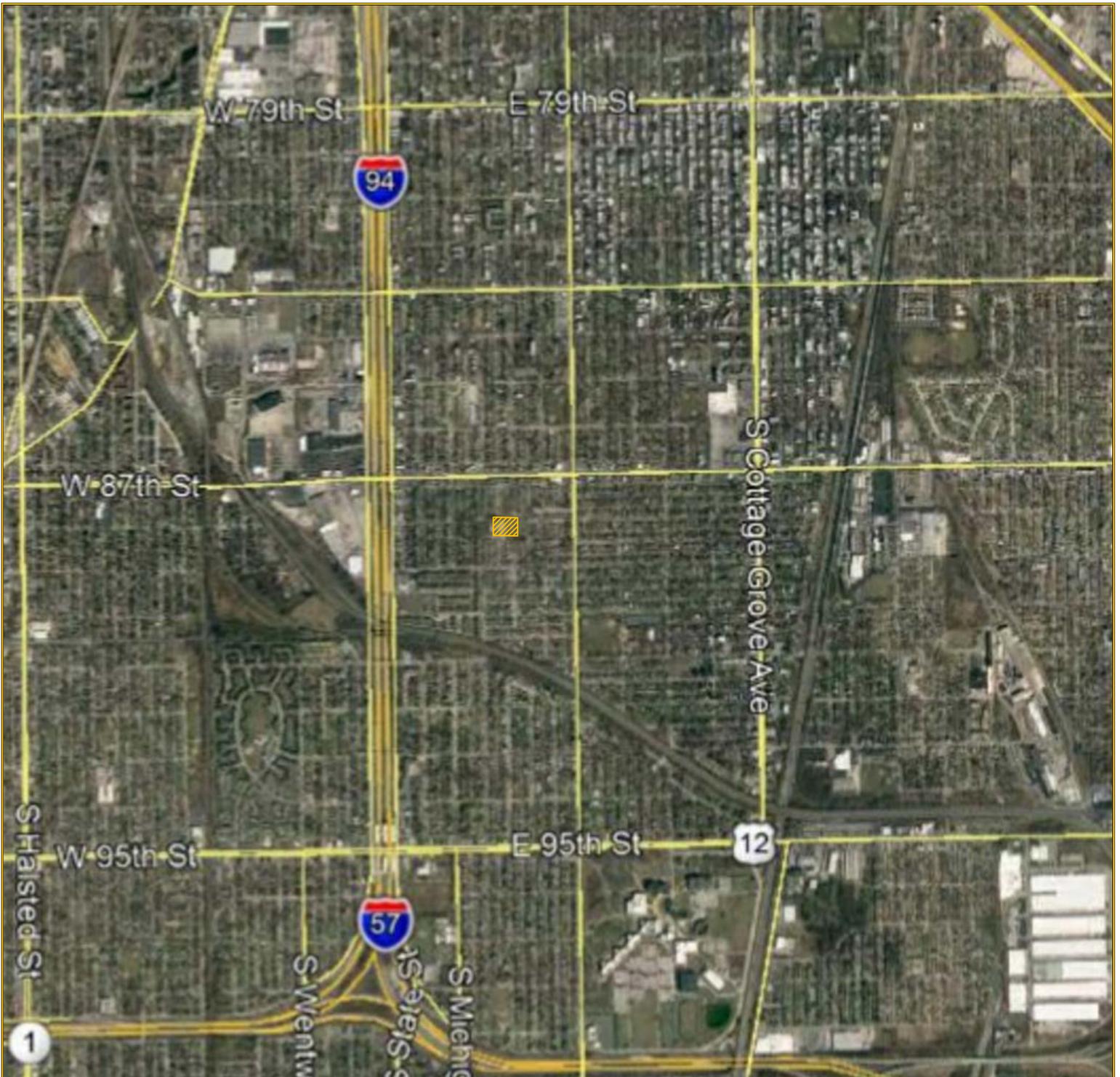
8 LIMITATIONS

WCG has prepared this report in accordance with generally accepted geotechnical engineering practices to aid in the evaluation of the site subsurface soils. No other warranty, expressed or implied, is made.

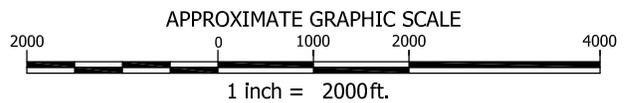
The scope of this report is limited to the specific project and location described herein, and our description of this project represents our understanding of the project. The geotechnical engineering analysis and foundation recommendations presented herein were developed based on the information obtained during the subsurface investigation. It should be noted that the borehole data reflects the subsurface conditions only at the specific locations designated on the borehole logs, and that soil and groundwater conditions could vary widely throughout the Site. If variations do appear during construction activities, it may become necessary to re-evaluate the recommendations of this report.

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

FIGURES



PROPERTY LOCATION



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<p>PREPARED FOR:</p> <p>PUBLIC BUILDING COMMISSION OF CHICAGO</p>	<p>PROPERTY LOCATION MAP McDADE SCHOOL IMPROVEMENTS 8801 S. INDIANA AVE. CHICAGO, IL</p> <p><small>REUSE OF DOCUMENTS THIS DOCUMENT, AND THE DESIGNS INCORPORATED HEREIN, AS AN INSTRUMENT OF PROFESSIONAL SERVICE, IS THE PROPERTY OF WEAVER CONSULTANTS GROUP, AND IS NOT TO BE USED IN WHOLE OR IN PART, WITHOUT THE WRITTEN AUTHORIZATION OF WEAVER CONSULTANTS GROUP.</small></p>	 <p>Weaver Consultants Group CHICAGO, ILLINOIS (312) 922-1030 www.wcgrp.com</p>	<p>DRAWN BY: SAS REVIEWED BY: JT DATE: 10/30/2018 FILE: 1012-325-19-01 CAD: Boring Location Plan.dwg</p> <p>FIGURE 1</p>
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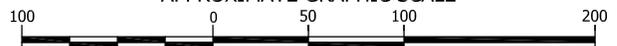


SOIL BORING LOCATION



SOIL CROSS-SECTION LOCATION

APPROXIMATE GRAPHIC SCALE



1 inch = 100 ft.



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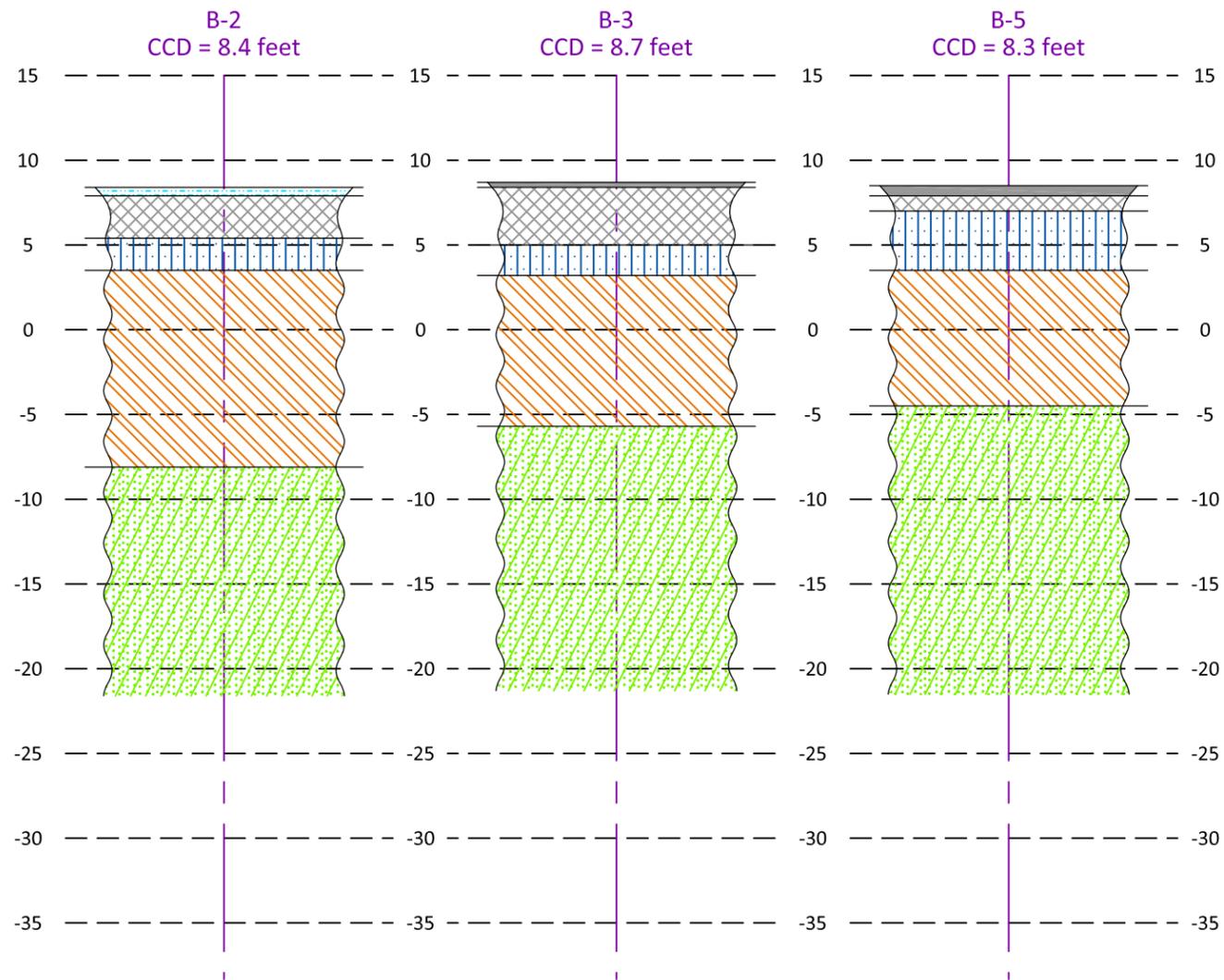
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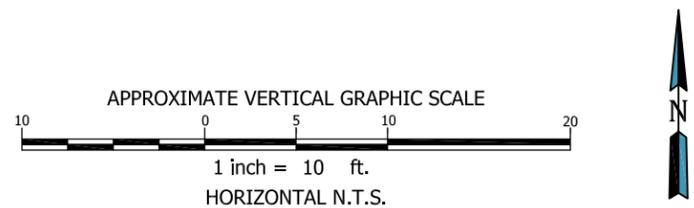
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FIGURE 2



-  ASPHALT PAVEMENT
-  TOPSOIL
-  FILL
-  SAND WITH FINES (SM-SC)
-  MEDIUM STIFF TO STIFF CLAY (CL)
-  VERY STIFF TO HARD HARDPAN (CL,ML,CL-ML)



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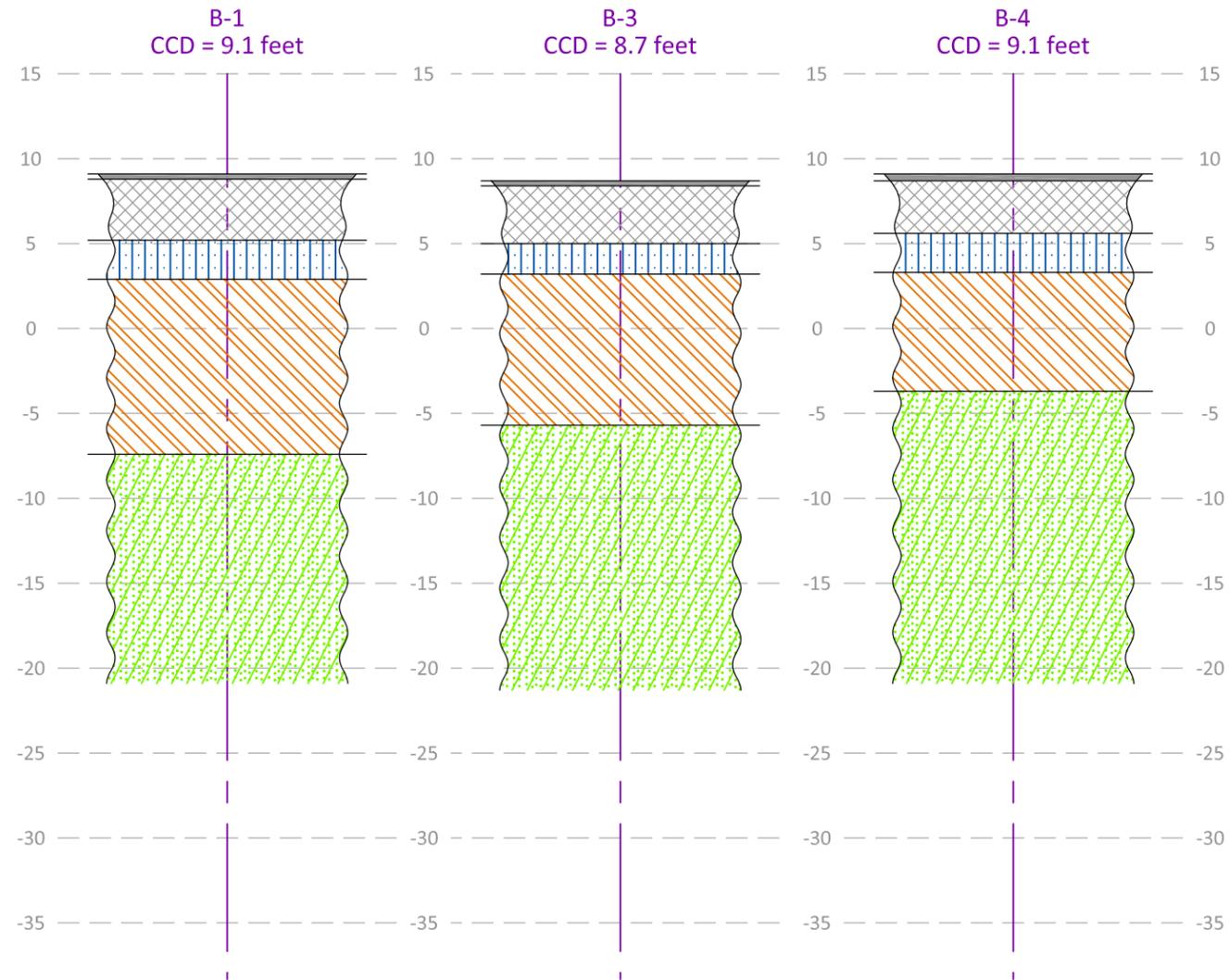
SOIL CROSS-SECTION A-A'
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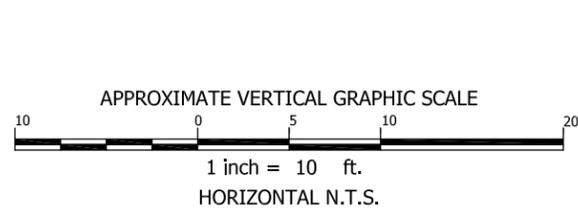
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CAD: Fig 3&4 - Soil Profiles.dwg

FIGURE 3



-  ASPHALT PAVEMENT
-  TOPSOIL
-  FILL
-  SAND WITH FINES (SM-SC)
-  MEDIUM STIFF TO STIFF CLAY (CL)
-  VERY STIFF TO HARD HARDPAN (CL,ML,CL-ML)



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SOIL CROSS-SECTION B-B'
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FIGURE 4

BORING LOGS

WATER LEVEL DATA
 NE = Not Encountered

Started: 10/23/2018

Completed: 10/23/2018

Engineer: S. Schubert

Driller: Strata

Drilling Equip.: D-50

Drilling Method: 4" SSA/Mud Rotary

PROJECT: Mc Dade Classical School Annex

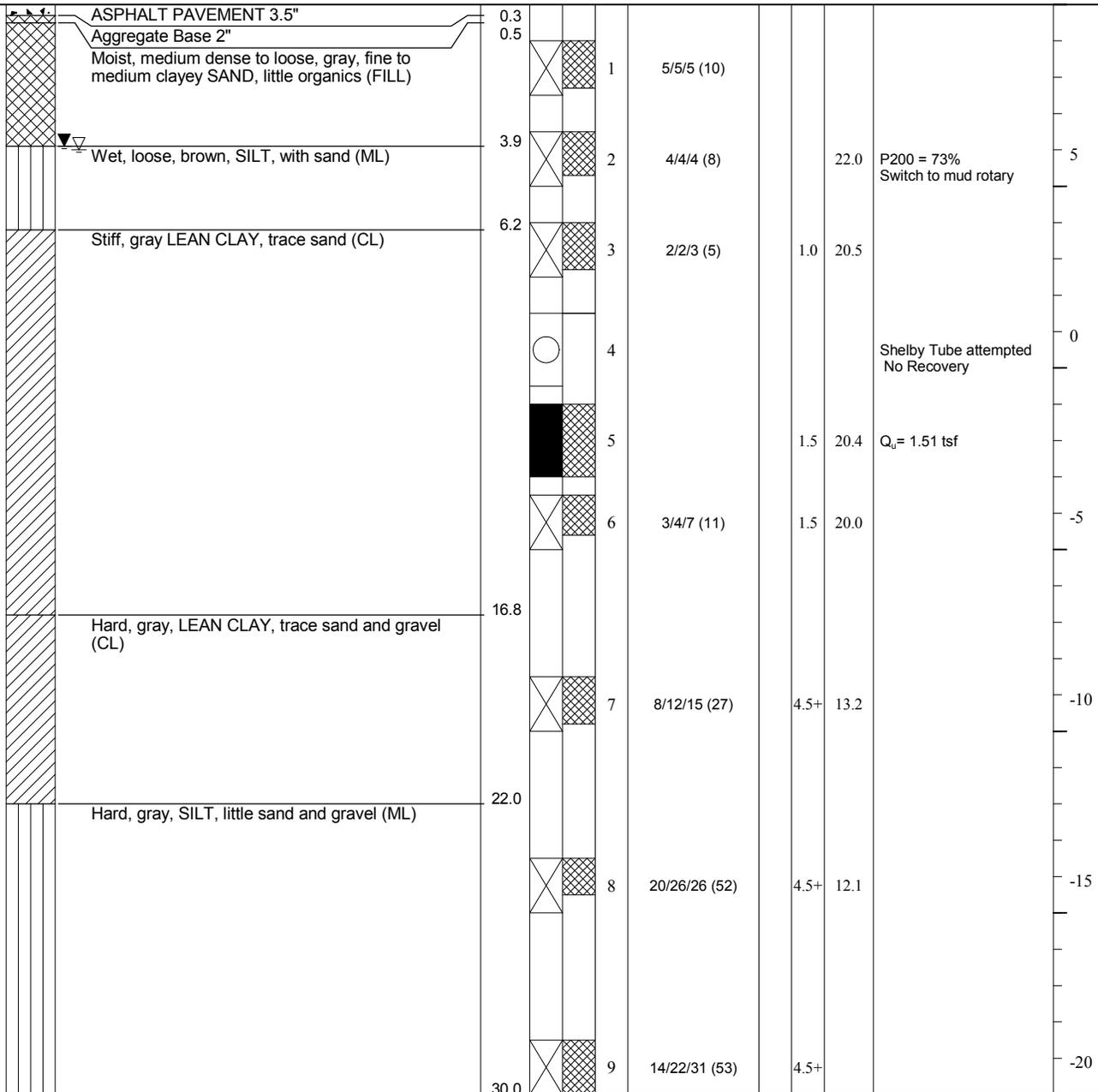
8801 South Indiana Avenue

Chicago, IL

CLIENT: Public Building Commission of Chicago

Chicago Illinois

Depth (ft)	Symbol	DATUM: SURFACE ELEVATION (ft): 9.1		Strata Depth (ft)	Type	Recovery	Number	Standard Penetration Test-Blows/6" (#) = "N" Value	LOI (%)	Qp (tsf)	Moisture Content %	BORING AND SAMPLING NOTES	Elevations (ft)
		SOIL DESCRIPTION, CLASSIFICATION and USCS or AASHTO GROUP SYMBOL											



NOTES: Boring Terminated at 30 ft

- Weather: Partly Cloudy, 51°F
- Used automatic hammer
- Backfilled with auger cuttings

LEGEND

	= Auger		= No Recovery		= Split-Spoon Sample
	= Geoprobe		= Core Sample		= Vane Shear Test
	= Grab Sample		= Shelby Tube		

LLC- ELEVATIONS 1012-377-19-01 MC DADE.GPJ 11/7/18

WATER LEVEL DATA
 NE = Not Encountered

Started: 10/24/2018

Completed: 10/24/2018

Engineer: S. Schubert

Driller: Strata

Drilling Equip.: D-50

Drilling Method: 4" SSA/Mud Rotary

PROJECT: Mc Dade Classical School Annex

8801 South Indiana Avenue

Chicago, IL

CLIENT: Public Building Commission of Chicago

Chicago Illinois

Depth (ft)	DATUM: SURFACE ELEVATION (ft): 8.4		Strata Depth (ft)	Type	Recovery	Number	Standard Penetration Test-Blows/6" (#) = "N" Value	LOI (%)	Qp (tsf)	Moisture Content %	BORING AND SAMPLING NOTES	Elevations (ft)
	Symbol	SOIL DESCRIPTION, CLASSIFICATION and USCS or AASHTO GROUP SYMBOL										
0.5		Moist, dark brown, TOPSOIL (OL)	0.5									
3.0		Moist, very stiff, brown, LEAN CLAY, little gravel (FILL)	3.0			1	4/8/11 (19)		3.0	19.2		
4.9		Moist to wet, loose, brown and gray, fine to medium SILTY SAND (SM)	4.9			2	2/3/2 (5)					5
5		Stiff to medium stiff, gray, LEAN CLAY, trace sand (CL)				3	2/3/4 (7)	1.25	18.2		Switch to mud rotary	
10		Trace gravel below 11 feet				4	2/4/4 (8)	0.75	19.7			0
15						5	2/3/5 (8)	1.0	19.2		PL= 17 LL= 31 PI= 14	-5
16.5		Hard, gray, LEAN CLAY, trace sand and gravel (CL)	16.5			6	2/4/6 (10)	1.5	18.8			
20						7	7/11/15 (26)	4.5+	14.7			-10
25						8	12/18/22 (40)	4.5+	12.6			-15
30.0			30.0			9	12/16/19 (35)	4.5+	13.6			-20

NOTES: Boring Terminated at 30 ft

- Weather: Partly Cloudy, 51°F
- Used automatic hammer
- Backfilled with auger cuttings

LEGEND

-  = Auger
-  = No Recovery
-  = Split-Spoon Sample
-  = Geoprobe
-  = Core Sample
-  = Vane Shear Test
-  = Grab Sample
-  = Shelby Tube

WATER LEVEL DATA
 NE = Not Encountered

Started: 10/23/2018

Completed: 10/23/2018

Engineer: S. Schubert

Driller: Strata

Drilling Equip.: D-50

Drilling Method: 4" SSA/Mud Rotary

PROJECT: Mc Dade Classical School Annex

8801 South Indiana Avenue

Chicago, IL

CLIENT: Public Building Commission of Chicago

Chicago Illinois

Depth (ft)	DATUM: SURFACE ELEVATION (ft): 8.7		Strata Depth (ft)	Type	Recovery	Number	Standard Penetration Test-Blows/6" (#) = "N" Value	LOI (%)	Qp (tsf)	Moisture Content %	BORING AND SAMPLING NOTES	Elevations (ft)
	Symbol	SOIL DESCRIPTION, CLASSIFICATION and USCS or AASHTO GROUP SYMBOL										
0.1		ASPHALT PAVEMENT 1.5"	0.1									
0.3		Aggregate Base 2"	0.3									
3.7		Moist, very stiff, gray, SANDY CLAY, trace gravel, little organics (FILL)	3.7	X		1	3/6/6 (12)	5.8	2.0	21.6		
5.5		Wet, loose, brown, fine SAND, with silt (SP-SM)	5.5	X		2	4/3/2 (5)			24.5		5
5.5		Medium stiff to stiff, gray, LEAN CLAY, trace sand (CL)	5.5	X		3	1/3/4 (7)		1.25		Switch to mud rotary PL=16 LL=34 PI=18	
14.4		Very stiff to hard, gray, LEAN CLAY, trace sand and gravel (CL)	14.4	X		4	3/6/9 (15)		1.25	21.7	Q _u = 0.56 tsf	0
14.4			14.4	X		5	2/4/4 (8)		1.0	20.5		
20.0			20.0	X		6	6/10/11 (21)		4.5+	12.8		-5
25.0			25.0	X		7	11/15/19 (34)		4.5+	11.9		-10
30.0			30.0	X		8	11/25/27 (52)		4.5+	13.1		-15
30.0			30.0	X		9						-20

NOTES: Boring Terminated at 30 ft

- Weather: Partly Cloudy, 51°F
- Used automatic hammer
- Backfilled with auger cuttings

LEGEND

-  = Auger
-  = No Recovery
-  = Split-Spoon Sample
-  = Geoprobe
-  = Core Sample
-  = Vane Shear Test
-  = Grab Sample
-  = Shelby Tube

WATER LEVEL DATA
 NE = Not Encountered

Started: 10/23/2018

Completed: 10/23/2018

Engineer: S. Schubert

Driller: Strata

Drilling Equip.: D-50

Drilling Method: 4" SSA/Mud Rotary

PROJECT: Mc Dade Classical School Annex

8801 South Indiana Avenue

Chicago, IL

CLIENT: Public Building Commission of Chicago

Chicago Illinois

Depth (ft)	DATUM: SURFACE ELEVATION (ft): 9.1		Strata Depth (ft)	Type	Recovery	Number	Standard Penetration Test-Blows/6" (#) = "N" Value	LOI (%)	Qp (tsf)	Moisture Content %	BORING AND SAMPLING NOTES	Elevations (ft)
	Symbol	SOIL DESCRIPTION, CLASSIFICATION and USCS or AASHTO GROUP SYMBOL										
0.2		Asphalt Pavement 2"	0.2									
0.4		Aggregate Base 2.5"	0.4									
2.2		Moist, loose, brown, fine to medium SAND, little clay, trace gravel (FILL)	2.2	X		1	6/5/3 (8)					
3.5		Moist, loose, black, SANDY ORGANIC TOPSOIL (OL)	3.5	X								
5.8		Wet, loose, brown, fine to medium SAND, with silt (SP-SM)	5.8	X		2	4/5/3 (8)					5
5.8		Stiff to medium stiff, gray, LEAN CLAY, trace sand (CL)	5.8	X		3	2/2/3 (5)	1.0	20.0			
12.8		Very stiff to hard, gray, LEAN CLAY, trace sand and gravel (CL)	12.8	X		4	2/2/4 (6)	1.25	20.3		Switch to mud rotary	0
12.8			12.8	X		5	1/2/4 (6)	0.75	20.6		PL= 16 LL= 33 PI= 17	
15.0				X		6	3/7/10 (17)	3.25	19.7			-5
20.0				X		7	6/10/13 (23)	4.5+	13.2			-10
25.0				X		8	8/13/16 (29)	4.5+	12.9			-15
30.0			30.0	X		9	9/14/20 (34)	4.5+	11.2			-20

NOTES:

Boring Terminated at 30 ft

- Weather: Partly Cloudy, 51°F
- Used automatic hammer
- Backfilled with auger cuttings

LEGEND

-  = Auger
-  = No Recovery
-  = Split-Spoon Sample
-  = Geoprobe
-  = Core Sample
-  = Vane Shear Test
-  = Grab Sample
-  = Shelby Tube

APPENDIX A

Field Exploration

WEAVER CONSULTANTS GROUP, LLC

- ☒ 35 East Wacker Drive, Suite 1250, Chicago, IL 60601 • (312) 922-0201
- ☐ 6420 Southwest Boulevard, Suite 206, Fort Worth, TX 76109 • (817) 735-9770
- ☐ 7121 Grape Road, Granger, IN 46530 • (574) 271-3447

LOG OF SOIL BORING - GENERAL NOTES

*In order to provide uniformity throughout our projects,
the following system has been adopted to describe each soil sample.
Rock, shale and other materials will be described in detail as encountered.*

CONSISTENCY OF COHESIVE SOILS		RELATIVE DENSITY OF GRANULAR SOILS		
UNCONFINED COMPRESSIVE STRENGTH, Q_u (tsf)	CONSISTENCY	Safety Hammer	Automatic Hammer	RELATIVE DENSITY
<0.25	Very Soft	<4	<3	Very Loose
0.25 - 0.49	Soft	4 - 9	3 - 7	Loose
0.50 - 0.99	Medium Stiff	10 - 29	8 - 21	Medium Dense
1.00 - 1.99	Stiff	30 - 50	22 - 35	Dense
2.00 - 3.99	Very Stiff	51 - 80	36 - 60	Very Dense
4.00 - 8.00	Hard	>80	>60	Extremely Dense
>8.00	Very Hard	*Number of blows per foot required to drive a 2-in. O.D. split-spoon sampler using a 140-lb. weight falling freely for 30 in., except where otherwise noted.		

COLOR - AS DETERMINED ON THE FRESH, MOIST SAMPLES	
PREDOMINATE COLORS	
Black	Yellow
Brown	Red
Gray	Blue
SHADES	MODIFYING ADJECTIVES
Light	Vari-colored
Dark	Streaked
	Mottled

ABBREVIATIONS	
DRILLING AND SAMPLING	
A.D. - After Drilling	PMT - Pressuremeter Test
BA - Bucket Auger (3/4-in. O.D.), except where noted	QC - Static Cone Penetrometer Reading (tsf)
CFA - Continuous Flight Auger	RC - Rock Core with diamond bit NX size, except where noted
C.I. - Cave-In Depth	RQD - Rock Quality Designation
CS - Continuous Sampling	SPT - Standard Penetration Test
DP - Direct Push	SS - 1 3/8-in. I.D. Split-Spoon Sample (2-in.O.D.)
GP - Geoprobe	ST - 3-in. O.D. Thin-Walled Shelby Tube Sample, except where noted
HA - Hand Auger	
HSA - Hollow Stem Auger	
HPR - Hollow Probe Rod	
MR - Mud Rotary	
NR - No Recovery	WOH - Weight of Hammer

GRADATION DESCRIPTION AND TERMINOLOGY	
COMPONENTS	SIZE RANGE
Boulders	Over 8 inches
Cobbles	8 inches to 3 inches
Gravel	3 inches to # 4 sieve (4.75 mm)
Sand	#4 sieve to #200 sieve (0.075 mm)
Silt	Passing #200 sieve to 0.005 mm
Clay	Smaller than 0.005 mm

LABORATORY TESTS	
DD - Dry Density (pcf)	MD - Moist Density (pcf)
LL - Liquid Limit %	pH - Soil Alkalinity/Acidity
LOI - Loss-on-Ignition, Organic Content (%)	PID - Photoionization Detector (ppm)
MC - Moisture Content (%)	PI - Plasticity Index (%)
P200 - Percentage of Soil Particles, by dry weight, Passing a No. 200 U.S. Standard Sieve	PL - Plastic Limit (%)
	QP - Calibrated Hand Penetrometer Reading (tsf)
	QU - Unconfined Compressive Strength (tsf)

DESCRIPTION OF COMPONENT ALSO PRESENT IN SAMPLE	PERCENT OF DRY WEIGHT
Trace	1 - 9
Little	10 - 19
Some	20 - 34
And	35 - 50

GROUNDWATER LEVELS

Water levels are those observed when borings were drilled, or as noted. Porosity of soil strata, variations of rainfall, site topography, etc., may cause changes in these levels.

WATER LEVEL MEASUREMENTS	
BF - Backfilled	D@C.I. - Dry at Cave-In Depth
D - Dry	NE - Not Encountered

ORGANIC CLASSIFICATION BY LOSS-ON-IGNITION¹

Category	Name	Organic Content (% by dry weight)	Group Symbols	Category	Name	Organic Content (% by dry weight)	Group Symbols
ORGANIC MATTER	FIBROUS PEAT (woody, mats, etc.)	75 to 100 % Organics either visible or inferred	PT	ORGANIC SOILS	Clayey ORGANIC SILT	5 to 30% Organics either visible or inferred	OH
	FINE GRAINED PEAT (amorphous)				Organic SAND or SILT		OL
HIGHLY ORGANIC SOILS	Silty Peat	30 to 75% Organics either visible or inferred	PT	SLIGHTLY ORGANIC SOILS	SOIL FRACTION add slightly Organic	Less than 5% Organics combined visible and inferred	Depend upon inorganic fraction
	Sandy Peat						

¹U.S. Navy, (May 1982), Naval Facilities Engineering Command, Design Manual DM 7.1, "Soil Mechanics," Dept. of Navy, Alexandria, VA.

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- (817) 735-9770
- (630) 717-4848
- (574) 271-3447

UNIFIED SOIL CLASSIFICATION SYSTEM

Major Divisions			Group Symbol	Typical Names	Classification on basis of percentage of fines by dry wt.	Laboratory Classification Criteria		
COARSE-GRAINED SOILS	GRAVELS	Clean Gravels	GW	Well-graded gravels and gravel-sand mixtures, little or no fines	< 5% passing #200 sieve = GW, GP, SW, SP	$C_u = D_{60}/D_{10}$ Greater Than 4; $C_c = \frac{(D_{30})^2}{D_{10} \times D_{60}}$ between 1 and 3		
			GP	Poorly-graded gravels and gravel-sand mixtures, little or no fines		Not meeting both criteria for GW		
		Gravels w/fines	GM	Silty gravels, gravel-sand-silt mixtures		Atterberg limits plot below "A" line or plasticity index less than 4	Atterberg limits plotting in hatched area are borderline classification requiring use of dual symbols	
			GC	Clayey gravels and gravel-sand-clay mixtures		Atterberg limits plot above "A" line and plasticity index greater than 7		
	SANDS	Clean Sands	SW	Well-graded sands and gravelly sands, little or no fines	5% to 12% passing #200 sieve = Borderline Classifications requiring use of dual symbols	$C_u = D_{60}/D_{10}$ Greater Than 6; $C_c = \frac{(D_{30})^2}{D_{10} \times D_{60}}$ between 1 and 3		
			SP	Poorly-graded sands and gravelly sands, little or no fines		Not meeting both criteria for SW		
		Sands w/fines	SM	Silty sands and sand-silt mixtures		Atterberg limits plot below "A" line and plasticity index less than 4	Atterberg limits plotting in hatched area are borderline classifications requiring use of dual symbols	
			SC	Clayey sands and sand-clay mixtures		Atterberg limits plot above "A" line and plasticity index greater than 7		
	FINE-GRAINED SOILS	SILTS & CLAYS	Liquid Limit 50% or less	ML	Inorganic silts, very fine sands, rock flour, silty or clayey fine sands	Equation of "A" line: $PI = 0.73 (LL - 20)$		
				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 & CLAYS		Liquid Limit greater than 50%	MH	Inorganic silts, micaceous or diatomaceous fine sands or silts, elastic silts				
			CH	Inorganic clays of high plasticity Fat clays				
			OH	Organic clays of medium to high plasticity				
HIGHLY ORGANIC SOILS		PT	Peat, Muck and other highly organic soils					

Plasticity Chart

FIELD EXPLORATION PROCEDURES

Standard Penetration Test Soil Borings

General

We wish to point out that the soils actually recovered from our borings for observation and testing represent a very small percentage of the site soils. Our records depict subsurface conditions only at specific locations and at the particular time when drilling. Soil conditions at other locations may differ from conditions occurring at these boring locations. The passage of time may result in a change in the subsurface soil and groundwater conditions at the boring locations. The interface between differing subsurface materials on the logs and profiles represent approximate boundaries. The transition between materials may be gradual. Also, thin strata that occur between sample depths may be present, but remain undetected by routine sampling procedures.

Drilling Procedures

Soil borings were performed at the approximate locations shown on the attached boring plan. The soil borings were advanced by mechanically twisting a continuous steel-flight, solid-stem augers and rotary bits into the soil. The outside diameter (O.D.) of the solid-stem auger is typically 4 in. When mud rotary is used, cuttings are circulated out of the borehole in drilling mud.

The auger is turned into the ground, which displaces the soil upwards as it advances. Once the desired sample depth is achieved, the advancement of the auger is stopped. The borehole is then cleaned of any soil and the sampling tools are inserted, and the sampling is performed. When drilling below the water table in pervious soils, a head of water is maintained in the hollow-stem, to prevent a "quick" condition at the auger tip.

Penetration Testing and Split-Barrel Sampling

Standard Penetration Testing and split-barrel sampling are normally conducted in the borings to provide relative density information and soil samples for visual classification and laboratory testing. The standard split-barrel (commonly called split-spoon) sampler is a 2-in. O.D., 1.375-in. I.D., typically 18 to 24 in. long and is connected to an AW or N size drilling rod. The sampler is then driven into the soil with a force of a 140 lb. hammer free-falling a distance of 30 in. The number of hammer blows required to drive the sampler into the soil is recorded for each 6-in. interval. The sampler is typically driven a total of 18 in., and the last two 6-in.

interval blow counts are added together and commonly referred to as the "N" value, blow count or penetration resistance. Representative samples are placed in airtight glass jars and returned to our laboratory for further observation and testing. Descriptions of the split-barrel samples and the penetration resistances are shown on the boring logs.

Shelby Tube Sampling Procedure

In the Shelby tube sampling procedure, a thin-walled steel seamless tube with a sharp cutting edge is pushed hydraulically into the soil and a relatively undisturbed sample is obtained. This procedure is generally employed in cohesive soils. The tubes are carefully handled in the field to avoid excessive disturbance and are returned to the laboratory for extrusion and further analysis and testing.

Calibrated Pocket Penetrometer Testing

The strength of cohesive soils does not correlate as well as granular materials with the Standard Penetration Testing described above. Typically, we test split-barrel samples of cohesive soils with a calibrated pocket penetrometer in the field. This test involves pushing a spring-loaded piston, 0.25-in. in diameter, into the sample and measuring the spring deflection, which has been correlated to shear strength. This test is used as a rough approximation method only. More refined results require undisturbed Shelby tube sampling and laboratory unconfined compressive strength testing.

Water Level Readings

When the drilling crew notices groundwater or significant variations in soil moisture, they are recorded on the boring logs. Generally, the level of water at the time of drilling is measured and recorded. The readings may indicate the approximate level of the hydrostatic water table at the time of our drilling activities.

Where low permeability soils are encountered, the water seeps into the borings at a slow rate, and it is generally not possible to establish accurate groundwater level readings in an open borehole during the drilling operations. If water-drilling methods are used, a local groundwater "mound" could be created, taking several days to dissipate. Also, the groundwater level typically fluctuates on a long-term or seasonal basis, due to variations in precipitation, surface run-off, evaporation, etc. When these long-term readings are required, piezometers or monitoring wells are necessary to maintain an open hole.

Boring Log Preparation

The subsurface conditions encountered during drilling are reported on a field log recorded by the chief driller. The driller's field record contains information concerning the boring method, samples attempted and recovered, indications of the presence of various materials such as coarse gravel, cobbles, etc., and observations between samples. Therefore, these records contain both factual and interpretive information. The field logs are on file in our office.

The soil samples, plus the field logs, are reviewed by a geotechnical engineer, geologist, or geotechnician. The engineer/geologist/geotechnician then classifies the soil in general accordance with the Unified Soil Classification System and prepares the final boring logs, which are the basis for our evaluations and recommendations. The group symbol for each soil type is indicated in parentheses following the soil descriptions on the boring logs. The final boring logs represent our interpretation of the contents of the field logs based on the results of the engineering review and laboratory testing of the field samples. The final boring logs are included in this section.

LABORATORY TESTING PROCEDURES

Representative soil samples were selected and tested in our laboratory in order to check field classifications and to evaluate pertinent engineering properties. The laboratory testing program included visual classification of all samples and hand penetrometer tests on all cohesive samples. In the hand penetrometer test, the unconfined compressive strength of a cohesive soil is estimated by measuring the resistance of the soil sample to penetration by a small spring calibrated cylinder. Any additional tests are described below or on the following sheet(s). Appropriate data obtained from laboratory tests are also included on the respective boring logs.

A geotechnical engineer classified each soil sample on the basis of texture and plasticity in accordance with the Unified Soil Classification System (ASTM D 2487 and/or ASTM D 2488). The group symbol for each soil type is indicated in parentheses following the soil descriptions on the boring logs. A brief explanation of the Unified System is included with this report.

Data obtained from the field logs and appropriate laboratory tests have been shown on the boring logs. The procedures used in preparing the final boring logs are described on the sheet entitled "Field Exploration Procedures."

It should be noted that the geotechnical engineer grouped the various soil types into the major zones noted on the boring logs. The stratification lines designating the interfaces between earthen materials shown on the boring logs and profiles are approximate; in-situ, the transitions may be gradual.

All samples will be retained in our Granger, Indiana laboratory for a period of thirty (30) days after which they will be discarded unless other instructions as to their disposal are received.

Calibrated Pocket (Hand) Penetrometer (Q_P) Testing

This test involves pushing a spring-loaded piston, 0.25-in. in diameter, into the sample and measuring the spring deflection, which has been correlated to shear strength. This test is used as a rough approximation method only. More refined results require undisturbed Shelby tube sampling and laboratory unconfined compressive strength testing.

Moisture Content Test

Moisture content tests were performed on selected soil samples. The moisture content has a significant effect on the strength, compressibility and general behavior of the soil.

Loss-On-Ignition Test

Loss-on-ignition (L.O.I.) tests are performed on samples to determine the percent of organic material present. Generally, organic material is undesirable when present in soil to be used as the foundation for structures or as engineered (structural) fill.

Atterberg Limits

To provide a quantitative appraisal of the soil and define the plastic characteristics, Atterberg limits are determined. The liquid limit is defined as the moisture content above which the soil would tend to act as a liquid, and below which the soil would tend to act as a solid. The difference between the liquid and the plastic limits is the plasticity index, which provides a measure of the plasticity of the soil.

Past experience and research studies indicate that if the natural moisture content of the soil is close to the liquid limit, the soil is likely normally consolidated and could be expected to settle under any increase in effective stress. However, if the moisture content is close to the plastic limit, the soil is likely over-consolidated and would not readily settle under a small increase in effective stress.

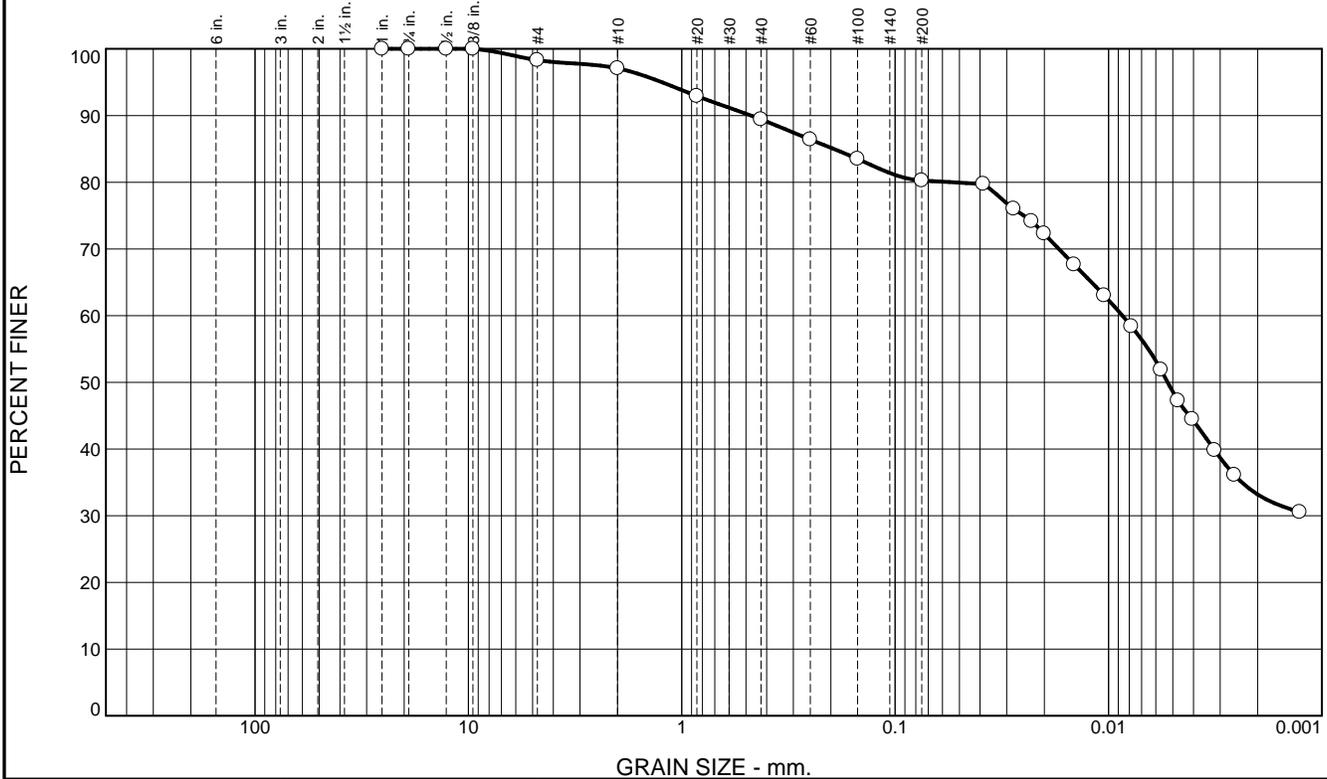
Grain-Size Test

Grain-size tests are performed to determine the soil classification and the grain-size distribution. The soil samples are prepared for testing according to ASTM D 421 (dry preparation) or ASTM D 2217 (wet preparation). The grain-size distribution of soils coarser than a No. 200 U.S. Standard sieve (0.074 mm opening) is determined by passing the samples through a standard set of nested sieves. Materials passing the No. 200 U.S. Standard sieve are suspended in water and the grain-size distribution calculated in accordance with ASTM D 422, or washed over the No. 200 sieve in accordance with ASTM D 1140.

APPENDIX B

Laboratory Test Results

ASTM D7928 (Air Dried) & ASMT D6913: Method B (Oven Dried)



% Cobbles	% Gravel		% Sand			% Fines	
	Coarse	Fine	Coarse	Medium	Fine	Silt	Clay
0.0	0.0	1.7	1.2	7.7	9.1	31.7	48.6

SIEVE SIZE	PERCENT FINER	SPEC.* PERCENT	PASS? (X=NO)
1"	100.0		
0.75"	100.0		
0.50"	100.0		
0.375"	100.0		
#4	98.3		
#10	97.1		
#20	92.9		
#40	89.4		
#60	86.4		
#100	83.5		
#200	80.3		
0.0386 mm.	79.7		
0.0279 mm.	76.0		
0.0230 mm.	74.2		
0.0201 mm.	72.3		
0.0145 mm.	67.7		
0.0105 mm.	63.0		
0.0078 mm.	58.4		
0.0057 mm.	51.9		
0.0047 mm.	47.3		
0.0041 mm.	44.5		
0.0032 mm.	39.8		
0.0026 mm.	36.1		
0.0013 mm.	30.5		

Soil Description

Atterberg Limits
 PL= LL= PI=

Coefficients
 D₈₅= 0.1941 D₆₀= 0.0086
 D₅₀= 0.0053 D₁₅=
 D₁₀= C_u= C_c=

Classification
 USCS= AASHTO=

Remarks

* (no specification provided)

Source of Sample: B-1 Depth: 3.5 - 5.0 ft
 Sample Number: 2

Date: 10-31-2018

Weaver Consultants Group

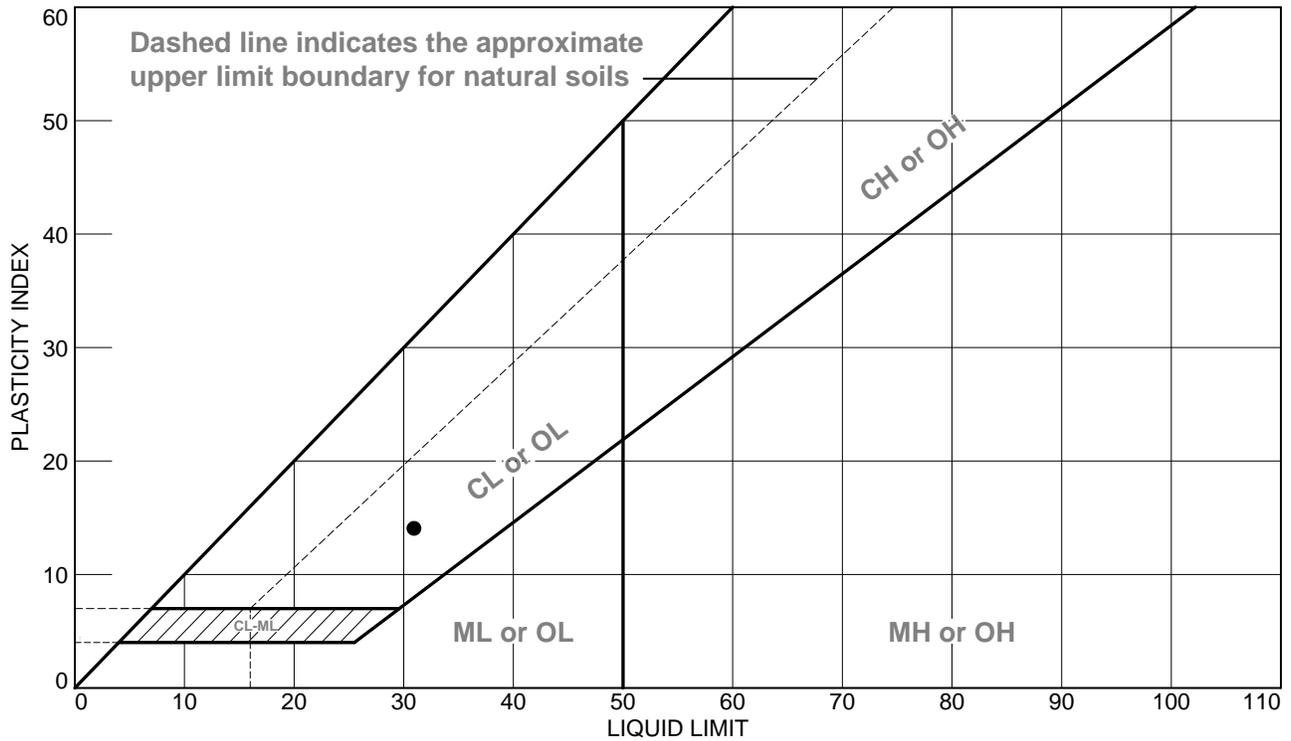
Granger, Indiana

Client: Public Building Commission of Chicago
Project: McDade Classical School Annex
 Chicago, Illinois
Project No: 1012-325-19-01

Figure

Tested By: pl Checked By: jjw

ATTERBERG LIMITS TEST REPORT ASTM D 4318



SOIL DATA

SOURCE	SAMPLE NO.	DEPTH	NATURAL WATER CONTENT (%)	PLASTIC LIMIT (%)	LIQUID LIMIT (%)	PLASTICITY INDEX (%)	LIQUIDITY INDEX	USCS
● B-2	5	11.0 - 12.5 ft		17	31	14		

Weaver Consultants Group

Granger, Indiana

Client: Public Building Commission of Chicago

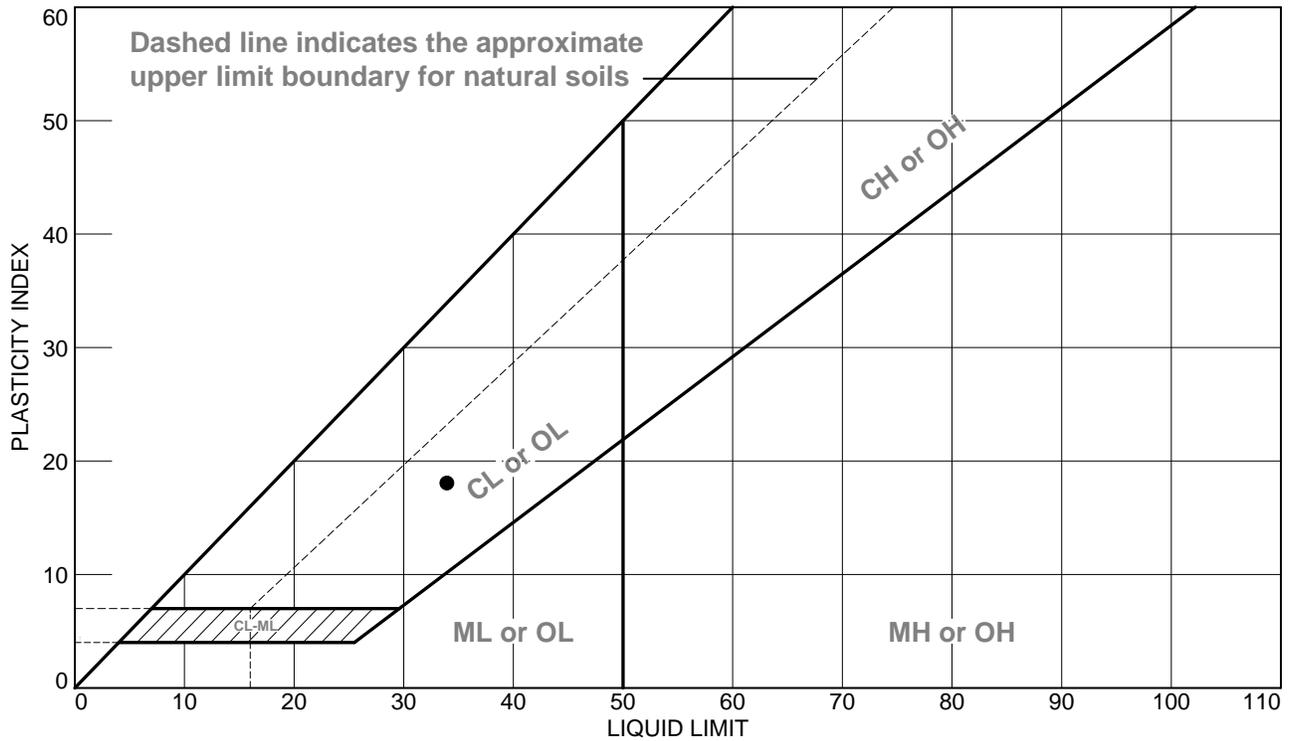
Project: McDade Classical School Annex
Chicago, Illinois

Project No.: 1012-325-19-01

Figure

Tested By: dw Checked By: jjw

ATTERBERG LIMITS TEST REPORT ASTM D 4318



SOIL DATA

	SOURCE	SAMPLE NO.	DEPTH	NATURAL WATER CONTENT (%)	PLASTIC LIMIT (%)	LIQUID LIMIT (%)	PLASTICITY INDEX (%)	LIQUIDITY INDEX	USCS
●	B-3	3	6.0 - 7.5 ft		16	34	18		

Weaver Consultants Group

Granger, Indiana

Client: Public Building Commission of Chicago

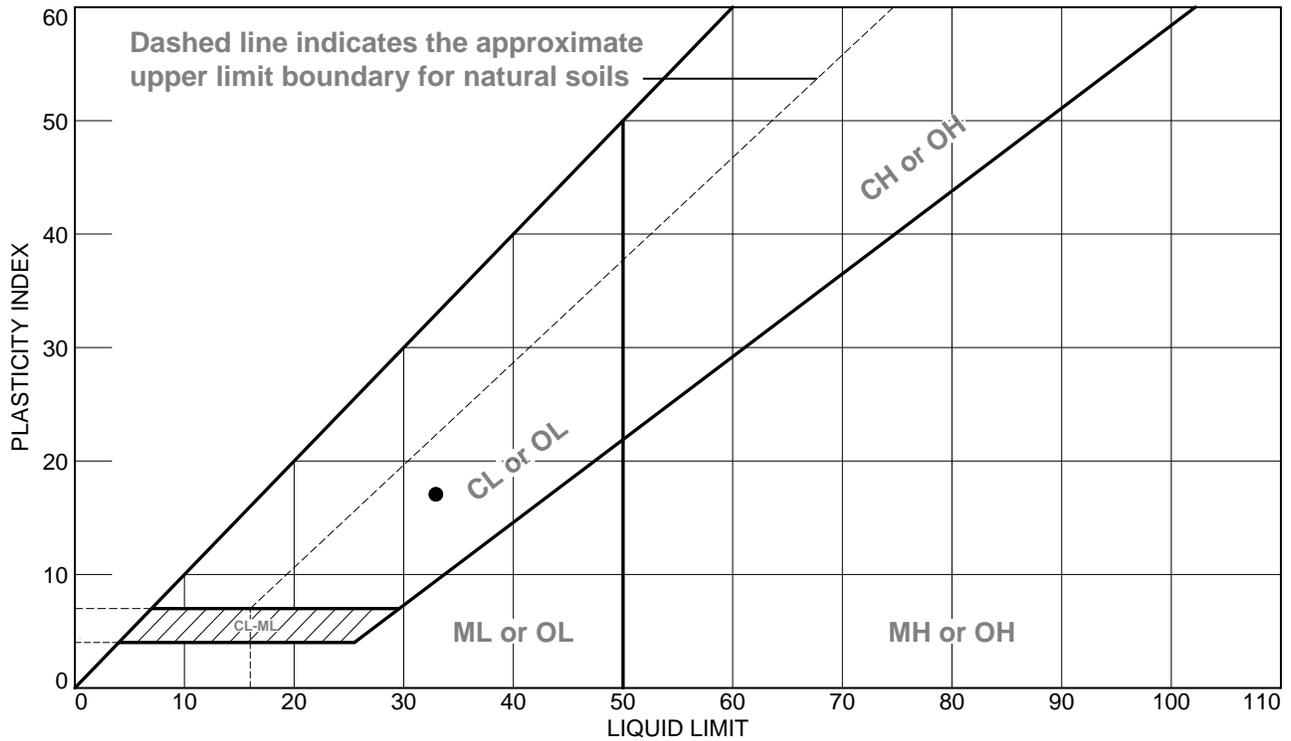
Project: McDade Classical School Annex
Chicago, Illinois

Project No.: 1012-325-19-01

Figure

Tested By: dw Checked By: jjw

ATTERBERG LIMITS TEST REPORT ASTM D 4318



SOIL DATA

	SOURCE	SAMPLE NO.	DEPTH	NATURAL WATER CONTENT (%)	PLASTIC LIMIT (%)	LIQUID LIMIT (%)	PLASTICITY INDEX (%)	LIQUIDITY INDEX	USCS
●	B-4	5	11.0 - 12.5 ft		16	33	17		

Weaver Consultants Group

Granger, Indiana

Client: Public Building Commission of Chicago

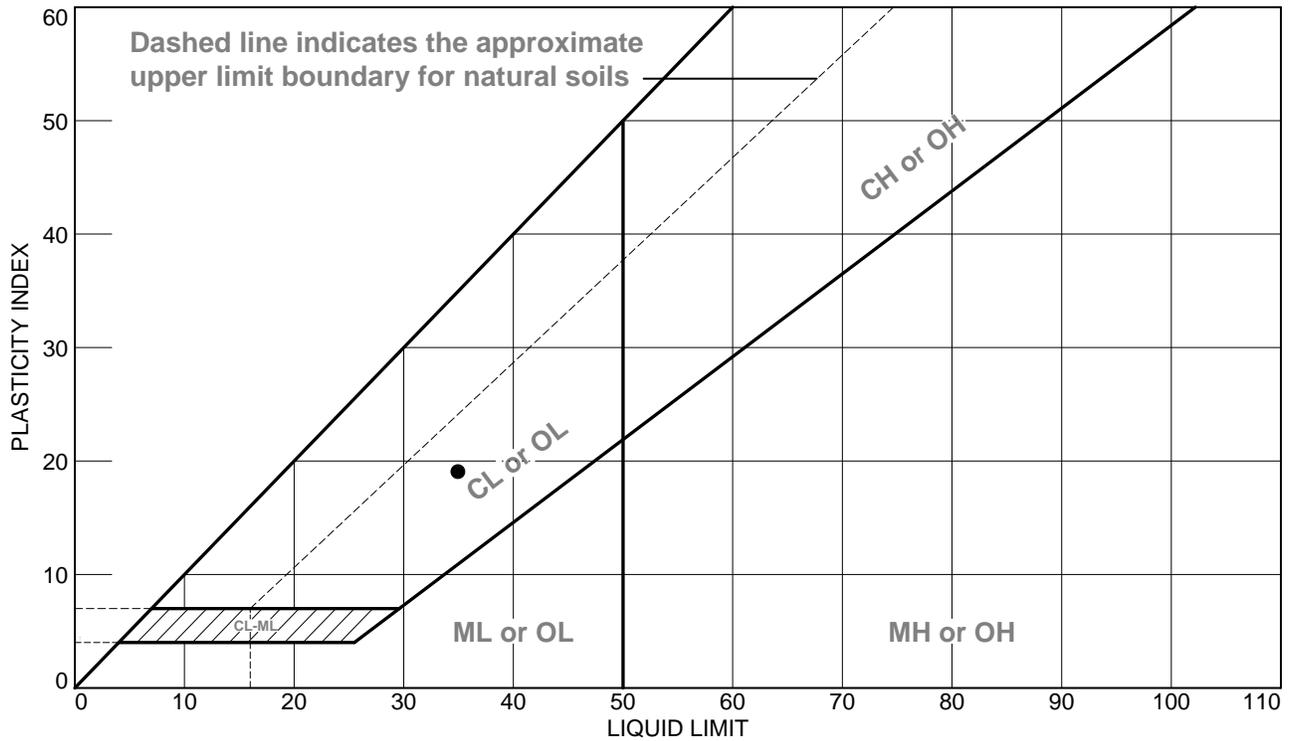
Project: McDade Classical School Annex
Chicago, Illinois

Project No.: 1012-325-19-01

Figure

Tested By: dw Checked By: jjw

ATTERBERG LIMITS TEST REPORT ASTM D 4318



SOIL DATA

SOURCE	SAMPLE NO.	DEPTH	NATURAL WATER CONTENT (%)	PLASTIC LIMIT (%)	LIQUID LIMIT (%)	PLASTICITY INDEX (%)	LIQUIDITY INDEX	USCS
● B-5	4	8.5 - 10.0 ft		16	35	19		

Weaver Consultants Group

Granger, Indiana

Client: Public Building Commission of Chicago

Project: McDade Classical School Annex
Chicago, Illinois

Project No.: 1012-325-19-01

Figure

Tested By: dw Checked By: jjw



UNCONFINED COMPRESSION TEST (QU-TEST) B-1 (11.0 - 13.0 ft)

Date: 11/2/2018

Project No.: 1012-377-19-01

Project: Mc Dade Classical School Annex

Visual Classification: Gray, Lean Clay, with sand (CL)

Sample Diameter: 2.833 inches

Area of Sample: 6.30 in²

Sample Height: 5.560 inches

Weight(g): 1225.5

Moisture: 20.4%

Sample Density: 133.1 pcf

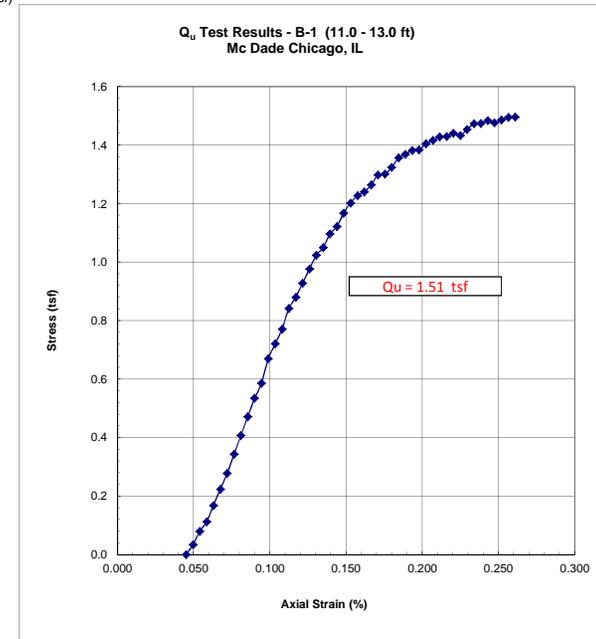
Dry Density: 110.6 pcf

L/D Ratio: 2.0

Correction: 1

Load Rate: 1200 sec/in

Time (sec)	Dial Reading	Strain		Load Dial Read.	Load (lbs)	(1-c)	Area (in ²)	Stress (psi)	Stress (tsf)
		Dial Reading x 10 ⁻³ (inches) ΔH	ε = ΔH/H (inch/inch)						
0	0	0.000	0.000	0	0.0	1.000	6.30	0.00	0.00
0:30	0.025	0.025	0.004	3	3.0	0.996	6.33	0.47	0.03
1:00	0.050	0.050	0.009	7	7.0	0.991	6.36	1.10	0.08
1:30	0.075	0.075	0.013	10	10.0	0.987	6.39	1.57	0.11
2:00	0.100	0.100	0.018	15	15.0	0.982	6.42	2.34	0.17
2:30	0.125	0.125	0.022	20	20.0	0.978	6.45	3.10	0.22
3:00	0.150	0.150	0.027	25	25.0	0.973	6.48	3.86	0.28
3:30	0.175	0.175	0.031	31	31.0	0.969	6.51	4.76	0.34
4:00	0.200	0.200	0.036	37	37.0	0.964	6.54	5.66	0.41
4:30	0.225	0.225	0.040	43	43.0	0.960	6.57	6.55	0.47
5:00	0.250	0.250	0.045	49	49.0	0.955	6.60	7.42	0.53
5:30	0.275	0.275	0.049	54	54.0	0.951	6.63	8.14	0.59
6:00	0.300	0.300	0.054	62	62.0	0.946	6.66	9.31	0.67
6:30	0.325	0.325	0.058	67	67.0	0.942	6.69	10.01	0.72
7:00	0.350	0.350	0.063	72	72.0	0.937	6.73	10.70	0.77
7:30	0.375	0.375	0.067	79	79.0	0.933	6.76	11.69	0.84
8:00	0.400	0.400	0.072	83	83.0	0.928	6.79	12.22	0.88
8:30	0.425	0.425	0.076	88	88.0	0.924	6.82	12.89	0.93
9:00	0.450	0.450	0.081	93	93.0	0.919	6.86	13.56	0.98
9:30	0.475	0.475	0.085	98	98.0	0.915	6.89	14.22	1.02
10:00	0.500	0.500	0.090	101	101.0	0.910	6.93	14.58	1.05
10:30	0.525	0.525	0.094	106	106.0	0.906	6.96	15.23	1.10
11:00	0.550	0.550	0.099	109	109.0	0.901	6.99	15.58	1.12
11:30	0.575	0.575	0.103	114	114.0	0.897	7.03	16.22	1.17
12:00	0.600	0.600	0.108	118	118.0	0.892	7.07	16.70	1.20
12:30	0.625	0.625	0.112	121	121.0	0.888	7.10	17.04	1.23
13:00	0.650	0.650	0.117	123	123.0	0.883	7.14	17.23	1.24
13:30	0.675	0.675	0.121	126	126.0	0.879	7.17	17.56	1.26
14:00	0.700	0.700	0.126	130	130.0	0.874	7.21	18.03	1.30
14:30	0.725	0.725	0.130	131	131.0	0.870	7.25	18.07	1.30
15:00	0.750	0.750	0.135	134	134.0	0.865	7.29	18.39	1.32
15:30	0.775	0.775	0.139	138	138.0	0.861	7.32	18.84	1.36
16:00	0.800	0.800	0.144	140	140.0	0.856	7.36	19.02	1.37
16:30	0.825	0.825	0.148	142	142.0	0.852	7.40	19.19	1.38
17:00	0.850	0.850	0.153	143	143.0	0.847	7.44	19.22	1.38
17:30	0.875	0.875	0.157	146	146.0	0.843	7.48	19.52	1.41
18:00	0.900	0.900	0.162	148	148.0	0.838	7.52	19.68	1.42
18:30	0.925	0.925	0.166	150	150.0	0.834	7.56	19.84	1.43
19:00	0.950	0.950	0.171	151	151.0	0.829	7.60	19.86	1.43
19:30	0.975	0.975	0.175	153	153.0	0.825	7.64	20.02	1.44
20:00	1.000	1.000	0.180	153	153.0	0.820	7.69	19.91	1.43
20:30	1.025	1.025	0.184	156	156.0	0.816	7.73	20.19	1.45
21:00	1.050	1.050	0.189	159	159.0	0.811	7.77	20.46	1.47
21:30	1.075	1.075	0.193	160	160.0	0.807	7.81	20.48	1.47
22:00	1.100	1.100	0.198	162	162.0	0.802	7.86	20.62	1.48
22:30	1.125	1.125	0.202	162	162.0	0.798	7.90	20.50	1.48
23:00	1.150	1.150	0.207	164	164.0	0.793	7.95	20.64	1.49
23:30	1.175	1.175	0.211	166	166.0	0.789	7.99	20.77	1.50
24:00	1.200	1.200	0.216	167	167.0	0.784	8.04	20.78	1.50
24:30	1.225	1.225	0.220	168	168.0	0.780	8.08	20.78	1.50
25:00	1.250	1.250	0.225	170	170.0	0.775	8.13	20.91	1.51
25:30	1.275	1.275	0.229	171	171.0	0.771	8.18	20.91	1.51
26:00	1.300	1.300	0.234	172	172.0	0.766	8.23	20.91	1.51
26:30	1.325	1.325	0.238	173	173.0	0.762	8.27	20.91	1.51
27:00	1.350	1.350	0.243	174	174.0	0.757	8.32	20.90	1.51
27:30	1.375	1.375	0.247	175	175.0	0.753	8.37	20.90	1.50
28:00	1.400	1.400	0.252	176	176.0	0.748	8.42	20.89	1.50
28:30	1.425	1.425	0.256	177	177.0	0.744	8.48	20.88	1.50
29:00	1.450	1.450	0.261	178	178.0	0.739	8.53	20.88	1.50





UNCONFINED COMPRESSION TEST (QU -TEST) B-3 (8.5-10.5 ft)

Date: 10/29/2018

Project No.: 1012-377-19-01

Project: Mc Dade Classical School Annex

Moisture: 21.8%

Sample Density: 133.1 pcf

Visual Classification: Gray, Lean Clay, trace sand (CL)

Sample Diameter: 2.824 inches

Sample Height: 5.554 inches

Weight(g): 1216.6

Dry Density: 109.3 pcf

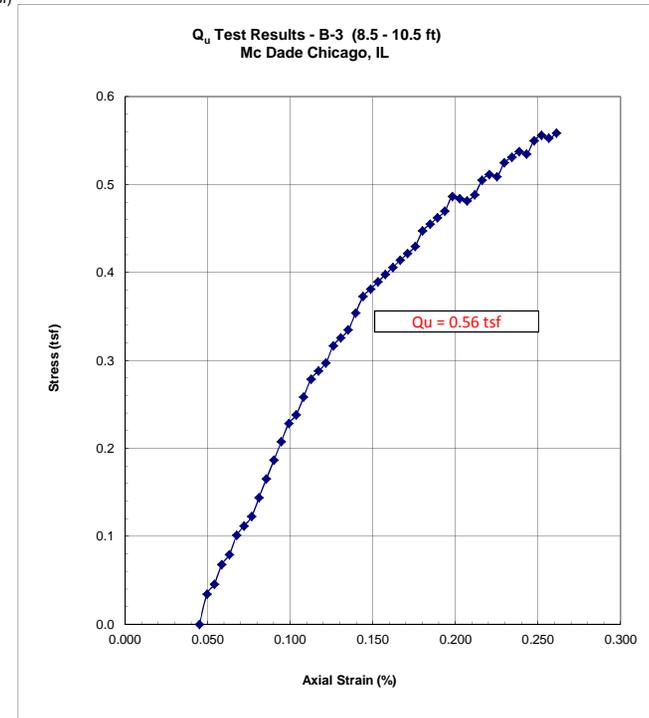
Area of Sample: 6.27 in²

L/D Ratio: 2.0

Correction: 1

Load Rate: 1200 sec/in

Time (sec)	Dial Reading	Dial Reading x 10 ³ (inches) ΔH	Strain ε = ΔH/H (inch/inch)	Load Dial Read.	Load (lbs)	(1-e)	Correc. Area (in ²)	Stress (psi)	Stress (tsf)
0	0	0.000	0.000	0	0.0	1.000	6.27	0.00	0.00
0:30	0.025	0.025	0.005	3	3.0	0.995	6.29	0.48	0.03
1:00	0.050	0.050	0.009	4	4.0	0.991	6.32	0.63	0.05
1:30	0.075	0.075	0.014	6	6.0	0.986	6.35	0.94	0.07
2:00	0.100	0.100	0.018	7	7.0	0.982	6.38	1.10	0.08
2:30	0.125	0.125	0.023	9	9.0	0.977	6.41	1.40	0.10
3:00	0.150	0.150	0.027	10	10.0	0.973	6.44	1.55	0.11
3:30	0.175	0.175	0.032	11	11.0	0.968	6.47	1.70	0.12
4:00	0.200	0.200	0.036	13	13.0	0.964	6.50	2.00	0.14
4:30	0.225	0.225	0.041	15	15.0	0.959	6.53	2.30	0.17
5:00	0.250	0.250	0.045	17	17.0	0.955	6.56	2.59	0.19
5:30	0.275	0.275	0.050	19	19.0	0.950	6.59	2.88	0.21
6:00	0.300	0.300	0.054	21	21.0	0.946	6.62	3.17	0.23
6:30	0.325	0.325	0.059	22	22.0	0.941	6.65	3.31	0.24
7:00	0.350	0.350	0.063	24	24.0	0.937	6.69	3.59	0.26
7:30	0.375	0.375	0.068	26	26.0	0.932	6.72	3.87	0.28
8:00	0.400	0.400	0.072	27	27.0	0.928	6.75	4.00	0.29
8:30	0.425	0.425	0.077	28	28.0	0.923	6.78	4.13	0.30
9:00	0.450	0.450	0.081	30	30.0	0.919	6.82	4.40	0.32
9:30	0.475	0.475	0.086	31	31.0	0.914	6.85	4.52	0.33
10:00	0.500	0.500	0.090	32	32.0	0.910	6.88	4.65	0.33
10:30	0.525	0.525	0.095	34	34.0	0.905	6.92	4.91	0.35
11:00	0.550	0.550	0.099	36	36.0	0.901	6.95	5.18	0.37
11:30	0.575	0.575	0.104	37	37.0	0.896	6.99	5.29	0.38
12:00	0.600	0.600	0.108	38	38.0	0.892	7.02	5.41	0.39
12:30	0.625	0.625	0.113	39	39.0	0.887	7.06	5.52	0.40
13:00	0.650	0.650	0.117	40	40.0	0.883	7.10	5.64	0.41
13:30	0.675	0.675	0.122	41	41.0	0.878	7.13	5.75	0.41
14:00	0.700	0.700	0.126	42	42.0	0.874	7.17	5.86	0.42
14:30	0.725	0.725	0.131	43	43.0	0.869	7.21	5.97	0.43
15:00	0.750	0.750	0.135	45	45.0	0.865	7.24	6.21	0.45
15:30	0.775	0.775	0.140	46	46.0	0.860	7.28	6.32	0.45
16:00	0.800	0.800	0.144	47	47.0	0.856	7.32	6.42	0.46
16:30	0.825	0.825	0.149	48	48.0	0.851	7.36	6.52	0.47
17:00	0.850	0.850	0.153	50	50.0	0.847	7.40	6.76	0.49
17:30	0.875	0.875	0.158	50	50.0	0.842	7.44	6.72	0.48
18:00	0.900	0.900	0.162	50	50.0	0.838	7.48	6.69	0.48
18:30	0.925	0.925	0.167	51	51.0	0.833	7.52	6.78	0.49
19:00	0.950	0.950	0.171	53	53.0	0.829	7.56	7.01	0.50
19:30	0.975	0.975	0.176	54	54.0	0.824	7.60	7.11	0.51
20:00	1.000	1.000	0.180	54	54.0	0.820	7.64	7.07	0.51
20:30	1.025	1.025	0.185	56	56.0	0.815	7.68	7.29	0.52
21:00	1.050	1.050	0.189	57	57.0	0.811	7.73	7.38	0.53
21:30	1.075	1.075	0.194	58	58.0	0.806	7.77	7.47	0.54
22:00	1.100	1.100	0.198	58	58.0	0.802	7.81	7.42	0.53
22:30	1.125	1.125	0.203	60	60.0	0.797	7.86	7.64	0.55
23:00	1.150	1.150	0.207	61	61.0	0.793	7.90	7.72	0.56
23:30	1.175	1.175	0.212	61	61.0	0.788	7.95	7.68	0.55
24:00	1.200	1.200	0.216	62	62.0	0.784	7.99	7.76	0.56





UNCONFINED COMPRESSION TEST (QU - TEST) B-5 (6.0-8.0 ft)

Date: 10/29/2018

Project No.: 1012-377-19-01

Project: Mc Dade Classical School Annex

Visual Classification: Gray, Lean Clay, with sand (CL)

Sample Diameter: 2.822 inches

Area of Sample: 6.25 in²

Moisture: 25.7%

Sample Density: 131.3 pcf

Dry Density: 104.4 pcf

Sample Height: 5.556 inches

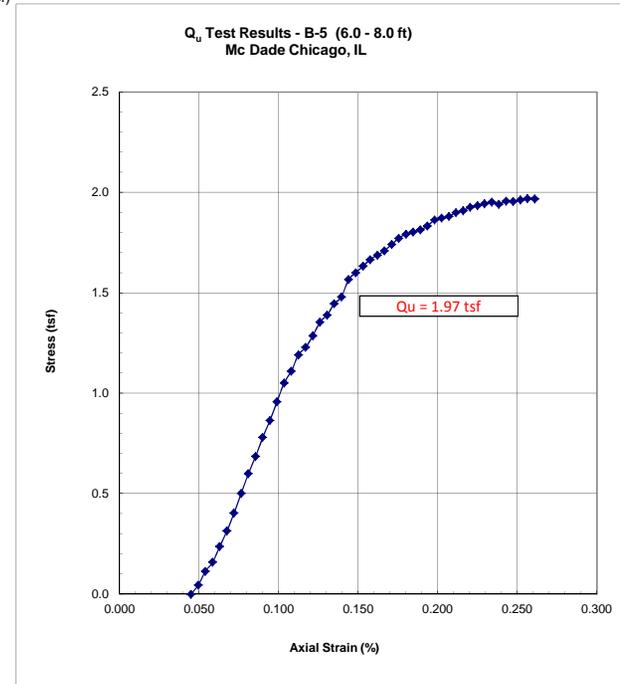
Weight(g): 1198.2

L/D Ratio: 2.0

Correction: 1

Load Rate : 1200 sec/in

Time (sec)	Dial Reading	Dial Reading x 10 ⁻³ (inches) ΔH	Strain (inch/inch) ε = ΔH/H	Load Dial Read.	Load (lbs)	(1-ε)	Area (in ²)	Stress (psi)	Stress (tsf)
0	0	0.000	0.000	0	0.0	1.000	6.25	0.00	0.00
0:30	0.025	0.025	0.004	4	4.0	0.996	6.28	0.64	0.05
1:00	0.050	0.050	0.009	10	10.0	0.991	6.31	1.58	0.11
1:30	0.075	0.075	0.013	14	14.0	0.987	6.34	2.21	0.16
2:00	0.100	0.100	0.018	21	21.0	0.982	6.37	3.30	0.24
2:30	0.125	0.125	0.022	28	28.0	0.978	6.40	4.38	0.32
3:00	0.150	0.150	0.027	36	36.0	0.973	6.43	5.60	0.40
3:30	0.175	0.175	0.031	45	45.0	0.969	6.46	6.97	0.50
4:00	0.200	0.200	0.036	54	54.0	0.964	6.49	8.33	0.60
4:30	0.225	0.225	0.040	62	62.0	0.960	6.52	9.51	0.69
5:00	0.250	0.250	0.045	71	71.0	0.955	6.55	10.84	0.78
5:30	0.275	0.275	0.049	79	79.0	0.951	6.58	12.01	0.86
6:00	0.300	0.300	0.054	88	88.0	0.946	6.61	13.31	0.96
6:30	0.325	0.325	0.058	97	97.0	0.942	6.64	14.61	1.05
7:00	0.350	0.350	0.063	103	103.0	0.937	6.67	15.44	1.11
7:30	0.375	0.375	0.067	111	111.0	0.933	6.71	16.55	1.19
8:00	0.400	0.400	0.072	115	115.0	0.928	6.74	17.07	1.23
8:30	0.425	0.425	0.076	121	121.0	0.924	6.77	17.87	1.29
9:00	0.450	0.450	0.081	128	128.0	0.919	6.80	18.81	1.35
9:30	0.475	0.475	0.085	132	132.0	0.915	6.84	19.31	1.39
10:00	0.500	0.500	0.090	138	138.0	0.910	6.87	20.08	1.45
10:30	0.525	0.525	0.094	142	142.0	0.906	6.90	20.56	1.48
11:00	0.550	0.550	0.099	151	151.0	0.901	6.94	21.76	1.57
11:30	0.575	0.575	0.103	155	155.0	0.897	6.97	22.22	1.60
12:00	0.600	0.600	0.108	159	159.0	0.892	7.01	22.68	1.63
12:30	0.625	0.625	0.112	163	163.0	0.888	7.05	23.14	1.67
13:00	0.650	0.650	0.117	166	166.0	0.883	7.08	23.44	1.69
13:30	0.675	0.675	0.121	169	169.0	0.879	7.12	23.75	1.71
14:00	0.700	0.700	0.126	173	173.0	0.874	7.15	24.18	1.74
14:30	0.725	0.725	0.130	177	177.0	0.870	7.19	24.61	1.77
15:00	0.750	0.750	0.135	180	180.0	0.865	7.23	24.90	1.79
15:30	0.775	0.775	0.139	182	182.0	0.861	7.27	25.05	1.80
16:00	0.800	0.800	0.144	184	184.0	0.856	7.30	25.19	1.81
16:30	0.825	0.825	0.148	187	187.0	0.852	7.34	25.47	1.83
17:00	0.850	0.850	0.153	191	191.0	0.847	7.38	25.87	1.86
17:30	0.875	0.875	0.157	193	193.0	0.843	7.42	26.01	1.87
18:00	0.900	0.900	0.162	195	195.0	0.838	7.46	26.14	1.88
18:30	0.925	0.925	0.166	198	198.0	0.834	7.50	26.39	1.90
19:00	0.950	0.950	0.171	200	200.0	0.829	7.54	26.52	1.91
19:30	0.975	0.975	0.175	203	203.0	0.825	7.58	26.77	1.93
20:00	1.000	1.000	0.180	205	205.0	0.820	7.62	26.89	1.94
20:30	1.025	1.025	0.184	207	207.0	0.816	7.67	27.00	1.94
21:00	1.050	1.050	0.189	209	209.0	0.811	7.71	27.11	1.95
21:30	1.075	1.075	0.193	209	209.0	0.807	7.75	26.96	1.94
22:00	1.100	1.100	0.198	212	212.0	0.802	7.80	27.19	1.96
22:30	1.125	1.125	0.202	213	213.0	0.798	7.84	27.17	1.96
23:00	1.150	1.150	0.207	215	215.0	0.793	7.88	27.27	1.96
23:30	1.175	1.175	0.211	217	217.0	0.789	7.93	27.37	1.97
24:00	1.200	1.200	0.216	218	218.0	0.784	7.98	27.34	1.97
24:30	1.225	1.225	0.220	219	219.0	0.780	8.02	27.30	1.97
25:00	1.250	1.250	0.225	220	220.0	0.775	8.07	27.27	1.96
25:30	1.275	1.275	0.229	222	222.0	0.771	8.11	27.36	1.97
26:00	1.300	1.300	0.234	222	222.0	0.766	8.16	27.20	1.96



APPENDIX C

Calculations

Objective:

Determine the allowable end bearing resistance for drilled shafts

Given:

Borings B-1 through B-5
Qp values ranged from 3.25 to over 4.5 tsf

Assumptions:

-Very stiff to hard clay will be the bearing layer
- Shaft diameter = 2.5 feet D = 2.5 ft

Base Resistance:

Use base resistance calculation method described in FHWA *Drilled Shafts: Construction Procedures and LRFD Design Methods*

$$q_{BN} = N_c^* s_u \quad \text{13-16}$$

where N_c^* = bearing capacity factor and s_u = mean undrained shear strength of the cohesive soil over a depth of $2B$ below the base. For cases where the shaft depth is at least 3 times the diameter and the mean undrained shear strength is at least 2,000 psf, the bearing capacity factor can be taken as 9.0. For smaller values of undrained shear strength, N_c^* can be approximated as a function of undrained shear strength as given in Table 13-2. Linear interpolation can be used for values between those tabulated. Note that it is unusual to locate the base of a drilled shaft in cohesive soil with s_u less than 2,000 psf when compression loads are supported.

$$\begin{aligned}
 S_u = Q_p/2, \quad Q_p \text{ (min.)} &= 3.75 \text{ tsf}, \quad Q_p = 7,500 \text{ psf}, \quad S_u = 3,750 \text{ psf} \\
 S_u &= 3750 \quad \text{psf} \quad \text{(Reference minimum } Q_p \text{ value)} \\
 N_c &= 9.0 \\
 q_B &= 33750 \quad \text{psf}
 \end{aligned}$$

Allowable Resistance:

**Use Factor of Safety of 3.0 for allowable resistances.

$$\begin{aligned}
 q_B &= \quad \quad \quad \mathbf{33750 \text{ psf}} \\
 q_{B - \text{allowable}} &= \quad \quad \quad \mathbf{11250 \text{ psf}}
 \end{aligned}$$

Settlement:

$$(q_m/q_u) = (\delta/\delta_u)^g$$

$q_m/q_u =$	0.333	(FS of 3)	($q_m/q_u =$ applied load/ unfactored capacity)
$\delta_u =$	0.25 ft		(Settlement required to mobilize resistance) (D/10, per Coduto 2016)
$g =$	0.5		(assumed for clay)
$\delta =$	0.02772225 ft 0.332667 in		

Conclusion:

Drilled shafts should be designed for a base resistance 10,00 psf and bear into the very stiff to hard clay layer with a Q_p of 3.75 tsf or greater. Side resistance should be neglected when considering

Settlement of the Drilled shaft was calculated to be less than 0.5 inches

References:

Federal Highway Administration, FHWA-MHI-10-016, Drilled Shafts: Construction Procedures and LRFD Design Methods, May 2010

Coduto, D. (2016). Foundation Design: Principles and Practices. Pearson.

Objective: Determine the squeeze potential of the clays for a 30-inch diameter drilled shaft at the McDade School Annex

Given: Borings B-1 through B-5
 ASCE Geotechnical Special Publication 312, Advances in Deep Foundations, 2005
 Budiman, Keifer, and Baker

Approach: Squeeze can occur if:

$$\frac{\sigma_v}{S_u} > \left(\frac{D + B}{4} \right) + 5$$

Squeeze Analysis:

Depth	CCD	Overburden		D/B ⁽²⁾	(D/B)/4 + 5	σ _v /S _u	Squeeze (Y or N)
		Pressure ⁽¹⁾	S _u (psf) ⁽³⁾				
7	2	875	1000	2.8	5.7	0.875	N
8.5	0.5	1062.5	560	3.4	5.85	1.897321	N
10	-1	1250	560	4	6	2.232143	N
11.5	-2.5	1437.5	750	4.6	6.15	1.916667	N
13	-4	1625	1500	5.2	6.3	1.083333	N
15	-6	1875	3250	6	6.5	0.576923	N
20	-11	2500	3750	8	7	0.666667	N
25	-16	3125	4500	10	7.5	0.694444	N

(1) Based on depth x assumed unit weight of 125 pcf

(2) B = 2.5 feet diameter

(3) = Based on minimum Q_p value at that depth
 = Based on minimum Q_u value at that depth

Conclusion:

Based on minimum shear strength values, we do not anticipate squeeze in the clay deposits.

Objective: Determine the design infiltration rate

Given: 12 inch ID casing ID= 12 inch
Assumptions: Porosity "N" = 0.20 n= 0.2
Approach: Chicago Stormwater Ordinance Manual

Test Data:

Elapsed Time (min)	Δ Time (min)	Water Decline (in)	Water Decline Volume (CF)	Cum. Water Vol. (CF)
0	0		0.000	0.000
1	1	0.5	0.033	0.033
2	1	0.0	0.000	0.033
3	1	0.0	0.000	0.033
5	2	0.3	0.016	0.049
10	5	0.3	0.016	0.065
15	5	0.0	0.000	0.065
30	15	0.0	0.000	0.065
45	15	0.1	0.008	0.074
60	15	0.4	0.025	0.098
75	15	0.1	0.008	0.106
90	15	0.1	0.008	0.115
TOTAL		1.75		0.115

r = 0.5 ft

$i_n = 1.15741E-05$ fps

z = 2 ft

$y_t = 0.145833333$ ft

$i_w = 5.14403E-06$ fps

$$i_w = \frac{i_n r^2}{\pi(r+x)^2}$$

L = 0.324074074 ft

$$L = \frac{y_t \pi r^2}{n\pi(r+x)^2}$$

K = 7.17295E-07 fps

$$K = \frac{i_w L}{(z+L)}$$

0.030987162 in/hr

Design K = 0.030987162 in/hr

APPENDIX D

Qualifications

GENERAL QUALIFICATIONS

This report has been prepared at the request of our client for his use on this project. The work, including the field work, laboratory testing, and engineering analysis, was performed in accordance with generally accepted Geotechnical Engineering practices. For this study, we were not retained to address environmental or land use restriction concerns. This warranty is in lieu of all other warranties either expressed or implied.

This report may not contain sufficient information for purposes of other parties or other uses. Should there be any sufficient differences in structural arrangement, loading or location of the structure, our analysis should be reviewed.

The analysis, conclusions, and recommendations contained in our report are based on site conditions as they existed at the time of our exploration and further assume that the borings are representative of the subsurface conditions throughout the site.

If during construction, different subsurface conditions from those encountered during our exploration are observed or appear to be present beneath excavations, we must be advised promptly so that we can review these conditions and reconsider our recommendations where necessary.

If there is a substantial lapse of time between the submission of our report and the start of work at the site, or if conditions have changed due to natural causes or construction operations at or adjacent to the site, we urge that our report be reviewed to determine the applicability of the conclusions and recommendations considering the changed conditions and time lapse.

We urge that we be retained to review those portions of the plans and specifications that pertain to earthwork and foundations to determine whether they are consistent with our recommendations. In addition, we are available to observe construction, particularly the compaction of structural backfill and preparation of the foundations, and such other field observations as may be necessary.

In order to fairly consider changed or unexpected conditions that might arise during construction, we recommend the following verbiage to be included in the project contract.

STANDARD CLAUSE FOR UNANTICIPATED SUBSURFACE CONDITIONS

The owner has had a subsurface exploration performed by a Geotechnical consultant, the results of which are contained in the consultant's report. The consultant's report presents his conclusions on the subsurface conditions based on his interpretation of the data obtained in the exploration. The contractor acknowledges that he has reviewed the consultant's report and any addenda thereto, and that his bid for earthwork operations is based on the subsurface conditions as described in that report. It is recognized that a subsurface exploration may not disclose all conditions as they actually exist and further, conditions may change, particularly groundwater conditions, between the time of a subsurface exploration and the time of earthwork operations. In recognition of these facts, this clause is entered in the contract to provide a means of equitable additional compensation for the contractor if adverse unanticipated conditions are encountered and to provide a means of rebate to the owner if the conditions are more favorable than anticipated.

Should the contractor encounter conditions that are different than those anticipated by the Geotechnical consultant's report at any time during construction operations, he shall immediately (within 24 hours) bring this fact to the owner's attention. If the owner's representative on the construction site observes subsurface conditions which are different than those anticipated by the consultant's report, he shall immediately (within 24 hours) bring this fact to the contractor's attention. Once a fact of unanticipated conditions has been brought to the attention of either the owner or the contractor, and the consultant has concurred, immediate negotiations will be undertaken between the owner and the contractor to arrive at a change in contract price for additional work or reduction in work. The contractor agrees that the following unit prices would apply for additional or reduced work under the contract. For changed conditions in which unit prices are not provided, the additional work shall be paid for on a time and material basis.