CGC, Inc.

Construction • Geotechnical
Consulting Engineering/Testing

August 20, 2021
C21051-10

Ms. Joanna OBrien
City of Madison – Engineering Department
210 Martin Luther King, Jr. Boulevard, Room 115
Madison, WI 53710

Re: Geotechnical Exploration Report - Updated
Proposed Storm Sewer Box Culvert
Mendota-Grassman Greenway
Madison, Wisconsin

Dear Ms. OBrien:

Construction • Geotechnical Consultants, Inc. (CGC) has completed the geotechnical exploration program for the project referenced above. The purpose of this exploration program was to evaluate the subsurface conditions within the proposed construction area and to provide geotechnical recommendations regarding storm sewer culvert design/construction and roadway reconstruction. An initial report dated August 20, 2021, was issued prior to the request for additional borings at Universty Avenue and support structures to address existing underlying utility protection. This report should be considred comprehensive and the initial report can be set aside. An electronic copy of this report is provided for your use, and a paper copy can be provided upon request.

PROJECT AND SITE DESCRIPTION

We understand that new precast concrete box culverts will be installed to replace existing steel and concrete culverts which cross below Camelot Drive just northwest of Baker Avenue, and below University Avenue between Baker Avenue and Hickory Hollow Drive. The box culverts will allow stormwater to travel beneath the roads and along an existing drainage/greenway towards Lake Mendota to the northeast.

Preliminary plans provided to CGC indicate that the Camelot Drive culvert will be 188 ft long and consist of dual (side by side) rectangular culverts each with an inside opening of 10 ft wide by 4 ft tall. The base of culvert elevations will be near EL 850 ft and 851 ft at the east and west ends of the culvert, respectively, which is about 6 ft below the pavement surface near the center of the road near EL 857 ft. The base of the culverts will be between about 3 to 4 ft below existing site grades outside of the roadway. The sides and bottoms of the structures will be approximately 12 in. thick.

The University Avenue culvert will be 140 ft long and consist of dual rectangular culverts each with an inside opening of 9 ft wide by 6 ft tall. The base of culvert elevations will be near EL 862.8 ft and 863.1 ft at the north and south ends of the pipe, respectively, which is about 19 ft below the pavement surface near the center of the road near EL 882 ft. The base of the culvert will be between about 3 to 4 ft below existing site grades outside of the roadway. The sides and bottoms of the structures will be approximately 12 in. thick.

2921 Perry Street, Madison WI 53713
Telephone: 608/288-4100
FAX: 608/288-7887

Preliminary plans show that roadway grades will remain unchanged or be minimally altered following installation of the new culverts and the pavement will be replaced in-kind.

SUBSURFACE CONDITIONS

A total of four Standard Penetration Test (SPT) soil borings were completed for this project. Borings 1 and 2 were completed along Camelot Drive to a depth of 25 ft, at which point auger refusal occurred on possible bedrock or cobbles/boulders. Borings 3 and 4 were completed along University Avenue to depths of 42 to 43.8 ft, at which point drill-string advancement was very slow under high down-pressure (i.e. "practical refusal" occurred on apparent bedrock). The borings were drilled by Badger State Drilling (under subcontract to CGC) on July 21 (Borings 1 and 2) and October 8 and 10 (Borings 3 and 4), 2021 using a truck-mounted, rotary drill-rig equipped with hollow-stem augers, mud-rotary tools and an automatic SPT hammer. The borings were located in the field by CGC and ground surface elevations at the boring locations were estimated using preliminary plans provided, which contain 1-ft contour lines. Therefore, the elevations should be considered approximate (± 1 ft). The boring locations are shown in plan on the Soil Boring Location Map attached in Appendix B.

The subsurface conditions at the boring locations were similar and a generalized profile includes the following strata, in descending order:

- About 11 to 12 in. of ***pavement layers***, including 4 in. of *asphalt* over 7 to 8 in. of *base course*, or 8 in. of ***topsoil fill*** at Boring 3; underlain by

- About 5 to 17 ft of existing roadway embankment ***fill***, comprised of mixture of medium stiff to stiff clay and granular soils; over

- Loose to very dense ***silt*** and ***sand*** having variable gravel contents, and extending to auger refusal on possible bedrock, or potentially cobbles/boulders within the granular deposits at a depth of about 25 to 44 ft below existing site grades. Scattered soft to stiff ***lean clay*** seams were encountered within the silt and sand deposits at Borings 3 and 4.

Groundwater was encountered in the Camelot Drive borings at depths of about 8.5 ft below current site grades during drilling, corresponding to an approximate groundwater elevation of 849.5 ft. Water level readings approximately 30 minutes and 3 hrs after the completion of drilling at B-2 and B-1, respectively, showed that groundwater levels rose to a depth of 6 ft, or approximately EL 852 ft. At University Avenue, groundwater levels were encountered at 12 to 14 ft below current site grades, corresponding to an approximate elevation of 867 and 869 ft at Borings 3 and 4, respectively. Longer-term water level readings were obscured due to the use of drilling fluid.

Groundwater levels on these sites are generally expected to be influenced by the water level in nearby Lake Mendota, as well as due to seasonal variations in precipitation (e.g., flow through the greenway). For reference, on the days the soil borings were conducted, the water level in Lake Mendota was recorded at about EL 850.01 ft (July 21, 2021) and 850.05 (October 10, 2021), according to the Dane County Land & Water Resources Department *Lake Levels & Information* online platform. A more detailed description of the site soil and groundwater conditions is presented on the Soil Boring Logs attached in Appendix B.

DISCUSSION AND RECOMMENDATIONS

Subject to the limitations discussed below and based on the subsurface exploration, it is our opinion that the site appears generally suitable for box culvert replacement and support. *Note that dewatering and subgrade stabilization will likely be required during culvert installation due to the presence of groundwater at or just below proposed structure elevations.* In addition, due to the presence of existing utilities which will remain below the planned culvert crossings, special culvert support considerations will be required. The following subsections provide our recommendations for box culvert and storm sewer foundation design/construction and pavement reconstruction.

1. Box Culvert Design

A. General

Based on bottom of culvert elevations and the groundwater conditions encountered in the borings, *dewatering should be anticipated such that bearing soils do not become disturbed during excavation or structure installation.* In addition, a plan to divert water from the upstream side of the crossings away from the excavation will also likely be necessary during and after periods of precipitation.

We recommend that groundwater levels be lowered at least 2 ft below the bottom of the planned excavation depths (e.g., bottom of stabilization layer excavation) *in advance* of excavation. For excavations extending less than about 1 to 2 ft below the groundwater table, dewatering can likely be accomplished using pumps operating from filtered sumps. Where excavations extend more than 2 ft below the groundwater table, effective dewatering generally requires a series of deep wells or a vacuum well-point system. Dewatering means and methods are the responsibility of the contractor.

The medium dense silt and clayey soils anticipated at the bottom of the excavations are expected to be difficult to dewater and can be susceptible to disturbance due to typical construction activity. We therefore recommend including a minimum 12-in. thick layer of compacted clear stone at the base of the excavation to help protect the subgrade from disturbance and create a working platform. Shallow sump-pumps can also be placed in the clear stone to provide supplemental dewatering, if needed. The clear stone should be enveloped in a non-woven geotextile fabric (e.g., Mirafi 160N or equivalent) to prevent soil migration into the clear stone layer.

The foundation analysis for the culverts was completed in general accordance with procedures in Chapter 36 of the WisDOT *LRFD Bridge Manual*, which is largely based upon and references procedures in the AASHTO *LRFD Bridge Manual.*

Because the replacement structures will largely be constructed within the existing embankments, the weight of the new concrete structure and soil/pavement cover over (and around) will be less than the weight of the removed embankment fill materials. Therefore, the subgrade soils will generally experience a minimal increase (potential net decrease) in pressure and bearing capacity thus settlement below the structures is generally not expected to be an issue. Further details regarding bearing resistance and settlement estimates are discussed in the following sections.

General geotechnical recommendations for design and installation of the structures include the following:

- The unit weight of soil placed above the culverts should be taken as 120 lb/cu ft (pcf), per WisDOT *Bridge Manual*, Chapter 36.

- Recommended parameters for calculating lateral earth pressures are as follow:
  - Coefficient of lateral earth pressure, $K_o = 0.5$ for *at-rest* conditions.
  - Angle of internal friction = 30° for granular backfill
- Unit weight for a typical granular backfill would be the same as soil above the structure, 120 pcf.

- To control infiltrating surface water following installation of the culverts, standard drainage provisions should be included, such as backfilling with reasonably free-draining (similar to WisDOT Grade 1) granular backfill. The existing embankment fill soils which will be removed are not considered suitable for re-use as backfill. Therefore, importing of suitable granular backfill soils, which is a typical requirement for City projects, will be necessary.

- A minimum 12-in. thick layer of compacted 1-in. crushed clear stone is recommended below the base of the structures to protect the subgrade from disturbance, aid in dewatering efforts and act as a working platform during construction as previously discussed. The stone layer should be compacted and enveloped on the top, bottom and sides with non-woven geotextile fabric (e.g., Mirafi 160N or equivalent) to prevent migration of surrounding soil into the void spaces of the stone. The stone stabilization layer should be installed in small sections with the subgrade covered in fabric and stone shortly after the subgrades are exposed in order to reduce the potential for degradation from water.

- Appropriate scour protection should be provided to prevent undermining of the box culvert.

B. Calculated Bearing Resistance - Conventional Structure Support

While the medium dense natural silt and loose to medium dense natural sands expected at the base of the stabilization layer can provide a higher factored bearing resistance, to account for potential variations in subgrade conditions during construction, and because the actual contact pressure of the culvert is anticipated to be relatively low, we recommend that a factored bearing resistance of **2,000 psf** be used for design. Note that this value is above the estimated increase in pressure below the new structures, as described above, so the Capacity to Demand Ratio (CDR) will exceed 1.0. The recommended bearing resistance is contingent on unsuitable existing fill and softer natural clay soils being removed as Excavation Below Subgrade (EBS), as well as the subgrade being effectively dewatered *in advance* of excavation (if required).

Although generally similar soil conditions were observed in the soil borings, some variability in subgrade conditions should be expected. Therefore, the quality and suitability of the soil exposed at storm sewer subgrade elevations should be carefully evaluated for culvert support at the time of foundation excavation. We recommend that a CGC geotechnical engineer or a qualified construction inspector be present during culvert excavation to check whether suitable bearing conditions are present at the base of the culvert or EBS excavation, and to provide corrective measures, if necessary. Based on the borings and proposed bottom of culvret elevations, EBS of 1 to 2 ft of existing fill may be required in the vicinity of Boring 1, and removal of 1 to 2 ft of soft to very soft clay may be required in the vicinity of Borings 3 and 4.

The primary concern with silt and clay soils is their tendency to soften/loosen, lose bearing capacity and increase the potential for settlement when saturated. The width of EBS, where required, should extend about 1 ft beyond the base of the structure on each side. Where the thickness of unsuitable soil removed (where required) below the culvert exceeds the recommended minimum of 12 in. of clear stone described above, the soil should be removed as EBS and the subgrade restored with additional clear stone (enveloped in geotextile) compacted with a large vibratory pate compactor (or hoe-pak) until no deflection is evident. As an alternative to the clear stone layer, a 4 to 6-in. thick layer of "lean mix" concrete having a minimum 28-day compressive strength of 1000 psi can be used to protect the subgrade during culvert installation, as well as to restore subgrade in areas where EBS is required. Similar to the fabric/clear stone alternative, lean mix should be applied to the subgrade shortly after being exposed to reduce the potential for subgrade disturbance. *In addition to the recommended minimum 12-in. stabilization layer, we recommend the project budget include a contingency for additional EBS/stabilization.*

C. Estimated Settlement

As noted previously, because the new structures will be installed within the existing roadway embankment and because the existing grades outside of the road are also higher than proposed bottom of culvert elevations, minimal net increase in pressure is expected due to the weight of the new structures and soil/pavement cover above it. Because of this, and provided the subgrade is prepared as described in detail above, total settlement less than about 1 in. is expected where culverts bear directly on soil (or stabilization layer over soil). Typically, differential settlement will be equal to about half of the total settlement, or less than about 0.5 in.

D. Special Considerations – Culvert Support Over Existing Utility Crossing

It is our understanding that Madison Metropolitan Sewerage District (MMSD) has identified four sanitary sewer and storm sewer crossings which will be located below the new culvert crossings. Although we expect that the net increase in stress applied by the new structures will be minimal, we understand that there is concern regarding stress induced upon, and settlement of the existing utilities which will remain in-place below the new structures. Therefore, in order to limit (or eliminate) potential negative impacts of the new culverts, we are providing additional foundation support recommendations for portions of the proposed structures which cross above the existing utilities.

It is our opinion that helical piers or micropiles could be considered for support of the culverts at existing utility crossings. Helical piers and micropiles are deep foundation systems designed to transfer loads vertically to bear within deeper soils or bedrock. In this case, the deep foundation systems would be utilized in combination with a structural, cast-in-place concrete pier/pile cap or slab installed over the utility and below the pre-cast culvert in order to transfer the loads to greater depths while bypassing the existing utilities. The following subsections provide specific recommendations regarding helical piers and micropile design and installation. While driven piles could also be considered, helical piers and micropiles are typically more economically favorable. In addition, smaller equipment (e.g., excavator or skidsteer) compared to pile driving equipment is generally required, which also may be favorable for these projects. Driven piles are not further discussed in this report, but we can provide additional details, if desired.

1. Helical Piers

Helical pier capacity will vary depending on the number and size of helices, depth of installation and bearing stratum. In general, we anticipate that helical piers will develop adequate capacity within the medium dense to very dense natural granular soils underlying each of the sites. Somewhat variable depths should be expected in order to develop target capacities, with actual installation depth dependent on required capacity and helix configuration.

The installation torque is correlated with capacity, although static load tests can also be completed to confirm the ultimate and allowable capacities. A minimum factor of safety of 2.0 to 3.0 is generally used for helical pier design. If a factor of safety of 2.0 is used to determine the allowable helical pier capacity, we recommend that at least three static load tests be performed to confirm the helical pier design satisfies the project requirements. The static load tests should be performed on piers installed to similar installation depths and torques as production piers. Additionally, the torque of each pier should be monitored during installation to document that each pier is torqued to the minimum torque established by the static load tests or empirical correlations to ultimate capacity. If static load tests are not performed, we recommend using a minimum factor of safety of 2.5 to 3.0 in determining the allowable capacity, and the installation torque of each pier should be monitored, which is empirically correlated to the ultimate capacity.

Since helical piers are proprietary, the helical pier capacities should be considered approximate, and the helical pier installer should determine the appropriate helices configuration and depth necessary to satisfy project requirements. Soil stratigraphy and properties should be expected to vary across the site, as shown in the borings, which will affect helical pier installation depths to achieve given capacity. Actual design depths should be determined by an independent analysis using specific helix configurations proposed on the project.

Other helical pier considerations include the following:

- *Prospective helical pier contractors should be aware of the presence of cobbles and boulders within the deeper, predominantly dense natural sand strata, that may impact helical pier installation.* The helical pier installer should have provisions to deal with the presence of potential obstructions. If obstructions are encountered, removing obstructions with an excavator would be one method to deal with the obstructions. Using smaller diameter helix configuration may also be necessary to assist in the installation process, but may require deeper piers to develop capacity.

- The loose silt and softer clay soils have relatively low lateral capacity. As such, round helical pier shafts, which have higher resistance to buckling, are recommended over square shafts. A buckling analysis should be completed to check that the pier shaft has adequate buckling resistance.

II. Micropiles

Micropiles are a drilled foundation system that can be advanced into soil (or bedrock) through the use of different drilling techniques. Micropile diameters typically range from about 5 to 9 in., and the upper part of the borehole is usually cased, with the bottom part of the hole not cased. After drilling, a high-strength threaded steel bar is placed in the borehole and grouted in-place. Grout placement can occur under pressure to improve the bond strength between the grout and formation, which is recommended in the deeper granular soils or bedrock where capacity is likely to be developed. End bearing of micropiles should be neglected.

Other items that should be considered in the micropile design:

- The minimum spacing between micropiles should be the larger of 30 in. or three micropile diameters.

- Appropriate corrosion protection should be provided since this application is considered a permanent installation.

2. Pavement Reconstruction

A. General

In our opinion, the mixed clayey to sandy fill materials encountered beneath the base course may prove generally satisfactory for proposed roadway support beyond the limits of culvert construction and associated backfill. Where areas of softer clays are encountered (such as where pocket penetrometer values are near 1 tsf or less), they may need to be undercut/removed and replaced with granular fill or additional base course. Furthermore, significant construction traffic could destabilize the existing materials and increase the potential for undercuts. Granular materials should be thoroughly compacted and evaluated for stability before the placement of additional fill and/or base course. Pockets of excessively organic soil should also be removed. Standard earthwork-related techniques that should be used during roadway construction include:

- Proof-rolling of the exposed subgrades;
- Undercutting and/or stabilization in soft areas; and
- Compaction control of fill/backfill materials.

B. Pavement Design

Clays will control the pavement design, as we anticipate that the pavement subgrades will generally consist of fill materials containing clay. The following *generalized* parameters should be used to develop the design pavement section:

AASHTO classification	A-6
Frost group index	F-3
Design group index	14
Soil support value	3.9
Subgrade modulus, k (pci)	125
Estimated percent shrinkage	20 - 30
Estimated CBR value	2-5

Assuming University Avenue is considered a local business/arterial street, we estimate it could receive between 51 to 275 ESALs (18,000 pound Equivalent Single Axle Loads). A typical pavement design per WisDOT Standard Specifications should meet MT (E-3) requirements. Thicker pavements could be necessary pending traffic counts. If Camelot Drive experiences traffic volumes of up to 3000 cars and 100 trucks per day per design lane, a typical pavement design per WisDOT Standard Specifications should meet LT (E-1) requirements.

C. Compaction Requirements

Regarding backfilling along and above the proposed box culverts, we anticipate that imported sands will be necessary which is a typical requirement for City projects. On-site sands could be considered for reuse as backfill but they should be separated from clay soils and selectively stockpiled. We recommend that at least a level of 95% compaction be achieved within backfill material placed within the final 3 feet below finished subgrades (including undercut backfill - if any), with 90% compaction required at depths greater than 3 feet. The specified levels of compaction are based on modified Proctor methods (ASTM D1557). In addition, the backfill material should be placed and compacted in accordance with our Recommended Compacted Fill Specifications presented in Appendix C.

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:

- 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 during construction. Fill/backfill 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 standards.

- Based on observations made during the field exploration, groundwater should be anticipated during culvert installation. Temporary cofferdams/storm sewer diversions and dewatering will be required so that culvert installation can occur "in the dry", as discussed in detail above. Additional seeping groundwater or infiltrating surface water accumulating at the base of the excavations should be controlled and removed using pumps operating from filtered sump pits. A layer of clear stone enveloped in a geotextile fabric should be placed below the base of the culvert and utility excavations to create a working platform, as discussed above, and also to assist in dewatering efforts.

RECOMMENDED CONSTRUCTION MONITORING

The level of care exercised during culvert subgrade preparation will largely determine the quality of the foundation subgrades. To check that earthwork and foundation construction proceed in accordance with our recommendations, qualified construction inspectors should monitor the following operations:

- Subgrade preparation;
- Placement of compacted fill/backfill; and
- Concrete and asphalt placement.

* * * * *

We trust this report addresses your present needs. General limitations regarding the conclusions and opinions presented in this report are discussed in Appendix B. If you have any questions, please contact us.

Sincerely,

CGC, Inc.

Eric S. Fair
Senior Staff Engineer/Geologist

Alex J. Bina, P.E.
Consulting Professional

Encl: Appendix A - Subsurface Exploration
Appendix B - Soil Boring Location Maps (2)
Logs of Test Borings (4)
Log of Test Boring-General Notes
Unified Soil Classification System
Appendix C - Recommended Compacted Fill Specifications
Appendix D - Document Qualifications

APPENDIX A

SUBSURFACE EXPLORATION

APPENDIX A

SUBSURFACE EXPLORATION

A total of four Standard Penetration Test (SPT) soil borings were completed for this project. Borings 1 and 2 were completed along Camelot Drive to a depth of 25 ft, at which point auger refusal occurred on possible bedrock or cobbles/boulders. Borings 3 and 4 were completed along University Avenue to depths of 42 to 43.8 ft, at which point practical refusal occurred on apparent bedrock. The borings were drilled by Badger State Drilling (under subcontract to CGC) on July 21 (Borings 1 and 2) and October 8 and 10 (Borings 3 and 4), 2021 using a truck-mounted rotary drill-rig equipped with hollow-stem augers, mud-rotary tools and an automatic SPT hammer. The borings were located in the field by CGC and ground surface elevations at the boring locations were estimated using preliminary plans provided, which contain 1-ft contour lines. Therefore, the elevations should be considered approximate (± 1 ft).

Standard penetration test (SPT) soil samples were obtained at 2.5-foot intervals to approximate invert elevations and then continuously for 15 feet as requested. Sampling beyond 15 feet below invert elevations occurred at 5-foot intervals. The soil samples were obtained in general accordance with specifications for standard penetration testing, ASTM D 1586. The specific procedures used for drilling and sampling are described below.

1. <u>Boring Procedures between Samples</u>

   The boring is extended downward to the next sample interval by a hollow-stem auger or rotary-bit string to to the maximum depth explored.

2. <u>Standard Penetration Test and Split-Barrel Sampling of Soils</u>
   (ASTM Designation: D 1586)

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 then driven 12 inches. The number of blows required to drive the sampler the final 12 inches is recorded on the log of boring and is known as the Standard Penetration Resistance.

During the field exploration, the driller visually classified the soil and prepared a field log. Water level observations were made in the boring during and shortly after drilling which are shown at the bottom of the individual boring logs. Upon completion of drilling, the borings were backfilled to satisfy WDNR regulations and the soil samples delivered to our laboratory for visual classification. The soils were classified by CGC using the Unified Soil Classification System. The final logs prepared by the engineer and a description of the Unified Soil Classification System are presented in Appendix B.

APPENDIX B

SOIL BORING LOCATION MAPS (2)
LOGS OF TEST BORINGS (4)
LOG OF TEST BORING-GENERAL NOTES
UNIFIED SOIL CLASSIFICATION SYSTEM

Legend

⌖ Denotes Boring Location

N

Scale: Reduced

Notes

1. Soil Borings performed by Badger State Drilling in July 2021
2. Boring locations are approximate

Date: 7/2021	CGC, Inc.	**Soil Boring Location Map** **Mendota-Grassman Greenway** **Camelot Drive** **Madison, WI**
Job No. C21051-10		

Legend

⊕ Denotes Boring Location

N

Scale: Reduced

Notes
1. Soil Borings performed by Badger State Drilling in October 2021
2. Boring locations are approximate

Date: 10/2021	CGC, Inc.	**Soil Boring Location Map**
Job No. C21051-10		**Mendota-Grassman Greenway** **University Avenue** **Madison, WI**

CGC Inc.

LOG OF TEST BORING

Project: Mendota-Grassman Greenway

Location: Madison, WI

Boring No. 1
Surface Elevation (ft) 858±
Job No. C21051-10
Sheet 1 of 1

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

SAMPLE No.	TYPE	Rec (in.)	Moist	N	Depth (ft)	VISUAL CLASSIFICATION and Remarks	SOIL PROPERTIES qu (qa) (tsf)	W	LL	PL	LI
						4 in. Asphalt Pavement/8 in. Base Course					
1		18	M	11		FILL: Medium Dense Brown and Dark Brown Sand with Clay to 3'					
2		18	M	4		Soft to Medium Stiff Bluish-Gray Sandy Clay to 5'	(0.5)				
					5	Medium Dense Sand with Gravel to 8'					
3		18	M/W	27							
4		18	W	20		Medium Dense, Brown Sandy SILT, Trace to Little Gravel and Clay (ML)					
5		20	W	21	10						
6		24	W	41		Dense to Very Dense, Brown Silty Fine SAND, Some Gravel, Trace Clay (SM)					
7		20	W	58/10"	15						
8		10	W	8		Loose, Light Brown Fine SAND, Some Silt, Trace Gravel (SM)					
9		24	W	18		Medium Dense, Brown Fine to Medium SAND, Some Silt and Gravel, Scattered Cobbles and Boulders (SM)					
10		24	W	19	20						
11		15	W	49							
12		0	-	50/2"	25	Presumed Bedrock (Hard Drilling)					
					30	End Boring at 25 ft Due to Auger Refusal on Presumed Bedrock/Possible Boulder Borehole backfilled with bentonite chips and asphalt patch					

WATER LEVEL OBSERVATIONS

While Drilling	∇ 8.5'	Upon Completion of Drilling			6'
Time After Drilling					3 Hour
Depth to Water					6' ▼
Depth to Cave in					8'

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

GENERAL NOTES

Start 7/21/21 End 7/21/21
Driller BSD Chief MC Rig CME-55
Logger GB Editor ESF
Drill Method 2.25" HSA; Autohammer

LOG OF TEST BORING

CGC Inc.

Project: Mendota-Grassman Greenway
Location: Madison, WI

Boring No. 2
Surface Elevation (ft) 858±
Job No. C21051-10
Sheet 1 of 1

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

SAMPLE No.	Rec (in.)	Moist	N	Depth (ft)	VISUAL CLASSIFICATION and Remarks	qu (qa) (tsf)	W	LL	PL	LI
					4 in. Asphalt Pavement/7.5 in. Base Course					
1	10	M	8		FILL: Stiff Dark Brown to Black Sandy Clay with Gravel to 3.5'	(1.25)				
2	18	M/W	16	5	Medium Dense Brown Sand with Gravel and Silt to 5.5'					
3	18	M	29		Medium Dense, Brown Sandy SILT, Trace to Little Gravel and Clay (ML)					
4	18	W	14							
5	20	W	17	10						
6	24	W	4		Loose to Very Loose, Brown Silty Fine SAND, Some Gravel, Trace Clay (SM)					
7	24	W	5	15						
8	24	W	29		Medium Dense, Light Brown Fine SAND, Some Silt and Gravel (SM)					
9	24	W	32		Dense, Light Brown Fine to Medium SAND, Some Gravel, Trace Silt (SP)					
10	24	W	32	20	Dense, Brown Fine to Medium SAND, Some Silt and Gravel, Scattered Cobbles and Boulders (SM)					
11	1	M	50/4"		Very Dense, Brown to Gray GRAVEL, Trace to Little Sand, Scattered Cobbles (GP)					
12	0	-	50/1"		Presumed Bedrock (Hard Drilling)					
				25	End Boring at 25 ft Auger Refusal on Presumed Bedrock/Possible Boulder Borehole backfilled with bentonite chips and asphalt patch					
				30						

WATER LEVEL OBSERVATIONS

While Drilling ▽ 8.5'   Upon Completion of Drilling 6'
Time After Drilling: 30 Min.
Depth to Water: 6' ▼
Depth to Cave in: 12'

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

GENERAL NOTES

Start 7/21/21 End 7/21/21
Driller BSD Chief MC Rig CME-55
Logger GB Editor ESF
Drill Method 2.25" HSA; Autohammer

CGC Inc.

LOG OF TEST BORING

Project	Mendota-Grassman Greenway
	University Avenue
Location	Madison, WI
Boring No.	3
Surface Elevation (ft)	881±
Job No.	C21051-10
Sheet	1 of 1

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

SAMPLE

No.	Type	Rec (in.)	Moist	N
1		18	M	14
2		14	M	4
3		18	W	7
4		16	M	5
5		18	M	15
6		18	W	15
7		18	W	7
8		18	M	4
9		14	W	23
10		12	W	24
11		18	W	18
12		16	W	15
13		14	W	7
14		16	W	12
15		16	W	27
16		10	M	7
17		18	W	56
18		1	W	50/3"

Depth (ft): 5, 10, 15, 20, 25, 30, 35, 40, 45, 50

VISUAL CLASSIFICATION and Remarks / SOIL PROPERTIES

Visual Classification and Remarks	qu (qa) (tsf)	W	LL	PL	LI
8 in. TOPSOIL					
FILL: Medium Dense Brown Sand with Silt and Gravel to 3'					
Loose to Very Loose Light Brown Sand with Silt to 8'					
Soft Brown Clay with Sand to 10'	(0.3)				
Medium Dense Brown Sand with Silt to 13'					
Medium Dense, Light Brown Sandy SILT (ML - Possible Fill)					
Loose, Dark Brown SILT, Some Sand, Trace Clay and Organics (ML)					
Soft, Bluish-Gray Lean CLAY, Trace to Little Sand (CL - Sandy Near 20')	(0.4)				
Medium Dense, Brown Fine to Medium SAND, Some Silt and Gravel (SM)					
Scattered Thin (<1/2-in.) Clay Seams Beginning Near 25'					
Becoming Loose Near 29'					
Medium Dense, Light Brown Fine to Coarse SAND, Trace Silt and Gravel (SP)					
Medium Dense, Brown Fine to Coarse SAND and GRAVEL, Some Silt (SM/GM)					
Medium Stiff to Soft, Gray Sandy Lean CLAY, Trace Gravel and Organics (CL)	(0.75) (0.4)				
Very Dense, Gray Fine to Medium SAND, Some Silt and Gravel (SM)					
Firm Drilling Beginning at 42'					
Apparent Bedrock Hard Drilling Beginning at 43'					

End of Boring at 43.75 ft Due to Refusal on Apparent Bedrock

Backfilled with Bentonite Slurry and Chips

WATER LEVEL OBSERVATIONS

While Drilling ∇ 14.0' 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 10/8/21 End 10/8/21

Driller BSD Chief MC Rig CME-55

Logger KD Editor ESF

Drill Method 2.25" HSA; to 10', 3 7/8" RB w/Mud to 43.75'; Autohammer

CGC Inc.

LOG OF TEST BORING

Project: Mendota-Grassman Greenway
University Avenue
Location: Madison, WI

Boring No. 4
Surface Elevation (ft) 881±
Job No. C21051-10
Sheet 1 of 1

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

SAMPLE / SOIL PROPERTIES

No.	Rec (in.)	Moist	N	qu (qa) (tsf)	W	LL	PL	LI
1	4	M	13					
2	14	M	25					
3	18	M	9					
4	8	M	7					
5	8	W	15 ∇					
6	14	W	14					
7	18	W	7					
8	12	W	4	(0.25)				
9	10	W	8					
10	12	W	6					
11	14	W	15	(0.2)				
12	16	W	17					
13	16	W	18					
14	6	W	19					
15	4	W	46					
16	16	W	24					
17	0	W	30					

VISUAL CLASSIFICATION and Remarks

4 in. Asphalt Pavement/7 in. Base Course

FILL: Medium Dense Dark Brown and Brown Sand with Silt, Gravel and Clay to 5'

Loose Brown and Light Brown Sandy Silt and Silty Sand to 16.5'

Medium Dense Light Brown Sand with Silt to 17.5'

Soft to Very Soft, Dark Gray Lean CLAY, Some Sand, Trace Organics (CL)

Loose, Gray Fine to Medium SAND, Some Silt and Gravel, Trace Clay (SM)

Very Soft, Gray Lean CLAY, Trace to Little Sand (CL)

Medium Dense, Brown and Gray Fine to Medium SAND, Some Silt and Gravel, Trace to Little Clay (SM)

Dense to Medium Dense, Brown Fine to Coarse SAND, Some Silt and Gravel (SM)

Firm Drilling Beginning at 40'

Apparent Bedrock Hard Drilling Beginning at 40.5'

End Boring at 42 ft Due to Rufusal on Apparent Bedrock

Backfilled with bentonite slurry, chips and asphalt patch

WATER LEVEL OBSERVATIONS

While Drilling ∇ 12.0' 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 10/12/21 End 10/12/21
Driller BSD Chief MC Rig CME-55
Logger KD Editor ESF
Drill Method 2.25" HSA; to 10', 3 7/8" RB w/Mud to 42'; Autohammer

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

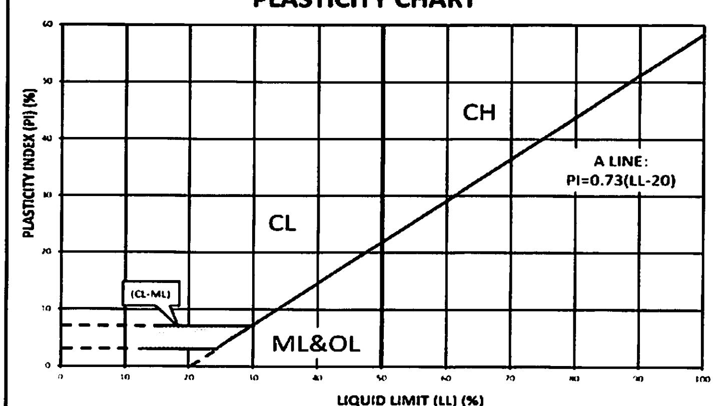
LABORATORY CLASSIFICATION CRITERIA

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

PLASTICITY CHART



APPENDIX C

RECOMMENDED COMPACTED FILL SPECIFICATIONS

APPENDIX C

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

Area	Percent Compaction (1) Clay/Silt	Percent Compaction (1) 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 D

DOCUMENT QUALIFICATIONS

APPENDIX D
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