CGC, Inc.

Construction • Geotechnical
Consulting Engineering/Testing

August 17, 2022
C22051-13

Maria Delestre
City of Madison - Engineering Division
City-County Bldg., Rm. 115
210 Martin Luther King, Jr. Blvd.
Madison, WI 53703-3342

Re: Preliminary Geotechnical Report
Proposed Student Housing
Theory Madison – 415 North Lake Street
Madison, Wisconsin

Dear Ms. Delestre:

Construction • Geotechnical Consultants, Inc. (CGC) has completed the review and evaluation of the provided subsurface information for the above-referenced project site. The purpose of this report is to summarize the subsurface conditions within the proposed construction area and to provide *preliminary* geotechnical recommendations regarding site preparation, foundation, floor slab and below-grade wall design/construction. A determination of the site class for seismic design is also included. We are sending you an electronic copy of this report, and we can provide a paper copy upon request.

PROJECT AND SITE DESCRIPTION

We understand a student housing development is planned to replace the existing parking ramp at 415 North Lake Street in Madison, Wisconsin. Preliminary development plans include one level of below-grade parking, six levels of above-grade parking, and nine residential levels above. A Madison Metro bus terminal is planned on the ground floor. The existing ramp includes four parking decks and is connected to the adjacent ramp at 434 North Frances Street. Hawthorne Court runs between the two ramps, and ground surface elevations along North Lake Street and Hawthorne Court in the vicinity of the project site appear to range between about EL 859 and 861 ft, based on publicly-available topographic data (DCiMap; 1-ft contour lines).

According to the provided concept plans for the new development, finished first floor elevation (G/P1) is planned to be established near the current street level, or about EL 860 ft; the bottom of the below-grade parking level (P-1) is indicated near EL 845 ft. The new building is envisioned to be connected to the North Frances Street ramp on the above-grade parking levels P2 and P3. We anticipate fairly high structural loads from the new construction. The building footprint is expected to occupy the entire site.

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SUBSURFACE CONDITIONS

A geotechnical exploration program was completed by others in 1962 in preparation of constructing the existing parking ramp, and the results were provided to us as a fence diagram (with boring location map) by the City of Madison. The 1962 geotechnical exploration program involved nine Standard Penetration Test (SPT) soil borings to depths between roughly 17 and 35 ft below (then) site grades, and boring locations as they relate to the planned development are shown in plan on the Soil Boring Location Exhibit included in Appendix A. Our interpretation of the original fence diagram is shown on the Fence Diagram which is also included in Appendix A. Refer to the original notes included on the fence diagram for a description of drilling and sampling procedures.

For our evaluation, we have assumed that elevation 0 shown on the provided fence diagram refers to the "old" City of Madison datum EL 845.4 ft (NAVD 88). We have further assumed that the SPT blow counts shown on the fence diagram are N_{60}-values (i.e., an SPT hammer with lower efficiency compared to modern, automatic SPT hammers was used).

The subsurface profiles at the boring locations varied to some degree, but the following strata were typically encountered (in descending order):

- Roughly 1 to 4 ft of ***fill***, which appears to be in loose to very dense condition based on N_{60}-values between about 8 and 115 bpf; underlain by
- About 4 to 12 ft of stiff to hard ***clay*** layers (based on q_a-values[1] between about 1.3 to more than 5 tsf), as well as loose to dense ***silt*** strata (based on N_{60}-values between about 8 and 37 bpf) in Borings 6 and 7; over
- Medium dense to very dense ***sand and gravel*** deposits (based on N_{60}-values between about 12 and more than 60 bpf), including very stiff to hard *clayey zones* (based on q_a-values between about 3.5 to more than 5 tsf) in Borings 3, 5 and 9, to the maximum depths explored.

Groundwater levels in the boreholes were observed at about 14 hours to 1 week after the completion of drilling, ranging between about 11 and 15 ft below the ground surface (corresponding to approximately EL 846 to 851 ft). The site is located roughly 1,100 ft south of Lake Mendota and approximately 3,500 ft north/northwest of Lake Monona (Monona Bay). Therefore, groundwater levels on the site are generally expected to be between the water levels in the lakes. For reference, typical summer maximum levels are EL 850.1 and 845.2 ft for Lakes Mendota and Monona, respectively, while a 1% flood event is defined by water levels of EL 852.8 and 847.7 ft. In addition to the influence from the water levels in Lakes Mendota and Monona, groundwater levels are expected to fluctuate with pumping rates in nearby wells, dewatering below nearby buildings and seasonal variations in precipitation, infiltration, evapotranspiration, as well as other factors.

[1] An estimate of the unconfined compressive strength of cohesive soils.

DISCUSSION AND RECOMMENDATIONS

Subject to the limitations discussed below and based on the subsurface exploration, it is our *preliminary* opinion that the site is generally suitable for the planned redevelopment and that the new building can be supported by a conventional spread footing foundation system, with the understanding that fairly shallow undercutting of marginal native soils may potentially be required below the bottom of footings on a fairly isolated basis. *In addition, temporary dewatering of the below-grade level excavation should generally be expected, which could be a significant cost and effort depending on time of year construction occurs. We also recommend that a permanent subfloor dewatering system be included below the parking level floor slab, or that the portions of the lower level that extend below the groundwater table on a permanent or intermittent basis be designed/constructed as watertight (i.e., "bathtub").* Our *preliminary* geotechnical recommendations for site preparation, foundation, floor slab and below-grade wall design/construction, along with our assessment of the site class for seismic design, are presented in the following subsections. Additional information regarding the conclusions and recommendations presented in this report is discussed in Appendix B.

1. Site Preparation

Following the demolition of the existing parking ramp, we anticipate that mass-excavation to planned lower-level subgrade elevations will commence. Note that we recommend the existing structure be demolished and removed in its entirety, including floor slabs, footings, and associated utilities. With the footprint of the new building expected to border public right-of-ways and neighboring parcels, and anticipated mass-excavation depths on the order of 15 ft below current site grades, we expect that temporary earth retention/shoring will generally be required on all sides of the excavation. We recommend shoring systems be designed by an appropriately qualified professional engineer.

It is important to note that the lower-level mass excavation is generally anticipated to extend on the order of 1 to 6 ft below the groundwater table, based on the water level observations in the soil borings and current typical lake levels, but deeper footing, elevator pit and undercut excavations also need to be taken into consideration. Seasonally higher groundwater levels could potentially also be experienced on this site, which may increase the dewatering effort. *In light of the observed groundwater conditions, dewatering is generally anticipated to play a critically important role in order to facilitate excavation and develop suitable foundation and floor slab subgrades*. To allow for construction "in the dry", water levels should be lowered a <u>minimum</u> of 2 ft below the bottom of excavations *in advance of final excavation*. It has been our experience that groundwater drawdowns on the order of 1 to 2 ft can typically be achieved using submersible pumps that operate from filtered sump pits. Drawdowns exceeding about 2 ft will likely require alternative dewatering measures, such as deep well or vacuum well point systems. Note, however, that due to the fairly fine-grained nature of some of the soils on this site, the excavation may be difficult to dewater, potentially requiring the use of a vacuum well-point system regardless of drawdown depths. Supplemental dewatering from shallow sumps outside the footing line may also be required. *Dewatering means and methods are the contractor's responsibility, and the construction dewatering system should be designed by a qualified* *professional engineer in conjunction with the temporary earth retention system such that appropriate hydrostatic pressures are accounted for. If groundwater is not adequately controlled, significantly deeper undercuts, subgrade stabilization and modifications to the temporary earth retention system could be required.* Ineffective dewatering could also cause loosening of the subgrades, resulting in settlement of foundations and slabs. The construction dewatering system will need to remain in-place until the permanent sub-floor dewatering system becomes operational, or sufficient building load is applied for a watertight ("bathtub") system (see Floor Slab section).

2. <u>Preliminary Foundation Design</u>

We understand that the finished parking level elevation is planned to be established near EL 845 ft, about 15 ft below the North Lake Street level. Footings and elevator shafts are generally anticipated to extend a few feet below the finished parking level elevation, and foundation subgrades are therefore expected to generally consist of dense to very dense sand/gravel deposits, including occasional very stiff to hard clayey zones, which are expected to control the foundation design. *Undercutting could potentially be required if loose to medium dense (or disturbed) sand soils, medium stiff to stiff clays or existing fill are present at or below footing grades. Based on the available subsurface information, undercutting is generally expected to be relatively isolated and shallow.*

As mentioned in the previous section, the below-grade level excavation is expected to require temporary dewatering during construction. *Where groundwater control is difficult, we recommend a <u>minimum</u> 1-ft thick clear stone layer be included below the bottom of footings and elevator pit base slabs, as the stability of the natural sand/gravel soils with significant fines-contents may rapidly deteriorate upon exposure, making the soils unsuitable for foundation support.* If such conditions occur, the final 1 ft of the excavations should occur in small sections and, once exposed, the subgrades should be evaluated by CGC and then quickly covered with non-woven geotextile fabric (e.g., Mirafi 160N or equivalent), followed by backfilling with clear stone that is placed in maximum loose lifts of 12 in. and compacted with a large vibratory plate compactor or excavator-mounted hoe-pack until deflection ceases. Supplemental dewatering can also occur from filtered sump pits within the clear stone layer outside of the footing line. Note that the geotextile fabric should be wrapped up the sides and over the top of the clear stone layer to prevent migration of fines from the surrounding soils into the void spaces of the clear stone. As an alternative to using clear stone below footings and elevator pits where groundwater control is difficult, foundation subgrades could be also stabilized with a <u>minimum</u> 4-in. thick "lean mix" concrete mud mat. The lean mix concrete should be capable to develop a minimum 28-day design strength of 1,000 psi. As with the use of clear stone, it is important that the final about 1 ft of the excavation occurs in small sections, followed by immediate placement of the lean mix concrete. *It is imperative that close coordination be developed by the contractor and CGC to facilitate prompt approval and clear stone or lean mix concrete placement in small sections to reduce the chance of subgrade degradation.*

In conjunction with the above recommendations, we recommend the following parameters be used for *preliminary* foundation design:

- Maximum net allowable bearing pressure:
  - Footings below the parking level: 8,000 psf
  - Shallow footings at frost depth (if any): 3,000 psf

- Minimum foundation widths:
  - Continuous wall footings: 18 in.
  - Column pad footings: 30 in.

- Minimum footing depths below finish site grades:
  - Exterior/perimeter footings: 4 ft
  - Interior footings: no minimum requirement

As a variety of subsurface conditions is expected to be encountered across the site, footing subgrades should be checked by a CGC field representative, including the performance of dynamic cone penetrometer (DCP) testing, to document that the subgrade soils are suitable for footing support or otherwise advise on corrective measures, such as undercutting. We recommend using a smooth-edged backhoe bucket for footing and undercut excavations. Where required, the base of undercut excavations should be widened beyond the footing edges at least 0.5 ft in each direction for each foot of undercut depth for stress distribution purposes. Granular soils exposed at footing grade or the bottom of undercut excavations *well above the water table or with an effective dewatering system in-place* should be thoroughly recompacted with a large vibratory plate compactor or an excavator-mounted hoe-pack prior to backfilling or formwork/concrete placement to densify soils loosened during the excavation process. Soils potentially susceptible to disturbance from vibratory compaction (e.g., cohesive/fine-grained soils or sands with elevated moisture-content) should be hand-trimmed. OSHA slope guidelines should be followed if workers need to enter footing excavations.

In order to re-establish footing grades in undercut areas *above the water table or with an effective dewatering system in-place*, we recommend using 3-in. DGB that is placed in loose 10-in. lifts and compacted until deflection ceases. Alternatively, granular backfill (including sand/gravel soils excavated on-site) compacted to at least 95% compaction based on modified Proctor methods (ASTM D1557), in accordance with the Recommended Compacted Fill Specifications presented in Appendix C, can also be used to restore foundation grades where groundwater is not a concern. *However, note that sand/gravel backfill should not be used below footings designed for an allowable bearing pressure in excess of 5,000 psf. In areas where saturated soils remain despite concerted dewatering effort*, undercut excavations should be backfilled with crushed clear stone or lean mix concrete as described previously. Note that with the use of lean mix concrete as backfill, undercut excavations should be laterally oversized 0.5 ft from the edges of the foundation, but can then be extended vertically (i.e., without sloping of the undercut sidewalls) provided the excavation is stable and workers do not need to enter the excavation.

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

3. Preliminary Seismic Site Class

In our *preliminary* opinion, the average soil properties in the upper 100 ft of the site (based on N-values projected to be greater than 50 blows/ft, on average, in the granular soils underlying the site) may be characterized as a very dense soil/soft rock profile. This characterization would place the site in Site Class C for seismic design according to the International Building Code and ASCE 7.

4. Preliminary Floor Slab Design

With the finished parking level elevation anticipated around EL 845 ft, we expect that the floor slab will generally be supported on medium dense to very dense sand/gravel deposits, including occasional very stiff to hard clayey zones. Prior to slab construction, granular subgrade soils should be thoroughly recompacted with a vibratory smooth-drum roller to densify soils that may become disturbed or loosened during construction activities. Cohesive or fine-grained subgrades should be statically recompacted and subsequently proof-rolled to check for soft/yielding areas. Areas of disturbed soil or where soils remain loose after recompaction should be undercut and replaced with compacted 3-in. DGB or granular fill. *Floor slab subgrades should be adequately dewatered during construction, as previously discussed.*

Finished parking level grades are expected to be established roughly 1 to 6 ft below the typical groundwater table, but seasonally elevated groundwater levels may also be experienced on this site. Although building below the water table can be accomplished, *the owner should understand that there are additional risks and considerable costs associated with such plans*. Where the lower level will extend below the water table on a permanent or intermittent basis, two typical strategies can be used to deal with the water table:

1. Install a sub-floor drainage system that *permanently* lowers the water table below the lower-level slab elevation for the life of the structure; or

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

On past projects with groundwater drawdowns of similar magnitude to this project, the permanent sub-floor dewatering alternative has typically been chosen based on economics, and additional recommendations for this alternative are provided below. However, we have also been involved in several projects that have utilized a watertight design, and we can provide additional recommendations upon request. We understand that there are higher up-front costs associated with constructing a watertight structure (from additional concrete and waterproofing), but *it should be recognized that there will be higher long-term operation and maintenance (O&M) costs and greater risk associated with permanently lowering the water table below the floor slab during the lifetime of the building*. The costs associated with building below the water table will be reduced, at least somewhat, if the lower floor slab grades are raised. *However, raising building grades will require a re-evaluation of the building foundation.*

If a permanent sub-floor dewatering system will be utilized below the parking level floor slab, we anticipate the system will involve the following components, based on past projects of similar nature:

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

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

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

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

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

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

- Exterior lower-level walls below approximately EL 853 ft should be waterproofed with a waterproofing membrane.

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

Note that there is considerable flexibility in the details of the drainage system, and we can work with the design team, *which should include an adequately qualified plumbing designer*, to develop a system suitable for the project. We recommend the installation of a temporary groundwater monitoring well in order to obtain longer-term groundwater data that can assist in sub-floor dewatering evaluation, and the well can also be used to conduct drawdown and recovery or slug tests in order to estimate the *in-situ* hydraulic conductivity, which can in turn be used to provide *preliminary* sub-floor dewatering rates based on the building footprint and anticipated groundwater drawdown. Sub-floor dewatering rates should be re-evaluated during construction to reflect actual construction dewatering rates, which will more accurately reflect anticipated long-term dewatering rates.

We expect that the parking level floor slab will be supported on the compacted stone drainage layer over the native sand/gravel or clay soils, and a subgrade modulus of 150 pci may be used in floor slab design. The design subgrade modulus is based on a firm, recompacted subgrade such that non-yielding conditions are developed, as discussed above. The floor slab should be isolated from the building walls and columns with compressible filler, and the design should include an adequate number of isolation and contraction joints. A vapor barrier can be installed below the parking level floor slab to further reduce the potential for moisture migration through the slab.

5. Below-Grade Walls

We anticipate that below-grade walls will be laterally supported by the lower-level slab and upper-level framing. Therefore, *at-rest* lateral earth pressures should be used during design of these walls. To reduce the buildup of such pressures, high-quality backfill should be placed within 4 to 6 ft of the walls. We recommend that a perimeter drainage system be installed to intercept potential surface water infiltration, and that the granular backfill be continuously connected to the drainage system. The perimeter drainage system, in turn, should be connected to the sub-floor dewatering system which discharges water to one or more sumps. The granular backfill should be well-graded sand or gravel having no more than 12% by weight passing the No. 200 U.S. standard sieve (i.e., USCS designations SP, SP-SM, GP or GP-GM). Some of the sand soils on this site are expected to contain higher amounts of fines, but may potentially also be used as below-grade wall backfill if a three-dimensional drainage board is included in the wall design. Soils containing cobbles/boulders should not be used in direct contact with below-grade walls. To impede the inflow of surface moisture, the final 2 ft of backfill in unpaved areas should consist of a clayey fill cap. The clayey cap (or pavement) should be graded to promote positive drainage away from the walls.

Before placing the wall backfill, the exterior walls should be damp-proofed with spray-applied or mopped-on rubber or bituminous sealer. Compaction of the backfill within 3 to 5 ft of the walls should be performed with lightweight equipment to avoid the development of excessive lateral earth pressures. The backfill should generally be compacted to a minimum compaction level of 93% modified Proctor following Appendix C guidelines. However, we recommend a minimum of 95% compaction where shallow footings or stoops will bear on the wall backfill, as well as in the upper 2± ft in pavement areas. Lower-level walls constructed in accordance with the above recommendations may be designed for an equivalent fluid pressure of 55 psf per ft of depth (*at-rest* conditions). Additionally, the wall design should also account for surcharge effects that could be applied during or after construction.

Where portions of the lower level may extend below the water table on a permanent or intermittent basis, such as elevator pits, they should be design and constructed as a watertight (i.e., “bath tub”) structure capable of resisting hydrostatic uplift pressures below the base and along the walls.

CONSTRUCTION CONSIDERATIONS

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

- Due to the potentially sensitive nature of some of the on-site soils, we recommend that final site grading activities be completed during dry weather, if possible. Construction traffic should be avoided on prepared subgrades to minimize potential disturbance.

- Contingencies in the project budget for subgrade stabilization with coarse aggregate in floor slab areas should be increased if the project schedule requires that work proceed during adverse weather conditions.

- Earthwork construction during the late fall through early spring could be complicated as a result of wet weather and freezing temperatures. During cold weather, exposed subgrades should be protected from freezing before and after footing construction. Fill should never be placed while frozen or on frozen ground.

- Excavations extending greater than 4 ft in depth below the existing ground surface should be sloped or braced in accordance with current OSHA standards. Earth retention systems should be designed by an appropriately qualified, registered professional engineer in conjunction with the dewatering system. *Care should be exercised not to undermine foundations of nearby existing buildings.* The completion of a pre-condition survey prior to and settlement monitoring on nearby buildings during construction is recommended.

- Dewatering of the below-grade excavation should generally be expected during construction, which could be a significant effort depending on the time of year construction occurs, and dewatering was discussed previously. In addition, water accumulating at the bottom of excavations as a result of precipitation or seepage should be quickly removed in a similar manner, with dewatering means and methods being the contractor's responsibility.

RECOMMENDED CONSTRUCTION MONITORING

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

- Subgrade proof-rolling/compaction;
- Fill/backfill placement and compaction;
- Foundation excavation/subgrade preparation; and
- Concrete placement.

CLOSING REMARKS

The recommendations presented in this report are *preliminary* in nature. As development plans progress, we recommend that information pertaining to proposed building and foundation grades, as well structural loads, be provided to us, and CGC should be allowed to review the recommendations contained herein and adjust them as needed.

If a higher allowable bearing pressure is desired for foundation design in order to reduce foundation sizes, a pressuremeter testing (PMT) program should be considered. The installation of a temporary groundwater monitoring well should also be considered in order to obtain longer-term groundwater data and conduct drawdown/recovery or slug tests. If desired, we can develop a proposal for supplemental subsurface exploration at the appropriate time.

* * * * *

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

Sincerely,

CGC, Inc.

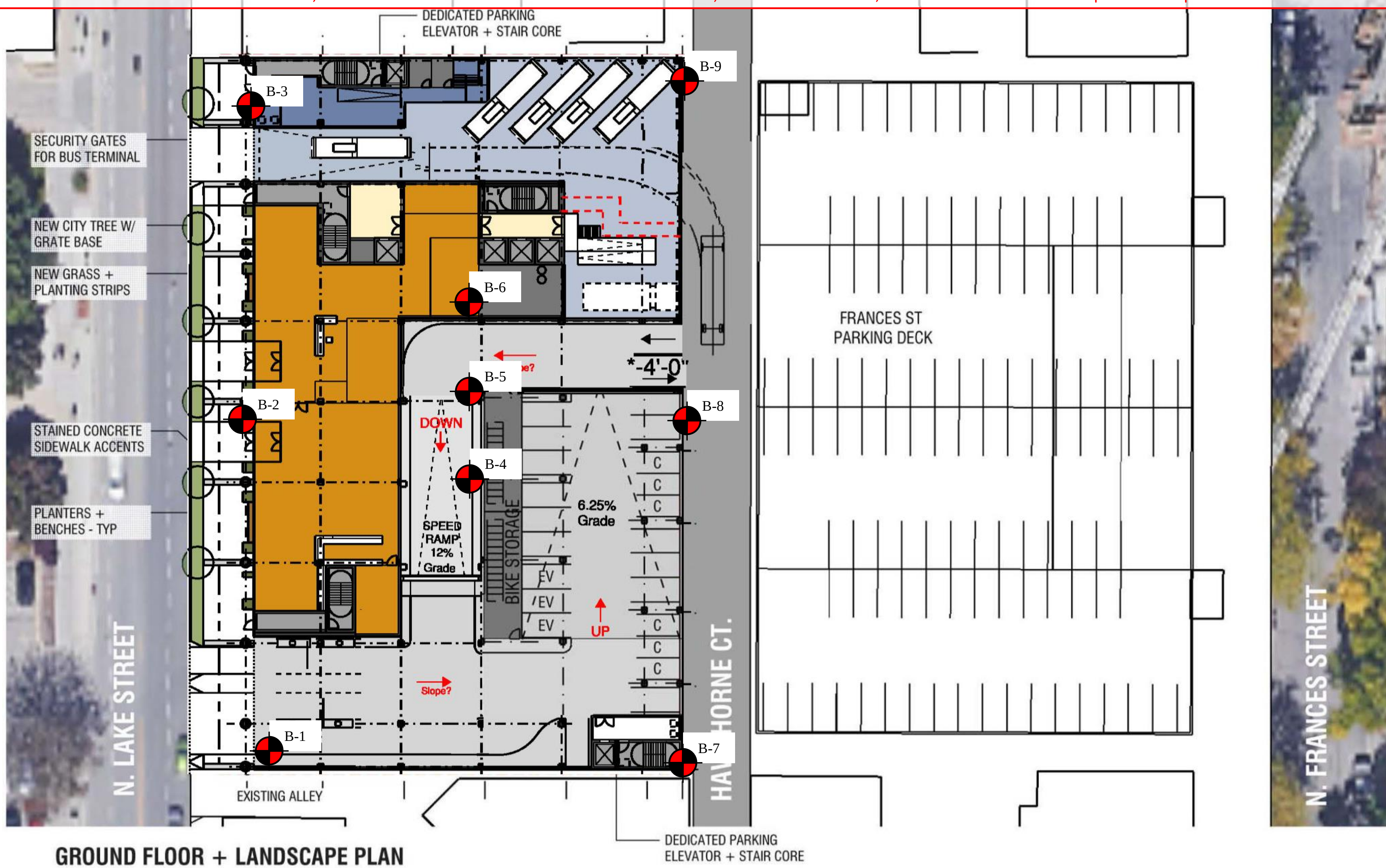
Tim F. Gassenheimer, PE
Senior Staff Engineer

Ryan J. Portman, PE
Senior Consulting Professional/Field Supervisor

Encl: Appendix A - Soil Boring Location Exhibit
Fence Diagram
Appendix B - Document Qualifications
Appendix C - Recommended Compacted Fill Specifications

APPENDIX A

SOIL BORING LOCATION EXHIBIT
FENCE DIAGRAM



Scale: Reduced

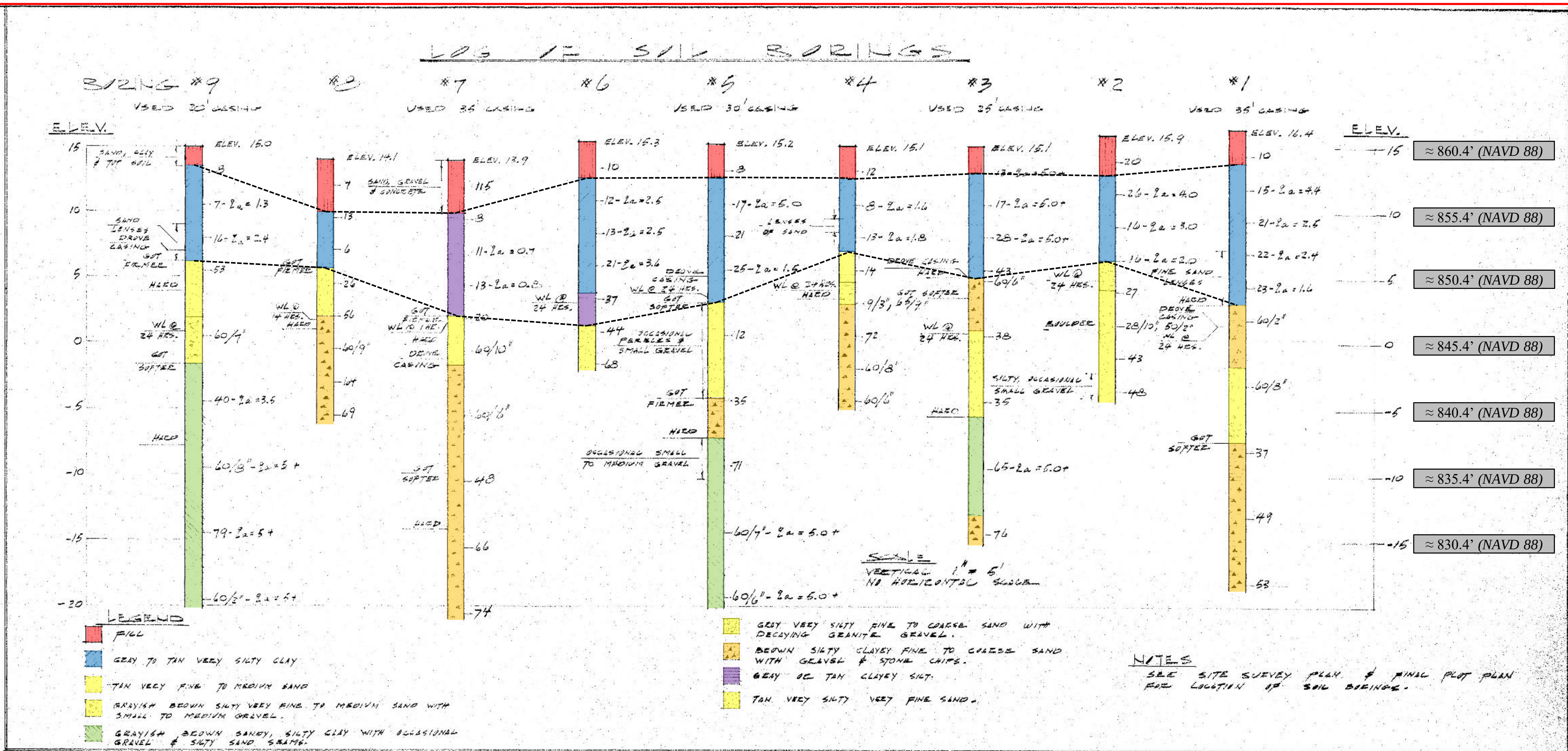
Legend

Denotes Soil Boring Location and Number

Notes

1. Borings were drilled by others in 1962.
2. Boring locations are very approximate.
3. Base map provided by City of Madison.

Job No.: C22051-13	**CGC, Inc.**	**SOIL BORING LOCATION EXHIBIT**
Date: 08/2022		**Proposed Student Housing** **Theory Madison – 415 North Lake Street** **Madison, Wisconsin**



NOTES

1. ALL ELEVATIONS ARE CITY DATUM.
2. BORINGS WERE MADE USING STANDARD PROCEDURES WITH A 2" O.D. SPLIT SPOON & 2½" CASING AS REQUIRED.
3. FIGURES TO THE RIGHT OF EACH BORING LOG INDICATES THE NUMBER OF BLOWS REQUIRED TO DRIVE THE 2" O.D. SPLIT SPOON 12" USING A 140 LB. WEIGHT FALLING 30".
4. NOTATION "DROVE CASING" INDICATES DEPTH TO WHICH BORING HAD PROGRESSED WHEN CASING WAS FIRST USED.
5. WATER LEVEL (WL) INDICATES WERE AT TIMES SHOWN AFTER COMPLETION OF THE BORINGS.

LABORATORY NOTES

Qa = APPROXIMATE UNCONFINED COMPRESSIVE STRENGTH DETERMINED WITH PENETROMETER, IN TONS / SQ. FT.

Not to scale

CGC Notes

1. Borings were drilled by others in 1962.
2. Fence diagram provided by City of Madison; interpretation by CGC.
3. Assuming elevation 0 is "old" City of Madison datum = 845.4 ft *(NAVD 88)*.
4. Blow counts are assumed to be N_{60}-values.

Job No.: C22051-13	CGC, Inc.	**FENCE DIAGRAM**
Date: 08/2022		**Proposed Student Housing** **Theory Madison – 415 North Lake Street** **Madison, Wisconsin**

APPENDIX B

DOCUMENT QUALIFICATIONS

APPENDIX B
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.

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

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)

CGC, Inc.

Construction • Geotechnical
Consulting Engineering/Testing

December 22, 2022
C22051-31

Maria Delestre
City of Madison - Engineering Division
City-County Bldg., Rm. 115
210 Martin Luther King, Jr. Blvd.
Madison, WI 53703-3342

Re: Supplemental Geotechnical Exploration Report
Proposed Student Housing
Theory Madison – 415 North Lake Street
Madison, Wisconsin

Dear Ms. Delestre:

Construction • Geotechnical Consultants, Inc. (CGC) has completed the supplemental subsurface exploration program for the above-referenced project. The purpose of this program was to further evaluate the subsurface conditions on the project site and to provide updated geotechnical recommendations regarding foundation design and construction. We are sending you an electronic copy of this report, and we can provide a paper copy upon request.

For geotechnical recommendations pertaining to site preparation, floor slab and below-grade wall design/construction, as well as a discussion of the site class for seismic design, please refer to our previously issued *Preliminary Geotechnical Report* (CGC Project No. C22051-13; dated August 17, 2022).

SUPPLEMENTAL SUBSURFACE EXPLORATION

Two additional Standard Penetration Test (SPT) soil borings (labeled PMT-1 and PMT-9) were drilled to supplement the 1962 SPT borings and to facilitate pressuremeter testing to assist with bearing capacity assessment and settlement estimation. The boring locations were selected and marked in the field by CGC.

The supplemental borings were conducted by America's Drilling Company (ADC; under subcontract to CGC) on November 28 and 29, 2022 using a truck-mounted D-50 rotary drill rig equipped with hollow stem augers, mud-rotary tooling and an automatic SPT hammer. The specific procedures used for drilling and sampling are described in Appendix A, and the boring locations are shown in plan on the Soil Boring Location Exhibit presented in Appendix B. Ground surface elevations at the boring locations were estimated by CGC based on DCiMap 1-ft contour lines, and the elevations should therefore be considered approximate.

Pressuremeter test zones were prepared using a split-barrel sampler and specially sized roller bits. The pressuremeter testing in the boreholes prepared by ADC was performed by GEI Consultants. In a

2921 Perry Street, Madison WI 53713
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pressuremeter test, a cylindrical probe is inserted to the (then current) borehole bottom, and hydraulically expanded radially against the borehole sides. Probe volume versus pressure is recorded. A more detailed description of pressuremeter testing is provided on the sheet titled "Pressuremeter Procedures" attached in Appendix D, along with plots of probe volume versus pressure for each individual test. The results of the pressuremeter tests are further discussed in the subsequent section of this report.

The subsurface profiles in the supplemental soil borings were in general agreement with the findings in the previously performed borings. A more detailed description of the encountered soil conditions is presented on the supplemental soil boring logs attached in Appendix B

DISCUSSION AND RECOMMENDATIONS

Pressuremeter test results are presented in Table 1 below.

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

TABLE 1 – Pressuremeter Test Results Summary

Boring No.	Test Elevation (ft)	P_o (tsf)	P_f (tsf)	P_l (tsf)	E_d (tsf)	E^+ (tsf)	E_d/E^+
PMT-1	842.8	1.7	18.0	42.7	492	1,952	0.25
	831.3	1.7	19.0	-- [(a)]	448	1,941	0.23
	812.3	3.5	16.3	-- [(b)]	633	--	--
PMT-9	840.3	3.0	18.0	37.4	423	992	0.43
	832.8	4.0	21.3	-- [(c)]	398	--	--
	829.3	3.5	10.0	-- [(d)]	186	--	--
						Average	**0.30**

Notes: [(a)] Pressurized to 19.0 tsf, P_l not reached; coarse gravel in the test zone.
[(b)] Membrane ruptured pressurizing to 20 tsf.
[(c)] Large test zone; probe reached maximum expansion prior to test completion.

[d] Membrane ruptured after pressurizing to 10 tsf.

Many of the pressuremeter tests were terminated prior to reaching completion due to soil conditions, large test zones, and membrane ruptures.

We understand the foundations for the proposed student housing structure will bear either at about EL 844 or 842 ft, roughly 16 to 20 ft below adjacent street/alley grades. The foundations are expected to bear on medium dense to very dense sand with a maximum anticipated column load of 1,800 kips.

A. Allowable Bearing Pressure

Based on lower-bound pressuremeter test program results, the maximum net allowable bearing pressure for foundations bearing on medium dense to very dense sand is 15,000 psf. The recommended maximum net allowable bearing pressure is the stress transmitted by the foundation to the soil in excess of the minimum final adjacent overburden stress. The recommended maximum net allowable bearing pressure incorporates a minimum safety factor of 3.0 against bearing capacity failure.

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

Due to the limited PMT data and difficulties during testing, CGC recommends additional pressuremeter testing to confirm the maximum allowable bearing pressure. We understand that one additional PMT boring can be performed near the center of the project area after the existing parking structure has been demolished.

B. Settlement Estimates

Based on lower-bound pressuremeter testing soil deformation moduli and the recommended maximum net allowable bearing pressure presented above, potential foundation settlement is estimated to be within typically tolerable levels of 1 in. or less for foundations bearing on medium dense to very dense sand.

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

CGC recommends additional pressuremeter testing to confirm the settlement estimate.

C. Site Preparation

We recommend that CGC provide observation and testing at the base of each foundation excavation to confirm that the soils match those encountered by the subsurface exploration and pressuremeter testing program, and that the soils meet minimum strength criteria associated with the maximum net allowable bearing pressure presented previously. The granular subgrade soils should exhibit a minimum equivalent SPT blow count ("N-value", comparable to the SPT hammer used) of 30 blows per foot.

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

If soils of the anticipated type or exhibiting the minimum required strength criteria are not found at the base of foundation excavations, it will be necessary to extend excavations deeper, or the affected foundations will have to be re-designed for a lower bearing pressure. We recommend that any foundation excavations that must be extended below their design bearing elevation be backfilled with "lean mix" concrete, which should have a minimum 28-day compressive strength of 1,000 psi, to restore the design bearing grade, and the lean mix backfill should extend laterally at least 0.5 ft beyond the edges the foundation it supports.

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

The building and foundation excavations are expected to encroach upon or extend below the groundwater table. Dewatering and measures to protect bearing subgrades (prior to concrete placement) should be anticipated, which was previously discussed in our Preliminary Geotechnical Report. *Foundation bearing soils which become disturbed due to standing water or construction activities must be removed and the excavation backfilled with lean mix concrete, as described above.*

Due to the relatively high recommended soil bearing pressure, it is imperative that the foundation subgrades remain undisturbed from the time of excavation to the time of footing concrete placement. Groundwater inflow into the excavations may cause softening/loosening of the exposed foundation subgrade soils, and worker traffic can further contribute to disturbance. As such, it will be extremely important that effective dewatering be provided as necessary during the construction of foundations. To protect subgrades from disturbance or when footings will not be cast on the exposed subgrade the same day of excavation, we recommend that a nominal 3-in. thick lean mix concrete "mud mat" be placed immediately after excavation to protect the sensitive subgrade soils from disturbance and loosening/softening. This is especially of concern where silty soils are exposed in the excavations. These soils will have a tendency to loosen upon removal of the overburden soils, especially when adequate dewatering in not provided. If this occurs, increased dewatering and additional removal will be required.

* * * * *

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

Sincerely,

CGC, Inc.

Tim F. Gassenheimer, PE
Senior Staff Engineer

Ryan J. Portman, PE
Consulting Professional/Field Supervisor

Encl: Appendix A - Field Exploration
Appendix B - Soil Boring Location Exhibit
Logs of Supplemental Test Borings (2)
Log of Test Boring-General Notes
Unified Soil Classification System
Appendix C - Document Qualifications
Appendix D - Pressuremeter Test Results
Pressuremeter Procedures

APPENDIX A

FIELD EXPLORATION

APPENDIX A

FIELD EXPLORATION

Subsurface conditions for this study were explored by drilling two supplemental Standard Penetration Test (SPT) soil borings to depths between 50 and 52.5 ft below current site grades, which were generally sampled at 2.5-ft intervals to a depth of 10 ft, and at 5-ft intervals below 10 ft; additional samples or samples at deviating depths were taken to prepare the pressuremeter test zones. The soil samples were obtained in general accordance with specifications for standard penetration testing, ASTM D1586, and the specific procedures used for drilling and sampling are described below.

1. Boring Procedures between Samples

   The boring is extended downward, between samples, by a hollow-stem auger. Note that the pressuremeter test borings were advanced, below depths of 25 ft, implementing mud-rotary drilling techniques in order to stabilize the sides of the borehole, preventing blow-up/loosening of the borehole bottom due to the presence of groundwater, and prepare the borehole bottom for pressuremeter testing. Mud-rotary drilling involves drilling with a roller bit, with drill cuttings being transported to the surface in the drilling slurry that is used to stabilize the borehole.

2. Standard Penetration Test and Split-Barrel Sampling of Soils
   (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 borings and is known as the Standard Penetration Resistance.

During the field exploration, the driller visually classified the soil and prepared a field log. *Field screening of the soil samples for possible environmental contaminants was not conducted by the drillers as these services were not part of CGC's work scope.* Water level observations were made in each boring during drilling and are shown at the bottom of each boring log. Upon completion of drilling, the borings were backfilled with bentonite to satisfy WDNR regulations and the soil samples were delivered to our laboratory for visual classification and limited geotechnical laboratory testing. The soils were visually classified by a geotechnical engineer using the Unified Soil Classification System. The final logs prepared by the engineer, as well as a Soil Boring Location Exhibit and a description of the Unified Soil Classification System are presented in Appendix B.

APPENDIX B

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

GROUND FLOOR + LANDSCAPE PLAN

N

Scale: Reduced

Legend

Denotes Supplemental Soil Boring Location and Number

Denotes Previous Soil Boring Location and Number

Notes

1. PMT-1 and PMT-9 were drilled by ADC on November 28 and 29, 2022.
2. B-1 through B-9 were drilled by others in 1962.
3. Boring locations are very approximate.
4. Base map was obtained through DCiMap.

Job No.: C22051-31	CGC, Inc.	**SOIL BORING LOCATION EXHIBIT** **Proposed Student Housing** **Theory Madison – 415 North Lake Street** **Madison, Wisconsin**
Date: 12/2022		

CGC Inc.

LOG OF TEST BORING

Project: Proposed Student Housing
Theory Madison - 415 North Lake Street
Location: Madison, Wisconsin

Boring No.: PMT-1
Surface Elevation (ft): 861.5±
Job No.: C22051-31
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	qu (qa) (tsf)	W	LL	PL	LI
					0	4± in. Concrete Pavement / 7± in. Base Course					
1	■	14	M	8		FILL: Loose, Tan Fine Sand, Trace to Little Silt, Scattered Lean Clay Seams/Pockets					
2	■	12	M	7	5						
3	■	6	M	6		FILL: Loose, Mixed Brown Silt and Black Organic Silt, Little Sand and Gravel					
4	■	6	M	4	10	FILL: Loose, Mixed Grayish Brown to Tan Silt and Reddish Brown to Brown Lean Clay, Little Sand and Gravel, Scattered Asphalt Pieces or Possible Cinders/Foundry Sand	(2.25-2.5)				
5	■	8	M	50/4"	15	Very Dense, Brown/Gray Fine to Coarse SAND and GRAVEL, Little Silt (SP-SM/GP-GM)					
6	■	14	M/W	30	20	Medium Dense to Very Dense, Light Brown to Grayish Brown Fine to Medium SAND, Some Silt and Gravel, Scattered Cobbles/Boulders (SM) PMT-1-1: 17 to 19.5 ft					
7	■	16	W	22	25						
8	■	18	W	32	30	PMT-1-2: 29 to 31.5 ft					
					35						
9	■	18	W	47	40						
10	■	10	W	50/4"	45						
11	■	12	W	96/9"	50	PMT-1-3: 48 to 52.5 ft					
					55	End of Boring at 52.5 ft Borehole Backfilled with Bentonite Chips and Asphalt Cold Patch					

WATER LEVEL OBSERVATIONS

While Drilling ▽ 22.0' 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 11/29/22 End 11/29/22
Driller ADC Chief KD Rig D-50
Logger DB Editor TFG
Drill Method 4.25" HSA (0-25') / 3.875" RB-DM (25-52.5'); Autohammer

CGC Inc.

LOG OF TEST BORING

Project: Proposed Student Housing
Theory Madison - 415 North Lake Street
Location: Madison, Wisconsin

Boring No. **PMT-9**
Surface Elevation (ft) 860.0±
Job No. C22051-31
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	qu (qa) (tsf)	W	LL	PL	LI
						7± in. Asphalt Pavement / 6± in. Base Course					
1	■	12	M	6		FILL: Loose, Grayish Brown to Brown Silt, Little to Some Sand, Trace Gravel, Scattered Asphalt Pieces or Possible Cinders/Foundry Sand					
2	■	10	M	4	5		(1.75-2.0)				
3	■	14	M	9		Stiff, Brownish Gray/Brown (Mottled) Lean CLAY, Trace Sand (CL)					
4	■	16	M	19	10	Loose to Medium Dense, Light Brown Fine SAND, Some Silt, Trace Gravel, Scattered Silt Seams (SM)					
5	■	14	M/W	26	15	Medium Dense, Light Brown SILT, Little Sand (ML)					
6	■	16	M	34		Medium Dense to Dense, Light Brown Fine to Medium SAND, Some Silt and Gravel, Slightly Clayey, Scattered Cobbles/Boulders (SM) PMT-9-1: 15.5 to 18 ft					
7	■	18	M	24	20						
8	■	20	W	43	25	Dense, Light Brown Fine SAND, Some Silt, Trace Gravel, Scattered Silt Seams (SM)					
9	■	20	W	60		Very Stiff to Hard, Brownish Gray/Reddish Brown (Varved) Lean CLAY, Trace Sand, Scattered Sand Seams (CL) PMT-9-2: 26 to 29 ft	(3.5-4.5+)	13.3	29	12	
10	■	16	W	89/9"	30	Very Dense, Brownish Gray Fine to Medium SAND, Some Silt and Gravel, Scattered Cobbles/Boulders (SM) PMT-9-3: 29 to 32 ft					
					35						
11	■	0	W	50/3"	40						
12	■	0	W	50/2"	45						
13	■	0	W	50/2"	50	End of Boring at 50 ft					
					55	Borehole Backfilled with Bentonite Chips and Asphalt Cold Patch					

WATER LEVEL OBSERVATIONS

While Drilling ▽ 23.0' 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 11/28/22 End 11/28/22
Driller ADC Chief KD Rig D-50
Logger DB Editor TFG
Drill Method 4.25" HSA (0-25') / 3.875" RB-DM (25-50'); 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)		
GRAVELS More than 50% of coarse fraction larger than No. 4 sieve size — Clean Gravels (Less than 5% fines)	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
SANDS 50% or more of coarse fraction smaller than No. 4 sieve size — Clean Sands (Less than 5% fines)	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

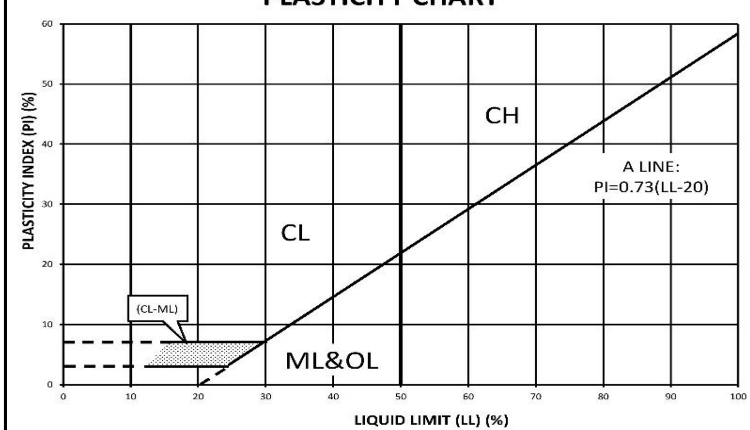
LABORATORY CLASSIFICATION CRITERIA

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

DOCUMENT QUALIFICATIONS

APPENDIX C
DOCUMENT QUALIFICATIONS

I. GENERAL RECOMMENDATIONS/LIMITATIONS

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

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

II. IMPORTANT INFORMATION ABOUT YOUR GEOTECHNICAL ENGINEERING REPORT

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

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

READ THE FULL REPORT

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

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

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

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

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

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

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

SUBSURFACE CONDITIONS CAN CHANGE

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

MOST GEOTECHNICAL FINDINGS ARE PROFESSIONAL OPINION

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

A REPORT'S RECOMMENDATIONS ARE NOT FINAL

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

A GEOTECHNICAL ENGINEERING REPORT IS SUBJECT TO MISINTERPRETATION

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

DO NOT REDRAW THE ENGINEER'S LOGS

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

GIVE CONSTRUCTORS A COMPLETE REPORT AND GUIDANCE

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

READ RESPONSIBILITY PROVISIONS CLOSELY

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

ENVIRONMENTAL CONCERNS ARE NOT COVERED

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

OBTAIN PROFESSIONAL ASSISTANCE TO DEAL WITH MOLD

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

RELY ON YOUR GEOTECHNICAL ENGINEER FOR ADDITIONAL ASSISTANCE

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

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

APPENDIX D

PRESSUREMETER TEST RESULTS
PRESSUREMETER PROCEDURES

GEI Consultants
Consulting Engineers & Scientists

Project Name: 415 N. Lake Street
GEI Project Number: 2204584
Operator: R.Rusk
Date: December 1, 2022

PRESSUREMETER TEST RESULTS

Boring ID	Test Depth (ft)	Test Midpoint Depth (ft)	P_o (tsf)	P_f (tsf)	P_L (tsf)	P^*_L (tsf)	E_d (tsf)	E^+ (tsf)	E_d/E^+	E_d/P^*_L	P_L/P_f
PMT-1	17.5 to 20.0	18.8	1.7	18.0	42.7	41.0	492	1952	0.25	12.0	2.4
	29.0 to 31.5	30.3	1.7	19.0	-	-	448	1941	0.23	-	-
	48.0 to 50.5	49.3	3.5	>16.3	-	-	633	-	-	-	-
PMT-9	18.5 to 21.0	19.8	3.0	18.0	37.4	34.4	423	992	0.43	12.3	2.1
	26.0 to 28.5	27.3	4.0	>21.3	-	-	398	-	-	-	-
	29.5 to 32.0	30.8	3.5	>10.1	-	-	186	-	-	-	-

GEI Consultants
Consulting Engineers & Scientists

Project Name: 415 N. Lake St.
GEI Job #: 2204584
Test Date: Tuesday, November 29, 2022
Boring No.: PMT-1
Test Depth (ft): 17.5 to 20.0
Test Zone Preparation: 2-1/2 In. Split Spoon

Probe Size: BX (58 mm)
Probe Deflated Vol. (cc): 535
Volumetric Zero Offset (cc): 48
Assumed Poisson's Ratio, ν: 0.33
Instrument Height (ft): 4
Operator: R.Rusk

No.	Field Pressure Readings (bars)	Field 30 Sec. Volume (cc)	Field 60 Sec. Volume (cc)	Creep Volume (cc)	Hydrostatic Pressure (tsf)	Membrane Resistance (tsf)	Corrected Pressure (tsf)	Corrected 30 Sec. Volume (cc)	Corrected 60 Sec. Volume, v_{60} (cc)	60 Sec. Radial Strain, $\Delta R/R_o$ (%)
1	0.0	64	77	13	0.7	0.18	0.5	16.0	29.0	2.67
2	0.5	111	135	24	0.7	0.51	0.7	63.0	87.0	7.82
3	1.0	175	198	23	0.7	0.83	0.9	126.9	149.9	13.15
4	1.5	234	255	21	0.7	1.07	1.2	185.9	206.9	17.76
5	2.0	274	277	3	0.7	1.15	1.7	225.9	228.9	19.49
6	3.0	285	285	0	0.7	1.18	2.7	236.8	236.8	20.11
7	5.0	295	295	0	0.7	1.21	4.7	246.7	246.7	20.88
8	8.0	307	307	0	0.7	1.26	7.8	258.5	258.5	21.78
9	12.0	325	325	0	0.7	1.31	11.9	276.2	276.2	23.14
10	16.0	344	345	1	0.7	1.37	16.1	294.9	295.9	24.62
11	20.0	366	369	3	0.7	1.44	20.2	316.6	319.6	26.39
12	11.0	366	366	0	0.7	1.43	10.8	317.3	317.3	26.21
13	18.0	375	375	0	0.7	1.45	18.1	325.8	325.8	26.84
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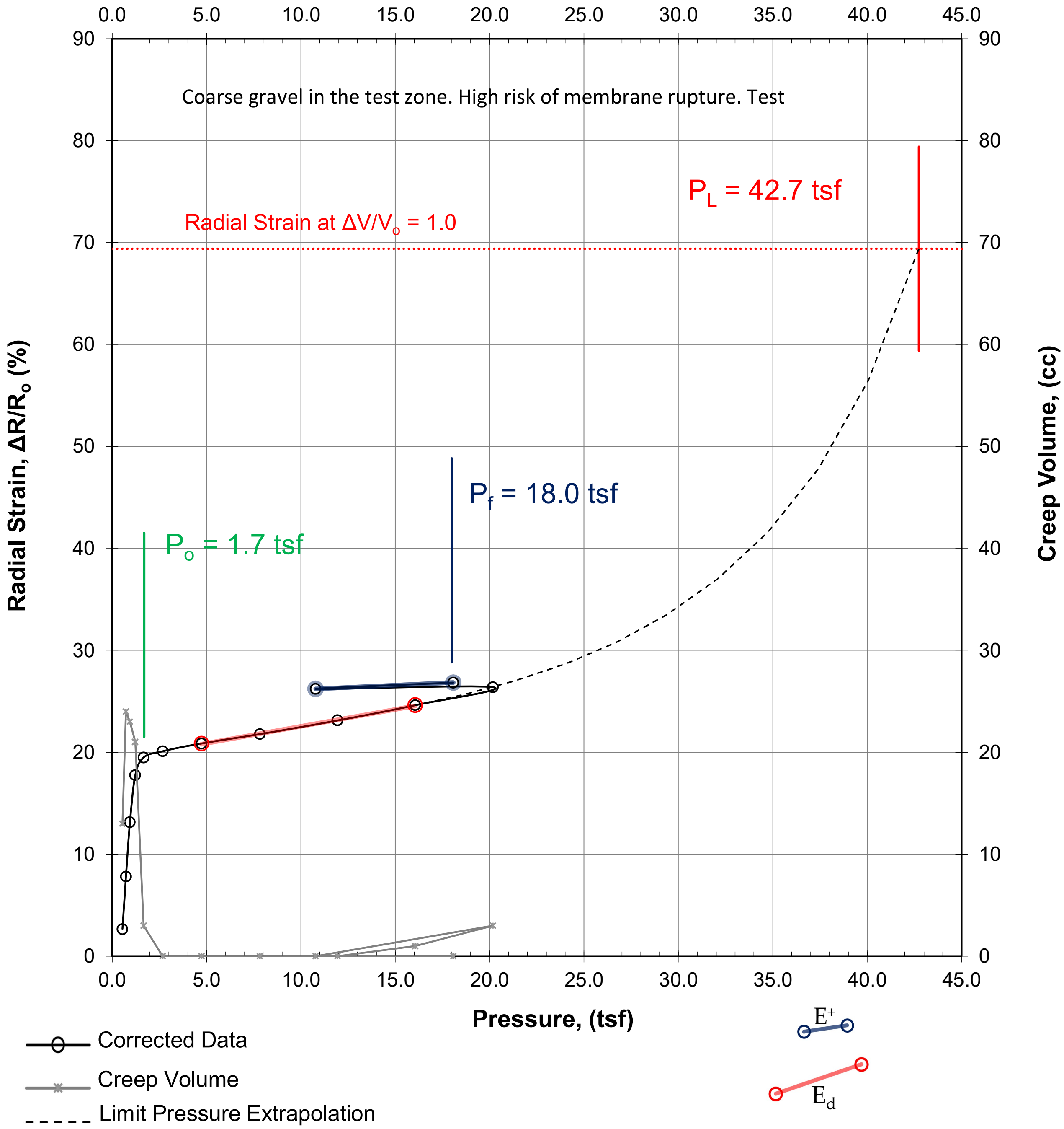
GEI Consultants
Consulting Engineers & Scientists

Project Name: 415 N. Lake St.
GEI Job #: 2204584

Boring No.: PMT-1
Test Depth (ft): 17.5 to 20.0



Test Results

P_o (tsf)	P_f (tsf)	P_L (tsf)	P^*_L (tsf)	E_d (tsf)	E^+ (tsf)	E_d/E^+	E_d/P^*_L	P_L/P_f
1.7	18.0	42.7	41.0	492	1952	0.25	12.0	2.4

GEI Consultants
Consulting Engineers & Scientists

Project Name: 415 N. Lake St.
GEI Job #: 2204584
Test Date: Tuesday, November 29, 2022
Boring No.: PMT-1
Test Depth (ft): 29.0 to 31.5
Test Zone Preparation: 2-1/2 In. Split Spoon

Probe Size: BX (58 mm)
Probe Deflated Vol. (cc): 535
Volumetric Zero Offset (cc): 48
Assumed Poisson's Ratio, ν: 0.33
Instrument Height (ft): 4
Operator: R.Rusk

No.	Field Pressure Readings (bars)	Field 30 Sec. Volume (cc)	Field 60 Sec. Volume (cc)	Creep Volume (cc)	Hydrostatic Pressure (tsf)	Membrane Resistance (tsf)	Corrected Pressure (tsf)	Corrected 30 Sec. Volume (cc)	Corrected 60 Sec. Volume, v_{60} (cc)	60 Sec. Radial Strain, $\Delta R/R_o$ (%)
1	0.5	85	103	18	1.1	0.33	1.3	36.9	54.9	5.01
2	1.0	129	147	18	1.1	0.57	1.6	80.9	98.9	8.85
3	2.0	192	205	13	1.1	0.86	2.3	143.8	156.8	13.72
4	3.0	221	223	2	1.1	0.94	3.3	172.8	174.8	15.18
5	5.0	234	234	0	1.1	0.98	5.3	185.6	185.6	16.06
6	8.0	249	250	1	1.1	1.05	8.4	200.4	201.4	17.32
7	11.0	264	265	1	1.1	1.11	11.5	215.2	216.2	18.50
8	15.0	280	280	0	1.1	1.16	15.6	230.9	230.9	19.65
9	8.0	279	279	0	1.1	1.16	8.3	230.4	230.4	19.61
10	14.0	286	286	0	1.1	1.18	14.5	237.0	237.0	20.13
11	19.0	302	305	3	1.1	1.25	19.7	252.7	255.7	21.57
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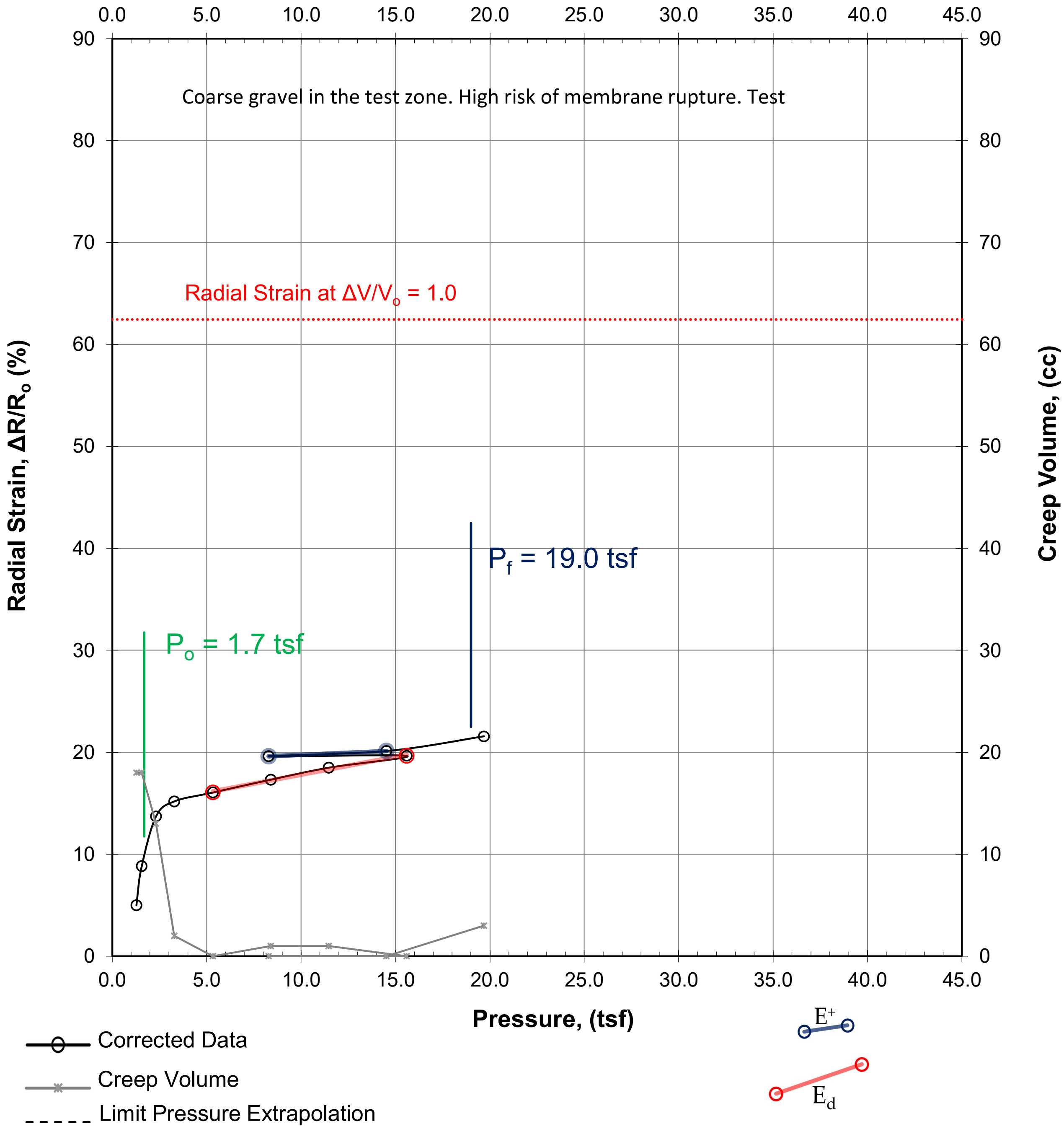
GEI Consultants
Consulting Engineers & Scientists

Project Name: 415 N. Lake St.
GEI Job #: 2204584
Boring No.: PMT-1
Test Depth (ft): 29.0 to 31.5



Test Results

P_o (tsf)	P_f (tsf)	P_L (tsf)	P^*_L (tsf)	E_d (tsf)	E^+ (tsf)	E_d/E^+	E_d/P^*_L	P_L/P_f
1.7	19.0	-	-	448	1941	0.23	-	-

GEI Consultants
Consulting Engineers & Scientists

Project Name: 415 N. Lake St.
GEI Job #: 2204584
Test Date: Tuesday, November 29, 2022
Boring No.: PMT-1
Test Depth (ft): 48.0 to 50.5
Test Zone Preparation: 2-1/2 In. Split Spoon

Probe Size: BX (58 mm)
Probe Deflated Vol. (cc): 535
Volumetric Zero Offset (cc): 48
Assumed Poisson's Ratio, ν: 0.33
Instrument Height (ft): 4
Operator: R.Rusk

No.	Field Pressure Readings (bars)	Field 30 Sec. Volume (cc)	Field 60 Sec. Volume (cc)	Creep Volume (cc)	Hydrostatic Pressure (tsf)	Membrane Resistance (tsf)	Corrected Pressure (tsf)	Corrected 30 Sec. Volume (cc)	Corrected 60 Sec. Volume, v_{60} (cc)	60 Sec. Radial Strain, $\Delta R/R_o$ (%)
1	0.5	74	86	12	1.7	0.23	2.0	25.9	37.9	3.48
2	1.0	110	125	15	1.7	0.46	2.3	61.8	76.8	6.94
3	1.5	150	166	16	1.7	0.67	2.6	101.8	117.8	10.46
4	2.0	187	194	7	1.7	0.81	3.0	138.8	145.8	12.81
5	3.0	209	209	0	1.7	0.88	4.0	160.7	160.7	14.04
6	6.0	226	226	0	1.7	0.95	7.0	177.5	177.5	15.40
7	10.0	239	239	0	1.7	1.00	11.1	190.2	190.2	16.43
8	15.0	255	255	0	1.7	1.07	16.3	205.9	205.9	17.68
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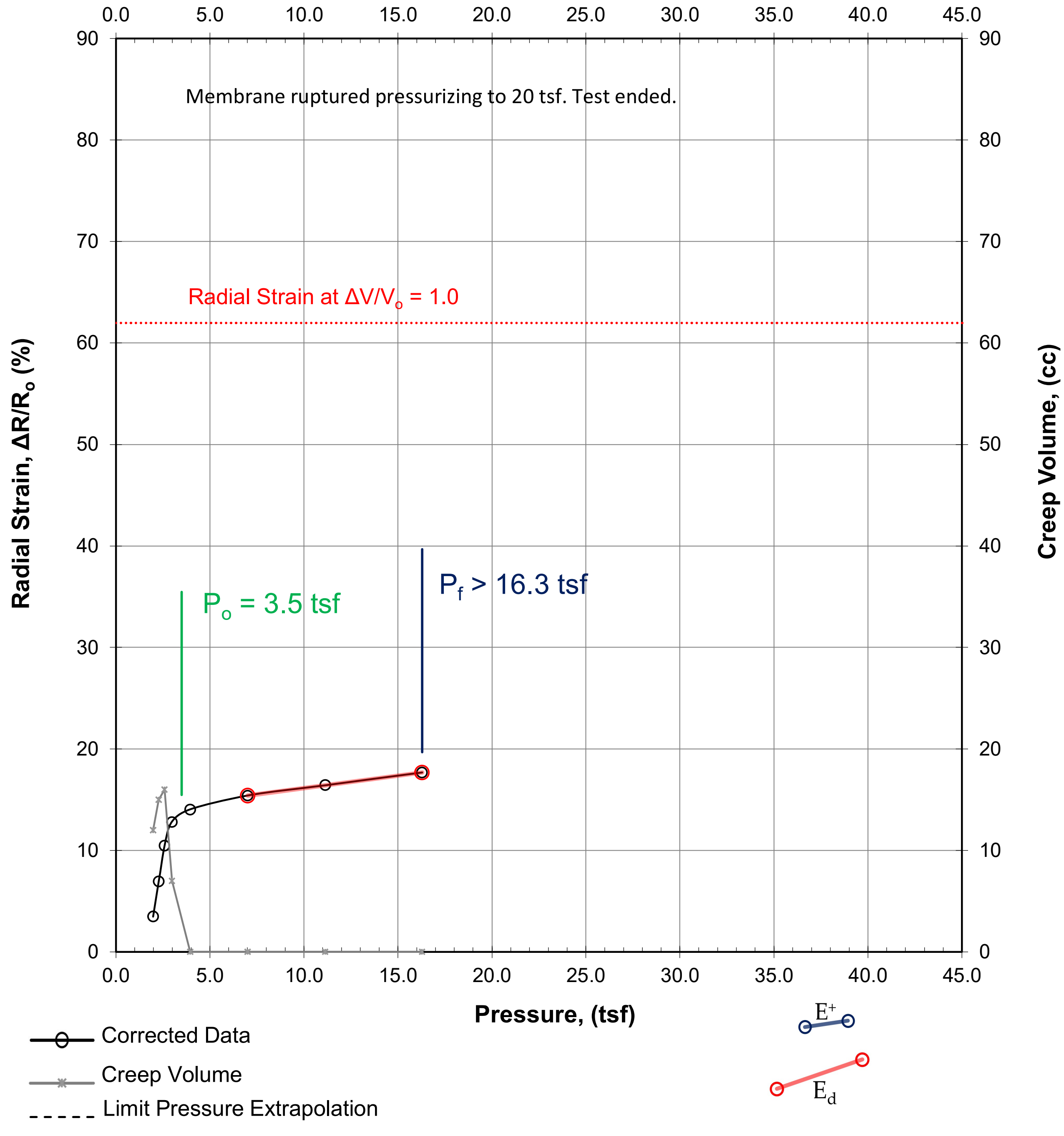
GEI Consultants
Consulting Engineers & Scientists

Project Name: 415 N. Lake St. | Boring No.: PMT-1
GEI Job #: 2204584 | Test Depth (ft): 48.0 to 50.5



Test Results

P_o (tsf)	P_f (tsf)	P_L (tsf)	P^*_L (tsf)	E_d (tsf)	E^+ (tsf)	E_d/E^+	E_d/P^*_L	P_L/P_f
3.5	>16.3	-	-	633	-	-	-	-

GEI Consultants
Consulting Engineers & Scientists

Project Name: 415 N. Lake St.
GEI Job #: 2204584
Test Date: Monday, November 28, 2022
Boring No.: PMT-9
Test Depth (ft): 15.5 to 18.0
Test Zone Preparation: 2-1/2 In. Split Spoon

Probe Size: BX (58 mm)
Probe Deflated Vol. (cc): 535
Volumetric Zero Offset (cc): 11
Assumed Poisson's Ratio, ν: 0.33
Instrument Height (ft): 4
Operator: R.Rusk

No.	Field Pressure Readings (bars)	Field 30 Sec. Volume (cc)	Field 60 Sec. Volume (cc)	Creep Volume (cc)	Hydrostatic Pressure (tsf)	Membrane Resistance (tsf)	Corrected Pressure (tsf)	Corrected 30 Sec. Volume (cc)	Corrected 60 Sec. Volume, v_{60} (cc)	60 Sec. Radial Strain, $\Delta R/R_o$ (%)
1	0.0	33	48	15	0.7	0.23	0.4	22.0	37.0	3.40
2	0.5	81	103	22	0.7	0.54	0.6	70.0	92.0	8.25
3	1.0	135	150	15	0.7	0.77	0.9	123.9	138.9	12.24
4	2.3	192	205	13	0.7	1.02	2.0	180.9	193.9	16.72
5	3.0	226	235	9	0.7	1.13	2.7	214.8	223.8	19.09
6	5.0	256	261	5	0.7	1.23	4.7	244.7	249.7	21.11
7	8.0	276	277	1	0.7	1.28	7.7	264.5	265.5	22.32
8	12.0	295	297	2	0.7	1.34	11.9	283.2	285.2	23.82
9	16.0	317	320	3	0.7	1.40	16.0	304.9	307.9	25.52
10	20.0	345	352	7	0.7	1.48	20.1	332.6	339.6	27.86
11	12.0	365	366	1	0.7	1.52	11.7	353.2	354.2	28.92
12	19.0	383	384	1	0.7	1.55	18.9	370.7	371.7	30.18
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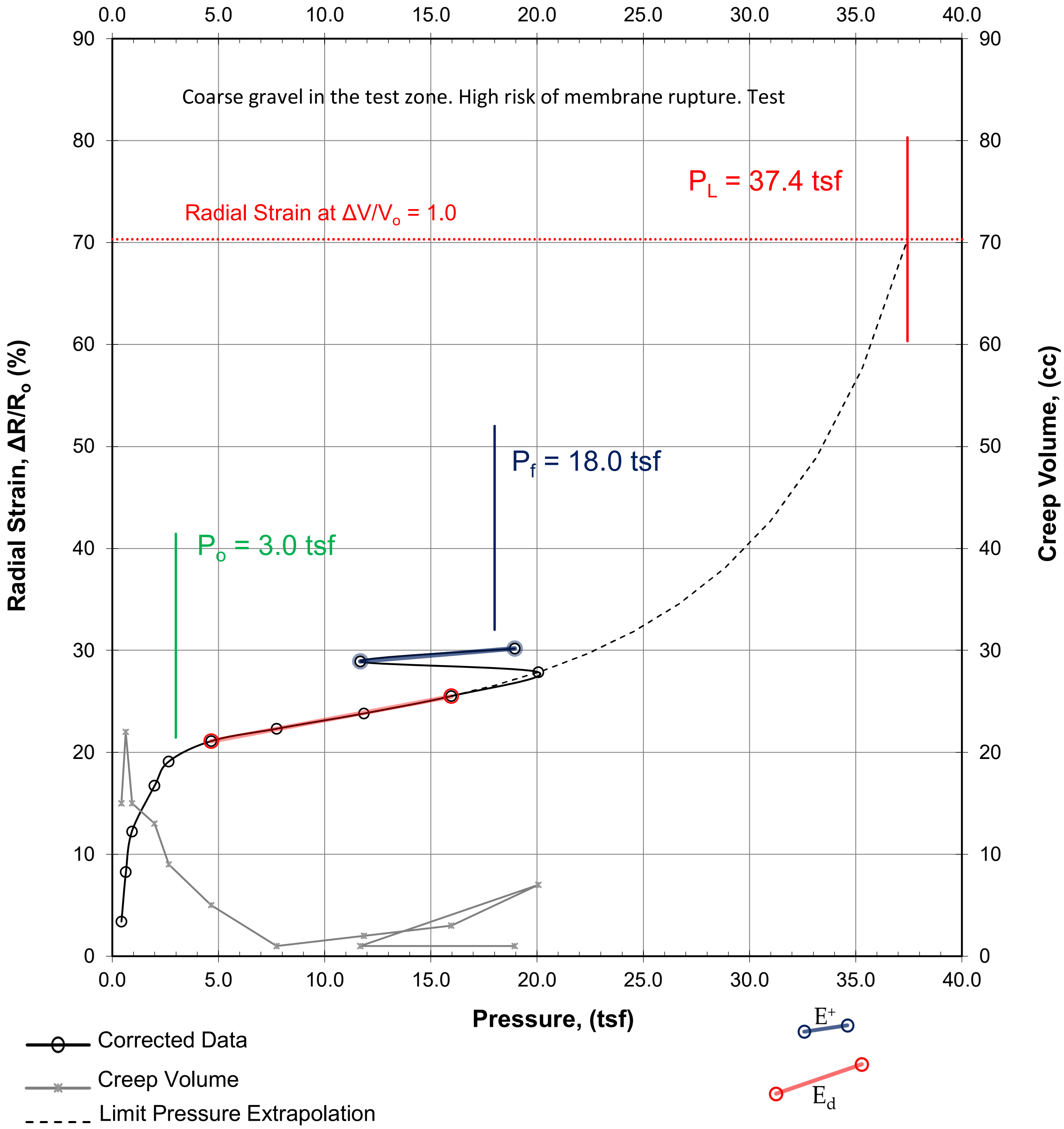
Project Name:	415 N. Lake St.	Boring No.:	PMT-9		
GEI Job #:	2204584	Test Depth (ft):	15.5	to	18.0



Test Results

P_o (tsf)	P_f (tsf)	P_L (tsf)	P^*_L (tsf)	E_d (tsf)	E^+ (tsf)	E_d/E^+	E_d/P^*_L	P_L/P_f
3.0	18.0	37.4	34.4	423	992	0.43	12.3	2.1

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Project Name: 415 N. Lake St.
GEI Job #: 2204584
Test Date: Monday, November 28, 2022
Boring No.: PMT-9
Test Depth (ft): 26.0 to 28.5
Test Zone Preparation: 2-1/2 In. Rock Bit

Probe Size: BX (58 mm)
Probe Deflated Vol. (cc): 535
Volumetric Zero Offset (cc): 11
Assumed Poisson's Ratio, ν: 0.33
Instrument Height (ft): 4
Operator: R.Rusk

No.	Field Pressure Readings (bars)	Field 30 Sec. Volume (cc)	Field 60 Sec. Volume (cc)	Creep Volume (cc)	Hydrostatic Pressure (tsf)	Membrane Resistance (tsf)	Corrected Pressure (tsf)	Corrected 30 Sec. Volume (cc)	Corrected 60 Sec. Volume, v_{60} (cc)	60 Sec. Radial Strain, $\Delta R/R_o$ (%)
1	0.0	36	45	9	1.0	0.21	0.8	24.9	33.9	3.12
2	1.0	80	135	55	1.0	0.70	1.3	68.9	123.9	10.98
3	2.0	176	236	60	1.0	1.14	1.9	164.9	224.9	19.18
4	3.0	268	300	32	1.0	1.35	2.8	256.8	288.8	24.09
5	4.0	318	324	6	1.0	1.42	3.8	306.7	312.7	25.88
6	6.0	348	351	3	1.0	1.48	5.8	336.6	339.6	27.86
7	9.0	374	379	5	1.0	1.54	8.9	362.4	367.4	29.87
8	12.0	396	400	4	1.0	1.58	11.9	384.2	388.2	31.36
9	15.0	416	420	4	1.0	1.61	15.0	404.0	408.0	32.76
10	18.0	436	440	4	1.0	1.64	18.2	423.8	427.8	34.15
11	21.0	456	460	4	1.0	1.66	21.3	443.6	447.6	35.52
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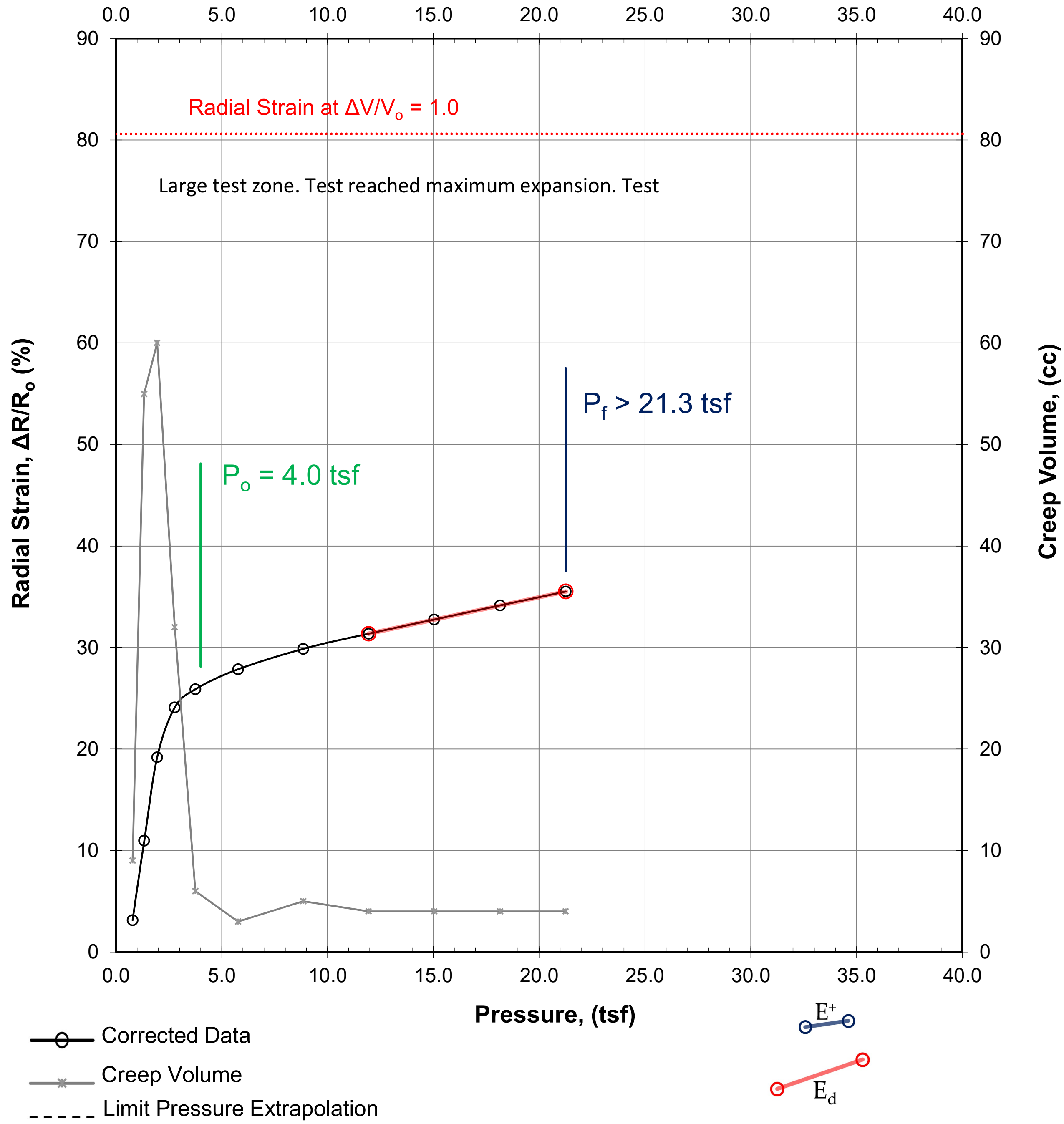
Project Name: 415 N. Lake St.
GEI Job #: 2204584

Boring No.: PMT-9
Test Depth (ft): 26.0 to 28.5



Test Results

P_o (tsf)	P_f (tsf)	P_L (tsf)	P^*_L (tsf)	E_d (tsf)	E^+ (tsf)	E_d/E^+	E_d/P^*_L	P_L/P_f
4.0	>21.3	-	-	398	-	-	-	-

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Consulting Engineers & Scientists

Project Name: 415 N. Lake St.
GEI Job #: 2204584
Test Date: Monday, November 28, 2022
Boring No.: PMT-9
Test Depth (ft): 29.5 to 32.0
Test Zone Preparation: 2-1/2 In. Rock Bit

Probe Size: BX (58 mm)
Probe Deflated Vol. (cc): 535
Volumetric Zero Offset (cc): 11
Assumed Poisson's Ratio, ν: 0.33
Instrument Height (ft): 4
Operator: R.Rusk

No.	Field Pressure Readings (bars)	Field 30 Sec. Volume (cc)	Field 60 Sec. Volume (cc)	Creep Volume (cc)	Hydrostatic Pressure (tsf)	Membrane Resistance (tsf)	Corrected Pressure (tsf)	Corrected 30 Sec. Volume (cc)	Corrected 60 Sec. Volume, v_{60} (cc)	60 Sec. Radial Strain, $\Delta R/R_o$ (%)
1	0.0	21	25	4	1.1	0.09	1.0	9.9	13.9	1.29
2	1.0	65	134	69	1.1	0.70	1.5	53.9	122.9	10.89
3	2.0	161	188	27	1.1	0.95	2.2	149.8	176.8	15.35
4	3.0	214	224	10	1.1	1.09	3.1	202.8	212.8	18.23
5	5.0	246	256	10	1.1	1.21	5.1	234.7	244.7	20.72
6	7.0	277	286	9	1.1	1.31	7.1	265.5	274.5	23.01
7	10.0	318	322	4	1.1	1.41	10.1	306.3	310.3	25.70
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Project Name: 415 N. Lake St.
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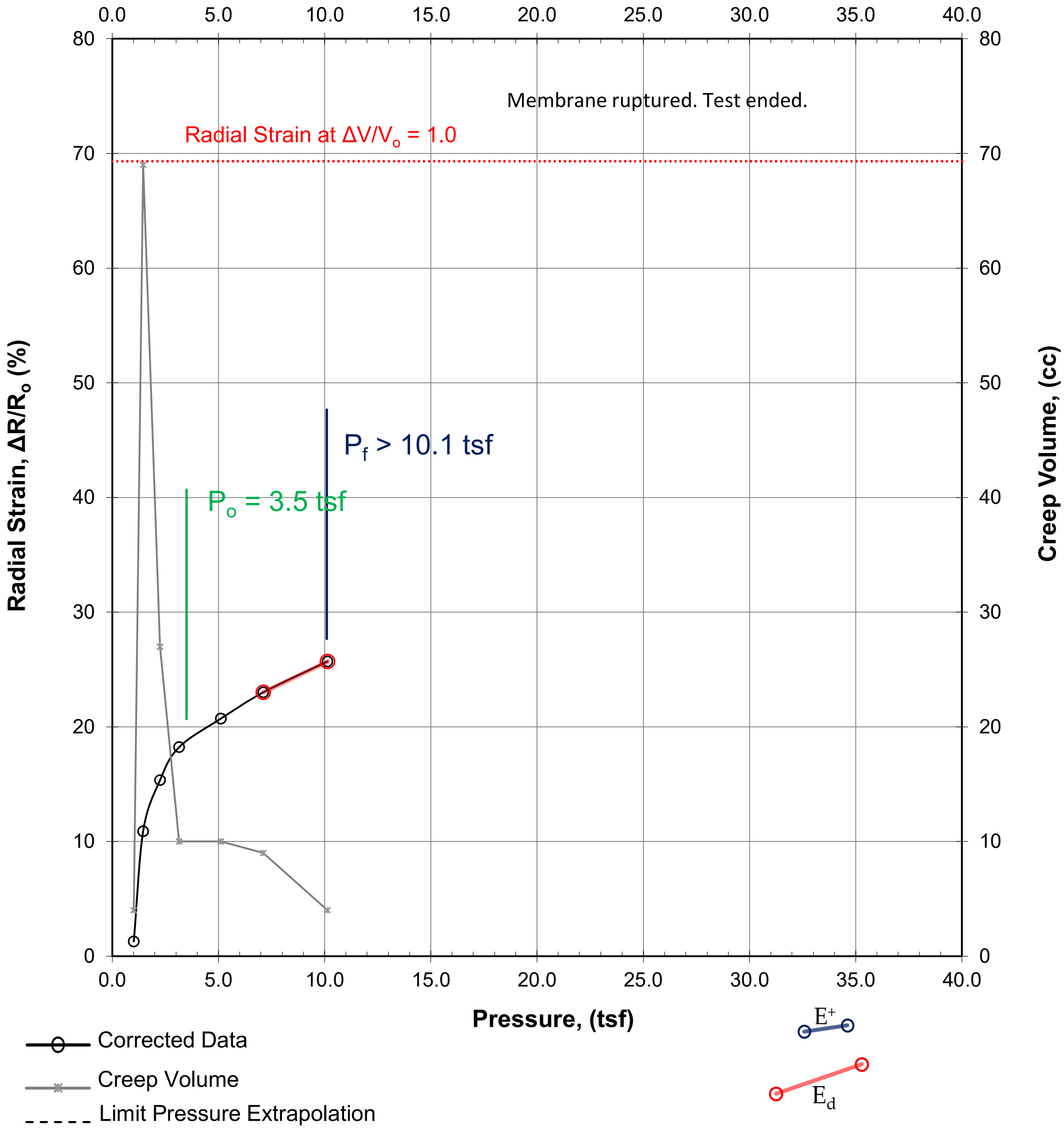
Test Depth (ft): 29.5 to 32.0



Test Results

P_o (tsf)	P_f (tsf)	P_L (tsf)	P^*_L (tsf)	E_d (tsf)	E^+ (tsf)	E_d/E^+	E_d/P^*_L	P_L/P_f
3.5	>10.1	-	-	186	-	-	-	-

Pressuremeter Procedures

Introduction

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

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

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

Pressuremeter Test

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

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

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

General Uses

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

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

Apparatus

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

Interpretation of Test Results

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

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

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

General Equations

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

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

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

where q = Ultimate bearing capacity

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

P_L = Limit pressure of the soil

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

P_V = Overburden pressure at foundation level

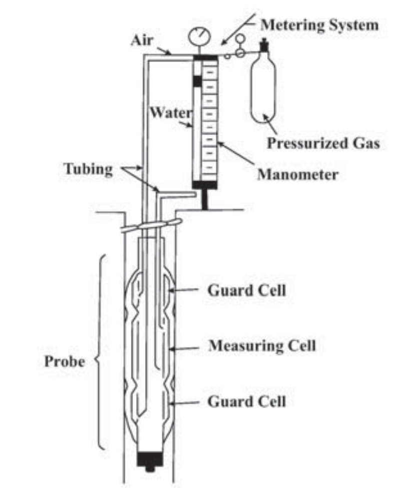
The calculations of settlements for a foundation are based upon the following formula:

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

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

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

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



Schematic Diagram Showing Pressuremeter in a Borehole

Figure 1

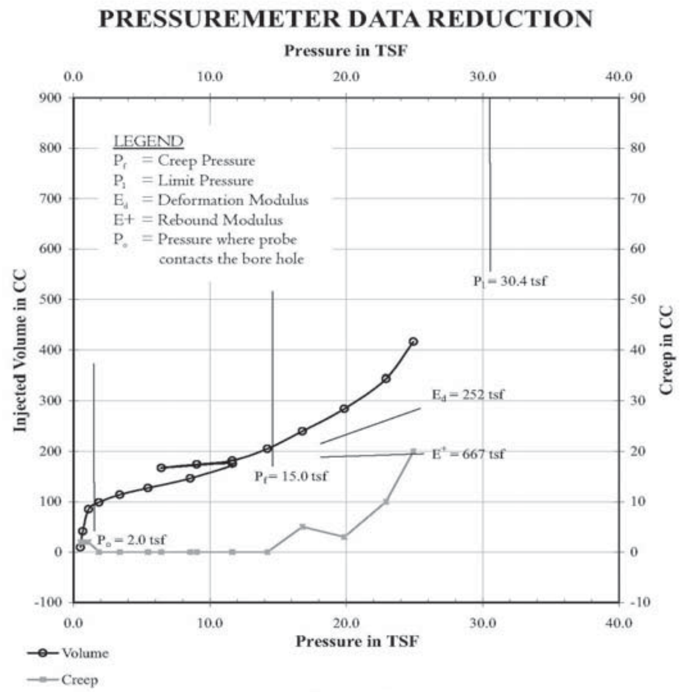


Figure 2

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