Geotechnical Engineering Report

Proposed Mountain View Tower Improvements

1600 Downing Street

Denver, Colorado

September 29, 2015

Terracon Project No. 25155132

Prepared for:

Urban Land Conservancy
Denver, Colorado

Prepared by:

Terracon Consultants, Inc.
Wheat Ridge, Colorado

terracon.com

Terracon

Environmental ■ Facilities ■ Geotechnical ■ Materials

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September 29, 2015

Urban Land Conservancy
305 Park Avenue West, Unite B
Denver, Colorado 80205

Attn: Ms. Kellie Myers
kmyers@urbanlandc.org
303.377.4477 ext. 19

Re: Geotechnical Engineering Report
Proposed Mountain View Tower Improvements
1600 Downing Street
Denver, Colorado
Terracon Project Number: 25155132

Ms. Myers:

Terracon Consultants, Inc. (Terracon) has completed the geotechnical engineering exploration for the above referenced project. This study was performed in general accordance with our proposal number P25150842 dated August 20, 2015. This report presents the findings of the subsurface exploration and provides geotechnical recommendations concerning earthwork and the design and construction of foundations and slabs-on-grade for the proposed project.

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

Sincerely,
TERRACON CONSULTANTS, INC.

COLORADO REGISTERED
WILLIAM D. RETHAMEL
36508
9/29/15
PROFESSIONAL ENGINEER

William D. Rethamel, P.E.
Senior Project Engineer

Scott Myers, P.E.
Geotechnical Department Manager

Enclosures
cc: 4 – Above
1 – File

EXECUTIVE SUMMARY

A geotechnical engineering exploration has been performed for the proposed elevator addition to the existing building located at 1600 Downing Street in Denver, Colorado. Based on the information obtained from our subsurface exploration and the laboratory testing completed, the site appears suitable for the proposed construction; however, the following geotechnical conditions will need to be considered:

- Up to about 1/2 foot of fill materials was encountered in the boring drilled for this exploration. We anticipate deeper backfill on the order of about 9 feet will be encountered where the elevator addition approaches the existing structure. It is our opinion the existing fill should not be used to support foundation and interior slab construction without complete removal and modification. Support of exterior slab construction on the existing fill materials can be considered, provided a portion of the existing fill materials are overexcavated, processed, moisture conditioned and recompacted and some movement can be tolerated.

- We understand the addition is planned to be constructed on a helical pile foundation. Based on the geotechnical engineering analyses, it is our opinion the proposed addition could be constructed on deep foundations such as helical piles or drilled piers bottomed in bedrock. Alternatively, spread footing foundation systems underlain by native sand/silt soils or new engineered fill could also be used, provided the owner is willing to accept a higher associated risk of movement.

- Provided the owner is willing to accept some risk of movement, conventional slabs-on-grade may be constructed on the native soils. If the owner is not willing to accept some risk of movement, the interior floor system should be designed as a structurally supported floor.

- Based on the 2012 International Building Code, Table 1613.5.2, and the City of Denver Amendments to the 2009 International Building Code (IBC), the IBC seismic site classification for this site is D.

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

- The amount of movement associated with foundations and flatwork, etc. will be related to the wetting of the underlying soil materials. Therefore, it is imperative the recommendations outlined in the "Grading and Drainage" section of this report be followed to reduce potential movement. Fill placement (if any) should follow the recommendations outlined in the "Earthwork" section of this report.

This summary should be used in conjunction with the entire report for design purposes. It should be recognized that details were not included or fully developed in this section, and this report must be read in its entirety for a comprehensive understanding of the items contained herein. The section titled **GENERAL COMMENTS** should be read for an understanding of the report limitations.

GEOTECHNICAL ENGINEERING REPORT
PROPOSED MOUNTAIN VIEW TOWER IMPROVEMENTS
1600 DOWNING STREET
DENVER, COLORADO

Terracon Project No. 25155132
September 29, 2015

1.0 INTRODUCTION

A geotechnical engineering report has been completed for the proposed Mountain View Tower Improvements located at 1600 Downing Street in Denver, Colorado.

As part of our subsurface exploration, one (1) exploratory boring was drilled to a depth of about 30 feet below the existing site grade. The Boring Log and the Boring Location Plan are included in Appendix A of this report.

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

- Subsurface soil and bedrock conditions
- Groundwater levels
- Earthwork
- Foundation design and construction
- Floor slab design and construction
- Lateral earth pressure
- Seismic considerations
- Grading and drainage

2.0 PROJECT INFORMATION

2.1 Project Description

Item	Description
Proposed construction	We understand the project will consist of the addition of a seven-story elevator shaft on the south side of the existing building. The addition will be about 10 feet by 8 feet in plan area. Plans indicate the elevator addition will be constructed on a helical pile foundation with cast-in-place elevator pit walls. The elevator shaft will be steel-framed. We assume the addition will have an exterior to match the existing building. The elevator pit will extend about 5 feet below the first level finished floor.
Anticipated foundation systems	Helical piles

Item	Description
Below grade areas	None
Maximum loads	Total service compression: 82 kips (provided) Net service tension: (16 kips (provided) Slabs: 150 to 300 psf (assumed)
Grading	None anticipated
Excavation depth	4 to 6 feet (anticipated)
Infrastructure	No exterior infrastructure anticipated

2.2 Site Location and Description

Item	Description
Location	The proposed elevator addition will be located on the south side of the existing Mountain View Tower at 1600 Downing Street in Denver, Colorado. The general location of the proposed project is 39.7418° N 104.9729° W.
Existing improvements	An existing 7-story office building is located on the subject site. We understand the building has a basement level extending about 9 feet below the first level finished floor. The foundation system of the existing building is not known at this time. The area of the proposed elevator addition is currently occupied by concrete sidewalk and gravel landscaping.
Current ground cover	Ground cover in the area of the proposed boring consists of irrigated landscaping.
Existing topography	The topography appears to be generally level with an elevation difference of about 2 feet or less in the area of the proposed boring.

3.0 SUBSURFACE CONDITIONS

3.1 Typical Profile

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

Material Description	Approximate Depth to Bottom of Stratum Below Existing Site Grade	Relative Density / Consistency
Fill materials consisting of lean	About 1/2 foot	N/A

Material Description	Approximate Depth to Bottom of Stratum Below Existing Site Grade	Relative Density / Consistency
clay with varying amounts of organics and sand		
Native soils consisting of sand with varying amounts of silt, clay and gravel	About 20-1/2 feet	Sand: loose to medium dense
Bedrock consisting of claystone	About 30 feet	Hard

Stratification boundaries on the boring log represent the approximate location of changes in soil and material types; in-situ, the transition between materials may be gradual. Further details of the boring can be found on the Boring Log in Appendix A.

Based on the laboratory testing and our experience in the area, the clay fill materials and native sand and silt soils have nil to low expansion potential. The claystone bedrock is considered to have low expansion potential.

The samples tested have the following physical and engineering properties:

Boring No.	Depth (ft.)	Fines Content (%)	Liquid Limit (%)	Plasticity Index (%)	Expansion/ Consolidation (%)[1]
1	4	65	20	3	
1	9				-0.2
1	19				-0.9

[1] Expansion/consolidation testing was performed under a 500 psf surcharge load

Laboratory testing for water soluble sulfate concentrations indicated a level of about 1 mg/l. A summary of the laboratory test results is included in Appendix B.

3.2 Groundwater

The boring was observed while drilling for the presence and level of groundwater. The groundwater levels observed while drilling and after completion are presented on the Boring Log included in Appendix A, and are summarized below.

Boring No.	Depth to groundwater while drilling (ft.)	Depth to groundwater at least one day after drilling (ft.)
1	None encountered	--*

* Due to safety concerns, the boring was backfilled immediately after obtaining the initial groundwater measurements.

These observations represent groundwater conditions at the time of the field exploration, and may not be indicative of other times or at other locations. Groundwater levels can be expected to fluctuate with varying seasonal and weather conditions, and other factors.

Based upon review of USGS maps, ([1]Hillier, et al, 1983), regional groundwater beneath the project area is generally located in the Denver Aquifer, below a depth of 20 feet, and commonly more than a depth of 100 feet below present ground surface. Locally, shallow groundwater can be found in alluvial and colluvial deposits along modern streambeds.

Zones of perched and/or trapped groundwater may occur at times in the subsurface soils overlying bedrock, on top of the bedrock surface or within permeable fractures in the bedrock materials. The location and amount of perched water is dependent upon several factors, including hydrologic conditions, type of site development, irrigation demands on or adjacent to the site, fluctuations in water features, seasonal and weather conditions.

Groundwater level fluctuations occur due to seasonal variations in the amount of rainfall, runoff and other factors not evident at the time the boring was performed. Therefore, groundwater levels during construction or at other times in the life of the structure may be higher or lower than the levels indicated on the boring log. The possibility of groundwater level fluctuations should be considered when developing the design and construction plans for the project.

4.0 RECOMMENDATIONS FOR DESIGN AND CONSTRUCTION

4.1 Geotechnical Considerations

Based on subsurface conditions encountered in the boring, the site appears suitable for the proposed construction from a geotechnical point of view provided certain precautions and design and construction recommendations outlined in this report are followed. We have identified geotechnical conditions that could impact design and construction of the proposed addition and other site improvements.

[3]Hillier, Donald E.; Schneider, Paul A., Jr.; and Hutchinson, E. Carter, 1983, ***Depth to Water Table (1976-1977) in the Greater Denver Area, Front Range Urban Corridor, Colorado***, United States Geological Survey, Map I-856-K.

4.1.1 Existing Fill Materials

Up to about ½-foot of fill materials was encountered in the exploratory boring drilled at this site. It should be noted that fill depth presented in the boring log is approximate and the depth and composition of fill should be expected to vary. We anticipate deeper fill on the order of about 9 feet will be encountered where the elevator addition approaches the existing structure. We do not possess any information regarding whether the fill was placed under the observation of a geotechnical engineer.

Based upon the results of our field exploration and laboratory testing, it is our opinion the existing fill should not be used to support foundations, interior slabs or exterior slabs-on-grade construction without complete removal and modification.

If the owner is willing to accept a higher risk of movement for exterior slabs, consideration could be given to overexcavating the existing fill materials below these elements to a depth of about 2 feet, then processing, moisture conditioning and compacting the materials back to subgrade elevation.

It should be noted that there exists the potential for construction debris and/or domestic trash to be encountered within the fill on some portions of the site. Since construction debris was not encountered within the boring, the potential for encountering construction debris and/or domestic trash is considered to be low. This should be verified by additional geotechnical exploration or evaluation at the site. If additional exploration is not performed, the owner should make allowances for such conditions to exist in the preparation of the project budget and/or construction plans.

The existing fill can be reused as engineered fill below foundations and slabs-on-grade, provided any deleterious materials are removed and some movement can be tolerated. Some removal and replacement may be required if unsuitable or soft materials are exposed.

4.1.2 Foundation Interaction

Based on our observations and review of the proposed elevator plans, the foundation for the elevator addition will be approximately 5 feet above the foundation elevation of the existing building. Care should be used while excavating adjacent to the existing foundations of the building to avoid disturbing these foundation elements. If excavations need to extend below the depths of the existing foundations, we should be contacted to provide additional recommendations. Shoring of the existing foundations may be required.

Differential movement between the existing building and the proposed elevator addition will likely occur; therefore, if possible, we recommend the addition be structurally independent of the existing building. We estimate the differential movement between the addition and the existing building could be about 1 inch, if the addition is constructed on a spread footing foundation system. If the proposed addition is constructed on a drilled pier foundation system, we anticipate the differential movement to be on the order of about 1/4 inch or less.

4.2 Earthwork

The following presents recommendations for site preparation, excavation, subgrade preparation and placement of engineered fills on the project. All earthwork on the project should be observed and evaluated by Terracon. The evaluation of earthwork should include overexcavation operations, observation and testing of engineered fills, subgrade preparation, foundation bearing soils and other geotechnical conditions exposed during the construction of the project.

4.2.1 Site Preparation

Strip and remove existing vegetation, debris, organics and other deleterious materials from proposed building and pavement areas. All exposed surfaces should be free of mounds and depressions that could prevent uniform compaction.

Stripped materials consisting of vegetation, unsuitable fills and organic materials should be wasted from the site or used to revegetate landscaped areas or exposed slopes after completion of grading operations.

The site should be initially graded to create a relatively level surface to receive fill and to provide for a relatively uniform thickness of fill beneath the proposed structure. All exposed areas that will receive fill, once properly cleared, should be scarified to a minimum depth of 12 inches, conditioned to near optimum moisture content and compacted.

Although evidence of underground facilities such as grease pits, septic tanks and cesspools was not observed during the site reconnaissance, such features could be encountered during construction. If unexpected fills or underground facilities are encountered, such features should be removed and the excavation thoroughly cleaned prior to fill placement and/or construction.

It is anticipated that excavations for the proposed construction can be accomplished with conventional earthmoving equipment.

Depending upon depth of excavation and seasonal conditions, groundwater could be encountered in excavations on the site. Pumping from sumps may be utilized to control water within excavations, if necessary.

The stability of subgrade soils may be affected by precipitation and seasonal groundwater conditions, repetitive construction traffic or other factors. Where unstable conditions are encountered or develop during construction, workability may be improved by overexcavation of wet zones and mixing these soils with crushed gravel or recycled concrete and recompaction. Use of geotextiles could also be considered as a stabilization technique. Lightweight excavation equipment may be required to reduce subgrade pumping.

The individual contractor(s) is responsible for designing and constructing stable, temporary excavations as required maintaining stability of both the excavation sides and bottoming. All excavations should be sloped or shored in the interest of safety following local and federal regulations, including current Occupational Safety and Health Administration (OSHA) excavation and trench safety standards.

4.2.2 Material Types

Engineered fill should meet the following material property requirements:

Fill Type [1]	USCS Classification	Acceptable Location for Placement
On-Site Silt and Clay Soils	ML, CL	On-site silt and clay soils are considered suitable for reuse as compacted fill below foundations and slabs-on-grade.
On-Site Sand Soils	SC, SM, SP-SM	On-site sand soils are considered suitable for reuse as compacted fill below foundations and slabs-on-grade.
Imported Soils	Varies	Imported soils meeting the gradation outlined herein can be considered acceptable for use as engineered fill beneath foundations, slabs and pavements.

1. Controlled, compacted fill should consist of approved materials that are free of organic matter and debris. Frozen material should not be used, and fill should not be placed on a frozen subgrade. A sample of each material type should be submitted to the geotechnical engineer for evaluation.
2. Care should be taken during the fill placement process to avoid zones of dis-similar fill. Improvements constructed over varying fill types are at a higher risk of differential movement compared to improvements over a uniform fill zone.

On-site and imported soils (if required) should meet the following material property requirements:

Gradation	Percent finer by weight (ASTM C136)
3”	100
No. 4 Sieve	50-100
No. 200 Sieve	15-35

- Liquid Limit……30 (max)
- Plasticity Index……..15 (max)
- Maximum Expansive Potential (%)……..1.0*

*Measured on a sample compacted to approximately 95 percent of the ASTM D698 maximum dry density at optimum water content. The sample is confined under a 200 psf surcharge and submerged.

4.2.3 Compaction Requirements

Engineered fill should be placed and compacted in horizontal lifts, using equipment and procedures that will produce recommended moisture contents and densities throughout the lift.

Item	Description
Fill Lift Thickness	8-inches or less in loose thickness when heavy, self-propelled compaction equipment is used 4 to 6-inches in loose thickness when hand-guided equipment (i.e. jumping jack, plate compactor) is used
Compaction Requirements	Minimum 98% of the material's standard Proctor maximum dry density (ASTM D698)
Moisture Content Cohesive Soil (clay)	0 to +3 % of the optimum moisture content
Moisture Content Cohesive Soil (silt)	-2 to +2 % of the optimum moisture content
Moisture Content Cohesionless Soil (sand)	-2 to +2 % of the optimum moisture content

1. We recommend engineered fill be tested for moisture content and compaction during placement. Should the results of the in-place density tests indicate the specified moisture or compaction limits have not been met, the area represented by the test should be reworked and retested as required until the specified moisture and compaction requirements are achieved.
2. Specifically, moisture levels should be maintained low enough to allow for satisfactory compaction to be achieved without the fill material pumping when proofrolled.
3. Moisture conditioned clay soils should not be allowed to dry out. A loss of moisture within these materials could result in an increase in the materials expansive potential. Subsequent wetting of these materials could result in undesirable movement.

4.2.4 Excavation and Trench Construction

Excavations into the on-site soils will encounter a variety of conditions. Excavations within the clay fill may remain stable for a short period of time. Excavations into the native sand soils may be subject to sloughing. The individual contractor(s) should be made responsible for designing and constructing stable, temporary excavations as required to maintain stability of both the excavation sides and bottom. All excavations should be sloped or shored in the interest of safety following local and federal regulations, including current OSHA excavation and trench safety standards.

Soils penetrated by the proposed excavations may vary significantly across the area of the proposed elevator addition. The soil classifications are based solely on the materials

encountered in the exploratory boring. The contractor should verify that similar conditions exist throughout the proposed area of excavation. If different subsurface conditions are encountered at the time of construction, the actual conditions should be evaluated to determine any excavation modifications necessary to maintain safe conditions.

As a safety measure, it is recommended that all vehicles and soil piles be kept to a minimum lateral distance from the crest of the slope equal to no less than the slope height. The exposed slope face should be protected against the elements.

4.2.5 Grading and Drainage

All grades must be adjusted to provide positive drainage away from the addition and building during construction and maintained throughout the life of the proposed project. Infiltration of water into utility or foundation excavations must be prevented during construction. Landscaped irrigation adjacent to the foundation systems should be minimized or eliminated. Water permitted to pond near or adjacent to the perimeter of the structure (either during or post-construction) can result in significantly higher soil movements than those discussed in this report. As a result, any estimations of potential movement described in this report cannot be relied upon if positive drainage is not obtained and maintained, and water is allowed to infiltrate the fill and/or subgrade.

Exposed ground should be sloped at a minimum of 10 percent grade for at least 10 feet beyond the perimeter of the addition. Where possible, asphalt pavement or concrete flatwork should be sloped at a minimum of 2 percent beyond the building perimeter. Where Americans with Disabilities Act (ADA) or other requirements or existing site features limit the gradient, slopes on the order of ½ to 1 percent minimum are considered acceptable. Backfill against exterior walls and in utility and sprinkler line trenches should be well compacted and free of all construction debris to reduce the possibility of water infiltration. After building construction and prior to project completion, we recommend that verification of final grading be performed to document that positive drainage, as described above, has been achieved.

Flatwork will be subject to post construction movement. Maximum grades practical should be used for paving and flatwork to prevent areas where water can pond. In addition, allowances in final grades should take into consideration post-construction movement of flatwork, particularly if such movement would be critical. Where paving or flatwork abuts the structure, care should be taken that joints are properly sealed and maintained to prevent the infiltration of surface water.

Where landscape or xeriscape areas are within 10 feet of the foundation systems, the areas shall have positive drainage away from the foundation that is not hindered by landscape edging, grade variations or vegetation. In addition, consideration should be given to snow removal practices that will minimize the stockpiling of snow in planter and landscaped areas adjacent to structural improvements.

Planters located adjacent to the structure should preferably be self-contained. Sprinkler mains and spray heads should be located a minimum of 10 feet away from the building line. Roof drains should discharge on pavements or be extended away from the structure a minimum of 10 feet through the use of splash blocks or downspout extensions. A preferred alternative is to have the roof drains discharge to storm sewers by solid pipe or daylighted to a detention pond or other appropriate outfall.

4.2.6 Earthwork Construction Considerations

Upon completion of grading operations, care should be taken to maintain the moisture content of the subgrade prior to construction of foundations, slabs-on-grade, etc. Construction traffic over prepared subgrade should be minimized and avoided to the extent practical. Construction traffic over processed clay subgrade will eventually reduce the moisture content and increase the density of the subgrade. Subsequent wetting of these materials will result in undesirable movement.

The site should also be graded to prevent ponding of surface water on the prepared subgrades or in excavations. If the subgrade should become frozen, desiccated, saturated, or disturbed, the affected material should be removed or these materials should be scarified, moisture conditioned, and recompacted prior to foundation or floor slab construction.

Although the exposed subgrade is anticipated to be relatively stable upon initial exposure, unstable subgrade conditions could develop during general construction operations, particularly if the soils are wetted and/or subjected to repetitive construction traffic. Should unstable subgrade conditions develop, stabilization measures will need to be employed. Options for subgrade stabilization can include removal of unsuitable material and replacement with approved fill material. An alternative can include the use of TX-140 Tensar geogrid (or approved equivalent) overlain by Colorado Department of Transportation (CDOT) Class 5 or 6 aggregate base course. The depth of aggregate base course will depend on the severity of unstable soils.

The geotechnical engineer should be retained during the construction phase of the project to observe earthwork and to perform necessary tests and observations during overexcavation operations, excavations, subgrade preparation; proof-rolling; placement and compaction of controlled compacted fills; backfilling of excavations into the completed subgrade, and just prior to construction of slabs-on-grade.

4.2.7 Soluble Sulfate Test Results

The following table lists the results of laboratory soluble sulfate testing. This value may be used to estimate potential corrosive characteristics of the on-site soils with respect to contact with the various underground materials which will be used for project construction.

Boring No.	Sample Depth(feet)	Soluble Sulfate[1] (mg/l)
1	2	1

1. Results of soluble sulfate testing indicate that samples of the on-site soils tested possess negligible sulfate concentrations when classified in accordance with Table 4.3.1 of the ACI Design Manual. The results of the testing indicate ASTM Type I Portland Cement is suitable for project concrete on and below grade. However, if there is no (or minimal) cost differential, use of ASTM Type II Portland Cement is recommended for additional sulfate resistance of construction concrete. Concrete should be designed in accordance with the provisions of the ACI Design Manual, Section 318, Chapter 4.

4.3 Foundations

Based upon the results of the field exploration and laboratory testing program, the following foundation systems were evaluated for use in supporting the proposed elevator addition:

- Shallow Foundations – Spread Footings
- Deep Foundations – Drilled Piers or Helical Piles

We understand a helical pile foundation is planned for the addition. Based on the geotechnical engineering analyses, it is our opinion the proposed addition could be constructed on a deep foundation system such as helical piles or drilled piers bottomed in bedrock. With a deep foundation system, we estimate movement between the existing building and the addition to be on the order of about ¼ inch.

Alternatively, the addition could be constructed on spread footing foundation systems underlain by native soils or new, properly engineered fill, provided the owner is willing to accept a higher associated risk of movement; we estimate movement on the order of about 1 inch is possible.

Foundation design and construction considerations for all three systems are provided below.

4.3.1 Drilled Pier Design Recommendations

If the owner would like to reduce the amount of movement between the proposed addition and existing building to about 1/4 inch or less, the proposed addition should be constructed on drilled pier foundation systems bottomed in the underlying claystone bedrock.

Design recommendations for a drilled shaft foundation system at this site are presented in the following paragraphs.

Description	Value
Minimum pier length	25 feet

Description	Value
Minimum bedrock embedment [1]	5 feet
Maximum end-bearing pressure	20,000 psf
Skin friction [2]	1,500 psf
Uplift force (tension due to soil uplift in kips)	N/A
Minimum pier diameter	18 inches or Length/Diameter < 30
Void Thickness	N/A

1. Drilled shafts should be embedded into firm or harder bedrock materials.
2. Required shaft penetration should be balanced against uplift forces for the portion of the shaft in bedrock below a depth of 15 feet from top of pier to resist axial loads and uplift forces.
3. The upper 3 feet of the pier should be ignored for load carrying capacity.
4. Movement on the order of less than 1/4 inch should be anticipated for the drilled piers, provided the piers and properly designed and constructed.

Piers should be considered to work in group action if the horizontal spacing between pier centers is less than three pier diameters. A minimum practical horizontal clear spacing between piers of at least three diameters should be maintained, and adjacent piers should bottom at the same elevation. The capacity of individual piers must be reduced when considering the effects of group action. Capacity reduction is a function of pier spacing and the number of piers within a group. For drilled piers spaced 3 diameters apart (center to center), pier capacities should be reduced by 30 percent.

To satisfy forces in the horizontal direction using the computer program LPile©, piers may be designed for the following lateral load criteria:

Soil Layer	Unit Weight (pcf)	Undrained Shear Strength (psf)	Angle of Internal Friction, Φ(degrees)	Coeff. of Subgrade Reaction, k (pci)	Strain, ε_{50} (%)
Native sand materials	115	N/A	30	60	N/A
Claystone bedrock	120	3,500	0	2,000-static 800-cyclic	0.004

* - The claystone bedrock may be modeled as hard clay.

Lateral analysis should account for the center to center spacing and P-Y multiplier values per the following table:

Pier center to center spacing (in direction of loading)	P-multiplier, P_M Row 1	P-multiplier, P_M Row 2	P-multiplier, P_M Row 3 and higher
3 x diameter	0.8	0.4	0.3
5 x diameter	1.0	0.85	0.7

4.3.2 Drilled Pier Construction Considerations

All piers should be reinforced full depth for the applied axial, lateral and uplift stresses imposed. The amount of reinforcing steel for expansion should be determined by the tensile force created by the uplift force on each pier, with allowance for dead load.

Drilling to design depth should be possible with conventional single-flight power augers; however, potentially cemented bedrock layers could require the use of heavy-duty equipment.

If caving soils or groundwater are encountered, temporary steel casing could be required in some areas to properly drill and clean some piers prior to concrete placement. If casing is used for pier construction, it should be withdrawn in a slow continuous manner maintaining a sufficient head of concrete to prevent infiltration of water or caving soils or the creation of voids in pier concrete. Pier concrete should have a relatively high fluidity when placed in cased pier holes or through a tremie. Pier concrete with slump in the range of 5 to 7 inches is recommended for uncased piers. For cased piers, concrete slump should be in the range of 7 to 9 inches.

Groundwater should be removed from each pier hole prior to concrete placement. Pier concrete should be placed immediately after completion of drilling and cleaning. If pier concrete cannot be placed in dry conditions, a tremie should be used for concrete placement. The use of a bottom-dump hopper, or an elephant's trunk discharging near the bottom of the hole where concrete segregation will be minimized, is recommended. Due to potential sloughing and raveling, foundation concrete quantities may exceed calculated geometric volumes.

We recommend the sides of each pier should be mechanically roughened in claystone, if encountered. This should be accomplished by a roughening tooth placed on the auger. Shaft bearing surfaces must be cleaned prior to concrete placement. A representative of the geotechnical engineer should observe the bearing surface and shaft configuration.

Free-fall concrete placement in piers will only be acceptable if provisions are taken to avoid striking the concrete on the sides of the hole or reinforcing steel. The use of a bottom-dump hopper, or an elephant's trunk discharging near the bottom of the hole where concrete segregation will be minimized, is recommended.

The top of the piers should be cylindrical in shape. Forms may be necessary at the top of the piers in order to minimize the disturbance of the soils and to maintain a cylindrical shape.

4.3.3 Helical Pile Design Recommendations

Another deep foundation system that could be used for the proposed elevator addition is helical piles bottomed in the underlying bedrock. Design recommendations for a helical pile foundation system are presented below.

Description	Value
Minimum pile length	25 feet[1]
Minimum bedrock embedment [1]	N/A

1. From current ground elevation from Boring No. 1.
2. Any pile that encounters refusal conditions prior to the recommended minimum length should be predrilled in order to achieve the recommended depth.
3. Load testing is recommended to confirm design capacity.

We do not recommend using vertically installed helical piles to resist lateral loads without approved lateral load test data, as these types of foundations are typically designed to resist axial loads. Helical piles installed at a batter may be used to resist lateral loads. Typically, helical piles can be installed to a batter of up to 45 degrees from the horizontal. Only the horizontal component of the allowable axial load should be considered to resist the lateral loading and only in the direction of the batter.

The helical piles should be designed to terminate in the claystone bedrock. The pile capacity should be determined through a combination of typical bearing capacity analysis, and results of the load tests correlated to helical pile installation torque.

We recommend in addition to minimum torque, piles have a minimum length of 25 feet. For any piles that encounter refusal conditions prior to the recommended minimum length, predrilling may be required to achieve the recommended depth. We recommend a load test be performed to confirm pile capacity.

Piers should be considered to work in group action if the horizontal spacing between pier centers is less than three pier diameters. A minimum practical horizontal clear spacing between piers of at least three diameters should be maintained, and adjacent piers should bottom at the same elevation. The capacity of individual piers must be reduced when considering the effects of group action. Capacity reduction is a function of pier spacing and the number of piers within a group. For drilled piers spaced 3 diameters apart (center to center), pier capacities should be reduced by 30 percent.

The actual design of the piles including the pile capacity, helix diameter(s), shaft length, bracket attachment and configuration, and shaft diameter should be performed by an experienced helical pile contractor or structural engineer.

4.3.4 Helical Pile Construction Considerations

An experienced helical pile contractor should review the data from this report to assess the equipment required to achieve the minimum length and capacity. We recommend a minimum of one load test be conducted at the site to confirm anticipated capacities and to finalize design information.

We should be consulted to review load test data, and a representative of the geotechnical engineer should be present to observe test and production helical pile installation to verify that piles have been installed to the recommended torque and/or minimum depth and to confirm pile capacity.

4.3.5 Spread Footing Design Recommendations

As an alternative to deep foundations, a spread footing foundation system may be considered for support of the proposed addition when constructed on native soils or new engineered fill, provided the potential for movement can be tolerated. New fill materials beneath foundations (if required) should be placed and compacted as outlined in the "Earthwork" section of this report; Terracon should be contacted to observe fill placement on a full-time basis.

Design recommendations for spread footing foundation systems are presented in the following paragraphs.

Description	Value
Overexcavation/modification depth	All fill below foundations must be overexcavated to native soils
Supporting stratum	Native soils or new engineered fill
Maximum net allowable bearing pressure[1,2]	2,000 psf
Minimum dead load pressure	N/A
Void Thickness, if needed	N/A
Coefficient of friction (sliding)	0.3
Minimum footing dimensions	Isolated footings: 24 inches Continuous footings: 16 inches
Minimum embedment below finished grade for frost protection	3 feet
Approximate total movement from foundation loads[3]	About 1 inch
Estimated differential movement between existing building foundation and elevator addition[3]	About 1 inch

1. The recommended net allowable bearing pressure is the pressure in excess of the minimum surrounding overburden pressure at the footing base elevation. This pressure assumes that any existing fill or lower strength soils, if encountered, will be excavated and replaced with engineered fill.
2. Maximum allowable soil bearing pressure can be increased by 1/3 for transient loading conditions.
3. Foundation movement will depend upon the variations within the subsurface soil profile, the structural loading conditions, the embedment depth of the footings, the thickness of engineered fill, and the quality of the earthwork operations and footing construction.

Footings should be proportioned on the basis of equal total dead load pressure to reduce differential movement between adjacent footings. Total movement resulting from the anticipated structural loads is estimated to be on the order of 1 inch. Additional foundation movements could occur if water from any source infiltrates the foundation soils; therefore, proper drainage should be provided in the final design and during construction and throughout the life of the structure. Failure to maintain the proper drainage as recommended in the "Grading and Drainage" section of this report will nullify the movement estimates provided above.

4.3.6 Spread Footing Construction Considerations

Spread footing foundation systems should only be considered if some foundation movement can be tolerated. Spread footing foundations should be constructed on native soils or new structural fill. Fill below foundations should be moisture conditioned and compacted as recommended in the "Compaction Requirements" section of this report.

Care needs to be taken when excavating adjacent to existing foundations and slabs-on-grade. It may be necessary to underpin or shore existing structural elements during construction of new foundations. We should be contacted to provide additional recommendations, if necessary.

Footings and foundation walls should be detailed and reinforced as necessary to reduce the potential for distress caused by differential foundation movement.

Unstable soil conditions may be encountered in the base of the excavations for the proposed footings. If unstable conditions are encountered, these will need to be stabilized prior to the placement of structural fill or constructing the footings. The use of angular rock, recycled concrete and/or gravel pushed into the yielding subgrade are generally considered suitable means of stabilizing soft soils below foundations. Alternatively, the use of geogrid materials in conjunction with gravel have also been successfully used to stabilize soft soil conditions below foundations.

If unstable soil conditions are encountered, a representative from our office should observe the conditions to provide site specific recommendations.

4.4 Seismic Considerations

Based on our subsurface exploration and laboratory testing, it is our opinion that the soils have a low risk of liquefaction. The following table presents the seismic site classification based on the 2012 International Building Code and the City of Denver Amendments to the 2009 International Building Code (IBC):

Code Used	Site Classification
2012 International Building Code (IBC) [1]	D

1. In general accordance with the *2012 International Building Code*, Section 1613.3.2, which refers to ASCE 7, Chapter 20, Table 9.4.1.2.
2. The City of Denver has amended the IBC to indicate that most sites shall have a Site Classification of D unless bedrock is encountered within 15 feet of the foundations or site-specific geophysical testing is performed. Because the bedrock was not encountered within 15 feet from the current ground surface and based on the subsurface conditions encountered within the borings, the subject site has a Site Classification of D. Alternatively, a geophysical exploration could be utilized in order to attempt to justify a higher seismic site class.

4.5 Interior Floors

We anticipate backfills up to about 9 feet may be encountered adjacent to the building. We recommend interior slabs-on-grade are constructed on new, properly engineered fills. We estimate potential movement of slabs-on-grade constructed on properly compacted engineered fill will be on the order of about 1 inch or less. If this amount of movement cannot be tolerated, then a structurally supported floor should be used. We can provide recommendations for a structural floor, if desired.

4.5.1 Interior Floors Design Recommendations

If the owner is willing to accept the risk associated with movement of a slab-on-grade floor system, slabs-on-grade should be constructed on a new engineered fill, as recommended above. If the owner cannot accept the risk of slab movement, a structural floor should be used.

For structural design of concrete slabs-on-grade, a modulus of subgrade reaction of 120 pounds per cubic inch (pci) may be used for point or limited area loads for floors supported on an engineered fill.

Additional floor slab design and construction recommendations are as follows:

- Positive separations and/or isolation joints should be provided between slabs and all foundations, columns or utility lines to allow independent movement.

- Control joints should be provided in slabs to control the location and extent of cracking.
- Interior trench backfill placed beneath slabs should be compacted in accordance with recommended specifications outlined below.
- Floor slabs should not be constructed on frozen subgrade.
- Other design and construction considerations, as outlined in Section 302.1R of the *ACI Design Manual*, are recommended.

4.5.2 Interior Floors Construction Considerations

Movements of slab-on-grades using the above outlined technique will likely be reduced and tend to be more uniform. The estimates outlined previously assume that the other recommendations in this report are followed. Additional movement could occur should the subsurface soils become wetted to significant depths, which could result in potential excessive movement causing uneven floor slabs and severe cracking. This could be due to over watering of landscaping, poor drainage, improperly functioning drain systems, and/or broken utility lines. Therefore, it is imperative that the recommendations outlined in this section and in the "Grading and Drainage" section of this report be followed.

4.6 Exterior Flatwork

Exterior slabs-on-grade and flatwork constructed on the existing backfill will have a low to moderate risk of movement. The risk of movement can be reduced if the backfill materials are modified by overexcavating the materials to a depth of at least 2 feet and properly moisture conditioning and compacting the material to grade. Where native soils are encountered, the upper 1 foot of materials should be scarified, moisture conditioned and recompacted.

For areas of exterior flatwork where the owner would like to reduce the movement to about 1 inch, the subgrade soils below exterior flatwork should be overexcavated and prepared as recommended in the "Interior Floors" section of this report.

4.6.1 Exterior Flatwork Design Recommendations

For structural design of exterior concrete slabs-on-grade, a modulus of subgrade reaction of 120 pci may be used for point or limited area loads for exterior slabs-on-grade at this site.

Additional slab design and construction recommendations are as follows:

- Minimizing moisture increases in the backfill.
- Controlling moisture-density during placement of backfill.

- Positive separations and/or isolation joints should be provided between exterior slabs and the addition to allow independent movement.
- Control joints should be provided in slabs to control the location and extent of cracking.
- Exterior slabs should not be constructed on frozen subgrade
- Other design and construction considerations, as outlined in Section 302.1R of the *ACI Design Manual*, are recommended.

4.6.2 Exterior Flatwork Construction Considerations

Movements of exterior slab-on-grades using the above technique will likely be reduced and tend to be more uniform. Additional movement could occur should the subsurface soils become wetted to significant depths, which could result in potential excessive movement causing uneven exterior slabs and severe cracking. This could be due to over watering of landscaping, poor drainage, and/or broken utility lines. Therefore, it is imperative that the recommendations outlined in the "Grading and Drainage" section of this report be followed.

4.7 Below-grade Construction

Groundwater was not encountered in the boring drilled during our investigation to a depth of 30 feet A below-grade elevator pit will be included. Groundwater levels may fluctuate seasonally and in response to landscaping irrigation (if any).

To reduce the potential for perched groundwater entering the subterranean elements of the structures, installation of a perimeter drainage system is recommended. The drainage system should be constructed around the exterior perimeter of the elevator pit foundation and sloped at a minimum 1/8 inch per foot to a suitable outlet, such as a sump and pump system.

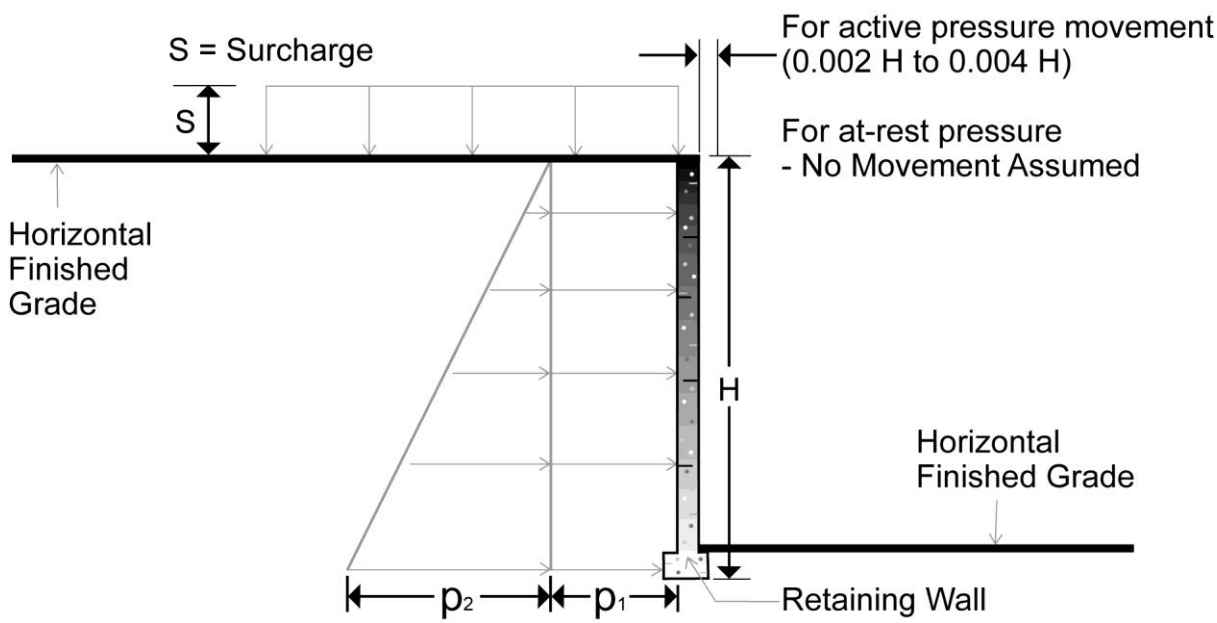
The drainage system should consist of a minimum 4-inch diameter perforated or slotted pipe, embedded in free-draining gravel, placed in a trench at least 12 inches in width. If footings are used, the edge of the trench should be sloped at a 1:1 slope beginning at the bottom outside edge of the footing. The trench should not be cut vertically at the edge of the footing.

The drainage system should consist of a properly sized, perforated pipe that is embedded in free-draining gravel and placed in a trench at least 12 inches in width. Gravel should extend a minimum of 3 inches beneath the bottom of the pipe and at least 2 feet above the bottom of the foundation wall. The system should be underlain with a polyethylene moisture barrier that is sealed to the foundation walls and extended to at least the edge of the backfill zone. The gravel should be covered with drainage fabric prior to placement of foundation backfill.

Since groundwater was not encountered in the boring to a depth of 30 feet, we do not anticipate the groundwater will enter the foundation drain system unless in the very unlikely but possible event that the groundwater level rises significantly in the future. Therefore, we anticipate the amount of current groundwater entering the foundation drain should be less than 0.5 cubic feet per second (cfs). A representative from our office should observe the subsurface soils within the excavation in order to confirm the subsurface conditions.

4.8 Lateral Earth Pressures

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



EARTH PRESSURE COEFFICIENTS

Earth Pressure Conditions	Coefficient For Backfill Type[1]	Equivalent Fluid Density (pcf)	Surcharge Pressure, p_1 (psf)	Earth Pressure, p_2 (psf)
Active (Ka)	Granular - 0.33	40	(0.33)S	(40)H
	Lean Clay - 0.53	65	(0.53)S	(65)H
At-Rest (Ko)	Granular - 0.50	60	(0.50)S	(60)H
	Lean Clay - 0.69	85	(0.69)S	(85)H
Passive (Kp)	Granular - 3.0	360	---	---
	Lean Clay – 1.9	225	---	---

1. Granular materials are considered to be sands or gravels with a maximum of 20 percent passing the No. 200 sieve.

Applicable conditions to the above include:

- For active earth pressure, wall must rotate about base, with top lateral movements of about 0.002 **H** to 0.004 **H**, where **H** is wall height
- For passive earth pressure to develop, wall must move horizontally to mobilize resistance.
- Uniform surcharge, where S is surcharge pressure
- In-situ soil backfill weight a maximum of 120 pcf
- Horizontal backfill, compacted to at least 95 percent of standard Proctor maximum dry density
- Loading from heavy compaction equipment not included
- No hydrostatic pressures acting on wall
- No dynamic loading
- No safety factor included in soil parameters

To control hydrostatic pressure behind earth-retaining walls we recommend that a drain be installed below the foundation of the wall with a collection pipe leading to a reliable discharge. If this is not possible, then combined hydrostatic and lateral earth pressures should be calculated for lean clay backfill using an equivalent fluid weighing 90 and 100 pcf for active and at-rest conditions, respectively. For granular backfill, an equivalent fluid weighing 85 and 90 pcf should be used for active and at-rest, respectively. These pressures do not include the influence of surcharge, equipment or floor loading, which should be added. Heavy equipment should not operate within a distance closer than the exposed height of retaining walls to prevent lateral pressures more than those provided.

The preceding data are applicable only to cast-in-place concrete or modular block walls up to 4 feet in height. **If taller single walls, tiered walls, or Mechanically Stabilized Earth (MSE) walls will be included in the proposed development, additional site-specific studies and laboratory testing will be required.** In addition, the wall designer should perform standard wall design practices including analysis for overturning, sliding, bearing capacity and global stability, and results of these analyses should be provided for our review. Additional sampling,

laboratory testing and document review associated with retaining walls is beyond the original scope of work but can be performed as a separate scope, for a separate fee.

5.0 GENERAL COMMENTS

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

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

The scope of services for this project does not include either specifically or by implication any environmental or biological (e.g., mold, fungi, bacteria) assessment of the site or identification or prevention of pollutants, hazardous materials or conditions. If the owner is concerned about the potential for such contamination or pollution, other studies should be undertaken.

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

APPENDIX A

FIELD EXPLORATION

Field Exploration Description

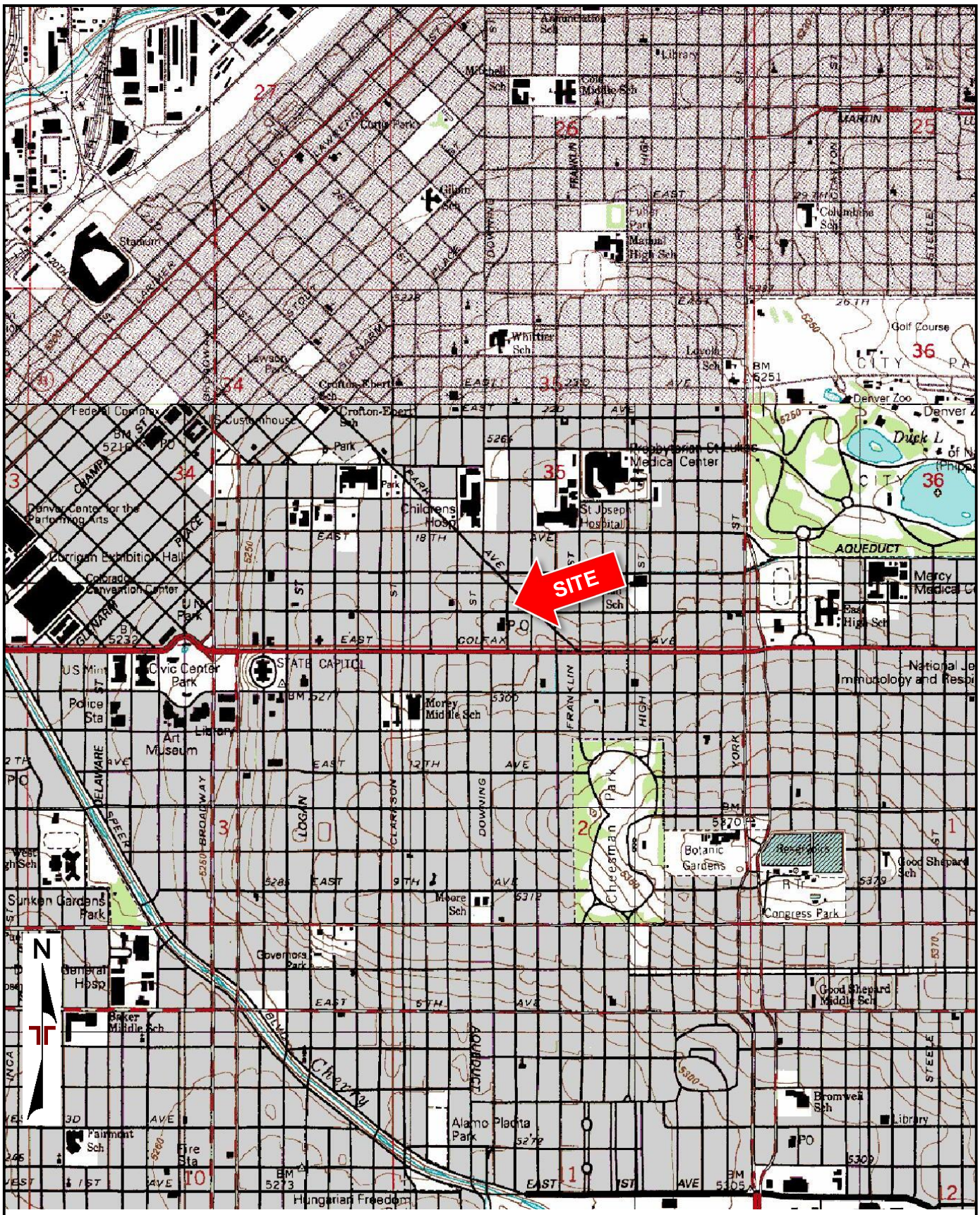
The location of the boring is presented in Exhibit A-3. The boring was located in the field by measuring with a measuring wheel from property lines and/or existing site features. The latitude and longitude coordinates of the boring location was obtained by locating the boring on Google Earth and recording the value. The accuracy of the latitude and longitude values is typically about +/- 25 feet when obtaining the values using this method. The accuracy of the boring location should only be assumed to the level implied by the methods used.

The boring was drilled with a CME-75 truck mounted rotary drill rig with solid-stem augers. During the drilling operations, a lithologic log of the boring was recorded by the field engineer. Relatively undisturbed samples were obtained at selected intervals utilizing a 3-inch outside diameter ring barrel sampler (RS). Bulk samples were obtained from auger cuttings. Penetration resistance values were recorded in a manner similar to the standard penetration test (SPT). This test consists of driving the sampler into the ground with a 140-pound hammer free-falling through a distance of 30 inches. The number of blows required to advance the ring-barrel sampler 12 inches (18-inches for standard split-spoon samplers, final 12-inches are recorded) or the interval indicated, is recorded and can be correlated to the standard penetration resistance value (N-value). The blow count values are indicated on the boring log at the respective sample depths, ring barrel sample blow counts are not considered N-values.

An automatic hammer was used to advance the samplers in the boring performed on this site. A greater efficiency is typically achieved with the automatic hammer compared to the conventional safety hammer operated with a cathead and rope. Published correlations between the SPT values and soil properties are based on the lower efficiency cathead and rope method. This higher efficiency affects the standard penetration resistance blow count value by increasing the penetration per hammer blow over what would be obtained using the cathead and rope method. The effect of the automatic hammer's efficiency has been considered in the interpretation and analysis of the subsurface information for this report.

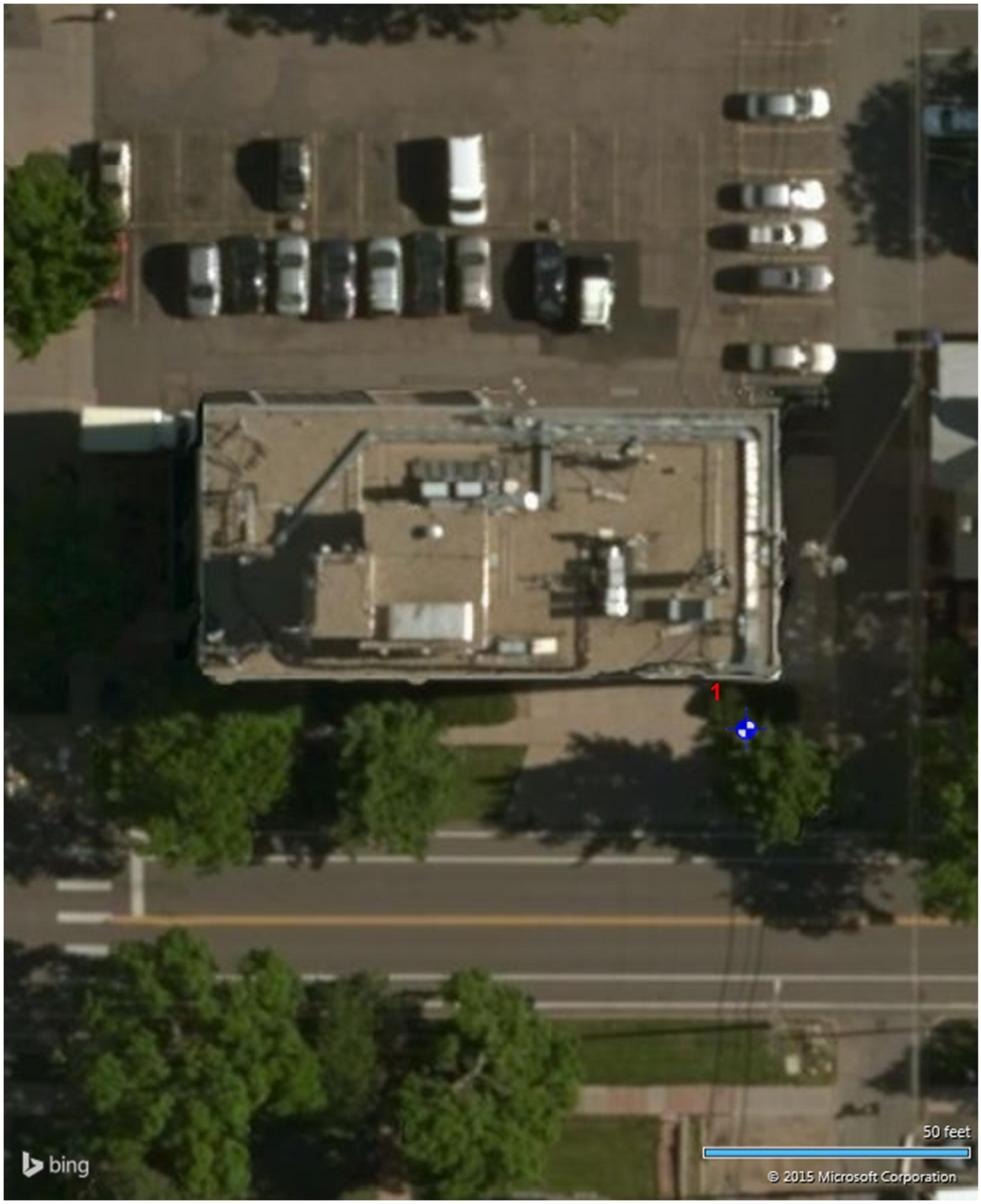
The standard penetration test provides a reasonable indication of the in-place density of sandy type materials, but only provides an indication of the relative stiffness of cohesive materials since the blow count in these soils may be affected by the soils moisture content. In addition, considerable care should be exercised in interpreting the N-values in gravelly soils, particularly where the size of the gravel particle exceeds the inside diameter of the sampler.

Groundwater measurements were obtained in the boring at the time of drilling. Due to safety concerns, the boring was backfilled with auger cuttings after drilling. Some settlement of the backfill may occur and should be repaired as soon as possible.



TOPOGRAPHIC MAP IMAGE COURTESY OF THE U.S. GEOLOGICAL SURVEY
QUADRANGLES INCLUDE: COMMERCE CITY, CO (1/1/1994) and ENGLEWOOD, CO (1/1/1997).

Project Manager: WDR	Project No. 25155132	Terracon	SITE LOCATION	Exhibit
Drawn by: CMW	Scale: 1"=24,000 SF	10625 W I-70 Frontage Rd N Ste 3	PROPOSED MOUNTAIN VIEW TOWER IMPROVEMENTS	A-2
Checked by: WDR	File Name: A-2 & A-3	Wheat Ridge, CO 80033	1600 DOWNING STREET	
Approved by: SBM	Date: 9/21/2015		DENVER, COLORADO	



N

LEGEND 1 Approximate Boring Location

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

AERIAL PHOTOGRAPHY PROVIDED BY MICROSOFT BING MAPS

Project Manager: WDR	Project No. 25155132	Terracon	AERIAL IMAGE EXPLORATION PLAN
Drawn by: CMW	Scale: AS SHOWN	10625 W I-70 Frontage Rd N Ste 3	PROPOSED MOUNTAIN VIEW TOWER IMPROVEMENTS
Checked by: WDR	File Name: A-2 & A-3	Wheat Ridge, CO 80033	1600 DOWNING STREET DENVER, COLORADO
Approved by: SBM	Date: 9/21/2015		Exhibit A-3

BORING LOG NO. 1

PROJECT: Proposed Mountain View Tower Improvements

CLIENT: Urban Land Conservancy

SITE: 1600 Downing Street
Denver, Colorado

LOCATION See Exhibit A-2

Latitude: 39.7418° Longitude: -104.9729°

DEPTH	ELEVATION (Ft.)	DESCRIPTION
0.5		**FILL - LEAN CLAY (CL)**, with organics and sand, dark brown
		SILTY SAND (SP-SM), fine grained, brown, loose, varies to SANDY SILT (ML)
9.0		**CLAYEY SAND (SC)**, medium grained, brown, loose
14.0		**SILTY SAND (SM)**, with gravel, coarse grained, yellow to brown, loose to medium dense
20.5		**CLAYSTONE**, yellow to dark brown, hard
30.0		***Boring Terminated at 30 Feet***

DEPTH (Ft.)	SAMPLE TYPE	RECOVERY (In.)	FIELD TEST RESULTS	Swells (%)	WATER CONTENT (%)	DRY UNIT WEIGHT (pcf)	ATTERBERG LIMITS LL-PL-PI	PERCENT FINES
		12	5-4		2	110		
		12	4-5		4	110	20-17-3	65
10		12	4-5	-0.2 @ 500 psf	8	120		
15		12	7-5					
20		12	10-30	-0.9 @ 500 psf	13	119		
25		9	19-50/3"					
30								

WATER LEVEL OBSERVATIONS: 28

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

Hammer Type: Automatic

Advancement Method:
4-inch diameter continuous flight auger

Abandonment Method:
Boring backfilled with cuttings upon completion.

See Exhibit A-1 for description of field procedures
See Appendix B for description of laboratory procedures and additional data (if any).
See Appendix C for explanation of symbols and abbreviations.

Notes:

WATER LEVEL OBSERVATIONS
None encountered while drilling
Boring caved at 28 feet

Terracon
10625 W I-70 Frontage Road N., Ste. 3
Wheat Ridge, Colorado

Boring Started: 8/28/2015
Boring Completed: 8/28/2015
Drill Rig: CME-75
Driller:
Project No.: 25155132
Exhibit: A-4

APPENDIX B

LABORATORY TESTING

Laboratory Testing

Samples retrieved during the field exploration were returned to the laboratory for observation by the project geotechnical engineer, and were classified in general accordance with the Unified Soil Classification System described in Appendix C.

At this time, an applicable laboratory-testing program was formulated to determine engineering properties of the subsurface materials. Following the completion of the laboratory testing, the field descriptions were confirmed or modified as necessary, and the Boring Log was prepared. The Boring Log is included in Appendix A.

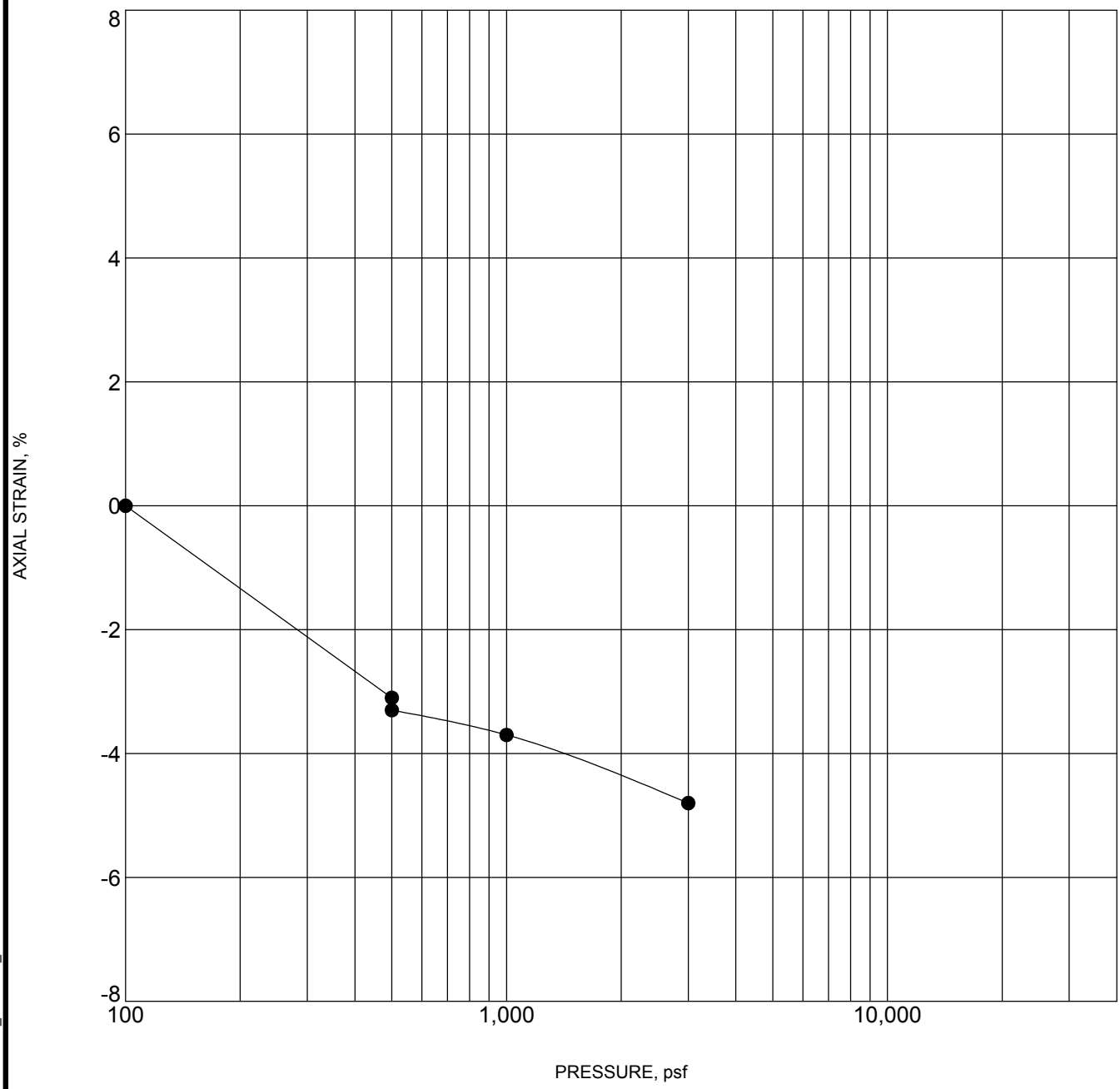
Laboratory test results are included in Appendix B. These results were used for the geotechnical engineering analyses and the development of foundation and earthwork recommendations. All laboratory tests were performed in general accordance with the applicable local or other accepted standards.

Selected soil and bedrock samples were tested for the following engineering properties:

- Water content
- Dry density
- Grain size distribution
- Swell/consolidation
- Atterberg limits
- Water soluble sulfate content

SWELL CONSOLIDATION TEST

ASTM D4546



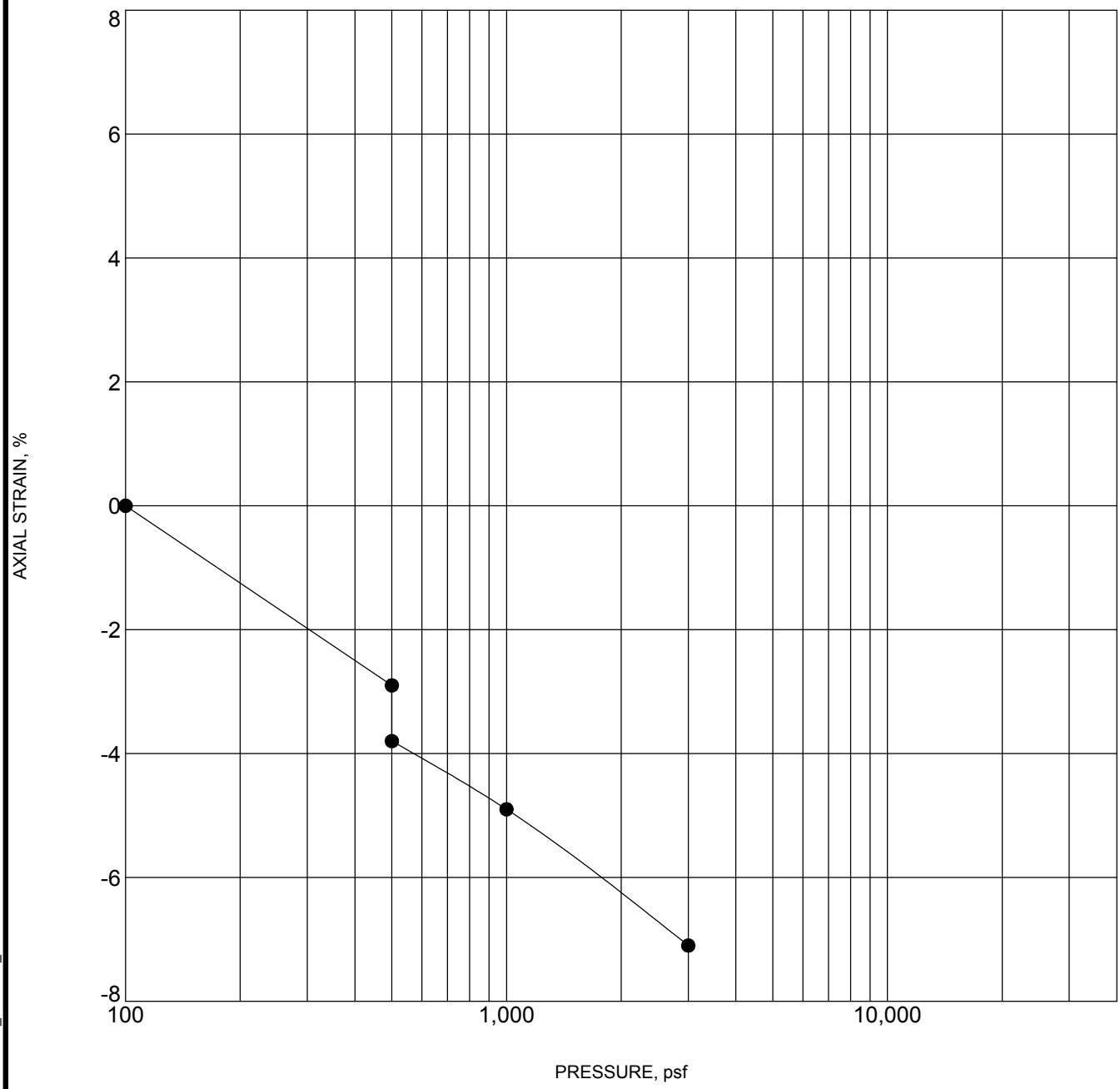
Specimen Identification			Classification	γ_d, pcf	WC, %
●	1	9 - 10 ft	CLAYEY SAND	120	8

NOTES: Water added at 500 psf.

PROJECT: Proposed Mountain View Tower Improvements	Terracon	PROJECT NUMBER: 25155132
SITE: 1600 Downing Street Denver, Colorado	10625 W I-70 Frontage Road N., Ste. 3 Wheat Ridge, Colorado	CLIENT: Urban Land Conservancy
		EXHIBIT: B-2

SWELL CONSOLIDATION TEST

ASTM D4546



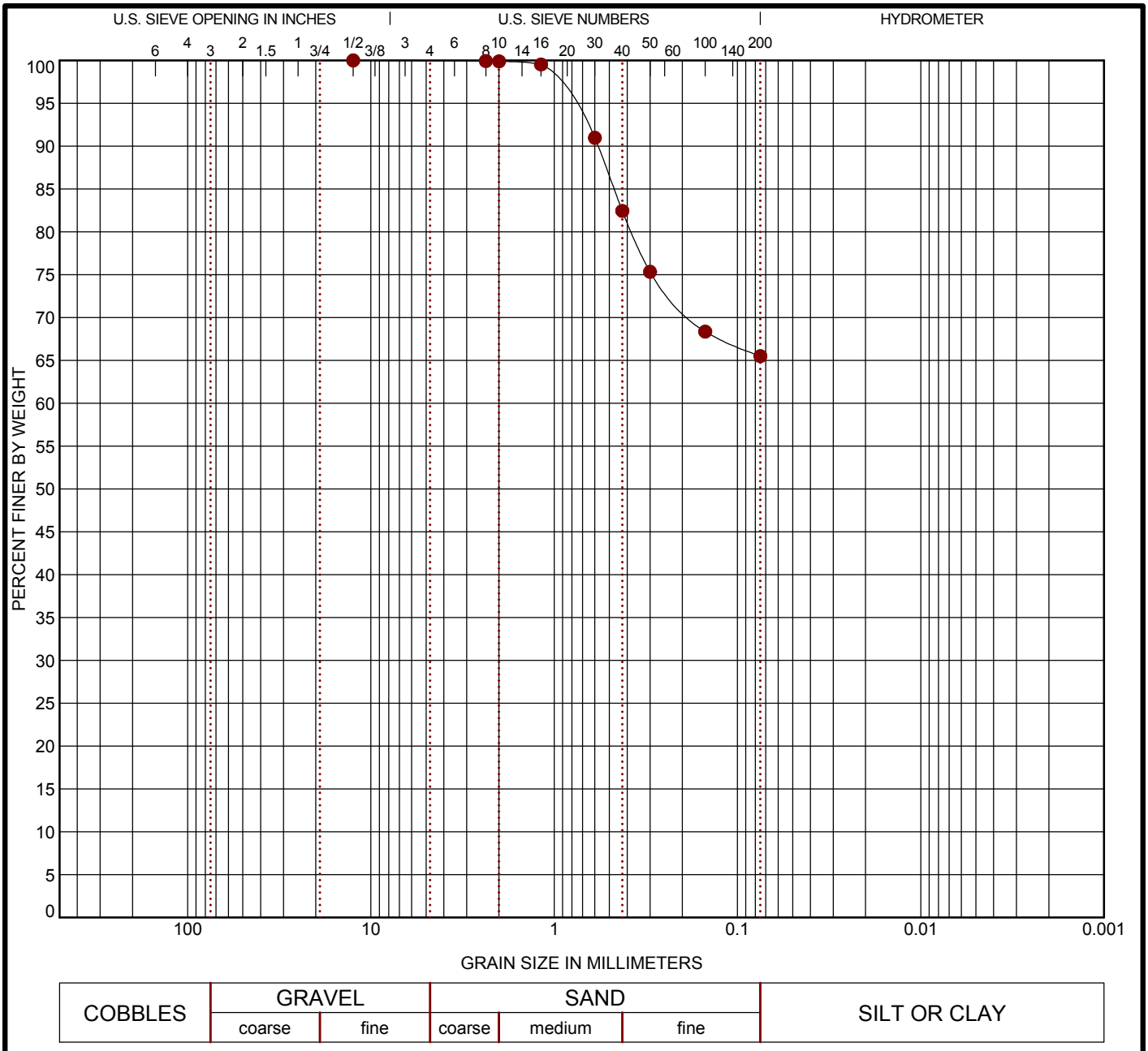
Specimen Identification			Classification	γ_d, pcf	WC, %
●	1	19 - 20 ft	SILTY SAND	119	13

NOTES: Water added at 500 psf.

PROJECT: Proposed Mountain View Tower Improvements	Terracon	PROJECT NUMBER: 25155132
SITE: 1600 Downing Street Denver, Colorado	10625 W I-70 Frontage Road N., Ste. 3 Wheat Ridge, Colorado	CLIENT: Urban Land Conservancy
		EXHIBIT: B-3

GRAIN SIZE DISTRIBUTION

ASTM D422



COBBLES	GRAVEL		SAND			SILT OR CLAY
	coarse	fine	coarse	medium	fine	

Boring ID	Depth	USCS Classification	AASHTO Classification	LL	PL	PI	Cc	Cu
1	4 - 5	SANDY SILT(ML)	A-4(0)	20	17	3		

Boring ID	Depth	D_{100}	D_{60}	D_{30}	D_{10}	%Gravel	%Sand	%Silt	%Clay
1	4 - 5	12.5				0.0	34.5	65.5	

PROJECT: Proposed Mountain View Tower Improvements

SITE: 1600 Downing Street
Denver, Colorado

Terracon
10625 W I-70 Frontage Road N., Ste. 3
Wheat Ridge, Colorado

PROJECT NUMBER: 25155132

CLIENT: Urban Land Conservancy

EXHIBIT: B-4

SUMMARY OF LABORATORY TEST RESULTS

Proposed Mountain View Tower Improvements - Denver, Colorado

Terracon Project No. 25155132

Boring No.	Depth (ft.)	USCS Class.	Initial Dry Density (pcf)	Initial Water Content (%)	Swell/Consolidation		Particle Size Distribution, Percent Passing by Weight					Atterberg Limits		Water Soluble Sulfates (mg/l)	Remarks
					Surcharge (ksf)	Swell (%)	1/2"	#4	#10	#40	#200	LL	PI		
1	2	SP-SM	110	2										1	4
1	4	ML	110	4			100	100	99	82	65	20	3		4
1	9	SC	120	8	0.5	-0.2									3,4
1	19	SM	119	13	0.5	-0.9									3,4

Notes:
Initial Dry Density and Initial Water Content are in-situ values unless otherwise noted.
* = Partially disturbed sample
- = Compression/settlement
NV = no value
NP = non-plastic

Remarks:
1 Remolded Compacted density (about 95% of ASTM D698 maximum density near optimum moisture content)
2 Remolded Compacted density (about 95% of ASTM D1557 maximum density near optimum moisture content)
3 Water added to sample
4 Dry density and/or moisture content determined from one ring of a multi-ring sample
5 Minus #200 Only
6 Moisture-Density Relationship Test Method ASTM D698/AASHTO T99
7 Moisture-Density Relationship Test Method ASTM D1557/AASHTO T180

Exhibit B-5

Terracon

APPENDIX C

SUPPORTING DOCUMENTS

GENERAL NOTES

DESCRIPTION OF SYMBOLS AND ABBREVIATIONS

SAMPLING

Auger	Shelby Tube	Split Spoon
Rock Core	Macro Core	Modified California Ring Sampler
Grab Sample	No Recovery	Modified Dames & Moore Ring Sampler

WATER LEVEL

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

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

FIELD TESTS

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

DESCRIPTIVE SOIL CLASSIFICATION

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

LOCATION AND ELEVATION NOTES

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

STRENGTH TERMS

RELATIVE DENSITY OF COARSE-GRAINED SOILS (More than 50% retained on No. 200 sieve.) Density determined by Standard Penetration Resistance Includes gravels, sands and silts.			CONSISTENCY OF FINE-GRAINED SOILS (50% or more passing the No. 200 sieve.) Consistency determined by laboratory shear strength testing, field visual-manual procedures or standard penetration resistance				BEDROCK		
Descriptive Term (Density)	Standard Penetration or N-Value Blows/Ft.	Ring Sampler Blows/Ft.	Descriptive Term (Consistency)	Unconfined Compressive Strength, Qu, psf	Standard Penetration or N-Value Blows/Ft.	Ring Sampler Blows/Ft.	Ring Sampler Blows/Ft.	Standard Penetration or N-Value Blows/Ft.	Descriptive Term (Consistency)
Very Loose	0 - 3	0 - 6	Very Soft	less than 500	0 - 1	< 3	< 30	< 20	Weathered
Loose	4 - 9	7 - 18	Soft	500 to 1,000	2 - 4	3 - 4	30 - 49	20 - 29	Firm
Medium Dense	10 - 29	19 - 58	Medium-Stiff	1,000 to 2,000	5 - 7	5 - 9	50 - 89	30 - 49	Medium Hard
Dense	30 - 50	59 - 98	Stiff	2,000 to 4,000	8 - 14	10 - 18	90 - 119	50 - 79	Hard
Very Dense	> 50	≥ 99	Very Stiff	4,000 to 8,000	15 - 30	19 - 42	> 119	>79	Very Hard
			Hard	> 8,000	> 30	> 42			

RELATIVE PROPORTIONS OF SAND AND GRAVEL

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

GRAIN SIZE TERMINOLOGY

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

RELATIVE PROPORTIONS OF FINES

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

PLASTICITY DESCRIPTION

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

Terracon

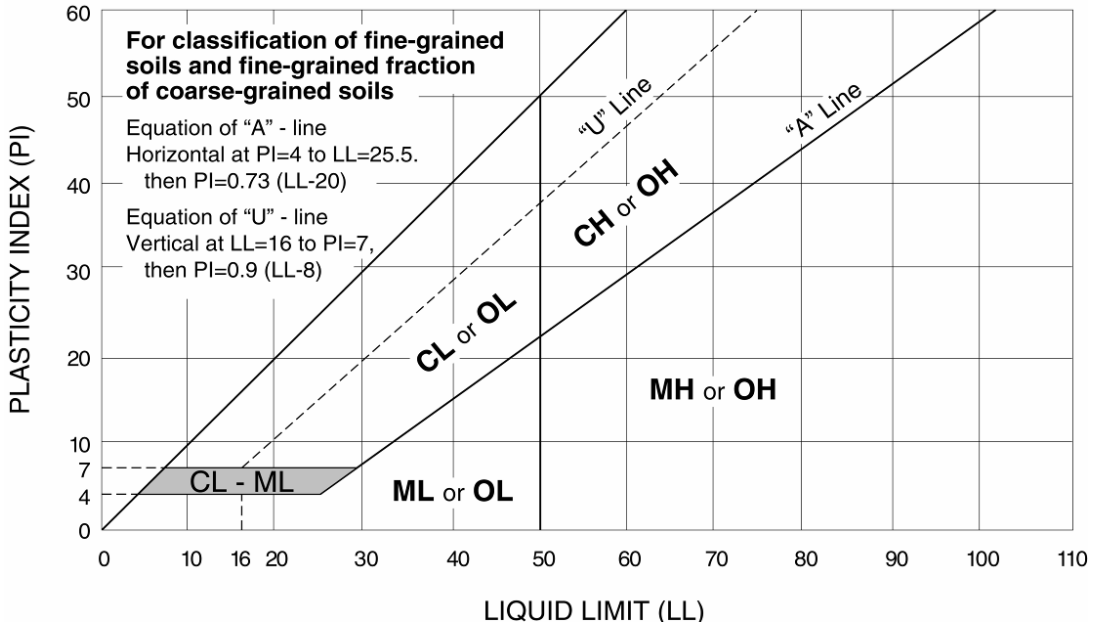
UNIFIED SOIL CLASSIFICATION SYSTEM

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

[A] Based on the material passing the 3-inch (75-mm) sieve
[B] If field sample contained cobbles or boulders, or both, add "with cobbles or boulders, or both" to group name.
[C] Gravels with 5 to 12% fines require dual symbols: GW-GM well-graded gravel with silt, GW-GC well-graded gravel with clay, GP-GM poorly graded gravel with silt, GP-GC poorly graded gravel with clay.
[D] Sands with 5 to 12% fines require dual symbols: SW-SM well-graded sand with silt, SW-SC well-graded sand with clay, SP-SM poorly graded sand with silt, SP-SC poorly graded sand with clay

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

[F] If soil contains ≥ 15% sand, add "with sand" to group name.
[G] If fines classify as CL-ML, use dual symbol GC-GM, or SC-SM.
[H] If fines are organic, add "with organic fines" to group name.
[I] If soil contains ≥ 15% gravel, add "with gravel" to group name.
[J] If Atterberg limits plot in shaded area, soil is a CL-ML, silty clay.
[K] If soil contains 15 to 29% plus No. 200, add "with sand" or "with gravel," whichever is predominant.
[L] If soil contains ≥ 30% plus No. 200 predominantly sand, add "sandy" to group name.
[M] If soil contains ≥ 30% plus No. 200, predominantly gravel, add "gravelly" to group name.
[N] PI ≥ 4 and plots on or above "A" line.
[O] PI < 4 or plots below "A" line.
[P] PI plots on or above "A" line.
[Q] PI plots below "A" line.



Terracon

Exhibit C-2

DESCRIPTION OF ROCK PROPERTIES

WEATHERING

Fresh	Rock fresh, crystals bright, few joints may show slight staining. Rock rings under hammer if crystalline.
Very slight	Rock generally fresh, joints stained, some joints may show thin clay coatings, crystals in broken face show bright. Rock rings under hammer if crystalline.
Slight	Rock generally fresh, joints stained, and discoloration extends into rock up to 1 in. Joints may contain clay. In granitoid rocks some occasional feldspar crystals are dull and discolored. Crystalline rocks ring under hammer.
Moderate	Significant portions of rock show discoloration and weathering effects. In granitoid rocks, most feldspars are dull and discolored; some show clayey. Rock has dull sound under hammer and shows significant loss of strength as compared with fresh rock.
Moderately severe	All rock except quartz discolored or stained. In granitoid rocks, all feldspars dull and discolored and majority show kaolinization. Rock shows severe loss of strength and can be excavated with geologist's pick.
Severe	All rock except quartz discolored or stained. Rock "fabric" clear and evident, but reduced in strength to strong soil. In granitoid rocks, all feldspars kaolinized to some extent. Some fragments of strong rock usually left.
Very severe	All rock except quartz discolored or stained. Rock "fabric" discernible, but mass effectively reduced to "soil" with only fragments of strong rock remaining.
Complete	Rock reduced to "soil". Rock "fabric" not discernible or discernible only in small, scattered locations. Quartz may be present as dikes or stringers.

HARDNESS (for engineering description of rock – not to be confused with Moh's scale for minerals)

Very hard	Cannot be scratched with knife or sharp pick. Breaking of hand specimens requires several hard blows of geologist's pick.
Hard	Can be scratched with knife or pick only with difficulty. Hard blow of hammer required to detach hand specimen.
Moderately hard	Can be scratched with knife or pick. Gouges or grooves to ¼ in. deep can be excavated by hard blow of point of a geologist's pick. Hand specimens can be detached by moderate blow.
Medium	Can be grooved or gouged 1/16 in. deep by firm pressure on knife or pick point. Can be excavated in small chips to pieces about 1-in. maximum size by hard blows of the point of a geologist's pick.
Soft	Can be gouged or grooved readily with knife or pick point. Can be excavated in chips to pieces several inches in size by moderate blows of a pick point. Small thin pieces can be broken by finger pressure.
Very soft	Can be carved with knife. Can be excavated readily with point of pick. Pieces 1-in. or more in thickness can be broken with finger pressure. Can be scratched readily by fingernail.

Joint, Bedding, and Foliation Spacing in Rock [a]

Spacing	Joints	Bedding/Foliation
Less than 2 in.	Very close	Very thin
2 in. – 1 ft.	Close	Thin
1 ft. – 3 ft.	Moderately close	Medium
3 ft. – 10 ft.	Wide	Thick
More than 10 ft.	Very wide	Very thick

a. Spacing refers to the distance normal to the planes, of the described feature, which are parallel to each other or nearly so.

Rock Quality Designator (RQD) a

RQD, as a percentage	Diagnostic description
Exceeding 90	Excellent
90 – 75	Good
75 – 50	Fair
50 – 25	Poor
Less than 25	Very poor

a. RQD (given as a percentage) = length of core in pieces 4 in. and longer/length of run.

Joint Openness Descriptors

Openness	Descriptor
No Visible Separation	Tight
Less than 1/32 in.	Slightly Open
1/32 to 1/8 in.	Moderately Open
1/8 to 3/8 in.	Open
3/8 in. to 0.1 ft.	Moderately Wide
Greater than 0.1 ft.	Wide

References: American Society of Civil Engineers. Manuals and Reports on Engineering Practice - No. 56. Subsurface Investigation for Design and Construction of Foundations of Buildings. New York: American Society of Civil Engineers, 1976. U.S. Department of the Interior, Bureau of Reclamation, Engineering Geology Field Manual.