APPENDIX B

GEOTECHNICAL REPORT

Limited Feasibility Level Geotechnical Investigation for a Proposed Multi-Level Apartment Complex, 1300 Bristol Street N, Newport Beach, CA 92660.

PN 21016-00

April 19, 2021

KCG KLING
Consulting
Group, Inc.

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April 19, 2021 **PN 21016-00**

Mr. Dan Matula
The Picerne Group
5000 Birch Street,
Newport Beach, CA 92660

Subject: **Limited Feasibility Level Geotechnical Investigation for a Proposed Multi-Level Apartment Complex, 1300 Bristol Street N, Newport Beach, CA 92660**

Dear Mr Matula:

At your request and authorization, Kling Consulting Group, Inc. (KCG) has performed a limited feasibility level geotechnical investigation for a proposed multi-level apartment complex at 1300 Bristol Street N, Newport Beach, California (**see Figure 1 - Site Location Map**). Our findings from subsurface exploration, laboratory testing, and geotechnical analyses are presented herein. Preliminary conclusions and recommendations are provided regarding the existing geotechnical conditions and the design of the proposed development.

We appreciate this opportunity to be of continued service and to work with you on this project. Should you have any questions regarding this report, please do not hesitate to call.

Respectfully,

KLING CONSULTING GROUP

Sean M. Webb
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Notes:

KCG KLING Consulting Group, Inc.

Client:
The Picerne Group

Address:
1300 Bristol Street North,
Newport Beach

Site Location Map

Drawn: S.W
Date: 04/19/2021
P/N: 21016-00
Figure: 1

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1.0 INTRODUCTION

1.1 PURPOSE AND SCOPE

The purpose of our limited geotechnical investigation was to evaluate near-surface soil conditions to provide preliminary feasibility level geotechnical design recommendations for a proposed multi-level apartment building complex. The scope of work undertaken included the following tasks:

- Compilation and interpretation of available, previously documented geologic and geotechnical data for the property;

- Coordination with Underground Service Alert to mark and identify buried utilities;

- Subsurface exploration, including the sampling and logging of two (2) hollow-stem auger borings to depths of 50 feet. Continuous logs of the subsurface conditions, as encountered in the borings, were recorded and are presented in Appendix B. The locations of the borings are shown in **Figure 2 - Exploration Location Map**;

- Laboratory testing and analysis of bulk and drive samples obtained in the field, including in-place moisture/densities, direct shear, consolidation, grain size distribution, expansion index, and soluble sulfates, to determine pertinent engineering properties. Laboratory descriptions and results are presented in Appendix C;

- Geotechnical engineering analysis and evaluation of collected data; and

- Preparation of this report along with accompanying maps and illustrations. This report presents our findings, conclusions, and feasibility level recommendations.

1.2 SITE AND PROJECT DESCRIPTION

The subject site is located along the north side of Bristol Street, north of the San Joaquin Hills Transportation Corridor, Highway 73, in Newport Beach, California. The subject site is identified as APN 427-342-01, and is addressed as 1300 Bristol Street N. The square-shaped subject site encompasses approximately 1.97-acres, and is currently occupied by an existing commercial office building along with paved drive and parking areas. The site is bordered by Bristol Street North to the south, Spruce Avenue to the west and existing commercial/retail buildings and paved parking to the north and east. The approximate location of the site is illustrated in **Figure 1 - Site Location Map**.

Through discussions with the client and a review of conceptual yield study plans provided by TCA Architects, dated March 23, 2021, it is understood the proposed development preliminarily consists of a multi-level podium III style building entailing multiple levels of studio, one-bedroom and two-bedroom apartments with both ground floor parking and two levels of subterranean parking. Access will be provided via a driveway that extends along the northern and eastern property boundaries.

Specific grading plans are not available; however, grading is anticipated to include cut excavations of at least 24-feet below existing grades to achieve the proposed grades for subterranean parking.

2.0 GEOLOGIC CONDITIONS

2.1 Subsurface Investigation and Sampling

On April 5, 2021, two borings were advanced using a hollow-stem auger. The borings were excavated to a depth of 50 feet below the existing ground surface in the vicinity of the proposed development area. The borings were observed and logged by a geologist from KCG. Logs of the exploratory borings are included in Appendix B. The approximate location of the boreholes is illustrated in **Figure 2 - Exploration Location Map**.

During our field exploration, selected ring and bulk samples were obtained for laboratory testing. Bulk composite samples were collected from drilling excavation spoils. All samples collected were returned to our laboratory for evaluation and testing. Laboratory tests included in-situ moisture and density, direct shear, Atterberg limits, sieve and hydrometer, consolidation test, soluble sulfate and expansion index. Test descriptions and results are presented in the summary of laboratory testing within Appendix C.

2.2 Regional and Site Specific Geologic Setting

The subject site is located in the Peninsular Ranges Geomorphic Province, at the south-eastern edge of the Los Angeles Basin and within the nearly flat-lying area of the Tustin Plain. The site is primarily underlain by elevated Pleistocene and late Pliocene marine terrace deposits established by progressive and (or) episodic tectonic uplift of coastal southern California.

The National Geologic Map Database maps the site as being underlain by late to middle Pleistocene Old Paralic Deposits. The Old Paralic Deposits comprise a poorly sorted, moderately permeable, reddish-brown, interfingered strandline, beach, estuarine, and colluvial deposits composed of silt, sand, and cobbles. These deposits rest on now emergent wave-cut abrasion platforms preserved by regional uplift.

Notes:

KCG KLING Consulting Group, Inc.

Client:
Picerne Group

Address:
1300 Bristol Street North,
Newport Beach

Geotechnical Map

Drawn: S.W
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Figure: 2

2.3 Subsurface Conditions

2.3.1 Asphalt

The site is mantled by a relatively thin veneer of asphalt to a depth of approximately 4-inches from the existing ground in the vicinity of all of the boreholes.

2.3.2 Old Paralic Deposits (Qopf$_a$)

The site is underlain by clay, clayey-silt and sand associated with the Old Paralic Deposits to an observed depth of up to 50 feet below the current ground level in the vicinity of the boreholes. The Old Paralic Deposits encountered during our subsurface exploration were generally light brown to dark gray, sandy clays and sandy to clayey silts, that were medium stiff to hard and moist to wet, with silty sands that were medium dense to very dense. Exploratory boring logs are presented in Appendix B.

2.4 Groundwater

A perched groundwater condition was encountered in boreholes KB-1 and KB-2 at depths of 26 feet and 35 feet, respectively. These perched zones are considered typical in this area where water accumulates in relatively permeable sand layers that are situated above or below relatively impermeable clay layers that act as confining layers and may not, in our opinion, necessarily represent the static groundwater level which may be closer to 50 feet below existing grades. However, these perched zones of groundwater will potentially impact the site construction and should be considered and incorporated into project design. We anticipate the perched groundwater may significantly impact two subterranean parking levels founded at a depth of 24 feet below existing grades. It should be noted that groundwater variation may result from fluctuations in the ground surface topography, subsurface stratification, rainfall, irrigation and other factors that may not be evident at the time of our subsurface exploration. Until the regional ground water table can be clearly established, the perched water discussed above should be considered in construction planning and final design.

The nearest groundwater observation well, monitored by the California Department of Water Resources, is located along Baker Street E, Costa Mesa, at a ground surface elevation of 42.290 feet above sea level. The highest recorded groundwater was recorded at 31.8 feet below the ground surface in October 1969 (Reference 4). The subject site is approximately 1.7 miles from this observation well. According to the California Geologic Survey (CGS), Seismic Hazard Zone Report for the Tustin 7.5-Minute and Newport Beach 7.5 minute Quadrangle (References 6 and 7), the reported and mapped historical high groundwater level is approximately 10 feet to 30 feet below the current ground level in the vicinity of the site.

3.0 GEOTECHNICAL ENGINEERING CONSIDERATIONS

3.1 Expansive Soil Characteristics

Expansion Index (EI) laboratory testing on representative soil samples from KB-1 and KB-2 resulted in indices of 62 and 61, respectively, which is considered "medium" expansion potential (EI 51-90) according to the CBC.

3.2 Sulfate Content

Sulfate testing was performed on a representative sample of the soil. The soils tested during this investigation indicated a class "S1" sulfate per ACI-318 (Reference 2), with a soluble sulfate content of 270 ppm or 0.027%.

3.3 Moisture and Density

Samples were retrieved at various depths below the ground surface from each boring location and used to determine in-place dry density and moisture content. Moisture results indicate the sampled soils have a moisture content of ranging from 6.2 to 23.3 percent and a dry density ranging from 102.9 to 117.2 pcf.

3.4 Faulting and Seismicity

The subject site is not located within a State of California Earthquake Fault Zone (formerly known as Alquist-Priolo Zones, Jennings and Bryant, 2010; Hart and Bryant, 1997). The property is not located where a site-specific investigation to determine the locations of any active faults would be required. However, the Southern California region is seismically active. Active and potentially active faults within Southern California are capable of producing seismic shaking at the site. It is anticipated that the site will periodically experience ground acceleration due to exposure to moderate to large magnitude earthquakes occurring on distant faults.

The distances of the closest major active faults from the property were generated from information provided on the USGS online resource (USGS, 2008, National Seismic Hazards Maps, Source Parameters,), with the approximate center of the site being at latitude 33.6589°N and longitude 117.86855°W. The San Joaquin Hills Fault is located approximately 2.2 miles from the site, the Newport Inglewood Fault Zone approximately 4.9 miles from the site and the Newport Inglewood (Offshore segment) located approximately 5.4 miles from the site. However, no active faults are known to exist at the site, and the risk of surface fault rupture is considered to be low.

3.5 Seismic Design Parameters

Presented below are the site seismic parameters utilizing generic geologic, seismic, and geotechnical data gathered for the site using the SEAC/OSHPD web based tool (Reference 10). All structures should be designed for earthquake-induced strong ground motions in accordance with the 2019 CBC procedures utilizing the following parameters:

Seismic Design Parameters

Site Class (Soil Profile)	**D**
Latitude	**33.6589334**
Longitude	**-117.8685508**
Short Period Spectral Acceleration, Ss:	**1.298**
1-Second Period Spectral Acceleration, S1:	**0.463**
Site Coefficient, Fa:	**1.0**
Site Coefficient, Fv:	**1.8**
Maximum Considered Earthquake Spectral Response Acceleration, SMS:	**1.298**
Maximum Considered Earthquake Spectral Response Acceleration, SM1:	**0.8334**
Design Spectral Response Acceleration, SDS:	**0.865**
Design Spectral Response Acceleration, SD1:	**0.555**
Site modified peak ground acceleration PGA_M	**0.612**
Seismic Design Category	**D**

Note: A site specific ground motion analysis was not included in the scope of this investigation. Per ASCE 7-16, 11.4.8, structures on Site Class D with S_1 greater than or equal to 0.2 may require Site Specific Ground Motion Analysis. However, a site specific ground motion analysis may not be required based on exceptions listed in ASCE 7-16, 11.4.8. The project structural engineer should verify whether exceptions are valid for this site and if a Site Specific Ground Motion Analysis is required.

3.6 Seismic Hazards

3.6.1 Liquefaction Potential

Liquefaction occurs when ground water pressure in loose sandy soil becomes greater than overburden pressure due to seismic-induced cyclic shear stresses from earth quakes. The result is a near complete loss of soil shear strength and ground settlement. The California Geological Survey (CGS), Seismic Hazard Zone Report for the Tustin Quadrangle (Reference 5) reports the site is not situated in a liquefaction zone. Our review of the Seismic Hazard Zone Reports for the Tustin and Newport Beach Quadrangles (References 6 & 7), indicate the historic groundwater is reported to be approximately 10 feet to 30 feet from existing grades in the vicinity of the property. Our liquefaction analysis conservatively incorporates the historic high groundwater depth of 10 feet. Our geotechnical evaluation indicated that localized and isolated sandy layers within the Old Paralic Deposits that underlie the site are susceptible to relatively minor amounts of liquefaction due to a design-level earthquake along a nearby fault and incorporating the historical high groundwater level of 10 feet below existing grades. Overall seismic induced liquefaction settlement would be reduced with the removal of materials for the subterranean excavations. The portions of the site that appear to be susceptible to liquefaction and the magnitudes of seismic-induced settlement described above appear to be somewhat localized. The state of California has not established a seismic hazard zone for the area.

3.6.2 Liquefaction Settlement Analysis

The total earthquake-induced liquefaction settlement potential was calculated using CivilTech Software's *Liquefy Pro* computer program. Our evaluation was based on the site class and adjusted peak ground acceleration of 0.612g, as presented in the Seismic Design Parameters Table above, and a probabilistic 2,475-year modal magnitude of 6.89. Our analysis indicated the estimated settlement due to earthquake-induced liquefaction settlement is approximately 0.36 inches to 0.54 inches. These settlement values are considered preliminary, and further geotechnical investigations would be required to provide refinement of the estimated differential settlement of the site. The results of our analysis are included herein in **Appendix D - Seismic Settlement Analysis.**

3.6.3 Lateral Spreading

Lateral spreading, a phenomenon associated with seismically induced soil liquefaction, is the lateral displacement of soils due to inertial motion and lack of lateral support during or post liquefaction. Lateral spreading generally occurs on gently sloping ground or level ground with nearby free surface faces such as a drainage or stream channel. The San Diego Creek is located approximately 2,180 ft to the southeast of the proposed development and consists of an approximately 10ft high free face. According to studies undertaken by Zhang et al. (2004), Cubrinovski (2012), lateral displacements occur between 300 and 1000 feet from a "free face". As such, lateral spreading would likely be nil within the project site.

4.0 CONCLUSIONS

The following conclusions are preliminary and based upon our analysis and data review obtained during our limited subsurface field investigation and laboratory testing. It is our opinion that the proposed development concept is considered geotechnically feasible provided the recommendations presented herein are implemented during design and construction.

- Based upon our review of the site and the proposed development plans, the underlying soils on-site are considered to have sufficient bearing capacity to support the proposed development, provided the recommendations herein are implemented.

- Based on our laboratory testing, soils underlying the site have a moderate expansion potential. However, the underlying soils are variable and anticipated to possess a very low to high expansion potential.

- Our geotechnical evaluation indicates that sandy layers within the Paralic Deposits that underlie the site are susceptible to liquefaction due to a design-level earthquake incorporating a historical high groundwater level of 10 feet below existing grades (CGS/CDMG, 1998). The estimated settlements are in the range of 0.37 inches to 0.58 inches and appear to be limited to isolated and localized relatively thin zones between 25 and 40 feet below existing grades at the site during seismic events. Overall seismic induced liquefaction settlement would be reduced with the removal of the upper materials for the subterranean excavations, as summarized in Section 2.6. The liquefaction assessment is considered preliminary, and further study is required to refine the estimates and determine likely differential settlement. Lateral spreading is deemed unlikely due to the lack of "free face" in the vicinity of the subject site.

- No active fault is known to exist at the site, and the risk of surface fault rupture is considered to be low. However, the project site lies within a region of historical seismicity and will likely be subject to seismic shaking in the future.

- KCG's professional opinion is that liquefaction-induced ground displacements are relatively minor overall and can be mitigated through the use of a reinforced concrete structural mat foundation system for the support of the proposed apartment building and parking structure.

- Soils underlying the subject site are not considered to be susceptible to hydrocollapse;

- A perched groundwater condition was encountered in our exploratory borings at depths of 26 and 35 feet below the existing ground surface. Although these perched zones are considered typical in the site vicinity where water accumulates in relatively permeable sand layers situated above and/or below confining layers, it may not, in our opinion, represent the static groundwater level which is more likely close to 50 feet below existing grades. However, this perched zone of groundwater will potentially impact and pose a problem for the site construction for two subterranean levels at or near a depth of 24 feet below existing grades. Temporary dewatering or other measures should be considered and groundwater levels incorporated into the project design for the proposed development;

- Based on laboratory soil test results, the on-site soils indicated a soluble sulfate content considered "Class S1", moderate severity per the 2014 ACI Concrete Manual of Practice.

- The proposed development should not adversely affect neighboring properties.

5.0 PRELIMINARY RECOMMENDATIONS

Preliminary recommendations presented below are based on plans obtained from the client and the limited geotechnical information gathered and analyzed to date. To provide uniform soil support for the proposed structures, we recommend that the upper soils be removed and re-compacted in those areas to receive buildings or other settlement - sensitive improvements, where not removed by planned excavation. The depth of removals will be dependent upon the type of foundation system selected.

5.1 Supplemental Subsurface Exploration

During this limited investigation, the subsurface exploration was limited to two boring locations in readily accessible areas. We recommend that a supplemental geotechnical investigation be performed that includes additional borings (including installation of piezometers) and CPT soundings (with ground water measuring capabilities). The supplemental investigation should also include additional laboratory testing, foundation and settlement analysis; ground water measurements and to verify subsurface conditions. Recommendations would be updated as warranted.

5.2 Earthwork Specifications

All grading should be performed per the General Earthwork and Grading Specifications presented in Appendix F unless specifically revised or amended below. Grading should also conform to all applicable governing agency requirements. Prior to the commencement of grading operations, all vegetation, organic topsoil and human-made structure should be cleared and disposed of off-site. Any undocumented fill or back-fill encountered should be removed and re-compacted. All areas receiving fill should be scarified to 6 inches and/or over-excavated, moisture conditioned to between optimum moisture and two to four percent above optimum moisture content, and re-compacted to a minimum of 90 percent relative compaction as determined by ASTM D1557. Soil material excavated from the site should be adequate for re-use as compacted fill provided it is free of trash, vegetation and other deleterious material. All earthwork and grading operation should be performed under the observation and testing of the geotechnical consultant of record.

5.3 Remedial Earthwork

5.3.1 Foundation Design Option: Conventional Foundations –Two Levels Subterranean

We recommend the subterranean level pad area be over excavated a minimum of five feet below the subterranean level finish grade elevation, or a minimum of two feet below proposed foundations, whichever is deeper. This would account for soil variability, expansion potential, reduce the potential for settlement and differential settlement and maintain a uniform fill blanket beneath the bottom of the foundations. The over-excavation should be extended a minimum of five feet laterally beyond the proposed building footprint and/or foundations or equal to the depth of the over-excavation, which-ever is deeper, where practical. Footings should be underlain by a minimum of two-feet of engineered fill below the bottom of footings.

5.3.2 Foundation Design Option: Mat Slab Foundations –Two Levels Subterranean

For Mat slab foundation systems, the removals can be reduced to reprocessing (i.e. 12-inch scarification and recompaction) and proof rolling of the subgrade soils exposed at the subterranean level.

5.3.3 Proposed Pavement and Flatwork Areas

In areas outside of proposed structural areas that would support pavement and flatwork, the exposed sub-grade soils should be processed and re-compacted to a depth of 12-inches. If soils are disturbed during the removal of existing improvements, the disturbed soil should be removed and replaced with compacted fill. After removals are made, exposed soils should be scarified to a depth of 6-inches, brought to near optimum moisture content, and re-compacted.

5.4 Processing of Natural Soils and Fill Placement

Processing of in-place soils exposed after clearing, grubbing, and removal of unsuitable material and before placing fill should include the following items of work:

Scarification of the materials exposed after remedial removals should be accomplished to a depth of at least 6 inches or as dictated by actual soil conditions encountered;

The scarified soils should be brought to 2 to 4 percent above optimum moisture content by watering or drying, as required;

Compaction of the processed soils to at least 90 percent of the laboratory maximum dry density before placing fill.

Fill should be placed in relatively thin (6 to 8-inch) uniform lifts; moisture conditioned to 2 to 4 percent above optimum moisture content and compacted to at least 90 percent relative compaction based on ASTM D 1557. Actual lift thickness would depend on soil type and compaction equipment being used.

5.5 Proposed Building Foundations

All foundation criteria are considered minimum requirements that may be superseded by more stringent requirements from the architect, structural engineer, or governing agencies; recommended geotechnical design parameters are being provided for conventional spread footing and reinforced mat slab foundation systems for the residential building; and drilled auger-cast pile for the parking structure at grade, and mat slab for the proposed parking structure with subterranean levels.

5.5.1 Subterranean-Conventional Shallow Foundations

The following geotechnical design parameters are provided to design proposed conventional foundations for the proposed multi-level apartment building, with two levels of subterranean parking. The proposed foundations for the proposed building may be supported by square pad footings utilizing a maximum allowable bearing pressure of 3000 pounds per square foot. The maximum width of the continuous footings should be no more than 8-feet with a minimum depth of 3-feet below the lowest adjacent grade (which included the top of the slab on grade). A coefficient of friction of 0.40 may be used, along with a passive lateral resistance of 250 pounds per square foot per foot of embedment. Sub-grade soil would likely require recompaction or processing for up to three feet.

If normal code requirements are used for seismic design, the allowable bearing value and coefficient of friction may be increased by 1/3 for short duration loads, such as the effect of wind or seismic forces.

If any utility lines are within a 1:1 (horizontal: vertical) projection from the bottom of a footing, they may be within the influence zone of the proposed footing load; if this condition exists, the proposed footing should be deepened so that the utility is outside the zone of influence; the utility line could also be relocated or encased with concrete with concrete slurry. These conditions should be evaluated on a case by case basis.

5.5.2 Subterranean- Mat Slab

A rigid mat foundation may be used to support the buildings at the site, provided the mat foundation is bearing within soils that are properly compacted, and proof rolled in accordance with the recommendations contained herein. When properly designed and constructed, a structural mat foundation system can be expected to support high structural loads and provide relatively uniform settlement across a structure while being able to "bridge" over local areas of dynamic and anticipated static settlement. Mat foundations should be properly reinforced to form a relatively rigid structural unit in accordance with the structural engineer's design. For designing a mat foundation, we recommend using a modulus of subgrade reaction of 65 pounds per square inch per inch (pci). This value can be further refined as part of the supplemental investigation.

5.6 Settlement

Static settlement of proposed foundations is not expected to exceed one inch for total and one half inch differential over 50 horizontal feet, provided the minimum remedial earthwork recommendations provided in Section 5.2 is performed for the specific foundation system type. For preliminary design purposes, seismic induced liquefaction settlement for the apartment site is 0.37 to 0.58 inches. Overall seismic induced liquefaction settlement would be reduced with the removal of static settlements can be additive. It may be prudent to assume a lesser horizontal distance should adjacent footings be substantially different in size. Further study is required to refine and calculate likely differential settlement beneath the proposed development.

5.7 Slab-On-Grade

These recommendations are considered minimum requirements for residential applications that may be superseded by more stringent requirements from the architect, structural engineer, or governing agencies.

Concrete slabs should be at least 5-inches in thickness. Actual slab thickness and reinforcement should be determined by the structural engineer based on structural loads and soil interaction. Our recommendations should be superseded by the recommendations of the structural engineer or architect.

Subgrade soils should be placed wet of the optimum moisture content, and moisture should be maintained until placement of the concrete slab. Additional testing should be performed after precise grading to verify our recommendations.

The slab should be underlain by a minimum two-inch layer of sand, with a sand equivalent of 30 or greater. The sand layer should be underlain by a 15-mil Stego Wrap vapor retarder or equivalent product with a permeance rate of 0.012 perms and a puncture resistance of Class “A” or “B” per ASTM E 1745-97. As per the manufacturer’s recommendations, all seams should overlap a minimum of 6 inches and should be sealed in accordance with the specifications provided by the vapor retarder manufacturer. All penetrations should be sealed using a combination of Stego Wrap, Stego Tape and/or Stego Mastic or approved equivalent product. The vapor retarder should be lapped downward a minimum of 12 inches where the vapor retarder encounters an interior footing or exterior thickened edge or footing. The vapor retarder should be placed on top of the sand layer if the sand is expected to become wet before pouring concrete. If the sand can be kept dry before pouring concrete, the vapor retarder should be placed under the sand layer. The water-cement ratio should be a minimum of 0.45 for all concrete within the structure that will contact the on-site soil.

If moisture sensitive floor coverings are utilized, interior concrete slabs should be designed and constructed in accordance with the applicable floor covering manufacturer’s specifications.

Slab subgrade soil should be pre-saturated to at least optimum moisture content to a depth of at least 12 inches below the sand layer.

5.7.1 Basement Slab on Grade Floors

Parking garage basement slab in grade floors, other than a mat slab, should be a minimum of 5-inches in thickness and reinforced to resist shrinkage and temperature warping cracking. Actual slab thickness and steel reinforcement should be determined by the structural engineer based on environmental factors and concrete shrinkage considerations. An aggregate base layer may be required depending on the subgrade soils exposed during construction or determined from the supplemental investigation.

5.8 Permanent Subterranean Walls

We anticipate that where temporary shoring is installed, the permanent restrained retaining walls for the subterranean level will predominantly be placed directly against the temporary shoring. The design parameters provided below assume that granular non-expansive soils (Expansion Index <20 and SE≥30) are used to back-fill any retaining walls. Permanent subterranean walls should be designed to resist the pressure exerted by retained soils plus any additional lateral forces due to loads placed adjacent to or near the wall. Retaining walls that are free-draining, are situated above groundwater and are to be restrained from movement at the top, such as basement walls, should be designed for an equivalent fluid weight of 60 pcf for at-rest conditions (for a level surface of retained earth). If traffic loads are planned adjacent to the walls, the walls should be designed for an additional uniform horizontal pressure of 75 and 150 psf for passenger car and truck traffic, respectively. For other surcharge loads, we recommend the walls be designed to resist a uniform horizontal pressure equal to 30 percent of the uniform surcharge load.

If back-fill conditions (including the slope of the retained ground surface) differ from those assumed herein, Kling Consulting Group should be consulted to provide additional evaluation and/or recommendations as warranted. All retaining structures should be fully free draining. Building walls below grade should be waterproofed or damp-proofed, depending on the degree of moisture protection desired. The foundation system for the retaining walls should be designed in accordance with the recommendations presented in the preceding sections of this report, as appropriate. Footings should be embedded at a minimum of 18-inches below adjacent grade (excluding the 6-inch landscape layer).

For resistance to lateral loads, an allowable coefficient of friction of 0.35 between the base of the foundation elements and underlying material is recommended. In addition, an allowable passive resistance equal to an equivalent fluid weighing 300 pcf acting against the foundation may be used to resist lateral forces. Passive pressure in the upper 1.0-foot should be neglected unless confined by concrete slabs-on-grade or asphaltic pavement. These values may be increased by one-third for transient wind or seismic loads. A seismic surcharge of 19 H should be applied as an equivalent fluid pressure with the resultant acting at 1/3-height above the base of the wall, where H= the retained height of the wall greater than 6 feet.

The permanent subterranean wall should be provided with an adequate back drain system to reduce the potential for build-up of hydrostatic pressures.

Adequate drainage should be provided behind all retaining walls. The drainage system should consist of a minimum of four-inch diameter perforated PVC pipe (schedule 40 or approved equivalent) placed at the base of the retaining wall and surrounded by ¾-inch clean crushed rock wrapped in a Mirafi 140N filter fabric, or equivalent approved by the Geotechnical Engineer. The drain rock wrapped in fabric should be at least 12-inches wide and extend from the base of the wall to within two feet of the ground surface. The upper two feet of back-fill should consist of compacted native soil. The retaining wall drainage system should be sloped to outfall to the storm drain system or other appropriate facility.

For those portions of the wall not placed against shoring, the above values assume granular back-fill and free-draining conditions to prevent buildup of hydrostatic pressure in the back-fill. Back-fill materials should meet the recommendations described in the following section of this report. Import fill materials should be approved by the soils engineer prior to placement. Wall back-fill should be compacted by mechanical methods to at least 90 percent of the maximum dry density as determined by ASTM D 1557.

5.9 Temporary Excavations

We anticipate the on-site soils can be excavated using conventional heavy duty earthmoving equipment in good condition. Shoring systems, if used, may yield during excavation causing adjacent facilities and improvements to settle slightly. The magnitude of shoring movements and the resulting settlements are difficult to estimate because they depend on many factors, including the method of installation and the contractor's skill with installing the shoring system. Lateral deflections for a properly designed and constructed shoring system would likely be within ordinarily accepted limits of approximately 1-inch. A monitoring program should be established to evaluate the effects of shoring construction on other facilities.

Provided the excavations are above groundwater, temporary excavations and trench walls to a depth of four feet may be made vertically without shoring, subject to verification of safety by the contractor. Deeper excavations should be no steeper than 1.5:1 (horizontal to vertical) or braced or shored in accordance with CAL OSHA standards and guidelines. The contractor is assumed responsible for maintaining safety at the jobsite. All excavation work should be in compliance with current CAL OSHA standards. Under no circumstances should excavations be made deeper than four feet or below groundwater without shoring, bracing or laying-back, in accordance with CAL OSHA standards and guidelines. No surcharge loads should be allowed within five feet from the top of the cuts.

Existing utility lines, roadways and other easements/right-of-ways may be impacted by the temporary excavations may require shoring to obtain the full depth of the excavation.

5.10 Shoring

It is understood that a temporary or permanent shoring system may be warranted for those areas of proposed excavation for the proposed structures to achieve the subterranean level grades where space is not available for properly sloped backcuts. The shoring contractor should coordinate with the earthmoving contractor regarding sequence and requirements of installing the shoring system. The shoring contractor should also consider the potential for localized perched groundwater in the design and installation procedures of the shoring system.

We anticipate that the shoring system will be designed as a cantilever system and may consist of closely spaced steel H-Pile soldier piles and wooden lagging. Preliminary design considerations are presented in the following section for this anticipated shoring method. Please note that the method of temporary support can impact the design earth pressures. As such, Kling Consulting Group should perform a review of the shoring design and provide additional recommendations, as warranted.

Shoring systems, during excavation, may yield causing adjacent facilities and improvements to settle slightly. The magnitude of shoring movements and the resulting settlements are difficult to estimate because they depend on many factors, including the method of installation and the contractor’s skill with installing the shoring system. Lateral deflections for a properly designed and constructed shoring system would likely be within ordinarily accepted limits of approximately 1-inch. A monitoring program should be established to evaluate the effects of shoring construction on other facilities.

Horizontal and vertical movements of the shoring system should be monitored by a licensed surveyor. The construction monitoring and performance of the shoring system are ultimately the contractor’s responsibility. At a minimum, we recommend that the tops of the soldier beams should be surveyed prior to excavation and that the top and bottom of the soldier beams be surveyed on a weekly basis until the foundation is completed. The surveyed soldier beam data points should be located at approximately 50 feet on-center. Surveying should consist of measuring movements in vertical and two perpendicular horizontal directions.

The shoring system should be designed to resist the pressure exerted by the retained soils plus any additional lateral forces due to loads applied near the top of the excavations. Cantilever shoring walls with a level back-fill surface should be designed for an equivalent fluid pressure of 40 pcf. For surcharge loads due to traffic, the shoring should be designed for an additional uniform horizontal pressure of 75 psf for passenger car traffic and 150 psf for heavy truck traffic. For other surcharge loads, the wall should be designed for a uniform horizontal pressure equal to one-third the anticipated surcharge pressure. These parameters all assume a level ground surface and that temporary shoring will not be subject to hydrostatic pressures. The shoring system should be properly embedded beneath the toe of the excavation to provide adequate structural stability.

It is recommended that the design of the shoring system incorporate a passive equivalent fluid weight of 300 pcf for the shoring embedded within relatively competent old paralic deposits material. The soldier piles should be spaced no closer than 3 diameters on center. The soldier piles should be drilled and back-filled with concrete to the full depth of the passive resistance zone. The area providing the passive resistance can be assumed to have a width equal to twice the concrete pile diameter.

The recommended passive pressure for the shoring assumes a horizontal surface for the soil mass extending at least 10 feet in front of the face of the shoring, or three times the height of the surface generating passive pressure, whichever is greater. The shoring system should be embedded a sufficient depth beneath the toe of the excavation so as to provide structural stability. We recommend that a factor of safety of at least 1.2 be applied to the calculated embedment depth and that the passive pressure be limited to 2,500 psf. The assumed geotechnical conditions should be verified as necessary during shoring construction by a representative of the geotechnical consultant.

Timber lagging may be used between the soldier piles to help support the exposed soils. If lagging is to remain after construction, treated lumber should be used. Lagging should be designed for the full lateral pressure recommended above. If possible, structural walls should be cast directly against the shoring, thus eliminating the need for placing back-fill within a narrow space. Voids between the soil and lagging should be properly grouted or slurried to reduce the potential for the voids to propagate to the surface.

Special provisions for wall drainage (such as the use of prefabricated composite drain) may be necessary above the groundwater table where this type of construction is used.

The performance of the proposed shoring system is highly dependent on the means and methods utilized by the contractors involved in the work and the judgment of the shoring design engineer. The shoring engineer and contractor shall be solely responsible for locating the existing improvements surrounding the site, controlling settlements of the surrounding structures and improvements within the structural and aesthetic limits. Load path and loading determination for underpinning design is the purview of the structural underpinning designer.

If the anticipated depth of excavation requires shoring that extends to depths where a cantilever shoring system is not feasible, we would be pleased to provide geotechnical recommendations for an anchored (tie-back) shoring system upon request. With deep excavations required to allow for the construction of subterranean levels that would normally require tie-back anchors, due to the proximity to the adjacent properties or structures tie-back systems may not be allowed and other options such as H-beam and lagging or rakers may be required.

5.11 Preliminary Pavement Design

Pavement section design is provided below based on near surface soil conditions encountered during our investigation and assumed traffic loading.

5.11.1 Asphalt Concrete Pavement

The upper on-site subgrade soils were classified as sandy clays and clayey silts and sandy silts. To allow for soil variability, we are assuming an R-Value of 10 for preliminary design purposes.

Based on an R-value of 10, the parameters below are provided for preliminary design purposes. Pavement sections were calculated for traffic indices of 4.0 and 5.5, which are commonly used for parking stalls and drive aisles subject to passenger vehicles, respectively. However, the selection of actual traffic index should be the purview of the project civil or traffic engineer.

Pavement Section Design

Location	**R-Value**	**Traffic Index**	**Multiple Layered**	
			Asphalt Concrete (inches)	**Aggregate Base* (inches)**
Parking Stall	10	4.0	3.0	6.0
Drive Aisles	10	5.5	4.0	9.0

*Aggregate base material should consist of Class 2 aggregate base materials or Crushed Miscellaneous Base (CMB).

The upper 12 inches of the subgrade soils should be compacted to at least 90 percent of the laboratory maximum dry density (ASTM D1557). All base materials should be compacted to at least 95 percent of the laboratory maximum dry density (ASTM D1557).

5.11.2 Portland Cement Concrete Pavement

For preliminary design of concrete pavement, it is recommended that a concrete pavement section consisting of 6-inches of concrete underlain by at least 4-inches of either Class 2 or crushed miscellaneous base be used for preliminary design. Concrete Compressive strength should be 4000 psi or greater. Aggregate base material should be compacted to a minimum of 95 percent relative compaction as per ASTM D1557. Subgrade soil should be compacted to at least 90 percent of the laboratory maximum dry density in accordance with ASTM D1557. If concrete crack control is desired, the slabs should be minimally reinforced with No. 4 rebar, placed every 24 inches on center, both ways. A 10-foot square or less grid system should be used in the construction of continuous sections of concrete pavement or as recommended by the structural engineer.

For trash enclosures, concrete pavement should consist of a minimum 8-inch thick concrete slab placed over a minimum of 6-inches of either Class 2 or crushed miscellaneous base material, compacted to 95 percent relative compaction. Concrete should have a minimum strength of 4000 psi and be reinforced with a minimum of No. 4 bars placed at 24 inches on center, in each direction, positively supported (with concrete chairs or other devices) at mid-height in the slab. Crack control joints should be placed at a 10-foot maximum spacing in each direction in the slab or as recommended by the structural engineer. Concrete mix design should incorporate the recommendations presented in the slab on grade section of this report for improved geotechnical performance.

5.12 Exterior Flatwork

Laboratory testing of onsite soils by and our experience with similar soils in the site vicinity indicate that the upper on-site soil materials present possess a very low to high expansion potential. **Appendix E** contains a table listing our hardscape recommendations for varying degrees of expansive soils. This table should be preliminarily followed for a low to high expansion potential for Expansion Index (E.I.) = 21 to 130. Additional testing should be performed during future supplemental investigation and subsequently during earthwork construction to confirm the as graded conditions.

The following general recommendations may be considered for concrete hardscape including expansive soils mitigation and may be superseded by the requirements of the City of Newport Beach. These recommendations are based on "medium" expansion potential and are preliminary.

5.12.1 Sidewalk, Pedestrian Walkways

Expansion Potential	Minimum Concrete Thickness (in)	Subgrade Pre-Soaking Depth	Reinforcement	Joint * Spacing
Medium (EI >51 & <90)	4 (Full)	120% of Optimum to 18" (or 5% over optimum,	#3 @ 16" OC, EW	4-5 Feet

* Joints at curves and angle points are recommended.

The above recommendations may be superseded by the project architect, structural engineer or the governing agency's requirements. These recommendations are not intended to mitigate cracking caused by shrinkage and temperature warping.

5.13 Drainage

Positive drainage should be maintained away from any building or graded slope face and directed to suitable areas via non-erosive devices, as designed by the project civil engineer. For drainage over soil and paved areas immediately adjacent to structures, please refer to Section 1804.4 of the 2019 CBC.

5.14 Geotechnical Observation and Testing

Geotechnical observation and testing should be conducted during the following stages of grading:

- During all phases of precise grading, footing excavations, etc.
- During slab subgrade pre-saturation and moisture conditioning.
- During utility trench excavation and compaction.
- During placement of retaining wall sub-drainage, back-fill, and compaction.
- For any unusual conditions encountered during grading.

6.0 PROFESSIONAL LIMITATIONS

Geotechnical services are provided by KCG in accordance with generally accepted professional engineering and geologic practice in the area where these services are to be rendered. Client acknowledges that the present standard in the engineering and geologic and environmental profession does not include a guarantee of perfection and, except as expressly set forth in the conditions above, no warranty, expressed or implied, is extended by KCG.

Geotechnical reports are based on the project description and proposed scope of work as described in the proposal. Our conclusions and recommendations are based on the results of the field, laboratory, and office studies, combined with an interpolation and extrapolation of soil conditions as described in the report. The results reflect our geotechnical interpretation of the limited direct evidence obtained. Our conclusions and recommendations are made contingent upon the opportunity for KCG to continue to provide geotechnical services beyond the scope in the proposal to include all geotechnical services. If parties other than KCG are engaged to provide such services, they must be notified that they will be required to assume complete responsibility for the geotechnical work of the project by concurring with the recommendations in our report or providing alternate recommendations.

It is the reader's responsibility to verify the correct interpretation and intention of the recommendations presented herein. KCG assumes no responsibility for misunderstandings or improper interpretations that result in unsatisfactory or unsafe work products. It is the reader's further responsibility to acquire copies of any supplemental reports, addenda, or responses to public agency reviews that may supersede recommendations in this report.

APPENDIX A

REFERENCES

APPENDIX A

REFERENCES

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APPENDIX B

EXPLORATION BORING LOGS

LOG OF EXPLORATORY BORING

Sheet **1** of **3**

Project: **The Picerne Group**
Project Number: **21016-00**
Date Drilled: **4/5/21**
Logged By: **S.W**

Boring No.: **KB-1**
Driller: **Cal-Pac**
Drill Type: **Hollow-Stem Auger**
Hammer Wt. / Drop: **18in**
Ground Elev. [ft]: **---**

◪ Standard Split Spoon
■ Shelby Tube
▽ Water Level ATD
⧗ California
☒ Bulk Sample
▼ Static Water Table

Depth [ft]	Graphic Log	Sample Type	Blows/6"	Moisture Content [%]	Dry Density, [pcf]	SOIL DESCRIPTION and CLASSIFICATION (USCS)	Pocket Pen. [tsf]	Lab Tests	Remarks
						Surficial Ashphalt Layer **Old Paralic Deposits (Qop)**			
						@ 0.3 feet - Sandy CLAY (CL): light brown, very stiff, dry. **@ 2.0 feet - Sandy SILT (ML):** light brown, very stiff, dry.			
		⧗	7 13 13	6.2	116.7				
5		⧗	7 12 13	15.7	105.6	**@ 5.0 feet - Clayey SILT (ML):** dark grayish brown, very stiff, moist.			
10		⧗	8 19 33	16.3	117.2				
15		◪	7 9 17			**@ 15.0 feet - Clayey SILT (ML):** dark gray, very stiff, moist.			
20		◪	3 8 13						

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LOG OF EXPLORATORY BORING

Sheet **2** of **3**

Project: **The Picerne Group**
Project Number: **21016-00**
Date Drilled: **4/5/21**
Logged By: **S.W**

Boring No.: **KB-1**
Driller: **Cal-Pac**
Drill Type: **Hollow-Stem Auger**
Hammer Wt. / Drop: **18in**
Ground Elev. [ft]: **---**

◩ Standard Split Spoon
■ Shelby Tube
▽ Water Level ATD
⧓ California
☒ Bulk Sample
▼ Static Water Table

Depth [ft]	Graphic Log	Sample Type	Blows/6"	Moisture Content [%]	Dry Density, [pcf]	SOIL DESCRIPTION and CLASSIFICATION (USCS)	Pocket Pen. [tsf]	Lab Tests	Remarks
		◩	10 24 26			**@ 26.0 feet - Silty Sand (SM):** fine grained, light brownish orange, very dense, wet.			▼
30		◩	2 13 21			**@ 31.0 feet - Sand (SM):** medium-coarse grained, light gray, dense, wet, shell fragments.			
35		◩	4 15 25						
40		◩	1 2 3			**@ 40.0 feet - CLAY (CH):** dark gray, firm, wet.			
45		◩	1 3 3						

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LOG OF EXPLORATORY BORING

Sheet **3** of **3**

Project: **The Picerne Group**
Project Number: **21016-00**
Date Drilled: **4/5/21**
Logged By: **S.W**

Boring No.: **KB-1**
Driller: **Cal-Pac**
Drill Type: **Hollow-Stem Auger**
Hammer Wt. / Drop: **18in**
Ground Elev. [ft]: **---**

◩ Standard Split Spoon
■ Shelby Tube
▽ Water Level ATD
⧓ California
⊠ Bulk Sample
▼ Static Water Table

Depth [ft]	Graphic Log	Sample Type	Blows/6"	Moisture Content [%]	Dry Density, [pcf]	SOIL DESCRIPTION and CLASSIFICATION (USCS)	Pocket Pen. [tsf]	Lab Tests	Remarks
		◩	2			Total Depth: 50 feet.			
			2			Groundwater encountered at 26ft.			
			3			No caving.			

LOG OF EXPLORATORY BORING

Sheet **1** of **3**

Project:	**The Picerne Group**	Boring No.:	**KB-2**
Project Number:	**21016-00**	Driller:	**Cal-Pac**
Date Drilled:	**4/5/21**	Drill Type:	**Hollow-Stem Auger**
Logged By:	**S.W**	Hammer Wt. / Drop:	**18in**
		Ground Elev. [ft]:	**---**

◩ Standard Split Spoon
■ Shelby Tube
▽ Water Level ATD
⧗ California
☒ Bulk Sample
▼ Static Water Table

Depth [ft]	Graphic Log	Sample Type	Blows/6"	Moisture Content [%]	Dry Density, [pcf]	SOIL DESCRIPTION and CLASSIFICATION (USCS)	Pocket Pen. [tsf]	Lab Tests	Remarks
	■					**Surficial Ashphalt Layer**			
	▨					**Old Paralic Deposits (Qop)** **@ 0.4 feet - Clayey SILT (ML):** dark grayish brown, very stiff, moist.			
	▨	⧗	5 8 7	12.6	115.9				
5	⁚	⧗	3 6 11	23.3	102.9	**@ 5.0 feet - Sandy SILT (ML):** light brown, very stiff, dry.			
10	⁚	⧗	6 10 16	20.9	107.7				
15	⁚	◩	3 6 7			**@ 15.0 feet - SILT (ML):** dark grayish brown, some clay, stiff, moist.			
20	⁚	◩	3 6 6			**@ 20.0 feet - Clayey SILT (ML):** dark gray, stiff, moist.			

LOG OF EXPLORATORY BORING

Sheet **2** of **3**

Project: **The Picerne Group**
Project Number: **21016-00**
Date Drilled: **4/5/21**
Logged By: **S.W**

Boring No.: **KB-2**
Driller: **Cal-Pac**
Drill Type: **Hollow-Stem Auger**
Hammer Wt. / Drop: **18in**
Ground Elev. [ft]: **---**

◩	Standard Split Spoon	■	Shelby Tube
⧓	California	☒	Bulk Sample
▽	Water Level ATD	▼	Static Water Table

Depth [ft]	Graphic Log	Sample Type	Blows/6"	Moisture Content [%]	Dry Density, [pcf]	SOIL DESCRIPTION and CLASSIFICATION (USCS)	Pocket Pen. [tsf]	Lab Tests	Remarks
		◩	3 7 9						
30		◩	2 4 5						
35		◩	4 6 9			**@ 35.0 feet - Sand (SM):** fine grained, light brownish orange, medium dense, wet.			▼
40		◩	10 7 7			**@ 41.0 feet - CLAY (CH):** dark gray, firm, wet.			
45		◩	2 2 4						

LOG OF EXPLORATORY BORING

Sheet **3** of **3**

Project:	**The Picerne Group**	Boring No.:	**KB-2**
Project Number:	**21016-00**	Driller:	**Cal-Pac**
Date Drilled:	**4/5/21**	Drill Type:	**Hollow-Stem Auger**
Logged By:	**S.W**	Hammer Wt. / Drop:	**18in**
		Ground Elev. [ft]:	**---**

◩ Standard Split Spoon ■ Shelby Tube ▽ Water Level ATD

⧓ California ☒ Bulk Sample ▼ Static Water Table

Depth [ft]	Graphic Log	Sample Type	Blows/6"	Moisture Content [%]	Dry Density, [pcf]	SOIL DESCRIPTION and CLASSIFICATION (USCS)	Pocket Pen. [tsf]	Lab Tests	Remarks
		◩	2 3 3			Total Depth: 50 feet. Groundwater encountered at 35ft. No caving.			

APPENDIX C

LABORATORY TEST PROCEDURES AND RESULTS

APPENDIX C

LABORATORY TEST PROCEDURES

VISUAL CLASSIFICATION OF SOILS

As a part of the routine laboratory soil testing, the soil samples are visually classified in accordance with the Unified Soil Classification System by experienced laboratory technicians. If necessary, in order to verify the visual classification, selected samples are classified utilizing the results of Standard Classification tests performed in accordance with ASTM D2487-00.

MOISTURE CONTENT AND DRY DENSITY DETERMINATION

Moisture content and dry density determinations were performed on relatively undisturbed samples obtained during our field exploration. The field moisture content is obtained by methods described in ASTM D2216-05. The in-situ dry unit weight was computed using the net weight and volume of the relatively undisturbed samples. The results of these tests are presented on the borings logs in Appendix B.

DIRECT SHEAR TESTS

Direct shear tests were performed in general accordance with ASTM D3080-98 on selected remolded and relatively undisturbed samples that were pre-soaked for a minimum of 24 hours. The samples were then tested under various normal loads with a different specimen being used for each normal load. The samples were sheared in a motor driven, strain-controlled direct shear testing apparatus at a strain rate of 0.05 inches per minute. The results of this test are presented in the Laboratory Summary.

EXPANSION INDEX TEST

The expansion potential of selected materials was evaluated by the Expansion Index Test, U.B.C. Standard No. 18-2. The specimen was molded under a given compactive energy and moisture content to achieve approximately 50 percent saturation. The prepared 1-inch thick by 4-inch diameter specimen was then loaded with a 144 psf surcharge and inundated with water until volumetric equilibrium is reached. The result of this test is presented in the Laboratory Summary.

CONSOLIDATION TESTS

Consolidation tests were performed in general accordance with ASTM D2435 on selected, relatively undisturbed, ring samples recovered from the exploratory excavations. Samples are placed in a consolidometer where increasing load increments are applied in geometric progression. The soil specimen is placed between porous stones that allow water to infiltrate and to flow of the soil sample. During the loading stages prior to the addition of water, the soil sample is sealed in order to prevent evaporation of soil water. The load increment where water was added is indicated on the consolidation pressure curves. The percent consolidation for each load cycle is recorded as the ratio of the amount of vertical compression to the original 1-inch height. The results of these tests are presented graphically as an attachment in this Appendix.

SOLUBLE SULFATES

Soluble sulfate tests determined in general accordance with California Test Method No. 417 were also performed on representative samples collected during the field investigation. Soils with a sulfate concentration greater than 0.07% may be corrosive to metals; concentrations greater than 0.10% are considered potentially harmful to concrete and would require following the current ACI or CBC for "moderate" or more severe sulfate exposure requirements. The results of this test are presented in the Laboratory Summary.

LABORATORY TEST RESULTS

PROJECT: THE PICERNE GROUP **NO.:** 21016-00 **TECHNICIAN :** RB DATE : 07-Apr-21

Hole No.		B1			B2							
Sample No.												
Sample Depth:		3'	5'	10'	3'	5'	10'					
Visual Soil Classification	Top	DK. BROWN SILTY FINE SAND (SM)	LT. BROWN SANDY CLAY (CL)	BROWN SANDY CLAY (CL)	DK. BROWN CLAYEY SAND (SC)	BROWN SANDY CLAY (CL)	SAME					
	Bottom		BR. SILTY FINE SAND (SM)									
Pocket Penetrometer Reading, (tsf)		> 4.5	> 4.5	> 4.5	> 4.5	> 4.5	> 4.5					
Weight of Moist Soil and Rings, (gms.)		969.90	959.20	1043.60	1009.20	987.60	1007.70					
No. of Rings		5	5	5	5	5	5					
Dish No.		A5	B1	A8	B12	B2	B11					
Weight of Moist Soil and Dish, (gms.)		141.81	162.58	195.43	163.08	168.66	174.19					
Weight of Dry Soil and Dish, (gms.)		134.99	143.95	171.65	147.55	141.36	148.42					
Weight of Dish, (gms.)		25.49	25.50	25.46	24.31	24.17	25.15					
Weight of Dry Soil, (gms.)		109.50	118.45	146.19	123.24	117.19	123.27					
Wet Density, (pcf)		123.9	122.2	136.2	130.5	126.9	130.3					
Moisture Content, (%)		6.2	15.7	16.3	12.6	23.3	20.9					
Dry Density, (pcf)		116.7	105.6	117.2	115.9	102.9	107.7					
Degree of Saturation, (%) (G =2.68)		38.5	72.1	102.1	76.2	99.9	101.4					
Void Ratio		0.433	0.584	0.427	0.443	0.625	0.552					
Porosity		0.302	0.369	0.299	0.307	0.385	0.356					
Remarks :												

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MOISTURE - DENSITY TEST

Project Number : 21016-00 Tested by RB Date 4/12/2021

Project Name : THE PICERNE GROUP Sampled by S.W Date 4/6/2021

Sample No. B2 Depth/Elev. 5' - 10' Location:

Sample Descriptions / Classification : DK. BROWN SANDY CLAY (CL)

HYDROMETER ANALYSIS (ASTM STD HYDROMETER 152H)

Temp.(^{0}C)	Meniscus Corr.	K Value	Hygroscopic Moisture		Wt.of Air Dry Sample, (g)	62.61
22	8.5	0.01312	Wet Weight of Soil,(g)	149.62	Wt.of Oven Dry Sample, (g)	60.38
21	8.5	0.01328	Dry Weight of Soil,(g)	144.28	Material Passing Sieve No.	10
			Moisture Content,(%)	3.70		

Specific Gravity (γ) = 2.7 (Assumed) Correction Factor (α) = 0.99

Date	Time	apsed Time(min)	Temp. (^{0}C)	R'	C	R	% P	% P Corrected	L (cm)	k Value	L/T (cm/min)	Diameter (mm)
12-Apr	7:42	0.25										
		0.50										
		1.00	22	36	8.5	27.5	45.1	44.9	10.4	0.01312	10.4000	0.04231
		2.00										
		4.00										
		5.00										
		15.0										
		30.0										
		60.0	22	26	8.5	17.5	28.7	28.6	12.0	0.01312	0.20000	0.00587
		240.0										
		1440.0	21	21	8.5	12.5	20.5	20.4	12.9	0.01328	0.00896	0.00126

SIEVE ANALYSIS

Sieve		Weight Retained		Cummulative		Specification
Size	Opening (mm)	Individual (g)	Cummulative (g)	% Retained	% Passing	% Passing
3"	75.0					
2"	50.0					
1-1/2"	38.1					
1"	25.0					
3/4"	19.0					
1/2"	12.5					
3/8"	9.5			0.0	100.0	
#4	4.75		-	0.0	100.0	
#8	2.36					
#10	2.00		1.00	0.3	99.7	
#16	1.18					
#30	0.600		11.69	3.7	96.3	
#50	0.300		30.59	9.8	90.2	
#100	0.150		87.61	27.9	72.1	
#200	0.075		133.18	42.5	57.5	

Total Wt. of Dry Soil,(g)	313.55

	Moist	Dry
(+)#10 Sieve,(g)	-	1
(-)#10 Sieve,(g)	111.89	107.90

Sand & Gravel Particle Descriptions

Shape	Rounded	
	Angular	
Hardness	Hard & Durable	
	Soft	
	Weathered & Friable	

D_{10}		D_{60}	
D_{30}			
Coefficient of Uniformity,C_u			#DIV/0!
Coefficient of Curvature,C_c			#DIV/0!

Remarks :

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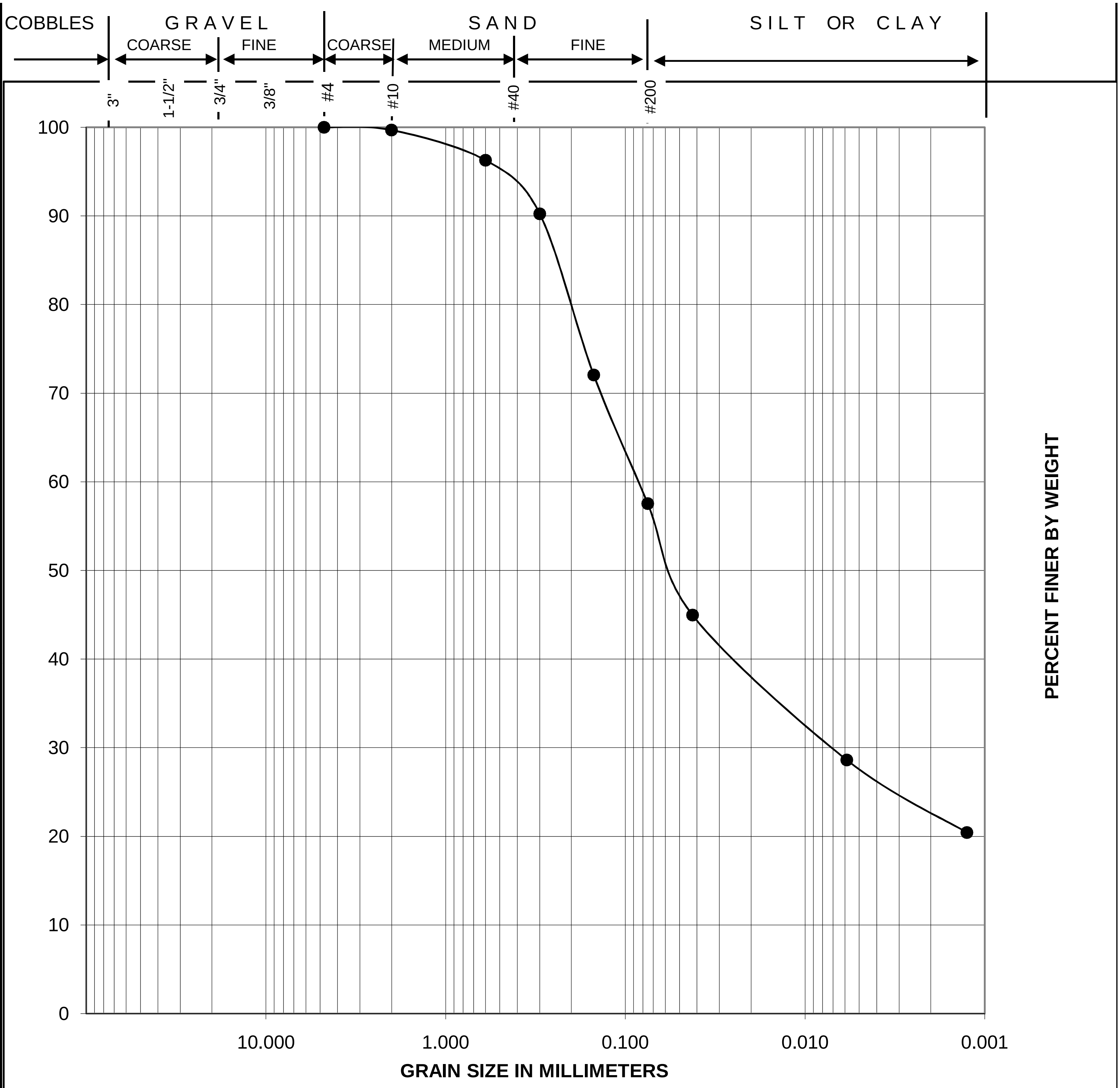
GRAIN - SIZE ANALYSIS

US STD. SIEVE OPENING IN INCHES US STD. SIEVE NUMBERS HYDROMETER



PROJECT NUMBER : 21016-00

PROJECT NAME : THE PICERNE GROUP

SAMPLE NO.	DEPTH	SYMBOL	CLASSIFICATION	NAT.W%	LL	PL	PI
B2	5' - 10'	CL	DK. BROWN SANDY CLAY		34	15	19

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GRAIN - SIZE CURVE

PROJECT NAME : THE PICERNE GROUP
PROJECT NO: 21016-00
DATE : 13-Apr-21
BORING NUMBER : B2
SAMPLE NO./DEPTH : 5' - 10'
TESTED BY : RB
SAMPLE DESCRIPTIONS/CLASSIFICATION : DK. RBOWN LEAN CLAY (CL)

PLASTIC LIMIT

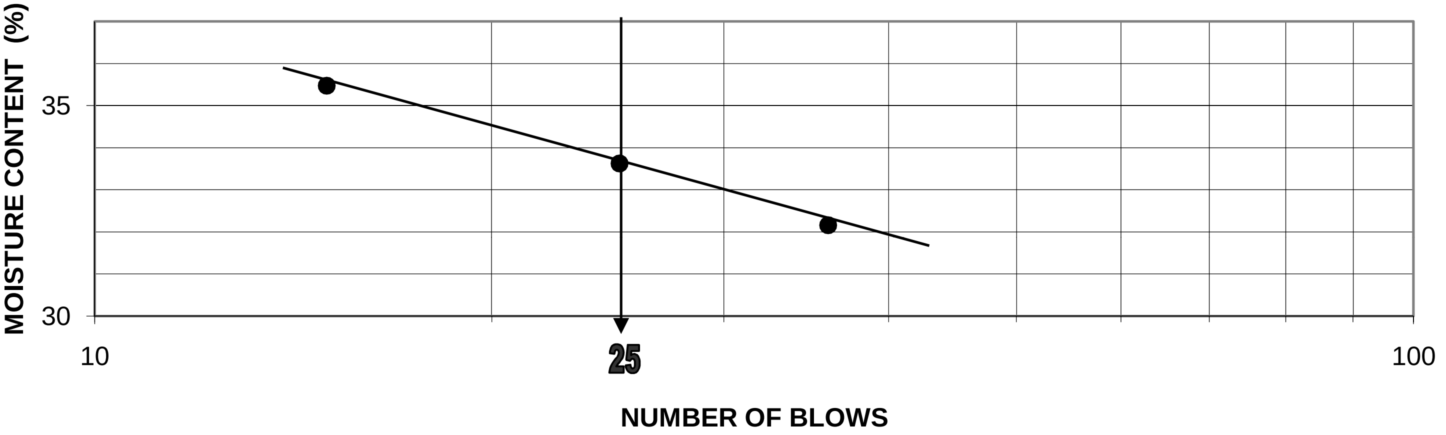
DETERMINATION NO	1	2
DISH NO.	10	
MASS OF DISH + WET SOIL,(Gms)	26.58	
MASS OF DISH + DRY SOIL,(Gms)	24.65	
MASS OF WATER,(Gms)	1.93	
MASS OF DISH,(Gms)	11.4	
MASS OF DRY SOIL,(Gms)	13.25	
MOISTURE CONTENT,(%)	14.6	

LIQUID LIMIT

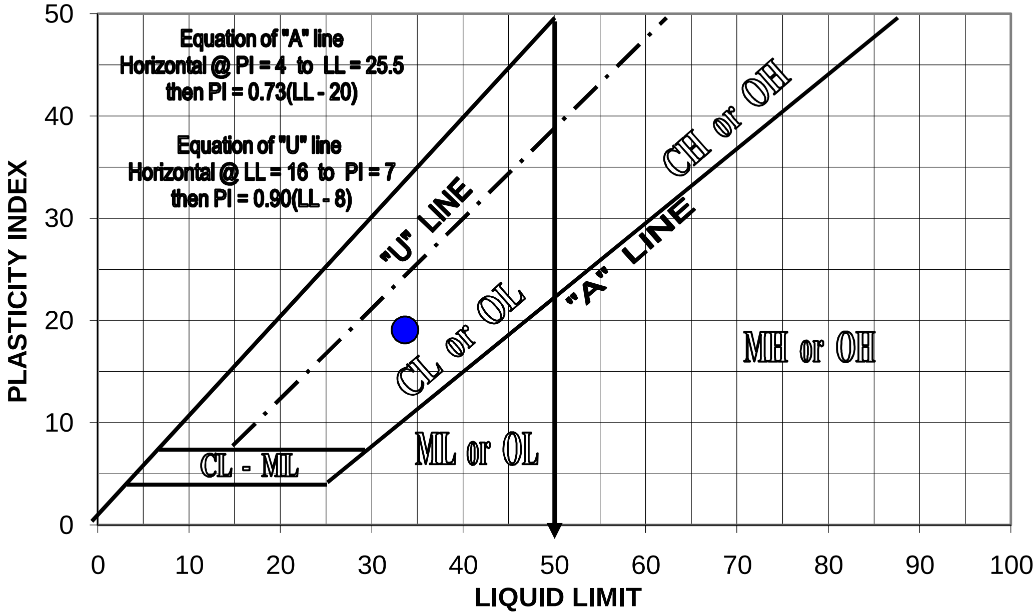
DETERMINATION NO.	1	2	3
DISH NUMBER	1	16	18
MASS, DISH + WET SOIL,(Gms)	11.81	11.80	11.69
MASS, DISH + DRY SOIL,(Gms)	9.82	9.89	9.86
MASS OF WATER,(Gms)	1.99	1.91	1.83
MASS OF DISH,(Gms)	4.21	4.21	4.17
MOISTURE CONTENT,(%)	35.5	33.6	32.2
NUMBER OF BLOWS	15	25	36

NATURAL MOISTURE CONTENT,%
0.00
#DIV/0!

FLOW CURVE



PLASTICITY CHART



RESULT SUMMARY

NATURAL MOISTURE CONTENT, (%) -
LIQUID LIMIT (LL) 34
PLASTIC LIMIT (PL) 15
PLASTICITY INDEX (PI) 19
SYMBOL FROM PLASTICITY CHART (CL)

METHOD OF PREPARATION		METHOD OF LL DETERMINATION	
DRY	X	MULTIPOINT	X
WET		ONE-POINT	

REMARKS :

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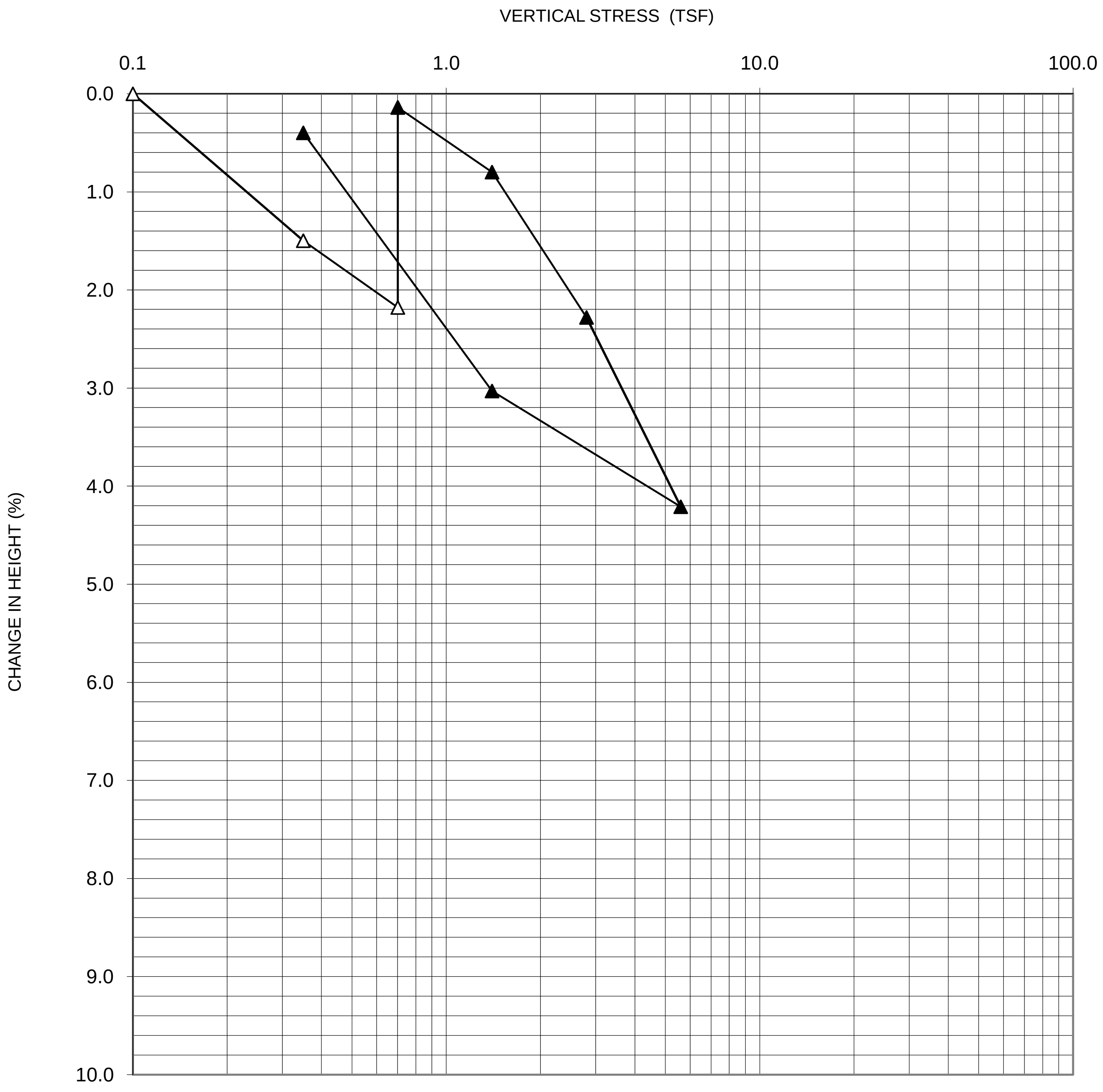
ATTERBERG LIMITS
(ASTM D4318-00)



PROJECT NO.: 21016-00

SOIL DESCRIPTIONS: BROWN SANDY CLAY (CL)

BORING NO./LOCATION : B2

DEPTH / ELEV. : 5'

LIQUID LIMIT : -

SPECIFIC GRAVITY : 2.68 (Assumed)

PLASTIC LIMIT: -

REMARKS :

	SPECIMEN HEIGHT (INCHES)	MOISTURE CONTENT (%)	DRY DENSITY (PCF)	SATURATION (%)	VOID RATIO
INITIAL	1.0000	25.7	102.2	108.4	0.636
FINAL	0.9960	27.2	102.6	115.8	0.630

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CONSOLIDATION TEST CURVE

Project Name : THE PICERNE GROUP
Project No. : 21016-00
Boring / Sample No : B1
Depth : 3' (ft.)
Tested By : RB
Date: 8-Apr-21
Sample Descriptions / Classification : DR. BROWN SILTY FINE SAND (SM)

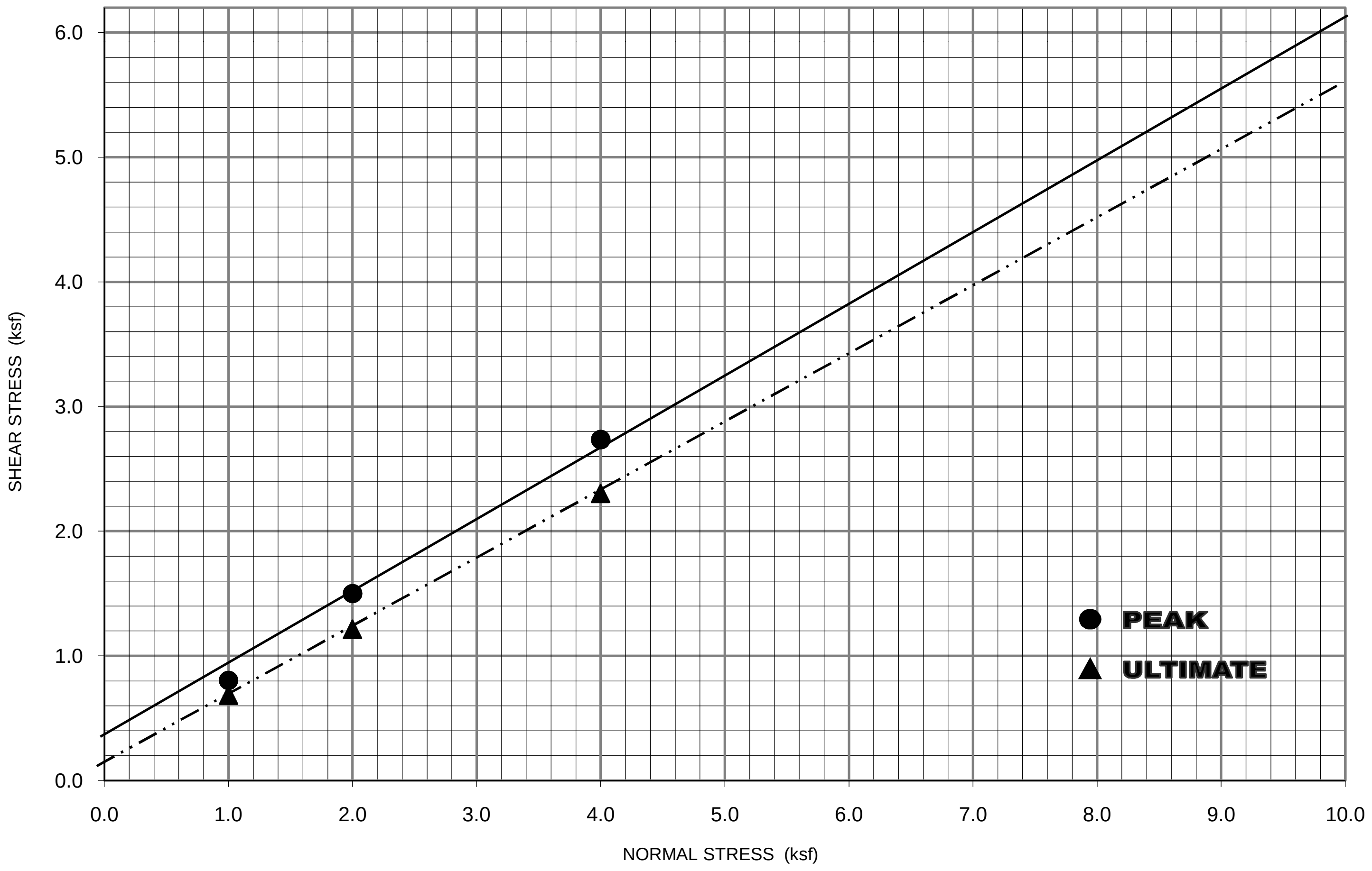
Applied Normal Load (ksf)	1.0		2.0		4.0	
Shear Stress,(Peak) (ksf)	0.804		1.500		2.736	
Shear Stress,(Ultimate) (ksf)	0.684		1.212		2.304	
Density and Saturation	Initial	Final	Initial	Final	Initial	Final
Wet Weight of Soil + Ring (gms)	190.67	201.09	195.82	205.42	198.52	206.55
Dry Weight of Soil + Ring (gms)		182.12		187.07		189.58
Weight of Water (gms)	-	46.54	-	47.28	-	45.81
Weight of Ring (gms)	-	44.19	-	45.87	-	45.43
Weight of Dry Soil (gms)	-	137.93	-	141.20	-	144.15
Moisture Content (%)	6.2	33.7	6.2	33.5	6.2	31.8
Wet Density (pcf)	122.3	131.0	125.2	133.2	127.8	134.5
Dry Density (pcf)	-	97.9	-	99.8	-	102.1
Specific Gravity,G_s (Assumed)	2.68					
Thickness of Specimen, (in.)	1.00					
Degree of Saturation, (%)	23.5	127.7	24.6	132.7	26.0	133.3
Void Ratio	-	0.708	-	0.676	-	0.639

Lateral Displacement, d_h 0.36 (in.)
Displacement Rate, d_r 0.05 (in./min.)
Elapsed Time of Test, t_e 7.20 (min.)

Specimen : Undisturbed : X
Remolded : -
Reconstituted : -

	PEAK	ULTIMATE
Cohesion,c (psf)	300	100
Friction Angle, ϕ	30	29

Remarks :



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DIRECT SHEAR TEST

(ASTM D3080)

PROJECT NAME : THE PICERNE GROUP

PROJECT NUMBER : 21016-00

TRACT NUMBER :

TESTED BY : RB DATE : 9-Apr-21

LOT NUMBER :

SAMPLED BY: S.W DATE : 07-Apr-21

SAMPLE NO. :

LOCATION : B1 @ 0' -10'

SOIL DESCRIPTIONS / CLASSIFICATION : DK. BROWN SANDY CLAY (CL)

TRIAL NUMBER		1	2	3	4
WET WT. OF SOIL + RING	(g)	570.03	616.00		
WEIGHT OF RING	(g)	204.27	204.27		
WET WEIGHT OF SOIL	(g)	365.76	411.73		
FACTOR		0.3030	0.3030		
WET DENSITY	(pcf)	110.8	124.8		
DRY DENSITY	(pcf)	105.4	114.8		
DEGREE OF SATURATION	(%)	23.1	50.2		

MOISTURE DETERMINATION

WET WEIGHT OF SOIL	(g)	308.14	305.27		
DRY WEIGHT OF SOIL	(g)	293.14	280.84		
MOISTURE CONTENT	(%)	5.1	8.7		

RACK NO. : 2

SURCHARGE : 144 psf

FINAL DENSITY & SATURATION

WET WT. + RING	(g)	
DRY WT. + RING	(g)	
MOISTURE CONTENT	(%)	
SAMPLE LENGTH	(cm)	
SAMPLE AREA	(cm^2)	
VOLUME	(cc)	
WT. OF RING	(g)	
DRY DENSITY	(pcf)	
SPEC.GRAVITY (assumed)		2.70
SATURATION	(%)	
% RETAINED ON #4 SIEVE		

DATE	TIME	ELAPSED TIME (min.)	DIAL READING (in.)	DEFLECTION (in.)
9-Apr	9:00		0.221	
9-Apr	11:00		0.245	
12-Apr	7:10		0.283	0.062

E. I. 62

SO_4 270 ppm

REMARKS :

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EXPANSION INDEX

(UBC 18-2)

PROJECT NAME : THE PICERNE GROUP

PROJECT NUMBER : 21016-00

TRACT NUMBER :

TESTED BY : RB

DATE : 9-Apr-21

LOT NUMBER :

SAMPLED BY: S.W

DATE : 07-Apr-21

SAMPLE NO. :

LOCATION : B2 @ 5' -10'

SOIL DESCRIPTIONS / CLASSIFICATION : DK. BROWN SANDY CLAY (CL)

TRIAL NUMBER		1	2	3	4
WET WT. OF SOIL + RING	(g)	574.19	606.95		
WEIGHT OF RING	(g)	204.39	204.39		
WET WEIGHT OF SOIL	(g)	369.80	402.56		
FACTOR		0.3030	0.3030		
WET DENSITY	(pcf)	112.0	122.0		
DRY DENSITY	(pcf)	105.6	111.4		
DEGREE OF SATURATION	(%)	27.6	50.0		

MOISTURE DETERMINATION

WET WEIGHT OF SOIL	(g)	312.59	308.03		
DRY WEIGHT OF SOIL	(g)	294.64	281.31		
MOISTURE CONTENT	(%)	6.1	9.5		

RACK NO. : 3

SURCHARGE : 144 psf

FINAL DENSITY & SATURATION

WET WT. + RING (g)	
DRY WT. + RING (g)	
MOISTURE CONTENT (%)	
SAMPLE LENGTH (cm)	
SAMPLE AREA (cm^2)	
VOLUME (cc)	
WT. OF RING (g)	
DRY DENSITY (pcf)	
SPEC.GRAVITY (assumed)	2.70
SATURATION (%)	
% RETAINED ON #4 SIEVE	

DATE	TIME	ELAPSED TIME (min.)	DIAL READING (in.)	DEFLECTION (in.)
9-Apr	9:43		0.597	
9-Apr	11:00		0.613	
12-Apr	7:10		0.658	0.061

E. I. 61

SO_4 ppm

REMARKS :

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EXPANSION INDEX

(UBC 18-2)

APPENDIX D

SEISMIC SETTLEMENT ASSESSMENT

Hole No.= Water Depth=10 ft

Magnitude=6.89
Acceleration=0.612g

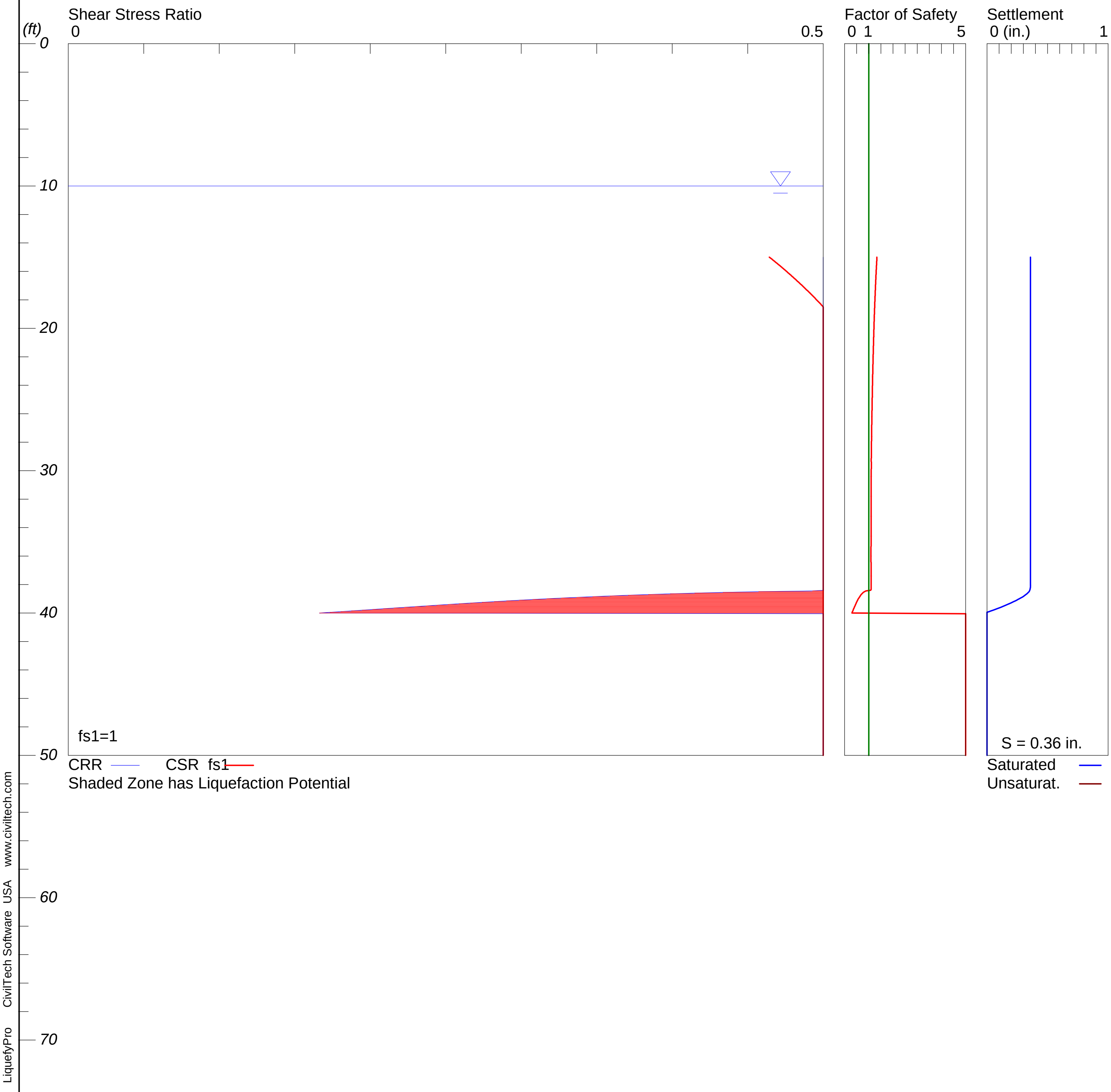

Plate A-1

Hole No.=   Water Depth=10 ft

Magnitude=6.89
Acceleration=0.612g

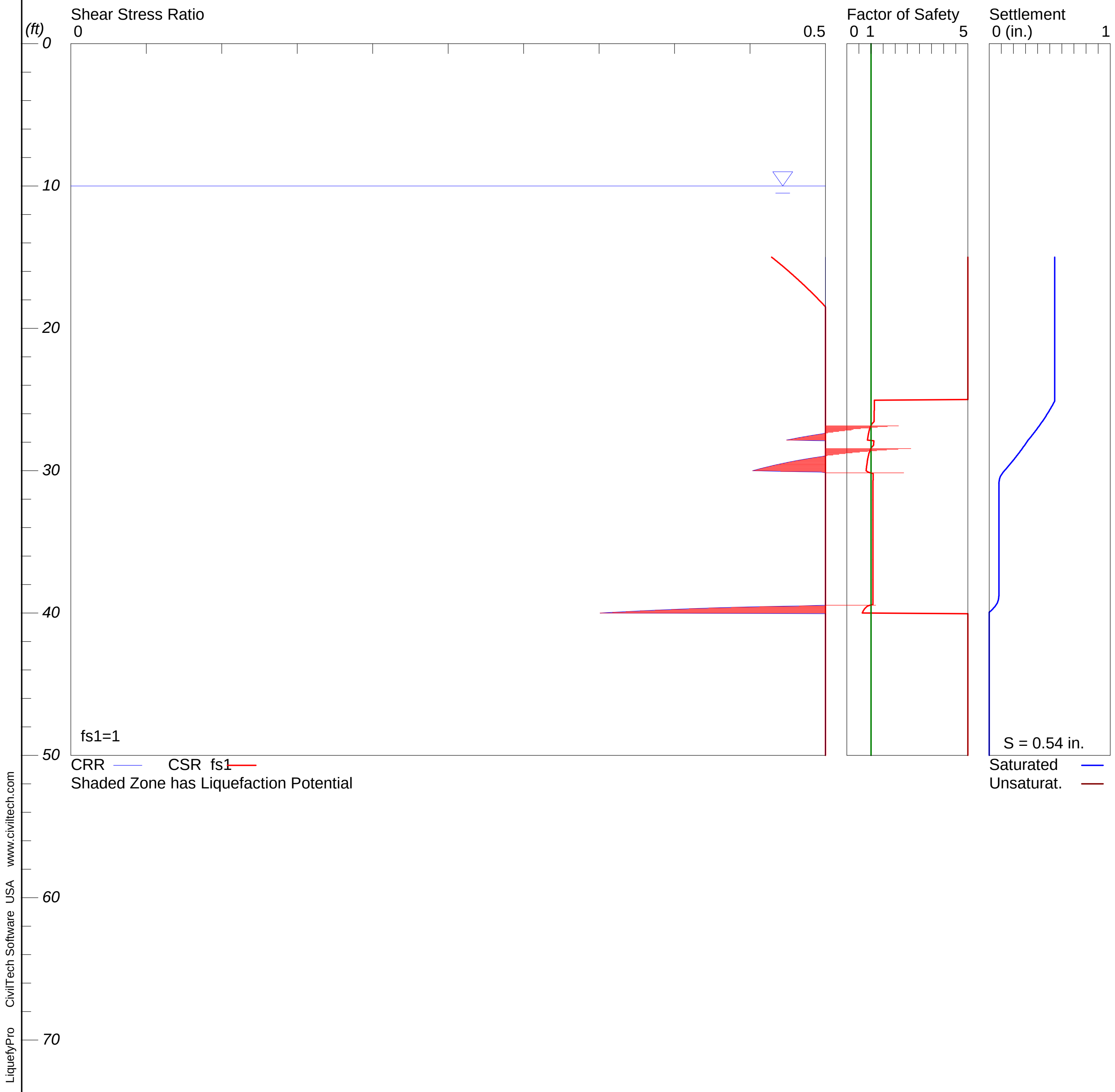

Plate A-1

APPENDIX E

HARDSCAPE RECOMMENDATIONS

KCG KLING Consulting Group, Inc.

HARDSCAPE RECOMMENDATIONS FOR EXPANSIVE SOILS
(Residential)

Description	Minimum Concrete Thickness (Inches)	Subgrade Pre-Soaking Depth	Reinforcement (1), (3), (5)	Cutoff Barrier or Edge Thickness (Inches)	Joint (2), (5) Spacing (Max)	Base
Common Sidewalks - Isolated			N.R.		4-5 Feet	N.R.
EI<21	4 (Nominal)	Optimum to 12"		N.R.		
EI 21-50	4 (Nominal)	120% of Optimum to 12"		N.R.		
EI 51-90	4 (Nominal)	120% of Optimum to 18"		18		
EI 91-130	4 (Nominal)	130% of Optimum to 18"		18		
EI>130	4 (Nominal)	140% of Optimum to 18"		24		
		(or 5% over optimum, whichever is greater)				
Common Sidewalks - Not Isolated (N.I.)				For exposed edges utilize recommendations for isolated condition	4-5 Feet	N.R.
EI<21	4 (Nominal)	Optimum to 12"	N.R.			
EI 21-50	4 (Nominal)	120% of Optimum to 12"	N.R.			
EI 51-90	4 (Nominal)	120% of Optimum to 18"	#3 @ 18" OC,EW			
EI 91-130	4 (Nominal)	130% of Optimum to 24"	#3 @ 12" OC,EW			
EI>130	4 (Full)	140% of Optimum to 24"	#4 @ 12" OC,EW			
		(or 5% over optimum, whichever is greater)				
City/County Standard Sidewalks	4 (Nominal) or C.S.	Same as Common Sidewalk Not Isolated or C.S.	Same as Common Sidewalk Not Isolated or C.S.	Same as Common Sidewalk No Isolated or C.S.	4-5 Feet	N.R.
Private Driveways (1 Unit)	4 (Full)	Same as Common Sidewalk Not Isolated	6x6 - W2.9xW2.9 Mesh	Same as Common Sidewalk Isolated	10 Feet	N.R.
Shared Driveways (2 Units)	5 (Full)	Same as Common Sidewalk Not Isolated	6x6 - W2.9xW2.9 Mesh	Same as Common Sidewalk Isolated	10 Feet	N.R.
Courts or Enhanced Concrete (where higher degree of crack control is desired)						Sand Leveling course, if desired
EI<21	5 (Full)	Optimum to 12"	6x6 - W1.4xW1.4 Mesh	N.R.	10 Feet	
EI 21-50	5 (Full)	120% of Optimum to 12"	6x6 - W1.4xW1.4 Mesh	N.R.	10 Feet	
EI 51-90	5 (Full)	120% of Optimum to 18"	#3 @ 18" OC,EW	18	10 Feet	
EI 91-130	6 (Full)	130% of Optimum to 24"	#3 @ 12" OC,EW	24	10 Feet	
EI>130	6 (Full)	140% of Optimum to 24"	#4 @ 12" OC,EW	24	8 Feet	
		(or 5% over optimum, whichever is greater)				
Concrete Pavement (4)	6.0	N.R.	Same as for Courts	Same as for Courts	10 Feet	6" aggregate base
Patios and Entryways						N.R.
E<21	4 (Nominal)	Optimum to 12"	6x6 - W1.4xW1.4 Mesh	N.R.	10 Feet	
EI 21-50	4 (Nominal)	120% of Optimum to 12"	6x6 - W1.4xW1.4 Mesh	N.R.	10 Feet	
EI 51-90	4 (Nominal)	120% of Optimum to 18"	6x6 - W2.9xW2.9 Mesh or		10 Feet	
			#3 @ 24" OC,EW	12		
EI 91-130	4 (Nominal)	130% of Optimum to 18"	#3 @ 18" OC,EW	18	10 Feet	
EI>130	4 (Full)	130% of Optimum to 24"	#3 @ 12" OC,EW	24	5 Feet	
		(or 5% over optimum, whichever is greater)				

Other Considerations:

Square concrete panels when possible

Maintain positive drainage for concrete flatwork
NR = Not required; C.S. = City/County Standard;
OC = On Center
EW = Each Way
NI = Not Isolated

Notes:

(1) Reinforcement to extend into cutoff barrier in thickened edge

(2) Joint at curves or angle points
(3) Reinforcement may be superseded by the structural engineer
(4) Actual design should be based on soil "k" value and proposed traffic loads.
Reinforcement and joint spacing requirements should be determined by the structural engineer.
(5) These recommendations are not intended to mitigate against cracking caused by shrinkage and temperature warping.
Mitigation of these aspects should be provided by the structural engineer.

General Notes:

The recommendations herein should be considered as general guidelines and should be implemented if a "moderate" degree of crack control is desired. Should a higher degree of risk management be desired, these recommendations could be revised upon request.

APPENDIX F

ASFE INSERT

Important Information About Your

Geotechnical Engineering Report

Subsurface problems are a principal cause of construction delays, cost overruns, claims, and disputes

The following information is provided to help you manage your risks.

Geotechnical Services Are Performed for Specific Purposes, Persons, and Projects

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 engineering 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 alight 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. *Geotechnical engineers cannot accept responsibility or liability for problems that occur because their reports do not consider developments of which they were not informed.*

Subsurface Conditions Can Change

A geotechnical engineering report is based on conditions that existed at the time the study was performed. *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 Opinions

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 judgment 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 overrely on the construction recommendations included in your report. *Those recommendations are not final*, because geotechnical engineers develop them principally from judgment and opinion. Geotechnical engineers can finalize their recommendations only by observing actual subsurface conditions revealed during construction. The geotechnical engineer who developed your report cannot assume responsibility or liability for the report's recommendations if that engineer does not perform construction observation.

A Geotechnical Engineering Report Is Subject to Misinterpretation

Other design team members' misinterpretation of geotechnical engineering reports has resulted in costly problems. Lower 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. Contractors can also misinterpret a geotechnical engineering report. Reduce that risk by having your geotechnical engineer participate in prebid and preconstruction conferences, and by providing 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 Contractors a Complete Report and Guidance

Some owners and design professionals mistakenly believe they can make contractors liable for unanticipated subsurface conditions by limiting what they provide for bid preparation. To help prevent costly problems, give contractors the complete geotechnical engineering report, *but* preface it with a clearly written letter of transmittal. In that letter, advise contractors 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 contractors have sufficient time* to perform additional study. Only then might you be in a position to give contractors 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 contractors 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 engineers' 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.

Geoenvironmental Concerns Are Not Covered

The equipment, techniques, and personnel used to perform a *geoenvironmental* study differ significantly from those used to perform a *geotechnical* study. For that reason, a geotechnical engineering report does not usually relate any geoenvironmental 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 geoenvironmental 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, a number of 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 ASFE-Member Geotechnical Engineer For Additional Assistance

Membership in ASFE/The Best People on Earth exposes geotechnical engineers to a wide array of risk management techniques that can be of genuine benefit for everyone involved with a construction project. Confer with your ASFE-member geotechnical engineer for more information.

ASFE
The Best People on Earth

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