Appendix C

Geotechnical Study

advanced geotechnical services, inc.

December 6, 2019
Client Number 4994
Report Number 10422

The Molasky Group of Companies
600 Galleria Parkway, Suite 995
Atlanta, Georgia 30339
Attn: William Valentine

Geotechnical Engineering Study
Proposed Department of Veterans Affairs
Community Based Outpatient Clinic
5250 Ralston Street
Ventura, California

In accordance with our proposal and your authorization, Advanced Geotechnical Services, Inc., (AGS) has prepared this *Geotechnical Engineering Study* for the proposed Department of Veterans Affairs Community Based Outpatient Clinic to be constructed at the subject site. This report presents the results of our data research, subsurface exploration, laboratory testing, and our professional opinions regarding the geotechnical engineering factors that may affect the proposed development.

Based on the results of this study, it is our opinion that the site is *suitable* for construction of the proposed development, provided recommendations of this report are properly incorporated in the design and implemented during construction.

This opportunity to be of service is sincerely appreciated. This report should be read from cover to cover to understand its limitations and to avoid taking a recommendation out-of-context. If you have any questions, or if we may be of any further assistance, please do *not* hesitate to call. We look forward to being of continued service.

Respectfully submitted,
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GEOTECHNICAL ENGINEERING STUDY

Proposed Department of Veterans Affairs
Community Based Outpatient Clinic
5250 Ralston Street
Ventura, California

Report to
The Molasky Group of Companies
Atlanta, Georgia

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1. INTRODUCTION

1.1 General Remarks

This *Geotechnical Engineering Study* has been prepared for the proposed Department of Veterans Affairs Community Based Outpatient Clinic to be constructed at the subject site. The proposed development will consist of the construction of a single-story building, to be constructed at or near existing site grade, subsequent to site grading. New parking areas, access driveways, infiltration features for stormwater management and landscaping will surround the proposed structure. The purposes of this study are to (1) evaluate the seismicity of the site and potential seismic hazards; (2) identify on-site soil conditions that may affect the proposed development, and (3) provide geotechnical recommendations for the development of the site. This report presents the findings of our data review, subsurface exploration, laboratory testing, engineering analyses and evaluations, and our conclusions and recommendations.

Appendices are attached following the main report. Appendix A includes an explanation of the field exploration, and the boring logs; Appendix B includes the laboratory test results; Appendix C includes the results of the seismicity study; Appendix D includes the results of the liquefaction evaluation; Appendix E includes the references used in this study, and the Figures and Plates referenced in this report are included in Appendix F.

1.2 Scope of Services

This geotechnical engineering study included:

a. Site observation and review of geotechnical and geologic data of the general study area. A *Site Location Map* showing a broad view of the overall area where the site is located is provided as Figure 1, and an *Existing Site Plan* showing more detailed current site conditions, boring locations, the proposed building location and liquefaction hazard area is provided as Figure 2. Figure 1 was created using an image obtained from the Google Earth web app, and Figure 2 was created using an image obtained from the County of Ventura *County View* website, which uses a Google Earth image as a base map, with County hazard information overlain. A *Proposed Site Plan* is provided as Plate 1, which was created utilizing a *Preliminary Grading Plan* provided by the client, dated November 9, 2018.

b. Reconnaissance of the site and the immediate vicinity of the subject site.

c. Drilling, sampling, and logging of 14 borings to depths between approximately 5 and 51.5 feet below the existing ground surface. The exploratory boring locations were determined in the field using a tape measure and approximate reference points. Thus, the actual locations of the exploratory borings may deviate slightly from the locations shown on the attached Figure 2, and Plates 1 and 2. The boring logs are included in Appendix A, along with a general description of the field operations.

d. Laboratory testing of selected samples to determine the engineering properties of the earth materials underlying the site. The results of laboratory testing are presented in Appendix B, and on the boring logs in Appendix A. Soil samples will be *discarded* 30 days after the date of this report, unless this office receives a specific request and fee to retain the samples for a longer period of time.

e. Determination of seismic parameters for potential on-site ground motion.

f. Engineering analysis of the data and information obtained from our field study, laboratory testing, and literature review.

g. Development of geotechnical recommendations for site preparation and grading, and geotechnical design criteria for building foundations, floor slabs, temporary excavations, pavement, retaining walls, drainage and infiltration.

h. Preparation of this report summarizing our findings, conclusions, and recommendations regarding the geotechnical aspects of the project site.

The scope of this geotechnical study did *not* include an assessment of potential environmental issues.

1.3 Site Description

The site of the proposed development consists of approximately 8 acres located at 5250 Ralston Street, in City of Ventura, County of Ventura, California. The site is irregular in shape, and is shown in detail on the attached Figure 2, *Existing Site Plan*, Plate 1, *Proposed Site Plan*, and Plate 2, *ALTA Survey Map*. The site measures a maximum of approximately 720 feet in the north-south direction, and 740 feet in the east-west direction, when measuring across the central portion of the site. At the time of our field exploration program, the eastern portion of the site was occupied by an existing single-story commercial building, and surrounding parking lots, driveways and landscape areas. The western portion of the site is a vacant, generally trapezoidal-shaped undeveloped area, between Glacier Avenue to the west and the existing building to the east, measuring approximately 135 feet wide at the north end, and 325 feet wide at the south end. The existing building was not in use at the time of our field exploration. The existing building floor level roughly matches the adjacent existing grade surrounding the northern portion of the building, and is dock-high (approximately 4 feet higher) relative to the existing grade surrounding the southern portion of the building.

The topography of the subject site and immediately surrounding area is generally roughly level to gently south to southwesterly sloping, with a few relatively small areas of more pronounced sloping in landscaped areas, based on the topographic data shown on the ALTA Survey provided by the client, dated November 8, 2005. Vegetation on the site consists of numerous trees and shrubs in landscaped areas surrounding the existing building, and within the existing parking areas, and only scattered weeds in the majority of the western, vacant portion of the site.

The surrounding development consists of Ralston Street to the north, Glacier Avenue to the west, Walker Street to the south, Saratoga Avenue to the east, and also an existing commercial property being utilized as a beverage distribution operation to the south and east of portions of the site.

1.4 Proposed Development

The proposed development will consist of the construction of a new single-story outpatient clinic building, and surrounding parking, driveway and landscape areas, as shown on the attached Plate 1, *Proposed Site Plan*. This plan was developed from a *Preliminary Grading Plan* provided by the client, dated November 9, 2018. The proposed building will be constructed utilizing steel framing, and a concrete floor slab on grade. Maximum dead plus live column loads are expected to be approximately 101 kips, and maximum gravity plus seismic column loads are expected to be approximately 153 kips.

Site grading is expected to consist of removal and recompaction of the upper site soils for support of the proposed structure, pavement and other miscellaneous site improvements, backfill of related new utilities and retaining walls, and modifications of the existing site topography of up to approximately 8 feet, to establish grade for the building pad, driveway, parking and landscape areas, and site drainage.

2. GEOLOGIC SETTING

2.1 Geology

Geologic conditions beneath the subject property have been interpreted and characterized based upon our review of published and unpublished references, and our subsurface exploration. Our interpretations involve projections of

data and assume that geologic conditions are reasonably constant between borings. Work should continue under the review of the Geotechnical Engineer to ensure that geologic conditions different from those described below are recognized and evaluated as soon as possible. Certain subsurface conditions such as groundwater levels and the consistency of near-surface soils will vary with the seasons.

The subject site is located within the Saticoy USGS 7.5-minute quadrangle. According to Dibblee (1992), as shown on the attached *Regional Geological Map*, Figure 3, the subject site is underlain by younger alluvial materials (Qa), which was confirmed during our site exploration.

2.2 Faulting

Southern California is a tectonically active region subject to hazards associated with earthquakes and faulting. Faults are classified as either active, potentially active, or inactive. Active faults are defined by the State of California as faults that have exhibited surface displacement within the last 11,000 years. Potentially active faults are defined by the State as those with a history of movement between 11,000 and 1.6 million years ago. Alquist-Priolo Earthquake Fault Zones are zones that have been established by the State that contain active faults, and projects that are located within these zones require that a fault investigation be performed to determine if active faulting affects the site.

The subject site is *not* located in an Alquist-Priolo Earthquake Fault Zone, and therefore a detailed subsurface fault investigation is not required. The nearest Alquist-Priolo Earthquake Fault Zone is located approximately 1.4 miles north of the subject site, as shown on the attached Figure 4, *Earthquake Fault Zones Map* (CDMG, 1978). The concealed (i.e. below existing ground surface) trace of the Oak Ridge fault is shown by Dibblee (1992) as being located approximately 215 feet southeast of the subject site (approximately 520 feet southeast of the proposed building area), as shown on the attached *Regional Geological Map*, Figure 3, and as also shown on the hazard map available on the County of Ventura *County View* website (2019). This fault is included in the USGS seismic database utilized to compute the seismic design parameters provided in a subsequent section of this report, and therefore the presence and potential activity of this fault have been accounted for in the seismic design parameters.

3. EARTH MATERIALS AND SUBSURFACE CONDITIONS

3.1 Artificial Fill (af)

Artificial fill was encountered in Borings B-4 through B-9 to depths between approximately 3 and 4 feet below existing site grade, and in Borings P-3 and P-4 to depths of 2.5 and 3 feet, respectively. The artificial fill consists of gray to brown to reddish brown silty sand, clayey sand, clay and clayey silt, which range from dry to moist and medium dense to stiff, and contained occasional small pieces of concrete

3.2 Alluvium (Qa)

Native, younger alluvial soils were encountered in all of the exploratory borings, beginning at the ground surface in some areas, and below the existing artificial in other areas, and extending to the maximum depth explored, 51.5 feet below the existing site grade. The native soils consist primarily of silt and clay in varying combinations, with generally lesser fractions of sand, and occasional interbedded sandy lenses. The alluvial soils range from light brown to brown to dark yellowish brown and dark gray brown, dry to slightly moist and loose to medium firm near the ground surface in some areas, becoming moist and then very moist to saturated and firm to very stiff with depth. More detailed descriptions of the earth materials encountered are provided on the enclosed boring logs.

3.3 Soil Parameters

3.3.1 Compaction

Compaction curves were developed in this study for representative samples of the upper site soils. The maximum dry density was 118.5 pcf, at an optimum moisture content of 13.0% for a sample obtained from Boring B-1 between the ground surface and a depth of 5 feet; the maximum dry density was 122.0 pcf, at an optimum moisture content of 12.5% for a sample obtained from Boring B-3 between the ground surface and a depth of 7 feet, and the maximum

dry density was 118.5 pcf, at an optimum moisture content of 13.5% for a sample obtained from Boring B-8 between the ground surface and a depth of 5 feet. The upper site soils should be removed and recompacted for support of the proposed structure, pavement and other miscellaneous site improvements, as discussed in subsequent sections of this report. The results of the compaction testing are presented in Appendix B of this report.

3.3.2 Compressibility

Consolidation tests were performed on representative undisturbed samples of the onsite soils, and a remolded sample intended to represent the future compacted fill. The consolidation test results showed only a slight tendency to hydroconsolidate, and a low to moderate overall level of compressibility on the undisturbed samples at 5 feet and below, and a negligible tendency to hydroconsolidate, and very low level of overall compressibility on the sample of the upper site soils remolded to 90% of the maximum dry density. The results of the consolidation testing are presented in Appendix B of this report.

3.3.3 Shear Strength

Direct shear testing was used to measure the peak and ultimate shear strength properties of representative samples of the onsite soils, both remolded and undisturbed, in terms of a cohesion value and a friction angle. The results of the direct shear testing are presented in Appendix B of this report.

3.3.4 Grain Size Analysis

Grain size testing was performed on representative samples of the earth materials underlying the site by using mechanical sieve analysis, and hydrometer testing. The results of the grain size testing are presented in Appendix B of this report.

3.3.5 Atterberg Limits

Atterberg limits testing was performed on representative samples of the clayey soils, in order to determine the plastic and liquid limits, and plasticity index. The results of the Atterberg limits testing are presented in Appendix B of this report.

3.3.6 Expansion Category

The potential of the soil to swell or expand increases with an increase in soil density, a decrease in initial moisture content, an increase in clay content, and an increase in the activity of the clay content. Expansive soils change in volume (shrink or swell) due to changes in the soil moisture content. The risk of soil expansion increases with an increase in expansion index.

The expansion indices of representative samples of the upper site soils were found to range from 78 to 86, which are within the upper end of the *medium* expansion category. The expansion index test results are shown on the boring logs in Appendix A of this report. Representative samples of the blended, recompacted soils should be obtained from the surface of the building pad after grading to confirm the expansion index.

3.3.7 Corrosivity

The risk of corrosion of construction materials relates to the potential for soil-induced chemical reaction. The rate of deterioration depends on soil resistivity, texture, acidity, and chemical concentration. Representative samples of the upper site soils were transported to an outside laboratory for corrosivity testing, and the results of these tests are attached in Appendix B, and summarized in the following table. Sulfate and chloride concentrations are expressed in mg/kg on a dry weight basis.

Boring Number	Depth (ft)	Description	pH	Chloride (mg/kg)	Sulfate (mg/kg)	Specific Conductance (umhos/cm)
B-1	0 – 5	SILTY CLAY TO CLAYEY SILT	7.8	19	33	330
B-3	0 – 7	SILTY CLAY TO CLAYEY SILT	8.0	15	360	730

The sulfate content is negligible (less than 1000 mg/kg) based on ACI 318, and therefore special considerations are not required for concrete which will be in contact with the onsite soils.

3.4 Groundwater

At the time of our field exploration, groundwater was encountered at depths between approximately 20 and 21 feet below the existing ground surface, although in one of the borings, excavated to a depth of 21.5 feet, groundwater was not encountered. Based on the *Depth to Historically High Groundwater Map*, Figure 5 (CGS, 2003), the historically highest groundwater level below the site was approximately 11 feet below the existing ground surface.

Groundwater elevations are dependent on seasonal precipitation, irrigation, land use, and climatic conditions, among other factors, and as a result fluctuate. Therefore, water levels at the time of construction and during the life of the development may vary from the observations or conditions at the time of our field exploration.

3.5 Field Percolation Testing and Infiltration

The field percolation testing program was performed in accordance with the specifications for the falling-head borehole infiltration test method as specified in section C.6 of Appendix C of the **Ventura County Technical Manual for Stormwater Quality Control Measures, Manual Update 2011** (Geosyntec, 2011). Our percolation testing program consisted of:

a. Drilling, sampling, and logging of four percolation test borings (Borings P-1 through P-4) in areas along the western and southern perimeters of the site where it has been proposed to install stormwater infiltration features. The percolation test borings were excavated to depths ranging from 5 feet to 16 feet below the existing ground surface with a truck mounted hollow stem auger drill rig at each of the locations for the purpose of evaluating the subsurface soil conditions, and to perform percolation testing. The locations of the percolation test borings are shown on the enclosed Figure 2, and Plates 1 and 2.

b. Performing field percolation testing as described below.

The falling head borehole infiltration test method used consisted of the excavation of the percolation test holes utilizing a truck mounted hollow stem auger drill rig with an 8-inch diameter auger. Two of the test holes (P-2 and P-4) were excavated to a depth of approximately 5 feet below existing grade, and then another test hole (P-1 and P-3) was excavated in close proximity to each of these holes a depth of approximately 16 feet below grade, in order to verify the earth material conditions and infiltration characteristics to a depth of 11 feet below the proposed infiltration features, as required. Upon completion of the excavation operations, each test hole was prepared by installing an appropriate length of 3-inch diameter perforated PVC pipe, surrounded by ¾-inch gravel.

The next day, the percolation test holes were pre-soaked by filling the holes with water within the percolation test zone, the bottom 12 inches of the hole. After maintaining water in the bottom 12 inches of the percolation test holes for a minimum period of 4-½ hours, the percolation testing was performed by filling each hole to a depth of 12 inches, and measuring the water drop after a period of 30 minutes, and then repeating this procedure. The testing was done until the highest and lowest rates of three consecutive readings were all within 10% of each other, at which point the testing was considered complete.

The final stabilized field percolation rates obtained through field testing are summarized in the following table, along with the corrected rates utilizing the correction factor provided in the County of Los Angeles Department of Public Works **Guidelines for Design, Investigation, and Reporting Low Impact Development Stormwater Infiltration** document GS200.1, dated December 31, 2014, as required by most Ventura County agencies. Shallow infiltration basins utilize infiltration through the bottom of the pit, and therefore the L.A. County correction factors are utilized to eliminate the component of infiltration which occurs through the sides of the percolation test borings, thereby more accurately modeling the infiltration rate at the bottom of typical infiltration basins. It is recommended that

the lowest corrected infiltration rate of 0.53 inches per hour obtained in Boring P-2 be utilized in the design for the entire site.

Percolation Test Data Summary

Boring #	Depth (ft)	Stabilized Field Absorption Rate (min/in)	Stabilized Field Absorption Rate (in/hr)	Correction Factor	Corrected Infiltration Rate (min/in)	Corrected Infiltration Rate (in/hr)
P-1	16.0	20.00	3.00	3.63	72.50	0.83
P-2	5.0	30.00	2.00	3.78	113.44	0.53
P-3	16.0	18.46	3.25	3.59	66.34	0.90
P-4	5.0	26.67	2.25	3.72	99.18	0.60

Upon completion of the field testing program, the PVC piping was pulled from each test hole, and the test holes were backfilled.

3.5.1 Infiltration Discussion

The intentional introduction of enormous amounts of water into the ground via the infiltration of onsite stormwater is a relatively new concept, and is inherently risky, regardless of any precautions which may be taken. Infiltration features should be located entirely within landscape areas, and should be as far as possible from the proposed structure, pavements, site walls, sidewalks and other improvements. Some potential settlement and cracking of any improvements located in the vicinity of infiltration features should be anticipated, due to shrinking and swelling of the expansive soils, and possible hydroconsolidation. There should also be careful coordination of the of the site utility locations with any proposed stormwater infiltration features. The proposed stormwater infiltration features should *not* be allowed to come into contact with, or to even be in close proximity to utility trench backfill, and utilities are not allowed to cross above, below or through any proposed infiltration features.

One of the restrictions related to onsite infiltration is that there must be a minimum of 5 feet of vertical separation between the depth of infiltration and either the current or historically highest groundwater level, whichever is higher. Therefore, since the historically highest groundwater level was approximately 11 feet below grade, the bottom of any proposed infiltration features should not exceed a depth of 6 feet below the natural site grade. Note that although the 'bottom' elevations of the infiltration areas shown on the attached Plate 1 indicate an elevation within approximately 1 foot of surrounding grade, these infiltration areas are typically filled with an infiltration medium ranging from a few to several feet thick, which includes some type of granular material (typically sand and/or small gravel) at the bottom, and some type of mulch, growing medium and/or vegetative layer at the top. The 'depth of infiltration' is considered to be at the bottom of this infiltration medium (i.e. at the top of the underlying native soil). Infiltration is not allowed into artificial fill.

4. SEISMICITY

4.1 Seismic Design Criteria

The California Building Code (CBC) is utilized in the seismic design of structures, and is based on the *Maximum Considered Earthquake Ground Motion*. The maximum considered earthquake spectral response accelerations are then adjusted for the general type of earth materials within approximately the upper 100 feet underlying the site, termed a *Site Class*, which would be D for the subject site. The *Site Class* is based on parameters such as shear wave velocity, standard penetration test resistance, undrained shear strength, and earth material type.

The site-specific seismic design criteria required by the CBC were determined utilizing the SEAOC/OSHPD (2019) Seismic Design Maps web app, utilizing both the ASCE 7-10 and ASCE 7-16 Standards, since it is unknown which of these design criteria may be required at the time the proposed development goes through the plan check process. The output from the Seismic Design Maps web app is included as an attachment in Appendix C of this report, for both the ASCE 7-10 and 7-16 Standards, and the primary design criteria are summarized in the table below.

ASCE Standard	F_a	F_v	PGA	PGA_M	S_S	S_1	S_{MS}	S_{M1}	S_{DS}	S_{D1}
7-10	1.0	1.5	0.998	0.998	2.587	1.006	2.587	1.508	1.725	1.006
7-16	1.5	Null*	0.865	0.952	1.964	0.737	1.964	Null*	1.309	Null*

*** See Section 11.4.8 of ASCE 7-16**

Conformance to these criteria does *not* constitute a guarantee or assurance that significant structural damage will *not* occur if a maximum level earthquake occurs. The primary goal of seismic design is to protect life and *not* to avoid all damage, since such design may be economically prohibitive.

4.2 Earthquake Effects

The intensity of ground shaking during an earthquake can result in a number of phenomena classified as ground failure, which include ground rupture due to faulting, landslides, liquefaction, and seismically induced dry settlement. Other seismic hazards include seiches, tsunamis and flooding due to dam failure during an earthquake. Descriptions of each of these phenomena and an assessment of each, as it may affect the subject site, are included in the following sections. The Seismic Hazards Mapping Act of 1990, which became effective in 1991, requires mitigation of seismic hazards to a level that does *not* cause collapse of a building intended for human occupancy, but it does *not* require mitigation to a level of no ground failure or structural damage.

4.2.1 Shallow Ground Rupture

Ground surface rupture occurs when movement along a fault is sufficient to cause a gap or rupture where the upper edge of the fault zone intersects the ground surface, and such ruptures rarely occur as single breaks or are confined to a narrow zone. More commonly, ground rupture associated with faulting is characterized by relatively short segments of ground rupture that occur over a broad area of the upper plate. In some cases, particularly in unconsolidated alluvial sediments, *secondary ground ruptures* can develop from a number of causes not necessarily related directly to surface rupture of the causative fault. The secondary ruptures may be caused by phenomena such as seismically-induced settlement, landslides, and liquefaction.

4.2.2 Earthquake-Induced Landsliding

Landslides are slope failures that occur where the horizontal seismic forces act to induce soil failure. Seismic Hazard Maps have been released by the California Geological Survey that delineate areas that have been subject to, or are potentially subject to landsliding or permanent ground displacement as a result of earthquake-induced ground shaking. The subject site is not located in a Seismic Hazard Zone for landslides, and the subject site and surrounding areas are relatively flat. Therefore, earthquake-induced landsliding is *not* considered to be a hazard at the subject site.

4.2.3 Tsunamis, Seiches and Dam Inundation

Tsunamis are large sea waves produced by submarine earthquakes or volcanic eruptions. Seiches are an oscillation of the surface of an inland body of water that varies in period from a few minutes to several hours. Seismic ground motions can induce such oscillations. Since the site is *not* located close to an inland body of water, and is at an elevation sufficiently above sea level to be outside the zone of a tsunami runup, the risk of these two hazards is *not* considered pertinent to the subject site.

The subject site lies at the edge a potential *Dam Inundation Area* as identified on Figure F-4A of the 2015 Ventura County Multi-Hazard Mitigation Plan (AECOM, 2015), which means that there is the possibility of flooding at the subject site in the event that there is failure of one of the dams upstream of the subject site during a major earthquake.

4.2.4 Evaluation of Liquefaction Potential

Liquefaction is a phenomenon in which soils below the groundwater level lose strength as a result of ground shaking due to earthquakes, and may result in phenomena such as ground settlement, lateral spreading, sand boils, and

ground fissuring. Only the eastern portion of the subject site, and the extreme southeast corner of the proposed building area is located in an area designated as potentially liquefiable on the *Seismic Hazard Zones Map* of the Saticoy Quadrangle, as shown on the attached Figure 6 (CGS, 2003), and as shown in more detail on the attached Figure 2, which was created from an image obtained from the County of Ventura *County View* website (Ventura County, 2019), where the yellow hachured area in the lower right (southeast) portion of this map indicates a potentially liquefiable area. Boring B-1 was excavated at the southeast corner of the proposed building area, within the potentially liquefiable area, in order to assess the potential for liquefaction affecting the proposed development.

The results of the liquefaction analysis are included in Appendix D, and the results of this analysis, along with other geologic information about the area, were then used to evaluate the potential for the occurrence of liquefaction, and the different liquefaction-induced phenomena mentioned herein. The geotechnical data obtained from the boring, and our laboratory test results, including standard penetration test data (SPT), percent fines, clay fraction and Atterberg limits results, were utilized in our evaluation of liquefaction hazard potential at the site. Younger alluvial soils consisting primarily of clay and silt, with lesser amounts of sand, were encountered from the ground surface to the total depth of exploration, 51.5 feet.

At the time of our field exploration, groundwater was encountered in Boring B-1 at a depth of 20 feet. Based on the enclosed Figure 5, *Depth to Historically High Groundwater Map* (CGS, 2003), the historically highest groundwater level at the site was approximately 11 feet below the existing ground surface. The liquefaction hazard analysis was therefore performed conservatively using the historically highest groundwater depth of 11 feet below the existing ground surface, as required.

The methods following the recommendations of the NCEER (Youd and Idriss, 1997; Youd et al, 2001) were used in the liquefaction analysis, supplemented by the recommendations of Bray and Sancio (2006), and Boulanger and Idriss (2006) in the analysis of fine grained soils (clays and silts). A design-level earthquake magnitude of 6.90, and a site acceleration of 0.998g (PGA_M) were utilized to perform the liquefaction evaluation.

Blow counts used for the liquefaction evaluation were based on the blow counts measured with the driven sampler, a modified California sampler. Blow counts using the modified California sampler were adjusted to equivalent blows of a standard penetration test (SPT) sampler using a standard multiplier of ⅔. The measured blow counts were also adjusted for borehole diameter, rod length, fines content, overburden pressure, and delivered energy (Youd and Idriss, 1997 and 2001) to correspond to a driving-energy level of 60%, to obtain the final corrected blow count, $N_{1|60}$.

The earth materials underlying the site from the historically highest groundwater level to the total depth explored are relatively clayey, and range from firm to very stiff , based on our observations, and on the equivalent SPT blow counts obtained during sampling operations. Representative samples of these materials were also determined to have plasticity indices greater than 7, and greater than 20% particle sizes smaller than 0.005 millimeters. Therefore, based on the preliminary screening criteria contained within SP117A (CGS, 2008), the earth materials underlying the site would be expected to behave like clays (Boulanger and Idriss, 2006), and would not be considered susceptible to liquefaction, or any of the related phenomena.

4.2.5 Dynamic Dry Settlement

Dynamic settlement of relatively loose, dry sandy soils above the groundwater level can be caused by strong earthquake ground shaking. A minimum of the upper 7 feet of earth materials underlying the proposed new structure will consist of newly placed compacted fill, as described in subsequent sections of this report, and this densified, recompacted material would not be considered prone to dynamic dry settlement. The earth materials from the bottom of this recompaction zone down to the historically highest and current groundwater level are clayey in nature, and would also not be considered prone to dynamic dry settlement. Therefore, dynamic dry settlement would not be considered to be a potential issue at the subject site.

5. CONCLUSIONS AND RECOMMENDATIONS

5.1 Conclusions and Design Requirements

Based on the findings of our data review, subsurface exploration, laboratory testing, field testing, and engineering analysis, and within the scope of this study, the proposed development is considered *feasible* from a geotechnical engineering viewpoint, provided the recommendations in this report are incorporated into the project plans and implemented during construction. The following is a general discussion of the main geotechnical issues affecting the site, and recommended mitigation measures. More detailed recommendations are provided in subsequent sections of this report.

Due to the inconsistent nature of the upper site soils, and in order to provide more uniform foundation support due to the significant elevation difference across the site, it is recommended that the upper site soils be over-excavated and recompacted to a minimum depth of 5 feet below the existing site grade, and a minimum of 5 feet below the bottom of foundations, whichever is deeper, to provide a relatively uniform blanket of newly placed compacted fill for support of the proposed structure. The existing ground surface elevation within the proposed building area varies by approximately 8 feet, and therefore the requirement to over-excavate a minimum of 5 feet below existing grade will control in some areas, and the requirement to over-excavate 5 feet below the bottom of future foundations will control in other areas. The ratio of the maximum to minimum depths of newly placed compacted fill below bottom of footings should not exceed approximately 2.5:1.

After removal and recompaction of the upper site soils, the proposed structure may then be constructed on a system of conventional, shallow spread footings bearing exclusively in newly placed, certified compacted fill. It is also recommended that all existing fill, and any loose or disturbed upper site soils be over-excavated and recompacted for support of pavements and other miscellaneous exterior surface improvements, and that a minimum of 12 inches of newly placed compacted fill be provided for support of pavement and all other exterior improvements.

5.1.1 Faults / Seismicity

Although no known active surface fault traces traverse through the subject site, like most of Southern California, the site lies within a seismically active area. Earthquake resistant structural design is recommended, and is required by the building code. Designing structures to be earthquake-proof is generally considered to be impractical, due to both design and cost limitations, and significant damage to structures may be unavoidable during large earthquakes.

Structural design based on the California Building Code structural analysis procedures requires the use of the seismic parameters given previously in the *Seismic Design Criteria* section of this report. These minimum code values are intended to protect life and may not provide an acceptable level of protection against significant cosmetic damage and serious economic loss. Significantly higher than code parameters may be necessary to further reduce potential economic loss during a major seismic event. Structural Engineers, however, often regard higher than code values or procedures as impractical for use in structural design. The Structural Engineer and project Owner must decide if the level of risk associated with code values is acceptable and, if not, to assign appropriate seismic values above code values for use in structural design.

5.1.2 Hazardous Materials

AGS has *not* been retained to provide any type of environmental assessment of the subject property, *nor* to provide recommendations with respect to any contamination that might be present.

5.1.3 Site Grade Adjustments

Grading for the proposed structure and other improvements will consist of removal and recompaction of the upper site soils, and the placement of up to approximately 8 feet of fill above the current existing site grade to achieve the future building pad and surrounding finished grades, and to provide proper site drainage.

5.1.4 Excavation Characteristics

Difficult excavation of the onsite earth materials in terms of extremely hard or cemented materials is not anticipated.

5.1.5 Shrinkage

Shrinkage results when the soil being placed as fill is compacted to a dry density greater than the in-place source materials. Based on experience, it is estimated that an average shrinkage factor of about 10% to 15% will result from recompaction of the upper onsite soils. This estimate is based on an average relative compaction of 92% for recompacted materials and average densities of the undisturbed ring samples. The above shrinkage figures do not account for the effects of volume losses due to clearing, grubbing and stripping operations, or uncertainty in the density of the in-place materials. If the actual average degree of compaction differs from that used to estimate shrinkage, the actual shrinkage may also differ. Variations in the estimated shrinkage factors should be anticipated and provisions for such variations should be included in the project specifications.

5.1.6 Drainage

All surface runoff must be carefully controlled and must remain a crucial element of site maintenance. Proper drainage and irrigation are important to reduce the potential for excessive infiltration adjacent to foundations. Final grading should provide positive drainage away from footings. All pad drainage shall be collected and diverted away from proposed buildings and foundations in non-erosive devices. Gutters and roof drains should be provided, properly maintained, and discharge directly into glue-joined, watertight subsurface piping. All drainage piping should be watertight and discharge to an appropriate location, as determined by the project Civil Engineer.

All underground plumbing fixtures should be absolutely leak-free. As part of the maintenance program, utility lines should be checked for leaks for early detection of water infiltrating the soils that could cause detrimental soil movements. Detected leaks should be promptly repaired. Proper drainage shall also be provided away from the building footings during construction. This is especially important when construction takes place during the rainy season.

Seepage of surface irrigation water or the spread of extensive root systems into the subgrade of footings, slabs, concrete flatwork or pavements can cause differential movements and consequent distress in these structural elements. Trees and large shrubbery should *not* be planted so that roots grow under foundations and flatwork when they reach maturity. Landscaping and watering schedules should be planned with consideration for these potential problems.

Drainage systems should be well maintained, and care should be taken to *not over* or *under* irrigate the site. Landscape watering should be held to a minimum while maintaining a uniformly moist condition without allowing the soil to dry out. During extreme hot and dry periods, adequate watering may be necessary to keep soil from separating or pulling back from the foundations. Cracks in paved surfaces should be sealed to limit infiltration of surface waters.

5.1.7 Plan Review

When final Grading Plans become available, they should be reviewed by AGS *prior* to submittal to regulatory agencies for approval. Additional analysis *may* be required at that time depending on specific details of the proposed grading and improvements. Approval by this office will be indicated on the plans by *manual* signature and stamp.

Please be aware that the contract fee for our services to prepare this report does not include additional work that may be required, such as grading observation and testing, footing observations, plan review, or responses to governmental (regulatory) plan reviews associated with you obtaining a building permit. Where additional services are requested or required, you will be billed on an hourly basis for consultation or analysis. AGS requests a minimum of 24 hours be provided for plan reviews. Please anticipate additional time for plan corrections if all of our geotechnical recommendations have not been added to the plans, prior to our approving and stamping the plans.

5.1.8 Additional Recommendations

The following additional geotechnical recommendations should be incorporated into the final design and construction plans. All such work and design should be in conformance with applicable governmental regulations or the recommendations contained herein, whichever are more restrictive. The following recommendations have *not* been reviewed or approved by any governing agency at this time. These recommendations may change based on obtaining approval from the City. Design of the proposed project should be made following approval from the City.

5.2 Site Preparation

The area of the proposed new structure should be prepared so that foundations are founded entirely within newly placed compacted fill, with a ratio of thicknesses of compacted fill below bottom of footings across the building not to exceed approximately 2.5:1. General guidelines are presented below to provide a basis for quality control during site grading. It is recommended that all compacted fills be placed and compacted with engineering control under continuous observation and testing by the Geotechnical Engineer and/or their field representative, and in accordance with the following requirements.

5.2.1 Removals

a. When demolishing any existing improvements or subsurface structures in the vicinity of the proposed new structure, the contractor should locate all existing foundations, floor slabs, debris pits, artificial fill, and subsurface trash which may be present. These soils and structures should be completely removed. The resulting excavations should be cleaned of all loose or organic material. In areas to receive fill or to support the proposed structure, deeper removals will be required, as discussed below.

b. Remove all vegetation and loose soil *prior* to fill placement. The general depth of stripping should be sufficiently deep to remove any root systems or organic topsoil which may be present. A careful search shall be made for subsurface trash, abandoned masonry, abandoned tanks and septic systems, and other debris during grading. All such materials, which are *not* acceptable fill material, shall be removed *prior* to fill placement. The removal of trees and large shrubs should include complete removal of their root structures, where applicable.

c. The proposed building area should be over-excavated to a minimum depth of 5 feet below the existing site grade, or a minimum of 5 feet below the bottom of the proposed new foundations, whichever is deeper. Note that in some areas, such as the west and southern portions of the proposed building area, where the existing grade will need to be raised up to an estimated 8 feet to achieve the future finished pad grade, the requirement to over-excavate a minimum of 5 feet below existing grade will control. In other areas, such as the northeast portion of the proposed building area, where the existing grade is at or near the proposed future building pad finished grade, the requirement to over-excavate a minimum of 5 feet below the bottom of proposed new footings will control (these general estimates are based on limited existing and future grade data shown on the *ALTA Survey* and *Preliminary Grading Plan* provided). In addition, all foundations should be underlain by a relatively uniform blanket of compacted fill, where the ratio of the maximum to minimum depths of newly placed compacted fill below bottom of footings does not exceed approximately 2.5:1.

d. The limits of over-excavation should extend laterally to a minimum distance of 7 feet beyond the outside perimeter of all foundations, or for a distance equal to the depth of fill below the bottom of footings, whichever is greater.

e. In areas to receive new exterior hardscape (i.e. concrete or asphalt paving, walkways, patios, pavers, etc.) or other miscellaneous improvements, all existing fill materials and any other loose or disturbed soil should be removed and recompacted. The depth of over-excavation in these

areas should be a minimum of either 12 inches below existing grade, or 12 inches below the bottom of any improvements, or supporting aggregate base section, whichever is deeper.

f. A careful search shall be made for any deeper areas of existing fill or loose soil during grading operations. If encountered, these loose areas should be properly removed to the firm underlying soil and properly backfilled and compacted as directed by a field representative of the Project Geotechnical Engineer.

5.2.2 Bottom Stabilization

a. Depending on the time of year and recent precipitation levels, stabilization of the removal bottom may be required prior to the placement of compacted fill. Acceptable stabilization methods include using (1) float rock worked into the soft soils; (2) geofabric, such as Mirafi Fabric 600X, with a 24-inch wide overlap, or (3) a combination of these methods. Some compaction effort should be used when working thin lifts of float rock into the excavation bottom. A 12- to 24-inch thick layer of float rock may be required to adequately bridge an unstable bottom, although this process is always a trial and error procedure, and is dependent upon many factors, including the type and size of equipment utilized, and the time of year and weather conditions. Smaller track-mounted equipment would be preferable to wheel-mounted equipment. The float rock zone should not be included in the required thickness of fill beneath footings. The use of scrapers to perform the over-excavation will likely not be possible, due to the presence of the relatively high moisture content soils. The use of large excavators staged outside the excavation will likely be required to achieve the required over-excavation.

5.2.3 Suitable Fill Material

a. The excavated site soils, cleaned of deleterious material, may be utilized for fill, however, significant moisture conditioning (drying and/or the addition of water) and thorough mixing will likely be required in most or all areas, depending on the time of year and recent precipitation. Rock larger than 6 inches should *not* be buried or placed in compacted fill. Rock fragments less than 6 inches may be used provided the fragments are *not* placed in concentrated pockets, and a sufficient percentage of finer grained material surrounds and infiltrates the rock voids.

b. Imported material should generally have engineering properties similar to, or more favorable than those on the subject site, including an expansion index less than 70, and an R-value greater than 10. Imported material will require testing to verify the engineering properties, and must be approved by the Geotechnical Engineer *prior* to placement on the site.

5.2.4 Placement of Compacted Fill

a. All fill materials should be placed in controlled, horizontal layers *not* exceeding 6 to 8 inches thick, and moisture conditioned to at least 2%, but no more than approximately 5% above the optimum moisture content. Fill materials should be compacted to a minimum 90% of the laboratory maximum dry density, as determined by ASTM D1557. If the relative compaction does not meet this criteria, the contractor should rework the fill until it does meet the criteria. If the fill materials pump (flex) under the weight of construction equipment, difficulties in obtaining the required minimum compaction may be experienced. Therefore, if soil pumping occurs, it may be necessary to control the moisture content to a closer tolerance (e.g., 2 to 3% above optimum), use construction equipment that is not as prone to cause pumping, and/or a stabilizing layer of float rock, gravel, geogrid, or a combination of these, as described above.

b. The field test methods to be used to determine the in-place dry density of the compacted fill shall be in conformance with either ASTM D1556 (sand cone test method) or ASTM D2922 (nuclear gauge method).

c. Subgrade for the support of exterior asphalt or concrete pavement shall be moisture conditioned as required, and be recompacted to at least 95% of the maximum dry density to a depth of at least 12 inches.

5.2.5 Testing of Compacted Fill

a. At least one compaction test shall be performed for every 500 yd^3 of the fill material. In addition, at least one test shall be performed for every 2 feet of fill thickness.

5.2.6 Inclement Weather and Construction Delays

a. If construction delays or the weather result in the surface of the fill drying, the surface should be scarified and moisture conditioned before the next layer of fill is added. Each new layer of fill should be placed on a rough surface so planes of weakness are not created in the fill.

b. During periods of wet weather and before stopping work, all loose material shall be spread and compacted, surfaces shall be sloped to drain to areas where water can be removed, and erosion protection or drainage provisions shall be made in accordance with the plans provided by the Civil Engineer. After the rainy period, the Geotechnical Engineer and or their field representative shall *review* the site for authorization to resume grading and to provide any specific recommendations that may be required. As a minimum, however, surface materials previously compacted before the wet weather shall be scarified, brought to the proper moisture content, and recompacted *prior* to placing additional fill.

c. During foundation construction, including any concrete flatwork, construction sequences should be scheduled to reduce the time interval between subgrade preparation and concrete placement to avoid drying and cracking of the subgrade or the surface should be covered or periodically wetted to prevent drying and cracking.

5.2.7 Responsibilities

a. Representative samples of material to be used as compacted fill should be analyzed in the laboratory by the Geotechnical Engineer to determine the physical properties of the materials. If any materials other than those previously tested are encountered during grading, the appropriate analysis of this material shall be conducted by the Geotechnical Engineer as soon as practicable. Any imported soil from off-site sources shall be approved *prior* to placement.

b. All grading work shall be observed and tested by the Project Geotechnical Engineer or their field representative to confirm proper site preparation, excavation, scarification, compaction of on-site soil, selection of satisfactory fill materials, and placement and compaction of fill. All removal areas and footing excavations shall be observed by the field representative of the Project Geotechnical Engineer before any fill or steel is placed.

c. The lateral limits and the depths of the required over-excavation should be shown by the Civil Engineer on the grading plans.

d. The grading contractor has the ultimate responsibility to achieve uniform compaction in accordance with the geotechnical report and grading specifications.

5.3 Utility Trench Backfill

The on-site soils are suitable for backfill of utility trenches from 1-foot above the top of the pipe to the surface, provided the material is free of organic matter and deleterious substances. The natural soils should provide a firm foundation for site utilities, but any soft or unstable material encountered at pipe invert should be removed and replaced with an adequate bedding material.

The site Civil Engineer in accordance with manufacturer's requirements should specify the type of bedding materials. Granular soils will need to be imported for bedding or shading of utilities. Jetting of bedding materials should *not* be permitted unless appropriate drainage is provided and the bedding has a sand equivalent greater than 50.

Trench backfill should be placed in 6 to 8-inch lifts, moisture conditioned to at least 2% above the optimum moisture content, but not more than approximately 5% above optimum, and compacted to at least 90% of the maximum density as determined by ASTM D1557, with the exception of the 1 foot below subgrade in any areas subject to vehicular traffic, which should be compacted to a minimum of 95% of the maximum dry density. Jetting of trench backfill is *not* acceptable to compact the backfill.

In areas where utility trenches pass through an existing pavement section, the trench width at the surface shall be enlarged a minimum of 6 inches on each side to provide bearing on undisturbed material for the new base and paving section to match the existing section.

Major underground utilities shall *not* cross beneath buildings unless specifically approved by the Project Civil Engineer and respective utility company. If approved, trenches crossing building areas shall be backfilled with a select gravelly sand compacted to 95% relative compaction.

5.4 Temporary Excavations

It is anticipated that temporary excavations made as part of the required removal and recompaction operations may be made to a maximum vertical height of approximately 5 feet, with any portion over 5 feet in height sloped back at a uniform 1:1 gradient. Excavations should *not* be allowed to become soaked with water or to dry out. Surcharge loads should *not* be permitted within a horizontal distance equal to the height of the excavation from the top of the excavation, unless the excavation is properly shored. Excavations that might extend below an imaginary plane inclined at 45 degrees below the edge of an existing foundation should be properly shored to maintain foundation support for the existing structure. All excavations should be observed by a representative of the Geotechnical Engineer soon after excavation, to confirm the anticipated soil conditions, and recommend modifications, if necessary.

5.5 Foundation Design

After removal and recompaction of the upper site soils as previously discussed in this report, conventional, shallow footings founded exclusively in newly placed, certified compacted fill can be utilized for foundation support for the proposed new structure. The following design parameters may be used in the design of conventional, shallow footings.

5.5.1 Minimum Footing Dimensions

Minimum required foundation depths and widths are provided below. The embedment depths are into the recommended bearing material (compacted fill), or below the lowest adjacent, final grade, whichever is deeper. Where located adjacent to utility trenches, footings shall extend below a 1:1 plane projected upward from the inside bottom of the trench.

Minimum Footing Embedment Depth (inches)	Minimum Wall Footing Width (inches)	Minimum Isolated or Pad Footing Width (inches)
24	12	24

5.5.2 Allowable Bearing Pressure and Lateral Resistance

Allowable net vertical soil bearing pressure, including dead and live loads, are given below for footings founded in the recommended bearing material, at the minimum required width and embedment depth. The bearing capacity can be increased by ⅓ when considering short duration wind or seismic loads.

Allowable Bearing Pressure (psf)	Allowable Bearing Increases – for Each Additional Foot of Width (psf)	Maximum Allowable Bearing Pressure (psf)
2500	100	3000

Resistance to lateral loads can be assumed to be provided by friction acting along the base of the foundation, and by passive earth pressure on the side of the footing, utilizing the values provided below. The allowable friction coefficient may be used with the vertical dead loads, and the passive pressure can be utilized for the sides of footings poured against the compacted fill. These allowable passive and friction values can be increased by a factor of 1.5 to convert from allowable to ultimate values. When combining passive and friction for lateral resistance, the passive component should be reduced by one-third.

Allowable Sliding Friction Coefficient	Allowable Passive Resistance (psf/ft)	Maximum Passive Resistance (psf)
0.3	250	2500

5.5.3 Foundation Settlement

Static settlement of proposed foundations due to dead and frequently applied live loads is not expected to exceed approximately ½-inch under the assumed loading conditions, and is expected to occur primarily upon initial application of loading. Differential settlement is not expected to exceed approximately ¼-inch.

5.5.4 Steel Reinforcement

All foundations should be reinforced with a minimum of four #4 steel bars. Two of these should be placed near the top of the foundation, and two should be placed near the bottom. Final structural details of the footings, such as footing thickness, concrete strength, and amount of reinforcement, should be established by the project Structural Engineer, but should comply with the above minimums. The upper site soils were determined to have an expansion index category of *medium.* Expansion index tests should be performed at the completion of grading to confirm the expansion index at subgrade level of the finished building pad.

5.5.5 Required Observations

Prior to placing concrete in the footing excavations, an observation should be made by a field representative of the Project Geotechnical Engineer to confirm that the footing excavations are free of loose and disturbed soils and are embedded in the recommended bearing materials. The proper as-graded moisture conditions should be maintained in the earth materials exposed in the foundation excavations prior to pouring concrete, or the excavations should be pre-moistened as required, prior to pouring concrete.

5.6 Floor Slab Design

All floor slabs should be supported on a 4-inch layer of sand, overlying newly placed compacted fill, with a vapor barrier placed where necessary, as described in a subsequent section of this report. If earthwork operations are conducted such that the construction sequence is not continuous, or if construction operations disturb the surface soils, it is recommended that the exposed subgrade to support concrete slabs be tested within a day of the concrete pour or placement of sand and vapor barrier to verify adequate compaction *and moisture conditions*. If adequate compaction *and moisture conditions* are not demonstrated, the disturbed subgrade should be over-excavated, scarified, and recompacted in accordance with the guidelines provided in the *Site Preparation* section of this report *prior* to the slab being poured, or placement of the sand and vapor barrier.

5.6.1 Reinforcing

Concrete slabs should be a minimum of 5 inches thick, and be reinforced with a minimum of #4 steel bars placed on 18-inch centers each way. The final structural details, such as (1) slab thickness, (2) concrete strength, (3) type, amount, and placement of reinforcing, and (4) joint spacing, should be established by the project Structural Engineer, but should comply with the above minimums. The upper site soils were found to have an expansion index at the upper end of the *medium* range.

Cracking of concrete floor slabs and exterior flatwork is relatively common, and difficult to completely prevent. Steel reinforcement and crack control joints are intended to reduce the risk of concrete slab cracking, as are the use of fiber reinforced concrete and proper concrete curing. In addition, concrete slabs are generally not perfectly level, but they should be within tolerances included in the project specifications.

Tile flooring can crack, reflecting cracks in the underlying concrete slab. Therefore, if tile flooring is used, the slab designer should consider additional steel reinforcement, above minimum requirements, in the design of concrete slab-on-grade where tile will be installed. Furthermore, the tile installer should consider installation methods, such as using a vinyl crack isolation membrane between the tile and concrete slab, to reduce the potential for tile cracking.

5.6.2 Vapor Barrier

It is recommended that a minimum 15-mil thick plastic vapor barrier be used under floor slabs in moisture sensitive areas. The vapor barrier should be installed in accordance with the recommendations contained in the latest version of ASTM E1643. In accordance with our understanding of the latest standard of practice, it is recommended that the concrete slab be poured directly on top of the vapor barrier. It is recommended that no sand be placed between the vapor barrier and concrete slab, however, due to the expansive nature of the onsite soils, it is recommended that 4 inches of sand be placed on top of the finished subgrade, and below the vapor barrier. Seams of the vapor barrier should be overlapped and sealed. Where pipes extend through the vapor barrier, the barrier should be sealed to the pipes. Tears or punctures in the vapor barrier should be completely repaired *prior* to placement of concrete. The concrete mix should be designed so as to minimize possible curling of the slab. The concrete slab should be allowed to cure properly before placing vinyl or other moisture-sensitive floor coverings.

5.7 Asphalt and Concrete Pavement Design

5.7.1 Grading

All exterior areas to be paved with asphalt or concrete should be graded in accordance with the general recommendations for site grading as described in the *Site Preparation* section of this report. In proposed parking and driveway areas, and any other exterior flatwork areas (walkways, patios, etc.), all existing fill material and loose or disturbed soils should be removed and recompacted. The depth of over-excavation should extend to a minimum of 12 inches below either existing grade, or the bottom of future concrete, asphalt or aggregate base section, whichever is deeper. If test results show that proper moisture and compaction requirements do not exist just *prior* to placing base or placing pavements, the surface should be scarified, moisture conditioned, and properly recompacted. The subgrade should be proof-rolled to check for soft spots.

Compaction tests will be required for all asphalt and aggregate base. A minimum relative compaction of 95% is required for all asphalt, aggregate base, and upper 12 inches of subgrade soils. The aggregate base should have a minimum *R*-value of 78 and meet recognized industry specifications for aggregate base. Base materials should be placed and compacted in lifts not exceeding 6 inches. Asphalt should *not* be placed if the base is pumping. Base materials are *not* required beneath curbs and gutters, however, if base materials are not utilized beneath the curbs and gutters, it is recommended that the subgrade soils be recompacted to at least 95% relative compaction to a minimum depth of 12 inches below the bottom of curbs and gutters.

5.7.2 Maintenance

Pavement section design assumes that proper maintenance practices, such as sealing and repair of localized areas of distress, are employed throughout the design life of the pavement.

5.7.3 Asphalt Pavement Design

Pavement section calculations were performed for asphalt pavement design for a range in traffic indices. Selection of the appropriate traffic index to use should be made by the Project Civil Engineer based on their knowledge of traffic flow and loadings.

The structural sections for asphalt pavement were computed in general accordance with the Caltrans method (**California Department of Transportation Highway Design Manual**). The results of the analyses, using an *R*-value of 4, as determined from testing of a representative sample of the upper onsite soils (results attached in Appendix B), are summarized in the following table:

Traffic Index	Thickness, Inches	
	Asphalt	Aggregate Base
5.0	4.5	7.0
6.0	5.0	10.0
7.0	5.0	14.0

5.7.4 Confirmation of R-Value

It is recommended that additional testing to determine the *R*-value of the subgrade soils in parking and driveway areas be performed near the completion of parking lot grading, in order to confirm the pavement structural section. It should be noted that the pavement structural section design recommendations presented in this report may change if a different *R*-value is obtained for the actual subgrade soils.

5.7.5 Concrete Pavement Design

It is recommended that all exterior concrete pavement subject to regular vehicular traffic be a minimum of 6 inches thick, and concrete pavement subject to truck traffic be a minimum of 8 inches thick. All concrete pavement subject to traffic loads should be underlain by a minimum of 6 inches of aggregate base. Concrete flatwork subject only to pedestrian traffic (i.e. walkways, patios, etc.) should be a minimum of 5 inches thick, and need not be underlain by base. All exterior concrete should be reinforced with a minimum of #4 steel bars placed on 18-inch centers each way.

5.8 Retaining Wall Design Criteria

The following retaining wall design information is provided for use in the design of loading dock retaining walls, and any other miscellaneous small retaining walls which may be incorporated into the project. It is anticipated that any proposed retaining walls will retain less than 6 feet of earth materials, and therefore seismic lateral forces need not be incorporated into the design. Retaining wall foundations may be designed utilizing the criteria provided in the *Foundation Design* section of this report.

5.8.1 Lateral Earth Pressures

The earth pressure behind retaining walls depends on the allowable wall movement, type of backfill materials, backfill slopes, surcharges, and hydrostatic pressures if any. The following equivalent fluid pressures are recommended for vertical walls with no hydrostatic pressure, no surcharge, no seismic effects, and a backfill slope with a gradient less (flatter) than 5(H):1(V). Seismic lateral forces would be in addition to the static wall pressures provided below, and would be required for walls retaining more than 6 feet, which is not currently anticipated.

	Equivalent Fluid Unit Weight, pcf			
Wall Movement	Clean Sand or Gravel Backfill (GW, GP, SW, SP)	Silty Sand or Silty Gravel Backfill (SM, GM)	Clayey Sand, Clayey Gravel Backfill (SC, GC)	Silts, Clays (ML, CL)
FREE TO DEFLECT	30	40	45	55
RESTRAINED	40	50	60	80

In areas where the retaining walls retain sloping ground steeper than 5(H):1(V), the equivalent unit weights in the above table should be increased by 13 pcf for gradients up to 2(H):1(V).

These values are applicable for backfill placed between the wall stem and an imaginary plane rising at a 45-degree angle from below the edge (heel) of the wall footing. If the onsite soil is used as backfill within this zone, the equivalent fluid unit weight associated with a soil classification of CL should be used.

The surcharging effect of anticipated adjacent loads on the wall backfill due to traffic, footings, or other loads, should be included in the wall design. The magnitude of lateral load due to surcharging depends on the magnitude of the surcharge, the size of the surcharge-loaded area, the distance of the surcharge from the wall, and the restraint of the wall. We can provide assistance in evaluating the effects of surcharge loading, if desired, once details are known and provided.

5.8.2 Backfill and Drainage

Except for the upper 18 to 24 inches, the soil immediately adjacent to backfilled retaining walls should be free-draining filter material (such as Caltrans Class 2 permeable material), or gravel wrapped in filter fabric, within a minimum horizontal distance of 1-foot from the back face of the wall. As an alternative to either one of these, a drainage tile product such as Miradrain may be applied to the back face of wall, over the waterproofing. Weep holes and/or a subdrain pipe, as appropriate, should be installed at the base of retaining walls. Subdrain pipe should consist of a minimum 4-inch diameter perforated PVC pipe meeting ASTM D2729 or better, surrounded by a minimum of 1 cubic foot of gravel per lineal foot of pipe, and the entire pipe and gravel system should be wrapped in filter fabric, such as Mirafi 140N. Accordion or similar type pipe is *not* acceptable for subdrain pipe. The top 18 to 24 inches should be backfilled with less permeable compacted fill to reduce infiltration.

During grading and backfilling operations adjacent to any wall, heavy equipment should not be allowed to operate within 5 feet laterally of the wall or within a lateral distance equal to the wall height, whichever is greater, to avoid developing excessive lateral pressures. Within this zone, only hand-operated equipment should be used to compact the backfill soils.

6. OBSERVATIONS AND TESTING

Prior to the start of site preparation and/or construction, *it is recommended that a meeting be held with the Contractor to discuss the project.* We recommend that AGS be retained to perform the following tasks prior to, and/or during construction. Please *advise* AGS a minimum 24 hours *prior* to any required site visit. *All approved plans, permits, and geotechnical reports must be at the jobsite and be made available during inspections.*

a. *Review* grading, foundation, and drainage plans to *verify* that the recommendations contained in this report have been properly interpreted and are incorporated into the project specifications. *If we are not accorded the opportunity to review these documents, we can take no responsibility for misinterpretation of our conclusions and recommendations.*

b. *Observe and advise* during all grading activities, including site preparation and placement of fill, and all foundation excavations, to *confirm* that suitable fill soils are placed upon competent material, and to allow design changes if subsurface conditions differ from those anticipated, *prior* to the start of construction.

c. *Observe* the installation of all drainage devices.

d. *Test* all fill placed for engineering purposes to *confirm* that suitable fill materials are used and properly compacted.

7. LIMITS AND LIABILITY

All building sites are subject to elements of risk that cannot be wholly identified and/or entirely eliminated. Building sites are subject to many detrimental geotechnical hazards, including but *not* limited to the effects of water infiltration, erosion, concentrated drainage, total settlement, differential settlement, expansive soil movement, seismic shaking, fault rupture, landsliding, and slope creep. The risks from these hazards can be reduced by employing subsurface exploration, laboratory testing, analyses, and experienced geotechnical judgment. Many geotechnical hazards, however, are highly dependent on the property owner properly maintaining the site, drainage facilities, and slope and by correcting any deficiencies found during occupancy of the property in a timely manner. Even with a thorough subsurface exploration and testing program, significant variability between test locations and between sample intervals may exist. Ultimately, geotechnical recommendations are based on the experience and judgment of the geotechnical professionals in evaluating the available data from site observations, subsurface exploration, and laboratory tests. Latent defects can be concealed by earth materials, deposition, geologic history, and existing improvements. If such defects are present, they are beyond the evaluation of the geotechnical professionals. No warranty, expressed or implied, is made or intended in connection with this report, by furnishing of this report, or by any other oral or written statement. Owners and developers are responsible for retaining appropriate design professionals and qualified contractors in developing their property and for properly maintaining the property. Retaining the services of a geotechnical consultant should *not* be construed to relieve the Owner, Developer, or Contractors of their responsibilities or liabilities.

The analysis and recommendations submitted in this report are based in part on our subsurface exploration, laboratory testing, site observations, and provided data on geology and the proposed site development. Our descriptions and the boring logs may show distinctions between fill and native soils, between native (e.g., alluvium, colluvium, slopewash) and bedrock formation, and between soil type (e.g., sands and silty sands). Such distinctions were based on geologic information, grading plans when available, intermittent recovered soil/bedrock samples, and judgment. Delineations between these categories of materials may not be perfect and may be subject to change as more information becomes available. For example, judgments may be clouded when recovered samples are intermittent and small in comparison to the volume of soil under study, and macrostructure that would aid the identification process are not as apparent as they would be when the borehole is geologically downhole logged by entering the excavation. When the age of the fill is old, the difference between the structure of the fill and native materials may be less pronounced, or the degree of bedrock formation weathering sometimes makes it difficult to distinguish between overlying alluvium, colluvium, or slopewash and weathered bedrock formational material. In general, our recommendations are based more on the properties of the materials than on the category of the material type such as fill, alluvium, colluvium, slopewash, or bedrock formation. Furthermore, the actual stratigraphy may be more variable than shown on the logs.

Although this report may comment on or discuss construction techniques or procedures for the design engineer's guidance, this report should *not* be interpreted to prescribe or dictate construction procedures or to relieve the contractor in any way of their responsibility for the construction.

Please be aware that the contract fee for our services to prepare this report does not include additional work that may be required, such as grading observation and testing, footing observations, plan review, or responses to governmental (regulatory) plan reviews associated with you obtaining a building permit. Where additional services are requested or required, you will be billed for any equipment costs and on an hourly basis for consultation or analysis.

The Geotechnical Engineer's actual scope of work during construction is very limited and does *not* assume the day-to-day physical direction of the work, minute examination of the elements, or responsibility for the safety of the contractor's workers. Our scope of services during construction consists of taking soil tests and making visual observations, sometimes on only an intermittent basis, relating to earthwork or foundation excavations for the project. We do *not* guarantee the contractor's performance, but rather look for general conformance to the intent of the plans and geotechnical report. Any discrepancy noted by us regarding earthwork or foundations will be referred to the Owner, project Engineer, Architect, or Contractor for action.

This report is issued with the understanding that it is the responsibility of the Owner, or of their representative, to ensure that the information and recommendations contained herein are called to the attention of the Architect and Engineers for the project and incorporated into the plan and that the necessary steps are taken to see that the Contractor carry out such recommendations in the field. Advanced Geotechnical Services, Inc., (AGS) has prepared this report for the exclusive use of the Client and authorized agents, and this report should *not* be considered transferable. We do recommend, however, that the report be given to future property Owners for the sole purpose of disclosing the report findings.

Findings of this report are valid as of the date of issuance. Changes in conditions of a property may occur with the passage of time whether attributable to natural processes or works of man on this or adjacent properties. Furthermore, changes in applicable or appropriate standards occur due, for example, to legislation and broadening of knowledge. Accordingly, findings of this report may be invalidated wholly or partially by changes outside our control. Therefore, *this report is subject to our review and remains valid for a maximum period of one year*, unless we issue a written opinion of its continued applicability thereafter.

In the event of any changes in the nature and design of the proposed improvements, the conclusions and recommendations contained in this report shall *not* be considered valid unless the changes are reviewed and conclusions of this report modified or verified in writing.

This report may be subject to review by controlling agencies, and any modifications they deem necessary should be made a part thereof, subject to our technical acceptance of such modifications. All submissions of this report should be in its entirety. Under no circumstances should this report be summarized and synthesized to be quoted out of context for any purpose.

Test findings and statements of professional opinion do *not* constitute a guarantee or warranty, and *no* warranties, either expressed or implied, are made as to the professional advice provided under the terms of this agreement. We have strived, however, to provide our services in accordance with generally accepted geotechnical engineering practices in this community at the time of this report.

Appendix A

Field Exploration and Boring Logs

Appendix A
Field Exploration and Boring Logs

The field exploration included a site reconnaissance and subsurface exploration. During the site reconnaissance, the surface site conditions were noted, and the approximate locations of any exploration points were determined. The following descriptions of exploration methods are generic and may include methods not used on this project. Reference to the boring logs can be made to determine which methods are applicable to this project, and any differences between what is described below and actually occurred is described on the boring logs or in the main body of the report.

The test borings were advanced by either hand digging, digging with a backhoe, or drilling. In the case of drilling, a truck-mounted rotary drilling rig with a hollow-stem auger or bucket was used to advance the borings. When we expect to encounter shallow groundwater, a wet rotary drilling operation is usually used. The method actually used is noted on the boring logs. For geologic studies when the need for visual examination of the bedding and other stratigraphic features is needed along with engineering data, the larger bucket augers are used to allow a geologist to enter the excavation for visually logging the hole. When geologically logging borings and trenches, the sides are scraped prior to logging. A prefix B is used to designate a boring made with a drilling rig. When hand dug, the boring numbers have a prefix HB. When a backhoe was used, prefixes TP (test pit) or T (trench) are used. The difference between a trench and test pit being the length of the exploration; a trench being a long narrow exploration, most commonly used for fault studies. In each case, the soils were logged by technical personnel from our office and visually classified in the field in general accordance with the Unified Soil Classification system. The field descriptions have been modified as appropriate to reflect laboratory results when preparing the final boring logs.

Relatively undisturbed samples of the subsurface materials were obtained at appropriate intervals in the borings using a steel drive sampler (2.5-inches inside diameter, 3-inches outside diameter) lined with brass, one-inch-high sample rings with a diameter of 2.4 inches. This is referred to as a modified California sampler. The boring may be advanced by drilling with a hollow-stem auger or with a wet rotary operation. If below the groundwater, the hollow-stem is filled with water or drilling mud to counteract the fluid pressure of the groundwater. The sampler was usually driven into the bottom of the borehole with successive drops of a 140-pound safety hammer connected to the sampler with either A or AW rod and falling 30 inches. An automatic hammer is usually used when drilling with a CME dill rig, and a Safe-T-Driver is used when drilling with a Mobile drill rig. When above the groundwater level, a downhole Safe-T-Driver is usually used. Studies have shown that hammer efficiencies of the automatic hammer is over 90% while that of the Safe-T-Driver is about 70%, based on impact velocities. When a bucket auger is used to advance the boring, the driving weights change with depth, depending on the weight characteristics of the telescoping kelley bar, but the height of fall is usually 18 inches. Sampler driving resistance, expressed as blows per 6 inches of penetration, is presented on the boring logs at the respective sampling depths. When the borings or trenches are excavated with a backhoe, the sampler is pushed into the soil with the force of the backhoe. A hand sampler is used when the borings or trenches are advanced by hand digging or in some cases when a backhoe is used to make the excavation. This hand sampler is similar to the conventional California sampler, but lighter weight. An approximately 8-pound hammer falling about 18 inches is used to drive the hand sampler about 6 inches into the bottom of the exploration. The type of sampler used is noted on the boring logs. In some cases the hammer weight and falling distance deviate from those given above. The actual conditions are shown on the boring logs and supersede the conditions given above.

Ring samples were retained in close-fitting, moisture tight containers for transport to our laboratory for testing. Bulk samples, which were collected from cuttings, were placed in bags and transported to our laboratory for testing.

When noted on the boring logs, standard penetration test (SPT) samples were obtained using either a 20-inch or a 32-inch long split-barrel sampler with a 2-inch outside diameter and a 1.375-inch inside diameter when liners are used (1.5-inch inside diameter without liners). Unless noted otherwise, liners are used. This sampler is driven into

the soil with successive drops of a 140-pound, safety hammer falling 30 inches. The blows are recorded for each 6 inches of penetration for a total penetration of 18 or 24 inches. The sum of the number of blows for the last 12 inches of an 18-inch penetration or the middle 12 inches of a 24-inch penetration is referred to as the *N* value.

Logs, which are presented on Plates at the end of this Appendix, include a description and classification of each stratum, sample locations, blow counts, groundwater conditions encountered during drilling, results from selected types of laboratory tests, and drilling information. Keys to *Soil and Bedrock Symbols and Terms* are included on Plate A-1 and Plate A-2.

Each boring or trench, unless noted otherwise, was backfilled with cuttings at the completion of the logging and sampling. The backfill, however, may settle with time, and it is the responsibility of our client to ensure that such settlement does *not* become a liability.

Advanced Geotechnical Services

Key to Soil Symbols and Terms

Major Divisions			USCS Group Symbols	Typical Names
Coarse-Grained Soils (More than half of material is larger than No. 200 sieve)	Gravels (More than half of coarse fraction is larger than No. 4 sieve)	Clean gravels (Little or no fines)	GW	Well-graded gravels, gravel-sand mixtures, little or no fines
			GP	Poorly graded gravels, gravel-sand mixtures, little or no fines
		Gravels with fines (Appreciable amount of fines)	GM	Silty gravels, gravel-sand-silt mixtures
			GC	Clayey gravels, gravel-sand, clay mixtures
	Sands (More than half of coarse fraction is smaller than No. 4 sieve)	Clean sands (Little or no fines)	SW	Well-graded sands, gravelly sand, little or no fines
			SP	Poorly graded sands, gravelly sands little or no fines
		Sands and fines (Appreciable amount of fines)	SM	Silty sands, sand-silt mixtures
			SC	Clayey sands, sand-clay mixtures
Fine-Grained Soils (More than half of material is smaller than No. 200 sieve)	Silts and Clays Liquid Limit < 50		ML	Silts and very fine sands, rock-flour, silty or clayey fine sands, or clayey silts with slight plasticity
			CL	Inorganic clays of low or medium plasticity, gravelly clays, sandy clays, silty clays, lean clays
			OL	Organic silts and organic silty clays of low plasticity
	Silts and Clays Liquid Limit > 50		MH	Inorganic silts, micaceous or diatomaceous fine sandy or silty soils, elastic silts
			CH	Inorganic clays of high plasticity, fat clays
	Highly Organic Soils		OH	Organic clays of medium to high plasticity, organic silts
			Pt	Peat and other highly organic soils

Legend of Laboratory Tests

G - Grain Size	C - Consolidation	PP - Pocket Penetrometer
A - Atterberg Limits	DS - Direct Shear	CH - Chemical
P - Compaction	U - Unconfined	
S - Swell/Expansion	T - Triaxial	

Sampler Type

Modified California / Hand Sampler — SPT / Shelby Tube — Rock Core / Bulk — No Recovery

Terms used in this report for describing soils according to their texture or grain size distributions are generally in accordance with the Unified Soil Classification System.

Terms Describing Density and Consistency

Coarse Grained soils (major portion retained on No. 200 sieve) include (1) clean gravels, (2) silty or clayey gravels, and (3) silty, clayey, or gravelly sands. Relative density is related to SPT blow count corrected for overburden pressure or drive energy.

Density		SPT *N* Value Blows/Ft	Relative Density %
Very Loose	vl	0 to 4	0 to 15
Loose	l	4 to 10	15 to 35
Medium Dense	md	10 to 30	35 to 65
Dense	d	30 to 50	65 to 85
Very Dense	vd	> 50	85 to 100

Fine Grained soils (major portions passing No. 200 sieve) include (1) inorganic and organic silts and clays, (2) gravelly, sandy, or silty clays, and (3) clayey silts. Consistency is rated according to shear strength as indicated by penetrometer readings, direct shear, or SPT blow count.

Consistency	Shear Strength, ksf	SPT *N* Value
Very Soft	< 0.25	0 to 2
Soft	0.25 to 0.50	2 to 4
Firm	0.50 to 1.00	4 to 8
Stiff	1.00 to 2.00	8 to 16
Very Stiff	2.00 to 4.00	16 to 32
Hard	> 4.00	> 32

Terms Characterizing Soil Structure

Slickensided	Having inclined planes of weakness that are slick and glossy in appearance.
Fissured	Containing shrinkage cracks, frequently filled with fine sand or silt; usually more or less vertical.
Laminated	Composed of thin layers of varying color and texture.
Interbedded	Composed of alternate layers of different soil types.
Calcareous	Containing appreciable quantities of calcium carbonate.
Well Graded	Having wide range in grain sizes and substantial amounts of intermediate particle sizes.
Poorly Graded	Predominately one grain size, or having a range of grain sizes with some intermediate sizes missing.
Porous	Having visibly apparent void spaces through which water, air, or light may pass.

Soil Moisture

From low to high, the moisture content is indicated by:

Dry	D
Slightly Moist	Sl M
Moist (near optimum for compaction)	M
Very Moist	V M
Wet	W

Size Proportions

Designation	Percent by Weight
Trace	< 5
Few	5 to 10
Little	15 to 25
Some	30 to 45

Grain Size Distribution

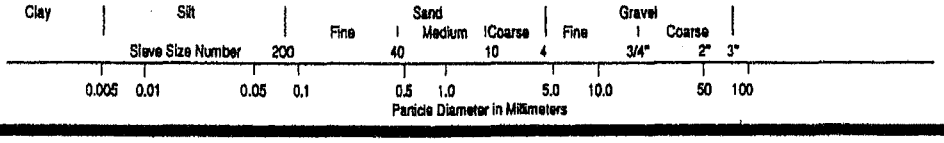


Plate A-1

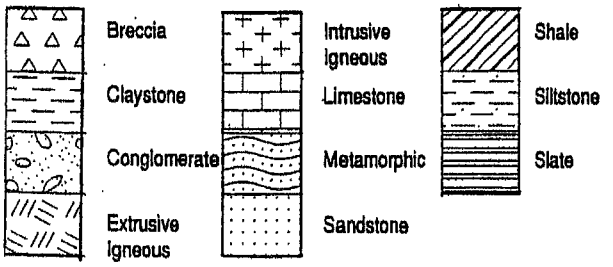
Degree of Weathering
Diagnostic Feature

Descriptive Term	Discoloration Extent	Fracture Condition	Surface Characteristics	Original Texture	Grain Boundary Condition
Unweathered	None	Closed or discolored	Unchanged	Preserved	Tight
Slightly Weathered	Less 20% of fracture spacing on both sides of fracture	Discolored, may contain thin filling	Partial discoloration	Preserved	Tight
Moderately Weathered	Greater than 20% of fracture spacing on both sides of fracture	Discolored, may contain thick filling, cemented rock	Partial to complete discoloration, not friable except poorly cemented rocks	Preserved	Partial Opening
Highly Weathered	Throughout		Friable and possibly pitted	Mainly Preserved	Partial Separation
Completely Weathered	Throughout		Resembles a soil	Partly Preserved	Complete Separation

Discontinuity Spacing

Description for Structural Feature: Bedding, Foliation, or Flow Banding	Spacing		Description for Joints, Faults, or Other Fractures
Very Thickly (Bedded, Foliated, or Banded)	More than 2 m	More than 6 ft	Very Widely (Fractured or Jointed)
Thickly	60 cm to 2 m	2 to 6 ft	Widely
Moderately	20 to 60 cm	8 to 24 in.	Medium
Thinly	60 to 200 mm	2.5 to 8 in.	Closely
Very Thinly	20 to 60 mm	0.75 to 2.5 in.	Very Closely
Description for Microstructural Features: Bedding, Foliation, or Cleavage			
Intensely (Laminated, Foliated, or Cleaved)	6 to 20 mm	0.25 to 0.75 in.	Extremely Close
Very Intensely	< 6 mm	< 0.25 in.	

Graphic Symbols - Bedrock



Rock Hardness

Classification	Field Test
Very Weak	Can be dug by hand and crushed with fingers.
Weak	Friable, can be gouged deeply with a knife and will crumble readily under light hammer blows.
Moderately Strong	Can be peeled with a knife. Material crumbles under firm blows with the sharp end of a geologic pick.
Strong	Cannot be scaped or peeled with a knife point. Hand held specimen breaks with firm blows of the pick.
Very Strong	Difficult to scratch with knife point. Cannot break hand held specimen.

Separation of Fracture Walls

Description	Separation of Walls, mm
Closed	0
Very Narrow	0 to 0.1
Narrow	0.1 to 1.0
Wide	1.0 to 5.0
Very Wide	> 5.0

Fracture Filling

Description	Definition
Clean	No fracture filling material
Stained	Discoloration of rock only. No recognizable filling material.
Filled	Fracture filled with recognizable filling material.

Surface Roughness

Description	Classification
Smooth	Appears smooth and is essentially smooth to the touch. May be slickensided.
Slightly Rough	Asperities on the fracture surfaces are visible and can be distinctly felt.
Medium Rough	Asperites are clearly visible and fracture surface feels abrasive to the touch.
Rough	Large angular asperites can be seen. Some ridge and high-side angle steps evident.
Very Rough	Near vertical steps and ridges occur on the fracture surface.

Where slickensides are observed, the direction of the slickensides should be recorded after the standard discontinuity surface description.

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Boring Log B-1

Sheet 1 of 2

Project **The Molasky Group of Companies** Client No. **4994** Date Drilled **11/6/19**

Comment **5250 Ralston Street, Ventura**

Drilling Company/Driller **Choice Drilling** Equipment **Hollow-Stem Auger**

Driving Weight (lbs) **140** Average Drop (in.) **30** Hole Diameter (in.) **8**

Elevation _____ ft Depth to Water _____ ft After _____ hrs on _____ Logged By **CMW**

Depth, ft	Sample	Blows/6"	Graphic Symbol	Description of Material	Attitudes	Dry Unit Weight, pcf	Moisture Content, %	-#200, %	Other Tests
				This log, which is part of the report prepared by Advanced Geotechnical Services, Inc. for the named project, should be read together with that report for complete interpretation. This summary applies only at this boring location and at the time of drilling. Subsurface conditions may differ at other locations and may change at this location with the passage of time. The data presented is a simplification of actual conditions encountered.					
				Allivium (Qa) Medium brown Silty CLAY to Clayey SILT, dry, medium firm, porous					E.I.=82
		10 15 17		Dark gray brown speckled with red Silty CLAY, grading to brown Clayey SILT, slightly moist, firm		100.7	5.6		
5		8 12 15				101.1	14.6		
		11 12 18				104.3	18.3		
				Brown Clayey SILT, moist, very stiff					
10		15 19 21				104.2	16.2		
		9 12 15				102.0	15.9		
15		10 10 13				102.6	19.2		
				Dark yellow brown Sandy to Silty CLAY, moist, stiff					
20		15 17 19		**@20 feet-Groundwater**		96.3	21.4		
25		4 7 7		Brown Silty CLAY, very moist to saturated, firm		90.3	29.1		

Plate A- 3

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Boring Log B-1

Sheet 2 of 2

Project **The Molasky Group of Companies** Client No. **4994** Date Drilled **11/6/19**

Comment **5250 Ralston Street, Ventura**

Drilling Company/Driller **Choice Drilling** Equipment **Hollow-Stem Auger**

Driving Weight (lbs) **140** Average Drop (in.) **30** Hole Diameter (in.) **8**

Elevation _____ ft Depth to Water _____ ft After _____ hrs on _____ Logged By **CMW**

Depth, ft	Sample	Blows/6"	Graphic Symbol	Description of Material	Attitudes	Dry Unit Weight, pcf	Moisture Content, %	-#200, %	Other Tests
		7 9 10		This log, which is part of the report prepared by Advanced Geotechnical Services, Inc. for the named project, should be read together with that report for complete interpretation. This summary applies only at this boring location and at the time of drilling. Subsurface conditions may differ at other locations and may change at this location with the passage of time. The data presented is a simplification of actual conditions encountered.		94.8	25.2		
35		4 4 5				92.2	27.0		
40		5 7 9		Dark brown Sandy to Silty CLAY, wet to saturated, stiff		96.1	22.8		
45		4 5 5		Grades to Brown, very moist, firm, trace hard white calcium carbonate nodules (<1/8" diameter)		95.0	25.6		
50		7 7 9				94.2	24.8		
55				**Total Depth Explored 51.5 feet** **Groundwater Encountered at 20 feet** **Boring Backfilled with Cuttings 11/6/2019**					

Plate A- 4

advanced geotechnical services, inc.

Boring Log B-2

Sheet 1 of 2

Project **The Molasky Group of Companies** Client No. **4994** Date Drilled **11/6/19**

Comment **5250 Ralston Street, Ventura**

Drilling Company/Driller **Choice Drilling** Equipment **Hollow-Stem Auger**

Driving Weight (lbs) **140** Average Drop (in.) **30** Hole Diameter (in.) **8**

Elevation _____ ft Depth to Water _____ ft After _____ hrs on _____ Logged By **CMW**

Depth, ft	Sample	Blows/6"	Graphic Symbol	Description of Material	Attitudes	Dry Unit Weight, pcf	Moisture Content, %	#200, %	Other Tests
				This log, which is part of the report prepared by Advanced Geotechnical Services, Inc. for the named project, should be read together with that report for complete interpretation. This summary applies only at this boring location and at the time of drilling. Subsurface conditions may differ at other locations and may change at this location with the passage of time. The data presented is a simplification of actual conditions encountered.					
		7 12 15		**Allivium (Qa)** Gray brown and light brown Silty CLAY and SILT, weakly laminated, slightly moist to moist, medium firm		102.9	9.3		
5		5 7 9		Brown Clayey SILT with fine SAND interbeds, moist, medium firm		95.8	23.5		
		6 6 8		Brown Clayey SILT, moist, stiff		100.0	20.8		
10		6 7 7		Dark brown Silty CLAY, moist, stiff		93.0	23.4		
		6 8 11		Gray brown to yellowish brown Silty CLAY to CLAY, moist, stiff		94.4	21.2		
15		5 7 7				98.5	18.4		
20		2 5 5		Yellowish brown Clayey SILT, very moist to wet, firm		89.5	31.4		
				Brown Clayey SILT to Silty CLAY, saturated to very moist, very stiff to stiff					
25		7 12 15				91.9	28.6		

Plate A- 5

advanced geotechnical services, inc.

Boring Log B-2

Sheet 2 of 2

Project **The Molasky Group of Companies** Client No. **4994** Date Drilled **11/6/19**

Comment **5250 Ralston Street, Ventura**

Drilling Company/Driller **Choice Drilling** Equipment **Hollow-Stem Auger**

Driving Weight (lbs) **140** Average Drop (in.) **30** Hole Diameter (in.) **8**

Elevation _____ ft Depth to Water _____ ft After _____ hrs on _____ Logged By **CMW**

Depth, ft	Sample	Blows/6"	Graphic Symbol	Description of Material	Attitudes	Dry Unit Weight, pcf	Moisture Content, %	#200, %	Other Tests
		7 9 12		This log, which is part of the report prepared by Advanced Geotechnical Services, Inc. for the named project, should be read together with that report for complete interpretation. This summary applies only at this boring location and at the time of drilling. Subsurface conditions may differ at other locations and may change at this location with the passage of time. The data presented is a simplification of actual conditions encountered.		91.1	28.0		
35		10 10 11		Dark brown Silty CLAY, minor white calcium carbonates, very moist, stiff		96.2	25.4		
40		5 7 10				92.7	26.6		
				Total Depth Explored 41.5 feet **Groundwater Encountered at 20 feet** **Boring Backfilled with Cuttings 11/6/2019**					
45									
50									
55									

Plate A- 6

advanced geotechnical services, inc.

Boring Log B-3

Sheet 1 of 1

Project **The Molasky Group of Companies** Client No. **4994** Date Drilled **11/6/19**

Comment **5250 Ralston Street, Ventura**

Drilling Company/Driller **Choice Drilling** Equipment **Hollow-Stem Auger**

Driving Weight (lbs) **140** Average Drop (in.) **30** Hole Diameter (in.) **8**

Elevation _____ ft Depth to Water _____ ft After _____ hrs on _____ Logged By **CMW**

Depth, ft	Sample	Blows/6"	Graphic Symbol	Description of Material	Attitudes	Dry Unit Weight, pcf	Moisture Content, %	#200, %	Other Tests
				This log, which is part of the report prepared by Advanced Geotechnical Services, Inc. for the named project, should be read together with that report for complete interpretation. This summary applies only at this boring location and at the time of drilling. Subsurface conditions may differ at other locations and may change at this location with the passage of time. The data presented is a simplification of actual conditions encountered.					
				Allivium (Qa) Dark gray brown and light brown Silty CLAY to Clayey SILT, dry, loose, porous					E.I.=78
		10 12 12		Dark gray brown and light brown Silty CLAY and Clayey SILT, slightly moist to moist, firm		104.1	7.1		
5		6 7 9				90.8	23.6		
		7 9 10				100.5	20.5		
				Yellowish gray brown Sandy SILT, moist, stiff					
10		9 10 12				93.4	17.7		
				Dark brown Clayey SILT, moist, firm					
15		3 5 7				95.5	23.2		
20		3 5 7		Gray brown Clayey SILT to Silty CLAY, minor white calcium carbonate staining, very moist, firm **@ 20 feet-Groundwater**		92.9	28.9		
				Total Depth Explored 21.5 feet **Groundwater Encountered at 20 feet** **Boring Backfilled with Cuttings 11/6/2019**					
25									

Plate A- 7

advanced geotechnical services, inc.

Boring Log B-4

Sheet 1 of 1

Project **The Molasky Group of Companies** Client No. **4994** Date Drilled **11/6/19**

Comment **5250 Ralston Street, Ventura**

Drilling Company/Driller **Choice Drilling** Equipment **Hollow-Stem Auger**

Driving Weight (lbs) **140** Average Drop (in.) **30** Hole Diameter (in.) **8**

Elevation _____ ft Depth to Water _____ ft After _____ hrs on _____ Logged By **CMW**

Description of Material

This log, which is part of the report prepared by Advanced Geotechnical Services, Inc. for the named project, should be read together with that report for complete interpretation. This summary applies only at this boring location and at the time of drilling. Subsurface conditions may differ at other locations and may change at this location with the passage of time. The data presented is a simplification of actual conditions encountered.

Depth, ft	Sample	Blows/6"	Graphic Symbol	Description of Material	Attitudes	Dry Unit Weight, pcf	Moisture Content, %	-#200, %	Other Tests
		12 18 21		**Artificial Fill (af)** Asphalt and Base over Reddish brown Clayey SAND and Gravel, moist, medium dense		103.2	20.8		
5		9 12 15		**Allivium (Qa)** Dark gray brown speckled with red Silty CLAY, minor white calcium carbonate staining, grading to dark brown Clayey SILT, moist, very stiff to stiff		95.4	22.4		
		9 9 10				93.4	25.5		
10		9 10 12		Yellowish brown Clayey SILT, slightly moist to moist, firm		103.2	19.8		
15		9 12 15		Dark brown Silty CLAY, moist, very stiff		99.0	21.7		
20		7 9 12		@ 21 feet-Groundwater		96.7	25.7		
				Total Depth Explored 21.5 feet **Groundwater Encountered at 21 feet** **Boring Backfilled with Cuttings 11/6/2019**					
25									

Plate A- 8

advanced geotechnical services, inc.

Boring Log B-5

Sheet 1 of 1

Project **The Molasky Group of Companies** Client No. **4994** Date Drilled **11/6/19**

Comment **5250 Ralston Street, Ventura**

Drilling Company/Driller **Choice Drilling** Equipment **Hollow-Stem Auger**

Driving Weight (lbs) **140** Average Drop (in.) **30** Hole Diameter (in.) **8**

Elevation _____ ft Depth to Water _____ ft After _____ hrs on _____ Logged By **CMW**

Description of Material

This log, which is part of the report prepared by Advanced Geotechnical Services, Inc. for the named project, should be read together with that report for complete interpretation. This summary applies only at this boring location and at the time of drilling. Subsurface conditions may differ at other locations and may change at this location with the passage of time. The data presented is a simplification of actual conditions encountered.

Depth, ft	Blows/6"	Description of Material	Attitudes	Dry Unit Weight, pcf	Moisture Content, %	-#200, %	Other Tests
	7 12 18	**Artificial Fill (af)** Asphalt and Base over Reddish brown Clayey SAND and Gravel, moist, medium dense		109.1	18.8		
		Gray brown Clayey SILT with some coarse grains and gray CLAY inclusions, moist, firm					
5	5 7 9	**Allivium (Qa)** Brown Clayey SILT to Silty CLAY, moist, stiff, trace white calcium carbonate staining		100.3	21.2		
	10 13 17	Yellow brown Sandy to Clayey SILT, moist, firm		106.5	17.2		
10	7 11 15			102.0	16.3		
		Medium brown Silty CLAY to Clayey SILT, moist, very stiff, minor white calcium carbonate staining					
15	10 13 14			99.9	22.9		
		Yellow brown Clayey SILT, moist, firm					
20	5 8 8			95.4	15.7		
		Total Depth Explored 21.5 feet **No Groundwater Encountered** **Boring Backfilled with Cuttings 11/6/2019**					
25							

Plate A- 9

advanced geotechnical services, inc.

Boring Log B-6

Sheet 1 of 1

Project **The Molasky Group of Companies** Client No. **4994** Date Drilled **11/6/19**

Comment **5250 Ralston Street, Ventura**

Drilling Company/Driller **Choice Drilling** Equipment **Hollow-Stem Auger**

Driving Weight (lbs) **140** Average Drop (in.) **30** Hole Diameter (in.) **8**

Elevation _____ ft Depth to Water _____ ft After _____ hrs on _____ Logged By **CMW**

Description of Material

This log, which is part of the report prepared by Advanced Geotechnical Services, Inc. for the named project, should be read together with that report for complete interpretation. This summary applies only at this boring location and at the time of drilling. Subsurface conditions may differ at other locations and may change at this location with the passage of time. The data presented is a simplification of actual conditions encountered.

Depth, ft	Sample	Blows/6"	Graphic Symbol	Description of Material	Attitudes	Dry Unit Weight, pcf	Moisture Content, %	#200, %	Other Tests
0		7 8 13		**Artificial Fill (af)** Asphalt and Base over Brown Clayey SAND with Gravel, slightly moist, medium dense		103.1	18.4		
5				**Allivium (Qa)** Brown Clayey SILT to SILT, moist, stiff, weakly laminated		101.3	19.0		
				Total Depth Explored 6.5 feet **No Groundwater Encountered** **Boring Backfilled with Cuttings 11/6/2019**					
10									
15									
20									
25									

Plate A- 10

advanced geotechnical services, inc.

Boring Log B-7

Sheet 1 of 1

Project **The Molasky Group of Companies** Client No. **4994** Date Drilled **11/6/19**

Comment **5250 Ralston Street, Ventura**

Drilling Company/Driller **Choice Drilling** Equipment **Hollow-Stem Auger**

Driving Weight (lbs) **140** Average Drop (in.) **30** Hole Diameter (in.) **8**

Elevation _____ ft Depth to Water _____ ft After _____ hrs on _____ Logged By **CMW**

Description of Material

This log, which is part of the report prepared by Advanced Geotechnical Services, Inc. for the named project, should be read together with that report for complete interpretation. This summary applies only at this boring location and at the time of drilling. Subsurface conditions may differ at other locations and may change at this location with the passage of time. The data presented is a simplification of actual conditions encountered.

Depth, ft	Sample	Blows/6"	Graphic Symbol	Description of Material	Attitudes	Dry Unit Weight, pcf	Moisture Content, %	#200, %	Other Tests
		7 9 15		**Artificial Fill (af)** Asphalt and Base over brown Clayey SAND with Gravel, slightly moist, medium dense		101.6	12.9		
5		9 9 9		**Allivium (Qa)** Brown Clayey SILT, moist, stiff, weakly laminated		107.8	15.2		
				Total Depth Explored 6.5 feet **No Groundwater Encountered** **Boring Backfilled with Cuttings 11/6/2019**					
10									
15									
20									
25									

Plate A- 11

advanced geotechnical services, inc.

Boring Log B-8

Sheet 1 of 1

Project **The Molasky Group of Companies** Client No. **4994** Date Drilled **11/7/19**

Comment **5250 Ralston Street, Ventura**

Drilling Company/Driller **Choice Drilling** Equipment **Hollow-Stem Auger**

Driving Weight (lbs) **140** Average Drop (in.) **30** Hole Diameter (in.) **8**

Elevation _____ ft Depth to Water _____ ft After _____ hrs on _____ Logged By **CMW**

Description of Material

This log, which is part of the report prepared by Advanced Geotechnical Services, Inc. for the named project, should be read together with that report for complete interpretation. This summary applies only at this boring location and at the time of drilling. Subsurface conditions may differ at other locations and may change at this location with the passage of time. The data presented is a simplification of actual conditions encountered.

Depth, ft	Sample	Blows/6"	Graphic Symbol	Description of Material	Attitudes	Dry Unit Weight, pcf	Moisture Content, %	#200, %	Other Tests
				Artificial Fill (af) Asphalt and Base over brown Silty CLAY with Gravel, slightly moist, stiff					E.I.=86
		7 10 12				139.1	11.2		
				Allivium (Qa) Gray brown Silty CLAY, slightly moist, stiff, very weakly laminated					
5		9 11 14		Brown Clayey SILT to SILT, slightly moist, stiff, weakly laminated		106.8	15.7		
				Total Depth Explored 6.5 feet **No Groundwater Encountered** **Boring Backfilled with Cuttings 11/7/2019**					
10									
15									
20									
25									

Plate A- 12

advanced geotechnical services, inc.

Boring Log B-9

Sheet 1 of 1

Project **The Molasky Group of Companies** Client No. **4994** Date Drilled **11/7/19**

Comment **5250 Ralston Street, Ventura**

Drilling Company/Driller **Choice Drilling** Equipment **Hollow-Stem Auger**

Driving Weight (lbs) **140** Average Drop (in.) **30** Hole Diameter (in.) **8**

Elevation _____ ft Depth to Water _____ ft After _____ hrs on _____ Logged By **CMW**

Depth, ft	Sample	Blows/6"	Graphic Symbol	Description of Material	Attitudes	Dry Unit Weight, pcf	Moisture Content, %	-#200, %	Other Tests
				This log, which is part of the report prepared by Advanced Geotechnical Services, Inc. for the named project, should be read together with that report for complete interpretation. This summary applies only at this boring location and at the time of drilling. Subsurface conditions may differ at other locations and may change at this location with the passage of time. The data presented is a simplification of actual conditions encountered.					
				Artificial Fill (af) Brown Silty CLAY, small pieces of concrete, dry to slightly moist, stiff					
		15 17 27				108.1	17.7		
				Allivium (Qa) Very dark brown Silty CLAY, moist, very stiff					
5		12 18		Yellow brown SILT, slightly moist, stiff		106.9	12.6		
		18		Brown Clayey SILT, moist, very stiff, weakly laminated					
				Total Depth Explored 6.5 feet **No Groundwater Encountered** **Boring Backfilled with Cuttings 11/7/2019**					
10									
15									
20									
25									

Plate A- 13

advanced geotechnical services, inc.

Boring Log B-10

Sheet 1 of 1

Project **The Molasky Group of Companies** Client No. **4994** Date Drilled **11/7/19**

Comment **5250 Ralston Street, Ventura**

Drilling Company/Driller **Choice Drilling** Equipment **Hollow-Stem Auger**

Driving Weight (lbs) **140** Average Drop (in.) **30** Hole Diameter (in.) **8**

Elevation _____ ft Depth to Water _____ ft After _____ hrs on _____ Logged By **CMW**

Description of Material

This log, which is part of the report prepared by Advanced Geotechnical Services, Inc. for the named project, should be read together with that report for complete interpretation. This summary applies only at this boring location and at the time of drilling. Subsurface conditions may differ at other locations and may change at this location with the passage of time. The data presented is a simplification of actual conditions encountered.

Depth, ft	Sample	Blows/6"	Graphic Symbol	Description of Material	Attitudes	Dry Unit Weight, pcf	Moisture Content, %	-#200, %	Other Tests
		6 8 12		**Allivium (Qa)** Brown Silty CLAY, slightly moist to moist, stiff, weakly laminated with light yellow SILT		106.9	14.5		
5		6 12 18		Yellowish brown SILT, slightly moist to moist, stiff		108.3	14.4		
				Total Depth Explored 6.5 feet **No Groundwater Encountered** **Boring Backfilled with Cuttings 11/7/2019**					
10									
15									
20									
25									

Plate A- 14

advanced geotechnical services, inc.

Boring Log P-1

Sheet 1 of 1

Project **The Molasky Group of Companies** Client No. **4994** Date Drilled **11/7/19**

Comment **5250 Ralston Street, Ventura**

Drilling Company/Driller **Choice Drilling** Equipment **Hollow-Stem Auger**

Driving Weight (lbs) **140** Average Drop (in.) **30** Hole Diameter (in.) **8**

Elevation _____ ft Depth to Water _____ ft After _____ hrs on _____ Logged By **CMW**

Depth, ft	Sample	Blows/6"	Graphic Symbol	Description of Material	Attitudes	Dry Unit Weight, pcf	Moisture Content, %	#200, %	Other Tests
				This log, which is part of the report prepared by Advanced Geotechnical Services, Inc. for the named project, should be read together with that report for complete interpretation. This summary applies only at this boring location and at the time of drilling. Subsurface conditions may differ at other locations and may change at this location with the passage of time. The data presented is a simplification of actual conditions encountered.					
				Allivium (Qa) Dark brown Silty CLAY, moist, medium firm					
5									
				Yellow brown Clayey Sandy SILT, moist, stiff					
10									
				Gray yellow Silty CLAY, very moist, stiff					
15		5 7 10				89.9	24.1		
				Total Depth Explored 16 feet **No Groundwater Encountered** **Perforated Pipe Placed in Boring 11/7/2019**					
20									
25									

Plate A- 15

advanced geotechnical services, inc.

Boring Log P-2

Sheet 1 of 1

Project **The Molasky Group of Companies** Client No. **4994** Date Drilled **11/7/19**

Comment **5250 Ralston Street, Ventura**

Drilling Company/Driller **Choice Drilling** Equipment **Hollow-Stem Auger**

Driving Weight (lbs) **140** Average Drop (in.) **30** Hole Diameter (in.) **8**

Elevation _____ ft Depth to Water _____ ft After _____ hrs on _____ Logged By **CMW**

Description of Material

This log, which is part of the report prepared by Advanced Geotechnical Services, Inc. for the named project, should be read together with that report for complete interpretation. This summary applies only at this boring location and at the time of drilling. Subsurface conditions may differ at other locations and may change at this location with the passage of time. The data presented is a simplification of actual conditions encountered.

Depth, ft	Sample	Blows/6"	Graphic Symbol	Description of Material	Attitudes	Dry Unit Weight, pcf	Moisture Content, %	-#200, %	Other Tests
0				**Allivium (Qa)** Dark brown Silty CLAY, moist, stiff					
5		6 7 13				94.2	25.5		
				Total Depth Explored 5 feet **No Groundwater Encountered** **Perforated Pipe Placed in Boring 11/7/2019**					
10									
15									
20									
25									

Plate A- 16

advanced geotechnical services, inc.

Boring Log P-3

Sheet 1 of 1

Project **The Molasky Group of Companies** Client No. **4994** Date Drilled **11/7/19**

Comment **5250 Ralston Street, Ventura**

Drilling Company/Driller **Choice Drilling** Equipment **Hollow-Stem Auger**

Driving Weight (lbs) **140** Average Drop (in.) **30** Hole Diameter (in.) **8**

Elevation _____ ft Depth to Water _____ ft After _____ hrs on _____ Logged By **CMW**

Description of Material

This log, which is part of the report prepared by Advanced Geotechnical Services, Inc. for the named project, should be read together with that report for complete interpretation. This summary applies only at this boring location and at the time of drilling. Subsurface conditions may differ at other locations and may change at this location with the passage of time. The data presented is a simplification of actual conditions encountered.

Depth, ft	Sample	Blows/6"	Graphic Symbol	Description of Material	Attitudes	Dry Unit Weight, pcf	Moisture Content, %	#200, %	Other Tests
				Artificial Fill (af) Asphalt and Base over brown Silty SAND and Gravel, slightly moist, medium dense					
				Alluvium (Qa) Dark brown silty CLAY, moist, firm					
5									
10				Yellowish brown Clayey, Sandy SILT, moist, stiff					
15		7 12 18		Dark yellow brown Silty CLAY to Clayey SILT, moist, very stiff		104.1	18.5		
				Total Depth Explored 16 feet **No Groundwater Encountered** **Perforated Pipe Placed in Boring 11/7/2019**					
20									
25									

Plate A- 17

advanced geotechnical services, inc.

Boring Log P-4

Sheet 1 of 1

Project **The Molasky Group of Companies** Client No. **4994** Date Drilled **11/7/19**

Comment **5250 Ralston Street, Ventura**

Drilling Company/Driller **Choice Drilling** Equipment **Hollow-Stem Auger**

Driving Weight (lbs) **140** Average Drop (in.) **30** Hole Diameter (in.) **8**

Elevation _____ ft Depth to Water _____ ft After _____ hrs on _____ Logged By **CMW**

Depth, ft	Sample	Blows/6"	Graphic Symbol	Description of Material	Attitudes	Dry Unit Weight, pcf	Moisture Content, %	-#200, %	Other Tests
				This log, which is part of the report prepared by Advanced Geotechnical Services, Inc. for the named project, should be read together with that report for complete interpretation. This summary applies only at this boring location and at the time of drilling. Subsurface conditions may differ at other locations and may change at this location with the passage of time. The data presented is a simplification of actual conditions encountered.					
				Artificial Fill (af) Asphalt and Base over brown Silty SAND and Gravel, slightly moist					
		7 7 11		**Alluvium (Qa)** Dark brown with red speckled Silty CLAY, moist @ 4.5 feet-grades to Gray brown Silty CLAY, moist, stiff, very weakly laminated with yellow SILT		105.0	17.6		
5				**Total Depth Explored 5 feet** **No Groundwater Encountered** **Perforated Pipe Placed in Boring 11/7/2019**					
10									
15									
20									
25									

Appendix B

Laboratory Testing

Appendix B
Laboratory Testing

A laboratory test program is designed for each project to evaluate the physical and mechanical properties of the soil and bedrock materials encountered at the site during our field exploration program. Laboratory tests were conducted on representative samples for the purpose of classification and determining their properties for use in analyses and evaluations. The most common laboratory tests include moisture-density, Atterberg limits, grain-size analyses (sieve and hydrometer analyses), sand equivalent, direct shear, consolidation, compaction, expansion index, and *R*-values. The following descriptions of test methods are generic and may include methods not used on this project. Reference to the boring logs and test results on Plates attached to this appendix will show which tests were performed for this project. Laboratory testing is performed in general accordance with the most recent ASTM (2007) test designations available at the time of testing.

Classification Tests

Classification testing is performed to identify differences in material behavior and to correlate the results with shear strength and volume change characteristics of the materials. Classification testing includes unit weight (e.g., dry density), moisture content, Atterberg limits, grain size analyses (sieve and hydrometer), and sand equivalent.

Moisture-Density Test

Site soils were classified in the laboratory in accordance with the Unified Soil Classification System. Moisture contents are performed in general accordance with ASTM Test Designation D2216 and unit weights were determined in general accordance with ASTM Test Designation D2937. Field moisture contents and dry unit weights were determined for the ring samples obtained in the field. Field moisture contents and dry unit weights are shown on the boring logs in Appendix A.

Sieve Analysis

Sieve analysis tests were conducted on the on-site soils in general accordance with sieve analysis test procedure from ASTM Test Designation D422. This method covers the quantitative determination of the distribution of particle sizes in soils. If this test was performed, the results are presented on Plates attached to this appendix.

Hydrometer Test

Hydrometer tests were performed in general accordance with ASTM Test Designation D422. If this test was performed, the results are presented on Plates attached to this appendix. Samples with obviously little course material and a high percentage of fines were prepared with a wet method (ASTM Test Designation D2217) rather than air-drying the sample and pulverizing with a mortar and pedestal.

Shear Tests

Direct shear tests were performed in general accordance with ASTM D3080 to determine the shear strength parameters of undisturbed on-site soils or remolded soil specimens. The samples are usually tested in an artificially saturated condition. This is accomplished by soaking the specimens in a confined container for a period of one or 2 days, depending on the permeability of the material. The specimen, 1-inch-high and 2.4-inch-diameter, is placed in the shear device, and a vertical stress is applied to the specimen. The specimen is allowed to reach an equilibrium state (swell or consolidate). The specimen is then sheared under a constant rate of deformation. The rate of deformation for a slow test, sufficiently slow to presumably allow drainage, is selected from computed or measured consolidation rates to simulate full drainage (full dissipation of any tendency for pore water pressure changes) during shear. A rate of displacement of 0.005 inches per minute was used for the most tests. The process usually is repeated for 3 specimens, each under different vertical stresses. The results from the 3 tests are plotted on a diagram of shear stress and normal (vertical) stress at failure, and linear approximations are drawn of the failure curves to determine the angle of internal friction and cohesion. The first moisture content shown on the graphs (associated with peak values) is for either the in-situ condition or the remolded condition, and the second moisture content (associated with ultimate value) is for the soaked condition.

Consolidation Test

Consolidation tests were performed in general accordance with ASTM D2435 and D5333 on selected samples to evaluate the load-deformation characteristics of the earth soils. The tests were performed primarily on material that would be most susceptible to consolidation under anticipated foundation loading. The soil specimen, contained in a 2.4-inch-diameter, 1.0-inch-high sampling ring, is placed in a loading frame under a seating pressure of 0.1 ksf. Vertical loads are applied to the samples in several geometric increments, and the resulting deformations were recorded at selected time intervals. When the pressure reaches a preselected effective overburden pressure (often 2 ksf) and the specimen has consolidated under that pressure, the laboratory technician adds water to the test cell and records the vertical movement. After the specimen reaches equilibrium with the addition of water, the technician continues the loading process, usually up to a pressure of about 8 ksf. The specimen is then unloaded in increments, and the test is dismantled. The results of the test are presented in terms of percent volume change versus applied vertical stress. If this test was performed, the results are presented on Plates attached to this appendix.

Compaction Test

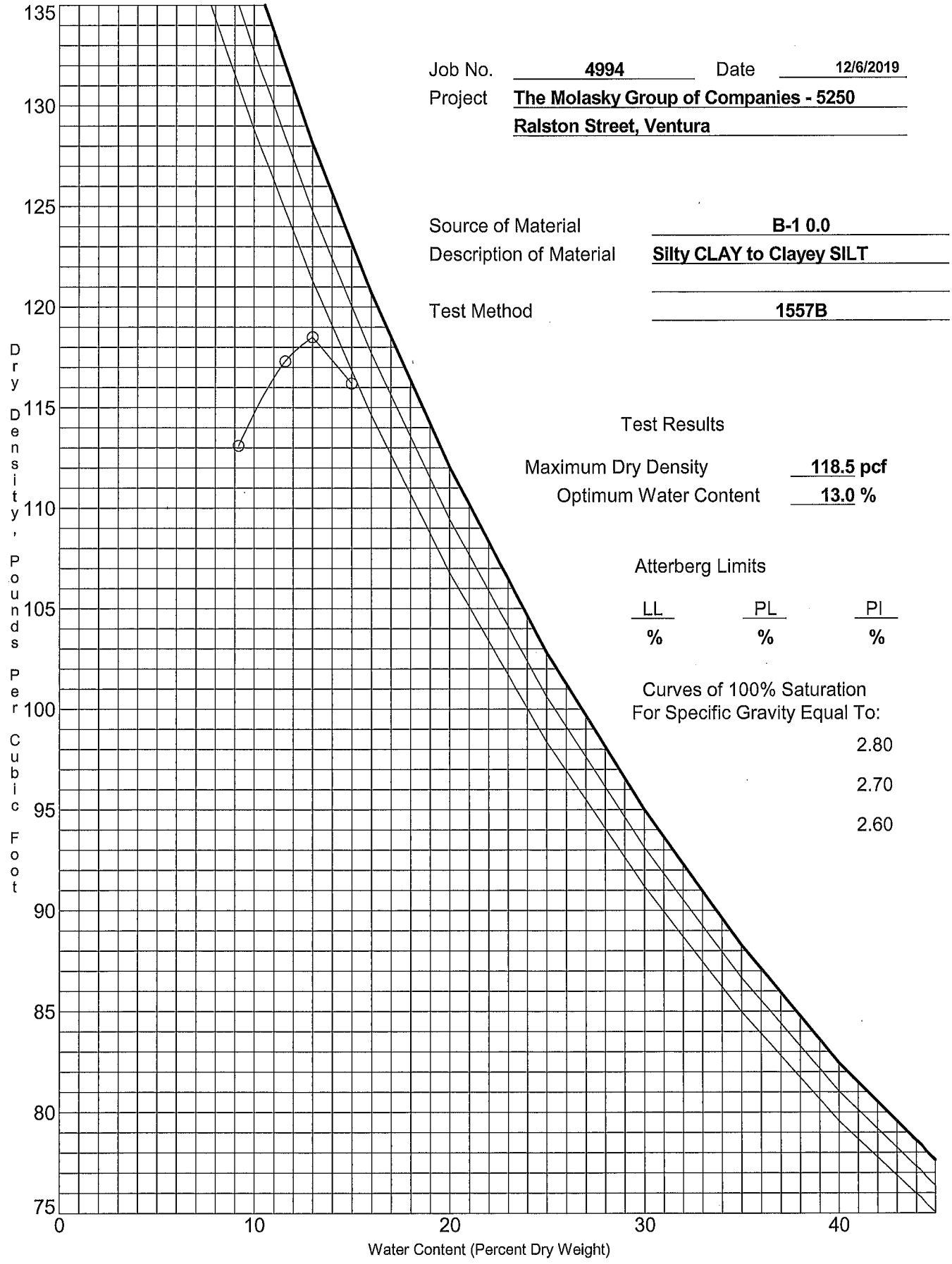
Compaction tests provide information on the relationship between moisture content and dry density of the soil compacted in a given manner. The maximum density is obtained for a given compaction effort at an optimum moisture content. Specifications for earthwork are in terms of the unit weight (or dry density) expressed as a percentage of the maximum density, and the moisture content compared to the optimum moisture content. Compaction tests were performed in general accordance with ASTM Test Designation D1557 to determine the maximum dry densities and optimum moisture contents of the on-site soils. If this test was performed, the results are presented on Plates attached to this appendix.

Expansion Index Test

The expansion index test provides an assessment of the potential for expansion or heave that could be detrimental to foundation or slab performance. Expansion Index tests are performed on shallow on-site soils in general accordance with expansion test procedures in ASTM D4829. In this test, a specimen is compacted at a degree of saturation between 45% and 55% in a 4.01-inch-diameter, 1.0-inch-high ring. The specimen is subjected to a seating pressure of 144 psf, water is added to the test cell, and swell is monitored until the expansion stops. The volume of swell is converted to an expansion index. Any test results are summarized on the boring logs in Appendix A.

Sample Remolding

In some cases remolded samples are used when performing direct shear tests and consolidation tests. Samples are remolded to a specified moisture and density by compacting the soil in a 2.42-inch-diameter sample ring. The specified moisture content is either at optimum or a few percentage points above optimum. The specified dry density is usually at a relative compaction of 90%. The required moisture is added to and mixed with dry soil, providing a homogeneous mixture. A 2.42-inch-diameter ring is placed in a 6-inch-diameter compaction mold, and soil is placed in the mold to above the ring. The soil is then compacted with a 5.5-pound hammer with a free-fall drop of 12 inches. The sample is trimmed, and the dry density is determined. If the dry density deviates more than about one pound per cubic foot from the specified dry density, the process is repeated with the number of blows altered to better achieve the specified dry density.



Job No. 4994 Date 12/6/2019

Project The Molasky Group of Companies - 5250 Ralston Street, Ventura

Source of Material B-1 0.0

Description of Material Silty CLAY to Clayey SILT

Test Method 1557B

Test Results

Maximum Dry Density 118.5 pcf

Optimum Water Content 13.0 %

Atterberg Limits

LL	PL	PI
%	%	%

Curves of 100% Saturation For Specific Gravity Equal To:

2.80

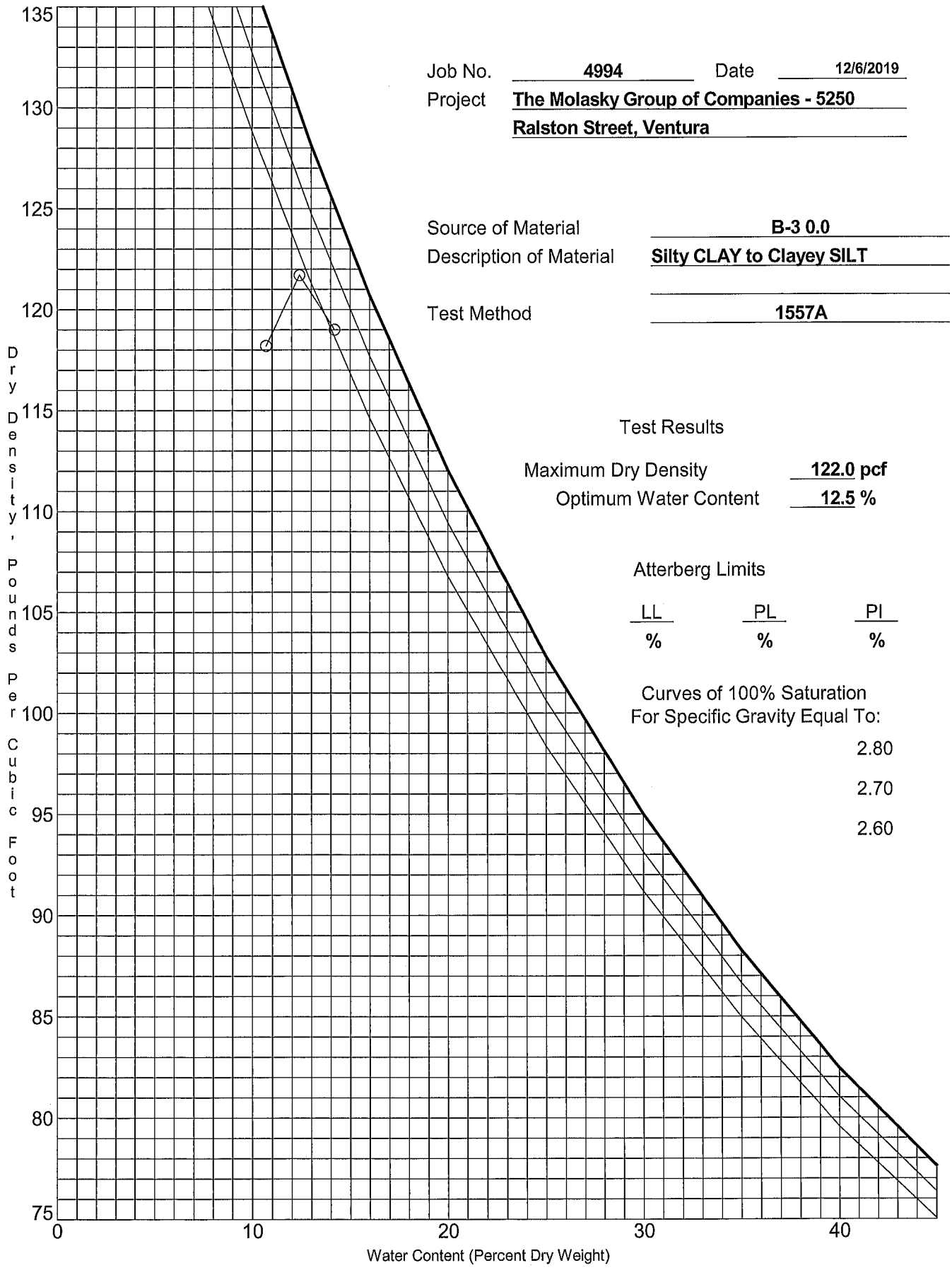
2.70

2.60

Moisture-Density Relationship

Advanced Geotechnical Services, Inc.

Plate B- 3



Job No. 4994 Date 12/6/2019

Project The Molasky Group of Companies - 5250 Ralston Street, Ventura

Source of Material B-3 0.0

Description of Material Silty CLAY to Clayey SILT

Test Method 1557A

Test Results

Maximum Dry Density 122.0 pcf

Optimum Water Content 12.5 %

Atterberg Limits

LL	PL	PI
%	%	%

Curves of 100% Saturation For Specific Gravity Equal To:

2.80

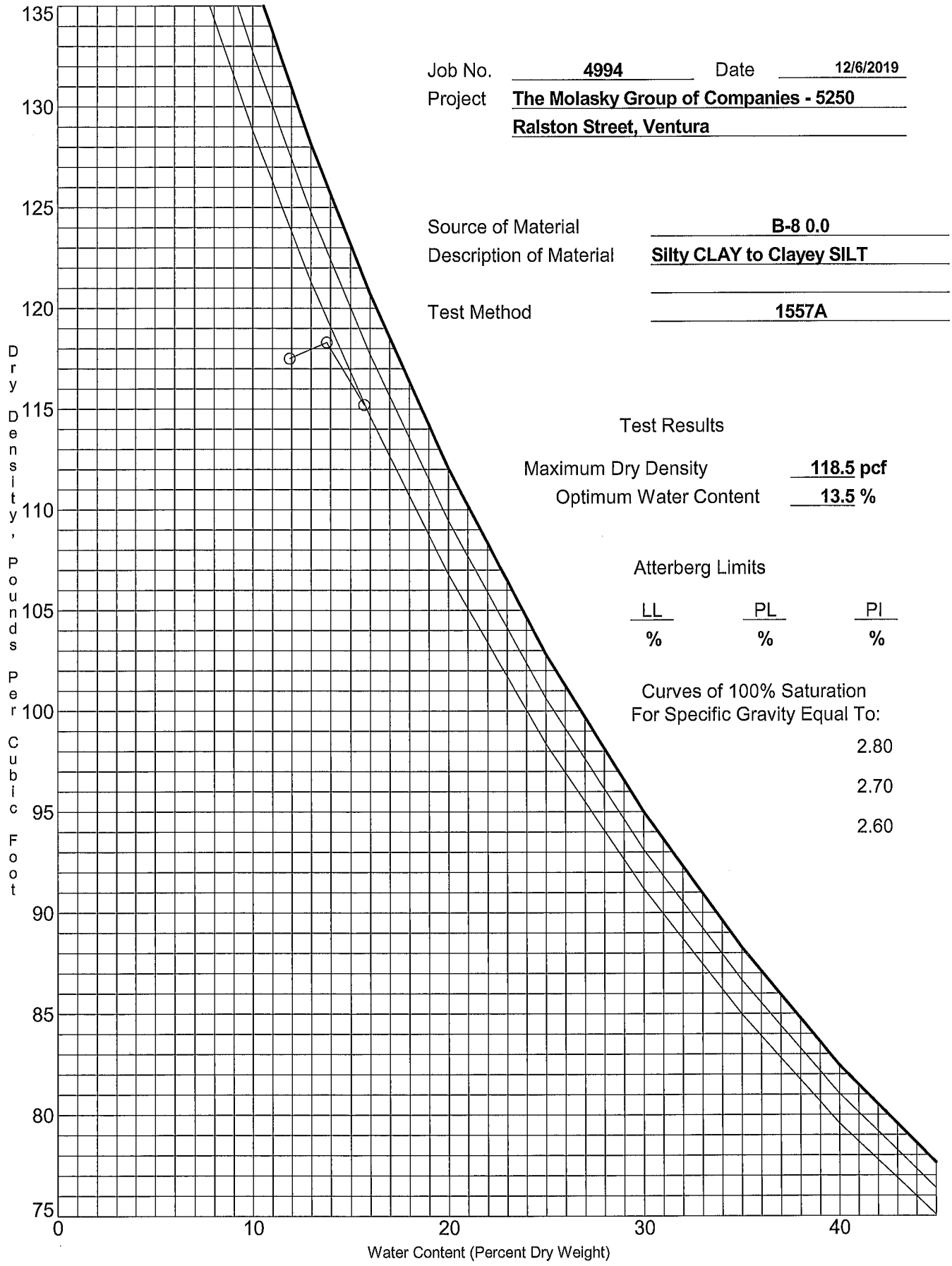
2.70

2.60

Moisture-Density Relationship

Advanced Geotechnical Services, Inc.

Plate B- 4



Job No. 4994 Date 12/6/2019

Project The Molasky Group of Companies - 5250 Ralston Street, Ventura

Source of Material B-8 0.0

Description of Material Silty CLAY to Clayey SILT

Test Method 1557A

Test Results

Maximum Dry Density 118.5 pcf

Optimum Water Content 13.5 %

Atterberg Limits

LL	PL	PI
%	%	%

Curves of 100% Saturation For Specific Gravity Equal To:

2.80

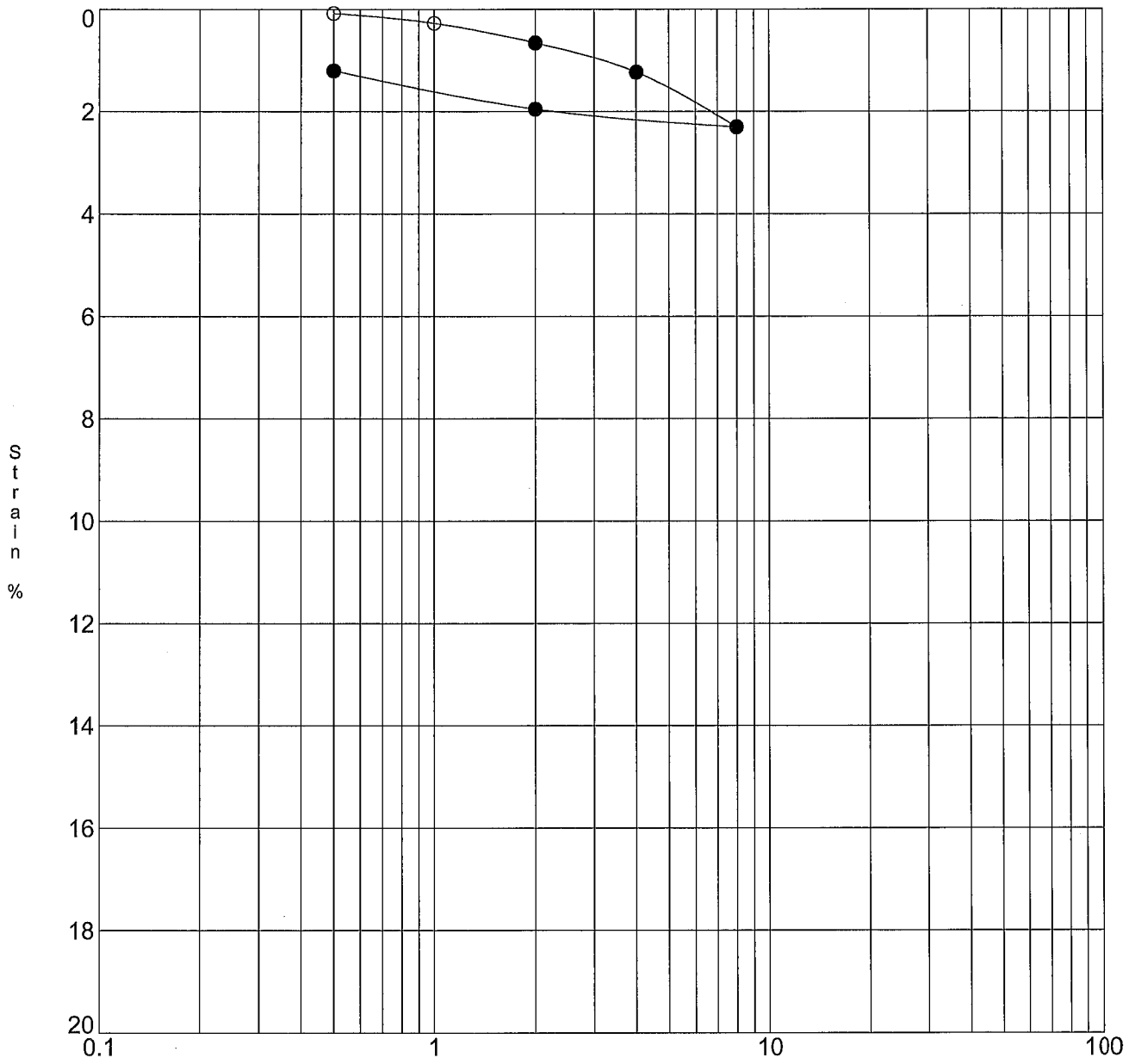
2.70

2.60

Moisture-Density Relationship

Advanced Geotechnical Services, Inc.

Plate B- 5



Open Symbol At Field Moisture, Solid Symbol After Submersion in Water

Specimen Identification			Classification	DD	MC%
○	B-1	0.0	Silty CLAY to Clayey SILT	112.5	8.8
●	B-1	0.0	*REMOLD*	115.3	13.1

Project: The Molasky Group of Companies - 5250 Ralston Street, Ventura

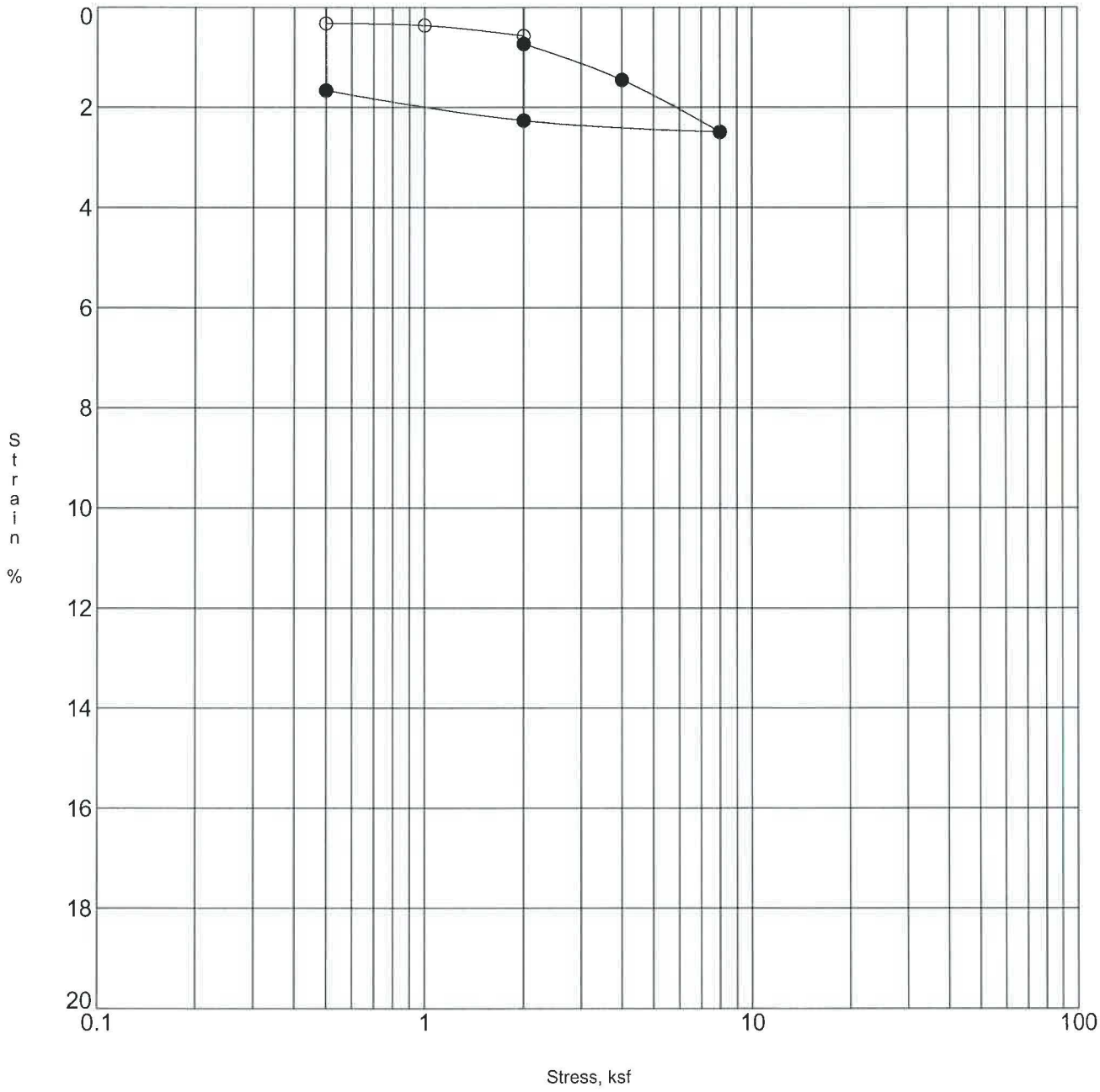
Client No.: 4994

Date: 12/6/19

Consolidation Test

Advanced Geotechnical Services, Inc.

Plate B- 6



Open Symbol At Field Moisture, Solid Symbol After Submersion in Water

	Specimen Identification		Classification	DD	MC%
○	B-3	5.0	Silty CLAY to Clayey SILT	102.6	14.5
●	B-3	5.0	*UNDISTURBED*	104.3	16.2

Project: The Molasky Group of Companies - 5250 Ralston Street, Ventura

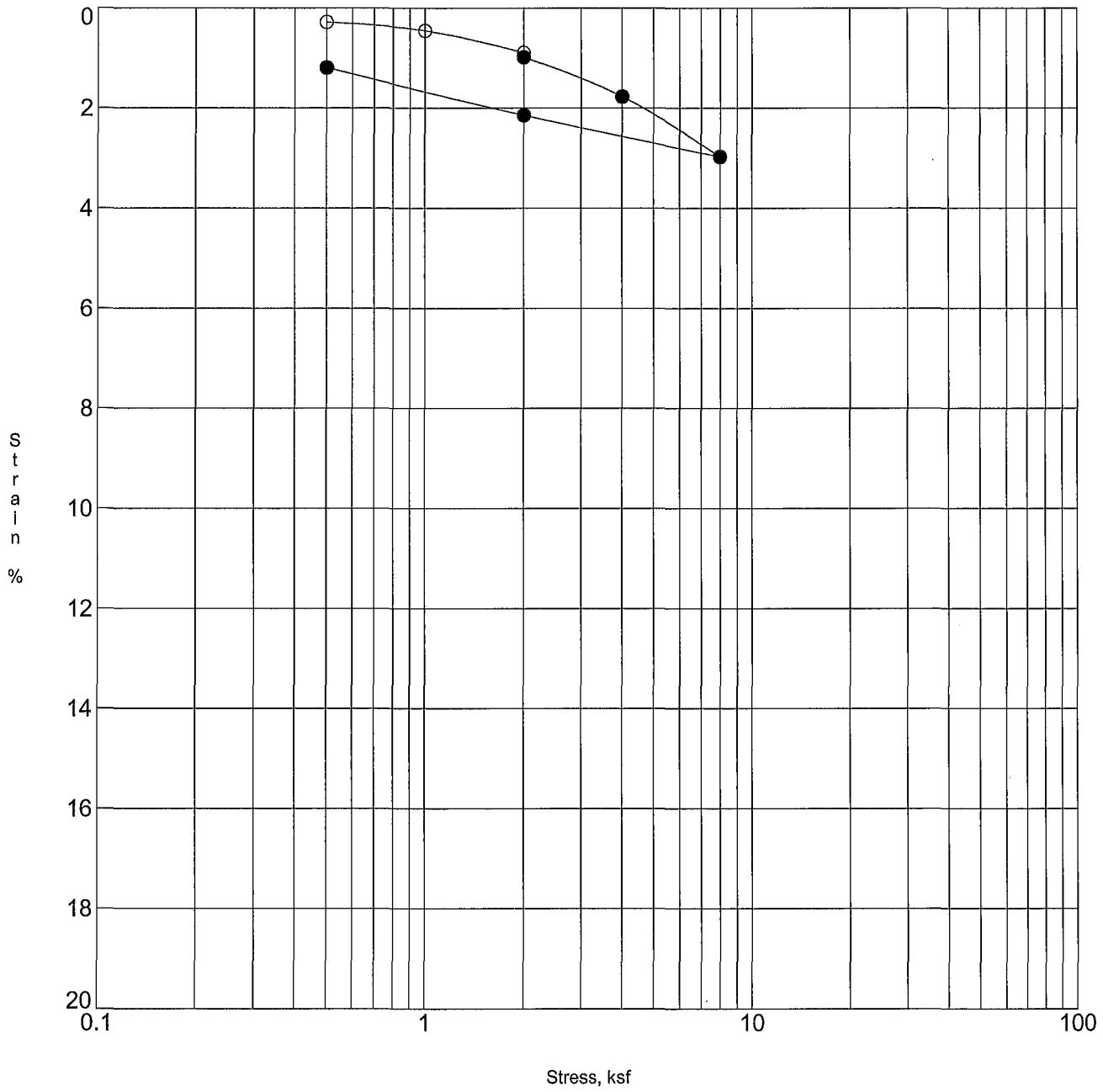
Client No.: 4994

Date: 12/6/19

Consolidation Test

Advanced Geotechnical Services, Inc.

Plate B- 7



Open Symbol At Field Moisture, Solid Symbol After Submersion in Water

	Specimen Identification		Classification	DD	MC%
○	B-1	7.5	Silty CLAY to Clayey SILT	108.4	13.0
●	B-1	7.5	*UNDISTURBED*	109.7	15.3

Project: The Molasky Group of Companies - 5250 Ralston Street, Ventura

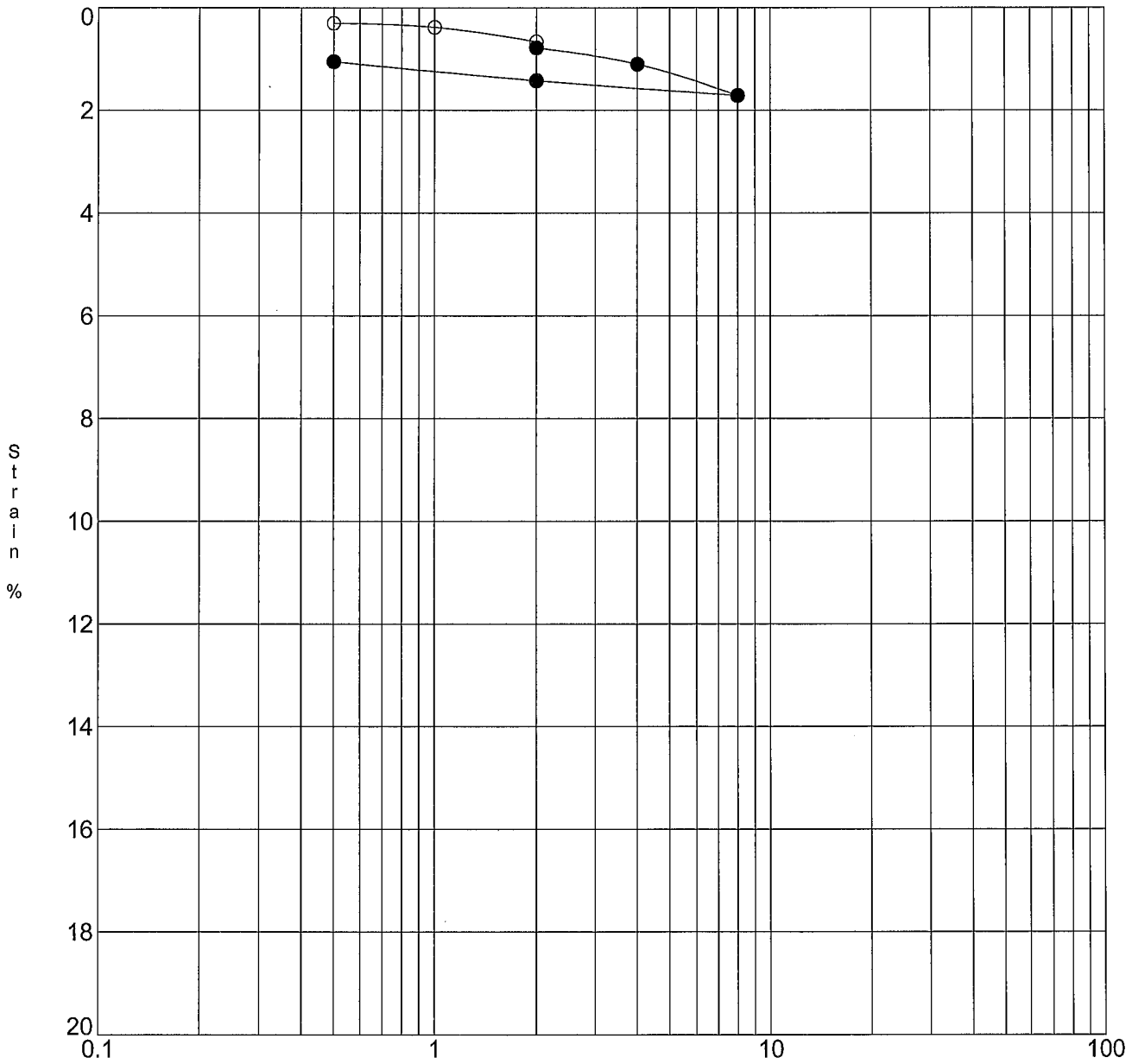
Client No.: 4994

Date: 12/6/19

Consolidation Test

Advanced Geotechnical Services, Inc.

Plate B- 8



Open Symbol At Field Moisture, Solid Symbol After Submersion in Water

	Specimen Identification		Classification	DD	MC%
○	B-4	10.0	Clayey SILT	111.7	8.9
●	B-4	10.0	*UNDISTURBED*	112.9	12.7

Project: The Molasky Group of Companies - 5250 Ralston Street, Ventura

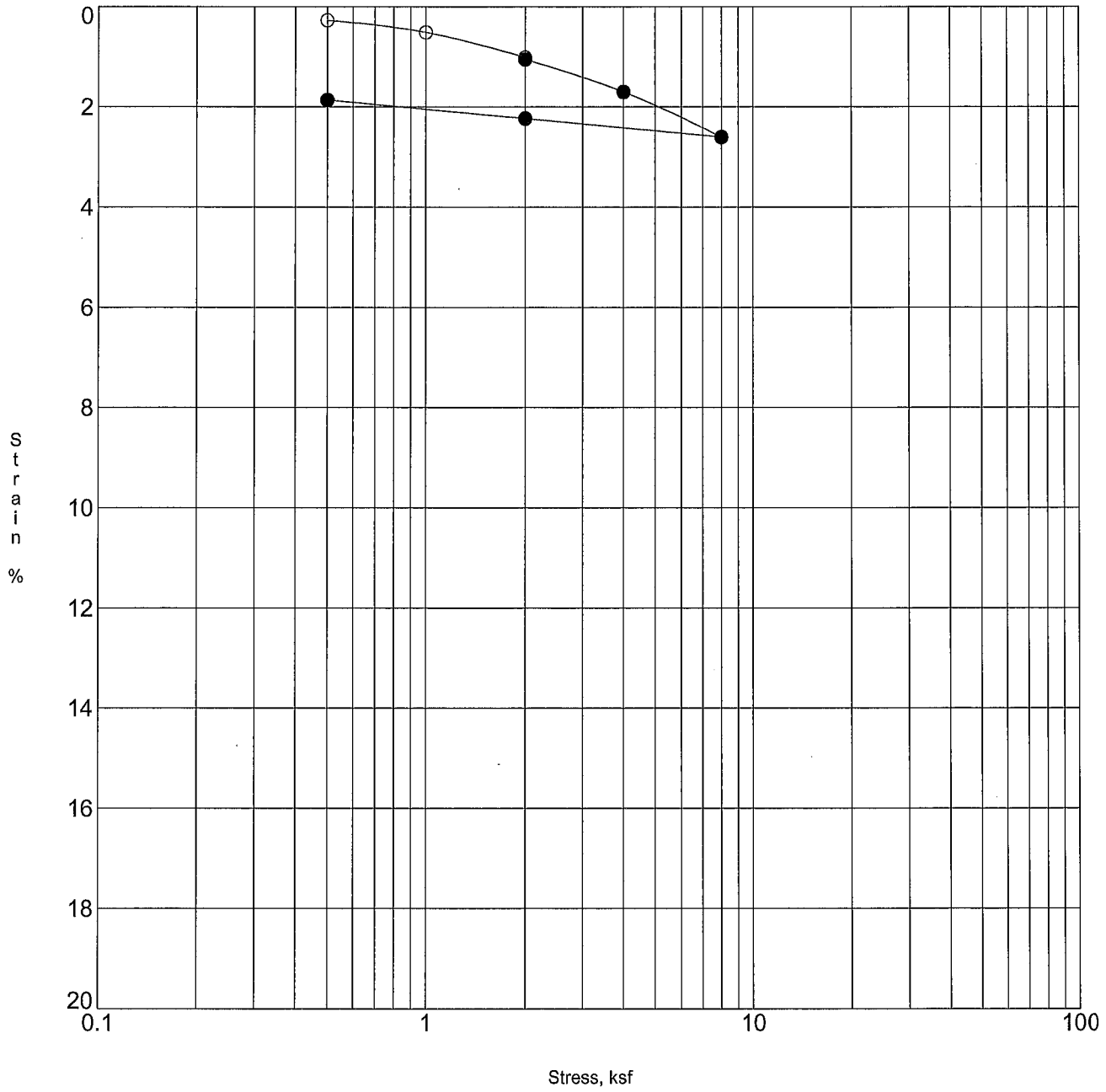
Client No.: 4994

Date: 12/6/19

Consolidation Test

Advanced Geotechnical Services, Inc.

Plate B- 9



Open Symbol At Field Moisture, Solid Symbol After Submersion in Water

	Specimen Identification		Classification	DD	MC%
○	B-5	15.0	Silty CLAY to Clayey SILT	106.7	14.0
●	B-5	15.0	*UNDISTURBED*	110.3	14.0

Project: The Molasky Group of Companies - 5250 Ralston Street, Ventura

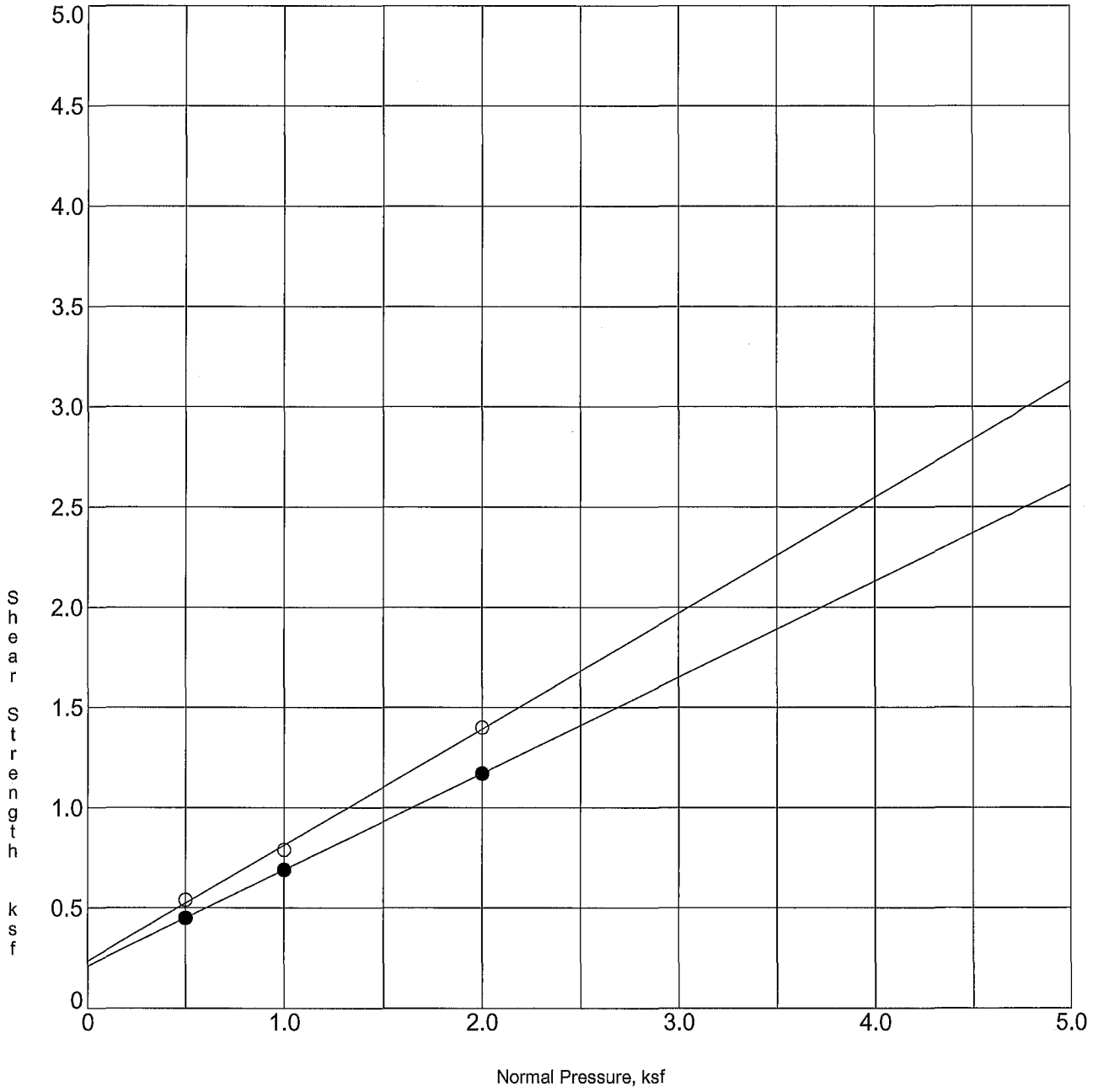
Client No. 4994

Date 12/6/19

Consolidation Test

Advanced Geotechnical Services, Inc.

Plate B- 10



○ - Peak Shear ● - Ultimate Shear △ - Residual Shear

	Specimen Identification		Classification	DD	MC%	c, ksf	phi
○	B-1	0.0	Silty CLAY to Clayey SILT	113.3	14.7	0.24	30
●	B-1	0.0	*REMOLD*	106.7	18.6	0.21	26

Project: The Molasky Group of Companies - 5250 Ralston Street, Ventura

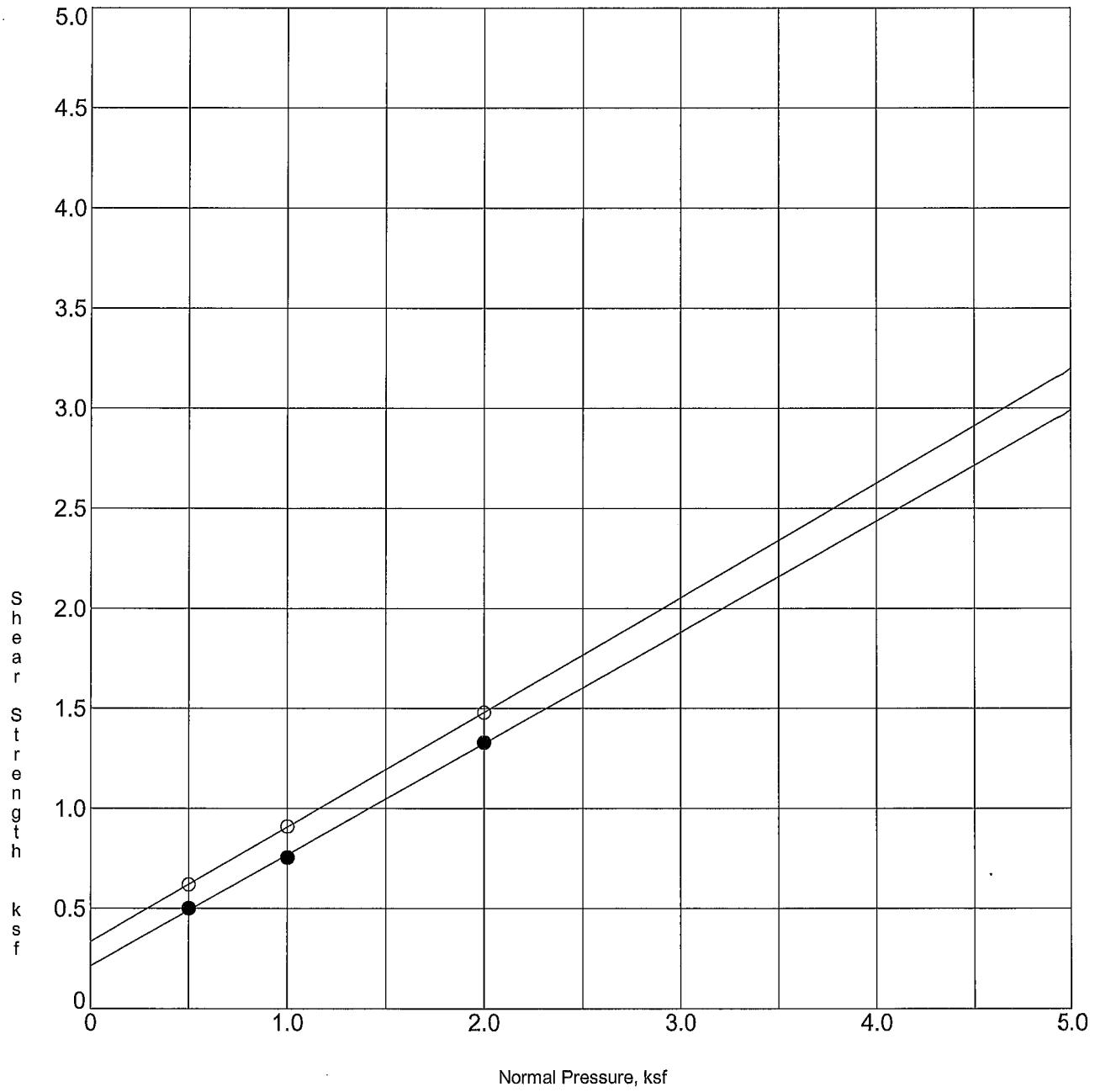
Client No.: 4994

Date: 12/6/19

Shear Test Diagram

Advanced Geotechnical Services, Inc.

Plate B- 11



○ - Peak Shear ● - Ultimate Shear △ - Residual Shear

	Specimen Identification		Classification	DD	MC%	c, ksf	phi
○	B-4	5.0	Silty CLAY	94.9	24.5	0.34	30
●	B-4	5.0	*UNDISTURBED*	94.9	30.9	0.21	29

Project: The Molasky Group of Companies - 5250 Ralston Street, Ventura

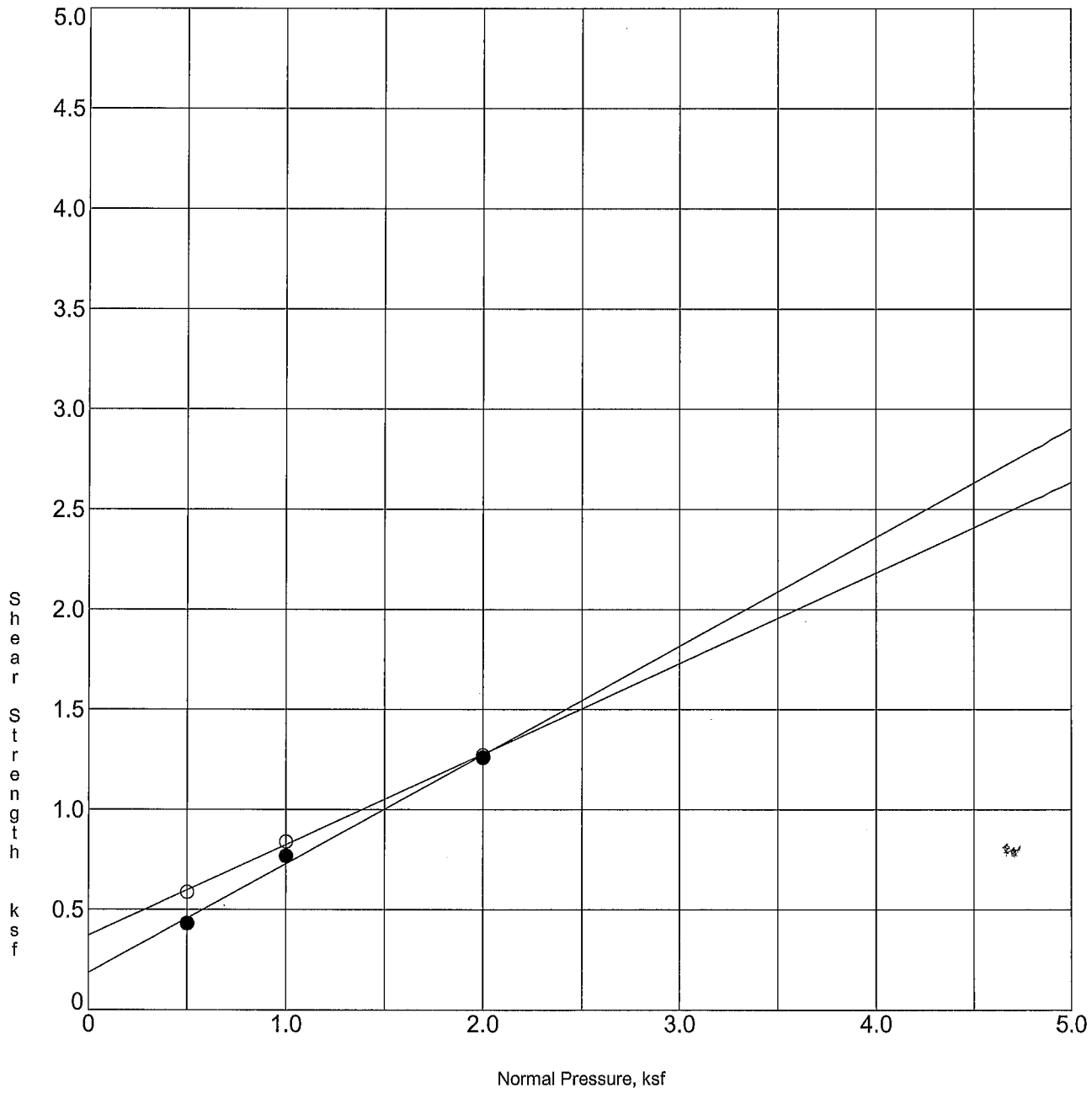
Client No.: 4994

Date: 12/6/19

Shear Test Diagram

Advanced Geotechnical Services, Inc.

Plate B- 12



○ - Peak Shear ● - Ultimate Shear △ - Residual Shear

	Specimen Identification		Classification	DD	MC%	c, ksf	phi
○	B-2	7.5	Clayey SILT	98.8	21.5	0.37	24
●	B-2	7.5	*UNDISTURBED*	98.8	24.4	0.19	29

Project: The Molasky Group of Companies - 5250 Ralston Street, Ventura

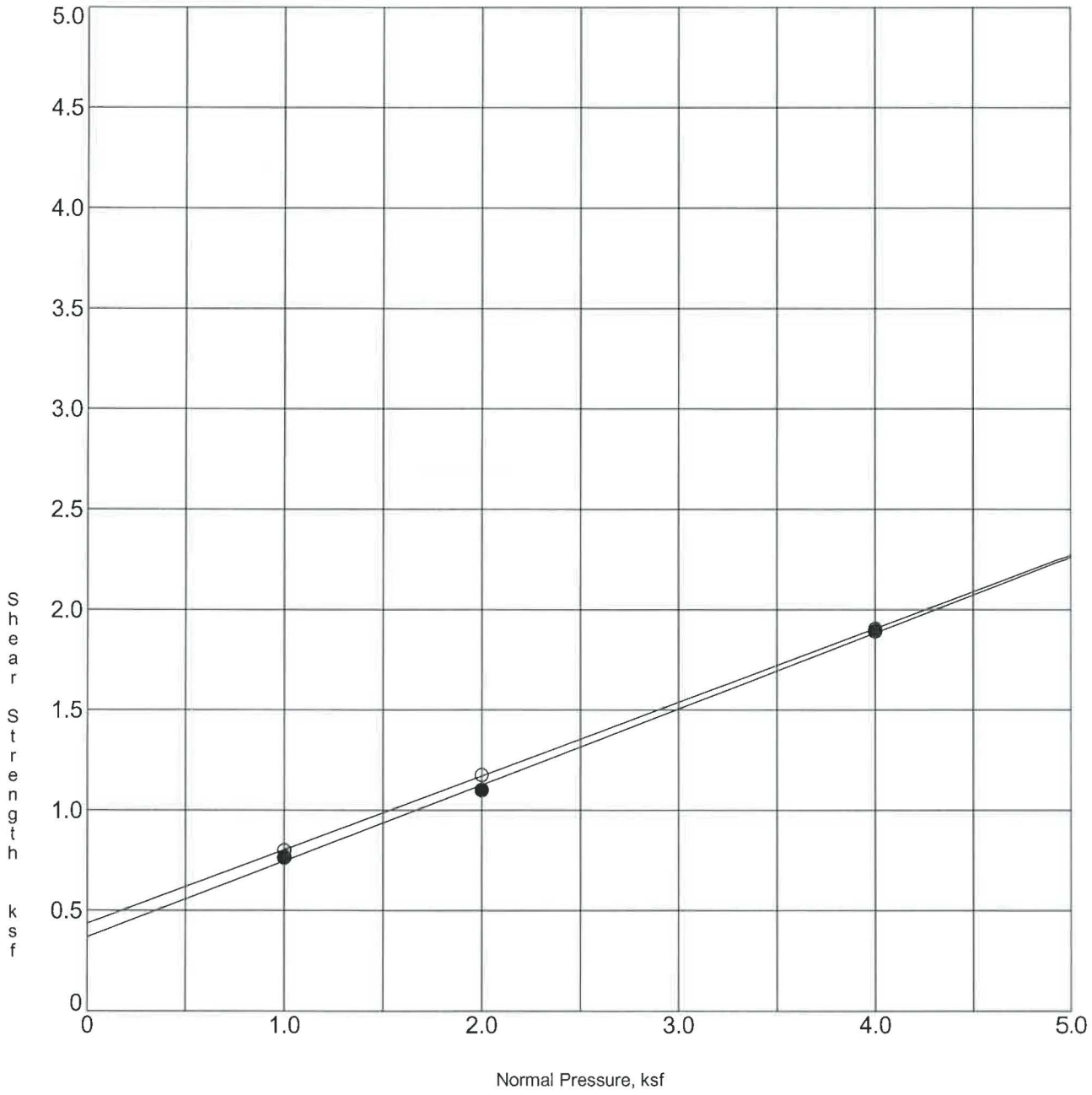
Client No.: 4994

Date: 12/6/19

Shear Test Diagram

Advanced Geotechnical Services, Inc.

Plate B- 13



○ - Peak Shear ● - Ultimate Shear △ - Residual Shear

	Specimen Identification		Classification	DD	MC%	c, ksf	phi
○	B-3	10.0	Sandy SILT	91.6	22.2	0.44	20
●	B-3	10.0	*UNDISTURBED*	91.6	28.9	0.37	21

Project: The Molasky Group of Companies - 5250 Ralston Street, Ventura

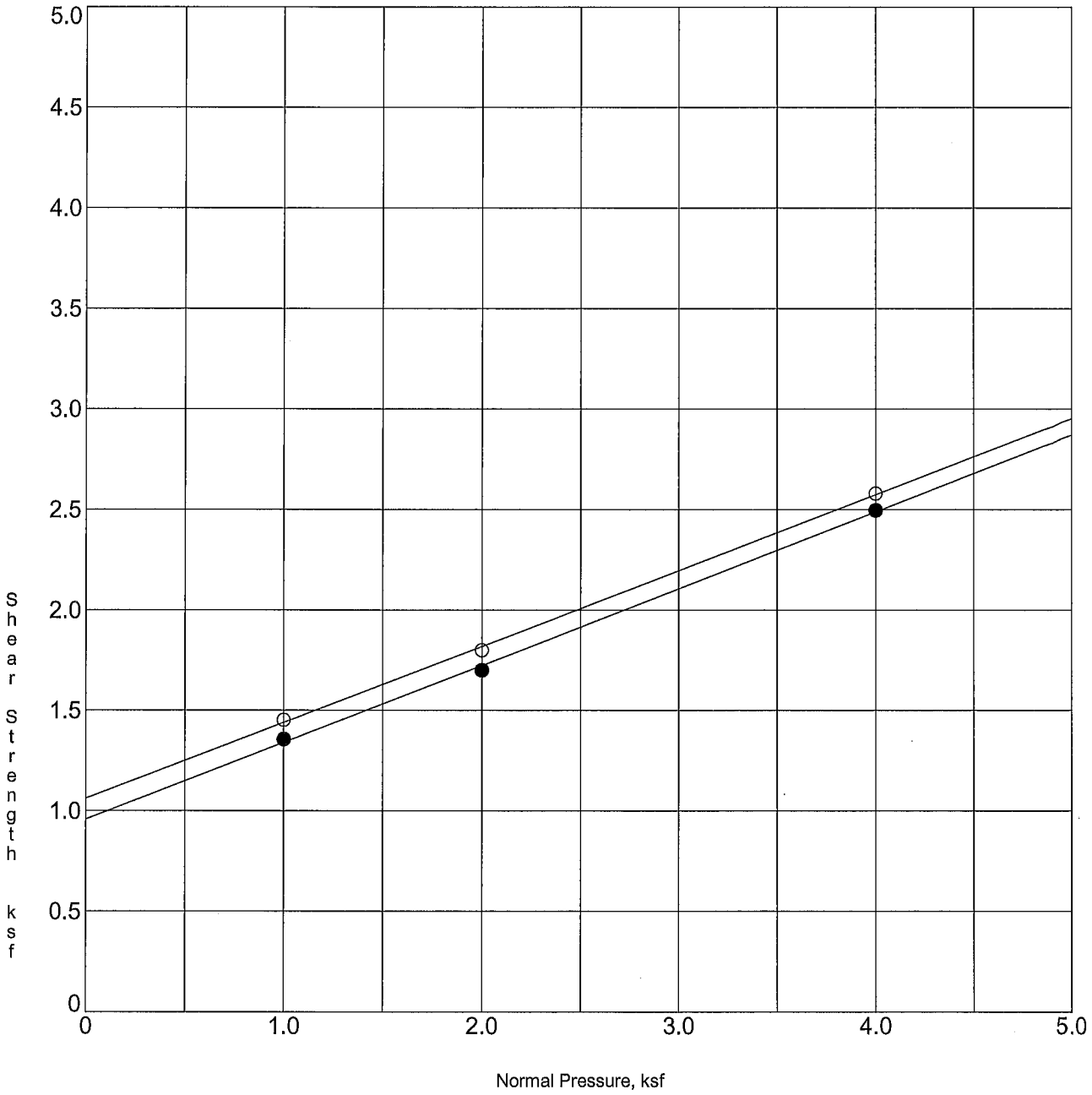
Client No.: 4994

Date: 12/6/19

Shear Test Diagram

Advanced Geotechnical Services, Inc.

Plate B- 14



○ - Peak Shear ● - Ultimate Shear △ - Residual Shear

	Specimen Identification		Classification	DD	MC%	c, ksf	phi
○	B-1	15.0	High PLASTICITY SILT	98.2	22.2	1.06	21
●	B-1	15.0	*UNDISTURBED*	98.2	26.9	0.96	21

Project: The Molasky Group of Companies - 5250 Ralston Street, Ventura

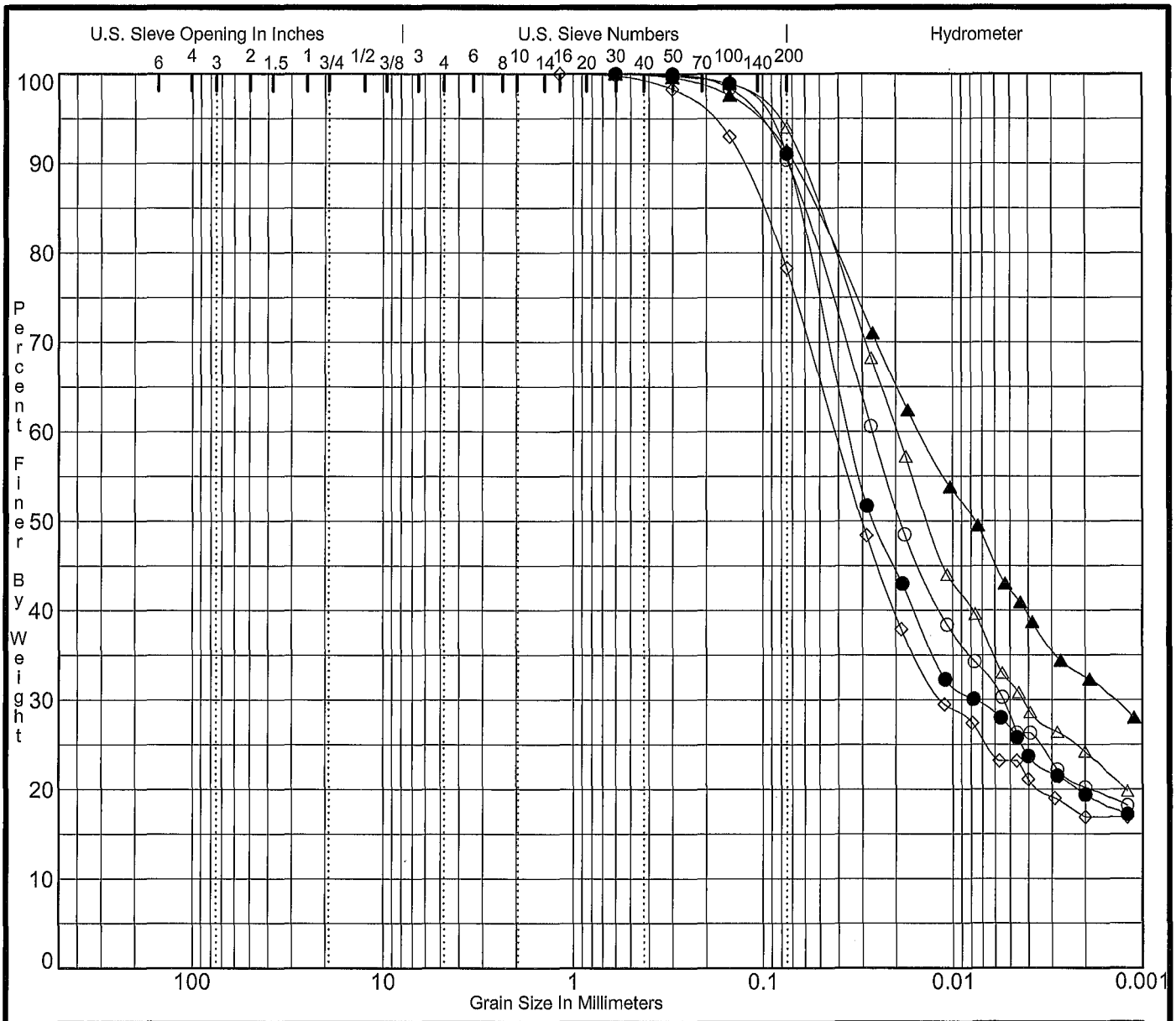
Client No.: 4994

Date: 12/6/19

Shear Test Diagram

Advanced Geotechnical Services, Inc.

Plate B- 15



Cobbles	Gravel		Sand			Silt Or Clay
	coarse	fine	coarse	medium	fine	

	Specimen Identification		Classification	MC%	LL	PL	PI	Cc	Cu
○	B-1	15.0	High Plasticity Clayey SILT		74	39	35		
●	B-1	20.0	Sandy to Silty CLAY						
△	B-1	25.0	Silty CLAY						
▲	B-1	35.0	Silty CLAY						
◇	B-1	40.0	Sandy to Silty CLAY						

	Specimen Identification		D100	D60	D30	D10	%Gravel	%Sand	%Silt	%Clay
○	B-1	15.0	0.30	0.03	0.005		0.0	9.6	62.2	28.2
●	B-1	20.0	0.60	0.03	0.008		0.0	8.9	64.4	26.7
△	B-1	25.0	0.30	0.02	0.004		0.0	6.0	62.0	32.0
▲	B-1	35.0	0.60	0.01	0.001		0.0	8.5	49.2	42.3
◇	B-1	40.0	1.18	0.04	0.011		0.0	21.7	55.1	23.2

Project: **The Molasky Group of Companies - 5250 Ralston Street, Ventura**

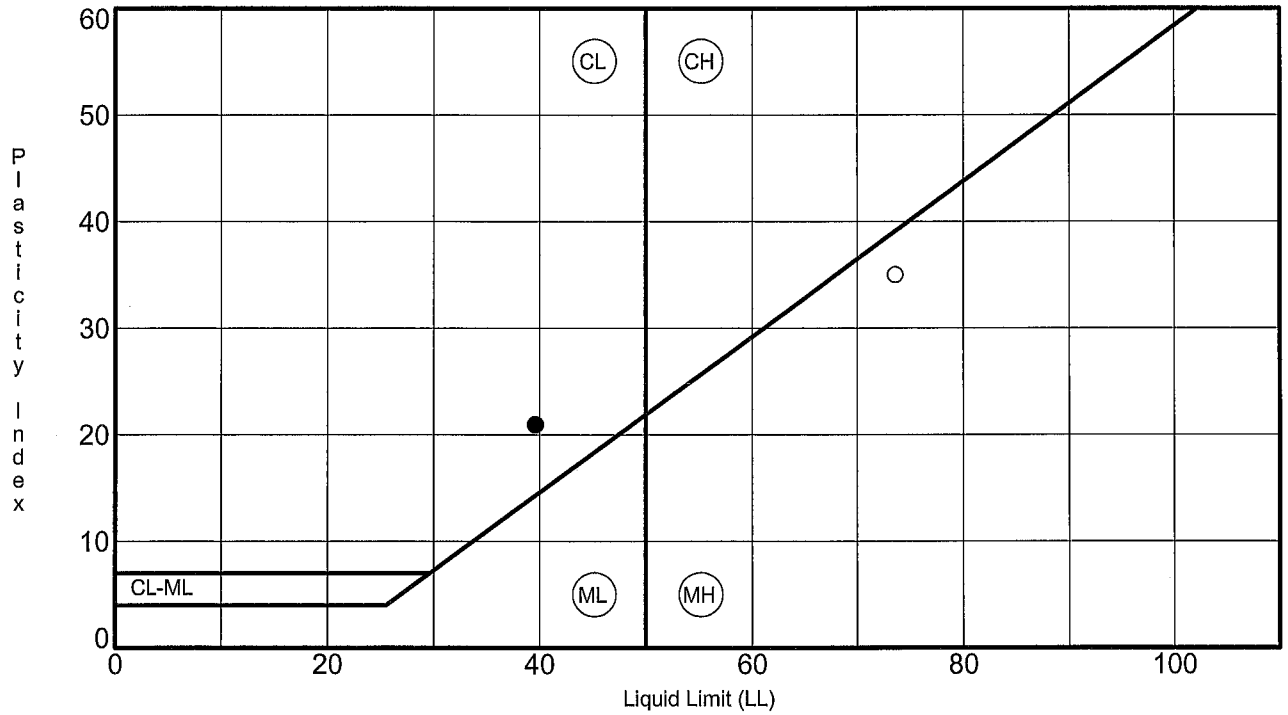
Client No.: **4994**

Date: **12/6/19**

Gradation Curves

Advanced Geotechnical Services, Inc.

Plate B- 16



	Specimen Identification		LL	PL	PI	Fines	Classification
○	**B-1**	**15.0**	**74**	**39**	**35**		
●	**B-1**	**30.0**	**40**	**19**	**21**		

Project **The Molasky Group of Companies - 5250 Ralston Street, Ventura**

Client No. **4994**

Date **12/6/19**

Atterberg Limits' Results

Advanced Geotechnical Services, Inc.

Plate B- 17

LABORATORY ANALYSIS RESULTS

Client: Advanced Geotechnical Services, Inc.
Project No: 4994
Project Name: Molaskey

AA Project No: A975176
Date Received: 11/11/19
Date Reported: 11/22/19

ANALYTICAL DATA SUMMARY

Analyte	Sample Name	Result	MRL	Units	Dilution	Prepared	Analyzed	Method
Chloride by Ion Chromatography								
Chloride	B-1@0'-5'	**19**	5.0	mg/kg	1	11/14/19	11/14/19	EPA 300.0
Chloride	B-3@0'-7'	**15**	5.0	mg/kg	1	11/14/19	11/14/19	EPA 300.0
General Chemistry Analyses								
pH	B-1@0'-5'	**7.8**	0.50	pH Units	1	11/14/19	11/14/19	9045C
Specific Conductance (EC)	B-1@0'-5'	**330**		umhos /cm	1	11/14/19	11/14/19	EPA 120.1
pH	B-3@0'-7'	**8.0**	0.50	pH Units	1	11/14/19	11/14/19	9045C
Specific Conductance (EC)	B-3@0'-7'	**730**		umhos /cm	1	11/14/19	11/14/19	EPA 120.1
Sulfate by Ion Chromatography								
Sulfate	B-1@0'-5'	**33**	5.0	mg/kg	1	11/14/19	11/14/19	EPA 300.0
Sulfate	B-3@0'-7'	**360**	10	mg/kg	2	11/14/19	11/14/19	EPA 300.0

Allen Aminian
QA/QC Manager

Pacific Materials Laboratory, Inc.
Serving Ventura County since 1963

November 14, 2019
Lab No. 35719-3
File No. 19-7059-3

AGS
5251 Verdugo Way, Suite L
Camarillo, CA 93012

SUBJECT: R-Value Testing
Samples Delivered to Laboratory

Gentlemen:

Pursuant to your request, R-Value testing was performed on soil samples delivered to our laboratory. R-Value testing was performed in accordance with California Test 301-F criteria. The test results follow:

R-VALUE RESULTS

PROJECT: Mulasky Group #4994
LOCATION: B-10 @ 0-5'
Soil Description: Grey/Brown Silty Clay

ITEM	1	2	3
Compaction Pressure – psi	150/200	150/175	100/125
Initial Moisture - %	18.3	18.3	18.3
Moisture at Compaction - %	21.3	22.2	24.2
Density – pcf	105.1	104.4	102.0
R-Value	12	7	4
Exudation Pressure	491	412	298
Expansion Pressure thickness ft.	0.73	0.60	0.30

Assigned R-Value: **4***

Footnote:
** Please verify R-value based upon expansion thickness (see California Test 301-F procedures)*Thank you for allowing

Pacific Materials Laboratory, Inc. to be of service. If we may be of further service regarding this or other geotechnical issues, please do not hesitate to call (805) 482-9801, fax (805) 445-6551 or email at pacificmaterialslab@msn.com.

Respectfully Submitted,
PACIFIC MATERIALS LABORATORY, INC.

Douglas C. Papay, CE 29,565
President

REGISTERED PROFESSIONAL ENGINEER
DOUGLAS C. PAPAY
CE 29565
Exp: 3/31/21
CIVIL
STATE OF CALIFORNIA

DCP:gp
cc: Addressee (Email)

Appendix C

Seismicity Study

Ventura VA Outpatient Clinic - ASCE 7-10

Latitude, Longitude: 34.258534, -119.220539

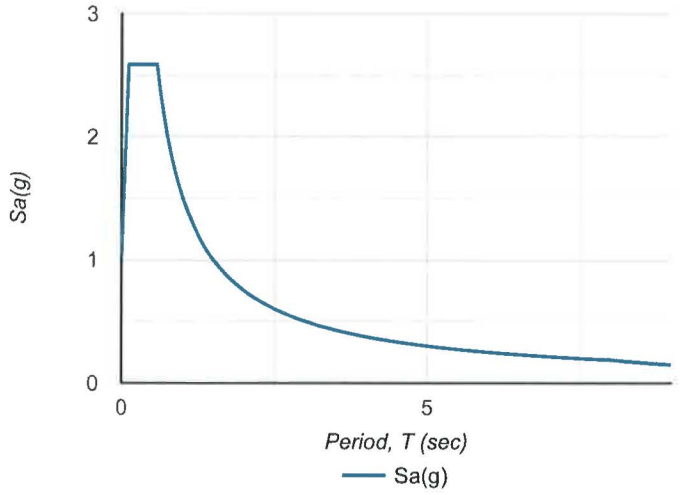


Date	11/26/2019, 2:32:47 PM
Design Code Reference Document	ASCE7-10
Risk Category	II
Site Class	D - Stiff Soil

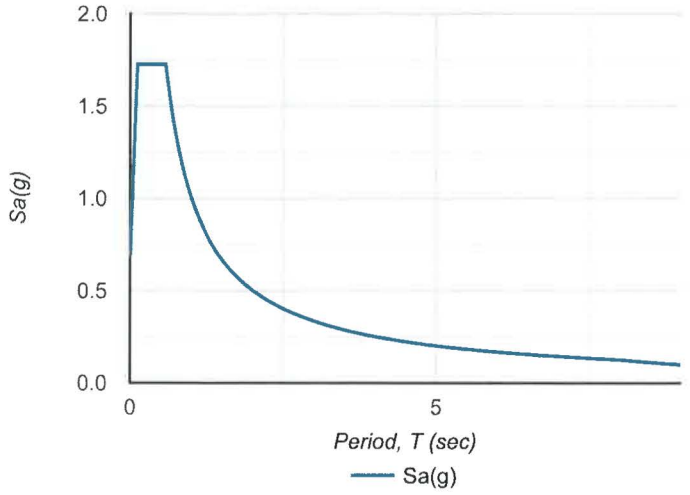
Type	Value	Description
S_S	2.587	MCE_R ground motion. (for 0.2 second period)
S_1	1.006	MCE_R ground motion. (for 1.0s period)
S_{MS}	2.587	Site-modified spectral acceleration value
S_{M1}	1.508	Site-modified spectral acceleration value
S_{DS}	1.725	Numeric seismic design value at 0.2 second SA
S_{D1}	1.006	Numeric seismic design value at 1.0 second SA

Type	Value	Description
SDC	E	Seismic design category
F_a	1	Site amplification factor at 0.2 second
F_v	1.5	Site amplification factor at 1.0 second
PGA	0.998	MCE_G peak ground acceleration
F_{PGA}	1	Site amplification factor at PGA
PGA_M	0.998	Site modified peak ground acceleration
T_L	8	Long-period transition period in seconds
SsRT	2.587	Probabilistic risk-targeted ground motion. (0.2 second)
SsUH	2.807	Factored uniform-hazard (2% probability of exceedance in 50 years) spectral acceleration
SsD	2.614	Factored deterministic acceleration value. (0.2 second)
S1RT	1.006	Probabilistic risk-targeted ground motion. (1.0 second)
S1UH	1.107	Factored uniform-hazard (2% probability of exceedance in 50 years) spectral acceleration.
S1D	1.055	Factored deterministic acceleration value. (1.0 second)
PGAd	0.998	Factored deterministic acceleration value. (Peak Ground Acceleration)
C_{RS}	0.922	Mapped value of the risk coefficient at short periods
C_{R1}	0.908	Mapped value of the risk coefficient at a period of 1 s

MCER Response Spectrum



Design Response Spectrum



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Ventura VA Outpatient Clinic - ASCE 7-16

Latitude, Longitude: 34.258534, -119.220539



Date	11/26/2019, 2:33:38 PM
Design Code Reference Document	ASCE7-16
Risk Category	II
Site Class	D - Stiff Soil

Type	Value	Description
S_S	1.964	MCE_R ground motion. (for 0.2 second period)
S_1	0.737	MCE_R ground motion. (for 1.0s period)
S_{MS}	1.964	Site-modified spectral acceleration value
S_{M1}	null -See Section 11.4.8	Site-modified spectral acceleration value
S_{DS}	1.309	Numeric seismic design value at 0.2 second SA
S_{D1}	null -See Section 11.4.8	Numeric seismic design value at 1.0 second SA

Type	Value	Description
SDC	null -See Section 11.4.8	Seismic design category
F_a	1	Site amplification factor at 0.2 second
F_v	null -See Section 11.4.8	Site amplification factor at 1.0 second
PGA	0.865	MCE_G peak ground acceleration
F_{PGA}	1.1	Site amplification factor at PGA
PGA_M	0.952	Site modified peak ground acceleration
T_L	8	Long-period transition period in seconds
SsRT	1.964	Probabilistic risk-targeted ground motion. (0.2 second)
SsUH	2.229	Factored uniform-hazard (2% probability of exceedance in 50 years) spectral acceleration
SsD	2.558	Factored deterministic acceleration value. (0.2 second)
S1RT	0.737	Probabilistic risk-targeted ground motion. (1.0 second)
S1UH	0.835	Factored uniform-hazard (2% probability of exceedance in 50 years) spectral acceleration.
S1D	0.809	Factored deterministic acceleration value. (1.0 second)
PGAd	1.018	Factored deterministic acceleration value. (Peak Ground Acceleration)
C_{RS}	0.881	Mapped value of the risk coefficient at short periods
C_{R1}	0.883	Mapped value of the risk coefficient at a period of 1 s

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Appendix D

Liquefaction Evaluation

Client Number	4994	Client Name	Molasky
Date Drilled	11/6/19	Boring	B-1

a_{max}/g	0.998		N Adjustments - (for Calif. Sampler)	0.667	Field Groundwater Depth (ft)	20	(Current)
Magnitude	6.90		N Adjustments - Hole Diameter	1.15	Method (S = SPT)	S	(Ring Sample Data Converted to Equiv. SPT)
Groundwater Depth (ft)	11.0	(Historic High)	N Adjustments - Energy (from Calibration)	1.33	Unit Weight of Water (kcf)	0.0624	
Reference Pressure, $p_{a\,(liq)}$	2.1164		Nc	10.08			
Reference Pressure, $p_{a\,(dry)}$	1.0582						

NL = Not Susceptible to Liquefaction

References

Seed, H. B., Tokimatsu, K., (1985), *Influence of SPT Procedures in Soil Liquefaction Resistance Evaluations* , Journal of Geotechnical Engineering, ASCE, Vol. 111, No. 12, pp. 1425 - 1445.

Yould, T. L. and Idriss, I. M. (1997), *Summary Report* , **Proceedings of the NCEER Workshop on Evaluation of liquefaction Resistance of Soils**

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Youd, T. L. et al (2001), *Summary Report on Evaluation of Liquefaction Resistance of Soils, Journal of Geotechnical and Geoenvironmental Engineering,* ASCE, Vol. 127, No. 10, pp. 817-833.

B-1 Liquefaction Evaluation

Depth, Feet	Total Unit Weight, γ_t	Overburden Pressure, σ_v	LIQ Effective Overburden Pressure, σ_v'	Field Effective Overburden Pressure, σ_v'	C_N	r_d	$CSR_{M=7.5}$	Soil Type*	% Fines	N for California Sampler **	$(N_1)_{60}$	Adjusted for Fines Content $(N_1)_{60}$	Rod Length Adjust	K_s	$CRR_{M=7.5}$	Safety Factor, SPT Method	Volumetric Strain	Layer Settlement, (inches)	Cumulative Liquefaction Settlement, (inches)
0.0		0.00	0.00	0.00															
3.5	0.135	0.47	0.47	0.47	1.70	0.99	0.521			50.0	65.0	65.0	0.75	1.00	5.000	Above GWT	0.000	0.000	0.000
7.0		0.95	0.95	0.95															
8.0	0.135	1.08	1.08	1.08	1.40	0.98	0.516			30.0	32.1	32.1	0.75	1.00	0.409	Above GWT	0.000	0.000	0.000
9.0		1.22	1.22	1.22															
10.0	0.135	1.35	1.35	1.35	1.25	0.98	0.513			40.0	43.4	43.4	0.85	1.00	5.000	Above GWT	0.000	0.000	0.000
11.0		1.49	1.49	1.49															
13.0	0.135	1.76	1.63	1.76	1.10	0.97	0.549	C	90.4	27.0	25.7	35.8	0.85	1.00	5.000	NL	0.000	0.000	0.000
15.0		2.03	1.78	2.03															
17.0	0.135	2.30	1.92	2.30	0.96	0.96	0.578	C	90.4	23.0	21.4	30.7	0.95	1.00	0.195	NL	0.000	0.000	0.000
19.0		2.57	2.07	2.57															
22.0	0.135	2.97	2.28	2.85	0.86	0.95	0.613	C	91.1	36.0	30.1	41.1	0.95	1.00	5.000	NL	0.000	0.000	0.000
25.0		3.38	2.50	3.06															
27.5	0.135	3.71	2.68	3.24	0.81	0.93	0.634	C	94.0	14.0	11.5	18.8	1.00	0.97	0.200	NL	0.000	0.000	0.000
30.0		4.05	2.86	3.43															
32.5	0.135	4.39	3.05	3.61	0.77	0.91	0.643	C	91.5	19.0	14.8	22.8	1.00	0.96	0.242	NL	0.000	0.000	0.000
35.0		4.73	3.23	3.79															
37.5	0.135	5.06	3.41	3.97	0.73	0.87	0.645	C	91.5	9.0	6.7	13.0	1.00	0.94	0.134	NL	0.000	0.000	0.000
40.0		5.40	3.59	4.15															
42.5	0.135	5.74	3.77	4.33	0.70	0.83	0.642	C	78.3	16.0	11.4	18.7	1.00	0.92	0.187	NL	0.000	0.000	0.000
45.0		6.08	3.95	4.52															
48.3	0.135	6.51	4.19	4.75	0.67	0.77	0.635	C	78.3	10.0	6.8	13.2	1.00	0.90	0.130	NL	0.000	0.000	0.000
51.5		6.95	4.43	4.99															

* Note: C = Clayey soils not susceptible to liquefaction

Total, Inches = 0.00

** Note: Cal Blow Counts of 50 assumed for the future newly placed compacted fill in the upper 7 feet

Plate D-1

Appendix E
References

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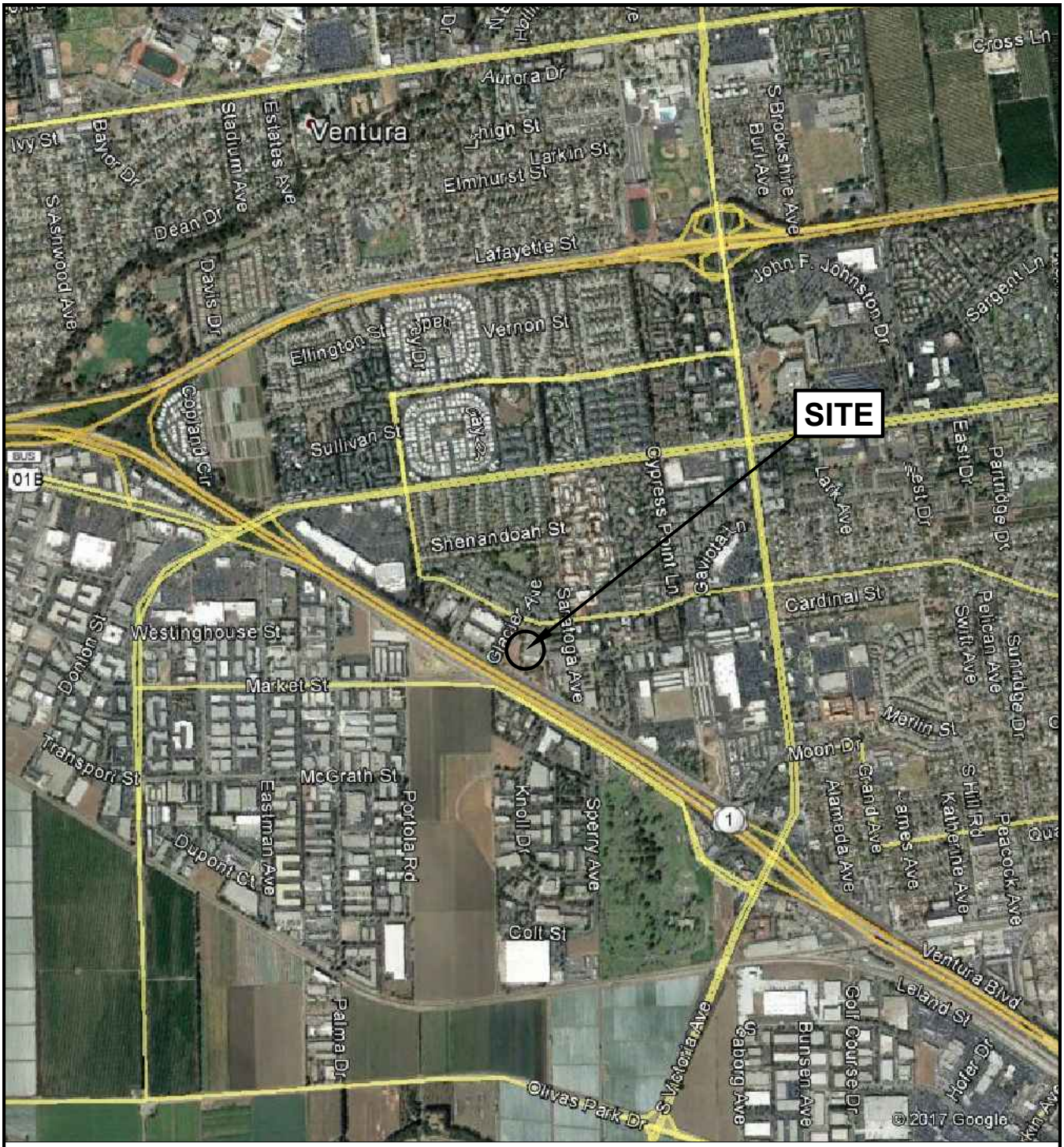
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Appendix F

Report Figures and Plates

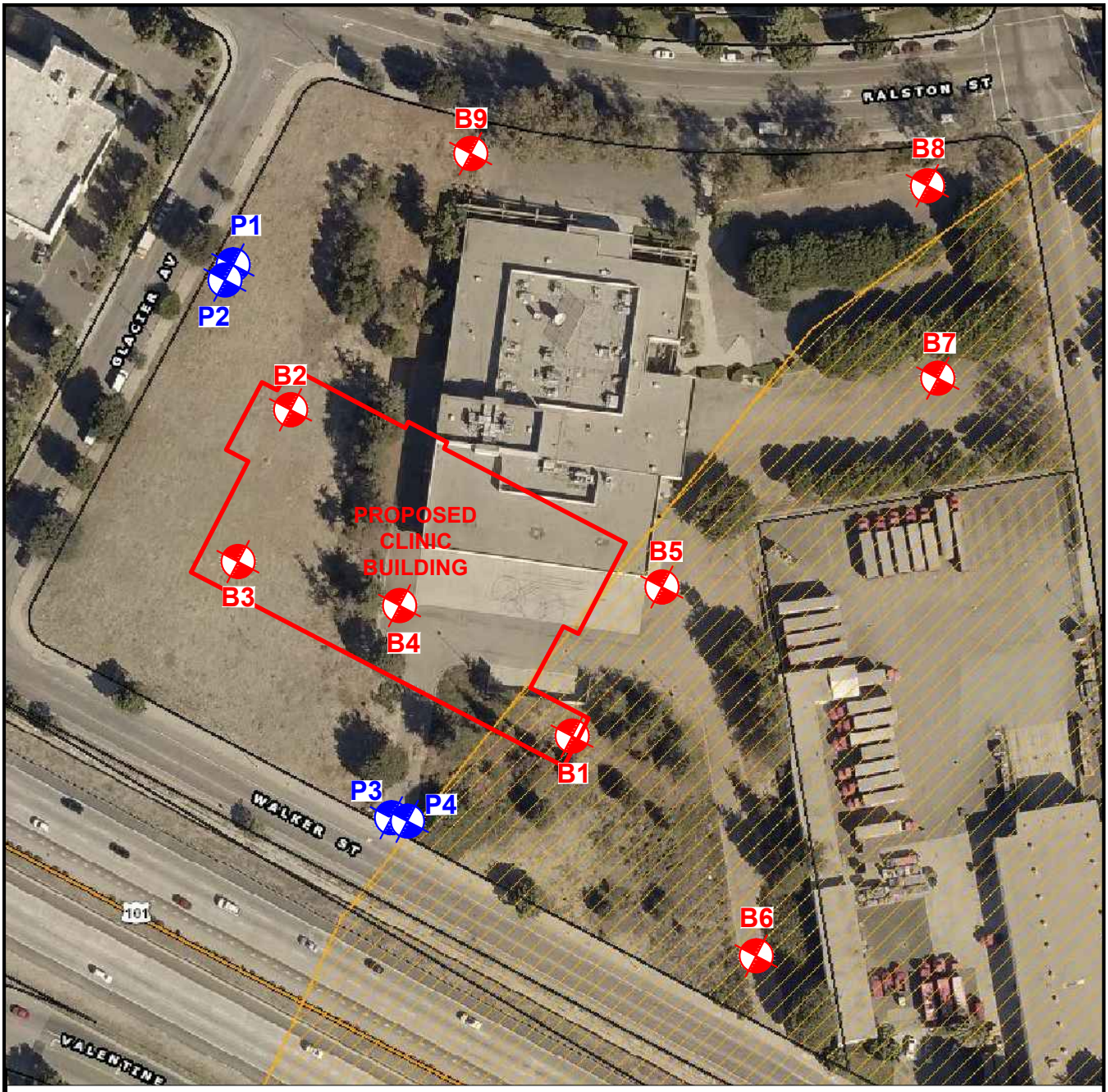


Reference: Google Earth 2019

N

No Scale

Advanced Geotechnical Services	**SITE LOCATION MAP**	**The Molasky Group** **5250 Ralston Street** **Ventura, California**	
		Client # 4994 **Report # 10422**	**FIGURE 1**



Reference: http://gis.ventura.org/countyview/

EXPLANATION

B10 Approximate Location of Exploratory Boring

P4 Approximate Location of Percolation Boring

N

Scale: 1" = 120'

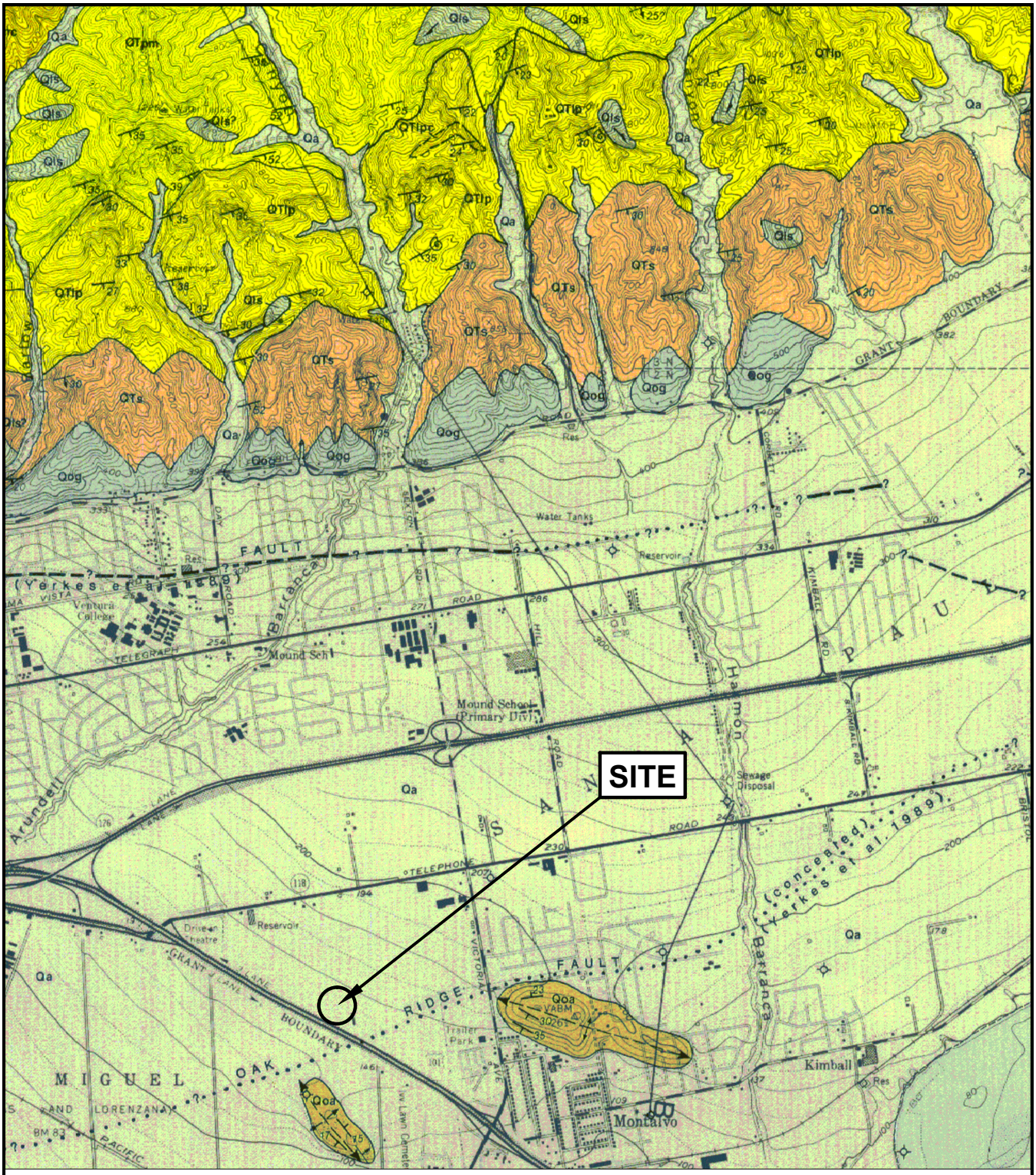
Advanced Geotechnical Services

EXISTING SITE PLAN

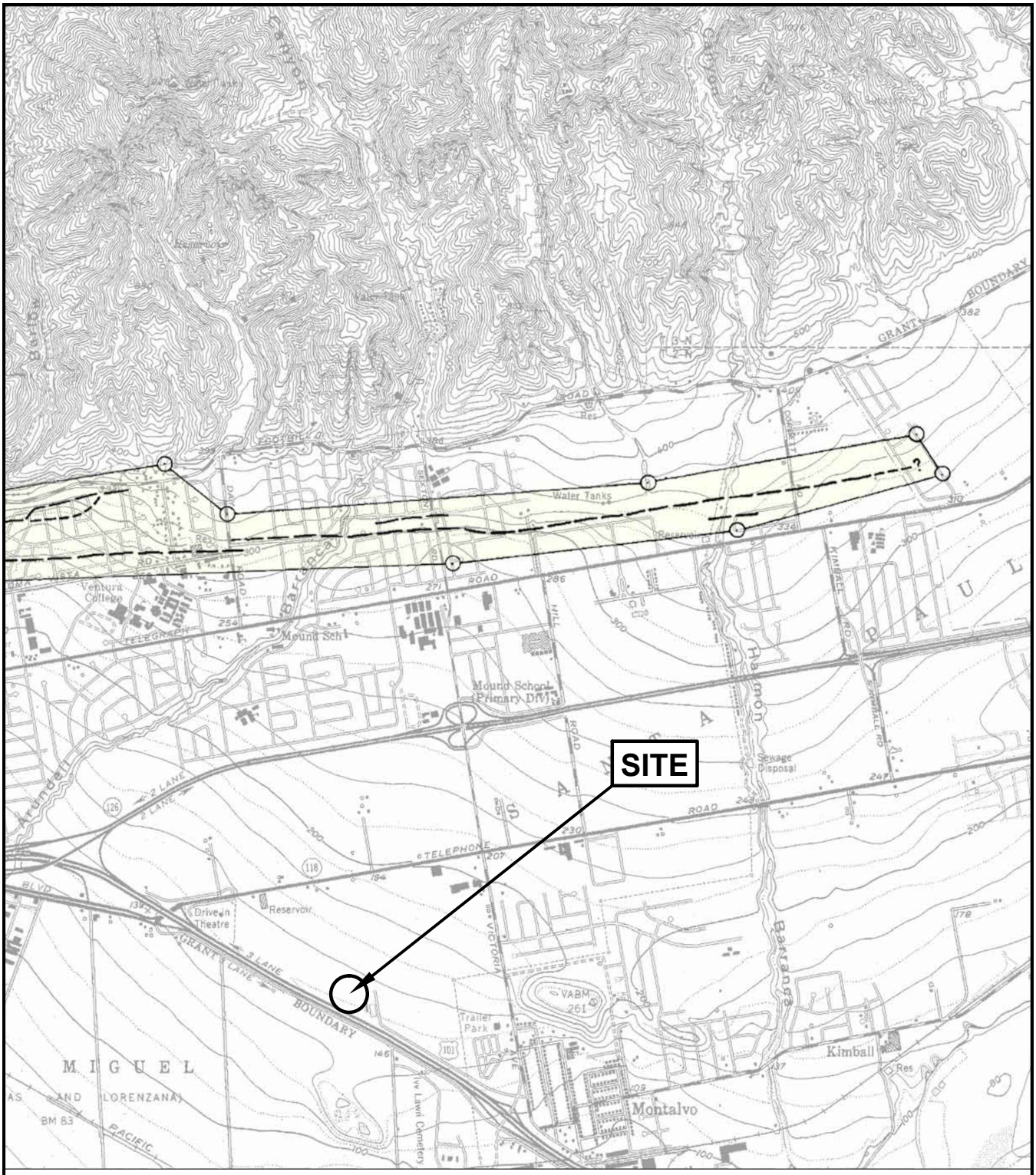
The Molasky Group
5250 Ralston Street
Ventura, California

Client # 4994
Report # 10422

FIGURE 2



Advanced Geotechnical Services	**REGIONAL GEOLOGICAL MAP**	The Molasky Group 5250 Ralston Street Ventura, California
		Client # 4994 Report # 10422 — **FIGURE 3**



Reference: CDMG, 1978, Special Studies Zones - Saticoy Quadrangle

N

Scale: 1" = ½ mile

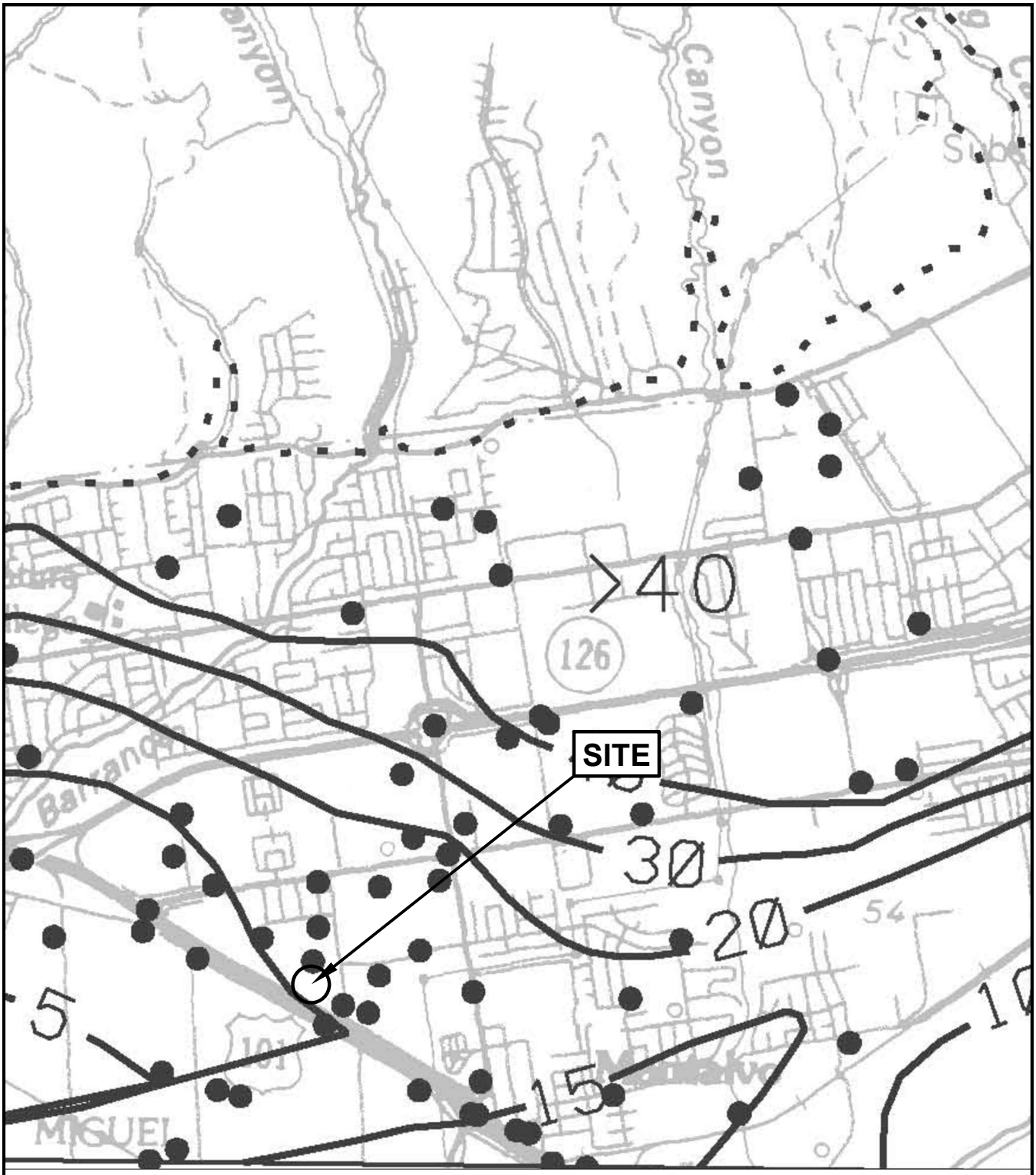
Advanced Geotechnical Services

EARTHQUAKE FAULT ZONES MAP

The Molasky Group
5250 Ralston Street
Ventura, California

Client # 4994
Report # 10422

FIGURE 4



Reference: CGS, 2003, SHZR 066

N

Scale: 1" = $\frac{1}{2}$ mile

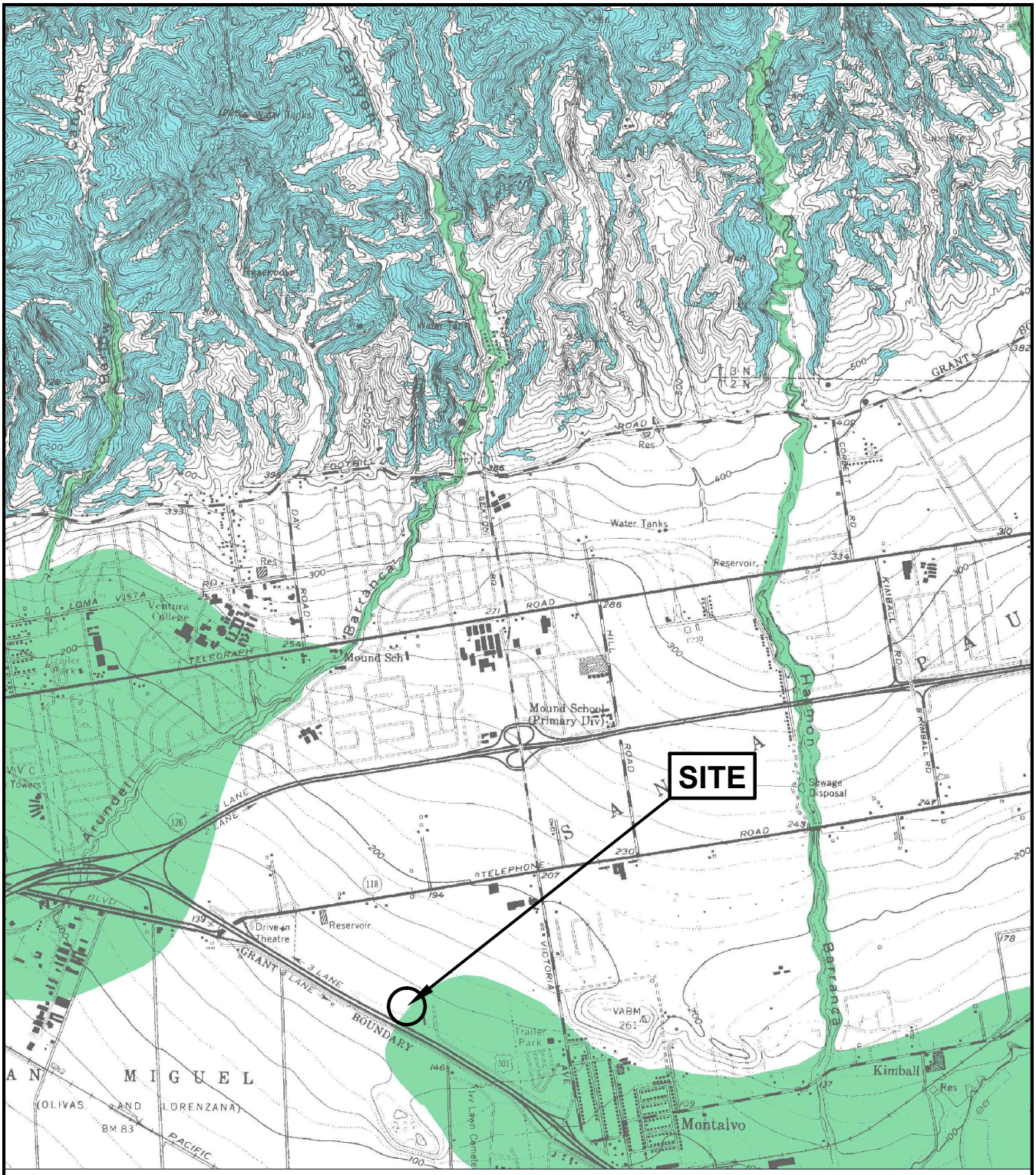
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DEPTH TO HISTORICALLY HIGH GROUNDWATER MAP

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Ventura, California

Client # 4994
Report # 10422

FIGURE 5



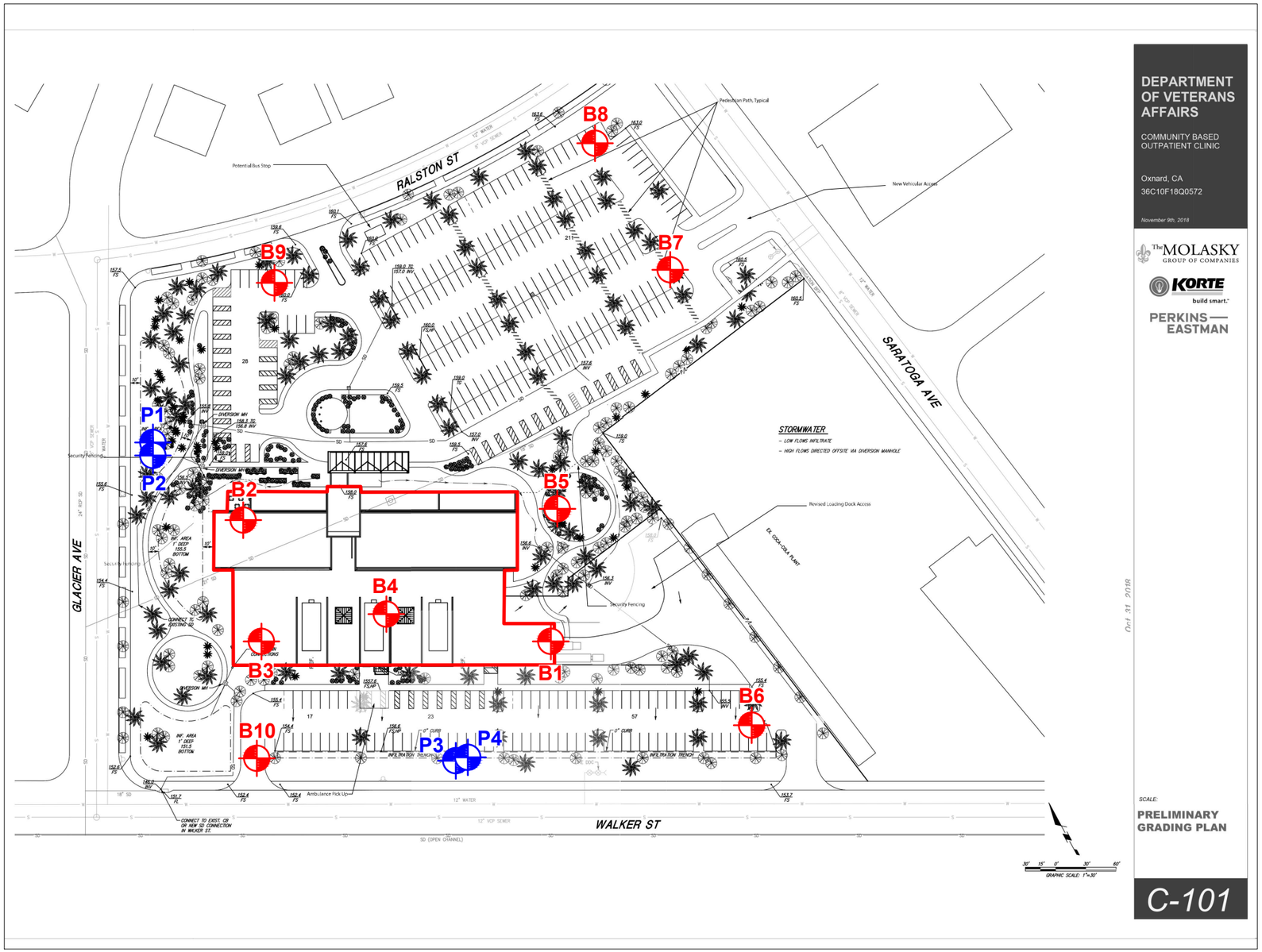
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SEISMIC HAZARD ZONES MAP

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Ventura, California

Client # 4994
Report # 10422

FIGURE 6



PROPOSED SITE PLAN

120' 0' 120' 240'

SCALE: 1" = 120'

EXPLANATION

B10 — Approximate Location of Geotechnical Boring

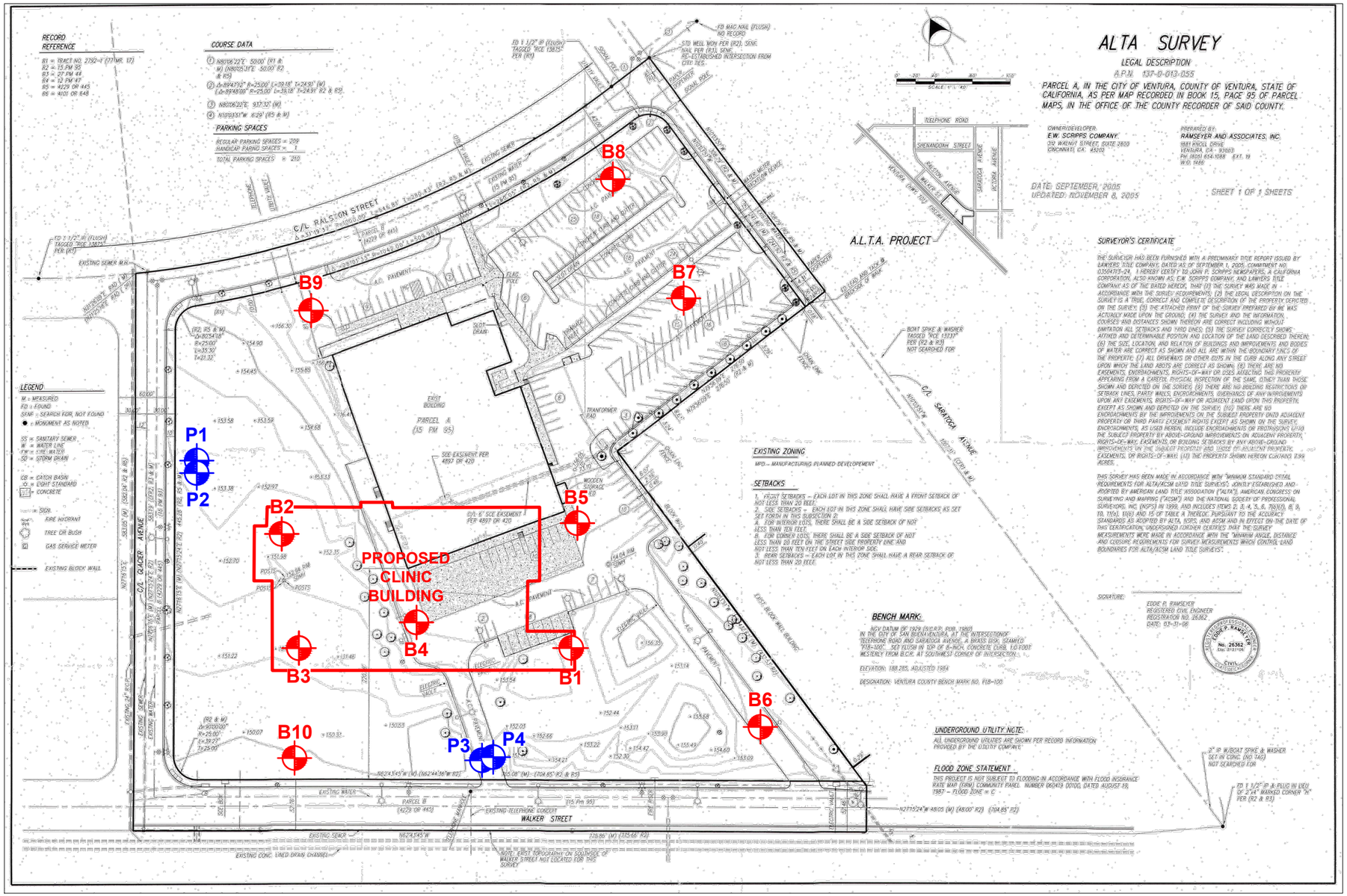
P4 — Approximate Location of Field Percolation Test Boring

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Geotechnical Engineering Study
Proposed Veterans Affairs Clinic
5250 Ralston Street
Ventura, California

Client No.	4994	PLATE 1
Report No.	10422	
Date	12/6/2019	
Drawing No.	10422cn4994	



ALTA SURVEY MAP

120' 0' 120' 240'

SCALE: 1" = 120'

EXPLANATION

B10 — Approximate Location of Geotechnical Boring

P4 — Approximate Location of Field Percolation Test Boring

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Client No.	4994	PLATE 2
Report No.	10422	
Date	12/6/2019	
Drawing No.	10422cn4994	