NMG Geotechnical, Inc.

March 3, 2017

Project No. 96079-03

To: C.J. Segerstrom and Sons
3315 Fairview Road
Costa Mesa, California 92626

Attention: Mr. Jeffery M. Reese

Subject: Geotechnical Exploration and Design for Proposed Building Replacing Existing Souplantation, 1555 Adams Avenue, Mesa Verde Shopping Center, Costa Mesa, California

EXECUTIVE SUMMARY

NMG Geotechnical, Inc. (NMG) has performed geotechnical exploration for a new proposed building at the Mesa Verde Center, in the city of Costa Mesa, California. The purpose of this exploration was to evaluate the existing subsurface conditions and perform soil characterization to provide geotechnical findings and recommendations for the project. Based on the initial information provided and the conceptual site plan prepared by Fuscoe Engineering, the project will consist of demolition of the existing Souplantation building and construction of a new building. The project will also include minor changes to the adjacent parking lot area. The geotechnical exploration performed included a total of five borings located around the existing building. Two of the borings were used to perform deep percolation testing and determine if infiltration is feasible. Laboratory testing was also performed on selected soil samples for soil characterization and to evaluate the engineering soil properties. This report summarizes our findings, conclusion and provides our recommendations for design, grading and construction.

The subject site is underlain by artificial fill and marine terrace deposits that consist predominantly of fine-grained clays and sandy clays in the upper 10 to 20 feet. The onsite clays have low to high plasticity and moderate expansion potential. Below the upper clays, there is a zone of relatively clean fine- to coarse-grained sand. The preliminary field testing and findings indicate that the sand layer is suitable for infiltration. Below 25 to 30 feet, the soils vary from sands to inter-layered sand, sandy clay and clay. No groundwater was encountered in our borings drilled to depths of 51.5 feet below ground surface (bgs). We anticipate the groundwater level will remain deep in this area based on review of published groundwater data.

Based on our field exploration and findings, we conclude that the proposed development is feasible from a geotechnical viewpoint provided it is designed, graded, and constructed in accordance with the recommendations presented in this report and the city of Costa Mesa requirements. Some remedial grading will be required to provide suitable foundation soil

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conditions for the new proposed building and improvements. Environmental evaluation of onsite soil or imported soil is not a part of this report and is the purview of others.

If you have any questions regarding this report, please contact our office. We appreciate the opportunity to provide our services.

Respectfully submitted,

NMG GEOTECHNICAL, INC.

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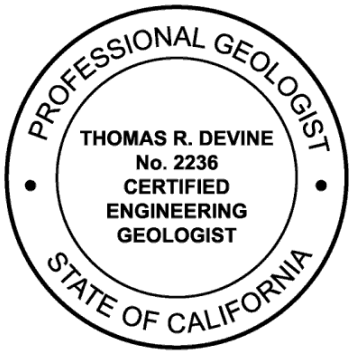

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1.0 INTRODUCTION

1.1 Purpose and Scope of Work

In accordance with your request, NMG Geotechnical, Inc. (NMG) has conducted geotechnical exploration and testing to characterize the existing subsurface soil at the subject site in the city of Costa Mesa, California. The subject site consists of the existing Souplantation building at the Mesa Verde Center that will be demolished and replaced with a new proposed building. The purpose of our study was to evaluate the geotechnical conditions in light of the proposed development in order to provide recommendations for the project design, grading, and construction. This study is based on the conceptual plan provided by Fuscoe Engineering on January 27, 2017. Pertinent geotechnical data from prior studies at or near the Mesa Verde Center was also utilized for this study.

Our scope of work included the following:

- Acquisition and review of available geologic maps and data for the subject site and surrounding area. This includes geotechnical data from a prior NMG study at the Mesa Verde Center, as well as other nearby projects by others. A list of references is included in Appendix A.
- Notification and coordination with Underground Service Alert to identify and locate any underground utilities was also performed prior to subsurface exploration.
- Drilling, sampling, and logging of five hollow-stem auger borings (H-1 through H-3, P-1, and P-2) to depths of 26.5 to 51.5 feet. Borings P-1 and P-2 were utilized for the deep percolation testing. The approximate location of our borings is provided on the Boring Location Map (Figure 2). Boring logs are included in Appendix B and the percolation test data sheets are included in Appendix C. Logs from prior studies at the site are included in Appendix B.
- Laboratory testing of selected samples for soil characterization included in-situ moisture and density, grain-size distribution, Atterberg limits, consolidation, shear strength, maximum dry density and optimum moisture content, expansion index, and soil corrosivity. A soil corrosivity report and corrosion protection recommendations were also prepared by HDR Schiff. Laboratory test results from this and prior studies are included in Appendix D.
- Evaluation of faulting, seismicity and liquefaction potential in accordance with the 2016 California Building Code (CBC).
- Geotechnical evaluation and analysis of the compiled data with respect to the anticipated development, determination of remedial grading measures, soil engineering parameters for foundation design, and infiltration.
- Preparation of this report, including our findings, conclusions and recommendations for the subject project.

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1.2 Site Description and Existing Conditions

The subject site is the existing Souplantation building within the Mesa Verde Center in the city of Costa Mesa, California (Figure 1). The site is approximately 3/4 acres and is located southeast of the intersection of Adams Avenue and Royal Palms Street. The elevations at the center varying from 59 to 61 feet above mean sea level (msl). There are existing parking lots, interior drives and concrete walks adjacent to the building. The existing landscaping around the buildings includes a few trees, shrubs and lawn areas, along with some planters that have various smaller plants and flowers.

Electrical, sanitary sewer, storm drain, telephone, gas, and domestic water pipelines are located at the site near the existing building and in the adjacent roadways. The location and/or depth of these and other utilities are unknown at this time.

1.3 Prior Studies

NMG has performed previous services in the Mesa Verde Shopping Center that includes geotechnical investigation, design and construction services for the CVS (Sav-On) building, Albertsons (Vons)/Bank of America expansion, and for parking lot rehabilitation. Additionally, we conducted a limited geotechnical investigation of the subgrade soils in the former ice skating rink. Our previous data indicates a total of 29 borings have been drilled throughout the site, ranging from 6.5 to 51.5 feet deep. A separate geotechnical study was conducted in June 2011 by others for the nearby Azulón at Mesa Verde senior apartment community. The prior study included nine borings and had three percolation tests. The prior geotechnical studies are referenced in Appendix A.

1.4 Proposed Development and Improvements

Based on the preliminary information provided and the conceptual site plan, a new Building A consisting of three shops (Units A-100, A-101, and A-103) will be constructed to replace the existing Souplantation restaurant building. The new building will be approximately the same overall size as the existing building (approximately 10,500 sf) and at the same general location. The new building may be shifted somewhat to the west into the existing parking lot. The individual shops vary from 3,000 to 4,400 sf. Patio spaces approximately 2,300 sf in size will be added on the south side of the building. The new building will also require some changes for the utilities. Minor parking lot improvements in adjacent areas will also be performed.

We understand that future infiltration BMP's are planned in the northwest portion of the site, which is where NMG conducted our preliminary infiltration testing.

1.5 Field Exploration

Our field exploration consisted of excavation of five (H-1 through H-3, P-1, and P-2) 8 to 12-inch-diameter, hollow-stem auger borings to depths of 26.5 to 51.5 feet below the ground surface (bgs). The borings were geotechnically logged, and both in-situ and bulk samples were collected at selected intervals. Borings P-1 and P-2 were drilled to depths of 31.5 and 26.5 feet respectively, and were used for preliminary percolation testing as described below.

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We obtained relatively undisturbed soil ring samples in the exploratory borings with a 2.5-inch inside-diameter, split-barrel sampler. The samplers were driven into the soil with a 140-pound automatic safety hammer, free-falling 30 inches. The drive samples were also used to obtain a measure of resistance of the soil to penetration (recorded as blows-per-foot on our geotechnical boring logs). Representative bulk samples of onsite soil were collected from the drill cuttings and used for additional soil identification purposes. Borings were backfilled with cuttings, tamped, and AC patched.

The approximate locations of the borings are shown on Figure 2. Boring logs are provided in Appendix B.

1.6 Laboratory Testing

Laboratory tests performed on selected bulk and relatively undisturbed soil samples include:

- Moisture content and dry density;
- Maximum dry density and optimum moisture content;
- Consolidation;
- Shear strength;
- Atterberg limits;
- Grain-size distribution;
- Hydrometer;
- Expansion index; and
- Corrosivity testing, including report.

Laboratory tests were conducted in general conformance with applicable ASTM standards. Laboratory test results are presented in Appendix D. In-situ moisture and dry density results are included on the geotechnical boring logs (Appendix B). The laboratory test results characterize the onsite soil and established the soil parameters utilized for our geotechnical analysis as presented in Section 2.6.

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2.0 GEOTECHNICAL FINDINGS

2.1 Geologic Setting and Earth Units

The subject site is located on the Newport Mesa, within the Peninsular Range geomorphic province of California. Geologic mapping by the California Geological Survey indicates that the site is underlain by marine terrace deposits (USGS, 2004).

Marine terrace deposits generally accumulate during periods of oceanic transgression when sea levels were higher than they are at present. The site would have been either underwater, or near a shore/beach environment. During this period sediment would have accumulated.

Early topographic maps of the area (USGS, 1932) show a small stream and canyon area at the north end of the site draining to the north. The topography shows 5-foot contours below the site indicating the canyon was deeper than 5 feet and shallower than 10 feet. Based on our subsurface exploration, the site is underlain by approximately 10 feet of fill overlying native marine terrace.

The underlying fill at the subject site generally consist of fine-grained clays and silts in the upper 10 to 15 feet. The fill was locally weathered and porous, with scattered carbonate deposits at the contact of the terrace deposits. The fill is stiff to very stiff, damp to wet, silty and sandy clay. Below this depth, there is a relatively clean, fine to coarse grained sandy zone that is 10 feet or more thick. The sandy layer is moist, medium dense to dense. Below 30 feet, the marine terrace was found to consist of interlayered clean sands and clays.

2.2 Regional Faulting and Seismicity

The site is not located within a fault-rupture hazard zone as defined by the Alquist-Priolo Special Studies Zones Act (Hart and Bryant, 2007) and no evidence of active faulting was observed during our site exploration. Also, based on mapping by the State (Jennings, 2010), there are no active faults mapped at the site.

Using the USGS deaggregation computer program (2013a) and the site coordinates of 33.672 degrees north latitude and 117.921 degrees west longitude, the closest major active fault to the site is the San Joaquin Hills Blind Thrust Fault, located at depth one mile northwest of the site.

The primary seismic hazard at the subject site is ground shaking due to a future earthquake on one of the major regional active faults.

2.3 Seismic Hazards

The subject site is not located in a mapped zone for liquefaction or earthquake induced landslides (CGS, 1998). Secondary earthquake hazards, including liquefaction, seismically-induced subsidence, earthquake-induced landslides, tsunami, and seiche should not impact the subject site due to the generally favorable geologic and topographic site conditions. Tsunami and seiche are not considered hazards at this site due to the elevation (approximately 60 feet above msl across the site) and the distance of the site from the ocean (3.5 miles). No confined bodies of water are located immediately adjacent to the site.

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2.4 Groundwater

No groundwater was encountered during excavation of the borings at the subject site. Historic high groundwater levels mapped by the California Geological Survey (CGS) have been reported as greater than 30 feet bgs in the general area where the site is located (CGS, 1997). This data provided by the CDMG is broad and not site specific. We found Geotracker data from multiple groundwater monitoring wells near the site that have groundwater level data over the past 15 years that ranges from a high of 65 feet bgs to a low of 90 feet bgs.

2.5 Percolation and Infiltration Rate Testing

Deep percolation testing was performed onsite on February 9, 2017 in Borings P-1 and P-2 that were 8-inch-diameter and 26.5 to 31.5 feet in depth. The location of the test borings takes into account information provided by Fuscoe indicating that the future BMPs are planned for the northeast portion of the site. The percolation testing targeted the sandy soil zone that underlies the site. The test borings were installed with 2-inch-diameter pipes and were backfilled with clean, graded, coarse sand to prevent caving during field percolation testing. The lower 10 feet of the pipes were perforated and upper portions were solid.

The field percolation testing was conducted per the 2011 County of Orange WQMP Technical Guidance Document (TGD). The percolation test method for sandy soils was utilized as described by the Riverside County Department of Environmental Health (as outlined in the County of Orange WQMP TGD). For the borehole method of testing, the percolation rate obtained from the measurements was converted to an infiltration rate. Note that the measured percolation rate overestimates the infiltration rate (by a factor of 10 or more) because it incorporates both downward and horizontal fluxes of water, whereas infiltration only refers to a downward flux of water.

The first two measurements for the deep percolation test were used to confirm that the criteria for sandy soil was achieved. The percolation testing was continued for an additional hour, with measurements taken every 5 to 10 minutes. The final measurement at the end of the testing period was used to calculate the tested infiltration rate. The infiltration rates were calculated using the Porchet Method (Inverse Borehole Method) as outlined by the TGD (and corrects for vertical flow and removes the effects of lateral flow). The percolation data sheets are presented in Appendix C.

Based on NMG's testing, the measured percolation rate and calculated infiltration rates are as listed in the table below.

DEEP FIELD PERCOLATION TEST RESULTS

Boring No.	*Total Depth (ft.)*	*Depth of Sand Zone (ft.)*	*Measured Percolation Rate (in./hr.)*	*Measured Infiltration Rate (in./hr.)*
P-1	31.5	16 – 30	39.6	2.17
P-2	26.5	10 – 25	43.2	2.56

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Both tests measured high percolation rates due to the clean sand layers that were targeted. The percolation rates are about 17 to 18 times higher than the infiltration rates. The existing fine-grained materials found in the upper soils and at various other depths in the borings will have significantly lower rates. Per the County guidelines, a minimum factor of safety of 2 should be provided to the infiltration rate for design purposes.

The geotechnical study for the nearby apartment site (Leighton, 2011) included two deep field percolation tests that were conducted in borings that were between 10 to 30 feet deep. These two tests had percolation rates of 33 to 38 gallons per square foot, per day. They also performed a third shallower percolation test in a boring that was only 10 feet deep and found a much lower percolation rate of 0.3 gallons per square foot, per day.

2.6 Geotechnical Conditions

The geotechnical soil properties and conditions provided below are based on the results of our field exploration and laboratory testing, along with our review of data from prior projects at or near the Mesa Verde center.

2.6.1 Soil Characterization and Classification: Grain-size distribution tests were conducted on six selected samples considered representative of the existing deposits. The existing fills have fines content (passing No. 200 sieve) from 56 to 79 percent in the upper 7.5 feet and the Marine Terrace Deposits have 6 percent fines for the sample at 15 feet. The Atterberg limits test were performed on near-surface soil samples (0 to 7.5 feet) to classify the materials. Based on the Atterberg limits tests, the clayey soils have a liquid limit in the range of 29 to 43 percent, with plasticity index varying from 10 to 23.

The boring and testing indicates the onsite soils consist mainly of clay, clayey and silty sand and sand. The existing soils are classified as CL, SC, SM and SP based on the Unified Soil Classification System (USCS). The upper 10 to 20 feet was found to be predominately clayey soils. Prior testing found some of the clay soils in the area to have high plasticity (classified as CH). The sandy soils were encountered mainly between depths of 10 to 30 feet. Below 25 to 30 feet deep, the soils vary and are interlayered.

2.6.2 Soil Moisture Content and Density: The soil in the upper 50 feet has a moisture content between 3 to 24 percent, total density between 100 to 134 pcf and dry density between 94 to 116 pcf. The degree of saturation ranged between 17 to 100 percent. The clayey soils had higher moisture content and saturation. The sandy soil had lower moisture content.

Based on our laboratory testing, the two bulk samples collected from the site and representative of the sandy clay/clayey sand materials in the upper 3 feet have maximum dry densities varying from 127 pounds per cubic foot (pcf) at optimum moisture contents of 9.0 to 9.5 percent. Prior testing for a sample of lean clay with sand taken nearby had a maximum dry density of 115 pounds per cubic foot (pcf) and an optimum moisture content of 12 percent.

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2.6.3 Direct Shear: The shear strength parameters for the shallow fill at the site was estimated based on laboratory direct shear testing of a remolded sampled from Boring H-1 and a relatively undisturbed sample from Boring H-2. These results are considered representative of the sandy clay. The test results found soil cohesion to vary from 0 to 700 psf with a soil friction angle of 27 to 29 degrees. Prior direct shear testing generally found similar strength parameters and, in some cases, slightly higher test results.

2.6.4 Consolidation: Three consolidation tests were performed on relatively undisturbed clayey soil samples obtained from Borings H-1 and P-1 in the upper 10 feet. The consolidation test results indicate these soil have low consolidation potential. The samples appear to be somewhat overconsolidated. The sample from Boring H-1 was found to swell near 1 percent upon the addition of water with a pressure of 1,600 psf. Prior consolidation testing found some of the clayey soil to be somewhat more compressible and also indicated some potential for swelling.

2.6.5 Expansion Potential and R-Value: Two expansion index tests were performed from bulk samples of the existing fill material obtained from the upper 2 feet in Borings H-1 and P-2. The result of the expansion index (EI) tests indicates a low expansion potential for the existing fill with laboratory test results of 39 and 46. Prior testing found the nearby soils to have a medium expansion potential (EI up to 90). Also, based on the upper clayey soils and prior testing, the site is considered to have poor pavement subgrade soil conditions for pavements.

2.7 Corrosivity

A representative soil sample was tested to evaluate pH, minimum electrical resistivity, and chemical content. The corrosivity testing was performed by HDR, Inc. and the results are presented in Appendix D. The results of the minimum resistivity tests indicated a minimum electrical resistivity of 2,240 ohm-cm (saturated), which is considered to be moderately corrosive to ferrous metals. The soil pH was measured to be 7.6, which is considered slightly alkaline. Sulfate and chloride concentrations for the tested soil indicate negligible corrosion potential for concrete.

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3.0 CONCLUSION AND RECOMMENDATIONS

3.1 General Conclusions

Based on the results of our subsurface exploration, the construction of the proposed new building, as described herein, is considered feasible from a geotechnical standpoint provided the recommendations in this report are implemented during design, grading, and construction.

This report provides geotechnical earthwork and foundation design recommendations for the new building. The recommendations herein are considered minimum and may be superseded by more stringent requirements of others. Our investigation and recommendations are based on preliminary project information. More specific recommendations may need to be provided once a final plan is developed.

3.2 Remedial Grading Measures (Overexcavation and Recompaction)

All soils disturbed during demolition of the existing building and foundation system will require reprocessing and recompaction. Existing footings and any underground conduits in the building area will have to be removed and disposed of offsite. Existing utility pipelines that will be abandoned should be completely removed and properly backfilled.

At minimum, remedial removals and overexcavation of the existing soils should be performed to provide a compacted fill blanket of at least 3 feet for the new building pad. Removal bottoms should expose relatively moist and stiff soil and should be observed and accepted by the geotechnical consultant. Locally, deeper removals may be required if the bottom soils are found to be unsuitable or disturbed. The remedial grading measures should include areas of foundations for canopies, patios and other architectural projections associated with the new structure. Where space permits, the remedial grading should extend a minimum of 5 feet beyond the building footprint and other foundations. Special care/measures may be required during any removal excavations next to the existing structures, utilities or improvements to be protected in-place.

3.3 General Earthwork and Grading

Prior to commencement of grading operations, all building debris, vegetation, asphalt and concrete, and other deleterious material should be cleared and disposed of offsite. All areas receiving fill, subsequent to the appropriate removals, should be scarified 6 inches and recompacted. Fill should be thoroughly mixed and placed in loose lifts of no more than 8 inches in thickness to a minimum of 90 percent of the maximum density (as established by ASTM Test Method D1557), at a moisture content approximately 120 to 130 percent of optimum moisture content.

Oversize material, if any, with a maximum dimension greater than 8inches should not be placed in the fill, but disposed of offsite. Any import soils should be evaluated and approved by the geotechnical consultant prior to use onsite.

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3.4 Foundation and Slab Design

The slab and foundations should be designed by the project structural engineer based on the proposed structure type and the near-surface soil materials properties. Onsite soils are expansive and CBC 2016 (Section 1808.6) guidelines for foundations and slabs on expansive soils apply. The slabs should also be designed to satisfy the settlement criteria presented in this report (Section 3.7). The following foundation recommendations are provided with the assumption that the remedial recommendations included in this report are implemented during grading of the building pad and provided a compacted fill blanket with proper compaction and moisture conditions.

The recommended allowable bearing capacity for footings founded in approved compacted fill may be calculated based on the following equation:

$$q_{all} = 1000 + 800D + 250B \quad \text{(but not to exceed 3,000 psf)}$$

where:

D = embedment depth of footing, in feet
B = width of footing, in feet

The building footings (exterior and interior) should be a minimum of 24 inches below the lowest adjacent grade. All isolated column footings should have a minimum depth and width of 24 inches.

Allowable bearing pressure may be increased by one-third for wind or seismic loading. A coefficient of friction of 0.3 may be assumed for the foundation soil contact. For concrete placed over aggregate base, the coefficient of resistance may be increased to 0.45. An effective plasticity index (PI) of 25 may be considered representative of the upper 15 feet of soils at the site. Passive lateral resistance of foundations against competent soil may be assumed to be equivalent to a fluid pressure of 300 pcf.

At minimum, we recommend the floor-slab section consist of a 5-inch reinforced PCC slab over an approved moisture retarder, over 4 inches of aggregate base (compacted to a minimum relative compaction of 95 percent per ASTM D1557). The slab subgrade should be presaturated to 130 percent of the optimum moisture content to a depth of 18 inches just prior to concrete placement. The concrete slab should have minimum strength of 3,250 psi and provided with reinforcement consisting, at minimum, of No. 4 bars spaced at a maximum of 24 inches on center, both directions. The recommendations for slab thickness and reinforcement are considered minimum and may be superseded by more restrictive requirements of the structural engineer, who is ultimately responsible for design of foundations and slabs based on the anticipated dead and live loads.

All footing excavations should be observed by the geotechnical consultant to verify that the footings are founded in approved soils. If clayey soils are encountered in the footing excavations or foundation soils they should be prevented from drying.

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3.5 Interior Slab Moisture Mitigation

In addition to geotechnical and structural considerations, the project owner should also consider moisture mitigation when designing and constructing slabs-on-grade. The intended use of the interior space, type of flooring, and the type of goods in contact with the floor may dictate the need for, and design of, measures to mitigate potential effects of moisture emission from and/or moisture vapor transmission through the slab. Typically, for human occupied structures, a vapor retarder or barrier has been recommended under the slab to help mitigate moisture transmission through slabs.

The current guidelines by the American Concrete Institute (ACI 302.1R-04) recommend that the vapor retarder be placed directly under the slab (no sand layer). However, the location of the vapor retarder may also be subject to the builder's past successful practice. Specifying the strength of the retarder to resist puncture and its permeance rating is important. These qualities are not necessarily a function of the retarder thickness. A minimum of 10-mil is typical but some materials, such as 10-mil polyethylene ("Visqueen"), may not meet the desired standards for toughness and permeance.

The vapor retarder, when used, should be installed in accordance with standards such as ASTM E1643-98 and/or those specified by the manufacturer.

Concrete mix design and curing are also significant factors in mitigating slab moisture problems. Concrete with lower water/cement ratios results in denser, less permeable slabs. They also "dry" faster with regard to when flooring can be installed (reduced moisture emissions quantities and rates). Rewetting of the slab following curing should be avoided since this can result in additional drying time required prior to flooring installation. Proper concrete slab testing prior to flooring installation is also important.

Also, the concrete mix design and the type and location of the vapor retarder should be determined in coordination with all parties involved in the finished product, including the project owner, architect, structural engineer, geotechnical consultant, concrete subcontractors, and flooring subcontractors.

3.6 Seismic Design Guidelines

The seismic design criteria based on the 2016 California Building Code (CBC) is presented in the following table:

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Selected Seismic Design Parameters from 2016 CBC/ASCE 7-10	*Seismic Design Values*	*Reference*
Latitude	33.672 North	
Longitude	117.921 West	
Nearest Seismic Source	San Joaquin Hills Blind Thrust Fault	USGS, 2013a
Distance to Nearest Seismic Source	1.86 Miles (3.0 km)	USGS, 2013a
Site Class per Table 20.3-1 of ASCE 7-10	D	USGS, 2013b
Spectral Acceleration for Short Periods (Ss)	1.604 g	USGS, 2013b
Spectral Accelerations for 1-Second Periods (S1)	0.592 g	USGS, 2013b
Site Coefficient F_a, Table 11.4-1 of ASCE 7-10	1.0	USGS, 2013b
Site Coefficient F_v, Table 11.4-2 of ASCE 7-10	1.5	USGS, 2013b
Design Spectral Response Acceleration at Short Periods (S_{DS}) from Equation 11.4-3 of ASCE 7-10	1.069 g	USGS, 2013b
Design Spectral Response Acceleration at 1-Second Period (S_{D1}) from Equation 11.4-4 of ASCE 7-10	0.592 g	USGS, 2013b
Peak Ground Acceleration (MCE_R) Corrected for Site Class Effects from Equation 11.8-1 of ASCE 7-10	0.633 g	USGS, 2013b
Seismic Design Category, Section 11.6 of ASCE 7-10	D	USGS, 2013b

3.7 Settlement Estimate

Total and differential settlements at the site for the proposed fill and structural loads should be within tolerable limits of the proposed structures provided the recommendations herein are implemented. Total settlements on the order of 1.0 inch or less are estimated. Differential settlements are not expected to exceed ½ inch over a 40 foot span; however, this should be evaluated more closely once the foundation plan is prepared. Differential settlements may depend on the footing design and layout. The foundation system designed by the structural engineer should be reviewed by the geotechnical consultant.

3.8 Groundwater and Infiltration Systems

Based on review of the collected groundwater information, including long-term monitoring data for adjacent observation wells (Geotracker), we anticipate the groundwater in the area to remain at least 50 or more feet deep.

The percolation testing conducted at the northwest portion of the site at Borings P-1 and P-2 found a zone of sandy soil that is considered suitable for infiltration (Figure 2). A deep infiltration/dry well system that specifically targets the existing sandy soil zone that is between 19 to 29 feet deep is feasible from a geotechnical standpoint.

The infiltration rate for the sand zone found between 19 to 29 feet deep was determined to be approximately 2.2 inches per hour. We recommend a minimum factor of safety of 2 be used for design of the onsite storm water infiltration/dry well system that would be a design infiltration

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rate of 1.1 inches per hour. The upper soils are clayey and are likely not suitable for infiltration and we would expect that they have an infiltration rate of less than 0.3 inch per hour.

The final design of the infiltration system/dry wells should be reviewed to confirm that the location and depths are within the sand layer. The underlying sand layer can vary across the site but is generally 10 feet or more in thickness in most locations. If alternative systems are proposed, additional field testing may be required to confirm the infiltration rate for the given locations and depths.

3.9 Cement Type and Corrosion

Sulfate content test results indicate that soluble sulfate exposure of onsite soils may be classified as "S0" per Table 19.3.1.1 of ACI-318-14 for structural concrete elements, including footings and building slabs-on-grade. Concrete mix for these elements may be based on the "S0" soluble sulfate exposure class per Table 19.3.2.1 in ACI-318-14, and/or the structural design requirements. Flatwork and sidewalk concrete are typically not considered structural concrete elements.

The corrosion control methods presented in the corrosivity report in Appendix C should be implemented to protect underground conduits and appurtenances

3.10 Exterior Concrete Construction

Exterior concrete elements, such as patios, sidewalks, driveways and curb and gutter are susceptible to lifting and cracking when constructed over expansive soils. With expansive soils, the impacts to flatwork/hardscape can be significant, generally requiring removal and replacement of the affected improvements. Please also note that reducing concrete problems is often a function of proper slab design, concrete mix design, placement, and curing/finishing practices. Adherence to the standard guidelines of the American Concrete Institute (ACI) and Portland Cement Association (PCA) is recommended. Also, the amount of post-construction watering, or lack thereof, can have a very significant impact on the adjacent concrete flatwork.

For reducing the potential effects of expansive soils, we recommend a combination of presaturation of subgrade soils, reinforcement, moisture barriers/drains, and a sub-layer of granular material. Though these types of measures may not completely eliminate adverse impacts, application of these measures can significantly reduce the impacts from post-construction expansion of soil. The degrees and combinations of these measures will depend upon:

- The expansion potential of the subgrade soils;
- The potential for moisture migration to the subgrade;
- The feasibility of the measures (especially presaturation); and
- The economics of these measures versus the benefits.

These factors should be weighed by the project owner determining the measures to be applied on a project-by-project basis, subject to the requirements of the local building/grading department.

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The following table provides our recommendations to be used during design and construction based on moderately expansive subgrade soils.

RECOMMENDATIONS FOR CONCRETE FLATWORK/HARDSCAPE	
Slab Thickness (Min.): Nominal thickness except where noted.	**4"**
Subbase: Thickness of sand or gravel layer below concrete	**2" – 4"**
Presaturation: Degree of optimum moisture content (opt.) and depth of saturation	**1.3 x opt. to 18"**
Joints: Maximum spacing of control joints. Joint should be ¼ of total thickness	**6'**
***Reinforcement*:** Rebar or equivalent welded wire mesh placed near mid-height of slab	**No. 3 rebar, 24" O.C. both ways or equivalent wire mesh**
***Restraint*:** Slip dowels across cold joints; between sidewalk and curb	**Across cold joints**

The more expansive soils, because they are clayey, can take significantly longer to achieve recommended presaturation levels. Therefore, the procedure and timing should be carefully planned in advance of construction. For exterior slabs, the use of a granular sublayer is primarily intended to facilitate presaturation and subsequent construction by providing a better working surface over the saturated soil. It also helps retain the added moisture in the native soil in the event that the slab is not placed immediately. Where these factors are not significant, the layer may be omitted.

Additional measures, such as thickened concrete edges/footings, subdrains, moisture barriers and/or root barriers, should be considered where planter or natural areas with irrigation are located adjacent to the concrete improvements. If the concrete will be subject to heavy loading from cars/trucks or other heavy objects, thicker slabs and a base layer may be warranted.

For Portland Cement Concrete (PCC) pavement at trash enclosure areas or other areas with heavier loads, we recommend the section consist of 6-inch concrete, reinforced with No. 3 rebar at 24-inches-on-center, both ways, placed over 4 inches of AB, compacted to a minimum relative compaction of 95 percent per ASTM D1557.

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3.11 Surface Drainage

Inadequate control of run-off water, heavy irrigation after development of the site may result in shallow groundwater conditions where previously none existed. Maintaining adequate surface drainage, proper disposal of run-off water, and control of irrigation will help reduce the potential for future moisture-related problems and differential movements from soil heave/settlement.

Surface drainage should be carefully taken into consideration during all landscaping and building construction. Positive surface drainage should be provided to direct surface water away from structures and hardscape areas and toward the street or suitable drainage devices. Ponding of water adjacent to the structures should be avoided. Buildings should have roof gutters systems and the run-off should be carried to street gutters by area drain pipes or by sheet flow over paved areas. The minimum gradient within 5 feet of the building will depend upon surface landscaping. In general, we suggest that unpaved lawn and landscape areas have a minimum gradient of 2 percent away from structures.

Construction of planter areas immediately adjacent to structures should be avoided. If planter boxes are constructed adjacent near buildings, the sides and bottoms of the planter should be provided with a moisture barrier to prevent penetration of the irrigation water into the subgrade. Provisions should be made to drain excess irrigation water from the planters without saturating the subgrade below or adjacent to the planters. Raised planter boxes, may be drained with weepholes. Deep planter (such as tree planters) should be drained with below-ground, water-tight drainage lines connected to a suitable outlet.

3.12 Utility Installation and Trench Backfill

Excavations should be performed in accordance with the requirements set forth by Cal/OSHA Excavation Safety Regulations (Construction Safety Orders, Section 1504, 1539 through 1547, Title 8, California Code of Regulations). In general, onsite soils may be classified as Type "B" Soil. Cal/OSHA regulations indicate that, for workmen in confined conditions, the steepest allowable slopes in Type B soil are 1:1 (horizontal to vertical) for excavations less than 20 feet deep. Where there is no room for these layback slopes, we anticipate that shoring will be necessary.

Excavations should be reviewed periodically by the contractor's qualified person to confirm compliance with Cal OSHA requirements.

Native soils should be suitable for use as trench backfill. Native backfill materials should be compacted to a minimum of 90 percent relative compaction. Select granular backfill may be used in lieu of native soils, but should also be compacted.

Trenches, including interior utility, should be either backfilled with native soil and compacted to 90 percent relative compaction, or backfilled with clean sand (SE 30 or better), which can be densified with water jetting and flooding (if allowed by the governing agency).

Trenches excavated next to structures and foundations should also be properly backfilled and compacted to provide full lateral support and reduce settlement potential.

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3.13 Geotechnical Plan Reviews

The building foundation plan and related structural calculations, along with the grading plan, should be reviewed and accepted by the geotechnical consultant for verification of geotechnical design parameters. Once grading and water quality management plans have been prepared, they should also be reviewed by the geotechnical consultant during the design phase of the project. Additional infiltration testing may need to be performed once infiltration BMP types and locations are selected.

3.14 Geotechnical Observation and Testing During Construction

We recommend that geotechnical observation and testing be performed at the following stages:

- During site preparation, prior to any earthwork operations, including demolition operation and removal of existing footings or underground pipelines/conduits;
- During all earthwork operations, including remedial removals and placement of fill;
- After footing excavations for buildings prior to placement of concrete;;
- After presaturation of slab subgrade soil, prior to pouring concrete;
- During pavement and hardscape subgrade preparation (including presoaking), prior to pouring of concrete;
- During placement of backfill for utility trenches; and
- When any unusual soil conditions are encountered.

The geotechnical consultant may need to provide additional recommendations and/or testing during grading based on the actual conditions encountered.

3.15 Limitations

This report has been prepared for the exclusive use of our client, C.J. Segerstrom and Sons, within the specific scope of services requested by them for the subject project. This report or its contents should not be used or relied upon for other projects or purposes or by other parties without the written consent of NMG and the involvement of a geotechnical professional. The means and methods used by NMG for this study are based on local geotechnical standards of practice, care, and requirements of governing agencies. No warranty or guarantee, express or implied is given.

The findings, conclusions, and recommendations herein are professional opinions based on interpretations and inferences made from geologic and engineering data from specific locations and depths, observed or collected at a given time. By nature, geologic conditions can vary from point to point, can be very different in between points, and can also change over time. Our conclusions and recommendations are subject to verification and/or modification during excavation and construction when more subsurface conditions are exposed.

NMG's expertise and scope of services did not include assessment of potential subsurface environmental contaminants or environmental health hazards.

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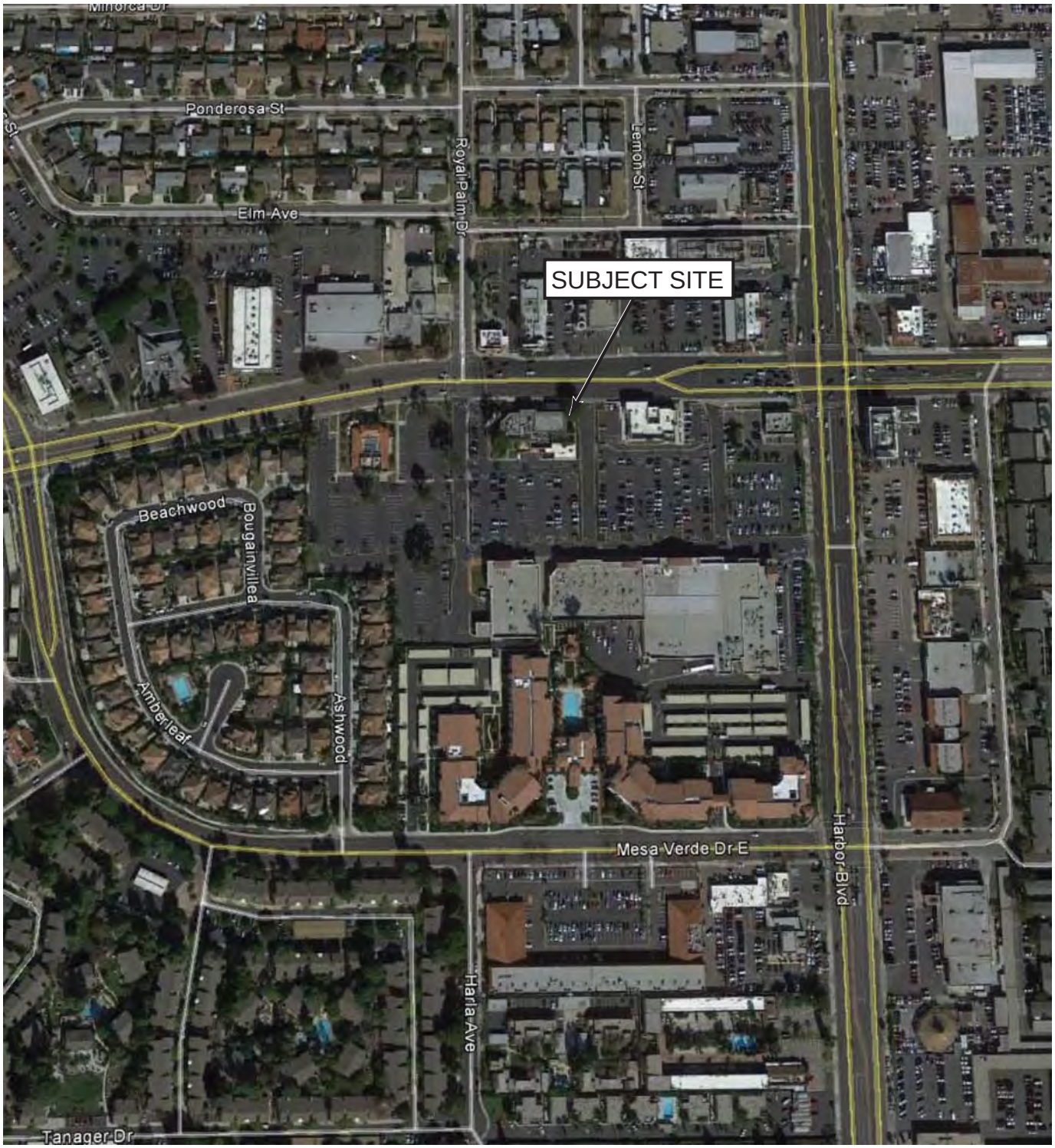
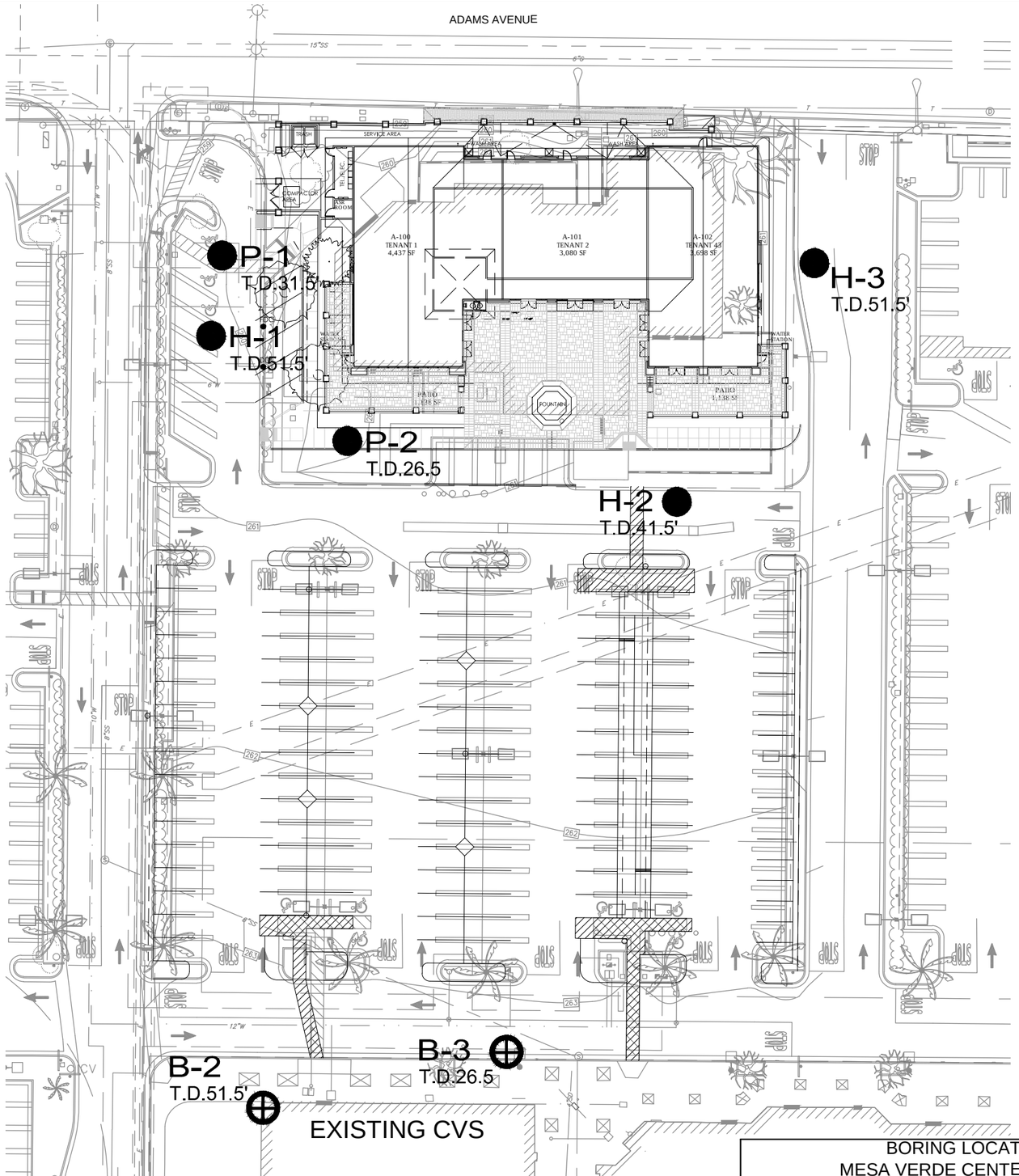


Base: Google Earth, 2016

SITE LOCATION MAP

N

MESA VERDE CENTER - BUILDING A 1555 ADAMS AVENUE (FORMER SOUPLANTATION) CITY OF COSTA MESA, CALIFORNIA	Project Number: 96079-03 Project Name: Segerstrom / Mesa Verde Date: 3/3/2017 Figure No. 1	NMG Geotechnical, Inc.



BASE MAP: FUSCOE ENGINEERING

L E G E N D

OTHER SYMBOLS - LOCATIONS ARE APPROXIMATE

P-2 T.D.31.5'	HOLLOW STEM AUGER BORING AND PERCOLATION TEST HOLE, SHOWING TOTAL DEPTH
H-3 T.D.51.5'	HOLLOW STEM AUGER BORING, SHOWING TOTAL DEPTH
B-3 T.D.26.5'	HOLLOW STEM AUGER BORING, BY NMG 1996

FIGURE 2

BORING LOCATION MAP
MESA VERDE CENTER-BUILDING A
1555 ADAMS AVE. (FORMER SOUPLANTATION)
CITY OF COSTA MESA, CALIFORNIA

Project No.: 96079-03 By: KGM/TD
Project Name:Segerstrom/MesaVerde
Date: 3/3/2017 N.T.S.

NMG Geotechnical, Inc.

Drawing: P:\1996\96079-03\Drafting_WP\PLATE 1.dwg Layout: Layout1 Last Saved: Tue Feb 28, 2017 - 10:18am Last Plotted: Wed Mar 01, 2017 - 9:57am By: jschneider

APPENDIX A

APPENDIX A

REFERENCES

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APPENDIX A

REFERENCES (Continued)

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APPENDIX B

SOIL CLASSIFICATION CHART

MAJOR DIVISIONS			SYMBOLS	TYPICAL DESCRIPTIONS
COARSE GRAINED SOILS MORE THAN 50% OF MATERIAL IS LARGER THAN NO. 200 SIEVE SIZE	*GRAVEL AND GRAVELLY SOILS* MORE THAN 50% OF COARSE FRACTION RETAINED ON 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
	SAND AND SANDY SOILS MORE THAN 50% OF COARSE FRACTION PASSING NO. 4 SIEVE	*CLEAN SANDS* (LITTLE OR NO FINES)	SW	WELL-GRADED SANDS, GRAVELLY SANDS, LITTLE OR NO FINES
			SP	POORLY GRADED SANDS, GRAVELLY SANDS, LITTLE OR NO FINES
		SANDS WITH FINES (APPRECIABLE AMOUNT OF FINES)	SM	SILTY SANDS, SAND - SILT MIXTURES
			SC	CLAYEY SANDS, SAND - CLAY MIXTURES
FINE GRAINED SOILS MORE THAN 50% OF MATERIAL IS SMALLER THAN NO. 200 SIEVE SIZE	*SILTS AND CLAYS*	LIQUID LIMIT LESS THAN 50	ML	INORGANIC SILTS AND VERY FINE SANDS, ROCK FLOUR, SILTY OR CLAYEY FINE SANDS OR CLAYEY SILTS WITH SLIGHT PLASTICITY
			CL	INORGANIC CLAYS OF LOW TO MEDIUM PLASTICITY, GRAVELLY CLAYS, SANDY CLAYS, SILTY CLAYS, LEAN CLAYS
			OL	ORGANIC SILTS AND ORGANIC SILTY CLAYS OF LOW PLASTICITY
	SILTS AND CLAYS	LIQUID LIMIT GREATER THAN 50	MH	INORGANIC SILTS, MICACEOUS OR DIATOMACEOUS FINE SANDY OR SILTY SOILS, ELASTIC SILTS
			CH	INORGANIC CLAYS OF HIGH PLASTICITY
			OH	ORGANIC CLAYS OF MEDIUM TO HIGH PLASTICITY, ORGANIC SILTS
HIGHLY ORGANIC SOILS			PT	PEAT, HUMUS, SWAMP SOILS WITH HIGH ORGANIC CONTENTS

NOTE: Dual symbols are used to indicate gravels or sand with 5-12% fines and soils with fines classifying as CL-ML. Symbols separated by a slash indicate borderline soil classifications.

Sampler and Symbol Descriptions

- Modified California sample (D-#)
- Standard Penetration Test (S-#)
- Undisturbed pushed tube sample (U-#)
- Large bulk sample (B-#)
- Small bulk sample (b-#)
- Approximate depth of groundwater

Note: Number of blows required to advance driven sample 12 inches (or length noted) is recorded.

Laboratory and Field Test Abbreviations

AL	Atterberg limits
CC	Chemical Testing incl. Soluble Sulfate
CN	Consolidation test
DS	Direct shear test
EI	Expansion Index
GS	Grain Size Analysis (Sieve and/or Hydro.)
MD	Compaction test
RV	Resistance Value (R-Value)
SE	Sand Equivalent
UU	Unconsolidated Shear Strength

GENERAL NOTES

1. Soil classifications are based on the Unified Soil System and include color, moisture, and relative density or consistency. Field descriptions have been modified to reflect results of laboratory tests where deemed appropriate. Bedrock descriptions are based on visual classification and include rock type, moisture, color, grain size, strength, and weathering.
2. Descriptions on these boring logs apply only at the specific boring locations and at the time the borings were made. They are not warranted to be representative of subsurface conditions at other locations or times.

KEY TO LOG OF BORING

Segerstrom/Mesa Verde
Costa Mesa, California
PROJECT NO. 96079-03

NMG Geotechnical, Inc.

Report: HOLLOW STEM; Project: 96079-03.GPJ; Data Template: NMG_GINT_2016.GDT; Printed: 2/28/17

Date(s) Drilled	2/8/17	Logged By	TBF
Drilling Company	2R Drilling	Drill Bit Size/Type	8"
Drill Rig Type	CME 75	Hammer Data	140 lbs @ 30" Drop
Sampling Method(s)	California Modified		
Approximate Groundwater Depth:	No Groundwater Encountered.		
Comments			

H-1

Sheet 1 of 2

Total Depth Drilled (ft)	51.5
Approximate Ground Surface Elevation (ft)	260.0

Elevation (ft)	Depth (ft)	SAMPLES Type	Number	Blows per foot	Graphic Log	USCS	MATERIAL DESCRIPTION	Moisture Content (%)	Dry Density (pcf)	OTHER TESTS and REMARKS
260	0						Surface: 5" AC over 6" AB.			
							Artficial Fill (Af)			B-1 @ .9 to 5', GS, MD, DS
			B-1 D-1	17		CL	@ 2.5': Dark brown CLAY with some fine to coarse sand, very stiff, wet, trace root hairs.	18.3	110.7	GS, AL
	5		D-2	25			@ 5': Dark brown CLAY with some fine to coarse sand, very stiff, wet, trace root hairs.	17.9	112.2	CN
			D-3	24		SM	@ 7.5': Upper: Dark brown to yellowish brown CLAY with some fine sand, very stiff, wet, trace root hairs, mottled. @ 8': Lower: Yellow silty fine SAND, medium dense, wet.	13.2	119.0	
250	10		D-4	21		SC	**Marine Terrace (Qtm)** @ 10': Yellow fine to coarse clayey SAND, medium dense, moist to wet.	11.9	115.9	
	15		D-5	40		SP	@ 15': Very pale brown to yellow fine to medium SAND, medium dense, damp, FeO staining.	4.0	104.5	-200
240	20		D-6	66			@ 20': Very pale brown fine to coarse SAND, dense, moist to wet.	7.7	113.8	
	25		D-7	41		CL	@ 25': Olive silty CLAY, very stiff, wet to saturated, trace caliche stringers.	31.1	94.5	
230	30									

LOG OF BORING

Segerstrom/Mesa Verde
Costa Mesa, California
PROJECT NO. 96079-03

NMG

Elevation (ft)	Depth (ft)	SAMPLES Type	SAMPLES Number	SAMPLES Blows per foot	Graphic Log	USCS	MATERIAL DESCRIPTION	Moisture Content (%)	Dry Density (pcf)	OTHER TESTS and REMARKS
230	30		D-8	47		CL	@ 30': Olive to yellow silty CLAY with some fine to coarse sand and silt, very hard, moist to wet.	14.7	114.4	
	35		D-9	52			@ 35': Upper: Olive silty CLAY, hard, moist to wet.	14.7	114.5	
						SC	@ 36': Lower: Gray clayey fine to coarse SAND, dense, moist, trace sea shells.			
220	40		D-10	54		CL	@ 40': Dark brown fine to coarse sandy CLAY, hard, wet, caliche.	23.9	100.9	
	45		D-11	28			@ 45': Olive silty CLAY, very stiff, wet, some caliche.	27.6	96.5	
210	50		D-12	20			@ 50': Light olive CLAY, stiff, wet, abundant caliche nodules.	18.8	105.9	
	55						Notes: Total Depth 51.5 Feet. No Groundwater Encountered. Backfilled With Cuttings, Tamped, And AC Patched.			
200	60									
	65									

LOG OF BORING

Segerstrom/Mesa Verde
Costa Mesa, California

PROJECT NO. 96079-03

NMG

Report: HOLLOW STEM; Project: 96079-03.GPJ; Data Template: NMG_GINT_2016.GDT; Printed: 2/27/17

Date(s) Drilled	2/8/17	Logged By	RTW
Drilling Company	2R Drilling	Drill Bit Size/Type	8"
Drill Rig Type	CME 75	Hammer Data	140 lbs @ 30" Drop
Sampling Method(s)	California Modified, Bulk		
Approximate Groundwater Depth:	No Groundwater Encountered.		
Comments			

H-2

Sheet 1 of 2

Total Depth Drilled (ft)	41.4
Approximate Ground Surface Elevation (ft)	261.0

Elevation (ft)	Depth (ft)	SAMPLES Type	Number	Blows per foot	Graphic Log	USCS	MATERIAL DESCRIPTION	Moisture Content (%)	Dry Density (pcf)	OTHER TESTS and REMARKS
	0						Surface: 5" AC over 9" AB.			
260							**Artificial Fill (Af)**			B-1 @ 1.5 to 5'
			B-1 D-1	8		CL	@ 2.5': Brown silty CLAY, medium stiff, wet.	18.9	107.2	
	5		D-2	27		CL	@ 5': Olive silty CLAY, very stiff, wet, mottled components.	18.0	109.6	GS, AL, DS
			D-3	59			@ 7.5': Mottled olive and white silty CLAY, hard, wet.	21.2	105.9	
	10		D-4	75		CL	**Marine Terrace (Qtm)** @ 10': Mottled olive and white silty CLAY, hard, wet.	17.9	111.0	
250										
	15		D-5	40			@ 15': Olive silty CLAY, very stiff, wet.	25.5	97.8	
	20		D-6	36			@ 20': Upper: Olive silty CLAY, very stiff, wet, micaceous.	28.4	92.7	
240						SP	Lower: Olive gray fine SAND, medium dense, moist, highly micaceous.			
	25		D-7	88/10"			@ 25': Pale yellow fine SAND, very dense, damp, micaceous.	3.7	102.3	
	30									

LOG OF BORING

Segerstrom/Mesa Verde
Costa Mesa, California
PROJECT NO. 96079-03

NMG

Elevation (ft)	Depth (ft)	SAMPLES Type	Number	Blows per foot	Graphic Log	USCS	MATERIAL DESCRIPTION	Moisture Content (%)	Dry Density (pcf)	OTHER TESTS and REMARKS
	30					SP	@ 30': Pale yellow fine to medium SAND, very dense, damp, dark mineral bands, clean.	3.4		
230			D-8	89/10"						
	35		D-9	88/11"			@ 35': Light gray fine to medium SAND, very dense, damp, micaceous.	5.0		
	40		D-10	50/4"			@ 40': Light gray fine SAND, very dense, damp, highly micaceous.	6.3		
220										
	45						Notes: Total Depth 41.4 Feet. No Groundwater Encountered. Backfilled With Cuttings, Tamped, And AC Patched.			
	50									
210										
	55									
	60									
200										
	65									

LOG OF BORING

Segerstrom/Mesa Verde
Costa Mesa, California

PROJECT NO. 96079-03

NMG

Report: HOLLOW STEM; Project: 96079-03.GPJ; Data Template: NMG_GINT_2016.GDT; Printed: 2/27/17

Date(s) Drilled	2/8/17	Logged By	TBF
Drilling Company	2R Drilling	Drill Bit Size/Type	8"
Drill Rig Type	CME 75	Hammer Data	140 lbs @ 30" Drop
Sampling Method(s)	California Modified, Bulk		
Approximate Groundwater Depth:	No Groundwater Encountered.		
Comments			

H-3

Sheet 1 of 2

Total Depth Drilled (ft)	51.5
Approximate Ground Surface Elevation (ft)	261.0

Elevation (ft)	Depth (ft)	SAMPLES Type	Number	Blows per foot	Graphic Log	USCS	MATERIAL DESCRIPTION	Moisture Content (%)	Dry Density (pcf)	OTHER TESTS and REMARKS
	0						Surface: 7" AC over 7" AB.			
260						CL	**Artificial Fill (Af)**			B-1 @ 1.1 to 5'
			B-1 D-1	16			@ 2.5': Dark reddish brown to dark brown CLAY, stiff, wet, trace wood debris, root hairs, mottled. @ 3.2': Tip: Pinkish white to brown silty CLAY, stiff, wet, mottled, trace root hairs.	17.9	109.7	
	5		D-2	26			@ 5': White to brown CLAY, very stiff, wet, abundant caliche nodules.	23.4	101.1	
			D-3			ML	@ 7.5': Brown clayey SILT, stiff, moist.	18.5	100.7	
	10		D-4	44		CL	**Marine Terrace (Qtm)** @ 10': Olive brown to Pale brown silty CLAY, very stiff, wet, blocky structure, strong cementation.	20.7	104.3	
250										
	15		D-5	34			@ 15': Brown silty CLAY, very stiff, wet, trace pencil-tip porosity, slightly stratified.	27.3	95.2	
	20		D-6	32			@ 20': Olive to brown silty CLAY, very stiff, wet, slight FeO staining.	28.1	93.5	
240										
	25		D-7	33			@ 25': Olive to brown silty CLAY, very stiff, wet to saturated, slight FeO staining, caliche.	26.6	98.4	
						SM	@ 25.7': Lower: Pale brown fine SAND with some silt, medium dense, wet.			
	30									

LOG OF BORING

Segerstrom/Mesa Verde
Costa Mesa, California
PROJECT NO. 96079-03

NMG

Elevation (ft)	Depth (ft)	SAMPLES Type	Number	Blows per foot	Graphic Log	USCS	MATERIAL DESCRIPTION	Moisture Content (%)	Dry Density (pcf)	OTHER TESTS and REMARKS
230	30		D-8	50/11"		SM	@ 30': Olive yellow fine SAND with some silt, very dense, damp to moist, FeO staining, slightly micaceous.	5.3	104.8	
	35		D-9	50/5"		SP	@ 35': Upper: Yellow to brown fine to medium SAND, very dense, damp to moist, micaceous, abundant seashells that are flat and up to 1" in diameter. @ 35.7': Lower: White fine SAND, very dense, damp to moist, micaceous.	6.7	98.1	
220	40		D-10	50/5"			@ 40': Very pale brown fine SAND, very dense, damp to moist, micaceous.	6.2	94.0	
	45		D-11	50			@ 45': Upper: Pink fine to coarse SAND, very dense, damp to moist, rounded to subrounded mostly quartz sand grains. @ 45.5': Lower: Light brown fine SAND, very dense, damp to moist, micaceous. @ 46.3': Tip: Pink fine to coarse SAND, very dense, damp to moist, rounded to subrounded mostly quartz sand grains.	9.2	99.3	
210	50		D-12	50/4"			@ 50': Light brown fine SAND, very dense, damp to moist, micaceous.	6.6	95.8	
	55						Notes: Total Depth 51.5 Feet. No Groundwater Encountered. Backfilled With Cuttings, Tamped, And AC Patched. The Drill Rig Experienced Mechanical Issues With The Hammer Drop While Taking Samples D-1, D-2, and D-3. Thes Blow Counts Recorded for These Samples may not be accuarte.			
200	60									
	65									

LOG OF BORING

Segerstrom/Mesa Verde
Costa Mesa, California

PROJECT NO. 96079-03

NMG

Report: HOLLOW STEM; Project: 96079-03.GPJ; Data Template: NMG_GINT_2016.GDT; Printed: 2/27/17

Date(s) Drilled	2/8/17	Logged By	TBF
Drilling Company	2R Drilling	Drill Bit Size/Type	8"
Drill Rig Type	CME 75	Hammer Data	140 lbs @ 30" Drop
Sampling Method(s)	California Modified, Bulk		
Approximate Groundwater Depth:	No Groundwater Encountered.		
Comments			

P-1

Sheet 1 of 2

Total Depth Drilled (ft)	31.5
Approximate Ground Surface Elevation (ft)	259.5

Elevation (ft)	Depth (ft)	SAMPLES Type	Number	Blows per foot	Graphic Log	USCS	MATERIAL DESCRIPTION	Moisture Content (%)	Dry Density (pcf)	OTHER TESTS and REMARKS
	0						Surface: 6" AC over 5 " AB.			
							Artificial Fill (Af)			
			D-1	22		CL	@ 2.5': Dark brown CLAY with some sand, stiff, wet, trace mica.	18.6	109.4	
	5		D-2	24			@ 5': Dark yellowish brown CLAY with some fine to coarse sand, very stiff, wet.	15.3	115.0	
			D-3	25		SC	@ 7.5': Upper: Brown silty CLAY, very stiff, wet, abundant calcihe.	13.7	119.1	
							Marine Terrace (Qtm) @ 7.7': Lower: Brownish yellow clayey fine to medium SAND, medium dense, wet.			
250	10		D-4	12			@ 10': Brownish yellow clayey fine to coarse SAND, loose, moist, trace root hairs, trace pinhole pores.	14.5	103.7	
	15		D-5	36		CL	@ 15': Upper: Olive CLAY, very stiff, moist.	15.7	98.4	
						SP	@ 16': Lower: Reddish yellow fine to coarse SAND, medium dense, moist, fine subrounded gravel.			
240	20		D-6	83			@ 20': Reddish yellow fine to coarse SAND, very dense, damp, fine subrounded gravel.	4.4	111.2	
	25		D-7	51			@ 25': Gray fine to coarse SAND, dense, moist, few thin clay layers throughout sample.	12.0	116.2	
230	30									

LOG OF BORING

Segerstrom/Mesa Verde
Costa Mesa, California
PROJECT NO. 96079-03

NMG

Elevation (ft)	Depth (ft)	SAMPLES Type	SAMPLES Number	SAMPLES Blows per foot	Graphic Log	USCS	MATERIAL DESCRIPTION	Moisture Content (%)	Dry Density (pcf)	OTHER TESTS and REMARKS
	30		D-8	28		CL	@ 30': Upper: Dark brown CLAY, very stiff, wet.	17.5	109.5	
						SC	@ 31': Lower: Light brown to gray fine to medium SAND with some clay, medium dense, wet, trace fine subdrounded gravel.			
	35						Notes: Total Depth 31.5 Feet. No Groundwater Encountered. Percolation Well Installed On 2/8/2017. Backfilled With Cuttings, Tamped, And AC Patched On 2/9/2017.			
220	40									
	45									
210	50									
	55									
200	60									
	65									

LOG OF BORING

Segerstrom/Mesa Verde
Costa Mesa, California

PROJECT NO. 96079-03

NMG

Report: HOLLOW STEM; Project: 96079-03.GPJ; Data Template: NMG_GINT_2016.GDT; Printed: 2/27/17

Date(s) Drilled	2/8/17	Logged By	RTW
Drilling Company	2R Drilling	Drill Bit Size/Type	8"
Drill Rig Type	CME 75	Hammer Data	140 lbs @ 30" Drop
Sampling Method(s)	California Modified, Bulk		
Approximate Groundwater Depth:	No Groundwater Encountered.		
Comments			

P-2

Sheet 1 of 1

Total Depth Drilled (ft)	26.5
Approximate Ground Surface Elevation (ft)	261.0

Elevation (ft)	Depth (ft)	SAMPLES Type	Number	Blows per foot	Graphic Log	USCS	MATERIAL DESCRIPTION	Moisture Content (%)	Dry Density (pcf)	OTHER TESTS and REMARKS
	0						Surface: Grass - landscaping area.			B-1 @ 0-5', GS
260							**Artificial Fill (Af)**			
			B-1 D-1	19		SM	@ 2.5': Upper: Yellowish brown silty fine SAND, medium dense, wet.	16.8	112.3	
						CL	@ 3': Lower: Dark brown silty CLAY, stiff, wet.			
	5		D-2	14		CL	@ 5': Yellowish brown silty CLAY with some fine sand, stiff, wet.	18.1	110.2	
			D-3	13			@ 7.5': Yellowish brown silty CLAY with some fine sand, stiff, wet, trace roots.	17.2	109.4	GS, AL
	10		D-4	16		SP	**Marine Terrace (Qtm)** @ 10': Yellowish brown medium to coarse SAND, medium dense, damp to moist.	7.9	106.3	
250										
	15		D-5	45		SM	@ 15': Upper: Yellowish brown silty fine to medium SAND, dense to very dense, moist.	12.9	99.8	
						SP	@ 16': Lower: Very pale brown medium SAND, very dense, moist.			
	20		D-6	84			@ 20': Very pale brown medium SAND, very dense, damp.	5.0	107.8	
240										
	25		D-7	34		CL	@ 25': Olive CLAY, very stiff, wet, slighlty stratified, trace coal.	24.3	99.7	
	30						Notes: Total Depth 26.5 Feet. No Groundwater Encountered. Percolation Well Installed On 2/8/2017. Backfilled With Cuttings, Tamped, And AC Patched On 2/9/2017.			

LOG OF BORING

Segerstrom/Mesa Verde
Costa Mesa, California
PROJECT NO. 96079-03

NMG

SOIL CLASSIFICATION CHART

MAJOR DIVISIONS			SYMBOLS	TYPICAL DESCRIPTIONS
COARSE GRAINED SOILS MORE THAN 50% OF MATERIAL IS LARGER THAN NO. 200 SIEVE SIZE	*GRAVEL AND GRAVELLY SOILS* MORE THAN 50% OF COARSE FRACTION RETAINED ON 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
	SAND AND SANDY SOILS MORE THAN 50% OF COARSE FRACTION PASSING NO. 4 SIEVE	*CLEAN SANDS* (LITTLE OR NO FINES)	SW	WELL-GRADED SANDS, GRAVELLY SANDS, LITTLE OR NO FINES
			SP	POORLY GRADED SANDS, GRAVELLY SANDS, LITTLE OR NO FINES
		SANDS WITH FINES (APPRECIABLE AMOUNT OF FINES)	SM	SILTY SANDS, SAND - SILT MIXTURES
			SC	CLAYEY SANDS, SAND - CLAY MIXTURES
FINE GRAINED SOILS MORE THAN 50% OF MATERIAL IS SMALLER THAN NO. 200 SIEVE SIZE	*SILTS AND CLAYS*	LIQUID LIMIT LESS THAN 50	ML	INORGANIC SILTS AND VERY FINE SANDS, ROCK FLOUR, SILTY OR CLAYEY FINE SANDS OR CLAYEY SILTS WITH SLIGHT PLASTICITY
			CL	INORGANIC CLAYS OF LOW TO MEDIUM PLASTICITY, GRAVELLY CLAYS, SANDY CLAYS, SILTY CLAYS, LEAN CLAYS
			OL	ORGANIC SILTS AND ORGANIC SILTY CLAYS OF LOW PLASTICITY
	SILTS AND CLAYS	LIQUID LIMIT GREATER THAN 50	MH	INORGANIC SILTS, MICACEOUS OR DIATOMACEOUS FINE SANDY OR SILTY SOILS, ELASTIC SILTS
			CH	INORGANIC CLAYS OF HIGH PLASTICITY
			OH	ORGANIC CLAYS OF MEDIUM TO HIGH PLASTICITY, ORGANIC SILTS
HIGHLY ORGANIC SOILS			PT	PEAT, HUMUS, SWAMP SOILS WITH HIGH ORGANIC CONTENTS

NOTE: Dual symbols are used to indicate gravels or sand with 5-12% fines and soils with fines classifying as CL-ML. Symbols separated by a slash indicate borderline soil classifications.

Sampler and Symbol Descriptions

- Modified California sample (63.5 mm diameter)
- Standard Penetration Test
- Undisturbed pushed tube sample
- Large bulk sample
- Small bulk sample
- Approximate depth of perched water or groundwater

Note: Number of blows required to advance driven sample 300 mm (or length noted) is recorded; blow count recorded for seating interval (initial 150 mm of drive) is indicated by an asterisk.

Laboratory and Field Test Abbreviations

MD	Laboratory compaction test
CN	Laboratory consolidation test
DS	Laboratory direct shear test
AL	Atterberg limits
SE	Sand equivalent, test result
SA	Sieve analysis
-#200	Wash analysis
CC	Chemical Testing
EI	Expansion Index

GENERAL NOTES

1. Station location is indicated with offset to right (R) or left (L) of centerline (CL).
2. Soil classifications are based on the Unified Soil System and include color, moisture, and relative density or consistency. Field descriptions have been modified to reflect results of laboratory tests where deemed appropriate. Bedrock descriptions are based on visual classification and include rock type, moisture, color, grain size, strength, and weathering.
3. Descriptions on these boring logs apply only at the specific boring locations and at the time the borings were made. They are not warranted to be representative of subsurface conditions at other locations or times.

KEY TO LOG OF BORING

Mesa Verde SAVE-ON

PROJECT NO. 95130-1

NMG Geotechnical, Inc.

Date(s) Drilled	3/24/96	Logged By	ML
Drilling Method	Hollow Stem	Drill Bit Size/Type	8"
Drill Rig Type	Mobile 57	Hammer Data	30" Drop with 140 lbs
Sampling Method(s)	Modified California, SPT and Bulk Sampling		
Approximate Groundwater Depth and Date Measured	No GW Encountered		
Comments	N/A		

B- 2

Sheet 1 of 2

Total Depth Drilled (ft)	51.5
Approximate Ground Surface Elevation (ft)	63.6

Elevation (ft)	Depth (ft)	SAMPLES Type	Number	Blows per foot	Graphic Log	USCS	MATERIAL DESCRIPTION	Moisture Content (%)	Dry Density (pcf)	OTHER TESTS and REMARKS
	0					SM&CL	FILL:			
			B1 D1	23			@ 1.5' Yellow brown silty SAND, moist, medium dense, micaceous, and dark brown silty CLAY, wet, with root hairs	13	108	DS
60			D2	34		SM	@ 3' Brown silty SAND, wet, medium dense, micaceous.	15	111	
	5						MARINE TERRACE:			
			D3	20		CL	@ 6' Dark brown silty CLAY, wet, stiff.	20	108	CC
						SP				
	10		S1	30			@ 10' Orange brown fine to medium SAND, damp, medium dense, micaceous.	14		
50										
	15		D4	93			@ 16' Brown fine to medium SAND, dry, dense, micaceous.	2		
	20		D5	29			@ 20' Brown fine SAND, damp, medium dense.	12		
						CL SP	@ 21' 4-5" thick layer of light gray silty CLAY, wet, very stiff.			
							@ 21.5' Brown coarse SAND, dry, medium dense.			
40										
	25			45			@ 25' Brown coarse SAND, with coarse pebbles, dry, medium dense, some gravel up to 1" in diameter.	3		
	30									

LOG OF BORING
MESA VERDE SAVE-ON
COSTA MESA
PROJECT NO. 95130-1

NMG Geotechnical, Inc.

MESA VERDE SAVE-ON COSTA MESA

FOR:

B- 2

Sheet 2 of 2

Elevation (ft)	Depth (ft)	SAMPLES Type	SAMPLES Number	SAMPLES Blows per foot	Graphic Log	USCS	MATERIAL DESCRIPTION	Moisture Content (%)	Dry Density (pcf)	OTHER TESTS and REMARKS
	30			83			@ 30' Yellow brown fine SAND, dry, very dense, micaceous.	4		
30										
	35		D6	70/ 4"		SP	@ 35' White fine to coarse SAND, dry, very dense, with shell fragments, micaceous.	4	94	
						SP	@ 35.8' White fine SAND, dry, very dense, micaceous, possibly disturbed.			
	40		S4	77			@ 40' Brown fine SAND, with interbeds of brown fine to medium SAND, dry, very dense, micaceous.	4		
20										
	45									
	50		S5	107			@ 50' Light gray brown fine SAND, damp, very dense, micaceous.	6		
10							Total Depth 51.5' No Groundwater Encountered Backfilled 3/24/96.			
	55									
	60									
0										
	65									

NMG Geotechnical, Inc.

Figure

Template: NMG4; Prj ID: MVSON; Printed: 4/9/96

Date(s) Drilled	3/24/96	Logged By	ML
Drilling Method	Hollow Stem	Drill Bit Size/Type	8"
Drill Rig Type	Mobile 57	Hammer Data	30" Drop with 140 lbs
Sampling Method(s)	Modified California, SPT and Bulk Sampling		
Approximate Groundwater Depth and Date Measured	No GW Encountered		
Comments	N/A		

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Sheet 1 of 1

Total Depth Drilled (ft)	26.5
Approximate Ground Surface Elevation (ft)	62.8

Elevation (ft)	Depth (ft)	SAMPLES Type	Number	Blows per foot	Graphic Log	USCS	MATERIAL DESCRIPTION	Moisture Content (%)	Dry Density (pcf)	OTHER TESTS and REMARKS
	0						ASPHALT			
						SM	FILL: @ 0.5' Yellow brown silty SAND, with some pebbles.			
			D1	45		CL	@ 1.5' Brown sandy CLAY, moist to wet, very stiff, micaceous.	15	116	CN,AL,-#200 EI,CC
60			B1			SM				
			D2	63		SP	@ 2.5' Brown silty SAND, damp to moist, medium dense, micaceous.	5	126	
	5						@ 3' Brown fine SAND, with trace of silt, damp, medium dense, micaceous. MARINE TERRACE:			
			D3	100		CL	@ 6' Dark brown silty CLAY, moist, hard, with abundant carbonate SILTS.	13	120	
	10		D4	90/ 4"			@ 10' Light brown silty CLAY/clayey SILT, moist, hard, abundant carbonate SILTS, porous.	13	107	
			B2							
50						CL				
	15		D5	90/ 4"			@ 15' Light brown silty CLAY, moist, hard, abundant carbonate SILTS, porous.	18	102	
	20		D6	96			@ 20' Mottled light gray green silty CLAY, moist, hard, abundant carbonate SILTS.	21	99	
40										
	25		D7	84		SM	@ 25' Light brown silty fine SAND, wet, dense.	24	94	
						CL	@ 26' Olive brown green silty CLAY, moist, hard, with carbonate SILTS.			
							Total Depth 26.5 No Groundwater Encountered Backfilled 3/24/96.			
	30									

LOG OF BORING
MESA VERDE SAVE-ON
COSTA MESA
PROJECT NO. 95130-1

NMG Geotechnical, Inc.

Template: NMG4; Prj ID: MVSON; Printed: 4/9/96

APPENDIX C

Percolation Data Sheet

Project Name: Segerstrom/Mesa Verde
Test Hole Number: P-1
Depth (in): 374.4 Radius (in.): 4
Tested By: TBF

Project Number: 96079-03
Date Excavated: 2/8/2017
Date Presoak: 2/9/2017
Date Tested: 2/9/2017

Sandy Soil Criteria

Trial Number	Time	Time Interval (mins.)	Initial Water Level (in.)	Final Water Level (in.)	Δ in Water Level (in.)
1	9:37 9:41	4	296.4	322.8	26.4
2	9:46 9:50	4	300	312	12

Percolation Data

Time	Time Interval (mins.)	Total Elapsed Time (mins)	Initial Depth to Water (in.)	Final Depth to Water (in.)	Δ in Water Level (in.)	Percolation Rate (in./hr.)
10:38 10:44	6	6	308.4	324	15.6	156
10:44 10:54	10	16	324	334.8	10.8	64.8
10:54 11:04	10	26	334.8	342.6	7.8	46.8
11:04 11:14	10	36	342.6	348	5.4	32.4
11:18 11:22	4	40	304.8	319.8	15	225
11:22 11:28	6	46	319.8	326.4	6.6	66
11:28 11:38	10	56	326.4	337.2	10.8	64.8
11:38 11:48	10	66	337.2	343.8	6.6	39.6
11:48 11:58	10	76	343.8	349.2	5.4	32.4

Initial Height of Water (Ho) = 30.6
Final Height of Water (Hf) = 25.2
Change in Height Over Time (ΔH) = 5.4
Average Head Over Time (Havg) = 27.9

$I_t = \Delta H(60r)/\Delta t(r+2Havg)$
$I_t = 2.17$ in./hr.

Percolation Data Sheet

Project Name: Segerstrom/Mesa Verde
Test Hole Number: P-2
Depth (in): 330 Radius (in.): 4
Tested By: TBF

Project Number: 96079-03
Date Excavated: 2/9/2017
Date Presoak: 2/9/2017
Date Tested: 2/9/2017

Sandy Soil Criteria

Trial Number	Time	Time Interval (mins.)	Initial Water Level (in.)	Final Water Level (in.)	Δ in Water Level (in.)
1	12:33 12:34	1	230.4	312	81.6
2	12:34 12:40	6	312	322.8	10.8

Percolation Data

Time	Time Interval (mins.)	Total Elapsed Time (mins)	Initial Depth to Water (in.)	Final Depth to Water (in.)	Δ in Water Level (in.)	Percolation Rate (in./hr.)
12:45 12:50	5	5	228	310.2	82.2	986.4
12:50 12:55	5	10	310.2	316.8	6.6	79.2
12:55 13:00	5	15	316.8	319.2	2.4	28.8
13:00 13:05	5	20	319.2	322.2	3	36
13:11 13:16	5	31	225.6	287.4	61.8	741.6
13:16 13:21	5	36	287.4	297	9.6	115.2
13:21 13:26	5	41	297	301.8	4.8	57.6
13:26 13:31	5	46	301.8	305.4	3.6	43.2
13:31 13:36	5	51	305.4	308.4	3	36
13:43 13:48	5	63	216	279	63	756
13:48 13:53	5	68	279	290.4	11.4	136.8
13:53 13:58	5	73	290.4	296.4	6	72
13:58 14:03	5	78	296.4	300	3.6	43.2

Initial Height of Water (Ho) = 33.6
Final Height of Water (Hf) = 30
Change in Height Over Time (ΔH) = 3.6
Average Head Over Time (Havg) = 31.8

$I_t = \Delta H(60r)/\Delta t(r+2Havg)$
$I_t = 2.56$ in./hr.

APPENDIX D

Segerstrom/Mesa Verde

APPENDIX
SUMMARY OF SOIL LABORATORY DATA

Costa Mesa, California

Boring/Sample Information									Sieve/ Hydrometer		Atterberg Limits			Direct Shear				Compaction					
														Ultimate		Peak							
Boring No.	Sample No.	Depth (feet)	Elevation (feet)	Blow Count (N)	Field Wet Density (pcf)	Field Dry Density (pcf)	Field Moisture Content (%)	Degree of Sat. (%)	Fines Content (% pass. #200)	Clay Content (% pass. 2μ)	LL (%)	PI (%)	USCS Group Symbol	Cohesion (psf)	Friction Angle (°)	Cohesion (psf)	Friction Angle (°)	Maximum Dry Density (pcf)	Optimum Moisture Content (%)	Expansion Index	R-Value	Soluble Sulfate Content (% by wt)	Remarks / Other Laboratory Tests
H-1	B-1	0.9	259.1						56				CL	0	29			127.0	9.5	39			
H-1	D-1	2.5	257.5	17	130.9	110.7	18.3	94.4	79	25	39	21	CL										
H-1	D-2	5.0	255.0	25	132.4	112.2	17.9	96.6					CL										
H-1	D-3	7.5	252.5	24	134.7	119.0	13.2	85.9					CL										
H-1	D-4	10.0	250.0	21	129.7	115.9	11.9	71.0					SC										
H-1	D-5	15.0	245.0	40	108.7	104.5	4.0	17.7	6				SP										
H-1	D-6	20.0	240.0	66	122.6	113.8	7.7	43.5					SP										
H-1	D-7	25.0	235.0	41	123.9	94.5	31.1	100.0					CL										
H-1	D-8	30.0	230.0	47	131.3	114.4	14.7	84.2					CL										
H-1	D-9	35.0	225.0	52	131.3	114.5	14.7	84.0					CL										
H-1	D-10	40.0	220.0	54	125.1	100.9	23.9	96.4					CL										
H-1	D-11	45.0	215.0	28	123.1	96.5	27.6	99.8					CL										
H-1	D-12	50.0	210.0	20	125.7	105.9	18.8	85.7					CL										
H-2	B-1	1.5	259.5										CL										
H-2	D-1	2.5	258.5	8	127.4	107.2	18.9	89.3					CL										
H-2	D-2	5.0	256.0	27	129.4	109.6	18.0	90.7	79	26	43	23	CL	325	27								
H-2	D-3	7.5	253.5	59	128.3	105.9	21.2	96.8					CL										
H-2	D-4	10.0	251.0	75	130.9	111.0	17.9	93.3					CL										
H-2	D-5	15.0	246.0	40	122.8	97.8	25.5	95.2					CL										
H-2	D-6	20.0	241.0	36	119.0	92.7	28.4	93.6					CL										
H-2	D-7	25.0	236.0	88/10"	106.1	102.3	3.7	15.3					SP										
H-2	D-8	30.0	231.0	89/10"			3.4						SP										
H-2	D-9	35.0	226.0	88/11"			5.0						SP										
H-2	D-10	40.0	221.0	50/4"			6.3						SP										
H-3	B-1	1.1	259.9										CL										
H-3	D-1	2.5	258.5	16	129.3	109.7	17.9	90.3					CL										
H-3	D-2	5.0	256.0	26	124.7	101.1	23.4	94.7					CL										
H-3	D-3	7.5	253.5		119.3	100.7	18.5	74.1					CL										
H-3	D-4	10.0	251.0	44	126.0	104.3	20.7	91.0					CL										
H-3	D-5	15.0	246.0	34	121.2	95.2	27.3	95.6					CL										
H-3	D-6	20.0	241.0	32	119.7	93.5	28.1	94.4					CL										
H-3	D-7	25.0	236.0	33	124.7	98.4	26.6	100.0					CL										
H-3	D-8	30.0	231.0	50/11"	110.4	104.8	5.3	23.7					SM										
H-3	D-9	35.0	226.0	50/5"	104.6	98.1	6.7	25.3					SP										
H-3	D-10	40.0	221.0	50/5"	99.8	94.0	6.2	21.1					SP										

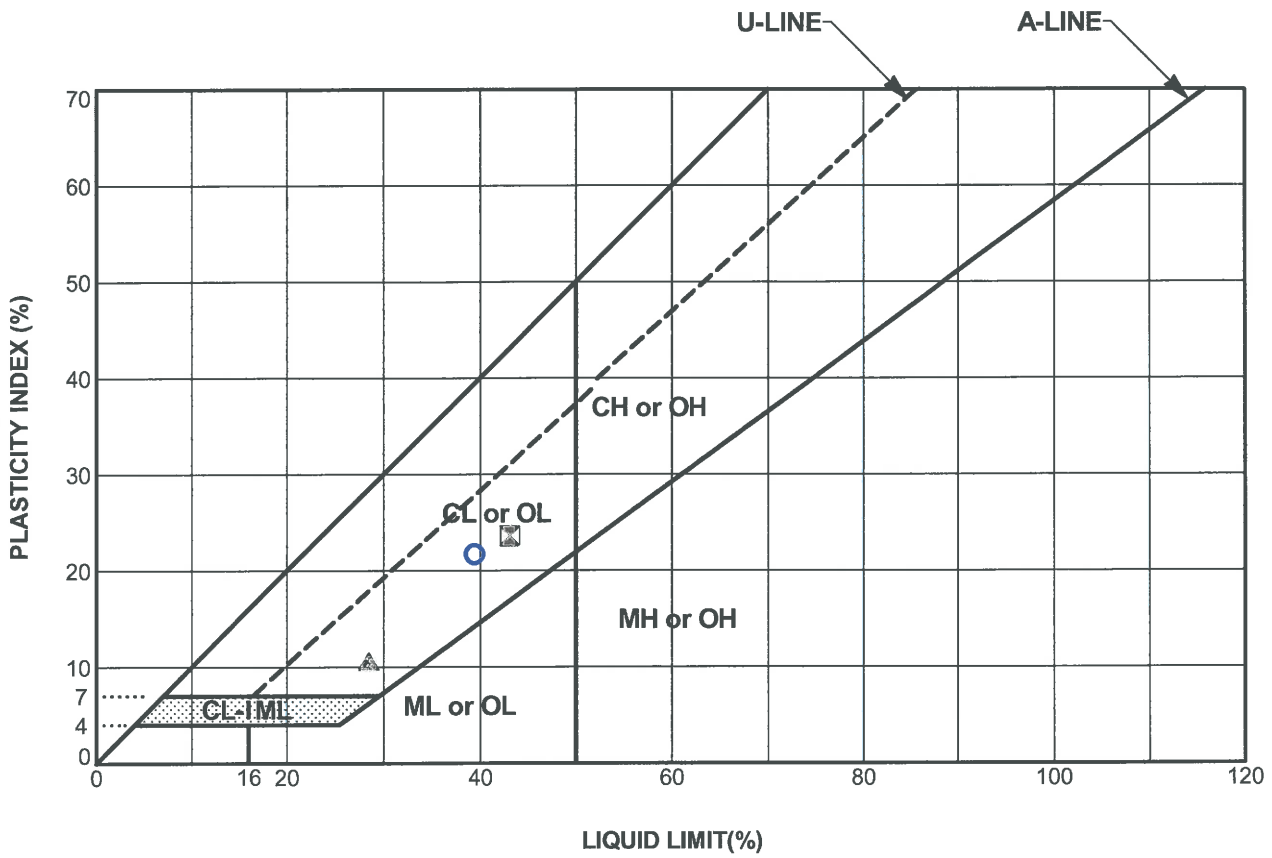
NMG Geotechnical, Inc.

Printed: 3/3/17; Template: SUM_SOIL_LAB; Proj ID: 96079-03.GPJ

APPENDIX

SUMMARY OF SOIL LABORATORY DATA

Boring/Sample Information									Sieve/ Hydrometer		Atterberg Limits			Direct Shear									
														Ultimate		Peak							
Boring No.	Sample No.	Depth (feet)	Elevation (feet)	Blow Count (N)	Field Wet Density (pcf)	Field Dry Density (pcf)	Field Moisture Content (%)	Degree of Sat. (%)	Fines Content (% pass. #200)	Clay Content (% pass. 2μ)	LL (%)	PI (%)	USCS Group Symbol	Cohesion (psf)	Friction Angle (°)	Cohesion (psf)	Friction Angle (°)	Maximum Dry Density (pcf)	Optimum Moisture Content (%)	Expansion Index	R-Value	Soluble Sulfate Content (% by wt)	Remarks / Other Laboratory Tests
H-3	D-11	45.0	216.0	50	108.4	99.3	9.2	35.7					SP										
H-3	D-12	50.0	211.0	50/4"	102.1	95.8	6.6	23.6					SP										
P-1	D-1	2.5	257.0	22	129.7	109.4	18.6	92.8					CL										
P-1	D-2	5.0	254.5	24	132.5	115.0	15.3	88.6					CL										
P-1	D-3	7.5	252.0	25	135.4	119.1	13.7	89.0					CL										
P-1	D-4	10.0	249.5	12	118.7	103.7	14.5	62.7					SC										
P-1	D-5	15.0	244.5	36	113.8	98.4	15.7	59.4					CL										
P-1	D-6	20.0	239.5	83	116.2	111.2	4.4	23.2					SP										
P-1	D-7	25.0	234.5	51	130.2	116.2	12.0	72.1					SP										
P-1	D-8	30.0	229.5	28	128.6	109.5	17.5	87.8					CL										
P-2	B-1	1.0	260.0						58				CL					127.0	9.0	46			
P-2	D-1	2.5	258.5	19	131.2	112.3	16.8	90.9					SM										
P-2	D-2	5.0	256.0	14	130.1	110.2	18.1	92.2					CL										
P-2	D-3	7.5	253.5	13	128.2	109.4	17.2	86.1	66	12	29	10	CL										
P-2	D-4	10.0	251.0	16	114.7	106.3	7.9	36.5					SP										
P-2	D-5	15.0	246.0	45	112.7	99.8	12.9	50.8					SM										
P-2	D-6	20.0	241.0	84	113.3	107.8	5.0	24.2					SP										
P-2	D-7	25.0	236.0	34	124.0	99.7	24.3	95.3					CL										

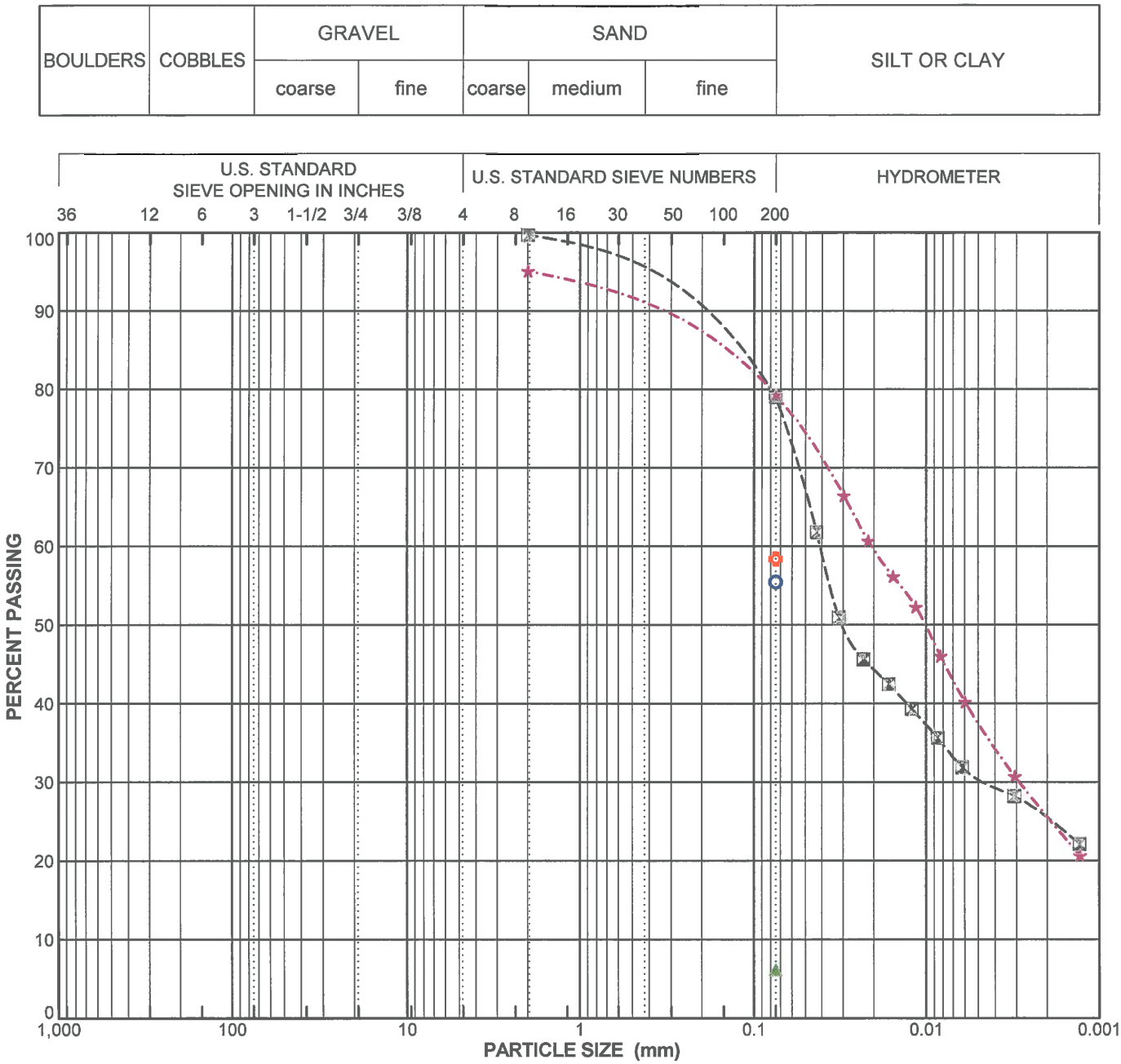


Symbol	Boring Number	Sample Number	Depth (feet)	Passing No. 200 Sieve (%)	LL	PI	USCS	Description
○	H-1	D-1	2.5	79	39	21	CL	Brown silty CLAY
⊠	H-2	D-2	5.0	79	43	23	CL	Light brown silty CLAY
▲	P-2	D-3	7.5	66	29	10	CL	Yellowish brown sandy CLAY

PLASTICITY CHART

Segerstrom/Mesa Verde
Costa Mesa, California
PROJECT NO. 96079-03

NMG Geotechnical, Inc.

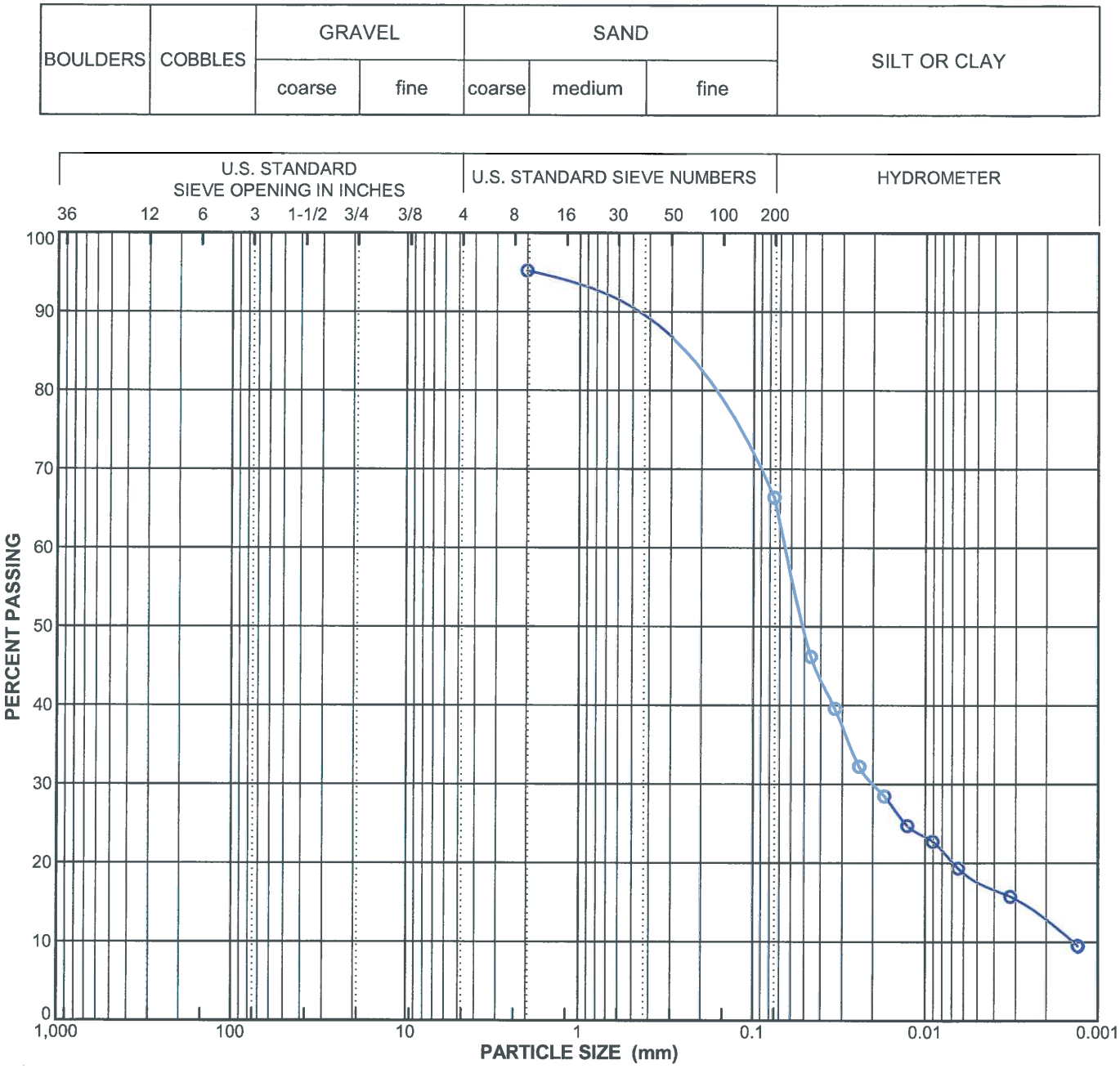


Symbol	Boring Number	Sample Number	Depth (feet)	Field Moisture (%)	LL	PI	Activity PI/-2μ	C_u	C_c	Passing No. 200 Sieve (%)	Passing 2μ (%)	USCS
○	H-1	B-1	0.9							56		CL
⊠	H-1	D-1	2.5	18	39	21	0.84			79	25	CL
▲	H-1	D-5	15.0	4						6		SP
★	H-2	D-2	5.0	18	43	23	0.88			79	26	CL
✿	P-2	B-1	1.0							58		CL

PARTICLE SIZE DISTRIBUTION

Segerstrom/Mesa Verde
Costa Mesa, California
PROJECT NO. 96079-03

NMG Geotechnical, Inc.



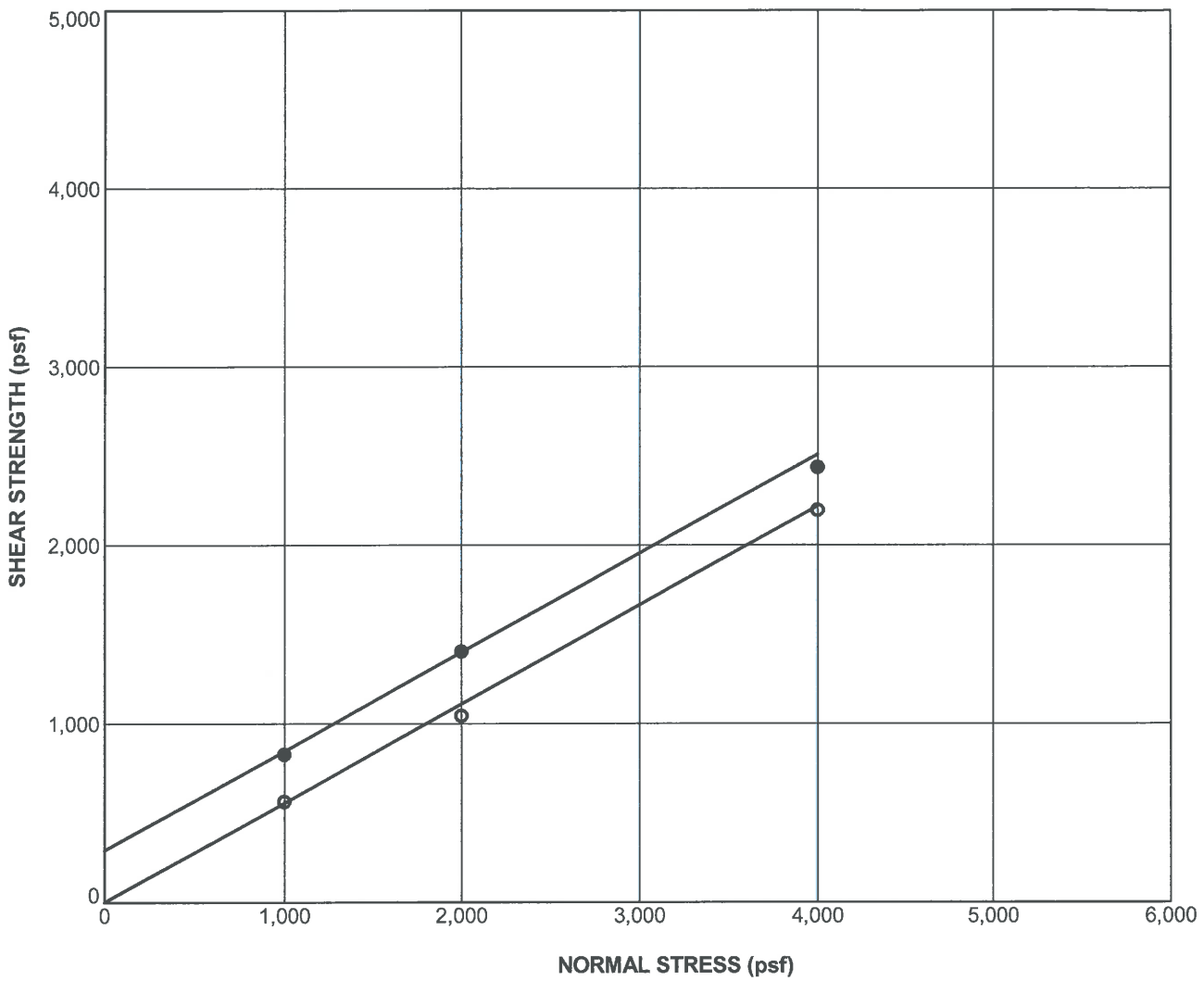
Symbol	Boring Number	Sample Number	Depth (feet)	Field Moisture (%)	LL	PI	Activity PI/-2μ	C_u	C_c	Passing No. 200 Sieve (%)	Passing 2μ (%)	USCS
O	P-2	D-3	7.5	17	29	10	0.83			66	12	CL

PARTICLE SIZE DISTRIBUTION

Segerstrom/Mesa Verde
Costa Mesa, California
PROJECT NO. 96079-03

NMG Geotechnical, Inc.

Template: NMSIV; Prj ID: 96079-03.GPJ; Printed: 2/23/17



Boring No. H-1		Sample No. B-1		Depth: 0.9 ft	
Sample Description: Brown sandy CLAY				USCS:	CL
Liquid Limit:		Plasticity Index:		Percent Passing No. 200 Sieve:	56
Moisture Content (%):	19.3	Dry Density (pcf):	115.2	Degree of Saturation (%):	100
Sample Type:	Remolded to 90% RC	Rate of Shear (in./min.):	0.005		

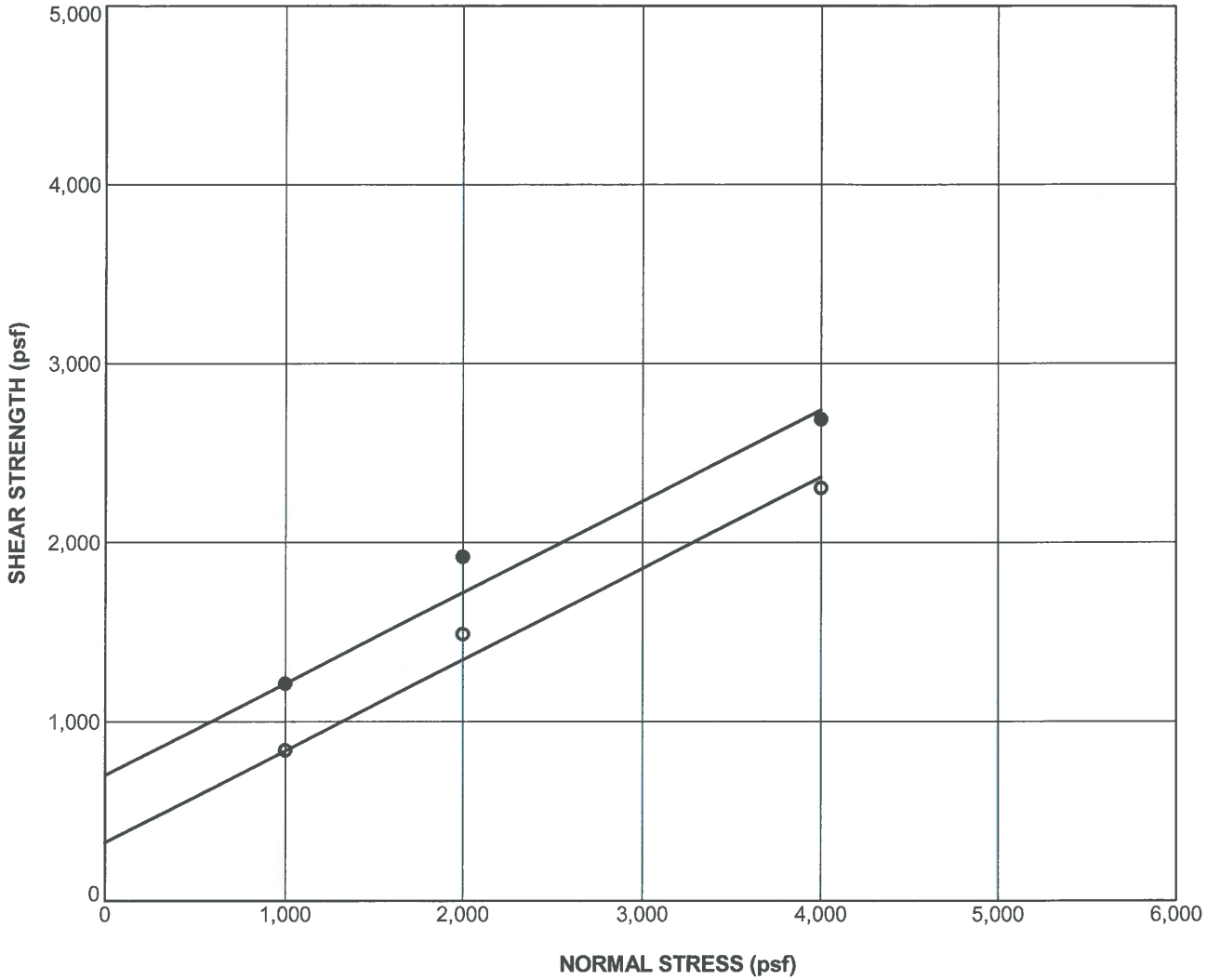
SHEAR STRENGTH PARAMETERS

Parameter	Peak ●	Ultimate O
Cohesion (psf)	290	0
Friction Angle (degrees)	29.0	29.0

DIRECT SHEAR TEST RESULTS

Segerstrom/Mesa Verde
Costa Mesa, California
PROJECT NO. 96079-03

NMG Geotechnical, Inc.



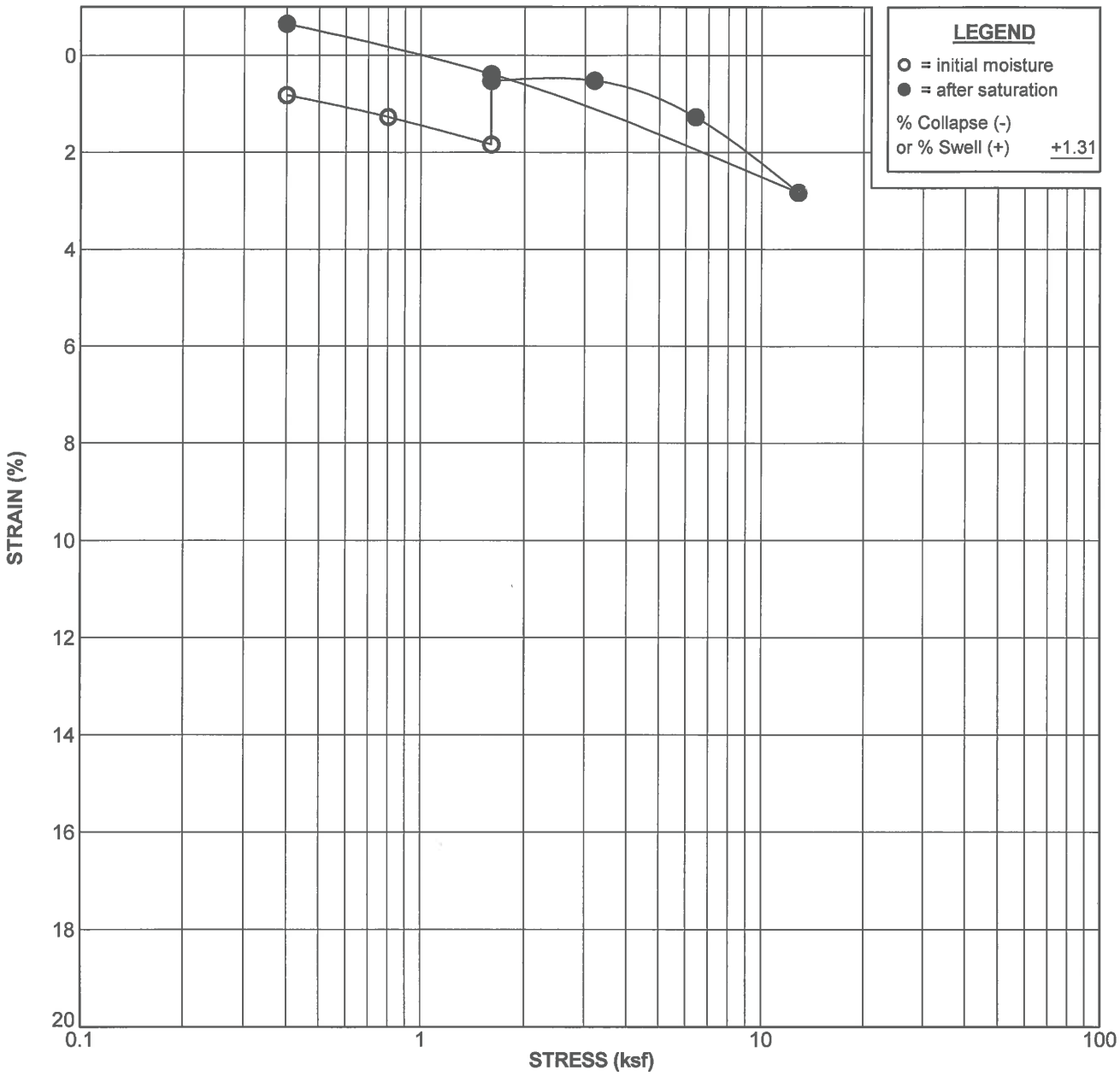
Boring No. H-2		Sample No. D-2		Depth: 5.0 ft	
Sample Description: Light brown silty CLAY				USCS:	CL
Liquid Limit:	43	Plasticity Index:	23	Percent Passing No. 200 Sieve:	79
Moisture Content (%):	21.4	Dry Density (pcf):	108.6	Degree of Saturation (%):	100
Sample Type:	Undisturbed	Rate of Shear (in./min.):	0.005		

SHEAR STRENGTH PARAMETERS		
Parameter	Peak ●	Ultimate ○
Cohesion (psf)	700	325
Friction Angle (degrees)	27.0	27.0

DIRECT SHEAR TEST RESULTS

Segerstrom/Mesa Verde
Costa Mesa, California
PROJECT NO. 96079-03

NMG Geotechnical, Inc.



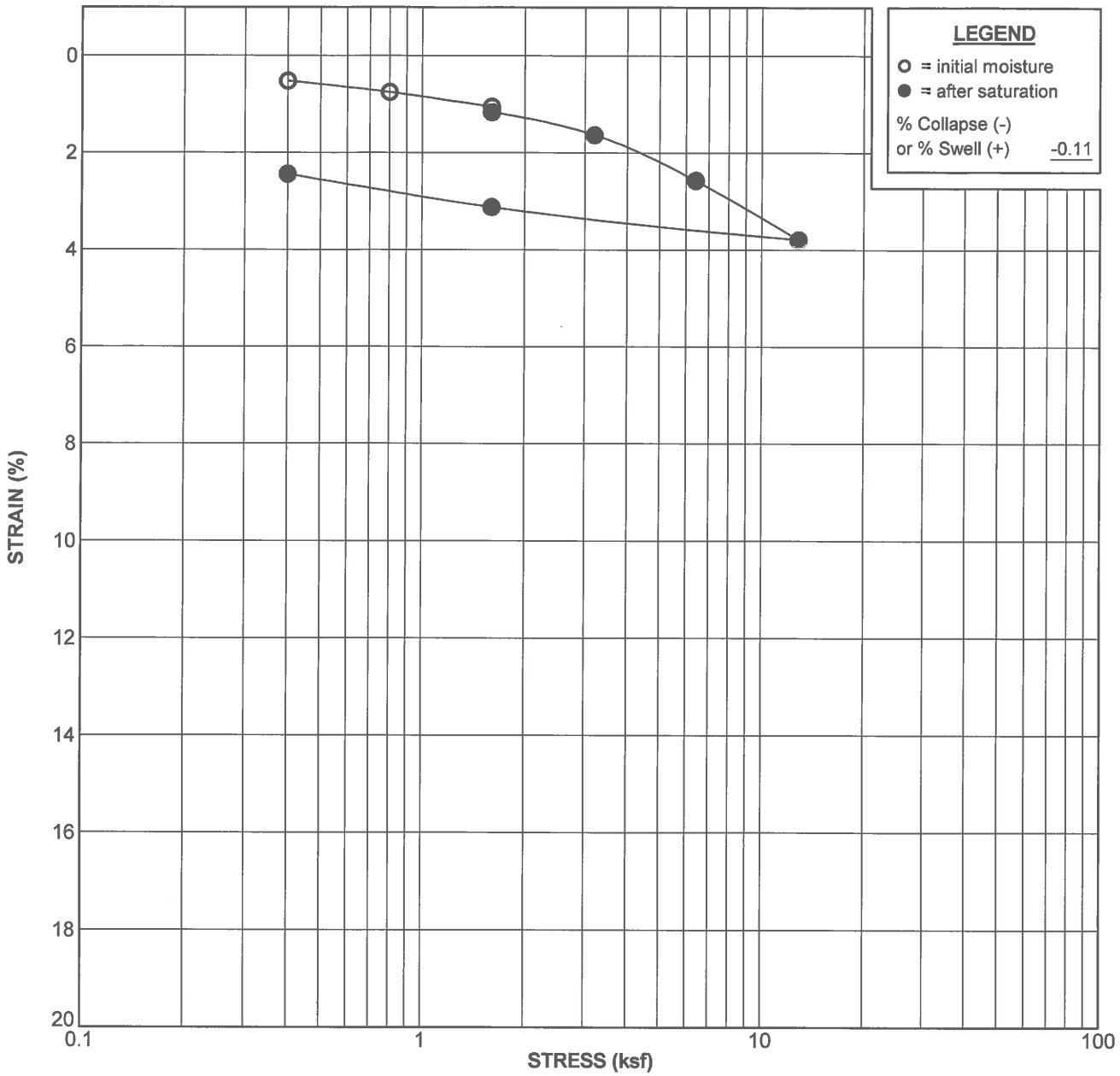
Boring No. H-1	Sample No. D-2	Depth: 5.0 ft
Sample Description: Brown silty sandy CLAY		USCS: CL
Liquid Limit:	Plasticity Index:	Percent Passing No. 200 Sieve:

Test Stage	Moisture Content (%)	Dry Density (pcf)	Degree of Saturation (%)	Void Ratio
Initial	19.4	109.1	88.7	0.619
Final	22.2	108.4	99.9	0.629

CONSOLIDATION TEST RESULTS

Segerstrom/Mesa Verde
Costa Mesa, California
PROJECT NO. 96079-03

NMG Geotechnical, Inc.



Boring No. P-1	Sample No. D-4	Depth: 10.0 ft
Sample Description: Brownish yellow clayey SAND		USCS: SC
Liquid Limit:	Plasticity Index:	Percent Passing No. 200 Sieve:

Test Stage	Moisture Content (%)	Dry Density (pcf)	Degree of Saturation (%)	Void Ratio
Initial	12.5	109.3	62.3	0.541
Final	17.1	112.0	91.6	0.504

CONSOLIDATION TEST RESULTS

Segerstrom/Mesa Verde
Costa Mesa, California
PROJECT NO. 96079-03

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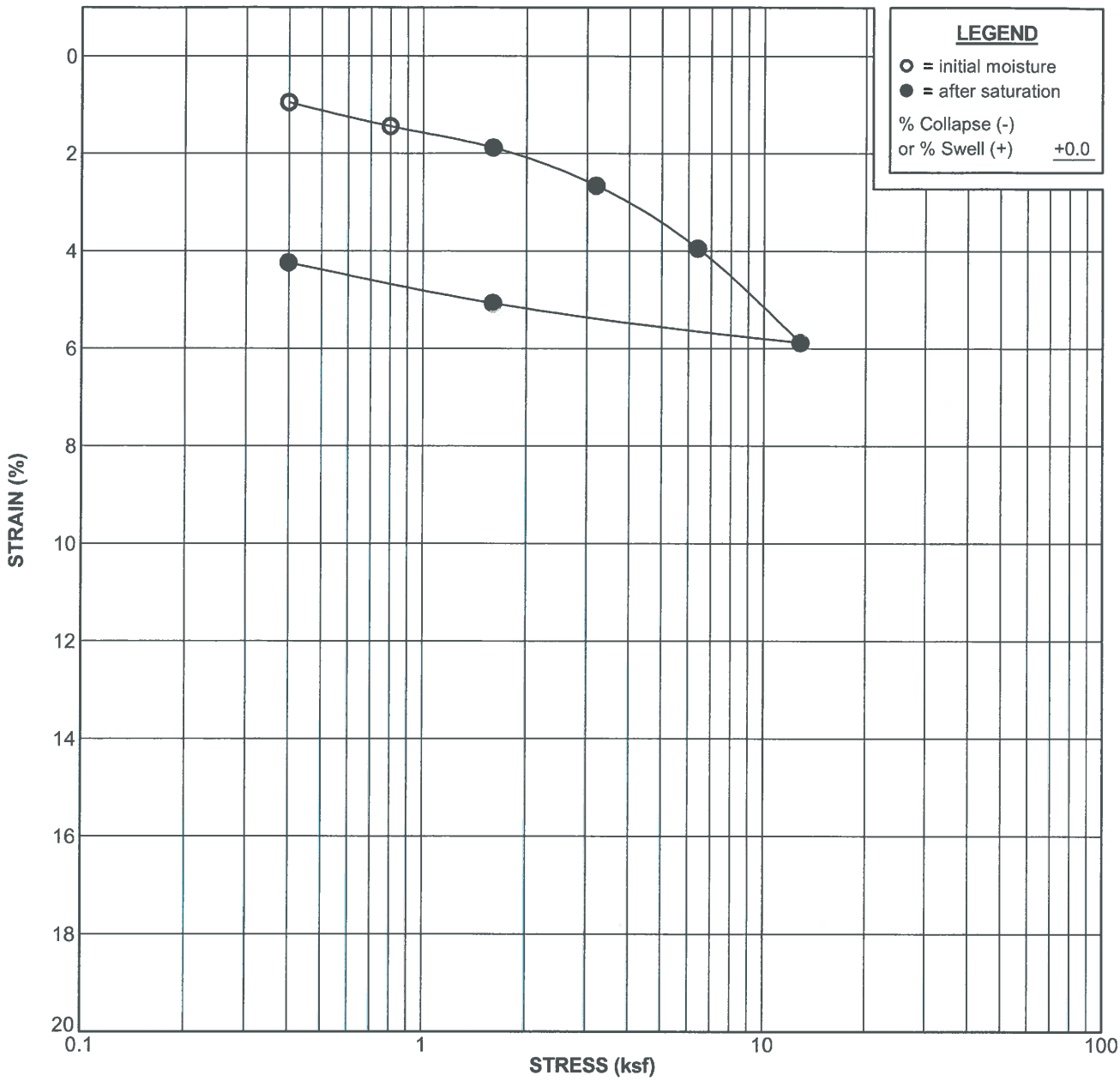


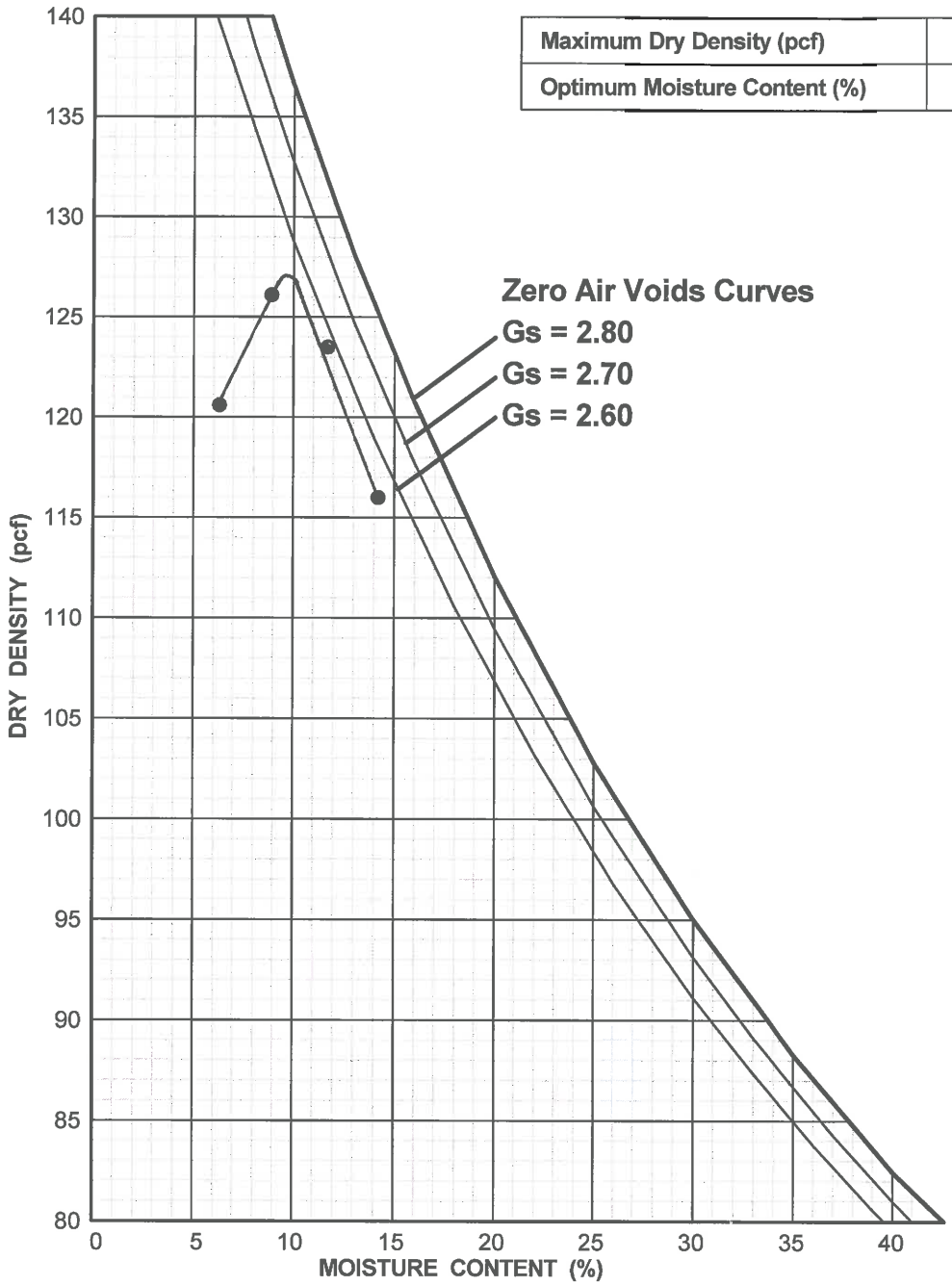
Boring No. P-2	Sample No. D-3	Depth: 7.5 ft
Sample Description: Yellowish brown sandy CLAY		USCS: CL
Liquid Limit: 29	Plasticity Index: 10	Percent Passing No. 200 Sieve: 66

Test Stage	Moisture Content (%)	Dry Density (pcf)	Degree of Saturation (%)	Void Ratio
Initial	17.6	110.7	89.3	0.537
Final	17.4	115.4	100.0	0.475

CONSOLIDATION TEST RESULTS

Segerstrom/Mesa Verde
Costa Mesa, California
PROJECT NO. 96079-03

NMG Geotechnical, Inc.



Maximum Dry Density (pcf)	127.0
Optimum Moisture Content (%)	9.5

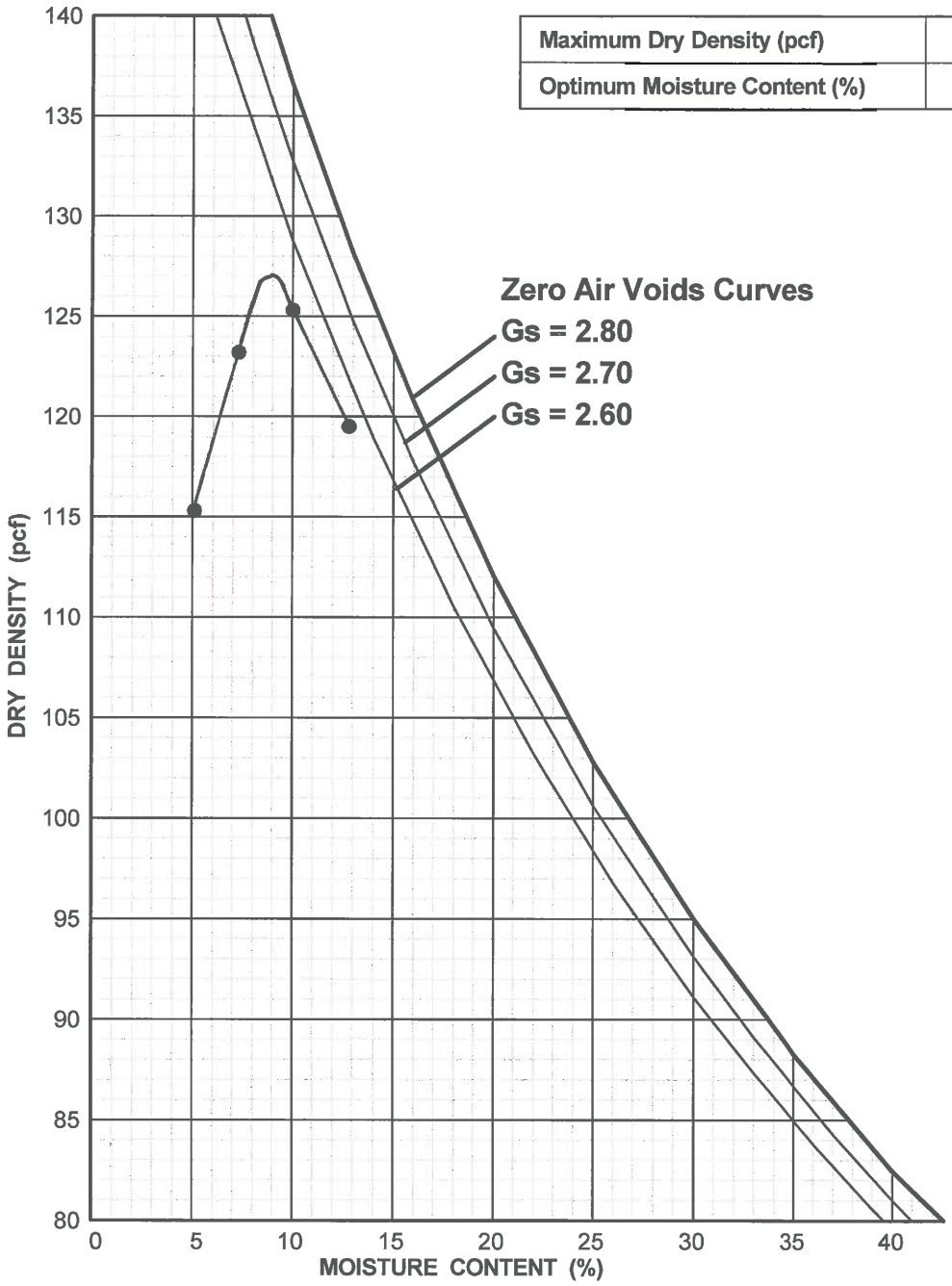
Boring No. H-1	**Sample No. B-1**	**Depth: 0.9 ft**
Sample Description: Brown sandy CLAY		**USCS:** CL
Liquid Limit:	**Plasticity Index:**	**Percent Passing No. 200 Sieve:** 56
Comments: 1557A		

COMPACTION TEST RESULTS

Segerstrom/Mesa Verde
Costa Mesa, California
PROJECT NO. 96079-03

NMG Geotechnical, Inc.

Template: NMCOMP; Prj ID: 96079-03.GPJ; Printed: 3/3/17



Maximum Dry Density (pcf)	127.0
Optimum Moisture Content (%)	9.0

Boring No. P-2	**Sample No. B-1**	**Depth: 1.0 ft**
Sample Description: Brown sandy CLAY		**USCS:** CL
Liquid Limit:	**Plasticity Index:**	**Percent Passing No. 200 Sieve:** 58
Comments: 1557A		

COMPACTION TEST RESULTS

Segerstrom/Mesa Verde
Costa Mesa, California
PROJECT NO. 96079-03

NMG Geotechnical, Inc.

Sample	Compacted Moisture (%)	Compacted Dry Density (pcf)	Final Moisture (%)	Volumetric Swell (%)	Expansion Index[1] Value/Method		Expansive Classification[2]	Soluble Sulfate (%)	Sulfate Exposure[3]
H-1 B-1 1-5'	9.5	113.6	17.6	3.7	39	B	Low		
P-2 B-1 0-5'	8.0	111.7	19.0	5.1	46	B	Low		

Test Method:
ASTM D4829

HACH SF-1 (Turbidimetric)

Notes:
1. Expansion Index (EI) method of determination:
   [A] E.I. determined by adjusting water content to achieve a 50 ±1% degree of saturation
   [B] E.I. calculated based on measured saturation within the range of 40% and 60%
2. ASTM D4829 *(Classification of Expansive Soil)*
3. ACI-318 Table 4.2.1 *(Requirement for Concrete Exposed to Sulfate-Containing Solutions)*

Expansion Index and Soluble Sulfate Test Results

(FRM001 Rev.5)

Project No. 96079-03

Project Name: Segerstrom/Mesa Verde

NMG

February 28, 2017 via email: bjimenez@nmggeotech.com

NMG GEOTECHNICAL, INC.
17991 Fitch
Irvine, CA 92614

Attention: Mr. Bryan Jimenez

Re: Soil Corrosivity Study
Segerstrom/Mesa Verde
Costa Mesa, California
HDR #17-0107SCS, NMG #96079-03

Introduction

Laboratory tests have been completed on one soil sample provided for the referenced project. The purpose of these tests was to determine if the soils might have deleterious effects on underground utility piping and concrete structures. HDR Engineering, Inc. (HDR) assumes that the sample provided is representative of the most corrosive soil at the site.

The proposed structure has one story and no subterranean levels. The site is located at 1555 Adams Avenue in Costa Mesa, California, and the water table is reportedly greater than 50 feet deep. Prior uses of the site include a single story retail building.

The scope of this study is limited to a determination of soil corrosivity and general corrosion control recommendations for materials likely to be used for construction. HDR's recommendations do not constitute, and are not meant as a substitute for, design documents for the purpose of construction. If the architects and/or engineers desire more specific information, designs, specifications, or review of design, HDR will be happy to work with them as a separate phase of this project.

431 W. Baseline Road, Claremont, CA 91711-1608
(909) 626-0967

Laboratory Soil Corrosivity Tests

The electrical resistivity of the soil sample was measured in a soil box per ASTM G187 in its as-received condition and again after saturation with distilled water. Resistivities are at about their lowest value when the soil is saturated. The pH of the saturated sample was measured per CTM 643. A 5:1 water:soil extract from the sample was chemically analyzed for the major soluble salts commonly found in soil per ASTM D4327, ASTM D6919, and Standard Method 2320-B[1]. Laboratory test results are shown in the attached Table 1.

Soil Corrosivity

A major factor in determining soil corrosivity is electrical resistivity. The electrical resistivity of a soil is a measure of its resistance to the flow of electrical current. Corrosion of buried metal is an electrochemical process in which the amount of metal loss due to corrosion is directly proportional to the flow of electrical current (DC) from the metal into the soil. Corrosion currents, following Ohm's Law, are inversely proportional to soil resistivity. Lower electrical resistivities result from higher moisture and soluble salt contents and indicate corrosive soil.

A correlation between electrical resistivity and corrosivity toward ferrous metals is:[2]

Soil Resistivity in ohm-centimeters	Corrosivity Category
Greater than 10,000	Mildly Corrosive
2,001 to 10,000	Moderately Corrosive
1,001 to 2,000	Corrosive
0 to 1,000	Severely Corrosive

Other soil characteristics that may influence corrosivity towards metals are pH, soluble salt content, soil types, aeration, anaerobic conditions, and site drainage.

---

[1] *American Public Health Association (APHA). 2012. Standard Methods of Water and Wastewater. 22nd ed. American Public Health Association, American Water Works Association, Water Environment Federation publication. APHA, Washington D.C.*

[2] *Romanoff, Melvin. Underground Corrosion, NBS Circular 579. Reprinted by NACE. Houston, TX, 1989, pp. 166–167.*

The electrical resistivity was in the moderately corrosive category with as-received moisture and at saturation.

Soil pH was 7.6. This value is mildly alkaline and does not particularly increase soil corrosivity.[3]

The soluble salt content of the sample was low.

Ammonium was detected in a low concentration.

Tests were not made for sulfide and oxidation-reduction (redox) potential because the sample did not exhibit characteristics typically associated with anaerobic conditions.

This soil is classified as moderately corrosive to ferrous metals.

Corrosion Control Recommendations

The life of buried materials depends on thickness, strength, loads, construction details, soil moisture, etc., in addition to soil corrosivity, and is, therefore, difficult to predict. Of more practical value are corrosion control methods that will increase the life of materials that would be subject to significant corrosion.

The following recommendations are based on the soil conditions discussed in the Soil Corrosivity section above. Unless otherwise indicated, these recommendations apply to the entire site or alignment.

Steel Pipe

Implement *all* the following measures:

1. Underground steel pipe with rubber gasketed, mechanical, grooved end, or other nonconductive type joints should be bonded for electrical continuity. Electrical continuity is necessary for corrosion monitoring and the possible future application of cathodic protection.

2. Install corrosion monitoring test stations to facilitate corrosion monitoring and the possible future application of cathodic protection:

---

[3] *Romanoff, Melvin. Underground Corrosion, NBS Circular 579. Reprinted by NACE. Houston, TX, 1989, p. 8.*

a. At each end of the pipeline.

b. At each end of all casings.

c. Other locations as necessary so the interval between test stations does not exceed 1,200 feet.

3. To prevent dissimilar metal corrosion cells and to facilitate the possible future application of cathodic protection, electrically isolate each buried steel pipeline per NACE SP0286 from:

   a. Dissimilar metals.

   b. Dissimilarly coated piping (cement-mortar vs. dielectric).

   c. Above ground steel pipe.

   d. All existing piping.

4. Choose one of the following corrosion control options:

   OPTION 1

   a. Apply a suitable dielectric coating intended for underground use such as:

      i. Polyurethane per AWWA C222 *or*

      ii. Extruded polyethylene per AWWA C215 *or*

      iii. A tape coating system per AWWA C214 *or*

      iv. Hot applied coal tar enamel per AWWA C203 *or*

      v. Fusion bonded epoxy per AWWA C213.

   b. Although it is customary to cathodically protect bonded dielectrically coated structures, cathodic protection is not recommended at this time due to moderately corrosive soils. Joint bonds, test stations, and insulated joints should still be installed and will facilitate the application of cathodic protection in the future if needed to control leaks.

OPTION 2

a. As an alternative to dielectric coating and possible future cathodic protection, apply a ¾-inch cement mortar coating per AWWA C205 or encase in concrete 3 inches thick, using any type of ASTM C150 cement. Joint bonds, test stations, and insulated joints are still recommended for these alternatives.

NOTE: Some steel piping systems, such as for oil, gas, and high-pressure piping systems, have special corrosion and cathodic protection requirements that must be evaluated for each specific application.

Iron Pipe

Implement *all* the following measures:

1. To prevent dissimilar metal corrosion cells and to facilitate the possible future application of cathodic protection, electrically insulate underground iron pipe from dissimilar metals and from above ground iron pipe with insulating joints per NACE SP0286.

2. Bond all nonconductive type joints for electrical continuity. Electrical continuity is necessary for corrosion monitoring and possible future application of cathodic protection.

3. Install corrosion monitoring test stations to facilitate corrosion monitoring and the possible future application of cathodic protection:

   a. At each end of the pipeline.

   b. At each end of any casings.

   c. Other locations as necessary so the interval between test stations does not exceed 1,200 feet.

4. Choose one of the following corrosion control options:

   OPTION 1

   a. Apply a suitable coating intended for underground use such as:

      i. Polyethylene encasement per AWWA C105; *or*

ii. Epoxy coating; *or*

iii. Polyurethane; *or*

iv. Wax tape.

NOTE: The thin factory-applied asphaltic coating applied to ductile iron pipe for transportation and aesthetic purposes does not constitute a corrosion control coating.

b. Although it is customary to cathodically protect coated structures, cathodic protection is not recommended at this time due to moderately corrosive soils. Joint bonds, test stations, and insulated joints should still be installed and will facilitate the application of cathodic protection in the future if needed to control leaks.

OPTION 2

a. As an alternative to coating systems described in Option 1 and possible future cathodic protection, concrete encase all buried portions of metallic piping so that there is a minimum of 3 inches of concrete cover provided over and around surfaces of pipe, fittings, and valves using any type of ASTM C150 cement.

Copper Tubing

Implement *all* the following measures:

1. Electrically insulate underground copper pipe from dissimilar metals and from above ground copper pipe with insulating devices per NACE SP0286.

2. Electrically insulate cold water piping from hot water piping systems.

3. Place cold water copper tubing in an 8-mil polyethylene sleeve or encase in double 4-mil thick polyethylene sleeves and bed and backfill with clean sand at least 2 inches thick surrounding the tubing. Clean sand should have a minimum resistivity of no less than 3,000 ohm-cm, and a pH of 6.0–8.0. Copper tubing for cold water can also be treated the same as for hot water.

4. Hot water tubing may be subject to a higher corrosion rate. Protect hot copper tubing by one of the following measures:

a. Preventing soil contact. Soil contact may be prevented by placing the tubing above ground or encasing the tubing with PVC pipe with solvent-welded joints. *or*

b. Applying cathodic protection per NACE SP0169. The amount of cathodic protection current needed can be minimized by coating the tubing.

Plastic and Vitrified Clay Pipe

1. No special precautions are required for plastic and vitrified clay piping placed underground from a corrosion viewpoint.

2. Protect all metallic fittings and valves with wax tape per AWWA C217 or epoxy.

All Pipe

1. On all pipes, appurtenances, and fittings not protected by cathodic protection, coat bare metal such as valves, bolts, flange joints, joint harnesses, and flexible couplings with wax tape per AWWA C217 after assembly.

2. Where metallic pipelines penetrate concrete structures such as building floors, vault walls, and thrust blocks use plastic sleeves, rubber seals, or other dielectric material to prevent pipe contact with the concrete and reinforcing steel.

Concrete

1. From a corrosion standpoint, any type of ASTM C150 cement may be used for concrete structures and pipe because the sulfate concentration is negligible, 0 to 0.10 percent.[4,5,6]

2. Standard concrete cover over reinforcing steel may be used for concrete structures and pipe in contact with these soils due to the low chloride concentration[7] found onsite.

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[4] *2015 International Building Code (IBC) which refers to American Concrete Institute (ACI) 318 Table 19.3.2.1*

[5] *2012 International Residential Code (IRC) which refers to American Concrete Institute (ACI) 318 Table 19.3.2.1*

[6] *2013 California Building Code (CBC) which refers to American Concrete Institute (ACI) 318 Table 19.3.2.1*

[7] *Design Manual 303: Concrete Cylinder Pipe. Ameron. p.65*

Post Tensioning Slabs: Unbonded Single-Stranded Tendons and Anchors

Soil is considered an aggressive environment for post-tensioning strands and anchors. Protect post-tensioning strands and anchors against corrosion by implementing *all* the following measures:[8,9,10]

1. Prior to grouting the pocket, apply a corrosion protection cap filled with corrosion protection material that provides a watertight seal for the strand end and wedge cavity such as Tiger Industries' PocketCap or equal. Ensure the cap fully seats against the face of the standard anchor at the live end.
2. All components exposed to the job site should be protected within one working day after their exposure during installation.
3. Ensure the minimum concrete cover over the tendon tail is 1 inch, or greater if required by the applicable building code.
4. Caps should be installed within one working day after the cutting of the tendon tails and acceptance of the elongation records by the engineer.
5. Limit the access of direct runoff onto the anchorage area by designing proper drainage.
6. Provide at least 2 inches of space between finish grade and the anchorage area, or more if required by applicable building codes.

---

[8] *Post-Tensioning Manual, sixth edition. Post-Tensioning Institute (PTI), Phoenix, AZ, 2006.*

[9] *PTI M10.2-00: Specification for Unbonded Single Strand Tendons. Post-Tensioning Institute (PTI), Phoenix, AZ, 2000.*

[10] *ACI 423.6-01: Specification for Unbonded Single Strand Tendons. American Concrete Institute (ACI), 2001*

Closure

The analysis and recommendations presented in this report are based upon data obtained from the laboratory sample. This report does not reflect variations that may occur across the site or due to the modifying effects of construction. If variations appear, HDR should be notified immediately so that further evaluation and supplemental recommendations can be provided.

HDR's services have been performed with the usual thoroughness and competence of the engineering profession. No other warranty or representation, either expressed or implied, is included or intended.

Please call if you have any questions.

Respectfully Submitted,
HDR Engineering, Inc.

REGISTERED PROFESSIONAL ENGINEER
GREGORY K FROST
No. 83780
EXP. 3/31/17
CIVIL
STATE OF CALIFORNIA

James Keegan

Greg Frost, PE

Enc: Table 1

17-0107SCS SCS JK-GF.docx

Table 1 - Laboratory Tests on Soil Samples

NMG Geotechnical, Inc.
Segerstrom/Mesa Verde
Your #96079-03, HDR Lab #17-0107SCS
27-Feb-17

Sample ID			H-1 @ 1'-5'
Resistivity		**Units**	
as-received		ohm-cm	8,800
saturated		ohm-cm	2,240
pH			7.6
Electrical Conductivity		mS/cm	0.15
Chemical Analyses			
Cations			
calcium	Ca^{2+}	mg/kg	54
magnesium	Mg^{2+}	mg/kg	6.4
sodium	Na^{1+}	mg/kg	132
potassium	K^{1+}	mg/kg	15
Anions			
carbonate	CO_3^{2-}	mg/kg	ND
bicarbonate	HCO_3^{1-}	mg/kg	308
fluoride	F^{1-}	mg/kg	8.1
chloride	Cl^{1-}	mg/kg	20
sulfate	SO_4^{2-}	mg/kg	64
phosphate	PO_4^{3-}	mg/kg	3.5
Other Tests			
ammonium	NH_4^{1+}	mg/kg	1.3
nitrate	NO_3^{1-}	mg/kg	ND
sulfide	S^{2-}	qual	na
Redox		mV	na

Resistivity per ASTM G187, Cations per ASTM D6919, Anions per ASTM D4327, and Alkalinity per APHA 2320-B.
Electrical conductivity in millisiemens/cm and chemical analyses were made on a 1:5 soil-to-water extract.
mg/kg = milligrams per kilogram (parts per million) of dry soil.
Redox = oxidation-reduction potential in millivolts
ND = not detected
na = not analyzed

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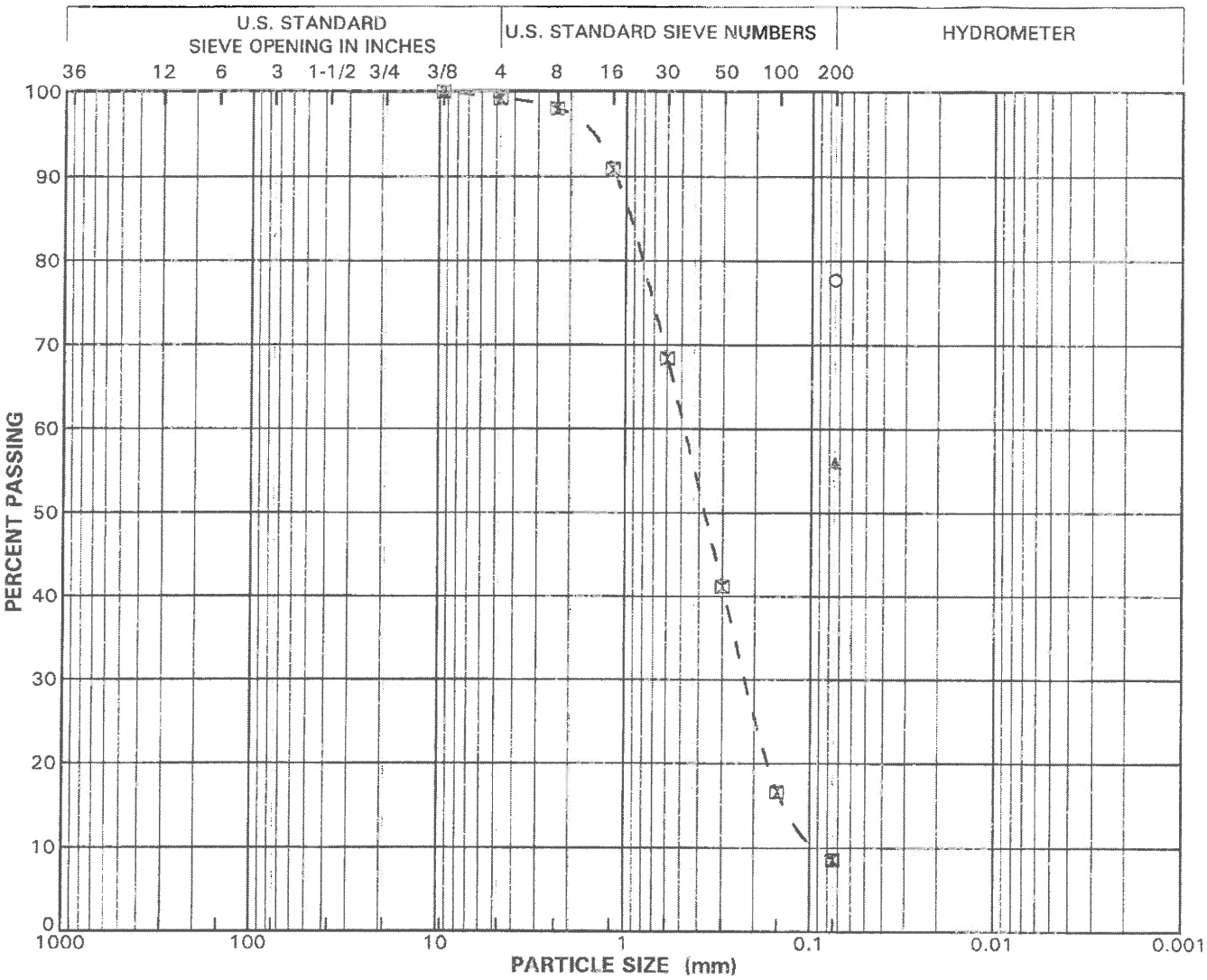
APPENDIX D

(NMG, 1996)

BOULDERS	COBBLES	GRAVEL		SAND			SILT OR CLAY
		coarse	fine	coarse	medium	fine	



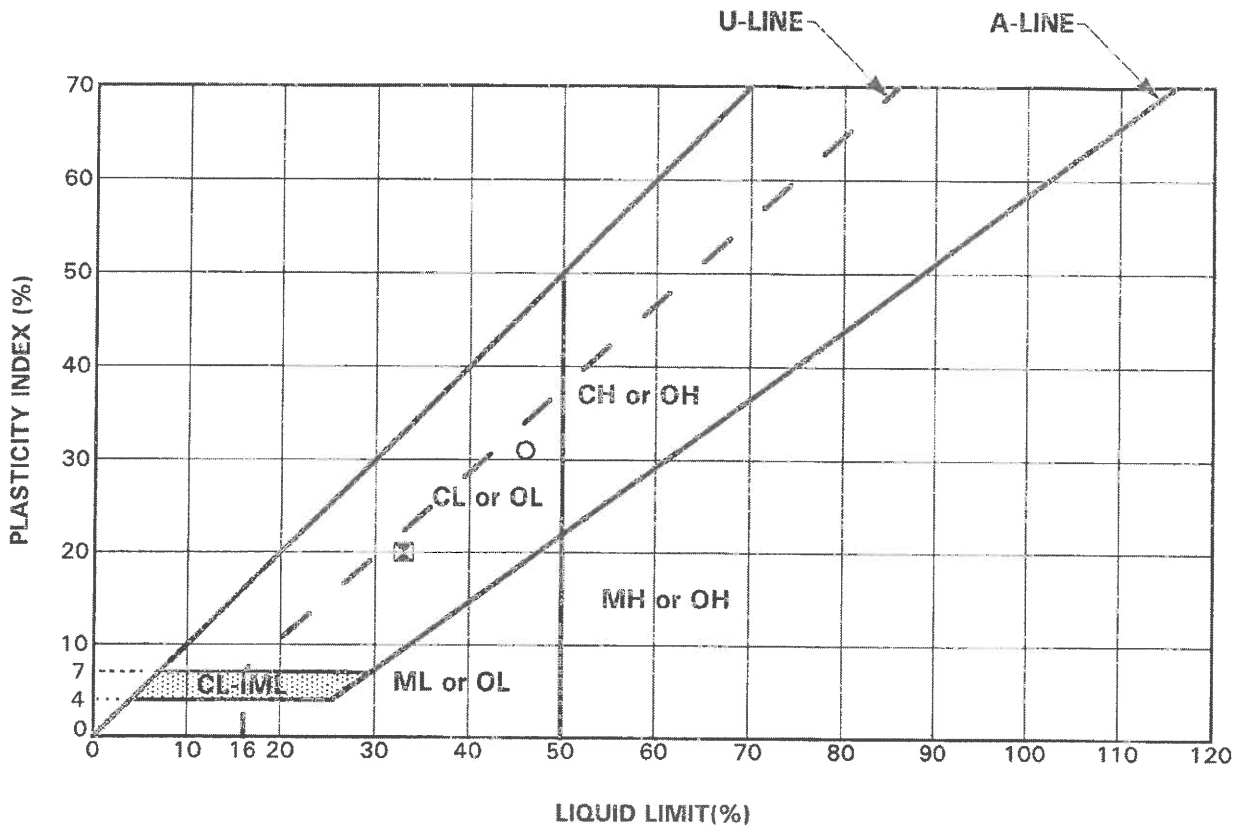
Symbol	Boring Number	Sample Number	Depth (feet)	Field Moisture (%)	LL	PI	Activity PI/-2μ	C_u	C_c	Passing No. 200 Sieve (%)	Passing 2μ (%)	USCS
○	B- 1	D3	6.0	21	46	31				78		CL
⊠	B- 1	D5	15.0	3				5.73	1.17	9		SP
▲	B- 3	D1	1.5	15	33	20				56		CL

PARTICLE SIZE DISTRIBUTION

MESA VERDE SAVE-ON
COSTA MESA
PROJECT NO. 95130-1

NMG Geotechnical, Inc.

Template: NMSIV; Prj ID: MVSON; Printed: 4/9/96

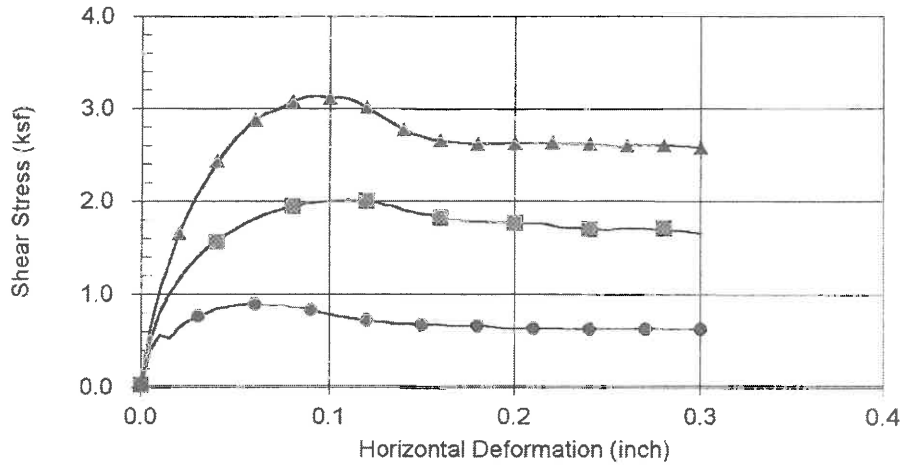


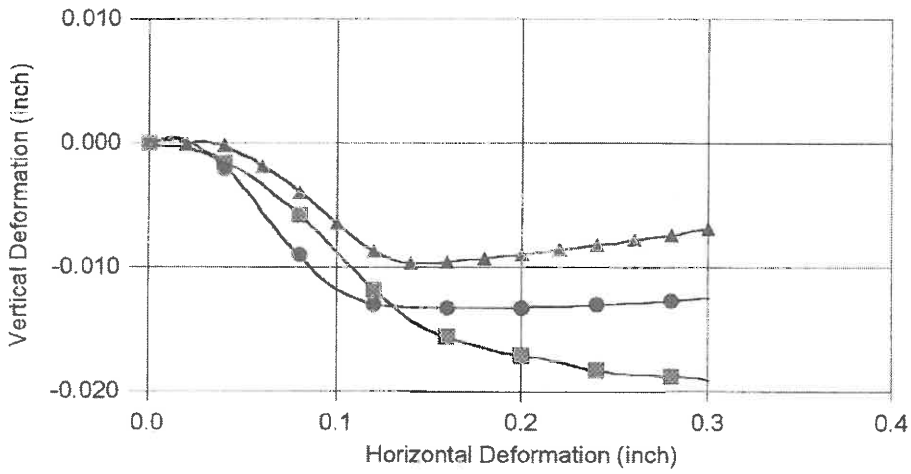
Symbol	Boring Number	Depth (feet)	Sample Number	Passing No. 200 Sieve (%)	LL	PI	USCS	Description
O	B- 1	6.0	D3	78	46	31	CL	Silty CLAY
⊠	B- 3	1.5	D1	56	33	20	CL	Sandy CLAY

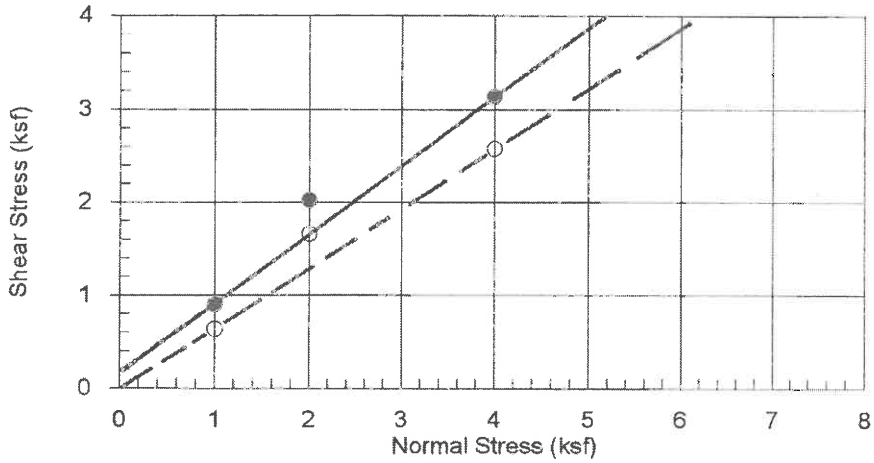
PLASTICITY CHART
MESA VERDE SAVE-ON
COSTA MESA
PROJECT NO. 95130-1

NMG Geotechnical, Inc.

Template: NMATT; Prj ID: MVSON; Printed: 4/9/96







Project Name : Mesa Verde Sav-on
Boring No. : B-2
Sample No. : D-1
Depth (ft) : 1.5-2.5
Sample Type : Undisturbed
Soil Type : Olive Brown Silty Sand
Initial Dry Density : 104.7 pcf
Moisture Content : 11.9 % (before)

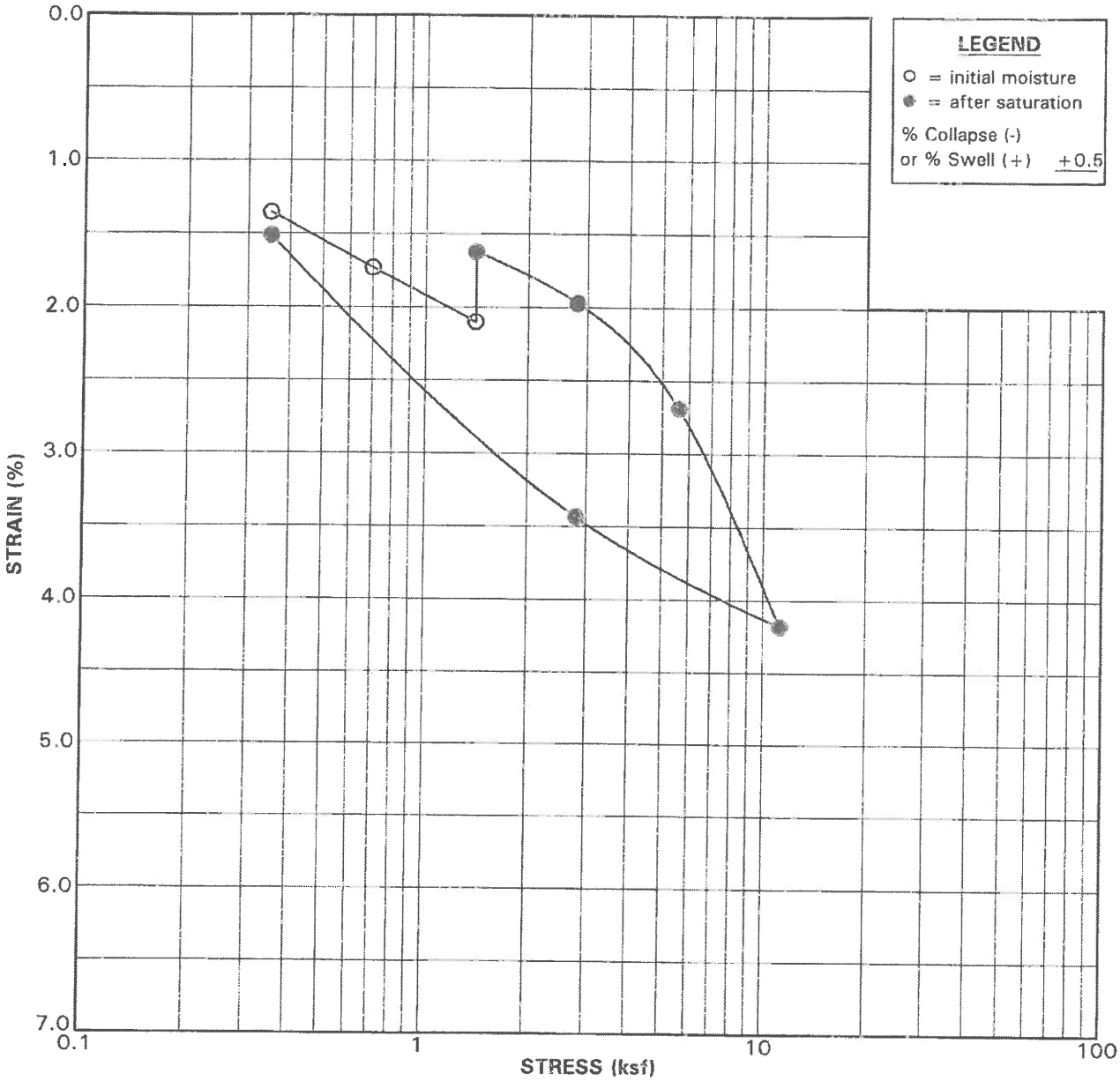
	Peak	Residual
Cohesion (ksf)	0.2	0.0
Phi (Degrees)	36	33

NMG Geotechnical, Inc.

DIRECT SHEAR
TEST RESULTS
(ASTM D3080)

4-96

Figure No.



Boring No. B- 3	Sample No. D1	Depth: 1.5 ft
Sample Description: Sandy CLAY		
Liquid Limit: 33	Plasticity Index: 20	Percent Passing No. 200 Sieve: 56

Test Stage	Moisture Content (%)	Dry Density (pcf)	Degree of Saturation (%)	Void Ratio
Initial	17.3	113.1	95.3	0.490
Final	18.1			

CONSOLIDATION TEST RESULTS

MESA VERDE SAVE-ON
COSTA MESA
PROJECT NO. 95130-1

NMG Geotechnical, Inc.

Template: NMCONS; Prj ID: MVSON; Printed: 4/9/96

Expansion Index Test Results

Sample No.	Compacted Moisture (%)	Compacted Dry Density (pcf)	Final Moisture (%)	Compacted Expans. Index	U.B.C. Classification
B-3, Bag 1, 1.5'-3'	9.1	110.3	20.2	51	Medium

TEST METHOD, U.B.C. STANDARD NO. 29-2

APPENDIX E

*** Deaggregation of Seismic Hazard at One Period of Spectral Accel. ***
*** Data from U.S.G.S. National Seismic Hazards Mapping Project, 2008 version ***
PSHA Deaggregation. %contributions. site: Mesa_Verde_ long: 117.921 W., lat: 33.672 N.
Vs30(m/s)= 300.0 (some WUS atten. models use Site Class not Vs30).
NSHMP 2007-08 See USGS OFR 2008-1128. dM=0.2 below
Return period: 2475 yrs. Exceedance PGA =0.6654 g. Weight * Computed_Rate_Ex
0.399E-03
#Pr[at least one eq with median motion>=PGA in 50 yrs]=0.00418
#This deaggregation corresponds to Mean Hazard w/all GMPEs

DIST(KM)	MAG(MW)	ALL_EPS	EPSILON>2	1<EPS<2	0<EPS<1	-1<EPS<0	-2<EPS<-1	EPS<-2
7.1	5.05	1.073	0.787	0.286	0.000	0.000	0.000	0.000
12.6	5.05	0.088	0.088	0.000	0.000	0.000	0.000	0.000
7.2	5.20	2.270	1.373	0.896	0.000	0.000	0.000	0.000
12.9	5.20	0.242	0.242	0.000	0.000	0.000	0.000	0.000
7.3	5.40	2.363	1.122	1.241	0.000	0.000	0.000	0.000
13.2	5.40	0.339	0.339	0.000	0.000	0.000	0.000	0.000
7.4	5.60	2.290	0.934	1.248	0.108	0.000	0.000	0.000
13.4	5.60	0.432	0.431	0.001	0.000	0.000	0.000	0.000
7.4	5.80	2.076	0.723	1.178	0.175	0.000	0.000	0.000
13.7	5.80	0.505	0.483	0.022	0.000	0.000	0.000	0.000
7.0	6.01	2.619	0.703	1.701	0.214	0.000	0.000	0.000
14.1	6.01	0.661	0.592	0.069	0.000	0.000	0.000	0.000
7.0	6.20	3.360	0.765	2.203	0.392	0.000	0.000	0.000
14.1	6.20	0.834	0.682	0.152	0.000	0.000	0.000	0.000
23.5	6.21	0.099	0.099	0.000	0.000	0.000	0.000	0.000
7.2	6.40	3.248	0.587	2.118	0.543	0.000	0.000	0.000
14.1	6.39	0.978	0.671	0.306	0.000	0.000	0.000	0.000
24.3	6.42	0.251	0.251	0.000	0.000	0.000	0.000	0.000
3.8	6.61	13.553	1.215	5.362	6.489	0.487	0.000	0.000
13.2	6.60	0.538	0.343	0.194	0.000	0.000	0.000	0.000
23.7	6.60	0.350	0.349	0.000	0.000	0.000	0.000	0.000
33.1	6.59	0.126	0.126	0.000	0.000	0.000	0.000	0.000
3.8	6.80	16.444	1.472	6.097	8.039	0.836	0.000	0.000
13.4	6.79	0.581	0.315	0.266	0.000	0.000	0.000	0.000
25.4	6.79	0.554	0.547	0.006	0.000	0.000	0.000	0.000
34.4	6.77	0.310	0.310	0.000	0.000	0.000	0.000	0.000
3.9	6.97	20.431	1.759	7.798	9.617	1.257	0.000	0.000
13.4	6.98	0.361	0.168	0.190	0.003	0.000	0.000	0.000
26.3	7.01	1.093	0.928	0.165	0.000	0.000	0.000	0.000
35.1	6.97	0.289	0.288	0.001	0.000	0.000	0.000	0.000
3.9	7.15	9.186	0.734	3.448	4.216	0.787	0.001	0.000
24.6	7.20	0.932	0.660	0.272	0.000	0.000	0.000	0.000
35.3	7.18	0.200	0.187	0.013	0.000	0.000	0.000	0.000
44.5	7.25	0.056	0.056	0.000	0.000	0.000	0.000	0.000
4.8	7.39	4.309	0.441	1.899	1.833	0.135	0.001	0.000
23.3	7.37	0.883	0.526	0.357	0.000	0.000	0.000	0.000
35.2	7.35	0.205	0.172	0.033	0.000	0.000	0.000	0.000
44.3	7.39	0.094	0.094	0.000	0.000	0.000	0.000	0.000
5.0	7.55	2.373	0.281	1.339	0.731	0.022	0.000	0.000
24.6	7.59	0.478	0.264	0.214	0.000	0.000	0.000	0.000
35.1	7.57	0.168	0.133	0.035	0.000	0.000	0.000	0.000
47.7	7.58	0.120	0.118	0.002	0.000	0.000	0.000	0.000
77.0	7.57	0.063	0.063	0.000	0.000	0.000	0.000	0.000
5.0	7.71	0.357	0.037	0.166	0.146	0.009	0.000	0.000
23.8	7.74	0.967	0.440	0.527	0.000	0.000	0.000	0.000
35.2	7.76	0.341	0.255	0.086	0.000	0.000	0.000	0.000
49.2	7.76	0.063	0.060	0.004	0.000	0.000	0.000	0.000
76.5	7.78	0.150	0.150	0.000	0.000	0.000	0.000	0.000
24.3	7.91	0.137	0.057	0.079	0.001	0.000	0.000	0.000
78.9	7.98	0.166	0.166	0.000	0.000	0.000	0.000	0.000

Summary statistics for above PSHA PGA deaggregation, R=distance, e=epsilon:
Contribution from this GMPE(%): 100.0

Mean src-site R= 7.4 km; M= 6.70; eps0= 0.69. Mean calculated for all sources.
Modal src-site R= 3.9 km; M= 6.97; eps0= 0.23 from peak (R,M) bin
MODE R*= 3.2km; M*= 6.96; EPS.INTERVAL: 0 to 1 sigma % CONTRIB.= 9.617

Principal sources (faults, subduction, random seismicity having > 3% contribution)

Source Category:	% contr.	R(km)	M	epsilon0 (mean values).
California B-faults Char	40.40	6.2	7.08	0.44
California B-faults GR	30.16	4.5	6.79	0.32
California A-faults	3.16	39.2	7.35	2.16
CA Compr. crustal gridded	26.27	8.6	5.93	1.35

Individual fault hazard details if its contribution to mean hazard > 2%:

Fault ID	% contr.	Rcd(km)	M	epsilon0	Site-to-src azimuth(d)
Newport-Inglewood (Offshore) Cha	4.80	9.1	6.89	1.30	176.2
Newport-Inglewood, alt 1 Char	2.55	5.1	7.15	0.67	-128.5
Newport-Inglewood, alt 2 Char	2.55	4.9	7.15	0.68	-129.1
San Joaquin Hills Char	21.90	2.8	6.97	-0.07	4.3
Newport Inglewood Connected alt	2.44	5.1	7.51	0.57	-128.5
Newport Inglewood Connected alt	2.48	4.9	7.51	0.56	-129.1
San Joaquin Hills GR	22.90	3.0	6.74	0.06	18.0

#*********End of deaggregation corresponding to Mean Hazard w/all GMPEs *********#

PSHA Deaggregation. %contributions. site: Mesa_Verde_ long: 117.921 W., lat: 33.672 N.
Vs30(m/s)= 300.0 (some WUS atten. models use Site Class not Vs30).
NSHMP 2007-08 See USGS OFR 2008-1128. dM=0.2 below
Return period: 2475 yrs. Exceedance PGA =0.6654 g. Weight * Computed_Rate_Ex
0.139E-03
#Pr[at least one eq with median motion>=PGA in 50 yrs]=0.00194
#This deaggregation corresponds to Boore-Atkinson 2008

DIST(KM)	MAG(MW)	ALL_EPS	EPSILON>2	1<EPS<2	0<EPS<1	-1<EPS<0	-2<EPS<-1	EPS<-2
6.5	5.05	0.099	0.099	0.000	0.000	0.000	0.000	0.000
6.6	5.20	0.246	0.246	0.000	0.000	0.000	0.000	0.000
6.8	5.40	0.311	0.289	0.023	0.000	0.000	0.000	0.000
7.0	5.60	0.361	0.302	0.058	0.000	0.000	0.000	0.000
12.9	5.61	0.026	0.026	0.000	0.000	0.000	0.000	0.000
7.1	5.80	0.390	0.314	0.076	0.000	0.000	0.000	0.000
13.6	5.81	0.059	0.059	0.000	0.000	0.000	0.000	0.000
6.6	6.02	0.637	0.421	0.216	0.000	0.000	0.000	0.000
14.3	6.01	0.119	0.119	0.000	0.000	0.000	0.000	0.000
6.6	6.20	0.883	0.525	0.359	0.000	0.000	0.000	0.000
14.4	6.20	0.189	0.189	0.000	0.000	0.000	0.000	0.000
24.1	6.22	0.044	0.044	0.000	0.000	0.000	0.000	0.000
6.9	6.40	0.851	0.470	0.382	0.000	0.000	0.000	0.000
14.5	6.40	0.254	0.254	0.000	0.000	0.000	0.000	0.000
25.3	6.42	0.129	0.129	0.000	0.000	0.000	0.000	0.000
32.0	6.41	0.030	0.030	0.000	0.000	0.000	0.000	0.000
4.1	6.62	4.064	0.540	1.852	1.625	0.048	0.000	0.000
13.5	6.60	0.245	0.190	0.055	0.000	0.000	0.000	0.000
24.3	6.60	0.251	0.250	0.000	0.000	0.000	0.000	0.000
33.2	6.59	0.119	0.119	0.000	0.000	0.000	0.000	0.000
4.2	6.80	5.313	0.623	2.275	2.287	0.128	0.000	0.000
13.8	6.79	0.289	0.197	0.092	0.000	0.000	0.000	0.000
25.9	6.79	0.444	0.438	0.006	0.000	0.000	0.000	0.000
34.4	6.77	0.307	0.307	0.000	0.000	0.000	0.000	0.000
44.6	6.77	0.017	0.017	0.000	0.000	0.000	0.000	0.000
4.4	6.97	7.146	0.756	3.271	2.923	0.196	0.000	0.000
13.8	6.98	0.196	0.105	0.089	0.002	0.000	0.000	0.000
26.4	7.01	0.923	0.773	0.151	0.000	0.000	0.000	0.000
35.1	6.97	0.284	0.283	0.001	0.000	0.000	0.000	0.000
47.5	7.02	0.026	0.026	0.000	0.000	0.000	0.000	0.000
4.2	7.16	3.226	0.292	1.432	1.355	0.148	0.000	0.000
13.2	7.19	0.023	0.007	0.015	0.001	0.000	0.000	0.000
24.4	7.20	0.708	0.475	0.233	0.000	0.000	0.000	0.000

35.3	7.18	0.189	0.176	0.013	0.000	0.000	0.000	0.000
44.5	7.25	0.056	0.056	0.000	0.000	0.000	0.000	0.000
52.7	7.19	0.032	0.032	0.000	0.000	0.000	0.000	0.000
4.9	7.39	1.796	0.166	0.872	0.745	0.013	0.000	0.000
14.3	7.40	0.020	0.006	0.013	0.001	0.000	0.000	0.000
23.3	7.37	0.696	0.393	0.303	0.000	0.000	0.000	0.000
35.2	7.35	0.190	0.157	0.033	0.000	0.000	0.000	0.000
44.3	7.39	0.091	0.091	0.000	0.000	0.000	0.000	0.000
52.9	7.35	0.034	0.034	0.000	0.000	0.000	0.000	0.000
5.0	7.55	1.072	0.130	0.657	0.286	0.000	0.000	0.000
24.9	7.60	0.347	0.174	0.173	0.000	0.000	0.000	0.000
35.1	7.57	0.147	0.112	0.035	0.000	0.000	0.000	0.000
47.8	7.58	0.115	0.113	0.002	0.000	0.000	0.000	0.000
77.0	7.57	0.063	0.063	0.000	0.000	0.000	0.000	0.000
5.0	7.71	0.157	0.015	0.079	0.063	0.000	0.000	0.000
23.9	7.74	0.665	0.264	0.401	0.000	0.000	0.000	0.000
35.2	7.76	0.283	0.197	0.086	0.000	0.000	0.000	0.000
49.2	7.76	0.057	0.054	0.004	0.000	0.000	0.000	0.000
76.5	7.78	0.150	0.150	0.000	0.000	0.000	0.000	0.000
24.5	7.91	0.092	0.032	0.060	0.001	0.000	0.000	0.000
35.2	7.93	0.026	0.016	0.011	0.000	0.000	0.000	0.000
78.9	7.98	0.163	0.163	0.000	0.000	0.000	0.000	0.000
79.3	8.21	0.018	0.018	0.000	0.000	0.000	0.000	0.000

Summary statistics for above PSHA PGA deaggregation, R=distance, e=epsilon:
Contribution from this GMPE(%): 34.8
Mean src-site R= 10.4 km; M= 6.89; eps0= 0.93. Mean calculated for all sources.
Modal src-site R= 4.4 km; M= 6.97; eps0= 0.46 from peak (R,M) bin
MODE R*= 5.1km; M*= 6.98; EPS.INTERVAL: 0 to 1 sigma % CONTRIB.= 3.271

Principal sources (faults, subduction, random seismicity having > 3% contribution)

Source Category:	% contr.	R(km)	M	epsilon0 (mean values).
California B-faults Char	16.00	8.3	7.11	0.72
California B-faults GR	10.44	6.0	6.82	0.62
CA Compr. crustal gridded	5.40	8.9	6.11	1.49

Individual fault hazard details if its contribution to mean hazard > 2%:

Fault ID	% contr.	Rcd(km)	M	epsilon0	Site-to-src azimuth(d)
Newport-Inglewood (Offshore) Cha	2.46	9.1	6.89	1.27	176.2
Newport-Inglewood, alt 1 Char	1.17	5.1	7.15	0.68	-128.5
Newport-Inglewood, alt 2 Char	1.12	4.9	7.15	0.72	-129.1
San Joaquin Hills Char	6.43	2.8	6.97	0.11	4.3
Newport Inglewood Connected alt	1.08	5.1	7.50	0.61	-128.5
Newport Inglewood Connected alt	1.08	4.9	7.50	0.61	-129.1
San Joaquin Hills GR	6.60	3.0	6.74	0.23	18.0

#*********End of deaggregation corresponding to Boore-Atkinson 2008 *********#

PSHA Deaggregation. %contributions. site: Mesa_Verde_ long: 117.921 W., lat: 33.672 N.
Vs30(m/s)= 300.0 (some WUS atten. models use Site Class not Vs30).
NSHMP 2007-08 See USGS OFR 2008-1128. dM=0.2 below
Return period: 2475 yrs. Exceedance PGA =0.6654 g. Weight * Computed_Rate_Ex
0.841E-04
#Pr[at least one eq with median motion>=PGA in 50 yrs]=0.00245
#This deaggregation corresponds to Campbell-Bozorgnia 2008

DIST(KM)	MAG(MW)	ALL_EPS	EPSILON>2	1<EPS<2	0<EPS<1	-1<EPS<0	-2<EPS<-1	EPS<-2
6.9	5.05	0.149	0.149	0.000	0.000	0.000	0.000	0.000
7.1	5.20	0.393	0.375	0.018	0.000	0.000	0.000	0.000
7.3	5.40	0.522	0.436	0.086	0.000	0.000	0.000	0.000
12.5	5.41	0.036	0.036	0.000	0.000	0.000	0.000	0.000
7.4	5.60	0.543	0.428	0.115	0.000	0.000	0.000	0.000
12.9	5.60	0.065	0.065	0.000	0.000	0.000	0.000	0.000
7.4	5.80	0.467	0.348	0.119	0.000	0.000	0.000	0.000
13.2	5.80	0.077	0.077	0.000	0.000	0.000	0.000	0.000

7.1	6.01	0.522	0.401	0.120	0.000	0.000	0.000	0.000
13.6	6.01	0.101	0.101	0.000	0.000	0.000	0.000	0.000
7.1	6.20	0.676	0.502	0.174	0.000	0.000	0.000	0.000
13.8	6.20	0.135	0.135	0.000	0.000	0.000	0.000	0.000
7.2	6.40	0.715	0.454	0.262	0.000	0.000	0.000	0.000
13.9	6.39	0.168	0.168	0.000	0.000	0.000	0.000	0.000
22.6	6.43	0.022	0.022	0.000	0.000	0.000	0.000	0.000
3.3	6.61	4.015	0.348	1.383	2.073	0.210	0.000	0.000
13.3	6.60	0.058	0.056	0.002	0.000	0.000	0.000	0.000
22.4	6.59	0.034	0.034	0.000	0.000	0.000	0.000	0.000
3.0	6.81	5.258	0.356	1.473	2.926	0.503	0.000	0.000
13.2	6.80	0.058	0.055	0.003	0.000	0.000	0.000	0.000
22.7	6.77	0.028	0.028	0.000	0.000	0.000	0.000	0.000
3.0	6.96	4.435	0.344	1.339	2.519	0.233	0.000	0.000
13.3	6.95	0.026	0.024	0.002	0.000	0.000	0.000	0.000
24.3	6.97	0.026	0.025	0.001	0.000	0.000	0.000	0.000
3.0	7.13	2.089	0.155	0.602	1.090	0.242	0.000	0.000
27.7	7.14	0.017	0.016	0.001	0.000	0.000	0.000	0.000
4.0	7.37	0.381	0.109	0.159	0.093	0.021	0.000	0.000
5.0	7.61	0.046	0.020	0.025	0.000	0.000	0.000	0.000
5.0	7.71	0.016	0.007	0.009	0.000	0.000	0.000	0.000

Summary statistics for above PSHA PGA deaggregation, R=distance, e=epsilon:
Contribution from this GMPE(%): 21.1
Mean src-site R= 4.4 km; M= 6.64; eps0= 0.42. Mean calculated for all sources.
Modal src-site R= 3.0 km; M= 6.81; eps0= -0.04 from peak (R,M) bin
MODE R*= 2.8km; M*= 6.81; EPS.INTERVAL: 0 to 1 sigma % CONTRIB.= 2.926

Principal sources (faults, subduction, random seismicity having > 3% contribution)

Source Category:	% contr.	R(km)	M	epsilon0 (mean values).
California B-faults Char	8.02	3.3	6.99	0.06
California B-faults GR	7.84	3.0	6.74	0.05
CA Compr. crustal gridded	5.24	8.0	5.97	1.52

Individual fault hazard details if its contribution to mean hazard > 2%:

Fault ID	% contr.	Rcd(km)	M	epsilon0	Site-to-src azimuth(d)
Newport-Inglewood (Offshore) Cha	0.09	9.1	6.93	2.23	176.2
Newport-Inglewood, alt 1 Char	0.12	5.1	7.15	1.63	-128.5
Newport-Inglewood, alt 2 Char	0.12	4.9	7.15	1.62	-129.1
San Joaquin Hills Char	7.36	2.8	6.97	-0.10	4.3
Newport Inglewood Connected alt	0.11	5.1	7.50	1.59	-128.5
Newport Inglewood Connected alt	0.11	4.9	7.50	1.58	-129.1
San Joaquin Hills GR	7.76	3.0	6.74	0.02	18.0

#*********End of deaggregation corresponding to Campbell-Bozorgnia 2008 *********#

PSHA Deaggregation. %contributions. site: Mesa_Verde_ long: 117.921 W., lat: 33.672 N.
Vs30(m/s)= 300.0 (some WUS atten. models use Site Class not Vs30).
NSHMP 2007-08 See USGS OFR 2008-1128. dM=0.2 below
Return period: 2475 yrs. Exceedance PGA =0.6654 g. Weight * Computed_Rate_Ex
0.176E-03
#Pr[at least one eq with median motion>=PGA in 50 yrs]=0.00816
#This deaggregation corresponds to Chiou-Youngs 2008

DIST(KM)	MAG(MW)	ALL_EPS	EPSILON>2	1<EPS<2	0<EPS<1	-1<EPS<0	-2<EPS<-1	EPS<-2
7.3	5.05	0.825	0.692	0.133	0.000	0.000	0.000	0.000
12.6	5.05	0.087	0.087	0.000	0.000	0.000	0.000	0.000
7.3	5.20	1.630	1.304	0.326	0.000	0.000	0.000	0.000
12.9	5.20	0.232	0.232	0.000	0.000	0.000	0.000	0.000
7.4	5.40	1.530	1.044	0.486	0.000	0.000	0.000	0.000
13.3	5.40	0.296	0.296	0.000	0.000	0.000	0.000	0.000
7.5	5.60	1.387	0.821	0.566	0.000	0.000	0.000	0.000
13.6	5.60	0.342	0.342	0.000	0.000	0.000	0.000	0.000
7.5	5.80	1.219	0.651	0.569	0.000	0.000	0.000	0.000
13.8	5.80	0.370	0.370	0.000	0.000	0.000	0.000	0.000

7.2	6.01	1.460	0.682	0.778	0.000	0.000	0.000	0.000
14.1	6.01	0.441	0.432	0.009	0.000	0.000	0.000	0.000
7.2	6.20	1.801	0.748	1.052	0.000	0.000	0.000	0.000
14.1	6.20	0.510	0.477	0.032	0.000	0.000	0.000	0.000
23.1	6.21	0.049	0.049	0.000	0.000	0.000	0.000	0.000
7.4	6.40	1.681	0.579	1.099	0.003	0.000	0.000	0.000
14.0	6.39	0.556	0.495	0.061	0.000	0.000	0.000	0.000
22.9	6.41	0.086	0.086	0.000	0.000	0.000	0.000	0.000
4.0	6.61	5.474	0.549	2.111	2.588	0.226	0.000	0.000
12.8	6.60	0.235	0.169	0.067	0.000	0.000	0.000	0.000
22.6	6.60	0.076	0.076	0.000	0.000	0.000	0.000	0.000
4.1	6.80	6.526	0.690	2.486	2.967	0.382	0.000	0.000
12.9	6.79	0.234	0.160	0.074	0.000	0.000	0.000	0.000
23.9	6.79	0.076	0.076	0.000	0.000	0.000	0.000	0.000
4.0	6.98	8.197	0.686	3.074	3.793	0.645	0.000	0.000
12.9	6.98	0.139	0.085	0.053	0.001	0.000	0.000	0.000
25.8	7.03	0.182	0.160	0.022	0.000	0.000	0.000	0.000
4.1	7.15	3.871	0.288	1.414	1.771	0.397	0.001	0.000
24.5	7.20	0.179	0.151	0.028	0.000	0.000	0.000	0.000
4.8	7.39	2.132	0.166	0.869	0.995	0.102	0.001	0.000
23.4	7.35	0.185	0.130	0.055	0.000	0.000	0.000	0.000
5.0	7.55	1.255	0.131	0.657	0.445	0.022	0.000	0.000
23.9	7.57	0.130	0.088	0.042	0.000	0.000	0.000	0.000
5.0	7.71	0.184	0.014	0.078	0.083	0.009	0.000	0.000
23.4	7.74	0.299	0.173	0.126	0.000	0.000	0.000	0.000
35.2	7.76	0.058	0.058	0.000	0.000	0.000	0.000	0.000
23.9	7.92	0.045	0.025	0.020	0.000	0.000	0.000	0.000

Summary statistics for above PSHA PGA deaggregation, R=distance, e=epsilon:
Contribution from this GMPE(%): 44.1
Mean src-site R= 6.5 km; M= 6.58; eps0= 0.64. Mean calculated for all sources.
Modal src-site R= 4.0 km; M= 6.98; eps0= 0.16 from peak (R,M) bin
MODE R*= 3.3km; M*= 6.97; EPS.INTERVAL: 0 to 1 sigma % CONTRIB.= 3.793

Principal sources (faults, subduction, random seismicity having > 3% contribution)

Source Category:	% contr.	R(km)	M	epsilon0 (mean values).
California B-faults Char	16.38	5.5	7.09	0.35
California B-faults GR	11.88	4.2	6.80	0.23
CA Compr. crustal gridded	15.63	8.7	5.86	1.24

Individual fault hazard details if its contribution to mean hazard > 2%:

Fault ID	% contr.	Rcd(km)	M	epsilon0	Site-to-src azimuth(d)
Newport-Inglewood (Offshore) Cha	2.25	9.1	6.89	1.28	176.2
Newport-Inglewood, alt 1 Char	1.26	5.1	7.15	0.57	-128.5
Newport-Inglewood, alt 2 Char	1.30	4.9	7.15	0.56	-129.1
San Joaquin Hills Char	8.10	2.8	6.97	-0.19	4.3
Newport Inglewood Connected alt	1.26	5.1	7.51	0.44	-128.5
Newport Inglewood Connected alt	1.29	4.9	7.51	0.42	-129.1
San Joaquin Hills GR	8.54	3.0	6.73	-0.04	18.0

#*********End of deaggregation corresponding to Chiou-Youngs 2008 *********#

********** Southern California **********

USGS Design Maps Detailed Report

ASCE 7-10 Standard (33.673°N, 117.9215°W)

Site Class D – "Stiff Soil", Risk Category I/II/III

Section 11.4.1 — Mapped Acceleration Parameters

Note: Ground motion values provided below are for the direction of maximum horizontal spectral response acceleration. They have been converted from corresponding geometric mean ground motions computed by the USGS by applying factors of 1.1 (to obtain S_S) and 1.3 (to obtain S_1). Maps in the 2010 ASCE-7 Standard are provided for Site Class B. Adjustments for other Site Classes are made, as needed, in Section 11.4.3.

From Figure 22-1 [1] $S_S = 1.604$ g

From Figure 22-2 [2] $S_1 = 0.592$ g

Section 11.4.2 — Site Class

The authority having jurisdiction (not the USGS), site-specific geotechnical data, and/or the default has classified the site as Site Class D, based on the site soil properties in accordance with Chapter 20.

Table 20.3–1 Site Classification

Site Class	$\bar{v}_s$	$\bar{N}$ or $\bar{N}_{ch}$	$\bar{s}_u$
A. Hard Rock	>5,000 ft/s	N/A	N/A
B. Rock	2,500 to 5,000 ft/s	N/A	N/A
C. Very dense soil and soft rock	1,200 to 2,500 ft/s	>50	>2,000 psf
D. Stiff Soil	600 to 1,200 ft/s	15 to 50	1,000 to 2,000 psf
E. Soft clay soil	<600 ft/s	<15	<1,000 psf
	Any profile with more than 10 ft of soil having the characteristics: • Plasticity index $PI > 20$, • Moisture content $w \geq 40\%$, and • Undrained shear strength $\bar{s}_u < 500$ psf		
F. Soils requiring site response analysis in accordance with Section 21.1	See Section 20.3.1		

For SI: 1ft/s = 0.3048 m/s 1lb/ft² = 0.0479 kN/m²

Section 11.4.3 — Site Coefficients and Risk-Targeted Maximum Considered Earthquake (MCE_R) Spectral Response Acceleration Parameters

Table 11.4-1: Site Coefficient F_a

Site Class	Mapped MCE_R Spectral Response Acceleration Parameter at Short Period				
	$S_S \leq 0.25$	$S_S = 0.50$	$S_S = 0.75$	$S_S = 1.00$	$S_S \geq 1.25$
A	0.8	0.8	0.8	0.8	0.8
B	1.0	1.0	1.0	1.0	1.0
C	1.2	1.2	1.1	1.0	1.0
D	1.6	1.4	1.2	1.1	1.0
E	2.5	1.7	1.2	0.9	0.9
F	See Section 11.4.7 of ASCE 7				

Note: Use straight-line interpolation for intermediate values of S_S

For Site Class = D and S_s = 1.604 g, F_a = 1.000

Table 11.4-2: Site Coefficient F_v

Site Class	Mapped MCE_R Spectral Response Acceleration Parameter at 1-s Period				
	$S_1 \leq 0.10$	$S_1 = 0.20$	$S_1 = 0.30$	$S_1 = 0.40$	$S_1 \geq 0.50$
A	0.8	0.8	0.8	0.8	0.8
B	1.0	1.0	1.0	1.0	1.0
C	1.7	1.6	1.5	1.4	1.3
D	2.4	2.0	1.8	1.6	1.5
E	3.5	3.2	2.8	2.4	2.4
F	See Section 11.4.7 of ASCE 7				

Note: Use straight-line interpolation for intermediate values of S_1

For Site Class = D and S_1 = 0.592 g, F_v = 1.500

Equation (11.4-1): $S_{MS} = F_a S_S = 1.000 \times 1.604 = 1.604$ g

Equation (11.4-2): $S_{M1} = F_v S_1 = 1.500 \times 0.592 = 0.889$ g

Section 11.4.4 — Design Spectral Acceleration Parameters

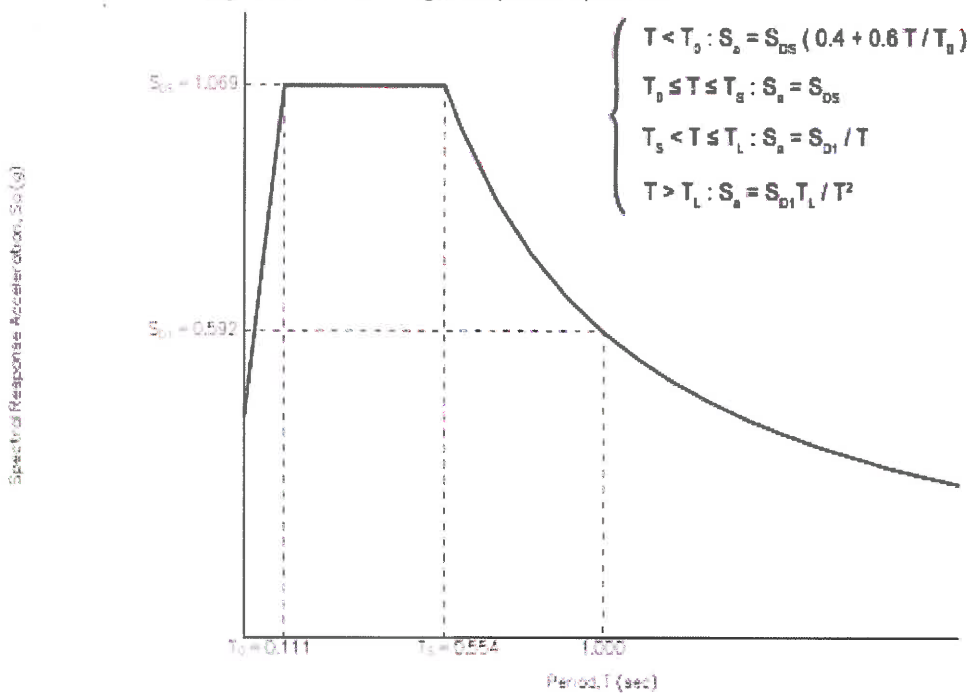
Equation (11.4-3): $S_{DS} = ⅔\ S_{MS} = ⅔ \times 1.604 = 1.069$ g

Equation (11.4-4): $S_{D1} = ⅔\ S_{M1} = ⅔ \times 0.889 = 0.592$ g

Section 11.4.5 — Design Response Spectrum

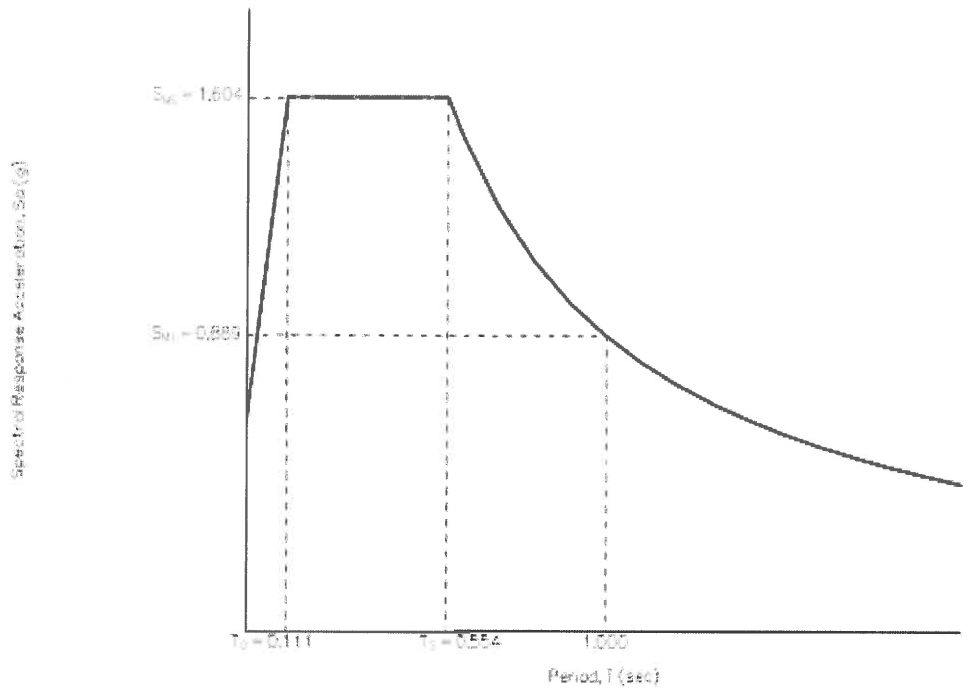
From Figure 22-12 [3] $T_L = 8$ seconds

Figure 11.4-1: Design Response Spectrum



Section 11.4.6 — Risk-Targeted Maximum Considered Earthquake (MCE_R) Response Spectrum

The MCE_R Response Spectrum is determined by multiplying the design response spectrum above by 1.5.



Section 11.8.3 — Additional Geotechnical Investigation Report Requirements for Seismic Design Categories D through F

From Figure 22-7[4] PGA = 0.633

Equation (11.8-1): $PGA_M = F_{PGA}PGA = 1.000 \times 0.633 = 0.633$ g

Table 11.8-1: Site Coefficient F_{PGA}

Site Class	Mapped MCE Geometric Mean Peak Ground Acceleration, PGA				
	PGA ≤ 0.10	PGA = 0.20	PGA = 0.30	PGA = 0.40	PGA ≥ 0.50
A	0.8	0.8	0.8	0.8	0.8
B	1.0	1.0	1.0	1.0	1.0
C	1.2	1.2	1.1	1.0	1.0
D	1.6	1.4	1.2	1.1	1.0
E	2.5	1.7	1.2	0.9	0.9
F	See Section 11.4.7 of ASCE 7				

Note: Use straight-line interpolation for intermediate values of PGA

For Site Class = D and PGA = 0.633 g, F_{PGA} = 1.000

Section 21.2.1.1 — Method 1 (from Chapter 21 - Site-Specific Ground Motion Procedures for Seismic Design)

From Figure 22-17[5] C_{RS} = 0.943

From Figure 22-18[6] C_{R1} = 0.970

Section 11.6 — Seismic Design Category

Table 11.6-1 Seismic Design Category Based on Short Period Response Acceleration Parameter

VALUE OF S_{DS}	RISK CATEGORY		
	I or II	III	IV
$S_{DS} < 0.167g$	A	A	A
$0.167g \leq S_{DS} < 0.33g$	B	B	C
$0.33g \leq S_{DS} < 0.50g$	C	C	D
$0.50g \leq S_{DS}$	D	D	D

For Risk Category = I and S_{DS} = 1.069 g, Seismic Design Category = D

Table 11.6-2 Seismic Design Category Based on 1-S Period Response Acceleration Parameter

VALUE OF S_{D1}	RISK CATEGORY		
	I or II	III	IV
$S_{D1} < 0.067g$	A	A	A
$0.067g \leq S_{D1} < 0.133g$	B	B	C
$0.133g \leq S_{D1} < 0.20g$	C	C	D
$0.20g \leq S_{D1}$	D	D	D

For Risk Category = I and S_{D1} = 0.592 g, Seismic Design Category = D

Note: When S_1 is greater than or equal to 0.75g, the Seismic Design Category is **E** for buildings in Risk Categories I, II, and III, and **F** for those in Risk Category IV, irrespective of the above.

Seismic Design Category ≡ "the more severe design category in accordance with Table 11.6-1 or 11.6-2" = D

Note: See Section 11.6 for alternative approaches to calculating Seismic Design Category.

References

1. *Figure 22-1*: https://earthquake.usgs.gov/hazards/designmaps/downloads/pdfs/2010_ASCE-7_Figure_22-1.pdf
2. *Figure 22-2*: https://earthquake.usgs.gov/hazards/designmaps/downloads/pdfs/2010_ASCE-7_Figure_22-2.pdf
3. *Figure 22-12*: https://earthquake.usgs.gov/hazards/designmaps/downloads/pdfs/2010_ASCE-7_Figure_22-12.pdf
4. *Figure 22-7*: https://earthquake.usgs.gov/hazards/designmaps/downloads/pdfs/2010_ASCE-7_Figure_22-7.pdf
5. *Figure 22-17*: https://earthquake.usgs.gov/hazards/designmaps/downloads/pdfs/2010_ASCE-7_Figure_22-17.pdf
6. *Figure 22-18*: https://earthquake.usgs.gov/hazards/designmaps/downloads/pdfs/2010_ASCE-7_Figure_22-18.pdf

APPENDIX F

APPENDIX F

GENERAL EARTHWORK AND GRADING SPECIFICATIONS

1.0 General

1.1 Intent: These General Earthwork and Grading Specifications are for the grading and earthwork shown on the approved grading plan(s) and/or indicated in the geotechnical report(s). These Specifications are a part of the recommendations contained in the geotechnical report(s). In case of conflict, the specific recommendations in the geotechnical report shall supersede these more general Specifications. Observations of the earthwork by the project Geotechnical Consultant during the course of grading may result in new or revised recommendations that could supersede these specifications or the recommendations in the geotechnical report(s).

1.2 Geotechnical Consultant: Prior to commencement of work, the owner shall employ a geotechnical consultant. The geotechnical consultant shall be responsible for reviewing the approved geotechnical report(s) and accepting the adequacy of the preliminary geotechnical findings, conclusions, and recommendations prior to the commencement of the grading.

Prior to commencement of grading, the Geotechnical Consultant shall review the "work plan" prepared by the Earthwork Contractor (Contractor) and schedule sufficient personnel to perform the appropriate level of observation, mapping, and compaction testing.

During the grading and earthwork operations, the Geotechnical Consultant shall observe, map, and document the subsurface exposures to verify the geotechnical design assumptions. If the observed conditions are found to be significantly different than the interpreted assumptions during the design phase, the Geotechnical Consultant shall inform the owner, recommend appropriate changes in design to accommodate the observed conditions, and notify the review agency where required. Subsurface areas to be geotechnically observed, mapped, elevations recorded, and/or tested include natural ground after it has been cleared for receiving fill but before fill is placed, bottoms of all "remedial removal" areas, all key bottoms, and benches made on sloping ground to receive fill.

The Geotechnical Consultant shall observe the moisture-conditioning and processing of the subgrade and fill materials and perform relative compaction testing of fill to determine the attained level of compaction. The Geotechnical Consultant shall provide the test results to the owner and the Contractor on a routine and frequent basis.

1.3 The Earthwork Contractor: The Earthwork Contractor (Contractor) shall be qualified, experienced, and knowledgeable in earthwork logistics, preparation and processing of ground to receive fill, moisture-conditioning and processing of fill, and compacting fill. The Contractor shall review and accept the plans, geotechnical report(s), and these Specifications prior to commencement of grading. The Contractor shall be solely responsible for performing the grading in accordance with the plans and specifications.

The Contractor shall prepare and submit to the owner and the Geotechnical Consultant a work plan that indicates the sequence of earthwork grading, the number of "spreads" of work and the estimated quantities of daily earthwork contemplated for the site prior to commencement of grading. The Contractor shall inform the owner and the Geotechnical Consultant of changes in work schedules and updates to the work plan at least 24 hours in advance of such changes so that appropriate observations and tests can be planned and accomplished. The Contractor shall not assume that the Geotechnical Consultant is aware of all grading operations.

The Contractor shall have the sole responsibility to provide adequate equipment and methods to accomplish the earthwork in accordance with the applicable grading codes and agency ordinances, these Specifications, and the recommendations in the approved geotechnical report(s) and grading plan(s). If, in the opinion of the Geotechnical Consultant, unsatisfactory conditions, such as unsuitable soil, improper moisture condition, inadequate compaction, insufficient buttress key size, adverse weather, etc., are resulting in a quality of work less than required in these specifications, the Geotechnical Consultant shall reject the work and may recommend to the owner that construction be stopped until the conditions are rectified.

2.0 Preparation of Areas to be Filled

2.1 Clearing and Grubbing: Vegetation, such as brush, grass, roots, and other deleterious material shall be sufficiently removed and properly disposed of in a method acceptable to the owner, governing agencies, and the Geotechnical Consultant.

The Geotechnical Consultant shall evaluate the extent of these removals depending on specific site conditions. Earth fill material shall not contain more than 1 percent of organic materials (by volume). No fill lift shall contain more than 5 percent of organic matter. Nesting of the organic materials shall not be allowed.

If potentially hazardous materials are encountered, the Contractor shall stop work in the affected area, and a hazardous material specialist shall be informed

immediately for proper evaluation and handling of these materials prior to continuing to work in that area.

As presently defined by the State of California, most refined petroleum products (gasoline, diesel fuel, motor oil, grease, coolant, etc.) have chemical constituents that are considered to be hazardous waste. As such, the indiscriminate dumping or spillage of these fluids onto the ground may constitute a misdemeanor, punishable by fines and/or imprisonment, and shall not be allowed.

2.2 Processing: Existing ground that has been declared satisfactory for support of fill by the Geotechnical Consultant shall be scarified to a minimum depth of 6 inches. Existing ground that is not satisfactory shall be overexcavated as specified in the following section. Scarification shall continue until soils are broken down and free of large clay lumps or clods and the working surface is reasonably uniform, flat, and free of uneven features that would inhibit uniform compaction.

2.3 Overexcavation: In addition to removals and overexcavations recommended in the approved geotechnical report(s) and the grading plan, soft, loose, dry, saturated, spongy, organic-rich, highly fractured or otherwise unsuitable ground shall be overexcavated to competent ground as evaluated by the Geotechnical Consultant during grading.

2.4 Benching: Where fills are to be placed on ground with slopes steeper than 5:1 (horizontal to vertical units), the ground shall be stepped or benched. Please see the Standard Details for a graphic illustration. The lowest bench or key shall be a minimum of 15 feet wide and at least 2 feet deep, into competent material as evaluated by the Geotechnical Consultant. Other benches shall be excavated a minimum height of 4 feet into competent material or as otherwise recommended by the Geotechnical Consultant. Fill placed on ground sloping flatter than 5:1 shall also be benched or otherwise overexcavated to provide a flat subgrade for the fill.

2.5 Evaluation/Acceptance of Fill Areas: All areas to receive fill, including removal and processed areas, key bottoms, and benches, shall be observed, mapped, elevations recorded, and/or tested prior to being accepted by the Geotechnical Consultant as suitable to receive fill. The Contractor shall obtain a written acceptance from the Geotechnical Consultant prior to fill placement. A licensed surveyor shall provide the survey control for determining elevations of processed areas, keys, and benches.

3.0 Fill Material

3.1 General: Material to be used as fill shall be essentially free of organic matter and other deleterious substances evaluated and accepted by the Geotechnical Consultant prior to placement. Soils of poor quality, such as those with unacceptable gradation, high expansion potential, or low strength shall be placed in areas acceptable to the Geotechnical Consultant or mixed with other soils to achieve satisfactory fill material.

3.2 Oversize: Oversize material defined as rock, or other irreducible material with a maximum dimension greater than 12 inches, shall not be buried or placed in fill unless location, materials, and placement methods are specifically accepted by the Geotechnical Consultant. Placement operations shall be such that nesting of oversized material does not occur and such that oversize material is completely surrounded by compacted or densified fill. Oversize material shall not be placed within 10 vertical feet of finish grade or within 2 feet of future utilities or underground construction.

3.3 Import: If importing of fill material is required for grading, proposed import material shall meet the requirements of Section 3.1. The potential import source shall be given to the Geotechnical Consultant at least 48 hours (2 working days) before importing begins so that its suitability can be determined and appropriate tests performed.

4.0 Fill Placement and Compaction

4.1 Fill Layers: Approved fill material shall be placed in areas prepared to receive fill (per Section 3.0) in near-horizontal layers not exceeding 8 inches in loose thickness. The Geotechnical Consultant may accept thicker layers if testing indicates the grading procedures can adequately compact the thicker layers. Each layer shall be spread evenly and mixed thoroughly to attain relative uniformity of material and moisture throughout.

4.2 Fill Moisture Conditioning: Fill soils shall be watered, dried back, blended, and/or mixed, as necessary to attain a relatively uniform moisture content at or slightly over optimum. Maximum density and optimum soil moisture content tests shall be performed in accordance with the American Society of Testing and Materials (ASTM Test Method D1557-91).

4.3 Compaction of Fill: After each layer has been moisture-conditioned, mixed, and evenly spread, it shall be uniformly compacted to not less than 90 percent of maximum dry density (ASTM Test Method D1557-91). Compaction equipment shall be adequately sized and be either specifically designed for soil compaction or of proven reliability to efficiently achieve the specified level of compaction with uniformity.

4.4 Compaction of Fill Slopes: In addition to normal compaction procedures specified above, compaction of slopes shall be accomplished by backrolling of slopes with sheepsfoot rollers at increments of 3 to 4 feet in fill elevation, or by other methods producing satisfactory results acceptable to the Geotechnical Consultant. Upon completion of grading, relative compaction of the fill, out to the slope face, shall be at least 90 percent of maximum density per ASTM Test Method D1557-91.

4.5 Compaction Testing: Field tests for moisture content and relative compaction of the fill soils shall be performed by the Geotechnical Consultant. Location and frequency of tests shall be at the Consultant's discretion based on field conditions encountered. Compaction test locations will not necessarily be selected on a random basis. Test locations shall be selected to verify adequacy of compaction levels in areas that are judged to be prone to inadequate compaction (such as close to slope faces and at the fill/bedrock benches).

4.6 Frequency of Compaction Testing: Tests shall be taken at intervals not exceeding 2 feet in vertical rise and/or 1,000 cubic yards of compacted fill soils embankment. In addition, as a guideline, at least one test shall be taken on slope faces for each 5,000 square feet of slope face and/or each 10 feet of vertical height of slope. The Contractor shall assure that fill construction is such that the testing schedule can be accomplished by the Geotechnical Consultant. The Contractor shall stop or slow down the earthwork construction if these minimum standards are not met.

4.7 Compaction Test Locations: The Geotechnical Consultant shall document the approximate elevation and horizontal coordinates of each test location. The Contractor shall coordinate with the project surveyor to assure that sufficient grade stakes are established so that the Geotechnical Consultant can determine the test locations with sufficient accuracy. At a minimum, two grade stakes within a horizontal distance of 100 feet and vertically less than 5 feet apart from potential test locations shall be provided.

5.0 Subdrain Installation

Subdrain systems shall be installed in accordance with the approved geotechnical report(s), the grading plan, and the Standard Details. The Geotechnical Consultant may recommend additional subdrains and/or changes in subdrain extent, location, grade, or material depending on conditions encountered during grading. All subdrains shall be surveyed by a land surveyor/civil engineer for line and grade after installation and prior to burial. Sufficient time should be allowed by the Contractor for these surveys.

6.0 Excavation

Excavations, as well as over-excavation for remedial purposes, shall be evaluated by the Geotechnical Consultant during grading. Remedial removal depths shown on geotechnical plans are estimates only. The actual extent of removal shall be determined by the Geotechnical Consultant based on the field evaluation of exposed conditions during grading. Where fill-over-cut slopes are to be graded, the cut portion of the slope shall be made, evaluated, and accepted by the Geotechnical Consultant prior to placement of materials for construction of the fill portion of the slope, unless otherwise recommended by the Geotechnical Consultant.

7.0 Trench Backfills

7.1 Contractor shall follow all OHSA and Cal/OSHA requirements for safety of trench excavations.

7.2 Bedding and backfill of utility trenches shall be done in accordance with the applicable provisions of Standard Specifications of Public Works Construction. Bedding material shall have a Sand Equivalent greater than 30 (SE>30). The bedding shall be placed to 1 foot over the top of the conduit and densified by jetting. Backfill shall be placed and densified to a minimum 90 percent of maximum from 1 foot above the top of the conduit to the surface, except in traveled ways (see Section 7.6 below).

7.3 Jetting of the bedding around the conduits shall be observed by the Geotechnical Consultant.

7.4 Geotechnical Consultant shall test the trench backfill for relative compaction. At least one test should be made for every 300 feet of trench and 2 feet of fill.

7.5 Lift thickness of trench backfill shall not exceed those allowed in the Standard Specifications of Public Works Construction unless the Contractor can demonstrate to the Geotechnical Consultant that the fill lift can be compacted to the minimum relative compaction by his alternative equipment and method.

7.6 Trench backfill in the upper foot measured from finish grade within existing or future traveled way, shoulder, and other paved areas (or areas to receive pavement) should be placed to a minimum 95 percent relative compaction.