LGC Geotechnical, Inc.

April 28, 2014

Project No. 13192-01

Mr. Eric Nelson
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450 Newport Center Drive, Suite 300
Newport Beach, CA 92660

Subject: ***Preliminary Geotechnical Evaluation for the Proposed Residential Development at 1239 Victoria Street, City of Costa Mesa, California***

In accordance with your request and authorization, LGC Geotechnical, Inc. has performed a preliminary geotechnical evaluation for the proposed approximately 1.75 acre residential development located at 1239 Victoria Street within the City of Costa Mesa, California. The purpose of our study was to evaluate the existing onsite geotechnical conditions and to provide preliminary geotechnical recommendations relative to the proposed residential development.

Should you have any questions regarding this report, please do not hesitate to contact our office. We appreciate this opportunity to be of service.

Respectfully Submitted,

LGC Geotechnical, Inc.

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1.0 INTRODUCTION

1.1 Purpose and Scope of Services

This report presents the results of our preliminary geotechnical evaluation for the proposed approximately 1.75-acre residential development located at 1239 Victoria Street in the City of Costa Mesa, California. Refer to the Site Location Map (Figure 1).

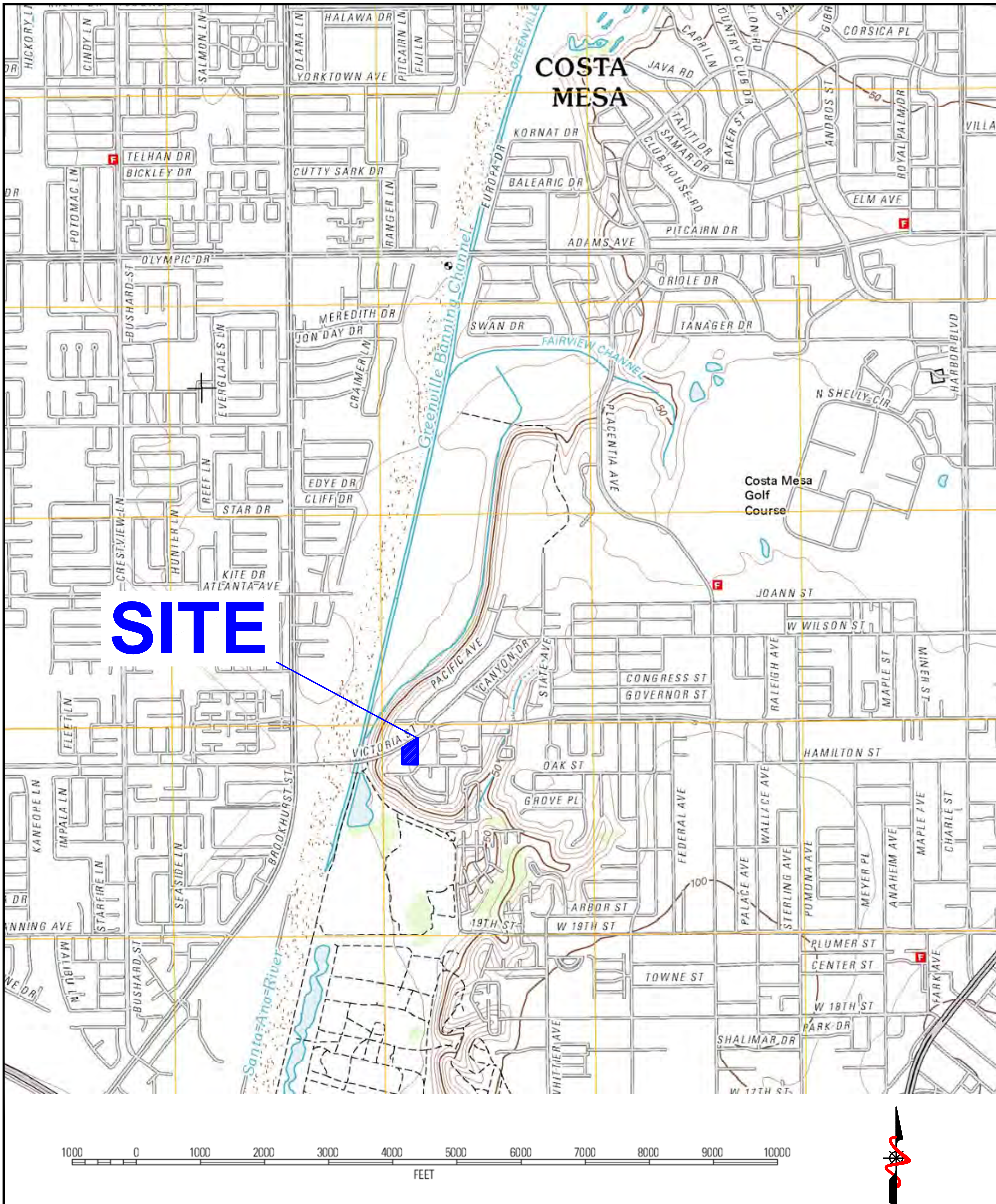
The purpose of our study was to provide a preliminary geotechnical evaluation relative to the proposed residential development. As part of our scope of work, we have: 1) reviewed available geotechnical reports and in-house geologic maps pertinent to the site (Appendix A); 2) performed a subsurface geotechnical evaluation of the site consisting of the excavation of six small diameter borings from approximately 10 to 50 feet below existing ground surface; 3) performed one field infiltration test; 4) performed laboratory testing of select soil samples obtained during our subsurface evaluation; and 5) prepared this preliminary geotechnical summary report presenting our findings and preliminary conclusions and recommendations for the development of the proposed project.

1.2 Project Description

The project consists of an approximately 1.75-acre sized parcel located at 1239 Victoria Street in the City of Costa Mesa (Figure 1). It is bound to the north by Victoria Street, to the west by a new 17-unit residential project that is under construction and to the south and east by existing residential/commercial buildings. The site is currently a two-story office building and associated parking area. The site has approximately 10 feet of relief sloping downward toward Victoria Street.

We understand the proposed improvements include construction of a residential development and associated improvements. The proposed development will be at-grade with relatively light building loads (column and wall loads maximum of 20 kips and 2 kips per lineal foot, respectively). A preliminary grading plan depicting existing and planned elevations was not available at the time of this report.

The recommendations given in this report are based upon at-grade structures with the estimated structural loads indicated above. LGC Geotechnical should be provided with any updated project information, plans and/or any changes to estimated structural loads when they become available, in order to either confirm or modify the recommendations provided herein.



LGC Geotechnical, Inc.	**FIGURE 1** **Site Location Map**	PROJECT NAME	Victoria Street, Costa Mesa
		PROJECT NO.	13192-01
		ENG. / GEOL.	DJB/KTM
		SCALE	1" = 2,000'
		DATE	April 2014

1.3 Subsurface Geotechnical Evaluation

A geotechnical evaluation of the site was performed by LGC Geotechnical. The exploration program consisted of drilling and sampling five small-diameter exploratory hollow-stem borings (HS-1 through HS-6) in February of 2014. The borings were drilled by Cal Pac Drilling, Inc., under subcontract to LGC Geotechnical. The depth of the borings ranged from approximately 10 to 50 feet below existing grade. An LGC Geotechnical representative observed the drilling operations, logged the borings, and collected soil samples for laboratory testing. The borings were performed using a B-53 truck-mounted drill rig equipped with 6-inch diameter hollow-stem augers. Bulk samples of the near surface soils were logged and collected for laboratory testing from select borings. Driven soil samples were collected by means of the Standard Penetration Test (SPT) and Modified California Drive (MCD) samplers, generally at 5-foot vertical increments. The MCD is a split-barrel sampler with a tapered cutting tip and lined with a series of 1-inch tall brass rings. The SPT sampler (1.4-inch ID) and MCD sampler (2.4-inch ID, 3.0-inch OD) were driven using a 140-pound automatic hammer falling 30 inches to advance the sampler a total depth of 18 inches or until refusal. The raw blow counts for each 6-inch increment of penetration were recorded on the boring logs. At the completion of drilling, the borings were backfilled with native soils and tamped. The approximate locations of our explorations are provided on the Boring Location Map, Figure 2. The boring logs are provided in Appendix B. It should be noted that elevations provided on the boring logs are sourced from Google Earth and should be considered approximate.

1.4 Infiltration Testing

A field infiltration test was performed in boring HS-6 (Figure 2). Estimation of infiltration rates was accomplished in general accordance with the guidelines set forth by the County of Orange (2011). A 3-inch diameter perforated PVC pipe was placed in the borehole, and the annulus was backfilled with gravel including placement of approximately 2 inches of gravel at the bottom of the borehole. Due to the water level dropping more than 6 inches in 25 minutes for two readings, the test procedure for coarse-grained soils was followed. The infiltration well was pre-soaked prior to testing. The procedure for coarse-grained soils requires performing the test for one hour and taking one reading every 10 minutes from a fixed reference point. Based on the County of Orange methodology, the calculated infiltration rate was 1.4 inches per hour for HS-6. This infiltration rate includes a factor of safety of 3 per County guidelines. This factor of safety is based only on geotechnical conditions. It should be emphasized that the result of the infiltration test is only representative of the location and depth where performed. Varying subsurface conditions may exist outside of the test location which could alter the calculated infiltration rate indicated above. Infiltration tests are performed using relatively clean water, free of particulates, silt, etc. Refer to the discussion provided in Section 4.6 and infiltration test data provided in Appendix B.

1.5 Laboratory Testing

Representative bulk and driven (relatively undisturbed) samples were retained for laboratory testing during our field evaluation. Laboratory testing included in-situ moisture content and in-situ dry density, grain size analysis, fines content, Atterberg Limits (liquid limit and plastic limit), expansion index, consolidation, direct shear, laboratory compaction and corrosion (sulfate, chloride, pH, and resistivity).

The following is a summary of the laboratory test results:

- Dry density of the samples collected ranged from approximately 91 pounds per cubic foot (pcf) to 120 pcf, with an average of 108 pcf. Field moisture contents ranged from approximately 2 to 27 percent, with an average of 9 percent.
- One gradation and four fines content tests were performed and indicated a fines content (passing No. 200 sieve) ranging from approximately 4 to 53 percent. Based on the Unified Soils Classification System (USCS), four of the five tested samples would be classified as "coarse-grained."
- Four Atterberg Limits (liquid limit and plastic limit) tests were performed. Results indicated Plasticity Index (PI) values ranging from approximately 6 to 38.
- Expansion potential testing indicated expansion index values of 42 and 51, corresponding to "Low" and "Medium" expansion potential, respectively.
- Three consolidation tests were performed on samples from various depths. The load versus deformation plots are provided in Appendix C.
- A direct shear test was performed on a driven sample from HS-5. The plot is provided in Appendix C.
- A laboratory compaction test resulted in a maximum dry density of 125.0 pcf at an optimum moisture content of 10.0 percent.
- Corrosion testing indicated soluble sulfate contents of approximately 0.01 percent or less, a chloride content of 40 parts per million (ppm), pH of 8.5 and a minimum resistivity of 2,335 ohm-cm.

A summary of the laboratory test results are presented in Appendix C. The moisture and dry density results are presented on the boring logs in Appendix B.

2.0 GEOTECHNICAL CONDITIONS

2.1 Regional Geology

The subject site is generally located within the Peninsular Ranges Geomorphic Province of California, more specifically at the southern boundary of the Los Angeles Sedimentary Basin. The Los Angeles Basin is a northwest-plunging synclinal sedimentary deposit that is bounded near the subject site by the broadly uplifted coastal mesa of Newport Beach. The local Santa Ana River System deeply incised the local sedimentary deposits. A channelized portion of the Santa Ana River passes close by the site, trending roughly south. The river deposited widely dispersed sheet deposits prior to construction of the upstream Prado Dam and channelization (Morton, 2004 & CGS, 2001).

2.2 Site-Specific Geology

Based on our review of available geologic maps (Morton, 2004), the primary geologic unit underlying the site is Quaternary Old Paralic deposits (Qop). These materials are defined as late to middle Pleistocene interfingered estuarine, beach, and colluvial deposits. The unit is known to consist of silt, sand and cobbles in general. At the subject site, there are likely thin layers of artificial fill associated with past uses of the area, not differentiated with this study. As encountered, these soils generally consist of clayey sand, sandy clay, and silty sand. Recommendations for remedial grading of the site are presented in the recommendations section of this report. From a geotechnical perspective, the existing soils are generally acceptable to be utilized as artificial fill materials provided they are moisture conditioned and properly compacted.

2.3 Generalized Subsurface Conditions

The field explorations (borings) indicate primarily medium dense clayey sands and stiff sandy clays transitioning at greater depths to dense to very dense sands with varying amounts of silt to the maximum explored depth of approximately 50 feet. In general, the upper approximate 5 feet of the site consisted of medium dense clayey sands and stiff sandy clays. A dense to very dense fine silty sand layer was encountered in HS-4 beginning at a depth of approximately 30 feet below current grade to the maximum explored depth.

It should be noted that borings are only representative of the location where they are performed and varying subsurface conditions may exist outside of the performed location. In addition, subsurface conditions can change over time. The soil descriptions provided above should not be construed to mean that the subsurface profile is uniform and that soil is homogeneous within the project area. The position of the site adjacent to a major riverbed leads to especially variable conditions such as those related to overbank deposits. For details on the stratigraphy at the exploration locations, refer to Appendix B.

2.4 *Groundwater*

Groundwater was not encountered to the maximum explored depth of 50 feet below existing ground surface during our subsurface evaluation. Historic high groundwater is estimated on the order of approximately 30 feet below existing ground surface (CGS, 2001).

Seasonal fluctuations of groundwater elevations should be expected over time. In general, groundwater levels fluctuate with the seasons and local zones of perched groundwater may be present due to local seepage caused by irrigation and/or recent precipitation. Local perched groundwater conditions or surface seepage may develop once site development is completed.

2.5 *California Seismic Hazard Zones*

A small portion of the site in the northeast corner along Victoria Street with relatively minor topographic relief is located within a within a State of California seismic hazard zone for seismically-induced landslide. Due to the limited area of minor relief and the overall relatively flat nature of site, it is our opinion that the potential for seismically-induced landslides is very low and is not a design constraint. The site is not located in a State of California liquefaction hazard zone (CGS, 2001).

2.6 *Seismic Design Criteria*

The site seismic characteristics were evaluated per the guidelines set forth in Chapter 16, Section 1613 of the 2013 California Building Code (CBC). Representative site coordinates of latitude 33.6506 degrees north and longitude -117.9479 degrees west, were utilized in our analyses. The maximum considered earthquake (MCE) spectral response accelerations (S_{MS} and S_{M1}) and adjusted design spectral response acceleration parameters (S_{DS} and S_{D1}) for Site Class D are provided in Table 1.

TABLE 1

Seismic Design Parameters

Selected Parameters from 2013 CBC, Section 1613 - Earthquake Loads	Seismic Design Values
Site Class per Chapter 20 of ASCE 7	D
Risk-Targeted Spectral Acceleration for Short Periods (S_S)*	1.679g
Risk-Targeted Spectral Accelerations for 1-Second Periods (S_1)*	0.621g
Site Coefficient F_a per Table 1613.3.3(1)	1.0
Site Coefficient F_v per Table 1613.3.3(2)	1.5
Site Modified Spectral Acceleration for Short Periods (S_{MS}) for Site Class D [Note: $S_{MS} = F_a S_S$]	1.679g
Site Modified Spectral Acceleration for 1-Second Periods (S_{M1}) for Site Class D [Note: $S_{M1} = F_v S_1$]	0.932g
Design Spectral Acceleration for Short Periods (S_{DS}) for Site Class D [Note: $S_{DS} = (^2/_3)S_{MS}$]	1.119g
Design Spectral Acceleration for 1-Second Periods (S_{D1}) for Site Class D [Note: $S_{D1} = (^2/_3)S_{M1}$]	0.621g
Mapped Risk Coefficient at 0.2 sec Spectral Response Period, C_{RS} (per ASCE 7)	0.916
Mapped Risk Coefficient at 1 sec Spectral Response Period, C_{R1} (per ASCE 7)	0.931

*From USGS, 2014

Section 1803.5.12 of the 2013 CBC (per Section 11.8.3 of ASCE 7) states that the maximum considered earthquake geometric mean (MCE_G) Peak Ground Acceleration (PGA) should be used for geotechnical evaluations such as liquefaction potential. The PGA_M for the site is equal to 0.681g (USGS, 2014).

A deaggregation of the PGA based on a 2,475-year average return period indicates that an earthquake magnitude of 7.0 at a distance of approximately 2.1 miles (3.4 km) from the site would contribute the most to this ground motion (USGS, 2008b).

2.7 Faulting

Prompted by damaging earthquakes in Northern and Southern California, State legislation and policies concerning the classification and land-use criteria associated with faults have been developed. Their purpose was to prevent the construction of urban developments across the trace of active faults, resulting in the Alquist-Priolo Earthquake Fault Zoning Act. Earthquake Fault Zones have been delineated along the traces of active faults within California. Where developments for human occupation are proposed within these zones, the state requires detailed fault evaluations be performed so that engineering geologists can mitigate the hazards associated with active faulting by identifying the location of active faults and allowing for a setback from the zone of previous ground rupture.

The subject site is not located within an Alquist-Priolo Earthquake Fault Zone and no faults were identified on the site during our site evaluation. The possibility of damage due to ground rupture is considered low since no active faults are known to cross the site.

Secondary effects of seismic shaking resulting from large earthquakes on the major faults in the Southern California region, which may affect the site, include ground lurching, shallow ground rupture, soil liquefaction and dynamic settlement. These secondary effects of seismic shaking are a possibility throughout the Southern California region and are dependant on the distance between the site and causative fault and the onsite geology. Some of the major active faults that could produce these secondary effects include the Newport Inglewood, Whittier-Elsinore, and San Andreas Faults, among others. A discussion of these secondary effects is provided in the following sections.

2.7.1 Liquefaction and Dynamic Settlement

Liquefaction is a seismic phenomenon in which loose, saturated, granular soils behave similarly to a fluid when subject to high-intensity ground shaking. Liquefaction occurs when three general conditions coexist: 1) shallow groundwater; 2) low density non-cohesive (granular) soils; and 3) high-intensity ground motion. Studies indicate that saturated, loose to medium dense, near surface cohesionless soils exhibit the highest liquefaction potential, while dry, dense, cohesionless soils and cohesive soils exhibit low to negligible liquefaction potential. In general, cohesive soils are not considered susceptible to liquefaction, depending on their plasticity and moisture content (Bray & Sancio, 2006). Effects of liquefaction on level ground include settlement, sand boils, and bearing capacity failures below structures. Dynamic settlement of dry loose sands can occur as the sand particles tend to settle and densify as a result of a seismic event.

The site is not located within a State California Seismic Hazard Zone for liquefaction potential (CGS, 1999). Site soils below the historic high groundwater level of approximately 30 feet are not considered susceptible to liquefaction due to the dense to very dense nature of the sandy soils encountered at depth.

2.7.2 Lateral Spreading

Lateral spreading is a type of liquefaction-induced ground failure associated with the lateral displacement of surficial blocks of sediment resulting from liquefaction in a subsurface layer. Once liquefaction transforms the subsurface layer into a fluid mass, gravity plus the earthquake inertial forces may cause the mass to move downslope towards a free face (such as a river channel or an embankment). Lateral spreading may cause large horizontal displacements and such movement typically damages pipelines, utilities, bridges, and structures.

Due to the very low potential for liquefaction, the potential for lateral spreading is also considered very low.

3.0 CONCLUSIONS

Based on the results of our geotechnical evaluation, it is our opinion that the proposed development is feasible from a geotechnical standpoint, provided the following conclusions and recommendations are implemented.

The following is a summary of the primary geotechnical factors that may affect future development of the site:

- In general, our borings indicate that the site contains primarily medium dense clayey sands and stiff sandy clays transitioning at greater depths to dense to very dense sands with varying amounts of silt to the maximum explored depth of approximately 50 feet. The near surface loose and compressible soils are not suitable for the planned improvements.
- Groundwater was not encountered during our subsurface evaluation to the maximum explored depth of approximately 50 feet below current grade. Historic high groundwater is estimated at about 30 feet below current grade (CGS, 1997).
- Active or potentially active faults are not known to exist on or immediately adjacent to the site. The closest known active fault is the onshore segment of the Newport-Inglewood Fault located approximately 2.1 miles (3.4 km) from the site (USGS, 2008a).
- The main seismic hazard that may affect the site is ground shaking from one of the active regional faults. The subject site will likely experience strong seismic ground shaking during its design life.
- The site is not located in a seismic hazard zone for liquefaction and site soils are not considered susceptible to liquefaction due to the dense to very dense nature of soils encountered at depths below historic high groundwater.
- Based on the results of preliminary laboratory testing, site soils are anticipated to have “Low” to “Medium” expansion potential. Mitigation measures are required for foundations and site improvements like concrete flatwork to minimize the impacts of expansive soils.
- From a geotechnical perspective, the existing onsite soils are suitable material for use as fill, provided that they are relatively free from rocks (larger than 8 inches in maximum dimension), construction debris, and significant organic material.
- Site contains clayey soils with high fines content and expansion potential that are not suitable for backfill of any planned site retaining walls. Therefore, import or select grading/stockpiling of sandy soils meeting project recommendations will be required.

4.0 PRELIMINARY RECOMMENDATIONS

The following recommendations are to be considered preliminary, and should be confirmed upon completion of grading and earthwork operations. In addition, they should be considered minimal from a geotechnical viewpoint, as there may be more restrictive requirements from the architect, structural engineer, building codes, governing agencies, or the owner.

It should be noted that the following geotechnical recommendations are intended to provide sufficient information to develop the site in general accordance with the 2013 CBC requirements. With regard to the potential occurrence of potentially catastrophic geotechnical hazards such as fault rupture, earthquake-induced landslides, liquefaction, etc. the following geotechnical recommendations should provide adequate protection for the proposed development to the extent required to reduce seismic risk to an "acceptable level." The "acceptable level" of risk is defined by the California Code of Regulations as "that level that provides reasonable protection of the public safety, though it does not necessarily ensure continued structural integrity and functionality of the project" [Section 3721(a)]. Therefore, repair and remedial work of the proposed improvements may be required after a significant seismic event. With regards to the potential for less significant geologic hazards to the proposed development, the recommendations contained herein are intended as a reasonable protection against the potential damaging effects of geotechnical phenomena such as expansive soils, fill settlement, groundwater seepage, etc. It should be understood, however, that our recommendations are intended to maintain the structural integrity of the proposed development and structures given the site geotechnical conditions, but cannot preclude the potential for some cosmetic distress or nuisance issues to develop as a result of the site geotechnical conditions.

The geotechnical recommendations contained herein must be confirmed to be suitable or modified based on the actual as-graded conditions.

4.1 Site Earthwork

We anticipate that earthwork at the site will consist of the removal of existing improvements associated with the former land use followed by the required earthwork removals, precise grading and construction of the proposed new improvements, including the residential structures, neighborhood amenities, subsurface utilities, interior streets, etc.

We recommend that earthwork onsite be performed in accordance with the following recommendations, future grading plan review report(s), the 2013 CBC/City of Costa Mesa grading requirements, and the General Earthwork and Grading Specifications for Rough Grading included in Appendix D. In case of conflict, the following recommendations shall supersede those included in Appendix D. The following recommendations should be considered preliminary and may be revised within the future grading plan review report or based on the actual conditions encountered during site grading.

4.1.1 Site Preparation

Prior to grading of areas to receive structural fill or engineered improvements, the areas should be cleared of existing asphalt, surface obstructions, and demolition debris. Vegetation and debris should be removed and properly disposed of off-site. Holes resulting from the removal of buried obstructions, which extend below proposed finish grades, should be replaced with suitable compacted fill material.

If cesspools or septic systems are encountered they should be removed in their entirety. The resulting excavation should be backfilled with properly compacted fill soils. As an alternative, cesspools can be backfilled with lean sand-cement slurry. Any encountered wells should be properly abandoned in accordance with regulatory requirements. At the conclusion of the clearing operations, a representative of LGC Geotechnical should observe and accept the site prior to further grading.

4.1.2 Removal Depths and Limits

In order to provide a relatively uniform bearing condition for the planned improvements, we recommend a minimum removal depth for building pad areas of 5 feet below existing grade or 2 feet below planned footings, whichever is greater. The envelope for removals should extend laterally a minimum distance of 5 feet beyond the edges of the proposed improvements.

For minor site structures such as free-standing and screen walls, the removals should extend at least 3 feet beneath the existing grade or 2 feet beneath the base of foundations, whichever is deeper. Within pavement and hardscape areas, removals should extend to a depth of at least 2 feet below the existing grade. The over-excavation in any design cut areas of the pavement may be reduced by the depth of the design cut, but should not be less than 1-foot below the finished subgrade (i.e., below planned aggregate base/asphalt concrete). In general, the envelope for over-excavation should extend laterally a minimum distance of 2 feet beyond the edges of the proposed improvements.

It is anticipated that a perimeter property line free-standing wall will be constructed around a majority of the site. Earthwork removals may be limited due to proximity of the adjacent property line. Footings may need to be deepened due to grading limitations caused by property line constraints. Recommendations will be provided in a geotechnical grading plan review report based on the proposed grading plan.

Local conditions may be encountered during excavation that could require additional over-excavation beyond the above noted minimum in order to obtain an acceptable subgrade. The actual depths and lateral extents of grading will be determined by the geotechnical consultant, based on subsurface conditions encountered during grading. Areas to be over-excavated should be accurately staked in the field by the Project Surveyor.

4.1.3 Temporary Excavations

Temporary excavations should be performed in accordance with project plans, specifications, and all Occupational Safety and Health Administration (OSHA) requirements. Excavations should be laid back or shored in accordance with OSHA requirements before personnel or equipment are allowed to enter.

Based on our field evaluation, the majority of the site soils upper approximate 5 feet are anticipated to be OSHA Type “B” soils (refer to the attached boring logs). Sandy soils were encountered at greater depths and should be considered Type “C” soils. Soil conditions should be regularly evaluated during construction to verify conditions are as anticipated. The contractor shall be responsible for providing the “competent person”, required by OSHA standards, to evaluate soil conditions. Sandy soils are present and should be considered susceptible to caving. The contractor shall be responsible for providing the “competent person”, required by OSHA standards, to evaluate soil conditions. Close coordination with the geotechnical consultant should be maintained to facilitate construction while providing safe excavations. Excavation safety is the sole responsibility of the contractor.

Vehicular traffic, stockpiles, and equipment storage should be set back from the perimeter of excavations a distance equivalent to a 1:1 projection from the bottom of the excavation. Once an excavation has been initiated, it should be backfilled as soon as practical. Prolonged exposure of temporary excavations may result in some localized instability. Excavations should be planned so that they are not initiated without sufficient time to shore/fill them prior to weekends, holidays, or forecasted rain.

It should be noted that any excavation that extends below a 1:1 (horizontal to vertical) projection of an existing foundation will remove existing support of the structure foundation. If requested, temporary shoring parameters will be provided.

4.1.4 Removal Bottoms and Subgrade Preparation

In general, removal bottom areas and any areas to receive compacted fill should be scarified to a minimum depth of 6 inches, brought to a near-optimum moisture condition, and re-compacted per project recommendations. Removal bottoms and areas to receive fill should be observed and accepted by the geotechnical consultant prior to subsequent fill placement.

4.1.5 Material for Fill

From a geotechnical perspective, the onsite soils are generally considered suitable for use as general compacted fill, provided they are screened of organic materials, construction debris and oversized material (8 inches in greatest dimension).

From a geotechnical viewpoint, required import soils for general fill (i.e., non-retaining wall backfill) should consist of clean, granular soils of “Low” expansion potential (expansion index 50 or less based on ASTM D 4829) free of organic materials, construction debris and any material greater than 3 inches. Import for any required retaining wall backfill should meet the criteria outlined in the following paragraph. Source samples should be provided to the geotechnical consultant for laboratory testing a minimum of four working days prior to any planned importation.

Any required retaining wall backfill should consist of sandy soils with a maximum of 35 percent fines (passing the No. 200 sieve) per American Society for Testing and Materials (ASTM) Test Method D1140 (or ASTM D6913/D422) and a “Very Low” expansion potential (EI of 20 or less per ASTM D4829). Soils should also be screened of organic materials, construction debris, and any material greater than 3 inches. The site contains soils that are not suitable for retaining wall backfill due to their fines content, therefore import or potentially select grading and stockpiling of site soils will be required by the contractor for obtaining suitable retaining wall backfill soil.

Aggregate base (crushed aggregate base or crushed miscellaneous base) should conform to the requirements of Section 200-2 of the Standard Specifications for Public Works Construction ("Greenbook") for untreated base materials (except processed miscellaneous base) or Caltrans Class 2 aggregate base.

From a geotechnical viewpoint, the placement of asphalt concrete (AC) fragments in compacted fill is acceptable provided the AC is broken up into pieces not larger than typically used for aggregate base (1-inch in maximum dimension) and limited to compacted fill placed within planned street areas (i.e., not within building pads). Asphalt concrete fragments should be well blended into fill soils with no resulting voids. Approval from an environmental viewpoint and/or the City of Costa Mesa may be required for the use of asphalt concrete in site fills and is not the purview of the geotechnical consultant.

4.1.6 __Placement and Compaction of Fills__

Material to be placed as fill should be brought to near optimum moisture content (generally within optimum and 2 percent above optimum moisture content) and recompacted to at least 90 percent relative compaction (per ASTM D1557). Moisture conditioning of site soils will be required in order to achieve adequate compaction. The optimum lift thickness to produce a uniformly compacted fill will depend on the type and size of compaction equipment used. In general, fill should be placed in uniform lifts not exceeding 8 inches in compacted thickness. Each lift should be thoroughly compacted and accepted prior to subsequent lifts. Generally, placement and compaction of fill should be performed in accordance with local grading ordinances and with observation and testing by the geotechnical consultant. Oversized material as previously defined should be removed from site fills.

During backfill of excavations, the fill should be properly benched into firm and competent soils of temporary backcut slopes as it is placed in lifts.

Aggregate base material should be compacted to at least 95 percent relative compaction at or slightly above optimum moisture content per ASTM D1557. Subgrade below aggregate base should be compacted to at least 90 percent relative compaction per ASTM D1557 at or slightly above optimum moisture content.

4.1.7 __Trench and Retaining Wall Backfill and Compaction__

The onsite soils may generally be suitable as trench backfill, provided the soils are screened of rocks and other material greater than 6 inches in diameter and organic matter. If trenches are shallow or the use of conventional equipment may result in damage to the utilities, sand having a sand equivalent (SE) of 30 or greater may be used to bed and shade the pipes. Sand backfill within the pipe bedding zone may be densified by jetting or flooding and then tamping to ensure adequate compaction. Subsequent trench backfill should be compacted in uniform thin lifts by mechanical means to at least the recommended minimum relative compaction (per ASTM D1557).

Retaining wall backfill should consist of sandy soils as outlined in preceding Section 4.1.5. The limits of select sandy backfill should extend at minimum ½ the height of the retaining wall or the width of the heel (if applicable), whichever is greater (Figure 3). Retaining wall backfill soils should be compacted in relatively uniform thin lifts to at least 90 percent relative compaction (per ASTM D1557). Jetting or flooding of retaining wall backfill materials should not be permitted.

A representative from LGC Geotechnical should observe, probe, and test the backfill to verify compliance with the project recommendations.

4.1.8 Shrinkage and Subsidence

Allowance in the earthwork volumes budget should be made for an estimated 5 to 10 percent reduction in volume of near surface (upper approximate 5 feet) soils. It should be stressed that these values are only estimates and that an actual shrinkage factor would be extremely difficult to predetermine. Subsidence, due to earthwork operations, is expected to be on the order of 0.1 to 0.2 feet. These values are estimates only and exclude losses due to removal of any vegetation or debris. The effective shrinkage of onsite soils will depend primarily on the type of compaction equipment and method of compaction used onsite by the contractor and accuracy of the topographic survey.

4.2 Preliminary Foundation Recommendations

Provided that the remedial grading recommendations provided herein are implemented, the site may be considered suitable for the support of the residential structures using a post-tensioned or conventional foundation system designed to resist the impacts of expansive soils. Please note that the following foundation recommendations are preliminary and must be confirmed by LGC Geotechnical.

Preliminary foundation recommendations are provided in the following sections. Recommended soil bearing and estimated settlement due to structural loads are provided in Section 4.3.

4.2.1 Provisional Conventional Foundation Design Parameters

Conventional foundations may be designed in accordance with Wire Reinforcement Institute (WRI) procedure for slab-on-ground foundations per Section 1808 of the 2013 CBC to resist expansive soils. The following preliminary soil parameters may be used:

- Effective Plasticity Index: 30
- Climatic Rating: Cw = 15
- Reinforcement: Per structural designer
- Minimum Footing Depth: 18 inches below lowest adjacent grade.
- Moisture condition (presoak) slab subgrade to 120% of optimum moisture content to a minimum depth of 18 inches prior to trenching.

The recommended moisture content should be maintained up to the time of concrete placement.

4.2.2 Provisional Post-Tensioned Foundation Design Parameters

The geotechnical parameters provided in Table 2 may be used for post-tensioned slab foundations. These parameters have been determined in general accordance with the Post-Tensioning Institute (PTI) Standard Requirements for Design of Shallow Post-Tensioned Concrete Foundations on Expansive Soils referenced in Chapter 18 of the 2013 CBC. In utilizing these parameters, the foundation engineer should design the foundation system in accordance with the allowable deflection criteria of applicable codes and the requirements of the structural designer/architect. Other types of stiff slabs may be used in place of the CBC post-tensioned slab design provided that, in the opinion of the foundation structural designer, the alternative type of slab is at least as stiff and strong as that designed by the CBC/PTI method to resist expansive soils.

Our design parameters are based on our experience with similar residential projects and the anticipated nature of the soil (with respect to expansion potential). Please note that implementation of our recommendations will not eliminate foundation movement (and related distress) should the moisture content of the subgrade soils fluctuate. It is the intent of these recommendations to help maintain the integrity of the proposed structures and reduce (not eliminate) movement, based upon the anticipated site soil conditions. Should future owners not properly maintain the areas surrounding the foundation, for example by overwatering, then we anticipate for highly expansive soils the maximum differential movement of the perimeter of the foundation to the center of the foundation to be on the order of a couple of inches. Soils of lower expansion potential are anticipated to show less movement.

4.2.3 Post-Tensioned Foundation Subgrade Preparation and Maintenance

Moisture conditioning of the subgrade soils is recommended prior to trenching the foundation. The recommendations specific to the anticipated site soil conditions are presented in Table 2. The subgrade moisture condition of the building pad soils should be maintained at near optimum moisture content up to the time of concrete placement. This moisture content should be maintained around the immediate perimeter of the slab during construction and up to occupancy of the homes.

The geotechnical parameters provided herein assume that if the areas adjacent to the foundation are planted and irrigated, these areas will be designed with proper drainage and adequately maintained so that ponding, which causes significant moisture changes below the foundation, does not occur. Our recommendations do not account for excessive irrigation and/or incorrect landscape design. Plants should only be provided with sufficient irrigation for life and not overwatered to saturate subgrade soils. Sunken planters placed adjacent to the foundation, should either be designed with an efficient drainage system or liners to prevent moisture infiltration below the foundation. Some lifting of the perimeter foundation beam should be expected even with properly constructed planters.

In addition to the factors mentioned above, future homeowners should be made aware of the potential negative influences of trees and/or other large vegetation. Roots that extend near the vicinity of foundations can cause distress to foundations. Future homeowners (and the owner's landscape architect) should not plant trees/large shrubs closer to the foundations than a distance equal to half the mature height of the tree or 20 feet, whichever is more conservative unless specifically provided with root barriers to prevent root growth below the house foundation.

It is the homeowner's responsibility to perform periodic maintenance during hot and dry periods to ensure that adequate watering has been provided to keep soils from separating or pulling back from the foundation. Future homeowners should be informed and educated regarding the importance of maintaining a constant level of soil-moisture. The homeowners should be made aware of the potential negative consequences of both excessive watering, as well as allowing potentially expansive soils to become too dry. Expansive soils can undergo shrinkage during drying, and swelling during the rainy winter season or when irrigation is resumed. This can result in distress to building structures and hardscape improvements. The builder should provide these recommendations to future homeowners.

TABLE 2

Provisional Geotechnical Parameters for Post-Tensioned Foundation Slab Design

Parameter	PT Slab with Perimeter Footing	PT Mat with Thickened Edge
Expansion Index	Medium[1]	Medium[1]
Thornthwaite Moisture Index	-20	-20
Constant Soil Suction	PF 3.9	PF 3.9
Center Lift Edge moisture variation distance, e_m Center lift, y_m	 9.0 feet 0.50 inch	 9.0 feet 0.60 inch
Edge Lift Edge moisture variation distance, e_m Edge lift, y_m	 4.7 feet 1.1 inch	 4.7 feet 1.3 inch
Modulus of Subgrade Reaction, k (assuming presoaking as indicated below)	100 pci	100 pci
Minimum Perimeter footing/thickened edge embedment below finish grade	18 inches	6 inches
1. Assumed for preliminary design purposes. Further evaluation is needed at the completion of grading. 2. Presoak to 120% of optimum moisture content to a minimum depth of 18 inches prior to trenching.		

4.2.4 Slab Underlayment Guidelines

The following is for informational purposes only since slab underlayment (e.g., moisture retarder, sand or gravel layers for concrete curing and/or capillary break) is unrelated to the geotechnical performance of the foundation and thereby not the purview of the geotechnical consultant. Post-construction moisture migration should be expected below the foundation. The foundation engineer/architect should determine whether the use of a capillary break (sand or gravel layer), in conjunction with the vapor retarder, is necessary or required by code. Sand layer thickness and location (above and/or below vapor retarder) should also be determined by the foundation engineer/architect.

4.3 Soil Bearing and Lateral Resistance

Provided our earthwork recommendations are implemented, an allowable soil bearing pressure of 1,500 pounds per square foot (psf) may be used for the design of footings having a minimum width of 12 inches and minimum embedment of 18 inches below lowest adjacent ground surface. This value may be increased by 300 psf for each additional foot of embedment of 100 psf for each additional foot of foundation width to a maximum value of 2,500 psf. A mat foundation a minimum of 6-inches below lowest adjacent grade may be designed for an allowable soil bearing pressure of 1,200 psf. These allowable bearing pressures are applicable for level (ground slope equal to or flatter than 5H:1V) conditions only. Bearing values indicated are for total dead loads and frequently applied live loads and may be increased by ⅓ for short duration loading (i.e., wind or seismic loads).

In utilizing the above-mentioned allowable bearing capacity and provided our earthwork recommendations are implemented, foundation settlement due to structural loads is anticipated to be 1-inch or less. Differential settlement may be taken as half of the total settlement (i.e., ½-inch over a horizontal span of 40 feet).

Resistance to lateral loads can be provided by friction acting at the base of foundations and by passive earth pressure. For concrete/soil frictional resistance, an allowable coefficient of friction of 0.30 may be assumed with dead-load forces. An allowable passive lateral earth pressure of 240 psf per foot of depth (or pcf) to a maximum of 2,400 psf may be used for the sides of footings poured against properly compacted fill. Allowable passive pressure may be increased to 320 pcf to a maximum of 3,200 psf for short duration seismic loading. This passive pressure is applicable for level (ground slope equal to or flatter than 5H:1V) conditions. Frictional resistance and passive pressure may be used in combination without reduction. We recommend that the upper foot of passive resistance be neglected if finished grade will not be covered with concrete or asphalt. The provided allowable passive pressures are based on a factor of safety of 1.5 and 1.1 for static and seismic loading conditions, respectively.

4.4 Lateral Earth Pressures for Retaining Walls

The following lateral earth pressures may be used for the preliminary design of the subject site retaining walls up to about 6 feet in height.

Lateral earth pressures for approved native sandy soils meeting indicated project requirements are provided below. Lateral earth pressures are provided as equivalent fluid unit weights, in psf per foot of depth (or pcf). These values do not contain an appreciable factor of safety, so the retaining wall designer should apply the applicable factors of safety and/or load factors during design. A soil unit weight of 120 pcf may be assumed for calculating the actual weight of soil over the wall footing.

The following lateral earth pressures are presented in Table 3 for approved select granular soils a maximum of 35 percent fines (passing the No. 200 sieve per ASTM D1140) and an Expansion Index of 20 or less per ASTM D4829. The retaining wall designer should clearly indicate on the retaining wall plans the required select sandy soil backfill.

TABLE 3

Lateral Earth Pressures – Approved Select Material

Conditions	**Equivalent Fluid Unit Weight (pcf)**
	Level Backfill
	Approved Soils
Active	35
At-Rest	55

The lateral earth pressures provided above may be increased by a factor of 1.5 for a 2:1 (horizontal to vertical) sloping backfill condition.

If the wall can yield enough to mobilize the full shear strength of the soil, it can be designed for "active" pressure. If the wall cannot yield under the applied load, the earth pressure will be higher. This would include 90-degree corners of retaining walls. Such walls should be designed for "at-rest." The equivalent fluid pressure values assume free-draining conditions. If conditions other than those assumed above are anticipated, the equivalent fluid pressure values should be provided on an individual-case basis by the geotechnical engineer.

Surcharge loading effects from any adjacent structures should be evaluated by the retaining wall designer. In general, structural loads within a 1:1 (horizontal: vertical) upward projection from the bottom of the proposed retaining wall footing will surcharge the proposed retaining wall. In addition to the recommended earth pressure, retaining walls adjacent to streets should be designed to resist a uniform lateral pressure of 100 pounds per square foot (psf) due to normal street vehicle traffic if applicable. The retaining wall designer should contact the geotechnical engineer for any required geotechnical input in estimating surcharge loads.

If required, the retaining wall designer may use a seismic lateral earth pressure increment of 7 pcf. This increment should be applied in addition to the provided static lateral earth pressure using a triangular distribution with the resultant acting at H/3 in relation to the base of the retaining structure (where H is the retained height). Per Section 1803.5.12 of the 2013 CBC, the seismic lateral earth pressure is applicable to structures assigned to Seismic Design Category D through F for retaining wall structures supporting more than 6 feet of backfill height. This seismic lateral earth pressure is estimated using the procedure outlined by the Structural Engineers Association of California (Lew, et al, 2010). The provided seismic lateral earth pressure is for a level backfill condition; a sloping backfill condition is not anticipated. However, if a sloping backfill condition is proposed, the retaining wall designer should contact the geotechnical engineer for specific seismic lateral earth pressure increments based on the configuration of the planned retaining walls.

Retaining wall structures should be provided with appropriate drainage and appropriately waterproofed. To reduce, but not eliminate, saturation of near surface (upper approximate 1 foot) soils in front of the retaining walls, the perforated subdrain pipe should be located as low as possible behind the retaining wall. The outlet pipe should be sloped to drain to a suitable outlet. In general, we do not recommend retaining wall outlet pipes be connected to area drains. If subdrains are connected to area drains, special care and information should be provided to homeowners to maintain these drains. Typical retaining wall drainage is illustrated in Figure 3. It should be noted that the recommended subdrain does not provide protection against seepage through the face of the wall and/or efflorescence. Efflorescence is generally a white crystalline powder (discoloration) that results when water containing soluble salts migrates over a period of time through the face of a retaining wall and evaporates. If such seepage or efflorescence is undesirable, retaining walls should be waterproofed to reduce this potential.

Soil bearing and lateral resistance (friction coefficient and passive resistance) are provided in Section 4.3. Earthwork considerations (temporary backcuts, backfill, compaction, etc.) for retaining walls are provided in Section 4.1 (Site Earthwork) and the subsequent earthwork related sub-sections.

4.5 Control of Surface Water and Drainage Control

Positive drainage of surface water away from structures is very important. Water should not be allowed to pond adjacent to buildings or to flow freely down a graded slope. Per the 2013 CBC, positive drainage may be accomplished by providing drainage away from buildings at a gradient of at least 5 percent for earthen surfaces for a distance of at least 10 feet away from the face of wall. If a distance of 10 feet cannot be achieved, an alternative of a gradient of at least 5 percent to an area drain or swale having a gradient of 2 percent is acceptable. Where necessary, drainage paths or drainage gradients may be shortened or reduced by use of area drains and collector pipes. A drainage gradient of 1 percent may be used for sideyard swales provided a 5 percent gradient is maintained away from the building. Impervious surfaces within 10 feet of the building foundation shall be sloped a minimum of 2 percent away from the building. Drainage swales used in conjunction with area drains should be designed by the project civil engineer so that a properly constructed and maintained system will prevent ponding within 5 feet of the foundation. Eave gutters are recommended and should reduce water infiltration into the subgrade soils if the downspouts are properly connected to appropriate outlets.

Planters with open bottoms adjacent to buildings should be avoided. Planters should not be designed adjacent to buildings unless provisions for drainage, such as catch basins, liners, and/or area drains, are made. Overwatering must be avoided.

4.6 Subsurface Water Infiltration

Recent regulatory changes mandate that storm water be infiltrated rather than discharged via conventional storm drainage systems. Typically, a combination of methods are implemented to reduce surface water runoff and increase infiltration including; permeable pavements/pavers for roadways and walkways, directing surface water runoff to grass-lined swales, retention areas, and/or drywells.
In general, the vast majority of geotechnical distress issues are directly related to improper drainage. Distress in the form of movement of improvements could occur as a result of soil saturation and loss of soil support, settlement, collapse, internal soil erosion, and/or expansion. Infiltrated water may enter underground utility pipe zones and migrate along the pipe backfill, potentially impacting other improvements located far away from the point of infiltration.

From a geotechnical perspective, we do not recommend that water be intentionally infiltrated. Impacts from infiltration could result in additional foundation settlement beyond the amount estimated due to structural loads. This additional settlement could impact structural foundations and site existing and planned improvements. If it is determined that water must be infiltrated due to regulatory requirements, we recommend the absolute minimum amount of water be infiltrated into site soils. Any infiltrated water will have to be located at adequate setback distances with respect to both depth and distance from foundations. If necessary, LGC Geotechnical should be provided with details for any planned required infiltration system early in the design process for geotechnical input.

4.7 *Preliminary Asphalt Pavement Sections*

The following provisional minimum street sections are provided in Table 4 based on an assumed R-value of 15 for Traffic Indices (TI) of 4.5 through 6.0. These recommendations must be confirmed with R-value testing of representative near-surface soils at the completion of grading and after underground utilities have been installed and backfilled. Final street sections should be confirmed by the project civil engineer based upon the final design Traffic Index. If requested, LGC Geotechnical will provide sections for alternate TI values.

TABLE 4

Paving Section Options

Assumed Traffic Index	4.5	5	6
R -Value Subgrade	15	15	15
AC Thickness	4.0 inches	4.0 inches	5 inches
Base Thickness	4.0 inches	6.5 inches	8.5 inches

The thicknesses shown are for minimum thicknesses. Increasing the thickness of any or all of the above layers will reduce the likelihood of the pavement experiencing distress during its service life. The above recommendations are based on the assumption that proper maintenance and irrigation of the areas adjacent to the roadway will occur through the design life of the pavement. Failure to maintain a proper maintenance and/or irrigation program may jeopardize the integrity of the pavement.

Earthwork recommendations regarding aggregate base and subgrade are provided in the previous section "Site Earthwork" and the related sub-sections of this report.

4.8 *Soil Corrosivity*

Although not corrosion engineers (LGC Geotechnical is not a corrosion consultant), several governing agencies in Southern California require the geotechnical consultant to determine the corrosion potential of soils to buried concrete and metal facilities. We therefore present the results of our testing with regard to corrosion for the use of the client and other consultants, as they determine necessary.

Corrosion testing of a near-surface bulk samples indicated a soluble sulfate content of approximately 0.01 percent or less, a chloride content of 40 parts per million (ppm), pH of 8.5 and a minimum resistivity of 2,355 ohm-cm. Based on Caltrans Corrosion Guidelines (Caltrans, 2012), soils are considered corrosive to structural elements if the pH is 5.5 or less, or the chloride concentration is 500 ppm or greater, or the sulfate concentration is 2,000 ppm or greater.

Based on laboratory sulfate test results, the near surface soils have a severity categorization of “Not Applicable” and are designated to a class “S0” per ACI 318, Table 4.2.1 with respect to sulfates. Concrete in direct contact with the onsite soils can be designed according to ACI 318, section 4.3 using the “S0” sulfate classification. This must be verified based on as-graded conditions.

4.9 Nonstructural Concrete Flatwork

Concrete flatwork (such as walkways, bicycle trails, patio slabs, etc.) has a potential for cracking due to changes in soil volume related to soil-moisture fluctuations. To reduce the potential for excessive cracking and lifting, concrete may be designed in accordance with the minimum guidelines outlined in Table 5. These guidelines will reduce the potential for irregular cracking and promote cracking along construction joints, but will not eliminate all cracking or lifting. Thickening the concrete and/or adding additional reinforcement will further reduce cosmetic distress.

TABLE 5

Nonstructural Concrete Flatwork for Medium Expansion Potential

	Homeowner Sidewalks	**Private Drives**	**Patios/Entryways**	**City Sidewalk Curb and Gutters**
Minimum Thickness (in.)	4 (nominal)	5 (full)	5 (full)	City/Agency Standard
Presoaking	Wet down prior to placing	Presoak to 12 inches	Presoak to 12 inches	City/Agency Standard
Reinforcement	—	No. 3 at 24 inches on centers	No. 3 at 24 inches on centers	City/Agency Standard
Thickened Edge (in.)	—	8 x 8	—	City/Agency Standard
Crack Control Joints	Saw cut or deep open tool joint to a minimum of $^{1}/_{3}$ the concrete thickness	Saw cut or deep open tool joint to a minimum of $^{1}/_{3}$ the concrete thickness	Saw cut or deep open tool joint to a minimum of $^{1}/_{3}$ the concrete thickness	City/Agency Standard
Maximum Joint Spacing	5 feet	10 feet or quarter cut whichever is closer	6 feet	City/Agency Standard
Aggregate Base Thickness (in.)	—	—	2	City/Agency Standard

To reduce the potential for driveways to separate from the garage slab, the builder may elect to install dowels to tie these two elements together. Similarly, future homeowners should consider the use of dowels to connect flatwork to the foundation.

4.10 Grading and Foundation Plan Review

When available, grading and foundation plans should be reviewed by LGC Geotechnical in order to verify our geotechnical recommendations are implemented. Updated recommendations and/or additional field work may be necessary.

4.11 Geotechnical Observation and Testing During Construction

The recommendations provided in this report are based on limited subsurface observations and geotechnical analysis. The interpolated subsurface conditions should be checked in the field during construction by a representative of LGC Geotechnical. Geotechnical observation and testing is required per Section 1705 of the 2013 California Building Code (CBC).

Geotechnical observation and/or testing should be performed by LGC Geotechnical at the following stages:

- During grading (removal bottoms, fill placement, etc);
- During utility trench and retaining wall backfill and compaction;
- After presoaking building pads and other concrete-flatwork subgrades, and prior to placement of aggregate base or concrete;
- Preparation of pavement subgrade and placement of aggregate base;
- After building and wall footing excavation and prior to placing reinforcement and/or concrete; and
- When any unusual soil conditions are encountered during any construction operation subsequent to issuance of this report.

5.0 LIMITATIONS

Our services were performed using the degree of care and skill ordinarily exercised, under similar circumstances, by reputable soils engineers and geologists practicing in this or similar localities. No other warranty, expressed or implied, is made as to the conclusions and professional advice included in this report.

This report is based on data obtained from limited observations of the site, which have been extrapolated to characterize the site. While the scope of services performed is considered suitable to adequately characterize the site geotechnical conditions relative to the proposed development, no practical evaluation can completely eliminate uncertainty regarding the anticipated geotechnical conditions in connection with a subject site. Variations may exist and conditions not observed or described in this report may be encountered during grading and construction.

This report is issued with the understanding that it is the responsibility of the owner, or of his/her representative, to ensure that the information and recommendations contained herein are brought to the attention of the other consultants (at a minimum the civil engineer, structural engineer, landscape architect) and incorporated into their plans. The contractor should properly implement the recommendations during construction and notify the owner if they consider any of the recommendations presented herein to be unsafe, or unsuitable.

The findings of this report are valid as of the present date. However, changes in the conditions of a site can and do occur with the passage of time, whether they be due to natural processes or the works of man on this or adjacent properties. The findings, conclusions, and recommendations presented in this report can be relied upon only if LGC Geotechnical has the opportunity to observe the subsurface conditions during grading and construction of the project, in order to confirm that our preliminary findings are representative for the site. This report is intended exclusively for use by the client, any use of or reliance on this report by a third party shall be at such party's sole risk.

In addition, changes in applicable or appropriate standards may occur, whether they result from legislation or the broadening of knowledge. Accordingly, the findings of this report may be invalidated wholly or partially by changes outside our control. Therefore, this report is subject to review and modification.

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FIGURE 2
Boring Location Map

PROJECT NAME	1239 Victoria St. Costa Mesa
PROJECT NO.	13192-01
ENG. / GEOL.	DJB
SCALE	1" : 60'
DATE	April 2014

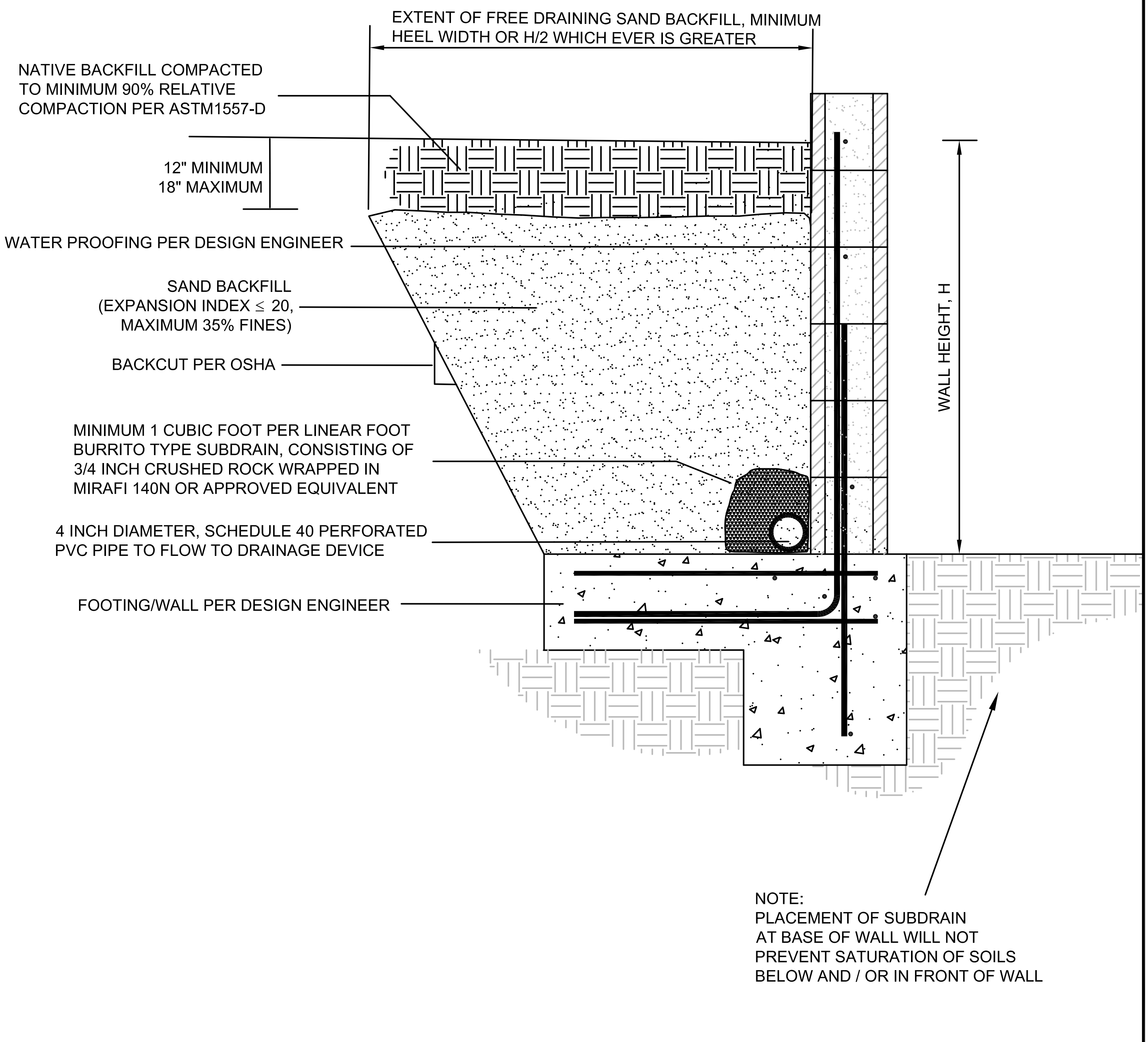

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FIGURE 3
Retaining Wall Backfill Detail

PROJECT NAME	1239 Victoria St, Costa Mesa
PROJECT NO.	13192-01
ENG.	DJB / BTZ
SCALE	Not to Scale
DATE	April 2014

Appendix A
References

APPENDIX A

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Appendix B
Field Exploration Logs & Infiltration Data

Geotechnical Boring Log Borehole LGC-HS-1

Date: 2/27/2014	**Drilling Company:** CALPAC
Project Name: 1239 Victoria St	**Type of Rig:** Hollow Stem Auger Mounted Rig
Project Number: 13192-01	**Drop:** 30" **Hole Diameter:** 8"
Elevation of Top of Hole: ~89' MSL	**Drive Weight:** 140 pounds
Hole Location: See Geotechnical Map	Page 1 of 1

Logged By CDH
Sampled By CDH
Checked By BTZ

Elevation (ft)	Depth (ft)	Graphic Log	Sample Number	Blow Count	Dry Density (pcf)	Moisture (%)	USCS Symbol	DESCRIPTION	Type of Test
	0							Approximately 6 inches of Asphalt Concrete (AC) (No Aggregate Base)	
		B-1	R-1	5 9 7	119.6	12.3	CL	@ 2.5' - Sandy CLAY; orangish brown, moist, fine to medium grain, stiff	EI, CR
84	5		SPT-1	4 6 10		13.0	CL	@ 5' - Sandy CLAY; brown, moist, fine to medium grain, very stiff	S&H
79	10		R-2	4 9 17	106.3	4.4	SP-SM	@ 10' - Poorly Graded SAND with Silt; yellowish brown, slightly moist, fine to medium grain, medium dense	CN
74	15		SPT-2	4 6 8		5.9	SP-SM	@ 15' - Poorly Graded SAND with Silt; yellowish brown, slightly moist, fine to medium grain, medium dense	
69	20		R-3	6 23 46	110.3	7.3	SM	@ 20' - Silty SAND; mottled (light and dark brown), slightly moist, fine grain, dense	
64	25							Total Depth = 20' Groundwater Not Encountered Backfilled with Cuttings and Capped with AC Cold Patch on 2/27/2014	
	30								

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THIS SUMMARY APPLIES ONLY AT THE LOCATION OF THIS BORING AND AT THE TIME OF DRILLING. SUBSURFACE CONDITIONS MAY DIFFER AT OTHER LOCATIONS AND MAY CHANGE AT THIS LOCATION WITH THE PASSAGE OF TIME. THE DATA PRESENTED IS A SIMPLIFICATION OF THE ACTUAL CONDITIONS ENCOUNTERED. THE DESCRIPTIONS PROVIDED ARE QUALITATIVE FIELD DESCRIPTIONS AND ARE NOT BASED ON QUANTITATIVE ENGINEERING ANALYSIS.

SAMPLE TYPES:
- B BULK SAMPLE
- R RING SAMPLE (CA Modified Sampler)
- G GRAB SAMPLE
- SPT STANDARD PENETRATION TEST SAMPLE
- GROUNDWATER TABLE

TEST TYPES:
- DS DIRECT SHEAR
- MD MAXIMUM DENSITY
- SA SIEVE ANALYSIS
- S&H SIEVE AND HYDROMETER
- EI EXPANSION INDEX
- CN CONSOLIDATION
- CR CORROSION
- AL ATTERBERG LIMITS
- CO COLLAPSE/SWELL
- RV R-VALUE
- -#200 % PASSING # 200 SIEVE

Last Edited: 3/20/2014

Geotechnical Boring Log Borehole LGC-HS-2

Date: 2/27/2014	**Drilling Company:** CALPAC
Project Name: 1239 Victoria St	**Type of Rig:** Hollow Stem Auger Mounted Rig
Project Number: 13192-01	**Drop:** 30" **Hole Diameter:** 8"
Elevation of Top of Hole: ~91' MSL	**Drive Weight:** 140 pounds
Hole Location: See Geotechnical Map	Page 1 of 1

Logged By CDH
Sampled By CDH
Checked By BTZ

Elevation (ft)	Depth (ft)	Graphic Log	Sample Number	Blow Count	Dry Density (pcf)	Moisture (%)	USCS Symbol	DESCRIPTION	Type of Test
	0							Approximately 6 inches of Asphalt Concrete (AC) over Approximately 10 inches Aggregate Base	
			R-1	4 15 21	116.3	8.4	SC	@ 2.5' - Clayey SAND, dark brown, slightly moist, fine to medium grain, medium dense	
86	5		R-2	9 20 31	120.0	10.6	CL	@ 5' - Lean CLAY with sand; olive brown, slightly moist, fine to medium grain, hard	AL, CN
			SPT-2	5 9 9		6.8	SM	@ 7.5' Silty SAND; brown, moist, fine grain, medium dense	
81	10		R-3	4 16 22	116.0	13.5	SM	@ 10' - Silty SAND, brown, very moist, fine to medium grain, medium dense	
76	15		SPT-3	5 10 14		6.1	SM	@ 15' - Silty SAND; brown, slightly moist, fine to medium grain, medium dense	
71	20		R-4	12 20 29	95.8	5.5	SM	@ 20' - Silty SAND, mottled (brown, orange, olive), slightly moist, fine grain, dense.	
								Total Depth = 20' Groundwater Not Encountered Backfilled with Cuttings and Capped with AC Cold Patch on 2/27/2014	
66	25								
	30								

LGC Geotechnical, Inc.

THIS SUMMARY APPLIES ONLY AT THE LOCATION OF THIS BORING AND AT THE TIME OF DRILLING. SUBSURFACE CONDITIONS MAY DIFFER AT OTHER LOCATIONS AND MAY CHANGE AT THIS LOCATION WITH THE PASSAGE OF TIME. THE DATA PRESENTED IS A SIMPLIFICATION OF THE ACTUAL CONDITIONS ENCOUNTERED. THE DESCRIPTIONS PROVIDED ARE QUALITATIVE FIELD DESCRIPTIONS AND ARE NOT BASED ON QUANTITATIVE ENGINEERING ANALYSIS.

SAMPLE TYPES:
- B BULK SAMPLE
- R RING SAMPLE (CA Modified Sampler)
- G GRAB SAMPLE
- SPT STANDARD PENETRATION TEST SAMPLE
- GROUNDWATER TABLE

TEST TYPES:
- DS DIRECT SHEAR
- MD MAXIMUM DENSITY
- SA SIEVE ANALYSIS
- S&H SIEVE AND HYDROMETER
- EI EXPANSION INDEX
- CN CONSOLIDATION
- CR CORROSION
- AL ATTERBERG LIMITS
- CO COLLAPSE/SWELL
- RV R-VALUE
- -#200 % PASSING # 200 SIEVE

Last Edited: 3/20/2014

Geotechnical Boring Log Borehole LGC-HS-3

Date: 2/27/2014	**Drilling Company:** CALPAC
Project Name: 1239 Victoria St	**Type of Rig:** Hollow Stem Auger Mounted Rig
Project Number: 13192-01	**Drop:** 30" **Hole Diameter:** 8"
Elevation of Top of Hole: ~87' MSL	**Drive Weight:** 140 pounds
Hole Location: See Geotechnical Map	Page 1 of 1

Logged By CDH
Sampled By CDH
Checked By BTZ

Elevation (ft)	Depth (ft)	Graphic Log	Sample Number	Blow Count	Dry Density (pcf)	Moisture (%)	USCS Symbol	DESCRIPTION	Type of Test
	0							Approximately 5 inches of Asphalt Concrete (AC) (No Aggregate Base)	
			R-1	8 21 31	120.3	11.1	SC	@ 2.5' - Clayey SAND; brown, slightly moist, fine to medium, dense	
82	5		SPT-1	5 6 6		5.9	SC-SM	@ 5' - Silty Clayey SAND, brown, slightly moist, fine to medium grain, medium dense	#200, AL
			R-2	10 14 19	104.1	3.1	SM	@ 7.5' - Silty SAND, brown, slightly moist, fine to medium grain, medium dense	
77	10		SPT-2	8 10 12		2.8	SM	@ 10' - Silty SAND; brown, slightly moist, fine to medium grain, medium dense	
72	15		R-3	6 22 22	90.7	10.6	SM	@ 15' - Silty SAND, orangish brown, moist, fine grained, dense	
67	20		SPT-3	14 19 21		5.3	SM	@ 20' - Silty SAND; mottled (brown, orange, olive), slightly moist, fine grained, dense	
62	25							Total Depth = 20' Groundwater Not Encountered Backfilled with Cuttings and Capped with AC Cold Patch on 2/27/2014	
	30								

LGC Geotechnical, Inc.

THIS SUMMARY APPLIES ONLY AT THE LOCATION OF THIS BORING AND AT THE TIME OF DRILLING. SUBSURFACE CONDITIONS MAY DIFFER AT OTHER LOCATIONS AND MAY CHANGE AT THIS LOCATION WITH THE PASSAGE OF TIME. THE DATA PRESENTED IS A SIMPLIFICATION OF THE ACTUAL CONDITIONS ENCOUNTERED. THE DESCRIPTIONS PROVIDED ARE QUALITATIVE FIELD DESCRIPTIONS AND ARE NOT BASED ON QUANTITATIVE ENGINEERING ANALYSIS.

SAMPLE TYPES:
- B BULK SAMPLE
- R RING SAMPLE (CA Modified Sampler)
- G GRAB SAMPLE
- SPT STANDARD PENETRATION TEST SAMPLE
- GROUNDWATER TABLE

TEST TYPES:
- DS DIRECT SHEAR
- MD MAXIMUM DENSITY
- SA SIEVE ANALYSIS
- S&H SIEVE AND HYDROMETER
- EI EXPANSION INDEX
- CN CONSOLIDATION
- CR CORROSION
- AL ATTERBERG LIMITS
- CO COLLAPSE/SWELL
- RV R-VALUE
- -#200 % PASSING # 200 SIEVE

Last Edited: 3/20/2014

Geotechnical Boring Log Borehole LGC-HS-4

Date: 2/27/2014	**Drilling Company:** CALPAC
Project Name: 1239 Victoria St	**Type of Rig:** Hollow Stem Auger Mounted Rig
Project Number: 13192-01	**Drop:** 30" **Hole Diameter:** 8"
Elevation of Top of Hole: ~89' MSL	**Drive Weight:** 140 pounds
Hole Location: See Geotechnical Map	Page 1 of 2

Logged By CDH
Sampled By CDH
Checked By BTZ

Elevation (ft)	Depth (ft)	Graphic Log	Sample Number	Blow Count	Dry Density (pcf)	Moisture (%)	USCS Symbol	DESCRIPTION	Type of Test
	0							Approximately 4 inches of Asphalt Concrete (AC) over Approximately 5 inches of Aggregate Base	
		B-1	R-1	4 12 22	116.9	15.1	SC	@ 2.5' - Clayey SAND; brown, very moist, fine to medium grain, medium dense	MD, EI, SO_4
84	5		SPT-1	6 9 13		11.4	SC	@ 5' - Clayey SAND; brown, moist, fine to medium grain, medium dense	#200, AL
			R-2	7 17 20	102.7	3.8	SP-SM	@ 7.5' - Clayey SAND; brown, slightly moist, fine to medium grain, medium dense	
79	10		SPT-2	7 9 11		3.1	SP	@ 10' - SAND; brown, slightly moist, fine to medium grain, medium dense	#200
74	15		R-3	6 17 20	96.5	4.8	SP-SM	@ 15' - SAND with Silt; light brown, slightly moist, fine grain, medium dense	
69	20		SPT-3	11 17 20		9.8	SM	@ 20' - Silty SAND; mottled (brown, orange, olive), moist, fine grain, dense	
64	25		R-4	12 22 30	97.0	27.1	CH	@ 25' - CLAY; olive brown, very moist, fine grain, hard	AL, CN
	30								

LGC Geotechnical, Inc.

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- EI EXPANSION INDEX
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- CR CORROSION
- AL ATTERBERG LIMITS
- CO COLLAPSE/SWELL
- RV R-VALUE
- -#200 % PASSING # 200 SIEVE

Last Edited: 3/20/2014

Geotechnical Boring Log Borehole LGC-HS-4

Date: 2/27/2014	**Drilling Company:** CALPAC
Project Name: 1239 Victoria St	**Type of Rig:** Hollow Stem Auger Mounted Rig
Project Number: 13192-01	**Drop:** 30" **Hole Diameter:** 8"
Elevation of Top of Hole: ~89' MSL	**Drive Weight:** 140 pounds
Hole Location: See Geotechnical Map	Page 2 of 2

Logged By CDH
Sampled By CDH
Checked By BTZ

Elevation (ft)	Depth (ft)	Graphic Log	Sample Number	Blow Count	Dry Density (pcf)	Moisture (%)	USCS Symbol	DESCRIPTION	Type of Test
	30		SPT-4	8 20 25		9.2	SM	@ 30' - Silty SAND; brown, moist, fine grain, dense	
59	35		R-5	21 41 50/5"	112.3	2.4	SP-SM	@ 35' - SAND with Silt; light brown, slightly moist, fine to coarse grain, very dense	#200
54	40		SPT-5	22 26 28		2.8	SP-SM	@ 40' - SAND with Silt; orangish brown, white, slightly moist, fine to coarse grain, very dense	
49	45		R-6	13 31 50/6"	94.6	25.9	SM	@ 45' - Silty SAND; mottled (brown, orange, olive), very moist, fine grained, very dense	
44	50		SPT-6	27 50 50/6"		8.8	SM	@ 50' - Fine Silty SAND; mottled (brown, orange, olive), moist, fine grained, very dense	
39	55							Total Depth = 50' Groundwater Not Encountered Backfilled with Cuttings and Capped with AC Cold Patch on 2/27/2014	
	60								

LGC Geotechnical, Inc.

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- EI EXPANSION INDEX
- CN CONSOLIDATION
- CR CORROSION
- AL ATTERBERG LIMITS
- CO COLLAPSE/SWELL
- RV R-VALUE
- -#200 % PASSING # 200 SIEVE

Geotechnical Boring Log Borehole LGC-HS-5

Date: 2/27/2014	**Drilling Company:** CALPAC
Project Name: 1239 Victoria St	**Type of Rig:** Hollow Stem Auger Mounted Rig
Project Number: 13192-01	**Drop:** 30" **Hole Diameter:** 8"
Elevation of Top of Hole: ~82' MSL	**Drive Weight:** 140 pounds
Hole Location: See Geotechnical Map	Page 1 of 1

Logged By CDH
Sampled By CDH
Checked By BTZ

Elevation (ft)	Depth (ft)	Graphic Log	Sample Number	Blow Count	Dry Density (pcf)	Moisture (%)	USCS Symbol	DESCRIPTION	Type of Test
	0							Approximately 5.5 inches of Asphalt Concrete (AC) over Approximately 5 inches of Aggregate Base	
			R-1	8 9 16	119.1	12.8	SC	@ 2.5' - Clayey SAND; dark brown, moist, fine to medium grain, medium dense	
77	5		SPT-1	6 7 9		12.2	SC	@ 5' - Clayey SAND; dark brown, moist, fine to medium grain, medium dense	
			R-2	8 10 10	112.3	14.7	SM	@ 7.5' - Silty SAND; brown, very moist, fine to medium grain, medium dense	DS
72	10							Total Depth = 10' Groundwater Not Encountered Backfilled with Cuttings and Capped with AC Cold Patch on 2/27/2014	
67	15								
62	20								
57	25								
	30								

LGC Geotechnical, Inc.

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- G GRAB SAMPLE
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- GROUNDWATER TABLE

TEST TYPES:
- DS DIRECT SHEAR
- MD MAXIMUM DENSITY
- SA SIEVE ANALYSIS
- S&H SIEVE AND HYDROMETER
- EI EXPANSION INDEX
- CN CONSOLIDATION
- CR CORROSION
- AL ATTERBERG LIMITS
- CO COLLAPSE/SWELL
- RV R-VALUE
- -#200 % PASSING # 200 SIEVE

Last Edited: 3/20/2014

Geotechnical Boring Log Borehole LGC-HS-6

Date: 2/27/2014	**Drilling Company:** CALPAC
Project Name: 1239 Victoria St	**Type of Rig:** Hollow Stem Auger Mounted Rig
Project Number: 13192-01	**Drop:** 30" **Hole Diameter:** 8"
Elevation of Top of Hole: ~82' MSL	**Drive Weight:** 140 pounds
Hole Location: See Geotechnical Map	Page 1 of 1

Logged By CDH
Sampled By CDH
Checked By BTZ

Elevation (ft)	Depth (ft)	Graphic Log	Sample Number	Blow Count	Dry Density (pcf)	Moisture (%)	USCS Symbol	DESCRIPTION	Type of Test
	0							Approximately 5 inches of Asphalt Concrete (AC) over Approximately 6 inches of Aggregate Base	
77	5								
			R-1	6 8 10	102.1	11.2	SM	@ 7.5' - Silty SAND; brown, moist, fine to medium grain, medium dense	
72	10								
								Total Depth = 10.67' Groundwater Not Encountered Drilled to 10.67'; Backfilled with 3" Diameter PVC Pipe and 3/4" Gravel on 2/27/14; Removed Pipe and Topped with Gravel and AC Cold Patch on 2/27/2014	
67	15								
62	20								
57	25								
	30								

LGC Geotechnical, Inc.

THIS SUMMARY APPLIES ONLY AT THE LOCATION OF THIS BORING AND AT THE TIME OF DRILLING. SUBSURFACE CONDITIONS MAY DIFFER AT OTHER LOCATIONS AND MAY CHANGE AT THIS LOCATION WITH THE PASSAGE OF TIME. THE DATA PRESENTED IS A SIMPLIFICATION OF THE ACTUAL CONDITIONS ENCOUNTERED. THE DESCRIPTIONS PROVIDED ARE QUALITATIVE FIELD DESCRIPTIONS AND ARE NOT BASED ON QUANTITATIVE ENGINEERING ANALYSIS.

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- R RING SAMPLE (CA Modified Sampler)
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- GROUNDWATER TABLE

TEST TYPES:
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- MD MAXIMUM DENSITY
- SA SIEVE ANALYSIS
- S&H SIEVE AND HYDROMETER
- EI EXPANSION INDEX
- CN CONSOLIDATION
- CR CORROSION
- AL ATTERBERG LIMITS
- CO COLLAPSE/SWELL
- RV R-VALUE
- -#200 % PASSING # 200 SIEVE

Last Edited: 3/20/2014

Infiltration Test Data Sheet

LGC Geotechnical, Inc
131 Calle Iglesia Suite 200, San Clemente, CA 92672 tel. (949) 369-6141

Project Name: 1239 Victoria
Project Number: 13192-01
Date: 2/27/2014
Boring Number: HS-6

Test hole dimensions (if circular)	
Boring Depth (feet)*:	9.45
Boring Diameter (inches):	8
Pipe Diameter (inches):	3

*measured at time of test

Test pit dimensions (if rectangular)	
Pit Depth (feet):	
Pit Length (feet):	
Pit Breadth (feet):	

*Pre-Test (Sandy Soil Criteria)**

Trial No.	Start Time (24:HR)	Stop Time (24:HR)	Time Interval (min)	Initial Depth to Water (feet)	Final Depth to Water (feet)	Total Change in Water Level (feet)	Greater Than or Equal to 0.5 feet (yes/no)
1	4:38	5:08	30.0	6.7	8.66	1.96	yes
2	5:10	5:40	30.0	6.63	8.55	1.92	yes

*If two consecutive measurements show that six inches of water seeps away in less than 25 minutes, the test shall be run for an additional hour with measurements taken every 10 minutes. Otherwise, pre-soak (fill) overnight, and then obtain at least twelve measurements per hole over at least six hours (approximately 30 minute intervals) with a precision of at least 0.25 inches

Main Test Data

Trial No.	Start Time (24:HR)	Stop Time (24:HR)	Time Interval, Δt (min)	Initial Depth to Water, D_o (feet)	Final Depth to Water, D_f (feet)	Change in Water Level, ΔD (feet)	Raw Infiltration Rate (in/hr)	Design Infiltration Rate (in/hr)
1	5:42	5:52	10.0	6.61	7.57	0.96	4.6	1.5
2	5:53	6:03	10.0	6.71	7.71	1	5.0	1.7
3	6:05	6:15	10.0	6.77	7.68	0.91	4.6	1.5
4	6:17	6:27	10.0	6.63	7.59	0.96	4.6	1.5
5	6:28	6:38	10.0	6.62	7.54	0.92	4.4	1.5
6	6:40	6:50	10.0	6.68	7.53	0.85	4.1	1.4

Calculated Infiltration Rate (Including Factor of Safety of 3) **1.4**

Sketch:

Notes:

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Appendix C
Laboratory Test Results

APPENDIX C

Laboratory Test Results

The laboratory testing program was directed towards providing quantitative data relating to the relevant engineering properties of the soils. Samples considered representative of site conditions were tested in general accordance with American Society for Testing and Materials (ASTM) procedure and/or California Test Methods (CTM), where applicable. The following summary is a brief outline of the test type and a table summarizing the test results.

Moisture and Density Determination Tests: Moisture content (ASTM Test Method D2216) and dry density determinations (ASTM D2937) were performed on relatively undisturbed samples obtained from the test borings. The results of these tests are presented in the boring logs. Where applicable, only moisture content was determined from undisturbed or disturbed samples.

Grain Size Distribution/Fines Content: Representative samples were dried, weighed, and soaked in water until individual soil particles were separated (per ASTM D421) and then washed on a No. 200 sieve (ASTM D1140). Where applicable, the portion retained on the No. 200 sieve was dried and then sieved on a U.S. Standard brass sieve set in accordance with ASTM D6913 (sieve) or ASTM D422 (sieve and hydrometer).

Sample Location	Description	% Passing # 200 Sieve
HS-1 @ 5 ft	Sandy Clay	53
HS-3 @ 5 ft	Silty Clayey Sand	49
HS-4 @ 5 ft	Clayey Sand	44
HS-4 @ 10 ft	Sand	4
HS-4 @ 35 ft	Sand with Silt	5

Atterberg Limits: The liquid and plastic limits ("Atterberg Limits") were determined per ASTM D4318 for engineering classification of fine-grained material and presented in the table below. It should be noted that the USCS soil classification provided in the table below is only based the portion of the sample passing the No. 40 sieve. The plots are provided in this Appendix.

Sample Location	Liquid Limit (%)	Plastic Limit (%)	Plasticity Index (%)	USCS Soil Classification
HS-2 @ 5 ft	33	15	18	CL
HS-3 @ 5 ft	21	15	6	CL-ML
HS-4 @ 5 ft	28	14	14	CL
HS-4 @ 25 ft	69	31	38	CH

Laboratory Test Results

Direct Shear: A direct shear test was performed on select driven, relatively undisturbed sample. The samples were soaked for a minimum of 24 hours prior to testing. The samples were tested under various normal loads, a motor-driven, strain-controlled, direct-shear testing apparatus (ASTM D3080). The plot is presented in this Appendix.

Consolidation: Consolidation tests were performed per ASTM D2435. Samples (2.4 inches in diameter and 1 inch in height) were placed in a consolidometer and increasing loads were applied. The samples were allowed to consolidate under "double drainage" and total deformation for each loading step was recorded. The percent consolidation for each load step was recorded as the ratio of the amount of vertical compression to the original sample height. The consolidation pressure curves are presented in this Appendix.

Expansion Index: The expansion potential of selected representative samples was evaluated by the Expansion Index Test per ASTM D4829.

Sample Location	Expansion Index	Expansion Potential*
HS-1 @ 2-5 ft	42	Low
HS-4 @ 2-5 ft	51	Medium

* Per ASTM D4829

Laboratory Compaction: The maximum dry density and optimum moisture content of typical materials were determined in accordance with ASTM D1557. The results of these tests are presented in the table below.

Sample Location	Sample Description	Maximum Dry Density (pcf)	Optimum Moisture Content (%)
HS-4 @ 2-5 ft	Yellowish Brown Clayey Sand	125.0	10.0

Soluble Sulfates: The soluble sulfate contents of selected samples were determined by standard geochemical methods (CTM 417). The test results are presented in the table below.

Sample Location	Sulfate Content
HS-1 @ 2-5 ft	< 0.01%
HS-4 @ 2-5 ft	0.01%

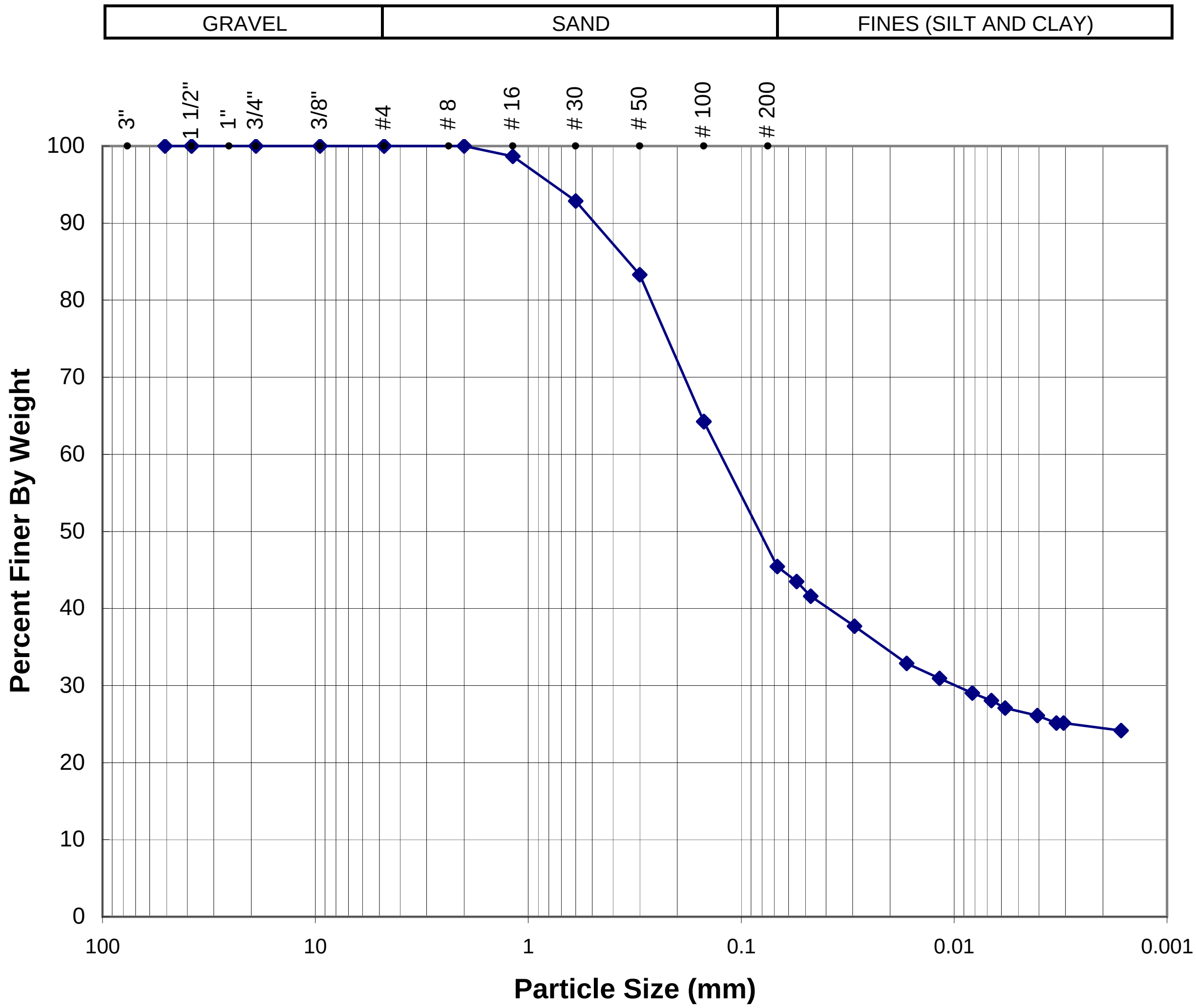
<u>Laboratory Test Results</u>

<u>Chloride Content</u>: Chloride content was tested per CTM 422. The results are presented below.

Sample Location	**Chloride Content, ppm**
HS-1 @ 2-5 ft	40

<u>Minimum Resistivity and pH Tests</u>: Minimum resistivity and pH tests were performed in general accordance with CTM 643 and standard geochemical methods. The results are presented in the table below.

Sample Location	**pH**	**Minimum Resistivity (ohms-cm)**
HS-1 @ 2-5 ft	8.5	2,335



Location:	Sample No.:	Depth (ft.)	Soil Type	Gravel (%)	Sand (%)	Fines (%)
HS-1	**SPT-1**	**5'**	**CL**	**0**	**47**	**53**

Sample Description: Sandy Clay

LGC Geotechnical, Inc.

PARTICLE SIZE ANALYSIS
(ASTM D 422)

Project Number: 13192-01
Date: Mar-14

1239 Victoria Costa Mesa

ATTERBERG LIMITS

ASTM D 4318

Project Name:	Costa Mesa	Tested By:	G. Bathala	Date:	03/14/14
Project No. :	13192-01	Input By:	J. Ward	Date:	03/19/14
Boring No.:	HS-2	Checked By:	J. Ward		
Sample No.:	R-2	Depth (ft.)	5.0		
Soil Identification:	Olive brown lean clay with sand (CL)s, caliche noted				

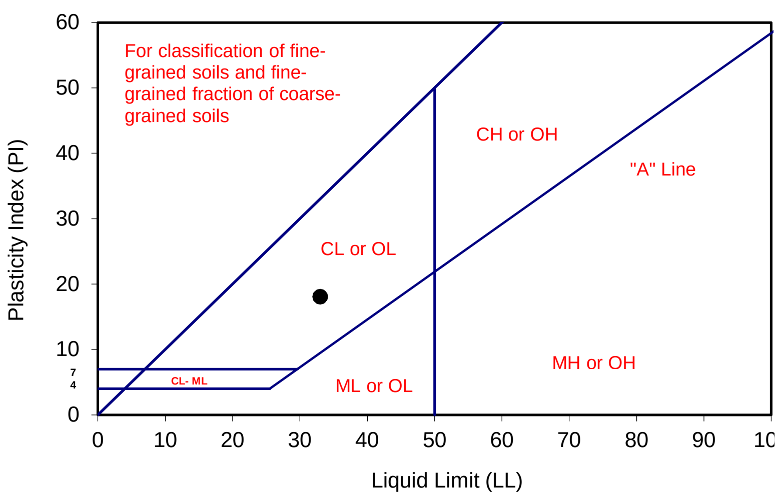
TEST	PLASTIC LIMIT		LIQUID LIMIT			
NO.	1	2	1	2	3	4
Number of Blows [N]			35	27	20	
Wet Wt. of Soil + Cont. (g)	26.23	24.30	28.42	28.17	31.13	
Dry Wt. of Soil + Cont. (g)	24.58	22.91	24.83	24.53	26.62	
Wt. of Container (g)	13.59	13.57	13.54	13.55	13.54	
Moisture Content (%) [Wn]	15.01	14.88	31.80	33.15	34.48	

Liquid Limit	**33**
Plastic Limit	**15**
Plasticity Index	**18**
Classification	**CL**

PI at "A" - Line = 0.73(LL-20) | 9.49

One - Point Liquid Limit Calculation

$LL = Wn(N/25)^{0.121}$



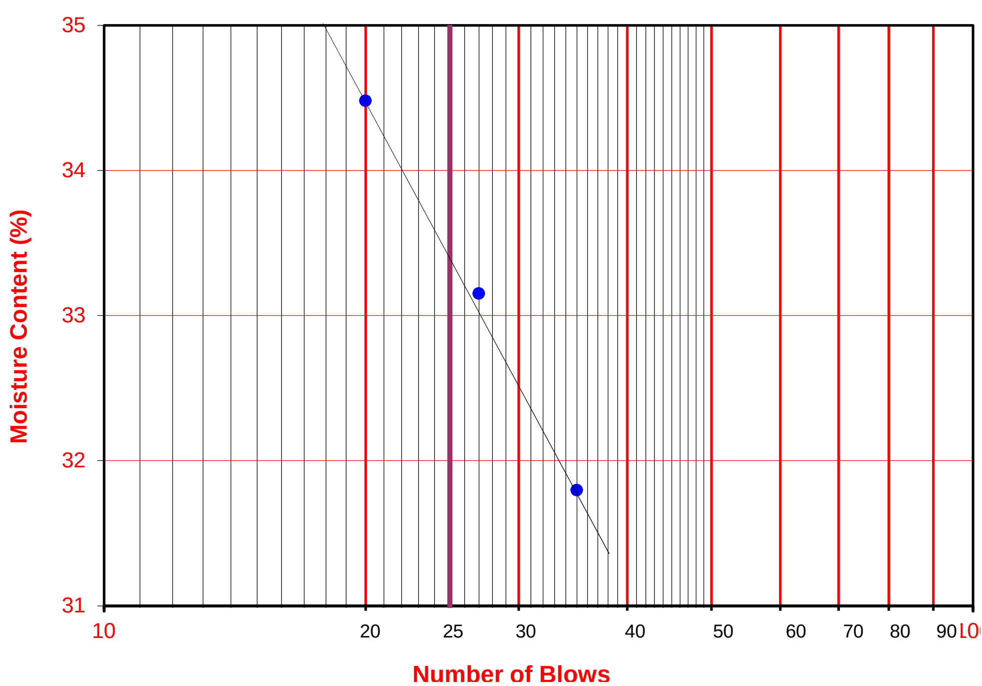
PROCEDURES USED

[ ] Wet Preparation
Multipoint - Wet

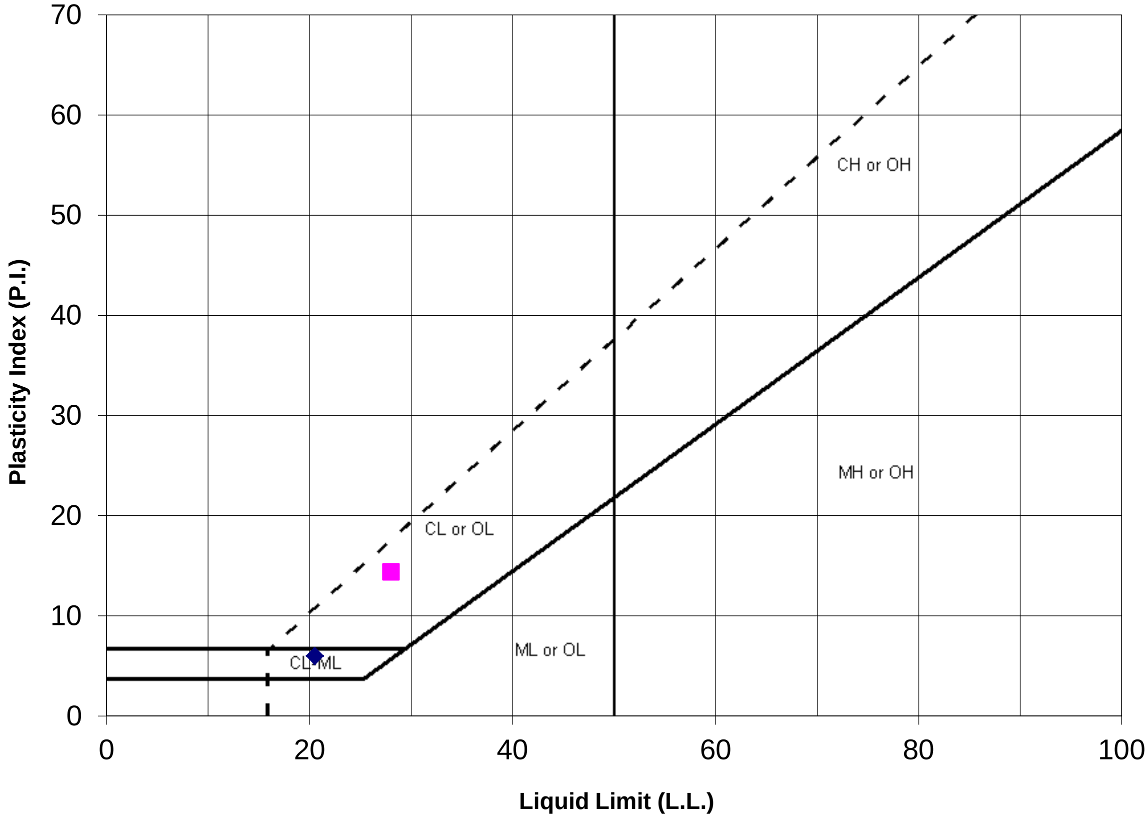
[X] Dry Preparation
Multipoint - Dry

[X] Procedure A
Multipoint Test

[ ] Procedure B
One-point Test



PLASTICITY CHART - CLASSIFICATION OF FINE-GRAINED SOILS



Symbol	Location.:	Sample No.:	Depth (ft)	Passing No. 200 Sieve (%)	Liquid Limit (%) LL	Plastic Limit (%) PL	Plasticity Index (%) PI	USCS
◆	**HS-3**	**SPT-1**	**5**	**49**	**21**	**15**	**6**	**CL-ML**
■	**HS-4**	**SPT-1**	**5**	**44**	**28**	**14**	**14**	**CL**

LGC Geotechnical, Inc.

ATTERBERG LIMITS
(ASTM D 4318)

Project Number: 13192-01
Date: Mar-14

1239 Victoria, Costa Mesa

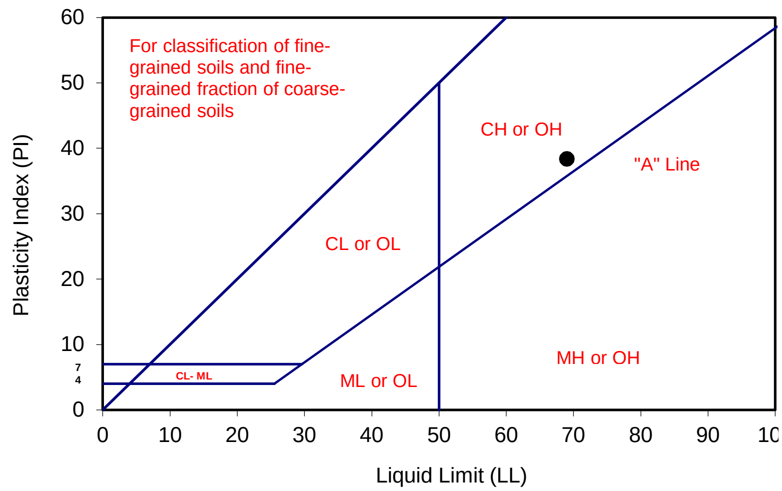
ATTERBERG LIMITS

ASTM D 4318

Project Name:	Costa Mesa	Tested By:	G. Bathala	Date:	03/14/14
Project No. :	13192-01	Input By:	J. Ward	Date:	03/19/14
Boring No.:	HS-4	Checked By:	J. Ward		
Sample No.:	R-4	Depth (ft.)	25.0		
Soil Identification:	Olive fat clay (CH)				

TEST	PLASTIC LIMIT		LIQUID LIMIT			
NO.	1	2	1	2	3	4
Number of Blows [N]			32	25	17	
Wet Wt. of Soil + Cont. (g)	25.64	26.19	28.12	27.32	26.51	
Dry Wt. of Soil + Cont. (g)	22.81	23.24	22.26	21.71	21.12	
Wt. of Container (g)	13.55	13.62	13.60	13.59	13.53	
Moisture Content (%) [Wn]	30.56	30.67	67.67	69.09	71.01	

Liquid Limit	**69**
Plastic Limit	**31**
Plasticity Index	**38**
Classification	**CH**



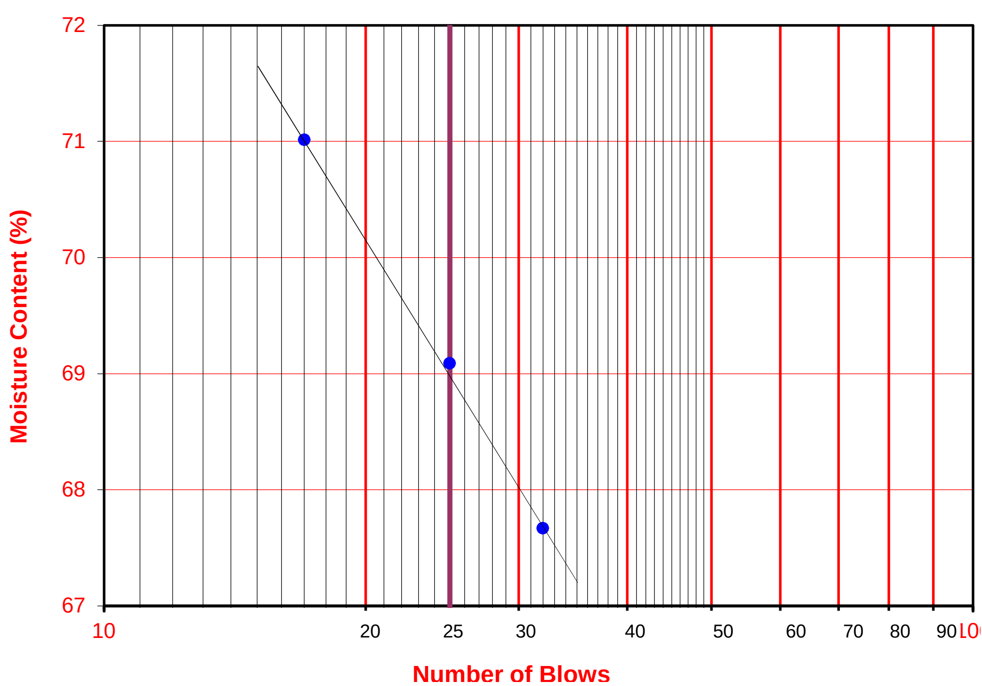
PI at "A" - Line = 0.73(LL-20) | 35.77

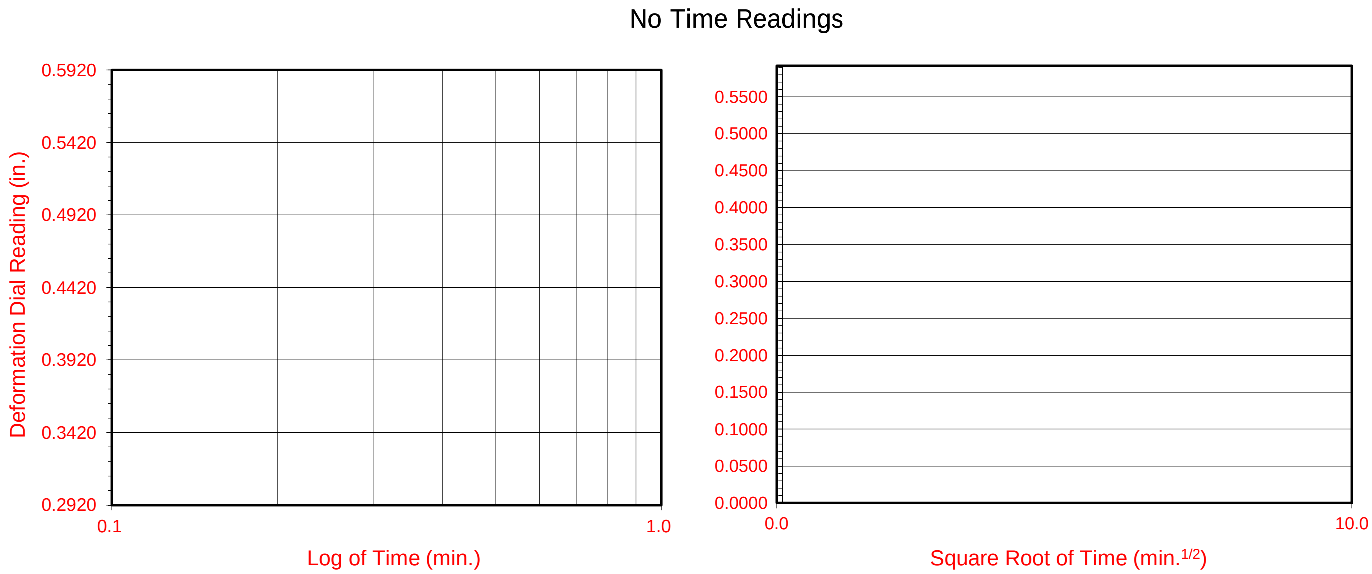
One - Point Liquid Limit Calculation

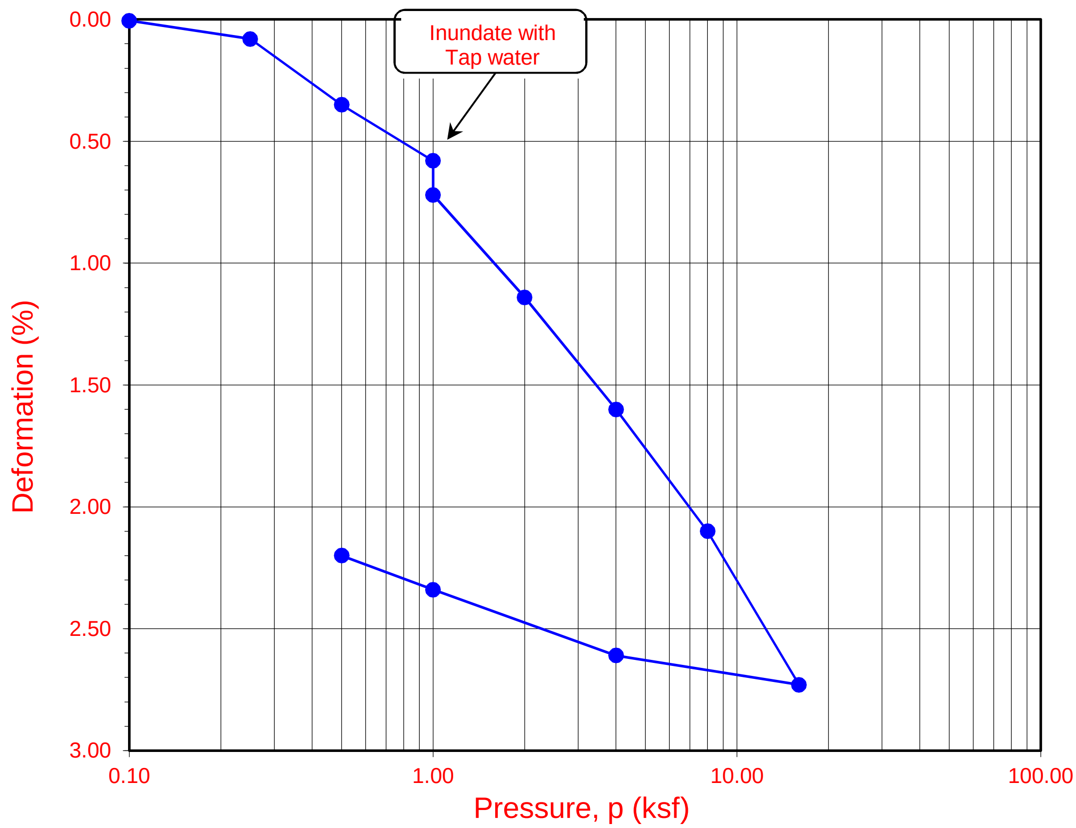
$LL = Wn(N/25)^{0.121}$

PROCEDURES USED

☐	Wet Preparation Multipoint - Wet
X	Dry Preparation Multipoint - Dry
X	Procedure A Multipoint Test
☐	Procedure B One-point Test







Boring No.	Sample No.	Depth (ft.)	Moisture Content (%)		Dry Density (pcf)		Void Ratio		Degree of Saturation (%)	
			Initial	Final	Initial	Final	Initial	Final	Initial	Final
HS-1	**R-2**	**10.0**	**4.4**	**18.2**	**104.5**	**105.7**	**0.613**	**0.577**	**20**	**83**

Soil Identification: Yellowish brown poorly-graded sand with silt (SP-SM)

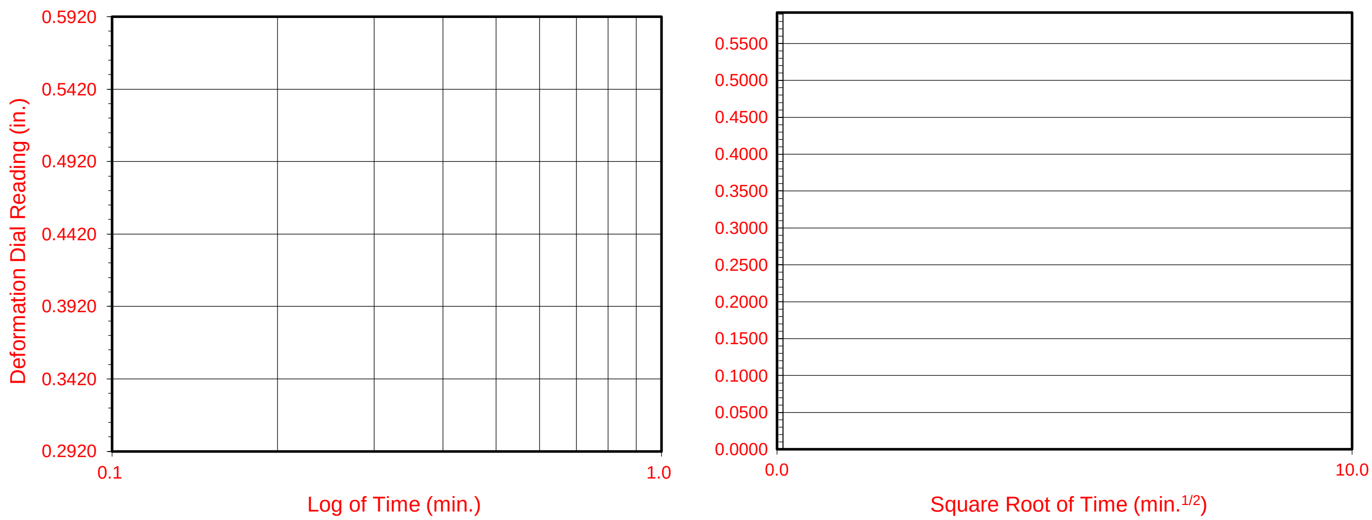
ONE-DIMENSIONAL CONSOLIDATION PROPERTIES of SOILS
ASTM D 2435

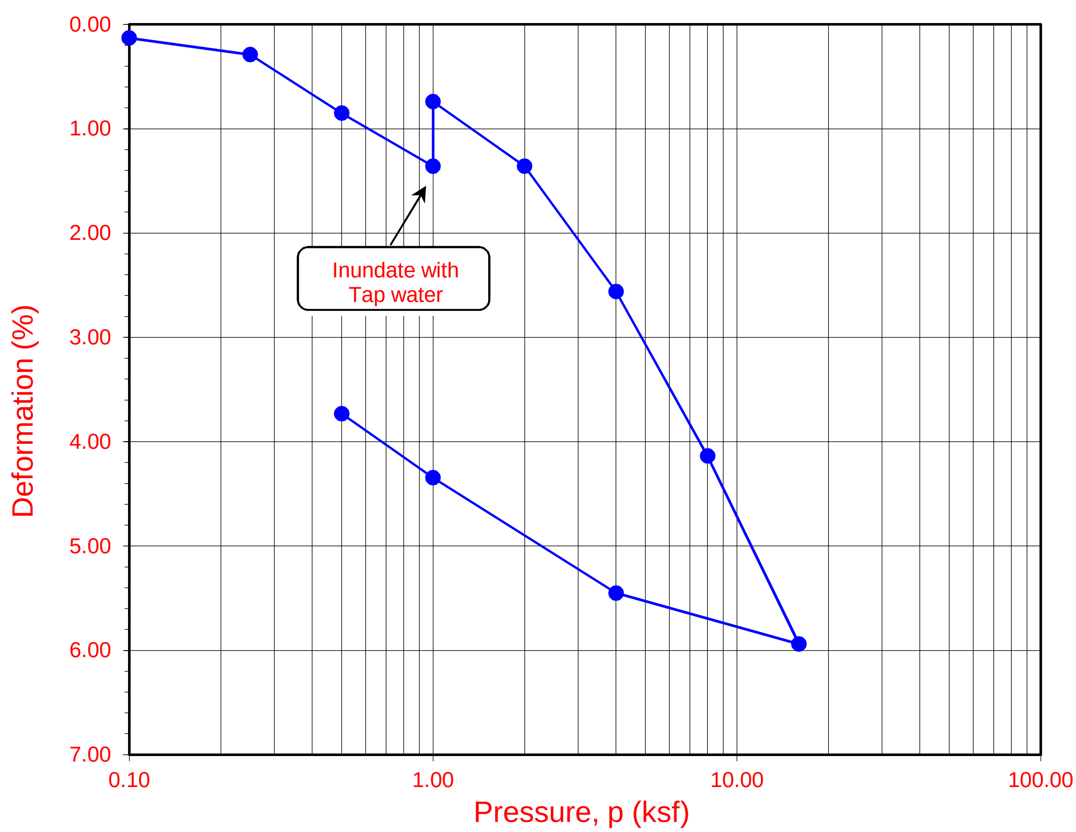
Project No.: 13192-01

Costa Mesa

03-14

No Time Readings





Boring No.	Sample No.	Depth (ft.)	Moisture Content (%)		Dry Density (pcf)		Void Ratio		Degree of Saturation (%)	
			Initial	Final	Initial	Final	Initial	Final	Initial	Final
HS-2	**R-2**	**5.0**	**10.6**	**14.2**	**117.8**	**121.3**	**0.430**	**0.377**	**66**	**98**

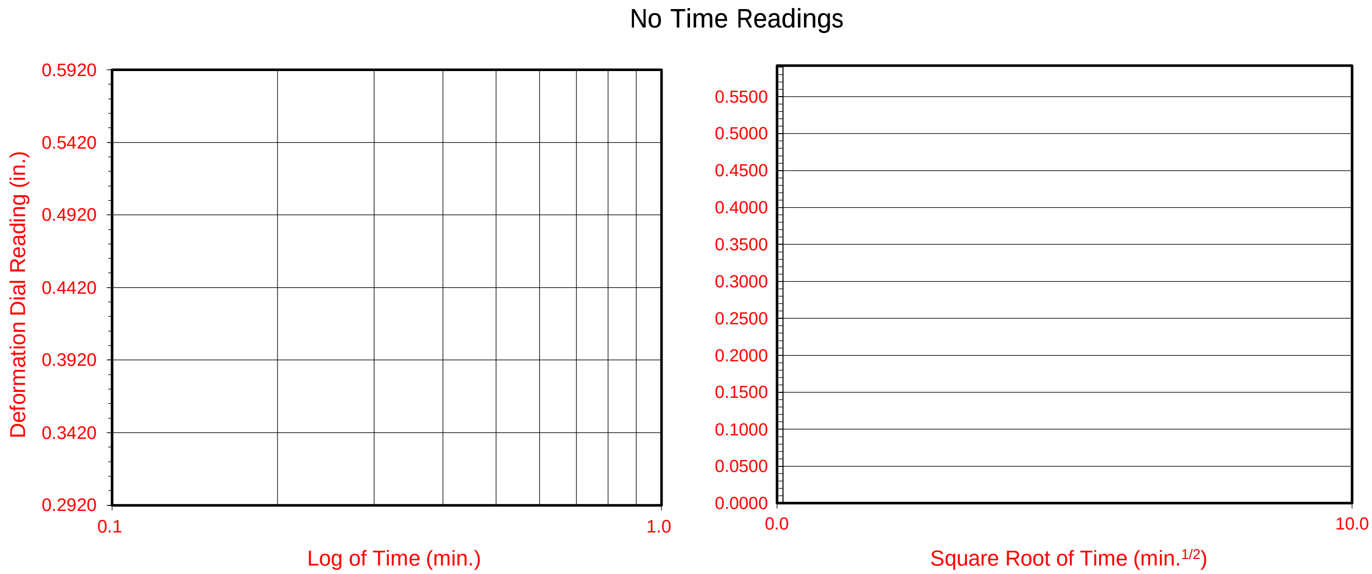
Soil Identification: Olive brown lean clay with sand (CL)s, caliche noted

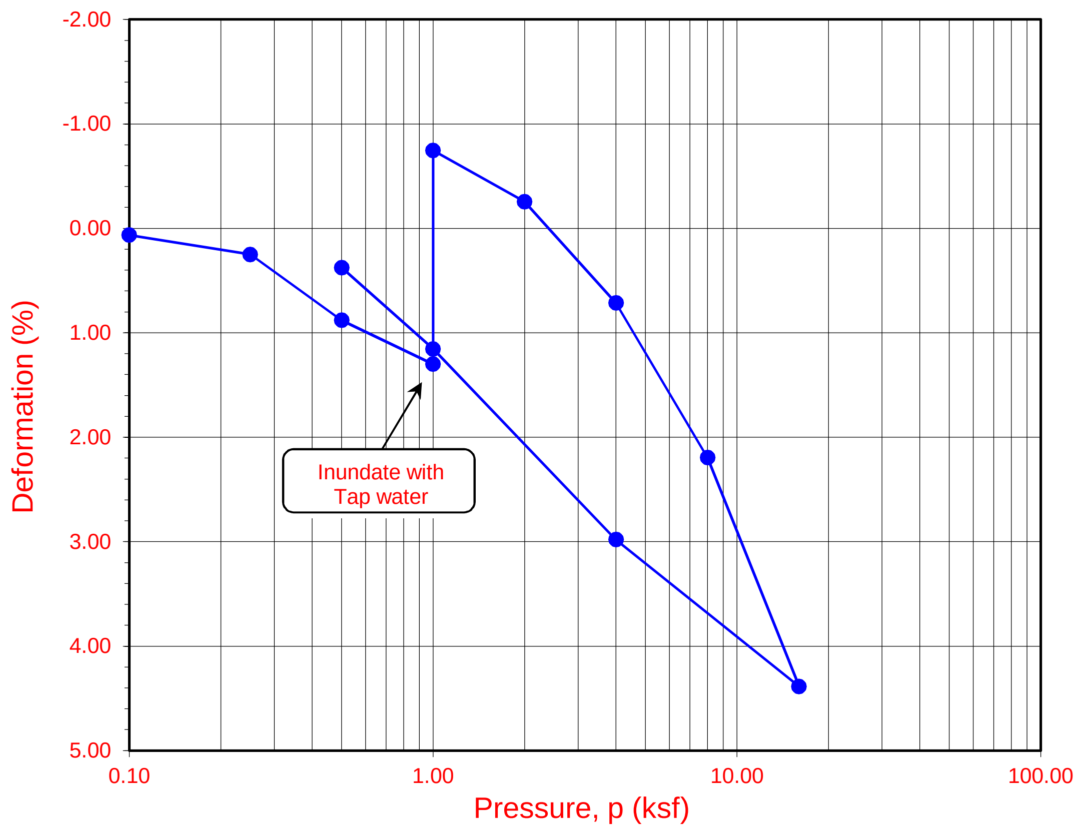
ONE-DIMENSIONAL CONSOLIDATION PROPERTIES of SOILS
ASTM D 2435

Project No.: 13192-01

Costa Mesa

03-14





Boring No.	Sample No.	Depth (ft.)	Moisture Content (%)		Dry Density (pcf)		Void Ratio		Degree of Saturation (%)	
			Initial	Final	Initial	Final	Initial	Final	Initial	Final
HS-4	**R-4**	**25.0**	**27.1**	**29.1**	**95.1**	**95.8**	**0.819**	**0.812**	**91**	**100**

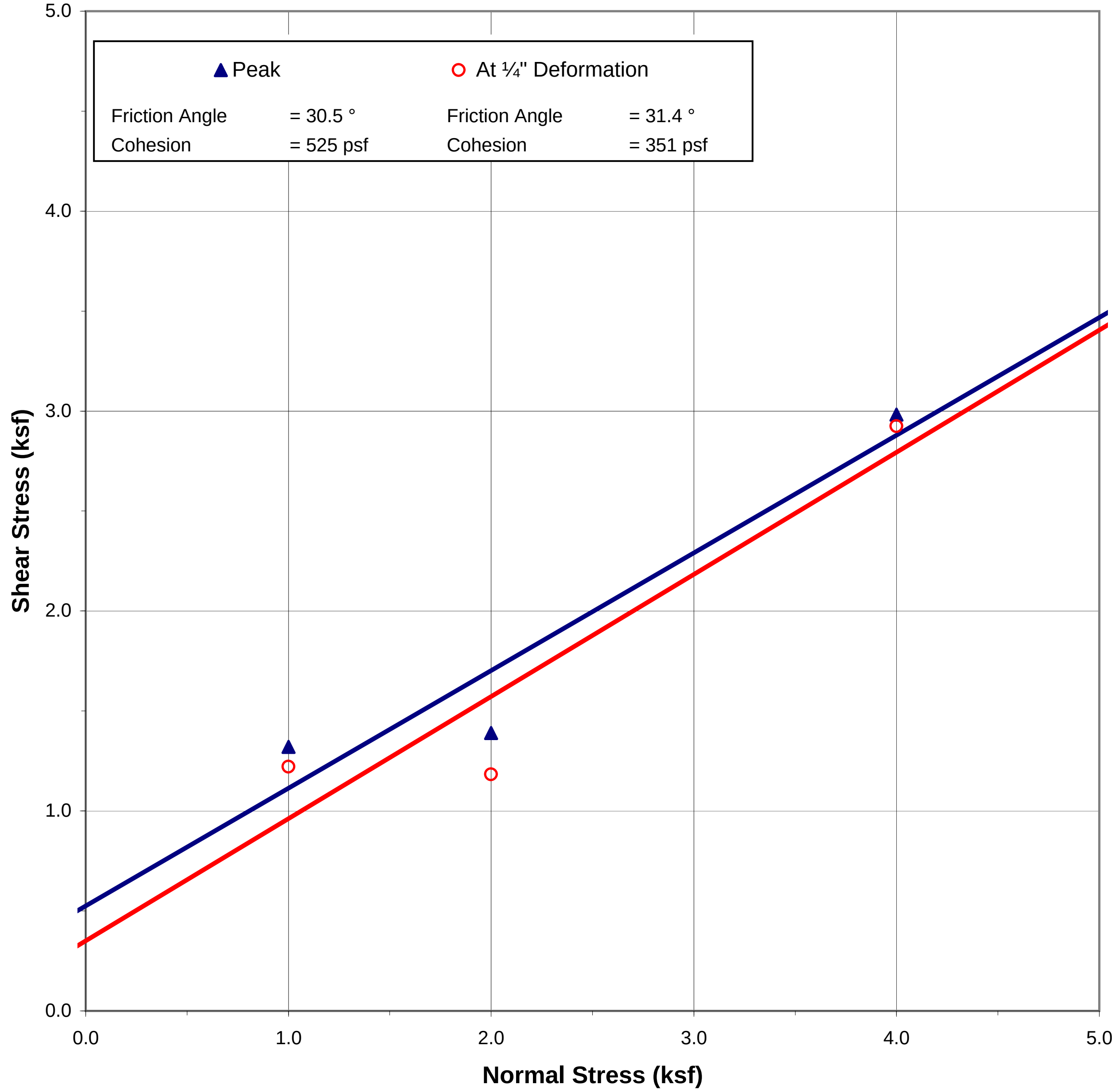
Soil Identification: Olive fat clay (CH)

ONE-DIMENSIONAL CONSOLIDATION PROPERTIES of SOILS
ASTM D 2435

Project No.: 13192-01

Costa Mesa

03-14



Location:	Sample No.:	Depth (ft)	Sample Type	Shear Rate (inch/min)	Dry Density (pcf)	Initial Moisture Content (%)	Final Moisture Content (%)
HS-5	**R-2**	**7.5**	**Ring**	**0.0005**	**112.3**	**14.7**	**15.3**

Sample Description: Silty Sand

LGC Geotechnical, Inc.

DIRECT SHEAR PLOT

Project Number: 13192-01
Date: Mar-14

1239 Victoria, Costa Mesa

Location	Sample No.	Depth (ft)	Molding Moisture Content (%)	Initial Dry Density (pcf)	Final Moisture Content (%)	Expansion Index	Expansion Classification[1]
HS-1	B-1	2-5	8.6	113.8	17.3	42	Low
HS-4	B-1	2-5	10.0	110.2	19.0	51	Medium

[1] Per ASTM D4829

LGC Geotechnical, Inc.

EXPANSION INDEX
(ASTM D 4829)

Project Number: 13192-01
Date: Mar-14

Victoria Costa Mesa

MODIFIED PROCTOR COMPACTION TEST

ASTM D 1557

Project Name:	Costa Mesa	Tested By:	O. Figueroa	Date:	03/13/14
Project No.:	13192-01	Input By:	J. Ward	Date:	03/19/14
Boring No.:	HS-4	Depth (ft.):	2-5		
Sample No.:	B-1				
Soil Identification:	Yellowish brown clayey sand (SC)				

Preparation Method: [X] Moist [ ] Dry — [X] Mechanical Ram [ ] Manual Ram

Mold Volume (ft³) 0.03300 — *Ram Weight = 10 lb.; Drop = 18 in.*

TEST NO.	1	2	3	4	5	6
Wt. Compacted Soil + Mold (g)	3798.0	3909.0	3900.0			
Weight of Mold (g)	1859.0	1859.0	1859.0			
Net Weight of Soil (g)	1939.0	2050.0	2041.0			
Wet Weight of Soil + Cont. (g)	478.30	474.10	477.20			
Dry Weight of Soil + Cont. (g)	449.70	436.50	428.20			
Weight of Container (g)	54.20	51.00	52.70			
Moisture Content (%)	7.23	9.75	13.05			
Wet Density (pcf)	129.5	137.0	136.4			
Dry Density (pcf)	120.8	124.8	120.6			

Maximum Dry Density (pcf) 125.0 **Optimum Moisture Content (%)** 10.0

PROCEDURE USED

[X] **Procedure A**
Soil Passing No. 4 (4.75 mm) Sieve
Mold : 4 in. (101.6 mm) diameter
Layers : 5 (Five)
Blows per layer : 25 (twenty-five)
May be used if +#4 is 20% or less

[ ] **Procedure B**
Soil Passing 3/8 in. (9.5 mm) Sieve
Mold : 4 in. (101.6 mm) diameter
Layers : 5 (Five)
Blows per layer : 25 (twenty-five)
Use if +#4 is >20% and +3/8 in. is 20% or less

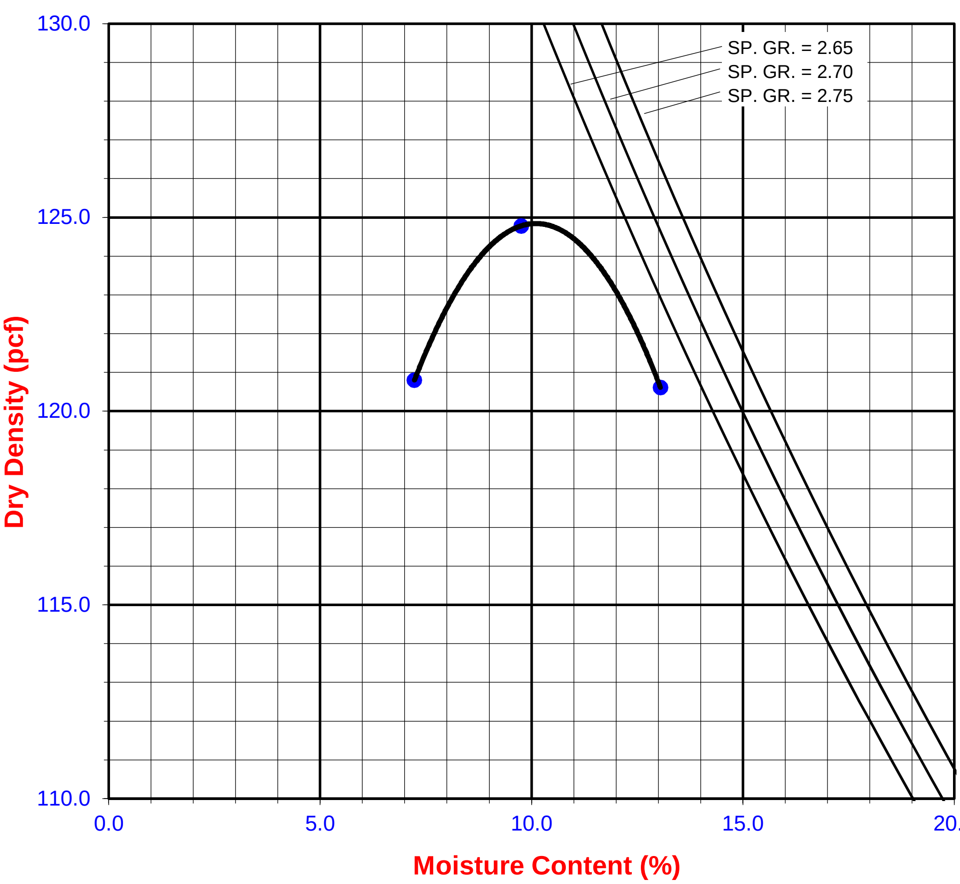
[ ] **Procedure C**
Soil Passing 3/4 in. (19.0 mm) Sieve
Mold : 6 in. (152.4 mm) diameter
Layers : 5 (Five)
Blows per layer : 56 (fifty-six)
Use if +3/8 in. is >20% and +¾ in. is <30%

Particle-Size Distribution:

GR:SA:FI

Atterberg Limits:

LL,PL,PI



SOIL RESISTIVITY TEST

DOT CA TEST 532 / 643

Project Name:	Costa Mesa	Tested By :	G. Berdy	Date: 03/13/14
Project No. :	13192-01	Data Input By:	J. Ward	Date: 03/19/14
Boring No.:	HS-1	Depth (ft.) :	2-5
Sample No. :	B-1		

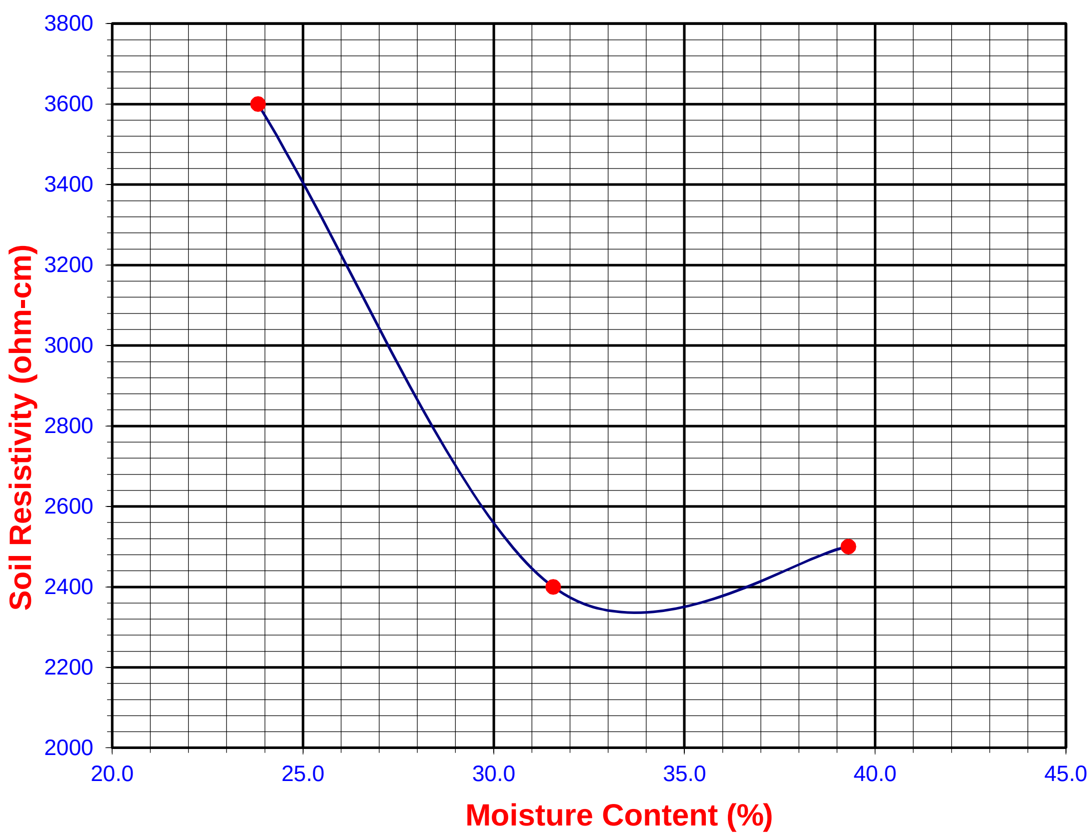
Soil Identification:* Yellowish brown SC

*California Test 643 requires soil specimens to consist only of portions of samples passing through the No. 8 US Standard Sieve before resistivity testing. Therefore, this test method may not be representative for coarser materials.

Specimen No.	Water Added (ml) (Wa)	Adjusted Moisture Content (MC)	Resistance Reading (ohm)	Soil Resistivity (ohm-cm)
1	30	23.82	3600	3600
2	40	31.56	2400	2400
3	50	39.30	2500	2500
4				
5				

Moisture Content (%) (MCi)	0.61
Wet Wt. of Soil + Cont. (g)	223.60
Dry Wt. of Soil + Cont. (g)	222.60
Wt. of Container (g)	57.60
Container No.	
Initial Soil Wt. (g) (Wt)	130.00
Box Constant	1.000
MC =(((1+Mci/100)x(Wa/Wt+1))-1)x100	

Min. Resistivity (ohm-cm)	Moisture Content (%)	Sulfate Content (ppm)	Chloride Content (ppm)	Soil pH	
				pH	Temp. (°C)
DOT CA Test 532 / 643		DOT CA Test 417 Part II	DOT CA Test 422	DOT CA Test 532 / 643	
2335	**33.7**	**91**	**40**	**8.45**	**20.8**



TESTS for SULFATE CONTENT
CHLORIDE CONTENT and pH of SOILS

Project Name: Costa Mesa   Tested By : G. Berdy   Date: 03/12/14

Project No. : 13192-01   Data Input By: J. Ward   Date: 03/19/14

Boring No.	HS-4			
Sample No.	B-1			
Sample Depth (ft)	2-5			
Soil Identification:	Yellowish brown SC			
Wet Weight of Soil + Container (g)	196.90			
Dry Weight of Soil + Container (g)	195.80			
Weight of Container (g)	53.10			
Moisture Content (%)	0.77			
Weight of Soaked Soil (g)	100.60			

SULFATE CONTENT, DOT California Test 417, Part II

Beaker No.	17			
Crucible No.	3			
Furnace Temperature (°C)	875			
Time In / Time Out	9:45/10:30			
Duration of Combustion (min)	45			
Wt. of Crucible + Residue (g)	18.5342			
Wt. of Crucible (g)	18.5318			
Wt. of Residue (g) (A)	0.0024			
PPM of Sulfate (A) x 41150	98.76			
PPM of Sulfate, Dry Weight Basis	**100**			

CHLORIDE CONTENT, DOT California Test 422

ml of Extract For Titration (B)				
ml of AgNO3 Soln. Used in Titration (C)				
PPM of Chloride (C -0.2) * 100 * 30 / B				
PPM of Chloride, Dry Wt. Basis	**N/A**			

pH TEST, DOT California Test 532/643

pH Value	**N/A**			
Temperature °C				

Appendix D
General Earthwork & Grading Specifications

General Earthwork and Grading Specifications for Rough Grading

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 The Geotechnical Consultant of Record

Prior to commencement of work, the owner shall employ a qualified Geotechnical Consultant of Record (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.

The Geotechnical Consultant shall observe the moisture-conditioning and processing of the subgrade and fill materials and perform relative compaction testing of fill to confirm that the attained level of compaction is being accomplished as specified. 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 project 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 “equipment” 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 personnel will be available for observation and testing. 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. It is the contractor's sole responsibility to provide proper fill compaction.

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). 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. The contractor is responsible for all hazardous waste relating to his work. The Geotechnical Consultant does not have expertise in this area. If hazardous waste is a concern, then the Client should acquire the services of a qualified environmental assessor.

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 over-excavated as specified in the following section. Scarification shall continue until soils are broken down and free of oversize material and the working surface is reasonably uniform, flat, and free of uneven features that would inhibit uniform compaction.

2.3 Over-excavation

In addition to removals and over-excavations 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 over-excavated 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 over-excavated 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 8 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 the geotechnical consultant. 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).

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). 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.

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

7.2 All 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 of 90 percent of maximum from 1 foot above the top of the conduit to the surface.

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

7.4 The 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.

W.O. 3593-2

December 9, 2014 Project No. 13192-01

Mr. Eric Nelson
Trumark Homes
450 Newport Center Drive, Suite 300
Newport Beach, CA 92660

Subject: ***Geotechnical Response to City of Costa Mesa Review Comments dated November 19, 2014, Proposed Residential Development at 1239 Victoria Street, Tentative Tract No. 17779, City of Costa Mesa, California***

Reference: Preliminary Geotechnical Evaluation for the Proposed Residential Development at 1239 Victoria Street, City of Costa Mesa, California, Project No. 13192-01, dated April 7, 2014.

Introduction

In accordance with your request, LGC Geotechnical, Inc. is responding to the city of Costa Mesa review comments dated November 19, 2014 for the proposed development located at 1239 Victoria Street, Tentative Tract No. 17779, Costa Mesa, California.

This response-report should be considered as part of the project design documents in conjunction with our previous geotechnical report (LGC, 2014). In the case of conflict, the recommendations contained herein should supersede those provided in our previous report. The remaining recommendations provided in our previous geotechnical report (LGC, 2014) remain valid and applicable.

Geotechnical Review Comments dated November 19, 2014

For your convenience, the applicable review comments have been repeated below along with our responses. A copy of the review sheet is provided in Appendix A.

Comment No. 1

"Please explain the large difference between the Pre-Test Water Level drop rate (1.96'/30 min = 0.65'/10 min) and Main Test Data (ranging from 0.85 to 1 ft/10 min.)."

Response to Comment No. 1

The drop between the pre-test water level and main test data is a factor of approximately 1.5. Based on our experience regarding infiltration test data, a factor of approximately 1.5 is not unreasonable. It should be noted that the provided infiltration rate of 1.4 inches per hour was the lowest test value and included a geotechnical factor of safety of 3.

Comment No. 2

"There are no bearing capacity, earth pressures, and settlement calculations included in the appendices."

Response to Comment No. 2

Acknowledged. The values provided for bearing capacity, earth pressures and settlement are supported by the provided laboratory test data and the provided values are within industry standards.

Comment No. 3

"What are the preliminary recommendations for infiltration zones setback from foundations?"

Response to Comment No. 3

Foundations should be set back a minimum of 10 feet from the infiltration facility and the bottom of footings should be a minimum of 10 feet from the expected zone of saturation. Once designed, the infiltration system will be reviewed from a geotechnical perspective.

Closure

Our services were performed using the degree of care and skill ordinarily exercised, under similar circumstances, by reputable engineers and geologists practicing in this or similar localities. No other warranty, expressed or implied, is made as to the conclusions and professional advice included in this report.

Should you have any questions regarding this report, please do not hesitate to contact our office. We appreciate this opportunity to be of service.

Respectfully,

LGC Geotechnical, Inc.

REGISTERED PROFESSIONAL ENGINEER BRAD ZELLMER NO. GE 2618 EXP. 12-31-14 GEOTECHNICAL STATE OF CALIFORNIA

Brad Zellmer, GE 2618
Project Engineer

Dennis Boratynec, GE 2770
Vice President

REGISTERED PROFESSIONAL ENGINEER DENNIS BORATYNEC No. 2770 Exp. 12/31/14 GEOTECHNICAL STATE OF CALIFORNIA

BTZ/DJB/kmb

Attachment: Appendix A – City of Costa Mesa Review Sheet Dated November 19, 2014

Distribution: (3) Addressee (wet-signed copies for agency submittal)
(1) URS Corporation (via e-mail)
Attention: Mr. Dan Stoica
(1) Hunsaker & Associates (via e-mail)
Attention: Mr. Dave Frattone

Appendix A
City of Costa Mesa Review Sheet

Technical Report: Preliminary Geotechnical Evaluation for the Proposed Residential Development at 1239 Victoria Street, City of Costa Mesa, California; report by LGC Geotechnical, Inc., dated April 28, 2014
Reviewer: Dan Stoica, Geotechnical Engineer, URS Corp.; review date Nov. 19, 2014

Comment Number	Location in Document	Comments
General Comments		
1.	Sect 1.4 and App B-Infiltration Test Data	Please explain the large difference between the Pre-Test Water Level drop rate (1.96'/30min = 0.65'/10min) and Main Test Data (ranging from 0.85 to 1 ft/10min.).
2.	Sect 4.3	There are no bearing capacity, earth pressures, and settlement calculations included in the appendices.
3.	Sect 4.6	What are the preliminary recommendations for infiltration zones setback from foundations?
4.		
Other Comments		
1.		
2.		
3.		
4.		
5.		
6.		
7.		
8.		
9.		

Technical Report: Preliminary Geotechnical Evaluation for the Proposed Residential Development at 1239 Victoria Street, City of Costa Mesa, California; report by LGC Geotechnical, Inc., dated April 28, 2014

Reviewer: Dan Stoica, Geotechnical Engineer, URS Corp.; review date Nov. 19, 2014

Comment Number	Location in Document	Comments
10.		