advanced geotechnical services, inc.

August 7, 2018
Client Number 4844
Report Number 10139

Channel Islands Beach Community Services District
353 Santa Monica Drive
Oxnard, CA 93035

Feasibility Level Geotechnical Engineering Study
Future Single-Family Residence
112 Las Palmas Street
Oxnard, California

In accordance with our proposal and your authorization, Advanced Geotechnical Services, Inc., (AGS) has prepared this *Feasibility Level Geotechnical Engineering Study* for the construction of a future single-family residence at the subject site. This report presents the results of our data research, subsurface exploration, laboratory testing, and our professional opinions regarding the geotechnical engineering factors that may affect the future development.

Based on the results of our study, it is our opinion that the site is suitable for construction of the future single-family residence and associated improvements, provided recommendations of this report are properly incorporated in the design and implemented during construction.

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

Respectfully submitted,
Advanced Geotechnical Services, Inc.

Kenneth J. Palos
President

Scott Moore, GE
Principal Engineer

REGISTERED PROFESSIONAL ENGINEER
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STATE OF CALIFORNIA

Enclosure: *Report No. 10139*

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advanced geotechnical services, inc.

FEASIBILITY LEVEL GEOTECHNICAL ENGINEERING STUDY

Future Single-Family Residence
112 Las Palmas Street
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1. INTRODUCTION

1.1 General Remarks

This *Feasibility Level Geotechnical Engineering Study* has been prepared for the construction of a future single-family residence at the subject site. The purposes of this study are to (1) identify on-site soil conditions, (2) evaluate potential seismic hazards at the site, and (3) provide feasibility level geotechnical recommendations to be utilized in the design and construction of the future single-family residence, and other associated improvements. This report presents the findings of our data review, subsurface exploration, laboratory testing, engineering analyses and evaluations, and our conclusions and recommendations.

Appendices are attached following the main report. Appendix A includes an explanation of the field exploration, and boring logs; Appendix B includes the laboratory test results; Appendix C includes the results of the seismicity study; Appendix D includes the results of the site liquefaction analysis; Appendix E includes the references used in this study; Appendix F includes the documentation regarding the previously existing water well abandonment, and Appendix G includes the Figures referenced in this report.

1.2 Site Description and Future Development

The subject site is located at 112 Las Palmas Street, in the Oxnard area of Ventura County, California. A *Site Location Map* is provided as Figure 1, and an *Existing Site Plan* is provided as Figure 2, both based on images obtained from the Google Earth web app (2018). The subject site consists of a relatively flat, rectangular shaped parcel, elevated up to a few feet above the adjacent Las Palmas Street level.

At the time of our field exploration program, the subject site was vacant, with scattered vegetation consisting of ice plant and weeds. The site is bounded by Las Palmas Street to the north, and existing residential properties to the east, south and west.

The future development will consist of a new single-family residence, and the typical associated site improvements, including site flatwork such as driveways and walkways. Plans of the proposed future residence are not yet available, however it is likely that the future residence will have the same front, rear and side yard setbacks as the neighboring residences, and will be constructed at or near the current existing grade in the central portion of the property. It is anticipated that the future single-family residence will be a typical wood-framed structure, with maximum loads not expected to exceed approximately 50 kips for columns, and 1 to 2 kips per foot for walls.

Plans of the future development were not available as of the date of this report, however site grading is expected to consist of removal and recompaction of the upper site soils, and the existing artificial fill associated with the abandonment of the previously existing water well, for support of the future structure and other improvements, and backfill of new utilities. Only minor alterations to the site topography of less than approximately 1 to 2 feet are anticipated.

1.3 Scope of Services

This feasibility level geotechnical engineering study included:

a. Site observation and review of geotechnical and geologic data of the general study area. A *Site Location Map* is provided as Figure 1, and an *Existing Site Plan* is provided as Figure 2, both based on images from the Google Earth web app (2018).

b. Drilling, sampling, and logging of two borings to depths between approximately 16.5 and 51.5 feet below the existing ground surface for soils evaluation. The exploratory borings were located in the field using a tape measure and approximate reference points. Thus, the actual locations of the exploratory borings may deviate slightly from the locations shown on the attached Figure 2. The Boring Logs are included in Appendix A, along with a general description of the field operations.

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

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

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

f. Development of feasibility level geotechnical recommendations for site preparation and grading, and geotechnical design criteria for foundations, floor slabs, site flatwork, underground utility trenches, temporary excavations, and drainage.

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

The scope of this feasibility level geotechnical study did *not* include environmental issues.

2. GEOLOGIC SETTING

2.1 Geology

Geologic conditions beneath the subject property have been interpreted and characterized based upon our review of published and unpublished references, and our subsurface exploration. Our interpretations involve projections of data and assume that geologic conditions are reasonably constant between points of exposure. Work should continue under the review of the Geotechnical Engineer to ensure that geologic conditions different from those described below are recognized and evaluated as soon as possible. Certain subsurface conditions such as groundwater levels and the consistency of near-surface soils will vary with the seasons. The subject site is located within the Oxnard USGS 7.5-minute quadrangle.

2.2 Faulting

Southern California is a tectonically active region subject to hazards associated with earthquakes and faulting. Faults are classified as either active, potentially active, or inactive. Active faults are defined by the State of California as faults that have exhibited surface displacement within the last 11,000 years. Potentially active faults are defined by the State of California as those with a history of movement between 11,000 and 1.6 million years ago.

Alquist-Priolo Earthquake Fault Zones are zones that have been established by the State that contain active faults, and projects that are located within these zones require that a fault investigation be performed to determine if active faulting affects the site. Other undiscovered active faults without surface expression, called blind faults, are also capable of generating earthquakes, and may be present beneath the subject site. The site is *not* located in an Alquist-Priolo Earthquake Fault Zone, and therefore a detailed subsurface fault investigation is not required.

3. EARTH MATERIALS AND SUBSURFACE CONDITIONS

3.1 Earth Materials

The earth materials encountered during exploration from the ground surface to a depth of approximately 40 feet consist of beach sand (Qs), likely interlayered with sandy alluvial deposits at depth. At a depth of 40 feet, a stiff sandy silt with minor clay was encountered, which continued to the total depth explored, 51.5 feet. The beach sand is relatively dry and loose at the ground surface, and becomes moderately dense and slightly moist by a depth of approximately 1 to 2 feet, and generally increasingly moist with depth. The sand becomes wet at a depth of 8.5 to 9 feet, where groundwater was encountered. The sand is tan to light gray, and ranges from moderately dense to dense below a depth of 1 to 2 feet. The sandy silt at a depth of 40 feet is light gray, fine grained, very moist and stiff. More detailed descriptions of the earth materials encountered can be found on the enclosed boring logs.

3.2 Existing Abandoned Well

There was a previously existing water well on the subject site that was abandoned in 2003. Based on the well abandonment documents provided to us by our client (included in Appendix F), there is no information available regarding the exact location of the abandoned well, or the exact depth and lateral extent of the excavation made during the abandonment of the well. The location of the abandoned well can only be approximately inferred from the photographs included with the well abandonment documents. The 'Conditions for Well Destruction' contained on the *Well Permit Application*, dated 1/28/03, indicates that the well casing was to be filled with cement from the bottom of the well up to a depth of 5 feet below finish grade, and the well casing above that was to be removed. Therefore, the minimum depth of uncertified fill placed back into the well abandonment excavation is likely 5 feet ('uncertified fill' being any man-made fill that was not inspected, tested, and certified by a geotechnical engineering company). The lateral extent of the uncertified fill is unknown however.

All of the uncertified fill resulting from the well abandonment will have to be removed, stockpiled onsite, and then properly compacted back into the resulting excavation, under the observation and testing of a geotechnical engineering company. A qualified representative of a geotechnical engineering company would be onsite during the excavation process to help determine the depth and lateral extent of the existing uncertified fill, which is mainly a visual determination.

3.3 Soil Parameters

3.3.1 Maximum Density

A compaction curve was developed in this study for a sample of the beach sand material between the depths of approximately 0 and 5 feet. The maximum dry density for this material was 106.5 pcf, at an optimum moisture content of 13.5%. This value may be utilized as a guideline during the required removal and recompaction of the upper onsite soils during grading.

3.3.2 Expansion Category

The potential of the soil to swell or expand increases with an increase in soil density, a decrease in initial moisture content (low percent saturation), an increase in clay content, and an increase in the activity of the clay content. Expansive soils change in volume (shrink or swell) due to changes in the soil moisture content. In addition to swell potential of the soil, the amount of volume change depends on (1) the availability of water, (2) the restraining pressure, and (3) time. The risk of soil expansion increases with an increase in expansion index. These test results show that the upper site soils are non-expansive (therefore in the *very low* expansion range, with an expansion index of 0).

3.3.3 Compressibility

A consolidation test was performed on a representative undisturbed sample of the onsite soils from a depth of 5 feet below the existing ground surface. The consolidation test results showed only a slight tendency to hydroconsolidate, and a relatively low potential of compressibility. Consolidation test results are included in Appendix B.

3.3.4 Shear Strength

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

3.3.5 Corrosivity

The risk of corrosion of construction materials relates to the potential for soil-induced chemical reaction. The rate of deterioration depends on soil resistivity, texture, acidity, and chemical concentration. To provide a basis for a preliminary corrosion evaluation, one sample of the near surface soils on the site was analyzed. The results of these tests are summarized in the following table, and the test results data sheet from American Analytics is attached in Appendix B. Sulfate and chloride concentrations are expressed in mg/kg on a dry weight basis.

Boring	Depth, Ft	Description	pH	Chloride, mg/kg	Sulfate, mg/kg	Specific Conductance, umhos/cm
B-1	0-5	SAND	7.5	22	16	270

The sulfate content is below 1000 mg/kg (*S0* exposure category based on ACI 318), and therefore special considerations for concrete which will be in contact with the onsite soils are not required.

3.4 Groundwater

At the time of our field exploration, groundwater was encountered at a depth of approximately 8.5 to 9 feet below the existing ground surface. Based on the *Depth to Historically High Groundwater* Map (CGS, 2002), Figure 3, the historically highest groundwater level in the site vicinity was approximately 5 feet below the existing ground surface. Groundwater elevations are dependent on seasonal precipitation, irrigation, land use, climatic conditions, among other factors, and as a result fluctuate. Therefore, water levels at the time of construction and during the life of the structure may vary from the observations or conditions at the time of our field exploration.

4. SEISMICITY

4.1 Seismic Design Criteria

The 2016 CBC specifies the use of the *Mapped Maximum Considered Geometric Mean (MCE_G) Peak Ground Acceleration, PGA*, which is adjusted for site class effects to obtain PGA_M. For the subject site, PGA and PGA_M are both 0.768g, as indicated on page 5 of the *USGS Design Maps Detailed Report* included as an attachment in Appendix C of this report.

The 2016 California Building Code (CBC) is utilized in the seismic design of structures, and is based on the *Maximum Considered Earthquake Ground Motion*. The earth materials underlying the site are classified based on parameters such as shear wave velocity, standard penetration test resistance, undrained shear strength, and earth material type. The maximum considered earthquake spectral response accelerations are then adjusted for general type of earth materials underlying the site, or *Site Class*, which would be D for the subject site. The remaining seismic parameters used in structural analyses are computed by the Structural Engineer from the values shown below.

The following seismic design coefficients and parameters for the project site have been determined utilizing the U.S. Seismic Design Maps web program developed by the United States Geological Survey (2014). The program incorporates seismic provisions set forth in the 2016 California Building Code (CBC) and 2015 International Building Code (IBC) procedures. Printout data generated by the USGS program is included in Appendix C of this report for reference.

Site Class	Spectral Accelerations, 0.2-Second Period, S_S	Spectral Accelerations, 1-Second Period, S_1	Site Coefficient, F_a	Site Coefficient, F_v	Adjusted Spectral Accelerations, 0.2-Second Period, S_{MS}	Adjusted Spectral Accelerations, 1-Second Period, S_{M1}	Adjusted Spectral Accelerations, 0.2-Second Period, S_{DS}	Adjusted Spectral Accelerations, 1-Second Period, S_{D1}
D	2.053	0.727	1.0	1.5	2.053	1.090	1.369	0.727

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

4.2 Earthquake Effects

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

4.2.1 Shallow Ground Rupture

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

Since there are *no* known active or potentially active surface fault traces passing through the site, the potential for on-site ground rupture due to movement on an underlying fault in *not* considered a significant hazard, although it is a possibility at any site. The potential for ground rupture due to other causes is discussed in the following sections.

4.2.2 Earthquake-Induced Landsliding

Landslides are slope failures that occur where the horizontal seismic forces act to induce soil failure. Seismic Hazard Maps have been released by the California Geological Survey that delineate areas that have been subject to, or are potentially subject to landsliding or permanent ground displacement as a result of earthquake-induced ground shaking. Since the site and surrounding area are relatively flat, on-site earthquake-induced landsliding is *not* considered to be a hazard. The site is *not* located in an area designated on the *Seismic Hazard Zones Map* (CGS, 2002), Figure 4, as being susceptible to hazards associated with earthquake-induced landslides.

4.2.3 Seiches and Tsunamis

Seiches are an oscillation of the surface of an inland body of water that varies in period from a few minutes to several hours. Seismic ground motions can induce such oscillations. Tsunamis are large sea waves produced by submarine earthquakes or volcanic eruptions. The site is *not* located close to an inland body of water, but the site is located within an area designated on the Ventura County Seismic Risk Maps as being at risk for tsunami runup.

4.2.4 Evaluation of Liquefaction Potential

Liquefaction is a phenomenon in which soils below the groundwater level lose strength as a result of ground shaking due to earthquakes. The site is located in an area designated as potentially liquefiable on the *Seismic Hazard Zones Map* of the Oxnard Quadrangle (CDMG 2002), Figure 4. The results of field exploration and laboratory testing conducted as part of this investigation indicate that the subject site meets the criteria of being potentially susceptible to liquefaction. A detailed liquefaction analysis was therefore performed to further evaluate the potential and extent of possible liquefaction at this site.

Exploratory Boring B-1 was excavated to a depth of 51.5 feet to assess the liquefaction hazard potential at the site. The geotechnical data obtained from the boring and our laboratory test results, including standard penetration test data (SPT), percent fines and clay fraction, were utilized in our evaluation of liquefaction hazard potential at the site. Beach sand likely interlayered with alluvial sand deposits were encountered from the ground surface to a depth of approximately 40 feet, followed by sandy silt to the total depth explored, 51.5 feet

At the time of our field exploration, groundwater was encountered and stabilized at a depth of approximately 9 feet below the existing ground surface in Boring B-1. Based on the *Depth to Historically High Groundwater* Map (CGS, 2002), Figure 3, the historically highest groundwater level below the existing ground surface at the site is approximately 5 feet. The liquefaction hazard analysis was therefore performed utilizing the historically highest groundwater level of 5 feet below the ground surface.

The methods following the recommendations of the NCEER (Youd and Idriss, 1997; Youd et al, 2001) were used in the liquefaction analysis, supplemented by the recommendations of Bray and Sancio (2006), and Boulanger and Idriss (2006) in the analysis of fine grained soils (clays and silts). A design-level earthquake magnitude of 6.9, and a site acceleration of 0.768 (PGA_M) were utilized to perform the liquefaction evaluation.

Blow counts used for the liquefaction evaluation were based on the blow counts measured with an unlined, Standard Penetration Test sampler, or a modified California sampler, utilizing a 140-pound automatic trip hammer, falling 30 inches. The blow counts obtained when utilizing the modified California sampler were then multiplied by a factor of $^2/_3$ to convert to equivalent SPT blow counts, and then divided by 1.2 to cancel the unlined sample correction factor utilized in the liquefaction analysis spreadsheet. The measured blow counts were further adjusted for borehole diameter, rod length, sampling method and delivered energy (Youd and Idriss, 1997 and 2001) to correspond to a driving-energy level of 60% (N_{60}). The adjusted blow counts (N_{60}) were then adjusted for overburden pressure to obtain $N_1|_{60}$.

The results of the liquefaction analysis indicate that there are potentially liquefiable soils between the depths of approximately 7.5 and 15 feet, and 45 and 50 feet below the existing ground surface. Utilizing the procedures of Tokimatsu and Seed (1987), the maximum potential liquefaction induced settlement is anticipated to be approximately 2.35 inches. Potential differential settlement due to liquefaction is typically considered to be up to a maximum of approximately two-thirds of the total settlement, which would be approximately 1.57 inches, and is typically assumed to occur over a span of 30 feet. Therefore, as discussed in subsequent sections of this report, a mat foundation is recommended for support of the future structure. The remainder of the earth materials consist of either moderately dense to dense sand or stiff sandy silt, with corrected SPT blow counts all above 30, and would therefore not be considered susceptible to liquefaction (CGS, 2008).

The subject site is located in a relatively flat to only gently sloping area, with no open channel faces or descending slopes in the immediate vicinity, and the corrected, equivalent SPT blow counts are all above 15. Therefore, the risk of liquefaction-induced lateral spreading is considered to be negligible (Bartlett and Youd, 1992).

Based on the relative thicknesses of non-liquefiable soils overlying potentially liquefiable layers, and the fact that the first potentially liquefiable layer is at a depth of only 7.5, there is the potential for localized loss of bearing capacity, and other surface manifestations of liquefaction such as sand boils and fissures. Therefore, as discussed in subsequent sections of this report, a mat foundation is recommended for support of the future structure.

4.2.5 Dynamic Dry Settlement

The upper site soils will be removed and recompacted for support of the future structure, and the existing groundwater level is relatively shallow. Therefore, the potential for any significant dynamic dry settlement of dry sandy soils during seismic shaking is considered to be negligible.

5. CONCLUSIONS AND RECOMMENDATIONS

5.1 Conclusions and Design Requirements

Based on the findings of our data review, subsurface exploration, laboratory testing, field testing, and engineering analysis, and within the scope of this study, the future improvements are considered *feasible* from a geotechnical engineering viewpoint, provided the recommendations in this report are incorporated into the building plans and implemented during construction. The following sections discuss conditions that should be anticipated, and provide specific recommendations for mitigation of adverse conditions during the design and construction phase of improvements.

The upper onsite soils should be removed and properly recompacted for support of the proposed structure and other site improvements, as detailed in the following sections. Within the proposed building area, it is recommended that the existing earth materials be removed and recompacted to a depth of approximately 3 feet below existing and future site grade, and a minimum of 1 foot below the bottom of foundations, whichever is deeper. In the area of the proposed driveways, walkways and other miscellaneous surface improvements, it is recommended that a minimum of approximately 1 foot of newly placed compacted fill be provided for support of these elements.

Due to the presence of potentially liquefiable soils, and the potential for total and differential liquefaction-induced settlements of an estimated 2.35 and 1.57 inches, respectively, and the potential for surface manifestation of liquefaction including sand boils and loss of bearing capacity, it is recommended that a mat foundation be utilized for support of the future structure. It is recommended that the proposed foundation system be supported entirely on newly placed compacted fill.

All uncertified fill associated with the abandonment of the previously existing water well onsite should be removed, stockpiled onsite, and properly recompacted during site grading. A qualified representative of a geotechnical engineering company would be onsite during the excavation process to help determine the depth and lateral extent of the existing uncertified fill, which is mainly a visual determination.

5.1.1 Faults / Seismicity

Although no known active surface fault traces cross the subject site, like most of Southern California, the site lies within a seismically active area. Earthquake resistant structural design is recommended. Designing structures to be earthquake-proof is generally considered to be impractical, especially for private projects, due to cost limitations. Significant damage to structures may be unavoidable during large earthquakes. Structural design based on the 2016 CBC (California Building Code) structural analysis procedures calls for the seismic parameters given previously in the *Seismic Design Criteria* section of this report. These minimum code values are intended to protect life and may not provide an acceptable level of protection against significant cosmetic damage and serious economic loss. Significantly higher than code parameter values would be necessary to further reduce potential economic loss during a major seismic event. Structural Engineers, however, often regard higher than code values or procedures as impractical for use in structural design. The Structural Engineer and project Owner must decide if the level of risk associated with code values is acceptable and, if not, to assign appropriate seismic values above code values for use in structural design.

5.1.2 Hazardous Materials

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

5.1.3 Site Grade Adjustments

Grading for the future single-family residence is expected to consist of removal and recompaction of the existing well abandonment backfill, and the upper site soils for support of the future structure and associated improvements. The finished building pad elevation is expected to be within approximately 1 to 2 feet of the existing grade at the site.

5.1.4 Excavation Characteristics

The upper earth materials underlying the site consist of sand, and therefore hard to excavate materials should not be encountered. Caving of the sandy soils should be expected.

5.1.5 Drainage

All surface runoff must be carefully controlled and must remain a crucial element of site maintenance. Proper drainage and irrigation are important to reduce the potential for excessive infiltration adjacent to foundations. Final grading should provide positive drainage away from footings and other improvements in compliance with the local jurisdiction's grading requirements. All pad drainage shall be collected and diverted away from future structure and foundations in non-erosive devices. Gutters and roof drains should be provided, properly maintained, and discharge directly into glue-joined, watertight subsurface piping. A drainage system consisting of area drains, catch basins, and connecting lines should be provided to capture landscape/hardscape sheet flow discharge water. All drainage piping should be watertight and discharge to an appropriate location, as determined by the project Civil Engineer.

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

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

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

5.1.6 Plan Review

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

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

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

5.2 Site Preparation
The area of the future single-family residence should be prepared so that foundations are founded entirely within newly placed compacted fill. General guidelines are presented below to provide a basis for quality control during site grading. It is recommended that all compacted fills be placed and compacted with engineering control under continuous observation and testing by the Geotechnical Engineer and/or his field representative, and in accordance with the following requirements.

5.2.1 Removals
a. When demolishing any existing improvements in the vicinity of the future structure and other improvements, the contractor should locate all existing foundations, floor slabs, debris pits, uncertified fill, and subsurface trash which may be present. This would include all of the materials placed to backfill the excavations made during the abandonment of the previously existing well onsite. These soils and structures should be completely removed. The resulting excavations should be cleaned of all loose soils and organic material, the exposed native soils should be scarified to a depth of 8 inches and compacted, and the excavation backfilled with compacted fill. Minimum over-excavation depths are required within the areas of the future structure and other improvements, as discussed below.

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

c. The future building area should be over-excavated to a minimum depth of approximately 3 feet below the existing site grade, or a minimum of approximately 1 foot below the bottom of the proposed foundations, whichever is deeper. The limits of over-excavation should extend a minimum of approximately 1 to 2 feet beyond the outside perimeter of foundations, where possible. The excavated onsite earth materials may then be replaced as compacted fill, as described below.

d. The removal and recompaction of all existing uncertified fill should include the uncertified fill associated with the abandonment of the previously existing water well onsite.

e. In areas to receive new exterior hardscape or other miscellaneous improvements, all existing fill materials and any other loose or disturbed soil should be removed and recompacted. The depth of over-excavation in these areas should be a minimum of either 12 inches below existing grade, or 12 inches below the bottom of any improvements, or supporting aggregate base section, whichever is deeper.

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

g. The exposed bottom of removal areas should be scarified, mixed, and moisture conditioned to a minimum depth of 8 inches. This thickness of scarification is included in the thickness of removal and recompaction mentioned above, unless the bottom is unstable and requires stabilization. The scarified soil shall be moisture conditioned to near optimum moisture content and compacted to a minimum of 90% of the laboratory maximum dry density as determined by ASTM D1557. Additional lifts should *not* be placed until the present lift has been tested and shown to meet the compaction requirements.

5.2.2 Bottom Stabilization

a. Depending on the time of year and recent precipitation, or should the bottom of over-excavation become flooded by rain during grading, or be found to be wet or 'pumping' due to influence from the groundwater below, additional stabilization of the bottom of over-excavation may be required. If the bottom is unstable, the use of track-mounted equipment and/or excavators should be considered to reduce the potential for disturbing the soils in the excavations near the groundwater level. If the bottom is extremely wet and pumping, the use of stabilization gravel and/or geogrid such as Mirafi 600X, may be required.

5.2.3 Suitable Fill Material

a. The excavated site soils, cleaned of deleterious material, can be re-used for fill. Rock larger than 6 inches should *not* be buried or placed in compacted fill. Rock fragments less than 6 inches may be used provided the fragments are *not* placed in concentrated pockets, and a sufficient percentage of finer grained material surrounds and infiltrates the rock voids. Furthermore, the placement of any rock must be under the continuous observation of the Geotechnical Engineer, and/or his field representative.

b. Imported material should generally have engineering properties similar to, or more favorable than those on the subject site. Imported material will require testing to verify the engineering properties, and must be approved by the Geotechnical Engineer *prior* to placement on the site.

5.2.4 Placement of Compacted Fill

a. All fill materials should be placed in controlled, horizontal layers *not* exceeding 6 to 8 inches thick, and moisture conditioned to near optimum moisture content. Fill materials should be compacted to a minimum 90% of the laboratory maximum dry density, as determined by ASTM D1557. If either the moisture content or relative compaction does *not* meet these criteria, the Contractor should rework the fill until it does meet the criteria. If the fill materials pump (flex) under the weight of construction equipment, difficulties in obtaining the required minimum compaction may be experienced. Therefore, if soil pumping occurs, it may be necessary to control the moisture content to a closer tolerance (e.g., 2 to 3% above optimum) or use construction equipment that is not as prone to cause pumping.

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

c. Subgrade for the support of exterior concrete flatwork such as the proposed driveway and walkways shall be moisture conditioned, as required, to near optimum moisture content, and recompacted to at least 90% of the maximum dry density to a depth of at least 12 inches. For the proposed driveway area, the same procedures should be followed, but a minimum of 95% compaction should be obtained.

5.2.5 Testing of Compacted Fill

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

5.2.6 Inclement Weather and Construction Delays

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

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

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

5.2.7 Responsibilities

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

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

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

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

5.3 Utility Trench Backfill

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

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

Trench backfill should be placed in 8-inch lifts, moisture conditioned to near optimum moisture content, and compacted to at least 90% of the maximum density as determined by ASTM D1557, with the exception of the 1 foot below subgrade in the proposed driveway area, which should be compacted to 95% of the maximum dry density.

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

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

5.4 Temporary Excavations

Temporary excavations made as part of the required removal and recompaction operations may be made to a maximum vertical height of 3 feet. Excavations should *not* be allowed to become soaked with water or to dry out. Surcharge loads should *not* be permitted within a horizontal distance equal to the height of the excavation from the top of the excavation, unless the excavation is properly shored. Excavations that might extend below an imaginary plane inclined at 45 degrees below the edge of an existing foundation should be properly shored to maintain foundation support of the existing structure.

5.5 Foundation Design

Due to the presence of potentially liquefiable soils, and the potential for surface manifestation of liquefaction including sand boils and ground fissuring, and potential total and differential liquefaction-induced settlements of up to an estimated 2.35 and 1.57 inches, respectively, a mat foundation is recommended for support of the future structure. It is recommended that the proposed foundation be supported entirely on newly placed compacted fill.

It is recommended that the perimeter of the proposed mat foundation be embedded a minimum of 18 inches in depth below the lowest adjacent grade, and 18 inches into the newly placed compacted fill. Where located adjacent to utility trenches, foundations shall extend below a 1:1 plane projected upward from the inside bottom of the trench.

5.5.1 Allowable Bearing Pressure and Lateral Resistance

The proposed mat foundation may be designed using a modulus of subgrade reaction of 200 kcf (kips per cubic foot). The allowable vertical and lateral bearing values given below may also be utilized in the design of the mat foundation. The bearing capacity can be increased by ⅓ when considering short duration wind or seismic loads.

Support Material	Allowable Bearing Pressure, psf	Allowable Sliding Friction Coefficient	Allowable Passive Resistance, psf per foot of depth	Maximum Passive Resistance, psf
Compacted Fill	1500	0.3	225	2250

Resistance to lateral loads can be assumed to be provided by friction along the base of the foundation, and by passive earth pressure against the side of foundation. The allowable friction coefficient may be used with the vertical dead loads, and the allowable lateral passive pressure can be utilized for the side of the foundation poured against newly placed compacted fill. These allowable values can be increased by a factor of 1.5 to convert from allowable to ultimate values. Where the soil on the resistance side of the passive wedge in not covered by a hard surface (e.g., concrete or pavement), however, the upper 1-foot of soil shall be neglected when computing resistance due to the potential for the material to become disturbed or degraded during the life of the structure.

5.5.2 Foundation Settlement
Static settlement of proposed foundation due to dead and frequently applied live loads is not expected to exceed approximately ½ to ¾ inch under the assumed loading conditions, and is expected to occur primarily upon initial application of loading. Static differential settlement is not expected to exceed approximately ¼ to ½ inch.

As described previously in this report, the maximum potential settlement due to liquefaction is anticipated to be up to approximately 2.35 inches, with potential differential settlement of up to approximately 1.57 inches over a span of 30 feet.

5.5.3 Steel Reinforcement
Steel reinforcing for the proposed mat foundation should be designed by the project structural engineer.

5.5.4 Required Observations
Prior to placing concrete in the foundation excavations, an observation should be made by a field representative of the Geotechnical Engineer to confirm that the excavations are free of loose and disturbed soils, and are embedded in the recommended earth materials.

5.5.5 Vapor Barrier
It is recommended that a minimum 10-mil plastic vapor barrier be used under the mat foundation slab in moisture sensitive areas. The vapor barrier should be installed in accordance with the recommendations contained in the latest version of ASTM E1643. In accordance with the latest standard of practice, it is recommended that the concrete mat foundation slab be poured directly on top of the vapor barrier. No sand should be placed atop the vapor barrier. Seams of the vapor barrier should be overlapped and sealed. Where pipes extend through the vapor barrier, the barrier should be sealed to the pipes. Tears or punctures in the vapor barrier should be completely repaired *prior* to placement of concrete. The concrete mix should be designed so as to minimize possible curling of the slab. The concrete slab should be allowed to cure properly before placing vinyl or other moisture-sensitive floor covering.

5.6 Concrete Pavement and Patio Design
All areas to be paved or covered with concrete flatwork (driveway, patios, walkways, etc.) or other hardscape should be graded in accordance with the recommendations provided in the *Site Preparation* section of this report.

All exterior concrete pavement, patios, walkways, etc., should be a minimum of 5 inches thick, and should be reinforced with a minimum of #4 steel bars on 18-inch centers each way. Concrete subject to vehicular traffic should be underlain by a minimum of 4 inches of aggregate base.

Cracking of concrete pavement, flatwork and other hardscape can occur and is relatively common. Steel reinforcement and crack control joints are intended to reduce the risk of concrete slab cracking, as are the use of fiber reinforced concrete and proper concrete curing. Cracking can never be completely eliminated, but can be controlled through the use of proper jointing and curing.

6. OBSERVATIONS AND TESTING

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

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

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

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

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

7. LIMITS AND LIABILITY

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

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

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

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

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

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

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

In the event that any changes in the nature and design (including structural loadings different from those anticipated), or other improvements are planned, the conclusions and recommendations contained in this report shall *not* be considered valid unless the changes are reviewed and conclusions of this report modified or verified in writing.

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

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

Appendix A

Field Exploration and Boring Logs

Appendix A
Field Exploration and Boring Logs

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

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

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

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

When noted on the boring logs, standard penetration test (SPT) samples were obtained using either a 20-inch or a 32-inch long split-barrel sampler with a 2-inch outside diameter and a 1.375-inch inside diameter when liners are used (1.5-inch inside diameter without liners). Unless noted otherwise, liners are used. This sampler is driven into the soil with successive drops of a 140-pound, safety hammer falling 30 inches. The blows are recorded for each 6 inches of penetration for a total penetration of 18 or 24 inches. The sum of the number of blows for the last 12 inches of an 18-inch penetration or the middle 12 inches of a 24-inch penetration is referred to as the *N* value.

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

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

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

Legend of Laboratory Tests

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

Sampler Type

Modified California

SPT

Rock Core

No Recovery

Hand Sampler

Shelby Tube

Bulk

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

Terms Describing Density and Consistency

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

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

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

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

Terms Characterizing Soil Structure

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

Soil Moisture

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

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

Size Proportions

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

Grain Size Distribution

Clay | Silt | Sand: Fine, Medium, Coarse | Gravel: Fine, Coarse

Sieve Size Number: 200, 40, 10, 4, 3/4", 2", 3"

Particle Diameter in Millimeters: 0.005, 0.01, 0.05, 0.1, 0.5, 1.0, 5.0, 10.0, 50, 100

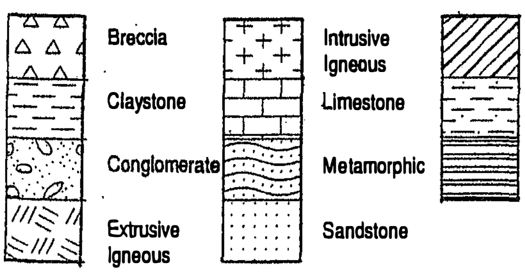
Degree of Weathering
Diagnostic Feature

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

Discontinuity Spacing

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

Graphic Symbols - Bedrock



Rock Hardness

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

Separation of Fracture Walls

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

Fracture Filling

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

Surface Roughness

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

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

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Boring Log B-1

Project **Channel Islands Beach Community Services District** Client No. **4844** Date Drilled **7/23/18**

Comment **112 Las Palmas Street, Oxnard**

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

Driving Weight (lbs) **140** Average Drop (in.) **30** Hole Diameter (in.) **6**

Elevation ____ ft Depth to Water **9.0** ft After ____ hrs on ____ Logged By **BW**

Description of Material

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

Depth, ft	Sample	Blows/6"	Graphic Symbol	Description of Material	Attitudes	Dry Unit Weight, pcf	Moisture Content, %	#200, %	Other Tests
				Beach Sand (Qs) Tan medium to coarse grained SAND, dry, loose at surface, becomes slighty moist and moderately dense by a depth of approximately 1 to 2 feet					
		11 16 19				101.0	3.3		
5		10 14 15				103.7	1.6		E.I.= 0
		5 7 9		becomes very moist @ 7.5 ft.		96.4	4.5		
				▼ groundwater @ 9 ft.					
10		6 9 12		no sample recovery @ 10 ft.					
15		6 9 15		becomes fine to coarse grained sand, wet, dense			22.5		
20		4 8 13					13.7		
25		14 26 28					14.7		

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Boring Log B-1

Sheet 2 of 2

Project **Channel Islands Beach Community Services District** Client No. **4844** Date Drilled **7/23/18**

Comment **112 Las Palmas Street, Oxnard**

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

Driving Weight (lbs) **140** Average Drop (in.) **30** Hole Diameter (in.) **6**

Elevation ____ ft Depth to Water **9.0** ft After ____ hrs on ____ Logged By **BW**

Description of Material

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

Depth, ft	Sample	Blows/6"	Graphic Symbol	Description of Material	Attitudes	Dry Unit Weight, pcf	Moisture Content, %	#200, %	Other Tests
	X	6 6 12		becomes medium to coarse grained sand, wet, dense silty lense at bottom of 30 ft. sample, firm, very moist			24.7 19.3		
35	X	6 8 15		becomes gray, medium to coarse grained, slightly silty			19.1		
40	X	8 10 27		Light gray fine grained Sandy SILT, stiff, very moist, minor clay			22.1		
45	X	4 5 7		grades siltier			37.6		
50	X	7 16 19					24.0		
55				Total Depth Explored = 51.5 ft. Groundwater Encountered @ 9 ft. No Groundwater Encountered					

advanced geotechnical services, inc.

Boring Log B-2

Project **Channel Islands Beach Community Services District** Client No. **4844** Date Drilled **7/23/18**

Comment **112 Las Palmas Street, Oxnard**

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

Driving Weight (lbs) **140** Average Drop (in.) **30** Hole Diameter (in.) **6**

Elevation ____ ft Depth to Water **8.5** ft After ____ hrs on ____ Logged By **BW**

Description of Material

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

Depth, ft	Sample	Blows/6"	Graphic Symbol	Description of Material	Attitudes	Dry Unit Weight, pcf	Moisture Content, %	-#200, %	Other Tests
				Beach Sand (Qs) Tan medium to coarse grained SAND, dry, loose at surface, becomes slighty moist and moderately dense by a depth of approximately 1 to 2 feet					
		15 18 18				85.1	26.5		
5		9 13 15		becomes medium to coarse grained SAND, slightly moist, dense		102.8	2.5		
		5 7 7		becomes very moist @ 7.5 ft.		94.5	20.2		
				▼ groundwater encountered @ 8.5 ft.					
10		2 2 3					24.5		
15		14 16 17		becomes coarse grained sand, wet, dense			17.5		
20				Total Depth Explored = 16.5 ft. Groundwater Encountered @ 8.5 ft. Backfilled with Spoils 7/23/2018					
25									

Appendix B

Laboratory Testing

Appendix B
Laboratory Testing

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

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

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

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

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

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

shown on the graphs (associated with peak values) is for either the in-situ condition or the remolded condition, and the second moisture content (associated with ultimate value) is for the soaked condition.

Consolidation Test

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

Compaction Test

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

Expansion Index Test

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

Sample Remolding

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

Job No. 4844 Date 8/7/2018

Project Channel Islands Beach Community Services District - 112 Las Palmas Street, Oxnard

Source of Material B-1 5.0

Description of Material Tan medium to coarse grained SAND

Test Method

Test Results

Maximum Dry Density 106.5 pcf

Optimum Water Content 13.5 %

Atterberg Limits

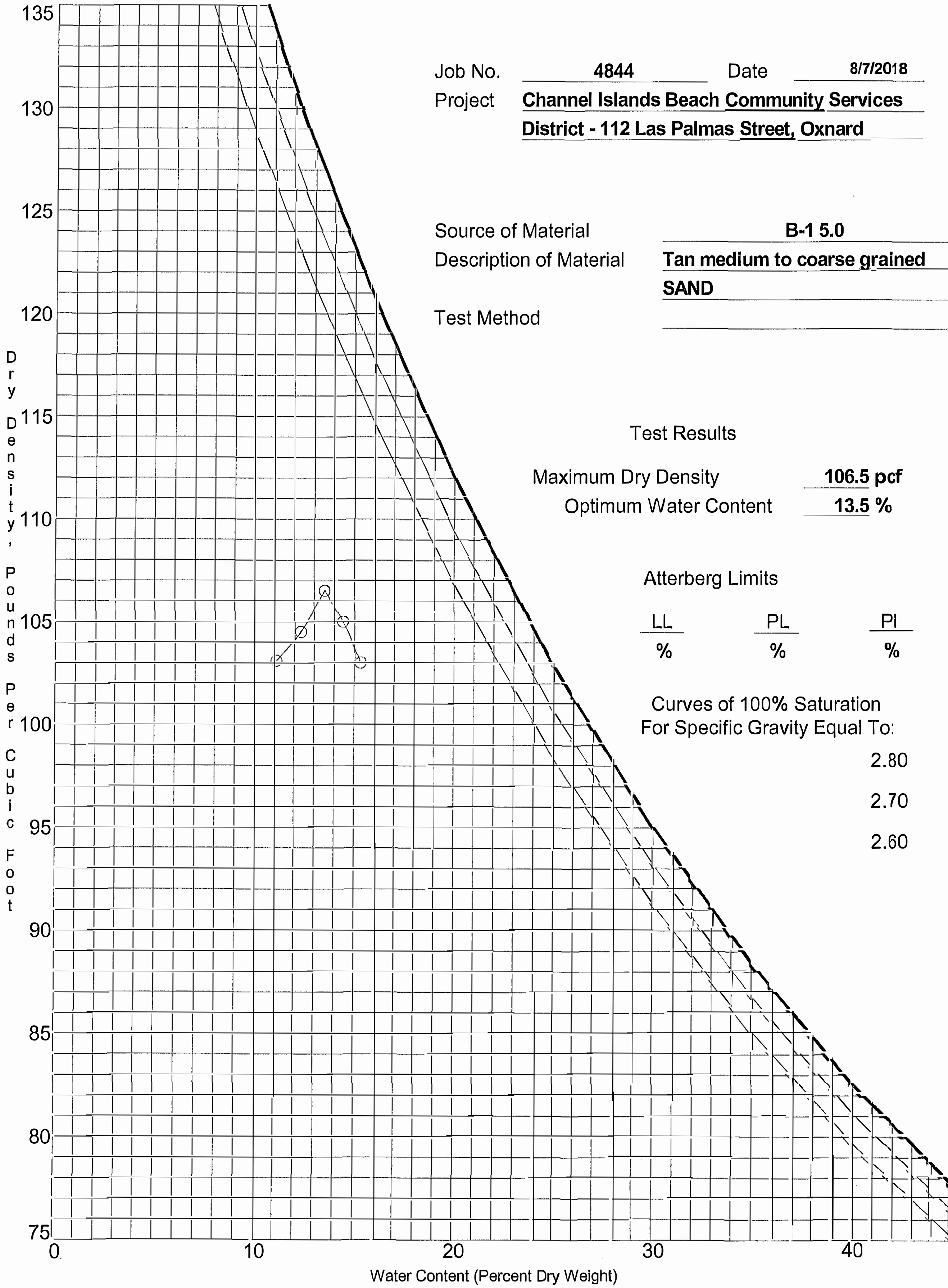
LL	PL	PI
%	%	%

Curves of 100% Saturation
For Specific Gravity Equal To:

2.80

2.70

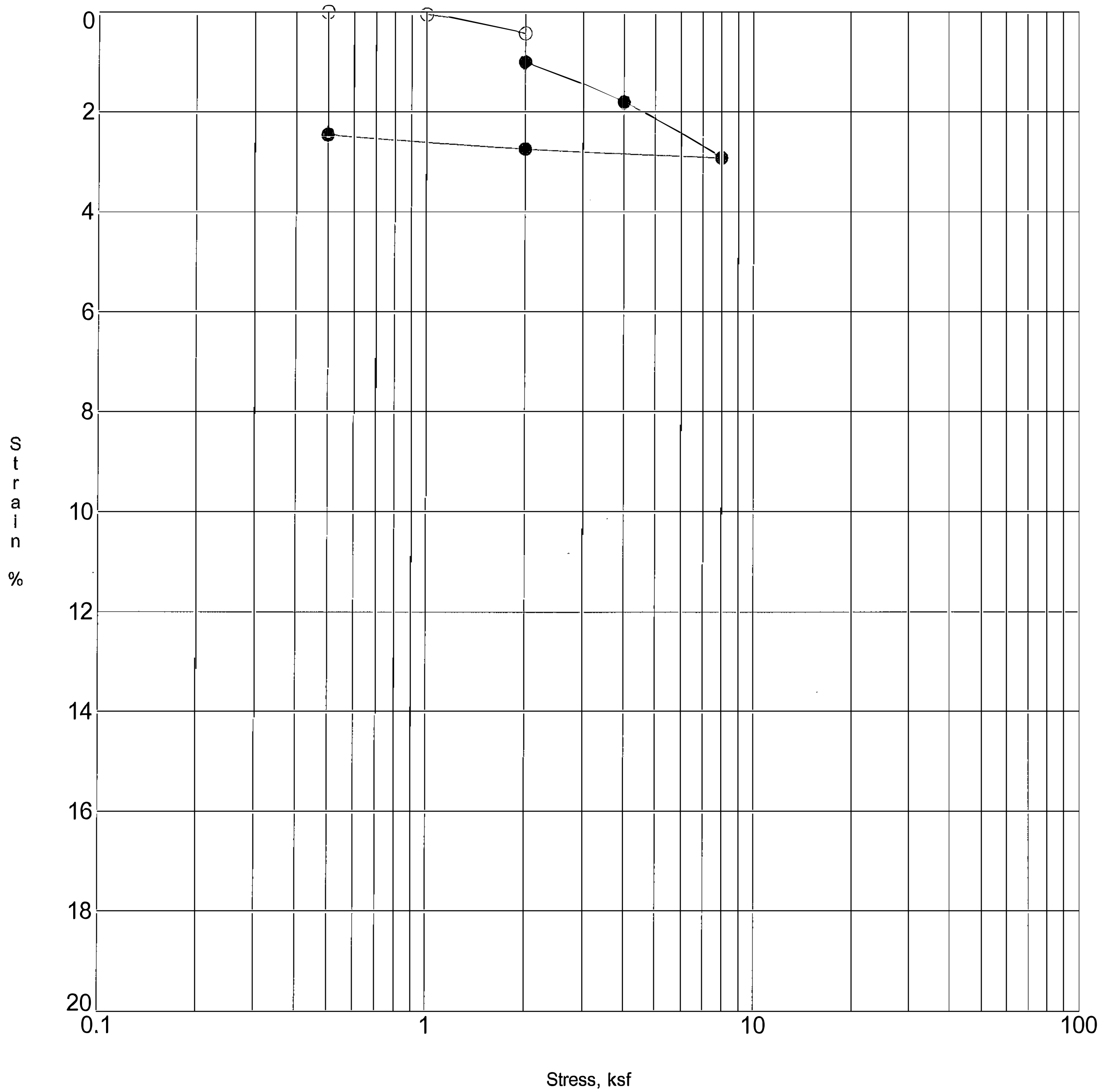
2.60



Moisture-Density Relationship

Advanced Geotechnical Services, Inc.

Plate B- 3



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

	Specimen Identification		Classification	DD	MC%
○	**B-1**	**5.0**	**Tan medium to coarse grained SAND**	**99.3**	**4.2**
●	**B-1**	**5.0**	***UNDISTURBED***	**101.8**	**16.7**

Project **Channel Islands Beach Community Services District - 112 Las Palmas Street, Oxnard**

Client No. **4844**

Date **8/7/18**

Consolidation Test

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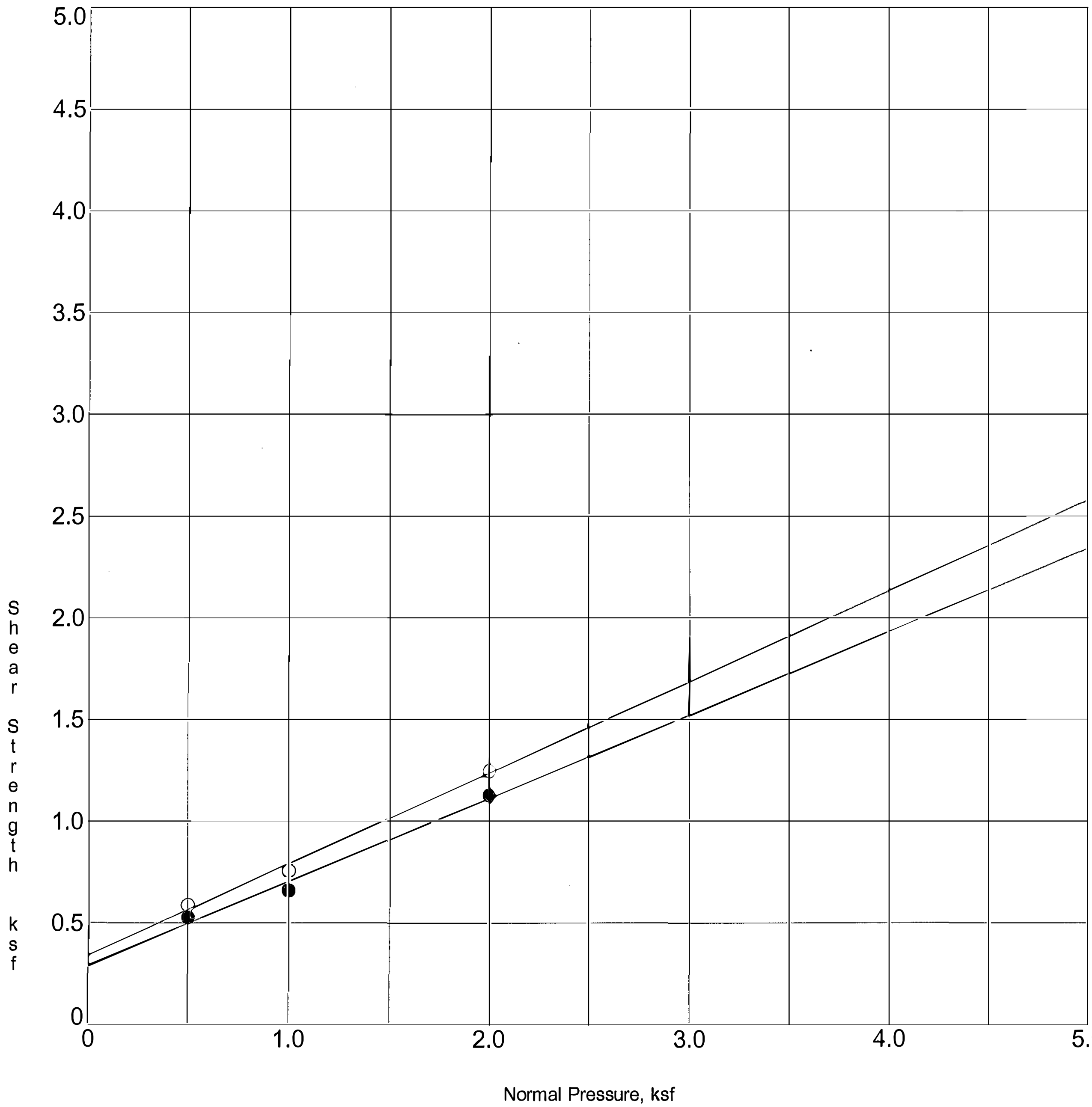
Plate B- 4



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

	Specimen Identification		Classification	DD	MC%	c, ksf	phi
○	B-1	5.0	Tan medium to coarse grained SAND	96.0	15.5	0.34	24
●	B-1	5.0	*REMOLD*	96.0	19.8	0.29	22

Project **Channel Islands Beach Community Services District - 112 Las Palmas Street, Oxnard**

Client No. **4844**

Date **8/7/18**

Shear Test Diagram

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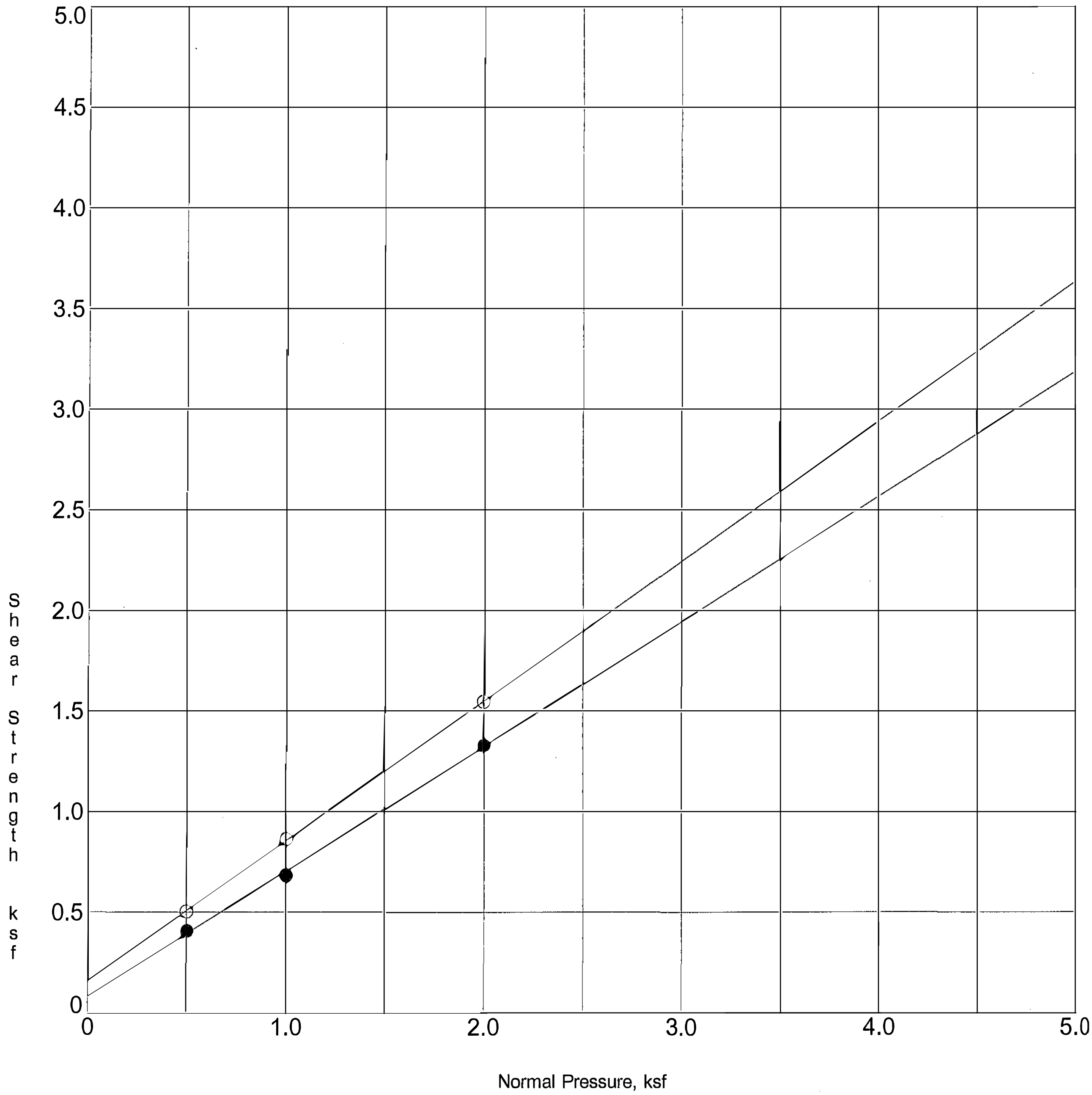
Plate B- 5



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

	Specimen Identification		Classification	DD	MC%	c, ksf	phi
○	**B-2**	**5.0**	**Tan fine to coarse grained SAND**	**95.7**	**6.9**	**0.16**	**35**
●	**B-2**	**5.0**	***UNDISTURBED***	**95.7**	**23.3**	**0.08**	**32**

Project **Channel Islands Beach Community Services District - 112 Las Palmas Street, Oxnard**

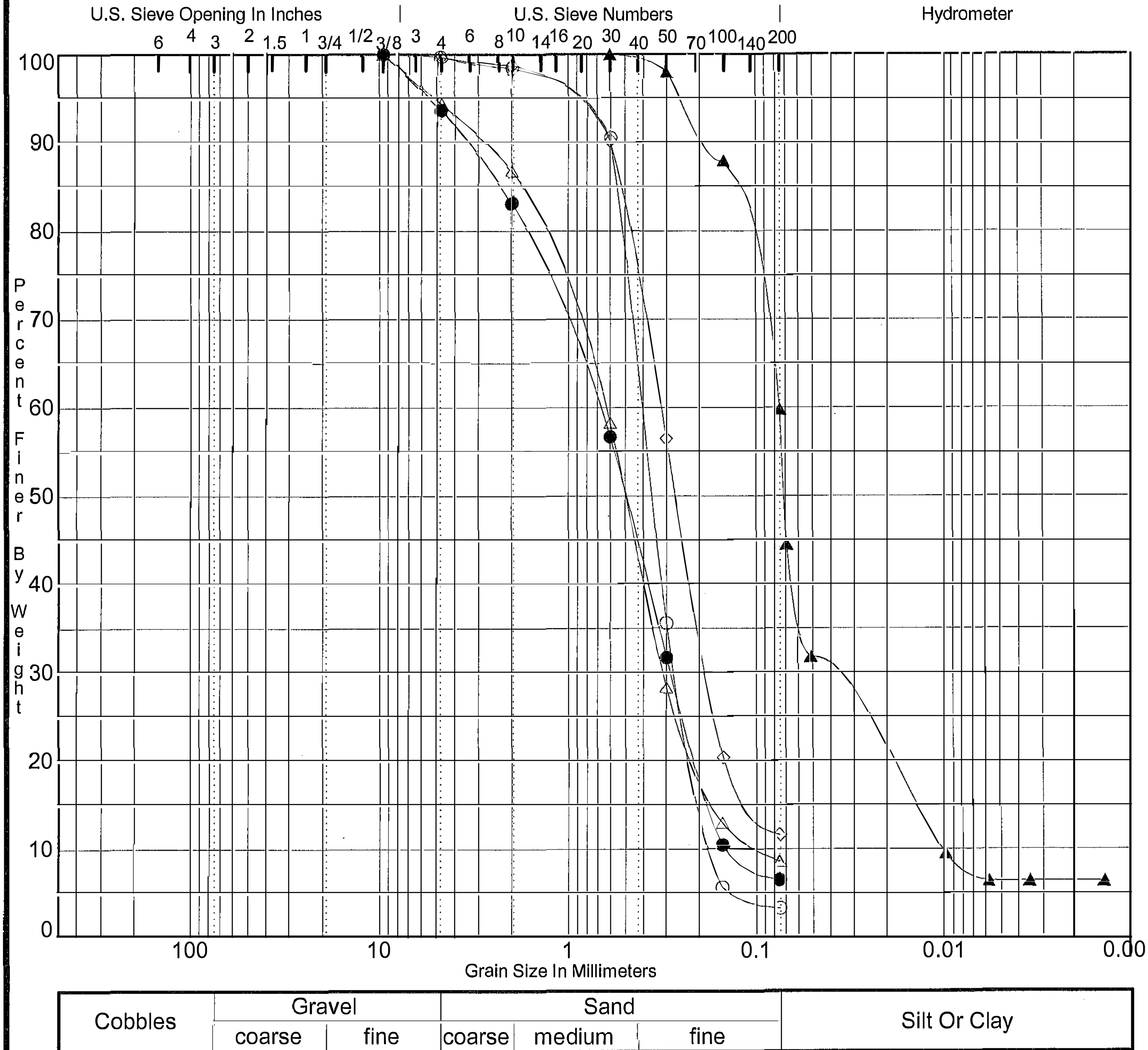
Client No. **4844**

Date **8/7/18**

Shear Test Diagram

Advanced Geotechnical Services, Inc.

Plate B- 6



Specimen Identification			Classification	MC%	LL	PL	PI	Cc	Cu
○	B-1	15.0	Tan fine to coarse grained SAND					1.02	2.5
●	B-1	20.0	Tan fine to coarse grained SAND					0.83	5.0
△	B-1	25.0	Tan fine to coarse grained SAND					1.61	6.9
▲	B-1	30.0	Gray Sandy SILT					2.58	7.3
◇	B-1	35.0	Gray medium to coarse grained SAND					1.53	4.9

Specimen Identification			D100	D60	D30	D10	%Gravel	%Sand	%Silt	%Clay
○	B-1	15.0	9.50	0.41	0.264	0.1661	0.3	96.4	3.3	
●	B-1	20.0	9.50	0.70	0.284	0.1397	6.4	87.1	6.5	
△	B-1	25.0	9.50	0.64	0.312	0.0940	5.7	85.7	8.6	
▲	B-1	30.0	0.60	0.08	0.045	0.0103	0.0	40.2	53.4	6.4
◇	B-1	35.0	9.50	0.32	0.180		0.3	88.1	11.6	

Project **Channel Islands Beach Community Services District - 112 Las Palmas Street, Oxnard**

Client No. **4844**

Date **8/7/18**

Gradation Curves

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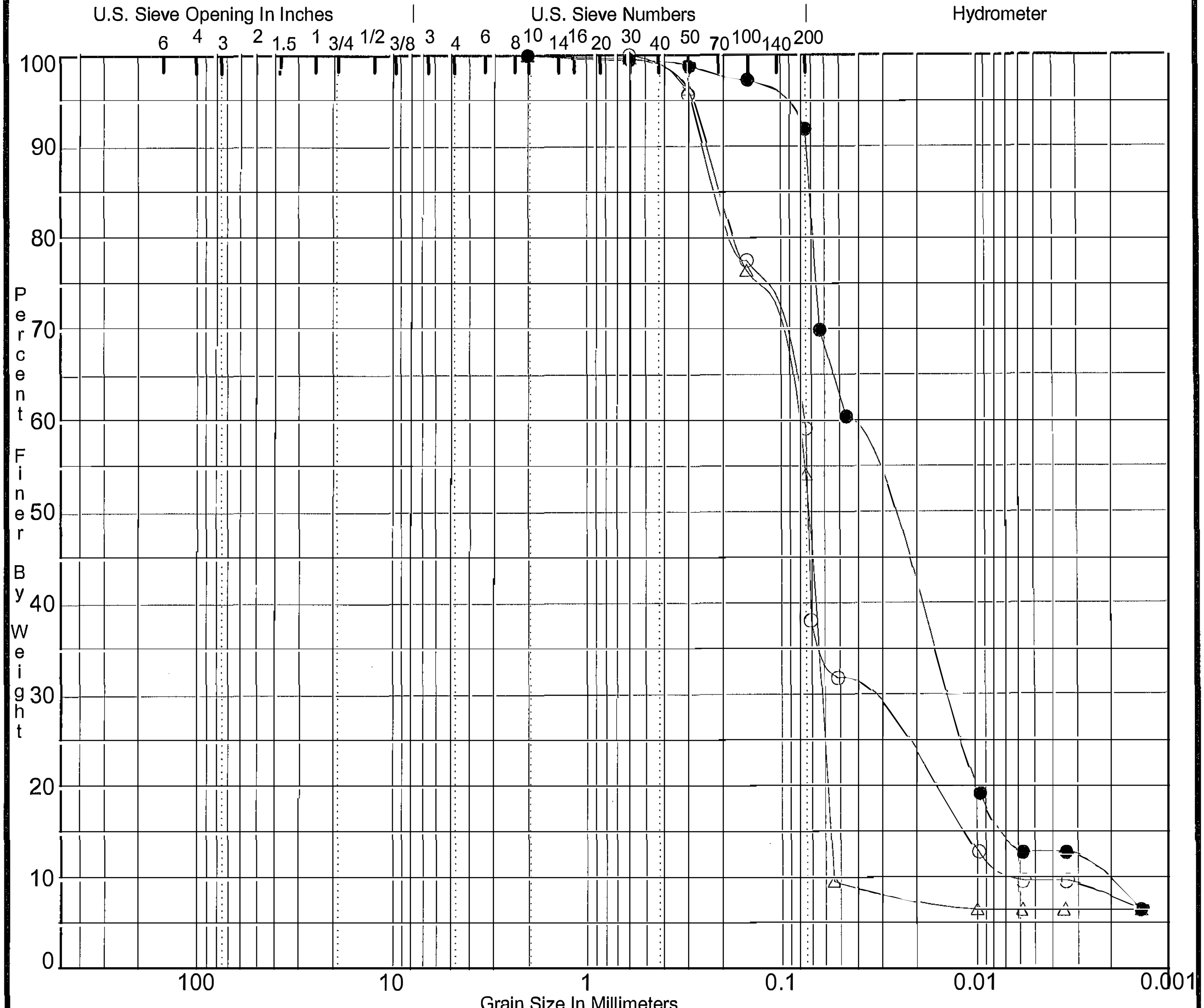
Advanced Geotechnical Services, Inc.

Plate B- 7



Cobbles	Gravel		Sand			Silt Or Clay
	coarse	fine	coarse	medium	fine	

	Specimen Identification		Classification	MC%	LL	PL	PI	Cc	Cu
○	**B-1**	**40.0**	**Light gray fine grained Sandy SILT**					**3.96**	**12.5**
●	**B-1**	**45.0**	**Light gray fine grained Sandy SILT**					**1.99**	**19.6**
△	**B-1**	**50.0**	**Light gray fine grained Sandy SILT**					**0.81**	**1.7**

	Specimen Identification		D100	D60	D30	D10	%Gravel	%Sand	%Silt	%Clay
○	**B-1**	**40.0**	**0.60**	**0.08**	**0.044**	**0.0062**	**0.0**	**40.9**	**49.6**	**9.5**
●	**B-1**	**45.0**	**2.00**	**0.05**	**0.015**	**0.0023**	**0.0**	**8.1**	**79.2**	**12.7**
△	**B-1**	**50.0**	**2.00**	**0.09**	**0.063**	**0.0545**	**0.0**	**45.9**	**47.7**	**6.4**

Project **Channel Islands Beach Community Services District - 112 Las Palmas Street, Oxnard**

Client No. **4844**

Date **8/7/18**

Gradation Curves

Advanced Geotechnical Services, Inc.

Plate B- 8

AMERICAN ANALYTICS

LABORATORY ANALYSIS RESULTS

Client: Advanced Geotechnical Services, Inc.
Project No: 4894
Project Name: Channel Island Beach CSD

AA Project No: A97589
Date Received: 07/24/18
Date Reported: 08/06/18

ANALYTICAL DATA SUMMARY

Analyte	Sample Name	Result	MRL	Units	Dilution	Prepared	Analyzed	Method
Chloride by Ion Chromatography								
Chloride	B-1@5'	**22**	5.0	mg/kg	1	07/30/18	07/30/18	EPA 300.0
General Chemistry Analyses								
pH	B-1@5'	**7.5**	0.50	pH Units	1	07/27/18	07/27/18	9045C
Specific Conductance (EC)	B-1@5'	**270**		umhos /cm	1	07/27/18	07/27/18	EPA 120.1
Sulfate by Ion Chromatography								
Sulfate	B-1@5'	**16**	5.0	mg/kg	1	07/30/18	07/30/18	EPA 300.0

Allen A

Allen Aminian
QA/QC Manager

Appendix C

Seismicity Study

Design Maps Summary Report

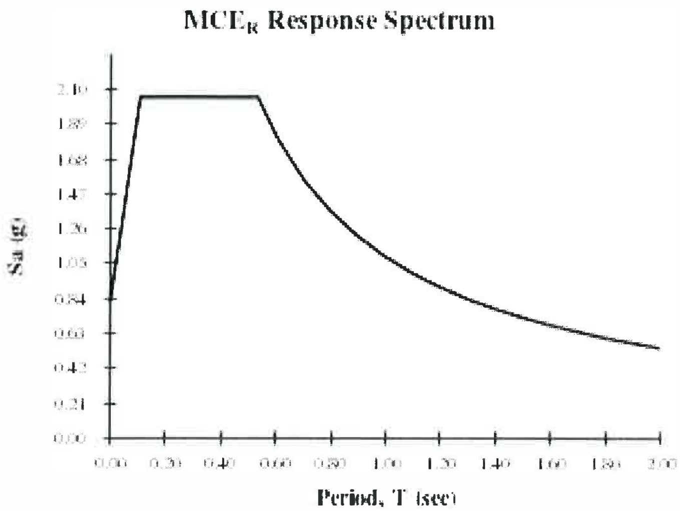
User–Specified Input

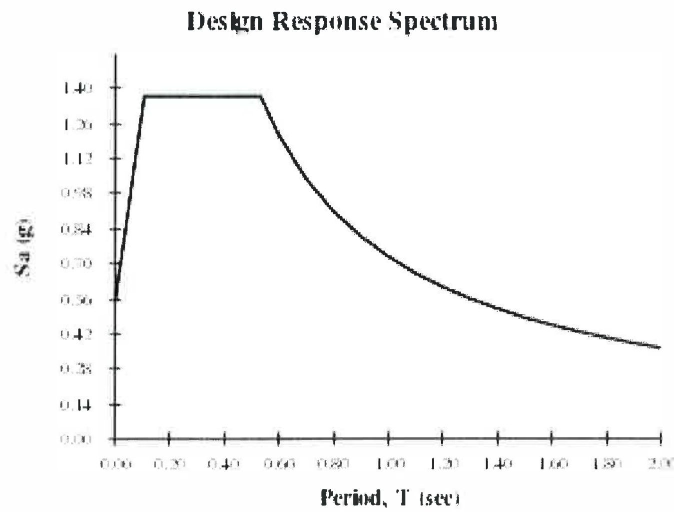
Report Title	C.I.Beach Comm. Serv. Dist. 112 Las Palmas Thu July 5, 2018 23:08:32 UTC
Building Code Reference Document	ASCE 7-10 Standard (which utilizes USGS hazard data available in 2008)
Site Coordinates	34.16466°N, 119.22851°W
Site Soil Classification	Site Class D – "Stiff Soil"
Risk Category	I/II/III

USGS–Provided Output

S_S =	2.053 g	S_{MS} =	2.053 g	S_{DS} =	1.369 g
S_1 =	0.727 g	S_{M1} =	1.090 g	S_{D1} =	0.727 g

For information on how the SS and S1 values above have been calculated from probabilistic (risk-targeted) and deterministic ground motions in the direction of maximum horizontal response, please return to the application and select the "2009 NEHRP" building code reference document.





For PGA_M, T_L, C_{RS}, and C_{R1} values, please view the detailed report.

Although this information is a product of the U.S. Geological Survey, we provide no warranty, expressed or implied, as to the accuracy of the data contained therein. This tool is not a substitute for technical subject-matter knowledge.

USGS Design Maps Detailed Report

ASCE 7-10 Standard (34.16466°N, 119.22851°W)

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

Section 11.4.1 — Mapped Acceleration Parameters

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

From Figure 22-1 [1]	S_S = 2.053 g
From Figure 22-2 [2]	S_1 = 0.727 g

Section 11.4.2 — Site Class

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

Table 20.3–1 Site Classification

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

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

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

Table 11.4–1: Site Coefficient F_a

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

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

For Site Class = D and S_S = 2.053 g, F_a = 1.000

Table 11.4–2: Site Coefficient F_v

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

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

For Site Class = D and S_1 = 0.727 g, F_v = 1.500

Equation (11.4-1): $S_{MS} = F_a S_S = 1.000 \times 2.053 = 2.053$ g

Equation (11.4-2): $S_{M1} = F_v S_1 = 1.500 \times 0.727 = 1.090$ g

Section 11.4.4 — Design Spectral Acceleration Parameters

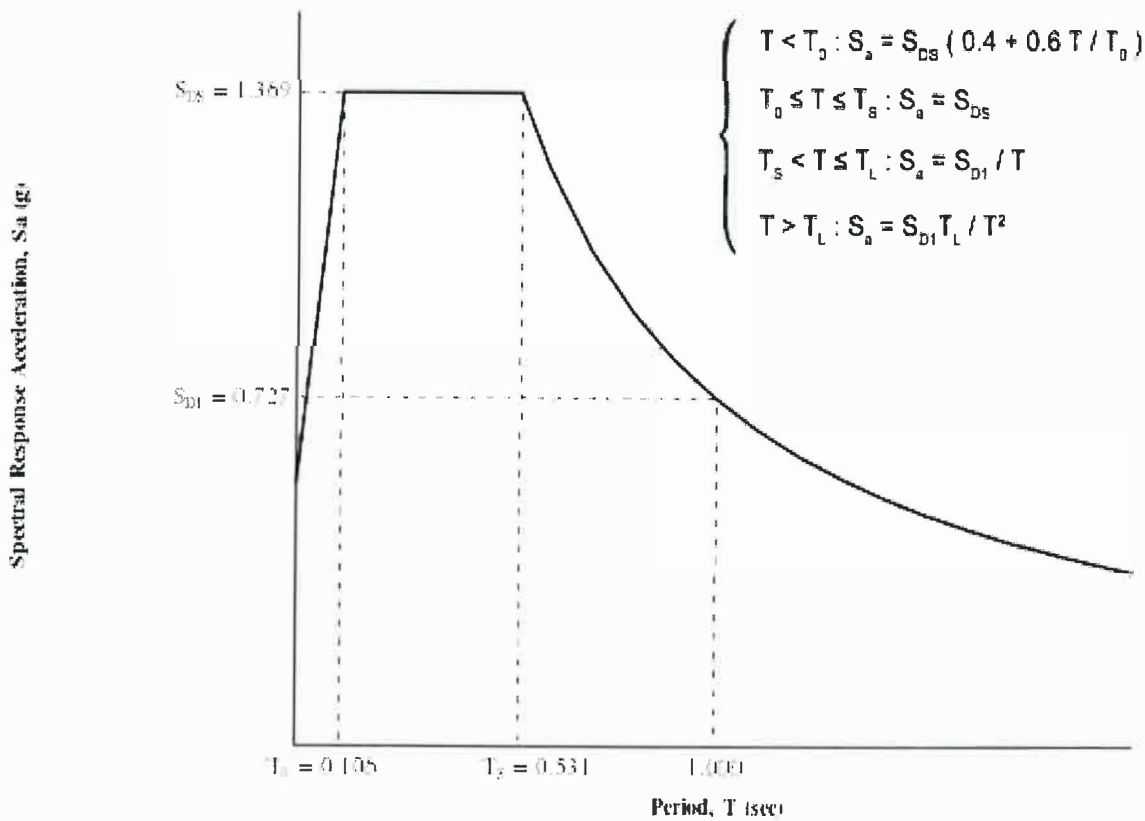
Equation (11.4-3): $S_{DS} = ⅔\, S_{MS} = ⅔ \times 2.053 = 1.369$ g

Equation (11.4-4): $S_{D1} = ⅔\, S_{M1} = ⅔ \times 1.090 = 0.727$ g

Section 11.4.5 — Design Response Spectrum

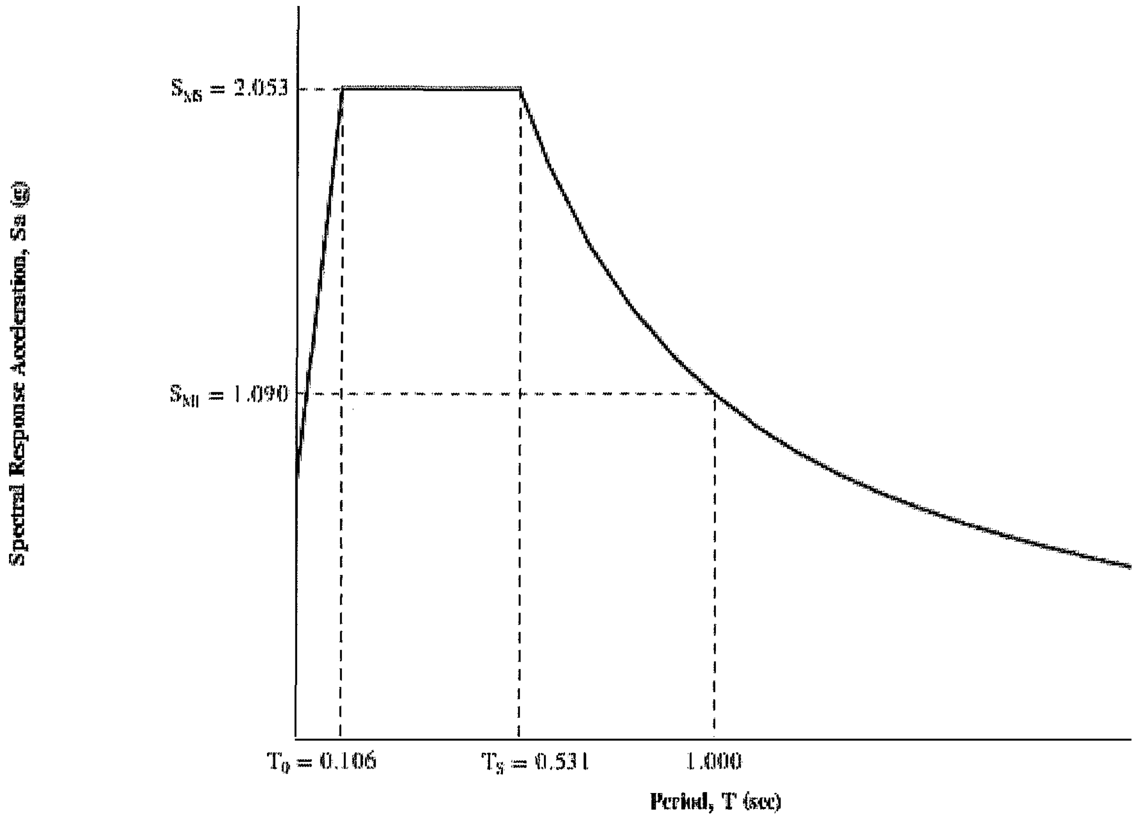
From Figure 22-12 [3] $T_L = 8$ seconds

Figure 11.4-1: Design Response Spectrum



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

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



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

From Figure 22-7 [4] PGA = 0.768

Equation (11.8–1): $PGA_M = F_{PGA}PGA = 1.000 \times 0.768 = 0.768$ g

Table 11.8–1: Site Coefficient F_{PGA}

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

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

For Site Class = D and PGA = 0.768 g, F_{PGA} = 1.000

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

From Figure 22-17 [5] C_{RS} = 0.926

From Figure 22-18 [6] C_{R1} = 0.936

Section 11.6 — Seismic Design Category

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

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

For Risk Category = I and S_{DS} = 1.369 g, Seismic Design Category = D

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

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

For Risk Category = I and S_{D1} = 0.727 g, Seismic Design Category = D

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

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

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

References

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

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

Appendix D

Liquefaction Evaluation

Input Data in Shaded Areas

Client Number	**4844**	**Client Name**	**C.I. Beach Community Services Dist.**
Date Drilled	**7/23/18**	**Boring**	**B-1**

a_{max}/g	0.768		N Adjustments - Liners (SPT Samples)	1.20	Field Groundwater Depth (ft)	9	(Current)
Magnitude	6.90		N Adjustments - Hole Diameter	1.00	Method (S = SPT)	S	
Groundwater Depth (ft)	5.0	(Historic High)	N Adjustments - Energy	1.33	Unit Weight of Water (kcf)	0.0624	
Reference Pressure, $p_{a\,(liq)}$	2.1164		Nc	10.08			
Reference Pressure, $p_{a\,(dry)}$	1.0582						

NL = Not Susceptible to Liquefaction

B1 Liquefaction Evaluation

Depth, Feet	Total Unit Weight, γ_t	Overburden Pressure, σ_v	LIQ Effective Overburden Pressure, σ_v'	Field Effective Overburden Pressure, σ_v'	C_N	r_d	$CSR_{M=7.5}$	Soil Type*	% Fines	N	$(N_1)_{60}$	Adjusted for Fines Content $(N_1)_{60}$	Rod Length Adjust	K_s	$CRR_{M=7.5}$	Safety Factor, SPT Method	Volumetric Strain	Layer Settlement, (inches)	Cumulative Liquefaction Settlement, (inches)
0.0		0.00	0.00	0.00															
1.5	0.125	0.19	0.19	0.19	1.70	1.00	0.403			50.0	101.7	101.7	0.75	1.00	5.000	Above GWT	0.000	0.000	0.000
3.0		0.38	0.38	0.38															
4.0	0.125	0.50	0.50	0.50	1.70	0.99	0.400			19.4	39.6	39.6	0.75	1.00	5.000	Above GWT	0.000	0.000	0.000
5.0		0.63	0.63	0.63															
6.3	0.125	0.78	0.70	0.78	1.65	0.99	0.442			16.1	31.7	31.7	0.75	1.00	0.387	NL	0.000	0.000	0.000
7.5		0.94	0.78	0.94															
8.8	0.125	1.09	0.86	1.09	1.39	0.98	0.504			8.9	16.8	16.8	0.85	1.00	0.182	0.36	0.018	0.549	0.549
10.0		1.25	0.94	1.19															
12.5	0.125	1.56	1.09	1.34	1.25	0.97	0.543			11.7	19.9	19.9	0.85	1.00	0.216	0.40	0.016	0.972	1.521
15.0		1.88	1.25	1.50															
16.3	0.125	2.03	1.33	1.58	1.16	0.97	0.571		3.3	24.0	42.1	42.1	0.95	1.00	5.000	NL	0.000	0.000	1.521
17.5		2.19	1.41	1.66															
18.8	0.125	2.34	1.49	1.74	1.10	0.96	0.582		3.3	24.0	40.2	40.2	0.95	1.00	5.000	NL	0.000	0.000	1.521
20.0		2.50	1.56	1.81															
21.3	0.125	2.66	1.64	1.89	1.06	0.95	0.589		6.5	21.0	33.7	34.0	0.95	1.00	0.502	NL	0.000	0.000	1.521
22.5		2.81	1.72	1.97															
23.8	0.125	2.97	1.80	2.05	1.02	0.95	0.593		6.5	21.0	32.4	32.6	0.95	1.00	0.435	NL	0.000	0.000	1.521
25.0		3.13	1.88	2.13															
26.3	0.125	3.28	1.96	2.20	0.98	0.94	0.596		8.6	54.0	84.4	86.2	1.00	1.00	5.000	NL	0.000	0.000	1.521
27.5		3.44	2.03	2.28															
28.8	0.125	3.59	2.11	2.36	0.95	0.93	0.596		8.6	54.0	81.6	83.3	1.00	1.00	5.000	NL	0.000	0.000	1.521
30.0		3.75	2.19	2.44															
31.3	0.125	3.91	2.27	2.52	0.92	0.91	0.595		59.8	18.0	26.3	36.6	1.00	1.00	5.000	NL	0.000	0.000	1.521
32.5		4.06	2.35	2.60															
33.8	0.125	4.22	2.42	2.67	0.89	0.90	0.593		59.8	18.0	25.6	35.7	1.00	0.99	5.000	NL	0.000	0.000	1.521
35.0		4.38	2.50	2.75															
36.3	0.125	4.53	2.58	2.83	0.86	0.88	0.591		11.6	23.0	31.7	34.1	1.00	0.98	0.499	NL	0.000	0.000	1.521
37.5		4.69	2.66	2.91															
38.8	0.125	4.84	2.74	2.99	0.84	0.86	0.587		11.6	23.0	30.9	33.2	1.00	0.97	0.451	NL	0.000	0.000	1.521
40.0		5.00	2.82	3.07															
41.3	0.125	5.16	2.89	3.14	0.82	0.84	0.583		59.1	37.0	48.5	63.1	1.00	0.96	5.000	NL	0.000	0.000	1.521
42.5		5.31	2.97	3.22															
43.8	0.125	5.47	3.05	3.30	0.80	0.82	0.578		59.1	37.0	47.3	61.7	1.00	0.96	5.000	NL	0.000	0.000	1.521
45.0		5.63	3.13	3.38															
46.3	0.125	5.78	3.21	3.46	0.78	0.79	0.573		54.1	12.0	15.0	23.0	1.00	0.95	0.242	0.42	0.014	0.410	1.931
47.5		5.94	3.29	3.54															
48.8	0.125	6.09	3.36	3.61	0.77	0.77	0.568		54.1	12.0	14.7	22.6	1.00	0.94	0.235	0.41	0.014	0.418	2.349
50.0		6.25	3.44	3.69															
																		total =	2.35

* C indicates clay or other non-liquefiable, fine grained soils (based on hydrometer and/or Atterberg testing), otherwise assumed to be potentially liquefiable.

Appendix E
References

The following list includes the citations of references referred to in this report.

American Society for Testing and Materials (2013), **ASTM Standards**, Section 4: Construction, Volume 04.08 Soil and Rock (I): D 420 – D 5611, West Conshohocken, PA.

Bartlett, S. F. and Youd, T. L. (2002), *Revised Multilinear Regression Equations for Prediction of Lateral Spread Displacement*, **Journal of Geotechnical and Geoenvironmental Engineering**, Vol. 128, No. 12, ASCE, ISSN 1090-0241 / 2002/12-1007-1017, December 1, 2002.

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California Geological Survey (2002), **Seismic Hazard Zone Report for the Oxnard 7.5-Minute Quadrangle, Ventura County, California**, Seismic Hazard Zone Report 052.

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International Conference of Building Officials and California Building Standards Commission (2016), **2016 California Building Code**.

Robertson, P. K. and Wride, C. E. (1997), *Cyclic Liquefaction and Its Evaluation based on the SPT and CPT*, **Proceedings of the NCEER Workshop on Evaluation of Liquefaction Resistance of Soils,** National Center for Earthquake Engineering Research, Technical Report NCEER-97-0022, pp. 41 – 87.

Southern California Earthquake Center (1999), **Recommended Procedures for Implementation of DMG Special Publication 117, Guidelines for Analyzing and Mitigating Liquefaction Hazards in California**, Martin, G. R. and Lew, M. Co-Chairs and Editors, University of Southern California, March 1999.

Tokimatsu, K. and Seed, H. B. (1987), *Evaluation of Settlements in Sands Due to Earthquake Shaking*, **Journal Geotechnical Engineering**, ASCE, Vol. 113, No. 8, pp. 861-878.

United States Geological Survey, (2014), **Seismic Design Maps** web program, last modified March 10, 2014
http://earthquake.usgs.gov/designmaps/us/application.php

Youd, T. L. and Idriss, I. M. (1997), *Summary Report*, **Proceedings of the NCEER Workshop on Evaluation of Liquefaction Resistance of Soils,** National Center for Earthquake Engineering Research, Technical Report NCEER-97-0022, pp. 1 – 40.

Youd et al (2001), Liquefaction Resistance of Soils: Summary Report from the 1996 and 1998 NCEER/NSF Workshop on Evaluation of Liquefaction Resistance of Soils, **Journal Geotechnical Engineering**, ASCE, Vol. 127, No. 10, pp. 817-833.

Appendix F

Well Abandonment Documents

County of Ventura

WELL PERMIT APPLICATION

800 South Victoria Avenue, Ventura CA 93009 Permit No. 5331

NAME OF WELL OWNER	OWNER MAILING ADDRESS (STREET, CITY, ZIP)	
Channel Islands Beach Community Services	353 Santa Monica Dr. Oxnard CA	
OWNER TELEPHONE NUMBER	NAME OF WELL DRILLER	LICENSE NUMBER
(805) 985-6021	Ike's Pump & Drilling Inc.	670-983

I hereby agree to comply with all regulations pertaining to well construction, repair, modification and destruction. Within 30 days of completion of work, I will furnish the Ventura County Public Works Agency, Water Resources and Engineering Department, Water Resources Division with a complete and accurate log of the well. Any modification of this permit requires approval by the Director. Call (805) 654-2907 or 654-2904. ***When signed by the Director of Public Works, this application is a permit.***

Owner's Signature ______ Date ______

Driller's Signature [signature] Date 1-27-03 (805) 485-5912

Estimated Dates of Work: Start 1-29-03 Completion ______ FAX: (805) 983-3050

TYPE OF WORK *(Check)*	USE *(Check)*	EQUIPMENT *(Check)*	WELL DEPTH	PROPOSED CASING
Water Supply Well ☒	Public ☒ Domestic ☐	Air Rotary ☐	250' FEET	Steel ☒
Repair or Modification ☐	Agricultural ☐	Mud Rotary ☐	DIAMETER WELL BORE	PVC ☐
Destruction ☒	LUFT Invest/Clean-up ☐	Hollow Stem ☐	12" INCHES	Other ______ ☐
Monitoring (No. ______) ☐	Site Assess/Leak Detect ☐	Cable Tool ☒		Diameter 12"
Borehole (No. ______) ☐	Other ______ ☐	Other ______ ☐		Wall or Gage ______

PROPOSED SEALING ZONE(s)	SEALING MATERIAL *(Check)*	PERFORATIONS OR SCREEN
From T.D. to 5' Ft with Neat Cement	Neat Cement ☒ Bentonite Chips ☐	From 10' to 160' Ft
From ______ to ______ Ft with ______	Cement Grout ☐ Concrete ☐	From ______ to ______ Ft
From ______ to ______ Ft with ______	*(10 Sack/Yd³ Mix)* *(6 Sack/Yd³ Mix)*	From ______ to ______ Ft

FOR OFFICE USE

Permit Start Date 01/28/03 $ 610.00 Fee Paid On 01/14/03 Receipt No. 5295

Expiration Date 07/28/03 Date Sealed 2/4/03 Inspector's Signature MSD

Disposition of Application: ☒ Approved with conditions below ☐ Denied

Agency: ☐ GMA ☐ UWCD ☐ City ______ ☐ Other ______

Permit conditioned and checked by Barbara Chinn **Date** 01/28/03

CONDITIONS FOR WELL DESTRUCTION:

1. Pump, motor, debris, pollutants, and contaminants, including oil from oil-lubricated pumps, shall be removed from the well.
2. Well casing shall be perforated over the depth interval of 240 ft to 210 ft. and from 160 ft. to within 10 ft. of finish grade. Perforations shall be one hole per foot, placed on alternating sides of the casing.
3. Neat cement sealing material shall be applied from the total depth of the well to within 5 ft. of finish grade by means of a grout pipe placed within 2 ft. of the base of the sealing zone.
4. The Contractor shall retain all discharges within the drilling site. No fluids shall drain offsite to flood control channels, creeks, rivers, sewers or any watercourses.
5. Casing shall be removed to a depth of 5 ft. below finish grade, and work area backfilled with native materials.
6. Public Works Inspector shall be present during casing perforation work and placement of all sealing material. **(NOTE: 24-hour advance notice is required: call (805) 654-2904 or 654-2024.)**
7. All work shall be performed by a well contractor licensed in the State of CA and registered with the County.

Public Works Director Larry Yamamoto Date 1/28/03

VA-86-000003A (Rev.12/98)

Permit 5331 SWN 01N22W18P01S

County of Ventura

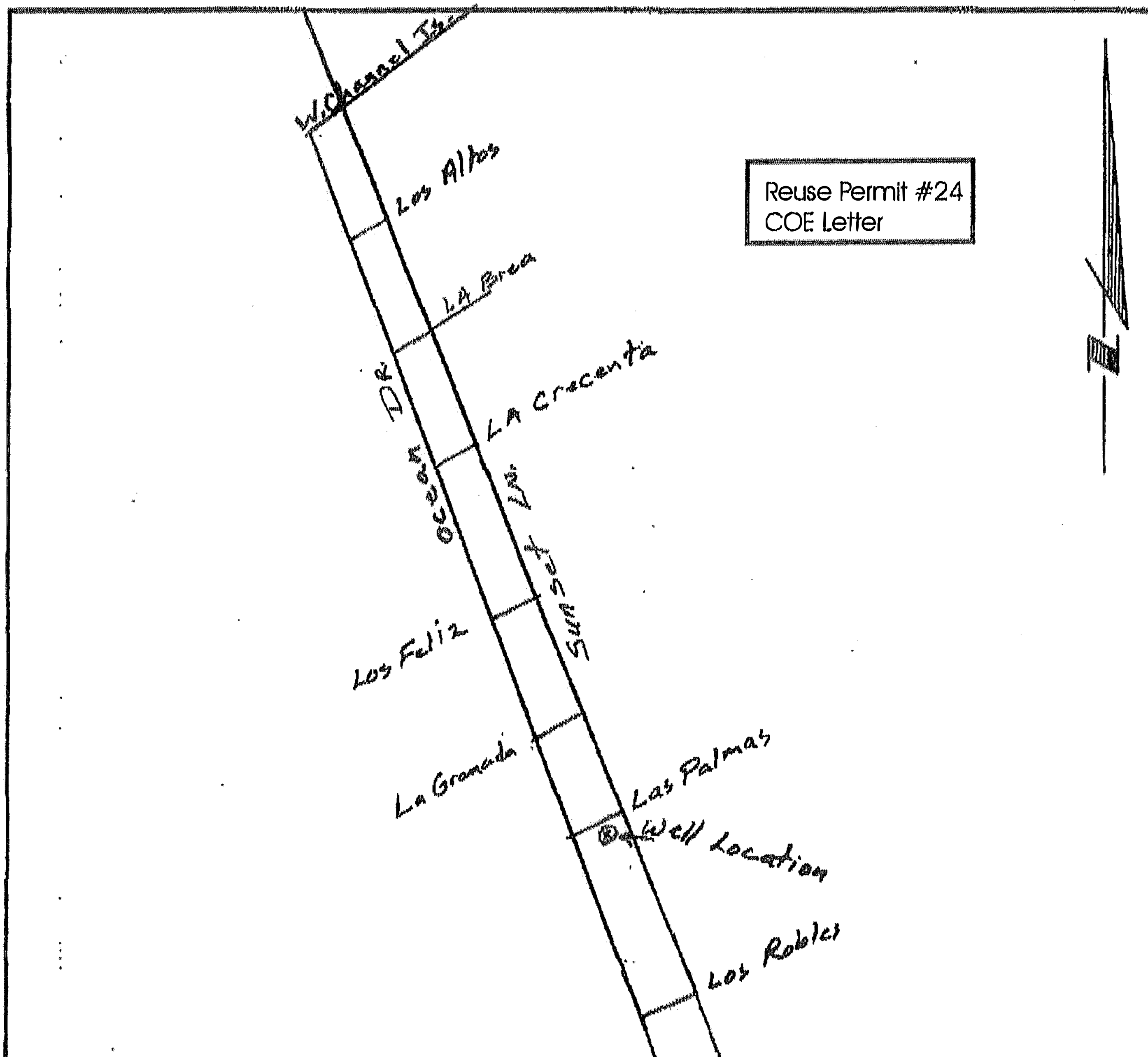
WELL PERMIT APPLICATION

800 South Victoria Avenue, Ventura CA 93009

Permit No. 5331

LOCATION

INDICATE BELOW THE EXACT LOCATION OF WELL WITH RESPECT TO THE FOLLOWING ITEMS: PROPERTY LINES, WATER BODIES OR WATER COURSES, DRAINAGE PATTERN, ROADS, EXISTING WELLS, SEWERS AND PRIVATE SEWAGE DISPOSAL SYSTEMS. INCLUDE DIMENSIONS. LIST ASSESSOR'S PARCEL NUMBER AND THOMAS BROS. GUIDE NUMBER.



Assessor's Parcel Number 206 - 0 - 231 - 030 Thomas Bros. Guide 552-A3

FOR OFFICE USE

01N22W18P01S STATE WELL NUMBER | 6343 QUAD NUMBER | 5331 PERMIT NUMBER

WATER WELL SEALING RECORD

SP SN34° 09' 52.8"
W 119° 13' 39.5"

PERMIT # 5331

START DATE 1 / 28 / 03

EXPIRATION DATE 7 / 28 / 03

☐ NEW WELL ☑ DESTRUCTION ☐ OTHER ____

TYPE OF MATERIAL USED Neat cement

WELL #	DELIVERED TO SITE Cu. Yd.	LEFT OVER Cu. Yd.	USED FOR SEALING Cu. Yd.	BOREHOLE Diameter (NEW WELLS)	WELL CASING Diameter	DEPTH OF SEAL FROM	DEPTH OF SEAL TO
☐ 1st load: ☐ MIX ON SITE	7.0	0	7.0	—	12	228'	5'
☐ 2nd load: ☐ MIX ON SITE	7.0	2 ~~[illegible]~~	5.0	—			
☐ ____ ☐ MIX ON SITE			~~11.5~~ 13.0				

METHOD OF SEAL PLACEMENT: ☑ GROUT PIPE ☐ DROP ☐ OTHER ____

NUMBER OF GROUT PIPE SECTIONS 11 LENGTH OF EACH SECTION 20' FEET

(DESTRUCTION ONLY)

CONFIRMATION THAT THE CASING WAS RIPPED OR PERFORATED AS REQUIRED BY THE PERMIT.

- Could only get knife to 225'. Perforated from 225-210'. Change to conditions verbally approved by G. Luscon.
- Perforated from 160' - 10' bgs as specified, in addition to perfs from 225'-210'.

REMARKS:
- Onsite 1010 - Cap still on well, requiring cutting of concrete to remove
- Cap removed 1100.
- Difficulty getting knife to depth. Started perforating at ≈ 1300 - Completed perfs @ ≈ 1430
- Sounded depth before perforations = 227'. After perforations 222'.
- Pump on site 1430. 1st truckload of cement @ 1445. 2nd truckload started pumping 1520.

DESCRIBE ANY VARIANCE IN THE SEALING METHOD OR MATERIAL FROM PERMIT CONDITIONS, OR ANY OTHER FACTOR WHICH, IN YOUR ESTIMATION, MIGHT HAVE CAUSED THE SEALING OPERATION TO BE LESS THAN SATISFACTORY ____

IN MY OPINION, THE WELL SEALING WAS:

☑ SATISFACTORY

☐ UNSATISFACTORY FOR REASONS DESCRIBED ABOVE

INSPECTION SERVICES

START 1010

COMPLETED 1540

OPTION:

☑ ATTACHED PHOTO OF SITE AND IMMEDIATE VICINITY

☑ ATTACHED CEMENT TRUCK REPORT

☐ OTHER ____

DATE SEALED: 2 / 4 / 03

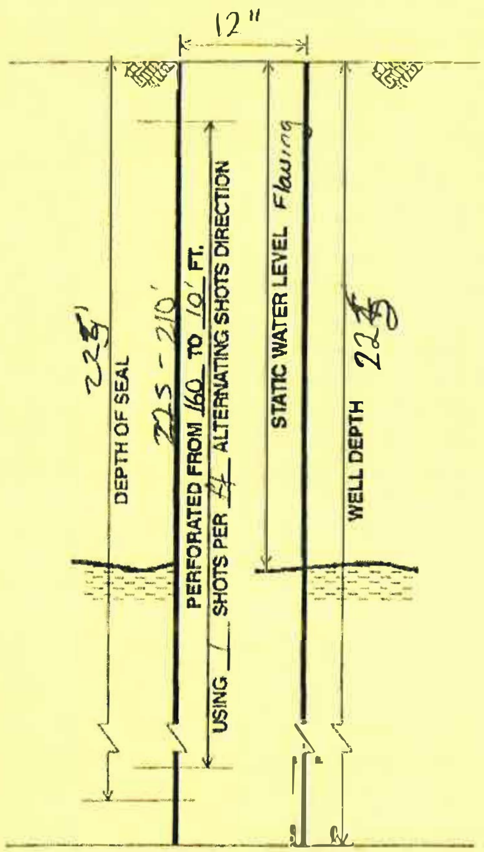
MAK [signature]
INSPECTOR

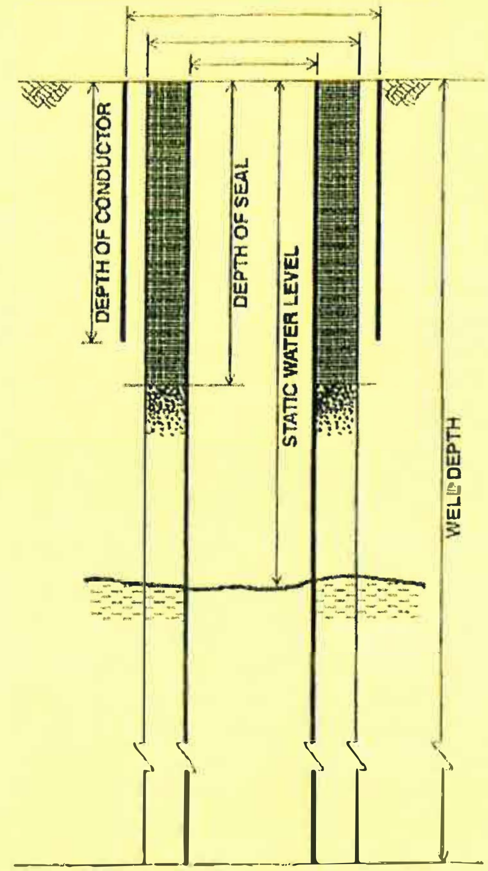
2/4/03
DATE

INSPECTION NOTES

PERMIT # S331

☑ DESTRUCTION	☐ NEW WELL





QUANTITIES OF		WATER	CEMENT	BENTONITE	SAND	GRAVEL	CLAY
☐ NEAT CEMENT	(CEMENT SLURRY): CEMENT + WATER				—	—	—
☐ CEMENT GROUT:	CEMENT + WATER + SAND					—	—
☐ CONCRETE:	CEMENT + WATER + SAND + GRAVEL						—
☐ CLAY PELLETS:	SOMETIMES USED AS A SEAL BETWEEN GRAVEL SURROUNDING PERFORATIONS AND CONCRETE SEAL IN SHALLOW (MONITORING) WELLS						

CEMEX

CEMXE FINANCIAL SERVICES, L.P.
430 N. Vineyard Ave., Suite 500 • Ontario, CA 91764-4463
(909) 974-5500 • Fax (909) 974-5524

TICKET NO. 109278

1:49PM

SANTA PAULA PLANT 748915

ORDER DESK
(800) 966-7796

Azusa (21) 1201 W. Gladstone	**Irvine (09)** 16161 Construction Circle E	**S. J. Capistrano (01)** 31601 Ortega Hwy	**Walnut (04)** 20903 Courier Rd.
Compton (05) 2722 N. Alameda St.	**Los Angeles (14)** 625 Lamar	**Santa Paula (44)** 1430 Santa Clara	
Hollywood (12) 1000 N. La Brea	**Moorpark (42)** 9035 Roseland Ave.	**Simi Valley (43)** 300 W. Los Angeles Ave.	
Inglewood (06) 505 Railroad Pl.	**Orange (31)** 1730 N. Main St.	**Sun Valley (10)** 8960 Bradley	

DATE	LEAVE PLANT	ARRIVE JOB	START UNLOAD	FINISH UNLOAD	LEAVE JOB	ARRIVE PLANT	STANDING TIME
Feb03	153	2:13	247	310			

PLANT/ALLEY	SALESMAN	ORDER NO.	ENTERED BY	APPLICATION
44				WELL CAP

CEMENT	FLYASH	W/C SAND	3/8" ROCK	1" ROCK	1 1/2" ROCK	LWT SAND	CU FT LWT AGG

GALS. FREE MOIST	GALS. ADDED	MAX GALS. ALLOW	GALS. ADDED ON JOB (BEGIN / MIDDLE / END)	DRUM REVOLUTIONS (BEGIN / MIDDLE / END)	SLUMP / CYL (BEGIN / MIDDLE / END)	SLUMP METER
						500

CUSTOMER P.O.	CUSTOMER JOB NO.	ZONE	MAP PAGE	TIME	TRUCK	DRIVER
		VC	VC552A4	14:45	2194	RICK WEAVER

CUSTOMER NAME	CUSTOMER NO.	SPECIAL INSTRUCTIONS
KES PUMP & DRILLING INC.	80053	ON LAS PALMAS SOUTH SIDE 805-797-4238

DELIVERY ADDRESS	PROJECT NO.
OXNARD, LAS PALMAS & OCEAN	

CROSS STREETS	TOTAL ORDERED	TOTAL SHIPPED
	14.00	7.00

LOAD #	QUANTITY	UOM	MIX NO.	SLUMP	PRODUCT DESCRIPTION	UNIT PRICE	EXTENSION
1	7.00	CY	[illegible]		NEAT CEMENT		

Aggregate Target Actual Status

2/4/03 1st truckload.

CEMENT 12852 LB 12[illegible] In Tol MSD

WATER *1 666 Gal 666 In Tol

2/4/03 - 2nd truck load MSD

ENVIRONMENTAL	
BALANCE FORWARD	
MINIMUM LOAD	
SALES TAX	
SUB-TOTAL	
STANDING TIME	
TOTAL	

❑ CHARGE ❑ COD

WEIGHMASTER CERTIFICATE

THIS TO CERTIFY that the following described commodity was weighed, measured or counted by a weighmaster, whose signature is on this certificate, who is a recognized authority of accuracy, as prescribed by Chapter 7 (commencing with Section 12700) of Division 5 of the California Business and Professions code, administered by the Division of Measurement Standards of the California Department of Food and Agriculture.

Cemex
Deputy MIGUEL MARCADO
X

STANDING TIME: Customer is allowed 4 minutes per cu. yd. unloading time from arrival at job to finish unloading. Excess time will be charged at the prevailing rate per hour or any part thereof.

REASON FOR DELAY TIME:
❑ JOB BROKE DOWN ❑ JOB NOT READY ❑ ADDING WATER
❑ TRUCK BROKE DOWN ❑ SLOW PLACING ❑ ARRIVED EARLY
OTHER EXPLAIN

Received by and terms agreed to X

CX04514 (4/01)

INSPECTOR

CEMEX

CEMXE FINANCIAL SERVICES, L.P.
430 N. Vineyard Ave., Suite 500 • Ontario, CA 91764-4463
(909) 974-5500 • Fax (909) 974-5524

ORDER DESK
(800) 966-7796

Azusa (21) 1201 W. Gladstone	**Irvine (09)** 16161 Construction Circle E.	**S. J. Capistrano (01)** 31601 Ortega Hwy	**Walnut (04)** 20903 Courier Rd.
Compton (05) 2722 N. Alameda St.	**Los Angeles (14)** 625 Lamar	**Santa Paula (44)** 1430 Santa Clara	
Hollywood (12) 1000 N. La Brea	**Moorpark (42)** 9035 Roseland Ave.	**Simi Valley (43)** 300 W. Los Angeles Ave.	
Inglewood (06) 505 Railroad Pl.	**Orange (31)** 1730 N. Main St.	**Sun Valley (10)** 8960 Bradley	

TICKET NO 092800

SANTA PAULA PLANT 748916

DATE	LEAVE PLANT	ARRIVE JOB	START UNLOAD	FINISH UNLOAD	LEAVE JOB	ARRIVE PLANT	STANDING TIME
	1425	1500	1515	1530			

PLANT/ALLEY	SALESMAN	ORDER NO	ENTERED BY	APPLICATION

CEMENT	FLYASH	W/C SAND	3/8" ROCK	1" ROCK	1 1/2" ROCK	LWT SAND	CU FT LWT AGG

GALS. FREE MOIST	GALS. ADDED	MAX GALS. ALLOW	GALS. ADDED ON JOB (BEGIN / MIDDLE / END)	DRUM REVOLUTIONS (BEGIN / MIDDLE / END)	SLUMP ☐ CYL ☐ SLUMP ☐ CYL ☐ SLUMP ☐ CYL ☐ (BEGIN / MIDDLE / END)	SLUMP METER

CUSTOMER P.O.	CUSTOMER JOB NO.	ZONE	MAP PAGE	TIME	TRUCK	DRIVER

CUSTOMER NAME	CUSTOMER NO.	SPECIAL INSTRUCTIONS
P & DRILLING INC.		SOUTH SIDE 805-797-4238

DELIVERY ADDRESS	PROJECT NO.
PALMAS & OCEAN	

CROSS STREETS	TOTAL ORDERED	TOTAL SHIPPED

LOAD #	QUANTITY	U OM	MIX NO.	SLUMP	PRODUCT DESCRIPTION	UNIT PRICE	EXTENSION

Aggregate Target Actual Status

2/4/03 - 2nd truck load.

MSD

CEMENT 12852 LB 12830 In Tol

WATER *1 666 Gal 665 In Tol

ENVIRONMENTAL	
BALANCE FORWARD	
MINIMUM LOAD	
SALES TAX	
SUB-TOTAL	
STANDING TIME	
TOTAL	

☐ CHARGE ☐ COD

WEIGHMASTER CERTIFICATE

THIS TO CERTIFY that the following described commodity was weighed, measured or counted by a weighmaster, whose signature is on this certificate, who is a recognized authority of accuracy, as prescribed by Chapter 7 (commencing with Section 12700) of Division 5 of the California Business and Professions code, administered by the Division of Measurement Standards of the California Department of Food and Agriculture.

Weighmaster MIGUEL MERCADO
Deputy

X

STANDING TIME: Customer is allowed 4 minutes per cu. yd. unloading time from arrival at job to finish unloading. Excess time will be charged at the prevailing rate per hour or any part thereof.

REASON FOR DELAY TIME:
☐ JOB BROKE DOWN ☐ JOB NOT READY ☐ ADDING WATER
☐ TRUCK BROKE DOWN ☐ SLOW PLACING ☐ ARRIVED EARLY
OTHER EXPLAIN:

Received by and terms agreed to X SYKES PUMP

CX04514 (4/01)

INSPECTOR

Unit 5331 - 2/4/03 MSO

↑ Location shot

↑ Perforating casing

↑ Mills Knife

↑ Pumping 1st truckload of cement

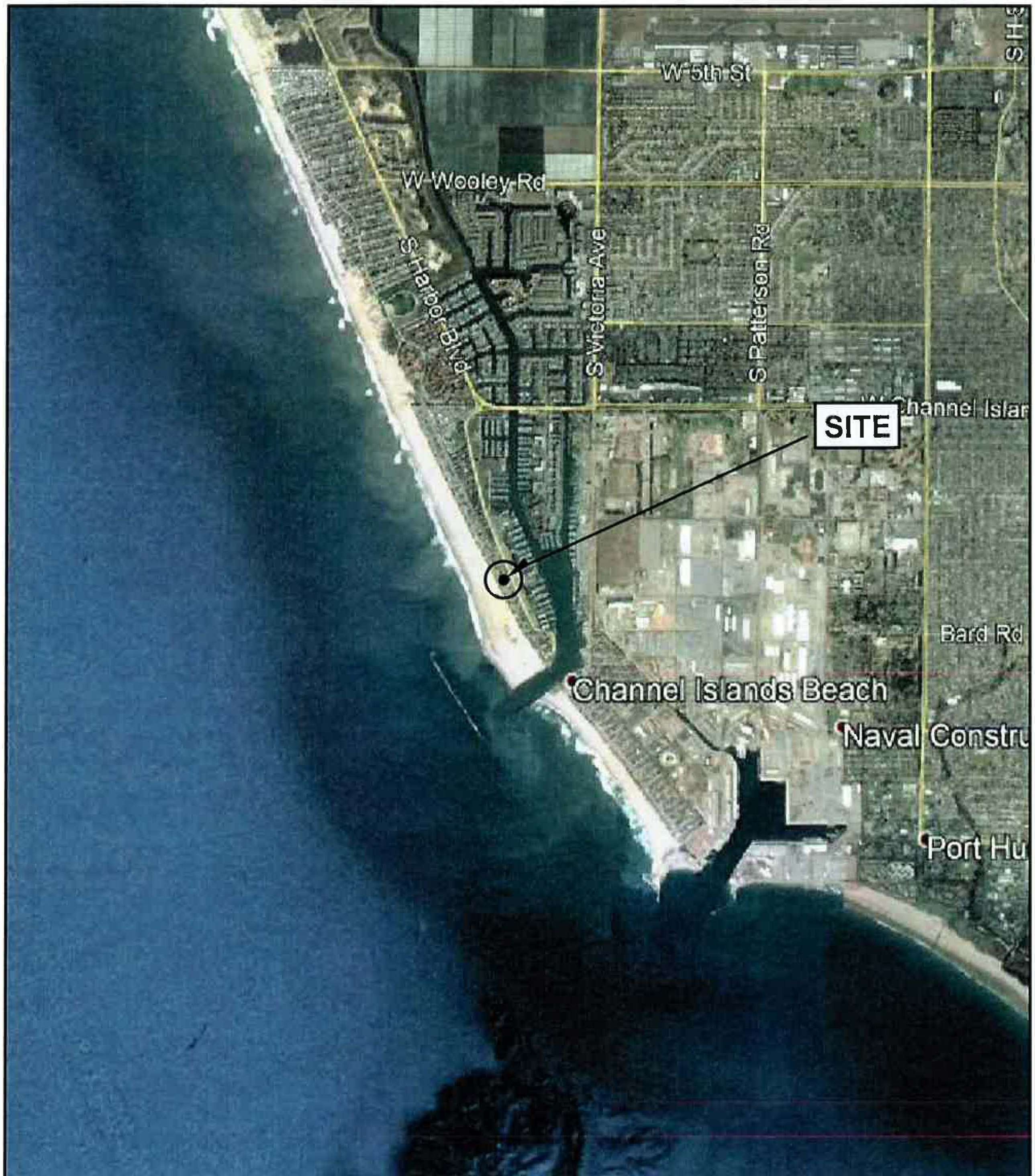
Permit 5331 - 2/4/03 MSD

↑ Pumping 2nd truckload of cement

↑ Completed seal - cement at top of casing, under ~12" water.

Appendix G

Report Figures



Reference: Google Earth 2018

No Scale

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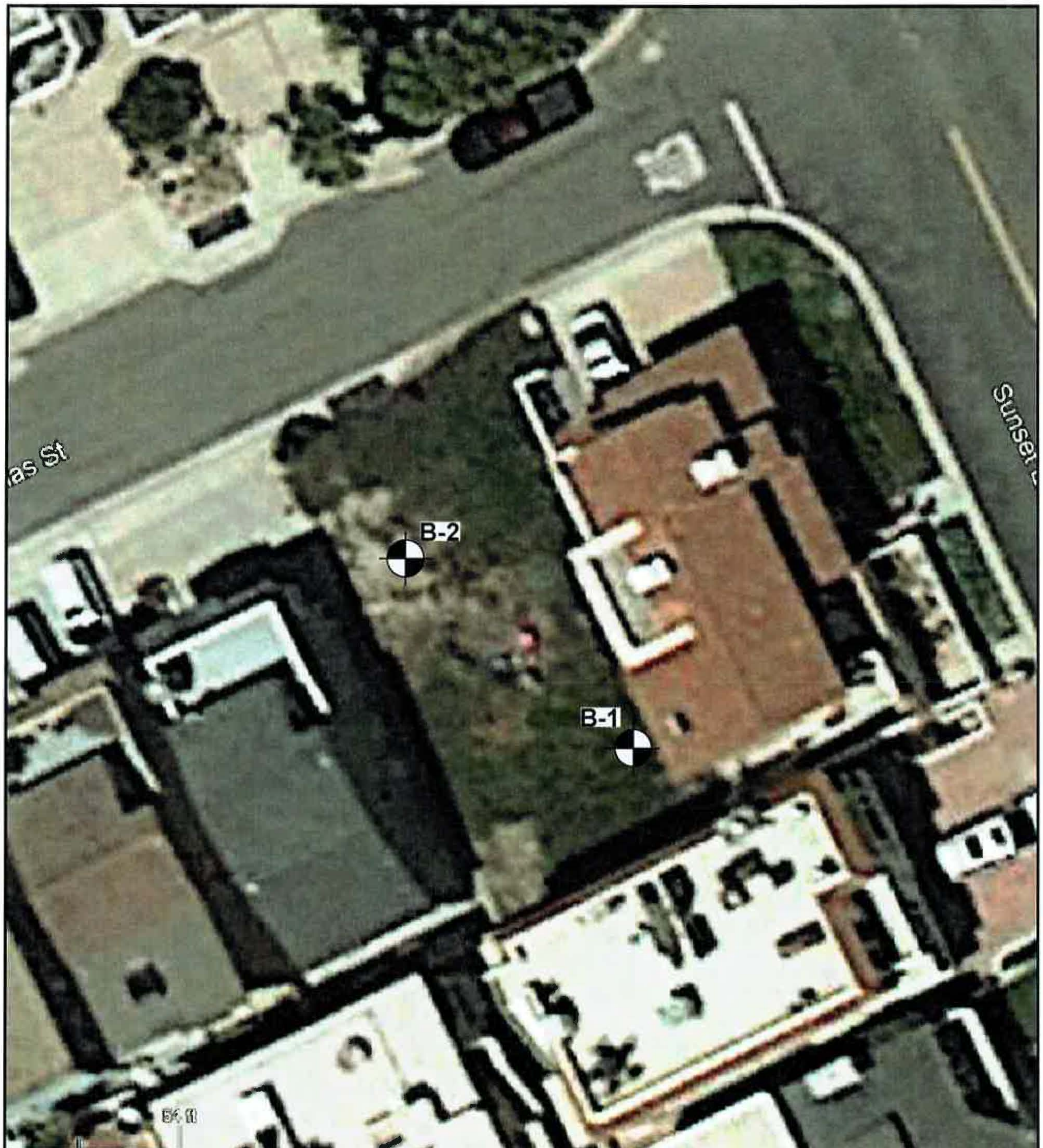
SITE LOCATION MAP

C.I. BEACH
112 Las Palmas Steet
Oxnard, California

Client # 4844
Report # 10139

FIGURE 1



EXPLANATION

B-2 Approximate Location of Exploratory Boring

N

Scale: 1" = 20'

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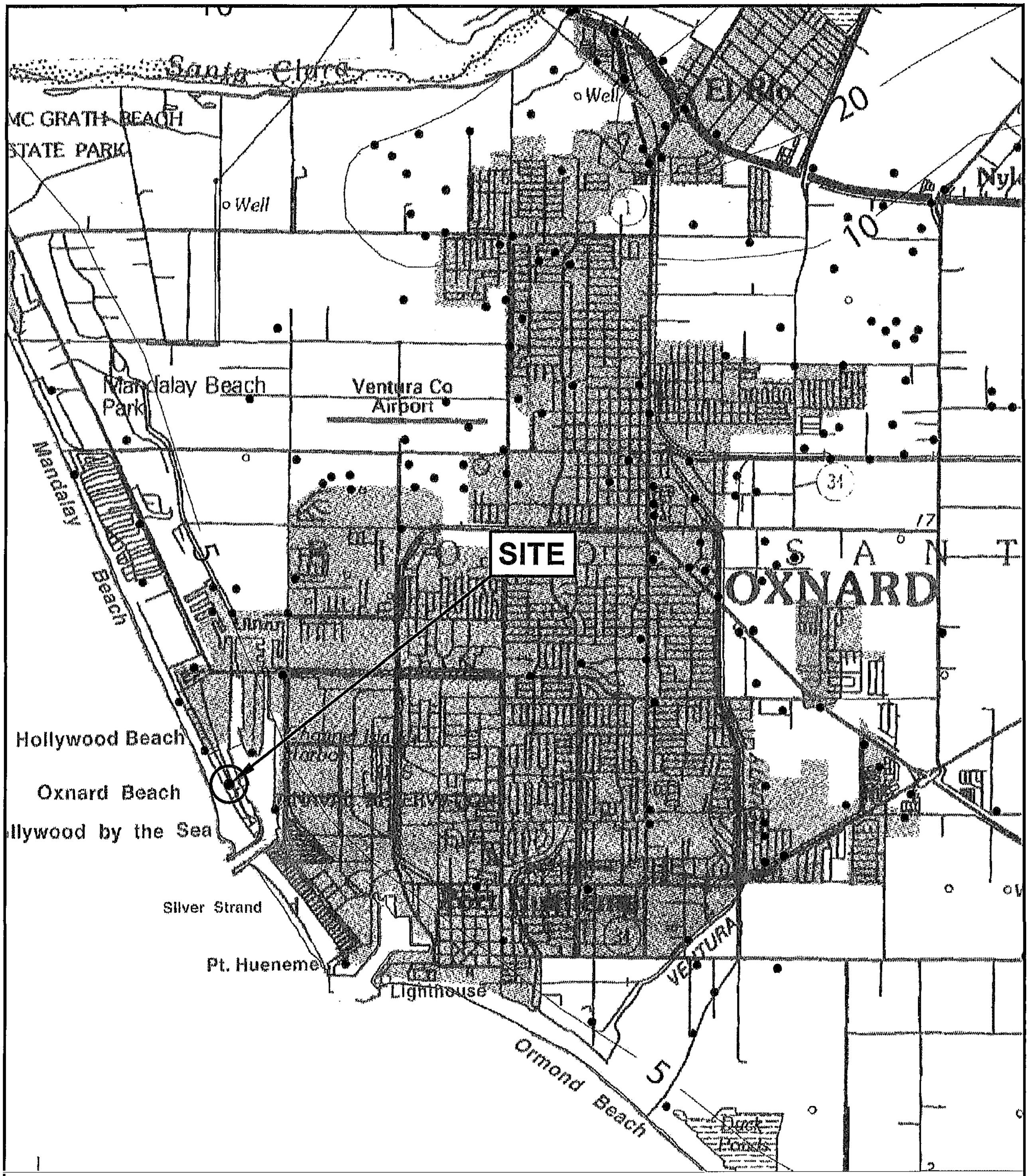
EXISTING SITE PLAN

C.I. BEACH
112 Las Palmas Steet
Oxnard, California

Client # 4844
Report # 10139

FIGURE 2



Reference: CDMG, 2002, Seismic Hazard Zone Report 052

Advanced Geotechnical Services	**DEPTH TO HISTORICALLY HIGH GROUNDWATER**	C.I. BEACH 112 Las Palmas Steet Oxnard, California
		Client # 4844 Report # 10139 — FIGURE 3

Reference: CDMG, 2002, Seismic Hazard Zones - Oxnard Quadrangle

Scale: 1" = ¼ mile

Advanced Geotechnical Services	SEISMIC HAZARD ZONES MAP	C.I. BEACH 112 Las Palmas Steet Oxnard, California	
		Client # 4844 Report # 10139	FIGURE 4