EnGEN CORPORATION

GEOTECHNICAL/GEOLOGICAL ENGINEERING STUDY

Zolfaghari Commercial
Assessors Parcel Number: 326-250-003
Clinton Keith Road and George Avenue
City of Murrieta, County of Riverside, California
Project Number: M3551-GS

April 6, 2007

Prepared for:
Mr. Reza Zolfaghari
c/o Mr. Reza Kassraian
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Laguna Niguel, California 92677

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Regarding: **GEOTECHNICAL / GEOLOGICAL ENGINEERING STUDY**
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Dear Mr. Zolfaghari:

According to your request and signed authorization, we have performed a Geotechnical/Geological Engineering Study for the subject project. The purpose of this study was to evaluate the existing geologic and geotechnical conditions within the subject property with respect to recommendations for rough grading of the site and design recommendations for foundations, slabs-on-grade, etc., for the proposed development. Submitted, herewith, are the results of this firm's findings and recommendations, along with the supporting data.

1.0 EXECUTIVE SUMMARY

A geotechnical/geological study of the subsurface conditions of the subject site has been performed for the proposed development. Exploratory excavations have been completed and earth material samples subjected to laboratory testing. The data has been analyzed with respect to the project information furnished to us for the proposed development. It is the opinion of this firm that the proposed development is feasible from a geotechnical/geological standpoint, provided that the recommendations presented in this report are implemented in the design and construction of the project.

The site primarily consists of shallow undocumented fill and alluvium overlying Pauba Formation bedrock. Some undocumented fill occurs on-site in the form of disked or tilled agricultural soils expected across the site in the upper 2 to 3 feet below ground surface (bgs). In addition, there was one backfilled and four open fault trenches with associated stockpiles on-site.

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The stockpiles and backfilled trench are considered undocumented fill material. The undocumented fill and shallow unsuitable alluvium should be removed to competent alluvium or bedrock according to the recommendations as provided in order to maintain tolerable settlement predictions. Anticipated removal depths to reach competent bottoms are on the order of 2 to 10-feet below existing ground surface, more specific recommendations based upon the Referenced No. 1 grading plans are discussed in section 8.2.5.

2.0 INTRODUCTION

2.1 **Authorization:** This report presents the results of the geotechnical engineering and engineering geology study performed on the subject site for the proposed development. Authorization to perform this study was in the form of a signed proposal.

2.2 **Scope of Work:** The scope of work performed for this study was designed to determine and evaluate the surface and subsurface conditions within the subject site with respect to its geotechnical characteristics, and to provide recommendations and criteria for use by the design engineer and architect for the development of the site and for design and construction of the proposed development. The scope of work included the following: site reconnaissance, surface geologic mapping; subsurface exploration; sampling of on-site earth materials; laboratory testing; engineering analysis of field and laboratory data; and the preparation of this report.

2.3 **Previous Site Studies:** A geologic study for on-site faulting was performed by Leighton Consulting, Inc. (Leighton), which identified on-site faulting and established a building setback zone. The documents produced as a result of this study, and provided to this firm include:

- **Leighton Consulting, Inc.**, *Geological Fault Hazard Investigation, Proposed ± 5-acre parcel, Assessor's Parcel Number 362-250-003, Wildomar, Riverside County, California*, prepared for Mr. Reza Zolfaghari, dated September 19, 2005.
- **Leighton Consulting, Inc.**, *Response to Review Comments, County of Riverside, Building and Safety Department, GEO Report No. 1524, Assessor's Parcel Number 362-250-003, Wildomar, County of Riverside, California,* Attention: Mr. Reza Zolfaghari, dated November 22, 2006.

- **Leighton Consulting, Inc.**, *2006, Fault Location Map, Geologic Location Fault Investigation, Assessor's Parcel Numbers 362-250-003 and 362-250-004*, Wildomar, California, Scale 1"=40', dated December 2005, Revised November 2006.

It is our understanding that the findings of the previous Fault Hazard Investigation were approved in the County of Riverside, Transportation and Land Management Agency, Planning Department letter dated January 5, 2007 and titled *Approval Comments, County Geologic Report No. 1524 (Fault Hazard), Geologic Fault Investigation, Proposed +/- 5 Acre Parcel, Assessor's Parcel Number 362-250-003, Wildomar, Riverside, California.* No additional geotechnical/geologic studies are known to have occurred for the subject site.

3.0 PROPOSED DEVELOPMENT / PROJECT DESCRIPTION

Grading plans were not available for review at the time of this study. The Referenced Fault Location Map provided proposed building footprints, finished floor elevations and site topography that were used to provide preliminary earthwork and foundation design recommendations. However the grading plans should be made available to this office for subsequent review so that additional recommendations may be prepared, if necessary. It is our understanding that the proposed development will consist of six (6) buildings to be one or two-story, concrete tilt-up, or block wall type structures with slab-on-grade foundations and associated landscape and hardscape improvements. It is assumed that relatively light loads will be imposed on the foundation soils. The foundation loads are not anticipated to exceed 2,000 pounds per lineal foot (plf) for continuous footings. The above project description and assumptions were used as the basis for the field and laboratory exploration and testing programs as well as the engineering analysis for the conclusions and recommendations presented in this report. This office should be notified if structures, foundation loads, grading, and/or details other than those represented herein are proposed for final development of the site so a review can be performed, supplemental evaluation prepared, and revised recommendations submitted, if necessary.

4.0 SITE DESCRIPTION

The subject site consists of approximately 5-acres located at the northeast corner of the intersection of Clinton Keith Road and George Avenue, in the City of Murrieta, County of Riverside, California. The site is comprised of rolling hills with regional drainage, primarily by seasonal drainages, generally flowing to the southwest. A concrete V-ditch parallels the eastern portion of the southern property line. The site was covered by a moderately thick amount of grasses, weeds, and brush at the time of our field study. It is our understanding that Clinton Keith Road may have been formerly aligned within roughly the southeastern most 200-feet, measured from the southeast corner and it is possible that related abandoned improvements may exist in this area of the site.

5.0 FIELD STUDY

Field reconnaissance and geologic mapping was conducted on March 13, 2007, by our Field Geologist. A study of the property's subsurface condition was performed to evaluate underlying earth strata and the presence of groundwater. Three (3) exploratory soil borings were excavated on the study site by **2 Bit Drilling** utilizing a CME 45 truck-mounted drill rig, equipped with 7.0-inch outside diameter continuous flight hollow-stem augers. The maximum depth explored on site was approximately 50.5-feet below the existing ground surface (bgs). Relatively undisturbed ring samples of the earth materials encountered were obtained at various depths in the exploratory borings. Bulk samples were collected from the soil borings. All soil samples were subsequently returned to our soils laboratory for verification of field classifications and testing. Bulk samples were obtained from cuttings developed during the excavation process and represent a mixture of the soils within the depth indicated on the logs. Relatively undisturbed samples of the earth materials encountered in the soil borings were obtained by driving a thin-walled steel sampler lined with 1.0-inch high, 2.416-inch inside diameter brass rings. The sampler was driven with successive drops of a 140-pound weight having a free fall of approximately 30-inches. The blow counts for each successive 6.0-inches of penetration, or fraction thereof, are shown in the Geotechnical Boring Logs presented in the Appendix. The ring samples were retained in close-fitting moisture-proof containers and returned to our laboratory for testing. The approximate locations of the soil borings are denoted on the Geotechnical Site Plan (Plate 1). The exploratory soil borings were backfilled with soil cuttings.

6.0 LABORATORY TESTING

6.1 **General:** The results of laboratory tests performed on samples of earth material obtained during the field investigation are presented in the Geotechnical Boring Logs in the Appendix. Following is a listing and brief explanation of the laboratory tests performed. The samples obtained during the field investigation will be discarded 30 days after the date of this report. This office should be notified immediately if retention of samples will be needed beyond 30 days.

6.2 **Classification:** The field classification of soil materials encountered in the geotechnical borings was verified in the laboratory, in general accordance with the Unified Soils Classification System, ASTM D 2488-00, Standard Practice for Determination and Identification of Soils (Visual-Manual Procedures).

6.3 **In-Situ Moisture Content and Density Test:** The in-situ moisture content and dry density were determined in general accordance with ASTM D 2216-98 and ASTM D 2937-00 procedures, respectively, for each selected undisturbed sample obtained. The dry density is determined in pounds per cubic foot and the moisture content is determined as a percentage of the oven dry weight of the soil.

6.4 **Maximum Dry Density / Optimum Moisture Content Relationship Test:** Maximum dry density/optimum moisture content relationship determinations were performed on samples of near-surface earth material in general accordance with ASTM D 1557-02 procedures using a 4.0-inch diameter mold. Samples were prepared at various moisture contents and compacted in five (5) layers using a 10-pound weight dropping 18-inches and with 25 blows per layer. A plot of the compacted dry density versus the moisture content of the specimens is constructed and the maximum dry density and optimum moisture content determined from the plot.

6.5 **Consolidation Test:** Settlement predictions of the on-site soil and compacted fill behavior under load were made, based on consolidation tests that were performed in general accordance with ASTM D 2435-03 procedures. The consolidation apparatus is designed to receive a 1.0-inch high, 2.416-inch diameter ring sample. Porous stones are placed in contact with the top and bottom of each specimen to permit addition and release of pore water and pore pressure. Loads normal to the face of the specimen are applied in several increments in a geometric progression under both field moisture and submerged conditions. The resulting changes in sample thickness are recorded at selected time intervals. Water was added to the test apparatus at various loads to create a submerged condition and to measure the collapse potential (hydroconsolidation) of the sample. The resulting change in sample thickness was recorded.

6.6 **Direct Shear Test (Remolded):** Direct shear tests were performed on selected samples of near-surface earth material in general accordance with ASTM D 3080-03 procedures. The shear machine is of the constant strain type. The shear machine is designed to receive a 1.0-inch high, 2.416-inch diameter ring sample. Specimens from the sample were sheared at various pressures normal to the face of the specimens. The specimens were tested in a submerged condition. The maximum shear stresses were plotted versus the normal confining stresses to determine the shear strength (cohesion and angle of internal friction).

6.7 **Expansion Test:** Laboratory expansion tests were performed on samples of near-surface earth material in general accordance with the California Building Code Standard (CBC 18-2). In this testing procedure, a remolded sample is compacted in two (2) layers in a 4.0-inch diameter mold to a total compacted thickness of approximately 1.0-inch by using a 5.5-pound weight dropping 12-inches and with 15 blows per layer. The sample is compacted at a saturation of between 49 and 51 percent. After remolding, the sample is confined under a pressure of 144 pounds per square foot (psf) and allowed to soak for 24 hours. The resulting volume change due to the increase in moisture content within the sample is recorded and the Expansion Index (EI) calculated.

6.8 **Soluble Sulfate Test:** Samples of near-surface earth material were obtained for soluble sulfate testing for the site. The concentration of soluble sulfates was determined in general conformance with California Test Method 417 procedures.

6.9 **pH/Minimum Resistivity:** Sample(s) of near surface soils were tested for pH and minimum resistivity in general conformance to CTM 643.

6.10 **Chloride Content:** Sample(s) of near surface soils were test4ed for chloride content in general conformance to CTM 422.

6.11 **R-Value Test:** An evaluation was performed on a selected representative soil sample in general accordance with California Test Method 301. The resistance (R-Value) test method is used to measure the potential strength of subgrade, subbase, and base course materials for use in road pavements.

7.0 ENGINEERING GEOLOGY

7.1 **Geologic Setting:** The site is located in the Northern Peninsular Range on the structural unit known as the Perris Block. The Perris Block is bounded on the northeast by the San Jacinto

Fault Zone, on the southwest by the Elsinore Fault Zone, and on the north by the Cucamonga Fault Zone. The southern boundary of the Perris Block is not as distinct, but is believed to coincide with a complex group of faults trending southeast from the Murrieta, California area. The Peninsular Range is characterized by large Mesozoic age intrusive rock masses flanked by volcanic, metasedimentary, and sedimentary rocks. Various thicknesses of alluvial sediments derived from the erosion of the elevated portions of the region fill the low-lying areas. Undocumented fill, alluvium and Pauba Formation bedrock underlie the subject property and surrounding area. The earth materials encountered on the subject site are described in more detail in Section 7.4, Earth Materials, of this report.

7.2 **Faulting:** The site is not located within an Alquist-Priolo Earthquake Fault Zone (Hart and Bryant, updated 1999). However, known active faults traverse the property (Leighton, 2005, 2006a, 2006b). The nearest Alquist-Priolo faults to the site are described below:

7.2.1 **Elsinore Fault Zone:** The Elsinore Fault - Temecula Valley Segment is located approximately 1.2 kilometers (0.76 miles) southwest of the site. The Elsinore Fault Zone generally trends northwest-southeast and is a major right lateral strike-slip fault, that has displayed Holocene displacement and associated strong earthquakes in 1856, 1894, and 1910.

7.3 **Seismicity:** The project lies within an active area of faulting and seismicity in the Southern California region. This predominance of seismic activity has been associated with the San Jacinto Fault Zone along its southeast section in the vicinity of the Salton Sea, and within the northwest portion near its junction with the San Andreas Fault Zone. The predominance of the remaining recorded activity has been associated with the San Andreas Fault Zone. A list of faults within 62 miles (100 kilometers) of the site is shown on Table A in the Appendix. Based on computer software by Thomas F. Blake (EQSEARCH, Blake 2004b), the maximum peak ground acceleration experienced at the site since 1800 was approximately 0.26g from a magnitude 6.8 earthquake located approximately 28 kilometers from the site that occurred in 1918.

Because active faults were found to exist within the project limits, the site may experience strong ground motion, including ground rupture and effects from earthquakes generated along active faults located on-site and effects from off-site active faulting.

To estimate the potential ground shaking, **EnGEN Corporation** has analyzed the seismic parameters using the probabilistic ground motion analysis. The probabilistic ground motion analysis requires information regarding fault geometry, the magnitude of the maximum credible earthquake on each fault, and the regional attenuation equation, which relates the considered seismic parameters to the magnitude and the source-site distance.

To perform this analysis, **EnGEN Corporation** utilized the computer software FRISKSP developed by Thomas F. Blake (Blake, 2004c).

The attenuation relationships by Boore et al. (1997) for soil type SD (stiff soil – shear wave velocity 250 m/s) was utilized. For a complete discussion of the software and probabilistic methods the reader is referred to Blake (2000a, b, c).

The intensity of ground shaking at a given location depends primarily upon the earthquake magnitude, distance from the source (epicenter), and the site response characteristics. The Elsinore Fault – Temecula Valley Segment is potentially capable of producing the most intense horizontal ground acceleration at the site, due to its proximity and associated maximum credible earthquake magnitude of 6.8. **Such an earthquake near the site could produce seismic shaking with an estimated maximum credible peak horizontal ground acceleration of 0.68g. The maximum credible peak horizontal acceleration is the maximum acceleration that appears capable of occurring under the presently known tectonic framework, and has a 10 percent chance of exceedance in 50 years.**

In sum, these results are based on many unavoidable geological and statistical uncertainties, but are consistent with current standard-of-practice. As engineering seismology evolves, and as more fault-specific geological data are gathered, more certainty and different methodologies may also evolve.

7.4 <u>**Earth Materials:**</u> A brief description of the earth materials encountered in the exploratory excavations is presented in the following sections. A more detailed description of the earth materials encountered is presented on the Geotechnical Boring Logs in the Appendix. The earth material strata as shown on the logs represent the conditions in the actual exploratory locations and other variations may occur between the excavations. Lines of demarcation between the earth materials on the logs represented the approximate boundary between the material types; however, the transition may be gradual.

7.4.1 **Undocumented Fill Material (Afu):** Undocumented fill material exists at the subject site in the form of tilled agricultural soils. Based on the soil samples collected on-site, it appears that the disturbed soils are limited to the upper 2 to 3-feet bgs. However, some deeper areas may occur. Some additional undocumented fill material exists in the form of stockpiled fault trench excavation soils. It is possible that additional undocumented fill material may be encountered in the southeastern most corner of the site, extending into the property approximately 200-feet. This area appears to be the location of former Catt Road/Clinton Keith Road and also appears to be a cut area, but may have subsurface abandoned utilities in the area associated with the former road. These materials and any other encountered undocumented fills should be removed and may be reused in fill provided they are properly cleaned of organics and debris. Also, based on our review of the referenced Fault Hazard Investigations, fault trench LFT-2 was backfilled without proper documentation. Therefore, the backfilled materials associated with this trench are considered undocumented fills.

7.4.2 **Alluvium (Qal):** Alluvium underlies the undocumented fill material and overlies the Pauba Formation bedrock. Based on our observations and the regional topography, the relatively thin alluvium generally thickens to the northwest across the site. The alluvium was observed to be primarily comprised of silty sands, and to a lesser extent, of clean or clayey sands. The alluvium exhibited conditions that were loose to medium dense, dry to moist, and porous to non-porous.

7.4.3 **Pauba Formation (Qps):** Pauba Formation bedrock was encountered below the undocumented fill or alluvium to the maximum depth explored (50.5-feet bgs). It was found to consist predominantly of clean, clayey, and silty sandstone with occasional sandy siltstone or sandy claystone. Soils encountered were found to be moist to wet and very dense or stiff to very hard in-place.

7.5 **Groundwater:** Groundwater was encountered at a depth of approximately 15-feet bgs in boring B-2 (approximately 1326 elevation based upon topography of the Leighton Fault Map, Plate 1) at the time of the field study. Groundwater was not encountered in the shallow borings, B-1 and B-3.

7.6 **Liquefaction Evaluation:** Liquefaction is a phenomenon where a sudden large decrease of shearing resistance takes place in fine-grained cohesionless and/or low plasticity cohesive soils due to the cyclic stresses produced by earthquakes causing a sudden, but temporary, increase of porewater pressure. The increased porewater pressure occurs below the water table, but can cause propagation of groundwater upward into overlying soil and possibly to the ground surface and cause sand boils as excess porewater escapes. Potential hazards due to liquefaction include significant total and/or differential settlements of the ground surface and structures as well as possible collapse of structures due to loss of support of foundations. It has been shown by laboratory testing and from the analysis of soil conditions at sites where liquefaction has occurred that the soil types most susceptible to liquefaction are saturated, fine-grained sand to sandy silt with a mean grain size ranging from approximately 0.075 mm to 0.5 mm. These soils derive their shear strength from intergranular friction and do not drain quickly during earthquakes. Published studies and field and laboratory test data indicate that coarse-grained sands and silty or clayey sands beyond the above-mentioned grain size range are considerably less vulnerable to liquefaction. To a large extent, the relative density of the soil also controls the susceptibility to liquefaction for a given number of cycles and acceleration levels during a seismic event. Other characteristics such as confining pressure and the stresses created within the soil during a seismic event also affect the liquefaction potential of a site. Liquefaction of soil does not generally occur at depths of greater than 40 to 50-feet below ground surface due to the confining pressure at that depth. To perform the liquefaction analysis, the computer software LIQUEFY2 (Blake, 1998) is typically utilized. The potential for liquefaction of the site is considered to be low due to the following conditions:

- High relative densities (based upon corrected SPT blow counts of 30 or more blows per foot) were encountered in the majority of the soils below the zone of proposed recompaction.

The total and differential potential settlement in the event of liquefaction has been calculated at 0-inches, assuming a maximum groundwater elevation of 10-feet bgs. Based on these findings, no further evaluation of liquefaction is considered necessary and settlement due to liquefaction is anticipated to be negligible.

7.7 **Secondary Effects of Seismic Activity:** The secondary effects of seismic activity normally considered as possible hazards to a site include various types of ground failure and induced flooding from dam failure. The nearest large confined body of water to the site is Lake Elsinore, located approximately 5 miles to the northwest and approximately

100-feet lower than the subject site. It is our understanding that the regional body of water has water surface elevation controls designed to provide appropriate safety factors with respect to the seismic forces that are expected to act on them. Therefore, seismically-induced flooding and earthquake-induced surface flooding due to seiches is considered low. Due to the distance from the Pacific Ocean and the site elevation of approximately 1,340 above mean sea level, the probability of a tsunami impacting the site is considered nil. The probability of occurrence of each type of ground failure depends on the severity of the earthquake, the distance of the site from the zone of maximum energy release of the quake, the topography of the site, the subsurface materials at the site, and groundwater conditions beneath the site, besides other factors. Since there are active faults on the site, the probability of hazards due to fault ground surface rupture is considered high. A structural setback zone was established by Leighton where structures for human occupancy should not be located (Leighton, 2006b). Due to the low topographic relief on-site, it is considered that the potential for earthquake-induced landslides is low.

8.0 CONCLUSIONS AND RECOMMENDATIONS

8.1 **General:** The conclusions and recommendations presented in this report are based on the results of field and laboratory data obtained from the exploratory excavations located across the property, and the project description and assumptions presented in Section 3.0, Proposed Development/Project Description, of this report. Based on the field and laboratory data and the engineering analysis performed, it is considered that the proposed development is feasible from a geotechnical/geological standpoint. The actual conditions of the near-surface supporting material across the site may vary. The nature and extent of variations of the surface and subsurface conditions between the exploratory excavations may not become evident until construction. If variations of the material become evident during grading, this office should be notified so that **EnGEN Corporation** can evaluate the characteristics of the material and, if needed, prepare revisions to the recommendations presented herein. Recommendations for general site grading, foundations, slab support, pavement design, slope maintenance, etc., are presented in the subsequent paragraphs.

8.2 Earthwork Recommendations:

8.2.1 **General:** The grading recommendations presented in this report are intended for: 1) the use of a conventional shallow foundation system and concrete slabs cast on-grade; and 2) the rework of unsuitable near-surface earth materials to create an engineered building pad and suitable support for exterior hardscape (sidewalks, patios, etc.) and pavement. If pavement subgrade soils are prepared at the time of rough grading of the building site and the areas are not paved immediately, additional observations and testing of the subgrade soil will have to be performed before placing aggregate base material or asphaltic concrete or PCC pavement to locate areas which may have been damaged by construction traffic, construction activities, and/or seasonal wetting and drying. The following recommendations may need to be modified and/or supplemented during rough grading as field conditions require.

8.2.2 **Clearing:** All debris, refuse, roots, grasses, weeds, brush and other deleterious materials should be removed from the proposed structure, exterior hardscape and pavement areas, as well as any areas to receive structural fill before grading is performed. No discing or mixing of organic material into the soils should be performed. Man-made objects encountered should be overexcavated and exported from the site. Any water wells encountered should be abandoned by a licensed well contractor.

8.2.3 **Excavation Characteristics:** Excavation and trenching within the undocumented fill, alluvium and Pauba Formation is anticipated to be relatively easy. These materials are anticipated to be rippable with conventional large grading equipment. A rippability study is not considered to be necessary.

8.2.4 **Suitability of On-Site Materials as Fill:** In general, the on-site earth materials present are considered suitable for reuse as fill. Fill materials should be free of significant amounts of organic materials and/or debris. Fill materials should not contain rocks greater than 6-inches in maximum diameter in the upper 5.0-feet of fill. Fill materials should not contain rocks greater than 12-inches in maximum diameter between 5 and 10-feet below proposed pad grade. Fills deeper than 10-feet may be used for oversize disposal. Oversize disposal of rocks greater than 12-inches maximum diameter may be conducted in accordance with Section 8.2.7, Oversize Material, of this report.

8.2.5 **Removal and Recompaction:** All existing undocumented fills and/or unsuitable, loose, or disturbed near-surface soil in areas that will support structural fills, structures, exterior hardscape (sidewalks, patios, etc.), and pavement should be removed. The grading plans should be made available for review by this office in order to prepare additional recommendations, if necessary. The following recommendations are based on field and laboratory results:

1. Any undocumented fill material to be encountered at the time of grading will require removal to competent alluvium.
2. The proposed detention area system and any improvements crossing the on-site fault zone should be designed and constructed in a manner that is compatable with potential fault related movement, including strong shaking and ground rupture.
3. All alluvium should be removed to competent Pauba Formation bedrock in the area of the proposed building pads. Based on our field and laboratory findings, removals deeper than 10-feet below existing grade, were not proposed or are not expected, except for proposed Building E. However, the exact removal depth should be determined based on exposed conditions to be encountered during grading. Based upon the field and laboratory findings, and the provided proposed Finished Floor elevations for the proposed structures on-site, the preliminary anticipated removal depths are as follows:

Building	Proposed Finished Floor Elevation	Preliminary Anticipated Removal Recommendation
"Drug Store" A B	1339.6 1342.1 1340.7	Competent bedrock is anticipated at the depth of the proposed pad grades for buildings A, B and "Drug Store". Bedrock bottoms should be inspected for competency and uniformity and tested for final expansion index. If expansive soils are found (EI 20 or more), selective grading or updated design may be necessary.
C	1343.2	Remove existing soils in area to receive fill to expose. Competent bedrock, anticipated at 3 to 4-feet bgs. Overexcavate cut and shallow fill areas a minimum of 3-feet below proposed grade.
D	1342.7	Remove existing soils in area to receive fill to expose. Competent bedrock, anticipated at 3 to 4-feet bgs. Overexcavate cut and shallow fill areas a minimum of 4-feet below proposed grade.

* Table continued see next page

Building	Proposed Finished Floor Elevation	Preliminary Anticipated Removal Recommendation
E	1339.1	Remove alluvium to competent bedrock or groundwater, if first encountered. Removals to bedrock are anticipated to be 10-feet bgs in the area of B-3 (south portion of Building E) and approximately 15-feet below original ground surface near station 2+35 of LFT-1 (north portion of Building E). It appears that the bedrock is deepest near the middle of Building E based upon its absence within the excavated depth of LFT-1 (approximately 12-feet below original ground surface) and the interpreted axis of the surface drainage through this general area. If ground water is encountered at the time of removals, specialized equipment or alternatives may be necessary to perform grading in this area.
Fault Trenches (LFT-1 and LFT 7) underlying building areas	Not Applicable	Should be backfilled at 95% relative compaction in areas to remain underlying building areas and 90% relative compaction in other areas.
Previously backfilled Fault Trench LFT-2	Not Applicable	Remove and recompact the undocumented fill to 90% relative compaction.

4. Horizontal building removals outside of building footprints should extend a distance equal to half the maximum fill depth below proposed grade with a minimum of 5-feet.

5. Bedrock bottoms should be inspected by the Project Geologist/Engineer or his/her representative to verify competency. Bottoms which are not found to be competent should be deepened.

6. Removals in the remaining hardscape portions of the site should be performed to 2-feet below the undocumented fill and alluvial bedrock in fill areas, and to 2-feet below proposed grade in cut areas the undocumented fill and alluvial bedrock in fill areas.

7. All exposed removal and overexcavation bottoms should be inspected by the Project Geologist, and/or his representative prior to placement of any fill.

8. The approved exposed bottoms of all removal areas should be scarified 12-inches, brought to near optimum moisture content, and compacted to a minimum of 90 percent

relative compaction before placement of fill. Maximum dry density and optimum moisture content for compacted materials should be determined in accordance with ASTM D 1557-02 procedures.

9. Geologic contacts as shown on the attached site plan are approximate. Final determination of removal and overexcavation depths should be made during grading.
10. If import material, selective grading or other specific activity associated with maintaining very low or low expansion soils near the finish surface is planned to be used, this firm should be notified immediately to perform additional testing and provide further recommendations, as necessary.

8.2.6 **Fill Placement Requirements:** All fill material, whether on-site material or import, should be approved by the Project Geotechnical Engineer and/or his representative before placement. All fill should be free of vegetation, organic material, and debris. Oversized material should be disposed of in accordance with Section 8.2.7, Oversize Material, of this report. Import fill should be no more expansive than the existing on-site material. Approved fill material should be placed in horizontal lifts not exceeding 10-inches in compacted thickness and watered or aerated to obtain near optimum moisture content (±2.0 percent of optimum). Each lift should be spread evenly and should be thoroughly mixed to ensure uniformity of soil moisture. Structural fill should meet a minimum relative compaction of 90 percent. Maximum dry density and optimum moisture content for compacted materials should be determined in accordance with ASTM D 1557-02 procedures. Moisture content of fill materials should not vary more than 2.0 percent from optimum, unless approved the Project Geotechnical Engineer.

8.2.7 **Oversize Material:** Oversize material is defined as rock, or other irreducible material with a dimension greater than 12-inches. Oversize material shall not be buried or placed in fill unless location, materials, and placement methods are specifically accepted by the Project Geotechnical Engineer. Placement operations shall be such that nesting of oversize material does not occur, and such that the oversize material is completely surrounded by compacted fill (windrow). Alternative methods, such as water jetting or wheel rolling with a backhoe may be required to achieve compaction in the fill materials immediately adjacent to the windrow. Oversize material shall not be placed within ten (10) vertical feet of finish grade, within fifteen (15) lateral feet of a finished slope face, or within two (2) feet of future utilities.

8.2.8 **Compaction Equipment:** It is anticipated that fill compaction for the project will be achieved by the use of a combination of rubber-tired and track-mounted heavy construction equipment. Compaction by rubber-tired or track-mounted equipment, by itself, may not be sufficient. Adequate water trucks, water pulls, and/or other suitable equipment should be available to provide sufficient moisture and dust control. The actual selection of equipment is the responsibility of the contractor performing the work and should be such that uniform and proper compaction of the fill is achieved. Specialized equipment may be required for activities near the groundwater table.

8.2.9 **Shrinkage and Bulking:** There will be a material loss due to the clearing and grubbing operations. Shrinkage of undocumented fill (Afu) and alluvium (Qal) that is excavated and replaced as compacted fill should be anticipated. It is estimated that the average shrinkage of these materials will be on the order of 5 percent, based on fill volumes when compacted to a minimum of 90 percent relative compaction. A higher relative compaction would mean a larger shrinkage value.

8.2.10 **Fill Slopes:** Finish fill slopes should not be inclined steeper than 2:1 (horizontal to vertical). Fill slope surfaces should be compacted to 90 percent relative compaction based on a maximum dry density for the soil as determined by ASTM D 1557-02 procedures to the face of the finished slope. Fill slopes should be constructed in a skillful manner so that they are positioned at the design orientations and slope ratio. Achieving a uniform slope surface by subsequent thin wedge filling should be avoided. Any add-on correction to a fill slope should be conducted under the observation and recommendations of the Project Geotechnical Engineer. The proposed add-on correction procedures should be submitted in writing by the contractor prior to commencement of corrective grading and reviewed by the Project Geotechnical Engineer. Compacted fill slopes should be backrolled with suitable equipment for the type of soil being used during fill placement at intervals not exceeding 4.0-feet in vertical height. As an alternative to the backrolling of the fill slopes, over-filling of the slopes will be considered acceptable and preferred. The fill slope should be constructed by over-filling with compacted fill a minimum of 3.0-feet horizontally, and then trimmed back to exposed the dense inner core of the slope surface.

8.2.11 **Cut Slopes:** All cut slopes should not be inclined steeper than 2:1 (horizontal to vertical). Steeper cut slopes will require slope stability analysis to verify stability. All cut slopes should be inspected by the Project Engineering Geologist to check for any adverse geologic conditions. Cut slopes with adverse geologic conditions may require flattening or buttressing to maintain stability.

8.2.12 **Keyways:** A keyway excavated into competent native earth materials should be constructed at the toe of all fill slopes that are proposed on natural grades of 5:1 (horizontal to vertical) or steeper. Keyways should be a minimum of 15-feet wide (equipment width) and tilted a minimum of 2 percent into the hillside. A series of level benches should be constructed into competent native earth materials on natural grades of 5:1 (horizontal to vertical) or steeper prior to placing fill.

8.2.13 **Subdrains:** Although the need for subdrains is not anticipated at this time, final recommendations should be made during grading by the Project Engineering Geologist.

8.2.14 **Observation and Testing:** During grading, observation and testing should be conducted by the Project Geotechnical Engineer and/or his representative to verify that the grading is being performed according to the recommendations presented in this report. The Project Geotechnical Engineer and/or his representative should observe the scarification and the placement of fill and should take tests to verify the moisture content, density, uniformity and degree of compaction obtained. Where testing demonstrates insufficient density, additional compaction effort, with the adjustment of the moisture content where necessary, should be applied until retesting shows that satisfactory relative compaction has been obtained. The results of observations and testing services should be presented in a formal Finish Grading Report following completion of the grading operations. Grading operations undertaken at the site without the Project Geotechnical Engineer and/or his representative present may result in exclusions of the affected areas from the finish grading report for the project. The presence of the Project Geotechnical Engineer and/or his representative will be for the purpose of providing observations and field testing and will not include any supervision or directing of the actual work of the contractor or the contractor's employees or agents. Neither the presence and/or the non-presence of the Project Geotechnical Engineer and/or his field representative nor the field observations and testing shall excuse the contractor in any way for defects discovered in the contractor's work.

8.2.15 **Soil Expansion Potential:** Upon completion of fine grading of the building pad, near-surface samples should be obtained for expansion potential testing to identify the expansion potential for each lot and assign appropriate foundation and slab-on-grade recommendations for construction. Our Expansion Index (EI) testing on-site indicates that soil expansivity is EI=18, which is classified as having a very low expansion potential. **Based on our observations of the subsurface soils, expansive soils may be present on site.** Mixing of these soils during grading could affect the overall EI of the fill. If selective grading is desired in order to ensure that expansive soils are not used near pad grade, this option should be discussed with this firm and the grading contractor prior to grading the site. Should alternative foundation design be preferred over these recommendations, it is recommended that the client or the clients authorized representative coordinate with this office at the earliest possible date in order to ensure final foundation plans reflect the recommendations of this firm. **Final foundation design parameters should be based on EI testing of near-surface soils and be performed at the conclusion of rough grading.**

8.2.16 **Soil Corrosivity:** Test results for pH, minimum resistivity, sulfate content and chloride content (CTM 417, CT 643, CTM 422 procedures) were analyzed and processed by Prime Testing, Inc. A non-detectable concentration (less than 0.001%) by weight) of water soluble sulfates were reported. As a result, normal Type II cement may be used in concrete that will come in contact with native soils. Additional corrosivity related test results provided but not interpreted included a pH of 7.1, a minimum resistively of 2,500 ohm-cm, and a chloride content of 170 ppm. Should additional corrosivity analysis be desires, a Corrosion Engineer should be consulted. Laboratory analytical results are included in the Appendix.

8.3 Foundation Design Recommendations:

8.3.1 **General:** Foundations for the proposed structures may consist of conventional column footings and continuous wall footings founded upon properly compacted fill, as recommended in Section 8.2, the Earthwork Recommendations, of this report. The recommendations presented in the subsequent paragraphs for foundation design and construction are based on geotechnical characteristics and a very low expansion potential for the supporting soils and are not intended to preclude more restrictive structural requirements. The Structural Engineer for the project should determine the actual footing width and depth to resist design vertical, horizontal, and uplift forces.

8.3.2 **Foundation Size:** Continuous footings should have a minimum width of 12-inches. Continuous footings should be continuously reinforced with a minimum of one (1) No. 4 steel reinforcing bar located near the top and one (1) No. 4 steel reinforcing bar located near the bottom of the footings to minimize the effects of slight differential movements which may occur due to minor variations in the engineering characteristics or seasonal moisture change in the supporting soils. Final foundation size and reinforcing should be determined by the Project Structural Engineer based on structural loads and the expansive potential of the supporting soils. Column footings should have a minimum width of 18-inches by 18-inches and be suitably reinforced, based on structural requirements. A grade beam, founded at the same depths and reinforced the same as the adjacent footings, should be provided across the garage, doorways, or any other types of perimeter openings.

8.3.3 **Depth of Embedment:** Exterior and interior footings founded in properly compacted fill should extend to a minimum depth of 12-inches below lowest adjacent finish grade for one story structures and 18-inches below lowest adjacent final grade for two story structures. Deeper footings may be necessary for expansive soils purposes, depending on the final determination of pad specific expansive potential.

8.3.4 **Bearing Capacity:** Provided the recommendations for site earthwork, minimum footing width, and minimum depth of embedment for footings are incorporated into the project design and construction, the allowable bearing value for design of continuous and column footings for the total dead plus frequently-applied live loads is 2,000 psf for continuous footings, and 2,000 psf for column footings in properly compacted fill or bedrock. The allowable bearing value has a factor of safety of at least 3.0 and may be increased by 33.3 percent for short durations of live and/or dynamic loading, such as wind or seismic forces.

8.3.5 **Settlement:** Footings designed according to the recommended bearing values and the maximum assumed wall and column loads are not expected to exceed a maximum settlement of 0.75-inch or a differential settlement of 0.50-inch in properly compacted fill under static load conditions.

No liquefaction is anticipated for the subject site; therefore, it is our opinion that dynamic settlement is not a design consideration for this site.

8.3.6 **Lateral Capacity:** Additional foundation design parameters for resistance to static lateral forces are as follows:

Allowable Lateral Pressure (Equivalent Fluid Pressure), Passive Case:
Competent Bedrock or Compacted Fill– 250 pcf

Allowable Coefficient of Friction: Competent Bedrock or Compacted Fill –
0.35

Lateral load resistance may be developed by a combination of friction acting on the base of foundations and slabs and passive earth pressure developed on the sides of the footings and stem walls below grade when in contact with properly compacted fill or competent bedrock. The above values are allowable design values and have safety factors of at least 2.0 incorporated into them and may be used in combination without reduction in evaluating the resistance to lateral loads. The allowable values may be increased by 33.3 percent for short durations of live and/or dynamic loading, such as wind or seismic forces. For the calculation of passive earth resistance, the upper 1.0-foot of material should be neglected unless confined by a concrete slab or pavement. The maximum recommended allowable passive pressure is 5.0 times the recommended design value.

8.3.7 **Seismic Design Parameters:** The following seismic design factors apply:

Design Fault: Elsinore – Temecula Segment

Fault Type: Type B Fault

Closest Distance to Fault: 1.2 Km

Soil Profile Type: S_D

8.4 **Slab-on-Grade Recommendations:** The recommendations for concrete slabs, both interior and exterior, excluding PCC pavement, are based upon the expansion potential for the supporting material. Concrete slabs should be designed to minimize cracking as a result of shrinkage. Joints (isolation, contraction, and construction) should be placed in accordance with the American Concrete Institute (ACI) guidelines. Special precautions should be taken during placement and curing of all concrete slabs. Excessive slump (high water / cement ratio) of the concrete and/or improper curing procedures used during either hot or cold weather conditions could result in excessive shrinkage, cracking, or curling in the slabs. It is recommended that all concrete proportioning, placement, and curing be performed in accordance with ACI recommendations and procedures.

8.4.1 **Interior Slabs:** Interior concrete slabs-on-grade should be a minimum of 4.0-inches nominal in thickness and be underlain by a 1.0 to 2.0-inches of clean coarse sand or other approved granular material placed on properly prepared subgrade per Section 8.2, Earthwork

Recommendations, of this report. Minimum slab reinforcement should consist of No. 3 reinforcing bars placed 24-inches on center in both directions, or a suitable equivalent as determined by the Project Structural Engineer. Final pad identification and slab construction requirements will be presented in the compaction report upon completion of grading. It is essential that the reinforcing be placed at mid-depth in the slab. The concrete section and/or reinforcing steel should be increased appropriately for anticipated excessive or concentrated floor loads. In areas where moisture sensitive floor coverings are anticipated over the slab, we recommend the use of a polyethylene vapor barrier with a minimum of 10.0 mil in thickness be placed beneath the slab. The moisture barrier should be overlapped or sealed at splices and covered top and bottom by a 1.0 to 2.0-inch minimum layer of clean, moist (not saturated) sand to aid in concrete curing and to minimize potential punctures.

8.4.2 **Exterior Slabs:** All exterior concrete slabs cast on finish subgrade (patios, sidewalks, etc., with the exception of PCC pavement) should be a minimum of 4.0-inches nominal in thickness and should be underlain by a minimum of 12.0-inches of soil that has been prepared in accordance with Section 8.2, Earthwork Recommendations, of this report. Reinforcing in the slabs and the use of a compacted sand or gravel base beneath the slabs should be according to the current local standards.

8.5 **Pavement Design Recommendations:** The following recommendations for the structural pavement section for the proposed parking and driveway areas for the subject development are presented for preliminary design purposes only. **The final design should be based on the soils located near subgrade.** The pavement section has been determined in general accordance with **CalTrans** design procedures and is based on an assumed Traffic Index (TI) and an assumed R-Value of 34, which corresponds to the test results from B-1 at 0 to10-feet. The R-Value of any imported fill material may vary from the assumed value thereby changing the proposed pavement section design.

The sections listed below are provided for reference purposes and are calculated as a minimum based on varying Traffic Indexes:

Traffic Index	Calculated Section
5.0	3-inches Asphaltic Concrete over 5-inches Aggregate Base, placed properly compacted subgrade.

6.0	3-inches Asphaltic Concrete over 7.5-inches Aggregate Base, placed properly compacted subgrade.

Asphalt concrete pavement materials should be as specified in Sections 39-2.01 and 39-2.02 of the current **CalTrans** Standard Specifications or a suitable equivalent. Aggregate base should conform to ¾-inch Class II material as specified in Section 26-1.02B of the current **CalTrans** Standard Specifications or a suitable equivalent. In public roadways the subgrade soil, including utility trench backfill, should be compacted to at least 95 percent relative compaction. The aggregate base material should be compacted to at least 95 percent relative compaction. Maximum dry density and optimum moisture content for subgrade and aggregate base materials should be determined according to ASTM D 1557-02 procedures. If pavement subgrade soils are prepared at the time of rough grading of the building site and the areas are not paved immediately, additional observations and testing will have to be performed before placing aggregate base material, asphaltic concrete, or PCC pavement to locate areas that may have been damaged by construction traffic, construction activities, and/or seasonal wetting and drying. In the proposed pavement areas, soil samples should be obtained at the time the subgrade is graded for R-Value testing according to California Test Method 301 procedures to verify the pavement design recommendations.

8.6 **Utility Trench Recommendations:** Utility trenches within the zone of influence of foundations or under building floor slabs, exterior hardscape, and/or pavement areas should be backfilled with properly compacted soil. All utility trenches within the building pad and extending to a distance of 5.0-feet beyond the building exterior footings should be backfilled with on-site or similar soil. Where interior or exterior utility trenches are proposed to pass beneath or parallel to building, retaining wall, and/or decorative concrete block perimeter wall footings, the bottom of the trench should not be located below a 1:1 plane projected downward from the outside bottom edge of the adjacent footing unless the utility lines are designed for the footing surcharge loads. It is recommended that all utility trenches excavated to depths of 5.0-feet or deeper be cut back according to Section 8.9, Temporary Construction Excavation Recommendations, of this report or be properly shored during construction. Backfill material should be placed in a lift thickness appropriate for the type of backfill material and compaction equipment used. Backfill material should be compacted to a minimum of 90 percent relative compaction by mechanical means. In public roadway areas, backfill material should be compacted to a minimum 95-percent relative compaction. Jetting or flooding of the backfill material will not be considered a satisfactory method for compaction unless the procedures are reviewed and approved in writing by the Project Geotechnical Engineer. Maximum dry density and optimum moisture content for backfill material should be determined according to ASTM D 1557-02 procedures.

8.7 **Finish Lot Drainage Recommendations:** Positive drainage should be established away from the tops of slopes, the exterior walls of structures, the back of retaining walls, and the decorative concrete block perimeter walls. Finish lot surface gradients in unpaved areas should be provided next to tops of slopes and buildings to guide surface water away from foundations and slabs and from flowing over the tops of slopes. The surface water should be directed toward suitable drainage facilities. Ponding of surface water should not be allowed next to structures or on pavements. In unpaved areas, a minimum positive gradient of 4.0 percent away from the structures and tops of slopes for a minimum distance of 3.0-feet and a minimum of 1.0 percent pad drainage off the property in a non-erosive manner should be provided. Landscape trees and plants with high water needs should be planted at least 5.0-feet away from the walls of the structures. Downspouts from roof drains should discharge to a surface which slopes away from the structure a minimum of 5.0-feet from the exterior building walls. In no case should downspouts from roof drains discharge into planter areas immediately adjacent to the building unless there is positive drainage away from the structure at a minimum gradient of 2.0 percent, directed onto a permanent all-weather surface or subdrain system.

8.8 **Planter Recommendations:** Planters around the perimeter of the structures should be designed to ensure that adequate drainage is maintained and minimal irrigation water is allowed to percolate into the soils underlying the buildings.

8.9 **Temporary Construction Excavation Recommendations:** Temporary construction excavations for rough grading, foundations, retaining walls, utility trenches, etc., more than 5.0-feet in depth and to a maximum depth of 15-feet should be properly shored or cut back to the following inclinations:

Earth Material	Inclination
Compacted Fill, Alluvium or Pauba Formation Bedrock	1.5:1

No surcharge loads (spoil piles, earthmoving equipment, trucks, etc.) should be allowed within a horizontal distance measured from the top of the excavation slope equal to 1.5 times the depth of the excavation. Excavations should be initially observed by the Project Geotechnical Engineer, Project Engineering Geologist, and/or their representative to verify our recommendations or to make additional recommendations to maintain stability and safety. Moisture variations, differences in the cohesive or cementation characteristics, or changes in the coarseness of the deposits may require slope flattening or, conversely, permit steepening upon review by the Project Geotechnical Engineer, Project Engineering Geologist, and/or their representative. Deep utility trenches may experience caving which will require special considerations to stabilize the walls and expedite trenching operations. Surface drainage should be controlled along the top of the slope to preclude erosion of the slope face. If excavations are to be left open for long periods, the slopes should be sprayed with a protective compound and/or covered to minimize drying out, raveling, and/or erosion of the slopes. For excavations more than 5.0-feet in depth which will not be cut back to the recommended slope inclination, the contractor should submit to the owner and/or the owner's designated representative detailed drawings showing the design of shoring, bracing, sloping, or other provisions to be made for worker protection. If the drawings do not vary from the requirements of the OSHA Construction Safety Orders (CAL OSHA or FED OSHA, whichever is applicable for the project at the time of construction), a statement signed by a registered Civil or Structural Engineer in the State of California, engaged by the contractor at his expense, should be submitted certifying that the contractor's excavation safety drawings comply with OSHA Construction Orders. If the drawings vary from the applicable OSHA Construction Safety Orders, the drawings should be prepared, signed, and sealed by a Registered Civil or Structural Engineer in the State of California. The contractor should not proceed with any excavations until the project owner or his designated representative has received and acknowledged the properly prepared excavation safety drawings.

8.10 **Retaining Wall Recommendations:**

8.10.1 **Earth Pressures:** Retaining walls backfilled with non-expansive granular soil (EI=0) or very low expansive potential materials (Expansion Index of 20 or less) within a zone extending upward and away from the heel of the footing at a slope of 0.5:1 (horizontal to vertical) or flatter can be designed to resist the following static lateral soil pressures:

Condition	**Level Backfill**	**2:1 Slope**

Active	30 pcf	45 pcf
At Rest	60 pcf	--

Walls that are free to deflect 0.01 radian at the top may be designed for the above-recommended active condition. Walls that need to be restricted from such movement should be assumed rigid and designed for the at-rest condition. The above values assume well-drained backfill and no buildup of hydrostatic pressure. Surcharge loads, dead and/or live, acting on the backfill behind the wall or directly on the wall should also be considered in the design.

8.10.2 **Foundation Design:** Retaining wall footings should be founded to the same depths into properly compacted fill as standard foundations and may be designed for the same average allowable bearing value across the footing (as long as the resultant force is located in the middle one-third of the footing), and with the same allowable static lateral bearing pressure and allowable sliding resistance as previously recommended. When using the allowable lateral pressure and allowable sliding resistance, a factor of safety of 1.0 may be used. If ultimate values are used for design, an approximate factor of safety of 1.5 should be achieved.

8.10.3 **Subdrain:** A subdrain system should be constructed behind and at the base of all retaining walls to allow drainage and to prevent the buildup of excessive hydrostatic pressures. Typical subdrains may include weep holes with a continuous gravel gallery, perforated pipe surrounded by filter rock, or some other approved system. Gravel galleries and/or filter rock, if not properly designed and graded for the on-site and/or import materials, should be enclosed in a geotextile fabric such as Mirafi 140N, Supac 4NP, or a suitable substitute in order to prevent infiltration of fines and clogging of the system. The perforated pipes should be at least 4.0-inches in diameter. Pipe perforations should be placed downward. Gravel filters should have volume of at least 1.0 cubic foot per lineal foot of pipe. Subdrains should maintain a positive flow gradient and have outlets that drain in a non-erosive manner. In the case of subdrains for basement walls, they need to empty into a sump provided with a submersible pump activated by a change in the water level.

8.10.4 **Backfill:** Backfill directly behind retaining walls (if backfill width is less than 3 feet) may consist of 0.5 to 0.75-inch diameter, rounded to subrounded gravel enclosed in a geotextile fabric such as Mirafi 140N, Supac 4NP, or a suitable substitute or a clean sand (Sand

Equivalent Value greater than 50) water jetted into place to obtain proper compaction. If water jetting is used, the subdrain system should be in place. Even if water jetting is used, the sand should be densified to a minimum of 90 percent relative compaction. If the specified density is not obtained by water jetting, mechanical methods will be required. If other types of soil or gravel are used for backfill, mechanical compaction methods will be required to obtain a relative compaction of at least 90 percent of maximum dry density. Backfill directly behind retaining walls should not be compacted by wheel, track or other rolling by heavy construction equipment unless the wall is designed for the surcharge loading. If gravel, clean sand or other imported backfill is used behind retaining walls, the upper 18-inches of backfill in unpaved areas should consist of typical on-site material compacted to a minimum of 90 percent relative compaction in order to prevent the influx of surface runoff into the granular backfill and into the subdrain system.

Maximum dry density and optimum moisture content for backfill materials should be determined in accordance with ASTM D 1557-02 procedures.

9.0 PLAN REVIEW

Grading and foundation plans for the proposed development should be provided for review by **EnGEN Corporation** to verify compatibility with site geotechnical conditions and conformance with the recommendations contained in this report. If **EnGEN Corporation** is not accorded the opportunity to make the recommended review, we will assume no responsibility for misinterpretation of the recommendations presented in this report.

10.0 PRE-BID CONFERENCE

It may be desirable to hold a pre-bid conference with the owner or an authorized representative, the Project Architect, the Project Civil Engineer, the Project Geotechnical Engineer, and the proposed contractors present. This conference will provide continuity in the bidding process and clarify questions relative to the grading and construction requirements of the project.

11.0 PRE-GRADING CONFERENCE

Before the start of grading, a conference should be held with the owner or an authorized representative, the contractor, the Project Architect, the Project Civil Engineer, and the Project Geotechnical Engineer present. The purpose of this meeting should be to clarify questions relating to the intent of the grading recommendations and to verify that the project specifications comply with the recommendations of this geotechnical engineering report. Any special grading procedures and/or difficulties proposed by the contractor can also be discussed at that time.

12.0 CONSTRUCTION OBSERVATIONS AND TESTING

Rough grading of the property should be performed under engineering observation and testing performed by **EnGEN Corporation.** Rough grading includes, but is not limited to, overexcavation cuts, fill placement, and excavation of temporary and permanent cut and fill slopes. In addition, **EnGEN Corporation** should observe all foundation excavations.

Observations should be made before installation of concrete forms and/or reinforcing steel to verify and/or modify the conclusions and recommendations in this report. Observations of overexcavation cuts, fill placement, finish grading, utility or other trench backfill, pavement subgrade and base course, retaining wall backfill, slab presaturation, or other earthwork completed for the subject development should be performed by **EnGEN Corporation.** If the observations and testing to verify site geotechnical conditions are not performed by **EnGEN Corporation**, liability for the performance of the development is limited to the actual portions of the project observed and/or tested by **EnGEN Corporation.** If parties other than **EnGEN Corporation** are engaged to perform soils and materials observations and testing, they must be notified that they will be required to assume complete responsibility for the geotechnical aspects of the project by concurring with the recommendations in this report or providing alternative recommendations. Neither the presence of the Project Geotechnical Engineer and/or his field representative, nor the field observations and testing, shall excuse the contractor in any way for defects discovered in the contractor's work. The Project Geotechnical Engineer and/or his representative shall not be responsible for job or project safety. Job or project safety shall be the sole responsibility of the contractor.

13.0 CLOSURE

This report has been prepared for use by the parties or project named or described in this document. It may or may not contain sufficient information for other parties or purposes. In the event that changes in the assumed nature, design, or location of the proposed development as described in this report are planned, the conclusions and recommendations contained in this report will not be considered valid unless the changes are reviewed and the conclusions and recommendations of this report modified or verified in writing. This study was conducted in general accordance with the applicable standards of our profession and the accepted geotechnical engineering principles and practices at the time this report was prepared. No other warranty, implied or expressed beyond the representations of this report, is made. Although every effort has been made to obtain information regarding the geotechnical and subsurface conditions of the site, limitations exist with respect to the knowledge of unknown regional or localized off-site conditions which may have an impact at the site. The recommendations presented in this report are valid as of the date of the report. However, changes in the conditions of a property can occur with the passage of time, whether they are due to natural processes or to the works of man on this and/or adjacent properties.

If conditions are observed or information becomes available during the design and construction process which are not reflected in this report, **EnGEN Corporation** should be notified so that supplemental evaluations can be performed and the conclusions and recommendations presented in this report can be modified or verified in writing. This report is not intended for use as a bid document. Any person or company using this report for bidding or construction purposes should perform such independent studies and explorations as he deems necessary to satisfy himself as to the surface and subsurface conditions to be encountered and the procedures to be used in the performance of the work on this project. Changes in applicable or appropriate standards of care or practice occur, whether they result from legislation or the broadening of knowledge and experience. Accordingly, the conclusions and recommendations presented in this report may be invalidated, wholly or in part, by changes outside the control of **EnGEN Corporation** which occur in the future.

Thank you for the opportunity to provide our services. If we can be of further service or you should have questions regarding this report, please contact this office at your convenience.

Respectfully submitted,
EnGEN Corporation

PROFESSIONAL GEOLOGIST ERIC DAVISSON No 8231 EXP. STATE OF CALIFORNIA

Eric Davisson, PG 8231
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Osbjorn Bratene, GE 162
President
Expires 09-30-07

APPENDIX

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TABLE 1

DISTANCE TO STATE DESIGNATED ACTIVE FAULTS

ABBREVIATED FAULT NAME	APPROXIMATE DISTANCE Mi	(Km)	MAXIMUM EARTHQUAKE MAG (Mw)
Elsinore – Temecula	0.76	1.2	6.8
Elsinore – Glen Ivy	6.9	11.1	6.8
Elsinore – Julian	20.1	32.4	7.1
San Jacinto – San Jacinto Valley	20.3	32.6	6.9
San Jacinto – Anza	21.1	33.9	7.2
Chino – Central Avenue (Elsinore)	24.6	39.6	6.7
San Joaquin Hills	24.9	40.1	6.6
Newport – Inglewood (Offshore)	28.6	46.1	7.1
Whittier	28.9	46.5	6.8
San Jacinto – San Bernardino	29.0	46.6	6.7
Rose Canyon	34.5	55.5	7.2
San Andreas – San Bernardino	35.4	56.9	7.5
San Andreas – SB-Coach.	35.4	56.9	7.7
San Andreas – Whole	35.4	56.9	8.0
San Andreas –SB-Coach.	35.4	56.9	7.7
Newport – Inglewood (L.A. Basin)	39.4	63.4	7.1
Cucamonga	41.6	67.0	6.9
Puente Hills Blind Thrust	42.6	68.6	7.1
San Jacinto – Coyote Creek	43.2	69.6	6.6
North Frontal Fault Zone (West)	43.5	70.0	7.2
Pinto Mountian	43.5	70.0	7.2
San Jose	43.9	70.6	6.4
Coronado Bank	44.9	72.3	7.0
Palos Verdes	45.7	73.6	7.3
Sierra Madre	46.3	74.5	7.2
Cleghorn	46.8	75.3	6.5
Lenwood-Lockhart-Old Woman Sprgs	47.3	76.1	7.5
Earthquake Valley	47.6	76.6	6.5
North Frontal Fault Zone (East)	49.6	79.9	6.7
San Andreas – Coachella	49.7	80.0	7.2
San Andreas – 1857 Rupture	50.0	80.5	7.8
San Andreas – Cho Moj	50.0	80.5	7.8
San Andreas – Mojave	50.0	80.5	7.4
Burnt Mountain	54.2	87.2	6.5
Clamshell - Sawpit	56.4	90.8	6.5
Eureka Peak	57.5	92.5	6.4
Helendale – South Lockhardt	57.5	92.5	7.3
Raymond	58.2	93.6	6.5
Upper Elysian Park Blind Thrust	59.2	95.2	6.4
Landers	60.9	98.0	7.3

TABLE 2
POTENTIAL SETTLEMENT DUE TO LIQUEFACTION CALCULATIONS
BORING NO. B-2

Layer No.	Depth Range (ft)	SPT	$(N_1)_{60}$	FS	Ev %	Layer Thickness (ft)	ΔH
1	0-10	77	81	>1.5	Non Liquefiable (H_2O density)	10	0
2	10-15	90	77	>1.5	Non Liquefiable (density)	5	0
3	15-35	95	68	>1.5	Non Liquefiable (density)	20	0
4	35-40	100	60	>1.5	Non Liquefiable (density)	5	0
5	40-50	100	59	>1.5	Non Liquefiable (density)	10	0

Total ΔH = 0-inches
Differential ΔH = 0-inches

- Non-Liquefiable (H_20) = Non-Liquefiable due to lack of groundwater
- Non-Liquefiable (clay) = Non-Liquefiable due to clay content in excess of 15 percent
- Non-Liquefiable (density) = Non-Liquefiable due to high relative densities, $(N_1)_{60}$ of 30 or more.

- Groundwater set at 10-feet bgs
- Earthquake Magnitude (M)w= 6.8
- Horizontal Ground Acceleration (probabilistic method)=0.68 g

GEOTECHNICAL BORING LOGS

(B-1 through B-3)

GEOTECHNICAL BORING LOG

Project Number: M3551-GS
Boring Number: B1
Date: 03/13/07

Project: Zolfaghari Commercial
Surface Elevation: 1352
Logged By: ED

Soil Graphic	Description	Sampler	Sample Depth	USCS	Blow Count	Dry Density	In-Situ Moisture Content	Maximum Density	Optimum Moisture Content
	<u>UNDOCUMENTED FILL (Afu)</u> Silty fine- to medium-grained sand, light olive brown (5Y 5/3), dry, medium dense.		0	SM					
	<u>PAUBA FORMATION (Qps)</u> Fine sandy siltstone, light olive brown (2.5Y 5/4), moist, very dense.			ML	16-29-50/1	109.1	6.7		
			5					126.4	10.4
	Fine sandy claystone, strong brown (7.5YR 5/6), moist, very hard.				48-50/4	120.7	11.9		
	Silty fine sandstone, light olive brown (2.5 Y 5/4), moist, very dense.		10		18-38-50/5	113.3	8.4		
	Cobble or boulder.					113.3	8.4		
	Silty fine to medium sandstone, olive brown (2.5Y 4/4), moist, very dense. Total Depth 15-feet bgs. No Groundwater Encountered.		15		24-37-50/6	126.8	8.4		
			20						
			25						
			30						
			35						

Notes:

EnGEN Corporation

GEOTECHNICAL BORING LOG

Project Number: M3551-GS
Boring Number: B2
Date: 03/13/07

Project: Zolfaghari Commercial
Surface Elevation: 1341
Logged By: ED

Soil Graphic	Description	Sampler	Sample Depth	USCS	Blow Count	Dry Density	In-Situ Moisture Content	Maximum Density	Optimum Moisture Content
	UNDOCUMENTED FILL (Afu) Silty fine- to medium-grained sand, brown (10YR 4/3), dry, dense.		0	SM					
	PAUBA FORMATION (Qps) Silty fine sandstone, Light Yellowish Brown (2.5Y 6/3), moist, very dense.			SM	23-34-47		22.9		
			5		22-50/5	116.3	12.5		
					35-46-50/3		16.0		
	Fine sandy siltstone, olive (5Y 5/4), moist, very dense, a few fine gravel, angular.		10	ML	15-50/4		16.5		
	Fine to medium sandstone, pale olive (5Y 6/3), wet, very dense.		15	SP	50/6		10.6		
	Fine to medium sandstone, pale olive (5Y 6/3), wet, very dense.		20	SP	40-50		13.5		
	Fine to medium sandstone, pale olive (5Y 6/3), wet, very dense.		25	SP	50/5		12.8		
	Fine to medium sandstone, pale olive (5Y 6/3), wet, very dense.		30	SP	23-50/3		13.1		
	Medium to coarse sandstone, light gray (2.5Y 7/2), wet, very dense.		35	SP	45-50/3		11.0		

Notes:

GEOTECHNICAL BORING LOG

Project Number: M3551-GS
Boring Number: B2
Date: 03/13/07

Project: Zolfaghari Commercial
Surface Elevation: 1341
Logged By: ED

Soil Graphic	Description	Sampler	Sample Depth	USCS	Blow Count	Dry Density	In-Situ Moisture Content	Maximum Density	Optimum Moisture Content
	Fine to medium sandstone, light gray (2.5Y 7/2), wet, very dense, 1-inch recovery.		40	SP	50/2		20.1		
	No recovery.		45		50/5				
	Fine to medium sandstone, pale olive (5Y 6/3), wet, very dense. Total Depth 50.5-feet bgs. Groundwater Encountered at 15-feet bgs.		50	SP	55/2		15.4		
			55						
			60						
			65						
			70						

Notes:

GEOTECHNICAL BORING LOG

Project Number: M3551-GS
Boring Number: B3
Date: 03/13/07

Project: Zolfaghari Commercial
Surface Elevation: 1533
Logged By: ED

Soil Graphic	Description	Sampler	Sample Depth	USCS	Blow Count	Dry Density	In-Situ Moisture Content	Maximum Density	Optimum Moisture Content
	UNDOOCUMENTED FILL (Afu) Silty fine- to medium-grained sand, yellowish brown (10YR 5/4), dry, loose.		0	SM					
	ALLUVIUM (Qal) Silty fine- to medium-grained sand, yellowish brown (10YR 5/4), dry, very dense, slightly porous.			SM	20-31-50/6	124.5	3.4	134.4	8.0
	Silty fine- to medium-grained sand, dark yellowish brown (10YR 4/4), moist, dense.		5	SM	19-30-33	121.8	6.4		
	Silty fine- to medium-grained sand, dark yellowish brown (10YR 4/4), moist, dense.			SM	16-22-31	130.0	8.0		
	PAUBA FORMATION (Qps) Clayey fine sandstone, light olive brown (2.5Y 5/4), moist, very dense. Total Depth 11.5-feet bgs. No Groundwater Encountered.		10	SC	27-34-50/6	127.0	8.5		
			15						
			20						
			25						
			30						
			35						

Notes:

KEY TO SYMBOLS

Symbol Description

Strata symbols

Silty sand

Silt

Poorly graded sand

Clayey sand

Misc. Symbols

Boring continues

Soil Samplers

California sampler

Standard penetration test

Notes:

1. Exploratory borings were drilled on 03/13/07 using a 7-inch diameter continuous flight power auger.

2. Water was encountered at the time of drilling at the depths shown.

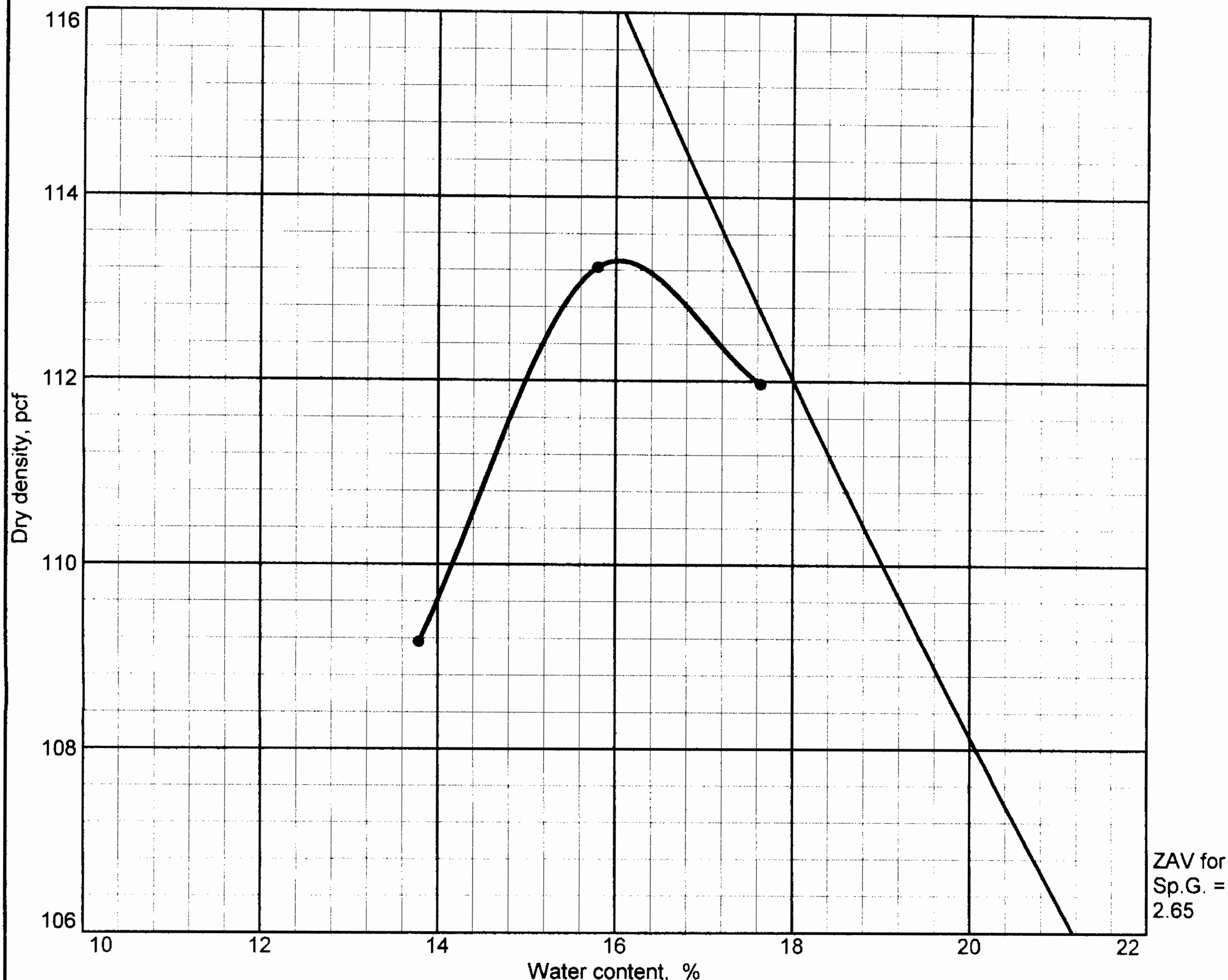
3. Boring locations were measured from existing features and elevations extrapolated from the final design plan.

4. These logs are subject to the limitations, conclusions, and recommendations in this report.

5. Results of tests conducted on samples recovered are reported on the logs.

LABORATORY TEST RESULTS

OPTIMUM MOISTURE/MAXIMUM DENSITY TEST REPORT



Test specification: ASTM D 1557-02 Method A Modified

Elev/ Depth	Classification		Nat. Moist.	Sp.G.	LL	PI	% > No.4	% < No.200
	USCS	AASHTO						
	ML		17.6					

TEST RESULTS	MATERIAL DESCRIPTION
Maximum dry density = 113.3 pcf Optimum moisture = 16.0 %	SANDY SILT, LIGHT OLIVE BROWN

Project No. M3551-GS **Client:** REZA ZOLFAGHARI

Project: ZOLFAGHARI COMMERCIAL

● **Location:** CLINTON KEITH ROAD

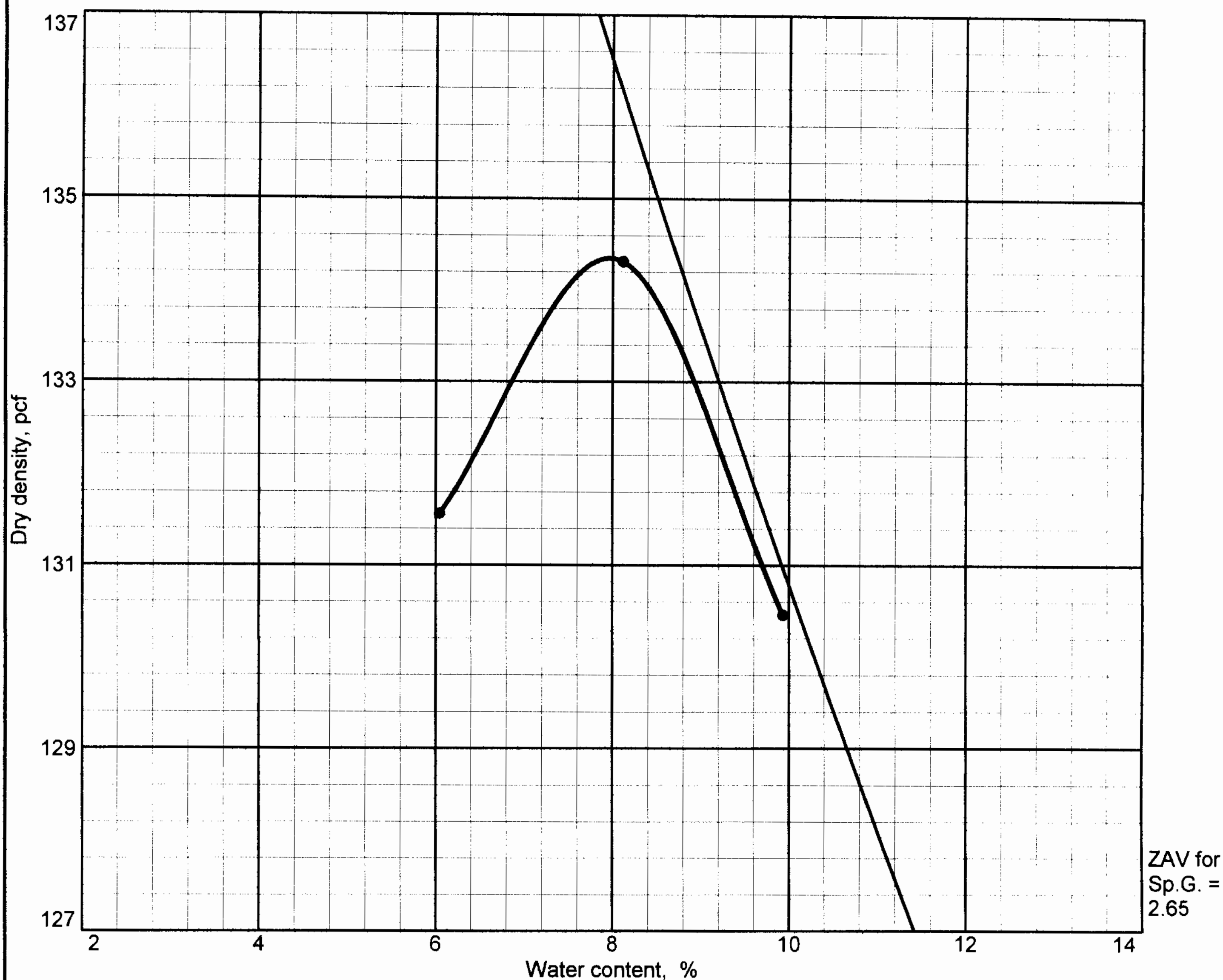
Remarks:
SAMPLE # B2 @ 0-10
BUILDING C
COLLECTED BY ED
COLLECTED ON (3/13/07)

OPTIMUM MOISTURE/MAXIMUM DENSITY TEST REPORT

ENVIRONMENTAL AND GEOTECHNICAL ENGINEERING NETWORK CORPORATION

Figure

OPTIMUM MOISTURE/MAXIMUM DENSITY TEST REPORT



Test specification: ASTM D 1557-02 Method A Modified

Elev/ Depth	Classification		Nat. Moist.	Sp.G.	LL	PI	% > No.4	% < No.200
	USCS	AASHTO						
	SM		4.8					

TEST RESULTS	MATERIAL DESCRIPTION
Maximum dry density = 134.4 pcf Optimum moisture = 8.0 %	SILTY SAND W/CLAY, BROWN

Project No. M3551-GS **Client:** REZA ZOLFAGHARI

Project: ZOLFAGHARI COMMERCIAL

● **Location:** CLINTON KEITH ROAD

Remarks:

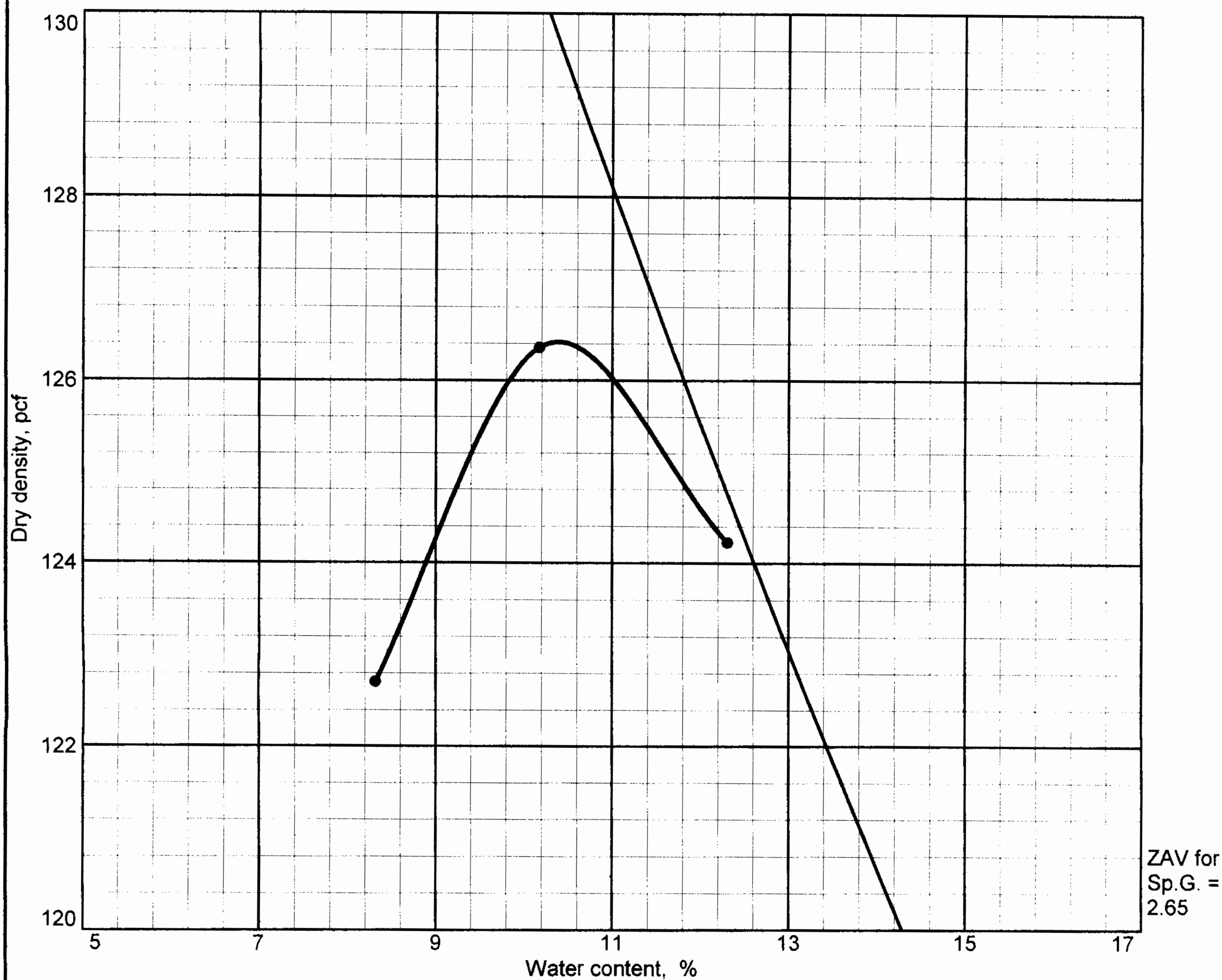
SAMPLE # B3 @ 0-5
BUILDING E
COLLECTED BY ED
COLLECTED ON (3/13/07)

OPTIMUM MOISTURE/MAXIMUM DENSITY TEST REPORT

ENVIRONMENTAL AND GEOTECHNICAL ENGINEERING NETWORK CORPORATION

Figure

OPTIMUM MOISTURE/MAXIMUM DENSITY TEST REPORT



Test specification: ASTM D 1557-02 Method A Modified

Elev/ Depth	Classification		Nat. Moist.	Sp.G.	LL	PI	% > No.4	% < No.200
	USCS	AASHTO						
	SM		8.3					

TEST RESULTS	MATERIAL DESCRIPTION
Maximum dry density = 126.4 pcf Optimum moisture = 10.4 %	SILTY SAND, LIGHT OLIVE BROWN

Project No. M3551-GS **Client:** REZA ZOLFAGHARI

Project: ZOLFAGHARI COMMERCIAL

● **Location:** CLINTON KEITH ROAD

Remarks:

SAMPLE # B1 @ 0-10
BUILDING A
COLLECTED BY ED
COLLECTED ON (3/13/07)

OPTIMUM MOISTURE/MAXIMUM DENSITY TEST REPORT

ENVIRONMENTAL AND GEOTECHNICAL ENGINEERING NETWORK CORPORATION

Figure

UBC Laboratory Expansion Test Results

3/16/2007

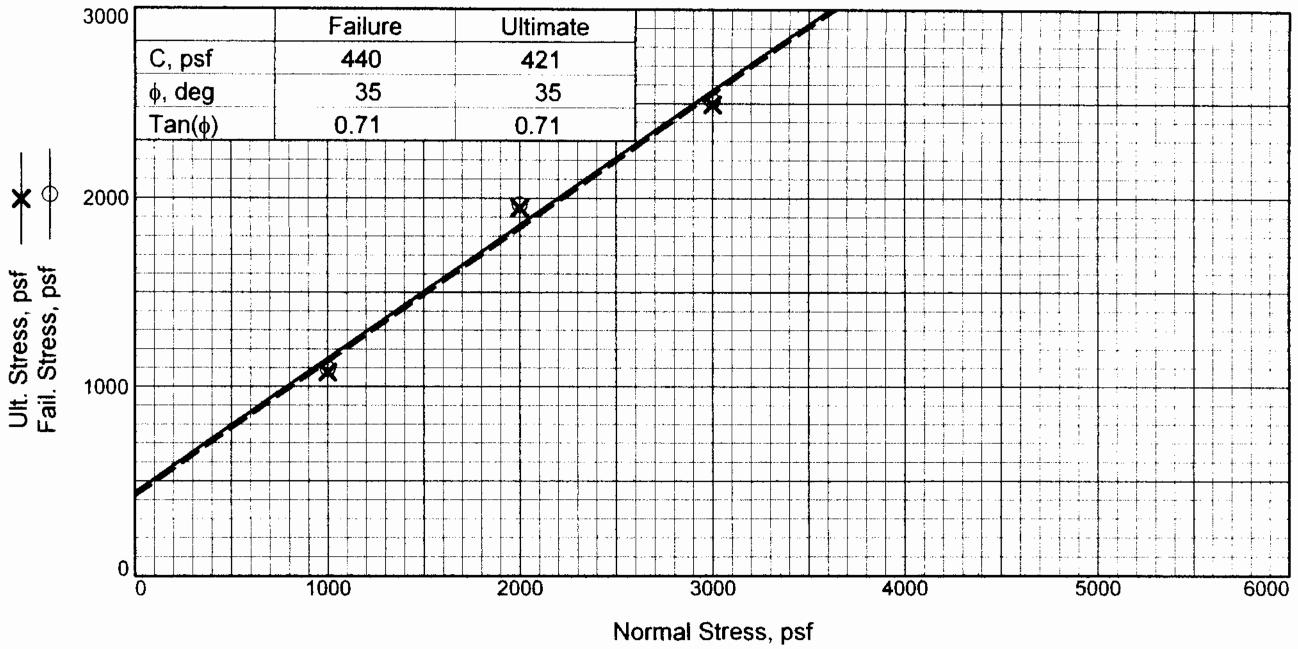
Job Number: M3551-GS
Job Name: ZOLFAGHARI COMMERCIAL
Location: CLINTON KEITH ROAD
Sample Source: B1 @ 0-10 (BUILDING A)
Sampled by: ED (3/13/07)
Lab Technician: SW
Sample Descr: SILTY SAND, LIGHT OLIVE BROWN

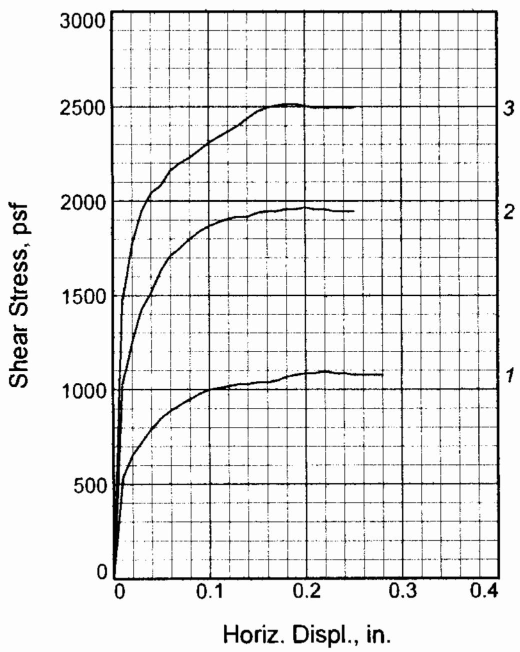
Wet Compacted Wt.:	587.9
Ring Wt.:	188.0
Net Wet Wt.:	399.9
Wet Density:	120.8
Wet Soil:	202.8
Dry Soil:	185.2
Initial Moisture (%):	9.5%
Initial Dry Density:	110.3
% Saturation:	48.6%
Final Wt. & Ring Wt.:	620.7
Net Final Wt.:	432.7
Dry Wt.:	365.2
Loss:	67.5
Net Dry Wt.:	361.9
Final Density:	109.3
Saturated Moisture:	18.7%

	Dial	Change	Time
Reading 1:	0.100	N/A	11:15
Reading 2:	0.110	0.010	11:30
Reading 3:	0.117	0.017	11:45
Reading 4:	0.118	0.018	16-Mar

Expansion Index:	18
Adjusted Index:	17.3
(UBC 18-2)	

EnGEN Corporation
25759 Jefferson Avenue, Murrieta, CA
92562
(951) 834-9000
Fax: (951) 834-9001





		1	2	3
Sample No.		1	2	3
Initial	Water Content, %	11.3	11.3	11.3
	Dry Density, pcf	113.7	113.7	113.7
	Saturation, %	65.6	65.6	65.6
	Void Ratio	0.4556	0.4556	0.4556
	Diameter, in.	2.42	2.42	2.42
	Height, in.	1.00	1.00	1.00
At Test	Water Content, %	N/A	N/A	N/A
	Dry Density, pcf			
	Saturation, %			
	Void Ratio			
	Diameter, in.			
	Height, in.			
Normal Stress, psf		1000	2000	3000
Fail. Stress, psf		1096	1966	2514
Displacement, in.		0.22	0.20	0.18
Ult. Stress, psf		1076	1946	2494
Displacement, in.		0.25	0.23	0.21
Strain rate, in./min.		0.20	0.20	0.20

Sample Type: REMOLDED
Description: SILTY SAND, LIGHT OLIVE BROWN
LL= **PL=** **PI=**
Specific Gravity= 2.65
Remarks: BUILDING A
COLLECTEED BY ED
COLLECTED ON (3/13/07)

Figure ________

Client: REZA ZOLFAGHARI

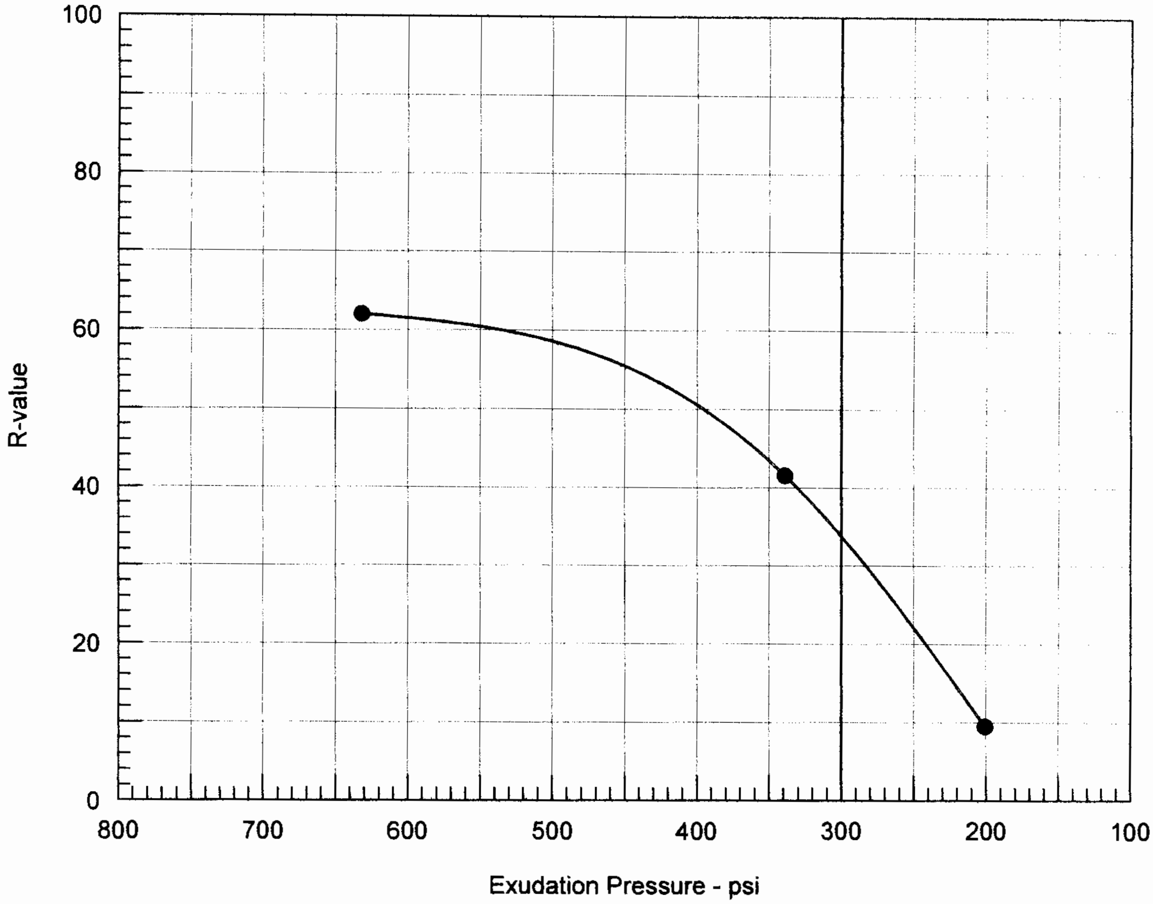
Project: ZOLFAGHARI COMMERCIAL

Source of Sample: SHEAR
Sample Number: B1 @ 1-10
Proj. No.: M3551-GS **Date:** 3/19/07

DIRECT SHEAR TEST REPORT
ENVIRONMENTAL AND GEOTECHNICAL ENGINEERING NETWORK CORPORATION

Tested By: SB **Checked By:** JH

R-VALUE TEST REPORT



Resistance R-Value and Expansion Pressure - Cal Test 301

No.	Compact. Pressure psi	Density pcf	Moist. %	Expansion Pressure psi	Horizontal Press. psi @ 160 psi	Sample Height in.	Exud. Pressure psi	R Value	R Value Corr.
1	350	127.1	11.5	20.01	50	2.47	632	62	62
2	300	126.5	12.6	13.95	85	2.53	339	42	42
3	200	123.6	13.6	8.49	140	2.68	201	9	10

Test Results	Material Description
R-value at 300 psi exudation pressure = 34	SILTY SAND, LIGHT OLIVE BROWN

Project No.: M3551-GS

Project:ZOLFAGHARI COMMERCIAL

Source of Sample: R-VALUE

Sample Number: B1 @ 0-10

Date: 3/19/2007

Tested by: SW

Checked by: JH

Remarks:
BUILDING A
COLLECTED BY ED
COLLECTED ON (3/13/07)

R-VALUE TEST REPORT

EnGEN Corporation

Figure 1

Prime Testing, Inc.
38372 Innovation Ct Ste 102 Murrieta, CA 92563
ph (951) 894-2682 • fx (951) 894-2683

Client: EnGEN Corporation
Report Date: March 22, 2007
Client No: A02
Work Order: 7C2
Project No: M3551-GS [P.O. #3264]
Project Name: Zolfaghari Commercial

Laboratory Test(s) Results Summary

The subject soil sample was processed in accordance with California Test Method CTM 643 and tested for pH / Minimum Resistivity (CTM 643), Sulfate Content (CTM 417) and Chloride Content (CTM 422). The test results follow:

Client Data			pH	Minimum Resistivity (ohm-cm)	Sulfate Content (mg/kg)	Sulfate Content (% by wgt)	Chloride Content (ppm)
Sample No.	Sample Location	Depth (ft)					
–	B1	0-10	7.1	2,500	ND	ND	170

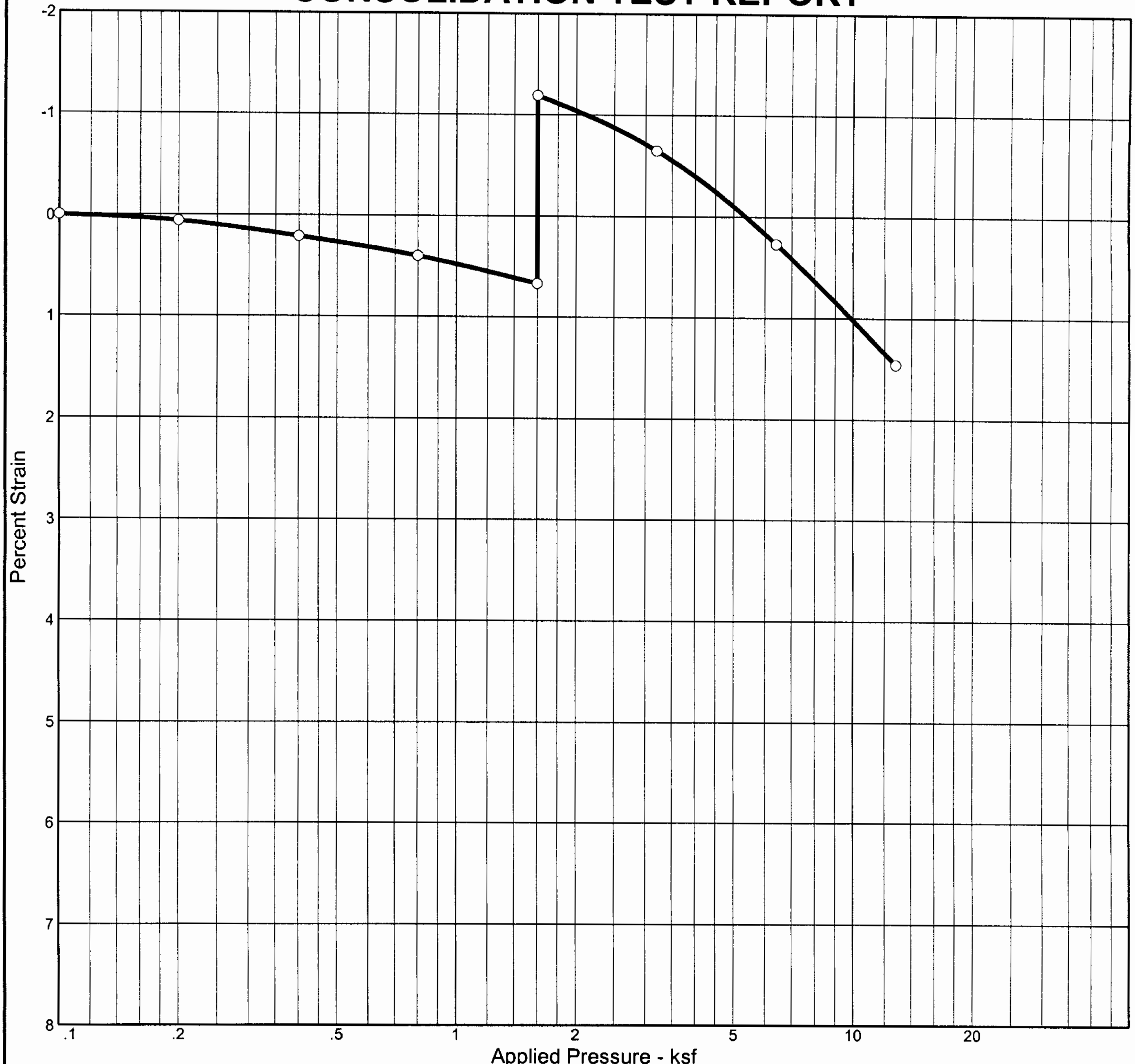
*ND=No Detection

We appreciate the opportunity to serve you. Please do not hesitate to contact us with any questions or clarifications regarding these results or procedures.

Ahmet K. Kaya, Laboratory Manager

INTERNATIONAL
ORGANIZATIONAL
MEMBER

CONSOLIDATION TEST REPORT



Natural		Dry Dens. (pcf)	LL	PI	Sp. Gr.	Overburden (ksf)	P_c (ksf)	C_c	C_r	Swell Press. (ksf)	Swell %	e_o
Sat.	Moist.											
89.7 %	11.9 %	120.7			2.60		6.70	0.05		8.16	1.9	0.345

MATERIAL DESCRIPTION	USCS	AASHTO
SANDY SILT, BROWN	ML	

Project No. M3551-GS **Client:** REZA ZOLFAGHARI

Project: ZOLFAGHARI COMMERCIAL

Source: CONSOL **Sample No.:** B1 @ 7.5

Remarks:

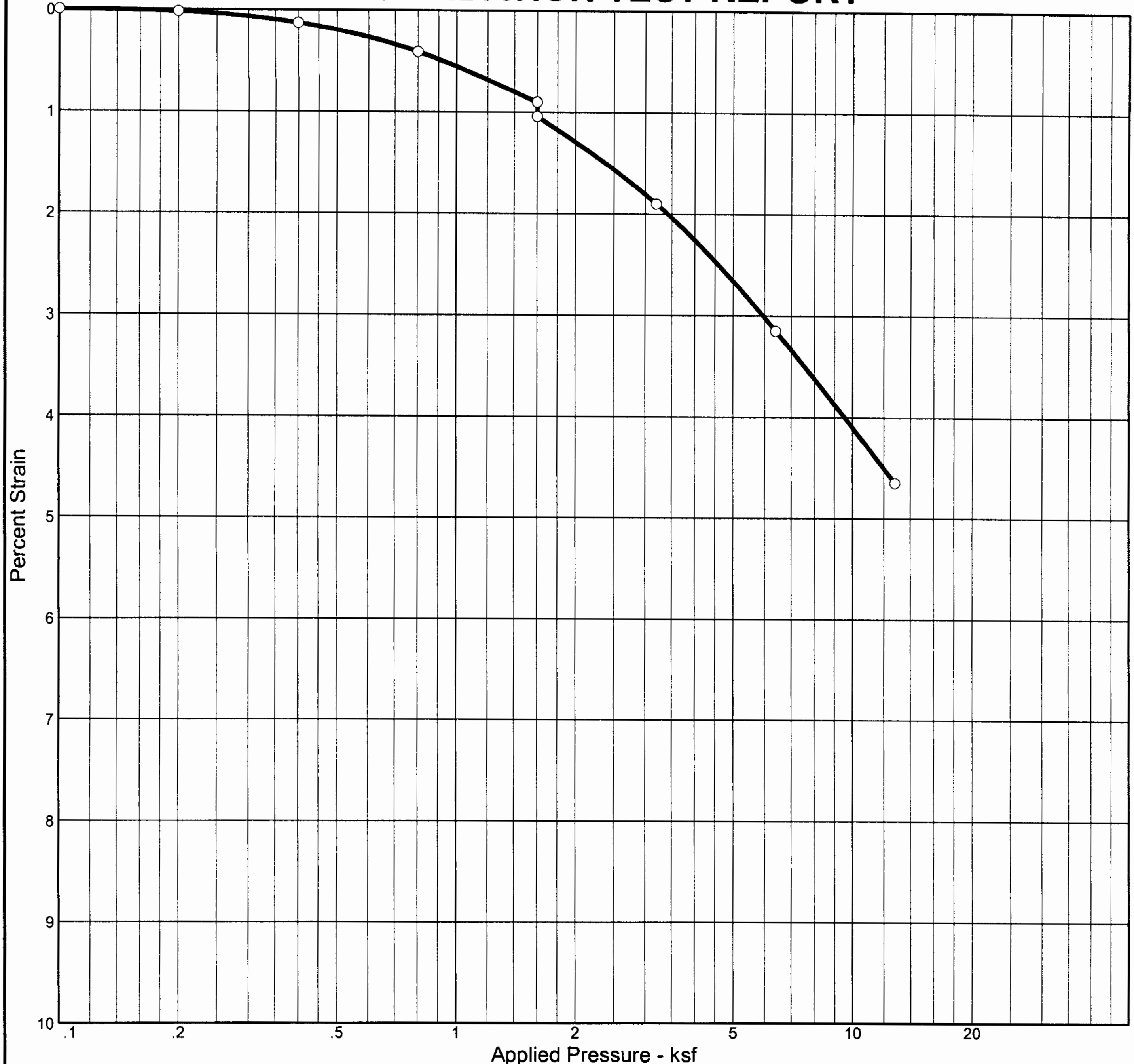
COLLECTED BY ED
COLLECTED ON (3/13/07)

CONSOLIDATION TEST REPORT

ENVIRONMENTAL AND GEOTECHNICAL ENGINEERING NETWORK CORPORATION

Figure

CONSOLIDATION TEST REPORT



Natural		Dry Dens. (pcf)	LL	PI	Sp. Gr.	Overburden (ksf)	P_c (ksf)	C_c	C_r	Swell Press. (ksf)	Clpse. %	e_o
Sat.	Moist.											
78.6 %	12.5 %	116.3			2.65		3.89	0.07			0.1	0.422

MATERIAL DESCRIPTION	USCS	AASHTO
SILTY SAND, LIGHT OLIVE BROWN	SM	

Project No. M3551-GS **Client:** REZA ZOLFAGHARI

Project: ZOLFAGHARI COMMERCIAL

Source: CONSOL **Sample No.:** B2 @ 5

Remarks:
COLLECTED BY ED
COLLECTED ON (3/13/07)

CONSOLIDATION TEST REPORT

ENVIRONMENTAL AND GEOTECHNICAL ENGINEERING NETWORK CORPORATION

Figure

CONSOLIDATION TEST REPORT

Natural		Dry Dens. (pcf)	LL	PI	Sp. Gr.	Overburden (ksf)	P_c (ksf)	C_c	C_r	Swell Press. (ksf)	Clpse. %	e_o
Sat.	Moist.											
27.8 %	3.4 %	124.5			2.65		3.56	0.12			1.7	0.329

MATERIAL DESCRIPTION	USCS	AASHTO
SILTY SAND, BROWN	SM	

Project No. M3551-GS **Client:** REZA ZOLFAGHARI

Project: ZOLFAGHARI COMMERCIAL

Source: CONSOL **Sample No.:** B3 @ 2.5

CONSOLIDATION TEST REPORT

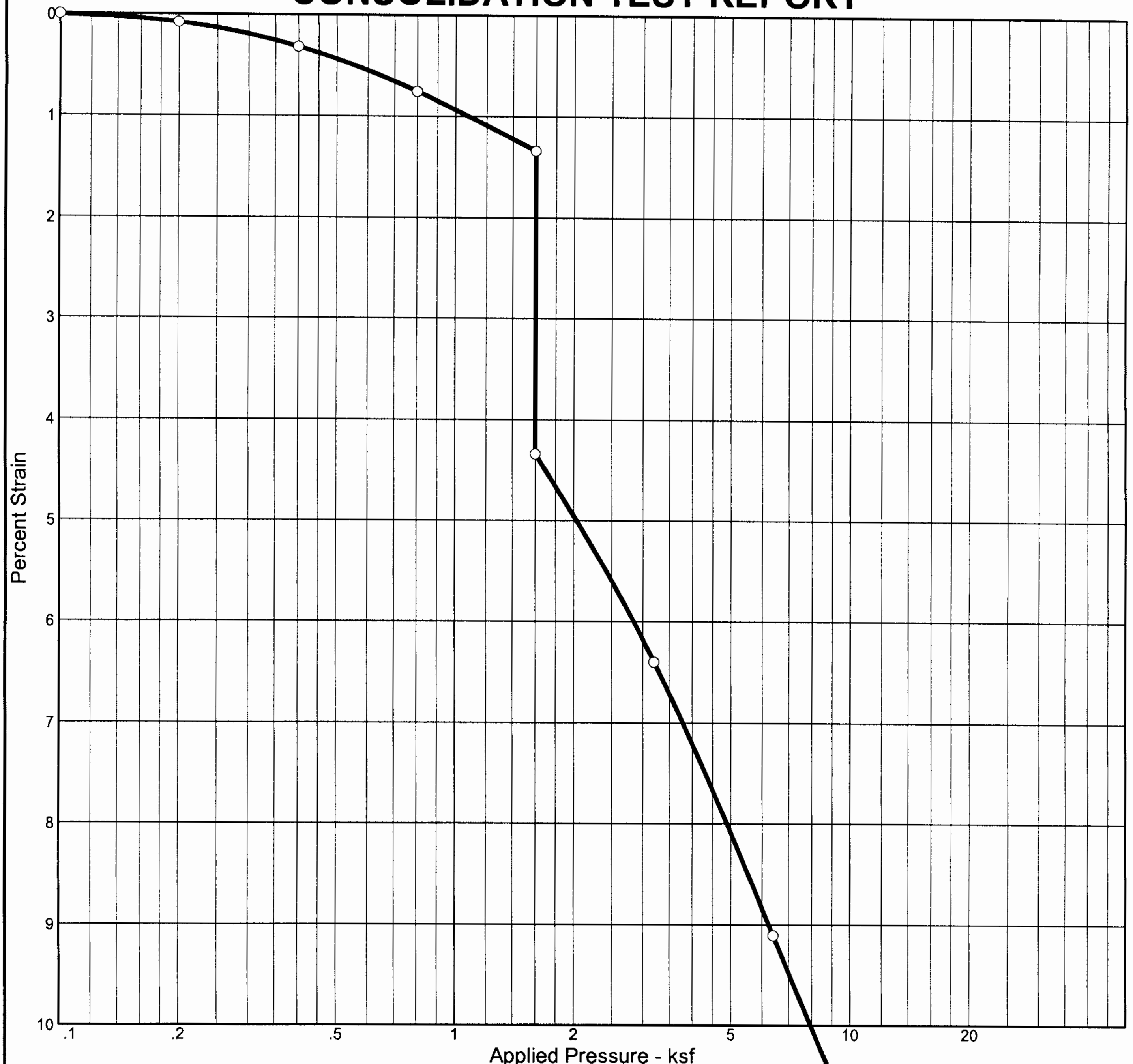
ENVIRONMENTAL AND GEOTECHNICAL ENGINEERING NETWORK CORPORATION

Remarks:

COLLECTED BY ED
COLLECTED ON (3/13/07)

Figure

CONSOLIDATION TEST REPORT



Natural Sat.	Natural Moist.	Dry Dens. (pcf)	LL	PI	Sp. Gr.	Overburden (ksf)	P_c (ksf)	C_c	C_r	Swell Press. (ksf)	Clpse. %	e_o
47.6 %	6.4 %	121.7			2.65		3.40	0.13			3.0	0.359

MATERIAL DESCRIPTION	USCS	AASHTO
SILTY SAND, BROWN	SM	

Project No. M3551-GS **Client:** REZA ZOLFAGHARI

Project: ZOLFAGHARI COMMERCIAL

Source: CONSOL **Sample No.:** B3 @ 5

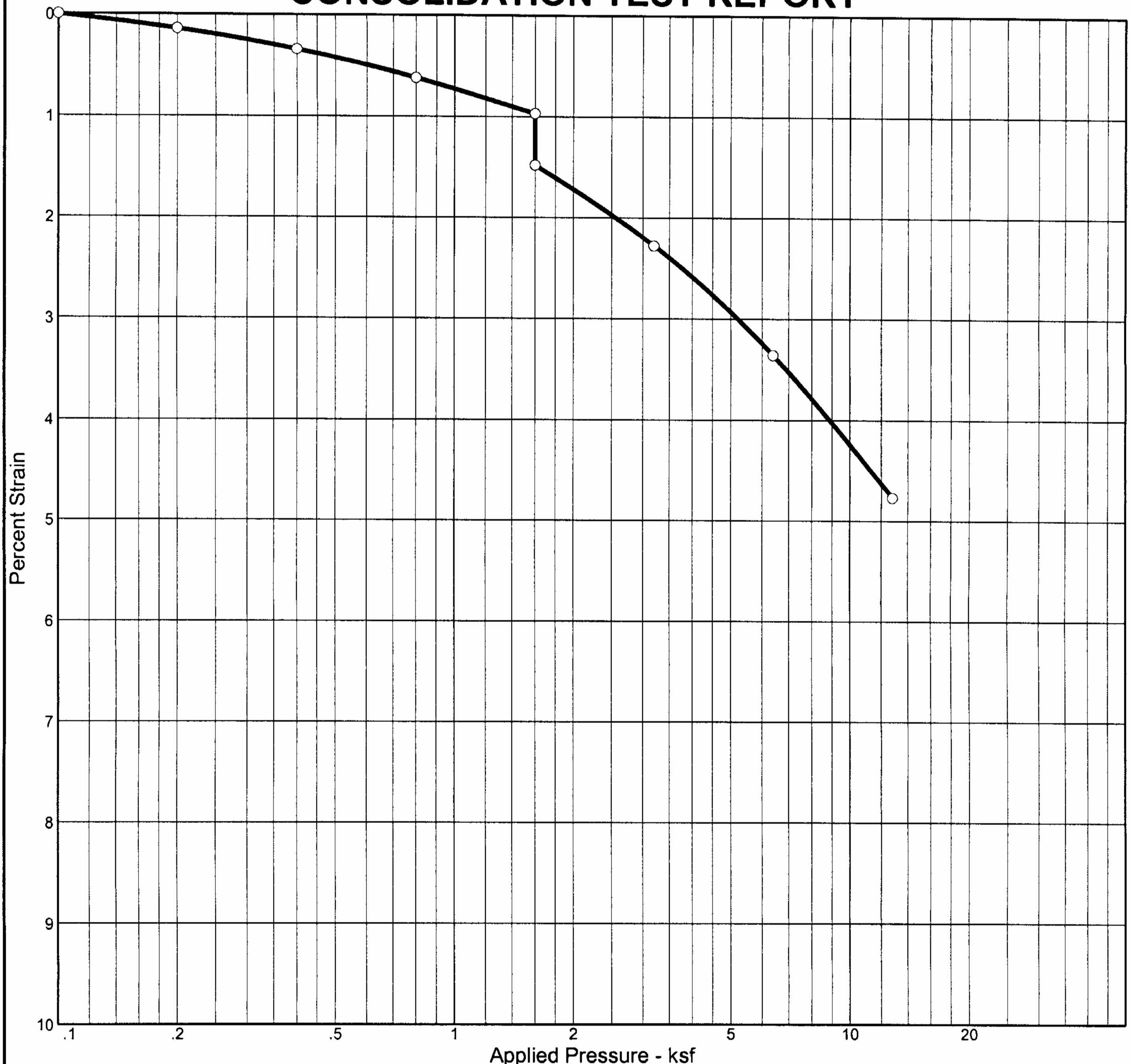
Remarks:
COLLECTED BY ED
COLLECTED ON (3/13/07)

CONSOLIDATION TEST REPORT

ENVIRONMENTAL AND GEOTECHNICAL ENGINEERING NETWORK CORPORATION

Figure

CONSOLIDATION TEST REPORT



Natural		Dry Dens. (pcf)	LL	PI	Sp. Gr.	Overburden (ksf)	P_c (ksf)	C_c	C_r	Swell Press. (ksf)	Clpse. %	e_o
Sat.	Moist.											
78.0 %	8.0 %	130.0			2.65		4.32	0.06			0.5	0.272

MATERIAL DESCRIPTION	USCS	AASHTO
SILTY SAND, BROWN	SM	

Project No. M3551-GS **Client:** REZA ZOLFAGHARI

Project: ZOLFAGHARI COMMERCIAL

Source: CONSOL **Sample No.:** B3 @ 7.5

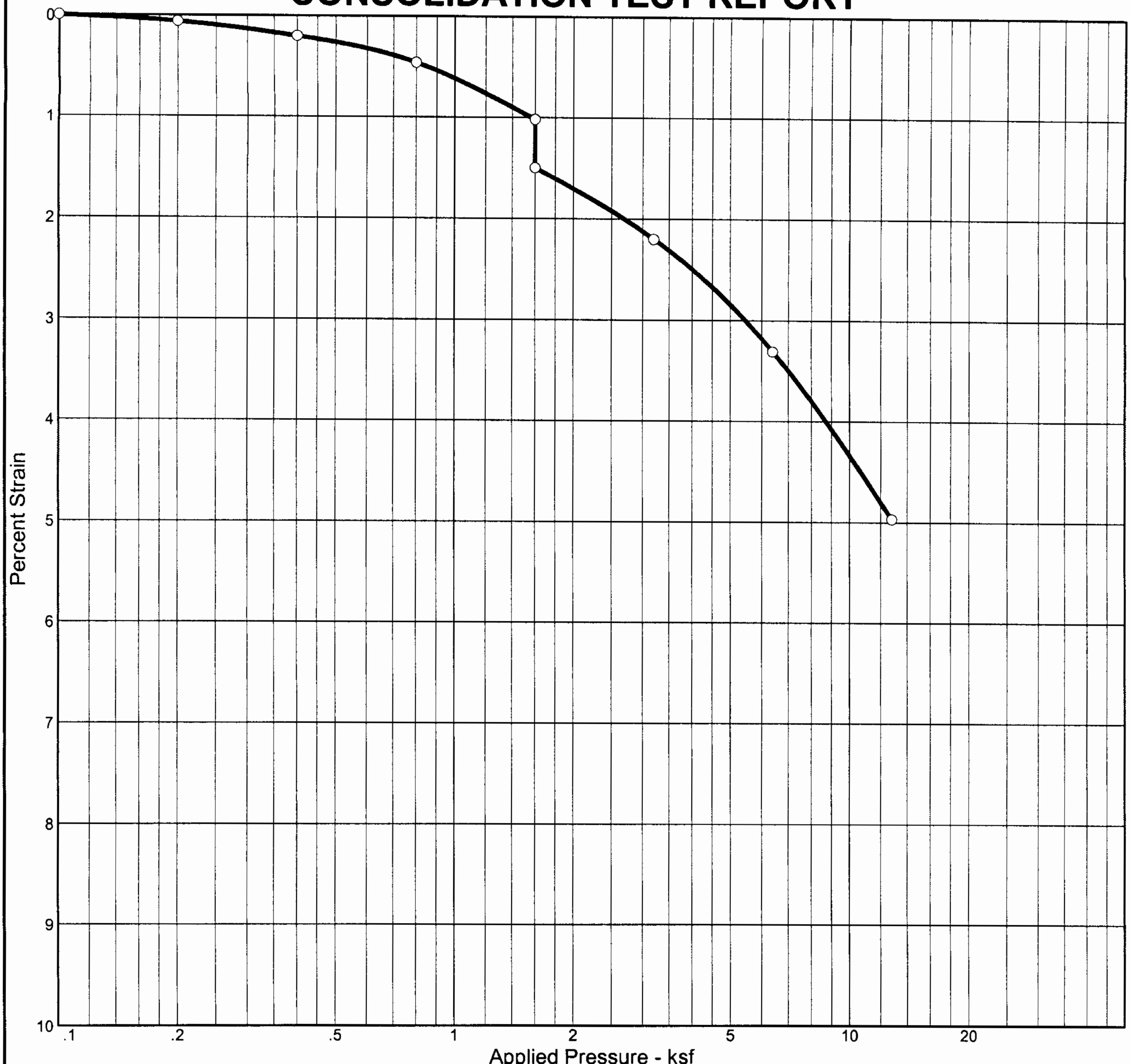
Remarks:
COLLECTED BY ED
COLLECTED ON (3/13/07)

CONSOLIDATION TEST REPORT

ENVIRONMENTAL AND GEOTECHNICAL ENGINEERING NETWORK CORPORATION

Figure

CONSOLIDATION TEST REPORT



Natural		Dry Dens. (pcf)	LL	PI	Sp. Gr.	Overburden (ksf)	P_c (ksf)	C_c	C_r	Swell Press. (ksf)	Clpse. %	e_o
Sat.	Moist.											
74.5 %	8.5 %	127.1			2.65		2.32	0.07			0.5	0.302

MATERIAL DESCRIPTION	USCS	AASHTO
SILTY SAND W/CLAY, BROWN	SM	

Project No. M3551-GS **Client:** REZA ZOLFAGHARI

Project: ZOLFAGHARI COMMERCIAL

Source: CONSOL **Sample No.:** B3 @ 10

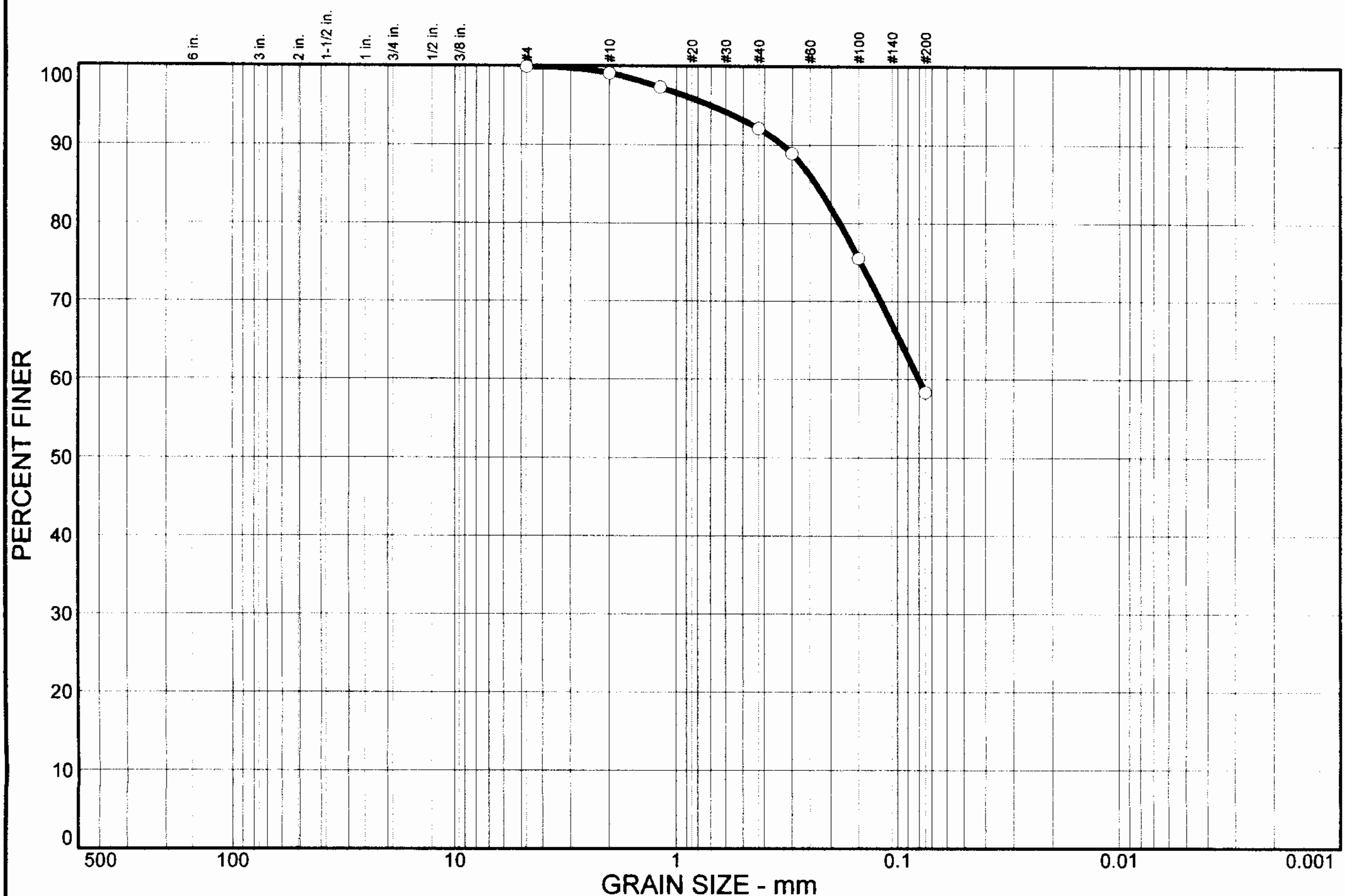
Remarks:
COLLECTED BY ED
COLLECTED ON (3/13/07)

CONSOLIDATION TEST REPORT

ENVIRONMENTAL AND GEOTECHNICAL ENGINEERING NETWORK CORPORATION

Figure

Particle Size Distribution Report



% COBBLES	% GRAVEL		% SAND			% FINES	
	CRS.	FINE	CRS.	MEDIUM	FINE	SILT	CLAY
			0.8	7.0	33.8	58.4	

SIEVE SIZE	PERCENT FINER	SPEC.* PERCENT	PASS? (X=NO)
#4	99.9		
#10	99.1		
#16	97.3		
#40	92.1		
#50	88.9		
#100	75.5		
#200	58.3		

* (no specification provided)

Material Description

FINE SANDY SILT, BROWN

Atterberg Limits

PL= LL= PI=

Coefficients

D_{85}= 0.234 D_{60}= 0.0802 D_{50}=
D_{30}= D_{15}= D_{10}=
C_u= C_c=

Classification

USCS= ML AASHTO=

Remarks

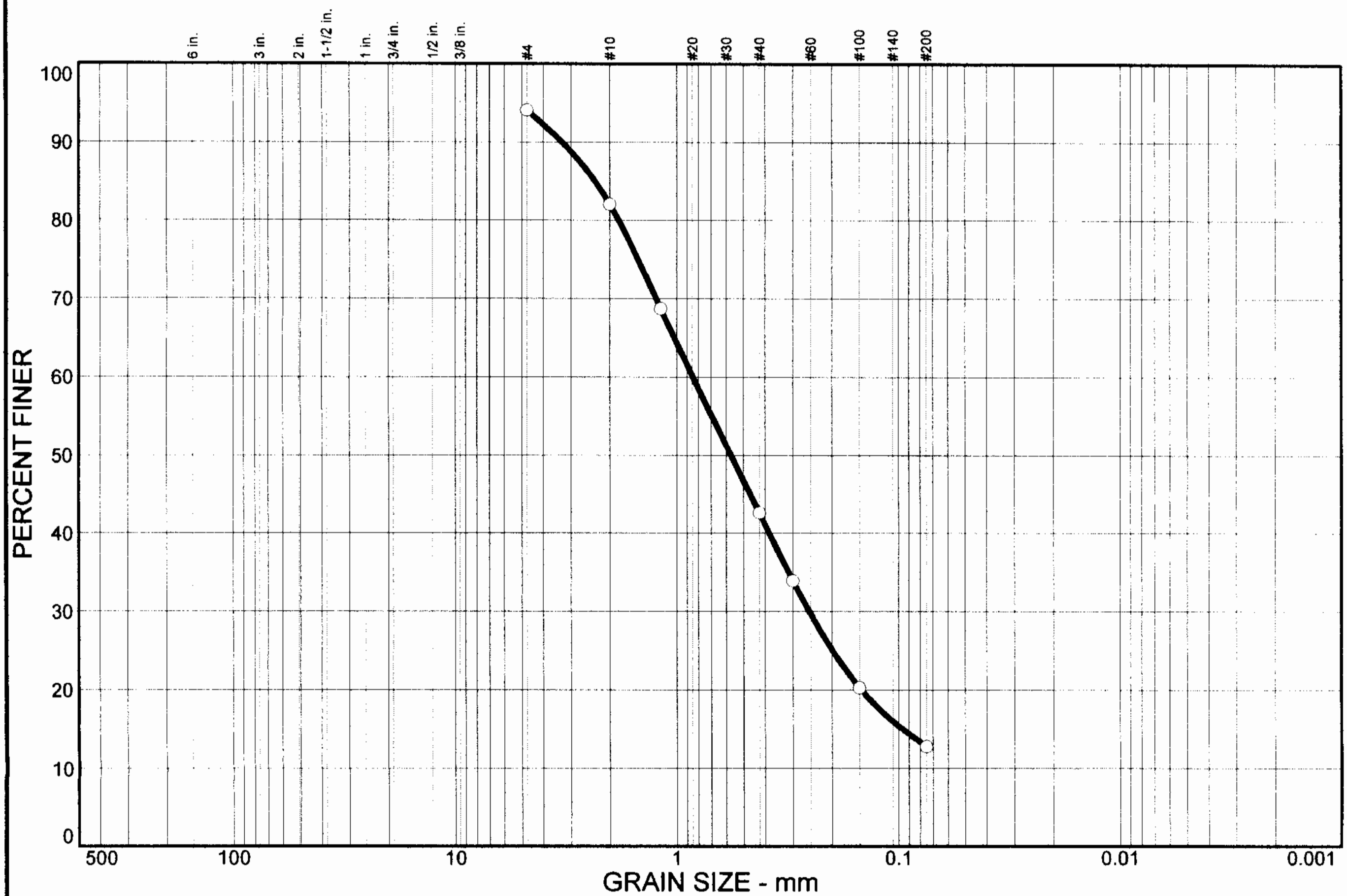
COLLECTED BY ED
COLLECTED ON (3/13/07)

Sample No.: B2 @ 10 **Source of Sample:** SIEVE **Date:** 3/22/07
Location: **Elev./Depth:**

ENVIRONMENTAL AND GEOTECHNICAL ENGINEERING NETWORK CORPORATION

Client: REZA ZOLFAGHARI
Project: ZOLFAGHARI COMMERCIAL
Project No: M3551-GS **Figure**

Particle Size Distribution Report



% COBBLES	% GRAVEL		% SAND			% FINES	
	CRS.	FINE	CRS.	MEDIUM	FINE	SILT	CLAY
			12.1	39.4	29.8	12.8	

SIEVE SIZE	PERCENT FINER	SPEC.* PERCENT	PASS? (X=NO)
#4	94.1		
#10	82.0		
#16	68.7		
#40	42.6		
#50	33.9		
#100	20.3		
#200	12.8		

* (no specification provided)

Material Description

SAND, GREY

Atterberg Limits

PL= LL= PI=

Coefficients

D_{85}= 2.34 D_{60}= 0.846 D_{50}= 0.570
D_{30}= 0.253 D_{15}= 0.0954 D_{10}=
C_u= C_c=

Classification

USCS= SP AASHTO=

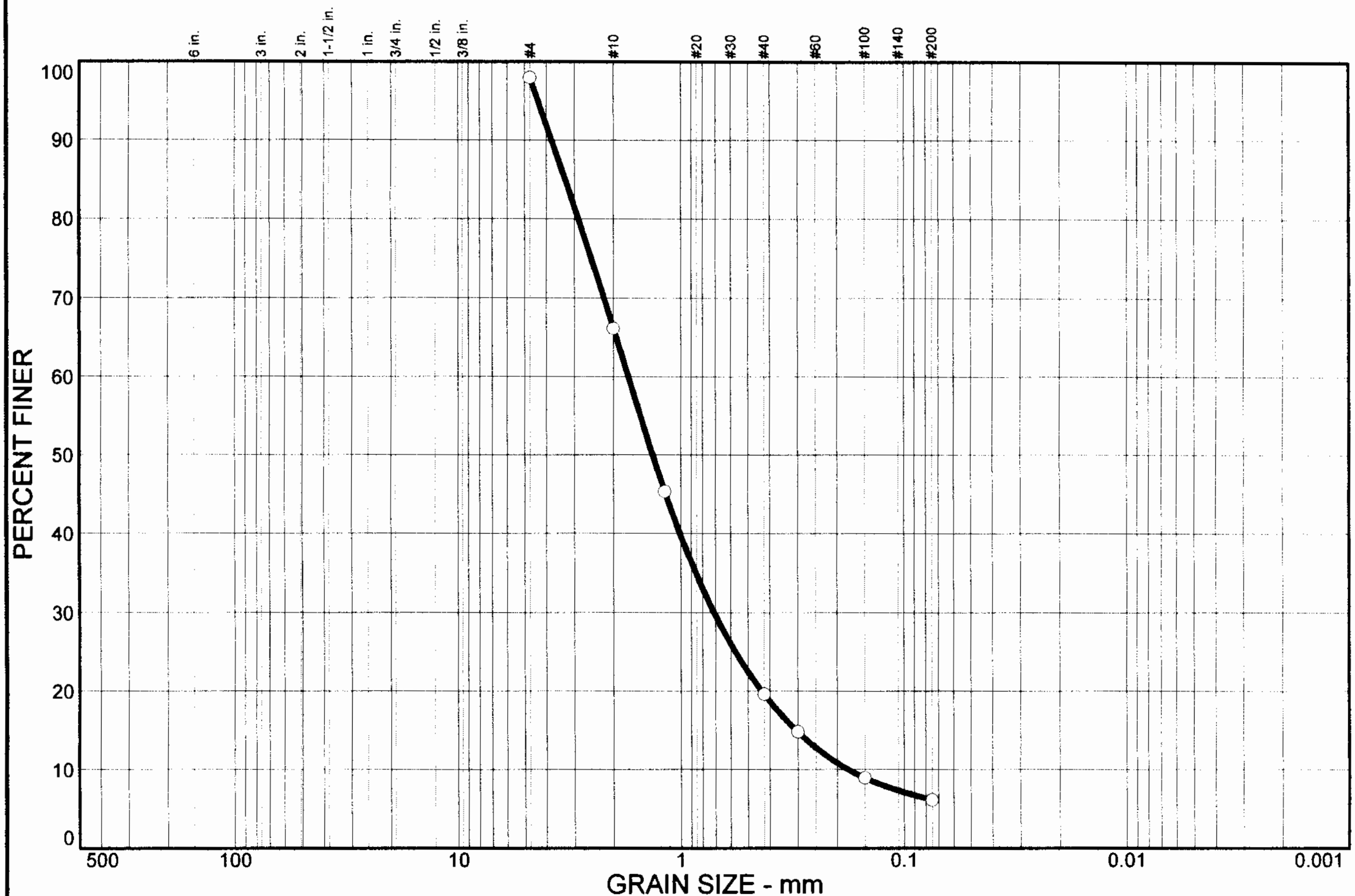
Remarks

COLLECTED BY ED
COLLECTED ON (3/13/07)

Sample No.: B2 @ 15 **Source of Sample:** SIEVE **Date:** 3/22/07
Location: **Elev./Depth:**

ENVIRONMENTAL AND GEOTECHNICAL ENGINEERING NETWORK CORPORATION	**Client:** REZA ZOLFAGHARI **Project:** ZOLFAGHARI COMMERCIAL **Project No:** M3551-GS **Figure**

Particle Size Distribution Report



% COBBLES	% GRAVEL		% SAND			% FINES	
	CRS.	FINE	CRS.	MEDIUM	FINE	SILT	CLAY
			31.9	46.5	13.5	6.1	

SIEVE SIZE	PERCENT FINER	SPEC.* PERCENT	PASS? (X=NO)
#4	98.0		
#10	66.1		
#16	45.3		
#40	19.6		
#50	14.8		
#100	8.9		
#200	6.1		

* (no specification provided)

Material Description

MEDIUM TO COARSE SAND, LIGHT BROWN

Atterberg Limits

PL= LL= PI=

Coefficients

D_{85}= 3.30 D_{60}= 1.72 D_{50}= 1.34
D_{30}= 0.711 D_{15}= 0.305 D_{10}= 0.179
C_u= 9.63 C_c= 1.65

Classification

USCS= SP AASHTO=

Remarks

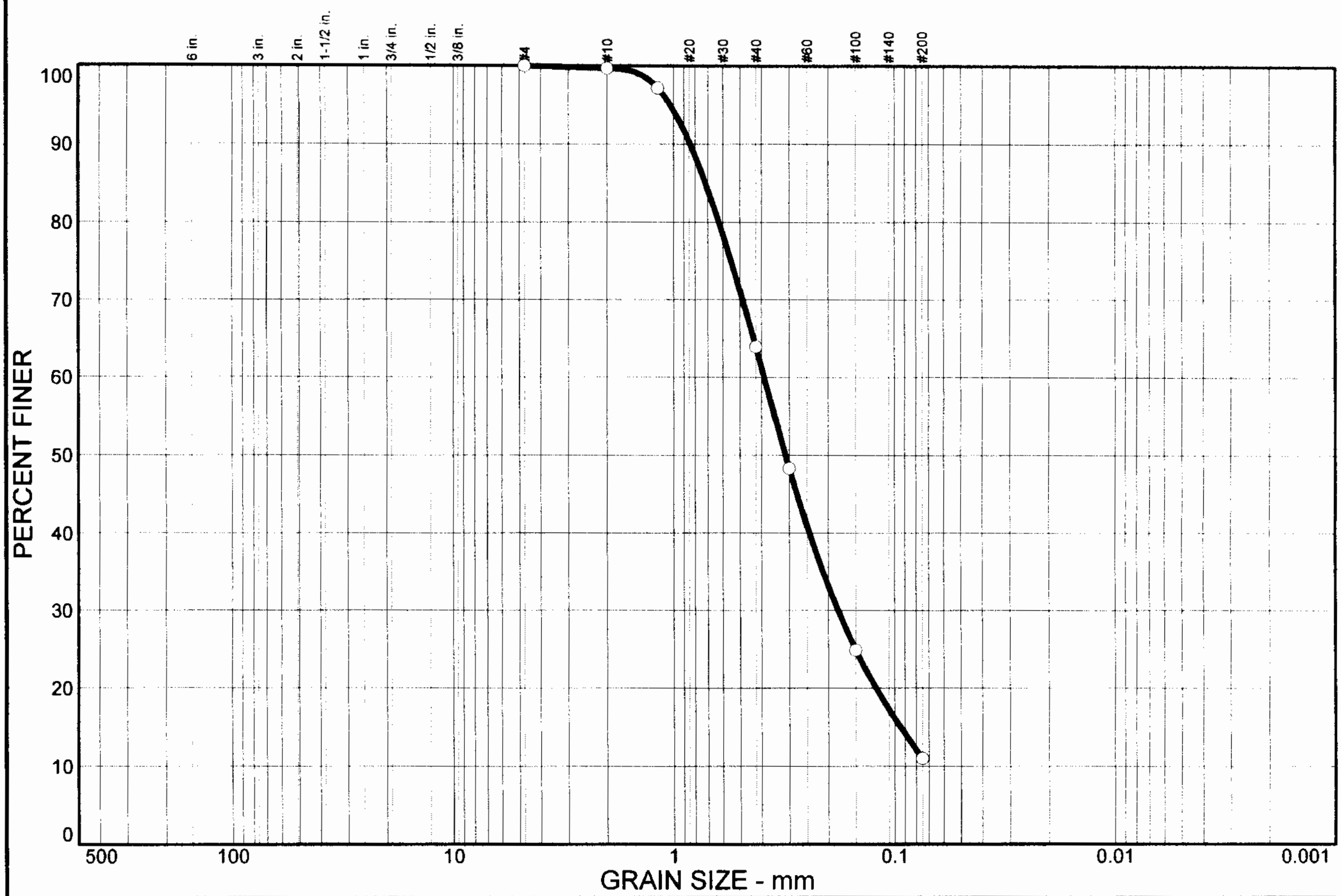
COLLECTED BY ED
COLLECTED ON (3/13/07)

Sample No.: B2 @ 35 **Source of Sample:** SIEVE **Date:** 3/22/07
Location: **Elev./Depth:**

ENVIRONMENTAL AND GEOTECHNICAL ENGINEERING NETWORK CORPORATION

Client: REZA ZOLFAGHARI
Project: ZOLFAGHARI COMMERCIAL
Project No: M3551-GS **Figure**

Particle Size Distribution Report



% COBBLES	% GRAVEL		% SAND			% FINES	
	CRS.	FINE	CRS.	MEDIUM	FINE	SILT	CLAY
0.0	0.0	0.0	0.3	35.8	52.9	11.0	

SIEVE SIZE	PERCENT FINER	SPEC.* PERCENT	PASS? (X=NO)
#4	100.0		
#10	99.7		
#16	97.2		
#40	63.9		
#50	48.3		
#100	24.9		
#200	11.0		

* (no specification provided)

Material Description

FINE SAND, LIGHT BROWN

Atterberg Limits

PL= LL= PI=

Coefficients

D_{85}= 0.717 D_{60}= 0.390 D_{50}= 0.312
D_{30}= 0.180 D_{15}= 0.0941 D_{10}=
C_u= C_c=

Classification

USCS= SP AASHTO=

Remarks

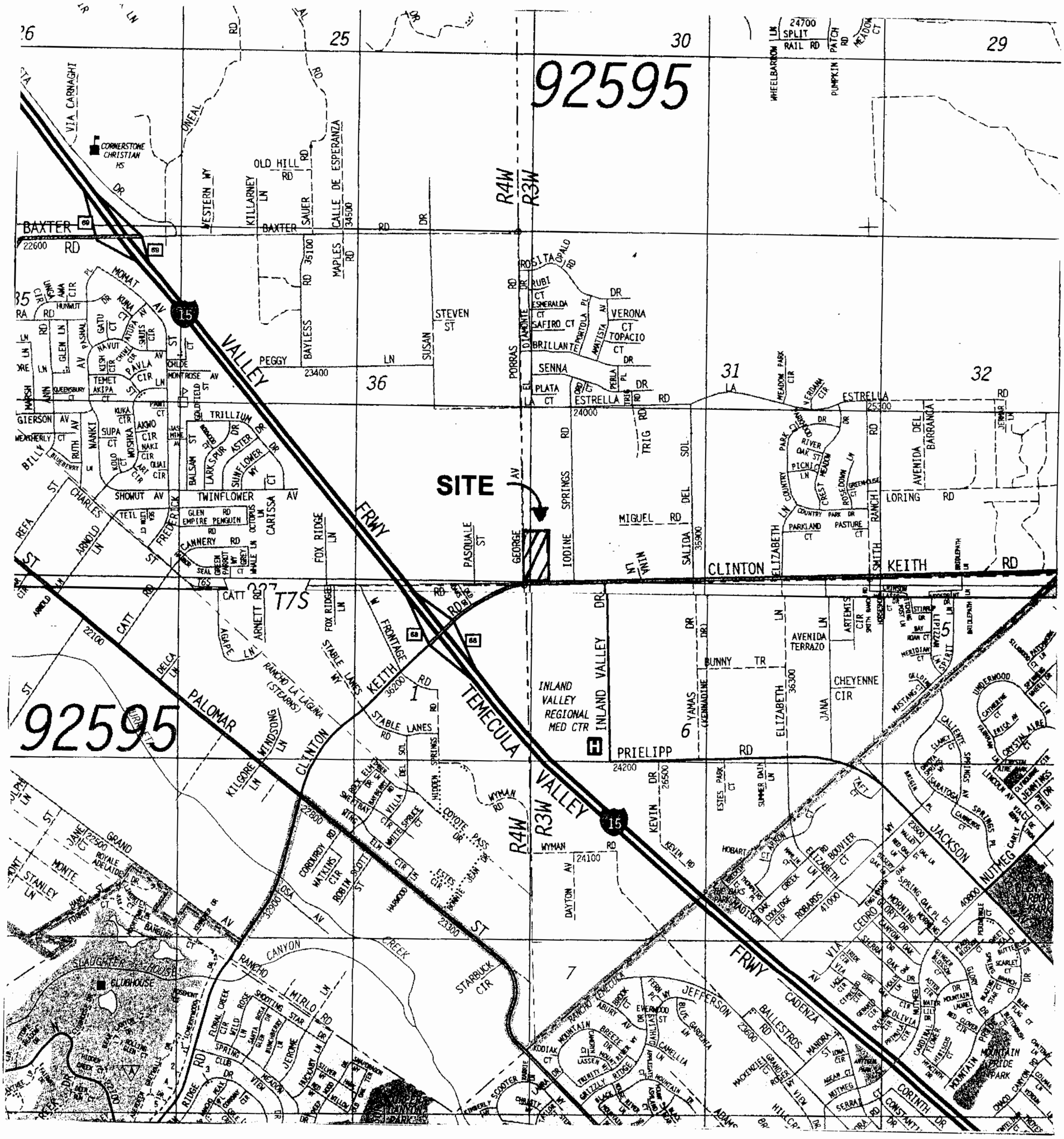
COLLECTED BY ED
COLLECTED ON (3/13/07)

Sample No.: B2 @ 40 **Source of Sample:** SIEVE **Date:** 3/22/07
Location: **Elev./Depth:**

ENVIRONMENTAL AND GEOTECHNICAL ENGINEERING NETWORK CORPORATION

Client: REZA ZOLFAGHARI
Project: ZOLFAGHARI COMMERCIAL
Project No: M3551-GS **Figure**

DRAWINGS



LEGAL DESCRIPTION: A.P.N.: 326-250-003	SITE LOCATION MAP	PROJECT NUMBER: M3551-GS
		DATE: APRIL 2007

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