Appendix D

Updated Geotechnical Investigation

UPDATED GEOTECHNICAL INVESTIGATION
for the
OAK CREEK PARK PROJECT FOR GRANUM HOLDINGS
at
Mount Hermon Road at Scotts Valley Drive
Scotts Valley, California
APN 022-162-76

Prepared For
MR. GREG EGER
GRANUM HOLDINGS
P.O. Box 2460
Saratoga, California

Prepared By
HARO, KASUNICH AND ASSOCIATES, INC.
Geotechnical & Coastal Engineers
Project No. SC11427
January 2018

RECEIVED
APR 12 2018
CITY OF SCOTTS VALLEY

Project No. SC11427
5 January 2018

Mr. Greg Eger
GRANUM HOLDINGS
P.O. Box 2460
Saratoga, California 95070

Subject: Updated Geotechnical Investigation Report

Reference: Oak Creek Park
APN 022-162-76
Mount Hermon Road at Scotts Valley Drive
Scotts Valley, California

Dear Mr. Eger,

In accordance with your authorization, we have prepared an updated geotechnical investigation report for the referenced project in Scotts Valley, California. The project consists of a new development which will include four new buildings: a mixed use building, a commercial building, and two apartment buildings. In addition, there will be new access driveways, garage parking and outdoor parking, sidewalks, landscaping, trash/recycling enclosure, and underground utilities. We also anticipate there will be new signage and lighting standards.

The original report for the project is entitled, *"Geotechnical Investigation Proposed Oak Creek Park Office Center Mount Hermon and Glen Canyon Roads Scotts Valley, California,"* by Kleinfelder, Inc., dated 8 April 1997.

In preparation of this updated geotechnical investigation report, we reviewed the 8 April 1997 geotechnical investigation report by Kleinfelder, and the preliminary architectural drawings by Thacher & Thompson Architects, dated October 2016 and February 2018. We also briefly visited the site to observe the current site conditions, and reviewed aerial photographs and geologic maps of the site. Additional subsurface investigation was not included in our scope of work. We performed engineering evaluation of the recommendations in the 8 April 1997 geotechnical report, updated recommendations as needed based on the results of our analyses, and provided updated seismic recommendations based on the current 2016 California Building Code (2016 CBC).

The primary geotechnical considerations for the project include loose fill soil which should be removed and replaced as engineered fill, moderate to highly expansive near-surface soils, buildings supported on a combination of cuts and fill, basements, proper foundation embedment, proper drainage, liquefaction and seismic considerations.

Our office should perform a geotechnical review of the pre-final project plans prior to city submittal. Our office should also be retained to perform geotechnical grading and foundation excavation observations and testing during construction.

If you have any questions concerning the data or conclusions presented in this updated geotechnical investigation report, please call our office. We are pleased to be of service on this project.

REGISTERED PROFESSIONAL ENGINEER
ROBERT G. HASSELER
GE 3074
GEOTECHNICAL
EXP. 12-31-2018
STATE OF CALIFORNIA

Respectfully Submitted,

HARO, KASUNICH AND ASSOCIATES, INC.

Robert Hasseler, PE, GE 3074

RH/rh

Copies: 4 to Addressee, Greg Eger, c/o Charles Eadie (charlie@eadieconsultants.com)

TABLE OF CONTENTS

GEOTECHNICAL INVESTIGATION

Introduction

This report summarizes the findings, conclusions, and recommendations of our updated geotechnical investigation report for the new Oak Creek Park project at Mount Hermon Road at Scotts Valley Drive, in Scotts Valley, California. Refer to Site Vicinity Map, Figure 1 in Appendix A of this report.

The original report for the project is entitled, *"Geotechnical Investigation Proposed Oak Creek Park Office Center Mount Hermon and Glen Canyon Roads Scotts Valley, California,"* by Kleinfelder, Inc., dated 8 April 1997.

We have updated the site and project descriptions to conform to our understanding of the currently planned project, and have provided additional analysis and recommendations for design considerations based on the currently planned project.

Based on our review, we are in general agreement with the recommendations presented in the April 1997 report except where changed herein. Within the text of this update report, we have incorporated both the accepted geotechnical design recommendations from the original report, and our revised or additional recommendations, to provide a complete standalone report for the project.

Purpose and Scope

The purpose of our work was to prepare an updated geotechnical investigation report with geotechnical design parameters and recommendations for the proposed improvements. Our specific scope of work included:

A. Review of aerial photographs and geologic maps of the site, and project administration.

B. Site visit to asses existing site conditions. Review of available data in our files regarding the neighborhood and region.

C. Review of the geotechnical investigation report by Kleinfelder, Inc., dated 8 April 1997.

D. Perform engineering evaluation of the recommendations in the 8 April 1997 geotechnical report, update recommendations as needed based on the results of our analyses, and provide updated seismic recommendations based on the current 2016 California Building Code (2016 CBC).

E. Preparation of a geotechnical update report to the existing 8 April 1997 geotechnical report and transfer of responsibility.

F. Preparation of a County of Santa Cruz soils engineer transfer of responsibility form. The soils engineer transfer of responsibility form can be found in Appendix B of this report.

Site Description and Condition

The site is located at Mount Hermon Road at Scotts Valley Drive, in Scotts Valley, California, as shown on the Site Vicinity Map, Figure 1 in Appendix A. Mount Hermon Road is located on the western side of the site and Glen Canyon Road is located on the southern side of the site. An existing office building and parking lot is located on the southeastern side of the site. Residential buildings along Lucia Lane are located to the northeast of the site, and an undeveloped area and a Shell service station are located to the north of the site at the intersection of Mount Hermon Road and Scotts Valley Drive. The site currently consists of an empty grass covered lot with several trees along the perimeter of the lot, especially on the northwestern portion of the lot. Sidewalks exist along Mount Hermon Road and Glen Canyon Road on the western and southern side of the site. A power pole is located on the western side of the site, and overhead powerlines cross the southern third of the site. The lot slopes gently to moderately downwards to the south, and drainage is generally to the southwest to a low area located on the southwestern portion of the site.

Project Description

We understand the proposed development will consist of four new buildings and associated improvements. The new buildings will include a mixed use building, a commercial building, and two apartment buildings. The four-story proposed mixed use building, Building A, will include commercial floor area, parking, and residential commons areas on the lower two floors, and residential apartments on the upper two floors. Building A will have a footprint of approximately 34,000 square feet and will include a partial basement. The commercial building, Building B, will have a ground floor and a mezzanine, and an approximately 8,900 square foot footprint. This building will include commercial floor area. The apartment buildings, Buildings C and D, will be the same as each other, and each will have two stories of residential apartments with a footprint of approximately 1,400 square feet each. Building A will be located on the southeastern portion of the property, cut into the existing slope to the east. Building B will be located on the southwestern portion of the site near the intersection of Mount Hermon Road and Glen Canyon Road, and the two apartment buildings, Buildings C and D, will be located on the northwestern portion of the site aligned with Mount Hermon Road. Outdoor parking lots will be located between Buildings A and B, and on the northeastern side of Buildings C and D. Primary access to the site will be off of Mount Hermon Road to the southwest of the site at the approximate middle of the site. A plan with the proposed improvements is shown on the Site Plan, Figure 2.

Specifics regarding the building construction are not known at this time, however we anticipate that Building A will consist of a combination of reinforce concrete construction on the lower floors, of the building and light weight wood frame construction on the upper floors of the building. Building A will also incorporate interior retaining walls up to about 21 feet in height. We assume that Buildings B, C and D will consist of light weight wood frame construction. We anticipate that all of the buildings will be supported on conventional spread footing foundations, with concrete slab-on-grade ground floors. Additional improvements will consist of sidewalks, pavements, landscaping, a trash/recycling enclosure and underground utilities. We also assume there will be new signage and lighting standards.

Field Exploration

Subsurface conditions were explored by Kleinfelder on 28 February 1997. A total of nine (9) borings were drilled to depths of between 11½ and 26½ feet below existing grade using a truck-mounted drill rig using 8-inch diameter hollow stem augers. Refer to the Boring Site Plan, Figure 3, for approximate boring locations and the Logs of Test Borings by Others in Appendix A of this report. Note that the boring log for Boring B-6 is missing from the 8 April 1997 Kleinfelder report.

Representative soil samples were obtained from the exploratory borings at selected depths using either a 2-inch inside diameter Modified California sampler or a 2-inch outside diameter split-spoon sampler (standard penetration test sampler, i.e. SPT). The penetration blow counts noted on the boring logs were obtained by driving a sampler into the soil with a 140-pound hammer dropping through a 30-inch fall. The sampler was driven up to 18 inches into the soil and the number of blows counted for each 6-inch penetration interval. The number of blows required to drive the sampler the last 12 inches (blows per foot) was converted to equivalent standard penetration blow counts and recorded on the boring logs. The soils encountered in the borings were logged in the field and described in accordance with the Unified Soil Classification System. The Logs of Test Borings are included in Appendix A of this report. The logs depict subsurface conditions at the approximate locations shown on the Boring Site Plan.

Additionally, five borings drilled by Jacobs, Raas & Associates in June and July 1988 were included as part of the 8 April 1997 report by Kleinfelder. These borings are Boring No's. 1, 2, 3, 5, and 6. Boring No. 4 was missing from the report. We assume similar drilling and sampling methods were used. Based on our experience, we anticipate that either hollow-stem or solid flight auger was used and that the L on the samples indicates a 2.5-inch inside diameter California Sampler (i.e. Large Sampler). We assume that the blowcounts (blows per foot) are raw field blowcounts unadjusted for sampler size.

Subsurface conditions at other locations may differ from those encountered at the explored locations. Stratification lines shown on the logs represent the approximate boundaries between soil types. The actual transitions may be gradual.

Laboratory Testing

Laboratory tests were performed on selected samples by Kleinfelder to evaluate their physical characteristics and engineering properties. Tests that were performed included in-situ moisture and unit weight, pocket penetrometer, and Atterberg limits. The pocket penetrometer gives an evaluation of the consistency and approximate unconfined compressive strength of soil. The Atterberg Limits test offers a relative potential of the expansivity of the soil sample. The results suggest a medium to high potential for shrinking and swelling of the near-surface soil.

Laboratory tests that were performed by Jacobs, Raas & Associates and depicted on their boring logs include in-situ moisture and unit weight, pocket penetrometer, unconfined compression and direct shear. Unconfined compression tests evaluate the unconfined compressive strength of soil, and direct shear tests provide the strength characteristics of the soil as cohesion and internal friction angle of the soil.

The results of the laboratory testing can be found in Appendix A on the "Logs of Test Borings by Others" opposite the sample tested. The results of the Atterberg limits test by Kleinfelder is also presented graphically in Appendix A.

Subsurface Conditions

Surface conditions at the site consist of grass and weeds, as well as numerous trees along the perimeter of the site, and a cluster of trees on the northern portion of the site. The ground surface at the boring locations consists primarily of grass and weeds.

The upper soil layers across the site consisted of sandy clay and silty clay materials as shown on the 28 February 1997 borings by Kleinfelder. These fine grained, upper soil layers extended from the ground surface to the maximum depths of the borings of approximately 26½ feet below exiting grade, with a couple of exceptions. In Kleinfelder Boring B-5, the silty clay layer extended to about 22 feet below existing grade and was underlain by medium dense poorly graded sand to the bottom of the boring at 26½ feet below existing grade, and in Boring B-8 was a thin layer of medium dense silty sand from about 4½ to 7½ feet below existing grade. The fine grained sandy clay and silty clay soils were typically medium stiff to stiff in the upper 0 to 5½ feet below the existing ground surface. Deeper fine grained soils were typically very stiff to hard in consistency.

Soils logged in the June and July 1988 borings by Jacobs, Raas & Associates showed various layers of soil described as sand, silty sand, silty sand with clay, sandy silt, silt, sandy clayey silt, sandy clay, clayey silt, and silty clay. Fine grained soils were typically described as firm to very hard in consistency. Pocket penetrometer tests on the soils were typically very stiff to hard. Coarse grained soils were described as loose to dense.

Groundwater was encountered in Kleinfelder Boring B-4 at 16½ feet below the existing ground surface (according to the report text) and in Boring B-5 at 19 feet below the existing ground surface. Groundwater was not encountered in the other borings drilled by Kleinfelder at the site. Groundwater was encountered in Jacobs, Raas & Associates Boring 1, 2, 3, 5, and 6 at 15¾, 17½, 25, 19, and 8½ feet below the existing ground surface, respectively. Refer to the "Logs of Test Borings by Others" in Appendix A of this report.

General Geologic Setting

Based on the County of Santa Cruz Geographic Information Services website at gis.co.santa-cruz.ca.us/PublicGISWeb, the site is described as underlain by Quartz Diorite on the eastern portion of the property and Alluvial Deposits on the western portion of the property. No faults or landslides are mapped at the site. The western third of the site is mapped as having a moderate potential for liquefaction, with the mapped liquefaction zone grazing the western corner of Building A near the planned apartment entrance, as well as passing under parts of Buildings B, C and D.

The site is located in the seismically active Santa Cruz County area, but not within any of the Alquist-Priolo Earthquake Fault Zones established by the Alquist-Priolo Earthquake Fault Zoning Act of 1972. Therefore, the risk of ground rupture occurring across the site is low

California Building Code Seismic Site Class

The improvements should be designed in conformance with the most current California Building Code (2016 CBC). Because this is a potentially liquefiable site, it should be classified as site soil profile F, which requires a site specific seismic response evaluation. However, if the structure has a fundamental period less than or equal to 0.5 seconds, the site may be classified as site soil profile D. If the structural period is in excess of 0.5 seconds, Haro, Kasunich and Associates should be consulted to perform a site-specific analysis. For seismic design, the soil properties at the site are classified as Site Class "D" based on definitions presented in Table 1613.5.2 in the 2016 CBC. The longitude and latitude were determined using a satellite image generated by Google Earth. These coordinates were taken from the approximate middle of the area of the proposed improvements:

Longitude = -122.02375, Latitude = 37.04190

The coordinates listed above were used as inputs in the Java Ground Motion Parameter Calculator created by the USGS to determine the ground motion associated with the maximum considered earthquake (MCE) S_M and the reduced ground motion for design S_D. The results are as follows:

<u>Site Class D</u>
S_S= 1.507 g
S_1= 0.603 g
F_A= 1.000
F_V= 1.500
S_{MS}= 1.507 g
S_{M1}= 0.904 g
S_{DS}= 1.005 g
S_{D1}= 0.603 g

A maximum considered earthquake geometric mean (MCE_G) peak ground acceleration (PGA) was estimated using the Figure 22-7 of the ASCE Standard 7-10. The mapped PGA was 0.556 g and the site coefficient F_{PGA} for Site Class D is 1.0. The MCE_G peak ground acceleration adjusted for Site Class effects is $PGA_M = F_{PGA} * PGA$

$$\boxed{PGA_M = 1.0 * 0.556\text{ g} = 0.556\text{ g}}$$

Liquefaction

Liquefaction is a phenomenon where the loose or medium dense sand or in some cases firm silt deposited below the groundwater table experiences a loss of shear strength while cyclically loaded by the ground shaking during an earthquake. Modern geotechnical engineering practice assumes ground failures can occur from soil deposits liquefying within 50 to 60 feet of the ground surface. The primary effect of liquefaction is settlement of the ground surface after an earthquake, with the potential for lateral movement, and in some cases ground disruption in the form of sand boils or ground cracking.

In general, the upper 16½ to 26½ feet bgs at the subject site were described as sandy clay and silty clay. These clay type soils had measured plasticity index (PI) of 25, natural moisture content less than 90 percent of the liquid limit (LL) of 45, contained predominately fine grained material, and plotted on the Atterberg Limits test as lean clay to sandy lean clay with a moderate to high plasticity. Cohesive type soils with these properties have very low potential for liquefaction and related effects (SP117A).

However, some isolated discontinuous soil deposits susceptible to liquefaction were encountered below the groundwater table, and within the upper 50 feet of the ground surface. Groundwater was encountered at the site in Kleinfelder Boring B-4 at 16½ feet, and in Boring B-5 at 19 feet bgs. Groundwater was encountered in Jacobs, Raas & Associates' borings from 8½ feet to 25 feet bgs. An estimated 5 foot thick liquefiable zone was encountered at about 22 feet below existing grade in Kleinfelder Boring B-5, a potentially up to 2½ foot thick liquefiable zone was encountered at about 13½ feet below existing grade in Jacobs, Raas & Associates' Boring 1, and an estimated up to 5 foot thick liquefiable zone was encountered at about 14 feet below existing grade in Jacobs, Raas & Associates' Boring 6. It should be noted that Kleinfelder Boring B-5, and Jacobs, Raas & Associates' Boring 6, were terminated in the potentially liquefiable layer. The estimated thicknesses of their liquefiable layers were estimated based on the subsurface profiles of the other nearby borings, site geology, and engineering judgement. Note also that Kleinfelder Boring B-5, and Jacobs, Raas & Associates' Boring 1, were drilled on a portion of the original site that is located to the south of the current site boundary. The boring locations indicating potential liquefaction were in general agreement with the mapped moderately liquefiable zones as limited to the subject site as based on the County of Santa Cruz Geographic Information Services website.

Liquefaction analysis was performed using Liquefy Pro Version 5.8d by Civiltech Software. The software allows users to input ground acceleration and soil profiles with field and laboratory test results. The software determines a factor of safety (FS) against liquefaction. Soil layers with FS < 1.0 are considered to have liquefied and related settlement of the soil layer is estimated. Liquefaction analyses were performed on the profiles of Borings B-5, 1, and 6. Since these test holes were not advanced to depths of 50 feet bgs our evaluation was limited to the upper 15 to 26½ feet below existing grade. However, the potential liquefiable soils are discontinuous and not anticipated to extend significantly past this depth, extrapolating from the hard clays, siltstone, and drilling refusal encountered at depth in nearby borings.

Estimated total settlement at the location of Kleinfelder Boring B-5 was calculated to be approximately 1½ inches with a differential settlement of about ¾ inches over 50 horizontal feet. Estimated total settlement at the location of Jacobs, Raas & Associates Boring 1 was calculated to be approximately ½ inches with a differential settlement of about ¼ inches over 50 horizontal feet, and estimated total settlement at the location of Jacobs, Raas & Associates Boring 6 was calculated to be approximately 1¾ inches with a differential settlement of about 1 inch over 50 horizontal feet.

Ishihara (1985) presented criteria for assessing the potential for ground disruption at liquefaction sites. Those criteria are based on relationship between thickness of liquefiable layers beneath a site and corresponding thickness of the overlying non-liquefiable soil. The criteria were graphically summarized as boundary curves for discriminating between occurrence and non-occurrence of surface effects of liquefaction. Based on the results of our liquefaction analysis for this site liquefiable layers ranged between 13½ to 22 feet thick. Non-liquefiable soil layers overlying the liquefiable soil layers were estimated to be between 2½ to 5 feet thick. Using Ishihara's criteria, the potential for occurrence of surface effects from the liquefiable soils below 13½ feet bgs is low to moderate and the potential for occurrence of surface effects from the liquefiable soils below 22 feet bgs is low.

We estimate that some of the liquefaction related settlement in the areas of the borings could reflect to the surface through the non-liquefiable soil layers. We do not anticipate ground failures to occur in the form of sand boils or ground cracking, however the ground surface may settle or depress between an estimated ½ to 1¾ inches total and ¼ to 1 inches differentially in the areas of Buildings B, C, and D. Total post-earthquake settlements over 1 inch could result in non-uniform settlement of masonry-walled structures. Differential settlements of 1 inch in 50 feet (i.e. angular distortion of 1/600) could result in cracking of brick walls or veneer and plaster, and is just at the limit of danger for frames with diagonals. We would not typically expect structural failure at this low angular distortion (although this should be verified by the structural engineer), and the need for some repairs (and possible releveling of the building) after a major earthquake near the site should be expected. In general, buildings are designed for life safety after a major earthquake, as opposed to a fully functioning building, and some repairs after a major earthquake are expected.

Where buildings cannot tolerate the estimated potential liquefaction settlements, the potential settlement can be reduced by supporting the buildings on deep foundations. Generally, the expense of deep foundations to reduce limited settlement is not justified, as the cost for deep foundations is usually greater than the cost of repairs. Recommendations for drilled pier foundations (primarily for use for support of signage, and lighting standards) are presented in the report, and may be used for the new buildings. Alternatively, conventional foundation can be reinforced to limit differential movement in critical areas. Since Building A will have a partial basement, is outside the mapped area of Liquefaction, and also is outside of the areas where Liquefaction is indicated in the borings, we estimate a low probability of seismic settlement at Building A.

Lateral spreading (horizontal displacement) of the ground surface can occur where liquefaction occurs in areas of sloping ground or at a free face, such as a creekbank or riverbank. Due to lateral confinement, the relatively thick layer of overlying clayey soils, and the discontinuous nature of the sandy layers, the potential for lateral spreading is considered to be low.

Initial Settlement

Due to the very stiff to hard consistency of the deep fine-grained soils underlying the site, the site is not subject to settlement from consolidation settlement or dynamic compaction. Provided our recommendations are incorporated into the design and construction of the project, post-construction total and differential settlement of foundations due to static loading are considered to be low. Potential initial foundation settlements (total and differential) are expected to be less than about an inch total settlement and ¾ inch differential settlement if the entire structure is properly founded on conventional foundations on engineered fill as recommended in the section on "Recommendations" herein.

Expansive Soils

Based on the results of our Atterberg Limits test for the near surface soils, the site is judged to have a very moderate to high expansion potential. Expansive soils undergo volume changes (i.e. shrinking and swelling) as a result of moisture content changes. These changes can come from rainfall, roof drainage, irrigation, perched water, leaking utilities, drought or other considerations. Moisture changes can result in unacceptable movement of structures supported on expansive soils, with the structures settling in dry conditions and heaving in wet conditions. This can result in cracking and damage to foundations, concrete slabs-on-grade, and pavements. Expansive soils can be mitigated by deepening foundations, supporting concrete slabs-on-grade on a layer of non-expansive fill material, and thickening pavement sections. Although these methods will greatly reduce the amount of movement, some movement should still be anticipated. Specific recommendations regarding expansive soils are presented in the pertinent sections in the section on "Recommendations," herein.

Loose/Weak Fill Soils

Loose or weak near surface fill soils were encountered on portions of the project site. These soils were primarily encountered on the southern portions of the site. If these soils are not to be removed as part of cuts, the loose/weak soil where encountered during general site grading, should be removed and replaced as engineered fill as recommended in the section on "Recommendations" herein.

Cut-Fill Transitions

Based on the layouts presented in the preliminary architectural drawings by Thacher & Thompson Architects, we understand that portions of the buildings will be supported on transitions between cuts and engineered fills. Due the relatively dense incompressible nature of the deeper, less weathered soils on the site, we anticipate that structures supported on both cuts and engineered fill materials will experience differential settlement between the cut and fill zone. Specifically, the portions off the buildings supported on fills are expected to settle more than the portions of the buildings supported on the cuts. To reduce the differential settlement, we recommend that the entire building pads of buildings supported on both cuts and engineered fills be over-excavated at least 2 feet below the bottoms of the conventional foundations, and the entire building be supported on a layer of engineered fill. If buildings are supported on drilled pier and grade beam foundations (optional) then the drilled piers would derive their support from the deeper firm soils and this remedial over-excavation will not be needed, except for removal of expansive soils from under the concrete slab-on-grade floor slabs, as discussed in the section on "Expansive Soils," herein.

RECOMMENDATIONS

Based on the results of our study, the proposed development is feasible from a geotechnical standpoint, provided the design criteria and recommendations presented in this report are incorporated into the design and construction of the project.

The key site-specific geotechnical concerns include proper site grading and earthwork, foundation embedment, proper drainage, and seismic considerations. The proposed structures, and associated improvements such as retaining walls, light standards, and signage, can be supported on conventional spread footing foundations or a drilled pier foundation system. Recommendations for both foundation systems are presented in the report.

Waterproofing and control of all surface and roof runoff and sub-surface seepage is of key importance to the performance of the structure.

Refer to the following criteria and recommendations for grading, foundations, retaining walls, slabs-on-grade, pavements, and drainage.

Plan Review Notice

Haro, Kasunich & Associates should be provided an opportunity to review the project plans during the design phase prior to cost estimating and city submittal. Allow at least one week for this task. The review provides an opportunity to check if our recommendations have been interpreted properly, which could reduce possible confusion and costly changes and time delays during construction. Once the plans meet our recommendations sufficiently we can provide the city-required plan review letter. Please contact our office at (reference Project Number SC11427):

Haro, Kasunich & Associates
116 East Lake Avenue
Watsonville, California 95076
(831) 722-4175 ext 115
bhasseler@harokasunich.com

Construction Observation Notice

Haro, Kasunich and Associates must provide observation and testing services for earthwork performed at the project site. The observation and testing of earthwork allows for evaluation of contractors' compliance with our geotechnical recommendations. It also allows us the opportunity to confirm that actual soil conditions encountered during construction are essentially the same as those anticipated based on the subsurface exploration. Unusual or unforeseen soil conditions may require supplemental evaluation by the geotechnical engineer.

The City usually requires a final grading and/or foundation compliance letter. We can prepare this letter only if we are called to the site to observe and test, as necessary, any grading and excavation operations **from the start of construction**. We cannot prepare a letter if we are not afforded the opportunity of observation from the **beginning of the grading operation**. The contractor must be made aware of this and earthwork testing and observation must be scheduled accordingly. Refer to contact information above.

Site Grading

1. The geotechnical engineer should be notified **at least four (4) working days prior to any grading or foundation excavating** so the work in the field can be coordinated with the grading contractor and arrangements for testing and observation can be made. The recommendations of this report are based on the assumption that the geotechnical engineer will perform the required testing and observation during grading and construction. It is the owner's responsibility to make the necessary arrangements for these required services.

2. Compaction during inclement weather or wet conditions may hamper compaction efforts and over-excavation may be necessary.

3. Where referenced in this report, Percent Relative Compaction and Optimum Moisture Content shall be based on ASTM Test Designation D1557.

4. Areas to be graded or designated to receive engineered fill, should be cleared of all obstructions, buried utilities, and tree roots.

5. In areas to be graded or designated to receive engineered fill, foundations, flatwork, pavements or hardscape, all loose or weak soil and other unsuitable material must be subexcavated to its full depth. Existing depressions or voids created during site clearing should be backfilled with engineered fill.

6. Cleared and subexcavated areas should then be stripped of organic-laden topsoil. Strippings should be wasted off-site or stockpiled for use in landscaped areas if desired.

7. Buildings with conventional footing foundations supported entirely on cuts, and buildings supported on drilled pier foundations, should have their building pad subexcavated as needed to allow placement of 15 inches of compacted non-expansive fill between the prepared native subgrade and the bottoms of any concrete slabs-on-grade. Exposed subgrade should be prepared as discussed in Item Number 10, herein. Non-expansive fill should meet the requirements for import fill as presented in Item Number 13, herein.

8. Buildings with conventional footing foundations, supported entirely on fill, or on both cut and fill, should be supported on at least 24 inches thickness of engineered fill below the bottoms of the foundations. This will require subexcavation in thin fill areas and in cut areas. Exposed subgrade should be prepared as discussed in Item Number 10, herein. Engineered fill consisting of approved on-site material or import fill material should be installed to finished subgrade elevation, except that the upper 15 inches of the engineered fill located below concrete slabs-on-grade should consist of non-expansive fill meeting the requirements for import fill as presented in Item Number 13, herein.

9. Exterior concrete slabs on grade should be supported on at least 8 inches thickness of non-expansive engineered fill over properly prepared subgrade soil. Exposed subgrade should be prepared as discussed in Item Number 10, herein. Non-expansive fill should meet the requirements for import fill as presented Item Number 13, herein.

10. Exposed subgrade should be scarified at least 8 inches; moisture conditioned to at least 2 percent over optimum moisture and compacted to 90 percent relative compaction. Subgrade

preparation should extend a minimum of 5 feet horizontally from new building footing perimeters and adjacent sidewalks, and a minimum of 2 feet horizontally from non-adjacent sidewalks and pavements (back of curbs) and areas to receive engineered fill, except where obstructed by improvements to remain. Where obstructed by improvements to remain, the subgrade preparation may be terminated at the edge of the improvement. Engineered fill consisting of onsite soils should be placed in thin lifts not exceeding 8 inches in loose thickness; moisture conditioned to at least 2 percent over optimum moisture, and compacted to a minimum of 90 percent relative compaction, up to desired grade. Engineered fill consisting of import non-expansive soils should be placed in thin lifts not exceeding 8 inches in loose thickness; moisture conditioned to over the optimum moisture, and compacted to a minimum of 90 percent relative compaction, up to desired grade. See the section on Flexible Pavements for compaction recommendations under asphalt concrete pavements.

11. Moisture conditioning consists of adding water where the soils are below the optimum moisture percentage and drying back soils that are too high over the optimum moisture percentage to achieve the recommended relative compaction. The contractor should anticipate that the on-site soils will require moisture conditioning and processing, either adding water or drying the soils. In addition to meeting the recommended percentage compaction, compacted soils should be firm and unyielding under proof-rolling with construction equipment, such as a full water truck.

12. The aggregate base should be moisture conditioned and compacted to at least 95 percent relative compaction.

13. On-site material at optimal moisture contents may be used in engineered fill, except where non-expansive fill is required. Imported fill material should consist of a predominantly granular soil conforming to the quality and gradation requirements as follows: The soil should be relatively free of organic material and contain no rocks or clods greater than 4 inches in diameter, with no more than 15 percent larger than 2 inches. The material should be predominately granular with a plasticity index less than 15, a liquid limit less than 30, and not more than 20 percent passing the #200 sieve.

14. Subgrade in areas to receive engineered fill should be prepared as discussed in Item Number 10, herein. Engineered fill slopes should be inclined no steeper than 2:1 (horizontal to vertical) for heights up to 10 feet. Where 2:1 fill slopes are not attainable, retaining walls or slope reinforcement, or other means, will be needed to steepen the slope. Fill embankments situated on slopes between 20% and 50% in gradient should be drained, keyed and benched into firm on-site material. All keys and benches should be drained. Fills should not be situated on slopes steeper than 50% in gradient. The toe of fill embankments should be setback at least 10 feet from a steep break in slope.

15. Cut and fill slopes should be protected from erosion by preventing runoff from spilling over fresh slopes. Lined V-ditches and/or berms at the top of the slope may be considered for the short and long term.

16. Driveway, access roads, flatwork and landscape features should be set back at least 5 feet from the crest of a slope. This criterion can be revised on a case by case basis.

17. Following grading, exposed bare slopes and soil should be planted or covered as soon as possible with erosion resistant vegetation or blanket.

18. After the earthwork operations have been completed and the geotechnical engineer has finished his observation of the work, no further earthwork operations shall be performed except with the approval of and under the observation of the geotechnical engineer.

Excavations

19. The Owner/Client and the contractor should make themselves aware of and become familiar with applicable local, state and federal safety regulations, excavation and trench safety standards. Construction site safety and temporary shoring generally is the sole responsibility of the contractor, who shall also be solely responsible for the means and methods, and sequencing of construction operations. Under no circumstances should the information provided below be interpreted to mean that HKA is assuming responsibility for construction site safety for the contractors' activities; such responsibility is not implied and should not be inferred.

20. Excavations are highly susceptible to sloughing. Protection and safety of workers is a key element of design. Excavation should not be performed in inclement weather. Excavations should not remain open or exposed to runoff.

21. Shallow temporary excavations for conventional foundations and shallow utilities up to about 5 feet deep are expected to stand vertical for short periods, provided the sidewalls of the excavations are kept moist and are not allowed to dry out or become saturated with water. Deeper excavations into the native soils, such as for the parking garage basement for Building A, will require shoring and bracing or the sidewalls laid back at a maximum slope of inclination of 1½:1 (horizontal to vertical). Shoring will be necessary for steeper temporary cuts or where sloping or benching back is not feasible. Actual configuration of benching and or shoring for temporary cuts should be handled on a case by case basis.

22. Temporary shoring walls may be designed to be part of the permanent walls.

23. Retaining walls may be designed to be a part of the structure or remain independent of the structure.

24. Permanent cut slopes exposing firm on-site soil should be cut no steeper than 2:1 (horizontal to vertical) for heights up to 10 feet.

Spread Footing Foundation System

25. Shallow reinforced concrete spread footings, founded in engineered fill or firm on-site soil, may support new structures. Please refer to Item Number 7 and Item Number 8, herein for additional requirements for conventional foundation support.

26. Footings should be deep enough to accommodate a horizontal distance of at least 5 feet between the bottom of all foundation elements and the surface of adjacent slopes.

27. Continuous footings and isolated footings should be embedded into engineered fill or firm on-site soil at least a full 24 inches for Building A, and at least 18 inches for Buildings B, C and D. Continuous footings should be at least 18 inches wide and isolated footings should be at least 24 inches wide. The foundation trenches should have horizontal bases and be stepped where situated on sloping ground.

28. Foundations designed in accordance with the above, may be designed for an allowable soil bearing pressure of 2,500 psf. This value may be increased by one-third to include short-term seismic and wind loads.

29. Lateral load resistance for structures supported on spread footings may be developed by a combination of friction between the foundation bottom and the supporting subgrade and passive resistance against the buried portion of the footing. A friction coefficient of 0.30 is considered applicable. In addition, a passive resistance of an equivalent fluid weight of 300 pounds per cubic foot against the footings may be used. The passive resistance may be increased by one-third to include short-term seismic and wind loads. The passive resistance can be assumed to start at the top of the lowest adjacent grade in pavement and concrete flatwork areas, and at a depth of 1 foot below grade in unpaved areas. These lateral load resistance values only apply to footings placed directly against undisturbed engineered fill or on-site soil, or where the voids from forming are backfilled with engineered fill.

30. All footings should be reinforced in accordance with applicable CBC and/or ACI standards.

31. New footings located adjacent to other footings or utility trenches should have their bearing surfaces founded below a 1½:1 (horizontal: vertical) line projected upward from the bottom edge of the adjacent footings or utility trenches.

32. The foundation trenches must be thoroughly cleaned of all slough or loose material prior to pouring concrete.

33. The footing excavations must be **observed by the geotechnical engineer prior to placement of forms and rebar** to verify subsurface soil conditions are consistent with the anticipated soil conditions so that the city required foundation excavation conformance letter can be prepared.

Reinforced Concrete Pier Foundation System

34. The proposed structures, and associated improvements such as retaining walls, light standards, and signage, can be supported on a drilled pier foundation system. These may consist of a pier and grade beam system, such as for the buildings and retaining walls, or individual piers such as for light standards and signage.

35. Piers must accommodate a horizontal distance of 5 feet between the base of the pier and the surface of the adjacent slopes. Actual pier depth will be determined by structural engineer based on loading, pier diameter and spacing.

36. Pier spacing must not be closer than three times the pier diameter. Actual spacing and depth of piers will be determined by the structural engineer.

37. Piers for short retaining walls, light standards, and signage should be at least 5 feet deep. Piers for the new buildings should be at least 10 feet deep. An average skin friction capacity of 350 psf may be used for piers at least 5 feet deep and an average skin friction capacity of 500 psf may be used for piers at least 10 feet deep. These values may be increased by one-third to include short-term seismic and wind loads. These short piers do not provide a reduction to liquefaction induced settlement. Anticipated initial total and differential initial settlement for drilled piers designed as above is anticipated to be ½ inches.

38. Based on our understanding of the project, and limited amount of possible liquefaction settlement estimated at the site, we assume the buildings will be supported on conventional footing foundations. If piers are used to reduce anticipated settlement due to liquefaction for Buildings B, C and D, we estimate that they will need to be at least 25 feet deep to penetrate the anticipated liquefiable near-surface layers. Initial total and differential settlement for drilled piers at least 25 feet deep is anticipated to be ½ inches. Anticipated additional settlement due to liquefaction is estimated at under 1 inch total and under ½ inch differential settlement over 50 horizontal feet. We anticipate that the drilled piers will need to be at least 2.5 feet in diameter. If buildings are supported on piers to reduce potential liquefaction settlement, we recommend that additional fieldwork using cone penetrometer testing be performed at the proposed building footprints to maximize the efficiency of the design. Piers less than 25 feet deep will not provide any significant reduction to liquefaction settlement.

39. For uplift resistance, an average skin friction value of 200 psf plus the weight of the piers may be used for piers at least 5 feet deep, and an average skin friction value of 300 psf plus the weight of the piers may be used for piers at least 10 feet deep. The skin friction value may be increased by one-third to include short-term seismic and wind loads.

40. A passive resistance equivalent to a fluid weighing 300 pcf (acting on 1.5 pier diameters) may be used. This value may be increased by one-third to include short-term seismic and wind loads.

41. Piers located adjacent to other piers, footings or utility trenches should have their bearing surfaces founded below a 1½:1 (horizontal: vertical) line projected upward from the bottom edge of the adjacent piers, footings or utility trenches.

42. All piers should be reinforced in accordance with the structural engineer's design.

43. It is possible some piers will require casing to control possible cave-ins during drilling. This is especially the case where the piers penetrate sandy soils, wet soils, or saturated soils below the groundwater elevation. Casing should be available on-site in case it is needed during drilling.

44. The pier excavations should be thoroughly cleaned of all slough or loose material prior to pouring concrete.

45. The pier drilling must be **observed by the geotechnical engineer <u>prior to placement of forms and rebar</u>** to verify subsurface soil conditions are consistent with the anticipated soil conditions so that the city required foundation excavation conformance letter can be prepared.

Retaining Wall Lateral Pressures

46. Foundations for retaining walls should follow the foundation criteria in the previous sections of this report.

47. To account for seismic loading, a horizontal line load surcharge equal to $16H^2$ pounds per linear foot of wall may be assumed to act at 0.6H above the base of the wall (where H is the height of the wall in feet).

48. Retaining walls should be designed to resist both lateral earth pressures and any additional surcharge loads. For design of retaining walls up to about 21 feet high and fully drained, the following design criteria may be used:

    a. Active earth pressure on the back of retaining walls that are **free-to-yield** may be considered to be equivalent to a fluid weight of 55 pcf for level backfills, acting in a triangular distribution.

    b. At-rest earth pressure may be considered to be equivalent to a fluid weight of 75 pcf, for level backfills, acting in a triangular distribution.

49. In addition, the walls should be designed for any adjacent live or dead loads that exert a force on the wall. For surcharges from adjacent live or dead loads within 1½ wall heights of the back of the wall, use 45% of the surcharge load as a uniform horizontal pressure. For light vehicle loads (cars and pickup trucks) within 1½ wall heights of the back of the wall, use 85 pounds per square foot as a uniform horizontal pressure. Seismic surcharge loads, and live surcharge loads (such as traffic) should be evaluated separately (not be combined). For other loading conditions HKA should be consulted for additional recommendations.

50. The above lateral pressure values assume that the walls are fully drained to prevent hydrostatic pressure behind the walls. Drainage materials behind the wall should consist of Class 1, Type A permeable material complying with Section 68 of Caltrans Standard Specifications, latest edition.

51. The drainage material should be at least 12 inches thick and extend from the base of the wall to within 12 inches of the top of the backfill.

52. Wall backdrains should be capped at the surface with clayey material to prevent infiltration of surface runoff into the backdrains. A layer of filter fabric (Mirafi 140N or equivalent) should separate the subdrain material from the overlying soil cap.

53. Retaining walls that act as interior building walls should be **thoroughly waterproofed** their full height, especially at the cold joint at the base of the wall.

54. The base of the gravel column should be made impermeable. The heel of the foundation should be water proofed to allow water to build up and enter drainpipe. A perforated rigid drain pipe should be placed (holes down) about 1 inch above the heel of the wall foundation and be tied to a suitable solid rigid drain outlet. The cold joint at the heel should be plugged with a wedge of concrete or poured with rubber gasket type plug.

55. We defer moisture proofing and water proofing recommendations to interior wall and floor covering manufacturer's suggested specifications and/or a moisture/water-proofing expert.

Flatwork, Hardscape and Concrete Slabs-on-Grade

56. Building floor slabs and exterior slabs, hardscapes, statuary, pavements, etc. should not be supported on loose soil. They should be supported on engineered fill supported on properly prepared subgrade soil. To reduce the effects of shrink/swell from the on-site medium to highly expansive clayey soils, the upper 15 inches of the engineered fill supporting interior concrete slabs-on-grade, and the upper 8 inches of the engineered fill supporting exterior concrete slabs-on-grade, should consist of non-expansive fill meeting the recommendations for import fill under Item 13, herein. Where a capillary break material is used, the capillary break thickness may be counted as part of the recommended non-expansive fill thickness. Where slabs and etcetera are within 5' of the break in slope, they may be supported on a deepened footing or drilled pier foundation system. Soil subgrades should be prepared as recommended in the section entitled "Site Grading". Final slab grades should be proof rolled to provide a smooth uniform working surface.

57. Slab reinforcing should be provided in accordance with the anticipated use and loading of the slab.

58. Where floor dampness must be minimized or where floor coverings will be installed, concrete slabs-on-grade should be constructed on a capillary break layer at least 4 inches thick and covered with a membrane vapor barrier. Capillary break material should be free draining, clean gravel or rock, such as 3/4-inch gravel. The gravel should be washed to remove fines and dust prior to placement on the slab subgrade. The vapor barrier should be a high quality membrane, such as Moistop by Fortifiber Corporation. The capillary break thickness may be counted as part of the recommended non-expansive fill thickness. We defer moisture proofing recommendations to floor covering manufacturer's suggested specifications and/or a moisture proofing expert.

59. Exterior slab reinforcement should **not** be tied to the building foundations.

60. Slabs and flatwork can be expected to suffer some cracking and movement. However, thickened exterior edges, a well-prepared subgrade including pre-moistening prior to pouring concrete, adequately spaced expansion and crack control joints at least at 8 foot intervals and good workmanship should minimize cracking and movement.

Site Drainage

61. Proper drainage is key to this project. Control of runoff from upslope of the site, control of infiltration and ponding adjacent to the foundation, and control of roof, surface and subsurface seepage is important to proper performance of the structure foundations, concrete slabs-on-grade, and pavements. Discharged collected water in a way so as not to cause erosion.

62. Surface water should not be allowed to flow towards improvements during construction and for the lifetime of the development. Surface drainage should be directed away from the building foundations, flatwork and pavements. Surface drainage should include provisions

for positive gradients (5% for 10 feet) so that water is not permitted to pond adjacent to foundations, flatwork and pavements. Otherwise, drainage devises (e.g. area or strip drains) must be used.

63. Provide provisions for surface water control and dispersion. Surface drainage improvements may consist of v-ditches, catch basins or drain inlets in association with grading; all connected to a storm drain system consisting of solid rigid pipe and clean outs.

64. Runoff and discharge must not be allowed to spill over graded slopes. Water should be directed to drain inlets connected to a drainage system that discharges into the storm drain system.

65. Rain gutters should be placed around roof eaves and conveyed to the storm drain system.

66. Conveyance and storm drain lines should consist of rigid, solid sturdy pipe.

67. Where lined perimeter foundation drains in association with waterproofed stem walls and footings are included in the design, the drains should follow recommendations in the Retaining Wall Section of this report.

68. The migration of irrigation water or spread of extensive root systems below foundations, slabs, or pavements may cause undesirable differential movements and subsequent damage to these structures. Landscaping should be planned accordingly and avoided adjacent to structures.

69. Never connect subdrains and storm drain lines. Never surcharge one into the other. Both systems should drain independently through discharge.

Utility Trenches

70. Trenches must be properly shored and braced during construction or laid back at an appropriate angle to prevent sloughing and caving at sidewalls. The project plans and specifications should direct the attention of the contractor to all CAL OSHA and local safety requirements and codes dealing with excavations and trenches.

71. Utility trenches that are parallel to the sides of buildings should be placed so that they do not extend below an imaginary line sloping down and away at a 1½:1 (horizontal to vertical) slope from the bottom outside edge of all footings. The structural design professional should coordinate this requirement with the utility layout plans for the project.

72. Trenches should be backfilled with granular-type material and uniformly compacted by mechanical means to the relative compaction as required by city specifications, but not less than 95 percent under paved areas and 90 percent elsewhere. The relative compaction will be based on the maximum dry density obtained from a laboratory compaction curve run in accordance with ASTM Procedure D1557.

73. We strongly recommend placing a concrete plug in the trench where it passes under foundation lines. Care should be taken not to damage utility lines.

74. Trenches should be capped with about 1½ feet of relatively impermeable soil.

Erosion Control

75. Do not discharge water directly on to slopes. Collected water should be discharged into the storm drain system.

76. All bare soil and cut and fill slopes should be seeded and mulched immediately after grading with barley, rye, grass and crimson clover or otherwise provided with erosion control measures.

77. Erosion control measures must be maintained during construction. Refer to construction timeframe constraints and local requirements.

Flexible Pavements

78. Anticipated traffic patterns and loading conditions for new on-site asphalt concrete pavements were not available at the time this report was prepared. Therefore, we have selected three different traffic indices 4.0, 5.0 and 6.0 for consideration and possible selection by the design Civil Engineer. The project Civil Engineer should be afforded the opportunity of specifying the most appropriate traffic index for the proposed traffic and usage. If a different traffic index from those used herein is desired or required, please contact our office and a suitable recommended asphalt structural section design can be provided.

79. Flexible asphalt concrete pavement sections have been designed based on the Caltrans design method, which includes a gravel equivalent safety factor of 0.2 feet applied to the asphalt thickness. A design R-Value of 5 was used in the pavement design. The pavement section designs are shown in the table below:

Flexible Pavement Section Design					
Subgrade R-Value	Traffic Index	Traffic	Pavement Section, Inches		
–	–	–	Asphalt Concrete	Aggregate Base/Subbase	
–	–	–	–	Class 2 AB	Class 2 ASB
5	4.0	Auto Parking	2.5	7.5	-
			2.5	3.5	4.5
5	5.0	Light Duty	2.5	11.0	-
			2.5	5.0	6.5
5	6.0	Medium Duty	3.0	13.5	-
			3.0	6.5	8.0

80. Asphalt concrete should meet the requirements for 1/2- or 3/4-inch maximum, medium Type A or Type B asphalt concrete. Asphalt concrete should comply with the specifications presented in Section 39 of the Caltrans Standard Specifications, latest edition.

81. Class 2 aggregate base materials should conform with Section 26 of the Caltrans Standard Specifications, latest edition. Class 2 aggregate subbase materials should conform with Section 25 of the Caltrans Standard Specifications, latest edition. Aggregate subbase materials should have a minimum R-value of 50.

82. Provide adequate drainage for both the pavement surface and subgrade. Landscaped and irrigated planters that are adjacent to the pavement should have cut-off curbing constructed around them that extends a minimum of 6 inches into subgrade soil (subgrade in this case is that beneath the pavement structural section).

83. Where heavy trucks, such as garbage trucks, busses will travel or make sharp turns, rigid concrete pavements should be considered.

84. To have asphaltic concrete, aggregate base and sub-base sections perform to their greatest efficiency, it is important that the following items be considered:

    A. Grading should not be performed during inclement weather.
    B. Remove unsuitable material, sub-excavate to specified grade, scarify exposed subgrade, moisture condition the subgrade and compact to a relative compaction of 92 percent and at least 2 percent over the optimum moisture content and tested by HKA. (Refer to Site Grading Section of this report).
    C. Any fill material should be placed in thin lifts as engineered fill and compacted to 92% and at least 2 percent over the optimum moisture content and tested by HKA for on-site materials, and at least 95% and at over the optimum moisture content and tested by HKA for import materials.
    D. Provide sufficient gradient to prevent ponding of water.
    E. Base rock section should meet Caltrans Standard Specifications for Class II Aggregate Base, and be angular in shape.
    F. Compact all baserock and subbase rock sections to a relative dry density of 95 percent at over the optimum moisture content. Contact HKA 4 days prior to earthwork so that the compaction curves samples of subgrade and baserock materials may be secured and tested in laboratory so that results are ready when field testing of compaction starts.
    G. Place the asphaltic concrete in two lifts or as per Caltrans section 39-6.01. Place during periods of fair weather when the free air temperature is within prescribed limits per Caltrans specifications.
    H. Provide a routine maintenance program, such as crack sealing, etc.

Plan Review, Construction Observation and Testing

85. Haro, Kasunich and Associates should be provided an opportunity to review project plans prior to construction to evaluate if our recommendations have been properly interpreted and implemented. We should also provide foundation excavation observations and earthwork observations and testing during construction. This allows us to confirm anticipated soil conditions and evaluate conformance with our recommendations and project plans. If we do not review the plans and provide observation and testing services during the earthwork phase of the project, we assume no responsibility for misinterpretation of our recommendations.

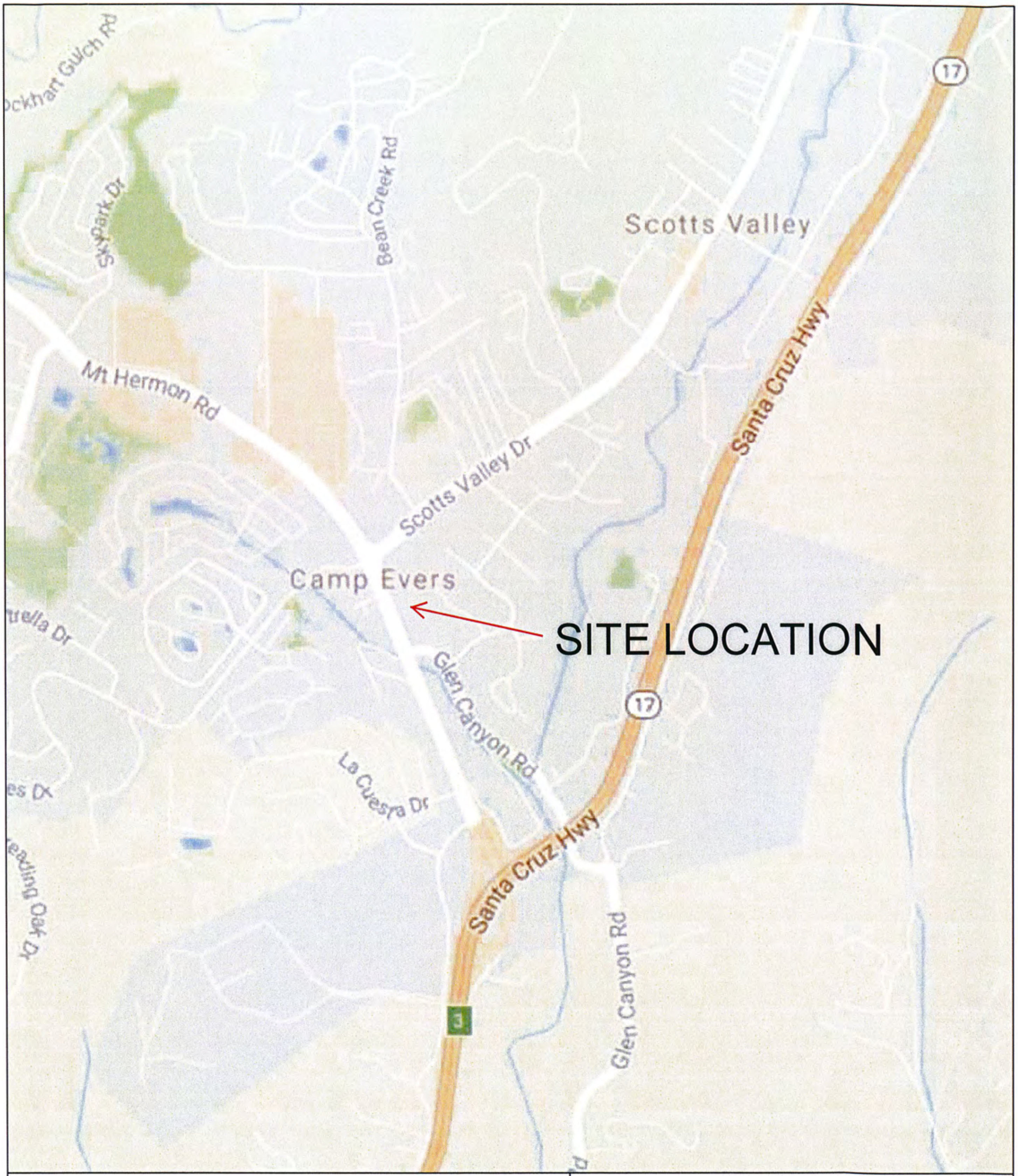
LIMITATIONS AND UNIFORMITY OF CONDITIONS

1. The recommendations of this report are based upon the assumption that the soil conditions do not deviate from those disclosed in the borings. If any variations or undesirable conditions are encountered during construction, or if the proposed construction will differ from that planned at the time, our firm should be notified so that supplemental recommendations can be given.

2. This report is issued with the understanding that it is the responsibility of the owner, or his representative, to ensure that the information and recommendations contained herein are called to the attention of the Architects and Engineers for the project and incorporated into the plans, and that the necessary steps are taken to ensure that the Contractors and Subcontractors carry out such recommendations in the field. The conclusions and recommendations contained herein are professional opinions derived in accordance with current standards of professional practice. No other warranty expressed or implied is made.

3. The findings of this report are valid as of the present date. 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 or adjacent properties. In addition, changes in applicable or appropriate standards occur whether they result from legislation or the broadening of knowledge. Accordingly, the findings of this report may be invalidated, wholly or partially, by changes outside our control. Therefore, this report should not be relied upon after a period of three years without being reviewed by a geotechnical engineer.

APPENDIX A



FROM:

GOOGLE MAPS

SITE VICINITY MAP
OAK CREEK PARK
SCOTTS VALLEY, CALIFORNIA
GRANUM HOLDINGS

SCALE: NTS
DRAWN BY: RH
DATE: JAN 2018
REVISED:
JOB NO. SC11427

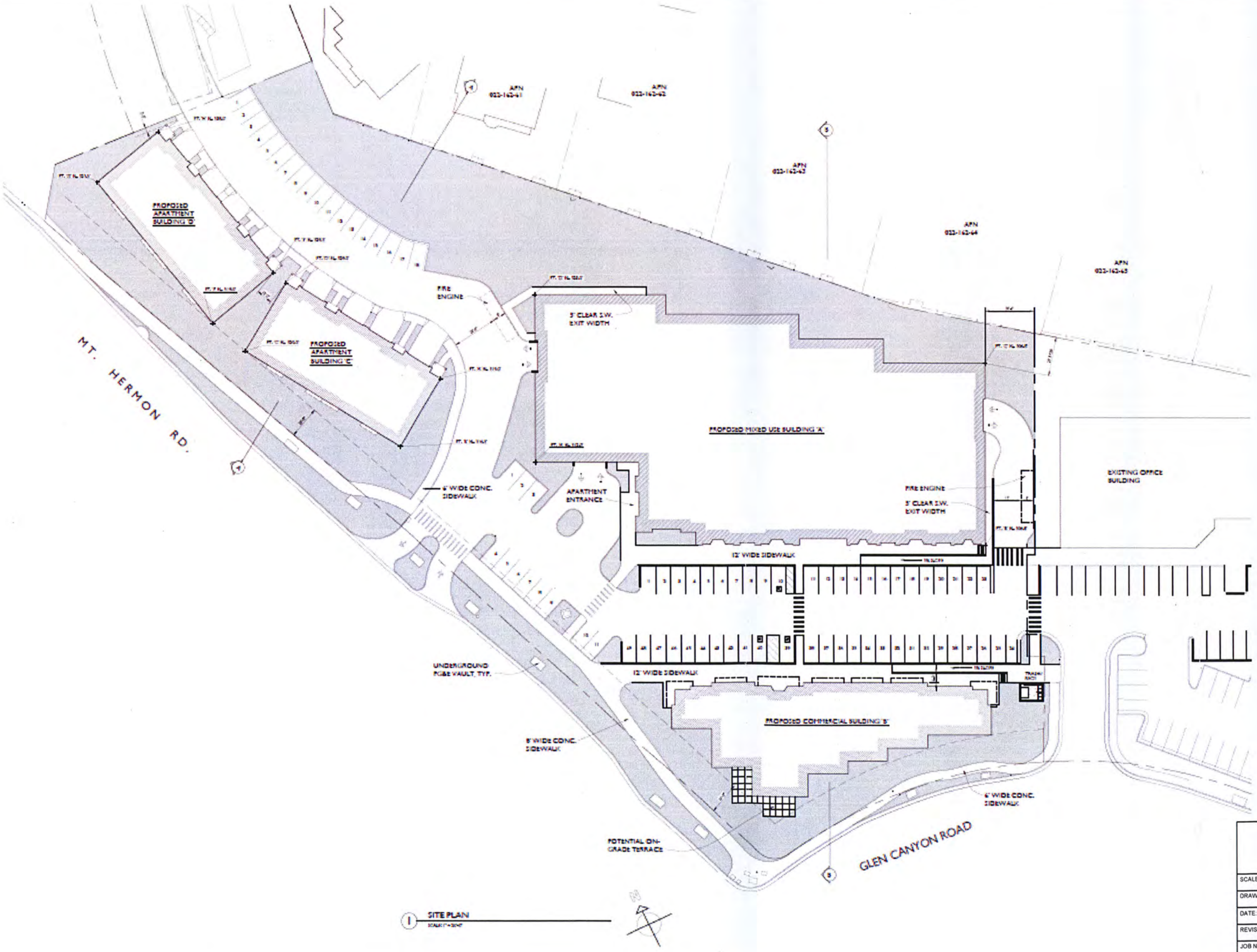
APN 022-162-76

HARO, KASUNICH & ASSOCIATES, INC.
GEOTECHNICAL AND COASTAL ENGINEERS
116 E. LAKE AVENUE, WATSONVILLE, CA 95076
(831) 722-4175

FIGURE NO. 1

SHEET NO.

Base Map by:
THACHER &
THOMPSON
ARCHITECTS
October 2016



SITE PLAN
OAK CREEK PARK
SCOTTS VALLEY, CALIFORNIA
GRANUM HOLDINGS

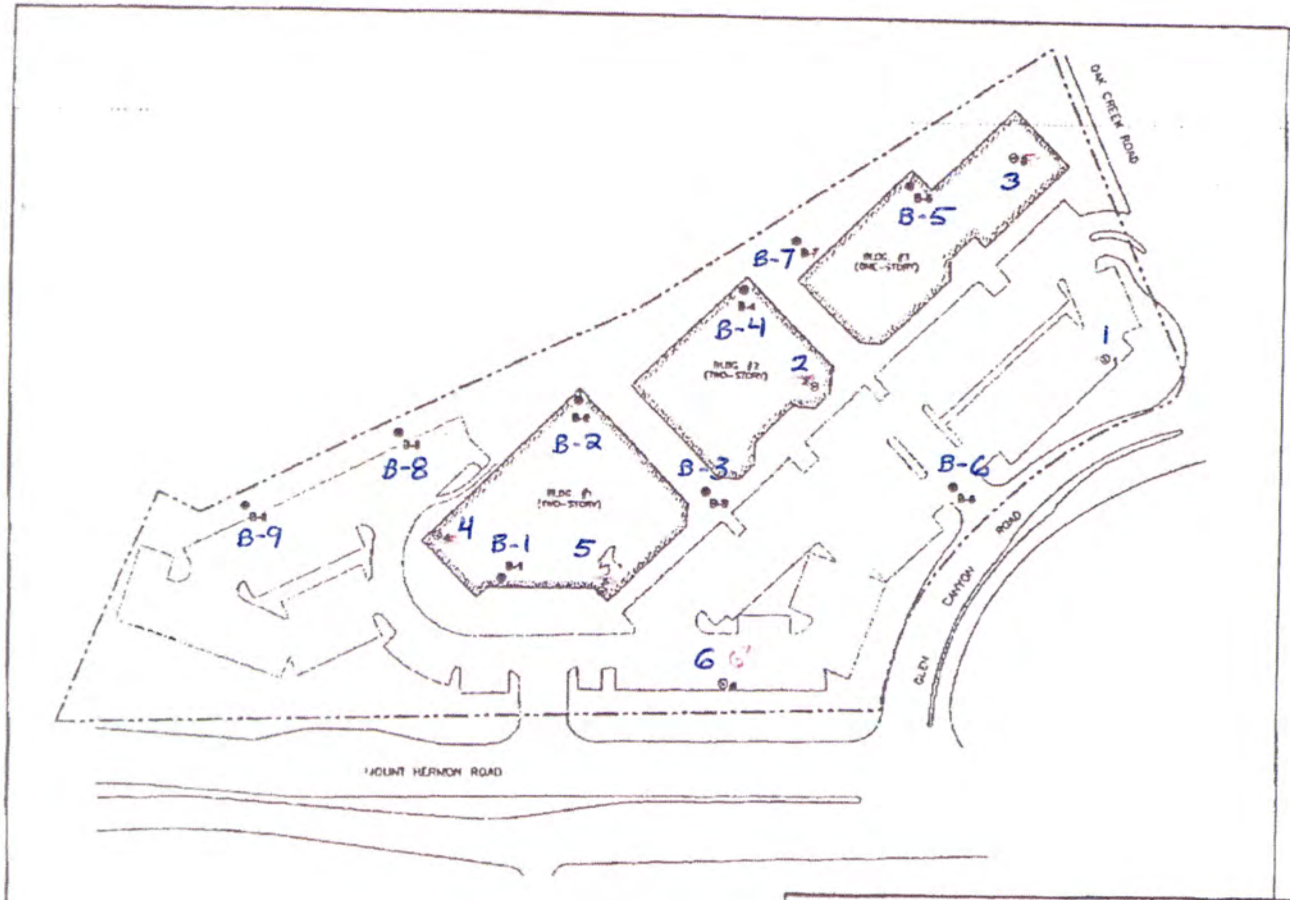
SCALE: NTS
DRAWN BY: RH
DATE: JAN 2018
REVISED:
JOB NO. SC11427

APN 022-162-76

HARO, KASUNICH & ASSOCIATES, INC.
GEOTECHNICAL AND COASTAL ENGINEERS
116 E. LAKE AVENUE, WATSONVILLE, CA 95076
(831) 722-4175

FIGURE NO. 2

SHEET NO.



LEGEND

PROPERTY BOUNDARY

● SOIL BORING
(drilled by Kleinfelder, 1997)

⊘ SOIL BORING
(drilled by others, 1988)

NOTE: Locations are approximate

80 0 80
APPROXIMATE SCALE (feet)

KLEINFELDER	**SITE PLAN** OAK CREEK PARK OFFICE CENTER SCOTTS VALLEY, CALIFORNIA	PLATE 2
DRAFTED BY: L. Sue DATE: 3-26-97		
CHECKED BY: S. Leek DATE: 3-27-97	PROJECT NO. 12-30251C-001	

CAD FILE: C:_KA-PROJ\SJ\12302510\SITEPLAN.dwg

MAJOR DIVISIONS				TYPICAL NAMES
COARSE GRAINED SOILS More than Half > #200 sieve	GRAVELS MORE THAN HALF COARSE FRACTION IS LARGER THAN NO. 4 SIEVE	CLEAN GRAVELS WITH LITTLE OR NO FINES	GW	WELL GRADED GRAVELS, GRAVEL-SAND
			GP	POORLY GRADED GRAVELS, GRAVEL-SAND MIXTURES
		GRAVELS WITH OVER 12% FINES	GM	SILTY GRAVELS, POORLY GRADED GRAVEL-SAND-SILT MIXTURES
			GC	CLAYEY GRAVELS, POORLY GRADED GRAVEL-SAND-CLAY MIXTURES
	SANDS MORE THAN HALF COARSE FRACTION IS SMALLER THAN NO. 4 SIEVE	CLEAN SANDS WITH LITTLE OR NO FINES	SW	WELL GRADED SANDS, GRAVELLY SANDS
			SP	POORLY GRADED SANDS, GRAVELLY SANDS
		SANDS WITH OVER 12% FINES	SM	SILTY SANDS, POOORLY GRADED SAND-SILT MIXTURES
			SC	CLAYEY SANDS, POORLY GRADED SAND-CLAY MIXTURES
FINE GRAINED SOILS More than Half < #200 sieve	SILTS AND CLAYS LIQUID LIMIT LESS THAN 50		ML	INORGANIC SILTS AND VERY FINE SANDS, ROCK FLOUR, SILTY OR CLAYEY FINE SANDS, OR CLAYEY SILTS WITH SLIGHT PLASTICITY
			CL	INORGANIC CLAYS OF LOW TO MEDIUM PLASTICITY, GRAVELLY CLAYS, SANDY CLAYS, SILTY CLAYS, LEAN CLAYS
			OL	ORGANIC CLAYS AND ORGANIC SILTY CLAYS OF LOW PLASTICITY
	SILTS AND CLAYS LIQUID LIMIT GREATER THAN 50		MH	INORGANIC SILTS, MICACEOUS OR DIATOMACIOUS 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

UNIFIED SOIL CLASSIFICATION SYSTEM

PS	Percent Saturation
SG	Specific Gravity
Consol	Consolidation
LL	Liquid Limit (in %)
PL	Plastic Limit (in %)
PI	Plasticity Index
TS	Total Saturation Moisture Content
SA	Sieve Analysis
■	Undisturbed Sample
☒	Bulk Sample
◩	Standard Penetration Test
◨	Sample Attempt with No Recovery

Shear Strength, psf
Confining Pressure, psf

Tx	2630 (240)	Unconsolidated Undrained Triaxial
Tx sat	2100 (575)	Unconsolidated Undrained Triaxial, saturated prior to test
DS	3740 (960)	Consolidated Drained Direct Shear
FVS	1320	Field Vane Shear
UC	4200	Unconfined Compression
LVS	500	Laboratory Vane Shear
C	Concrete Compressive Strength	
PE	Petrographic Examination	
Perm	Permeability	
PP	Pocket Penetrometer (tsf) Unc. Compressive Strength	

KEY TO TEST DATA

KLEINFELDER
Geotechnical, Materials and Environmental Engineering

PROJECT NUMBER 12-3025-10 DATE APR 1997

SOIL CLASSIFICATION CHART
AND KEY TO TEST DATA
OAK CREEK PARK
OFFICE CENTER
SCOTTS VALLEY, CALIFORNIA

PLATE
A-1

EQUIPMENT: 8"-Dia. Hollow Auger

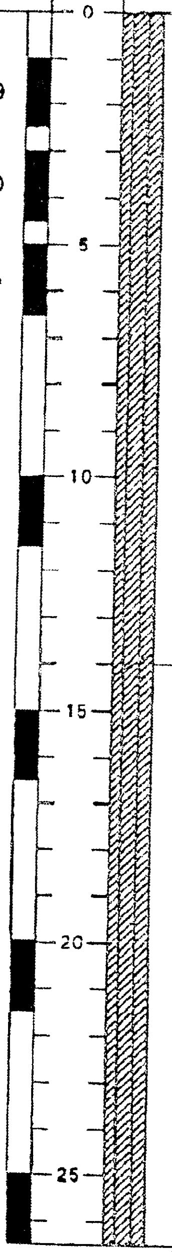
LOGGED BY: M.Brisano

ELEVATION: **

START DATE: 2-28-97

FINISH DATE: 2-28-97

Other Laboratory Tests	Dry Density (pcf)	Moisture Content (%)	Blows/ Foot *
PP = 3.5			19
PP = 4.5 +			40
PP = 4.5 +			55
PP = 4.5 +			59
PP = 4.5 +			48
PP = 4.5 +			44
PP = 4.5 +			41



MOTTLED YELLOW TO GRAY BROWN SILTY CLAY (CL) very stiff to hard, dry to moist, with traces of sand, medium plasticity

hard

GRAY SILTY CLAY (CL) hard, dry to moist, medium plasticity

Bottom of Boring B-1 @ 26.5 feet.
No Free Groundwater Observed.

* Converted to equivalent standard penetration blow counts.

** Existing ground surface at time of drilling

KLEINFELDER
Geotechnical, Materials and Environmental Engineering

PROJECT NUMBER **12-3025-10** DATE **APR 1997**

LOG OF BORING B-1

OAK CREEK PARK
OFFICE CENTER
SCOTTS VALLEY, CALIFORNIA

PLATE
A-2

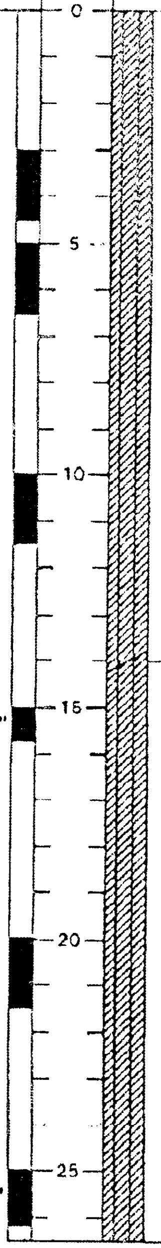
EQUIPMENT: 8"-Dia. Hollow Auger

LOGGED BY: M.Briseno

ELEVATION: **

START DATE: 2-28-97

FINISH DATE: 2-28-97



Other Laboratory Tests	Dry Density (pcf)	Moisture Content (%)	Blows/ Foot *	DEPTH (FEET)	
				0	**MOTTLED YELLOW TO GRAY BROWN SILTY CLAY (CL)** very stiff, dry to moist, medium plasticity
PP = 4.5 +			33		
PI = 25, see Plate B-1 minus #200 = 75% PP = 4.5 +			51	5	hard
PP = 4.5 +			50	10	mottled gray and yellow brown
					GRAY SILTY CLAY (CL) hard, dry to moist, medium plasticity
PP = 4.5 +			70/8"	15	
PP = 4.5 +			72	20	
PP = 4.5 +			76/8"	25	

Bottom of Boring B-2 @ 26.5 feet.

* Converted to equivalent standard penetration blow counts.

** Existing ground surface at time of drilling.

KLEINFELDER
Geotechnical, Materials and Environmental Engineering

PROJECT NUMBER 12-3025-10 DATE APR 1997

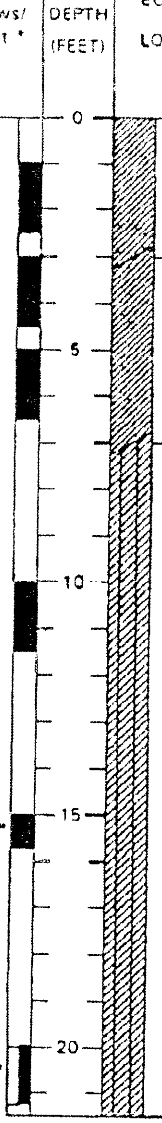
LOG OF BORING B-2

OAK CREEK PARK
OFFICE CENTER
SCOTTS VALLEY, CALIFORNIA

PLATE
A-3

EQUIPMENT: 8"-Dia. Hollow Auger
ELEVATION: **
LOGGED BY: M.Brisano
START DATE: 2-28-97
FINISH DATE: 2-29-97

Other Laboratory Tests	Dry Density (pcf)	Moisture Content (%)	Blows/ Foot *
PP = 1.5	87	26.0	5
PP = 1.0	94	25.0	6
PP = 2.5	94	22.0	7
PP = 4.5 +			68
PP = 4.5 +			70/8"
PP + 4.5 +			73/9"



YELLOW TO DARK BROWN SANDY CLAY (CL) medium stiff, moist to wet, trace of roots, medium plasticity (FILL)

DARK BROWN SILTY CLAY (CL) medium stiff, moist, medium plasticity (FILL)

MOTTLED YELLOW BROWN SILTY CLAY (CL) hard, dry to moist, medium plasticity

Bottom of Boring B-3 @ 21.5 feet.
No Free Groundwater Observed.

* Converted to equivalent standard penetration blow counts.
** Existing ground surface at time of drilling.

KLEINFELDER
Geotechnical, Materials and Environmental Engineering

PROJECT NUMBER 12-3025-10 DATE APR 1997

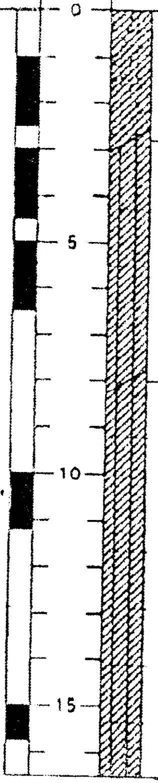
LOG OF BORING B-3

OAK CREEK PARK
OFFICE CENTER
SCOTTS VALLEY, CALIFORNIA

PLATE
A-4

EQUIPMENT: 8" Dia. Hollow Auger
LOGGED BY: M Brisano
ELEVATION: **
START DATE: 2-28-97
FINISH DATE: 2-28-97

Other Laboratory Tests	Dry Density (pcf)	Moisture Content (%)	Blows/ Foot *
PP = 1.75			5
PP = 2.0	102	20.0	8
PP = 4.5	98	22.0	16
PP = 4.5 +			44/8"
PP = 4.5 +			44/8"



BROWN SILTY CLAY (CL) medium stiff, moist, with trace of roots, medium plasticity

MOTTLED YELLOW BROWN SILTY CLAY (CL) stiff to hard, moist with trace of roots, medium plasticity

MOTTLED YELLOW BROWN SILTY CLAY (CL) hard, moist, medium plasticity

Auger refusal @ 16.5 feet.

Bottom of Boring B-4 @ 16.5 feet.
No Free Groundwater Observed.

* Converted to equivalent standard penetration blow counts.
** Existing ground surface at time of drilling.

KLEINFELDER
Geotechnical, Materials and Environmental Engineering

PROJECT NUMBER 12-3025-10 DATE APR 1997

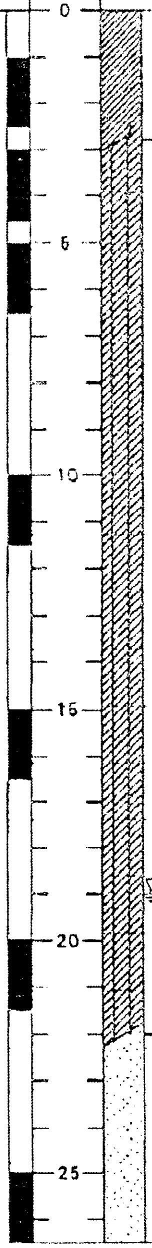
LOG OF BORING B-4

OAK CREEK PARK
OFFICE CENTER
SCOTTS VALLEY, CALIFORNIA

PLATE
A-5

EQUIPMENT: 8"-Dia. Hollow Auger
LOGGED BY: M.Brisano
ELEVATION: **
START DATE: 2-28-97
FINISH DATE: 2-28-97

Other Laboratory Tests	Dry Density (pcf)	Moisture Content (%)	Blows/ Foot *
PP = 1.5			11
PP = 1.0			2
PP = 2.0 PP = 4.0			9
PP = 3.5			10
PP = 3.5			16
PP = 2.75			14
			14



BROWN SANDY CLAY (CL) stiff, moist, medium plasticity

MOTTLED YELLOW BROWN SILTY CLAY (CL) stiff to very stiff, moist to wet, with traces of roots and sand, medium plasticity

visible moisture on sample

grading olive gray and sandier

GRAY CLEAN SAND (SP) medium dense, wet, medium to coarse-grained

Bottom of Boring B-5 @ 26.5 feet.

* Converted to equivalent standard penetration blow counts.
** Existing ground surface at time of drilling.

KLEINFELDER
Geotechnical, Materials and Environmental Engineering

PROJECT NUMBER 12-3025-10 DATE APR 1997

LOG OF BORING B-5

OAK CREEK PARK
OFFICE CENTER
SCOTTS VALLEY, CALIFORNIA

PLATE
A-6

EQUIPMENT: 8"-Dia. Hollow Auger

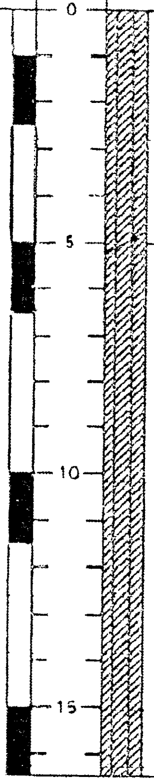
LOGGED BY: M.Brisano

ELEVATION: **

START DATE: 2-28-97

FINISH DATE: 2-28-97

Other Laboratory Tests	Dry Density (pcf)	Moisture Content (%)	Blows/ Foot *
PP = 1.5			2
PP = 3.0			11
PP = 4.5			7
PP = 4.0			22



MOTTLED YELLOW BROWN SILTY CLAY (CL) stiff to very stiff, moist, with traces of roots, medium plasticity

BROWN SILTY CLAY (CL) stiff, moist, trace of sand

Bottom of Boring B-7 @ 16.5 feet.
No Free Groundwater Observed.

* Converted to equivalent standard penetration blow counts.

** Existing ground surface at time of drilling.

KLEINFELDER
Geotechnical, Materials and Environmental Engineering

PROJECT NUMBER 12-3025-10 DATE APR 1997

LOG OF BORING B-7

OAK CREEK PARK
OFFICE CENTER
SCOTTS VALLEY, CALIFORNIA

PLATE
A-7

EQUIPMENT: 8"-Dia Hollow Auger

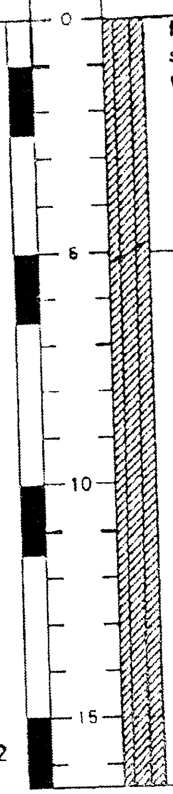
LOGGED BY M Briseno

ELEVATION **

START DATE: 2-28-97

FINISH DATE: 2-28-97

Other Laboratory Tests	Dry Density (pcf)	Moisture Content (%)	Blows/ Foot *
PP = 1.5			2
PP = 3.0			11
PP = 4.5			7
PP = 4.0			22



MOTTLED YELLOW BROWN SILTY CLAY (CL) stiff to very stiff, moist, with traces of roots, medium plasticity

BROWN SILTY CLAY (CL) stiff, moist, trace of sand

Bottom of Boring B-7 @ 16.5 feet.
No Free Groundwater Observed.

- * Converted to equivalent standard penetration blow counts.
- ** Existing ground surface at time of drilling

KLEINFELDER
Geotechnical, Materials and Environmental Engineering

PROJECT NUMBER 12-3025-10 DATE APR 1997

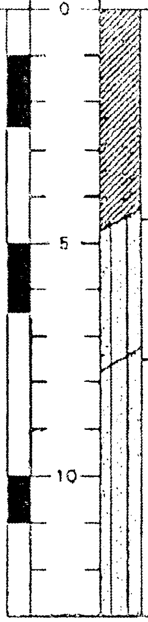
LOG OF BORING B-7

OAK CREEK PARK
OFFICE CENTER
SCOTTS VALLEY, CALIFORNIA

PLATE
A-8

EQUIPMENT: 8"-Dia. Hollow Auger
LOGGED BY: M Briseno
ELEVATION: **
START DATE: 2-28-97
FINISH DATE: 2-28-97

Other Laboratory Tests	Dry Density (pcf)	Moisture Content (%)	Blows/ Foot *	DEPTH (FEET)
				0
			7	
				5
			11	
				10
PP = 4.5 +			53	



MOTTLED YELLOW BROWN SANDY CLAY (CL) medium stiff, moist, low plasticity

YELLOW BROWN SILTY SAND (SM) medium dense, moist, fine-grained

GRAY SILTY CLAY (CL) hard, moist, medium plasticity

Auger refusal @ 13 feet

Bottom of Boring B-8 @ 13 feet.
No Free Groundwater Observed.

* Converted to equivalent standard penetration blow counts.

** Existing ground surface at time of drilling.

KLEINFELDER
Geotechnical, Materials and Environmental Engineering

PROJECT NUMBER 12-3025-10 DATE APR 1997

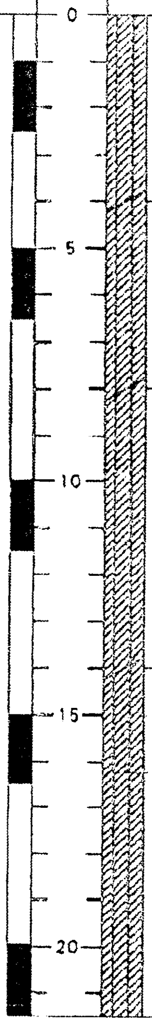
LOG OF BORING B-8

OAK CREEK PARK
OFFICE CENTER
SCOTTS VALLEY, CALIFORNIA

PLATE
A-9

EQUIPMENT: 8"-Dia. Hollow Auger
LOGGED BY: M.Briseno
ELEVATION: **
START DATE: 2-28-97
FINISH DATE: 2-28-97

Other Laboratory Tests	Dry Density (pcf)	Moisture Content (%)	Blows/ Foot *
PP = 3.25			7
PP = 4.5 +			10
PP = 4.5 +			53
PP = 4.5 +			26
PP = 4.5 +			40



BROWN SILTY CLAY (CL) stiff to very stiff, moist with traces of roots and sand, medium plasticity

MOTTLED YELLOW BROWN SILTY CLAY (CL) hard, moist, medium plasticity

GRAY BROWN SILTY CLAY (CL) very stiff to hard, moist

GRAY SILTY CLAY (CL) hard, moist, medium plasticity

Bottom of Boring B-9 @ 21.5 feet.
No Free Groundwater Observed.

* Converted to equivalent standard penetration blow counts.
** Existing ground surface at time of drilling.

KLEINFELDER
Geotechnical, Materials and Environmental Engineering

PROJECT NUMBER 12-3026-10 DATE APR 1997

LOG OF BORING B-9

OAK CREEK PARK
OFFICE CENTER
SCOTTS VALLEY, CALIFORNIA

PLATE
A-10

July 26, 1988

LOGGED BY SAS DATE DRILLED 6/21/88 BORING DIAMETER 6" BORING NO. 1

Depth, ft.	Sample No. and type	Symbol	SOIL DESCRIPTION	Unified Soil Classification	Blows/foot 350 ft-lbs.	Qu - t.s.f. Penetrometer	Dry Density p.c.f.	Moisture % dry wt.	MISC. LAB RESULTS
1			Brown Silty fine SAND with subrounded gravels and occasional cobbles, damp, loose						
2	1-1		Dark gray brown very fine SAND with organic debris, strong organic odor, moist, stiff		7	4.5+	99.8	27.3	
3	L								
4									
5									
6									
7	1-2		NATIVE						
8	L		Light gray Silty CLAY with red brown mottling, moist, stiff		12	2.9	108.5	18.0	Qu=4900 psf
9									
10									
11			Light green brown Silty CLAY, moist, stiff						
12									
13									
14			Tan Silty medium SAND with subrounded granitic gravels, wet, medium dense						
15	1-3		▽ Perched						
16	L		Dark gray SILT (weakly indurated), very damp, very stiff		31	4.5+	109.5	17.7	
17									
18									
19									
20									
21			Increase in hardness - hard						
22			Break-in-log						
25									
			Boring Terminated @ 25'						

Jacobs, Raas & Associates FIGURE NO. 2 Log of Test Borings

LITHO WATSONVILLE PRESS

July 28, 1988

LOGGED BY SAS DATE DRILLED 6-21-88 BORING DIAMETER 6" BORING NO. 2

Depth, ft.	Sample No. and type	Symbol	SOIL DESCRIPTION	Unified Soil Classification	Blows/foot 350 ft-lbs.	Qu - t.s.f. Penetrometer	Dry Density p.c.f.	Moisture % dry wt.	MISC. LAB RESULTS
			FILL						
1			Brown Silty very fine SAND, dry, loose						
			wood debris						
2			NATIVE-Dark brown Silty fine SAND, damp loose		5	4.5	97.7	21.7	Qu=1090 psf
3	2-1		Dark gray Clayey SILT, wet, firm						
4	L								
5	2-2		Brown very fine Sandy Clayey SILT, very moist, stiff (no sample recovered)		13				
6	L		Light green brown Clayey SILT with light gray mottling, moist, very stiff						
7	2-3								
8	L				20	4.4	20.8	103.3	Ø=30° C=1650 psf
9									
10									
11									
12									
13									
14									
15									
16									
17			∇ Perched						
18									
19			Green brown very fine Sandy SILT, wet, firm to stiff						
20									
21									
22			Dark gray SILT (weakly indurated), very damp, hard						
			Break-in-log						
25			Boring Terminated @ 25'						

Jacobs, Raas & Associates FIGURE NO. 3 Log of Test Borings

LOGGED BY SAS DATE DRILLED 7/5/88 BORING DIAMETER 6" BORING NO. 3

Depth, ft.	Sample No. and type	Symbol	SOIL DESCRIPTION	Unified Soil Classification	Blows/foot 350 ft-lbs.	Qu - t. s. f. Penetrometer	Dry Density p.c.f.	Moisture % dry wt.	MISC. LAB RESULTS
			FILL						
1			Brown Silty fine SAND with subrounded gravel and cobbles, dry, loose						
2	3-1		NATIVE-Gray brown Silty fine SAND damp, medium dense						
3	L		Gray Clayey SILT with red brown mottling very damp, hard		36	4.5+	112.7	14.4	Qu= 10,000 psf
4			Brown Silty fine SAND, damp, dense						
6			Red brown Silty CLAY with gray mottling moist, hard						
7	3-2								
8	L				56	4.5+	108.4	19.0	Ø=26 C=2670 psf
18	3-3								
19	L				48	2.7	102.5	21.0	
22			Dark brown fine Sandy CLAY, very moist, very stiff						
25			▽ W/L 7-5/88 Break-in-log						
			Boring Terminated @ 25'						

Jacobs, Raas & Associates

FIGURE NO. 4 Log of Test Borings

LOGGED BY SAS DATE DRILLED 7/5/88 BORING DIAMETER 6" BORING NO. 5

Depth, ft.	Sample No. and type	Symbol	SOIL DESCRIPTION	Unified Soil Classification	Blows/foot 350 ft-lbs.	Qu - t.s.f. - Penetrometer	Dry Density p.c.f.	Moisture % dry wt.	MISC. LAB RESULTS
1			Light yellow-brown very fine Sandy SILT dry, stiff						
2	5-1								
3	L		orange-brown mottling, very stiff		30	4.5+	105.	10.9	Qu=14780 psf
4									
5	5-2		Yellow-brown very fine Sandy Clayey SILT with orange-brown mottling, very damp, hard						
6	L				60	4.5+	106.0	19.6	Qu=13390 psf
7									
8			Brown very fine Sandy SILT (weakly indurated silt stone) damp, hard						
9									
10	5-		very hard						
11	L				50/8'	2.9	94.5	13.4	
12									
13									
14									
15			Gray brown very fine Sandy SILT with fine silt stone (moderately indurated silt stone) damp, very hard						
16									
17									
18									
19			▽ W/L 7/5/88						
20									
21									
22									
			Break-in-log						
25									
			Boring Terminated @ 25'						

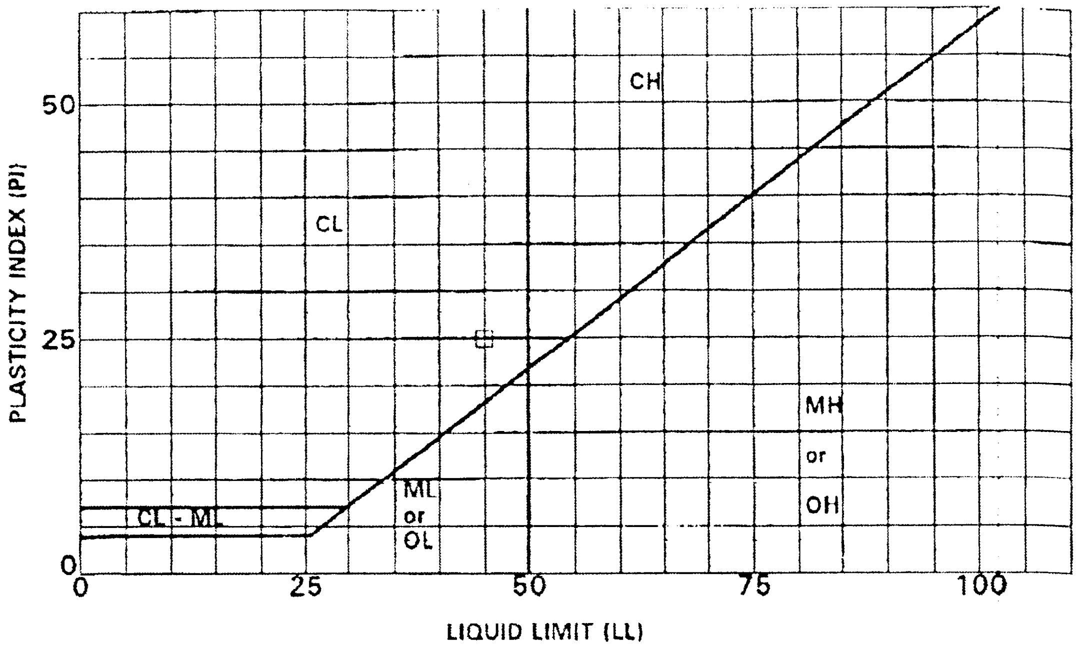
Jacobs, Raas & Associates FIGURE NO. 6 Log of Test Borings

LITHO WATSONVILLE PRESS

LOGGED BY SAS DATE DRILLED 7/5/88 BORING DIAMETER 6" BORING NO. 6

Depth, ft.	Sample No. and type	Symbol	SOIL DESCRIPTION	Unified Soil Classification	Blows/foot 350 ft-lbs.	Qu - t.s.f. Penetrometer	Dry Density p.c.f.	Moisture % dry wt.	MISC. LAB RESULTS
			FILL						
	6-1								
1			Orange Silty fine SAND with fine granitic rock fragments and occasional gravels and cobbles, dry, moist dense		36	4.5+	110.	15.	
2	L								
3	6-2		NATIVE Dark gray brown very fine Sandy SILT, damp, stiff very moist						
4	L				8	4.5+	118.	12.	Qu= 6470 ps
5									
6									
7									
8			▽ W/L 7/5/88						
9			Yellow brown very fine Sandy Clayey SILT with orange-brown mottling, very moist, stiff						
10	6-								
11	L		Very stiff, light gray brown stockwork						
12					16	2.8	98.7	25.3	
13									
14			Blue gray Silt fine SAND with clay binder, very moist, loose to medium dense						
15			Boring Terminated @ 15'						
16									
17									
18									
19									
20									
21									
22									
23									
24									

Jacobs, Raas & Associates FIGURE NO. 7 Log of Test Borings



Symbol	Boring	Depth	LL	PL	PI	Sample Description
☐	B-3	5.5	45	20	25	Gray Brown Silty Clay (CL)

Unified Soil Classification
Fine Grained Soil Groups

Symbol	LL < 50	Symbol	LL > 50
ML	Inorganic clayey silts to very fine of slight plasticity	MH	Inorganic silts and clayey silts of high plasticity
CL	Inorganic clays of low to medium plasticity	CH	Inorganic clays of high plasticity
OL	Organic silts and organic silty clays of low plasticity	OH	Organic clays of medium to high plasticity, organic silts

KLEINFELDER

PROJECT NO. 12-3025-10

PLASTICITY CHART

OAK CREEK PARK
OFFICE CENTER
SCOTTS VALLEY, CALIFORNIA

PLATE

B-1

APPENDIX B

County of Santa Cruz – Soils Engineer Transfer of Responsibility

COUNTY OF SANTA CRUZ

PLANNING DEPARTMENT
701 OCEAN STREET, 4TH FLOOR, SANTA CRUZ, CA 95060
(831) 454-2580 FAX: (831) 454-2131 TDD: (831) 454-2123
KATHLEEN MOLLOY PREVISICH, PLANNING DIRECTOR

SOILS ENGINEER TRANSFER OF RESPONSIBILTY

APN: 022 - 162 - 76 DATE: 5 January 2018
OWNER: Mr. Greg Eger Granum Holdings
PROJECT LOCATION: Mount Hermon Road at Scotts Valley Drive, Scotts Valley, California

PROJECT DESCRIPTION:
New mixed use development including 4 buildings; a mixed use building, a commercial building, and two apartment buildings. Will include new access driveways, garage parking, outdoor parking, sidewalks, landscaping, trash/recycling enclosure & underground utilities.

Our firm is taking over the above referenced project as the project soils engineer of record.

We have reviewed the original geotechnical work for this project. Completed work reviewed to date is as follows (detail all reports including author, title, date and project number):
"Geotechnical Investigation Proposed Oak Creek Park Office Center Mount Hermon and Glen Canyon Roads Scotts Valley, California," by Kleinfelder, Inc., dated 8 April 1997, Project Number 12-3025-10 (& also listed as 12-3024-00 on letter of transmittal & T.O.C.)

Based upon our review, we offer our professional opinions as follows (check where applicable):

____ We concur with all of the technical conclusions and recommendations.

_X_ We do not agree with or support geotechnical conclusions or recommendations as detailed on the attached report (attach new conclusions and recommendations and all new supporting data and reasoning). In part. We have prepared an updated report with 2016 CBC, updated liquefaction analyis & modified some of the original recommendations. <u>Please read prior to signature</u>

By signing below, we agree to accept responsibility within our area of technical competence for approval of this project upon completion of the work.

SIGNED: 1/5/18
(Apply California State-registered civil or soils engineer's signature and wet stamp here)

ROBERT G. HASSLER
GE 3074
GEOTECHNICAL
EXP. 12-31-2018

RETURN TO: ____

KLEINFELDER

318-00

COPY TO KEE WONG

12-3025-10

GEOTECHNICAL INVESTIGATION
PROPOSED OAK CREEK PARK OFFICE CENTER
MOUNT HERMON AND GLEN CANYON ROADS
SCOTTS VALLEY, CALIFORNIA

FOR: Hal Porter Homes
1210 Central Boulevard
PO Drawer 1088
Brentwood, California 94513

Attention: Mr. Brent Aasen

This document was prepared for use only by Hal Porter Homes, only for the purposes stated, and within a reasonable time from its issuance. Please read the "Limitations" section of this report. Unauthorized use or copying of this document is strictly prohibited. See "Application for Authorization to Use" located at the end of this document if use or copying is desired by anyone other than Hal Porter Homes and for the project identified above.

April 8, 1997

KLEINFELDER

April 8, 1997
File: 12-3025-10

Hal Porter Homes
1234 Central Avenue
Brentwood, California 92614

Attention: Mr. Brent Aasen

Subject: Geotechnical Investigation Report For the Proposed Oak Creek Park Office Center in Scotts Valley, California

Gentlemen:

Kleinfelder, Inc. is pleased to submit our geotechnical report for the proposed Oak Creek Park Office Center in Scotts Valley, California. The attached report provides a description of the investigation performed and our recommendations for design of foundations, retaining walls, earthwork and flexible pavements.

In summary, it is our opinion that the site may be developed as presently proposed, provided that the recommendations presented in our report are followed. The primary geotechnical considerations with respect to the proposed construction are 1) the loose fill soils which will have to be removed and replaced as engineered fill, 2) the moderate to high expansion potential of the site soils, and 3) the proposed loction of the buildings across cut-fill sections of the site. This condition will likely result in differential settlement and therefore, the cut portions of the buildings should be undercut and the soils replaced as engineered fill to provide an uniform section of fill below the building pads. The proposed building structures may be supported on shallow foundations with an allowable bearing capacity of 2,500 pounds per square foot at a minimum embedment depth of 18 inches below the lowest adjacent finished grade. Post-construction total and differential settlements for the building are anticipated to be on the order of about 3/4-inch and 1/2-inch, respectively, based on the structural loads discussed in this report.

We appreciate the opportunity of providing our services to you on this project and trust this report meets your needs at this time. If you have any questions concerning the information presented, please contact this office.

Sincerely,
KLEINFELDER, INC.

Scott M. Leck

Scott M. Leck, G.E. 2067
Senior Geotechnical Engineer
cc: Addressee (4)

Reviewed by

Scott M. Leck for

Gregory R. Ruf, C.E.
Area Manager

GEOTECHNICAL INVESTIGATION
PROPOSED OAK CREEK PARK OFFICE CENTER
MOUNT HERMON AND GLEN CANYON ROADS
SCOTTS VALLEY, CALIFORNIA

TABLE OF CONTENTS

GEOTECHNICAL INVESTIGATION
PROPOSED OAK CREEK PARK OFFICE CENTER
MOUNT HERMON AND GLEN CANYON ROADS
SCOTTS VALLEY, CALIFORNIA

TABLE OF CONTENTS (continued...)

GEOTECHNICAL INVESTIGATION REPORT
OAK CREEK OFFICE PARK CENTER
MOUNT HERMON ROAD AND GLEN CANYON ROAD
SCOTTS VALLEY, CALIFORNIA

1. INTRODUCTION

This report presents the results of a geotechnical investigation performed for the proposed Oak Creek Office Park Center to be located at the corner of Mt. Hermon Road and Glen Canyon Road in Scotts Valley, California. The location of the site is shown on the Site Location Map, Plate 1. This investigation has been performed for Hal Porter Homes, the project developer.

1.1. Project Description

The project plan includes the construction of two two-story and one single-story office buildings with footprint areas of about 12,000 to 15,000 square feet with finished floor levels at about elevation 505 feet. On this basis, grading consisting of cuts of up to about 10 feet and fills up to about 5 feet are anticipated. Based on the proposed construction of wood and/or steel framing, we anticipate that the building loads will consist of perimeter wall loads of 2 to 4 kips per lineal foot and interior column loads of 40 to 80 kips; Slab-on-grade floors are also anticipated. Additional site improvements will include paved parking and driveway areas and retaining walls up to a maximum height of about 5 feet.

1.2. Purpose and Scope of Services

The purpose of this investigation was to explore and evaluate the soil characteristics at the site with respect to the proposed development. The Scope of Services performed for this investigation, as presented in our proposal dated February 21, 1997, consisted of a site reconnaissance by a registered engineer, review of published geologic maps of the area, subsurface exploration, laboratory testing, engineering analyses of field and laboratory data, and preparation of this report.

The data obtained and analyses performed were for the purpose of providing design and construction recommendations for site earthwork, underground utilities, foundations, interior and exterior slabs-on-grade, retaining walls and flexible pavements.

2. GEOLOGY

2.1. Geologic Setting

The project site lies within the Santa Cruz Mountains which are located along the western edge of the Coast Range geomorphic province, a more or less discontinuous series of northwest trending mountain ranges, ridges, and intervening valleys characterized by intense, complex folding and faulting.

The project site is located within an area of hilly terrain approximately 1,000 feet west of Carbonera Creek which drains into the Monterey Bay approximately 5-1/2 miles south of the site. Based on a review of geologic maps of the project site, bedrock units present include the Santa Cruz Mudstone, a thick-bedded siliceous organic mudstone and the Santa Margarita Sandstone, a fine to medium grained friable sandstone, which are locally overlain by colluvium or residual soils.

2.2. Faulting and Seismicity

The Scotts Valley area is seismically dominated by the active San Andreas Fault system, the general boundary between the northward moving Pacific Plate (west of the fault) and the southward moving North American Plate (east of the fault). This movement is distributed across a complex system of generally strike-slip, right lateral, parallel and sub-parallel faults, which include among others the regional San Andreas, Seal Cove-San Gregorio, Hayward and Calaveras faults, located at distances of 7-1/2 miles northeast, 15-1/2 miles southwest, 20-1/2 miles northeast and 23-1/2 miles northeast, respectively. The site is not located within a State of California Earthquake Fault Study Zone (formerly known as Alquist-Priolo Special Study Zone) and no mapped fault traces are known to traverse the site. However, the site will be subjected to strong ground shaking. Therefore, the proposed structure should be designed accordingly.

3. SITE INVESTIGATION

3.1 Site Description

We understand that the proposed project will consist of development of an approximately five-acre, undeveloped site bounded by Mount Hermon Road, Glen Canyon Road, Oak Creek Road and an exiting residential subdivision in Scotts Valley, California. Based on the topographic information available, the site slopes down from north to south with maximum and minimum elevations of about 540 feet and 495 feet, respectively. The slope of the ground surface is generally in the range of 6 to 1 (horizontal to vertical) or flatter at the southern part of the site, while at the north part of the site, slopes of up to 3 to 1 are present. At the time of our site exploration, the ground surface was covered with some grasses and weeds. A geotechnical investigation performed at the site in 1988 by others indicated that some loose fill soils were present along Mount Hermon and Glen Canyon Roads.

3.2 Field Investigation

A field investigation was performed on February 28, 1997. The field investigation consisted of a site reconnaissance, and drilling and sampling of nine borings to depths of between 11-1/2 and 26-1/2 feet below the existing ground surface. The borings were drilled using a truck-mounted drill rig equipped with 8-inch diameter hollow stem augers. The soils encountered in the borings were visually classified in the field and a continuous log of each boring was recorded by one of our engineers. Samples were obtained from the borings by driving either a 2-inch inside diameter Modified California tube sampler, or a 2-inch outside diameter split-spoon sampler, to a depth of 18 inches into the underlying soil using a 140-pound hammer falling 30 inches. The number of blows required to drive the sampler was recorded for each 6-inch penetration interval. The number of blows required to drive the sampler the last 12 inches are noted on the boring logs.

Visual classification of the soils encountered in our exploratory borings was made in general accordance with the Unified Soil Classification System (ASTM D2487). A key for the classification of the soils is presented in Appendix A on the Boring Log Legend, Plate A-1. The logs of the borings drilled for this investigation are presented on Plates A-2 through A-10 of Appendix A. The approximate locations of the borings are shown on Plate 2 and were estimated by our engineer in the field based on measurements from the existing site corners. The locations of the borings should be considerated accurate only to the degree implied by the method used.

3.3. Laboratory Testing

Laboratory tests were performed on selected soil samples to evaluate their physical characteristics and engineering properties. The laboratory testing program included measurements of in-situ unit weights, moisture contents and Atterberg limits. The laboratory test results are presented on the individual boring logs. Graphic presentations of the results of Atterberg limits are presented on Plate B-1 in Appendix B.

3.4. Subsurface Conditions

In general, the site soil conditions consist of an upper layer of medium stiff to stiff silty clay soils. The thickness of the upper silty clay soils vary from about 2 to 10 feet at the boring locations. At the locations of Borings B-3 and B-6, the upper 7 feet and 5-1/2 feet, respectively, consists of medium stiff silty and sandy clay and loose clayey sand fill. Below the upper silty clay and fill soils, the deeper soils consist of stiff to hard silty clays; the auger met drilling refusal at depths of about 16 feet in Boring B-4 and 13 feet in Boring B-8, indicating very hard materials. We obtained a Plasticity Index of 25 on a sample taken from the silty clay which indicates the soils to have a moderate to high expansion potential. At the location of Boring B-5, a stratum of medium dense sand was encountered at a depth of about 23 feet and extended to the bottom of the boring at 26-1/2 feet, and at the location of Boring B-8, a medium dense silty sand was encountered from about 4½ to 7½ feet.

Groundwater was encountered in Borings B-4 and B-5 at depths of 16-1/2 and 19 feet respectively; groundwater was not encountered in the other borings drilled at the site. It should be noted that the borings were not left open and the groundwater level may not have had time to stabilize in the boreholes. Groundwater levels may fluctuate depending on factors such as seasonal rainfall, groundwater withdrawal and construction activities on this or adjacent properties. Due to the presence of the fill and less stiff upper soils overlying the site, there is also a possibility that a perched groundwater condition could exist, or develop, particularly after periods of rainfall.

The above is a general description of the subsurface conditions encountered at the site in the borings drilled for this investigation. For a more detailed description of the soil conditions encountered, refer to the logs of borings presented in Appendix A.

4. CONCLUSIONS

4.1. General

Based upon the data collected during this investigation, and from a geotechnical engineering standpoint, it is our opinion that the site may be developed as presently planned provided that the recommendations presented in this report are incorporated in the design and construction of the project.

Our primary geotechnical consideration with respect to the proposed construction is 1) the presence of loose fills which if not removed and recompacted, would likely result in unacceptable total and differential settlements of the structures and pavements; 2) the moderate to high expansion and shrinkage potential of the native site soils; and 3) the location of buildings across cut-fill transitions where very stiff to hard soils will underlie part of the buildings. To reduce the potential for differential settlement across these two different materials, the cut section of the building pad should be undercut and reconstructed as engineered fill to provide a pad uniformly underlain by engineered fill.

Detailed geotechnical engineering recommendations addressing the surficial soils, site clearing and preparation, site earthwork, foundations, slabs-on-grade, retaining walls and asphalt pavements are presented in the remaining portions of this report. The following opinions, conclusions, and recommendations are based on the properties of the materials encountered in our borings and the results of the laboratory testing program.

4.2. Fill Soils

As discussed previously, loose fill soils were identified at the southern part of the site. The placement of building foundations or pavement sections on these existing fills will likely result in unacceptable total and differential settlements of the ground surface due to compression.

The existing fills appear to be located within areas which will be filled to create the building pads, parking lots and driveways. Therefore, these fills should be overexcavated and replaced as engineered fills prior to placing the fills required for site grading.

4.3 Expansive Soils

Based on the results of our field investigation and laboratory testing programs, the silty clay soils located at the site appear to have moderate to high expansion potential. Expansive soils are characterized by their ability to undergo significant volume changes (shrink or swell) due to changes in moisture content. Changes in moisture content can result from rainfall, landscape irrigation, utility leakage, roof drainage, perched groundwater, drought or other factors. Changes in soil moisture may result in unacceptable settlement or heave of structures, concrete slabs supported on grade or pavements supported on these materials. Depending on the extent and location below finished subgrade, these soils could have a detrimental effect on the proposed construction.

4.4. Cut-Fill Transition Considerations

Based on the proposed grading plan, the buildings will be situated on both cut and fill areas of the site. Because of the dense, relatively incompressible nature of the less weathered soils underlying the site, it is our opinion that structures founded on both undisturbed native materials and fill soils will be subject to excessive differential settlements. For this reason, the cut portion of the building pads should be overexcavated a minimum of two feet below the proposed bearing level of the footings and the excavated materials replaced as engineered fill to provide a relatively uniform fill thickness below the buildings footprints.

4.5. Liquefaction

Soil liquefaction is a phenomenon in which saturated, cohesionless soils lose their strength due to the build-up of excess pore water pressure during cyclic loading such as that induced by earthquakes. The primary factors affecting the liquefaction potential of a soil deposit are 1) level and duration of earthquake shaking; 2) soil type and consistency; 3) existing overburden; and 4) depth to groundwater. Soils most susceptible to liquefaction are clean, loose, fine-grained sands, and silts which are saturated and uniformly graded. Silty sands have also been proven to be susceptible to liquefaction. The occurrence of liquefaction is generally limited to soils located within about 50 feet of the ground surface.

The subsurface soils encountered in our exploratory borings consist generally of clays and silts, with the exception of two borings where sands were encountered. At the location of boring B-5, a medium dense medium to coarse grained sand was encountered below the groundwater level at a depth of about 22 feet. There is a low to moderate possibility that this sand could liquefy during strong earthquake shaking. However, even if it did experience liquiefaction, because it is confined below relatively stiff, non-liquefiable soils, and does not appear to be continuous across the site, there is a very low possibility of ground surface failure. The consequences of liquefaction would be limited to minor settlement of the ground surface. The silty sand idenfied in Boring B-8 appears to be located above the groundwater level, and therefore liquefaction of this sand stratum is unlikely.

4.6 Soil Profile Co-Efficient and Near Source Factors

The project site is located in Seismic Zone 4, and can be classified , from a seismic standpoint, as being a relatively stiff soil site with soil thickness less than 200 feet. Based on the blow count data from the boring logs, the appropriate soil profile coefficient factor, S_2, equal to an "S" factor of 1.0 according to Table 16A-J of the 1995 California Building Code.

Structures located near a seismic source may have a higher potential for damage due to near-fault motions. The Near Source Acceleration Factor, N_a, and the Near Source Velocity Factor, N_v, were estimated for the site from Tables 16-S and 16-T as proposed by the Structural Engineers Association of California (SEAOC) to the International Conference of Building Officials (ICBO) for adoption into the 1997 Uniform Building Code (UBC). For the San Andreas fault, a Type A fault, located a distance of about 12 kilometers from the site, N_a is estimated at 1.0 and N_v is estimated at 1.1.

5. RECOMMENDATIONS

5.1. Site Preparation and Grading

5.1.1. General

Grading is most economically performed during the summer months when the on-site soils are driest. Delays should be anticipated in site grading performed during the rainy season due to excessive soil moisture. Comparatively expensive construction procedures should be anticipated if grading must be performed or initiated soon after the rainy season. Where excessively wet soils are present, chemical stabilization, removal and replacement, or allowance for natural drying through evaporation will be required if grading is to be accomplished as recommended below. As discussed, the cut portions of the building pads should be overexcavated to a level two feet below the bottom of the footing bearing level and the material replaced as engineered.

5.1.2. Stripping and Site Clearing

Prior to the start of site grading, all surface vegetation, organic-landen soils and other deleterious matter should be removed from areas to be graded. The actual depth of stripping should be determined by Kleinfelder at the time of grading. Materials resulting from the stripping operations may be stockpiled on-site for subsequent use as topsoil in landscaped areas or should be removed from the site. These materials are not acceptable for use in engineered fills. In the event that the site is cleared and vegetation allowed to grow in the cleared areas prior to start of construction, additional stripping may be required prior to initiating earth moving activities.

Prior to start of construction, the site should be cleared of all fill soils, abandoned and/or designated buried utility lines and other below grade obstacles encountered during this operation. All rubble and debris generated during the clearing operation should be removed from the site. To allow us to substantiate that our recommendations for site clearing have been adhered to, the site clearing work should be performed under the observation of a representative from Kleinfelder. Existing fills should be completely remove a horizontal distance of 5 feet beyond the building perimeter.

5.1.3. Subgrade Preparation

Upon the completion of site clearing, we recommend that the cut portions of the building pads be overexcavated to a level two feet below the proposed footing bearing level and this material be replaced as engineering fill as described below. This area of overexcavation should extend at least 5 feet beyond the building perimeters. Prior to placing fill, upper 12 inches of soil below the pad grade or in areas to receive new construction should be scarified and moisture conditioned to between 2 and 5 percent above optimum moisture content and recompacted to at least 90 percent of the laboratory maximum dry density as obtained by the ASTM D1557-91 test method. The scarified soil should be compacted in accordance with the recommendations presented below in Section 5.1.5., "Fill Placement and Compaction." This area should extend at least 5 feet beyond the building perimeters as well as adjacent concrete slabs-on-grade and parking areas.

5.1.4. Material for Fill

In general, the existing site soils with an organic content of less than three percent or without visible organic matter deemed excessive by Kleinfelder and free of any deleterious materials or hazardous substances, may be used as general engineered fill to achieve project grades except where "non-expansive" fill is recommended. Fill soil materials placed within the upper 18 inches beneath interior slabs-on-grades should consist of imported "non-expansive" materials. The "non-expansive" fill materials should extend horizontally at least five feet beyond the plan areas of all slabs-on-grade **including** adjoining exterior (perimeter) sidewalks. Non-expansive fill material should be primarily granular but should not contain any rocks or lumps larger than three inches in greatest dimension with no more that 15 percent of the material larger than 1.5 inches. The material should have a minimum of 20 percent passing the Number 200 mesh sieve and a Plasticity Index of 12 or less.

5.1.5 Fill Placement and Compaction

In general, all on-site soils should be moisture conditioned to between two and five percent above the optimum moisture content and compacted to at least 90 percent relative compaction. Subgrade soils in sidewalk areas should be compacted to between 85 and 90 percent to a depth of eight inches. Subgrade soils in pavement areas should be compacted to a minimum 95 percent in the upper 12 inches. Aggregate base for pavement sections should also be compacted to 95 percent relative compaction. The percent relative compaction should be based on the maximum dry density obtained in general accordance with ASTM D1557-91. Moisture conditioning may include the drying of soils with moisture contents exceeding 5 percent above the optimum moisture content.

Imported, non-expansive fill soils in building pad areas should be moisture conditioned to between optimum and two percent above optimum moisture content, and compacted to at least 90 percent relative compaction. Fill materials should be placed in lifts not exceeding 8 inches in uncompacted thickness. We recommend that compaction be done by mechanical means only. Due to equipment limitations, thinner lifts may be necessary to achieve the recommended level of compaction.

5.1.6. Trenches

All underground utility trenches should be backfilled with compacted engineered fill. Either the existing site soils or imported sand may be used for backfilling utility trenches. The trench backfill should be compacted to at least 90 percent relative compaction and capped with a minimum 12-inch thick layer of compacted, on-site fill soil similar to that of the adjoining subgrade. In addition, the upper 12 inches of all trench backfill in areas to be paved should be compacted to at least 95 percent relative compaction. The backfill material should be placed in lifts not exceeding eight inches in uncompacted thickness. Thinner lifts may be necessary to achieve the recommended level of compaction of the backfill due to equipment limitations. Compaction should be performed by mechanical means only. Water jetting to attain compaction should not be permitted. All trenches should be constructed in accordance with OSHA and Cal-OSHA Safety Standards.

5.1.7. Surface Drainage

Final site grading should provide surface drainage away from structures and slabs-on-grade to reduce the percolation of water into the underlying soils. Ponding of surface water should not be allowed adjacent to structures. Grades should be sloped away from the structures a minimum of 4 percent in landscaped areas and 2 percent in paved areas for a horizontal distance of at least 5 feet. Rainwater collected on the roofs of the building should be channeled through gutters, downspouts and closed pipes which discharge on the pavement or lead directly to the site storm sewer system.

5.1.8. Subsurface Drainage

To reduce the potential for surface or subsurface water from entering the building underfloor areas, we recommend that a continuous subdrain be located across the site immediately adjacent to the rear (upslope) of the buildings. The subdrain should consist of a four-inch diameter perforated drain pipe, placed in the bottom of a trench at least 12 inches wide and three feet deep. The trench should be backfilled with drain material, such as 3/4-inch by 1/2-inch drain rock wrapped in a non-woven geotextile such as Mirafi 140N. The upper 12 inches of the subdrain should be backfilled with compacted on-site clayey soils. The subdrain pipe should be connected to a solid drain pipe leading to a suitable discharge point.

5.1.9. Seepage Control

Where slabs or pavements abut against landscaped areas, some method should be used to protect the base rock layer and subgrade soils against saturation from water in the landscaped areas. If landscape water or surface runoff is allowed to seep into the pavement section, the service life of the pavement may be reduced. Methods of reducing seepage into the pavement sections may include vertical curbs extending at two inches below the base rock/subgrade interface, or subdrains behind curbs in landscape areas. Also, care should be taken to prevent over-watering of landscaped areas adjacent to pavements. Cut-offs should be carefully constructed such that they extend below the base rock section and are poured neat against undisturbed native soil or compacted clayey fill. The cut-offs should be continuous and any utility trenches (irrigation lines, electrical conduit, etc.) that extend through, or under the curbs, should be sealed with compacted clayey soil or poured in-place concrete.

Where utility lines extend through or beneath perimeter footings, permeable backfill should be terminated at least one-foot from the footing. Concrete should be used around the pipe to act as a seepage cutoff. Beneath footings, the pipe should be "sleeved" through concrete cut-offs, and the annular space around the pipe should be filled with a waterproof caulk. This will help reduce the amount of water seeping through the previous trench backfill and collecting under the buildings.

5.1.10. Construction Observation

Variations in soil types and conditions are possible and may be encountered during construction. In order to permit correlation between the soil data and the actual soil conditions encountered during construction, and to check for conformance with our recommendations as originally contemplated, it is imperative that Kleinfelder be retained to perform continuous review as required during earthwork, excavation, and foundation phases of construction. All earthwork should be performed in accordance with the recommendations presented in this report, or as recommended by Kleinfelder during construction.

5.2 Foundations

5.2.1. Shallow Foundations

The proposed structures may be supported on conventional spread footings bearing on engineered fill soils. Continuous footings should have a minimum width of 18 inches and isolated spread footings a minimum width of 24 inches. Exterior footings should be embedded a minimum of 18 inches below the lowest adjacent grade or bottom of slab, whichever is deeper. Interior footings surrounded by concrete slabs-on-grade should be embedded a minimum of 18 inches below the bottom of the slab. All shallow footings may be designed for an allowable bearing pressure of 2,500 pounds per square foot due to dead plus live loads. The allowable bearing pressure may be increased by one-third for transient loads such as seismic or wind.

To maintain the desired support for the foundations, footings located adjacent to utility trenches or other footings should be deepened as necessary so that the bearing surfaces are below an imaginary plane having an inclination of 1.5 horizontal to 1.0 vertical, extending upward from the bottom edge of the adjacent footing or utility trench. If utility trenches will be located such that the footings will bear in the zone above the above described imaginary plane, we recommend that the trenches be backfilled with a sand-cement slurry (controlled density backfill) consisting of a two sack mix.

All visible cracks in the bottom of footing excavations should be closed by moisture conditioning for a minimum of two days prior to placement of concrete. Water should not be allowed to pond in the bottom of the excavations. Areas which become excessively wet should be over-excavated to a firm base. We recommend that Kleinfelder be retained to observe the footing excavations prior to placing reinforcing steel or concrete to check that footings are founded in the anticipated bearing soil.

5.2.2. Lateral Load Resistance

For footings bearing on engineered fill materials, lateral loads may be resisted by friction between the foundations and the supporting subgrade. A friction coefficient of 0.30 can be used for design. Additionally, lateral resistance may be provided by passive pressures acting against the sides of spread footings provided they are poured neat against undisturbed soil or properly compacted fill soils. We recommend that a uniform passive pressure of 300 pounds per cubic foot be used for design purposes. This passive pressure can be assumed to act starting at the top of the lowest adjacent grade in paved areas and at a depth of one-foot below grade in unpaved areas. The allowable passive pressure may be increased by one-third for lateral loading due to wind or seismic forces.

5.2.3 Settlement

Provided the building areas are properly graded and foundations designed and constructed in accordance with our recommendations, we estimate the maximum total post-construction settlement resulting from surcharges imposed by the foundation loads will be about one inch, and that post-construction differential settlements will be on the order of ¾ inch over a horizontal distance of about 40 feet. It should be noted that specific foundation loads were not available at the time this report was prepared. Our assumptions regarding the building loads are discussed under Section 1.1. Further settlement analyses may be warranted when structural design loads have been finalized.

5.3. Slabs-on-Grade

5.3.1. General

We recommend that concrete slabs-on-grade in the building pads be placed on non-expansive fill soils, as described above. Prior to the placement of the non-expansive fill, the exposed subgrade should be moisture conditioned and compacted as described above. A four-inch thick layer of capillary break material (free-draining crushed rock), and a two-inch thick sand cover, as discussed below should also be placed under the slabs.

Although the recommended subgrade preparation methods presented in this report will help to reduce the potential problems associated with expansive soils, it may not fully eliminate them. Therefore, a mitigation measures including reinforcement and construction joints, as discussed below, are recommended for inclusion in the structural design to further reduce the effects of the expansive soils. Frequent construction or control joints should be provided in all concrete slabs where cracking is objectionable.

5.3.2. Interior Slabs-on-Grade

Imported, non-expansive materials as discussed in Section 5.1.4 "Materials for Fill" should be used within the upper 15 inches of the building areas. Prior to the placement of the non-expansive fill, the subgrade soils in the slab-on-grade areas of the buildings should be scarified, moisture conditioned and compacted as discussed in Section 5.1.3. "Subgrade Preparation." Once moisture conditioned and compacted, the subgrade soils should not be allowed to dry out prior to the placement of non-expansive fill. A capillary break consisting of at least four inches of free draining gravel, such as 3/4-inch by 1/2-inch crushed rock, should be provided beneath the slabs. Slabs should be provided with a plastic vapor barrier of at least 10-mil thickness over the free draining gravel layer where vinyl floor covering, carpets, tile, or other moisture sensitive flooring will be placed on the floor, or where moisture protection is desired. To promote more uniform curing of the concrete, and to provide protection for the membrane, two inches of clean sand should be placed on top of the membrane prior to the placement of concrete. The gravel and sand layer may be considered as the upper six inches of the recommended non-expansive fill section.

As a minimum, slabs should be at least four inches thick, and should be reinforced to reduce cracking due to the expansive soils. The slab thickness and reinforcement should be designed by a Structural Engineer. Joints spaced no farther apart that 12 feet should be provided to control cracking. As a minimum, we suggest that slabs contain No. 3 reinforcing bars at 18 inches on center each way. Care should be taken to see that the reinforcing is placed a slab mid-height. The minimum recommended steel will not prevent the development of slab cracks but will aid in keeping joints relatively tight and reduce the potential for differential movement between adjacent panels.

5.3.3. Exterior Flatwork

For either exterior non-vehicular use concrete slabs-on-grade or walkways, the hardscape should be underlain by four inches of Class 2 aggregate baserock compacted to a minimum of 90 percent relative compaction.

Exterior flatwork will be subjected to edge effects due to the drying out of the subgrade soils along the outer edge of the slab. Deepened edge sections and controlled irrigation of landscaped areas adjacent to the flatwork will aid in reducing the potential for the shrinkage and swelling of the underling expansive soils. By maintaining the soil moisture content, the resulting soil displacement or shrink/swell cycles will also be reduced.

To reduce the amount of vertical movement of exterior concrete flatwork, which will not be subjected to vehicular traffic, we recommend that subgrades for these slabs be scarified to a minimum depth of 12 inches, moisture conditioned to between two and five percent above the optimum moisture content and compacted to a relative compaction of between 85 and 90 percent.

Subgrade soils beneath exterior flatwork subject to vehicle traffic should be compacted to 90 percent. Increased depth of moisture conditioning or the use of a non-expansive fill layer beneath exterior slabs-on-grade will further reduce the potential for displacement of overlying concrete slabs as a result of soil swell with moisture increase.

5.4. Retaining Walls

We anticipate there will be retaining walls for grade separation at the north part of the site. We recommend that cantilevered retaining walls be designed to resist an equivalent fluid pressure of 55 pounds per cubic foot (pcf) for the case of a horizontal backfill. The earth pressures presented do not include hydrostatic pressures and assumes that the wall will be backfilled with compacted native soil. Wherever the walls will be subjected to surcharge loads, the walls should be designed for an additional uniform lateral pressure equal to one-third of the anticipated surcharge load.

To reduce the build-up of hydrostatic pressures, drainage should be provided behind the retaining wall. The wall may be drained by providing at least a 12-inch wide zone of drain material, such as 3/4-inch by 1/2-inch drain rock wrapped in a non-woven geotextile such as Mirafi 140N, behind the back face of the wall. Alternatively, drainage may be provided by the placement of a commercially produced composite drainage blanket, such as Miradrain, extending continuously up from the base of the wall. The wall drain should extend from the base of the wall to about 18 inches below the top of the wall, and should be capped with a layer of compacted impervious native soil. The wall drain should be connected to a four-inch diameter perforated drain pipe surrounded with drain rock wrapped in filter fabric. Drainage outlet should be provided by a solid drain pipe leading to a suitable discharge point. To reduce the potential adverse affects of water seeping through the walls, we recommend that the backs of the walls be water/damp proofed prior to backfilling.

The retaining wall may be supported on a spread foundations bearing on undisturbed natural soils or engineered fill. Footings should be designed as discussed in Section 5.2.1. "Shallow

Foundations." Resistance to lateral loads can be obtained as discussed in Section 5.2.2. "Lateral Load Resistance" of this report.

5.6. Pavements

Pavement for this project will consist of parking and driveway areas. We assume vehicle loading for this project will be variable and consist primarily of passenger vehicles. For this use, we estimate traffic indices of 4.5 to 7. The actual traffic indices used should be determined by the Civil Engineer in consultation with Hal Porter Homes.

We have made our pavement section recommendations assuming the pavement subgrade soil will be similar to the silty clay soils described in the boring logs. If site grading exposes soil other than that assumed, we should perform additional tests to confirm or revise the recommended pavement sections for actual field conditions.

Based on the soil particle size and plasticity, we have estimated an (R)-Value of 5. In general, Test Method 301-F of the State of California Department of Transportation, as modified by the "Flexible Pavement Structural Section Design Guide for California Cities and Counties," was used to develop recommended pavement design sections. Development of the California pavement design method is based on empirical formulas and data, primarily related to highways and moving loads. Therefore, selection of a Traffic Index (TI) cannot be made strictly based on the estimated volume of traffic for the desired life span.

We recommend that the anticipated traffic and the alternate pavement sections presented be reviewed by the project civil engineer in consultation with the owner during the development of the final grading and paving plans. As a minimum, we recommend the use of a section based on a TI of 5 for areas subject to automobiles traffic lanes. Where moderate to frequent light truck traffic is anticipated, we suggest the use of a pavement section for a TI of at least 6. Rigid paving consisting of Portland cement concrete should be considered for use for use at truck docks, refuse bin pick-up locations and areas where trucks may be parked.

Asphalt concrete, aggregate base and subbase, and preparation of the subgrade should conform to, and be placed in accordance with, the California Department of Transportation Standard Specification, latest revision, except as noted herein. ASTM Test procedures should be used to assess the percent relative compaction of soils, aggregate base and asphalt concrete. Asphalt concrete should be compacted to a minimum of 92 percent of the maximum theoretical unity weight (Rice Gravity).

Parking areas should be sloped at a minimum of 2 percent and drainage gradients maintained to carry all surface water off the site. Surface water ponding should not be allowed anywhere on the site during construction. Seepage cut-offs should be constructed as presented above in Section 5.1.9. "Seepage Control."

The pavement subgrade should be scarified to a depth of 12 inches and compacted to a minimum of 95 percent relative compaction at a moisture content of between 2 and 5 percent above optimum moisture. The Class 2 aggregate base course material should be compacted to at least 95 percent relative compaction. The percent relative compaction should be based on the maximum dry density obtained in general accordance with ASTM D1557-91.

FLEXIBLE PAVEMENT DESIGN ALTERNATIVES

(Subgrade R-value = 5)

Traffic Index	Asphalt Concrete (inches)	Aggregate Base Class 2 (inches)	Total Pavement Thickness (inches)
4.5	2.5	9.5	12.0
5.0	2.5	11.0	13.5
5.5	3.0	12.0	15.0
6.0	3.0	14.0	17.0
6.5	3.5	15.0	18.5
7.0	4.0	15.5	19.5

AC = Type B Asphalt Concrete
AB = Class 2 Aggregate Base (Minimum R-Value = 78)

Each traffic index represents a different level of use. The owner or designer should determine which level of use best reflects the project and select appropriate pavement section(s).

6. ADDITIONAL SERVICES AND LIMITATIONS

6.1. Additional Services

The review of plans and specifications, and the observation and testing by Kleinfelder of earthwork related construction activities, are an integral part of the conclusions and recommendations made in this report. If Kleinfelder is not retained for these services, the client will be assuming our responsibility for any potential claims that may arise during or after construction. The required tests, observations, and consultation by Kleinfelder during construction includes, but is not limited to:

- review of plans and specifications;
- observation of site clearing;
- observation of retaining wall construction;
- construction observation and density testing of fill material placement, trench backfill and subgrade preparation;
- observation of foundation excavations and foundation construction.

6.2. Limitations

Recommendations contained in this report are for the proposed Oak Creek Park Office Center in Scotts Valley, California, as described in this report. Our recommendations are based upon field observations, data from nine exploratory borings drilled for this study and five drilled previously by others, laboratory tests, our analyses and our present knowledge of the proposed construction. It is possible that subsurface conditions could vary between and beyond the points explored. If soil and groundwater conditions are encountered during construction which differ from those described herein, our firm should be notified immediately in order that a review may be made and any supplemental recommendations provided. If the scope of the proposed construction, including proposed loads, grades, or structure locations change from that described in this report, our recommendations should also be reviewed.

Our firm has prepared this report for exclusive use of Hal Porter Homes in substantial accordance with the generally accepted geotechnical engineering practice as it exists in the site area at the time of our study. No warranty is expressed or implied. The recommendations provided in this report are based on the assumption that an adequate program of tests and observations will be conducted by our firm during the construction phase in order to evaluate compliance with our recommendations. If we are not retained for these services, the client agrees to assume Kleinfelder's responsibility for any potential claims that may arise during or after construction.

This report is issued with the understanding that the client chooses the risk he wishes to bear by the expenditures involved with the construction alternatives and scheduling that is chosen. It is the client's responsibility to see that all parties to the project including the designer, contractor, subcontractors, etc., are made aware of this report in its entirety including the Additional Services and Limitations section.

****

Base: U.S.G.S. Felton 7.5' Quadrangle

Scale 1 inch = 2000 feet

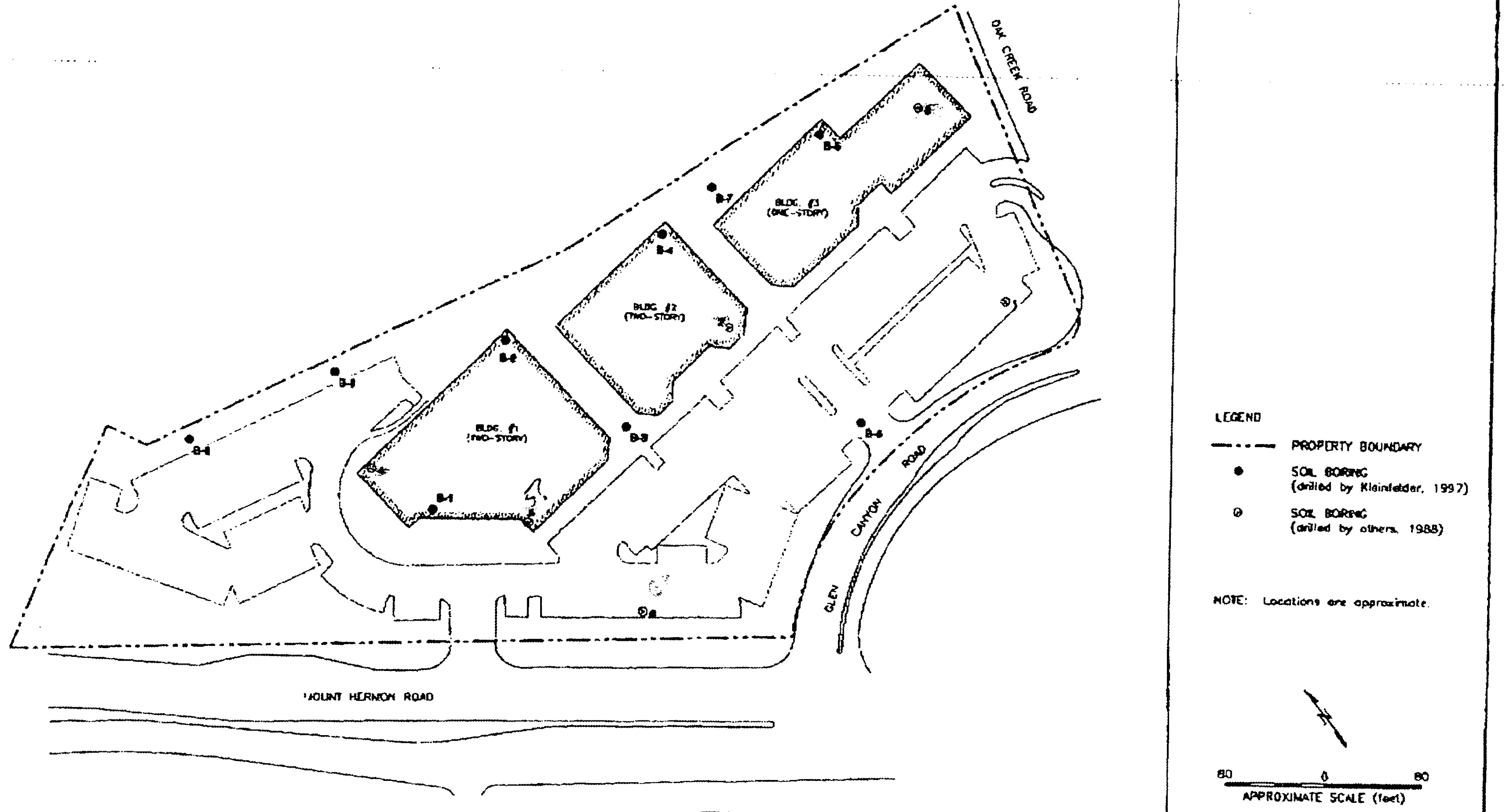
KLEINFELDER

PROJECT NO. 12-3025-10

VICINITY MAP

OAK CREEK PARK OFFICE CENTER
MOUNT HERMON ROAD
SCOTTS, VALLEY CALIFORNIA

PLATE
1



LEGEND

—·—·— PROPERTY BOUNDARY

● SOIL BORING (drilled by Kleinfelder, 1997)

⊘ SOIL BORING (drilled by others, 1988)

NOTE: Locations are approximate.

KLEINFELDER

DRAFTED BY: L. Sue DATE: 3-26-97

CHECKED BY: S. Leck DATE: 3-27-97

SITE PLAN

OAK CREEK PARK OFFICE CENTER
SCOTTS VALLEY, CALIFORNIA

PROJECT NO. 12-302510-001

PLATE

2

MAJOR DIVISIONS					TYPICAL NAMES
COARSE GRAINED SOILS More than Half > #200 sieve	GRAVELS MORE THAN HALF COARSE FRACTION IS LARGER THAN NO. 4 SIEVE	CLEAN GRAVELS WITH LITTLE OR NO FINES	GW	⁘	WELL GRADED GRAVELS, GRAVEL-SAND
			GP	●	POORLY GRADED GRAVELS, GRAVEL-SAND MIXTURES
		GRAVELS WITH OVER 12% FINES	GM	▥	SILTY GRAVELS, POORLY GRADED GRAVEL-SAND-SILT MIXTURES
			GC	▨	CLAYEY GRAVELS, POORLY GRADED GRAVEL-SAND-CLAY MIXTURES
	SANDS MORE THAN HALF COARSE FRACTION IS SMALLER THAN NO. 4 SIEVE	CLEAN SANDS WITH LITTLE OR NO FINES	SW	⁘	WELL GRADED SANDS, GRAVELLY SANDS
			SP	⁚	POORLY GRADED SANDS, GRAVELLY SANDS
		SANDS WITH OVER 12% FINES	SM	▥	SILTY SANDS, POOORLY GRADED SAND-SILT MIXTURES
			SC	▨	CLAYEY SANDS, POORLY GRADED SAND-CLAY MIXTURES
FINE GRAINED SOILS More than Half < #200 sieve	SILTS AND CLAYS LIQUID LIMIT LESS THAN 50		ML	▥	INORGANIC SILTS AND VERY FINE SANDS, ROCK FLOUR, SILTY OR CLAYEY FINE SANDS, OR CLAYEY SILTS WITH SLIGHT PLASTICITY
			CL	▨	INORGANIC CLAYS OF LOW TO MEDIUM PLASTICITY, GRAVELLY CLAYS, SANDY CLAYS, SILTY CLAYS, LEAN CLAYS
			OL	▥	ORGANIC CLAYS AND ORGANIC SILTY CLAYS OF LOW PLASTICITY
	SILTS AND CLAYS LIQUID LIMIT GREATER THAN 50		MH	▮	INORGANIC SILTS, MICACEOUS OR DIATOMACIOUS 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

UNIFIED SOIL CLASSIFICATION SYSTEM

			Shear Strength, psf Confining Pressure, psf	
PS	Percent Saturation			
SG	Specific Gravity			
Consol	Consolidation	Tx	2630 (240)	Unconsolidated Undrained Triaxial
LL	Liquid Limit (in %)	Tx sat	2100 (575)	Unconsolidated Undrained Triaxial, saturated prior to test
PL	Plastic Limit (in %)	DS	3740 (960)	Consolidated Drained Direct Shear
PI	Plasticity Index	FVS	1320	Field Vane Shear
TS	Total Saturation Moisture Content	UC	4200	Unconfined Compression
SA	Sieve Analysis	LVS	500	Laboratory Vane Shear
■	Undisturbed Sample	C	Concrete Compressive Strength	
☒	Bulk Sample	PE	Petrographic Examination	
◪	Standard Penetration Test	Perm	Permeability	
◨	Sample Attempt with No Recovery	PP	Pocket Penetrometer (tsf) Unc. Compressive Strength	

KEY TO TEST DATA

KLEINFELDER
Geotechnical, Materials and Environmental Engineering

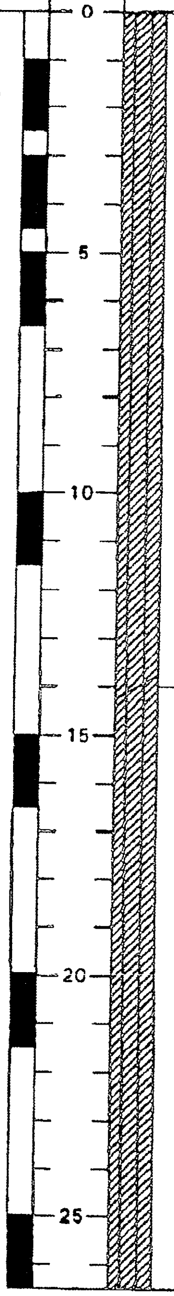
PROJECT NUMBER 12-3025-10 DATE APR 1997

SOIL CLASSIFICATION CHART AND KEY TO TEST DATA
OAK CREEK PARK OFFICE CENTER
SCOTTS VALLEY, CALIFORNIA

PLATE
A-1

EQUIPMENT: 8"-Dia. Hollow Auger
LOGGED BY: M.Briseno
ELEVATION: **
START DATE: 2-28-97
FINISH DATE: 2-28-97

Other Laboratory Tests	Dry Density (pcf)	Moisture Content (%)	Blows/ Foot *
PP = 3.5			19
PP = 4.5 +			40
PP = 4.5 +			55
PP = 4.5 +			59
PP = 4.5 +			48
PP = 4.5 +			44
PP = 4.5 +			41



MOTTLED YELLOW TO GRAY BROWN SILTY CLAY (CL) very stiff to hard, dry to moist, with traces of sand, medium plasticity

hard

GRAY SILTY CLAY (CL) hard, dry to moist, medium plasticity

Bottom of Boring B-1 @ 26.5 feet.
No Free Groundwater Observed.

* Converted to equivalent standard penetration blow counts.
** Existing ground surface at time of drilling.

KLEINFELDER
Geotechnical, Materials and Environmental Engineering

PROJECT NUMBER 12-3025-10 DATE APR 1997

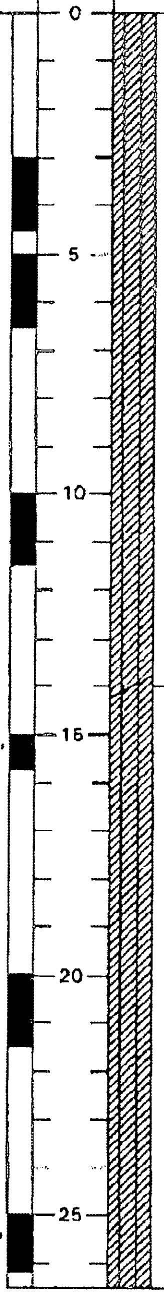
LOG OF BORING B-1

OAK CREEK PARK
OFFICE CENTER
SCOTTS VALLEY, CALIFORNIA

PLATE
A-2

EQUIPMENT: 8"-Dia. Hollow Auger
LOGGED BY: M.Briseno
ELEVATION: **
START DATE: 2-28-97
FINISH DATE: 2-28-97

Other Laboratory Tests	Dry Density (pcf)	Moisture Content (%)	Blows/ Foot *
PP = 4.5 +			33
PI = 25, see Plate B-1 minus #200 = 75% PP = 4.5 +			51
PP = 4.5 +			50
PP = 4.5 +			70/8"
PP = 4.5 +			72
PP = 4.5 +			76/8"



MOTTLED YELLOW TO GRAY BROWN SILTY CLAY (CL) very stiff, dry to moist, medium plasticity

hard

mottled gray and yellow brown

GRAY SILTY CLAY (CL) hard, dry to moist, medium plasticity

Bottom of Boring B-2 @ 26.5 feet.

* Converted to equivalent standard penetration blow counts.

** Existing ground surface at time of drilling.

KLEINFELDER
Geotechnical, Materials and Environmental Engineering

PROJECT NUMBER 12-3025-10 DATE APR 1997

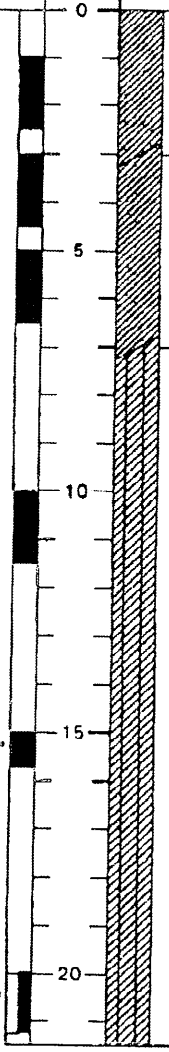
LOG OF BORING B-2

OAK CREEK PARK OFFICE CENTER SCOTTS VALLEY, CALIFORNIA

PLATE
A-3

EQUIPMENT: 8"-Dia. Hollow Auger
LOGGED BY: M.Briseno
ELEVATION: **
START DATE: 2-28-97
FINISH DATE: 2-28-97

Other Laboratory Tests	Dry Density (pcf)	Moisture Content (%)	Blows/ Foot *
PP = 1.5	87	26.0	5
PP = 1.0	94	25.0	6
PP = 2.5	94	22.0	7
PP = 4.5 +			68
PP = 4.5 +			70/8"
PP + 4.5 +			73/9"



YELLOW TO DARK BROWN SANDY CLAY (CL) medium stiff, moist to wet, trace of roots, medium plasticity (FILL)

DARK BROWN SILTY CLAY (CL) medium stiff, moist, medium plasticity (FILL)

MOTTLED YELLOW BROWN SILTY CLAY (CL) hard, dry to moist, medium plasticity

Bottom of Boring B-3 @ 21.5 feet.
No Free Groundwater Observed.

* Converted to equivalent standard penetration blow counts.
** Existing ground surface at time of drilling.

KLEINFELDER
Geotechnical, Materials and Environmental Engineering

PROJECT NUMBER 12-3025-10 DATE APR 1997

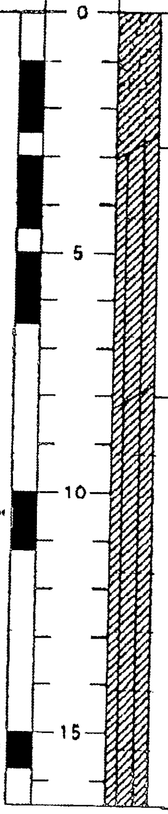
LOG OF BORING B-3

OAK CREEK PARK
OFFICE CENTER
SCOTTS VALLEY, CALIFORNIA

PLATE
A-4

EQUIPMENT: 8"-Dia. Hollow Auger
LOGGED BY: M.Briseno
ELEVATION: **
START DATE: 2-28-97
FINISH DATE: 2-28-97

Other Laboratory Tests	Dry Density (pcf)	Moisture Content (%)	Blows/ Foot *
PP = 1.75			5
PP = 2.0	102	20.0	8
PP = 4.5	98	22.0	16
PP = 4.5 +			44/8"
PP = 4.5 +			44/8"



BROWN SILTY CLAY (CL) medium stiff, moist, with trace of roots, medium plasticity

MOTTLED YELLOW BROWN SILTY CLAY (CL) stiff to hard, moist with trace of roots, medium plasticity

MOTTLED YELLOW BROWN SILTY CLAY (CL) hard, moist, medium plasticity

Auger refusal @ 16.5 feet.

Bottom of Boring B-4 @ 16.5 feet.
No Free Groundwater Observed.

* Converted to equivalent standard penetration blow counts.

** Existing ground surface at time of drilling.

KLEINFELDER
Geotechnical, Materials and Environmental Engineering

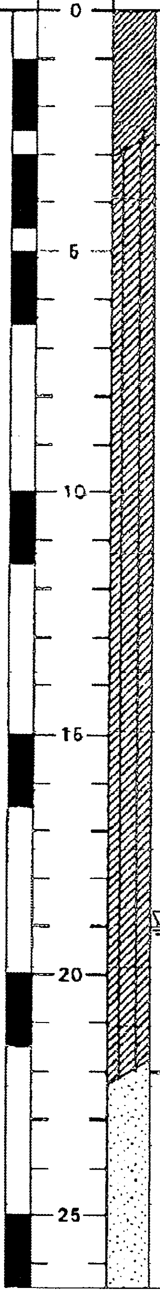
PROJECT NUMBER 12-3025-10 DATE APR 1997

LOG OF BORING B-4

OAK CREEK PARK
OFFICE CENTER
SCOTTS VALLEY, CALIFORNIA

PLATE
A-5

EQUIPMENT: 8"-Dia. Hollow Auger
LOGGED BY: M.Brisano
ELEVATION: **
START DATE: 2-28-97
FINISH DATE: 2-28-97



Other Laboratory Tests	Dry Density (pcf)	Moisture Content (%)	Blows/ Foot *	
PP = 1.5			11	**BROWN SANDY CLAY (CL)** stiff, moist, medium plasticity
PP = 1.0			2	**MOTTLED YELLOW BROWN SILTY CLAY (CL)** stiff to very stiff, moist to wet, with traces of roots and sand, medium plasticity
PP = 2.0				
PP = 4.0			9	
PP = 3.5			10	visible moisture on sample
PP = 3.5			16	
PP = 2.75			14	grading olive gray and sandier
				GRAY CLEAN SAND (SP) medium dense, wet, medium to coarse-grained
			14	

Bottom of Boring B-5 @ 26.5 feet.

* Converted to equivalent standard penetration blow counts.

** Existing ground surface at time of drilling.

KLEINFELDER
Geotechnical, Materials and Environmental Engineering

PROJECT NUMBER 12-3026-10 DATE APR 1997

LOG OF BORING B-5

OAK CREEK PARK
OFFICE CENTER
SCOTTS VALLEY, CALIFORNIA

PLATE
A-6

EQUIPMENT: 8"-Dia. Hollow Auger
LOGGED BY: M.Briseno
ELEVATION: **
START DATE: 2-28-97
FINISH DATE: 2-28-97

Other Laboratory Tests	Dry Density (pcf)	Moisture Content (%)	Blows/ Foot *	DEPTH (FEET)	
				0	**MOTTLED YELLOW BROWN SILTY CLAY (CL)** stiff to very stiff, moist, with traces of roots, medium plasticity
PP = 1.5			2		
PP = 3.0			11	5	BROWN SILTY CLAY (CL) stiff, moist, trace of sand
PP = 4.5			7	10	
PP = 4.0			22	15	

Bottom of Boring B-7 @ 16.5 feet.
No Free Groundwater Observed.

* Converted to equivalent standard penetration blow counts.

** Existing ground surface at time of drilling.

KLEINFELDER
Geotechnical, Materials and Environmental Engineering

PROJECT NUMBER 12-3025-10 DATE APR 1997

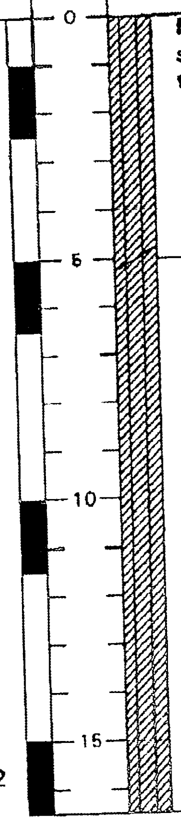
LOG OF BORING B-7

OAK CREEK PARK
OFFICE CENTER
SCOTTS VALLEY, CALIFORNIA

PLATE
A-7

EQUIPMENT: 8"-Dia. Hollow Auger
LOGGED BY: M.Briseno
ELEVATION: **
START DATE: 2-28-97
FINISH DATE: 2-28-97

Other Laboratory Tests	Dry Density (pcf)	Moisture Content (%)	Blows/ Foot *
PP=1.5			2
PP=3.0			11
PP=4.5			7
PP=4.0			22



MOTTLED YELLOW BROWN SILTY CLAY (CL) stiff to very stiff, moist, with traces of roots, medium plasticity

BROWN SILTY CLAY (CL) stiff, moist, trace of sand

Bottom of Boring B-7 @ 16.5 feet.
No Free Groundwater Observed.

- * Converted to equivalent standard penetration blow counts.
- ** Existing ground surface at time of drilling.

KLEINFELDER
Geotechnical, Materials and Environmental Engineering

PROJECT NUMBER 12-3025-10 DATE APR 1997

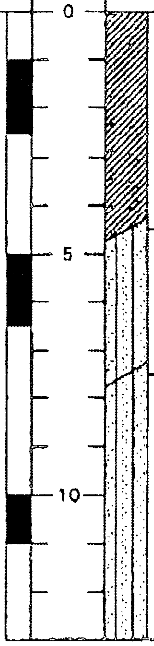
LOG OF BORING B-7

OAK CREEK PARK
OFFICE CENTER
SCOTTS VALLEY, CALIFORNIA

PLATE
A-8

EQUIPMENT: 8"-Dia. Hollow Auger
ELEVATION: **
LOGGED BY: M.Briseno
START DATE: 2-28-97
FINISH DATE: 2-28-97

Other Laboratory Tests	Dry Density (pcf)	Moisture Content (%)	Blows/ Foot *
			7
			11
PP = 4.5 +			53



MOTTLED YELLOW BROWN SANDY CLAY (CL) medium stiff, moist, low plasticity

YELLOW BROWN SILTY SAND (SM) medium dense, moist, fine-grained

GRAY SILTY CLAY (CL) hard, moist, medium plasticity

Auger refusal @ 13 feet

Bottom of Boring B-8 @ 13 feet.
No Free Groundwater Observed.

* Converted to equivalent standard penetration blow counts.

** Existing ground surface at time of drilling.

KLEINFELDER
Geotechnical, Materials and Environmental Engineering

PROJECT NUMBER 12-3025-10 DATE APR 1997

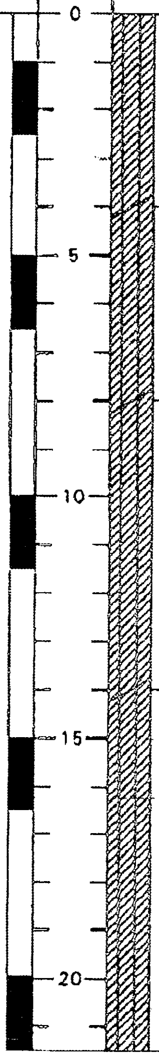
LOG OF BORING B-8

OAK CREEK PARK
OFFICE CENTER
SCOTTS VALLEY, CALIFORNIA

PLATE
A-9

EQUIPMENT: 8"-Dia. Hollow Auger
LOGGED BY: M.Briseno
ELEVATION: **
START DATE: 2-28-97
FINISH DATE: 2-28-97

Other Laboratory Tests	Dry Density (pcf)	Moisture Content (%)	Blows/ Foot *
PP = 3.25			7
PP = 4.5 +			10
PP = 4.5 +			53
PP = 4.5 +			26
PP = 4.5 +			40



BROWN SILTY CLAY (CL) stiff to very stiff, moist with traces of roots and sand, medium plasticity

MOTTLED YELLOW BROWN SILTY CLAY (CL) hard, moist, medium plasticity

GRAY BROWN SILTY CLAY (CL) very stiff to hard, moist

GRAY SILTY CLAY (CL) hard, moist, medium plasticity

Bottom of Boring B-9 @ 21.5 feet.
No Free Groundwater Observed.

* Converted to equivalent standard penetration blow counts.

** Existing ground surface at time of drilling.

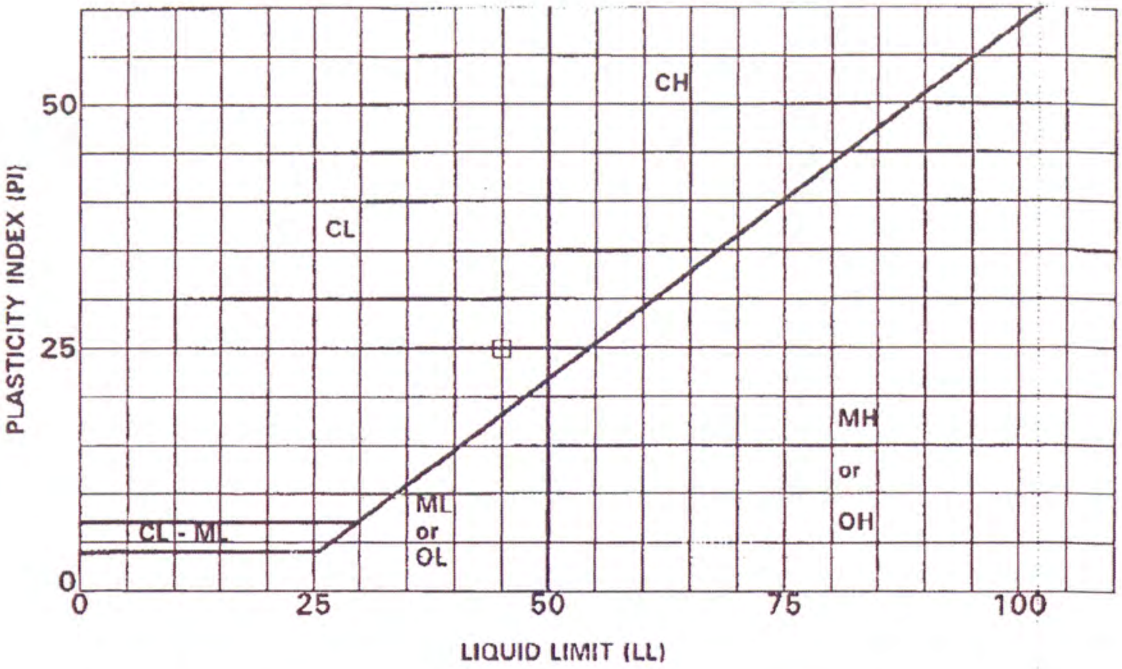
KLEINFELDER
Geotechnical, Materials and Environmental Engineering

PROJECT NUMBER 12-3025-10 DATE APR 1997

LOG OF BORING B-9

OAK CREEK PARK
OFFICE CENTER
SCOTTS VALLEY, CALIFORNIA

PLATE
A-10



Symbol	Boring	Depth	LL	PL	PI	Sample Description
☐	B-3	5.5	45	20	25	Gray Brown Silty Clay (CL)

Unified Soil Classification
Fine Grained Soil Groups

Symbol	LL < 50	Symbol	LL > 50
ML	Inorganic clayey silts to very fine of slight plasticity	MH	Inorganic silts and clayey silts of high plasticity
CL	Inorganic clays of low to medium plasticity	CH	Inorganic clays of high plasticity
OL	Organic silts and organic silty clays of low plasticity	OH	Organic clays of medium to high plasticity, organic silts

KLEINFELDER

PROJECT NO. 12-3025-10

PLASTICITY CHART

OAK CREEK PARK
OFFICE CENTER
SCOTTS VALLEY, CALIFORNIA

PLATE

B-1

LOGGED BY SAS DATE DRILLED 6/21/88 BORING DIAMETER 6" BORING NO. 1

Depth, ft.	Sample No. and type	Symbol	SOIL DESCRIPTION	Unified Soil Classification	Blows/foot 350 ft-lbs.	Qu - t. s. f. - Penetrometer	Dry Density p.c.f.	Moisture % dry wt.	MISC. LAB RESULTS
1			Brown Silty fine SAND with subrounded gravels and occasional cobbles, damp, loose						
2	1-1		Dark gray brown very fine SAND with organic debris, strong organic odor, moist, stiff		7	4.5+	89.8	27.3	
3	L								
4									
5									
6									
7	1-2		NATIVE						
8	L		Light gray Silty CLAY with red brown mottling, moist, stiff		12	2.9	108.	18.0	Qu=4900 psf
9									
10									
11			Light green brown Silty CLAY, moist, stiff						
12									
13									
14			Tan Silty medium SAND with subrounded granitic gravels, wet, medium dense						
15	1-3								
16	L		∇ Perched		31	4.5+	109.	17.7	
17			Dark gray SILT (weakly indurated), very damp, very stiff						
18									
19									
20									
21			Increase in hardness - hard						
22			Break-in-log						
25									
			Boring Terminated @ 25'						

Jacobs, Raas & Associates | FIGURE NO. 2 Log of Test Borings

LITHO WATSONVILLE PRESS

LOGGED BY SAS DATE DRILLED 6-21-88 BORING DIAMETER 6" BORING NO. 2

Depth, ft.	Sample No. and type	Symbol	SOIL DESCRIPTION	Unified Soil Classification	Blows/foot 350 ft-lbs.	Qu - t.s.f. Penetrometer	Dry Density p.c.f.	Moisture % dry wt.	MISC. LAB RESULTS
			FILL						
1			Brown Silty very fine SAND, dry, loose wood debris						
2			NATIVE-Dark brown Silty fine SAND, damp loose		5	4.5+	97.7	21.7	Qu=1090 psf
3	2-1 L		Dark gray Clayey SILT, wet, firm						
5	2-2 L		Brown very fine Sandy Clayey SILT, very moist, stiff (no sample recovered)		13				
6	2- L		Light green brown Clayey SILT with light gray mottling, moist, very stiff		20	4.4	20.8	103.	Ø=30° C=1650 psf
17			▽ Perched						
18			Green brown very fine Sandy SILT, wet, firm to stiff						
22			Dark gray SILT (weakly indurated), very damp, hard Break-in-log						
25			Boring Terminated @ 25'						

July 2[illegible], 1988

LOGGED BY SAS DATE DRILLED 7/5/88 BORING DIAMETER 6" BORING NO. 3

Depth, ft.	Sample No. and type	Symbol	SOIL DESCRIPTION	Unified Soil Classification	Blows/foot 350 ft-lbs.	Qu - t. s. f., Penetrometer	Dry Density p.c.f.	Moisture % dry wt.	MISC. LAB RESULTS
			FILL						
1			Brown Silty fine SAND with subrounded gravel and cobbles, dry, loose						
2	3-1		NATIVE-Gray brown Silty fine SAND damp, medium dense						
3	L		Gray Clayey SILT with red brown mottling very damp, hard		36	4.5+	112.7	14.4	Qu= 10,000 psf
4			Brown Silty fine SAND, damp, dense						
6			Red brown Silty CLAY with gray mottling moist, hard						
7–8	3-2 L				56	4.5+	108.4	19.0	Ø=26 C=2670 psf
18–19	3-3 L				48	2.7	102.5	21.0	
22–25			Dark brown fine Sandy CLAY, very moist, very stiff [illegible] ▽ W/L 7-5/88 Break-in-log						
			Boring Terminated @ 25'						

Jacobs, Raas & Associates

FIGURE NO. 4 Log of Test Borings

July 28, 1988

LOGGED BY SAS DATE DRILLED 7/5/88 BORING DIAMETER 6" BORING NO. 5

Depth, ft.	Sample No. and type	SOIL DESCRIPTION	Unified Soil Classification	Blows/foot 350 ft-lbs.	Qu - t. s. f. - Penetrometer	Dry Density p.c.f.	Moisture % dry wt.	MISC. LAB RESULTS
1		Light yellow-brown very fine Sandy SILT dry, stiff						
2	5-1							
3	L	orange-brown mottling very stiff		30	4.5+	105.	10.9	Qu=14780 psf
5	5-2	Yellow-brown very fine Sandy Clayey SILT with orange-brown mottling, very damp, hard						
6	L			60	4.5+	106.0	19.6	Qu=13390 psf
8		Brown very fine Sandy SILT (weakly indurated silt stone) damp, hard						
10	5-	very hard						
11	L			50/8"	2.9	94.5	13.4	
15		Gray brown very fine Sandy SILT with fine silt stone (moderately indurated silt stone) damp, very hard						
19		▽ W/L 7/5/88						
22–25		Break-in-log						
25		Boring Terminated @ 25'						

Jacobs, Raas & Associates

FIGURE NO. 6 Log of Test Borings

LITHO WATSONVILLE PRESS

LOGGED BY SAS DATE DRILLED 7/5/88 BORING DIAMETER 6" BORING NO. 6

Depth, ft.	Sample No. and type	Symbol	SOIL DESCRIPTION	Unified Soil Classification	Blows/foot 350 ft-lbs.	Qu - t. s. f. - Penetrometer	Dry Density p.c.f.	Moisture % dry wt.	MISC. LAB RESULTS
1 2	6-1 L		FILL Orange Silty fine SAND with fine granitic rock fragments and occasional gravels and cobbles, dry, moist dense		36	4.5+	110.6	15.1	
3 4 5 6 7	6-2 L		NATIVE Dark gray brown very fine Sandy SILT, damp, stiff very moist		8	4.5+	118.0	12.1	Qu= 6470 ps
8 9 10 11 12 13	6-3 L		▽ W/L 7/5/88 Yellow brown very fine Sandy Clayey SILT with orange-brown mottling, very moist, stiff Very stiff, light gray brown stockwork		16	2.8	98.7	25.3	
14 15			Blue gray Silt fine SAND with clay binder, very moist, loose to medium dense						
16 17 18 19 20 21 22 23 24			Boring Terminated @ 15'						

Jacobs, Raas & Associates **FIGURE NO.** 7 Log of Test Borings