Detterman, Mark, Env. Health

From: Costello, Timothy [Timothy.Costello@tetratech.com]
Sent: Friday, June 05, 2015 4:50 PM
To: Detterman, Mark, Env. Health
Cc: Kingsley Aduaka
Subject: Rockridge Shopping Center Site, 51st & Broadway, Oakland
Attachments: Geotechnical Investigation Report_Rockridge Shopping Ctr_11-10-14.pdf

Categories: Red Category

Mark,
The most recent Geotechnical Report for the Rockridge Shopping Center project site is attached; 15MB.

We have two more post-meeting items to provide – the soil profiling sample report, and the project schedule. The soil profiling report is under review by our PG and the report will be provided early next week. We are working on the schedule and will have that to you next week as well.

Feel free to contact me with any questions.

Thank you,
Tim.

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GEOTECHNICAL INVESTIGATION REPORT
SHOPS AT THE RIDGE
SAFEWAY STORE #3132
BROADWAY & PLEASANT VALLEY
5130 BROADWAY
OAKLAND, CALIFORNIA

PROJECT NO. 00136146.001A

NOVEMBER 10, 2014

KLEINFELDER
Bright People. Right Solutions.

November 10, 2014
Project No. 00136146.001A

Mr. Kingsley Aduaka
Property Development Centers
5918 Stoneridge Mall Road
Pleasanton, California 94588
kingsley.aduaka@pdcenters.com

Subject: **Geotechnical Investigation Report**
Shops at the Ridge, Safeway Store #3132
Broadway & Pleasant Valley
5130 Broadway, Oakland, California

Dear Mr. Aduaka:

Kleinfelder is pleased to submit this geotechnical investigation report for the Shops at the Ridge, including Safeway Store #3132, located at 5130 Broadway in Oakland, California. The enclosed report provides a description of the investigation performed previously and geotechnical recommendations for site grading and foundation design.

Kleinfelder previously completed a geotechnical investigation for this site, the results of which were presented in our report entitled *Geotechnical Investigation Safeway Replacement Store #3132, 5130 Broadway, Oakland, California*, project number 82546/GEO, dated September 14, 2007. We also performed an *Engineering Geologic Slope Reconnaissance and Preliminary Geotechnical Evaluation for the Existing Safeway Store No. 3132 and Adjacent Buildings Located at 5130 Broadway in Oakland, California*, project number 82546/slope, dated December 13, 2011. Since our prior reports were issued, the redevelopment plans for the shopping center have been modified and advanced, and this report includes updates to the geotechnical recommendations for the currently proposed project.

In summary, it is our opinion that the site is suitable for the proposed development and construction provided that the recommendations presented in our report are followed. The main geotechnical concern for the project site is the presence of old undocumented fills, rock rippability, potential instability of the existing cut slopes along the north side of the site, and the variability of the existing subsurface conditions including deep fills and shallow bedrock. Grading methods and foundation types including allowable bearing pressures will not be the same for each proposed building. Grading operations and foundation excavations will need to be closely monitored during construction to ensure the validity of the recommendations given in this report. Another concern on this site is rippability and excavation in those areas of shallow bedrock including the planned basement in Building G. The site development will also require cuts into the rock slope along the northern portion of the site, which will require a design-build contractor for implementation of slope retention measures and construction of retaining walls.

Groundwater was measured at a depth of about 18 feet near the southwest corner of the property, and is not anticipated to be encountered in the planned excavations except for possible isolated zones of perched water that might require localized dewatering during excavating. The soils and/or rock anticipated at the bottom of the footing excavations will be

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able to support the building loads on shallow footings. The floor slabs can be supported on grade over a prepared subgrade. These items, as well as our investigative methods, and our specific recommendations for design and construction, are contained in the following report.

It should be noted that the conclusions and recommendations presented in this report are based on limited subsurface exploration, and, as a result, variations between anticipated and actual soil conditions may be found in localized areas during construction. It is recommended that Kleinfelder be retained during construction to observe earthwork, installation of foundations and retaining walls, and slope retention systems to make changes deemed necessary to our recommendations due to encountered varying subsurface conditions.

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 at (925) 484-1700.

Sincerely,

KLEINFELDER, INC.

Don Adams, PE
Project Manager

Brian O'Neill, PE, GE
Principal Geotechnical Engineer

E. Morley John, PE
Project Engineer

Sadek M. Derrega, PG, CEG
Senior Principal Professional

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LIST OF ATTACHMENTS

1 INTRODUCTION

This report presents the results of a supplemental geotechnical investigation for the proposed redevelopment of the Property Development Centers (PDC) Rockridge Shopping Center, 51st and Broadway Safeway Store #3132 in Oakland, California. A Vicinity Map showing the site location is presented on Plate 1. The locations of our borings for this and previous investigations as well as our seismic refraction survey lines are shown on the Site Plan, Plate 2. This geotechnical investigation was performed for PDC.

Kleinfelder previously completed a preliminary geotechnical investigation on this site, the results of which were presented in our report entitled *"Geotechnical Investigation Safeway Replacement Store #3132"*, project number 82546/GEO, dated September 14, 2007. We also performed an *"Engineering Geologic Slope Reconnaissance and Preliminary Geotechnical Evaluation of the Existing Safeway Store No. 3132 and Adjacent Buildings"*, project number 82546/slope, dated December 13, 2011. Since our prior reports were issued, the redevelopment plans for the shopping center have changed and the results of the previous investigations have been referenced throughout this report, and updated where necessary.

The conclusions and recommendations presented in this report are based on the reconnaissance of the existing cut slopes to the north of the shopping center, the subsurface conditions encountered at the locations of our explorations, and the provisions and requirements outlined in the Limitations section of this report. The findings, conclusions and recommendations presented herein should not be extrapolated to other areas or be used for other projects without our prior review.

1.1 PROPOSED PROJECT

The existing Safeway store is located within the shopping center at the northeast corner of Broadway and Pleasant Valley Road in the Rockridge area of Oakland, California. To fit within the existing shopping center site, the existing commercial buildings were located at the northern (rear) portion of the property, and were configured with a jog. The two anchor tenants of the shopping center are a Safeway store (located in the western portion of the configuration) and a former CVS Pharmacy (located in the eastern portion).

It is understood that the six existing buildings, which were generally constructed in the 1960s, will be demolished and removed during several phases. Phase 1 will consist of redeveloping the eastern part of the site and includes:

- Demolition of existing Buildings 5 & 6;
- A new 64,000 square feet (sq. ft.), one-story Safeway store (Building A) will be constructed in the northeastern part of the site;
- Building C, a 3,100 sq. ft. one-story building on the eastern side of Phase 1, adjacent to the existing water reservoir;
- A bank in Building D, 8,800 sq. ft., two-story building in the southern portion of Phase 1. The southeast corner of building will be at the edge of the reservoir outflow pipe easement ;
- Building K, a one- and two-story retail building with 26,000 sq. ft. adjacent to the southwest corner of Building A.
- An upper level (roof) parking deck that will span over Buildings A and K;
- Low retaining walls less than about 10 feet high that may include planter walls;
- Truck loading dock pits and other non-building walls;
- Taller retaining walls and slope retention measures at the area of the existing cut slope along the northern boundary of the site; and
- Reconfiguration of the at-grade parking south of the new Safeway location (Building A).

The second phase (Phase 2) will redevelop the western portion of the shopping center after the Phase 1 work is completed and consists of the following as shown on the schematic plans;

- Demolition of existing Buildings 1 through 4. Building 1 (bank building) contains a basement which is deeper than the new Building G planned in this area and will therefore need to be backfilled;
- Building E, a one-story restaurant covering about 2,900 sq. ft.;
- Building F, a two-story retail space covering about 19,000 sq. ft.;
- Building G, two-stories of offices and a restaurant with a 24,000 sq. ft. basement;
- Building H covering about 12,000 sq. ft. with two stories of retail and restaurant space;
- Building J, three levels of parking over a one-story retail shop area covering about 32,500 sq. ft.; and,
- Street improvements along Pleasant Valley Avenue.

We have assumed the general site grades will remain about the same except for minor changes to facilitate drainage and truck unloading depressions and the planned cuts into the northern slope. Storm water (bio-retention) basins up to about 4 feet deep are planned in the surface parking lot planter areas and adjacent to drive aisles. The on-grade parking areas, driveways and truck access will be asphalt paved during the respective phases. No basements are planned except for the lower level of Building G and elevator pits. Extensive flatwork, including pavers, is anticipated.

The extent of offsite improvements that may become associated with the project and subject to needing design input are not known, and so not included in this report. However, we recognize the need for recommendations regarding the future traffic signals as well as the proposed 20-foot tall monument sign along the southern boundary of the property. The location of these features has not been determined, but general recommendations have been provided as part of this report based on the conditions encountered at the locations of our exploration.

The magnitude of building column loads for each building will vary. At the largest Building A, loads will vary and be in excess of 1,000 kips dead plus live loads for some column locations based on information provided to us by ARUP, the project structural engineers.

General site grading is anticipated to be minor, with cuts and fills generally on the order of 1 to 2 feet to establish finish grades. Significant cuts may be necessary along the northern slope. The approximate locations of the existing buildings are shown on our Site Plan, Plate 2.

If the project differs from that presented above, we should be contacted to review the applicability and potential modifications to our recommendations.

1.2 PURPOSE AND SCOPE OF SERVICES

The purpose of our current geotechnical investigation is to supplement our previous subsurface conditions, and amend our previous recommendations to fit the new planned site development plan. Our focus is to better assess the rippability of the varying surficial and subsurface materials and to help evaluate the potential for the existing old fill to impact construction of the new development. Our supplemental scope of work for this investigation as outlined in our June 21, 2013 proposal (File Number: 134295) included:

- Review of historic aerial photos and historic topographic base maps of the area prior to the construction of the existing shopping center (i.e. 1960s) and review historical information on the "Rockridge Quarry";
- Mapping of the rear (north) cut slope and recommendations of areas where rock fall protection should be considered per our December 13, 2011 letter;
- Site reconnaissance and mapping of the northern slopes by our Certified Engineering Geologists;
- Subsurface field exploration, including the drilling of twenty-two exploratory borings to depths of between about 1.5 ft. and 36.5 ft. below the ground surface;
- Geophysical surveys using 4 seismic lines in the area of Phase 1 to help evaluate the extent of the infill within the old quarry and excavatability for proposed construction;
- Laboratory testing including natural moisture content and unit weight, Atterberg limits, Resistance Value for pavement design, and strength tests; and,
- Engineering analysis and report preparation including design and construction recommendations for site preparation, grading and compaction, engineered fill, excavation, drainage, foundations, estimated settlements, seismic design parameters, concrete slabs-on-grade, pavements, retaining walls, and reuse of onsite materials.

This investigation specifically excludes the assessment of site environmental characteristics, particularly those involving hazardous substances.

2 GEOLOGY, FAULTING, AND SEISMICITY

Presented below is a discussion of the regional and local geology at the site, as well as a general discussion of the faulting and seismicity at the site and within the San Francisco Bay Area.

2.1 REGIONAL GEOLOGY

The site of the Rockridge shopping center project lies within the Coast Ranges geomorphic province, a series of discontinuous northwest-trending mountain ranges, ridges, and intervening valleys characterized by complex folding and faulting. The general geologic framework of the San Francisco Bay Area is illustrated in regional studies by Schlocker (1971), Helley et al. (1979), Wagner et al. (1991), Chin et al. (1993), Ellen and Wentworth (1995), Graymer et al. (1996), Graymer (2000), Graymer et al. (2006), and Witter et al. (2006).

Geologic and geomorphic structures within the San Francisco Bay Area are dominated by the San Andreas fault (SAF), a right-lateral strike-slip fault that extends from the Gulf of California in Mexico to Cape Mendocino on the Coast of Humboldt County in northern California. It forms a portion of the boundary between two independent tectonic plates on the surface of the earth. To the west of the SAF is the Pacific Plate, which moves north relative to the North American Plate, located east of the fault. In the San Francisco Bay Area, movement across this plate boundary is concentrated on the SAF; however, it is also distributed, to a lesser extent across a number of other faults that include the Hayward, Calaveras and Concord among others. Together, these faults are referred to as the SAF System. Movement along the SAF system has been ongoing for about the last 25 million years. The northwest trend of the faults within this fault system is largely responsible for the strong northwest structural orientation of geologic and geomorphic features in the San Francisco Bay Area.

Basement rocks west of the SAF are generally granitic, while to the east they consist of a chaotic mixture of highly deformed marine sedimentary, submarine volcanic and metamorphic rocks of the Franciscan Complex. Both are typically Jurassic to Cretaceous in age (200-65 million years old). Overlying the basement rocks are Cretaceous (about 145 to 65 million years old) marine, as well as Tertiary (about 65 to 1.8 million years old [USGS, 2006]) marine and non-marine sedimentary rocks with some continental volcanic rock. These Cretaceous and Tertiary rocks have been extensively folded and faulted as a result of late Tertiary and

Quaternary regional compressional forces. The inland valleys, as well as the structural depression within which the San Francisco Bay is located, are filled with unconsolidated to semi-consolidated continental deposits of Quaternary age (about the last 1.8 million years). Continental surficial deposits (alluvium, colluvium, and landslide deposits) consist of unconsolidated to semi-consolidated sand, silt, clay, and gravel while the Bay deposits typically consist of very soft organic-rich silt and clay (Bay Mud) or sand.

The portion of Graymer et al. (1996) geologic map that covers the site area is shown on Plate 3, Area Geologic Map.

2.2 SITE AND AREA GEOLOGY

Certified Engineering Geologists (CEGs) with our firm performed a reconnaissance of the slope areas bordering the shopping center along its northern side beyond the access perimeter paved roadway on September 26, 2013 and again on August 21 and 22, 2014 as part of our current scope. The main emphasis of our 2013 engineering geologic reconnaissance of the slope area was to assess the potential for erosion and/or slope instability and to evaluate whether conditions along the noted northern slope have changed significantly since our initial slope reconnaissance and mapping performed on November 16, 2011. The main emphasis of our 2014 engineering geologic reconnaissance of the slope areas was to assess the feasibility of further cutting portions of the northern slope, obtain geologic structure data pertaining to bedding and fractures in the shale/sandstone and tonalite bedrock, evaluate the constructability of retaining walls, and other slope retention measures under consideration by the project team along the east and west ridges, and to review and revise prior mapping information due to any observed changed conditions at the site.

The results of our initial 2011 slope reconnaissance were presented in our letter entitled *Engineering Geologic Slope Reconnaissance and Preliminary Geotechnical Evaluation for the Existing Safeway Store No. 3123 and Adjacent Buildings*, project number 82546/slope, dated December 13, 2011.

Our CEGs observed the slope to vary in height from less than about 5 feet along the western and eastern ends to as high as about 70 feet near the west side of the existing CVS Pharmacy building. The slope measured about 1,400 lineal feet in length and there are existing low wooden retaining walls measuring about 4 feet in height along the toe of the slope at several locations. The noted wooden walls are made up of stacked railroad ties that are situated immediately behind a chain link fence line that extends along the entire length of the toe of the slope.

Our CEGs observations were mapped on a topographic base map prepared by BKF Engineers and included herein as Plate 4, Northern Slope Geologic Map. No subsurface explorations or slope stability analyses were performed to assess the stability of the subject northern, south-facing slope as part of our previous or current assessments.

As is indicated by the topographic base map of the subject slope, some portions are relatively steep to near vertical and are relatively high. These portions of the slope are generally underlain by the intrusive igneous diorite bedrock, which is chiefly composed of the feldspar mineral andesine. This variety of diorite is known as tonalite since it contains more than about 10 percent of the mineral quartz (silica). The tonalite bedrock belongs to the Cretaceous age Franciscan Complex and was observed to be fresh along cuts made during the construction of the shopping center and during previous quarrying activities. It is moderately strong to strong, and moderately fractured to highly-fractured. Areas where fresh and strong tonalite bedrock is exposed along the slope face have been colored purple and are delineated as “Rocky Tonalite Bedrock Areas” on Plate 4.

Along some areas of the exposed slopes, our CEGs observed the presence of highly weathered to disintegrated Cretaceous shale and sandstone bedrock near the top of the slope generally overlying the Cretaceous diorite volcanic rock and along areas where topographic hollows and swales are blanketed with shale and sandstone debris. The weak to very weak shale is interbedded with medium strong to strong sandstone. These in-place shale and sandstone areas have been colored blue, and are delineated as “Shale/Sandstone” on Plate 4.

Blocks of bedrock may become dislodged along fracture planes and topple down slope, especially during moderate to strong seismic shaking on nearby faults like the Hayward fault. The feldspar mineral in the tonalite is prone to physical weathering and chemical decomposition in place, becoming clay where it is exposed, at the ground surface, to weathering elements and agents for a prolonged periods of time such as along the top of the slope. Accordingly, slope areas where fresh bedrock is present are veneered along their top by residual sandy clay soils that have developed through the chemical weathering of the underlying bedrock. Such surficial residual clay zones could measure up to about 10 feet in thickness.

Where the bedrock has been highly weathered into a sandy clay matrix supporting sand to cobble-sized bedrock fragments along sheared and/or highly and closely fractured zones in the bedrock, the slope gradient is less steep and the slope face is veneered with sloughing residual soil, colluvial sediment, and intact bedrock blocks. Our CEGs observed several areas behind the existing CVS Pharmacy where the slope is covered with loose sediment containing bedrock fragment float, which we labeled “Loose and Unstable Deposits” and colored green on Plate 4. These deposits have a moderate to high potential for failure if they become wet or are subjected to a moderate to strong seismic shaking.

In addition to the noted two types of materials we described above, our CEGs observed several areas where the tonalite bedrock is present at shallow depths and is veneered with a relatively thin layer of colluvial soils. These areas are shown in yellow on Plate 4 and labeled as “Thin Colluvial Veneer”. These areas include colluvium collection areas along the toe and top of the slope and are generally present along the eastern and western portions of the overall slope. These areas are not considered a significant source of loose material and their potential for failure and adversely impacting the development is considered low.

As part of their on-going Seismic Hazard Zonation Program, the California Geological Survey (CGS, 2003 a and b) has mapped the area of the site, and their Landslide Inventory Maps (Plates 2.1) do not show landslide deposits along the subject slope.

Although our CEGs did not observe signs indicative of recent and fresh gross slope instabilities along these areas, historical rock topples and slope failures consisting of loose sediment and rock debris could have occurred at the site but have been disturbed and/or masked by quarrying activities. The bedrock and “Loose and Unstable Deposits” zones remain a material source for rockfalls and mudflows, especially if significant rainstorms and/or earthquake shaking occur, or if surface water runoff from upslope areas is allowed to uncontrollably cascade down the slope.

2.3 FAULTING AND SEISMICITY

The site area and the entire San Francisco Bay Area are located within an active seismic region due to the presence of several active earthquake faults. No active faults cross the site area and the site is not located within an Alquist-Priolo Earthquake Fault Zone associated with active faults and where detailed fault investigations are required to characterize the presence, activity, and potential for fault-related ground surface rupture.

The site is located less than 1.5 miles to the southwest of the active Hayward fault zone, about 11.5 miles to the west of the Calaveras fault, and more than 17 miles to the northeast of the San Andreas fault zone.

3 SITE INVESTIGATION

3.1 SITE DESCRIPTION

The existing approximately 15-acre shopping center site is located northeast of the corner of Broadway and Pleasant Valley Road in the Rockridge area of Oakland, California. The shopping center was constructed in the 1960s at what appears to have been the western portion of a previous rock quarry. The site is relatively level and has been created by cuts in the northern area, and fill in the western, eastern and southern areas of the site.

This site is bounded by Pleasant Valley Road to the southwest and Broadway to the west. An existing cut slope is visible at the north (rear) of the property measuring up to about 70 feet in height. Commercial and residential properties are located to the north of the site, on top of the existing cut slope. A pond is located to the east and southeast of the site in a portion of the former quarry, beyond which are more residential properties.

The inclination of the cut slope at the north (rear) of the property varies, with some areas near vertical. Previous information regarding this slope was included in our letter report dated December 13, 2011. In general, the slope is mostly underlain by coherent, strong, and widely-fractured bedrock consisting of tonalite. Several shale/sandstone zones were observed to be underlain by loose and unstable rock and soil debris. The fractured in-place tonalite bedrock poses rockfall hazard while the shale and sandstone debris-filled topographic swales that also house relatively large dislodged tonalite blocks represent a mudflow and rock debris flow source area that may mobilize if wetted or subjected to strong seismic shaking.

To the east and southeast of the shopping center site is a pond that has been created as a result of the quarry operations. The groundwater in the pond is about 20 feet below the shopping center grade, and the top of the bank is about 30 feet from the former CVS Pharmacy building; asphalt parking and driveway is present between the building and the bank of the pond. The bank of the pond exposes tonalite rock, and is nearly vertical. On the opposite side of the pond to the east is a steep cut slope (nearly vertical) into rock measuring about 80 to 100 feet high. This cut slope decreases in height towards the south. It is unknown how, and to what extent the height of the water in the pond is managed.

The existing shopping center buildings are all one-story in height, with sidewalks and asphalt parking covering the southern half of the site with some landscaping. An asphalt driveway and loading area is located on the northern portion of the site, between the buildings and the cut slope. A separate building that functions as a Bank of America is located at the southwest corner of the shopping center. The pad grade at the location of the bank building is about 5 feet higher than street grade (both Broadway and Pleasant Valley Road); which may be the result of fill placement in this area. The limits of the existing buildings and the proposed buildings are shown on the Site Plan, Plate 2.

3.2 FINDINGS FROM REVIEW OF HISTORIC AERIAL PHOTOS AND MAPS

The quarry was originally owned by the Oakland Paving Company. It was mined for Franciscan quartz Diorite (tonalite) which was crushed and used for roadway and railroad bed material. The quarry was primarily operational in the late 1800s and early 1900s, prior to the earliest aerial photo reviewed from 1930.

Ponding in the northeastern portion of the site occurred in the late 1950s (1957/1958) as is evident in both aerial photos and a 1959 topographic map. Based on aerial photo review, portions of the ponded areas were essentially filled in by 1959. Based on topographic map and photo review, the existing shopping center was constructed between 1963 and 1968, and Pleasant Valley Road was constructed between 1953 and 1957. The school campus to the north was primarily constructed prior to 1930.

Topographic contours at the intersection of Broadway and Pleasant Valley are essentially unchanged. It appears no significant grading of the intersection occurred in the 1900s.

The western portion of the site (near the existing Boston Market and Bank of America) was a staging or parking area from about 1950 to 1960.

The ridge on the western edge of the main quarry would have extended through the eastern half of the existing Safeway building. This ridge was slowly reduced between 1930 and 1963 based on the aerial photo evidence. It stands to reason that the subgrade beneath the eastern half of the Safeway building would be relatively hard rock, while the footprint of the existing CVS Pharmacy and parking lot are likely to contain fill as they are within the old quarry limits. The CVS Pharmacy building in particular, is constructed in the approximate location of the former main quarry pit, and as such is underlain by fill material.

No significant grading or excavation of the western half of the site (from the existing Safeway building east to Broadway) was evident in review of topographic maps or aerial photos.

3.3 SUBSURFACE EXPLORATION

The subsurface exploration for this study included the drilling of seven borings on September 3, 2013, and the drilling of an additional 15 borings between September 9 and September 12, 2014. The 2013 borings were evenly distributed throughout the building, parking and outparcel areas of the site, while the additional borings drilled in 2014 were focused in areas along the northern margin of the site near the base of the existing steep slopes. Logs of the subsurface conditions, as encountered in the borings, are presented on the Boring Logs, Plates A-4 through A-25, in Appendix A. The borings were located approximately as shown on the Site Plan, Plate 2. A key to soil and rock symbols and terms used with this report is presented on Plates A-1 to A-3, in Appendix A.

The locations of the borings were estimated by our geologist based on rough measurements from existing features at the site. Prior to the start of our field investigation, Underground Services Alert (USA) was contacted to locate utilities within the pertinent street rights-of-way. In addition, we retained a private utility locator to confirm that our exploratory locations were not in conflict with known underground utilities. As required by local ordinance, a drilling permit was obtained from the Alameda County Public Works Department.

Exploration Geoservices of San Jose, California was subcontracted to provide drilling services in September 2013, and GREGG Drilling of Martinez, California and Woodward Drilling of Rio Vista, California were subcontracted to provide drilling services in September 2014. The soil/rock borings were drilled using a truck-mounted B-53 rig (Exploration Geoservices and GREGG Drilling) or a BK-81 rig (Woodward) utilizing eight inch diameter hollow stem augers to depths of between 1½ feet and 36½ feet below ground surface level. Disturbed and relatively undisturbed samples were taken at the direction of our engineer during drilling. Relatively disturbed samples of the subsurface materials were obtained using a Standard Penetration Split Spoon Sampler (SPT) with a 1⅜inch inside diameter (I.D.) and a 2 inch outside diameter (O.D.), and a California Sampler with a 2½ inch inside diameter (I.D.) and a 3 inch outside diameter (O.D.). The samplers were driven 18 inches using a 140 pound hammer falling 30 inches, and blow counts for successive 6 inch penetration intervals or to refusal were recorded.

After the sampler was withdrawn from the borehole, the samples were removed, sealed to reduce moisture loss, labeled, and returned to our laboratory. Prior to sealing the samples, strength characteristics of the cohesive soil samples recovered were evaluated using a hand-held pocket penetrometer. The results of these tests are shown adjacent to the samples on the boring logs.

Soil classifications made in the field from auger cuttings and samples were re-evaluated in the laboratory for further examination and testing. The soils were classified in general accordance with the Unified Soil Classification System (USCS) presented on Plate A-1, Graphics Key. Soil and rock description definitions are presented on Plate A-2, Soil Description Key and Plate A-3 Rock Description Key. Sample classifications, blow counts recorded during sampling, and other related information were recorded on the soil boring logs. The boring logs for borings K-1 to K-7 are presented on Plates A-4 to A-10, and the boring logs for borings K100 to K114 are presented on Plates A-11 to A-25, in Appendix A. Upon completion of drilling and sampling, the borings were backfilled with cement grout.

3.4 SEISMIC REFRACTION SURVEY

Seismic refraction surveys were completed in the eastern half of the site by Advanced Geological Services, Inc., during the night of September 11 and 12, 2013. Four seismic lines were completed during the survey. Generally, the results indicate that the bedrock depth ranges from 0 feet to 60 feet below ground surface, with the top of bedrock occurring mostly between 20 feet and 40 feet below ground surface. A complete report of the results of the seismic refraction survey is included in Appendix D.

4 LABORATORY TESTING

Laboratory testing was performed on selected soil and rock samples collected from the borings to evaluate their engineering characteristics. The following laboratory tests were used to develop the design geotechnical parameters:

- Atterberg Limits;
- Percent Fines (-#200 Sieve);
- Moisture Content and Unit Weight;
- Resistance Value;
- Unconsolidated Undrained Triaxial Compression; and,
- Point Load Strength Index (Rock).

Some of the laboratory test results are presented on the boring logs in Appendix A. Graphic presentation of the results of the Atterberg Limits, R-Value, Unconsolidated Undrained Triaxial Compression, and Point Load Strength Index testing results are presented in Appendix B. Laboratory results for the samples collected in 2013 (borings K-1 through K-7) are presented on plates B-1 through B-6, and results for samples collected in 2014 (borings K100 through K114) are presented on plates B-7 through B-21. In addition, two samples were submitted to AP Engineering & Testing of Pomona, California and one sample was submitted to CERCO Analytical of Concord, California for brief corrosion analyses. The results of the corrosion testing are presented in Appendix C.

5 SURFACE AND SUBSURFACE CONDITIONS

Presented below is a general description of surface and subsurface conditions encountered at the site. For a more detailed description of the materials encountered in the borings, refer to the Boring Logs in Appendix A. It should be noted that subsurface conditions can deviate from those conditions encountered at the boring locations. In general, this site is underlain with highly variable material. Kleinfelder should be present during construction to confirm that our recommendations presented herein are appropriate based on actual conditions encountered during grading and foundation construction and to recommend adjustments as necessary.

5.1 SUBSURFACE CONDITIONS

The subsurface conditions across the site vary; from shallow bedrock encountered directly below the pavement materials in borings K-5 and K-6, to deep fill soils encountered to the limits of the investigation in borings K-1 through K-4, K-7, K103, K106, K110, and K113. In borings K100 through K114, drilled along the base of the existing rock slopes at the north of the site, bedrock was generally encountered at depths of 1 to 15 feet. Based on the results of our previous investigations on this site, our recent review of historic air photos, the results of the recent borings and the results of the seismic refraction survey, we believe the subsurface conditions can be summarized into areas as presented in Table 1 below. However, it must be noted that subsurface conditions can deviate from those conditions encountered during this investigation.

Table 1
Summary of Inferred Subsurface Conditions

General Region of the Site	Existing Site Features	Proposed Buildings	Brief Summary of Subsurface Conditions
Western	Buildings 1, 2, 3 & 4, driveways and parking	Buildings E, F, G, H & J, driveways and parking	Shallow bedrock, localized shallow fill
Southern	Driveways and parking	Buildings C & D, driveways and parking	Deep fill, with depth to bedrock decreasing towards the south
Eastern	Buildings 5 & 6, driveways and parking	Buildings A, B & K, driveways and parking	Deep fill, with localized areas of very shallow bedrock along the eastern boundary (near the pond)

Based on our most recent field investigation, the existing asphalt pavement at the site measured approximately 4 to 5 inches thick over about 2 to 10 inches of aggregate base material.

Underlying the pavement in boring K-6 and beneath a thin layer of clayey gravel in boring K-5, highly weathered sandstone bedrock was encountered. In K-6 the SPT results in the bedrock were 50 blows for 2 inches penetration at 2½ feet depth, and 50 blows for 3 inches penetration at auger refusal at 5 feet depth.

Underlying the pavement in borings K-1 to K-4, K-7, K103, K106, K110, and K113, variable fill was encountered to bore termination depths of between 14½ feet and 36½ feet below ground surface (bgs). The fill generally comprised gravelly lean clays and clayey sands and gravels. The clays were generally firm to hard, of low to medium plasticity and brown, dark brown or grey in color. The sand and gravel layers were generally medium dense to dense, sub-rounded to sub-angular and ranged from fine to coarse.

Groundwater was encountered in boring K-2 at approximately 18 feet bgs. Free groundwater was not encountered in any of the other borings during this investigation. In our previous investigation in 2007, groundwater was encountered in B-1 at approximately 20 feet bgs. Seasonal fluctuations in the groundwater level may occur due to variations in rainfall, temperature, water levels in the adjacent pond, pumping from local wells, and possibly changes as the result of other factors that were not evident at the time of our investigation.

Soil, rock and groundwater conditions can deviate from those conditions encountered at the boring locations. Should this be revealed during construction, Kleinfelder should be notified immediately for possible revisions to the recommendations that follow.

As described in Section 3.4, seismic refraction geophysical surveys were performed for purposes of subsurface profile data collection and rock rippability (excavatability) evaluation of the bedrock present at the site. Generally, the results indicate that the bedrock depth ranges from 0 feet to 60 feet below ground surface (bgs), with the top of bedrock occurring mostly between 20 and 40 feet bgs. However, the depth to bedrock is shallower in some areas (i.e. west side of the site) and highly variable, especially considering that a large portion of the site was a former rock quarry. A complete report of the results and interpretations of the seismic refraction survey is included in Appendix D. The locations of the seismic refraction lines were chosen to obtain relative depth to bedrock and rock rippability in the study area, and subject to available space constraints at the active shopping center site.

5.2 EXISTING SLOPE CONDITIONS

As discussed in Section 2.2, a site reconnaissance was completed by our CEGs along the existing slopes along the northern portion of the project site. As part of our reconnaissance, geological structural mapping was completed to collect data related to the overall rock mass conditions and the discontinuities present in the rock mass. We collected information in accordance with *The Rock Slopes Reference Manual* (FHWA A-HI-99-007, 1998). Discontinuity information that we collected includes the following:

- Location of the discontinuity
- Type of discontinuity
- Discontinuity orientation (strike direction and dip magnitude and direction)
- Bedding attitudes (strike and dip)

Rock mass information that we collected includes the following:

- Locality type
- Slope length
- Slope height
- Rock mass color
- Rock mass grain size
- Field estimates of intact rock uniaxial compressive strength
- Rock mass fabric
- Rock mass block size
- Rock mass state of weathering
- Number of discontinuity sets

As part of our assessment of the rock mass and the existing slope conditions, we completed geomechanical rock mass classifications. These classifications are a design tool and are estimated using the field data. Two of the more widely accepted classifications systems are the Rock Mass Rating System (RMR) by Bieniawski (1989) and the Geological Strength Index (GSI) from Hoek (1997).

The base RMR, also referred to as the geomechanics classification system, is based on the algebraic sum of five rock mass property ratings, namely:

- Rock quality designation (RQD)

- Strength of intact rock material
- Spacing of discontinuities
- Condition of discontinuities
- Groundwater conditions

To estimate the RMR, we compared field data to published tables by Bieniawski (1989). Values for RMR can range from zero to 100. From the ratings, rock class and corresponding descriptions and engineering properties are assigned to the overall rock mass.

Bieniawski's (1989) RMR classification can be related to Hoek's (1997) GSI rating. The GSI rating can also be estimated directly from the information that we collected during our field mapping. Table 2 summarizes the geomechanical rock mass information collected during the field mapping.

Table 2
Summary of Rock Mass Characteristics Collected During Field Reconnaissance

Outcrop	Lithology	Rock Strength (MPa)	Rock Strength (psi)	RQD	Base RMR	GSI (RMR-5)	GSI Field
B-B'	Tonalite	62.5	9062.5	62	66	61	55
C-C'	Tonalite	62.5	9062.5	62	57	52	55
E-E'	Tonalite	62.5	9062.5	52	58	53	50
F-F'	Tonalite	62.5	9062.5	36	63	58	50
F-F'	Shale	15.0	2175	36	NA	NA	35

5.2.1 Stereonets and Markland Analyses

Using the discontinuity data we collected at existing rock outcrops, we constructed pole plots on equal area stereonets. A pole represents an individual discontinuity. Stereonets provide a two-dimensional representation of the three-dimensional discontinuity data. Next we outlined pole clusters or populations of discontinuities. Great circles were then plotted for each major pole cluster. The pole clusters represent major discontinuity sets, which generally strike in a similar direction. We plotted the orientations of the discontinuities on stereonets using the computer programs Dips V5.0 by Rocscience®.

All the discontinuity data collected at the site was combined to develop stereonets. We estimated 6 discontinuity sets including bedding from our mapping. Dip angles range from 7 to 89 degrees for average dip values for each set.

Where the stability of a rock cut is controlled by the structure of the rock mass, a Markland analysis was used to estimate the kinematic potential for rock blocks to fail out of the existing or proposed slopes. The information required to perform an analysis are the design slope dip and dip direction, the orientation of the discontinuities within the rock mass, and the friction angle of the discontinuities. A kinematically potential wedge failure is identified when a point defining the line of intersection of two planes falls within the area included between the great circle defining the slope face and a circle defined by the angle of friction. A planar failure is a specialized form of a wedge failure that follows the same criteria above and also must fall within ± 20 degrees of the dip direction of the slope face. We plotted the orientations of the discontinuities on stereonets using the computer programs Dips V5.0 and ROCKPACK III by C. F. Watts (2001). We plotted both poles and dip vectors. The poles tend to accentuate the orientation of steeply dipping discontinuities while the dip vectors lend themselves to performing Markland analyses.

The Markland analysis does not consider a cohesion intercept when modeling the strength of discontinuities. This method also assumes that the discontinuities are continuous and through going with no "bridging" within the discontinuity. The effect of "bridging" would allow a tensional component (or cohesion intercept) of discontinuity strength. The Markland Analysis assumes that the factor of safety of individual rock blocks may be estimated as follows. When the dip of a discontinuity or the plunge of the line of intersection is greater than the friction angle, the factor of safety is less than 1.0. When the dip of a discontinuity or the plunge of the line of intersection is less than the friction angle, the factor of safety is greater than 1.0. In either case, the dip or plunge has to be less than the dip of the slope face, or the structure will not daylight the slope.

We assumed that the tonalite rock had a discontinuity friction angle of 30 degrees and the shale has a discontinuity friction angle of 25 degrees based on the geomechanical information that we collected in the field, experience with similar rock types and guidance from the Rock Slopes Reference Manual (FHWA, 1998). Based on the results of our analyses, there is a potential for planar and wedge-type failures out of the existing and proposed cut slopes at inclinations ranging from approximately 30 to 70 degrees.

6 DISCUSSION AND CONCLUSIONS

The significant geotechnical issue for the proposed development is the presence of old undocumented fills, rock rippability, potential instability of the existing cut slopes along the north side of the site, and the variability of the existing subsurface conditions including deep fills and shallow bedrock. Grading methods and foundation types including bearing pressures will not be the same for each proposed building. Grading operations and foundation excavations will need to be closely monitored during construction to ensure the validity of the recommendations given in this report.

Another concern on this site is excavation and rock rippability in those areas of shallow bedrock including the planned basement in Building G. While blasting is not likely to be required, hard digging with a backhoe or excavator should be anticipated, along with the possible use of hoerams, pneumatic hammers or other measures. Further discussion of the anticipated rippability of the site is included in Section 7.6.

The new buildings can be supported on shallow footings bearing in either the native bedrock materials, existing fill, or in new engineered fill, provided the foundation excavations are inspected. In the event that unsuitable soil is encountered, Kleinfelder will need to provide recommendations for mitigation, likely to include limited over-excavation to a firm bottom with the resulting excavation backfilled with engineered fill or lean mix concrete.

Care should be taken during demolition to remove all debris, concrete footings, etc., and properly moisture condition and compact any backfill required as a result of demolition (e.g. removal of old footings).

It is important to note that it was not within our scope to work to assess or evaluate the stability of the pond bank to the east of the buildings and shopping center, the impact of the possible rise of the groundwater in pond, or possible impact of waves generated from rockfall originating from the surrounding pond slopes. Consideration should be given to evaluating these issues to define and assess their potential adverse impact to the proposed development. However, detailed topographic contour mapping and geotechnical data along the base of the pond would be required.

Design recommendations for foundations, concrete floor slabs, demolition, exterior flatwork, earthwork, low retaining walls, site drainage, and pavements as well as a discussion of seismic design and corrosion considerations for this project, are presented in Section 7 "Recommendations" of this report.

7 RECOMMENDATIONS

Presented below are recommendations for foundations, concrete floor slabs, demolition, exterior flatwork, earthwork, retaining walls, retention and stabilization of high cut slopes, site drainage, and pavements as well as a discussion of seismic design and corrosion considerations for this project.

We recommend that Kleinfelder be retained to provide observation and testing services during site earthwork and foundation construction. This will allow us the opportunity to compare conditions exposed during construction with those interpreted in our investigation and, if necessary, to expedite supplemental recommendations if warranted by the exposed subsurface conditions.

7.1 FOUNDATIONS

7.1.1 Building Foundations

Based on our investigation, the anticipated loads for the proposed building can be supported by continuous and isolated footings bearing on the native soils and bedrock as encountered in the western area of the site, or on existing fill as encountered in the eastern and southern areas of the site. Foundations can also be supported on newly placed engineered fill in any area of the site. All footings need to extend to a minimum depth of 24 inches below the bottom of the floor slab for interior footings or below adjacent finished grade for exterior footings. The recommended allowable soil bearing pressures for each area of the site are given in Table 3 below. For interior and exterior continuous footings, a minimum width of 18 inches is recommended. Isolated interior and exterior footings should measure a minimum of 18 inches by 18 inches.

As much as is practicable, all foundations of each structure should be founded in similar material (i.e. all in fill (new or existing) or all in bedrock) to reduce the risk of differential settlement between adjacent footings which could damage the new structures.

Table 3
Foundation Bearing Capacity Recommendations

General Region of the Site	Existing Site Features	Proposed Buildings	Founding Material*	Allowable Bearing Pressure (psf)**
Western	Buildings 1, 2, 3 & 4, and pavement	Buildings E, F, G, H & J	Existing Gravel Fill, Bedrock or New Engineered Fill[1]	4,000
Southern	Pavement	Buildings C & D	Fill (existing or new)[2]	3,000
Eastern	Buildings 5 & 6, and pavement	Buildings A & K	Fill (existing or new)[3]	4,000
Northern	Buildings 5 & 6, and pavement	Building K	Bedrock	15,000

* Anticipated site preparation (generalized and excluding demolition related grading requirements)
1 Hard rock excavation and minor fill placement
2 Engineered fill placement including that for soft spots encountered in the existing fill
3 Engineered fill placement including that for soft spots encountered in the existing fill as well as hard rock excavation
** Pounds per square foot, dead plus live load, includes a factor of safety

Allowable soil bearing pressures may be increased by one-third for transient loads such as wind and seismic loads. Total estimated settlement of an individual spread foundation will vary depending on the plan dimensions of the foundation and the actual load supported. Based on foundation dimensions and loads we were provided for Buildings A, K, C, and D, estimated total settlement of footings is expected to typically range up to 1 to 1½-inch for Buildings A and K for heavily loaded square footings for basecase 4,000 psf bearing pressure, and ¾ to 1-inch if bearing value was limited to 2,000 psf. The estimated total settlement of footings is expected to typically range up to ¾ inch for Buildings C and D. For footings founded on similar subgrade materials at all building locations, the estimated magnitude of differential settlements between adjacent footings are expected to be up to ½ of the magnitudes provided for total settlement. For footings within Building K that bear entirely on competent tonalite bedrock, bearing pressure may be increased to 15,000 psf. Estimated total settlements of foundations on rock is approximately ¼ inch; differential settlements between adjacent foundations on rock and soil may be up to the estimated total settlement.

Where footings are located adjacent to below-grade structures or near major underground utilities, the footings should extend below a 2:1 (horizontal to vertical) plane projected upward from the structure footing or bottom of the underground utility to avoid surcharging the below grade structure and underground utility with building loads. Also, where utilities cross the perimeter footings line, the trench backfill should consist of a vertical barrier of impervious type of material or lean concrete, as explained in the Section 7.6 “Earthwork” of this report. In addition, where utilities cross through or under exterior footings, flexible waterproof caulking should be provided between the sleeve and the pipe. Utility plans should be reviewed by Kleinfelder prior to trenching for conformance to these requirements.

Concrete for footings should be placed neat against native soil, bedrock, newly placed engineered fill or suitable existing fill. It is critical that footing excavations not be allowed to dry before placing concrete. If shrinkage cracks appear in the footing excavations, the excavations should be thoroughly moistened to close all cracks prior to concrete placement. The footing excavations should be monitored by a representative of Kleinfelder for compliance with appropriate moisture control and to confirm the adequacy of the bearing materials. In the event that unsuitable existing fill is encountered, Kleinfelder will need to provide recommendations for mitigation, likely to include over-excavation to a firm bottom with the resulting excavation backfilled with engineered fill or lean mix concrete.

Lateral loads may be resisted by a combination of friction between the foundation bottoms and the supporting subgrade, and by passive resistance acting against the vertical faces of the foundations, including grade beams. Allowable friction coefficients and an allowable equivalent fluid pressure for passive resistance for each area of the site are presented in Table 4 below. Passive pressure should be neglected in the upper one foot unless the adjacent surface is confined by paving or flatwork. The friction coefficient and passive resistance may be used concurrently, and the passive resistance can be increased by one-third for wind and/or seismic loading.

Table 4
Lateral Resistance Recommendations

General Region of the Site	Proposed Buildings	Founding Material	Allowable Friction Coefficient	Allowable Equivalent Fluid Pressure (pcf)
Western	Buildings E, F, G, H & J	Existing Gravel Fill, Bedrock or New Engineered Fill	0.5	300
Southern	Buildings C & D	Fill (existing or new)	0.25	200
Eastern	Buildings A & K	Fill (existing or new)	0.25	200

7.1.2 Tie-Down Ground Anchors

The structural engineer's foundation designs at Buildings C, D, and K may include vertical tie-down ground anchors for uplift resistance. The final layout and design tension loads were not yet known at the time this report was prepared. Tie-downs are expected to be small-diameter (typically <18-inch) drilled and grouted non-displacement elements similar to drilled piers that are typically reinforced with a steel central bar. Geotechnical evaluation assumes that construction will include grouting the drilled holes by gravity, and not under pressure. Tie-downs can withstand transient axial loads in shaft skin resistance (geotechnical capacity). Tie-downs should be designed to gain their support in shaft skin resistance, have a minimum diameter of 12-inches, a minimum length of 25 feet below the bottom of concrete 'caps', and have a minimum spacing (center to center) of 3 diameters.

For design, we recommend that tie-downs with embedment in soil layers use an ultimate bond strength of 1,200 psf, and tie-downs with embedment in bedrock use ultimate bond strength of 20,000 psf (for the vertical interval in rock). At Buildings C & D, soil layers extend to depth of approximately 45 feet, underlain by the bedrock. At Building K, soil layers are interpreted to extend to depths of approximately 15 to 25 feet (or more), underlain by bedrock. These recommended values assume primary grout placed under gravity head, and no secondary pressure grouting. For allowable stress design, a safety factor of at least 2.0 should be applied to the grout-to-ground ultimate bond strengths. These estimated grout-to-ground bond ultimate strengths are based on interpretation of subsurface conditions, and past experience using Federal Highway Administration guidelines.

An unbonded or "free length" of the tie-downs should be considered and extend to a minimum depth of about 10 feet below the bottom of foundations. However, the design-build tie-down contractor may consider the addition of steel casing in the unbonded zone to increase the stiffness and lateral load resistance, if deemed necessary. The actual unbonded and bonded lengths should be developed by the design-build contractor in consultation with the structural engineer, who will provide the final structural load demands. Site specific load testing should be performed in advance of production tie-down installation to verify (or modify) the design values recommended. We anticipate that the ultimate design tensile load demands can be achieved with tie-downs of a reasonable and constructible embedment depth of less than about 30 to 50 feet below the bottom of foundations. The tie-downs should be provided by a design-build type foundation contractor who will be responsible for the final design, tip elevations and grouting methods required to provide the structural design load demands, maximum deflection tolerance, axial stiffness, cone pullout resistance, and acceptance criteria. The structural material details and type of grouting should be designed to meet the required acceptance criteria.

7.1.3 Traffic Signal, Pylon Sign and Light Pole Foundations

For traffic signals, pylon sign structures or light poles, cast-in-drilled-hole (CIDH) pile (i.e. drilled pier) foundation systems may be used. These piles can be designed to derive support from skin friction developed along the shaft of the pile, and lateral resistance from passive soil pressure against the side of the pile.

The ultimate 'unit' side resistance capacity for drilled piers should be taken as 1,000 psf in native soils, newly placed engineered fill, suitable existing fill or bedrock. For the structural designer's analysis of the axial capacity of drilled piers, skin friction of the upper 2 feet of soil should be neglected. A factor of safety of at least 2 should be applied. A one-third increase in the allowable capacity may be used for consideration of transient loads such as wind or seismic. The foundation weight may be added when evaluating uplift resistance. Drilled pier embedment length may be controlled by various load cases in either axial loading (compression or uplift) or lateral loading, however a minimum pier depth of 10 feet below final ground surface is recommended for embedment in soil (or rock) layers with competent bearing characteristics.

End bearing should be neglected due to strain incompatibility issues and because adequate bottom clean-out will be unlikely during installation of the shafts, thereby preventing visual inspection of the bottoms of shaft excavations. For a shaft constructed in accordance with the recommendations presented in this report and under slurry, with no verifiable bottom clean-out, the vertical movement that would be required to mobilize end-bearing resistance is expected to be on the order of 2 inches or more. The long-term settlement may be much greater where the quality of the drilled hole is marginal to poor. Additional discussion of construction considerations is provided below.

For drilled shafts designed and constructed in accordance with the recommendations presented in this report, total settlement of each drilled shaft is expected to be less than about ¾ inch, with differential settlement between adjacent supports of up to about ½ inch. These values include elastic compression of the shaft under design loads. The majority of the settlement should occur during and shortly after application of the structure loads.

Pier foundation resistance to lateral loads will be provided by passive resistance of the soil against shafts, pier caps, and grade beams (if present) and by the bending stiffness of the pier shafts. The lateral resistance of a drilled pier is a function of the surrounding soil strength and stiffness, size and stiffness of the pier, pier top connection, and induced moments and forces at the top of the pier. For pier caps and grade beams, the allowable passive pressure available in undisturbed native soil or compacted engineered fill may be taken as equivalent to the pressure exerted by a fluid weighing 200 pounds per cubic foot (pcf) acting on one pile diameter for the portion of the pier foundation embedded in firm soil. For piers in bedrock, an allowable equivalent fluid weight of 300 pcf can be used. This passive pressure value is an allowable value derived using an estimated shaft head deflection of about ½ inch. We anticipate that there may be a variety of pier lateral loading conditions due to the configuration of the proposed structure. The appropriate factor of safety for lateral load resistance will depend on the design condition and should be selected by the designer.

The structural engineer should determine the actual embedded depth based on the lateral loads transmitted to the foundations. Once the structural loading information is available, if requested, Kleinfelder can perform an L-Pile analysis to assist in determining the shear, moments and lateral displacement for the piles based on the design loads. CIDH piles should be located no closer together than three pile diameters on-center.

We note that attention must be given to the method of CIDH pile construction to satisfy the above recommendations. For the anticipated lightly loaded structures, the need for slurry is not anticipated due to groundwater being at about 20 feet below existing grade. The existing site soils were highly variable and included some sand and gravel; therefore, casing should be available onsite to facilitate supporting the excavations if needed. Steel reinforcement and concrete should be placed within about 4 to 6 hours of completion of each drilled hole. As a minimum, the holes should be poured the same day they are drilled. The bottom of the drilled holes should be cleaned to remove as much loose soil as practical prior to placement of concrete. A representative from Kleinfelder should be present to observe drilled holes to confirm the soils encountered are capable of carrying the design loads and that bottom conditions are satisfactory prior to placing steel reinforcement.

The steel reinforcement should be centered in the drilled hole. Concrete should be discharged vertically with a tremie pipe from the shaft bottom upward at a rate in which the tremie nozzle does not become separated from the placed concrete by more than three feet. Under no circumstances should concrete be allowed to free-fall against either the steel reinforcement or the sides of the excavation during construction. Sufficient vibration should be performed while the concrete is tremied to minimize voids and properly derive the frictional shaft surface to satisfy design requirements.

Prior to mobilizing drilling equipment to the site, the foundation contractor should submit to Kleinfelder a construction plan describing the procedures it intends to utilize in the CIDH pile construction process. Kleinfelder should review this plan and confirm that the procedures conform to the recommendations provided herein.

7.2 SLABS-ON-GRADE

7.2.1 General Considerations

Concrete slabs-on-grade will include building interior floor slabs. The slabs should be placed on 6 inches of capillary break material over a prepared subgrade. The capillary break material should be at least 6 inches thick, and should consist of free-draining crushed rock or gravel (no rounded rock) graded such that 100 percent will pass the 1 inch sieve and none will pass the No. 4 sieve. All slabs should be supported on properly prepared subgrade soils, as described in Section 7.6 “Earthwork” of this report.

Where the risk of moisture penetration through interior floor slabs is to be reduced, the slab should be constructed on a layer of capillary break material covered by a continuous impermeable membrane vapor barrier. The impermeable membrane should consist of a minimum thickness of 10-mil polyethylene sheeting or similar moisture barrier. Lapped joints and perforations in the vapor barrier should be kept to a minimum, and should be sealed. To provide protection for the membrane, 2 inches of slightly moistened clean fine sand should be placed on top of the membrane prior to placement of concrete. The 2 inches of sand can replace 2 inches of the capillary break material. Where crushed rock is used as the capillary break material, seating of the rock with a vibratory plate compactor may aid in reducing the potential for damage to the vapor barrier as the reinforcing steel and the concrete are placed.

It should be emphasized that we are not floor moisture proofing experts. While the current industry standard is to place a vapor barrier over a gravel layer as described above, this system may not be completely effective in preventing floor slab moisture problems. These systems typically will not necessarily assure that floor slab moisture transmission rates will meet floor-covering manufacturing standards and that indoor humidity levels be appropriate to inhibit mold growth. The design and construction of such systems are totally dependent on the proposed use and design of the proposed building. All elements of building design and function should be considered in the slab-on-grade floor design. Building design and construction may have a greater role in perceived moisture problems since sealed buildings/rooms or inadequate ventilation may produce excess moisture in a building and affect indoor air quality.

The structural engineer should design the slab thickness, reinforcing, and control joint spacing. However, a minimum floor slab thickness of 5 inches is recommended for interior floor slabs, and 6 inches for exterior flatwork subject to vehicle traffic.

7.2.2 Modulus of Subgrade Reaction

Rigid concrete slabs consisting of Portland cement concrete may be considered for use in certain areas of the new development. Using the Portland Cement Association Simplified Design Procedure and the R-value laboratory testing results, we recommend the use of the values given in Table 5 below for modulus of subgrade reaction. These values are based on subgrade preparation as per the recommendations in this report and outlined in Exhibit 1. Our design is based on estimated modulus of subgrade reaction values as presented in the table below at the top of the compacted subgrade, with doweled joints or aggregate-interlock joints, and a modulus of rupture for the concrete of 550 pounds per square inch. The modulus is a function of the bearing pressure (e.g. the footing size and load) and the estimated settlement discussed in Section 7.1.

Table 5
Rigid Concrete Slab Recommendations

General Region of the Site	Anticipated Subgrade Material	Modulus of Subgrade Reaction (pci)
Western	Existing Gravel Fill, Bedrock, or New Engineered Fill	180
Southern	Fill (existing or new)	70
Eastern	Fill (existing or new)	70

The modulus should be adjusted for the actual slab size using appropriate formulas or software. For full slab sizes, the subgrade modulus may be estimated using the following formulas:

Square $Ks = K_{V1}\ ((B+1)/2B)^2$
Rectangular $Kr = K_{V1}\ ((m+0.5)/1.5m)$

In the relationships above, B is the foundation width and m is the ratio obtained by dividing the slab length by its width.

It should be noted that the modulus of rupture for concrete is based on flexural strength, not compressive strength, and should be specified accordingly. Concrete with a compressive strength of 3,000 psi is not expected to provide the desired flexural strength. Our experience is that the compressive strength will be on the order of 4,500 to 5,000 psi to achieve the required flexural strength. Laboratory testing to evaluate the design strength is recommended.

Subgrade preparation should extend a minimum of 2 feet laterally beyond the edge of the slab and consist of scarifying, moisture conditioning, and compacting as recommended in Exhibit 1. Compacted subgrade should be non-yielding. Removal and subsequent replacement of some material (i.e., areas of excessively wet materials, unstable subgrade, or pumping soils) may be required to obtain the minimum compaction to the recommended depth.

7.3 RETAINING WALLS

Cast in place walls up to 10 feet in height may be constructed for planter walls, truck loading dock pits, ramps, and other non-building walls. Higher cast in place retaining walls over about 10 feet in height are not planned for this project. At the area of the existing slope (due to former rock quarry activities) along the northern boundary of the site, other retaining walls on the order of 5 feet tall as well as cut slope retention measures for higher slopes are planned.

The following wall design recommendations are intended for variable retaining wall types that are expected for the project. Separate sets of geotechnical recommendations are provided, divided as described above for the cast in place type construction, and then for retaining walls at the existing cut slope along the northern boundary. Recommendations for rock slope retention measures needed at higher elevations (above the top of walls) for the high cut slopes at the northern end of the site along the vehicle/truck access driveway are provided in Section 7.4 of this report.

Retaining wall and slope designs shall be coordinated with other project design elements that might interfere with or impact the design or construction of the wall or slope. Selection of appropriate earth retention system for a given setting shall be based on design constraints, geotechnical subsurface investigations, and surface and groundwater issues. Consideration must be given to the presence of (and potential conflicts with) drainage features; buried and overhead utilities, lighting or sign structures, adjacent structures, traffic barriers, and/or fences and guardrails. These design elements shall be located in a manner that will minimize the impacts to the retaining wall or reinforced slope elements. The potential effect that site constraints might have on the constructability of the specific wall/slope shall be considered. Additional constraints to be considered include but are not limited to site geometry, access, time required to construct the wall, environmental issues, and impact on traffic flow and other construction activities.

The structural elements of the wall or slope and the soil below, behind, and/or within the structure shall be designed together as a system. The wall or slope system shall be designed for overall external stability as well as internal stability. Overall external stability includes stability of the slope the wall/reinforced slope is a part of and the local external stability (overturning, sliding, and bearing capacity). Internal stability includes resistance of the structural members to load and, in the case of anchored walls and reinforced slopes, pull-out capacity of the structural members from the rock/soil.

Wall types considered to be unacceptable include mortar rubble gravity walls, timber or metal bin walls, and “rockery” walls.

7.3.1 Lateral Earth Pressures and Foundations for Cast in Place Retaining Walls

Cast in place retaining walls up to 10 feet high are included in the development plans. These walls should be designed to resist the lateral earth pressures exerted by the retained, compacted backfill plus any additional lateral force that will be applied to the wall due to surface surcharge loads placed at or near the wall.

Active earth pressure should be used where the walls are allowed to deflect, and at-pressure should be used for restrained walls. The at-rest earth pressure against walls that are restrained at the top and with level backfill may be taken as equivalent to the pressure exerted by a fluid weighing 60 pounds per cubic foot (pcf). Fifty percent of any uniform area surcharge load placed at the top of a restrained wall may be assumed to act as a uniform horizontal pressure over the entire height of the wall.

Retaining walls that are not restrained at the top and also with level backfill may be designed for an active earth pressure developed by an equivalent fluid weighing 45 pcf. Thirty percent of any uniform surcharge load may be assumed to act as a uniform horizontal pressure over the entire height of the wall.

For cast in place walls over 5 feet high, horizontal accelerations during seismic events will apply additional (incremental) lateral earth pressures. A recommended value of 43 pounds per cubic foot (pcf) equivalent fluid pressure should be used to calculate the additional seismically-induced earth pressure. The additional seismic pressure for level backfill conditions will have a triangular pressure distribution, with the resultant seismic force assumed to act at a height of 0.33H above the base of the wall. The seismic earth pressures are in addition to the static earth pressures. The seismic earth pressure increment does not need to be included for design of proposed walls retaining the existing rock cut slope.

The above recommended lateral earth pressure values for backfill assume that “non-expansive” granular soil is used as wall backfill within the zone defined by a 1:1 (H:V) line extended up from the base of the heel of the wall. These lateral pressures assume properly compacted backfill, and use of subdrains behind the walls. The above-recommended values do not include lateral pressures due to hydrostatic forces. Therefore, wall backfill should be free draining and provisions should be made to collect and dispose of excess water that may accumulate behind earth retaining structures.

Continuous spread footing type foundations should be used for support of the low walls bearing in the anticipated subsurface conditions described above. In the event that unsuitable fill soil is encountered, Kleinfelder will need to provide recommendations for mitigation, likely to include limited over-excavation to a firm bottom with the resulting excavation backfilled with engineered fill or lean mix concrete.

All footings should extend to a minimum embedment depth of 24 inches below the lowest adjacent finished grade. A minimum footing width of 36 inches is recommended for cast in place walls over 5 feet tall. For shorter walls, a minimum footing width of 24 inches is recommended. The recommended allowable foundation bearing pressures for dead plus live loading is 3,000 pounds per square foot (psf). Allowable bearing pressure may be increased by one-third for transient loads such as wind and seismic loads.

For wall foundations with design bearing pressures equal to or less than the net allowable pressure provided above, and under static loading conditions, total post-construction foundation settlement is expected to be less than about 1 inch. Post-construction differential settlement is expected to be up to about ½ inch over an approximate 25 foot span. Footing excavation bottoms should not be allowed to dry out between the time of excavation and concrete placement. As much as is practicable, all foundations of each wall structure segment should be founded in similar material (i.e. all in engineered fill, or all in bedrock) to reduce the risk of differential settlement between adjacent footings which could potentially damage the new structures.

Resistance to lateral loads can be provided by passive pressure on the vertical faces of foundation elements and frictional resistance between the footing bottoms and underlying soil. This assumes the footing excavation sidewalls remain stable during construction, and the concrete is placed neat with the sides of the excavation. For design of retaining walls (including adjacent to the buildings) and related features (such as bio-retention basins) in Phase 1, allowable values for passive pressure and friction coefficient of 200 psf and 0.25 may be used, respectively. Foundations of retaining walls should extend below the depth of potential maintenance adjacent to bio-retention basins. The wall designer should neglect the bio-retention basin depth (the backfilled vertical interval above the invert elevation) for embedment due to potential soil saturation. Where bio-retention basins are located immediately adjacent to retaining walls, the basins should have durable impermeable liners installed.

Passive pressure should be neglected within the upper 12 inches of subgrade soil, unless the area in front of the footing is protected slabs or pavements that confine the surface. The friction coefficient and passive resistance may be used concurrently without reduction, and the passive resistance can be increased by one-third for wind and/or seismic loading.

7.3.2 Retaining Walls at Existing Cut Slope

Retaining walls approximately 5 feet and higher will be needed at the area of the existing rock cut slope (due to former quarry activities) along the northern boundary of the site. The longest wall segment planned is located between the local street driveway entrance at Broadway (at Station 1+00, West Ridge) and the parking deck ramp (at Station 6+78, West Ridge). Two additional minor retaining walls (shorter segment lengths) are planned in the area to the east of the ramp located from approximately Stations 2+90 to 3+25, and Stations 3+90 to 4+50 (on the East Ridge stationing line). Behind the main building ‘A’, current grading plans require additional cut excavations in the existing rock slope to make space for widened driveway lanes and delivery truck access. Recommendations for rock slope retention measures needed for these cut slopes east of the parking deck ramp are provided in Section 7.4 of this report.

Because the proposed vertical walls will be retaining existing cut slopes ascending above the tops of the walls, new retaining structures will also require tied back anchorage using grouted rock anchors penetrating in competent bedrock. Cantilever type walls that are free to rotate are not permissible. Based on review of subsurface conditions and evaluation of alternative wall types and consultation with the engineering design and development team, the recommended 'basecase' wall system is a Soldier Pile and Lagging (SPL) type wall, with one or possibly two rows of grouted anchors. A restriction on grouted anchor lengths is the shopping center property line boundary, which reportedly corresponds to the top of existing slopes. Lagging between soldier piles is expected to consist of reinforced concrete precast planks or panels. Lagging must be approved by the owner. Steel wide flange sections for soldier beams should be painted and subject to approval by the owner. Another anchored wall type option that may be considered is a cast in place wall system, however the anticipated larger extent (larger) of excavations needed for spread footings, temporary shoring, and overall higher costs are expected to be prohibitive compared to SPL walls.

The SPL wall and grouted anchor systems at rock cut slope areas should be provided by a design-build type specialty contractor who will be responsible for the final engineering design, assessment of loads, configurations, dimensions, elevations, grouting methods etc. required to meet wall/slope acceptance criteria. Final design and construction of restrained SPL walls for the cut slope areas must meet applicable codes. The design-build (D-B) contractor should develop their design and construction criteria, including QA/QC provisions and testing, and submit for approval by the Owner.

Preliminary geotechnical parameters for consideration corresponding to anchored SPL type walls are presented below in Table 6. The values for fill in Table 6 assume imported, granular fill material is used for backfill of the SPL walls. However, D-B retaining wall and slope retention contractors bidding the work at the cut slope areas should not rely solely on investigation data and information presented in this report for the variable subsurface conditions etc., and are responsible to examine the site and make their own interpretations for engineering design. Besides the variable subsurface rock and soil conditions, since the cut slope configurations, retention requirements, and surcharge loading scenarios (due to ascending slopes above walls) will vary significantly along the new walls on a segment by segment basis, then refined analysis will be required by the D-B.

Table 6
Lateral Earth Pressures for Retaining Walls at Existing Cut Slope

Soil/Rock Type	Effective Unit Weight (pcf)	Internal Angle of Friction (degrees)	Ultimate Passive Equivalent Fluid Pressure for Level Backfill (pcf)	Ultimate At-Rest Equivalent Fluid Pressure for Level Backfill (pcf)
Tonalite	140	40	640	50
Fill	125	34	440	55
Colluvium	120	32	390	56

Vertical end-bearing capacity of soldier piles embedded into competent tonalite bedrock may be taken as 15,000 psf. For preliminary wall design, the recommended minimum embedment depth for soldier piles (penetration into competent bedrock) is 5 feet, however lateral and vertical load resistance analysis and design is the responsibility of the D-B contractor for walls. The maximum center-to-center soldier pile spacing is typically 8 feet.

Tie-back anchors are typically designed with a minimum un-bonded length of 10 to 15 feet. The bonded portion of the anchor typically extends beyond a 45 degree line projected behind the retaining wall beginning approximately two feet below the bottom of the excavation. For preliminary design and sizing of the retention systems, post-grouted rock anchors in tonalite bedrock should be designed for an ultimate bond strength of 200 psi. For allowable stress design, a safety factor of at least 2.0 should be applied to the grout-to-ground ultimate bond strengths. These estimated/presumptive grout-to-ground bond ultimate strengths are based on interpretation of subsurface conditions, and past experience using Post Tensioning Institute (PTI 2004) guidelines. The anchors should be inclined 10 degrees below horizontal. The actual unbonded and bonded lengths should be developed by the D-B contractor. Site specific load testing should be performed in advance of production anchor installation to verify (or modify) the design values. Retaining wall submittals and products will be subject to review with the engineering design team and owner for acceptability.

Considering the walls will be situated adjacent to cut slopes in rock ascending above the walls, to improve protection of completed walls and performance of the overall wall 'systems' we recommend that rockfall resistant design measures be considered, such as incorporation of a rockfall 'catchment' vertical barrier at upper portion of the wall structures. At a minimum, retaining walls should extend at least 1.5 feet above the elevation of top surface of engineered backfill or slope.

Additional aspects identified for consideration at the slope areas in cut wall design and construction include the following:

- Corrosivity (of soil and rock materials) and potential corrosion loss must be accounted for in design of steel and concrete elements,
- For SPL walls, the minimum wall lagging embedment depth below lowest adjacent finished grade (at the adjacent pavement surface) is 12 inches,
- Continuous barrier perimeter fencing should be designed for and installed at top of all slopes (or northern property line) as well as at the base of retained cut slopes to keep out pedestrians and wildlife (i.e. local deer),
- Access space behind fences should be provided (approximately 2 foot minimum width) for periodic inspection and 'clearing' of debris expected to collect at the base of slopes and tops of walls by maintenance staff.

7.3.3 Drainage at Walls and Cut Slopes

The recommendations above for retaining wall designs do not include lateral pressures due to hydrostatic forces from groundwater. Therefore, wall backfill should be free draining and provisions should be made to collect and dispose of excess water that may accumulate behind earth retaining structures.

The wall structures may be designed without hydrostatic pressures if they are fully drained. Backdrainage should consist of either a prefabricated drainage material or a layer of drain rock. Prefabricated drainage material (such as Miradrain® 2000 or an approved alternative) may be used directly behind walls. Prefabricated drainage material, typically in either continuous (large) panel or narrow vertical strip configurations, should be installed in accordance with the manufacturer's recommendations.

As an alternative to prefabricated drainage material, a drain rock layer may be used for low walls. The drain rock layer should be at least 12 inches thick (section width), and extend to within 1.5 feet of the finished ground surface behind walls. A four-inch diameter, perforated, schedule 40 PVC (or equivalent) pipe should be installed (with perforations facing down) along the base of the wall. Drain pipes should rest on a 2-inch-thick bed of drain rock. Drain pipes should be sloped to drain by gravity to a sump or other drainage facility. Alternately, weep holes at least 3 inches in diameter and spaced no farther than 8 feet apart may be used where drainage from the holes does not create a hazard, and is acceptable. Drain rock should conform to Caltrans specifications for Class 2 Permeable material. Alternatively, clean, 1/2 to 3/4-inch maximum size crushed rock or gravel could be used, provided it is fully encapsulated in a non-woven geotextile filter fabric, such as Mirafi 140N or an approved alternative.

Subdrainage recommendations (at foundation level) presented above should also be used for the proposed structural features supporting the elevated parking deck located at the base of cut slopes in the vicinity adjacent to intersection of planned buildings ‘K’ and ‘A’.

Surface drainage provisions are required for design of cut slopes and walls. Surface drainage should be accomplished through the use of drainage ditches and berms located above the top of the cut, around the sides of the cut, and at the base of the cut. Surface drainage at base of slopes (and tops of walls) should be diverted using v-ditches to the storm drain system.

7.3.4 Backfill Placement

Engineered backfill behind the retaining walls should conform to the material requirements in this report, and consist of granular, imported soil or approved on-site soils of a low expansion potential. Clays with moderate to high expansion potential shall not be used as backfill behind retaining walls. Over-compaction of wall backfill should be avoided because increased compaction effort can result in lateral pressures significantly greater than those recommended above. We recommend that all backfill placed within 3 feet of the walls be compacted with hand-operated equipment to minimize possible overstressing of the wall.

7.3.5 Construction Considerations

Landscape planting (trees, shrubs etc.) should not be situated located behind the walls. Excavations at slope areas for installation of landscaping are not advisable, and irrigation watering as well as future root growth etc. could contribute to hazardous conditions and negative impacts for wall/slope stability and integrity over time.

Existing trees and shrubs should be cleared from the slopes along segments where slope cuts are planned, including retaining wall alignments and areas where slope cut retention measures will be constructed. Remaining root systems can be left in place.

Retaining wall and slope retention construction activities will need to consider potential disturbance to the adjoining properties, residents, and other features above the site to the north of the slopes. This includes construction-related vibration, noise, dust, etc. but also control (by the contractors) of local slope stability and potential ground movements or deformation. Contractors will be responsible for maintaining these aspects within acceptable levels and tolerances, once established. It is strongly recommended that property condition (interior and exterior) surveys be conducted prior to, during and after construction including use of videography for recording and documentation purposes. In addition, the contractor should establish and frequently monitor fixed survey points around and at the top of slopes for deformation/movements, and have advance contingency plans in place for corrective actions as necessary.

For fill walls, properly compacted retaining wall backfill may experience some settlement or deflection after construction. This is a result of normal deflection of the wall and settling of engineered fills. This post-construction "settling in period" will vary with wall type, size, and construction and should be taken into account in overall site design.

7.4 ROCK SLOPE DESIGN AND STABILIZATION

7.4.1 Rockfall Protection Alternatives

Based on our field reconnaissance, there appear to be two priority areas for rock fall protection. Both areas are on the East Ridge, and extend from approximately Stations 3+25 to 3+90 and Stations 4+50 to 5+24. Along both of these slopes there is potential for rockfall from loose rock that could present a significant hazard to facilities and/or human activity in these and adjoining areas.

We have considered stabilization options for the two areas outlined above:

1. Potential Rockfall Protection Methods for East Ridge
    - *Shotcrete Slope Areas* – It is our opinion that shotcrete alone on these steep slope areas would not be effective. Shotcrete does not have adequate strength to stabilize large potentially unstable blocks. Shotcrete in combination with rock bolting could be effective but with potential seepage issues on the slope, there are other more effective and less costly stabilization methods.

- *Slope Reconfiguration* – The existing slopes could be excavated and reconfigured to a kinematically stable slope inclination based on the stereonet analyses. However, based on the range of inclinations for the potential failures noted, this option may not be practical.

- *Scaling Steep Slopes* – Scaling some of the loose blocks and debris in the steep slope areas would reduce the risk of rockfall but with the potential for seepage in this area, rockfall would be an ongoing issue. Scaling to remove all overhanging blocks and debris would require removing a substantial amount of material and this could require removal beyond the property line at the top of the slope to be effective. Complete removal of all overhanging blocks and debris would likely not be cost effective compared to other options.

- *Rock Anchors to Stabilize Slopes* – The installation of rock anchors can be used to stabilize the potential wedge and planar type failures identified from the field mapping. As new cut slopes are excavated, rock anchors can be installed on a patterned system to reduce the potential for larger-scale rock block failures. Rock anchors will not control rockfall from smaller loose blocks. Rock anchors can be used in conjunction with a slope drape to reduce the potential for rockfall.

- *Drape Steep Slopes* – A wire mesh drape system could be placed over the steep slope areas that would direct the rockfall down along the slope and reduce the risk of loose rock bouncing off the cut and impacting people on the ground. The larger blocks would likely fall closer to the slope with this method but would still have the potential of rolling out from the slope and blocking the activity areas behind the proposed facilities. This method could be used in conjunction with a barrier to control the risk of loose rock impacting people on the ground and stopping the larger blocks from rolling away from the slope and blocking the activity areas at the base of the slopes.

- *Bolted Mesh Stabilization on Steep Slopes* – Bolted mesh stabilization would involve the use of high tensile strength wire mesh secured by rock bolts at a spacing of approximately ten feet. There are many commercially available systems such as Teccomesh ® that would be appropriate for use. The mesh would be placed in panels with an overlap of approximately 5 feet on either side of the potentially unstable areas and would secure large and small blocks to the slope. This protection method would reduce the rockfall hazards at the base of the slopes.

Many of the potential rockfall protection methods discussed may potentially require access to the top of the rock cut for scaling, rock anchor, and/or drape installation. During our field reconnaissance, it was observed that access to the top from the above slope would be difficult due to uneven steep terrain, rock outcrops, and property line constraints. It is anticipated that access from the bottom by a crane or lift is feasible in this area. Construction access should be discussed with a contractor that specializes in rock slope stabilization.

We recommend that a combination of the options listed above be used in the areas with proposed rock cut slopes. We recommend the following:

- Excavate the proposed rock cut slopes to an inclination of 76 degrees (0.25H:1V). This will not eliminate the potential for block failures, but will help reduce it.
- Scale loose blocks from the existing slope above the cut areas and scale loose rock from the constructed slopes.
- Install fully-grouted 25-kip capacity rock anchors on 5-foot centers horizontally and vertically on the constructed cut slopes. For the proposed cut slope from approximate Station 3+25 to 3+90, we recommend 10-foot long rock anchors. For the proposed cut slope from Approximate Station 4+50 to 5+24, we recommend 20-foot long rock anchors.
- In the areas of the proposed cut slopes, drape the slope with a mesh drape system (Teccomesh® drape or similar) to the top of the existing cut slope. There is the potential for rockfall to originate in the existing slope and launch of the steeper proposed cut below and onto the access road. The installation of a mesh drape system over the full slope will reduce this potential.

We recommend that Kleinfelder provide cut slope stabilization observation by our rock engineers to validate design recommendations. Our engineers will observe rock features such as jointing, faulting, joint irregularity, and orientation, and if conditions vary, will use this information for evaluation and potential modifications to design.

As described above, constructing the proposed retaining walls and installing rock anchor slope retention measures along planned cut slope areas is intended to create additional level space for various features at the shopping center redevelopment project. The planned walls and rock cut retention measures are not intended or have been designed to improve the global and/or local slope stability hazards that may currently exist, or potentially develop in the future. The current scope of services requested and approved by the owner did not include the assessment of global and/or local slope stability hazards for purposes of small or large scale mitigation programs (stability improvement) that could be intrusive, costly, and potentially extensive. Considering the current conditions and prior rock quarrying activities over the past decades, these slope hazards may exist, especially during rare or extreme loading events such as intense precipitation during wet-season storms or seismic shaking. The shopping center owner/developer has acknowledged this, and we understand that it has been considered as part of their overall risk management process for the full development. This was discussed at the developer/design team progress meeting on September 24, 2014 in our Pleasanton office.

7.5 DEMOLITION

7.5.1 Existing Improvements

As part of the demolition process, existing foundations and other improvements should be removed. Excavations from removal of foundations, underground utilities or other below ground obstructions where located outside of the planned excavation for the underground parking should be cleaned of loose soil and deleterious material, and backfilled with compacted engineered fill. Recommendations for compaction of fill are included in Section 7.6 "Earthwork" of this report and presented in Exhibit 1.

7.5.2 Existing Utilities

Active or inactive utilities within the construction area should be protected, relocated, or abandoned. Pipelines that are 2 inches in diameter or less may be left in place beneath the planned buildings. Pipelines between 2 and 6 inches in diameter may be removed or left in place within the limits of the buildings provided they are filled with sand/cement slurry and capped at both ends. Pipelines larger than 6 inches in diameter within the planned buildings should be removed. Active utilities to be reused should be carefully located and protected during demolition and during construction.

7.5.3 Existing Trees

Tree stumps and roots over 1 inch in diameter and over 3 feet in length should be removed within the building footprints and areas for planned improvements. From a geotechnical standpoint, existing landscaping may be left in place as landscaping provided that it is outside of the area to be graded.

7.5.4 Landscape and Paved Areas

Based on our experience, areas covered by landscape or that are paved have above optimum moisture contents. We recommend that sprinklers in the area be turned off at least two weeks before earthwork if possible. Consideration may also be given to planning for additional time to allow these areas to dry out or for over-excavation to suitable material below the wet areas.

7.6 EXTERIOR FLATWORK

Prior to construction of exterior flatwork, including concrete pavements, the subgrade should be moisture conditioned and compacted according to Section 7.6 “Earthwork” of this report and Exhibit 1 attached. Where flatwork is to be exposed to vehicular traffic, we recommend that it be underlain by 6 inches of compacted Class 2 Aggregate Base material. Where flatwork is adjacent to curbs, reinforcing bars should be placed between the flatwork and the curbs. Expansion joint material should be used between flatwork and curbs, and flatwork and buildings. The design of concrete pavements should incorporate the drainage and pavement specific earthwork recommendations provided in Section 7.8 “Pavements” of this report. If the flatwork is not exposed to vehicle traffic, it should be underlain by a minimum of 4 inches of baserock or sand to provide a leveling course.

It is our understanding that aesthetic pavers may be used on this site. Our experience indicates that pavers set in sand underlain by aggregate base do not perform well in heavy vehicle traffic areas. Therefore, the roadway base for the pavers should be a Portland cement concrete (PCC) rigid pavement. The concrete pavement should be a minimum of 6 inches of concrete over a minimum of 6 inches Class 2 aggregate base. A minimum concrete compressive strength of 3,000 psi should be used based on our design assumptions. The subgrade should be moisture conditioned and compacted according to Section 7.7 “Earthwork” of this report and Exhibit 1 attached. The pavers should be adhered to the concrete pavement using cement mortar.

7.7 EARTHWORK

Earthwork at the site will generally consist of minor cuts and fills to create the building pads, to fill the disturbed soil and voids resulting from the demolition of the existing buildings, subgrade preparation and placement of baserock or crushed rock for concrete flatwork and pavements, and excavation and backfill of underground utility line trenches. Kleinfelder should review the final grading plans for conformance to our design recommendations prior to construction bidding. In addition, it is important that a representative of Kleinfelder observe and evaluate the competency of existing soils or new fill underlying structures, concrete flatwork, and pavements. In general, soft/loose or unsuitable materials encountered should be overexcavated, removed, and replaced with compacted engineered fill material.

Construction debris consisting of aggregate base, concrete, and asphalt concrete generated during the demolition operation may be used as general fill material provided that it meets the grading and expansive criteria for import material specified in Section 7.6.3 “Fill Material” of this report. Note that construction debris consisting of organic material (i.e., wood, mulch, etc.), metal, or similar degradable materials should not be used as fill material at the site and should be hauled offsite.

Site preparation and grading for this project should be performed in accordance with the site-specific recommendations provided below. A summary of soil compaction recommendations for this project is presented in Exhibit 1. Additional earthwork recommendations are presented in related sections of this report.

Prior to the start of the excavation for the below ground portion of the project, it is suggested that a survey of nearby structures be made to document existing conditions. In addition, survey points should be established around the perimeter of the site to measure both vertical and horizontal movements.

7.7.1 Site Preparation and Grading

Prior to the start of grading and subgrade preparation operations, the site should first be cleared of debris generated during the demolition of existing pavements, concrete slabs and flatwork, foundations, and landscaping, and, in planter areas, stripped to remove all surface vegetation and organic laden topsoil. Stripped topsoil from landscaped areas may be stockpiled for later use in landscaping areas; however, this material should not be reused for engineered fill.

Following stripping and removal of deleterious materials, areas of the site to receive fill should be scarified to a minimum depth of 12 inches, moisture-conditioned, and recompacted as indicated on Exhibit 1. Scarification should extend laterally a minimum of 5 feet beyond the building limits and 2 feet beyond flatwork and pavements, where achievable, and any debris uncovered by this process should be removed. All fills should be compacted in lifts of 8-inch maximum uncompacted thickness. A summary of compaction requirements for the project is presented in Exhibit 1. Laboratory maximum dry density and optimum moisture content relationships should be evaluated based on ASTM Test Designation D-1557 (latest edition). Caution should be taken during grading and compaction to reduce the "pumping" of soft or wet soil. This could result in the need to use light weight compaction equipment in low areas and rerouting truck traffic to avoid overstressing the haul roads.

All site preparation and fill placement should be observed by a Kleinfelder representative. It is important that, during the stripping and scarification process, our representative be present to observe whether any undesirable material is encountered in the construction area and whether exposed soils are similar to those encountered during our field investigation.

7.7.2 Rippability

As described in Section 3.4, seismic refraction geophysical surveys were performed for purposes of subsurface profile data collection and rock rippability (excavatability) evaluation of the bedrock present at the site. This pertains to the areas of the site located south of the rock cut slope. Generally, the results indicate that the bedrock depth ranges from 0 feet to 60 feet below ground surface (bgs), with the top of bedrock occurring mostly between 20 and 40 feet bgs. However, the depth to bedrock is shallower in some areas (i.e. west side of the site) and highly variable, especially considering that a large portion of the site was a former rock quarry. A complete report of the results and interpretations of the seismic refraction survey is included in Appendix D. The locations of the seismic refraction lines were chosen to obtain relative depth to bedrock and rock rippability in the study area, and subject to available space constraints at the active shopping center site.

The seismic velocities obtained during our seismic-refraction survey are necessarily averages across differing soil and rock conditions. Bedrock units can have coherent rock masses separated by discontinuities (fractures, bedding planes, joints, highly weathered zones, etc.). This fact should be considered and allowed for when using seismic-refraction to estimate rippability. Seismic waves travel through coherent rock relatively fast and travel through intervening discontinuities relatively slow. The resulting seismic velocity through rock units will be the sum of velocities across coherent rock masses and discontinuities, and will not be a true velocity through either rock type. As a result, variable rippability or excavation conditions both harder and softer can be encountered along the geophysical survey lines.

The velocity with which rock transmits induced shock waves is controlled by its strength and degree of induration, and these characteristics materially affect the rock's rippability. Conditions that are favorable for seismic-wave transmission and therefore unfavorable for rippability include the following:

- Massive or homogeneous rock units;
- Absence of planes of structural weakness;
- High degree of cementation;
- High compressive strength; and,
- High rock quality determination (RQD).

Rock conditions that are favorable for rippability include:

- Presence of fractures, faults, and planes of weakness;
- Weathering and brittleness;
- High degree of stratification or lamination;
- Loose cementation;
- Low compressive strength; and,
- Low RQD.

In evaluating the seismic-refraction velocities with respect to rock rippability, we used the technical reference "Handbook of Ripping - 12th Edition", by Caterpillar Inc. Tractor Company (Caterpillar), for heavy-duty ripper performance. The ripper performance chart provided in the reference document by Caterpillar is for a large track-mounted D9 model bulldozer fitted with single or multi shank rippers attached. The rippability ratings are used as an indicator of the relative difficulty anticipated in excavating soil and rock at the project site, and should be adjusted based on the equipment selected by the project grading contractor. It should be expected that even though upper (near surface) layers of soil and weathered rock at this site are within the rippable range, harder areas might be encountered that marginally rippable to non-rippable.

The seismic-refraction surveys suggest that there is an undulating or irregular contact between a rippable upper layer (soil to weathered rock), and underlying much harder non-rippable bedrock. The seismic data also indicates that the bedrock slopes (dives) downward in the direction of the former quarry pit on the east side of the site, as expected. The high velocity bedrock at depth is interpreted to be more indurated (hard) and very resistant to ripping. The hardness and strength of both the weathered, upper velocity layer and the deeper bedrock material appear to have significant variability, and will probably have areas that range from rippable and marginally rippable to areas that are very hard and non-rippable.

The interpreted “layers” shown on the geophysics plates presented in Appendix D are velocity layers and reflect interpreted zones of relatively consistent velocities and may not represent actual rock contacts or other physical characteristics. It is important to note that the earthmoving equipment operator’s experience, working condition of excavation equipment and the selection of excavation tools used will be critical factors in the excavatability of rock. During construction, modifications to tool selection or replacement of equipment being used may be necessary to improve performance and production rates. It is recommended that the contractor who uses the rippability data in this report, visit the site to observe bedrock conditions, and that the contractor have options available in order to address differing bedrock conditions.

7.7.3 Excavation

In general, our test borings encountered shallow bedrock near the ground surface on the western part of the site, and variable existing fill on the eastern and southern areas of the site. For the majority of the site, only minor cuts are anticipated to achieve desired grade, for construction of foundations, and for installation of underground utilities. However, in the southwestern corner of the site at the location of the proposed Building G, it is understood a basement level is to be constructed.

It is expected, based on the test borings, that conventional earth moving equipment can be used for the eastern and southern areas of the site where the variable fill was primarily encountered in the borings. Special equipment may be required to remove any existing concrete slabs, footings, and other buried obstructions associated with the previously existing structures on the site. On the western area of the site, hard bedrock was encountered at shallow depth. As discussed previously, greater effort will be needed for excavations that extend into this material, such as for the footings, utilities and the proposed Building G basement level. In some local areas, the use of hoerams, pneumatic hammers or other equipment, or other measures to break the rock may be needed.

If the proposed basement excavation is graded with open cuts, the cuts should not be any steeper than 1:1 (horizontal to vertical). Equipment and materials should be kept a minimum of 10 feet back from the top of any cuts. In addition, safety precautions to keep people and equipment away from the top of the slope should be provided.

7.7.4 Fill Material

Except for organic laden topsoil in landscaped areas, and any material containing organics, the on-site soil is suitable for use as general engineered fill if it is free of deleterious material matter. Maximum particle size for fill material should be limited to 3 inches, with at least 90 percent by weight passing the 1 inch sieve. Where imported material is required, it is recommended that it be granular in nature, adhere to the above gradation recommendations, and conform to the following minimum criteria:

Plasticity Index	15 or less
Liquid Limit	less than 30%
Percent Soil Passing #200 Sieve	8% to 40%

Existing asphalt concrete may be pulverized and mixed with the underlying aggregate base for use as engineered fill provided it meets the following gradation requirements:

Table 7
Reuse of Crushed Asphalt Concrete

Sieve Size	Percentage Passing
3 inch	100 minimum
1½ inch	85 minimum
No. 200	8 – 40

Similarly, concrete slabs and foundations may be recycled by crushing. The processed asphalt concrete and aggregate base material and/or crushed concrete may be used as "non-expansive" fill if the material meets the gradation and plasticity requirements outlined above. The processed asphalt and concrete may also be used as general fill or used as a "base course" surface for paved or unpaved access roads.

Highly pervious materials such as pea gravel are not recommended because they permit transmission of water to the underlying soils, except as bedding material for utilities and in relatively narrow excavations resulting from removal of existing piles. In addition, imported fill material should be tested for corrosion, and should not be any more corrosive than the on-site soils. We recommend that representative samples of the material proposed for use as fill be submitted to Kleinfelder for testing and approval at least two weeks prior to the start of grading and import of this material. All on-site and import fill material should be compacted to the recommendations provided for engineered fill in Exhibit 1.

The moisture conditioning should be performed in accordance with Exhibit 1. Where low expansion potential soils or baserock in paved areas are used, it should be placed immediately over the prepared subgrade to avoid drying of the subgrade. Prior to the placement of the capillary break or drainage gravel (if applicable) over the subgrade for the building, the subgrade should be conditioned to the moisture content indicated in Exhibit 1; if the subgrade for the underground parking is not disturbed during excavation, it is anticipated that moisture conditioning will not be needed. The subgrade for exterior concrete flatwork should be conditioned to the required moisture content prior to their construction, and may require additional conditioning if it is allowed to dry. Caution should be taken during compaction to reduce “pumping” up of groundwater by repeated or heavy vehicle traffic.

7.7.5 Weather/Moisture Considerations

If earthwork operations and construction for this project are scheduled to be performed during the rainy season or in areas containing saturated soils, provisions may be required for drying of soil or providing admixtures to the soil prior to compaction. If desired, we can provide recommendations for wet weather earthwork and alternatives for drying the soil prior to compaction. Conversely, additional moisture may be required during dry months. Water trucks should be made available in sufficient numbers to provided adequate water during earthwork operations.

Since portions of the site are currently capped with concrete slab or AC pavement, the moisture content of the subgrade soils in these areas may be significantly above the optimum moisture content. This occurrence is usually caused by the migration of irrigation water from landscaped areas into the aggregate base material and/or the entrapment of subsurface moisture underneath slab and pavement areas. As a result, the subgrade soils may need to be dried prior to undergoing recompaction. It is also recommended that any landscape watering in the area be turned off at least two weeks prior to the start of grading activities at the site. If site grading is performed during the rainy months, the site soils could become very wet and difficult to compact without undergoing significant drying. This may not be feasible without delaying the construction schedule. For this reason, drier import soils could be required or lime treating may be needed if construction takes place during winter months.

7.7.6 Footing and Trench Excavation and Backfill

We anticipate that excavation for foundations and utility trenches in the eastern and southern areas of the site can be made with either a backhoe or trencher, or similar earthwork equipment. In the western area of the site, heavier equipment and the use of hoerams, pneumatic hammers or other measures will likely be required.

Where trenches or other excavations (i.e. Building G basement) are extended deeper than 4 feet, the excavation may become unstable and should be evaluated to monitor stability prior to personnel entering the trenches. Shoring or sloping of any trench wall may be necessary to protect personnel and to provide stability. Bids should be obtained for continuous shoring, which may include solid plate shoring, trench boxes, sheet piling, or other propriety systems that provide continuous shoring. The contractor's proposed shoring system(s) should be submitted to the architect for approval prior to use. We recommend that incremental bid items be included in the bid item list for the various types of shoring.

All trenches or other excavations should conform to the current California Occupational Safety and Health Administration (Cal-OSHA) requirements for work safety. It is the contractor's responsibility to follow Cal-OSHA temporary excavation guidelines and grade the slopes with adequate layback or provide adequate shoring and underpinning of existing structures and improvements, as needed. Slope layback and/or shoring measures should be adjusted as necessary in the field during construction to suit the actual conditions encountered, in order to protect personnel and equipment within excavations. These recommendations assume minimal equipment vibration and adequate setbacks of excavated materials and construction equipment from the top edge of the excavation. We recommend that the minimum setback distance be one-half the excavation depth. We have also assumed that the moisture content of the soil in the cut face will not be allowed to change significantly.

Care should be taken during construction to reduce the impact of trenching on adjacent structures and pavements (if applicable). Excavations should be located so that no structures, foundations, and slabs, existing or new, are located above a plane projected 1:1 (horizontal to vertical) upward from any point in an excavation, regardless of whether it is shored or unshored.

Backfill for trenches and other small excavations beneath slabs and within pavement areas should be compacted as noted in Exhibit 1. Care should be taken in the control of utility trench backfilling under structures, pavements, and flatwork/slab areas. Poor compaction may cause excessive settlements resulting in damage to overlying structures, slabs, and the pavement structural section. Where backfill is to be placed against walls, care should be taken to not use equipment that could overload and damage the wall. Equipment other than small hand propelled equipment is to be used for compaction should be approved by the Structural Engineer.

Pipe bedding should consist of sand or similar granular material having a minimum sand equivalent value of 30. The majority of the near-surface, onsite soils is relatively fine grained and is not suitable for pipe bedding. The imported sand should be placed in a zone that extends a minimum of 6 inches below and 12 inches above the pipe for the full trench width. Crushed rock bedding may be used but should be wrapped in geotextile drain fabric to help prevent fines from surrounding trench walls or overlying backfill from migrating into the open pore spaces of the crushed rock, which may cause settlement of overlying construction. The bedding material should be compacted to a minimum of 90 percent of the maximum dry density. Trench backfill above pipe bedding may consist of approved, onsite or import soils placed in lifts no greater than 8 inches in loose thickness and compacted to 90 percent of the maximum dry density with 95 percent compaction of the upper 12 inches under pavement sections. Jetting of pipe bedding or trench backfill materials is not permitted.

7.7.7 Temporary Dewatering

We anticipate that excavations will not encounter groundwater at the site. However, perched water may be encountered during excavation. As such, temporary dewatering may be required.

Temporary dewatering for construction is the responsibility of the contractor. The selection of equipment and methods of dewatering should be left up to the contractor, who should be aware that modifications to the dewatering system may be required during construction depending on the conditions encountered. The dewatering method selected should have minimal impact on the groundwater level surrounding the proposed excavation. We recommend that temporary dewatering of the site be carried out in such a manner as to maintain the groundwater a minimum of 2 feet below the base of excavations for utilities or structures.

As a minimum, provisions should be made to ensure that conventional sump pumps used in typical trenching and excavation projects are available during construction in case groundwater is found to be higher than observed during our investigation, and/or if substantial runoff water accumulates within the excavations as a result of wet weather conditions.

7.7.8 Seepage Control

Where utility lines extend through or beneath perimeter foundations or curbs at pavement areas, permeable backfill should be terminated at least 1 foot from the footings or curbs. Concrete or compacted clayey soil should be used around the pipes to act as a seepage cutoff. Beneath footings, the pipes should be “sleeved” through concrete cutoffs, and the annular space around the pipes should be filled with waterproof caulk. This will help reduce the amount of water seeping through the pervious trench backfill and collecting under the building or pavements.

Where slabs or pavements abut against landscaped areas, the base rock and subgrade soil should be protected against saturation. If landscape water or surface runoff is allowed to seep into the pavement section or subgrade, the service life of the pavement will be reduced dramatically. Subdrains behind curbs in landscape areas or vertical cut-off structures may be used to reduce lateral seepage under pavements or slabs from adjacent landscaped areas. Vertical cut-off structures may consist of deepened curb sections, or equivalent, extending at least 2 inches below the baserock/subgrade interface. Subdrains should discharge to a proper outlet or through weep holes in the vertical curbs as determined by the project civil engineer. Cut-off structures should be carefully constructed such that they extend below the base section and are poured neat against native soil or compacted clayey fill. The cut-off structures should be continuous. 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. In addition, care should be taken to prevent over-watering of landscaped areas.

7.7.9 Construction Observation

Variations in soil and rock types and conditions are likely on this site and are anticipated to be encountered during construction. To permit correlation between the subsurface data obtained during this investigation and the actual conditions encountered during construction, we recommend that Kleinfelder be retained to provide observation and testing services during site earthwork and foundation construction. This will allow us the opportunity to compare actual conditions exposed during construction with those encountered in our investigation and to provide supplemental recommendations if warranted by the exposed conditions. Earthwork should be performed in accordance with the recommendations presented in this report, or as recommended by Kleinfelder during construction. Kleinfelder should be notified at least two working days prior to the start of construction and prior to when observation and testing services are needed.

7.8 SITE DRAINAGE

Proper site drainage is important for the long-term performance of the planned structures, pavements, and concrete flatwork. The site should generally be graded so as to carry surface water away from the building foundation. The ground surface should slope away from the building at a minimum inclination of 4 percent in landscape areas and 2 percent in paved areas, for a minimum distance of 5 feet. The maintenance department should be instructed to not decrease the drainage gradient during future landscaping or other improvements. In addition, all roof gutters should be connected directly into a storm drainage system, or drain onto impervious surface (not splash blocks) that drain away from the structure, provided that a safety hazard is not created.

Project design includes plans for construction of onsite stormwater bio-retention basins (connected to the storm drain system) up to 4 feet deep in various areas of the at-grade parking lots and near drive aisles. Due to the presence of clay subgrade soils that were used to backfill the rock quarry pit, the infiltration/permeability characteristics of the clay soils are expected to be limited (too low to be viable for all stormwater drainage purposes).

7.9 PAVEMENTS

7.9.1 Flexible Pavements

Pavements for this project will consist of asphalt concrete (AC) access driveways and parking areas, and loading dock slabs. We have made our pavement designs assuming the pavement subgrade soil will be similar to the near surface soils described in the boring logs. This assumption is based on our anticipation that grading and soil removal in the paved areas will be minimal. If site grading exposes soil other than that assumed, or import fill is used to construct pavement subgrades, we should perform additional tests to confirm or revise the recommended pavement sections for actual field conditions.

Asphalt pavement sections for this project have been calculated using Caltrans Flexible Pavement Design Method. For our analysis, a Resistance (R)-value of 9 was assumed for the eastern and southern areas of the site and an R-value of 49 was assumed for the western area of the site. These values area based on the results of the laboratory testing for samples collected from the respective areas of the site. Additional sampling and subgrade testing during grading should be performed to verify or revise these design assumptions.

Various alternative pavement sections for various different Traffic Indices (TIs) are presented below. Each TI represents a different level of use. The owner or designer should determine which level of use best reflects the project and select appropriate pavement sections.

Table 8
Asphalt Concrete Pavement Design

Traffic Index	AC* (inches)	AB** (inches)
R-Value = 9 (eastern and southern areas of the site)		
4.0	3.0	7.0
4.5	3.0	8.5
5.0	3.0	10.0
5.5	3.0	11.0
6.5	3.5	14.0
R-Value = 49 (western area of the site)		
4.0	3.0	6.0
4.5	3.0	6.0
5.0	3.0	6.0
5.5	3.0	6.0
6.5	3.5	6.0

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

We recommend that the subgrade soil, over which the pavement sections are to be placed, be moisture conditioned and compacted according to the recommendations in Exhibit 1. Subgrade preparation should extend a minimum of 2 feet laterally beyond the back of curb or edge of pavement.

Subgrade preparation should extend a minimum of 2 feet laterally beyond the face of the curb (or edge of pavement if there is no curb) and consist of scarifying, moisture conditioning, and compacting as recommended in Exhibit 1. Compacted pavement subgrade should be non-yielding. Removal and subsequent replacement of some material (i.e., areas of excessively wet materials, unstable subgrade, or pumping soils) may be required to obtain the minimum compaction to the recommended depth.

Asphalt concrete should meet the requirements for ½- or ¾- 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. Class 2 aggregate base materials should conform to Section 26 of these specifications and aggregate subbase materials should conform to Section 25 with a minimum R-Value of 50. Class 2 aggregate base and aggregate subbase materials should be compacted to at least 95 percent relative compaction at near the optimum moisture content by ASTM Test Method D 1557, latest edition. Asphalt concrete should be compacted to a minimum of 96 percent of the maximum laboratory compacted (Hveem) unit weight. ASTM test procedures should be used to assess the percent relative compaction of the pavement subgrade soils, aggregate base and asphalt concrete.

Pavement surface should be sloped at a minimum of 2 percent and drainage gradients maintained to carry all surface water off the site due to the slightly porous or permeable nature of asphalt concrete. Surface water ponding should not be allowed anywhere on the site during or after construction. We recommend that the pavement section be isolated from non-developed areas and areas of intrusion of irrigation water from landscaped areas. Concrete curbs should extend a minimum of 2 inches below the baserock and into the subgrade to provide a barrier against drying of the subgrade soils, and a reduction of migration of landscape water into the pavement section. Weep holes on 4 feet on centers should also be provided. In lieu of the weep holes, a more effective system is to install subdrain behind the curbs.

In addition, we recommend that all pavements conform to the following criteria:

- All trench backfills, including utility and sprinkler lines, should be properly placed and adequately compacted to provide a stable subgrade, in accordance with the compaction recommendations in Exhibit 1;
- An adequate drainage system should be provided to prevent surface water or subsurface seepage from saturating the subgrade soil;
- The asphalt concrete, aggregate base, and aggregate subbase materials should conform to Caltrans Specifications, latest edition; and
- Placement and compaction of pavements should be performed and tested in accordance to appropriate ASTM test procedures.

7.9.2 Rigid Concrete Pavements

Rigid pavements consisting of Portland cement concrete may be considered for use in certain areas of the new development.

Using the Portland Cement Association Simplified Design Procedure and the R-value laboratory testing results, we recommend the use of the values given in Table 9 below for minimum concrete pavement thickness, minimum Class 2 Aggregate Base thickness. These values are based on subgrade preparation as pre the recommendations in this report and outlined in Exhibit 1. Our design is based on an estimated modulus of subgrade reaction values as presented in the table below at the top of the compacted subgrade, with doweled joints or aggregate-interlock joints and no concrete shoulder or curb, and a modulus of rupture for the concrete of 550 pounds per square inch.

Table 9
Concrete Pavement Recommendations

General Region of the Site	Anticipated Subgrade Material	Modulus of Subgrade Reaction (pci)	Minimum Concrete Pavement Thickness (in)	Minimum Class 2 Aggregate Base Thickness (in)
Western	Existing Gravel Fill, Bedrock, or New Engineered Fill	180	6.5	6
Southern	Fill (existing or new)	70	7.5	6
Eastern	Fill (existing or new)	70	7.5	6

It should be noted that the modulus of rupture for concrete is based on flexural strength, not compressive strength, and should be specified accordingly. Concrete with a compressive strength of 3,000 psi is not expected to provide the desired flexural strength. Our experience is that the compressive strength will be on the order of 4,500 to 5,000 psi to achieve the required flexural strength. Laboratory testing to evaluate the design strength is recommended. Alternatives to this design may be considered, based on the final design of the site grading plans and more accurate traffic data.

Subgrade preparation should extend a minimum of 2 feet laterally beyond the face of the curb (or edge of pavement if there is no curb) and consist of scarifying, moisture conditioning, and compacting as recommended in Exhibit 1. Compacted pavement subgrade should be non-yielding. Removal and subsequent replacement of some material (i.e., areas of excessively wet materials, unstable subgrade, or pumping soils) may be required to obtain the minimum compaction to the recommended depth.

7.10 SEISMIC DESIGN CRITERIA

The seismicity of the region surrounding the site is discussed in detail in Section 2 "Geology, Faulting, and Seismicity" of this report. From that discussion it is important to note that the site is in a region of high seismic activity and is expected to be subjected to major shaking during the design life of the store. As a result, structures to be constructed on the site should be designed in accordance with applicable seismic provisions contained in the 2013 California Building Code (CBC).

7.10.1 Liquefaction, Lateral Spread and Dynamic Compaction

The term liquefaction describes a phenomenon in which saturated, cohesionless soils temporarily lose shear strength (liquefy) due to increased pore water pressures induced by strong, cyclic ground motions. Such motions can be induced by construction activities such as blasting or pile installation; however, the majority of observed liquefaction events have resulted from earthquakes with magnitudes greater than 5½ to 6. Soils most susceptible to liquefaction are saturated, loose, clean, uniformly graded, and fine-grained sand deposits. If liquefaction occurs, foundations resting on or within the liquefiable layer may undergo settlements. This will result in reduction of foundation stiffness and capacities.

Based on the subsurface data obtained from the borings performed at the site, the soils encountered contained sufficient clayey soils or were of sufficient density to trigger liquefaction. As such, the potential for liquefaction at the site is low.

Lateral spreading is a potential consequence of liquefaction, resulting in lateral movement towards a slope. Because liquefaction is considered to be low at this site, lateral spreading is also considered to be low.

Dynamic compaction is the densification of granular soils as the result of earthquake shaking. This generally occurs in loose to medium dense sand above groundwater. The potential impact of dynamic compaction is settlement of the ground surface and structures/improvements supported above the layer. Although some medium dense near-surface sandy soil layers were encountered on the site, our analysis indicates that total and differential settlement due to dynamic compaction at the site as a result of a nearby major earthquake should be negligible; i.e. less than ¼ inches of total and differential settlement.

7.10.2 Seismic Design Criteria

The site is located in a seismically active region and the proposed new development can be expected to be subjected to moderate to strong seismic shaking during its design life. Potential seismic hazards include ground shaking, localized liquefaction, ground rupture due to faulting, and seismic settlement. Of these, ground shaking is the only seismic hazard that may impact the site based on our investigation.

In developing seismic design criteria, the characteristics of the soils underlying the site are an important input to evaluate the site response. Based on information obtained from the investigation, published geologic literature and maps, and on our interpretation of the 2013 California Building Code (CBC) criteria, it is our opinion that the west portion of the site where Buildings E, F, G, H, and J will be located may be classified as Site Class C, and eastern and southern portions of the site where Buildings A, C, D, and K will be located may be classified as Site Class D according to Section 1613.3.2 of 2013 CBC and Table 20.3-1 of American Society of Civil Engineers(ASCE) 7-10 (2010). The seismic parameters corresponding to Site Class C should also be used for design of the retaining wall at base of cut slopes at the northern boundary of the site.

The Risk-Targeted Maximum Considered Earthquake (MCER) mapped spectral accelerations for 0.2 second and 1 second periods (SS and S1) were estimated using Section 1613.3 of the 2013 CBC and the U.S. Geological Survey (USGS) web based application (available at http://geohazards.usgs.gov/designmaps/us/application.php). The mapped acceleration values and associated soil amplification factors (Fa and Fv) based on the 2013 CBC and corresponding site modified (SMS and SM1) and design spectral accelerations (SDS and SD1) are presented in Table 10 below.

Table 10
Ground Motion Parameters Based On 2013 CBC

Parameter	Value	Reference
Western Area Of The Site – Site Class C, Buildings E, F, G, H, & J		
S_S	2.185	2013 CBC Section 1613.3.1
S_1	0.902	2013 CBC Section 1613.3.1
F_a	1.0	2013 CBC Table 1613.3.3(1)
F_v	1.3	2013 CBC Table 1613.3.3(2)
S_{MS}	2.185	2013 CBC Section 1613.3.3
S_{M1}	1.173	2013 CBC Section 1613.3.3
S_{DS}	1.456	2013 CBC Section 1613.3.4
S_{D1}	0.782	2013 CBC Section 1613.3.4
PGA_M	0.839	ASCE 7-10 Section 11.8.3
Eastern and Southern Areas Of The Site – Site Class D, Buildings A, C, D, & K		
S_S	2.210	2013 CBC Section 1613.3.1
S_1	0.914	2013 CBC Section 1613.3.1
F_a	1.0	2013 CBC Table 1613.3.3(1)
F_v	1.5	2013 CBC Table 1613.3.3(2)
S_{MS}	2.210	2013 CBC Section 1613.3.3
S_{M1}	1.371	2013 CBC Section 1613.3.3
S_{DS}	1.474	2013 CBC Section 1613.3.4
S_{D1}	0.914	2013 CBC Section 1613.3.4
PGA_M	0.853	ASCE 7-10 Section 11.8.3

Seismic Design Category should be taken as “E” for design of the structures since the S1 value is greater than 0.75g. According to Section 1803.5.12 of the 2013 CBC, in the absence of a site-specific ground motion hazard analysis, the MCE geometric mean peak ground acceleration adjusted for Site Class effects (PGAM) can be determined based on Equation 11.8-1 in Section 11.8.3 of ASCE 7-10.

7.11 CORROSION

Three samples of soils from the borings were collected during our field investigation and submitted for corrosion testing. The samples were from borings K-3, K-7 and K111 at depths of about 3 to 6 feet. The soils in this area were selected for corrosion testing because they will likely be in direct contact with concrete or buried metal utility lines, or are representative of other soils at the site. The samples were tested by AP Engineering & Testing, Inc. of Pomona California (borings K-3 and K-7) and CERCO Analytical of Concord, California (boring K111) for pH, resistivity, chloride, sulfide and sulfate. The results are presented in Appendix C.

Kleinfelder has completed laboratory testing to provide data regarding corrosivity of on-site soils. Our scope of services does not include corrosion engineering and, therefore, a detailed analysis of the corrosion test results is not included in this report. However, based upon the resistivity measurement, the soil samples tested can be classified as "corrosive." This classification is consistent with CERCO's brief letter report in Appendix C. A qualified corrosion engineer should be retained to review the test results and design protective systems that may be required. Kleinfelder may be able to provide those services.

Consideration should be given to soils in contact with concrete that will be imported to the site during construction, such as topsoil and landscaping materials. For instance, any imported soil materials should not be any more corrosive than the on-site soils and should not be classified as being more corrosive than "moderately corrosive." Also, on-site cutting and filling may result in soils contacting concrete that were not anticipated at the time of this investigation.

8 LIMITATIONS AND ADDITIONAL SERVICES

The scope of services for this investigation was limited to conducting a reconnaissance of the northern slope, drilling of 22 borings and performing a seismic refraction survey. It should be recognized that definition and evaluation of subsurface conditions are difficult. Judgments leading to conclusions and recommendations are generally made with incomplete knowledge of the subsurface conditions present due to the limitations of data from field studies. The conclusions of this assessment are based on our subsurface exploration including seven borings to depths of between 1½ feet and 36½ feet below the ground surface, the seismic refraction survey, laboratory testing, and engineering analyses.

It is important to understand that it was not within the scope of this current investigation to evaluate the global stability of the overall northern slope. Furthermore, recommendations presented herein and pertaining to stabilization of localized slope portions where cuts are currently proposed are not intended to stabilize higher slope portions. It is also important to understand that slope failures originating higher up the slope may impact lower slope areas where improvements are planned. Slope failures could also occur within topographic hollows and swales where there exists relatively thick loose and unstable soil and rock debris deposits where no current cuts are planned. Such failures could encroach on the new planned perimeter roadway and associated improvements.

Kleinfelder offers various levels of investigative and engineering services to suit the varying needs of different clients. Although risk can never be eliminated, more detailed and extensive studies yield more information, which may help understand and manage the level of risk. Since detailed study and analysis involves greater expense, our clients participate in determining levels of service, which provide information for their purposes at acceptable levels of risk. The client and key members of the design team should discuss the issues covered in this report with Kleinfelder, so that the issues are understood and applied in a manner consistent with the owner's budget, tolerance of risk and expectations for future performance and maintenance.

Recommendations contained in this report are based on our field observations and subsurface explorations, seismic refraction survey, limited laboratory tests, and our present knowledge of the proposed construction. It is possible that soil, rock or groundwater conditions could vary between or beyond the points explored. If soil, rock or groundwater conditions are encountered during construction that differ from those described herein, the client is responsible for ensuring that Kleinfelder is notified immediately so that we may reevaluate the recommendations of this report. If the scope of the proposed construction, including the estimated structural loads, and the design depths or locations of the foundations, changes from that described in this report, the conclusions and recommendations contained in this report are not considered valid unless the changes are reviewed, and the conclusions of this report are modified or approved in writing, by Kleinfelder.

As the geotechnical engineering firm that performed the geotechnical evaluation for this project, Kleinfelder should be retained to confirm that the recommendations of this report are properly incorporated in the design of this project, and properly implemented during construction. This may avoid misinterpretation of the information by other parties and will allow us to review and modify our recommendations if variations in the soil conditions are encountered. As a minimum Kleinfelder should be retained to provide the following continuing services for the project:

- Review any revisions or modifications to the project plans and specifications;
- Observe and evaluate the site earthwork operations to confirm subgrade soils and rock are suitable for construction of foundations, slabs-on-grade, pavements and placement of engineered fill;
- Confirm engineered fill for the structure and other improvements is placed and compacted per the project specifications;
- Observe foundation bearing soils to confirm conditions are as anticipated; and
- Observe installation of aggregate base and flexible and rigid pavements.

The scope of services for this subsurface exploration and geotechnical report did not include environmental assessments or evaluations regarding the presence or absence of wetlands or hazardous substances in the soil, surface water, or groundwater at this site.

Kleinfelder cannot be responsible for interpretation by others of this report or the conditions encountered in the field. Kleinfelder must be retained so that all geotechnical aspects of construction will be monitored on a full-time basis by a representative from Kleinfelder, including site preparation, earthwork grading, preparation of foundations, installation of tie-downs and/or piles or drilled piers, and placement of engineered fill and trench backfill. These services provide Kleinfelder the opportunity to observe the actual soil, rock and groundwater conditions encountered during construction and to evaluate the applicability of the recommendations presented in this report to the site conditions. If Kleinfelder is not retained to provide these services, we will cease to be the engineer of record for this project and will assume no responsibility for any potential claim during or after construction on this project. If changed site conditions affect the recommendations presented herein, Kleinfelder must also be retained to perform a supplemental evaluation and to issue a revision to our original report.

This report, and any future addenda or reports regarding this site, may be made available to bidders to supply them with only the data contained in the report regarding subsurface conditions and laboratory test results at the point and time noted. Bidders may not rely on interpretations, opinion, recommendations, or conclusions contained in the report. Because of the limited nature of any subsurface study, the contractor may encounter conditions during construction which differ from those presented in this report. In such event, the contractor should promptly notify the owner so that Kleinfelder's geotechnical engineer can be contacted to confirm those conditions. We recommend the contractor describe the nature and extent of the differing conditions in writing and that the construction contract include provisions for dealing with differing conditions. Contingency funds should be reserved for potential problems during earthwork and foundation construction.

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PLATES

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APPROXIMATE SCALE (feet)

REFERENCE:
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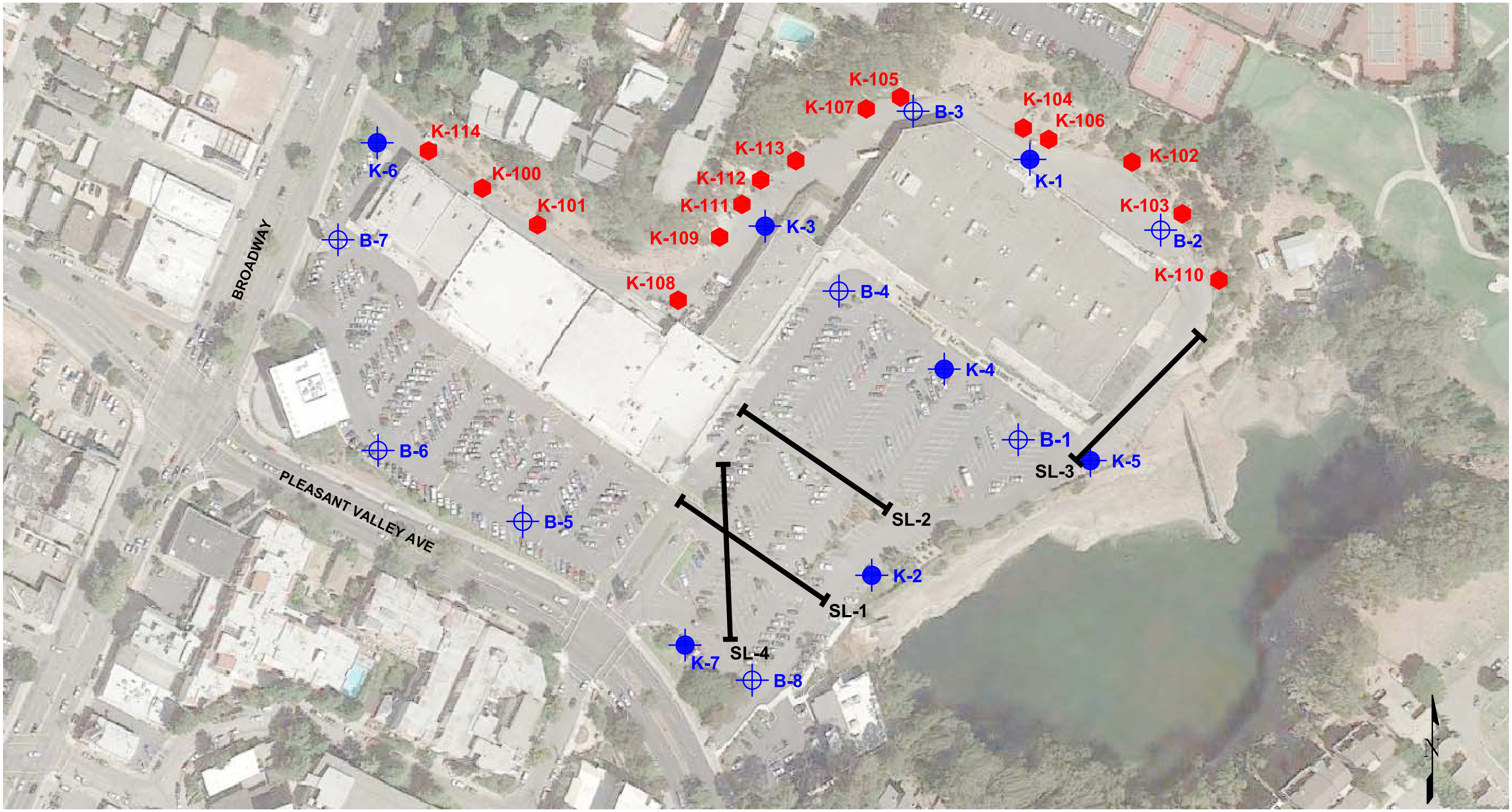
CHECKED BY: DA	PDC ROCKRIDGE SHOPPING CENTER	1
DATE: 10-06-2014	5130 PROADWAY	
REVISED: 10-08-2014	OAKLAND, CALIFORNIA	

CAD FILE: L:\2014\14Projects\00136146.001A - Safeway Rockridge Oakland\GRAPHICs\136146-VicMap.dwg



LEGEND

K-101	APPROXIMATE BORING LOCATION (by Kleinfelder 2014)
K-1	APPROXIMATE BORING LOCATION (by Kleinfelder 2013)
B-8	APPROXIMATE BORING LOCATION (by Kleinfelder 2007)
SL-4	APPROXIMATE LOCATION OF SEISMIC LINE (by Kleinfelder 2013)

200 100 0 200

APPROXIMATE SCALE (feet)

REFERENCE:
GOOGLE EARTH PRO 2013

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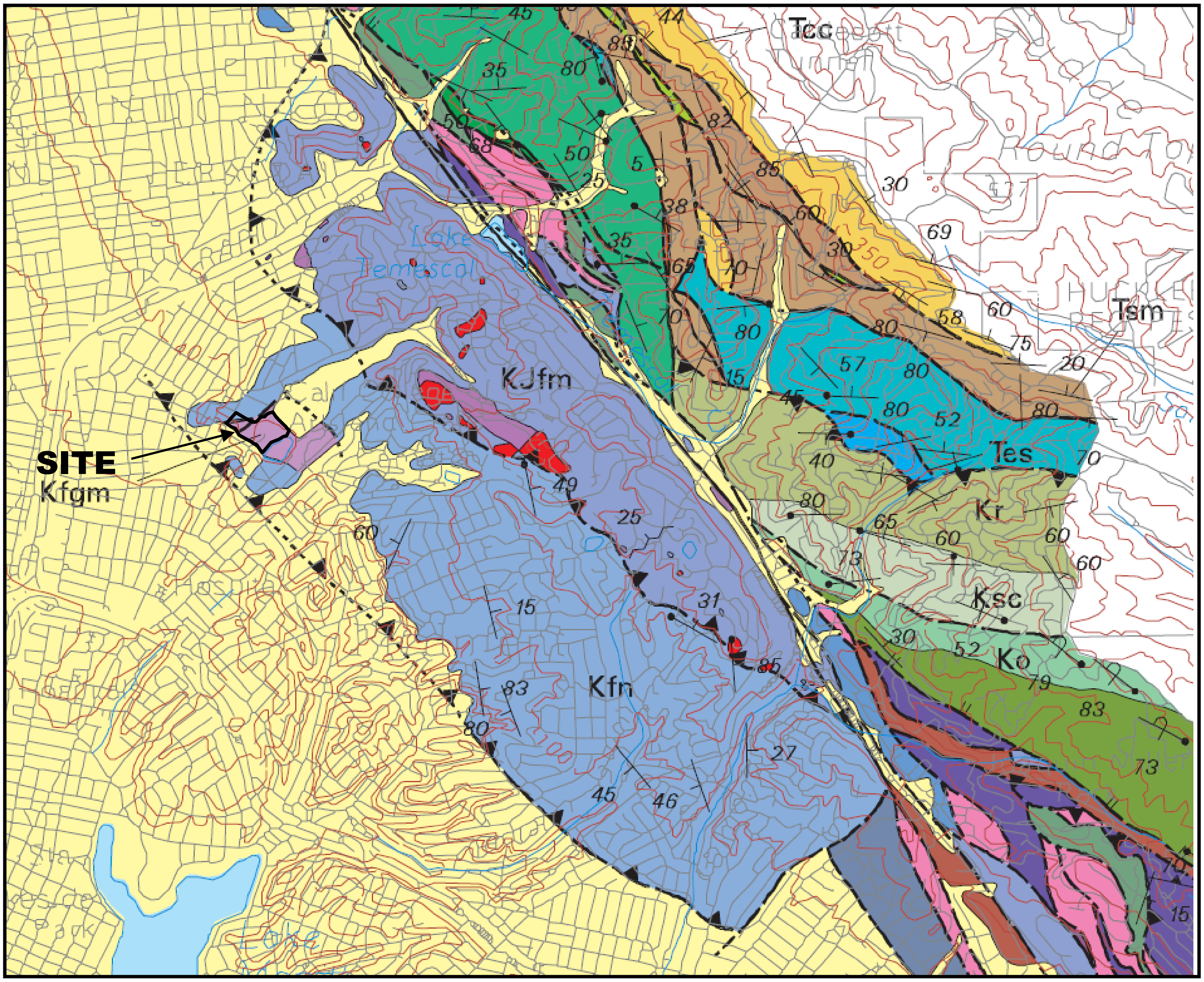
PROJECT NO.	00136146
DRAWN BY:	AG/JDS
CHECKED BY:	DA
DATE:	10-06-2014
REVISED:	10-14-2014

SITE PLAN

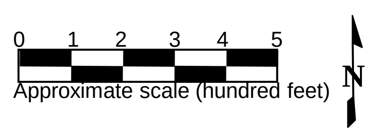
PDC ROCKRIDGE SHOPPING CENTER
5130 BROADWAY
OAKLAND, CALIFORNIA

PLATE
2



Qpaf	Alluvial Terrace Deposits
Kfn	Novato Quarry terrane (sandstone and siltstone)
Kfgm	fine grained quartz diorite

Graymer, R.W., Jones, D.L., and Brabb, E.E., 1997, Preliminary geologic map emphasizing bedrock formations in Alameda County, California: a digital database: U.S. Geological Survey, Open-File Report OF-96-252, scale 1:75000.



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PROJECT NO:	136146
DRAWN:	SEPT 2013
COMPILED BY:	JDS
CHECKED BY:	SD
FILE NAME: AREA GEOLOGIC	

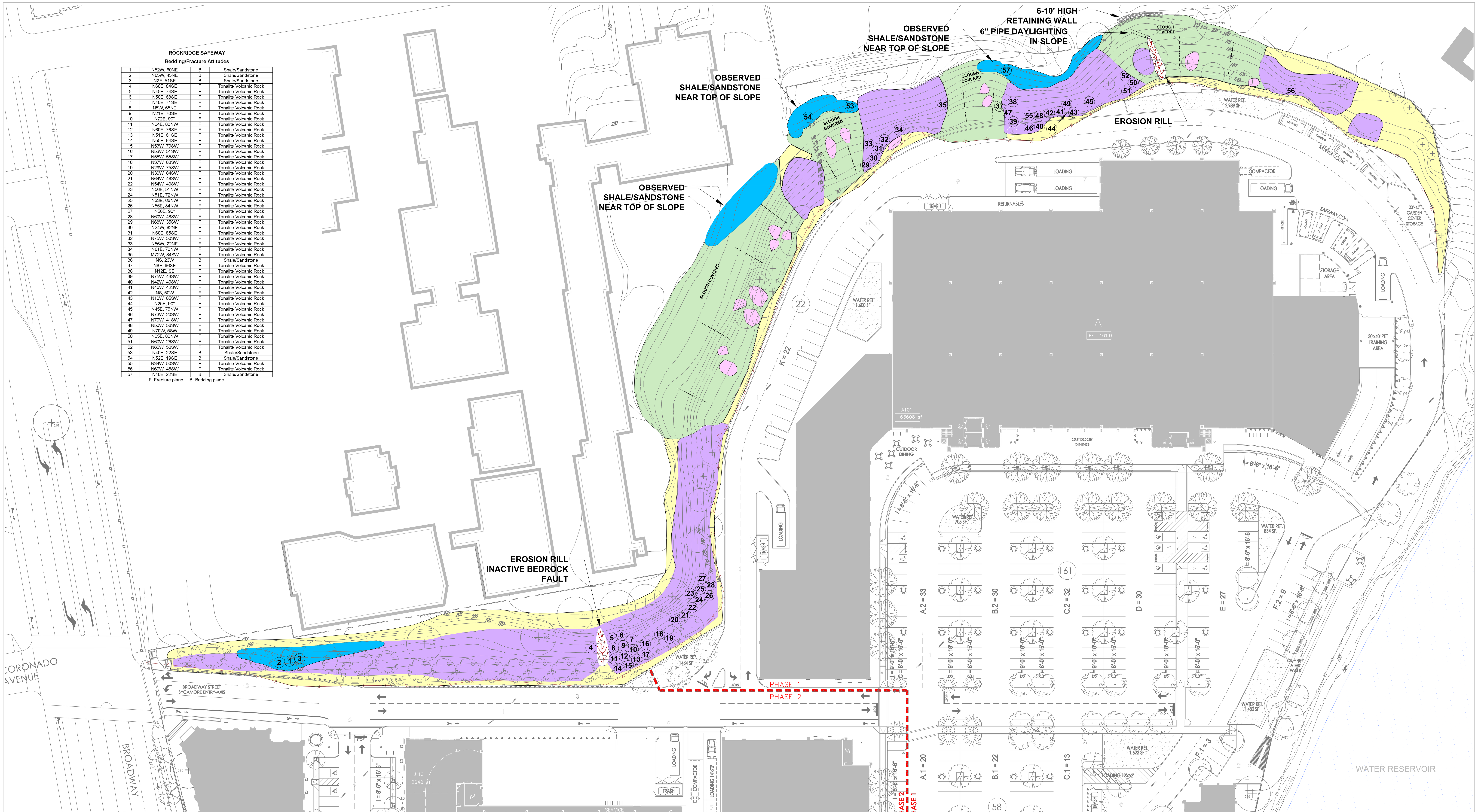
AREA GEOLOGIC MAP

PDC ROCKRIDGE SHOPPING CENTER
5130 BROADWAY
OAKLAND, CALIFORNIA

PLATE

3

Library file: L:\2013\library\projects\136146\AERA GEOLOGIC.ppt



ROCKRIDGE SAFEWAY

Bedding/Fracture Attitudes

1	N52W, 60NE	B	Shale/Sandstone
2	N65W, 45NE	B	Shale/Sandstone
3	N2E, 51SE	B	Shale/Sandstone
4	N60E, 84SE	F	Tonalite Volcanic Rock
5	N45E, 74SE	F	Tonalite Volcanic Rock
6	N50E, 68SE	F	Tonalite Volcanic Rock
7	N40E, 71SE	F	Tonalite Volcanic Rock
8	N5W, 65NE	F	Tonalite Volcanic Rock
9	N21E, 70SE	F	Tonalite Volcanic Rock
10	N72E, 90°	F	Tonalite Volcanic Rock
11	N34E, 80NW	F	Tonalite Volcanic Rock
12	N60E, 76SE	F	Tonalite Volcanic Rock
13	N51E, 61SE	F	Tonalite Volcanic Rock
14	N55E, 64SE	F	Tonalite Volcanic Rock
15	N53W, 70SW	F	Tonalite Volcanic Rock
16	N53W, 51SW	F	Tonalite Volcanic Rock
17	N55W, 55SW	F	Tonalite Volcanic Rock
18	N37W, 83SW	F	Tonalite Volcanic Rock
19	N29W, 75SW	F	Tonalite Volcanic Rock
20	N30W, 84SW	F	Tonalite Volcanic Rock
21	N64W, 48SW	F	Tonalite Volcanic Rock
22	N54W, 40SW	F	Tonalite Volcanic Rock
23	N56E, 51NW	F	Tonalite Volcanic Rock
24	N51E, 72NW	F	Tonalite Volcanic Rock
25	N33E, 66NW	F	Tonalite Volcanic Rock
26	N55E, 84NW	F	Tonalite Volcanic Rock
27	N56E, 90°	F	Tonalite Volcanic Rock
28	N60W, 48SW	F	Tonalite Volcanic Rock
29	N68W, 35SW	F	Tonalite Volcanic Rock
30	N24W, 82NE	F	Tonalite Volcanic Rock
31	N60E, 85SE	F	Tonalite Volcanic Rock
32	N75W, 50SW	F	Tonalite Volcanic Rock
33	N56W, 22NE	F	Tonalite Volcanic Rock
34	N61E, 70NW	F	Tonalite Volcanic Rock
35	M72W, 34SW	F	Tonalite Volcanic Rock
36	NS, 23W	B	Shale/Sandstone
37	N8E, 66SE	F	Tonalite Volcanic Rock
38	N12E, SE	F	Tonalite Volcanic Rock
39	N75W, 43SW	F	Tonalite Volcanic Rock
40	N42W, 40SW	F	Tonalite Volcanic Rock
41	N46W, 42SW	F	Tonalite Volcanic Rock
42	NS, 50W	F	Tonalite Volcanic Rock
43	N10W, 85SW	F	Tonalite Volcanic Rock
44	N25E, 90°	F	Tonalite Volcanic Rock
45	N45E, 75NW	F	Tonalite Volcanic Rock
46	N73W, 20SW	F	Tonalite Volcanic Rock
47	N70W, 41SW	F	Tonalite Volcanic Rock
48	N50W, 56SW	F	Tonalite Volcanic Rock
49	N70W, 5SW	F	Tonalite Volcanic Rock
50	N35E, 80NW	F	Tonalite Volcanic Rock
51	N60W, 26SW	F	Tonalite Volcanic Rock
52	N65W, 50SW	F	Tonalite Volcanic Rock
53	N40E, 22SE	B	Shale/Sandstone
54	N52E, 19SE	B	Shale/Sandstone
55	N34W, 50SW	F	Tonalite Volcanic Rock
56	N60W, 45SW	F	Tonalite Volcanic Rock
57	N40E, 22SE	B	Shale/Sandstone

F: Fracture plane B: Bedding plane

60 30 0 60
APPROXIMATE SCALE (feet)

REFERENCE:
JRDV Architects Inc., Site Plan for 51st & Broadway
Store No. 3132 Oakland, Ca., dated May 22, 2013.

NOTE:
The shown street and building layout is proposed.

LEGEND

- ROCKY TONALITE BEDROCK AREAS
- LOOSE & UNSTABLE DEPOSITS
- THIN COLLUVIAL VENEER
- SHALE/SANDSTONE
- EXISTING CHAIN LINK FENCE LINE
- ROCK FRAGMENTS WITHIN SANDY CLAY MATRIX (FLOAT)
- 1 BEDDING/FRACTURE ATTITUDES (See Table)

NOTE: LOCATIONS ARE APPROXIMATE

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PROJECT NO. 136146	NORTHERN SLOPE GEOLOGIC MAP		PLATE
DRAWN BY JDS			4
CHECKED BY DA	PDC ROCKRIDGE SHOPPING CENTER		
DATE: 10-03-2014	5130 BROADWAY		
REVISED: 10-10-2014	OAKLAND, CALIFORNIA		

APPENDIX A

FIELD EXPLORATIONS

SAMPLE/SAMPLER TYPE GRAPHICS

BULK SAMPLE

CALIFORNIA SAMPLER
(3 in. (76.2 mm.) outer diameter)

STANDARD PENETRATION SPLIT SPOON SAMPLER
(2 in. (50.8 mm.) outer diameter and 1-3/8 in. (34.9 mm.) inner diameter)

GROUND WATER GRAPHICS

WATER LEVEL (level where first observed)

WATER LEVEL (level after exploration completion)

WATER LEVEL (additional levels after exploration)

OBSERVED SEEPAGE

NOTES

1. The report and log key are an integral part of these logs. All data and interpretations in this log are subject to the explanations and limitations stated in the report.

2. Lines separating strata on the logs represent approximate boundaries only. Actual transitions may be gradual or differ from those shown.

3. No warranty is provided as to the continuity of soil or rock conditions between individual sample locations.

4. Logs represent general soil or rock conditions observed at the point of exploration on the date indicated.

5. In general, Unified Soil Classification System designations presented on the logs were based on visual classification in the field and were modified where appropriate based on gradation and index property testing.

6. Fine grained soils that plot within the hatched area on the Plasticity Chart, and coarse grained soils with between 5% and 12% passing the No. 200 sieve require dual USCS symbols, ie., GW-GM, GP-GM, GW-GC, GP-GC, GC-GM, SW-SM, SP-SM, SW-SC, SP-SC, SC-SM.

7. If sampler is not able to be driven at least 6 inches, 50/X indicates number of blows required to drive the identified sampler X inches with a 140 pound hammer falling 30 inches.

UNIFIED SOIL CLASSIFICATION SYSTEM (ASTM D 2487)

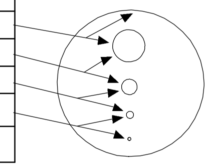
COARSE GRAINED SOILS (More than half of material is larger than the #200 sieve)	GRAVELS (More than half of coarse fraction is larger than the #4 sieve)	CLEAN GRAVEL WITH <5% FINES	Cu ≥4 and 1≤ Cc≤3	GW	WELL-GRADED GRAVELS, GRAVEL-SAND MIXTURES WITH LITTLE OR NO FINES
			Cu <4 and/ or 1>Cc >3	GP	POORLY GRADED GRAVELS, GRAVEL-SAND MIXTURES WITH LITTLE OR NO FINES
		GRAVELS WITH 5% TO 12% FINES	Cu ≥4 and 1≤ Cc≤3	GW-GM	WELL-GRADED GRAVELS, GRAVEL-SAND MIXTURES WITH LITTLE FINES
				GW-GC	WELL-GRADED GRAVELS, GRAVEL-SAND MIXTURES WITH LITTLE CLAY FINES
			Cu <4 and/ or 1>Cc >3	GP-GM	POORLY GRADED GRAVELS, GRAVEL-SAND MIXTURES WITH LITTLE FINES
				GP-GC	POORLY GRADED GRAVELS, GRAVEL-SAND MIXTURES WITH LITTLE CLAY FINES
		GRAVELS WITH > 12% FINES		GM	SILTY GRAVELS, GRAVEL-SILT-SAND MIXTURES
				GC	CLAYEY GRAVELS, GRAVEL-SAND-CLAY MIXTURES
				GC-GM	CLAYEY GRAVELS, GRAVEL-SAND-CLAY-SILT MIXTURES
	SANDS (More than half of coarse fraction is smaller than the #4 sieve)	CLEAN SANDS WITH <5% FINES	Cu ≥6 and 1≤ Cc≤3	SW	WELL-GRADED SANDS, SAND-GRAVEL MIXTURES WITH LITTLE OR NO FINES
			Cu <6 and/ or 1>Cc >3	SP	POORLY GRADED SANDS, SAND-GRAVEL MIXTURES WITH LITTLE OR NO FINES
		SANDS WITH 5% TO 12% FINES	Cu ≥6 and 1≤ Cc≤3	SW-SM	WELL-GRADED SANDS, SAND-GRAVEL MIXTURES WITH LITTLE FINES
				SW-SC	WELL-GRADED SANDS, SAND-GRAVEL MIXTURES WITH LITTLE CLAY FINES
			Cu <6 and/ or 1>Cc >3	SP-SM	POORLY GRADED SANDS, SAND-GRAVEL MIXTURES WITH LITTLE FINES
				SP-SC	POORLY GRADED SANDS, SAND-GRAVEL MIXTURES WITH LITTLE CLAY FINES
		SANDS WITH > 12% FINES		SM	SILTY SANDS, SAND-GRAVEL-SILT MIXTURES
				SC	CLAYEY SANDS, SAND-GRAVEL-CLAY MIXTURES
				SC-SM	CLAYEY SANDS, SAND-SILT-CLAY MIXTURES

FINE GRAINED SOILS (More than half of material is smaller than the #200 sieve)	SILTS AND CLAYS (Liquid Limit less than 50)	ML	INORGANIC SILTS AND VERY FINE SANDS, SILTY OR CLAYEY FINE SANDS, SILTS WITH SLIGHT PLASTICITY
		CL	INORGANIC CLAYS OF LOW TO MEDIUM PLASTICITY, GRAVELLY CLAYS, SANDY CLAYS, SILTY CLAYS, LEAN CLAYS
		CL-ML	INORGANIC CLAYS-SILTS OF LOW PLASTICITY, GRAVELLY CLAYS, SANDY CLAYS, SILTY CLAYS, LEAN CLAYS
		OL	ORGANIC SILTS & ORGANIC SILTY CLAYS OF LOW PLASTICITY
	SILTS AND CLAYS (Liquid Limit greater than 50)	MH	INORGANIC SILTS, MICACEOUS OR DIATOMACEOUS FINE SAND OR SILT
		CH	INORGANIC CLAYS OF HIGH PLASTICITY, FAT CLAYS
		OH	ORGANIC CLAYS & ORGANIC SILTS OF MEDIUM-TO-HIGH PLASTICITY

KLEINFELDER Bright People. Right Solutions.	PROJECT NO.: 136146	GRAPHICS KEY	PLATE
	DRAWN BY: AG		
	CHECKED BY: AJB	PDC ROCKRIDGE SHOPPING CENTER 5130 BROADWAY OAKLAND, CALIFORNIA	A-1
	DATE: 9/3/2013		
	REVISED:		

GRAIN SIZE

DESCRIPTION		SIEVE SIZE	GRAIN SIZE	APPROXIMATE SIZE
Boulders		>12 in. (304.8 mm.)	>12 in. (304.8 mm.)	Larger than basketball-sized
Cobbles		3 - 12 in. (76.2 - 304.8 mm.)	3 - 12 in. (76.2 - 304.8 mm.)	Fist-sized to basketball-sized
Gravel	coarse	3/4 -3 in. (19 - 76.2 mm.)	3/4 -3 in. (19 - 76.2 mm.)	Thumb-sized to fist-sized
	fine	#4 - 3/4 in. (#4 - 19 mm.)	0.19 - 0.75 in. (4.8 - 19 mm.)	Pea-sized to thumb-sized
Sand	coarse	#10 - #4	0.079 - 0.19 in. (2 - 4.9 mm.)	Rock salt-sized to pea-sized
	medium	#40 - #10	0.017 - 0.079 in. (0.43 - 2 mm.)	Sugar-sized to rock salt-sized
	fine	#200 - #10	0.0029 - 0.017 in. (0.07 - 0.43 mm.)	Flour-sized to sugar-sized
Fines		Passing #200	<0.0029 in. (<0.07 mm.)	Flour-sized and smaller

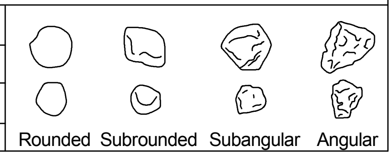


Munsell Color

NAME	ABBR
Red	R
Yellow Red	YR
Yellow	Y
Green Yellow	GY
Green	G
Blue Green	BG
Blue	B
Purple Blue	PB
Purple	P
Red Purple	RP

ANGULARITY

DESCRIPTION	CRITERIA
Angular	Particles have sharp edges and relatively plane sides with unpolished surfaces
Subangular	Particles are similar to angular description but have rounded edges
Subrounded	Particles have nearly plane sides but have well-rounded corners and edges
Rounded	Particles have smoothly curved sides and no edges



PLASTICITY

DESCRIPTION	LL	FIELD TEST
Non-plastic	NP	A 1/8-in. (3 mm.) thread cannot be rolled at any water content.
Low (L)	< 30	The thread can barely be rolled and the lump or thread cannot be formed when drier than the plastic limit.
Medium (M)	30 - 50	The thread is easy to roll and not much time is required to reach the plastic limit. The thread cannot be rerolled after reaching the plastic limit. The lump or thread crumbles when drier than the plastic limit
High (H)	> 50	It takes considerable time rolling and kneading to reach the plastic limit. The thread can be rerolled several times after reaching the plastic limit. The lump or thread can be formed without crumbling when drier than the plastic limit

MOISTURE CONTENT

DESCRIPTION	FIELD TEST
Dry	Absence of moisture, dusty, dry to the touch
Moist	Damp but no visible water
Wet	Visible free water, usually soil is below water table

REACTION WITH HYDROCHLORIC ACID

DESCRIPTION	FIELD TEST
None	No visible reaction
Weak	Some reaction, with bubbles forming slowly
Strong	Violent reaction, with bubbles forming immediately

APPARENT / RELATIVE DENSITY - COARSE-GRAINED SOIL

APPARENT DENSITY	SPT-N_{60} (# blows/ft)	MODIFIED CA SAMPLER (# blows/ft)	CALIFORNIA SAMPLER (# blows/ft)	RELATIVE DENSITY (%)
Very Loose	<4	<4	<5	0 - 15
Loose	4 - 10	5 - 12	5 - 15	15 - 35
Medium Dense	10 - 30	12- 35	15 - 40	35 - 65
Dense	30 - 50	35 - 60	40 - 70	65 - 85
Very Dense	>50	>60	>70	85 - 100

NOTE: AFTER TERZAGHI AND PECK, 1948

CONSISTENCY - FINE-GRAINED SOIL

CONSISTENCY	UNCONFINED COMPRESSIVE STRENGTH (Qu)(psf)	CRITERIA
Very Soft	< 1000	Thumb will penetrate soil more than 1 in. (25 mm.)
Soft	1000 - 2000	Thumb will penetrate soil about 1 in. (25 mm.)
Firm	2000 < 4000	Thumb will indent soil about 1/4-in. (6 mm.)
Hard	4000 < 8000	Thumb will not indent soil but readily indented with thumbnail
Very Hard	> 8000	Thumbnail will not indent soil

STRUCTURE

DESCRIPTION	CRITERIA
Stratified	Alternating layers of varying material or color with layers at least 1/4-in. thick, note thickness
Laminated	Alternating layers of varying material or color with the layer less than 1/4-in. thick, note thickness
Fissured	Breaks along definite planes of fracture with little resistance to fracturing
Slickensided	Fracture planes appear polished or glossy, sometimes striated
Blocky	Cohesive soil that can be broken down into small angular lumps which resist further breakdown
Lensed	Inclusion of small pockets of different soils, such as small lenses of sand scattered through a mass of clay; note thickness
Homogeneous	Same color and appearance throughout

CEMENTATION

DESCRIPTION	FIELD TEST
Weakly	Crumbles or breaks with handling or slight finger pressure
Moderately	Crumbles or breaks with considerable finger pressure
Strongly	Will not crumble or break with finger pressure

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PROJECT NO.: 136146	SOIL DESCRIPTION KEY	PLATE
DRAWN BY: AG		
CHECKED BY: AJB	PDC ROCKRIDGE SHOPPING CENTER 5130 BROADWAY OAKLAND, CALIFORNIA	A-2
DATE: 9/3/2013		
REVISED:		

INFILLING TYPE

NAME	ABBR	NAME	ABBR
Albite	Al	Muscovite	Mus
Apatite	Ap	None	No
Biotite	Bi	Pyrite	Py
Clay	Cl	Quartz	Qz
Calcite	Ca	Sand	Sa
Chlorite	Ch	Sericite	Ser
Epidote	Ep	Silt	Si
Iron Oxide	Fe	Talc	Ta
Manganese	Mn	Unknown	Uk

BEDDING CHARACTERISTICS

TERM	Thickness (in.)	Thickness (mm.)
Very Thick Bedded	> 36	> 915
Thick Bedded	12 - 36	305 - 915
Medium Bedded	4 - 12	102 - 305
Thin Bedded	1 - 4	25 - 102
Very Thin Bedded	0.4 - 1	10 - 25
Laminated	0.1 - 0.4	2.5 - 10
Thinly Laminated	< 0.1	< 2.5

Bedding Planes Planes dividing the individual layers, beds, or stratigraphy of rocks.
Joint Fracture in rock, generally more or less vertical or traverse to bedding.
Seam Applies to bedding plane with unspecified degree of weather.

DENSITY/SPACING OF DISCONTINUITIES

DESCRIPTION	SPACING CRITERIA
Unfractured	> 6 ft. (> 1.83 meters)
Slightly Fractured	2 - 6 ft. (.061 - 1.83 meters)
Moderately Fractured	8 in - 2 ft. (203.20 - 609.60 mm.)
Highly Fractured	2 - 8 in. (50.80 - 203.30 mm.)
Intensely Fractured	< 2 in. (< 50.80 mm.)

APERTURE

DESCRIPTION	CRITERIA [in.(mm.)]
Tight	< 0.04 (< 1)
Open	0.04 - 0.20 (1 - 5)
Wide	> 0.20 (> 5)

ADDITIONAL TEXTURAL ADJECTIVES

DESCRIPTION	RECOGNITION
Pit (Pitted)	Pinhole to 0.03 ft. (3/8 in.) (>1 to 10 mm.) openings
Vug (Vuggy)	Small openings (usually lined with crystals) ranging in diameter from 0.03 ft. (3/8 in.) to 0.33 ft. (4 in.) (10 to 100 mm.)
Cavity	An opening larger than 0.33 ft. (4 in.) (100 mm.), size descriptions are required, and adjectives such as small, large, etc., may be used
Honeycombed	If numerous enough that only thin walls separate individual pits or vugs, this term further describes the preceding nomenclature to indicate cell-like form
Vesicle (Vesicular)	Small openings in volcanic rocks of variable shape and size formed by entrapped gas bubbles during solidification

DISCONTINUITY TYPE

DESCRIPTION
Fault
Joint
Shear
Foliation
Vein
Bedding

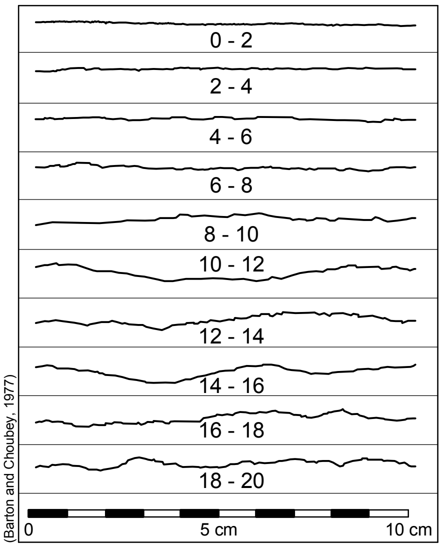
INFILLING AMOUNT

DESCRIPTION
Surface Stain
Spotty
Partially Filled
Filled
None

ROCK QUALITY DESIGNATION (RQD)

DESCRIPTION	RQD (%)
Very Poor	0 - 25
Poor	25 - 50
Fair	50 - 75
Good	75 - 90
Excellent	90 - 100

JOINT ROUGHNESS COEFFICIENT (JRC)



RQD Rock-quality designation (RQD) Rough measure of the degree of jointing or fracture in a rock mass, measured as a percentage of the drill core in lengths of 10 cm. or more.

DEGREES OF WEATHERING

DESCRIPTION	CRITERIA
Unweathered	No evidence of chemical/mechanical alternation; rings with hammer blow.
Slightly Weathered	Slight discoloration on surface; slight alteration along discontinuities; <10% rock volume altered.
Moderately Weathered	Discoloring evident; surface pitted and alteration penetration well below surface; Weathering "halos" evident; 10-50% rock altered.
Highly Weathered	Entire mass discolored; Alteration pervading most rock, some slight weathering pockets; some minerals may be leached out.
Decomposed	Rock reduced to soil with relict rock texture/structure; Generally molded and crumbled by hand.

RELATIVE HARDNESS / STRENGTH DESCRIPTIONS

GRADE		UCS (MPa)	FIELD TEST
R0	Extremely Weak	0.25 - 1.0	Indented by thumbnail
R1	Very Weak	1.0 - 5.0	Crumbles under firm blows of geological hammer, can be peeled by a pocket knife
R2	Weak	5.0 - 25	Can be peeled by a pocket knife with difficulty, shallow indentations made by firm blow with point of geological hammer
R3	Medium Strong	25 - 50	Cannot be scraped or peeled with a pocket knife, specimen can be fractured with a single firm blow of a geological hammer
R4	Strong	50 - 100	Specimen requires more than one blow of geological hammer to fracture it
R5	Very Strong	100 - 250	Specimen requires many blows of geological hammer to fracture it
R6	Extremely Strong	> 250	Specimen can only be chipped with a geological hammer

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PROJECT NO.: 136146	ROCK DESCRIPTION KEY	PLATE
DRAWN BY: AG		
CHECKED BY: AJB	PDC ROCKRIDGE SHOPPING CENTER 5130 BROADWAY OAKLAND, CALIFORNIA	A-3
DATE: 9/3/2013		
REVISED:		

Date Begin - End:	9/03/2013	**Drill Company:**	Exploration Geo		**BORING LOG K-1**
Logged By:	AJB	**Drill Crew:**	Loren & Dee		
Hor.-Vert. Datum:	Not Available	**Drill Equipment:**	Mobile B53 Blue	**Hammer Type - Drop:**	140 lb. Cathead - 30 in.
Exploration Plunge:	-90 degrees	**Exploration Method:**	Hollow Stem Auger		
Weather:		**Bit Type - Auger Dia.:**	8" - 8" in. O.D.		

FIELD EXPLORATION / LABORATORY RESULTS

No Coordinates Available
No Elevation Available
Location Offset: See Site Plan
Surface Condition: Asphalt

Depth (feet)	Graphical Log	Description	Sample Number	Exploration Method	Sample Type	Blow Counts(BC)= Uncorr. blows/6 in.	Recovery (NR=No Recovery)	USCS Symbol	Water Content (%)	Dry Density (pcf)	Passing No.4 Sieve (%)	Passing #200 Sieve (%)	Liquid Limit (NV=No Value)	Plasticity Index (NP=No Plasticity)	Other Tests/ Remarks
		5 inches asphalt concrete over 10 inches aggregate base													
		Gravelly Lean CLAY (CL): medium plasticity, brown, moist, firm to hard, subrounded to subangular fine to coarse gravel, with some fine to coarse grained sand (FILL)	1C 1B 1A			BC=25 20 18	100%	CL	10.1	117		27	37	19	
5		gray and brown with some cobbles	2C 2B 2A			BC=18 18 20	28%								
10		**Lean CLAY with Gravel (CL)**: medium plasticity, dark brown, moist, firm to hard, fine to coarse rounded to subangular gravel, fine to coarse grained sand, with some asphalt (FILL)	3C 3B 3A			BC=20 20 29	100%	CL	16.6	112					
															Obstruction encountered.
			4A			BC=7 50/3""	NR								
15															
20															
25															
30															

The exploration was terminated at approximately 14.5 ft. below ground surface. The exploration was backfilled with bentonite on September 03, 2013.

GROUNDWATER LEVEL INFORMATION:
Groundwater was not encountered during drilling or after completion.

KLEINFELDER Bright People. Right Solutions.	PROJECT NO.:	136146	BORING LOG K-1	PLATE
	DRAWN BY:	AG		
	CHECKED BY:	AJB	PDC ROCKRIDGE SHOPPING CENTER 5130 BROADWAY OAKLAND, CALIFORNIA	A-4
	DATE:	9/3/2013		
	REVISED:			PAGE: 1 of 1

Date Begin - End:	9/03/2013	**Drill Company:**	Exploration Geo		**BORING LOG K-2**
Logged By:	AJB	**Drill Crew:**	Loren & Dee		
Hor.-Vert. Datum:	Not Available	**Drill Equipment:**	Mobile B53 Blue	**Hammer Type - Drop:**	140 lb. Cathead - 30 in.
Exploration Plunge:	-90 degrees	**Exploration Method:**	Hollow Stem Auger		
Weather:		**Bit Type - Auger Dia.:**	8" - 8" in. O.D.		

FIELD EXPLORATION / LABORATORY RESULTS

No Coordinates Available
No Elevation Available
Location Offset: See Site Plan
Surface Condition: Asphalt

Depth (feet)	Graphical Log	Description	Sample Number	Exploration Method	Sample Type	Blow Counts(BC)= Uncorr. blows/6 in.	Recovery (NR=No Recovery)	USCS Symbol	Water Content (%)	Dry Density (pcf)	Passing No.4 Sieve (%)	Passing #200 Sieve (%)	Liquid Limit (NV=No Value)	Plasticity Index (NP=No Plasticity)	Other Tests/ Remarks
		4 inches asphalt concrete over 3 inches aggregate base													
		Clayey GRAVEL (GC): medium plasticity, brown, moist, medium dense, fine to coarse subangular gravel, with some fine to coarse grained sand (FILL)	1A			BC=5/50"	NR	GC							
5		**Lean CLAY (CL)**: medium plasticity, dark brown, moist, hard, fine to coarse subangular gravel, fine to coarse grained sand (FILL) with some dark gray	2C 2B 2A			BC=25 10 17	61%	CL	16.6	110		53	34	20	
10		medium plasticity, moist, firm to hard	3C 3B 3A			BC=9 11 14	94%		15.9	112					
15		medium to high plasticity, moist to wet	4C 4B 4A			BC=4 5 5	66%								
20			5C 5B 5A			BC=4 4 5	100%								
25		**Clayey SAND (SC)**: dark brown, wet, medium dense, fine to medium grained sand, with some subrounded weathered sandstone gravel (FILL)	6C 6B 6A			BC=20 15 11	66%		14.0	111		29			
		Poorly-Graded SAND with Clay and Gravel (SP-SC): wet, medium dense, fine to coarse grained sand, fine to coarse subrounded gravel (FILL)	7A			BC=26 50/1""	100%	SP-SC							
30		loose	8A			BC=6 4 4	100%								
		dense	9A			BC=8 12 24	NR								

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PROJECT NO.: 136146	BORING LOG K-2		PLATE
DRAWN BY: AG			A-5
CHECKED BY: AJB	PDC ROCKRIDGE SHOPPING CENTER 5130 BROADWAY OAKLAND, CALIFORNIA		
DATE: 9/3/2013			
REVISED:			PAGE: 1 of 2

Date Begin - End:	9/03/2013	**Drill Company:**	Exploration Geo		**BORING LOG K-2**
Logged By:	AJB	**Drill Crew:**	Loren & Dee		
Hor.-Vert. Datum:	Not Available	**Drill Equipment:**	Mobile B53 Blue	**Hammer Type - Drop:**	140 lb. Cathead - 30 in.
Exploration Plunge:	-90 degrees	**Exploration Method:**	Hollow Stem Auger		
Weather:		**Bit Type - Auger Dia.:**	8" - 8" in. O.D.		

Depth (feet)	Graphical Log	FIELD EXPLORATION: No Coordinates Available No Elevation Available Location Offset: See Site Plan Surface Condition: Asphalt	Sample Number	Exploration Method	Sample Type	Blow Counts(BC)= Uncorr. blows/6 in.	Recovery (NR=No Recovery)	LABORATORY RESULTS: USCS Symbol	Water Content (%)	Dry Density (pcf)	Passing No.4 Sieve (%)	Passing #200 Sieve (%)	Liquid Limit (NV=No Value)	Plasticity Index (NP=No Plasticity)	Other Tests/ Remarks
		very dense, with trace organics	10A			BC=20 35 36	100%								

The exploration was terminated at approximately 36.5 ft. below ground surface. The exploration was backfilled with bentonite on September 03, 2013.

GROUNDWATER LEVEL INFORMATION:
Groundwater was observed at approximately 17.5 ft. below ground surface during drilling.

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KLEINFELDER Bright People. Right Solutions.	PROJECT NO.: 136146 DRAWN BY: AG CHECKED BY: AJB DATE: 9/3/2013 REVISED:	BORING LOG K-2 PDC ROCKRIDGE SHOPPING CENTER 5130 BROADWAY OAKLAND, CALIFORNIA	PLATE A-5 PAGE: 2 of 2

Date Begin - End:	9/03/2013	**Drill Company:**	Exploration Geo
Logged By:	AJB	**Drill Crew:**	Loren & Dee
Hor.-Vert. Datum:	Not Available	**Drill Equipment:**	Mobile B53 Blue
Exploration Plunge:	-90 degrees	**Exploration Method:**	Hollow Stem Auger
Weather:		**Bit Type - Auger Dia.:**	8" - 8" in. O.D.

BORING LOG K-3

Hammer Type - Drop: 140 lb. Cathead - 30 in.

Depth (feet)	Graphical Log	FIELD EXPLORATION: No Coordinates Available No Elevation Available Location Offset: See Site Plan Surface Condition: Asphalt	Sample Number	Exploration Method	Sample Type	Blow Counts(BC)= Uncorr. blows/6 in.	Recovery (NR=No Recovery)	LABORATORY RESULTS: USCS Symbol	Water Content (%)	Dry Density (pcf)	Passing No.4 Sieve (%)	Passing #200 Sieve (%)	Liquid Limit (NV=No Value)	Plasticity Index (NP=No Plasticity)	Other Tests/ Remarks
		5 inches asphalt concrete over 8 inches aggregate base													
		Clayey GRAVEL (GC): medium plasticity, brown, moist, dense, fine to coarse subrounded to angular gravel, with fine to coarse grained sand, with some cobbles (FILL)	1C 1B 1A			BC=15 30 24	66%	GC							
		brown with gray													
5		**Gravelly Lean CLAY (CL)**: medium plasticity, brown with gray, moist, firm to hard, fine to coarse grained sand, fine to coarse subangular gravel, with some cobbles (FILL)	2C 2B 2A			BC=10 17 21	100%	CL	12.3	118		33	34	15	
		gray with some dark brown, less gravel	3C 3B 3A			BC=21 24 20	100%		13.2	110					
10															
		greenish gray with red and dark brown, more sand with some concrete	4C 4B 4A			BC=20 28 12	100%								
15															

The exploration was terminated at approximately 15 ft. below ground surface. The exploration was backfilled with bentonite on September 03, 2013.

GROUNDWATER LEVEL INFORMATION:
Groundwater was not encountered during drilling or after completion.

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Date Begin - End:	9/03/2013	**Drill Company:**	Exploration Geo		**BORING LOG K-4**
Logged By:	AJB	**Drill Crew:**	Loren & Dee		
Hor.-Vert. Datum:	Not Available	**Drill Equipment:**	Mobile B53 Blue	**Hammer Type - Drop:**	140 lb. Cathead - 30 in.
Exploration Plunge:	-90 degrees	**Exploration Method:**	Hollow Stem Auger		
Weather:		**Bit Type - Auger Dia.:**	8" - 8" in. O.D.		

FIELD EXPLORATION / LABORATORY RESULTS

No Coordinates Available
No Elevation Available
Location Offset: See Site Plan
Surface Condition: Asphalt

Depth (feet)	Graphical Log	Description	Sample Number	Exploration Method	Sample Type	Blow Counts(BC)= Uncorr. blows/6 in.	Recovery (NR=No Recovery)	USCS Symbol	Water Content (%)	Dry Density (pcf)	Passing No.4 Sieve (%)	Passing #200 Sieve (%)	Liquid Limit (NV=No Value)	Plasticity Index (NP=No Plasticity)	Other Tests/ Remarks
		4 inches asphalt concrete over 8 inches aggregate base													
		Clayey GRAVEL (GC): medium plasticity, brown, moist, dense, fine to coarse subrounded to subangular gravel, fine to coarse grained sand (FILL)						GC							
			1C 1B 1A			BC=24 25 31	66%		8.5	129		24			
5			2C 2B 2A			BC=32 19 17	66%								
		Lean CLAY with Sand and Gravel (CL): medium plasticity, dark brown, moist, firm to hard, fine to coarse subrounded to subangular gravel, fine to coarse grained sand (FILL)						CL	11.3	115					
			3C 3B 3A			BC=3 4 4	66%								
10															
			4C 4B 4A			BC=7 15 20	66%								
		organics layer, black, lots of organics							20.0	97					
15		no organics, trace asphalt													

The exploration was terminated at approximately 15 ft. below ground surface. The exploration was backfilled with bentonite on September 03, 2013.

GROUNDWATER LEVEL INFORMATION:
Groundwater was not encountered during drilling or after completion.

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PROJECT NO.: 136146	BORING LOG K-4	PLATE
DRAWN BY: AG		
CHECKED BY: AJB	PDC ROCKRIDGE SHOPPING CENTER 5130 BROADWAY OAKLAND, CALIFORNIA	A-7
DATE: 9/3/2013		
REVISED:		PAGE: 1 of 1

Date Begin - End:	9/03/2013	**Drill Company:**	Exploration Geo		**BORING LOG K-5**
Logged By:	AJB	**Drill Crew:**	Loren & Dee		
Hor.-Vert. Datum:	Not Available	**Drill Equipment:**	Mobile B53 Blue	**Hammer Type - Drop:**	140 lb. Cathead - 30 in.
Exploration Plunge:	-90 degrees	**Exploration Method:**	Hollow Stem Auger		
Weather:		**Bit Type - Auger Dia.:**	8" - 8" in. O.D.		

		FIELD EXPLORATION						LABORATORY RESULTS							
Depth (feet)	Graphical Log	No Coordinates Available No Elevation Available Location Offset: See Site Plan Surface Condition: Asphalt	Sample Number	Exploration Method	Sample Type	Blow Counts(BC)= blows/6 in.	Recovery (NR=No Recovery)	USCS Symbol	Water Content (%)	Dry Density (pcf)	Passing No.4 Sieve (%)	Passing #200 Sieve (%)	Liquid Limit (NV=No Value)	Plasticity Index (NP=No Plasticity)	Other Tests/ Remarks
		4 inches asphalt concrete over 2 inches aggregate base						GC							Auger refusal at 1.5 feet. Tried three locations to confirm rock.
		Clayey GRAVEL (GC): medium plasticity, brown, moist, very dense, fine to coarse subangular gravel, fine to coarse grained sand (FILL)													
5		**SANDSTONE**: gray, high strength, slightly weathered to fresh, fine to coarse grained sand													
		The exploration was terminated at approximately 1.5 ft. below ground surface. The exploration was backfilled with bentonite on September 03, 2013.													
10															
15															
20															
25															
30															

GROUNDWATER LEVEL INFORMATION:
Groundwater was not encountered during drilling or after completion.

KLEINFELDER Bright People. Right Solutions.	PROJECT NO.: 136146 DRAWN BY: AG CHECKED BY: AJB DATE: 9/3/2013 REVISED:	BORING LOG K-5 PDC ROCKRIDGE SHOPPING CENTER 5130 BROADWAY OAKLAND, CALIFORNIA	PLATE A-8 PAGE: 1 of 1

Date Begin - End:	9/03/2013	**Drill Company:**	Exploration Geo		**BORING LOG K-6**
Logged By:	AJB	**Drill Crew:**	Loren & Dee		
Hor.-Vert. Datum:	Not Available	**Drill Equipment:**	Mobile B53 Blue	**Hammer Type - Drop:**	140 lb. Cathead - 30 in.
Exploration Plunge:	-90 degrees	**Exploration Method:**	Hollow Stem Auger		
Weather:		**Bit Type - Auger Dia.:**	8" - 8" in. O.D.		

Depth (feet)	Graphical Log	FIELD EXPLORATION: No Coordinates Available / No Elevation Available / Location Offset: See Site Plan / Surface Condition: Asphalt	Sample Number	Exploration Method	Sample Type	Blow Counts(BC)= Uncorr. blows/6 in.	Recovery (NR=No Recovery)	LABORATORY RESULTS: USCS Symbol	Water Content (%)	Dry Density (pcf)	Passing No.4 Sieve (%)	Passing #200 Sieve (%)	Liquid Limit (NV=No Value)	Plasticity Index (NP=No Plasticity)	Other Tests/ Remarks
		5 inches asphalt concrete over 9 inches aggregate base													
		SANDSTONE: pale brown, low strength, highly weathered, fine to coarse grained sand	1A			BC=50/2""	100%								
5		pale brown and gray, low to medium strength													Auger refusal
		gray with some brown, highly weathered to slightly weathered	2A			BC=50/3""	100%								

The exploration was terminated at approximately 5.5 ft. below ground surface. The exploration was backfilled with bentonite on September 03, 2013.

GROUNDWATER LEVEL INFORMATION:
Groundwater was not encountered during drilling or after completion.

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PROJECT NO.: 136146	BORING LOG K-6	PLATE
DRAWN BY: AG		
CHECKED BY: AJB	PDC ROCKRIDGE SHOPPING CENTER 5130 BROADWAY OAKLAND, CALIFORNIA	A-9
DATE: 9/3/2013		
REVISED:		PAGE: 1 of 1

Date Begin - End:	9/03/2013	**Drill Company:**	Exploration Geo		**BORING LOG K-7**
Logged By:	AJB	**Drill Crew:**	Loren & Dee		
Hor.-Vert. Datum:	Not Available	**Drill Equipment:**	Mobile B53 Blue	**Hammer Type - Drop:**	140 lb. Cathead - 30 in.
Exploration Plunge:	-90 degrees	**Exploration Method:**	Hollow Stem Auger		
Weather:		**Bit Type - Auger Dia.:**	8" - 8" in. O.D.		

FIELD EXPLORATION / LABORATORY RESULTS

No Coordinates Available
No Elevation Available
Location Offset: See Site Plan

Surface Condition: Asphalt

Depth (feet)	Graphical Log	Description	Sample Number	Exploration Method	Sample Type	Blow Counts(BC)= Uncorr. blows/6 in.	Recovery (NR=No Recovery)	USCS Symbol	Water Content (%)	Dry Density (pcf)	Passing No.4 Sieve (%)	Passing #200 Sieve (%)	Liquid Limit (NV=No Value)	Plasticity Index (NP=No Plasticity)	Other Tests/ Remarks
		4 inches asphalt concrete over 8 aggregate base													
		Clayey GRAVEL (GC): medium plasticity, brown, dry, very dense, fine to coarse subrounded to angular gravel, with some cobbles, fine to coarse grained sand (FILL)	1C 1B 1A			BC=20 22 16	66%	CL							
5		**Gravelly Lean CLAY (CL)**: medium plasticity, red brown with gray, moist, firm to hard, fine to coarse grained sand, fine to coarse subrounded to subangular gravel, with gray sandstone gravel (FILL) brown	2C 2B 2A			BC=18 20 30	66%	GC	6.3	114		18	26	12	
		Sandy Lean CLAY (CL): medium plasticity, dark gray, moist, firm to hard, fine to coarse grained sand, fine to coarse subangular to angular gravel (FILL)	3C 3B 3A			BC=15 6 7	66%	CL	10.5	116					
10															
		less sand	4C 4B 4A			BC=6 10 24	33%								
15															

The exploration was terminated at approximately 15 ft. below ground surface. The exploration was backfilled with bentonite on September 03, 2013.

GROUNDWATER LEVEL INFORMATION:
Groundwater was not encountered during drilling or after completion.

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PROJECT NO.:	136146	BORING LOG K-7	PLATE
DRAWN BY:	AG		A-10
CHECKED BY:	AJB	PDC ROCKRIDGE SHOPPING CENTER 5130 BROADWAY OAKLAND, CALIFORNIA	
DATE:	9/3/2013		
REVISED:			PAGE: 1 of 1

Date Begin - End:	9/09/2014	**Drilling Company:**	Woodward		**BORING LOG K100**
Logged By:	O. Khan	**Drill Crew:**	German/Joe		
Hor.-Vert. Datum:	Not Available	**Drilling Equipment:**	BK 81	**Hammer Type - Drop:**	140 lb. Auto - 30 in.
Plunge:	-90 degrees	**Drilling Method:**	Hollow Stem Auger		
Weather:	overcast	**Bore Diameter:**	8 in. O.D.		

FIELD EXPLORATION / LABORATORY RESULTS

Coordinates Not Available
Ground Surface Elevation Not Available
Surface Condition: Asphalt

Depth (feet)	Graphical Log	Lithologic Description	Sample Type	Blow Counts(BC)= Uncorr. Blows/6 in. Pocket Pen(PP)= tsf	Recovery (NR=No Recovery)	USCS Symbol	Water Content (%)	Dry Unit Wt. (pcf)	Passing #4 (%)	Passing #200 (%)	Liquid Limit	Plasticity Index (NP=NonPlastic)	Additional Tests/ Remarks
		approximate 8-inches of asphalt											
		approximate 6-inches of concrete											
		Clayey SAND with Gravel (SC): fine to medium grained, gray, moist, fine angular gravel (FILL)											
		Poorly-graded SAND (SP): fine grained, light yellowish brown, moist, (FILL)		BC=6 7	12"					1.5			
5				BC=1 2 3	3"								
		SHALE: gray, slightly weathered, weak, highly fractured to intensely fractured, medium plasticity		BC=4 19 32									
10		**TONALITE**: gray, highly to intensely fractured, fine grain, fractures filled with quartz			12"								switched to rock coring at 10'
													hard coring
15													
20													

The exploration was terminated at approximately 11.5 ft. below ground surface. The exploration was backfilled with grout and capped with concrete on September 09, 2014. Rock was encountered at a depth of 6.5 ft. during this exploration.

GROUNDWATER LEVEL INFORMATION:
Groundwater was not encountered during drilling or after completion.
GENERAL NOTES:

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PROJECT NO.: 00136146	BOARING LOG K100	PLATE
DRAWN BY: JDS		
CHECKED BY: OK	ROCKRIDGE SAFEWAY 51st & BROADWAY STORE NO. 3132 OAKLAND, CALIFORNIA	A-11
DATE: 9/24/2014		
REVISED: -		PAGE: 1 of 1

Date Begin - End:	9/09/2014	**Drilling Company:**	Woodward		**BORING LOG K101**
Logged By:	O. Khan	**Drill Crew:**	German/Joe		
Hor.-Vert. Datum:	Not Available	**Drilling Equipment:**	BK 81	**Hammer Type - Drop:**	140 lb. Auto - 30 in.
Plunge:	-90 degrees	**Drilling Method:**	Hollow Stem Auger		
Weather:	clear hot	**Bore Diameter:**	8 in. O.D.		

FIELD EXPLORATION / LABORATORY RESULTS

Coordinates Not Available
Ground Surface Elevation Not Available
Surface Condition: Asphalt

Depth (feet)	Graphical Log	Lithologic Description	Sample Type	Blow Counts(BC)= Uncorr. Blows/6 in. Pocket Pen(PP)= tsf	Recovery (NR=No Recovery)	USCS Symbol	Water Content (%)	Dry Unit Wt. (pcf)	Passing #4 (%)	Passing #200 (%)	Liquid Limit	Plasticity Index (NP=NonPlastic)	Additional Tests/ Remarks
		approximate 8-inches of asphalt											
		Well-graded GRAVEL with Sand (GW-GC): fine to medium grained, light yellowish brown, dry, fine to coarse subangular gravel, rock fragments (FILL)		BC=50-6"	6"								hard drilling
		Clayey SAND with Gravel (SC): fine to medium grained, yellow, dry, very dense, fine subangular gravel (FILL)											
		Rock Fragments within a **Clayey SAND matrix**: fine to medium grained, olive gray to gray, dry, fine to coarse subangular gravel, cobbles (FILL)		BC=43 21 38	9"								hard drilling
5				BC=50-3"									
		Poorly-graded SAND (SP): fine grained, yellowish brown, dry to moist, trace fine gravel (FILL)											
		SANDSTONE: fine grain, gray, unweathered, medium strong, intensely to highly fractured											

The exploration was terminated at approximately 6.5 ft. below ground surface. The exploration was backfilled with grout and capped with concrete on September 09, 2014. Rock was encountered at a depth of 5 ft. during this exploration.

GROUNDWATER LEVEL INFORMATION:
Groundwater was not encountered during drilling or after completion.
GENERAL NOTES:

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PROJECT NO.: 00136146	BORING LOG K101	PLATE
DRAWN BY: JDS		
CHECKED BY: OK	ROCKRIDGE SAFEWAY 51st & BROADWAY STORE NO. 3132 OAKLAND, CALIFORNIA	A-12
DATE: 9/24/2014		
REVISED: -		PAGE: 1 of 1

Date Begin - End:	9/10/2014	Drilling Company:	Woodward	**BORING LOG K102**
Logged By:	O. Khan	Drill Crew:	German/Joe	
Hor.-Vert. Datum:	Not Available	Drilling Equipment:	BK 81	Hammer Type - Drop: 140 lb. Auto - 30 in.
Plunge:	-90 degrees	Drilling Method:	Hollow Stem Auger	
Weather:	clear hot	Bore Diameter:	8 in. O.D.	

FIELD EXPLORATION / LABORATORY RESULTS

Coordinates Not Available
Ground Surface Elevation Not Available
Surface Condition: Asphalt

Depth (feet)	Lithologic Description	Sample Type	Blow Counts(BC)= Uncorr. Blows/6 in. Pocket Pen(PP)= tsf	Recovery (NR=No Recovery)	USCS Symbol	Water Content (%)	Dry Unit Wt. (pcf)	Passing #4 (%)	Passing #200 (%)	Liquid Limit	Plasticity Index (NP=NonPlastic)	Additional Tests/ Remarks
	approximate 8-inches of asphalt											
	Clayey GRAVEL with Sand (GC): fine to medium grained, greenish gray to gray, moist, fine to coarse subangular gravel, rock fragments (FILL)											
	increase in gravel content subangular to angular gravel, medium grain sand		BC=12 13 23	11"								
5			BC=20 10 7									cobble within shoe
	Lean CLAY with Sand (CL): fine grained, medium plasticity, greenish gray, moist, trace of fine gravel (FILL)		BC=50-1/2"	27"								switched to coring at 7'
	TONALITE: fine grain, light bluish gray, strong to very strong, slightly weathered, highly, fractured, fracturing in filled with quartz & feldspar, fractures @ about 45 degrees											hard drilling at 7.5' RQD = 67%
10												

The exploration was terminated at approximately 10 ft. below ground surface. The exploration was backfilled with grout and capped with concrete on September 10, 2014. Rock was encountered at a depth of 7 ft. during this exploration.

GROUNDWATER LEVEL INFORMATION:
Groundwater was not encountered during drilling or after completion.
GENERAL NOTES:

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KLEINFELDER Bright People. Right Solutions.	PROJECT NO.: 00136146	BORING LOG K102	PLATE
	DRAWN BY: JDS		A-13
	CHECKED BY: OK	ROCKRIDGE SAFEWAY 51st & BROADWAY STORE NO. 3132 OAKLAND, CALIFORNIA	
	DATE: 9/24/2014		
	REVISED: -		PAGE: 1 of 1

Date Begin - End:	9/10/2014	**Drilling Company:**	Woodward		**BORING LOG K103**
Logged By:	O. Khan	**Drill Crew:**	German/Joe		
Hor.-Vert. Datum:	Not Available	**Drilling Equipment:**	BK 81	**Hammer Type - Drop:**	140 lb. Auto - 30 in.
Plunge:	-90 degrees	**Drilling Method:**	Hollow Stem Auger		
Weather:	clear hot	**Bore Diameter:**	8 in. O.D.		

FIELD EXPLORATION / LABORATORY RESULTS

Coordinates Not Available
Ground Surface Elevation Not Available
Surface Condition: Asphalt

Depth (feet)	Lithologic Description	Sample Type	Blow Counts(BC)= Uncorr. Blows/6 in. Pocket Pen(PP)= tsf	Recovery (NR=No Recovery)	USCS Symbol	Water Content (%)	Dry Unit Wt. (pcf)	Passing #4 (%)	Passing #200 (%)	Liquid Limit	Plasticity Index (NP=NonPlastic)	Additional Tests/ Remarks
	approximate 8-inches of asphalt											
	Rock fragments with in a Clayey SAND (SC): fine to medium grained, dark yellowish brown, moist, cobbles, fine to coarse subangular to angular gravel (FILL)											
			BC=9 29 30	17"								
	dark greenish gray, dry to moist, dense, increase in sand and gravel content, decrease in clay content											
5			BC=24 22 20	14"								
	moist, dense, cobble size rock fragments mottled with sandy lean clay											
	wood fragments											
10	wood fragments		BC=8 5 11									no recovery sample fell out , drove SPT to recover
	Lean CLAY with Sand (CL): fine grained, medium plasticity, dark greenish gray, moist, trace fine gravel, wood fragments (FILL)											
			BC=50-6"									possible bedrock, hard drilling at 14-1/2 ft
15	cobbles mottled with clay		BC=50-4"									
20												

The exploration was terminated at approximately 16.5 ft. below ground surface. The exploration was backfilled with grout and capped with concrete on September 10, 2014.

GROUNDWATER LEVEL INFORMATION:
Groundwater was not encountered during drilling or after completion.
GENERAL NOTES:

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PROJECT NO.: 00136146	BORING LOG K103		PLATE
DRAWN BY: JDS			A-14
CHECKED BY: OK	ROCKRIDGE SAFEWAY 51st & BROADWAY STORE NO. 3132 OAKLAND, CALIFORNIA		
DATE: 9/24/2014			
REVISED: -			PAGE: 1 of 1

Date Begin - End:	9/10/2014	**Drilling Company:**	Woodward		**BORING LOG K104**
Logged By:	O. Khan	**Drill Crew:**	German/Joe		
Hor.-Vert. Datum:	Not Available	**Drilling Equipment:**	BK 81	**Hammer Type - Drop:**	140 lb. Auto - 30 in.
Plunge:	-90 degrees	**Drilling Method:**	Hollow Stem Auger		
Weather:	clear hot	**Bore Diameter:**	8 in. O.D.		

FIELD EXPLORATION / LABORATORY RESULTS

Coordinates Not Available
Ground Surface Elevation Not Available
Surface Condition: Asphalt

Depth (feet)	Graphical Log	Lithologic Description	Sample Type	Blow Counts(BC)= Uncorr. Blows/6 in. Pocket Pen(PP)= tsf	Recovery (NR=No Recovery)	USCS Symbol	Water Content (%)	Dry Unit Wt. (pcf)	Passing #4 (%)	Passing #200 (%)	Liquid Limit	Plasticity Index (NP=NonPlastic)	Additional Tests/ Remarks
		approximate 8-inches of asphalt											
		Clayey SAND with Gravel (SC): fine to medium grained, yellowish brown, moist, medium dense to dense, fine to coarse subangalur gravels, cobbles (FILL)											
				BC=17 13 13									
		dark grayish brown											rock fragment in shoe
		increase in rock fragments											
5				BC=12 14 9									
		Clayey GRAVEL with Sand (GC): fine to medium grained, dark grayish brown to dark gray, moist, medium dense, fine to coarse subangalur gravel (FILL)											
10				BC=5 3 50/5"									
		TONALITE: fine grained, light bluish gray, slightly weathered, strong to very strong, highly fractured		BC=50/2"									hard drilling at 11'

The exploration was terminated at approximately 12 ft. below ground surface. The exploration was backfilled with grout and capped with concrete on September 10, 2014. Rock was encountered at a depth of 11 ft. during this exploration.

GROUNDWATER LEVEL INFORMATION:
Groundwater was not encountered during drilling or after completion.
GENERAL NOTES:

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Date Begin - End:	9/10/2014 - 9/11/2014	**Drilling Company:**	Woodward				**BORING LOG K105**
Logged By:	SN/OK	**Drill Crew:**	German/Joe				
Hor.-Vert. Datum:	Not Available	**Drilling Equipment:**	BK-59	**Hammer Type - Drop:**	140 lb. Auto - 30 in.		
Plunge:	-90 degrees	**Drilling Method:**	Hollow Stem Auger				
Weather:	clear hot	**Bore Diameter:**	8 in. O.D.				

FIELD EXPLORATION / LABORATORY RESULTS

Coordinates Not Available
Ground Surface Elevation Not Available
Surface Condition: Asphalt

Depth (feet)	Graphical Log	Lithologic Description	Sample Type	Blow Counts(BC)= Uncorr. Blows/6 in. Pocket Pen(PP)= tsf	Recovery (NR=No Recovery)	USCS Symbol	Water Content (%)	Dry Unit Wt. (pcf)	Passing #4 (%)	Passing #200 (%)	Liquid Limit	Plasticity Index (NP=NonPlastic)	Additional Tests/ Remarks
		approximate 8-inches of asphalt											
		Sandy Lean CLAY (CL): non-plastic, light brown, dry, hard, root present											
				BC=32 24 50	10"						36	18	
		Gravelly Lean CLAY with Sand (CL): low plasticity, dark brown, moist, hard											
5				BC=14 28 25	16"								
		increase in gravel content											
10		**TONALITE**: fine to medium grain, dark greenish gray to bluish gray, slightly weathered, strong, highly to intensely fractured, quartz			NR								switched to coring at 10'
					12"								RQD = 0%
					3"								
													RQD = 0%
		intensely fractured											
15													
20													

The exploration was terminated at approximately 16 ft. below ground surface. The exploration was backfilled with grout and capped with concrete on September 10, 2014. Rock was encountered at a depth of 10 ft. during this exploration.

GROUNDWATER LEVEL INFORMATION:
Groundwater was not encountered during drilling or after completion.
GENERAL NOTES:

KLEINFELDER Bright People. Right Solutions.	PROJECT NO.: 00136146	BORING LOG K105		PLATE
	DRAWN BY: JDS			
	CHECKED BY: OK	ROCKRIDGE SAFEWAY 51st & BROADWAY STORE NO. 3132 OAKLAND, CALIFORNIA		A-16
	DATE: 9/24/2014			
	REVISED: -			PAGE: 1 of 1

Date Begin - End:	9/10/2014 - 9/11/2014	**Drilling Company:**	Woodward		**BORING LOG K106**
Logged By:	O. Khan	**Drill Crew:**	German/Joe		
Hor.-Vert. Datum:	Not Available	**Drilling Equipment:**	BK-59	**Hammer Type - Drop:**	140 lb. Auto - 30 in.
Plunge:	-90 degrees	**Drilling Method:**	Rotary		
Weather:	clear hot	**Bore Diameter:**	8 in. O.D.		

FIELD EXPLORATION / LABORATORY RESULTS

Coordinates Not Available
Ground Surface Elevation Not Available
Surface Condition: Asphalt

Depth (feet)	Lithologic Description	Sample Type	Blow Counts(BC)= Uncorr. Blows/6 in. Pocket Pen(PP)= tsf	Recovery (NR=No Recovery)	USCS Symbol	Water Content (%)	Dry Unit Wt. (pcf)	Passing #4 (%)	Passing #200 (%)	Liquid Limit	Plasticity Index (NP=NonPlastic)	Additional Tests/ Remarks
	approximate 8-inches of asphalt											using auger as casing to about 4'
	Gravelly Lean CLAY with Sand (CL): fine to medium grained, yellowish brown, moist, fine to coarse subangular to angular gravel (FILL) increase in gravel, cobbles											
5	**Rock Fragments with in a Sandy Lean CLAY matrix (CL)**: fine to medium grained, dark greenish gray, moist, fine to coarse subrounded to subangular rock fragments (FILL)			6"								
	Sandy Lean CLAY with Gravel (CL): fine to medium grained, dark grayish brown, mottled, fine to coarse gravel, chert, organic odor (FILL)			28"								
10												
15	wood fragment with clayey gravels		BC=8 10 10	3"								difficulties coring drove mod cal at 15', switched to augers
			BC=8 8 5	2"								
20	fine to medium grained, mottled dark greenish gray, moist, fine to coarse gravel, rock fragments (FILL)		BC=7 11 8	12"								

The exploration was terminated at approximately 21.5 ft. below ground surface. The exploration was backfilled with grout and capped with concrete on September 10, 2014.

GROUNDWATER LEVEL INFORMATION:
Groundwater was not encountered during drilling or after completion.
GENERAL NOTES:

KLEINFELDER Bright People. Right Solutions.	PROJECT NO.: 00136146 DRAWN BY: JDS CHECKED BY: OK DATE: 9/24/2014 REVISED: -	BORING LOG K106 ROCKRIDGE SAFEWAY 51st & BROADWAY STORE NO. 3132 OAKLAND, CALIFORNIA	PLATE A-17 PAGE: 1 of 1

Date Begin - End:	9/10/2014 - 9/11/2014	**Drilling Company:**	Woodward		**BORING LOG K107**
Logged By:	SN	**Drill Crew:**	German/Joe		
Hor.-Vert. Datum:	Not Available	**Drilling Equipment:**	BK-59	**Hammer Type - Drop:**	140 lb. Auto - 30 in.
Plunge:	-90 degrees	**Drilling Method:**	Hollow Stem Auger		
Weather:	clear hot	**Bore Diameter:**	8 in. O.D.		

FIELD EXPLORATION / LABORATORY RESULTS

Coordinates Not Available
Ground Surface Elevation Not Available
Surface Condition: Asphalt

Depth (feet)	Lithologic Description	Sample Type	Blow Counts(BC)= Uncorr. Blows/6 in. Pocket Pen(PP)= tsf	Recovery (NR=No Recovery)	USCS Symbol	Water Content (%)	Dry Unit Wt. (pcf)	Passing #4 (%)	Passing #200 (%)	Liquid Limit	Plasticity Index (NP=NonPlastic)	Additional Tests/ Remarks
	approximate 8-inches of asphalt											
	Sandy Lean CLAY with Gravel (CL): non-plastic, light brown, dry											
	less gravel											
5			BC=13 25 42	6"								
	Lean CLAY with Sand (CL): dark brown, wet, soft, plastic											
10			BC=13 28 50-3"	NR								no recovery SPT driven
	large gravel pieces											
	TONALITE: fine grained, bluish gray, strong, highly fractured, some fractures @ about 45 degrees, infilling with quartz, fresh to slightly weathered		BC=50-2"									switched to core
15												RQD = 60%

The exploration was terminated at approximately 17.5 ft. below ground surface. The exploration was backfilled with grout and capped with concrete on September 10, 2014. Rock was encountered at a depth of 13.5 ft. during this exploration.

20

GROUNDWATER LEVEL INFORMATION:
Groundwater was not encountered during drilling or after completion.
GENERAL NOTES:

KLEINFELDER
Bright People. Right Solutions.

PROJECT NO.:	00136146
DRAWN BY:	JDS
CHECKED BY:	OK
DATE:	9/24/2014
REVISED:	-

BORING LOG K107

ROCKRIDGE SAFEWAY
51st & BROADWAY
STORE NO. 3132
OAKLAND, CALIFORNIA

PLATE
A-18

Date Begin - End:	9/11/2014	Drilling Company:	Gregg Drilling	**BORING LOG K108**
Logged By:	SN	Drill Crew:		
Hor.-Vert. Datum:	Not Available	Drilling Equipment:	B-53	Hammer Type - Drop: 140 lb. Auto - 30 in.
Plunge:	-90 degrees	Drilling Method:	Hollow Stem Auger	
Weather:	clear hot	Bore Diameter:	8 in. O.D.	

FIELD EXPLORATION — Coordinates Not Available; Ground Surface Elevation Not Available; Surface Condition: Asphalt

LABORATORY RESULTS

Depth (feet)	Graphical Log	Lithologic Description	Sample Type	Blow Counts(BC)= Uncorr. Blows/6 in. Pocket Pen(PP)= tsf	Recovery (NR=No Recovery)	USCS Symbol	Water Content (%)	Dry Unit Wt. (pcf)	Passing #4 (%)	Passing #200 (%)	Liquid Limit	Plasticity Index (NP=NonPlastic)	Additional Tests/ Remarks
		approximate 8-inches of asphalt											
		Well-graded GRAVEL with Sand (GW): fine to medium grained, dark brown, dry, dense, angular (FILL)											difficult drilling
				BC=14 17 8	NR								
5													drilling faster
		very dense		BC=13 18 50-3									
10		light brown, less sand											
		smaller gravel pieces, powder from pulverization											
		TONALITE: fine grain, gray, slightly weathered, medium strong to strong, highly fractured at about 17-1/2, thin clay seam approx 45 degrees, fractures filled with feldspars		BC=50-3									
15				BC=50-1/2"	NR 54"								switched to coring
													RQD = 67%
20													

The exploration was terminated at approximately 20 ft. below ground surface. The exploration was backfilled with grout and capped with concrete on September 11, 2014. Rock was encountered at a depth of 13 ft. during this exploration.

GROUNDWATER LEVEL INFORMATION:
Groundwater was not encountered during drilling or after completion.

GENERAL NOTES:

KLEINFELDER
Bright People. Right Solutions.

PROJECT NO.:	00136146	BORING LOG K108	PLATE
DRAWN BY:	JDS		
CHECKED BY:	OK	ROCKRIDGE SAFEWAY 51st & BROADWAY STORE NO. 3132 OAKLAND, CALIFORNIA	A-19
DATE:	9/24/2014		
REVISED:	-		PAGE: 1 of 1

Date Begin - End:	9/11/2014	**Drilling Company:**	Gregg Drilling		**BORING LOG K109**
Logged By:	SN	**Drill Crew:**			
Hor.-Vert. Datum:	Not Available	**Drilling Equipment:**	B-53	**Hammer Type - Drop:**	140 lb. Auto - 30 in.
Plunge:	-90 degrees	**Drilling Method:**	Hollow Stem Auger		
Weather:	clear hot	**Bore Diameter:**	8 in. O.D.		

FIELD EXPLORATION / LABORATORY RESULTS

Coordinates Not Available
Ground Surface Elevation Not Available
Surface Condition: Asphalt

Depth (feet)	Graphical Log	Lithologic Description	Sample Type	Blow Counts(BC)= Uncorr. Blows/6 in. Pocket Pen(PP)= tsf	Recovery (NR=No Recovery)	USCS Symbol	Water Content (%)	Dry Unit Wt. (pcf)	Passing #4 (%)	Passing #200 (%)	Liquid Limit	Plasticity Index (NP=NonPlastic)	Additional Tests/ Remarks
		approximate 8-inches of asphalt											
		TONALITE: fine grain, bluish-gray, slightly weathered, strong, highly fractured											
					3"								RQD = 0%
5					6"								RQD = 0%
10													
15													
20													

The exploration was terminated at approximately 7 ft. below ground surface. The exploration was backfilled with grout and capped with concrete on September 11, 2014. Rock was encountered at a depth of 7 ft. during this exploration.

GROUNDWATER LEVEL INFORMATION:
Groundwater was not encountered during drilling or after completion.

GENERAL NOTES:

KLEINFELDER
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PROJECT NO.: 00136146	BORING LOG K109	PLATE
DRAWN BY: JDS		A-20
CHECKED BY: OK	ROCKRIDGE SAFEWAY 51st & BROADWAY STORE NO. 3132 OAKLAND, CALIFORNIA	
DATE: 9/24/2014		
REVISED: -		PAGE: 1 of 1

Date Begin - End:	9/11/2014	Drilling Company:	Woodward	BORING LOG K110
Logged By:	O. Khan	Drill Crew:	German/Joe	
Hor.-Vert. Datum:	Not Available	Drilling Equipment:	BK 81	Hammer Type - Drop: 140 lb. Auto - 30 in.
Plunge:	-90 degrees	Drilling Method:	Hollow Stem Auger	
Weather:	clear hot	Bore Diameter:	8 in. O.D.	

FIELD EXPLORATION / LABORATORY RESULTS

Coordinates Not Available
Ground Surface Elevation Not Available
Surface Condition: Asphalt

Depth (feet)	Graphical Log	Lithologic Description	Sample Type	Blow Counts(BC)= Uncorr. Blows/6 in. Pocket Pen(PP)= tsf	Recovery (NR=No Recovery)	USCS Symbol	Water Content (%)	Dry Unit Wt. (pcf)	Passing #4 (%)	Passing #200 (%)	Liquid Limit	Plasticity Index (NP=NonPlastic)	Additional Tests/ Remarks
		approximate 8-inches of asphalt											
		Clayey GRAVEL with Sand (GC): fine grained, medium plasticity, yellowish brown, moist, fine to coarse subangular gravel, rock fragments (FILL)											
		increase in clay content		BC=18 25 26	3"								rock fragment within shoe, hard drilling at 3'
5		increase in rock fragments		BC=22 14 15	8"								
		Lean CLAY with Sand (CL): fine grained, medium plasticity, greenish gray to grayish brown, trace fine gravel (FILL)											
10		gravels within clay matrix		BC=33 10 7	3"								rubble within shoe
15		wet, wood fragments		BC=4 10 4	5"								
20		wet, wood fragments within sandy clay to clayey sand matrix		BC=10 4 4	NR								sample fell out, captured with SPT

The exploration was terminated at approximately 21.5 ft. below ground surface. The exploration was backfilled with grout and capped with concrete on September 11, 2014.

GROUNDWATER LEVEL INFORMATION:
Groundwater was not encountered during drilling or after completion.
GENERAL NOTES:

KLEINFELDER Bright People. Right Solutions.	PROJECT NO.: 00136146	BORING LOG K110	PLATE
	DRAWN BY: JDS		A-21
	CHECKED BY: OK	ROCKRIDGE SAFEWAY 51st & BROADWAY STORE NO. 3132 OAKLAND, CALIFORNIA	
	DATE: 9/24/2014		
	REVISED: -		PAGE: 1 of 1

Date Begin - End:	9/11/2014	**Drilling Company:**	Gregg Drilling
Logged By:	SN	**Drill Crew:**	
Hor.-Vert. Datum:	Not Available	**Drilling Equipment:**	B-53
Plunge:	-90 degrees	**Drilling Method:**	Hollow Stem Auger
Weather:	clear hot	**Bore Diameter:**	8 in. O.D.

BORING LOG K111

Hammer Type - Drop: 140 lb. Auto - 30 in.

FIELD EXPLORATION

Coordinates Not Available
Ground Surface Elevation Not Available
Surface Condition: Asphalt

LABORATORY RESULTS

Depth (feet)	Lithologic Description	Sample Type	Blow Counts(BC)= Uncorr. Blows/6 in. Pocket Pen(PP)= tsf	Recovery (NR=No Recovery)	USCS Symbol	Water Content (%)	Dry Unit Wt. (pcf)	Passing #4 (%)	Passing #200 (%)	Liquid Limit	Plasticity Index (NP=NonPlastic)	Additional Tests/ Remarks
	approximate 8-inches of asphalt											
	Sandy Lean CLAY with Gravel (CL): fine grained, low plasticity, dark brown, moist, firm, fine to coarse gravels (FILL)											
			BC=17 21 20	12"								
5	light brown, smaller gravel		BC=3 50-2"	6"								
												drilling difficult
	Sandy Lean CLAY (CL): low plasticity, dark brown, moist, firm											
	dark brown											
10			BC=37 50-4"	18"								
	rock fragments											
15	**TONALITE**: fine grain, bluish gray, strong, slightly weathered, highly fractured		BC=50-4"	3"								
												drilling difficult
			BC=50-3"	3"								
20												

The exploration was terminated at approximately 19.5 ft. below ground surface. The exploration was backfilled with grout and capped with concrete on September 11, 2014. Rock was encountered at a depth of 15 ft. during this exploration.

GROUNDWATER LEVEL INFORMATION:
Groundwater was not encountered during drilling or after completion.
GENERAL NOTES:

KLEINFELDER
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PROJECT NO.: 00136146	BORING LOG K111	PLATE
DRAWN BY: JDS		A-22
CHECKED BY: OK	ROCKRIDGE SAFEWAY 51st & BROADWAY STORE NO. 3132 OAKLAND, CALIFORNIA	
DATE: 9/24/2014		
REVISED: -		PAGE: 1 of 1

Date Begin - End:	9/12/2014	**Drilling Company:**	Gregg Drilling
Logged By:	SN	**Drill Crew:**	
Hor.-Vert. Datum:	Not Available	**Drilling Equipment:**	B-53
Plunge:	-90 degrees	**Drilling Method:**	Hollow Stem Auger
Weather:	clear hot	**Bore Diameter:**	8 in. O.D.

BORING LOG K112

Hammer Type - Drop: 140 lb. Auto - 30 in.

FIELD EXPLORATION: Coordinates Not Available; Ground Surface Elevation Not Available; Surface Condition: Asphalt

Depth (feet)	Lithologic Description	Sample Type	Blow Counts(BC)= Uncorr. Blows/6 in. Pocket Pen(PP)= tsf	Recovery (NR=No Recovery)	USCS Symbol	Water Content (%)	Dry Unit Wt. (pcf)	Passing #4 (%)	Passing #200 (%)	Liquid Limit	Plasticity Index (NP=NonPlastic)	Additional Tests/ Remarks
	approximate 8-inches of asphalt											
	Sandy Lean CLAY with Gravel (CL): fine to medium grained, medium plasticity, brown, dry to moist, hard, subangular gravel (FILL)											
			BC=16 22 24							29	13	
5	medium plasticity, dry to moist, roots, weaklty cemented, less gravel		BC=14 10 50-4"									
10	decrease in gravel content, roots, brick fragments		BC=6 6 10									
	medium plasticity, dry, firm to hard											
	TONALITE: fine grain, bluish gray, strong to very strong, slightly weathered, fractured		BC=50-3"	28"								difficult drilling switched to coring at 13' RQD = 60%
15												
20												

The exploration was terminated at approximately 15.5 ft. below ground surface. The exploration was backfilled with grout and capped with concrete on September 12, 2014. Rock was encountered at a depth of 12.5 ft. during this exploration.

GROUNDWATER LEVEL INFORMATION:
Groundwater was not encountered during drilling or after completion.
GENERAL NOTES:

KLEINFELDER
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PROJECT NO.:	00136146
DRAWN BY:	JDS
CHECKED BY:	OK
DATE:	9/24/2014
REVISED:	-

BORING LOG K112

ROCKRIDGE SAFEWAY
51st & BROADWAY
STORE NO. 3132
OAKLAND, CALIFORNIA

PLATE
A-23

Date Begin - End:	9/12/2014	**Drilling Company:**	Gregg Drilling		**BORING LOG K113**
Logged By:	SN	**Drill Crew:**			
Hor.-Vert. Datum:	Not Available	**Drilling Equipment:**	B-53	**Hammer Type - Drop:**	140 lb. Auto - 30 in.
Plunge:	-90 degrees	**Drilling Method:**	Hollow Stem Auger		
Weather:	clear hot	**Bore Diameter:**	8 in. O.D.		

FIELD EXPLORATION / LABORATORY RESULTS

Coordinates Not Available
Ground Surface Elevation Not Available
Surface Condition: Asphalt

Depth (feet)	Graphical Log	Lithologic Description	Sample Type	Blow Counts(BC)= Uncorr. Blows/6 in. Pocket Pen(PP)= tsf	Recovery (NR=No Recovery)	USCS Symbol	Water Content (%)	Dry Unit Wt. (pcf)	Passing #4 (%)	Passing #200 (%)	Liquid Limit	Plasticity Index (NP=NonPlastic)	Additional Tests/ Remarks
		approximate 8-inches of asphalt											
		Sandy Lean CLAY with Gravel (CL): fine to medium grained, medium plasticity, light brown to yellow, dry, hard, angular gravel (FILL)											bumpy drilling
				BC=21 24 22	12"				70	34			
5		brown, smaller size gravel		BC=9 16 8	8"								
		Lean CLAY with Sand (CL): fine grained, medium plasticity, black, dry, firm, traces of brick, roots (FILL)											
10				BC=6 6 8									
15		wet		BC=13 19 19	NR								no recovery drove SPT to obtain sample
		tile and morter pieces, very wet		BC=4 2 1									
20													

The exploration was terminated at approximately 20 ft. below ground surface. The exploration was backfilled with grout and capped with concrete on September 12, 2014.

GROUNDWATER LEVEL INFORMATION:
Groundwater was not encountered during drilling or after completion.
GENERAL NOTES:

KLEINFELDER
Bright People. Right Solutions.

PROJECT NO.:	00136146
DRAWN BY:	JDS
CHECKED BY:	OK
DATE:	9/24/2014
REVISED:	-

BORING LOG K113

ROCKRIDGE SAFEWAY
51st & BROADWAY
STORE NO. 3132
OAKLAND, CALIFORNIA

PLATE
A-24

Date Begin - End:	9/12/2014	**Drilling Company:**	Woodward		**BORING LOG K114**
Logged By:	O. Khan	**Drill Crew:**	German/Joe		
Hor.-Vert. Datum:	Not Available	**Drilling Equipment:**	BK 81	**Hammer Type - Drop:**	140 lb. Auto - 30 in.
Plunge:	-90 degrees	**Drilling Method:**	Hollow Stem Auger/Rotary		
Weather:	overcast	**Bore Diameter:**	8 in. O.D.		

Depth (feet)	Graphical Log	FIELD EXPLORATION: Coordinates Not Available Ground Surface Elevation Not Available Surface Condition: Asphalt Lithologic Description	Sample Type	Blow Counts(BC)= Uncorr. Blows/6 in. Pocket Pen(PP)= tsf	Recovery (NR=No Recovery)	LABORATORY RESULTS: USCS Symbol	Water Content (%)	Dry Unit Wt. (pcf)	Passing #4 (%)	Passing #200 (%)	Liquid Limit	Plasticity Index (NP=NonPlastic)	Additional Tests/ Remarks
		approximate 8-inches of asphalt											
		Clayey GRAVEL (GC): yellowish brown, moist, fine to coarse subangular gravel, rock fragements (FILL)											
		TONALITE: fine grain, dark greenish gray to bluish gray, strong to very strong, slightly weathered, some fractures at 45 degrees or steeper, interbedded with dark gray shale		BC=50-2	24" NR								switched to coring
													hard coring
5													

The exploration was terminated at approximately 5.5 ft. below ground surface. The exploration was backfilled with grout and capped with concrete on September 12, 2014. Rock was encountered at a depth of 1.5 ft. during this exploration.

GROUNDWATER LEVEL INFORMATION:
Groundwater was not encountered during drilling or after completion.
GENERAL NOTES:

10

15

20

KLEINFELDER
Bright People. Right Solutions.

PROJECT NO.: 00136146	BORING LOG K114	PLATE
DRAWN BY: JDS		
CHECKED BY: OK	ROCKRIDGE SAFEWAY 51st & BROADWAY STORE NO. 3132 OAKLAND, CALIFORNIA	A-25
DATE: 9/24/2014		
REVISED: -		PAGE: 1 of 1

APPENDIX B

LABORATORY TESTING

Exploration ID	Approx. Sample Depth (ft.)	Sample No.	Sample Description	Moisture Content (%)	Dry Density (pcf)	Sieve Analysis			Atterberg Limits			Swell/Compression	Other Tests
						Passing 3/4 inch Sieve (%)	Passing #4 Sieve (%)	Passing #200 Sieve (%)	LL	PL	PI		
K-1	3.5		DARK BROWN CLAYEY SAND WITH GRAVEL (SC)	10.1	117			27	37	18	19		
K-1	9.5			16.6	112								
K-2	6.0		DARK GRAYISH BROWN SANDY LEAN CLAY (CL)	16.6	110			53	34	14	20		
K-2	9.5			15.9	112								
K-2	23.5	6C	DARK GRAYISH BROWN CLAYEY SAND WITH GRAVEL (SC)	14.0	111			29					
		6B											
		6A											
K-3	6.0		DARK OLIVE BROWN CLAYEY SAND WITH GRAVEL (SC)	12.3	118			33	34	19	15		
K-3	9.5			13.2	110								
K-4	3.5		DARK OLIVE BROWN CLAYEY SAND WITH GRAVEL (SC)	8.5	129			24					
K-4	6.0			11.3	115								
K-4	14.0			20.0	97								
K-7	6.0		DARK OLIVE BROWN CLAYEY GRAVEL WITH SAND (GC)	6.3	114			18	26	14	12		
K-7	9.5			10.5	116								

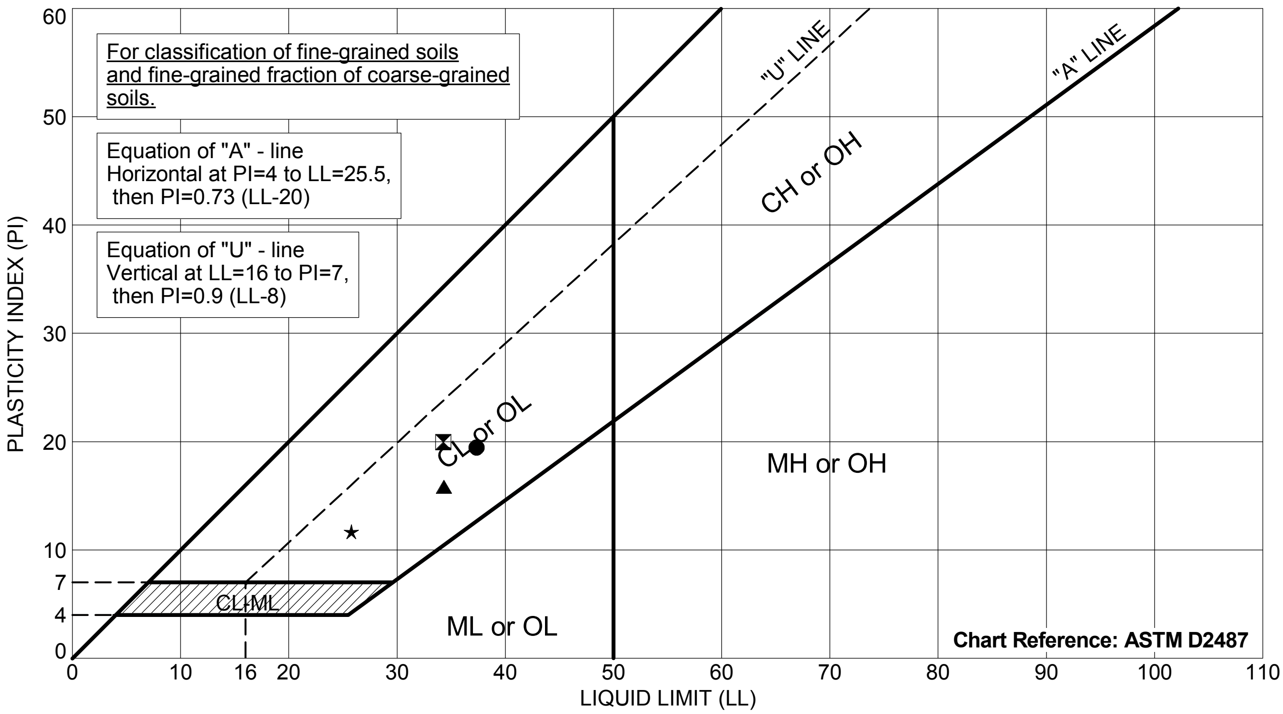
KLEINFELDER
Bright People. Right Solutions.

PROJECT NO.: 136146
DRAWN BY: AG
CHECKED BY: AJB
DATE: 9/3/2013
REVISED:

LABORATORY TEST RESULT SUMMARY

PDC ROCKRIDGE SHOPPING CENTER
5130 BROADWAY
OAKLAND, CALIFORNIA

TABLE
B-1



Symbol	Exploration ID	Approx. Depth (ft.)	Sample No.	Liquid Limit	Plastic Limit	Plasticity Index	Sample Description
●	K-1	3.5	N/A	37	18	19	DARK BROWN CLAYEY SAND WITH GRAVEL (SC)
⧗	K-2	6	N/A	34	14	20	DARK GRAYISH BROWN SANDY LEAN CLAY (CL)
▲	K-3	6	N/A	34	19	15	DARK OLIVE BROWN CLAYEY SAND WITH GRAVEL (SC)
★	K-7	6	N/A	26	14	12	DARK OLIVE BROWN CLAYEY GRAVEL WITH SAND (GC)

KLEINFELDER
Bright People. Right Solutions.

PROJECT NO.: 136146
DRAWN BY: AG
CHECKED BY: AJB
DATE: 9/3/2013
REVISED:

ATTERBERG LIMITS

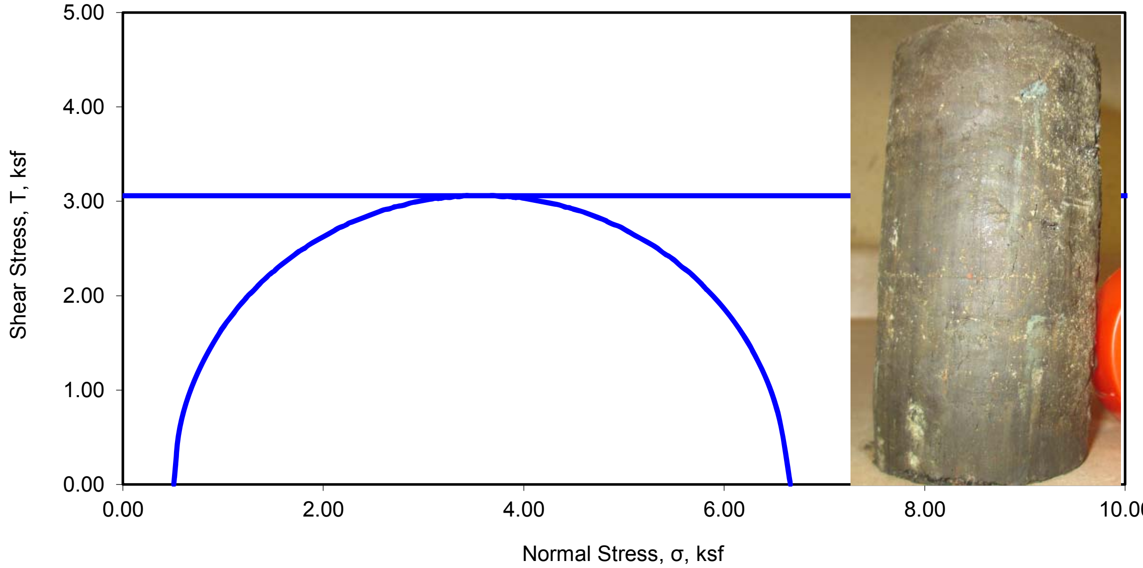
PDC ROCKRIDGE SHOPPING CENTER
5130 BROADWAY
OAKLAND, CALIFORNIA

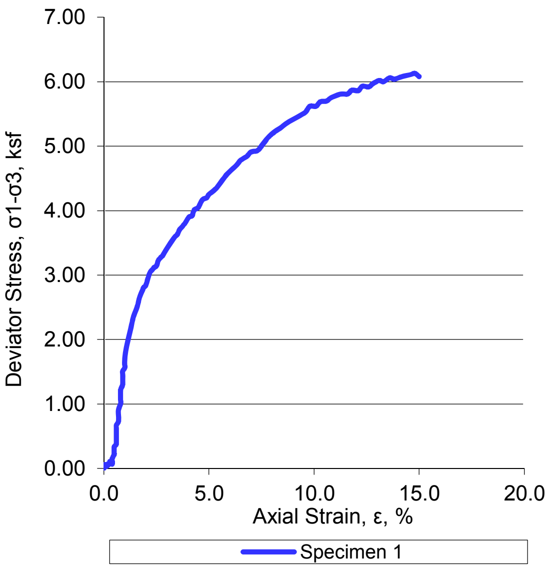
PLATE

B-2

Total		
c =	3.06	ksf

Specimen Shear Picture





Specimen No.			1
Initial	Diameter, in	D_O	2.41
Initial	Height, in	H_O	5.73
Initial	Water Content, %	ω_O	16.6
Initial	Dry Density, lbs/ft3	γ_{d_O}	111.8
Initial	Saturation, %	S_O	92
Initial	Void Ratio	e_O	0.480
Minor Principal Stress, ksf		σ_3	0.50
Maximum Deviator Stress, ksf		$(\sigma1-\sigma3)_{max}$	6.13
Time to $(\sigma_1-\sigma_3)_{max}$, min		t_f	14.83
Deviator Stress @ 15% Axial Strain, ksf		$(\sigma_1-\sigma_3)_{15\%}$	6.08
Ultimate Deviator Stress, ksf		$(\sigma_1-\sigma_3)_{ult}$	na
Rate of strain, %/min		'ε	1.00
Axial Strain at Failure, %		ε_f	14.83

Description of Specimen: Dark Olive Brown Lean Clay with Sand (CL)

Amount of Material Finer than the No. 200, %: nm

LL: nm	PL: nm	PI: nm	G_S: 2.65 Assumed	Specimen Type: Undisturbed	Test Method: ASTM D2850

Membrane correction applied

Boring:	K-1
Sample:	3A
Depth, ft:	9.5
Test Date:	9/10/13

Remarks: nm= not measured, na = not applicable

KLEINFELDER
Bright People. Right Solutions.
2601 Barrington Ct, Hayward, CA 94545

Project Number:	136146
Date:	9/16/13
Entry By:	CP
Checked By:	AJB
File Name:	HL5981

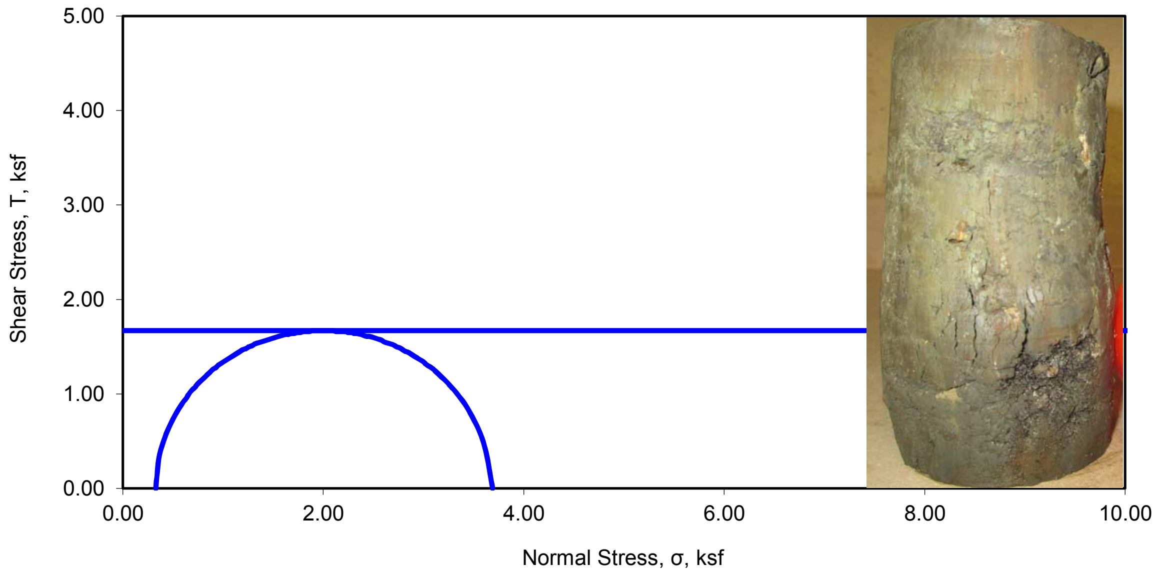
TRIAXIAL COMPRESSION TEST (UU)

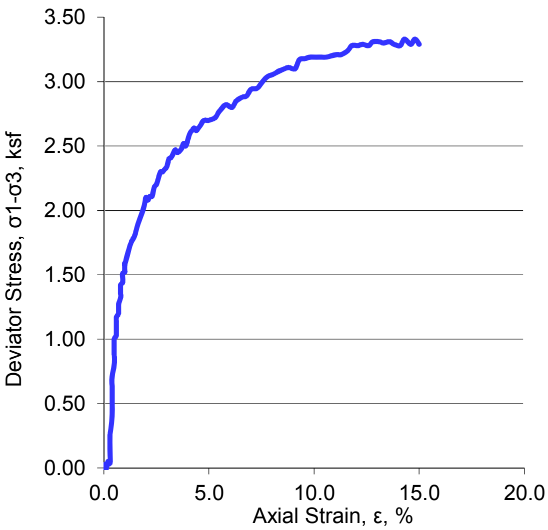
PDC ROCKRIDGE SHOPPING CENTER
5130 BROADWAY
OAKLAND, CALIFORNIA

Plate
1 of 1

B-3

Total	
c = 1.67	ksf





Specimen No.			1
Initial	Diameter, in	D_O	2.40
	Height, in	H_O	5.67
	Water Content, %	ω_O	16.6
	Dry Density, lbs/ft3	γ_{d_o}	110.3
	Saturation, %	S_O	88
	Void Ratio	e_O	0.499
Minor Principal Stress, ksf		σ_3	0.33
Maximum Deviator Stress, ksf		$(\sigma1-\sigma3)_{max}$	3.33
Time to $(\sigma_1\text{-}\sigma_3)_{max}$, min		t_f	14.33
Deviator Stress @ 15% Axial Strain, ksf		$(\sigma_1-\sigma_3)_{15\%}$	3.29
Ultimate Deviator Stress, ksf		$(\sigma_1-\sigma_3)_{ult}$	na
Rate of strain, %/min		$'\varepsilon$	1.00
Axial Strain at Failure, %		ε_f	14.33

Description of Specimen: Dark Olive Brown Sandy Lean Clay (CL)

Amount of Material Finer than the No. 200, %: 53

LL: 34 | PL: 14 | PI: 20 | G_S: 2.65 Assumed | Specimen Type: Undisturbed | Test Method: ASTM D2850

Membrane correction applied

Boring:	K-2
Sample:	2A
Depth, ft:	6.0
Test Date:	9/10/13

Remarks: nm= not measured, na = not applicable

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2601 Barrington Ct, Hayward, CA 94545

Project Number:	136146
Date:	9/16/13
Entry By:	cp
Checked By:	
File Name:	HL5981

TRIAXIAL COMPRESSION TEST (UU)

PDC ROCKRIDGE SHOPPING CENTER
5130 BROADWAY
OAKLAND, CALIFORNIA

Plate
1 of 1
B-4

KLEINFELDER
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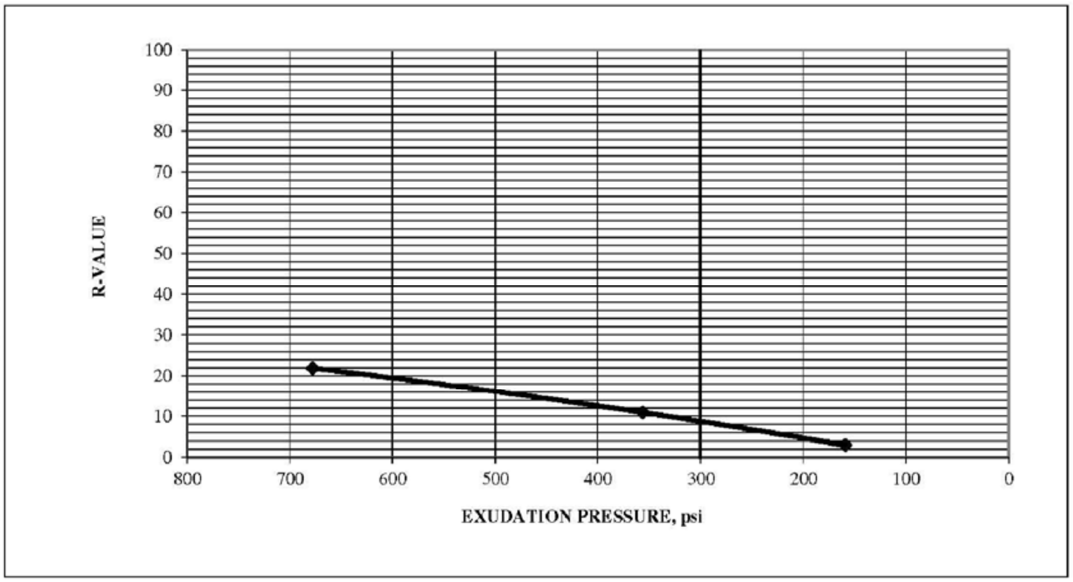
Laboratory Test Report

Project Name: PDC ROCKRIDGE
Project No.: 136146
Lab No.: HL5981
Sample Date: September 3, 2013
Sample No.: BULK - HL5981A
Sample Location: K-1 @ 1.0'-5.0'
Material Description: Olive Gray Clayey Sand with Gravel (SC)
Report Date: September 16, 2013

Resistance R-Value and Expansion Pressure of Compacted Soils (ASTM D2844, CTM 301)



Briquette No.	A	B	C
Moisture at Test, %	9.6	8.7	10.5
Dry Unit Weight at Test, pcf	132.0	131.0	130.1
Expansion Pressure, psf	30	52	4
Exudation Pressure, psi	356	678	159
Resistance Value	11	22	3
R - Value at 300 psi Exudation Pressure:			**9**

Reviewed By on 9/13/2013:

for **Aaron Kidd**
Laboratory Manager

Limitations: Pursuant to applicable building codes, the results presented in this report are for the exclusive use of the client and the registered design professional in responsible charge. The results apply only to the samples tested. If changes to the specifications were made and not communicated to Kleinfelder, Kleinfelder assumes no responsibility for pass/fail statements (meets/did not meet), if provided.

KLEINFELDER Bright People. Right Solutions. www.kleinfelder.com	PROJECT NO. 136146	R-VALUE LABORATORY TEST REPORT
	DRAWN: SEPT 2013	
	DRAWN BY: AG	
	CHECKED BY: AJB	PDC ROCKRIDGE SHOPPING CENTER 5130 BROADWAY OAKLAND, CALIFORNIA
	FILE NAME: Lab-b5-6.dwg	PLATE B-5

KLEINFELDER
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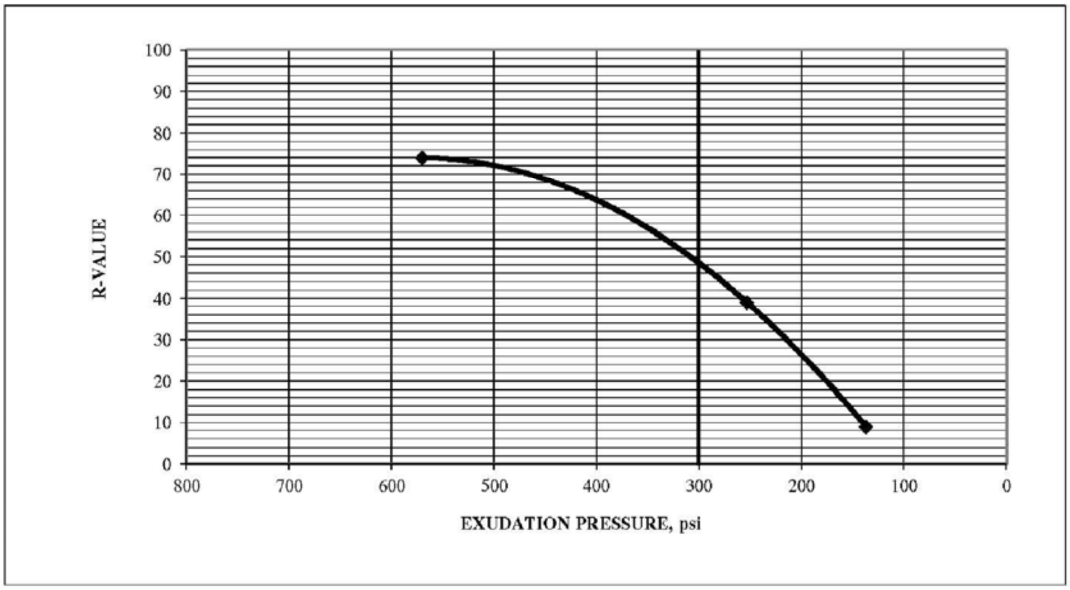
Laboratory Test Report

Project Name: PDC ROCKRIDGE
Project No.: 136146
Lab No.: HL5981
Sample Date: September 3, 2013
Sample No.: BULK - HL5981B
Sample Location: K-6 @ 1.0'-5.0'
Material Description: Brown Clayey Sand with Gravel (SC)
Report Date: September 16, 2013

Resistance R-Value and Expansion Pressure of Compacted Soils (ASTM D2844, CTM 301)



Briquette No.	A	B	C
Moisture at Test, %	9.3	7.5	6.2
Dry Unit Weight at Test, pcf	132.0	135.1	134.7
Expansion Pressure, psf	13	52	186
Exudation Pressure, psi	137	253	570
Resistance Value	9	39	74
R - Value at 300 psi Exudation Pressure:			**49**

Reviewed By on 9/13/2013:

for **Aaron Kidd**
Laboratory Manager

Limitations: Pursuant to applicable building codes, the results presented in this report are for the exclusive use of the client and the registered design professional in responsible charge. The results apply only to the samples tested. If changes to the specifications were made and not communicated to Kleinfelder, Kleinfelder assumes no responsibility for pass/fail statements (meets/did not meet), if provided.

Exploration ID	Depth (ft.)	Sample Description	Water Content (%)	Dry Unit Wt. (pcf)	Sieve Analysis (%)			Atterberg Limits			Additional Tests
					Passing 3/4"	Passing #4	Passing #200	Liquid Limit	Plastic Limit	Plasticity Index	
K100	2.5	DARK OLIVE BROWN POORLY GRADED SAND (SP)					1.5				
K105	2.5	DARK OLIVE BROWN LEAN CLAY (CL)						36	18	18	
K112	2.5	YELLOWISH BROWN SANDY LEAN CLAY WITH GRAVEL (CL)						29	16	13	
K113	3.5	LIGHT OLIVE BROWN CLAYEY SAND WITH GRAVEL (SC)			86	70	34				

Refer to the Geotechnical Evaluation Report or the supplemental plates for the method used for the testing performed above.
NP = NonPlastic

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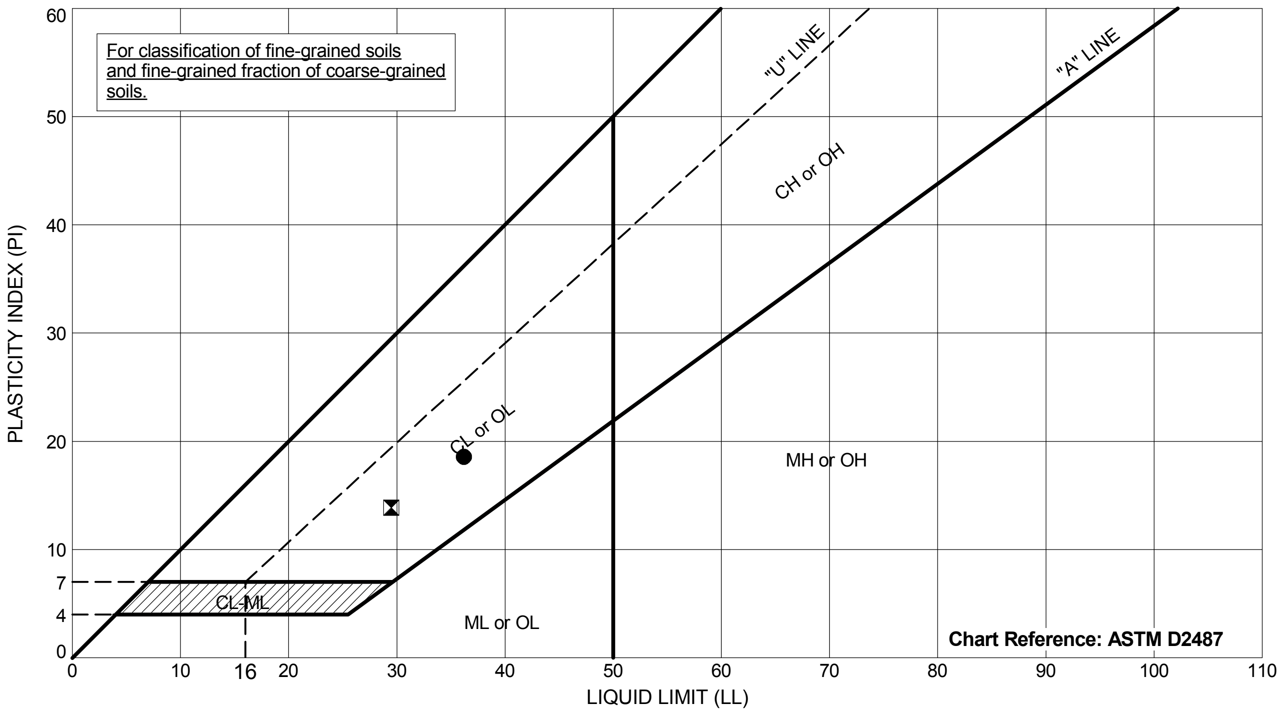
PROJECT NO.: 00136146
DRAWN BY: JDS
CHECKED BY: OK
DATE: 9/22/2014
REVISED: -

LABORATORY TEST RESULT SUMMARY

ROCKRIDGE SAFEWAY
51st & BROADWAY
STORE NO. 3132
OAKLAND, CALIFORNIA

TABLE
B-7



	Exploration ID	Depth (ft.)	Sample Description	Passing #200	LL	PL	PI
●	K105	2.5	DARK OLIVE BROWN LEAN CLAY (CL)	NM	36	18	18
⧗	K112	2.5	YELLOWISH BROWN SANDY LEAN CLAY WITH GRAVEL (CL)	NM	29	16	13

Testing perfomed in general accordance with ASTM D4318.
NP = Nonplastic
NM = Not Measured

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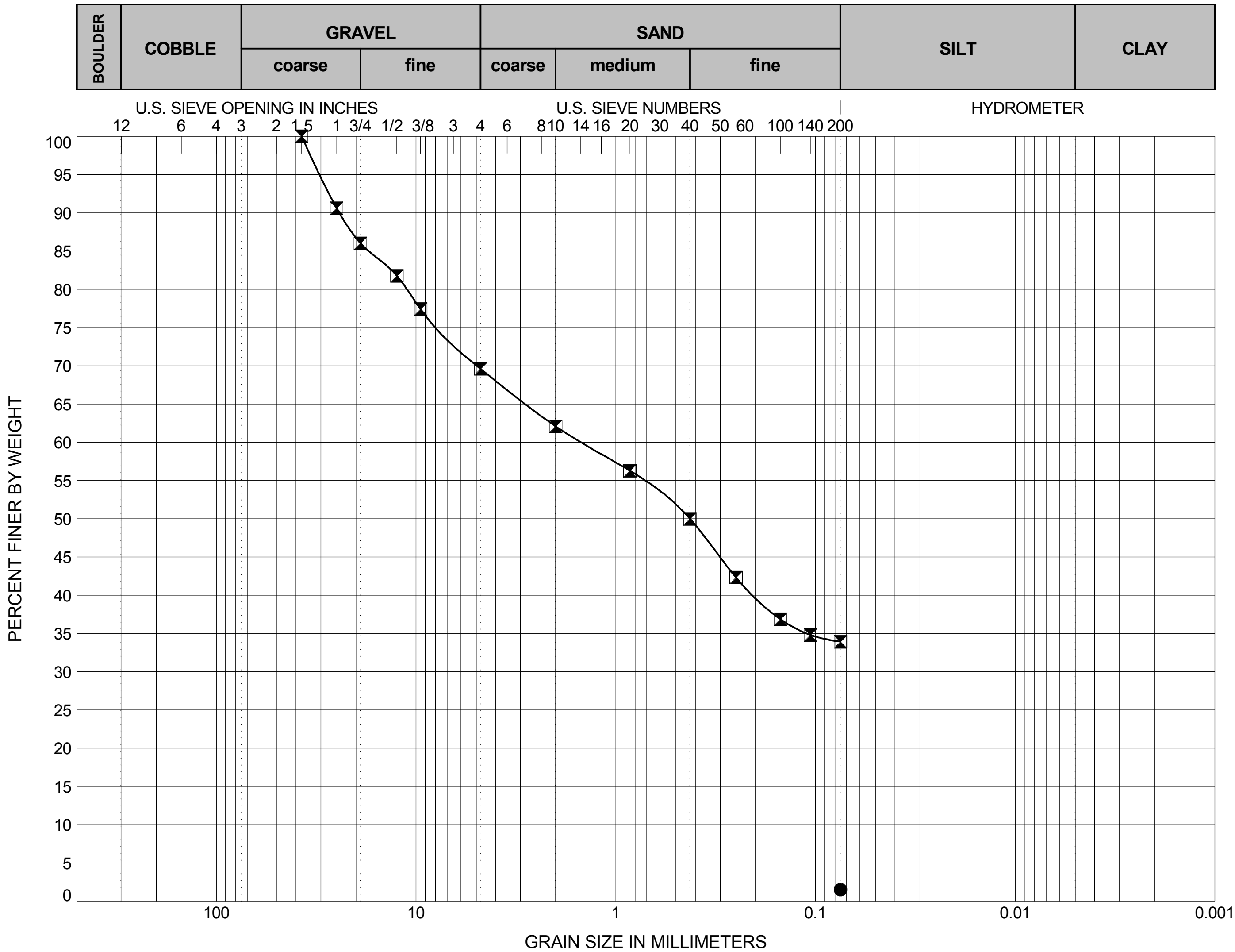
PROJECT NO.:	00136146	ATTERBERG LIMITS	PLATE
DRAWN BY:	JDS		B-8
CHECKED BY:	OK	ROCKRIDGE SAFEWAY 51st & BROADWAY STORE NO. 3132 OAKLAND, CALIFORNIA	
DATE:	9/24/2014		
REVISED:	-		



Exploration ID		Depth (ft.)	Sample Description	LL	PL	PI
●	K100	2.5 - 3.5	DARK OLIVE BROWN POORLY GRADED SAND (SP)	NM	NM	NM
⧗	K113	3.5	LIGHT OLIVE BROWN CLAYEY SAND WITH GRAVEL (SC)	NM	NM	NM

Exploration ID		Depth (ft.)	D_{100}	D_{60}	D_{30}	D_{10}	Cc	Cu	Passing 3/4"	Passing #4	Passing #200	%Silt	%Clay
●	K100	2.5 - 3.5	0.075	NM	NM	NM	NM	NM			1.5	NM	NM
⧗	K113	3.5	37.5	1.472	NM	NM	NM	NM	86	70	34	NM	NM

Sieve Analysis and Hydrometer Analysis testing performed in general accordance with ASTM D422.
NP = Nonplastic
NM = Not Measured

Coefficients of Uniformity - $C_u = D_{60} / D_{10}$
Coefficients of Curvature - $C_C = (D_{30})^2 / D_{60} D_{10}$
D_{60} = Grain diameter at 60% passing
D_{30} = Grain diameter at 30% passing
D_{10} = Grain diameter at 10% passing

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PROJECT NO.: 00136146	SIEVE ANALYSIS	PLATE
DRAWN BY: JDS		B-9
CHECKED BY: OK	ROCKRIDGE SAFEWAY 51st & BROADWAY STORE NO. 3132 OAKLAND, CALIFORNIA	
DATE: 9/24/2014		
REVISED: -		

Data checked by:_CRL____________ Date:_09/30/14___________

Point Load Strength Index Test Results ASTM D-5731

PROJECT:	Safeway Rockridge Oakland	LAB SAMPLE NO.:	11605A
PROJECT NO.:	136146.001A	SAMPLE NO.:	K-102 7'-9'
PROJECT LOCATION:	Oakland, CA	SAMPLE DESCRIP:	Tonalite
SAMPLED BY:	KLF	DATE TESTED:	9/25/2014
DATE SAMPLED:	9/14	REPORTED BY:	B. Kochanski

Boring No.	Depth (ft)	Test Number	Test Type*	Rock Type	Width, W (in)	Depth or Diameter, D (in)	Failure Load, P (lbs)	De^2 (in^2)	Point Load Strength Index, $I_{s(50)}$ (PSI)	Uniaxial Compressive Strength, UCS (PSI)	Valid/ Invalid
K-102	7' - 9'	1	a	Tonalite	2.41	1.12	1.3	3.43	374	9172	Y
K-103	7' - 9'	2	a	Tonalite	2.40	1.12	0.5	3.40	144	3537	Y
K-104	7' - 9'	3	a	Tonalite	2.41	0.83	1.1	2.55	383	9375	Y
K-105	7' - 9'	4	a	Tonalite	2.40	1.22	1.8	3.73	481	11788	Y
K-106	7' - 9'	5	a	Tonalite	2.40	1.17	1.2	3.59	329	8052	Y
K-107	7' - 9'	6	a	Tonalite	2.41	1.10	0.3	3.37	74	1822	Y
K-108	7' - 9'	7	a	Tonalite	2.39	1.05	1.3	3.20	378	9258	Y
K-109	7' - 9'	8	a	Tonalite	2.40	1.16	1.1	3.55	317	7760	Y
K-110	7' - 9'	9	i	Tonalite	2.41	1.52	2.4	4.65	538	13180	Y
K-111	7' - 9'	10	i	Tonalite	2.40	1.46	1.9	4.45	444	10888	Y

Point Load Strength Index

$I_{s(50)}$	**371 psi**	or	**53,424 psf**	or	**2.56 MPa**		

Uniaxial Compressive Strength

σ_c=	**9,084 psi**	or	**1,308,096 psf**	or	**63 MPa**

*Test Type
d = diametral
a = axial
b = block
i = irregular lump

PROJECT NO.: 00136146	POINT LOAD STRENGTH INDEX TEST RESULTS	TABLE	B-12
DRAWN BY: JDS			
CHECKED BY: DA	ROCKRIDGE SAFEWAY 51st & BROADWAY STORE NO. 3132 OAKLAND, CALIFORNIA		
DATE: 10/9/2014			
REVISED: -			

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Point Load Strength Index Test Results ASTM D-5731

KLEINFELDER Bright People. Right Solutions.	PROJECT NO.: 00136146 DRAWN BY: JDS CHECKED BY: OK DATE: 9/24/2014 REVISED: -	POINT LOAD STRENGTH INDEX TEST RESULTS ROCKRIDGE SAFEWAY 51st & BROADWAY STORE NO. 3132 OAKLAND, CALIFORNIA	PLATE B-13

Data checked by:_CRL____________ Date:_09/30/14___________

Point Load Strength Index Test Results ASTM D-5731

PROJECT:	Safeway Rockridge Oakland	LAB SAMPLE NO.:	11605B
PROJECT NO.:	136146.001A	SAMPLE NO.:	K-107 12.5' - 15'
PROJECT LOCATION:	Oakland, CA	SAMPLE DESCRIP:	Tonalite
SAMPLED BY:	KLF	DATE TESTED:	9/29/2014
DATE SAMPLED:	9/14	REPORTED BY:	B. Kochanski

Boring No.	Depth (ft)	Test Number	Test Type*	Rock Type	Width, W (in)	Depth or Diameter, D (in)	Failure Load, P (lbs)	De^2 (in^2)	Point Load Strength Index, $I_{s(50)}$ (PSI)	Uniaxial Compressive Strength, UCS (PSI)	Valid/ Invalid
K-107	12.5' - 15'	1	d	Tonalite	1.11	2.39	1.0	4.83	224	5488	Y
K-108	12.5' - 15'	2	d	Tonalite	1.20	2.41	2.7	4.78	595	14586	Y
K-109	12.5' - 15'	3	a	Tonalite	1.07	2.40	0.9	3.28	257	6303	Y
K-110	12.5' - 15'	4	d	Tonalite	1.24	2.39	0.6	4.91	119	2917	Y
K-111	12.5' - 15'	5	a	Tonalite	2.40	1.11	2.8	3.38	798	19551	Y
K-112	12.5' - 15'	6	a	Tonalite	2.40	1.15	2.6	3.50	719	17619	Y
K-113	12.5' - 15'	7	a	Tonalite	2.40	1.10	1.8	3.35	520	12736	Y
K-114	12.5' - 15'	8	a	Tonalite	2.40	1.15	2.8	3.50	793	19424	Y
K-115	12.5' - 15'	9	a	Tonalite	2.41	1.45	2.1	4.44	487	11935	Y
K-116	12.5' - 15'	10	i	Tonalite	2.41	0.89	1.3	2.73	424	10380	Y

Point Load Strength Index

$I_{s(50)}$ **500 psi** or **72,000 psf** or **3.45 MPa**

Uniaxial Compressive Strength

σ_c= **12,260 psi** or **1,765,440 psf** or **85 MPa**

*Test Type
d = diametral
a = axial
b = block
i = irregular lump

KLEINFELDER Bright People. Right Solutions.	PROJECT NO.: 00136146 DRAWN BY: JDS CHECKED BY: DA DATE: 10/9/2014 REVISED: -	POINT LOAD STRENGTH INDEX TEST RESULTS ROCKRIDGE SAFEWAY 51st & BROADWAY STORE NO. 3132 OAKLAND, CALIFORNIA	TABLE B-14

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Point Load Strength Index Test Results ASTM D-5731

KLEINFELDER Bright People. Right Solutions.	PROJECT NO.: 00136146 DRAWN BY: JDS CHECKED BY: OK DATE: 9/24/2014 REVISED: -	POINT LOAD STRENGTH INDEX TEST RESULTS ROCKRIDGE SAFEWAY 51st & BROADWAY STORE NO. 3132 OAKLAND, CALIFORNIA	PLATE B-15

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Data checked by:_CRL____________ Date:_09/30/14___________

Point Load Strength Index Test Results ASTM D-5731

PROJECT:	Safeway Rockridge Oakland	LAB SAMPLE NO.:	11605C
PROJECT NO.:	136146.001A	SAMPLE NO.:	K-108 17.5' - 20'
PROJECT LOCATION:	Oakland, CA	SAMPLE DESCRIP:	Tonalite
SAMPLED BY:	KLF	DATE TESTED:	9/29/2014
DATE SAMPLED:	9/14	REPORTED BY:	B. Kochanski

Boring No.	Depth (ft)	Test Number	Test Type*	Rock Type	Width, W (in)	Depth or Diameter, D (in)	Failure Load, P (lbs)	De^2 (in^2)	Point Load Strength Index, $I_{s(50)}$ (PSI)	Uniaxial Compressive Strength, UCS (PSI)	Valid/ Invalid
K-108	17.5' - 20'	1	i	Tonalite	2.40	0.95	0.2	2.92	60	1477	Y
K-109	17.5' - 20'	2	i	Tonalite	2.40	0.92	0.1	2.83	48	1169	Y
K-110	17.5' - 20'	3	i	Tonalite	2.39	0.45	0.3	1.36	158	3880	Y
K-111	17.5' - 20'	4	i	Tonalite	2.41	1.12	0.1	3.45	24	576	Y
K-112	17.5' - 20'	5	i	Tonalite	2.40	0.57	0.2	1.75	105	2575	Y
K-113	17.5' - 20'	6	i	Tonalite	2.37	0.85	0.2	2.55	79	1948	Y
K-114	17.5' - 20'	7	i	Tonalite	2.40	0.91	0.2	2.79	78	1911	Y
K-115	17.5' - 20'	8	i	Tonalite	2.14	0.58	0.1	1.58	73	1798	Y
K-116	17.5' - 20'	9	i	Tonalite	1.90	1.14	0.2	2.75	70	1720	Y
K-117	17.5' - 20'	10	i	Tonalite	1.60	0.72	0.3	1.46	152	3717	Y

Point Load Strength Index

$I_{s(50)}$ **78 psi** or **11,232 psf** or **0.54 MPa**

Uniaxial Compressive Strength

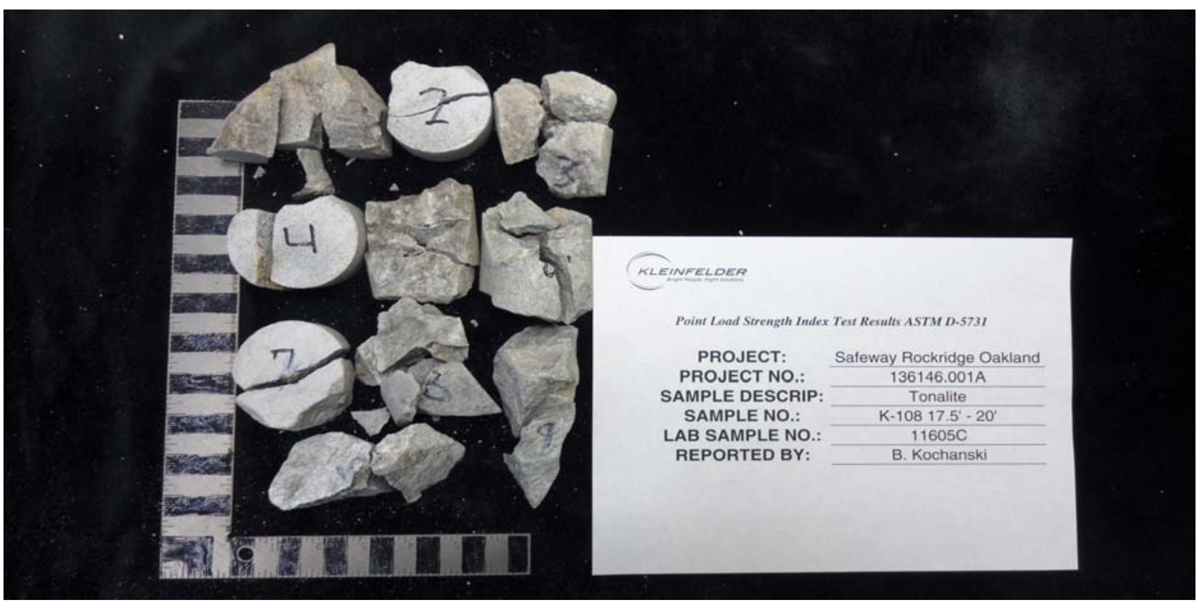
σ_c= **1,905 psi** or **274,320 psf** or **13 MPa**

*Test Type
d = diametral
a = axial
b = block
i = irregular lump

KLEINFELDER Bright People. Right Solutions.	PROJECT NO.: 00136146 DRAWN BY: JDS CHECKED BY: DA DATE: 10/9/2014 REVISED: -	POINT LOAD STRENGTH INDEX TEST RESULTS ROCKRIDGE SAFEWAY 51st & BROADWAY STORE NO. 3132 OAKLAND, CALIFORNIA	TABLE B-16

PLOTTED: 10/08/2014 05:19 PM BY: jsala

Point Load Strength Index Test Results ASTM D-5731



KLEINFELDER Bright People. Right Solutions.	PROJECT NO.: 00136146 DRAWN BY: JDS CHECKED BY: OK DATE: 9/24/2014 REVISED: -	POINT LOAD STRENGTH INDEX TEST RESULTS ROCKRIDGE SAFEWAY 51st & BROADWAY STORE NO. 3132 OAKLAND, CALIFORNIA	PLATE B-17

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Data checked by:_CRL____________ Date:_09/30/14___________

Point Load Strength Index Test Results ASTM D-5731

PROJECT:	**Safeway Rockridge Oakland**	**LAB SAMPLE NO.:**	11605D
PROJECT NO.:	**136146.001A**	**SAMPLE NO.:**	K-112 13' - 15'
PROJECT LOCATION:	**Oakland, CA**	**SAMPLE DESCRIP:**	Tonalite
SAMPLED BY:	**KLF**	**DATE TESTED:**	9/30/2014
DATE SAMPLED:	**9/14**	**REPORTED BY:**	B. Kochanski

Boring No.	Depth (ft)	Test Number	Test Type*	Rock Type	Width, W (in)	Depth or Diameter, D (in)	Failure Load, P (lbs)	De^2 (in^2)	Point Load Strength Index, $I_{s(50)}$ (PSI)	Uniaxial Compressive Strength, UCS (PSI)	Valid/ Invalid
K-112	13' - 15'	1	a	Tonalite	2.41	1.13	1.0	3.46	289	7089	Y
K-113	13' - 15'	2	a	Tonalite	2.40	1.18	2.2	3.60	591	14485	Y
K-114	13' - 15'	3	a	Tonalite	2.40	1.12	1.8	3.42	523	12804	Y
K-115	13' - 15'	4	i	Tonalite	2.39	1.42	0.5	4.31	130	3197	Y
K-116	13' - 15'	5	i	Tonalite	2.39	1.19	2.5	3.61	673	16497	Y
K-117	13' - 15'	6	a	Tonalite	2.39	1.15	2.0	3.49	564	13815	Y
K-118	13' - 15'	7	a	Tonalite	2.39	1.61	2.0	4.92	423	10355	Y
K-119	13' - 15'	8	a	Tonalite	2.39	1.17	0.8	3.55	229	5607	Y
K-120	13' - 15'	9	i	Tonalite	2.41	1.57	0.5	4.82	116	2835	Y
K-121	13' - 15'	10	i	Tonalite	1.84	1.45	1.0	3.40	284	6963	Y

Point Load Strength Index

$I_{s(50)}$ **385 psi** or **55,440 psf** or **2.66 MPa**

Uniaxial Compressive Strength

σ_c= **9,439 psi** or **1,359,216 psf** or **65 MPa**

*Test Type
d = diametral
a = axial
b = block
i = irregular lump

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PROJECT NO.: 00136146	POINT LOAD STRENGTH INDEX TEST RESULTS	TABLE
DRAWN BY: JDS		B-18
CHECKED BY: DA	ROCKRIDGE SAFEWAY 51st & BROADWAY STORE NO. 3132 OAKLAND, CALIFORNIA	
DATE: 10/9/2014		
REVISED: -		

PLOTTED: 10/08/2014 05:19 PM BY: jsala

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Point Load Strength Index Test Results ASTM D-5731

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KLEINFELDER Bright People. Right Solutions.	PROJECT NO.: 00136146	POINT LOAD STRENGTH INDEX TEST RESULTS	PLATE
	DRAWN BY: JDS		B-19
	CHECKED BY: OK	ROCKRIDGE SAFEWAY 51st & BROADWAY STORE NO. 3132 OAKLAND, CALIFORNIA	
	DATE: 9/24/2014		
	REVISED: -		

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Data checked by:_CRL____________ Date:_09/30/14___________

Point Load Strength Index Test Results ASTM D-5731

PROJECT:	**Safeway Rockridge Oakland**	**LAB SAMPLE NO.:**	11605E
PROJECT NO.:	**136146.001A**	**SAMPLE NO.:**	K-114 @ 2.5' - 5'
PROJECT LOCATION:	**Oakland, CA**	**SAMPLE DESCRIP:**	Tonalite
SAMPLED BY:	**KLF**	**DATE TESTED:**	9/30/2014
DATE SAMPLED:	**9/14**	**REPORTED BY:**	B. Kochanski

Boring No.	Depth (ft)	Test Number	Test Type*	Rock Type	Width, W (in)	Depth or Diameter, D (in)	Failure Load, P (lbs)	De^2 (in^2)	Point Load Strength Index, $I_{s(50)}$ (PSI)	Uniaxial Compressive Strength, UCS (PSI)	Valid/ Invalid
K-114	2.5' - 5'	1	i	Tonalite	2.40	0.78	0.3	2.39	118	2881	Y
K-115	2.5' - 5'	2	i	Tonalite	2.34	0.73	0.1	2.18	57	1392	Y
K-116	2.5' - 5'	3	i	Tonalite	2.40	2.40	0.1	7.31	19	468	Y
K-117	2.5' - 5'	4	i	Tonalite	2.39	1.61	0.5	4.90	118	2892	Y
K-118	2.5' - 5'	5	i	Tonalite	1.77	0.96	0.2	2.16	79	1942	Y
K-119	2.5' - 5'	6	i	Tonalite	2.39	1.30	1.7	3.95	423	10353	Y
K-120	2.5' - 5'	7	i	Tonalite	2.40	0.80	2.0	2.44	729	17862	Y
K-121	2.5' - 5'	8	i	Tonalite	2.39	1.00	0.5	3.04	166	4066	Y
K-122	2.5' - 5'	9	i	Tonalite	2.42	1.51	0.4	4.65	88	2151	Y
K-123	2.5' - 5'	10	i	Tonalite	2.40	1.78	1.3	5.42	255	6238	Y

Point Load Strength Index

$I_{s(50)}$ **137 psi** or **19,728 psf** or **0.94 MPa**

Uniaxial Compressive Strength

σ_c= **3,362 psi** or **484,128 psf** or **23 MPa**

*Test Type
d = diametral
a = axial
b = block
i = irregular lump

KLEINFELDER
Bright People. Right Solutions.

PROJECT NO.: 00136146	POINT LOAD STRENGTH INDEX TEST RESULTS	TABLE	
DRAWN BY: JDS			B-20
CHECKED BY: DA	ROCKRIDGE SAFEWAY 51st & BROADWAY STORE NO. 3132 OAKLAND, CALIFORNIA		
DATE: 10/9/2014			
REVISED: -			

KLEINFELDER
Bright People. Right Solutions.

Point Load Strength Index Test Results ASTM D-5731

KLEINFELDER Bright People. Right Solutions.	PROJECT NO.: 00136146 DRAWN BY: JDS CHECKED BY: OK DATE: 9/24/2014 REVISED: -	POINT LOAD STRENGTH INDEX TEST RESULTS ROCKRIDGE SAFEWAY 51st & BROADWAY STORE NO. 3132 OAKLAND, CALIFORNIA	PLATE B-21

APPENDIX C

CORROSIVITY ANALYSIS RESULTS

AP Engineering & Testing, Inc.

CORROSION TEST RESULTS

Client Name: Kleinfelder
Project Name: PDC Rockridge GEO
Project No.: 136146

AP Job No.: 13-0917
Date 09/11/13

Boring No.	Sample No.	Depth (feet)	Soil Type	Minimum Resistivity (ohm-cm)	pH	Sulfate Content (ppm)	Chloride Content (ppm)
K-3	1a & 1b	3-4	CL	3263	7.2	216	224
K-7	1a & 1b	3-4	CL	3283	8.1	74	207

NOTES:
Resistivity Test and pH: California Test Method 643
Sulfate Content : California Test Method 417
Chloride Content : California Test Method 422
ND = Not Detectable
NA = Not Sufficient Sample
NR = Not Requested

2607 Pomona Boulevard, Pomona, CA 91768
Tel. (909) 869-6316 Fax. (909)869-6318

CERCO analytical
1100 Willow Pass Court, Suite A
Concord, CA 94520-1006
925 **462 2771** Fax. 925 **462 2775**
www.cercoanalytical.com

23 September, 2014

Job No.1409175
Cust. No.10527

Mr. Omar Khan
Kleinfelder
4670 Willow Road, Ste. 100
Pleasanton, CA 94566

Subject: Project No.: 136146
Project Name: Rockridge Safeway, 51st & Broadway, Oakland
Corrosivity Analysis – ASTM Test Methods

Dear Mr. Khan:

Pursuant to your request, CERCO Analytical has analyzed the soil sample submitted on September 18, 2014. Based on the analytical results, this brief corrosivity evaluation is enclosed for your consideration.

Based upon the 100% saturated resistivity measurement, this sample is classified as "corrosive". All buried iron, steel, cast iron, ductile iron, galvanized steel and dielectric coated steel or iron should be properly protected against corrosion depending upon the critical nature of the structure. All buried metallic pressure piping such as ductile iron firewater pipelines should be protected against corrosion.

The chloride ion concentration reflects none detected with a detection limit of 15 mg/kg.

The sulfate ion concentration is 230 mg/kg and is determined to be sufficient to potentially be detrimental to reinforced concrete structures and cement mortar-coated steel at these locations. Therefore, concrete that comes into contact with this soil should use sulfate resistant cement such as Type II, with a maximum water-to-cement ratio of 0.55.

The sulfide ion concentrations reflect none detected with a detection limit of 50 mg/kg.

The pH of the soil is 8.11, which does not present corrosion problems for buried iron, steel, mortar-coated steel and reinforced concrete structures.

The redox potential is 400-mV, is indicative of potentially "slightly corrosive" soils resulting from anaerobic soil conditions.

This corrosivity evaluation is based on general corrosion engineering standards and is non-specific in nature. For specific long-term corrosion control design recommendations or consultation, please call *JDH Corrosion Consultants, Inc. at (925) 927-6630.*

We appreciate the opportunity of working with you on this project. If you have any questions, or if you require further information, please do not hesitate to contact us.

Very truly yours,
CERCO ANALYTICAL, INC.

Cheryl McMillen for

J. Darby Howard, Jr., P.E.
President

JDH/jdl
Enclosure

California State Certified Laboratory No. 2153

CERCO analytical
1100 Willow Pass Court, Suite A
Concord, CA 94520-1006
925 462 2771 Fax. 925 462 2775
www.cercoanalytical.com

Client: Kleinfelder
Client's Project No.: 136146
Client's Project Name: Rockridge Safeway, 51st & Broadway, Oakland
Date Sampled: 11-Sep-14
Date Received: 18-Sep-14
Matrix: Soil
Authorization: Signed Chain of Custody

Date of Report: 23-Sep-2014

Job/Sample No.	Sample I.D.	Redox (mV)	pH	Resistivity ("As Received") (ohms-cm)	Resistivity (100% Saturation) (ohms-cm)	Sulfide (mg/kg)*	Chloride (mg/kg)*	Sulfate (mg/kg)*
1409175-001	K111, Sample 1B @3-6'	400	8.11	22,000	1,600	N.D.	N.D.	230

Method:	ASTM D1498	ASTM D4972	ASTM G57	ASTM G57	ASTM D4658M	ASTM D4327	ASTM D4327
Detection Limit:	-	-	-	-	50	15	15
	22-Sep-2014	22-Sep-2014	22-Sep-2014	22-Sep-2014	22-Sep-2014	22-Sep-2014	22-Sep-2014

Cheryl McMillen
Cheryl McMillen
Laboratory Director

* Results Reported on "As Received" Basis
N.D. - None Detected

<u>Quality Control Summary</u> - All laboratory quality control parameters were found to be within established limits

APPENDIX D

SEISMIC REFRACTION RESULTS

ADVANCED GEOLOGICAL SERVICES

1605 School Street, #4
Moraga CA 94556
925 (808-8965)

Andi J. Bord
Kleinfelder
40 Clark Street, Suite J
Salinas, CA 93901

Subject: Report
Seismic Refraction Survey
Proposed Safeway #3132 Remodel
5130 Broadway, Oakland, California

Dear Ms. Bord:

1.0 INTRODUCTION

This letter presents the results of Advanced Geological Services, Inc. (AGS) seismic refraction survey in support of Kleinfelder's geotechnical investigation for the proposed Safeway #3132 remodeling project at 5130 Broadway in Oakland, California (Figure 1). The investigation objective was to assess the depth to bedrock.

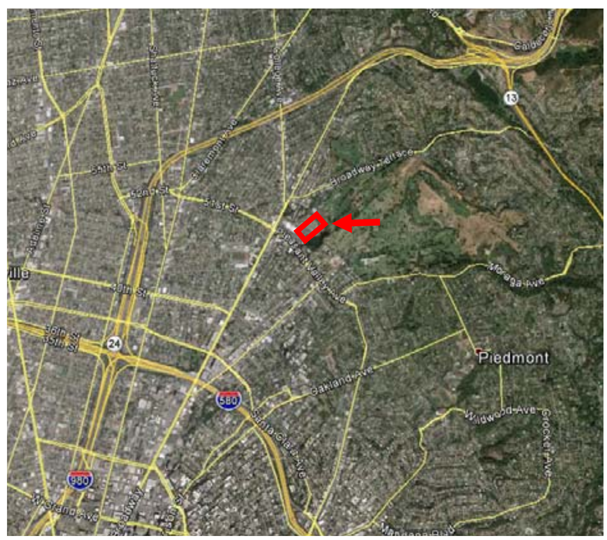


Figure 1 - Site Location- Oakland, California

The primary investigation technique used to assess bedrock depth was seismic refraction. Seismic refraction data were obtained along four lines located in the Safeway parking lot. As a back-up to the refraction work, AGS also performed additional data processing to analyze seismic surface-wave data, which was collected along with the refraction data. Surface-wave processing was performed to look for changes in shear (S-) wave velocity with depth that could also indicate bedrock depth. The investigation was performed during the night of September 11 and 12, 2013 by AGS senior geophysicist Roark Smith, with invaluable assistance provided by Mr. Dan Dockendorf of Kleinfelder.

2.0 SUMMARY OF FINDINGS

- The seismic refraction survey results indicate that the overall bedrock depth ranges from 0 to 60 feet below ground surface (bgs), with the top-of-bedrock surface mostly occurring in the 20- to 40-foot bgs range.

- Two velocity layers were identified; they are designated as Layers V_1 and V_2. Layer V_1

is a 0- to 60-foot thick upper layer exhibiting P-wave velocities between 2,300 and 4,600 feet per second (fps) that is interpreted to represent fill material. Layer V_2, the lower layer, exhibits P-wave velocities between 12,000 and 14,800 fps and is interpreted to represent bedrock.

- An anomalously low V_2 velocity was observed along seismic line SL-2, located in the center of the parking lot. The low V_2 velocity along SL-2 could mean that bedrock is highly weathered and/or fractured in this area. It could also mean that bedrock is deeper than the investigation depth of the refraction survey, which is estimated to be approximately 60 or 70 feet bgs.

- The surface-wave data indicate that bedrock depth is on the order of 45 feet bgs and that Vs30 (the average shear-wave velocity of the upper 30 meters of subsurface) is approximately 1,025 fps. The surface-wave data also indicate the presence of a velocity inversion in the subsurface, wherein a layer of higher-velocity material overlies lower velocity material. It is worth noting that a velocity inversion violates one of the prime assumptions of the refraction method (that velocity increases with depth) and could result in a velocity layer model with erroneous layer depths.

3.0 SITE DESCRIPTION

The investigation was performed within the 300- by 500-foot topographically flat asphalt-paved parking lot between the Safeway and CVS store buildings (Figure 2). To keep vehicles from running over the equipment, the parking lot was cordoned off with traffic cones, and, to minimize vibration noise from passing traffic, the field work was performed at night.

4.0 FIELD PROCEDURES

The four planned seismic lines were shown on a map provided by Kleinfelder. For each line (SL-1 through SL-4), AGS first laid a fiberglass tape measure on the ground surface along the planned survey line. AGS then placed 24 geophones on the ground at 10-foot spacings to form a 230-foot long seismic line. Because the survey was performed on asphalt pavement, the 4-inch long spikes on each geophone were replaced with metal base plates, which were screwed onto the threaded bolt that normally receives the geophone spike. The geophone-plate assembly was secured to the pavement with quick-drying modeling clay. Three shot points were used for each line— one was placed in the middle of the geophone array and one was placed at each end of the array, 5 feet beyond the nearest geophone.

AGS produced seismic energy through multiple impacts with a 16-lb sledge hammer against the asphalt pavement at each shotpoint location. Ten to 15 hammer blows were used (“stacked”) at the end-of-line shotpoints and five blows were used at the mid-line shotpoints. The P-waves produced by the hammer impacts were detected using 4.5-Hz geophones from GeoSpace Corp. The detected seismic signals were recorded using a Geometrics StrataView R24 signal enhancement seismograph.

After seismic refraction data were collected along the first line, AGS moved the tape measure to the second line (SL-2) and repositioned the geophones along the second line and collected refraction data in same manner as at the first line. The process was repeated for the third and fourth lines.

5.0 GEOPHYSICAL METHOD OVERVIEW

5.1 Seismic Refraction

The seismic refraction method uses compressional (P-) wave energy to delineate seismic velocity layers within the subsurface. Interpretation entails correlating the velocity layers to geologic features such as soil and various types of bedrock. To perform a refraction survey, an elastic wave (compressional, or P-wave) is generated at certain locations (shotpoints) along a survey line. The P-wave energy is usually produced by striking the ground with a sledgehammer, although for deeper investigations with a small explosive charge can be used. As the P-wave propagates through the ground it is refracted along boundaries between geologic layers with different seismic velocities.

Part of the refracted P-wave energy returns to the ground surface where it is detected by vibration-sensitive devices called geophones, which are placed in a co-linear array along the seismic survey line. The geophone data are fed to a seismograph, where they are recorded, and then to a computer, where they are analyzed to determine the depth and velocities of subsurface seismic layers. Key data for refraction analysis are the positions of the geophones and shotpoints along a seismic line, and the amount of time it takes for the refracted wave to travel from the shotpoint to each geophone location. Because the P-wave is the fastest traveling of all types of seismic waves, it can be readily identified as the first deflection ("first break") on a seismic trace.

Additional discussion of the refraction method, its limitations, and the relationship between seismic velocity and geologic materials is presented in Appendix A.

5.2 Seismic Surface Wave

Seismic surface-wave surveys use essentially the same field set-up as a refraction survey, but a different part of the recorded seismic signal— the Rayleigh (surface) waves is analyzed instead of the P-wave. Briefly, a surface-wave survey entails measuring the velocity of surface waves using an array of motion detectors (geophones) placed on the ground surface. Because surface-wave velocity closely follows shear-wave velocity (90 to 95% of V_S), surface-wave velocity data can be used to estimate shear wave velocity (V_S). Surface-Waves are seismic waves that travel along or near the surface of the earth; they are generated by both natural (e.g., wind, ocean waves) and man-made (e.g., hammer blow, traffic noise, factory vibration) sources. Surface-Waves travel in assemblages of frequencies, with each frequency having a corresponding wavelength. Because surface-waves are influenced by subsurface material to a depth approximately equal to the surface-wave's wavelength, a velocity vs. depth profile can be generated by measuring the velocity of surface-waves of varying wavelengths. Short wavelengths (higher frequencies) respond to the material properties (e.g., stiffness) of shallower materials while longer wavelengths (lower frequency) respond to deeper materials.

Specialized computer software is used to identify surface-waves in the recorded data and prepare a 'velocity spectrum' image, which the geophysical analyst interprets to produce a 'dispersion curve' to depict how velocity varies with frequency. The geophysicist then prepares a velocity layer model from which a synthetic dispersion curve is produced. The analyst then adjusts the model to obtain a 'best fit' between the synthetic dispersion curve and the actual dispersion curve that was interpreted from the velocity spectrum. The degree or closeness of the fit between the interpreted and synthetic curves provides an indication of how well the model represents actual subsurface conditions.

Surface-wave surveys produce a 1-dimenstional (1-D) profile showing S-wave velocity variations with depth at a point that is taken to be at the center of the geophone array.

6.0 DATA PROCESSING AND ANALYSIS

The seismic refraction data quality for this project was fair to good and first break picks were made with confidence. Data quality was enhanced by "stacking," which entailed using multiple hammer blows at each shotpoint location to improve the signal-to-noise ratio. The additive affect of stacking of multiple hammer blows at the same location enhances or increases the amplitude of the signal (i.e., the refracted wave arrival) while amplitude of the background noise, which, being random in nature, tends to cancel itself on successive hammer blows and remains largely unchanged. Stacking was made necessary by the wind gusts and vibratory noise from vehicle traffic along the nearby roadways. AGS stacked 10 hammer blows at the end-of-line shotpoints and 5 blows at the center shotpoint.

Seismic data were transferred from the seismograph to a desktop computer where they were processed using the *SeisImager* and *SeisImager/SW* software packages by Geometrics, Inc. to process the refraction and surface-wave data, respectively. Briefly, *SeisImager* is a computer inversion program that generates an initial velocity layer model, produces synthetic data from the model, and then adjusts the model so that the synthetic data better matches the observed field data. The agreement between the synthetic and observed data provides an indication of how well the model represents the true subsurface conditions.

6.1 Refraction Processing and Analysis

First, AGS used the *SeisImager* module *PickWin* to interpret ("pick") the P-wave arrivals ("first breaks") for each of the shotpoint data sets ("shot gathers") per line. *PickWin* was also used to check (against the geophysicst's field log) that the proper locations were assigned to the geophones and shotpoints. Next, the first break files were fed to the SeisImager module *PlotRefra*, which was used review time-distance (TD) plots for the two seismic lines and assign a seismic layer to each arrival time. For the initial refraction analysis, each P-wave arrival is considered to have refracted from a distinct seismic layer. The number of layers resolved by the seismic survey, and their thickness and average velocity, are revealed by straight line segments on the TD plot; because these straight-line segments represent a constant velocity condition within the subsurface, they often represent a distinct geologic layer. Normally, data depicting topographic variations along the seismic line are incorporated at this point; however, because the parking lot site was topographically flat, no topographic data were obtained. Next, a time-term inversion was performed to produce layered velocity models.

6.2 Surface-Wave Processing and Analysis

In general, surface wave data processing entails first producing a velocity spectrum image, which shows the phase velocity for the various frequencies of surface waves detected (Figure 7). This image is used as the basis for interpreting ("picking") a dispersion curve, which is a graph that depicts how surface-wave velocity varies with frequency (hence, depth). The dispersion curve is then used to prepare an initial 1D model of surface-wave velocity versus depth using a one-third wavelength approximation (i.e., a given phase velocity is assigned to a depth that is one-third of the wavelength of the corresponding surface-wave). The initial velocity layer model is then adjusted using an inversion process until the corresponding synthetic dispersion curve achieves a "best-fit" match to the original dispersion curve that was interpreted from the observed data (i.e., the velocity spectrum image). The degree or closeness of the fit between the interpreted and synthetic curves provides an indication of how well the model represents actual subsurface conditions.

The seismic surface-wave data were processed using *SeisImager/SW,* which comprises the software models *Pickwin*, and *WaveEq*. *Pickwin* displays the raw field data and the corresponding velocity spectrum image, and it enables the geophysical analyst to pick a dispersion curve. *Pickwin* automatically creates a dispersion curve by picking the mathematical maximum amplitude for each frequency. *WaveEq* was then used to prepare the initial velocity layer model from the interpreted dispersion curve and perform the subsequent inversion that refines the initial model into the final "best fit" model. Inputs to *WaveEq* included the number of layers and the number of iterations to be performed by the inversion process. AGS specified 6 layers and 10 inversions.

7.0 RESULTS

The investigation results are presented on Figures 2 through 7. Figure 2 is a site map showing the seismic line locations. Figures 3 through 6 present compressional (P-) wave velocity layer models generated from the seismic refraction data. Figure 7 presents the seismic surface-wave survey results for two of the seismic lines (SL-2 and SL-3). The upper portion of the Figure 7 contains 1-D models that show S-wave velocity variations with depth while the lower portion of the figure shows the velocity spectrum images and associated dispersion curves from which the models were generated.

In general, the seismic refraction survey results indicate that the bedrock depth ranges from 0 to 60 feet below ground surface (bgs), with the top-of-bedrock surface mostly occurring in the 20- to 40-foot bgs range. Two velocity layers were identified; they are designated as Layers V_1 and V_2. Layer V_1 is a 0- to 60-foot thick upper layer exhibiting P-wave velocities between 2,300 and 4,600 feet per second (fps) that is interpreted to represent fill material. Layer $V_{2,}$ the lower layer, exhibits P-wave velocities between 12,000 and 14,800 fps and is interpreted to represent bedrock.

An anomalously low V_2 velocity of 3,600 fps was observed along seismic line SL-2, located in the center of the parking lot. The low V_2 velocity along SL-2 could mean that bedrock is highly weathered and/or fractured in this area. It could also mean that bedrock is deeper than the investigation depth of the refraction survey, which is estimated to be approximately 60 or 70 feet bgs.

The surface-wave data indicate that bedrock depth is on the order of 45 feet bgs. As shown on Figure 7, the onset of bedrock is indicated by a 300 fps increase in S-wave velocity, from approximately 900 fps to over 1,200 fps. The surface-wave data also indicate that Vs30 is approximately 1,025 fps. Vs30 is the average shear-wave velocity of the upper 30 meters of subsurface, a value that is used to establish a site's IBC seismic site classification. A value of 1,025 fps places the site in class D, with a profile name of "stiff soil."

Both the surface-wave data and the TD plots of the refraction arrival-time data indicate the presence of a velocity inversion in the subsurface, wherein a layer of higher-velocity material overlies lower velocity material. It is worth noting that a velocity inversion violates one of the prime assumptions of the refraction method (that velocity increases with depth) and could result in velocity layer models with erroneous layer depths.

8.0 CLOSING

All geophysical data and field notes collected as a part of this investigation will be archived at the AGS office. The data collection and interpretation methods used in this investigation are consistent with standard practices applied to similar geophysical investigations. The correlation of geophysical responses with probable subsurface features is based on the past results of similar surveys although it is possible that some variation could exist at this site. Due to the nature of geophysical data, no guarantees can be made or implied regarding the targets identified or the presence or absence of additional objects or targets.

AGS appreciates working for you and we look forward to working with you again.

Sincerely,

Roark W. Smit
Senior Geophysicist
Advanced Geological Services, Inc.

Figures:	Figure 1	Site Location (imbedded in Report text)
	Figure 2	Site Map and Seismic Survey Lines
	Figure 3	Seismic Refraction Survey Results, SL-1
	Figure 4	Seismic Refraction Survey Results, SL-2
	Figure 5	Seismic Refraction Survey Results, SL-3
	Figure 6	Seismic Refraction Survey Results, SL-4
	Figure 7	Surface-Wave Survey Results, SL-2 and SL-3

Attachments: Appendix A: Seismic Velocity and Limitations of the Refraction Method

APPENDIX A

SEISMIC VELOCITY AND LIMITATIONS OF THE REFRACTION METHOD

The physical properties of earth materials (fill, sediment, rock) such as compaction, density, hardness, and induration dictate the corresponding seismic velocity of the material. Additionally, other factors such as bedding, fracturing, weathering, and saturation can also affect seismic velocity. In general, low velocities indicate loose soil, poorly compacted fill material, poorly to semi-consolidated sediments, deeply weathered, and highly fractured rock. Conversely, high velocities are indicative of competent rock or dense and highly compacted sediments and fill. The highest velocities are measured in unweathered and little fractured rock.

There are certain limitations associated with the seismic refraction method as applied for this investigation. These limitations are primarily based on assumptions that are made by the data analysis routine. The data analysis routine assumes that the velocities along the length of each spread are uniform. If there are localized zones within each layer where the velocities are higher or lower than indicated, the analysis routine will interpret these zones as changes in the surface topography of the underlying layer. A zone of higher velocity material would be interpreted as a low in the surface of the underlying layer. Zones of lower velocity material would be interpreted as a high in the underlying layer. The data analysis routine also assumes that the velocity of subsurface materials increase with depth. Therefore, if a layer exhibits velocities that are slower than those of the material above it, the slower layer will not be resolved. Also, a velocity layer may simply be too thin to be detected.

The quality of the field data is critical to the construction of an accurate depth and velocity profile. Strong, clear “first-break” information from refracted interfaces will make the data processing, analysis, and interpretation much more accurate and meaningful. Vibrational noise or poor subsurface conditions can decrease the ability to accurately locate and pick seismic waves from the interfaces.

Due to these and other limitations inherent to the seismic refraction method, resultant velocity cross-sections should be considered only as approximations of the subsurface conditions. The actual conditions may vary locally.

EXPLANATION

SEISMIC REFRACTION LINE

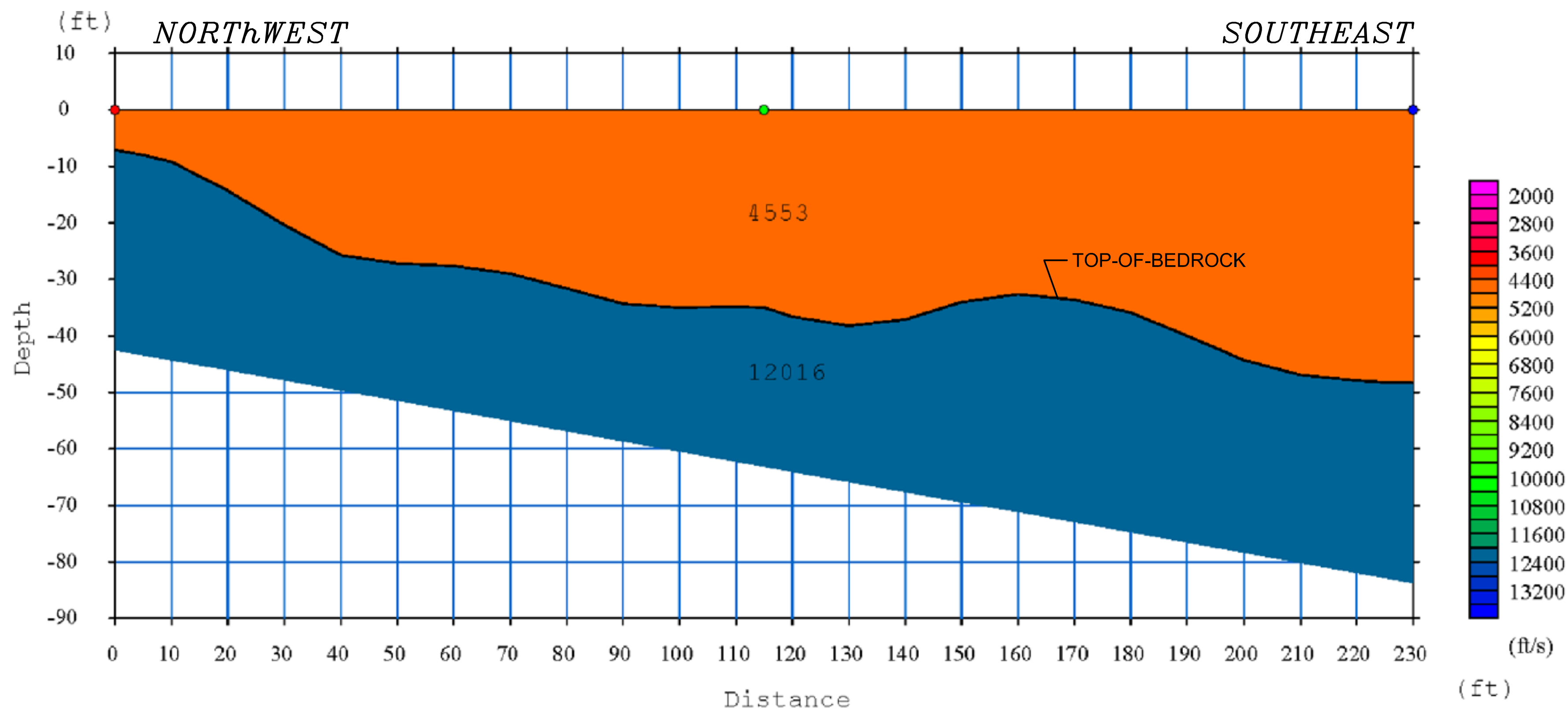
SHOTPOINT LOCATION

ADVANCED GEOLOGICAL SERVICES

1605 School Street
Suite 4
Moraga, CA 94556
(925) 808-8965

Seismic Survey Line Locations
Proposed Safeway #3132 Remodel
5130 Broadway

LOCATION: Oakland, California	
CLIENT: Kleinfelder	FIGURE
PROJECT #: 13-058-1CA	2
DATE: Sept 20, 2013	DRAWN BY: R. SMITH



ADVANCED GEOLOGICAL SERVICES

1605 School Street
Suite 4
Moraga, CA 94556
(925) 808–8965

Seismic Refraction Line SL-1
Velocity Layer Model
Proposed Safeway #3132 Remodel
5130 Broadway

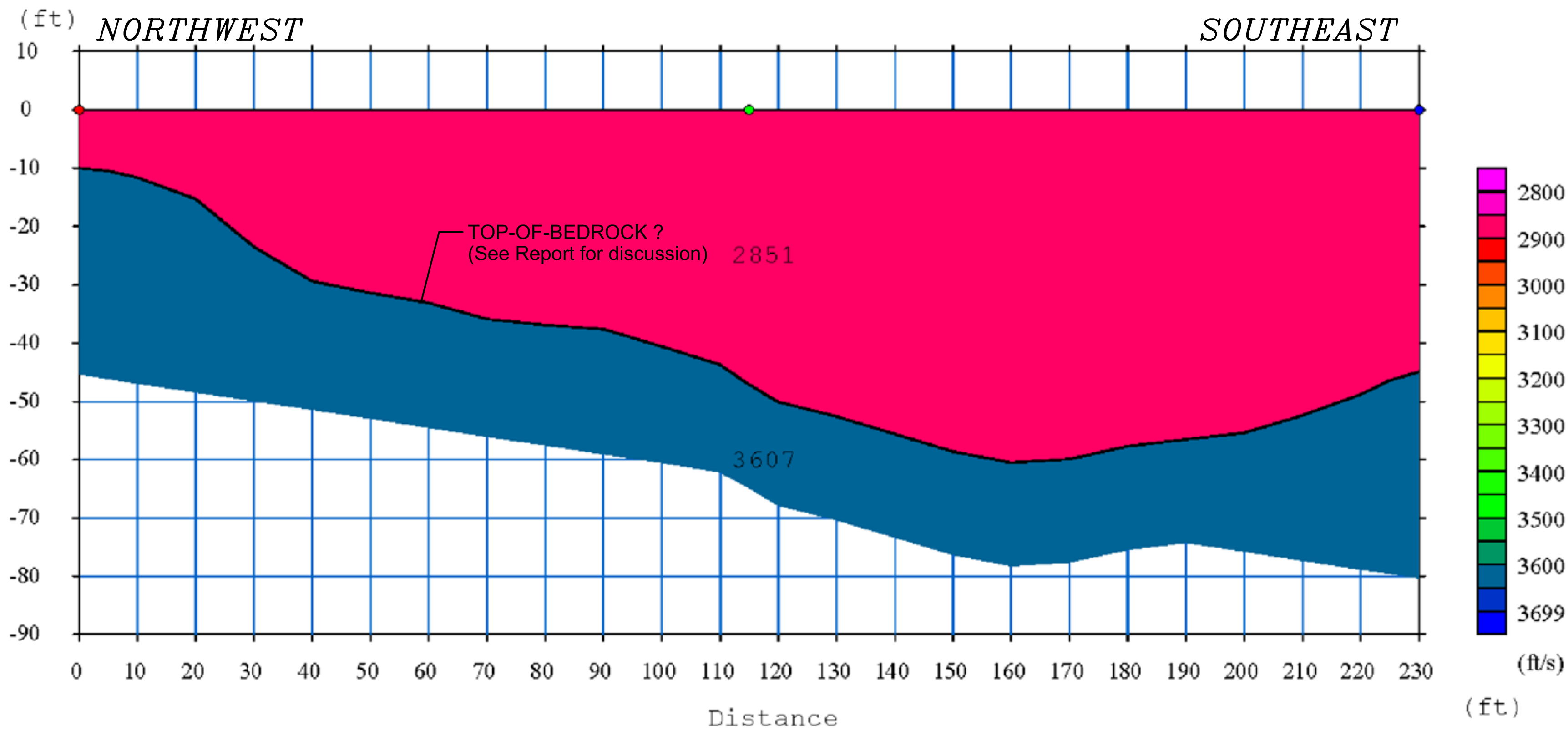
LOCATION: Oakland, California

CLIENT: Kleinfelder

PROJECT #: 13–058–1CA

DATE: Sept 20, 2013 DRAWN BY: R. SMITH

FIGURE 3



ADVANCED GEOLOGICAL SERVICES

1605 School Street
Suite 4
Moraga, CA 94556
(925) 808-8965

Seismic Refraction Line SL-2
Velocity Layer Model
Proposed Safeway #3132 Remodel
5130 Broadway

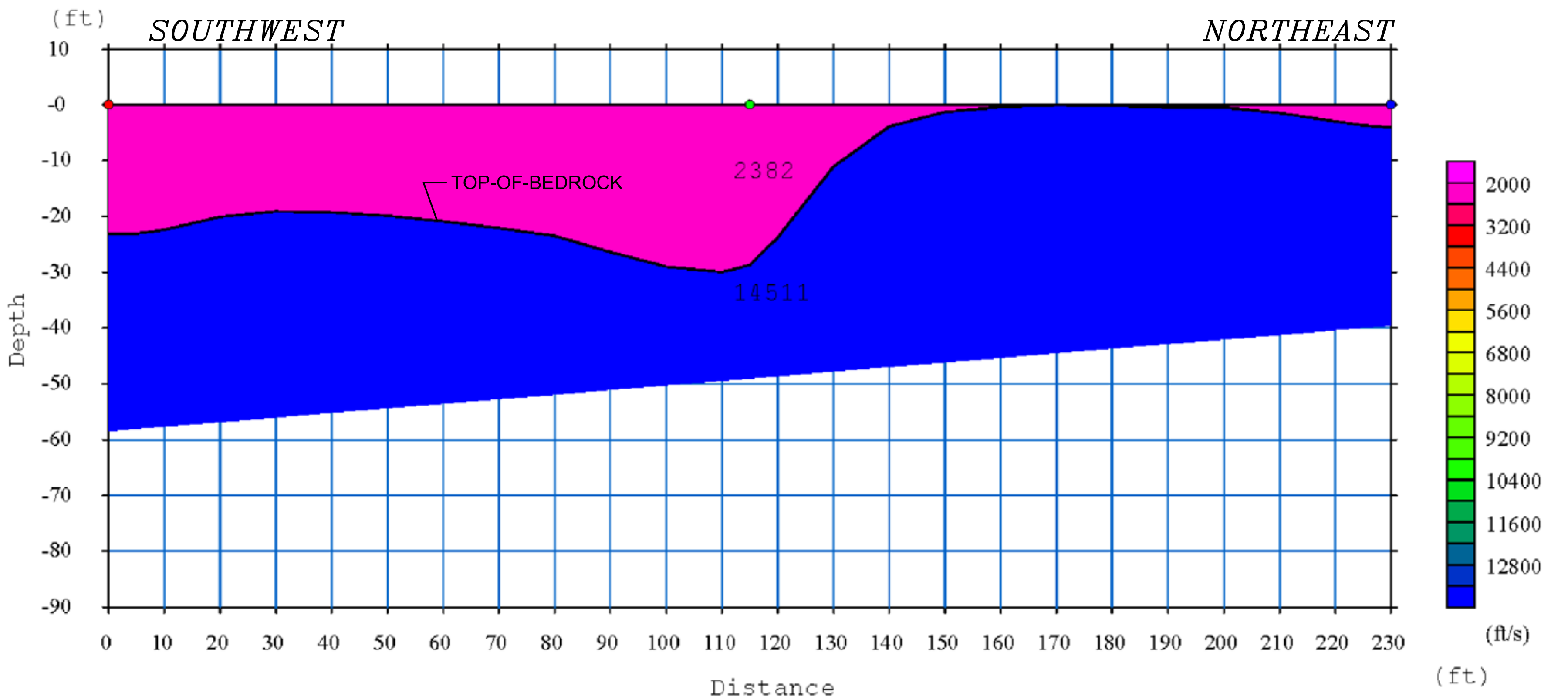
LOCATION: Oakland, California

CLIENT: Kleinfelder

PROJECT #: 13-058-1CA

DATE: Sept 20, 2013 DRAWN BY: R. SMITH

FIGURE
4

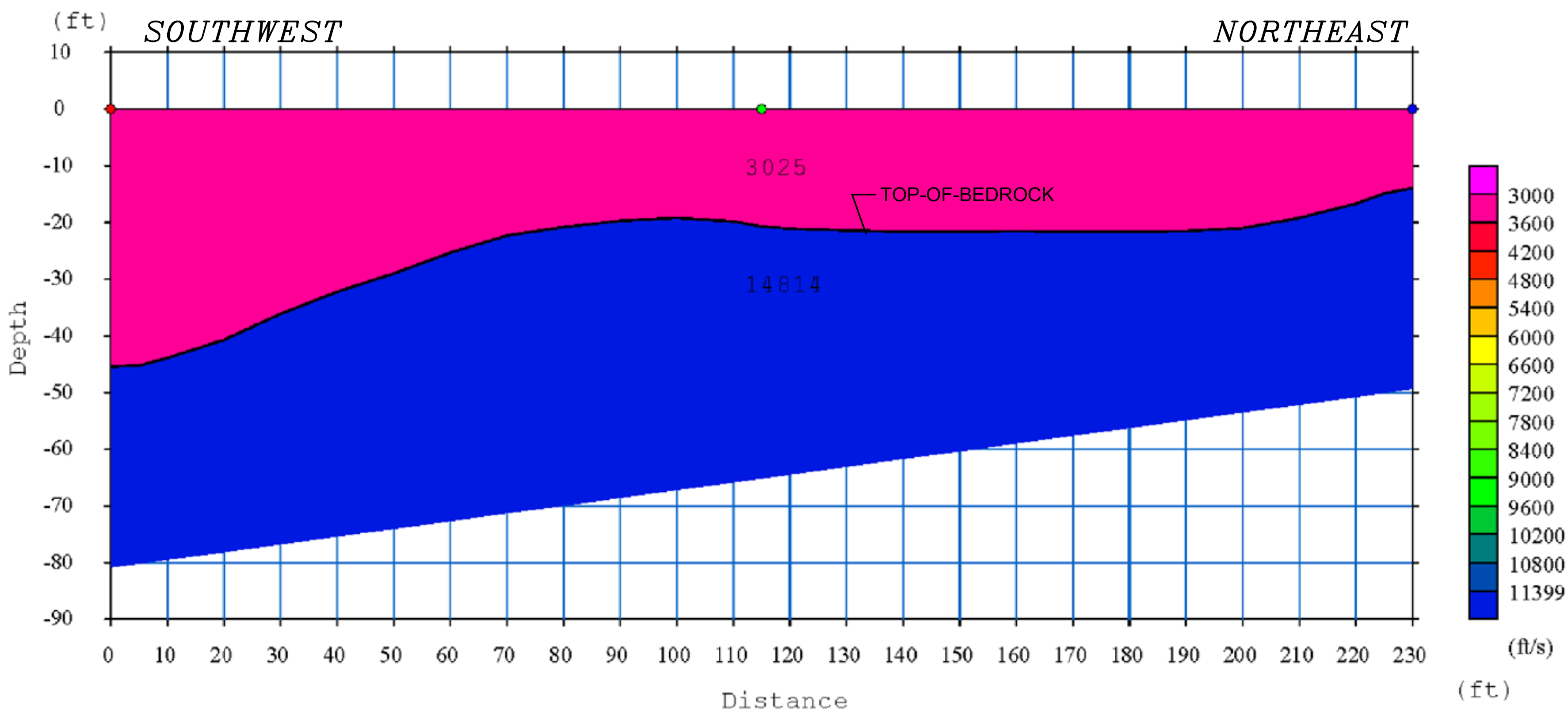


ADVANCED GEOLOGICAL SERVICES

1605 School Street
Suite 4
Moraga, CA 94556
(925) 808-8965

Seismic Refraction Line SL-3
Velocity Layer Model
Proposed Safeway #3132 Remodel
5130 Broadway

LOCATION: Oakland, California	
CLIENT: Kleinfelder	FIGURE 5
PROJECT #: 13-058-1CA	
DATE: Sept 20, 2013	DRAWN BY: R. SMITH



ADVANCED GEOLOGICAL SERVICES

1605 School Street
Suite 4
Moraga, CA 94556
(925) 808-8965

Seismic Refraction Line SL-4
Velocity Layer Model
Proposed Safeway #3132 Remodel
5130 Broadway

LOCATION: Oakland, California

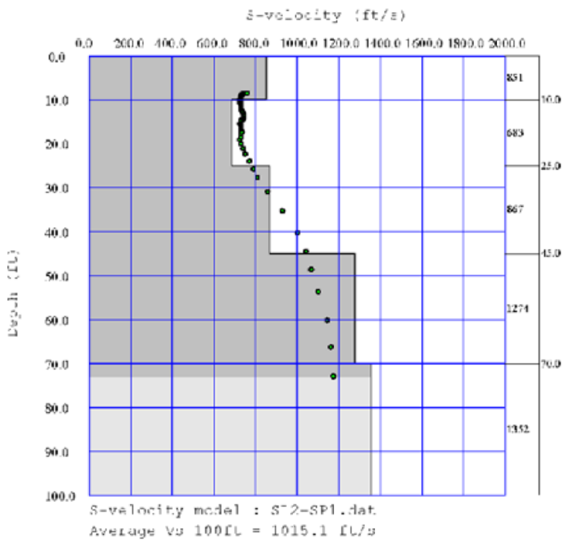
CLIENT: Kleinfelder

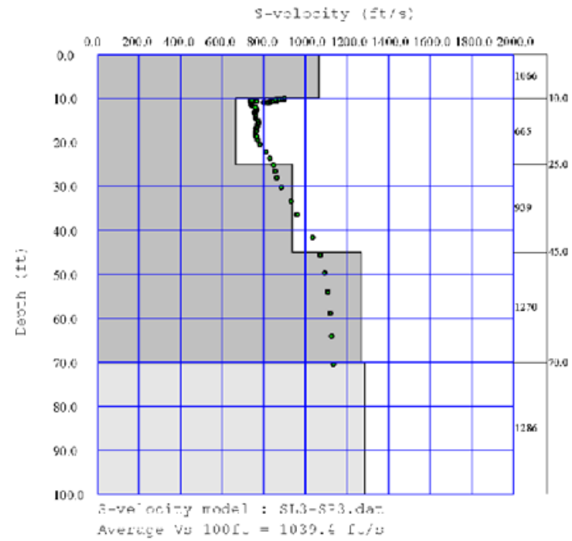
PROJECT #: 13-058-1CA

DATE: Sept 20, 2013

DRAWN BY: R. SMITH

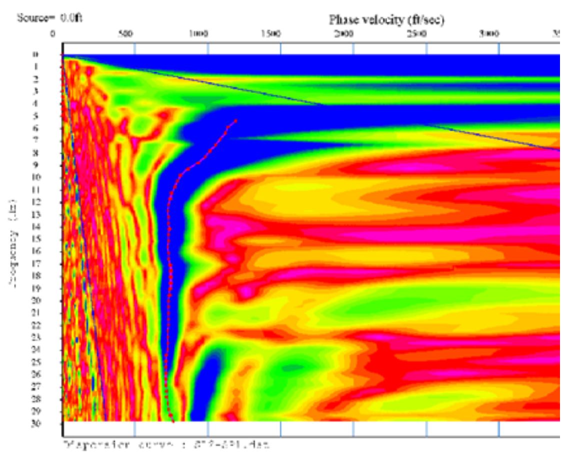
FIGURE
6

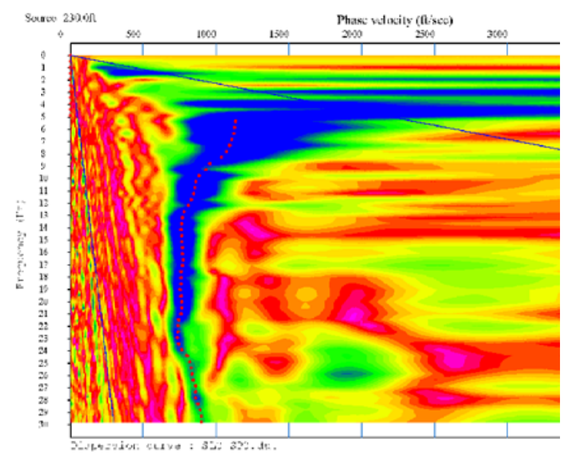




SL- 2

SL- 3





ADVANCED GEOLOGICAL SERVICES

Seismic Surface-Wave Survey Results,
SL-2 and SL-3
Proposed Safeway #3132 Remodel
5130 Broadway

1605 School Street
Suite 4
Moraga, CA 94556
(925) 808-8965

LOCATION: Oakland, California	
CLIENT: Kleinfelder	FIGURE
PROJECT #: 13-058-1CA	7
DATE: Sept 20, 2013 DRAWN BY: R. SMITH	

APPENDIX E

BORING LOGS AND LABORATORY TEST RESULTS FROM KLEINFELDER 2007 INVESTIGATION

UNIFIED SOIL CLASSIFICATION SYSTEM

MAJOR DIVISIONS		LTR	ID	DESCRIPTION
COARSE GRAINED SOILS	GRAVEL AND GRAVELLY	GW		Well-graded gravels or gravel with sand, little or no fines.
		GP		Poorly-graded gravels or gravel with sand, little or no fines.
		GM		Silty gravels, silty gravel with sand mixture.
		GC		Clayey gravels, clayey gravel with sand mixture
	SAND AND SANDY	SW		Well-graded sands or gravelly sands, little or no fines.
		SP		Poorly-graded sands or gravelly sands, little or no fines.
		SM		Silty sand.
		SC		Clayey sand.

MAJOR DIVISIONS		LTR	ID	DESCRIPTION
FINE GRAINED SOILS	SILTS AND CLAYS	ML		Inorganic silts and very fine sands, rock flour or clayey silts with slight plasticity.
		CL		Inorganic lean clays of low to medium plasticity, gravelly clays, sandy clays, silty clays.
		OL		Organic silts and organic silt-clays of low plasticity.
	SILTS AND CLAYS	MH		Inorganic elastic silts, micaceous or diatomaceous or silty soils.
		CH		Inorganic fat clays (high plasticity).
		OH		Organic clays of medium high to high plasticity.
HIGHLY ORGANIC SOILS		Pt		Peat and other highly organic soils.

Standard Penetration Split Spoon Sampler 2.0 inch O.D., 1.4 inch I.D.

Modified California Sampler 2.5 inch O.D., 2.0 inch I.D.

Bulk Sample

California Sampler, 3.0 inch O.D., 2.5 inch I.D.

Shelby Tube 3.0 inch O.D.

▽ 0745, 5/31 Approximate water level first observed in boring. Time recorded in reference to a 24 hour clock.

▼ 0800, 5/31 Approximate water level observed in boring following drilling

PEN	Pocket Penetrometer reading, in tsf
TV:Su	Torvane shear strength, in ksf

LL	LIQUID LIMIT	TX	TRIAXIAL SHEAR
PI	PLASTICITY INDEX	CONSOL	CONSOLIDATION
%-#200	SIEVE ANALYSIS (#200 SCREEN)	R-Value	RESISTANCE VALUE
DS	DIRECT SHEAR	SE	SAND EQUIVALENT
C	COHESION (PSF)	EI	EXPANSION INDEX
PHI	FRICTION ANGLE	FS	FREE SWELL (U.S.B.R.)

Notes: Blow counts represent the number of blows a 140-pound hammer falling 30 inches required to drive a sampler through the last 12 inches of an 18 inch penetration, unless otherwise noted.

The lines separating strata on the logs represent approximate boundaries only. The actual transition may be gradual. No warranty is provided as to the continuity of soil strata between borings. Logs represent the soil section observed at the boring location on the date of drilling only.

KLEINFELDER	BORING LOG LEGEND	*PLATE*
PROJECT NO. **82546**	REPLACEMENT STORE SAFEWAY STORE #3132 5130 BROADWAY OAKLAND, CALIFORNIA	**A-1**

9/5/2007 1:53:47 PM

Date Completed: 4/30/07

Logged By: C. Varela

Total Depth: Approximately 50.5 ft

Drilling method: 2.5" O.D., 2.0 inch I.D., Modified California Sampler

Hammer Wt: 140 lbs., 30" drop

Notes:

Depth, ft	FIELD: Sample	FIELD: Blows/ft	LABORATORY: Dry Density pcf	LABORATORY: Moisture Content %	LABORATORY: Compress. Strength tsf	LABORATORY: Other Tests	Pen, tsf	DESCRIPTION (Surface Elevation: Estimated 152 feet (MSL))
								ASPHALT - approximately 2 inches thick
								AGGREGATE - approximately 12" thick
								SANDY CLAY (CL) - gray, moist, hard, coarse gravel, moderately cemented, highly weathered claystone
5		27	125	8.9		LL=26; PI=12 -#200=57%		
10		17						- olive/yellow brown, moist, some organics (wood chips)
15		9	95	20.5				- very moist, medium stiff, clayey
20		10	122	12.4		-#200=24.6%		- some coarse gravel (possible slough)
25		12				-#200=52.1%		- brown, wet, stiff, with coarse gravel
30								- no recovery

KLEINFELDER

PROJECT NO. 82546

LOG OF BORING NO. B-1

REPLACEMENT STORE
SAFEWAY STORE #3132
5130 BROADWAY
OAKLAND, CALIFORNIA

PLATE **A-2**

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Depth, ft	Sample	Blows/ft	Dry Density pcf	Moisture Content %	Compress. Strength tsf	Other Tests	Pen, tsf	DESCRIPTION
								(Continued from previous plate)
		11						
								SANDY CLAY (CL) - continued
35		14	106	19.8				- gray, medium stiff
40		21						- trace organics
45		50/5"						- with coarse gravel, less weathered
50		39						Boring terminated at approximately 50.5 feet below ground surface. Boring backfilled with grout.
55								
60								

KLEINFELDER

PROJECT NO. 82546

LOG OF BORING NO. B-1

REPLACEMENT STORE
SAFEWAY STORE #3132
5130 BROADWAY
OAKLAND, CALIFORNIA

PLATE **A-2 (cont'd)**

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9/14/2007 9:01:10 AM

Date Completed:	4/30/07	*Drilling method:*	2.5" O.D., 2.0 inch I.D., Modified California Sampler
Logged By:	C. Varela	*Hammer Wt:*	140 lbs., 30" drop
Total Depth:	Approximately 15.5 ft	*Notes:*	

Depth, ft	FIELD: Sample	FIELD: Blows/ft	LABORATORY: Dry Density pcf	LABORATORY: Moisture Content %	LABORATORY: Compress. Strength tsf	LABORATORY: Other Tests	Pen, tsf	DESCRIPTION (Surface Elevation: **Estimated 152 feet (MSL)**)
								ASPHALT - approximately 4 inches thick
								AGGREGATE - approximately 8" thick
								SANDY CLAY (CL) - gray, dry, cobbles and coarse gravel, (highly weathered claystone)
5	■	29	121	5.8		LL=29; PI=12		
								- difficult drilling
								- moist, coarse gravel, trace organics
10	■	11	107	16.5		-#200=58.4%		
								- moist, soft, sandier, possible slough
15	■	6	91	24.1				
								Boring terminated at approximately 15.5 feet below ground surface. No free groundwater encountered. Boring backfilled with grout.
20								
25								
30								

KLEINFELDER

PROJECT NO. 82546

LOG OF BORING NO. B-2

REPLACEMENT STORE
SAFEWAY STORE #3132
5130 BROADWAY
OAKLAND, CALIFORNIA

PLATE **A-3**

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Date Completed:	4/30/07	Drilling method:	2.5" O.D., 2.0 inch I.D., Modified California Sampler
Logged By:	C. Varela	Hammer Wt:	140 lbs., 30" drop
Total Depth:	Approximately 15.5 ft	Notes:	

Depth, ft	FIELD: Sample	FIELD: Blows/ft	LABORATORY: Dry Density pcf	LABORATORY: Moisture Content %	LABORATORY: Compress. Strength tsf	LABORATORY: Other Tests	Pen, tsf	DESCRIPTION (Surface Elevation: Estimated 152 feet (MSL))
								ASPHALT - approximately 5 inches thick
								AGGREGATE - approximately 12" thick
								SANDY CLAY (CL) - gray, moist, coarse gravel (highly weathered claystone)
5	■	30	114	10.1	1.61 @ 3.0%			
10	■	56						- dry
15	■	10				Passing -#200=51%		- moist to wet, coarse gravel and rock fragments
								Boring terminated at approximately 15.5 feet below ground surface. No free groundwater encountered. Boring backfilled with grout.
20								
25								
30								

KLEINFELDER

PROJECT NO. 82546

LOG OF BORING NO. B-3

REPLACEMENT STORE
SAFEWAY STORE #3132
5130 BROADWAY
OAKLAND, CALIFORNIA

PLATE **A-4**

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Date Completed:	5/1/07	Drilling method:	2.5" O.D., 2.0 inch I.D., Modified California Sampler
Logged By:	C. Varela	Hammer Wt:	140 lbs., 30" drop
Total Depth:	Approximately 10.5 ft	Notes:	

Depth,ft	FIELD: Sample	FIELD: Blows/ft	LABORATORY: Dry Density pcf	LABORATORY: Moisture Content %	LABORATORY: Compress. Strength tsf	LABORATORY: Other Tests	Pen, tsf	DESCRIPTION (Surface Elevation: **Estimated 155 feet (MSL)**)
								ASPHALT - approximately 5 inches thick
								AGGREGATE - approximately 12" thick
								CLAYEY SAND (SC) - red-brown, moist, fine gravel (highly weathered sandstone)
5	■	42	118	8.3				
								- rock fragments
10	■	50/3"						- auger refusal
								Boring terminated at approximately 10.5 feet below ground surface. No free groundwater encountered. Boring backfilled with grout.
15								
20								
25								
30								

KLEINFELDER

PROJECT NO. 82546

LOG OF BORING NO. B-4

REPLACEMENT STORE
SAFEWAY STORE #3132
5130 BROADWAY
OAKLAND, CALIFORNIA

PLATE **A-5**

Date Completed:	4/30/07	*Drilling method:*	2.5" O.D., 2.0 inch I.D., Modified California Sampler
Logged By:	C. Varela	*Hammer Wt:*	140 lbs., 30" drop
Total Depth:	Approximately 7.0 ft	*Notes:*	

Depth, ft	FIELD: Sample	FIELD: Blows/ft	LABORATORY: Dry Density pcf	LABORATORY: Moisture Content %	LABORATORY: Compress. Strength tsf	LABORATORY: Other Tests	Pen, tsf	DESCRIPTION (*Surface Elevation:* **Estimated 158 feet (MSL)**)
								ASPHALT - approximately 4 inches thick
								AGGREGATE - approximately 12" thick
								SANDY GRAVEL (GP) - brown, dry, very dense, with clay (weathered sandstone)
5	■	96						
								- moist
								- auger refusal
								Boring terminated at approximately 7 feet below ground surface. No free groundwater encountered. Boring backfilled with grout.
10								
15								
20								
25								
30								

KLEINFELDER

PROJECT NO. 82546

LOG OF BORING NO. B-5

REPLACEMENT STORE
SAFEWAY STORE #3132
5130 BROADWAY
OAKLAND, CALIFORNIA

PLATE

A-6

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Date Completed:	**4/30/07**	*Drilling method:*	**2.5" O.D., 2.0 inch I.D., Modified California Sampler**
Logged By:	**C. Varela**	*Hammer Wt:*	**140 lbs., 30" drop**
Total Depth:	**Approximately 10.5 ft**	*Notes:*	

Depth,ft	FIELD: Sample	FIELD: Blows/ft	LABORATORY: Dry Density pcf	LABORATORY: Moisture Content %	LABORATORY: Compress. Strength tsf	LABORATORY: Other Tests	Pen, tsf	DESCRIPTION — *Surface Elevation:* **Estimated 157 feet (MSL)**
								ASPHALT - approximately 4 inches thick
								AGGREGATE - approximately 12" thick
								GRAVEL (GP) - olive-brown, moist, dense, with clay
5	■	35						- water encountered, most likely perched from adjacent landscape island
10	■	50/3"						- grading less silt, moist, hard
								Boring terminated at approximately 10.5 feet below ground surface. Boring backfilled with grout.
15								
20								
25								
30								

KLEINFELDER

PROJECT NO. 82546

LOG OF BORING NO. B-6

REPLACEMENT STORE
SAFEWAY STORE #3132
5130 BROADWAY
OAKLAND, CALIFORNIA

PLATE

A-7

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Date Completed:	4/30/07	Drilling method:	2.5" O.D., 2.0 inch I.D., Modified California Sampler
Logged By:	C. Varela	Hammer Wt:	140 lbs., 30" drop
Total Depth:	Approximately 10.5 ft	Notes:	

Depth, ft	FIELD: Sample	FIELD: Blows/ft	LABORATORY: Dry Density pcf	LABORATORY: Moisture Content %	LABORATORY: Compress. Strength tsf	LABORATORY: Other Tests	Pen, tsf	DESCRIPTION (Surface Elevation: Estimated 158 feet (MSL))
0								**ASPHALT** - approximately 3 inches thick
								AGGREGATE - approximately 4" thick
								SANDY GRAVEL (GP) - dark brown, dry, coarse gravel, (weathered sandstone)
5	■	50/2"						
10	■	50/3"						- auger refusal
								Boring terminated at approximately 10.5 feet below ground surface. No free groundwater encountered. Boring backfilled with grout.
15								
20								
25								
30								

KLEINFELDER

PROJECT NO. 82546

LOG OF BORING NO. B-7

REPLACEMENT STORE
SAFEWAY STORE #3132
5130 BROADWAY
OAKLAND, CALIFORNIA

PLATE **A-8**

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Date Completed:	4/30/07	Drilling method:	2.5" O.D., 2.0 inch I.D., Modified California Sampler
Logged By:	C. Varela	Hammer Wt:	140 lbs., 30" drop
Total Depth:	Approximately 5.0 ft	Notes:	

Depth,ft	FIELD: Sample	FIELD: Blows/ft	LABORATORY: Dry Density pcf	LABORATORY: Moisture Content %	LABORATORY: Compress. Strength tsf	LABORATORY: Other Tests	Pen, tsf	DESCRIPTION (Surface Elevation: **Estimated 161 feet (MSL)**)
								ASPHALT - approximately 2 inches thick
								AGGREGATE BASEROCK - approximately 8 to 12" thick
								SANDY GRAVEL (GP) - brown, dry, dense, coarse gravel (weathered sandstone)
5	■	50/5"						- auger refusal
								Boring terminated at approximately 5 feet below ground surface. No free groundwater encountered. Boring backfilled with grout.
10								
15								
20								
25								
30								

KLEINFELDER

PROJECT NO. 82546

LOG OF BORING NO. B-8

REPLACEMENT STORE
SAFEWAY STORE #3132
5130 BROADWAY
OAKLAND, CALIFORNIA

PLATE **A-9**

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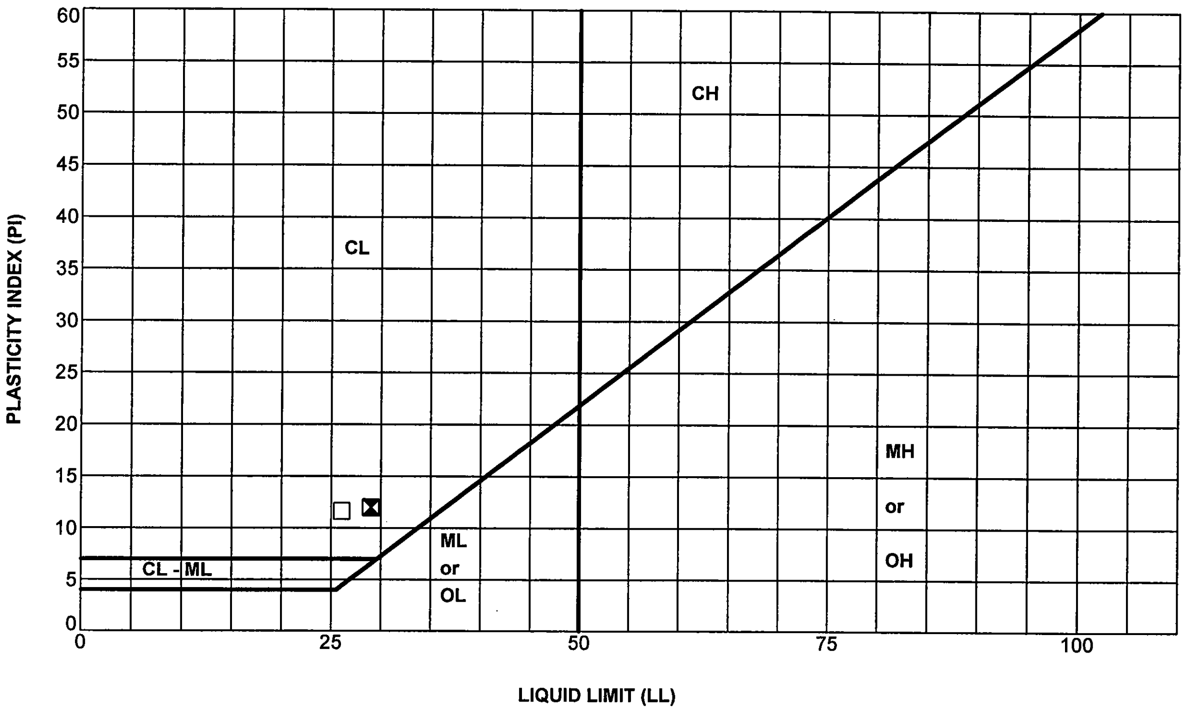

SYMBOL	BORING	DEPTH, ft	LL	PL	PI	SAMPLE DESCRIPTION
□	B-1	5.0	26	14	12	Sandy Lean Clay
⊠	B-2	4.5	29	17	12	Sandy Lean Clay

Unified Soil Classification

Fine Grained Soil Groups

Symbol	LL < 50	Symbol	LL > 50
ML	Inorganic clayey silts to very fine sands 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

*PERFORMED IN GENERAL ACCORDANCE WITH ASTM D 4318 (DRY PREP)

KLEINFELDER

PROJECT NO. 82546

ATTERBERG LIMITS*

REPLACEMENT STORE
SAFEWAY STORE #3132
5130 BROADWAY
OAKLAND, CALIFORNIA

PLATE

B-1

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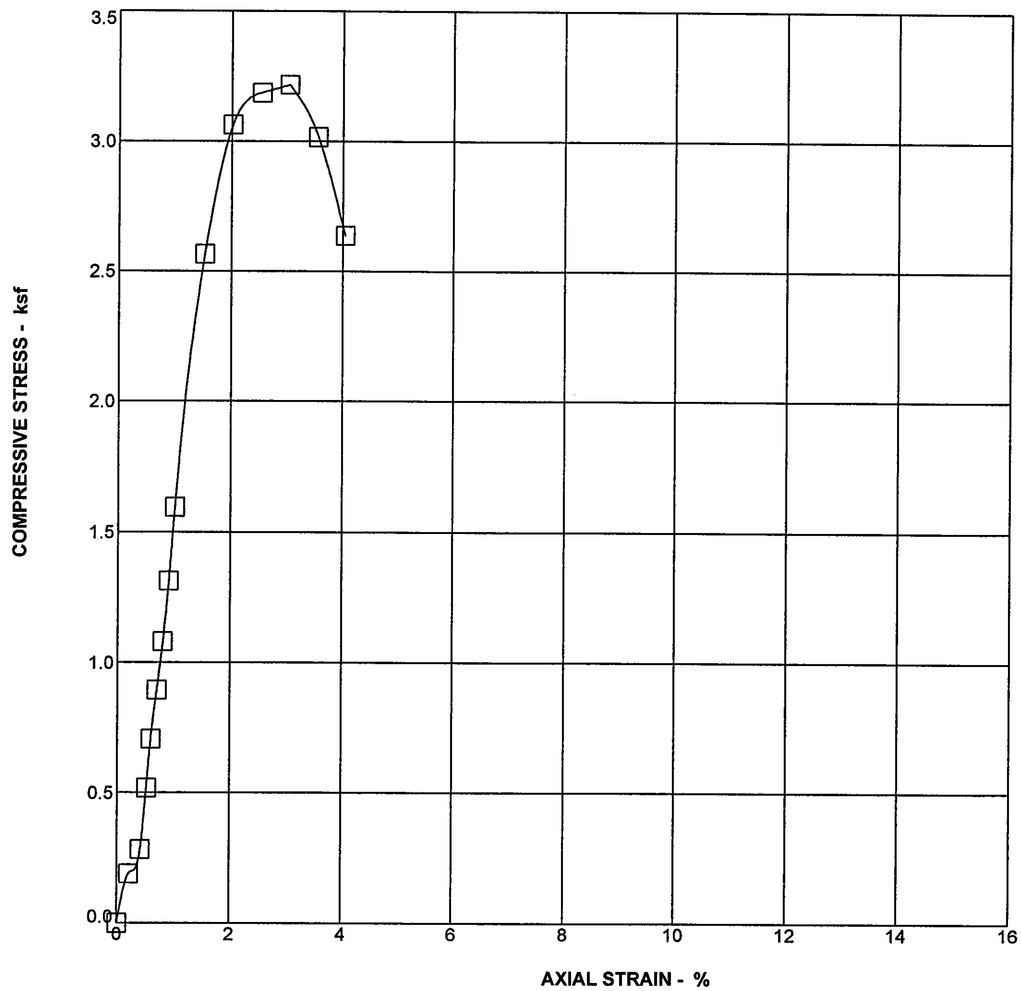

BORING NO. B-3

DEPTH - ft 5

SAMPLE DESCRIPTION Dark Olive Brown Clayey Gravel with Sand (GC)

DRY DENSITY - pcf 114

WATER CONTENT - % 10.1

MAXIMUM COMPRESSIVE STRESS= 3.22 ksf at 3.0 % STRAIN

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*PERFORMED IN GENERAL ACCORDANCE WITH ASTM D 2166

KLEINFELDER

PROJECT NO. *82546*

UNCONFINED COMPRESSION*

REPLACEMENT STORE
SAFEWAY STORE #3132
5130 BROADWAY
OAKLAND, CALIFORNIA

PLATE

B-3

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APPENDIX F

EXHIBIT 1
SUMMARY OF COMPACTION RECOMMENDATIONS

EXHIBIT 1
SUMMARY OF COMPACTION RECOMMENDATIONS

Area	Compaction Recommendation (1,2,3,4)
Subgrade Preparation and General Engineered Fill	Compact clayey material to a minimum of 92 percent compaction at a minimum of 2 percent over the optimum moisture content. Compact granular material to a minimum of 95 percent compaction at above the optimum moisture content.
Utility Trenches (5) and Exterior Flatwork (6)	Compact clayey material to a minimum of 90 percent compaction at a minimum of 2 percent over the optimum moisture content. Where utility trenches or exterior flatwork is exposed to vehicular traffic, compact clayey material in the upper 12 inches of subgrade to a minimum of 92 percent relative compaction at a minimum of 2 percent over the optimum moisture content. Compact granular material to a minimum of 90 percent compaction at above the optimum moisture content. Where utility trenches or exterior flatwork is exposed to vehicular traffic, compact granular material in the upper 12 inches of subgrade to a minimum of 95 percent relative compaction at above the optimum moisture content. Compact baserock (use optional) to a minimum of 95 percent compaction at above the optimum moisture content.
Building Pads (6)	Compact clayey material to a minimum of 92 percent compaction at a minimum of 2 percent over the optimum moisture content. Compact granular material to a minimum of 95 percent compaction at above the optimum moisture content. Compact baserock (use optional) to a minimum of 95 percent compaction at above the optimum moisture content.
Parking and Access Driveways (6)	Compact upper 12 inches of clayey subgrade to a minimum of 92 percent relative compaction at a minimum of 2 percent over the optimum moisture content. Compact upper 12 inches of granular subgrade to a minimum of 95 percent relative compaction at above the optimum moisture content. Compact baserock and subbase materials to a minimum of 95 percent compaction at above the optimum moisture content.

Notes:

1. All compaction requirements refer to relative compaction as a percentage of the laboratory standard described by ASTM D-1557.
2. All lifts to be compacted shall be a maximum of 8 inches loose thickness, unless otherwise recommended.
3. All compacted surfaces should be firm, stable, and unyielding under compaction equipment.
4. Where fills are deeper than 7 feet, the portion below 7 feet should be compacted to a minimum of 95 percent.
5. In landscaping areas, this percent compaction in trenches may be reduced to 85 percent.
6. Depths are below finished subgrade elevation.

APPENDIX G

GBA IMPORTANT INFORMATION ABOUT YOUR GEOTECHNICAL REPORT

Important Information about Your

Geotechnical-Engineering Report

Subsurface problems are a principal cause of construction delays, cost overruns, claims, and disputes.

While you cannot eliminate all such risks, you can manage them. The following information is provided to help.

Geotechnical Services Are Performed for Specific Purposes, Persons, and Projects

Geotechnical engineers structure their services to meet the specific needs of their clients. A geotechnical-engineering study conducted for a civil engineer may not fulfill the needs of a construction contractor or even another civil engineer. Because each geotechnical-engineering study is unique, each geotechnical-engineering report is unique, prepared *solely* for the client. No one except you should rely on your geotechnical engineering report without first conferring with the geotechnical engineer who prepared it. *And no one — not even you* — should apply the report for any purpose or project except the one originally contemplated.

Read the Full Report

Serious problems have occurred because those relying on a geotechnical-engineering report did not read it all. Do not rely on an executive summary. Do not read selected elements only.

A Geotechnical-Engineering Report Is Based on a Unique Set of Project-Specific Factors

Geotechnical engineers consider many unique, project-specific factors when establishing the scope of a study. Typical factors include: the client's goals, objectives, and risk-management preferences; the general nature of the structure involved, its size, and configuration; the location of the structure on the site; and other planned or existing site improvements, such as access roads, parking lots, and underground utilities. Unless the geotechnical engineer who conducted the study specifically indicates otherwise, do not rely on a geotechnical engineering report that was:

- not prepared for you,
- not prepared for your project,
- not prepared for the specific site explored, or
- completed before important project changes were made.

Typical changes that can erode the reliability of an existing geotechnical-engineering report include those that affect:

- the function of the proposed structure, as when it's changed from a parking garage to an office building, or from a light-industrial plant to a refrigerated warehouse,
- elevation, configuration, location, orientation, or weight of the proposed structure,
- composition of the design team, or
- project ownership.

As a general rule, *always* inform your geotechnical engineer of project changes—even minor ones—and request an assessment of their impact. *Geotechnical engineers cannot accept responsibility or liability for problems that occur because their reports do not consider developments of which they were not informed.*

Subsurface Conditions Can Change

A geotechnical-engineering report is based on conditions that existed at the time the study was performed. *Do not rely on a geotechnical-engineering report* whose adequacy may have been affected by: the passage of time; by man-made events, such as construction on or adjacent to the site; or by natural events, such as floods, droughts, earthquakes, or groundwater fluctuations. *Always* contact the geotechnical engineer before applying the report to determine if it is still reliable. A minor amount of additional testing or analysis could prevent major problems.

Most Geotechnical Findings Are Professional Opinions

Site exploration identifies subsurface conditions only at those points where subsurface tests are conducted or samples are taken. Geotechnical engineers review field and laboratory data and then apply their professional judgment to render an opinion about subsurface conditions throughout the site. Actual subsurface conditions may differ—sometimes significantly—from those indicated in your report. Retaining the geotechnical engineer who developed your report to provide construction observation is the most effective method of managing the risks associated with unanticipated conditions.

A Report's Recommendations Are *Not* Final

Do not overrely on the construction recommendations included in your report. *Those recommendations are not final,* because geotechnical engineers develop them principally from judgment and opinion. Geotechnical engineers can finalize their recommendations *only* by observing actual subsurface conditions revealed during construction. *The geotechnical engineer who developed your report cannot assume responsibility or liability for the report's recommendations if that engineer does not perform construction observation.*

A Geotechnical Engineering Report Is Subject to Misinterpretation

Other design team members' misinterpretation of geotechnical-engineering reports has resulted in costly problems. Lower that risk by having your geotechnical engineer confer with appropriate members of the design team after submitting the report. Also retain your geotechnical engineer to review pertinent elements of the design team's plans and specifications. Contractors can also misinterpret a geotechnical-engineering report. Reduce that risk by having your geotechnical engineer participate in prebid and preconstruction conferences, and by providing construction observation.

Do Not Redraw the Engineer's Logs

Geotechnical engineers prepare final boring and testing logs based upon their interpretation of field logs and laboratory data. To prevent errors or omissions, the logs included in a geotechnical engineering report should *never* be redrawn for inclusion in architectural or other design drawings. Only photographic or electronic reproduction is acceptable, *but recognize that separating logs from the report can elevate risk.*

Give Contractors a Complete Report and Guidance

Some owners and design professionals mistakenly believe they can make contractors liable for unanticipated subsurface conditions by limiting what they provide for bid preparation. To help prevent costly problems, give contractors the complete geotechnical-engineering report, *but* preface it with a clearly written letter of transmittal. In that letter, advise contractors that the report was not prepared for purposes of bid development and that the report's accuracy is limited; encourage them to confer with the geotechnical engineer who prepared the report (a modest fee may be required) and/or to conduct additional study to obtain the specific types of information they need or prefer. A prebid conference can also be valuable. *Be sure contractors have sufficient time* to perform additional study. Only then might you be in a position to give contractors the best information available to you, while requiring them to at least share some of the financial responsibilities stemming from unanticipated conditions.

Read Responsibility Provisions Closely

Some clients, design professionals, and contractors do not recognize that geotechnical engineering is far less exact than other engineering disciplines. This lack of understanding has created unrealistic expectations that have led to disappointments, claims, and disputes. To help reduce the risk of such outcomes, geotechnical engineers commonly include a variety of explanatory provisions in their reports. Sometimes labeled "limitations," many of these provisions indicate where geotechnical engineers' responsibilities begin and end, to help others recognize their own responsibilities and risks. *Read these provisions closely.* Ask questions. Your geotechnical engineer should respond fully and frankly.

Geoenvironmental Concerns Are Not Covered

The equipment, techniques, and personnel used to perform a *geoenvironmental* study differ significantly from those used to perform a *geotechnical* study. For that reason, a geotechnical-engineering report does not usually relate any geoenvironmental findings, conclusions, or recommendations; e.g., about the likelihood of encountering underground storage tanks or regulated contaminants. *Unanticipated environmental problems have led to numerous project failures.* If you have not yet obtained your own geoenvironmental information, ask your geotechnical consultant for risk management guidance. *Do not rely on an environmental report prepared for someone else.*

Obtain Professional Assistance To Deal with Mold

Diverse strategies can be applied during building design, construction, operation, and maintenance to prevent significant amounts of mold from growing on indoor surfaces. To be effective, all such strategies should be devised for the *express purpose* of mold prevention, integrated into a comprehensive plan, and executed with diligent oversight by a professional mold-prevention consultant. Because just a small amount of water or moisture can lead to the development of severe mold infestations, many mold-prevention strategies focus on keeping building surfaces dry. While groundwater, water infiltration, and similar issues may have been addressed as part of the geotechnical-engineering study whose findings are conveyed in this report, the geotechnical engineer in charge of this project is not a mold-prevention consultant; ***none of the services performed in connection with the geotechnical engineer's study were designed or conducted for the purpose of mold prevention. Proper implementation of the recommendations conveyed in this report will not of itself be sufficient to prevent mold from growing in or on the structure involved.***

Rely, on Your GBA-Member Geotechncial Engineer for Additional Assistance

Membership in the Geoprofessional Business Association exposes geotechnical engineers to a wide array of risk confrontaton techniques that can be of genuine benefit for everyone involved with a construction project. Confer with your GBA-member geotechnical engineer for more information.

GEOPROFESSIONAL BUSINESS ASSOCIATION

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