APPENDIX 4.6

Geology and Soils

REVISED DESIGN-LEVEL
GEOTECHNICAL INVESTIGATION
UOP PROPERTY
D STREET AND WINDSOR
PETALUMA, CALIFORNIA

FOR
DAVIDON HOMES
September 22, 2004

BERLOGAR GEOTECHNICAL CONSULTANTS

September 22, 2004
Job No. 2616.100

BGC
BERLOGAR
GEOTECHNICAL
CONSULTANTS

Mr. Steve Abbs
Davidon Homes
1600 South Main Street, Suite 150
Walnut Creek, California 94596

Subject: Revised Design-Level Geotechnical Investigation
UOP Property
D Street and Windsor Drive
Petaluma, California

Dear Mr. Abbs:

INTRODUCTION

This report contains the results of our revised design-level geotechnical investigation for the UOP property located at the intersection of D Street and Windsor Drive in Petaluma, California. The site is shown on the Vicinity Map, Plate 1. We previously performed a feasibility investigation of the site and presented the results in a report dated March 7, 2002.

PURPOSE AND SCOPE OF SERVICES

The purpose of this investigation has been to characterize the engineering properties of soil and bedrock at the site and provide design-level geotechnical recommendations for site development. Our scope of services for this project were as follows:

1. Review of previous information covering the site and vicinity,
2. Review of stereo-paired aerial photographs covering the site and vicinity,
3. Site geologic reconnaissance and mapping,
4. Drilling and logging of 14 borings,
5. Excavation and logging of 36 backhoe test pits,
6. Laboratory testing of selected representative samples collected during the field investigation,
7. Engineering and geologic analysis, and
8. Preparation of this report.

PROPOSED DEVELOPMENT

The site includes two parcels totaling about 58 acres that are separated by Windsor Drive. The site is bound by residential developments to the north and west, by D Street to the east, and open land to the south. We have received plans from BKF via electronic file on March 15, 2004, that show the site being developed into 93 single-family residential lots separated by existing creek channels. The existing creek channels are to remain undeveloped open space. New streets providing access to the site are to branch off of Windsor Drive and D Street. Two creek crossings are currently planned: a five-foot-wide pedestrian foot bridge located roughly in the center of the site and a roadway crossing over a 60-inch-diameter culvert located in the southeast part of the site, near D Street. A low bridge intended as a roadway crossing is also planned to cross a shallow swale designated as a wetland located roughly in the center of the site. We understand that Davidon Homes is currently exploring other options for the roadway crossing such as a box culvert or an arch culvert are under consideration. Each of the culvert options would include a natural bottom. An existing stock pond and berm located in a swale in the southern portion of the site are to remain. In order to achieve design grades, cuts of up to about 36 feet and fills of up to about 32 feet are planned. The design grading will result in cut slopes up to about 80 feet tall and fill slopes up to about 30 feet high. Retaining walls up to 10 feet high are planned to achieve design grades. Pedestrian footpaths connected by the footbridge are planned along both sides of Kelly Creek.

FIELD INVESTIGATION

Our field investigation was conducted between March 28 and April 3, 2003. The investigation included a site reconnaissance, geologic mapping of creek bank exposures, drilling and logging of two rotary wash borings (B-1 and B-2) to depths ranging from 24 to 50½ feet, drilling and logging of 12 auger borings (B-3 through B-14) to depths ranging from 8 to 22 feet, and excavation and logging of 36 backhoe test pits (TP2-1 through TP2-36) to depths of up to 14 feet. Materials encountered in the borings and test pits were visually classified and logs were recorded. Bulk and relatively undisturbed samples of bedrock and soils were collected from the borings and test pits for laboratory testing.

Where ground water was encountered, the borings were backfilled with neat cement grout in accordance with Sonoma County requirements. Borings that did not encounter ground water were backfilled with soil cuttings. Test pits were loosely backfilled with excavated materials at the completion of logging. The locations of borings and test pits are shown on the attached Geologic Map. Boring logs are presented in Appendix A (Plates A-1 through A-20), a Key to Boring Symbols is included as Plate A-21, and Rock Description is presented as Plate A-22. Test Pit Logs are also included in Appendix A (Plates A-23 through A-31).

FINDINGS

SURFACE CONDITIONS

Site topography ranges from about 100 feet above mean sea level in the eastern portion of the site to about 380 feet near the southwest corner of the site. The site contains a relatively flat alluvial plain in the central portion of the site that is bordered by moderately steep bedrock slopes to the north and south. Kelly Creek crosses the site in an east-west direction and intersects an unnamed tributary that crosses the eastern portion of the site in a north-south direction near D Street. Two drainage gullies cross the central plain and drain to Kelly Creek. Drainage at the site flows to the northeast and enters an existing box culvert beneath D Street.

Existing site improvements include scattered wood-framed houses and barns located near D Street. An open, concrete-lined water well about 3 feet in diameter and about 15 feet deep, is located beneath the trees along the edge of the westernmost drainage gully on the south side of Kelly Creek. Water in the well was roughly at the ground surface. A stock pond and berm are located in a swale south of Kelly Creek. An existing storm drain outlet is located on the southwest side of D Street, about 5 feet east of the property line. Water from the storm drain outlet is planned to discharge to the rear of proposed Lot 93.

REGIONAL GEOLOGY

The site is situated along the southwest margin of the Petaluma River valley. This valley is part of a series of small basins and ranges characteristic to the Coast Ranges geomorphic province of California. In this portion of the province, the oldest bedrock consists of sedimentary and meta-volcanic rocks of the Franciscan Complex which were deposited during the Jurassic and Cretaceous Periods of geologic time (about 65 to 208 million years before present). Small lenses of sheared and/ or altered bodies of rock are inherent to the Franciscan Complex. Tertiary aged (10.6 to 65 million years before present) volcanic rocks are present in scattered patches throughout the region (Blake et al., 1974).

Bedrock in this region has been folded and faulted during the past several million years due to relative strike-slip and convergent motion between tectonic plates. Much of the deformation (shearing, faulting and folding) of the Franciscan Complex occurred during past convergent plate motions. Most convergent plate motion in the region ended millions of years ago. This deformation is believed by many researchers to be intrinsic to the Franciscan Complex and is separate from the active strike-slip fault motion in the region.

SUBSURFACE CONDITIONS

During the course of this investigation, we encountered artificial fill, landslide deposits, colluvium, alluvium, and bedrock units of the Franciscan Complex. A description of each material excluding landslide deposits which are discussed separately are listed in order from youngest to oldest as follows:

ARTIFICIAL FILL

Isolated areas of artificial fill at the site were encountered in three main areas: beneath and around existing buildings, the stock pond earthen berm, and along the downslope (south) side of Windsor Drive beneath D Street. Fill beneath and around existing structures encountered in Test Pits TP2-35 and-36 was found to consist of dense sandy silt and gravel that extended to a depth of about 1 foot. The stock pond berm fill encountered in Boring B-5 was found to consist of stiff to very stiff silty clay that extended to a depth of about 12 feet. Fill along the downslope edge of Windsor Drive and D Street is assumed to have been engineered along with roadway construction and was not investigated. Areas of artificial fill are delineated by the symbol "Qaf" on the Geologic Map.

COLLUVIUM

Areas of soil accumulation referred to as colluvium are present in the lower portions of the site. Colluvium is material that is generated by the in-place weathering of underlying bedrock on a slope and then migrates downslope under the influence of gravity. Colluvium mantles all slopes to some degree and forms particularly thick deposits at the toes of slopes and in swales. At the site, colluvium was found to be brown to light red-brown, stiff to very stiff silty clay with minor amounts of gravel. Laboratory testing suggests that the colluvium on site is moderately expansive. Areas of colluvium thicker than a few feet are delineated on the Geologic Map by the symbol "Qc."

ALLUVIUM

Alluvium is material that has been transported and deposited by way of flowing water. At the site, alluvium was found to consist of orange-brown to yellow-brown sandy clays and clayey sands with various amounts of gravel that are stiff to very stiff and medium dense to dense. Alluvium is generally found in relatively flat lying areas bordering drainage courses and at the downslope end of swales as shown on the Geologic Map by the symbol "Qal." Based on the information provided by the borings and creek exposures, the alluvium unit reaches a maximum thickness of about 25 feet at a point about half way between the Kelly Creek channel and the base of the hills to the south. Laboratory testing suggests that the alluvium on site is moderately expansive.

SHEAR ZONE MATERIAL

Three shear zones were encountered during our current and previous investigations: one north of Windsor Drive, and two located in the southwest portion of the site. The shear zones at the site are not related to the active regional strike-slip system of faulting. The shear zones at the site are interpreted as deformation concentrated within the relatively weak shale. The deformation likely occurred as flexural slip during regional folding resulting from the past convergent tectonic regime. Our previous investigation described material within the shear zone as containing serpentine minerals. Serpentine minerals are often found in ultramafic rocks. Based on the test pits excavated within shear zones during this investigation (TP2-2, TP2-21, and TP2-31), ultramafic rocks were not encountered. We also re-excavated test pits from our previous investigation (TP-17 and TP-20) and reclassified the shear zone materials. It was found that the shear zone materials are composed of sheared clayey shale, and no serpentine minerals or ultramafic rocks were encountered. The gray-green alteration colors previously described are interpreted to be the result of a localized chemical reduction of the clayey material. Based on these findings, we conclude that the potential for significant volumes of serpentine-bearing ultramafic rocks being present at the site is low.

FRANCISCAN COMPLEX BEDROCK

Bedrock at the site consists of sandstone and shale of the Franciscan Complex. The sandstone was found to be moderately strong to strong and highly fractured with scattered areas of very strong-cemented beds. The shale is weak to moderately strong, thinly laminated, and crushed to sheared. Where sheared, the shale was weathered to clay and displayed a faint residual bedrock structure. Bedding was found in general to strike northwest and dip southwest at inclinations between about 33 and 73 degrees.

LANDSLIDES

A total of 18 landslides were mapped within the site, and are shown and designated as Landslides A through R on the Geologic Map. Landslides A, B, C, D, and G are located on the flanks of the hillsides in the southern portion of the site. Landslides E, F, and H are located on the flank of the large bedrock knob in the northwest portion of the site. The remaining landslides (Landslides I through R) are located along the banks of Kelly Creek and are the result of typical creek bank oversteepening. Landslides encountered at the site are relatively shallow with depths up to about 15 feet and are believed to involve soils and the upper 2 to 3 feet of highly weathered bedrock.

FAULTING

The site is not located within a State of California designated earthquake fault zone for active faults (Davis, 2000; Hart and Bryant, 1982). The State of California considers a fault active if it has demonstrated Holocene activity (within the past 11,000 years). We did not encounter evidence of an active fault crossing or trending toward the site.

The table below lists the seven known active faults believed to present the highest potential levels of ground shaking at the site, their distances from the site, and their potential maximum moment-magnitude earthquakes. The faults in the table are arranged in order of their decreasing potential level of ground shaking at the site.

SIGNIFICANT POTENTIAL FAULT EARTHQUAKE SOURCES IN SITE VICINITY			
Fault Name	Approx. Distance to Fault Trace (mi)[a]	Compass Direction to Fault	Maximum E.Q. mag. (Mw)[b]
Rodgers Creek	7	NE	7.0
San Andreas, 1906 Rupture	14	SW	7.9
Hayward, Total Length	18	SE	7.1
San Gregorio	23	S	7.3
Point Reyes	22	SW	6.8
West Napa	19	E	6.5
Maacama, south	25	N	6.9

1. Potential fault earthquake sources given by Peterson et al. (1998).
2. Maximum earthquake moment magnitude calculated by Peterson et al. (1998).

GROUNDWATER

Groundwater was encountered in Test Pit TP2-7 at a depth of about 2 feet and is likely the result of the storm drain outfall next to D Street. Groundwater was encountered in Borings B-1, B-5, B-7, B-8, B-12, and B-13 at depths of about 21, 17, 7, 14½, 17½, and 9½ feet, respectively. Marshy ground and groundwater seepage have been observed in various places across the site mainly following periods of higher rainfall. Areas of perched groundwater are expected in the lower portions of the site. Groundwater levels are expected to undergo significant fluctuations based on seasonal rainfall and time of year.

CONCLUSIONS AND RECOMMENDATIONS

GENERAL

From a geotechnical standpoint, the proposed residential development can generally be constructed as planned, provided the conclusions and recommendations contained within this report are incorporated into the project design and construction. The primary geotechnical issues for site development are landslide remediation, treatment of existing fill, fill slope construction, stability of proposed cut slopes, and the potential for expansion and settlement of on-site earth materials.

LANDSLIDE REMEDIATION

The recommended remedial treatment of landslide hazards is dependent on many factors such as the size of the landslide, the landslide's spatial relationship to proposed improvements, and the individual characteristics of each landslide. In general, the preferred remedial measure from a geotechnical standpoint is complete removal of landslide debris located within the development area. A number of factors can make complete removal of landslide debris impractical, such as property line limitations or the presence of trees. Provided the risks associated with movement of part of a given landslide located outside the development are acceptable, the potential adverse impacts to the planned development can be minimized by implementing remedial measures such as construction of engineered fill, below-grade MSE walls, and catchment areas.

Landslides A, B, D, and E will require remedial treatment for the currently planned development. Landslide F will be removed with the design cut. Landslides C, G, and H are located outside of the planned development area and require no remediation. Similarly, landslides located along Kelly Creek (I through R) do not impact the development and require no remediation. We recommend that landslides be treated as summarized in the following table:

RECOMMENDED LANDSLIDE REMEDIATION SUMMARY

Landslide Designation	Est. Ave. Thickness (feet)	Relationship of Landslide to Proposed Development	Recommended Remedial Measures
A	12	Within and upslope of limit of grading	• Remove portion within development and replace with engineered fill with proper subdrainage. • Remove portion upslope of development extending to property line and replace with engineered fill with proper subdrainage. • Construct a 40 feet wide (minimum) keyway with proper subdrainage. • Construct a geogrid reinforced MSE retaining wall.
B	9	Partially within limit of grading	• Remove portion within development and replace with engineered, drained fill. • Remove portion in area of fill slope on west sides of Lots 74 and 75 and replace with engineered, drained fill. • Construct 40-foot-wide (min.) keyway and below-grade MSE wall at toe of fill slope along creek.
C	6	Outside limit of grading	• None required
D	4	Within limits of grading	• Remove and replace all landslide debris with engineered fill and proper subdrainage.
E	6	Within limits of grading	• Remove portion within development and replace with engineered fill provided with proper subdrainage. • Construct a 20 feet wide (minimum) keyway.
F	7	Within limits of grading	• None required (removed by design grading).
G	10	Outside limit of grading	• None required.
H	10	Outside limit of grading	• None required.
I through R	3 to 5	Along creek bank	• None required.

The remediation for Landslide A will include removing all landslide debris within the development area upslope to the property line to the south. The excavation will need to extend to bedrock at a depth of about 18 feet at the downslope extent of the repair and include a 40-foot-wide (minimum) keyway. The excavation should be backfilled with engineered fill to design grade within the development and to match existing grade upslope of the development. Grading limitations due to the proximity of the unnamed Kelly Creek tributary will result in the need for a below-grade geogrid-reinforced MSE retaining wall in order to achieve design grade and repair the landslide.

The design of the below-grade MSE retaining wall will be determined when the final grading plans are approved. The below-grade geogrid-reinforced MSE retaining wall should extend beyond the limits of the landslide by about 25 feet to the north and 50 feet south as shown on the Geologic Map. It is expected that the bottom of the excavation will be deeper than a free-draining gravity discharge for a subdrain at that elevation would allow. Therefore, it will be acceptable to place fill in the excavation beneath the subdrain system. The subdrain should then be placed at the lowest elevation that will allow gravity discharge to an acceptable outlet point. We anticipate that a subdrain placed at about Elevation 116 feet will allow for a free draining gravity discharge. The subdrain should discharge at the box culvert planned east of Lot 73. Our intended remediation for Landslide A is shown on Cross Section A-A', on Plate 4.

Remediation of Landslide B should remove roughly the lower half of the landslide including all landslide debris from within the development area. Remediation of Landslide B will remove all landslide debris from within the development and, because of grading constraints due to existing trees, will leave the uppermost portion of landslide debris upslope of proposed lots. The toe of the proposed fill slope on the westside of Lots 74 and 75 is located within the landslide limits. Landslide debris should be removed down to expose competent bedrock in this area and replaced with engineered fill. Section B1-B1' and B3-B3' shows this recommended removal and replacement. Along the northern limit of the proposed fill slope where it parallels Kelly Creek, the landslide removal should include a 40 feet wide (minimum) keyway. Grading limitations due to the proximity of Kelly Creek will result in the need for a below-grade geogrid-reinforced MSE retaining wall along the north side of Lot 74 in order to maintain a stable fill slope toe without extending needed remedial grading into the creek channel. Section B2-B2' shows the recommended subsurface remedial work. The upper portion of Landslide B would be left in place, as shown in Section B1-B1'.

DEBRIS FLOW/SEDIMENTATION POTENTIAL

The potential for debris flows to impact the site is low across most of the development. Areas where we evaluated debris flow potential are at the mouths of the two large swales on the south side of the site.

Aerial photographs and field observations suggest that the swale upslope of Lots 78 and 79 may have experienced small debris flows in the past. The gradient down this swale is sufficient to allow movement of debris flows, which potentially would allow deposition of debris within the development area. The current grading plan shows a concrete headwall located in this swale and a drainage inlet box just upslope of the headwall. The headwall will create a catchment area for potential debris flows. The catchment volume appears roughly adequate for intercepting potential debris flows, depending on the final height of the wall and pad grades, the design of which we expect will be refined as project planning proceeds. The drainage inlet should be designed with means to reduce blockage by solids and allow storm water runoff into the drain system at that entry point.

The swale located upslope of Lots 82 and 83 has a gradient of 8H: 1V for a substantial distance, which is too low a gradient to allow significant movement of debris flows. However, the topography suggests that sedimentation has been occurring in the swale; potential sedimentation could clog the proposed drainage inlet box at that location. This inlet should be provided with measures to reduce the potential for excessive sediment entering the storm drain system at that entry point.

Sedimentation around these two drainage inlets is expected. Regular inspection of the catchment basins should be performed in order to determine if sedimentation has occurred. We recommend inspection of these two areas be performed prior to the rainy season and after large periods of rainfall. If it is found that debris or sediment has reduced the water-drainage or debris-holding capacity, then the accumulated material should be removed.

GRADED SLOPES

CUT SLOPES

All cut slopes should be inspected at the time of construction by an engineering geologist focusing on evidence of potential instability. Cut slopes should be constructed at gradients no steeper than 2H:1V. Where cut slopes over 30 feet in height are planned, intermediate surface benches should be spaced no more than 25 feet vertically on the slope. The benches should be a minimum of 8 feet wide and include a concrete lined V-ditch to intercept surface water runoff.

Based on bedding attitudes measured in test pits, areas of adverse bedrock structure were not encountered at the locations of proposed cut slopes. However, due to folding and shearing of the bedrock, localized areas of adverse bedrock structure or other zones of geologic weakness could be exposed during grading of cut slopes. If areas of adverse bedrock structure are encountered, we anticipate that the remedial measures for these slopes will involve overexcavation of the affected portion of slope and construction of a slope buttress with appropriate subdrainage. We should provide specific remedial design recommendations based on the conditions exposed in areas of concern identified during grading.

FILL SLOPES

The stability of proposed fill slopes is dependent on proper keyways, benching, subdrainage, fill compaction, and slope gradient. Fill slopes should be constructed at gradients no steeper than 2H:1V. Where fill slopes over 30 feet in height are planned, intermediate surface drainage benches should be spaced no more than 25 feet vertically on the slope. The benches should be a minimum of 8 feet wide and include a concrete-lined V ditch to intercept surface water runoff. Fill slopes should be overbuilt and cut back to expose firm compacted materials. Fill slopes should be constructed with a 6 feet deep (minimum) keyway with a width equal to ½ the slope height or 20 feet, whichever is greater, and provided with proper subdrainage. All keyway excavations should be mapped by an engineering geologist prior to backfilling. Typical Fill Slope Details are presented as Plate 5.

The northern limit of the proposed fill slope below (north of) Lot 74 shows grading below the top of the creek bank. This will require a below-grade retaining wall to achieve design grade. Additionally this portion of the fill slope encroaches into Landslide B. The portion of the slope underlain by landslide debris should be overexcavated to expose competent bedrock and replaced with engineered fill as discussed in the section titled *Landslide Remediation*.

Based on the current grading plan, the fill slope located on the west side of the driveway for Lots 92 and 93 will require a 20-feet-wide (minimum) keyway. The required keyway excavation would encroach into the adjacent creek bed. If grading near the creek bed is not feasible, then it may be necessary to construct a retaining wall to support the driveway rather than a fill slope as currently planned. Other options to support the driveway could include using rip-rap grouted in-place or gabian baskets. We anticipate such a structure could be up to about 4 feet high and about 100 feet long.

All cut and fill slopes should be planted with fast growing, deep-rooted vegetation before the first winter to reduce erosion. Consideration should be given to the irrigation of some slopes; specific details regarding irrigation systems, locations, and discharge should be reviewed by BGC prior to their approval.

TREATMENT OF EXISTING FILL

From a geotechnical standpoint, the on-site existing fill is considered suitable for re-use as engineered fill provided it is free of rock fragments greater than 12 inches in size and deleterious material. Inasmuch as the proposed development does not encroach onto these fill areas, based on our field observations, existing fill along D Street and Windsor Drive does not require additional treatment. All other fill located at the site (with the exception of the stock pond berm) should be completely removed and reworked as engineered fill.

Test pits excavated during our current as well as previous investigations were loosely backfilled with excavated materials. Where not removed by design cut, loose backfill at the test pit locations should be subexcavated and replaced with engineered fill.

We understand that it is preferred to leave the existing stock pond berm in place in order to preserve the pond. Fill encountered in Boring B-5 drilled at the top of the stock pond berm was found suitable for the intended purpose of retaining water for the pond. However, due to the proximity of proposed lots we judge that the gradient on the north berm slope is oversteepened and that a proper keyway does not support the fill slope. To protect downslope lots, we recommend that the stock pond berm be upgraded with additional fill placed on the northern slope of the berm. The additional fill should be constructed with a 20-feet-wide (minimum) keyway at the toe of the slope and provided with a subdrain along the upslope side. The additional fill should be benched into the existing fill slope, and the upper few feet of the existing earth berm should be reworked and compacted. Drain rock finger drains should be placed to the back slope of the excavation in 15 feet increments on center. The finished fill slope gradient on the north side of the stock pond berm should be a maximum of 2H:1V, and the top of the berm should be a minimum of 20 feet wide.

Low-permeability soil should be used for constructing the berm upgrade fill, as approved in the field by the soil engineer. Details for upgrading the existing stock pond berm are presented on Plate 6.

SUBDRAINAGE

Ground water seepage is expected to occur in swales, at the bases of slopes, and in isolated pockets in the lower portions of the site. Subdrainage should be provided to intercept ground water in the following locations:

1. On the uphill side of all keyways and proposed fill,
2. Along swales and gullies to receive fill,
3. At all springs and seepage areas,
4. At the toes of major cut slopes,
5. At geologic contacts known to transmit water, and
6. In other areas of the site where seepage is observed during and after grading or as determined in the field by the soil engineer.

Subdrains should consists of perforated PVC pipe conforming to ASTM D 2751, Type SDR 35, for fill less than 30 feet deep and Type SDR 23.5 for fills over 30 feet deep. Subdrains should be at least 6 inches in diameter. All subdrains should be surrounded by and underlain by at least 6 inches of Class 2 Permeable Material as defined in Section 68-1.025 of the Caltrans's Standard Specifications (July 1999). Subdrain trenches should be at least 18 inches wide and at least 4 feet deep. Final trench configurations should be approved by the soil engineer. Subdrain trenches should be capped with engineered fill or topsoil, depending on the location of the subdrain. Subdrain systems should be discharged into a storm drain structure (manhole, inlet) where possible. Subdrain details are provided on Plate 7.

Some areas of seepage may develop after house construction is completed. Additional subdrains will likely be needed in these areas should seepage develop.

EXCAVATION CHARACTERISTICS

Conditions encountered during our field investigations at the site as well as our experience in the area suggest that, in general, excavation to planned depths should be achievable using conventional grading equipment. Based on the high degree of fracturing and the fracture spacing encountered in the test pits, and the rock quality designation (RQD) logged in Boring B-2, we believe that large grading equipment such as a Caterpillar dozer D-10 with rippers should be adequate. Areas of very hard bedrock should be anticipated in deep cut areas at the site that are likely to generate oversize material. Modified excavation techniques such as using a single shank on a D-10 should generally be capable of ripping very hard-cemented areas of bedrock. Areas of hard rock were encountered in Boring B-2 and Test Pits TP2-2 through TP2-4, TP2-8, and TP2-10.

SELECTIVE GRADING

Special care should be taken to reduce the size of bedrock derived fill material so that the material can be properly compacted. Oversized material (greater than 12 inches) is expected to be generated from bedrock cuts at the site. Oversize material can be broken down mechanically or placed in deeper areas of fill and not within 10 feet of pad grade or street subgrade. Oversize material to be used in deeper areas of fill should be spread out so that large rocks are not concentrated in pockets and are surrounded by engineered fill. Placement of oversize material should be subject to approval by the soil engineer.

SITE PREPARATION AND GRADING

All grading operations should be performed in accordance with the following recommendations:

1. Existing earth materials on-site are considered suitable for re-use as engineered fill provided it does not contain rock fragments greater than 12 inches and is free of deleterious material as determined in the field by the soil engineer.

2. If import fill is used, it should have a Plasticity Index (PI) less than 12 and should be subject to evaluation and approval by the soil engineer prior to use.

3. All fill materials to be used at the site should be subject to evaluation and approval by the soil engineer prior to use.

4. Areas to be graded should be cleared and stripped of all vegetation. Strippings can be stockpiled and re-used as topsoil in landscape areas. Strippings can also be blended with clean on-site soils at a ratio of 10 loads of clean soil to 1 load of strippings, to create a soil mixture suitable for use as engineered fill.

5. Existing foundations, wells, septic systems, leach fields (and any similar subsurface structures) should be completely removed prior to grading. Any soft soils encountered during excavation should be removed as determined in the field by the soil engineer.

6. The upper three feet of soil in areas mapped as colluvium should be reworked as engineered fill. This depth of reworking can be reduced as discussed under *Colluvium/Alluvium Overexcavation* below.

7. Low-expansion-potential bedrock cut derived material should be used in keyways for landslide remediation and buttress fill slopes.

8. Where zones of soft or saturated soils are encountered during excavation and compaction, deeper excavation may be required to expose competent materials. This should be determined in the field by the soil engineer.

9. Areas to receive fill should be scarified to a minimum depth of 12 inches, brought to at least 3 percent over optimum moisture content, and compacted to not less than 90 percent relative compaction.

   Relative compaction refers to the in-place density of a soil expressed as a percentage of the maximum dry density determined by Test Method ASTM D1557-00. Optimum moisture is the water content (percentage by weight) corresponding to the maximum dry density.

10. If significant subgrade pumping and/ or yielding occur during scarification or recompaction, it may be necessary to stabilize the exposed subgrade. The actual stabilization method, if warranted, will depend on exposed conditions and should be judged suitable by the soil engineer.

11. Fill should be placed in thin lifts (normally 6 to 8 inches thick, depending on compaction equipment used), moisture conditioned to at least 3 percent over optimum moisture, and compacted to at least 90 percent relative compaction. Modification to acceptable lift thickness should be determined in the field by the soil engineer and based on the demonstrated compaction performance during fill placement, which will depend on the equipment and methods used.

12. Fill placed on ground sloping greater than 7H:1V should be benched into firm materials as determined in the field by the soil engineer.

13. Fill slopes should be over built and cut back to expose a firm compacted surface.

14. Observation and soil density testing should be performed during grading to assist the contractor in achieving the required degree of compaction and the proper moisture content. Where compaction is less than required, additional compactive effort should be made with an adjustment in the moisture content where necessary until the specified compaction is obtained.

15. The soil engineer should be informed at least 48 hours prior to any grading operation. The procedures and methods can then be discussed between the developer, contractor, and soil engineer. This can facilitate the performance of grading operations and minimize potential construction delays.

CUT/ FILL TRANSITION LOT TREATMENT

Because the proposed fill and bedrock at the site will have different expansion and settlement potential, structures and slabs placed across the transition line between cut and fill could experience significant differential expansion and/ or settlement. This condition can be mitigated by overexcavating the cut portion of the cut/ fill transition lots to a depth of about 3 feet below rough pad grade. The exposed excavation bottom should then be scarified to a minimum depth of 12 inches, properly moisture conditioned to not less than 3 percent over optimum moisture content and compacted to at least 90 percent relative compaction. The overexcavation should be restored with engineered fill. Typical Cut/Fill Transition Lot Overexcavation Details are provided on Plate 9.

The horizontal and vertical extent of overexcavation should be determined in the field by the soil engineer. We recommend that the contract documents provide for add-and-deduct unit prices for excavation and replacement as engineered fill to allow for unanticipated variations in excavation quantities.

BEDROCK CUT LOT TREATMENT

Cut lots that have subgrades exposing bedrock should be overexcavated and recompacted a minimum depth of 3 feet. The exposed surface should be scarified to a depth of about 12 inches, moisture conditioned to not less than 3 percent over optimum moisture content, and compacted to at least 90 percent relative compaction. This is to allow for easier excavation of utility trenches and planting of vegetation.

COLLUVIUM/ ALLUVIUM OVEREXCAVATION

Depending on the time of year that grading operations occur at the site, it may be necessary to rework the upper 3 feet of areas mapped as colluvium and alluvium prior to placement of fill. The necessity to rework these areas will depend on the presence of desiccation cracks in the soil. Desiccation cracks in these types of soils often extend to a depth of about 3 feet and occur late in the dry seasons as the soil moisture content decreases. We anticipate that the upper about 1½ feet will be reworked during normal stripping and scarification processes. If desiccation cracks extend below the depth of scarification, additional reworking will be required as determined in the field by the soil engineer. The need for additional reworking of colluvium and alluvium can be reduced if grading occurs early in the grading season, prior to drying of the soil and the formation of desiccation cracks.

EXPANSION POTENTIAL

As indicated by the results of our Atterberg limits and single-point consolidation/swell tests on the on-site soil and bedrock materials, the expansion potential of the on-site soil material is generally moderate. The total swell of fill placed and compacted following the recommendations presented under *Site Preparation and Grading* are estimated as follows:

ESTIMATED POTENTIAL SWELL OF COMPACTED FILL	
Fill Thickness (feet)	**Swell (inches)**
5	¾
10	1
15	1¼

ESTIMATED POTENTIAL SWELL OF COMPACTED FILL	
Fill Thickness (feet)	**Swell (inches)**
35	1½

The above preliminary estimates of potential swells are based on a uniform mixture of soil and bedrock generated from the design cuts planned at the site. The actual swell in fill areas will depend on the total depth of fill, the depths of placement of various materials in the fill, and the in-place moisture content and density.

SETTLEMENT

The results of single-point consolidation tests on remolded soil samples from the site, representing proposed fill, are summarized in Appendix B. Based on these results, we estimate that on-site soil and bedrock materials used as fill will undergo some settlement during placement and for a duration following mass grading. The total settlement of the fill placed and compacted following the recommendations presented under *Site Preparation and Grading* are estimated as follows:

PRELIMINARY ESTIMATED POTENTIAL SETTLEMENT OF COMPACTED FILL	
Fill Thickness (feet)	**Preliminary Estimate of Total Settlement (inches)**
5	¼
10	½
15	1
20	1¾
25	2½
30	3½
35	4½

Based on our laboratory test results and our experience, we anticipate that about 70 percent of the estimated total settlement of the fill should occur during mass grading. Therefore, we estimate that the maximum post-grading settlement will be less than about 1½ inches.

SETTLEMENT OF COLLUVIUM AND ALLUVIUM

In some areas at the site, up to about 32 feet of fill is planned at locations underlain with colluvium and alluvium extending to depths of about 25 feet down to bedrock. Based on our boring log data and the results of our laboratory testing, we believe that the colluvium and alluvium at the site consist of stiff to very stiff, silty to sandy clays and clayey sands. Settlement of these deposits should take place upon application of the new fill loads, and should be on the order of less than about 2 inches. This settlement should not adversely affect the proposed development.

RESIDENTIAL FOUNDATIONS

GENERAL

The site soils generally consist of stiff colluvial and alluvial soils with shallow bedrock between 5 to 20 feet deep. Provided the grading recommendations presented in this report are adhered to, the proposed homes may be supported on either structural mat/slab or drilled cast-in-place concrete pier and grade beam foundations. Recommendations and design parameters for these foundation types are as follows:

STRUCTURAL MAT/SLAB FOUNDATIONS

Structural mat/slab foundations may consist of either conventional reinforced or post-tensioned concrete slab foundations. The slab foundation should be designed by a structural engineer to accommodate 2-inches of total soil movement and 1-inch in 25 horizontal feet of differential soil movement without structural distress to the slab and excessive deflections in the building framing and wall finishes. We recommend that the following criteria be incorporated in the design of the slab foundations:

Allowable Bearing Capacity (may be increased by ⅓ for seismic and wind load)	1,500 psf
Passive Equivalent Fluid Pressure (neglect the upper 1 foot if ground surface is not confined by slabs or pavement)	300 psf
Base Friction Coefficient	0.3
Minimum Interior Span	15 feet
Minimum Perimeter Cantilever	5 feet
Edge Variation Distance Center Lift Edge Lift	 4.5 feet 5.5 feet
Differential Swell Center Lift Edge Lift	 0.85 inch 0.64 inch
Minimum Slab Thickness	10 inches

The upper 12 inches of subgrade soil should be presaturated to at least 5 percent above optimum moisture content. The presaturated pad should not be allowed to dry out to less than this recommended moisture content prior to the construction of the slab.

Where moisture vapor transmission through the slab would be objectionable, the use of a vapor retarder and capillary moisture break should be considered by the designer of the slab and floor covering. The thickness of the slab, capillary break, and vapor retarder should be determined by the slab and floor covering designers.

PIER AND GRADE BEAM

Drilled cast-in-place reinforced concrete friction piers and grade beams are suitable foundation support for the proposed homes. Foundation support would be provided by skin friction between the pier shaft and surrounding soil. The reinforced concrete piers and grade beams should be designed by a structural engineer with the following minimum parameters.

Minimum depth below finish soil pad grade (feet)	8
Minimum diameter (inches)	12
Minimum pier spacing	6 pier diameters measured center-to-center
Allowable skin friction (psf)	450
Passive pressure (pcf, equivalent fluid pressure)	300
Minimum Grade beam embedment (inches)	6

Skin friction should be neglected in the upper 1 foot below adjacent grade. Passive pressure should be neglected in the portion of pier shaft that is less than 10 horizontal feet from a slope face. The recommended friction and passive pressure values may be increased by ⅓ for short-term wind and seismic effects. The minimum parameters above are preliminary in nature and should be re-evaluated as site grading exposes soil and bedrock conditions at the locations where drilled pier foundations are being considered.

Prior to placement of reinforcing steel and concrete, the bottom of the pier excavations should be free of excess loose soil and debris. Water that has collected in pier hole excavations should be pumped out or displaced by means of a tremie method.

RETAINING WALLS

Retaining walls up to about 10 feet high are planned on the east side of Windsor Drive. Numerous shorter walls, up to about 6 feet high, are planned at grade breaks between lots and at toes of slopes. Additionally, a proposed vehicle crossing for the unnamed Kelly Creek tributary located near the southwest corner of the site will include a culvert covered with fill supported by headwalls. The vehicle crossing over the shallow swale designated as a wetland is under consideration for similar construction of a culvert supported by retaining walls as well. It is our opinion that these retaining walls can be supported on footing foundations founded on engineered fill or firm native soils. However, headwalls for the earth and culvert crossings over the unnamed Kelley Creek tributary should be pier supported. We recommend that the following geotechnical criteria be incorporated in the design of retaining walls:

Active Equivalent Fluid Pressure Level Backfill Sloping Backfill	 50 pcf 65 pcf
At-rest Equivalent Fluid Pressure	75 pcf
Allowable Bearing Capacity (may be increased by one-third for seismic and wind loads)	2,500 psf

Passive Equivalent Fluid Pressure (neglect the upper 1 foot if the ground surface is not confined by slabs or pavement)	350 pcf
Friction Coefficient	0.3
Minimum Footing Depth	18 inches below the lowest adjacent grade
Minimum Footing Width	24 inches

Piers used for the support of headwalls of the unnamed Kelley Creek tributary should be designed per the parameters presented in the table under *Vehicle Bridge Foundations*. Additionally, the above recommended lateral pressures are based on drained conditions, and do not include any surcharges; therefore, the designer should include the appropriate surcharge loads to the retaining walls.

To prevent hydrostatic pressure build-up, retaining walls should be constructed with permanent backdrains. The backdrain should consist of a blanket of Class 2 Permeable Material and a 4-inch diameter perforated PVC pipe (SDR 35). The permeable materials should be in conformance with Section 68-1.025 of the 1999 Caltrans "Standard Specifications." The permeable material blanket should be at least 12 inches thick and should be placed from the base of the retaining wall to about 1 foot below the finished grade behind the retaining wall. Alternatively, a geo-composite drain, such as Miradrain 6000 or an approved equivalent, may be used in lieu of the Class 2 Permeable Material blanket. The perforated pipe should be placed near the bottom of the wall to carry collected water to a suitable gravity discharge.

VEHICLE AND PEDESTRIAN BRIDGE FOUNDATIONS

If the vehicle crossing over the shallow swale designated as a wetlands site is to be constructed as a vehicle bridge, we recommend that the vehicle bridge can be supported on drilled cast-in-place concrete pier foundations. The piers should be designed in accordance with Caltrans seismic design criteria. The following criteria should be incorporated in the design and construction of the bridge foundations:

Allowable Skin Friction (may be increased by one-third for seismic and wind loads)*	600 psf (downward) 300 psf (uplift)
Passive Equivalent Fluid Pressure*	350 pcf
Minimum Pier Diameter	18 inches
Minimum Pier Depth	15 feet below lowest adjacent grade
Minimum Pier Spacing	3 pier diameters

* = Neglect upper 5 ± foot of pier embedment.

The piers should be tied together with pier caps to improve their overall resistance to lateral loads. We estimate that total bridge settlement will be less than about 1-inch, with differential settlement between bridge supports being less than about ½-inch.

Abutment and wing walls should also be supported on piers as discussed above, and designed for active and at-rest pressures of 50 and 75 pcf, respectively. The walls should be provided with permanent drainage to prevent buildup of hydrostatic pressure. The drains should consist of a blanket (1 foot minimum thickness) of Class 2 Permeable Material (conforming to Section 68-1.025) State of California Standard Specifications, dated July 1999, and weep holes near the bottom of the wall. The weep holes should be at least 3 inches in diameter at a spacing of not more than 6 feet center to center.

The backfill of abutment and wing walls should consist of Structure Backfill conforming to Section 19-3.0-6 of the 1999 edition of the Caltrans Standard Specifications. The abutment backfill should be moisture conditioned to above optimum moisture content and compacted to at least 95 percent relative compaction per ASTM D1557-00.

SEISMIC DESIGN CRITERIA

The following Caltrans seismic design parameters should be incorporated in the structural design of the proposed vehicle bridge:

Closest Distance to Known Seismic Source (Rodgers Creek)	7 km
Maximum Credible Earthquake Magnitude	7.0 Mw
Soil Profile Type	S_D
Peak Ground Acceleration	0.5g

PRELIMINARY PAVEMENT SECTIONS

The following recommendations for asphalt concrete pavement sections are preliminary only. Pavement analyses are based on an assumed "R" (resistance) value of 5, which we expect to be representative of final pavement subgrade materials, Caltrans *Design Method for Flexible Pavement*, and traffic indices (TI's), which are indications of traffic load frequency and intensity. Assigned TI's should include provisions for heavy truck traffic related to construction activities. We recommend the following preliminary pavement sections:

PRELIMINARY RECOMMENDED PAVEMENT SECTIONS		
Traffic Index (TI)	Thickness (inches)	
	Asphalt Concrete Type B	Class 2 Aggregate Base
4	2½	8
4½	2½	10
5	2½	11
5½	3	12
6	3	14

Since on-site materials vary from sandstone to clay, samples should be obtained from the rough roadway subgrade after mass grading. R-value tests should be performed on these samples. Final pavement section recommendations should be made on the basis of these test results.

Prior to subgrade preparation, all utility trench backfill should be properly placed and compacted. Subgrade soils should be rolled to at least 95 percent relative compaction to provide a smooth, unyielding surface. Subgrade soils should be maintained in a moist and compacted condition until covered with the complete pavement section.

Class 2 aggregate base should conform to the requirements in Section 26 of Caltrans's *Standard Specifications* (July, 1999). The aggregate base should be placed in thin lifts in a manner to prevent segregation, uniformly moisture conditioned, and compacted to at least 95 percent relative compaction to provide a smooth, unyielding surface. Relative compaction refers to the in-place dry density of soil expressed as a percentage of the maximum dry density of the same soil, as determined by the ASTM D1557-00 compaction test method.

Where drop inlets or other surface drainage structures are to be installed, slots or weep holes should be provided to allow free drainage of the contiguous aggregate base section.

EXTERIOR FLATWORK

It is our opinion the exterior concrete flatwork may be placed directly on the finish soil subgrade. The soil subgrade should be compacted to a minimum 90 percent relative compaction at a moisture content not less than 3 percent over optimum. All exterior concrete flatwork be cast free from adjacent footings or building slabs. The moisture-conditioned subgrade should not be allowed to lose moisture prior to concrete placement. If the subgrade dries out and shrinkage cracks appear, the subgrade should be reconditioned in accordance with the recommendations of the geotechnical engineer in the field.

UTILITY TRENCHES

All excavations should conform to applicable state and federal industrial safety requirements. Where trench excavations are deeper than 5 feet, they should be sloped no steeper than 1H:1V and/ or shored. Flatter side slopes may be required if seepage is encountered during construction or if the exposed materials differ from those described in the test pit and boring logs. If fully sloped trench walls cannot be excavated due to site constraints, shoring should be provided to ensure trench stability for worker safety. We can provide parameters for shoring design on request.

Material quality, placement procedures, and compaction requirements for utility line bedding and shading materials should meet the City of Petaluma and/or applicable utility agency requirements. From a geotechnical standpoint, the material above the shading material may consist of native materials, compacted to no less than 90 percent relative compaction and 3 percent over optimum moisture content.

Depending on time of year, location, and recent rainfall, ground water may be intercepted during trench excavation, in which case local dewatering will be required. The actual dewatering technique to be used should be approved by the soil engineer before implementation.

CORROSION TESTING

We have obtained three soil samples from the site for corrosion testing. The corrosion testing was performed by CERCO Analytical, Inc., of Pleasanton, California, and the test results are included in Appendix C. The corrosion test results should be transmitted to your structural engineer and underground utility designer, and should be incorporated in the design of the concrete and pipes to be placed directly against the on-site soils.

SEISMIC HAZARDS

SURFACE FAULT RUPTURE

We did not encounter evidence of Quaternary fault traces crossing, passing near, or trending toward the site. The site is not located within an official State of California earthquake fault zone (Davis 2000; Hart and Bryant, 1999) for active faults. According to the State of California, a fault is considered active if it has demonstrated Holocene activity (within the past 11,000 years). We conclude that the potential for surface fault rupture at the site is low.

GROUND SHAKING

The site is located in a region of high seismicity given the proximity of the Rodgers Creek fault, San Andreas fault, and other active faults in the San Francisco Bay Area. As for all sites in the Bay Area, the project can be expected to experience at least one moderate to severe earthquake during the life span of the development. Ground shaking is a hazard that cannot be eliminated but can be partially mitigated through proper attention to seismic structural design and observance of good construction practices.

The table below presents seismic design parameters in accordance with the 2001 California Building Code (CBC). The governing fault is the Rodgers Creek fault, considered a Type A fault, located 10 km from the site.

SEISMIC DESIGN PARAMETERS PER 2001 CBC	
Description	Parameter
Seismic Zone	4
Soil Profile Type	S_D
Seismic Source Type	A
Closest Distance to Known Seismic Source (Rodgers Creek fault)	10 km

The S_D soil profile corresponds to a stiff soil. This soil profile assumes residences will be constructed on the future engineered fill and the existing alluvium and colluvium.

SECONDARY EFFECTS OF GROUND SHAKING

Liquefaction is the temporary transformation of a saturated, cohesionless soil into a viscous liquid during strong ground shaking from a major earthquake. Dynamic densification can occur when dry, loose, cohesionless soil is subjected to earthquake vibrations of high amplitude. We did not encounter earth materials susceptible to liquefaction or significant dynamic densification at the site. Strong ground shaking during a major earthquake is liable to initiate landsliding in parts of the region. The stability of all slopes is lower during earthquake disturbances than at other times. Grading in accordance with the recommendations presented above (under *Landslide Remediation* and *Graded Slopes*) is expected to result in a low risk of seismically induced landslides.

ADDITIONAL SERVICES

Our firm should be afforded the opportunity to review the final plans and specifications to determine if the recommendations contained herein are incorporated into those documents. The review would be acknowledged in writing. Field observation and testing are essential and integral parts of this geotechnical investigation. Our firm should be retained to monitor earthwork and other relevant construction operations; the recommendations of this report are contingent on this.

LIMITATIONS

The conclusions and recommendations contained herein are based upon the information provided to us regarding proposed improvements, our geologic reconnaissance of the site, subsurface conditions encountered during the course of our field investigation, the results of our laboratory testing program, our experience in the area, and professional judgment. This study has been conducted in accordance with current professional geotechnical engineering and engineering geology standards; no other warranty is expressed or implied.

The locations of borings were determined by pacing from existing cultural features and other points of reference depicted on plans prepared by BKF and are considered approximate only. Site conditions described in the text are those existing at the time of our last site visit in April 2003 and are not necessarily representative of such conditions at other locations or times.

If it is found during construction that the conditions differ from those described on the boring and test pit logs, then the conclusions and recommendations contained within this report shall be considered invalid unless the changes are reviewed and the conclusions and recommendations modified or approved in writing by BGC.

Respectfully submitted,

BERLOGAR GEOTECHNICAL CONSULTANTS

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CERTIFIED ENGINEERING GEOLOGIST
STATE OF CALIFORNIA

Frank Berlogar

KJR/MGM/FG/FB:kr

Attachments:

- References
- Plate 1 - Vicinity Map
- Plate 2 - Geologic Map
- Plate 3 - Geologic Cross Sections A-A', B1-B1',B2-B2', B3-B3' and C-C'
- Plate 4 - Remedial Cross Sections A-A', B1-B1',B2-B2', B3-B3' and C-C'
- Plate 5 - Typical Fill Slope Details
- Plate 6 - Stock Pond Berm Upgrade Details
- Plate 7 - Typical Subdrain Details
- Plate 8 - Typical Cut/ Fill Transition Lot Overexcavation Details
- Appendix A - Field Investigation Data
  - A-1 through A-20 - Boring Logs
  - A-21 - Unified Soil Classification System
  - A-22 - Key to Rock Descriptions
  - A-23 through A-31 - Test Pit Logs
- Appendix B - Laboratory Test Results
  - B-1 – Atterberg Limits Test Results
  - B-2 through B-7 – Direct Shear Test Results
  - B-8 through B-11 – Compaction Test Data
  - B-12 through B-14 – Gradation Test Data
  - B-15 through B-17 – Consolidation Test Data
- Appendix C - CERCO Analytical, Inc. Corrosion Test Data

Copies: Addressee (6)
BKF (1)
Attention: Mr. Tom Morse

word/report/13211

REFERENCES

Blake, M.C., Jr., Bartow, J.A., Frizzell, V.A., Jr., Schlocker, J., Sorg, D., Wentworth, C.M., and Wright, R.H., 1974, Preliminary geologic map of Marin and San Francisco Counties and parts of Alameda, Contra Costa and Sonoma Counties, California: U.S. Geological Survey Miscellaneous Field Studies Map MF-574.

Davis, J., 2000, Digital images of official maps of the Alquist-Priolo earthquake fault zones of California, Central Coast Region, California Division of Mines and Geology: Division of Mines and Geology CD 2000-004-2000.

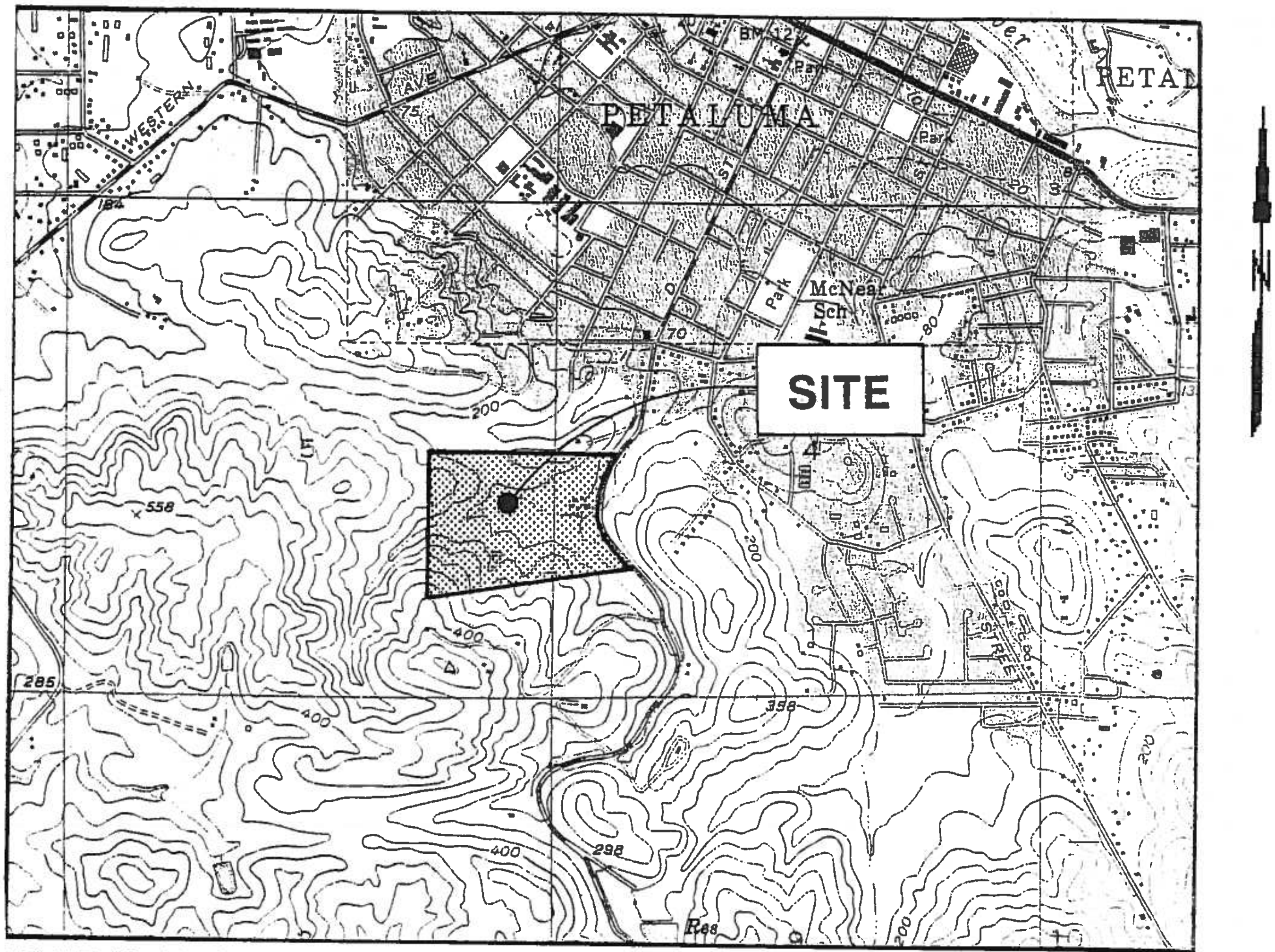
Hart, E.W., and Bryant, W.A., 1999, Fault-rupture hazard zones in California, Alquist-Priolo earthquake fault zoning act with index to earthquake fault zones maps: California Division of Mines and Geology Special Publication 42.

Peterson, M.D., Toppoxada, T., Cao, T., Cramer, C., Reichle, M., Maher, M., and Atchinson, L., 1998, Determining distances from faults within and bordering the state of California for the 1997 Uniform Building Code, in CDMG and SEAC, Maps of Known Active Fault Near-Source Zones in California and Adjacent Portions of Nevada, p. ix-xvii: International Conference of Building Officials, Whittier, California.

AERIAL PHOTOGRAPHS

Date	Photographer	Project, flight line, frames	Nominal scale
09/25/73	U.S. Geological Survey	2-58 and -59	1:24,000
04/19/86	Pacific Aerial Surveys	AV-2860-7-31, -32	1:12,000
06/15/00	Pacific Aerial Surveys	SON-AV-6540-19-34, -35	1:12,000

BY: FF DATE: 5-28-03 JOB NUMBER: 2616.100



SCALE: 1"= 2000'

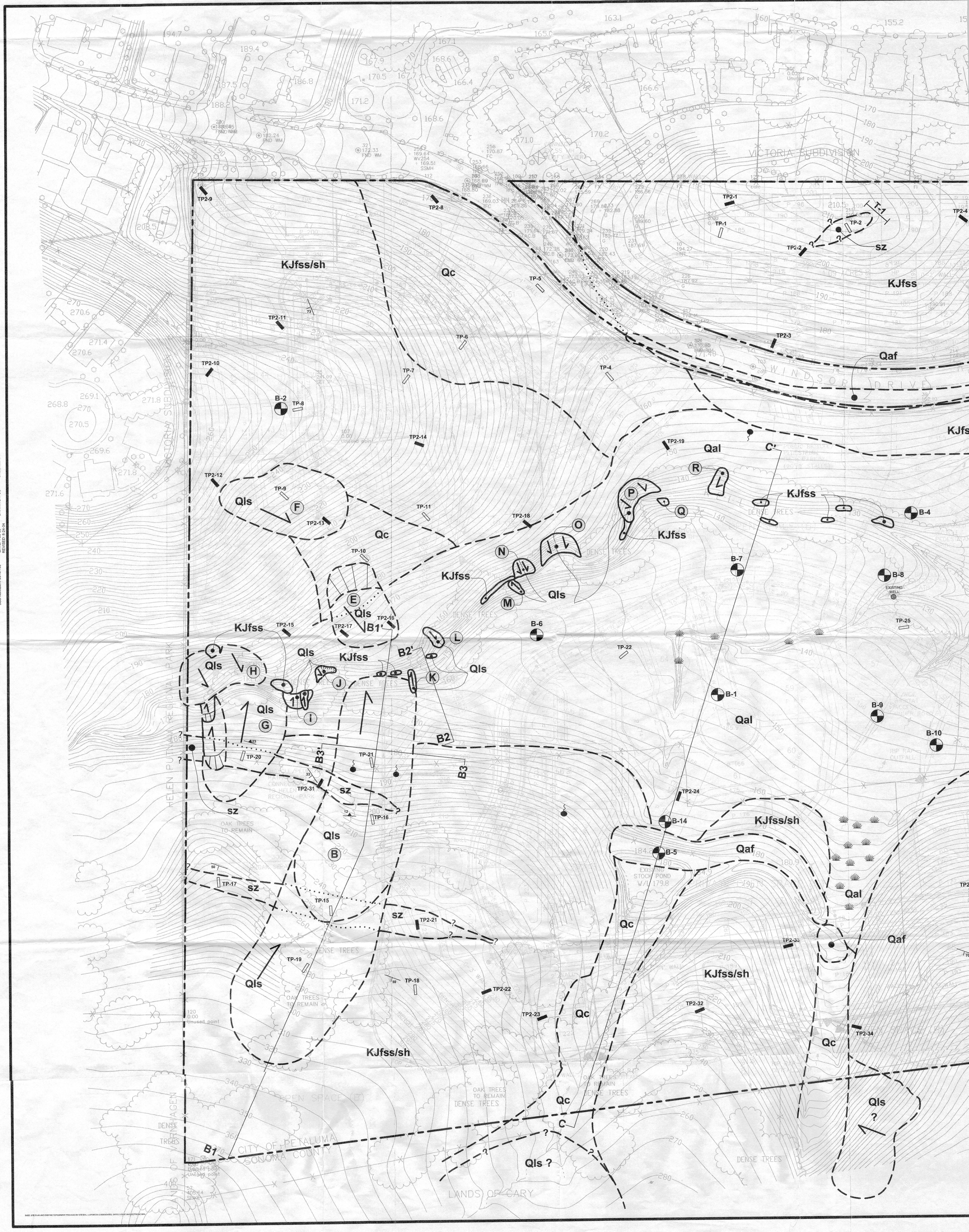
VICINITY MAP

UOP PROPERTY

"D" STREET AND WINDSOR DRIVE
PETALUMA, CALIFORNIA
FOR
DAVIDON HOMES

BASE: PORTION OF U.S.G.S. 7.5 MINUTE TOPOGRAPHIC QUADRANGLE,BENECIA AND CORDELIA, CALIFORNIA, PHOTOREVISED 1980, AT A SCALE OF 1:24,000.

PLATE 1



JOB NUMBER: 2616.100
DATE: 4-28-03
REVISED: 8-24-04
DRAWN BY: CC
CHECKED BY:
VICTORIA SUBDIVISION
WINDSOR DRIVE
HELEN PUTNAM REGIONAL PARK
CITY OF PETALUMA
SONOMA COUNTY
LANDS OF CARY
OPEN SPACE
DENSE TREES
OAK TREES TO REMAIN
EXISTING STOCK POND W/L 179.8
EXISTING WELL
KJfss/sh
KJfss
Qc
Qls
Qal
Qaf
sz
Qls ?
B-1
B-2
B-4
B-5
B-6
B-7
B-8
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B1
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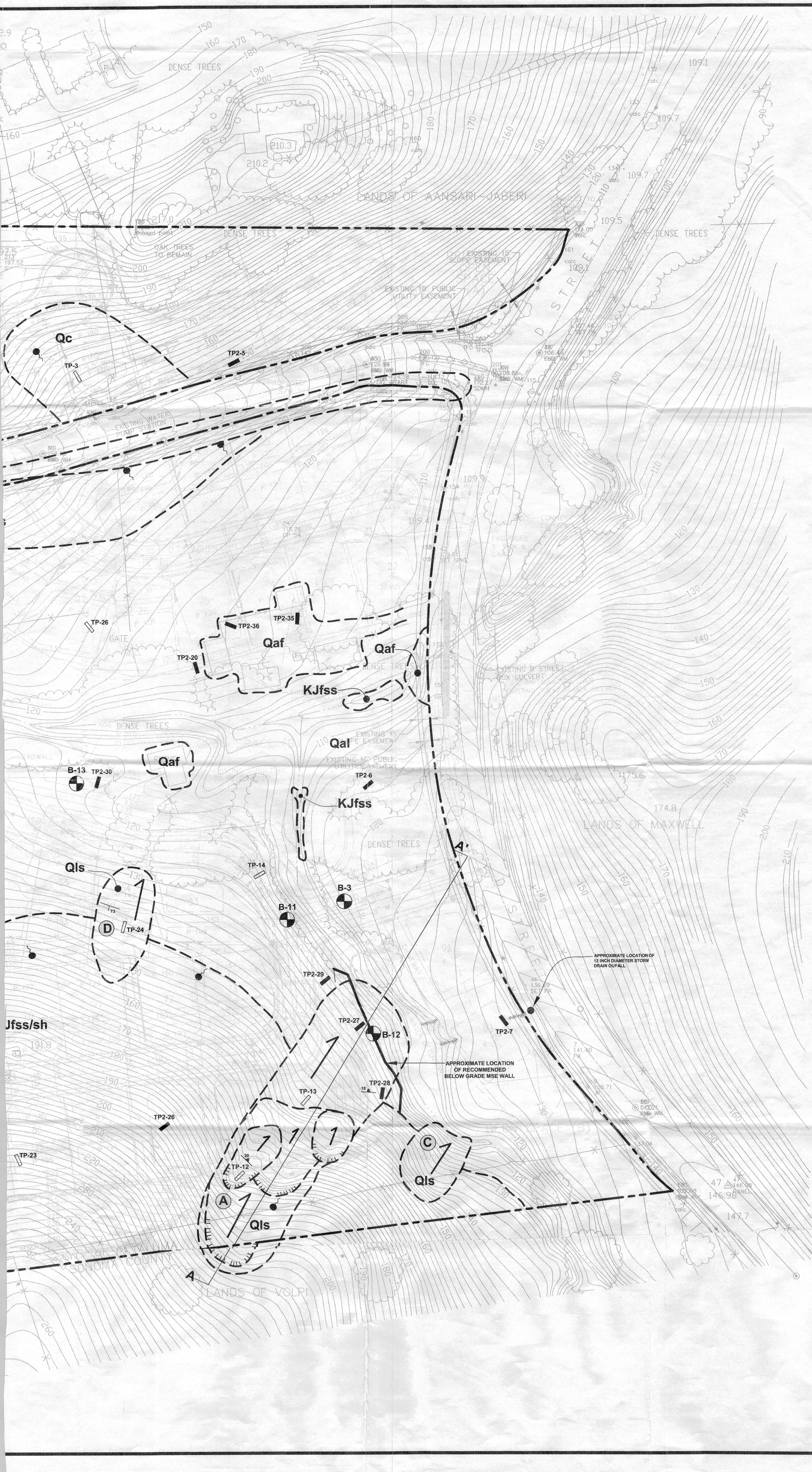
B2
B2'
B3
B3'
C
C'
T-1



SCALE
1"=50'

N

EXPLANATION

Symbol	Description
	PROPERTY LINE
?	GEOLOGIC CONTACT, DASHED WHERE APPROXIMATE, DOTTED WHERE CONCEALED, QUERIED WHERE UNCERTAIN
C C'	GEOLOGIC CROSS SECTION
T-1	TRENCH LOCATION, BGC 2002
B-14	BORING LOCATION, BGC 2003
TP2-36	TEST PIT LOCATION, BGC 2003
TP-26	TEST PIT LOCATION, BGC 2002
Qaf	ARTIFICIAL FILL
Qls	LANDSLIDE DEBRIS
Qc	COLLUVIUM
Qal	ALLUVIUM
KJfss	FRANCISCAN COMPLEX SANDSTONE
KJfss/sh	FRANCISCAN COMPLEX SANDSTONE AND SHALE
sz	SHEAR ZONE
73	STRIKE AND DIP OF BEDDDING
23	STRIKE AND DIP OF SHEAR
	LANDSLIDE, RECENTLY ACTIVE
	LANDSLIDE, SURFICIAL
	SPRING
	EROSION GULLY
	WATER SEEKING VEGETATION
R	LANDSLIDE DESIGNATION

GEOLOGIC MAP

UOP PROPERTY

"D" STREET AND WINDSOR DRIVE

PETALUMA, CALIFORNIA

FOR

DAVIDON HOMES

Berlogar Geotechnical Consultants

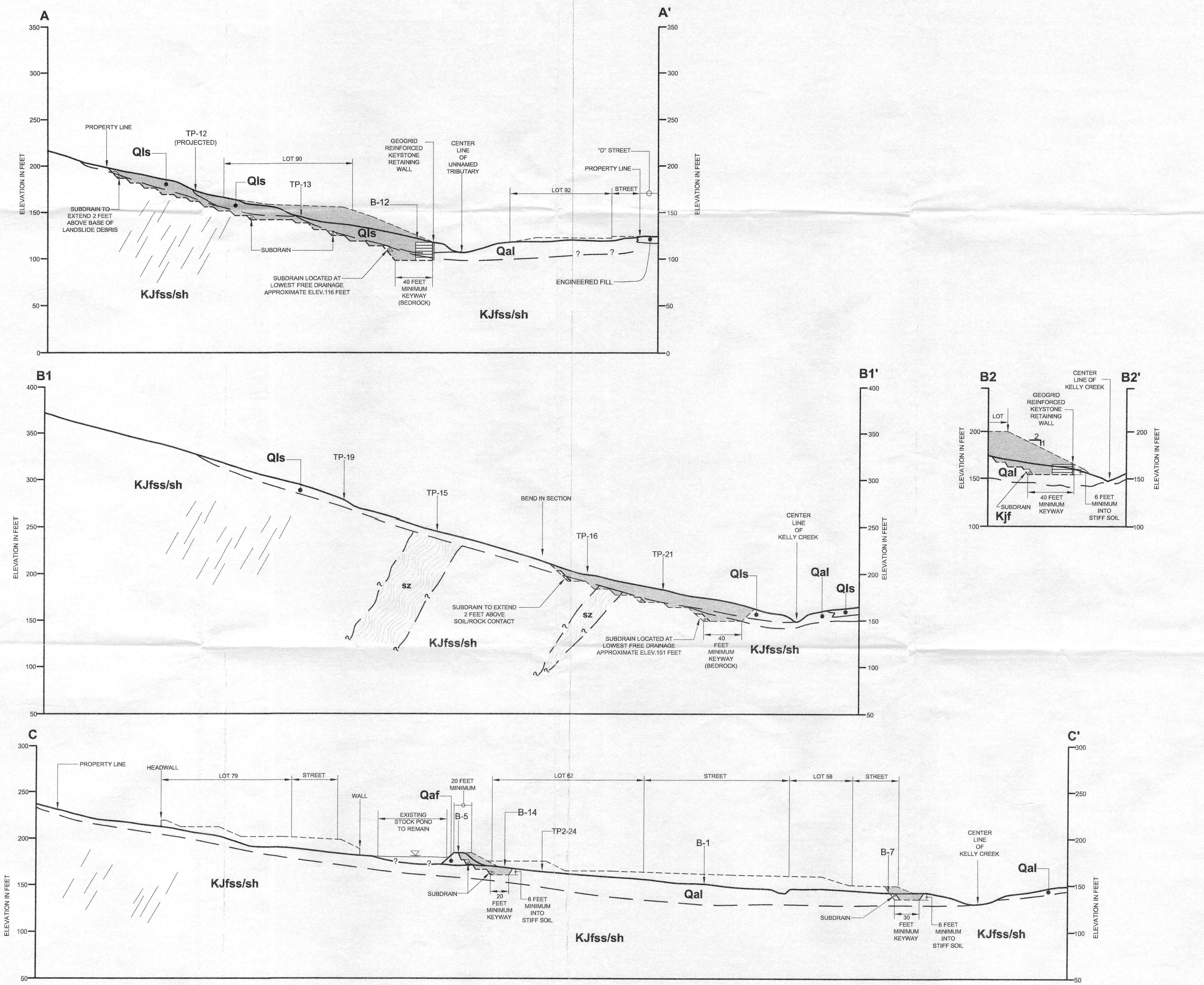
SOIL ENGINEERS * ENGINEERING GEOLOGISTS

JOB NUMBER: 2616.100 DATE: 5-3-03 REVISED: 8-24-04 DRAWN BY: CC CHECKED BY:

GEOLOGIC CROSS SECTIONS A-A', B1-B1', B2-B2', B3-B3' AND C-C'

UOP PROPERTY
"D" STREET AND WINDSOR DRIVE
PETALUMA, CALIFORNIA
FOR
DAVIDON HOMES
Berlogar Geotechnical Consultants
SOIL ENGINEERS * ENGINEERING GEOLOGISTS

PLATE 3



SCALE
HORIZONTAL AND VERTICAL
1"=50'

EXPLANATION

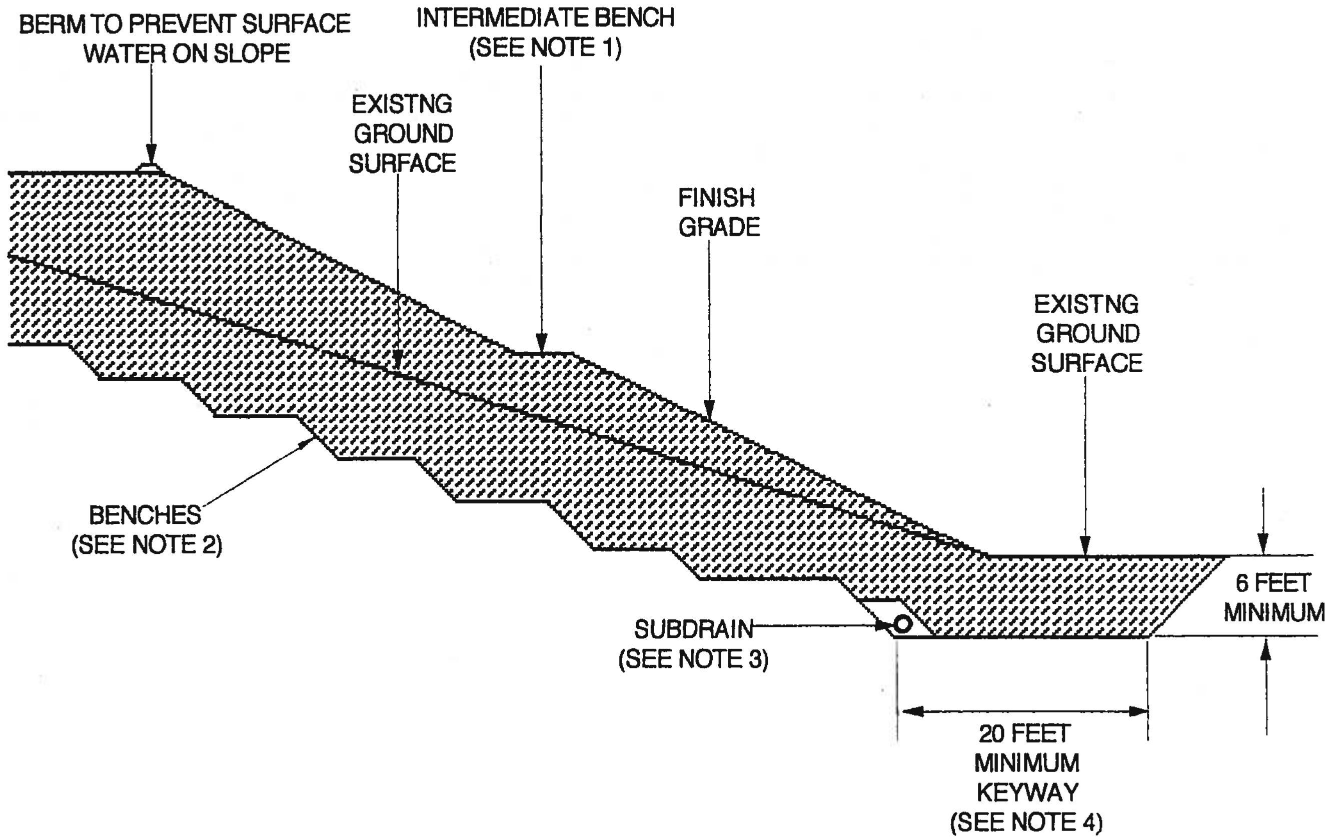
———	GROUND SURFACE
— — — ?	GEOLOGIC CONTACT, DASHED WHERE APPROXIMATE, QUERIED WHERE UNCERTAIN
B-14	BORING LOCATION, BGC THIS INVESTIGATION
TP2-24	TEST PIT LOCATION, BGC THIS INVESTIGATION
TP-21	TEST PIT LOCATION, BGC 2002
Qaf	ARTIFICIAL FILL
Qls	LANDSLIDE DEBRIS
Qal	ALLUVIUM
KJfss/sh	FRANCISCAN COMPLEX SANDSTONE
sz	SHEAR ZONE

REMEDIAL CROSS SECTIONS A-A', B1-B1', B2-B2', B3-B3' AND C-C'

UOP PROPERTY
"D" STREET AND WINDSOR DRIVE
PETALUMA, CALIFORNIA
FOR
DAVIDON HOMES
Berlogar Geotechnical Consultants
SOIL ENGINEERS * ENGINEERING GEOLOGISTS

PLATE 4

BY: FF
DATE: 5-6-03
JOB NUMBER: 2616.100

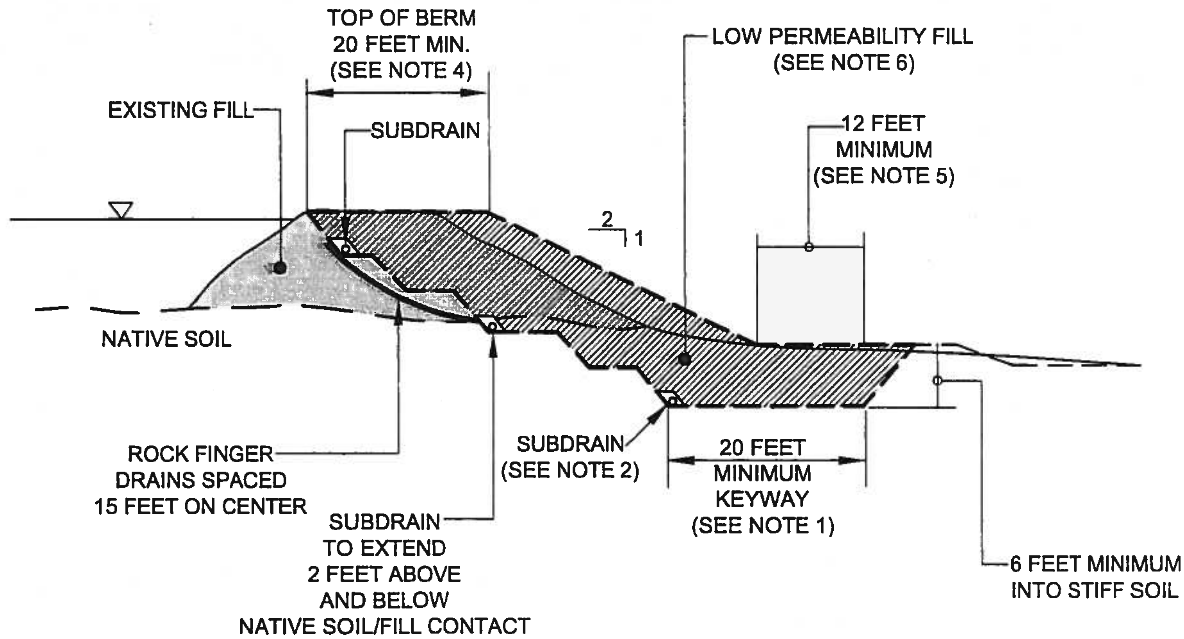


NOTES:

1. INTERMEDIATE BENCHES SHOULD BE SPACED EVERY 25 VERTICAL FEET ON SLOPES HIGHER THAN 30 FEET
2. WHERE NATURAL GRADE IS STEEPER THAN 7:1, BENCH INTO STIFF SOIL OR BEDROCK AS DETERMINED BY SOIL ENGINEER.
3. SUBDRAIN SHOULD DISCHARGE VIA A CLOSED PIPE TO STORM DRAIN OR SUITABLE NATURAL DRAINAGE.
4. KEYWAY SHOULD EXTEND AT LEAST (SIX) 6 FEET AND FOUNDED ON STIFF SOIL AS DETERMINED BY THE SOIL ENGINEER. KEYWAY WIDTH SHOULD BE A MINMIMUM OF 20 FEET OR 1/2 OF THE FILL SLOPE HEIGHT, WHICHEVER IS GREATER.

TYPICAL FILL SLOPE DETAIL

CHECKED BY:
DRAWN BY: FF
DATE: 6-12-03
JOB NUMBER: 2616.100



NOTES:

1. KEYWAY SHOULD EXTEND AT LEAST SIX(6) FEET INTO STIFF SOIL AS DETERMINED BY SOIL ENGINNER. KEYWAY WIDTH SHOULD BE A MINIMUM OF 20 FEET.

2. SUBDRAIN SHOULD DISCHARGE VIA A CLOSED PIPE TO STORM DRAIN OR SUITABLE NATURAL DRAINAGE.

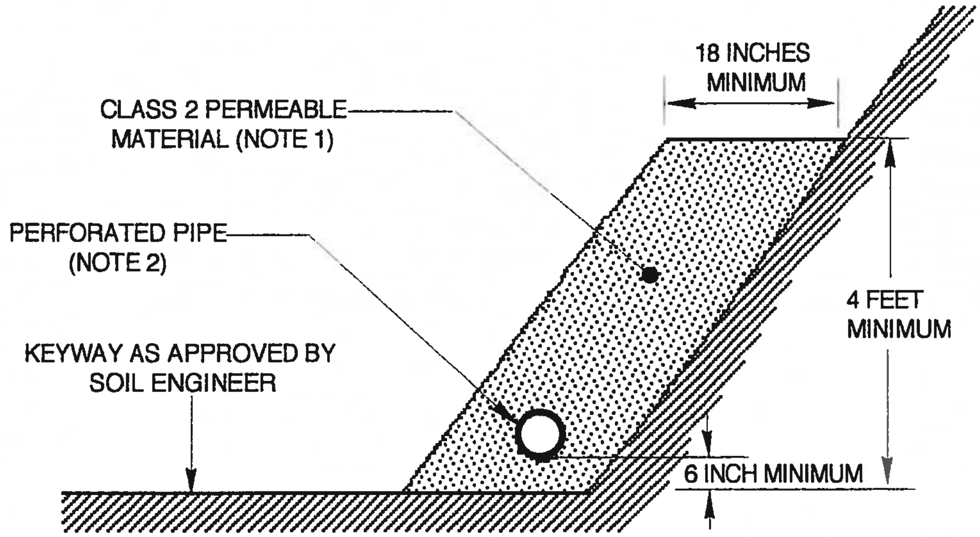
3. ANY DEBRIS ENCOUNTERED ON THE UPSLOPE SIDE OF THE KEYWAY EXCAVATION SHOULD BE REMOVED AND REPLACED WITH ENGINEERING FILL.

4. FINISHED TOP OF BERM SHOULD BE 20 FEET WIDE MINIMUM.

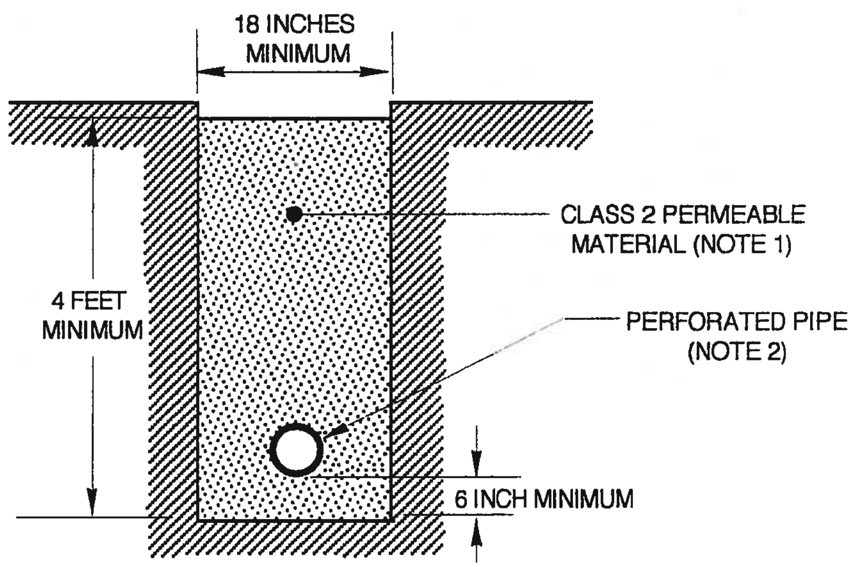
5. FRONT EDGE OF KEYWAY SHOULD BE AT LEAST A 2H:1V PROJECTION FROM TOE OF SLOPE. FOR 6 FEET DEEP KEYWAY THIS DISTANCE IS 12 FEET.

6. FILL SHOULD CONSIST OF LOW PERMEABILITY CLAYEY SOIL TYPE MATERIAL AS APPROVED IN THE FIELD BY THE SOIL ENGINEER.

STOCK POND BERM UPGRADE DETAILS



KEYWAY SUBDRAIN



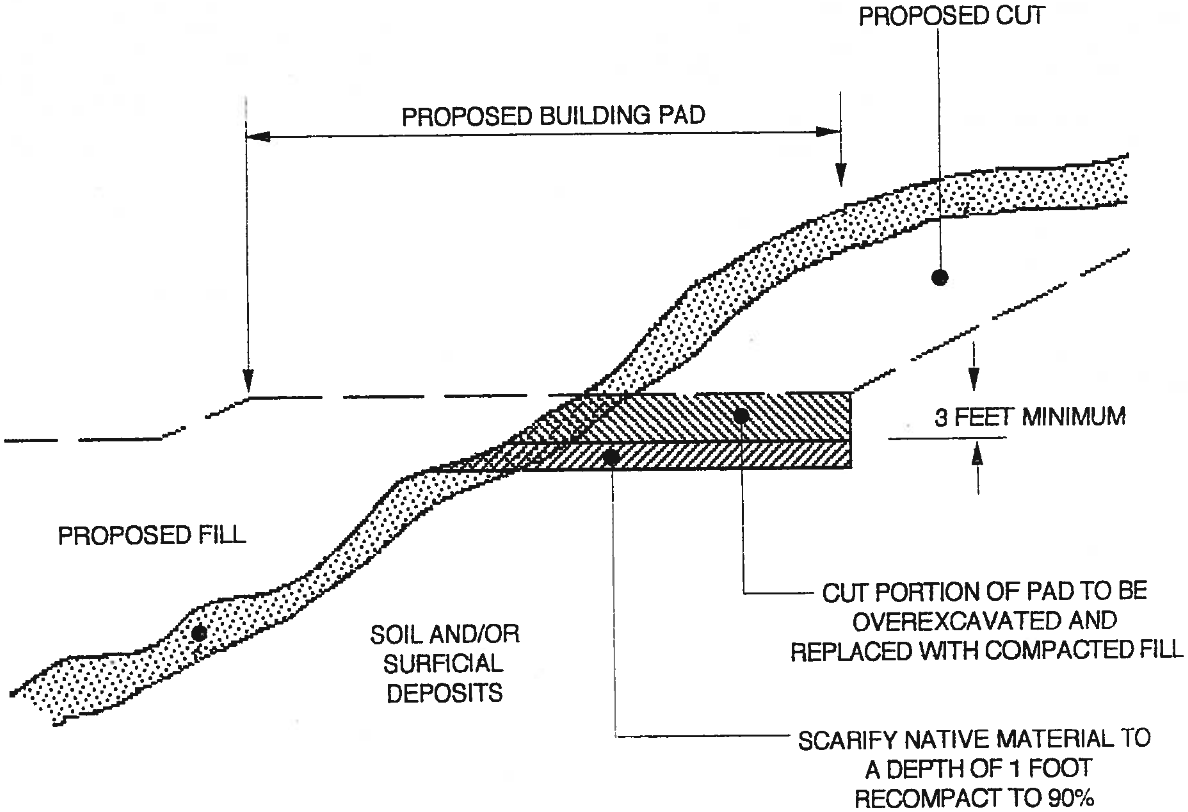
TRENCH SUBDRAIN

NOTES:

1. CLASS 2 PERMEABLE MATERIAL AS GIVEN IN SECTION 68-1.025, STATE OF CALIFORNIA STANDARD SPECIFICATION, JANUARY, 1999 EDITION.

2. PERFORATED PIPE PLACED PERFORATIONS DOWN, PVC PIPE WITH A MINIMUM DIAMETER OF SIX (6) INCHES, CONFORMING TO ASTM D-2751 SDR35 FOR FILLS LESS THAN 30 FEET AND SDR 23.5 FOR FILLS GREATER THAN 30 FEET.

SUBDRAIN DETAILS



TYPICAL CUT/FILL TRANSITION LOT OVEREXCAVATION DETAILS

PLATE 8

APPENDIX A

Field Investigation Data

BORING LOG B-1

JOB NUMBER: 2616.100 **DATE DRILLED:** 3-28-03

JOB NAME: UOP Property **SURFACE ELEVATION:** 151 feet

DRILL RIG: Rotary Wash **DATUM:** Mean Sea Level

SAMPLER TYPE:	DRIVE WEIGHT - LB	HEIGHT OF FALL - IN
2.5 inch I.D. Split Barrel	140	30
Standard Penetration Test	140	30

BLOWS PER FT.	MOISTURE CONTENT %	DRY UNIT WEIGHT p.c.f.	DEPTH IN FEET	USCS CLASSI-FICATION	DESCRIPTION
				CL	SILTY CLAY, dark gray-brown, moist, medium stiff, trace fine- grained sand
10	18.9	102			
				CL	SILTY CLAY, gray-brown, moist, stiff to very stiff, trace fine-grained sand
22	19.1	110	5		
25	22.3	103	10	CL	SANDY CLAY, light to medium gray-brown, moist, very stiff, fine to medium-grained sand, some silt
				SC	CLAYEY SAND, gray-brown, wet, medium dense, fine to medium-grained sand, trace silt, trace fine gravel
25	20.1	104	15		
14	-	-			
				CL	SILTY CLAY, gray-brown, wet, stiff to very stiff, trace fine-grained sand
22	19.9	108	20		

JOB NUMBER: 2616.100 **SHEET:** 2 **OF:** 2

JOB NAME: UOP Property **DEPTH:** 20 feet **TO** 24.5 feet

NOTES:

BLOWS PER FT.	MOISTURE CONTENT %	DRY UNIT WEIGHT p.c.f.	DEPTH IN FEET	USCS CLASSI-FICATION	DESCRIPTION
22	19.9	108		CL	SILTY CLAY, gray-brown, wet to saturated, stiff to very stiff, trace to some fine-grained sand
				CL	SANDY CLAY, light gray-brown, saturated, very stiff, fine to medium-grained sand
					SANDSTONE, fine-grained, tan-brown, highly weathered, strong
66	-	-			SHALE, black, slightly weathered, fractured, low hardness,
			25		Boring terminated at 24-1/2 feet. Free water encountered at 21 feet.
			30		
			35		
			40		

CORE LOG

BORING NO.: B-2 JOB NO.: 2616.100
DATE BEGUN: 3-28-03
DATE COMPLETED: 3-28-03
DEPTH OF HOLE: 50-1/2 feet
NUMBER OF CORE BOXES: 7
LOGGED BY: ROV

PROJECT: UOP Property
DRILLING COMPANY: Spectrum
DRILLING METHODS: Rotary Wash
ELEVATION (FEET): 249 feet

RUN NO.	DRILLRATE (MIN/FT.)	CUT	REC	% REC.	% DRILLING FLUID LOSS	RQD (%)
1	1 1.5 2.0 3.0 2.5/6"	4.5	4.0	80	0	0
2	1.5 1.0 2.0 3.5 7.0	5.0	4.0	80	0	0
3	2.0 4.5 7.0 6.5 4/6"	4.5	1.1	24	0	0

DEPTH	DESCRIPTION
0	SILTY CLAY, gray-brown, moist, stiff, some fine-grained sand
2	
	SANDSTONE, fine-grained, light orange-brown, highly weathered, crushed with some clay
4	
	SANDSTONE, fine-grained, light to medium brown-gray, highly weathered, weak, highly fractured to crushed
6	at 5.6 feet, joint 60° dip
	from 7.5 to 8.5 feet, light gray-brown, friable zone
8	from 8.5 to 9 feet, clay layer, 50° dip
	below 9 feet, becomes crushed sandstone
10	at 9.6 feet, joint 60° dip
	at 11.5 feet, joint 60° dip
12	from 12 to 12.5 feet, abundant calcite veinlets
14	SHALE, dark gray, highly weathered, moderately strong, crushed
16	

CORE LOG

PROJECT UOP Property BORING NUMBER B-2 JOB NUMBER 2616.100

RUN NO.	DRILLRATE (MIN/FT.)	CUT	REC	% REC.	% DRILLING FLUID LOSS	RQD (%)	DEPTH	LOG	DESCRIPTION
4	2.5	5.0	4.8	96	0	60			SHALE, dark gray, highly weathered, moderately strong, crushed SANDSTONE, fine-grained, gray, moderately to highly weathered, weak, highly fractured, limonite stains at 16.8 feet, joint 60° dip
	2.0						18		SHALE , dark gray, highly weathered, moderately strong, crushed SANDSTONE, fine-grained, gray, moderately to highly weathered, weak to moderately strong, highly fractured, limonite stains
	2.0								at 19 feet, joint 60° dip limonite stains on fracture surfaces
	2.0						20		
	2.0								
5	2.0	5.0	5.0	100	0	60			at 21.2 feeet, joint 65° dip at 21.8 feet, fracture 70° dip
	1.5						22		
	1.5								
	1.5						24		
	2.0								
6	2.0	5.0	5.0	100	0	60	26		at 26.4 feet, joint 55° dip
	2.0								at 27.2 feet, crushed zone
	2.0						28		
	2.0								at 29.5 feet, thin bedding laminations 55° to 60° dip
	2.0						30		at 30.4 feet, bedding 55° dip SHALE, black, moderately to highly weathered, weak, crushed
7	2.5	5.0	5.0	100	0	10	32		

BERLOGAR GEOTECHNICAL CONSULTANTS

CORE LOG

PROJECT: UOP Property BORING NUMBER: B-2 JOB NUMBER: 2616.100

RUN NO.	DRILLRATE (MIN/FT.)	CUT	REC	% REC.	% DRILLING FLUID LOSS	RQD (%)
7	2.0, 1.5, 2.0, 2.0	5.0	5.0	100	0	10
8	6.0	0.5	0.5	100	0	0
9	3.0, 3.0, 2.0, 3.0	4.0	3.6	90	0	10
10	3.0, 1.5, 2.0, 2.0, 3.0	5.0	5.0	100	0	16
11	3.0, 2.5, 2.0	5.0	5.0	100	0	14

DEPTH: 34, 36, 38, 40, 42, 44, 46, 48

DESCRIPTION

SHALE, black, moderately to highly weathered, weak, crushed

SANDSTONE, fine-grained, gray, moderately weathered, weak to moderately strong, highly fractured, thickly bedded with shale, black, highly weathered, weak, curshed

at 35.5 feet, 2-inch thick clay seam, 40° dip

from 36.8 to 37 feet, shale layer, bedding 50° dip

SANDSTONE, fine-grained, gray, slightly weathered, moderately strong

at 42.2 feet, bedding 50° dip

SHALE, black, highly weathered, weak, crushed, faintly sheared

SANDSTONE, fine-grained, gray, moderately weathered, strong, highly fractured

CORE LOG

PROJECT UOP Property BORING NUMBER B2 JOB NUMBER 2616.100

RUN NO.	DRILLRATE (MIN/FT.)	CUT	REC	% REC.	% DRILLING FLUID LOSS	RQD (%)	DEPTH	LOG	DESCRIPTION
11	2.0 2.0 2.5	5.0	5.0	100	0	14	50		SANDSTONE, fine-grained, gray, moderately weathered, strong, highly fractured
							52 54 56 58 60 62 64		Boring terminated at 50-1/2 feet.

BORING LOG B-3

JOB NUMBER: 2616.100 **DATE DRILLED:** 4-2-03

JOB NAME: UOP Property **SURFACE ELEVATION:** 118 feet

DRILL RIG: Solid Flight Auger **DATUM:** Mean Sea Level

SAMPLER TYPE:	DRIVE WEIGHT - LB	HEIGHT OF FALL - IN
2.5 inch I.D. Split Barrel	140	30
Standard Penetration Test	140	30

BLOWS PER FT.	MOISTURE CONTENT %	DRY UNIT WEIGHT p.c.f.	DEPTH IN FEET	USCS CLASSIFICATION	DESCRIPTION
				ML	SANDY SILT, brown, dry to moist, medium dense, fine-grained sand, trace to some clay, rootlets
20	20.0	101		CL	SILTY CLAY, dark brown, moist, stiff, trace fine-grained sand, trace well rounded gravel up to 1/4-inch diameter, faint iron oxide mottling
				CL	SILTY CLAY, dark yellow-brown, moist, stiff, some fine-grained sand, faint iron oxide mottling
33	17.4	106	5		
				CL	SANDY CLAY, dark yellow-brown, moist, very stiff, medium-grained sand, trace to some well rounded gravel up to 1/8-inch diameter
43	19.8	106	10	SC	CLAYEY SAND, mottled orange-brown and gray, moist.dense, fine to medium-grained sand, trace well rounded gravel up to 1/4-inch diameter
50/6"	15.1	92	15		SHALE, gray, highly weathered, weak, crushed, thinly laminated at 65° to 70°.
					Boring terminated at 15 feet. No free water encountered.
			20		

BORING LOG B-4

JOB NUMBER: 2616.100 **DATE DRILLED:** 4-2-03

JOB NAME: UOP Property **SURFACE ELEVATION:** 137 feet

DRILL RIG: Solid Flight Auger **DATUM:** Mean Sea Level

SAMPLER TYPE:	DRIVE WEIGHT - LB	HEIGHT OF FALL - IN
2.5 inch I.D. Split Barrel	140	30
Standard Penetration Test	140	30

BLOWS PER FT.	MOISTURE CONTENT %	DRY UNIT WEIGHT p.c.f.	DEPTH IN FEET	USCS CLASSI-FICATION	DESCRIPTION
				ML	SANDY SILT, brown, moist, medium dense, fine-grained sand
57	19.5	102		SM	SILTY SAND, orange-brown, moist, dense to very dense, medium-grained sand, trace clay
50/3"	-	-			
50/4"	10.4	107	5		SANDSTONE, fine-grained, tan-brown, highly weathered, weak to moderately strong, moderately fractured with manganese oxide on surfaces
60/6"	-	-			
					Boring terminated at 8 feet. No free water encountered.
			10		
			15		
			20		

BORING LOG B-5

JOB NUMBER: 2616.100 **DATE DRILLED:** 4-2-03

JOB NAME: UOP Proeprty **SURFACE ELEVATION:** 185 feet

DRILL RIG: Solid Flight Auger **DATUM:** Mean Sea Level

SAMPLER TYPE:	DRIVE WEIGHT - LB	HEIGHT OF FALL - IN
■ 2.5 inch I.D. Split Barrel	140	30
◿ Standard Penetration Test	140	30

BLOWS PER FT.	MOISTURE CONTENT %	DRY UNIT WEIGHT p.c.f.	DEPTH IN FEET	USCS CLASSI-FICATION	DESCRIPTION
13	12.8	105		CL	SILTY CLAY, mixed brown and gray-brown, moist, medium stiff to stiff, trace fine to coarse-grained sand, trace cobbles (fill)
32	15.7	108	5	CL	SILTY CLAY, dark gray-brown to brown, moist, stiff, trace gravel, trace fine-grained sand (fill)
25	18.7	108	10		
22	17.1	110	15	CL	SILTY CLAY, dark brown, moist, stiff, some medium-grained sand, trace subrounded gravel up to 1/8-inch diameter
36	16.7	113	20	CL	SILTY CLAY, brown to dark yellow-brown, stiff, trace fine-grained sand, trace subrounded gravel up to 1/2-inch diameter , 1/16-inch thick gray clay films

JOB NUMBER: 2616.100 **SHEET:** 2 **OF:** 2

JOB NAME: UOP Property **DEPTH:** 20 feet **TO** 30-1/4 feet

NOTES:

BLOWS PER FT.	MOISTURE CONTENT %	DRY UNIT WEIGHT p.c.f.	DEPTH IN FEET	USCS CLASSI-FICATION	DESCRIPTION
36	16.7	113		CL	SILTY CLAY, brown to dark yellow-brown, moist, stiff, trace fine-grained sand, trace subrounded gravel up to 1/2-inch diameter, 1/16-inch gray clay films
25	17.5	116	25	SC / CL	SANDY CLAY / CLAYEY SAND, yellow-brown-brown, moist, stiff to medium dense, medium-grained sand
65/6"	-	-			SHALE, gray, highly weathered, weak, highly fractured from 65° to 70° .
50/ 2.5"	-	-	30		
			35		Boring terminated at 30-1/4 feet. Free water encountered at 20 feet, rose to 17 feet in 4 hours.
			40		

BORING LOG B-6

JOB NUMBER: 2616.100 **DATE DRILLED:** 4-2-03

JOB NAME: UOP Property **SURFACE ELEVATION:** 157 feet

DRILL RIG: Solid Flight Auger **DATUM:** Mean Sea Level

SAMPLER TYPE:	DRIVE WEIGHT - LB	HEIGHT OF FALL - IN
2.5 inch I.D. Split Barrel	140	30
Standard Penetration Test	140	30

BLOWS PER FT.	MOISTURE CONTENT %	DRY UNIT WEIGHT p.c.f.	DEPTH IN FEET	USCS CLASSIFICATION	DESCRIPTION
				ML	CLAYEY SILT, brown, moist, stiff, trace fine-grained sand
13	14.6	115		SC	CLAYEY SAND, gray-brown, moist, medium dense, medium-grained sand
17	-	-	5	CL	SANDY CLAY, mottled gray-brown and brown, moist, stiff, medium-grained sand, trace well rounded gravel up to 1/8-inch diameter, iron oxide stains
39	15.9	111		SC	CLAYEY SAND, mottled light gray and orange-brown, moist, dense, medium to coarse-grained sand, trace well rounded to subrounded gravel up to 1/8-inch diameter
74	14.4	108	10	SC	CLAYEY SAND AND SILT, mottled orange-brown and gray, moist, very dense, coarse-grained sand, weakly cemented
50/6"	8.5	129	15		SHALE, gray to black, highly weathered, weak to moderately strong, crushed, 40° joints possible bedding
60/2"	-	-			
			20		Boring terminated at 19-1/2 feet. No free water encountered.

BORING LOG B-7

JOB NUMBER: 2616.100 **DATE DRILLED:** 4-2-03

JOB NAME: UOP Property **SURFACE ELEVATION:** 144 feet

DRILL RIG: Solid Flight Auger **DATUM:** Mean Sea Level

SAMPLER TYPE:	DRIVE WEIGHT - LB	HEIGHT OF FALL - IN
2.5 inch I.D. Split Barrel	140	30
Standard Penetration Test	140	30

BLOWS PER FT.	MOISTURE CONTENT %	DRY UNIT WEIGHT p.c.f.	DEPTH IN FEET	USCS CLASSI-FICATION	DESCRIPTION
				ML	SANDY SILT, brown, moist, medium stiff
7	18.2	107		SC	CLAYEY SAND, gray, wet, medium dense, coarse-grained sand
22	-	-		CL	SILTY CLAY, mottled tan-brown and gray-brown, moist, very stiff, trace fine-grained sand, trace well rounded gravel up to 1/4-inch diameter
38	15.2	112	5	CL	SANDY CLAY, mottled gray-brown and gray, moist, very stiff, fine to medium-grained sand, trace subrounded gravel up to 1/4-inch diameter
				SM	SILTY SAND, gray, saturated, medium dense, some clay
40	15.5	111	10	SC	CLAYEY SAND, mottled orange-brown and gray, moist, very dense, coarse-grained sand, trace subrounded gravel up to 1/4-inch diameter
60/2"	-	-			SANDSTONE, fine to medium-grained, tan-brown, highly weathered, weak, highly fractured
			15		Boring terminated at 14-1/6 feet. Free water encountered at 7 feet.
			20		

BORING LOG B-8

JOB NUMBER: 2616.100 **DATE DRILLED:** 4-2-03

JOB NAME: UOP Property **SURFACE ELEVATION:** 134 feet

DRILL RIG: Solid Flight Auger **DATUM:** Mean Sea Level

SAMPLER TYPE:	DRIVE WEIGHT - LB	HEIGHT OF FALL - IN
2.5 inch I.D. Split Barrel	140	30
Standard Penetration Test	140	30

BLOWS PER FT.	MOISTURE CONTENT %	DRY UNIT WEIGHT p.c.f.	DEPTH IN FEET	USCS CLASSI-FICATION	DESCRIPTION
				CL	SANDY CLAY, brown, moist, stiff, fine to medium-grained sand
35	16.2	114		SC	CLAYEY SAND, mottled orange-brown and gray, moist, dense, coarse-grained sand
				CL	SANDY CLAY, brown, moist, stiff to very stiff, fine-grained sand, iron oxide stains
50/6"	10.5	108	5	SC	CLAYEY SAND, tan-brown, dry to moist, very dense, fine to medium-grained sand, trace subrounded gravel up to 1/4-inch diameter, weakly cemented
				SC	CLAYEY SAND, mottled orange-brown and gray, moist, very dense, fine to coarse-grained sand, trace subrounded gravel up to 1/4-inch diameter
65	17.9	104		SP	SAND, orange-brown, moist, very dense, coarse-grained sand
18	-	-	10	SC	CLAYEY SAND, mottled orange-brown and gray, moist, medium dense, fine-grained sand
15	20.8	105	15	SC	CLAYEY SAND, orange-brown and gray, saturated, medium dense, coarse-grained sand
13	-	-			
50/6"	-	-			SANDSTONE, fine-grained, tan-brown, highly weathered, moderately strong, crushed
			20		Boring terminated at 19-1/2 feeet. Free water encounted at 16 feet, rose to 14-1/2 feet in 1 hour.

JOB NUMBER: 2616.100 **DATE DRILLED:** 4-3-03

JOB NAME: UOP Property **SURFACE ELEVATION:** 140 feet

DRILL RIG: Solid Flight Auger **DATUM:** Mean Sea Level

SAMPLER TYPE:	DRIVE WEIGHT - LB	HEIGHT OF FALL - IN
2.5 inch I.D. Split Barrel	140	30
Standard Penetration Test	140	30

BLOWS PER FT.	MOISTURE CONTENT %	DRY UNIT WEIGHT p.c.f.	DEPTH IN FEET	USCS CLASSI-FICATION	DESCRIPTION
				ML	CLAYEY SILT, gray-brown, moist, stiff
32	17.4	110		CL	SILTY CLAY, dark yellow-brown to brown, moist, stiff, trace well-rounded gravel up to 1/8-inch diameter
36	21.0	105	5	CL	SILTY CLAY, yellow-brown, moist, stiff, trace to some fine to medium-grained sand, trace subrounded to subangular gravel up to 3/4-inch diameter, faint iron oxide stains
				CL	SILTY CLAY, tan-brown, moist, stiff, trace to some fine-grained sand, trace subrounded gravel up to 1/4-inch diameter
58	16.3	110	10	CL	SANDY CLAY, mottled orange-brown and gray, moist, very stiff, medium-grained sand, trace to some subangular gravel up to 2-inch diameter
60/6"	-	-			SANDSTONE, medium to coarse-grained, green-gray to black, highly weathered, strong, crushed, 60° joints.
			15		Boring terminated at 14 feet. No free water encountered.
			20		

BORING LOG B-10

JOB NUMBER: 2616.100 **DATE DRILLED:** 4-3-03

JOB NAME: UOP Property **SURFACE ELEVATION:** 142 feet

DRILL RIG: Solid Flight Auger **DATUM:** Mean Sea Level

SAMPLER TYPE:	DRIVE WEIGHT - LB	HEIGHT OF FALL - IN
■ 2.5 inch I.D. Split Barrel	140	30
◪ Standard Penetration Test	140	30

BLOWS PER FT.	MOISTURE CONTENT %	DRY UNIT WEIGHT p.c.f.	DEPTH IN FEET	USCS CLASSI-FICATION	DESCRIPTION
				SM	SANDY SILT, brown, wet, medium dense, fine-grained sand
23	19.6	108		CL	SILTY CLAY, dark yellow-brown, moist, stiff, trace coarse-grained sand, iron oxide stains
37	20.8	102		CL	SILTY CLAY, mottled orange-brown and tan-brown and gray, moist, stiff, some fine-grained sand, trace well rounded gravel up to 1/8-inch diameter, iron oxide stains
			5		SANDSTONE, fine to medium-grained, tan-brown, highly weathered, weak to moderately strong, highly fractured
50/6"	-	-			SHALE, black, highly weathered, strong, crushed
			10		Boring terminated at 9-1/2 feet. No free water encountered.
			15		
			20		

BORING LOG B-11

JOB NUMBER: 2616.100 **DATE DRILLED:** 4-3-03

JOB NAME: UOP Property **SURFACE ELEVATION:** 133 feet

DRILL RIG: Solid Flight Auger **DATUM:** Mean Sea Level

SAMPLER TYPE:	DRIVE WEIGHT - LB	HEIGHT OF FALL - IN
■ 2.5 inch I.D. Split Barrel	140	30
⧄ Standard Penetration Test	140	30

BLOWS PER FT.	MOISTURE CONTENT %	DRY UNIT WEIGHT p.c.f.	DEPTH IN FEET	USCS CLASSI-FICATION	DESCRIPTION
				CL	SANDY CLAY, moist, stiff, fine-grained sand
41	19.2	108		CL	SILTY CLAY, mottled tan-brown and gray, moist, stiff, trace fine-grained sand
				CL	SILTY CLAY, mottled brown and orange-brown, moist, very stiff to hard, trace coarse-grained sand, trace subrounded gravel up to 1/4-inch diameter at 4-1/2 feet, approximately 6-inch diameter sandstone cobble
50/6"	-	-	5		
				CL	SANDY CLAY, mottled orange-brown and gray, moist, hard, fine-grained sand
60	-	-			
			10		
50/6"	-	-			SHALE, gray to orange-brown, highly weathered, moterately strong to strong, trace clay
65/6"	-	-			
			15		Broing terminated at 14 feet. No free water encountered.
			20		

A-16

BORING LOG B-12

JOB NUMBER: 2616.100 **DATE DRILLED:** 4-3-03

JOB NAME: UOP Property **SURFACE ELEVATION:** 124 feet

DRILL RIG: Solid Flight Auger **DATUM:** Mean Sea Level

SAMPLER TYPE:	DRIVE WEIGHT - LB	HEIGHT OF FALL - IN
2.5 inch I.D. Split Barrel	140	30
Standard Penetration Test	140	30

BLOWS PER FT.	MOISTURE CONTENT %	DRY UNIT WEIGHT p.c.f.	DEPTH IN FEET	USCS CLASSI-FICATION	DESCRIPTION
23	19.2	109		CL	SANDY CLAY, dark yellow-brown, moist, stiff, fine-grained, some silt, iron oxide stains
42	16.7	113	5	CL	SILTY CLAY, dark yellow-brown to brown, moist, very stiff, trace to some subangular gravel up to 1/2-inch diameter, trace fine to medium-grained sand
42	16.7	109	10		
45	18.5	110	15	CL	SANDY CLAY, mottled orange-brown and gray-brown, moist, hard, medium-grained sand, trace subangular to subrounded gravel up to 1/4-inch diameter
65/6"	-	-			SHALE, gray, highly weathered, strong, 70° fractures.
			20		Boring terminated at 18-1/2 feet. Free water encountered at 17-1/2 feet.

BORING LOG B-13

JOB NUMBER: 2616.100 **DATE DRILLED:** 4-3-03

JOB NAME: UOP Property **SURFACE ELEVATION:** 123 feet

DRILL RIG: Solid Flight Auger **DATUM:** Mean Sea Level

SAMPLER TYPE:	DRIVE WEIGHT - LB	HEIGHT OF FALL - IN
2.5 inch I.D. Split Barrel	140	30
Standard Penetration Test	140	30

BLOWS PER FT.	MOISTURE CONTENT %	DRY UNIT WEIGHT p.c.f.	DEPTH IN FEET	USCS CLASSI-FICATION	DESCRIPTION
				SM	SANDY SILT, brown, dry, medium dense, fine-grained sand
29	17.9	104		CL	SILTY CLAY, dark gray-brown, moist, stiff, trace coarse-grained sand, iron oxide stains
				CL	SANDY CLAY, tan-brown to brown, moist, stiff, coarse-grained sand
28	18.7	104	5	SC/ CL	CLAYEY SAND / SANDY CLAY, tan-brown to brown, moist, medium dense to stiff, fine to medium-grained sand, trace well rounded gravel up to 1/8-inch diameter, iron oxide stains
				SC	CLAYEY SAND, mottled orange-brown and gray, moist, medium dense, medium-grained sand, trace subrounded to well rounded gravel up to 1/4-inch diameter
30	17.1	108	10		
15	-	-			
20	20.1	107.3	15	SM	SILTY SAND, mottled gray and orange-grown, moist, medium dense, coarse-grained sand, some clay
17	-	-			
50/6"	11.1	125	20		SANDSTONE, fine-grained, gray, highly weathered, highly fractured, strong, 60° joints

BORING LOG B-13

JOB NUMBER: 2616.100 SHEET: 2 OF: 2

JOB NAME: UOP Property DEPTH: 20 feet TO 21 feet

NOTES:

BLOWS PER FT.	MOISTURE CONTENT %	DRY UNIT WEIGHT p.c.f.	DEPTH IN FEET	USCS CLASSI-FICATION	DESCRIPTION
50/6"	11.1	125			SANDSTONE, fine-grained, gray, highly weathered, highly fractured, strong, 60° joints
					Boring terminated at 21 feet. Free water encountered at 9-1/2 feet, dropped to 17 feet in 2 hours
			25		
			30		
			35		
			40		

BORING LOG B-14

JOB NUMBER: 2616.100 **DATE DRILLED:** 4-3-03

JOB NAME: UOP Property **SURFACE ELEVATION:** 170 feet

DRILL RIG: Solid Flight Auger **DATUM:** Mean Sea Level

SAMPLER TYPE:	DRIVE WEIGHT - LB	HEIGHT OF FALL - IN
2.5 inch I.D. Split Barrel	140	30

BLOWS PER FT.	MOISTURE CONTENT %	DRY UNIT WEIGHT p.c.f.	DEPTH IN FEET	USCS CLASSIFICATION	DESCRIPTION
19	19.8	100		CL	SILTY CLAY, dark, gray-brown, moist, stiff, trace subrounded gravel up to 1/4-inch diameter
20	19.6	107	5	CL	SILTY CLAY, dark brown, moist, stiff, trace well rounded gravel up to 1/4-inch diameter, trace medium to coarse-grained sand
30	14.4	117	10	CL	SILTY CLAY, yellow-brown, moist, stiff to very stiff, some coarse-grained sand, trace subrounded gravel up to 1/2-inch diameter
50/6"	-	-	15		at 15 feet, sandstone boulder approximately18 inches in diameter
50/3"	-	-			SANDSTONE, fine-grained, gray, highly weathered, strong, highly fractured
			20		Boring terminated at 19-1/2 feet. No free water encountered.

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

UNIFIED SOIL CLASSIFICATION SYSTEM

Blows per ft.	Moisture Content (%)	Dry Unit Weight (pcf)	Depth in Feet	USCS Classifi-cation	
					Bulk Sample
					2.5" I.D. Split Barrel Sample
					2.8" I.D. Shelby Tube Sample
					No sample recovered
					Standard Penetration Test interval
					Well defined stratum change
					Gradual stratum change
					Interpreted stratum change
					Apparent ground water level at date noted. Seasonal weather conditions, site topography, etc., may cause changes in water level indicated on logs.

Note: Soils described as dry, moist, and wet are estimated to be dry of optimum, near optimum, and wet of optimum moisture content, respectively. Saturated soils are estimated to be within areas of free groundwater.

KEY TO BORING LOG SYMBOLS

ROCK DESCRIPTION

ROCK TYPE
GRAIN SIZE (if Applicable)
COLOR
WEATHERING

Highly - Moderate to complete mineral decomposition, extensive disintegration, deep and through discoloration, fractures extensively coated or filled with oxides, carbonates and/or silt and clay.

Moderately - Slight change or partial decomposition of minerals, little disintegration, cementation little to unaffected, moderate to occasionally intense discoloration, moderately coated fractures.

Slightly - No megascopic decompositon of minerals, little to no effect on cementation, slight and intermittent or localized discoloration, few stains on fracture surfaces.

Unweathered - Unaffected by weathering agents, no discoloration or disintegration.

STRENGTH

Friable - Crumbles easily with fingers

Waek - Crumbles under light hammer blows

Moderately Strong - Specimen will withstand a few hammer blows before breaking

Strong - Specimen will withstand a few eavy ringing hammer blows before breaking into large fragments

Very Strong - Specimen will resist heavy ringing hammer blows and will yield with difficulty only dust and small flying fragments

FRACTURING - Intensity, coating or filling, attitude(s)

Intensity	Size of Pieces
Occasionally Fractured	Greater than 12 inches
Moderately Fractured	6 inches to 12 inches
Highly Fractured	1/2 inch to 6 inches
Crushed	Less than 1/2 inch

BEDING - Stratification, Attitude

Stratification	Thickness
Very Thickly Bedded	Greater than 4 feet
Thickly Bedded	2 to 4 feet
Thinly Bedded	1 inch to 2 feet
Thinly Laminated	Less than 1 inch

MISCELLANEOUS - Shearing of rock, veins, caliche, etc.

Source: Modified from Civil Engineers Reference Book (Blake, 1975)

JOB NUMBER: 2670.100 DATE: 4-23-03 BY: FF

TEST PIT LOGS

Test Pit Number	Depth (feet)	Description
TP2-1	0-1	Sandy Clay, light brown, moist, stiff, fine-grained sand, some silt, rootlets.
	1-3	Silty Sand, orange-brown, moist, very dense, coarse-grained sand, some clay.
	3-6	Sandstone, coarse-grained, tan-brown, highly weathered, moderately strong, highly fractured with manganese oxide on surfaces.
		Total Depth 6 feet No free ground water encountered
TP2-2	0-2	Sandy Clay, light brown, moist, stiff, fine-grained sand, some silt, rootlets.
	2-6	Sandstone, medium-grained, gray and red-brown, moderately weathered, very strong, highly fractured. Joints N30W 40S.
		Total Depth 6 feet No free ground water encountered
TP2-3	0-3	Sandy Clay, light brown, moist, stiff, fine-grained sand.
	3-5	Clayey Sand, mottled orange-brown and gray, moist, dense to very dense, medium-grained sand, trace subangular sandstone clasts.
	5-7	Sandstone, coarse-grained, orange-brown, moderately weathered, strong to very strong, highly fractured to crushed.
		Total Depth 7 feet No free ground water encountered
TP2-4	0-3½	Sandstone, fine- to medium-grained, tan-brown, moderately weathered, very strong, highly fractured. Joints N10E 45N, N45E vertical.
		Total Depth 3½ feet No free ground water encountered

TEST PIT LOGS

Test Pit Number	Depth (feet)	Description
TP2-5	0-3½	Silty Clay, dark gray-brown, moist, stiff, trace fine-grained sand, subrounded sandstone cobbles from 2 to 3 feet.
	3½-4½	Clayey Sand, orange-brown, moist, dense, trace subangular sandstone clasts.
	4½-6½	Sandstone, coarse-grained, orange-brown, highly weathered, strong, crushed.
		Total Depth 6½ feet No free ground water encountered
TP2-6	0-2	Clayey Silt, brown, moist, stiff, trace fine-grained sand.
	2-6	Sandy Clay, mottled brown and orange-brown, moist, very stiff, some well-rounded gravel, coarse-grained sand.
		Total Depth 6 feet No free ground water encountered
TP2-7	0-2	Sandy Clay, light brown, moist, stiff, fine-grained sand.
	2-2¼	Sandy Clay, light brown, saturated, medium stiff, fine-grained sand.
	2¼-4	Sandy Clay, mottled brown and orange-brown, moist, very stiff, coarse-grained sand, trace to some well rounded gravel.
		Total Depth 4 feet Ground water encountered at 2 feet
TP2-8	0-4	Sandy Clay, light brown, moist, stiff, fine-grained sand.
	4-6	Sandy Clay, orange-brown, moist, very stiff, trace angular sandstone clasts.
	6-8	Sandstone, coarse-grained, orange-brown, highly weathered, strong, highly fractured. Joints N30W 60SW.
		Total Depth 8 feet No free ground water encountered

TEST PIT LOGS

Test Pit Number	Depth (feet)	Description
TP2-9	0-3	Silty Clay, brown, moist, stiff, trace fine-grained sand.
	3-6	Sandy Clay, mottled brown and orange-brown, moist, stiff to very stiff, medium grained sand, trace well-rounded gravel.
	6-8	Shale, gray, highly weathered, weak to moderately strong, crushed.
		Total Depth 8 feet No free ground water encountered
TP2-10	0-½	Sandy Silt, brown, moist, stiff, fine-grained sand.
	½-5	Sandstone, coarse-grained, orange-brown, highly weathered, strong, highly fractured. Joints N25E 65N, N70W 30S.
		Total Depth 5 feet No free ground water encountered
TP2-11	0-3½	Silty Clay, dark brown, moist, stiff, trace fine-grained sand.
	3½-7	Shale, gray to orange-brown, highly weathered, weak to moderately strong, highly fractured, thinly laminated. Bedding N20W 73SW.
		Total Depth 7 feet No free ground water encountered
TP2-12	0-2½	Silty Clay, dark brown, moist, stiff, trace gravel.
	2½-7	Shale, gray, highly weathered, weak, crushed, some clay.
		Total Depth 7 feet No free ground water encountered
TP2-13	0-3	Silty Clay, dark brown, moist, stiff.
	3-6	Silty Clay, brown, moist, stiff, trace subangular gravel.
	6-9	Shale, gray, highly weathered, crushed, some clay.
		Total Depth 9 feet No free ground water encountered

TEST PIT LOGS

Test Pit Number	Depth (feet)	Description
TP2-14	0-3	Silty Clay, dark brown, moist, stiff.
	3-8	Sandstone, fine-grained, gray, highly weathered, crushed, moderately strong. Joints N70W 70N, N30W 85SW.
		Total Depth 8 feet No free ground water encountered
TP2-15	0-4	Silty Clay, dark brown, moist, stiff, trace gravel.
	4-8	Sandstone, fine-grained, gray, highly weathered, moderately strong, crushed. Possible bedding N24W 73SW. Joint N85W 20N.
		Total Depth 8 feet No free ground water encountered
TP2-16	0-4	Silty Clay, dark gray-brown, moist, stiff.
	4-6	Silty Clay, brown, moist, stiff, trace coarse-grained sand, trace subangular gravel up to ½-inch diameter, fairly sharp basal contact with faint horizontal clay film.
	6-13	Clayey Sand/Sandy Clay, orange-brown, moist, very dense, coarse-grained sand, trace well-rounded gravel up to ½-inch diameter.
	13-15	Shale, gray, highly weathered, weak to moderately strong, crushed.
		Total Depth 15 feet No free ground water encountered
TP2-17	0-3½	Silty Clay, dark gray-brown, moist, stiff.
	3½-5	Silty Clay, brown, moist, stiff, trace subangular gravel up to ½-inch diameter, sharp basal contact with 1/16-inch clay. N20E 12SE.
	5-8	Clayey Sand, orange-brown, moist, very dense, coarse-grained sand, trace well-rounded gravel up to ½-inch diameter.
		Total Depth 8 feet No free ground water encountered

TEST PIT LOGS

Test Pit Number	Depth (feet)	Description
TP2-18	0-3½	Silty Clay, dark gray-brown, moist, stiff, trace fine-grained sand.
	3½-7	Sandstone, fine-grained, tan-brown, highly weathered, strong, crushed.
		Total Depth 7 feet No free ground water encountered
TP2-19	0-1	Sandy Silt, light brown, dry to moist, fine-grained sand, trace clay.
	1-9	Clayey Sand, mottled orange-brown and gray, moist, very dense, coarse-grained sand.
	9-12	Sandstone, medium-grained, tan-brown, highly weathered, moderately strong, crushed.
		Total Depth 12 feet No free ground water encountered
TP2-20	0-3	Silty Sand, brown, moist, stiff, fine-grained sand, some clay.
	3-9	Silty Sand, orange-brown and gray, moist, very dense, fine-grained sand, trace well-grounded gravel up to ½-inch diameter, trace clay.
	9-14	Clayey Sand, mottled orange-brown and gray, moist to wet, dense to very dense, coarse-grained sand, some subrounded gravel up to 1-inch diameter.
	14-15	Shale, gray, highly weathered, weak, crushed.
		Total Depth 15 feet Free ground water encountered at 15 feet
TP2-21	0-1	Sandy Silt, brown, dry to moist, stiff, fine-grained sand, some clay.
	1-5	Sandstone, fine-grained, tan-brown, highly weathered, strong, highly fractured, bedded with shale, black, highly weathered, weak to moderately strong, crushed with some clay, disrupted structure. Bedding N62W 87S.
		Total Depth 5 feet No free ground water encountered

TEST PIT LOGS

Test Pit Number	Depth (feet)	Description
TP2-22	0-1½	Sandy Clay, brown, moist, stiff, fine-grained sand, some silt.
	1½-5	Shale, gray to orange-brown, highly weathered, weak to moderately strong, crushed with trace to some clay, disrupted structure.
		Total Depth 5 feet No free ground water encountered
TP2-23	0-2	Sandy Clay, brown, moist, stiff, fine-grained sand, faint blocky ped structure.
	2-4	Sandstone, medium-grained, tan-brown, highly weathered, highly fractured, trace clay on fracture surfaces.
		Total Depth 4 feet No free ground water encountered
TP2-24	0-4½	Silty Clay, dark gray-brown, moist, stiff, trace well-rounded gravel up to ¼-inch diameter.
	4½-6½	Silty Clay, dark brown, moist, stiff to very stiff, trace well-rounded gravel up to ¼-inch diameter, trace medium- to coarse-grained sand.
	6½-11½	Silty Clay, brown, moist, stiff to very stiff, some coarse-grained sand, trace subrounded to rounded gravel up to ½-inch diameter.
		Total Depth 11½ feet No free ground water encountered
TP2-25	0-2	Sandy Clay, brown, moist, stiff, fine-grained sand, trace subrounded cobbles up to 2-inches diameter.
	2-6	Shale, black to orange-brown, highly weathered, moderately strong, crushed with some clay, fairly undulating, disrupted structure N70W 70-85SW.
		Total Depth 6 feet No free ground water encountered

TEST PIT LOGS

Test Pit Number	Depth (feet)	Description
TP2-26	0-2	Sandy Clay, brown, moist, stiff, fine-grained sand.
	2-6	Shale, black to orange-brown, highly weathered, weak, crushed with some clay, thinly bedded with sandstone, fine-grained, tan-brown, moderately strong, crushed. Bedding N20E 65S.
TP2-27	0-3	Sandy Clay, brown to dark yellow-brown, moist, stiff, fine-grained sand, some silt, iron oxide stains.
	3-14	Silty Clay, dark yellow-brown to brown, moist, very stiff, trace to some subangular to subrounded gravel up to ½-inch, angular friable sandstone clasts, trace fine- to medium-grained sand (Qls).
		Total Depth 14 feet No free ground water encountered
TP2-28	0-1½	Sandy Clay, brown, moist, stiff, fine-grained sand.
	1½-3½	Cobbles and orange-brown Clay (matrix), moist, dense, clast supported.
	3½-6	Sandy Clay, mottled orange-brown and gray, moist, very stiff, coarse-grained sand, trace subangular gravel up to ½-inch diameter, sharp basal contact with ⅛-inch orange-brown clay. N50E 18SE.
	6-8	Sandstone, fine-grained, tan-brown, highly weathered, weak to moderately strong, crushed with trace clay.
		Total Depth 8 feet No free ground water encountered
TP2-29	0-2½	Sandy Clay, brown, moist, stiff, fine-grained sand.
	2½-8	Silty Clay, mottled tan-brown and gray, moist, stiff to very stiff, trace fine-grained sand, trace well-rounded gravel up to ¼-inch diameter, gradational roughly horizontal basal contact over approximately 8 inches.
	8-11	Sandstone, fine-grained, tan-brown, weak to strong, highly fractured to crushed.
		Total Depth 11 feet No free ground water encountered

TEST PIT LOGS

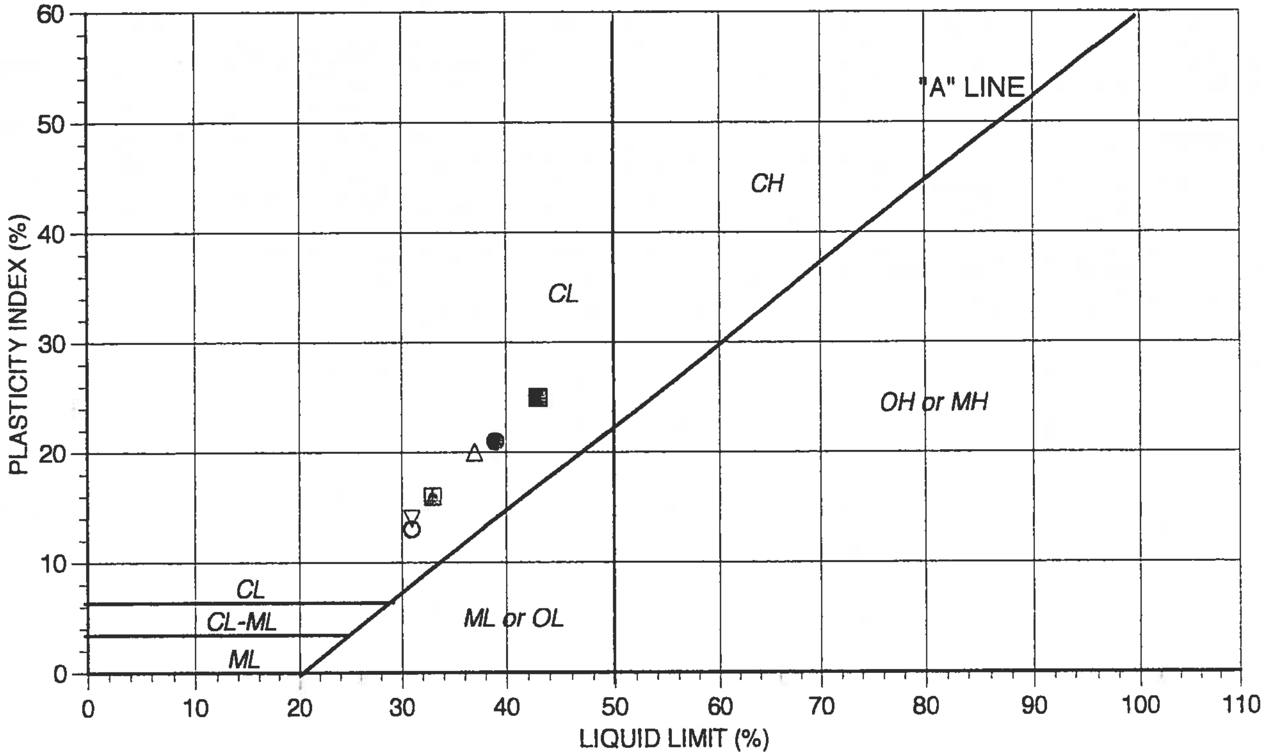
Test Pit Number	Depth (feet)	Description
TP2-30	0-1	Sandy Silt, brown, dry, stiff, fine-grained sand.
	1-2½	Silty Clay, dark gray-brown, moist, stiff.
	2½-6	Sandy Clay, tan-brown to brown, moist, stiff, coarse-grained sand.
	6-8	Clayey Sand, brown to gray, moist, dense to very dense, coarse-grained sand.
	8-14	Sandy Clay, mottled orange-brown and gray, moist, very stiff, coarse-grained sand.
		Total Depth 14 feet No free ground water encountered
TP2-31	0-1½	Sandy Clay, brown, moist, stiff, some rounded cobbles up to 4-inches diameter.
	1½-3	Sandstone, fine-grained, tan-brown, highly weathered, crushed.
	3-5	Sheared Clay/Shale, black, highly weathered, weak, inclusions of shale and sandstone, faint foliation parallel to contact, faint residual bedrock structure. Bedding N40W 33S.
		Total Depth 5 feet No free ground water encountered
TP2-32	0-1½	Sandy Clay, brown, moist, stiff, fine-grained sand, trace subangular cobbles up to 4-inches diameter.
	1½-5	Shale, gray to orange-brown, highly weathered, weak to moderately strong, crushed. Bedding N70W 63S.
		Total Depth 5 feet No free ground water encountered
TP2-33	0-2½	Silty Clay, brown, moist, stiff, trace fine-grained sand.
	2½-6	Shale, black, highly weathered, weak to moderately strong, crushed, thinly bedded. Bedding N60W 70S.
		Total Depth 6 feet No free ground water encountered

TEST PIT LOGS

Test Pit Number	Depth (feet)	Description
TP2-34	0-3	Silty Clay, dark brown, moist, stiff, trace well-rounded gravel up to ¼-inch diameter.
	3-5	Silty Clay, brown, moist, very stiff, gradational basal contact over approximately 10-inches.
	5-8	Shale, gray to black, highly weathered, weak to moderately strong, crushed with trace to some clay.
		Total Depth 8 feet No free ground water encountered
TP2-35	0-½	Sandy Silt and Gravel, tan-brown, dry, hard (Fill).
	½-1½	Sandy Clay, brown, moist, very stiff, fine-grained sand.
		Total Depth 1½ feet No free ground water encountered
TP2-36	0-1	Sandy Silt and Gravel, tan-brown, dry, hard (Fill).
	1-1½	Sandy Clay, brown, moist, very stiff, fine-grained sand.
		Total Depth 1½ feet No free ground water encountered

APPENDIX B

Laboratory Test Results

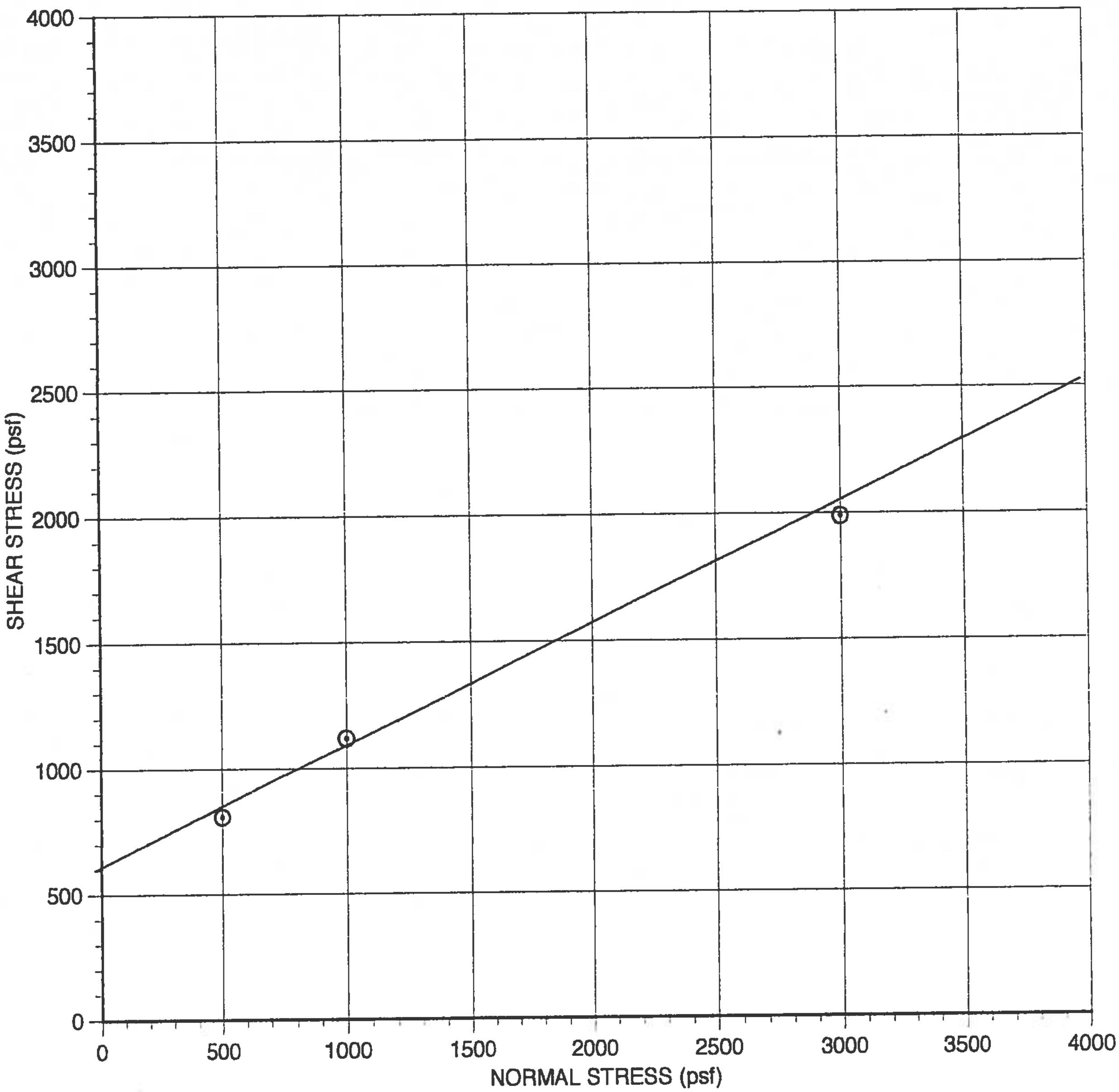


BY: PW DATE: 4-23-03 JOB NUMBER: 2070.100

SYMBOLS	LOCATION	LIQUID LIMIT	PLASTICITY INDEX	USCS CLASSIFICATION
●	B-1 at 1 foot	39	21	C L
□	B-3 at 5 feet	33	16	C L
▲	B-13 at 1 foot	33	16	C L
○	TP2-2 at 0 to 2 feet	31	13	C L
■	TP2-5 at 0-3 feet	43	25	C L
△	TP2-13 at 6-9 feet	37	20	C L
▽	TP2-24 at 6-1/2 feet	31	14	C L

ATTERBERG LIMITS TEST DATA

JOB NUMBER: 2616.100 DATE: 4-23-03 BY: PW



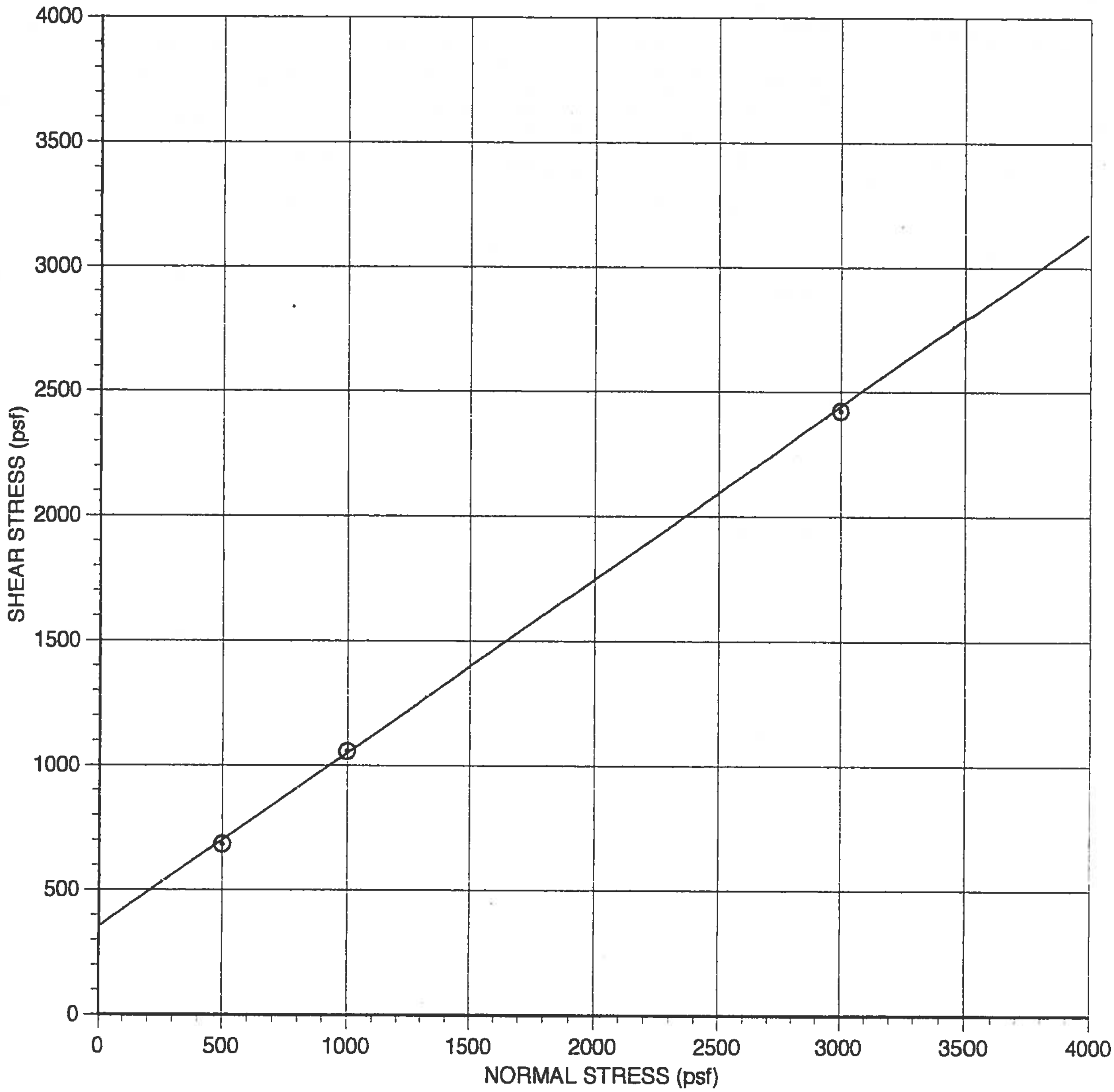
LOCATION: B-3 at 9-1/2 feet

SAMPLE: SANDY CLAY, mottled yellow brown olive gray

TEST TYPE:	Consolidated Drained
RATE OF SHEAR (in/min):	0.00099
FRICTION ANGLE:	25°
COHESION:	600 psf

SPECIMEN	A	B	C
DRY DENSITY (pcf)	107.6	105.6	106.0
INITIAL WATER CONTENT (%)	19.3	20.8	19.4
FINAL WATER CONTENT (%)	20.8	22.2	19.1
NORMAL STRESS (psf)	500	1000	3000
MAXIMUM SHEAR (psf)	808	1118	1988

DIRECT SHEAR TEST



LOCATION: B-5 at 9 feet

SAMPLE: SILTY CLAY, dark gray, trace sand

TEST TYPE: Consolidated Drained

RATE OF SHEAR (in/min): 0.00099

FRICTION ANGLE: 35°

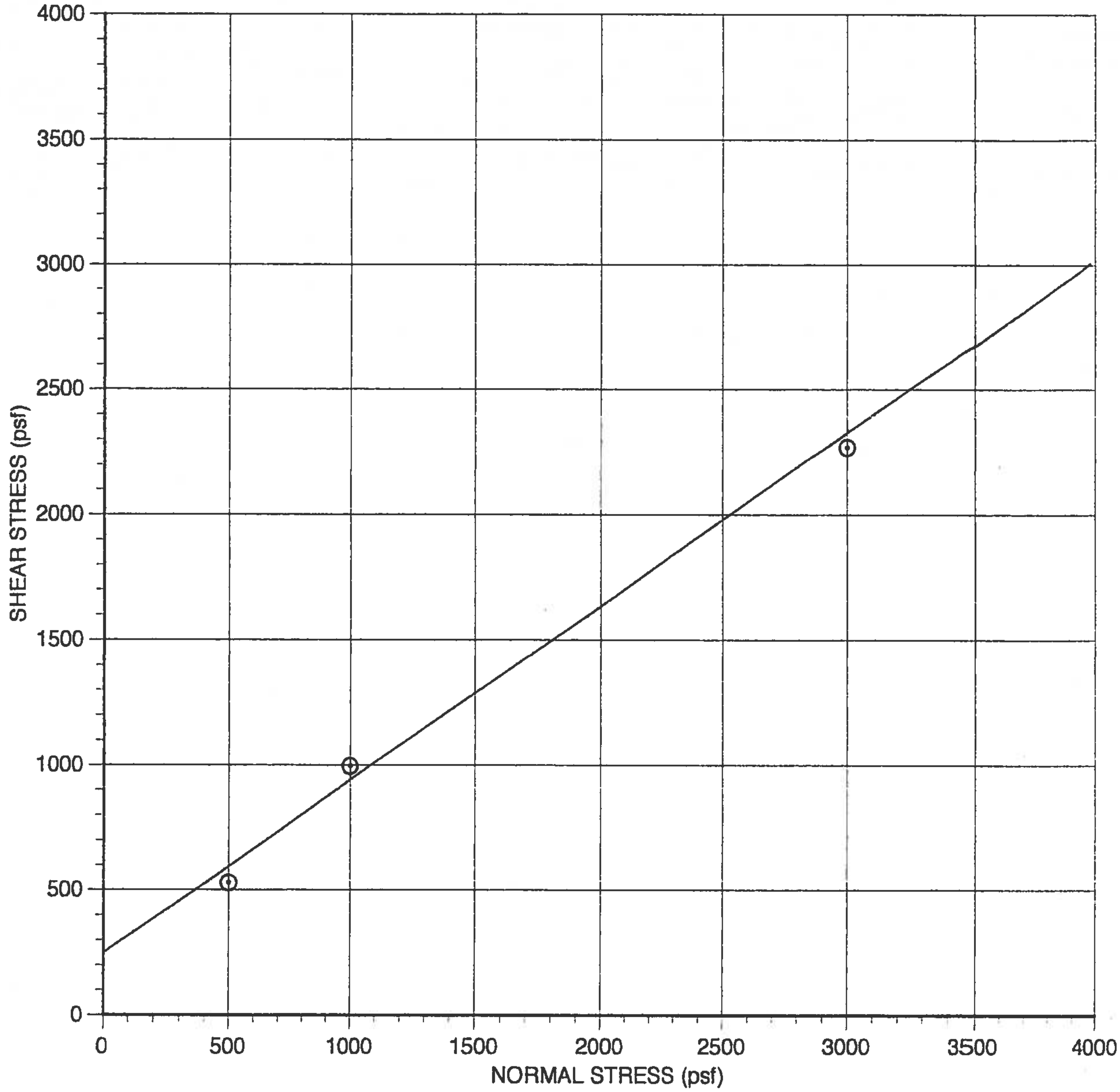
COHESION: 350 psf

SPECIMEN	A	B	C
DRY DENSITY (pcf)	107.8	108.7	108.1
INITIAL WATER CONTENT (%)	19.0	17.4	17.2
FINAL WATER CONTENT (%)	20.6	18.7	16.9
NORMAL STRESS (psf)	500	1000	3000
MAXIMUM SHEAR (psf)	683	1056	2423

DIRECT SHEAR TEST

JOB NUMBER: 2616.100 DATE: 4-23-03 BY: PW

JOB NUMBER: 2110.100 DATE: 4-25-03 BY: PW



LOCATION: B-14 at 5 feet

SAMPLE: SILTY CLAY with sand, brown

TEST TYPE: Consolidated Drained

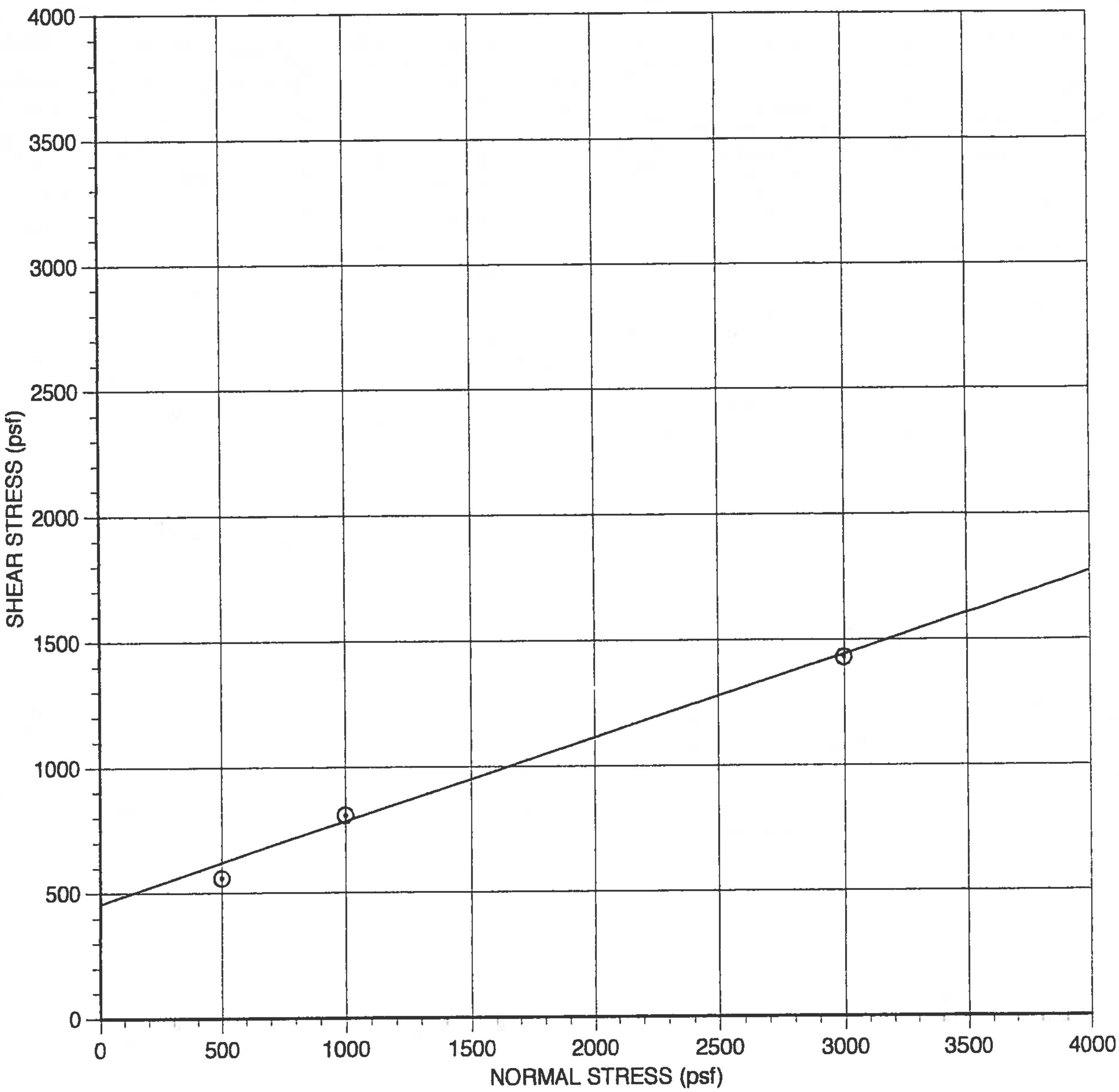
RATE OF SHEAR (in/min): 0.00099

FRICTION ANGLE: 34.5°

COHESION: 250 psf

SPECIMEN	A	B	C
DRY DENSITY (pcf)	106.3	106.7	107.2
INITIAL WATER CONTENT (%)	20.9	20.3	17.6
FINAL WATER CONTENT (%)	20.9	20.3	18.8
NORMAL STRESS (psf)	500	1000	3000
MAXIMUM SHEAR (psf)	528	994	2268

DIRECT SHEAR TEST



LOCATION: TP2-2 at 2 feet

SAMPLE: SANDY SILT CLAY, olive-brown, Remolded to 90% Relative Compaction

TEST TYPE: Consolidated Drained

RATE OF SHEAR (in/min): 0.00099

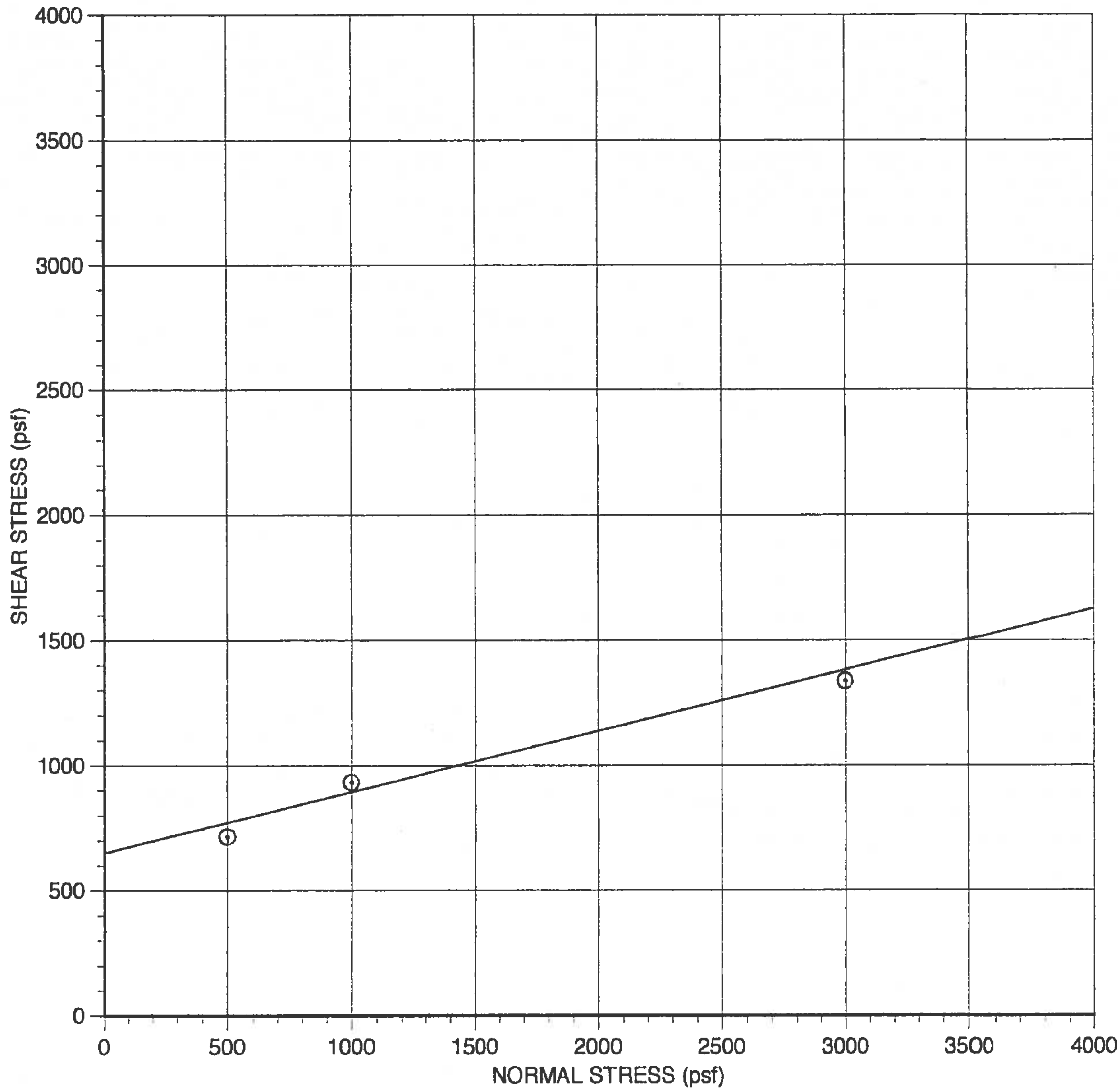
FRICTION ANGLE: 18°

COHESION: 450 psf

SPECIMEN	A	B	C
DRY DENSITY (pcf)	110.5	110.8	110.8
INITIAL WATER CONTENT (%)	14.9	14.9	15.0
FINAL WATER CONTENT (%)	18.4	16.0	15.4
NORMAL STRESS (psf)	500	1000	3000
MAXIMUM SHEAR (psf)	559	808	1429

DIRECT SHEAR TEST

JOB NUMBER: 2616.100 DATE: 4-23-03



LOCATION: TP2-13 at 6 to 9 feet

SAMPLE: SANDY CLAY, olive-brown with gray, Remolded to 90% Relative Compaction

TEST TYPE: Consolidated Drained

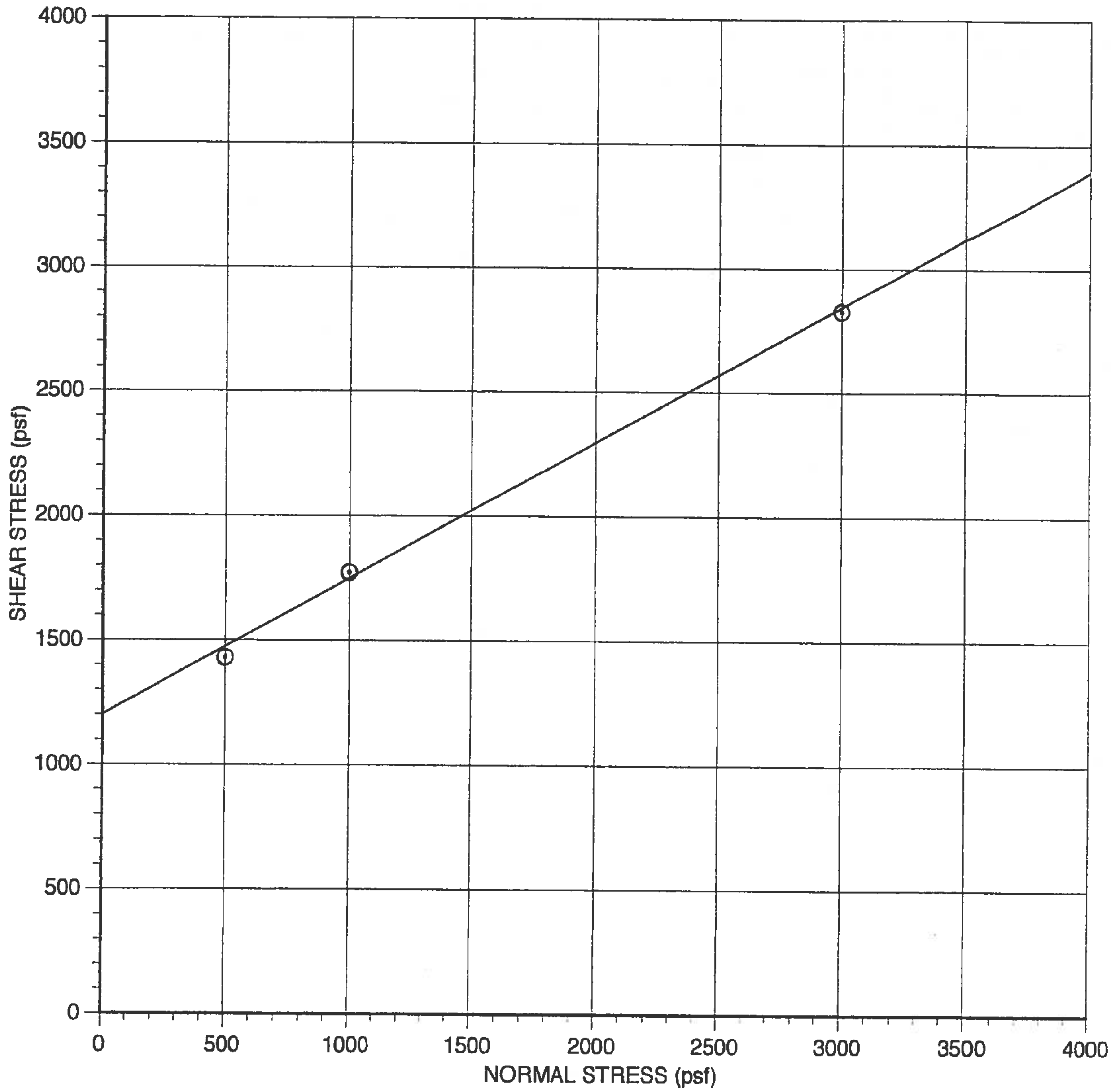
RATE OF SHEAR (in/min): 0.00099

FRICTION ANGLE: 14°

COHESION: 650 psf

SPECIMEN	A	B	C
DRY DENSITY (pcf)	120.2	119.0	120.3
INITIAL WATER CONTENT (%)	11.4	11.7	11.4
FINAL WATER CONTENT (%)	13.2	13.9	13.6
NORMAL STRESS (psf)	500	1000	3000
MAXIMUM SHEAR (psf)	715	932	1336

DIRECT SHEAR TEST



LOCATION: TP2-15 at 4 to 8 feet

SAMPLE: CLAYEY SANDSTONE, olive-gray, Remolded to 90% Relative Comaction

TEST TYPE: Consolidated Drained

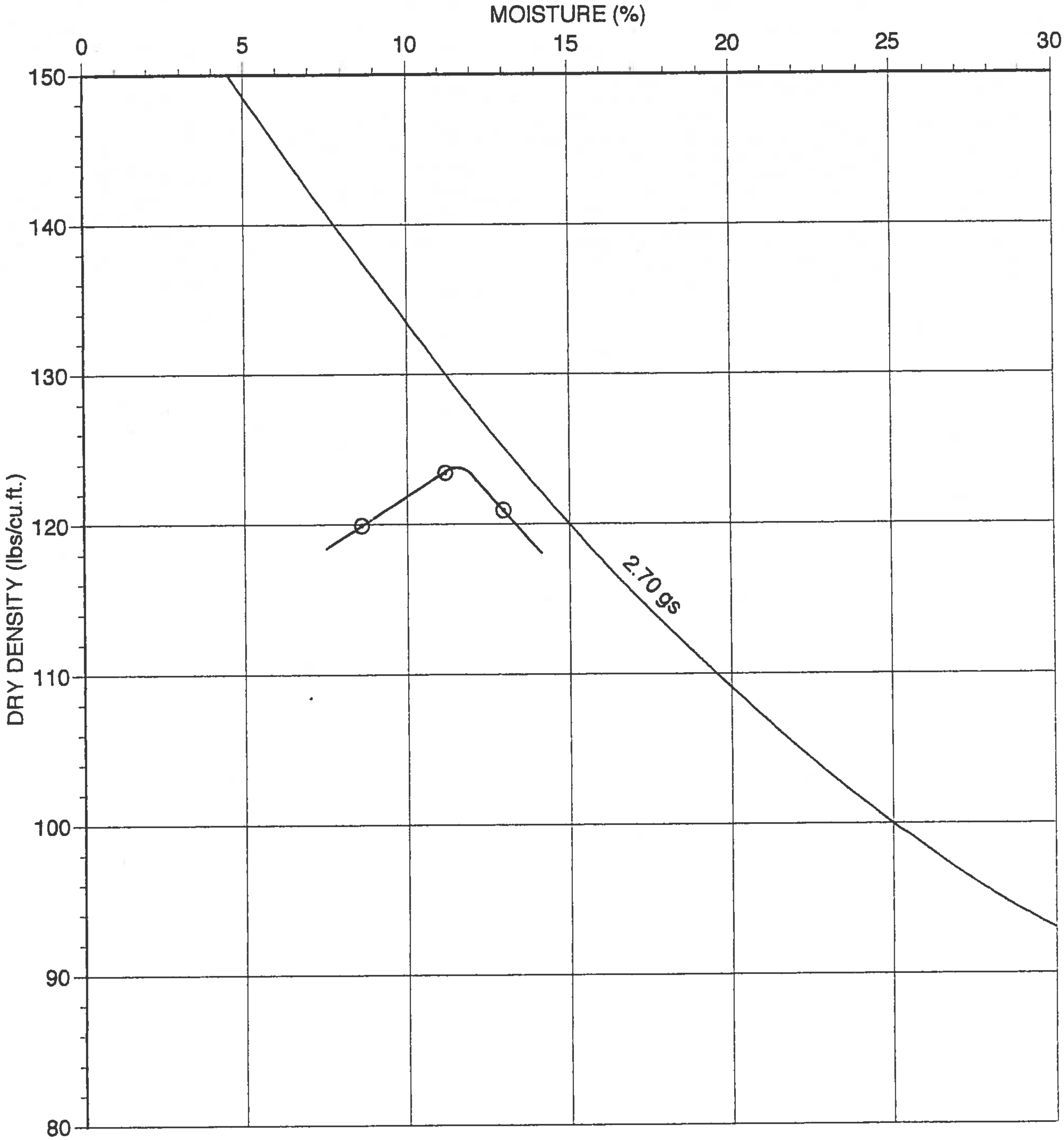
RATE OF SHEAR (in/min): 0.00099

FRICTION ANGLE: 29°

COHESION: 1200 psf

SPECIMEN	A	B	C
DRY DENSITY (pcf)	124.8	124.8	124.4
INITIAL WATER CONTENT (%)	9.5	9.7	9.8
FINAL WATER CONTENT (%)	10.3	11.0	10.5
NORMAL STRESS (psf)	500	1000	3000
MAXIMUM SHEAR (psf)	1429	1771	2827

DIRECT SHEAR TEST



SYMBOL	LOCATION	DESCRIPTION	OPTIMUM MOISTURE CONTENT (%)	MAX. DRY DENSITY (pcf)
⊙	TP2-2 at 0-2 feet	SILTY CLAY, brown	11.5	123.6

COMPACTION TEST DATA

JOB NUMBER: 2616.100

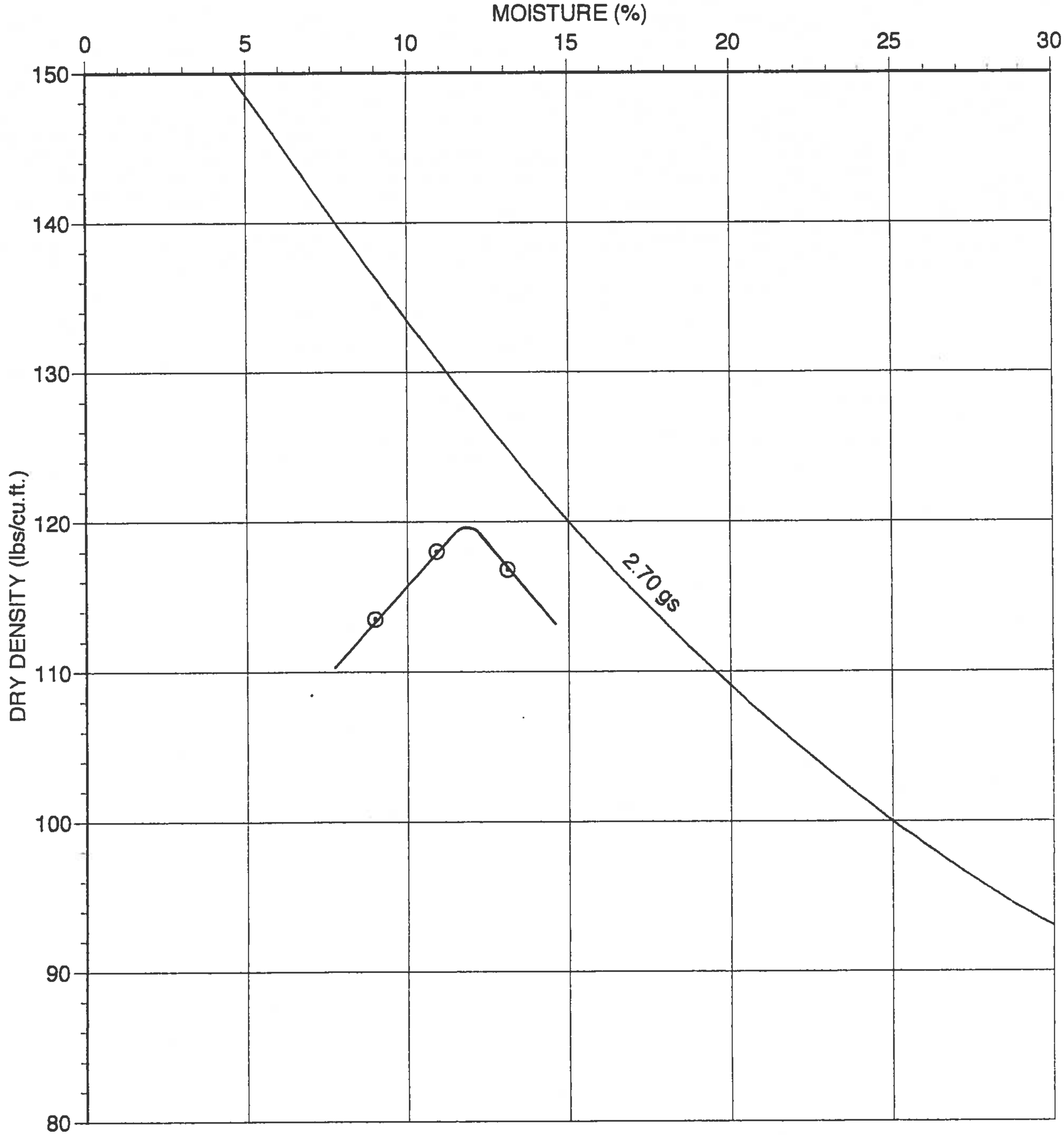
DATE: 4-23-03

BY: PW

JOB NUMBER: 2616.100

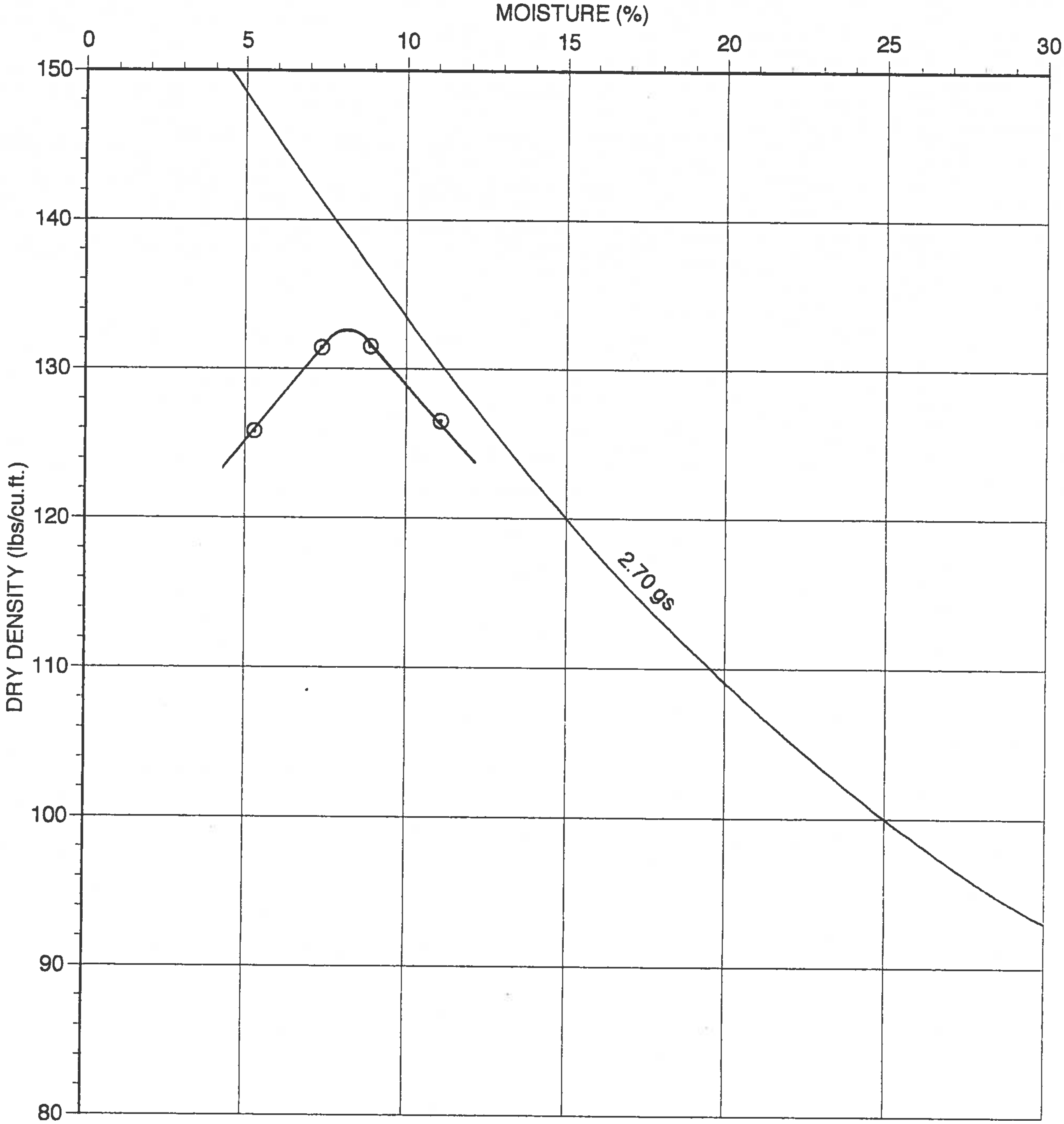
DATE: 4-23-03

BY: FW



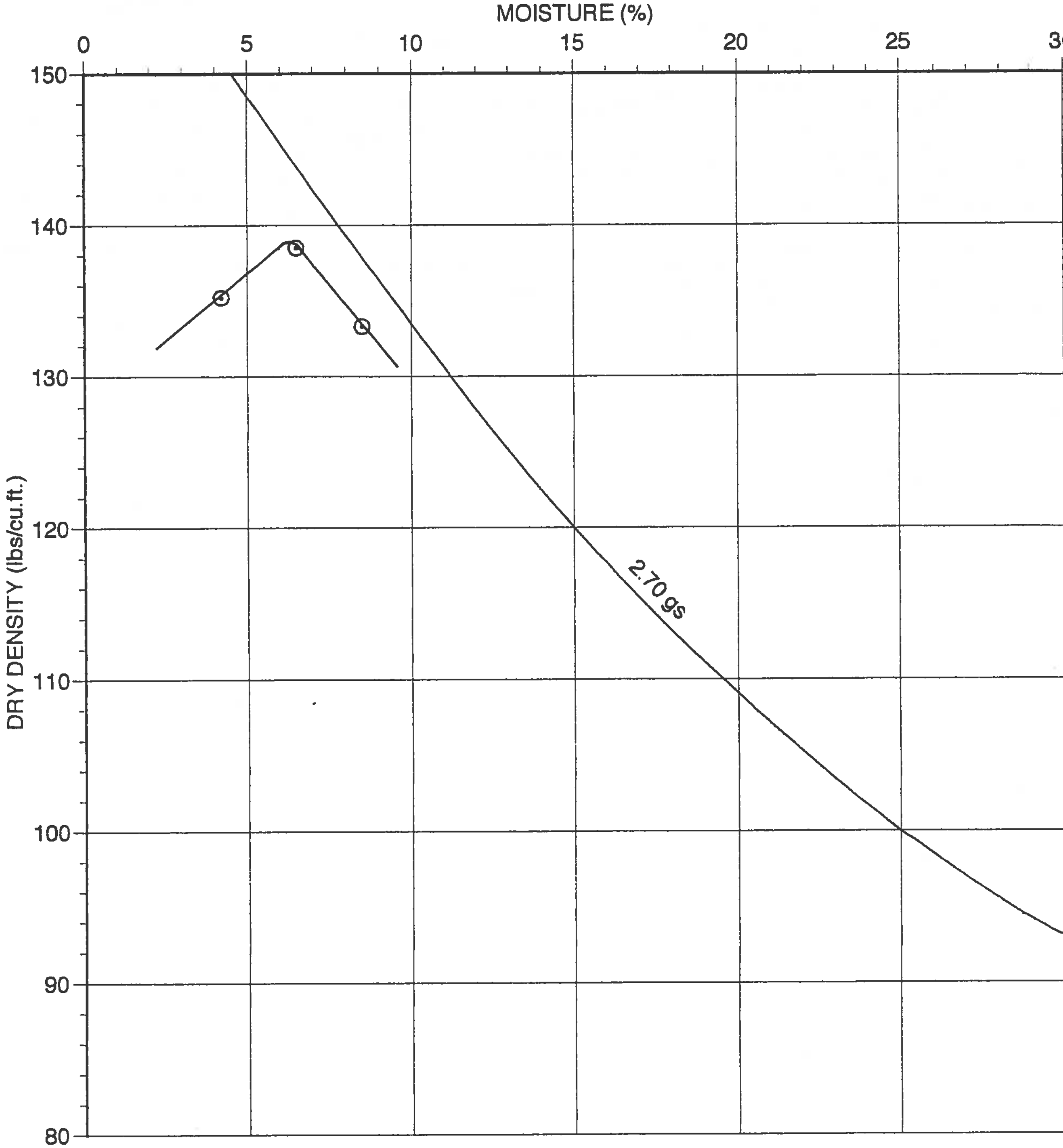
SYMBOL	LOCATION	DESCRIPTION	OPTIMUM MOISTURE CONTENT (%)	MAX. DRY DENSITY (pcf)
⊙	TP2-5 at 0-3-1/2 feet	SILTY CLAY, gray brown	11.8	119.5

COMPACTION TEST DATA



SYMBOL	LOCATION	DESCRIPTION	OPTIMUM MOISTURE CONTENT (%)	MAX. DRY DENSITY (pcf)
⊙	TP2-13 at 6 to 9 feet	CLAYEY SILTSTONE, gray-brown	8.2	132.2

COMPACTION TEST DATA



SYMBOL	LOCATION	DESCRIPTION	OPTIMUM MOISTURE CONTENT (%)	MAX. DRY DENSITY (pcf)
⊙	TP2-15 at 4 to 8 feet	SILTY CLAY, gray brown	6.2	138.7

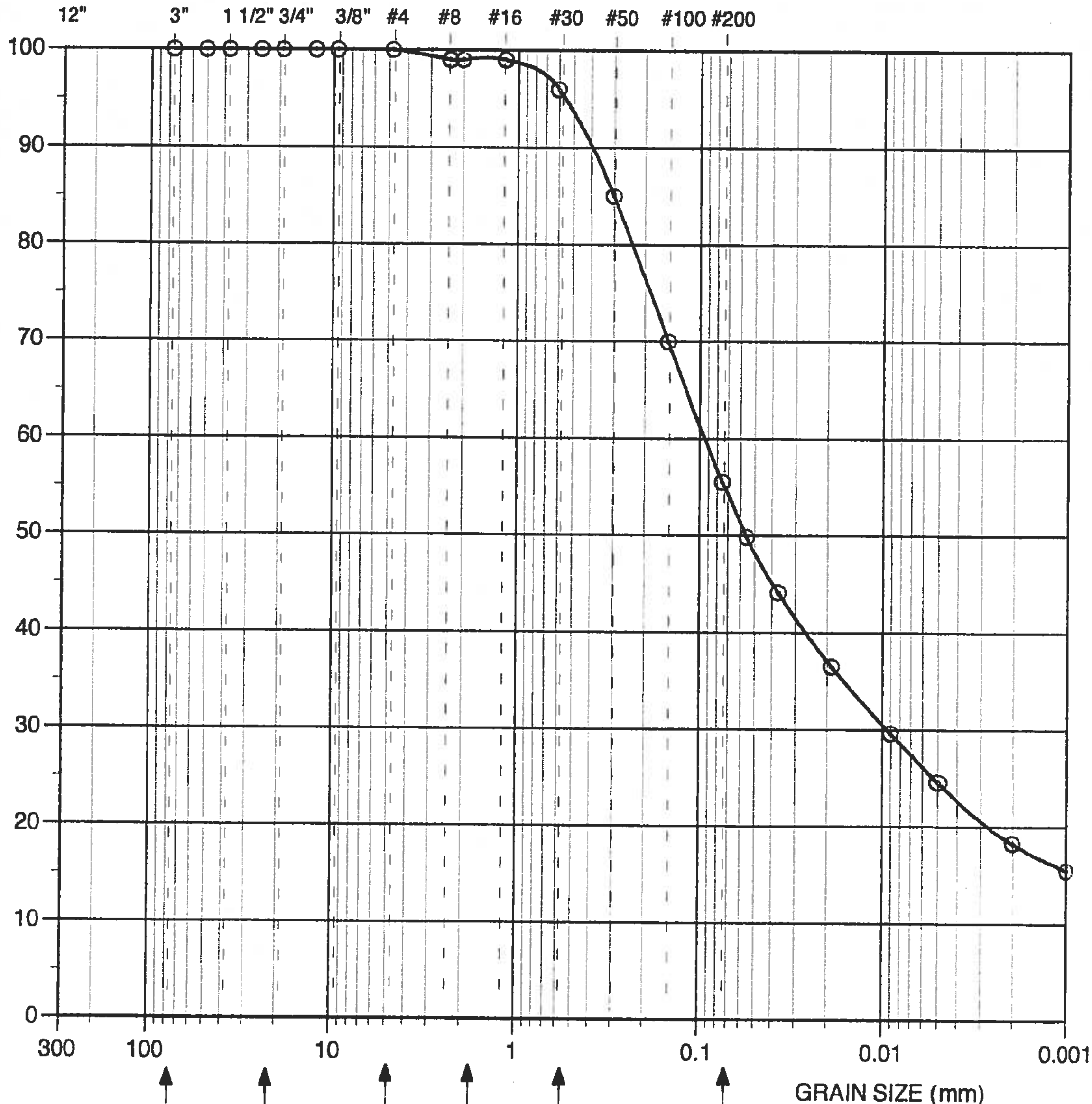
COMPACTION TEST DATA

BY: PW
DATE: 4-23-03
JOB NUMBER: 2616.100

BY: FF

DATE: 5-1-03

JOB NUMBER: 2616.100

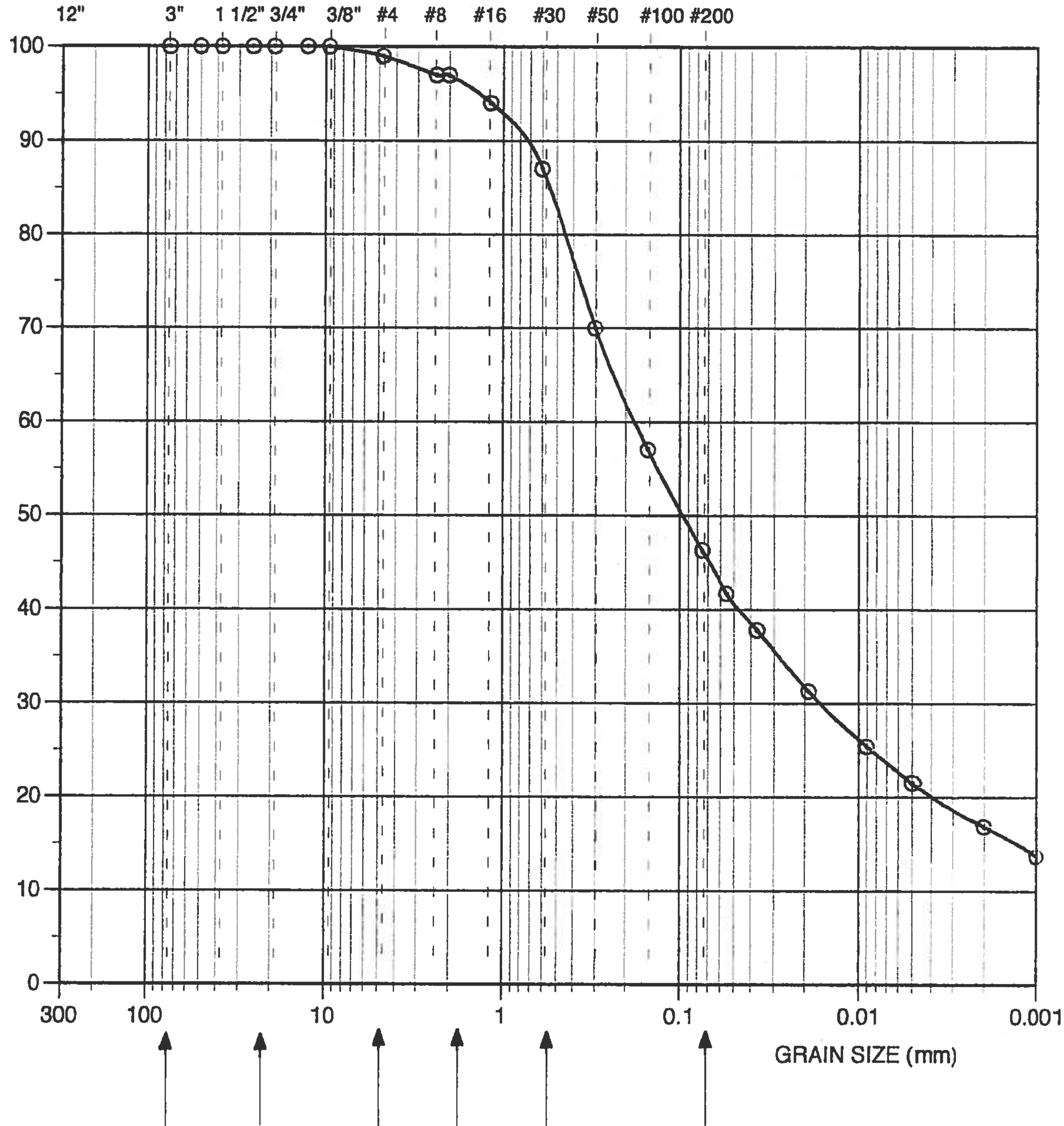


COBBLES	GRAVEL		SAND			SILT/CLAY
	coarse	fine	coarse	medium	fine	

SYMBOLS	LOCATION	DESCRIPTION
⊙	B-8 at 10-1/2 to 12 feet	SANDY SILT, light brown with clay

GRADATION TEST DATA

JOB NUMBER: 2616.100 DATE: 5-1-03 BY: FF



COBBLES	GRAVEL		SAND			SILT/CLAY
	coarse	fine	coarse	medium	fine	

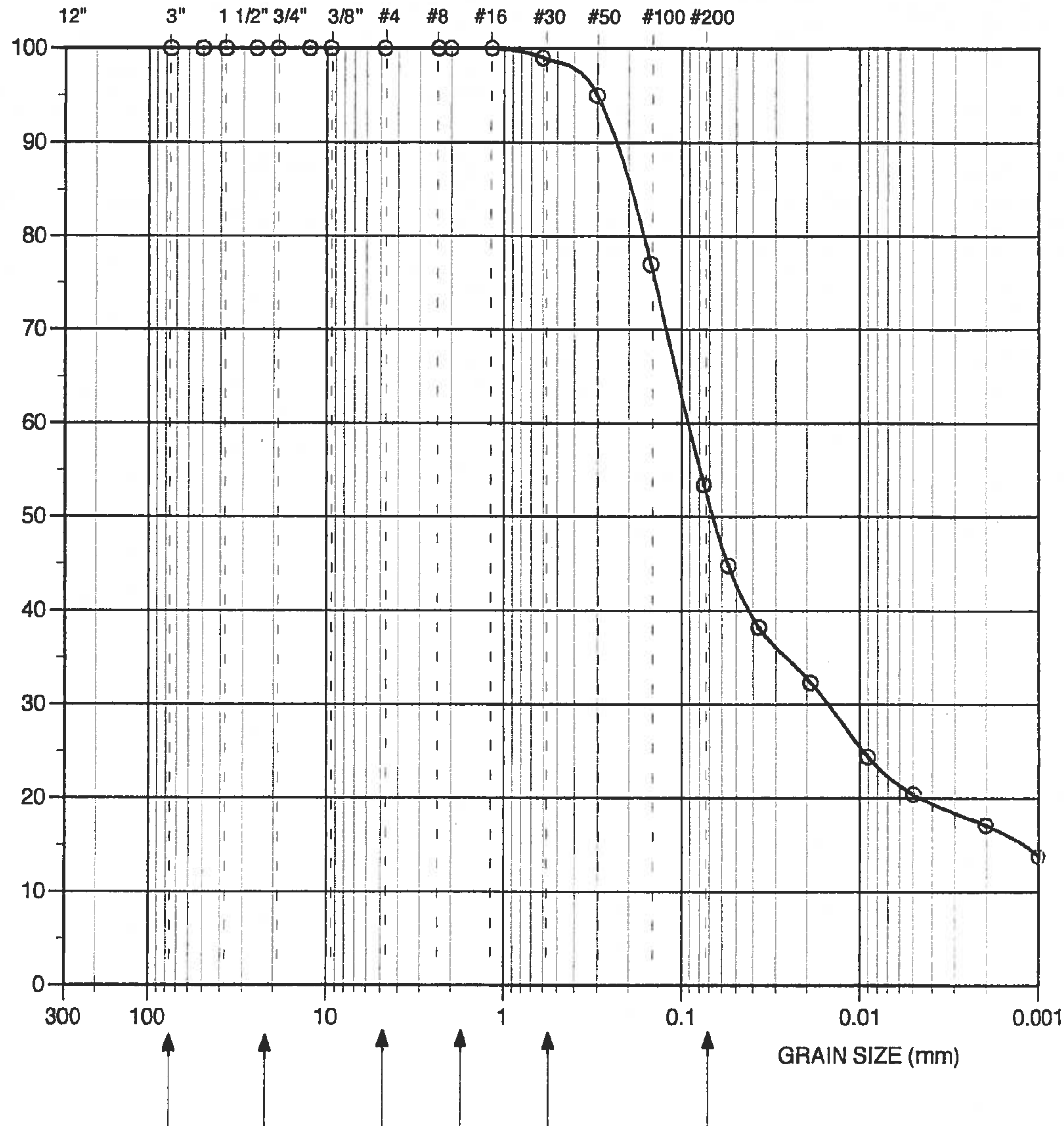
SYMBOLS	LOCATION	DESCRIPTION
⊙	B-8 at 16-1/2 to 18 feet	SILTY SAND, light brown with clay

GRADATION TEST DATA

BY: FF

DATE: 5-1-03

JOB NUMBER: 2616.100

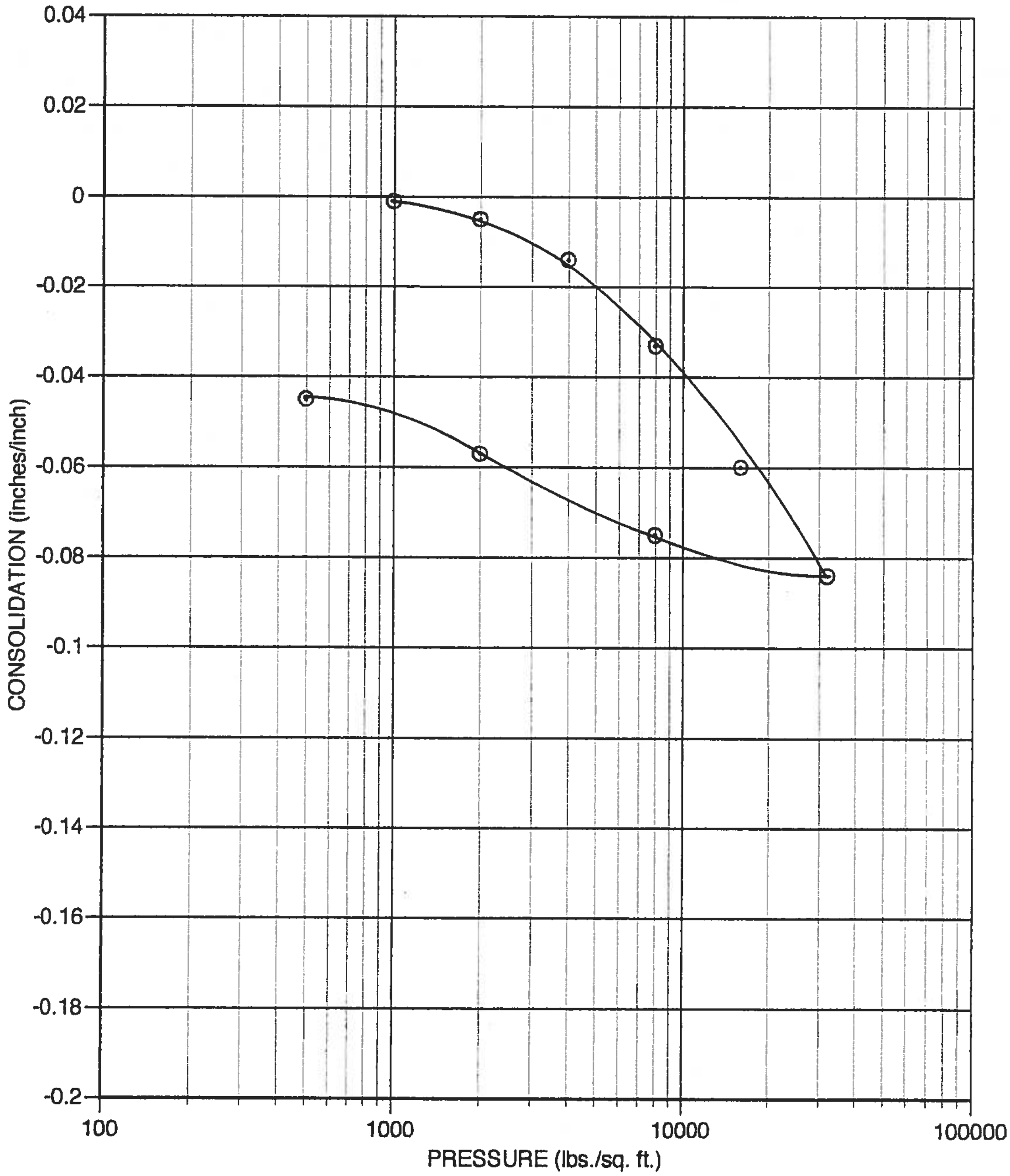


COBBLES	GRAVEL		SAND			SILT/CLAY
	coarse	fine	coarse	medium	fine	

SYMBOLS	LOCATION	DESCRIPTION
⊙	B-13 at 10-1/2 to 12 feet	SANDY CLAYEY SILT, red-brown

GRADATION TEST DATA

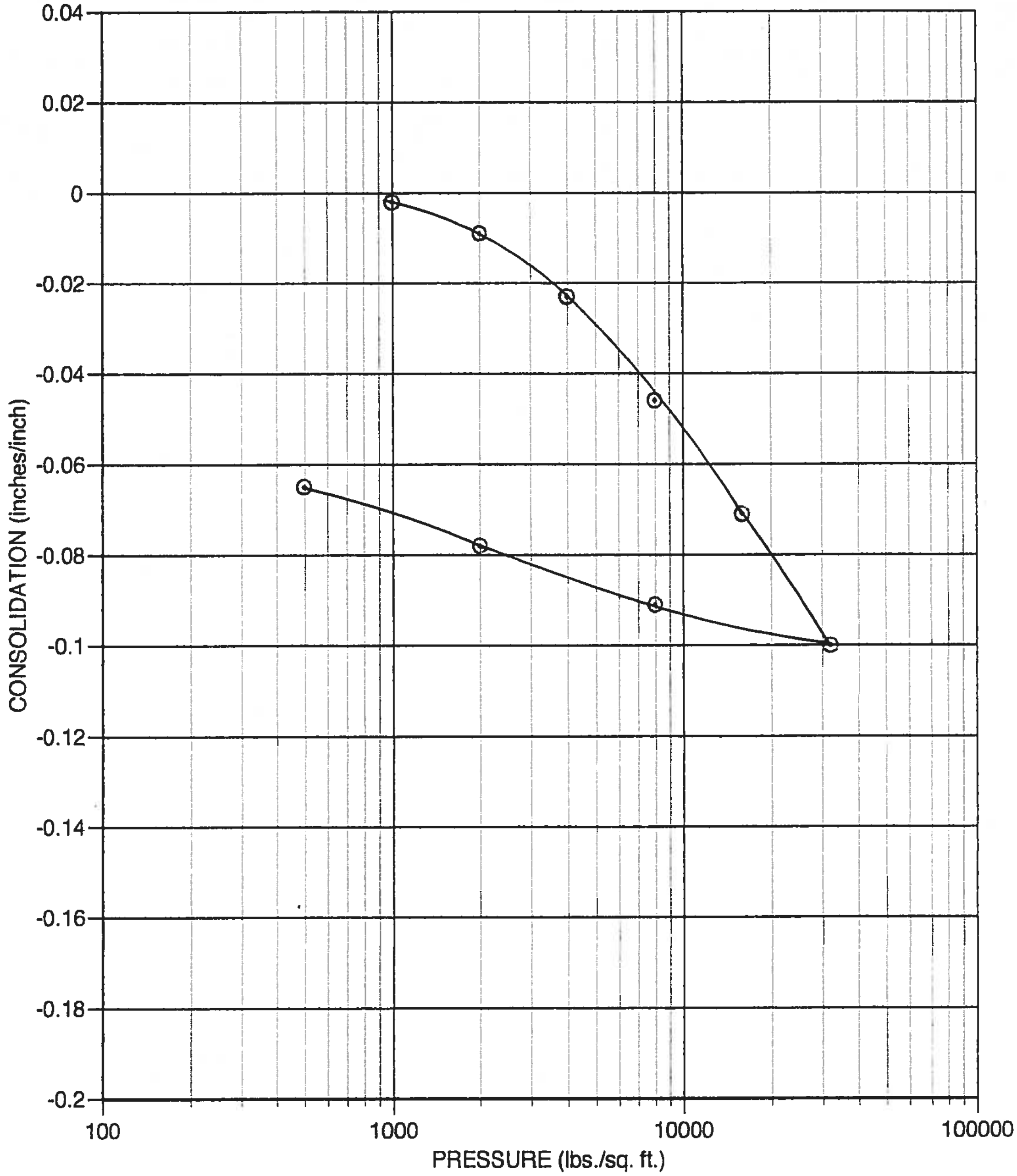
JOB NUMBER: 2626.100 DATE: 5-1-03 BY:FF



SYMBOL	LOCATION	DESCRIPTION	INITIAL MOISTURE CONTENT (%)	INITIAL DRY DENSITY (pcf)
⊙	B-1 at 5 feet	SILTY CLAY	19.1	109.5

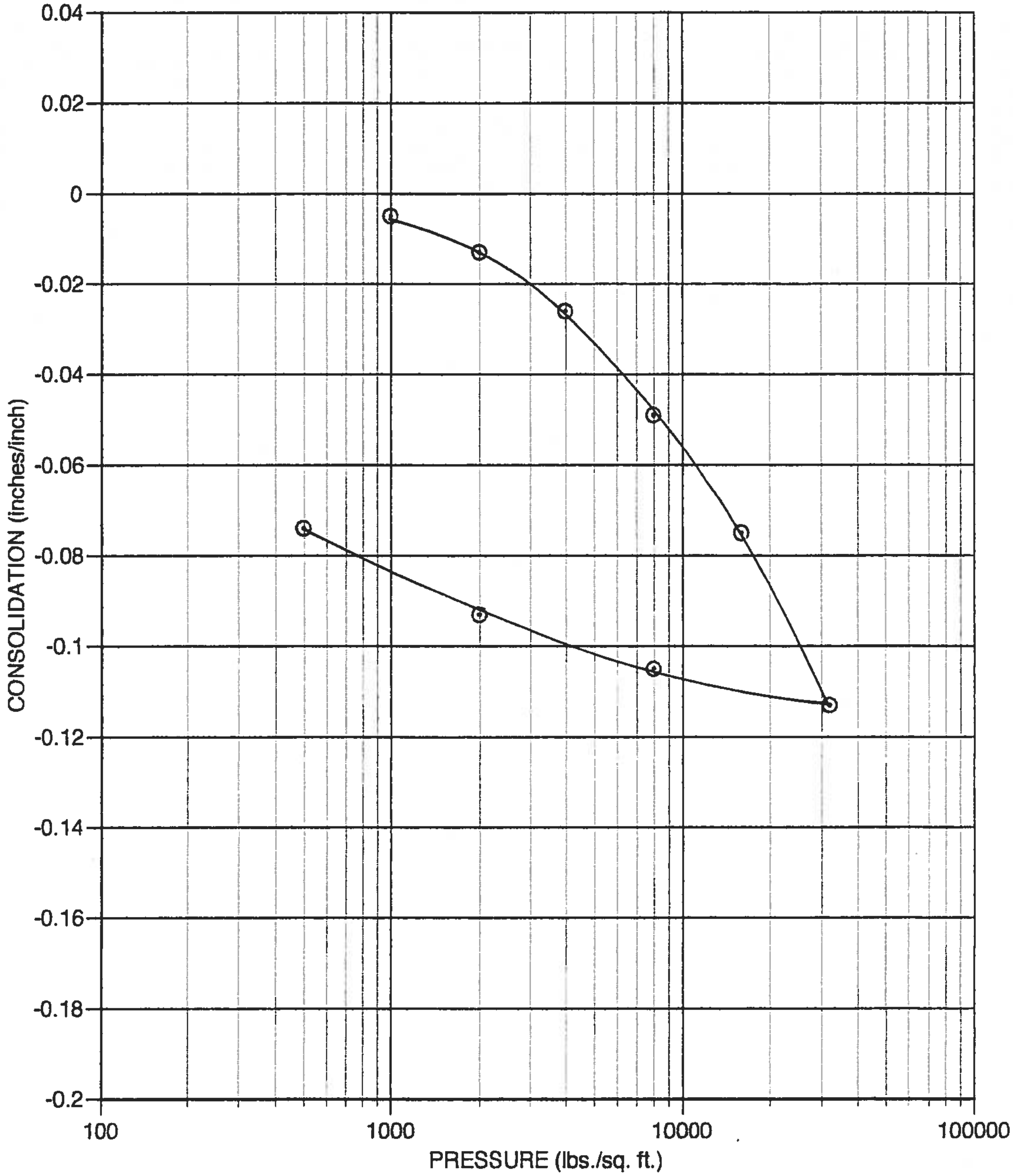
CONSOLIDATION TEST DATA

JOB NUMBER: 2626.100 DATE: 5-1-03 BY: FF



SYMBOL	LOCATION	DESCRIPTION	INITIAL MOISTURE CONTENT (%)	INITIAL DRY DENSITY (pcf)
⊙	B-1 at 10 feet	SANDY CLAY, yellow-brown	22.3	103.0

CONSOLIDATION TEST DATA



SYMBOL	LOCATION	DESCRIPTION	INITIAL MOISTURE CONTENT (%)	INITIAL DRY DENSITY (pcf)
⊙	B-9 at 4 feet	SANDY SILTY CLAY, dark brown	21.0	105.1

CONSOLIDATION TEST DATA

APPENDIX C

CERCO Analytical, Inc.
Corrosion Test Data

California State Certified Laboratory No.2153

CERCO
analytical, inc.

3942-A Valley Avenue
Pleasanton, CA 94566-4715
Tel: 925.462.2771
Fax: 925.462.2775

24 April, 2003

Job No.0304121
Cust. No.10598

Mr. Paul Lai
Berlogar Geotechnical Consultants
5587 Sunol Blvd.
Pleasanton, CA 94566

Subject: Project No.: 2616.100
Project Name: UOP Property
Corrosivity Analysis – ASTM Test Methods

Dear Mr. Lai:

Pursuant to your request, CERCO Analytical has analyzed the soil samples submitted on April 11, 2003. Based on the analytical results, a brief corrosivity evaluation is enclosed for your consideration.

Based upon the resistivity measurements, Sample No.001 is classified as "corrosive" and Samples No.002 and No.003 are classified as "moderately 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 concentrations range from none detected to 76 mg/kg. Because the chloride ion concentrations are less than 300 mg/kg, they are determined to be insufficient to attack steel embedded in a concrete mortar coating.

The sulfate ion concentrations reflect none detected with a detection limit of 15 mg/kg and are determined to be insufficient to damage reinforced concrete structures and cement mortar-coated steel at these locations.

The pH of the soils range from 6.0 to 8.5 which does not present corrosion problems for buried iron, steel, mortar-coated steel and reinforced concrete structures. However, any soils with a pH of <6.0 are considered to be corrosive to buried iron, steel, mortar-coated steel and reinforced concrete structures, and corrosion prevention measures will need to be considered for structures to be placed in acidic soils.

The redox potentials range from 290 to 400-mV, which are 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.

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

JDH/jdl

CERCO Analytical, Inc.

3942-A Valley Avenue, Pleasanton, CA 94566-4715 (925) 462-2771 Fax (925) 462-2775

FINAL RESULTS

Client:	Berlogar	Date Sampled:	31-Mar-2003
Client's Project No.:	2616.100	Date Received:	11-Apr-2003
Client's Project Name:	UOP Property	Date of Report:	24-Apr-2003
Authorization:	Signed Chain of Custody	Matrix:	Soil

Job/Sample No.	Sample I.D.	Redox (mV)	pH	Conductivity (umhos/cm)*	Resistivity (100% Saturation) (ohms-cm)	Sulfide (mg/kg)*	Chloride (mg/kg)*	Sulfate (mg/kg)*
0304121-001	TP2-13 @ 6-9'	300	8.5	-	1,200	-	76	N.D.
0304121-002	TP2-15 @ 4-8'	290	8.4	-	2,500	-	N.D.	N.D.
0304121-003	TP2-19 @ 1-9'	400	6.0	-	4,800	-	N.D.	N.D.

Method:	ASTM D1498	ASTM D4972	ASTM D1125M	ASTM G57	ASTM D4658M	ASTM D4327	ASTM D4327
Detection Limit:	-	-	10	-	50	15	15
Date Analyzed:	17-Apr-2003	18-Apr-2003	-	23-Apr-2003	-	18-Apr-2003	18-Apr-2003

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

23 November 2004
Project No. 3965.01

Mr. Geoffrey A. Reilly
Christopher A. Joseph & Associates
101 H Street, Suite Q
Petaluma, California 94952

Subject: Third Party Geotechnical/Geological Review
Davidon Homes EIR
Petaluma, California

Dear Mr. Reilly:

This letter presents the third party geotechnical/geological review comments for the Davidon Homes project in Petaluma, California. The review of geotechnical/geological information was performed for the preparation of the Environmental Impact Report (EIR). The site located at the northwest and southwest corners of Windsor Drive and D Street, as indicated on the Site Location Map, Figure 1.

The scope of services for this third party geotechnical/geological review included:

- reviewing available published and unpublished geologic and seismicity data, including previous geotechnical and geologic reports for the site;

- reviewing select historical aerial photography of the site to identify features that may be associated with areas of slope instability, areas of fill, or other geologic conditions of concern; and

- performing a site reconnaissance to confirm features identified in the aerial photograph review and to observe the soil and site conditions for evidence of geologic hazards.

The project documents reviewed for this study included:

- vesting tentative map and planned unit district plans for the Davidon Homes/UOP Property, prepared by BKF Engineers, dated 28 January 2004;

- geotechnical feasibility investigation report for the UOP property, prepared by Berlogar Geotechnical Consultants, dated 7 March 2002; and

- design-level geotechnical investigation report for the UOP property, prepared by Berlogar Geotechnical Consultants, dated 22 September 2004.

This geotechnical/geological review was prepared with the technical assistance of Mr. David Simpson of Gilpin Geosciences, Inc. (GGI), who provided site mapping and consultation regarding geological and seismological issues. The scope of services did not include subsurface exploration or laboratory testing.

SITE DESCRIPTION

The project site includes two parcels totaling about 58 acres and is located at the northwest and southwest corners of Windsor Drive and D Street in Petaluma. The two parcels are separated by Windsor Drive. The site consists of a relatively flat east-west trending central alluvial plain with steep slopes to the north and south. Elevations on the site range from a low of about 100 feet above sea level along Kelly Creek to a high of about 380 feet above sea level at the southwest corner of the site (BKF, 2004).

The site is only slightly developed with one modular single-family residence, two barns, the burned remains of a house, and several agricultural structures near the intersection of D Street and Windsor Drive. A small stock pond embankment is present on the north-facing slope south of Kelly Creek, and remnants of an old stone and mortar foundation are present on the hill east of stock pond. Both the north and south parcels contain 10-foot public utility and 15-foot slope easements along the D Street and Windsor Drive right-of-ways. A water booster pump station is located within this easement along Windsor Drive.

Vegetation on the site consists primarily of grass and weeds with scattered oak trees present on the south slope and at the top of the slope north of Windsor Drive. Oak and bay laurel trees are present along the creek channels, and tall eucalyptus trees are present along D Street, near the farm buildings and to the south of the buildings. Blackberry bushes have overgrown the area around the burned farmhouse.

The stock pond embankment is about 15 feet high. A low “levee” of fill was graded to control potential overflow from the pond. The fill directs the pond overflow toward an existing swale located about 200 feet to the east. An apparently thin sliver of fill also underlies the southern edge of Windsor Drive and the western edge of D Street.

The creek channels were dry at the time of GGI’s reconnaissance on 12 August 2004. No springs were observed on the site. Also, the area labeled as “wetland status to be investigated” (Berlogar, 2004) was dry. A small pool of water in the creek channel near the southeast corner of the site and water in the stock pond were the only locations where surface water was observed at the site on 12 August 2004.

PROJECT DESCRIPTION

The propose project will consist of subdividing the site to create 93 building lots that will eventually be improved with single-family dwellings. Approximately 20 acres on the north and south sides of Kelly Creek are proposed as open space. Also, the project plans include preserving the stock pond and designating 2.35 acres to the east of the stock pond as open space.

New streets providing access to the site will branch off of Windsor Drive and D Street. Two creek crossings are being proposed: 1) a five-foot-wide pedestrian foot bridge located roughly in the center of the site and 2) a roadway crossing over a 60-inch-diameter culvert located in the southeast part of the site near D Street. A vehicular bridge is also planned to cross a shallow swale that extends north of the potential wetland area near the center of the site.

In order to achieve design grades, cuts of up to about 36 feet and fills of up to about 32 feet are planned. The design grading will result in cut slopes up to about 80 feet tall and fill slopes up to about 30 feet high (Berlogar, 2004).

SITE HISTORY

Historical site conditions were observed by reviewing aerial photographs dating back to 1950. Eight black and white stereo aerial photograph pairs and one black and white single aerial photograph were reviewed from Pacific Aerial Surveys, Oakland. The aerial photographs reviewed are listed in Table 1. Standard aerial photograph review and photogeologic mapping techniques were employed to identify significant geologic features at the site such as tonal contrasts, vegetation patterns, and abrupt changes in topographic slope. The following sections provide a limited chronology of site development and slope conditions based on the photographs.

Table 1
List of Aerial Photographs Reviewed[1]
Davidon Homes, Petaluma, California

Date	Photo Number	Scale	Type**
10/10/50	AV 41-02-24*	1: 6,000	B&W
06/12/56	AV 222-04-15, -16	1: 24,000	B&W
04/14/66	AV 71-02-11, -12	1: 36,000	B&W
04/10/68	AV 844-05-13, -14	1: 30,000	B&W
05/03/80	OIR-SON-19-29, -30	1: 24,000	B&W
04/19/86	AV 2860-07-31, -32	1: 12,000	B&W
04/23/92	AV 4252-24-49, -50	1: 12,000	B&W
03/15/96	AV 5132-110-02, -03	1: 24,000	B&W
06/15/00	AV 6540-19-34, -35	1: 12,000	B&W

* Single photograph
**B&W = black and white

Development History

The earliest available aerial photographs, dated 1950, showed site improvements consisting of a farmhouse, two barns, and associated structures near the east side of the site. Thick tree cover was also observed along Kelly Creek. In June 1956, most of the site north of the north-facing slope on the southern third of the site had been mowed and the grass collected into bales. The small stock pond embankment on the north-facing slope was visible in the April 1966 photographs. The stock pond was constructed by building an earth berm, possibly using on-site materials excavated nearby. The 1980 photographs indicated that the farmhouse at the site was partly burned and the modular home was constructed. Windsor Drive and residential developments north and northeast of the site are visible in the April 1992 photographs. A portion of Windsor Drive was constructed on the site property, and no residential development has been completed along this portion of the street. The site conditions remained relatively unchanged after 1992.

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1 Aerial photographs provided by Pacific Aerial Surveys in Oakland, California

Historical Slope Conditions

Several landslides are visible on the site hill slopes in many of the photographs reviewed. Two apparently active slides, landslides A and C (Berlogar, 2004), are visible near the southeast corner of the site, just south and upslope of the tributary channel to Kelly Creek. The ground surface of these two landslides are hummocky; however, bare soil or rock are not exposed in scarps at the crest or sides of these landslides. This may indicate either slow creeping movement of these features or it may indicate the passage of sufficient time since slide movement occurred that vegetation has become established on the scarp areas.

Shallow landslides and raveling at landslides N, O, and P (Berlogar, 2004) are visible on the steep slopes on the north bank of the incised Kelly Creek channel in the western half of the site, where the tree canopy does not obscure the underlying slopes in the photographs reviewed. Older landslides, landslides E and H (Berlogar, 2004), are visible immediately north of Kelly Creek.

REGIONAL GEOLOGY

The site is located in the Coast Ranges geomorphic province of California that is characterized by northwest-southeast trending valleys and ridges. These topographic features are controlled by folds and faults that resulted from the collision of the Farallon and North American plates and subsequent predominantly strike-slip faulting along the San Andreas Fault system.

Regional geologic mapping shows the site vicinity to be underlain by bedrock of the late Jurassic through Cretaceous age (about 160 through 65 million years old) Franciscan Complex and undifferentiated Miocene and Pliocene age (about 24 through 1.8 million years old) Sonoma Volcanic rocks (Blake et al., 1974; Huffman and Armstrong, 1980; Wagner and Bortugno, 1982; and Bezore et al., 2002). These conditions are shown on Regional Geologic Map, Figure 2. The Franciscan rocks mapped on the majority of the site are described as melange: primarily sheared shale and sandstone with resistant masses of chert, greenstone, and metagraywacke. The undifferentiated Sonoma Volcanic rocks are mapped on the northern edge of the site and to the north and east of the site and consist of rhyolite, andesite, basalt, and tuff.

SITE GEOLOGY

Berlogar identified eight large landslides, designated as landslides A through H, affecting the site and several smaller landslides on the oversteepened banks along the riparian corridor of Kelly Creek. The locations of the landslides are shown on the Site Geologic Map, Figure 3, which is based on "Plate 2" of the 2004 Berlogar report. The Berlogar report indicates the larger landslides are up to about 15 feet thick, but are typically not more than 7 or 8 feet thick. Three bedrock shear zones are identified by Berlogar: two across the southern half of the west site boundary and a third short zone was identified at the center of the north edge of the site, as shown on Figure 3.

GGI performed a reconnaissance of the site on 12 August 2004 to observe the site conditions and geology. During this visit, GGI mapped the geology of the site and checked the geologic mapping prepared by Berlogar. Two active landslides, designated as A and C by Berlogar and presented on Figure 3, are visible in the aerial photographs near the southeast corner of the site and were confirmed by GGI in the field. The ground surface of these two landslides is hummocky, and there was a small pool of water in the channel of the creek at the toe of the smaller slide (designated as landslide C in the Berlogar report). Several shallow landslides (designated as N, O, P, and R in the Berlogar report), as well as other smaller unnamed slides were confirmed by GGI on the steep slopes along the north side of Kelly Creek and on the western half of the site where the creek has eroded and oversteepened the bank.

GGI could not confirm the presence of landslides B, F, and G, as described in the Berlogar report. Berlogar explored landslide B by excavating and logging four test pits, one of which was located in a mapped shear zone. Based on GGI's review, the logs of these pits do not indicate the presence of landslide materials or a basal landslide plane. Berlogar explored landslide F by excavating and logging two test pits, one of which identified a "sharp basal contact, possible slide plane" with an 18 degree dip (direction not specified). If landslide F exists, the lack of surficial evidence defining the limits of the deposit indicates that it has not moved in a long time. Berlogar explored landslide G by excavating one test pit, where they encountered a 1/4-inch-thick slicken-sided clay slide plane. However, the test pit was excavated in the middle of a mapped shear zone; therefore, it is difficult to determine if the slicken-sided plane is associated with a landslide or is an inherent feature of the shear zone.

GGI observed a short steep 15- to 20-foot wide unvegetated slope near the west property line about 80 feet south of Kelly Creek. GGI indicates this feature was interpreted by Berlogar to be a narrow landslide scarp within landslide G. However, GGI notes that this interpretation was not confirmed with subsurface exploration, and no hummocky or bulging accumulation of slide debris was present downslope of this feature.

GGI identified rock outcrops of weak to moderately strong, moderately hard Franciscan Complex graywacke sandstone and shale are present along the channel of Kelly Creek and in the tributary that intersects Kelly Creek from the south near D Street. This bedrock is moderately weathered, fine-grained with a trace of lithic fragments, and is intensely fractured. Several scattered outcrops of Franciscan graywacke sandstone are also present on the steep north facing slopes on the southern third of the site. One outcrop of dark reddish brown and black banded Franciscan chert was observed by GGI along the west property line at the top of landslide H. This outcrop had previously been identified by Berlogar as Franciscan sandstone. A cut along the west side of D Street, on the east end of the hill north of Windsor Drive, exposes dark gray to dark brown, moderately weathered basalt of the Sonoma Volcanics. GGI indicates that it is possible that this rock also underlies the hill to the west, although volcanic rocks are not mapped by Berlogar on the site.

GGI indicates that topographic evidence of the three bedrock shear zones mapped by Berlogar was not visible on the surface of the site during the site reconnaissance on 12 August 2004.

SUBSURFACE CONDITIONS

Berlogar performed a design-level geotechnical investigation in 2003 to 2004 for the proposed development, the results of which are presented in a report dated 22 September 2004. Their field exploration included drilling 14 borings and excavating 36 test pits at the site to characterize the engineering properties of soil and bedrock at the site. Berlogar previously performed a geotechnical feasibility investigation in 2002. The feasibility investigation included excavating 26 test pits and one short trench. Based on the subsurface investigations, Berlogar concluded five types of soil/bedrock were encountered at the site: artificial fill (Qaf), landslide deposits (Qls), colluvium (Qc), alluvium (Qal), and Franciscan bedrock (KJf).

Berlogar indicated isolated areas of artificial fill were encountered in three main areas: 1) beneath and around existing buildings, 2) adjacent to the stock pond, and 3) along the downslope (south) side of Windsor Drive. Berlogar describes the fill as generally consisting of dense sandy silt and gravel and stiff to very stiff silty clay. Berlogar mapped the central half of the site along Kelly Creek as being covered with alluvium and the adjacent swales as covered with colluvium. Alluvium, consisting of sandy clays and clayey sands with various amounts of gravel was mapped by Berlogar in relatively flat areas bordering drainage courses and at the down-slope end of the swales. Colluvium, consisting of stiff to very stiff clay with minor amounts of gravel was present in the lower portions of the site. The colluvium at the site is moderately expansive. Berlogar mapped two types of Franciscan bedrock on the site. Sandstone (KJfss) is mapped along most of the northern edge of the site and at scattered outcrops in the channel of Kelly Creek. Sandstone and shale (KJfss/sh) is mapped on the majority of the upland portions of the site.

REGIONAL SEISMICITY

The coastal areas of Northern California are seismically active, and the site can be expected to experience periodic minor earthquakes and possibly a major earthquake (moment magnitude 7 or greater) on one of the nearby active faults during the life of the proposed project. The site will be subject to strong to very strong shaking during a large event on the nearby faults.

The seismicity in the site vicinity is related to activity on the San Andreas system of active faults. The faults in this system are characterized by right-lateral, strike-slip movements (movement is predominantly horizontal). The nearest major active fault is the Rodgers Creek fault located approximately 8.5 kilometers east of the site. Other major active faults in the area are the San Andreas, West Napa, Maacama, and Hayward faults (Jennings, 1994). These and other faults of the region are shown on Figure 4. A list of major active faults in the region, including the distance from the site and estimated maximum Moment magnitude[2] [Working Group on California Earthquake Probabilities (WGCEP) (2003) and Cao et al. (2003)] are summarized in Table 2.

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2 Moment magnitude is an energy-based scale and provides a physically meaningful measure of the size of a faulting event. Moment magnitude is directly related to average slip and fault rupture area.

Table 2
Regional Active Faults and Seismicity

Fault Name	Distance from Site (km)	Direction From Site	Maximum Moment Magnitude
Rodgers Creek	8.5	East	7.0
Total Hayward-Rodgers Creek	8.5	East	7.3
San Andreas - 1906 Rupture	22	Southwest	7.9
San Andreas- North Coast South	22	Southwest	7.5
North Hayward	24	Southeast	6.5
Total Hayward	24	Southeast	6.9
West Napa	30	East	6.5
Point Reyes	35	Southwest	6.8
Maacama-Garberville	40	North	6.9
Concord/Green Valley	43	East	6.7
Hunting Creek-Berryessa	47	Northeast	6.9
San Andreas - Peninsula	49	South	7.2
Northern San Gregorio	50	South	7.2
Total San Gregorio	50	South	7.4

Since 1800, four major earthquakes have been recorded on the San Andreas Fault. In 1836 an earthquake with an estimated maximum intensity of VII on the Modified Mercalli (MM) scale (Figure 5) occurred east of Monterey Bay on the San Andreas Fault (Toppozada and Borchardt 1998). The estimated Moment magnitude, $M_{w,}$ for this earthquake is about 6.25. In 1838, an earthquake occurred with an estimated intensity of about VIII-IX (MM), corresponding to a M_w of about 7.5. The San Francisco Earthquake of 1906 caused the most significant damage in the history of the Bay Area in terms of loss of lives and property damage. This earthquake created a surface rupture along the San Andreas Fault from Shelter Cove to San Juan Bautista approximately 470 kilometers in length. It had a maximum intensity of XI (MM), a M_w of about 7.9, and was felt 560 kilometers away in Oregon, Nevada, and Los Angeles. The Loma Prieta Earthquake of 17 October 1989 also affected the greater Bay Area. This earthquake occurred in the Santa Cruz Mountains with a M_w of 6.9, approximately 148 km from the site.

In 1868, an earthquake with an estimated maximum intensity of X on the MM scale occurred on the southern segment (between San Leandro and Fremont) of the Hayward Fault. The estimated M_w for the earthquake is 7.0. In 1861, an earthquake of unknown magnitude (probably a M_w of about 6.5) was reported on the Calaveras Fault. The most recent significant earthquake on this fault was the 1984 Morgan Hill earthquake ($M_w = 6.2$).

In 2002, the Working Group on California Earthquake Probabilities (WGCEP 2003) at the U.S. Geologic Survey (USGS) predicted a 62 percent probability of a magnitude 6.7 or greater earthquake occurring in the San Francisco Bay Area by the year 2031. More specific estimates of the probabilities for different faults in the Bay Area are presented in Table 3.

TABLE 3
WGCEP (2003) Estimates of 30-Year Probability (2002 to 2031) of a Magnitude 6.7 or Greater Earthquake

Fault	Probability (percent)
Hayward-Rodgers Creek	27
San Andreas	21
Calaveras	11
San Gregorio	10
Concord-Green Valley	4
Greenville	3

GEOLOGIC AND SEISMIC HAZARDS

Potential geologic and seismic hazards at the project site include fault rupture, landslide hazards, erosion, flooding, and expansive soil. These and other geologic and seismic hazards are discussed in the following sections.

Fault Rupture

Berlogar indicates the site is not located within a State of California Earthquake Fault Zone and no evidence of an active fault crossing or trending toward the site. Berlogar indicates the nearest mapped active fault to the site is the Rodgers Creek fault, about 11 kilometers (km) northeast of the site.

T&R concurs with Berlogar's assessment that the site is not located within an Earthquake Fault Zone (CDMG, 1974). No active faults or extensions of active faults are mapped on the site, and surficial indications of faulting on the site were not identified during GGI's site reconnaissance. T&R estimates the nearest mapped active fault, the Rodgers Creek fault, lies about 8.5 kilometers east of the project site. T&R concludes the potential for fault rupture at the site is low.

Seismic Hazards

In addition to triggering landslides, strong ground shaking caused by large earthquakes can induce ground failures, such as liquefaction[3], lateral spreading[4], and cyclic densification[5]. A site's susceptibility to these hazards relates to the site topography, soil conditions, and/or depth to groundwater.

Berlogar indicated material susceptible to liquefaction or significant dynamic densification was not encountered at the site. T&R reviewed the test pit and boring logs prepared by Berlogar and concluded that the soil at the site has sufficient fines and/or density to resist liquefaction and cyclic densification. Therefore, T&R concurs with Berlogar's evaluation and conclusion that the potential for liquefaction or seismically induced differential settlement to occur at the site is very low. In addition, T&R concludes the potential for liquefaction-induced hazards, such as lateral spreading, is also very low.

Seismically-induced landsliding could potentially be a hazard in areas of moderate to steep slopes underlain by thick soils, weak or fractured rock (i.e. much of the Franciscan melange bedrock), previously existing landslides, or loose fill. Mitigation alternatives for seismically induced landsliding include grading and drainage of existing landslides and steep slopes, and setbacks from incised stream channels. Berlogar stated that the stability of all slopes becomes

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3 Liquefaction is a phenomenon in which saturated, cohesionless soil experiences a temporary loss of strength due to the buildup of excess pore water pressure, especially during cyclic loading such as that induced by earthquakes. Soil most susceptible to liquefaction is loose, clean, saturated, uniformly graded, fine-grained sand; however, low plasticity silts and clay can also liquefy.

4 Lateral spreading is a phenomenon in which surficial soil displaces along a shear zone that has formed within an underlying liquefied layer. Upon reaching mobilization, the surficial blocks are transported downslope or in the direction of a free face by earthquake and gravitational forces.

5 Cyclic densification is a phenomenon in which non-saturated, cohesionless soil is densified by earthquake vibrations, causing ground surface settlement.

lowered during an earthquake event. However, grading in accordance with the recommendations presented in the Berlogar report will help to reduce the risk of seismically induced landslides. T&R concurs with Berlogar's assessment for mitigating seismically-induced landslide hazards.

Bedrock Shear Zones

The Berlogar report indicates the three bedrock shear zones identified on the site represent ancient shearing within the Franciscan Complex that occurred during its emplacement onto the North American continent, and therefore, the risk of surface displacement along the shear zones is very low. If the rock is sufficiently sheared and weathered, it may require mitigation if construction is proposed nearby. The current project documents prepared by BKF (2004) indicate that only the eastern end of the longest shear and the short shear zone at the north edge of the site will extend into the proposed developed portion of the site. The Berlogar report indicates that if zones of soft or saturated soil are encountered during site preparation and grading activities, excavations deeper than those recommended in the report may be required to expose competed materials. The limits of excavation will be determined in the field by the soil engineer. T&R concurs with Berlogar's assessment and proposed mitigation.

Landslides

A large-scale regional landslide map of Sonoma County identifies slides on most of the slopes of the site (Huffman and Armstrong, 1980). Berlogar identified eight large landslides and several smaller landslides on the banks along Kelly Creek. Five of the large landslides (as identified in the Berlogar report) are located partially or entirely within the proposed development area (BKF, 2004). Three of the large landslides and the smaller landslides along the creek bank are outside the proposed development area.

Berlogar stated the preferred remedial measure is complete removal of landslide debris located within the development area. Due to limitations, such as landslides extending beyond property lines, Berlogar concluded complete removal of landslide debris is impractical and the potential for adverse impacts to the planned development from partial removal of landslide can be minimized by implementing the following remedial measures:

- remove landslide debris and replace with engineered fill with proper subdrainage;
- construct keyways with proper subdrainage; and
- construct geogrid reinforced MSE retaining walls.

In the 2004 report, Berlogar recommended all keyway excavations should be mapped by an engineering geologist prior to backfilling. Also, Berlogar recommended subdrain locations and final subdrain trench configurations be approved by a soil engineer. Due to the potential for adverse impacts to the development associated with landslides and landslide mitigation activities, T&R concurs with Berlogar's recommendation. In addition, T&R suggests that a qualified engineering geologist and/or geotechnical engineer be retained by the site developer to observe all landslide remediation activities, and check that all landslide debris within designated excavation areas has been properly removed and engineered fill, keyways, subdrains, and MSE retaining walls have been properly constructed.

Debris Flow and Sedimentation

With the exception of two large swales on the south side of the site, Berlogar concluded the potential for debris flows to impact the site is low across most of the development.

Based on aerial photographs and field observations, Berlogar concluded the swale upslope of Lots 78 and 79 may have experienced small debris flows in the past. In addition, Berlogar concluded the downslope gradient of this swale is sufficient to allow movement of debris flows, which could potentially deposit debris within the development area. The current grading plan shows a proposed concrete headwall located in the swale and a drainage inlet box just upslope of the headwall. The Berlogar report indicates the headwall will create a catchment area for potential debris flows. The report also indicates the catchment volume appears "roughly adequate" for intercepting the potential debris flows, and the design of the headwall and catchment area will be refined as the project planning proceeds.

It is T&R's opinion that the feasibility of using a concrete headwall to create a catchment area for potential debris should be better assessed. T&R recommends the headwall be designed to resist the impact force associated with potential debris flows. The catchment area volume should be adequately sized to contain the anticipated volume of potential debris flow material. Also, T&R suggests a homeowner's association or similar entity be assigned the responsibility for periodically inspecting and maintaining the debris flow catchment system. Alternatively, the stability of debris flow material could be assessed, and mitigative actions should be implemented to either prevent debris flows from occurring or protect downslope properties from being adversely impacted by potential debris flows. The evaluation of debris flow hazards with mitigation recommendations should be provided by Berlogar.

Berlogar indicated the swale located upslope of Lots 82 and 83 has a gradient of 8:1 (horizontal: vertical) for a substantial distance, which is too low to allow significant movement of debris flow; however, sedimentation has been occurring in the swale. Berlogar concluded future sedimentation could clog the proposed drain inlet boxes located upslope of 82 and 83. The drainage inlets at these locations should be designed to reduce the potential for excessive sediment entering the drainage system. Based on discussions with GGI, T&R concurs with Berlogar's assessment of the debris flow hazard, but suggests a homeowner's association or similar entity be assigned the responsibility for periodically inspecting and maintaining the headwall and drain inlet system.

Temporary Cut Slopes

The Berlogar report indicates there is no adverse bedding conditions at the locations of proposed cut slopes; however, due to folding and shearing of the bedrock, localized areas of adverse bedrock structure or other zones of geologic weakness could be exposed during grading of cut slopes. Any adverse bedding that exists will increase the potential for landsliding.

T&R concurs with Berlogar's recommendation that all keyway excavations should be mapped by an engineering geologist prior to backfilling. Field supervision by a qualified engineering geologist will allow for the timely identification and mitigation of adverse bedding conditions, if they are encountered during construction.

Erosion

GGI's site reconnaissance and aerial photograph review indicate erosion is occurring along the incised channels of Kelly Creek and a southern tributary near the east side of the site. The presence of bedrock in the floors of these two channels indicates that downcutting is relatively slow along these seasonal streams, and lateral erosion of unconsolidated materials in the channel banks appears to be the main mode of erosion. A small gully was observed in the alluvium across the very gently northeast-sloping valley bottom north of the stock pond. The source of the water that caused the erosion is not known at this time.

To mitigate the potential for future erosion, Berlogar recommended all cut and fill slopes be planted with fast growing, deep-rooted vegetation before the first winter.

T&R concurs with Berlogar's recommendations to mitigate the potential for erosion. In addition, T&R suggests controlling surface runoff and directing it away from potentially unstable site slopes and proposed improvements, and using erosion control blankets and fiber rolls to temporarily protect the slope surfaces from erosion until adequate deep-rooted vegetation is established.

Flooding

The site is above the Federal Emergency Management Agency's (FEMA) 100-year and 500-year flood zones for the Petaluma River (FEMA, 1989). The site is not susceptible to tsunamis or seiches. However, if the small stock pond embankment were to fail while retaining water, that water would be released onto the downslope property. Berlogar recommended the north slope of the stock pond embankment be reinforced with a keyed earth buttress with a slope inclination not to exceed 2:1 (horizontal:vertical). Subsurface drainage should also be installed during the construction of the earth buttress.

T&R concurs with Berlogar's recommendation to buttress the existing stock pond embankment to reduce the potential for future flooding. Also, T&R recommends properly maintaining the earthen ditch system that directs overflow water toward the east to the existing swale. The periodic inspection and maintenance of this system should become the responsibility of the homeowner's association or similar entity.

Expansive Soil

Expansive soils shrink or swell with changes in moisture content. Clay mineralogy, clay content, and porosity of the soil influence the change in volume. The shrinking and swelling caused by expansive clay-rich soil can result in damage to overlying structures. Site soils encountered by Berlogar were found to be moderately expansive. The Berlogar report indicates that 5- to 35-foot-thick layers of compacted fill may swell between 3/4 and 1-1/2 inches, respectively. The actual swell will depend on the total thickness of fill, material in the fill, and in-place moisture content and density. The Berlogar report indicates mitigation alternatives for expansive soils include moisture conditioning and recompaction of expansive soil, use of non-expansive fill, or designing foundations to resist or tolerate differential movement of expansive soil.

Based on T&R's experience, the soil within the zone of seasonal soil fluctuation or within areas susceptible to flooding may experience differential movement associated with expansive soil. Typically, the depth of severe seasonal moisture change is limited to about 2 to 3 feet below the ground surface. Therefore, in T&R's opinion, Berlogar's swell estimates for a properly moisture-conditioned and compacted fill with a thickness greater than about three feet, does not seem to be consistent with the behavior of moderately expansive soil. Although, T&R concurs with Berlogar regarding the proposed mitigation alternatives that include moisture-conditioning and recompacting the soil, using non-expansive fill, or designing foundations to resist or tolerate differential movement. In addition, T&R suggests that site grades be designed to slope away from the proposed structures and water from roof drains be directed to suitable outlets.

GEOTECHNICAL AND FOUNDATION ISSUES

The Berlogar report concludes the site can be developed as proposed, provided the conclusions and recommendations presented in the report are incorporated into the project design and construction. According to Berlogar, the primary geotechnical considerations are landslide remediation, treatment of existing fill, fill slope construction, stability of proposed cut slopes, and the potential for expansion and settlement of on-site earth materials.

The primary conclusions and recommendations presented by Berlogar and our associated comments are summarized in the following sections.

Foundations and Settlement

Berlogar concluded proposed residential structures may be supported on either structural mat/slab foundations or drilled, cast-in-place concrete piers and grade beams.

Berlogar recommended structural mat/slab foundations should consist of either conventional reinforced or post-tensioned concrete slab foundations. The mat/slab foundation should be designed to accommodate two inches of total soil movement and one inch in 25 horizontal feet of differential soil movement. The mat/slab foundation may be designed for an allowable bearing pressure of 1,500 pounds per square feet (psf) for static loads; this pressure may be increased by 1/3 for seismic and wind loads. The upper 12 inches of subgrade soil should be presaturated to at least five percent above optimum moisture content. The presaturated pad should not be allowed to dry out to less than five percent above optimum moisture content prior to placement of concrete or moisture break.

Berlogar recommended that drilled piers be designed using a skin friction value of 450 psf; skin friction derived from the upper foot of soil below adjacent grade should be neglected.

Proposed new residences may be constructed on cut and/or fill slopes. Berlogar estimated that on-site soil and bedrock material used as fill will settle during and after mass grading. The Berlogar report indicates fills with a thickness between 5 and 35 feet may experience total settlement of between 1/4 and 4-1/2 inches, respectively. Berlogar estimates that about 70 percent of the estimated total settlement should occur during mass grading; therefore, maximum post-grading settlement will be less that about 1-1/2 inches.

The Berlogar report also indicates that up to about 32 feet of fill is planned at locations underlain with colluvium and alluvium that extends to depths of about 25 feet below the ground surface. Berlogar describes the colluvial and alluvial soil at stiff to very stiff, silty to sandy clay and clayey sand. Berlogar estimates the settlement of these deposits will be less than about two inches.

Because fill and bedrock at the site will have different expansion and settlement potentials, Berlogar indicates that structures and foundations constructed across the transition line between cut and fill could experience significant differential expansion and/or settlement. To mitigate the potential for differential movement at cut/fill transition lots, Berlogar recommended overexcavating the cut portion of the cut/fill transition lots to a depth of about three feet below rough pad grade and backfilling the overexcavated areas with engineered fill.

T&R generally concurs with Berlogar's recommendations; however, there are several issues that require technical comments:

Total and Differential Settlement of Fill
To reduce the potential for total and differential settlement of engineering fills, T&R suggests moisture conditioning new fills that are more than 20 feet below the ground surface to above the optimum moisture content and to at least 95 percent relative compaction. Subsequently, the settlement behavior of the new fill should be monitored to confirm that total and differential settlements are within tolerable limits for the new buildings and site improvements. In addition, T&R suggests estimating the seismic compression of new fills at the site. Seismic compression can result in sudden and abrupt ground settlement that can damage new structures and site improvements.

Split-Level Residences
In T&R's opinion, Berlogar should provide settlement estimates for the upper and lower levels of the proposed residences. Structural strengthening or stiffening may be required if the differential settlement between the two levels is too large. Also, below-grade walls will be needed in the design of the split-level residences. Berlogar should indicate whether a lateral seismic increment and surcharge load should be included in the design of the below-grade walls of the new residences.

Mat and Drilled Pier Foundations
Berlogar's mat foundation design recommendations did not provide a minimum depth of mat embedment. In T&R's opinion, the minimum embedment depth and a requirement for the site grades to slope away from the proposed buildings will help to reduce the potential for surface

water to enter beneath the new buildings, resulting in potential surface water and moisture intrusion problems. Also, as previously discussed for split-level residences, Berlogar should indicate whether the proposed mat can achieve bearing support on the soil immediately adjacent to the below-grade wall.

For the drilled pier and grade beam foundation system, T&R suggests that Berlogar provide recommendations for the floor slab. Typical floor slab systems consist of either structured floors that gain support on the pier and grade beam foundation or slab-on-grade floors that bear directly on the soil subgrade. Additional slab reinforcement may be required if slabs are placed adjacent to below-grade walls.

Bridge Foundations

The Berlogar report indicates that new bridges should be supported on drilled piers designed in accordance with Caltrans seismic design criteria. Berlogar recommended drilled piers be designed with allowable skin friction values of 600 psf for compression and 300 psf for uplift; these skin frictions may be increased by 1/3 for seismic and wind loads. The upper five feet of pier embedment should be neglected for skin friction resistance. The drilled piers should be founded at least 15 feet below lowest adjacent grade.

T&R recommends that Berlogar indicate whether creek scour will have an adverse impact on the proposed bridge foundations. Also, if the bridges are to be designed in accordance with Caltrans seismic design criteria, Berlogar should provide recommendations for the 5% damped elastic acceleration response spectrum (ARS).

Site Grading and Fill Placement

The Berlogar report indicates that onsite soil/rock are suitable for re-use as engineered fill provided it does not contain rock fragments greater than 12 inches and is free of deleterious material. Imported fill should have plasticity index less than 12 and approved by the soil engineer. Fill should be placed in 6- to 8- inch lifts, moisture conditioned to at least three percent above optimum moisture content, and compacted to at least 90 percent relative compaction.

Berlogar recommended that fill placed on a slope with inclinations steeper than 7:1 should be benched into firm materials as determined in the field by the geotechnical engineer. Fill slopes should be constructed at gradients no steeper than 2:1. Where fill slopes over 30 feet in height, intermediate surface drainage benches should be spaced no more than 25 feet vertically on the slope. The benches should be a minimum of eight feet wide and include a concrete-lined V ditch to intercept surface water runoff. Fill slopes should be over built and cut back to expose a firm compacted surface. Fill slopes should be constructed with a six feet deep (minimum) keyway with a width equal to 1/2 the slope height or 20 feet, which ever is greater, and constructed with proper subdrainage.

In general, T&R concurs with the Berlogar recommendations, with the exceptions that T&R suggests: 1) limiting the maximum size of rocks or lumps in fill to no greater than six inches in greatest dimension so that loose soil adjacent to large rocks and lumps can be adequately compacted, and 2) using a higher compaction standard and a lower moisture condition for fill placed at depths greater than 20 feet below finish grade.

Seismic Design Criteria

Berlogar concluded residential structures be designed in accordance with the 2001 California Building Code (CBC). Berlogar recommended using the following seismic design criteria:

- seismic zone (Z) of 4
- soil profile type of S_D
- seismic source type A
- closest distance to known seismic source (Rodgers Creek fault) of 10 km.

Berlogar concluded bridges should be designed in accordance with the Caltrans seismic design criteria. Berlogar recommended using the following Caltrans seismic design parameters:

- closest distance to known seismic source (Rodgers Creek fault) of 7 km
- maximum credible earthquake magnitude (M_w) of 7.0
- soil profile type of S_D
- peak ground acceleration of 0.5 times gravity.

There is a discrepancy with the closest distance to Rodgers Creek fault. T&R estimates the closest distance to Rodgers Creek fault is about 8.5 km. Berlogar should re-evaluate the closest distance to known seismic source and modify the design criteria in accordance with the CBC and Caltrans methodology, if appropriate.

Surface and Subsurface Drainage

The Berlogar report states that surface drainage benches should be spaced no more than 25 feet vertically on the slope. The benches should be a minimum of eight feet wide and include a concrete-lined V ditch to intercept surface water runoff. Berlogar did not address surface drainage to collect and/or redirect surface water away from the building foundations.

T&R recommends roof downspouts should be connected to tightlines consisting of rigid, PVC pipe that will convey water to suitable discharge areas. Concrete-lined V ditches should also be placed at strategic locations to protect slopes. Storm water should not be allowed to pond or flow in concentrated streams or channels on the site. The homeowner's association or similar entity should be responsible for inspecting and maintaining the drains for the project.

Berlogar indicated that subsurface drains should be constructed behind retaining walls, on the uphill side of all keyways and proposed fill, at all spring and seepage areas, at the toes of major cut slopes, at geologic contacts known to transmit water, and other areas where seepage is observed during and after grading or as determined in the field by the geotechnical engineer.

T&R believes the design of surface and subsurface systems are important to the success of the proposed project. Of particular concern is the high susceptibility to erosion of site soils and the likely presence of weak Franciscan melange adjacent to the sandstone bedrock at or near the site perimeter (including neighboring properties). Therefore, T&R believes that Berlogar (or other geologic consultant with experience in surface and groundwater controls) should provide technical input and review of the surface and subsurface drainage systems for the purpose of reducing the potential for adverse impacts, such as surface erosion and shallow landslides, on and adjacent to site. Common design issues that may require technical input from Berlogar include: 1) the location of surface and subsurface drainage alignments, especially within filled slopes, 2) selection of water discharge locations, 3) separation of surface and subsurface water collection pipes, 4) location of pipe cleanouts, and 5) recommendations for controlling groundwater flow through trench backfill.

Retaining Walls

Berlogar recommended that all retaining walls be designed to resist the active pressure corresponding to an equivalent fluid weight of 50 pounds per cubic feet (pcf) and 65 pcf for level and sloped backfill, respectively; and the at-rest pressure corresponding to an equivalent fluid weight of 75 pcf. Berlogar also recommended adequate subdrainage be constructed behind retaining walls.

Berlogar recommended retaining walls may be designed for an allowable bearing pressure of 2,500 psf for static loads; this pressure may be increased by 1/3 for seismic and wind loads. Piers used for support of headwalls of the Kelly Creek tributary should be designed per the parameters presented for bridge foundations.

Due to the relatively close proximity of the Rodgers Creek fault to the site and the potential for strong ground shaking, T&R suggests that Berlogar consider applying a uniform seismic increment to the design of new retaining walls at the site. Also, if retaining walls are constructed adjacent to roadways and/or buildings, appropriate surcharge loads from the adjacent improvements should also be incorporated in the wall design.

CONCLUSIONS

T&R concludes the proposed project is feasible, but potentially constrained by: 1) strong ground shaking, 2) slope instabilities associated with existing landslides, potential debris flows, and new and existing cuts and fills, 3) localized settlement of compacted fill, and 4) impact of scour on bridge foundations adjacent to existing creek and tributaries. Based on the review of geotechnical/geological studies presented for this project, T&R concludes the consultants have performed an adequate geotechnical/geological characterization of the site conditions and provided suitable geotechnical/geological recommendations for many of the site issues. However, T&R believes there are several issues that need further evaluation.

Based on the geological/geotechnical third party review, T&R has the following comments that should be addressed or commented upon by the project applicant. These comments were previously discussed in this letter and are summarized as follows.

<u>T&R Comment No. 1</u>
The current grading plan shows a concrete headwall, which will create a catchment area for potential debris flows. T&R recommends that Berlogar evaluate if the headwall is adequate for resisting the impact forces associated with debris flows and if the catchment area is large enough to contain the potential debris flow material without adversely impacting the downslope properties.

T&R Comment No. 2
T&R suggests that Berlogar evaluate the use of engineered fill that contains Franciscan rock fragments with dimensions of up to 12 inches, and indicate how adequate soil compaction will be achieved in areas adjacent to large rock pieces. Also, T&R suggests that Berlogar evaluate the compaction criteria for deep fills. Specifically, whether 90 percent relative compaction for fills that are up to about 32 feet high is adequate.

T&R Comment No. 3
T&R suggests that Berlogar evaluate the seismic performance of proposed fill slopes within areas to receive new buildings and/or site improvements. Subsequently, Berlogar should select a foundation system(s) that is compatible with the estimated movement or settlement of the fill slope (if any), and capable of safely supporting the new residences.

T&R Comment No. 4
T&R suggests that Berlogar evaluate the potential for differential settlement between upper and lower levels of the proposed residences. Berlogar should also indicate whether seismic and surcharge lateral loads should be incorporated into the design of proposed below-grade walls.

T&R Comment No. 5
T&R suggests that Berlogar provide recommendations for mat foundation embedment or alternate mitigation to reduce the potential for surface water and moisture intrusion beneath the new structures.

T&R Comment No. 6
T&R suggests that Berlogar provide recommendations for a ground floor system to be used in conjunction with a pier and grade beam foundation.

T&R Comment No. 7
T&R suggests that Berlogar evaluate the potential for adverse impacts associated with scour on bridge and headwall foundations, and if necessary, provide mitigating recommendations.

T&R Comment No. 8
Berlogar should clarify the closest distance to the nearest fault (Rodgers Creek fault) and modify seismic design parameters (2001 CBC and Caltrans), if appropriate. Also, Berlogar should provide recommendations for the appropriate ARS curves for use in designing the vehicular bridge in accordance with Caltrans criteria.

<u>T&R Comment No. 9</u>
T&R recommends that a qualified engineering geologist and/or geotechnical engineer should be retained by the site developer to observe all landslide remediation activities, and check that all landslide debris within designated excavation areas has been properly removed and engineering fill, keyways, subdrains, and MSE retaining walls have been properly constructed.

<u>T&R Comment No. 10</u>
The surface and subsurface drainage systems need to be functioning properly in order to reduce the potential for slope movement, debris flows, and surface erosion. T&R recommends that Berlogar provide technical input during design and construction of surface and subsurface drainage systems, including temporary and permanent erosion control systems. Also, the homeowner's association or similar entity should be responsible for the periodic inspection and maintenance of surface and subsurface drainage systems for the proposed development.

In conclusion, T&R recommends the project applicant and/or Berlogar provide a response to the comments presented above. The City of Petaluma should be given an opportunity to review the responses, and comment on whether any outstanding issues still remain. T&R and GGI appreciates the opportunity to assist you with the evaluation of geological and geotechnical issues for this project. If you have any questions or require additional information, please call.

Sincerely yours,
TREADWELL & ROLLO, INC.

Dean H. Iwasa
Geotechnical Engineer

Attachment: References
Figures 1 through 5

cc: Mr. David Simpson, Gilpin Geosciences

Via Hand Delivery

December 16, 2004
Job No. 2616.001

BGC
BERLOGAR
GEOTECHNICAL
CONSULTANTS

Mr. Steve Abbs
Davidon Homes
1600 South Main Street, Suite 150
Walnut Creek, California 94596

Subject: Response to Geotechnical Peer Review Comments, plus Supplemental Recommendations
UOP Property
D Street
Petaluma, California

Dear Mr. Abbs:

We previously prepared a report dated September 22, 2004, titled *Revised Design-Level Geotechnical Investigation, UOP Property, D Street and Windsor, Petaluma, California.* We received a copy of a letter with the subject line *Third Party Geotechnical/Geological Review, Davidon Homes EIR, Petaluma, California*, dated November 11, 2004, by Treadwell & Rollo (T&R) that included six summary comments for our response. This letter was followed by a second letter, dated November 23, 2004, which included the initial six comments with minor revisions plus four new comments. Our responses to the ten comments by T&R are presented below.

T&R Comment No. 1, Debris Flows and Headwall

The civil design shows a headwall proposed behind Lots 78 and 79. The headwall, in addition to directing surface water from offsite into an onsite underground stormwater pipeline, would form a collecting basin for debris flow deposits. T&R recommends that BGC evaluate (A) the headwall's adequacy for resisting impact forces associated with debris flows and (B) if the catchment area is large enough to contain potential debris flow material. We respond to these two comments separately below:

A) The headwall would not be a free-standing wall; in other words, it would not be a catchment wall or deflection wall. The grading plan shows it as a typical civil engineering headwall, which would serve the dual purpose of retaining a vertical grade split (fill on the high northern side) and funneling stormwater runoff into a pipeline. The headwall would also form one side of a basin south of the wall, offsite, that could potentially collect debris flow material.

B) We evaluated the potential debris flow volume south of the site in comparison to the volume of the basin that will result from grading (filling for) Lots 78 and 79. We found the future catchment basin volume adequate, and we state this on Page 8 of our September 2004 report. Note that elevations at the top of the headwall will be approximately 220 feet versus a basin bottom elevation of approximate 207 feet: the basin will be approximately 13 feet deep.

T&R Comment No. 2, Handling of Large Rock Fragments in Fill and Compaction Criteria in Deep Fills (up to 32 feet)

Due to the presence of isolated, strongly cemented sandstone beds in the proposed cut areas, large rock fragment, up to 12 inches in size and greater, will likely be generated and used in the fills. As discussed in our report, we recommend that these larger rock fragments not be placed in concentrated masses, so that they can be surrounded with finer compacted materials as the fills are being made. To achieve this, we will be on site during grading operations to advise the contractor as the fill is being placed and confirm that the larger rock fragments are properly blended in the fill.

To help reduce post-construction settlement in deep fill areas, we recommend compaction criteria below depths of 20 feet be increased to 95 percent relative compaction and 3 percent over optimum moisture content. Based on our experience, the results of our laboratory hydro-compression test results, and the increased compaction criteria, post-construction settlement in the deepest fill areas should be minimal and should not have any adverse impacts on the proposed development.

T&R Comment No. 3, Seismic Performance of Fill Slopes with Respect to House Foundations

Site grading will be performed in conformance with the recommendations presented in our report, which should provide stable fill slopes and no more than minor differential settlement. Therefore, we believe either post-tensioned concrete slabs-on-grade and/or pier-and-grade-beam foundations would be suitable foundations for the project from a geotechnical standpoint. However, both foundation types have advantages and disadvantages with respect to cost and constructability on single and split-level building pads, which are planned for this project. Therefore, we are allowing these optional foundation types for the homebuilder to select from.

T&R Comment No. 4, Seismic and Surcharge Loads and Differential Settlement Between Upper and Lower Levels of Split Level Pads

We recommend that potential seismic and surcharge loads be incorporated into the designs of retaining walls. These loads should be calculated by the retaining wall designer and provided to us for review as part of the retaining wall plan review. Should additional surcharge loading or increases in the design seismic coefficients be warranted based on our review, we will advise the designer accordingly.

With regard to differential settlement between the upper and lower levels of split-level pads, we estimate about ½-inch of post-construction differential settlement over a horizontal distance of about 40 feet in the deepest fill areas, which should not have adverse impacts on the proposed house foundations. This estimate is based on the results of our laboratory hydro-compression/swell testing and the assumption that site grading will be performed per our recommendations.

T&R Comment No. 5, Recommendations to Reduce Potential for Surface Water and Moisture Intrusion Beneath New Structures

Surface water and moisture intrusion beneath structures is typically mitigated by providing positive drainage around the perimeters of structures are shown on civil plot plans and the structural engineer's foundation plans. It is our opinion that perimeter subdrainage will be needed if pier-and-grade-beam foundations are used. Detailed recommendations for perimeter subdrainage will be provided as needed based on our review of the foundation plans and our findings in the field during site grading and foundation construction.

T&R Comment No. 6, Recommendations for Ground Floor System to Be Used in Conjunction with Pier and Grade Beam Foundations

We recommend that concrete slabs-on-grade for garages and living areas be structurally tied to the perimeter grade beams. As discussed in our report, where the transmission of vapor through the slab is objectionable, the use of an underlying vapor retarder and capillary moisture break should be considered by the designer of the slab. The thickness of the slab, capillary break, and vapor retarder should be determined by the slab and floor covering designers.

T&R Comment No. 7, Scour Along Bridge Foundations

The two vehicle crossings planned at the project site will include pier-supported head walls at each end of culverts covered with compacted fill embankments. Piers are recommended because of the potential presence of compressible and/or erodible creek bottom materials. However, to help reduce the potential for scour at the juncture between the creek bottom native materials and the head wall foundations, we believe it would be more appropriate for the project civil engineer, if they feel one is warranted, to design an energy dissipater. We would perform a grading plan review and check this area as such to consult with the project civil engineer as to modifying the design from a geotechnical perspective.

T&R Comment No. 8, Seismic Design Parameters

The 2001 California Building Code (CBC) requires the use of fault locations published most recently by the U.S. Geological Survey or California Geological Survey. CGS formerly published fault locations in a document by Petersen et al. (1996). The coordinates of the northeast corner of the site, which is the site corner closest to the Rodgers Creek fault, are Latitude 38.220 and Longitude -122.644. The distance from the site corner to that previous location of this fault was 10.2 kilometers.

The California Geological Survey recently published a new set of California fault locations and their magnitudes (Cao et al. 2003). Based on that slightly revised location for the Rodgers Creek fault, the resulting site-to-fault distance is 8.7 kilometers. We recommend that a distance of 8.7 kilometers be used in seismic designs for the UOP Petaluma site.

With regard to the design of vehicle bridges, the appropriate design ARS curves should be developed by the bridge designer. To help establish the appropriate ARS curves, the Caltrans seismic design criteria presented on page 19 of our September 2004 report may be used, with the exception of the site-to-fault distance which we have updated to 8.7 km as discussed above.

T&R Comment No. 9, Observation of Landslide Remediation and MSE Retaining Wall Construction

It is our common practice to have our engineering geologist observe landslide removals and associated keyway excavations and subdrain installation for landslide remediation, and we will do so on this project. Similarly we will provide geotechnical observation and testing services during the construction of MSE retaining walls to confirm that the geotechnical aspects have been properly implemented in their construction.

T&R Comment No. 10, Technical Input During Design and Construction of Surface and Subsurface Drainage Systems and Erosion Control

Our September 2004 report includes our geotechnical input for the design of surface and subsurface drainage in the construction of landslide remediation, fill slopes, re-built cut slopes, and retaining wall construction. During site grading we will be present to confirm that our recommendations are adhered to. Additionally, we have reviewed the surface drainage aspects in the project grading plans, which appear to be acceptable from a geotechnical perspective. If, in our opinion, additional surface drainage elements are warranted as site grading progresses, we will make additional recommendations at that time.

Temporary and permanent erosion control measures should be provided by the project civil engineer. As a minimum, we have recommended (September 2004 report, page 10) that finish graded slopes be planted with deep-rooted, fast-growing vegetation. We will provide additional geotechnical input, if needed, when we review the erosion control measures provided by the project civil engineer.

Summary of Supplemental Geotechnical Recommendations

The supplemental geotechnical recommendations discussed in the text of this letter have been summarized below to be used in conjunction with the recommendations presented in our September 2004 geotechnical report for the design and construction of the subject project.

1. Fills deeper than 20 feet should be compacted to a minimum 95 percent relative compaction at not less than 3 percent over optimum moisture content. Compaction criteria should be based on the ASTM D1557-00 laboratory test method.

2. Perimeter subdrainage should be included with pier and grade beam foundation systems. Details for perimeter subdrainage will be provided when we have an opportunity to review the project foundation plans.

3. Ground floor systems or concrete slabs-on-grade to be used in conjunction with pier and grade beam foundations should be structurally tied to perimeter grade beams. Where the vapor transmission through the slab would be objectionable, the use of a vapor retarder and capillary moisture break should be considered by the designer of the slab and floor covering. The thickness of the slab, capillary break, and vapor retarder should be determined by the slab and floor covering designers.

4. Our certified engineering geologist should be allowed to observe landslide removals and associated keyway excavations and subdrain installation for landslide remediation. Additionally, retaining wall construction should be observed by our firm to confirm that our geotechnical recommendations are properly incorporated.

If you have any questions regarding this letter, please do not hesitate to call one of us.

Respectfully submitted,

BERLOGAR GEOTECHNICAL CONSULTANTS

Michael G. Matusich
Project Engineer
C.E. 62536, Exp. 12/31/05

REGISTERED PROFESSIONAL ENGINEER
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Frank Berlogar

MGM/FJG/FB:fjg\pv

Copies: Addressee (3)

word/letter/15575

Treadwell&Rollo

21 December 2004
Project 3965.01

Mr. Geoffrey A. Reilly
Christopher A. Joseph & Associates
101 H Street, Suite Q
Petaluma, California 94952

Subject: Third Party Geotechnical/Geological Review
Davidon Homes EIR
Petaluma, California

Dear Mr. Reilly:

This letter acknowledges our receipt of the electronically mailed letter titled *Response to Geotechnical Peer Review Comments, plus Supplemental Recommendations, UOP Property, D Street, Petaluma, California*, prepared by Berlogar Geotechnical Consultants (Berlogar), dated 16 December 2004. The Berlogar letter provided responses to geotechnical/geological comments described in our third party review letters dated 11 and 23 November 2004. We reviewed the Berlogar responses and supplemental recommendations and concluded the responses satisfactorily address our concerns and no further response is required. However, it should be noted that some of the proposed mitigation will require reviewing and approving the final plans and details, and providing geotechnical/geological consultation based on field observations during construction. To ensure the applicant's geologic and geotechnical consultants are given the opportunity to participate in the final design and construction phases of the project, we suggest the City of Petaluma require the owner's consultants (Registered Geotechnical Engineer and Registered Engineering Geologist) to review and approve the final grading, drainage, and foundation plans and specifications. Also, upon completion of construction activities, the owner's consultant should provide a final statement indicating whether the work was performed in accordance with project plans and specifications, and the consultant's recommendations.

If you have any questions or require additional information, please call.

Sincerely yours,
TREADWELL & ROLLO, INC.

Dean H. Iwasa
Geotechnical Engineer

REGISTERED PROFESSIONAL ENGINEER
DEAN H. IWASA
GE 2285
Exp. 3/31/06
GEOTECHNICAL
STATE OF CALIFORNIA

cc: Mr. David Simpson, Gilpin Geosciences

Treadwell & Rollo, Inc. *Environmental & Geotechnical Consultants*
501 14th Street, Third Floor, Oakland, CA 94612
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Treadwell&Rollo

21 December 2004
Project 3965.01

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Christopher A. Joseph & Associates
101 H Street, Suite Q
Petaluma, California 94952

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If you have any questions or require additional information, please call.

Sincerely yours,
TREADWELL & ROLLO, INC.

Dean H. Iwasa
Geotechnical Engineer

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Gilpin Geosciences, Inc
Earthquake & Engineering Geology

October 30, 2012
3965.01

Mr. Geoff Reilly
WRA, Inc.
2169-G East Francisco Blvd.,
San Rafael, CA 94901

Subject: Geological Site Review Update
Davidon Homes Administrative Draft EIR Comments
UOP Property D Street
Petaluma, California

Dear Mr. Reilly:

We are pleased to present this update letter at your request. We have performed the following tasks based on our 22 June 2012 proposal:

- Review the existing reports and EIR;
- Review aerial photography dated 12/31/05; 8/5/06; 6/30/07; 6/6/09; 8/25/09; 9/16/10; 5/7/12, more recent than previously reported; and,
- Perform a site reconnaissance to observe present site conditions.

We visited the site on 29 October 2012 to review site conditions that were originally mapped by our staff geologist on 12 August 2004. We reviewed Google Earth photography archives for images of the site since 2004 because the most recent vertical stereo-paired photography available from Pacific Aerial Surveys is 2005.

We found no significant changes in the state of the site slope stability or level of erosion. We concur with the findings regarding the site geology presented in the Draft EIR and concur with the recommended mitigations. If you have any further questions or require clarification, please call.

Sincerely,
GILPIN GEOSCIENCES. INC.

REGISTERED GEOLOGIST
L. M. GILPIN
No. 1518
CERTIFIED ENGINEERING GEOLOGIST
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Lou M. Gilpin, EG, PhD
Engineering Geologist

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DESIGN LEVEL GEOTECHNICAL INVESTIGATION
OPTION A – 66 LOTS
OPTION B – 63 LOTS
SCOTT RANCH
D STREET AND WINDSOR DRIVE
PETALUMA, CALIFORNIA

FOR
DAVIDON HOMES
April 28, 2014

Job No. 2616.006

BERLOGAR STEVENS & ASSOCIATES

Via E-Mail and Mail

April 28, 2014
Job No. 2616.006

BERLOGAR
STEVENS &
ASSOCIATES

Mr. Jeff Thayer
Davidon Homes
1600 South Main Street, Suite 150
Walnut Creek, California 94596

Subject: Design-Level Geotechnical Investigation Report
Option A – 66 Lots
Option B – 63 Lots
Scott Ranch
D Street and Windsor Drive
Petaluma, California

Dear Mr. Thayer:

INTRODUCTION

This report contains the results of our Design-Level Geotechnical Investigation for the Option A – 66 Lots and Option B – 63 Lots plans for Scott Ranch located at the intersection of D Street and Windsor Drive in Petaluma, California. The site is shown on the Vicinity Map, Plate 1. The field investigation and laboratory testing utilized in this report were conducted in 2004 for a 93-lot plan under consideration at that time.

PURPOSE AND SCOPE OF SERVICES

The purpose of the investigation has been to characterize the engineering properties of soil and bedrock at the site and provide design-level geotechnical recommendations for site development. The scope of services for this project included:

1. Review of previous information covering the site and vicinity,
2. Review of stereo-paired aerial photographs covering the site and vicinity,
3. Site geologic reconnaissance and mapping,
4. Drilling and logging of 14 borings,
5. Excavation and logging of 36 backhoe test pits,
6. Laboratory testing of selected representative samples collected during the field investigation,
7. Engineering and geologic analysis, and
8. Preparation of this report.

PROPOSED DEVELOPMENT

The site includes two parcels totaling about 58.5 acres that are separated by Windsor Drive. The site is bound by residential developments to the north and west, by D Street to the east, and open land to the south. We have received Option A – 66 lot plan and Option B – 63 lot plan from BKF via electronic file, that show the site being developed into 66 or 63 single-family residential lots separated by existing creek channels. The existing creek channels are to remain undeveloped open space. New streets providing access to the site are to branch off of Windsor Drive and D Street. An existing stock pond and berm located in a swale in the southern portion of the site are to remain. In order to achieve design grades, cuts of up to about 25 feet and fills of up to about 15 feet are planned. The design grading will result in cut slopes up to about 60 feet tall and fill slopes up to about 30 feet high. Retaining walls up to 5 feet high are planned to achieve design grades. A developed trail will be constructed on the South side of Kelly Creek with a trailhead and bathroom facilities. The trail extends on site to the border of Helen Putnam Regional Park. There is an existing barn and 2 accessory structures. Option A proposes to relocate the barn only to a site on the eastern property line at D Street. The accessory structures will not be retained in Option A. Option B will leave the Barn and accessory structures in place. In both option A and Option B, a parking lot of approximately 45 spaces accessing the trailhead facilities is proposed.

FIELD INVESTIGATION

The field investigation was conducted between March 28 and April 3, 2003. The investigation included a site reconnaissance, geologic mapping of creek bank exposures, drilling and logging of two rotary wash borings (B-1 and B-2) to depths ranging from 24 to 502 feet, drilling and logging of 12 auger borings (B-3 through B-14) to depths ranging from 8 to 22 feet, and excavation and logging of 36 backhoe test pits (TP2-1 through TP2-36) to depths of up to 14 feet. Materials encountered in the borings and test pits were visually classified and logs were recorded. Bulk and relatively undisturbed samples of bedrock and soils were collected from the borings and test pits for laboratory testing.

Where ground water was encountered, the borings were backfilled with neat cement grout in accordance with Sonoma County requirements. Borings that did not encounter ground water were backfilled with soil cuttings. Test pits were loosely backfilled with excavated materials at the completion of logging. The locations of borings and test pits are shown on the attached Geologic Map. Boring logs are presented in Appendix A (Plates A-1 through A-20), a Key to Boring Symbols is included as Plate A-21, and Rock Description is presented as Plate A-22. Test Pit Logs are also included in Appendix A (Plates A-23 through A-31).

FINDINGS

SURFACE CONDITIONS

Site topography ranges from about 100 feet above mean sea level (msl) in the eastern portion of the site to about 380 feet near the southwest corner of the site. The site contains a relatively flat alluvial plain in the central portion of the site that is bordered by moderately steep bedrock slopes to the north and south. Kelly Creek crosses the site in an east-west direction and intersects an unnamed tributary that crosses the eastern portion of the site in a north-south direction near D Street. Two drainage gullies cross the central plain and drain to Kelly Creek. Drainage at the site flows to the northeast and enters an existing box culvert beneath D Street.

Existing site improvements include the remains of a wood-framed house, a vacant mobile home and the barn and accessory structures located near D Street. An open, concrete-lined water well about 3 feet in diameter and about 15 feet deep, is located beneath the trees along the edge of the westernmost drainage gully on the south side of Kelly Creek. Water in the well was roughly at the ground surface. A stock pond and berm are located in a swale south of Kelly Creek. An existing storm drain outlet is located on the southwest side of D Street, about 5 feet east of the property line.

REGIONAL GEOLOGY

The site is situated along the southwest margin of the Petaluma River valley. This valley is part of a series of small basins and ranges characteristic to the Coast Ranges geomorphic province of California. In this portion of the province, the oldest bedrock consists of sedimentary and meta-volcanic rocks of the Franciscan Complex which were deposited during the Jurassic and Cretaceous Periods of geologic time (about 65 to 208 million years before present). Small lenses of sheared and/ or altered bodies of rock are inherent to the Franciscan Complex. Tertiary aged (10.6 to 65 million years before present) volcanic rocks are present in scattered patches throughout the region (Blake et al., 1974).

Bedrock in this region has been folded and faulted during the past several million years due to relative strike-slip and convergent motion between tectonic plates. Much of the deformation (shearing, faulting and folding) of the Franciscan Complex occurred during past convergent plate motions. Most convergent plate motion in the region ended millions of years ago. This deformation is believed by many researchers to be intrinsic to the Franciscan Complex and is separate from the active strike-slip fault motion in the region.

SUBSURFACE CONDITIONS

During the course of this investigation, we encountered artificial fill, landslide deposits, colluvium, alluvium, and bedrock units of the Franciscan Complex. A description of each material excluding landslide deposits which are discussed separately are listed in order from youngest to oldest as follows:

ARTIFICIAL FILL

Isolated areas of artificial fill at the site were encountered in three main areas: beneath and around existing buildings, the stock pond earthen berm, and along the downslope (south) side of Windsor Drive beneath D Street. Fill beneath and around existing structures encountered in Test Pits TP2-35 and-36 was found to consist of dense sandy silt and gravel that extended to a depth of about 1 foot. The stock pond berm fill encountered in Boring B-5 was found to consist of stiff to very stiff silty clay that extended to a depth of about 12 feet. Fill along the downslope edge of Windsor Drive and D Street is assumed to have been engineered along with roadway construction and was not investigated. Areas of artificial fill are delineated by the symbol "Qaf" on the Geologic Map.

COLLUVIUM

Areas of soil accumulation referred to as colluvium are present in the lower portions of the site. Colluvium is material that is generated by the in-place weathering of underlying bedrock on a slope and then migrates downslope under the influence of gravity. Colluvium mantles all slopes to some degree and forms particularly thick deposits at the toes of slopes and in swales. At the site, colluvium was found to be brown to light red-brown, stiff to very stiff silty clay with minor amounts of gravel. Laboratory testing suggests that the colluvium on site is moderately expansive. Areas of colluvium thicker than a few feet are delineated on the Geologic Map by the symbol "Qc."

ALLUVIUM

Alluvium is material that has been transported and deposited by way of flowing water. At the site, alluvium was found to consist of orange-brown to yellow-brown sandy clays and clayey sands with various amounts of gravel that are stiff to very stiff and medium dense to dense. Alluvium is generally found in relatively flat lying areas bordering drainage courses and at the downslope end of swales as shown on the Geologic Map by the symbol "Qal." Based on the information provided by the borings and creek exposures, the alluvium unit reaches a maximum thickness of about 25 feet at a point about half way between the Kelly Creek channel and the base of the hills to the south. Laboratory testing suggests that the alluvium on site is moderately expansive.

SHEAR ZONE MATERIAL

Three shear zones were encountered during our current and previous investigations: one north of Windsor Drive, and two located in the southwest portion of the site. The shear zones at the site are not related to the active regional strike-slip system of faulting. The shear zones at the site are interpreted as deformation concentrated within the relatively weak shale. The deformation likely occurred as flexural slip during regional folding resulting from the past convergent tectonic regime.

Our previous investigation described material within the shear zone as containing serpentine minerals. Serpentine minerals are often found in ultramafic rocks. Based on the test pits excavated within shear zones during this investigation (TP2-2, TP2-21, and TP2-31), ultramafic rocks were not encountered. We also re-excavated test pits from our previous investigation (TP-17 and TP-20) and reclassified the shear zone materials. It was found that the shear zone materials are composed of sheared clayey shale, and no serpentine minerals or ultramafic rocks were encountered. The gray-green alteration colors previously described are interpreted to be the result of a localized chemical reduction of the clayey material. Based on these findings, we conclude that the potential for significant volumes of serpentine-bearing ultramafic rocks being present at the site is low.

FRANCISCAN COMPLEX BEDROCK

Bedrock at the site consists of sandstone and shale of the Franciscan Complex. The sandstone was found to be moderately strong to strong and highly fractured with scattered areas of very strong-cemented beds. The shale is weak to moderately strong, thinly laminated, and crushed to sheared. Where sheared, the shale was weathered to clay and displayed a faint residual bedrock structure. Bedding was found in general to strike northwest and dip southwest at inclinations between about 33 and 73 degrees.

LANDSLIDES

A total of 18 landslides were mapped within the site, and are shown and designated as Landslides A through R on the Geologic Map. Landslides A, B, C, D, and G are located on the flanks of the hillsides in the southern portion of the site. Landslides E, F, and H are located on the flank of the large bedrock knob in the northwest portion of the site. The remaining landslides (Landslides I through R) are located along the banks of Kelly Creek and are the result of typical creek bank oversteepening. Landslides encountered at the site are relatively shallow with depths up to about 15 feet and are believed to involve soils and the upper 2 to 3 feet of highly weathered bedrock.

FAULTING

The site is not located within a State of California designated earthquake fault zone for active faults (Davis, 2000; Hart and Bryant, 1982). The State of California considers a fault active if it has demonstrated Holocene activity (within the past 11,000 years). We did not encounter evidence of an active fault crossing or trending toward the site.

The table below lists the seven known active faults believed to present the highest potential levels of ground shaking at the site, their distances from the site, and their potential maximum moment-magnitude earthquakes. The faults in the table are arranged in order of their decreasing potential level of ground shaking at the site.

SIGNIFICANT POTENTIAL FAULT EARTHQUAKE SOURCES IN SITE VICINITY			
Fault Name	Approx. Distance to Fault Trace (mi)[a]	Compass Direction to Fault	Maximum E.Q. mag. (Mw)[b]
Rodgers Creek	7	NE	7.0
San Andreas, 1906 Rupture	14	SW	7.9
Hayward, Total Length	18	SE	7.1
San Gregorio	23	S	7.3
Point Reyes	22	SW	6.8
West Napa	19	E	6.5
Maacama, south	25	N	6.9

1. Potential fault earthquake sources given by Peterson et al. (1998).
2. Maximum earthquake moment magnitude calculated by Peterson et al. (1998).

GROUNDWATER

Groundwater was encountered in Test Pit TP2-7 at a depth of about 2 feet and is likely the result of the storm drain outfall next to D Street. Groundwater was encountered in Borings B-1, B-5, B-7, B-8, B-12, and B-13 at depths of about 21, 17, 7, 14½, 17½, and 9½ feet, respectively. Marshy ground and groundwater seepage have been observed in various places across the site mainly following periods of higher rainfall. Areas of perched groundwater are expected in the lower portions of the site. Groundwater levels are expected to undergo significant fluctuations based on seasonal rainfall and time of year.

CONCLUSIONS AND RECOMMENDATIONS

GENERAL

From a geotechnical standpoint, the proposed residential development can generally be constructed as planned, provided the conclusions and recommendations contained within this report are incorporated into the project design and construction. The primary geotechnical issues for site development are landslide remediation, treatment of existing fill, fill slope construction, stability of proposed cut slopes, and the potential for expansion and settlement of on-site earth materials.

LANDSLIDE REMEDIATION

The recommended remedial treatment of landslide hazards is dependent on many factors such as the size of the landslide, the landslide's spatial relationship to proposed improvements, and the individual characteristics of each landslide. In general, the preferred remedial measure from a geotechnical standpoint is complete removal of landslide debris located within the development area. A number of factors can make complete removal of landslide debris impractical, such as property line limitations or the presence of trees. Provided the risks associated with movement of part of a given landslide located outside the development are acceptable, the potential adverse impacts to the planned development can be minimized by implementing remedial measures such as construction of engineered fill, below-grade MSE walls, and catchment areas.

Landslides A, D, and E will require remedial treatment for the currently planned development. Landslide F will be removed with the design cut. Landslides B, C, G, and H are located outside of the planned development area and require no remediation. Similarly, landslides located along Kelly Creek (I through R) do not impact the development and require no remediation. We recommend that landslides be treated as summarized in the following table:

RECOMMENDED LANDSLIDE REMEDIATION SUMMARY

Landslide Designation	Est. Ave. Thickness (feet)	Relationship of Landslide to Proposed Development	Recommended Remedial Measures
A	12	Within and upslope of limit of grading	• Remove portion within development and replace with engineered fill with proper subdrainage. (See Plates 5 and 6, Remedial Grading Plans) • Construct a 40 feet wide (minimum) keyway with proper subdrainage.
B	9	In Open Space	• None Required

RECOMMENDED LANDSLIDE REMEDIATION SUMMARY			
Landslide Designation	Est. Ave. Thickness (feet)	Relationship of Landslide to Proposed Development	Recommended Remedial Measures
C	6	Outside limit of grading	• None required
D	4	Within limits of grading	• Remove and replace all landslide debris with engineered fill and proper subdrainage.
E	6	Within limits of grading	• Remove portion within development and replace with engineered fill provided with proper subdrainage. • Construct a 20 feet wide (minimum) keyway.
F	7	Within limits of grading	• None required (removed by design grading).
G	10	Outside limit of grading	• None required.
H	10	Outside limit of grading	• None required.
I through R	3 to 5	Along creek bank	• None required.

DEBRIS FLOW/SEDIMENTATION POTENTIAL

Debris flow potential was identified in the southwestern drainage courses. These drainage courses will remain in open space; thus, remedial treatment is not required.

GRADED SLOPES

CUT SLOPES

All cut slopes should be inspected at the time of construction by an engineering geologist focusing on evidence of potential instability. Cut slopes should be constructed at gradients no steeper than 2H:1V. Where cut slopes over 30 feet in height are planned, intermediate surface benches should be spaced no more than 25 feet vertically on the slope. The benches should be a minimum of 8 feet wide and include a concrete lined V-ditch to intercept surface water runoff.

Based on bedding attitudes measured in test pits, areas of adverse bedrock structure were not encountered at the locations of proposed cut slopes. However, due to folding and shearing of the bedrock, localized areas of adverse bedrock structure or other zones of geologic weakness could be exposed during grading of cut slopes. If areas of adverse bedrock structure are encountered, we anticipate that the remedial measures for these slopes will involve overexcavation of the affected portion of slope and construction of a slope buttress with appropriate subdrainage. We should provide specific remedial design recommendations based on the conditions exposed in areas of concern identified during grading.

FILL SLOPES

The stability of proposed fill slopes is dependent on proper keyways, benching, subdrainage, fill compaction, and slope gradient. Fill slopes should be constructed at gradients no steeper than 2H:1V. Fill slopes should be overbuilt and cut back to expose firm compacted materials. Fill slopes should be constructed with a 6 feet deep (minimum) keyway with a width equal to ½ the slope height or 20 feet, whichever is greater, and provided with proper subdrainage. All keyway excavations should be mapped by an engineering geologist prior to backfilling. Typical Fill Slope Details are presented as Plate 5.

All cut and fill slopes should be planted with fast growing, deep-rooted vegetation before the first winter to reduce erosion. Consideration should be given to the irrigation of some slopes; specific details regarding irrigation systems, locations, and discharge should be reviewed by this office prior to their approval.

TREATMENT OF EXISTING FILL

From a geotechnical standpoint, the on-site existing fill is considered suitable for re-use as engineered fill provided it is free of rock fragments greater than 12 inches in size and deleterious material. Inasmuch as the proposed development does not encroach onto these fill areas, based on our field observations, existing fill along D Street and Windsor Drive does not require additional treatment. All other fill located at the site (with the exception of the stock pond berm) should be completely removed and reworked as engineered fill.

Test pits excavated during our current as well as previous investigations were loosely backfilled with excavated materials. Where not removed by design cut, loose backfill at the test pit locations should be subexcavated and replaced with engineered fill.

The existing stock pond is to remain as-is in open space. Remedial treatment of the stock pond is no longer recommended because the stock pond and the area downhill of the stock pond are in planned open space and no longer in close proximity to planned residential construction.

SUBDRAINAGE

Ground water seepage is expected to occur in swales, at the bases of slopes, and in isolated pockets in the lower portions of the site. Subdrainage should be provided to intercept ground water in the following locations:

1. On the uphill side of all keyways and proposed fill,
2. Along swales and gullies to receive fill,
3. At all springs and seepage areas,
4. At the toes of major cut slopes,
5. At geologic contacts known to transmit water, and
6. In other areas of the site where seepage is observed during and after grading or as determined in the field by the soil engineer.

Subdrains should consist of perforated PVC pipe conforming to ASTM D 2751, Type SDR 35. Subdrains should be at least 6 inches in diameter. All subdrains should be surrounded by and underlain by at least 6 inches of Class 2 Permeable Material as defined in Section 68-1.025 of the Caltrans Standard Specifications (July 1999). Subdrain trenches should be at least 18 inches wide and at least 4 feet deep. Final trench configurations should be approved by the soil engineer. Subdrain trenches should be capped with engineered fill or topsoil, depending on the location of the subdrain. Subdrain systems should be discharged into a storm drain structure (manhole, inlet) where possible. Subdrain details are provided on Plate 8.

Some areas of seepage may develop after house construction is completed. Additional subdrains will likely be needed in these areas should seepage develop.

EXCAVATION CHARACTERISTICS

Conditions encountered during our field investigations at the site as well as our experience in the area suggest that, in general, excavation to planned depths should be achievable using conventional grading equipment. Based on the high degree of fracturing and the fracture spacing encountered in the test pits, and the rock quality designation (RQD) logged in Boring B-2, we believe that large grading equipment such as a Caterpillar dozer D-10 with rippers should be adequate. Areas of very hard bedrock should be anticipated in deep cut areas at the site that are likely to generate oversize material. Modified excavation techniques such as using a single shank on a D-10 should generally be capable of ripping very hard-cemented areas of bedrock. Areas of hard rock were encountered in Boring B-2 and Test Pits TP2-2 through TP2-4, TP2-8, and TP2-10.

SELECTIVE GRADING

Special care should be taken to reduce the size of bedrock derived fill material so that the material can be properly compacted. Oversized material (greater than 6 inches) is expected to be generated from bedrock cuts at the site. Oversize material can be broken down mechanically or placed in deeper areas of fill and not within 10 feet of pad grade or street subgrade. Oversize material to be used in deeper areas of fill should be spread out so that large rocks are not concentrated in pockets and are surrounded by engineered fill. Placement of oversize material should be subject to approval by the soil engineer.

SITE PREPARATION AND GRADING

All grading operations should be performed in accordance with the following recommendations:

1. Existing earth materials on-site are considered suitable for re-use as engineered fill provided it does not contain rock fragments greater than 6 inches and is free of deleterious material as determined in the field by the soil engineer.
2. If import fill is used, it should have a Plasticity Index (PI) less than 12 and should be subject to evaluation and approval by the soil engineer prior to use.
3. All fill materials to be used at the site should be subject to evaluation and approval by the soil engineer prior to use.
4. Areas to be graded should be cleared and stripped of all vegetation. Strippings can be stockpiled and re-used as topsoil in landscape areas. Strippings can also be blended with clean on-site soils at a ratio of 10 loads of clean soil to 1 load of strippings, to create a soil mixture suitable for use as engineered fill.

5. Existing foundations, wells, septic systems, leach fields (and any similar subsurface structures) should be completely removed prior to grading. Any soft soils encountered during excavation should be removed as determined in the field by the soil engineer.

6. The upper three feet of soil in areas mapped as colluvium should be reworked as engineered fill. This depth of reworking can be reduced as discussed under *Colluvium/Alluvium Overexcavation* below.

7. Low-expansion-potential bedrock cut derived material should be used in keyways for landslide remediation and buttress fill slopes.

8. Where zones of soft or saturated soils are encountered during excavation and compaction, deeper excavation may be required to expose competent materials. This should be determined in the field by the soil engineer.

9. Areas to receive fill should be scarified to a minimum depth of 12 inches, brought to at least 3 percent over optimum moisture content, and compacted to not less than 90 percent relative compaction.

   Relative compaction refers to the in-place density of a soil expressed as a percentage of the maximum dry density determined by Test Method ASTM D1557. Optimum moisture is the water content (percentage by weight) corresponding to the maximum dry density.

10. If significant subgrade pumping and/ or yielding occur during scarification or recompaction, it may be necessary to stabilize the exposed subgrade. The actual stabilization method, if warranted, will depend on exposed conditions and should be judged suitable by the soil engineer.

11. Fill should be placed in thin lifts (normally 6 to 8 inches thick, depending on compaction equipment used), moisture conditioned to at least 3 percent over optimum moisture, and compacted to at least 90 percent relative compaction. Modification to acceptable lift thickness should be determined in the field by the soil engineer and based on the demonstrated compaction performance during fill placement, which will depend on the equipment and methods used.

12. Fill placed on ground sloping greater than 7H:1V should be benched into firm materials as determined in the field by the soil engineer.

13. Fill slopes should be over built and cut back to expose a firm compacted surface.

14. Observation and soil density testing should be performed during grading to assist the contractor in achieving the required degree of compaction and the proper moisture content. Where compaction is less than required, additional compactive effort should be made with an adjustment in the moisture content where necessary until the specified compaction is obtained.

15. The soil engineer should be informed at least 48 hours prior to any grading operation. The procedures and methods can then be discussed between the developer, contractor, and soil engineer. This can facilitate the performance of grading operations and minimize potential construction delays.

CUT/ FILL TRANSITION LOT TREATMENT

Because the proposed fill and bedrock at the site will have different expansion and settlement potential, structures and slabs placed across the transition line between cut and fill could experience significant differential expansion and/ or settlement. This condition can be mitigated by overexcavating the cut portion of the cut/ fill transition lots to a depth of about 3 feet below rough pad grade. The exposed excavation bottom should then be scarified to a minimum depth of 12 inches, properly moisture conditioned to not less than 3 percent over optimum moisture content and compacted to at least 90 percent relative compaction. The overexcavation should be restored with engineered fill. Typical Cut/Fill Transition Lot Overexcavation Details are provided on Plate 9.

The horizontal and vertical extent of overexcavation should be determined in the field by the soil engineer. We recommend that the contract documents provide for add-and-deduct unit prices for excavation and replacement as engineered fill to allow for unanticipated variations in excavation quantities.

BEDROCK CUT LOT TREATMENT

Cut lots that have subgrades exposing bedrock should be overexcavated and recompacted a minimum depth of 3 feet. The exposed surface should be scarified to a depth of about 12 inches, moisture conditioned to not less than 3 percent over optimum moisture content, and compacted to at least 90 percent relative compaction. This is to allow for easier excavation of utility trenches and planting of vegetation.

COLLUVIUM/ ALLUVIUM OVEREXCAVATION

Depending on the time of year that grading operations occur at the site, it may be necessary to rework the upper 3 feet of areas mapped as colluvium and alluvium prior to placement of fill. The necessity to rework these areas will depend on the presence of desiccation cracks in the soil. Desiccation cracks in these types of soils often extend to a depth of about 3 feet and occur late in the dry seasons as the soil moisture content decreases. We anticipate that the upper about 12 feet will be reworked during normal stripping and scarification processes. If desiccation cracks extend below the depth of scarification, additional reworking will be required as determined in the field by the soil engineer. The need for additional reworking of colluvium and alluvium can be reduced if grading occurs early in the grading season, prior to drying of the soil and the formation of desiccation cracks.

EXPANSION POTENTIAL

As indicated by the results of our Atterberg limits and single-point consolidation/swell tests on the on-site soil and bedrock materials, the expansion potential of the on-site soil material is generally moderate. The total swell of fill placed and compacted following the recommendations presented under *Site Preparation and Grading* are estimated as follows:

ESTIMATED POTENTIAL SWELL OF COMPACTED FILL	
Fill Thickness (feet)	**Swell (inches)**
5	¾
10	1
15	1¼

The above preliminary estimates of potential swells are based on a uniform mixture of soil and bedrock generated from the design cuts planned at the site. The actual swell in fill areas will depend on the total depth of fill, the depths of placement of various materials in the fill, and the in-place moisture content and density. The maximum fill slope planned for this site is approximately 30 feet as measured from top of slope to toe of slope. The maximum depth of fill as measured vertically at the top of fill slope is approximately 15 feet. Swell of 1¼ inch measured vertically over the 15 feet maximum fill depth is 0.7% of the fill depth. This minor swell percentage is judged insignificant.

SETTLEMENT

The results of single-point consolidation tests on remolded soil samples from the site, representing proposed fill, are summarized in Appendix B. Based on these results, we estimate that on-site soil and bedrock materials used as fill will undergo some settlement during placement and for a duration following mass grading. The total settlement of the fill placed and compacted following the recommendations presented under *Site Preparation and Grading* are estimated as follows:

PRELIMINARY ESTIMATED POTENTIAL SETTLEMENT OF COMPACTED FILL	
Fill Thickness (feet)	**Preliminary Estimate of Total Settlement (inches)**
5	¼
10	½
15	1

Based on our laboratory test results and our experience, we anticipate that about 70 percent of the estimated total settlement of the fill should occur during mass grading. Therefore, we estimate that the maximum post-grading settlement should be less than 1 inch. The maximum fill slope planned for this site is approximately 30 feet as measured from top of slope to toe of slope. The maximum depth as measured vertically at the top of fill slope is approximately 15 feet. Settlement of 1 inch measured vertically over the 15 feet fill depth is 0.6% of the fill depth. This minor settlement percentage is judged insignificant.

SETTLEMENT OF COLLUVIUM AND ALLUVIUM

In some areas at the site, up to about 15 feet of fill is planned at locations underlain with colluvium and alluvium extending to depths of about 25 feet down to bedrock. Based on our boring log data and the results of our laboratory testing, we believe that the colluvium and alluvium at the site consist of stiff to very stiff, silty to sandy clays and clayey sands. Settlement of these deposits should take place upon application of the new fill loads, and should be on the order of less than about 1 inch. This settlement should not adversely affect the proposed development.

RESIDENTIAL FOUNDATIONS

GENERAL

The site soils generally consist of stiff colluvial and alluvial soils with shallow bedrock between 5 to 20 feet deep. Provided the grading recommendations presented in this report are adhered to, the proposed homes may be supported on either structural mat/slab or drilled cast-in-place concrete pier and grade beam foundations. Recommendations and design parameters for these foundation types are as follows:

STRUCTURAL MAT/SLAB FOUNDATIONS

Structural mat/slab foundations may consist of either conventional reinforced or post-tensioned concrete slab foundations. The slab foundation should be designed by a structural engineer to accommodate 2 inches of total soil movement and 1 inch in 25 horizontal feet of differential soil movement without structural distress to the slab and excessive deflections in the building framing and wall finishes. We recommend that the following criteria be incorporated in the design of the slab foundations:

Allowable Bearing Capacity (may be increased by a for seismic and wind load)	1,500 psf
Passive Equivalent Fluid Pressure (neglect the upper 1 foot if ground surface is not confined by slabs or pavement)	300 psf
Base Friction Coefficient	0.3
Minimum Interior Span	15 feet
Minimum Perimeter Cantilever	5 feet
Edge Variation Distance Center Lift Edge Lift	 9.0 feet 4.8 feet
Differential Swell Center Lift Edge Lift	 0.78 inch 1.14 inch
Minimum Slab Thickness	10 inches

The upper 12 inches of subgrade soil should be presaturated to at least 5 percent above optimum moisture content. The presaturated pad should not be allowed to dry out to less than this recommended moisture content prior to the construction of the slab.

Where moisture vapor transmission through the slab would be objectionable, the use of a vapor retarder and capillary moisture break should be considered by the designer of the slab and floor covering. The thickness of the slab, capillary break, and vapor retarder should be determined by the slab and floor covering designers.

PIER AND GRADE BEAM

Drilled cast-in-place reinforced concrete friction piers and grade beams are suitable foundation support for the proposed homes. Foundation support would be provided by skin friction between the pier shaft and surrounding soil. The reinforced concrete piers and grade beams should be designed by a structural engineer with the following minimum parameters.

Minimum depth below finish soil pad grade (feet)	8
Minimum diameter (inches)	12
Minimum pier spacing	6 pier diameters measured center-to-center
Allowable skin friction (psf)	450
Passive pressure (pcf, equivalent fluid pressure)	300
Minimum Grade beam embedment (inches)	6

Skin friction should be neglected in the upper 1 foot below adjacent grade. Passive pressure should be neglected in the portion of pier shaft that is less than 10 horizontal feet from a slope face. The recommended friction and passive pressure values may be increased by 1/3 for short-term wind and seismic effects. The minimum parameters above are preliminary in nature and should be re-evaluated as site grading exposes soil and bedrock conditions at the locations where drilled pier foundations are being considered.

Prior to placement of reinforcing steel and concrete, the bottom of the pier excavations should be free of excess loose soil and debris. Water that has collected in pier hole excavations should be pumped out or displaced by means of a tremie method.

RETAINING WALLS

Retaining walls, up to about 5 feet high, are planned at grade breaks between lots and at toes of slopes. We recommend that the following geotechnical criteria be incorporated in the design of retaining walls:

Active Equivalent Fluid Pressure Level Backfill Sloping Backfill	 50 pcf 65 pcf
At-rest Equivalent Fluid Pressure	75 pcf
Allowable Bearing Capacity (may be increased by one-third for seismic and wind loads)	2,500 psf
Passive Equivalent Fluid Pressure (neglect the upper 1 foot if the ground surface is not confined by slabs or pavement)	350 pcf
Friction Coefficient	0.3
Minimum Footing Depth	18 inches below the lowest adjacent grade
Minimum Footing Width	24 inches

The above recommended lateral pressures are based on drained conditions, and do not include any surcharges; therefore, the designer should include the appropriate surcharge loads to the retaining walls.

To prevent hydrostatic pressure build-up, retaining walls should be constructed with permanent backdrains. The backdrain should consist of a blanket of Class 2 Permeable Material and a 4-inch diameter perforated PVC pipe (SDR 35). The permeable materials should be in conformance with Section 68-1.025 of the 1999 Caltrans "Standard Specifications." The permeable material blanket should be at least 12 inches thick and should be placed from the base of the retaining wall to about 1 foot below the finished grade behind the retaining wall. Alternatively, a geo-composite drain, such as Miradrain 2000 or an approved equivalent, may be used in lieu of the Class 2 Permeable Material blanket. The perforated pipe should be placed near the bottom of the wall to carry collected water to a suitable gravity discharge.

MSE Retaining Wall Design Parameters

Mechanically Stabilized Earth (MSE) Retaining Walls are constructed of precast modular blocks and geogrid reinforcement.

Reinforced Fill, Retained Fill and Foundation	
Unit Weight	125 pcf
Friction Angle	25 degrees
Cohesion	200 psf

The base of the modular block walls should be at least 6 inches (level ground) and 18 inches (sloped ground) below the lowest adjacent finished grade.

SEISMIC DESIGN PARAMETERS

The site is located in a region of high seismicity given the proximity of the Rodgers Creek fault, San Andreas fault, and other active faults in the San Francisco Bay Area. As for all sites in the Bay Area, the project can be expected to experience at least one moderate to severe earthquake during the life span of the development. Ground shaking is a hazard that cannot be eliminated but can be partially mitigated through proper attention to seismic structural design and observance of good construction practices.

The Scott Ranch site is located at approximately 38.2174 degrees North latitude and 122.6470 degrees West longitude. The Peak Ground Acceleration (PGA) according to the 2013 CBC is 0.53 g. We are providing the following 2013 California Building Code seismic design criteria.

- USGS Seismic Design Maps program, Version 3.1.0 dated July 11, 2013.

California Building Code	2013
Mapped Spectral Acceleration for Short Periods, S_s	1.500 g
Mapped Spectral Acceleration for 1-Second Period, S_1	0.600 g
Site Class	D
Site Coefficient F_a (for Site Class D)	1.0
Site Coefficient F_v (for Site Class D)	1.5
Acceleration Parameter S_{MS} (adjusted for Site Class D)	1.500 g
Acceleration Parameter, S_{M1} (adjusted for Site Class D)	0.900 g
Acceleration Parameter, S_{DS}(adjusted for Site Class D)	1.000 g
Acceleration Parameter, S_{D1} (adjusted for Site Class D)	0.600 g

PRELIMINARY PAVEMENT SECTIONS

The following recommendations for asphalt concrete pavement sections are preliminary only. Pavement analyses are based on an assumed "R" (resistance) value of 5, which we expect to be representative of final pavement subgrade materials, Caltrans *Design Method for Flexible Pavement*, and traffic indices (TI's), which are indications of traffic load frequency and intensity. Assigned TI's should include provisions for heavy truck traffic related to construction activities. We recommend the following preliminary pavement sections:

PRELIMINARY RECOMMENDED PAVEMENT SECTIONS		
Traffic Index (TI)	Thickness (inches)	
	Asphalt Concrete Type B	Class 2 Aggregate Base
4	2½	8
4½	2½	10
5	2½	11
5½	3	12
6	3	14

Since on-site materials vary from sandstone to clay, samples should be obtained from the rough roadway subgrade after mass grading. R-value tests should be performed on these samples. Final pavement section recommendations should be made on the basis of these test results.

Prior to subgrade preparation, all utility trench backfill should be properly placed and compacted. Subgrade soils should be rolled to at least 95 percent relative compaction to provide a smooth, unyielding surface. Subgrade soils should be maintained in a moist and compacted condition until covered with the complete pavement section.

Class 2 aggregate base should conform to the requirements in Section 26 of Caltrans' *Standard Specifications* (July, 1999). The aggregate base should be placed in thin lifts in a manner to prevent segregation, uniformly moisture conditioned, and compacted to at least 95 percent relative compaction to provide a smooth, unyielding surface. Relative compaction refers to the in-place dry density of soil expressed as a percentage of the maximum dry density of the same soil, as determined by the ASTM D1557-00 compaction test method.

Where drop inlets or other surface drainage structures are to be installed, slots or weep holes should be provided to allow free drainage of the contiguous aggregate base section.

EXTERIOR FLATWORK

It is our opinion the exterior concrete flatwork may be placed directly on the finish soil subgrade. The soil subgrade should be compacted to a minimum 90 percent relative compaction at a moisture content not less than 3 percent over optimum. All exterior concrete flatwork be cast free from adjacent footings or building slabs. The moisture-conditioned subgrade should not be allowed to lose moisture prior to concrete placement. If the subgrade dries out and shrinkage cracks appear, the subgrade should be reconditioned in accordance with the recommendations of the geotechnical engineer in the field.

UTILITY TRENCHES

All excavations should conform to applicable state and federal industrial safety requirements. Where trench excavations are deeper than 5 feet, they should be sloped no steeper than 1H:1V and/ or shored. Flatter side slopes may be required if seepage is encountered during construction or if the exposed materials differ from those described in the test pit and boring logs. If fully sloped trench walls cannot be excavated due to site constraints, shoring should be provided to ensure trench stability for worker safety. We can provide parameters for shoring design on request.

Material quality, placement procedures, and compaction requirements for utility line bedding and shading materials should meet the City of Petaluma and/or applicable utility agency requirements. From a geotechnical standpoint, the material above the shading material may consist of native materials, compacted to no less than 90 percent relative compaction and 3 percent over optimum moisture content.

Depending on time of year, location, and recent rainfall, ground water may be intercepted during trench excavation, in which case local dewatering will be required. The actual dewatering technique to be used should be approved by the soil engineer before implementation.

CORROSION TESTING

We have obtained three soil samples from the site for corrosion testing. The corrosion testing was performed by CERCO Analytical, Inc., of Pleasanton, California, and the test results are included in Appendix C. The corrosion test results should be transmitted to your structural engineer and underground utility designer, and should be incorporated in the design of the concrete and pipes to be placed directly against the on-site soils.

SEISMIC HAZARDS

SURFACE FAULT RUPTURE

We did not encounter evidence of Quaternary fault traces crossing, passing near, or trending toward the site. The site is not located within an official State of California earthquake fault zone (Davis 2000; Hart and Bryant, 1999) for active faults. According to the State of California, a fault is considered active if it has demonstrated Holocene activity (within the past 11,000 years). We conclude that the potential for surface fault rupture at the site is low.

SECONDARY EFFECTS OF GROUND SHAKING

Liquefaction is the temporary transformation of a saturated, cohesionless soil into a viscous liquid during strong ground shaking from a major earthquake. Dynamic densification can occur when dry, loose, cohesionless soil is subjected to earthquake vibrations of high amplitude. We did not encounter earth materials susceptible to liquefaction or significant dynamic densification at the site.

Strong ground shaking during a major earthquake is liable to initiate landsliding in parts of the region. The stability of all slopes is lower during earthquake disturbances than at other times. Grading in accordance with the recommendations presented above (under *Landslide Remediation* and *Graded Slopes*) is expected to result in a low risk of seismically induced landslides.

RESPONSE TO THIRD PARTY GEOTECHNICAL REVIEWER'S COMMENTS

Our 2004 report for the subject site was peer reviewed by Treadwell & Rollo. Treadwell & Rollo presented their comments in a letter dated November 23, 2004. The following are our responses to their comments:

COMMENT 1

Debris flows are no longer a concern because this portion of the site will be in open space.

COMMENT 2

The maximum allowable fragment dimension for use in engineered fill has been reduced from 12 inches to 6 inches. Adequate soil compaction should be achievable without special equipment.

Maximum fill depths are about 15 feet, 90 percent relative maximum compaction should be satisfactory.

COMMENT 3

Slope stability analyses were performed using Geo-Slope International Ltd. Slope/W program using the Morgenstern-Price method. The following are the shear strength parameters utilized in the slope stability analyses.

Material	**Density, pcf**	**Friction Angle, degrees**	**Cohesion, psf**
Bedrock	125	30	1000
Engineered Fill	120	20	500

The following table presents the results of our slope stability analysis:

	Safety Factor	Safety Factor w/ Seismic Conditions
60 foot Cut Slope	2.7	1.5
25 foot Fill Slope	2.6	1.4

The pseudostatic factor to be applied was determined in accordance with Special Publication 117A, Guidelines for Evaluating and Mitigating Seismic Hazards in California, California Geologic Survey, 2008. A pseudostatic factor, Keq of 0.28 was determined utilizing the Chart on page 30 for a 5 cm threshold displacement, a magnitude of 7.0, distance of 8.9 km, and a 0.53 g maximum horizontal acceleration (PGA from the 2013 CBC).

COMMENT 4

We performed settlement analysis for the upper and lower pad for the proposed split lot residences. We estimate the potential differential settlement to be ½ inch.

The split-level residences are proposed to have a crawlspace and therefore no soil loads against the lower level walls. It is our opinion that the incorporation of seismic and surcharge lateral loads are not necessary.

COMMENT 5

It is not common practice to embed post-tensioned slab foundations because they are designed to primarily resist lateral loads based on base friction.

COMMENT 6

The proposed split-level residences are planned to use a wood flooring system supported on the drilled pier and grade beam foundation system.

COMMENT 7

The comment is not applicable since a bridge for vehicle crossing is no longer planned.

COMMENT 8

We have provided updated seismic design parameter in the *Seismic Design Parameter* section.

ARS curves are not necessary because the vehicular bridge is no longer planned.

COMMENT 9

We concur with T&R's comment

COMMENT 10

We have provided subsurface drainage recommendation in our report and on the two remedial grading plans. Surface drainage should be designed by qualified personnel retained by the project developer. We concur with T&R's comment that the homeowners' association or similar entity should be responsible for inspection and maintenance.

ADDITIONAL SERVICES

Our firm should be afforded the opportunity to review the final plans and specifications to determine if the recommendations contained herein are incorporated into those documents. The review would be acknowledged in writing. Field observation and testing are essential and integral parts of this geotechnical investigation. Our firm should be retained to monitor earthwork and other relevant construction operations; the recommendations of this report are contingent on this.

LIMITATIONS

The conclusions and recommendations contained herein are based upon the information provided to us regarding proposed improvements, our geologic reconnaissance of the site, subsurface conditions encountered during the course of our field investigation, the results of our laboratory testing program, our experience in the area, and professional judgment. This study has been conducted in accordance with current professional geotechnical engineering and engineering geology standards; no other warranty is expressed or implied.

The locations of borings were determined by pacing from existing cultural features and other points of reference depicted on plans prepared by BKF and are considered approximate only. Site conditions described in the text are those existing at the time of our last site visit in April 2003 and are not necessarily representative of such conditions at other locations or times.

If it is found during construction that the conditions differ from those described on the boring and test pit logs, then the conclusions and recommendations contained within this report shall be considered invalid unless the changes are reviewed and the conclusions and recommendations modified or approved in writing by BSA.

Respectfully submitted,

BERLOGAR STEVENS & ASSOCIATES

N. Cardanini

Nicholas Cardanini
Project Engineer

Frank Berlogar
RCE 20383

REGISTERED PROFESSIONAL ENGINEER
FRANK BERLOGAR
No. 20383
CIVIL
STATE OF CALIFORNIA

NC/FB:jmo

Attachments:
- References
- Plate 1 – Vicinity Map
- Plate 2 – Geologic Map, Option A
- Plate 3 – Geologic Map, Option B
- Plate 4 – Geologic Cross Sections A-A′, B1-B1′, B2-B2′, B3-B3′ and C-C′
- Plate 5 – Remedial Cross Sections A-A′, B1-B1′, B2-B2′, B3-B3′ and C-C′
- Plate 6 – Remedial Grading Plan – Option A
- Plate 7 – Remedial Grading Plan – Option B
- Plate 8 – Typical Fill Slope Details
- Plate 9 – Typical Subdrain Details
- Plate 10 – Typical Cut/ Fill Transition Lot Overexcavation Details
- Appendix A – Field Investigation Data
  - A-1 through A-20 – Boring Logs
  - A-21 – Unified Soil Classification System
  - A-22 – Key to Rock Descriptions
  - A-23 through A-31 – Test Pit Logs
- Appendix B – Laboratory Test Results
  - B-1 – Atterberg Limits Test Results
  - B-2 through B-7 – Direct Shear Test Results
  - B-8 through B-11 – Compaction Test Data
  - B-12 through B – Gradation Test Data
  - B-15 through B-17 – Consolidation Test Data
- Appendix C – CERCO Analytical, Inc. Corrosion Test Data

Copies: Addressee (6)

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REFERENCES

Blake, M.C., Jr., Bartow, J.A., Frizzell, V.A., Jr., Schlocker, J., Sorg, D., Wentworth, C.M., and Wright, R.H., 1974, Preliminary geologic map of Marin and San Francisco Counties and parts of Alameda, Contra Costa and Sonoma Counties, California: U.S. Geological Survey Miscellaneous Field Studies Map MF-574.

Davis, J., 2000, Digital images of official maps of the Alquist-Priolo earthquake fault zones of California, Central Coast Region, California Division of Mines and Geology: Division of Mines and Geology CD 2000-004-2000.

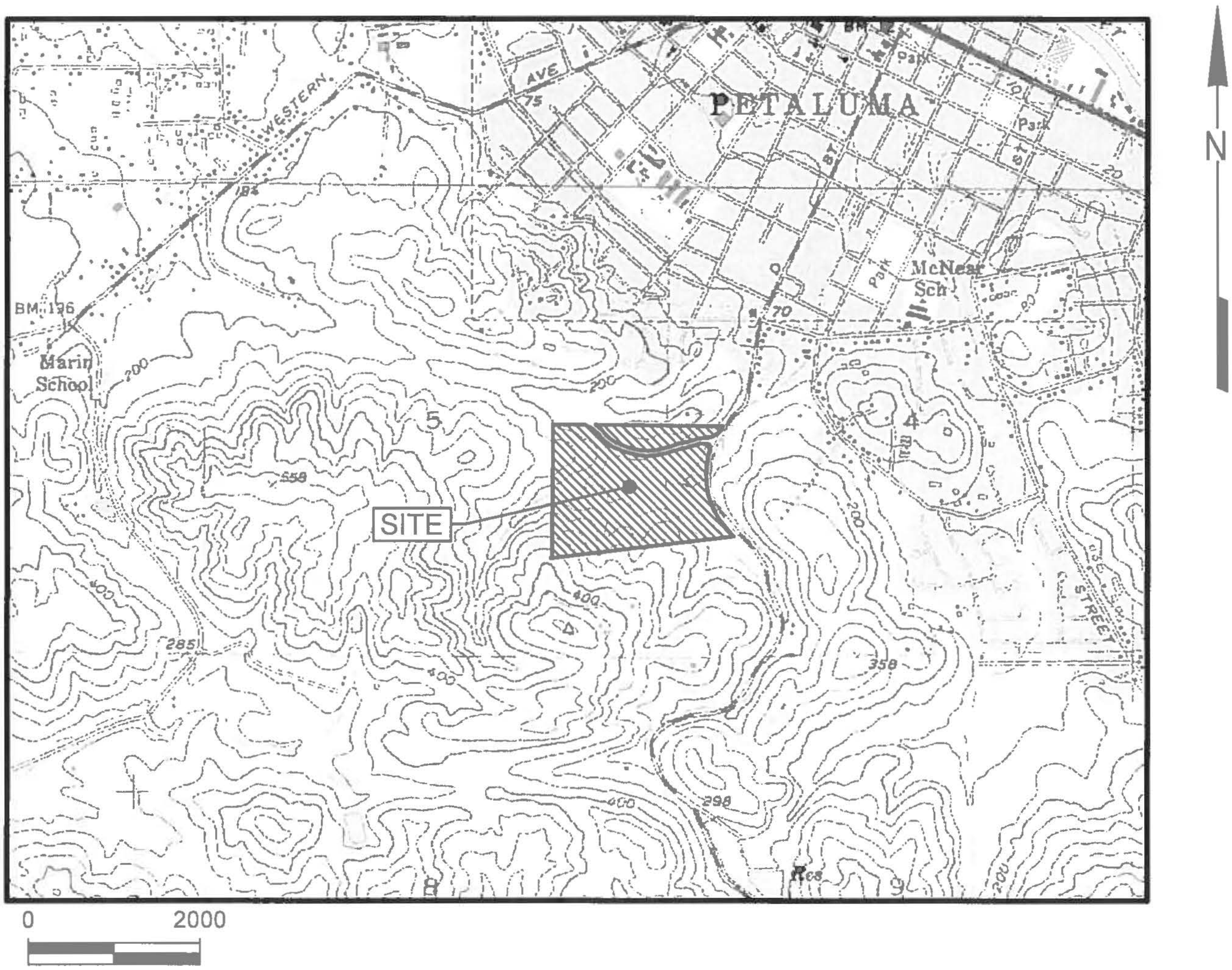
Hart, E.W., and Bryant, W.A., 1999, Fault-rupture hazard zones in California, Alquist-Priolo earthquake fault zoning act with index to earthquake fault zones maps: California Division of Mines and Geology Special Publication 42.

Peterson, M.D., Toppoxada, T., Cao, T., Cramer, C., Reichle, M., Maher, M., and Atchinson, L., 1998, Determining distances from faults within and bordering the state of California for the 1997 Uniform Building Code, in CDMG and SEAC, Maps of Known Active Fault Near-Source Zones in California and Adjacent Portions of Nevada, p. ix-xvii: International Conference of Building Officials, Whittier, California.

AERIAL PHOTOGRAPHS

Date	Photographer	Project, flight line, frames	Nominal scale
09/25/73	U.S. Geological Survey	2-58 and -59	1:24,000
04/19/86	Pacific Aerial Surveys	AV-2860-7-31, -32	1:12,000
06/15/00	Pacific Aerial Surveys	SON-AV-6540-19-34, -35	1:12,000

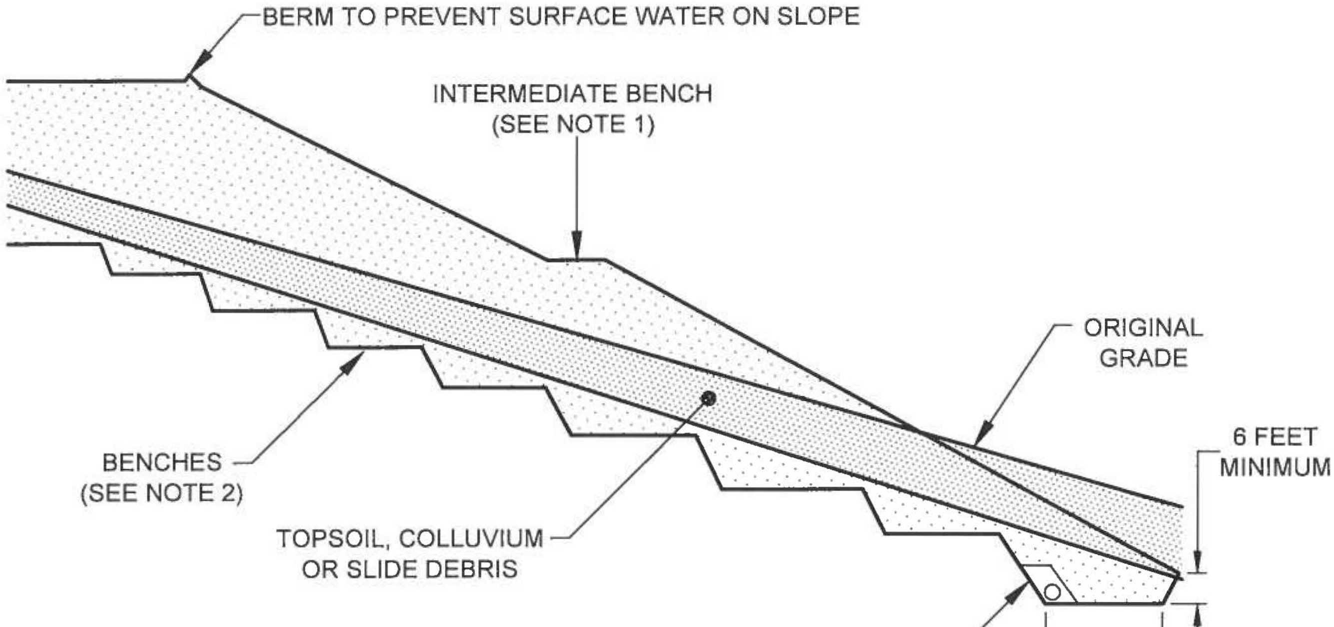
JOB NUMBER: 2616.006 DATE: 4-3-14 BY: CC



VICINITY MAP

SCOTT RANCH
PETALUMA, CALIFORNIA
FOR
DAVIDON HOMES

BASE: PORTION OF U.S.G.S. 7.5 MINUTE TOPOGRAPHIC QUADRANGLE, PETALUMA, CALIFORNIA, AT A SCALE OF 1:24,000.



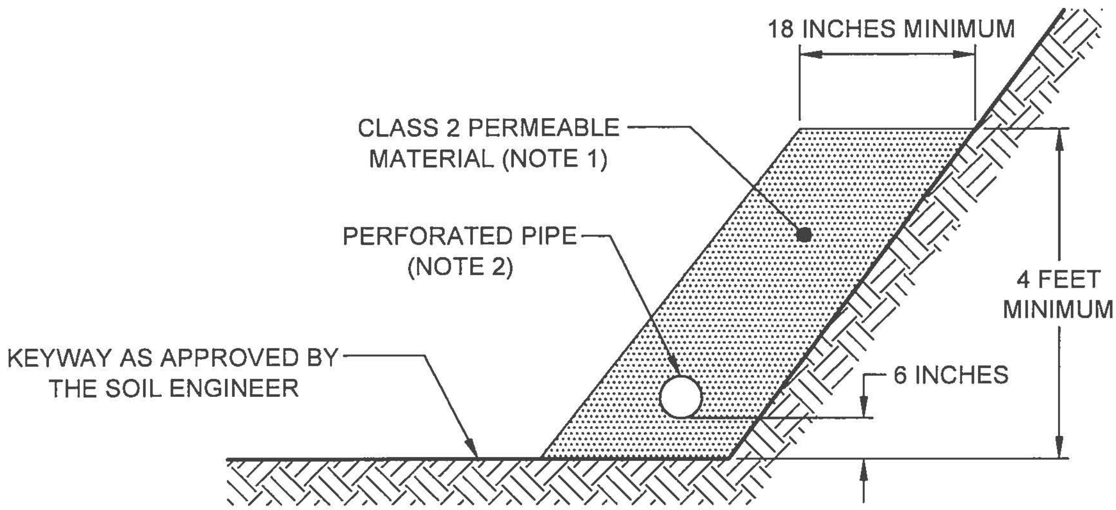
NOT TO SCALE

JOB NUMBER: 2616.006 DATE: 4-22-14 DRAWN BY: CC CHECKED BY:

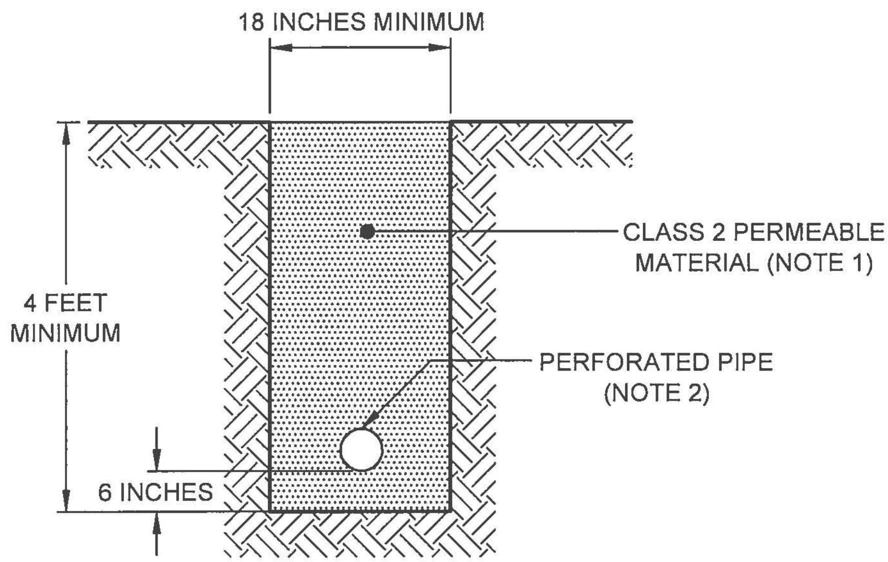
NOTES:

1. INTERMEDIATE BENCHES SHOULD BE SPACED EVERY 25 VERTICAL FEET ON SLOPES HIGHER THAN 30 FEET.
2. WHERE NATURAL GRADE IS STEEPER THAN 7:1, BENCH INTO STIFF SOIL OR BEDROCK AS DETERMINED BY SOIL ENGINEER.
3. SUBDRAIN SHOULD DISCHARGE VIA A CLOSED PIPE TO STORM DRAIN OR SUITABLE NATURAL DRAINAGE.
4. KEYWAY SHOULD EXTEND AT LEAST 6 FEET INTO STIFF SOIL OR BEDROCK AS DETERMINED BY THE SOIL ENGINEER. KEYWAY WIDTH SHOULD BE A MINIMUM OF 20 FEET OR 1/2 OF THE FILL SLOPE HEIGHT, WHICHEVER IS GREATER.

FILL SLOPE DETAIL



KEYWAY SUBDRAIN



NOT TO SCALE

COLLECTOR SUBDRAIN

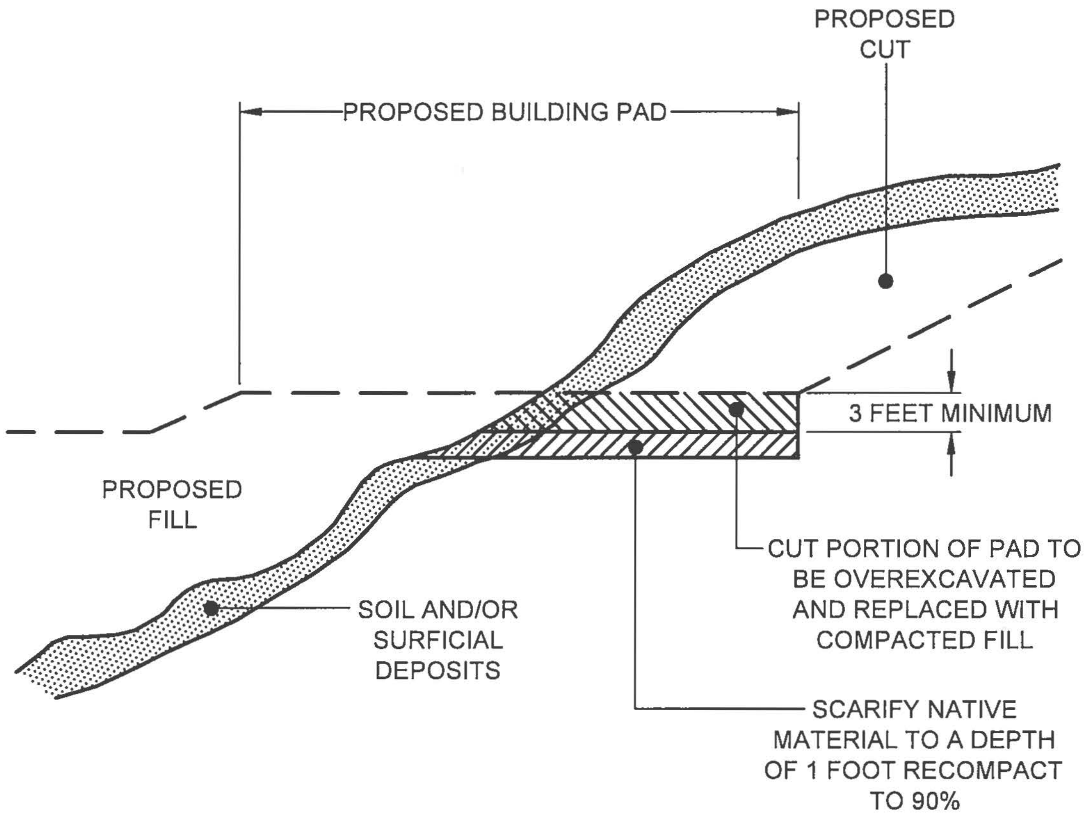
JOB NUMBER: 2616.006 DATE: 4-3-14 DRAWN BY: CC

NOTES:

1. CLASS 2 PERMEABLE MATERIAL AS GIVEN IN SECTION 68 - 1.025, STATE OF CALIFORNIA STANDARD SPECIFICATIONS, MAY, 2006 EDITION.

2. PERFORATED PIPE PLACED PERFORATIONS DOWN, PVC PIPE WITH A MINIMUM DIAMETER OF SIX (6) INCHES, CONFORMING TO ASTM D-3034 SDR 35, FOR DEPTHS LESS THAN 30 FEET, AND SDR 23.5 FOR DEPTHS GREATER THAN 30 FEET.

TYPICAL SUBDRAIN DETAILS



JOB NUMBER: 2616.006 DATE: 4-22-14 DRAWN BY: CC

NOT TO SCALE

TYPICAL CUT/FILL TRANSITION LOT OVEREXCAVATION DETAIL

APPENDIX A

Field Investigation Data

BORING LOG B-1

JOB NUMBER: 2616.100 **DATE DRILLED:** 3-28-03

JOB NAME: UOP Property **SURFACE ELEVATION:** 151 feet

DRILL RIG: Rotary Wash **DATUM:** Mean Sea Level

SAMPLER TYPE:	DRIVE WEIGHT - LB	HEIGHT OF FALL - IN
2.5 inch I.D. Split Barrel	140	30
Standard Penetration Test	140	30

BLOWS PER FT.	MOISTURE CONTENT %	DRY UNIT WEIGHT p.c.f.	DEPTH IN FEET	USCS CLASSIFICATION	DESCRIPTION
10	18.9	102		CL	SILTY CLAY, dark gray-brown, moist, medium stiff, trace fine- grained sand
22	19.1	110	5	CL	SILTY CLAY, gray-brown, moist, stiff to very stiff, trace fine-grained sand
25	22.3	103	10	CL	SANDY CLAY, light to medium gray-brown, moist, very stiff, fine to medium-grained sand, some silt
25	20.1	104	15	SC	CLAYEY SAND, gray-brown, wet, medium dense, fine to medium-grained sand, trace silt, trace fine gravel
14	-	-			
22	19.9	108	20	CL	SILTY CLAY, gray-brown, wet, stiff to very stiff, trace fine-grained sand

BORING LOG B-1

JOB NUMBER: 2616.100 **SHEET:** 2 **OF:** 2

JOB NAME: UOP Property **DEPTH:** 20 feet **TO** 24.5 feet

NOTES:

BLOWS PER FT.	MOISTURE CONTENT %	DRY UNIT WEIGHT p.c.f.	DEPTH IN FEET	USCS CLASSI-FICATION	DESCRIPTION
22	19.9	108		CL	SILTY CLAY, gray-brown, wet to saturated, stiff to very stiff, trace to some fine-grained sand
				CL	SANDY CLAY, light gray-brown, saturated, very stiff, fine to medium-grained sand
					SANDSTONE, fine-grained, tan-brown, highly weathered, strong
66	-	-			SHALE, black, slightly weathered, fractured, low hardness,
			25		Boring terminated at 24-1/2 feet. Free water encountered at 21 feet.
			30		
			35		
			40		

CORE LOG

PROJECT: UOP Property
DRILLING COMPANY: Spectrum
DRILLING METHODS: Rotary Wash
ELEVATION (FEET): 249 feet

BORING NO.: B-2 JOB NO.: 2616.100
DATE BEGUN: 3-28-03
DATE COMPLETED: 3-28-03
DEPTH OF HOLE: 50-1/2 feet
NUMBER OF CORE BOXES: 7
LOGGED BY: ROV

RUN NO.	DRILLRATE (MIN/FT.)	CUT	REC	% REC.	% DRILLING FLUID LOSS	RQD (%)
1	1 1.5 2.0 3.0 2.5/ 6"	4.5	4.0	80	0	0
2	1.5 1.0 2.0 3.5 7.0	5.0	4.0	80	0	0
3	2.0 4.5 7.0 6.5 4/6"	4.5	1.1	24	0	0

DEPTH	DESCRIPTION
0	SILTY CLAY, gray-brown, moist, stiff, some fine-grained sand
2	
4	SANDSTONE, fine-grained, light orange-brown, highly weathered, crushed with some clay
6	SANDSTONE, fine-grained, light to medium brown-gray, highly weathered, weak, highly fractured to crushed at 5.6 feet, joint 60° dip from 7.5 to 8.5 feet, light gray-brown, friable zone
8	from 8.5 to 9 feet, clay layer, 50° dip below 9 feet, becomes crushed sandstone at 9.6 feet, joint 60° dip
10	at 11.5 feet, joint 60° dip
12	from 12 to 12.5 feet, abundant calcite veinlets
14	SHALE, dark gray, highly weathered, moderately strong, crushed
16	

CORE LOG

PROJECT UOP Property BORING NUMBER B-2 JOB NUMBER 2616.100

RUN NO.	DRILLRATE (MIN/FT.)	CUT	REC	% REC.	% DRILLING FLUID LOSS	RQD (%)
4	2.5 2.0 2.0 2.0 2.0	5.0	4.8	96	0	60
5	2.0 1.5 1.5 1.5 2.0	5.0	5.0	100	0	60
6	2.0 2.0 2.0 2.0 2.0	5.0	5.0	100	0	60
7	2.5	5.0	5.0	100	0	10

DEPTH	DESCRIPTION
	SHALE, dark gray, highly weathered, moderately strong, crushed
	SANDSTONE, fine-grained, gray, moderately to highly weathered, weak, highly fractured, limonite stains at 16.8 feet, joint 60° dip
18	SHALE , dark gray, highly weathered, moderately strong, crushed
	SANDSTONE, fine-grained, gray, moderately to highly weathered, weak to moderately strong, highly fractured, limonite stains
20	at 19 feet, joint 60° dip limonite stains on fracture surfaces
	at 21.2 feeet, joint 65° dip
22	at 21.8 feet, fracture 70° dip
24	
26	at 26.4 feet, joint 55° dip
28	at 27.2 feet, crushed zone
30	at 29.5 feet, thin bedding laminations 55° to 60° dip at 30.4 feet, bedding 55° dip
	SHALE, black, moderately to highly weathered, weak, crushed
32	

CORE LOG

PROJECT UOP Property BORING NUMBER B-2 JOB NUMBER 2616.100

RUN NO.	DRILLRATE (MIN/FT.)	CUT	REC	% REC.	% DRILLING FLUID LOSS	RQD (%)	DEPTH
7	2.0 1.5 2.0 2.0	5.0	5.0	100	0	10	34 36
8	6.0	0.5	0.5	100	0	0	
9	3.0 3.0 2.0 3.0	4.0	3.6	90	0	10	38 40
10	3.0 1.5 2.0 2.0 3.0	5.0	5.0	100	0	16	42 44
11	3.0 2.5 2.0	5.0	5.0	100	0	14	46 48

DESCRIPTION

SHALE, black, moderately to highly weathered, weak, crushed

SANDSTONE, fine-grained, gray, moderately weathered, weak to moderately strong, highly fractured, thickly bedded with shale, black, highly weathered, weak, curshed

at 35.5 feet, 2-inch thick clay seam, 40° dip

from 36.8 to 37 feet, shale layer, bedding 50° dip

SANDSTONE, fine-grained, gray, slightly weathered, moderately strong

at 42.2 feet, bedding 50° dip

SHALE, black, highly weathered, weak, crushed, faintly sheared

SANDSTONE, fine-grained, gray, moderately weathered, strong, highly fractured

CORE LOG

PROJECT UOP Property BORING NUMBER B2 JOB NUMBER 2616.100

RUN NO.	DRILLRATE (MIN/FT.)	CUT	REC	% REC.	% DRILLING FLUID LOSS	RQD (%)	DEPTH	LOG	DESCRIPTION
11	2.0	5.0	5.0	100	0	14			SANDSTONE, fine-grained, gray, moderately weathered, strong, highly fractured
	2.0								
	2.5						50		
									Boring terminated at 50-1/2 feet.
							52		
							54		
							56		
							58		
							60		
							62		
							64		

BORING LOG B-3

JOB NUMBER: 2616.100 **DATE DRILLED:** 4-2-03

JOB NAME: UOP Property **SURFACE ELEVATION:** 118 feet

DRILL RIG: Solid Flight Auger **DATUM:** Mean Sea Level

SAMPLER TYPE:	DRIVE WEIGHT - LB	HEIGHT OF FALL - IN
■ 2.5 inch I.D. Split Barrel	140	30
◫ Standard Penetration Test	140	30

BLOWS PER FT.	MOISTURE CONTENT %	DRY UNIT WEIGHT p.c.f.	DEPTH IN FEET	USCS CLASSIFICATION	DESCRIPTION
				ML	SANDY SILT, brown, dry to moist, medium dense, fine-grained sand, trace to some clay, rootlets
20	20.0	101		CL	SILTY CLAY, dark brown, moist, stiff, trace fine-grained sand, trace well rounded gravel up to 1/4-inch diameter, faint iron oxide mottling
				CL	SILTY CLAY, dark yellow-brown, moist, stiff, some fine-grained sand, faint iron oxide mottling
33	17.4	106	5		
				CL	SANDY CLAY, dark yellow-brown, moist, very stiff, medium-grained sand, trace to some well rounded gravel up to 1/8-inch diameter
43	19.8	106	10	SC	CLAYEY SAND, mottled orange-brown and gray, moist,dense, fine to medium-grained sand, trace well rounded gravel up to 1/4-inch diameter
50/6"	15.1	92	15		SHALE, gray, highly weathered, weak, crushed, thinly laminated at 65° to 70°.
					Boring terminated at 15 feet. No free water encountered.
			20		

BORING LOG B-4

JOB NUMBER: 2616.100 **DATE DRILLED:** 4-2-03

JOB NAME: UOP Property **SURFACE ELEVATION:** 137 feet

DRILL RIG: Solid Flight Auger **DATUM:** Mean Sea Level

SAMPLER TYPE:	DRIVE WEIGHT - LB	HEIGHT OF FALL - IN
■ 2.5 inch I.D. Split Barrel	140	30
▱ Standard Penetration Test	140	30

BLOWS PER FT.	MOISTURE CONTENT %	DRY UNIT WEIGHT p.c.f.	DEPTH IN FEET	USCS CLASSI-FICATION	DESCRIPTION
				ML	SANDY SILT, brown, moist, medium dense, fine-grained sand
57	19.5	102		SM	SILTY SAND, orange-brown, moist, dense to very dense, medium-grained sand, trace clay
50/3"	-	-			
50/4"	10.4	107	5		SANDSTONE, fine-grained, tan-brown, highly weathered, weak to moderately strong, moderately fractured with manganese oxide on surfaces
60/6"	-	-			
					Boring terminated at 8 feet. No free water encountered.
			10		
			15		
			20		

BORING LOG B-5

JOB NUMBER: 2616.100 **DATE DRILLED:** 4-2-03

JOB NAME: UOP Proeprty **SURFACE ELEVATION:** 185 feet

DRILL RIG: Solid Flight Auger **DATUM:** Mean Sea Level

SAMPLER TYPE:	DRIVE WEIGHT - LB	HEIGHT OF FALL - IN
2.5 inch I.D. Split Barrel	140	30
Standard Penetration Test	140	30

BLOWS PER FT.	MOISTURE CONTENT %	DRY UNIT WEIGHT p.c.f.	DEPTH IN FEET	USCS CLASSI-FICATION	DESCRIPTION
13	12.8	105		CL	SILTY CLAY, mixed brown and gray-brown, moist, medium stiff to stiff, trace fine to coarse-grained sand, trace cobbles (fill)
32	15.7	108	5	CL	SILTY CLAY, dark gray-brown to brown, moist, stiff, trace gravel, trace fine-grained sand (fill)
25	18.7	108	10		
22	17.1	110	15	CL	SILTY CLAY, dark brown, moist, stiff, some medium-grained sand, trace subrounded gravel up to 1/8-inch diameter
36	16.7	113	20	CL	SILTY CLAY, brown to dark yellow-brown, stiff, trace fine-grained sand, trace subrounded gravel up to 1/2-inch diameter , 1/16-inch thick gray clay films

BORING LOG B-5

JOB NUMBER: 2616.100 **SHEET:** 2 **OF:** 2

JOB NAME: UOP Property **DEPTH:** 20 feet **TO** 30-1/4 feet

NOTES:

BLOWS PER FT.	MOISTURE CONTENT %	DRY UNIT WEIGHT p.c.f.	DEPTH IN FEET	USCS CLASSI-FICATION	DESCRIPTION
36	16.7	113		CL	SILTY CLAY, brown to dark yellow-brown, moist, stiff, trace fine-grained sand, trace subrounded gravel up to 1/2-inch diameter, 1/16-inch gray clay films
25	17.5	116	25		
				SC / CL	SANDY CLAY / CLAYEY SAND, yellow-brown-brown, moist, stiff to medium dense, medium-grained sand
65/6"	-	-			SHALE, gray, highly weathered, weak, highly fractured from 65° to 70° .
50/ 2.5"	-	-	30		
					Boring terminated at 30-1/4 feet. Free water encountered at 20 feet, rose to 17 feet in 4 hours.
			35		
			40		

BORING LOG B-6

JOB NUMBER: 2616.100 **DATE DRILLED:** 4-2-03

JOB NAME: UOP Property **SURFACE ELEVATION:** 157 feet

DRILL RIG: Solid Flight Auger **DATUM:** Mean Sea Level

SAMPLER TYPE:	DRIVE WEIGHT - LB	HEIGHT OF FALL - IN
2.5 inch I.D. Split Barrel	140	30
Standard Penetration Test	140	30

BLOWS PER FT.	MOISTURE CONTENT %	DRY UNIT WEIGHT p.c.f.	DEPTH IN FEET	USCS CLASSI-FICATION	DESCRIPTION
				ML	CLAYEY SILT, brown, moist, stiff, trace fine-grained sand
13	14.6	115		SC	CLAYEY SAND, gray-brown, moist, medium dense, medium-grained sand
17	-	-	5	CL	SANDY CLAY, mottled gray-brown and brown, moist, stiff, medium-grained sand, trace well rounded gravel up to 1/8-inch diameter, iron oxide stains
39	15.9	111		SC	CLAYEY SAND, mottled light gray and orange-brown, moist, dense, medium to coarse-grained sand, trace well rounded to subrounded gravel up to 1/8-inch diameter
74	14.4	108	10	SC	CLAYEY SAND AND SILT, mottled orange-brown and gray, moist, very dense, coarse-grained sand, weakly cemented
50/6"	8.5	129	15		SHALE, gray to black, highly weathered, weak to moderately strong, crushed, 40° joints possible bedding
60/2"	-	-			
			20		Boring terminated at 19-1/2 feet. No free water encountered.

BORING LOG B-7

JOB NUMBER: 2616.100 **DATE DRILLED:** 4-2-03

JOB NAME: UOP Property **SURFACE ELEVATION:** 144 feet

DRILL RIG: Solid Flight Auger **DATUM:** Mean Sea Level

SAMPLER TYPE:	DRIVE WEIGHT - LB	HEIGHT OF FALL - IN
2.5 inch I.D. Split Barrel	140	30
Standard Penetration Test	140	30

BLOWS PER FT.	MOISTURE CONTENT %	DRY UNIT WEIGHT p.c.f.	DEPTH IN FEET	USCS CLASSIFICATION	DESCRIPTION
				ML	SANDY SILT, brown, moist, medium stiff
7	18.2	107		SC	CLAYEY SAND, gray, wet, medium dense, coarse-grained sand
22	-	-		CL	SILTY CLAY, mottled tan-brown and gray-brown, moist, very stiff, trace fine-grained sand, trace well rounded gravel up to 1/4-inch diameter
38	15.2	112	5	CL	SANDY CLAY, mottled gray-brown and gray, moist, very stiff, fine to medium-grained sand, trace subrounded gravel up to 1/4-inch diameter
				SM	SILTY SAND, gray, saturated, medium dense, some clay
40	15.5	111	10	SC	CLAYEY SAND, mottled orange-brown and gray, moist, very dense, coarse-grained sand, trace subrounded gravel up to 1/4-inch diameter
60/2"	-	-			SANDSTONE, fine to medium-grained, tan-brown, highly weathered, weak, highly fractured
			15		Boring terminated at 14-1/6 feet. Free water encountered at 7 feet.
			20		

BORING LOG B-8

JOB NUMBER: 2616.100 **DATE DRILLED:** 4-2-03

JOB NAME: UOP Property **SURFACE ELEVATION:** 134 feet

DRILL RIG: Solid Flight Auger **DATUM:** Mean Sea Level

SAMPLER TYPE:	DRIVE WEIGHT - LB	HEIGHT OF FALL - IN
2.5 inch I.D. Split Barrel	140	30
Standard Penetration Test	140	30

BLOWS PER FT.	MOISTURE CONTENT %	DRY UNIT WEIGHT p.c.f.	DEPTH IN FEET	USCS CLASSI-FICATION	DESCRIPTION
				CL	SANDY CLAY, brown, moist, stiff, fine to medium-grained sand
35	16.2	114		SC	CLAYEY SAND, mottled orange-brown and gray, moist, dense, coarse-grained sand
				CL	SANDY CLAY, brown, moist, stiff to very stiff, fine-grained sand, iron oxide stains
50/6"	10.5	108	5	SC	CLAYEY SAND, tan-brown, dry to moist, very dense, fine to medium-grained sand, trace subrounded gravel up to 1/4-inch diameter, weakly cemented
				SC	CLAYEY SAND, mottled orange-brown and gray, moist, very dense, fine to coarse-grained sand, trace subrounded gravel up to 1/4-inch diameter
65	17.9	104		SP	SAND, orange-brown, moist, very dense, coarse-grained sand
18	-	-	10	SC	CLAYEY SAND, mottled orange-brown and gray, moist, medium dense, fine-grained sand
15	20.8	105	15	SC	CLAYEY SAND, orange-brown and gray, saturated, medium dense, coarse-grained sand
13	-	-			
50/6"	-	-			SANDSTONE, fine-grained, tan-brown, highly weathered, moderately strong, crushed
			20		Boring terminated at 19-1/2 feeet. Free water encounted at 16 feet, rose to 14-1/2 feet in 1 hour.

BORING LOG B-9

JOB NUMBER: 2616.100 **DATE DRILLED:** 4-3-03

JOB NAME: UOP Property **SURFACE ELEVATION:** 140 feet

DRILL RIG: Solid Flight Auger **DATUM:** Mean Sea Level

SAMPLER TYPE:	DRIVE WEIGHT - LB	HEIGHT OF FALL - IN
■ 2.5 inch I.D. Split Barrel	140	30
▱ Standard Penetration Test	140	30

BLOWS PER FT.	MOISTURE CONTENT %	DRY UNIT WEIGHT p.c.f.	DEPTH IN FEET	USCS CLASSIFICATION	DESCRIPTION
				ML	CLAYEY SILT, gray-brown, moist, stiff
32	17.4	110		CL	SILTY CLAY, dark yellow-brown to brown, moist, stiff, trace well-rounded gravel up to 1/8-inch diameter
36	21.0	105	5	CL	SILTY CLAY, yellow-brown, moist, stiff, trace to some fine to medium-grained sand, trace subrounded to subangular gravel up to 3/4-inch diameter, faint iron oxide stains
				CL	SILTY CLAY, tan-brown, moist, stiff, trace to some fine-grained sand, trace subrounded gravel up to 1/4-inch diameter
58	16.3	110	10	CL	SANDY CLAY, mottled orange-brown and gray, moist, very stiff, medium-grained sand, trace to some subangular gravel up to 2-inch diameter
60/6"	-	-			SANDSTONE, medium to coarse-grained, green-gray to black, highly weathered, strong, crushed, 60° joints.
			15		Boring terminated at 14 feet. No free water encountered.
			20		

BORING LOG B-10

JOB NUMBER: 2616.100 **DATE DRILLED:** 4-3-03

JOB NAME: UOP Property **SURFACE ELEVATION:** 142 feet

DRILL RIG: Solid Flight Auger **DATUM:** Mean Sea Level

SAMPLER TYPE:	DRIVE WEIGHT - LB	HEIGHT OF FALL - IN
2.5 inch I.D. Split Barrel	140	30
Standard Penetration Test	140	30

BLOWS PER FT.	MOISTURE CONTENT %	DRY UNIT WEIGHT p.c.f.	DEPTH IN FEET	USCS CLASSI-FICATION	DESCRIPTION
				SM	SANDY SILT, brown, wet, medium dense, fine-grained sand
23	19.6	108		CL	SILTY CLAY, dark yellow-brown, moist, stiff, trace coarse-grained sand, iron oxide stains
37	20.8	102		CL	SILTY CLAY, mottled orange-brown and tan-brown and gray, moist, stiff, some fine-grained sand, trace well rounded gravel up to 1/8-inch diameter, iron oxide stains
			5		SANDSTONE, fine to medium-grained, tan-brown, highly weathered, weak to moderately strong, highly fractured
50/6"	-	-			SHALE, black, highly weathered, strong, crushed
			10		Boring terminated at 9-1/2 feet. No free water encountered.
			15		
			20		

BORING LOG B-11

JOB NUMBER: 2616.100 **DATE DRILLED:** 4-3-03

JOB NAME: UOP Property **SURFACE ELEVATION:** 133 feet

DRILL RIG: Solid Flight Auger **DATUM:** Mean Sea Level

SAMPLER TYPE:	DRIVE WEIGHT - LB	HEIGHT OF FALL - IN
■ 2.5 inch I.D. Split Barrel	140	30
▨ Standard Penetration Test	140	30

BLOWS PER FT.	MOISTURE CONTENT %	DRY UNIT WEIGHT p.c.f.	DEPTH IN FEET	USCS CLASSI-FICATION	DESCRIPTION
				CL	SANDY CLAY, moist, stiff, fine-grained sand
41	19.2	108		CL	SILTY CLAY, mottled tan-brown and gray, moist, stiff, trace fine-grained sand
				CL	SILTY CLAY, mottled brown and orange-brown, moist, very stiff to hard, trace coarse-grained sand, trace subrounded gravel up to 1/4-inch diameter
					at 4-1/2 feet, approximately 6-inch diameter sandstone cobble
50/6"	-	-	5		
				CL	SANDY CLAY, mottled orange-brown and gray, moist, hard, fine-grained sand
60	-	-			
			10		
50/6"	-	-			SHALE, gray to orange-brown, highly weathered, moterately strong to strong, trace clay
65/6"	-	-			
			15		Broing terminated at 14 feet. No free water encountered.
			20		

BORING LOG B-12

JOB NUMBER: 2616.100 **DATE DRILLED:** 4-3-03

JOB NAME: UOP Property **SURFACE ELEVATION:** 124 feet

DRILL RIG: Solid Flight Auger **DATUM:** Mean Sea Level

SAMPLER TYPE:	DRIVE WEIGHT - LB	HEIGHT OF FALL - IN
2.5 inch I.D. Split Barrel	140	30
Standard Penetration Test	140	30

BLOWS PER FT.	MOISTURE CONTENT %	DRY UNIT WEIGHT p.c.f.	DEPTH IN FEET	USCS CLASSI-FICATION	DESCRIPTION
23	19.2	109		CL	SANDY CLAY, dark yellow-brown, moist, stiff, fine-grained, some silt, iron oxide stains
42	16.7	113	5	CL	SILTY CLAY, dark yellow-brown to brown, moist, very stiff, trace to some subangular gravel up to 1/2-inch diameter, trace fine to medium-grained sand
42	16.7	109	10		
45	18.5	110	15	CL	SANDY CLAY, mottled orange-brown and gray-brown, moist, hard, medium-grained sand, trace subangular to subrounded gravel up to 1/4-inch diameter
65/6"	-	-			SHALE, gray, highly weathered, strong, 70° fractures.
			20		Boring terminated at 18-1/2 feet. Free water encountered at 17-1/2 feet.

BORING LOG B-13

JOB NUMBER: 2616.100 **DATE DRILLED:** 4-3-03

JOB NAME: UOP Property **SURFACE ELEVATION:** 123 feet

DRILL RIG: Solid Flight Auger **DATUM:** Mean Sea Level

SAMPLER TYPE:	DRIVE WEIGHT - LB	HEIGHT OF FALL - IN
2.5 inch I.D. Split Barrel	140	30
Standard Penetration Test	140	30

BLOWS PER FT.	MOISTURE CONTENT %	DRY UNIT WEIGHT p.c.f.	DEPTH IN FEET	USCS CLASSI-FICATION	DESCRIPTION
				SM	SANDY SILT, brown, dry, medium dense, fine-grained sand
29	17.9	104		CL	SILTY CLAY, dark gray-brown, moist, stiff, trace coarse-grained sand, iron oxide stains
				CL	SANDY CLAY, tan-brown to brown, moist, stiff, coarse-grained sand
28	18.7	104	5	SC/ CL	CLAYEY SAND / SANDY CLAY, tan-brown to brown, moist, medium dense to stiff, fine to medium-grained sand, trace well rounded gravel up to 1/8-inch diameter, iron oxide stains
				SC	CLAYEY SAND, mottled orange-brown and gray, moist, medium dense, medium-grained sand, trace subrounded to well rounded gravel up to 1/4-inch diameter
30	17.1	108	10		
15	-	-			
20	20.1	107.3	15	SM	SILTY SAND, mottled gray and orange-grown, moist, medium dense, coarse-grained sand, some clay
17	-	-			
50/6"	11.1	125	20		SANDSTONE, fine-grained, gray, highly weathered, highly fractured, strong, 60° joints

BORING LOG B-13

JOB NUMBER: 2616.100 **SHEET:** 2 **OF:** 2

JOB NAME: UOP Property **DEPTH:** 20 feet **TO** 21 feet

NOTES:

BLOWS PER FT.	MOISTURE CONTENT %	DRY UNIT WEIGHT p.c.f.	DEPTH IN FEET	USCS CLASSI-FICATION	DESCRIPTION
50/6"	11.1	125			SANDSTONE, fine-grained, gray, highly weathered, highly fractured, strong, 60° joints
			25		Boring terminated at 21 feet. Free water encountered at 9-1/2 feet, dropped to 17 feet in 2 hours
			30		
			35		
			40		

BORING LOG B-14

JOB NUMBER: 2616.100 **DATE DRILLED:** 4-3-03

JOB NAME: UOP Property **SURFACE ELEVATION:** 170 feet

DRILL RIG: Solid Flight Auger **DATUM:** Mean Sea Level

SAMPLER TYPE:	DRIVE WEIGHT - LB	HEIGHT OF FALL - IN
■ 2.5 inch I.D. Split Barrel	140	30

BLOWS PER FT.	MOISTURE CONTENT %	DRY UNIT WEIGHT p.c.f.	DEPTH IN FEET	USCS CLASSI-FICATION	DESCRIPTION
19	19.8	100		CL	SILTY CLAY, dark, gray-brown, moist, stiff, trace subrounded gravel up to 1/4-inch diameter
20	19.6	107	5	CL	SILTY CLAY, dark brown, moist, stiff, trace well rounded gravel up to 1/4-inch diameter, trace medium to coarse-grained sand
30	14.4	117	10	CL	SILTY CLAY, yellow-brown, moist, stiff to very stiff, some coarse-grained sand, trace subrounded gravel up to 1/2-inch diameter
50/6"	-	-	15		at 15 feet, sandstone boulder approximately18 inches in diameter
50/3"	-	-			SANDSTONE, fine-grained, gray, highly weathered, strong, highly fractured
			20		Boring terminated at 19-1/2 feet. No free water encountered.

BY: FF

DATE: 4-23-03

JOB NUMBER: 2516.100

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

UNIFIED SOIL CLASSIFICATION SYSTEM

Blows per ft.	Moisture Content (%)	Dry Unit Weight (pcf)	Depth in Feet	USCS Classifi-cation	
					Bulk Sample
					2.5" I.D. Split Barrel Sample
					2.8" I.D. Shelby Tube Sample
					No sample recovered
					Standard Penetration Test interval
					Well defined stratum change
					Gradual stratum change
					Interpreted stratum change
					Apparent ground water level at date noted. Seasonal weather conditions, site topography, etc., may cause changes in water level indicated on logs.

Note: Soils described as dry, moist, and wet are estimated to be dry of optimum, near optimum, and wet of optimum moisture content, respectively. Saturated soils are estimated to be within areas of free groundwater.

KEY TO BORING LOG SYMBOLS

ROCK DESCRIPTION

ROCK TYPE
GRAIN SIZE (If Applicable)
COLOR
WEATHERING

Highly - Moderate to complete mineral decomposition, extensive disintegration, deep and through discoloration, fractures extensively coated or filled with oxides, carbonates and/or silt and clay.

Moderately - Slight change or partial decomposition of minerals, little disintegration, cementation little to unaffected, moderate to occasionally intense discoloration, moderately coated fractures.

Slightly - No megascopic decompositon of minerals, little to no effect on cementation, slight and intermittent or localized discoloration, few stains on fracture surfaces.

Unweathered - Unaffected by weathering agents, no discoloration or disintegration.

STRENGTH

Friable - Crumbles easily with fingers

Waek - Crumbles under light hammer blows

Moderately Strong - Specimen will withstand a few hammer blows before breaking

Strong - Specimen will withstand a few eavy ringing hammer blows before breaking into large fragments

Very Strong - Specimen will resist heavy ringing hammer blows and will yield with difficulty only dust and small flying fragments

FRACTURING - Intensity, coating or filling, attitude(s)

Intensity	Size of Pieces
Occasionally Fractured	Greater than 12 inches
Moderately Fractured	6 inches to 12 inches
Highly Fractured	1/2 inch to 6 inches
Crushed	Less than 1/2 inch

BEDING - Stratification, Attitude

Stratification	Thickness
Very Thickly Bedded	Greater than 4 feet
Thickly Bedded	2 to 4 feet
Thinly Bedded	1 inch to 2 feet
Thinly Laminated	Less than 1 inch

MISCELLANEOUS - Shearing of rock, veins, caliche, etc.

Source: Modified from Civil Engineers Reference Book (Blake, 1975)

Job No. 2616.100
UOP Property
D Street and Windsor Drive
Petaluma, California

TEST PIT LOGS

Test Pit Number	Depth (feet)	Description
TP2-1	0-1	Sandy Clay, light brown, moist, stiff, fine-grained sand, some silt, rootlets.
	1-3	Silty Sand, orange-brown, moist, very dense, coarse-grained sand, some clay.
	3-6	Sandstone, coarse-grained, tan-brown, highly weathered, moderately strong, highly fractured with manganese oxide on surfaces.
		Total Depth 6 feet No free ground water encountered
TP2-2	0-2	Sandy Clay, light brown, moist, stiff, fine-grained sand, some silt, rootlets.
	2-6	Sandstone, medium-grained, gray and red-brown, moderately weathered, very strong, highly fractured. Joints N30W 40S.
		Total Depth 6 feet No free ground water encountered
TP2-3	0-3	Sandy Clay, light brown, moist, stiff, fine-grained sand.
	3-5	Clayey Sand, mottled orange-brown and gray, moist, dense to very dense, medium-grained sand, trace subangular sandstone clasts.
	5-7	Sandstone, coarse-grained, orange-brown, moderately weathered, strong to very strong, highly fractured to crushed.
		Total Depth 7 feet No free ground water encountered
TP2-4	0-32	Sandstone, fine- to medium-grained, tan-brown, moderately weathered, very strong, highly fractured. Joints N10E 45N, N45E vertical.
		Total Depth 32 feet No free ground water encountered

Job No. 2616.100
UOP Property
D Street and Windsor Drive
Petaluma, California

TEST PIT LOGS

Test Pit Number	Depth (feet)	Description
TP2-5	0-32	Silty Clay, dark gray-brown, moist, stiff, trace fine-grained sand, subrounded sandstone cobbles from 2 to 3 feet.
	32-42	Clayey Sand, orange-brown, moist, dense, trace subangular sandstone clasts.
	42-62	Sandstone, coarse-grained, orange-brown, highly weathered, strong, crushed.
		Total Depth 62 feet No free ground water encountered
TP2-6	0-2	Clayey Silt, brown, moist, stiff, trace fine-grained sand.
	2-6	Sandy Clay, mottled brown and orange-brown, moist, very stiff, some well-rounded gravel, coarse-grained sand.
		Total Depth 6 feet No free ground water encountered
TP2-7	0-2	Sandy Clay, light brown, moist, stiff, fine-grained sand.
	2-23	Sandy Clay, light brown, saturated, medium stiff, fine-grained sand.
	23-4	Sandy Clay, mottled brown and orange-brown, moist, very stiff, coarse-grained sand, trace to some well rounded gravel.
		Total Depth 4 feet Ground water encountered at 2 feet
TP2-8	0-4	Sandy Clay, light brown, moist, stiff, fine-grained sand.
	4-6	Sandy Clay, orange-brown, moist, very stiff, trace angular sandstone clasts.
	6-8	Sandstone, coarse-grained, orange-brown, highly weathered, strong, highly fractured. Joints N30W 60SW.
		Total Depth 8 feet No free ground water encountered

TEST PIT LOGS

Test Pit Number	Depth (feet)	Description
TP2-9	0-3	Silty Clay, brown, moist, stiff, trace fine-grained sand.
	3-6	Sandy Clay, mottled brown and orange-brown, moist, stiff to very stiff, medium grained sand, trace well-rounded gravel.
	6-8	Shale, gray, highly weathered, weak to moderately strong, crushed.
		Total Depth 8 feet No free ground water encountered
TP2-10	0-2	Sandy Silt, brown, moist, stiff, fine-grained sand.
	2-5	Sandstone, coarse-grained, orange-brown, highly weathered, strong, highly fractured. Joints N25E 65N, N70W 30S.
		Total Depth 5 feet No free ground water encountered
TP2-11	0-32	Silty Clay, dark brown, moist, stiff, trace fine-grained sand.
	32-7	Shale, gray to orange-brown, highly weathered, weak to moderately strong, highly fractured, thinly laminated. Bedding N20W 73SW.
		Total Depth 7 feet No free ground water encountered
TP2-12	0-22	Silty Clay, dark brown, moist, stiff, trace gravel.
	22-7	Shale, gray, highly weathered, weak, crushed, some clay.
		Total Depth 7 feet No free ground water encountered
TP2-13	0-3	Silty Clay, dark brown, moist, stiff.
	3-6	Silty Clay, brown, moist, stiff, trace subangular gravel.
	6-9	Shale, gray, highly weathered, crushed, some clay.
		Total Depth 9 feet No free ground water encountered

Job No. 2616.100
UOP Property
D Street and Windsor Drive
Petaluma, California

TEST PIT LOGS

Test Pit Number	Depth (feet)	Description
TP2-14	0-3	Silty Clay, dark brown, moist, stiff.
	3-8	Sandstone, fine-grained, gray, highly weathered, crushed, moderately strong. Joints N70W 70N, N30W 85SW.
		Total Depth 8 feet No free ground water encountered
TP2-15	0-4	Silty Clay, dark brown, moist, stiff, trace gravel.
	4-8	Sandstone, fine-grained, gray, highly weathered, moderately strong, crushed. Possible bedding N24W 73SW. Joint N85W 20N.
		Total Depth 8 feet No free ground water encountered
TP2-16	0-4	Silty Clay, dark gray-brown, moist, stiff.
	4-6	Silty Clay, brown, moist, stiff, trace coarse-grained sand, trace subangular gravel up to 2-inch diameter, fairly sharp basal contact with faint horizontal clay film.
	6-13	Clayey Sand/Sandy Clay, orange-brown, moist, very dense, coarse-grained sand, trace well-rounded gravel up to 2-inch diameter.
	13-15	Shale, gray, highly weathered, weak to moderately strong, crushed.
		Total Depth 15 feet No free ground water encountered
TP2-17	0-32	Silty Clay, dark gray-brown, moist, stiff.
	32-5	Silty Clay, brown, moist, stiff, trace subangular gravel up to 2-inch diameter, sharp basal contact with 1/16-inch clay. N20E 12SE.
	5-8	Clayey Sand, orange-brown, moist, very dense, coarse-grained sand, trace well-rounded gravel up to 2-inch diameter.
		Total Depth 8 feet No free ground water encountered

Job No. 2616.100
UOP Property
D Street and Windsor Drive
Petaluma, California

TEST PIT LOGS

Test Pit Number	Depth (feet)	Description
TP2-18	0-32	Silty Clay, dark gray-brown, moist, stiff, trace fine-grained sand.
	32-7	Sandstone, fine-grained, tan-brown, highly weathered, strong, crushed.
		Total Depth 7 feet No free ground water encountered
TP2-19	0-1	Sandy Silt, light brown, dry to moist, fine-grained sand, trace clay.
	1-9	Clayey Sand, mottled orange-brown and gray, moist, very dense, coarse-grained sand.
	9-12	Sandstone, medium-grained, tan-brown, highly weathered, moderately strong, crushed.
		Total Depth 12 feet No free ground water encountered
TP2-20	0-3	Silty Sand, brown, moist, stiff, fine-grained sand, some clay.
	3-9	Silty Sand, orange-brown and gray, moist, very dense, fine-grained sand, trace well-grounded gravel up to 2-inch diameter, trace clay.
	9-14	Clayey Sand, mottled orange-brown and gray, moist to wet, dense to very dense, coarse-grained sand, some subrounded gravel up to 1-inch diameter.
	14-15	Shale, gray, highly weathered, weak, crushed.
		Total Depth 15 feet Free ground water encountered at 15 feet
TP2-21	0-1	Sandy Silt, brown, dry to moist, stiff, fine-grained sand, some clay.
	1-5	Sandstone, fine-grained, tan-brown, highly weathered, strong, highly fractured, bedded with shale, black, highly weathered, weak to moderately strong, crushed with some clay, disrupted structure. Bedding N62W 87S.
		Total Depth 5 feet No free ground water encountered

TEST PIT LOGS

Test Pit Number	Depth (feet)	Description
TP2-22	0-12	Sandy Clay, brown, moist, stiff, fine-grained sand, some silt.
	12-5	Shale, gray to orange-brown, highly weathered, weak to moderately strong, crushed with trace to some clay, disrupted structure.
		Total Depth 5 feet No free ground water encountered
TP2-23	0-2	Sandy Clay, brown, moist, stiff, fine-grained sand, faint blocky ped structure.
	2-4	Sandstone, medium-grained, tan-brown, highly weathered, highly fractured, trace clay on fracture surfaces.
		Total Depth 4 feet No free ground water encountered
TP2-24	0-42	Silty Clay, dark gray-brown, moist, stiff, trace well-rounded gravel up to 3-inch diameter.
	42-62	Silty Clay, dark brown, moist, stiff to very stiff, trace well-rounded gravel up to 3-inch diameter, trace medium- to coarse-grained sand.
	62-112	Silty Clay, brown, moist, stiff to very stiff, some coarse-grained sand, trace subrounded to rounded gravel up to 2-inch diameter.
		Total Depth 112 feet No free ground water encountered
TP2-25	0-2	Sandy Clay, brown, moist, stiff, fine-grained sand, trace subrounded cobbles up to 2-inches diameter.
	2-6	Shale, black to orange-brown, highly weathered, moderately strong, crushed with some clay, fairly undulating, disrupted structure N70W 70-85SW.
		Total Depth 6 feet No free ground water encountered

TEST PIT LOGS

Test Pit Number	Depth (feet)	Description
TP2-26	0-2	Sandy Clay, brown, moist, stiff, fine-grained sand.
	2-6	Shale, black to orange-brown, highly weathered, weak, crushed with some clay, thinly bedded with sandstone, fine-grained, tan-brown, moderately strong, crushed. Bedding N20E 65S.
TP2-27	0-3	Sandy Clay, brown to dark yellow-brown, moist, stiff, fine-grained sand, some silt, iron oxide stains.
	3-14	Silty Clay, dark yellow-brown to brown, moist, very stiff, trace to some subangular to subrounded gravel up to 2-inch, angular friable sandstone clasts, trace fine- to medium-grained sand (Qls).
		Total Depth 14 feet No free ground water encountered
TP2-28	0-12	Sandy Clay, brown, moist, stiff, fine-grained sand.
	12-32	Cobbles and orange-brown Clay (matrix), moist, dense, clast supported.
	32-6	Sandy Clay, mottled orange-brown and gray, moist, very stiff, coarse-grained sand, trace subangular gravel up to 2-inch diameter, sharp basal contact with c-inch orange-brown clay. N50E 18SE.
	6-8	Sandstone, fine-grained, tan-brown, highly weathered, weak to moderately strong, crushed with trace clay.
		Total Depth 8 feet No free ground water encountered
TP2-29	0-22	Sandy Clay, brown, moist, stiff, fine-grained sand.
	22-8	Silty Clay, mottled tan-brown and gray, moist, stiff to very stiff, trace fine-grained sand, trace well-rounded gravel up to 3-inch diameter, gradational roughly horizontal basal contact over approximately 8 inches.
	8-11	Sandstone, fine-grained, tan-brown, weak to strong, highly fractured to crushed.
		Total Depth 11 feet No free ground water encountered

Job No. 2616.100
UOP Property
D Street and Windsor Drive
Petaluma, California

TEST PIT LOGS

Test Pit Number	Depth (feet)	Description
TP2-30	0-1	Sandy Silt, brown, dry, stiff, fine-grained sand.
	1-2½	Silty Clay, dark gray-brown, moist, stiff.
	2½-6	Sandy Clay, tan-brown to brown, moist, stiff, coarse-grained sand.
	6-8	Clayey Sand, brown to gray, moist, dense to very dense, coarse-grained sand.
	8-14	Sandy Clay, mottled orange-brown and gray, moist, very stiff, coarse-grained sand.
		Total Depth 14 feet No free ground water encountered
TP2-31	0-1½	Sandy Clay, brown, moist, stiff, some rounded cobbles up to 4-inches diameter.
	1½-3	Sandstone, fine-grained, tan-brown, highly weathered, crushed.
	3-5	Sheared Clay/Shale, black, highly weathered, weak, inclusions of shale and sandstone, faint foliation parallel to contact, faint residual bedrock structure. Bedding N40W 33S.
		Total Depth 5 feet No free ground water encountered
TP2-32	0-1½	Sandy Clay, brown, moist, stiff, fine-grained sand, trace subangular cobbles up to 4-inches diameter.
	1½-5	Shale, gray to orange-brown, highly weathered, weak to moderately strong, crushed. Bedding N70W 63S.
		Total Depth 5 feet No free ground water encountered
TP2-33	0-2½	Silty Clay, brown, moist, stiff, trace fine-grained sand.
	2½-6	Shale, black, highly weathered, weak to moderately strong, crushed, thinly bedded. Bedding N60W 70S.
		Total Depth 6 feet No free ground water encountered

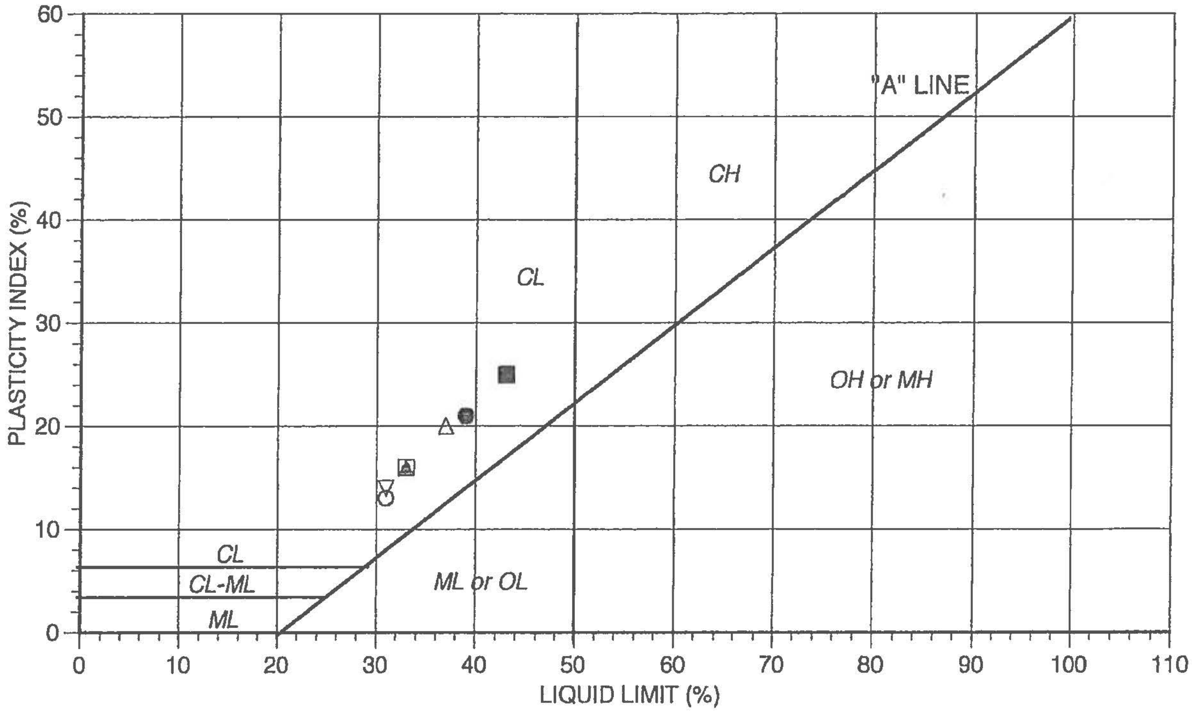
TEST PIT LOGS

Test Pit Number	Depth (feet)	Description
TP2-34	0-3	Silty Clay, dark brown, moist, stiff, trace well-rounded gravel up to 3-inch diameter.
	3-5	Silty Clay, brown, moist, very stiff, gradational basal contact over approximately 10-inches.
	5-8	Shale, gray to black, highly weathered, weak to moderately strong, crushed with trace to some clay.
		Total Depth 8 feet No free ground water encountered
TP2-35	0-2	Sandy Silt and Gravel, tan-brown, dry, hard (Fill).
	2-12	Sandy Clay, brown, moist, very stiff, fine-grained sand.
		Total Depth 12 feet No free ground water encountered
TP2-36	0-1	Sandy Silt and Gravel, tan-brown, dry, hard (Fill).
	1-12	Sandy Clay, brown, moist, very stiff, fine-grained sand.
		Total Depth 12 feet No free ground water encountered

APPENDIX B

Laboratory Test Results

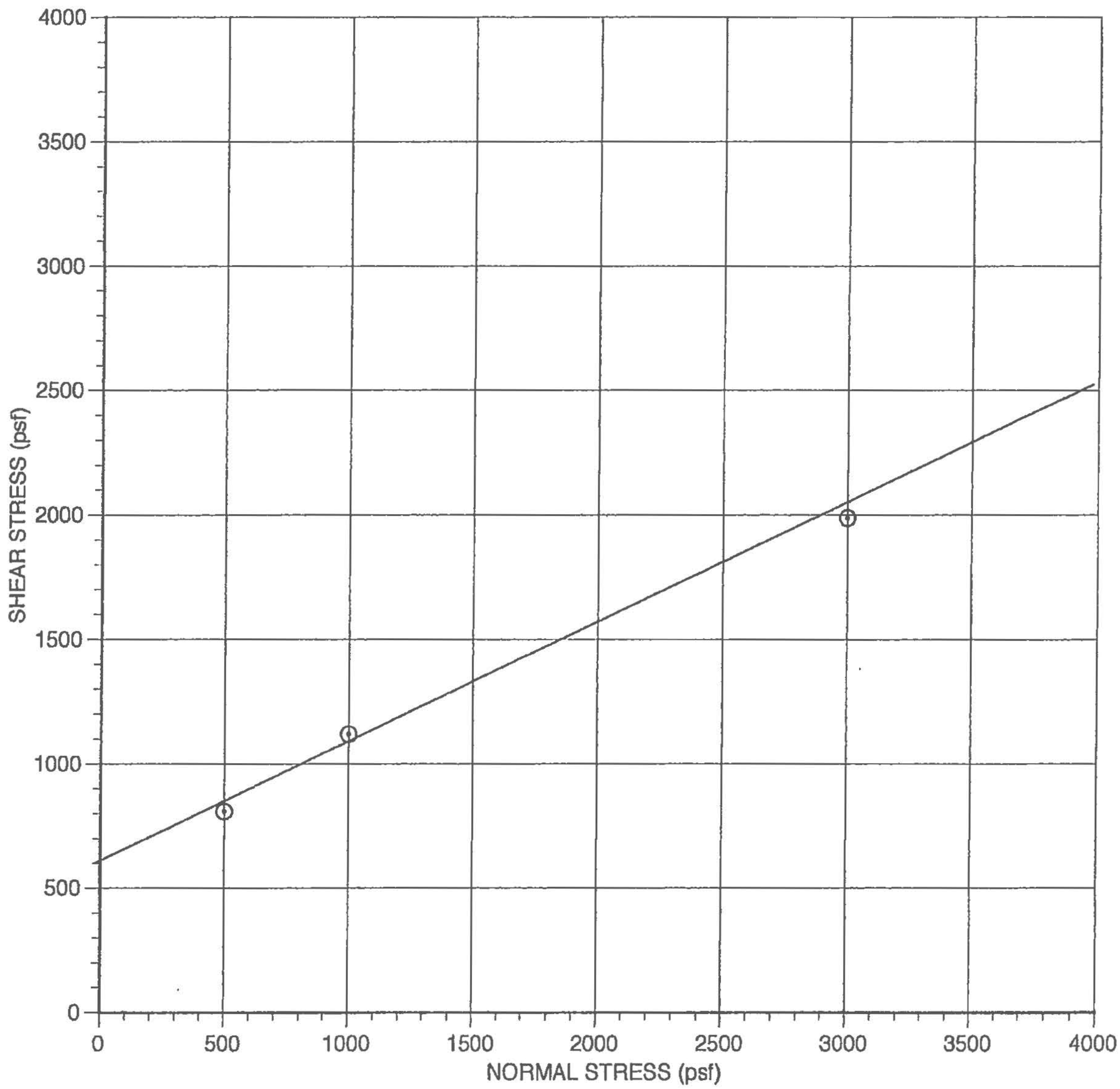
BY: PW DATE: 4-23-03 JOB NUMBER: 2616.100



SYMBOLS	LOCATION	LIQUID LIMIT	PLASTICITY INDEX	USCS CLASSIFICATION
●	B-1 at 1 foot	39	21	C L
□	B-3 at 5 feet	33	16	C L
▲	B-13 at 1 foot	33	16	C L
○	TP2-2 at 0 to 2 feet	31	13	C L
■	TP2-5 at 0-3 feet	43	25	C L
△	TP2-13 at 6-9 feet	37	20	C L
▽	TP2-24 at 6-1/2 feet	31	14	C L

ATTERBERG LIMITS TEST DATA

JOB NUMBER: 2616.100 DATE: 4-23-03 BY: PW



LOCATION: B-3 at 9-1/2 feet

SAMPLE: SANDY CLAY, mottled yellow brown olive gray

TEST TYPE: Consolidated Drained

RATE OF SHEAR (in/min): 0.00099

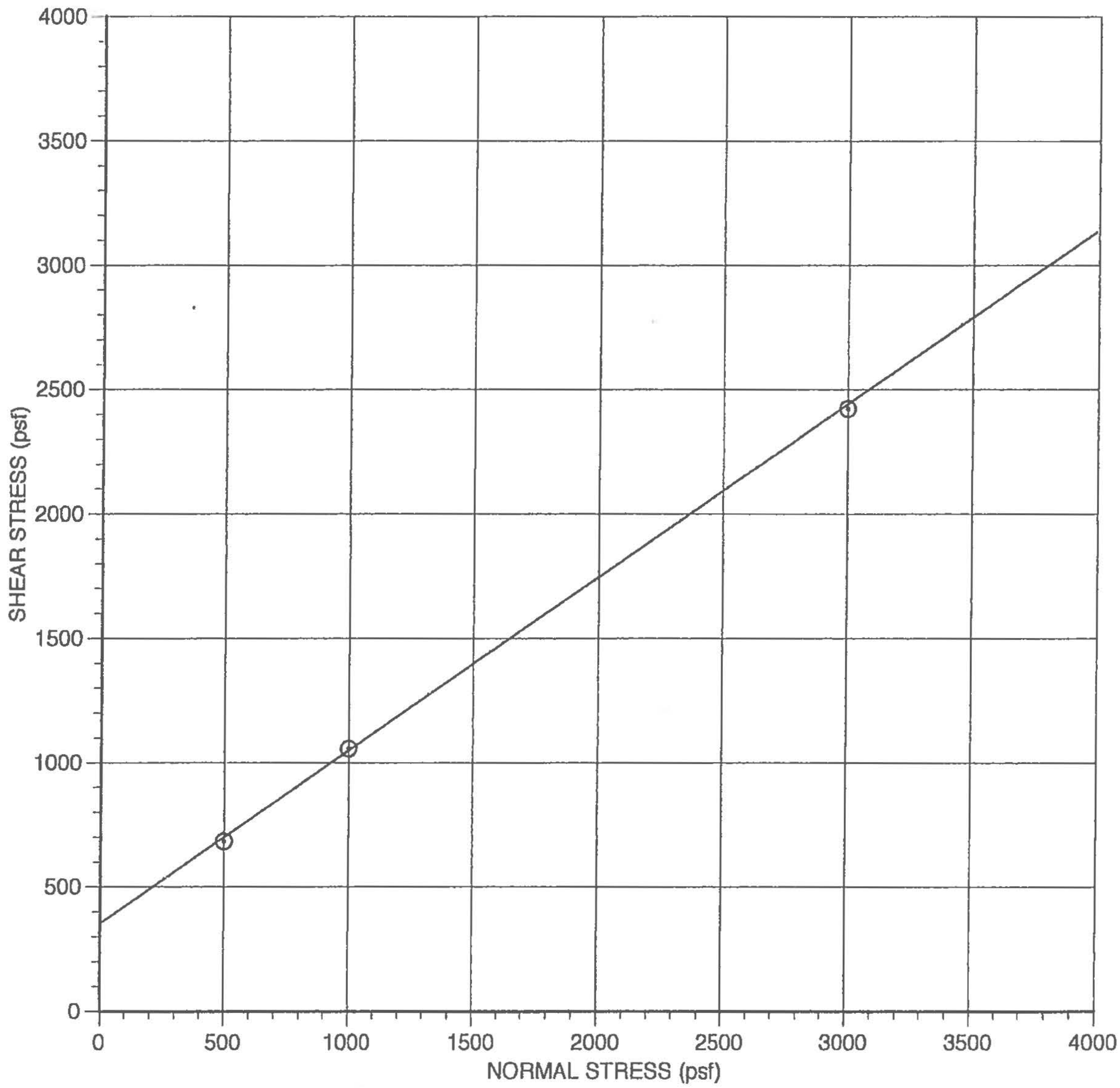
FRICTION ANGLE: 25°

COHESION: 600 psf

SPECIMEN	A	B	C
DRY DENSITY (pcf)	107.6	105.6	106.0
INITIAL WATER CONTENT (%)	19.3	20.8	19.4
FINAL WATER CONTENT (%)	20.8	22.2	19.1
NORMAL STRESS (psf)	500	1000	3000
MAXIMUM SHEAR (psf)	808	1118	1988

DIRECT SHEAR TEST

JOB NUMBER: 2616.100 DATE: 4-23-03 BY: PW



LOCATION: B-5 at 9 feet

SAMPLE: SILTY CLAY, dark gray, trace sand

TEST TYPE: Consolidated Drained

RATE OF SHEAR (in/min): 0.00099

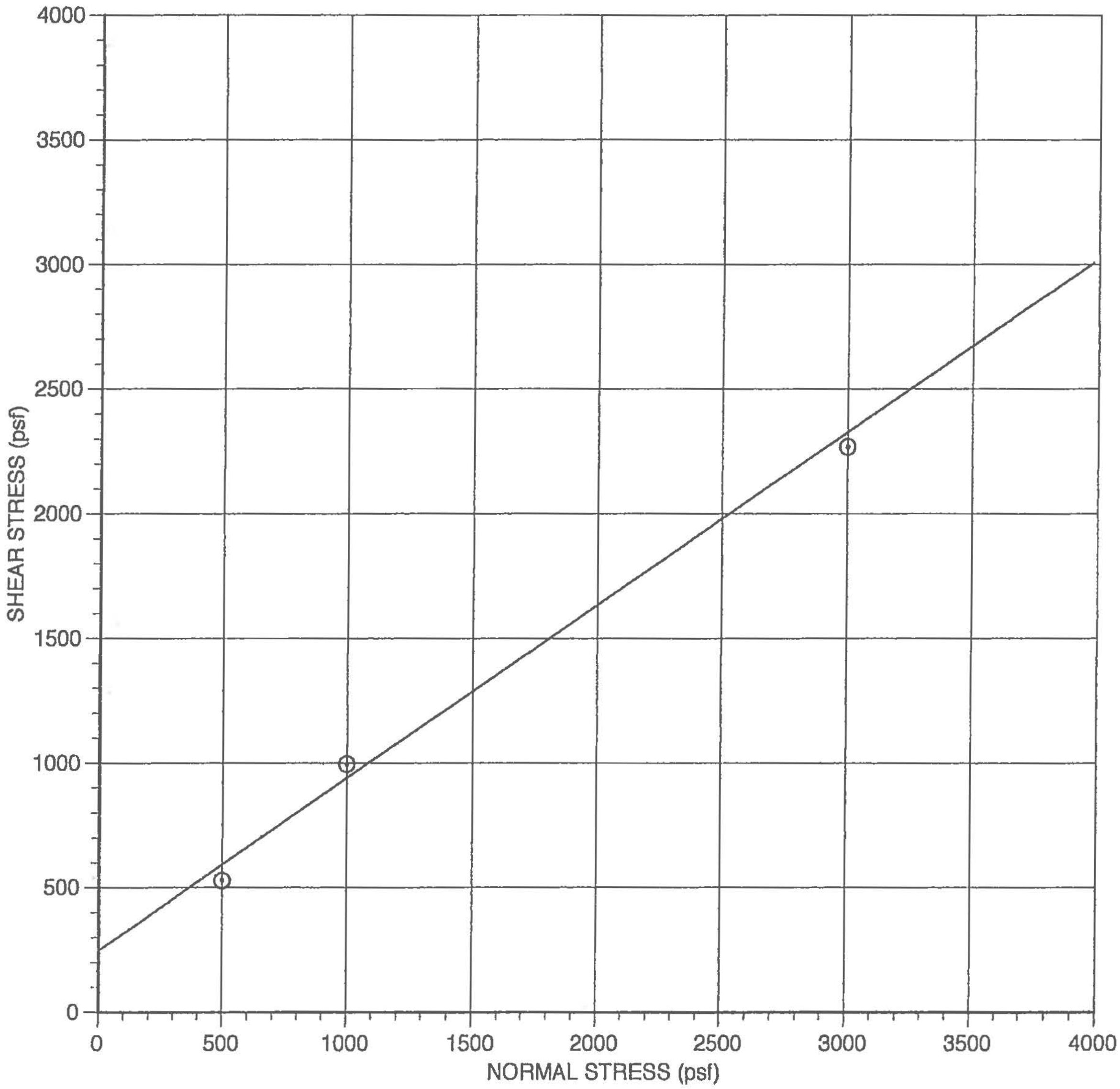
FRICTION ANGLE: 35°

COHESION: 350 psf

SPECIMEN	A	B	C
DRY DENSITY (pcf)	107.8	108.7	108.1
INITIAL WATER CONTENT (%)	19.0	17.4	17.2
FINAL WATER CONTENT (%)	20.6	18.7	16.9
NORMAL STRESS (psf)	500	1000	3000
MAXIMUM SHEAR (psf)	683	1056	2423

DIRECT SHEAR TEST

JOB NUMBER: 2616.100 DATE: 4-23-03 BY: PW



LOCATION: B-14 at 5 feet

SAMPLE: SILTY CLAY with sand, brown

TEST TYPE: Consolidated Drained

RATE OF SHEAR (in/min): 0.00099

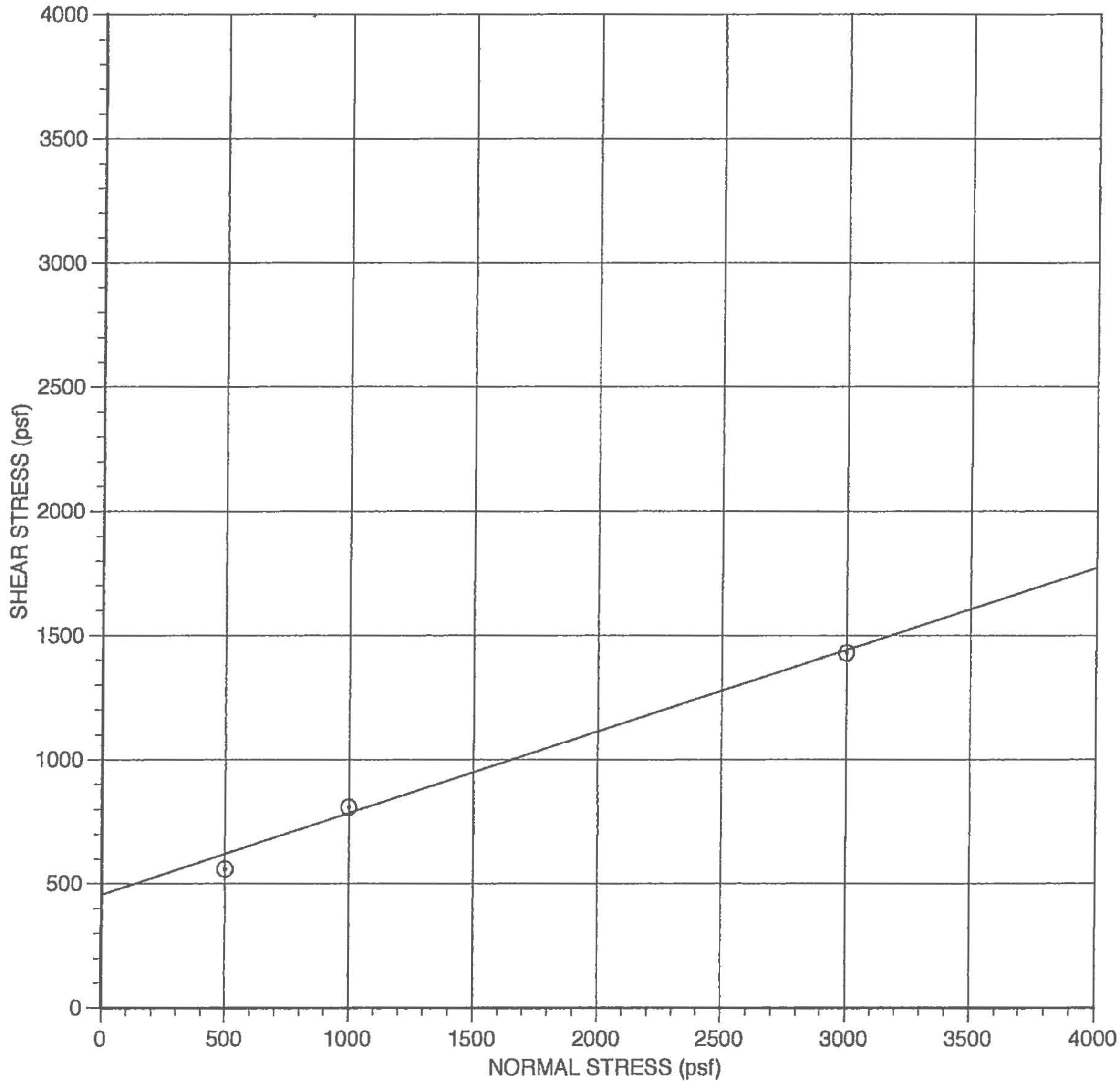
FRICTION ANGLE: 34.5°

COHESION: 250 psf

SPECIMEN	A	B	C
DRY DENSITY (pcf)	106.3	106.7	107.2
INITIAL WATER CONTENT (%)	20.9	20.3	17.6
FINAL WATER CONTENT (%)	20.9	20.3	18.8
NORMAL STRESS (psf)	500	1000	3000
MAXIMUM SHEAR (psf)	528	994	2268

DIRECT SHEAR TEST

JOB NUMBER: 2616.100 DATE: 4-23-03 BY: PW



LOCATION: TP2-2 at 2 feet

SAMPLE: SANDY SILT CLAY, olive-brown, Remolded to 90% Relative Compaction

TEST TYPE: Consolidated Drained

RATE OF SHEAR (in/min): 0.00099

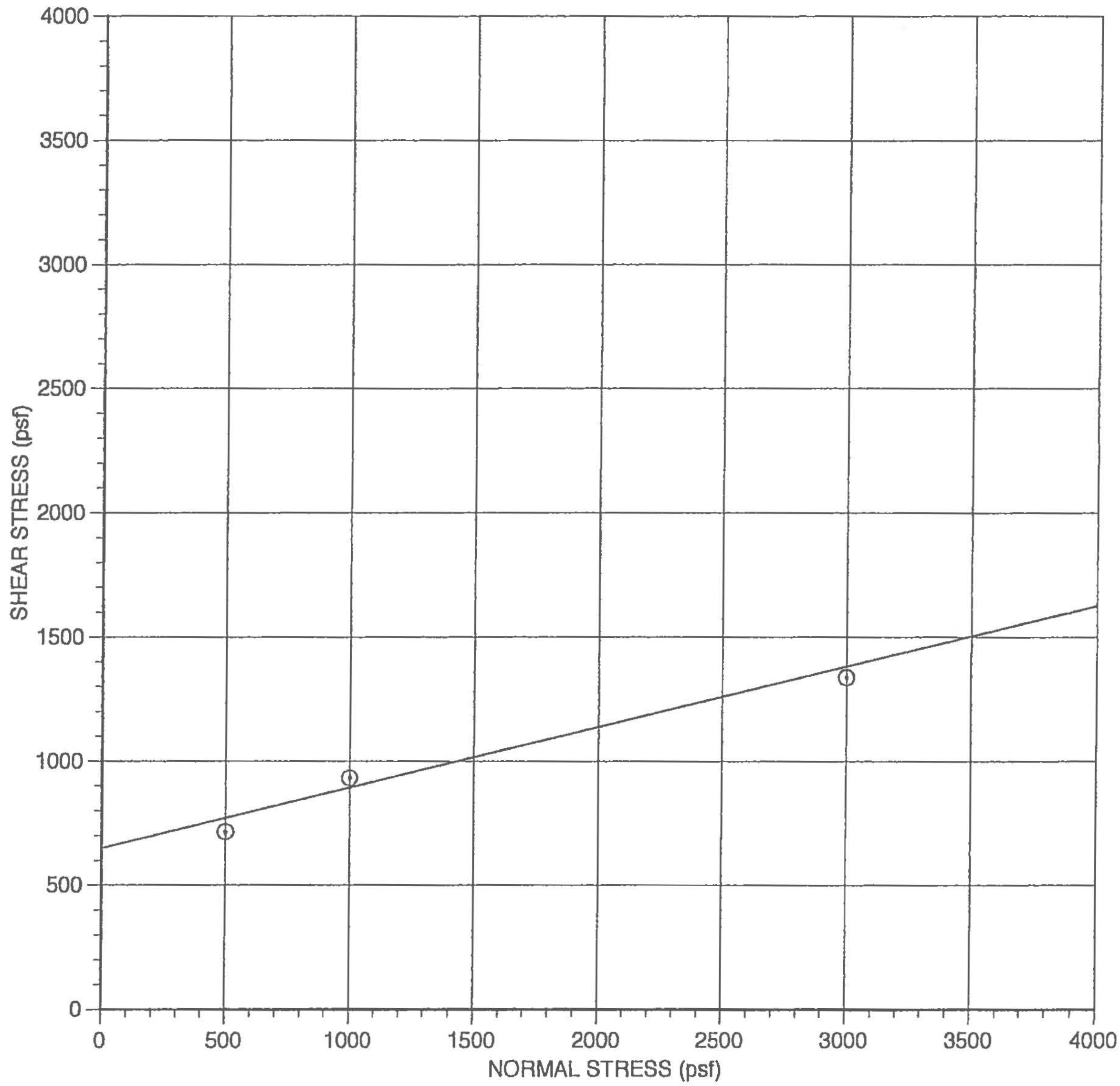
FRICTION ANGLE: 18°

COHESION: 450 psf

SPECIMEN	A	B	C
DRY DENSITY (pcf)	110.5	110.8	110.8
INITIAL WATER CONTENT (%)	14.9	14.9	15.0
FINAL WATER CONTENT (%)	18.4	16.0	15.4
NORMAL STRESS (psf)	500	1000	3000
MAXIMUM SHEAR (psf)	559	808	1429

DIRECT SHEAR TEST

JOB NUMBER: 2616.100 DATE: 4-23-03 BY: PW



LOCATION: TP2-13 at 6 to 9 feet

SAMPLE: SANDY CLAY, olive-brown with gray, Remolded to 90% Relative Compaction

TEST TYPE: Consolidated Drained

RATE OF SHEAR (in/min): 0.00099

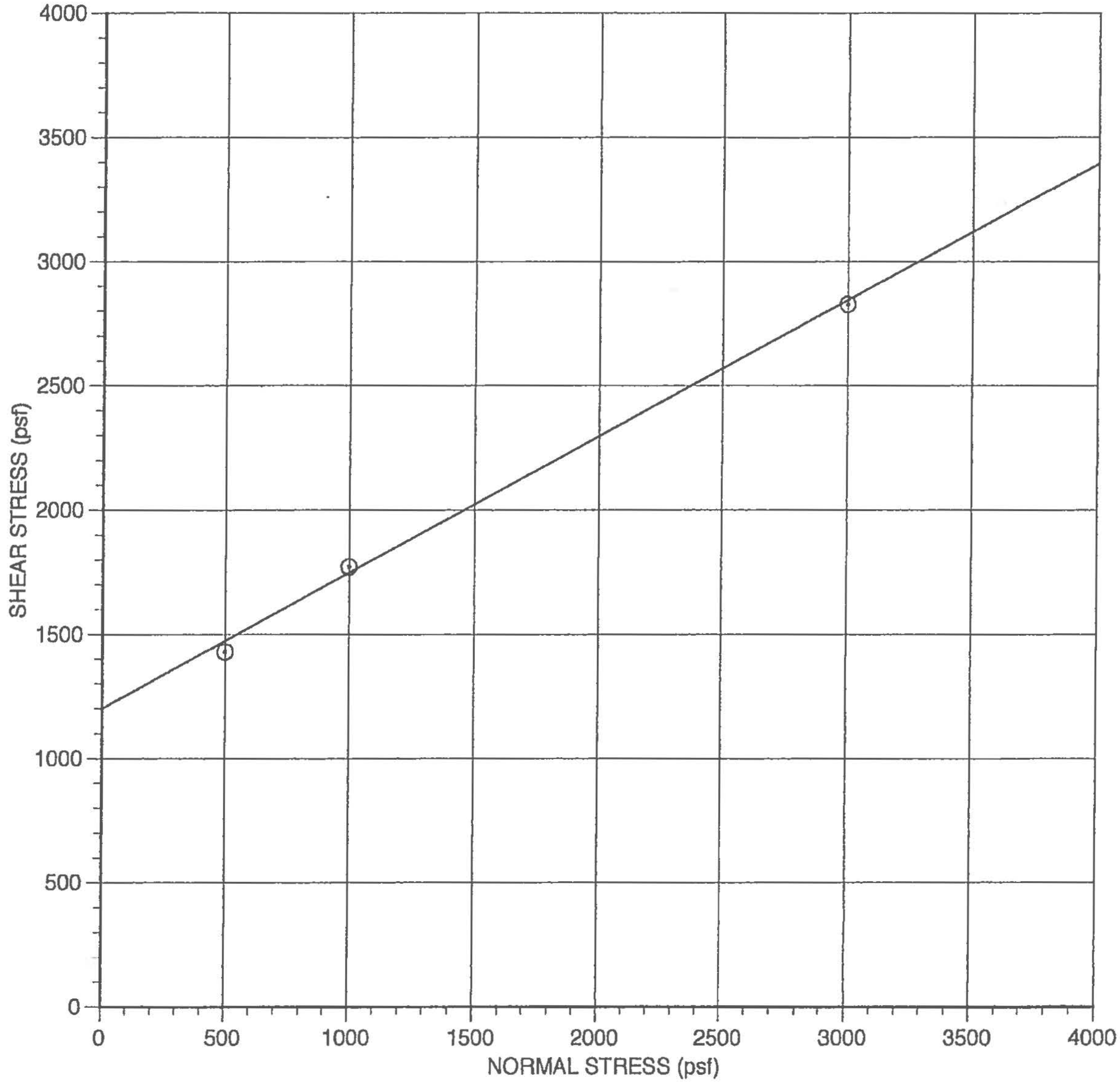
FRICTION ANGLE: 14°

COHESION: 650 psf

SPECIMEN	A	B	C
DRY DENSITY (pcf)	120.2	119.0	120.3
INITIAL WATER CONTENT (%)	11.4	11.7	11.4
FINAL WATER CONTENT (%)	13.2	13.9	13.6
NORMAL STRESS (psf)	500	1000	3000
MAXIMUM SHEAR (psf)	715	932	1336

DIRECT SHEAR TEST

JOB NUMBER: 2616.100 DATE: 4-23-03 BY: PW



LOCATION: TP2-15 at 4 to 8 feet

SAMPLE: CLAYEY SANDSTONE, olive-gray, Remolded to 90% Relative Comaction

TEST TYPE: Consolidated Drained

RATE OF SHEAR (in/min): 0.00099

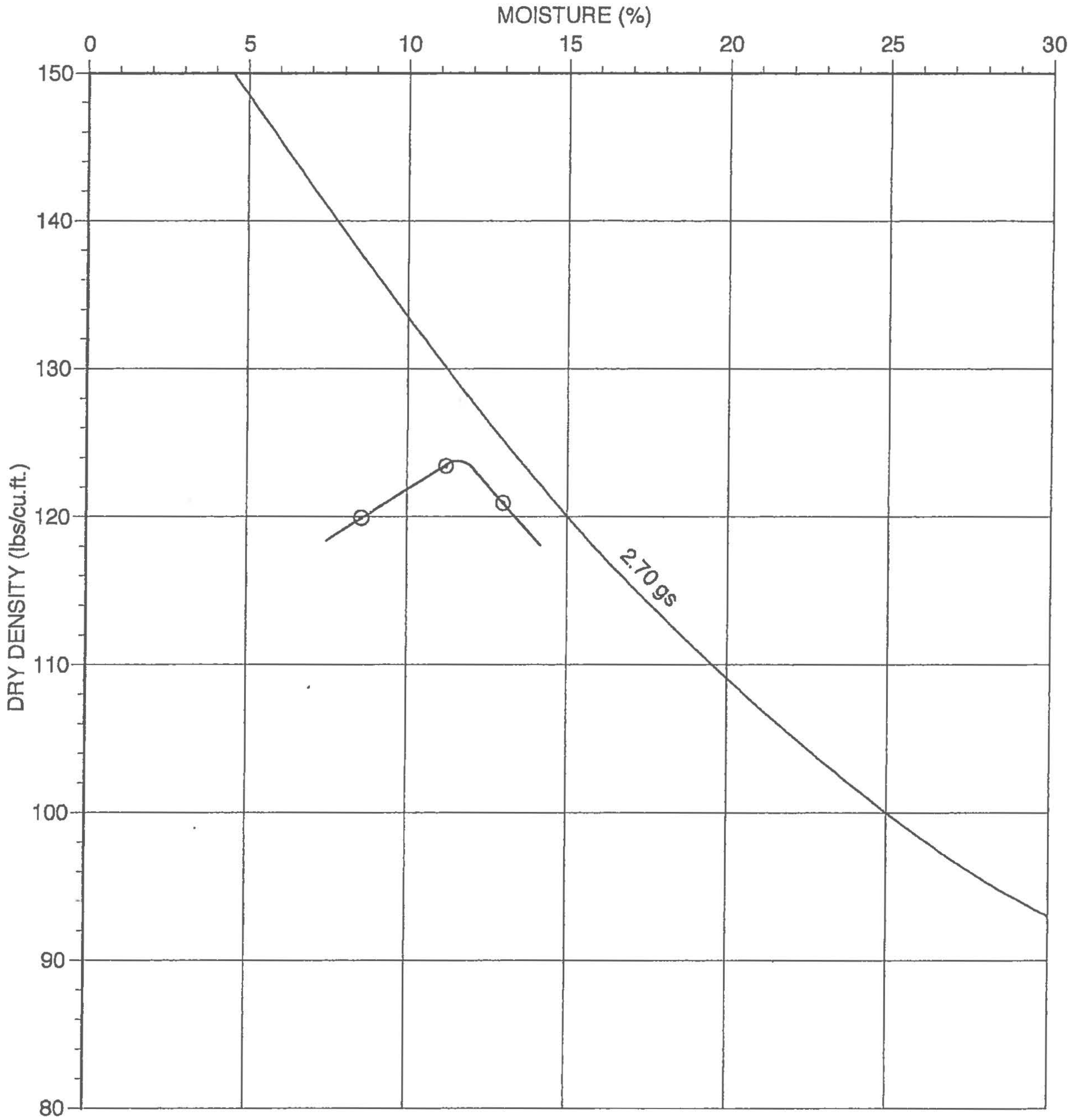
FRICTION ANGLE: 29°

COHESION: 1200 psf

SPECIMEN	A	B	C
DRY DENSITY (pcf)	124.8	124.8	124.4
INITIAL WATER CONTENT (%)	9.5	9.7	9.8
FINAL WATER CONTENT (%)	10.3	11.0	10.5
NORMAL STRESS (psf)	500	1000	3000
MAXIMUM SHEAR (psf)	1429	1771	2827

DIRECT SHEAR TEST

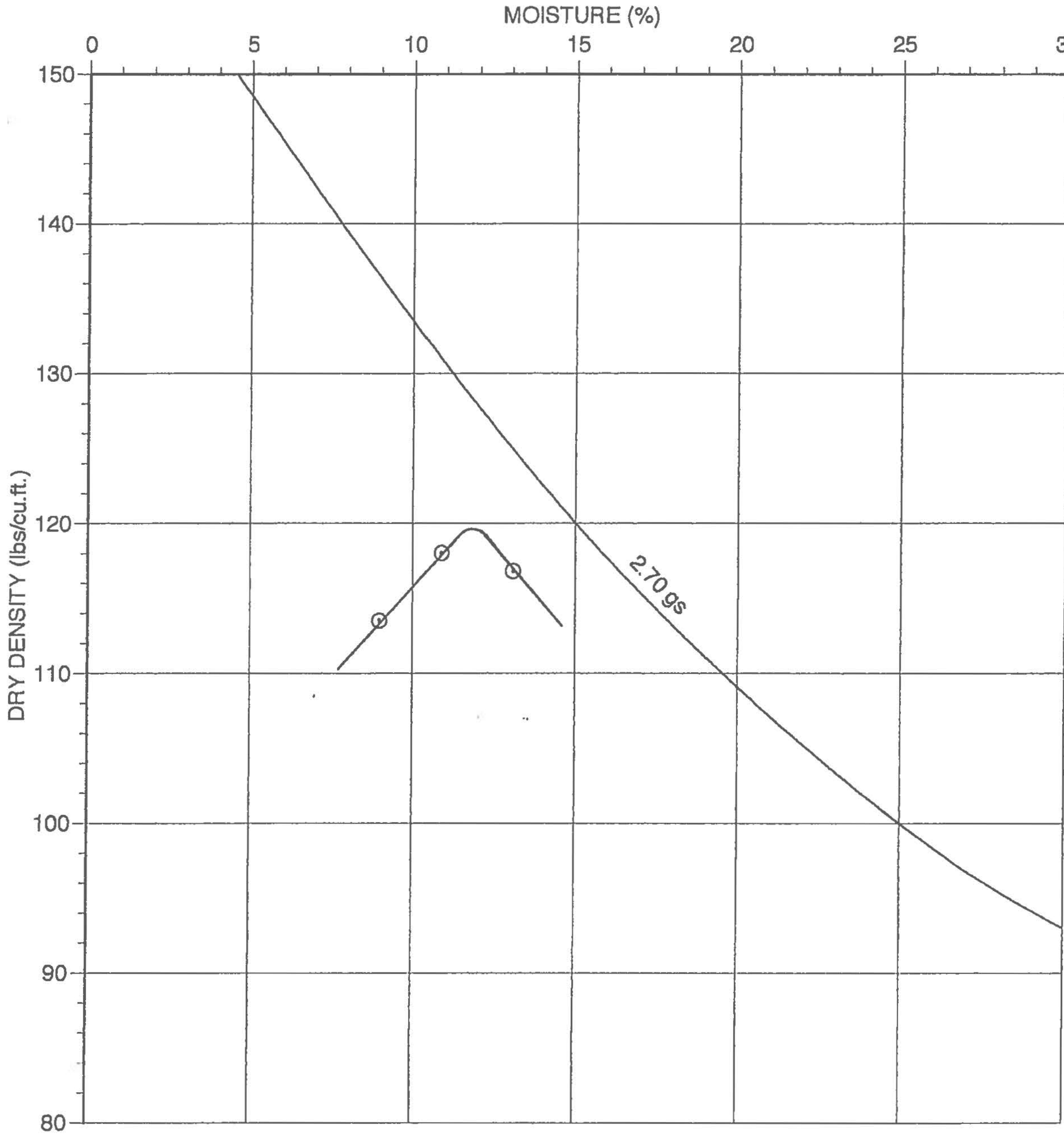
JOB NUMBER: 2616.100 DATE: 4-23-03 BY: PW



SYMBOL	LOCATION	DESCRIPTION	OPTIMUM MOISTURE CONTENT (%)	MAX. DRY DENSITY (pcf)
⊙	TP2-2 at 0-2 feet	SILTY CLAY, brown	11.5	123.6

COMPACTION TEST DATA

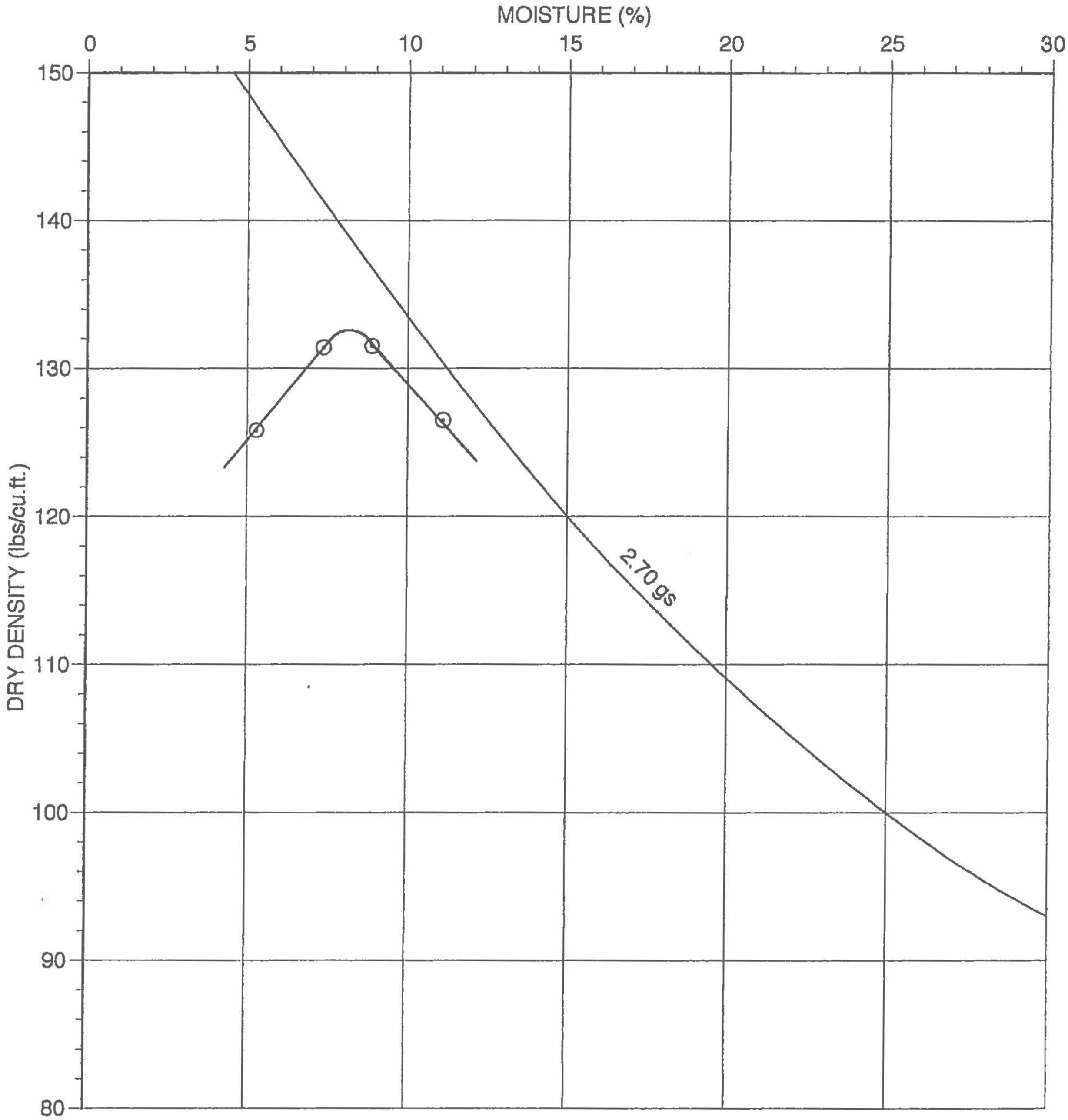
JOB NUMBER: 2616.100 DATE: 4-23-03 BY: PW



SYMBOL	LOCATION	DESCRIPTION	OPTIMUM MOISTURE CONTENT (%)	MAX. DRY DENSITY (pcf)
⊙	TP2-5 at 0-3-1/2 feet	SILTY CLAY, gray brown	11.8	119.5

COMPACTION TEST DATA

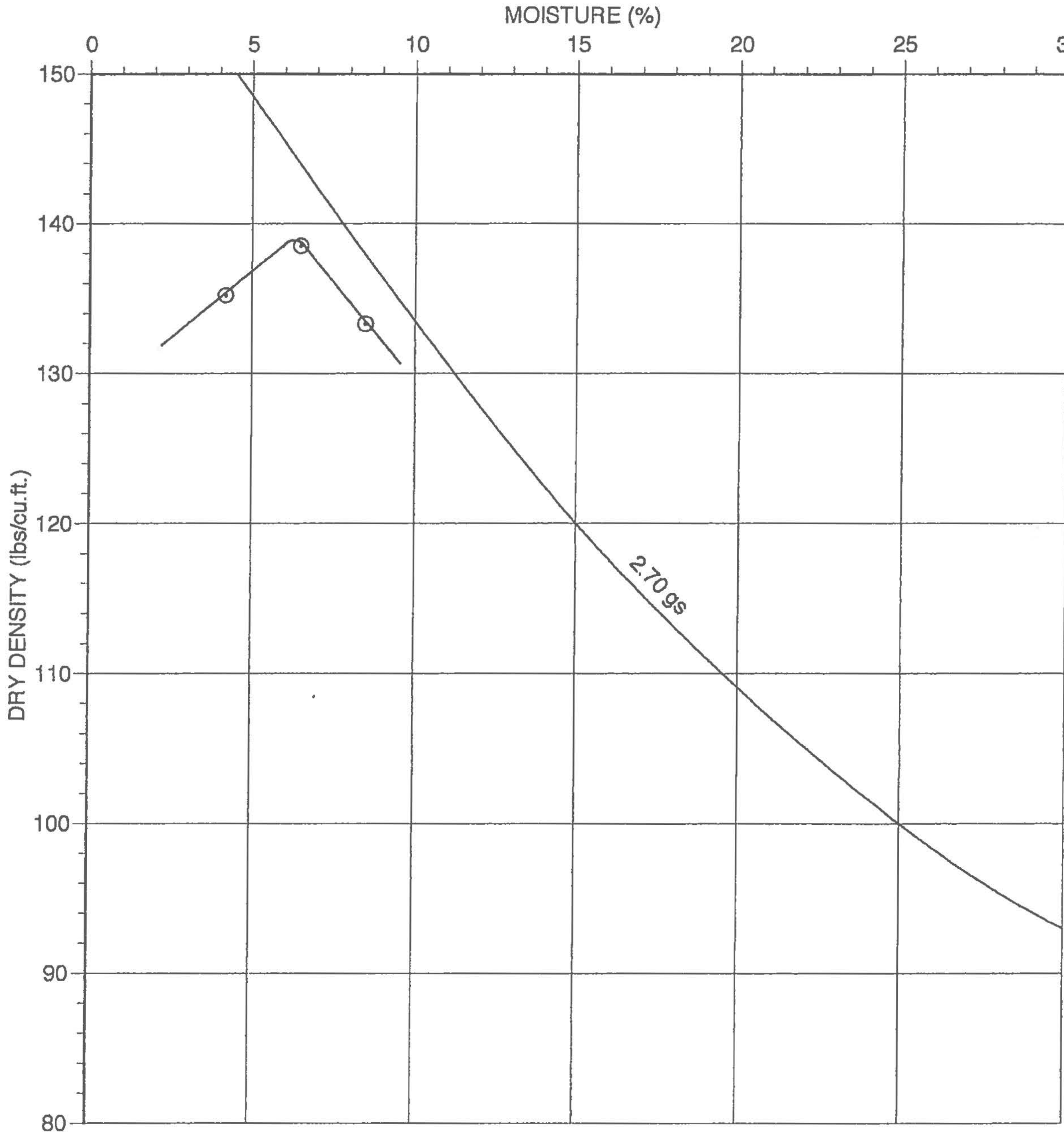
JOB NUMBER: 2616.100 DATE: 4-23-03 BY: PW



SYMBOL	LOCATION	DESCRIPTION	OPTIMUM MOISTURE CONTENT (%)	MAX. DRY DENSITY (pcf)
⊙	TP2-13 at 6 to 9 feet	CLAYEY SILTSTONE, gray-brown	8.2	132.2

COMPACTION TEST DATA

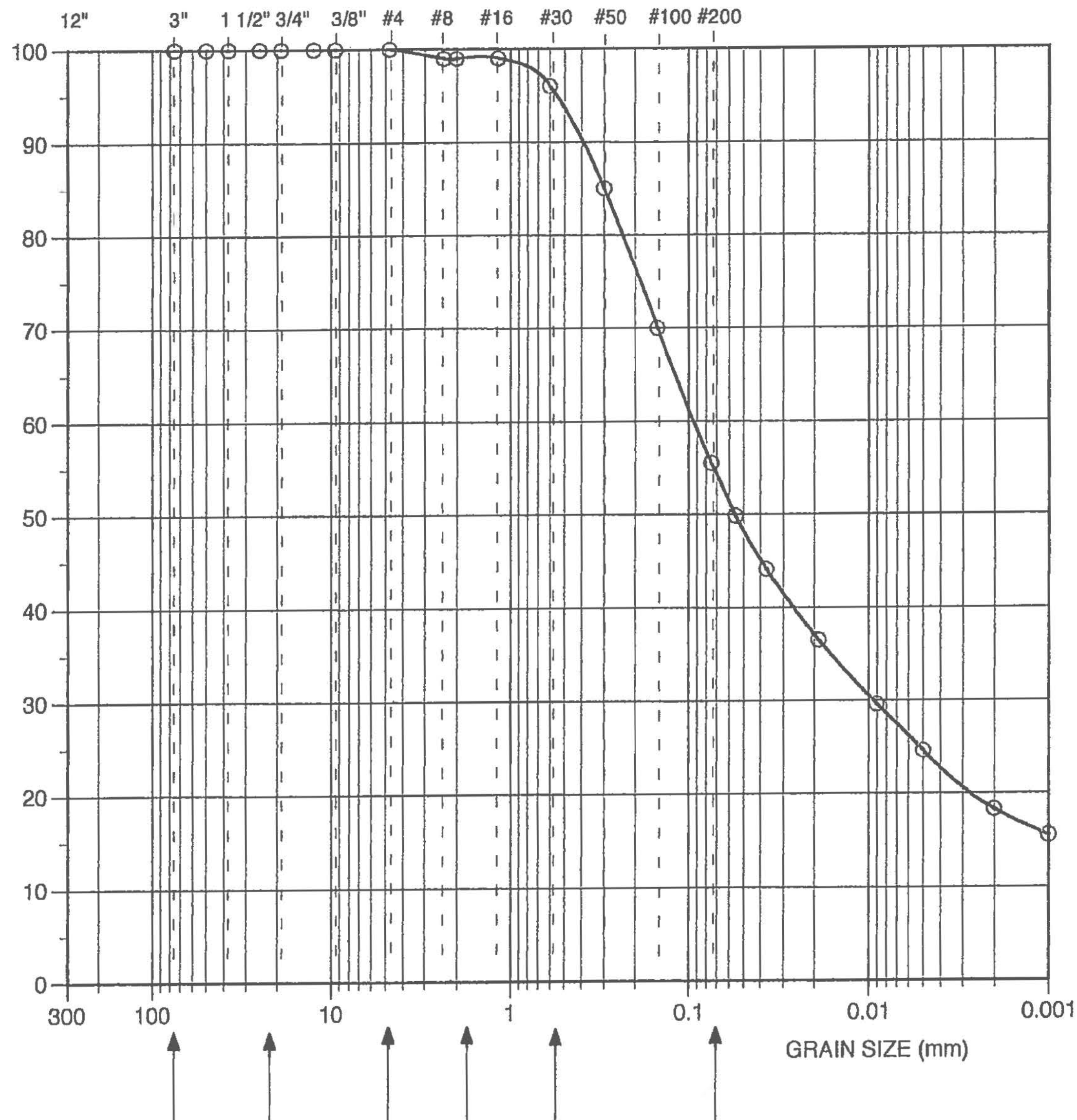
JOB NUMBER: 2616.100 DATE: 4-23-03 BY: PW



SYMBOL	LOCATION	DESCRIPTION	OPTIMUM MOISTURE CONTENT (%)	MAX. DRY DENSITY (pcf)
⊙	TP2-15 at 4 to 8 feet	SILTY CLAY, gray brown	6.2	138.7

COMPACTION TEST DATA

JOB NUMBER: 2616.100 DATE: 5-1-03 BY: FF



COBBLES	GRAVEL		SAND			SILT/CLAY
	coarse	fine	coarse	medium	fine	

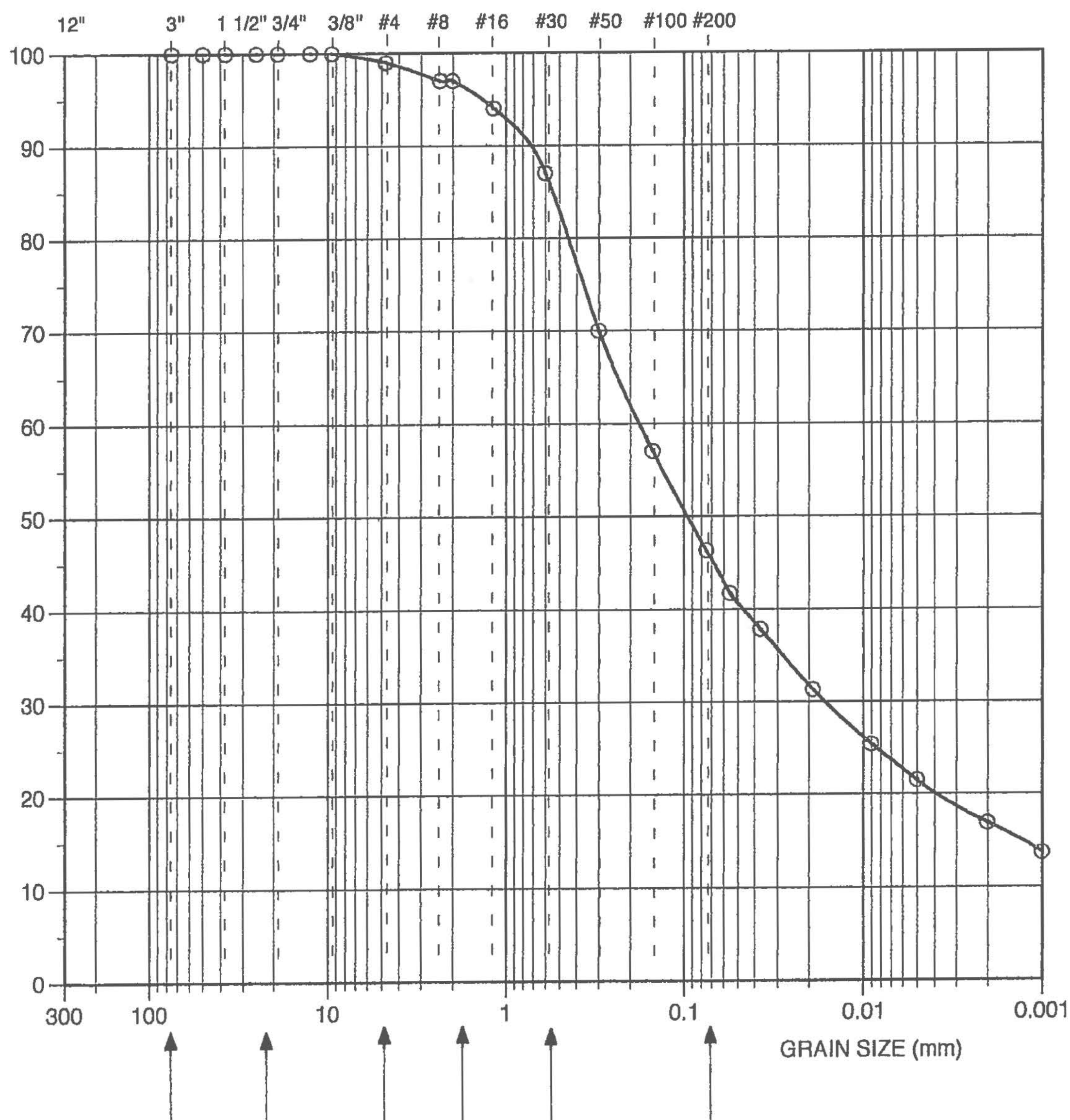
SYMBOLS	LOCATION	DESCRIPTION
⊙	B-8 at 10-1/2 to 12 feet	SANDY SILT, light brown with clay

GRADATION TEST DATA

JOB NUMBER: 2616.100

DATE: 5-1-03

BY: FF

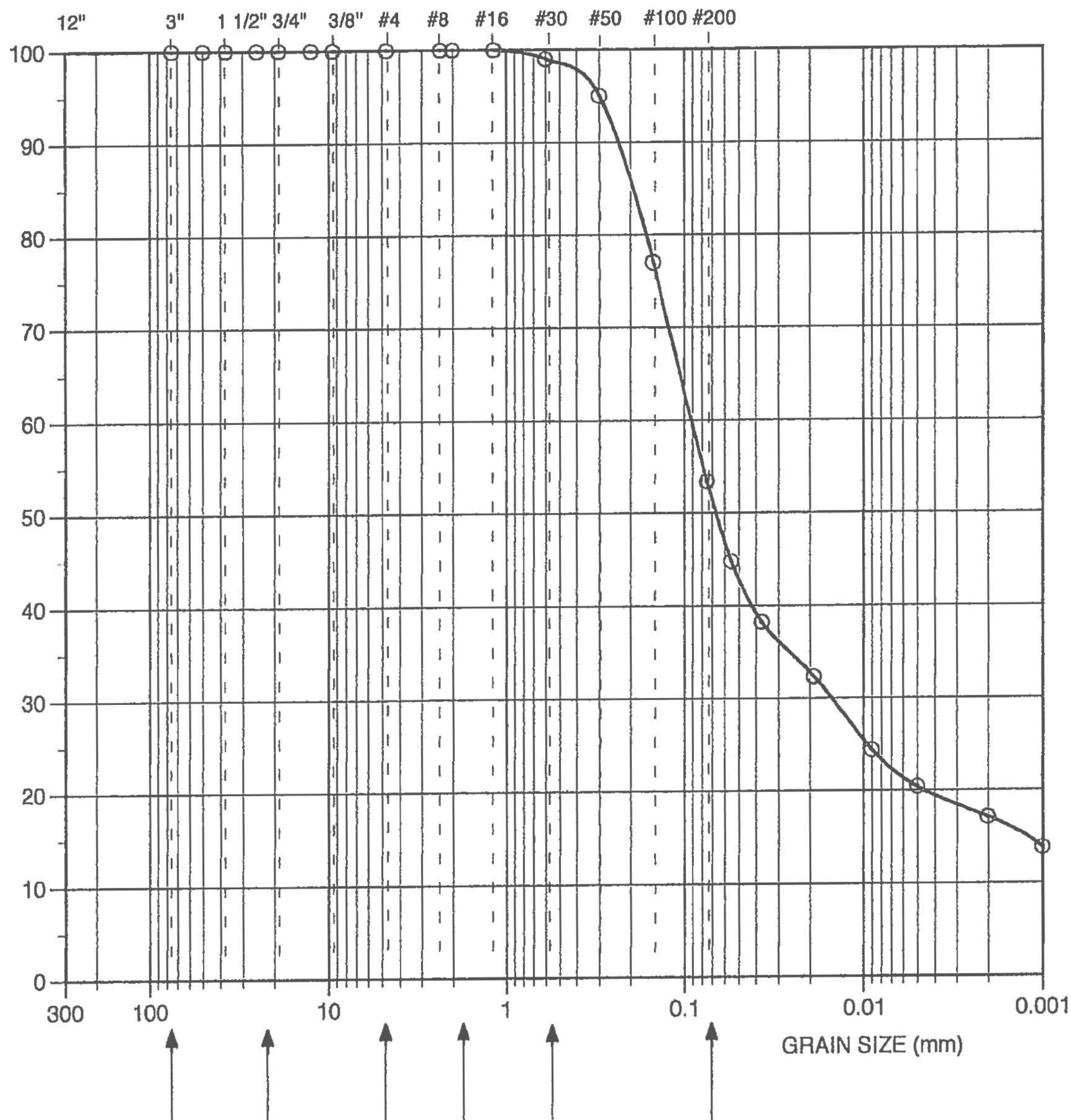


COBBLES	GRAVEL		SAND			SILT/CLAY
	coarse	fine	coarse	medium	fine	

SYMBOLS	LOCATION	DESCRIPTION
⊙	B-8 at 16-1/2 to 18 feet	SILTY SAND, light brown with clay

GRADATION TEST DATA

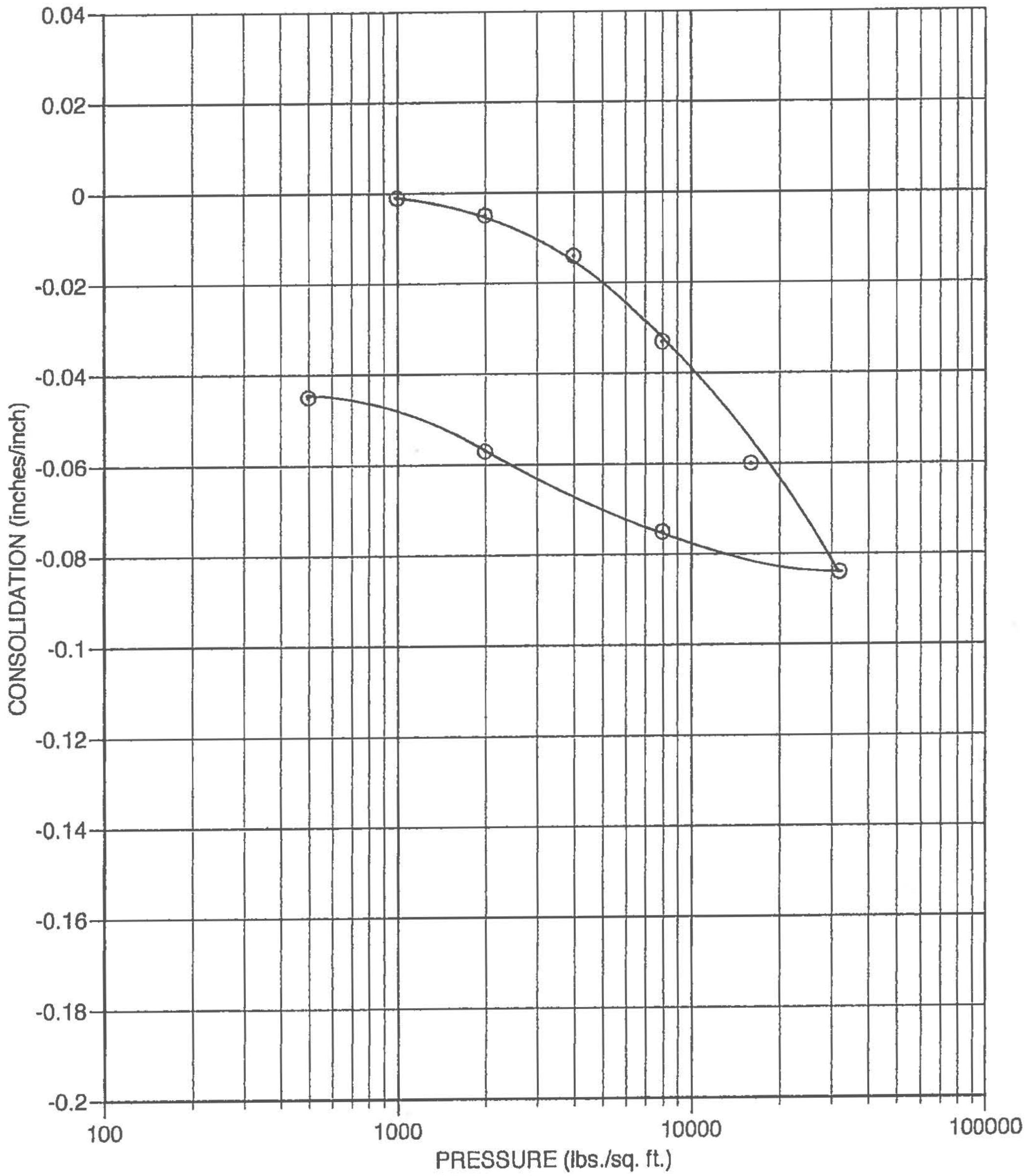
JOB NUMBER: 2616.100 DATE: 5-1-03 BY: FF



COBBLES	GRAVEL		SAND			SILT/CLAY
	coarse	fine	coarse	medium	fine	

SYMBOLS	LOCATION	DESCRIPTION
⊙	B-13 at 10-1/2 to 12 feet	SANDY CLAYEY SILT, red-brown

GRADATION TEST DATA



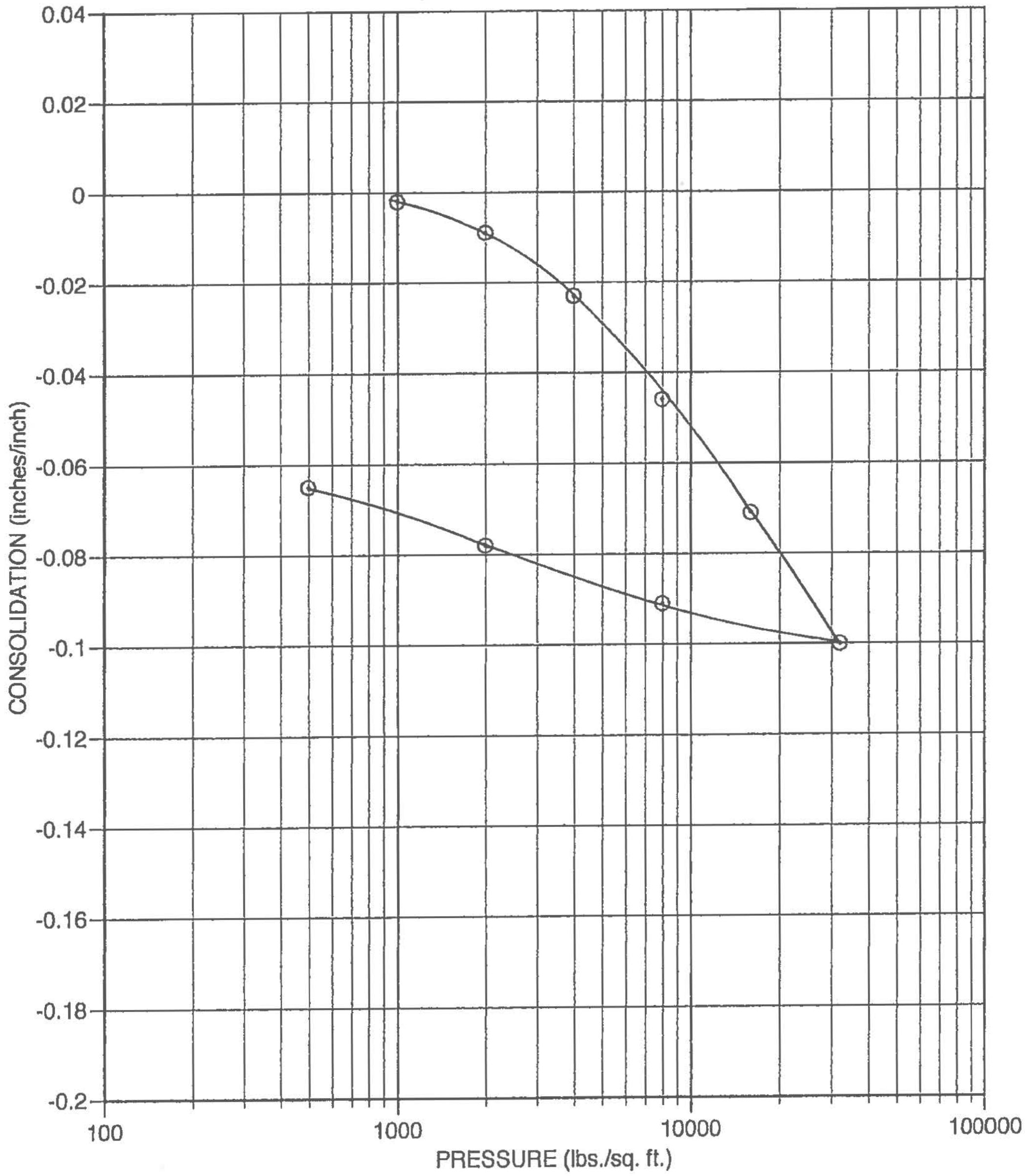
SYMBOL	LOCATION	DESCRIPTION	INITIAL MOISTURE CONTENT (%)	INITIAL DRY DENSITY (p c f)
⊙	B-1 at 5 feet	SILTY CLAY	19.1	109.5

CONSOLIDATION TEST DATA

BY:FF

DATE: 5-1-03

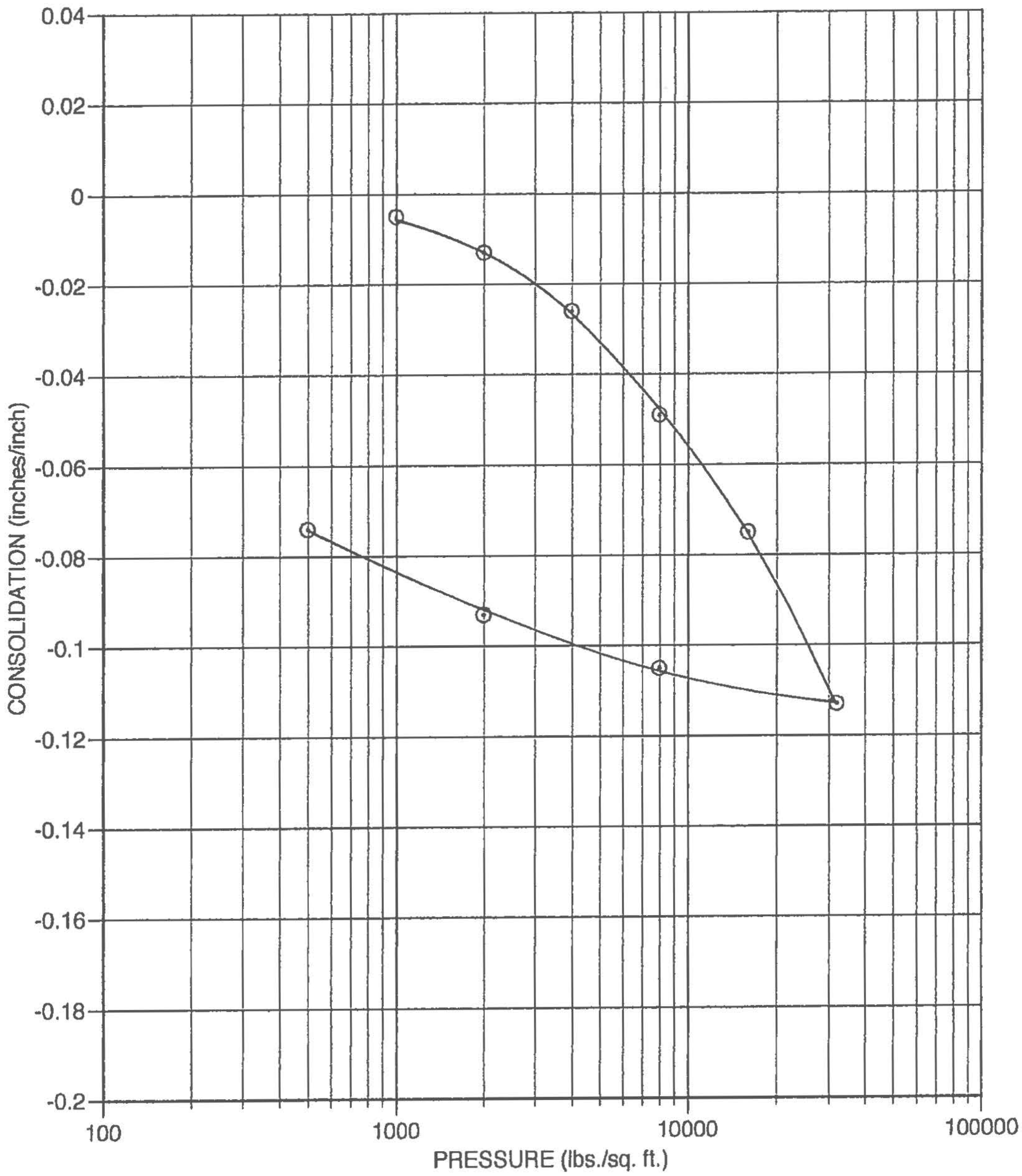
JOB NUMBER: 2626.100



SYMBOL	LOCATION	DESCRIPTION	INITIAL MOISTURE CONTENT (%)	INITIAL DRY DENSITY (pcf)
⊙	B-1 at 10 feet	SANDY CLAY, yellow-brown	22.3	103.0

CONSOLIDATION TEST DATA

JOB NUMBER: 2626.100 DATE: 5-1-03 BY:FF



SYMBOL	LOCATION	DESCRIPTION	INITIAL MOISTURE CONTENT (%)	INITIAL DRY DENSITY (pcf)
⊙	B-9 at 4 feet	SANDY SILTY CLAY, dark brown	21.0	105.1

CONSOLIDATION TEST DATA

APPENDIX C

CERCO Analytical, Inc. Corrosion Test Data

California State Certified Laboratory No.2153

CERCO
analytical, inc.

3942-A Valley Avenue
Pleasanton, CA 94566-4715
Tel: 925.462.2771
Fax: 925.462.2775

24 April, 2003

Job No.0304121
Cust. No.10598

Mr. Paul Lai
Berlogar Geotechnical Consultants
5587 Sunol Blvd.
Pleasanton, CA 94566

Subject: Project No.: 2616.100
Project Name: UOP Property
Corrosivity Analysis – ASTM Test Methods

Dear Mr. Lai:

Pursuant to your request, CERCO Analytical has analyzed the soil samples submitted on April 11, 2003. Based on the analytical results, a brief corrosivity evaluation is enclosed for your consideration.

Based upon the resistivity measurements, Sample No.001 is classified as "corrosive" and Samples No.002 and No.003 are classified as "moderately 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 concentrations range from none detected to 76 mg/kg. Because the chloride ion concentrations are less than 300 mg/kg, they are determined to be insufficient to attack steel embedded in a concrete mortar coating.

The sulfate ion concentrations reflect none detected with a detection limit of 15 mg/kg and are determined to be insufficient to damage reinforced concrete structures and cement mortar-coated steel at these locations.

The pH of the soils range from 6.0 to 8.5 which does not present corrosion problems for buried iron, steel, mortar-coated steel and reinforced concrete structures. However, any soils with a pH of <6.0 are considered to be corrosive to buried iron, steel, mortar-coated steel and reinforced concrete structures, and corrosion prevention measures will need to be considered for structures to be placed in acidic soils.

The redox potentials range from 290 to 400-mV, which are 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.

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

JDH/jdl

CERCO Analytical, Inc.

3942-A Valley Avenue, Pleasanton, CA 94566-4715 (925) 462-2771 Fax (925) 462-2775

FINAL RESULTS

Client:	Berlogar	Date Sampled:	31-Mar-2003
Client's Project No.:	2616.100	Date Received:	11-Apr-2003
Client's Project Name:	UOP Property	Date of Report:	24-Apr-2003
Authorization:	Signed Chain of Custody	Matrix:	Soil

Job/Sample No.	Sample I.D.	Redox (mV)	pH	Conductivity (umhos/cm)*	Resistivity (100% Saturation) (ohms-cm)	Sulfide (mg/kg)*	Chloride (mg/kg)*	Sulfate (mg/kg)*
0304121-001	TP2-13 @ 6-9'	300	8.5	-	1,200	-	76	N.D.
0304121-002	TP2-15 @ 4-8'	290	8.4	-	2,500	-	N.D.	N.D.
0304121-003	TP2-19 @ 1-9'	400	6.0	-	4,800	-	N.D.	N.D.

Method:	ASTM D1498	ASTM D4972	ASTM D1125M	ASTM G57	ASTM D4658M	ASTM D4327	ASTM D4327
Detection Limit:	-	-	10	-	50	15	15
Date Analyzed:	17-Apr-2003	18-Apr-2003	-	23-Apr-2003	-	18-Apr-2003	18-Apr-2003

Cheryl McMillen

Cheryl McMillen
Laboratory Director

* Results Reported on "As Received" Basis

N.D. - None Detected

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

Via E-Mail and Mail

BERLOGAR
STEVENS &
ASSOCIATES

August 12, 2014
Job No. 2616.006

Mr. Jeff Thayer
Davidon Homes
1600 South Main Street, Suite 150
Walnut Creek, California 94596

Subject: Landslide G
Option A – 66 Lots
Option B – 63 Lots
Scott Ranch
D Street and Windsor Drive
Petaluma, California

Dear Mr. Thayer:

This letter contains our opinion on Landslide G remedial work at your Scott Ranch project in Petaluma, California. We provided geotechnical recommendations in our Geotechnical Feasibility Investigation dated March 7, 2002 and our Design-Level Geotechnical Investigation dated April 28, 2014. Our 2002 report was based on 117 lot development.

We originally recommended over excavating the Landslide G debris and replacing it with engineered fill because of the following:

- Two lots were located in the landslide impact zone.
- Landslide G was to be extensively graded with fill depths up to 40 feet.
- Landscape irrigation would introduce excess water into Landslide G.

Our most recent report is based Grading Plans for 66 Lot and 63 Lot development, which designates the land south of Kelley Creek as Open Space. In our report we concluded that Landslide G was dormant. Aside from a minor 3 foot tall Keystone retaining wall, no remedial grading of Landslide G was recommended because:

- The new Grading Plans do not have lots within the landslide impact zone.
- There is no proposed grading within Landslide G.
- There will be no landscape irrigation within the landslide impact zone.

Common Practice for Landslides in Open Space in the Bay Area is to leave them in their natural state. Remedial work would call for the removal of all existing vegetation and trees. Given all the above, it is our opinion that remedial measures for Landslide G, except for the 3-foot tall Keystone retaining wall, need not be performed.

Respectfully submitted,

BERLOGAR STEVENS & ASSOCIATES

Nicholas Cardanini
Staff Engineer

Frank Berlogar
RCE 20383

REGISTERED PROFESSIONAL ENGINEER
FRANK BERLOGAR
No. 20383
CIVIL
STATE OF CALIFORNIA

NC/FB:jmo

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Haley & Aldrich, Inc.
2033 N. Main Street
Suite 309
Walnut Creek, CA 94596

Tel: 925.949.1012
Fax: 925.979.1456
HaleyAldrich.com

HALEY& ALDRICH

11 October 2014
File No. 41431-000

Impact Sciences, Inc.
555 12th Street, Suite 1650
Oakland, California 94607

Attention: Ms. Shabnam Barati, Ph.D.

Subject: Third Party Geological/Geotechnical Review
Davidon Homes – Scott Ranch EIR
Windsor Drive and D Street
Petaluma, California

Ladies and Gentlemen:

Haley & Aldrich, Inc. (Haley & Aldrich) performed a third party geotechnical/geological review for the planned Davidon Homes, Scott Ranch project in Petaluma, California ("site") and this letter presents our review comments. The review of geotechnical/geological information was performed for the preparation of the Environmental Impact Report (EIR). The site is located at the northwest and southwest corners of Windsor Drive and D Street, as indicated on the Site Location Map, Figure 1.

The scope of services for this third party geotechnical/geological review included:

- reviewing available geological data, including the applicant's geotechnical/geological report(s);
- reviewing available geologic and seismicity data;
- performing a site reconnaissance and mapping to update descriptions of site geology and geologic hazards affecting the site;
- reviewing historical, stereo-paired aerial photos;
- reviewing historical ground failures at adjacent properties; and
- reviewing available geotechnical data and reviewing previous public comments to the former draft EIR.

The project documents reviewed for this study included:

- a design level geotechnical investigation report for the subject site, prepared by Berloger Stevens & Associates, Soil Engineers and Engineering Geologists ("BSA"), dated 28 April 2014; and
- public comments in response to the project draft EIR, dated 11 March through 15 April 2013.

This geotechnical/geological review was prepared with the technical assistance of Mr. Chris Hundemer and Mr. Craig Reid of C2Earth, Inc. (C2), who performed site mapping and provided consultations regarding geological and seismological issues. The scope of services did not include subsurface exploration or laboratory testing.

A draft EIR for the subject site was previously published in 2004 and it was reviewed by the public. Based on public comments, the proposed development plans have changed and the draft EIR is currently being revised.

SITE DESCRIPTION

The project site is comprised of two parcels totaling about 58.6 acres located at the northwest and southwest corners of Windsor Drive and D Street in Petaluma. The two parcels are separated by Windsor Drive. The site consists of a relatively flat east-west trending central alluvial plain with steep slopes to the north, west, and south. Elevations on the site range from a low of about 100 feet above sea level along Kelly Creek to a high of about 380 feet above sea level at the southwest corner of the site (BSA, 2014). Kelly Creek bisects the alluvial plain and flows in the west to east direction.

The site is only slightly developed with one modular single-family residence, two barns, the burned remains of a house, and several agricultural structures all in the eastern central portion of the alluvial plain, near D Street. A small stock pond embankment is present in the southern central portion of the site on the north-facing slope south of Kelly Creek. Remnants of an old stone and mortar foundation are present on the hill east of the stock pond. Both the north and south parcels contain 10-foot-wide public utility easements and 15-foot-wide slope easements along the D Street and Windsor Drive right-of-ways. A water booster pump station is located within this easement along Windsor Drive.

Vegetation on the site consists primarily of grass and weeds with scattered oak trees present on the north-facing slopes in the southwestern corner of the site, and at the top of the south-facing slope on the parcel north of Windsor Drive. Oak and bay laurel trees are present along the creek channels, and tall eucalyptus trees are present along D Street near the farm buildings and to the south of the buildings. Blackberry bushes have overgrown the area around the burned firehouse.

The stock pond embankment is about 15 feet high. A low "levee" of fill was graded to control potential overflow toward an existing swale located about 200 feet to the east. An apparently thin sliver of fill also underlies the southern edge of Windsor Drive and the western edge of D Street.

The creek channels were dry at the time of C2's reconnaissance on 10 September 2014. One small seep was observed emanating from a west-facing slope below D Street, just south of where Kelly Creek flows beneath the roadway. A small pool of standing water was observed in the creek channel near the seep, and stagnant water in the stock pond were the only locations where surface water was observed at the site on 10 September 2014.

PROJECT UNDERSTANDING

The revised development application includes: 1) a General Plan Amendment to modify language contained in General Plan Policy 2-P-68; 2) a Zoning Amendment to rezone the property to Planned Unit District (PUD); and 3) a Tentative Subdivision Map to subdivide two existing parcels into either 66 single-family residential lots with plans to renovate and relocate the existing red barn to a different part of the site (Option A) or 63 single-family residential lots with the red barn cluster left in place (Option B). For Option B, the modification of the General Plan Policy 2-P-68 is not required. The proposed layouts for Option A and Option B were presented as an overlay on the 2014 BSA geologic map and are presented on the attached Figures 2 and 3, respectively.

The proposed development includes approximately 35 acres of open space which will include a public parking area, a bicycle and pedestrian path along Kelly Creek from D Street to the western property boundary, which is adjacent to Helen Putnam Park. The project applicant also proposes to maintain a 300-foot urban separator along the southern property line with a recreational trail. The trail is planned from the parking area (in the southeastern corner of the site) traversing the north-facing slopes along the southern property boundary to the property's southwest corner, then north along the western property line and subsequently connecting to an existing trail located within Helen Putnam Park. The trail will traverse existing mapped landslides and erosional features along the southern edge of the site and a relatively steep hillside at the southwestern portion of the proposed path alignment.

The 52-acre parcel on the south side of Windsor Drive contains a stock pond, several wetland areas that were dry at the time of C2's reconnaissance, and a section of Kelly Creek that flows east to west through the parcel. California Red-Legged frogs have been identified in the existing stock pond on the site. The project proposal is to retain the stock pond. Portions of existing drainage that lead from the stock pond to Kelly Creek will be rerouted in some areas to accommodate the proposed development.

SITE HISTORY

Historical site conditions were observed by reviewing aerial photographs dating back to 1950. Eight sets of black and white stereo-paired aerial photographs and one black and white single aerial photograph were reviewed from Pacific Aerial Surveys, Oakland, California. Three additional sets of stereo-paired historical black and white and infrared photographs were reviewed by C2 at the United States Geological Survey (USGS) library in Menlo Park, California to supplement the prior review. The aerial photographs reviewed are listed in Table I. Standard aerial photograph review and photo-geologic mapping techniques were employed to identify significant geologic features at the site and adjacent properties, such as tonal contrasts, vegetation patterns, and abrupt changes in topographic slope. The following sections provide a limited chronology of site development and slope conditions based on the photographs and site reconnaissance.

Table I
List of Aerial Photographs Reviewed[1]
Davidon Homes, Scott Ranch, Petaluma, California

Date	Photograph Number	Scale	Type
10/10/1950	AV 41-02-24*	1: 6,000	Black and White
06/12/1956	AV 222-04-15, -16	1: 24,000	Black and White
05/03/1961	CSH 2BB-031, -032**	1:20,000	Black and White
07/03/1965	SON 67-118, -118**	1:12,000	Black and White
04/14/1966	AV 71-02-11, -12	1: 36,000	Black and White
04/10/1968	AV 844-05-13, -14	1: 30,000	Black and White
09/25/1973	SFB 2-58, -59**	1:20,000	Infra-red
05/03/1980	OIR-SON-19-29, -30	1: 24,000	Black and White
04/19/1986	AV 2860-07-31, -32	1: 12,000	Black and White
04/23/1992	AV 4252-24-49, -50	1: 12,000	Black and White
03/15/1996	AV 5132-110-02, -03	1: 24,000	Black and White
06/15/2000	AV 6540-19-34, -35	1: 12,000	Black and White

* Single photograph
** Reviewed at the USGS Library in Menlo Park as part of this evaluation

Development History

The earliest available aerial photographs dated 1950 showed site improvements consisting of a farmhouse, two barns, and associated structures near the east side of the site. Thick tree cover was also observed along Kelly Creek. In June 1956, most of the site north of the north-facing slope on the southern third of the site had been mowed and the grass collected into bales. The small stock pond embankment on the north-facing slope was visible in the May 1961 photographs. The stock pond was constructed by building an earth berm, possibly using on-site materials excavated nearby. The 1980 photographs indicated that the farmhouse at the site was partly burned and the modular home was constructed. Windsor Drive and residential developments north and northeast of the site are visible in the April 1992 photographs. A portion of Windsor Drive was constructed on the site property, and no residential development has been completed along this portion of the street. The site conditions remain relatively unchanged after 1992.

Historical Slope Conditions

Several landslides are visible on the site hill slopes in many of the photographs reviewed. Two apparently active slides, landslides A and C (BSA, 2014), are visible near the southeast corner of the site, just south and upslope of the tributary channel to Kelly Creek. The ground surfaces of these two landslides in photographs taken after 1961 are hummocky; however, bare soil or rock is not exposed in scarps at the crest or sides of these landslides. This may indicate either slow creeping movement of these features or

[1] Aerial photographs provided by Pacific Aerial Surveys in Oakland, California unless otherwise noted.

it may indicate the passage of sufficient time since slide movement occurred that vegetation has become established on the scarp areas. A broad swale with slightly hummocky topography was observed in the area of Landslide F in the northwest corner of the property.

During C2's site reconnaissance on 10 September 2014, C2 identified a near vertical cut slope excavated south of the head-scarp of the mapped Landslide G. This cut appears to be a remnant from a former small quarry borrow pit.

Shallow landslides and raveling at landslides N, O, and P (BSA, 2014) are visible on the steep slopes on the north bank of the incised Kelly Creek channel in the western half of the site, where the tree canopy does not obscure the underlying slopes in the photographs reviewed. Older landslides, landslides E and H (BSA, 2014), are visible immediately north of Kelly Creek.

To the west of the project site and within Helen Putman Regional Park, the south-facing slopes on the north side of Kelly Creek were observed to be recently disturbed, having no visible vegetative cover, suggesting either recent shallow landsliding or erosion. West of Helen Putnam Regional Park, moderate-size fans from possible debris flows were observed emanating from north-facing drainage area of the current development along Cambridge Lane.

REGIONAL GEOLOGY

The site is located in the Coast Ranges geomorphic province of California, characterized by northwest-southeast trending valleys and ridges. These topographic features are controlled by folds and faults that resulted from the collision of the Farallon and North American plates and subsequent predominantly strike-slip faulting along the San Andreas Fault system.

Regional geologic mapping shows the site vicinity to be underlain by bedrock of the late Jurassic through Cretaceous age (about 160 through 65 million years old) Franciscan assemblage, and undifferentiated Miocene and Pliocene age (about 24 through 1.8 million years old) Sonoma Volcanic rocks (Blake et al., 2000). The Franciscan rocks mapped on the majority of the site are described as mélange, primarily sheared shale and sandstone with resistant masses of chert, greenstone, and meta-graywacke. The undifferentiated Sonoma Volcanic rocks are mapped on the northern edge of the site and to the north and east of the site and consist of rhyolite, andesite, basalt, and tuff.

SITE GEOLOGY

BSA identified eight large landslides, designated as landslides A through H, affecting the site and several smaller landslides on the over-steepened banks along the riparian corridor of Kelly Creek. The locations of the landslides are shown on the Site Geologic Maps, Figures 2 and 3, which are based on "Plate 2" and "Plate 3" of the 2014 BSA report. The BSA report indicates the larger landslides are relatively shallow, and up to about 12 feet thick. Three bedrock shear zones are identified by BSA: two across the southern half of the west site boundary and a third short zone was identified at the center of the north edge of the site, as shown on Figures 2 and 3.

C2 performed a reconnaissance of the site on 10 September 2014 to observe the site conditions and geology. During this visit, C2 mapped the geology of the site and reviewed the geologic mapping prepared by BSA. Two of the larger landslides (landslides A and E) and a smaller landslide that has developed within landslide G were observed to be recently active. The landslide activity associated with Landslides A and E near the southeast corner and the center of the western border of the site is visible in the aerial photographs taken after 1961 and was confirmed by C2 in the field. The ground surface of these two landslides is hummocky, and there was a small pool of water in the channel of the creek at the toe of the smaller slide (designated as a portion of landslide G in the BSA report). Several shallow landslides (designated as N, O, P, and R in the BSA report), as well as other smaller unnamed slides were confirmed by C2 on the steep slopes along the north side of Kelly Creek and on the western half of the site where the creek has eroded and over-steepened the bank.

C2 could not confirm the presence of landslides B, F, and the remainder of landslide G, as described in the BSA report. BSA explored landslide B by excavating and logging four test pits, one of which was located in a mapped shear zone. Based on C2's review, the logs of these pits do not indicate the presence of landslide materials or a basal landslide plane. BSA explored landslide F by excavating and logging two test pits, one of which identified a "sharp basal contact, possible slide plane" with an 18 degree dip (direction not specified). If landslide F exists, the lack of surficial evidence defining the limits of the deposit indicates that it has not moved in several years. BSA explored landslide G by excavating one test pit, where they encountered a 1/4-inch-thick, slicken-sided clay slide plane. However, the test pit was excavated in the middle of a mapped shear zone; therefore, it is difficult to determine if the slicken-sided plane is associated with a landslide or is an inherent feature of the shear zone.

C2 observed a short steep 15- to 20-foot wide unvegetated slope near the west property line about 80 feet south of Kelly Creek. C2 indicates this feature was interpreted by BSA to be a narrow landslide scarp within landslide G. However, C2 notes that this interpretation was not confirmed with subsurface exploration, and no hummocky or bulging accumulation of slide debris was present downslope of this feature.

C2 identified rock outcrops of weak to moderately strong, moderately hard Franciscan assemblage graywacke sandstone and shale are present along the channel of Kelly Creek and in the tributary that intersects Kelly Creek from the south near D Street. This bedrock is moderately weathered, fine-grained with a trace of lithic fragments, and is intensely fractured. Several scattered outcrops of Franciscan graywacke sandstone are also present on the steep north-facing slopes on the southern third of the site. One outcrop of dark reddish brown and black banded Franciscan chert was observed in the past by Gilpin Geosciences, Inc. in 2004 and during the recent reconnaissance by C2 located along the west property line at the top of landslide H. This outcrop had previously been identified by BSA as Franciscan sandstone.

A cut along the west side of D Street, on the east end of the hill north of Windsor Drive, exposes dark gray to dark brown, moderately weathered basalt of the Sonoma volcanics. C2 indicates that it is possible that this rock also underlies the hill to the west, although Franciscan meta-sandstone was observed in rock outcrops at the top of the slope in the northeast corner of the property. Local geologic maps for the subject site indicate that Sonoma volcanic rocks are present on the north portion of the site. However, tonal vegetation changes and an east-west trending lineament observed in the aerial photographs suggest the contact between the Franciscan assemblage and Sonoma volcanics daylights in areas that are north of the subject site.

C2 indicates that topographic evidence of the three bedrock shear zones mapped by BSA was not visible on the surface of the site during the site reconnaissance on 10 September 2014. In the area of the shear zone that BSA mapped in the southwest corner of the property, C2 observed outcrops of indurated sandstone and possibly silica-carbonate rock. C2 observations are presented on Figures 2 and 3.

SUBSURFACE CONDITIONS

BSA performed a design-level geotechnical investigation in 2003 to 2004 for the proposed development, the results of which are presented in a report dated 22 September 2004, and subsequently updated in a report dated 28 April 2014. Their field exploration included drilling 14 borings and excavating 36 test pits at the site to characterize the engineering properties of soil and bedrock at the site. BSA previously performed a geotechnical feasibility investigation in 2002. The feasibility investigation included excavating 26 test pits and one short trench. Based on the subsurface investigations, BSA concluded six types of soil/bedrock were encountered at the site: artificial fill (Qaf), landslide deposits (Qls), colluvium (Qc), alluvium (Qal), Franciscan complex sandstone (KJfss), and Franciscan complex sandstone and shale (KJfss/sh).

BSA indicated isolated areas of artificial fill were encountered in three main areas: 1) beneath and around existing buildings, 2) adjacent to the stock pond, and 3) along the downslope (south) side of Windsor Drive. BSA describes the fill as generally consisting of dense sandy silt and gravel and stiff to very stiff silty clay. BSA mapped the central half of the site along Kelly Creek as being covered with alluvium and the adjacent swales as covered with colluvium. Alluvium, consisting of stiff to very stiff and medium dense to dense sandy clays and clayey sands with various amounts of gravel was mapped by BSA in relatively flat areas bordering drainage courses and at the down-slope end of the swales. Colluvium, consisting of stiff to very stiff clay with minor amounts of gravel was present in the swales and lower portions of the site. Laboratory test results indicate the alluvium and colluvium at the site is moderately expansive. BSA mapped two types of Franciscan bedrock on the site. Sandstone (KJfss) is mapped along most of the northern edge of the site and at scattered outcrops in the channel of Kelly Creek. Sandstone and shale (KJfss/sh) is mapped on the majority of the upland portions of the site.

GEOLOGIC AND SEISMIC HAZARDS

Potential geologic and seismic hazards at the project site include fault rupture, landslide hazards, erosion, flooding, and expansive soil. These and other geologic and seismic hazards are discussed in the following sections.

Fault Rupture

BSA indicates the site is not located within a State of California Earthquake Fault Zone and there is no evidence of an active fault crossing or trending toward the site. BSA indicates the nearest mapped active fault to the site is the Rodgers Creek fault, about 11 kilometers (km) northeast of the site.

Haley & Aldrich concurs with BSA's assessment that the site is not located within an Earthquake Fault Zone. No active faults or extensions of active faults are mapped on the site, and surficial indications of faulting on the site were not identified during C2's site reconnaissance or in the review of historical aerial photographs. Haley & Aldrich estimates the nearest mapped active fault, the Rodgers Creek fault, lies about 8.5 kilometers east of the project site. Haley & Aldrich concludes the potential for fault rupture at the site is low.

Seismic Hazards

Seismically-induced landsliding could potentially be a hazard in areas of moderate to steep slopes underlain by thick soils, weak or fractured rock (i.e., Franciscan melange), previously existing landslides, or loose fill. Mitigation alternatives for seismically induced landsliding include grading and drainage measures in areas of existing landslides and steep slopes, and setbacks from incised stream channels. BSA stated that the stability of all slopes is reduced during an earthquake event. However, grading in accordance with the recommendations presented in the BSA report will help to mitigate the risk of seismically induced landslides. Haley & Aldrich concurs with BSA's assessment for mitigating seismically-induced landslide hazards.

In addition to triggering landslides, strong ground shaking caused by large earthquakes can induce ground failures, such as liquefaction,[2] lateral spreading,[3] and cyclic densification.[4] A site's susceptibility to these hazards relates to the site topography, soil conditions, and/or depth to groundwater. BSA indicated material susceptible to liquefaction or significant dynamic densification was not encountered at the site. Haley & Aldrich reviewed the test pit and boring logs prepared by BSA and conclude that the soil at the site appears to have sufficient fines and/or density to resist liquefaction and cyclic densification. Therefore, Haley & Aldrich concurs with BSA's evaluation and conclusion that the potential for liquefaction or seismically induced differential settlement to occur at the site is very low. In addition, Haley & Aldrich concludes the potential for liquefaction-induced hazards, such as lateral spreading, is also very low.

Bedrock Shear Zones

The BSA report indicates the three bedrock shear zones identified on the site represent ancient shearing within the Franciscan assemblage that occurred during its emplacement onto the North American continent, and therefore, the risk of surface displacement along the shear zones is very low. If the rock

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[2] Liquefaction is a phenomenon in which saturated, cohesionless soil experiences a temporary loss of strength due to the buildup of excess pore water pressure, especially during cyclic loading such as that induced by earthquakes. Soil most susceptible to liquefaction is loose, clean, saturated, uniformly graded, fine-grained sand; however, low plasticity silts and clay can also liquefy.

[3] Lateral spreading is a phenomenon in which surficial soil displaces along a shear zone that has formed within an underlying liquefied layer. Upon reaching mobilization, the surficial blocks are transported downslope or in the direction of a free face by earthquake and gravitational forces.

[4] Cyclic densification is a phenomenon in which non-saturated, cohesionless soil is densified by earthquake vibrations, causing ground surface settlement.

is sufficiently sheared and weathered, it may require mitigation if construction is proposed nearby. The current project documents prepared by BSA (2014) indicate that only the short shear zone at the north edge of the site will extend into the proposed developed portion of the site. The BSA report indicates that if zones of soft or saturated soil are encountered during site preparation and grading activities, excavations deeper than those recommended in the report may be required to expose competent materials. The limits of excavation will be determined in the field by the soil engineer. Haley & Aldrich concurs with BSA's assessment and proposed mitigation.

Landslides

A large-scale regional landslide map of Sonoma County identifies slides on most of the slopes of the site (Huffman and Armstrong, 1980). BSA identified eight large landslides on slopes within the subject site, and several smaller landslides on the banks along Kelly Creek. Four of the large landslides (as identified in the BSA report) are located partially or entirely within the proposed development area (BSA, 2014). The other four large landslides and the smaller landslides along the creek bank are outside the proposed development area.

BSA stated the preferred remedial measure is complete removal of landslide debris located within the development area, and rebuilding of slopes with engineered buttress fills. Due to limitations, such as landslides extending beyond property lines, BSA concluded complete removal of landslide debris is impractical in some areas, and the potential for adverse impacts to the planned development from the partial removal of a landslide can be minimized by implementing the following remedial measures:

- remove landslide debris where feasible and replace excavations with engineered fill provided with keyways and proper subdrainage; and

- construct geo-grid reinforced mechanically stabilized earth (MSE) retaining walls.

In the 2014 report, BSA recommended all keyway excavations should be mapped by an engineering geologist prior to backfilling. Also, BSA recommended subdrain locations and final subdrain trench configurations be approved by a soil engineer. Due to the potential for adverse impacts to the development associated with landslides and landslide mitigation activities, Haley & Aldrich concurs with BSA's recommendations. In addition, Haley & Aldrich suggests that a qualified engineering geologist and/or geotechnical engineer be retained by the site developer to observe all landslide remediation activities, and check that all landslide debris within designated excavation areas have been properly removed, and engineered fill, including fill for keyways, subdrains, and MSE retaining walls have been properly constructed.

<u>Debris Flow and Sedimentation</u>

BSA concluded that debris flow potential was identified in the southwestern drainage courses; however, the drainage courses are located in open space and remedial treatment is not required.

Haley & Aldrich recommends that the locations of previous debris flows be included in the site geologic maps. BSA should provide further explanation that the potential debris flow will not enter the development area by providing calculations of anticipated debris flow run-out.

If needed, a concrete headwall, berm, or other mitigation measures should be used to divert debris flows away from areas of proposed development or to create a catchment area for potential debris. Haley & Aldrich recommends the deflection structures be designed to resist the impact force associated with potential debris flows. The catchment area volume should be adequately sized to contain the anticipated volume of potential debris flow material. Also, Haley & Aldrich suggests a homeowner's association or similar entity be assigned the responsibility for periodically inspecting and maintaining the debris flow catchment system.

Alternatively, the stability of debris flow material could be assessed, and, if necessary, mitigative actions can be implemented to either prevent debris flows from occurring or protect downslope properties from being adversely impacted by potential debris flows. The evaluation of debris flow hazards with mitigation recommendations should be provided by BSA.

Temporary Cut Slopes

The BSA report indicates there are no adverse bedding conditions at the locations of proposed cut slopes; however, due to folding and shearing of the bedrock, localized areas of adverse bedrock structure or other zones of geologic weakness could be exposed during grading of cut slopes. Any adverse bedding that exists will increase the potential for landsliding.

Haley & Aldrich concurs with BSA's recommendation that all keyway excavations should be mapped by an engineering geologist prior to backfilling. Haley & Aldrich also recommends that an engineering geologist map any temporary cut slopes for fill benches or for proposed retaining walls. Field supervision by a qualified engineering geologist will allow for the timely identification and mitigation of adverse bedding conditions, if they are encountered during construction.

Erosion

C2's site reconnaissance and aerial photograph review indicate erosion is occurring along north-trending incised channels that discharge into Kelly Creek. The drainages extend from two north-tending swales in the central southern portion of the site, the western of which has since been truncated by the embankment for the stock pond. The head of the eastern of the two incised channels has eroded more than 80 feet towards the southwest since the time of BSA's survey. This channel extends into the area of several of the proposed home-sites.

The current location of the western incised channel (downslope of the pond embankment) appears to be in about the same state as when BSA performed their survey. During our review of aerial photographs, this western channel previously extended further south to limits near the embankment; portions of the channel appear to have been filled by man-made or natural depositional mechanisms since 1973. Undocumented fill may still exist in this area.

The presence of bedrock in the floors of these two channels indicates that down-cutting is relatively slow along these seasonal streams, and lateral erosion of unconsolidated materials in the channel banks appears to be the main mode of erosion. A small gully was observed in the alluvium across the very gently northeast-sloping valley bottom north of the stock pond. The source of the water that caused the erosion is not known at this time.

To mitigate the potential for future erosion, BSA recommended all cut and fill slopes be planted with fast growing, deep-rooted vegetation before the first winter.

Haley & Aldrich concurs with BSA's recommendations to mitigate the potential for erosion. In addition, Haley & Aldrich suggests controlling surface runoff and directing it away from potentially unstable site slopes and proposed improvements, and using erosion control blankets and fiber rolls to temporarily protect the slope surfaces from erosion until adequate deep-rooted vegetation is established.

Flooding

The site is above the Federal Emergency Management Agency's (FEMA) 100-year and 500-year flood zones for the Petaluma River (FEMA, 1989). The site is not susceptible to tsunamis or seiches. However, if the small stock pond embankment were to fail while retaining water, the water would be released downslope towards the open space area and Kelly Creek. Haley & Aldrich recommends that the volume of potential flow be used to evaluate the impact on the downslope properties and creek capacity. If the evaluation indicates that a potential release of pond water is not adversely impacting the planned development, then no further mitigation is necessary. However, if the sudden release of pond water is found to adversely impact Kelly Creek or the planned development, then further mitigation is necessary. Mitigation measures may include constructing a keyed earth buttress with a slope inclination not to exceed 2:1 (horizontal:vertical), rip-rap, or other stabilization measures. Subsurface drainage should also be installed during the construction of the earth buttress.

Expansive Soil

Expansive soils shrink or swell with changes in moisture content. Clay mineralogy, clay content, and porosity of the soil can influence the shrink-swell behavior of the soil. The shrinking and swelling caused by expansive clay-rich soil can result in damage to overlying structures and surface improvements. Site soils encountered by BSA were found to be moderately expansive. The BSA report indicates that 5- to 15-foot-thick layers of compacted fill may swell between 3/4 and 1-1/4 inches, respectively. The actual swell will depend on the total thickness of fill, material in the fill, and in-place moisture content and density. The BSA report concluded the percentage of fill was insignificant.

Haley & Aldrich believes that 3/4 to 1-1/4 inches of swelling is significant and should be mitigated. Haley & Aldrich suggests that mitigation alternatives for moderately expansive soils be considered, including moisture conditioning and re-compacting moderately expansive soil, using select, non-expansive fill beneath homes and rigid surface improvements, or designing foundations to resist or tolerate differential movement of moderately expansive soil. In addition, Haley & Aldrich suggests that site grades be designed to slope away from the proposed structures, water from roof drains be directed to suitable outlets, and that fill slopes constructed using moderately expansive soil be evaluated for stability or engineered with a flatter than 2:1 (horizontal:vertical) gradient, if necessary.

GEOTECHNICAL AND FOUNDATION ISSUES

The BSA report concludes the site can be developed as proposed, provided the conclusions and recommendations presented in the report are incorporated into the project design and construction. According to BSA, the primary geotechnical considerations are landslide remediation, treatment of existing fill, fill slope construction, stability of proposed cut slopes, and the potential for expansion and settlement of on-site earth materials.

The primary conclusions and recommendations presented by BSA and our associated comments are summarized in the following sections.

Foundations and Settlement

BSA concluded proposed residential structures may be supported on either structural mat/slab foundations or drilled, cast-in-place concrete piers and grade beams.

BSA recommended structural mat/slab foundations should consist of either conventional reinforced or post-tensioned concrete slab foundations. The mat/slab foundation should be designed to accommodate 2 inches of total soil movement and 1 inch in 25 horizontal feet of differential soil movement. The mat/slab foundation may be designed for an allowable bearing pressure of 1,500 pounds per square feet (psf) for static loads; this pressure may be increased for seismic and wind loads. The note for seismic and wind increase does not include the increase factor. The upper 12 inches of subgrade soil should be pre-saturated to at least 5 percent above optimum moisture content. The pre-saturated pad should not be allowed to dry out to less than 5 percent above optimum moisture content prior to placement of concrete or moisture break.

BSA recommended that drilled piers be designed using a skin friction value of 450 psf; skin friction derived from the upper foot of soil below adjacent grade should be neglected.

Proposed new residences may be constructed on cut and/or fill slopes. BSA estimated that on-site soil and bedrock materials used as fill will settle during and after mass grading. The BSA report indicates fills with a thickness between 5 and 15 feet may experience total settlement of between 1/4 and 1 inch, respectively. BSA estimates that about 70 percent of the estimated total settlement should occur during mass grading; therefore, maximum post-grading settlement will be less than 1 inch.

The BSA report also indicates that up to about 15 feet of fill is planned at locations underlain by colluvium and/or alluvium that extends to depths of about 25 feet below the ground surface. BSA describes the colluvial and alluvial soil as stiff to very stiff, silty to sandy clay and clayey sand. BSA estimates the settlement of these deposits will be less than 1 inch.

Because fill and bedrock at the site will have different expansion and settlement potentials, BSA indicates that structures and foundations constructed across the transition line between cut and fill could experience significant differential expansion and/or settlement. To mitigate the potential for differential movement at cut/fill transition lots, BSA recommended over-excavating the cut portion of the cut/fill transition lots to a depth of about 3 feet below rough pad grade and backfilling the over-excavated areas with engineered fill.

Haley & Aldrich generally concurs with BSA's recommendations; however, there are several items requiring additional clarification or analyses, as noted by the following technical comments:

Total and Differential Settlement of Fill

The settlement behavior of the new fill should be monitored to confirm that total and differential settlements are within tolerable limits for the new buildings and site improvements. In addition, Haley & Aldrich suggests estimating the seismic compression of new fills at the site. Seismic compression can result in sudden and abrupt ground settlement that can damage new structures and site improvements.

BSA provided estimated total and differential amounts for settlement and swell related to compacted fill and colluvium and alluvium; however, the sum of the total and differential settlements was not discussed. Haley & Aldrich suggests BSA provide further explanation for the sum of settlement for cases where compacted fill is placed over colluvium and alluvium.

Split-Level Residences

We understand that BSA performed settlement evaluations for the upper and lower levels of the proposed residences and estimate the differential settlement will be about 1/2 inch. Structural strengthening or stiffening may be required if the differential settlement between the two levels is too large. If below-grade walls are present, BSA should indicate whether a lateral seismic increment and surcharge load should be included in the design. In addition, the elevations for back-drains behind below-grade walls and/or waterproofing should be evaluated to mitigate moisture intrusion into the adjacent lower level.

Mat and Drilled Pier Foundations

BSA's mat foundation design recommendations did not provide a minimum depth of mat embedment. In Haley & Aldrich's opinion, the minimum embedment depth and a requirement for the site grades to slope away from the proposed buildings will help to reduce the potential for surface water to enter beneath the new buildings, causing surface water and/or moisture intrusion problems. Also, as previously discussed for split-level residences, BSA should indicate whether the proposed mat can achieve bearing support on the soil immediately adjacent to any below-grade walls.

For the drilled pier and grade beam foundation system, Haley & Aldrich suggests that BSA provide recommendations for the floor slab if a wood-framed flooring system is not used. Typical floor slab systems consist of either structured floors that gain support on the pier and grade beam foundation or floating slab-on-grade floors that bear directly on the soil subgrade. Additional slab reinforcement may be required if slabs are placed adjacent to below-grade walls.

Site Grading and Fill Placement

The BSA report indicates that on-site soil/rock are suitable for re-use as engineered fill provided it does not contain rock fragments greater than 12 inches and is free of deleterious material. Imported fill should have a plasticity index less than 12 and be approved by the soil engineer. Fill should be placed in 6- to 8-inch-thick lifts, moisture conditioned to at least 3 percent above optimum moisture content, and compacted to at least 90 percent relative compaction. Strippings can be blended with clean on-site soils at a ratio of 10 loads of clean soil to one load of strippings to create a soil mixture suitable for engineered fill.

BSA recommended that fill placed on a slope with inclinations steeper than 7:1 should be benched into firm materials as determined in the field by the geotechnical engineer. Fill slopes should be constructed at gradients no steeper than 2:1. Fill slopes should be over built and cut back to expose a firm compacted surface. Fill slopes should be constructed with a 6-foot-deep (minimum) keyway with a width equal to 1/2 the slope height or 20 feet, whichever is greater, and constructed with proper sub-drainage.

In general, Haley & Aldrich concurs with the BSA recommendations, with the exceptions that Haley & Aldrich suggests: 1) limiting the maximum size of rocks or lumps in fill to no greater than 6 inches in greatest dimension so that loose soil adjacent to large rocks and lumps can be adequately compacted; and 2) to reduce potential settlement, strippings should not be blended into the soil and used as engineered fill.

Seismic Design Criteria

BSA concluded residential structures be designed in accordance with the 2013 California Building Code (CBC). BSA recommended using the seismic design criteria for Site Class D. Haley & Aldrich concurs with BSA's seismic design parameters.

There is a discrepancy with the closest distance to Rodgers Creek fault. Haley & Aldrich estimates the closest distance to Rodgers Creek fault is about 8.5 km. BSA should re-evaluate the closest distance to known seismic sources and modify the design criteria, if appropriate.

Recreational Trail

The recreational trail planned along the southern site boundary will cross two drainage/erosional features. As shown on Figures 2 and 3, pedestrian bridges and culverts may be needed. A realignment of the trail may be needed in the southwest corner of the site to avoid the steep slopes near the borrow pit area. A proposed trail alignment is shown on Figures 2 and 3.

Surface and Subsurface Drainage

The BSA report states that surface drainage benches on cut slopes should be spaced no more than 25 feet vertically on the slope. The benches should be a minimum of 8 feet wide and include a concrete-lined V ditch to intercept surface water runoff. BSA did not address surface drainage to collect and/or redirect surface water away from the building foundations. BSA did not address surface drainage of fill slopes.

Haley & Aldrich recommends roof downspouts should be connected to tight-lines consisting of rigid, PVC pipes that will convey water to suitable discharge areas. Concrete-lined V ditches should also be placed at strategic locations to protect slopes. For example, concrete-lined V-ditches should be considered above newly graded fill slopes and at a vertical spacing of no more than 30 feet on constructed engineered fill slopes. Storm water should not be allowed to pond or flow in concentrated streams or channels on the site. The homeowner's association or similar entity should be responsible for inspecting and maintaining the drains for the project.

BSA indicated that subsurface drains should be constructed on the uphill side of all keyways and proposed fill, along swales and gullies to receive fill, at all spring and seepage areas, at the toes of major cut slopes, at geologic contacts known to transmit water, and other areas where seepage is observed during and after grading or as determined in the field by the geotechnical engineer. BSA also recommended retaining walls be constructed with backdrains.

Haley & Aldrich believes the design of surface and subsurface systems are important to the success of the proposed project. Of particular concern is the high susceptibility to erosion of site soils and the possible presence of weak Franciscan melange adjacent to the sandstone bedrock at or near the site perimeter (including neighboring properties). Therefore, Haley & Aldrich believes that BSA (or other geologic consultant with experience in surface and groundwater controls) should review the surface and subsurface drainage drawings for the purpose of reducing the potential for adverse impacts, such as surface erosion and shallow landslides, on and adjacent to the site. Common design issues that may require technical input from BSA include: 1) the location of surface and subsurface drainage alignments, especially within filled slopes; 2) selection of water discharge locations; 3) separation of surface and subsurface water collection pipes; 4) location of pipe cleanouts; and 5) recommendations for controlling groundwater flow through trench backfill.

Retaining Walls

BSA recommended that all active condition retaining walls be designed to resist an active pressure corresponding to an equivalent fluid weight of 50 pounds per cubic feet (pcf) and 65 pcf for level and sloped backfill, respectively; and restrained walls be designed to resist an at-rest pressure corresponding to an equivalent fluid weight of 75 pcf. BSA also recommended adequate backdrains be constructed behind retaining walls. BSA recommended retaining footings be designed for an allowable bearing pressure of 2,500 psf for static loads; this pressure may be increased by 1/3 for seismic and wind loads. BSA recommends retaining walls be designed using appropriate surcharge loads.

Due to the relatively close proximity of the Rodgers Creek fault to the site and the potential for strong ground shaking, Haley & Aldrich suggests that BSA consider applying a uniform seismic increment to the design of new retaining walls at the site. Also, if retaining walls are constructed adjacent to roadways and/or buildings, appropriate surcharge loads from the adjacent improvements should also be incorporated in the wall design, as well as special foundation design criteria, if wall footings are placed on sloping terrain.

CONCLUSIONS

Haley & Aldrich concludes from a geologic and geotechnical engineering perspective that the proposed project is feasible, but potentially constrained by: 1) strong ground shaking; 2) slope instabilities associated with existing landslides, potential debris flows, and new and existing cuts and fills; and 3) localized settlement of existing or proposed compacted fill. Based on the review of prior geotechnical/geological studies presented for this project, Haley & Aldrich concludes the consultants have performed an adequate geotechnical/geological characterization of the site conditions and provided suitable geotechnical/geological recommendations for many of the site issues. However, Haley & Aldrich believes there are some issues that need further evaluation.

Based on the geological/geotechnical third party review, Haley & Aldrich has the following comments which been divided between those that directly impact the CEQA review and others which are not CEQA related, but are relevant to the geological/geotechnical design of the project. Haley & Aldrich suggests that the comments be addressed or commented upon by the project applicant. The following comments are relevant to the CEQA review.

CEQA Review Comments

Haley & Aldrich CEQA Related Comment No. 1
Page 3: BSA reports that the fill along the downslope, southern side of Windsor Drive is assumed to have been engineered along with the roadway. During the site reconnaissance, C2 observed that sub-parallel cracks in the pavement are present and are likely the result of fill creep and/or settlement. Haley & Aldrich suggests that the assumptions and design recommendation for this area be re-evaluated.

Haley & Aldrich CEQA Related Comment No. 2
Page 5: BSA should clarify the closest distance to the nearest fault (Rodgers Creek fault) and modify seismic design parameters, if appropriate.

Haley & Aldrich CEQA Related Comment No. 3
Page 6: As shown on Plates 2 and 3, BSA recommends that a below-grade MSE wall should be constructed during the repair of landslide A. However, the addition of the below-grade MSE wall is not included in the table on page 6 or on Plates 5 and 6. In addition, recommendations for the below-grade wall are not discussed. These discrepancies should be corrected.

Haley & Aldrich CEQA Related Comment No. 4
In the third paragraph on page 8, the largest rock fragment is 12 inches, but is recommended to be 6 inches on the following page.

Haley & Aldrich CEQA Related Comment No. 5
Page 13: The "Structural Mat/Slab Foundations" table should be revised to include the seismic and wind load increase factor.

Haley & Aldrich CEQA Related Comment No. 6

Appendix A: The test pits from the 2002 investigation are shown on the site plans, but not included in the Test Pit Logs. The depths of geologic contacts and the total depths of the test pit logs appear to vary in unit measurements from inches to feet, but are all designated as feet. The logs where depths to contacts and total depths are measured in inches should be revised to be in feet.

Haley & Aldrich CEQA Related Comment No. 7
Haley & Aldrich suggests that the locations of "debris flows" that were previously described by BSA be included on the Geologic Maps. Haley & Aldrich understands that the debris flow areas are located in open space areas; however, further evaluation and explanation should be provided regarding the expected run-out lengths and impact loads of debris flows with respect to the proposed development and Kelly Creek. If needed, a concrete headwall, berm, or other measure to divert flows from Kelly Creek or development areas and/or a proposed catchment area should be shown on the plans. Debris flow diversion structures and catchment areas should be evaluated for adequacy for resisting the impact forces and if the catchment area is large enough to contain the potential debris flow material without adversely impacting the downslope properties.

Haley & Aldrich CEQA Related Comment No. 8
Haley & Aldrich recommends that BSA evaluate the seismic stability of the partially repaired Landslide A, including the unrepaired upper portion of landslide A to evaluate the risk of landslide impacts on the proposed development areas downslope.

Haley & Aldrich CEQA Related Comment No. 9
Haley & Aldrich suggests that BSA address the sum of total and differential settlement/swell amounts in areas anticipated to have new fill over potentially compressible soils.

Haley & Aldrich CEQA Related Comment No. 10
Haley & Aldrich suggests that BSA provide recommendations for mat foundation embedment, or alternate mitigation to reduce the potential for surface water and moisture intrusion beneath the new structures.

Haley & Aldrich CEQA Related Comment No. 11
Plate 5: A subdrain is not shown in the keyway of section A-A'. BSA should clarify if these cross-sections are applicable to both options with regards to grading elevations and proposed lot numbers and streets. In addition incorrect lot numbers are shown on sections D-D' and E-E'.

Haley & Aldrich CEQA Related Comment No. 12
Plate 8: A berm is shown on the top of the fill slope detail. The grading plan should be checked to confirm that the ground surface upslope of the berms are sloped to drain to reduce water infiltration into the slope.

Haley & Aldrich CEQA Related Comment No. 13
If the small stock pond embankment were to fail while retaining water, the water would be released downslope towards the open space area and Kelly Creek. Haley & Aldrich suggests that the volume of potential flow be used to evaluate the impact on the downslope properties and creek capacity. If the evaluation indicates that a potential release of pond water does not adversely impact the planned development, then no further mitigation is necessary. However, if the sudden release of pond water is found to adversely impact the planned development, then further mitigation is necessary.

Haley & Aldrich CEQA Related Comment No. 14
Erosion is occurring along north-trending incised channels that discharge into Kelly Creek. The drainages extend from two north-tending swales in the central southern portion of the site, the western of which has since been truncated by the embankment for the stock pond. The head of the eastern of the two incised channels has eroded more than 80 feet towards the southwest since the time of BSA's survey. This channel extends into the area of several of the proposed home-sites. The current location of the western incised channel (downslope of the pond embankment) appears to be in about the same state as when BSA performed their survey. During our review of aerial photographs, this western channel previously extended further south to limits near the embankment; portions of the channel appear to have been filled by man-made or natural depositional mechanisms since 1973. Undocumented fill may still exist in this area. Haley & Aldrich recommends the area of on-going erosion along the eastern incised channel be evaluated for potential impacts on the proposed home-sites in the area. Also, Haley & Aldrich recommends that the area immediately north of the existing stock pond embankment be evaluated for the presence of undocumented fill within the former western incised channel. If undocumented fill is present, BSA should indicate whether this material will adversely impact the performance of the embankment.

General Geological/Geotechnical Review Comments

The following comments are relevant to the general geological/geotechnical design of the project.

Haley & Aldrich General Comment No.1
In the first paragraph of the "Field Investigation" section, it describes the maximum boring depth ranging from 24 to 502 feet. The depth range should be revised to reflect the actual depths drilled.

Haley & Aldrich General Comment No.2
In the first paragraph on page 8, the "Typical Fill Slope Details" are presented on Plate 8, not on Plate 5.

Haley & Aldrich General Comment No.3
Page 9: The subdrain details are provided on Plate 9.

Haley & Aldrich General Comment No.4
Page 11: The "Typical Cut/Fill Transition Lot Over-excavation Details" are provided on Plate 10.

Haley & Aldrich General Comment No.5
Page 11: The third paragraph states that the upper 12 feet will be reworked during normal stripping and scarification. This statement should be revised.

Haley & Aldrich General Comment No.6
Page 16: ASTM D1557-00 is referenced. Haley & Aldrich suggests the most recent ASTM procedure be referenced.

Haley & Aldrich General Comment No.7
Page 16: Haley & Aldrich recommends that the slopes of trench excavations be sloped, benched or shored in accordance with OSHA standards.

Haley & Aldrich General Comment No.8
Haley & Aldrich understands that a wood flooring system is planned for use with pier and grade beam foundations. If changed, Haley & Aldrich suggests that BSA provide recommendations for where the ground floor slab will consist of a structured slab or slab-on-grade when used in conjunction with a pier and grade beam foundation.

Haley & Aldrich General Comment No.9
The recreational trail planned along the southern site boundary will cross two drainage courses. As shown on Figures 2 and 3, pedestrian bridges and culverts may be needed. A realignment of the trail may be needed in the southwest corner of the site to avoid the steep slopes near the borrow pit area. Design recommendations for these improvements should be provided by BSA.

Haley & Aldrich General Comment No.10
We understand that BSA performed settlement evaluations for the upper and lower levels of the proposed split-level residences. Structural strengthening or stiffening may be required if the differential settlement between the two levels is too large. If below-grade walls are present, BSA should indicate whether a lateral seismic increment and surcharge load should be included in the design. In addition, the elevations for back-drains behind below-grade walls and/or waterproofing should be evaluated to mitigate moisture intrusion into the adjacent lower level.

In conclusion, Haley & Aldrich recommends the project applicant and/or BSA provide a response to the comments presented above. The City of Petaluma should be given an opportunity to review the responses, and comment on whether any outstanding issues still remain. Haley & Aldrich and C2 appreciate the opportunity to assist you with the evaluation of geological and geotechnical issues for this project. If you have any questions or require additional information, please call.

Sincerely yours,
HALEY & ALDRICH, INC.

REGISTERED PROFESSIONAL ENGINEER
DEAN H. IWASA
GE 2285
Exp 3/31/16
GEOTECHNICAL
STATE OF CALIFORNIA

Dean H. Iwasa
Geotechnical Engineer

Attachment: References
Figures 1 through 3

cc: Mr. Chris Hundemer, C2Earth

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REFERENCES

Berlogar Geotechnical Consultants, 2003, Design-level geotechnical investigation, UOP property, D Street, Petaluma, California: consultant's report prepared for Davidon Homes, 22 July.

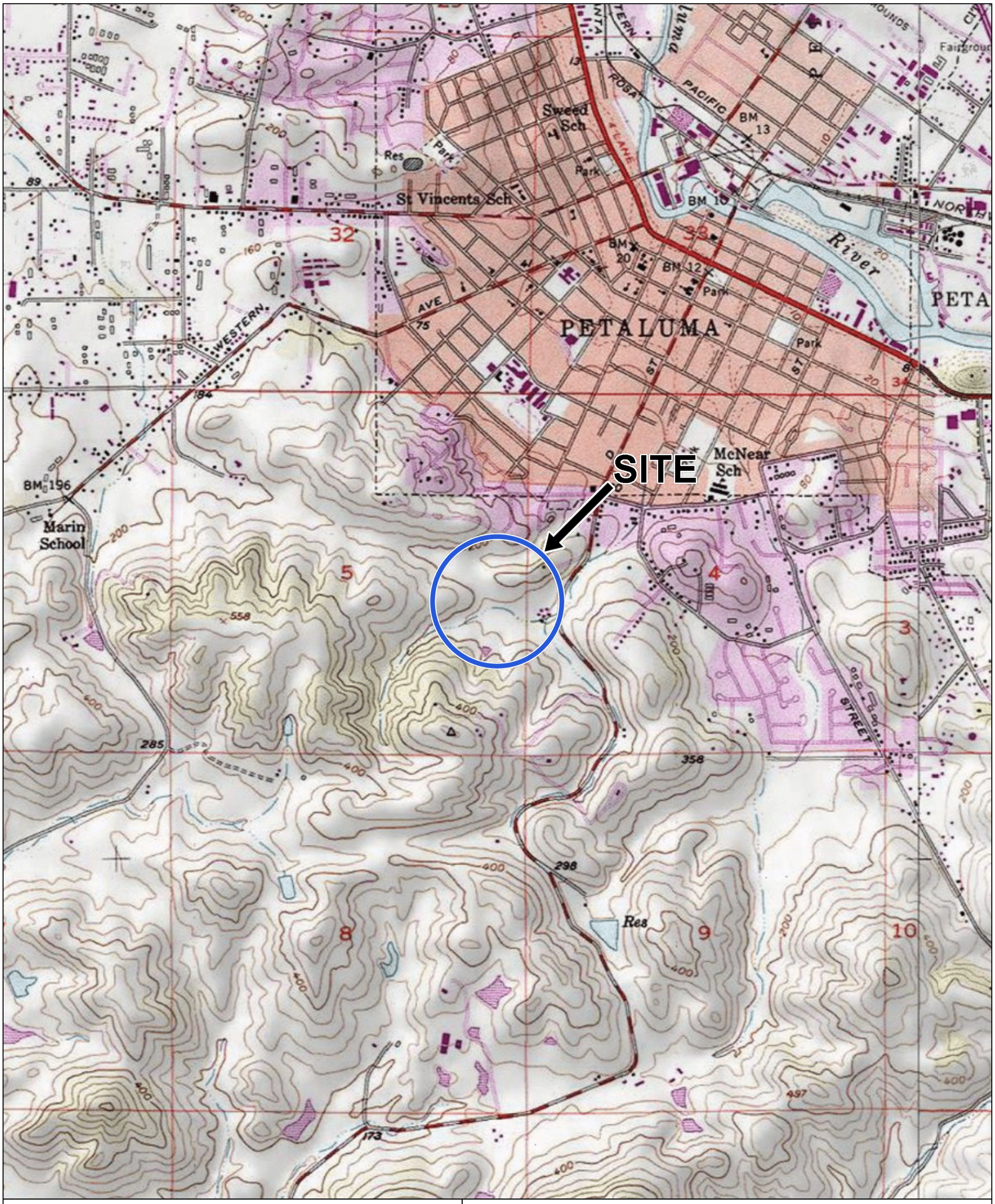
Berlogar Geotechnical Consultants, 2004, Response to geotechnical peer review comments plus supplemental recommendations, UOP property, D Street, Petaluma, California, 16 December.

Berlogar Stevens & Associates, 2014, Design-level geotechnical investigation, Option A – 66 Lots, Option B – 63 Lots, Scott Ranch, D Street and Windsor Drive, Petaluma, California: consultant's report prepared for Davidon Homes, 28 April.

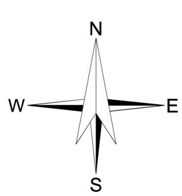
Blake, M.C., Jr., Graymar, R. W., and Jones, D.L., 2000, Geologic Map and Map Database of Parts of Marin, San Francisco, Alameda, Contra Costa, and Sonoma Counties, California: U.S. Geological Survey Miscellaneous Field Studies Map MF-2337, Version 1.0.

California Building Standards Commission, 2013. California Code of Regulations. Title 24, Volume 2.

Treadwell & Rollo, Inc., 2004, Third party geotechnical/geological review, Davidon Homes EIR, Petaluma, California, 23 November.



SITE COORDINATES: 38°13'7"N,122°38'52"W



U.S.G.S. QUADRANGLE: PETALUMA, CA

HALEY & ALDRICH

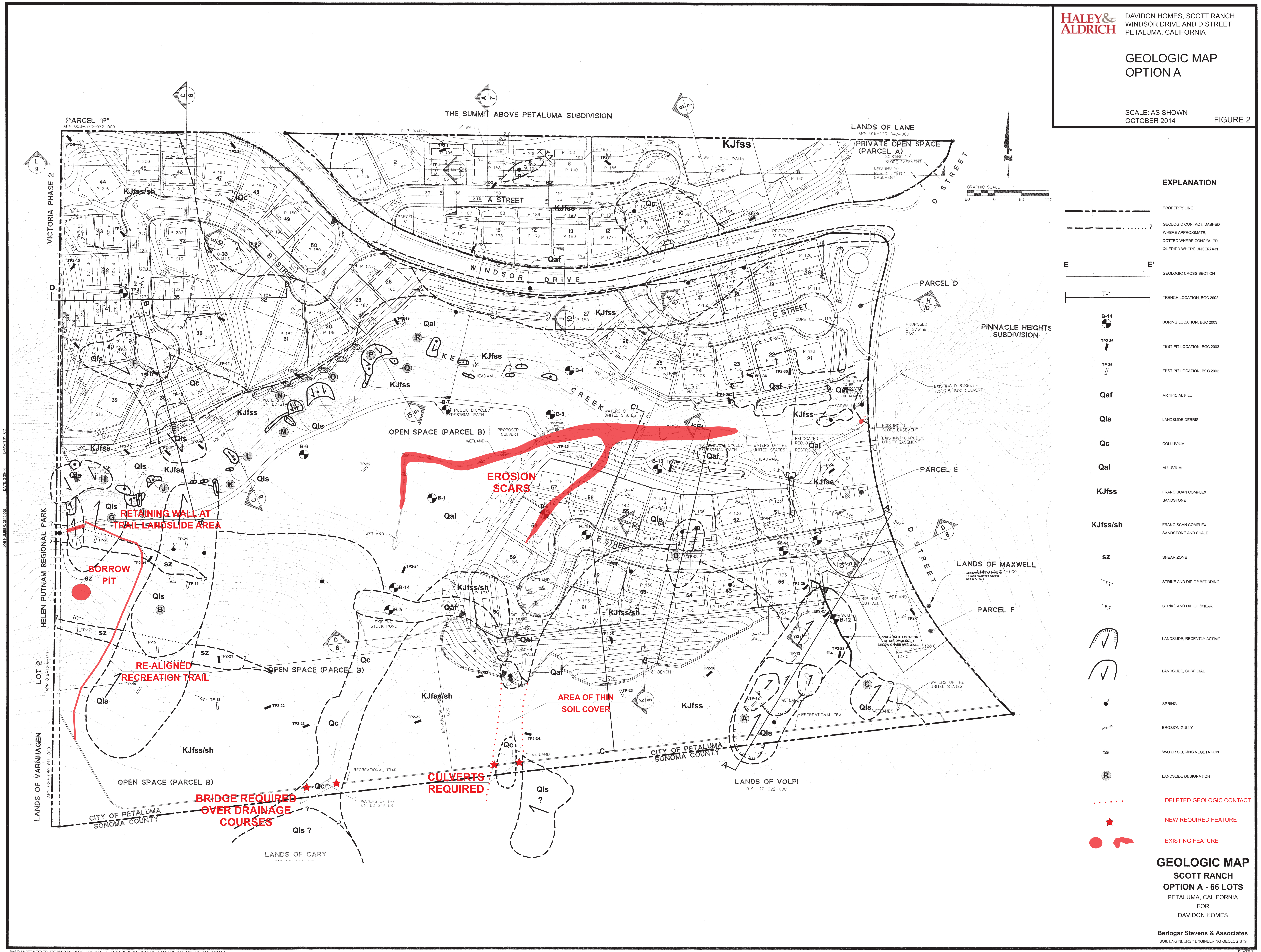
DAVIDON HOMES, SCOTT RANCH
WINDSOR DRIVE AND D STREET
PETALUMA, CALIFORNIA

PROJECT LOCUS

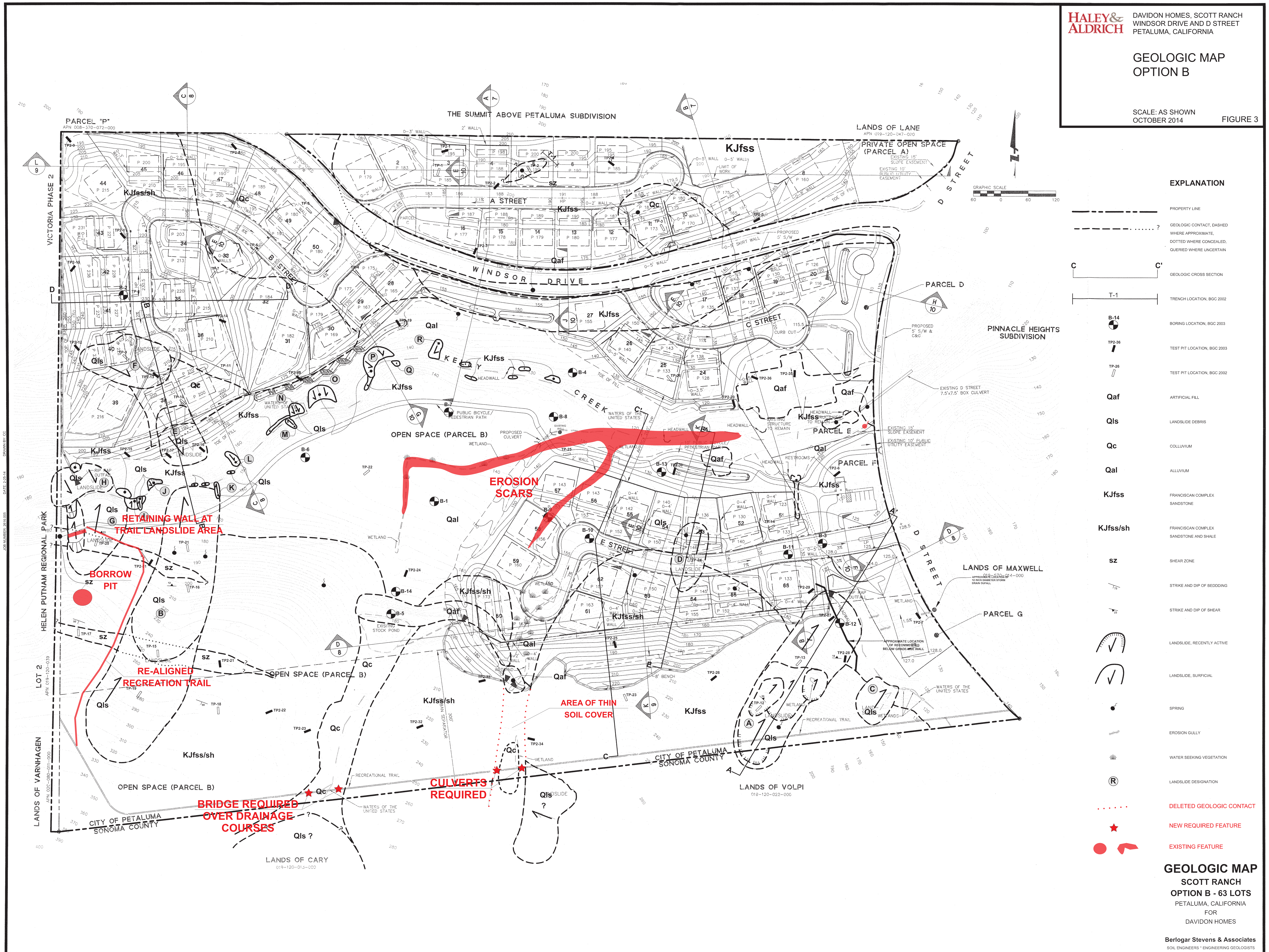
SCALE: 1:24,000
OCTOBER 2014

FIGURE 1

41431-000 1.PDF



HALEY & ALDRICH
DAVIDON HOMES, SCOTT RANCH
WINDSOR DRIVE AND D STREET
PETALUMA, CALIFORNIA
GEOLOGIC MAP
OPTION A
SCALE: AS SHOWN
OCTOBER 2014
FIGURE 2
EXPLANATION
PROPERTY LINE
GEOLOGIC CONTACT, DASHED WHERE APPROXIMATE, DOTTED WHERE CONCEALED, QUERIED WHERE UNCERTAIN
GEOLOGIC CROSS SECTION
TRENCH LOCATION, BGC 2002
BORING LOCATION, BGC 2003
TEST PIT LOCATION, BGC 2003
TEST PIT LOCATION, BGC 2002
Qaf ARTIFICIAL FILL
Qls LANDSLIDE DEBRIS
Qc COLLUVIUM
Qal ALLUVIUM
KJfss FRANCISCAN COMPLEX SANDSTONE
KJfss/sh FRANCISCAN COMPLEX SANDSTONE AND SHALE
sz SHEAR ZONE
STRIKE AND DIP OF BEDDING
STRIKE AND DIP OF SHEAR
LANDSLIDE, RECENTLY ACTIVE
LANDSLIDE, SURFICIAL
SPRING
EROSION GULLY
WATER SEEKING VEGETATION
LANDSLIDE DESIGNATION
DELETED GEOLOGIC CONTACT
NEW REQUIRED FEATURE
EXISTING FEATURE
GEOLOGIC MAP
SCOTT RANCH
OPTION A - 66 LOTS
PETALUMA, CALIFORNIA
FOR
DAVIDON HOMES
Berlogar Stevens & Associates
SOIL ENGINEERS * ENGINEERING GEOLOGISTS
PLATE 2
THE SUMMIT ABOVE PETALUMA SUBDIVISION
PARCEL "P"
VICTORIA PHASE 2
HELEN PUTNAM REGIONAL PARK
LOT 2
LANDS OF VARNHAGEN
LANDS OF LANE
PRIVATE OPEN SPACE (PARCEL A)
PARCEL D
PARCEL E
PARCEL F
PINNACLE HEIGHTS SUBDIVISION
LANDS OF MAXWELL
LANDS OF VOLPI
LANDS OF CARY
CITY OF PETALUMA
SONOMA COUNTY
A STREET
B STREET
C STREET
D STREET
E STREET
WINDSOR DRIVE
KELLY CREEK
OPEN SPACE (PARCEL B)
EROSION SCARS
RETAINING WALL AT TRAIL LANDSLIDE AREA
BORROW PIT
RE-ALIGNED RECREATION TRAIL
AREA OF THIN SOIL COVER
CULVERTS REQUIRED
BRIDGE REQUIRED OVER DRAINAGE COURSES
BASE: SHEET 5 TITLED, "REVISED PROJECT - OPTION A - 66 LOTS PROPOSED GRADING PLAN", PREPARED BY BKF, DATED 12-11-13


HALEY & ALDRICH
DAVIDON HOMES, SCOTT RANCH
WINDSOR DRIVE AND D STREET
PETALUMA, CALIFORNIA
GEOLOGIC MAP
OPTION B
SCALE: AS SHOWN
OCTOBER 2014
FIGURE 3
EXPLANATION
PROPERTY LINE
GEOLOGIC CONTACT, DASHED WHERE APPROXIMATE, DOTTED WHERE CONCEALED, QUERIED WHERE UNCERTAIN
GEOLOGIC CROSS SECTION
TRENCH LOCATION, BGC 2002
BORING LOCATION, BGC 2003
TEST PIT LOCATION, BGC 2003
TEST PIT LOCATION, BGC 2002
Qaf ARTIFICIAL FILL
Qls LANDSLIDE DEBRIS
Qc COLLUVIUM
Qal ALLUVIUM
KJfss FRANCISCAN COMPLEX SANDSTONE
KJfss/sh FRANCISCAN COMPLEX SANDSTONE AND SHALE
sz SHEAR ZONE
STRIKE AND DIP OF BEDDING
STRIKE AND DIP OF SHEAR
LANDSLIDE, RECENTLY ACTIVE
LANDSLIDE, SURFICIAL
SPRING
EROSION GULLY
WATER SEEKING VEGETATION
LANDSLIDE DESIGNATION
DELETED GEOLOGIC CONTACT
NEW REQUIRED FEATURE
EXISTING FEATURE
GRAPHIC SCALE
THE SUMMIT ABOVE PETALUMA SUBDIVISION
PARCEL "P"
VICTORIA PHASE 2
LANDS OF LANE
PRIVATE OPEN SPACE (PARCEL A)
D STREET
A STREET
B STREET
C STREET
E STREET
WINDSOR DRIVE
KELLY CREEK
PARCEL D
PARCEL E
PARCEL F
PARCEL G
PINNACLE HEIGHTS SUBDIVISION
LANDS OF MAXWELL
OPEN SPACE (PARCEL B)
HELEN PUTNAM REGIONAL PARK
LOT 2
LANDS OF VARNHAGEN
LANDS OF CARY
LANDS OF VOLPI
CITY OF PETALUMA
SONOMA COUNTY
EROSION SCARS
RETAINING WALL AT TRAIL LANDSLIDE AREA
BORROW PIT
RE-ALIGNED RECREATION TRAIL
AREA OF THIN SOIL COVER
CULVERTS REQUIRED
BRIDGE REQUIRED OVER DRAINAGE COURSES
GEOLOGIC MAP
SCOTT RANCH
OPTION B - 63 LOTS
PETALUMA, CALIFORNIA
FOR
DAVIDON HOMES
Berlogar Stevens & Associates
SOIL ENGINEERS * ENGINEERING GEOLOGISTS
PLATE 3

REVISED DESIGN LEVEL GEOTECHNICAL INVESTIGATION
OPTION A – 66 LOTS
OPTION B – 63 LOTS
SCOTT RANCH
D STREET AND WINDSOR DRIVE
PETALUMA, CALIFORNIA

FOR
DAVIDON HOMES
June 23, 2015

Job No. 2616.006

BERLOGAR STEVENS & ASSOCIATES

Via E-Mail and Mail

BERLOGAR
STEVENS &
ASSOCIATES

June 23, 2015
Job No. 2616.006

Mr. Jeff Thayer
Davidon Homes
1600 South Main Street, Suite 150
Walnut Creek, California 94596

Subject: Revised Design Level Geotechnical Investigation Report
Scott Ranch Option A – 66 Lots
Scott Ranch Option B – 63 Lots
D Street and Windsor Drive
Petaluma, California

Dear Mr. Thayer:

We are presenting our Revised Design-Level Geotechnical Investigation for Option A – 66 Lots and Option B – 63 Lots for the Scott Ranch residential development. The site is located at the intersection of D Street and Windsor Drive in Petaluma, California as shown on the Vicinity Map, Plate 1. This revised report incorporates responses to comments from Haley & Aldrich in their review of our April 28, 2014 Design Level Geotechnical Investigation report. The field investigation and laboratory testing utilized in this report were conducted in 2004 for a 93-lot plan under consideration at that time.

PURPOSE AND SCOPE OF SERVICES

The purpose of the investigation has been to characterize the engineering properties of soil and bedrock at the site and provide design-level geotechnical recommendations for site development. The scope of services for this project included:

1. Review of previous information covering the site and vicinity,
2. Review of stereo-paired aerial photographs covering the site and vicinity,
3. Site geologic reconnaissance and mapping,
4. Drilling and logging of 14 borings,
5. Excavating and logging 36 backhoe test pits,
6. Laboratory testing of selected representative samples collected during the field investigation,
7. Engineering and geologic analysis, and
8. Preparation of this report.

This report also addresses comments from two geotechnical peer reviewers. Treadwell & Rollo comments are contained in a letter dated November 23, 2004 on our 2004 geotechnical report. Haley & Aldrich provided comments to our geotechnical report dated April 28, 2014. This revised report includes relevant responses to the peer review comments that are within the body of this report. Test Pits TPA through TPD were excavated on June 11, 2015 along the south side of Windsor Drive to respond to CEQA comment 1 by H&A. The locations of these test pits are shown on Plates 2 and 3, and graphic test pits logs for TPA through TPD are shown on Plate 11. Laboratory test results from soil samples from TPA through TPD are contained in the graphic test pit logs. A brief summary of our responses to their comments are contained in Appendix D (including a discussion of the fill slope on the south side of Windsor Drive).

PROPOSED DEVELOPMENT

The site includes two parcels totaling about 58.5 acres that are separated by Windsor Drive. The site is bound by residential developments to the north and west, by D Street to the east, and open land to the south. We have received Option A – 66 lot plan and Option B – 63 lot plan from BKF via electronic file, that show the site being developed into 66 or 63 single-family residential lots separated by existing creek channels. The existing creek channels are to remain undeveloped open space. New streets providing access to the site are to branch off of Windsor Drive and D Street. An existing stock pond and berm located in a swale in the southern portion of the site are to remain.

In order to achieve design grades, cuts of up to about 25 feet and fills of up to about 15 feet are planned. The design grading will result in cut slopes up to about 60 feet tall and fill slopes up to about 30 feet high. Retaining walls up to 5 feet high are planned to achieve design grades. A developed trail will be constructed on the south side of Kelly Creek with a trailhead, bathroom facility, and a paved parking lot next to D Street. The trail extends westerly to the border of Helen Putnam Regional Park. There is an existing barn and 2 accessory structures located in the northeast corner of the site. Option A proposes to relocate the barn and demolish the accessory structures in order to construct Lots 21, 22 and 23. Option B will leave the barn and accessory structures in place.

FIELD INVESTIGATION

The field investigation was conducted between March 28 and April 3, 2003. The investigation included a site reconnaissance, geologic mapping of creek bank exposures, drilling and logging two rotary wash borings (B-1 and B-2) to depths ranging from 24 to 50 feet, drilling and logging 12 auger borings (B-3 through B-14) to depths ranging from 8 to 22 feet, and excavating and logging 36 backhoe test pits (TP2-1 through TP2-36) to depths of up to 14 feet. We also excavated and logged 26 test pits and a trench for our preliminary geotechnical investigation in 2002. Materials encountered in the borings and test pits were visually classified and logs were recorded. Bulk and relatively undisturbed samples of bedrock and soils were collected from the borings and test pits for laboratory testing.

Four additional test pits were excavated within the fill slope along the southern side of Windsor Drive on June 11, 2015. Test Pits TPA through TPD were excavated behind lots 117 to 120. These test pits were excavated to determine the quality of the fill placed during construction of

Windsor Drive as requested by Haley & Aldrich (see comment 1 in Appendix D). The graphic logs for these test pits and the laboratory test results are contained in Plate 11.

Where ground water was encountered, the borings were backfilled with neat cement grout in accordance with Sonoma County requirements. Borings that did not encounter ground water were backfilled with soil cuttings. Test pits were loosely backfilled with excavated materials at the completion of logging. The locations of borings and test pits are shown on the attached Geologic Maps, Plates 2 and 3 for Options A and B, respectively. Boring logs are presented in Appendix A (A-1 through A-20), and a Key to Boring Symbols and Rock Description is presented on pages A-21 and A-22. Test Pit Logs are also included in Appendix A (A-23 through A-31 from the 2004 investigation and A-32 through A-38 from the 2002 investigation).

FINDINGS

SURFACE CONDITIONS

Site elevations range from about 100 feet in the eastern portion of the site to about 380 feet near the southwest corner of the site. The site contains a relatively flat alluvial plain in the central portion of the site that is bordered by moderately steep bedrock slopes to the north and south. Kelly Creek crosses the site in an east-west direction and intersects an unnamed tributary that crosses the eastern portion of the site in a north-south direction near D Street. Two drainage gullies on the southern slope drain into Kelly Creek. Kelly Creek flows to the northeast and enters an existing box culvert beneath D Street.

Existing site improvements include the remains of a wood-framed house, a vacant mobile home, a barn and accessory structures located near D Street. An open, concrete-lined water well about 3 feet in diameter and about 15 feet deep, is located beneath the trees along the edge of the westernmost drainage gully on the south side of Kelly Creek. Water in the well was roughly at the ground surface. A stock pond and berm are located in a drainage swale south of Kelly Creek. An existing storm drain outlet is located on the southwest side of D Street, about 5 feet east of the property line.

REGIONAL GEOLOGY

The site is situated along the southwest margin of the Petaluma River valley. This valley is part of a series of small basins and ranges characteristic to the Coast Ranges geomorphic province of California. In this portion of the province, the oldest bedrock consists of sedimentary and meta-volcanic rocks of the Franciscan Complex which were deposited during the Jurassic and Cretaceous Periods of geologic time (about 65 to 208 million years before present). Small lenses of sheared and/or altered bodies of rock are inherent to the Franciscan Complex. Tertiary aged (10.6 to 65 million years before present) volcanic rocks are present in scattered patches throughout the region (Blake et al., 1974).

Bedrock in this region has been folded and faulted during the past several million years due to relative strike-slip and convergent motion between tectonic plates. Much of the deformation (shearing, faulting and folding) of the Franciscan Complex occurred during past convergent plate motions. Most convergent plate motion in the region ended millions of years ago. This deformation is believed by many researchers to be intrinsic to the Franciscan Complex and is separate from the active strike-slip fault motion in the region.

SUBSURFACE CONDITIONS

During the course of this investigation, we encountered artificial fill, landslide deposits, colluvium, alluvium, shear zone material, and bedrock units of the Franciscan Complex. A description of each material, excluding landslide deposits which are discussed separately, are listed below in order from youngest to oldest:

ARTIFICIAL FILL

Isolated areas of artificial fill at the site were encountered in three main areas: beneath and around existing buildings, the stock pond earthen berm, along the downslope (south) side of Windsor Drive, and beneath D Street in low areas. Fill beneath and around existing structures encountered in Test Pits TP2-35 and-36 was found to consist of dense sandy silt and gravel that extended to a depth of about 1 foot. The stock pond berm fill encountered in Boring B-5 was found to consist of stiff to very stiff silty clay that extended to a depth of about 12 feet. Areas of artificial fill are delineated by the symbol “Qaf” on the Geologic Map.

COLLUVIUM

Areas of soil accumulation referred to as colluvium are present in the lower portions of the site. Colluvium is material that is generated by the in-place weathering of underlying bedrock on a slope which migrates downslope under the influence of gravity. Colluvium mantles all slopes to some degree and forms particularly thick deposits at the toes of slopes and in swales. At the site, colluvium was found to be brown to light red-brown, stiff to very stiff silty clay with minor amounts of gravel. Laboratory testing suggests that the colluvium is moderately expansive. Areas of colluvium thicker than a few feet are delineated on the Geologic Map by the symbol “Qc.”

ALLUVIUM

Alluvium is material that has been transported and deposited by flowing water. Alluvium was found to consist of orange-brown to yellow-brown sandy clays and clayey sands with various amounts of gravel that are stiff to very stiff and medium dense to dense. Alluvium is generally found in relatively flat lying areas bordering drainage courses and at the downslope end of swales as shown on the Geologic Map by the symbol “Qal.” Based on the information provided by the borings and creek exposures, the alluvium unit reaches a maximum thickness of about 25 feet at a point about half way between the Kelly Creek channel and the base of the hills to the south. Laboratory testing suggests that the alluvium on site is moderately expansive.

SHEAR ZONE MATERIAL

Three shear zones were encountered during our 2002 and 2004 investigations: one north of Windsor Drive, and two located in the southwest portion of the site. The shear zones at the site are not related to the active regional strike-slip system of faulting. The shear zones at the site are interpreted as deformation concentrated within the relatively weak shale. The deformation likely occurred as flexural slip during regional folding resulting from the past convergent tectonic regime.

Our previous investigation described material within the shear zone as containing serpentine minerals. Serpentine minerals are often found in ultramafic rocks. Based on the test pits excavated within shear zones during this investigation (TP2-2, TP2-21, and TP2-31), ultramafic rocks were not encountered. We also re-excavated test pits from our previous investigation (TP-17 and TP-20) and reclassified the shear zone materials. It was found that the shear zone materials are composed of sheared clayey shale, and no serpentine minerals or ultramafic rocks were encountered. The gray-green alteration colors previously described are interpreted to be the result of a localized chemical reduction of the clayey material. Based on these findings, we conclude that the potential for significant volumes of serpentine-bearing ultramafic rocks being present at the site is low.

FRANCISCAN COMPLEX BEDROCK

Bedrock at the site consists of sandstone and shale of the Franciscan Complex. The sandstone was found to be moderately strong to strong and highly fractured with scattered zones of very strongly-cemented beds. The shale is weak to moderately strong, thinly laminated, and crushed to sheared. Where sheared, the shale was weathered to clay and displayed a faint residual bedrock structure. Bedding was found in general to strike northwest and dip southwest at inclinations between about 33 and 73 degrees.

LANDSLIDES

A total of 18 landslides were mapped within the site, and are shown and designated as Landslides A through R on the Geologic Map. Landslides A, B, C, D, and G are located on the flanks of the hillsides in the southern portion of the site. Landslides E, F, and H are located on the flank of the large bedrock knob in the northwest portion of the site. The remaining landslides (Landslides I through R) are located along the banks of Kelly Creek and are the result of typical creek bank oversteepening. Landslides encountered at the site are relatively shallow with depths up to about 15 feet and are believed to involve soils and the upper 2 to 3 feet of highly weathered bedrock.

FAULTING

The site is not located within a State of California designated earthquake fault zone for active faults (Davis, 2000; Hart and Bryant, 1982). The State of California considers a fault active if it has demonstrated activity within the Holocene Epoch of geologic time, within the past 11,700 years. Potentially active faults are faults with Quaternary displacement (within the past 1.6 million years) but no evidence for Holocene activity. The U.S. Geological Survey's Quaternary Fault and Fold Database shows several Quaternary aged faults in the region. The majority of the Quaternary faults shown in the database have slip rates less than 0.2 mm/yr and are considered secondary systems in the greater Bay Area fault hazard scenarios. Quaternary faults near the site include, but are not limited to, the Burdell Mountain fault located about 2 miles to the south, the

Tolay fault located about 2½ miles to the east, the Lakeview fault located about 3¾ miles to the northeast, the Bloomfield and Americano Creek faults fault located about 6½ miles to the northwest, and the Bennett Valley fault located about 14 miles to the northeast. These Quaternary faults disrupt the bedrock but rarely offset younger Late Pleistocene or Holocene sediments or soils; therefore they are believed to be either inactive or to play a small role in the regional hazard models.

The closest known active fault is the Rodgers Creek fault, which is located about 6½ miles northeast of the site. The table below lists the eight known active faults that are believed to present the highest potential levels of ground shaking at the site, their distances from the site, and their potential maximum moment-magnitude earthquakes. Faults listed below are those shown in the 2008 Fault Source Map contained in the 2014 Fault Parameters by the U.S. Geological Survey's Earthquake Hazards Program and are arranged in order of their decreasing potential level of ground shaking at the site.

SIGNIFICANT POTENTIAL EARTHQUAKE FAULT SOURCES

Fault Source[1]	Approx. Distance to Fault Trace (mi)[2]	Compass Direction to Fault	Maximum E.Q. mag. (Mw)[3]
Rodgers Creek	6½	NE	7.0
San Andreas, 1906 Rupture	13½	SW	7.9
Hayward, Total Length	18	SE	7.1
San Gregorio	23	S	7.3
Point Reyes	22	SW	6.8
West Napa	17	E	6.5
Maacama, south	20	N	6.9
Collayomi	37	N	6.5

1. 1. 2008 Fault sources included in the 2014 Fault Parameters provided by the U.S. Geological Survey's Earthquake Hazards Program on-line web tools.
2. 2. Fault locations and distances to the site were determined from the KML files provided from the Quaternary Fault and fold Database.
3. 3. Maximum earthquake moment magnitude calculated by Peterson et al. (1998).

GROUNDWATER

Groundwater was encountered in Test Pit TP2-7 at a depth of about 2 feet and is likely the result of the storm drain outfall next to D Street. Groundwater was encountered in Borings B-1, B-5, B-7, B-8, B-12, and B-13 at depths of about 21, 17, 7, 14½, 17½, and 9½ feet, respectively. Marshy ground and groundwater seepage have been observed in various places across the site, mainly following periods of higher rainfall. Areas of perched groundwater are expected in the lower portions of the site. Groundwater levels are expected to undergo significant fluctuations based on seasonal rainfall and time of year.

CONCLUSIONS AND RECOMMENDATIONS

GENERAL

From a geotechnical standpoint, the proposed residential development can generally be constructed as planned, provided the conclusions and recommendations contained within this report are incorporated into the project design and construction. The primary geotechnical issues for site development are landslide remediation, treatment of existing fill, fill slope construction, stability of proposed cut slopes, and the potential for expansion and settlement of on-site earth materials.

LANDSLIDE REMEDIATION

The recommended remedial treatment of landslide hazards is dependent on many factors such as the size of the landslide, the landslide's spatial relationship to proposed improvements, and the individual characteristics of each landslide. In general, the preferred remedial measure from a geotechnical standpoint is complete removal of landslide debris located within the development area. A number of factors can make complete removal of landslide debris impractical, such as property line limitations or the presence of trees. Provided the risks associated with movement of part of a given landslide located outside the development are acceptable, the potential adverse impacts to the planned development can be minimized by implementing remedial measures such as construction of engineered fill, below-grade MSE walls, and catchment areas.

Landslides A, D, and E will require remedial treatment for the currently planned development with either 63 or 66 lots. Landslide F will be removed with the design cut. Landslides B, C, G, and H are located outside of the planned development area and require no remediation. Similarly, landslides located along Kelly Creek (I through R) do not impact the development and require no remediation. We recommend that landslides be treated as summarized in the following table:

RECOMMENDED LANDSLIDE REMEDIATION SUMMARY			
Landslide Designation	**Est. Ave. Thickness (feet)**	**Relationship of Landslide to Proposed Development**	**Recommended Remedial Measures**
A	12	Within and upslope of limit of grading	• Remove portion within development and replace with engineered fill with proper subdrainage. (See Plates 6 and 7, Remedial Grading Plans) • Construct a 40 feet wide (minimum) keyway with proper subdrainage.
B	9	In Open Space	• None Required
C	6	Outside limits of grading	• None required
D	4	Within limits of grading	• Remove and replace all landslide debris with engineered fill and proper subdrainage.
E	6	Within limits of grading	• Remove and replace all landslide debris with engineered fill and proper subdrainage. • Construct a 40 feet wide (minimum) keyway.
F	7	Within limits of grading	• None required (removed by design grading).
G	10	Outside limit of grading	• None required.
H	10	Outside limit of grading	• None required.
I through R	3 to 5	Along creek bank in open space	• None required.

DEBRIS FLOW/SEDIMENTATION POTENTIAL

The potential for debris flow hazards was determined to be low across most of the site; however, potential debris flow hazards within the two southern drainage swales were discussed in our prior report. Our investigation included site reconnaissance's by engineering geologists, review of topographic maps, and a review of historical stereo pair aerial photographs.

The drainage swale above the stock pond leads out about 600 feet east of this project. The steepest gradient is about 3.2H:1V. The drainage swale above E Street heads out about 1000 feet east of the project. The gradient varies from 3H:1V at the head to 4H:1V for the remaining portion east of the project. In the event debris flows were to occur in either of these drainage swales, they would have very short runouts because of the relatively flat gradients in the drainage swales.

GRADED SLOPES

CUT SLOPES

All cut slopes should be inspected at the time of construction by an engineering geologist focusing on evidence of potential instability. Cut slopes should be constructed at gradients no steeper than 2H:1V. Where cut slopes over 30 feet in height are planned, intermediate surface benches should be spaced no more than 25 feet vertically on the slope. The benches should be a minimum of 8 feet wide and include a concrete lined V-ditch to intercept surface water runoff.

Based on bedding attitudes measured in test pits, areas of adverse bedrock structure were not encountered at the locations of proposed cut slopes. However, due to folding and shearing of the bedrock, localized areas of adverse bedrock structure or other zones of geologic weakness could be exposed during grading of cut slopes. If areas of adverse bedrock structure are encountered, we anticipate that the remedial measures for these slopes will involve overexcavation of the affected portion of slope and construction of a slope buttress with appropriate subdrainage. We should provide specific remedial design recommendations based on the conditions exposed in areas of concern identified during grading.

FILL SLOPES

The stability of proposed fill slopes is dependent on proper keyways, benching, subdrainage, fill compaction, and slope gradient. Fill slopes should be constructed at gradients no steeper than 2H:1V. Fill slopes should be overbuilt and cut back to expose firm compacted materials. Fill slopes should be constructed with a 6 feet deep (minimum) keyway with a width equal to ½ the slope height or 20 feet, whichever is greater, and provided with proper subdrainage. All keyway excavations should be mapped by an engineering geologist prior to backfilling. Typical Fill Slope Details are presented on Plate 8.

TREATMENT OF EXISTING FILL

From a geotechnical standpoint, the on-site existing fill material is considered suitable for re-use as engineered fill provided it is free of rock fragments greater than 6 inches in size and deleterious material. Inasmuch as the proposed development does not encroach onto these fill areas, based on our field observations, existing fill along D Street does not require additional treatment. All other fill located on site (with the exception of the stock pond berm) should be completely removed and reworked as engineered fill.

The existing stock pond is to remain as-is in open space. Remedial treatment of the stock pond is no longer recommended because the stock pond and the area downhill of the stock pond are in planned open space and no longer in close proximity to planned residential construction.

The existing fill material encountered in the test pits in the fill slope on the south side of Windsor Drive was found to be relatively loose. The clayey fill material has low expansion potential with Plasticity Indices of 9 and 13, and did have negligible swell upon saturation. We recommend that the fill slope behind Lots 17 through 20 be reconstructed by benching into the fill slope during grading. Currently, sliver cuts and fills less than 2 feet thick are proposed within the fill slope in this area. The outer approximate 5 feet of the fill slope should be reconstructed by benching into the fill slope and should extend up to about 1 foot from the back of curb on Windsor Drive. Plate 11 shows suggested benching and fill slope reconstruction sections in the locations of the 4 test pits.

Test pits excavated during our current as well as previous investigations were loosely backfilled with excavated materials. Where not removed by design cut, loose backfill at the test pit locations should be subexcavated and replaced with engineered fill.

SUBDRAINAGE

Ground water seepage is expected to occur in swales, at the bases of slopes, and in isolated pockets in the lower portions of the site. Subdrainage should be provided to intercept ground water in the following locations:

1. On the uphill side of all keyways and proposed fill,
2. Along swales and gullies to receive fill,
3. At all springs and seepage areas,
4. At the toes of major cut slopes,
5. At geologic contacts known to transmit water, and
6. In other areas of the site where seepage is observed during and after grading or as determined in the field by the soil engineer.

Subdrains should consist of perforated PVC pipe conforming to ASTM D 2751, Type SDR 35. Subdrains should be at least 6 inches in diameter. All subdrains should be surrounded by and underlain by at least 6 inches of Class 2 Permeable Material as defined in Section 68-1.025 of the Caltrans Standard Specifications. Subdrain trenches should be at least 18 inches wide and at least 4 feet deep. Final trench configurations should be approved by the soil engineer. Subdrain trenches should be capped with engineered fill or topsoil, depending on the location of the subdrain. Subdrain systems should be discharged into a storm drain structure (manhole, inlet) where possible. Subdrain details are provided on Plate 9.

Some areas of seepage may develop after house construction is completed. Additional subdrains may be needed in these areas should seepage develop.

EXCAVATION CHARACTERISTICS

Conditions encountered during our field investigations at the site as well as our experience in the area suggest that, in general, excavation to planned depths should be achievable using conventional grading equipment. Based on the high degree of fracturing and the fracture spacing encountered in the test pits, and the rock quality designation (RQD) logged in Boring B-2, we believe that large grading equipment such as a Caterpillar D-10 bulldozer with rippers should be adequate. Areas of very hard bedrock should be anticipated in deep cut areas at the site that are likely to generate oversize material. Modified excavation techniques such as using a single shank on a D-10 should generally be capable of ripping very hard-cemented areas of bedrock. Areas of hard rock were encountered in Boring B-2 and Test Pits TP2-2 through TP2-4, TP2-8, and TP2-10.

SELECTIVE GRADING

Special care should be taken to reduce the size of bedrock derived fill material so that the material can be properly compacted. Oversized material (greater than 6 inches) is expected to be generated from bedrock cuts at the site. Oversize material can be broken down mechanically or placed in deeper areas of fill and not within 10 feet of pad grade or street subgrade. Oversize material to be used in deeper areas of fill should be spread out so that large rocks are not concentrated in pockets and are surrounded by engineered fill. Placement of oversize material should be subject to approval by the soil engineer.

SITE PREPARATION AND GRADING

All grading operations should be performed in accordance with the following recommendations:

1. Existing earth materials on-site are considered suitable for re-use as engineered fill provided it does not contain rock fragments greater than 6 inches and is free of deleterious material as determined in the field by the soil engineer. Oversized material can either be buried at least 10 feet deep without nesting (see Selective Grading above) or removed from the site.
2. If import fill is used, it should have a Plasticity Index (PI) less than 15 and should be subject to evaluation and approval by the soil engineer prior to use.
3. All fill materials to be used at the site should be subject to evaluation and approval by the soil engineer prior to use.
4. Areas to be graded should be cleared and stripped of all vegetation. Strippings can be stockpiled and re-used as topsoil in landscape areas. Strippings can also be blended

with clean on-site soils at a ratio of 10 loads of clean soil to 1 load of strippings, to create a soil mixture suitable for use as engineered fill.

5. Existing foundations, wells, septic systems, leach fields and other subsurface structures should be completely removed prior to grading. Any soft soils encountered during excavation should be removed as determined in the field by the soil engineer.

6. The upper three feet of soil in areas mapped as colluvium should be reworked as engineered fill. This depth of reworking can be reduced as discussed under *Colluvium/Alluvium Overexcavation* below.

7. Low-expansion-potential bedrock cut derived material with a PI less than 20 should be used in keyways, landslide remediation and buttress fill slopes.

8. Where zones of soft or saturated soils are encountered during excavation and compaction, deeper excavation may be required to expose competent materials. This should be determined in the field by the soil engineer.

9. Areas to receive fill should be scarified to a minimum depth of 12 inches, brought to at least 3 percent over optimum moisture content, and compacted to not less than 90 percent relative compaction.

   Relative compaction refers to the in-place density of a soil expressed as a percentage of the maximum dry density determined by Test Method ASTM D1557. Optimum moisture is the water content (percentage by weight) corresponding to the maximum dry density.

10. If significant subgrade pumping and/ or yielding occur during scarification or compaction, it may be necessary to stabilize the exposed subgrade. The actual stabilization method, if warranted, will depend on exposed conditions and should be judged suitable by the soil engineer.

11. Fill should be placed in thin lifts (normally 6 to 8 inches thick, depending on compaction equipment used), moisture conditioned to at least 3 percent over optimum moisture, and compacted to at least 90 percent relative compaction. Modification to acceptable lift thickness should be determined in the field by the soil engineer and based on the demonstrated compaction performance during fill placement, which will depend on the equipment and methods used.

12. Fill placed on ground sloping greater than 7H:1V should be benched into firm materials as determined in the field by the soil engineer.

13. Fill slopes should be over built and cut back to expose a firm compacted surface.

14. Observation and soil density testing should be performed during grading to assist the contractor in achieving the required degree of compaction and the proper moisture content. Where compaction is less than required, additional compactive effort should be made with an adjustment in the moisture content where necessary until the specified compaction is obtained.

15. The soil engineer should be informed at least 48 hours prior to any grading operation. The procedures and methods can then be discussed between the developer, contractor,

and soil engineer. This can facilitate the performance of grading operations and minimize potential construction delays.

CUT/ FILL TRANSITION LOT TREATMENT

Because the proposed fill and bedrock at the site will have different expansion and settlement potential, structures and slabs placed across the transition line between cut and fill could experience differential expansion and/ or settlement. This condition can be mitigated by overexcavating the cut portion of the cut/ fill transition lots to a depth of about 3 feet below rough pad grade. The exposed excavation bottom should then be scarified to a minimum depth of 12 inches, moisture conditioned to not less than 3 percent over optimum moisture content and compacted to at least 90 percent relative compaction. The overexcavation should be restored with engineered fill. Typical Cut/Fill Transition Lot Overexcavation Details are provided on Plate 10.

The horizontal and vertical extent of overexcavation should be determined in the field by the soil engineer. We recommend that the contract documents provide for add-and-deduct unit prices for excavation and replacement as engineered fill to allow for unanticipated variations in excavation quantities.

BEDROCK CUT LOT TREATMENT

Cut lots that have subgrades exposing bedrock should be overexcavated and compacted a minimum depth of 3 feet. The exposed surface should be scarified to a depth of about 12 inches, moisture conditioned to not less than 3 percent over optimum moisture content, and compacted to at least 90 percent relative compaction. This is to allow for easier excavation of utility trenches and planting of vegetation.

COLLUVIUM/ ALLUVIUM OVEREXCAVATION

Depending on the time of year that grading operations occur at the site, it may be necessary to rework the upper 3 feet of areas mapped as colluvium and alluvium prior to placement of fill. The necessity to rework these areas will depend on the presence of desiccation cracks in the soil. Desiccation cracks in these types of soils often extend to a depth of about 3 feet and occur late in the dry seasons as the soil moisture content decreases. We anticipate that the upper about foot will be reworked during normal stripping and scarification processes. If desiccation cracks extend below the depth of scarification, additional reworking will be required as determined in the field by the soil engineer. The need for additional reworking of colluvium and alluvium can be reduced if grading occurs early in the grading season, prior to drying of the soil and the formation of desiccation cracks.

EXPANSION POTENTIAL

As indicated by the results of our Atterberg limits and single-point consolidation/swell tests on the on-site soil and bedrock materials, the expansion potential of the on-site soil material is generally moderate. The total swell of fill placed and compacted following the recommendations presented under *Site Preparation and Grading* are estimated as follows:

ESTIMATED POTENTIAL SWELL OF COMPACTED FILL	
Fill Thickness (feet)	**Swell (inches)**
5	¾
10	1
15	1¼

These preliminary potential swell estimates are based on a uniform mixture of soil and bedrock generated from the design cuts planned at the site. The actual swell in fill areas will depend on the total depth of fill, the depths of placement of various materials in the fill, and the in-place moisture content and density. The maximum fill slope planned for this site is approximately 30 feet as measured from top of slope to toe of slope. The maximum depth of fill as measured vertically at the top of fill slope is approximately 15 feet. Swell of 1¼ inch measured vertically over the 15 feet maximum fill depth is 0.7% of the fill depth. This minor swell percentage is judged to be insignificant.

SETTLEMENT OF COMPACTED FILL

The results of single-point consolidation tests on remolded soil samples from the site, representing proposed fill, are summarized in Appendix B. Based on these results, we estimate that on-site soil and bedrock materials used as fill will undergo some settlement during placement and for a duration following mass grading. The total settlement of the fill placed and compacted following the recommendations presented under *Site Preparation and Grading* are estimated as follows:

PRELIMINARY ESTIMATED POTENTIAL SETTLEMENT OF COMPACTED FILL	
Fill Thickness (feet)	**Preliminary Estimate of Total Settlement (inches)**
5	¼
10	½
15	1

Based on our laboratory test results and our experience, we anticipate that about 70 percent of the estimated total settlement of the fill should occur during mass grading. Therefore, we estimate that the maximum post-grading settlement should be less than 1 inch. The maximum fill slope planned for this site is approximately 30 feet as measured from top of slope to toe of slope. The maximum depth as measured vertically at the top of fill slope is approximately 15 feet. Settlement of 1 inch measured vertically over the 15 feet fill depth is 0.6% of the fill depth. This minor settlement percentage is judged to be insignificant.

SETTLEMENT OF COLLUVIUM AND ALLUVIUM

In some areas at the site, up to about 15 feet of fill is planned at locations underlain with colluvium and alluvium extending to depths of about 25 feet down to bedrock. Based on our boring log data and the results of our laboratory testing, we believe that the colluvium and alluvium at the site consist of stiff to very stiff, silty to sandy clays and clayey sands. Settlement of these deposits should take place upon application of the new fill loads, and should be on the order of less than about 1 inch. This settlement should not adversely affect the proposed development.

RESIDENTIAL FOUNDATIONS

GENERAL

The site soils generally consist of stiff colluvial and alluvial soils with shallow bedrock between 5 to 20 feet deep. Provided the grading recommendations presented in this report are adhered to, the proposed homes may be supported on either on grade structural mat, post tension slab or drilled cast-in-place concrete pier and grade beam foundations. Recommendations and design parameters for these foundation types are as follows:

ON GRADE STRUCTURAL MAT FOUNDATIONS

On grade structural mat foundations should be designed by a structural engineer to accommodate 1 inch of differential movement in 25 horizontal feet. We recommend that the following criteria be incorporated in the design of on grade structural mat foundations:

Allowable Bearing Capacity (may be increased by 1/3 for temporary seismic and wind loads at the discretion of the structural engineer)	1,500 psf
Passive Equivalent Fluid Pressure (neglect the upper 1 foot if ground surface is not confined by slabs or pavement)	300 psf
Base Friction Coefficient	0.3
Minimum Embedment at the Building Exterior	6 inches
Modulus of Subgrade Reaction	100 pci

The upper 12 inches of subgrade soil should be presaturated to at least 5 percent above optimum moisture content. The presaturated pad should not be allowed to dry out to less than this recommended moisture content prior to the construction of the slab. The on grade mat foundation can be placed directly upon the prepared subgrade soil. Where moisture vapor transmission through the slab would be objectionable, the use of a vapor retarder should be considered by the designer.

POST TENSION SLAB FOUNDATIONS

We recommend that the following criteria be incorporated in the design of PT slab foundations. These parameters are in general accordance with Post Tension Institute.

Allowable Bearing Capacity (may be increased by 1/3 for seismic and wind load a the discretion of the structural engineer)	1,500 psf
Passive Equivalent Fluid Pressure (neglect the upper 1 foot if ground surface is not confined by slabs or pavement)	300 psf
Base Friction Coefficient	0.3
Edge Variation Distance Center Lift Edge Lift	 9.0 feet 4.8 feet
Differential Swell Center Lift Edge Lift	 0.78 inch 1.14 inch

The upper 12 inches of subgrade soil should be presaturated to at least 5 percent above optimum moisture content. The presaturated pad should not be allowed to dry out to less than this recommended moisture content prior to the construction of the slab. The on PT slab foundation can be placed directly upon the prepared subgrade soil. Where moisture vapor transmission through the slab would be objectionable, the use of a vapor retarder should be considered by the designer.

PIER AND GRADE BEAM FOUNDATIONS

Drilled cast-in-place reinforced concrete friction piers and grade beams are suitable foundation support for the proposed homes. Foundation support would be provided by skin friction between the pier shaft and surrounding soil. The reinforced concrete piers and grade beams should be designed by a structural engineer with the following design parameters.

Minimum depth below finish soil pad grade (feet)	8
Minimum diameter (inches)	12
Minimum pier spacing	3 pier diameters measured center-to-center
Allowable skin friction (psf) in compression	450
Allowable skin friction (psf) in tension	300
Passive resistance (pcf, equivalent fluid pressure)	300
Minimum Grade beam embedment (inches)	6

Skin friction and passive resistance should be neglected in the upper 1 foot below adjacent grade. Passive pressure should only be applied for the portion with at least 10 feet of soil horizontally when near or on a slope.

Prior to placement of reinforcing steel and concrete, the bottom of the pier excavations should be free of excess loose soil and debris. Water that has collected in pier hole excavations should be pumped out or displaced by means of a tremie method.

RETAINING WALLS

Retaining walls, up to about 5 feet high, are planned at grade breaks between lots and at toes of slopes. We recommend that the following geotechnical criteria be incorporated in the design of retaining walls:

Active Equivalent Fluid Pressure Level Backfill Sloping Backfill	 50 pcf 65 pcf
At-rest Equivalent Fluid Pressure	75 pcf
Allowable Bearing Capacity (may be increased by one-third for seismic and wind loads at the discretion of the structural engineer)	2,500 psf
Passive Equivalent Fluid Pressure (neglect the upper 1 foot if the ground surface is not confined by slabs or pavement)	350 pcf
Friction Coefficient	0.3
Seismic Increment for Retaining Walls more than 6 feet tall.	25H psf
Minimum Footing Depth	18 inches below the lowest adjacent grade

The above recommended lateral pressures are based on drained conditions~~,~~ and do not include any surcharges**;** therefore, the designer should include the appropriate surcharge loads to the retaining walls.

To prevent hydrostatic pressure build-up, retaining walls should be constructed with permanent backdrains. The backdrain should consist of a blanket of Class 2 Permeable Material and a 4-inch diameter perforated PVC pipe (SDR 35). The permeable materials should be in conformance with Section 68-1.025 of the 1999 Caltrans “Standard Specifications.” The permeable material blanket should be at least 12 inches thick and should be placed from the base of the retaining wall to about 1 foot below the finished grade behind the retaining wall. Alternatively, a geo-composite drain, such as Miradrain 2000 or an approved equivalent, may be used in lieu of the Class 2 Permeable Material blanket. The perforated pipe should be placed near the bottom of the wall to carry collected water to a suitable gravity discharge.

MSE RETAINING WALL DESIGN PARAMETERS

Mechanically Stabilized Earth (MSE) Retaining Walls are constructed of precast modular blocks and geogrid reinforcement.

Reinforced Fill, Retained Fill and Foundation Unit Weight Friction Angle Cohesion	 125 pcf 25 degrees 200 psf

The base of the modular block walls should be at least 6 inches (level ground) and 18 inches (sloped ground) below the lowest adjacent finished grade.

SEISMIC DESIGN PARAMETERS

The site is located in a region of high seismicity given the proximity of the Rodgers Creek fault, San Andreas fault, and other active faults in the San Francisco Bay Area. As for all sites in the Bay Area, the project can be expected to experience at least one moderate to severe earthquake during the life span of the development. Ground shaking is a hazard that cannot be eliminated but can be partially mitigated through proper attention to seismic structural design and observance of good construction practices.

- The Scott Ranch site is located at approximately 38.2174 degrees North latitude and 122.6470 degrees West longitude. The Peak Ground Acceleration (PGA) according to the 2013 CBC is 0.53 g. We are providing the following 2013 California Building Code seismic design criteria using the USGS Seismic Design Maps program, Version 3.1.0 dated July 11, 2013.

Mapped Spectral Acceleration for Short Periods, S_s	1.500 g
Mapped Spectral Acceleration for 1-Second Period, S_1	0.600 g
Site Class	D
Site Coefficient F_a (for Site Class D)	1.0
Site Coefficient F_v (for Site Class D)	1.5
Acceleration Parameter S_{MS} (adjusted for Site Class D)	1.500 g
Acceleration Parameter, S_{M1} (adjusted for Site Class D)	0.900 g
Acceleration Parameter, S_{DS}(adjusted for Site Class D)	1.000 g
Acceleration Parameter, S_{D1} (adjusted for Site Class D)	0.600 g

PRELIMINARY PAVEMENT SECTIONS

The following recommendations for asphalt concrete pavement sections are preliminary only. Pavement analyses are based on an assumed “R” (resistance) value of 5, which we expect to be representative of final pavement subgrade materials, Caltrans *Design Method for Flexible Pavement*, and traffic indices (TI’s), which are indications of traffic load frequency and intensity. Assigned TI’s should include provisions for heavy truck traffic related to construction activities. We recommend the following preliminary pavement sections:

PRELIMINARY RECOMMENDED PAVEMENT SECTIONS		
Traffic Index (TI)	**Thickness (inches)**	
	Asphalt Concrete Type B	**Class 2 Aggregate Base**
4	2½	8
4½	2½	10
5	2½	11
5½	3	12
6	3	14

Since on-site materials vary from sandstone to clay, samples should be obtained from the rough roadway subgrade after mass grading. R-value tests should be performed on these samples. Final pavement section recommendations should be made on the basis of these test results.

Prior to subgrade preparation, all utility trench backfill should be properly placed and compacted. Subgrade soils should be rolled to at least 95 percent relative compaction to provide a smooth, unyielding surface. Subgrade soils should be maintained in a moist and compacted condition until covered with the complete pavement section.

Class 2 aggregate base should conform to the requirements in Section 26 of Caltrans’ *Standard Specifications*. The aggregate base should be placed in thin lifts in a manner to prevent segregation, uniformly moisture conditioned, and compacted to at least 95 percent relative compaction to provide a smooth, unyielding surface. Relative compaction refers to the in-place dry density of soil expressed as a percentage of the maximum dry density of the same soil, as determined by the ASTM D1557 compaction test method.

Where drop inlets or other surface drainage structures are to be installed, slots or weep holes should be provided to allow free drainage of the contiguous aggregate base section.

EXTERIOR FLATWORK

It is our opinion the exterior concrete flatwork may be placed directly on the finish soil subgrade. The soil subgrade should be compacted to a minimum 90 percent relative compaction at a moisture content not less than 3 percent over optimum. All exterior concrete flatwork be cast free from adjacent footings or building slabs. The moisture-conditioned subgrade should not be allowed to lose moisture prior to concrete placement. If the subgrade dries out and shrinkage cracks appear, the subgrade should be reconditioned in accordance with the recommendations of the geotechnical engineer in the field.

UTILITY TRENCHES

All excavations should conform to applicable state and federal OSHA standards. Where trench excavations are deeper than 5 feet, they should be sloped no steeper than 1H:1V and/ or shored. Flatter side slopes may be required if seepage is encountered during construction or if the exposed materials differ from those described in the test pit and boring logs. If fully sloped trench walls cannot be excavated due to site constraints, shoring should be provided to ensure trench stability for worker safety. We can provide parameters for shoring design on request.

Material quality, placement procedures, and compaction requirements for utility line bedding and shading materials should meet the City of Petaluma and/or applicable utility agency requirements. From a geotechnical standpoint, the material above the shading material may consist of native materials, compacted to no less than 90 percent relative compaction and 3 percent over optimum moisture content.

Depending on time of year, location, and recent rainfall, ground water may be intercepted during trench excavation, in which case local dewatering will be required. The actual dewatering technique to be used should be approved by the soil engineer before implementation.

CORROSION TESTING

We have obtained three soil samples from the site for corrosion testing. The corrosion testing was performed by CERCO Analytical, Inc., of Pleasanton, California, and the test results are included in Appendix C. The corrosion test results should be transmitted to your structural engineer and underground utility designer, and should be incorporated in the design of the concrete and pipes to be placed directly against the on-site soils.

SEISMIC HAZARDS

SURFACE FAULT RUPTURE

We did not encounter evidence of Quaternary fault traces crossing, passing near, or trending toward the site. The site is not located within an official State of California earthquake fault zone (Davis 2000; Hart and Bryant, 1999) for active faults. According to the State of California, a fault is considered active if it has demonstrated Holocene activity (within the past 11,000 years). We conclude that the potential for surface fault rupture at the site is low.

SECONDARY EFFECTS OF GROUND SHAKING

Liquefaction is the temporary transformation of a saturated, cohesionless soil into a viscous liquid during strong ground shaking from a major earthquake. Dynamic densification can occur when dry, loose, cohesionless soil is subjected to earthquake vibrations of high amplitude. We did not encounter earth materials susceptible to liquefaction or significant dynamic densification at the site.

Strong ground shaking during a major earthquake is liable to initiate landsliding in parts of the region. The stability of all slopes is lower during earthquake disturbances than at other times. Grading in accordance with the recommendations presented above (under *Landslide Remediation* and *Graded Slopes*) is expected to result in a low risk of seismically induced landslides.

ADDITIONAL SERVICES

Our firm should be afforded the opportunity to review the final plans and specifications to determine if the recommendations contained herein are incorporated into those documents. The review would be acknowledged in writing. Field observation and testing are essential and integral parts of this geotechnical investigation. Our firm should be retained to monitor earthwork and other relevant construction operations; the recommendations of this report are contingent on this.

LIMITATIONS

The conclusions and recommendations contained herein are based upon the information provided to us regarding proposed improvements, our geologic reconnaissance of the site, subsurface conditions encountered during the course of our field investigation, the results of our laboratory testing program, our experience in the area, and professional judgment. This study has been conducted in accordance with current professional geotechnical engineering and engineering geology standards; no other warranty is expressed or implied.

The locations of borings were determined by pacing from existing cultural features and other points of reference depicted on plans prepared by BKF and are considered approximate only. Site conditions described in the text are those existing at the time of our last site visit in April 2003 and are not necessarily representative of such conditions at other locations or times.

If it is found during construction that the conditions differ from those described on the boring and test pit logs, then the conclusions and recommendations contained within this report shall be considered invalid unless the changes are reviewed and the conclusions and recommendations modified or approved in writing by BSA.

Respectfully submitted,

BERLOGAR STEVENS & ASSOCIATES

W. R. Stevens

William R. Stevens
Principal Engineer
GE 2339

REGISTERED PROFESSIONAL ENGINEER
WILLIAM R. STEVENS
No. 2339
GEOTECHNICAL
STATE OF CALIFORNIA

Frank Berlogar

WRS/FB:jmo

Attachments:
- References
- Plate 1 – Vicinity Map
- Plate 2 – Geologic Map, Option A
- Plate 3 – Geologic Map, Option B
- Plate 4 – Geologic Cross Sections A-A′, B1-B1′, B2-B2′, B3-B3′ and C-C′
- Plate 5 – Remedial Cross Sections A-A′, B1-B1′, B2-B2′, B3-B3′ and C-C′
- Plate 6 – Remedial Grading Plan – Option A
- Plate 7 – Remedial Grading Plan – Option B
- Plate 8 – Typical Fill Slope Details
- Plate 9 – Typical Subdrain Details
- Plate 10 – Typical Cut/ Fill Transition Lot Overexcavation Details
- Plate 11 – Graphic Test Pit Logs, TP1 through TP4
- Appendix A – Field Investigation Data
  - A-1 through A-20 – Boring Logs
  - A-21 – Unified Soil Classification System
  - A-22 – Key to Rock Descriptions
  - A-23 through A-31 – Test Pit Logs 2004
  - A32 through A-38 – Test Pit Logs 2002
- Appendix B – Laboratory Test Results
  - B-1 – Atterberg Limits Test Results
  - B-2 through B-7 – Direct Shear Test Results
  - B-8 through B-11 – Compaction Test Data
  - B-12 through B – Gradation Test Data
  - B-15 through B-17 – Consolidation Test Data
- Appendix C – CERCO Analytical, Inc. Corrosion Test Data
- Appendix D – Responses to Treadwell & Rollo and Haley & Aldrich Comments

Copies: Addressee (6)
Mr. Steve Abbs, Davidon Homes (e-mail only)

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REFERENCES

Blake, M.C., Jr., Bartow, J.A., Frizzell, V.A., Jr., Schlocker, J., Sorg, D., Wentworth, C.M., and Wright, R.H., 1974, Preliminary geologic map of Marin and San Francisco Counties and parts of Alameda, Contra Costa and Sonoma Counties, California: U.S. Geological Survey Miscellaneous Field Studies Map MF-574.

Davis, J., 2000, Digital images of official maps of the Alquist-Priolo earthquake fault zones of California, Central Coast Region, California Division of Mines and Geology: Division of Mines and Geology CD 2000-004-2000.

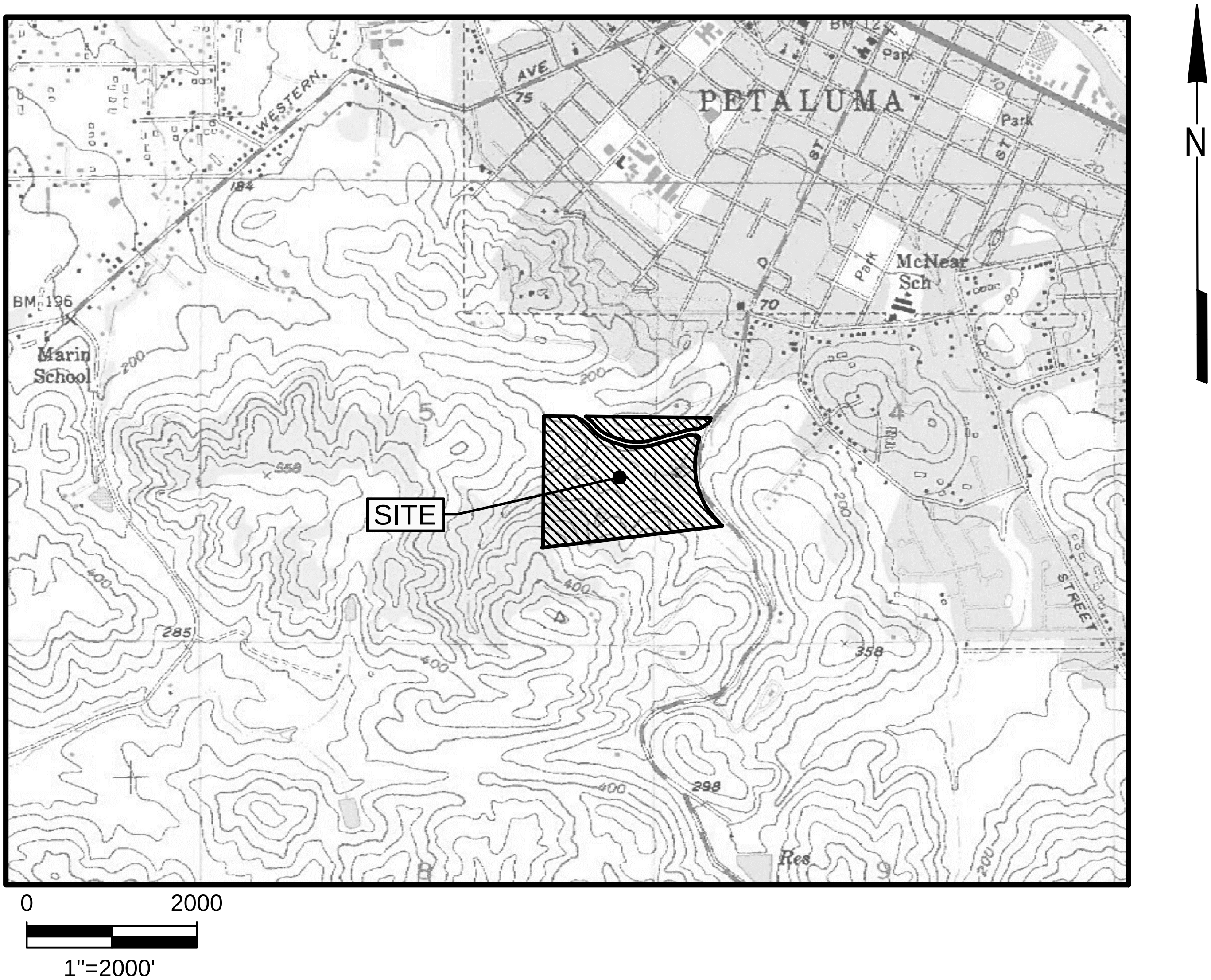
Hart, E.W., and Bryant, W.A., 1999, Fault-rupture hazard zones in California, Alquist-Priolo earthquake fault zoning act with index to earthquake fault zones maps: California Division of Mines and Geology Special Publication 42.

Peterson, M.D., Toppoxada, T., Cao, T., Cramer, C., Reichle, M., Maher, M., and Atchinson, L., 1998, Determining distances from faults within and bordering the state of California for the 1997 Uniform Building Code, in CDMG and SEAC, Maps of Known Active Fault Near-Source Zones in California and Adjacent Portions of Nevada, p. ix-xvii: International Conference of Building Officials, Whittier, California.

AERIAL PHOTOGRAPHS

Date	Photographer	Project, flight line, frames	Nominal scale
09/25/73	U.S. Geological Survey	2-58 and -59	1:24,000
04/19/86	Pacific Aerial Surveys	AV-2860-7-31, -32	1:12,000
06/15/00	Pacific Aerial Surveys	SON-AV-6540-19-34, -35	1:12,000

JOB NUMBER: 2616.006 DATE: 4-3-14 BY: CC



VICINITY MAP

SCOTT RANCH

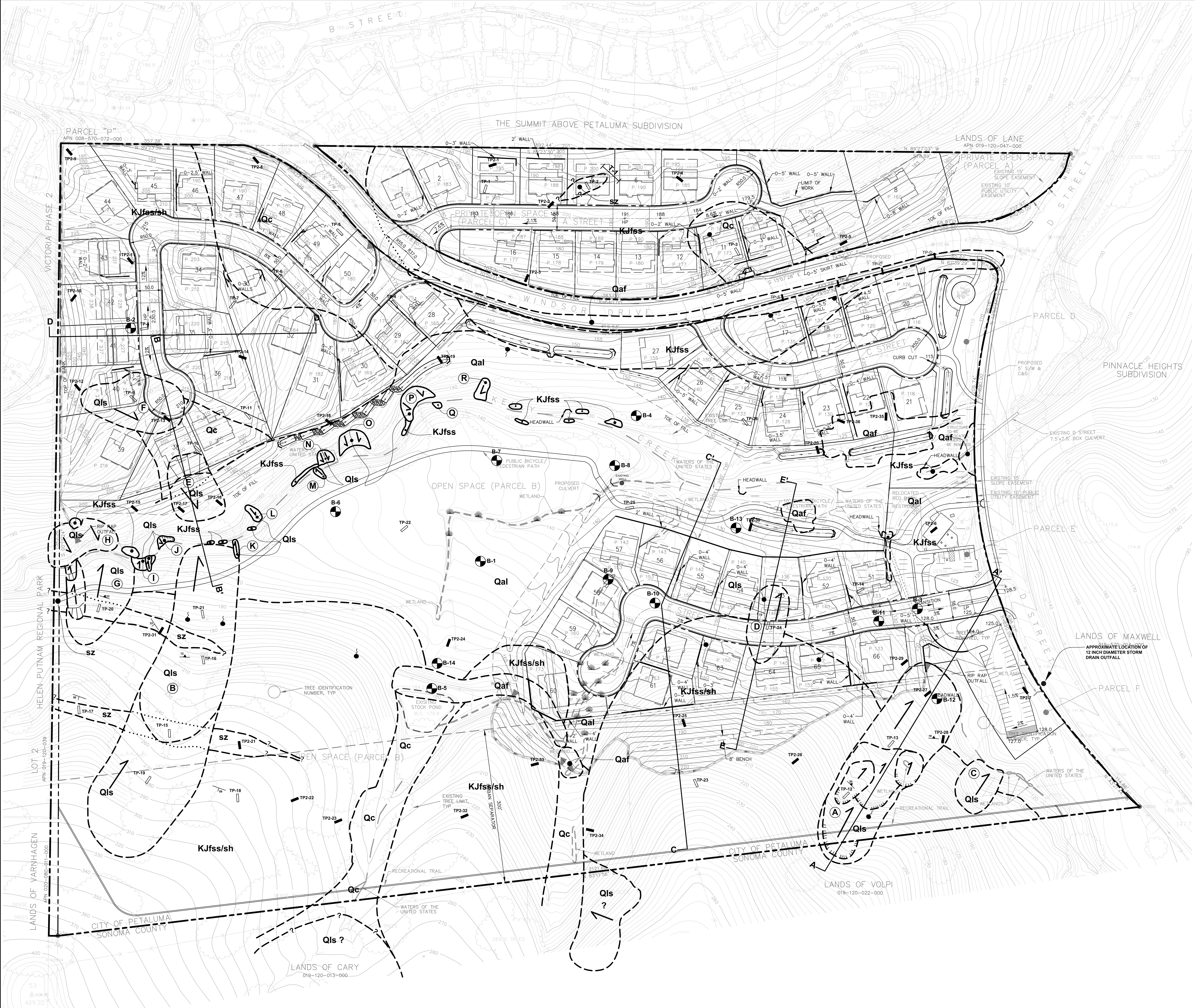
PETALUMA, CALIFORNIA

FOR

DAVIDON HOMES

BASE: PORTION OF U.S.G.S. 7.5 MINUTE TOPOGRAPHIC QUADRANGLE,
PETALUMA, CALIFORNIA, AT A SCALE OF 1:24,000.

PLATE 1



N

0 60
1"=60'

EXPLANATION

	PROPERTY LINE
?	GEOLOGIC CONTACT, DASHED WHERE APPROXIMATE, DOTTED WHERE CONCEALED, QUERIED WHERE UNCERTAIN
E E'	GEOLOGIC CROSS SECTION
T-1	TRENCH LOCATION, BGC 2002
B-14	BORING LOCATION, BGC 2003
TP-D	TEST PIT LOCATION (THIS STUDY)
TP2-36	TEST PIT LOCATION, BGC 2003
TP-26	TEST PIT LOCATION, BGC 2002
Qaf	ARTIFICIAL FILL
Qls	LANDSLIDE DEBRIS
Qc	COLLUVIUM
Qal	ALLUVIUM
KJfss	FRANCISCAN COMPLEX SANDSTONE
KJfss/sh	FRANCISCAN COMPLEX SANDSTONE AND SHALE
sz	SHEAR ZONE
73	STRIKE AND DIP OF BEDDING
23	STRIKE AND DIP OF SHEAR
	LANDSLIDE, RECENTLY ACTIVE
	LANDSLIDE, SURFICIAL
	SPRING
	EROSION GULLY
	WATER SEEKING VEGETATION
R	LANDSLIDE DESIGNATION

GEOLOGIC MAP

SCOTT RANCH
OPTION A - 66 LOTS

PETALUMA, CALIFORNIA
FOR
DAVIDON HOMES

Berlogar Stevens & Associates
SOIL ENGINEERS * ENGINEERING GEOLOGISTS

BASE: SHEET 5 TITLED, "REVISED PROJECT - OPTION A - 66 LOTS PROPOSED GRADING PLAN", PREPARED BY BKF, DATED 12-11-13

EXPLANATION

	PROPERTY LINE
?	GEOLOGIC CONTACT, DASHED WHERE APPROXIMATE, DOTTED WHERE CONCEALED, QUERIED WHERE UNCERTAIN
E E'	GEOLOGIC CROSS SECTION
T-1	TRENCH LOCATION, BGC 2002
B-14	BORING LOCATION, BGC 2003
TP-D	TEST PIT LOCATION (THIS STUDY)
TP2-36	TEST PIT LOCATION, BGC 2003
TP-26	TEST PIT LOCATION, BGC 2002
Qaf	ARTIFICIAL FILL
Qls	LANDSLIDE DEBRIS
Qc	COLLUVIUM
Qal	ALLUVIUM
KJfss	FRANCISCAN COMPLEX SANDSTONE
KJfss/sh	FRANCISCAN COMPLEX SANDSTONE AND SHALE
sz	SHEAR ZONE
73	STRIKE AND DIP OF BEDDING
23	STRIKE AND DIP OF SHEAR
	LANDSLIDE, RECENTLY ACTIVE
	LANDSLIDE, SURFICIAL
	SPRING
	EROSION GULLY
	WATER SEEKING VEGETATION
R	LANDSLIDE DESIGNATION

GEOLOGIC MAP

SCOTT RANCH

OPTION B - 63 LOTS

PETALUMA, CALIFORNIA

FOR

DAVIDON HOMES

Berlogar Stevens & Associates

SOIL ENGINEERS * ENGINEERING GEOLOGISTS

BASE: SHEET 5 TITLED, "REVISED PROJECT - OPTION B - 63 LOTS PROPOSED GRADING PLAN", DATED: 12-5-13, PREPARED BY BKF ENGINEERING

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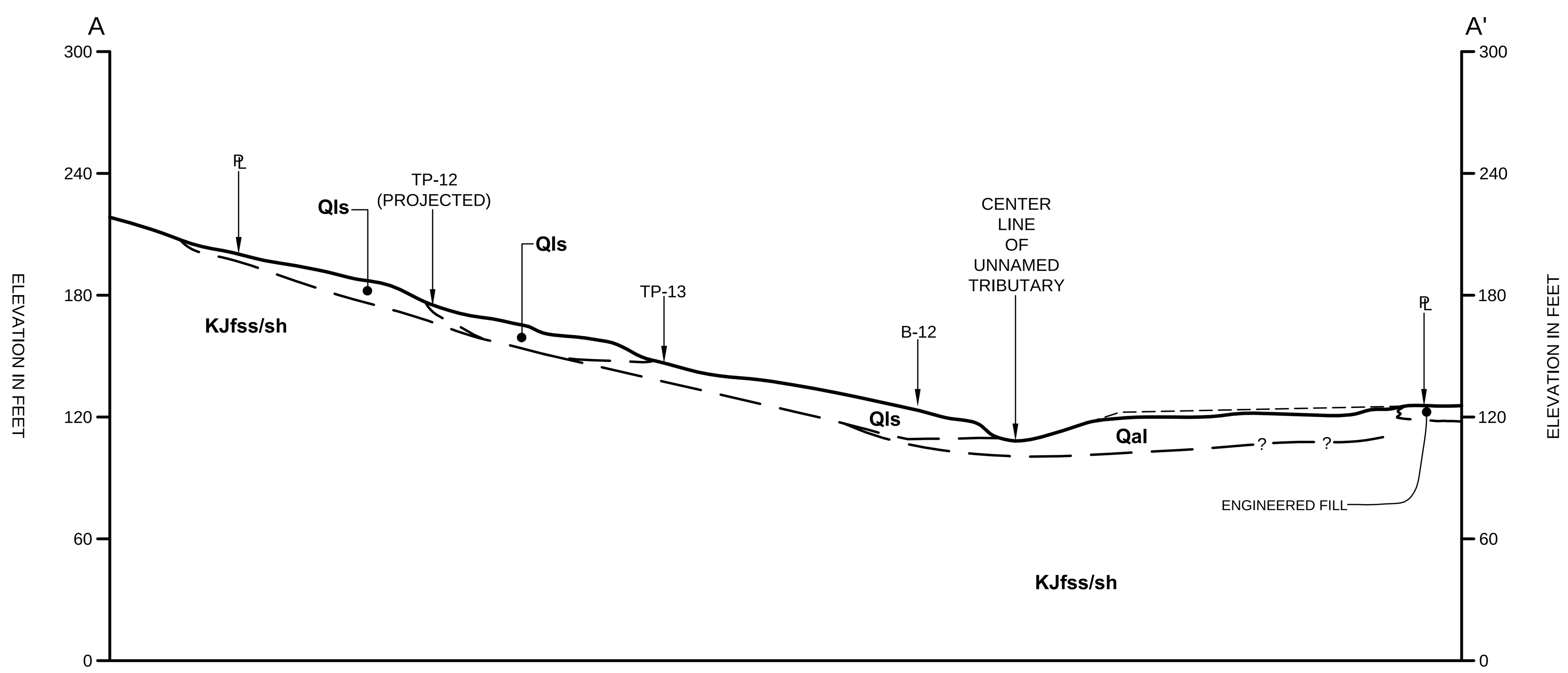

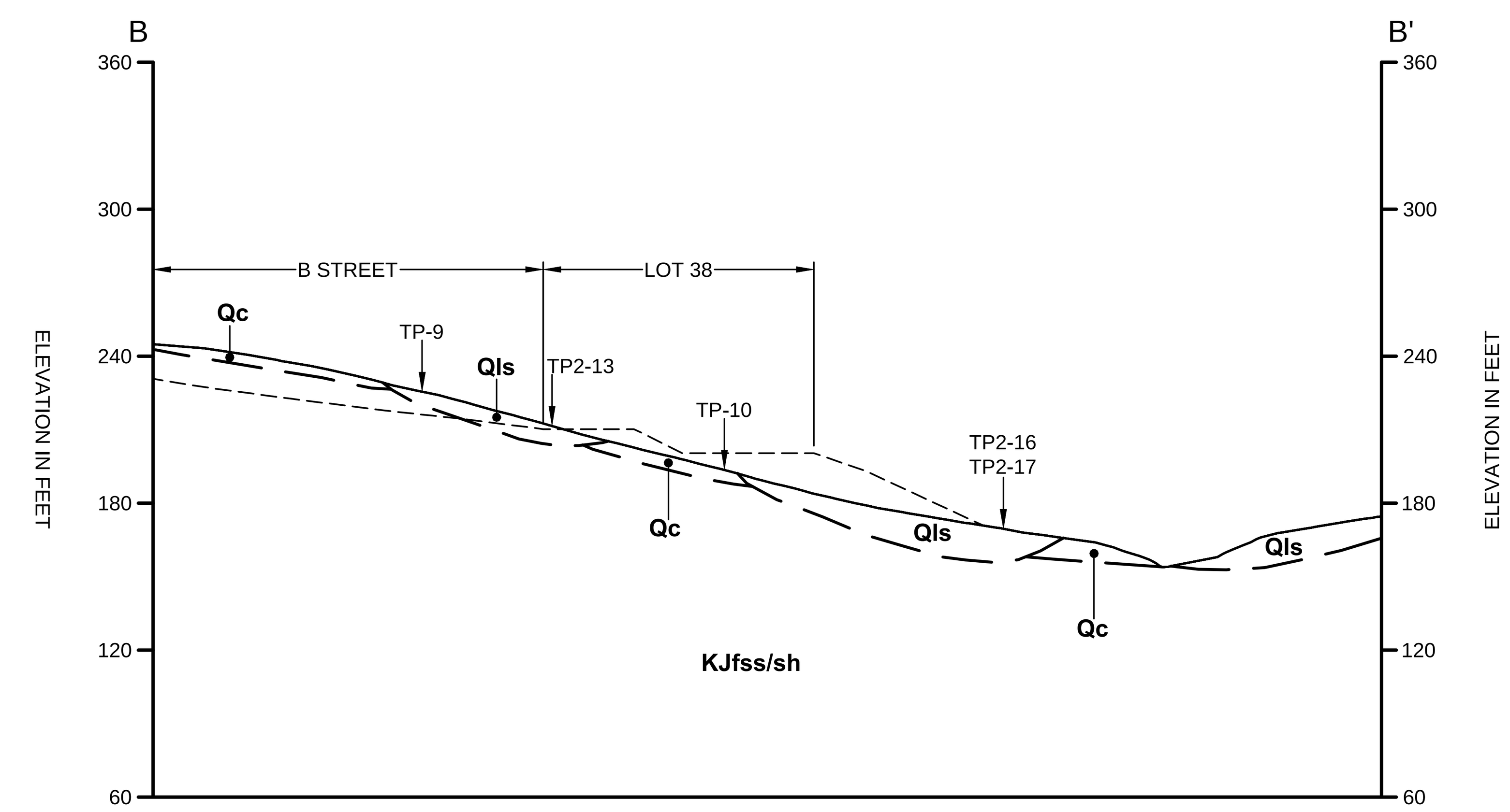

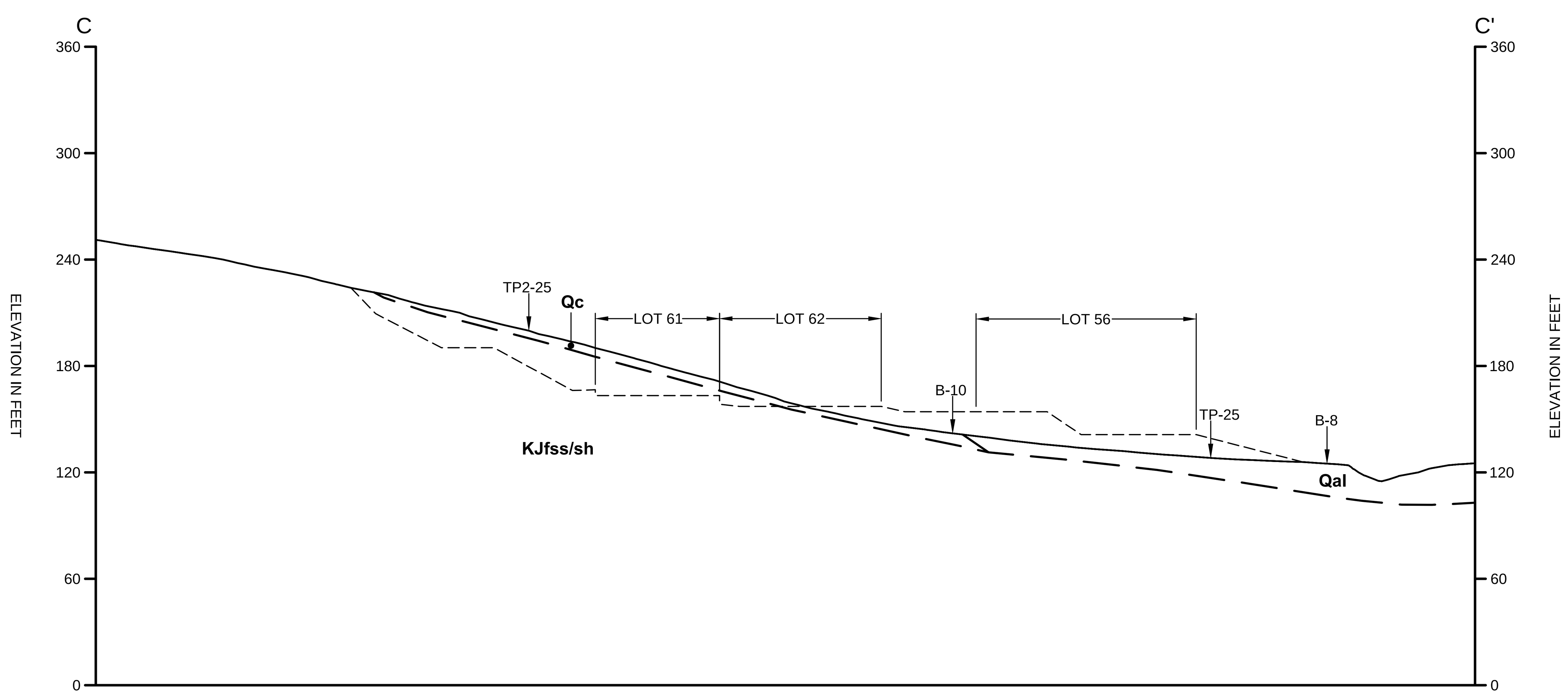

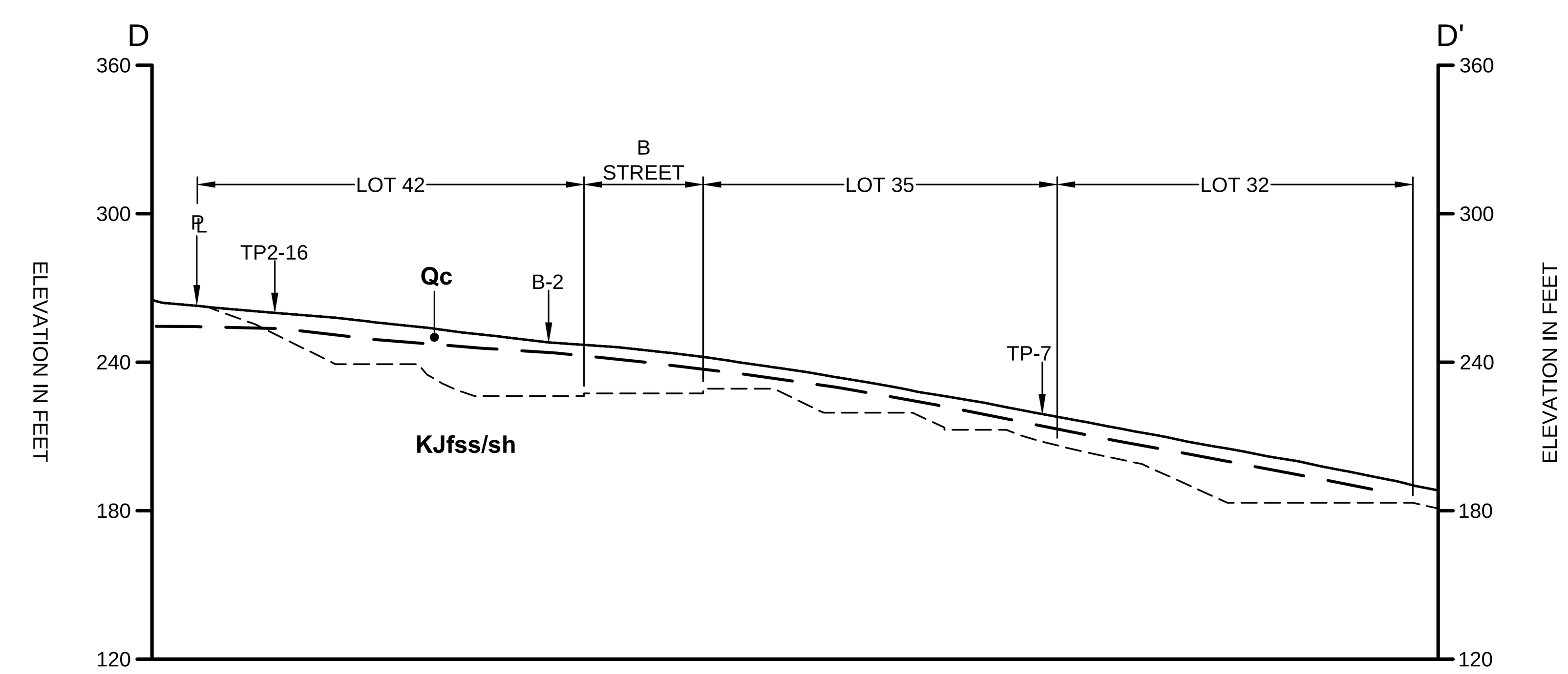

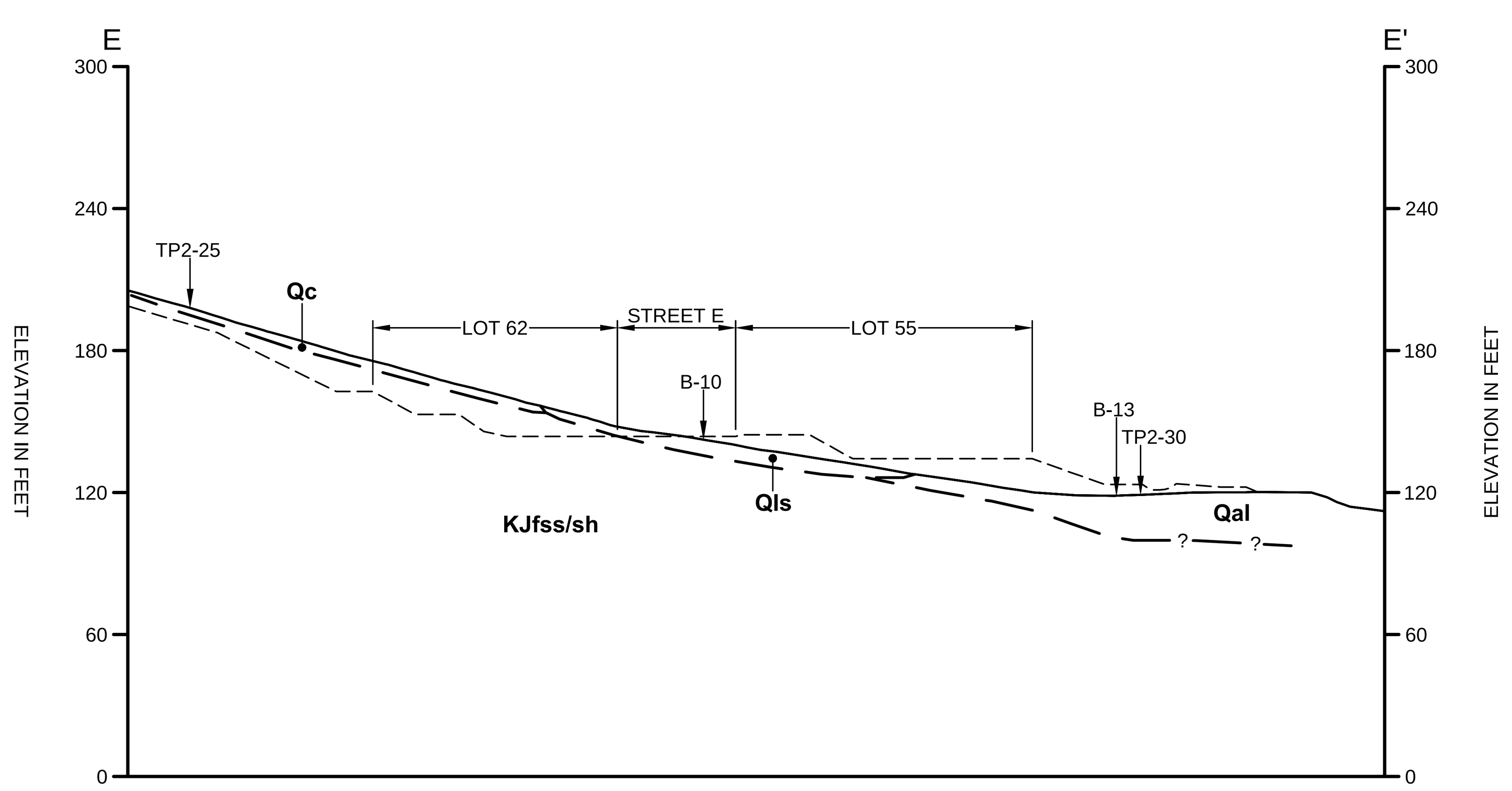

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DATE: 5-7-15

JOB NUMBER: 2616.006

EXPLANATION

EXISTING GRADE

PROPOSED GRADE (OPTIONS A AND B)

? GEOLOGIC CONTACT, DASHED WHERE APPROXIMATE, QUERIED WHERE UNCERTAIN

B-14 BORING LOCATION (BGC, 2003)

TP2-24 TEST PIT LOCATION (BGC, 2003)

TP-21 TEST PIT LOCATION (BGC, 2002)

Qaf	ARTIFICIAL FILL
Qls	LANDSLIDE DEBRIS
Qal	ALLUVIUM
KJfss/sh	FRANCISCAN COMPLEX SANDSTONE

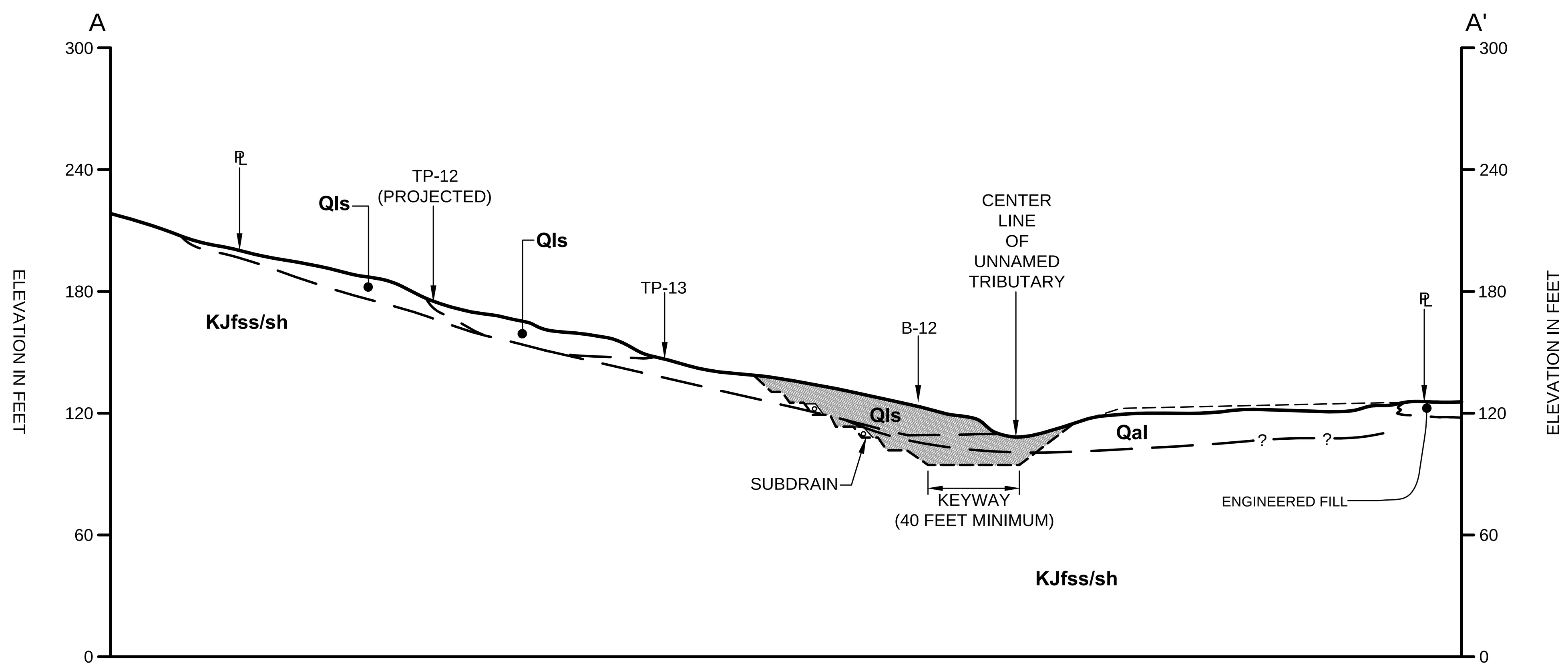
GEOLOGIC CROSS SECTIONS A-A' THROUGH E-E'

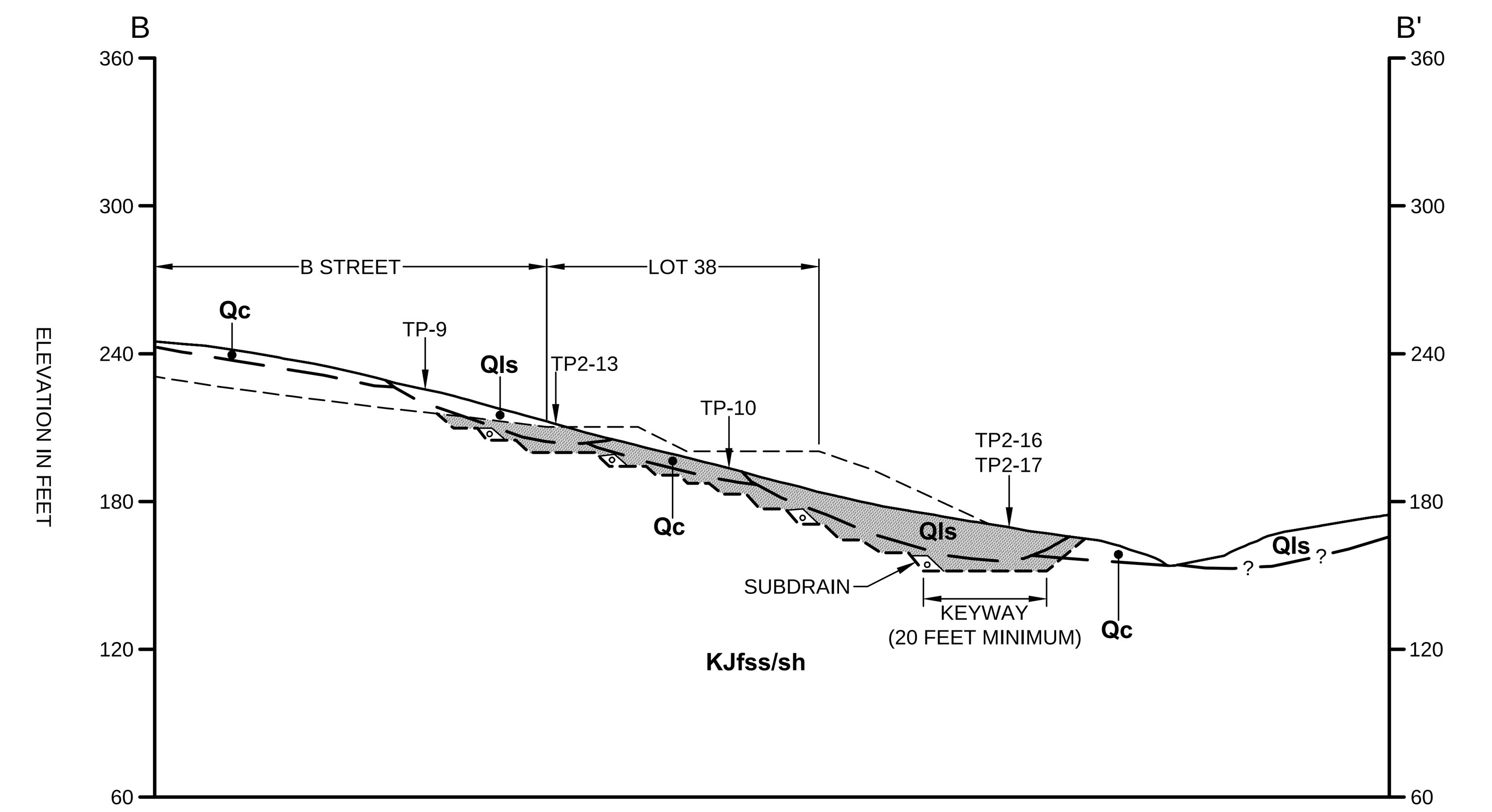
SCOTT RANCH

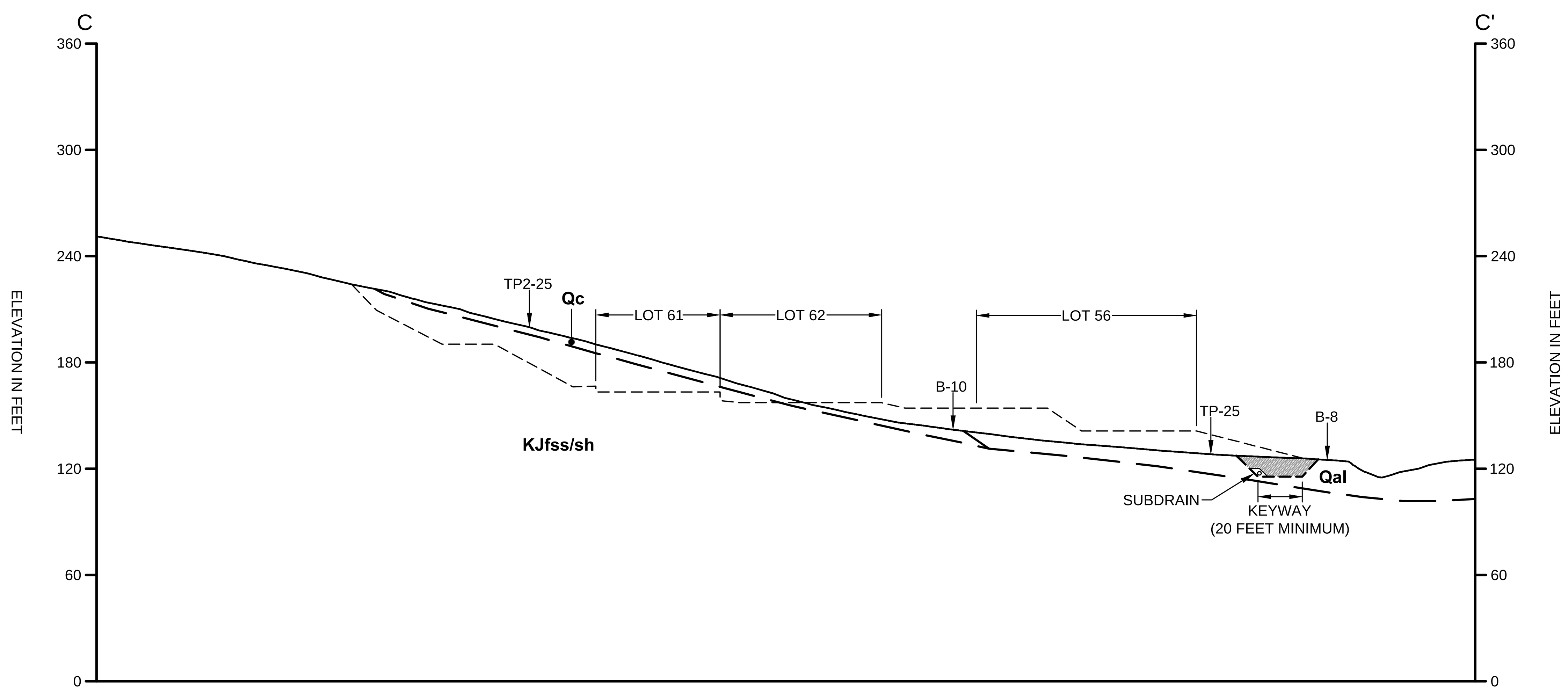
PETALUMA, CALIFORNIA
FOR
DAVIDON HOMES

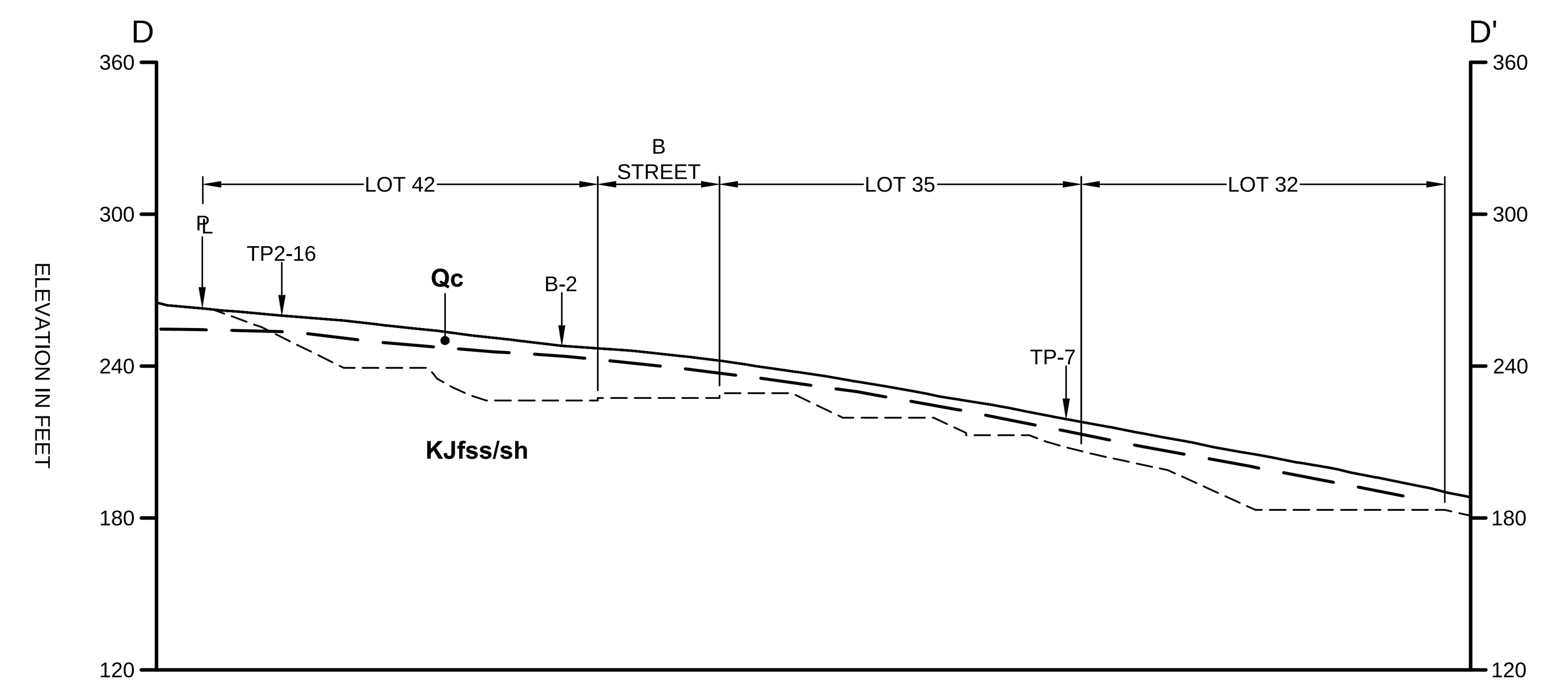
Berlogar Stevens & Associates
SOIL ENGINEERS * ENGINEERING GEOLOGISTS

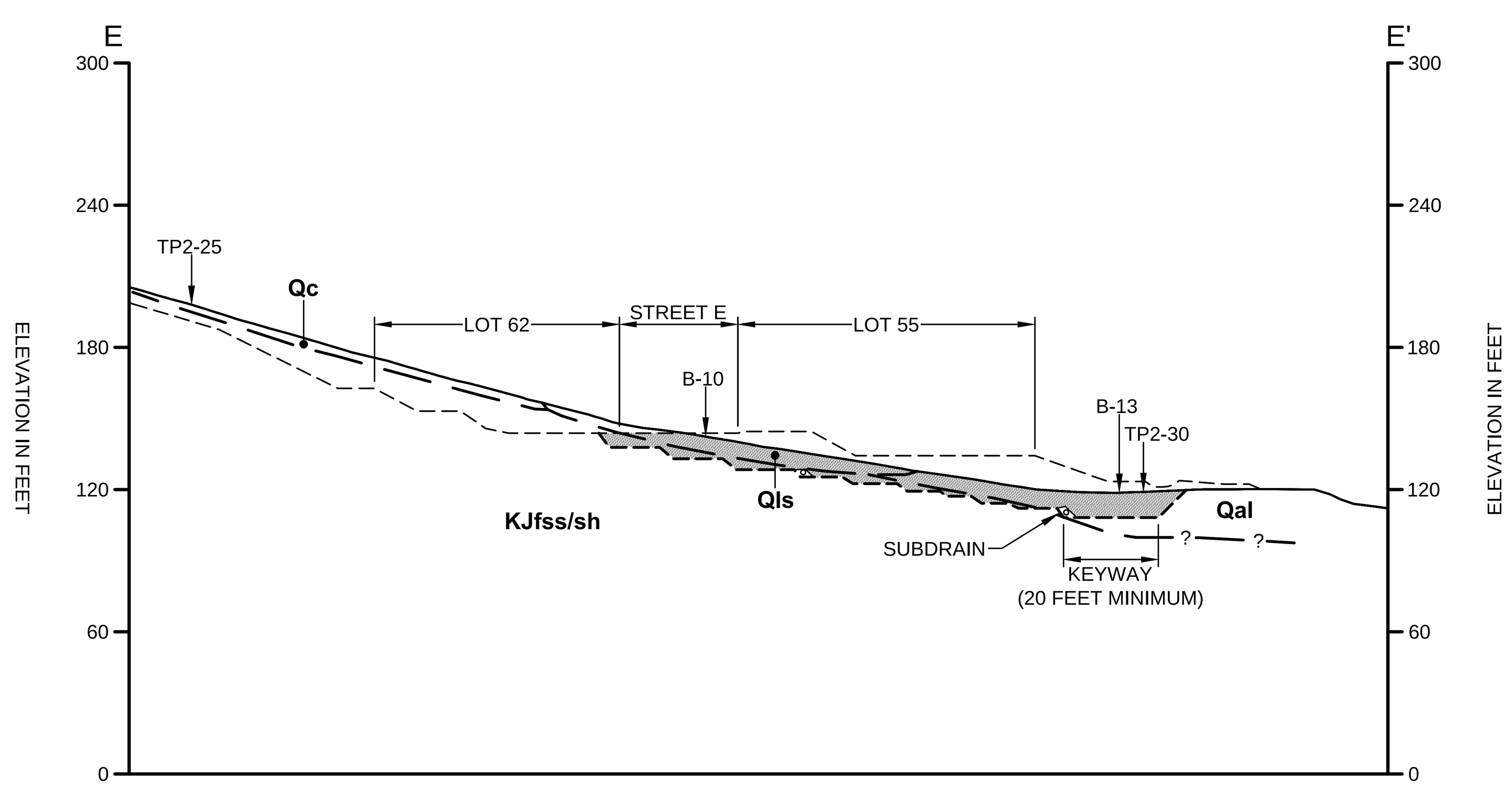
0 60
1"=60'











DRAWN BY: CC
DATE: 5-7-15
JOB NUMBER: 2616.006

EXPLANATION

	EXISTING GRADE
	PROPOSED GRADE (OPTIONS A AND B)
?	GEOLOGIC CONTACT, DASHED WHERE APPROXIMATE, QUERIED WHERE UNCERTAIN
	REMOVE AND REPLACE WITH ENGINEERED FILL
B-14	BORING LOCATION (BGC, 2003)
TP2-24	TEST PIT LOCATION (BGC, 2003)
TP-21	TEST PIT LOCATION (BGC, 2002)
Qaf	ARTIFICIAL FILL
Qls	LANDSLIDE DEBRIS
Qal	ALLUVIUM
KJfss/sh	FRANCISCAN COMPLEX SANDSTONE

REMEDIAL CROSS SECTIONS A-A' THROUGH E-E'

SCOTT RANCH

PETALUMA, CALIFORNIA
FOR
DAVIDON HOMES

Berlogar Stevens & Associates
SOIL ENGINEERS * ENGINEERING GEOLOGISTS

PLATE 5

EXPLANATION

PROJECT BOUNDARY

E E' CROSS SECTION LOCATION

KEYWAY MINIMUM 20 FEET WIDE, MINIMUM 6 FEET DEEP, CONSTRUCTED WITH BEDROCK CUT MATERIAL WITH A PI LESS THAN 20

REMOVE AND REPLACE LANDSLIDE DEBRIS WITH ENGINEERED FILL

SUBDRAIN LOCATION (SDR 35 PERFORATED PIPE)

G LANDSLIDE DESIGNATION

REMEDIAL GRADING PLAN

SCOTT RANCH

OPTION A - 66 LOTS

PETALUMA, CALIFORNIA

FOR

DAVIDON HOMES

Berlogar Stevens & Associates

SOIL ENGINEERS * ENGINEERING GEOLOGISTS

BASE: SHEET 5 TITLED, "REVISED PROJECT - OPTION A - 66 LOTS PROPOSED GRADING PLAN", PREPARED BY BKF, DATED 12-11-13

EXPLANATION

PROJECT BOUNDARY

E — E' CROSS SECTION LOCATION

KEYWAY MINIMUM 20 FEET WIDE, MINIMUM 6 FEET DEEP, CONSTRUCTED WITH BEDROCK CUT MATERIAL WITH A PI LESS THAN 20

REMOVE AND REPLACE LANDSLIDE DEBRIS WITH ENGINEERED FILL

SUBDRAIN LOCATION (SDR 35 PERFORATED PIPE)

(G) LANDSLIDE DESIGNATION

REMEDIAL GRADING PLAN

SCOTT RANCH

OPTION B - 63 LOTS

PETALUMA, CALIFORNIA

FOR

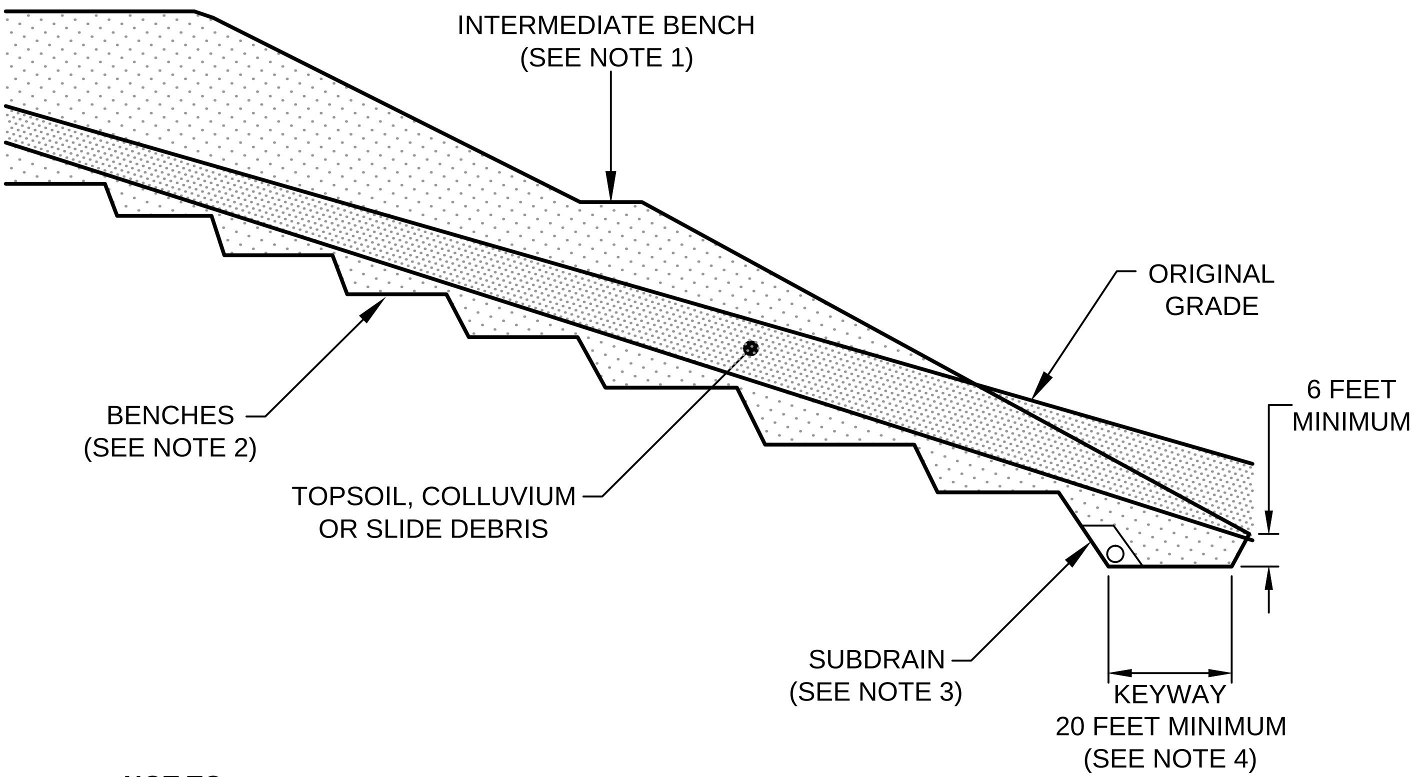
DAVIDON HOMES

Berlogar Stevens & Associates

SOIL ENGINEERS * ENGINEERING GEOLOGISTS

BASE: SHEET 5 TITLED, "REVISED PROJECT - OPTION B - 63 LOTS PROPOSED GRADING PLAN", PREPARED BY BKF, DATED 12-11-13

JOB NUMBER: 2616.606 DATE: 5-12-15 DRAWN BY: CC

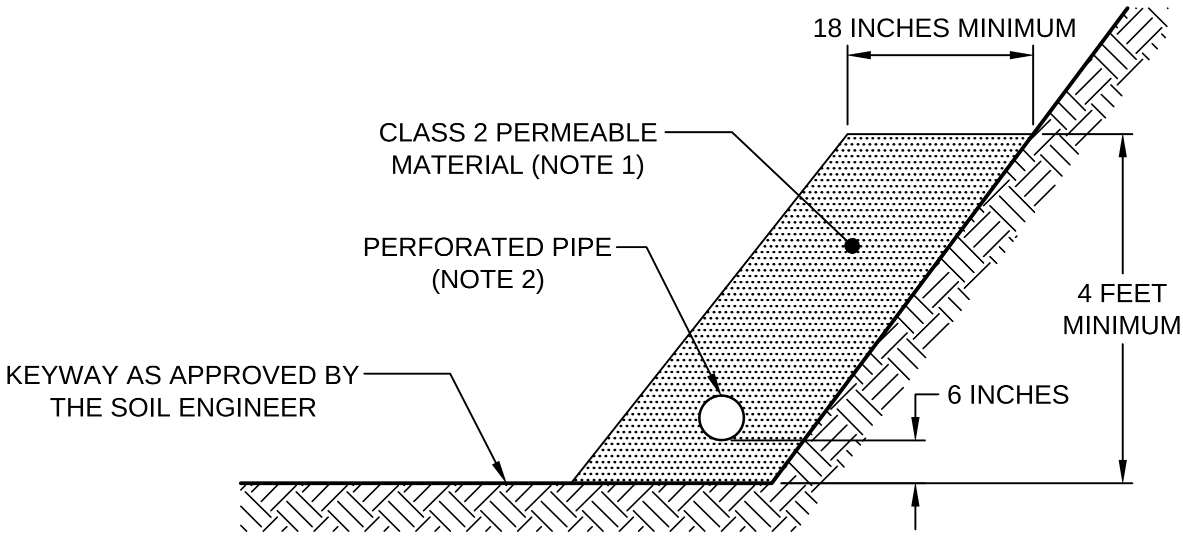


NOT TO SCALE

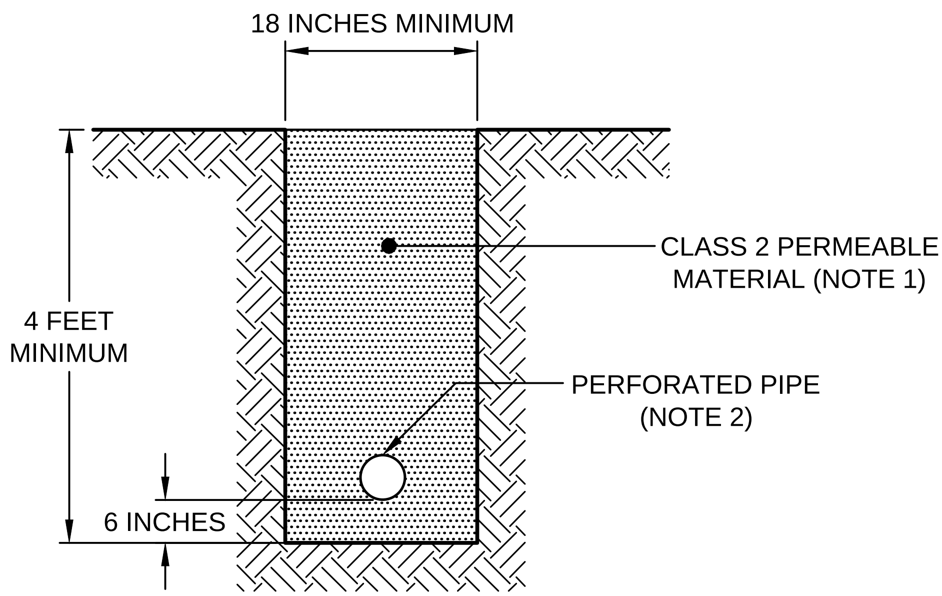
NOTES:

1. INTERMEDIATE BENCHES SHOULD BE SPACED EVERY 25 VERTICAL FEET ON SLOPES HIGHER THAN 30 FEET.
2. WHERE NATURAL GRADE IS STEEPER THAN 7:1, BENCH INTO STIFF SOIL OR BEDROCK AS DETERMINED BY SOIL ENGINEER.
3. SUBDRAIN SHOULD DISCHARGE VIA A CLOSED PIPE TO STORM DRAIN OR SUITABLE NATURAL DRAINAGE.
4. KEYWAY SHOULD EXTEND AT LEAST 6 FEET INTO STIFF SOIL OR BEDROCK AS DETERMINED BY THE SOIL ENGINEER. KEYWAY WIDTH SHOULD BE A MINIMUM OF 20 FEET OR 1/2 OF THE FILL SLOPE HEIGHT, WHICHEVER IS GREATER.

FILL SLOPE DETAIL



KEYWAY SUBDRAIN



NOT TO SCALE

COLLECTOR SUBDRAIN

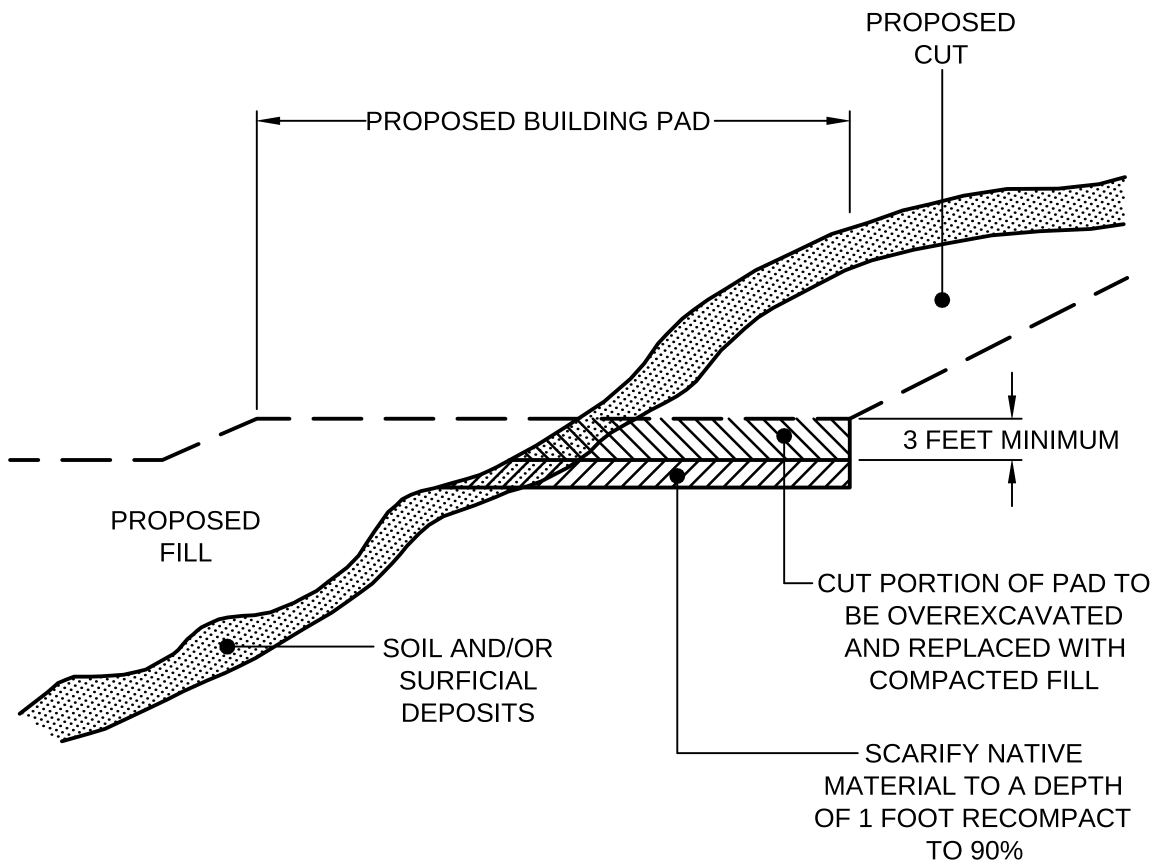
NOTES:

1. CLASS 2 PERMEABLE MATERIAL AS GIVEN IN SECTION 68 - 1.025, STATE OF CALIFORNIA STANDARD SPECIFICATIONS, MAY, 2006 EDITION.

2. PERFORATED PIPE PLACED PERFORATIONS DOWN, PVC PIPE WITH A MINIMUM DIAMETER OF SIX (6) INCHES, CONFORMING TO ASTM D-3034 SDR 35, FOR DEPTHS LESS THAN 30 FEET, AND SDR 23.5 FOR DEPTHS GREATER THAN 30 FEET.

TYPICAL SUBDRAIN DETAILS

JOB NUMBER: 2616.006 DATE: 4-3-14 DRAWN BY: CC



DRAWN BY: CC

DATE: 4-22-14

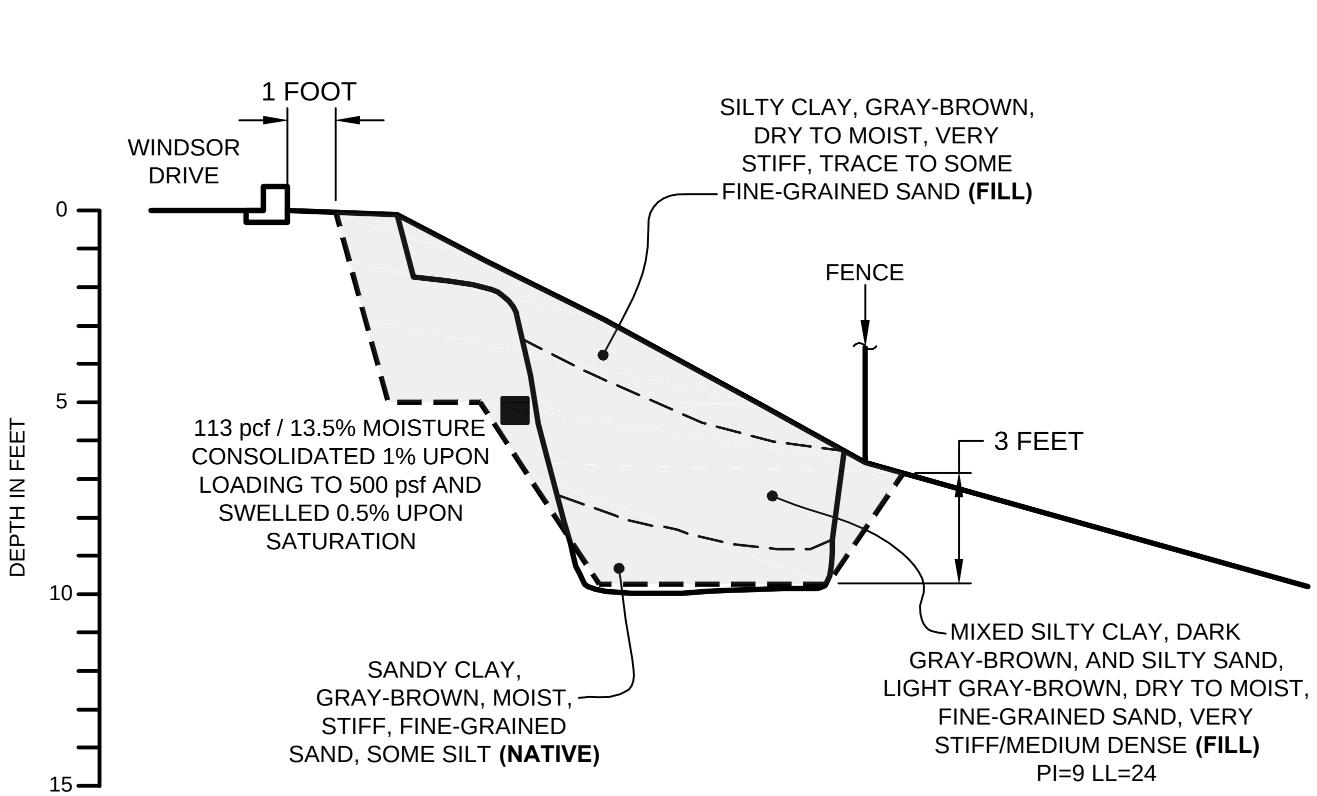
JOB NUMBER: 2616.006

NOT TO SCALE

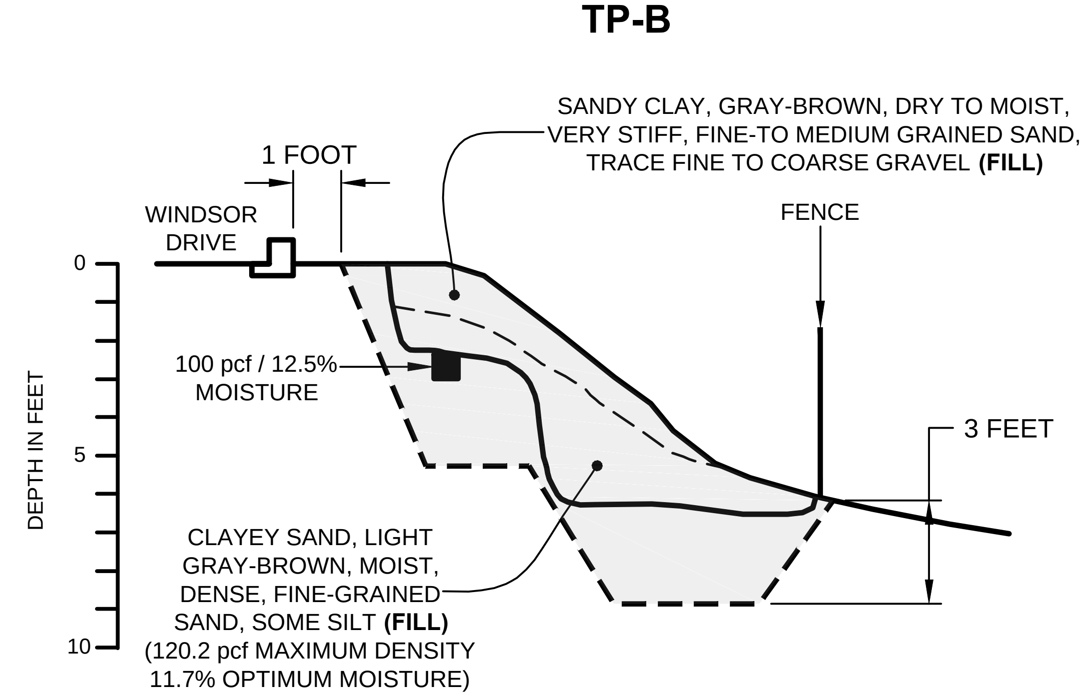
TYPICAL CUT/FILL TRANSITION LOT OVEREXCAVATION DETAIL

DRAWN BY: CC
DATE: 6-12-14
JOB NUMBER: 2616.006

TP-A



TP-B



0 4
1"=4'

EXPLANATION

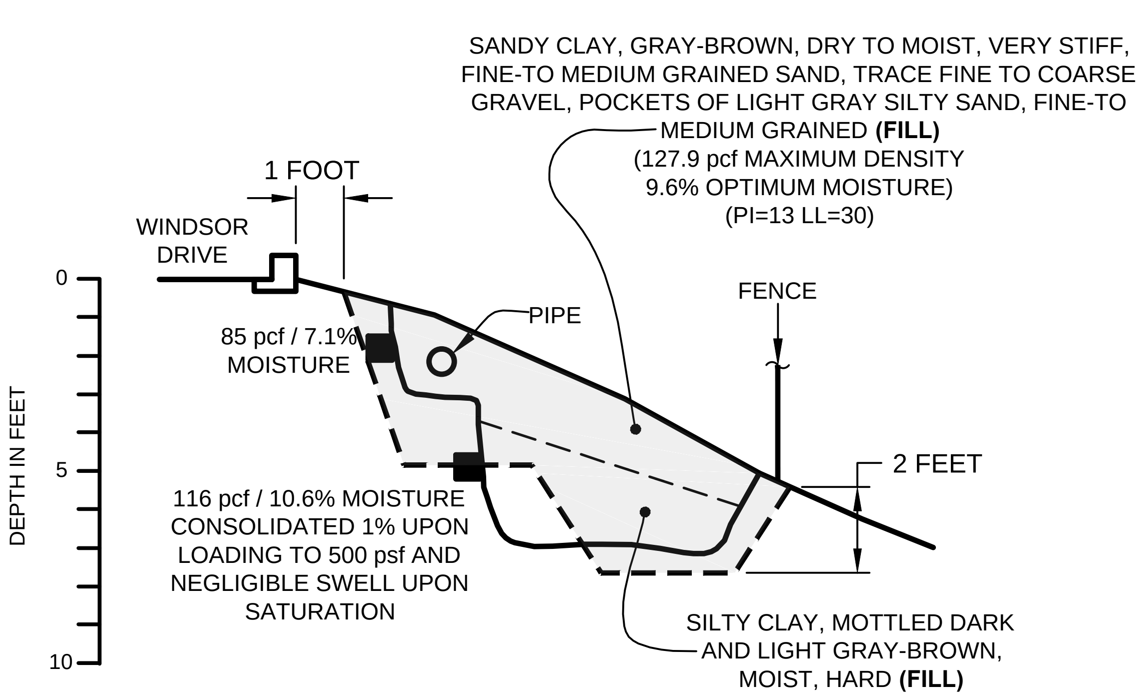
GROUND SURFACE AND TEST PIT LIMITS

GEOLOGIC CONTACT, SOLID WHERE SHARP, DASHED WHERE GRADATIONAL

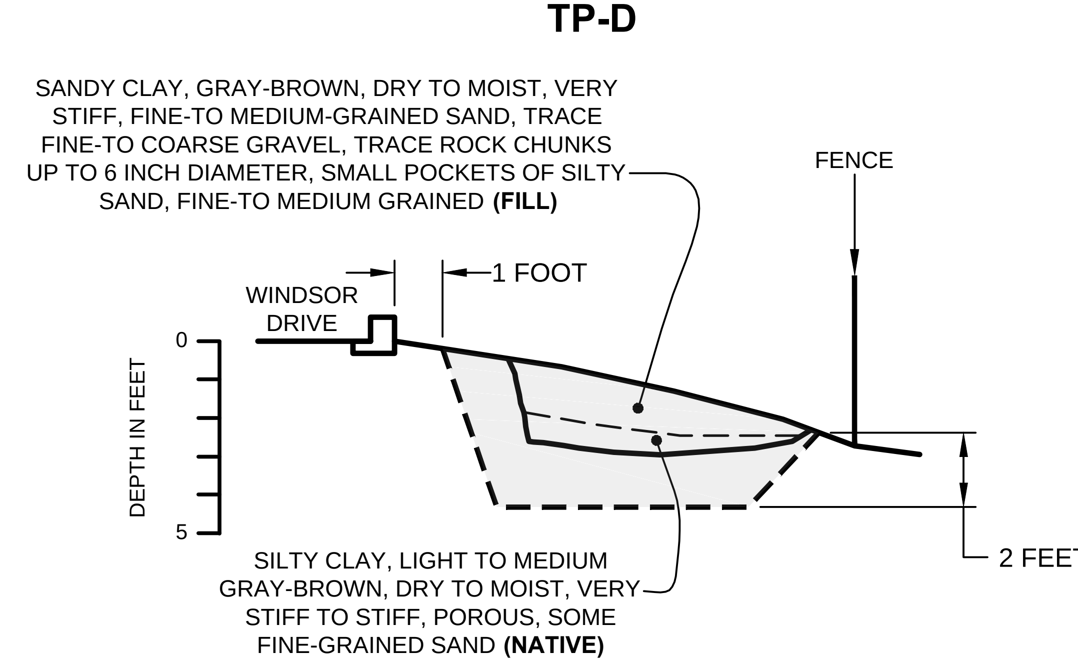
TUBE SAMPLE OF SOIL

FILL SLOPE BENCHING AND RECONSTRUCTION

TP-C



TP-D



GRAPHIC TEST PIT LOGS TP-A THROUGH TP-D

SCOTT RANCH

PETALUMA, CALIFORNIA
FOR
DAVIDON HOMES

Berlogar Stevens & Associates
SOIL ENGINEERS * ENGINEERING GEOLOGISTS

TEST PITS EXCAVATED AND LOGGED ON 6-11-15

APPENDIX A

Field Investigation Data

BORING LOG B-1

JOB NUMBER: 2616.100 **DATE DRILLED:** 3-28-03

JOB NAME: UOP Property **SURFACE ELEVATION:** 151 feet

DRILL RIG: Rotary Wash **DATUM:** Mean Sea Level

SAMPLER TYPE:	DRIVE WEIGHT - LB	HEIGHT OF FALL - IN
2.5 inch I.D. Split Barrel	140	30
Standard Penetration Test	140	30

BLOWS PER FT.	MOISTURE CONTENT %	DRY UNIT WEIGHT p.c.f.	DEPTH IN FEET	USCS CLASSI-FICATION	DESCRIPTION
10	18.9	102		CL	SILTY CLAY, dark gray-brown, moist, medium stiff, trace fine- grained sand
22	19.1	110	5	CL	SILTY CLAY, gray-brown, moist, stiff to very stiff, trace fine-grained sand
25	22.3	103	10	CL	SANDY CLAY, light to medium gray-brown, moist, very stiff, fine to medium-grained sand, some silt
25	20.1	104	15	SC	CLAYEY SAND, gray-brown, wet, medium dense, fine to medium-grained sand, trace silt, trace fine gravel
14	-	-			
22	19.9	108	20	CL	SILTY CLAY, gray-brown, wet, stiff to very stiff, trace fine-grained sand

BORING LOG B-1

JOB NUMBER: 2616.100 **SHEET:** 2 **OF:** 2

JOB NAME: UOP Property **DEPTH:** 20 feet **TO** 24.5 feet

NOTES:

BLOWS PER FT.	MOISTURE CONTENT %	DRY UNIT WEIGHT p.c.f.	DEPTH IN FEET	USCS CLASSI-FICATION	DESCRIPTION
22	19.9	108		CL	SILTY CLAY, gray-brown, wet to saturated, stiff to very stiff, trace to some fine-grained sand
				CL	SANDY CLAY, light gray-brown, saturated, very stiff, fine to medium-grained sand
					SANDSTONE, fine-grained, tan-brown, highly weathered, strong
66	-	-			SHALE, black, slightly weathered, fractured, low hardness,
			25		Boring terminated at 24-1/2 feet. Free water encountered at 21 feet.
			30		
			35		
			40		

CORE LOG

BORING NO.: B-2 JOB NO.: 2616.100
DATE BEGUN: 3-28-03
DATE COMPLETED: 3-28-03
DEPTH OF HOLE: 50-1/2 feet
NUMBER OF CORE BOXES: 7
LOGGED BY: ROV

PROJECT: UOP Property
DRILLING COMPANY: Spectrum
DRILLING METHODS: Rotary Wash
ELEVATION (FEET): 249 feet

RUN NO.	DRILLRATE (MIN/FT.)	CUT	REC	% REC.	% DRILLING FLUID LOSS	RQD (%)
1	1 1.5 2.0 3.0 2.5/ 6"	4.5	4.0	80	0	0
2	1.5 1.0 2.0 3.5 7.0	5.0	4.0	80	0	0
3	2.0 4.5 7.0 6.5 4/6"	4.5	1.1	24	0	0

DEPTH	DESCRIPTION
0	SILTY CLAY, gray-brown, moist, stiff, some fine-grained sand
2	
4	SANDSTONE, fine-grained, light orange-brown, highly weathered, crushed with some clay
6	SANDSTONE, fine-grained, light to medium brown-gray, highly weathered, weak, highly fractured to crushed at 5.6 feet, joint 60° dip
8	from 7.5 to 8.5 feet, light gray-brown, friable zone from 8.5 to 9 feet, clay layer, 50° dip below 9 feet, becomes crushed sandstone
10	at 9.6 feet, joint 60° dip at 11.5 feet, joint 60° dip
12	from 12 to 12.5 feet, abundant calcite veinlets
14	SHALE, dark gray, highly weathered, moderately strong, crushed
16	

CORE LOG

PROJECT UOP Property BORING NUMBER B-2 JOB NUMBER 2616.100

RUN NO.	DRILLRATE (MIN/FT.)	CUT	REC	% REC.	% DRILLING FLUID LOSS	RQD (%)	DEPTH	DESCRIPTION
4	2.5	5.0	4.8	96	0	60		SHALE, dark gray, highly weathered, moderately strong, crushed SANDSTONE, fine-grained, gray, moderately to highly weathered, weak, highly fractured, limonite stains
	2.0							at 16.8 feet, joint 60° dip
	2.0						18	SHALE , dark gray, highly weathered, moderately strong, crushed SANDSTONE, fine-grained, gray, moderately to highly weathered, weak to moderately strong, highly fractured, limonite stains
	2.0							at 19 feet, joint 60° dip limonite stains on fracture surfaces
	2.0						20	
5	2.0	5.0	5.0	100	0	60		at 21.2 feeet, joint 65° dip at 21.8 feet, fracture 70° dip
	1.5						22	
	1.5							
	1.5						24	
	2.0							
6	2.0	5.0	5.0	100	0	60	26	at 26.4 feet, joint 55° dip
	2.0							at 27.2 feet, crushed zone
	2.0						28	
	2.0							at 29.5 feet, thin bedding laminations 55° to 60° dip
	2.0						30	at 30.4 feet, bedding 55° dip SHALE, black, moderately to highly weathered, weak, crushed
7	2.5	5.0	5.0	100	0	10	32	

CORE LOG

PROJECT: UOP Property BORING NUMBER: B-2 JOB NUMBER: 2616.100

RUN NO.	DRILLRATE (MIN/FT.)	CUT	REC	% REC.	% DRILLING FLUID LOSS	RQD (%)
7	2.0 1.5 2.0 2.0	5.0	5.0	100	0	10
8	6.0	0.5	0.5	100	0	0
9	3.0 3.0 2.0 3.0	4.0	3.6	90	0	10
10	3.0 1.5 2.0 2.0 3.0	5.0	5.0	100	0	16
11	3.0 2.5 2.0	5.0	5.0	100	0	14

DEPTH	DESCRIPTION
	SHALE, black, moderately to highly weathered, weak, crushed
34	SANDSTONE, fine-grained, gray, moderately weathered, weak to moderately strong, highly fractured, thickly bedded with shale, black, highly weathered, weak, curshed
	at 35.5 feet, 2-inch thick clay seam, 40° dip
36	from 36.8 to 37 feet, shale layer, bedding 50° dip
38	
40	
42	SANDSTONE, fine-grained, gray, slightly weathered, moderately strong at 42.2 feet, bedding 50° dip
44	SHALE, black, highly weathered, weak, crushed, faintly sheared
46	
48	SANDSTONE, fine-grained, gray, moderately weathered, strong, highly fractured

CORE LOG

PROJECT UOP Property BORING NUMBER B2 JOB NUMBER 2616.100

RUN NO.	DRILLRATE (MIN/FT.)	CUT	REC	% REC.	% DRILLING FLUID LOSS	RQD (%)	DEPTH	LOG	DESCRIPTION
11	2.0 2.0 2.5	5.0	5.0	100	0	14	50		SANDSTONE, fine-grained, gray, moderately weathered, strong, highly fractured
							52 54 56 58 60 62 64		Boring terminated at 50-1/2 feet.

BORING LOG B-3

JOB NUMBER: 2616.100 **DATE DRILLED:** 4-2-03

JOB NAME: UOP Property **SURFACE ELEVATION:** 118 feet

DRILL RIG: Solid Flight Auger **DATUM:** Mean Sea Level

SAMPLER TYPE:	DRIVE WEIGHT - LB	HEIGHT OF FALL - IN
■ 2.5 inch I.D. Split Barrel	140	30
▨ Standard Penetration Test	140	30

BLOWS PER FT.	MOISTURE CONTENT %	DRY UNIT WEIGHT p.c.f.	DEPTH IN FEET	USCS CLASSI-FICATION	DESCRIPTION
20	20.0	101		ML	SANDY SILT, brown, dry to moist, medium dense, fine-grained sand, trace to some clay, rootlets
				CL	SILTY CLAY, dark brown, moist, stiff, trace fine-grained sand, trace well rounded gravel up to 1/4-inch diameter, faint iron oxide mottling
33	17.4	106	5	CL	SILTY CLAY, dark yellow-brown, moist, stiff, some fine-grained sand, faint iron oxide mottling
				CL	SANDY CLAY, dark yellow-brown, moist, very stiff, medium-grained sand, trace to some well rounded gravel up to 1/8-inch diameter
43	19.8	106	10	SC	CLAYEY SAND, mottled orange-brown and gray, moist,dense, fine to medium-grained sand, trace well rounded gravel up to 1/4-inch diameter
50/6"	15.1	92	15		SHALE, gray, highly weathered, weak, crushed, thinly laminated at 65° to 70°.
			20		Boring terminated at 15 feet. No free water encountered.

BORING LOG B-4

JOB NUMBER: 2616.100 **DATE DRILLED:** 4-2-03

JOB NAME: UOP Property **SURFACE ELEVATION:** 137 feet

DRILL RIG: Solid Flight Auger **DATUM:** Mean Sea Level

SAMPLER TYPE:	DRIVE WEIGHT - LB	HEIGHT OF FALL - IN
2.5 inch I.D. Split Barrel	140	30
Standard Penetration Test	140	30

BLOWS PER FT.	MOISTURE CONTENT %	DRY UNIT WEIGHT p.c.f.	DEPTH IN FEET	USCS CLASSI-FICATION	DESCRIPTION
				ML	SANDY SILT, brown, moist, medium dense, fine-grained sand
57	19.5	102		SM	SILTY SAND, orange-brown, moist, dense to very dense, medium-grained sand, trace clay
50/3"	-	-			
50/4"	10.4	107	5		SANDSTONE, fine-grained, tan-brown, highly weathered, weak to moderately strong, moderately fractured with manganese oxide on surfaces
60/6"	-	-			
			10		Boring terminated at 8 feet. No free water encountered.
			15		
			20		

BORING LOG B-5

JOB NUMBER: 2616.100 **DATE DRILLED:** 4-2-03

JOB NAME: UOP Proeprty **SURFACE ELEVATION:** 185 feet

DRILL RIG: Solid Flight Auger **DATUM:** Mean Sea Level

SAMPLER TYPE:	DRIVE WEIGHT - LB	HEIGHT OF FALL - IN
■ 2.5 inch I.D. Split Barrel	140	30
▨ Standard Penetration Test	140	30

BLOWS PER FT.	MOISTURE CONTENT %	DRY UNIT WEIGHT p.c.f.	DEPTH IN FEET	USCS CLASSI-FICATION	DESCRIPTION
13	12.8	105		CL	SILTY CLAY, mixed brown and gray-brown, moist, medium stiff to stiff, trace fine to coarse-grained sand, trace cobbles (fill)
32	15.7	108	5	CL	SILTY CLAY, dark gray-brown to brown, moist, stiff, trace gravel, trace fine-grained sand (fill)
25	18.7	108	10		
22	17.1	110	15	CL	SILTY CLAY, dark brown, moist, stiff, some medium-grained sand, trace subrounded gravel up to 1/8-inch diameter
36	16.7	113	20	CL	SILTY CLAY, brown to dark yellow-brown, stiff, trace fine-grained sand, trace subrounded gravel up to 1/2-inch diameter , 1/16-inch thick gray clay films

BORING LOG B-5

JOB NUMBER: 2616.100 **SHEET:** 2 **OF:** 2

JOB NAME: UOP Property **DEPTH:** 20 feet **TO** 30-1/4 feet

NOTES:

BLOWS PER FT.	MOISTURE CONTENT %	DRY UNIT WEIGHT p.c.f.	DEPTH IN FEET	USCS CLASSI-FICATION	DESCRIPTION
36	16.7	113		CL	SILTY CLAY, brown to dark yellow-brown, moist, stiff, trace fine-grained sand, trace subrounded gravel up to 1/2-inch diameter, 1/16-inch gray clay films
25	17.5	116	25	SC / CL	SANDY CLAY / CLAYEY SAND, yellow-brown-brown, moist, stiff to medium dense, medium-grained sand
65/6"	-	-			SHALE, gray, highly weathered, weak, highly fractured from 65° to 70° .
50/ 2.5"	-	-	30		
			35		Boring terminated at 30-1/4 feet. Free water encountered at 20 feet, rose to 17 feet in 4 hours.
			40		

BORING LOG B-6

JOB NUMBER: 2616.100 **DATE DRILLED:** 4-2-03

JOB NAME: UOP Property **SURFACE ELEVATION:** 157 feet

DRILL RIG: Solid Flight Auger **DATUM:** Mean Sea Level

SAMPLER TYPE:	DRIVE WEIGHT - LB	HEIGHT OF FALL - IN
■ 2.5 inch I.D. Split Barrel	140	30
⧄ Standard Penetration Test	140	30

BLOWS PER FT.	MOISTURE CONTENT %	DRY UNIT WEIGHT p.c.f.	DEPTH IN FEET	USCS CLASSI-FICATION	DESCRIPTION
				ML	CLAYEY SILT, brown, moist, stiff, trace fine-grained sand
13	14.6	115		SC	CLAYEY SAND, gray-brown, moist, medium dense, medium-grained sand
17	-	-	5	CL	SANDY CLAY, mottled gray-brown and brown, moist, stiff, medium-grained sand, trace well rounded gravel up to 1/8-inch diameter, iron oxide stains
39	15.9	111		SC	CLAYEY SAND, mottled light gray and orange-brown, moist, dense, medium to coarse-grained sand, trace well rounded to subrounded gravel up to 1/8-inch diameter
74	14.4	108	10	SC	CLAYEY SAND AND SILT, mottled orange-brown and gray, moist, very dense, coarse-grained sand, weakly cemented
50/6"	8.5	129	15		SHALE, gray to black, highly weathered, weak to moderately strong, crushed, 40° joints possible bedding
60/2"	-	-			
			20		Boring terminated at 19-1/2 feet. No free water encountered.

BORING LOG B-7

JOB NUMBER: 2616.100 **DATE DRILLED:** 4-2-03

JOB NAME: UOP Property **SURFACE ELEVATION:** 144 feet

DRILL RIG: Solid Flight Auger **DATUM:** Mean Sea Level

SAMPLER TYPE:	DRIVE WEIGHT - LB	HEIGHT OF FALL - IN
■ 2.5 inch I.D. Split Barrel	140	30
▱ Standard Penetration Test	140	30

BLOWS PER FT.	MOISTURE CONTENT %	DRY UNIT WEIGHT p.c.f.	DEPTH IN FEET	USCS CLASSI-FICATION	DESCRIPTION
				ML	SANDY SILT, brown, moist, medium stiff
7	18.2	107			
				SC	CLAYEY SAND, gray, wet, medium dense, coarse-grained sand
22	-	-		CL	SILTY CLAY, mottled tan-brown and gray-brown, moist, very stiff, trace fine-grained sand, trace well rounded gravel up to 1/4-inch diameter
38	15.2	112	5	CL	SANDY CLAY, mottled gray-brown and gray, moist, very stiff, fine to medium-grained sand, trace subrounded gravel up to 1/4-inch diameter
				SM	SILTY SAND, gray, saturated, medium dense, some clay
40	15.5	111	10	SC	CLAYEY SAND, mottled orange-brown and gray, moist, very dense, coarse-grained sand, trace subrounded gravel up to 1/4-inch diameter
60/2"	-	-			SANDSTONE, fine to medium-grained, tan-brown, highly weathered, weak, highly fractured
			15		Boring terminated at 14-1/6 feet. Free water encountered at 7 feet.
			20		

BORING LOG B-8

JOB NUMBER: 2616.100 **DATE DRILLED:** 4-2-03

JOB NAME: UOP Property **SURFACE ELEVATION:** 134 feet

DRILL RIG: Solid Flight Auger **DATUM:** Mean Sea Level

SAMPLER TYPE:	DRIVE WEIGHT - LB	HEIGHT OF FALL - IN
2.5 inch I.D. Split Barrel	140	30
Standard Penetration Test	140	30

BLOWS PER FT.	MOISTURE CONTENT %	DRY UNIT WEIGHT p.c.f.	DEPTH IN FEET	USCS CLASSI-FICATION	DESCRIPTION
				CL	SANDY CLAY, brown, moist, stiff, fine to medium-grained sand
35	16.2	114		SC	CLAYEY SAND, mottled orange-brown and gray, moist, dense, coarse-grained sand
				CL	SANDY CLAY, brown, moist, stiff to very stiff, fine-grained sand, iron oxide stains
50/6"	10.5	108	5	SC	CLAYEY SAND, tan-brown, dry to moist, very dense, fine to medium-grained sand, trace subrounded gravel up to 1/4-inch diameter, weakly cemented
				SC	CLAYEY SAND, mottled orange-brown and gray, moist, very dense, fine to coarse-grained sand, trace subrounded gravel up to 1/4-inch diameter
65	17.9	104		SP	SAND, orange-brown, moist, very dense, coarse-grained sand
18	-	-	10	SC	CLAYEY SAND, mottled orange-brown and gray, moist, medium dense, fine-grained sand
15	20.8	105	15	SC	CLAYEY SAND, orange-brown and gray, saturated, medium dense, coarse-grained sand
13	-	-			
50/6"	-	-			SANDSTONE, fine-grained, tan-brown, highly weathered, moderately strong, crushed
			20		Boring terminated at 19-1/2 feeet. Free water encountered at 16 feet, rose to 14-1/2 feet in 1 hour.

BORING LOG B-9

JOB NUMBER: 2616.100 **DATE DRILLED:** 4-3-03

JOB NAME: UOP Property **SURFACE ELEVATION:** 140 feet

DRILL RIG: Solid Flight Auger **DATUM:** Mean Sea Level

SAMPLER TYPE:	DRIVE WEIGHT - LB	HEIGHT OF FALL - IN
2.5 inch I.D. Split Barrel	140	30
Standard Penetration Test	140	30

BLOWS PER FT.	MOISTURE CONTENT %	DRY UNIT WEIGHT p.c.f.	DEPTH IN FEET	USCS CLASSI-FICATION	DESCRIPTION
				ML	CLAYEY SILT, gray-brown, moist, stiff
32	17.4	110		CL	SILTY CLAY, dark yellow-brown to brown, moist, stiff, trace well-rounded gravel up to 1/8-inch diameter
36	21.0	105	5	CL	SILTY CLAY, yellow-brown, moist, stiff, trace to some fine to medium-grained sand, trace subrounded to subangular gravel up to 3/4-inch diameter, faint iron oxide stains
				CL	SILTY CLAY, tan-brown, moist, stiff, trace to some fine-grained sand, trace subrounded gravel up to 1/4-inch diameter
58	16.3	110	10	CL	SANDY CLAY, mottled orange-brown and gray, moist, very stiff, medium-grained sand, trace to some subangular gravel up to 2-inch diameter
60/6"	-	-			SANDSTONE, medium to coarse-grained, green-gray to black, highly weathered, strong, crushed, 60° joints.
			15		Boring terminated at 14 feet. No free water encountered.
			20		

BORING LOG B-10

JOB NUMBER: 2616.100 **DATE DRILLED:** 4-3-03

JOB NAME: UOP Property **SURFACE ELEVATION:** 142 feet

DRILL RIG: Solid Flight Auger **DATUM:** Mean Sea Level

SAMPLER TYPE:	DRIVE WEIGHT - LB	HEIGHT OF FALL - IN
2.5 inch I.D. Split Barrel	140	30
Standard Penetration Test	140	30

BLOWS PER FT.	MOISTURE CONTENT %	DRY UNIT WEIGHT p.c.f.	DEPTH IN FEET	USCS CLASSI-FICATION	DESCRIPTION
				SM	SANDY SILT, brown, wet, medium dense, fine-grained sand
23	19.6	108		CL	SILTY CLAY, dark yellow-brown, moist, stiff, trace coarse-grained sand, iron oxide stains
37	20.8	102		CL	SILTY CLAY, mottled orange-brown and tan-brown and gray, moist, stiff, some fine-grained sand, trace well rounded gravel up to 1/8-inch diameter, iron oxide stains
			5		SANDSTONE, fine to medium-grained, tan-brown, highly weathered, weak to moderately strong, highly fractured
50/6"	-	-			SHALE, black, highly weathered, strong, crushed
			10		Boring terminated at 9-1/2 feet. No free water encountered.
			15		
			20		

BORING LOG B-11

JOB NUMBER: 2616.100 **DATE DRILLED:** 4-3-03

JOB NAME: UOP Property **SURFACE ELEVATION:** 133 feet

DRILL RIG: Solid Flight Auger **DATUM:** Mean Sea Level

SAMPLER TYPE:	DRIVE WEIGHT - LB	HEIGHT OF FALL - IN
■ 2.5 inch I.D. Split Barrel	140	30
▱ Standard Penetration Test	140	30

BLOWS PER FT.	MOISTURE CONTENT %	DRY UNIT WEIGHT p.c.f.	DEPTH IN FEET	USCS CLASSI-FICATION	DESCRIPTION
				CL	SANDY CLAY, moist, stiff, fine-grained sand
41	19.2	108		CL	SILTY CLAY, mottled tan-brown and gray, moist, stiff, trace fine-grained sand
				CL	SILTY CLAY, mottled brown and orange-brown, moist, very stiff to hard, trace coarse-grained sand, trace subrounded gravel up to 1/4-inch diameter
					at 4-1/2 feet, approximately 6-inch diameter sandstone cobble
50/6"	-	-	5		
				CL	SANDY CLAY, mottled orange-brown and gray, moist, hard, fine-grained sand
60	-	-			
			10		
50/6"	-	-			SHALE, gray to orange-brown, highly weathered, moterately strong to strong, trace clay
65/6"	-	-			
			15		Broing terminated at 14 feet. No free water encountered.
			20		

BORING LOG B-12

JOB NUMBER: 2616.100 **DATE DRILLED:** 4-3-03

JOB NAME: UOP Property **SURFACE ELEVATION:** 124 feet

DRILL RIG: Solid Flight Auger **DATUM:** Mean Sea Level

SAMPLER TYPE:	DRIVE WEIGHT - LB	HEIGHT OF FALL - IN
■ 2.5 inch I.D. Split Barrel	140	30
▨ Standard Penetration Test	140	30

BLOWS PER FT.	MOISTURE CONTENT %	DRY UNIT WEIGHT p.c.f.	DEPTH IN FEET	USCS CLASSI-FICATION	DESCRIPTION
23	19.2	109		CL	SANDY CLAY, dark yellow-brown, moist, stiff, fine-grained, some silt, iron oxide stains
42	16.7	113	5	CL	SILTY CLAY, dark yellow-brown to brown, moist, very stiff, trace to some subangular gravel up to 1/2-inch diameter, trace fine to medium-grained sand
42	16.7	109	10		
45	18.5	110	15	CL	SANDY CLAY, mottled orange-brown and gray-brown, moist, hard, medium-grained sand, trace subangular to subrounded gravel up to 1/4-inch diameter
65/6"	-	-			SHALE, gray, highly weathered, strong, 70° fractures.
			20		Boring terminated at 18-1/2 feet. Free water encountered at 17-1/2 feet.

BORING LOG B-13

JOB NUMBER: 2616.100 **DATE DRILLED:** 4-3-03

JOB NAME: UOP Property **SURFACE ELEVATION:** 123 feet

DRILL RIG: Solid Flight Auger **DATUM:** Mean Sea Level

SAMPLER TYPE:	DRIVE WEIGHT - LB	HEIGHT OF FALL - IN
2.5 inch I.D. Split Barrel	140	30
Standard Penetration Test	140	30

BLOWS PER FT.	MOISTURE CONTENT %	DRY UNIT WEIGHT p.c.f.	DEPTH IN FEET	USCS CLASSIFICATION	DESCRIPTION
				SM	SANDY SILT, brown, dry, medium dense, fine-grained sand
29	17.9	104		CL	SILTY CLAY, dark gray-brown, moist, stiff, trace coarse-grained sand, iron oxide stains
				CL	SANDY CLAY, tan-brown to brown, moist, stiff, coarse-grained sand
28	18.7	104	5	SC/ CL	CLAYEY SAND / SANDY CLAY, tan-brown to brown, moist, medium dense to stiff, fine to medium-grained sand, trace well rounded gravel up to 1/8-inch diameter, iron oxide stains
30	17.1	108	10	SC	CLAYEY SAND, mottled orange-brown and gray, moist, medium dense, medium-grained sand, trace subrounded to well rounded gravel up to 1/4-inch diameter
15	-	-			
20	20.1	107.3	15	SM	SILTY SAND, mottled gray and orange-grown, moist, medium dense, coarse-grained sand, some clay
17	-	-			
50/6"	11.1	125	20		SANDSTONE, fine-grained, gray, highly weathered, highly fractured, strong, 60° joints

BORING LOG B-13

JOB NUMBER: 2616.100 **SHEET:** 2 **OF:** 2

JOB NAME: UOP Property **DEPTH:** 20 feet **TO** 21 feet

NOTES:

BLOWS PER FT.	MOISTURE CONTENT %	DRY UNIT WEIGHT p.c.f.	DEPTH IN FEET	USCS CLASSI-FICATION	DESCRIPTION
50/6"	11.1	125			SANDSTONE, fine-grained, gray, highly weathered, highly fractured, strong, 60° joints
			25		Boring terminated at 21 feet. Free water encountered at 9-1/2 feet, dropped to 17 feet in 2 hours
			30		
			35		
			40		

BORING LOG B-14

JOB NUMBER: 2616.100 **DATE DRILLED:** 4-3-03

JOB NAME: UOP Property **SURFACE ELEVATION:** 170 feet

DRILL RIG: Solid Flight Auger **DATUM:** Mean Sea Level

SAMPLER TYPE: ■ 2.5 inch I.D. Split Barrel **DRIVE WEIGHT - LB** 140 **HEIGHT OF FALL - IN** 30

BLOWS PER FT.	MOISTURE CONTENT %	DRY UNIT WEIGHT p.c.f.	DEPTH IN FEET	USCS CLASSI-FICATION	DESCRIPTION
19	19.8	100		CL	SILTY CLAY, dark, gray-brown, moist, stiff, trace subrounded gravel up to 1/4-inch diameter
20	19.6	107	5	CL	SILTY CLAY, dark brown, moist, stiff, trace well rounded gravel up to 1/4-inch diameter, trace medium to coarse-grained sand
30	14.4	117	10	CL	SILTY CLAY, yellow-brown, moist, stiff to very stiff, some coarse-grained sand, trace subrounded gravel up to 1/2-inch diameter
50/6"	-	-	15		at 15 feet, sandstone boulder approximately18 inches in diameter
50/3"	-	-			SANDSTONE, fine-grained, gray, highly weathered, strong, highly fractured
			20		Boring terminated at 19-1/2 feet. No free water encountered.

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

UNIFIED SOIL CLASSIFICATION SYSTEM

Blows per ft.	Moisture Content (%)	Dry Unit Weight (pcf)	Depth in Feet	USCS Classification	
					Bulk Sample
					2.5" I.D. Split Barrel Sample
					2.8" I.D. Shelby Tube Sample
					No sample recovered
					Standard Penetration Test interval
					Well defined stratum change
					Gradual stratum change
					Interpreted stratum change
					Apparent ground water level at date noted. Seasonal weather conditions, site topography, etc., may cause changes in water level indicated on logs.

Note: Soils described as dry, moist, and wet are estimated to be dry of optimum, near optimum, and wet of optimum moisture content, respectively. Saturated soils are estimated to be within areas of free groundwater.

KEY TO BORING LOG SYMBOLS

BY: FF
DATE: 4-23-03
JOB NUMBER: 2616.100

ROCK DESCRIPTION

ROCK TYPE
GRAIN SIZE (if Applicable)
COLOR
WEATHERING

Highly - Moderate to complete mineral decomposition, extensive disintegration, deep and through discoloration, fractures extensively coated or filled with oxides, carbonates and/or silt and clay.

Moderately - Slight change or partial decomposition of minerals, little disintegration, cementation little to unaffected, moderate to occasionally intense discoloration, moderately coated fractures.

Slightly - No megascopic decompositon of minerals, little to no effect on cementation, slight and intermittent or localized discoloration, few stains on fracture surfaces.

Unweathered - Unaffected by weathering agents, no discoloration or disintegration.

STRENGTH

Friable - Crumbles easily with fingers

Waek - Crumbles under light hammer blows

Moderately Strong - Specimen will withstand a few hammer blows before breaking

Strong - Specimen will withstand a few eavy ringing hammer blows before breaking into large fragments

Very Strong - Specimen will resist heavy ringing hammer blows and will yield with difficulty only dust and small flying fragments

FRACTURING - Intensity, coating or filling, attitude(s)

Intensity	Size of Pieces
Occasionally Fractured	Greater than 12 inches
Moderately Fractured	6 inches to 12 inches
Highly Fractured	1/2 inch to 6 inches
Crushed	Less than 1/2 inch

BEDING - Stratification, Attitude

Stratification	Thickness
Very Thickly Bedded	Greater than 4 feet
Thickly Bedded	2 to 4 feet
Thinly Bedded	1 inch to 2 feet
Thinly Laminated	Less than 1 inch

MISCELLANEOUS - Shearing of rock, veins, caliche, etc.

Source: Modified from Civil Engineers Reference Book (Blake, 1975)

JOB NUMBER: 2616.100 DATE: 4-23-03 BY: FF

TEST PIT LOGS

Test Pit Number	Depth (feet)	Description
TP2-1	0-1	Sandy Clay, light brown, moist, stiff, fine-grained sand, some silt, rootlets.
	1-3	Silty Sand, orange-brown, moist, very dense, coarse-grained sand, some clay.
	3-6	Sandstone, coarse-grained, tan-brown, highly weathered, moderately strong, highly fractured with manganese oxide on surfaces.
		Total Depth 6 feet No free ground water encountered
TP2-2	0-2	Sandy Clay, light brown, moist, stiff, fine-grained sand, some silt, rootlets.
	2-6	Sandstone, medium-grained, gray and red-brown, moderately weathered, very strong, highly fractured. Joints N30W 40S.
		Total Depth 6 feet No free ground water encountered
TP2-3	0-3	Sandy Clay, light brown, moist, stiff, fine-grained sand.
	3-5	Clayey Sand, mottled orange-brown and gray, moist, dense to very dense, medium-grained sand, trace subangular sandstone clasts.
	5-7	Sandstone, coarse-grained, orange-brown, moderately weathered, strong to very strong, highly fractured to crushed.
		Total Depth 7 feet No free ground water encountered
TP2-4	0-3½	Sandstone, fine- to medium-grained, tan-brown, moderately weathered, very strong, highly fractured. Joints N10E 45N, N45E vertical.
		Total Depth 3½ feet No free ground water encountered

TEST PIT LOGS

Test Pit Number	Depth (feet)	Description
TP2-5	0-3½	Silty Clay, dark gray-brown, moist, stiff, trace fine-grained sand, subrounded sandstone cobbles from 2 to 3 feet.
	3½-4½	Clayey Sand, orange-brown, moist, dense, trace subangular sandstone clasts.
	4½-6½	Sandstone, coarse-grained, orange-brown, highly weathered, strong, crushed.
		Total Depth 6½ feet No free ground water encountered
TP2-6	0-2	Clayey Silt, brown, moist, stiff, trace fine-grained sand.
	2-6	Sandy Clay, mottled brown and orange-brown, moist, very stiff, some well-rounded gravel, coarse-grained sand.
		Total Depth 6 feet No free ground water encountered
TP2-7	0-2	Sandy Clay, light brown, moist, stiff, fine-grained sand.
	2-2¼	Sandy Clay, light brown, saturated, medium stiff, fine-grained sand.
	2¼-4	Sandy Clay, mottled brown and orange-brown, moist, very stiff, coarse-grained sand, trace to some well rounded gravel.
		Total Depth 4 feet Ground water encountered at 2 feet
TP2-8	0-4	Sandy Clay, light brown, moist, stiff, fine-grained sand.
	4-6	Sandy Clay, orange-brown, moist, very stiff, trace angular sandstone clasts.
	6-8	Sandstone, coarse-grained, orange-brown, highly weathered, strong, highly fractured. Joints N30W 60SW.
		Total Depth 8 feet No free ground water encountered

TEST PIT LOGS

Test Pit Number	Depth (feet)	Description
TP2-9	0-3	Silty Clay, brown, moist, stiff, trace fine-grained sand.
	3-6	Sandy Clay, mottled brown and orange-brown, moist, stiff to very stiff, medium grained sand, trace well-rounded gravel.
	6-8	Shale, gray, highly weathered, weak to moderately strong, crushed.
		Total Depth 8 feet No free ground water encountered
TP2-10	0-½	Sandy Silt, brown, moist, stiff, fine-grained sand.
	½-5	Sandstone, coarse-grained, orange-brown, highly weathered, strong, highly fractured. Joints N25E 65N, N70W 30S.
		Total Depth 5 feet No free ground water encountered
TP2-11	0-3½	Silty Clay, dark brown, moist, stiff, trace fine-grained sand.
	3½-7	Shale, gray to orange-brown, highly weathered, weak to moderately strong, highly fractured, thinly laminated. Bedding N20W 73SW.
		Total Depth 7 feet No free ground water encountered
TP2-12	0-2½	Silty Clay, dark brown, moist, stiff, trace gravel.
	2½-7	Shale, gray, highly weathered, weak, crushed, some clay.
		Total Depth 7 feet No free ground water encountered
TP2-13	0-3	Silty Clay, dark brown, moist, stiff.
	3-6	Silty Clay, brown, moist, stiff, trace subangular gravel.
	6-9	Shale, gray, highly weathered, crushed, some clay.
		Total Depth 9 feet No free ground water encountered

TEST PIT LOGS

Test Pit Number	Depth (feet)	Description
TP2-14	0-3	Silty Clay, dark brown, moist, stiff.
	3-8	Sandstone, fine-grained, gray, highly weathered, crushed, moderately strong. Joints N70W 70N, N30W 85SW.
		Total Depth 8 feet No free ground water encountered
TP2-15	0-4	Silty Clay, dark brown, moist, stiff, trace gravel.
	4-8	Sandstone, fine-grained, gray, highly weathered, moderately strong, crushed. Possible bedding N24W 73SW. Joint N85W 20N.
		Total Depth 8 feet No free ground water encountered
TP2-16	0-4	Silty Clay, dark gray-brown, moist, stiff.
	4-6	Silty Clay, brown, moist, stiff, trace coarse-grained sand, trace subangular gravel up to ½-inch diameter, fairly sharp basal contact with faint horizontal clay film.
	6-13	Clayey Sand/Sandy Clay, orange-brown, moist, very dense, coarse-grained sand, trace well-rounded gravel up to ½-inch diameter.
	13-15	Shale, gray, highly weathered, weak to moderately strong, crushed.
		Total Depth 15 feet No free ground water encountered
TP2-17	0-3½	Silty Clay, dark gray-brown, moist, stiff.
	3½-5	Silty Clay, brown, moist, stiff, trace subangular gravel up to ½-inch diameter, sharp basal contact with 1/16-inch clay. N20E 12SE.
	5-8	Clayey Sand, orange-brown, moist, very dense, coarse-grained sand, trace well-rounded gravel up to ½-inch diameter.
		Total Depth 8 feet No free ground water encountered

TEST PIT LOGS

Test Pit Number	Depth (feet)	Description
TP2-18	0-3½	Silty Clay, dark gray-brown, moist, stiff, trace fine-grained sand.
	3½-7	Sandstone, fine-grained, tan-brown, highly weathered, strong, crushed.
		Total Depth 7 feet No free ground water encountered
TP2-19	0-1	Sandy Silt, light brown, dry to moist, fine-grained sand, trace clay.
	1-9	Clayey Sand, mottled orange-brown and gray, moist, very dense, coarse-grained sand.
	9-12	Sandstone, medium-grained, tan-brown, highly weathered, moderately strong, crushed.
		Total Depth 12 feet No free ground water encountered
TP2-20	0-3	Silty Sand, brown, moist, stiff, fine-grained sand, some clay.
	3-9	Silty Sand, orange-brown and gray, moist, very dense, fine-grained sand, trace well-grounded gravel up to ½-inch diameter, trace clay.
	9-14	Clayey Sand, mottled orange-brown and gray, moist to wet, dense to very dense, coarse-grained sand, some subrounded gravel up to 1-inch diameter.
	14-15	Shale, gray, highly weathered, weak, crushed.
		Total Depth 15 feet Free ground water encountered at 15 feet
TP2-21	0-1	Sandy Silt, brown, dry to moist, stiff, fine-grained sand, some clay.
	1-5	Sandstone, fine-grained, tan-brown, highly weathered, strong, highly fractured, bedded with shale, black, highly weathered, weak to moderately strong, crushed with some clay, disrupted structure. Bedding N62W 87S.
		Total Depth 5 feet No free ground water encountered

TEST PIT LOGS

Test Pit Number	Depth (feet)	Description
TP2-22	0-1½	Sandy Clay, brown, moist, stiff, fine-grained sand, some silt.
	1½-5	Shale, gray to orange-brown, highly weathered, weak to moderately strong, crushed with trace to some clay, disrupted structure.
		Total Depth 5 feet No free ground water encountered
TP2-23	0-2	Sandy Clay, brown, moist, stiff, fine-grained sand, faint blocky ped structure.
	2-4	Sandstone, medium-grained, tan-brown, highly weathered, highly fractured, trace clay on fracture surfaces.
		Total Depth 4 feet No free ground water encountered
TP2-24	0-4½	Silty Clay, dark gray-brown, moist, stiff, trace well-rounded gravel up to ¼-inch diameter.
	4½-6½	Silty Clay, dark brown, moist, stiff to very stiff, trace well-rounded gravel up to ¼-inch diameter, trace medium- to coarse-grained sand.
	6½-11½	Silty Clay, brown, moist, stiff to very stiff, some coarse-grained sand, trace subrounded to rounded gravel up to ½-inch diameter.
		Total Depth 11½ feet No free ground water encountered
TP2-25	0-2	Sandy Clay, brown, moist, stiff, fine-grained sand, trace subrounded cobbles up to 2-inches diameter.
	2-6	Shale, black to orange-brown, highly weathered, moderately strong, crushed with some clay, fairly undulating, disrupted structure N70W 70-85SW.
		Total Depth 6 feet No free ground water encountered

TEST PIT LOGS

Test Pit Number	Depth (feet)	Description
TP2-26	0-2	Sandy Clay, brown, moist, stiff, fine-grained sand.
	2-6	Shale, black to orange-brown, highly weathered, weak, crushed with some clay, thinly bedded with sandstone, fine-grained, tan-brown, moderately strong, crushed. Bedding N20E 65S.
TP2-27	0-3	Sandy Clay, brown to dark yellow-brown, moist, stiff, fine-grained sand, some silt, iron oxide stains.
	3-14	Silty Clay, dark yellow-brown to brown, moist, very stiff, trace to some subangular to subrounded gravel up to ½-inch, angular friable sandstone clasts, trace fine- to medium-grained sand (Qls).
		Total Depth 14 feet No free ground water encountered
TP2-28	0-1½	Sandy Clay, brown, moist, stiff, fine-grained sand.
	1½-3½	Cobbles and orange-brown Clay (matrix), moist, dense, clast supported.
	3½-6	Sandy Clay, mottled orange-brown and gray, moist, very stiff, coarse-grained sand, trace subangular gravel up to ½-inch diameter, sharp basal contact with ⅛-inch orange-brown clay. N50E 18SE.
	6-8	Sandstone, fine-grained, tan-brown, highly weathered, weak to moderately strong, crushed with trace clay.
		Total Depth 8 feet No free ground water encountered
TP2-29	0-2½	Sandy Clay, brown, moist, stiff, fine-grained sand.
	2½-8	Silty Clay, mottled tan-brown and gray, moist, stiff to very stiff, trace fine-grained sand, trace well-rounded gravel up to ¼-inch diameter, gradational roughly horizontal basal contact over approximately 8 inches.
	8-11	Sandstone, fine-grained, tan-brown, weak to strong, highly fractured to crushed.
		Total Depth 11 feet No free ground water encountered

TEST PIT LOGS

Test Pit Number	Depth (feet)	Description
TP2-30	0-1	Sandy Silt, brown, dry, stiff, fine-grained sand.
	1-2½	Silty Clay, dark gray-brown, moist, stiff.
	2½-6	Sandy Clay, tan-brown to brown, moist, stiff, coarse-grained sand.
	6-8	Clayey Sand, brown to gray, moist, dense to very dense, coarse-grained sand.
	8-14	Sandy Clay, mottled orange-brown and gray, moist, very stiff, coarse-grained sand.
		Total Depth 14 feet No free ground water encountered
TP2-31	0-1½	Sandy Clay, brown, moist, stiff, some rounded cobbles up to 4-inches diameter.
	1½-3	Sandstone, fine-grained, tan-brown, highly weathered, crushed.
	3-5	Sheared Clay/Shale, black, highly weathered, weak, inclusions of shale and sandstone, faint foliation parallel to contact, faint residual bedrock structure. Bedding N40W 33S.
		Total Depth 5 feet No free ground water encountered
TP2-32	0-1½	Sandy Clay, brown, moist, stiff, fine-grained sand, trace subangular cobbles up to 4-inches diameter.
	1½-5	Shale, gray to orange-brown, highly weathered, weak to moderately strong, crushed. Bedding N70W 63S.
		Total Depth 5 feet No free ground water encountered
TP2-33	0-2½	Silty Clay, brown, moist, stiff, trace fine-grained sand.
	2½-6	Shale, black, highly weathered, weak to moderately strong, crushed, thinly bedded. Bedding N60W 70S.
		Total Depth 6 feet No free ground water encountered

TEST PIT LOGS

Test Pit Number	Depth (feet)	Description
TP2-34	0-3	Silty Clay, dark brown, moist, stiff, trace well-rounded gravel up to ¼-inch diameter.
	3-5	Silty Clay, brown, moist, very stiff, gradational basal contact over approximately 10-inches.
	5-8	Shale, gray to black, highly weathered, weak to moderately strong, crushed with trace to some clay.
		Total Depth 8 feet No free ground water encountered
TP2-35	0-½	Sandy Silt and Gravel, tan-brown, dry, hard (Fill).
	½-1½	Sandy Clay, brown, moist, very stiff, fine-grained sand.
		Total Depth 1½ feet No free ground water encountered
TP2-36	0-1	Sandy Silt and Gravel, tan-brown, dry, hard (Fill).
	1-1½	Sandy Clay, brown, moist, very stiff, fine-grained sand.
		Total Depth 1½ feet No free ground water encountered

TEST PIT LOGS AND TRENCH LOG

Test Pit Number	Depth (Feet)	Description
TP-1	0-1½	Sandy Clay, brown, wet to saturated, medium stiff, fine-grained sand.
	1½-2½	Sandy Clay, orange-brown, moist, very stiff, fine-grained sand, trace to some gravel.
	2½-6	Sandstone, coarse-grained, orange-brown, highly weathered, weak, highly fractured.
		Total Depth 6 feet Free ground water encountered at 1½ feet
TP-2	0-2½	Sandy Clay, brown, wet to saturated, medium stiff, fine-grained sand.
	2½-6	Sandstone, fine- to medium-grained, orange-brown to gray, highly weathered, crushed, sheared with serpentine rich clay.
		Total Depth 6 feet Free ground water encountered at 2½ feet
TP-3	0-3	Sandy Clay, brown, wet to saturated, medium stiff, fine-grained sand.
	3-5	Sandy Clay, orange-brown, moist, stiff, fine-grained sand.
	5-8	Sandstone, fine- to medium-grained, orange-brown, weak, highly fractured.
		Total Depth 8 feet Free ground water encountered at 2 feet
TP-4	0-2½	Sandy Clay, brown, wet to saturated, medium stiff, fine-grained sand.
	2½-5	Silty Clay, orange-brown, moist, very stiff, trace fine-grained sand.
	5-7	Sandy Clay, orange-brown and gray, moist, very stiff, fine-grained sand.
	7-10	Sandstone, fine-grained, gray-brown, highly weathered, highly fractured.
		Total Depth 10 feet Free ground water encountered at 2½ feet

TEST PIT LOGS AND TRENCH LOG

Test Pit Number	Depth (Feet)	Description
TP-5	0-2	Sandy clay, brown, wet to saturated, medium stiff, fine-grained sand.
	2-9	Sandy Clay, orange-brown and gray, moist, very stiff, fine-grained sand.
	9-12	Sandstone, fine-grained, gray-brown, highly weathered, weak, highly fractured.
		Total Depth 12 feet Free ground water encountered at 2 feet
TP-6	0-2	Sandy Clay, brown, wet to saturated, medium stiff, fine-grained sand.
	2-3	Silty clay, gray, moist, stiff, gradational upper and basal contacts.
	3-4½	Sandy Clay, mottled orange-brown and gray, moist, very stiff.
	4½-7	Sandstone, fine-grained, gray-brown, highly weathered, weak, highly fractured.
		Total Depth 7 feet Free ground water encountered at 2 feet
TP-7	0-2	Sandy Clay, brown, moist to wet, medium stiff, fine-grained sand.
	2-3	Silty Clay, mottled gray and orange-brown, moist, very stiff.
	3-6	Sandstone, fine-grained, gray-brown, highly weathered, weak to moderately strong, highly fractured.
		Total Depth 6 feet Free ground water encountered at 2 feet
TP-8	0-2	Sandy clay, gray-brown, moist to wet, stiff, fine-grained sand.
	2-6	Sandstone, fine-grained, gray, highly weathered, weak, highly fractured to crushed.
		Total Depth 6 feet No free ground water encountered

TEST PIT LOGS AND TRENCH LOG

Test Pit Number	Depth (Feet)	Description
TP-9	0-6	Silty Clay, dark gray-brown, moist, stiff.
	6-8	Silty Clay, brown, moist, stiff, trace gravel. Sharp basal contact, possible slide plane 18° dip.
	8-11	Sandstone, fine-grained, gray-brown, highly weathered, weak.
		Total Depth 11 feet No free ground water encountered
TP-10	0-4	Silty Clay, dark gray-brown, moist, stiff, trace caliche.
	4-5½	Silty Clay, gray-brown, moist, stiff to very stiff, trace gravel.
	5½-9	Sandstone, fine-grained, gray, highly weathered, weak to moderately strong.
		Total Depth 9 feet No free ground water encountered
TP-11	0-3	Silty Clay, dark gray-brown, moist, stiff.
	3-6	Sandstone, fine-grained, gray, highly weathered, weak, highly fractured.
		Total Depth 6 feet No free ground water encountered
TP-12	0-3	Sandy Clay, brown, moist to wet, stiff, trace gravel, fine-grained sand (surficial Qls fresh).
	3-7	Sandy Clay, brown, moist, stiff, fine-grained sand. Sharp basal contact, slide plane dipping 20° downslope.
	7-11	Sandstone, fine-grained, orange-brown, highly weathered.
		Total Depth 11 feet No free ground water encountered
TP-13	0-3½	Silty Clay, brown, wet to saturated, medium stiff to stiff.
	3½-5	Sandstone, fine-grained, gray, highly weathered, strong to very strong, highly fractured.

TEST PIT LOGS AND TRENCH LOG

Test Pit Number	Depth (Feet)	Description
		Total Depth 5 feet Free ground water encountered at surface and at 2 feet
TP-14	0-3	Silty Clay, dark brown, wet to saturated, medium stiff.
	3-8	Silty Clay, orange-brown, moist, very stiff, some fine-grained sand.
	8-11	Sandstone, fine-grained, gray, highly weathered, weak, highly fractured, thinly bedded with shale, gray, highly weathered, weak, highly fractured.
		Total Depth 11 feet Free ground water encountered at 3 feet
TP-15	0-3	Silty Clay, dark gray-brown, saturated, medium stiff, trace gravel, 3 foot diameter boulder.
	3-6	Silty Clay, brown, moist to wet, stiff, trace gravel.
	6-9	Serpentinite, gray-green to black, highly weathered, friable to moderately strong, crushed, shear foliation.
		Total Depth 9 feet Free ground water encountered at 3 feet
TP-16	0-5	Silty Clay, dark gray-brown, moist to saturated, medium stiff to stiff, trace gravel.
	5-7	Silty Clay, brown, moist, stiff, trace gravel, faint surface at 7 feet, possible slide plane dipping 12°.
	7-10	Sandstone, fine-grained, brown, highly weathered, moderately strong, highly fractured, thinly bedded with shale, gray, highly weathered, weak, crushed, bedding N50W 59S.
		Total Depth 10 feet Free ground water encountered at 3 feet

TEST PIT LOGS AND TRENCH LOG

Test Pit Number	Depth (Feet)	Description
TP-17	0-2	Silty Clay, brown, moist, stiff, trace gravel.
	2-4	Sheared Clayey Shale, black and gray-green, highly weathered, friable, crushed with clasts of SS, SH (MS).
	4-7	Sandstone, fine-grained, brown, highly weathered, moderately strong, bedding N50W 50S.
		Total Depth 7 feet No free ground water encountered
TP-18	0-2	Silty Clay, dark-brown, moist, stiff, trace fine-grained sand.
	2-6	Sandstone, fine-grained, gray-brown, highly weathered, moderately strong, highly fractured, thinly bedded with shale, black, highly weathered, weak, crushed, bedding N60W 59S.
		Total Depth 6 feet No free ground water encountered
TP-19	0-3	Silty Clay, brown, moist to wet, medium stiff to stiff, trace gravel.
	3-5	Silty Clay, gray-brown, moist, very stiff, trace gravel.
	5-7	Sandy Clay, tan-brown, moist, very stiff, fine-grained sand.
	7-10	Sandstone, medium-grained, tan-brown, highly weathered, weak to moderately strong, highly fractured.
		Total Depth 10 feet Free ground water encountered at 3 feet
TP-20	0-4	Silty Clay, dark brown, moist, stiff, trace fine-grained sand.
	4-6½	Silty Clay, tan-brown, moist, very stiff, trace gravel, at 6½ feet, slide plane, ¼-inch gray clay gouge with a slickensided surface and faint striations N75W 23N.
	6½-10	Sheared Clayey Shale, black, highly weathered, moderately strong, crushed, faint shear foliation.
		Total Depth 10 feet Free ground water encountered at 4 feet

TEST PIT LOGS AND TRENCH LOG

Test Pit Number	Depth (Feet)	Description
TP-21	0-4½	Silty clay, brown, wet, medium stiff, trace gravel.
	4½-12	Silty Clay, red-brown to brown, moist, very stiff, trace to some 1 to 3 inch angular gravel.
	12-13	Sandstone, fine-grained, gray, highly weathered, moderately strong, highly fractured.
		Total Depth 13 feet Free ground water encountered at 4 feet
TP-22	0-3	Silty Clay, dark gray-brown, saturated, medium stiff to stiff.
	3-5	Clayey Sand, brown, moist, dense, coarse-grained sand, trace fine-grained gravel, thinly laminated.
	5-11	Sandy Clay, brown, moist, stiff to very stiff, coarse-grained sand, trace gravel.
		Total Depth 11 feet Free ground water encountered at 3 feet
TP-23	0-2	Sandy Clay, brown, moist to wet, medium stiff.
	2-6	Sandstone, fine- to medium-grained, brown to gray, highly weathered, strong, highly fractured.
		Total Depth 6 feet No free ground water encountered
TP-24	0-4	Silty Clay, gray-brown, moist to wet, stiff, trace gravel, smooth slickensided surface at 4 feet slide plane, N82W 12S.
	4-7	Sandstone, fine-grained, gray, highly weathered, strong, highly fractured, thinly bedded, bedding N71W 73S.
		Total Depth 7 feet No free ground water encountered
TP-25	0-4	Silty Clay, dark gray-brown, moist to wet, stiff, trace gravel.
	4-8	Clayey Sand, tan-brown, moist, very dense, coarse-grained

TEST PIT LOGS AND TRENCH LOG

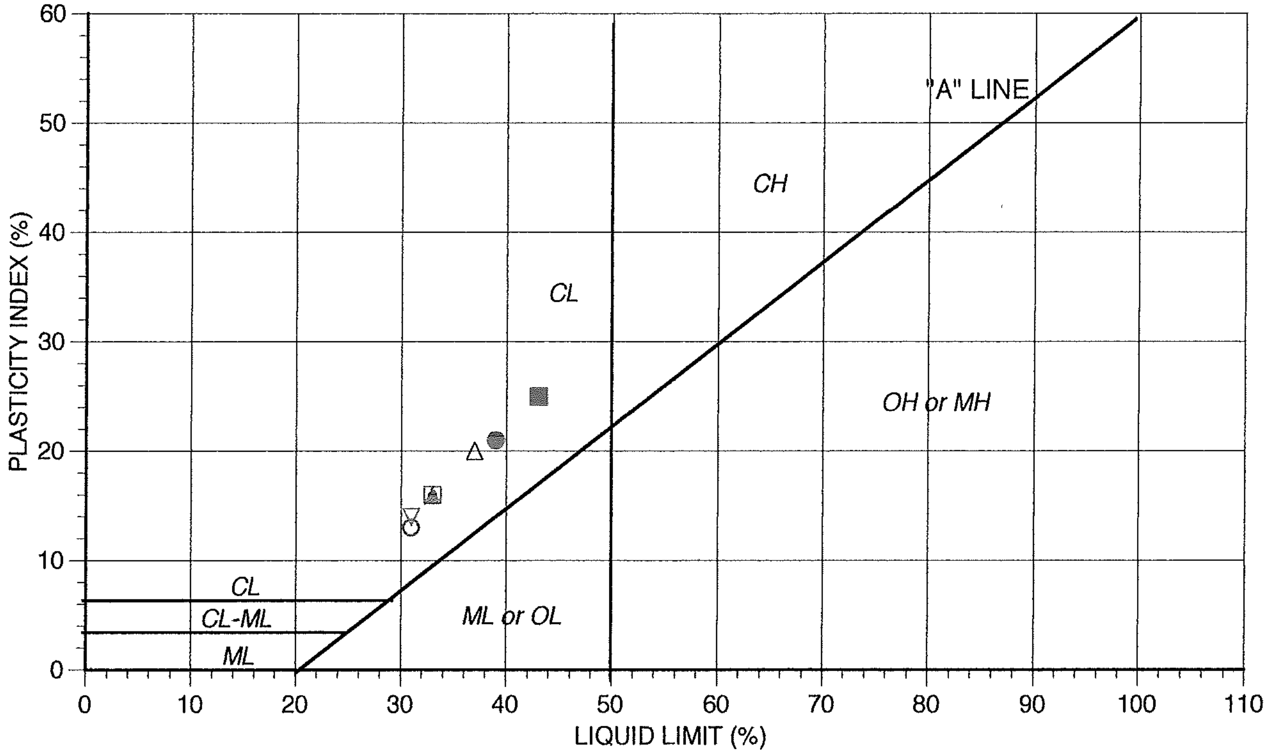
Test Pit Number	Depth (Feet)	Description
		sand, thin subhorizontal gravel laminations.
	8-11	Sandy Clay, tan-brown, moist, very stiff, trace gravel.
		Total Depth 11 feet Free ground water encountered at 4 feet
TP-26	0-1½	Clayey Sand, orange-brown, saturated, loose, coarse-grained sand.
	1½-4	Clayey Sand, orange-brown, moist, dense to very dense, coarse-grained sand.
	4-7	Sandy Clay, orange-brown, moist, very stiff, coarse-grained sand.
	7-10	Sandstone, fine-grained, gray-brown, highly weathered, weak, highly fractured.
		Total Depth 10 feet No free ground water encountered

Trench Number	Depth (Feet)	Description
T-1	0-1	Sandy Clay, brown, wet to saturated, medium stiff, fine-grained sand.
	1-2	Sandstone, orange-brown, coarse-grained, highly weathered, crushed.
		Excavated 1 foot deep into bedrock. Bedrock is all continuous strong sandstone.

APPENDIX B

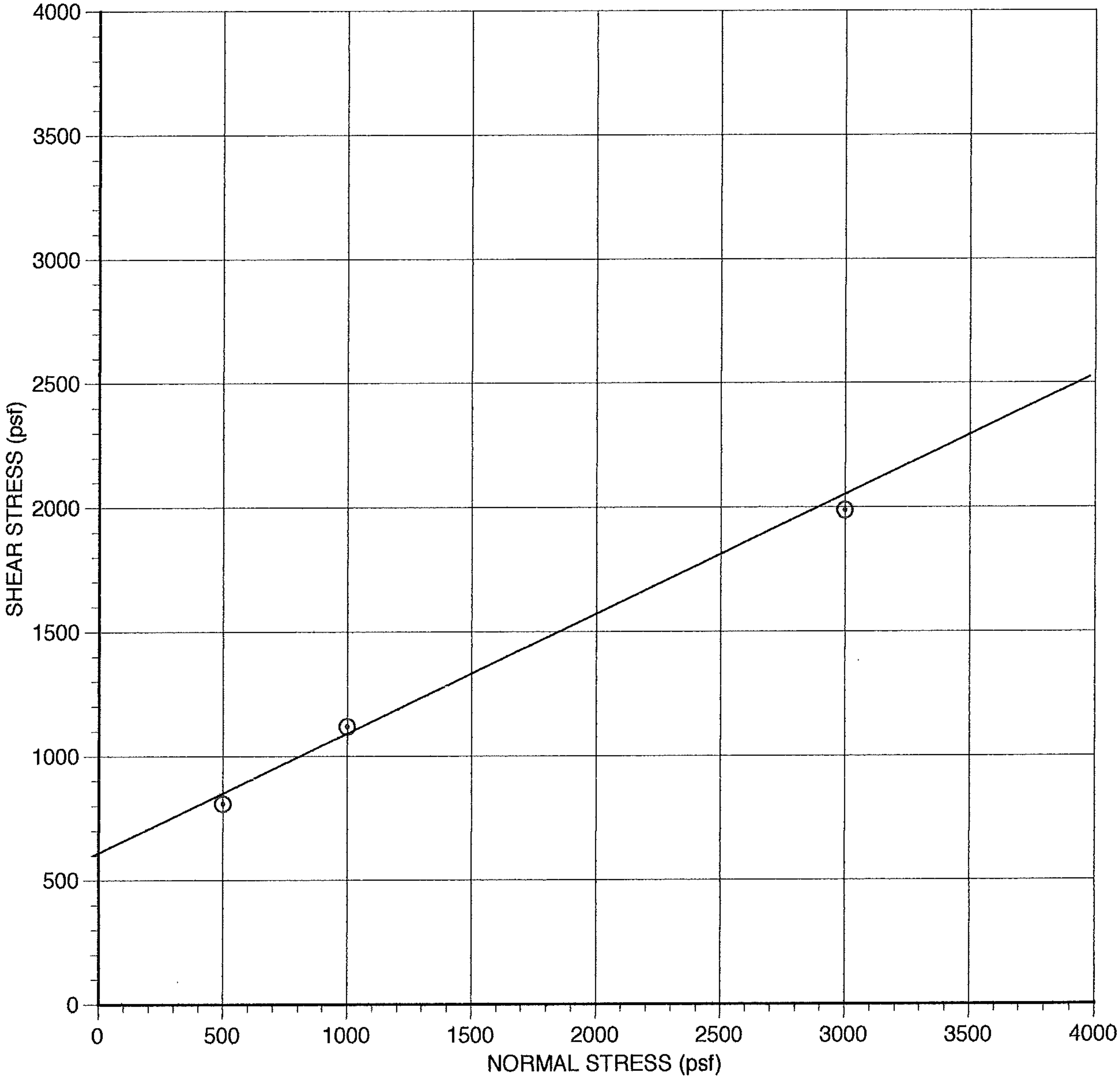
Laboratory Test Results

BY: PW DATE: 4-23-03 JOB NUMBER: 2616.100



SYMBOLS	LOCATION	LIQUID LIMIT	PLASTICITY INDEX	USCS CLASSIFICATION
●	B-1 at 1 foot	39	21	C L
□	B-3 at 5 feet	33	16	C L
▲	B-13 at 1 foot	33	16	C L
○	TP2-2 at 0 to 2 feet	31	13	C L
■	TP2-5 at 0-3 feet	43	25	C L
△	TP2-13 at 6-9 feet	37	20	C L
▽	TP2-24 at 6-1/2 feet	31	14	C L

ATTERBERG LIMITS TEST DATA



JOB NUMBER: 2616.100 DATE: 4-23-03 BY: PW

LOCATION: B-3 at 9-1/2 feet

SAMPLE: SANDY CLAY, mottled yellow brown olive gray

TEST TYPE: Consolidated Drained

RATE OF SHEAR (in/min): 0.00099

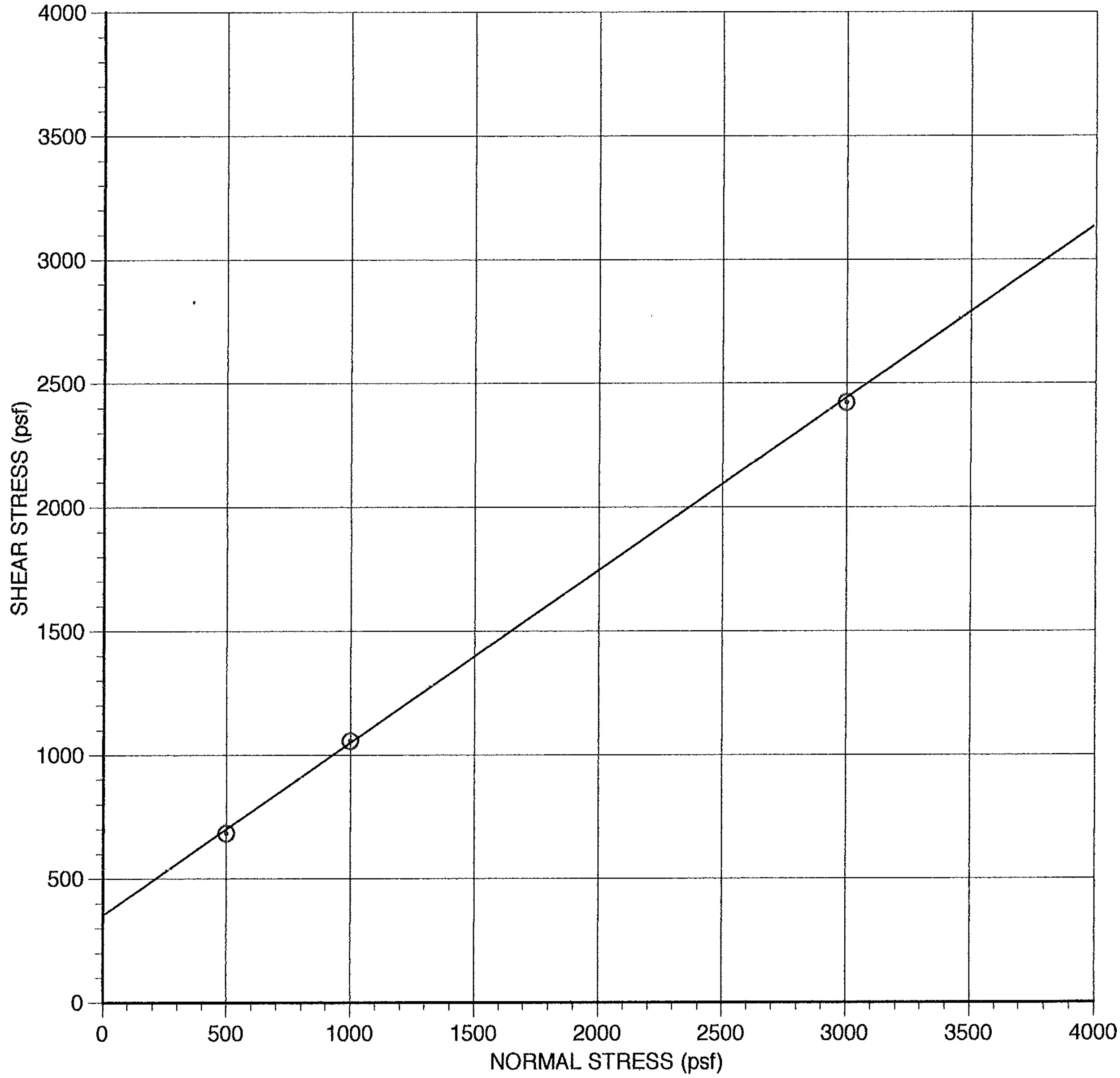
FRICTION ANGLE: 25°

COHESION: 600 psf

SPECIMEN	A	B	C
DRY DENSITY (pcf)	107.6	105.6	106.0
INITIAL WATER CONTENT (%)	19.3	20.8	19.4
FINAL WATER CONTENT (%)	20.8	22.2	19.1
NORMAL STRESS (psf)	500	1000	3000
MAXIMUM SHEAR (psf)	808	1118	1988

DIRECT SHEAR TEST

JOB NUMBER: 2616.100 DATE: 4-23-03 BY: PW



LOCATION: B-5 at 9 feet

SAMPLE: SILTY CLAY, dark gray, trace sand

TEST TYPE: Consolidated Drained

RATE OF SHEAR (in/min): 0.00099

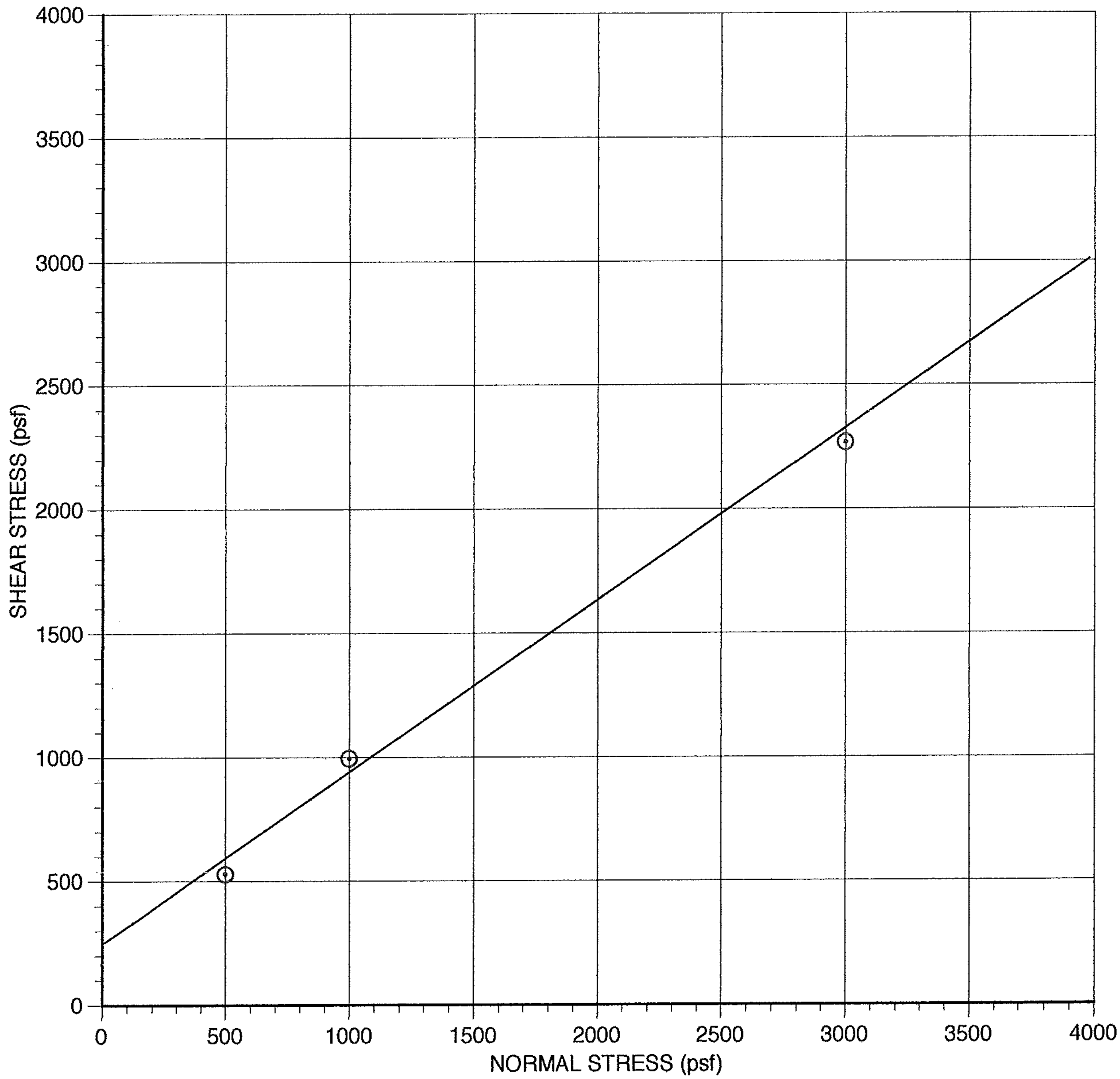
FRICTION ANGLE: 35°

COHESION: 350 psf

SPECIMEN	A	B	C
DRY DENSITY (pcf)	107.8	108.7	108.1
INITIAL WATER CONTENT (%)	19.0	17.4	17.2
FINAL WATER CONTENT (%)	20.6	18.7	16.9
NORMAL STRESS (psf)	500	1000	3000
MAXIMUM SHEAR (psf)	683	1056	2423

DIRECT SHEAR TEST

JOB NUMBER: 2616.100 DATE: 4-23-03 BY: PW



LOCATION: B-14 at 5 feet

SAMPLE: SILTY CLAY with sand, brown

TEST TYPE: Consolidated Drained

RATE OF SHEAR (in/min): 0.00099

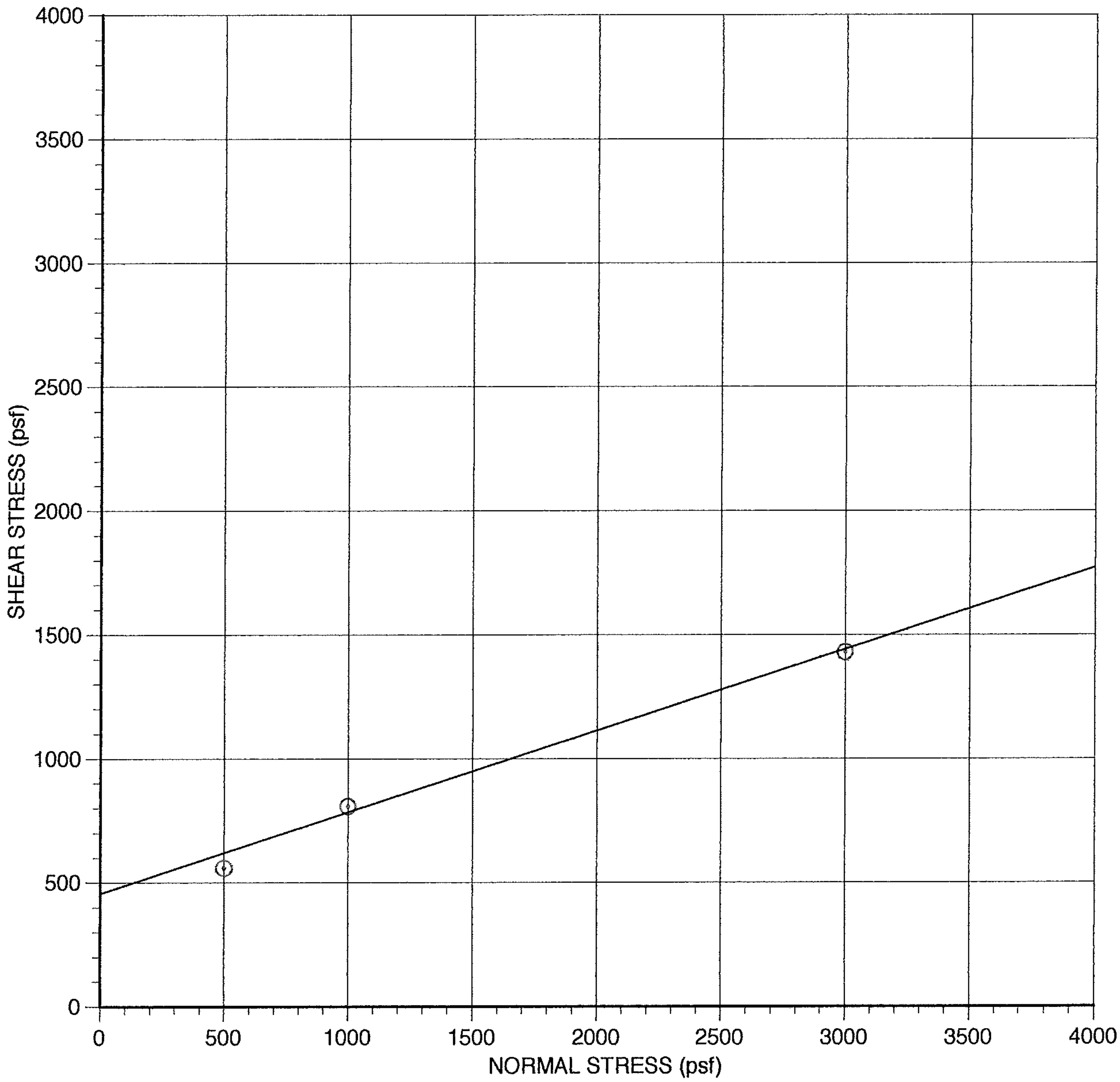
FRICTION ANGLE: 34.5°

COHESION: 250 psf

SPECIMEN	A	B	C
DRY DENSITY (pcf)	106.3	106.7	107.2
INITIAL WATER CONTENT (%)	20.9	20.3	17.6
FINAL WATER CONTENT (%)	20.9	20.3	18.8
NORMAL STRESS (psf)	500	1000	3000
MAXIMUM SHEAR (psf)	528	994	2268

DIRECT SHEAR TEST

JOB NUMBER: 2616.100 DATE: 4-23-03 BY: PW



LOCATION: TP2-2 at 2 feet

SAMPLE: SANDY SILT CLAY, olive-brown, Remolded to 90% Relative Compaction

TEST TYPE: Consolidated Drained

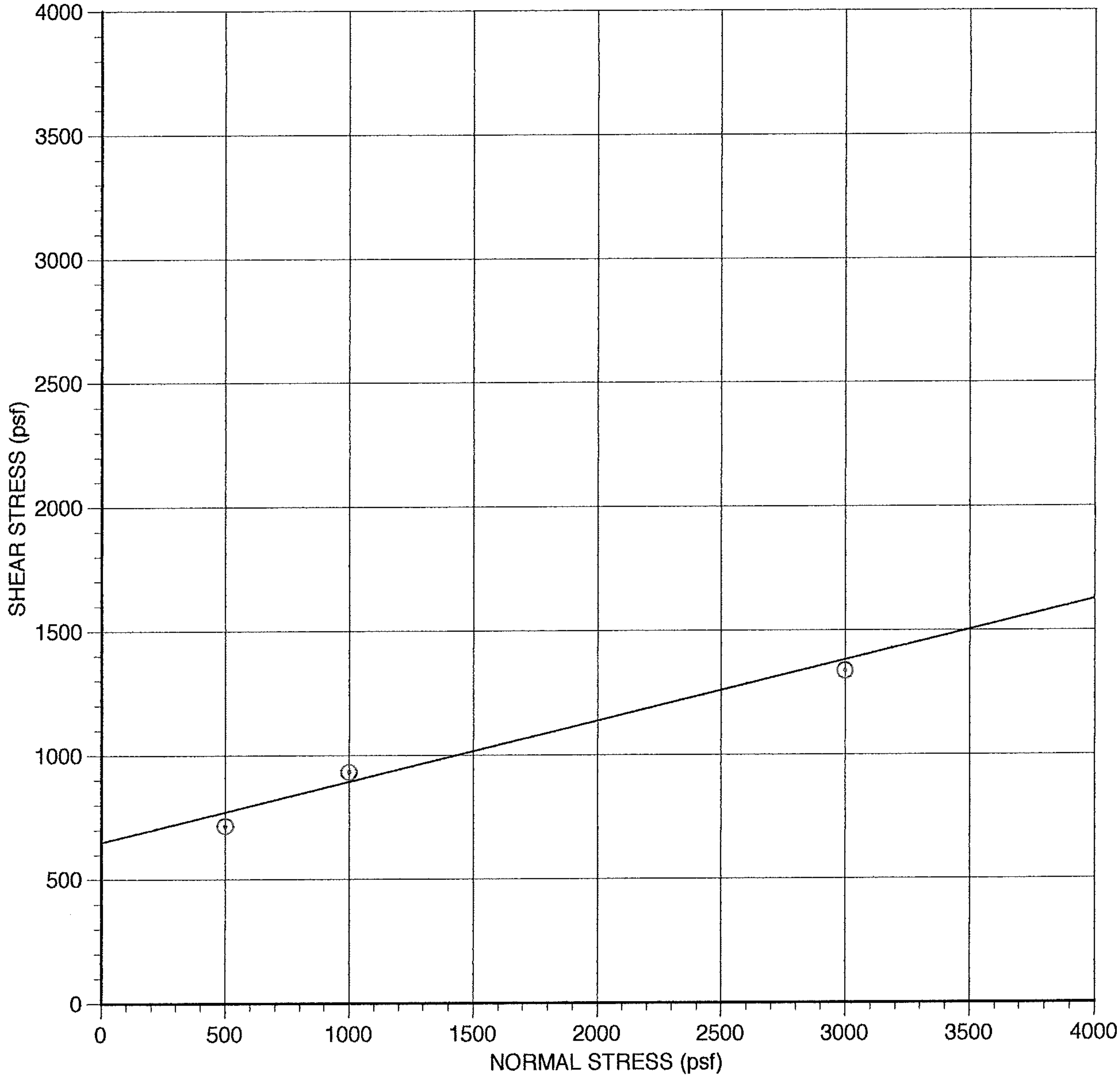
RATE OF SHEAR (in/min): 0.00099

FRICTION ANGLE: 18°

COHESION: 450 psf

SPECIMEN	A	B	C
DRY DENSITY (pcf)	110.5	110.8	110.8
INITIAL WATER CONTENT (%)	14.9	14.9	15.0
FINAL WATER CONTENT (%)	18.4	16.0	15.4
NORMAL STRESS (psf)	500	1000	3000
MAXIMUM SHEAR (psf)	559	808	1429

DIRECT SHEAR TEST



LOCATION: TP2-13 at 6 to 9 feet

SAMPLE: SANDY CLAY, olive-brown with gray, Remolded to 90% Relative Compaction

JOB NUMBER: 2616.100 DATE: 4-23-03 BY: PW

TEST TYPE: Consolidated Drained

RATE OF SHEAR (in/min): 0.00099

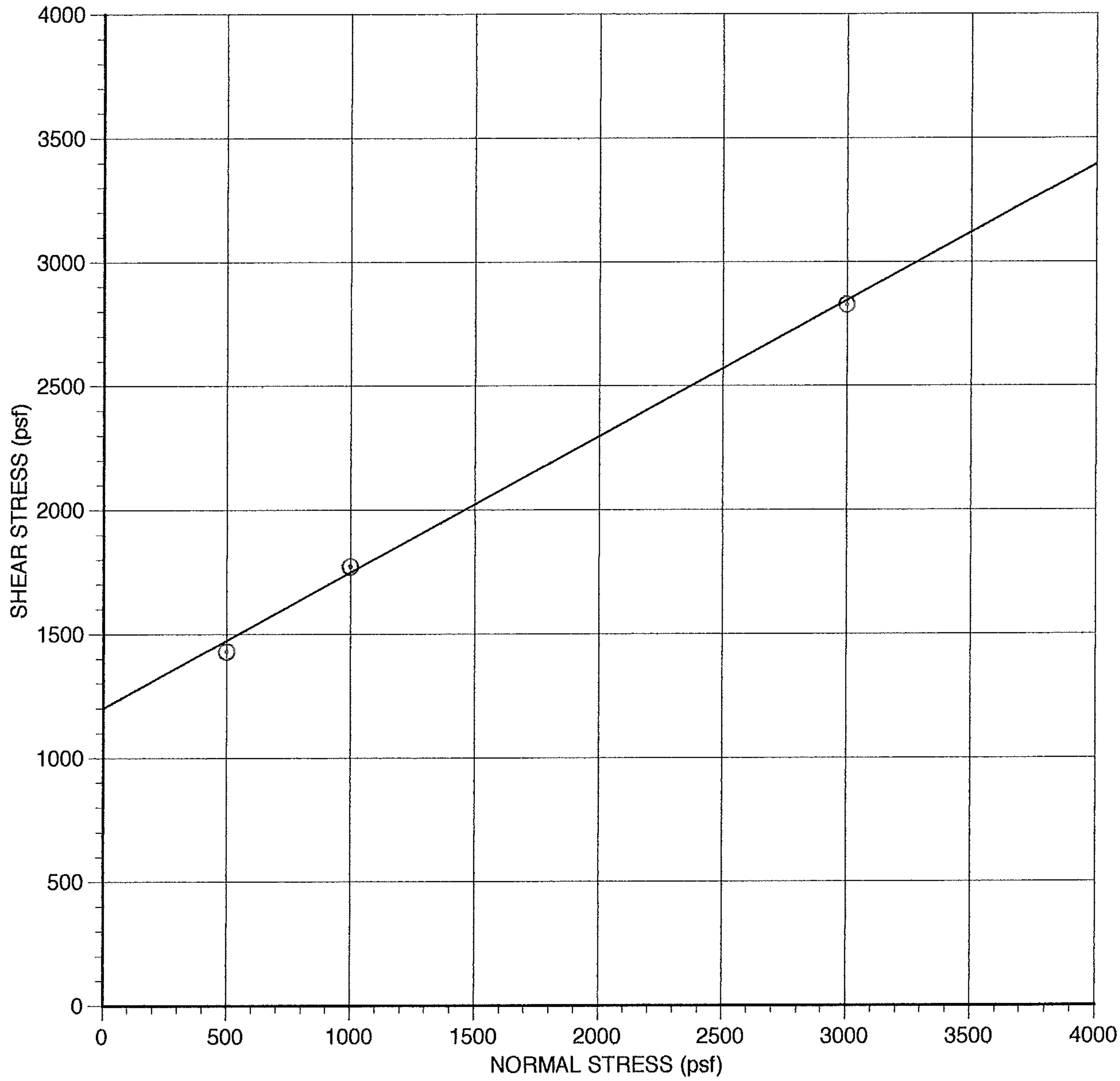
FRICTION ANGLE: 14°

COHESION: 650 psf

SPECIMEN	A	B	C
DRY DENSITY (pcf)	120.2	119.0	120.3
INITIAL WATER CONTENT (%)	11.4	11.7	11.4
FINAL WATER CONTENT (%)	13.2	13.9	13.6
NORMAL STRESS (psf)	500	1000	3000
MAXIMUM SHEAR (psf)	715	932	1336

DIRECT SHEAR TEST

JOB NUMBER: 2616.100 DATE: 4-23-03 BY: PW



LOCATION: TP2-15 at 4 to 8 feet

SAMPLE: CLAYEY SANDSTONE, olive-gray, Remolded to 90% Relative Comaction

TEST TYPE: Consolidated Drained

RATE OF SHEAR (in/min): 0.00099

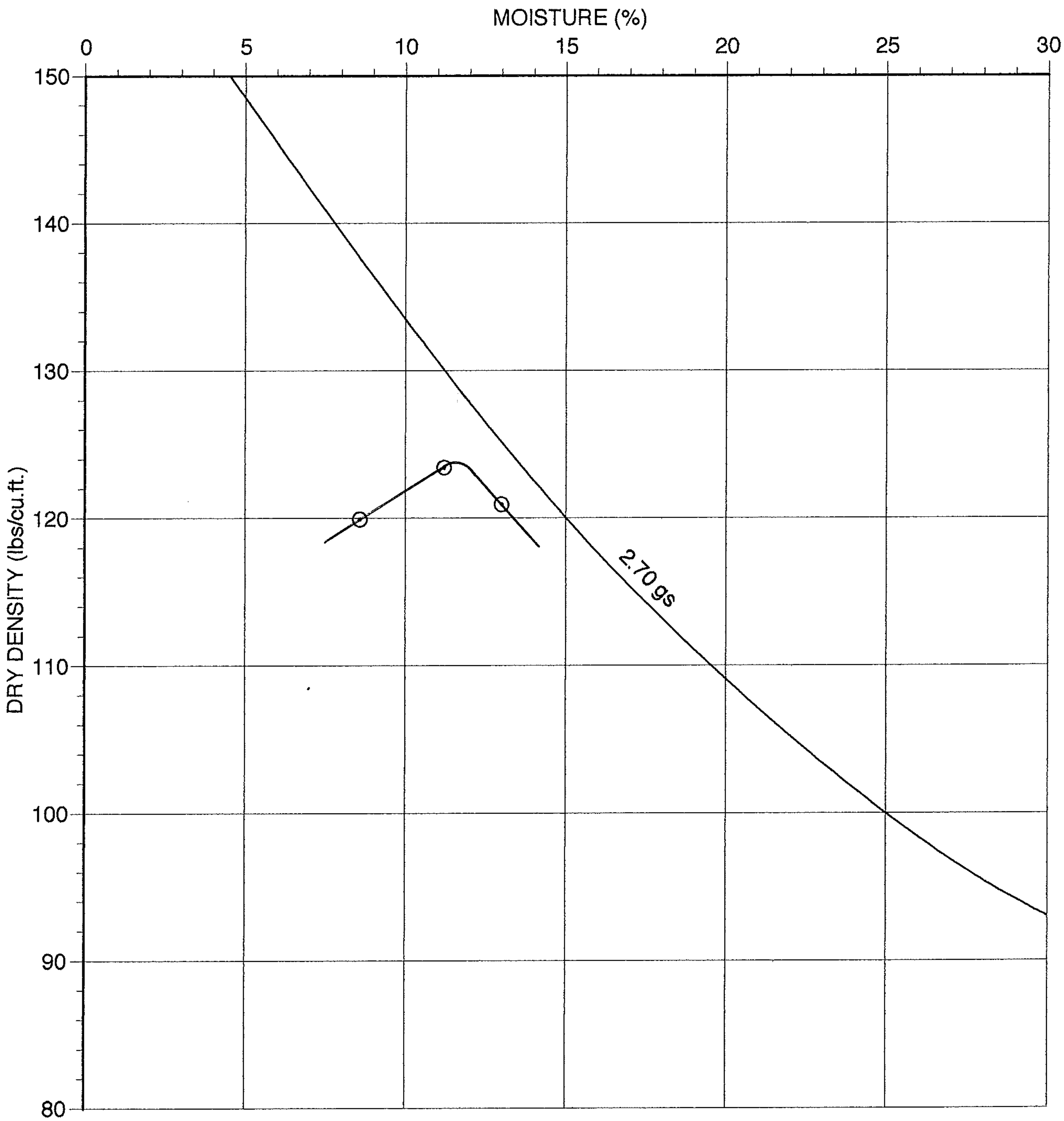
FRICTION ANGLE: 29°

COHESION: 1200 psf

SPECIMEN	A	B	C
DRY DENSITY (pcf)	124.8	124.8	124.4
INITIAL WATER CONTENT (%)	9.5	9.7	9.8
FINAL WATER CONTENT (%)	10.3	11.0	10.5
NORMAL STRESS (psf)	500	1000	3000
MAXIMUM SHEAR (psf)	1429	1771	2827

DIRECT SHEAR TEST

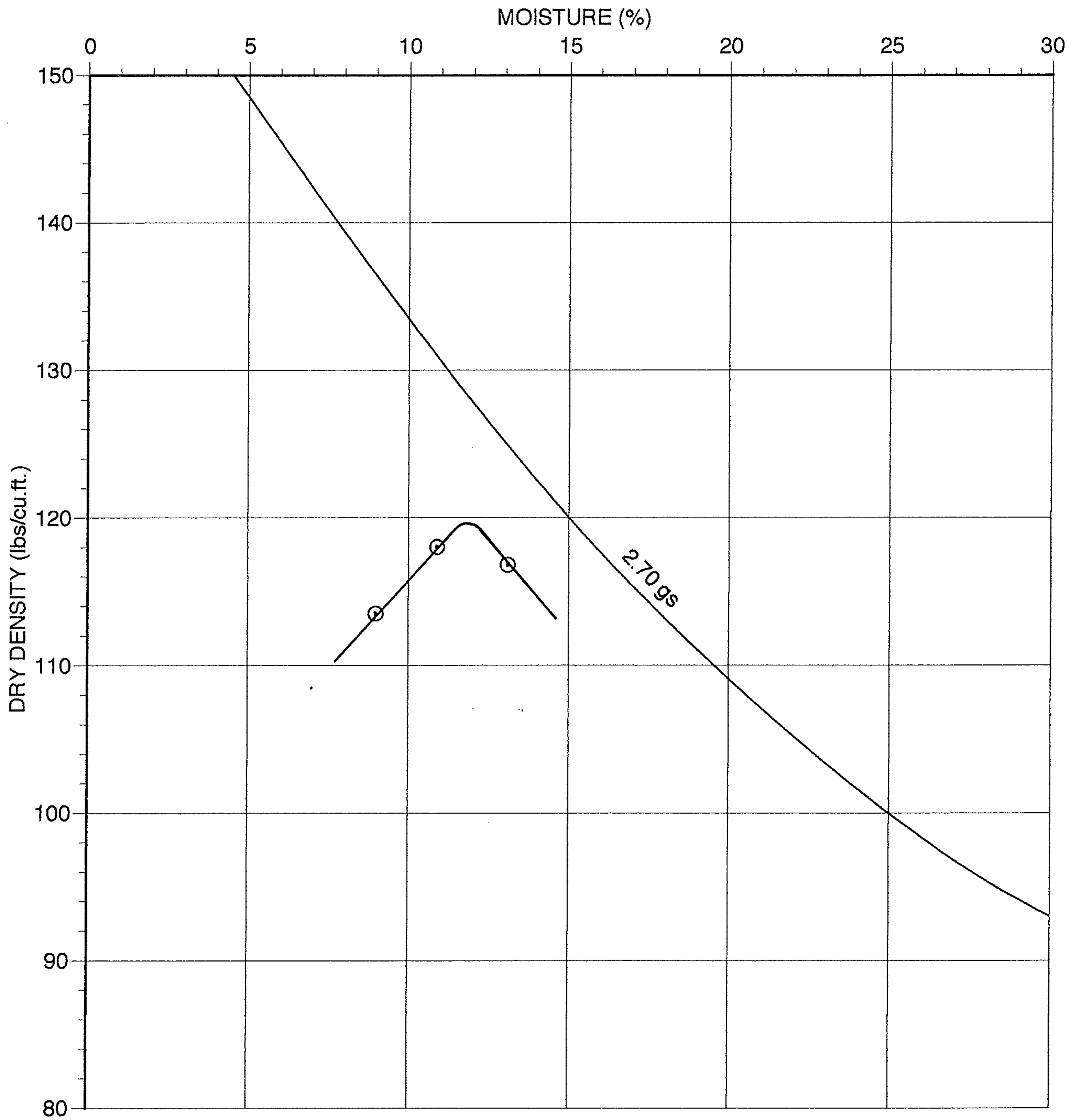
JOB NUMBER: 2616.100 DATE: 4-23-03 BY: PW



SYMBOL	LOCATION	DESCRIPTION	OPTIMUM MOISTURE CONTENT (%)	MAX. DRY DENSITY (pcf)
⊙	TP2-2 at 0-2 feet	SILTY CLAY, brown	11.5	123.6

COMPACTION TEST DATA

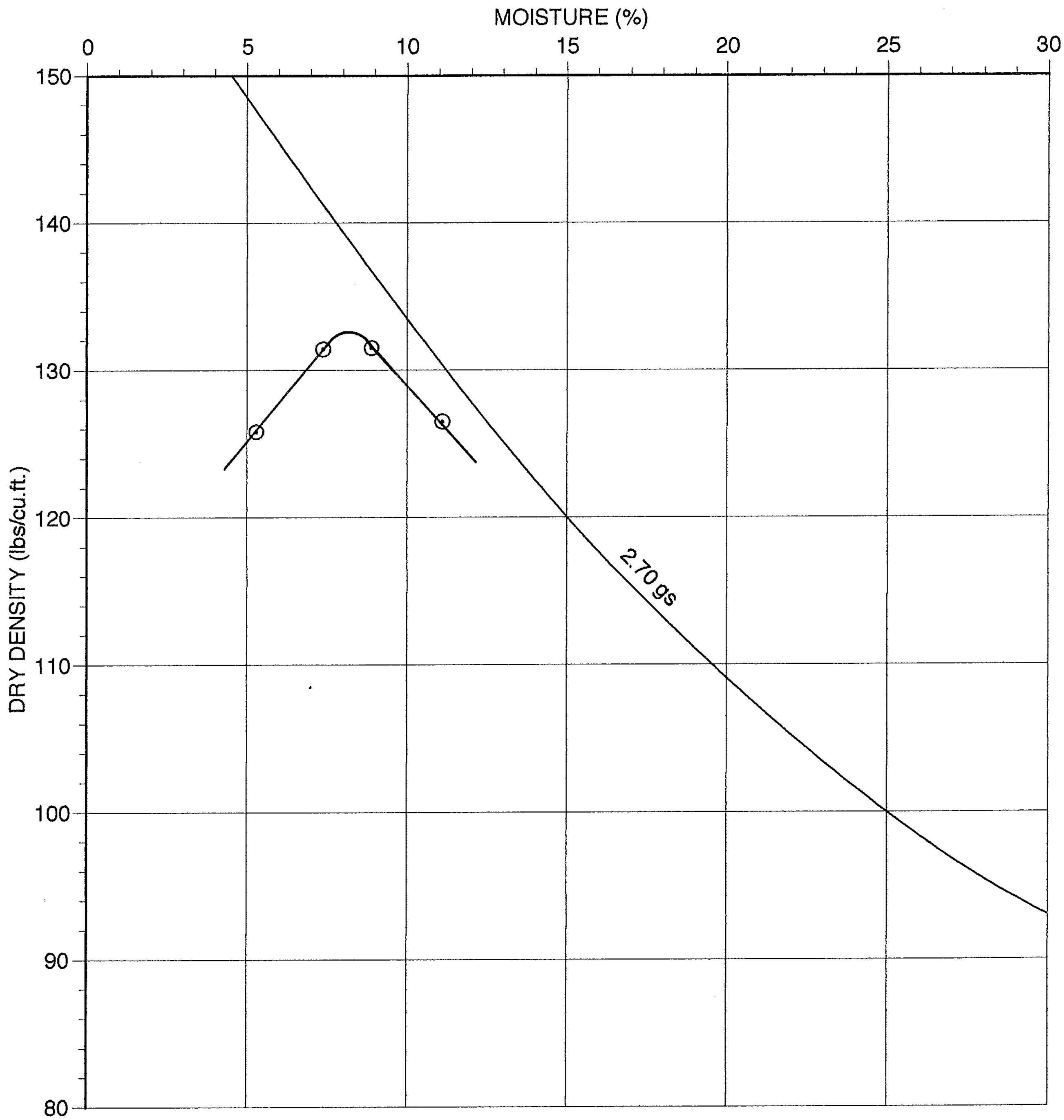
JOB NUMBER: 2616.100 DATE: 4-23-03 BY: PW



SYMBOL	LOCATION	DESCRIPTION	OPTIMUM MOISTURE CONTENT (%)	MAX. DRY DENSITY (pcf)
⊙	TP2-5 at 0-3-1/2 feet	SILTY CLAY, gray brown	11.8	119.5

COMPACTION TEST DATA

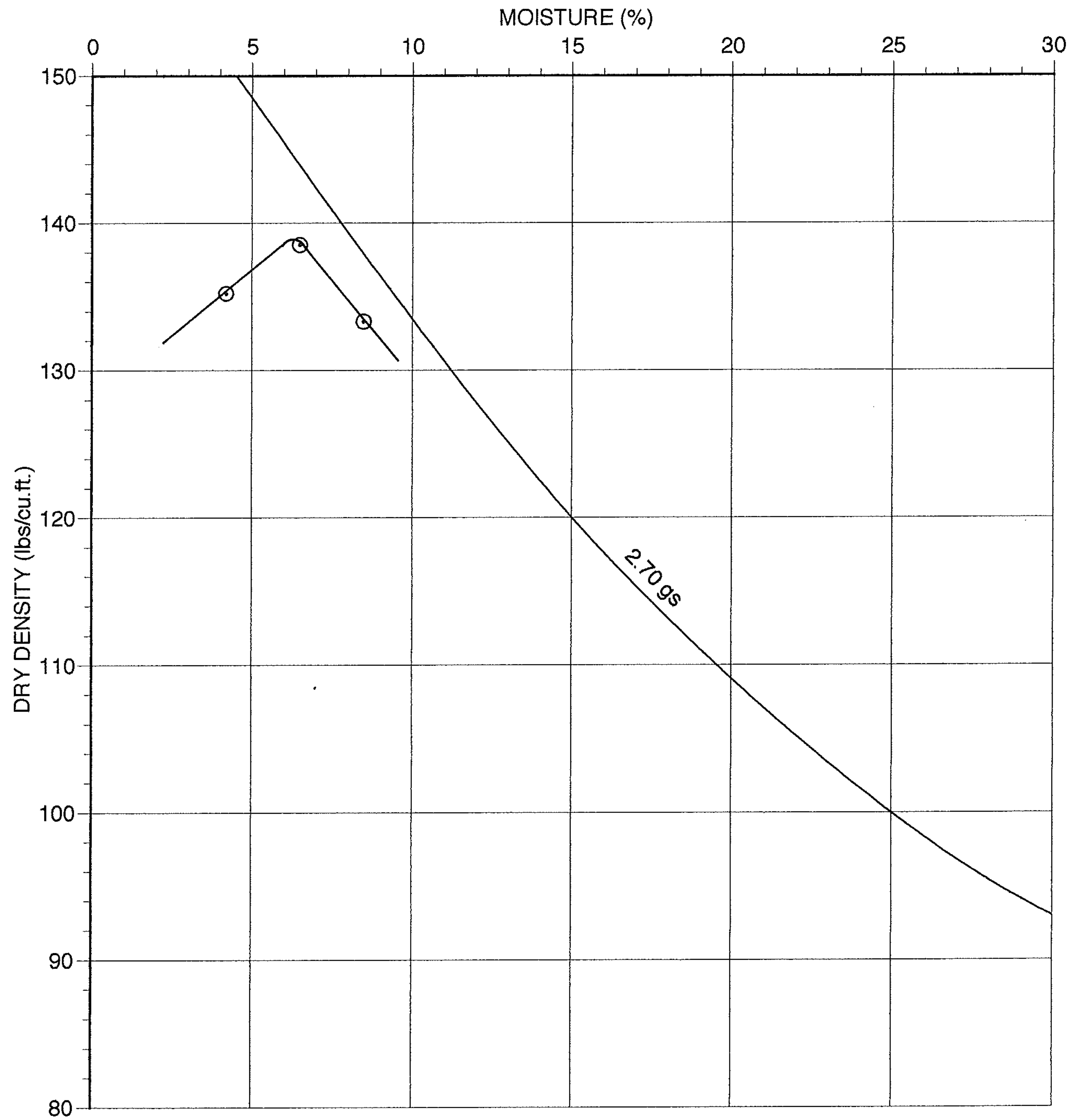
JOB NUMBER: 2616.100 DATE: 4-23-03 BY: PW



SYMBOL	LOCATION	DESCRIPTION	OPTIMUM MOISTURE CONTENT (%)	MAX. DRY DENSITY (pcf)
⊙	TP2-13 at 6 to 9 feet	CLAYEY SILTSTONE, gray-brown	8.2	132.2

COMPACTION TEST DATA

JOB NUMBER: 2616.100 DATE: 4-23-03 BY: PW



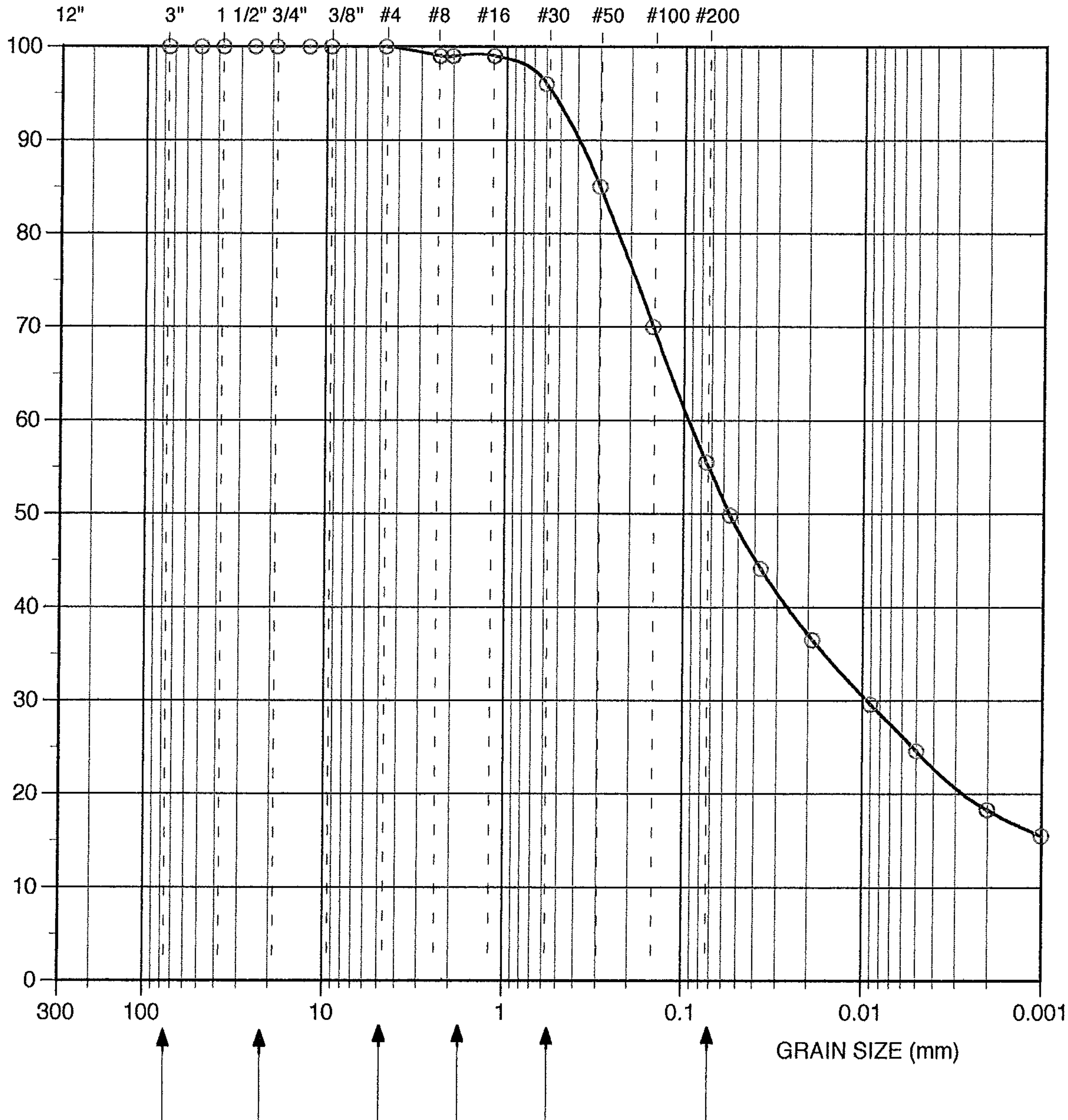
SYMBOL	LOCATION	DESCRIPTION	OPTIMUM MOISTURE CONTENT (%)	MAX. DRY DENSITY (pcf)
⊙	TP2-15 at 4 to 8 feet	SILTY CLAY, gray brown	6.2	138.7

COMPACTION TEST DATA

JOB NUMBER: 2616.100

DATE: 5-1-03

BY: FF



COBBLES	GRAVEL		SAND			SILT/CLAY
	coarse	fine	coarse	medium	fine	

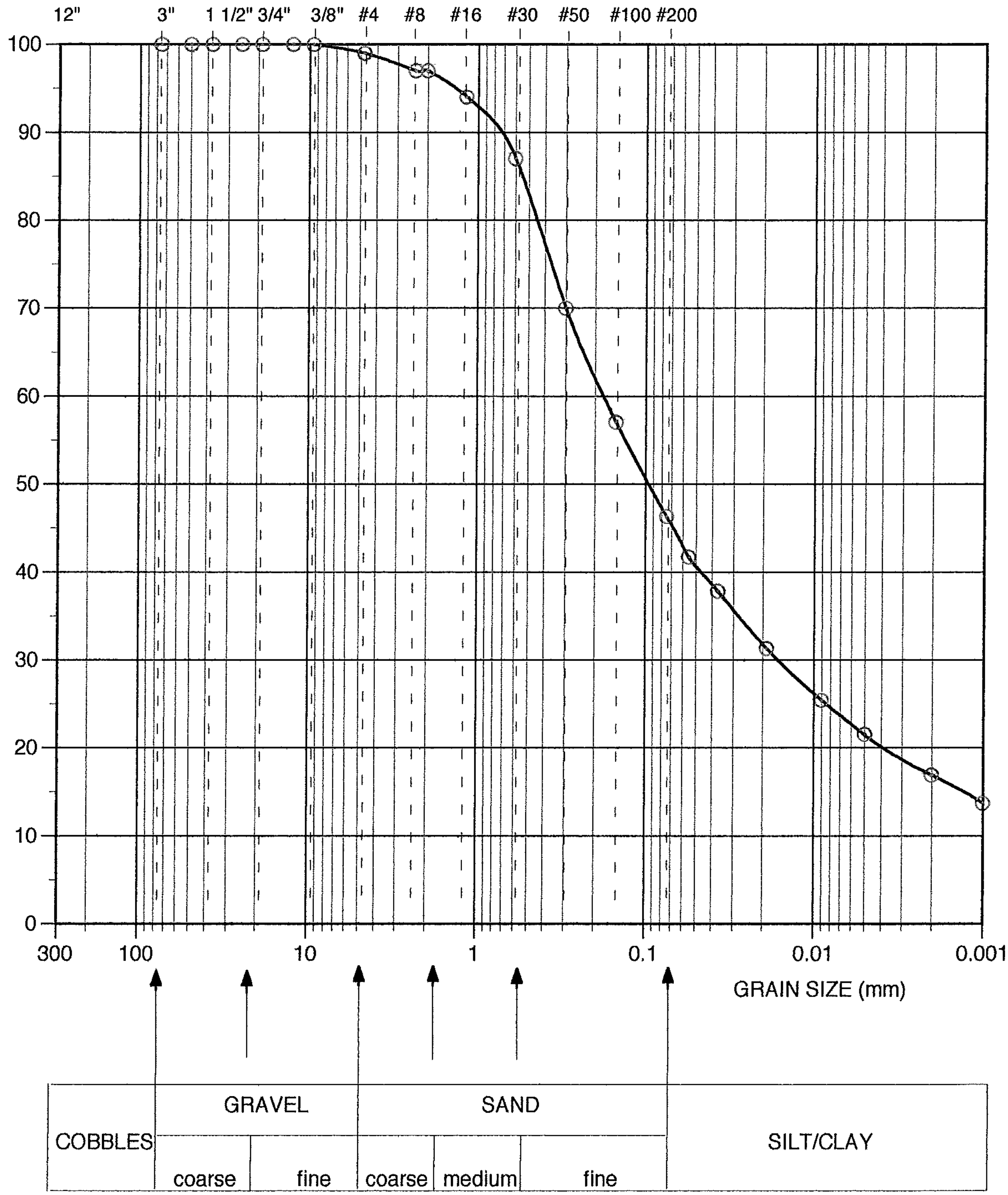
SYMBOLS	LOCATION	DESCRIPTION
⊙	B-8 at 10-1/2 to 12 feet	SANDY SILT, light brown with clay

GRADATION TEST DATA

BY: FF

DATE: 5-1-03

JOB NUMBER: 2616.100



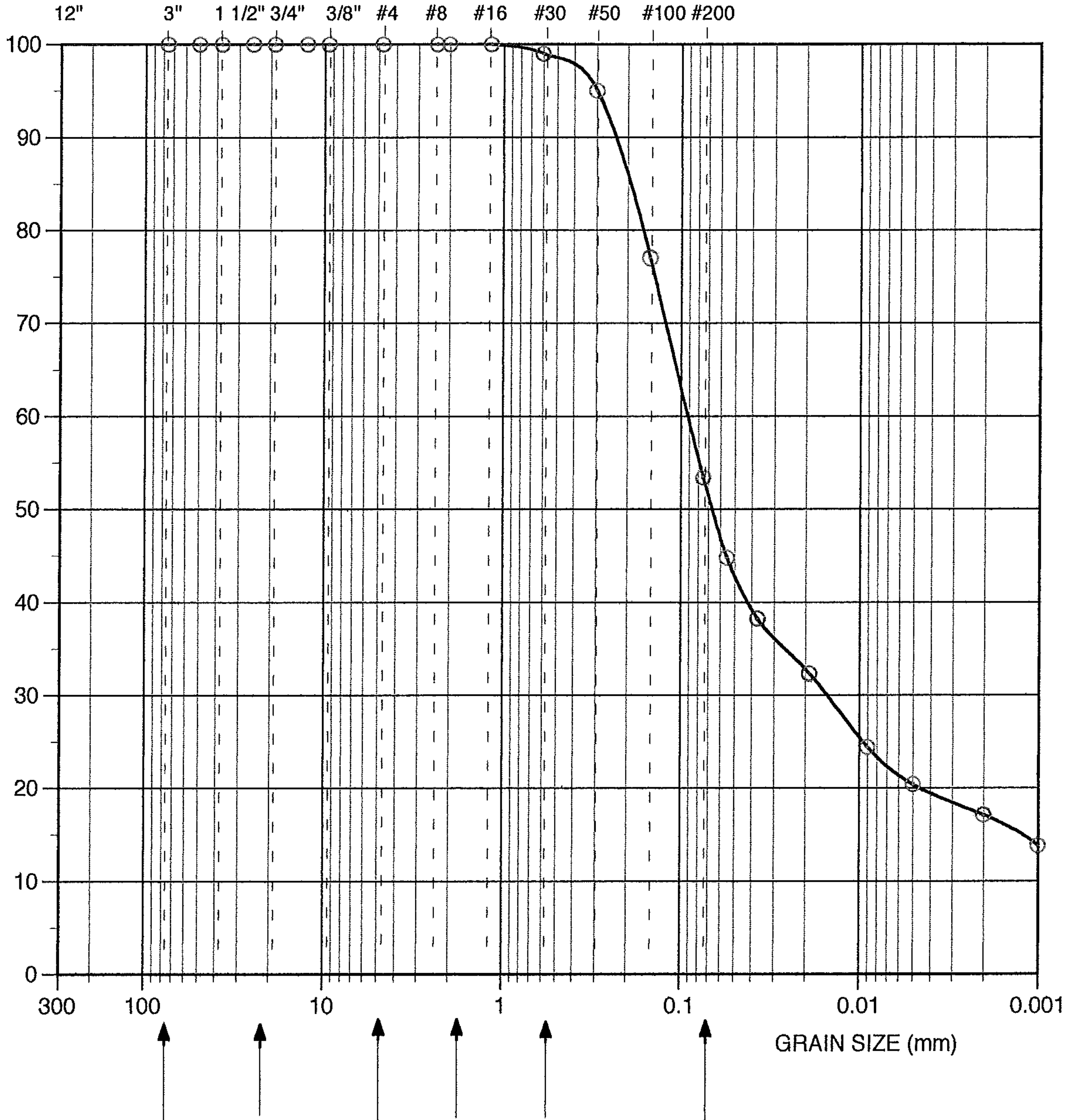
SYMBOLS	LOCATION	DESCRIPTION
⊙	B-8 at 16-1/2 to 18 feet	SILTY SAND, light brown with clay

GRADATION TEST DATA

JOB NUMBER: 2616.100

DATE: 5-1-03

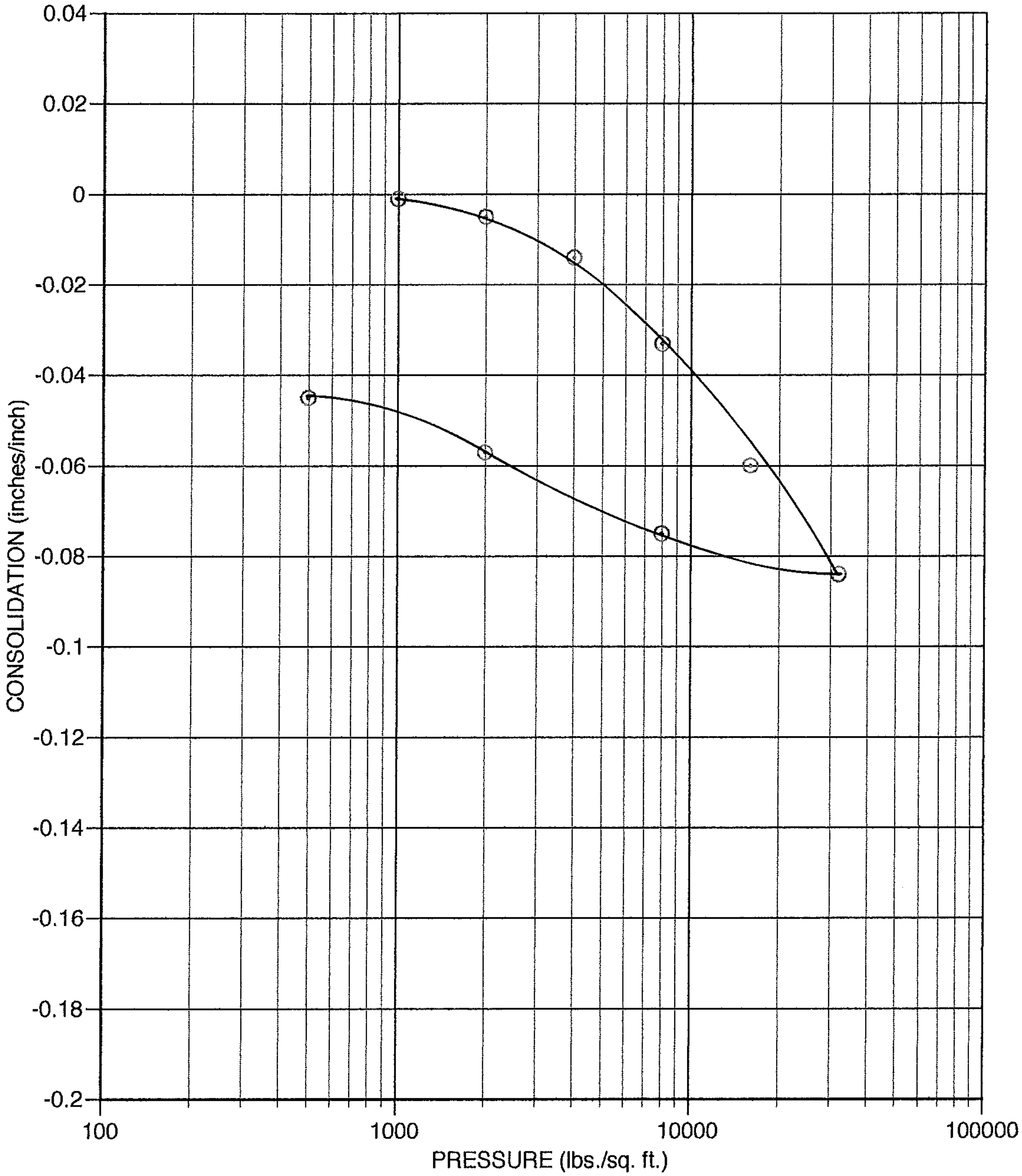
BY: FF



COBBLES	GRAVEL		SAND			SILT/CLAY
	coarse	fine	coarse	medium	fine	

SYMBOLS	LOCATION	DESCRIPTION
⊙	B-13 at 10-1/2 to 12 feet	SANDY CLAYEY SILT, red-brown

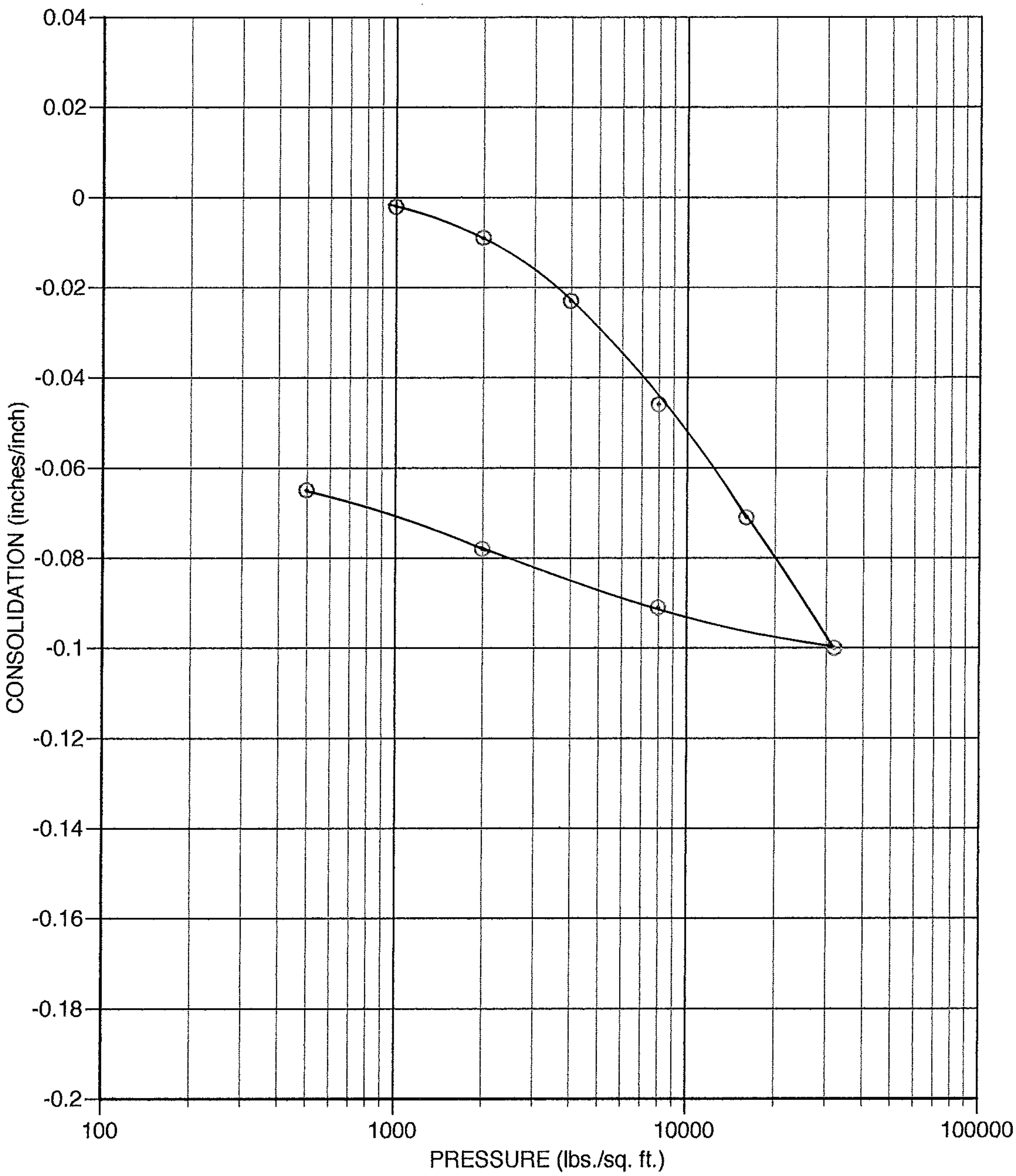
GRADATION TEST DATA



SYMBOL	LOCATION	DESCRIPTION	INITIAL MOISTURE CONTENT (%)	INITIAL DRY DENSITY (pcf)
⊙	B-1 at 5 feet	SILTY CLAY	19.1	109.5

CONSOLIDATION TEST DATA

JOB NUMBER: 2626.100 DATE: 5-1-03 BY:FF



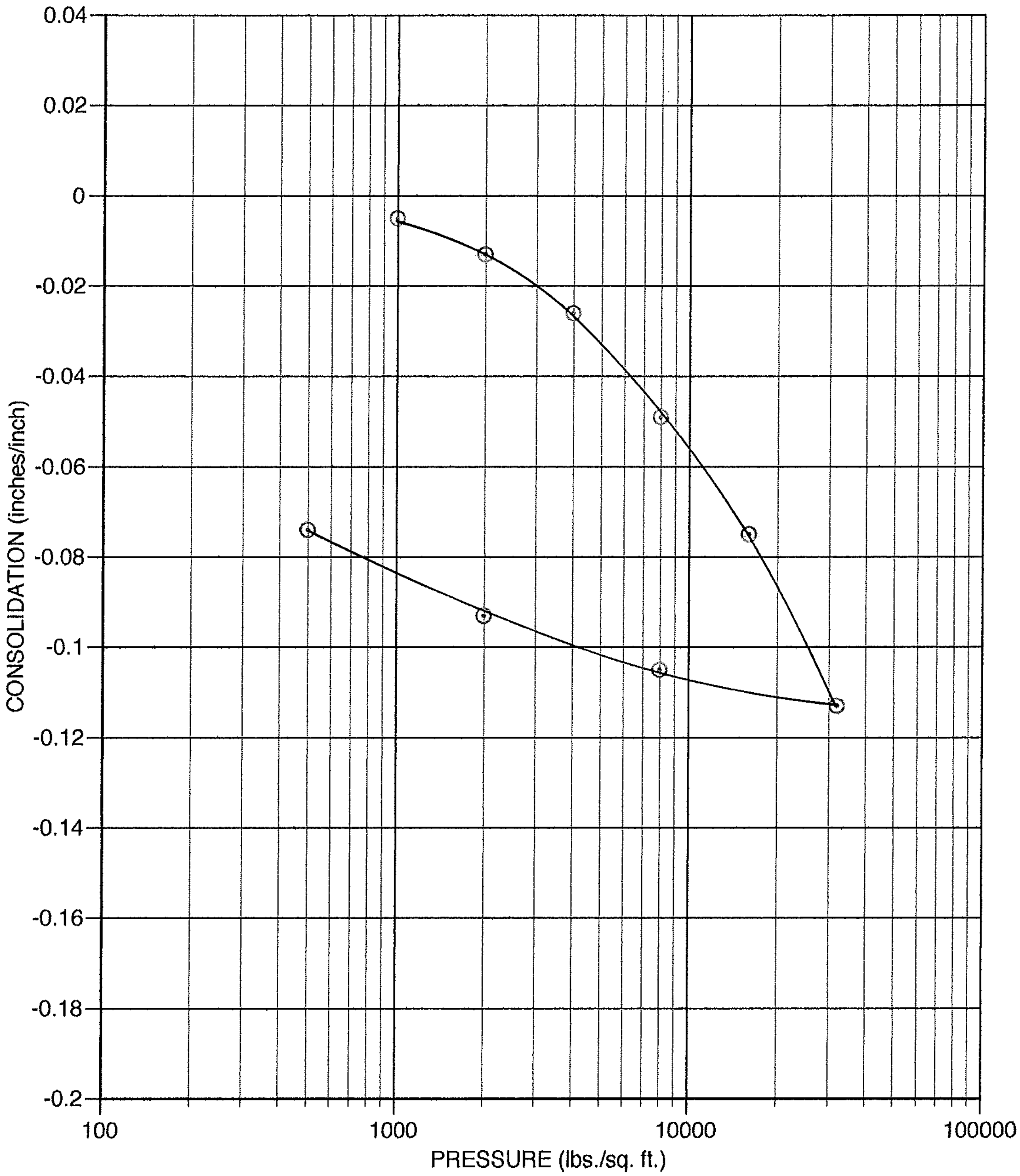
SYMBOL	LOCATION	DESCRIPTION	INITIAL MOISTURE CONTENT (%)	INITIAL DRY DENSITY (pcf)
⊙	B-1 at 10 feet	SANDY CLAY, yellow-brown	22.3	103.0

CONSOLIDATION TEST DATA

JOB NUMBER: 2626.100

DATE: 5-1-03

BY:FF



SYMBOL	LOCATION	DESCRIPTION	INITIAL MOISTURE CONTENT (%)	INITIAL DRY DENSITY (pcf)
⊙	B-9 at 4 feet	SANDY SILTY CLAY, dark brown	21.0	105.1

CONSOLIDATION TEST DATA

APPENDIX C

CERCO Analytical, Inc. Corrosion Test Data

California State Certified Laboratory No.2153

CERCO
analytical, inc.

3942-A Valley Avenue
Pleasanton, CA 94566-4715
Tel: 925.462.2771
Fax: 925.462.2775

24 April, 2003

Job No.0304121
Cust. No.10598

Mr. Paul Lai
Berlogar Geotechnical Consultants
5587 Sunol Blvd.
Pleasanton, CA 94566

Subject: Project No.: 2616.100
Project Name: UOP Property
Corrosivity Analysis – ASTM Test Methods

Dear Mr. Lai:

Pursuant to your request, CERCO Analytical has analyzed the soil samples submitted on April 11, 2003. Based on the analytical results, a brief corrosivity evaluation is enclosed for your consideration.

Based upon the resistivity measurements, Sample No.001 is classified as "corrosive" and Samples No.002 and No.003 are classified as "moderately 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 concentrations range from none detected to 76 mg/kg. Because the chloride ion concentrations are less than 300 mg/kg, they are determined to be insufficient to attack steel embedded in a concrete mortar coating.

The sulfate ion concentrations reflect none detected with a detection limit of 15 mg/kg and are determined to be insufficient to damage reinforced concrete structures and cement mortar-coated steel at these locations.

The pH of the soils range from 6.0 to 8.5 which does not present corrosion problems for buried iron, steel, mortar-coated steel and reinforced concrete structures. However, any soils with a pH of <6.0 are considered to be corrosive to buried iron, steel, mortar-coated steel and reinforced concrete structures, and corrosion prevention measures will need to be considered for structures to be placed in acidic soils.

The redox potentials range from 290 to 400-mV, which are 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.

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

JDH/jdl

CERCO Analytical, Inc.

3942-A Valley Avenue, Pleasanton, CA 94566-4715 (925) 462-2771 Fax (925) 462-2775

FINAL RESULTS

Client:	Berlogar	Date Sampled:	31-Mar-2003
Client's Project No.:	2616.100	Date Received:	11-Apr-2003
Client's Project Name:	UOP Property	Date of Report:	24-Apr-2003
Authorization:	Signed Chain of Custody	Matrix:	Soil

Job/Sample No.	Sample I.D.	Redox (mV)	pH	Conductivity (umhos/cm)*	Resistivity (100% Saturation) (ohms-cm)	Sulfide (mg/kg)*	Chloride (mg/kg)*	Sulfate (mg/kg)*
0304121-001	TP2-13 @ 6-9'	300	8.5	-	1,200	-	76	N.D.
0304121-002	TP2-15 @ 4-8'	290	8.4	-	2,500	-	N.D.	N.D.
0304121-003	TP2-19 @ 1-9'	400	6.0	-	4,800	-	N.D.	N.D.

	Redox	pH	Conductivity	Resistivity	Sulfide	Chloride	Sulfate
Method:	ASTM D1498	ASTM D4972	ASTM D1125M	ASTM G57	ASTM D4658M	ASTM D4327	ASTM D4327
Detection Limit:	-	-	10	-	50	15	15
Date Analyzed:	17-Apr-2003	18-Apr-2003	-	23-Apr-2003	-	18-Apr-2003	18-Apr-2003

* Results Reported on "As Received" Basis

N.D. - None Detected

Cheryl McMillen

Cheryl McMillen
Laboratory Director

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

APPENDIX D

Responses to Treadwell & Rollo and Haley & Aldrich Comments

APPENDIX D

RESPONSES TO TREADWELL & ROLLO AND HALEY & ALDRICH COMMENTS

RESPONSE TO TREADWELL & ROLLO COMMENTS

COMMENT 1 – The adequacy of the headwall behind Lots 78 and 79 to handle debris flows.

Debris flows in this area are no longer a concern because this portion of the site will not be developed and will be open space.

COMMENT 2 – Discuss the use of rock particles up to 12 inches in size in engineered fill and compaction criteria for fills up to 32 feet thick.

The maximum allowable fragment dimension for use in engineered fill has been reduced from 12 inches to 6 inches. Maximum fill depths are about 15 feet, 90 percent relative maximum compaction should be satisfactory. Adequate soil compaction should be achievable without special equipment.

COMMENT 3 – The seismic performance of proposed fill slopes should be evaluated and potential impacts to foundations.

Slope stability analyses were performed using Geo-Slope International Ltd. Slope/W program using the Morgenstern-Price method. The following are the shear strength parameters utilized in the slope stability analyses.

Material	Density, pcf	Friction Angle, degrees	Cohesion, psf
Bedrock	125	30	1000
Engineered Fill	120	20	500

The following table presents the results of our slope stability analysis. PT slab foundations are suitable for the site conditions.

	Safety Factor	Safety Factor w/ Seismic Conditions
60 foot Cut Slope	2.7	1.5
25 foot Fill Slope	2.6	1.4

The pseudostatic factor to be applied was determined in accordance with Special Publication 117A, Guidelines for Evaluating and Mitigating Seismic Hazards in California, California Geologic Survey, 2008. A pseudostatic factor, Keq of 0.28 was determined utilizing the Chart on page 30 for a 5 cm threshold displacement, a magnitude of 7.0, distance of 8.9 km, and a 0.53 g maximum horizontal acceleration (PGA from the 2013 CBC).

COMMENT 4 – The differential settlement between the upper and lower split level pads should be determined and if a seismic load should be applied to the retaining walls between the split levels.

We performed settlement analysis for the upper and lower pad for the proposed split lot residences. We estimate the potential differential settlement to be ½ inch. A seismic increment for retaining walls over 6 feet tall is included the retaining wall section of the report.

COMMENT 5 – Recommendations for mat slab embedment should be provided to reduce the potential for surface water intrusion beneath the mat slab.

We have included a perimeter turndown in our revised recommendations for the mat foundations. It is not common practice to embed post-tensioned slab foundations because they are designed to primarily resist lateral loads based on base friction. Additionally, PT slab foundations are constructed directly upon the prepared subgrade with no gravel layer (which could transmit water beneath the PT slab).

COMMENT 6 – A ground floor system should be recommended for the structures with pier and grade beam foundations.

The proposed split-level residences are planned to use a wood flooring system supported on the drilled pier and grade beam foundation system.

COMMENT 7 – Scour should be evaluated for bridge foundations.

The comment is not applicable since a bridge for vehicle crossing is no longer planned.

COMMENT 8 – The closest distance to nearest faults should be clarified and provide the appropriate ARS curves for bridge design.

We have provided updated seismic design parameter in the Seismic Design Parameter section. ARS curves are not necessary because the vehicular bridge is no longer planned.

COMMENT 9 – A qualified engineering geologist or geotechnical engineer should be retained during site grading.

We concur with T&R's comment

COMMENT 10 – Technical input should be provided during design and construction of surface and subsurface drainage systems and the homeowners association should be responsible for periodic maintenance.

We have provided subsurface drainage recommendation in our report and on the two remedial grading plans. Surface drainage should be designed by qualified personnel retained by the project developer. We concur with T&R's comment that the homeowners' association or similar entity should be responsible for inspection and maintenance.

RESPONSES TO HALEY & ALDRICH COMMENTS

CEQA Comment 1 – The downhill fill on the south side of Windsor Drive should be evaluated (to determine impacts from development on Windsor Drive).

Windsor Drive is outside the proposed improvement limits of the project. However, minor grading, with sliver cuts and fills up to 2 feet thick, are proposed on the downhill side of the southern Windsor Drive fill slope behind Lots 17 to 20. Windsor Drive was constructed in 1993 according to Google Earth imagery. Longitudinal cracks (with weeds) in the AC pavement totaling up to 1 inch of total lateral movement were observed in an approximate 400 foot long section on the downhill edge. Alligator cracking, rutting and previous trench repairs were also observed on Windsor Drive in this area.

We excavated 4 test pits in the southern fill slope behind Lots 17 through 20 as discussed with H&A and the City in a teleconference. The locations of the test pits are shown on Plates 2 and 3, Geologic Maps for options A and B, respectively. Graphic test pit logs for these test pits are shown on Plate 11. Results of the laboratory testing, including in-situ moisture/density, Atterberg Limits, and single point consolidation, are also shown on Plate 11.

The fill slope material was found to have relatively low expansion potential with PI's of 9 and 13. Hence, soil creep is not likely occurring along this section of Windsor Drive. The fill material was found to be relatively loose. We have recommended that the outer approximate 5 feet of this fill slope behind Lots 17 through 20 be reconstructed by benching and fill placement during mass grading. Plate 11 shows the suggested benching and recompaction of the fill slope in the locations of the 4 test pits.

CEQA Comment 2 – The distance to the closest faults should be clarified and modify seismic design parameters as needed.

See Treatment of Fault Section in the report. Seismic design parameters do not need to be modified.

CEQA Comment 3 – Plates 2 and 3 show a below grade MSE wall below Landslide A, but is not discussed in the report.

The below grade MSE wall has been removed since site development plans were modified. The wall shown previously on Plates 2 and 3 were for an older development plan.

CEQA Comment 4 – Clarify the largest dimension of 6 and 12 inches.

Engineered fill should not contain particles larger than 6 inches. Larger particles should either be broken down or removed from engineered fill.

<u>CEQA Comment 5 – Temporary seismic or wind load factor should be provided in the foundation recommendations table.</u>

This factor is now included in the table.

<u>CEQA Comment 6 – Test pits from the 2002 investigation should be included and the test pit logs for the 2004 investigation mixes up feet and inches.</u>

Test pit logs from the 2002 investigation are included as pages A-32 through A-38 in Appendix A. The typographical errors contained in the 2004 logs are corrected.

<u>CEQA Comment 7 – Debris flow hazards previously discussed be shown on the geologic map and the impacts and mitigation measures from debris flows should be evaluated.</u>

Our previous discussions on debris flow hazards consisted of site reconnaissance and review of topographic maps and historical aerial photographs. A more detailed discussion of this contained in the Debris Flow/Sedimentation section of this revised report. Debris flows were not mapped previously, hence, are not shown on the geologic maps.

<u>CEQA Comment 8 – The seismic stability of the partially repaired Landslide A should be evaluated and the risk to the proposed development.</u>

The average thickness of Landslide A is estimated to be 12 feet. Landslide A is a combination of several small slumps that occurred over time as the ground became oversaturated and perhaps triggered by seismic events (when the ground was wet). The topographical relief indicates that the slumps do not travel far down slope, rather soil at the toe builds up at most about 3 feet in thickness. It is our opinion that small slump failures will continue in the upper portion of the landslide when the ground becomes oversaturated. However, these small slumps will override the lower remediated portion for only a short distance. This area should be included in the periodic annual inspections and maintenance to be performed by the homeowner’s association.

<u>CEQA Comment 9 – The sum of total and differential settlement, including fill placed over potentially compressible material, should be addressed.</u>

It is our opinion that the estimated swell of the compacted fill combined with the settlement of the compacted fill and underlying colluvium and alluvium may produce a total settlement of approximately 1 inch, which is judged to be insignificant.

<u>CEQA Comment 10 – Mat foundation embedment and the potential for water intrusion beneath the mat should be discussed.</u>

We have included a perimeter turndown in our revised recommendations for mat foundations. It is not common practice to embed PT slab foundations. Water intrusion is not a concern since the PT slab sets directly upon the subgrade soil and the PT slab is underlain by a vapor retarder. The PT slabs design parameters are provided in accordance with the recommendations from the Post Tension Institute.

<u>CEQA Comment 11 – A keyway subdrain is not shown on section AA' on Plate 5 and correct the lot numbers depicted on sections DD' and EE'.</u>

A keyway subdrain cannot be installed in the keyway since the elevation of the keyway bottom is too high for gravity drainage. Instead, we have shown additional subdrains above the keyway to collect subsurface water. It should be noted that subdrains are typically field located during grading, and the subdrains shown on the plans and cross sections are for information only. As-built drawings showing the subdrains are typically made during grading. The keyways will be constructed with engineered fill from the site with a PI less than 20. Lot depictions have been corrected on the affected sections.

<u>CEQA Comment 12 – A berm is shown on Plate 8 at the top of a fill slope detail and the ground surface at the top of fill slopes should be checked for proper drainage.</u>

The top of fill slopes are typically graded flat during mass grading. Final drainage and grades are constructed during vertical construction. The fill slope detail has been clarified.

<u>CEQA Comment 13 – The impacts of failure of the small stock pond should be evaluated.</u>

It is our understanding that BKF addresses this comment in their letter dated June 6, 2015. BKF states "…the release of pond water would not adversely impact the planned development."

<u>CEQA Comment 14 – The two drainage channels extend down to the north on the south side of Kelly Creek should be evaluated for potential impacts to development. Fill placed potentially under and around the stockpond berm should also be evaluated.</u>

The western drainage channel is in open space, and development will not impact this drainage channel. If the stock pond berm were to fail, the water within the stock pond would not adversely impact the development according to BKF (see Comment 13 above). The eastern drainage channel will be filled in the development area, and the water will be routed into storm drain inlet structures. Since the water that used to flow down the drainage channel will be collected and redirected, the erosion occurring at the base of the drainage channel at Kelly Creek should be nearly eliminated. The homeowners assocation should be responsible for periodic inspection and maintenance of potential erosion areas.

<u>General Comments 1 through 7 – Typographical errors and inconsistencies were pointed out in the body of the report.</u>

These have been corrected.

General Comment 8 – Pier and grade beam residences will have raised wood floors, and if not, recommendations for slab on grade floors should be provided.

Pier and grade beam residences will all have raised wood floors and mat slab and PT slab foundation recommendations have been included in the geotechnical report.

General Comment 9 – Design recommendations for potential bridges or culverts for the pedestrian trail should be provided.

Once improvement plans have been finalized, we can provide the necessary recommendations as requested.

General Comment 10 – Discuss settlement between the upper and lower pads for split level lots. Recommendations for seismic increment for retaining walls should be provided. Water proofing and subdrainage recommendations for interior retaining walls should also be provided.

We have estimated that there is a potential for approximately ½ inch of differential settlement between the upper and lower levels for the split pad lots. It is our understanding that the foundations for residences on split level pads will be supported on drilled, reinforced concrete piers. Hence, differential settlement should not be a concern. Currently, the split lots will have graded slopes between the two levels. Seismic increment is provided in the retaining wall section of this report for retaining walls taller than 6 feet. We do not typically provide waterproofing recommendations since this is not our area of expertise. Locations of retaining wall subdrains are determined on a case by case basis, and are typically either on top of or next to the retaining wall foundations.

Via Email Only

July 3, 2018
Job No. 2616.009

BERLOGAR
STEVENS &
ASSOCIATES

Mr. Steve Abbs
Davidon Homes
1600 South Main Street, Suite 150
Walnut Creek, California 94596

Subject: Grading Exhibit Review
Scott Ranch
Petaluma, California

Dear Mr. Abbs:

We have received the following plan:

> "Grading Exhibit dated June 4, 2018, Davidon Homes/Scott Ranch by BKF, scale 1" =60'"

We conclude that the Grading Exhibit is in substantial conformance with the following report:

> "Revised Design-Level Geotechnical Investigation dated September 22, 2004, Berlogar Geotechnical Consultants"

Respectfully submitted,

BERLOGAR STEVENS & ASSOCIATES

REGISTERED PROFESSIONAL ENGINEER
FRANK BERLOGAR
No. 20383
CIVIL
STATE OF CALIFORNIA

Frank Berlogar
RCE 20383

FB:mc

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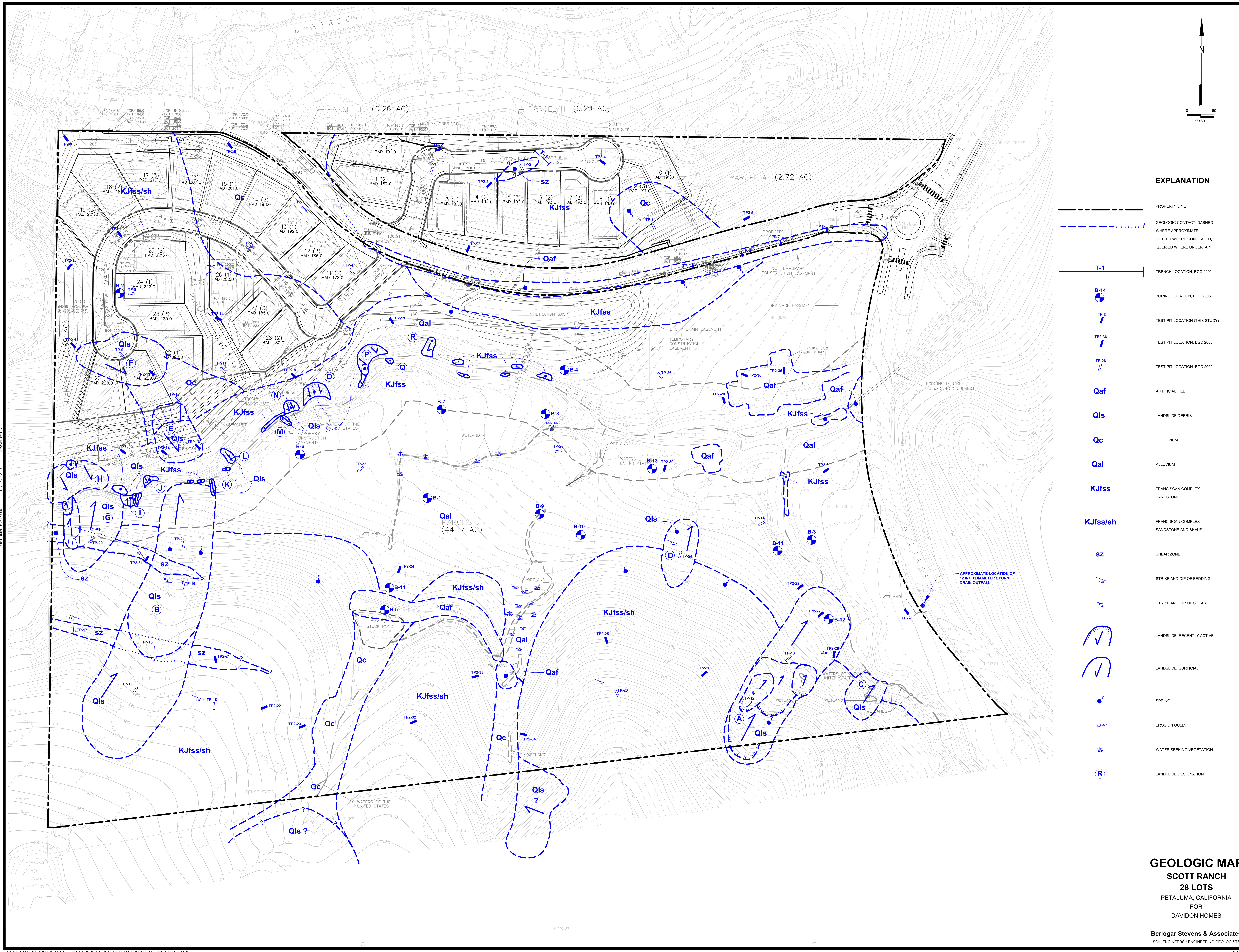
EXPLANATION
PROPERTY LINE
GEOLOGIC CONTACT, DASHED WHERE APPROXIMATE, DOTTED WHERE CONCEALED, QUERIED WHERE UNCERTAIN
TRENCH LOCATION, BGC 2002
BORING LOCATION, BGC 2003
TEST PIT LOCATION (THIS STUDY)
TEST PIT LOCATION, BGC 2003
TEST PIT LOCATION, BGC 2002
Qaf ARTIFICIAL FILL
Qls LANDSLIDE DEBRIS
Qc COLLUVIUM
Qal ALLUVIUM
KJfss FRANCISCAN COMPLEX SANDSTONE
KJfss/sh FRANCISCAN COMPLEX SANDSTONE AND SHALE
sz SHEAR ZONE
STRIKE AND DIP OF BEDDING
STRIKE AND DIP OF SHEAR
LANDSLIDE, RECENTLY ACTIVE
LANDSLIDE, SURFICIAL
SPRING
EROSION GULLY
WATER SEEKING VEGETATION
LANDSLIDE DESIGNATION
GEOLOGIC MAP
SCOTT RANCH
28 LOTS
PETALUMA, CALIFORNIA
FOR
DAVIDON HOMES
Berlogar Stevens & Associates
SOIL ENGINEERS * ENGINEERING GEOLOGISTS
BASE: TITLED, "REVISED PROJECT - 28 LOTS PROPOSED GRADING PLAN", PREPARED BY BKF, DATED 2-11-19
PLATE 2
APPROXIMATE LOCATION OF 12 INCH DIAMETER STORM DRAIN OUTFALL
PARCEL B (44.17 AC)
PARCEL A (2.72 AC)
WINDSOR DRIVE
D STREET