Appendix C

Geotechnical Report and Peer Review

UPDATED GEOTECHNICAL REPORT

OAK CREEK CANYON
5 LOTS - SUBDIVISION 6826
APN #119-070-008

CLAYTON, CALIFORNIA

SUBMITTED

TO

WEST COAST HOME BUILDERS INC.

CONCORD, CALIFORNIA

PREPARED

BY

ENGEO INCORPORATED

PROJECT NO. 3840.205.202

FEBRUARY 22, 2008

ENGEO INCORPORATED

GEOTECHNICAL
ENVIRONMENTAL
WATER RESOURCES
CONSTRUCTION SERVICES

Project No.
3840.205.202

February 22, 2008

Mr. Albert Seeno III
West Coast Home Builders, Inc.
4021 Port Chicago Highway
Concord, CA 94524-4113

Subject: Oak Creek Canyon
5 Lots - Subdivision 6826
APN #119-070-008
Clayton, California

UPDATED GEOTECHNICAL REPORT

Dear Mr. Seeno:

At your request and with your authorization, this report contains the results of our updated geotechnical report presenting our conclusions and recommendations regarding the current proposed development in Clayton, California.

It is our opinion that the proposed development is feasible from a geotechnical standpoint provided that the recommendations contained herein are incorporated into the project plans and implemented during construction. We are pleased to be of service to you on this project and will continue to consult with you and your design team as project planning progresses.

Very truly yours,

ENGEO Incorporated

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jb/tpb/mb: supplemental exploration

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INTRODUCTION

Purpose and Scope

This report is intended to provide supplemental exploration to address geologic and geotechnical peer review comments by James Joyce Associates on behalf of the City of Clayton, as well as provide an update to our previous work regarding geotechnical aspects for the current planned site development.

The scope of our services has included the following:

1. Review of previously published maps and reports regarding geological and geotechnical characteristics, and presence of landslides at the subject site and nearby properties.
2. Review of stereographic aerial photographs covering the site.
3. Excavation and logging of exploratory test pits and trenches.
4. Sampling and laboratory testing of subsurface materials.
5. Analysis of the geological and geotechnical data.
6. Preparation of this report summarizing our findings and geotechnical design recommendations.

This report was prepared for the exclusive use of West Coast Home Builders Inc. and their design team consultants. In the event that any changes are made in the character, design, or layout of the development, the conclusions and recommendations contained in this report should be reviewed by ENGEO to determine whether modifications to the report are necessary. This document may not be reproduced in whole or in part by any means whatsoever, nor may it be quoted or excerpted without the express written consent of ENGEO Incorporated.

Site Location and Description

The approximate 6.5-acre site is situated along the north side of Marsh Creek Road and Diablo Parkway in Clayton, California (Figure 1). The site is further bounded by a private driveway to the east, and Contra Costa County water tank property to the northwest and open space up slope to the north. Currently, the water tank access road is situated across the western portion of the site. The triangular-shaped parcel generally slopes from north to south, with a level fill area constructed along the southeast corner of the property, and an existing swale traversing the property in a northeast-southwest direction between the level fill area and the slope. At the time of our field exploration, the subject site was open space used for cattle grazing. Site vegetation consisted of native grasses.

It is our understanding that a Getty Oil pipeline exists along the eastern boundary of the property and an abandoned pipeline runs east-westerly across the upper portions of the slope in the northern portion of the site. Representatives of Shell Pipelines informed us of another pipeline adjacent to the property along Marsh Creek Road. According to the tentative site plan, setbacks will be provided for these areas.

Previous Work by ENGEO

ENGEO Incorporated previously conducted subsurface exploration at the Oak Creek project site that included 5 lots (formerly known as Oak Glen Property) in 1994, which included the drilling 1 auger boring (Figure 4). In December 1997, ENGEO performed supplemental subsurface investigation which consisted of drilling 4 additional auger borings (see References). The 1997 report was updated again in 2001 (Reference 11). This study updated the geologic and geotechnical data and provided updated geologic mapping, as well as updated recommendations. In 2006, updated remedial grading recommendations based on revised grading plans provided by Isakson and Associates were provided along with revised remedial grading plan (Reference 12). The City of Clayton's peer reviewer, Joyce Associates (JA), advised further characterization of the sites geology is warranted, including the existence of mapped Nilsen slide, the shallow mapped slide above Lots 1 and 2, and the geologic characteristics of the site bedrock. This report is intended to provide an update of previous findings, and address the comments provided by James Joyce Associates, and the recommendations in this report supersede those in all previous reports. We reviewed the previous reports and have incorporated data from those reports in our findings and conclusions, as appropriate.

Proposed Development

Based on grading plans by Isakson and Associates dated April 18, 2006, the current proposed development includes a 5-lot residential subdivision with interior subdivision roads and utilities servicing the development with a detention pond located in the southeast portion of the site. The majority of the development areas will have cut and fill slopes graded at 2:1. Lots 2 through 5 are flat lots. Lot 1 is a split lot with an 8 foot high 2:1 (horizontal to vertical) slope between the upper and lower pads. Lots 1 through 5 are cut/fill transition lots and Lot 1 is a fill lot. However, after the removal of the landslide material in the vicinity of Lots 1 and 2, Lot 2 will only require fill to achieve design grades. Slopes are generally 2:1 slopes up to 15 feet in height. Retaining walls are planned at the toe of slope in the rear portion of the lots. Cuts for the planned detention basin are approximately 5 feet.

REGIONAL GEOLOGY AND SEISMICITY

Regional Geology

The geologic deposits at the site are mapped as part of the Panoche Formation (Kp), Figure 2. These deposits typically consist of micaceous clay shale interbedded with sandstone (Dibblee, 2006). Surficial deposits along the eastern portion of the site are mapped by Dibblee as alluvium (Qa). Nilsen (1975) had mapped a landslide deposit covering the majority of the site with the eastern portion of the site consisting of a colluvial deposit or small alluvial fan deposit (Figure 3). The mapped Nilsen landslide has two main lobes, with the western lobe encompassing the ridge on the western portion of the site with the water tank, and the eastern lobe encompassing the less prominent ridge located in the center of the site. We did not find evidence of a landslide in the vicinity of the eastern lobe in our review of stereo aerial photographs or during our site visit. During our review of aerial photographs for the western lobe of the mapped landslide, we observed topographic features which could be indicative of an ancient landslide. However, these features could also be related to differential weathering of the bedrock.

The USDA Soil Conservation Services has classified the soil on the northern portion of the subject property as belonging to the Los Osos Series. These soils typically are low strength and consist of well-drained soils underlain by soft, fine-grained sandstone and shale. The USDA also characterized the Los Osos Series with a high shrink-swell potential, moderate to high erosion, and low permeability. The soils along the southwest and southeast portions of the property are classified by the USDA as belonging to the Capay Series and Perkins Series. These soils generally form in alluvial areas and have a moderate to high shrink/swell potential and are typically low to medium strength soils. The USDA describes these soils as having a high corrosivity to uncoated steel.

Faulting and Seismicity

The site is not located within an Alquist-Priolo Earthquake Fault Zone; however, large (>M6) earthquakes have historically occurred in the San Francisco Bay Area and many earthquakes of low magnitude occur every year. No active faults are known to pass through the project site, according to published geologic maps (Dibblee, 2006; Crane, 1988). The nearest active fault is the Greenville fault located approximately 1 mile southeast of the project site, which is capable of a maximum probable earthquake Richter magnitude of 6.9 with a maximum probable ground acceleration of 0.57g at the site (Blake, 1994). The Concord fault is located approximately 4 miles southwest of the site, and is capable of a maximum probable ground acceleration of 0.40g at the site. Other active faults in the San Francisco Bay Area capable of producing significant ground shaking at the site include the Calaveras fault, 10 miles southwest; the Cordelia fault, 22 miles northwest; the Green Valley Fault, 14 miles northwest, the Hayward fault, 17 miles west; and the San Andreas fault, 35 miles west.

The United States Geologic Survey has evaluated the Bay Area seismicity through a study by the Working Group on California Earthquake Probabilities (WGCEP, 2003). In their study, the WGCEP evaluated the 30-year probability of M6.7 or greater earthquakes in the Bay Area. According to their conclusions, the Bay Area has a 30-year probability of 62 percent for such an event. The Hayward - Rogers Creek and the Concord - Green Valley faults were assigned a 30-year probability of 27 percent and 4 percent, respectively. It should, therefore, be expected that the site will experience one or more episodes of strong ground shaking during the design life of the proposed improvements.

Clayton Fault. According to the Seismic Safety Element for Contra Costa County (1975), the Clayton fault is shown to dip easterly at approximately 70 degrees, with an east-side thrusting over the west block. Several studies have been performed on the nearby Clayton fault. Dibblee, 1980, shows the Clayton fault approximately 500 feet north of the northern boundary of the project.

According to previous site work performed by Brabb, et al., 1971, the Clayton fault is located approximately 2,000 feet north of the northern boundary of the project. A later study provided by Woodward-Lungren, 1974, mapped the possible southern limit of the Clayton fault at Marsh Creek Road, in a northwest-southeast line of projection along the western edge of the Contra Costa County reservoir.

An extensive study provided by Purcell, Rhoades & Associates in their 1978 soil and geological investigation for the neighboring Regency Meadows project south of Marsh Creek Road included the excavation of several trenches to determine the southern limits and location of the Clayton fault. Their findings did not indicate any signs of faulting on the proposed Regency Meadows development.

An independent study was concurrently performed by Purcell, Rhoades & Associates in 1978, which included the excavation of a trench along the northwestern boundary of the proposed Oak Creek Canyon (then Oak Glen) development, with the southeastern limits of the trench located at the rear of the Contra Costa County reservoir building pad. The results of this study indicated that the original fault delineation for the Clayton fault prepared by Woodward-Lundgren in 1974 did not extend into the proposed Oak Creek Canyon development, but rather followed either the orientation determined by Brabb, et al. in 1971, or extended further west at the base of the hills of the Keller Ranch property.

SUPPLEMENTAL FIELD EXPLORATION

To address several peer review comments by James Joyce Associates, ENGEO performed a supplemental field exploration on November 30 and 31, 2007. This exploration consisted of logging an additional 6 exploratory test pits and two exploratory trenches at the site. The approximate exploration locations of the test pits and trenches are shown on Figure 4. These locations were predetermined and reviewed by JA prior to field work. JA was consulted on the location of additional test pits and trench performed during the course of the field exploration. The test pits and trenches were located by pacing from existing features and the locations should be considered accurate to the degree implied by the method used.

The test pits were excavated throughout the site to a maximum depth of 13 feet at the locations shown on Figure 4. An ENGEO geologist logged the excavations. The test pits and trenches were excavated with an excavator equipped with a 30-inch bucket. The logs depict subsurface conditions within the test pits and at the time the exploration was conducted. Subsurface conditions at other locations may differ from conditions noted at these locations. In addition, stratification lines represent the approximate boundaries between soil types and the transitions may be gradual. The test pit and trench logs are presented in Figures 6 and 7.

LABORATORY TESTING

Following excavation, we reexamined the samples in our laboratory to confirm field classifications. Representative samples recovered from test pits were tested for the following physical characteristics:

Characteristic	Test Method	Location of Results Within this Report
Atterberg Limits	ASTM D-4318	Appendix A

Laboratory test results from samples recovered during our subsurface exploration of the site are included on the boring logs and in Appendix A as noted above. Laboratory testing from previous explorations has also been incorporated into our conclusions and recommendations where appropriate.

FINDINGS

Subsurface Conditions

Panoche Formation (Kp) - Bedrock at the site comprises interbedded sandstone, siltstone, and claystones of the Cretaceous Panoche Formation. In general, the sandstone is well cemented, moderately strong to strong, massive to laminated, orange brown where weathered. Siltstone is generally dark gray brown to orange brown, friable to moderately strong, and thin bedded. Claystone encountered is dark gray, friable to moderately strong, preferentially sheared, and thin bedded. Bedding observed in the test pits and trenches throughout the site ranged from a strike of S89W to N36W and dipping 10 to 50 degrees to the north or northeast.

Existing Fill (Qaf) - Existing man-made fills materials have been imported and placed in the lower lying flat portion of the site. Some of this material was placed as engineered fill and tested by ENGEO in 1995 in the southeast portion of the site as shown on the site geologic map. As of our final testing and observation report, the pad fills had not been completed. Of the planned fills, approximately 4 feet had been placed. A keyway and drain were constructed along the southern edge of the fill slope, draining to the ditch at the south western boundary of the site.

Alluvium (Qal) - The swale in the southeast portion of the site and the imported fills in the vicinity of the proposed detention basin are underlain by alluvium. Our previous explorations revealed several feet of existing fill are underlain by moderately expansive silty clay ranging from 2 to25 feet below ground surface. ENGEO drilled one boring associated with our 1994 exploration (Reference 9). The boring ended in alluvium at a depth of 26.5 feet. Bedrock was not encountered in the boring

Residual Soil and Colluvium (Qc). The site bedrock is typically mantled with about 2 to 3½ feet of residual soil formed from weathering and decomposition of the underlying bedrock. The residual soil and colluvial soils generally consist of silty clay with varying sand; these soils are moderate to high in plasticity and considered highly expansive.

Deposits of soils exceeding 3½ feet have been designated as colluvium (Qc) and these occur in the swales and ravines and at the base of the slope in the vicinity of Lots 3, 4, and 5. Colluvium is a soil deposit formed from downslope movement and deposition of residual soil by such processes as slope-wash, sloughing/shallow sliding, and creep. Soil creep is the slow, nearly continuous downhill movement of the soil mantle on steep terrain induced by gravity and moisture-related volume changes. Several of the test pits excavated in swale areas across the site encountered colluvium to depths ranging from 4 to 7 feet. The colluvium typically consists of silty clay or clayey silt with occasional scattered rock fragments.

Landslide (Qls). As previously discussed a large landslide was mapped at this site by Nilsen, (Figure 3), which was discussed in References 10 and 11 by ENGEO. A principal focus of this current supplemental exploration was to further characterize site conditions to determine if there was any evidence of the mapped Nilsen slide. In Reference 11, ENGEO had identified a relatively shallow landslide involving soil landslide debris in the western swale above Lots 1 and 2. Trenches T-1, T-2, and test pits TP-2, TP-3, and TP-4 were excavated near the limits of the previous postulated large slide as shown on Figure 4. We encountered soil to a depth of up to 8 feet in our trenches and test pits overlying bedrock units. Cross-Section A-A' on Figure 8 drawn longitudinally through the shallow soil landslide depicts the probable geometry of the slide feature.

As discussed in Reference 11, we did not find evidence of the postulated large ancient landslide mapped by Nilsen in our review of stereo aerial photographs, or during this or our previous explorations. To resolve peer review comments about whether or not there exists evidence of the postulated Nilsen landslide, ENGEO performed two exploratory trenches at the limits of the mapped feature at the approximate location shown on Figure 4. Both trenches encountered bedrock units of moderately weathered, and moderately to highly fractured claystone and siltstone interbedded with fine to medium grained, moderately to highly weathered sandstone typical of the Panoche Formation. The strike of bedding in trench T-1 ranged from N36W to N65W, dipping 10 to 38 degrees to the northeast. Increased weathering was noted from Stations 0+50 to 0+80 which coincided with the swale above the proposed Lots 1 and 2. In the same portion of the trench, the dip of bedding of the siltstone and sandstone became shallower, and we observed evidence of surficial expansive soil creep at the bedrock-soil contact. The strike of bedding in trench T-2 ranged from N55W to N62W; dipping from 35 to 39 degrees to the northeast. The bedding encountered in the trenches generally coincides with bedding observed in our exploratory test pits through out the subject property. We also observed continuous exposure of intact bedrock in both exploratory trenches. Based on the results of this supplemental exploration, we conclude that there is no evidence of the postulated large landslide feature mapped by Nilsen. Furthermore, during our supplemental trenching work, the City of Clayton contract geologic peer reviewer, Mr. Jim Joyce, CEG met with our Certified Engineering Geologist to observe the locations of and the conditions in the exploratory trenches and test pits; it was concurred by both ENGEO and Mr. Joyce that the length and locations of the trenches and test pits were adequate to determine there was no evidence of the deep-seated landslide as previously postulated by Nilsen.

Groundwater

Ground water was not encountered in the test pits or trenches at the time of excavation. Fluctuations in ground-water levels occur seasonally and over a period of years because of variations in precipitation, temperature, irrigation and other factors. Future irrigation may cause an overall rise in ground-water levels.

CONCLUSIONS

Based on our previous and current supplemental exploration, we conclude that the proposed development of site is feasible from a geotechnical standpoint. The recommendations included in this report, along with sound engineering practices, should be incorporated in the design and construction of the project.

Slope Stabilization Measures

ENGEO recommends that the surficial landslide and areas of colluvium mapped along slopes, in areas identified on Figures 4 and 5 be overexcavated and removed, and replaced with properly drained engineered fill. The location, extent and depth of the required overexcavation areas and anticipated subdrainage has been depicted on the Remedial Grading Plan (Figure 5). For clarity, remedial grading concepts are also depicted on the cross-sections provided in Figures 8 and 9.

Expansive Soils

The clayey soils at this site have Plasticity Indices (PI) ranging from 20 to 54, which indicates these are considered moderate to very high potential for expansion, shrink-swell behavior. Expansive soils shrink and swell as a result of Seasonal fluctuation in moisture content. This can cause heaving and cracking of slabs-on-grade, pavements, and structures founded on shallow foundations. Building damage due to volume changes associated with expansive soils can be reduced through proper foundation design. Successful construction on expansive soils requires special attention during construction. It is imperative that exposed soils be kept moist by watering for several days before placement of concrete. Mitigation measures should include the prevention of moisture variation.

Compressible Soils

During our field explorations, layers of soft, medium stiff to stiff clay and silty clay were encountered to depths between approximately 4 and 13 feet below existing grades; these layers were typically encountered in the swales in the western and eastern portion of the site and in the alluvium and imported fills in the southeastern portion of the site. The fine-grained deposits in these areas appear to be potentially compressible and could result in measurable consolidation settlements. Compressible soils should be removed and replaced prior to fill placement in these areas. The actual depth of removal of soft and compressible soils should be determined during grading by the Geotechnical Engineer.

Seismic Hazards

Potential seismic hazards resulting from a nearby moderate to major earthquake may include primary ground rupture, ground shaking, lurching, liquefaction, dynamic densification, lateral spreading, and earthquake-induced landsliding. These hazards are discussed below. Risks from seiches, tsunamis, and inundation due to embankment failure are currently considered low at the subject site.

Ground Rupture. No known seismogenic faults have been mapped within the Oak Creek Canyon project site; therefore, the potential for ground rupture is considered low. Sympathetic ground movements due to an earthquake on a nearby active fault are possible, but the risk is anticipated to be very minor.

Ground Shaking. An earthquake of moderate to high magnitude generated within the San Francisco Bay Region could cause considerable ground shaking at the site, similar to that which has occurred in the past. To mitigate the shaking effects, all structures should be designed using sound engineering judgment and the latest California Building Code (CBC) requirements as a minimum.

Seismic design provisions of current building codes generally prescribe minimum lateral forces, applied statically to the structure, combined with the gravity forces of dead-and-live loads. The prescribed lateral forces are generally considered to be substantially smaller than the equivalent forces that would be associated with a major earthquake. Structures should be able to: (1) resist minor earthquakes without damage, (2) resist moderate earthquakes without structural damage but with some nonstructural damage, and (3) resist major earthquakes without collapse but with some structural as well as nonstructural damage. Conformance to the current building code recommendations does not constitute any kind of guarantee that significant structural damage would not occur in the event of a maximum magnitude earthquake; however, it is reasonable to expect that a well-designed and well-constructed structure will not collapse or cause loss of life in a major earthquake (SEAOC, 1996).

Based on the subsurface soil conditions encountered and local seismic sources for seismic design the site can be classified as Soil Profile S_C in accordance with the 2007 California Building Code (CBC), and Site Class C in accordance with the 2006 International Building Code (IBC); the tables below provide seismic design criteria in accordance with the UBC and IBC.

TABLE I
2007 UNIFORM BUILDING CODE - Chapter 16

ITEM	DESIGN VALUE	SOURCE
Seismic Zone	4	Figure 16-2
Seismic Zone Factor	0.40	Table 16-I
Soil Profile Type	S_D	Table 16-J
Seismic Source Type	B	Table 16-U
Near Source Factor, N_a	1.3	Table 16-S
Near Source Factor, N_v	1.6	Table 16-T
Seismic Coefficient, C_a	$(0.44N_a)$	Table 16-Q
Seismic Coefficient, C_v	$(0.64N_v)$	Table 16-R

*Greenville fault located approximately 1.5 km from the site.

ITEM	DESIGN VALUE
Site Class	C
0.2 second Spectral Response Acceleration, S_s	1.5
1.0 second Spectral Response Acceleration, S_1	0.60
Site Coefficient, F_a	1.0
Site Coefficient, F_v	1.0
Maximum considered earthquake spectral response accelerations for short periods, S_{MS}	1.50
Maximum considered earthquake spectral response accelerations for 1-second periods, S_{M1}	0.90
Design spectral response acceleration at short periods, S_{DS}	1.00
Design spectral response acceleration at 1-second periods, S_{D1}	0.60

Lurching. Ground lurching is a result of the rolling motion imparted to the ground surface during energy released by an earthquake. Such rolling motion can cause ground cracks to form. The potential for the formation of these cracks is considered greater at contacts between deep alluvium and bedrock, such as those at the margins of valley flood plains. Such an occurrence is possible at the subject site as in other locations in the Bay Area, but the offset or strain is expected to be very minor. Proposed construction of engineered fills underlying all developed portions of the Oak Creek Canyon project is expected to mitigate this hazard.

Liquefaction. Liquefaction is a phenomenon in which saturated cohesionless soils are subject to a temporary loss of shear strength because of pore pressure build-up under the cyclic shear stresses associated with earthquakes. Based on the material types and densities (blow counts) of materials encountered in our borings, the risk of liquefaction is considered low to negligible at this site.

Earthquake-Induced Densification. Densification of loose sand above the groundwater level during earthquake shaking could cause settlement of the ground surface. In addition, densification of liquefiable soils, below the ground-water level, can cause detrimental settlement at the ground surface. Loose sand layers were generally not encountered above the groundwater level and, as described above, the liquefaction potential within the Oak Creek Canyon project site is considered low. Therefore, the potential for earthquake-induced densification can be considered low.

Lateral Spreading. Lateral spreading is a failure within a nearly horizontal soil zone, commonly associated with liquefaction, which causes the overlying soil mass to move towards a free face or down a gentle slope. Since the potential for liquefaction is considered low, and the proposed development area is not adjacent to a free face, it is our opinion that lateral spreading is unlikely.

RECOMMENDATIONS

Grading

All grading and site development plans have been coordinated and should continue to be coordinated with the Engineering Geologist and the Geotechnical Engineer to modify the plans such that they mitigate known soil and geologic hazards. Detailed locations of keyways, subdrains, debris benches, and subexcavation areas should be shown on the final grading plans upon their completion. Sequence of grading issues, such as placement of various cut materials in specific locations, should have also been evaluated during review of final 40-scale grading plans.

The Geotechnical Engineer or qualified representative should be present during all phases of grading operations to observe demolition, site preparation, grading operations, and subdrain placement. The Geotechnical Engineer should be notified a minimum of 72 hours prior to the commencement of any grading or stripping operations at the site. This is to provide time to coordinate the work with the Grading Contractor. After the grading operations commence, geologic observations of cut areas should be made at frequent intervals. This is advised so that revised geologic recommendations can be incorporated into updated grading plans as grading proceeds.

Ponding of storm water, other than within engineered detention basins, should not be permitted at the site, particularly during work stoppage for rainy weather. Before the grading is halted by rain, positive slopes should be provided to carry the surface runoff to storm drainage structures in a controlled manner to prevent erosion damage.

Demolition and Stripping

Grading should begin with the removal of existing structures and associated foundation systems, any buried pipes, septic tanks, leach fields, debris piles, designated fences, trees and associated root systems, and any other deleterious materials. Underground structures that will be abandoned or are expected to extend below proposed finished grades should be removed from the project site.

All existing non-engineered fill, vegetation and soft or compressible soils should be removed as necessary for project requirements. The depth of removal of these materials should be determined by the Geotechnical Engineer or qualified representative in the field at the time of grading. Evaluation of unsuitable deposits should be performed during grading by sampling and laboratory analyses.

Areas to receive fill, slabs-on-grade, or structural foundations and those areas that serve as borrow for fill should be stripped of existing vegetation. Topsoil is estimated to be from 4 to 8 inches in thickness depending on location. Actual depths will be determined by the Geotechnical Engineer or qualified representative in the field during grading. Site strippings should be reserved for placement on graded slopes prior to installation of proposed erosion control measures. After placement on graded slopes, any remaining strippings and organically contaminated soils which are not suitable for use as engineered fill may be used in approved open space areas or landscape areas. These materials may also be blended into engineered fills provided the organic content of the fill is increased less than 3 percent by weight of the non-stripping soils after blending. Any topsoil retained for future use in landscape areas should be approved by the Landscape Architect and stockpiled in areas where it will not interfere with mass grading operations.

All exploratory geologic test pits excavated during site explorations are shown on Figure 4. It will be necessary to remove and recompact all loose soil within the test pits, where it will remain below final grades and is located within proposed improvement areas. Within the development areas, excavations resulting from demolition, clearing, and/or stripping which extend below final grades should be cleaned to firm undisturbed soil as determined by the Geotechnical Engineer's representative.

Subgrade Preparation

Following demolition, clearing, and stripping, all areas to receive fill, slabs-on-grade or pavement should be scarified to a depth of at least 12 inches, moisture conditioned, and compacted to the requirements for engineered fill presented below. The finished subgrade should be firm and non-yielding under the weight of compaction equipment.

Fill Materials

The site soils and bedrock containing less than 3 percent organics are suitable for use as engineered fill. Import materials, if any are needed, must meet the requirements contained in Section 2.02B, Part I of the Guide Contract Specifications. The Geotechnical Engineer should be informed if any importation of soil is contemplated. A sample of the proposed import material should be submitted to the Geotechnical Engineer for evaluation prior to delivery at the site.

Placement of Fill

Overcompaction of expansive materials (PI >12) may produce an undesirable environment for expansion in the zone of significant seasonal moisture variation; therefore, special requirements for compaction of expansive soils are necessary within the upper 5 feet in building areas. This recommendation is not to be interpreted as a requirement to remove and replace the top five feet within all lots, but is to be used when fill is placed within the top 5 feet of finished grade. The following compaction control requirements should be generally applied to engineered fills.

TABLE II

DESCRIPTION	MATERIALS	MINIMUM RELATIVE COMPACTION (%)	MINIMUM MOISTURE CONTENT PERCENTAGE POINTS ABOVE OPTIMUM MOISTURE
Within the upper 5 ft	Expansive	87 to 92	+4 %
	Non-expansive	90	+2 %
From 5 to 20 ft	Expansive	90	+3 %
	Non-expansive	90	+2 %

Maximum dry densities and moisture contents should be determined in accordance with ASTM D-1557, latest edition. Plasticity Index determinations, and possibly supplemental swell test data, should be made as a part of grading control. All fills should be placed in lifts not exceeding 12 inches or the depth of penetration of the compaction equipment used, whichever is less.

Keyways

After stripping, mass grading should begin with construction of keyways and subdrains. All fills should be adequately keyed into firm natural materials unaffected by shrinkage cracks. Keyways should be compacted in accordance with the specification presented above for fills greater than 5 feet deep. Anticipated keyway sizes and locations should be determined based on the final grading plans by the Geotechnical Engineer. Typical minimum keyway sizes and subdrains are shown on Figure 10 and 11. The actual depth of the keyways will be determined in the field by the Geotechnical Engineer during grading. Filling above keyways should be benched into firm competent soil or bedrock and drained as appropriate. Unless otherwise recommended by the Geotechnical Engineer, benches should be constructed at vertical intervals of not less than 5 feet.

Debris Benches

Debris benches with keyways will be required at the toes of cut or natural slopes as shown on the remedial grading plan. The debris bench should be provided with a concrete V-ditch discharging into an approved outlet. All debris benches will require periodic maintenance consisting of the removal and disposal of accumulated slope detritus. Proper access should be provided for the heavy equipment which may be required for removal of slide debris from benches and paved areas. All debris benches and buttress fills should be jointly designed by the Civil and Geotechnical Engineers to optimize stability, cut/fill balance, and drainage concerns. Recommendations for mass grading are generally applicable to landslide reconstruction and buttress fill installation.

Construction of Subsurface Drainage Facilities

Subsurface drainage systems should be installed in all keyways, swales or natural drainage areas, and landslide removal areas. Swales and drainage courses should be overexcavated to a firm base as determined by the Geotechnical Engineer during grading. A trench subdrain should then be installed through the center of the subexcavation as shown in Figure 11. The approximate locations of the recommended subdrains should be shown on the final grading plans. Depending on the actual conditions encountered during grading, similar subsurface drainage facilities may be recommended within existing stock ponds, springs or low-lying areas.

Subdrains should also be added where wet conditions are encountered during excavations. Subdrain systems should consist of a minimum 6-inch-diameter perforated pipe encased in at least 18 inches of Caltrans Class 2 permeable material or coarse drain rock wrapped in geotextile filter fabric. For selected keyway and bench subdrains, premanufactured synthetic edge drains may be substituted for the perforated pipe and permeable material. Typical subdrain details are shown in Figure 11. The subdrain pipe should meet the requirements contained in Section 2.05, Part I of the Guide Contract Specifications. Discharge from the subdrains will generally be low but in some instances may be continuous. Subdrains should outlet into open drainages or the proposed storm drain system, and their locations should be documented for future maintenance.

In addition, we recommend installing subdrains along the toes of downhill slopes adjacent to cut lots within the residential development. The subdrains should be located at the toes of slopes used to transition between cut lots. The subdrain system should be at least 3 feet deep and 12 inches in width. The subdrain should consist of a 4-inch-diameter perforated pipe, perforations placed down, surrounded by a filter medium. The filter medium may consist of Class 2 permeable material or clean, crushed rock or gravel encapsulated in filter fabric. The top 12 inches of subdrain trench backfill should consist of native compacted soil. Where solid pipe is used as the collector to discharge to an approved outlet, the trench backfill material should consist of native compacted soil.

Not all sources of seepage have been uncovered during our field work because of the intermittent nature of some of these conditions and their dependence on long-term climatic conditions. Furthermore, new sources of seepage may be created by a combination of changed topography, manmade irrigation patterns and potential utility leakage. Since uncontrolled water movements are one of the major causes of detrimental soil movements, it is of utmost importance that the Geotechnical Engineer be advised of any seepage conditions encountered during grading so that remedial action may be initiated, if necessary.

Cut-Fill Transition Lots and Cut Lots

Some single-family lots in this project will likely be entirely in cut or traversed by a cut/fill transition. It can be anticipated that significant variations in material properties may occur in areas of cut or cut/fill transition if not mitigated during site grading. It is our opinion that there is a potential for significant differential in swell characteristics across cut areas and cut/fill transitions. Such situations can be detrimental to building performance. Figure 12 represents the typical overexcavation recommended to mitigate the effects of differential materials located under a structure. In summary, we recommend that cut lots be overexcavated 2 feet, scarified 12 inches, and recompacted; cut/fill transition lots should be overexcavated 3 feet to provide a uniform thickness of engineered fill within the entire foundation area.

Graded Slopes

In general the following slope gradient guidelines may be applied for mass grading design of both permanent cut and fill slopes:

TABLE III

ALLOWABLE SLOPE GRADIENT (H:V)	MAXIMUM ALLOWABLE SLOPE HEIGHT (FT)		
	GENERAL (On Site Material)	GENERAL FILL WITH GEOGRID REINFORCEMENT	SELECTED FILL On low to moderate expansive
2:1	8	20	20
2.5:1	15	40	40
3:1	>15	>40	>40

The current grading plan utilizes 15 foot high 2:1 slopes throughout the project. It is our opinion, that these planned 2:1 slopes are acceptable provided that stabilization measures are utilized, such as overexcavation and reconstruction as engineered fill buttress slope with select fill materials with a Plasticity index of 25 or less, or reconstruction as an engineered fill buttress slope with geogrid reinforcement for materials with PI's greater than 25. The geogrid reinforcement shall consist of Tensar BX1200 or approved equivalent and have a width of 11 feet minimum, measured from the face of the finished slope into the slope horizontally. For convenience, a full roll width of 13.1 feet can be used. The recommended spacing between layers shall be 3 feet typical from the toe of the slope to within 4 feet of the top of the reinforced slope. Verification of the actual slope gradient is the responsibility of the contractor and surveyor.

All cut slopes should be viewed by the Engineering Geologist during slope grading for adverse bedding, seepage, or bedrock conditions which may affect slope stability. In the event that adverse geologic conditions are detected during grading of the cut slopes, overexcavation and reconstruction of these slopes may be necessary. Track rolling to compact faces of slopes is not sufficient. Slopes should be overbuilt at least 2 feet and cut back to design grades.

Unsuitable Material Removal Area (Alternate)

As an alternative to generate additional onsite fill material, identified areas above Lots 1 and 2, as depicted on Figures 5 and 8, may be removed and such materials may be incorporated into engineered fills at the site. We estimate the final grades in these areas would be as depicted in Figures 8 as the "Optional Proposed Grades". During grading, supplemental recommendations related to remedial grading and/or subdrainage would be provided as necessary. If unsuitable bedrock conditions are encountered during grading the unsuitable material should be over-excavated 15 feet, measured horizontally, and grades restored using properly drained engineered fill. For slopes steeper than 3:1 additional slope stabilization measures, such as geo-grid reinforcement may also be necessary.

Foundation Recommendations

The proposed house structures may be supported utilizing a number of foundation alternates as discussed in the following sections of this report. It has been our experience that pier-and-grade-beam foundations are suitable for lots where building areas will be located in proximity to or along slopes, or where building areas may have a split-level condition. Where fills underlie building envelopes and subdrainage is present an alternate system such as shallow continuous footings may be appropriate. For relatively level pads setback at least 10 feet from downslope areas the use of post-tensioned slabs, structural mat foundations is preferred. If near-slope portions of lots are supported with properly designed retaining walls, spread footing or structural mat foundations may be designed for level-ground conditions may be acceptable. The following table summarizes the recommended and alternative foundation types for the subject lots:

TABLE IV
Recommended Foundation Types by Lot Number

Lot Numbers	Preferred Foundation Alternate	Optional Foundation Alternate
1	Continuous Spread Footings	Pier-and-grade-beam
2, 3, 4, and 5	Post-Tensioned Slab	Continuous Spread Footings ; Pier-and-grade-beam

Pier-and-Grade-Beam Foundations. The proposed houses may be supported on a friction pier-and-grade-beam foundation system as listed in Table IV. In pier foundation design, deeper more widely spaced piers with stiffer grade beams are preferred in order to make the foundation design less susceptible to changes in subgrade conditions over time. The following criteria should be used to design the piers:

TABLE V
Pier-and-Grade-Beam Recommendations

Minimum pier depth:	10 feet minimum and 5 feet into competent bedrock, whichever is greater in depth.
Minimum pier diameter:	16 inches for piers up to 20 feet deep; and 18 inches for piers greater than 20 feet deep.
Minimum pier spacing:	3 pier diameters, center-to-center. Where closer spacing is unavoidable, the piers should be designed with a reduced skin friction of 330 psf.
Maximum allowable skin friction:	500 pounds per square foot (psf). This value may be increased by one-third when considering seismic or wind loads. Friction in the upper 36 inches or as should be ignored.

Piers located on or within 5 feet (measured horizontally) of downhill slopes should be designed to resist lateral creep loads using a uniform pressure of 300 psf acting on 1½ times the pier diameter against the upper 3 feet of the pier. Lateral loads may be resisted by passive pressures generated by the soils below a depth of 3 feet. For passive resistance, an equivalent fluid weight of 300 pounds per cubic foot (pcf) acting on 2 times the pier diameter may be used for the portions below a depth of 3 feet. The pier reinforcement should be designed by the Structural Engineer. Where applicable, the pier reinforcement should be tied to the grade beam as recommended by the Structural Engineer.

The pier spacing should be determined from the load-bearing capacity of the piers. All exterior and interior piers should be tied together with a well-reinforced grade-beam system to act as a rigid grid. The grade-beam reinforcement will be dependent on the pier spacing and the structural loads to be supported, but in no case should less than four No. 5 rebar be used, two in the top and two in the bottom of the beam. Grade beams should be constructed to span between the piers without bearing on the underlying expansive soil. We recommend that a minimum 2-inch void be constructed below grade beams by placing a compressible material at the soil surface prior to casting concrete. The void-forming material should be approved by ENGEO prior to construction. Grade beams should be kept to the minimum width that is structurally practical to avoid uplift forces associated with swelling soils. Isolated piers may be used to support floor loads and isolated point loads; however, the number of isolated piers should be kept to a minimum. We will be glad to consult with your Structural Engineer on this matter on a case-by-case basis.

Provisions must be made to prevent surface water from flowing under the structure. To cause water to flow away from the structure, at least 6 inches of soil should be placed and compacted on the outside of the grade beam, and sloped away from the foundation at right angles to the grade beam. Pier hole drilling should be done under the observation of the Geotechnical Engineer or his/her qualified representative to confirm that the above recommendations are being complied with and so that alternative action may be implemented when subsurface conditions vary from those encountered in our explorations. If refusal to drilling is encountered, the Geotechnical Engineer, in consultation with the Structural Engineer, should determine what measures, if any, need to be taken. In order to minimize potential future pier settlements, all loose soil should be removed from the bottom of pier holes prior to placing concrete. Pier holes should not be allowed to desiccate before pouring concrete. Depressions at the top of the piers resulting from drilling operations or from any other cause should be backfilled to prevent ponding, and concrete collars occurring at the top of the piers as a result of excess concrete placement should be removed to prevent unnecessary uplift forces against the piers. The foundation plans should be reviewed by the project Geotechnical Engineer when they become available to check for conformance with the above recommendations.

Continuous Spread Footings. Structures may be supported on shallow continuous spread "T"-footings. This system may be combined with raised floor systems or slabs-on-grade. The footings should be interconnected and have a minimum width of 15 inches and have a minimum depth of embedment of 24 inches. The depth of the footings should be measured from the lowest adjacent finished grade. Embedment depth of footings should be increased to a minimum depth of 36 inches for footings along slopes and/or located closer than 5 feet (measured horizontally) to downslope areas that are steeper than 5:1 (horizontal to vertical).

Continuous footings should be designed by a Structural Engineer and reinforced with top and bottom steel to provide structural continuity and to permit spanning of local irregularities. Footings should be designed to form a rigid interconnected grid and reinforced to accommodate a differential movement of 1 inch over 20 feet. In addition, the structural engineer should consider designing the footing reinforcement to limit excessive deflections in the framing and wall finishes.

The shallow continuous footings should be designed for an allowable bearing pressure of 2,500 pounds per square foot (psf); this value may be increased by one-third for wind and seismic loads. A passive resistance pressure of 300 pounds per cubic foot (pcf), equivalent fluid weight, may be used for design if the area in front of the footing is level for at least 8 feet, where the upper 1 foot of footing embedment should be neglected for passive resistance pressure. For foundations located less than 8 feet from the edge of slopes (measured horizontally) passive resistance should be neglected in the upper 3 feet of foundation embedment. A base friction factor of 0.30 may be used in the design.

Footings founded in expansive soils may be subjected to detrimental uplift forces along the sides of the footings. To help reduce the potential for uplift pressures in expansive soils, we recommend the portion of these foundations above the top of the footings be formed and the top of the footings should be a minimum of 18 inches below the lowest adjacent grade. Footing excavations should be kept moist prior to placing foundation concrete and should be backfilled with native soil. The foundation plans should be reviewed by a Geotechnical Engineer when they become available to check for conformance with these recommendations.

Post–Tensioned Slabs. Post-tensioned slabs are suitable to support the proposed structures as listed in Table IV above. We recommend a 10-inch minimum slab thickness. The perimeter should be thickened an additional 2 inches, with a 6-inch minimum soil backfill height against the slab at the perimeter. The post-tensioned slabs should be designed to impose a maximum allowable bearing pressure of 1,000 pounds per square foot (psf) for dead-plus-live loads. This value may be increased by one-third when considering wind and seismic loads. The proposed structure may not be capable of undergoing the differential movements that the mat can sustain; hence, stiffeners may have to be considered. The Structural Engineer should be consulted on this matter.

The following recommendations reflect the latest California Building Code that requires PT criteria per the Post-Tensioning Institute "Design of Post-Tensioned Slabs-on-Ground" Third Edition:

Center Lift Condition:

Edge Moisture Variation Distance, em = 5.0 feet
Differential Soil Movement, ym = 4.0 inches

Edge Lift Condition:

Edge Moisture Variation Distance, em = 4.0 feet
Differential Soil Movement, ym = 1.7 inches

A uniform subgrade material should be provided under post-tensioned mats. The top 12 inches of pad subgrade should be moisture conditioned at least 2 percentage points above optimum moisture content by sprinkling subgrade soils uniformly immediately prior to concrete placement. Do not allow the subgrade to dry prior to concrete placement.

Slab Moisture Vapor Reduction. When buildings are constructed with concrete slabs-on-grade, such as post-tensioned mats, water vapor from beneath the slab will migrate through the slab and into the building. This water vapor can be reduced but not stopped. Vapor transmission can negatively affect floor coverings and lead to increased moisture within a building. When water vapor migrating through the slab would be undesirable, we recommend the following to reduce, but not stop, water vapor transmission upward through the slab on grade.

1. Install a vapor retarder membrane directly beneath the slab. Seal the vapor retarder at all seams and pipe penetrations. Vapor retarders shall be Class A vapor retarder in accordance with ASTM E 1745 "Standard Specification for Plastic Water Vapor Retarders used in Contact with Soil or Granular Fill under Concrete Slabs." Vaper retarders should be installed and sealed as recommended by the manufacturer and at all seams and pipe penetrations..
2. Concrete shall have a concrete water-cement ratio of no more than 0.5.
3. Provide inspection and testing during concrete placement to check that the proper concrete and water cement ratio are used.
4. Consider moist cure slabs for a minimum of 3 days.

The Structural Engineer should be consulted as to the use of a layer of clean sand (less than 5 percent passing the U.S. Standard No. 200 Sieve) placed on top of the vapor retarder membrane to assist in concrete curing. In our past experience, we have observed that concrete slabs retain moisture and may take several months to fully hydrate. Provide sufficient time to air dry floor slabs before floor covering application, such as vinyl floor tile and wood flooring placement. Alternatively, apply a floor sealant over the concrete to minimize moisture from accumulating under the flooring. Also, the use of a lower water/cement ratio and higher strength concrete will reduce the amount of water in the slab and help expedite the hydration time. Protect foundation subgrade soils from seepage by providing impermeable plugs within utility trenches as described in the "Utilities" section.

Foundation Drainage. For a raised floor system, it is recommended that subsurface drains be provided around the perimeter of the residential houses to help collect subsurface seepage beneath foundations, as illustrated on Figure 13. The subdrainage trench should be at least 12 inches wide and extend at least 6 inches below the bottom of the perimeter grade beam. The trench should be provided with a 4-inch-diameter perforated pipe (with perforations down) surrounded by either Class 2 permeable material or drain rock encapsulated in filter fabric (6-oz. minimum). All trenches and pipes should have a minimum slope of 1 percent, and must be constructed within 12 inches of the foundation. ENGEO should be consulted if these criteria can not be achieved.

The under-floor area should be sloped away from the foundation and drain into crawlspace drain inlets to remove any water that may enter the crawl space. This drain should outlet into an approved location well outside the structure, or if approved by the Geotechnical Engineer, may connect into the perimeter subdrain outlet system as shown on Figure 13. In addition, under-floor crawl spaces should be provided with a liberal number of ventilation openings to reduce differential soil moisture conditions.

Closed roof downspout collector pipe and perimeter subdrains can be constructed in a single trench, if desired; however, the closed collector pipe must be placed above the subdrain pipe and in no case may the subdrain pipe be connected to the closed drain pipe system. In addition, under-floor crawl spaces should be provided with a liberal number of ventilation openings to reduce differential soil moisture conditions in accordance with current building code requirements.

Secondary Slab-on-Grade Construction. This section provides guidelines for secondary slabs such as porch slabs, exterior patio slabs, walkways, driveways, and steps. Secondary slabs-on-grade should be constructed structurally independent of the foundation system. This allows slab movement to occur with a minimum of foundation distress. Where slab-on-grade construction is anticipated, care must be exercised in attaining a near-saturation condition of the subgrade soil before concrete placement. Slabs-on-grade should be designed specifically for their intended use and loading requirements. Some of the site soils have a high expansion potential; therefore, cracking of conventional slabs should be expected. As a minimum requirement, slabs-on-grade should be reinforced for control of cracking. Slab reinforcement should be designed by the Structural Engineer. In our experience, welded wire mesh is generally not sufficient to control slab cracking. Therefore, we recommend the Structural Engineer consider using a minimum of No. 3 bars for design of the slab reinforcement.

Slabs-on-grade should have a minimum thickness of 4 inches with a thickened edge extending at least 6 inches into compacted soil to minimize water infiltration. A 4-inch-thick layer of clean crushed rock or gravel should be placed under sidewalk and driveway slabs. As an alternative to providing a 6-inch-thick edge, a minimum 5½-inch-thick slab could be placed over 4 inches of clean crushed rock or gravel.

Retaining Walls

Small retaining walls may be used in conjunction with the planned development. If incorporated into house design, retaining walls not free to deflect (or rotate at the top) should be designed as restrained walls, and at-rest earth pressures should be used. Other retaining walls not adjoining house structures may be designed for active earth pressures since these walls are anticipated to be free to rotate at the top of the walls.

Retaining walls should be designed to withstand the following equivalent fluid pressures, which do not include increases due to surcharge and hydrostatic pressures.

Backfill Slope Condition (horizontal:vertical)	Active Pressure (pounds per cubic foot)	At-Rest Pressure (pounds per cubic foot)
Level	50	75
4:1	55	80
3:1	60	90
2:1	70	100

Retaining walls supported on shallow continuous footings should have a minimum width of 15 inches and have a minimum depth of embedment of 24 inches. The depth of the footings should be measured from the lowest adjacent finished grade. Embedment depth of footings should be increased to a minimum depth of 36 inches for footings along slopes and/or located closer than 5 feet (measured horizontally) to downslope areas that are steeper than 5:1 (horizontal:vertical). The shallow continuous wall footings should be designed for an allowable bearing pressure of 2,000 pounds per square foot (psf); this value may be increased by one-third for wind and seismic loads. A passive resistance pressure of 300 pounds per cubic foot (pcf), equivalent fluid weight, may be used for design if the area in front of the wall is level for at least 8 feet. The upper one foot of wall embedment should be neglected for passive resistance pressure. For foundations located less than 8 feet from the edge of slopes (measured horizontally) passive resistance should be neglected in the upper 3 feet of wall embedment. To develop passive resistance, the designer may consider incorporating a structural key incorporated into the footing, provided the key is located at least 8 feet from the face of the slope. A base friction factor of 0.35 may be used in the design.

For retaining walls supported on drilled piers, the following criteria are recommended. The drilled piers should be at least 12 inches in diameter and designed for an allowable skin friction of 500 psf. Skin friction should be disregarded in the upper 12 inches of embedment. Resistance to lateral loads can be obtained from passive resistance against the drilled pier face. Passive resistance can be calculated by using 300 pcf equivalent fluid weight, using a shape factor of 2.0. Passive pressure should be neglected in the upper one foot of embedment at the toe of the wall. For piers located along slopes, the uppermost 3 feet of embedment should be neglected for passive resistance.

Drilled piers should be free of loose soil and debris prior to concrete placement. If water collects in the pier shaft, it should be pumped out prior to the placement of concrete. Concrete should be placed by means of a tremie-pipe or similar device to avoid concrete contamination by soils dislodging from the pier shaft. Drilling below bedrock may be difficult and require drill rigs capable of drilling moderately strong sandstone bedrock materials, and the use of rock barrels/buckets may be needed to maintain plumbness and the integrity of piers.

All retaining walls should be provided with drainage facilities to prevent the build-up of hydrostatic pressures behind them. Wall drainage may be provided using a 4-inch-diameter perforated pipe (SDR 35 or approved equivalent) embedded in Class 2 permeable material, or free-draining gravel surrounded by synthetic filter fabric. The width of the drain blanket should be at least 12 inches. The drain blanket should extend to about one foot below the finished grades. As an alternative, prefabricated synthetic wall drain panels can be used. The upper foot of wall backfill should consist of on-site clayey soils. Drainage should be collected by perforated pipes and directed to an outlet approved by the Civil Engineer.

Retaining Wall Drainage.

All retaining walls should be provided with drainage facilities to prevent the build-up of hydrostatic pressures behind the walls. Wall drainage may be provided using a 4-inch-diameter perforated pipe (such as SDR-35 or approved equivalent) embedded in free-draining gravel surrounded by synthetic filter fabric (at least 6 ounces per square yard), or Class 2 permeable material. The thickness of the drainage medium extending up the back of wall should be at least 12 inches and should extend to approximately one foot below finished grades. The upper one foot of wall backfill should consist of compacted site soil materials. As an alternative, prefabricated synthetic wall drain panels, which meet the minimum requirements listed in the Guide Contract Specifications and are pre-approved by ENGEO, can replace the granular drainage medium. Drainage should be collected by solid pipes and directed to an outlet approved by the Civil Engineer. All backfill should be placed in accordance with the recommendations provided above for engineered fill. Light equipment should be used during backfill compaction to minimize possible overstressing of the walls. The foundation details and structural calculations for the walls should be submitted for review.

Preliminary Pavement Design

The following preliminary pavement section has been determined for a Traffic Index of 5 and the assumed R-value of 5 according to methods contained in Topic 608.4 of Highway Design Manual by Caltrans and City of Clayton requirements.

Traffic Index	AC (inches)	AB (inches)
5.0	3.0	10.0
6.0	3.5	13.0
7.0	4.0	15.5

Notes: AC is asphaltic concrete
AB is aggregate base Class 2 Material with minimum R = 78

The above pavement section is provided for estimating only. The actual subgrade material should be tested for R-value. The Traffic Index should be confirmed by the Civil Engineer and the City of Clayton. Pavement materials and construction should conform to the specifications and requirements of the Standard Specifications by the Division of Highways, Department of Public Works, State of California, city requirements and the following minimum requirements.

- All pavement subgrades should be scarified to a depth of 12 inches below finished subgrade elevation, moisture conditioned to at least 3 percentage points above optimum, and compacted to at least 90 percent relative compaction and in accordance with city requirements (ASTM Methods).
- Subgrade soils should be in a stable, non-pumping condition at the time aggregate baserock materials are placed and compacted.
- Adequate provisions must be made such that the subgrade soils and aggregate baserock materials are not allowed to become saturated.
- Aggregate base materials should meet current Caltrans specifications for Class 2 Aggregate Base and should be compacted to at least 95 percent of maximum dry density at a moisture content of at least optimum (ASTM Methods).
- Asphalt paving materials should meet current Caltrans specifications for asphalt concrete and should be compacted to at least 95 percent of maximum wet density (Caltrans Methods) unless otherwise noted by the City.
- All concrete curbs separating pavement and irrigated landscaped areas should extend into the subgrade and below the bottom of adjacent aggregate baserock materials. Alternatively, median and edge drains can be installed to help prevent infiltration of water under pavement areas.

Drainage Requirements

It is very important that all lots be positively graded at all times to provide for rapid removal of surface water. Ponding of water under floors or seepage toward foundation systems at any time during or after construction must be prevented. As a minimum requirement, finished grades should generally provide a slope of at least 3 percent within 5 feet from exterior walls at right angles to them to allow surface water to drain positively away from the structures. Care should be exercised to ensure that landscape mounds will not interfere with these requirements. All lots should be drained individually. Storm water from roof downspouts should be conveyed in closed drain systems to an outlet that extends through the curb or to an approved outlet.

If planting adjacent to a building is desired, the use of plants that require very little moisture is recommended. Trees should be avoided in close proximity to structures. Sprinkler systems should not be installed where they may cause ponding or saturation of foundation soils. Such ponding or saturation could result in undesirable soil swell, loss of compaction and consequent foundation and slab movements. Irrigation of landscape areas should be limited strictly to that necessary for plant growth.

Building Setback Distance

Where building pads are adjacent to uphill slopes, all permanent structures should be set back from the toe-of-slope a distance equal to one-half the vertical graded slope height or 15 feet, whichever is less. Where building pads are adjacent to downhill slopes, all permanent structures should generally be set back from the top-of-slope. Structures should be located no closer than 15-feet from the top-of-slope. If structures are to be located closer than 15-feet from the top-of-slope pier-and-grade-beam or continous spread footing foundations should be utilized. Slope set-back requirements should be evaluated on a lot-by-lot basis after the final grading plan is developed.

Erosion Control

In addition to vegetated cover, viable erosion mitigation measures may include concrete or asphalt-lined drainage facilities and slopes graded to 3:1 (horizontal:vertical) or less. These measures are typically used on slopes with heights greater than 30 feet. The purpose of the drainage facilities is to intercept and divert the surface water runoff from the slopes and, combined with the

3:1 or flatter slopes, reduce runoff velocities, water infiltration, and sloughing or erosion of the slope surfaces. Erosion of graded slopes can be mitigated by hydroseeding, landscaping, or placement of topsoil materials prior to the winter rains following rough grading. All landscaped slopes should be maintained in a vegetated state after project completion with drought tolerant vegetation requiring drip irrigation.

The tops of fill or cut slopes should be graded in such a way as to prevent water from flowing freely down the slopes. Due to the nature of the bedrock, slopes may experience severe erosion when grading is halted by heavy rain. Therefore, before work is stopped, a positive gradient away from the slopes should be provided to carry the surface runoff away from the slopes to areas where erosion can be controlled. It is vital that no completed slope be left standing through a winter season without erosion control measures having been provided.

Utilities

Allow the Geotechnical Engineer to observe all utility trench backfill. Use well-graded import or native material less than ¾ inch in maximum dimension for pipe zone backfill (i.e. material beneath and immediately surrounding the pipe). Use native soil for trench zone backfill (i.e. material placed between the pipe zone backfill and the ground surface). Compact backfill in accordance with the recommendations provided above for engineered fill. Use fine- to medium-grained sand or a well-graded mixture of sand and gravel for pipe zone backfill import material. Avoid using this material within 2 feet of finish grades. In general, avoid using uniformly graded gravel for pipe or trench zone backfill due to the potential for migration of: (1) soil into the relatively large void spaces present in this type of material; and (2) water along trenches backfilled with this type of material. Provide all utility trenches entering buildings and paved areas with an impervious seal consisting of native materials or concrete where the trenches pass under building perimeters or curb lines. Extend the impervious plug a minimum of 3 feet to either side of the crossing to prevent surface water percolation into the sands under foundations and pavements. Trapped water will remain trapped in a perched condition, allowing clays to develop their full expansion potential.

Avoid locating utility trenches upslope of any foundation area without a Geotechnical Engineer review of the placement, depth and proposed backfill material. Exercise care where utility trenches are located beside foundation areas. Locate utility trenches constructed parallel to foundations entirely above a plane extending down from the lower edge of the footing at an angle of 45 degrees. Provide utility companies and Landscape Architects with this information. Construct utility trenches in paved areas in accordance with City of Clayton requirements; however, avoid compaction of native trench backfill by jetting. Notify owner if a conflict between city or other agency requirements and the recommendations contained in this report is observed to provide a resolution prior to submitting bids.

LIMITATIONS AND UNIFORMITY OF CONDITIONS

This report is issued with the understanding that it is the responsibility of the owner to transmit the information and recommendations of this report to developers, contractors, buyers, architects, engineers, and designers for the project so that the necessary steps can be taken by the contractors and subcontractors to carry out such recommendations in the field. The conclusions and recommendations contained in this report are solely professional opinions.

The professional staff of ENGEO Incorporated strives to perform its services in a proper and professional manner with reasonable care and competence but is not infallible. There are risks of earth movement and property damages inherent in land development. We are unable to eliminate all risks or provide insurance; therefore, we are unable to guarantee or warrant the results of our work.

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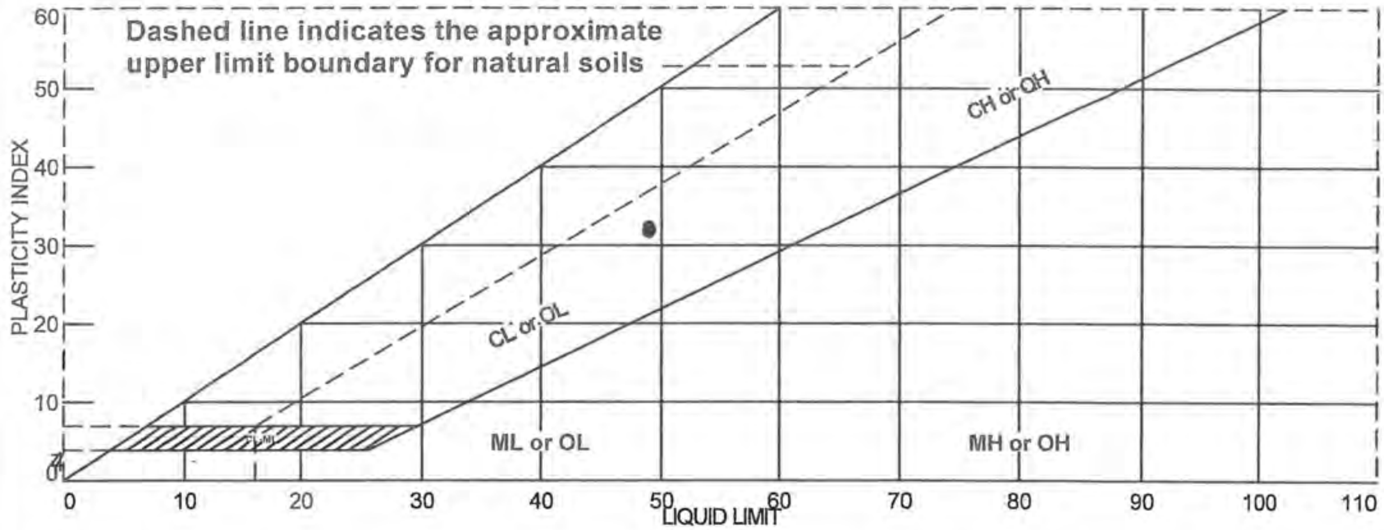
APPENDIX A

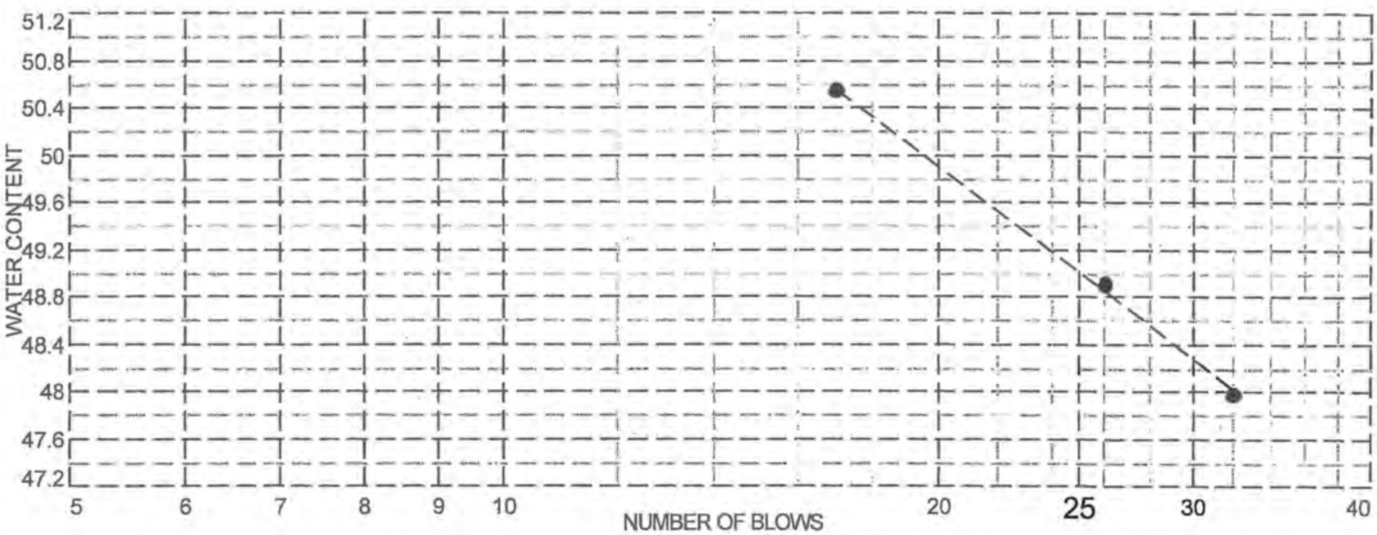
Laboratory Tests

1. Atterberg Limits (ASTM D-4318)

   Performed primarily on cohesive soils. Includes the Liquid Limit and the Plastic Limit. From these, a Plasticity Index can be computed which allows classification of the soil and is an indirect measure of its expansion characteristics.

LIQUID AND PLASTIC LIMITS TEST REPORT





	MATERIAL DESCRIPTION	LL	PL	PI	%<#40	%<#200	USCS
●	Dark yellowish brown silty CLAY to CLAY	49	17	32			CL-CH

Project No. 3840.2.052.02 **Client:**

Project: Oak Creek Canyon. Clayton, CA

● **Sample Number:** Bulk 1

Remarks:

ENGEO INCORPORATED GEOTECHNICAL AND ENVIRONMENTAL CONSULTANTS MATERIALS TESTING

Plate

APPENDIX B

Guide Contract Specification

3840.205.202
February 22, 2008

GUIDE CONTRACT SPECIFICATIONS

PART I - EARTHWORK

PREFACE

These specifications are intended as a guide for the earthwork performed at the subject development project. If there is a conflict between these specifications (including the recommendations of the geotechnical report) and agency or code requirements, it should be brought to the attention of ENGEO and Owner prior to contract bidding.

PART 1 - GENERAL

1.01 WORK COVERED

A. Grading, excavating, filling and backfilling, including trenching and backfilling for utilities as necessary to complete the Project as indicated on the Drawings.

B. Subsurface drainage as indicated on the Drawings.

1.02 CODES AND STANDARDS

A. Excavating, trenching, filling, backfilling, and grading work shall meet the applicable requirements of the Uniform Building Code and the standards and ordinances of state and local governing authorities.

1.03 SUBSURFACE SOIL CONDITIONS

A. The Owners' Geotechnical Exploration report is available for inspection by bidder or Contractor. The Contractor shall refer to the findings and recommendations of the Geotechnical Exploration report in planning and executing his work.

1.04 DEFINITIONS

A. Fill: All soil, rock, or soil-rock materials placed to raise the grades of the site or to backfill excavations.

B. Backfill: All soil, rock or soil-rock material used to fill excavations and trenches.

C. On-Site Material: Soil and/or rock material which is obtained from the site.

D. Imported Material: Soil and/or rock material which is brought to the site from off-site areas.

E. Select Material: On-site and/or imported material which is approved by ENGEO as a specific-purpose fill.

F. Engineered Fill: Fill upon which ENGEO has made sufficient observations and tests to confirm that the fill has been placed and compacted in accordance with specifications and requirements.

G. Degree of Compaction or Relative Compaction: The ratio, expressed as a percentage, of the in-place dry density of the fill and backfill material as compacted in the field to the maximum dry density of the same material as determined by ASTM D-1557 or California 216 compaction test method.

H. Optimum Moisture: Water content, percentage by dry weight, corresponding to the maximum dry density as determined by ASTM D-1557.

I. ENGEO: The project geotechnical engineering consulting firm, its employees or its designated representatives.

J. Drawings: All documents, approved for construction, which describe the Work.

1.05 OBSERVATION AND TESTING

A. All site preparation, cutting and shaping, excavating, filling, and backfilling shall be carried out under the observation of ENGEO, employed and paid for by the Owners. ENGEO will perform appropriate field and laboratory tests to evaluate the suitability of fill material, the proper moisture content for compaction, and the degree of compaction achieved. Any fill that does not meet the specification requirements shall be removed and/or reworked until the requirements are satisfied.

B. Cutting and shaping, excavating, conditioning, filling, and compacting procedures require approval of ENGEO as they are performed. Any work found unsatisfactory or any work disturbed by subsequent operations before approval is granted shall be corrected in an approved manner as recommended by ENGEO.

C. Tests for compaction will be made in accordance with test procedures outlined in ASTM D-1557, as applicable. Field testing of soils or compacted fill shall conform with the applicable requirements of ASTM D-2922.

D. All authorized observation and testing will be paid for by the Owners.

1.06 SITE CONDITIONS

A. Excavating, filling, backfilling, and grading work shall not be performed during unfavorable weather conditions. When the work is interrupted by rain, excavating, filling, backfilling, and grading work shall not be resumed until the site and soil conditions are suitable.

B. Contractor shall take the necessary measures to prevent erosion of freshly filled, backfilled, and graded areas until such time as permanent drainage and erosion control measures have been installed.

PART 2 - PRODUCTS

2.01 GENERAL

A. Contractor shall furnish all materials, tools, equipment, facilities, and services as required for performing the required excavating, filling, backfilling, and grading work, and trenching and backfilling for utilities.

2.02 SOIL MATERIALS

A. Fill

1. Material to be used for engineered fill and backfill shall be free from organic matter and other deleterious substances, and of such quality that it will compact thoroughly without excessive voids when watered and rolled. Excavated on-site material will be considered suitable for engineered fill and backfill if it contains no more than 3 percent organic matter, is free of debris and other deleterious substances and conforms to the requirements specified above. Rocks of maximum dimension in excess of two-thirds of the lift thickness shall be removed from any fill material to the satisfaction of ENGEO.

2. Excavated earth material which is suitable for engineered fill or backfill, as determined by ENGEO, shall be conditioned for reuse and properly stockpiled as required for later filling and backfilling operations. Conditioning shall consist of spreading material in layers not to exceed 8 inches and raking free of debris and rubble. Rocks and aggregate exceeding the allowed largest dimension, and deleterious material shall be removed from the site and disposed off site in a legal manner.

3. ENGEO shall be notified at least 48 hours prior to the start of filling and backfilling operations so that it may evaluate samples of the material intended for use as fill and backfill. All materials to be used for filling and backfilling require the approval of ENGEO.

B. Import Material: Where conditions require the importation of fill material, the material shall be an inert, nonexpansive soil or soil-rock material free of organic matter and meeting the following requirements unless otherwise approved by ENGEO.

Gradation (ASTM D-421):	Sieve Size	Percent Passing
	2-inch	100
	#200	15 - 70
Plasticity (ASTM D-4318):	Liquid Limit	Plasticity Index
	< 30	< 12
Swell Potential (ASTM D-4546B): (at optimum moisture)	Percent Heave	Swell Pressure
	< 2 percent	< 300 psf
Resistance Value (ASTM D-2844):	Minimum 25	
Organic Content (ASTM D-2974):	Less than 2 percent	

A sample of the proposed import material should be submitted to ENGEO for evaluation prior to delivery at the site.

2.03 SAND

A. Sand for sand cushion under slabs and for bedding of pipe in utility trenches shall be a clean and graded, washed sand, free from clay or organic material, suitable for the intended purpose with 90 to 100 percent passing a No. 4 U.S. Standard Sieve, not more than 5 percent passing a No. 200 U.S. Standard Sieve, and generally conforming to ASTM C33 for fine aggregate.

2.04 AGGREGATE DRAINAGE FILL

A. Aggregate drainage fill under concrete slabs and paving shall consist of broken stone, crushed or uncrushed gravel, clean quarry waste, or a combination thereof. The aggregate shall be free from fines, vegetable matter, loam, volcanic tuff, and other deleterious substances. It shall be of such quality that the absorption of water in a saturated surface dry condition does not exceed 3 percent of the oven dry weight of the samples.

B. Aggregate drainage fill shall be of such size that the percentage composition by dry weight as determined by laboratory sieves (U. S. Series) will conform to the following grading:

Sieve Size	Percentage Passing Sieve
1½-inches	100
1-inch	90 - 100
#4	0 - 5

2.05 SUBDRAINS

A. Perforated subdrain pipe of the required diameter shall be installed as shown on the drawings. The pipe(s) shall also conform to these specifications unless otherwise specified by ENGEO in the field.

Subdrain pipe shall be manufactured in accordance with one of the following requirements:

Design depths less than 30 feet

- Perforated ABS Solid Wall SDR 35 (ASTM D-2751)
- Perforated PVC Solid Wall SDR 35 (ASTM D-3034)
- Perforated PVC A-2000 (ASTM F949)
- Perforated Corrugated HDPE double-wall (AASHTO M-252 or M-294, Caltrans Type S, 50 psi minimum stiffness)

Design depths less than 50 feet

- Perforated PVC SDR 23.5 Solid Wall (ASTM D-3034)
- Perforated Sch. 40 PVC Solid Wall (ASTM-1785)
- Perforated ABS SDR 23.5 Solid Wall (ASTM D-2751)
- Perforated ABS DWV/Sch. 40 (ASTM D-2661 and D-1527)
- Perforated Corrugated HDPE double-wall (AASHTO M-252 or M-294, Caltrans Type S, 70 psi minimum stiffness)

Design depths less than 70 feet

- Perforated ABS Solid Wall SDR 15.3 (ASTM D-2751)
- Perforated Sch. 80 PVC (ASTM D-1785)
- Perforated Corrugated Aluminum (ASTM B-745)

B. Permeable Material (Class 2): Class 2 permeable material for filling trenches under, around, and over subdrains, behind building and retaining walls, and for pervious blankets shall consist of clean, coarse sand and gravel or crushed stone, conforming to the following grading requirements:

Sieve Size	Percentage Passing Sieve
1-inch	100
¾-inch	90 - 100
$^3/8$-inch	40 - 100
#4	25 - 40
#8	18 - 33
#30	5 - 15
#50	0 - 7
#200	0 - 3

C. Filter Fabric: All filter fabric shall meet the following Minimum Average Roll Values unless otherwise specified by ENGEO.

Grab Strength (ASTM D-4632)..180 lbs
Mass Per Unit Area (ASTM D-4751)..................................6 oz/yd^2
Apparent Opening Size (ASTM D-4751)...........................70-100 U.S. Std. Sieve
Flow Rate (ASTM D-4491)...80 gal/min/ft^2
Puncture Strength (ASTM D-4833) ....80 lbs

D. Vapor Retarder: Vapor Retarders shall consist of PVC, LDPE or HDPE impermeable sheeting at least 10 mils thick..

2.06 PERMEABLE MATERIAL (Class 1; Type A)

A. Class 1 permeable material to be used in conjunction with filter fabric for backfilling of subdrain excavations shall conform to the following grading requirements:

Sieve Size	Percentage Passing Sieve
¾-inch	100
½-inch	95 - 100
$^3/8$-inch	70 - 100
#4	0 - 55
#8	0 - 10
#200	0 - 3

PART 3 - EXECUTION

3.01 STAKING AND GRADES

A. Contractor shall lay out all his work, establish all necessary markers, bench marks, grading stakes, and other stakes as required to achieve design grades.

3.02 EXISTING UTILITIES

A. Contractor shall verify the location and depth (elevation) of all existing utilities and services before performing any excavation work.

3.03 EXCAVATION

A. Contractor shall perform excavating as indicated and required for concrete footings, drilled piers, foundations, floor slabs, concrete walks, and site leveling and grading, and provide shoring, bracing, underpinning, cribbing, pumping, and planking as required. The bottoms of excavations shall be firm undisturbed earth, clean and free from loose material, debris, and foreign matter.

B. Excavations shall be kept free from water at all times. Adequate dewatering equipment shall be maintained at the site to handle emergency situations until concrete or backfill is placed.

C. Unauthorized excavations for footings shall be filled with concrete to required elevations, unless other methods of filling are authorized by ENGEO.

D. Excavated earth material which is suitable for engineered fill or backfill, as determined by ENGEO, shall be conditioned for reuse and properly stockpiled for later filling and backfilling operations as specified under Section 2.02, "Soil Materials."

E. Abandoned sewers, piping, and other utilities encountered during excavating shall be removed and the resulting excavations shall be backfilled with engineered fill as required by ENGEO.

F. Any active utility lines encountered shall be reported immediately to the Owner's Representative and authorities involved. The Owner and proper authorities shall be permitted free access to take the measures deemed necessary to repair, relocate, or remove the obstruction as determined by the responsible authority or Owner's Representative.

3.04 SUBGRADE PREPARATION

A. All brush and other rubbish, as well as trees and root systems not marked for saving, shall be removed from the site and legally disposed of.

B. Any existing structures, foundations, underground storage tanks, or debris must be removed from the site prior to any building, grading, or fill operations. Septic tanks, including all drain fields and other lines, if encountered, must be totally removed. The resulting depressions shall be properly prepared and filled to the satisfaction of ENGEO.

C. Vegetation and organic topsoil shall be removed from the surface upon which the fill is to be placed and either removed and legally disposed of or stockpiled for later use in approved landscape areas. The surface shall then be scarified to a depth of at least eight inches until the surface is free from ruts, hummocks, or other uneven features which would tend to prevent uniform compaction by the equipment to be used.

D. After the foundation for the fill has been cleared and scarified, it shall be made uniform and free from large clods. The proper moisture content must be obtained by adding water or aerating. The foundation for the fill shall be compacted at the proper moisture content to a relative compaction as specified herein.

3.05 ENGINEERED FILL

A. Select Material: Fill material shall be "Select" or "Imported Material" as previously specified.

B. Placing and Compacting: Engineered fill shall be constructed by approved and accepted methods. Fill material shall be spread in uniform lifts not exceeding 8 inches in uncompacted thickness. Each layer shall be spread evenly, and thoroughly blade-mixed to obtain uniformity of material. Fill material which does not contain sufficient moisture as specified by ENGEO shall be sprinkled with water; if it contains excess moisture it shall be aerated or blended with drier material to achieve the proper water content. Select material and water shall then be thoroughly mixed before being compacted.

C. Unless otherwise specified in the Geotechnical Exploration report, each layer of spread select material shall be compacted to at least 90 percent relative compaction at a moisture content of at least three percent above the optimum moisture content. Minimum compaction in all keyways shall be a minimum of 95 percent with a minimum moisture content of at least 1 percentage point above optimum.

D. Unless otherwise specified in the Geotechnical Exploration report or otherwise required by the local authorities, the upper 6 inches of engineered fill in areas to receive pavement shall be compacted to at least 95 percent relative compaction with a minimum moisture content of at least 3 percentage points above optimum.

E. Testing and Observation of Fill: The work shall consist of field observation and testing to determine that each layer has been compacted to the required density and that the required moisture is being obtained. Any layer or portion of a layer that does not attain the compaction required shall be reworked until the required density is obtained.

F. Compaction: Compaction shall be by sheepsfoot rollers, multiple-wheel steel or pneumatic-tired rollers or other types of acceptable compaction equipment. Rollers shall be of such design that they will be able to compact the fill to the specified compaction. Rolling shall be accomplished while the fill material is within the specified moisture content range. Rolling of each layer must be continuous so that the required compaction may be obtained uniformly throughout each layer.

G. Fill slopes shall be constructed by overfilling the design slopes and later cutting back the slopes to the design grades. No loose soil will be permitted on the faces of the finished slopes.

H. Strippings and topsoil shall be stockpiled as approved by Owner, then placed in accordance with ENGEO's recommendations to a minimum thickness of 6 inches and a maximum thickness of 12 inches over exposed open space cut slopes which are 3:1 or flatter, and track walked to the satisfaction of ENGEO.

I. Final Prepared Subgrade: Finish blading and smoothing shall be performed as necessary to produce the required density, with a uniform surface, smooth and true to grade.

3.06 BACKFILLING

A. Backfill shall not be placed against footings, building walls, or other structures until approved by ENGEO.

B. Backfill material shall be Select Material as specified for engineered fill.

C. Backfill shall be placed in 6-inch layers, leveled, rammed, and tamped in place. Each layer shall be compacted with suitable compaction equipment to 90 percent relative compaction at a moisture content of at least 3 percent above optimum.

3.07 TRENCHING AND BACKFILLING FOR UTILITIES

A. Trenching:

1. Trenching shall include the removal of material and obstructions, the installation and removal of sheeting and bracing and the control of water as necessary to provide the required utilities and services.

2. Trenches shall be excavated to the lines, grades, and dimensions indicated on the Drawings. Maximum allowable trench width shall be the outside diameter of the pipe plus 24 inches, inclusive of any trench bracing.

3. When the trench bottom is a soft or unstable material as determined by ENGEO, it shall be made firm and solid by removing said unstable material to a sufficient depth and replacing it with on-site material compacted to 90 percent minimum relative compaction.

4. Where water is encountered in the trench, the contractor must provide materials necessary to drain the water and stabilize the bed.

B. Backfilling:

1. Trenches must be backfilled within 2 days of excavation to minimize desiccation.

2. Bedding material shall be sand and shall not extend more than 6 inches above any utility lines.

3. Backfill material shall be select material.

4. Trenches shall be backfilled as indicated or required and compacted with suitable equipment to 90 percent minimum relative compaction at the required moisture content.

3.08 SUBDRAINS

A. Trenches for subdrain pipe shall be excavated to a minimum width equal to the outside diameter of the pipe plus at least 12 inches and to a depth of approximately 2 inches below the grade established for the invert of the pipe, or as indicated on the Drawings.

B. The space below the pipe invert shall be filled with a layer of Class 2 permeable material, upon which the pipe shall be laid with perforations down. Sections shall be joined as recommended by the pipe manufacturer.

C. Rocks, bricks, broken concrete, or other hard material shall not be used to give intermediate support to pipes. Large stones or other hard objects shall not be left in contact with the pipes.

D. Excavations for subdrains shall be filled as required to fill voids and prevent settlement without damaging the subdrain pipe. Alternatively, excavations for subdrains may be filled with Class 1 permeable material (as defined in Section 2.06) wrapped in Filter Fabric (as defined in Section 2.05).

3.09 AGGREGATE DRAINAGE FILL

A. ENGEO shall approve finished subgrades before aggregate drainage fill is installed.

B. Pipes, drains, conduits, and any other mechanical or electrical installations shall be in place before any aggregate drainage fill is placed. Backfill at walls to elevation of drainage fill shall be in place and compacted.

C. Aggregate drainage fill under slabs and concrete paving shall be the minimum uniform thickness after compaction of dimensions indicated on Drawings. Where not indicated, minimum thickness after compaction shall be 4 inches.

D. Aggregate drainage fill shall be rolled to form a well-compacted bed.

E. The finished aggregate drainage fill must be observed and approved by ENGEO before proceeding with any subsequent construction over the compacted base or fill.

3.10 SAND CUSHION

A. A sand cushion shall be placed over the vapor retarder membrane under concrete slabs on grade. Sand cushion shall be placed in uniform thickness as indicated on the Drawings. Where not indicated, the thickness shall be 2 inches.

3.11 FINISH GRADING

A. All areas must be finish graded to elevations and grades indicated on the Drawings. In areas to receive topsoil and landscape planting, finish grading shall be performed to a uniform 6 inches below the grades and elevations indicated on the Drawings, and brought to final grade with topsoil.

3.12 DISPOSAL OF WASTE MATERIALS

A. Excess earth materials and debris shall be removed from the site and disposed of in a legal manner. Location of dump site and length of haul are the Contractor's responsibility.

PART II - GEOGRID SOIL REINFORCEMENT

1. DESCRIPTION:

Work shall consist of furnishing geogrid soil reinforcement for use in construction of reinforced soil slopes and retention systems.

2. GEOGRID MATERIAL:

2.1 The specific geogrid material shall be preapproved by ENGEO.

2.2 The geogrid shall be a regular network of integrally connected polymer tensile elements with aperture geometry sufficient to permit significant mechanical interlock with the surrounding soil or rock. The geogrid structure shall be dimensionally stable and able to retain its geometry under construction stresses and shall have high resistance to damage during construction, to ultraviolet degradation, and to all forms of chemical and biological degradation encountered in the soil being reinforced.

2.3 The geogrids shall have an Allowable Strength (T_a) and Pullout Resistance, for the soil type(s) indicated, as listed in Table I.

2.4 Certifications: The Contractor shall submit a manufacturer's certification that the geogrids supplied meet the respective index criteria set when geogrid was approved by ENGEO, measured in full accordance with all test methods and standards specified. In case of dispute over validity of values, the Contractor will supply test data from an ENGEO-approved laboratory to support the certified values submitted.

3. CONSTRUCTION:

3.1 Delivery, Storage, and Handling: Contractor shall check the geogrid upon delivery to ensure that the proper material has been received. During all periods of shipment and storage, the geogrid shall be protected from temperatures greater than 140 °F, mud, dirt, dust, and debris. Manufacturer's recommendations in regard to protection from direct sunlight must also be followed. At the time of installation, the geogrid will be rejected if it has defects, tears, punctures, flaws, deterioration, or damage incurred during manufacture, transportation, or storage. If approved by ENGEO, torn or punctured sections may be repaired by placing a patch over the damaged area. Any geogrid damaged during storage or installation shall be replaced by the Contractor at no additional cost to the owner.

3.2 On-Site Representative: Geogrid material suppliers shall provide a qualified and experienced representative on site at the initiation of the project, for a minimum of three days, to assist the Contractor and ENGEO personnel at the start of construction. If there is more than one slope on a project, this criterion will apply to construction of the initial slope only. The representative shall also be available on an as-needed basis, as requested by ENGEO, during construction of the remaining slope(s).

3.3 Geogrid reinforcement may be joined with mechanical connections or overlaps as recommended and approved by the Manufacturer. Joints shall not be placed within 6 feet of the slope face, within 4 feet below top of slope, nor horizontally or vertically adjacent to another joint.

3.4 Geogrid Placement: The geogrid reinforcement shall be installed in accordance with the manufacturer's recommendations. The geogrid reinforcement shall be placed within the layers of the compacted soil as shown on the plans or as directed.

The geogrid reinforcement shall be placed in continuous longitudinal strips in the direction of main reinforcement. However, if the Contractor is unable to complete a required length with a single continuous length of geogrid, a joint may be made with the Manufacturer's approval. Only one joint per length of geogrid shall be allowed. This joint shall be made for the full width of the strip by using a similar material with similar strength. Joints in geogrid reinforcement shall be pulled and held taut during fill placement.

Adjacent strips, in the case of 100 percent coverage in plan view, need not be overlapped. The minimum horizontal coverage is 50 percent, with horizontal spacings between reinforcement no greater than 40 inches. Horizontal coverage of less than 100 percent shall not be allowed unless specifically detailed in the construction drawings.

Adjacent rolls of geogrid reinforcement shall be overlapped or mechanically connected where exposed in a wrap around face system, as applicable.

The Contractor may place only that amount of geogrid reinforcement required for immediately pending work to prevent undue damage. After a layer of geogrid reinforcement has been placed, the next succeeding layer of soil shall be placed and compacted as appropriate. After the specified soil layer has been placed, the next geogrid reinforcement layer shall be installed. The process shall be repeated for each subsequent layer of geogrid reinforcement and soil.

Geogrid reinforcement shall be placed to lay flat and pulled tight prior to backfilling. After a layer of geogrid reinforcement has been placed, suitable means, such as pins or small piles of soil, shall be used to hold the geogrid reinforcement in position until the subsequent soil layer can be placed.

Under no circumstances shall a track-type vehicle be allowed on the geogrid reinforcement before at least six inches of soil have been placed. Turning of tracked vehicles should be kept to a minimum to prevent tracks from displacing the fill and the geogrid reinforcement. If approved by the Manufacturer, rubber-tired equipment may pass over the geosynthetic reinforcement at slow speeds, less than 10 mph. Sudden braking and sharp turning shall be avoided.

During construction, the surface of the fill should be kept approximately horizontal. Geogrid reinforcement shall be placed directly on the compacted horizontal fill surface. Geogrid reinforcements are to be placed within three inches of the design elevations and extend the length as shown on the elevation view unless otherwise directed by ENGEO. Correct orientation of the geogrid reinforcement shall be verified by ENGEO.

Table I
Allowable Geogrid Strength
With Various Soil Types
For Geosynthetic Reinforcement In
Mechanically Stabilized Earth Slopes

(Geogrid Pullout Resistance and Allowable Strengths vary with reinforced backfill used due to soil anchorage and site damage factors. Guidelines are provided below.)

SOIL TYPE		MINIMUM ALLOWABLE STRENGTH, T_a (lb/ft)*		
		GEOGRID Type I	GEOGRID Type II	GEOGRID Type III
A.	Gravels, sandy gravels, and gravel-sand-silt mixtures (GW, GP, GC, GM & SP)**	2400	4800	7200
B.	Well graded sands, gravelly sands, and sand-silt mixtures (SW & SM)**	2000	4000	6000
C.	Silts, very fine sands, clayey sands and clayey silts (SC & ML)**	1000	2000	3000
D.	Gravelly clays, sandy clays, silty clays, and lean clays (CL)**	1600	3200	4800

* All partial Factors of Safety for reduction of design strength are included in listed values. Additional factors of safety may be required to further reduce these design strengths based on site conditions.

** Unified Soil Classifications.

PART III - GEOTEXTILE SOIL REINFORCEMENT

1. DESCRIPTION:

Work shall consist of furnishing geotextile soil reinforcement for use in construction of reinforced soil slopes.

2. GEOTEXTILE MATERIAL:

2.1 The specific geotextile material and supplier shall be preapproved by ENGEO.

2.2 The geotextile shall have a high tensile modulus and shall have high resistance to damage during construction, to ultraviolet degradation, and to all forms of chemical and biological degradation encountered in the soil being reinforced.

2.3 The geotextiles shall have an Allowable Strength (T_a) and Pullout Resistance, for the soil type(s) indicated as listed in Table II.

2.4 Certification: The Contractor shall submit a manufacturer's certification that the geotextiles supplied meet the respective index criteria set when geotextile was approved by ENGEO, measured in full accordance with all test methods and standards specified. In case of dispute over validity of values, the Contractor will supply the data from an ENGEO-approved laboratory to support the certified values submitted.

3. CONSTRUCTION:

3.1 Delivery, Storage and Handling: Contractor shall check the geotextile upon delivery to ensure that the proper material has been received. During all periods of shipment and storage, the geotextile shall be protected from temperatures greater than 140 °F, mud, dirt, dust, and debris. Manufacturer's recommendations in regard to protection from direct sunlight must also be followed. At the time of installation, the geotextile will be rejected if it has defects, tears, punctures, flaws, deterioration, or damage incurred during manufacture, transportation, or storage. If approved by ENGEO, torn or punctured sections may be repaired by placing a patch over the damaged area. Any geotextile damaged during storage or installation shall be replaced by the Contractor at no additional cost to the owner.

3.2 On-Site Representative: Geotextile material suppliers shall provide a qualified and experienced representative on site at the initiation of the project, for a minimum of three days, to assist the Contractor and ENGEO personnel at the start of construction. If there is more than one slope on a project, this criterion will apply to construction of the initial slope only. The representative shall also be available on an as-needed basis, as requested by ENGEO, during construction of the remaining slope(s).

3.3 Geotextile Placement: The geotextile reinforcement shall be installed in accordance with the manufacturer's recommendations. The geotextile reinforcement shall be placed within the layers of the compacted soil as shown on the plans or as directed.

The geotextile reinforcement shall be placed in continuous longitudinal strips in the direction of main reinforcement. Joints shall not be used with geotextiles.

Adjacent strips, in the case of 100 percent coverage in plan view, need not be overlapped. The minimum horizontal coverage is 50 percent, with horizontal spacings between reinforcement no greater than 40 inches. Horizontal coverage of less than 100 percent shall not be allowed unless specifically detailed in the construction drawings.

Adjacent rolls of geotextile reinforcement shall be overlapped or mechanically connected where exposed in a wrap around face system, as applicable.

The Contractor may place only that amount of geotextile reinforcement required for immediately pending work to prevent undue damage. After a layer of geotextile reinforcement has been placed, the succeeding layer of soil shall be placed and compacted as appropriate. After the specified soil layer has been placed, the next geotextile reinforcement layer shall be installed. The process shall be repeated for each subsequent layer of geotextile reinforcement and soil.

Geosynthetic reinforcement shall be placed to lay flat and be pulled tight prior to backfilling. After a layer of geotextile reinforcement has been placed, suitable means, such as pins or small piles of soil, shall be used to hold the geotextile reinforcement in position until the subsequent soil layer can be placed.

Under no circumstances shall a track-type vehicle be allowed on the geotextile reinforcement before at least six inches of soil has been placed. Turning of tracked vehicles should be kept to a minimum to prevent tracks from displacing the fill and the geotextile reinforcement. If approved by the Manufacturer, rubber-tired equipment may pass over the geotextile reinforcement as slow speeds, less than 10 mph. Sudden braking and sharp turning shall be avoided.

During construction, the surface of the fill should be kept approximately horizontal. Geotextile reinforcement shall be placed directly on the compacted horizontal fill surface. Geotextile reinforcements are to be placed within three inches of the design elevations and extend the length as shown on the elevation view unless otherwise directed by ENGEO. Correct orientation of the geotextile reinforcement shall be verified by ENGEO.

Table II
Allowable Geotextile Strength
With Various Soil Types
For Geosynthetic Reinforcement In
Mechanically Stabilized Earth Slopes

(Geotextile Pullout Resistance and Allowable Strengths vary with reinforced backfill used due to soil anchorage and site damage factors. Guidelines are provided below.)

SOIL TYPE		MINIMUM ALLOWABLE STRENGTH, T_a (lb/ft)*		
		GEOTEXTILE Type I	GEOTEXTILE Type II	GEOTEXTILE Type III
A.	Gravels, sandy gravels, and gravel-sand-silt mixtures (GW, GP, GC, GM & SP)**	2400	4800	7200
B.	Well graded sands, gravelly sands, and sand-silt mixtures (SW & SM)**	2000	4000	6000
C.	Silts, very fine sands, clayey sands and clayey silts (SC & ML)**	1000	2000	3000
D.	Gravelly clays, sandy clays, silty clays, and lean clays (CL)**	1600	3200	4800

* All partial Factors of Safety for reduction of design strength are included in listed values. Additional factors of safety may be required to further reduce these design strengths based on site conditions.

** Unified Soil Classifications.

PART IV - EROSION CONTROL MAT OR BLANKET

1. DESCRIPTION:

Work shall consist of furnishing and placing a synthetic erosion control mat and/or degradable erosion control blanket for slope face protection and lining of runoff channels.

2. EROSION CONTROL MATERIALS:

2.1 The specific erosion control material and supplier shall be pre-approved by ENGEO.

2.2 Certification: The Contractor shall submit a manufacturer's certification that the erosion mat/blanket supplied meets the criteria specified when the material was approved by ENGEO. The manufacturer's certification shall include a submittal package of documented test results that confirm the property values. In case of a dispute over validity of values, the Contractor will supply property test data from an ENGEO-approved laboratory, to support the certified values submitted. Minimum average roll values, per ASTM D 4759, shall be used for conformance determinations.

3. CONSTRUCTION:

3.1 Delivery, Storage, and Handling: Contractor shall check the erosion control material upon delivery to ensure that the proper material has been received. During all periods of shipment and storage, the erosion mat shall be protected from temperatures greater than 140 °F, mud, dirt, and debris. Manufacturer's recommendations in regard to protection from direct sunlight must also be followed. At the time of installation, the erosion mat/blanket shall be rejected if it has defects, tears, punctures, flaws, deterioration, or damage incurred during manufacture, transportation, or storage. If approved by ENGEO, torn or punctured sections may be removed by cutting OUT a section of the mat. The remaining ends should be overlapped and secured with ground anchors. Any erosion mat/blanket damaged during storage or installation shall be replaced by the Contractor at no additional cost to the Owner.

3.2 On-Site Representative: Erosion control material suppliers shall provide a qualified and experienced representative on site, for a minimum of one day, to assist the Contractor and ENGEO personnel at the start of construction. If there is more than one slope on a project, this criteria will apply to construction of the initial slope only. The representative shall be available on an as-needed basis, as requested by ENGEO, during construction of the remaining slope(s).

3.3 Placement: The erosion control material shall be placed and anchored on a smooth graded, firm surface approved by the Engineer. Anchoring terminal ends of the erosion control material shall be accomplished through use of key trenches. The material in the trenches shall be anchored to the soil on maximum 1½ foot centers. Topsoil, if required by construction drawings, placed over final grade prior to installation of the erosion control material shall be limited to a depth not exceeding 3 inches.

3.4 Erosion control material shall be anchored, overlapped, and otherwise constructed to ensure performance until vegetation is well established. Anchors shall be as designated on the construction drawings, with a minimum of 12 inches length, and shall be spaced as designated on the construction drawings, with a maximum spacing of 4 feet.

3.5 Soil Filling: If noted on the construction drawings, the erosion control mat shall be filled with a fine grained topsoil, as recommended by the manufacturer. Soil shall be lightly raked or brushed on/into the mat to fill the mat voids or to a maximum depth of 1 inch.

PART V - GEOSYNTHETIC DRAINAGE COMPOSITE

1. DESCRIPTION:

Work shall consist of furnishing and placing a geosynthetic drainage system as a subsurface drainage medium for reinforced soil slopes.

2. DRAINAGE COMPOSITE MATERIALS:

2.1 The specific drainage composite material and supplier shall be preapproved by ENGEO.

2.2 The drain shall be of composite construction consisting of a supporting structure or drainage core material surrounded by a geotextile. The geotextile shall encapsulate the drainage core and prevent random soil intrusion into the Continuous Spread Footings structure. The drainage core material shall consist of a three dimensional polymeric material with a structure that permits flow along the core laterally. The core structure shall also be constructed to permit flow regardless of the water inlet surface. The drainage core shall provide support to the geotextile. The fabric shall meet the minimum property requirements for filter fabric listed in Section 2.05C of the Guide Earthwork Specifications.

2.3 A geotextile flap shall be provided along all drainage core edges. This flap shall be of sufficient width for sealing the geotextile to the adjacent drainage structure edge to prevent soil intrusion into the structure during and after installation. The geotextile shall cover the full length of the core.

2.4 The geocomposite core shall be furnished with an approved method of constructing and connecting with outlet pipes or weepholes as shown on the plans. Any fittings shall allow entry of water from the core but prevent intrusion of backfill material into the core material.

2.5 Certification and Acceptance: The Contractor shall submit a manufacturer's certification that the geosynthetic drainage composite meets the design properties and respective index criteria measured in full accordance with all test methods and standards specified. The manufacturer's certification shall include a submittal package of documented test results that confirm the design values. In case of dispute over validity of design values, the Contractor will supply design property test data from an ENGEO-approved laboratory, to support the certified values submitted. Minimum average roll values, per ASTM D 4759, shall be used for determining conformance.

3. CONSTRUCTION:

3.1 Delivery, Storage, and Handling: Contractor shall check the geosynthetic composite upon delivery to ensure that the proper material has been received. During all periods of shipment and storage, the geosynthetic drainage composite shall be protected from temperatures greater than 140 °F, mud, dirt, and debris. Manufacturer's recommendations in regards to protection from direct sunlight must also be followed. At the time of installation, the geosynthetic drainage composite shall be rejected if it has defects, tears, punctures, flaws, deterioration, or damage incurred during manufacture, transportation, or storage. If approved by ENGEO, torn or punctured sections may be removed or repaired. Any geosynthetic drainage composite damaged during storage or installation shall be replaced by the Contractor at no additional cost to the Owner.

3.2 On-Site Representative: Geosynthetic drainage composite material suppliers shall provide a qualified and experienced representative on site, for a minimum of one half day, to assist the Contractor and ENGEO personnel at the start of construction with directions on the use of drainage composite. If there is more than one application on a project, this criterion will apply to construction of the initial application only. The representative shall also be available on an as-needed basis, as requested by ENGEO, during construction of the remaining applications.

3.3 Placement: The soil surface against which the geosynthetic drainage composite is to be placed shall be free of debris and inordinate irregularities that will prevent intimate contact between the soil surface and the drain.

3.4 Seams: Edge seams shall be formed by utilizing the flap of the geotextile extending from the geocomposite's edge and lapping over the top of the fabric of the adjacent course. The fabric flap shall be securely fastened to the adjacent fabric by means of plastic tape or non-water-soluble construction adhesive, as recommended by the supplier. Where vertical splices are necessary at the end of a geocomposite roll or panel, an 8-inch-wide continuous strip of geotextile may be placed, centering over the seam and continuously fastened on both sides with plastic tape or non-water-soluble construction adhesive. As an alternative, rolls of geocomposite drain material may be joined together by turning back the fabric at the roll edges and interlocking the cuspidations approximately 2 inches. For overlapping in this manner, the fabric shall be lapped and tightly taped beyond the seam with tape or adhesive. Interlocking of the core shall always be made with the upstream edge on top in the direction of water flow. To prevent soil intrusion, all exposed edges of the geocomposite drainage core edge must be covered. Alternatively, a 12-inch-wide strip of fabric may be utilized in the same manner, fastening it to the exposed fabric 8 inches in from the edge and folding the remaining flap over the core edge.

3.5 Soil Fill Placement: Structural backfill shall be placed immediately over the geocomposite drain. Care shall be taken during the backfill operation not to damage the geotextile surface of the drain. Care shall also be taken to avoid excessive settlement of the backfill material. The geocomposite drain, once installed, shall not be exposed for more than seven days prior to backfilling.

LIST OF FIGURES

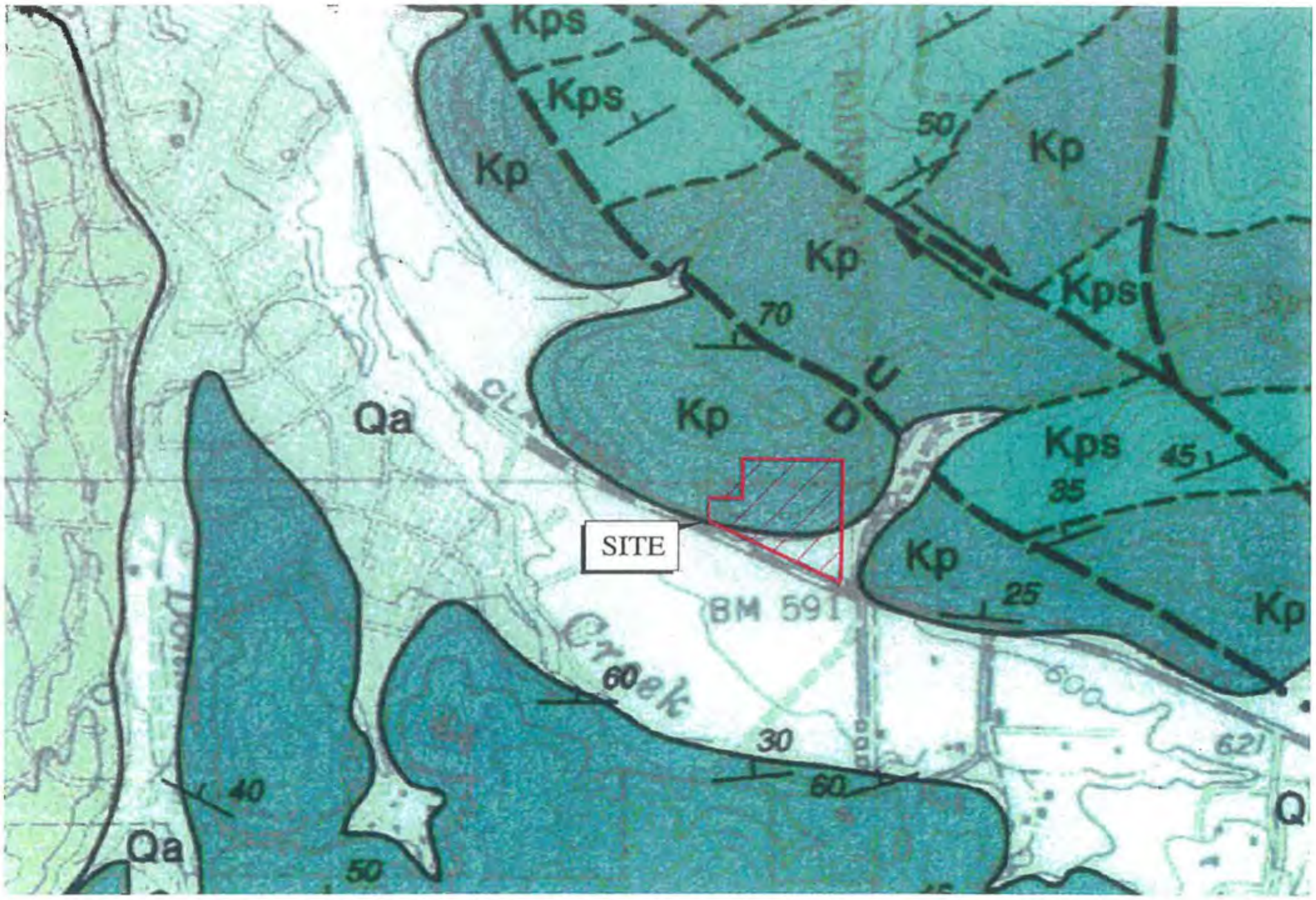
BASE MAP SOURCE: MS STREETS AND TRIPS

ENGEO INCORPORATED
EXCELLENT SERVICE SINCE 1971

VICINITY MAP
OAK CREEK CANYON
CLAYTON, CALIFORNIA

PROJECT NO.: 3840.205.202		FIGURE NO.
DATE: FEBRUARY 2008		1
DRAWN BY: JMG	CHECKED BY: TPB	

ORIGINAL FIGURE PRINTED IN COLOR



EXPLANATION

	BEDROCK CONTACT-DASHED WHERE GRADATIONAL OR APPROXIMATELY LOCATED		SANDSTONE BED
	FAULT-DASHED WHERE INFERRED, DOTTED WHERE CONCEALED, QUERIED WHERE EXISTENCE IS DOUBTFUL ; DOUBLE ARROWS INDICATE STRIKE-SLIP MOVEMENT	Qa	ALLUVIAL GRAVEL, SAND, AND CLAY OF VALLEY AREAS
U	UPTHROWN SIDE	Kp	PANOCHE FORMATION (CLAY SHALE AND CLAYTONE)
D	DOWNTHROWN SIDE RELATIVELY	Kps	PANOCHE FORMATION (SANDSTONE)

STRIKE AND DIP OF STRATA

INCLINED VERTICAL OVERTURNED

N

0 FEET 1000
0 METERS 500

BASE MAP SOURCE: DIBBLE, 2006

ENGEO INCORPORATED EXCELLENT SERVICE SINCE 1971	REGIONAL GEOLOGIC MAP OAK CREEK CANYON CLAYTON, CALIFORNIA	PROJECT NO.: 3840.205.202 DATE: FEBRUARY 2008 DRAWN BY: JMG CHECKED BY: TPB	FIGURE NO. 2

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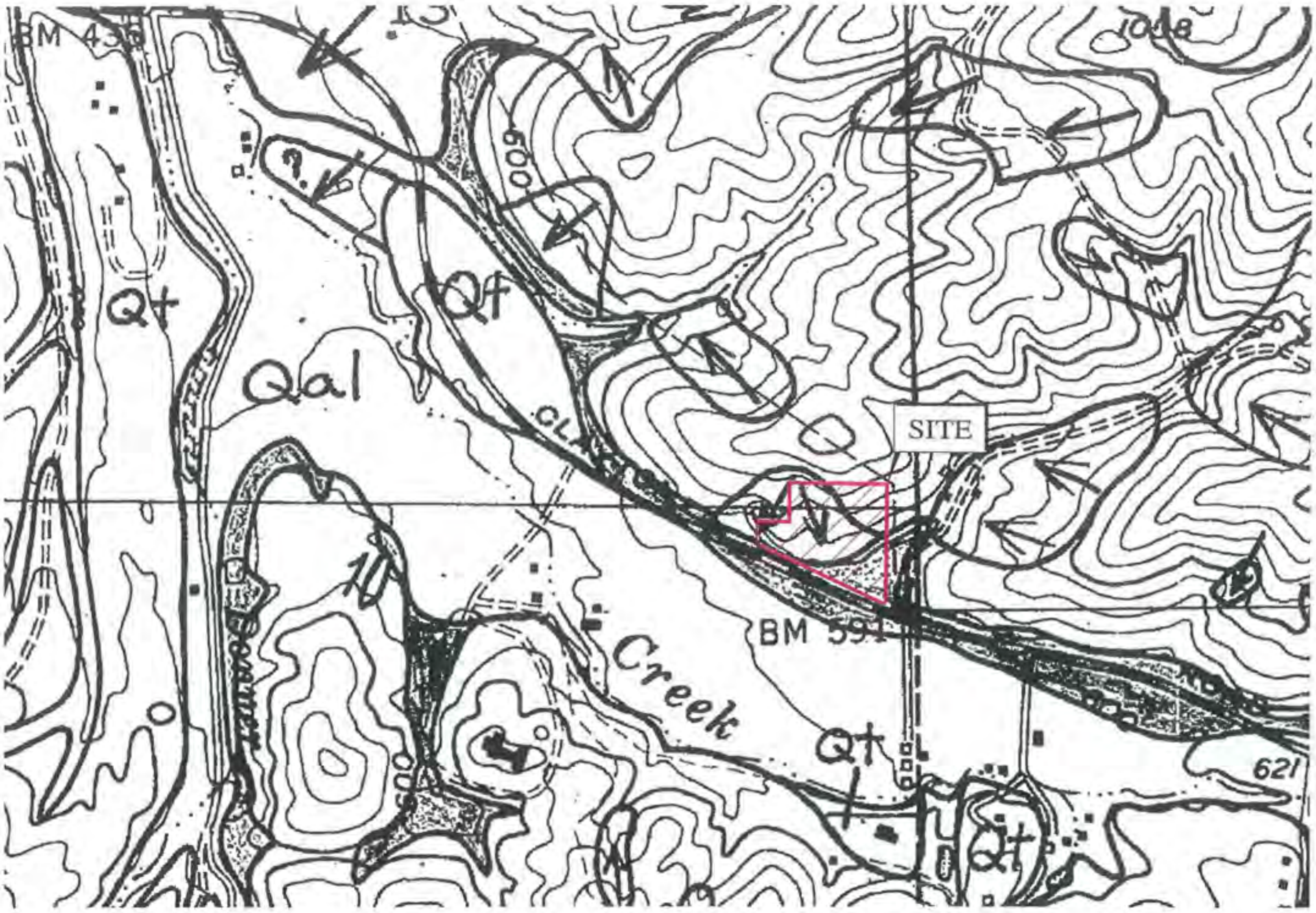
ORIGINAL FIGURE PRINTED IN COLOR



EXPLANATION

	LANDSLIDE DEPOSIT. ARROWS INDICATE GENERAL DIRECTION OF DOWNSLOPE MOVEMENT. QUERIED WHERE UNCERTAIN
Qal	ALLUVIAL DEPOSIT
Qt	ALLUVIAL TERRACE DEPOSIT. QUERIED WHERE UNCERTAIN
	COLLUVIAL DEPOSIT AND/OR SMALL ALLUVIAL FAN DEPOSIT
	BEDROCK. QUERIED WHERE IDENTIFICATION UNCERTAIN

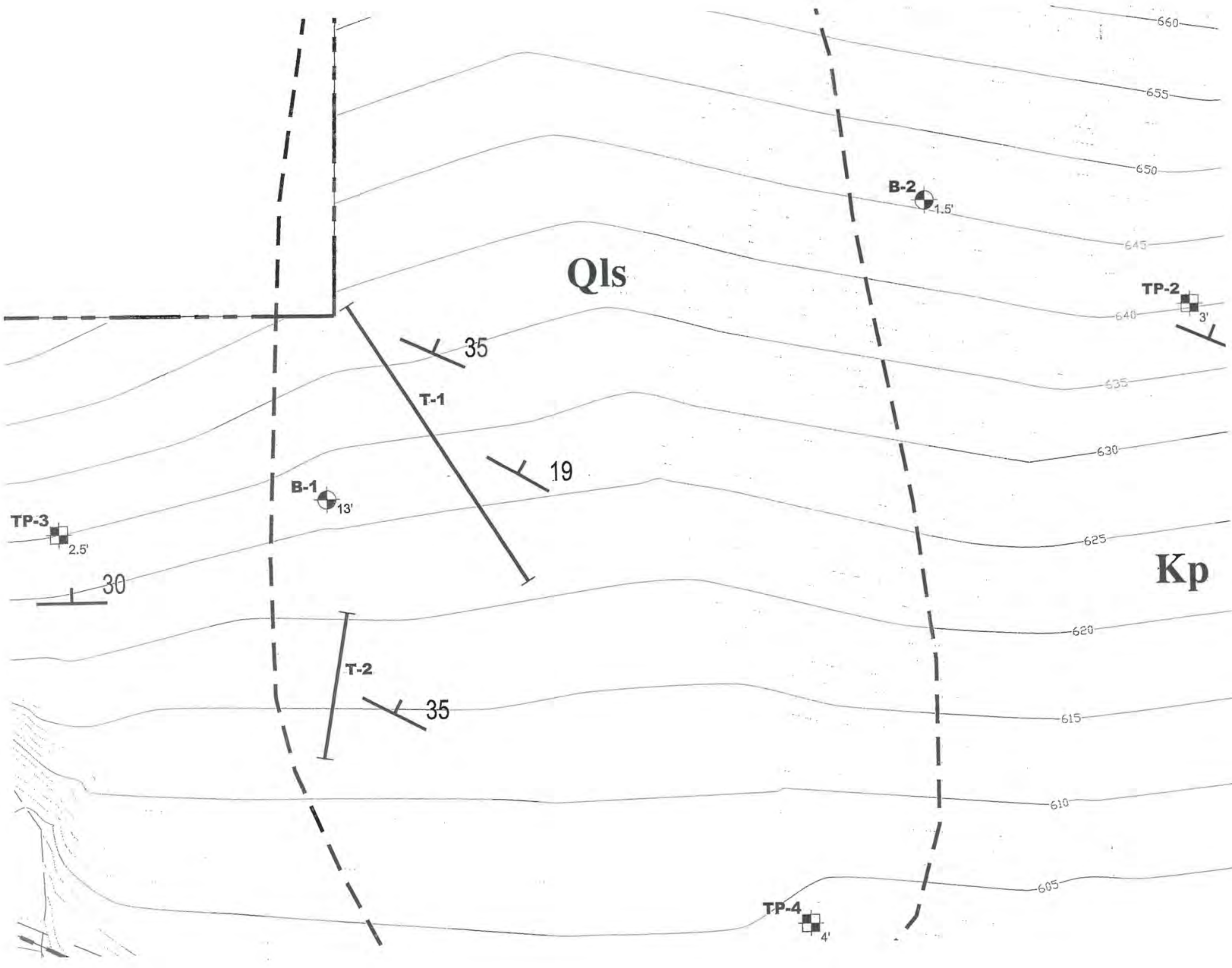
N

0 FEET 1000

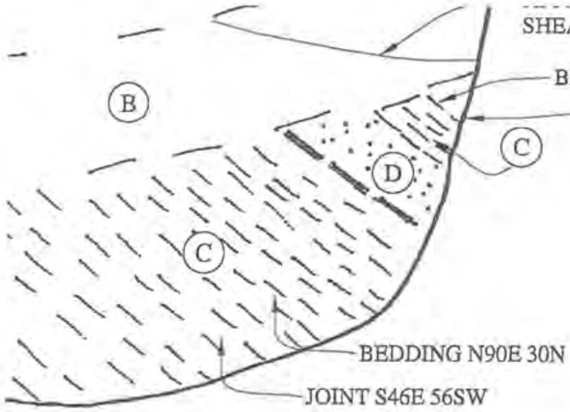
0 METERS 500

BASE MAP SOURCE: NILSEN, 1975

ENGEO INCORPORATED EXCELLENT SERVICE SINCE 1971	REGIONAL LANDSLIDE MAP OAK CREEK CANYON CLAYTON, CALIFORNIA	PROJECT NO.: 3840.205.202 DATE: FEBRUARY 2008 DRAWN BY: JMG CHECKED BY: TPB	FIGURE NO. 3



Qls
Kp
TP-2
3'
TP-3
2.5'
TP-4
4'
B-1
13'
B-2
1.5'
T-1
T-2
35
35
19
30
660
655
650
645
640
635
630
625
620
615
610
605



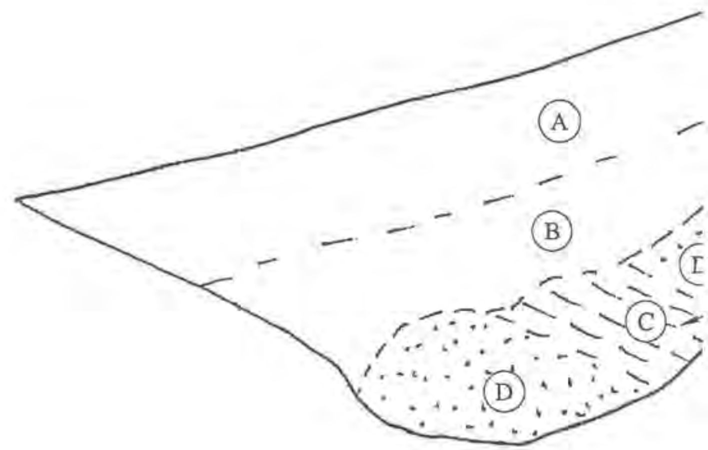
TP-1

0 FEET 5
0 METERS 2.5

TP-4

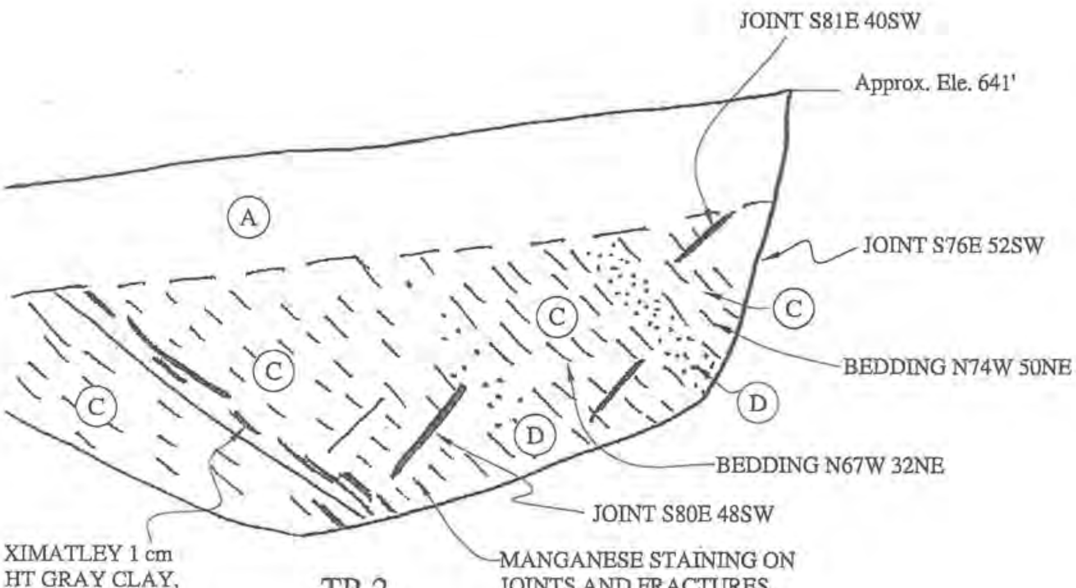
0 FEET 5
0 METERS 2.5

TP-5



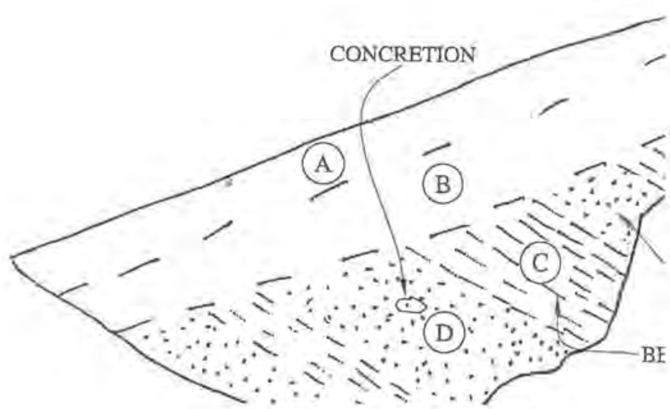
TP-5

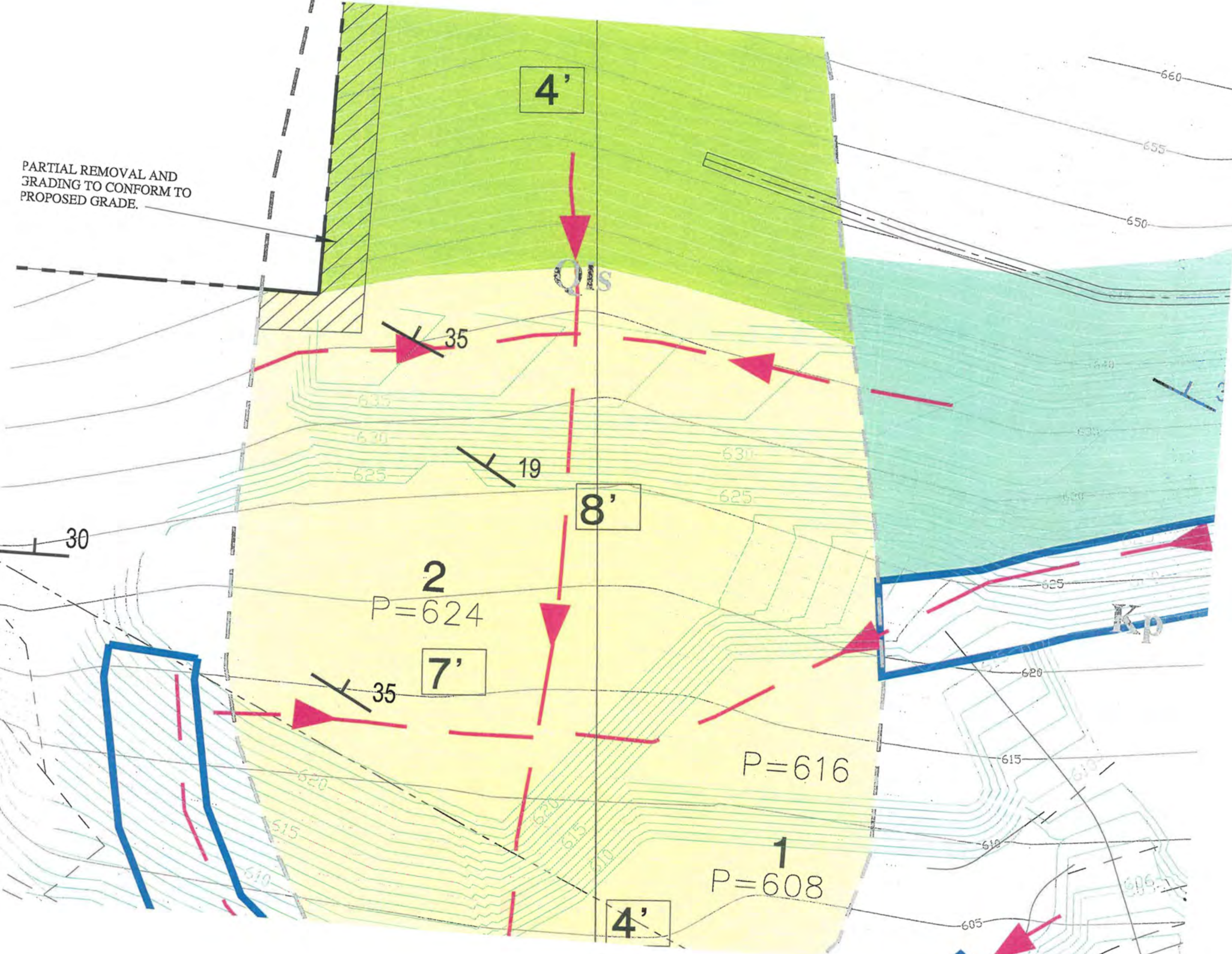
0 FEET 5
0 METERS 2.5



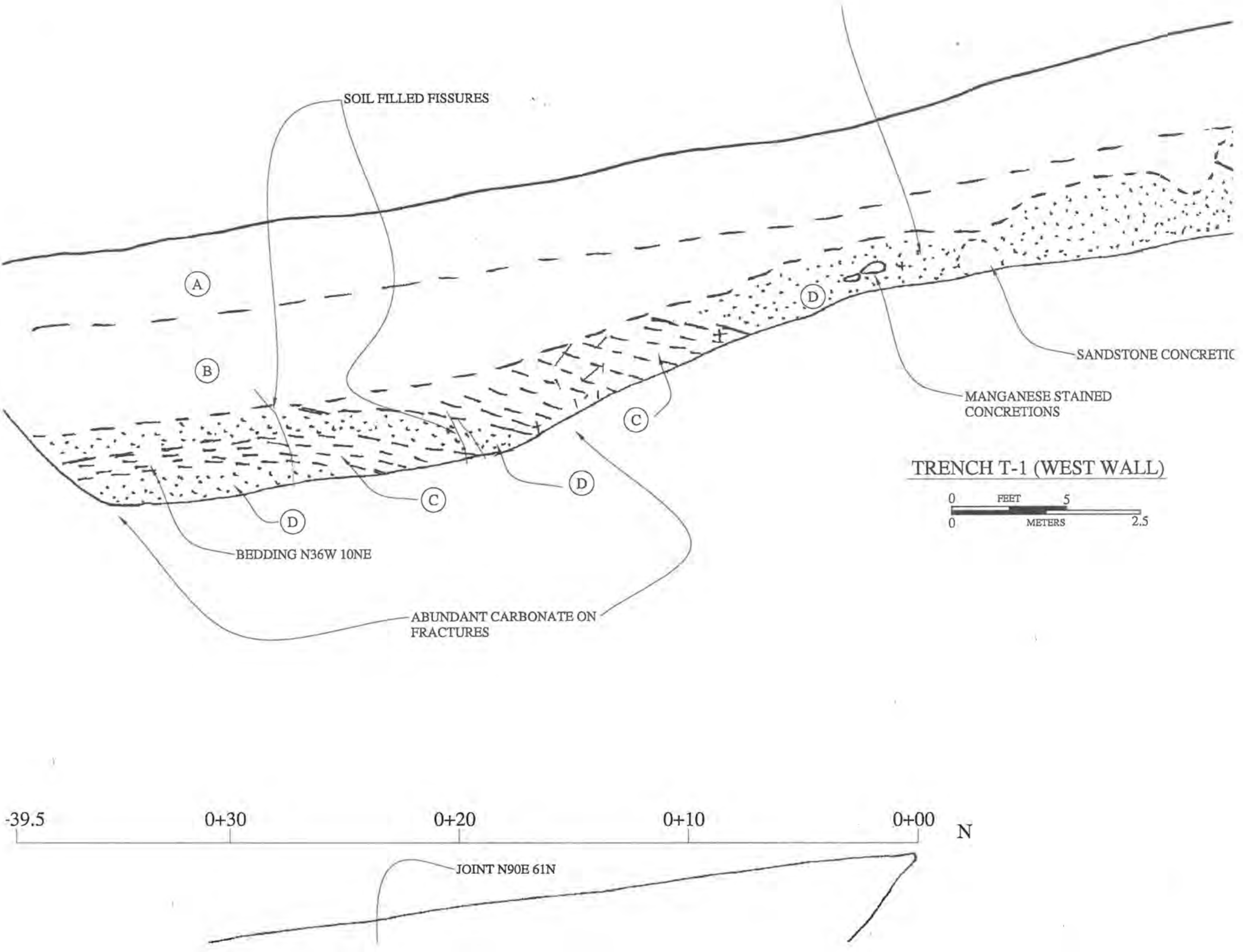
TP-2

0 FEET 5

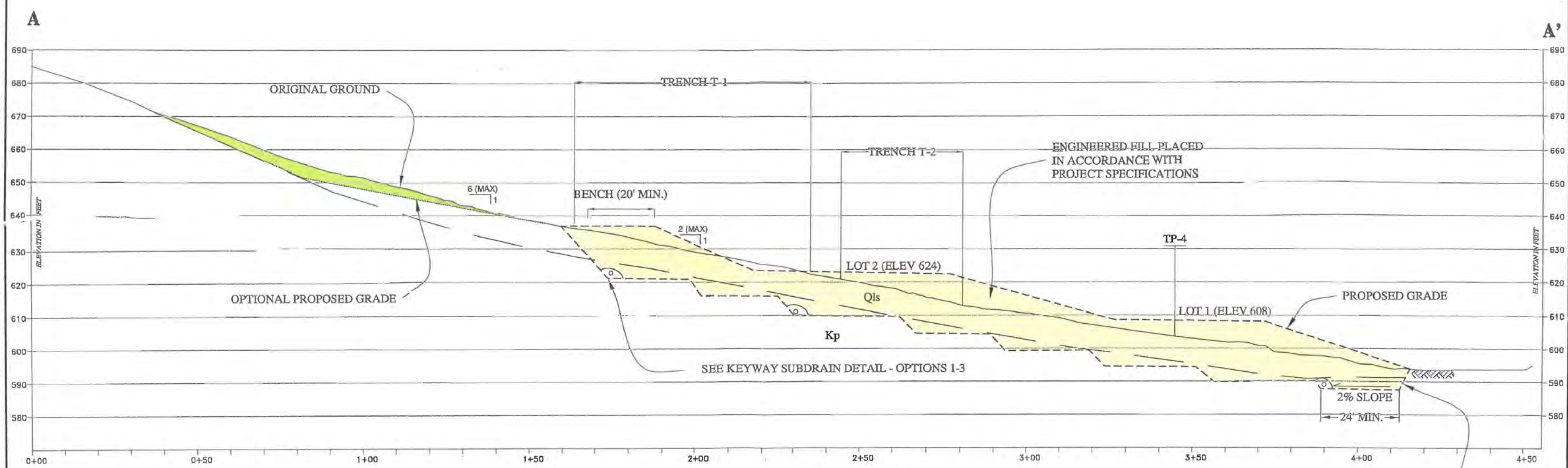




PARTIAL REMOVAL AND
GRADING TO CONFORM TO
PROPOSED GRADE.
4'
Qls
35
19
8'
30
2
P=624
7'
35
P=616
1
P=608
4'
Kp
660
655
650
640
635
630
625
620
615
610
605



SOIL FILLED FISSURES
A
B
C
D
SANDSTONE CONCRETIC
MANGANESE STAINED CONCRETIONS
TRENCH T-1 (WEST WALL)
0
FEET
5
0
METERS
2.5
BEDDING N36W 10NE
ABUNDANT CARBONATE ON FRACTURES
-39.5
0+30
0+20
0+10
0+00
N
JOINT N90E 61N



CROSS SECTION A-A'

1"=30'

EXPLANATION

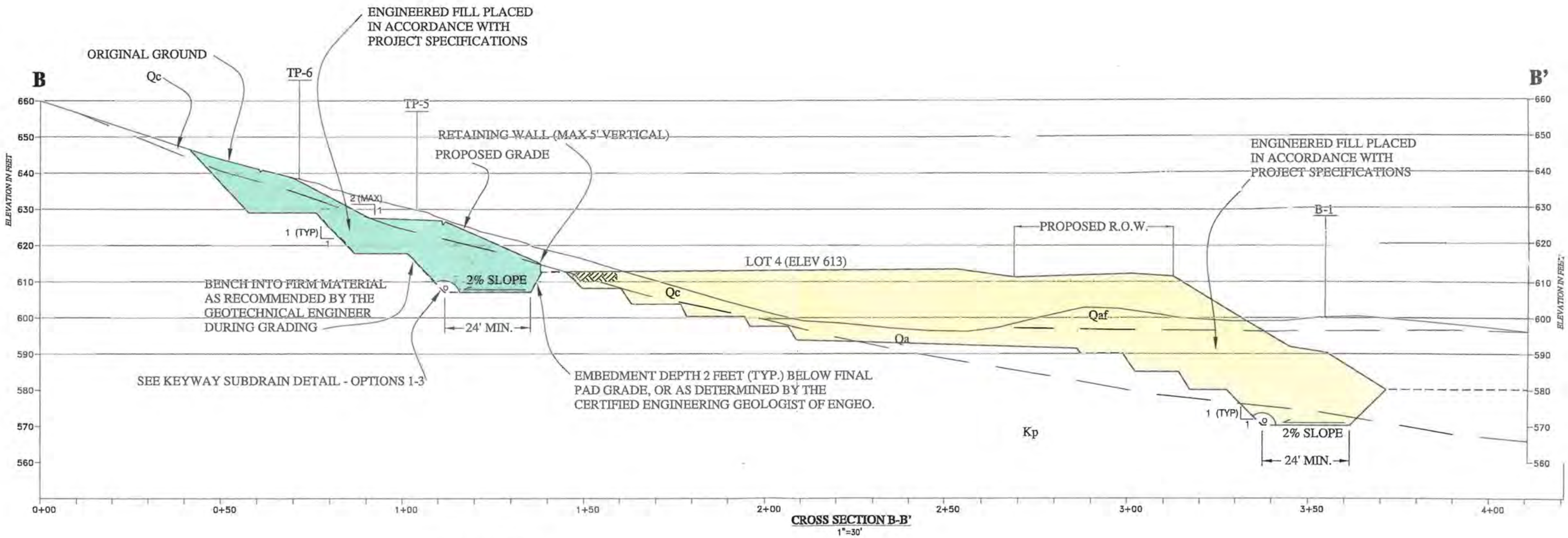
APPROXIMATE AREA OF OPTIONAL UNSUITABLE MATERIAL REMOVAL TO GENERATE FILL

APPROXIMATE LIMITS OF UNSUITABLE MATERIAL REMOVAL AND REPLACEMENT

0 FEET 30
0 METERS 15

ENGEO INCORPORATED
EXCELLENT SERVICE SINCE 1971

CROSS SECTION A-A' OAK CREEK CANYON CLAYTON, CALIFORNIA	PROJECT NO.: 3840.2.052.02		FIGURE NO. 8
	DATE: FEBRUARY 2008		
	DRAWN BY: JMG	CHECKED BY: JB	



EXPLANATION

APPROXIMATE LIMITS OF UNSUITABLE MATERIAL REMOVAL AND REPLACEMENT

APPROXIMATE LIMITS OF SLOPE REBUILD

0 FEET 30
0 METERS 15

ENGEO INCORPORATED
EXCELLENT SERVICE SINCE 1971

CROSS SECTION B-B'
OAK CREEK CANYON
CLAYTON, CALIFORNIA

PROJECT NO.: 3840.2.052.02	FIGURE NO. 9
DATE: FEBRUARY 2008	
DRAWN BY: JMG	CHECKED BY: XXX

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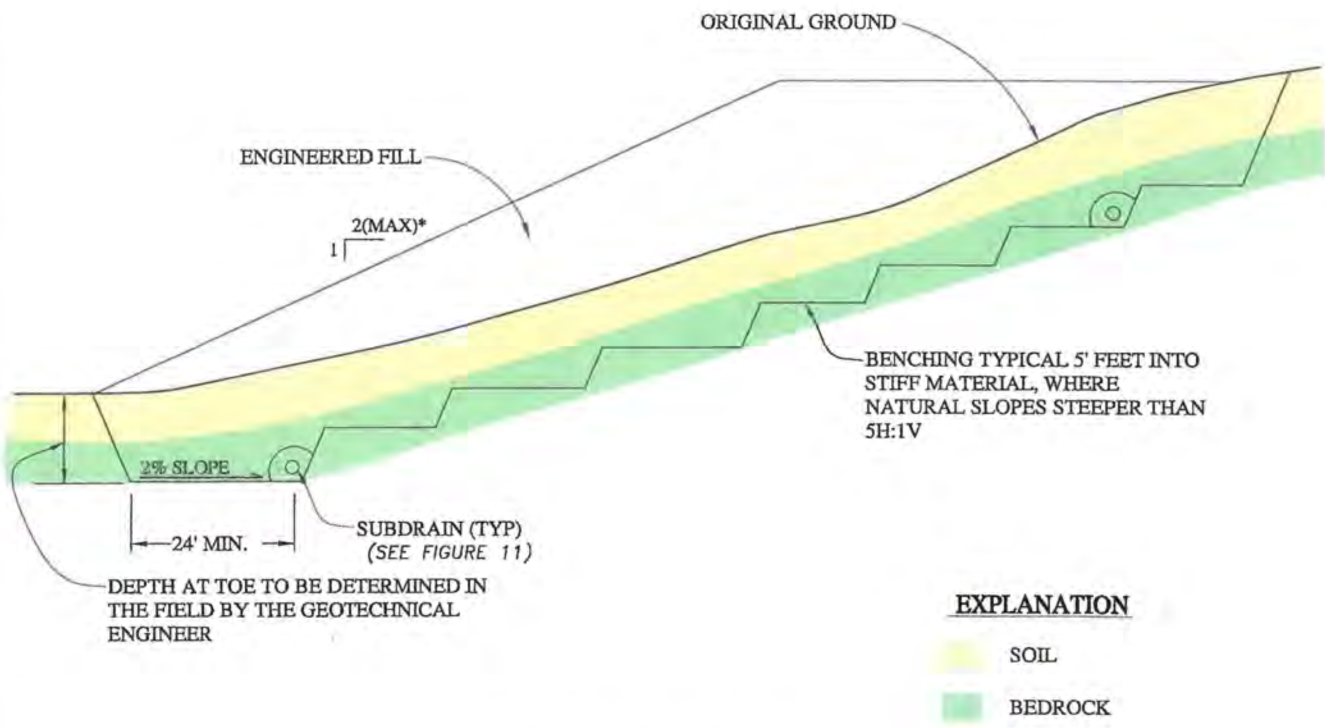
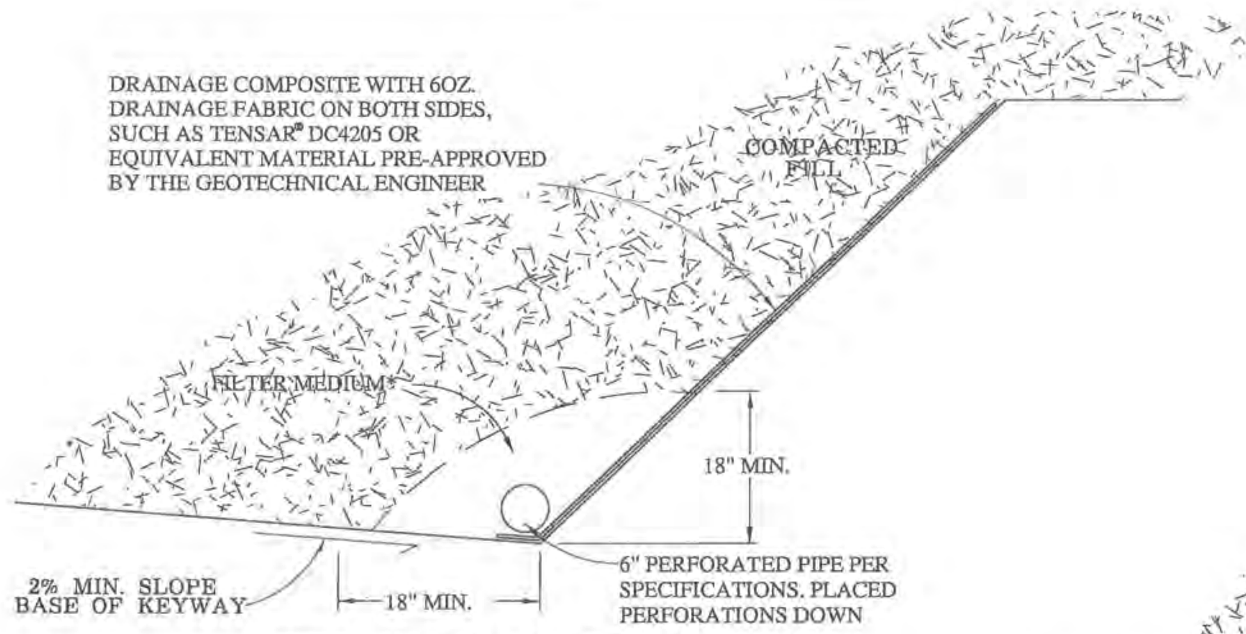


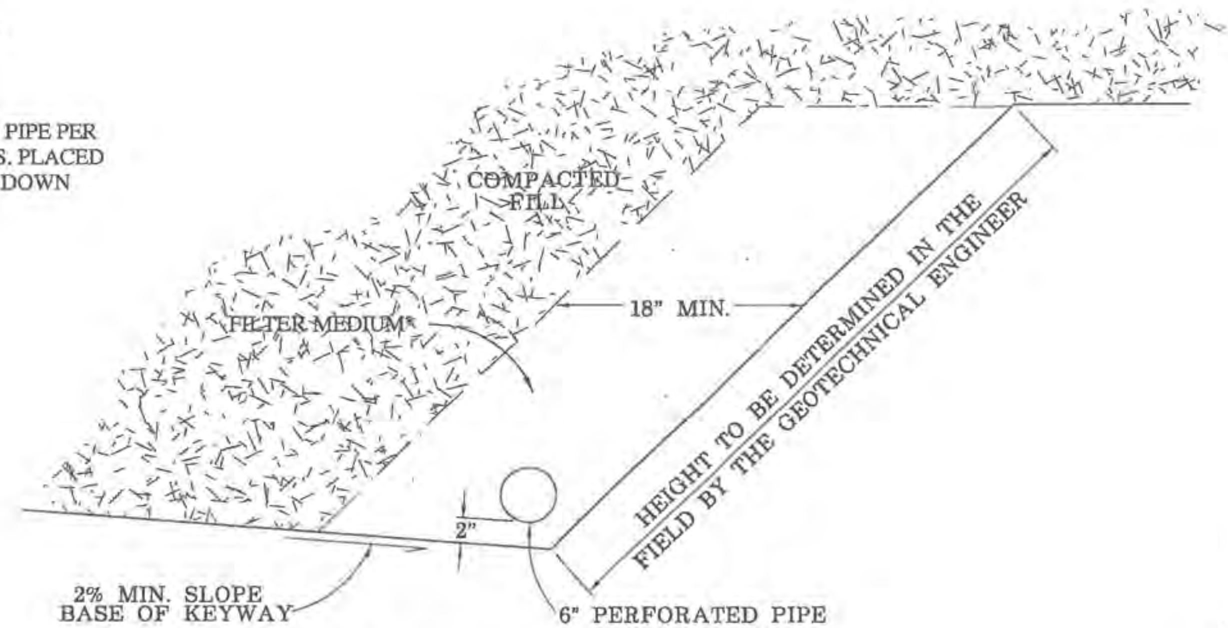
FOR SLOPE HEIGHT AND MAXIMUM SLOPE GRADIENT, REFER TO GRADED SLOPES SECTION OF THE TEXT
**2:1 SLOPE ACCEPTABLE WITH GEOGRID REINFORCEMENT OR THE USE OF SELECT FILL*

NO SCALE

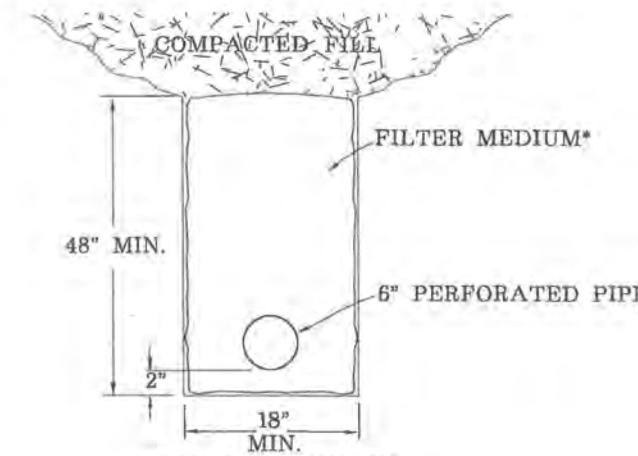
ENGEO INCORPORATED EXCELLENT SERVICE SINCE 1971	TYPICAL KEYWAY SECTION OAK CREEK CANYON CLAYTON, CALIFORNIA	PROJECT NO.: 3840.205.202 DATE: FEBRUARY 2008 DRAWN BY: JMG CHECKED BY: TPB	FIGURE NO. 10



KEYWAY SUBDRAIN - OPTION 1



KEYWAY SUBDRAIN - OPTION 2



SWALE SUBDRAIN

*FILTER MEDIUM

ALTERNATIVE A

CLASS 2 PERMEABLE MATERIAL

MATERIAL SHALL CONSIST OF CLEAN, COARSE SAND AND GRAVEL OR CRUSHED STONE, CONFORMING TO THE FOLLOWING GRADING REQUIREMENTS:

SIEVE SIZE	% PASSING SIEVE
1"	100
3/4"	90-100
3/8"	40-100
#4	25-40
#8	18-33
#30	5-15
#50	0-7
#200	0-3

ALTERNATIVE B

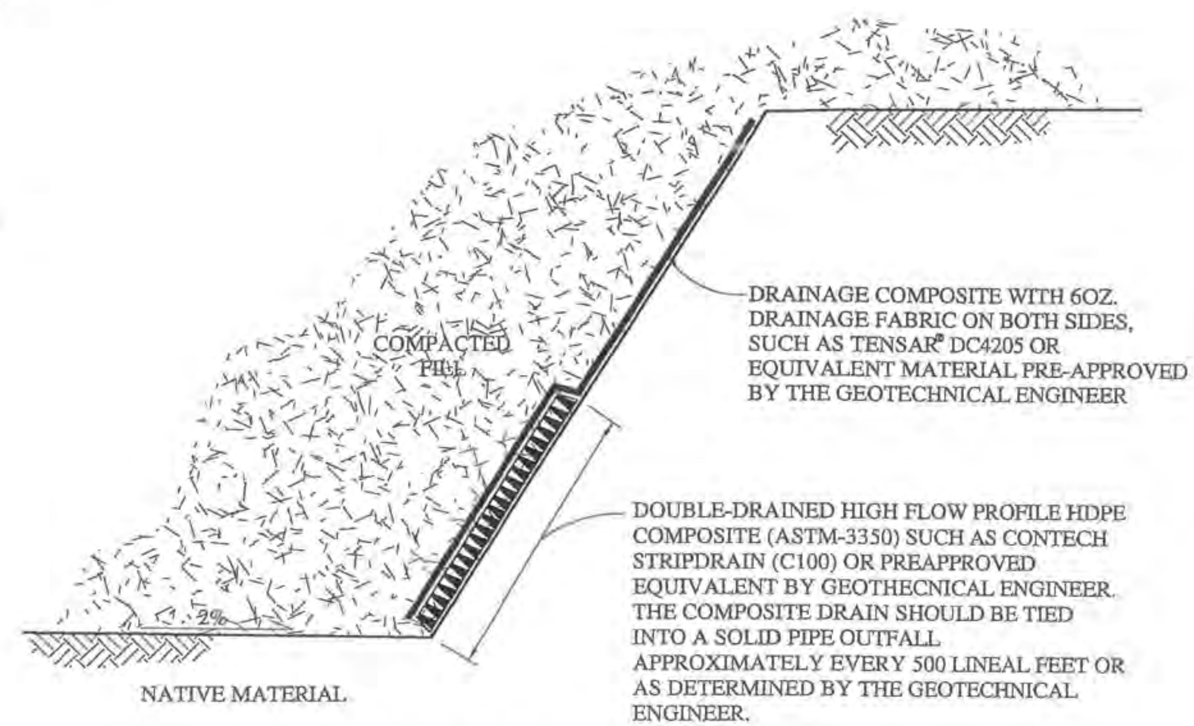
CLEAN CRUSHED ROCK OR GRAVEL WRAPPED IN FILTER FABRIC

ALL FILTER FABRIC SHALL MEET THE FOLLOWING MINIMUM AVERAGE ROLL VALUES UNLESS OTHERWISE SPECIFIED BY ENGEO:

GRAB STRENGTH (ASTM D-4632)	180 lbs
MASS PER UNIT AREA (ASTM D-4751)	6 oz/yd^2
APPARENT OPENING SIZE (ASTM D-4751)	70-100 U.S. STD. SIEVE
FLOW RATE (ASTM D-4491)	80 gal/min/ft
PUNCTURE STRENGTH (ASTM D-4833)	80 lbs

NOTES:

1. *ALL PIPE JOINTS SHALL BE GLUED*
2. *ALL PERFORATED PIPE PLACED PERFORATIONS DOWN*
3. *1% FALL (MINIMUM) ON ALL TRENCHES AND DRAIN LINES*



ALTERNATE KEYWAY SUBDRAIN - OPTION 3
(FOR DEPTHS LESS THAN 30 FEET)

NO SCALE

ENGEO INCORPORATED EXCELLENT SERVICE SINCE 1971	TYPICAL SUBDRAIN DETAILS OAK CREEK CANYON CLAYTON, CALIFORNIA	PROJECT NO.: 3840.205.202 DATE: FEBRUARY 2008 DRAWN BY: JMG CHECKED BY: TPB	FIGURE NO. 11

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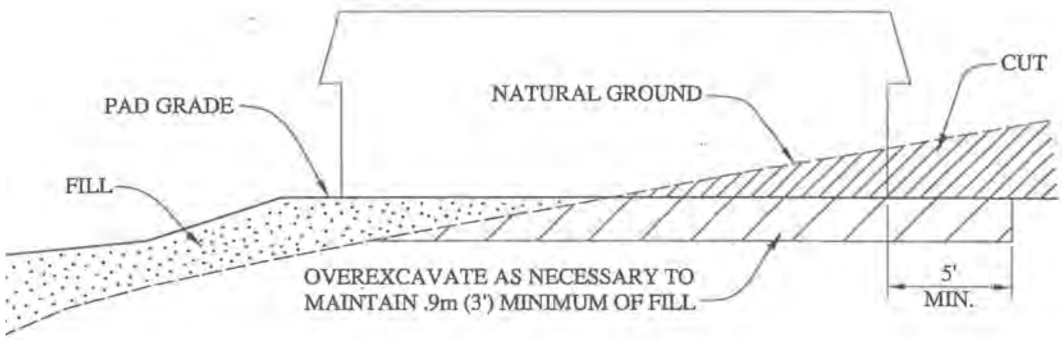


WHERE LOTS ARE PARTIALLY IN FILL, AND PARTIALLY IN CUT, THE CUT PORTION MUST BE OVEREXCAVATED AS SHOWN

CUT/FILL LOT

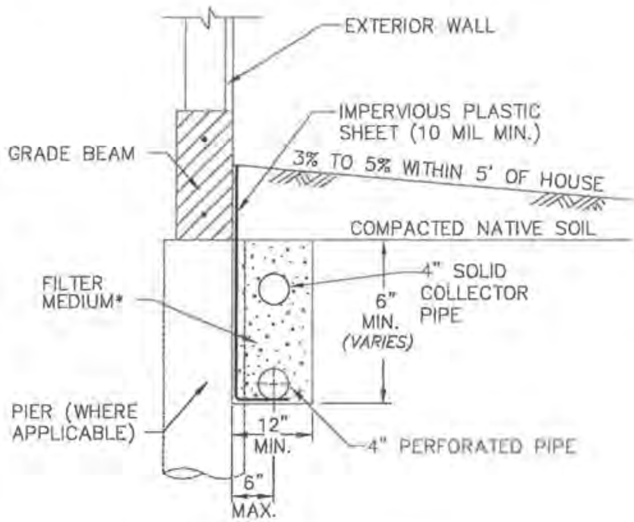
NO SCALE

ENGEO INCORPORATED
EXCELLENT SERVICE SINCE 1971

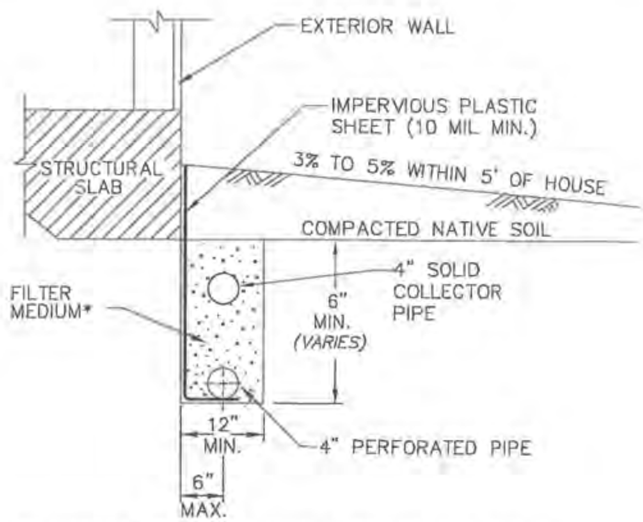
OVEREXCAVATION FOR CUT/FILL LOTS
OAK CREEK CANYON
CLAYTON, CALIFORNIA

PROJECT NO.: 3840.205.202		FIGURE NO.
DATE: FEBRUARY 2008		12
DRAWN BY: JMG	CHECKED BY: TPB	

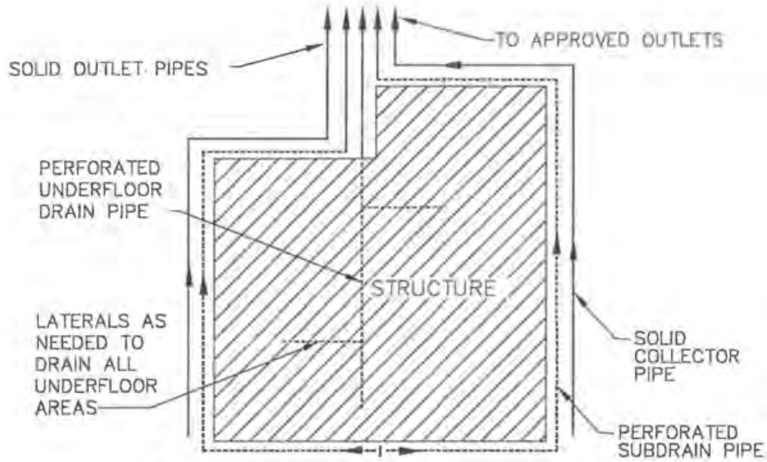
G:\Active Projects\3840\3840205201\Drafting\03840.205.202 13 CUT-FILL DETAIL C



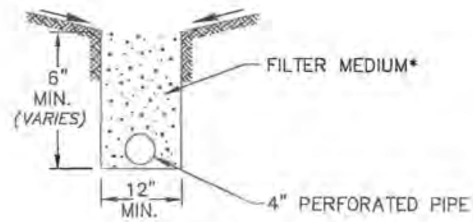
PIER AND GRADE BEAM



COMBINED SUBDRAIN AND PERIMETER DRAIN
STRUCTURAL SLAB



TYPICAL FOUNDATION SUBDRAIN PLAN



UNDERFLOOR DRAIN

NOTES:

1. *ALL PIPE JOINTS SHALL BE GLUED*
2. *ALL PERFORATED PIPE PLACED PERFORATIONS DOWN*
3. *1% FALL (MINIMUM) ON ALL TRENCHES AND DRAIN LINES*
4. *THE CLOSED COLLECTOR AND THE PERIMETER SUBDRAIN CAN BE CONSTRUCTED IN A SINGLE TRENCH, IF DESIRED. HOWEVER, THE CLOSED COLLECTOR PIPE MUST BE PLACED ABOVE THE SUBDRAIN PIPE, AND IN NO CASE SHOULD THE TWO SYSTEMS BE INTERCONNECTED*

*FILTER MEDIUM

ALTERNATIVE A

CLASS 2 PERMEABLE MATERIAL

MATERIAL SHALL CONSIST OF CLEAN, COARSE SAND AND GRAVEL OR CRUSHED STONE, CONFORMING TO THE FOLLOWING GRADING REQUIREMENTS:

SIEVE SIZE	% PASSING SIEVE
1"	100
3/4"	90–100
3/8"	40–100
#4	25–40
#8	18–33
#30	5–15
#50	0–7
#200	0–3

ALTERNATIVE B

CLEAN CRUSHED ROCK OR GRAVEL WRAPPED IN FILTER FABRIC

ALL FILTER FABRIC SHALL MEET THE FOLLOWING MINIMUM AVERAGE ROLL VALUES UNLESS OTHERWISE SPECIFIED BY ENGEO:

GRAB STRENGTH (ASTM D–4632)	180 lbs
MASS PER UNIT AREA (ASTM D–4751)	6 oz/yd^2
APPARENT OPENING SIZE (ASTM D–4751)	70–100 U.S. STD. SIEVE
FLOW RATE (ASTM D–4491)	80 gal/min/ft
PUNCTURE STRENGTH (ASTM D–4833)	80 lbs

NO SCALE

ENGEO INCORPORATED — EXCELLENT SERVICE SINCE 1971	FOUNDATION DRAIN DETAILS OAK CREEK CANYON CLAYTON, CALIFORNIA	PROJECT NO.: 3840.2.052.01 DATE: FEBRUARY 2008 DRAWN BY: JMP — CHECKED BY: JB	FIGURE NO. 13

O:\Active Projects\3840\3840205201\Drafting\03840.205.202 fndn subdrain BORROWED FROM 3193120(

ALAN KROPP
& ASSOCIATES, INC.
GEOTECHNICAL
CONSULTANTS

ALAN KROPP, CE, GE
JAMES R. LOTT, CE, GE
JEROEN VAN DEN BERG, CE
THOMAS M. BRENCIC, CE

February 25, 2020
P-8764, L-31991

Mr. Kevin English
West Coast Home Builders, Inc.
4021 Port Chicago Highway
Concord, CA 94520

RE: Geotechnical/Geological Peer Review
Oak Creek Canyon Project
Clayton, California

Dear Mr. English:

At your request, we performed a geotechnical and geological peer review of the geotechnical investigation and improvement plans for the proposed Oak Creek Canyon residential subdivision in Clayton, California. The purpose of this peer review was to evaluate whether the documents submitted conform to City standards and generally accepted geotechnical and geological practices. This peer review builds on the previous peer reviews performed by James Joyce, who also participated in the current peer review.

<u>DOCUMENTS REVIEWED</u>

The documents that we reviewed in our current evaluation include:

<u>Published Materials</u>

- Nilsen, Tor H., 1975, "Preliminary Photointerpretation Map of Landslide and Other Surficial Deposits of the Clayton 7-1/2' Quadrangle, Contra Costa County, California", U. S. Geological Survey Open File Map 75-277-12, 1:24,000.
- Dibblee, Thomas W., Jr., 1980, "Preliminary Geologic Map of the Clayton Quadrangle, Contra Costa County, California", U. S. Geological Survey Open-File Report 80-547, 1:24,000.
- Dibblee, Thomas W., Jr., 2006, "Geologic Map of the Clayton Quadrangle, Contra Costa County, California", Dibblee Geology Center Map #DF-192, 1:24,000.

<u>Consultant Materials</u>

- "Geotechnical Exploration – Oak Glen, Northeast Corner of Marsh Creek Road and Diablo Parkway, Contra Costa County", prepared by Engeo, dated March 31, 1994, Project No. 3840-E1.
- "Update of Geotechnical Exploration, Oak Creek Canyon, Subdivision 6826, APN 119-07-08, Clayton, California", prepared by Engeo, dated December 26, 2001 (Revised January 9, 2002), Project No. 3840.2.050.01.

- "Geotechnical Engineering Investigation, Seminary Tank Rehabilitation Project, Clayton", prepared by DCM Engineering, dated February 14, 2005, File: J-4904-1.
- "Geotechnical Peer Review, Oak Creek Canyon – Subdivision 6826, Marsh Creek Road, Clayton, California", dated February 23, 2007, Job No. 2965.000.
- "Geologic Peer Review, Subdivision 6826, Oak Creek Canyon, Clayton, California", prepared by Joyce Associates, dated October 22, 2007, Job Number 171.05.
- "Updated Geotechnical Report, Oak Creek Canyon, 5 Lots – Subdivision 6826, APN 119-070-008, Clayton, California", prepared by Engeo, dated February 22, 2008, Project No. 3840.205.202.
- "Geologic Peer Review, Subdivision 6826, Oak Creek Canyon, Clayton, California", prepared by Joyce Associates, dated March 19, 2008, Job Number 171.05.
- "Grading Plan Review, Oak Creek Canyon, 6 Lots – Subdivision 6826, APN 119-070-008, Clayton, California", prepared by Engeo, dated August 24, 2016, Project No. 3840.205.400.
- "Preliminary Grading Plan, Oak Creek Canyon, Subdivision 6826, City of Clayton, County of Contra Costa, State of California", prepared by Isakson and Associates, November 4, 2019, Job No. 200514.
- "Preliminary Grading Plan, Oak Creek Canyon, Subdivision 6826, City of Clayton, County of Contra Costa, State of California", prepared by Isakson and Associates, January 31, 2020, Job No. 200514.
- "Geotechnical Update and Plan Review, Oak Creek Canyon – Subdivision 6826 (6 Residential Lots) Clayton, California", prepared by Engeo, dated February 6, 2020, project number 3840.205.401.

In addtion, we received an undated draft copy of a Preliminary Corrective Grading Plan (prepared by Engeo) that used the 2019 grading plan (prepared by Isakson and Associates) as a base. However, we did not receive the March 21, 1997 Geotechnical Exploration Update by Engeo, which contained boring logs from borings drill in 1997.

It should also be noted that we received logs from borings drilled on the adjacent Contra Costa Water District (CCWD) Seminary Water Tank area in 1965, 1991, and 2001. These documents did not have an attached report.

PROPOSED CONSTRUCTION

The proposed project will consist of six residential lots, a new road, and related improvements. Access will be from Marsh Creek Road. Project grading will include a large cut along the uphill side of the development and a fill along the lower side. An engineering fill buttress with geogrid reinforcement will be constructed above the proposed road to improve stability and allow the use of slopes ranging up to 2:1 (h:v) in steepness. Short retaining walls will be built on Lots 2 and 3. The western portion of the property will not be developed.

BACKGROUND DATA

Published geologic maps such as Dibblee (2006; 1980) show that the site is underlain by Cretaceous-age sedimentary rocks of the Panoche Formation. These rocks consist principally of interbedded sandstone and shale. Traces of the Clayton fault are shown approximately 500 and 1500 feet northeast of the site. Bedding attitudes west of the Clayton fault are shown to dip moderately to steeply north in the project vicinity. The low-lying portion of the site is mapped as alluvium. No landslides are shown within the site.

Nilsen (1975) prepared a preliminary photo-interpretive map of landslides and surficial deposits covering the subject site. The central and western portions of the site and the CCWD water tank are mapped as a large landside, which extends from the edge of Marsh Creek Road to near the top of the ridge to the north. The eastern portion of the site is mapped as undifferentiated bedrock. The low-lying portion of the site adjacent at the mouth of Oak Creek Canyon is mapped as colluvium.

CONSULTANT'S DATA

In 1994, Engeo performed a boring in the lower portion of the site. Four additional borings were performed by Engeo in 1997. In response to peer review comments provided by Joyce Associates in 2007, an additional investigation was performed by Engeo, which is summarized in their 2008 report. This investigation included six test pits and two test trenches. The purpose of the pits and trenches was primarily to evaluate the extent of landslides within the site and evaluate the properties and bedding orientations of the Panoche Formation bedrock. The borings confirmed that the central and upper portions of the site are underlain by bedrock of the Panoche Formation. The borings show that the site is underlain by sediments consisting mainly of medium stiff to hard, silty and sandy clays, with some interbedded layers of sand, silt, and gravel. At depth, these materials are very dense.

SITE RECONNAISSANCE

The undersigned engineering geologist performed a site visit on December 26, 2019. Overall, the middle and upper portions of the site slope steeply to the south, with slopes ranging up to nearly 2:1 (h:v). The parcel is vacant and is covered with native grasses. A moderately large landslide is present in the western portion of the proposed develeopment area. The lower portion of the site is near level. A CCWD water reservoir (steel tank) is located on a graded pad along the western margin of the proposed development area.

Mr. Joyce also observed the two test trenches performed as a part of the 2008 investigation. At that time, discussions were held with Engeo's Engineering Geologist, Mr. Phil Stuecheli, and a general consensus was reached regarding the geologic conditions.

CONCLUSIONS

It is our opinion that the project documents conform to reasonable standard practices and City requirements regarding the geotechnical aspects of the project. We have the following comments:

1. The preliminary grading plan references a 1997 geotechnical report by Engeo. The grading plan should reference the more recent Engeo report and plan review.
2. There appear to be some differences between the corrective grading plans prepared by Engeo in 2008 and the recent draft copy we received. Key issues are the extent of remedial grading on Lots 3 to 5 and conforming remedial grading areas along the common property line with CCWD. The rationale for these differences should be provided. Also, the recent draft plan did not provide the locations of the 2008 trenches or the borings drilled on CCWD property, and this information should be added (assuming locations of borings on the CCWD property can be established).

3. During our recent reconnaissance, we observed a partially buried plastic pipe extending into the subject site near the southeast corner of the CCWD property. This pipe may be an outlet for subdrains extending beneath the fill that forms the outer portion of the pad for the water tank. We recommend that Engeo evaluate the pipe during project construction and connect it to an appropriate outlet.
4. A discussion of the anticipated future maintenance effort that will be required on the debris catchment bench should be provided by Engeo.
5. Subexcavation of the landslide area and keyways should be observed by an Engeo engineering geologist.
6. During construction, representatives of Engeo should observe the geotechnical aspects of the work, including grading, fill placement, surface and subsurface drainage measures, and foundation excavations. At the conclusion of the work, Engeo should prepare and submit to the City a final report summarizing their services during construction and indicating that the work was performed in accordance with their recommendations.

LIMITATIONS AND CLOSURE

This geotechnical peer review has been performed to provide technical advice to assist the City with its discretionary permit decisions. Our services have been limited to an independent review of the referenced documents. The opinions and conclusions presented in this letter are made in accordance with generally accepted geotechnical principles and practices. No other warranty, either expressed or implied, is made.

We trust this provides the information required at this time. If you have any questions, please call.

Very truly yours,

REGISTERED PROFESSIONAL ENGINEER
ALAN LEE KROPP
GE 487
EXP. 12/31/21
GEOTECHNICAL
STATE OF CALIFORNIA

ALAN KROPP & ASSOCIATES
Alan Kropp, G.E.
Principal Engineer

PROFESSIONAL GEOLOGIST
JAMES M. JOYCE
No. 1517
CERTIFIED
ENGINEERING
GEOLOGIST
STATE OF CALIFORNIA

JOYCE ASSOCIATES
James Joyce, CEG
Principal Geologist

AK/JJ/ab

Copies: Addressee (PDF) – kenglish@discoverybuilders.com
Engeo, Attention: Ted Bayham (PDF) – tbayham@engeo.com

Project No.
3840.205.401

March 10, 2020

Mr. Kevin English
West Coast Home Builders. Inc.
4021 Port Chicago Highway
Concord, CA 94520

Subject: Oak Creek Canyon – Subdivision 6826 (6 Residential Lots)
Clayton, California

RESPONSE TO REVIEW COMMENTS BY ALAN KROPP & ASSOCIATES, DATED FEBRUARY 25, 2020

Dear Mr. English:

At your request, this letter provides our response and clarification to several review comments provided by Alan Kropp & Associates (AKA) in their letter dated February 25, 2020, regarding the Oak Creek Canyon residential subdivision in Clayton, California.

Provided below are the AKA geotechnical comments in italics followed by our responses. Comment No. 1 requested information from the project Civil Engineer, Isakson and Associates, Inc., and therefore, not included in this letter.

Comment 2. *There appear to be some differences between the corrective grading plans prepared by Engeo in 2008 and the recent draft copy we received. Key issues are the extent of remedial grading on Lots 3 to 5 and conforming remedial grading areas along the common property line with CCWD. The rationale for these differences should be provided. Also, the recent draft plan did not provide the locations of the 2008 trenches or the borings drilled on CCWD property, and this information should be added (assuming locations of borings on the CCWD property can be established).*

ENGEO Response: The remedial grading plan (draft) provided to AKA as part of their review was tentative and considered a work in process. Once the Civil Engineer 40-scale design plans are final, a final remedial grading plan will be prepared. We have provided locations of the 2008 trenches and borings in the attached Appendix. Once the construction plans are completed, we will update our remedial grading plan to include previous exploration locations, as well as recommendations for conforming grading along the shared property line with the CCWD property, and delineation of areas containing unsuitable material that needs to be removed and replaced, as shown in the 2008 Remedial Grading Plan (Reference 2).

Comment 3. *During our recent reconnaissance, we observed a partially buried plastic pipe extending into the subject site near the southeast corner of the CCWD property. This pipe may be an outlet for subdrains extending beneath the fill that forms the outer portion of the pad for the water tank. We recommend that Engeo evaluate the pipe during project construction and connect it to an appropriate outlet.*

ENGEO Response: We appreciate this reconnaissance note by AKA, and ENGEO will evaluate this site condition during project construction to determine appropriate recommendations. If the pipe is a discharge pipeline for adjacent CCWD facility, the project Civil Engineer will include appropriate connections for future development in final plans.

Comment 4. *A discussion of the anticipated future maintenance effort that will be required on the debris catchment bench should be provided by Engeo.*

ENGEO Response: It is anticipated that the natural slope above the bench will periodically shed debris or accumulations of soil deposits onto the bench and/or within concrete lined drainage ditch, that these will need to be maintained on a periodic and as-needed basis. Bi-annual inspection of ditches is commonly performed on subdivisions with such facilities in the Bay Area to access the need for maintenance and clearing. Maintenance is further discussed in ENGEO's Updated Geotechnical Report, dated February 22, 2008 (Reference 2).

Comment 5. *Subexcavation of the landslide area and keyways should be observed by an Engeo engineering geologist.*

ENGEO Response: We concur with this comment by AKA, and recommend that an ENGEO Certified Engineering Geologist observe and approve all excavations of landslide areas and keyway for suitability to receive engineered fill.

Comment 6. *During construction, representatives of Engeo should observe the geotechnical aspects of the work, including grading, fill placement, surface and subsurface drainage measures, and foundation excavations. At the conclusion of the work, Engeo should prepare and submit to the City a final report summarizing their services during construction and indicating that the work was performed in accordance with their recommendations.*

ENGEO Response: We concur with this comment by AKA, and recommend that ENGEO representatives be present on site during construction to provide testing and observation recommendations in the field. Upon the conclusion of the project, a testing and observation report should be prepared by ENGEO documenting our services and whether or not the site work was completed in accordance with our recommendations or not.

If you have any questions or comments regarding this letter, please call and we will be glad to discuss them with you.

Sincerely,

ENGEO Incorporated

PROFESSIONAL GEOLOGIST
CURTIS E. HALL
No. 9557
STATE OF CALIFORNIA

Curtis E. Hall, PG

REGISTERED PROFESSIONAL ENGINEER
THEODORE P. BAYHAM
No. 2480
GEOTECHNICAL
STATE OF CALIFORNIA

Theodore P. Bayham, GE, CEG

Mary Bromfield
ceh/tpb/dt

Attachments: Selected References
Appendix A – Previous CCWD Exploration Information with Locations

SELECTED REFERENCES

1. Alan Kropp & Associates, Inc.; Geotechnical/Geological Peer Review, Oak Creek Canyon Project, Clayton, California; February 25, 2020; P-8764, L-31991.
2. ENGEO; Updated Geotechnical Report, Oak Creek Canyon, 5 Lots – Subdivision 6826, APN 119-070-008, Clayton, California; February 22, 2008; Project No. 3840.205.202.
3. ENGEO; Grading Plan Review, Oak Creek Canyon, 6 Lots – Subdivision 6826, APN 119-070-008, Clayton, California; August 24, 2016; Project No. 3840.205.400.
4. ENGEO; Geotechnical Update and Plan Review, Oak Creek Canyon – Subdivision 6826 (6 Residential Lots), Clayton, California; February 6, 2020; Project No. 3840.205.401.
5. Isakson and Associates, Inc.; Preliminary Grading Plan, Oak Creek Canyon, Subdivision 6826, City of Clayton, County of Contra Costa, State of California; January 31, 2020; Job No. 200514.

APPENDIX A

Previous CCWD Exploration Information with Locations

CONTRA COSTA WATER DISTRICT

ENGINEERING DEPARTMENT FAX TRANSMITTAL

Date: March 7, 2007

Number of Pages: 5
(including Cover Sheet)

NOTE: IF YOU DID NOT RECEIVE ALL OF THE PAGES, OR IF YOU HAVE A QUESTION, PLEASE CALL THE VERIFYING NUMBER BELOW.

TO:	FROM:
James Wang	Paul Lau
(Name)	(Name)
Discovery Builders	Contra Costa Water District
(Company Name)	(Company Name)
	688-8016
(Phone Number)	(Phone Number - Verifying)
689-2047	(925) 688-8303
(FAX number)	(FAX number)

Subject:

Comments:

FYI

LAND RIGHTS PROJECT ROUTING SLIP TREATED WATER

FILE: 701109 **REPLY BY:** 6-26-01

SUBJECT: Sprint PCS Wireless Boring at Seminary Pump Station site 67

ENCLOSURES: Routing slips, sprint drawing A2 and Boring Information

	CONCUR:	DATE:		CONCUR:	DATE:
1 ~~WELCH~~	JW	6/18/01	3 NICKERSON		6/19/01
			Laabach	L	19Jun01
ROUTON			AUGUST		
MARTIN			HANSON		
2 PISILA	LP	6-19-01	4 ANGELOSANTE		

- ☐ FOR YOUR INFORMATION
- ☒ FOR YOUR REVIEW/COMMENTS
- ☒ FOR YOUR APPROVAL
- ☐ DISTRICT/USBR OWNED OR MAINTAINED
- ☐ APPLICANT OWNED OR MAINTAINED
- ☐ INSPECTOR REQUIRED no L

MESSAGE: Sprint would like to make a Test bore at Seminary Pump Station. Please review and comment. If approved do we want to request a copy of the boring logs.

By: Dino Angelosante **Date:** 6-15-01
Dino Angelosante

COMMENTS:
1. Must coordinate all boring/access with O&M.
2. Must first have positively located all potential utilities @ boring prior to drilling. Any potential conflict will require a safe relocation of the bore.
3. Pls provide a copy of the boring logs to CCWD.

2 SAME COMMENT AS 3-9-01, ATTACHED; EXCEPT NOTE THAT SITE IS STILL IN UNINCORPORATED COUNTY [CLAYTON IS SUBMITTING AN APPLICATION TO ANNEX ADJACENT SEECON-OWNED LANDS (FOR SUB-DIVISION) + THE SEMINARY RESERVOIR INTO THE CITY]. ANY USE PERMIT/CEQA DOCUMENTATION WOULD BE BY THE COUNTY AT THIS TIME.

ATTACH SEPARATE PAGES WITH COMMENTS

By: ______ **Date:** ______

EXPLORATION DATE	LOGGED BY	TOTAL DEPTH	BORING NO.
August 15, 2001	Peter Schuman	20 feet	B-1
EXPLORATION EQUIPMENT		BACKFILL MATERIAL	
Mobile B-20 equipped with a 4-inch-diameter, solid-stem auger		Cement slurry	

SURFACE CONDITIONS: Gravel-surfaced vehicular access way

GROUNDWATER CONDITIONS: No free groundwater encountered

APPROX. GROUND SURFACE ELEVATION (FEET): N/A

FIELD						DESCRIPTION	LABORATORY		
DEPTH (IN FEET)	SAMPLE TYPE	SAMPLE NO.	BLOWS/FOOT	UNCONFINED COMP. STRENGTH (TSF)	USCS LETTER SYMBOL		DRY DENSITY (PCF)	MOISTURE CONTENT (%)	OTHER LAB TESTS SEE LOG LEGEND FOR ABBREVIATION DEFINITIONS
		1	24	4.0	SC	***Clayey SAND:*** Mottled yellow-brown, ligth brown, and brown, dry-to-moist, medium dense, fine-to-coarse grained, with some silt (FILL)			
5		2	64			***MARINE SEDIMENTARY ROCK:*** Consisting predominantly of sandstone/siltstone, light brown, completely weathered, friable			
10		3	81			grades highly weathered, weak			
15		4	77/11"			***MARINE SEDIMENTARY ROCK:*** Consisting predominantly of shale, mottled gray-brown and orange-brown, completely weathered, friable			
20		5	84/12"			grades highly weathered			

BMI PROJECT NO. 01S-666

LOG OF EXPLORATORY BORING
PROPOSED TELECOMMUNICATIONS FACILITY
SEMINARY PUMP STATION, SF54XC100-A
CONTRA COSTA COUNTY, CALIFORNIA

PLATE 5

UNIFIED SOIL CLASSIFICATION SYSTEM

MAJOR DIVISIONS			SYM	DESCRIPTION
COARSE-GRAINED SOILS MORE THAN 50% OF MATERIAL IS GREATER THAN NO. 200 SIEVE	GRAVELS MORE THAN 50% OF COARSE FRACTION RETAINED ON NO. 4 SIEVE	GRAVELS (LITTLE OR NO FINES)	GW	Well-graded gravels, gravel-sand mixtures, little or no fines
			GP	Poorly-graded gravels, gravel-sand mixtures, little or no fines
		GRAVELS (APPRECIABLE FINES)	GM	Silty gravels, poorly-graded gravel-sand-silt mixtures
			GC	Clayey gravels, poorly-graded gravel-sand-clay mixtures
	SANDS MORE THAN 50% OF COARSE FRACTION PASSES NO. 4 SIEVE	SANDS (LITTLE OR NO FINES)	SW	Well-graded sands, gravelly sands, little or no fines
			SP	Poorly-graded sands, gravelly sands, little or no fines
		SANDS (APPRECIABLE FINES)	SM	Silty sands, poorly-graded sand-gravel-silt mixtures
			SC	Clayey sands, poorly-graded sand-gravel-clay mixtures
FINE-GRAINED SOILS MORE THAN 50% OF MATERIAL IS SMALLER THAN NO. 200 SIEVE	SILTS AND CLAYS LIQUID LIMIT LESS THAN 50		ML	Inorganic silts and very fine sands, silty or clayey fine sands, clayey silts with slight plasticity
			CL	Inorganic clays of low-to-medium plasticity, gravelly clays, sandy clays, silty clays, lean clays
			OL	Organic silts and clays of low plasticity
	SILTS AND CLAYS LIQUID LIMIT GREATER THAN 50		MH	Inorganic silts, micaceous or diatomaceous fine sands or silts
			CH	Inorganic clays of high plasticity, fat clays
			OH	Organic silts and clays of high plasticity
HIGHLY ORGANIC SOILS			PT	Peat, humus, swamp soils with high organic content

NOTE: DUAL SYMBOLS ARE USED TO INDICATE BORDERLINE SOIL CLASSIFICATIONS

LOG SYMBOLS AND DEFINITIONS

FIELD	LABORATORY	
STANDARD PENETRATION SPLIT-SPOON SAMPLER (2-INCH OUTSIDE DIAMETER)	-4	% PASSING NO. 4 SIEVE (ASTM TEST METHOD C 136)
CALIFORNIA SAMPLER (3-INCH OUTSIDE DIAMETER)	-200	% PASSING NO. 200 SIEVE (ASTM TEST METHOD C 117)
MODIFIED CALIFORNIA SAMPLER (2.5-INCH OUTSIDE DIAMETER)	LL	LIQUID LIMIT (ASTM TEST METHOD D 4318)
BAG/BULK	PI	PLASTICITY INDEX (ASTM TEST METHOD D 4318)
THIN-WALLED SHELBY TUBE (3-INCH OUTSIDE DIAMETER)	R-VAL	RESISTANCE VALUE (CALTRANS TEST 301)
WATER LEVEL (LEVEL ESTABLISHED AS NOTED ON LOGS)	EI	EXPANSION INDEX (UBC STANDARD 29-2)
WATER OR SEEPAGE ENCOUNTERED (LEVEL NOT ESTABLISHED)	COL	COLLAPSE POTENTIAL (ASTM TEST METHOD D 5333)
	SP	SWELL POTENTIAL (under a specified load) (ASTM TEST METHOD D 4546)
	SL	SWELL PRESSURE (no consolidation) (ASTM TEST METHOD D 4546)

GENERAL NOTES:

1. Lines separating soil or rock strata on logs are approximate boundaries only. Actual transitions may be gradual and, in the case of selectively sampled borings, may vary by as much as the sample interval.
2. In general, Unified Soil Classification designations were evaluated using visual methods only. Actual designations (based on laboratory tests) may vary.
3. Logs represent general soil conditions on the date and at the location indicated. No warranty is provided as to the continuity of soil conditions between individual sample locations.
4. Unconfined compressive strengths reported on the logs (if any) were obtained using a pocket penetrometer.

BMI PROJECT NO. 01S-666

LOG LEGEND
PROPOSED TELECOMMUNICATIONS FACILITY
SEMINARY PUMP STATION, SF54XC100-A
CONTRA COSTA COUNTY, CALIFORNIA

PLATE 3

FRACTURING	
LOG TERM	DEFINITION
Very Widely	>6 feet
Widely	2 to 6 feet
Moderately	8 to 24 inches
Closely	2-1/2 to 8 inches
Very Closely	3/4 to 2-1/2 inches

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

WEATHERING	
LOG TERM	DESCRIPTION / DEFINITION
Fresh	No visible sign of decomposition or discoloration. Rings under hammer impact.
Slightly Weathered	Slight discoloration inwards from open fractures; otherwise similar to fresh.
Moderately Weathered	Discoloration throughout. Strength less than fresh rock; specimens cannot be broken by hand or scraped with knife.
Highly Weathered	Specimens can be broken by hand with effort and shaved with knife. Texture becoming indistinct but fabric preserved.
Completely Weathered	Minerals decomposed to soil but fabric and structure preserved. Specimens easily crumbled or penetrated.

COMPETENCY			
CLASS	LOG TERM	DESCRIPTION / DEFINITION	APPROXIMATE RANGE OF UNCONFINED COMPRES-SIVE STRENGTHS (tsf)
I	Extremely Strong	Many blows with geologic hammer required to break intact specimens.	>2000
II	Very Strong	Hand-held specimens break with pick-end of hammer under more than one blow.	1000 - 2000
III	Strong	Hand-held specimens can be broken with single, moderate blow with pick-end of hammer.	500 - 1000
IV	Moderately Strong	Specimens can be scraped with knife; light blow with pick-end of hammer causes indentations.	250 - 500
V	Weak	Specimens crumble under moderate blow with pick-end of hammer.	10 - 250
VI	Friable	Specimens crumble in hand.	N/A

BMI PROJECT NO. ▶ 01S-666

ROCK CLASSIFICATION LEGEND
PROPOSED TELECOMMUNICATIONS FACILITY
SEMINARY PUMP STATION, SF54XC100-A
CONTRA COSTA COUNTY, CALIFORNIA

PLATE 4

CONTRA COSTA WATER DISTRICT

ENGINEERING DEPARTMENT FAX TRANSMITTAL

Date: March 6, 2007

Number of Pages: 9
(including Cover Sheet)

NOTE: IF YOU DID NOT RECEIVE ALL OF THE PAGES, OR IF YOU HAVE A QUESTION, PLEASE CALL THE VERIFYING NUMBER BELOW.

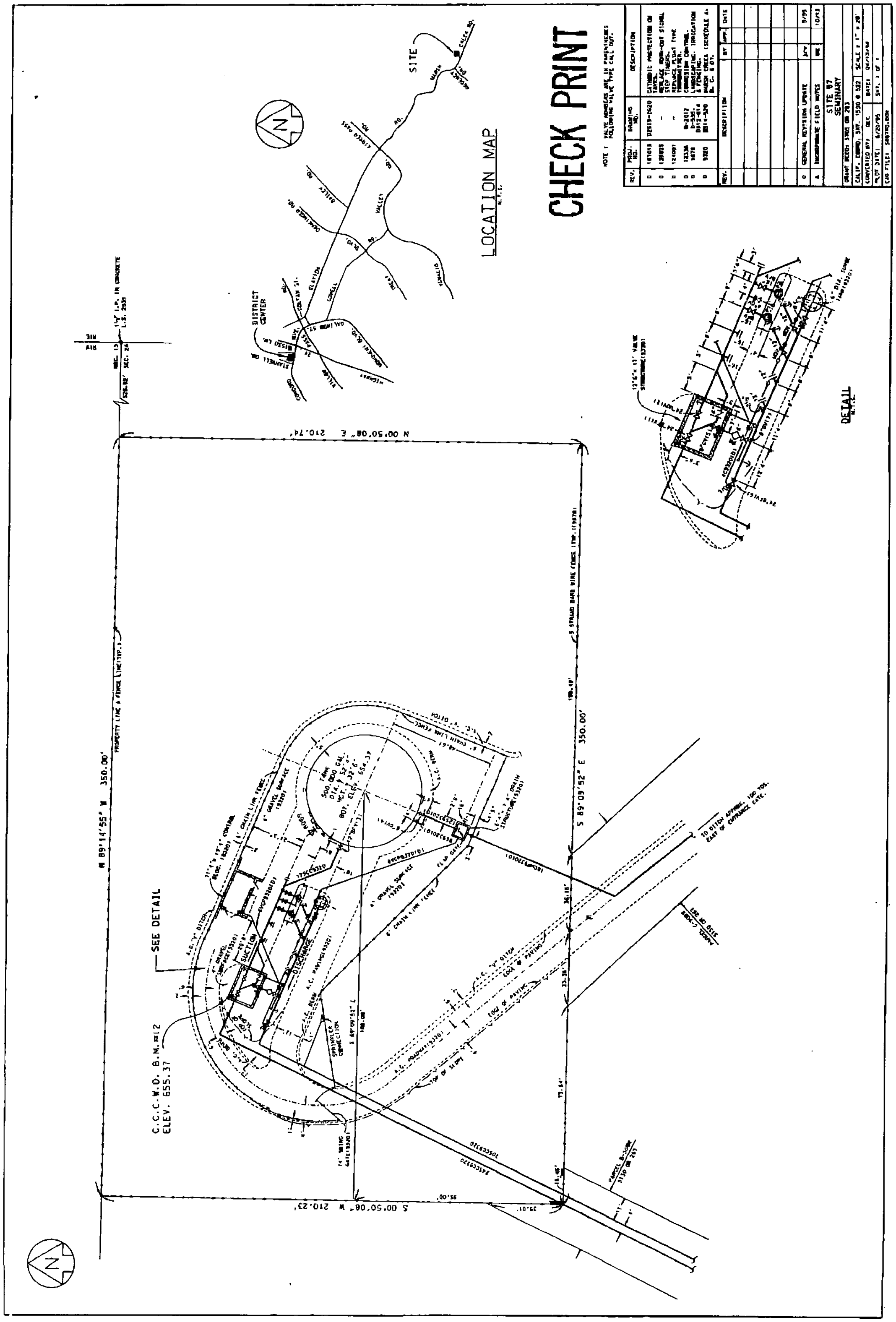
TO:	FROM:
JAMES WANG (Name)	PAUL LAU (Name)
Discovery Builders (Company Name)	Contra Costa Water District (Company Name)
(Phone Number)	688-8016 (Phone Number - Verifying)
689-2047 (FAX number)	(925) 688-8303 (FAX number)

Subject: Geotech Reports at Seminary

Comments:

I'm having problems finding an old Dames & Moore Report with some borings of the site. Here's what I could find.

Paul L.


CHECK PRINT
LOCATION MAP
N.T.S.
DETAIL
N.T.S.
SEE DETAIL
SITE
DISTRICT CENTER
SITE 97
SEMINARY
C.C.C.W.D. B.M. #12
ELEV. 655.37
N 89°14'55" W 350.00'
S 89°09'52" E 350.00'
N 00°50'08" E 210.74'
S 00°50'08" W 210.23'
SUCTION
DISCHARGE

DCM Engineering

David C. Mathy
Robert A. Kahl
Dru R. Nielson
Brian R. Dodge
Mark D. Sinclair
Marc M. Gelinas

February 14, 2005
File: J-4904-1

Ms. Jill Cunningham
Brown & Caldwell
201 N. Civic Drive, Suite 115
Walnut Creek, CA 94596

Subject: **Geotechnical Engineering Investigation
Seminary Tank Rehabilitation Project
Clayton, California**

Dear Ms. Cunningham:

This letter report summarizes our geotechnical engineering investigation at Contra Costa Water District's Seminary Tank in Clayton, California. The existing reservoir is an above grade welded steel tank with a capacity of 0.5 mg. The tank was constructed in about 1966 and retrofitted with rock anchor tie-downs for seismic stability in about 1992. The current project involves updated evaluations of seismic safety and water quality. This geotechnical investigation provides the specific geotechnical parameters requested by Brown & Caldwell. Background information referenced in this report has been provided by Brown & Caldwell.

1.0 FINDINGS

1.1 Review of Available Information

Previous Test Pits

Logs for four test pits that were apparently excavated for a geotechnical investigation by Soil Mechanics and Foundation Engineers in 1965 for the original design and placement of the Seminary tank are provided on Plate E-3, Log of Reference Test Pits. The location and elevation of these test pits relative to the existing tank location are not known to us at this time. These test pits describe topsoil depths from nothing to 7½ feet over bedrock consisting of sandstone and shale. The deepest test pit was 11 feet.

Previous Boring Logs

The logs for two test borings that were drilled for a geotechnical investigation performed by Dames & Moore in 1991 for design of the existing tank rock anchors are provided on Plates E-1 and E-2, Log of Reference Borings RB-1 and RB-2. The location of these reference borings are shown on Plate 1, Boring Location Map. These borings were logged to depths of 8 feet and 11 feet. Both borings were drilled on the pad cut for the tank, and based on the boring logs, both borings encountered sandstone to the maximum depth explored of 11 feet. We were not provided with copies of the reports from which these logs originated.

Mapping of Site Conditions

Geologic mapping by Graymer and others, *Preliminary Geologic Map Emphasizing Bedrock Formations in Contra Costa County, California, 1994*, describes bedrock at the tank site as shale with minor sandstone (see Plate 2). Bedrock bedding at the site is steeply dipping with no out of slope dip component around the tank perimeter.

Landslide mapping by Nilsen, *Preliminary Photointerpretation Map of Landslide and Other Surficial Deposits of Clayton 7½' Quadrangle, Contra Costa County, California, 1975*, indicates the Seminary tank site is located within a previously mapped landslide. The depth of this landslide and measures taken to mitigate the effects of this landslide for site development are not known to us at this time. However, it is most likely that the landslide mapped by Nilsen represents a failure of the topsoil not the bedrock. The topsoil was removed during grading for the tank pad. There was no evidence of landslide features in our test borings.

Fault zone mapping by the California Department of Mines and Geology, *Maps of Known Active Fault Near - Source Zones in California and Adjacent Portions of California, 1997*, to be used with the 1997 Uniform Building Code identifies the nearest known active fault trace to be the Greenville fault. The Greenville fault is less than 2 km north of the Seminary tank site. Some geology maps identify the section of the Greenville fault near the Seminary tank site as the Clayton fault. Peak ground (i.e., bedrock) acceleration at the site will be on the order of 0.50g (see Plate 3).

The tank is directly underlain by bedrock; therefore, the potential for liquefaction is nil.

Soil Conditions

According to the Soil Survey of Contra Costa County, California, by the Soil Conservation Service (SCS), 1977, the tank site is located near the boundary between two mapped soil units; Perkins gravelly loam and Los Osos clay loam. Although the topsoil has been removed from the tank site, it is noted that the Los Osos soils are described as being underlain by fine-grained sandstone and shale.

Tank Anchorage Design Plans

Plans for seismic improvements to the tank are detailed on *Design of Seismic Improvements, Tank Anchorage, Contra Costa County Water District Seminary Tanks, by Dames & Moore, May 8, 1992.* These plans show 28 rock anchors were installed around the tank at a spacing interval of approximately 6 feet. These rock anchors extend 49 feet beneath the surface of a ringwall that surrounds the tank. The bond length for these rock anchors is 29 feet which starts 20 feet below the top of the ringwall. The anchors were placed in 4-inch diameter pre-drilled holes. The plans do not specify the method of grouting (i.e., gravity vs. pressure). The plans show that the rock anchors are required to have a capacity of 95 kips for pullout.

Rock Anchor Performance Testing

We reviewed performance testing records for the Seminary tank rock anchors. The testing was performed for the Contra Costa Water District by AVAR Construction Systems on January 25, 1993.

All 28 of the Seminary tank rock anchors were performance tested successfully to 95 kips. No surcharge testing was performed on the rock anchors that would have exceeded their 95 kip design capacity. Creep testing performed on a few of the rock anchors was also successful. The creep test records show there was no movement over the 10 minute length of the test.

1.2 Borings

We drilled, logged, and sampled two borings (Borings B-1 and B-2) at the tank site on January 6, 2005, to depths of 34 feet. These borings were spaced between the locations of the two earlier referenced borings by Dames & Moore (RB-1 and RB-2). The location of all four borings is shown on Plate 1, Boring Location Map. Logs of our borings are provided in Appendix B and logs of reference borings and test pits are provided in Appendix E.

The two borings logged for our investigation were drilled using a tractor-mounted Mobile B-24 drill rig with 4-inch diameter continuous flight, solid-stem augers. Subsurface soil, bedrock and groundwater conditions were logged and representative subsurface bedrock samples were obtained from each boring. Bedrock samples were obtained in the test borings by driving a 2.5-inch inside diameter, 3.0-inch outside diameter Modified California Sampler (MCS) containing thin brass liners into the bottom of the boring or by driving a 1.4-inch inside diameter, 2.0-inch outside diameter Standard Penetration Test (SPT) sampler (ASTM D1586) into the bottom of the boring.

A 140-pound hammer falling 30 inches per blow was used to drive the samplers into the bottom of the borehole. The number of blows required to advance the samplers the last 12 inches of an 18-inch drive are recorded on the boring logs as penetration resistance (blows/ft). Sample penetration of less than 12 inches is noted on the boring logs with the number of blows per total increment of penetration. The penetration resistance values (blows/ft) for the SPT sampler given on the boring logs are actual ASTM D1586 N-values. The penetration resistance that is given on the boring logs for the MCS sampler is a field blow count for the sampler used and has not been correlated to an equivalent SPT N-value.

After the drive samplers were withdrawn from the borehole, the bedrock samples were removed, examined for classification, and sealed to preserve their natural moisture content for laboratory testing. Classification systems used to describe the bedrock are provided in Appendix A. Descriptions of bedrock and groundwater conditions provided in the boring logs are based on observations during drilling and sampling and on the results of laboratory tests. The borings were backfilled with cement slurry immediately after drilling.

1.3 Laboratory Tests

The following laboratory tests were performed on bedrock samples retrieved from our borings:

- Moisture Content
- Unit Weight
- Atterberg Limits
- Grain Size
- Corrosivity

The results of these laboratory tests are presented in Appendix B, Log of Borings B-1 and B-2. The results of testing for Atterberg limits, grain size, and corrosivity are shown graphically on plates in Appendix C. The Corrosion Engineering Investigation Report by Conceco/Matcor Engineering is included in Appendix D.

1.4 Summary of Subsurface Conditions

For a detailed description of the subsurface conditions encountered in our borings, see Appendix B.

Boring B-1 was drilled between the east side of the tank and the face of a cut slope made during the grading work for the tank pad. The upper 20 feet of the boring penetrated bedrock that consists predominantly of olive brown and dark yellowish brown weathered shale/claystone interbedded with sandstone layers of varying thicknesses. This same bedrock is visible in the cut slope face. At a depth of 20 feet in this boring, a dark gray weathered claystone was encountered that extended to the bottom of the boring. Groundwater seepage was encountered during drilling at a depth of about 24 feet.

Boring B-2 was drilled on the southwest side of the tank. This boring is located about 22 feet from the downhill side of the tank. Boring B-2 encountered about 2 feet of fill material over bedrock, which appears to indicate the cut pad for the tank was widened along the downhill side by placing fill over a benched slope. It is not likely that the fill extends beneath the tank. The fill appeared to consist of fat clays that are consistent with native topsoils. The underlying bedrock consists of olive brown and dark yellowish brown weathered shale/claystone interbedded with sandstone layers of varying thickness to the bottom of the boring. There was no groundwater seepage nor free groundwater encountered in this boring.

Received Time Mar. 6. 4:38PM

2.0 CONCLUSIONS AND RECOMMENDATIONS

2.1 Foundation Design Parameters

The Seminary tank rehabilitation project may require modification of the existing tank foundations and rock anchors (the exact scope of modifications is not known to us at this time). The following geotechnical engineering design parameters are for foundations constructed in undisturbed bedrock.

Allowable bearing capacity for footings having a minimum width of 12 inches and a minimum depth of embedment of 12 inches into undisturbed bedrock. An increase of 20% shall be allowed for each additional foot of width or depth to a maximum value of 6000 psf.	**3,000 psf**
Coefficient of friction between the base of the footing and undisturbed bedrock.	**0.40**
Allowable Passive Pressure (equivalent fluid load) for footings against undisturbed bedrock.	**500 pcf**
Seismic Design Coefficients per 1997 UBC. Based on the Greenville fault (Type B) at less than 2 km from the site.	**Soil Type = S_c** **$N_a = 1.3$** **$N_v = 1.6$** **$C_a = 0.40N_a = 0.52$** **$C_v = 0.56N_v = 0.90$**

2.2 Rock Anchors

The rock anchor performance test results verify the rock anchor capacity of 95 kips. Assuming a drilled hole diameter of 4 inches, a bond length of 29 feet, and a capacity of 95 kips, the rock anchorage bond stress is approximately 22 psi. At this time, we do not know if the rock anchors were gravity grouted or pressure grouted. A bond stress of 22 psi is more consistent with pressure grouting than gravity grouting.

If updated seismic evaluations of the Seminary tank determine that a small amount of additional uplift resistance is needed from these rock anchors (e.g., less than 10%), we recommend that they be performance tested to determine if the existing rock anchors have the desired capacity. However, assuming that the rock anchorage bond stress is 22 psi, it is doubtful if there is much additional capacity in these anchors, especially considering the bedrock is more shale/claystone than sandstone. As such, if significant additional uplift resistance is needed, it will most likely be necessary to install additional rock anchors. New rock anchors can be designed using the same criteria as the existing rock anchors.

3.0 LIMITATIONS

This report is to only be used for the Contra Costa Water District's Seminary Tank Rehabilitation project in Clayton, California. Recommendations provided in this report may require reevaluation once the final scope of foundation rehabilitation is determined.

4.0 CLOSURE

We appreciate the opportunity to serve Brown & Caldwell and the Contra Costa Water District on this project and trust that this report meets your needs and the needs of the Water District at this time.

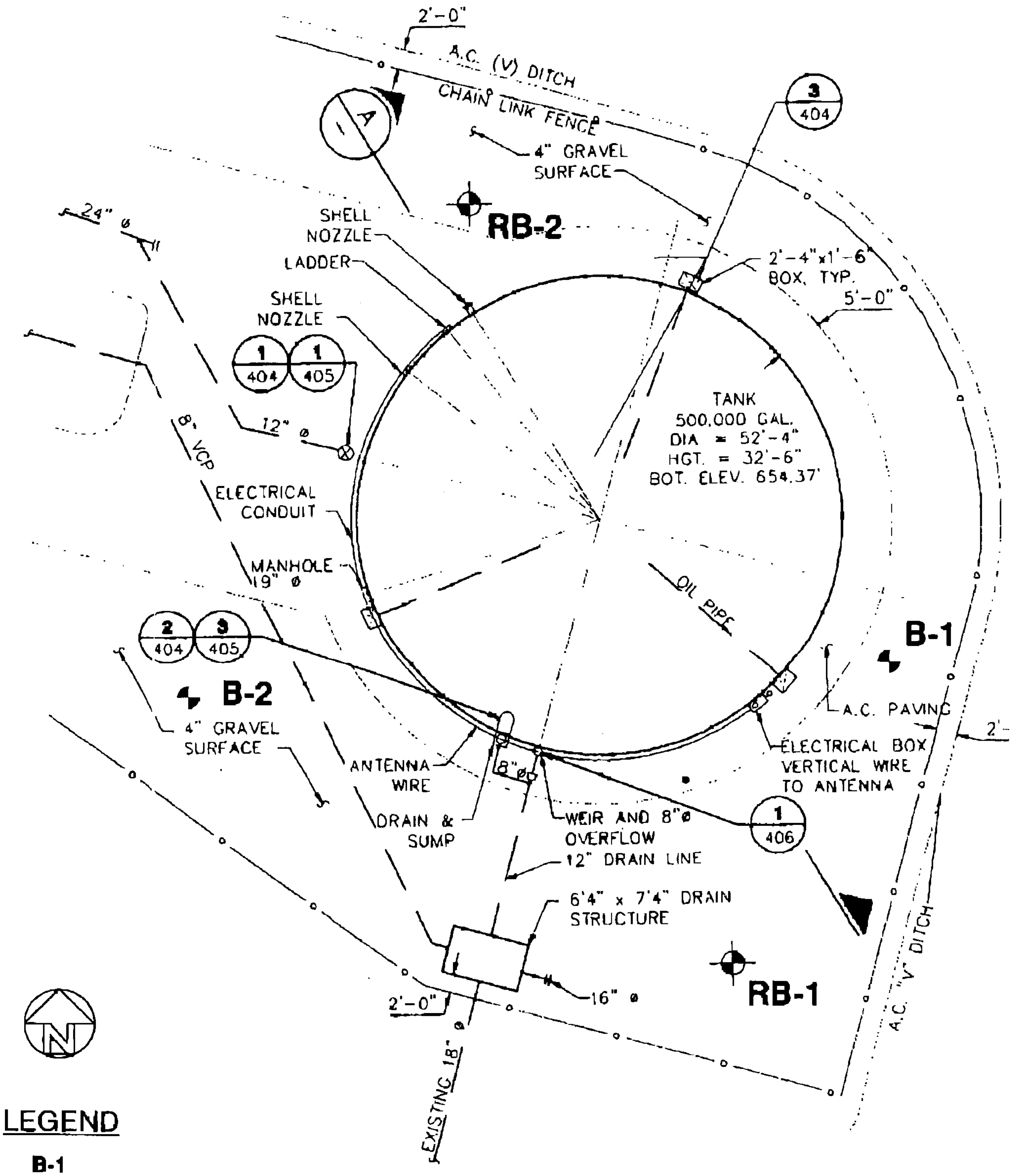
Very truly yours,

DCM ENGINEERING

Mark Sinclair
Staff Geologist

David C. Mathy
Principal Engineer
C.E. 28082
G.E. 569

Enclosures:
- Plate 1 – Boring Location Map
- Plate 2 – Geology Map
- Plate 3 – Seismic Shaking Map
- Appendix A (Plates A-1 and A-2)
- Appendix B (Log of Borings B-1 and B-2)
- Appendix C (Lab Plates C-1 through C-3)
- Appendix D (Corrosion Engineering Investigation Report)
- Appendix E (Reference Borings and Test Pits by others)



LEGEND

B-1 - Test Boring by DCM/Joyal Engineering

RB-1 - Test Boring by Others

Scale: 1 inch = 16 feet

Site plan reference not known

DCM Engineering	BROWN & CALDWELL Contra Costa Water District Seminary Tank Rehabilitation Project Clayton, California	PLATE NO. 1
FILE NO. J-4904-1 / FEBRUARY 2005	BORING LOCATION MAP	

SECTION E-3

FIELD EXPLORATION
Seminary Hill Site

The geological field exploration consisted of obtaining information about the subsurface soil and rock conditions by drilling 2 small diameter borings. The drilling program was performed and completed on October 15, 1991.

Our drilling contractor was Pitcher Drilling Co., from East Palo Alto, California. Pitcher used a truck-mounted solid flight auger. The primary purpose of the borings was to determine the depth of bedrock, the thickness and relative density of the surficial materials, and general soil conditions for the tank site. Location of the borings was coordinated with District Personnel prior to drilling.

All drilling operations were conducted under the direction of our field engineer. A continuous field log of each boring was maintained, based upon recovered samples, behavior and rate of penetration of the drill rig, and observation of the soil or rock cuttings being augered out of the holes.

Sampling was performed at three to five foot intervals, using the Standard Penetration Test (SPT). The sampler was driven with a 140-pound hammer falling 30 inches. SPT test were performed in accordance with ASTM Test Designation D-1586-64T. The sampling resistance, measured in blows per foot of sampler penetration, or fraction thereof, is shown on the logs adjacent to the appropriate sample. No coring was performed in the bedrock. Drilling operations were stopped upon refusal of the auger (generally at a depth of 8 to 10 feet). Cuttings were collected and examined to identify the bedrock type in preparation of the logs of the borings. Foundation materials were found to be extremely competent and no undisturbed sampling was necessary.

The samples were reviewed by our engineers and classified according to the Unified Soil Classification (USC) System, in accordance with the lithographic classification presented on Plate A-1. As the foundation material is very dense, most of the samples recovered were disturbed by the sampling process and were not suitable for strength testing. However, the high blowcounts obtained in the field confirm that the tank foundation is in a dense to very dense in-situ condition and has excellent bearing capacity. Explanations of the nomenclature and symbols presented on the logs of the borings are also presented on Plate A-1.

SEMINARY GENERAL SITE DESCRIPTION

The Seminary site is located on the southward facing slope of a ridge. The Seminary tank is built on an excavated pad. The site is underlain by an interbedded sequence of sandstone and shale. Rock is generally thin-bedded, dipping to the North at about 40 degrees, and weathered near the surface. Previous exploration with a backhoe (1965) indicate difficulties to excavate below 5 to 8 foot depth. Original surficial soils varied in thickness from 1/2 to two feet. Sandstone near surface is generally quite fractured but hardens with depth. Shale is thin-bedded and easily excavatable with a backhoe. Logs of backhoe pits excavated in 1965 are included after the logs of borings.

BORING B-1 (Seminary)

DATE DRILLED: 10/15/91

ELEVATION: 654 Feet ±

DEPTH IN FEET	SAMPLER TYPE	SAMPLING RESISTANCE	SAMPLES	SYMBOLS	DESCRIPTION
0				SP	BROWN SANDSTONE, rust color streaking
	SPT	50/4.5"	▬		
					Becomes harder
5	SPT	52	▬		BROWN SANDSTONE, some interbedded shale present
10	SPT	51/6"	▬		
15					
20					
25					
30					
35					

NOTES:

1. Boring completed at a depth of 11 feet on 10/15/91.
2. Sampling resistance is measured in blows per foot required to drive the sampler 12 inches with a 140 lb. hammer falling 30 inches after sampler has been seated 6 inches.
3. Boring log indicates interpreted subsurface conditions only at the location and the time the boring was drilled.
4. For an explanation of terms used see the Soils Classification Chart and Key to Test Data.
5. No groundwater encountered.

LOG OF BORING

Dames & Moore

BORING B-2 (Seminary)

DATE DRILLED: 10/15/91

ELEVATION: 654 Feet ±

DEPTH IN FEET	SAMPLER TYPE	SAMPLING RESISTANCE	SAMPLES	SYMBOLS	DESCRIPTION
0	SPT	50/5"	■	SP	BROWN TO OLIVE BROWN SANDSTONE, dry, hard, wire at approximately 6 inches, appears to be a welding electrode, (Fill)
5	SPT	50/3"	■		BROWN SANDSTONE, dry, hard Grading to a fine grained material - almost siltstone Drilling very difficult @ 7.5 feet Refusal
10					
15					
20					
25					
30					
35					

NOTES:
1. Boring completed at a depth of 8 feet on 10/15/91.
2. Sampling resistance is measured in blows per foot required to drive the sampler 12 inches with a 140 lb. hammer falling 30 inches after sampler has been seated 6 inches.
3. Boring log indicates interpreted subsurface conditions only at the location and the time the boring was drilled.
4. For an explanation of terms used see the Soils Classification Chart and Key to Test Data.
5. No groundwater encountered.

LOG OF BORING

Dames & Moore

TABLE B

SEMINARY NORTH RESERVOIR AND PUMP STATION SITE

BACKHOE PIT LOGS

Pit No.	Depth (feet)	Description
1	0 - 7	BEDROCK; SANDSTONE; light brown, dry, dense to hard; digs easily to 5', then much harder. Breaks out in angular fragments 6" to 18" across below 5'. 2" of topsoil at surface, bedrock crops out at surface near pit location.
2	0 - 7	TOPSOIL; Sandy Silty CLAY; red-brown, dry at surface to damp, hard to stiff; increasing rock fragments with depth.
	7 - 10	BEDROCK; interbedded SHALE and SANDSTONE; green-gray to reddish-brown and light brown, slightly damp, dense to hard; thin bedded, sandstone comes out in blocks to 1' across.
3	0 - 7½	TOPSOIL; Sandy Silty CLAY; red-brown, dry at surface to damp, hard to stiff.
	7½ - 11	BEDROCK; interbedded SANDSTONE and SHALE; light brown to red-brown, damp, dense to hard; very well weathered to 10', then hard.
4	0 - 2	TOPSOIL: Sandy Silty CLAY; red-brown, dry to slightly damp, hard to stiff.
	2 - 10	BEDROCK; SHALE; green-brown, slightly damp, dense; thin-bedded, digs very easily. Few sandstone interbeds 8' to 10'.
	10 - 11	BEDROCK; SANDSTONE; light brown, slightly damp, dense; breaks out fairly readily with backhoe.

ALAN KROPP
& ASSOCIATES, INC.
GEOTECHNICAL
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THOMAS M. BRENCIC, CE

March 18, 2020
3010-1, L-32011

Mr. Kevin English
West Coast Home Builders, Inc.
4021 Port Chicago Highway
Concord, CA 94520

RE: Supplemental Geotechnical/Geological Peer Review
Oak Creek Canyon Project
Clayton, California

Dear Mr. English:

At your request, we performed a supplemental geotechnical and geological peer review of the new documents we received for the proposed Oak Creek Canyon residential subdivision in Clayton, California. This review is part of our overall peer review work for this project. The purpose of our peer review analyses has been to evaluate whether the documents submitted conform to City standards and generally accepted geotechnical and geological practices.

We previously reviewed other documents sent to us and summarized our review of these documents in our letter to you dated February 25, 2020. In that letter, we indicated that additional materials should be transmitted to complete our review. A response to our comments was submitted by your geotechnical consultant (ENGEO) in their letter dated March 10, 2020.

It is our opinion that with the addition of the recent materials, the set of project documents we have now reviewed substantially conforms to reasonable standard practices and City requirements regarding the geotechnical aspects of the project. The project civil engineer (Isakson and Associates) is now apparently completing the final project plans, and ENGEO notes several items that will be added to these plans in the final stage. We believe these items are very straightforward, and we have confidence they will be added to the plans; therefore, it is our opinion we do not need to review the final drawings. As noted in our previous letter, ENGEO should provide the appropriate monitoring and testing during the geotechnical aspects of site development. Their observations and test results should be provided in a construction monitoring letter at the completion of the work.

This geotechnical peer review has been performed to provide technical advice to assist the City with its discretionary permit decisions. Our services have been limited to an independent review of the referenced documents. The opinions and conclusions presented in this letter are made in accordance with generally accepted geotechnical principles and practices. No other warranty, either expressed or implied, is made.

We trust this provides the information required at this time. If you have any questions, please call.

Very truly yours,

ALAN KROPP & ASSOCIATES
Alan Kropp, G.E.
Principal Engineer

JOYCE ASSOCIATES
James Joyce, CEG
Principal Geologist

AK/JJ/ab

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Engeo, Attention: Ted Bayham (PDF) – tbayham@engeo.com