Geotechnical Investigation and Rock Rippability Report for the Proposed Decker II Assemblage Industrial Site, Located at the Southwest Corner of Oleander Avenue and Decker Road, Assessor's Parcel Numbers (APN's): 314-020-010, -017, & -019, Western Perris Area, County of Riverside, California

Project No. M1103-008

Dated: February 19, 2015

Prepared For:

Mr. Neal Holdridge
TRAMMELL-CROW COMPANY
3501 Jamboree Road, Suite 230
Newport Beach, California 92660

MATRIX
Geotechnical Consulting, Inc.
P.O. BOX 2161, Temecula, California 92593
Phone 951.200.4747 Fax 760.692.1411 www.matrix-geotech.com

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TRAMMELL-CROW COMPANY
3501 Jamboree Road, Suite 230
Newport Beach, California 92660

Attention: Mr. Neal Holdridge, Principal

Subject: Geotechnical Investigation and Rock Rippability Report for the Proposed Decker II Assemblage Industrial Site, Located at the Southwest Corner of Oleander Avenue and Decker Road, Assessor's Parcel Numbers (APN's): 314-020-010, -017, & -019, Western Perris Area, County of Riverside, California

Matrix Geotechnical Consulting, Inc. (MATRIX) is pleased to submit herewith our Geotechnical Investigation and Rock Rippability report for the proposed Decker Assemblage Industrial Site, located at the southwest corner of Oleander Avenue and Decker Road, Assessor's Parcel Numbers (APN's): 314-020-010, -017 & -019, Western Perris Area, County of Riverside, California. This report presents the results of our review of pertinent geologic and geotechnical reports; the results of our field mapping and reconnaissance, laboratory testing, and presents our geologic and engineering judgment, opinions, conclusions, and recommendations pertaining to the geotechnical design and feasibility aspects of the proposed Decker II Assemblage project.

Based on the results of the above efforts, it is our opinion that the subject site is suitable for the proposed industrial use facility project, provided the recommendations presented herein are incorporated into the design of the project and implemented during site grading and construction. MATRIX should review and approve final rough grading plans and foundation plans when those become available and revise our recommendations presented herein, if we deem it necessary.

We are pleased that you retained Matrix to assist you on the preliminary design aspects of this project. Should you have any questions regarding the contents of this report or should you require additional information, please do not hesitate to contact this office at your convenience.

Very Truly Yours,

Chris Josef | Principal

MATRIX GEOTECHNICAL CONSULTING

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Geotechnical Consulting, Inc.
P.O. BOX 2161, Temecula, California 92593
Phone 951.200.4747 Fax 760.692.1411 www.matrix-geotech.com

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1.0 INTRODUCTION

1.1 Purpose and Scope of Services

The purpose of the work leading to the preparation of this geotechnical and rock rippability report was to evaluate the pertinent geologic and geotechnical conditions on the site. We included in this report our preliminary geotechnical design criteria for grading, foundation design and construction, and other relevant geotechnical considerations for use during the design and construction of the proposed industrial site.

Our scope of services consisted of:

- A review of existing geotechnical/geologic reports and geologic maps pertinent to the site (Appendix A).

- Analysis and review of stereoscopic aerial photographs of the property (Appendix A).

- Evaluation 16 test pits excavated to a depth of 8½ to 20 feet and rippable depth up to 25 feet. Logs of the test pits are presented in Appendix B, with the approximate locations depicted on the Geotechnical Map, Plate 1. Note: numbering of the test pits from TP-17 to TP-32 is a continuance of the test pits excavated on the Decker I property. The test pits were advanced to various depths throughout the site to evaluate the alluvial soil thicknesses onsite and classify the rock materials as rippable, marginally rippable, or non-rippable.

- Drilling of 25 rotary percussion “air-track” borings to depths of 15 to 40 feet. The logs of the air-track borings are located in Appendix B, with the approximate location depicted on the Geotechnical Map, Plate 1. Note: numbering of the test pits from AT-34 to AT-58 is a continuance of the air-tracks advanced on the Decker I property. The air-track borings are utilized to determine the relative hardness of the rock and suspected blasting depth. The potential blasting depth is classified into soft, medium, medium hard, and hard.

- Laboratory testing of representative soil samples obtained during the subsurface exploration (Appendix C).

- Six (6) seismic refraction survey lines labeled S-8 through S-13 were performed along representative areas delineated by Matrix Geotechnical Consulting field staff. Note: the refraction lines are labeled S-8 through S-13 as a continuation of the numbering from Decker I, to the east The traverses were located in the field by use of Google^{TM} Earth (2013) imagery and GPS coordinates. The approximate location of the seismic traverses is located on the Geotechnical Map – Plate 1. The seismic refraction survey is located in Appendix D.

- Excavation and logging of three exploratory trenches across the linear geomorphic features identified onsite. The depth of trenches was approximately 14 feet and 60 to 64 feet in length. The lineament analysis was performed by our subcontractor TerraGeosciences and is included in Appendix E.

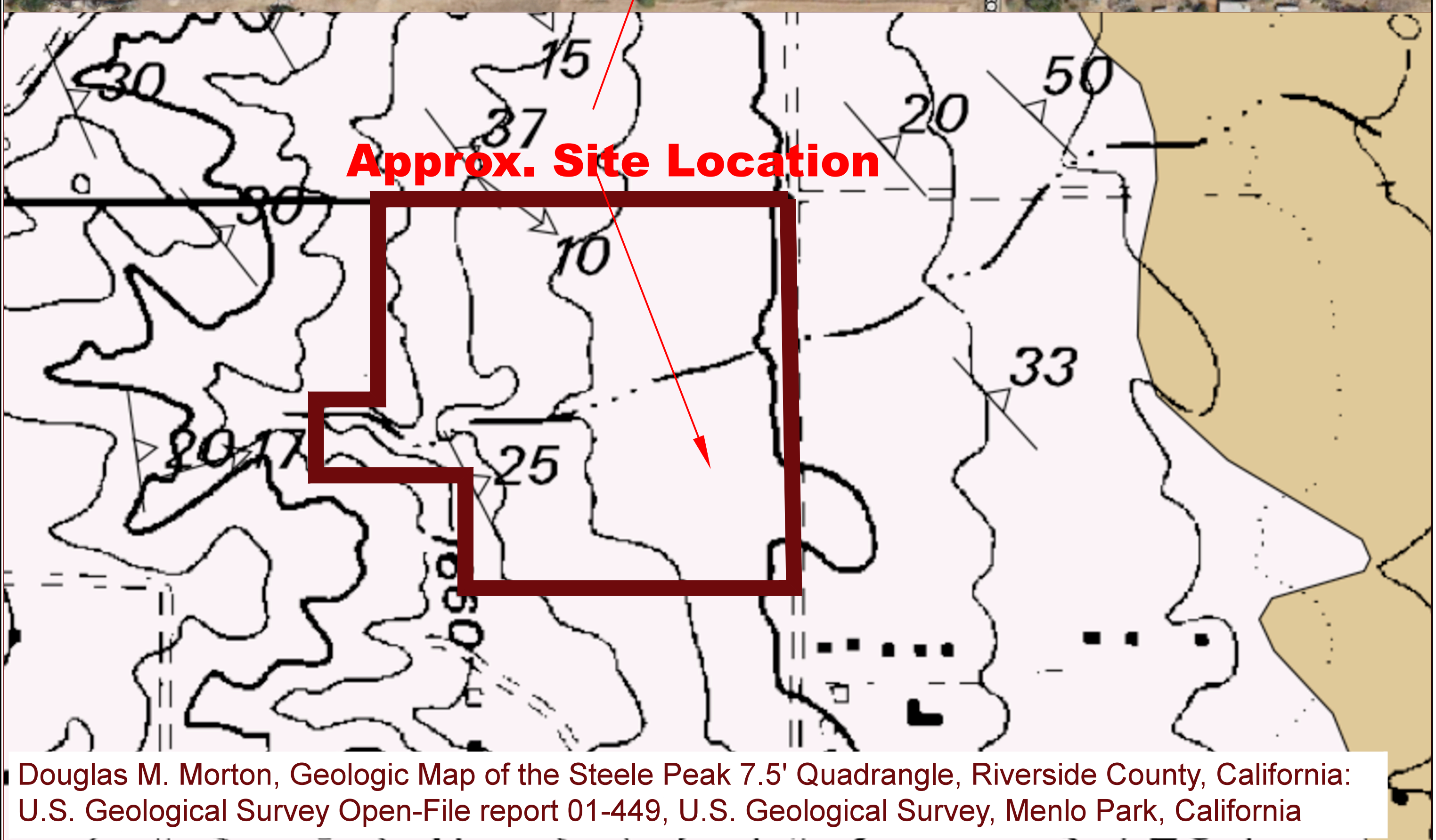
- Geologic site reconnaissance and mapping of surficial units.

- Engineering and geologic analyses of the data with respect to the design and construction of the proposed industrial site.

- Preparation of specific site seismicity, secondary seismic effects, and site response spectra (Appendix F).

- Preparation of General Earthwork and Grading Specifications (Appendix G).

- Preparation of this report presenting our review, conclusions and preliminary geotechnical design recommendations for the design and construction of the proposed industrial site.



Douglas M. Morton, Geologic Map of the Steele Peak 7.5' Quadrangle, Riverside County, California: U.S. Geological Survey Open-File report 01-449, U.S. Geological Survey, Menlo Park, California

MATRIX

FIGURE 1 - Site Location Map
FIGURE 2 - Geologic Map

Project Name	DECKER II ASSEMBLAGE
Project No.	M1103-004
Geo/Eng	RS
Scale	NOT TO SCALE
Date	FEBRUARY 2015

1.2 Location and Site Description

The project site consists of three parcels, APN's: 314-020-010, -017, & -019, located at the southwest corner of Decker Road and Oleander Road, in the Western Perris Area of Riverside County, California. The site is bounded on the north by undeveloped native land. Additionally the project is bounded on the east and west by undeveloped land a water tank, respectively and on the south by existing residential parcels and a roadway easement to the water tank. The general location and configuration of the site is shown on the Site Location Map (Figure 1).

Based upon our document and project background review the general area of the site property has experienced minor grading and is mainly in its natural condition. The portions of the site that are graded and/or have artificial undocumented fill are located in the south and southwest along the roadway easement for the tank and the excess of rocks that were generated from the excavation of the tank.

The remainder of the site consists of annual weeds and grasses, natural swales, and arroyos, and large granitic outcrop boulders. Although identified on the geological map as having a water-borne swale at the surface, no flowing water or surficial saturated soil was present in the near surface soil. However, groundwater was present at depth within the majority of the deep excavations near the swale that runs within the site and within the fault trench excavations and air-track bores. We anticipate that future elevations of the site will need to account for the potential for groundwater influence on the site.

From experience in the immediate area and the project to the east, the water condition is a perched condition, traversing across a large granitic bedrock shelf. The near surface bedrock (Val Verde Tonalite) is highly weathered and permeable, whereas the deeper bedrock is well indurated and non-weathered. The depth of groundwater is indirectly to directly associated with hard to very hard rock materials located within the site. Although some caution should be applied to reviewing the groundwater and hard rock within the swale portion of the site. The relatively thin presence of alluvial materials within the site over bedrock are consistent with the conditions observed within the western extent of the Decker I property to the east. The subsurface materials located in a majority of the site area have very shallow depths of older alluvium directly over weathered rock, which was also consistent with the western extent of Decker I.

Relatively large corestones and subsurface boulders were observed within the subsurface site area. These corestones and subsurface boulders, while being relatively hard and very dense in-place were rippable with the use of a large 60-inch ripper attached to the excavator. Below the corestones and boulders weathered and non-indurated Tonalite was observed and was readily excavated with the use of the excavator and air-track borings. An additional discussion of the rock and rippability is discussed in Section 2.7.

The general topography of the site is moderately sloping from west to east, with subtle grade changes from north to south. Elevations within the western to eastern central axis portion of the site vary from approximately 1665 (MSL) to 1600 above mean sea level (MSL). Comparatively, site elevations vary from approximately (+/-) 1620 through the central north to south axis of the site. Approximately 65 feet of relief occurs west to east.

1.3 Previous Geological and Geotechnical Investigations

Based on information provided to MATRIX, previous geotechnical reporting was performed on adjacent and nearby parcels. Representatives of MATRIX conducted a review of the files located within the County of Riverside Office of the County Geologist Building. The file review produced four (4) reports prepared by as follows: (a) Southern California Geotechnical (SCG), GEO 1659, November 4, 2004, (b) GEO 2085 December 13, 2005, and (c) GEO 2270 June 1, 2011; and (d) Salem Engineering Group, GEO 2311, November 30, 2012.

1.4 Proposed Development and Grading

It is our understanding that the proposed Decker II Assemblage – industrial building will consist of an approximate 600,000 square foot logistics building with truck bays located on the east and west and parking stalls located on the north and south of the proposed building. A detention basin is located along the eastern-southeastern portion of the site. The remainder of the site will provide asphaltic concrete paving for parking area drive aisles, concrete paving within the truck dock areas, the creation of a level building pad, construction of underground utilities, curbs, gutters, and other appurtenances. The preliminary configuration of the proposed building pad is shown on the Geotechnical Map, Plate 1.

1.5 Subsurface Investigation and Sampling Method

The subsurface exploration conducted for this project consisted of 25 rotary percussion "air-track" borings were advanced to depths of 15 to 40 feet within the site area. Sixteen (16) excavator pits were excavated to depths of 6½ to 20 feet within the site area. All of the air-track bores and test pits were logged during drilling and excavation by a member of our staff. Representative bulk and in-situ soil samples were taken during the excavations. Samples resulting from the excavator pits were sealed and transported to the laboratory as well. The air-track borings do not provide a sampling mechanism.

Six seismic refraction survey lines S-8 through S-13 were performed within the site area as delineated by MATRIX. The seismic traverse data collection was performed using twenty-four 14-Hertz geophones, spaced at eight to ten foot intervals to detect both the direct and refracted waves, with a 16-pound sledge-hammer being used as the energy source to produce the seismic waves.

Three fault trenches were excavated across two geomorphic lineaments that were identified to be within the site. The trenches were excavated in accordance with OSHA standards for layback and safe entry/exit. Logging of the fault trenches was performed by TerraGeosciences and a separate lineament report was prepared as part of this report and is included in Appendix E.

The approximate locations of the air-track bores, fault trenches, test pits and seismic lines are indicated on the Geotechnical Map, included as Plate 1 (Rear of Report). The test pit logs, and air-track bore profiles, which illustrate the soil conditions encountered at the bore and test pit locations, as well as the results of some of the laboratory testing, are included in Appendix B.

2.0 GEOTECHNICAL CONDITIONS

2.1 Soil and Geologic Conditions

The field investigation indicates that three geologic units occur on the site; undocumented fill, Quaternary Alluvial Deposits, and Cretaceous Val Verde Tonalite. The occurrence and distribution of the units encountered, including descriptions of the units, are shown on the excavator pits in Appendix B and on the Geotechnical Map – Plate 1 (map pocket). The geologic units are described below.

2.2 Site Geology

Based upon our understanding of the regional area and a review of the geotechnical test pits the surficial earth materials on the site are comprised of artificial fills placed by others in the southern and northeastern portion of the site, and Quaternary Alluvial Fan Deposits, and Cretaceous Val Verde Tonalite. The Alluvial fan deposits overlie the majority of the site area and to depths of 3 to 5 feet. The Val Verde Tonalite underlies the balance of the site both below the alluvium and in some areas, exposed at the surface. Large granitic outcroppings associated with the Tonalite exist on the western portion of the project. A general description of the earth materials observed on the site is provided in the following paragraphs:

2.2.1 Artificial Fill, by Others (Afo):

Artificial Fill, placed by others materials was mapped directly from the surface, mainly within the southwest and south and northeast portion boundaries of the site. The artificial fill was generated from placement of rock excavated from the tank construction and asphalt roadway along the easement of the tank area. Additional fill material was observed along the extents of the northeastern boundary of the site. The fill material is approximately 2 to 6½ feet, notably thicker in the area of the rock. The fill, consists of light-brown to brown silty sand, sand, silt, and boulders, dry to damp, and loose to medium dense.

2.2.2 Quaternary Alluvial Fan Deposits (Qal)

Quaternary Alluvial Fan Deposits, both youthful and older alluvium, were mapped directly below the fill materials and at the surface throughout the majority of the site. These materials were comprised of silt, clayey sand and silty sand, permeable to non-permeable, light pale brown to brown in color, medium dense to dense and were interfingered with caliche stringers, elluvial horizons directly above the bedrock, and colluvial deposits of silty-clayey material in the banks of the arroyos onsite.

2.2.3 Cretaceous Val Verde Tonalite (Kvt):

The Val Verde Tonalite underlies most of the site. Tonalite has a similar chemical composition to gabbro, but includes a higher percentage of quartz. Foliation within the Val Verde Tonalite mapped in the area generally strikes to the northeast (USGS, Steelepeak Quadrangle). The foliation generally has a vertical to near vertical dip and the direction of dip varies from a northeast to a southwest dip.

The Val Verde Tonalite was observed to be white-gray to gray and was found to be in a moderately hard to very hard state. In select areas, the upper 5 to 24 feet was more weathered and considered to be in a soft to moderately hard state. The unit was encountered throughout the majority of the site, beneath a veneer of topsoil or very old alluvium.

2.3 Landslides

Our review of the pertinent geologic literature did not indicate the presence of landslides on or directly adjacent to the site. The subject site is slightly to moderately sloping from west to east and not located within an area mapped as being potentially affected by earthquake-induced landsliding.

2.4 Groundwater

Groundwater was observed during field investigation. Based upon our knowledge of the site and local area, the groundwater observed is perched above the Tonalite. The granitic environment within the local and regional area is heavily weathered to non-weathered and contains zones of fresh very dense granitic bedrock with weathered fractures and seams that allow water to move freely within the rock. Depending upon the final design elevation of the proposed building pad groundwater may or may not adversely impact the proposed project development. Cuts in excess of 15 to 20 feet are likely to yield zones of seepage at the toe of slope (if configured within the site) or saturated conditions at the subgrade. The use of subdrains, curtain drains, or cut-off walls is very likely within areas of the site that water can travel from west to east. However, it is not uncommon for groundwater or seepage conditions to develop were none previously existed. Groundwater elevations are dependent on seasonal precipitation, irrigation, land use, among other factors, and may vary significantly as a result. Proper surface and subsurface drainage of irrigation and rainwater will be important to future performance of the project. Once a design pad elevation and plan is proposed MATRIX should review the plans and provide a design for recommendations of subdrains, curtain drains, or cut-off walls at that time.

In general, it is our opinion that those groundwater conditions will not have an influence on the subject site if properly managed through civil design with geotechnical input, although changes in ground conditions can occur. Based upon the dense to very dense to hard conditions of the Val Verde Tonalite, groundwater is not expected to be a constraint for the proposed industrial construction.

2.5 Surface Drainage

Existing surface drainage is evident within the site. A surficial arroyo traverses the site from west to east. This arroyo is likely to carry significant volumes of water during a small to peak storm event. Ponding areas were not noticeable during our geotechnical investigation. In general, during a storm event, excessive water sheet flow may likely traverse the site in a west to east pattern, as elevations suggest.

2.6 Seismicity

2.6.1 Faulting and Seismic Coefficients

The site is not located within an Alquist-Priolo Earthquake Fault Zone and there are not any faults (active, potentially active, or inactive) report on the pertinent literature onsite. Based on our background review, the site is not mapped in the vicinity of geologic hazards such as landslides, liquefaction areas, or faulting. The site is location in a seismically active region of Southern California. The possibility of damage from ground rupture is considered nil because active faults are not known to cross the site. However, two geomorphic lineaments were identified in our aerial photo analysis. These geomorphic lineaments were brought to the attention of our engineering geologist TerraGeosciences. TerraGeosciences performed a geologic lineament hazard analysis and prepared a report that is included within the rear of this report as Appendix E. The results of the lineament report indicate that an old bedrock fault was identified onsite that was deemed inactive. A small setback zone consisting of 15 feet on either side of the staked location onsite was recommended.

According to information obtained from the Acceleration Response Spectra (ARS) Online and the 2007 Caltrans Fault database, Table 1 lists the potential controlling fault located within a search radius of 50 miles from the property, its closest distance to the site and other information. The nearest known "active" fault is the San Jacinto Fault located approximately 8.0 miles northeast of the site. The San Jacinto Fault have been included in a State of California Earthquake Fault Zone. A maximum credible seismic event of magnitude 6.8 is postulated for the San Jacinto Fault with an estimated maximum credible peak site acceleration of 0.40g using the USGS acceleration-attenuation relationship.

TABLE 1
Nearby Faults

Fault Name	Fault Type	Dip (degrees) and Direction	Site Acceleration	Distance to Site[1]	Maximum Moment Magnitude (Mmax)
San Jacinto	Strike Slip		0.40	1.86	6.8[3]

[1] Closest distance from site to fault trace or surface projection of rupture area, based on Caltrans Design Manual Version 1.0 (2009)
[2] Site on footwall side of fault
3Based on review of the published reports on the San Jacinto Fault, a Mmax of 6.8 was used for the San Jacinto Section, consistent with Caltrans internal use.

Site accelerations were developed for the site based on the CBC, 2013 and Caltrans 2013 Acceleration Response Spectra (ARS) Online, Version 2.3.06. A site Coordinate of 33.857292° N, -117.267976° W was used to derive the seismic design parameters presented below in Table 2. MATRIX obtained its seismic design parameters in accordance with the California Building Code (CBC) Section 1613 using the United States Geological Survey (USGS) computer program, Earthquake Ground Motion Parameters and the site-specific Interactive Deaggregations software to develop further site analysis. The deaggregated site coefficients for 10 percent in 50 year (475-year recurrence interval) and for 2 percent in 50 year (2475-year recurrence interval) are listed herein using a V_s = 760 feet per second, associated with soil type D, a site specific PGA equivalent to 10% in 50 years = 0.39g and 2% in 50 years = 0.63g. The value of 2% in 50 years is associated with the 2013 California Building Code. However, the effective ground acceleration or (EGA) for the site is commonly taken as 2/3 to ¾ of the 2% in 50 years (2475 recurrence interval). MATRIX recommends that a site-specific coefficient of 0.42g be utilized for the subject site. (See Deaggregated Plots – Appendix F). The appropriate design spectrum should be selected by the project structural engineer.

TABLE 2
Seismic Design Parameters

Seismic Soil Parameters (2013 CBC Section 1613)	
Site Class Definition (Table 1613.5.2)	D
Mapped Spectral Response Acceleration Parameter Ss (for 0.2 second) (Figure 1613.5(3))	1.50
Mapped Spectral Response Acceleration Parameter, S_1 (for 1.0 second) (Figure 1613.5(4))	0.60
Site Coefficient Fa (short period) (Table 1613.5.3(1))	1.00
Site Coefficient F_v (1-second period) (Table 1613.5.2(2))	1.50
Adjusted Maximum Considered Earthquake (MCE) Spectral Response Acceleration Parameter S_{MS} (short period) (Eq. 16-37)	1.50
Adjusted Maximum Considered Earthquake (MCE) Spectral Response Acceleration Parameter S_{M1} (1-second period) (Eq. 16-38)	0.90
Design Spectral Response Acceleration Parameter, S_{DS} (short period) (Eq. 16-39)	1.00
Design Spectral Response Acceleration Parameter, S_{D1} (1-second period) (Eq. 16-40)	0.60

2.6.2 Liquefaction & Seismically Induced Settlement

Liquefaction is a seismic phenomenon in which loose, saturated, granular soil behaves similarly to a fluid when subjected to high-intensity ground shaking. Liquefaction occurs when three general conditions exist: 1) shallow groundwater; 2) low density non-cohesive (granular) soil; and 3) high-intensity ground motion. Studies indicate that saturated, loose to medium dense, near surface cohesionless soil exhibits the highest liquefaction potential. Dry cohesionless soil may experience dynamic compaction during an earthquake. In general, cohesive soil may not be susceptible to liquefaction. Groundwater was not identified below existing site grade. The potential for liquefaction to occur on the site is nil.

Dynamic settlement on the site of non-saturated fill and alluvium approximately 1-inch is anticipated, for proposed engineered fill and Val Verde Tonalite. A differential settlement of approximately ½-inch in 30-feet for engineered fill Val Verde Tonalite is expected because of seismic shaking. A corresponding angular distortion ratio of 1/500 may be utilized in the design of the site.

2.6.3 Shallow Ground Rupture

Shallow ground rupture cannot be completely precluded from occurring on the project site. However, based on our geologic mapping, literature review, and aerial photo analysis it appears that active faulting/potential shallow ground rupture is considered unlikely because of the absence of identified faults on the site. The potential for ground cracking because of shaking from distant seismic events is considered unlikely, although it is a possibility at any site.

2.6.4 Tsunami and Seiches

Based on the elevation of the of the site with respect to sea level and its distance from large open-bodies of water, the potential for seiche and/or tsunami waves to occur on the site is considered to be nil.

2.6.5 Lateral Spreading

Saturated soil that has experienced liquefaction may be subject to lateral spreading where located adjacent to free-faces, such as slopes, channels, and rivers. Therefore, lateral spreading does not appear to present a causative hazard to the site and the effects of lateral spreading on the site are considered to be nil.

2.7 Seismic Refraction Survey

A field seismic refraction survey was performed throughout the site. A total of six (6) seismc refraction lines were performed in areas designated by MATRIX personnel. Matrix Geotechnical Consulting subcontracted Terra Geosciences – Mr. Donn Schwartzkopf, PGP to perform the seismic lines. Mr. Schwartzkopf and TerraGeosciences personnel located the traverses in the field by using Google EarthTM imagery and GPS coordinates. The equipment utilized consisted of twenty-four 14-Hertz geophones, spaced at eight-to ten-foot intervals, on each line to detect both the direct and refracted waves; a 16-pound sledge-hammer being used as the energy source to produce the seismic waves.

In general the site can be broken down into three velocity layers, V1, V2, and V3, respectively. The V1 layer is the uppermost layer and consists of topsoil, colluvial soil, older alluvium, and/or completely weathered and fractured bedrock. An average weighed velocity of 1,279 to 1,657 feet per second, is applied to these materials. In general, this was observed to be accurate within the excavator pits, hollow-stem and air-track borings. Materials were readily excavatable and required very little to moderate effort to remove soil or advance a flight auger. Notably, the site was observed to excavate slightly easier than the site to the east.

The second layer V2 is located directly below the V1 layer. It has an average weighed velocity of 3,021 to 4,648 feet per second. From experience working within these rock materials, a value of 4,500 feet per second and higher yields heavy ripping and or a blasting requirement. For comparison, various charts of rippability and rock engineering properties have been provided within the Terra Geosciences report. This layer is dominated by the older alluvium and the highly weathered and fractured Tonalite bedrock and some fresh corestones and boulders which required switching from the excavator bucket to a ripper shank.

Although it should be noted that the excavatability was only reduced by some of the corestones that were observed within the excavator pit subgrade. These corestones and boulders will be fresh and well indurated and weathered depending upon location and depth. There did not appear to be a consistent depth or area of the site that presented similar characteristics for the presence of significantly hard material at depth. Additionally, it should be observed that air-track borings did encounter these corestones and boulders. However, in some instances the air-track rotary percussion hammer results, which were not an indication of hard bedrock as the hammer continued through the hard materials and advanced into semi-rippable material below.

The third layer V3 indicates the presence of slight weathering of the Tonalite bedrock. This layer has a seismic velocity range of 7,227 to 11,039 feet per second. These materials are unlikely to be excavated by conventional earth-moving equipment and will most likely require blasting. Large fractions of the fresh bedrock material was observed in some of the excavator pits and prevented the ripper shank from advancing deeper than the observed rock depth. These materials are generally limited to selective areas in the southeast, central, and central northern portion of the site, where rock is generally exposed at the surface. Notably, the velocities generated from V3 are significantly lower than the eastern Decker I site.

2.8 Slope Stability

The site is generally flat and we understand that significant slopes, greater than 30 feet in height, are not proposed to develop the site for its intended use. Once final grading plans become available, MATRIX should review the final proposed grading and provide supplemental recommendations with regards to slopes, as necessary.

2.9 Laboratory Testing

The following tests were performed on soil samples recovered from within the test pits: maximum density and optimum water content (ASTM D1557), direct shear, Expansion Index, soluble sulfates, pH, resitivity, chloride, and R-Value.

3.0 CONCLUSIONS

Based on the results of our geotechnical site reconnaissance, field and laboratory investigations, and our understanding of the site, it is our opinion that the proposed industrial facility and improvements are feasible from a geotechnical viewpoint, provided the conclusions and recommendations contained in this report are incorporated into the project design process and implemented during construction. The following is a summary of the primary geotechnical conclusions determined from our analysis of the site.

- Based on our review of some of the pertinent geologic maps, stereoscopic aerial photos, and reports, the site is underlain by Artificial Fill, Quaternary Alluvial Fan Deposits and the Cretaceous Val Verde Tonalite.
- The site is not located within a State of California Earthquake Fault Zone.
- Groundwater is not considered a constraint for the proposed industrial development, provided that the design elevation of the site is reviewed by MATRIX.
- The potential for liquefaction to occur is considered negligible.
- Active or potentially active faults were not identified, to exist on, or project toward the site.
- Known landslides do not occur on, or have the potential to impact the site.
- Laboratory test results of the near surface soil (fill and native) indicate a very low expansion potential as evaluated by the Expansion Index (EI) test. The EI test consists of remolding a soil to an arbitrary density that bears little or no relationship to field density conditions. At best the EI is an index of probable soil behavior. The Index is not useful to the engineer assigned the task of designing a foundation.
- Laboratory testing indicates that site soil has a negligible potential for soluble sulfate attack on Type II/V concrete.
- Laboratory test results of the near surface soil indicate that onsite soil has a moderate corrosion potential to buried metals.
- The Artificial Fill, previously placed by others and Very Old Fan Deposits has the potential to settle and should be overexcavated to underlying competent Val Verde Tonalite, within the entire site, areas of proposed structures, fill or new as remedial improvements. Anticipated removal depths range from approximately 3½ to 15 feet below the existing surface (See Geotechnical Map, Plate 1).
- Transition areas should be overexcavated to a depth of the Fill Height / 3, to minimize the effects of differential settlement.

- The existing onsite soil appears, from a geotechnical perspective, to be suitable material for use as fill, provided it is relatively free from rocks (larger than 3 inches in maximum dimension), construction debris, and organic material. It is anticipated that the onsite soil may be excavated with conventional heavy-duty earth moving equipment.

4.0 RECOMMENDATIONS

4.1 Site Earthwork

We anticipate that earthwork at the site will consist of site preparation and remedial grading, followed by the installation of underground utilities, and foundations for the proposed industrial site. All earthwork and grading should be performed in accordance with all applicable requirements of the County of Riverside and the Earthwork Specifications presented in Appendix G. In case of conflict, the following recommendations shall supersede those presented in Appendix G.

4.1.1 Site Preparation

Prior to grading of areas that may receive structural fill, structures or other improvements the areas should be cleared of surface obstructions, existing debris and stripped of vegetation. Vegetation and debris should be removed and properly disposed of offsite. All debris from the demolition of any onsite facilities of any type should be removed and properly disposed of offsite. Holes resulting from the removal of buried tree roots, obstructions, structures or utilities, which extend below finished site grades should be excavated to Val Verde Tonalite and replaced with a suitable compacted fill material. Areas to receive fill and/or other surface improvements should be scarified to a minimum depth of 6 inches, brought to a near-optimum water content, and recompacted to 90 percent or more relative compaction (based on American Standard of Testing and Materials [ASTM] Test Method D1557).

4.1.2 Overexcavation and Recompaction

The site is overlain with Artifical fill and Very Old Alluvial deposits. The site should be excavated within the entire site fill area to remove alluvial soil to the underlying Val Verde Tonalite. A fill keyway should be established on the eastern side and southern side of the project to commence filling of the site to reach design elevation. Prior to placement of fill material the bottom of the proposed fill keyway should be underlain with a gravel blanket approximately 12 inches thick, properly drain with subdrains connected to a solid piped outlet. The presence of grading water in a hard rock site and the influence of underground seasonal water conditions are likely to see a significant rise in the water on the fill portion of the site during rough grading. Control of this water will be necessary to achieve dense and stable conditions throughout the installation of the fill materials. Alternatively, dewatering wells could be established along the eastern perimeter of the project during the grading to prevent water from infiltrating into the subgrade of the compacted fill material. Note, these dewatering wells are only likely to be needed in the area of the swale that crosses the site from west to east.

Transition parcels should have the cut portion over-excavated at equal depth for fill depths of 0 to 5 feet, 5 feet for fill depths exceeding 5 feet and up to 10 feet, 10 feet for fill depths exceeding 10 and up to 15 feet, and H/3 (where "H" is the proposed fill height) for fills greater than 15 feet. Over–excavation in building areas should extend 5 feet or more beyond the proposed structure. Although not anticipated, localized, deeper over-excavation should be anticipated where deemed necessary by the geotechnical consultant based on observation during grading.

Following the over-excavation, the exposed rock subgrade should be surveyed by the project surveyor to determine that the underlying rock has not created a depression within the subgrade where water could pond. All bedrock should be shaped to drain with some percent fall away from structures. The onsite MATRIX engineering geologist and senior field technician will be observing these conditions and making recommendations if any grades do not appear to be in general accordance with our preliminary geotechnical report.

If a large area of loose/soft bottom is encountered (not likely in a rock project such as this), we recommend that a layer of geotextile fabric be placed to stabilize the bottom before placing the primary structural fill. Such additional subsurface treatment should be determined in the field by MATRIX during foundation subgrade preparation activities. Upon completion of the required overexcavation, backfill should be placed in accordance with recommendations presented later in this report.

Within any proposed roadway pavement areas 24 inches of the native soil below the design subgrade should be removed and recompacted, that is **below** the proposed structural section (total thickness of asphaltic concrete and aggregate base) of the roadway. However, localized, deeper overexcavation should be anticipated where deemed necessary by the geotechnical consultant based on observations during grading.

4.1.3 Import Soil for Grading

In the event import soil is needed to achieve final design grades, all import materials should be free of deleterious/oversize materials, have a very low expansion potential, negligible corrosion potential, and receive prior approval by Matrix Geotechnical Consulting 48 hours prior to commencement of delivery onsite. Laboratory testing of import soil must consist of maximum density and optimum water content, Expansion Index, sulfate, chloride, resistivity, pH, sieve analysis, and R-value.

4.1.4 Shrinkage and Bulking

Volumetric changes in earth quantities occur when excavated onsite earth materials are replaced as properly compacted fill or when fill is imported on a volumetric basis. The following (Table 3) is an estimate of losses from removal of organics, shrinkage and bulking factors for the various geologic units found on the site. These estimates are based on in-place densities of the various materials and on the estimated average degree of relative compaction specified during grading.

TABLE 3
Bulking and Shrinkage

GEOLOGIC UNIT	SHRINKAGE/BULKING PERCENT
Artificial Fill, by Others	10 to 15 (shrinkage)
Quaternary Alluvial Deposits	5 to 10 (shrinkage)
Cretaceous Val Verde Tonalite (weathered)	0 to 2 (shrinkage)*
Cretaceous Val Verde Tonalite (non-weathered)	0 to 5 (bulking)

*Negligible

The above estimates of shrinkage are intended as an aid for project engineers in determining earthwork quantities. **However, these estimates should be used with some caution because those are not absolute values,** rather preliminary estimates which may vary with depth of overexcavation, stripping losses, field conditions at the time of grading, etc. (Handling losses, and reduction in volume because of removal of oversized material, are not included in these estimates).

4.1.5 Fill Placement and Compaction

Areas prepared to receive structural fill should be scarified to a minimum depth of 6 inches, brought to optimum-water content, and recompacted to 90 percent or more relative compaction (based on ASTM Test Method D1557). The optimum lift thickness to produce a uniformly compacted fill will depend on the type and size of compaction equipment used. In general, fill should be placed in uniform lifts generally not exceeding 8 inches in uncompacted thickness. Fill materials shall be free of cobbles and boulders, with not more than 25% of the material being greater than 3 inches in size. Placement and compaction of fill should be performed in accordance with local grading ordinances under the observation and testing of the geotechnical consultant. In general, oversized material greater than 8 inches shall not be placed within 10 vertical feet of finish grade or within 2 feet of future utilities or underground construction. Oversize material may be incorporated into design fills in accordance with our standard grading details (see Appendix G).

4.1.6 Trench Backfill and Compaction

Onsite soil is generally considered to be suitable as trench backfill provided it is screened of rocks and other material over 3 inches in diameter and free of organic material. The trench backfill soil should possess a well-distributed grain size of coarse and fine gravel as well as coarse, medium, and fine sands. It is expected that onsite soil will meet this specification. Trench backfill should be compacted in uniform lifts (generally not exceeding 8 inches in uncompacted thickness) by mechanical means to 90 percent or more relative compaction (per ASTM Test Method D1557).

4.1.7 Temporary Stability of Trenches

All excavations for the proposed development must be performed in accordance with current OSHA (Occupational Safety and Health Agency) regulations and those of other regulatory agencies, as appropriate.

Based upon previous construction experience within the County of Riverside, working within Very Old Alluvial Deposits and Val Verde Tonalite, temporary vertical trenches or other cuts may be cut up to five feet. Those deeper than five feet shoud be slot-cut, shored, or cut to a 1H:1V (horizontal, H: vertical, V) slope gradient. Surface water should be diverted away from exposed cuts, and not be allowed to pond on or near the top of the cut slopes. Temporary cuts should not be left open for an extended period of time. Recommendations and stability calculations can be provided upon request for the use of cantilevered shoring, soldier piles, and underpinning. A foundation and/or shoring plan review must be completed by MATRIX prior to construction to confirm the location and suitability of potential shoring with respect to new structures.

If trenches are shallow and the use of conventional equipment may result in damage to the utilities, clean sand, having a sand equivalent (SE) of 30 or greater, should be used to bed and shade the utilities. Sand backfill should be densified. The densification may be accomplished by jetting or flooding. However, a representative of MATRIX shall observe the sub-soil conditions within the trench to determine the soil drainage condition potential. The presence of silt or clay bearing sub-soil within a trench suggests the use of a vibratory plate and then tamping to ensure adequate compaction of the trench backfill. A representative from MATRIX should observe, probe, and test the backfill to verify compliance with the project specifications.

4.1.8 Cal/OSHA Soil Classification

Based on the soil types encountered during our preliminary investigation, onsite soil can be generally classified as Type B. MATRIX does not limit the soil classification to one type as soil may locally change over short distances. Furthermore, this classification should not preclude a Cal/OSHA “competent person” from determining soil type on a case-by-case basis.

4.2 Foundation Selection

4.2.1 General

Preliminary recommendations for conventional foundation design construction are presented herein. When the final structural loads for the proposed structures become available, those should be provided to our office to verify the recommendations presented herein.

The information and recommendations presented in this section are minimums from a geotechnical point of view and are not meant to supersede design by the project structural engineer or civil engineer specializing in the structural design or those of a corrosion consultant.

4.2.2 Conventional Foundations

Place continuous footings at a minimum depth of 18-inch for exterior and interior construction into certified compacted fill. All continuous footings should have a minimum width of 15 inches.

Shallow foundations may be designed for a maximum allowable bearing capacity of 2,000 lb/ft^2, for continuous and spread footings. This value may be increased by 300 psf for each additional foot in depth and 150 psf for each additional foot of width to a maximum value of 3,000 psf, for dead load plus live load.

Spread or isolated pad footings shall be a minimum width of 24 inches and be founded 18 inches deep into certified compacted fill or approved Paralic Deposits or Friars Formation, where exposed. The bearing capacities should be re-evaluated when loads and footing sizes are finalized.

Lateral forces on footings may be resisted by passive earth resistance and friction along the bottom of the footing. Foundations may be designed for a coefficient of friction of 0.35, and a passive earth pressure of 250 lb/ft^2/ft. When combining passive and friction forces, passive resistance should be reduced by 1/3.

All footing trenches and bearing pads must be cut neat and level, and should be free of sloughed materials. See Table 4 for subgrade water conditioning for both continuous footing trenches and pads.

TABLE 4 CONVENTIONAL CONTINUOUS FOUNDATION DESIGN PARAMETERS	
Expansion Potential	**Very Low**
Footing Depth Below Lowest Adjacent Finish Grade	
Interior/Exterior	18
Footing Width	15
Footing Reinforcement	No. 4 Rebar Two (2) on Top Two (2) on Bottom
Slab Thickness	6 inches (minimum)
Under-Slab Requirements	A water and vapor retarding system (Stego or equivalent) should be placed below the slab on grade and on water sensitive areas as discussed in Section 4.2.3
Foundation and Slab Subgrade Water Content	At 10% above optimum water content prior to placement of concrete
Footing Embedment Next to Swales and Slopes If exterior footings are proposed adjacent to drainage swales are proposed within five (5) feet horizontally of a swale, the footing should be embedded 10" below the bottom of the swale. Footings adjacent to slopes should be placed at least five (5) feet horizontally from the edge of the footing to the face of the slope.	

*For Expansion potential greater than Low Expansion, alternative design guidelines will be provided by the Geotechnical Engineer.

4.2.4 Building Floor Slabs

We recommend a minimum floor slab thickness of 6 inches, reinforced with No. 3 bars spaced a maximum of 18 inches on center, both ways. Support slab reinforcement on concrete chairs to provide placement of the reinforcing near mid-depth of the slab, or as otherwise specified by the project structural engineer. Concrete should be either Type II/V having a minimum compressive strength of 4000 pounds per square inch (psi) and a water to cement ratio of 0.45.

Interior floor slabs with water sensitive floor coverings should be underlain by a 15-mil thick water/vapor barrier (Stego or equivalent), to mitigate the upward migration of water from the underlying subgrade soil. The water/vapor barrier product must meet the performance standards of an ASTM E 1745 Class A material and have a permeance rating less than 0.01 perms as described in ASTM E 96-95 and ASTM E 154-88, and be properly installed in accordance with ACI Publication 302. It is the responsibility of the contractor to ensure that the water-vapor barrier system is placed in accordance with the project plans and the manufacturers and architectural specifications, and that the water /vapor retarder materials are free of tears and punctures prior to concrete placement. Additional water reduction and/or prevention measures may be needed, depending on the performance requirements of future interior floor coverings. Lap the membrane twelve inches or more and tape the seams. Where water sensitive floor coverings are not anticipated, the water/vapor barrier may be eliminated.

Sand layer requirements are the purview of the structural engineer, and provided in accordance with ACI Publication 302 "Guide for Concrete Floor and Slab Construction". In general, two inches of sand above and below the water/vapor barrier can be used as a guide. The use of sand layers is not a soil engineering issue and hence outside our purview. Ultimately, the design of the water retarder system and recommendations for concrete placement and curing are the purview of the developer, architect, building designer or the engineer responsible for the design of the foundations and floor slabs on grade.

Subgrade preparation below the concrete and sand shall consist of 4-inches of ¾-inch crushed aggregate rock or an equivalent material. The crushed aggregate base should be thoroughly water conditioned and be compacted with a minimum of 3 passes, each way, with a vibratory plate compactor.

Prior to placing concrete, vapor barrier, and sand, the subgrade soil below all floor slabs should be pre-watered to achieve a water content that is at least equal to or slightly greater than optimum water content. This water content should penetrate to a minimum depth of 12 inches into the subgrade soil. The water content of the floor slab subgrade soil should be verified by the geotechnical engineer within 24-hours prior to concrete placement. Proper concrete curing techniques should be utilized to reduce the potential for slab curling or the formation of excessive shrinkage cracks.

4.3 Lateral Earth Pressures and Retaining Wall Design Considerations

Retaining walls should be founded on compacted fill per these recommendations or in dense Val Verde Tonalite. Foundations may be designed in accordance within the recommendations presented in Section 4.2.2. It should be noted that the values for lateral bearing presented therein are based upon level conditions at the toe. Reduced values may be appropriate for walls adjacent to descending slopes. In general, conventional walls may be designed to retain either native materials or select granular backfill. MATRIX must test and approve retaining wall backfill materials. Retaining walls should be backfilled with free draining materials (SE> 30) within one-half (½) the height of the wall, measured horizontally from the back of the wall, and compacted to project specifications. The upper twelve (12) inches of backfill should consist of

clayey soil. Drainage systems should be provided to walls to relieve potential hydrostatic pressure. Specifications for the quality of backfill soil should be defined on the retaining wall plans. It should be anticipated that suitable backfill material will have to be imported or selectively produced from onsite sources and should consist of granular, very low expansive materials. The following lateral earth pressures are recommended for retaining walls. The recommended lateral pressures for approved on-site soil (sand equivalency greater than 30 and non-expansive) for level or sloping backfill are presented on Table 5.

TABLE 5
Lateral Earth Pressures

Design Parameter		Soil Type	
		Imported Aggregate Base (Assumed)	**Val Verde Tonalite**
Internal Friction Angle (ϕ)		38°	32°
Unit Weight		130 lbs/ft^3	125 lbs/ft^3
Equivalent Fluid Pressure	Active Condition (Level backfill)	40 lbs/ft^3	55 lbs/ft^3
	Active Condition (2H:1V backfill)	55 lbs/ft^3	85 lbs/ft^3
	At-Rest Condition (Level backfill)	60 lbs/ft^3	75 lbs/ft^3
	At-Rest Condition (2H:1V backfill)	75 lbs/ft^3	95 lbs/ft^3
Passive Pressure		330	250

*Onsite backfill soil must be free from organics.

Equivalent fluid pressures are calculated utilizing a soil unit weight of γ = 130 pcf and γ = 125 pcf, for Imported Aggregate Base and Formational Soil, respectively. Restrained retaining walls should be designed for "at-rest" conditions, utilizing K_o.

- The design loads presented in the above table applied a horizontal loading. Friction between wall and retained soil should not be allowed in the retaining wall analyses.

- Additional allowances should be made in the retaining wall design to account for the influence of construction loads, temporary loads, and possible nearby structural footing loads.

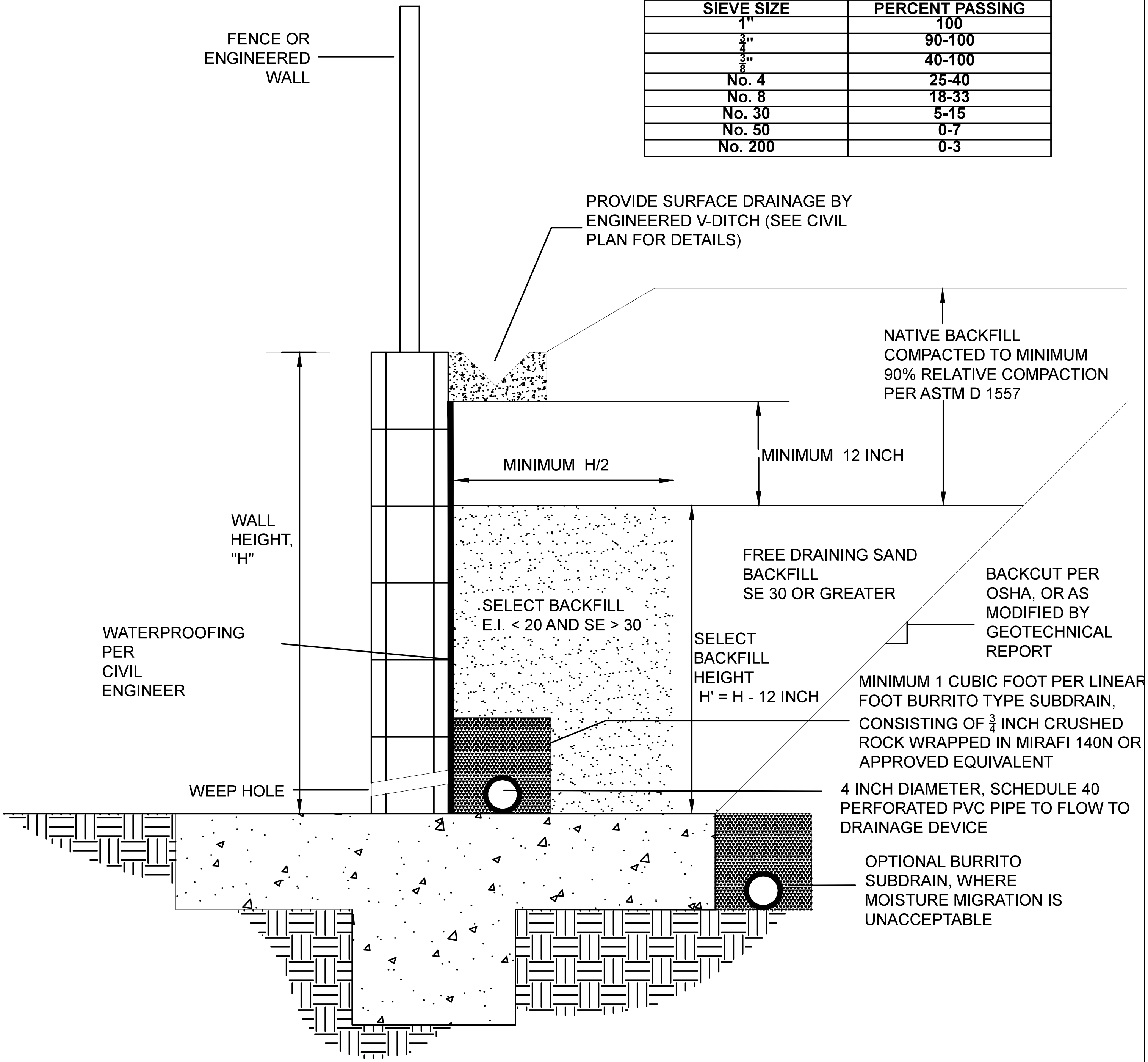
- Unit weights of 120 pcf and 130 pcf may be used to model the dry and wet density of onsite compacted fill materials.

- Select backfill should be granular, structural quality backfill with an Expansion index of 20 or less. The select backfill must extend at least one-half the wall height behind the wall. The upper one-foot of backfill should be comprised of native onsite soil.

- The wall design should include waterproofing (where appropriate) and back drains or weep holes for relieving possible hydrostatic pressures. The back drain should be comprised of a 4-ich perforated PVC pipe in a 1 foot by 1 foot, ¾-inch gravel matrix, wrapped with a geo-fabric, Mirafi 140N (or equivalent). The back drain should be installed with a minimum gradient of 2 percent and should be outletted to an appropriate location. For subterranean walls, this may include drainage by sump pumps.

- Backfill should not be placed against retaining wall concrete until the minimum design concrete strength (specified by others) is achieved by compression testing of field cast concrete cylinders.

CLASS 2 FILTER PERMEABLE MATERIAL GRADATION PER CALTRANS SPECIFICATIONS	
SIEVE SIZE	PERCENT PASSING
1"	100
$\frac{3}{4}$"	90-100
$\frac{3}{8}$"	40-100
No. 4	25-40
No. 8	18-33
No. 30	5-15
No. 50	0-7
No. 200	0-3



MATRIX

FIGURE 3
RETAINING WALL DIAGRAM

Project Name	DECKER II ASSEMBLAGE
Project No.	M1103-008
Geo/Eng	RS
Scale	NOT TO SCALE
Date	FEBRUARY 2015

4.4 Structural Setbacks

Structural setbacks, in addition to those required per the CBC, are not required because of geologic or geotechnical conditions within the site. Footing setbacks from basement foundation walls, if any, should be designed to minimize the effects of loading within the active zone of the subterranean walls. Where foundations are anticipated to be within the active zone for a potential subterranean wall, special design criteria for retaining wall active bearing pressures should be provided by MATRIX. The geotechnical and structural engineers must evaluate surcharge loading effects from the adjacent structures.

4.5 Corrosivity to Concrete and Metal

The National Association of Corrosion Engineers (NACE) defines corrosion as "a deterioration of a substance or its properties because of a reaction with its environment". The "environment" from a geotechnical viewpoint is the prevailing foundation soil and the "substances" are the reinforced concrete foundations or various buried metallic elements such as rebars, piles, pipes, etc., which are in direct contact with or within close vicinity of the foundation soil.

In general, soil environments that are detrimental to concrete possess high concentrations of soluble sulfates and/or pH values of less than 5.5. ACI 318R-05 Table 4.3.1 provides specific guidelines for the concrete mix design based on different amount of soluble sulfate content. The minimum amount of chloride ions in the soil environment that are corrosive to steel, either in the form of reinforcement protected by concrete cover, or plain steel substructures such as steel pipes or piles, is 500 ppm per California Test 532.

Based on testing performed during this investigation within the project site, the onsite soil is classified as having a negligible sulfate exposure condition in accordance with ACI 318R-05 Table 4.3.1. It is also our opinion that onsite soil should be considered to possess a moderate corrosion potential to buried metals because of its low resistivity.

Despite the minimum recommendation above, Matrix Geotechnical Consulting is not a corrosion-engineering firm. We recommend that you consult with a competent corrosion engineer and conduct additional testing to evaluate the actual corrosion potential of the site and to provide recommendations to reduce the corrosion potential with respect to the proposed improvements. The recommendations of the corrosion engineer may supersede our findings and recommendations.

4.6 Concrete Flatwork and Improvements

In an effort to minimize shrinkage cracking, concrete flatwork should be constructed of uniformly cured, low-slump concrete and should contain sufficient control/contraction joints (typically spaced at 8 to 10 feet, maximum).

Additional provisions need to be incorporated into the design and construction of all improvements exterior to the structures (walls, patios, walkways, planters, etc.). Design considerations may need to include provisions for differential bearing materials (bedrock versus compacted fill), ascending/descending slope conditions, bedrock structure, perched (irrigation) water, special surcharge loading conditions, potential expansive soil pressure, and differential settlement/heave.

Exterior improvements should be designed and constructed by qualified professionals using appropriate design methodologies that account for the onsite soil and geologic conditions. The above considerations should be used when designing, constructing, and evaluating long-term performance of the exterior improvements on the site.

The owner is advised of its maintenance responsibilities as well as geotechnical issues that could affect design and construction of future owner improvements. The information contained within this report should be considered for inclusion in owner packages (sale, transfer, lease, etc.) to inform the potential owner or lease-holder of issues relative to drainage, expansive soil, landscaping, irrigation, corrosive soil, and slope maintenance.

4.7 Preliminary Pavement Design

The following pavement recommendations assume proper drainage and construction monitoring, and are based on either the Portland Concrete Cement (PCA) or Caltrans design parameters for a twenty (20) year design period. However these designs also assume a routine pavement maintenance program to obtain the anticipated 20-year pavement service life.

Structural pavement sections presented herein for pavements are based on assumed subgrade soil conditions at the completion of grading and a review of the soil samples recovered during our subsurface exploration. However, it should be understood that the soil material exposed during grading may differ from the materials sampled and tested during this investigation. Therefore, preliminary pavement recommendations are subject to verification and possible revision based on any revised Traffic Indices (TI) as well as sampling and testing of subgrade soil present after grading. The client and/or civil engineer should verify that the TI's are representative of the anticipated traffic volumes. If the client and/or civil engineer determines that the expected traffic volume will exceed the assumed traffic indices, Matrix Geotechnical Consulting should be contacted for supplementary recommendations. The design traffic indices equate to the following approximate daily traffic volumes over a 20-year design life, assuming six operational traffic days per week.

Traffic Index	No. of Heavy Trucks per Day
4.0	5
5.0	8
6.0	10
7.0	15

With respect of the traffic volumes indicated above, a truck is defined as a 5-axle tractor-trailer unit with one 8-kip axle and two 32-kip tandem axles. All of the traffic indices allow for 1,000 automobiles per day.

Our laboratory testing determined an R-value of soil of 33 for design purposes we assumed an R-value of 30 for planning and prepared the following preliminary asphaltic concrete (AC) pavement sections (Table 6) based on assumed Traffic Indices (T.I.) of 5.0, 6.5, 7.0, and 7.5, and for Portland cement concrete (PCC) pavement sections (Table 7) for automobile parking and drive areas, light and moderate truck traffic.

TABLE 6
Preliminary Pavement Design – Asphaltic Concrete
Recommended Minimum Pavement Sections

ASPHALT PAVEMENTS (R = 30)				
Proposed Condition	**Thickness (inches)**			
	Private Drive/Parking Lot	Drive Aisles	Heavy Loaded Areas	Fire Lane
Assumed Traffic Index	5.0	6.5	7.0	7.5
Design R-value	30	30	30	30
AC Thickness (inches)	3.5	4.0	4.0	5.0
AB Thickness (inches)	6.0	9.0	10.0	10.0

Notes: AC – Asphaltic Concrete
AB – Aggregate Base

The thicknesses of the provided section are considered minimum thicknesses. We utilized a design R-Value of 30 for these minimum recommendations. Increasing the thickness of any or all of the above layers will reduce the likelihood of the pavement experiencing distress during its service life. The above recommendations are based on the assumption that proper maintenance and drainage of irrigation areas adjacent to the roadway will occur throughout the design life of the pavement. Failure to maintain a proper maintenance and/or irrigation program will jeopardize the integrity of the pavement.

TABLE 7
Preliminary Pavement Design – Portland Cement Concrete
Recommended Minimum Pavement Sections

PORTLAND CEMENT CONCRETE PAVEMENTS			
Materials	**Thickness (inches)**		
	Automobile Parking and Drive Areas	Light Truck Traffic Areas	Moderate Truck Traffic Areas
PCC	6	8	10
Compacted Subgrade (95% minimum compaction)	12	12	12

Crushed aggregate base should be compacted to a minimum of 95 percent relative compaction placed over a subgrade compacted to a minimum of 95 percent relative compaction per ASTM D1557 or the R-Value dry density, whichever is greater, throughout its upper 12 inches. Aggregate base should meet the specifications of the latest edition of the "Standard Specifications for Public Works Construction" (Greenbook) or the specifications of Caltrans Class 2 aggregate base. **Subgrade R-values shall be obtained by MATRIX upon completion of finished subgrade soil conditions within the site at the conclusion of rough or precise grading to confirm that our preliminary R-values remain applicable and valid for the as-graded conditions.** MATRIX should provide geotechnical observation and testing during construction.

The concrete should be a 28-day compressive strength of 4,000 pounds per square-inch (psi). Subgrade conditions assume a modulus of subgrade reaction of 100 pounds per cubic-inch (pci). Reinforcing within all pavements should be designed by the structural engineer. The maximum joint spacing within the entire PCC pavement is recommended to be equal to or less than 20 times the pavement thickness. The structural engineer should determine the actual joint spacing and reinforcing of the Portland cement concrete pavements.

4.8 Control of Surface Water and Drainage Control

Positive drainage of surface water away from structures is very important. Water must not be allowed to pond onsite or directly adjacent to or behind retaining walls. Design fine-grade elevations should be maintained throughout the life of the structure or if design fine grade elevations are altered, adequate area drains should be installed in order to provide rapid discharge of water, away from structures and slopes. Positive drainage may be accomplished by providing drainage away from buildings at a gradient of at least 2 percent to a location identified for drainage and further maintained by a suitable outlet or sump-pump (as necessary). Where existing conditions prevent 2 percent fall away from structures, alternative drainage methods should be incorporated by the civil engineer into his design of the drainage of the site. Additionally, MATRIX should review and comment on the use of alternative drainage devices within the site.

Planters with open bottoms adjacent to buildings should be avoided. Planters should not be located adjacent to buildings unless provisions for drainage, such as catch basins, and/or area drains, are made. Over watering must be avoided.

4.9 Slope Landscaping and Maintenance (as necessary)

Adequate slope and pad drainage facilities must be incorporated into the design of the finish grading for the subject site. The overall stability of graded slopes should not be adversely affected provided all drainage provisions are properly constructed and maintained thereafter and provided all engineered slopes are landscaped with a deep rooted, drought tolerant and maintenance free plant species, as recommended by the project landscape architect and reviewed by MATRIX.

4.10 Future Plan Reviews, Construction Observation and Testing

Future plan reviews are necessary to verify that recommendations and conclusions provided by Matrix Geotechnical Consulting preliminary studies are incorporated into the plans. Modifications to the plan or additional subsurface exploration/laboratory testing may be required based upon our review; therefore our review should be performed before any related construction is initiated. Such reviews should include, but are not limited to a review of :

- Precise Grading Plans
- Foundation and Structural Plans
- Retaining Wall and Shoring Plans
- Storm Drain/Sewer/Water/Dry Utility Plans

Plans should be forwarded to the project geotechnical engineer and/or engineering geologist for review and comments, as deemed necessary.

The recommendations provided in this report are based on limited subsurface observations and geotechnical analysis. A representative of MATRIX should check the interpolated subsurface conditions in the field during construction.

The geotechnical consultant should also perform construction observation and testing during future grading, excavations, backfill of utility trenches, preparation of pavement subgrade and placement of aggregate base, foundation or retaining wall construction or when an unusual soil condition is encountered at the site. Grading plans, foundation plans, and final project drawings should be reviewed by this office prior to construction.

5.0 LIMITATIONS

Our services were performed using the degree of care and skill ordinarily exercised, under similar circumstances, by California licensed civil or geotechnical engineers and geologists practicing in this or similar localities. Other warranties, expressed or implied are not made as to the conclusions and professional advice included in this report. The soil samples taken and submitted for laboratory testing, the observations made and the in-situ field testing performed are considered to be representative of the entire project; however, soil and geologic conditions revealed by future excavation may be different than our preliminary findings. If this occurs, the responsible party (client or contractor performing the work) must notify Matrix Geotechnical Consulting immediately of the changed conditions. These conditions must be evaluated by the project geotechnical engineer and geologist, and design(s) adjusted as required or alternate design(s) recommended.

This report is issued with the understanding that it is the responsibility of the owner, or of his/her representative, to ensure that the information and recommendations contained herein are brought to the attention of the architect and/or project engineer and incorporated into the plans, and that the necessary steps are taken to determine that the contractor and/or subcontractor properly implements our recommendations in the field. The contractor and/or subcontractor should notify the owner if they consider any of the recommendations presented herein to be unsafe.

Matrix Geotechnical Consulting, Inc. is not responsible for construction means, methods, techniques, sequences, or procedures, or for safety or precautionary programs in connection with the construction, for the acts and omissions of the CONTRACTOR, or any other person performing any of the construction, or for the failure of any of them to carry out the construction in accordance the final design drawings and specifications.

The findings of this report are valid as of the present date. However, changes in the conditions of a property can and do occur with the passage of time, whether they be because of natural processes or the works of man on this or adjacent properties.

In addition, changes in applicable or appropriate standards may occur, whether they result from legislation or the broadening of knowledge. Accordingly, the findings of this report may be invalidated wholly or partially by changes outside our control. This report should be reviewed and updated after a maximum period of 2-years or if the project concept changes from that described herein. This report has not been prepared for use by any parties or projects other than those specifically named or described herein. This report may not contain sufficient information for other parties or other purposes.

The opportunity to be of service is appreciated. Should you have any questions regarding the content of this report, or should you require additional information, please do not hesitate to contact this office.

Respectfully submitted,

MATRIX GEOTECHNICAL CONSULTING

Chris Josef
Principal

Richard Soltysiak, PE
Associate Engineer

REGISTERED PROFESSIONAL ENGINEER
RICHARD D. SOLTYSIAK
No. 37233
EXP. 06-30-16
STATE OF CALIFORNIA

CEJ/RS

APPENDIX A

REFERENCES

APPENDIX A

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APPENDIX B

TEST PIT /AIR-TRACK BORE LOGS

JOB# 4281	Hole/Stake# 1 / 1023		Matrix / West of Decker rd. x Oleander rd.				
DATE: 1/15/15			DRILL MAKE/MODEL: INGERSAL RAND ECM 720 CRAWLER DRILL #838				
RIPPABLE:			NEUTRAL NO LOAD: 100psi				
Marginal:			ROTATION UNDER LOAD: 900psi				
BLASTING req.:			DRILL W/PERCUSION : Feed 900psi / percusion 1200psi				
DEPTH	TIME	RQ	NOTES	DEPTH	TIME	RQ	NOTES
1	11			41			
2	11			42			
3	11			43			
4	11			44			
5	11			45			
6	11			46			
7	11			47			
8	11			48			
9	11			49			
10	11			50			
11	11			51			
12	11			52			
13	11			53			
14	11			54			
15	11	Hard	Grey granite	55			
16				56			
17				57			
18				58			
19				59			
20				60			
21				61			
22				62			
23				63			
24				64			
25				65			
26				66			
27				67			
28				68			
29				69			
30				70			
31				71			
32				72			
33				73			
34				74			
35				75			
36				76			
37				77			
38				78			
39				79			
40				80			

JOB# 4281	Hole/Stake# 2 / 1021	Matrix / West of Decker rd. x Oleander rd.
DATE: 1/15/15		DRILL MAKE/MODEL: INGERSAL RAND ECM 720 CRAWLER DRILL #838
RIPPABLE:		NEUTRAL NO LOAD: 100psi
Marginal:		ROTATION UNDER LOAD: 900psi
BLASTING req.:		DRILL W/PERCUSION : Feed 900psi / percusion 1200psi

DEPTH	TIME	RQ	NOTES	DEPTH	TIME	RQ	NOTES
1				41			
2				42			
3				43			
4				44			
5				45			
6				46			
7				47			
8				48			
9				49			
10				50			
11	N/A	Soft	Brown Dirt	51			
12	9			52			
13	9			53			
14	9			54			
15	9	Med.Hard	Grey	55			
16				56			
17				57			
18				58			
19				59			
20				60			
21				61			
22				62			
23				63			
24				64			
25				65			
26				66			
27				67			
28				68			
29				69			
30				70			
31				71			
32				72			
33				73			
34				74			
35				75			
36				76			
37				77			
38				78			
39				79			
40				80			

JOB# 4281	Hole/Stake# 3 / 1022		Matrix / West of Decker rd. x Oleander rd.				
DATE: 1/15/15		DRILL MAKE/MODEL: INGERSAL RAND ECM 720 CRAWLER DRILL #838					
RIPPABLE:		NEUTRAL NO LOAD: 100psi					
Marginal:		ROTATION UNDER LOAD: 900psi					
BLASTING req.:		DRILL W/PERCUSION : Feed 900psi / percusion 1200psi					
DEPTH	TIME	RQ	NOTES	DEPTH	TIME	RQ	NOTES
1				41			
2				42			
3				43			
4				44			
5				45			
6				46			
7				47			
8				48			
9				49			
10	N/A	Soft		50			
11	4			51			
12	4			52			
13	4			53			
14	4			54			
15	4	Med.Hard		55			
16				56			
17				57			
18				58			
19				59			
20				60			
21				61			
22				62			
23				63			
24				64			
25				65			
26				66			
27				67			
28				68			
29				69			
30				70			
31				71			
32				72			
33				73			
34				74			
35				75			
36				76			
37				77			
38				78			
39				79			
40				80			

JOB# 4281	Hole/Stake# 4 /1014		Matrix / West of Decker rd. x Oleander rd.				
DATE: 1/15/15		DRILL MAKE/MODEL: INGERSAL RAND ECM 720 CRAWLER DRILL #838					
RIPPABLE:		NEUTRAL NO LOAD: 100psi					
Marginal:		ROTATION UNDER LOAD: 900psi					
BLASTING req.:		DRILL W/PERCUSION : Feed 900psi / percusion 1200psi					

DEPTH	TIME	RQ	NOTES	DEPTH	TIME	RQ	NOTES
1				41			
2				42			
3	N/A	Soft	dirt	43			
4	8			44			
5	8			45			
6	8			46			
7	8			47			
8	8			48			
9	8			49			
10	8			50			
11	8			51			
12	8	Med.Hard	Granite	52			
13	11			53			
14	11			54			
15	11	Hard		55			
16				56			
17				57			
18				58			
19				59			
20				60			
21				61			
22				62			
23				63			
24				64			
25				65			
26				66			
27				67			
28				68			
29				69			
30				70			
31				71			
32				72			
33				73			
34				74			
35				75			
36				76			
37				77			
38				78			
39				79			
40				80			

JOB# 4281	Hole/Stake# 5 / 1006		Matrix / West of Decker rd. x Oleander rd.				
DATE: 1/15/15		DRILL MAKE/MODEL: INGERSAL RAND ECM 720 CRAWLER DRILL #838					
RIPPABLE:		NEUTRAL NO LOAD: 100psi					
Marginal:		ROTATION UNDER LOAD: 900psi					
BLASTING req.:		DRILL W/PERCUSION : Feed 900psi / percusion 1200psi					
DEPTH	TIME	RQ	NOTES	DEPTH	TIME	RQ	NOTES
1				41			
2				42			
3				43			
4				44			
5	N/A	Soft	Dirt	45			
6	21			46			
7	21			47			
8	21			48			
9	21			49			
10	21			50			
11	21			51			
12	21	X-Hard	Hard birds eye	52			
13			Granite	53			
14				54			
15				55			
16				56			
17				57			
18				58			
19				59			
20				60			
21				61			
22				62			
23				63			
24				64			
25				65			
26				66			
27				67			
28				68			
29				69			
30				70			
31				71			
32				72			
33				73			
34				74			
35				75			
36				76			
37				77			
38				78			
39				79			
40				80			

JOB# 4281	Hole/Stake# 6 / 1005	Matrix / West of Decker rd. x Oleander rd.
DATE: 1/15/15		DRILL MAKE/MODEL: INGERSAL RAND ECM 720 CRAWLER DRILL #838
RIPPABLE:		NEUTRAL NO LOAD: 100psi
Marginal:		ROTATION UNDER LOAD: 900psi
BLASTING req.:		DRILL W/PERCUSION : Feed 900psi / percusion 1200psi

DEPTH	TIME	RQ	NOTES	DEPTH	TIME	RQ	NOTES
1				41			
2				42			
3				43			
4				44			
5	N/A	Soft	DG	45			
6	11			46			
7	11			47			
8	11			48			
9	11			49			
10	11			50			
11	11			51			
12	11			52			
13	11			53			
14	11			54			
15	11	Hard		55			
16				56			
17				57			
18				58			
19				59			
20				60			
21				61			
22				62			
23				63			
24				64			
25				65			
26				66			
27				67			
28				68			
29				69			
30				70			
31				71			
32				72			
33				73			
34				74			
35				75			
36				76			
37				77			
38				78			
39				79			
40				80			

JOB# 4281	Hole/Stake# 7 / 1004		Matrix / West of Decker rd. x Oleander rd.				
DATE: 1/15/15		DRILL MAKE/MODEL: INGERSAL RAND ECM 720 CRAWLER DRILL #838					
RIPPABLE:		NEUTRAL NO LOAD: 100psi					
Marginal:		ROTATION UNDER LOAD: 900psi					
BLASTING req.:		DRILL W/PERCUSION : Feed 900psi / percusion 1200psi					
DEPTH	TIME	RQ	NOTES	DEPTH	TIME	RQ	NOTES
1				41			
2				42			
3				43			
4				44			
5				45			
6				46			
7				47			
8	N/A	Soft	dirt	48			
9				49			
10				50			
11				51			
12				52			
13				53			
14				54			
15				55			
16	N/A	Soft	DG	56			
17	7.5			57			
18	7.5			58			
19	7.5			59			
20	7.5	Medium	DG	60			
21	5			61			
22	5			62			
23	5			63			
24	5			64			
25	5			65			
26	5	Medium	Granite	66			
27				67			
28				68			
29				69			
30				70			
31				71			
32				72			
33				73			
34				74			
35				75			
36				76			
37				77			
38				78			
39				79			
40				80			

JOB# 4281	Hole/Stake# 8 / 1007	Matrix / West of Decker rd. x Oleander rd.
DATE: 1/15/15		DRILL MAKE/MODEL: INGERSAL RAND ECM 720 CRAWLER DRILL #838
RIPPABLE:		NEUTRAL NO LOAD: 100psi
Marginal:		ROTATION UNDER LOAD: 900psi
BLASTING req.:		DRILL W/PERCUSION : Feed 900psi / percusion 1200psi

DEPTH	TIME	RQ	NOTES	DEPTH	TIME	RQ	NOTES
1				41			
2				42			
3	N/A	Soft	dirt	43			
4	18.5			44			
5	18.5			45			
6	18.5			46			
7	18.5	Hard		47			
8	7			48			
9	7			49			
10	7	Med.Hard		50			
11	8			51			
12	8	Med.Hard		52			
13	6			53			
14	6			54			
15	6			55			
16	6			56			
17	6			57			
18	6			58			
19	6			59			
20	6			60			
21	6			61			
22	6			62			
23	6			63			
24	6			64			
25	6	Soft	Broken	65			
26				66			
27				67			
28				68			
29				69			
30				70			
31				71			
32				72			
33				73			
34				74			
35				75			
36				76			
37				77			
38				78			
39				79			
40				80			

JOB# 4281	Hole/Stake# 9 / 1013	Matrix / West of Decker rd. x Oleander rd.
DATE: 1/15/15		DRILL MAKE/MODEL: INGERSAL RAND ECM 720 CRAWLER DRILL #838
RIPPABLE:		NEUTRAL NO LOAD: 100psi
Marginal:		ROTATION UNDER LOAD: 900psi
BLASTING req.:		DRILL W/PERCUSION : Feed 900psi / percusion 1200psi

DEPTH	TIME	RQ	NOTES	DEPTH	TIME	RQ	NOTES
1				41			
2				42			
3				43			
4				44			
5				45			
6		Soft		46			
7	6			47			
8	6			48			
9	6			49			
10	6			50			
11	6	Med.Soft		51			
12	6			52			
13	6			53			
14	6			54			
15	6			55			
16	6			56			
17	6			57			
18	6			58			
19	6			59			
20	6	Medium		60			
21	6			61			
22	6			62			
23	6			63			
24	6			64			
25	6	Medium		65			
26	8			66			
27	8			67			
28	8			68			
29	8			69			
30	8	Med.Hard		70			
31				71			
32				72			
33				73			
34				74			
35				75			
36				76			
37				77			
38				78			
39				79			
40				80			

JOB# 4281	Hole/Stake# 10 / 1015		Matrix / West of Decker rd. x Oleander rd.				
DATE: 1/15/15		DRILL MAKE/MODEL: INGERSAL RAND ECM 720 CRAWLER DRILL #838					
RIPPABLE:		NEUTRAL NO LOAD: 100psi					
Marginal:		ROTATION UNDER LOAD: 900psi					
BLASTING req.:		DRILL W/PERCUSION : Feed 900psi / percusion 1200psi					

DEPTH	TIME	RQ	NOTES	DEPTH	TIME	RQ	NOTES
1				41			
2				42			
3				43			
4				44			
5				45			
6		Soft		46			
7	8			47			
8	8			48			
9	8			49			
10	8			50			
11	8	Med.Soft		51			
12	10			52			
13	10	Med.Hard		53			
14	8			54			
15	8			55			
16	8			56			
17	8			57			
18	8			58			
19	8			59			
20	8			60			
21	8			61			
22	8			62			
23	8			63			
24	8			64			
25	8	Med.Soft		65			
26	6			66			
27	6			67			
28	6			68			
29	6			69			
30	6	Medium		70			
31				71			
32				72			
33				73			
34				74			
35				75			
36				76			
37				77			
38				78			
39				79			
40				80			

JOB# 4281	Hole/Stake# 11 / 1020		Matrix / West of Decker rd. x Oleander rd.				
DATE: 1/15/15		DRILL MAKE/MODEL: INGERSAL RAND ECM 720 CRAWLER DRILL #838					
RIPPABLE:		NEUTRAL NO LOAD: 100psi					
Marginal:		ROTATION UNDER LOAD: 900psi					
BLASTING req.:		DRILL W/PERCUSION : Feed 900psi / percusion 1200psi					

DEPTH	TIME	RQ	NOTES	DEPTH	TIME	RQ	NOTES
1				41			
2				42			
3				43			
4				44			
5				45			
6				46			
7				47			
8		Soft	DG	48			
9	9			49			
10	9			50			
11	9	Med.Soft		51			
12	5			52			
13	5			53			
14	5			54			
15	5			55			
16	5			56			
17	5			57			
18	5			58			
19	5			59			
20	5			60			
21	5			61			
22	5			62			
23	5			63			
24	5			64			
25	5	Medium		65			
26				66			
27				67			
28				68			
29				69			
30				70			
31				71			
32				72			
33				73			
34				74			
35				75			
36				76			
37				77			
38				78			
39				79			
40				80			

JOB# 4281	Hole/Stake# 12 / 1024		Matrix / West of Decker rd. x Oleander rd.				
DATE: 1/15/15		DRILL MAKE/MODEL: INGERSAL RAND ECM 720 CRAWLER DRILL #838					
RIPPABLE:		NEUTRAL NO LOAD: 100psi					
Marginal:		ROTATION UNDER LOAD: 900psi					
BLASTING req.:		DRILL W/PERCUSION : Feed 900psi / percusion 1200psi					
DEPTH	TIME	RQ	NOTES	DEPTH	TIME	RQ	NOTES
1				41			
2				42			
3				43			
4				44			
5				45			
6				46			
7				47			
8				48			
9				49			
10				50			
11		Soft	Dirt	51			
12	5			52			
13	5			53			
14	5			54			
15	5			55			
16	5			56			
17	5			57			
18	5	Medium		58			
19				59			
20				60			
21		Soft		61			
22	3			62			
23	3			63			
24	3			64			
25	3	Med.Soft		65			
26				66			
27				67			
28				68			
29				69			
30				70			
31				71			
32				72			
33				73			
34				74			
35				75			
36				76			
37				77			
38				78			
39				79			
40				80			

JOB# 4281	Hole/Stake# 13 / 1025		Matrix / West of Decker rd. x Oleander rd.				
DATE: 1/15/15			DRILL MAKE/MODEL: INGERSAL RAND ECM 720 CRAWLER DRILL #838				
RIPPABLE:			NEUTRAL NO LOAD: 100psi				
Marginal:			ROTATION UNDER LOAD: 900psi				
BLASTING req.:			DRILL W/PERCUSION : Feed 900psi / percusion 1200psi				
DEPTH	TIME	RQ	NOTES	DEPTH	TIME	RQ	NOTES
1				41			
2		Soft	Dirt	42			
3	4			43			
4	4			44			
5	4			45			
6	4			46			
7	4			47			
8	4	Med.Soft	DG	48			
9	5			49			
10	5			50			
11	5			51			
12	5	Medium	DG	52			
13	6			53			
14	6	Med.Hard		54			
15	6			55			
16	6			56			
17	6			57			
18	6			58			
19	6			59			
20	6			60			
21	6			61			
22	6			62			
23	6	Medium		63			
24	10			64			
25	10	Soft		65			
26				66			
27				67			
28				68			
29				69			
30				70			
31				71			
32				72			
33				73			
34				74			
35				75			
36				76			
37				77			
38				78			
39				79			
40				80			

JOB# 4281	Hole/Stake# 14 / 1026		Matrix / West of Decker rd. x Oleander rd.				
DATE: 1/15/15			DRILL MAKE/MODEL: INGERSAL RAND ECM 720 CRAWLER DRILL #838				
RIPPABLE:			NEUTRAL NO LOAD: 100psi				
Marginal:			ROTATION UNDER LOAD: 900psi				
BLASTING req.:			DRILL W/PERCUSION : Feed 900psi / percusion 1200psi				
DEPTH	TIME	RQ	NOTES	DEPTH	TIME	RQ	NOTES
1				41			
2				42			
3		Soft	Dirt	43			
4	7			44			
5	7			45			
6	7			46			
7	7			47			
8	7			48			
9	7			49			
10	7			50			
11	7			51			
12	7	Med.Soft	DG	52			
13	7.5			53			
14	7.5			54			
15	7.5			55			
16	7.5			56			
17	7.5			57			
18	7.5			58			
19	7.5			59			
20	7.5	Medium		60			
21	6			61			
22	6			62			
23	6			63			
24	6			64			
25	6	Med.Hard		65			
26				66			
27				67			
28				68			
29				69			
30				70			
31				71			
32				72			
33				73			
34				74			
35				75			
36				76			
37				77			
38				78			
39				79			
40				80			

JOB# 4281	Hole/Stake# 15 / 1019		Matrix / West of Decker rd. x Oleander rd.				
DATE: 1/15/15		DRILL MAKE/MODEL: INGERSAL RAND ECM 720 CRAWLER DRILL #838					
RIPPABLE:		NEUTRAL NO LOAD: 100psi					
Marginal:		ROTATION UNDER LOAD: 900psi					
BLASTING req.:		DRILL W/PERCUSION : Feed 900psi / percusion 1200psi					

DEPTH	TIME	RQ	NOTES	DEPTH	TIME	RQ	NOTES
1				41			
2				42			
3				43			
4				44			
5				45			
6				46			
7				47			
8				48			
9				49			
10		Soft		50			
11	5			51			
12	5	Hard		52			
13	23	Soft		53			
14	7			54			
15	7			55			
16	7			56			
17	7			57			
18	7			58			
19	7			59			
20	7			60			
21	7			61			
22	7	Med.Hard		62			
23				63			
24				64			
25		Hard		65			
26				66			
27				67			
28				68			
29				69			
30				70			
31				71			
32				72			
33				73			
34				74			
35				75			
36				76			
37				77			
38				78			
39				79			
40				80			

JOB# 4281		Hole/Stake# 16 / 1016		Matrix / West of Decker rd. x Oleander rd.			
DATE: 1/15/15			DRILL MAKE/MODEL: INGERSAL RAND ECM 720 CRAWLER DRILL #838				
RIPPABLE:			NEUTRAL NO LOAD: 100psi				
Marginal:			ROTATION UNDER LOAD: 900psi				
BLASTING req.:			DRILL W/PERCUSION : Feed 900psi / percusion 1200psi				
DEPTH	TIME	RQ	NOTES	DEPTH	TIME	RQ	NOTES
1				41			
2		Soft		42			
3	12			43			
4	12			44			
5	12			45			
6	12			46			
7	12	Medium		47			
8	9			48			
9	9			49			
10	9			50			
11	9	Soft		51			
12	8			52			
13	8			53			
14	8	Medium		54			
15	5			55			
16	5			56			
17	5			57			
18	5			58			
19	5			59			
20	5	Soft		60			
21	6			61			
22	6			62			
23	6			63			
24	6			64			
25	6	Med.Hard		65			
26				66			
27				67			
28				68			
29				69			
30				70			
31				71			
32				72			
33				73			
34				74			
35				75			
36				76			
37				77			
38				78			
39				79			
40				80			

JOB# 4281	Hole/Stake# 17 / 1017		Matrix / West of Decker rd. x Oleander rd.				
DATE: 1/15/15		DRILL MAKE/MODEL: INGERSAL RAND ECM 720 CRAWLER DRILL #838					
RIPPABLE:		NEUTRAL NO LOAD: 100psi					
Marginal:		ROTATION UNDER LOAD: 900psi					
BLASTING req.:		DRILL W/PERCUSION : Feed 900psi / percusion 1200psi					
DEPTH	TIME	RQ	NOTES	DEPTH	TIME	RQ	NOTES
1				41			
2				42			
3				43			
4				44			
5				45			
6				46			
7				47			
8				48			
9		Soft		49			
10	4			50			
11	4			51			
12	4	Medium		52			
13	5			53			
14	5			54			
15	5			55			
16	5			56			
17	5			57			
18	5			58			
19	5			59			
20	5			60			
21	5			61			
22	5			62			
23	5	Med.Soft		63			
24	3			64			
25	3			65			
26	3	Soft		66			
27	8			67			
28	8			68			
29	8			69			
30	8			70			
31	8			71			
32	8			72			
33	8			73			
34	8	Med.Hard		74			
35	6.5			75			
36	6.5			76			
37	6.5			77			
38	6.5			78			
39	6.5			79			
40	6.5	Medium		80			

JOB# 4281	Hole/Stake# 18 / 1011	Matrix / West of Decker rd. x Oleander rd.
DATE: 1/15/15		DRILL MAKE/MODEL: INGERSAL RAND ECM 720 CRAWLER DRILL #838
RIPPABLE:		NEUTRAL NO LOAD: 100psi
Marginal:		ROTATION UNDER LOAD: 900psi
BLASTING req.:		DRILL W/PERCUSION : Feed 900psi / percusion 1200psi

DEPTH	TIME	RQ	NOTES	DEPTH	TIME	RQ	NOTES
1				41			
2				42			
3				43			
4				44			
5				45			
6				46			
7				47			
8				48			
9				49			
10		Soft		50			
11	7			51			
12	7	Med.Hard		52			
13	16			53			
14	16			54			
15	16			55			
16	16			56			
17	16			57			
18	16			58			
19	16			59			
20	16	Xtra Hard		60			
21	11			61			
22	11			62			
23	11			63			
24	11			64			
25	11	Hard		65			
26	17.5			66			
27	17.5			67			
28	17.5			68			
29	17.5			69			
30	17.5			70			
31	17.5			71			
32	17.5			72			
33	17.5			73			
34	17.5	Xtra Hard		74			
35	32			75			
36	32			76			
37	32			77			
38	32			78			
39	32			79			
40	32	Xtra Hard		80			

JOB# 4281	Hole/Stake# 19 / 1012	Matrix / West of Decker rd. x Oleander rd.
DATE: 1/15/15		DRILL MAKE/MODEL: INGERSAL RAND ECM 720 CRAWLER DRILL #838
RIPPABLE:		NEUTRAL NO LOAD: 100psi
Marginal:		ROTATION UNDER LOAD: 900psi
BLASTING req.:		DRILL W/PERCUSION : Feed 900psi / percusion 1200psi

DEPTH	TIME	RQ	NOTES	DEPTH	TIME	RQ	NOTES
1				41			
2				42			
3				43			
4		Soft		44			
5	12			45			
6	12			46			
7	12			47			
8	12			48			
9	12	Medium		49			
10	4			50			
11	4			51			
12	4	Soft		52			
13	9			53			
14	9			54			
15	9			55			
16	9			56			
17	9			57			
18	9			58			
19	9			59			
20	9			60			
21	9			61			
22	9			62			
23	9			63			
24	9			64			
25	9	Med.Hard	DG	65			
26	no			66			
27	time			67			
28	record			68			
29				69			
30			DG	70			
31		Med.		71			
32				72			
33				73			
34				74			
35				75			
36				76			
37				77			
38				78			
39				79			
40		Med.Hard		80			

JOB# 4281	Hole/Stake# 20 / 1008	Matrix / West of Decker rd. x Oleander rd.
DATE: 1/15/15		DRILL MAKE/MODEL: INGERSAL RAND ECM 720 CRAWLER DRILL #838
RIPPABLE:		NEUTRAL NO LOAD: 100psi
Marginal:		ROTATION UNDER LOAD: 900psi
BLASTING req.:		DRILL W/PERCUSION : Feed 900psi / percusion 1200psi

DEPTH	TIME	RQ	NOTES	DEPTH	TIME	RQ	NOTES
1				41			
2				42			
3				43			
4				44			
5				45			
6				46			
7				47			
8				48			
9				49			
10				50			
11		Soft	Dirt	51			
12	11			52			
13	11			53			
14	11	Medium	Dg	54			
15	6			55			
16	6			56			
17	6			57			
18	6			58			
19	6			59			
20	6	Medium	DG	60			
21	7			61			
22	7			62			
23	7			63			
24	7			64			
25	7	Soft	Dg	65			
26				66			
27				67			
28				68			
29				69			
30				70			
31				71			
32				72			
33				73			
34				74			
35				75			
36				76			
37				77			
38				78			
39				79			
40				80			

JOB# 4281	Hole/Stake# 21 / 1003		Matrix / West of Decker rd. x Oleander rd.				
DATE: 1/15/15		DRILL MAKE/MODEL: INGERSAL RAND ECM 720 CRAWLER DRILL #838					
RIPPABLE:		NEUTRAL NO LOAD: 100psi					
Marginal:		ROTATION UNDER LOAD: 900psi					
BLASTING req.:		DRILL W/PERCUSION : Feed 900psi / percusion 1200psi					
DEPTH	TIME	RQ	NOTES	DEPTH	TIME	RQ	NOTES
1				41			
2				42			
3				43			
4		Med.Hard		44			
5	6	Soft		45			
6	7.5			46			
7	7.5			47			
8	7.5			48			
9	7.5			49			
10	7.5			50			
11	7.5	Hard		51			
12	32			52			
13	32			53			
14	32			54			
15	32			55			
16	32			56			
17	32			57			
18	32			58			
19	32			59			
20	32	Xtra Hard		60			
21	7			61			
22	7			62			
23	7			63			
24	7			64			
25	7	Xtra Hard		65			
26	12			66			
27	12	Soft		67			
28	7			68			
29	7	Med.Hard		69			
30	36			70			
31	36			71			
32	36			72			
33	36			73			
34	36	Xtra Hard		74			
35	36			75			
36	36			76			
37	36			77			
38	36			78			
39	36	Xtra Hard		79			
40				80			

JOB# 4281	Hole/Stake# 22 / 1002	Matrix / West of Decker rd. x Oleander rd.
DATE: 1/15/15		DRILL MAKE/MODEL: INGERSAL RAND ECM 720 CRAWLER DRILL #838
RIPPABLE:		NEUTRAL NO LOAD: 100psi
Marginal:		ROTATION UNDER LOAD: 900psi
BLASTING req.:		DRILL W/PERCUSION : Feed 900psi / percusion 1200psi

DEPTH	TIME	RQ	NOTES	DEPTH	TIME	RQ	NOTES
1		Soft		41			
2	6			42			
3	6			43			
4	6			44			
5	6			45			
6	6	Hard		46			
7	4			47			
8	4			48			
9	4			49			
10	4			50			
11	4	Med.Hard		51			
12	25			52			
13	25			53			
14	25			54			
15	25			55			
16	25			56			
17	25	Xtra Hard		57			
18	30			58			
19	30			59			
20	30			60			
21	30			61			
22	30			62			
23	30			63			
24	30			64			
25	30	Xtra Hard		65			
26	28			66			
27	28			67			
28	28			68			
29	28			69			
30	28			70			
31	28			71			
32	28			72			
33	28			73			
34	28	Xtra Hard		74			
35	28.5			75			
36	28.5			76			
37	28.5			77			
38	28.5			78			
39	28.5			79			
40	28.5	Xtra Hard		80			

JOB# 4281	Hole/Stake# 23 / 1009	Matrix / West of Decker rd. x Oleander rd.
DATE: 1/15/15		DRILL MAKE/MODEL: INGERSAL RAND ECM 720 CRAWLER DRILL #838
RIPPABLE:		NEUTRAL NO LOAD: 100psi
Marginal:		ROTATION UNDER LOAD: 900psi
BLASTING req.:		DRILL W/PERCUSION : Feed 900psi / percusion 1200psi

DEPTH	TIME	RQ	NOTES	DEPTH	TIME	RQ	NOTES
1				41			
2				42			
3				43			
4				44			
5				45			
6		Med.Hard		46			
7	9			47			
8	9			48			
9	9			49			
10	9			50			
11	9	Med.Hard		51			
12	7			52			
13	7			53			
14	7			54			
15	3"-4"	crack	small crack with a lar-	55			
16	7		ge noticable volume	56			
17	7	Xtra Hard	of water encountered	57			
18	25		Only mentioned	58			
19	25		Because the water	59			
20	25		intrusion was very	60			
21	25		noticable at this point	61			
22	25		water was present in	62			
23	25		all other test holes	63			
24	25			64			
25	25	Xtra Hard		65			
26	36			66			
27	36			67			
28	36			68			
29	36			69			
30	36			70			
31	36			71			
32	36			72			
33	36			73			
34	36	Xtra Hard		74			
35	24			75			
36	24			76			
37	24			77			
38	24			78			
39	24			79			
40	24	Xtra Hard		80			

JOB# 4281	Hole/Stake# 24 / 1010		Matrix / West of Decker rd. x Oleander rd.				
DATE: 1/15/15		DRILL MAKE/MODEL: INGERSAL RAND ECM 720 CRAWLER DRILL #838					
RIPPABLE:		NEUTRAL NO LOAD: 100psi					
Marginal:		ROTATION UNDER LOAD: 900psi					
BLASTING req.:		DRILL W/PERCUSION : Feed 900psi / percusion 1200psi					

DEPTH	TIME	RQ	NOTES	DEPTH	TIME	RQ	NOTES
1				41			
2				42			
3				43			
4				44			
5				45			
6				46			
7				47			
8				48			
9		Soft		49			
10				50			
11				51			
12		Med. Hard		52			
13	10.5			53			
14	10.5			54			
15	10.5			55			
16	10.5			56			
17	10.5			57			
18	10.5			58			
19	10.5			59			
20	10.5			60			
21	10.5			61			
22	10.5			62			
23	10.5			63			
24	10.5			64			
25	10.5	Xtra Hard		65			
26	7			66			
27	7			67			
28	7			68			
29	7			69			
30	7			70			
31	7			71			
32	7			72			
33	7			73			
34	7			74			
35	7			75			
36	7			76			
37	7			77			
38	7			78			
39	7	Xtra Hard		79			
40				80			

JOB# 4281	Hole/Stake# 25 / 1018		Matrix / West of Decker rd. x Oleander rd.				
DATE: 1/15/15			DRILL MAKE/MODEL: INGERSAL RAND ECM 720 CRAWLER DRILL #838				
RIPPABLE:			NEUTRAL NO LOAD: 100psi				
Marginal:			ROTATION UNDER LOAD: 900psi				
BLASTING req.:			DRILL W/PERCUSION : Feed 900psi / percusion 1200psi				
DEPTH	TIME	RQ	NOTES	DEPTH	TIME	RQ	NOTES
1				41			
2				42			
3				43			
4		Soft		44			
5	6			45			
6	6			46			
7	6			47			
8	6			48			
9	6			49			
10	6			50			
11	6			51			
12	6	Med. Soft		52			
13	7			53			
14	7			54			
15	7			55			
16	7			56			
17	7			57			
18	7			58			
19	7			59			
20	7	Hard		60			
21	4			61			
22	4			62			
23	4			63			
24	4			64			
25	4			65			
26	4	Soft		66			
27	6			67			
28	6			68			
29	6			69			
30	6			70			
31	6			71			
32	6			72			
33	6			73			
34	6			74			
35	6			75			
36	6			76			
37	6			77			
38	6	Medium		78			
39	9			79			
40	9	Hard		80			

APPENDIX C

LABORATORY TESTING PROCEDURES AND TEST RESULTS

APPENDIX C

Laboratory Testing Procedures and Test Results

The laboratory-testing program was directed towards providing quantitative data relating to the relevant engineering properties of the soil. Samples considered representative of site conditions were tested in general accordance with American Society for Testing and Materials (ASTM) procedure and/or California Test Methods (CTM), where applicable. The following summary is a brief outline of the test type and a table summarizing the test results.

Soil Classification: Soil were classified according the Unified Soil Classification System (USCS) in accordance with ASTM Test Methods D2487 and D2488. The soil classifications (or group symbol) are shown on the laboratory test data and test pit logs.

Expansion Index: the Expansion Index Test, U.B.C. Standard No. 18 2 and/or ASTM D4829 evaluated the expansion potential of selected samples. Specimens are molded under a given compactive energy to approximately the optimum water content and approximately 50 percent saturation or approximately 90 percent relative compaction. The prepared 1-inch-thick by 2.42-inch-diameter specimens are loaded to an equivalent 144 psf surcharge and are inundated with tap water until volumetric equilibrium is reached. The results of these tests are presented in the table below:

SAMPLE LOCATION	SAMPLE DESCRIPTION	EXPANSION INDEX	EXPANSION POTENTIAL*
TP-21, Bulk 0-5’	Silty SAND	4	Non-Expansive
TP-30, Bulk 0-5’	Silty SAND	9	Non-Expansive

*Per ASTM D4829

Soluble Sulfates: The soluble sulfate contents of selected samples were determined by standard geotechnical methods (CTM 417). The soluble sulfate content is used to determine the appropriate cement type and maximum water-cement ratios. The test results are presented in the table below:

SAMPLE LOCATION	SAMPLE DESCRIPTION	SULFATE CONTENT (ppm)	SULFATE EXPOSURE*
TP-21, Bulk 0-5’	Silty SAND	75	*Negligible*
TP-30, Bulk 0-5’	Silty SAND	33	*Negligible*

*Per ACI 318R-05 Table 4.3.1

Minimum Resistivity and pH Tests: Minimum resistivity and pH tests were performed with CTM 643. The results are presented in the table below:

SAMPLE LOCATION	SAMPLE DESCRIPTION	pH	MINIMUM RESISTIVITY (ohm-cm)
TP-21, Bulk 0-5’	Silty SAND	6.9	2,180
TP-30, Bulk 0-5’	Silty SAND	7.3	1,700

Chloride Content: Chloride content was tested with CTM 422. The results are presented below:

SAMPLE LOCATION	SAMPLE DESCRIPTION	CHLORIDE CONTENT (ppm)
TP-21, Bulk 0-5'	Silty SAND	105
TP-30, Bulk 0-5'	Silty SAND	220

Maximum Dry Density Tests: The maximum dry density and optimum water content of typical materials were determined in accordance with ASTM D1557. The results of these tests are presented in the table below:

SAMPLE LOCATION	SAMPLE DESCRIPTION	MAXIMUM DRY DENSITY (% by weight)	OPTIMUM WATER CONENT (%)
TP-21, Bulk 0-5'	Silty SAND	126.4	11.1
TP-30, Bulk 0-5'	Silty SAND	131.9	8.7

Direct Shear: Direct shear tests were performed on selected remolded and/or undisturbed samples with ASTM D 3080. Results of these tests are presented in the table below.

SAMPLE LOCATION	SAMPLE DESCRIPTION	FRICTION ANGLE (degrees)*	APPARENT COHESION (psf)*	FRICTION ANGLE (degrees)**	APPARENT COHESION (psf)**
TP-21, Bulk 0-5'	Silty SAND	34	310	31	241
TP-30, Bulk 0-5'	Silty SAND	30	229	28	215
TP-30, Bulk 0-5'***	Silty SAND	31	265	31	233

*Peak Values; **Ultimate Values; ***Remolded

R-Value: The R-value of representative samples were determined with CTM 301. The test results are presented in the table below:

SAMPLE LOCATION	SAMPLE DESCRIPTION	R-VALUE
TP-25, Bulk @ 0-5 feet	Silty SAND	33

APPENDIX D

SEISMIC REFRACTION SURVEY

TERRA GEOSCIENCES

SEISMIC REFRACTION SURVEY

PROPOSED DECKER II PROJECT

SW CORNER OF DECKER ROAD AND OLEANDER AVENUE

PERRIS AREA, RIVERSIDE COUNTY, CALIFORNIA

Project No. 142740-2

January 12, 2015

Prepared for:

Matrix Geotechnical Consulting
P.O. Box 2161
Temecula, California 92593

Consulting Engineering Geology & Geophysics

P.O. Box 1090, Loma Linda, CA 92354 • 909 796-4667

Matrix Geotechnical Consulting
P.O. Box 2161
Temecula, California 92593

Attention: Mr. Chris Josef

Regarding: Seismic Refraction Survey
Proposed Decker II Project
SW Corner of Decker Road and Oleander Avenue
Perris Area, Riverside County, California
MGC Project No. M1103-008

INTRODUCTION

As requested, this firm has performed a geophysical survey using the seismic refraction method for the above-referenced site. The purpose of this investigation was to assess the general seismic velocity characteristics of the underlying earth materials and to evaluate whether high velocity earth materials (non-rippable) are present which could possibly indicate areas of potential excavation difficulties, and also to aid in evaluating the subsurface structure and seismic velocity distribution. The underlying earth materials have been mapped (Morton, 2001) to consist of Cretaceous age granitic rocks (locally referred to as the Val Verde tonalite) comprised of a gray-weathering, relatively homogeneous, massive to well-foliated, medium- to coarse-grained, biotite hornblende tonalite. It is also possible that very old alluvial fan deposits (early Pleistocene age) comprised of well-indurated sand deposits, may be found locally mantling portions of the site. The locations of the survey lines have been approximated on a captured Google™ Earth image (Google™ Earth, 2013), in turn overlain by a topographic base map, which is presented as the Seismic Line Location Map, Plate 1, for reference. As authorized by you, the following services were performed during this study:

- ***Review of available published and unpublished geologic/geophysical data in our files pertinent to the site.***

- ***Performing a geophysical survey by a State of California licensed Professional Geophysicist; to include six seismic refraction traverses.***

- ***Preparation of this report, presenting our findings and conclusions with respect to the bedrock velocity characteristics and the expected excavation potentials.***

Accompanying Map and Appendices

Plate 1 - Seismic Line Location Map
Appendix A - Layer Velocity Models
Appendix B - Refraction Tomographic Models
Appendix C - Excavation Considerations
Appendix D - References

SEISMIC REFRACTION SURVEY

Methodology

The seismic refraction method consists of measuring (at known points along the surface of the ground) the travel times of compressional waves generated by an impulsive energy source and can be used to estimate the layering, structure, and seismic acoustic velocities of subsurface horizons. Seismic waves travel down and through the soils and rocks, and when the wave encounters a contact between two earth materials having different velocities, some of the wave's energy travels along the contact at the velocity of the lower layer. The fundamental assumption is that each successively deeper layer has a velocity greater than the layer immediately above it. As the wave travels along the contact, some of the wave's energy is refracted toward the surface where it is detected by a series of motion-sensitive transducers (geophones). The arrival time of the seismic wave at the geophone locations can be related to the relative seismic velocities of the subsurface layers in feet per second (fps), which can then be used to aid in interpreting both the depth and type of materials encountered.

Field Procedures

Six seismic refraction survey lines (Seismic lines S-8 through S-13) were performed along representative areas as delineated by your firm. The traverses were located in the field by use of Google™ Earth (2013) imagery and GPS coordinates. Twenty-four 14-Hertz geophones, spaced at eight- to ten-foot intervals, were employed on each line to detect both the direct and refracted waves, with a 16-pound sledge-hammer being used as the energy source to produce the seismic waves. Seismic Line S-11 consisted of two overlapped individual spreads to provide a longer continuous profile. The seismic wave arrivals were digitally recorded in SEG-2 format on a Geometrics StrataVisor™ NZXP model signal enhancement refraction seismograph. Seven shot points were utilized along each spread using forward, reverse, and several intermediate locations in order to obtain high resolution survey data for velocity analysis and depth modeling purposes. The data was acquired using a sampling rate of 0.0625 milliseconds having a record length of 0.08 seconds with no acquisition filters. During acquisition, the seismograph displays the seismic wave arrivals on the computer screen which were used to analyze the arrival time of the primary seismic “P”-waves at each geophone station, in the form of a wiggle trace for quality control purposes in the field. Each geophone and seismic shot location was surveyed using a hand level and ruler for relative topographic correction, with “0” representing the lowest point along each line.

Data Processing

The recorded seismic data was subsequently transferred to our office computer for processing and analyzing purposes, using the computer programs **SIPwin** (**S**eismic Refraction **I**nterpretation **P**rogram for **Win**dows) developed by Rimrock Geophysics, Inc. (2004); **Refractor** (Geogiga, 2001-2013); and **Rayfract™** (Intelligent Resources, Inc., 1996-2014). All of the computer programs perform their analysis using exactly the same input data which includes the first-arrival “P”-waves and survey line geometry.

- **SIPwin** is a ray-trace modeling program that evaluates the subsurface using layer assignments based on time-distance curves and is better suited for layered media, using the "Seismic Refraction Modeling by Computer" method (Scott, 1973). The first step in the modeling procedure is to compute layer velocities by least-squares techniques. Then the program uses the delay-time method to estimate depths to the top of layer-2. A forward modeling routine traces rays from the shot points to each geophone that received a first-arrival ray refracted along the top of layer-2. The travel time of each such ray is compared with the travel time recorded in the field by the seismic system. The program then adjusts the layer-2 depths so as to minimize discrepancies between the computed ray-trace travel times and the first arrival times picked from the seismic waveform record. The process of ray tracing and model adjustment is repeated a total of six times to improve the accuracy of depths to the top of layer-2. This first-arrival picks were then used to generate the Layer Velocity Models using the **SIPwin** computer program, which presents the subsurface velocities as individual layers and are presented within Appendix A for reference. In addition, the associated Time-Distance Plot for the survey lines which shows the individual data picks of the first "P-wave" arrival times, also appears in Appendix A.

- **Refractor** is seismic refraction software that also evaluates the subsurface using layer assignments utilizing interactive and interchangeable analytical methods that include the Delay-Time method, the ABC method, and the Generalized Reciprocal Method (GRM). These methods are used for defining irregular non-planar refractors and are briefly described below. The Delay-Time method will measure the delay time depth to a refractor beneath each geophone rather than at shot points. Delay-time is the time spent by a wave to travel up or down through the layer (slant path) compared to the time the wave would spend if traveling along the projection of the slant path on the refractor. The ABC (intercept time) method makes use of critically refracted rays converging on a common surface position. This method involves using three surface to surface travel times between three geophones and the velocity of the first layer in an equation to calculate depth under the central geophone and is applied to all other geophones on the survey line. The GRM method is a technique for delineating undulating refractors at any depth from in-line seismic refraction data consisting of forward and reverse travel-times and is capable of resolving dips of up to 20% and does not over-smooth or average the subsurface refracting layers. In addition, the technique provides an approach for recognizing and compensating for hidden layer conditions.

- **Rayfract™** is seismic refraction tomography software that models subsurface refraction, transmission, and diffraction of acoustic waves which generally indicates the relative structure and velocity distribution of the subsurface using first break energy propagation modeling. An initial 1D gradient model is created using the DeltatV method (Gebrande and Miller, 1985) which gives a good initial fit between modeled and picked first breaks. The DeltatV method is a turning-ray inversion method which delivers continuous depth vs. velocity profiles for all profile stations. These profiles consist of horizontal inline offset, depth, and velocity triples. The method handles real-life geological conditions such as velocity gradients, linear increasing of velocity with depth, velocity inversions, pinched-out layers and outcrops, and faults and local velocity anomalies. This initial model is then refined automatically with a true 2D WET (Wavepath Eikonal Traveltime) tomographic inversion (Schuster and Quintus-Bosz, 1993).

WET tomography models multiple signal propagation paths contributing to one first break, whereas conventional ray tracing tomography is limited to the modeling of just one ray per first break. This computer program performs the analysis by using the same first-arrival P-wave times and survey line geometry that were generated during the layer velocity model analyses. The associated Refraction Tomographic Models which display the subsurface earth material velocity structure, is represented by the velocity contours (isobars displayed in feet/second), supplemented with the color-coded velocity shading for visual reference, and are presented within Appendix B.

The combined use of these computer programs provided a more thorough and comprehensive analysis of the subsurface structure and velocity characteristics. Each computer program has a specific purpose based on the objective of the analysis being performed. **SIPwin** and **Refractor** were primarily used for detecting generalized subsurface velocity layers providing “weighted average velocities.” The processed seismic data of these two programs were compared and averaged to provide a final composite layer velocity model which provided a more thorough representation of the subsurface. **Rayfract™** provided tomographic velocity and structural imaging that is very conducive to detecting strong lateral velocity characteristics such as imaging corestones, dikes, and other subsurface structural characteristics.

SUMMARY OF GEOPHYSICAL INTERPRETATION

To begin our discussion, it is important to consider that the seismic velocities obtained within bedrock materials are influenced by the nature and character of the localized major structural discontinuities (foliation, fracturing, relic bedding, etc.), creating anisotropic conditions. Anisotropy (direction-dependent properties of materials) can be caused by “micro-cracks,” jointing, foliation, layered or inter-bedded rocks with unequal layer stiffness, small-scale lithologic changes, etc (Barton, 2007). Velocity anisotropy complicates interpretation and it should be noted that the seismic velocities obtained during this survey may have been influenced by the nature and character of any localized structural discontinuities within the bedrock underlying the subject site. Generally, it is expected that higher (truer) velocities will be obtained when the seismic waves propagate along direction (strike) of the dominant structure, with a damping effect when the seismic waves travel in a perpendicular direction. Such variable directions can result in velocity differentials of between 2% to 40% depending upon the degree of the structural fabric (i.e., weakly-moderately-strongly foliated, respectively). Therefore, the seismic velocities obtained during our field study and as discussed below, should be considered minimum velocities at this time.

The first method described below used for data analysis is the traditional layer method (**SIPwin** and **Refractor**). Using this method, it should be understood that the data obtained represents an average of seismic velocities within any given layer. For example, high seismic velocity boulders, dikes, or other local lithologic inconsistencies, may be isolated within a low velocity matrix, thus yielding an average medium velocity for that layer. Therefore, in any given layer, a range of velocities could be anticipated, which can also result in a wide range of excavation characteristics. In general, the site where locally surveyed was noted to be characterized by three major subsurface layers with respect to seismic velocities. The following layer summaries have been prepared using the **SIPwin** and **Refractor** analysis, with the representative Layer Velocity Models presented within Appendix A along with their respective Time-Distance Plots.

- **Velocity Layer V1**:

  This uppermost velocity layer (V1) is most likely comprised of topsoil, colluvium, older alluvial sediments, and/or completely-weathered and fractured bedrock materials. This layer has an average weighted velocity of 1,279 to 1,657 fps, which is typical for these types of unconsolidated surficial earth materials.

- **Velocity Layer V2**:

  The second layer (V2) yielded a seismic velocity range of 3,021 to 4,648 fps, which is typical for highly- to moderately-weathered bedrock materials. This velocity range may indicate the presence of homogeneous weathered bedrock with a relatively wide spaced joint/fracture system and/or the possibility of buried relatively-fresher boulders within a very-highly decomposed bedrock matrix. Additionally, the presence of older alluvial sediments, such as mapped just east of the site (Morton, 2001), may also be locally present based upon the degree of sediment induration.

- **Velocity Layer V3**:

  The third layer (V3) indicates the presence of slightly-weathered to fresh granitic bedrock, having a seismic velocity range of 7,227 to 11,039 fps. These higher velocities signify the decreasing effect of weathering as a function of depth and could indicate the presence of abundant widely-scattered buried fresh large crystalline boulders in highly-weathered matrix, or possibly a slightly-weathered to fresher crystalline bedrock matrix, that has a wide-spaced fracture system.

Using **Rayfract™**, tomographic models were also prepared for comparative purposes to better illustrate the general structure and velocity distribution of the subsurface, as presented within Appendix B. Although no discrete velocity layers or boundaries are created, these models generally resemble the corresponding overall average layer velocities as presented within Appendix A. In general, the seismic velocity of the bedrock and/or alluvial deposits gradually increases with depth, with numerous strong lateral velocity differentials suggesting the presence of buried corestones and/or dike structures. The colors representing the velocity gradients have been standardized on all of the models for comparative purposes.

GENERALIZED RIPPABILITY CHARACTERISTICS OF BEDROCK

A summary of the generalized rippability characteristics of bedrock based on a compilation of rippability performance charts prepared by Caterpillar, Inc. (2004), Caltrans (Stephens, 1978), and Santi (2006), has been provided to aid in evaluating potential excavation difficulties with respect to the seismic velocities obtained along the local areas surveyed. These seismic velocity ranges and rippability potentials have been tabulated below for reference.

TABLE 1- CATERPILLAR RIPPABILITY CHART (D9 Ripper)

Granitic Rock Velocity	Rippability
< 6,800	Rippable
6,800 – 8,000	Moderately Rippable
> 8,000	Non-Rippable

Additionally, we have provided the Caltrans Rippability Chart as presented below within Table 2 for comparison. These values are from published Caltrans studies (Stephens, 1978) that are based on their experience which are more conservative than Caterpillar's rippability charts. It should be noted that the type of bedrock was not indicated.

TABLE 2- STANDARD CALTRANS RIPPABILITY CHART

Velocity (feet/sec ±)	Rippability
< 3,500	Easily Ripped
3,500 – 5,000	Moderately Difficult
5,000 – 6,600	Difficult Ripping / Light Blasting
> 6,600	Blasting Required

Table 3 is partially modified from the "Engineering Behavior from Weathering Grade" as presented by Santi (2006), which also provides velocity ranges with respect to rippability potentials, along with other rock engineering properties that may be pertinent.

TABLE 3- SUMMARY OF ROCK ENGINEERING PROPERTIES

ENGINEERING PROPERTY:	Slightly Weathered	Moderately Weathered	Highly Weathered	Completely Weathered
Excavatability	Blasting necessary	Blasting to rippable	Generally rippable	Rippable
Slope Stability	½ :1 to 1:1 (H:V)	1:1 (H:V)	1:1 to 1.5:1 (H:V)	1.5:1 to 2:1 (H:V)
Schmidt Hammer Value	51 – 56	37 – 48	12 – 21	5 – 20
Seismic Velocity (fps)	8,200 – 13,125	5,000 – 10,000	3,300 – 6,600	1,650 – 3,300

Additionally, as presented below on Figure 1, the Caterpillar D9R Ripper Performance Chart (Caterpillar, 2012) has been provided for reference.

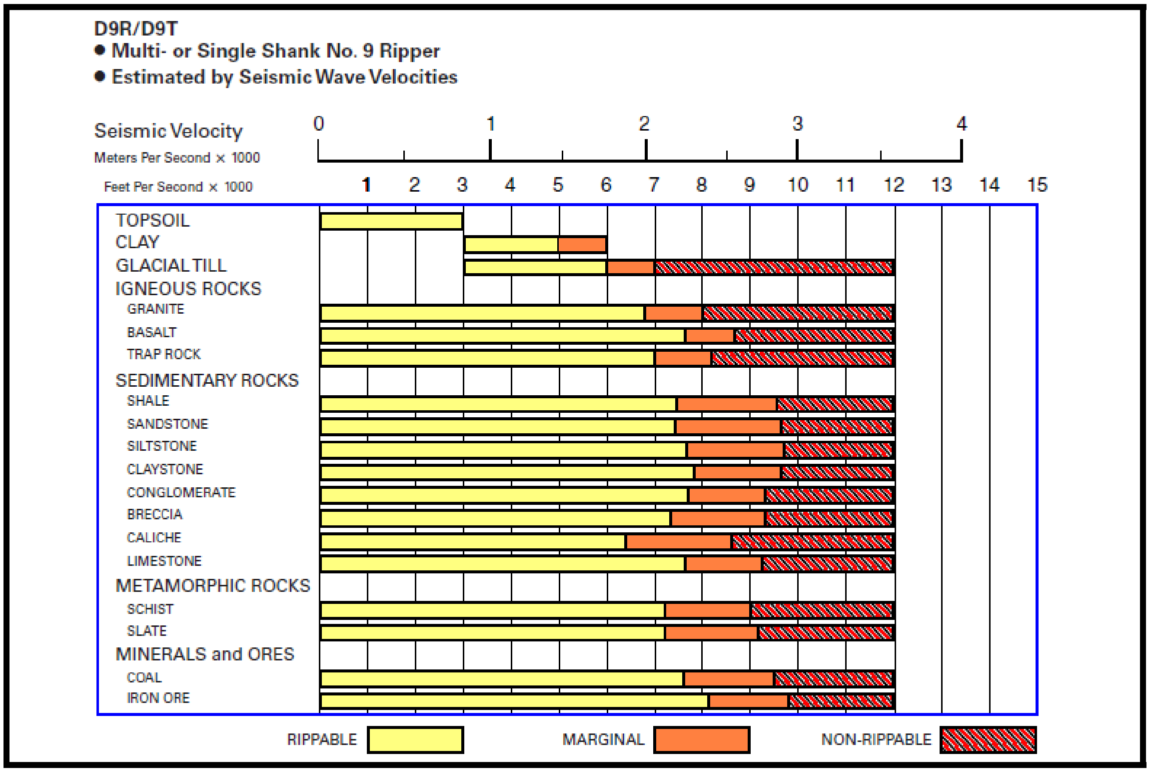


FIGURE 1- Caterpillar D9R Ripper Performance Chart

For purposes of the discussion in this report with respect to the expected bedrock rippability characteristics, we are assuming that a D9R/D9T dozer will be used as a minimum, such as illustrated above. Smaller excavating equipment will most likely result in slower production rates and possible refusal within relatively lower velocity bedrock materials. It should be noted that the decision for blasting of bedrock materials for facilitating the excavation process is sometimes made based upon economic production reasons and not solely on the rippability (velocity/hardness) characteristics of the bedrock.

A summary of the generalized rippability characteristics of granitic bedrock has been provided to aid in evaluating potential excavation difficulties with respect to the seismic velocities obtained along the local area surveyed. The velocity ranges described below are approximate and assume typical, good-working, heavy excavation equipment, such as single shank D9R dozer, such as described by Caterpillar, Inc. (2000 and 2012); however, different excavating equipment (i.e., trenching equipment) <u>may not</u> correlate well with these velocity ranges. Trenching operations which utilize large excavator-type equipment within granitic bedrock materials, typically encounter very difficult to non-productable conditions where seismic velocities are generally greater than 4,000± fps, and less for smaller backhoe-type equipment.

- **<u>Rippable Condition (0 - 4,000 ft/sec)</u>**:

  This velocity range indicates rippable materials which may consist of alluvial-type deposits and decomposed granitic bedrock, with random hardrock floaters. These materials typically break down into silty sands (depending on parent lithologic materials), whereas floaters will require special disposal. Some areas containing numerous hardrock floaters may present utility trench problems. Large floaters exposed at or near finished grade may present problems for footing or infrastructure trenching.

- **<u>Marginally Rippable Condition (4,000 - 7,000 ft/sec)</u>**:

  This range of seismic velocities indicates materials which may consist of moderately weathered bedrock and/or large areas of fresh bedrock materials separated by weathered fractured zones. These bedrock materials are generally rippable with difficulty by a Caterpillar D9R or equivalent. Excavations may produce material that will partially break down into a coarse, silty to clean sand, with a high percentage of very coarse sand to pebble-sized material depending on the parent bedrock lithology. Less fractured or weathered materials will probably require blasting to facilitate removal.

- **<u>Non-Rippable Condition (7,000 ft/sec or greater)</u>**:

  This velocity range includes non-rippable material consisting primarily of moderately fractured bedrock at lower velocities and only slightly fractured or unfractured rock at higher velocities. Materials in this velocity range may be marginally rippable, depending upon the degree of fracturing and the skill and experience of the operator. Tooth penetration is often the key to ripping success, regardless of seismic velocity. If the fractures and joints do not allow tooth penetration, the material may not be ripped effectively; however, pre-blasting or "popping" may induce sufficient fracturing to permit tooth entry. In their natural state, materials with these velocities are generally not desirable for building pad grade, due to difficulty in footing and utility trench excavation. Blasting will most likely produce oversized material, requiring special disposal.

GEOLOGIC & EARTHWORK CONSIDERATIONS

To evaluate whether a particular bedrock material can be ripped or excavated, this geophysical survey should be used in conjunction with the geologic and/or geotechnical report and/or information gathered for the subject project which may describe the physical properties of the bedrock. The physical characteristics of bedrock materials that favor ripping generally include the presence of fractures, faults, and other structural discontinuities, weathering effects, brittleness or crystalline structure, stratification or lamination, large grain size, moisture permeated clay, and low compressive strength. If the bedrock is foliated and/or fractured at depth, this structure could aid in excavation production.

Unfavorable bedrock conditions can include such characteristics as massive and homogeneous formations, non-crystalline structure, absence of planes of weakness, fine-grained materials, and formations of clay origin where moisture makes the material plastic. Use of these physical bedrock conditions along with the subsurface velocity characteristics as presented within this report should aid in properly evaluating the type of equipment that will be necessary and the production levels that can be anticipated for this project. A summary of excavation considerations is included within Appendix C in order to provide you with a better understanding of the complexities of excavation in bedrock materials. These concepts should be understood so that proper planning and excavation techniques can be employed by the selected grading contractor.

SUMMARY OF FINDINGS AND CONCLUSIONS

The raw field data was considered to be of good quality with moderate amounts of ambient "noise" being introduced during our survey, mostly from local air traffic and aircraft operations at the nearby March Air Reserve Base, and also from vehicular traffic along nearby roads and the 215 Freeway to the east. Analysis of the data and picking of the primary "P"-wave arrivals was therefore performed with some difficulty, with minor interpolation of data being necessary. Based on the results of our comparative seismic analyses of the computer programs **SIPwin**, **Refractor**, and **Rayfract™**, the seismic refraction survey line models appear to generally coincide with one another, with some minor variances due to the methods that these programs process and integrate the input data. The anticipated excavation potentials of the velocity layers encountered locally during our survey are as follows:

- **Velocity Layer V1**:

  No excavating difficulties are expected to be encountered within the uppermost, low-velocity layer V1 (average weighted velocity of 1,279 to 1,657 fps) and should excavate with conventional ripping. This layer is expected to be comprised of topsoil, colluvium, possible older alluvium sediments, and/or completely-weathered and fractured bedrock materials. Localized boulders should be anticipated based on surficial exposures, which may require more significant excavation techniques.

- **Velocity Layer V2**:

  The second layer V2 (average weighted velocity of 3,021 to 4,648 fps) is believed to consist of highly- to moderately-weathered granitic bedrock (within higher end of velocity range) and/or possibly older alluvial sediments (within lower end of velocity range). Using the rock classifications as presented within Tables 1 through 3, seismic wave velocities of less than 6,800± fps are generally noted to be within the threshold for conventional ripping. Isolated floaters (i.e., boulders, corestones, etc.) should be expected to be present within this layer and could produce somewhat difficult conditions locally. Placement of infrastructure within this velocity layer may require some breaking and/or light blasting to obtain desired grade.

- **Velocity Layer V3**:

  The third layer V3 is believed to consist of slightly-weathered to fresh bedrock. Extremely hard excavation difficulties within this deeper velocity layer (average weighted velocity range of 7,227 to 11,039 fps) will be encountered. This layer may consist of relatively fresher homogeneous bedrock, or may contain higher velocity scattered corestones, dikes, and other lithologic variables, within a relatively lower velocity bedrock matrix. Continuous blasting will most likely be required within this velocity layer to achieve desired grade, including any infrastructure.

The ray sampling coverage of the subsurface seismic waves that were acquired during the processing of the tomographic models appeared to be of very good quality which was verified by having a Root Mean Square Error (RMS) of 1.1 to 1.9 percent (see lower right-hand corner of each model). The RMS error (misfit between picked and modeled first break times) is automatically calculated during the processing routine, with a value of less than 2.0% being preferred, of which all of the models obtained. Based on the tomographic models and typical excavation characteristics observed within granitic bedrock of the southern California region, anticipation of gradual increasing hardness with depth should be anticipated during grading. Significant lateral velocity variations will most likely be encountered across the predominance of the site generally due to the presence of buried corestones and/or dikes such as imaged in some of the tomographic refraction models and as also expressed as scattered outcrops across the subject site.

CLOSURE

The field geophysical survey was performed by the undersigned on January 6 and January 7, 2015 using "state of the art" geophysical equipment and techniques along the selected portions of the subject study area as directed by you. The seismic data was further evaluated using recently developed tomographic inversion techniques to provide a more thorough analysis and understanding of the subsurface structural conditions. It should be noted that our data was obtained along only six specific locations therefore other areas in the local vicinity beyond the limits of our seismic lines may contain different velocity layers and depths not encountered during our field survey. Additional survey traverses may be necessary to further evaluate the excavation characteristics across other portions of the site where cut grading will be proposed.

In summary, the results of this seismic refraction survey are to be considered as an aid to assessing the rippability and excavation potentials of the bedrock locally. This information should be carefully reviewed by the grading contractor and representative “test” excavations with the proposed type of excavation equipment for the proposed construction should be considered, so that they may be correlated with the data presented within this report. Estimates of layer velocity boundaries as presented in this report are generally considered to be within 10± percent of the total depth of the contact.

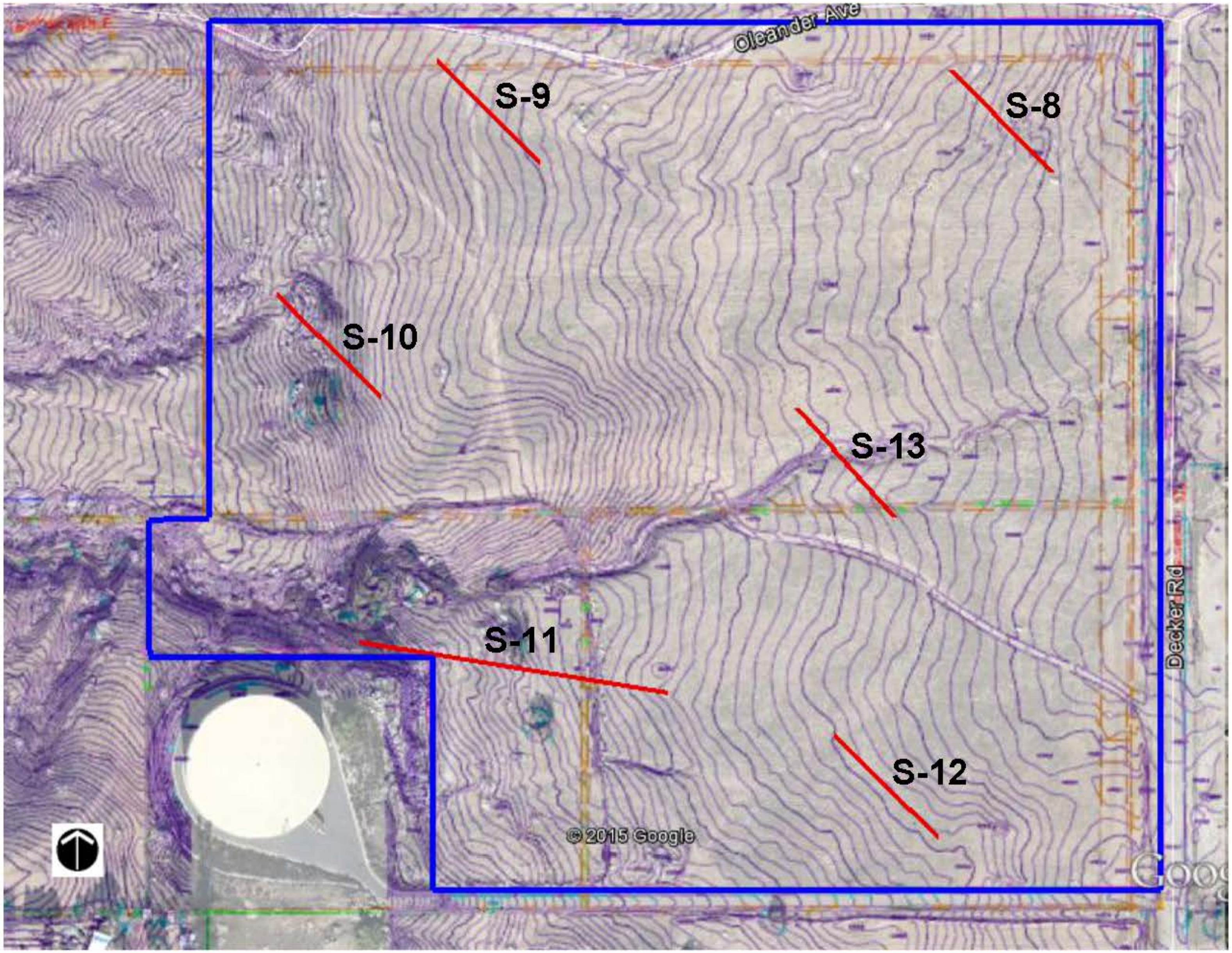
It is important to understand that the fundamental limitation for seismic refraction surveys is known as nonuniqueness, wherein a specific seismic refraction data set does not provide sufficient information to determine a single “true” earth model. Therefore, the interpretation of any seismic data set uses “best-fit” approximations along with the geologic models that appear to be most reasonable for the local area being surveyed. Client should also understand that when using the theoretical geophysical principles and techniques discussed in this report, sources of error are possible in both the data obtained and in the interpretation and that the results of this survey may not represent actual subsurface conditions. These are all factors beyond **Terra Geosciences** control and no guarantees as to the results of this survey can be made. We make no warranty, either expressed or implied.

This opportunity to be of service is sincerely appreciated. If you should have any questions regarding this report or do not understand the limitations of this study or the data and results that are presented, please do not hesitate to contact our office at your earliest convenience.

Respectfully submitted,
TERRA GEOSCIENCES

Donn C. Schwartzkopf
Principal Geophysicist
PGP 1002

SEISMIC LINE LOCATION MAP



PROJECT NO. 142740-2

PLATE 1

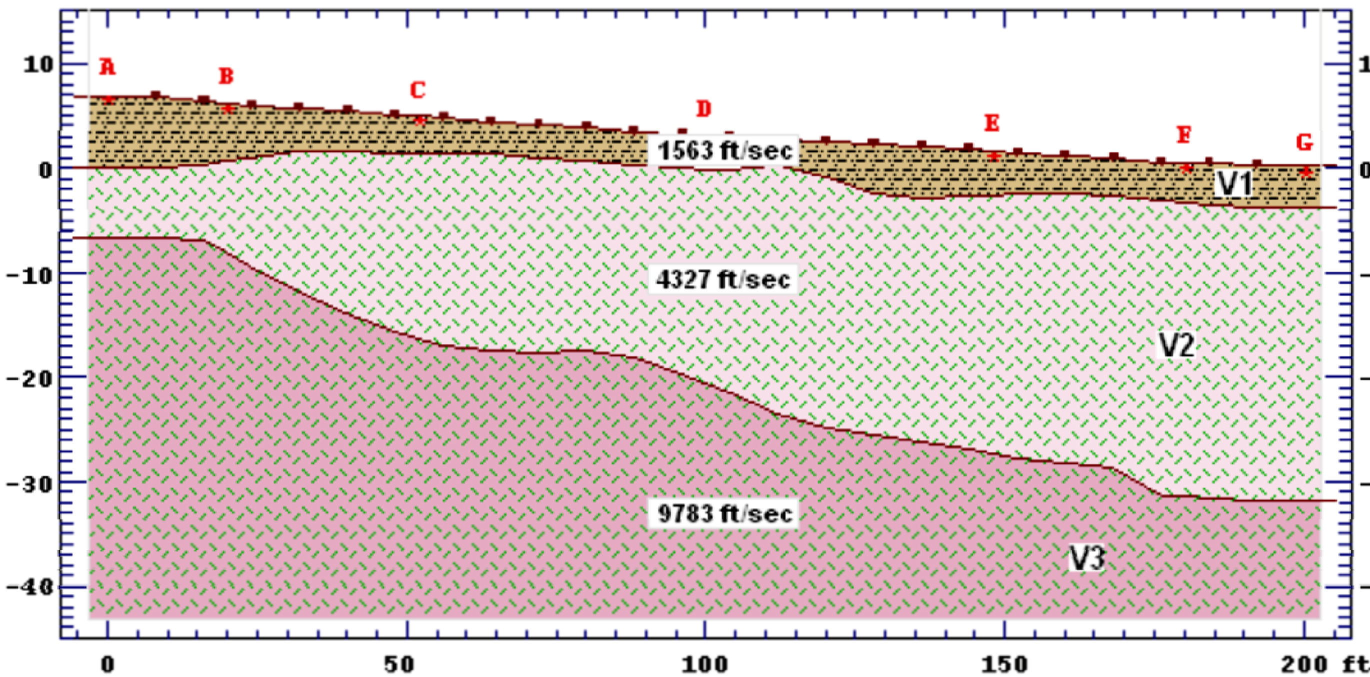
APPENDIX A

LAYER VELOCITY MODELS

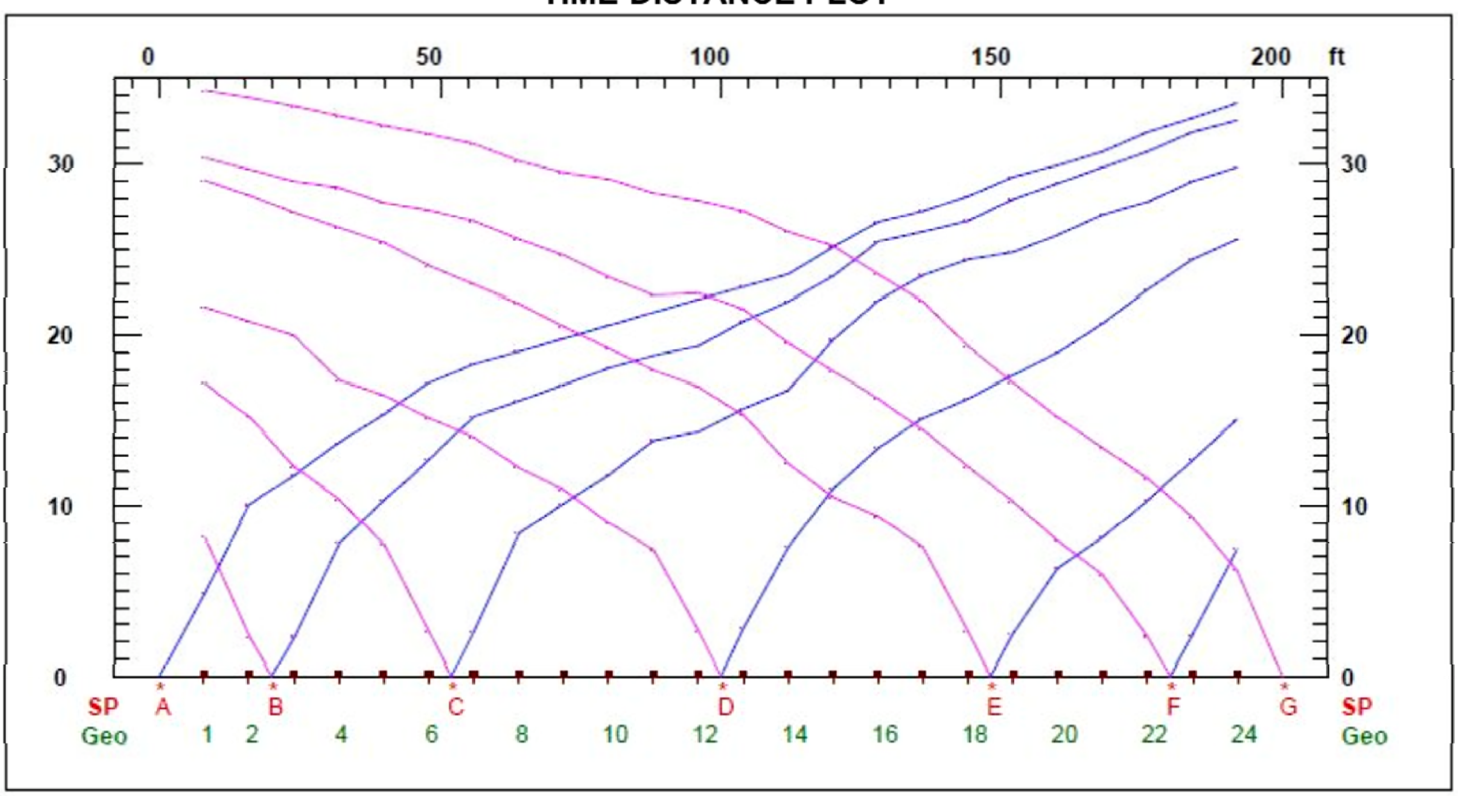
SEISMIC LINE S-8

South 45° East >

LAYER VELOCITY MODEL



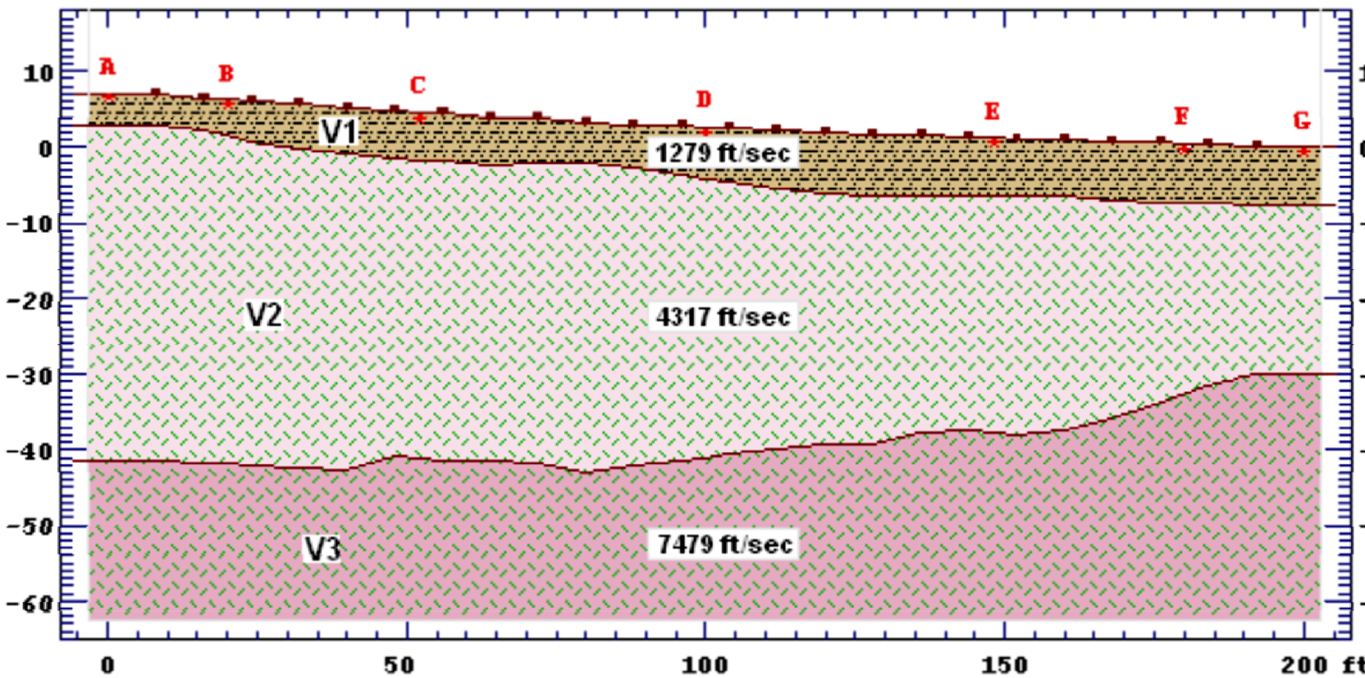
TIME-DISTANCE PLOT



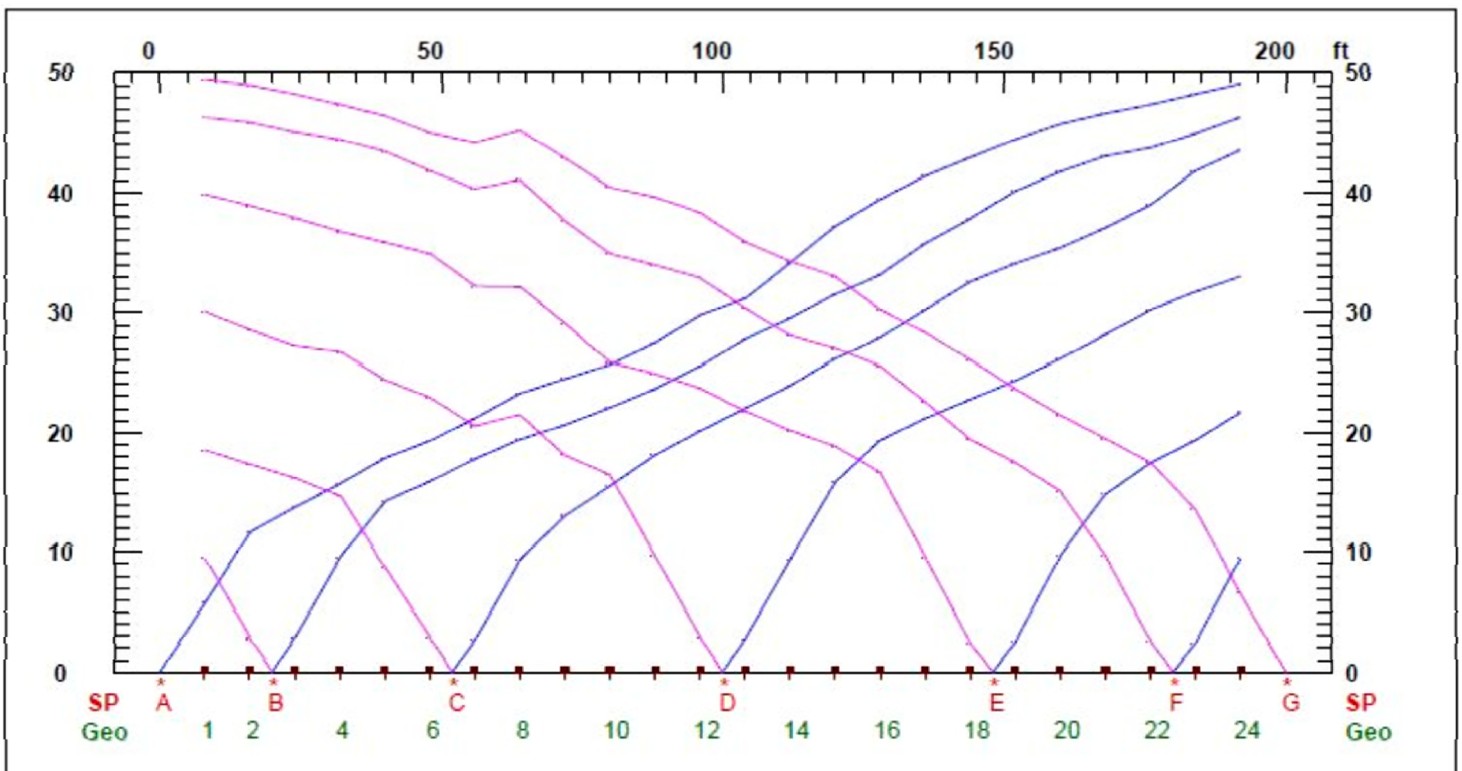
SEISMIC LINE S-9

South 45° East >

LAYER VELOCITY MODEL



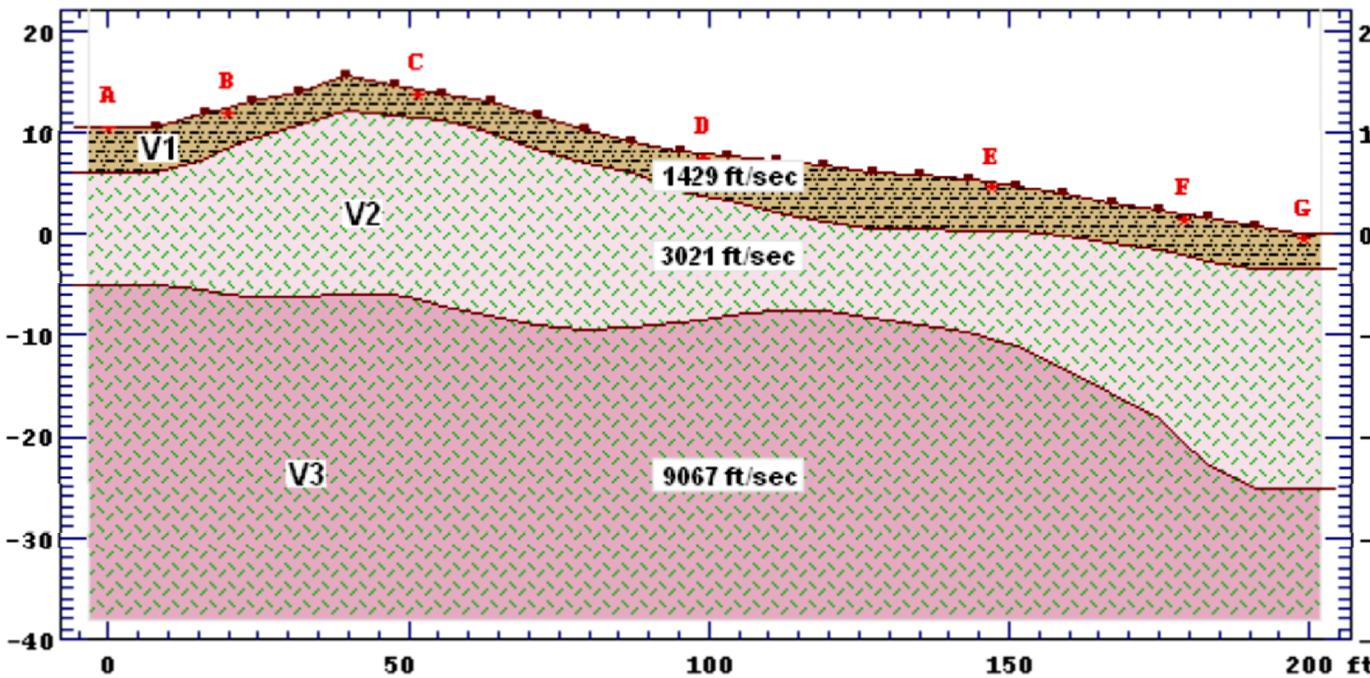
TIME-DISTANCE PLOT



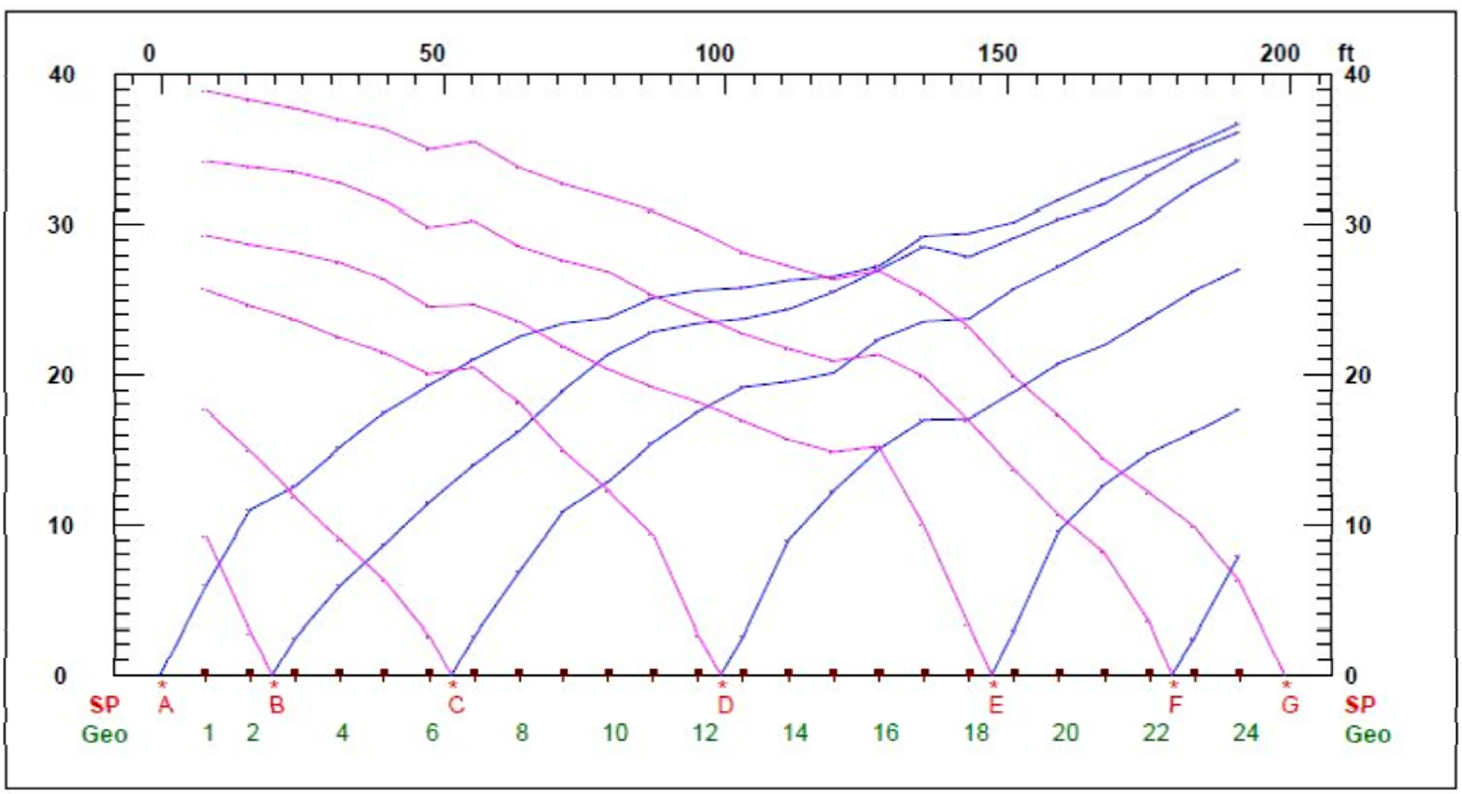
SEISMIC LINE S-10

South 45° East >

LAYER VELOCITY MODEL



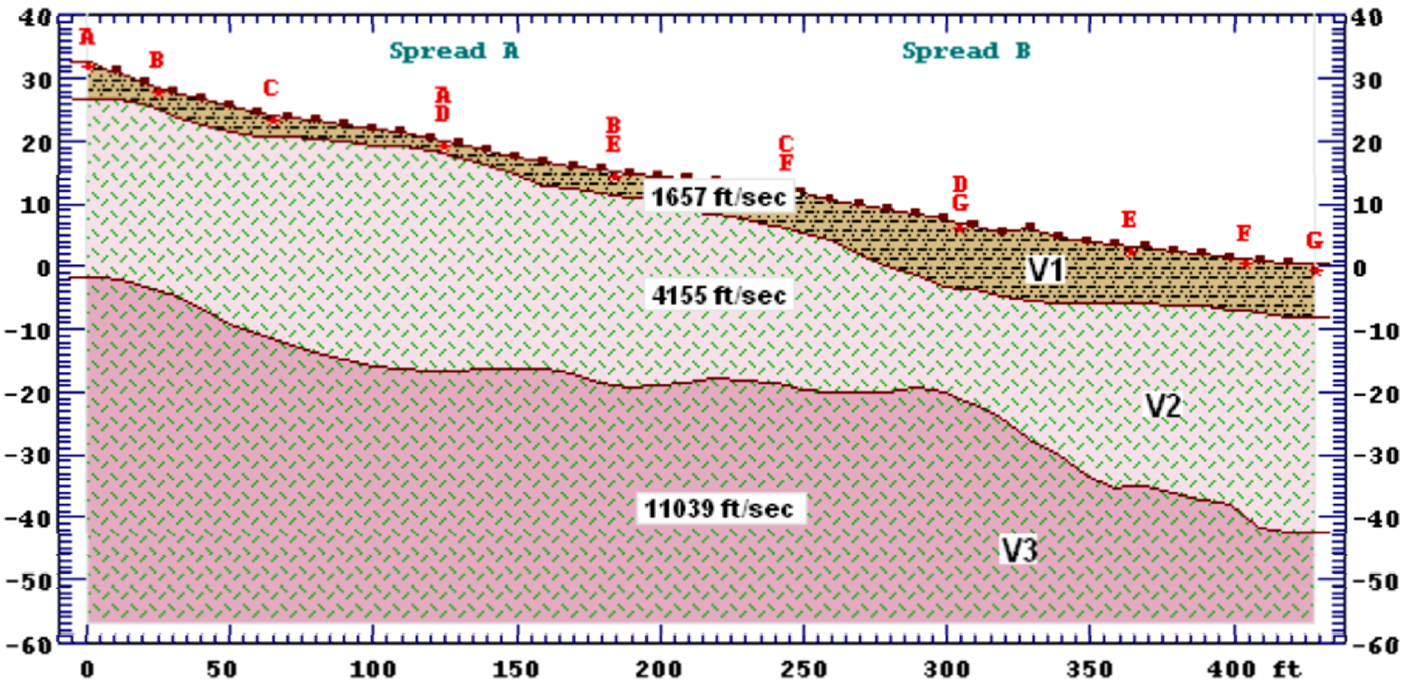
TIME-DISTANCE PLOT



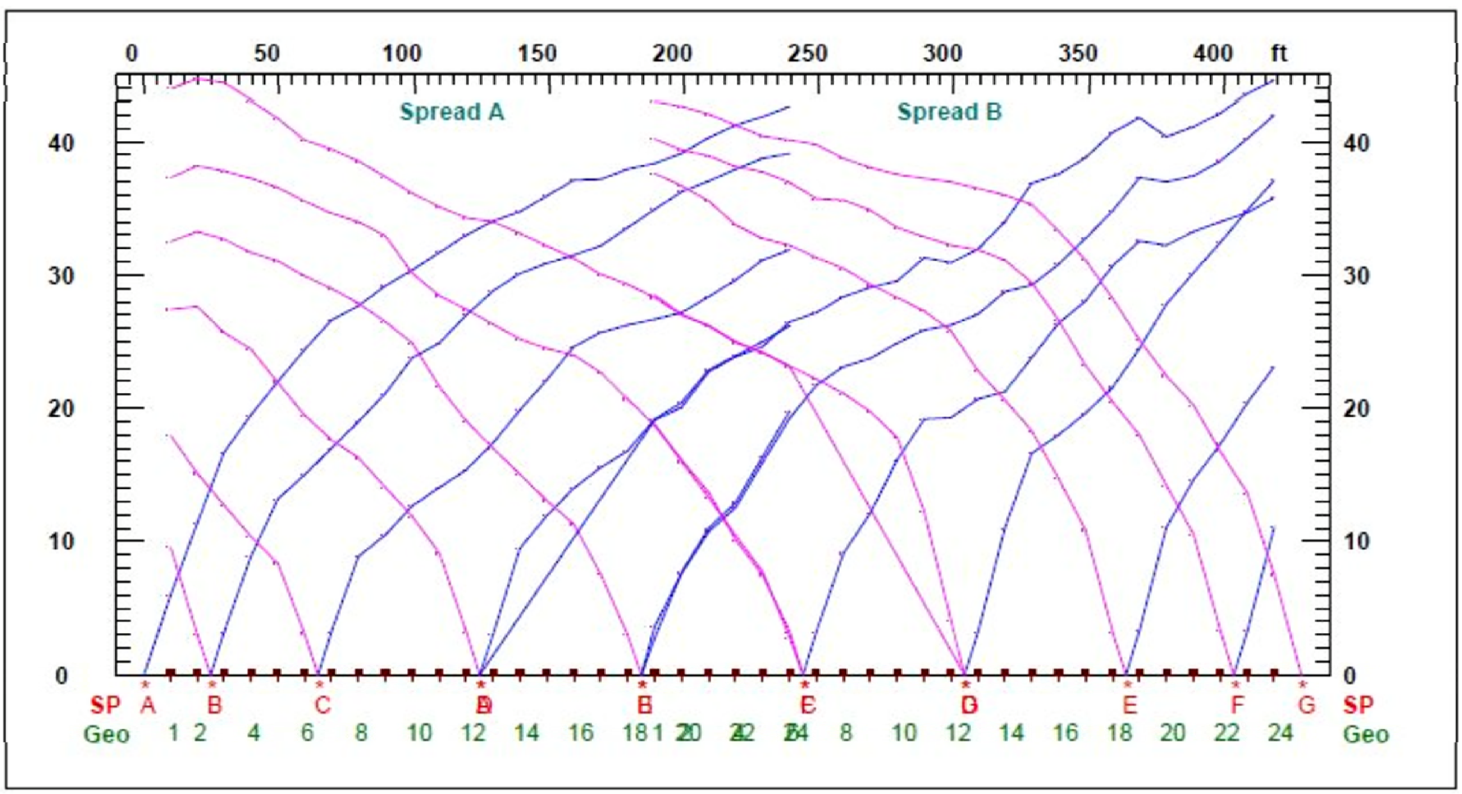
SEISMIC LINE S-11

South 80° East >

LAYER VELOCITY MODEL



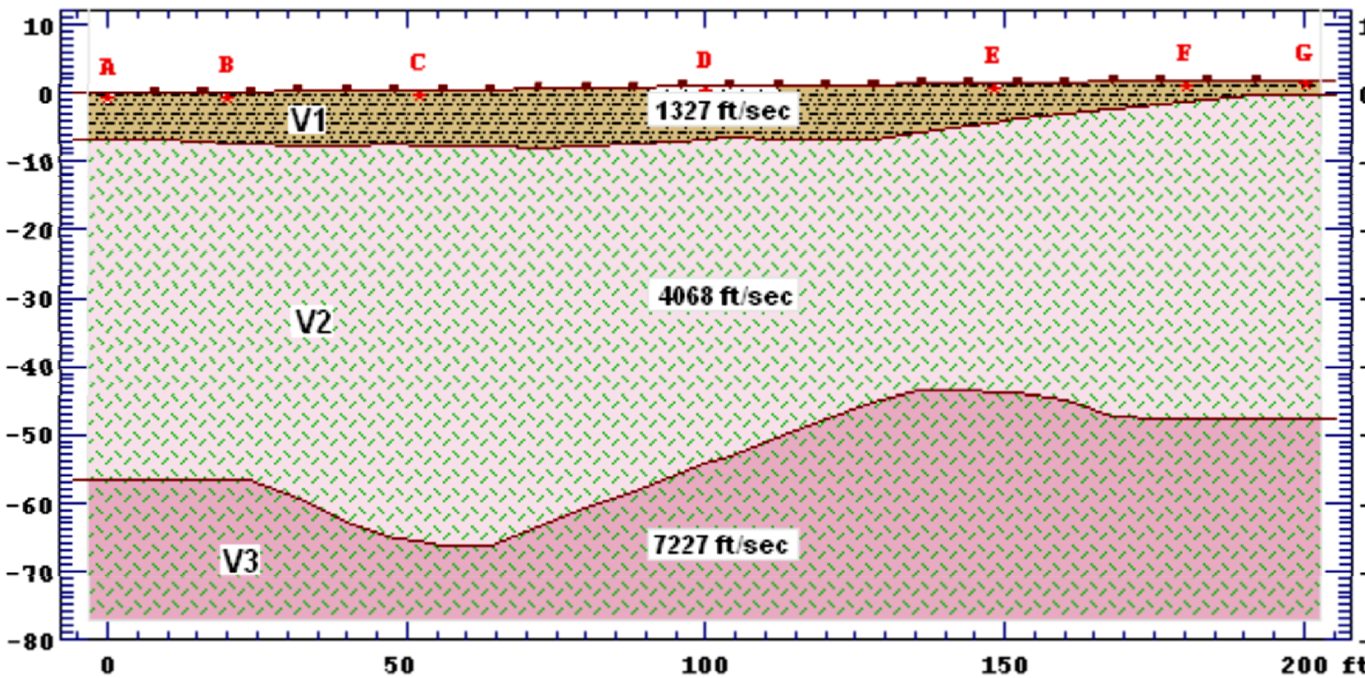
TIME-DISTANCE PLOT



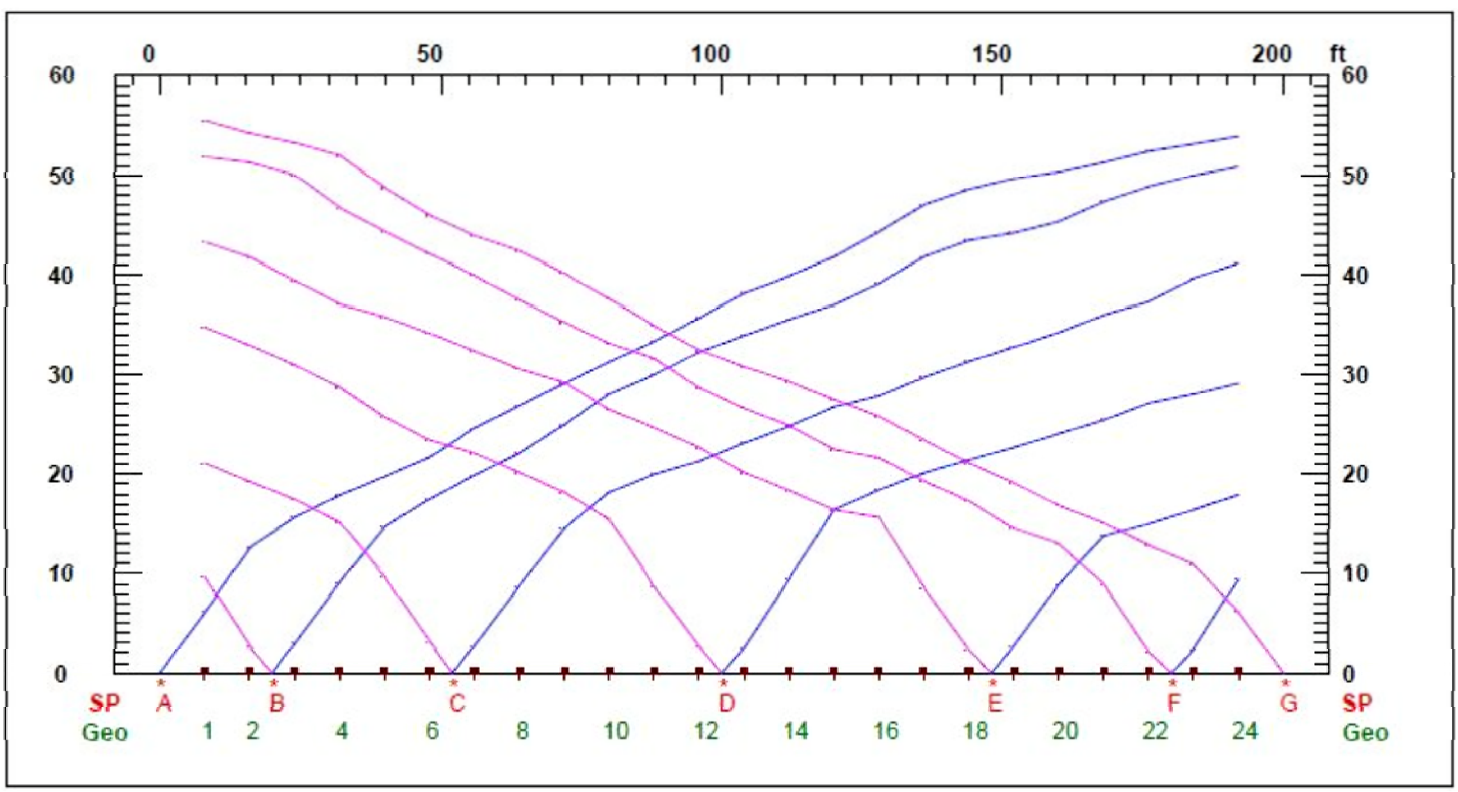
SEISMIC LINE S-12

South 45° East >

LAYER VELOCITY MODEL



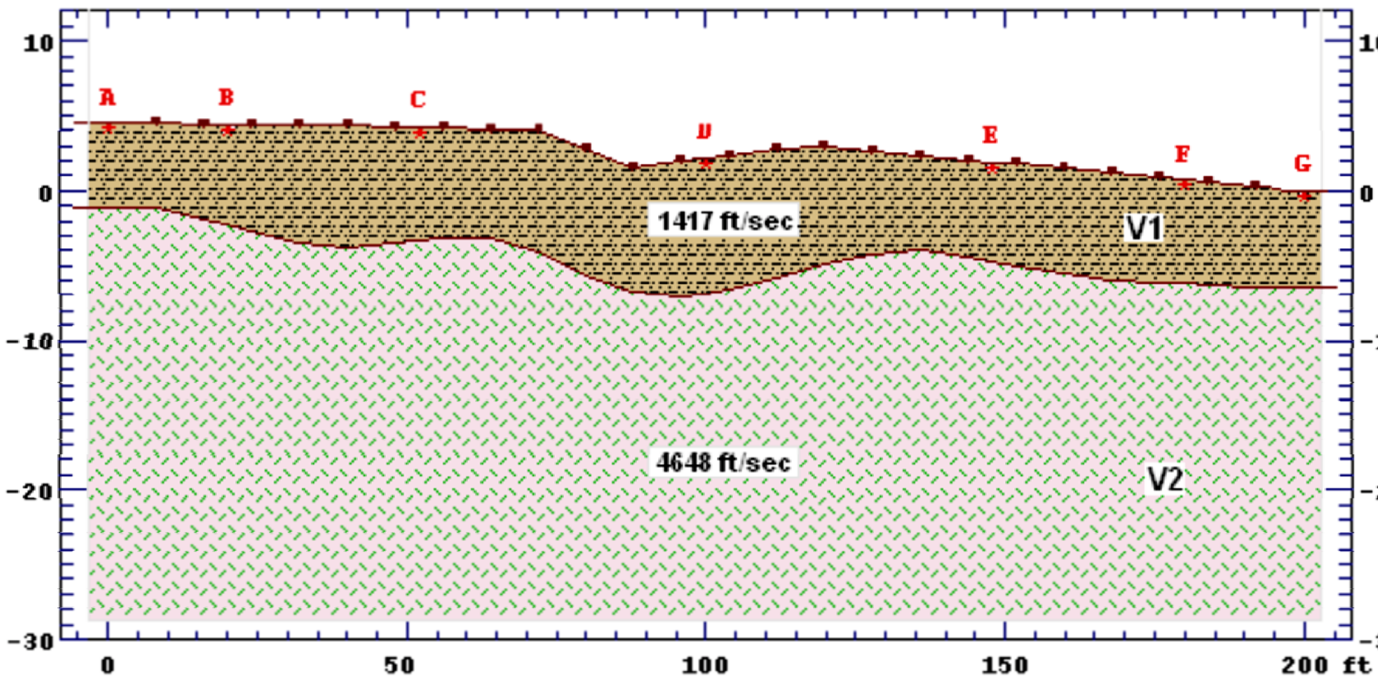
TIME-DISTANCE PLOT



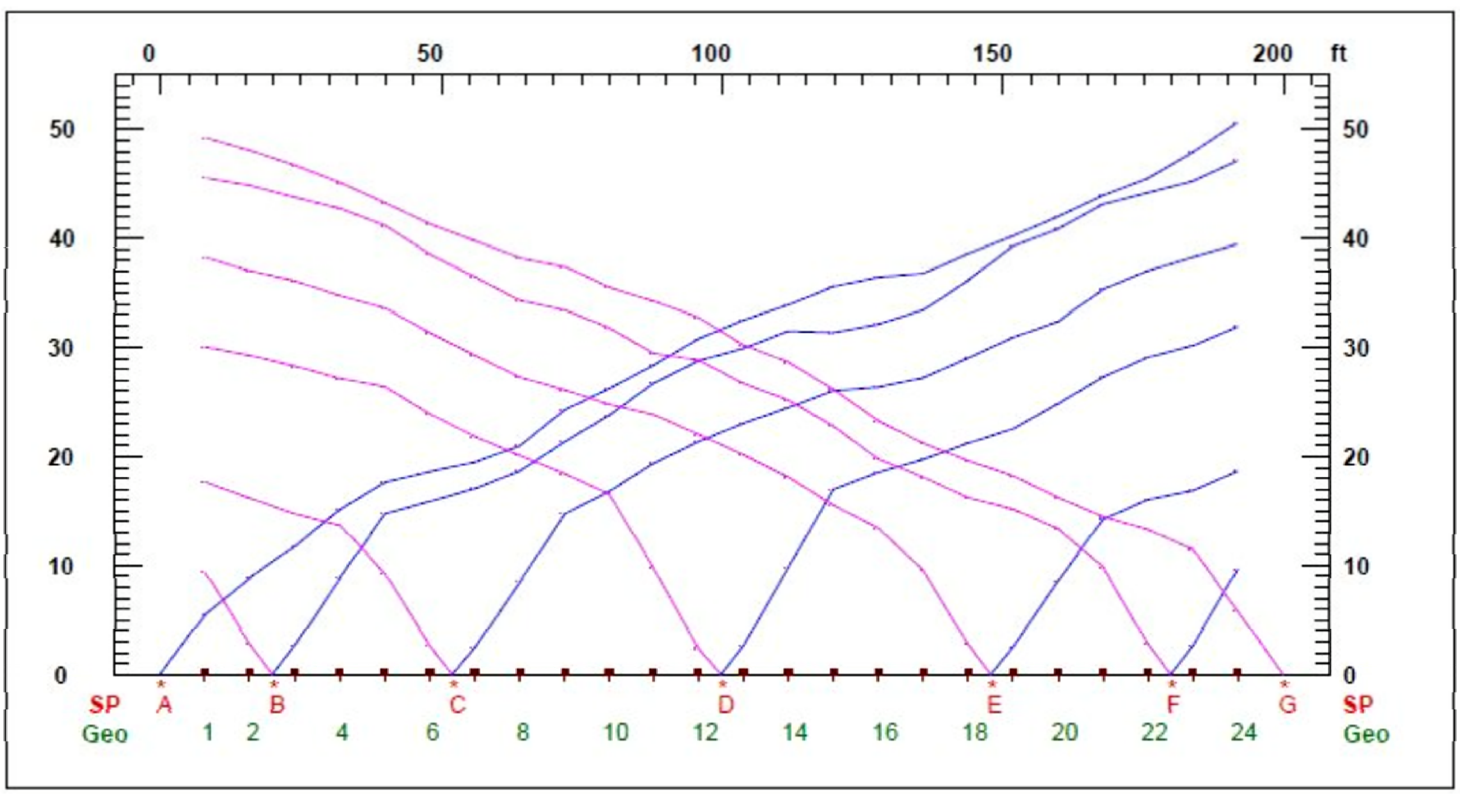
SEISMIC LINE S-13

South 45° East >

LAYER VELOCITY MODEL



TIME-DISTANCE PLOT



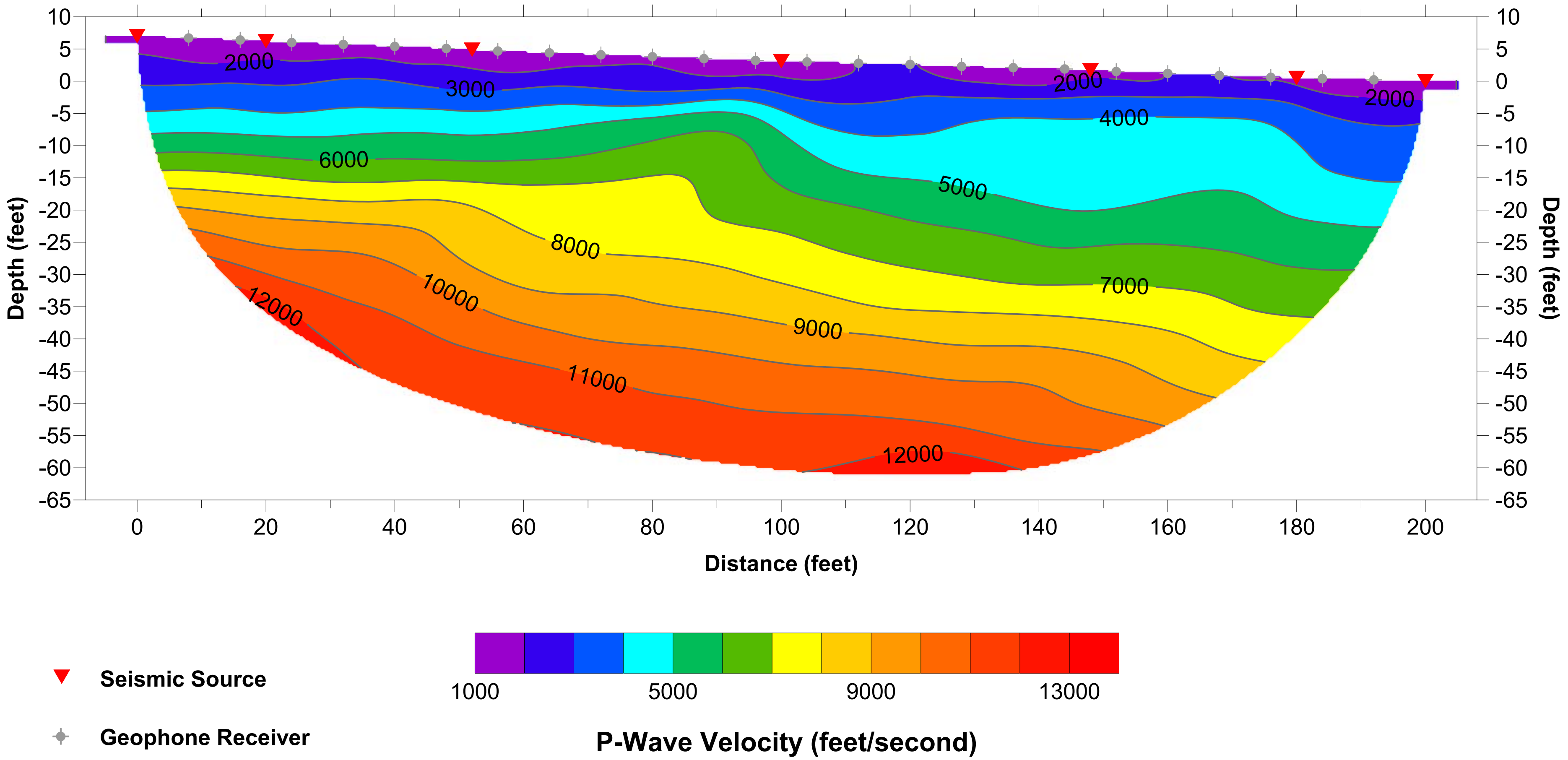
APPENDIX B

REFRACTION TOMOGRAPHIC MODELS

SEISMIC LINE S-8

South 45° East →

REFRACTION TOMOGRAPHIC MODEL



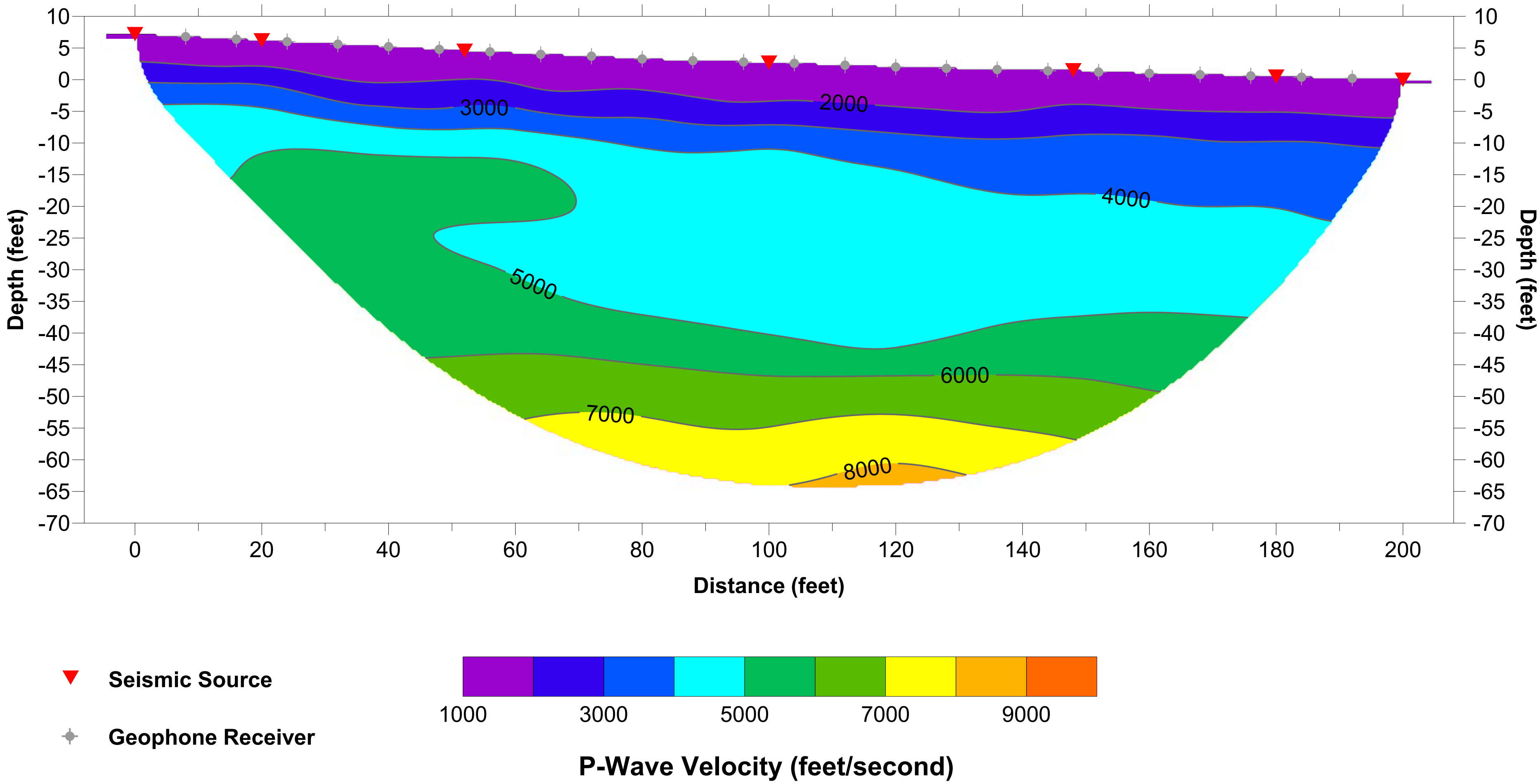
SCALE: 1" = 25' (Horizontal & Vertical)

RMS error 1.4 %, Rayfract Version 3.32

SEISMIC LINE S-9

South 45° East →

REFRACTION TOMOGRAPHIC MODEL



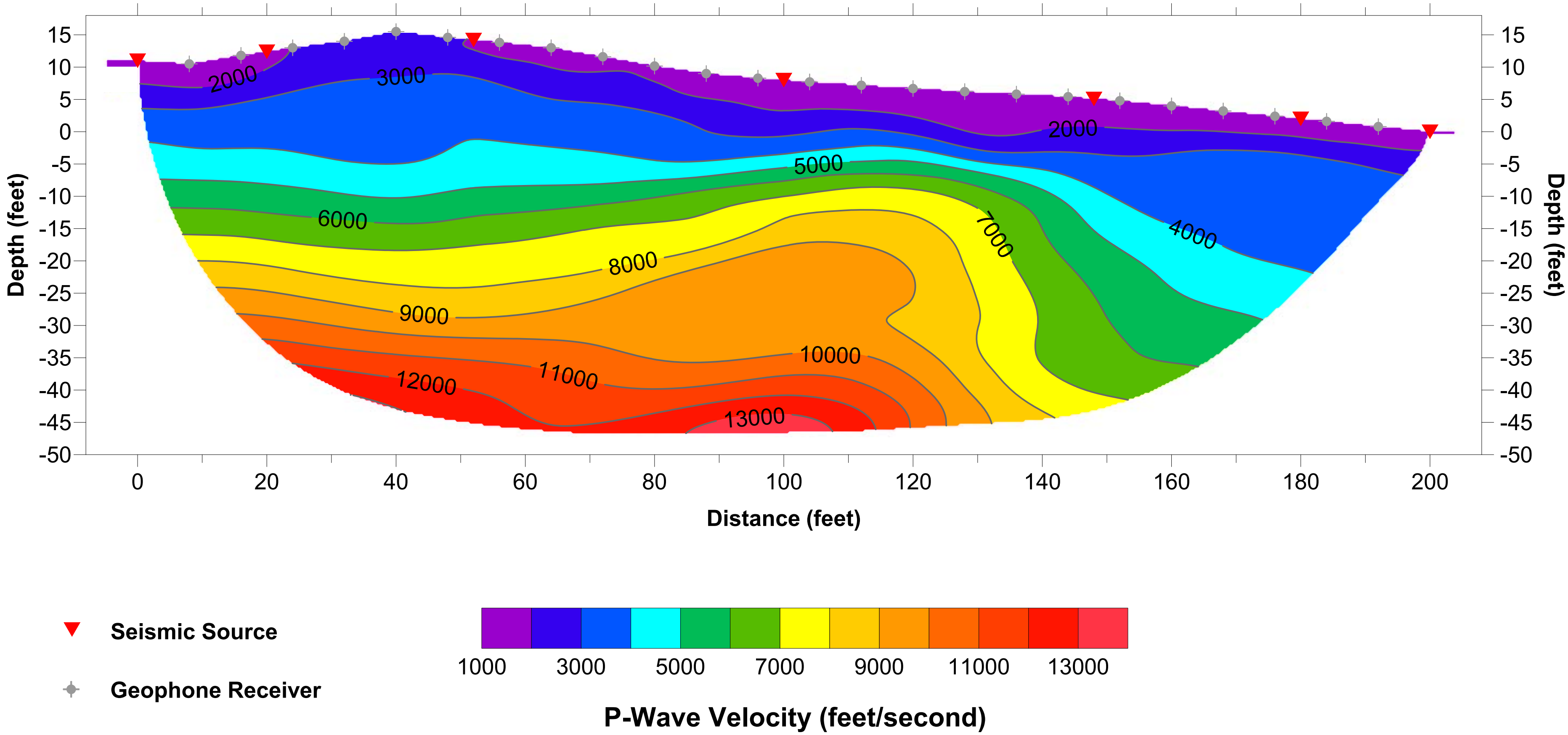
SCALE: 1" = 25' (Horizontal & Vertical)

RMS error 1.2 %, Rayfract Version 3.32

SEISMIC LINE S-10

South 45° East →

REFRACTION TOMOGRAPHIC MODEL



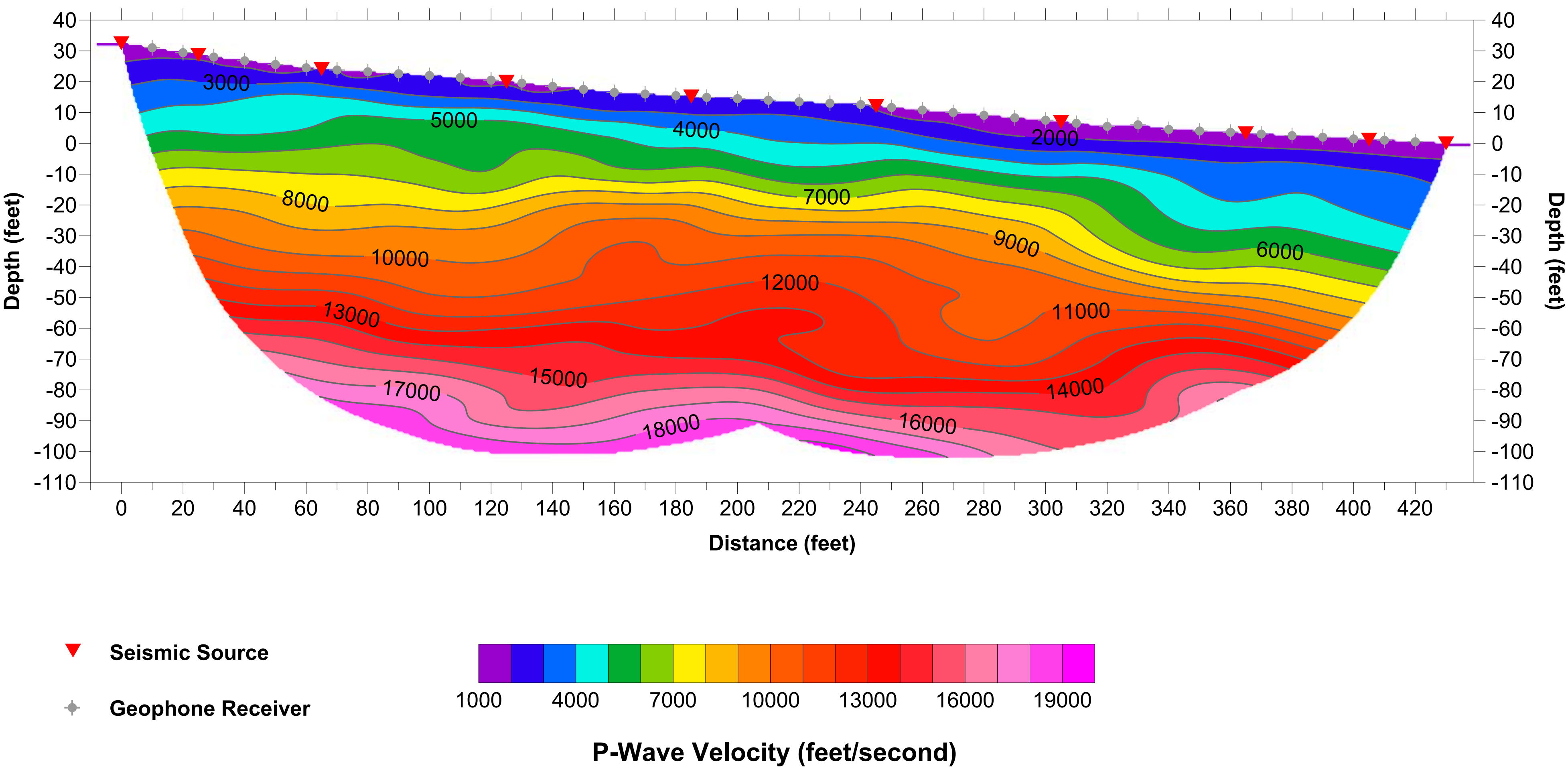
SCALE: 1" = 25' (Horizontal & Vertical)

RMS error 1.1 %, Rayfract Version 3.32

SEISMIC LINE S-11

South 80° East →

REFRACTION TOMOGRAPHIC MODEL



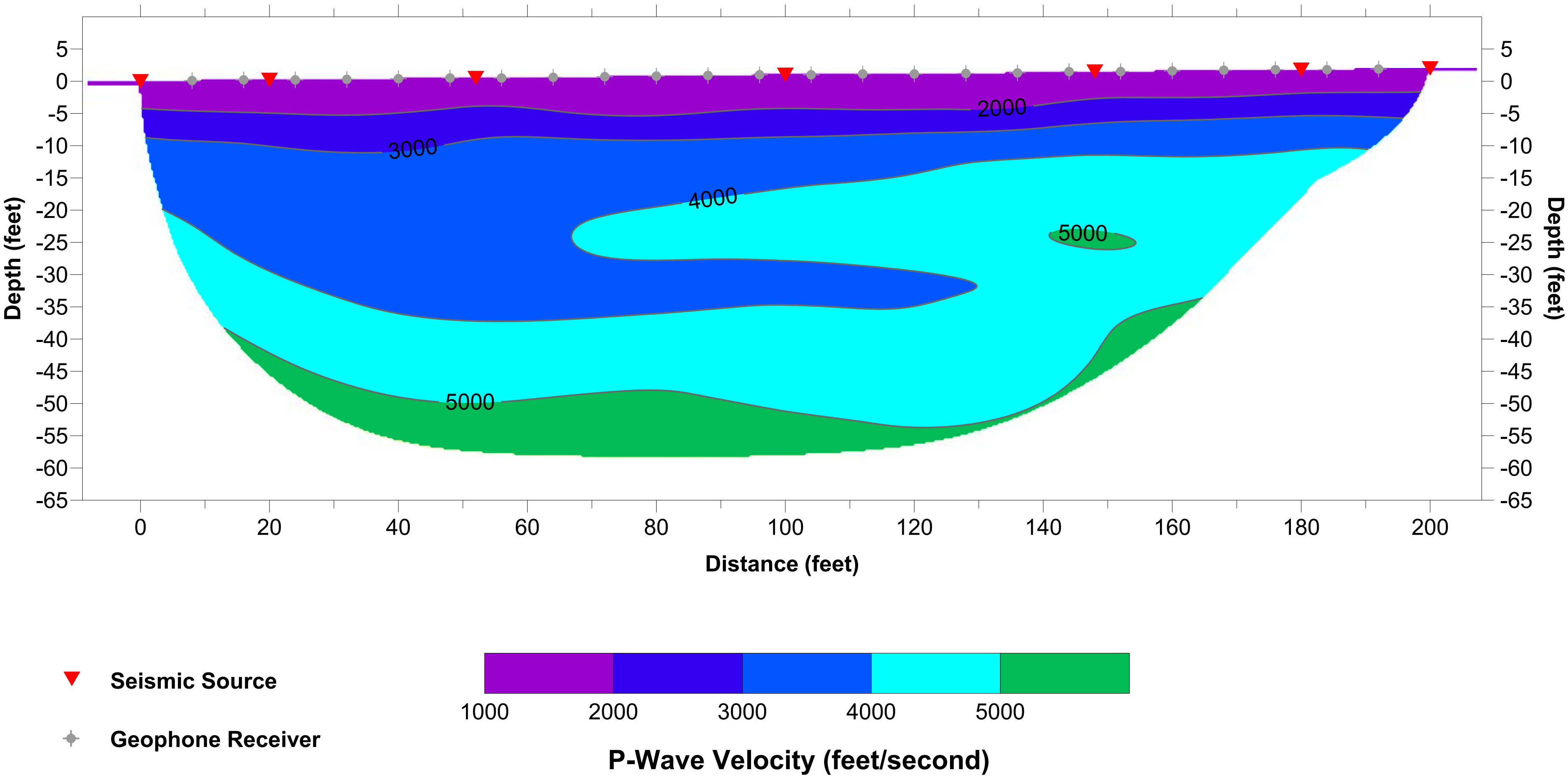
SCALE: 1" = 50' (Horizontal & Vertical)

RMS error 1.9 %, Rayfract Version 3.32

SEISMIC LINE S-12

South 45° East →

REFRACTION TOMOGRAPHIC MODEL



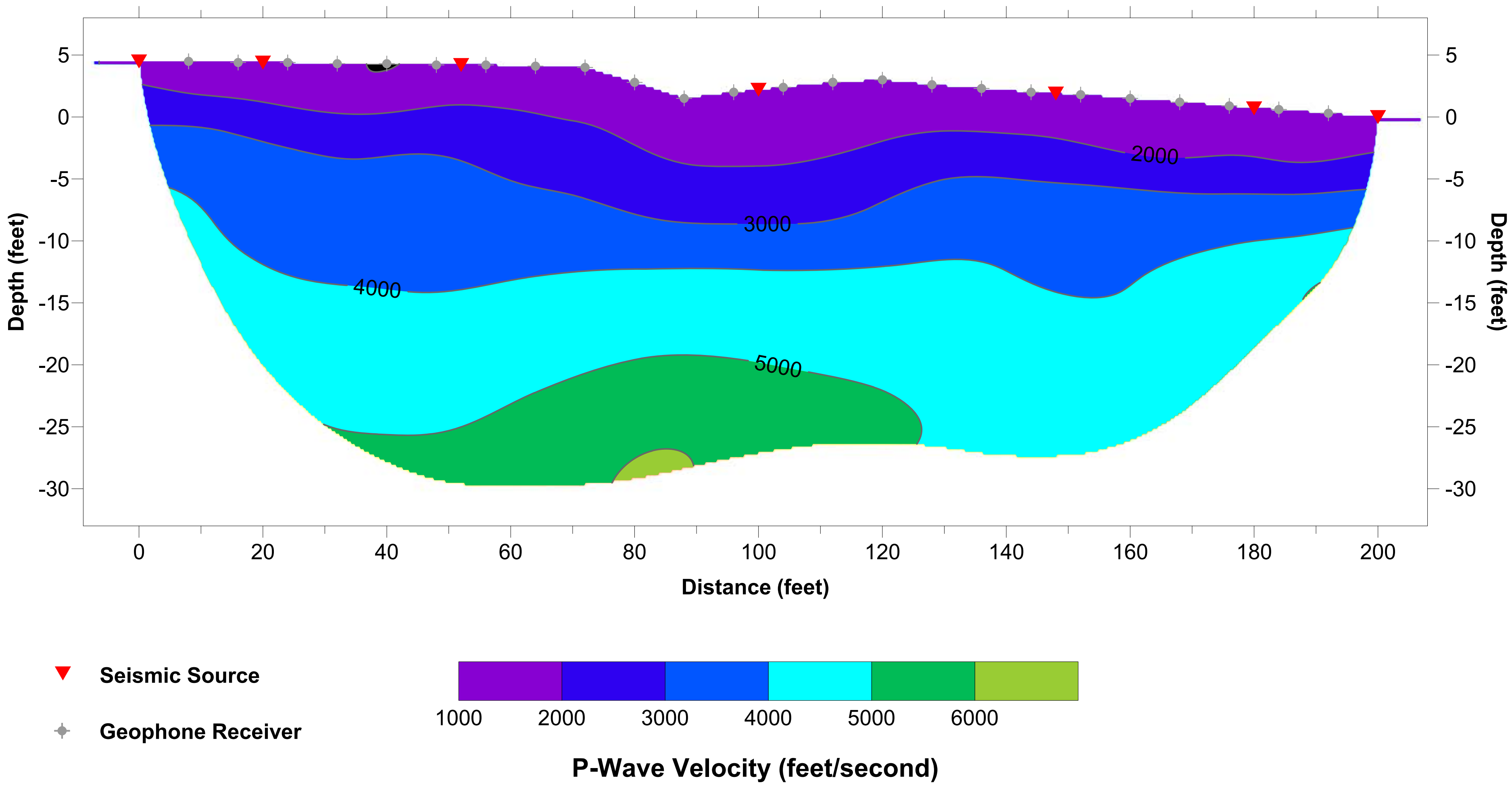
SCALE: 1" = 25' (Horizontal & Vertical)

RMS error 1.7 %, Rayfract Version 3.32

SEISMIC LINE S-13

South 45° East →

REFRACTION TOMOGRAPHIC MODEL



NOTE: Vertical Exaggeration 2X

RMS error 1.8 %, Rayfract Version 3.32

APPENDIX C

EXCAVATION CONSIDERATIONS

EXCAVATION CONSIDERATIONS

These excavation considerations have been included to provide the client with a brief overall summary of the general complexity of hard bedrock excavation. It is considered the clients responsibility to insure that the grading contractor they select is both properly licensed and qualified, with experience in hard-bedrock ripping processes. To evaluate whether a particular bedrock material can be ripped, this geophysical survey should be used in conjunction with the geologic or geotechnical report prepared for the project which describes the physical properties of the bedrock. The physical characteristics of bedrock materials that favor ripping generally include the presence of fractures, faults and other structural discontinuities, weathering effects, brittleness or crystalline structure, stratification of lamination, large grain size, moisture permeated clay, and low compressive strength. Unfavorable conditions can include such characteristics as massive and homogeneous formations, non-crystalline structure, absence of planes of weakness, fine-grained materials, and formations of clay origin where moisture makes the material plastic.

When assessing the potential rippability of the underlying bedrock of a given site, the above geologic characteristics along with the estimated seismic velocities can then be used to evaluate what type of equipment may be appropriate for the proposed grading. When selecting the proper ripping equipment there are three primary factors to consider, which are:

- ***Down Pressure available at the tip, which determines the ripper penetration that can be attained and maintained,***

- ***Tractor flywheel horsepower, which determines whether the tractor can advance the tip, and,***

- ***Tractor gross-weight, which determines whether the tractor will have sufficient traction to use the horsepower.***

In addition to selecting the appropriate tractor, selection of the proper ripper design is also important. There are basically three designs, being radial, parallelogram, and adjustable parallelogram, of which the contractor should be aware of when selecting the appropriate design to be used for the project. The penetration depth will depend upon the down-pressure and penetration angle, as well as the length of the shank tips (short, intermediate, and long).

Also important in the excavation process is the ripping technique used as well as the skill of the individual tractor operator. These techniques include the use of one or more ripping teeth, up- and down-hill ripping, and the direction of ripping with respect to the geologic structure of the bedrock locally. The use of two tractors (one to push the first tractor-ripper) can extend the range of materials that can be ripped. The second tractor can also be used to supply additional down-pressure on the ripper. Consideration of light blasting can also facilitate the ripper penetration and reduce the cost of moving highly consolidated rock formations.

All of the combined factors above should be considered by both the client and the grading contractor, to insure that the proper selection of equipment and ripping techniques are used for the proposed grading.

APPENDIX D

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REFERENCES

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<u>APPENDIX E</u>

GEOLOGIC LINEAMENT ANALYSIS

TERRA GEOSCIENCES

LINEAMENT EVALUATION

PROPOSED DECKER II PROJECT

SW CORNER OF DECKER ROAD AND OLEANDER AVENUE

PERRIS AREA, RIVERSIDE COUNTY, CALIFORNIA

Project No. 142740-3

January 25, 2015

Prepared for:

Matrix Geotechnical Consulting
41769 Enterprise Circle North
Suite 107
Temecula, California 92590

Consulting Engineering Geology & Geophysics

P.O. Box 1090, Loma Linda, CA 92354 • 909 796-4667

Matrix Geotechnical Consulting
41769 Enterprise Circle North, Suite 107
Temecula, California 92590

Attention: Mr. Chris Josef

Regarding: Lineament Evaluation
Proposed Decker II Project
SW Corner of Decker Road and Oleander Avenue
Perris Area, Riverside County, California
MGC Project No. M1103-008

INTRODUCTION

As requested, we have performed a Lineament Evaluation for the above-referenced site. The purpose for this study was to evaluate the nature and character of several prominent geomorphic linear features that have been identified to traverse through the subject site. Both photogeologic analysis and subsurface exploration were utilized to asses these features. A Photolineation Map has been prepared (see Plate 2) which identifies the project boundaries, topography, approximate locations of the exploratory trench excavations, and the location of the photographic lineaments observed during this study. This map was created from a captured Google™ Earth image (Google™ Earth, 2013), in turn overlain by the site topographic base map.
As authorized by you, the following services were performed during this study:

- **Review of available published and unpublished geologic data in our files pertinent to the site.**
- **Photogeologic analysis of seven stereographic pairs of aerial photographs obtained from the Riverside County Flood Control Department.**
- **Field geologic reconnaissance by a State of California Certified Engineering Geologist.**
- **Excavation and logging of three exploratory trenches across the linear geomorphic features that were identified during this study.**
- **Preparation of this report, presenting our findings, conclusions, and recommendations with respect to the lineaments observed.**

Accompanying Maps and Appendices

Plate 1 - Regional Geologic Map
Plate 2 - Photolineation Map
Appendix A - Exploratory Trench Logs
Appendix B - References

GEOMORPHIC SETTING

The subject site is situated within a natural geomorphic province in southwestern California known as the Peninsular Ranges, which is characterized by steep, elongated ranges and valleys that trend northwesterly. This province is believed to have begun as a thick accumulation of predominantly marine sedimentary and volcanic rocks during the late Paleozoic and early Mesozoic (pre-batholithic rocks). Following this accumulation, in mid-Cretaceous time, the province underwent a pronounced episode of mountain building. The accumulated rocks were then complexly metamorphosed and intruded by igneous rocks, known locally as the Peninsular Ranges Batholith. A period of erosion followed the mountain building, and during the late Cretaceous and Cenozoic time, sedimentary and subordinate volcanic rocks were deposited upon the eroded surfaces of the batholithic and pre-batholithic rocks (post-batholithic rocks). Most of these post-batholithic rocks occur along the western and northern potion of the province.

More specifically, the site is situated along the Perris Block, an eroded mass of Cretaceous and older crystalline rock. Thin sedimentary and volcanic units mantle the bedrock in a few places with alluvial deposits filling in the lower valley areas. The Perris Block, approximately 20 miles by 50 miles in extent, is bounded by the San Jacinto Fault Zone to the northeast, the Elsinore Fault Zone to the southwest, the Cucamonga Fault Zone to the northwest, and to the southeast by the fringes of the Temecula basin, where the boundary is ill-defined. The Perris Block in its entirety, is probably bounded everywhere by fault zones and has been repeatably uplifted and occasionally depressed since the beginning of Pliocene time (5.3± million years before present). These episodic movements have undoubtedly led to internal fracturing, shearing, and faulting, that are discontinuous and sporadic.

The Perris Block has had a complex history, apparently undergoing relative vertical land movements of several thousand feet in response to movement on the Elsinore and San Jacinto Fault Zones. These movements of the geologic past, in conjunction with the semi-arid climate and the weathering resistance of the rock, are responsible for the formation and preservation of ancient, generally flat-lying erosion surfaces now present at various elevations that give this region its unique geologic character, of which there are six recognized surfaces.

Of these geomorphic surfaces, the subject property appears to be located just within the easternmost fringe of the Perris Surface, as approximated on Figure 1 below. The Perris Surface, which developed sometime during the later Pliocene, is for the most part, a somewhat undulating erosional surface generally found between elevations of 1,600 to 1,800 feet above mean sea level. The 1,600 foot elevation is generally coincident with the eastern property boundary, which approximates the boundary between the Perris Surface to the west, and the Paloma Surface to the east. The Paloma Surface to the east is relatively flat lying and generally lies between elevations of 1,400 and 1,600 feet above mean sea level.

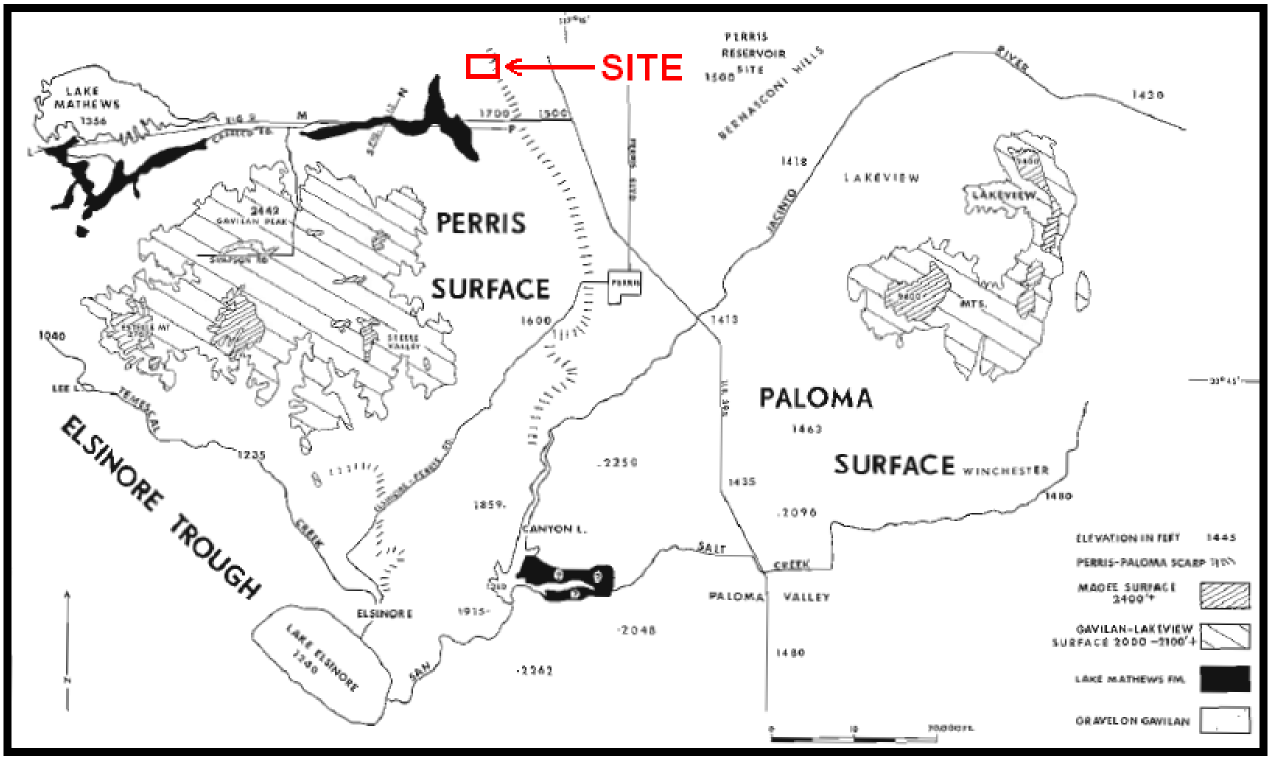


FIGURE 1- Geomorphic map of the central portion of the Perris Block (Woodford et al., 1971)

LOCAL GEOLOGY

As shown on the Geologic Site Map (see Plate 1), the underlying earth materials have been mapped (Morton, 2001) to consist of Cretaceous age granitic rocks, locally referred to as the Val Verde Tonalite. These rocks originated from the Val Verde Pluton, one of many such plutons that were emplaced into the northern part of the Peninsular Ranges Batholith. The Val Verde Pluton is relatively uniform in composition, which is predominantly comprised of biotite-hornblende tonalite, estimated to have been emplaced approximately 105.7 million years before present (Morton and Miller, 2014). The Val Verde Tonalite has been described by Morton and Miller (2014) to generally consist of gray-weathering, relatively homogeneous, massive to well-foliated, medium- to coarse-grained, hypautomorphic-granular biotite-hornblende tonalite.

Geographically, the site is located within the central portion of the Val Verde Pluton, where the tonalite is mostly massive, and contains few segregational masses of mesocratic to melanocratic tonalite. Some foliated rock can be locally observed, which for the most part, has a northwest-southeast trending structural orientation that is parallel to the regional structural grain of the batholith.

Additionally, it is also possible that very old alluvial fan deposits (early Pleistocene age) comprised of well-indurated sand deposits, may be found locally mantling portions of the site, such as mapped just east of the site.

AERIAL PHOTOGRAPHIC REVIEW SUMMARY

A detailed review of pertinent stereoscopic aerial photographs was performed for this study for the purposes of evaluating the geomorphology of the site, specifically for the presence of any photogeologic features (i.e. fractures, joints, dikes, faults, etc.) that may traverse through the subject property. Seven sets of photographs at various scales, were reviewed between the years 1962 to 2005 (see references in Appendix B for a listing), that were obtained from the Riverside County Flood Control Department. In addition, the historical imagery database of Google Earth (Google™ Earth, 2013) was also utilized.

Review of these photographs revealed several distinct linear features (lineations) that traverse through the subject property, which are depicted on the accompanying Photolineation Map (see Plate 2). The most prevalent lineament traverses along a north-south direction within the western portion of the site. This feature is expressed as linear topography that forms a fairly sharp geomorphic boundary delineated by a scarp, with the land being higher to the west. Additionally, this feature also forms a distinctive tonal vegetation lineament, with the presence of several large trees. Additionally, two smaller, discontinuous linear features were also observed within the southern central portion of the site which was also identified by tonal vegetation lineaments. It was noted that these lineations varied in expression, being more pronounced during various seasons and years, most likely due to periods of relatively wetter weather.

Other than the three delineated lineaments as presented on Plate 2, no other photogeologic features were observed to traverse through site based on the aerial photographs reviewed. To further evaluate the nature and character of these lineations with respect to potential impacts on the proposed construction, subsurface exploration was deemed necessary, of which is described in further detail below.

SUBSURFACE EXPLORATION

Three exploratory trenches, ranging from 60 to 64 feet in length, were excavated to depths of up to 14± feet. Exploratory Trenches ET-1 and ET-2 were excavated in an east-west direction and Exploratory Trench ET-3 was excavated in a northeast-southwest direction. These orientations were maintained in an attempt to place the trenches in a near perpendicular direction to the observable lineation trends to provide a proper undistorted perspective of the subsurface structural features (Hathaway and Leighton, 1979). Graphic logs of these exploratory trenches are provided within Appendix A, which were prepared at a scale of one inch equals four feet (horizontal and vertical) that depict the structure and lithologic nature of the earth materials encountered locally. The earth materials that were encountered within these exploratory trenches consisted of an overlying mantle of unconsolidated Holocene age younger alluvium/colluvium which is comprised of fine- to coarse-grained silty sand that has massive soil structure and is loose to moderately loose.

Partially underlying the younger alluvium along local areas is Pleistocene age older alluvium, generally comprised of fine- to medium-grained clayey silty sands which have a blocky soil structure and are slightly- to moderately-indurated. Underlying the younger and older alluvial deposits at depth locally where explored is highly weathered and decomposed coarse-grained granitic bedrock, which is tonalitic in composition as previously discussed.

Within both Exploratory Trenches ET-1 and ET-2, a very well-defined fracture structure zone was observed that is coincident with the photolineation as mapped on Plate 2. This zone ranged locally from 10 to 12 feet in width as is composed of highly sheared bedrock with whitish to greenish colored clayey gouge materials that line the shears up to a few inches thick. Portions of the internal bedrock within the shear zone are partially crushed creating a fine-grained granitic matrix. The shear zone was measured to traverse in a general North 6° West direction, of which the surficial lineation also trends. The clayey gouge materials within this shear zone have created a physical impedance boundary to transient groundwater wherein water was seeping from the bedrock along the west side of the shear zone, as high up as five to six feet from the surface. Beyond the shear zone in both directions the bedrock is only slightly fractured, generally along random orientations.

Within Exploratory Trench ET-1, there was a very distinctive older alluvial unit mantling the shear zone that had a very sharp and well-defined planar contact that was continuous and unbroken. These sediments appear to be at least late to middle Pleistocene in age based on the block soil structure, illuviated clay along the ped faces, and the indurated nature of the sediments. Overlying the shear zone within Exploratory Trench ET-2, only relatively younger alluvial deposits were observed. This contact was moderately sharp and undulating, but was unbroken and the shearing did not enter into the overlying alluvial materials.

It was determined that this fractured zone is a fault that has demonstrated past movement based on the fine-grained shearing of the bedrock and accumulation of clayey gouge. This fault has most likely ruptured during the episodic movements of the Perris Block since Cretaceous time, but does not show any indications of recent fault activity, which is defined as surface ground rupture within the past 11,000 years before present (Bryant and Hart, 2007).

Exploratory Trench ET-3 revealed a moderately well-defined joint/fracture zone within the bedrock being only a couple feet wide wherein the fracture planes are clay and caliche-lined, less than ¼-inch thick. The contact with the overlying alluvial sediments was noted to be fairly sharp and unbroken. This fracture structure was determined not to be associated with faulting but rather a joint zone within the regional structural trend that strikes to the northwest.

A more detailed description of the subsurface earth materials and structure for each exploratory trench is provided within the Exploratory Trench Logs (see Appendix A).

CONCLUSIONS AND RECOMMENDATIONS

CONCLUSIONS:

Based on review of published geologic data, field reconnaissance, photogeologic analysis, and our subsurface exploration, the continuous north-south photolineation located along the west appears to be related to faulting, and was assessed to be not active by definition (surface ground rupture within the last 11,000± years). This conclusion is supported by the apparent undisturbed and unbroken overlying Quaternary age alluvial sediments which were not ruptured by the fault. It is believed that this is an ancient fault associated with internal fracturing and shearing of the Perris Block during the episodic movements during Pliocene time. The highly sheared fault zone that consists of abundant clay gouge acts as an impedance barrier to the subsurface movement of groundwater, which has been trapped along the topographically higher western side of the fault zone, where the groundwater appears to migrate from west to east direction. This barrier has created a vegetation lineament that is vividly expressed on the aerial photographs, more so during periods of wetter seasons.

Geologic mapping by Rodgers (1966), Ziony and Jones (1989), Greenwood and Morton (1991), Jennings (1994), Morton (2001), Morton and Miller (2006), Bryant and Hart (2007), Jennings and Bryant (2010), and California Geological Survey (2010), do not indicate the presence of this fault or any others in the near vicinity. No other fault related features or photolineations were observed during this study. The linear structure that was encountered within Exploratory Trench ET-3 appears to be related to fracturing and/or jointing and therefore, the northwest-southeast trending lineations within the southern central portion of the site do not appear to be fault related. No shearing or evidence of previous ground movement was observed.

RECOMMENDATIONS:

Although the fault zone encountered along the western portion of the site is not “active” by definition, it is our opinion that a “Restricted Use Zone” should be created in order to completely mitigate any potential damage that could occur from hazards relating to differential settlement, shrinking/expansion along the clay-rich shear zone, secondary sympathetic movement associated with any potential nearby large future earthquakes, or any other such related hazards. No structures for human occupancy (2,000 person hours per year, or as defined by local agencies) should be constructed within this “Restricted-Use Zone” associated with the north-south trending fault zone encountered along the western portion of the site. A building setback line should be established by measuring 15 feet in both the east and west directions from the center of the fault zone (as defined by the staked surveyed fault locations for trenches ET-1 and ET-2), that is parallel to the fault zone, which will create a 30-foot wide “Restricted-Use Zone.” No building setbacks are necessary associated with the lineations along the southern central portion of the site.

CLOSURE

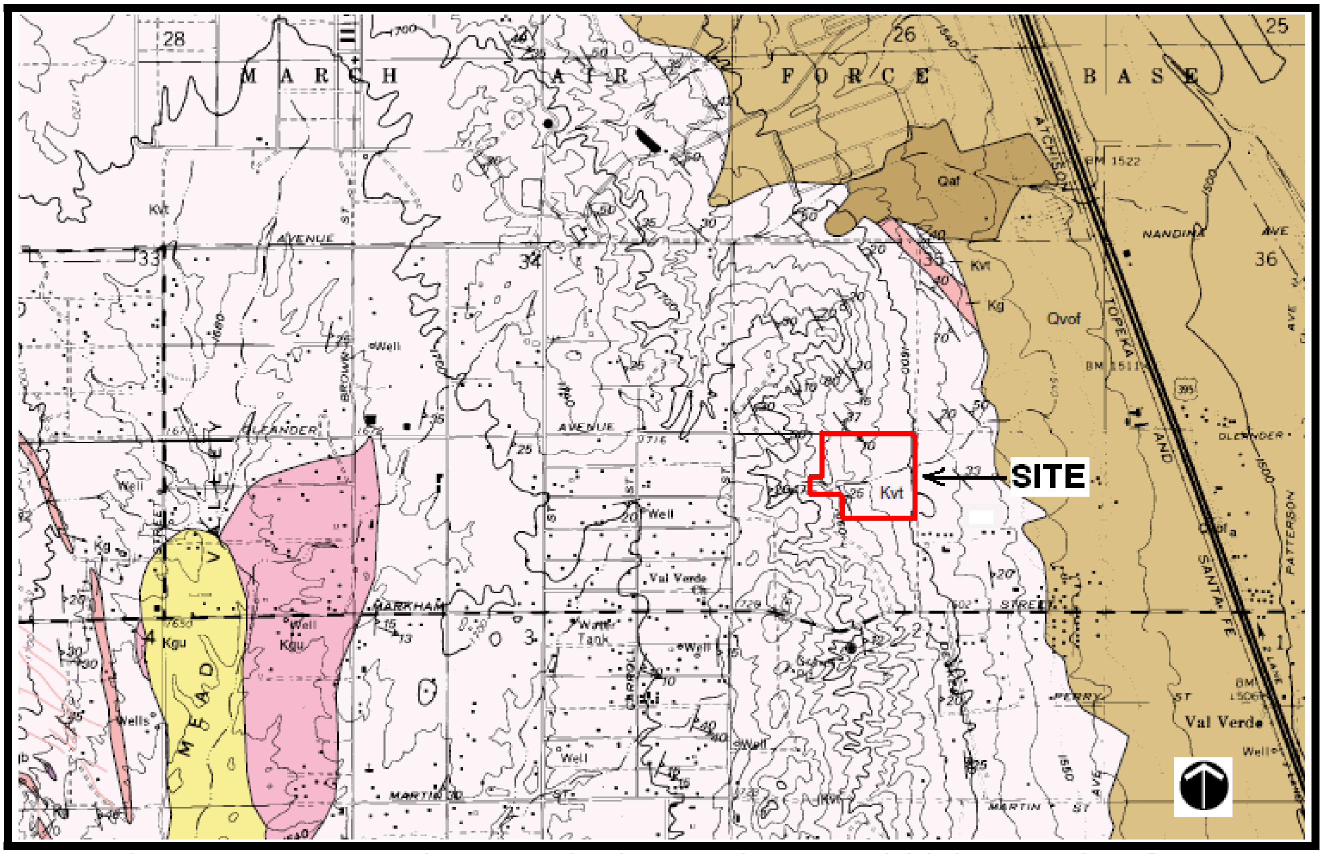
Our conclusions and recommendations are based on a surficial field reconnaissance, limited subsurface exploration, photogeologic analysis, and an interpretation of available geologic data. We make no warranty, either express or implied. Should conditions be encountered at a later date or more information becomes available that appear to be different than those indicated in this report, we reserve the right to reevaluate our conclusions and recommendations and provide appropriate mitigation measures, if warranted.

This opportunity to be of service is sincerely appreciated. If you should have any questions regarding this report or do not understand the limitations of this study or the data and results that are presented, please do not hesitate to contact our office at your earliest convenience.

Respectfully submitted,
TERRA GEOSCIENCES

Donn C. Schwartzkopf
Certified Engineering Geologist
CEG 1459

REGIONAL GEOLOGIC MAP

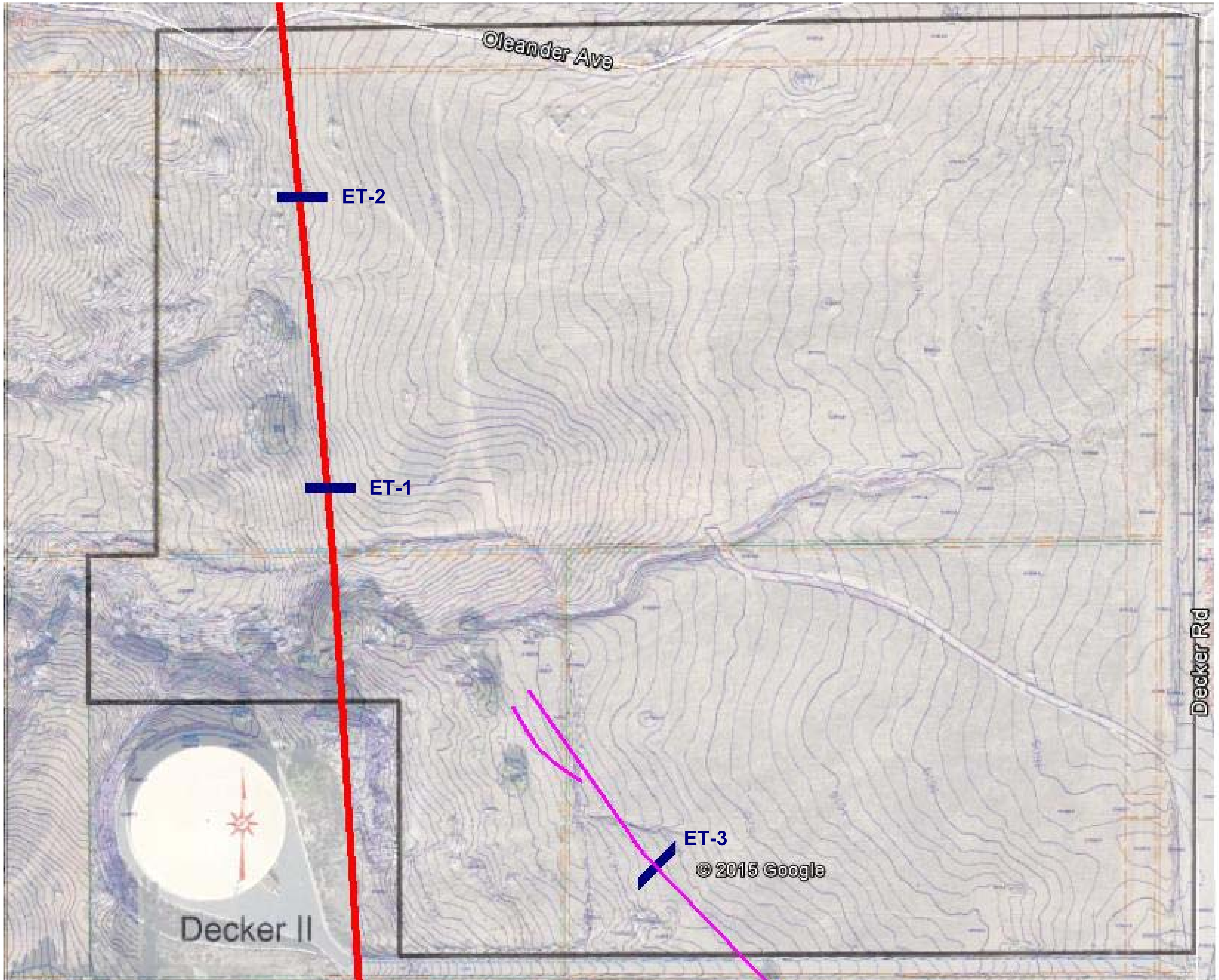


BASE MAP: Morton, D.M., 2001, Geologic Map of the Steele Peak Quadrangle, U.S.G.S. OFR 01-449, Scale 1”=2,750’

PARTIAL LEGEND

Qvof	**OLD FAN DEPOSITS**	Well indurated and dissected, reddish-brown sand deposits (early Pleistocene).
Kvt	**VAL VERDE TONALITE**	Homogeneous, massive-well foliated, medium-coarse grained, gray weathered (Cretaceous).
Kgu	**GRANITE**	Leucocratic fine-coarse grained massive granite and biotite monzogranite (Cretaceous).
Kg	**GRANITIC DIKES**	Leucocratic granitic dikes composed mainly of quartz and alkali (Cretaceous).
?	**GEOLOGIC CONTACT**	Generally located within 15±-meters.
50	**FOLIATION ATTITUDE**	Strike and dip of foliation

PHOTOLINEATION MAP



LEGEND

LINEATION / JOINT

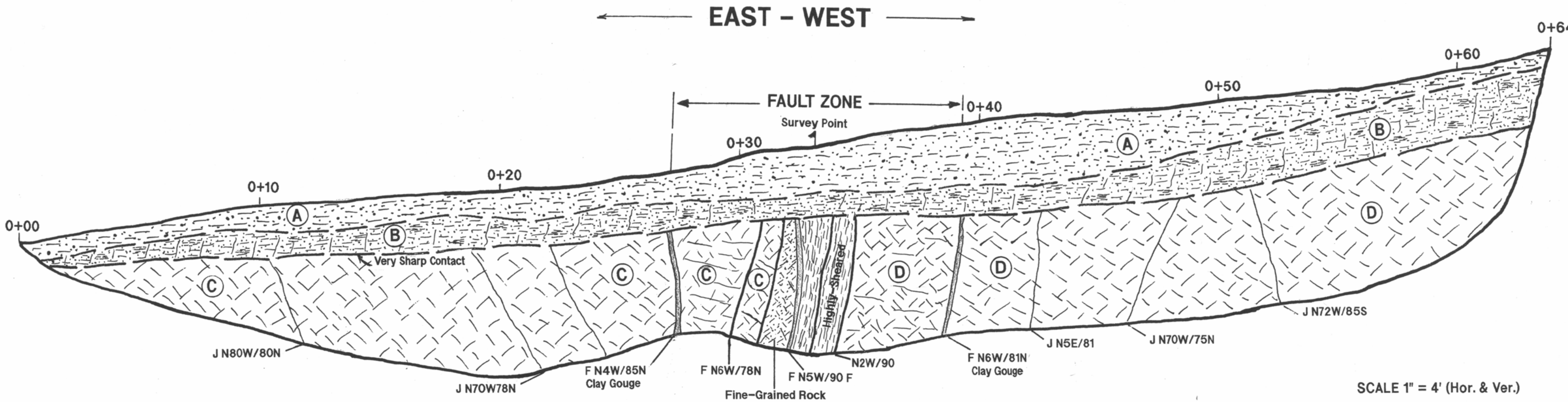
LINEATION / FAULT

ET-3 EXPLORATORY TRENCH

APPENDIX A

EXPLORATORY TRENCH LOGS

EXPLORATORY TRENCH ET-1



LITHOLOGIC DESCRIPTION

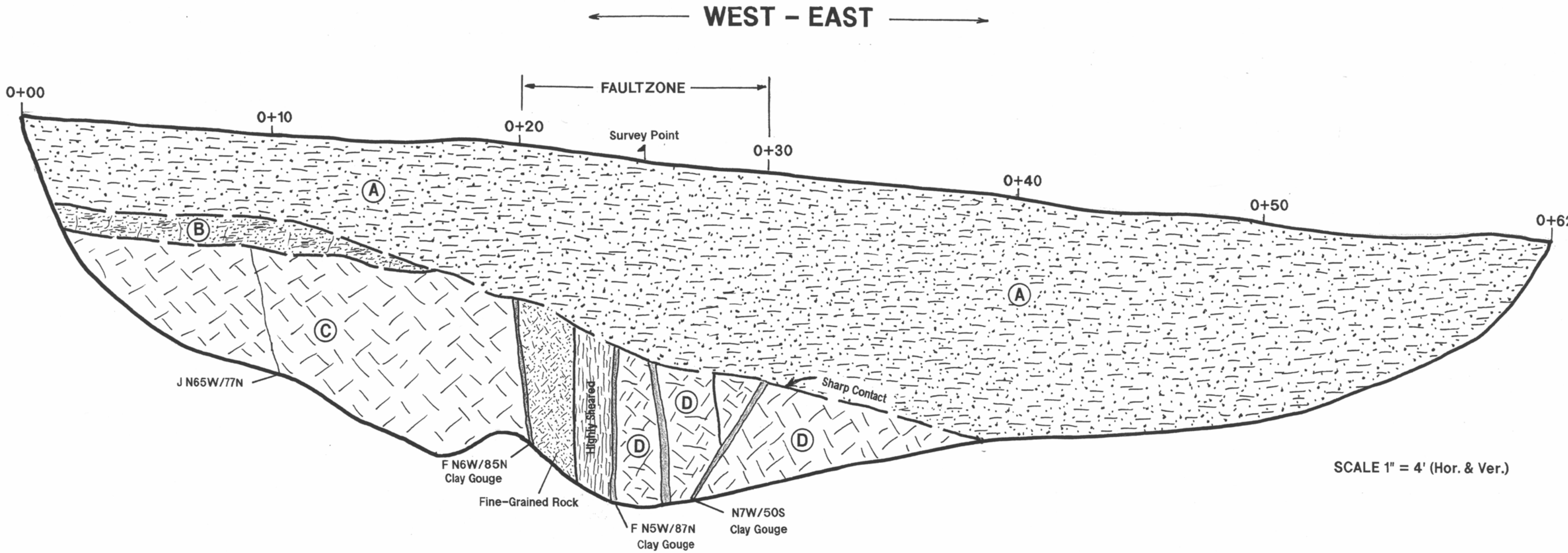
(A) - **YOUNGER ALLUVIUM** (Holocene): SILTY SAND; Brown (10YR 4/3), fine- to coarse-grained, massive structure, slightly moist, slightly cohesive, abundant rootlets, moderately loose, local small pieces of trash.

(B) - **OLDER ALLUVIUM** (Pleistocene): CLAYEY SILTY SAND; Dark Brown (10YR 3/3), fine-medium grained with minor coarse, moderately indurated, blocky soil structure, illuviated clay along ped faces, cohesive, moist, occasional pores.

(C) - **GRANITIC BEDROCK** (Cretaceous): VAL VERDE TONALITE; Olive Brown (2.5Y 4/3), fine- to coarse-grained, slightly fractured, slightly moist, highly weathered and decomposed, tonalite in composition.

(D) - **GRANITIC BEDROCK** (Cretaceous): VAL VERDE TONALITE; Dark Olive Gray (5Y 3/2), fine- to coarse-grained, slightly fractured, wet (water seeping out along fractures up to 5' from surface), completely weathered and decomposed, tonalite in composition.

EXPLORATORY TRENCH ET-2



LITHOLOGIC DESCRIPTION

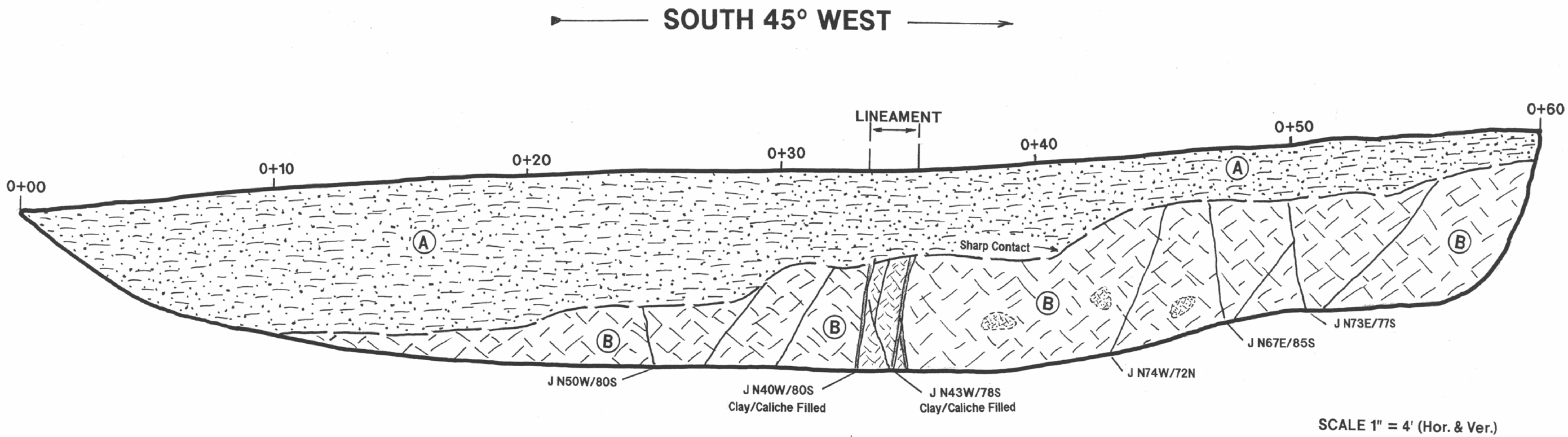
(A) - **YOUNGER ALLUVIUM** (Holocene): SILTY SAND; Brown (10YR 4/3), fine- to coarse-grained, massive structure, slightly moist, slightly cohesive, abundant rootlets, moderately loose, local small pieces of trash.

(B) - **OLDER ALLUVIUM** (Pleistocene): CLAYEY SILTY SAND; Dark Brown (10YR 3/3), fine-medium grained with minor coarse, moderately indurated, blocky soil structure, illuviated clay along ped faces, cohesive, moist, occasional pores.

(C) - **GRANITIC BEDROCK** (Cretaceous): VAL VERDE TONALITE; Dark Olive Gray (5Y 3/2), fine- to coarse-grained, slightly fractured, wet (water seeping out along fractures up to 5' from surface), completely weathered and decomposed, tonalite in composition.

(D) - **GRANITIC BEDROCK** (Cretaceous): VAL VERDE TONALITE; Dark Grayish Brown (2.5Y 4/2), fine- to coarse-grained, slightly fractured, slightly moist, highly weathered and decomposed, tonalite in composition.

EXPLORATORY TRENCH ET-3



LITHOLOGIC DESCRIPTION

(A) - **YOUNGER ALLUVIUM** (Holocene): SILTY SAND; Dark Brown (10YR 3/3), fine- to medium-grained with minor coarse, massive structure, slightly moist, slightly cohesive, abundant rootlets, slightly porous, moderately loose.

(B) - **GRANITIC BEDROCK** (Cretaceous): VAL VERDE TONALITE; Dark Grayish Brown (2.5Y 4/2), fine- to coarse-grained, highly fractured, slightly moist, highly weathered and decomposed, tonalite in composition, clay and caliche along joints/fractures between 33' to 37', occasional large mafic clasts present.

APPENDIX B

REFERENCES

REFERENCES

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MAPS UTILIZED

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AERIAL PHOTOGRAPHS

Riverside County Flood Control District, 1962, Photo Numbers 1-04 through 1-106, Scale 1”=2,000’, dated January 28, 1962.

Riverside County Flood Control District, 1974, Photo Numbers 306 and 307, Scale 1”=2,000’, dated June 20, 1974.

Riverside County Flood Control District, 1980, Photo Numbers 319 and 320, Scale 1”=2,000’, dated April 10, 1980.

Riverside County Flood Control District, 1984, Photo Numbers 1292 through 1294, Scale 1”=1,600’, dated February 4, 1984.

Riverside County Flood Control District, 1995, Photo Numbers 7-24 and 7-25, Scale 1”=1,600’, dated January 30, 1995.

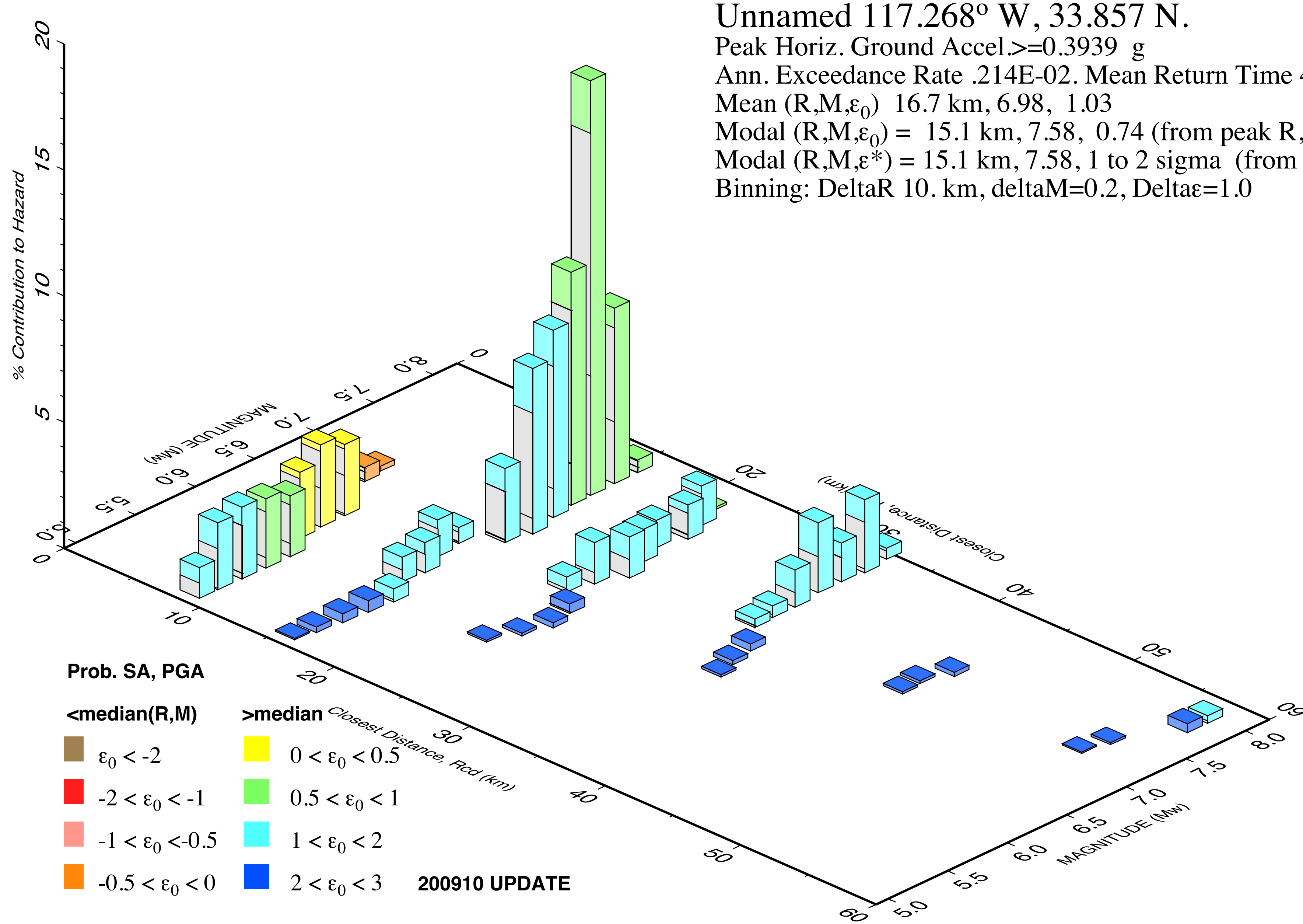
Riverside County Flood Control District, 2000, Photo Numbers 7-25 and 7-26, Scale 1”=1,600’, dated March 11, 2000.

Riverside County Flood Control District, 2005, Photo Numbers 7-24 and 7-25, Scale 1”=1,600’, dated April 14, 2005.

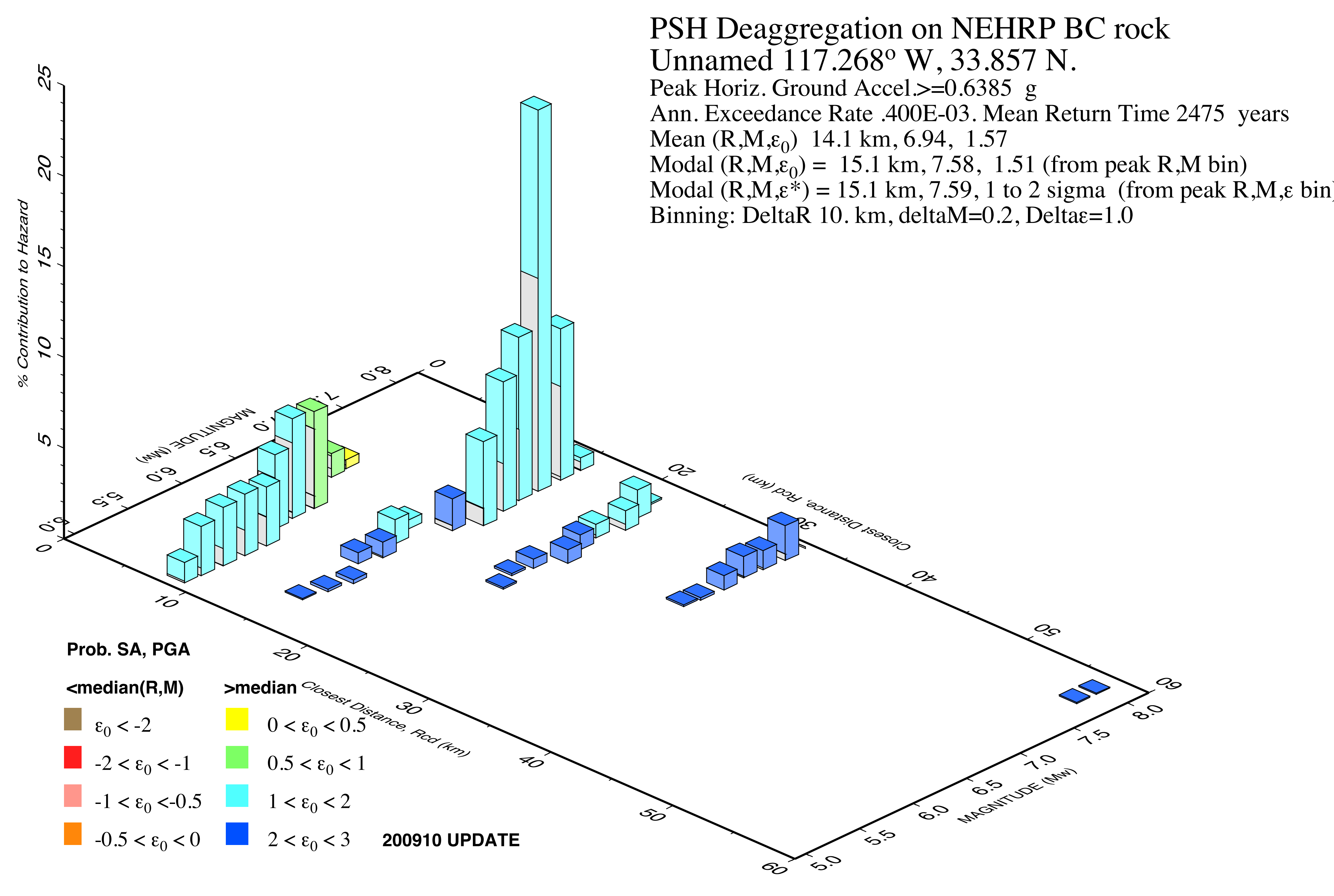
APPENDIX F

SEISMICITY

PSH Deaggregation on NEHRP BC rock
Unnamed 117.268° W, 33.857 N.
Peak Horiz. Ground Accel.>=0.3939 g
Ann. Exceedance Rate .214E-02. Mean Return Time 475 years
Mean (R,M,ε_0) 16.7 km, 6.98, 1.03
Modal (R,M,ε_0) = 15.1 km, 7.58, 0.74 (from peak R,M bin)
Modal (R,M,ε^*) = 15.1 km, 7.58, 1 to 2 sigma (from peak R,M,ε bin)
Binning: DeltaR 10. km, deltaM=0.2, Deltaε=1.0



GMT 2014 Oct 1 00:39:57 Distance (R), magnitude (M), epsilon (E0,E) deaggregation for a site on rock with average vs= 760. m/s top 30 m. USGS CGHT PSHA2008 UPDATE Bins with lt 0.05% contrib. omitted



GMT 2014 Oct 1 00:38:25 Distance (R), magnitude (M), epsilon (E0,E) deaggregation for a site on rock with average vs= 760. m/s top 30 m. USGS CGHT PSHA2008 UPDATE Bins with lt 0.05% contrib. omitted

APPENDIX G

EARTHWORK SPECIFICATIONS

APPENDIX G

MATRIX GEOTECHNICAL CONSULTING

EARTHWORK SPECIFICATIONS

These specifications present generally accepted standards and minimum earthwork requirements for the development of the project. These specifications shall be the guidelines for earthwork except where specifically superceded in preliminary geology and soil reports, grading plan review reports or by prevailing grading codes or ordinances of the controlling agency.

1.0 <u>GENERAL</u>

1.1 The contractor shall be responsible for the satisfactory completion of all earthwork in accordance with the project plans and specifications.

1.2 The project Soil Engineer and Engineering Geologist of their representative shall provide testing services, and Geotechnical consultation during the duration of the project.

1.3 All clearing, grubbing, stripping and site preparation for the project shall be accomplished by the Contractor to the satisfaction of the Soil Engineer.

1.4 It is the Contractor's responsibility to prepare the ground surface to receive the fills to the satisfaction of the Soil Engineer and to place, spread, mix and compact the fill in accordance with the job specifications and as requested by the Soil Engineer. The Contractor shall also remove all material considered by the Soil Engineer to be unsuitable for use in the construction of compacted fill.

1.5 The Contractor shall have suitable and sufficient equipment in operation to handle the amount of fill being placed. When necessary, equipment will be shut down temporarily in order to permit proper compaction of fills.

2.0 <u>GENERAL</u>

2.1 Excessive vegetation and all deleterious material should be disposed of offsite as required by the Soil Engineer. Existing fill, soil, alluvium or rock materials determined by the Soil Engineer as being unsuitable for placement in compacted fills shall be removed and wasted from the site. Where applicable, the Contractor may obtain the approval of the Soil Engineer and the controlling authorities for the project to dispose of the above-described materials, or a portion thereof, in designated areas onsite.

After removals as described above have been accomplished, earth materials deemed unsuitable in their natural, in-place condition, shall be removed as recommended by the Soil Engineer/Engineering Geologist.

2.2 After the removals as delineated in Item 2.0, 2.1 above, the exposed surfaces shall be disked or bladed by the Contractor to the satisfaction of the Soil Engineer. The prepared ground surfaces shall then be brought to the specified water content, mixed as required, and compacted and tested as specified. In areas where it is necessary to obtain the approval of the controlling agency, prior to placing fill, it will be the contractor's responsibility to notify the proper authorities.

2.3 Any underground structures such as cesspools, cisterns, mining shafts, tunnels, septic tanks, wells, pipelines or others not located prior to grading are to be removed or treated in a manner prescribed by the Soil Engineer and/or the controlling agency for the project.

3.0 COMPACTED FILLS

3.1 Any materials imported or excavated on the property may be utilized in the fill, provided each material has been determined to be suitable by the Soil Engineer. Deleterious material not disposed of during clearing or demolition shall be removed from the fill as directed by the Soil Engineer.

3.2 Rock or rock fragments less than eight inches in the largest dimension may be utilized in the fill, provided they are not placed in contracted pockets and the distribution of the rocks is approved by the Soil Engineer.

3.3 Rocks greater than eight inches in the largest dimension shall be taken offsite, or placed in accordance with the recommendations of the Soil Engineer in areas designated as suitable for rock disposal.

3.4 All fills, including onsite and import materials to be used for fill, shall be tested in the laboratory by the Soil Engineer. Proposed import materials shall be approved prior to importation.

3.5 The fill materials shall be placed by the Contractor in layers that when compacted shall not exceed six inches. Each layer shall be spread evenly and shall be thoroughly mixed during the spreading to obtain near uniform water content and a uniform blend of materials.

All compaction shall be achieved at optimum water content, or above, as determined by the applicable laboratory standard. No upper limit on the optimum water content is necessary; however, the Contractor must achieve the necessary compaction and will be alerted when the material is too wet and compaction cannot be attained.

3.6 Where the water content of the fill material is below the limit specified by the Soil Engineer, water shall be added and the materials shall be blended until a uniform water content, within specified limits, is achieved. Where the water content of the fill material is above the limits specified by the Soil Engineer, the fill materials shall be aerated by disked, blading or other satisfactory methods until the water content is within the limits specified.

3.7 Each fill layer shall be compacted to minimum project standards, in compliance with the testing methods specified by the controlled governmental agency and in accordance with recommendations for the Soil Engineer.

In the absence of specific recommendations by the Soil Engineer to the contrary, the compaction standard shall be ASTM D 1557.

3.8 Where a slope-receiving fill exceeds a ration of five-horizontal to one-vertical, the fill shall be keyed and benched through all unsuitable topsoil, colluvium, alluvium, or creep material, into sound bedrock or firm material, in accordance with the recommendations and approval of the Soil Engineer.

3.9 Side hill fills shall have a <u>minimum key width</u> of 15 feet into bedrock of firm material, unless otherwise specified in the soil report and approved by the Soil Engineer in the field.

3.10 Drainage terraces and subdrainage devices shall be constructed in compliance with the ordinances of the controlling governmental agency and/or with the recommendations of the Soil Engineer and Engineering Geologist.

3.11 The contractor shall be required to maintain the specified minimum relative compaction our to the finish slope face of fill slopes, buttresses, and stabilization fills as directed by the Soil Engineer and/or governing agency for the project. The may be achieved by either overbuilding the slope and cutting back to the compacted core, or by direct compaction of the slope face with suitable equipment, or by any other procedure which produces the designated result.

3.12 Fill-over-cut slopes shall be properly keyed through topsoil, colluvium or creep material into rock or firm material: and the transition shall be stripped of all soil or unsuitable materials prior to placing fill.

The cut portion should be made and evaluated by the Engineering Geologist prior to placed of fill above.

3.12 Pad areas in natural ground and cut shall be approved by the Soil Engineer. Finished surfaces of these pads may require scarification and recompaction.

4.0 <u>CUT SLOPES</u>

4.1 The Engineering Geologist shall inspect all cut slopes and shall be notified by the Contractor when cut slopes are started.

4.2 If, during the course of grading, unforeseen adverse or potentially adverse geologist conditions are encountered, the Engineering Geologist and Soil Engineer shall investigate, analyze and make recommendations to treat these problems.

4.3 Non-erodible interceptor swales shall be placed at the top of cut slopes that face the same direction as the prevailing drainage.

4.4 Unless otherwise specified in soil and geological reports, no cut slopes shall be excavated higher or steeper than allowed by the ordinances or controlling governmental agencies.

4.5 Drainage terraces shall be constructed in compliance with the ordinances of the controlling governmental agencies, and/or in accordance with the recommendations of the Soil Engineer or Engineering Geologist.

5.0 GRADING CONTROL

5.1 Fill placement shall be observed by the Soil Engineer and/or his representative during the progress of grading.

Field density tests shall be made by the Soil Engineer and/or his representative to evaluate the compaction and water content compliance of each layer of fill. Density tests shall be performed at intervals not to exceed two feet of fill height. Where sheepsfoot rollers are used, the soil may be disturbed to a depth of several inches. Density determinations shall be taken in the compacted material below the disturbed surface at a depth determined by the Soil Engineer or his representative.

5.2 Where tests indicate that the density of any layer of fill, or portion thereof, is below the required relative compaction, or improper water content is evident, the particular layer or portion shall be reworked until the required density and/or water content has been attained. No additional fill shall be placed over an area until the last placed lift of fill has been test and found to meet the density and water content requirements and that lift approved by the Soil Engineer.

5.3 Where the work is interrupted by heavy rains, fill operations shall not be resumed until field observations and tests by the Soil Engineer indicate the water content and density of the fill are within the limits previously specified.

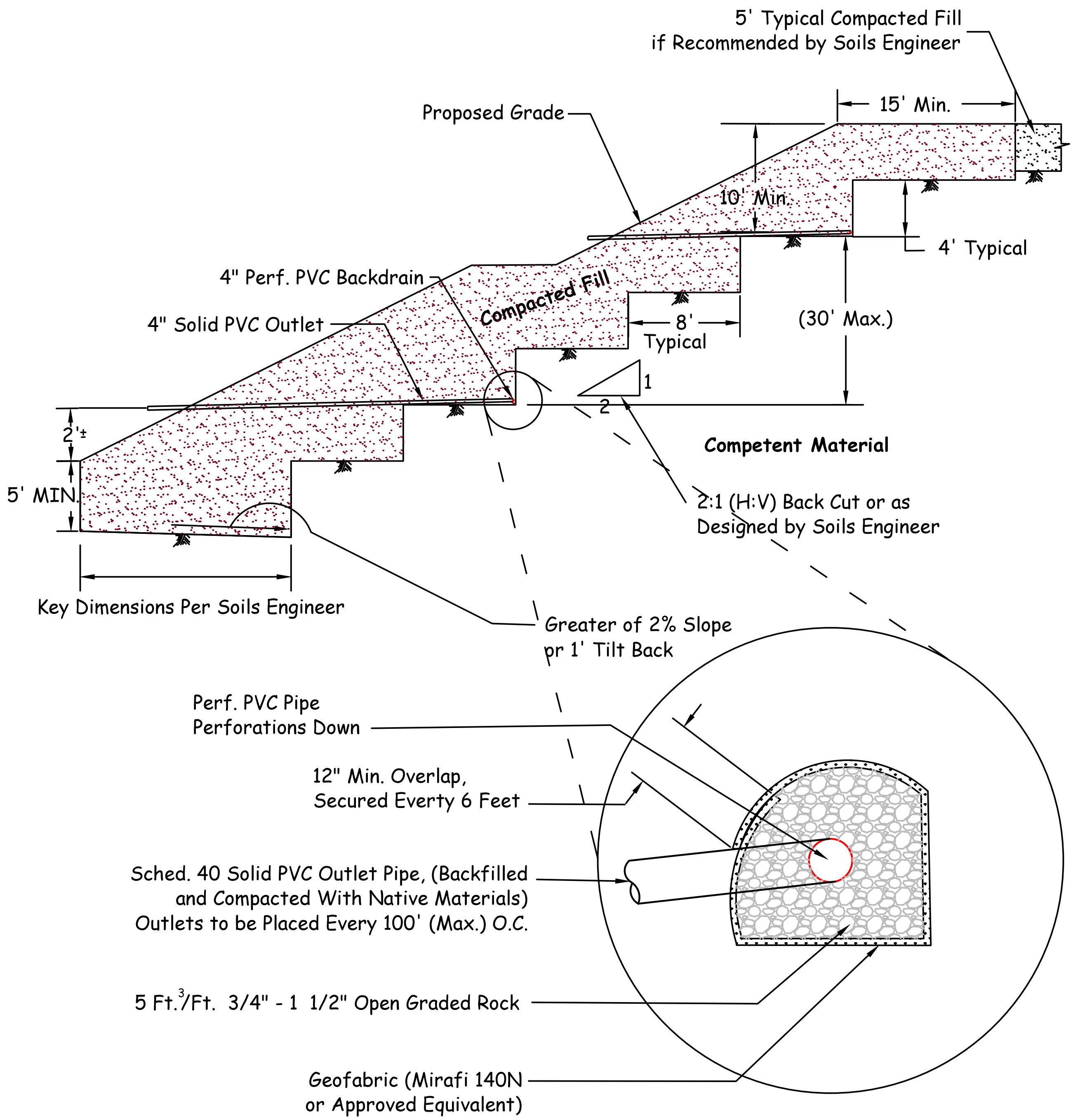
5.4 During construction, the Contractor shall properly grade all surfaces to maintain good drainage and prevent ponding of water. The Contractor shall take remedial measures to control surface water and to prevent erosion of graded area until such time as permanent drainage and erosion measures have been installed.

5.5 Observation and testing by the Soil Engineer shall be conducted during the filling and compacting operations in order that he will be able to state in his opinion all cut and filled areas area graded in accordance within the approved specifications.

5.6 After completion of grading and after the Soil Engineer and Engineering Geologist have finished their observations of the work, final reports shall be submitted. No further excavation or filling shall be undertaken without prior notification of the Soil Engineer and/or Engineering Geologist.

6.0 SLOPE

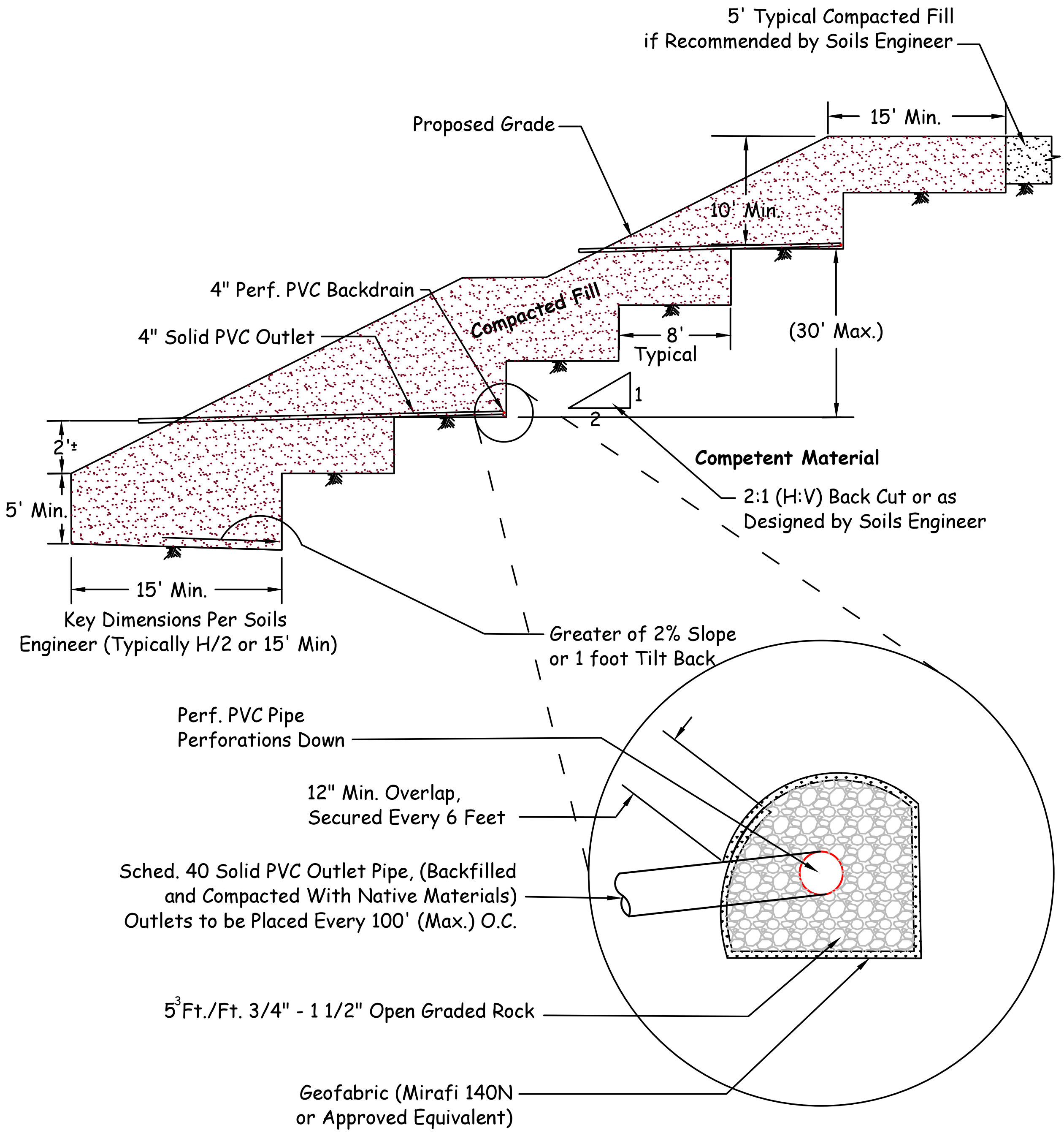
6.1 All finished cut and fill slopes shall be planted and/or protected from erosion in accordance with the project specification and/or recommended by a landscape architect.



MATRIX

TYPICAL BUTTRESS DETAIL

July 1, 2012



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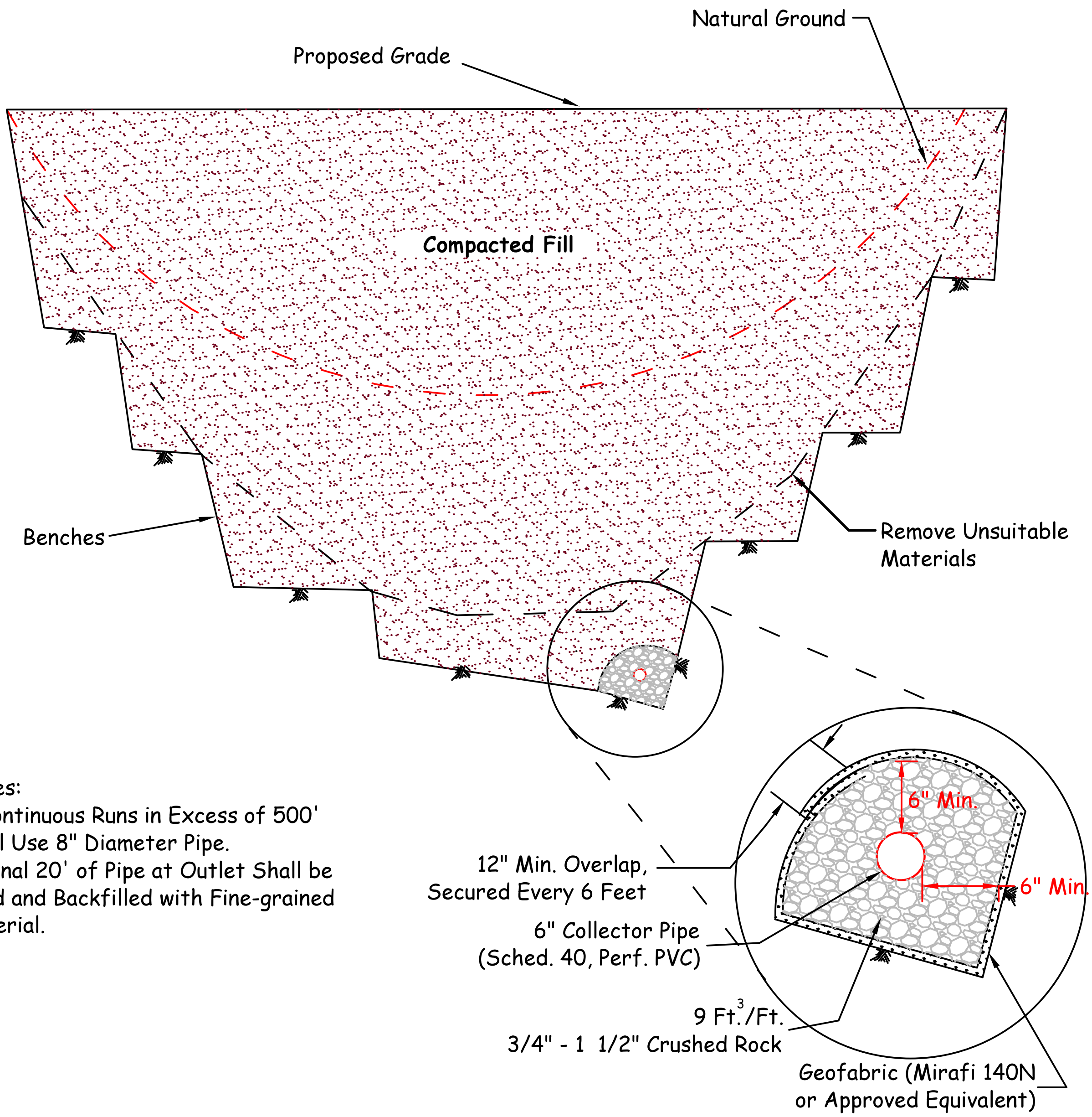
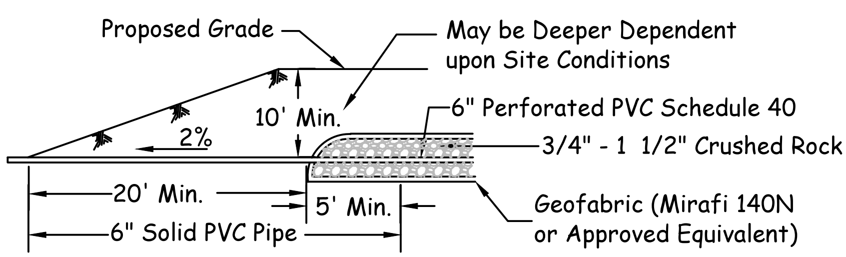
TYPICAL STABILIZATION FILL DETAIL

July 1, 2012



Notes:
1) Continuous Runs in Excess of 500' Shall Use 8" Diameter Pipe.
2) Final 20' of Pipe at Outlet Shall be Solid and Backfilled with Fine-grained Material.

Proposed Outlet Detail



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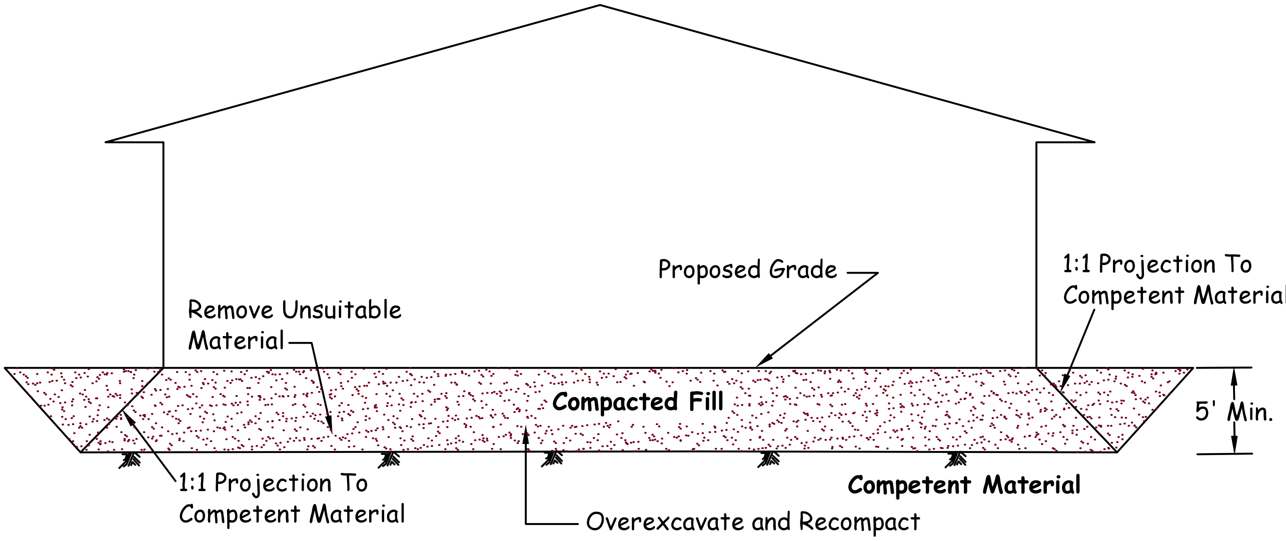
CANYON SUBDRAINS

July 1, 2012

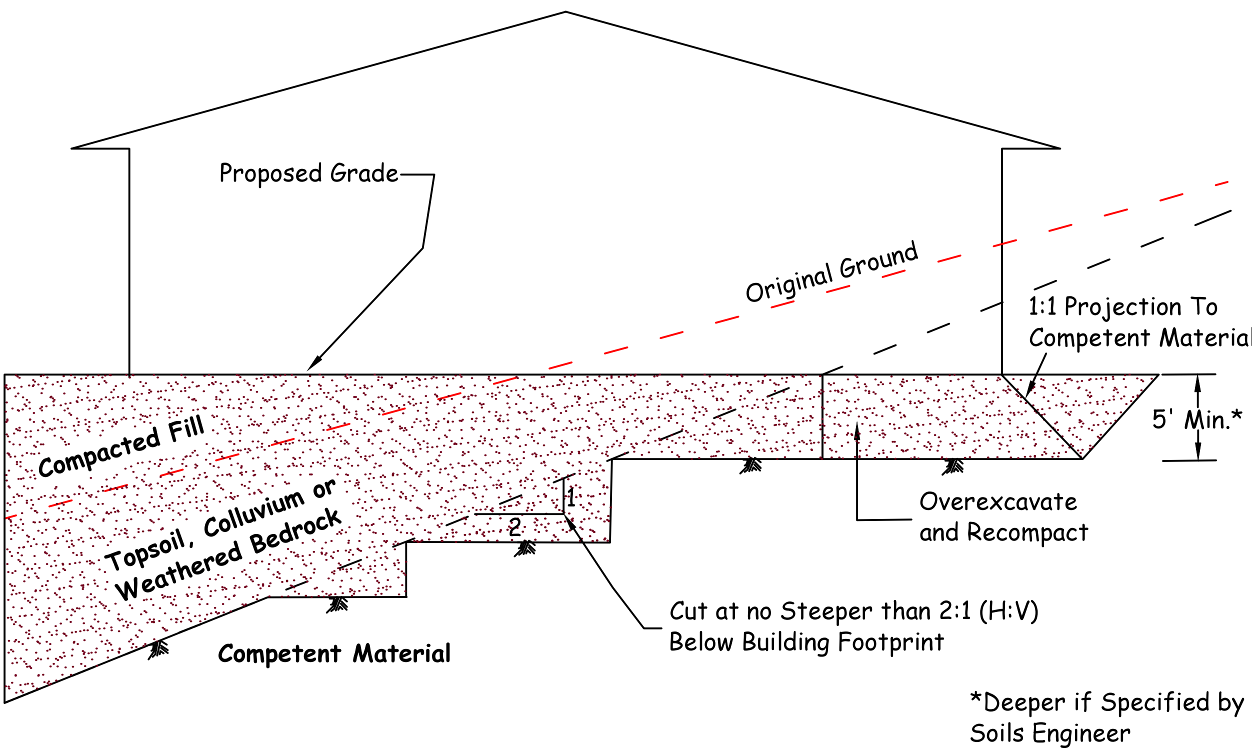
Cut Lot
(Exposing Unsuitable Soils at Design Grade)



Note 1: Removal Bottom Should be Graded With Minimum 2% Fall Towards Street or Other Suitable Area (as Determined by Soils Engineer) to Avoid Ponding Below Building

Note 2: Where Design Cut Lots are Excavated Entirely Into Competent Material, Overexcavation May Still be Required for Hard-Rock Conditions or for Materials With Variable Expansion Characteristics.

Cut/Fill Transition Lot



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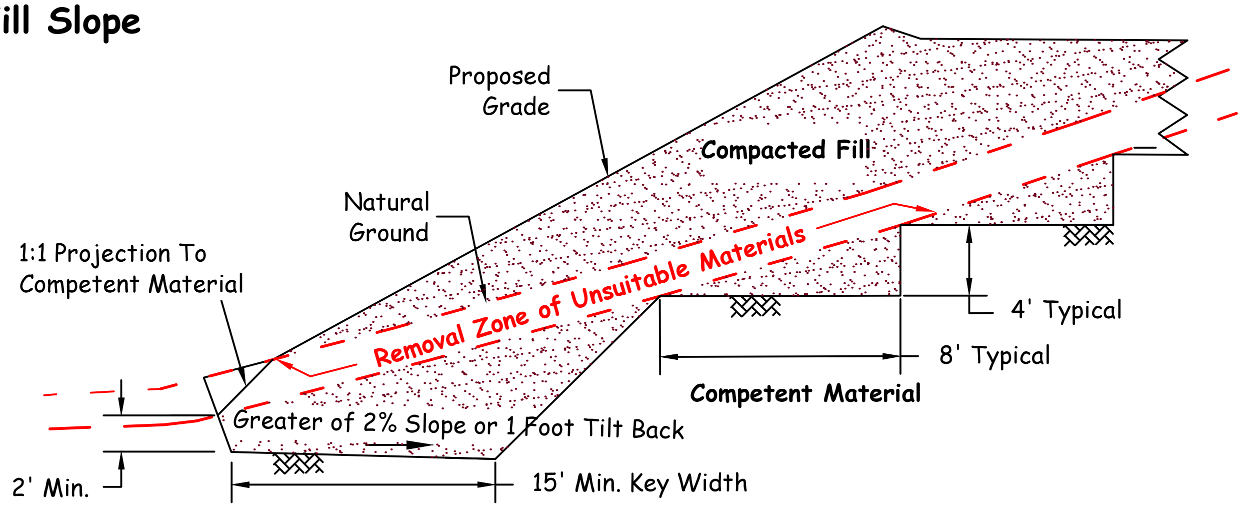
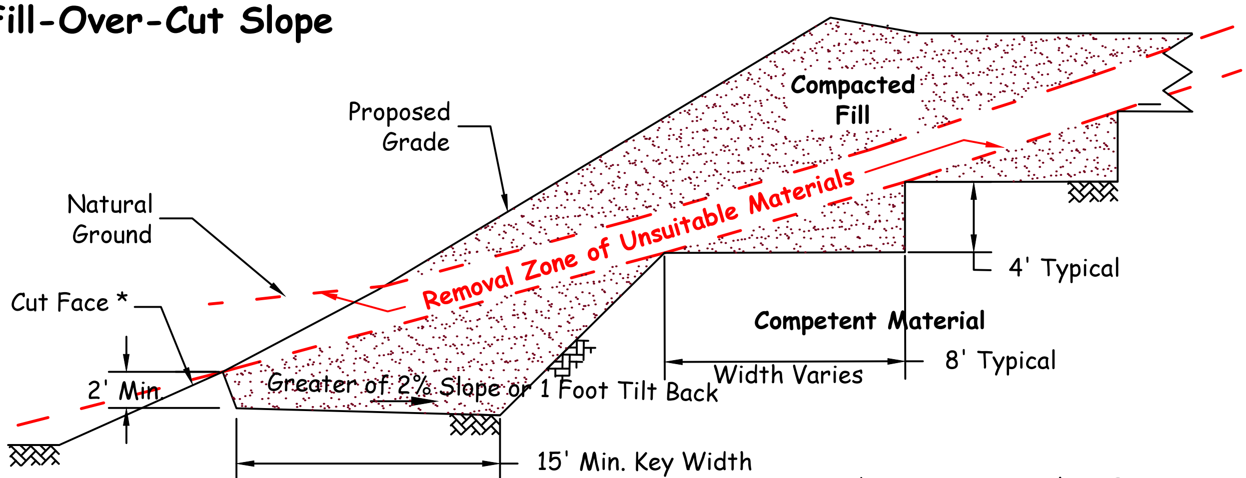
CUT AND TRANSITION LOT OVEREXCAVATION DETAIL

July 1, 2012

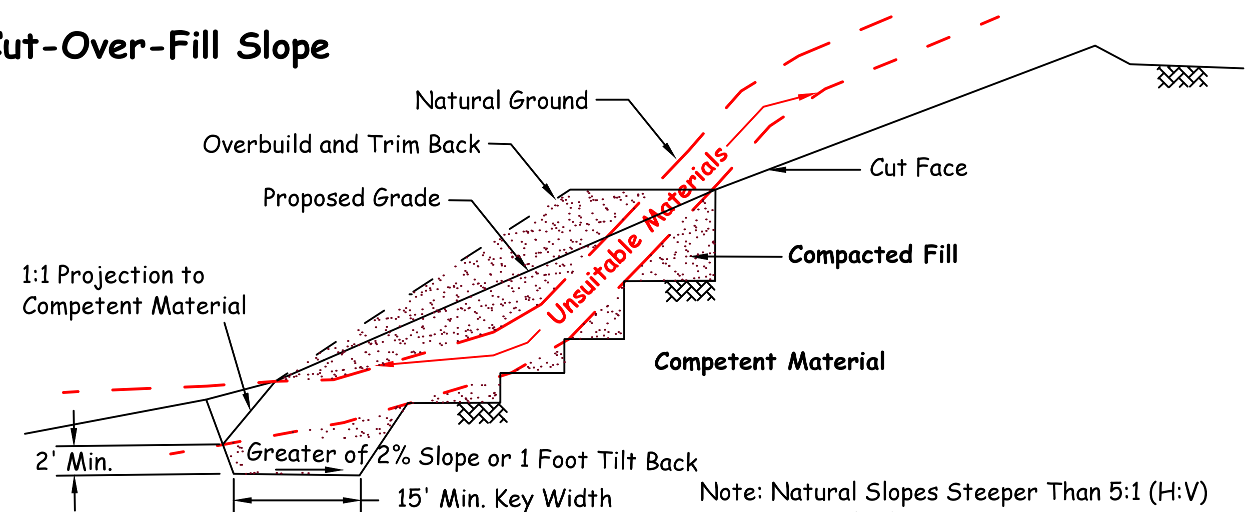
Fill Slope



Fill-Over-Cut Slope



Cut-Over-Fill Slope



Note: Natural Slopes Steeper Than 5:1 (H:V) Must Be Benched.

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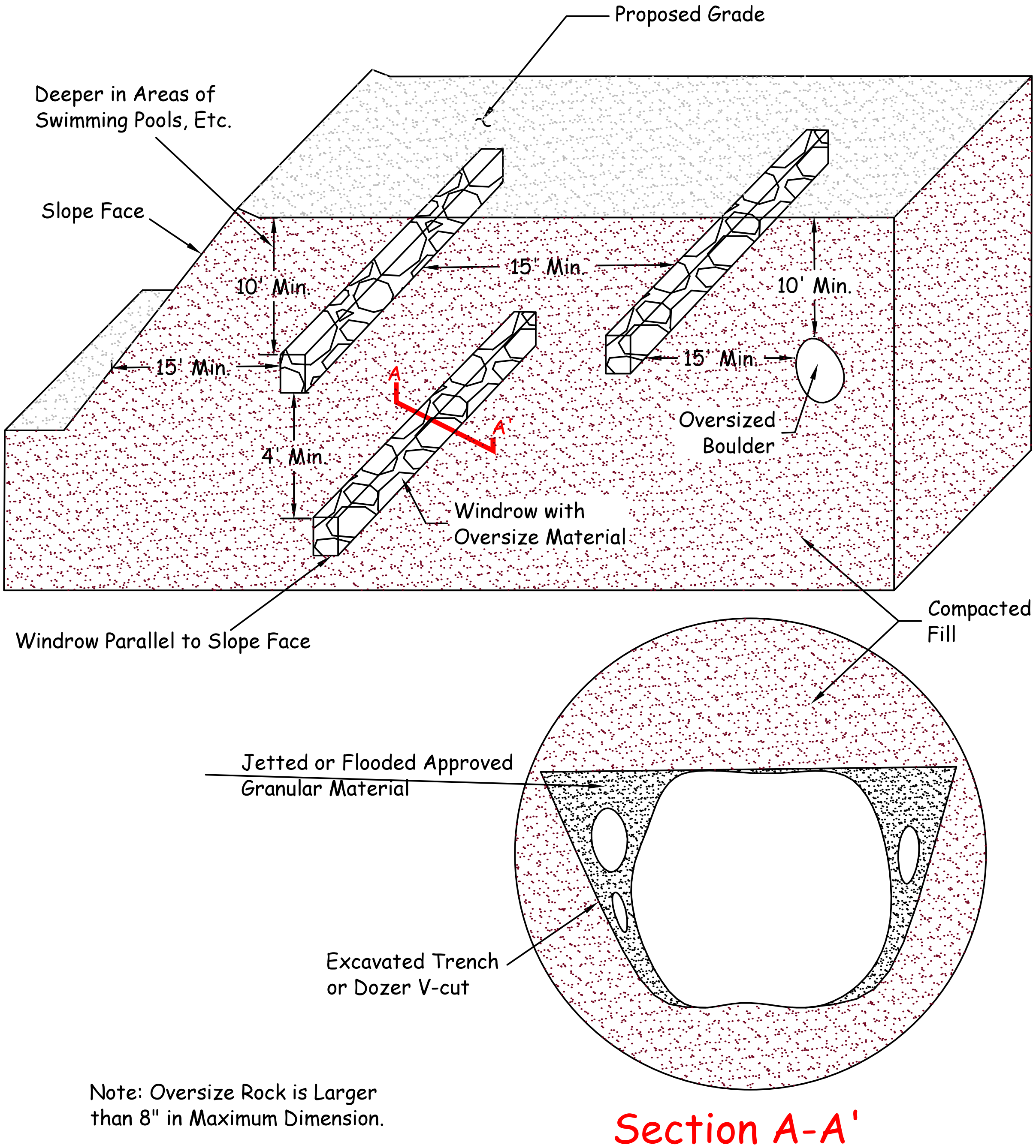
KEYING AND BENCHING



Note: Oversize Rock is Larger than 8" in Maximum Dimension.

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OVERSIZE ROCK DISPOSAL DETAIL

July 1, 2012