APPENDIX D

Preliminary Geotechnical Investigation and Report of Infiltration Testing

PRELIMINARY GEOTECHNICAL INVESTIGATION FOR DUE DILIGENCE PURPOSES, WEST PROPERTY, PROPOSED 19-ACRE (+/-), RESIDENTIAL DEVELOPMENT, SOUTHWEST OF LITTON AVENUE AND BOSTICK AVENUE, CITY OF COLTON, CALIFORNIA

Prepared for:

MR. SCOTT MCKHANN
1448 Andalusian Drive
Norco, California 92860

Project No. 021906-001

May 30, 2006

Leighton and Associates, Inc.
A LEIGHTON GROUP COMPANY

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To: Mr. Scott McKhann
1448 Andalusian Drive
Norco, California 92860

Attention: Mr. Scott McKhann

Subject: Preliminary Geotechnical Investigation for Due Diligence Purposes, West Property, Proposed 19-Acre (+/-) Residential Development, Southwest of Litton Avenue and Bostick Avenue, City of Colton, California

Introduction

In response to your request, Leighton and Associates, Inc. has conducted a preliminary geotechnical investigation for the proposed 19-acre (+/-), West Property residential development southwest of the intersection of Bostick Avenue and Litton Avenue in the City of Colton, California. The purpose of our investigation has been to evaluate the geotechnical conditions at the site with respect to the planned development and to provide preliminary geotechnical recommendations for design and construction. Our investigation was based on our discussions with you, as development plans for the site were unavailable at the time of our investigation. Based on our discussions with you, the proposed development is expected to consist of a series of terraced residential pads with ascending cut slopes on the south and west.

Based upon our investigation and analysis, the proposed development is feasible from a geotechnical viewpoint, provided our recommendations are incorporated into the design and construction of the project. The most significant geotechnical issues at the site are those related to the potential for strong seismic shaking, the presence of compressible soils, and the presence of hard bedrock at shallow depths beneath the site. These and other geotechnical issues are discussed in detail in the following report. Additional investigation may be warranted based on the actual designs for the project.

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We appreciate the opportunity to work with you on this project. If you have any questions or if we can be of further service, please call us at your convenience.

Respectfully submitted,

LEIGHTON AND ASSOCIATES, INC.

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1.0 INTRODUCTION

1.1 Site Location and Proposed Development

The site is approximately 19-acres in area and is roughly rectangular in shape. The site includes two parcels APN 0275-081-01 and -02 in San Bernardino County, California. The site is bounded by Bostick Avenue on the east, Litton Avenue on the north and the ascending slopes of the undeveloped La Loma Hills on the west and south. Existing single-family residences are present to the north and east of the site. The La Loma Hills rise approximately 400 feet above the lower portion of the site. The site is currently vacant and was previously used for agricultural purposes. The site drains toward the east, away from the La Loma Hills and ultimately towards the Santa Ana River which is located approximately ½ mile north of the site.

Development plans for the site were unavailable at the time of our investigation. Based on our discussions with you, the proposed development is expected to consist of a series of terraced residential pads with ascending cut slopes on the south and west. Improvements typical to a residential tract development are expected including streets and underground utilities.

1.2 Purpose of Investigation

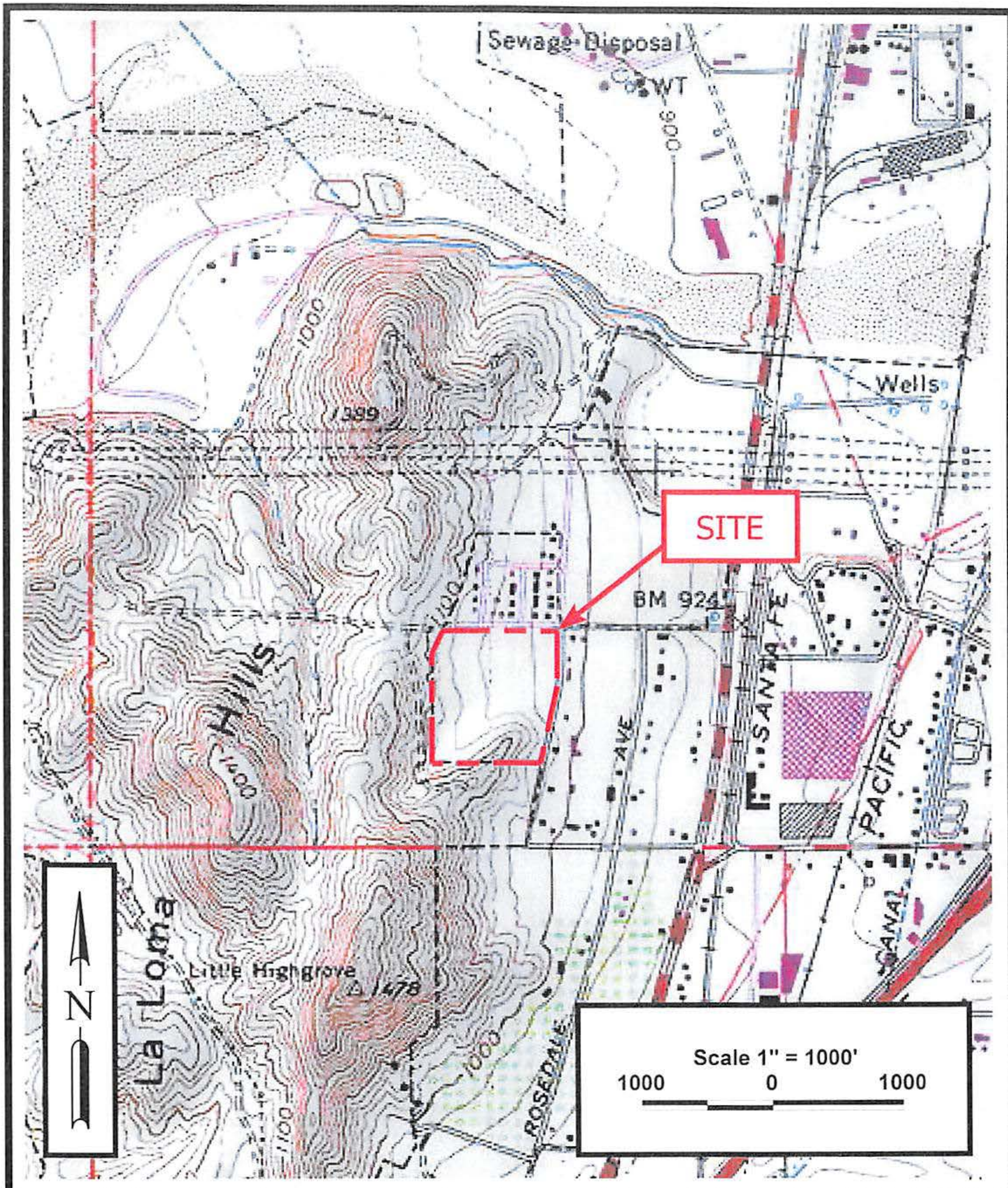
The purpose of this study has been to evaluate the general geotechnical conditions at the site to identify significant geotechnical or geologic issues that would impact development of the site, and to provide preliminary geotechnical recommendations for design and construction. A 100-scale version of the topographic base map for the project prepared by Hillwig-Goodrow, LLC has been used as the base for our Geotechnical Map, Plate 1.

1.3 Scope of Investigation

The scope of our investigation has included the following tasks:

- Background Review - A background review of readily available, relevant, in-house geotechnical reports, literature and historic aerial photographs was performed.

- Field Coordination - We coordinated with Underground Service Alert (USA) to have underground services and/or utility lines located prior to the beginning of our field investigation.



SITE LOCATION MAP

19-ACRE WEST PROPERTY

SOUTHWEST CORNER OF
LITTON AVENUE AND BOSTIC AVENUE
COLTON, CALIFORNIA

Leighton and Associates
A LEIGHTON GROUP COMPANY

Project No.
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Date
May, 2006

Figure 1

- Hollow Stem Auger Borings - We excavated, logged and sampled 6 hollow-stem auger borings (B-1 through B-6). The borings were excavated to a maximum depth of approximately 41 feet below the existing ground surface. Bulk and relatively undisturbed samples of representative soil types were obtained and transported to our affiliate laboratory for testing. The boring logs are presented in Appendix B. The results of the in situ moisture content and dry density tests are shown on the boring logs. Approximate boring locations are shown on the accompanying Geotechnical Map, Plate 1.

  Backhoe Test Pits - Fifteen backhoe test pits were excavated and logged in representative areas of the site to a maximum depth of 15 feet below the existing ground surface. Each test pit was logged by a member of our technical staff. Representative bulk soil samples were obtained from selected test pits. In-situ moisture content and dry density tests were performed in selected test pits using a nuclear density test gauge, the results of which are shown on the test pit logs, presented in Appendix C. The approximate test pit locations are shown on the accompanying Geotechnical Map, Plate 1.

- Laboratory Tests - Laboratory tests were performed on selected representative bulk and relatively undisturbed soil samples obtained during our field investigation. The laboratory testing program was designed to evaluate the engineering characteristics of the soils onsite. Tests performed include:

  - In situ moisture content and dry density.
  - Maximum dry density and optimum moisture content.
  - Grain Size Analysis.
  - Atterberg Limits.
  - Shear Strength.
  - Expansion Index.
  - Consolidation and hydrocollapse.
  - Soluble sulfate concentration.
  - Chloride, resistivity and pH.
  - R-Value.

  The in situ moisture and density test results are shown on the boring logs in Appendix B. The other laboratory test results are provided in Appendix D.

- Rippability Study - A geophysical rippability study was conducted to evaluate the density of the onsite bedrock. Terra Geosciences, acting as our sub-consultant, conducted the study. They placed six seismic lines in areas where shallow bedrock was suspected. The rippability study was conducted by sending an energy pulse (a sound wave) into the earth and then "listening" for it to be reflected off rock layers in the subsurface. The time it takes for the energy pulse to return to the surface at points along a seismic line was used to estimate the velocity of the sound wave, which in turn was used to correlate the rippability of rock layers. The geophysical rippability study is presented in Appendix E.

- Engineering Analysis - The data obtained from our background review, field exploration, laboratory-testing program and the geophysical study was evaluated and analyzed in order to provide the conclusions and recommendations in the following sections.

- Report Preparation - The results of our geotechnical investigation have been summarized in this report presenting our findings, conclusions and preliminary recommendations for design and construction of the proposed development.

2.0 FINDINGS

2.1 Site Geology

The site is located in the Peninsular Range's geomorphic province within a geologically complex region of Southern California, near the intersection of the Peninsular Range's province and the Transverse Range province. The Peninsular Ranges province is characterized by a series of northwest to southeast-oriented valleys, hills and mountains separated by faults associated with, and subparallel to, the San Andreas fault system. The Transverse Range is characterized by east-west trending folds and faults the southerly moving Transverse Ranges are opposed by the northwesterly moving Peninsular Range province. The site is located immediately east of the La Loma Hills, between the northern edges of the Jurupa and San Jacintö Mountain Ranges. The active San Jacinto and San Andreas faults are located approximately 3 and 11 miles northeast of the site, respectively. Both the San Andreas and San Jacinto faults have experienced significant activity in the recent geologic past. Regional geologic maps for the area indicate the site is underlain by shallow granitic bedrock and older alluvial soil deposits.

2.2 Subsurface Soil Conditions

The onsite alluvial soil was generally observed to consist of silty sand to sandy silt with varying amounts of clay. The alluvium was generally observed to be soft to medium stiff in the near subsurface, becoming increasingly stiff with depth. Moisture contents within the alluvial soil ranging from 4 to 14 percent, averaging around approximately 6 percent. The upper 0.5 to 2 feet of the onsite soil was observed to be highly disturbed and generally loose, likely due to the past agricultural activities onsite.

Outcrops of granitic bedrock are visible on the surface in local areas of the slopes in the southern and western portions of the site. Weathered granitic bedrock was encountered in Borings B-1 through B-4 and B-6 and Test Pits TP- 2, 8, 9, 10, 12, and 14 at depths of approximately 1 to 27 feet below the ground surface, locations of the explorations are presented on the Geotechnical Map, Plate 1. Where encountered within our explorations, the bedrock was observed to be slightly moist, very dense, and generally coarse to very coarse grained.

A geophysical study of the southern and western sloped portions of the site was conducted by Terra Geosciences, whose report is included as Appendix E. The geophysical study consisted of six geophysical lines located along the sloped areas of the site in which development is expected (based on our discussions with you), the locations of the seismic lines are shown on Plate 1. Based on our surficial observations, the data obtained from our field explorations and the geophysical study, bedrock increases in depth from 0 feet in portions of the sloped areas to over 25 feet in depth at the lower lying portions of the site.

2.3 Groundwater

Groundwater was not encountered in any of the borings or test pits excavated during this investigation to a maximum depth of 41 feet. During 1933, groundwater was at a depth on the order of 100 feet in the general site vicinity (CDWR, 1970). Because shallow groundwater has been absent both recently and historically, groundwater is not expected to pose a constraint for the proposed project. Perched water at the bedrock/alluvium contact may be present locally.

2.4 Faulting and Seismicity

Our review of available in-house literature indicates that there are no known active or potentially active faults that traverse the site, and the site is not located within an Alquist-Priolo Earthquake Fault Zone (CGS, 2000) nor a San Bernardino County designated fault zone (San Bernardino County, 1994). The principal seismic hazard that could affect the site is ground shaking resulting from an earthquake occurring along several major active or potentially active faults in southern California. The known regional active and potentially active faults that could produce the most significant ground shaking at the site include various segments of the San Jacinto and San Andreas fault zones.

PHGA for the site was estimated using California Geologic Survey (CGS) Probabilistic Seismic Hazards Mapping Ground Motion data (CGS, 2003), which utilizes a probabilistic seismic hazard analysis approach based on currently available earthquake and fault information. Based on information from the CGS, the PHGA with a 10 percent probability of being exceeded in 50 years is estimated to be approximately 0.77g.

2.5 Secondary Seismic Hazards

Liquefaction Potential

Liquefaction is the loss of soil strength due to a buildup of pore-water pressure during severe ground shaking. Liquefaction is associated primarily with loose (low density), saturated, fine- to medium-grained, clean cohesionless soils. As the shaking action of an earthquake progresses, the soil grains are rearranged and the soil densifies within a short period of time. Rapid densification of the soil results in a buildup of pore-water pressure. When the pore-water pressure approaches the total overburden pressure, the soil reduces greatly in strength and temporarily behaves similarly to a fluid. Effects of severe liquefaction can include sand boils, settlement, and bearing capacity failures below structural foundations.

The site is not located in an area mapped as potentially liquifiable on the San Bernardino County Geologic Hazards Overlay for the San Bernardino South Quadrangle (San Bernardino County, 1994). Based on the absence of shallow groundwater, the potential for liquefaction is considered very low.

Seismically Induced Settlement

Seismically induced settlement consists of dry dynamic settlement (above groundwater) and liquefaction-induced settlement (below groundwater). This settlement occurs primarily within loose to moderately dense, dry granular soil. Settlement caused by ground shaking is often non-uniformly distributed, which can result in differential settlement. We have performed analyses to estimate the seismically induced settlement using the methods set forth by Tokimatsu and Seed (1987). The potential total settlement resulting from seismic loading is estimated to be on the order of 1 inch. The potential seismically induced differential settlement is estimated to be half of the total settlement over a horizontal distance of 40 feet.

2.6 Slope Stability

A number of the test pits excavated during this investigation were excavated within the slopes in the western and southern portions of the site, as shown on Plate 1. The purpose of these test pits was to observe and evaluate the depth to bedrock and to observe any potential adverse geologic conditions within the bedrock (out of slope foliations or joint patterns). Based on our observations, the slope is blanketed by a layer of colluvial soil of varying thickness. The soil overlying the slope is of variable depth due to the variable weathering properties of the bedrock. The depth to bedrock as encountered during our investigation is shown on Plate 1. The slopes, when cut at an inclination of 2:1 are expected to be underlain by either very dense, massive granitic bedrock, or decomposed bedrock (silty sand) surrounding hard granitic corestones. Any loose colluvial soil exposed on the cutslopes will need to be removed during grading of the site. During grading, if the proposed cutslopes expose massive and hard granitic bedrock, the slopes may be cut at an inclination of 1.5:1. All cutslopes should be geologically mapped during grading to determine if any loose colluvial soils are present and to evaluate whether the slopes may be cut at an inclination of 1.5:1.

No landslides or other evidence of slope instability were observed during our investigations of the site and no evidence of slope instability was noted on geologic maps and aerial photographs of the area. Based on our understanding of the site conditions, we anticipate that the natural slopes and the planned manufactured slopes will be grossly stable and significant slope stability concerns at the site are not expected.

Loose surficial soils mantle most of the natural slopes above the site and surficial failures and mudflows could develop in these soils on steeper portions of the slopes. Measures to control erosion and water runoff will usually limit the impact of surficial slope instability and mudflows. Additional measures such as debris walls, and berms to direct water and potential mud flow debris to erosion control devices (such as brow ditches) may be needed in some areas based on the specific site conditions and development plans. Specific recommendations should be provided as the grading plans are developed.

2.7 Compressible and Collapsible Soil

Based on our investigation, the upper 5 to 15 feet of older alluvium is generally considered to be slightly compressible.

Hydrocollapse potential refers to the potential settlement of a soil under existing stresses upon being wetted. Four representative samples from the upper 5 feet of the subsurface soil were tested for hydrocollapse potential. Test results indicate that the near surface soil onsite has a slight hydrocollapse potential.

2.8 Expansive Soils

A representative sample of the subsurface soil was tested for expansion potential. Test results indicate an Expansion Index of 2. Based on these results and our experience in the general site vicinity, the onsite soil is expected to exhibit a very low expansion potential.

2.9 Sulfate Content

Water-soluble sulfates in soil can react adversely with concrete. However, concrete in contact with soil containing sulfate concentrations of less than 0.10 percent are considered to have negligible sulfate exposure (UBC, 1997 edition, Chapter 19).

Near-surface soil samples tested during this investigation had a soluble sulfate content of less than 0.02 percent by weight, indicating negligible sulfate exposure. As such, the soils exposed at pad grade are not expected to pose a significant potential for sulfate reaction with concrete.

2.10 Resistivity, Chloride and pH

Soil corrosivity to ferrous metals can be estimated by the soil's pH level, electrical resistivity, and chloride content. In general, soil having a minimum resistivity between 1,000 and 2,000 ohm-cm is considered corrosive. Soil with a chloride content of 500 ppm or more is considered corrosive to ferrous metals. As a screening for potentially corrosive soil, two representative soil samples were tested during this investigation to determine minimum resistivity, chloride content, and pH level. The minimum soil resistivity of the samples ranged from 5,300 to 6,450 ohm-cm, the chloride content was less than 50 ppm, and the pH level was 7.7 to 7.9. These results indicate that the soil is considered moderately corrosive to ferrous metal.

2.11 Rippability and Oversize Material

Based on the geophysical study conducted by Terra Geosciences (Appendix E), dense bedrock is present at the surface and shallow subsurface in the slopes located in the southern and western portions of the site. The majority of the bedrock in these areas has a seismic velocity ranging between 4,000 and 7,000 feet per second. This material would typically be considered borderline rippable with a Caterpillar D9 dozer. Seismic Line S-1 encountered bedrock with a velocity over 11,000 feet per second at a depth of between 20 to 40 feet below the existing ground surface. This material would likely be non rippable and require blasting if cuts are planned to these depths.

In general, if the onsite bedrock is heavily fractured, it could be ripped, however, if fracturing is found to be minimal, excavation of the earth material with conventional earthmoving equipment could be very difficult. Based on these results, it is probable that blasting of the granitic bedrock to achieve design grade may be required depending on final pad locations and planned depths of cuts. Oversize material will likely be generated from the excavation of the bedrock. Although not encountered during our investigation, oversized boulders may be encountered within the older alluvium material, due to the proximity of the site to the La Loma Hills.

3.0 CONCLUSIONS AND PRELIMINARY RECOMMENDATIONS

3.1 General Conclusion

Based upon this study, we conclude that development of the site is feasible from a geotechnical standpoint, provided the preliminary recommendations presented herein are considered in the design and construction of the site. No severe geologic or soil-related hazards or constraints that would preclude development of the site have been found during the course of this study. Additional geotechnical review, evaluation and investigation will be required based on the actual development plans.

3.2 General Earthwork and Grading

All grading should be performed in accordance with the General Earthwork and Grading Specifications presented in Appendix F, unless specifically revised or amended below or by future recommendations based on final development plans.

Site Preparation

Prior to construction, the site should be cleared of vegetation, trash, organics and debris, which should be disposed of offsite. Any underground obstructions onsite should be removed. The resulting cavities should be properly backfilled and compacted. Efforts should be made to locate any existing utility lines. Those lines should be removed or rerouted if they interfere with the proposed construction, and the resulting cavities should be properly backfilled and compacted. In addition, any uncontrolled artificial fill onsite should be removed. Any trees in the areas of the proposed improvements should be removed and grubbed out. Excavations to remove trees should be backfilled with compacted fill.

Overexcavation and Recompaction

To reduce the potential for adverse differential settlement of the proposed structures, the underlying subgrade soil should be prepared in such a manner that a uniform response to the applied loads is achieved. Based on our preliminary data, we recommend that the soil underneath conventional footings be overexcavated and recompacted to a minimum depth of 4 feet below the bottom of the proposed footings or 5 feet below the existing grade, whichever is deeper. The overexcavation and recompaction should extend a minimum distance of 5 feet away from the footings. These recommendations assume one- or two-story, conventional single-family residential structures. Revised recommendations may be warranted based on the actual development plans for the site, and on any future geotechnical investigations.

Areas outside the overexcavation limits of buildings planned for asphalt or concrete pavement and flatwork and areas to receive fill should be overexcavated to a depth of 2 to 3 feet below the existing ground surface or 1 foot below proposed finish grade, whichever is deeper.

After completion of the overexcavation, and prior to fill placement, the exposed surfaces should be scarified to a minimum depth of 6 inches, moisture-conditioned to, or slightly above, optimum moisture content, and recompacted to a minimum 90 percent relative compaction.

Fill Placement and Compaction

The onsite soil is generally suitable for use as compacted structural fill, provided it is free of debris, organic material, and oversized material (greater than 8 inches in largest dimension). Any soil to be placed as fill, whether onsite or imported material, should be accepted by Leighton and Associates.

All fill soil should be placed in thin, loose lifts, moisture-conditioned, as necessary, to near optimum moisture content, and compacted to a minimum 90 percent relative compaction as determined by ASTM Test Method D1557. Aggregate base should be compacted to a minimum of 95 percent relative compaction.

We are aware of the City of Colton Municipal Code which requires that graded pads, when located in areas designated as susceptible to earth movement due to the existing soil conditions (i.e., La Loma Hills), be compacted using a minimum standard of 95 percent relative compaction. It is our opinion that the slopes on and adjacent to the site are grossly stable. It is our opinion that there is no need to compact graded building pads on this site to a minimum of 95 percent relative compaction and that fill soil placed on this site during development can be compacted using the generally accepted minimum standard of 90 percent relative compaction.

Shrinkage and Subsidence

The change in volume of excavated materials upon recompaction as fill varies according to soil type and location. This volume change is represented as a percentage increase (bulking) or decrease (shrinkage) in volume of fill after removal and recompaction.

Subsidence occurs as natural ground is moisture conditioned and densified to receive fill. Field and laboratory data used in our calculation included laboratory-measured maximum dry density for soil types encountered at the subject site and the measured in-place density of soils encountered. We estimate the following earth volume changes will occur during grading:

Shrinkage (Alluvial Soil)	Approximately 10 percent
Bulkage (Granitic Bedrock)	Approximately 10 percent
Subsidence	Approximately 0.10 feet

The level of fill compaction, variations in the dry density of the existing soils and other factors influence the amount of volume change. Some adjustments to earthwork volume should be anticipated during grading of the site. These estimates do not include shrinkage due to removal of trees and vegetation, rock, trash or debris.

Rippability and Oversized Materials

The prevailing alluvial materials onsite should be rippable using conventional heavy equipment in good working condition and using modern earthmoving methods.

Based on the findings of the geophysical study, borderline rippable material was encountered within all seismic lines. The depth to the borderline rippable material ranged from 0 to 20 feet below the existing ground surface. Blasting may be required depending on the depth of cut and areas planned for development. In addition, hard rock may be encountered in some utility trench excavations.

No oversized material (greater than 8 inches in dimension) was encountered during our investigation. However, oversized materials may be encountered locally during excavation of the alluvial soils, particularly adjacent to the toe of the natural slopes; and during potential excavations in bedrock areas. Oversized rock should be placed in accordance with the recommendations presented in the General Earthwork and Grading Specifications (Appendix F). If cuts are planned requiring blasting, additional recommendations will be required for the placement of rock fill.

3.3 Preliminary Foundation Recommendations

Based on our lab results and the presence of near-surface granular soil onsite, we anticipate that soils with very low expansion potential (Expansion Index less than 20) will be exposed at pad grade across most of the graded lots onsite.

Conventional shallow foundation design recommendations are provided in this report for planning purposes. Additional testing of the soils present near finish grade will be required to provide final foundation design information. Typical 1- to 2-story residential houses can be supported on conventional shallow foundations founded in properly compacted fill (see Section 3.2). The footings should have a minimum embedment depth of 18 inches for two 2-story houses and 12 inches for one-story houses. Isolated footings should have a minimum width of 24 inches. Continuous footings should have a minimum width of 18 inches for 2-story buildings and 15 inches for 1-story buildings. An allowable bearing capacity of 2,000 psf may be used, based on the minimum embedment depth and width. The allowable bearing value may be increased by 250 psf per foot increase in depth or width to a maximum allowable bearing pressure of 3,500 psf. The allowable bearing pressures are for the total dead load and frequently applied live loads. These values may be increased by one third when considering loads of short duration, such as those imposed by wind and seismic forces. Footing reinforcement should be designed by the structural engineer.

Settlement

The total static settlement is estimated to be 1 inch. Static differential settlement is estimated to be on the order of ½ inch over a horizontal distance of 30 feet for shallow footings. Since settlement is a function of footing size and contact bearing pressure, differential settlement can be expected between adjacent columns or walls where a large differential loading condition exists.

We estimate the potential total settlement resulting from seismic loading is on the order of 1 inch. The potential differential settlement resulting from seismic loading is estimated to be half of the total seismic settlement over a horizontal distance of 40 feet.

Conventional Slabs-On Grade

The Expansion Index of representative soils at finish grade should be verified by the geotechnical consultant during grading. Where conventional light floor loading conditions exist, the following minimum recommendations for conventional slabs-on-grade should be used, which are based on a very low expansion potential:

- A minimum slab thickness of 4 inches (nominal). Slab reinforcement should consist of a minimum of No. 3 rebar placed at 24 inches on center in each direction and placed with adequate concrete cover.

Conventional shallow foundation design recommendations are provided in this report for planning purposes. Additional testing of the soils present near finish grade will be required to provide final foundation design information. Typical 1- to 2-story residential houses can be supported on conventional shallow foundations founded in properly compacted fill (see Section 3.2). The footings should have a minimum embedment depth of 18 inches for two 2-story houses and 12 inches for one-story houses. Isolated footings should have a minimum width of 24 inches. Continuous footings should have a minimum width of 18 inches for 2-story buildings and 15 inches for 1-story buildings. An allowable bearing capacity of 2,000 psf may be used, based on the minimum embedment depth and width. The allowable bearing value may be increased by 250 psf per foot increase in depth or width to a maximum allowable bearing pressure of 3,500 psf. The allowable bearing pressures are for the total dead load and frequently applied live loads. These values may be increased by one third when considering loads of short duration, such as those imposed by wind and seismic forces. Footing reinforcement should be designed by the structural engineer.

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- A minimum slab thickness of 4 inches (nominal). Slab reinforcement should consist of a minimum of No. 3 rebar placed at 24 inches on center in each direction and placed with adequate concrete cover.

- To reduce the potential for excessive cracking, concrete slabs-on-grade should be provided with construction or weakened plane joints at frequent intervals. Joints should be laid out to form approximately square panels.

- The slab subgrade soil should be moisture conditioned to at least optimum moisture content to a minimum depth of 12 inches prior to placing the moisture retarder, steel or concrete.

A moisture retarder consisting of 10-mil Visqueen (or equivalent) should be placed below slabs where moisture-sensitive floor coverings or equipment are planned. The moisture retarder should be sealed at the penetrations and laps. The moisture retarder should be covered by a minimum of 2 inches of sand. Prior to placement of the moisture retarder, all protruding gravel or other objects that could puncture the retarder should be removed.

Moisture retarders can impede, but not eliminate moisture vapor movement from the underlying soils up through the slabs. Moisture vapor transmission through the concrete slabs may be additionally reduced by the use of concrete additives. We recommend that the floor-covering contractor be consulted prior to attempting applications of the flooring.

Within concrete slabs, minor cracking of the concrete as it cures due to drying and shrinkage is normal and should be expected. However, cracking is often aggravated by a high water/cement ratio, high concrete temperature at the time of placement, small nominal aggregate size, and rapid moisture loss due to hot, dry, and/or windy weather conditions during placement and curing. Cracking due to temperature and moisture fluctuations can also be expected. The use of low slump concrete can reduce the potential for shrinkage cracking. Additionally, our experience indicates that the use of reinforcement in slabs and foundations can generally reduce the potential for concrete cracking.

Lateral Resistance of Shallow Foundations

The soil resistance available to withstand lateral loads on a shallow foundation is a function of the frictional resistance along the base of the footing and the passive resistance that may develop as the face of the structure tends to move into the soil. The frictional resistance between the base of the foundation and the subgrade soil may be computed using a coefficient of friction of 0.35. This value may be increased by one third when considering loads of short duration, such as those imposed by wind and seismic forces. The passive resistance may be computed using an equivalent fluid pressure of 300 pcf, assuming there is constant contact between the footing and undisturbed soil.

3.4 Seismic Recommendations

Seismic parameters presented in this report should be considered during project design. In order to reduce the effects of ground shaking produced by regional seismic events, seismic design should be performed in accordance with the most recent edition of the Uniform Building Code (UBC). The following data should be considered for the seismic analysis of the subject site:

1997 UBC Seismic Parameters	
Seismic Zone	4
Soil Profile Type	S_D
Seismic Source	San Jacinto-San Bernardino Fault
Seismic Source Type	A
Distance to Seismic Source	Approximately 4 km
Near Source Factor, N_a	1.3
Near Source Factor, N_v	1.7

3.5 Retaining Walls

Areas planned for retaining walls should be overexcavated in accordance with the recommendations provided for buildings in Section 3.2. We recommend that retaining walls be backfilled with onsite, very low expansive soil and constructed with a backdrain in accordance with the recommendations provided on Figure 2 (rear of text). Based on these recommendations, the following parameters may be used for the design of conventional retaining walls up to 6 feet tall:

Static Equivalent Fluid Weight (pcf)		
Condition	Level	2:1 Slope
Active	35	55
At-Rest	55	65
Passive	300 (Maximum of 3 ksf)	--

The above values do not contain an appreciable factor of safety, so the structural engineer should apply the applicable factors of safety and/or load factors during design.

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Cantilever walls that are designed to yield at least 0.001H, where H is equal to the wall height, may be designed using the active condition. Rigid walls and walls braced at the top should be designed using the at-rest condition.

Passive pressure is used to compute soil resistance to lateral structural movement. In addition, for sliding resistance, a frictional resistance coefficient of 0.35 may be used at the concrete and soil interface. The lateral passive resistance should be taken into account only if it is ensured that the soil providing passive resistance, embedded against the foundation elements, will remain intact with time.

In addition to the above lateral forces due to retained earth, surcharge due to improvements, such as an adjacent structure, should be considered in the design of the retaining wall. Loads applied within a 1:1 projection from the surcharging structure on the stem of the wall should be considered in the design.

A soil unit weight of 120 pcf may be assumed for calculating the actual weight of the soil over the wall footing.

Retaining wall footings should have a minimum width of 12 inches and a minimum embedment of 12 inches below the lowest adjacent grade. An allowable bearing capacity of 2,000 psf may be used for retaining wall footing design, based on the minimum footing width and depth. This bearing value may be increased by 250 psf per foot increase in width or depth to a maximum allowable bearing pressure of 3,500 psf.

3.6 Cement Type and Corrosion Protection

Preliminary laboratory testing indicates that the onsite soils have negligible concentrations of soluble sulfates. Additional testing should be performed on the soils present at finish grade. For planning purposes, it appears that Type II cement will be acceptable for use in concrete in contact with the soil.

Based on our laboratory testing of representative soil samples obtained during this investigation, soil considered corrosive to ferrous metals is present onsite. The corrosion information presented in this report should be provided to your underground subcontractors.

3.7 Preliminary Pavement Design

R-value tests, performed on representative soil samples of the existing near surface soils during our investigation, yielded R-value of 68. For design purposes, we have assumed an average R-value of 60 for pavement design.

Based on the design procedures outlined in the current Caltrans Highway Design Manual, and a design R-value of 60 for subgrade, preliminary flexible pavement sections may consist of the following for the Traffic Indices indicated. Final pavement design should be based on the Traffic Index determined by the project civil engineer and R-value testing of the street grading after grading.

AC Pavement Section Thickness		
Traffic Index	Asphaltic Concrete (AC) Thickness (foot)	Class 2 Aggregate Base (AB) Thickness (foot)
5 or less	0.25	0.35*
6	0.25	0.35*
7	0.25	0.35
8	0.35	0.35

* - minimum requirement

All pavement construction should be performed in accordance with the Standard Specifications for Public Works Construction. Field inspection and periodic testing, as needed during placement of the base course materials, should be undertaken to ensure that the requirements of the standard specifications are fulfilled. Prior to placement of aggregate base, the subgrade soil should be processed to a minimum depth of 6 inches, moisture-conditioned, as necessary, and recompacted to a minimum of 95 percent relative compaction. Aggregate base should be moisture conditioned, as necessary, and compacted to a minimum of 95 percent relative compaction.

3.8 Temporary Excavations

All temporary excavations, including utility trenches, retaining wall excavations and other excavations should be performed in accordance with project plans, specifications and all OSHA requirements.

No surcharge loads should be permitted within a horizontal distance equal to the height of cut or 5 feet, whichever is greater from the top of the slope, unless the cut is shored appropriately. Excavations that extend below an imaginary plane inclined at 45 degrees below the edge of any adjacent existing site foundation should be properly shored to maintain support of the adjacent structures.

Typical cantilever shoring should be designed based on the active fluid pressure presented in the retaining wall section. If excavations are braced at the top and at specific design intervals, the active pressure may then be approximated by a rectangular soil pressure distribution with the pressure per foot of width equal to 25H, where H is equal to the depth of the excavation being shored.

During construction, the soil conditions should be regularly evaluated to verify that conditions are as anticipated. The contractor should be responsible for providing the "competent person" required by OSHA, standards to evaluate soil conditions. Close coordination between the competent person and the geotechnical engineer should be maintained to facilitate construction while providing safe excavations.

3.9 Trench Backfill

Utility-type trenches onsite can be backfilled with the onsite material, provided it is free of debris, significant organic material and oversized material. Prior to backfilling the trench, pipes should be bedded and shaded in a granular material that has a sand equivalent of 30 or greater. The sand should extend 12 inches above the top of the pipe. The bedding/shading sand should be densified in-place with water. The native backfill should be placed in thin, loose layers, moisture conditioned, as necessary, and mechanically compacted using a minimum standard of 90 percent relative compaction.

3.10 Surface Drainage

Surface drainage should be designed to be directed away from foundations and toward approved drainage devices. Irrigation of landscaping should be controlled to maintain, as much as possible, a consistent moisture content sufficient to provide healthy plant growth without overwatering.

3.11 Grading Plan Review and Geotechnical Testing During Grading

This report was based, in part, upon data obtained from a limited number of observations, site visits, soil excavations, samples and tests. Such information is, by necessity, incomplete. The nature of many sites is such that differing soils or geologic conditions can be experienced within small distances and under varying climatic conditions. Changes in subsurface conditions can and do occur over time. As the geotechnical consultants of record for this project, Leighton and Associates should be onsite during grading to confirm that the preliminary data developed during this study is indeed representative of the actual site conditions. If a consultant other than Leighton and Associates is hired for future phases of this project and the data and recommendations from this study are used or implemented in the field, that consultant must assume the responsibility as geotechnical consultants of record.

We understand that plans for the site are still being developed. Leighton and Associates should review the grading plans for the site as they are developed. The preliminary geotechnical recommendations provided in this report are based on information available at the time the report was prepared and may change as plans are developed. Additional geotechnical investigation and/or analysis may be required based on the actual development plans.

Geotechnical observation and testing should be conducted during excavation and all phases of grading operations. Geotechnical observation and testing should be conducted at the following stages:

- Upon completion of clearing and grubbing.
- During all phases of rough grading, including removal and fill operations.
- At the completion of rough grading to conduct additional soil sampling, laboratory testing and analysis for final foundation design recommendations.
- During excavation of footings for foundations and retaining walls.
- During trench and retaining wall backfill operations.
- When any unusual conditions are encountered during grading.

A final report of rough grading accompanied by an as-graded geotechnical map should be prepared at the completion of rough grading.

3.12 ASFE Important Information About this Geotechnical Engineering Report

See ASFE insert on the following page.

Important Information About Your

Geotechnical Engineering Report

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

The following information is provided to help you manage your risks.

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

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

Read the Full Report

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

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

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

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

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

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

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

Subsurface Conditions Can Change

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

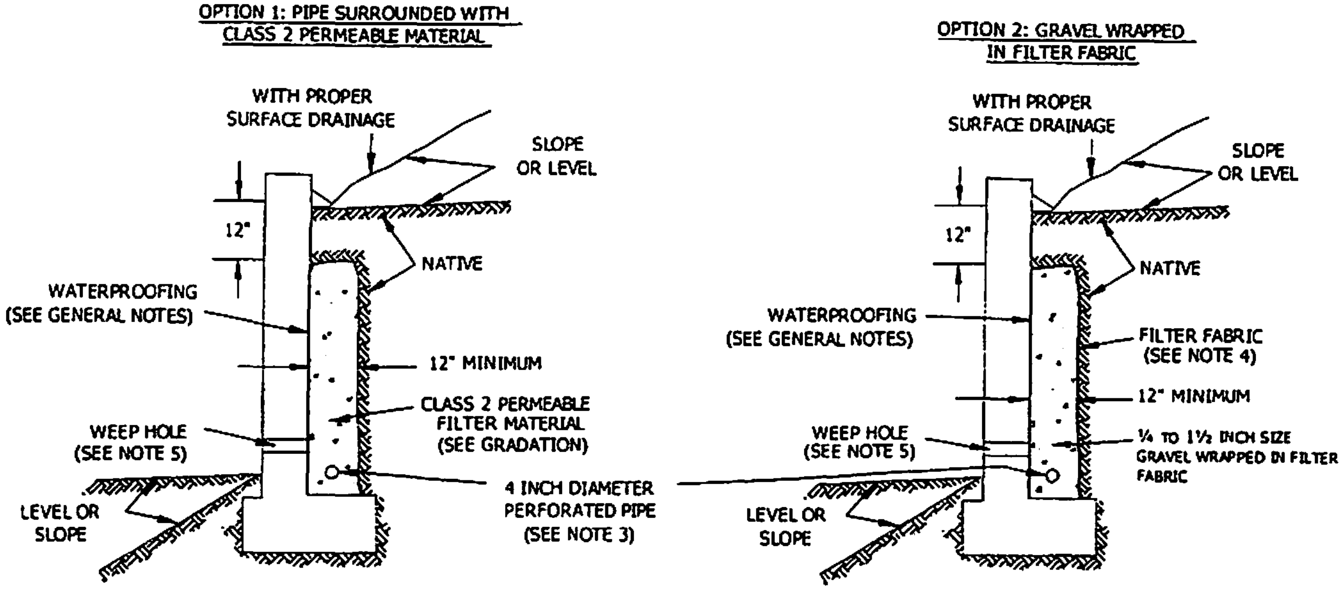
Most Geotechnical Findings Are Professional Opinions

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

A Report's Recommendations Are *Not* Final

Do not overrely on the construction recommendations included in your report. *Those recommendations are not final,* because geotechnical engineers develop them principally from judgment and opinion. Geotechnical engineers can finalize their recommendations only by observing actual

SUBDRAIN OPTIONS AND BACKFILL WHEN NATIVE MATERIAL HAS EXPANSION INDEX OF ≤50



Class 2 Filter Permeable Material Gradation
Per Caltrans Specifications

Sieve Size	Percent Passing
1"	100
3/4"	90-100
3/8"	40-100
No. 4	25-40
No. 8	18-33
No. 30	5-15
No. 50	0-7
No. 200	0-3

GENERAL NOTES:

* Waterproofing should be provided where moisture nuisance problem through the wall is undesirable.
* Water proofing of the walls is not under purview of the geotechnical engineer
* All drains should have a gradient of 1 percent minimum
*Outlet portion of the subdrain should have a 4-inch diameter solid pipe discharged into a suitable disposal area designed by the project engineer. The subdrain pipe should be accessible for maintenance (rodding)
*Other subdrain backfill options are subject to the review by the geotechnical engineer and modification of design parameters.

Notes:
1) Sand should have a sand equivalent of 30 or greater and may be densified by water jetting.
2) 1 Cu. ft. per ft. of 1/4- to 1 1/2-inch size gravel wrapped in filter fabric
3) Pipe type should be ASTM D1527 Acrylonitrile Butadiene Styrene (ABS) SDR35 or ASTM D1785 Polyvinyl Chloride plastic (PVC), Schedule 40, Armco A2000 PVC, or approved equivalent. Pipe should be installed with perforations down. Perforations should be 3/8 inch in diameter placed at the ends of a 120-degree arc in two rows at 3-inch on center (staggered)
4) Filter fabric should be Mirafi 140NC or approved equivalent.
5) Weephole should be 3-inch minimum diameter and provided at 10-foot maximum intervals. If exposure is permitted, weepholes should be located 12 inches above finished grade. If exposure is not permitted such as for a wall adjacent to a sidewalk/curb, a pipe under the sidewalk to be discharged through the curb face or equivalent should be provided. For a basement-type wall, a proper subdrain outlet system should be provided.
6) Retaining wall plans should be reviewed and approved by the geotechnical engineer.
7) Walls over six feet in height are subject to a special review by the geotechnical engineer and modifications to the above requirements.

RETAINING WALL BACKFILL AND SUBDRAIN DETAIL FOR WALLS 6 FEET OR LESS IN HEIGHT

WHEN NATIVE MATERIAL HAS EXPANSION INDEX OF ≤50

Figure 2

APPENDIX A

References

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Aerial Photographs

Date:	Flight No.	Frame:	Scale:	Agency:
11-8-55	22306	2-99	1:14,400	San Bernardino County Flood Control District (SBCFCD)
1-28-62	RCFC 62	1-151	1:24,000	Riverside County Flood Control District (RCFCD)
2-1-69	C-293	101	1:24,000	SBCFCD
1-21-78	C-279	53	1:24,000	SBCFCD
2-25-86	C-450	54 and 55	1:24,000	SBCFCD
2006	NA	NA	NA	Google Earth Image

GEOTECHNICAL BORING LOG B-1

Date 3-16-06 | Sheet 1 of 2
Project West Property, Colton | Project No. 021906-001
Drilling Co. 2-R Drilling Inc. | Type of Rig Hollow Stem Auger
Hole Diameter 8 Inch | Drive Weight 140 Pound Automatic Hammer | Drop 30"
Elevation Top of Hole ' | Location See Geotechnical Map

Logged By SFR
Sampled By SFR

Elevation Feet	Depth Feet	Graphic Log (N S)	Attitudes	Sample No.	Blows Per Foot	Dry Density pcf	Moisture Content, %	Soil Class. (U.S.C.S.)	DESCRIPTION
	0								**@ 0-26' :Alluvium**
				R-1	8			ML	@ 2': R-1: Sandy SILT, brown, slightly moist, medium stiff
	5			R-2	9	112.0	8.0	SM/ML	@ 5': R-2: Silty SAND to Sandy SILT with trace pebbles/ root hairs, medium brown, slightly moist, loose/medium stiff
	10			R-3	74	123.0	13.0	CL/ML	@ 10': R-3: Clayey SILT to Silty CLAY, red brown, slightly moist, hard
	15			R-4	82/11"	98.0	6.0	SM	@ 15': R-4: Silty SAND with some medium to fine pebbles, yellow orange to orange brown, moist, very dense
	20			R-5	80/10"			ML	@ 20': R-5: Sandy SILT with some medium to fine angular pebbles, tan to light gray, moist, very dense.
	25			R-6	50/6"			SM	@ 25': R-6: Silty Sand to SAND with few fine to medium pebbles, brown to red brown, moist, very dense
									@ 26' to Total Depth: Granitic Bedrock
									@ 26' Silty SAND to SAND, decomposed bedrock, gradational contact at 25' to 26'
	30								

SAMPLE TYPES:
S SPLIT SPOON
R RING SAMPLE
B BULK SAMPLE
T TUBE SAMPLE
G GRAB SAMPLE
SH SHELBY TUBE

LEIGHTON AND ASSOCIATES, INC.

GEOTECHNICAL BORING LOG B-1

Date 3-16-06 | Sheet 2 of 2
Project West Property, Colton | Project No. 021906-001
Drilling Co. 2-R Drilling Inc. | Type of Rig Hollow Stem Auger
Hole Diameter 8 Inch | Drive Weight 140 Pound Automatic Hammer | Drop 30"
Elevation Top of Hole ' | Location See Geotechnical Map

Logged By SFR
Sampled By SFR

Elevation Feet	Depth Feet	Graphic Log N S	Attitudes	Sample No.	Blows Per Foot	Dry Density pcf	Moisture Content, %	Soil Class. (U.S.C.S.)	DESCRIPTION
	30			R-7	50/2"	119.0	5.0		@ 30': R-7: Medium to coarse SAND with angular granite fragments, red brown to orange brown, slightly moist, very dense, decomposed to highly weathered bedrock
	35			S-1	50/3"				@ 35': S-1: Weathered GRANITE, light gray to tan, slightly moist, very dense
	40			S-2					@ 40': S-2: Weathered Granite with medium to coarse SAND, light gray to tan, very dense
									Total depth 41.5 feet Bedrock encountered @ 26 feet No ground water encountered Boring Backfilled with Native Soil
	45								
	50								
	55								
	60								

SAMPLE TYPES:
S SPLIT SPOON
R RING SAMPLE
B BULK SAMPLE
T TUBE SAMPLE
G GRAB SAMPLE
SH SHELBY TUBE

LEIGHTON AND ASSOCIATES, INC.

GEOTECHNICAL BORING LOG B-2

Date: 3-16-06 | Sheet 1 of 1

Project: West Property, Colton | Project No. 021906-001

Drilling Co.: 2-R Drilling Inc. | Type of Rig: Hollow Stem Auger

Hole Diameter: 8 Inch | Drive Weight: 140 Pound Automatic Hammer | Drop: 30"

Elevation Top of Hole: ' | Location: See Geotechnical Map

Logged By: SFR

Sampled By: SFR

Elevation Feet	Depth Feet	Graphic Log (N S)	Attitudes	Sample No.	Blows Per Foot	Dry Density pcf	Moisture Content, %	Soil Class. (U.S.C.S.)	DESCRIPTION
	0								**@ 0-14' Alluvium**
				R-1	8	111.0	7.0	SM/ML	@ 2': R-1: Sandy SILT to Silty SAND, brown, moist, medium stiff/loose
	5			R-2	22	115.0	10.0	CL	@ 5': R-2: Sandy Silty CLAY, brown, trace pebbles, moist, stiff
	10			R-3	42	122.0	14.0	CL	@ 10': R-3: CLAY to Sandy CLAY, brown, few pebbles and gravel, moist, very stiff
	15			R-4	50/6"				**@14' to Total Depth: Granitic Bedrock** @ 15': R-4: Weathered GRANITE, very dense, slightly moist to moist, very coarse grained
	20			R-5	50/2"				@ 20': R-5: No Recovery
	25								**Total depth 20.2 feet** **Bedrock encountered @ 14 feet** **No ground water encountered** **Boring backfilled with native soil**
	30								

SAMPLE TYPES:

- S SPLIT SPOON
- R RING SAMPLE
- B BULK SAMPLE
- T TUBE SAMPLE
- G GRAB SAMPLE
- SH SHELBY TUBE

LEIGHTON AND ASSOCIATES, INC.

GEOTECHNICAL BORING LOG B-3

Date 3-16-06 | Sheet 1 of 1
Project West Property, Colton | Project No. 021906-001
Drilling Co. 2-R Drilling Inc. | Type of Rig Hollow Stem Auger
Hole Diameter 8 Inch | Drive Weight 140 Pound Automatic Hammer | Drop 30"
Elevation Top of Hole ' | Location See Geotechnical Map

Logged By SFR
Sampled By SFR

Elevation Feet	Depth Feet	Graphic Log (N S)	Attitudes	Sample No.	Blows Per Foot	Dry Density pcf	Moisture Content, %	Soil Class. (U.S.C.S.)	DESCRIPTION
	0								@ 0-10': Alluvium
				R-1	7			ML	@ 2': R-1: Sandy SILT to SILT, brown, slightly moist, soft to medium stiff
	5			R-2	8	110.0	10.0	ML	@ 5': R-2: Sandy SILT, brown to slightly red brown, moist, medium stiff
									@ 9' to Total Depth: Granitic Bedrock
	10			R-3	90/7"	120.0	4.0		@ 10': R-3: Decomposed Granite with medium to coarse SAND and gravel, orange brown to tan, slightly moist, very dense
	15			R-4	50/1"	114.0	2.0		@ 15': R-4: Weathered GRANITE, tan, slightly moist to dry, very dense, fine to coarse grained
	20			R-5	bouncing				@ 20': R-5: No Recovery, sampler bouncing
	25								Total depth 20.0 feet Bedrock encountered @ 9 feet No ground water encountered Boring backfilled with native soil
	30								

SAMPLE TYPES:
S SPLIT SPOON
R RING SAMPLE
B BULK SAMPLE
T TUBE SAMPLE
G GRAB SAMPLE
SH SHELBY TUBE

LEIGHTON AND ASSOCIATES, INC.

GEOTECHNICAL BORING LOG B-4

Date 3-16-06 | Sheet 1 of 2
Project West Property, Colton | Project No. 021906-001
Drilling Co. 2-R Drilling Inc. | Type of Rig Hollow Stem Auger
Hole Diameter 8 Inch | Drive Weight 140 Pound Automatic Hammer | Drop 30"
Elevation Top of Hole ' | Location See Geotechnical Map

Logged By SFR
Sampled By SFR

Elevation Feet	Depth Feet	Graphic Log (N S)	Attitudes	Sample No.	Blows Per Foot	Dry Density pcf	Moisture Content, %	Soil Class. (U.S.C.S.)	DESCRIPTION
	0							ML	**@ 0-26.5 Alluvium**
				Bag-1					
				R-1	11	111.0	4.0	ML	@ 2': R-1: Sandy SILT, light brown, slightly moist, medium stiff
	5			R-2	11	111.0	7.0	ML	@ 5': R-2: Sandy SILT to Sandy CLAY with gravel, slightly moist to moist, medium stiff
	10			R-3	17	118.0	7.0	SM	@ 10': R-3: Silty SAND with some gravel, red brown, slightly moist, loose to medium dense
	15			R-4	25	119.0	6.0	SM	@ 15': R-4: Silty SAND with increasing gravel, red brown, slightly moist, medium dense
	20			R-5	37			SM	@ 20': R-5: Silty SAND with gravel, red brown, slightly moist, medium dense
	25			R-6	42			SM	@ 25': R-6: Silty SAND with gravel, red brown, slightly moist, medium dense
						'			**@26.5 to Total Depth: Granitic Bedrock**
	30								

SAMPLE TYPES:
S SPLIT SPOON
R RING SAMPLE
B BULK SAMPLE
T TUBE SAMPLE
G GRAB SAMPLE
SH SHELBY TUBE

LEIGHTON AND ASSOCIATES, INC.

GEOTECHNICAL BORING LOG B-4

Date 3-16-06 | Sheet 2 of 2
Project West Property, Colton | Project No. 021906-001
Drilling Co. 2-R Drilling Inc. | Type of Rig Hollow Stem Auger
Hole Diameter 8 Inch | Drive Weight 140 Pound Automatic Hammer | Drop 30"
Elevation Top of Hole ' | Location See Geotechnical Map

Elevation Feet	Depth Feet	Graphic Log (N S)	Attitudes	Sample No.	Blows Per Foot	Dry Density pcf	Moisture Content, %	Soil Class. (U.S.C.S.)	DESCRIPTION Logged By SFR Sampled By SFR
	30			R-7	50/5"				@ 30': R-7: Decomposed GRANITE, orange to red brown, slightly moist, dense
									Total depth 31.5 feet Bedrock encountered @ 26.5 feet No ground water encountered Boring backfilled with native soil
	35								
	40								
	45								
	50								
	55								
	60								

SAMPLE TYPES:
- S SPLIT SPOON
- R RING SAMPLE
- B BULK SAMPLE
- T TUBE SAMPLE
- G GRAB SAMPLE
- SH SHELBY TUBE

LEIGHTON AND ASSOCIATES, INC.

GEOTECHNICAL BORING LOG B-5

Date 3-16-06 | Sheet 1 of 1
Project West Property, Colton | Project No. 021906-001
Drilling Co. 2-R Drilling Inc. | Type of Rig Hollow Stem Auger
Hole Diameter 8 Inch | Drive Weight 140 Pound Automatic Hammer | Drop 30"
Elevation Top of Hole ' | Location See Geotechnical Map

Logged By SFR
Sampled By SFR

Elevation Feet	Depth Feet	Graphic Log (N / S)	Attitudes	Sample No.	Blows Per Foot	Dry Density pcf	Moisture Content, %	Soil Class. (U.S.C.S.)	DESCRIPTION
	0								@ 0-21.5': Alluvium
				R-1	10	112.0	4.0	ML	@ 2': R-1: Sandy SILT with some rounded gravel, light brown, slightly moist, medium stiff
	5			R-2	10	110.0	5.0	ML	@ 5': R-2: Sandy SILT with some rounded gravel, brown to light brown, slightly moist, medium stiff
	10			R-3	29	128.0	8.0	SM	@ 10': R-3: Silty SAND with some fine gravel, slightly moist to moist, medium dense
	15			R-4	23			SM	@ 15': R-4: Silty SAND with some fine gravel, slightly moist to moist, medium dense
	20			R-5	40	120.0	9.0	SM	@ 20': R-5: Silty SAND with some fine gravel, slightly moist to moist, medium dense
									Total depth 21.5 feet **No bedrock encountered** **No ground water encountered** **Boring backfilled with native soil**
	25								
	30								

SAMPLE TYPES:
S SPLIT SPOON
R RING SAMPLE
B BULK SAMPLE
T TUBE SAMPLE
G GRAB SAMPLE
SH SHELBY TUBE

LEIGHTON AND ASSOCIATES, INC.

GEOTECHNICAL BORING LOG B-6

Date 3-16-06 — Sheet 1 of 1
Project West Property, Colton — Project No. 021906-001
Drilling Co. 2-R Drilling Inc. — Type of Rig Hollow Stem Auger
Hole Diameter 8 Inch — Drive Weight 140 Pound Automatic Hammer — Drop 30"
Elevation Top of Hole ' — Location See Geotechnical Map

Logged By SFR
Sampled By SFR

Elevation Feet	Depth Feet	Graphic Log (N S)	Attitudes	Sample No.	Blows Per Foot	Dry Density pcf	Moisture Content, %	Soil Class. (U.S.C.S.)	DESCRIPTION
	0							ML	@ 0-21.5 Alluvium
				Bag-1					
				R-1	8	103.0	5.0	ML	@ 2': R-1: Sandy SILT, brown, moist,soft
	5			R-2	11			ML	@ 5': R-2: Sandy SILT, trace fine gravel, brown to red brown, moist, medium stiff
	10			R-3	25	121.0	10.0	ML	@ 10': R-3: Sandy SILT, clayey, minor fine gravel, red brown, moist, stiff to very stiff
	15			R-4	54	125.0	11.0		@ 15' to Total Depth: Granitic Bedrock @ 15': R-4: Decomposed GRANITE, moist, dense
	20			R-5	83				@ 20': R-5: Highly weathered GRANITE, red brown, slightly moist, very dense
									Total Depth 21.5 feet **Bedrock encountered @ 15 feet** **No ground water encountered** **Boring backfilled with native soil**
	25								
	30								

SAMPLE TYPES:
S SPLIT SPOON
R RING SAMPLE
B BULK SAMPLE
T TUBE SAMPLE
G GRAB SAMPLE
SH SHELBY TUBE

West Property, Colton

Project No. 021906-001

Logged By: SFR
Sampled By: SFR

Test Pit TP-1

Date Excavated: 03/21/2006

Location: See Geotechnical Map

Depth Top (ft)	Depth Bottom (ft)	Soil symbol (USCS)	Description	Geologic Unit	Test Results: Sample number	Depth (ft)	Density, Dry (pcf)	Moisture (%)
0.0	1.2	SM	Silty SAND, light brown, moist, very loose, rootlets					
					B-1	1-4		
1.2	5.0	SM/ML	Silty SAND to Sandy SILT, brown, moist, loose		N-1	2.0	90.3	7.1

Total Depth (ft): 5.0
No ground water encountered.
Test pit backfilled with native soil.

Test Pit TP-2

Date Excavated: 03/21/2006

Location: See Geotechnical Map

Depth Top (ft)	Depth Bottom (ft)	Soil symbol (USCS)	Description	Geologic Unit	Test Results: Sample number	Depth (ft)	Density, Dry (pcf)	Moisture (%)
0.0	0.6	SM/ML	Silty SAND to Sandy SILT, brown, moist, well rooted					
0.6	5.0	SM/ML	Silty SAND to Sandy SILT, brown, moist, medium dense		N-1	2.0	87.7	6.8
					N-2	5.0	88.9	11.0

Total Depth (ft): 5.0
No ground water encountered.
Test pit backfilled with native soil.

West Property, Colton

Project No. 021906-001

Logged By: SFR
Sampled By: SFR

Test Pit TP-3

Date Excavated: 03/21/2006

Location: See Geotechnical Map

Depth Top (ft)	Depth Bottom (ft)	Soil symbol (USCS)	Description	Geologic Unit	Test Results: Sample number	Depth (ft)	Density, Dry (pcf)	Moisture (%)
0.0	1.4	ML	Sandy SILT, brown, slightly moist to moist, loose, well rooted upper 7", decreasing with depth					
1.4	1.9	ML	Sandy SILT, light brown, slightly moist to moist, medium dense, trace fine gravel, rootlets thinning with depth					
1.9	5.0	ML	Sandy SILT with rounded fine Gravel, light brown, slightly moist, medium dense, trace root hairs					

Total Depth (ft): 5.0
No ground water encountered.
Test pit backfilled with native soil.

Test Pit TP-4

Date Excavated: 03/21/2006

Location: See Geotechnical Map

Depth Top (ft)	Depth Bottom (ft)	Soil symbol (USCS)	Description	Geologic Unit	Test Results: Sample number	Depth (ft)	Density, Dry (pcf)	Moisture (%)
0.0	1.0	ML	Sandy SILT, brown, moist, loose, well rooted					
1.0	1.6		Grades to Sandy SILT to Silty SAND, medium brown, slightly moist, rootlets, some coarse sand					
1.6	2.5	SM	Silty SAND, light brown, slightly moist, firm to medium dense, increasing gravel and size to 1/2", rootlets					
2.5	6.5		Decomposed GRANITE, orange brown, highly weathered, highly fractured					

Total Depth (ft): 6.5
No ground water encountered.
Test pit backfilled with native soil.

West Property, Colton Project No. 021906-001

Logged By: SFR
Sampled By: SFR

Test Pit TP-5

Date Excavated: 03/21/2006
Location: See Geotechnical Map

Depth Top (ft)	Depth Bottom (ft)	Soil symbol (USCS)	Description	Geologic Unit	Test Results Sample number	Test Results Depth (ft)	Test Results Density, Dry (pcf)	Test Results Moisture (%)
0.0	0.5	SM/ML	Sandy SILT to Silty SAND, brown, well rooted					
0.5	2.5	ML	Sandy SILT, medium brown, moist, abundant fine gravel, trace rootlets					
2.5	12.0	ML	Sandy SILT, medium brown, moist to slightly moist, medium dense, scarce fine gravel, trace rootlets, becomes blocky at 8', hard digging at 12'					
12.0	14.0	SM	Silty SAND, brown to orange brown, slightly moist to dry, very dense, scratching for depth					

Total Depth (ft): 14.0
No ground water encountered.
Test pit backfilled with native soil.

Test Pit TP-6

Date Excavated: 03/21/2006
Location: See Geotechnical Map

Depth Top (ft)	Depth Bottom (ft)	Soil symbol (USCS)	Description	Geologic Unit	Test Results Sample number	Test Results Depth (ft)	Test Results Density, Dry (pcf)	Test Results Moisture (%)
0.0	1.0	ML	Sandy SILT, brown, moist, well rooted					
1.0	3.3	SM/ML	Silty SAND to SAND with fine Gravel, brown to yellow brown, slightly moist to dry, dense, animal burrows from 1-2', rootlets					
3.3	5.0	SM	Silty fine SAND, orange brown, slightly moist, some fine gravel					

Total Depth (ft): 5.0
No ground water encountered.
Test pit backfilled with native soil.

West Property, Colton

Project No. 021906-001

Logged By: SFR
Sampled By: SFR

Test Pit TP-7

Date Excavated: 03/21/2006

Location: See Geotechnical Map

Depth Top (ft)	Depth Bottom (ft)	Soil symbol (USCS)	Description	Geologic Unit	Test Results: Sample number	Test Results: Depth (ft)	Test Results: Density, Dry (pcf)	Test Results: Moisture (%)
0.0	0.8	SM/ML	Sandy SILT to SILT, dark brown, moist, loose, well rooted					
0.8	2.0	ML	Sandy SILT with fine gravel, light brown, stiff, few rootlets					
2.0	3.4	ML	Sandy SILT, light brown to brown, slightly moist, firm to dense, some fine gravel and rootlets, 3% porosity					
3.4	10.0	ML	Sandy SILT, medium brown to light brown, slightly moist, trace fine gravel and rootlets, porous, color change at 8' to red brown					

Total Depth (ft): 10.0
No ground water encountered.
Test pit backfilled with native soil.

Test Pit TP-8

Date Excavated: 03/21/2006

Location: See Geotechnical Map

Depth Top (ft)	Depth Bottom (ft)	Soil symbol (USCS)	Description	Geologic Unit	Test Results: Sample number	Test Results: Depth (ft)	Test Results: Density, Dry (pcf)	Test Results: Moisture (%)
0.0	1.3	ML	Sandy SILT, dark brown, moist, abundant rootlets					
1.3	3.0		Granite, orange brown, raveling, scratching for depth, Joint: N20W,75SE at 2'					

Total Depth (ft): 3.0
No ground water encountered.
Test pit backfilled with native soil.

Leighton and Associates

West Property, Colton

Project No. 021906-001

Logged By: SFR
Sampled By: SFR

Test Pit TP-9

Date Excavated: 03/21/2006

Location: See Geotechnical Map

Depth Top (ft)	Depth Bottom (ft)	Soil symbol (USCS)	Description	Geologic Unit	Test Results: Sample number	Test Results: Depth (ft)	Test Results: Density, Dry (pcf)	Test Results: Moisture (%)
0.0	1.5	ML	Sandy SILT, brown, very moist, stringers, well rooted		B-1	0.5-3		
1.5	3.0		Granite, orange brown, slightly moist, very dense					

Total Depth (ft): 3.0
No ground water encountered.
Test pit backfilled with native soil.

Test Pit TP-10

Date Excavated: 03/21/2006

Location: See Geotechnical Map

Depth Top (ft)	Depth Bottom (ft)	Soil symbol (USCS)	Description	Geologic Unit	Test Results: Sample number	Test Results: Depth (ft)	Test Results: Density, Dry (pcf)	Test Results: Moisture (%)
0.0	1.7	SM/ML	Silty SAND to Sandy SILT, dark brown, moist, rootlets					
1.7	4.2	ML	Sandy SILT, light brown to brown, fine gravel, blocky, rootlets					
4.2	5.2		Decomposed GRANITE, orange brown, very dense, scratching for depth					

Total Depth (ft): 5.2
No ground water encountered.
Test pit backfilled with native soil.

Leighton and Associates

West Property, Colton

Project No. 021906-001

Logged By: SFR
Samp McKhar SFR

Test Pit TP-11

Date Excavated: 03/21/2006

Location: See Geotechnical Map

Depth Top (ft)	Depth Bottom (ft)	Soil symbol (USCS)	Description	Geologic Unit	Test Results Sample number	Depth (ft)	Density, Dry (pcf)	Moisture (%)
0.0	0.6	ML	Sandy SILT, brown, moist, loose, well rooted					
0.6	2.7	ML	Sandy SILT, medium brown, moist, firm, trace fine gravel, rootlets					
2.7	3.5	ML	Sandy SILT, medium brown, moist, firm, increasing gravel, decreasing rootlets					
3.5	5.0	ML	Sandy SILT with rounded fine Gravel, light brown, slightly moist, medium dense, trace root hairs					

Total Depth (ft): 5.0
No ground water encountered.
Test pit backfilled with native soil.

Test Pit TP-12

Date Excavated: 03/21/2006

Location: See Geotechnical Map

Depth Top (ft)	Depth Bottom (ft)	Soil symbol (USCS)	Description	Geologic Unit	Test Results Sample number	Depth (ft)	Density, Dry (pcf)	Moisture (%)
0.0	1.5	SM	Silty SAND, brown, moist, abundant rootlets					
1.5	4.2	SM	Silty SAND, brown, slightly moist, fine gravel, porous, rootlets					
4.2	8.0	SM	Silty SAND, brown to tan, slightly moist, increasing gravel, porous, gradational contact with decomposed granite					
8.0	9.0		Decomposed GRANITE, orange brown, slightly moist, very dense					

Total Depth (ft): 9.0
No ground water encountered.
Test pit backfilled with native soil.

West Property, Colton

Project No. 021906-001

Logged By: SFR
Sampled By: SFR

Test Pit TP-13

Date Excavated: 03/21/2006
Location: See Geotechnical Map

Depth Top (ft)	Depth Bottom (ft)	Soil symbol (USCS)	Description	Geologic Unit	Test Results Sample number	Depth (ft)	Density, Dry (pcf)	Moisture (%)
0.0	1.7	SM/ML	Sandy SILT to Silty SAND, brown, loose, roots in upper 6", rootlets below					
1.7	3.5	SM	Silty SAND with fine gravel, medium brown, moist, firm, trace rootlets		B-1	3-5		
3.5	4.5	ML	Sandy SILT, gray brown, moist, medium dense increasing fine gravel					
4.5	5.2	ML	Sandy SILT with rounded fine Gravel, light brown, slightly moist, medium dense, trace root hairs					

Total Depth (ft): 5.2
No ground water encountered.
Test pit backfilled with native soil.

Test Pit TP-14

Date Excavated: 03/21/2006
Location: See Geotechnical Map

Depth Top (ft)	Depth Bottom (ft)	Soil symbol (USCS)	Description	Geologic Unit	Test Results Sample number	Depth (ft)	Density, Dry (pcf)	Moisture (%)
0.0	0.8	ML	Sandy SILT, dark brown, moist, well rooted					
0.8	2.7	SM/ML	Sandy SILT to Silty SAND with fine gravel and rootlets, medium brown, burrow at 1.8'					
2.7	9.0	SM	Silty SAND, medium brown to orange brown, medium dense, some fine gravel, no rootlets, grading to light brown to orange brown at 6', blocky in cuttings, granite cobbles in wall below 6' (2-4")					
9.0	9.3		Decomposed GRANITE, orange brown, slightly moist, very dense					

Total Depth (ft): 9.3
No ground water encountered.
Test pit backfilled with native soil.

West Property, Colton

Project No. 021906-001

Logged By: SFR
Sampled By: SFR

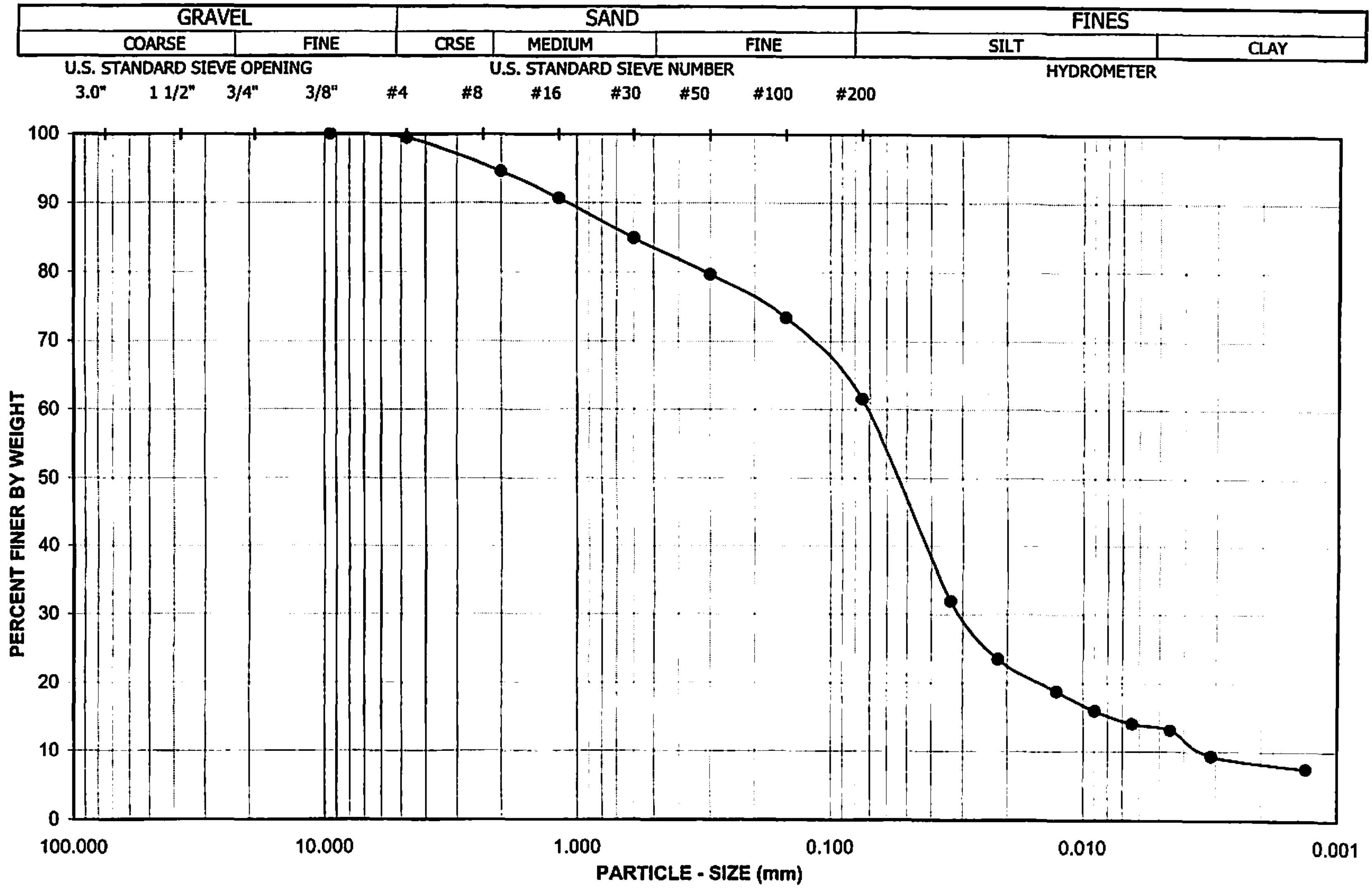
Test Pit TP-15

Date Excavated: 03/21/2006
Location: See Geotechnical Map

Depth		Soil symbol (USCS)	Description	Geologic Unit	Test Results			
Top (ft)	Bottom (ft)				Sample number	Depth (ft)	Density, Dry (pcf)	Moisture (%)
0.0	0.4	ML	Sandy SILT, dark brown, moist, loose, well rooted					
0.4	1.9	SM	Silty SAND with rootlets and some fine gravel and coarse sand, medium brown, moist, medium dense					
1.9	15.0	SM	Silty SAND, medium to light brown, slightly moist, trace fine gravel, rootlets, blocky, minor caving at 8.5', pockets of light gray decomposed granite					

Total Depth (ft): 15.0
No ground water encountered.
Test pit backfilled with native soil.

Leighton and Associates



Project Name: West Property / Colton

Project No.: 021906-001

Exploration No.: B-2 Sample No.: R-2

Depth (feet) : 5.0 Soil Type : s(CL-ML)

Soil Identification: Brown Sandy Silty Clay s(CL-ML)

GR:SA:FI : (%) 1 : 38 : 61

Leighton

PARTICLE - SIZE DISTRIBUTION ASTM D 422

Mar-06

S&H B-2, R-2 @ 5

ATTERBERG LIMITS

ASTM D 4318

Leighton

Project Name:	West Property / Colton	Tested By:	GB	Date:	03/30/06
Project No. :	021906-001	Input By:	JHW	Date:	03/31/06
Boring No.:	B-4	Checked By:	JHW		
Sample No.:	R-2	Depth (ft.)	5.0		
Soil Identification:	Brown Silty, Clayey Sand (SC-SM)				

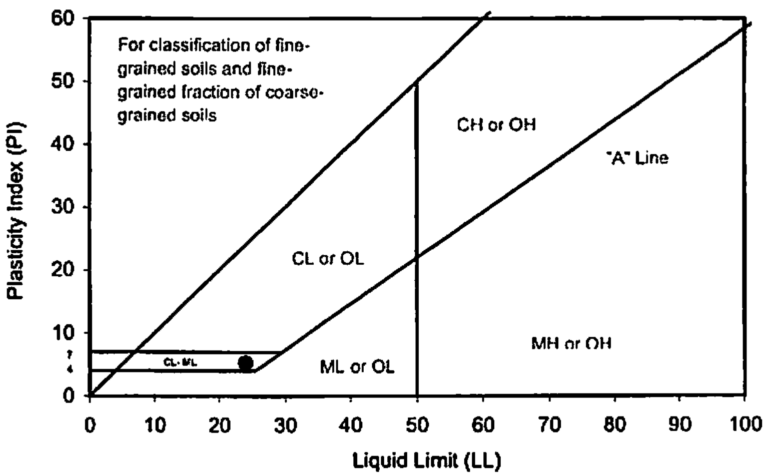
TEST	PLASTIC LIMIT		LIQUID LIMIT			
NO.	1	2	1	2	3	4
Number of Blows [N]			19	29	24	
Wet Wt. of Soil + Cont. (g)	8.00	8.27	13.00	11.42	12.27	
Dry Wt. of Soil + Cont. (g)	6.90	7.14	10.64	9.45	10.09	
Wt. of Container (g)	1.07	1.08	1.04	1.09	1.05	
Moisture Content (%) [Wn]	18.87	18.65	24.58	23.56	24.12	

Liquid Limit	**24**
Plastic Limit	**19**
Plasticity Index	**5**
Classification	**CL-ML**

PI at "A" - Line = 0.73(LL-20) | 2.92

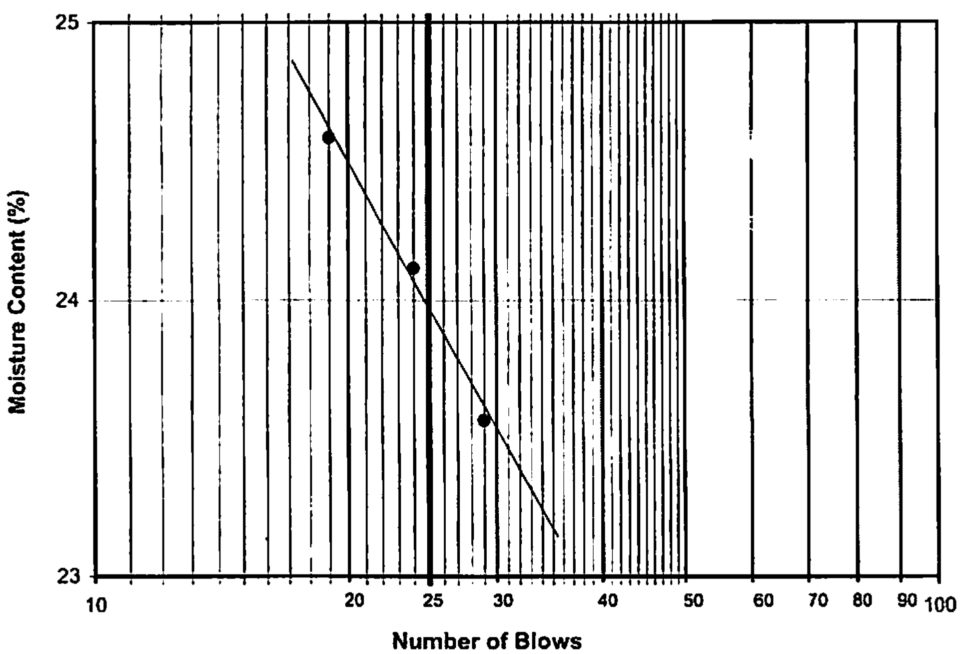
One - Point Liquid Limit Calculation

$LL = Wn(N/25)^{0.12}$



PROCEDURES USED

- [ ] Wet Preparation Multipoint - Wet
- [X] Dry Preparation Multipoint - Dry
- [X] Procedure A Multipoint Test
- [ ] Procedure B One-point Test



MODIFIED PROCTOR COMPACTION TEST

ASTM D 1557

Leighton

Project Name:	West Property / Colton	Tested By :	GEB	Date:	03/28/06
Project No.:	021906-001	Input By :	FT	Date:	03/29/06
Boring No.:	B-6	Depth (ft.)	0-5		
Sample No. :	B-1				
Soil Identification:	Dark Brown Silty Sand (SM)				

Preparation Method: [X] Moist [ ] Dry

[X] Mechanical Ram [ ] Manual Ram

Mold Volume (ft^3) 0.03319

Ram Weight = 10 lb.; Drop = 18 in.

TEST NO.	1	2	3	4	5	6
Wt. Compacted Soil + Mold (g)	3695.0	3818.0	3884.0	3837.0		
Weight of Mold (g)	1852.0	1852.0	1852.0	1852.0		
Net Weight of Soil (g)	1843.0	1966.0	2032.0	1985.0		
Wet Weight of Soil + Cont. (g)	556.00	528.60	510.80	518.90		
Dry Weight of Soil + Cont. (g)	534.30	496.70	471.90	467.90		
Weight of Container (g)	51.30	52.20	54.30	54.00		
Moisture Content (%)	4.49	7.18	9.32	12.32		
Wet Density (pcf)	122.4	130.6	135.0	131.9		
Dry Density (pcf)	117.2	121.8	123.5	117.4		

Maximum Dry Density (pcf) 123.5 **Optimum Moisture Content (%)** 9.5

PROCEDURE USED

[X] **Procedure A**
Soil Passing No. 4 (4.75 mm) Sieve
Mold : 4 in. (101.6 mm) diameter
Layers : 5 (Five)
Blows per layer : 25 (twenty-five)
May be used if +#4 is 20% or less

[ ] **Procedure B**
Soil Passing 3/8 in. (9.5 mm) Sieve
Mold : 4 in. (101.6 mm) diameter
Layers : 5 (Five)
Blows per layer : 25 (twenty-five)
Use if +#4 is >20% and +3/8 in. is 20% or less

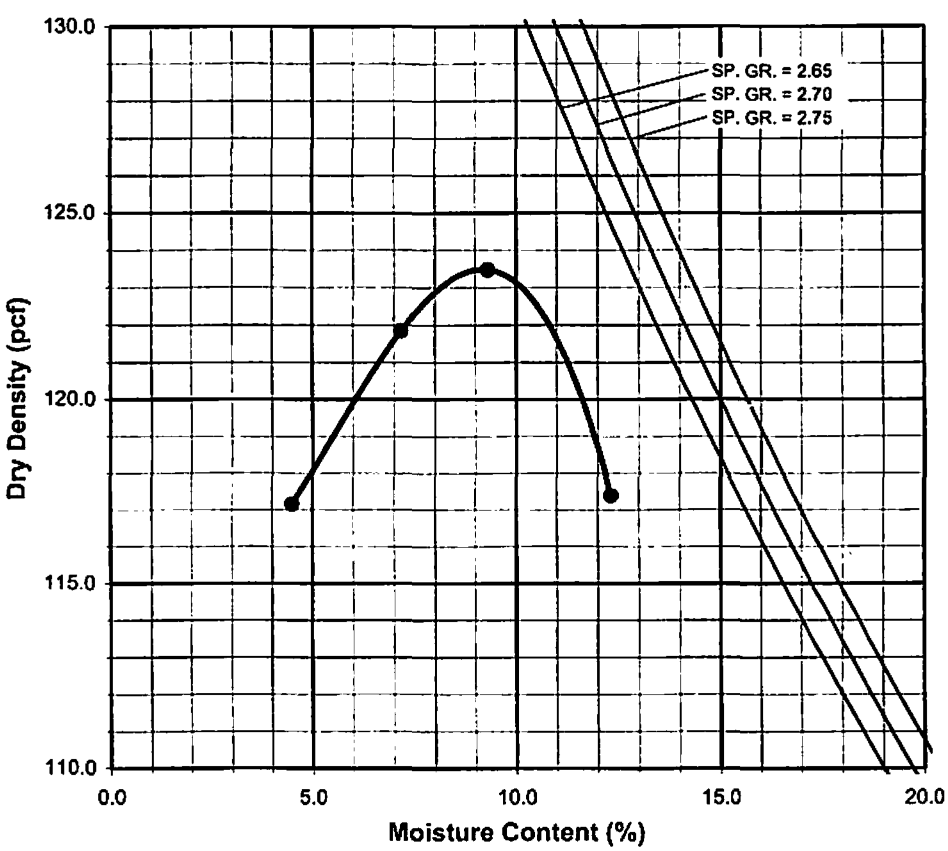
[ ] **Procedure C**
Soil Passing 3/4 in. (19.0 mm) Sieve
Mold : 6 in. (152.4 mm) diameter
Layers : 5 (Five)
Blows per layer : 56 (fifty-six)
Use if +3/8 in. is >20% and +¾ in. is <30%

Particle-Size Distribution:

GR:SA:FI

Atterberg Limits:

LL,PL,PI



MODIFIED PROCTOR COMPACTION TEST
ASTM D 1557

Leighton

Project Name:	West Property / Colton	Tested By :	GEB	Date:	03/28/06
Project No.:	021906-001	Input By :	JHW	Date:	03/31/06
Boring No.:	TP-13	Depth (ft.)	3-5		
Sample No. :	B-1				
Soil Identification:	Brown Poorly-graded Sand (SP)				

			Scalp Fraction (%)	
Preparation Method:	X	Moist	#3/4	
		Dry	#3/8	
Compaction Method	X	Mechanical Ram	#4	5.4
		Manual Ram		

Rammer Weight (lb.) = 10.0
Height of Drop (in.) = 18.0
Mold Volume (ft^3) 0.03319

TEST NO.	1	2	3	4	5	6
Wt. Compacted Soil + Mold (g)	3892.0	3977.0	3965.0			
Weight of Mold (g)	1852.0	1852.0	1852.0			
Net Weight of Soil (g)	2040.0	2125.0	2113.0			
Wet Weight of Soil + Cont. (g)	491.20	515.70	560.40			
Dry Weight of Soil + Cont. (g)	467.40	482.30	512.00			
Weight of Container (g)	52.40	55.00	54.00			
Moisture Content (%)	5.73	7.82	10.57			
Wet Density (pcf)	135.5	141.1	140.4			
Dry Density (pcf)	128.2	130.9	126.9			

Maximum Dry Density (pcf) 131.0 **Optimum Moisture Content (%)** 8.0

Corrected Dry Density (pcf) 132.5 **Corrected Moisture Content (%)** 7.5

X Procedure A
Soil Passing No. 4 (4.75 mm) Sieve
Mold : 4 in. (101.6 mm) diameter
Layers : 5 (Five)
Blows per layer : 25 (twenty-five)
May be used if +#4 is 20% or less

Procedure B
Soil Passing 3/8 in. (9.5 mm) Sieve
Mold : 4 in. (101.6 mm) diameter
Layers : 5 (Five)
Blows per layer : 25 (twenty-five)
Use if +#4 is >20% and +3/8 in. is 20% or less

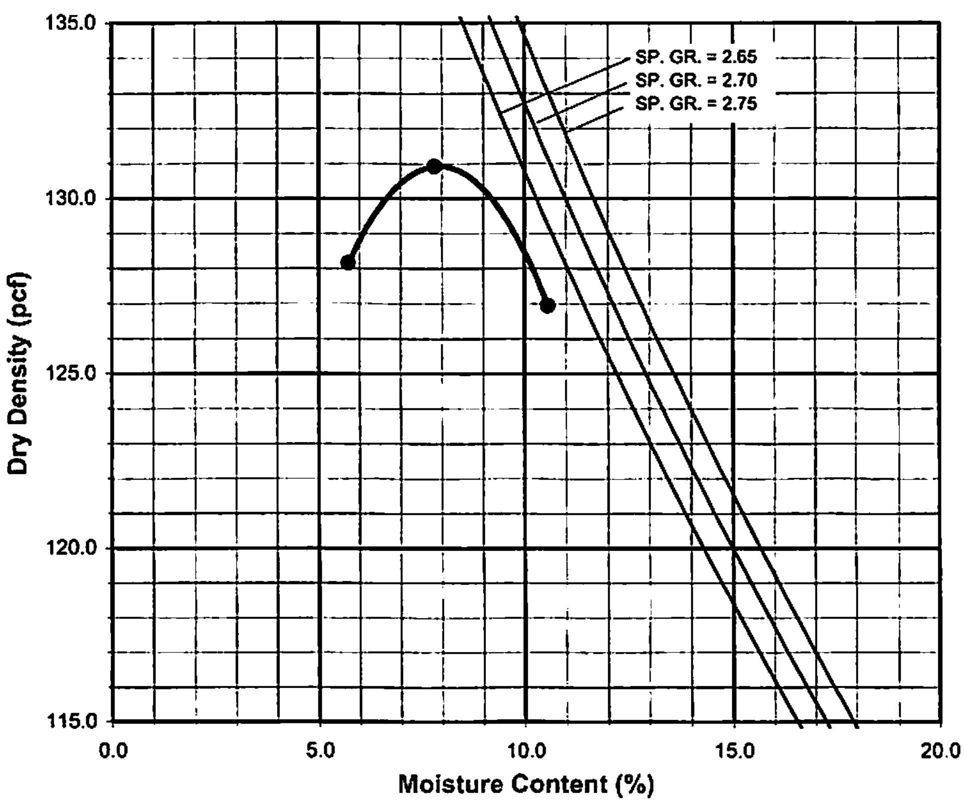
Procedure C
Soil Passing 3/4 in. (19.0 mm) Sieve
Mold : 6 in. (152.4 mm) diameter
Layers : 5 (Five)
Blows per layer : 56 (fifty-six)
Use if +3/8 in. is >20% and +¾ in. is <30%

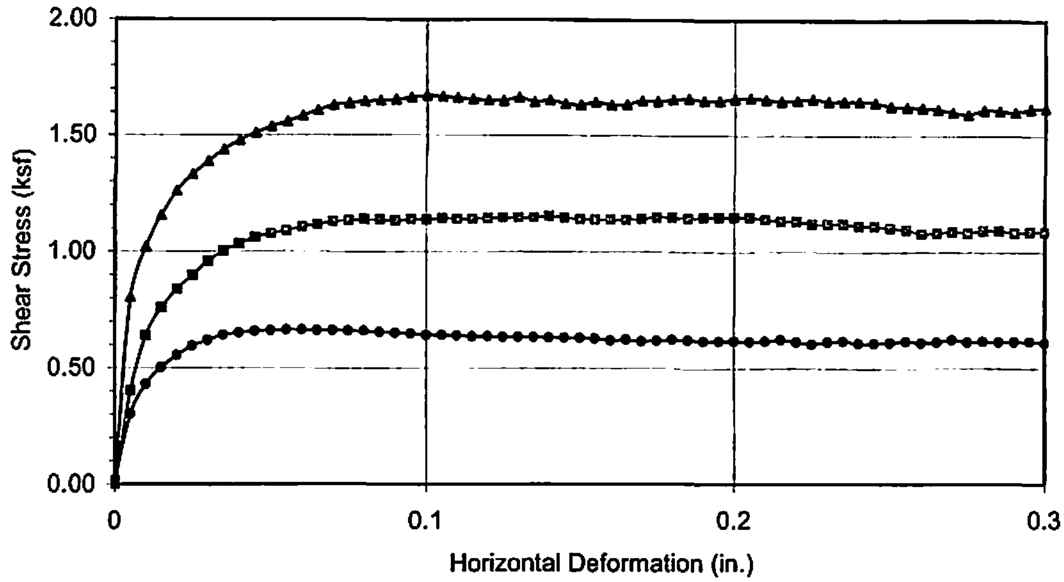
Particle-Size Distribution:

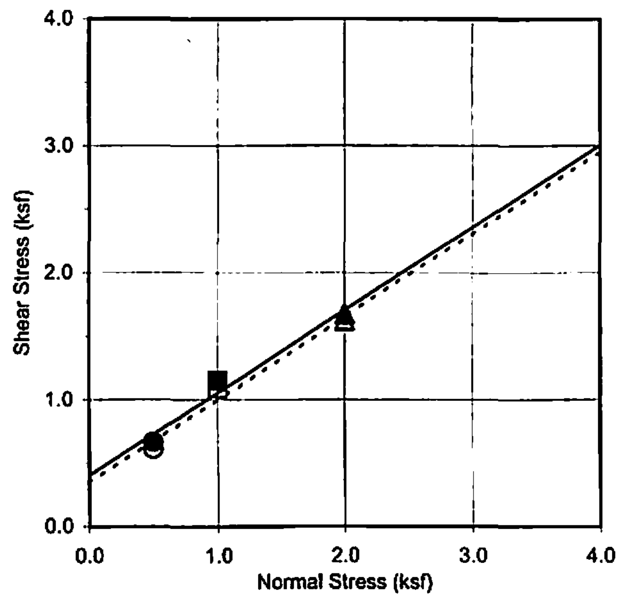
GR:SA:FI

Atterberg Limits:

LL,PL,PI







Boring No.	**TP-13**
Sample No.	**B-1**
Depth (ft)	**3-5**
Sample Type: 90% Remold	
Soil Identification: Brown Poorly-graded Sand (SP)	

Strength Parameters

	C (psf)	ϕ (°)
Peak	403.5	33.0
Ultimate	344.5	33.2

Normal Stress (kip/ft²)	0.500	1.000	2.000
Peak Shear Stress (kip/ft²)	● 0.664	■ 1.150	▲ 1.671
Shear Stress @ End of Test (ksf)	○ 0.611	□ 1.088	△ 1.621
Deformation Rate (in./min.)	0.0500	0.0500	0.0500
Initial Sample Height (in.)	1.000	1.000	1.000
Diameter (in.)	2.415	2.415	2.415
Initial Moisture Content (%)	8.00	8.00	8.00
Dry Density (pcf)	118.0	118.0	118.0
Saturation (%)	50.4	50.4	50.4
Soil Height Before Shearing (in.)	0.9925	0.9878	0.9879
Final Moisture Content (%)	14.9	14.7	14.4

Leighton

DIRECT SHEAR TEST RESULTS
Consolidated Undrained

Project No.: 021906-001

West Property / Colton

03-06

Leighton

EXPANSION INDEX of SOILS

ASTM D 4829

Project Name:	West Property / Colton	Tested By:	GB	Date:	04/15/06
Project No. :	021906-001	Checked By:	LF	Date:	04/17/06
Boring No.:	TP-1	Depth (ft.)	1-4		
Sample No. :	B-1				
Soil Identification:	Dark yellowish brown silty sand (SM)				

Dry Wt. of Soil + Cont.	(g)	1000.00
Wt. of Container No.	(g)	0.00
Dry Wt. of Soil	(g)	1000.00
Weight Soil Retained on #4 Sieve		0.00
Percent Passing # 4		100.00

MOLDED SPECIMEN		Before Test	After Test
Specimen Diameter	(in.)	4.01	4.01
Specimen Height	(in.)	1.0000	1.0025
Wt. Comp. Soil + Mold	(g)	591.00	434.70
Wt. of Mold	(g)	181.40	0.00
Specific Gravity (Assumed)		2.70	2.70
Container No.		O	O
Wet Wt. of Soil + Cont.	(g)	831.10	616.10
Dry Wt. of Soil + Cont.	(g)	766.00	558.90
Wt. of Container	(g)	0.00	181.40
Moisture Content	(%)	8.50	15.15
Wet Density	(pcf)	123.6	130.8
Dry Density	(pcf)	113.9	113.6
Void Ratio		0.480	0.484
Total Porosity		0.325	0.326
Pore Volume	(cc)	67.2	67.7
Degree of Saturation (%) [S_{meas}]		**47.8**	84.5

SPECIMEN INUNDATION in distilled water for the period of 24 h or expansion rate < 0.0002 in./h

Date	Time	Pressure (psi)	Elapsed Time (min.)	Dial Readings (in.)
04/15/06	9:57	1.0	0	0.5770
04/15/06	10:07	1.0	10	0.5770
Add Distilled Water to the Specimen				
04/15/06	11:25	1.0	78	0.5785
04/17/06	6:35	1.0	2668	0.5795
04/17/06	7:45	1.0	2738	0.5795

Expansion Index (EI_{meas})	= ((Final Rdg - Initial Rdg) / Initial Thick.) x 1000	2.5
Expansion Index (EI)$_{50}$	= EI_{meas} - (50 - S_{meas})x((65+EI_{meas}) / (220-S_{meas}))	**2**

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One-Dimensional Swell or Settlement Potential of Cohesive Soils
(ASTM D 4546)

Project Name:	West Property / Colton	Tested By:	FT, ESS	Date:	03/27/06
Project No.:	021906-001	Checked By:	JHW	Date:	03/30/06
Boring No.:	B-1	Sample Type:	Drive		
Sample No.:	R-2	Depth (ft.)	5.0		
Sample Description:	Brown Sandy Silt s(ML)				

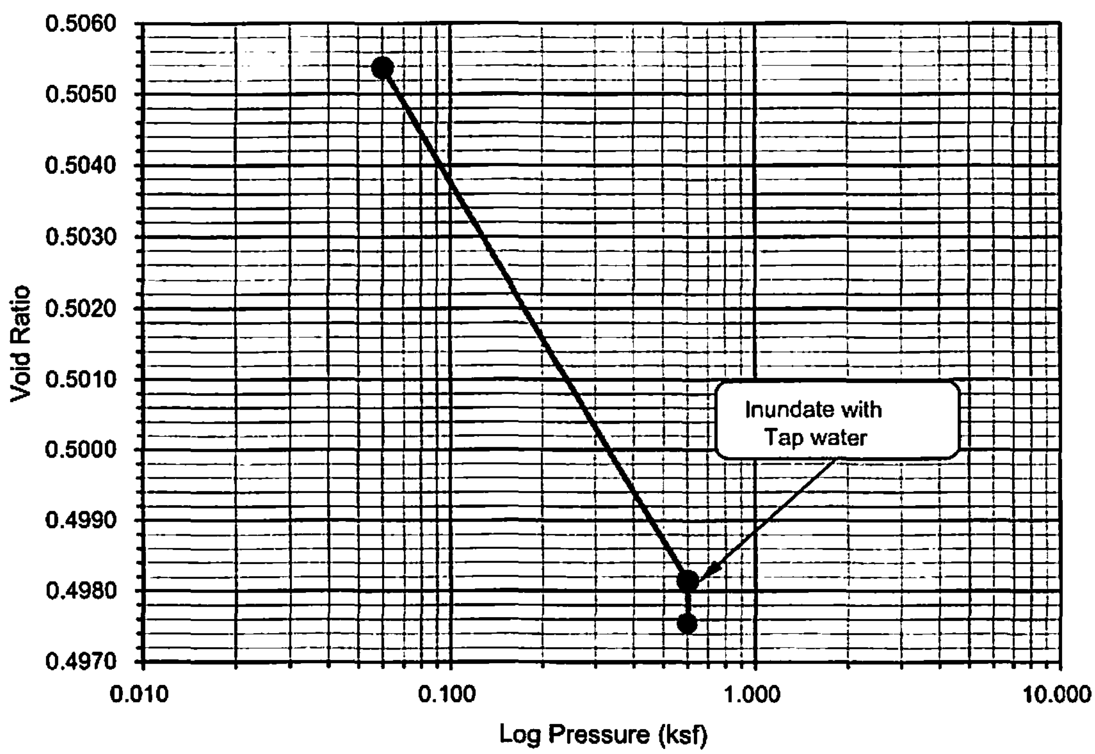
Initial Dry Density (pcf):	112.0
Initial Moisture (%):	8.40
Initial Length (in.):	1.0000
Initial Dial Reading:	0.1254
Diameter(in):	2.416

Final Dry Density (pcf):	110.5
Final Moisture (%) :	18.4
Initial Void ratio:	0.5054
Specific Gravity(assumed):	2.70
Initial Saturation (%)	44.9

Pressure (p) (ksf)	Final Reading (in)	Apparent Thickness (in)	Load Compliance (%)	Swell (+) Settlement (-) % of Sample Thickness	Void Ratio	Corrected Deformation (%)
0.060	0.1254	1.0000	0.00	0.00	0.5054	0.00
0.600	0.1302	0.9952	0.00	-0.48	0.4981	-0.48
H2O	0.1306	0.9948	0.00	-0.52	0.4975	-0.52

Percent Swell (+) / Settlement (-) After Inundation = -0.04

Void Ratio - Log Pressure Curve



Collapse B-1, R-2 @ 5

Leighton

One-Dimensional Swell or Settlement Potential of Cohesive Soils
(ASTM D 4546)

Project Name: West Property / Colton
Project No.: 021906-001
Boring No.: B-5
Sample No.: R-2
Sample Description: Brown Silty Sand / Silty, Clayey Sand (SM) / (SC-SM)

Tested By: FT, ESS Date: 03/27/06
Checked By: JHW Date: 03/30/06
Sample Type: Drive
Depth (ft.) 5.0

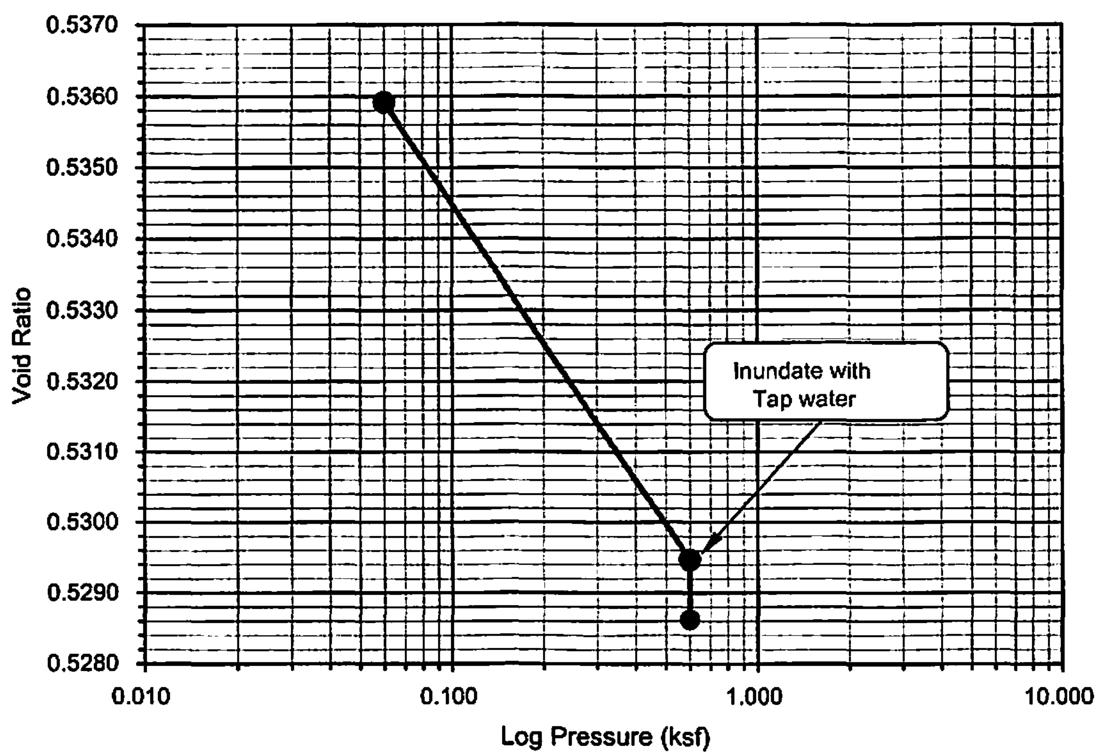
Initial Dry Density (pcf):	109.7
Initial Moisture (%):	4.88
Initial Length (in.):	1.0000
Initial Dial Reading:	0.1485
Diameter(in):	2.416

Final Dry Density (pcf):	109.0
Final Moisture (%) :	18.4
Initial Void ratio:	0.5359
Specific Gravity(assumed):	2.70
Initial Saturation (%)	24.6

Pressure (p) (ksf)	Final Reading (in)	Apparent Thickness (in)	Load Compliance (%)	Swell (+) Settlement (-) % of Sample Thickness	Void Ratio	Corrected Deformation (%)
0.060	0.1485	1.0000	0.00	0.00	0.5359	0.00
0.600	0.1527	0.9958	0.00	-0.42	0.5295	-0.42
H2O	0.1533	0.9953	0.00	-0.48	0.5286	-0.48

Percent Swell (+) / Settlement (-) After Inundation = -0.06

Void Ratio - Log Pressure Curve



Collapse B-5, R-2 @ 5

One-Dimensional Swell or Settlement Potential of Cohesive Soils
(ASTM D 4546)

Leighton

Project Name:	West Property / Colton	Tested By:	FT, ESS	Date:	03/27/06
Project No.:	021906-001	Checked By:	JHW	Date:	03/30/06
Boring No.:	B-6	Sample Type:	Drive		
Sample No.:	R-2	Depth (ft.)	5.0		

Sample Description: Brown Silty Sand / Sandy Silt (SM) / s(ML)

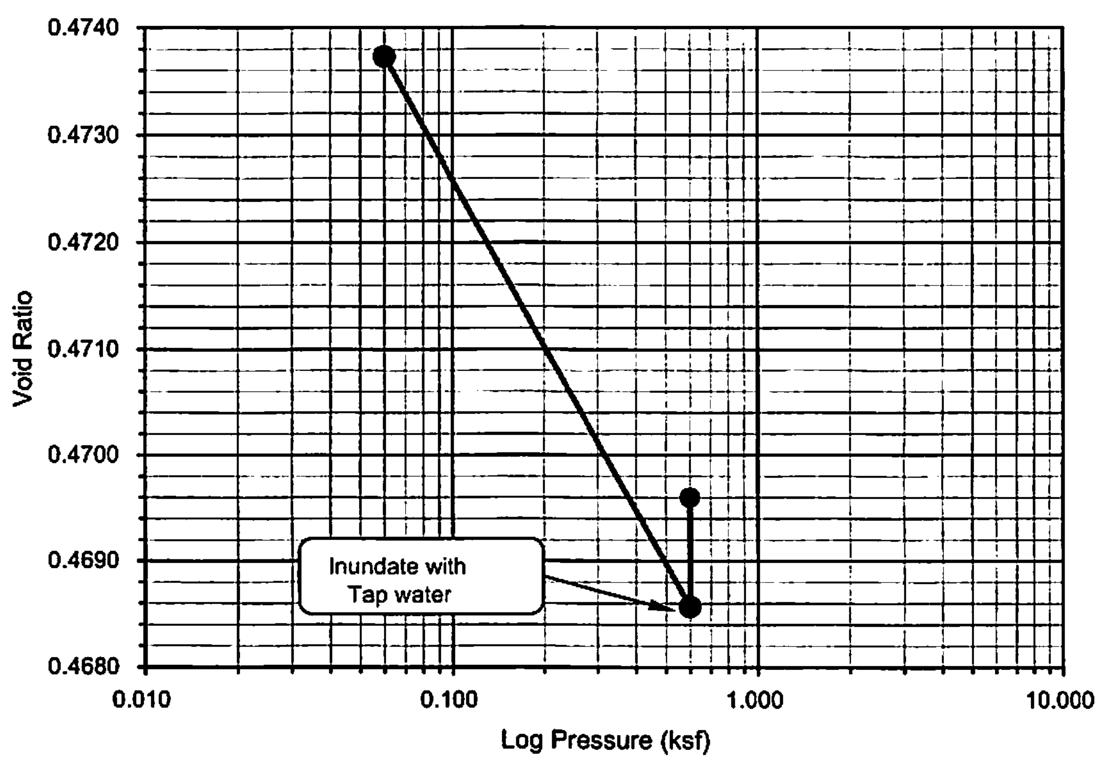
Initial Dry Density (pcf):	114.3
Initial Moisture (%):	6.23
Initial Length (in.):	1.0000
Initial Dial Reading:	0.1177
Diameter(in):	2.416

Final Dry Density (pcf):	113.7
Final Moisture (%) :	18.1
Initial Void ratio:	0.4745
Specific Gravity(assumed):	2.70
Initial Saturation (%)	35.5

Pressure (p) (ksf)	Final Reading (in)	Apparent Thickness (in)	Load Compliance (%)	Swell (+) Settlement (-) % of Sample Thickness	Void Ratio	Corrected Deformation (%)
0.060	0.1182	0.9995	0.00	-0.05	0.4737	-0.05
0.600	0.1217	0.9960	0.00	-0.40	0.4686	-0.40
H2O	0.1210	0.9967	0.00	-0.33	0.4696	-0.33

Percent Swell (+) / Settlement (-) After Inundation = 0.07

Void Ratio - Log Pressure Curve



Leighton

One-Dimensional Swell or Settlement Potential of Cohesive Soils
(ASTM D 4546)

Project Name:	West Property / Colton	Tested By:	FT, ESS	Date:	03/27/06
Project No.:	021906-001	Checked By:	JHW	Date:	03/30/06
Boring No.:	B-4	Sample Type:	Drive		
Sample No.:	R-2	Depth (ft.)	5.0		
Sample Description:	Brown Silty, Clayey Sand (SC-SM)				

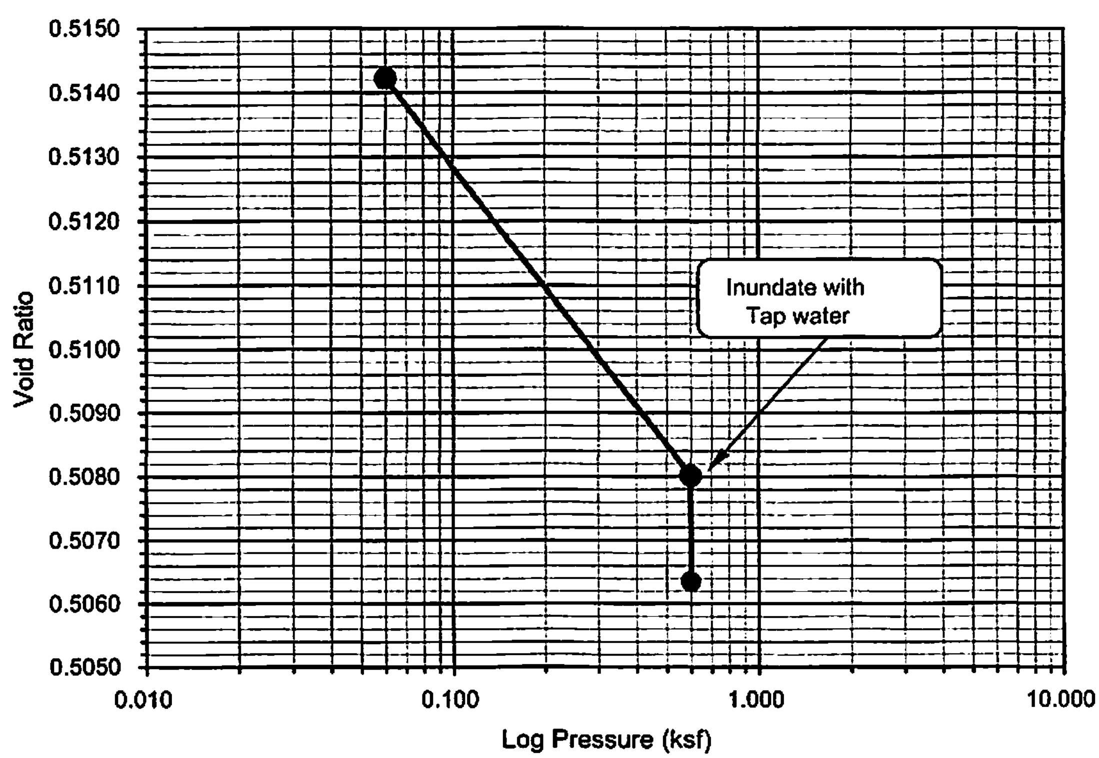
Initial Dry Density (pcf):	111.3
Initial Moisture (%):	7.12
Initial Length (in.):	1.0000
Initial Dial Reading:	0.1520
Diameter(in):	2.416

Final Dry Density (pcf):	111.5
Final Moisture (%) :	17.2
Initial Void ratio:	0.5144
Specific Gravity(assumed):	2.70
Initial Saturation (%)	37.4

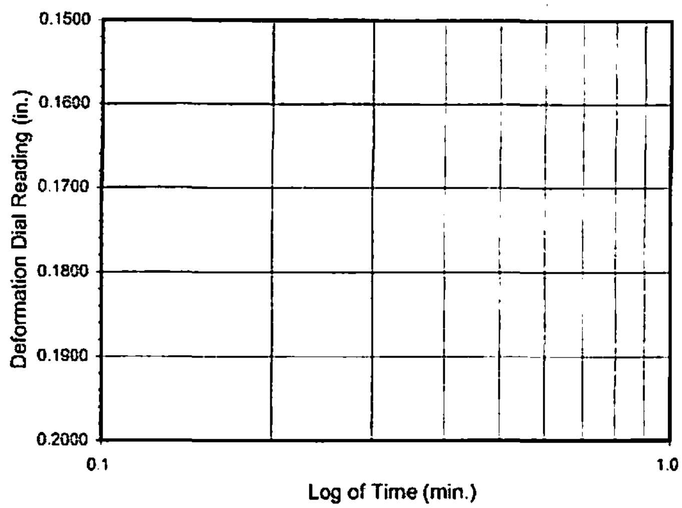
Pressure (p) (ksf)	Final Reading (in)	Apparent Thickness (in)	Load Compliance (%)	Swell (+) Settlement (-) % of Sample Thickness	Void Ratio	Corrected Deformation (%)
0.060	0.1521	0.9999	0.00	-0.01	0.5142	-0.01
0.600	0.1562	0.9958	0.00	-0.42	0.5080	-0.42
H2O	0.1573	0.9947	0.00	-0.53	0.5063	-0.53

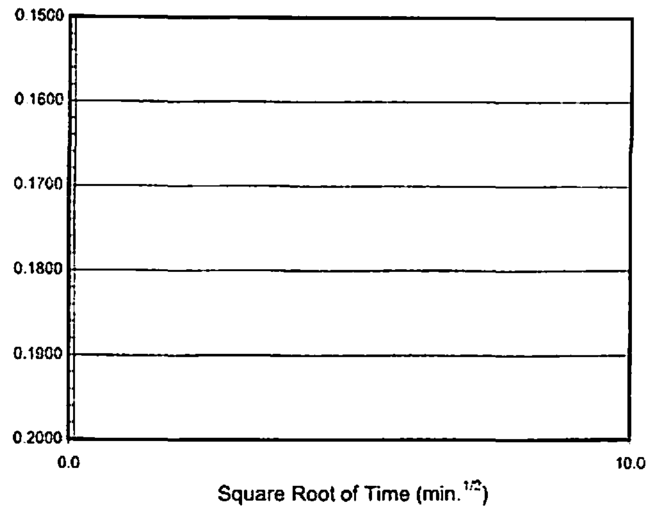
Percent Swell (+) / Settlement (-) After Inundation = -0.11

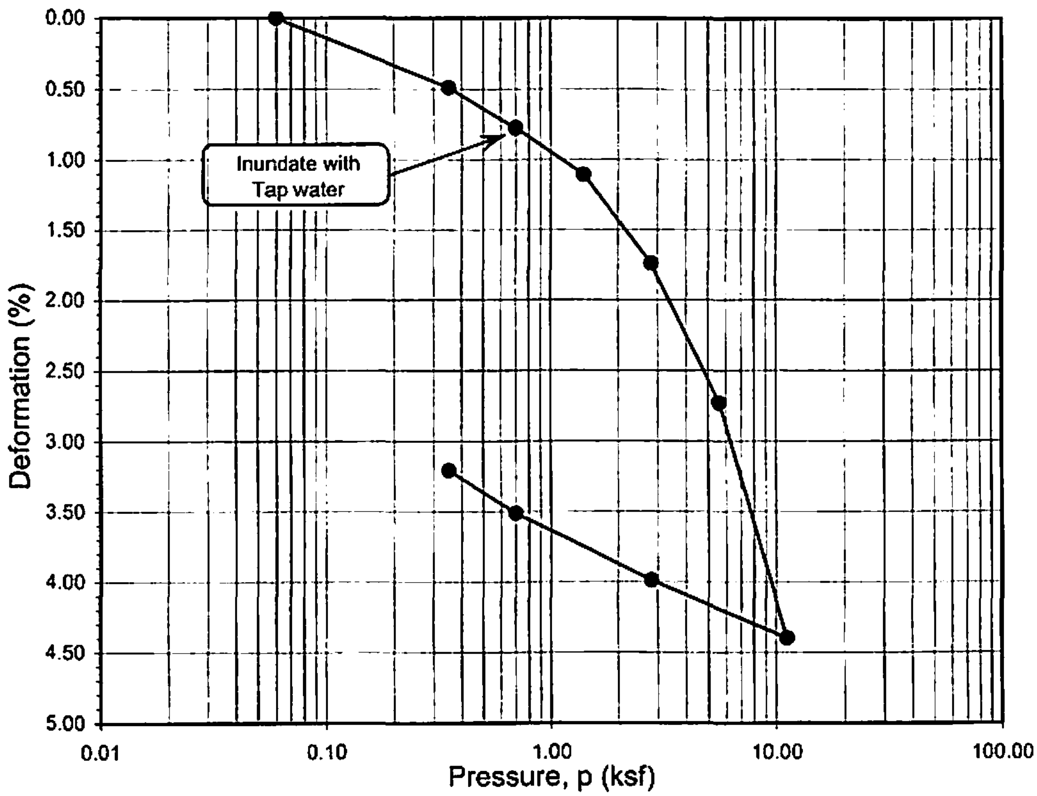
Void Ratio - Log Pressure Curve



No Time Readings







Boring No.	Sample No.	Depth (ft.)	Moisture Content (%)		Dry Density (pcf)		Void Ratio		Degree of Saturation (%)	
			Initial	Final	Initial	Final	Initial	Final	Initial	Final
B-3	**R-2**	**5**	**9.7**	**16.8**	**109.5**	**112.9**	**0.540**	**0.490**	**48**	**92**

Soil Identification: Brown Silty Sand / Silty Clayey Sand (SM) / (SC-SM)

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ONE-DIMENSIONAL CONSOLIDATION PROPERTIES of SOILS
(ASTM D 2435)

Project No.: 021906-001

West Property / Colton

03-06

Leighton

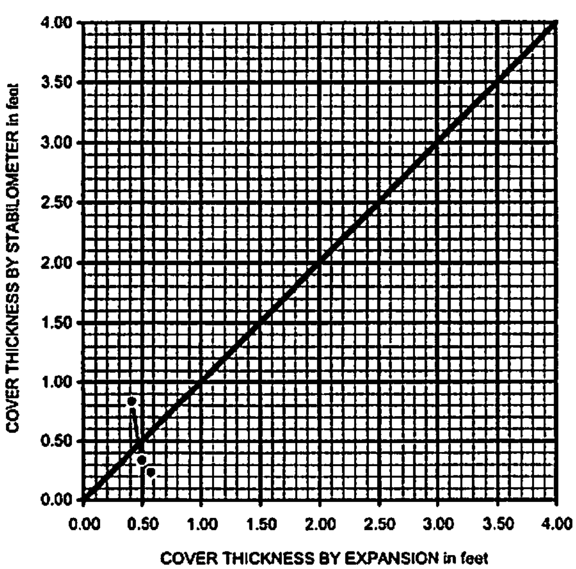
R-VALUE TEST RESULTS

PROJECT NAME:	W Propety / Colton	PROJECT NUMBER:	021906-001
SAMPLE NUMBER:	B-1	SAMPLE LOCATION:	B-6 0-5'
SAMPLE DESCRIPTION:	SM	TECHNICIAN:	SCF
		DATE SAMPLED	3/28/2006

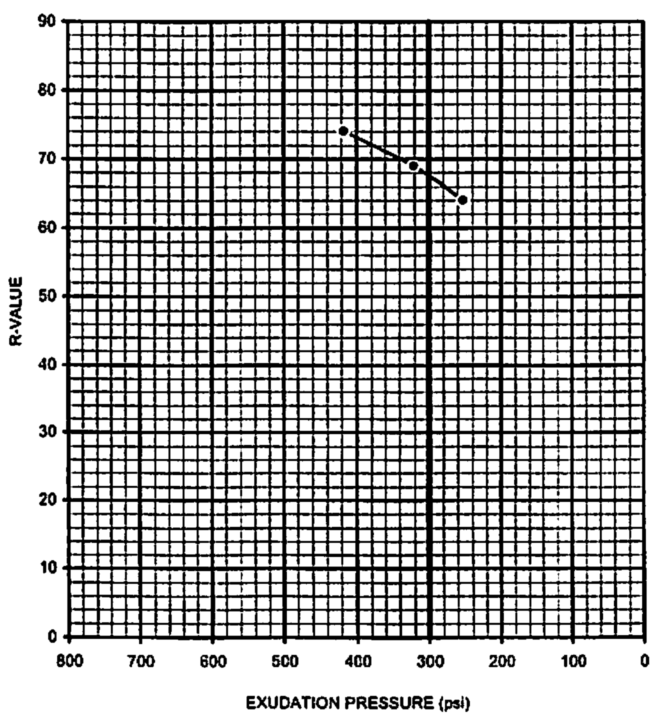
TEST SPECIMEN	a	b	c
MOISTURE AT COMPACTION %	10.5	10.6	10.9
HEIGHT OF SAMPLE, Inches	2.56	2.57	2.44
DRY DENSITY, pcf	122.6	122.2	123.6
COMPACTOR PRESSURE, psi	200	160	125
EXUDATION PRESSURE, psi	417	321	253
EXPANSION, Inches x 10exp-4	25	10	7
STABILITY Ph 2,000 lbs (160 psi)	27	32	36
TURNS DISPLACEMENT	4.40	4.47	4.52
R-VALUE UNCORRECTED	74	69	66
R-VALUE CORRECTED	74	69	64

DESIGN CALCULATION DATA	a	b	c
GRAVEL EQUIVALENT FACTOR	1.0	1.0	1.0
TRAFFIC INDEX	5.0	5.0	5.0
STABILOMETER THICKNESS, ft.	0.42	0.50	0.58
EXPANSION PRESSURE THICKNESS, ft.	0.83	0.33	0.23

EXPANSION PRESSURE CHART



EXUDATION PRESSURE CHART



R-VALUE BY EXPANSION:	71
R-VALUE BY EXUDATION:	68
EQUILIBRIUM R-VALUE:	68

Leighton

SOIL RESISTIVITY TEST

DOT CA TEST 532 / 643

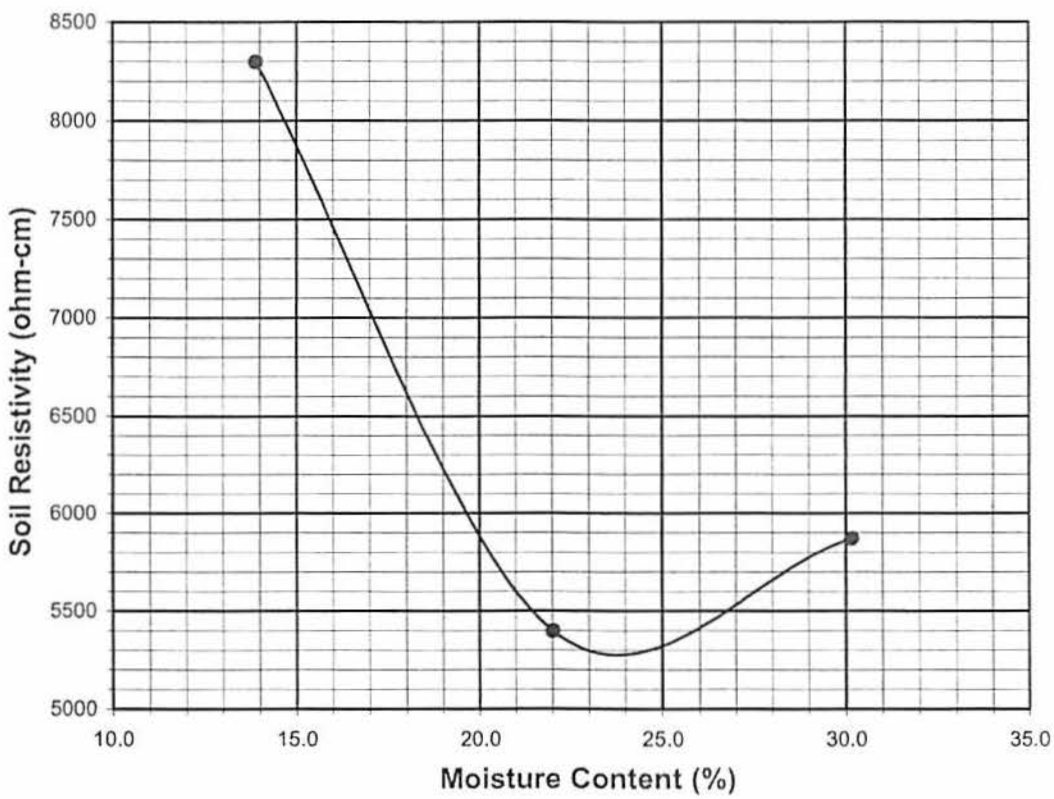
Project Name: West Property / Colton
Project No. : 021906-001
Boring No.: TP-13
Sample No. : B-1
Soil Identification: SP

Tested By : VJ Date: 04/03/06
Data Input By: LF Date: 04/06/06
Depth (ft.) : 3-5

Specimen No.	Water Added (ml) (Wa)	Adjusted Moisture Content (MC)	Resistance Reading (ohm)	Soil Resistivity (ohm-cm)
1	100	13.89	1230	8298
2	200	22.03	800	5397
3	300	30.16	870	5869
4				
5				

Moisture Content (%) (MCi)	5.76
Wet Wt. of Soil + Cont. (g)	182.97
Dry Wt. of Soil + Cont. (g)	176.07
Wt. of Container (g)	56.20
Container No.	
Initial Soil Wt. (g) (Wt)	1300.00
Box Constant	6.746
MC =(((1+Mci/100)x(Wa/Wt+1))-1)x100	

Min. Resistivity (ohm-cm)	Moisture Content (%)	Sulfate Content (ppm)	Chloride Content (ppm)	Soil pH	
				pH	Temp. (°C)
DOT CA Test 532 / 643		DOT CA Test 417 Part II	DOT CA Test 422	DOT CA Test 532 / 643	
5300	**23.8**	**144**	**42**	**7.85**	**21.6**



SEISMIC REFRACTION SURVEY

WEST PROPERTY PROJECT

CITY OF COLTON, CALIFORNIA

Project No. 262060-1

March 29, 2006

Prepared for:

Leighton and Associates, Inc.
10532 Acacia Street, Suite B-6
Rancho Cucamonga, CA 91730

TERRA GEOSCIENCES

Engineering Geology • Geophysics • Geotechnical Applications

P.O. Box 1099, Loma Linda, CA 92354 • 909-796-4667

Leighton and Associates, Inc.
10532 Acacia Street, Suite B-6
Rancho Cucamonga, CA 91730

Attention: Mr. Dan Jankly, Senior Project Geologist

Regarding: Seismic Refraction Survey
West Property Project
City of Colton, California
L & A Project No. 021906-001

INTRODUCTION

As requested, this firm has performed a geophysical survey using the seismic refraction method for the above-referenced site, along selected areas as delineated by you. 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 along local areas which could possibly indicate areas of potential excavation difficulties, and also to aid in evaluating the subsurface structure and seismic velocity distribution. The bedrock materials as mapped by Morton (1978) are comprised of Cretaceous age granitic rocks, generally described as being a light colored, coarse-grained, porphyritic foliated biotite quartz monzonite along the south, with coarse-grained gray biotite-hornblende quartz diorite along the west.

Representative Layer Velocity Profiles for each seismic line have been prepared and are presented in Appendix A, which indicates their respective "weighted average" subsurface velocities in generalized layers. In addition, associated Tomographic Models have also been prepared for comparative purposes, which generally indicate the relative structure and velocity distribution for each seismic line, and are presented within Appendix B. We understand that this report will be included as a technical appendix to your report, therefore as requested, the locations of our geophysical survey lines were transferred onto your field map for inclusion onto your final map.

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 seismic refraction survey by a State of California Professional Geophysicist, to include six seismic traverses along selected portions of the site.**
- **Preparation of this report, presenting our findings and conclusions with respect to the velocity characteristics and the expected rippability potentials of the subsurface earth materials.**

Accompanying Appendices

Appendix A - Layer Velocity Profiles
Appendix B - 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

Five, 195-foot long 12-channel and one, 300-foot long 24-channel seismic refraction survey lines were performed with a target depth of 40- to 50±-feet. A 16-pound sledge-hammer was used as an energy source to produce the seismic waves and twelve or twenty-four, 14-Hz geophones (with 70% damping), were spaced at 12- to 15-foot intervals along the traverse lines to detect both the direct and refracted waves. The seismic wave arrivals were recorded on a Geometrics StrataVisor™ NX model signal enhancement refraction seismograph. Seven shot points were utilized along each seismic line spread using offset, forward, reverse, and intermediate locations, in order to obtain sufficient data for velocity analysis and depth modeling purposes. Each geophone and shot location was surveyed using a hand level and ruler for relative topographic correction. During acquisition, the seismograph provides both a hard copy and screen display of the seismic wave arrival, of which are digitally recorded on the in-board seismograph computer and subsequently transferred to a disk and downloaded into our office computer for further processing, analyzing, and printing purposes.

Data Reduction

The data on the paper record and/or display screen were used to analyze the arrival time of the primary seismic "P"-waves at each geophone station, in the form of a wiggle trace, or wave travel-time curve, for quality control purposes in the field. All of the recorded data was transferred to our office computer for further processing, analyzing, and printing purposes, using the computer programs **SIP** (Seismic refraction Interpretation Program) developed by Rimrock Geophysics, Inc. (1995), and **Rayfract™** (Intelligent Resources, Inc., 1996-2005). **SIP** 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). In addition, **Rayfract™** was also used for comparative purposes. **Rayfract™** is seismic refraction tomography software that models subsurface refraction, transmission, and diffraction of acoustic waves. Both computer programs perform their analysis using exactly the same input data, which includes first-arrival P-waves and line geometry.

SUMMARY OF GEOPHYSICAL INTERPRETATION

It is important to understand that the data obtained during this survey represent an average of seismic velocities within any given layer. For example, high seismic velocity boulders/dikes or 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. Due to the geologic character of the area with respect to boulder outcrops in the region, this condition at depth within the subject property may be present. In general, the upper 40- to 50±-feet of the site where locally surveyed was noted to be characterized by three major subsurface layers (**SIP** analysis, see Appendix A) with respect to seismic velocities and are generally described below:

- Velocity Layer V1: This uppermost velocity layer (V1) is most likely comprised of topsoil, colluvium, aeolian deposits, and/or highly-weathered and fractured bedrock materials. This layer has an average weighted velocity ranging from 1,266 to 1,756 fps, which is typical for these types of near-surface materials.

- Velocity Layer V2: The second layer (V2) yielded a narrow range of seismic velocities between 4,032 to 4,830 fps, indicating moderate degrees of weathering. These velocities are typical for the near surface weathered zone commonly found in granitic rocks in the southern California region. This velocity zone may also include scattered buried fresh large boulders and/or dikes that are surrounded by a highly decomposed matrix.

- Velocity Layer V3: The third layer (V3) indicated a wide range of velocities of 7,026 to 11,625 fps. These velocities also indicate the possibility of granitic rock with abundant widely-scattered buried fresh large boulders and/or dikes within a moderately decomposed matrix or more likely a relatively fresher crystalline bedrock matrix with wider-spaced fractures.

It is also important to note that the seismic velocities obtained in bedrock are influenced by the nature and character of the localized major structural discontinuities (bedding, foliation, fracturing, etc.). 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. Therefore, the seismic velocities obtained during our field study and as discussed below, should be considered minimum velocities at this time, as the structure of the bedrock locally is not known.

Using **Rayfract™**, a tomographic model for each seismic line was also prepared and analyzed for comparative purposes, as presented in Appendix B, which generally indicates the relative structure and velocity distribution. Although no discrete velocity layers or boundaries are created, these models generally resemble the **SIP** analysis. **Rayfract™** allows imaging of subsurface velocity using first break energy propagation modeling. An initial 1D gradient model is created using the Delta-t-V method which gives a good initial fit between modeled and picked first breaks. This initial model is then refined automatically with a true 2D WET (Wavepath Eikonal Traveltime) tomographic inversion. 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.

It can be seen in these tomographic models that the seismic velocity (i.e., hardness) of the bedrock gradually increases with depth which is most likely the representative condition of the subsurface materials. It was also noted that for the most part, the seismic velocities on the Layer Velocity Profiles (Appendix A) appears to generally correlate with the average of the velocity gradients as shown on the Tomographic Models (Appendix B).

GENERALIZED RIPPABILITY CHARACTERISTICS OF GRANITICS

A summary of the generalized rippability characteristics of granitic bedrock has been provided in order to aid in evaluating potential excavation difficulties with respect to the seismic velocities obtained during our geophysical survey. For reference purposes, a summary of excavation considerations has been included within Appendix C in order to provide the client and contractor with a better understanding of the complexities of excavation within granitic bedrock materials. The seismic velocity ranges that are described below are considered to be approximate and assume typical, good-working, heavy excavation equipment, such as single shank or D9R dozer, such as described by Caterpillar, Inc. (2000 and 2004); however, different excavating equipment (i.e., trenching equipment) may not correlate well with these velocity ranges.

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

  This velocity range indicates rippable materials which may consist of alluvial deposits and decomposed granitics, with random hardrock floaters. These materials will break down into slightly silty, well-graded sand, 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.

- **Marginally Rippable Condition (4,000 - 8,000 ft/sec)**:

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

- **Non-Rippable Condition (8,000 ft/sec or greater)**:

  This velocity range includes non-rippable material consisting primarily of moderately fractured granitics 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.

SUMMARY OF FINDINGS AND CONCLUSIONS

The raw field data was considered to be of fair to good quality which had minor to moderate amounts of ambient "noise" that was introduced during our survey from overhead power lines (for S-1 and S-2), wind sources, and distant ground vibrations from the nearby railroad tracks and roadways. Analysis of the data and picking of the primary "P"-wave arrivals was performed with minor to moderate difficulty and some interpolation of data was necessary. Based on the results of our comparative seismic analyses of both **SIP** and **Rayfract™** (of which both software programs use exactly the same input data), the seismic refraction survey lines appear to generally coincide with one another, with some minor variances due to the methods that these programs process and integrate the input data. Anticipation of gradual increasing hardness with depth and lateral variations, with respect to excavation characteristics, should be anticipated across the subject site. The anticipated excavation potentials of the **SIP** analysis velocity layers encountered locally during our survey are as follows:

- <u>Velocity Layer V1</u>:

  No major excavating difficulties are expected within the uppermost, low seismic velocity layer V1 (velocity range of 1,266 to 1,756 fps). This surficial layer is expected to be comprised of topsoil, colluvium, aeolian deposits, and/or very highly-weathered and fractured bedrock materials. However, some fresher boulders should be anticipated to be encountered based on surface exposures of bedrock outcrops in the general region.

- <u>Velocity Layer V2</u>:

  The second layer V2 is believed to consist of moderately to moderately weathered bedrock (velocity range of 4,032 to 4,830 fps) and should excavate with minor to moderate difficulty, assuming appropriate good-working equipment for the proposed type of excavation. Isolated floaters (i.e., boulders, corestones, etc.) are presumably present and could produce difficult excavation conditions locally. Placement of infrastructures in this material may also be difficult in local areas. Although not anticipated, localized blasting in these materials cannot be completely ruled out, due to the presence of any encountered fresh, buried boulders/dikes.

- <u>Velocity Layer V3</u>:

  Areas of bedrock materials within the underlying higher velocity V3 layer (ranging from 7,026 to 11,625 fps) are anticipated to have excavation difficulties. Hard excavating areas consisting of localized buried boulders and/or relatively fresher homogeneous bedrock with wide-spaced fracturing will most likely be encountered during both remedial grading and placement of infrastructures, which may require minor blasting to achieve desired grade.

It was found that Seismic Lines S-1 through S-3 all had three velocity layers wherein Seismic Line S-4 through S-6 only indicated two velocity layers to the depths explored. Based on the geologic mapping by Morton (1978) and the data obtained during our survey, there appear to be two different rock types present within the site which have varying velocity characteristics. In summary, the results of this seismic refraction survey are to be considered as an aid in assessing the rippability potentials of the earth materials locally. This information should be carefully reviewed by the grading contractor and representative "test" excavations should be considered, so that they may be correlated with the data presented within this report.

CLOSURE

This survey was performed using "state of the art" geophysical techniques, computer processing, and equipment, in the localized areas delineated by you. We make no warranty, either expressed or implied. It should be noted that our data was obtained along only six specific areas; therefore, other local areas at the site beyond the limits of our seismic lines may contain different velocity layers and depths not encountered during our field survey. The Excavation Considerations provided within Appendix C should be understood so that proper planning and excavation techniques can be employed by the selected grading contractor. It should be understood that when using these theoretical geophysical principles and techniques, sources of error are possible in both the data obtained and in the interpretation.

If you should have any questions regarding this report or do not understand the limitations of this survey, please do not hesitate to contact our office.

Respectfully submitted,
TERRA GEOSCIENCES

Donn C. Schwartzkopf
Principal Geophysicist
PGP 1002

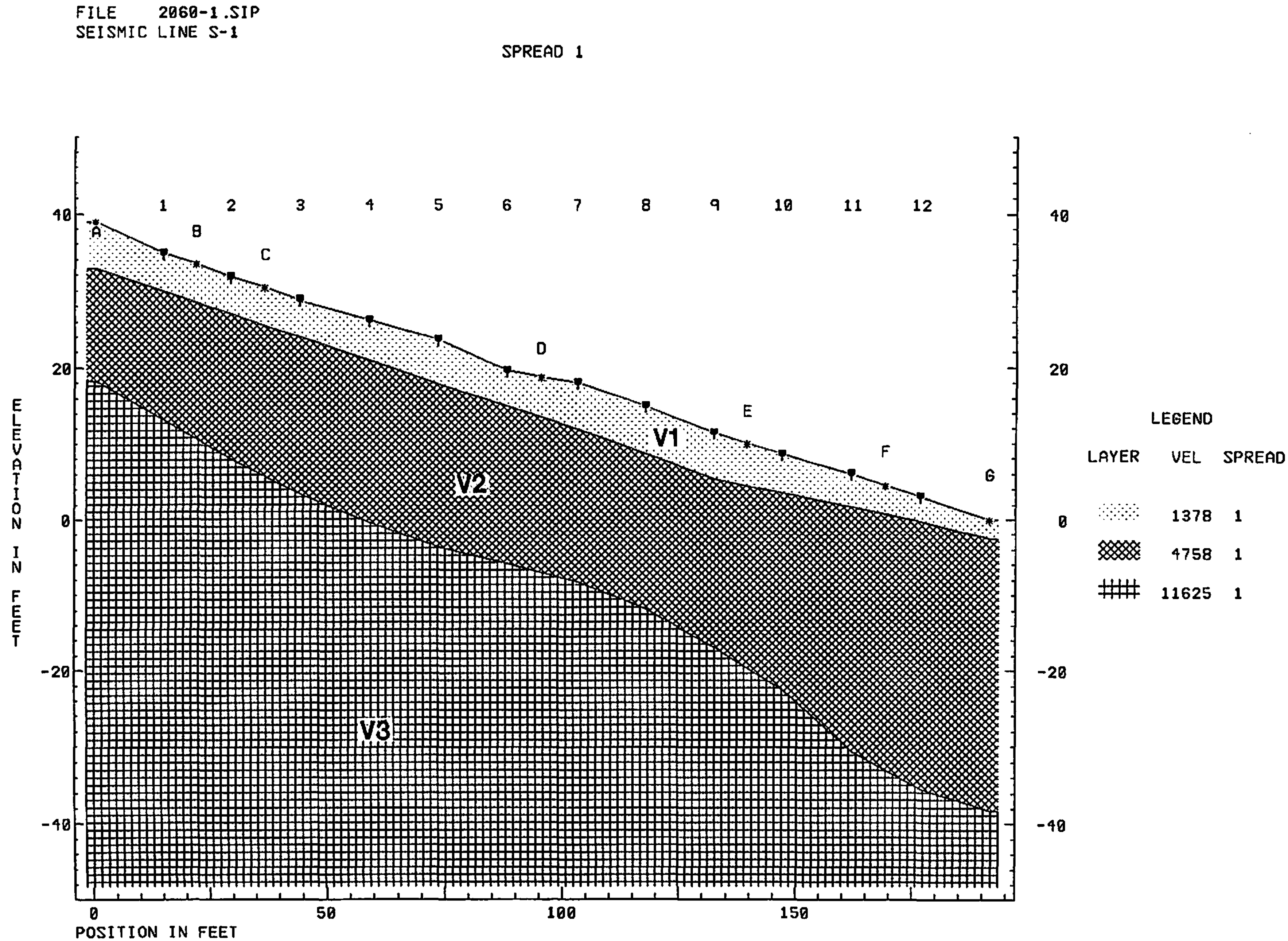
PROFESSIONAL GEOPHYSICIST
DONN C. SCHWARTZKOPF
No. 1002
STATE OF CALIFORNIA

APPENDIX A

LAYER VELOCITY PROFILES

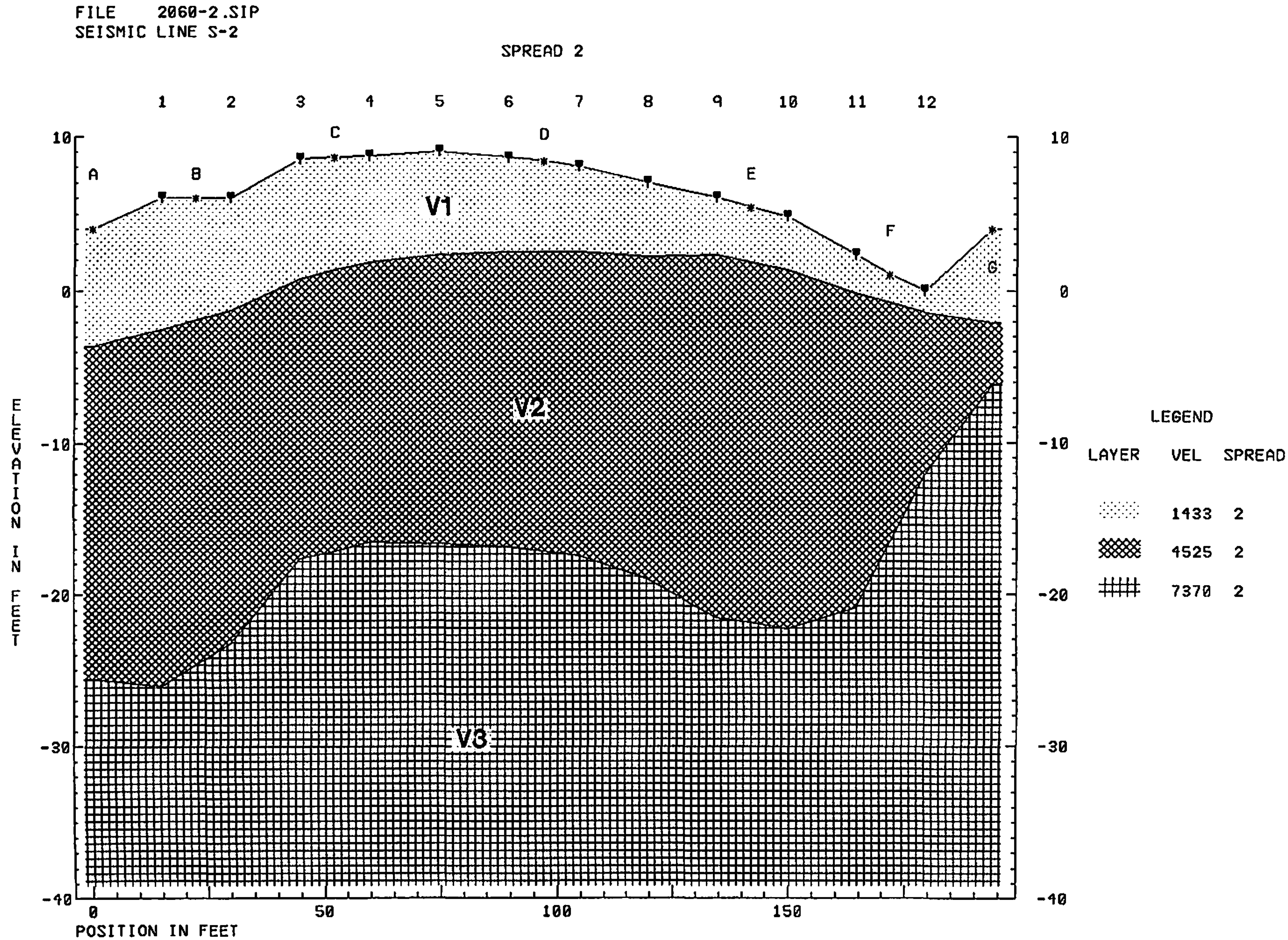
LAYER VELOCITY PROFILE S-1

South 85° East →



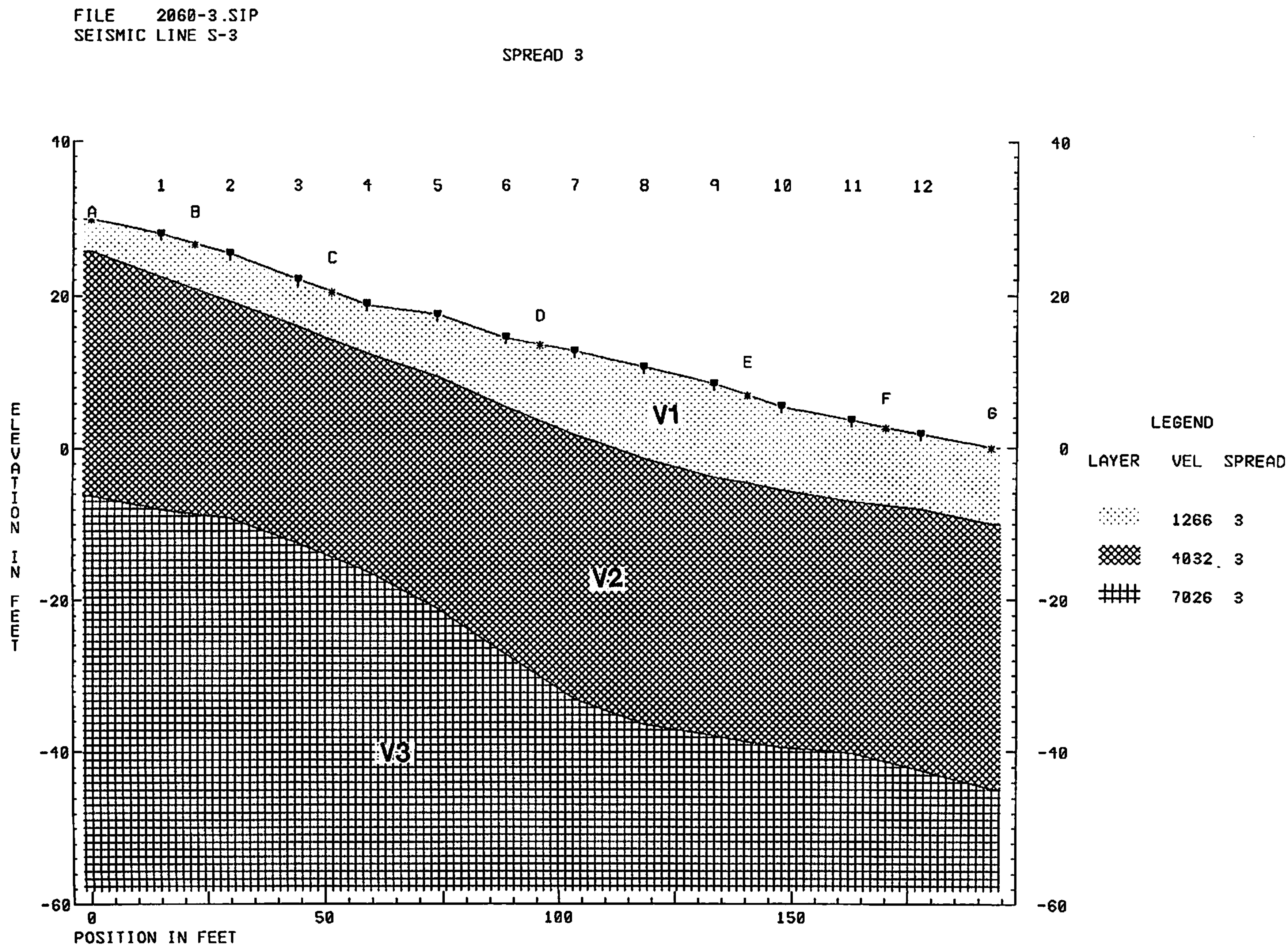
LAYER VELOCITY PROFILE S-2

North 5° East →



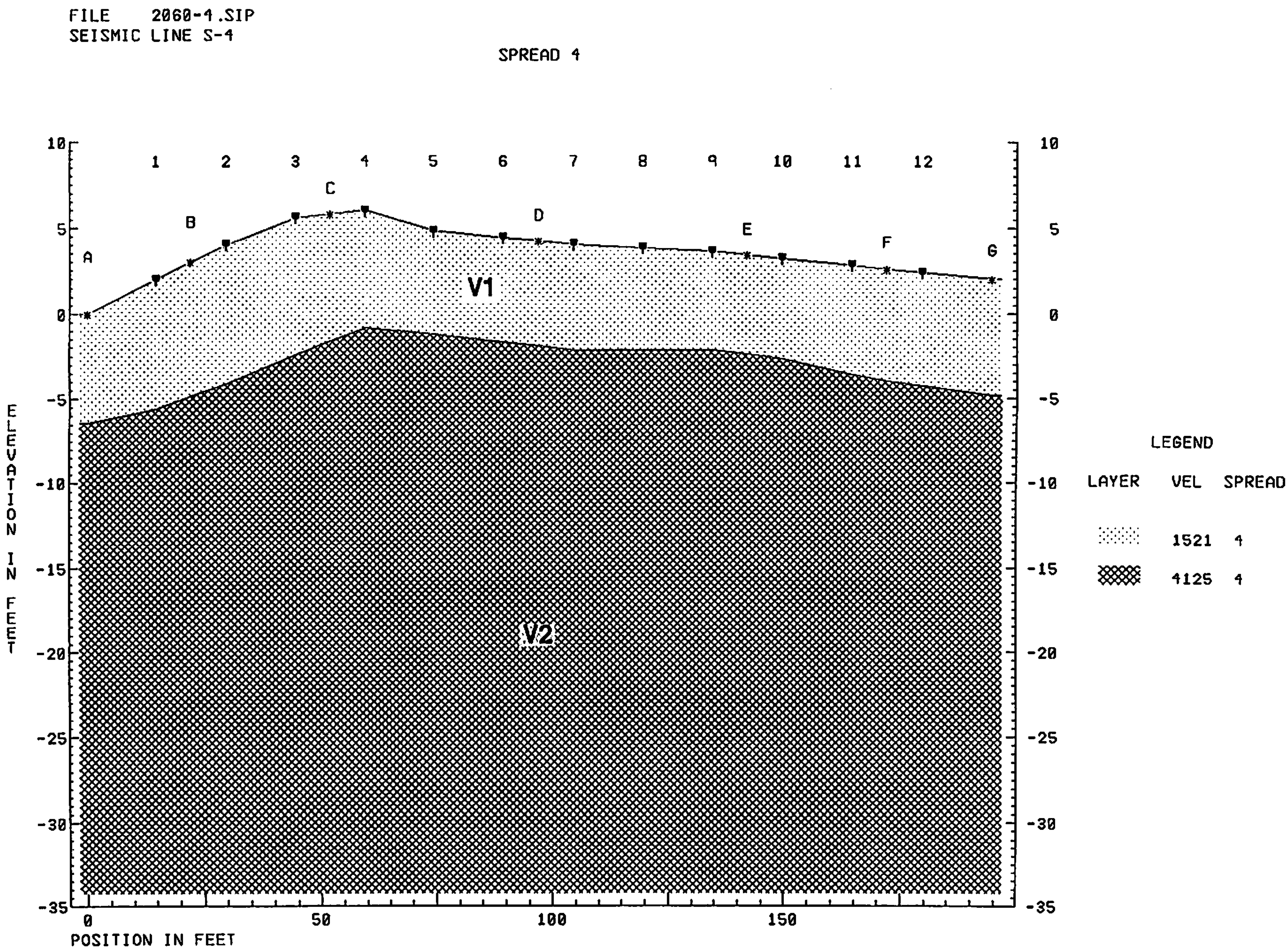
LAYER VELOCITY PROFILE S-3

South 85° East →



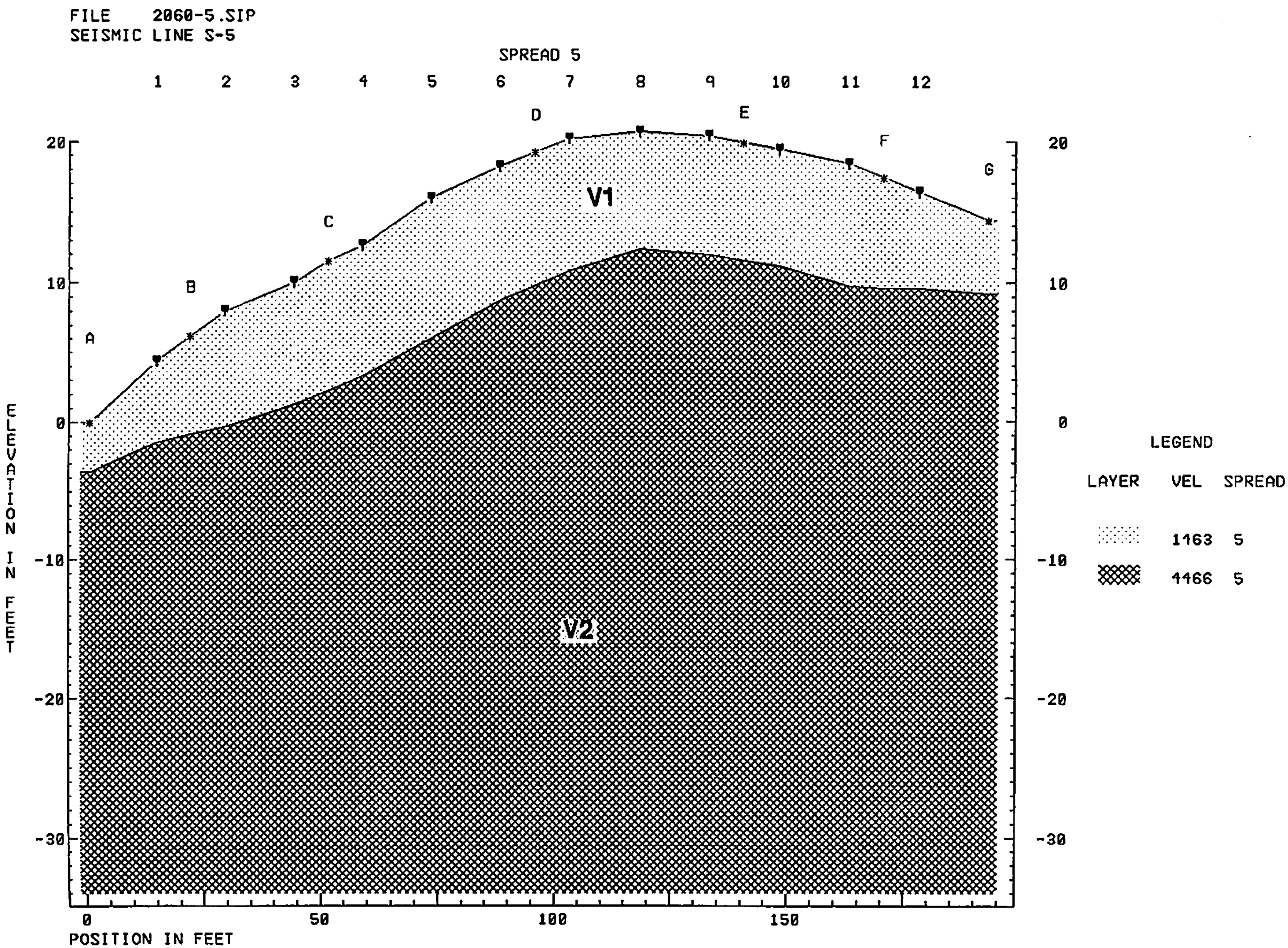
LAYER VELOCITY PROFILE S-4

South 60° West →



LAYER VELOCITY PROFILE S-5

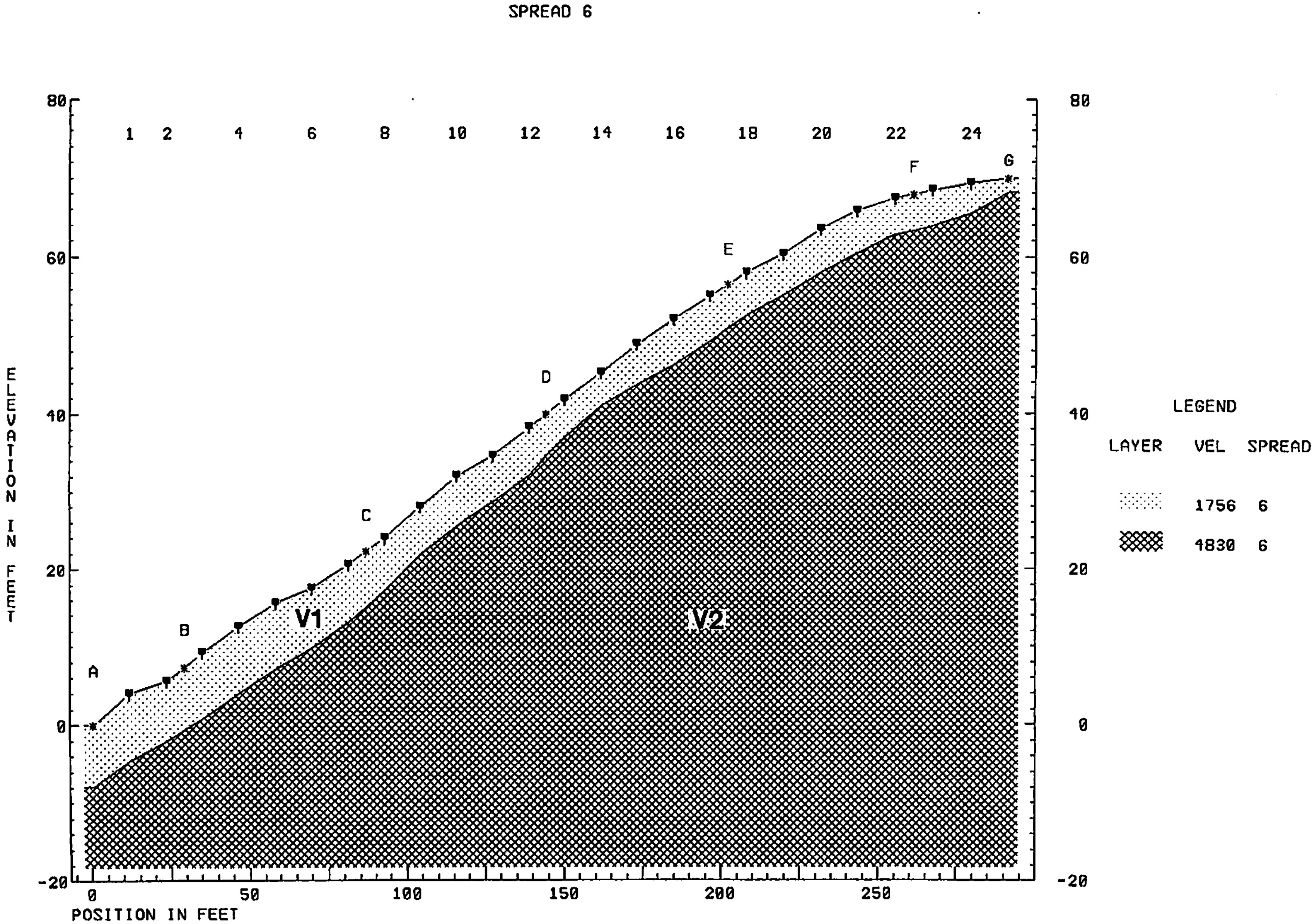
South 20° West →



LAYER VELOCITY PROFILE S-6

South 60° West →

FILE 2060-6.SIP
SEISMIC LINE S-6

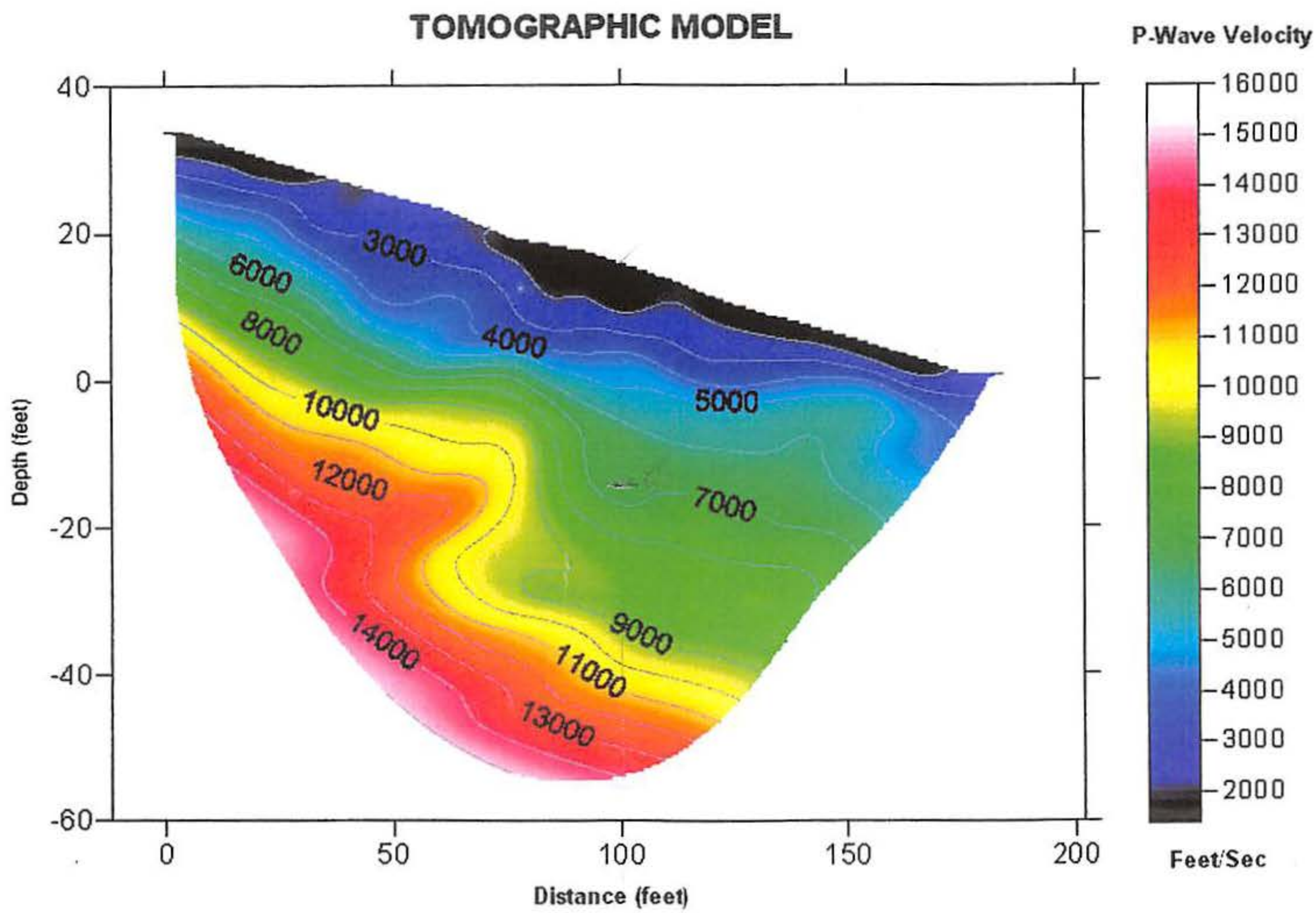


APPENDIX B

TOMOGRAPHIC MODELS

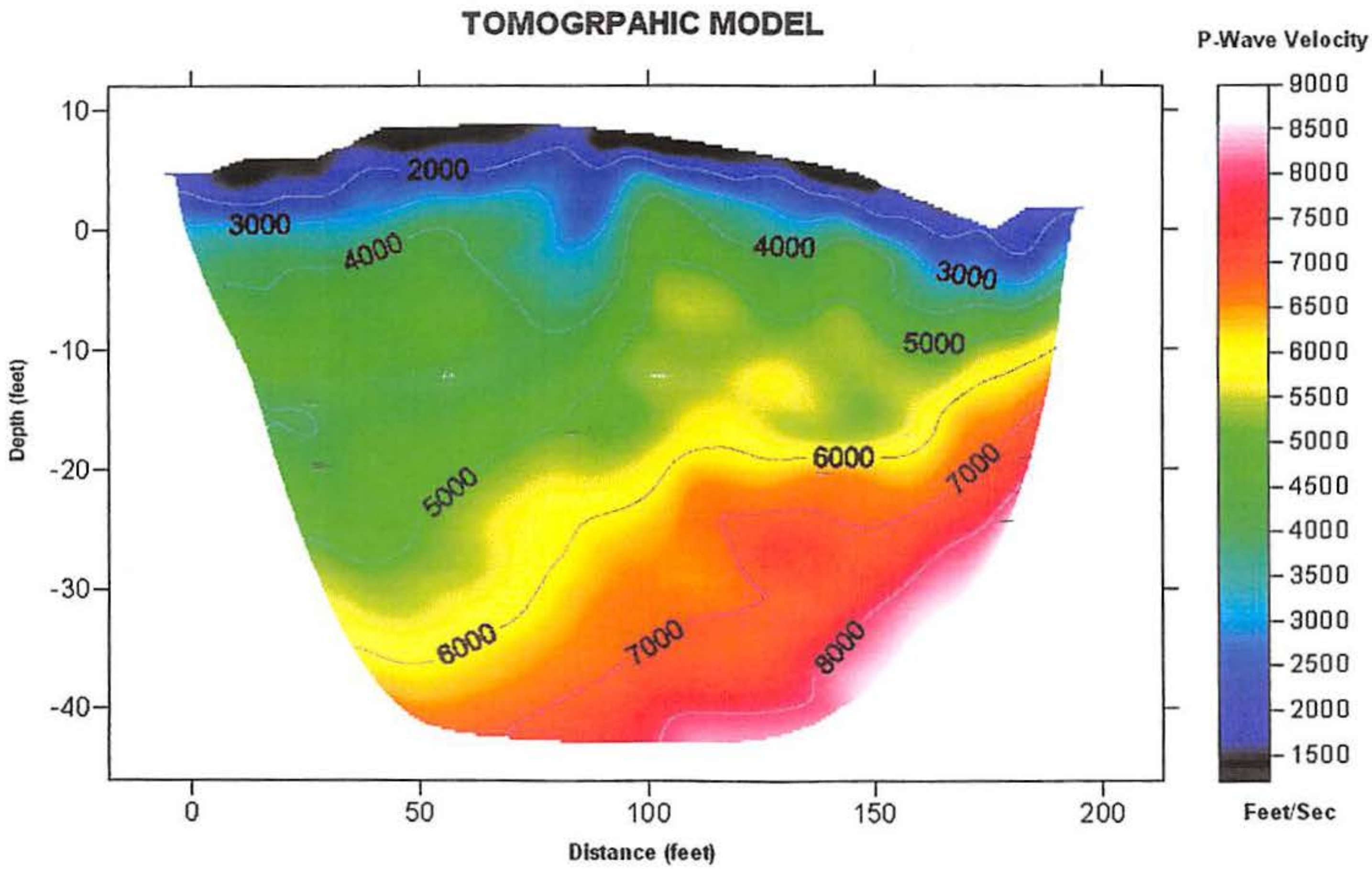
SEISMIC LINE S-1

South 85° East →



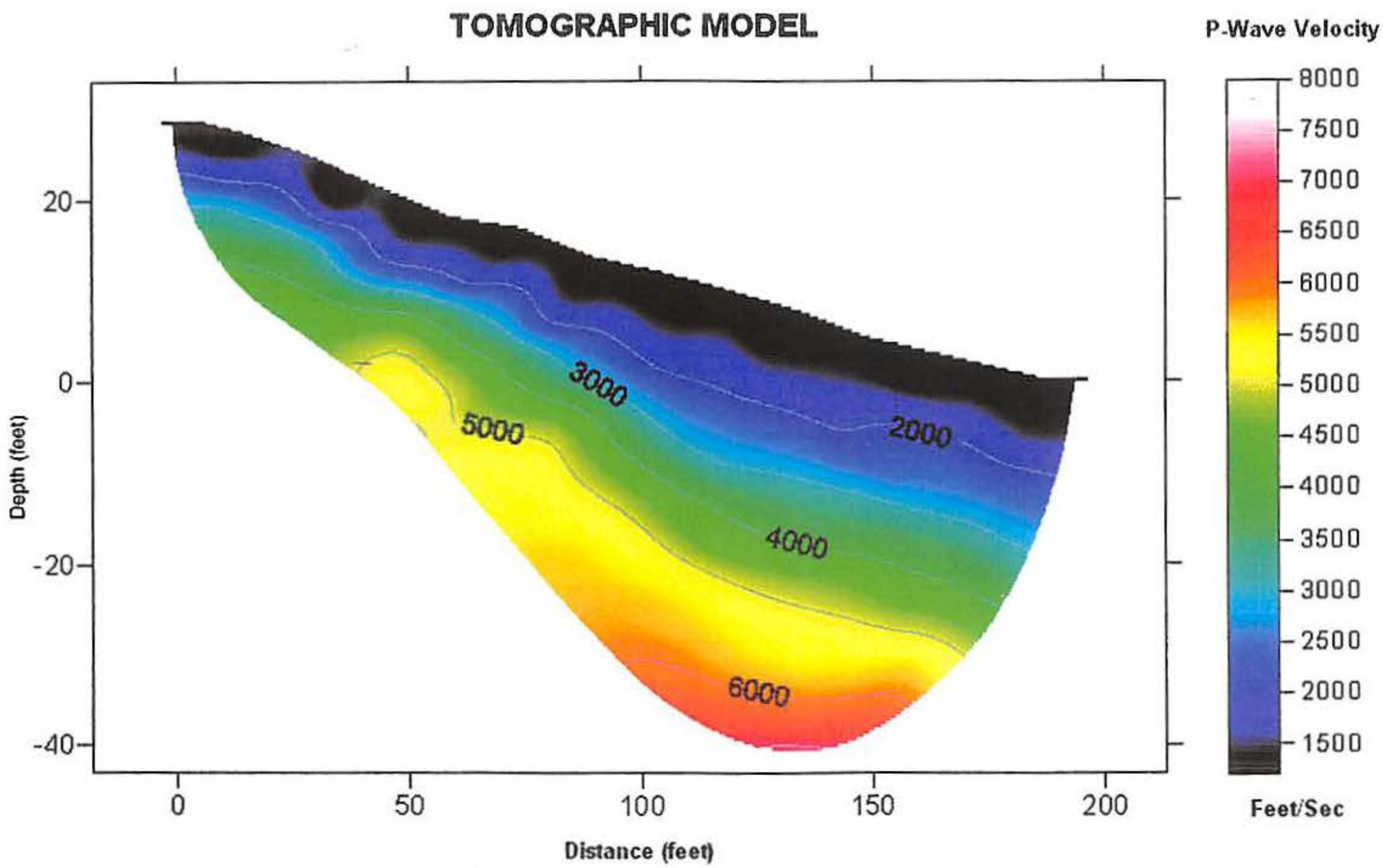
SEISMIC LINE S-2

North 5° East →



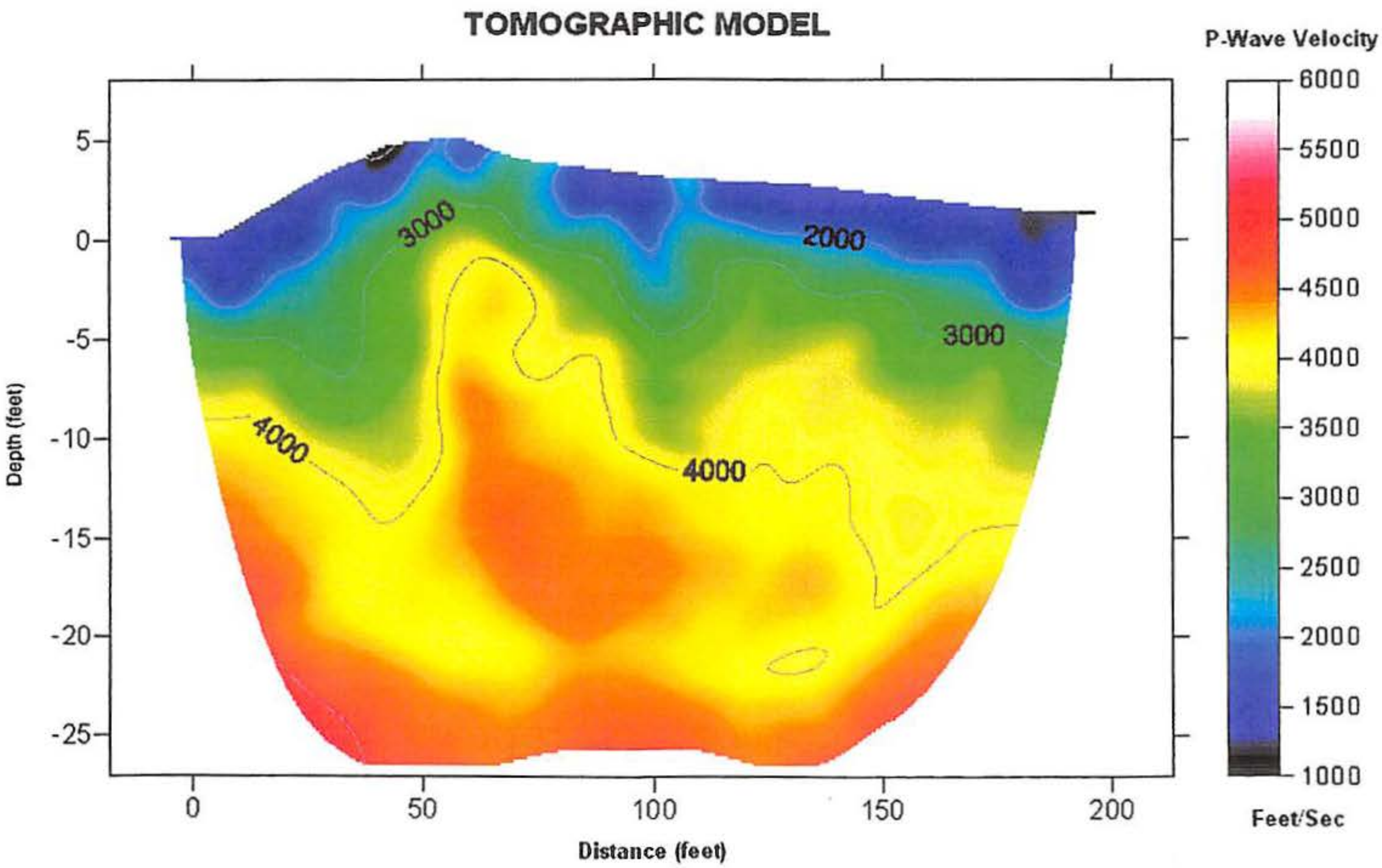
SEISMIC LINE S-3

South 85° East →



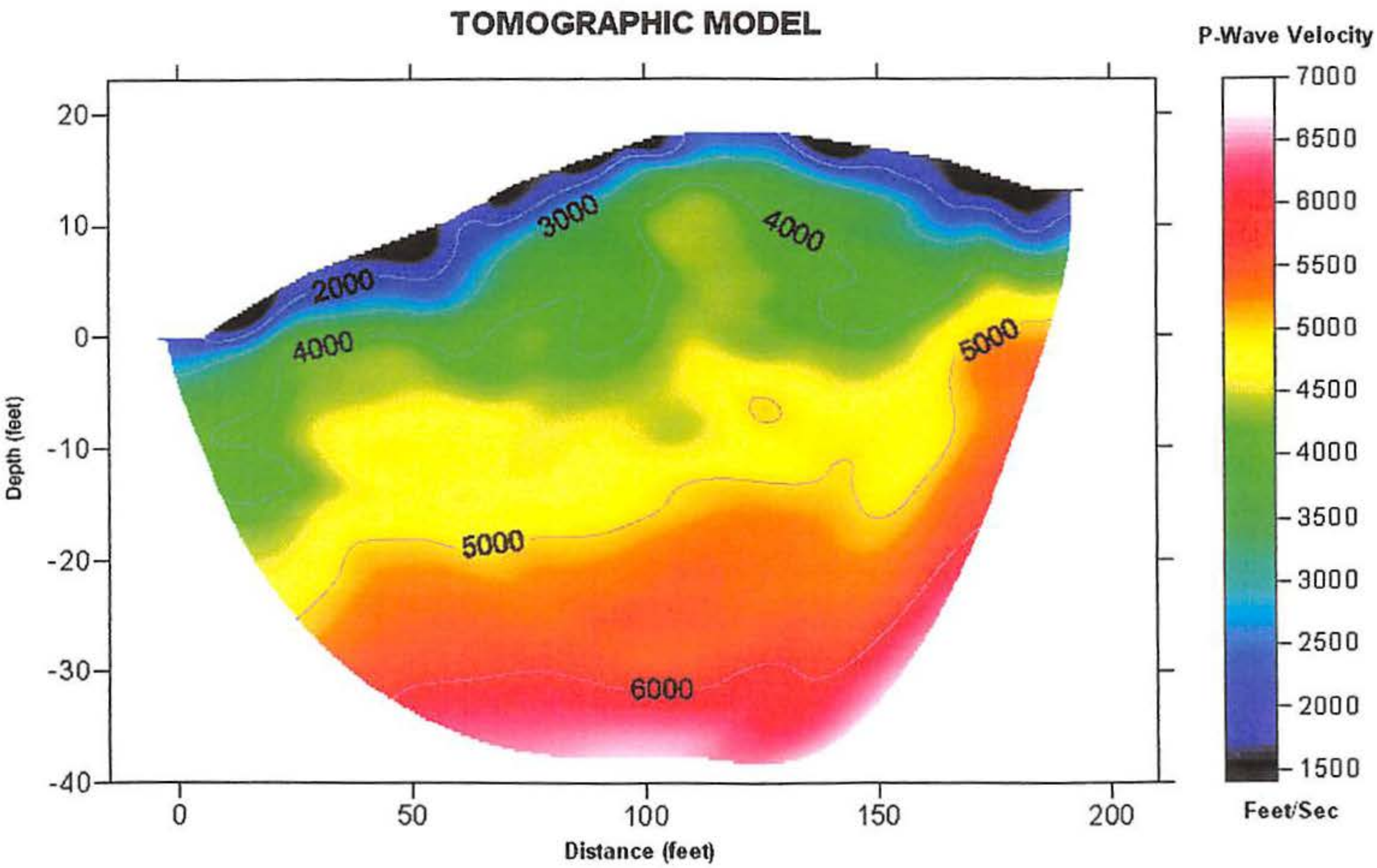
SEISMIC LINE S-4

South 60° West →



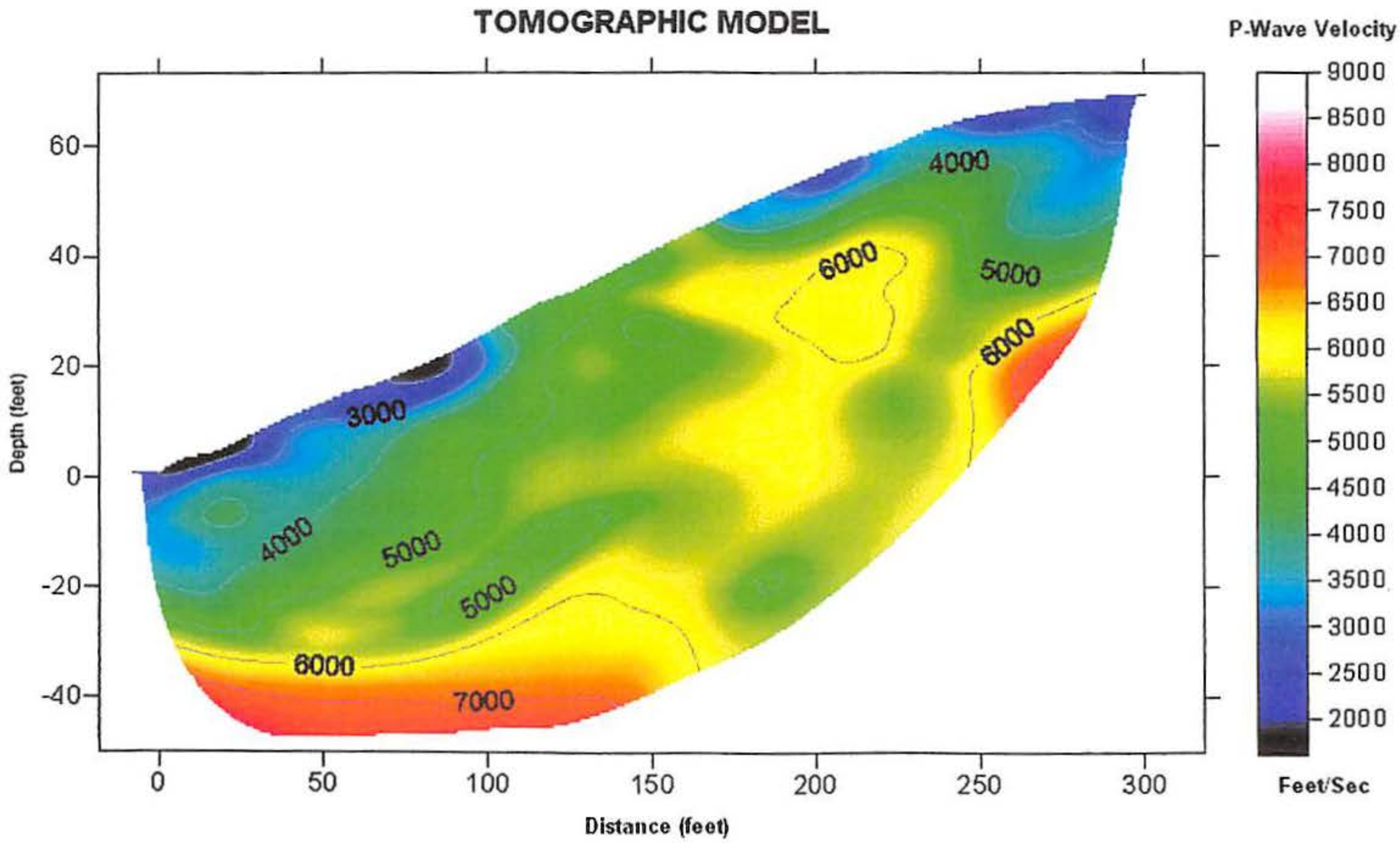
SEISMIC LINE S-5

South 20° West →



SEISMIC LINE S-6

South 60° West →



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

REFERENCES

REFERENCES

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APPENDIX F

LEIGHTON AND ASSOCIATES, INC.

GENERAL EARTHWORK AND GRADING SPECIFICATIONS FOR ROUGH GRADING

Table of Contents

GENERAL EARTHWORK AND GRADING SPECIFICATIONS FOR ROUGH GRADING

Table of Contents (Cont'd.)

Standard Details

1.0 General

1.1 Intent: These General Earthwork and Grading Specifications are for the grading and earthwork shown on the approved grading plan(s) and/or indicated in the geotechnical report(s). These Specifications are a part of the recommendations contained in the geotechnical report(s). In case of conflict, the specific recommendations in the geotechnical report shall supersede these more general Specifications. Observations of the earthwork by the project Geotechnical Consultant during the course of grading may result in new or revised recommendations that could supersede these specifications or the recommendations in the geotechnical report(s).

1.2 The Geotechnical Consultant of Record: Prior to commencement of work, the owner shall employ the Geotechnical Consultant of Record (Geotechnical Consultant). The Geotechnical Consultants shall be responsible for reviewing the approved geotechnical report(s) and accepting the adequacy of the preliminary geotechnical findings, conclusions, and recommendations prior to the commencement of the grading.

Prior to commencement of grading, the Geotechnical Consultant shall review the "work plan" prepared by the Earthwork Contractor (Contractor) and schedule sufficient personnel to perform the appropriate level of observation, mapping, and compaction testing.

During the grading and earthwork operations, the Geotechnical Consultant shall observe, map, and document the subsurface exposures to verify the geotechnical design assumptions. If the observed conditions are found to be significantly different than the interpreted assumptions during the design phase, the Geotechnical Consultant shall inform the owner, recommend appropriate changes in design to accommodate the observed conditions, and notify the review agency where required. Subsurface areas to be geotechnically observed, mapped, elevations recorded, and/or tested include natural ground after it has been cleared for receiving fill but before fill is placed, bottoms of all "remedial removal" areas, all key bottoms, and benches made on sloping ground to receive fill.

The Geotechnical Consultant shall observe the moisture-conditioning and processing of the subgrade and fill materials and perform relative compaction testing of fill to determine the attained level of compaction. The Geotechnical Consultant shall provide the test results to the owner and the Contractor on a routine and frequent basis.

1.3 The Earthwork Contractor: The Earthwork Contractor (Contractor) shall be qualified, experienced, and knowledgeable in earthwork logistics, preparation and processing of ground to receive fill, moisture-conditioning and processing of fill, and compacting fill. The Contractor shall review and accept the plans, geotechnical report(s), and these Specifications prior to commencement of grading. The

Contractor shall be solely responsible for performing the grading in accordance with the plans and specifications.

The Contractor shall prepare and submit to the owner and the Geotechnical Consultant a work plan that indicates the sequence of earthwork grading, the number of "spreads" of work and the estimated quantities of daily earthwork contemplated for the site prior to commencement of grading. The Contractor shall inform the owner and the Geotechnical Consultant of changes in work schedules and updates to the work plan at least 24 hours in advance of such changes so that appropriate observations and tests can be planned and accomplished. The Contractor shall not assume that the Geotechnical Consultant is aware of all grading operations.

The Contractor shall have the sole responsibility to provide adequate equipment and methods to accomplish the earthwork in accordance with the applicable grading codes and agency ordinances, these Specifications, and the recommendations in the approved geotechnical report(s) and grading plan(s). If, in the opinion of the Geotechnical Consultant, unsatisfactory conditions, such as unsuitable soil, improper moisture condition, inadequate compaction, insufficient buttress key size, adverse weather, etc., are resulting in a quality of work less than required in these specifications, the Geotechnical Consultant shall reject the work and may recommend to the owner that construction be stopped until the conditions are rectified.

2.0 Preparation of Areas to be Filled

2.1 Clearing and Grubbing: Vegetation, such as brush, grass, roots, and other deleterious material shall be sufficiently removed and properly disposed of in a method acceptable to the owner, governing agencies, and the Geotechnical Consultant.

The Geotechnical Consultant shall evaluate the extent of these removals depending on specific site conditions. Earth fill material shall not contain more than 1 percent of organic materials (by volume). No fill lift shall contain more than 5 percent of organic matter. Nesting of the organic materials shall not be allowed.

If potentially hazardous materials are encountered, the Contractor shall stop work in the affected area, and a hazardous material specialist shall be informed immediately for proper evaluation and handling of these materials prior to continuing to work in that area.

As presently defined by the State of California, most refined petroleum products (gasoline, diesel fuel, motor oil, grease, coolant, etc.) have chemical constituents that are considered to be hazardous waste. As such, the indiscriminate dumping or spillage of these fluids onto the ground may constitute a misdemeanor, punishable by fines and/or imprisonment, and shall not be allowed.

2.2 Processing: Existing ground that has been declared satisfactory for support of fill by the Geotechnical Consultant shall be scarified to a minimum depth of 6 inches. Existing ground that is not satisfactory shall be overexcavated as specified in the following section. Scarification shall continue until soils are broken down and free of large clay lumps or clods and the working surface is reasonably uniform, flat, and free of uneven features that would inhibit uniform compaction.

2.3 Overexcavation: In addition to removals and overexcavations recommended in the approved geotechnical report(s) and the grading plan, soft, loose, dry, saturated, spongy, organic-rich, highly fractured or otherwise unsuitable ground shall be overexcavated to competent ground as evaluated by the Geotechnical Consultant during grading.

2.4 Benching: Where fills are to be placed on ground with slopes steeper than 5:1 (horizontal to vertical units), the ground shall be stepped or benched. Please see the Standard Details for a graphic illustration. The lowest bench or key shall be a minimum of 15 feet wide and at least 2 feet deep, into competent material as evaluated by the Geotechnical Consultant. Other benches shall be excavated a minimum height of 4 feet into competent material or as otherwise recommended by the Geotechnical Consultant. Fill placed on ground sloping flatter than 5:1 shall also be benched or otherwise overexcavated to provide a flat subgrade for the fill.

2.5 Evaluation/Acceptance of Fill Areas: All areas to receive fill, including removal and processed areas, key bottoms, and benches, shall be observed, mapped, elevations recorded, and/or tested prior to being accepted by the Geotechnical Consultant as suitable to receive fill. The Contractor shall obtain a written acceptance from the Geotechnical Consultant prior to fill placement. A licensed surveyor shall provide the survey control for determining elevations of processed areas, keys, and benches.

3.0 Fill Material

3.1 General: Material to be used as fill shall be essentially free of organic matter and other deleterious substances evaluated and accepted by the Geotechnical Consultant prior to placement. Soils of poor quality, such as those with unacceptable gradation, high expansion potential, or low strength shall be placed in areas acceptable to the Geotechnical Consultant or mixed with other soils to achieve satisfactory fill material.

3.2 Oversize: Oversize material defined as rock, or other irreducible material with a maximum dimension greater than 8 inches, shall not be buried or placed in fill unless location, materials, and placement methods are specifically accepted by the Geotechnical Consultant. Placement operations shall be such that nesting of oversized material does not occur and such that oversize material is completely surrounded by compacted or densified fill. Oversize material shall not be placed within 10 vertical feet of finish grade or within 2 feet of future utilities or underground construction.

3.3 Import: If importing of fill material is required for grading, proposed import material shall meet the requirements of Section 3.1. The potential import source shall be given to the Geotechnical Consultant at least 48 hours (2 working days) before importing begins so that its suitability can be determined and appropriate tests performed.

4.0 Fill Placement and Compaction

4.1 Fill Layers: Approved fill material shall be placed in areas prepared to receive fill (per Section 3.0) in near-horizontal layers not exceeding 8 inches in loose thickness. The Geotechnical Consultant may accept thicker layers if testing indicates the grading procedures can adequately compact the thicker layers. Each layer shall be spread evenly and mixed thoroughly to attain relative uniformity of material and moisture throughout.

4.2 Fill Moisture Conditioning: Fill soils shall be watered, dried back, blended, and/or mixed, as necessary to attain a relatively uniform moisture content at or slightly over optimum. Maximum density and optimum soil moisture content tests shall be performed in accordance with the American Society of Testing and Materials (ASTM Test Method D1557-91).

4.3 Compaction of Fill: After each layer has been moisture-conditioned, mixed, and evenly spread, it shall be uniformly compacted to not less than 90 percent of maximum dry density (ASTM Test Method D1557-91). Compaction equipment shall be adequately sized and be either specifically designed for soil compaction or of proven reliability to efficiently achieve the specified level of compaction with uniformity.

4.4 Compaction of Fill Slopes: In addition to normal compaction procedures specified above, compaction of slopes shall be accomplished by backrolling of slopes with sheepsfoot rollers at increments of 3 to 4 feet in fill elevation, or by other methods producing satisfactory results acceptable to the Geotechnical Consultant. Upon completion of grading, relative compaction of the fill, out to the slope face, shall be at least 90 percent of maximum density per ASTM Test Method D1557-91.

4.5 Compaction Testing: Field tests for moisture content and relative compaction of the fill soils shall be performed by the Geotechnical Consultant. Location and frequency of tests shall be at the Consultant's discretion based on field conditions encountered. Compaction test locations will not necessarily be selected on a random basis. Test locations shall be selected to verify adequacy of compaction levels in areas that are judged to be prone to inadequate compaction (such as close to slope faces and at the fill/bedrock benches).

4.6 Frequency of Compaction Testing: Tests shall be taken at intervals not exceeding 2 feet in vertical rise and/or 1,000 cubic yards of compacted fill soils embankment. In addition, as a guideline, at least one test shall be taken on slope faces for each 5,000 square feet of slope face and/or each 10 feet of vertical height of slope. The Contractor shall assure that fill construction is such that the testing schedule can be accomplished by the Geotechnical Consultant. The Contractor shall stop or slow down the earthwork construction if these minimum standards are not met.

4.7 Compaction Test Locations: The Geotechnical Consultant shall document the approximate elevation and horizontal coordinates of each test location. The Contractor shall coordinate with the project surveyor to assure that sufficient grade stakes are established so that the Geotechnical Consultant can determine the test locations with sufficient accuracy. At a minimum, two grade stakes within a horizontal distance of 100 feet and vertically less than 5 feet apart from potential test locations shall be provided.

5.0 Subdrain Installation

Subdrain systems shall be installed in accordance with the approved geotechnical report(s), the grading plan, and the Standard Details. The Geotechnical Consultant may recommend additional subdrains and/or changes in subdrain extent, location, grade, or material depending on conditions encountered during grading. All subdrains shall be surveyed by a land surveyor/civil engineer for line and grade after installation and prior to burial. Sufficient time should be allowed by the Contractor for these surveys.

6.0 Excavation

Excavations, as well as over-excavation for remedial purposes, shall be evaluated by the Geotechnical Consultant during grading. Remedial removal depths shown on geotechnical plans are estimates only. The actual extent of removal shall be determined by the Geotechnical Consultant based on the field evaluation of exposed conditions during grading. Where fill-over-cut slopes are to be graded, the cut portion of the slope shall be made, evaluated, and accepted by the Geotechnical Consultant prior to placement of materials for construction of the fill portion of the slope, unless otherwise recommended by the Geotechnical Consultant.

7.0 Trench Backfills

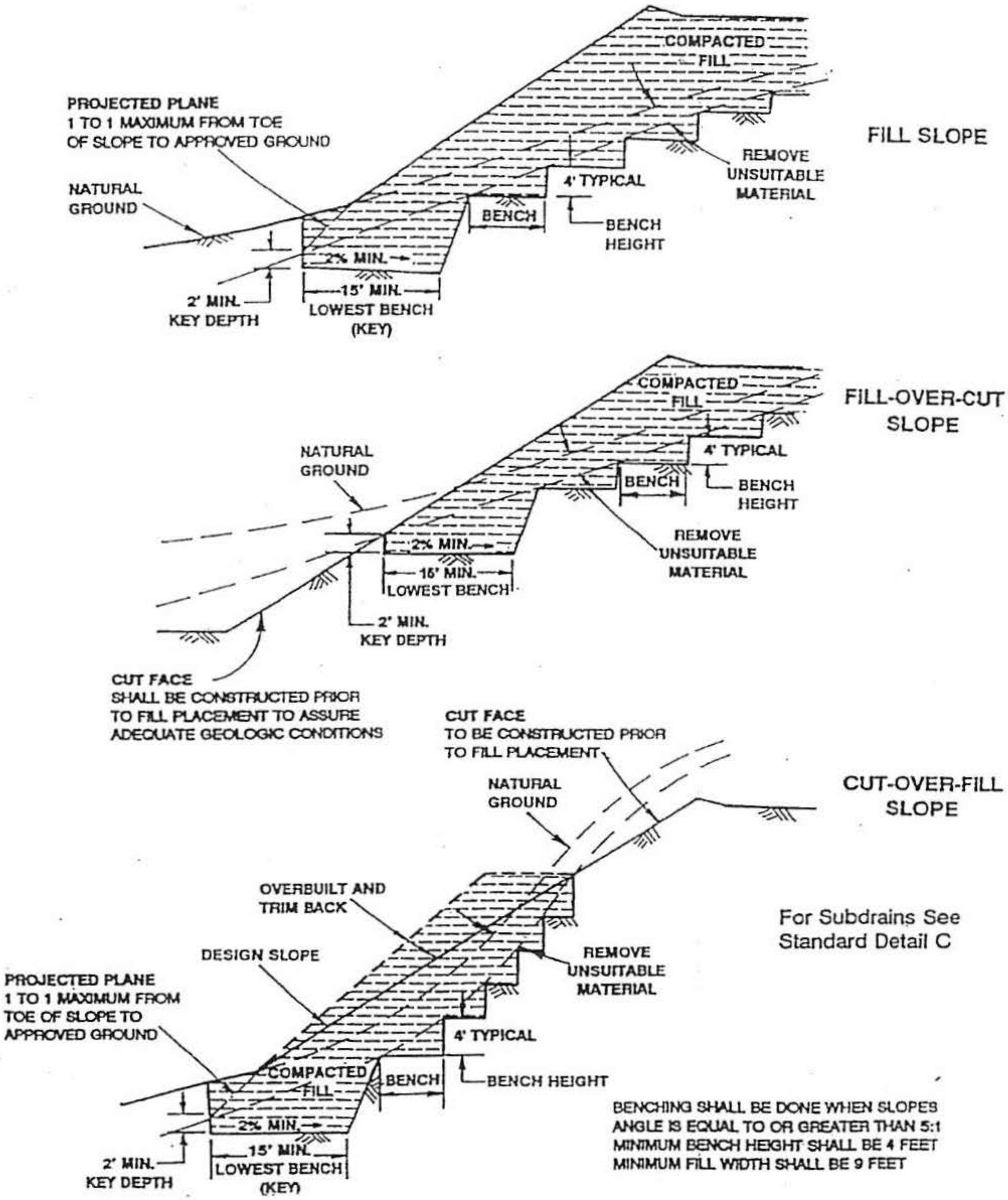
7.1 Safety: The Contractor shall follow all OHSA and Cal/OSHA requirements for safety of trench excavations.

7.2 Bedding and Backfill: All bedding and backfill of utility trenches shall be done in accordance with the applicable provisions of Standard Specifications of Public Works Construction. Bedding material shall have a Sand Equivalent greater than 30 (SE>30). The bedding shall be placed to 1 foot over the top of the conduit and densified by jetting. Backfill shall be placed and densified to a minimum of 90 percent of maximum from 1 foot above the top of the conduit to the surface.

The Geotechnical Consultant shall test the trench backfill for relative compaction. At least one test should be made for every 300 feet of trench and 2 feet of fill.

7.3 Lift Thickness: Lift thickness of trench backfill shall not exceed those allowed in the Standard Specifications of Public Works Construction unless the Contractor can demonstrate to the Geotechnical Consultant that the fill lift can be compacted to the minimum relative compaction by his alternative equipment and method.

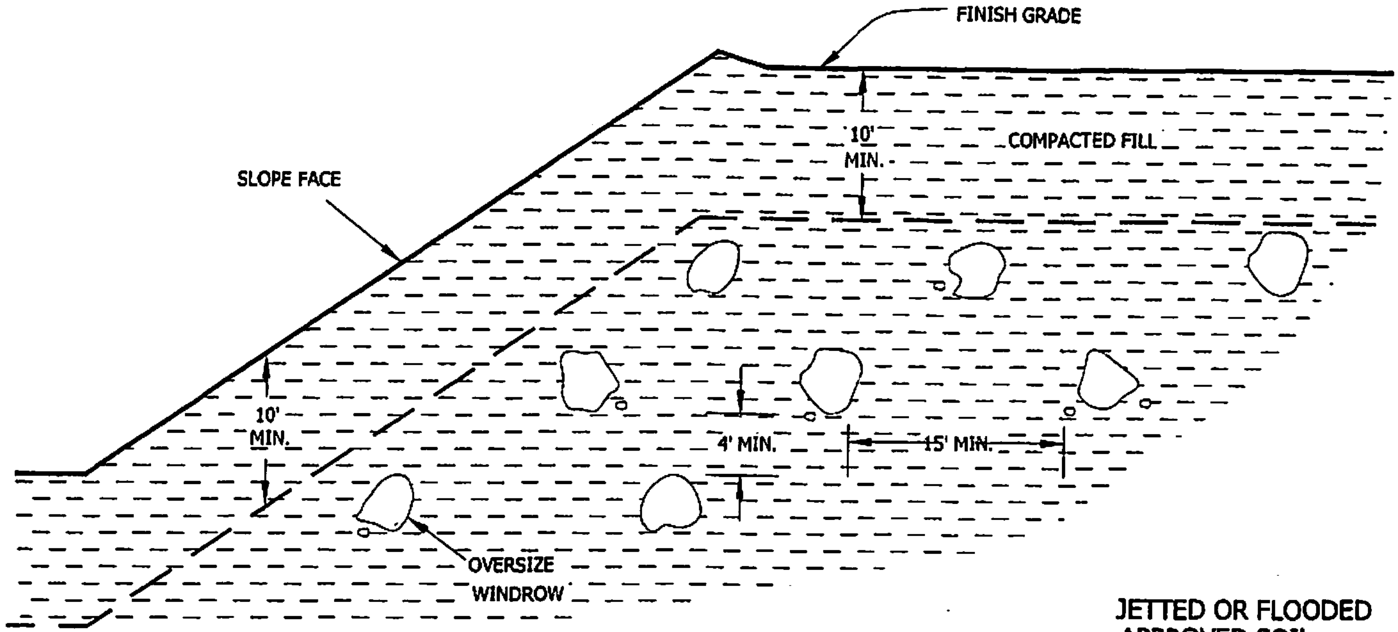
7.4 Observation and Testing: The jetting of the bedding around the conduits shall be observed by the Geotechnical Consultant.



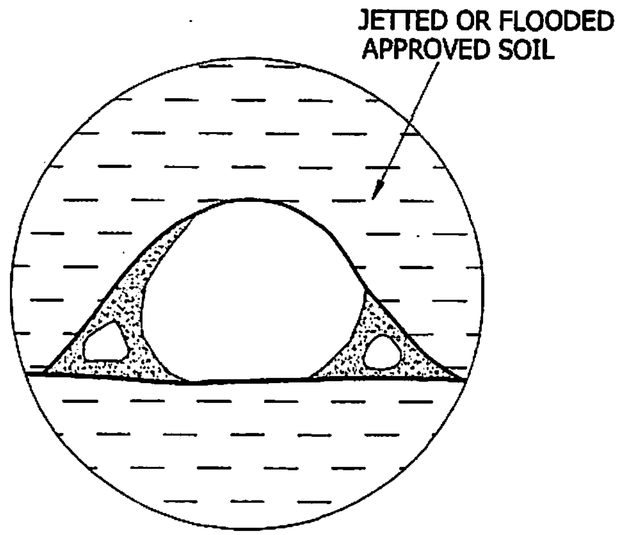
For Subdrains See
Standard Detail C

BENCHING SHALL BE DONE WHEN SLOPES
ANGLE IS EQUAL TO OR GREATER THAN 5:1
MINIMUM BENCH HEIGHT SHALL BE 4 FEET
MINIMUM FILL WIDTH SHALL BE 9 FEET

KEYING AND BENCHING	GENERAL EARTHWORK AND GRADING SPECIFICATIONS STANDARD DETAILS A	

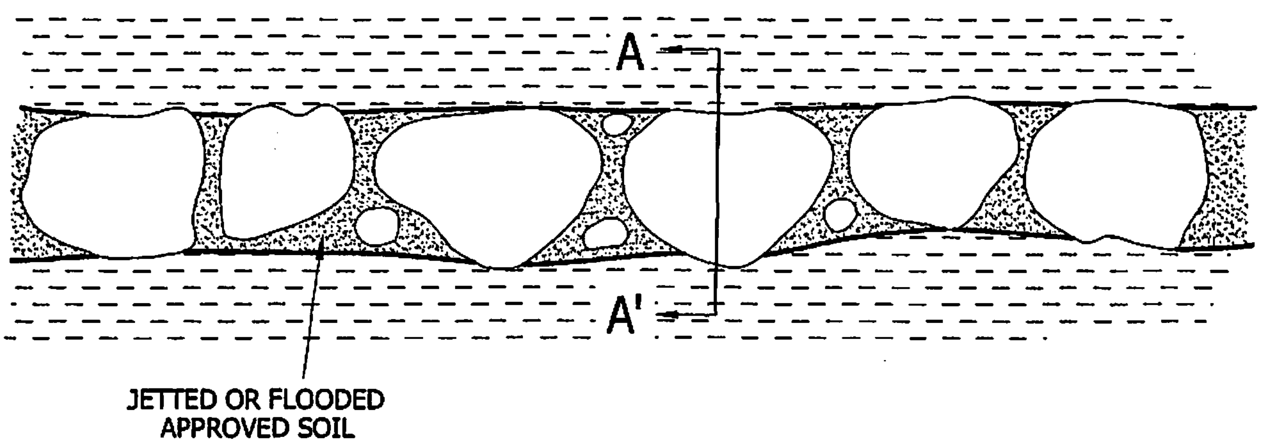


- Oversize rock is larger than 8 inches in largest dimension.
- Backfill with approved soil jetted or flooded in place to fill all the voids.
- Do not bury rock within 10 feet of finish grade.
- Windrow of buried rock shall be parallel to the finished slope face.



SECTION A-A'

PROFILE ALONG WINDROW



OVERSIZE ROCK DISPOSAL	GENERAL EARTHWORK AND GRADING SPECIFICATIONS STANDARD DETAILS B	A LEIGHTON GROUP COMPANY Rev. 7/00

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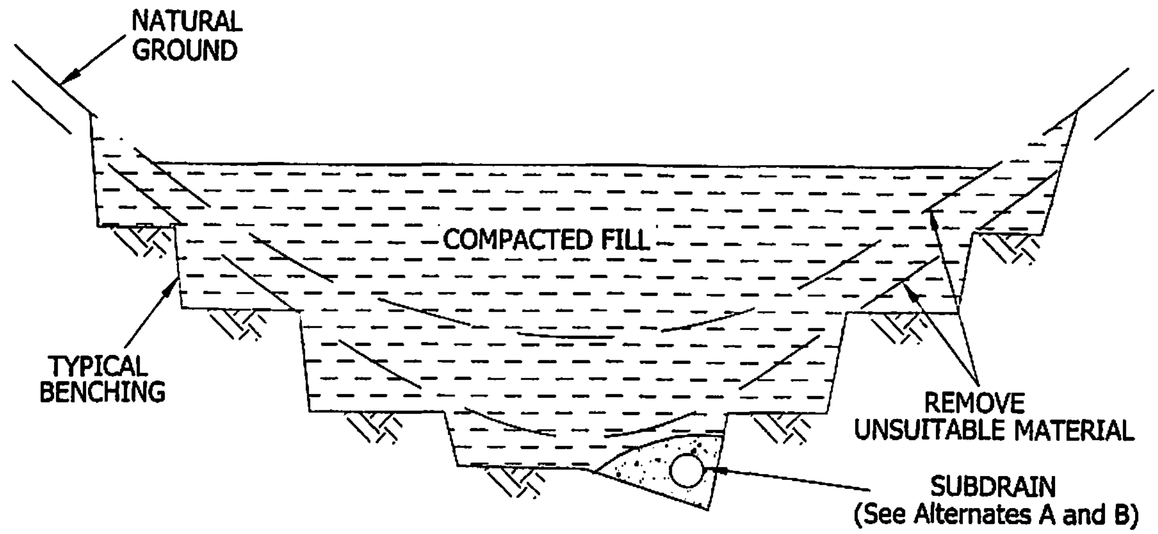
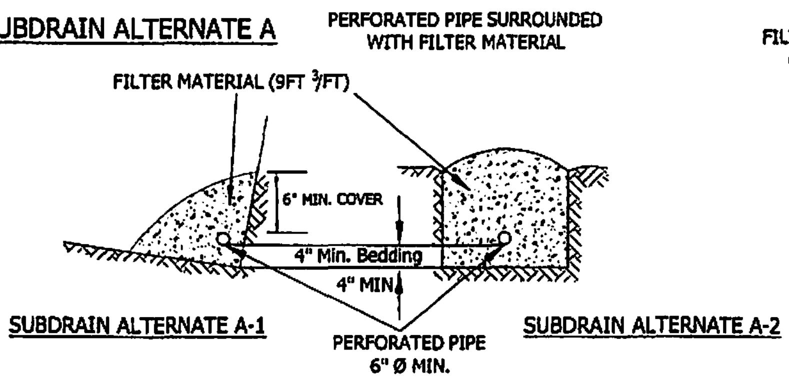


SUBDRAIN ALTERNATE A

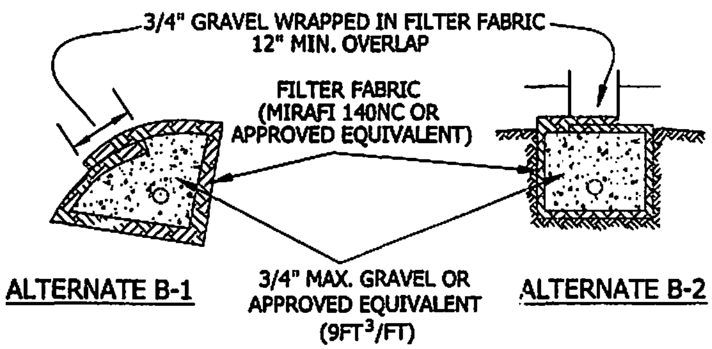


FILTER MATERIAL

FILTER MATERIAL SHALL BE CLASS 2 PERMEABLE MATERIAL PER STATE OF CALIFORNIA STANDARD SPECIFICATION, OR APPROVED ALTERNATE. CLASS 2 GRADING AS FOLLOWS:

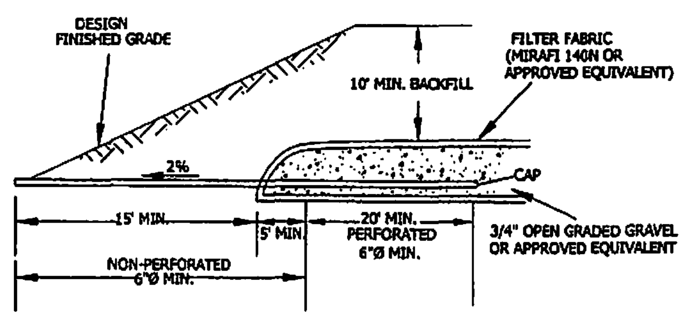
Sieve Size	Percent Passing
1"	100
3/4"	90-100
3/8"	40-100
No. 4	25-40
No. 8	18-33
No. 30	5-15
No. 50	0-7
No. 200	0-3

SUBDRAIN ALTERNATE B

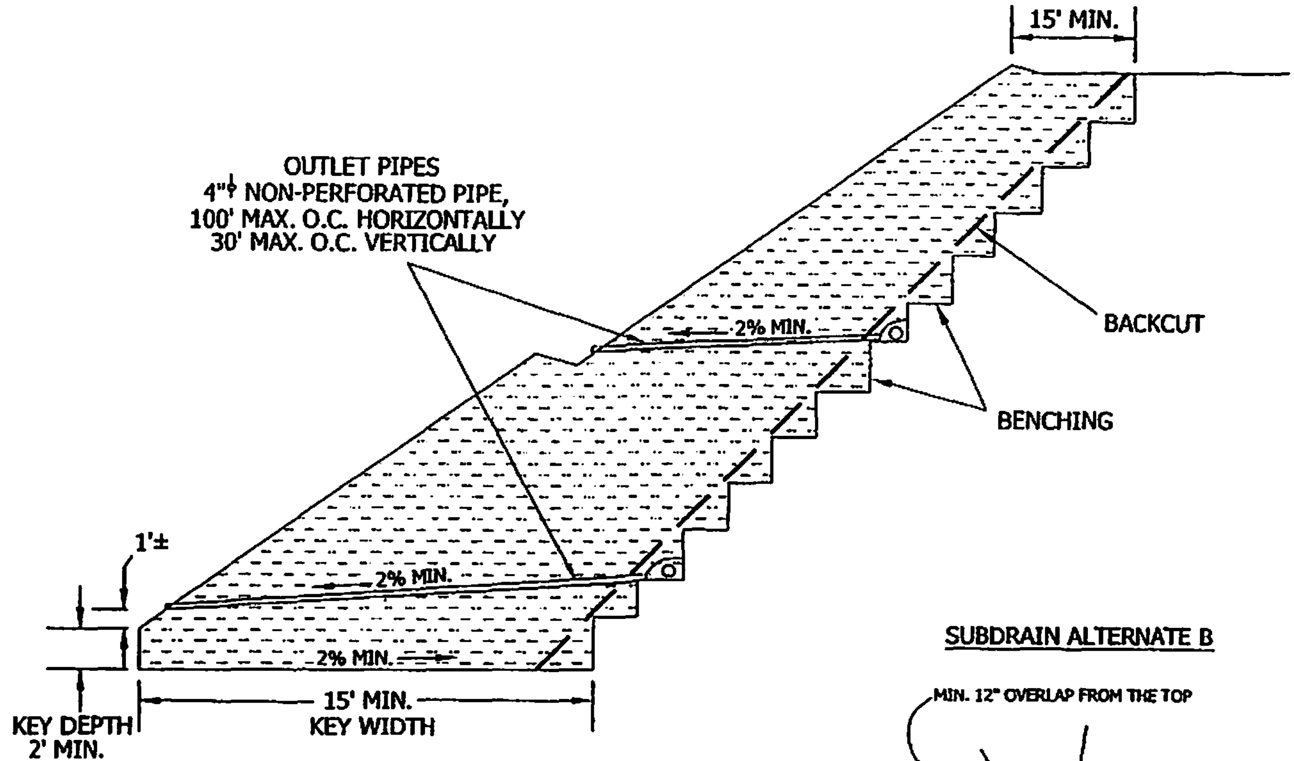


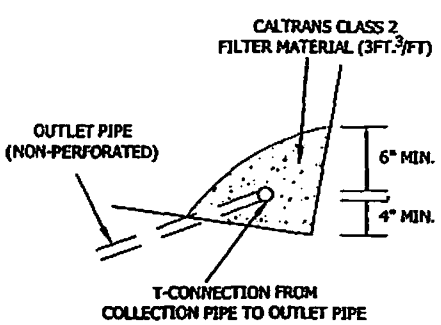
○ PERFORATED PIPE IS OPTIONAL PER GOVERNING AGENCY'S REQUIREMENTS

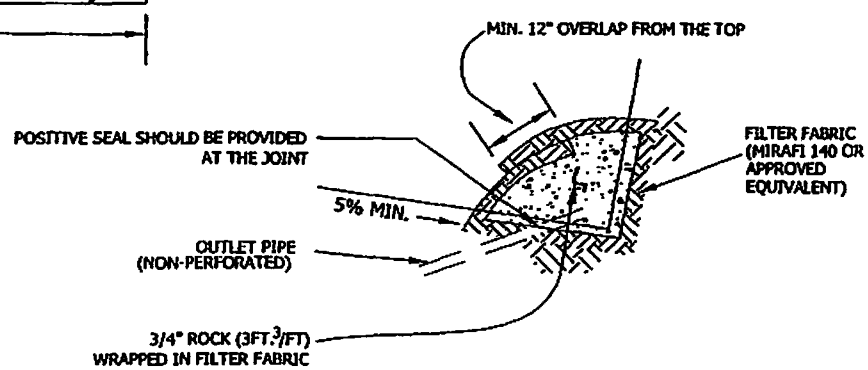
DETAIL OF CANYON SUBDRAIN TERMINAL



CANYON SUBDRAIN	GENERAL EARTHWORK AND GRADING SPECIFICATIONS STANDARD DETAILS C	A LEIGHTON GROUP COMPANY Rev. 7/00







- SUBDRAIN INSTALLATION - Subdrain collector pipe shall be installed with perforations down or, unless otherwise designated by the geotechnical consultant. Outlet pipes shall be non-perforated pipe. The subdrain pipe shall have at least 8 perforations uniformly spaced per foot. Perforation shall be 1/4" to 1/2" if drilled holes are used. All subdrain pipes shall have a gradient at least 2 % towards the outlet.
- SUBDRAIN PIPE - Subdrain pipe shall be ASTM D2751, ASTM D1527 (Schedule 40) or SDR 23.5 ABS pipe or ASTM D3034 (Schedule 40) or SDR 23.5 PVC pipe.
- All outlet pipe shall be placed in a trench and, after fill is placed above it, rodded to verify integrity.

BUTTRESS OR REPLACEMENT FILL SUBDRAINS	GENERAL EARTHWORK AND GRADING SPECIFICATIONS STANDARD DETAILS D	

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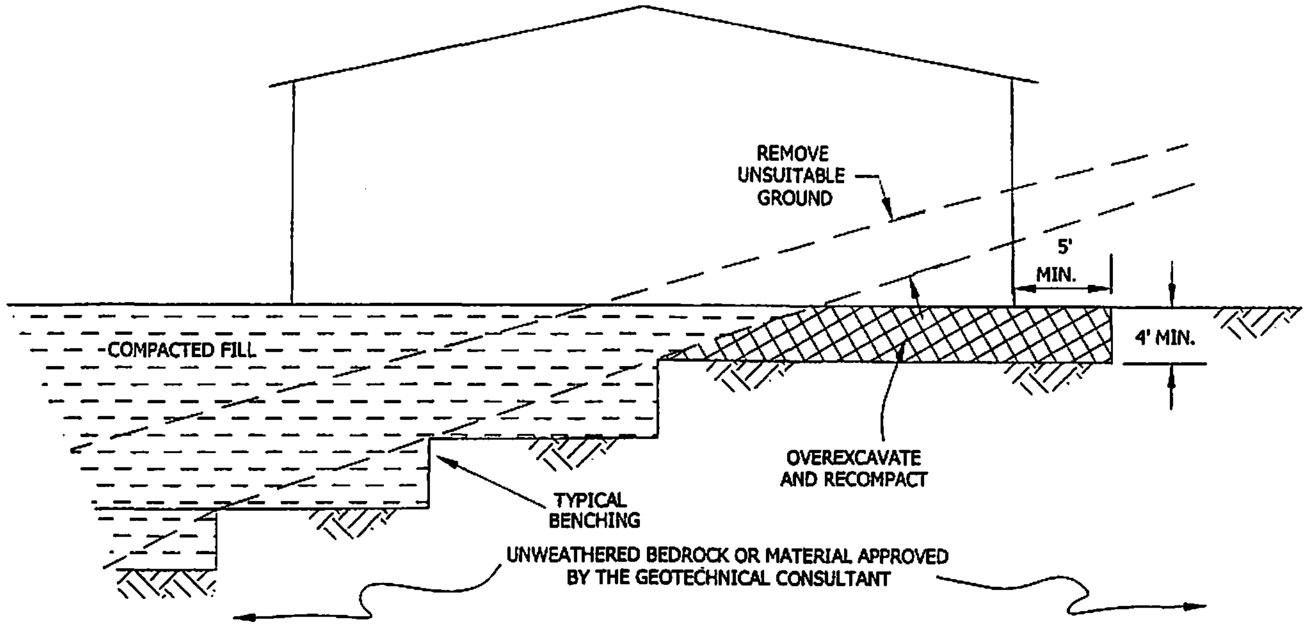
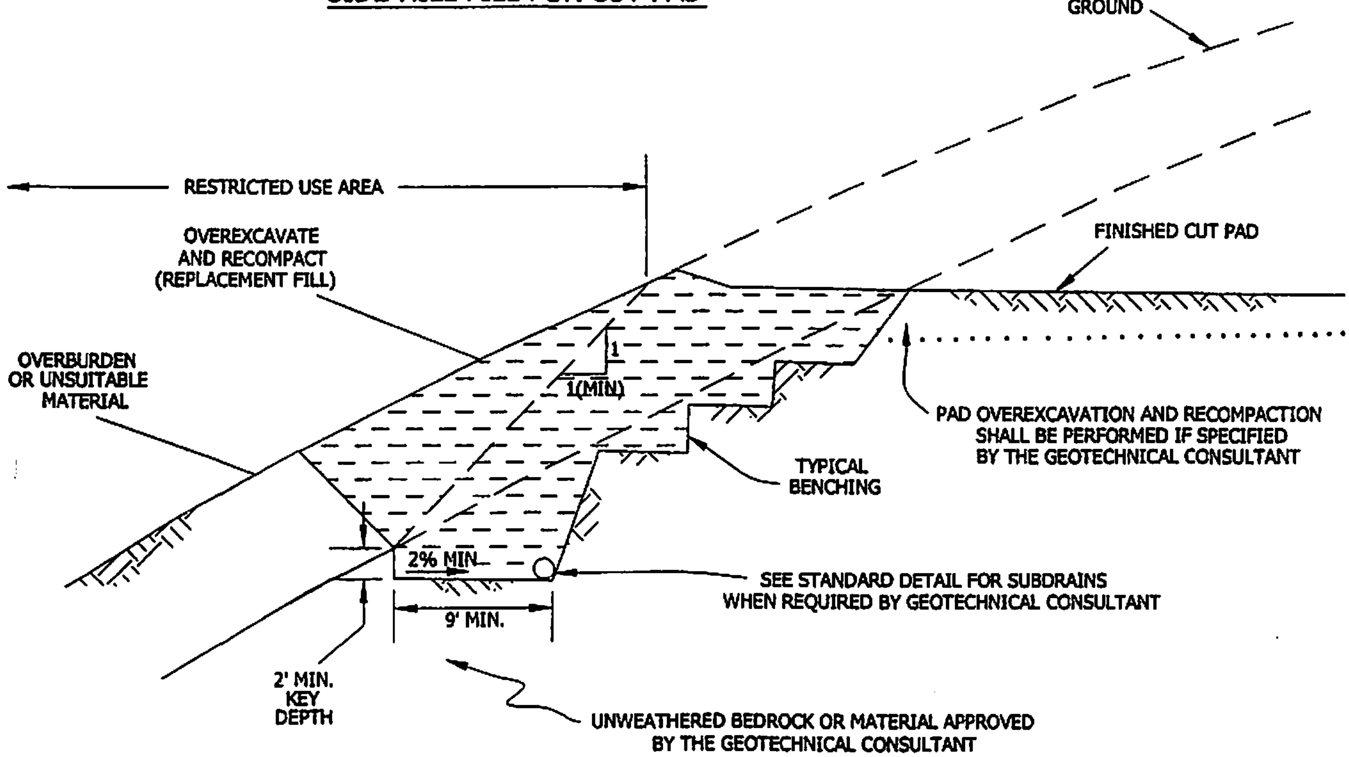
CUT-FILL TRANSITION LOT OVEREXCAVATION



SIDE HILL FILL FOR CUT PAD



TRANSITION LOT FILLS AND SIDE HILL FILLS	GENERAL EARTHWORK AND GRADING SPECIFICATIONS STANDARD DETAILS E	A LEIGHTON GROUP COMPANY Rev. 7/00

Leighton

SOIL RESISTIVITY TEST

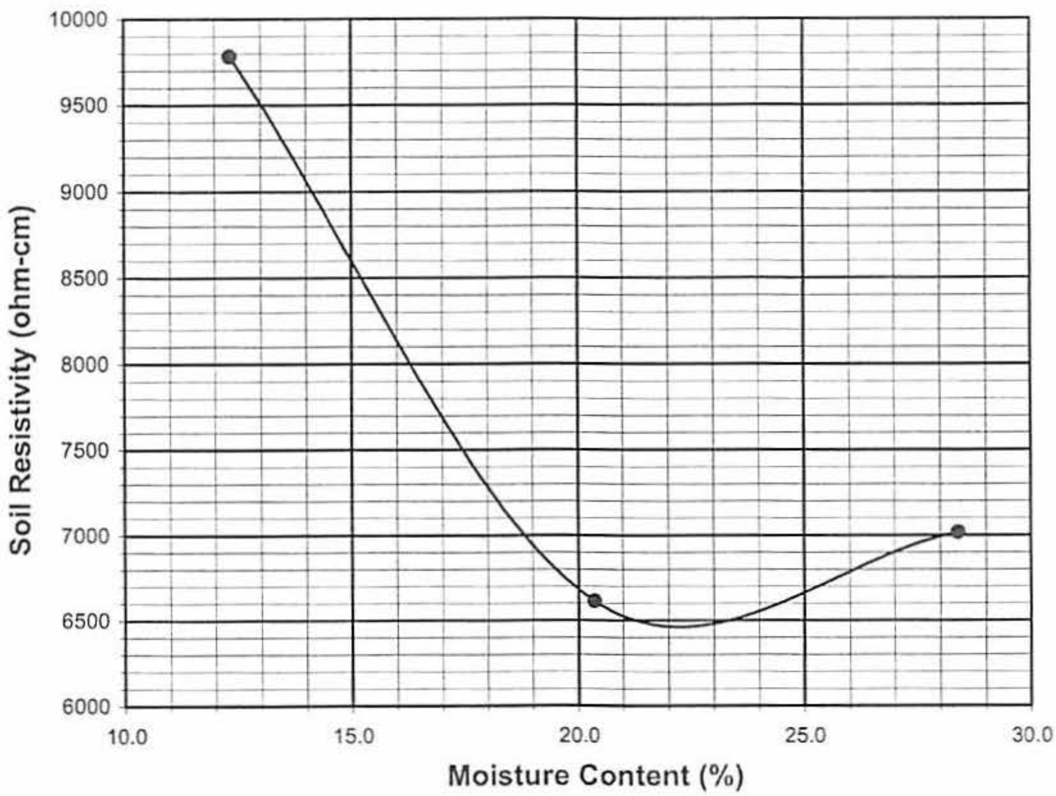
DOT CA TEST 532 / 643

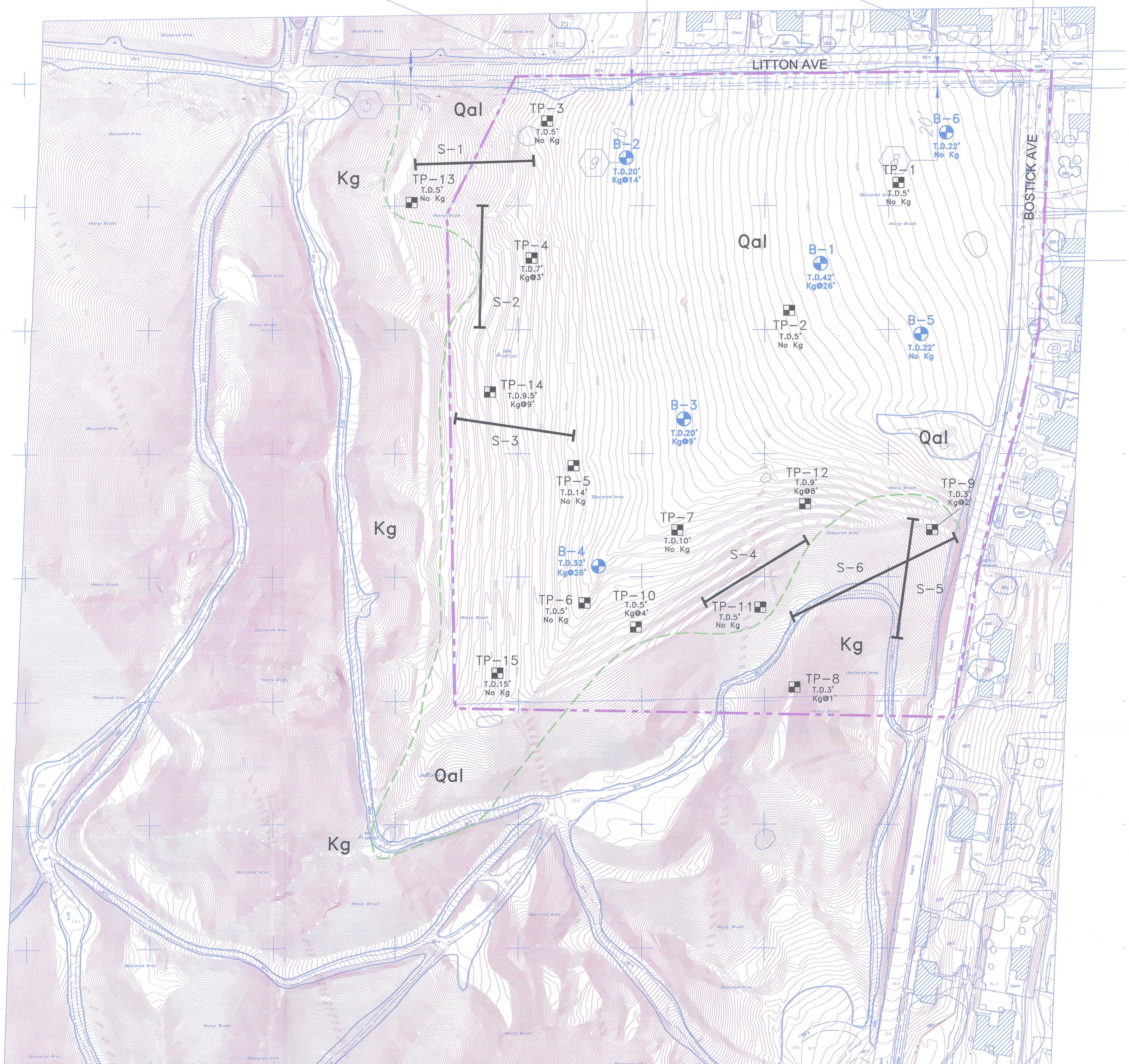
Project Name:	West Property / Colton	Tested By :	VJ	Date:	04/03/06
Project No. :	021906-001	Data Input By:	LF	Date:	04/06/06
Boring No.:	B-6	Depth (ft.) :	0-5		
Sample No. :	B-1				
Soil Identification:	SM				

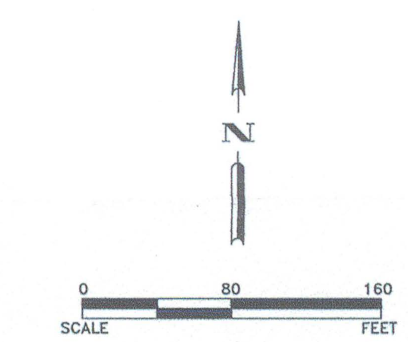
Specimen No.	Water Added (ml) (Wa)	Adjusted Moisture Content (MC)	Resistance Reading (ohm)	Soil Resistivity (ohm-cm)
1	100	12.34	1450	9782
2	200	20.36	980	6611
3	300	28.39	1040	7016
4				
5				

Moisture Content (%) (MCi)	4.32
Wet Wt. of Soil + Cont. (g)	177.02
Dry Wt. of Soil + Cont. (g)	171.80
Wt. of Container (g)	50.85
Container No.	
Initial Soil Wt. (g) (Wt)	1300.00
Box Constant	6.746
MC =(((1+Mci/100)x(Wa/Wt+1))-1)x100	

Min. Resistivity (ohm-cm)	Moisture Content (%)	Sulfate Content (ppm)	Chloride Content (ppm)	Soil pH	
				pH	Temp. (°C)
DOT CA Test 532 / 643		DOT CA Test 417 Part II	DOT CA Test 422	DOT CA Test 532 / 643	
6450	**22.0**	**108**	**42**	**7.70**	**21.6**







LEGEND

Qal	ALLUVIUM
Kg	GRANITIC BEDROCK
B-6 T.D.22' No Kg	APPROXIMATE LOCATION OF HOLLOW STEM AUGER BORING WITH TOTAL DEPTH AND LITHOLOGIC SUMMARY
TP-15 T.D.15' No Kg	APPROXIMATE LOCATION OF BACKHOE TEST PIT WITH TOTAL DEPTH AND LITHOLOGIC SUMMARY
S-6	APPROXIMATE LOCATION OF SEISMIC LINE
	APPROXIMATE LOCATION OF GEOLOGIC CONTACT; CONTACT BETWEEN Qal and Kg IS EXPECTED TO BE HIGHLY VARIABLE DUE TO THE VARIABLE COLLUVIAL THICKNESS AND WEATHERING OF THE UNDERLYING BEDROCK
	APPROXIMATE LIMITS OF THIS STUDY

PLATE 1

GEOTECHNICAL MAP
WEST PROPERTY
COLTON, CALIFORNIA

Leighton and Associates, Inc.
A LEIGHTON GROUP COMPANY

Proj: 021906-001	Scale: 1"=80'	Date: 5/06
Eng/Geol: KS/DPJ	Drafted By: BQT	CP By: BQT

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REPORT OF INFILTRATION TESTING AND UPDATED GEOTECHNICAL RECOMMENDATIONS FOR TRACT 18233, PROPOSED 30-ACRE RESIDENTIAL DEVELOPMENT, SOUTHWEST OF LITTON AVENUE AND BOSTICK AVENUE CITY OF COLTON, CALIFORNIA

Prepared for:

MODERN PACIFIC HOMES, LLC

P.O. Box 7538
Capistrano Beach, California 92629

Project No. 10811.001

September 23, 2016

Leighton and Associates, Inc.

A LEIGHTON GROUP COMPANY

Leighton and Associates, Inc.
A LEIGHTON GROUP COMPANY

September 23, 2016

Project No. 10811.001

Modern Pacific Homes, LLC
P.O. Box 7538
Capistrano Beach, California 92629

Attention: Mr. Scott McKhann

Subject: Report of Infiltration Testing and Updated Geotechnical Recommendations for Tract 18233, Proposed 40-Acre Residential Development, Southwest of Litton Avenue and Bostick Avenue, City of Colton, California

In response to your request, Leighton and Associates, Inc (Leighton) has conducted infiltration testing and is providing updated geotechnical recommendations for the proposed residential development of Tract 18233, located at the southwest corner of Litton and Bostick Avenues in the City of Colton, California. Leighton previously conducted a geotechnical investigation for proposed development in the northern portion of the property (Leighton, 2006). However, plans for the development have changed and development in the southern portion of the property is also now planned. The purpose of this study was to conduct geotechnical explorations in the area of new development in the southern portion of the site and to conduct infiltration testing in areas planned for water quality facilities. This report is considered an addendum to our previous report (Leighton, 2006).

In performing our study we used the Tentative Map for Tract 18233, dated May 2007, prepared by Mayers and Associates Civil Engineers. The civil engineer also provided recommended infiltration locations and depths.

SITE LOCATION AND DESCRIPTION

The entire property is approximately 40 acres in area, located southwest of Bostick and Litton Avenues. Development was originally planned for approximately the northern 19 acres of the property, and the southern portion was to remain undeveloped. A geotechnical investigation for the northern 19 acres was completed previously (Leighton, 2006). Development plans now include the addition of an approximately 1-acre parcel in the southeast portion of the site at the corner of Palm and Bostick Avenues.

Existing single-family residences are present to the north and east of the site. The La Loma Hills rise approximately 400 feet above the site to the west. The site is currently vacant and was previously used for agricultural purposes. The site drains toward the east, away from the La Loma Hills and ultimately towards the Santa Ana River, which is located approximately ½ mile north of the site.

PROPOSED DEVELOPMENT

Current plans include development of approximately 89 residential lots for single-family home sites. Six of these lots are planned in the southern area of the development. Design cuts and fills up to about 15 feet are planned to achieve design grade for the development, and design cut and fill slopes are proposed along the site perimeter and between lots. The development is expected to include drainage, utility, street, sidewalk, landscape and associated improvements.

SCOPE OF INVESTIGATION

The scope of our study has included the following tasks:

- Background Review: A review of the previously prepared geotechnical report, as well as literature, maps and historical aerial photographs relevant to the planned improvements was performed.

- Utility Coordination: We coordinated with Underground Service Alert (USA) to have underground services and/or utility lines located prior to our field investigation.

- Field Exploration: We excavated, logged and sampled three hollow-stem auger borings at the location of the northern water quality basin (LB-7, LB-8 and LB-9) and two hollow-stem auger borings in the newly proposed development in the southern area (LB-10, LB-11). Borings LB-7, LB-10 and LB-11 were drilled to a maximum depth of 20 feet below the existing ground surface (bgs) by a subcontracted drill rig operator. The remaining borings were drilled to the test depth for infiltration testing. The borings were logged by our field representative during drilling. Relatively undisturbed soil samples were obtained at selected intervals within the borings using a California Ring Sampler. Representative bulk soil samples were also collected at shallow depths from the borings.

Well permeameter tests were conducted at 4 boring locations on the site (LB-8 through LB-11) to evaluate general infiltration rates of the subsurface soils at the depths and locations tested. The well permeameter tests were conducted based on the USBR-89 method. The tests were conducted at depths ranging from about 7 to 15 feet (bgs) to estimate the infiltration rate for use of shallow infiltration trenches.

All excavations were backfilled with the soil cuttings. Logs of the geotechnical borings and the well permeameter test results are presented in Appendix B. Approximate boring and well permeameter test locations are shown on the accompanying Geotechnical Map, Plate 1.

- Geotechnical Laboratory Testing: Geotechnical laboratory tests were conducted on selected relatively undisturbed and bulk soil samples obtained during our field investigation. This laboratory testing program was designed to evaluate engineering characteristics of site soils. Laboratory tests conducted during this investigation include:

  - In situ moisture content and dry density
  - Maximum dry density and optimum moisture content
  - Sieve analysis for grain-size distribution
  - Swell and collapse potential
  - Water-soluble sulfate concentration
  - Resistivity, chloride content and pH

  The in situ moisture content and dry density test results are shown on the boring logs, Appendix B. The other laboratory test results are presented in Appendix C.

- Engineering Analysis: Data obtained from our background review, field exploration and geotechnical laboratory testing was evaluated and analyzed to develop geotechnical conclusions and provide preliminary recommendations presented in this report.

- Report Preparation: Results of our geotechnical study have been summarized in this report, presenting our findings, conclusions and updated geotechnical recommendations for design and construction of the proposed residential development.

SUBSURFACE SOIL CONDITIONS

The onsite alluvial soil encountered in our borings was very similar to that observed during our previous study. In the southern portion of the site, the soil was observed to consist of silty sand to sandy silt with varying amounts of clay and gravel. The alluvium was generally observed to be soft to medium stiff in the near subsurface, becoming increasingly stiff with depth. Moisture contents within the alluvial soil samples tested ranged from 1 to 15 percent, averaging around approximately 6 percent. The upper 0.5 to 2 feet of the onsite soil was observed to be highly disturbed and generally loose. In the southern borings (LB-10, LB-11), weathered granitic bedrock was encountered at 15 to 20 feet bgs, but granitic bedrock was also observed at the surface within the area planned for development. Where encountered within our explorations, the bedrock was observed to be slightly moist, very dense, and generally coarse to very coarse grained.

FAULTING AND SEISMICITY

Our review of available in-house literature indicates that there are no known active faults traversing the site. The closest known active or potentially active fault is the Chino-Elsinore fault, located approximately 3 miles southwest of the site.

The principal seismic hazard that could affect the site is ground shaking resulting from an earthquake occurring along several major active or potentially active faults in southern California. The known regional active and potentially active faults that could produce the most significant ground shaking at the site include the Chino-Elsinore, San Jose, Cucamonga, Sierra Madre, Whittier, Elsinore-Glen Ivy, and Elysian Park Thrust faults.

Based on ASCE 7-10 Equation 11.8-1, the F_{PGA} is 1, the PGA is 0.70g, and the PGA_M is 0.70g. As an added check, PGA and hazard deaggregation were also estimated using the United States Geological Survey's (USGS) Interactive Deaggregations utility. The results of this analysis indicate that the predominant modal earthquake has a PHGA of 0.87g with magnitude of approximately 7.0 (M_W) at a distance on the order of 7 kilometers for the Maximum Considered Earthquake (2% probability of exceedance in 50 years). Based on this, the corresponding PHGA for the design earthquake (2/3 of the MCE) is 0.58g. Based on these results, we have selected a design PHGA of 0.70g for seismic analysis of the onsite soils (seismic settlement).

CONCLUSIONS AND RECOMMENDATIONS

Based on our review the geotechnical conditions present in the area of the proposed new development at the southern portion of Tentative Tract 18233 are very similar to those in the area of the originally planned development (Leighton, 2006). As such the findings conclusion and recommendations contained in our original report remain applicable except where modified herein. Based upon this study, we conclude that development of the site is feasible from a geotechnical standpoint, provided the recommendations presented herein and previously (Leighton, 2006) are considered in the design and construction of the development. No severe geologic or soil-related hazards or constraints that would preclude development of the site have been found during the course of this study. Additional geotechnical review, evaluation and investigation may be required based on actual development plans.

GENERAL EARTHWORK AND GRADING

All grading should be performed in accordance with the General Earthwork and Grading Specifications presented in Appendix E, unless specifically revised or amended below or by future recommendations based on final development plans.

FILL PLACEMENT AND COMPACTION

The onsite soil is generally suitable for use as compacted structural fill, provided it is free of debris, organic material, and oversized material (greater than 8 inches in largest dimension). Any soil to be placed as fill, whether onsite or imported material, should be accepted by Leighton and Associates.

All fill soil should be placed in thin, loose lifts, moisture-conditioned, as necessary, to near optimum moisture content, and compacted to a minimum 90 percent relative compaction as determined by ASTM Test Method D1557. Aggregate base should be compacted to a minimum of 95 percent relative compaction.

We are aware of the City of Colton Municipal Code which requires that graded pads, when located in areas designated as susceptible to earth movement due to the existing soil conditions (i.e., La Loma Hills), be compacted using a minimum standard of 95 percent relative compaction. It is our opinion that the slopes on and adjacent to the site are grossly stable. It is our opinion that there is no need to compact graded building pads on this site to a minimum of 95 percent relative compaction and that fill soil placed on this site during development can be compacted using the generally accepted minimum standard of 90 percent relative compaction.

RIPPABILITY AND OVERSIZED MATERIALS

The prevailing alluvial materials onsite should be rippable using conventional heavy equipment in good working condition and using modern earthmoving methods.

Based on the findings of the previous geophysical study (Leighton, 2006), borderline rippable material was encountered onsite. The depth to the borderline rippable material ranged from 0 to 20 feet below the existing ground surface. Blasting may be required depending on the depth of cut and areas planned for development. In addition, hard rock may be encountered in some utility trench excavations.

No oversized material (greater than 8 inches in dimension) was encountered during our investigation. However, oversized materials may be encountered locally during excavation of the alluvial soils, particularly adjacent to the toe of the natural slopes; and during potential excavations in bedrock areas. Oversized rock should be placed in accordance with the recommendations presented in the General Earthwork and Grading Specifications (Appendix E). If cuts are planned requiring blasting, additional recommendations will be required for the placement of rock fill.

SLOPES

Current plans include grading to create manufactured cut and fill slops to a maximum height of about 40 feet. Fill slopes are designed at inclinations of 2:1 (horizontal to vertical) or flatter. The cut slope at the rear of Lots 21 through 30 has been designed at 1.5:1 and is expected to be underlain by granitic bedrock. The granitic bedrock is expected to be dense and generally suitable to support slopes excavated at 1.5:1 with adequate factors of safety for gross and surficial stability. Similar slopes in adjacent developments appear to be functioning well. All slopes should be mapped in detail during grading to confirm that geologic conditions are as anticipated.

UPDATED FOUNDATION RECOMMENDATIONS

Overexcavation and recompaction of the footing subgrade soil should be performed as detailed in Leighton (2006). The following recommendations are based on the onsite soil conditions and soils with a very low expansion potential.

MINIMUM EMBEDMENT AND WIDTH

Based on our preliminary investigation, footings should have a minimum embedment of 18 inches, with a minimum width of 24 and 15 inches for isolated and continuous footings, respectively.

ALLOWABLE BEARING

An allowable bearing pressure of 2,000 pounds-per-square-foot (psf) may be used, based on the minimum embedment depth and width above. This allowable bearing value may be increased by 250 psf per foot increase in depth or width to a maximum allowable bearing pressure of 3,500 psf. If higher bearing pressures are required, this should be reviewed on a case-by-case basis. These allowable bearing pressures are for total dead load and sustained live loads. Footing reinforcement should be designed by the structural engineer.

LATERAL LOAD RESISTANCE

Soil resistance available to withstand lateral loads on a shallow foundation is a function of the frictional resistance along the base of the footing and the passive resistance that may develop as the face of the structure tends to move into the soil. The frictional resistance between the base of the foundation and the subgrade soil may be computed using a coefficient of friction of 0.30. The passive resistance may be computed using an allowable equivalent fluid pressure of 300 pounds per cubic foot (pcf), assuming there is constant contact between the footing and undisturbed soil. The coefficient of friction and passive resistance may be combined without further reduction.

INCREASE IN BEARING AND FRICTION - SHORT DURATION LOADS

The allowable bearing pressure and coefficient of friction values may be increased by one-third when considering loads of short duration, such as those imposed by wind and seismic forces.

RECOMMENDATIONS FOR SLABS-ON-GRADE

Concrete slabs-on-grade should be designed by the structural engineer in accordance with the current CBC for a soil with a very low expansion potential. Where conventional light floor loading conditions exist, the following minimum recommendations should be used. More stringent requirements may be required by local agencies, the structural engineer, the architect, or the CBC. Laboratory testing should be conducted at finish grade to evaluate the Expansion Index (EI) of near-surface subgrade soils. Slabs-on-grade should have the following minimum recommended components:

- Subgrade Moisture Conditioning: The subgrade soil should be moisture conditioned to at least 2 percent above optimum moisture content to a minimum depth of 12 inches prior to placing steel or concrete.

- Moisture Vapor Retarder: A minimum of a 10-mil vapor retarder should be placed below slabs where moisture-sensitive floor coverings or equipment is planned. A heavier/stronger vapor retarder will provide increased protection. Since moisture will otherwise be transmitted up from the soil through the concrete, it is important that an intact vapor retarder be installed. We recommend that the vapor retarder meet the requirements of ASTM E1745 and be installed per ASTM E1643. The structural engineer should specify pertinent concrete design parameters and moisture migration prevention measures, such as whether a sand blotter layer should be placed over the vapor retarder. Gravel or other protruding objects that could puncture the moisture retarder should be removed from the subgrade prior to placing the vapor retarder, or a stronger vapor retarder intended for the specific conditions present can be used.

- Concrete Thickness: Slabs-on-grade should be at least 4 inches thick. Reinforcing steel should be designed by the structural engineer, but as a minimum should be No. 3 rebar placed at 18 inches on center, each direction, mid-depth in the slab.

Minor cracking of the concrete as it cures, due to drying and shrinkage is normal and should be expected. However, cracking is often aggravated by a high water/cement ratio, high concrete temperature at the time of placement, small nominal aggregate size, and rapid moisture loss due to hot, dry, and/or windy weather conditions during placement and curing. Cracking due to temperature and moisture fluctuations can also be expected. Low slump concrete can reduce the potential for shrinkage cracking. Additionally, our experience indicates that reinforcement in slabs and foundations can generally reduce the potential for concrete cracking. The structural engineer should consider these components in slab design and specifications.

Moisture retarders can reduce, but not eliminate moisture vapor rise from the underlying soils up through the slab. Floor covering manufacturers should be consulted for specific recommendations.

Leighton does not practice in the field of moisture vapor transmission evaluation, since this is not specifically a geotechnical issue. Therefore, we recommend that a qualified person, such as the flooring subcontractor and/or structural engineer, be consulted with to evaluate the general and specific moisture vapor transmission paths and any impact on the proposed construction. That person should provide recommendations for mitigation of potential adverse impact of moisture vapor transmission on various components of the structures as deemed appropriate.

SEISMIC DESIGN PARAMETERS

Seismic parameters presented in this report should be considered during project design. In order to reduce the effects of ground shaking produced by regional seismic events, seismic design should be performed in accordance with the most recent edition of the California Building Code (CBC). The following data should be considered for the seismic analysis of the subject site:

2013 CBC Categorization/Coefficient	Design Value
Site Longitude (decimal degrees)	-117.338
Site Latitude (decimal degrees)	34.037
Site Class Definition (ASCE 7 Table 20.3-1)	D
Mapped Spectral Response Acceleration at 0.2s Period, $\mathbf{S_s}$ (Figure 1613.3.1(1))	1.788 g
Mapped Spectral Response Acceleration at 1s Period, $\mathbf{S_1}$ (Figure 1613.3.1(2))	0.786 g
Short Period Site Coefficient at 0.2s Period, $\mathbf{F_a}$ (Table 1613.3.3(1))	1.0
Long Period Site Coefficient at 1s Period, $\mathbf{F_v}$ (Table 1613.3.3(2)	1.5
Adjusted Spectral Response Acceleration at 0.2s Period, $\mathbf{S_{MS}}$ (Eq. 16-37)	1.788 g
Adjusted Spectral Response Acceleration at 1s Period, $\mathbf{S_{M1}}$ (Eq. 16-38)	1.179 g
Design Spectral Response Acceleration at 0.2s Period, $\mathbf{S_{DS}}$ (Eq. 16-39)	1.192 g
Design Spectral Response Acceleration at 1s Period, $\mathbf{S_{D1}}$ (Eq. 16-40)	0.786 g

INFILTRATION TESTING

Four well permeameter tests (LB-8 through LB-11) were conducted to estimate the infiltration rate at specific points and depths of the site. The well permeameter tests were conducted at depths between 7 and 15 feet below ground surface.

A well permeameter test is useful for field measurements of soil infiltration rates, and is suited for testing when the design depth of the basin is deeper than current existing grades. The test consists of excavating a boring to the depth of the test (or deeper if it is partially backfilled with soil and a bentonite plug with a thin soil covering is placed just below the design test elevation). A layer of clean sand is placed in the boring bottom to support a float mechanism and temporary perforated well casing pipe. In addition, sand is poured around the outside of the well casing within the test zone to prevent the boring from caving/collapsing or eroding when water is added. The float mechanism, placed inside the casing, adds water stored in barrels at the top of the hole to the boring as water infiltrates into the soil, while maintaining a constant water head in the boring. The volume percolated during timed intervals is converted to an incremental infiltration velocity, or infiltration rate, such as inches per hour. The rate was converted to an estimate of infiltration rate using the Porchet Method (aka, Inverse Borehole Method). The test was conducted based on the USBR 7300-89 test method.

Infiltration rates were measured at the 4 well permeameter locations and ranged from approximately 0.0 to 0.5 inch per hour (no factor of safety applied). These rates are generally considered to be very low and not suitable for infiltration.

Infiltration test results are provided in Appendix B.

ADDITIONAL GEOTECHNICAL SERVICES

The geotechnical recommendations presented in this report are based on subsurface conditions as interpreted from limited subsurface explorations and limited laboratory testing. Our geotechnical recommendations provided in this report are based on information available at the time the report was prepared and may change as plans are developed. Additional geotechnical investigation and analysis may be required based on final improvement plans. Leighton should review the site and grading plans when available and comment further on the geotechnical aspects of the project. Geotechnical observation and testing should be conducted during excavation and all phases of grading operations. Our conclusions and preliminary recommendations should be reviewed and verified by Leighton during construction and revised accordingly if geotechnical conditions encountered vary from our preliminary findings and interpretations.

Geotechnical observation and testing should be provided:

- After completion of site clearing.
- During overexcavation of compressible soil.
- During compaction of all fill materials.
- After excavation of all footings and prior to placement of concrete.
- During utility trench backfilling and compaction.
- During pavement subgrade and base preparation.
- When any unusual conditions are encountered.

LIMITATIONS

This report was based in part on data obtained from a limited number of observations, site visits, soil excavations, samples, and tests. Such information is, by necessity, incomplete. The nature of many sites is such that differing soil or geologic conditions can be present within small distances and under varying climatic conditions. Changes in subsurface conditions can and do occur over time. Therefore, our findings, conclusions, and recommendations presented in this report are based on the assumption that Leighton and Associates, Inc. will provide geotechnical observation and testing during construction.

This report was prepared for the sole use of client for application to the design of the proposed residential development in accordance with generally accepted geotechnical engineering practices at this time in California.

See the attached GBA (Geoprofessional Business Association) insert for important information about this geotechnical engineering report.

We appreciate the opportunity to work with you on the development of this project. If you have any questions regarding this report, please call us at your convenience.

Respectfully submitted,

LEIGHTON AND ASSOCIATES, INC.

Jason D. Hertzberg, GE 2711
Associate Engineer

Philip A. Buchiarelli, CEG 1715
Principal Geologist

JMD/JDH/PB/lr

Attachments: Important Information About your Geotechnical Report

Figure 1 - Site Location Map

Appendix A - References
Appendix B - Geotechnical Boring Logs and Infiltration Test Results
Appendix C - Laboratory Test Results
Appendix D - Summary of Seismic Hazard Analysis
Appendix E - General Earthwork and Grading Specifications

Plate 1 - Geotechnical Map

Distribution: (1) Addressee

Important Information about This

Geotechnical-Engineering Report

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

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

The Geoprofessional Business Association (GBA) has prepared this advisory to help you – assumedly a client representative – interpret and apply this geotechnical-engineering report as effectively as possible. In that way, clients can benefit from a lowered exposure to the subsurface problems that, for decades, have been a principal cause of construction delays, cost overruns, claims, and disputes. If you have questions or want more information about any of the issues discussed below, contact your GBA-member geotechnical engineer. Active involvement in the Geoprofessional Business Association exposes geotechnical engineers to a wide array of risk-confrontation techniques that can be of genuine benefit for everyone involved with a construction project.

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

Geotechnical engineers structure their services to meet the specific needs of their clients. A geotechnical-engineering study conducted for a given civil engineer will not likely meet the needs of a civil-works constructor or even a different civil engineer. Because each geotechnical-engineering study is unique, each geotechnical-engineering report is unique, prepared *solely* for the client. *Those who rely on a geotechnical-engineering report prepared for a different client can be seriously misled.* No one except authorized client representatives should rely on this geotechnical-engineering report without first conferring with the geotechnical engineer who prepared it. *And no one – not even you – should apply this report for any purpose or project except the one originally contemplated.*

Read this Report in Full

Costly problems have occurred because those relying on a geotechnical-engineering report did not read it *in its entirety*. Do not rely on an executive summary. Do not read selected elements only. *Read this report in full.*

You Need to Inform Your Geotechnical Engineer about Change

Your geotechnical engineer considered unique, project-specific factors when designing the study behind this report and developing the confirmation-dependent recommendations the report conveys. A few typical factors include:

- the client's goals, objectives, budget, schedule, and risk-management preferences;
- the general nature of the structure involved, its size, configuration, and performance criteria;
- the structure's location and orientation on the site; and
- other planned or existing site improvements, such as retaining walls, access roads, parking lots, and underground utilities.

Typical changes that could erode the reliability of this report include those that affect:

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

As a general rule, *always* inform your geotechnical engineer of project changes – even minor ones – and request an assessment of their impact. *The geotechnical engineer who prepared this report cannot accept responsibility or liability for problems that arise because the geotechnical engineer was not informed about developments the engineer otherwise would have considered.*

This Report May Not Be Reliable

Do not rely on this report if your geotechnical engineer prepared it:

- for a different client;
- for a different project;
- for a different site (that may or may not include all or a portion of the original site); or
- before important events occurred at the site or adjacent to it; e.g., man-made events like construction or environmental remediation, or natural events like floods, droughts, earthquakes, or groundwater fluctuations.

Note, too, that it could be unwise to rely on a geotechnical-engineering report whose reliability may have been affected by the passage of time, because of factors like changed subsurface conditions; new or modified codes, standards, or regulations; or new techniques or tools. *If your geotechnical engineer has not indicated an "apply-by" date on the report, ask what it should be*, and, in general, *if you are the least bit uncertain* about the continued reliability of this report, contact your geotechnical engineer before applying it. A minor amount of additional testing or analysis – if any is required at all – could prevent major problems.

Most of the "Findings" Related in This Report Are Professional Opinions

Before construction begins, geotechnical engineers explore a site's subsurface through various sampling and testing procedures. *Geotechnical engineers can observe actual subsurface conditions only at those specific locations where sampling and testing were performed.* The data derived from that sampling and testing were reviewed by your geotechnical engineer, who then applied professional judgment to form opinions about subsurface conditions throughout the site. Actual sitewide-subsurface conditions may differ – maybe significantly – from those indicated in this report. Confront that risk by retaining your geotechnical engineer to serve on the design team from project start to project finish, so the individual can provide informed guidance quickly, whenever needed.

This Report's Recommendations Are Confirmation-Dependent

The recommendations included in this report – including any options or alternatives – are confirmation-dependent. In other words, *they are not final*, because the geotechnical engineer who developed them relied heavily on judgment and opinion to do so. Your geotechnical engineer can finalize the recommendations *only after observing actual subsurface conditions* revealed during construction. If through observation your geotechnical engineer confirms that the conditions assumed to exist actually do exist, the recommendations can be relied upon, assuming no other changes have occurred. *The geotechnical engineer who prepared this report cannot assume responsibility or liability for confirmation-dependent recommendations if you fail to retain that engineer to perform construction observation.*

This Report Could Be Misinterpreted

Other design professionals' misinterpretation of geotechnical-engineering reports has resulted in costly problems. Confront that risk by having your geotechnical engineer serve as a full-time member of the design team, to:

- confer with other design-team members,
- help develop specifications,
- review pertinent elements of other design professionals' plans and specifications, and
- be on hand quickly whenever geotechnical-engineering guidance is needed.

You should also confront the risk of constructors misinterpreting this report. Do so by retaining your geotechnical engineer to participate in prebid and preconstruction conferences and to perform construction observation.

Give Constructors a Complete Report and Guidance

Some owners and design professionals mistakenly believe they can shift unanticipated-subsurface-conditions liability to constructors by limiting the information they provide for bid preparation. To help prevent the costly, contentious problems this practice has caused, include the complete geotechnical-engineering report, along with any attachments or appendices, with your contract documents, *but be certain to note conspicuously that you've included the material for informational purposes only*. To avoid misunderstanding, you may also want to note that "informational purposes" means constructors have no right to rely on the interpretations, opinions, conclusions, or recommendations in the report, but they may rely on the factual data relative to the specific times, locations, and depths/elevations referenced. Be certain that constructors know they may learn about specific project requirements, including options selected from the report, *only* from the design drawings and specifications. Remind constructors that they may perform their own studies if they want to, and *be sure to allow enough time* to permit them to do so. Only then might you be in a position to give constructors the information available to you, while requiring them to at least share some of the financial responsibilities stemming from unanticipated conditions. Conducting prebid and preconstruction conferences can also be valuable in this respect.

Read Responsibility Provisions Closely

Some client representatives, design professionals, and constructors do not realize that geotechnical engineering is far less exact than other engineering disciplines. That lack of understanding has nurtured unrealistic expectations that have resulted in disappointments, delays, cost overruns, claims, and disputes. To confront that risk, geotechnical engineers commonly include explanatory provisions in their reports. Sometimes labeled "limitations," many of these provisions indicate where geotechnical engineers' responsibilities begin and end, to help others recognize their own responsibilities and risks. *Read these provisions closely*. Ask questions. Your geotechnical engineer should respond fully and frankly.

Geoenvironmental Concerns Are Not Covered

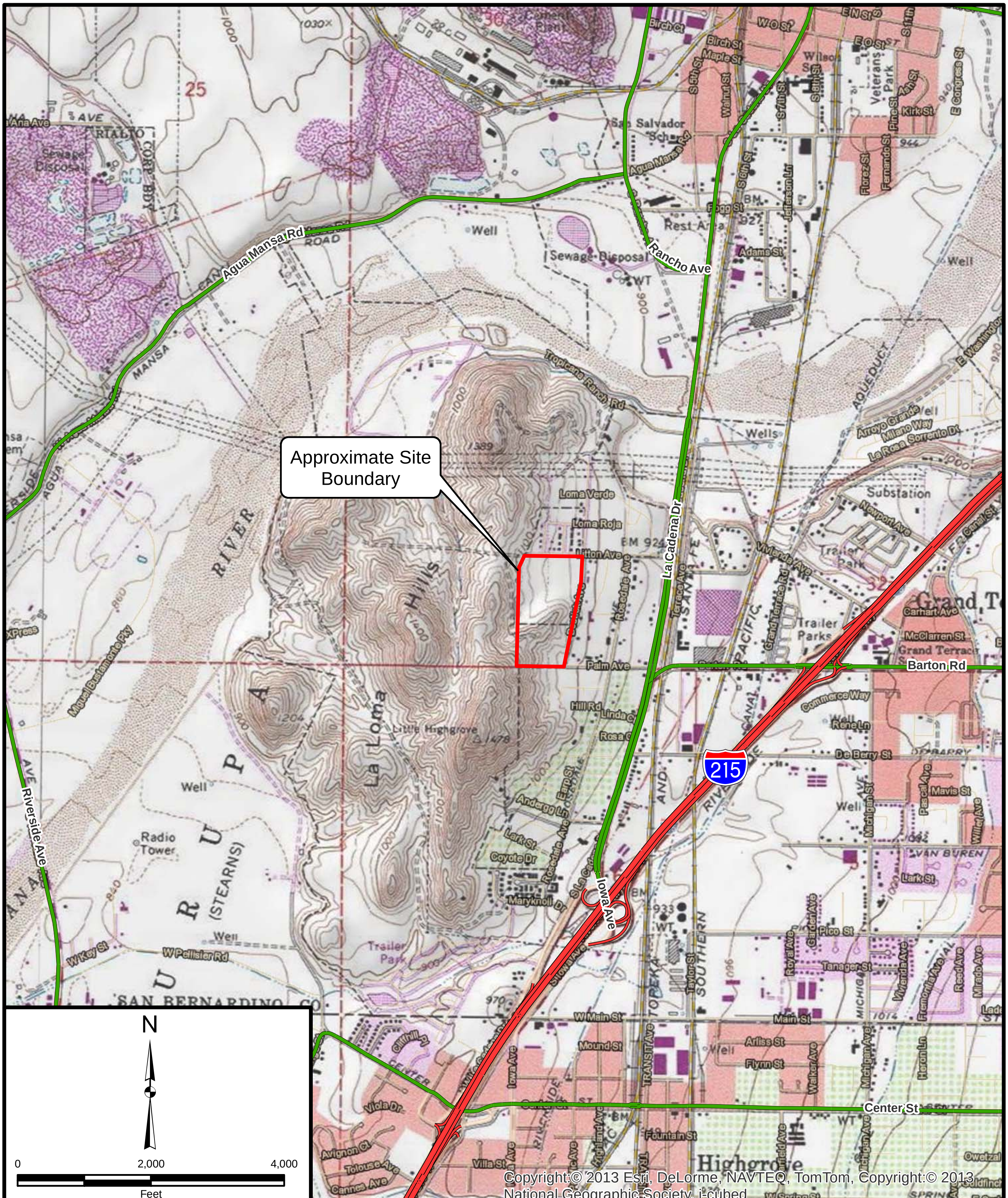
The personnel, equipment, and techniques used to perform an environmental study – e.g., a "phase-one" or "phase-two" environmental site assessment – differ significantly from those used to perform a geotechnical-engineering study. For that reason, a geotechnical-engineering report does not usually relate any environmental findings, conclusions, or recommendations; e.g., about the likelihood of encountering underground storage tanks or regulated contaminants. *Unanticipated subsurface environmental problems have led to project failures*. If you have not yet obtained your own environmental information, ask your geotechnical consultant for risk-management guidance. As a general rule, *do not rely on an environmental report prepared for a different client, site, or project, or that is more than six months old.*

Obtain Professional Assistance to Deal with Moisture Infiltration and Mold

While your geotechnical engineer may have addressed groundwater, water infiltration, or similar issues in this report, none of the engineer's services were designed, conducted, or intended to prevent uncontrolled migration of moisture – including water vapor – from the soil through building slabs and walls and into the building interior, where it can cause mold growth and material-performance deficiencies. Accordingly, ***proper implementation of the geotechnical engineer's recommendations will not of itself be sufficient to prevent moisture infiltration.*** Confront the risk of moisture infiltration by including building-envelope or mold specialists on the design team. ***Geotechnical engineers are not building-envelope or mold specialists.***

GEOPROFESSIONAL BUSINESS ASSOCIATION

Telephone: 301/565-2733
e-mail: info@geoprofessional.org www.geoprofessional.org



Project: 10811.001	Eng/Geol: JDH/PB
Scale:1 " = 2,000 '	Date: October 2014

Base Map: ESRI ArcGIS Online 2014
Thematic Information: Leighton
Author: (btran)

SITE LOCATION MAP

West Property
Southwest of Litton Avenue and Bostick Avenue
Colton, California

Figure 1

Leighton

Map Saved as P:\Drafting\10811\001\Maps\10811-001_F01_SLM_20141015.mxd on 10/15/2014 11:50:20 AM

APPENDIX A

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Leighton

APPENDIX A

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APPENDIX B

GEOTECHNICAL BORING LOGS AND INFILTRATION TEST RESULTS

Leighton

GEOTECHNICAL BORING LOG LB-7

Project No.	10811.001	**Date Drilled**	9-15-14
Project	McKhann Colton	**Logged By**	BER
Drilling Co.	2R Drilling	**Hole Diameter**	9"
Drilling Method	Hollow Stem Auger - 140lb - Autohammer - 30" Drop	**Ground Elevation**	'
Location	See Figure 2	**Sampled By**	BER

SOIL DESCRIPTION

This Soil Description applies only to a location of the exploration at the time of sampling. Subsurface conditions may differ at other locations and may change with time. The description is a simplification of the actual conditions encountered. Transitions between soil types may be gradual.

Elevation Feet	Depth Feet	Graphic Log (N S)	Attitudes	Sample No.	Blows Per 6 Inches	Dry Density pcf	Moisture Content, %	Soil Class. (U.S.C.S.)	Soil Description	Type of Tests
	0			BULK					**Alluvium**	SA, MD, CR
				R-1	6 7 7	101	4	SM	@2' SILTY SAND, stiff, medium brown to orange, dry, fine sand, 40% fines (field estimate), minor pore spaces	
	5			R-2	6 7 8	113	1	ML	@5' SANDY SILT, stiff, medium brown to orange, slightly moist, fine gravel, trace rounded gravel, minor pore spaces, trace rootlets, colluvium	CO
	10			R-3	13 19 34	128	7	ML	@10' SANDY SILT, hard, medium brown to orange/red, slightly moist, fine gravel, same, trace quartz fragements (max size 1/2"), rounded gravel	
	15			R-4	28 50/6"			GW-GM	**Weathered Granitic Bedrock** very dense, medium brown to orange, slightly moist, fine sand, 40% fines (field estimate), subrounded gravel (max size 1/4"), weathered bedrock	
	20			R-5	50/3"			GW-GM	SILTY GRAVEL with sand (GM), very dense, medium brown to orange, slightly moist, fine sand, "40% fines (field estimate), subrounded gravel (max size 1/4"")", weathered bedrock	
									Total Depth = 20' Bedrock @ 15' No groundwater Backfilled with spoils	
	25									
	30									

SAMPLE TYPES:
- B BULK SAMPLE
- C CORE SAMPLE
- G GRAB SAMPLE
- R RING SAMPLE
- S SPLIT SPOON SAMPLE
- T TUBE SAMPLE

TYPE OF TESTS:
- -200 % FINES PASSING
- AL ATTERBERG LIMITS
- CN CONSOLIDATION
- CO COLLAPSE
- CR CORROSION
- CU UNDRAINED TRIAXIAL
- DS DIRECT SHEAR
- EI EXPANSION INDEX
- H HYDROMETER
- MD MAXIMUM DENSITY
- PP POCKET PENETROMETER
- RV R VALUE
- SA SIEVE ANALYSIS
- SE SAND EQUIVALENT
- SG SPECIFIC GRAVITY
- UC UNCONFINED COMPRESSIVE STRENGTH

*** This log is a part of a report by Leighton and should not be used as a stand-alone document. ***

GEOTECHNICAL BORING LOG LB-10

Project No.	10811.001	**Date Drilled**	9-15-14
Project	McKhann Colton	**Logged By**	BER
Drilling Co.	2R Drilling	**Hole Diameter**	9"
Drilling Method	Hollow Stem Auger - 140lb - Autohammer - 30" Drop	**Ground Elevation**	'
Location	See Figure 2	**Sampled By**	BER

SOIL DESCRIPTION

This Soil Description applies only to a location of the exploration at the time of sampling. Subsurface conditions may differ at other locations and may change with time. The description is a simplification of the actual conditions encountered. Transitions between soil types may be gradual.

Elevation Feet	Depth Feet	Graphic Log (N S)	Attitudes	Sample No.	Blows Per 6 Inches	Dry Density pcf	Moisture Content, %	Soil Class. (U.S.C.S.)	SOIL DESCRIPTION	Type of Tests
	0			BULK					**Alluvium**	
				R-1	40/3" 50/3"	102	4	SM	@2' SILTY SAND with gravel (SM), very dense, medium brown, slightly moist, fine to medium sand, 10% fines (field estimate), some quartz chunks (max 1" size), some asphalt chunks.	
	5			R-2	50/5.5"	103	4	GW-GM	**Weathered Granitic Bedrock** @5' SILTY GRAVEL with sand (GM), very dense, medium brown, slightly moist, fine to medium sand, 10% fines (field estimate), quartz chunks, weathered bedrock	
	10			R-3	50/2"				@10' No Recovery, very dense	
	15			R-4	50/4"			GW-GM	@15' SILTY GRAVEL with sand (GM), very dense, light tan, dry, coarse sand, 1" max gravel/rock sizes, only 3 rings of recovery, weathered bedrock	
	20			R-5	50/1.5"			GW-GM	@20' SILTY GRAVEL with sand (GM), very dense, light tan, dry, coarse sand, 1" max rock fragments (granite)	
									Total Depth = 20' Bedrock @ 5' No groundwater Backfilled with spoils	
	25									
	30									

SAMPLE TYPES:
- B BULK SAMPLE
- C CORE SAMPLE
- G GRAB SAMPLE
- R RING SAMPLE
- S SPLIT SPOON SAMPLE
- T TUBE SAMPLE

TYPE OF TESTS:
- -200 % FINES PASSING
- AL ATTERBERG LIMITS
- CN CONSOLIDATION
- CO COLLAPSE
- CR CORROSION
- CU UNDRAINED TRIAXIAL
- DS DIRECT SHEAR
- EI EXPANSION INDEX
- H HYDROMETER
- MD MAXIMUM DENSITY
- PP POCKET PENETROMETER
- RV R VALUE
- SA SIEVE ANALYSIS
- SE SAND EQUIVALENT
- SG SPECIFIC GRAVITY
- UC UNCONFINED COMPRESSIVE STRENGTH

*** This log is a part of a report by Leighton and should not be used as a stand-alone document. ***

GEOTECHNICAL BORING LOG LB-11

Project No.	10811.001	**Date Drilled**	9-15-14
Project	McKhann Colton	**Logged By**	BER
Drilling Co.	2R Drilling	**Hole Diameter**	9"
Drilling Method	Hollow Stem Auger - 140lb - Autohammer - 30" Drop	**Ground Elevation**	'
Location	See Figure 2	**Sampled By**	BER

Elevation Feet	Depth Feet	Graphic Log (N S)	Attitudes	Sample No.	Blows Per 6 Inches	Dry Density pcf	Moisture Content, %	Soil Class. (U.S.C.S.)	SOIL DESCRIPTION *This Soil Description applies only to a location of the exploration at the time of sampling. Subsurface conditions may differ at other locations and may change with time. The description is a simplification of the actual conditions encountered. Transitions between soil types may be gradual.*	Type of Tests
	0			BULK					**Alluvium**	
				R-1	9 16 27	116	5	ML	@2' SANDY SILT with gravel (ML), very stiff, medium olive brown, slightly moist, medium sand, 60% fines (field estimates), subrounded gravel, broken up granite	
	5			R-2	24 31 36	105	10	ML	@5' SILTY SAND with gravel (SM), hard, medium olive brown, slightly moist, fine sand, subrounded gravel (max size 1/4"), colluvium	CO
	10			R-3	13 18 26	119	13	ML	@10' SANDY SILT with gravel (ML), very stiff, dark brown, moist, trace subrounded gravel	
	15			R-4	50/5"			CL	**Weathered Granitic Bedrock** @15' SANDY CLAY with gravel (CL), hard, dark olive brown, moist, coarse sand, low plasticity, chunks of granite rock fragments at bottom of bore (1.5" max size), subrounded gravel	
	20			R-5	50/1.5"				@20' Weathered Bedrock NO RECOVERY, Broken up granite	
									Total Depth = 20' Bedrock @ 15' No groundwater Backfilled with spoils	
	25									
	30									

SAMPLE TYPES:
- B BULK SAMPLE
- C CORE SAMPLE
- G GRAB SAMPLE
- R RING SAMPLE
- S SPLIT SPOON SAMPLE
- T TUBE SAMPLE

TYPE OF TESTS:
- -200 % FINES PASSING
- AL ATTERBERG LIMITS
- CN CONSOLIDATION
- CO COLLAPSE
- CR CORROSION
- CU UNDRAINED TRIAXIAL
- DS DIRECT SHEAR
- EI EXPANSION INDEX
- H HYDROMETER
- MD MAXIMUM DENSITY
- PP POCKET PENETROMETER
- RV R VALUE
- SA SIEVE ANALYSIS
- SE SAND EQUIVALENT
- SG SPECIFIC GRAVITY
- UC UNCONFINED COMPRESSIVE STRENGTH

Results of Well Permeameter Test, from USBR 7300-89 Method.

Leighton

Project: McKhann Colton, Project No. 10811.001

Exploration #/Location: ***LB-8*** Initial Depth to top of float rod (in.) 0

Field Data / **Calculations**

Date (and comments)	Time	Water Level in Supply Barrel (in.)	Depth to top of float rod (when changed) ft	in.	Water Temp in Barrel (deg F)	DL Interpre-tation? ("Y")	DL -- Head of Water in Barrel (in.)	h, Height of Water in Well (in.)	h/r	Total Elapsed Time (minutes) F	Δt (min) G	Vol Change (in.^3) H	Flow (in^3/min)	q, Flow (in^3/hr)	V (Fig 9)	K20, Coef. Of Permeability at 20 deg C (in./hr)	Infiltration Rate [flow/surf area] (in./hr) (FS=1)
Start Date 9/16/2014	Start time: 9:07:00 AM																
9/16/14	9:07	30			91			33.312	7.4	0					0.7		
9/16/14	9:41	29.5			93			32.952	7.3	0	34	208	6	366	0.7	0.07	0.13
9/16/14	10:11	29.25			96			33.312	7.4	0	30	104	3	208	0.7	0.04	0.07
9/16/14	10:39	29			97			33.672	7.5	0	28	104	4	222	0.7	0.04	0.08
9/16/14	11:06	28.75			98			33.552	7.5	0	27	104	4	231	0.7	0.04	0.08
9/16/14	11:36	28.375			100			34.032	7.6	0	30	156	5	311	0.7	0.06	0.10
9/16/14	12:07	28.25			101			33.672	7.5	0	31	52	2	100	0.7	0.02	0.03
9/16/14	12:37	27.875			103			34.512	7.7	0	30	156	5	311	0.7	0.06	0.10
9/16/14	13:04	27.75			104			33.792	7.5	0	27	52	2	115	0.7	0.02	0.04
9/16/14	13:33	27.75			105			33.792	7.5	0	29	0	0	0	0.7	0.00	0.00
										0							

Results of Well Permeameter Test, from USBR 7300-89 Method.

Leighton

Project: **McKhann Colton, Project No. 10811.001**

Exploration #/Location: ***LB-9***

Initial Depth to top of float rod (in.) 0

Field Data **Calculations**

Date (and comments)	Time	Water Level in Supply Barrel (in.)	Depth to top of float rod (when changed)		Water Temp in Barrel (deg F)	DL Interpre-tation? ("Y")	DL -- Head of Water in Barrel (in.)	h, Height of Water in Well (in.)	h/r	Total Elapsed Time (minutes)	Δt (min)	Vol Change (in.^3)	Flow (in^3/min)	q, Flow (in^3/hr)	V (Fig 9)	K20, Coef. Of Permeability at 20 deg C (in./hr)	Infiltration Rate [flow/surf area] (in./hr) (FS=1)
Start Date 9/16/2014	Start time: 9:15:00 AM		ft	in.						E	G	H					
9/16/14	9:15	26.75			91			39.672									
9/16/14	9:44	25.25			92			33.792	7.5	0	29	623	21	1289	0.7	0.25	0.45
9/16/14	10:13	23.5			94			32.712	7.3	0	29	727	25	1504	0.7	0.30	0.53
9/16/14	10:41	21.875			94			32.232	7.2	0	28	675	24	1446	0.7	0.29	0.52
9/16/14	11:08	20.125			95			32.712	7.3	0	27	727	27	1615	0.7	0.32	0.57
9/16/14	11:39	18.5			96			32.112	7.1	0	31	675	22	1306	0.7	0.26	0.47
9/16/14	12:09	16.625			97			32.592	7.2	0	30	779	26	1557	0.7	0.31	0.54
9/16/14	12:38	15			98			31.872	7.1	0	29	675	23	1396	0.7	0.28	0.50
9/16/14	13:09	13			99			32.712	7.3	0	31	831	27	1607	0.7	0.31	0.56
9/16/14	13:34	11.625			99			32.112	7.1	0	25	571	23	1370	0.7	0.27	0.48
										0							

Results of Well Permeameter Test, from USBR 7300-89 Method.

Leighton

Project: **McKhann Colton, Project No. 10811.001**

Exploration #/Location: ***LB-10***

Initial Depth to top of float rod (in.) 0

Field Data | Calculations

Date (and comments)	Time	Water Level in Supply Barrel (in.)	Depth to top of float rod (when changed)		Water Temp in Barrel (deg F)	DL Interpre-tation? ("Y")	DL -- Head of Water in Barrel (in.)	h, Height of Water in Well (in.)	h/r	Total Elapsed Time (minutes)	Δt (min)	Vol Change (in.^3)	Flow (in^3/min)	q, Flow (in^3/hr)	Cumulative Vol (gal)	V (Fig 9)	K20, Coef. Of Permeability at 20 deg C (in./hr)	Infiltration Rate [flow/surf area] (in./hr) (FS=1)
Start Date	Start time:																	
9/16/2014	9:25:00 AM		ft	in.						F	G	H						
9/16/14	**9:25**	26.5			93			37.08										
9/16/14	9:52	22.875			**94**			47.76	10.6	0	27	1505	56	3345	0	0.7	0.37	**0.81**
Flooded																		
9/16/14	10:19	22.875			96			36	8.0		27	0	0	0		0.7	0.00	**0.00**
9/16/14	10:46	22.75			97			38.28	8.5		27	52	2	115		0.7	0.02	**0.03**
9/16/14	11:14	22.75			99			34.32	7.6		28	0	0	0		0.7	0.00	**0.00**
9/16/14	11:45	22.5			100			34.68	7.7		31	104	3	201		0.7	0.04	**0.07**
9/16/14	12:18	21.875			102			34.32	7.6		33	260	8	472		0.7	0.08	**0.15**
9/16/14	12:43	21.625			103			34.8	7.7		25	104	4	249		0.7	0.04	**0.08**
9/16/14	13:14	21			104			34.44	7.7		31	260	8	502		0.7	0.09	**0.16**
9/16/14	13:39	20.75			**105**			33.24	7.4		25	104	4	249		0.7	0.05	**0.08**

Results of Well Permeameter Test, from USBR 7300-89 Method.

Leighton

Project: **McKhann Colton, Project No. 10811.001**

Exploration #/Location: ***LB-11***

Initial Depth to top of float rod (in.) 0

Field Data — **Calculations**

Date (and comments)	Time	**Water Level in Supply Barrel (in.)**	Depth to top of float rod (when changed)		Water Temp in Barrel (deg F)	DL Interpre-tation? ("Y")	DL -- Head of Water in Barrel (in.)	h, Height of Water in Well (in.)	h/r	Total Elapsed Time (minutes)	Δt (min)	Vol Change (in.^3)	Flow (in^3/min)	q, Flow (in^3/hr)	Cumulative Vol (gal)	V (Fig 9)	K20, Coef. Of Permeability at 20 deg C (in./hr)	**Infiltration Rate [flow/surf area] (in./hr) (FS=1)**
Start Date 9/16/2014	Start time: 9:13:00 AM		ft	in.														
9/16/14	**9:13**	29.75			93			21.912										
9/16/14	9:55	29.625			**95**			23.472	5.5	0	42	52	1	74	0	0.7	0.02	**0.04**
9/16/14	10:22	29.625			97			23.232	5.5		27	0	0	0		0.7	0.00	**0.00**
9/16/14	10:48	29.5			98			22.992	5.4		26	52	2	120		0.7	0.04	**0.07**
9/16/14	11:17	29.5			100			22.752	5.4		29	0	0	0		0.7	0.00	**0.00**
9/16/14	11:47	29.5			102			22.392	5.3		30	0	0	0		0.7	0.00	**0.00**
9/16/14	12:20	29.5			103			22.272	5.2		33	0	0	0		0.7	0.00	**0.00**
9/16/14	12:46	29.5			104			22.512	5.3		26	0	0	0		0.7	0.00	**0.00**
9/16/14	13:16	29.5			105			22.272	5.2		30	0	0	0		0.7	0.00	**0.00**
9/16/14	13:41	29.375			106			22.152	5.2		25	52	2	125		0.7	0.04	**0.07**

APPENDIX C

LABORATORY TEST RESULTS

Leighton

TESTS for SULFATE CONTENT
CHLORIDE CONTENT and pH of SOILS

Project Name:	McKhann Colton	Tested By :	G. Berdy	Date:	09/24/14
Project No. :	10811.001	Data Input By:	J. Ward	Date:	10/01/14

Boring No.	LB-1			
Sample No.	B-1			
Sample Depth (ft)	2.0			
Soil Identification:	Olive brown s(ML)			
Wet Weight of Soil + Container (g)	221.00			
Dry Weight of Soil + Container (g)	215.87			
Weight of Container (g)	51.43			
Moisture Content (%)	3.12			
Weight of Soaked Soil (g)	100.91			

SULFATE CONTENT, DOT California Test 417, Part II

Beaker No.	36			
Crucible No.	6			
Furnace Temperature (°C)	840			
Time In / Time Out	13:25/14:10			
Duration of Combustion (min)	45			
Wt. of Crucible + Residue (g)	23.3485			
Wt. of Crucible (g)	23.3435			
Wt. of Residue (g) (A)	0.0050			
PPM of Sulfate (A) x 41150	205.75			
PPM of Sulfate, Dry Weight Basis	**212**			

CHLORIDE CONTENT, DOT California Test 422

ml of Extract For Titration (B)	30			
ml of AgNO3 Soln. Used in Titration (C)	2.0			
PPM of Chloride (C -0.2) * 100 * 30 / B	180			
PPM of Chloride, Dry Wt. Basis	**186**			

pH TEST, DOT California Test 532/643

pH Value	**7.50**			
Temperature °C	22.3			

SOIL RESISTIVITY TEST

DOT CA TEST 532 / 643

Project Name:	McKhann Colton	Tested By :	G. Berdy Date: 09/29/14
Project No. :	10811.001	Data Input By:	J. Ward Date: 10/01/14
Boring No.:	LB-1	Depth (ft.) :	2.0
Sample No. :	B-1		

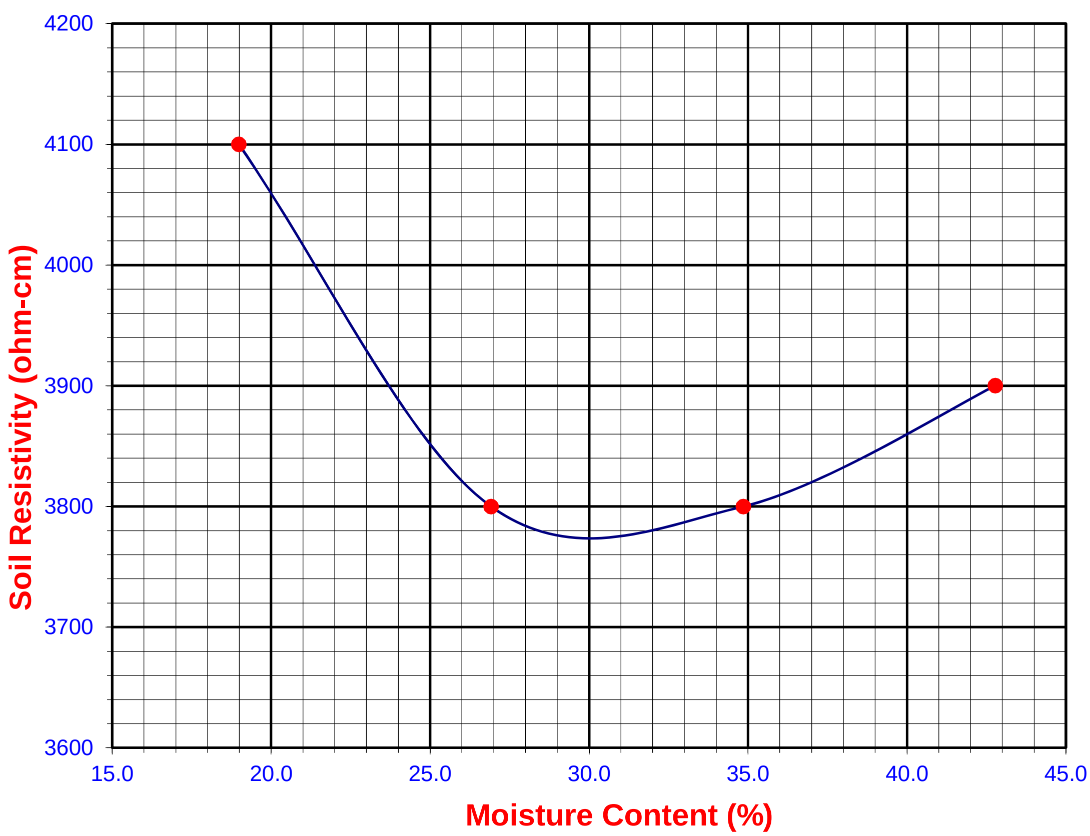
Soil Identification:* Olive brown s(ML)

*California Test 643 requires soil specimens to consist only of portions of samples passing through the No. 8 US Standard Sieve before resistivity testing. Therefore, this test method may not be representative for coarser materials.

Specimen No.	Water Added (ml) (Wa)	Adjusted Moisture Content (MC)	Resistance Reading (ohm)	Soil Resistivity (ohm-cm)
1	20	18.98	4100	4100
2	30	26.92	3800	3800
3	40	34.85	3800	3800
4	50	42.78	3900	3900
5				

Moisture Content (%) (MCi)	3.12
Wet Wt. of Soil + Cont. (g)	221.00
Dry Wt. of Soil + Cont. (g)	215.87
Wt. of Container (g)	51.43
Container No.	
Initial Soil Wt. (g) (Wt)	130.00
Box Constant	1.000
MC =(((1+Mci/100)x(Wa/Wt+1))-1)x100	

Min. Resistivity (ohm-cm)	Moisture Content (%)	Sulfate Content (ppm)	Chloride Content (ppm)	Soil pH	
				pH	Temp. (°C)
DOT CA Test 532 / 643		DOT CA Test 417 Part II	DOT CA Test 422	DOT CA Test 532 / 643	
3775	**30.0**	**212**	**186**	**7.50**	**22.3**



MODIFIED PROCTOR COMPACTION TEST

ASTM D 1557

Leighton

Project Name:	McKhann Colton	Tested By: O. Figueroa	Date: 09/26/14
Project No.:	10811.001	Input By: J. Ward	Date: 10/01/14
Boring No.:	LB-1	Depth (ft.): 2.0	
Sample No.:	B-1		
Soil Identification:	Olive brown sandy silt s(ML)		

Preparation Method: [X] Moist [ ] Dry — [X] Mechanical Ram [ ] Manual Ram

Mold Volume (ft³) 0.03320 — *Ram Weight = 10 lb.; Drop = 18 in.*

TEST NO.	1	2	3	4	5	6
Wt. Compacted Soil + Mold (g)	3796.0	3903.0	3929.0			
Weight of Mold (g)	1835.0	1835.0	1835.0			
Net Weight of Soil (g)	1961.0	2068.0	2094.0			
Wet Weight of Soil + Cont. (g)	441.80	444.40	476.60			
Dry Weight of Soil + Cont. (g)	421.60	416.00	436.90			
Weight of Container (g)	50.50	50.10	54.20			
Moisture Content (%)	5.44	7.76	10.37			
Wet Density (pcf)	130.2	137.3	139.0			
Dry Density (pcf)	123.5	127.4	126.0			

Maximum Dry Density (pcf) **127.5** **Optimum Moisture Content (%)** **8.5**

PROCEDURE USED

[X] **Procedure A**
Soil Passing No. 4 (4.75 mm) Sieve
Mold : 4 in. (101.6 mm) diameter
Layers : 5 (Five)
Blows per layer : 25 (twenty-five)
May be used if +#4 is 20% or less

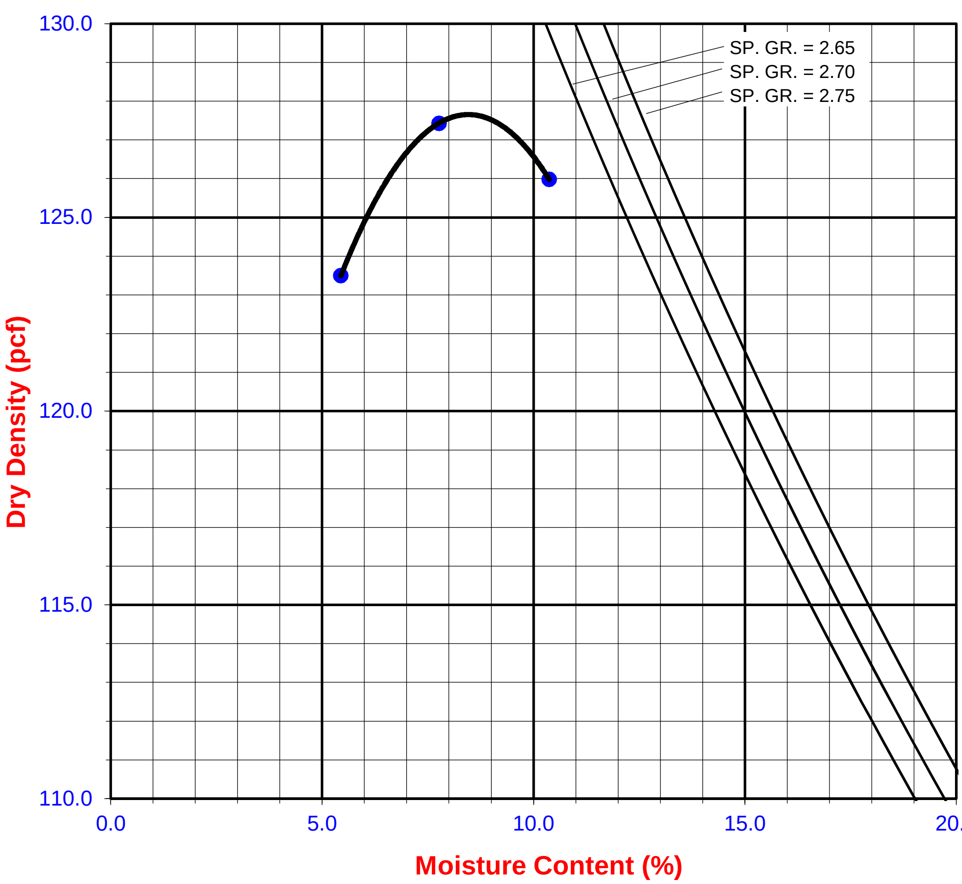
[ ] **Procedure B**
Soil Passing 3/8 in. (9.5 mm) Sieve
Mold : 4 in. (101.6 mm) diameter
Layers : 5 (Five)
Blows per layer : 25 (twenty-five)
Use if +#4 is >20% and +3/8 in. is 20% or less

[ ] **Procedure C**
Soil Passing 3/4 in. (19.0 mm) Sieve
Mold : 6 in. (152.4 mm) diameter
Layers : 5 (Five)
Blows per layer : 56 (fifty-six)
Use if +3/8 in. is >20% and +¾ in. is <30%

Particle-Size Distribution:
2:41:57
GR:SA:FI

Atterberg Limits:

LL,PL,PI



Leighton

PARTICLE-SIZE DISTRIBUTION (GRADATION) of SOILS USING SIEVE ANALYSIS

ASTM D 6913

Project Name: McKhann Colton

Project No.: 10811.001

Boring No.: LB-1

Sample No.: B-1

Soil Identification: Olive brown sandy silt s(ML)

Tested By: ACS/SF Date: 09/28/14

Checked By: J. Ward Date: 10/01/14

Depth (feet): 2.0

Calculation of Dry Weights	Whole Sample	Sample Passing #4	Moisture Contents	Whole Sample	Sample passing #4
Container No.:	1F	979	Wt. of Air-Dry Soil + Cont.(g)	0.00	0.00
Wt. Air-Dried Soil + Cont.(g)	2216.9	621.7	Wt. of Dry Soil + Cont. (g)	0.00	0.00
Wt. of Container (g)	223.7	111.8	Wt. of Container No._____(g)	1.00	1.00
Dry Wt. of Soil (g)	1993.2	509.9	Moisture Content (%)	0.00	0.00

Passing #4 Material After Wet Sieve		
	Container No.	979
	Wt. of Dry Soil + Container (g)	340.1
	Wt. of Container (g)	111.8
	Dry Wt. of Soil Retained on # 200 Sieve (g)	228.3

U. S. Sieve Size		Cumulative Weight of Dry Soil Retained (g)		Percent Passing (%)
	(mm.)	Whole Sample	Sample Passing #4	
3"	75.000			
1 1/2"	37.500			
3/4"	19.000	0.0		100.0
3/8"	9.500	12.3		99.4
#4	4.750	35.0		98.2
#8	2.360		20.6	94.2
#16	1.180		60.4	86.6
#30	0.600		92.8	80.3
#50	0.300		120.7	75.0
#100	0.150		149.2	69.5
#200	0.075		212.3	57.3
PAN				

GRAVEL: **2 %**

SAND: **41 %**

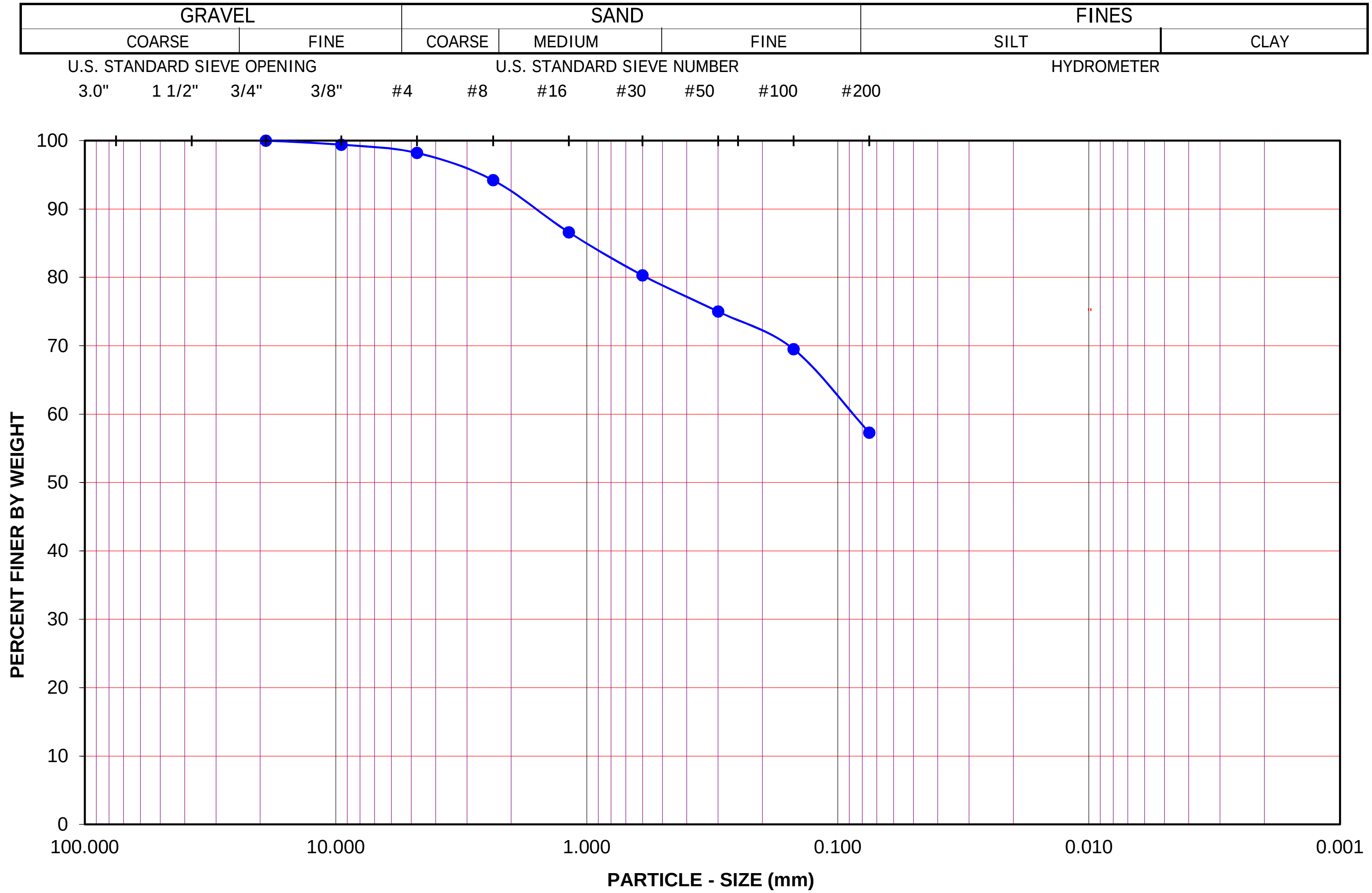
FINES: **57 %**

GROUP SYMBOL: **s(ML)**

Cu = D60/D10 = ______

Cc = $(D30)^2/(D60*D10)$ = ______

Remarks:



Project Name: McKhann Colton

Project No.: 10811.001

Boring No.: LB-1

Sample No.: B-1

Depth (feet): 2.0

Soil Type : s(ML)

Soil Identification: Olive brown sandy silt s(ML)

Leighton

PARTICLE - SIZE DISTRIBUTION ASTM D 6913

GR:SA:FI : (%) 2 : 41 : 57

Oct-14

PARTICLE-SIZE DISTRIBUTION (GRADATION) of SOILS USING SIEVE ANALYSIS

ASTM D 6913

Leighton

Project Name:	McKhann Colton	Tested By:	ACS/SF	Date:	09/26/14
Project No.:	10811.001	Checked By:	J. Ward	Date:	10/02/14
Boring No.:	LB-1A	Depth (feet):	13.0		
Sample No.:	R-1				
Soil Identification:	Olive brown sandy lean clay s(CL)				

		Moisture Content of Total Air - Dry Soil	
Container No.:	191	Wt. of Air-Dry Soil + Cont. (g)	0.00
Wt. of Air-Dried Soil + Cont.(g)	764.4	Wt. of Dry Soil + Cont. (g)	0.00
Wt. of Container (g)	161.0	Wt. of Container No.______ (g)	1.00
Dry Wt. of Soil (g)	603.4	Moisture Content (%)	0.00

After Wet Sieve	Container No.	191
	Wt. of Dry Soil + Container (g)	415.3
	Wt. of Container (g)	161.0
	Dry Wt. of Soil Retained on # 200 Sieve (g)	254.3

U. S. Sieve Size		Cumulative Weight Dry Soil Retained (g)	Percent Passing (%)
(in.)	(mm.)		
6"	152.400		
3"	75.000		
1 1/2	37.500		
3/4"	19.000		
3/8"	9.500	0.0	100.0
#4	4.750	6.4	98.9
#8	2.360	29.5	95.1
#16	1.180	67.0	88.9
#30	0.600	101.7	83.1
#50	0.300	132.1	78.1
#100	0.150	164.7	72.7
#200	0.075	236.8	60.8
PAN			

GRAVEL: **1 %**

SAND: **38 %**

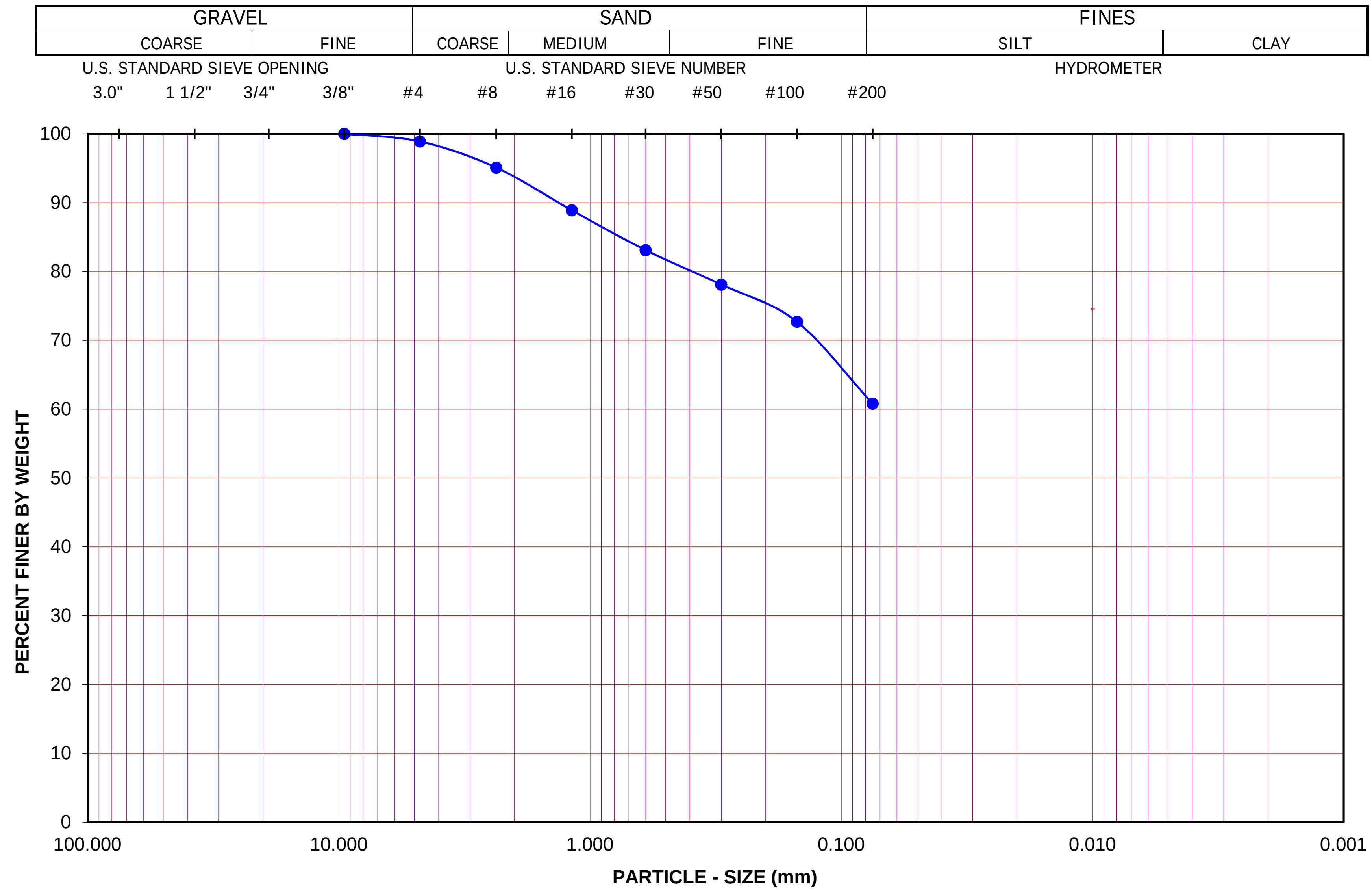
FINES: **61 %**

GROUP SYMBOL: **s(CL)**

Cu = D60/D10 = ______

Cc = $(D30)^2/(D60*D10)$ = ______

Remarks:



Project Name: McKhann Colton

Project No.: 10811.001

Boring No.: LB-1A

Sample No.: R-1

Depth (feet): 13.0

Soil Type : s(CL)

Soil Identification: Olive brown sandy lean clay s(CL)

Leighton

PARTICLE - SIZE DISTRIBUTION ASTM D 6913

GR:SA:FI : (%) 1 : 38 : 61

Oct-14

PARTICLE-SIZE DISTRIBUTION (GRADATION) of SOILS USING SIEVE ANALYSIS

ASTM D 6913

Leighton

Project Name:	McKhann Colton	Tested By:	ACS/SF	Date:	09/28/14
Project No.:	10811.001	Checked By:	J. Ward	Date:	10/01/14
Boring No.:	LB-1B	Depth (feet):	9.0		
Sample No.:	R-1				
Soil Identification:	Brown sandy lean clay s(CL)				

		Moisture Content of Total Air - Dry Soil	
Container No.:	969	Wt. of Air-Dry Soil + Cont. (g)	0.00
Wt. of Air-Dried Soil + Cont.(g)	810.7	Wt. of Dry Soil + Cont. (g)	0.00
Wt. of Container (g)	110.7	Wt. of Container No.______ (g)	1.00
Dry Wt. of Soil (g)	700.0	Moisture Content (%)	0.00

After Wet Sieve	Container No.	969
	Wt. of Dry Soil + Container (g)	376.4
	Wt. of Container (g)	110.7
	Dry Wt. of Soil Retained on # 200 Sieve (g)	265.7

U. S. Sieve Size		Cumulative Weight Dry Soil Retained (g)	Percent Passing (%)
(in.)	(mm.)		
6"	152.400		
3"	75.000		
1 1/2	37.500		
3/4"	19.000		
3/8"	9.500	0.0	100.0
#4	4.750	5.5	99.2
#8	2.360	26.4	96.2
#16	1.180	60.4	91.4
#30	0.600	95.7	86.3
#50	0.300	124.7	82.2
#100	0.150	158.6	77.3
#200	0.075	242.7	65.3
PAN			

GRAVEL: **1 %**

SAND: **34 %**

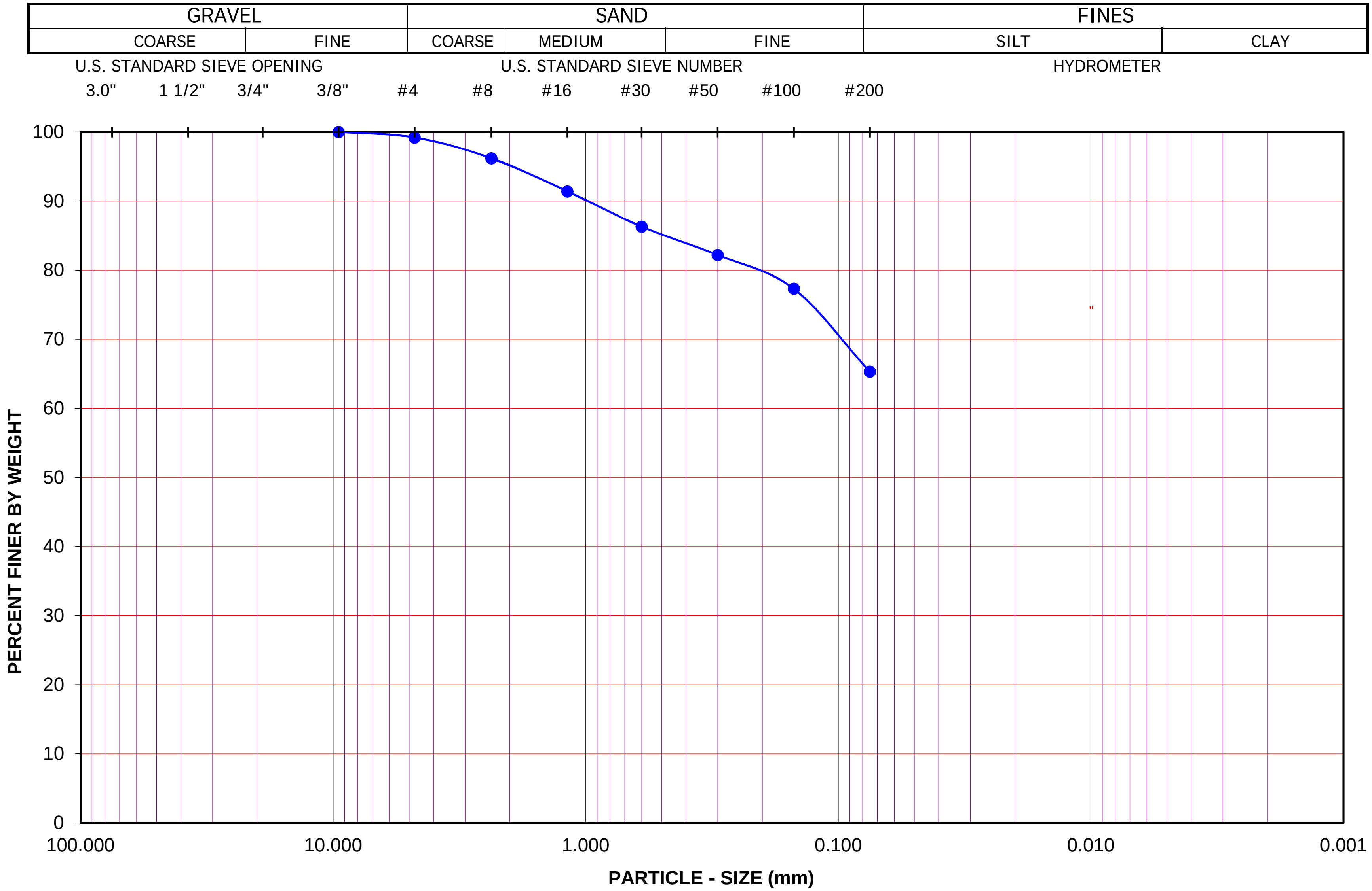
FINES: **65 %**

GROUP SYMBOL: **s(CL)**

Cu = D60/D10 = ______

Cc = (D30)²/(D60*D10) = ______

Remarks:



Project Name: McKhann Colton

Project No.: 10811.001

Boring No.: LB-1B | Sample No.: R-1

Depth (feet): 9.0 | Soil Type : s(CL)

Soil Identification: Brown sandy lean clay s(CL)

Leighton

PARTICLE - SIZE DISTRIBUTION ASTM D 6913

GR:SA:FI : (%) 1 : 34 : 65

Oct-14

SA LB-1B, R-1 @ 9

ONE-DIMENSIONAL SWELL OR SETTLEMENT POTENTIAL OF COHESIVE SOILS
(ASTM D 4546)

Leighton

Project Name:	McKhann Colton	Tested By:	G. Bathala	Date:	09/26/14
Project No.:	10811.001	Checked By:	J. Ward	Date:	10/01/14
Boring No.:	LB-1	Sample Type:	Ring		
Sample No.:	R-2	Depth (ft.)	5.0		
Sample Description:	Brown silty clay (CL-ML)				

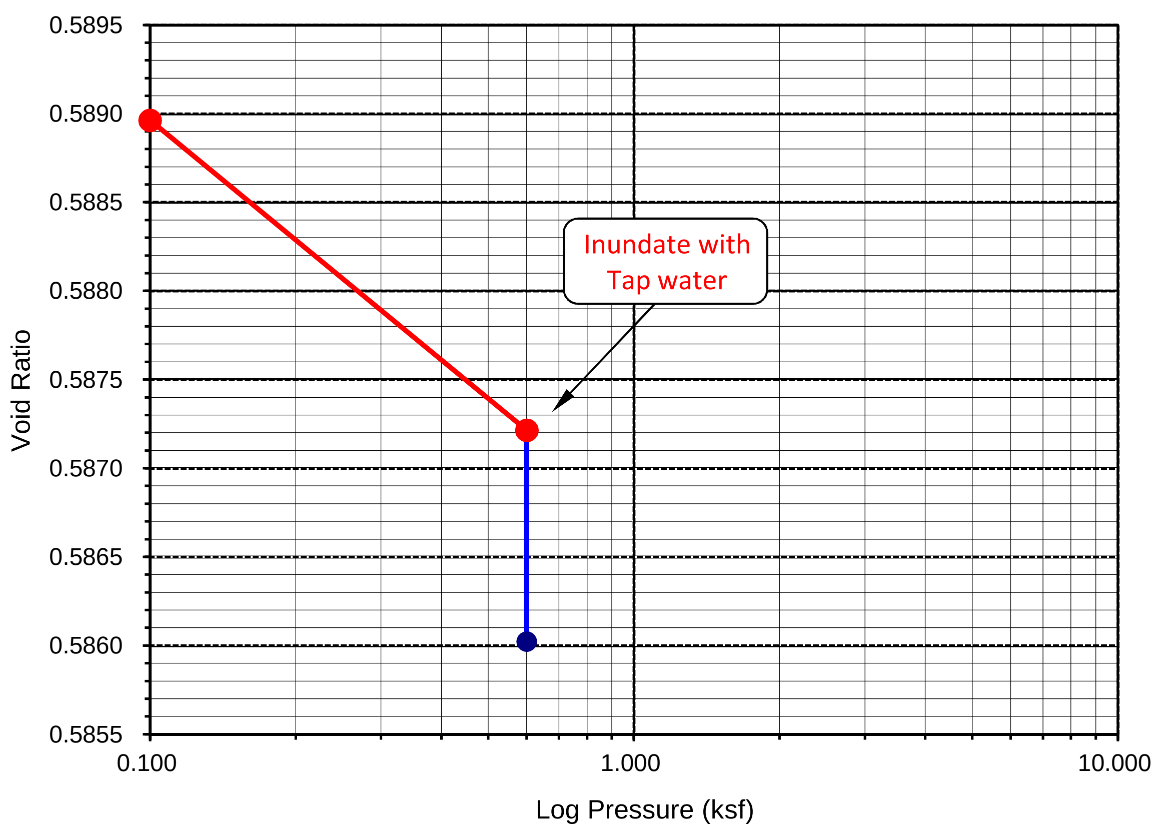
Initial Dry Density (pcf):	106.1
Initial Moisture (%):	4.29
Initial Length (in.):	1.0000
Initial Dial Reading:	0.2667
Diameter(in):	2.415

Final Dry Density (pcf):	106.3
Final Moisture (%) :	19.3
Initial Void Ratio:	0.5890
Specific Gravity(assumed):	2.70
Initial Saturation (%)	19.7

Pressure (p) (ksf)	Final Reading (in)	Apparent Thickness (in)	Load Compliance (%)	Swell (+) Settlement (-) % of Sample Thickness	Void Ratio	Corrected Deformation (%)
0.100	0.2667	1.0000	0.00	0.00	0.5890	0.00
0.600	0.2649	0.9982	0.07	-0.19	0.5872	-0.12
H_2O	0.2641	0.9974	0.07	-0.26	0.5860	-0.19

Percent Swell (+) / Settlement (-) After Inundation = -0.08

Void Ratio - Log Pressure Curve



ONE-DIMENSIONAL SWELL OR SETTLEMENT POTENTIAL OF COHESIVE SOILS
(ASTM D 4546)

Leighton

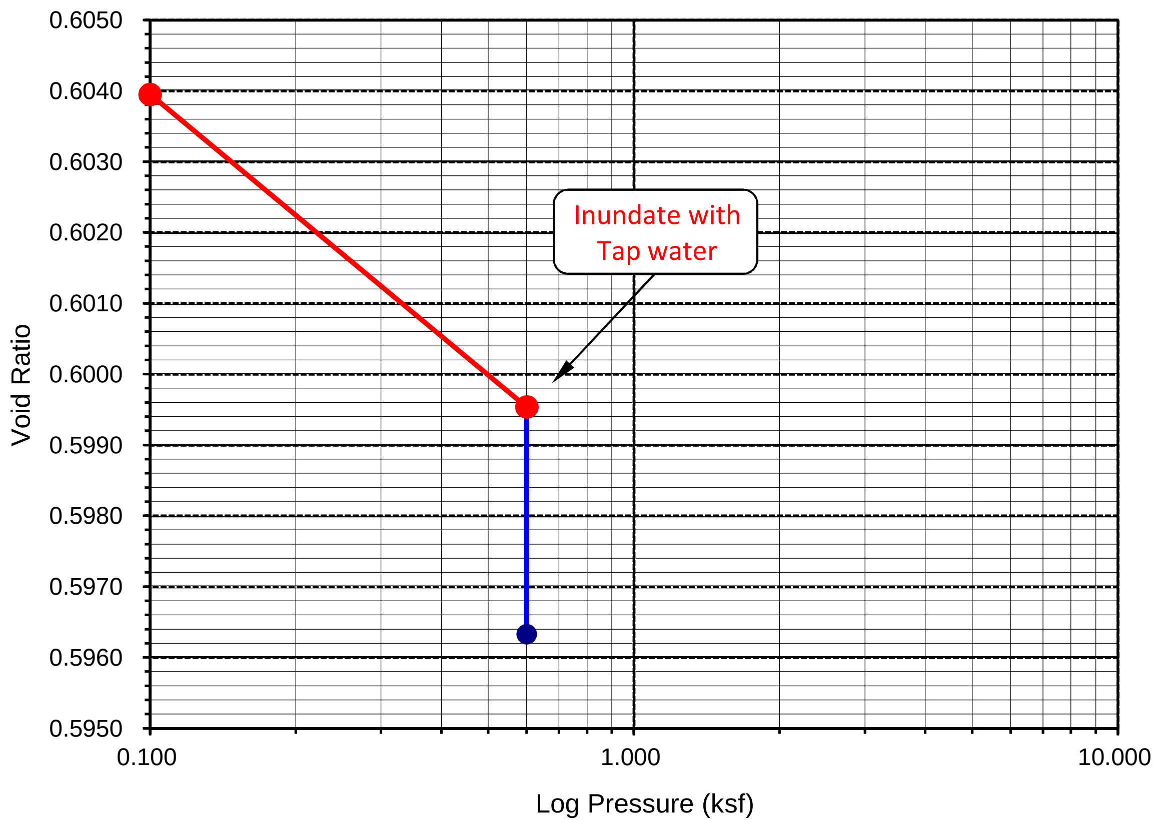
Project Name:	McKhann Colton	Tested By:	G. Bathala	Date:	09/26/14
Project No.:	10811.001	Checked By:	J. Ward	Date:	10/01/14
Boring No.:	LB-1	Sample Type:	Ring		
Sample No.:	R-2	Depth (ft.)	5.0		
Sample Description:	Olive brown silty clay (CL-ML)				

Initial Dry Density (pcf):	105.1	Final Dry Density (pcf):	105.6
Initial Moisture (%):	9.32	Final Moisture (%) :	19.2
Initial Length (in.):	1.0000	Initial Void Ratio:	0.6039
Initial Dial Reading:	0.3292	Specific Gravity(assumed):	2.70
Diameter(in):	2.415	Initial Saturation (%)	41.7

Pressure (p) (ksf)	Final Reading (in)	Apparent Thickness (in)	Load Compliance (%)	Swell (+) Settlement (-) % of Sample Thickness	Void Ratio	Corrected Deformation (%)
0.100	0.3292	1.0000	0.00	0.00	0.6039	0.00
0.600	0.3256	0.9965	0.08	-0.35	0.5995	-0.27
H2O	0.3236	0.9945	0.08	-0.55	0.5963	-0.47

Percent Swell (+) / Settlement (-) After Inundation = -0.20

Void Ratio - Log Pressure Curve



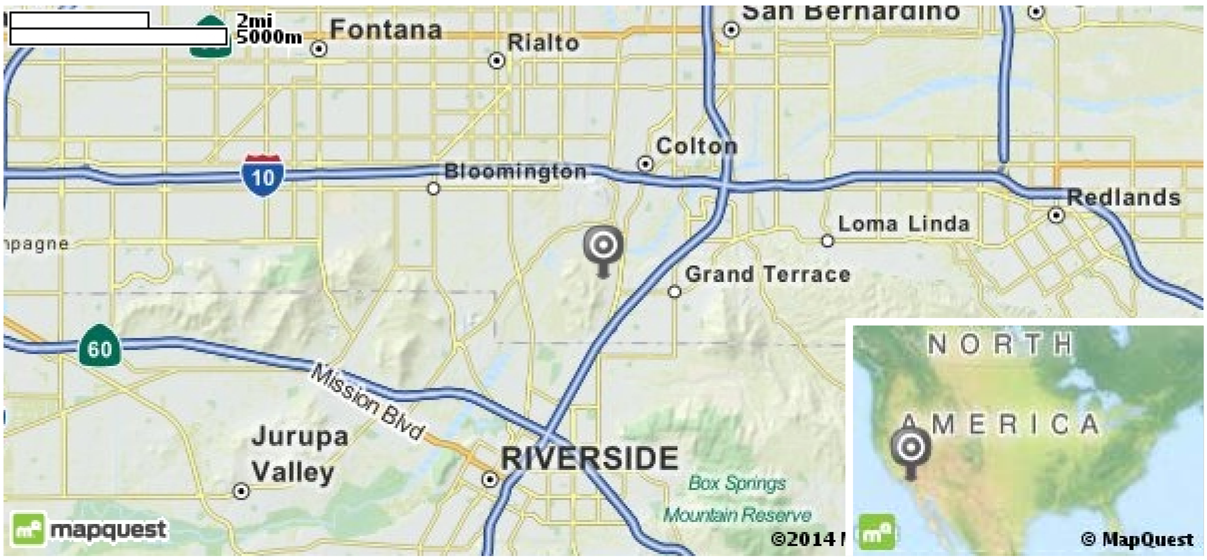
APPENDIX D

SUMMARY OF SEISMIC HAZARD ANALYSIS

USGS Design Maps Summary Report

User–Specified Input

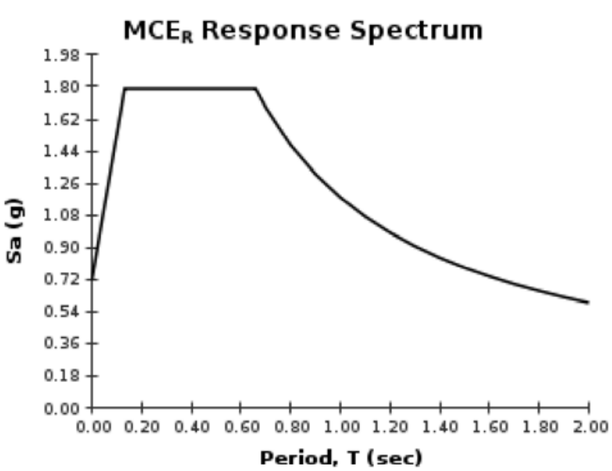
Report Title	McKhann Property Fri October 10, 2014 16:16:25 UTC
Building Code Reference Document	2012 International Building Code (which utilizes USGS hazard data available in 2008)
Site Coordinates	34.0371°N, 117.3379°W
Site Soil Classification	Site Class D – "Stiff Soil"
Risk Category	I/II/III

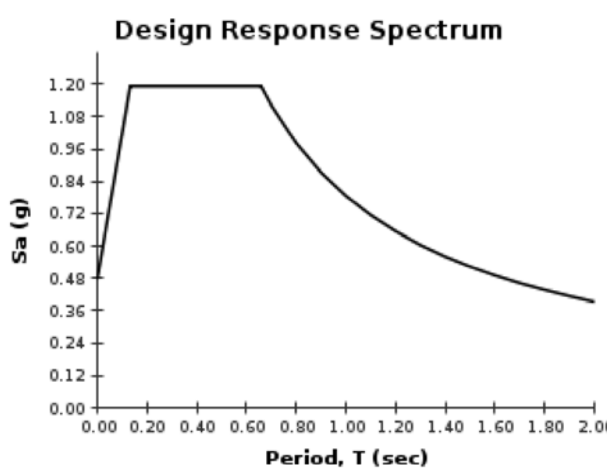


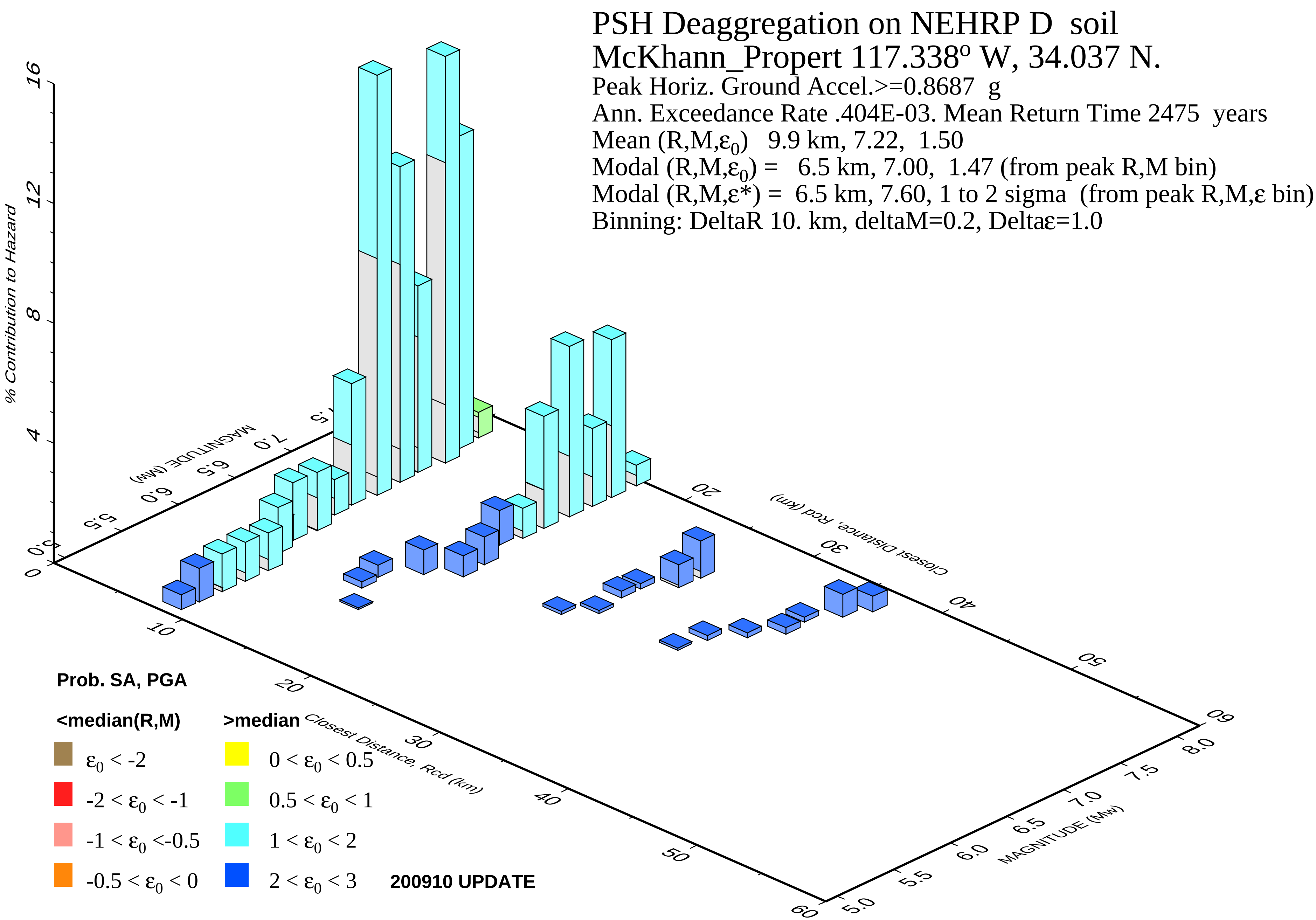
USGS–Provided Output

S_S =	1.788 g	S_{MS} =	1.788 g	S_{DS} =	1.192 g
S_1 =	0.786 g	S_{M1} =	1.179 g	S_{D1} =	0.786 g

For information on how the SS and S1 values above have been calculated from probabilistic (risk-targeted) and deterministic ground motions in the direction of maximum horizontal response, please return to the application and select the "2009 NEHRP" building code reference document.







GMT 2014 Oct 10 16:16:01 Distance (R), magnitude (M), epsilon (E0,E) deaggregation for a site on soil with average vs= 270. m/s top 30 m. USGS CGHT PSHA2008 UPDATE Bins with lt 0.05% contrib. omitted

APPENDIX E

GENERAL EARTHWORK AND GRADING SPECIFICATIONS

Leighton

APPENDIX E
LEIGHTON AND ASSOCIATES, INC.
GENERAL EARTHWORK AND GRADING SPECIFICATIONS FOR ROUGH GRADING

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LEIGHTON AND ASSOCIATES, INC.

GENERAL EARTHWORK AND GRADING SPECIFICATIONS FOR ROUGH GRADING

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Standard Details

Leighton

1.0 GENERAL

1.1 Intent

These General Earthwork and Grading Specifications are for the grading and earthwork shown on the approved grading plan(s) and/or indicated in the geotechnical report(s). These Specifications are a part of the recommendations contained in the geotechnical report(s). In case of conflict, the specific recommendations in the geotechnical report shall supersede these more general Specifications. Observations of the earthwork by the project Geotechnical Consultant during the course of grading may result in new or revised recommendations that could supersede these specifications or the recommendations in the geotechnical report(s).

1.2 The Geotechnical Consultant of Record

Prior to commencement of work, the owner shall employ the Geotechnical Consultant of Record (Geotechnical Consultant). The Geotechnical Consultants shall be responsible for reviewing the approved geotechnical report(s) and accepting the adequacy of the preliminary geotechnical findings, conclusions, and recommendations prior to the commencement of the grading.

Prior to commencement of grading, the Geotechnical Consultant shall review the "work plan" prepared by the Earthwork Contractor (Contractor) and schedule sufficient personnel to perform the appropriate level of observation, mapping, and compaction testing.

During the grading and earthwork operations, the Geotechnical Consultant shall observe, map, and document the subsurface exposures to verify the geotechnical design assumptions. If the observed conditions are found to be significantly different than the interpreted assumptions during the design phase, the Geotechnical Consultant shall inform the owner, recommend appropriate changes in design to accommodate the observed conditions, and notify the review agency where required. Subsurface areas to be geotechnically observed, mapped, elevations recorded, and/or tested include natural ground after it has been cleared for receiving fill but before fill is placed, bottoms of all "remedial removal" areas, all key bottoms, and benches made on sloping ground to receive fill.

The Geotechnical Consultant shall observe the moisture-conditioning and processing of the subgrade and fill materials and perform relative compaction testing of fill to determine the attained level of compaction.

The Geotechnical Consultant shall provide the test results to the owner and the Contractor on a routine and frequent basis.

1.3 The Earthwork Contractor

The Earthwork Contractor (Contractor) shall be qualified, experienced, and knowledgeable in earthwork logistics, preparation and processing of ground to receive fill, moisture-conditioning and processing of fill, and compacting fill. The Contractor shall review and accept the plans, geotechnical report(s), and these Specifications prior to commencement of grading. The Contractor shall be solely responsible for performing the grading in accordance with the plans and specifications.

The Contractor shall prepare and submit to the owner and the Geotechnical Consultant a work plan that indicates the sequence of earthwork grading, the number of "spreads" of work and the estimated quantities of daily earthwork contemplated for the site prior to commencement of grading. The Contractor shall inform the owner and the Geotechnical Consultant of changes in work schedules and updates to the work plan at least 24 hours in advance of such changes so that appropriate observations and tests can be planned and accomplished. The Contractor shall not assume that the Geotechnical Consultant is aware of all grading operations.

The Contractor shall have the sole responsibility to provide adequate equipment and methods to accomplish the earthwork in accordance with the applicable grading codes and agency ordinances, these Specifications, and the recommendations in the approved geotechnical report(s) and grading plan(s). If, in the opinion of the Geotechnical Consultant, unsatisfactory conditions, such as unsuitable soil, improper moisture condition, inadequate compaction, insufficient buttress key size, adverse weather, etc., are resulting in a quality of work less than required in these specifications, the Geotechnical Consultant shall reject the work and may recommend to the owner that construction be stopped until the conditions are rectified.

2.0 PREPARATION OF AREAS TO BE FILLED

2.1 Clearing and Grubbing

Vegetation, such as brush, grass, roots, and other deleterious material shall be sufficiently removed and properly disposed of in a method acceptable to the owner, governing agencies, and the Geotechnical Consultant.

The Geotechnical Consultant shall evaluate the extent of these removals depending on specific site conditions. Earth fill material shall not contain more than 1 percent of organic materials (by volume). No fill lift shall contain more than 5 percent of organic matter. Nesting of the organic materials shall not be allowed.

If potentially hazardous materials are encountered, the Contractor shall stop work in the affected area, and a hazardous material specialist shall be informed immediately for proper evaluation and handling of these materials prior to continuing to work in that area.

As presently defined by the State of California, most refined petroleum products (gasoline, diesel fuel, motor oil, grease, coolant, etc.) have chemical constituents that are considered to be hazardous waste. As such, the indiscriminate dumping or spillage of these fluids onto the ground may constitute a misdemeanor, punishable by fines and/or imprisonment, and shall not be allowed.

2.2 Processing

Existing ground that has been declared satisfactory for support of fill by the Geotechnical Consultant shall be scarified to a minimum depth of 6 inches. Existing ground that is not satisfactory shall be overexcavated as specified in the following section. Scarification shall continue until soils are broken down and free of large clay lumps or clods and the working surface is reasonably uniform, flat, and free of uneven features that would inhibit uniform compaction.

2.3 Overexcavation

In addition to removals and overexcavations recommended in the approved geotechnical report(s) and the grading plan, soft, loose, dry, saturated, spongy, organic-rich, highly fractured or otherwise unsuitable ground shall be overexcavated to competent ground as evaluated by the Geotechnical Consultant during grading.

2.4 Benching

Where fills are to be placed on ground with slopes steeper than 5:1 (horizontal to vertical units), the ground shall be stepped or benched. Please see the Standard Details for a graphic illustration. The lowest bench or key shall be a minimum of 15 feet wide and at least 2 feet deep, into competent material as evaluated by the Geotechnical Consultant. Other benches shall be excavated a minimum height of 4 feet into competent material or as otherwise recommended by the Geotechnical

Consultant. Fill placed on ground sloping flatter than 5:1 shall also be benched or otherwise overexcavated to provide a flat subgrade for the fill.

2.5 <u>Evaluation/Acceptance of Fill Areas</u>

All areas to receive fill, including removal and processed areas, key bottoms, and benches, shall be observed, mapped, elevations recorded, and/or tested prior to being accepted by the Geotechnical Consultant as suitable to receive fill. The Contractor shall obtain a written acceptance from the Geotechnical Consultant prior to fill placement. A licensed surveyor shall provide the survey control for determining elevations of processed areas, keys, and benches.

3.0 FILL MATERIAL

3.1 <u>General</u>

Material to be used as fill shall be essentially free of organic matter and other deleterious substances evaluated and accepted by the Geotechnical Consultant prior to placement. Soils of poor quality, such as those with unacceptable gradation, high expansion potential, or low strength shall be placed in areas acceptable to the Geotechnical Consultant or mixed with other soils to achieve satisfactory fill material.

3.2 <u>Oversize</u>

Oversize material defined as rock, or other irreducible material with a maximum dimension greater than 8 inches, shall not be buried or placed in fill unless location, materials, and placement methods are specifically accepted by the Geotechnical Consultant. Placement operations shall be such that nesting of oversized material does not occur and such that oversize material is completely surrounded by compacted or densified fill. Oversize material shall not be placed within 10 vertical feet of finish grade or within 2 feet of future utilities or underground construction.

3.3 <u>Import</u>

If importing of fill material is required for grading, proposed import material shall meet the requirements of Section 3.1. The potential import source shall be given to the Geotechnical Consultant at least 48 hours (2 working days) before importing begins so that its suitability can be determined and appropriate tests performed.

4.0 FILL PLACEMENT AND COMPACTION

4.1 Fill Layers

Approved fill material shall be placed in areas prepared to receive fill (per Section 3.0) in near-horizontal layers not exceeding 8 inches in loose thickness. The Geotechnical Consultant may accept thicker layers if testing indicates the grading procedures can adequately compact the thicker layers. Each layer shall be spread evenly and mixed thoroughly to attain relative uniformity of material and moisture throughout.

4.2 Fill Moisture Conditioning

Fill soils shall be watered, dried back, blended, and/or mixed, as necessary to attain a relatively uniform moisture content at or slightly over optimum. Maximum density and optimum soil moisture content tests shall be performed in accordance with the American Society of Testing and Materials (ASTM Test Method D1557).

4.3 Compaction of Fill

After each layer has been moisture-conditioned, mixed, and evenly spread, it shall be uniformly compacted to not less than 90 percent of maximum dry density (ASTM Test Method D1557). Compaction equipment shall be adequately sized and be either specifically designed for soil compaction or of proven reliability to efficiently achieve the specified level of compaction with uniformity.

4.4 Compaction of Fill Slopes

In addition to normal compaction procedures specified above, compaction of slopes shall be accomplished by backrolling of slopes with sheepsfoot rollers at increments of 3 to 4 feet in fill elevation, or by other methods producing satisfactory results acceptable to the Geotechnical Consultant. Upon completion of grading, relative compaction of the fill, out to the slope face, shall be at least 90 percent of maximum density per ASTM Test Method D1557.

4.5 Compaction Testing

Field-tests for moisture content and relative compaction of the fill soils shall be performed by the Geotechnical Consultant. Location and frequency of tests shall be at the Consultant's discretion based on field conditions encountered. Compaction test locations will not necessarily be selected on a random basis. Test locations shall be selected to verify adequacy of compaction levels in areas that are judged to be prone to inadequate compaction (such as close to slope faces and at the fill/bedrock benches).

4.6 Frequency of Compaction Testing

Tests shall be taken at intervals not exceeding 2 feet in vertical rise and/or 1,000 cubic yards of compacted fill soils embankment. In addition, as a guideline, at least one test shall be taken on slope faces for each 5,000 square feet of slope face and/or each 10 feet of vertical height of slope. The Contractor shall assure that fill construction is such that the testing schedule can be accomplished by the Geotechnical Consultant. The Contractor shall stop or slow down the earthwork construction if these minimum standards are not met.

4.7 Compaction Test Locations

The Geotechnical Consultant shall document the approximate elevation and horizontal coordinates of each test location. The Contractor shall coordinate with the project surveyor to assure that sufficient grade stakes are established so that the Geotechnical Consultant can determine the test locations with sufficient accuracy. At a minimum, two grade stakes within a horizontal distance of 100 feet and vertically less than 5 feet apart from potential test locations shall be provided.

5.0 SUBDRAIN INSTALLATION

Subdrain systems shall be installed in accordance with the approved geotechnical report(s), the grading plan, and the Standard Details. The Geotechnical Consultant may recommend additional subdrains and/or changes in subdrain extent, location, grade, or material depending on conditions encountered during grading. All subdrains shall be surveyed by a land surveyor/civil engineer for line and grade after installation and prior to burial. Sufficient time should be allowed by the Contractor for these surveys.

6.0 EXCAVATION

Excavations, as well as over-excavation for remedial purposes, shall be evaluated by the Geotechnical Consultant during grading. Remedial removal depths shown on geotechnical plans are estimates only. The actual extent of removal shall be determined by the Geotechnical Consultant based on the field evaluation of exposed conditions during grading. Where fill-over-cut slopes are to be graded, the cut portion of the slope shall be made, evaluated, and accepted by the Geotechnical Consultant prior to placement of materials for construction of the fill portion of the slope, unless otherwise recommended by the Geotechnical Consultant.

7.0 TRENCH BACKFILLS

7.1 Safety

The Contractor shall follow all OSHA and Cal/OSHA requirements for safety of trench excavations.

7.2 Bedding and Backfill

All bedding and backfill of utility trenches shall be performed in accordance with the applicable provisions of Standard Specifications of Public Works Construction. Bedding material shall have a Sand Equivalent greater than 30 (SE>30). The bedding shall be placed to 1 foot over the top of the conduit and densified by jetting. Backfill shall be placed and densified to a minimum of 90 percent of relative compaction from 1 foot above the top of the conduit to the surface.

The Geotechnical Consultant shall test the trench backfill for relative compaction. At least one test should be made for every 300 feet of trench and 2 feet of fill.

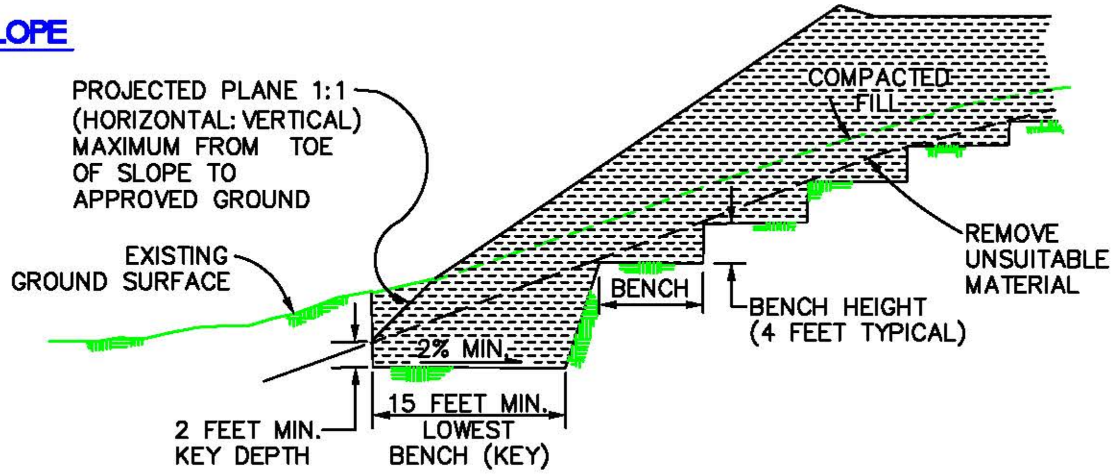
7.3 Lift Thickness

Lift thickness of trench backfill shall not exceed those allowed in the Standard Specifications of Public Works Construction unless the Contractor can demonstrate to the Geotechnical Consultant that the fill lift can be compacted to the minimum relative compaction by his alternative equipment and method.

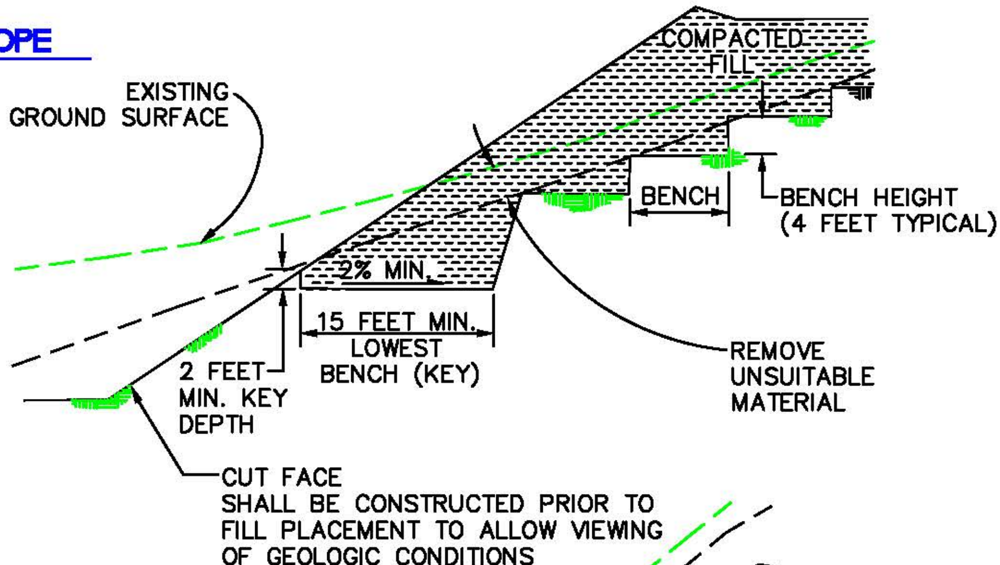
7.4 Observation and Testing

The jetting of the bedding around the conduits shall be observed by the Geotechnical Consultant.

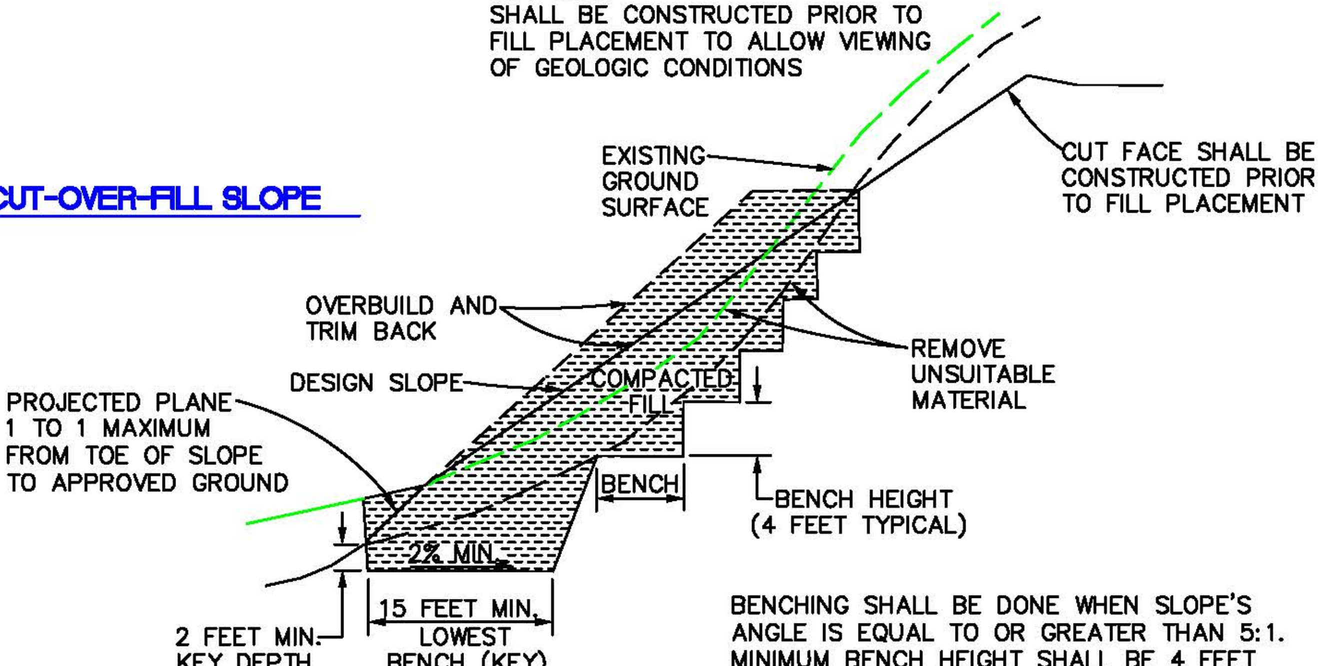
FILL SLOPE



FILL-OVER-CUT SLOPE

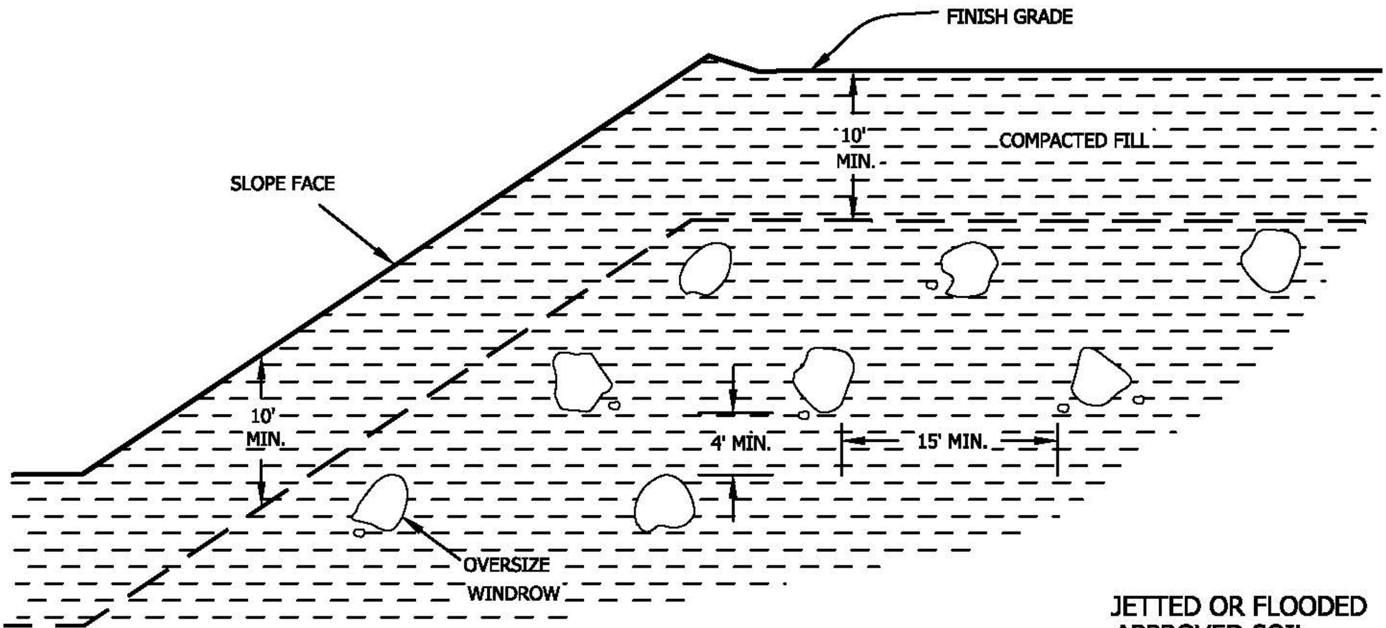


CUT-OVER-FILL SLOPE

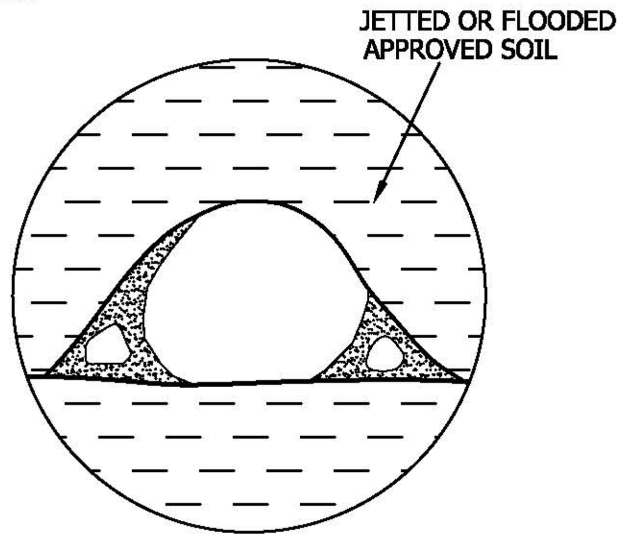


BENCHING SHALL BE DONE WHEN SLOPE'S ANGLE IS EQUAL TO OR GREATER THAN 5:1. MINIMUM BENCH HEIGHT SHALL BE 4 FEET AND MINIMUM FILL WIDTH SHALL BE 9 FEET.

KEYING AND BENCHING	GENERAL EARTHWORK AND GRADING SPECIFICATIONS STANDARD DETAILS A	Leighton

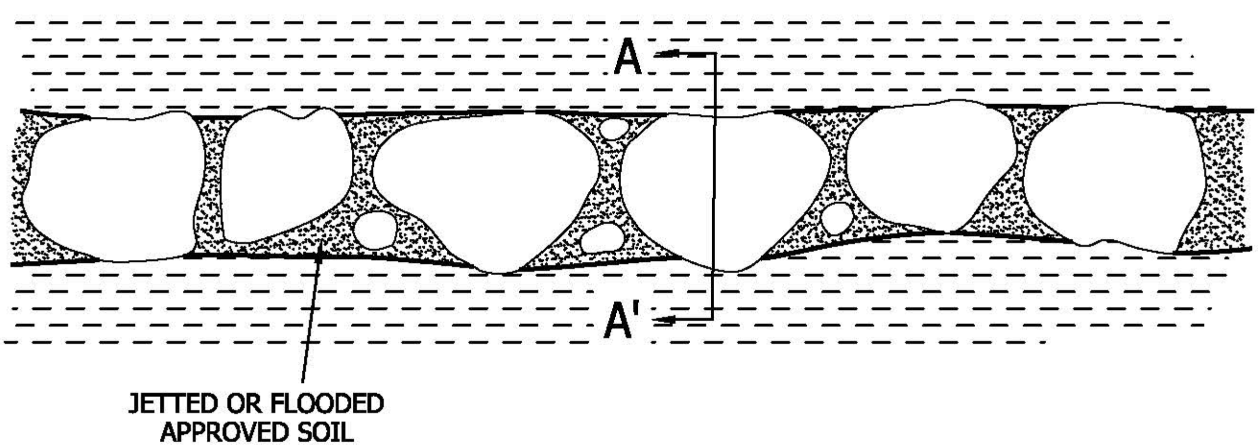


- Oversize rock is larger than 8 inches in largest dimension.
- Backfill with approved soil jetted or flooded in place to fill all the voids.
- Do not bury rock within 10 feet of finish grade.
- Windrow of buried rock shall be parallel to the finished slope face.



SECTION A-A'

PROFILE ALONG WINDROW

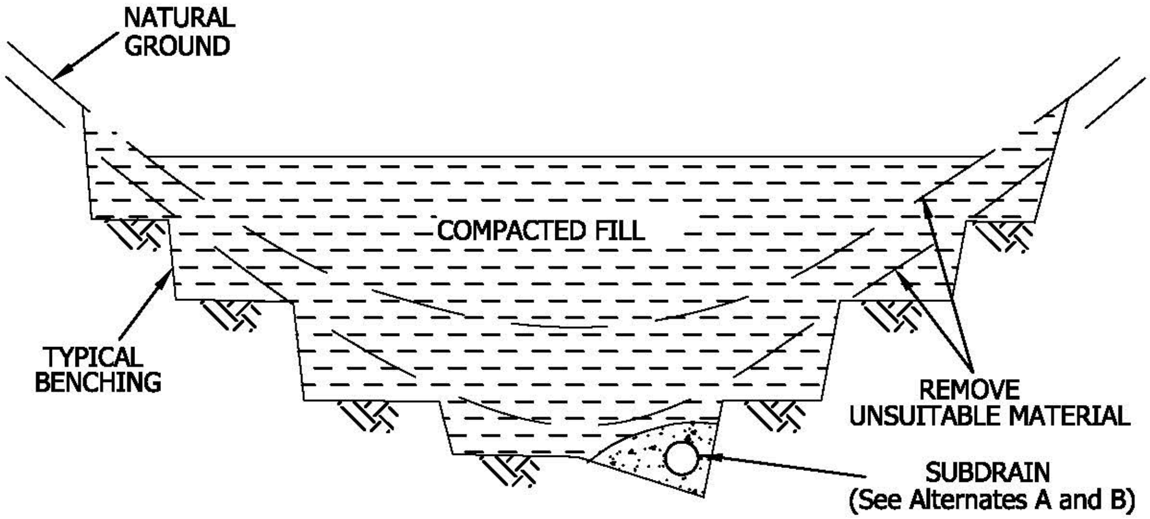


OVERSIZE ROCK DISPOSAL

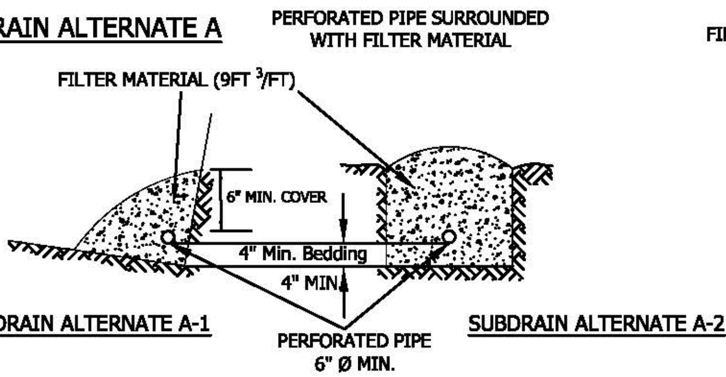
GENERAL EARTHWORK AND GRADING SPECIFICATIONS
STANDARD DETAILS B

Leighton

P:Drafting\templates\details\over-rock-disp.dwg (7/00)



SUBDRAIN ALTERNATE A

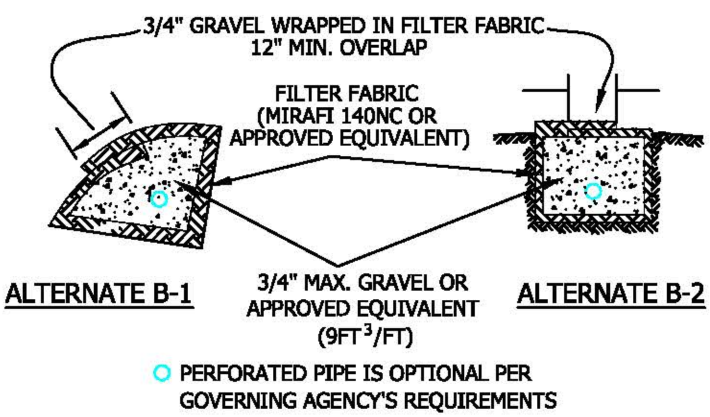


FILTER MATERIAL

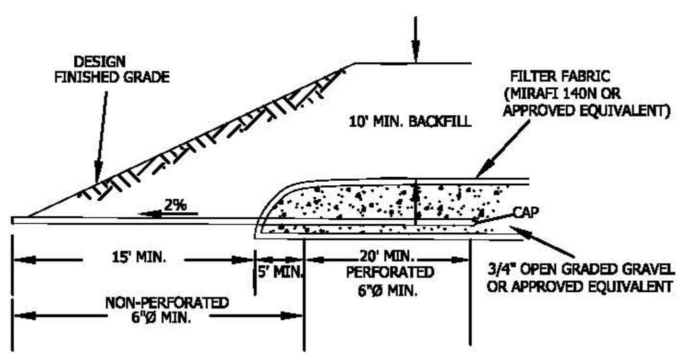
FILTER MATERIAL SHALL BE CLASS 2 PERMEABLE MATERIAL PER STATE OF CALIFORNIA STANDARD SPECIFICATION, OR APPROVED ALTERNATE. CLASS 2 GRADING AS FOLLOWS:

Sieve Size	Percent Passing
1"	100
3/4"	90-100
3/8"	40-100
No. 4	25-40
No. 8	18-33
No. 30	5-15
No. 50	0-7
No. 200	0-3

SUBDRAIN ALTERNATE B

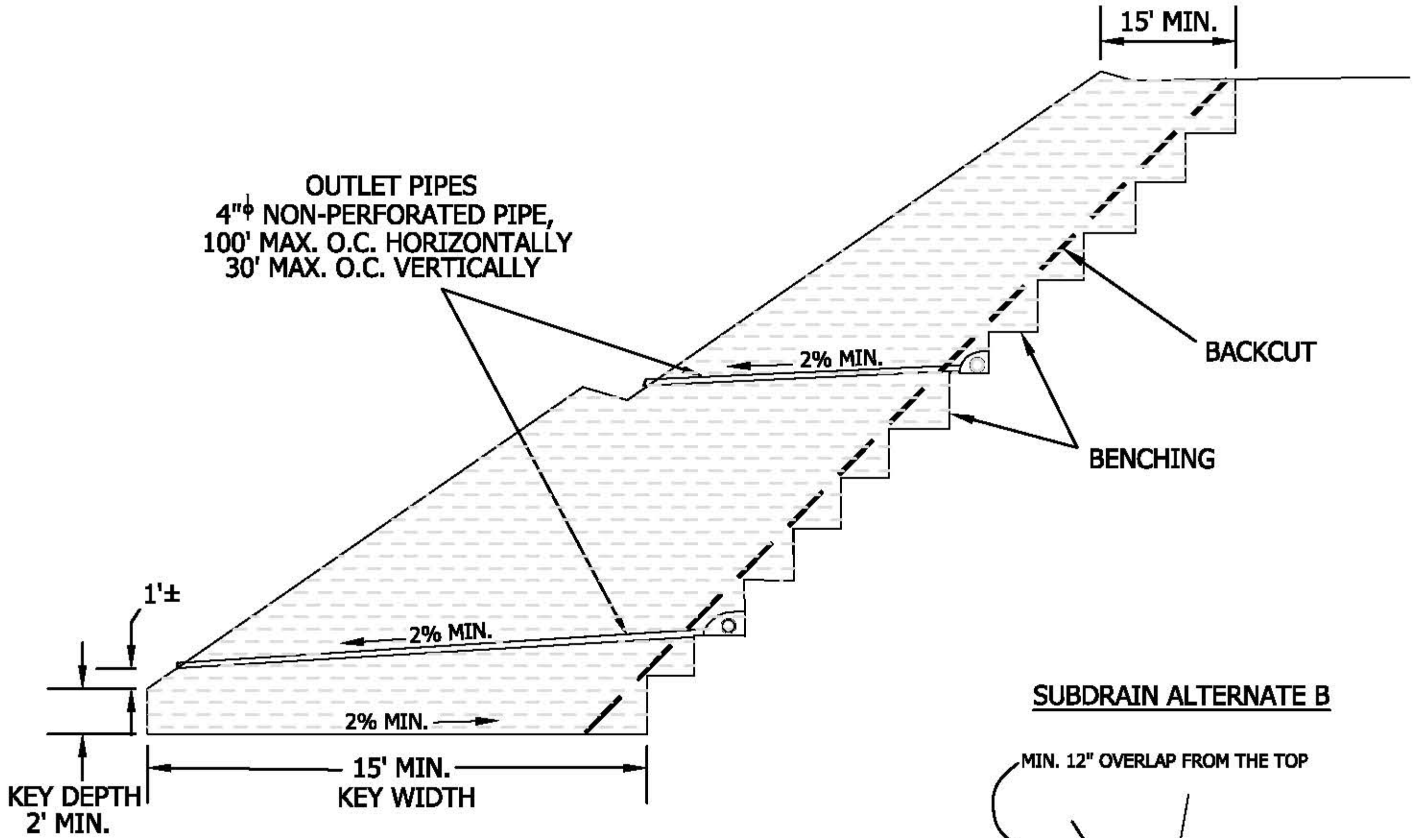


DETAIL OF CANYON SUBDRAIN TERMINAL

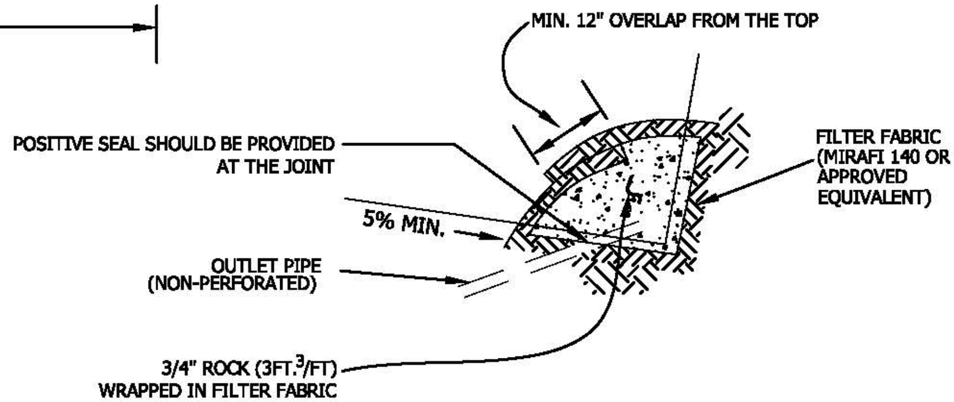


CANYON SUBDRAIN	GENERAL EARTHWORK AND GRADING SPECIFICATIONS STANDARD DETAILS C	Leighton

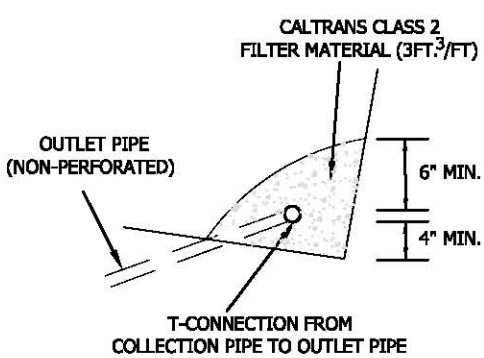
P:Drafting\templates\details\canyon_drains.dwg (7/00)



SUBDRAIN ALTERNATE B



SUBDRAIN ALTERNATE A



- SUBDRAIN INSTALLATION - Subdrain collector pipe shall be installed with perforations down or, unless otherwise designated by the geotechnical consultant. Outlet pipes shall be non-perforated pipe. The subdrain pipe shall have at least 8 perforations uniformly spaced per foot. Perforation shall be 1/4" to 1/2" if drilled holes are used. All subdrain pipes shall have a gradient at least 2% towards the outlet.
- SUBDRAIN PIPE - Subdrain pipe shall be ASTM D2751, ASTM D1527 (Schedule 40) or SDR 23.5 ABS pipe or ASTM D3034 (Schedule 40) or SDR 23.5 PVC pipe.
- All outlet pipe shall be placed in a trench and, after fill is placed above it, rodded to verify integrity.

BUTTRESS OR REPLACEMENT FILL SUBDRAINS	GENERAL EARTHWORK AND GRADING SPECIFICATIONS STANDARD DETAILS D	Leighton

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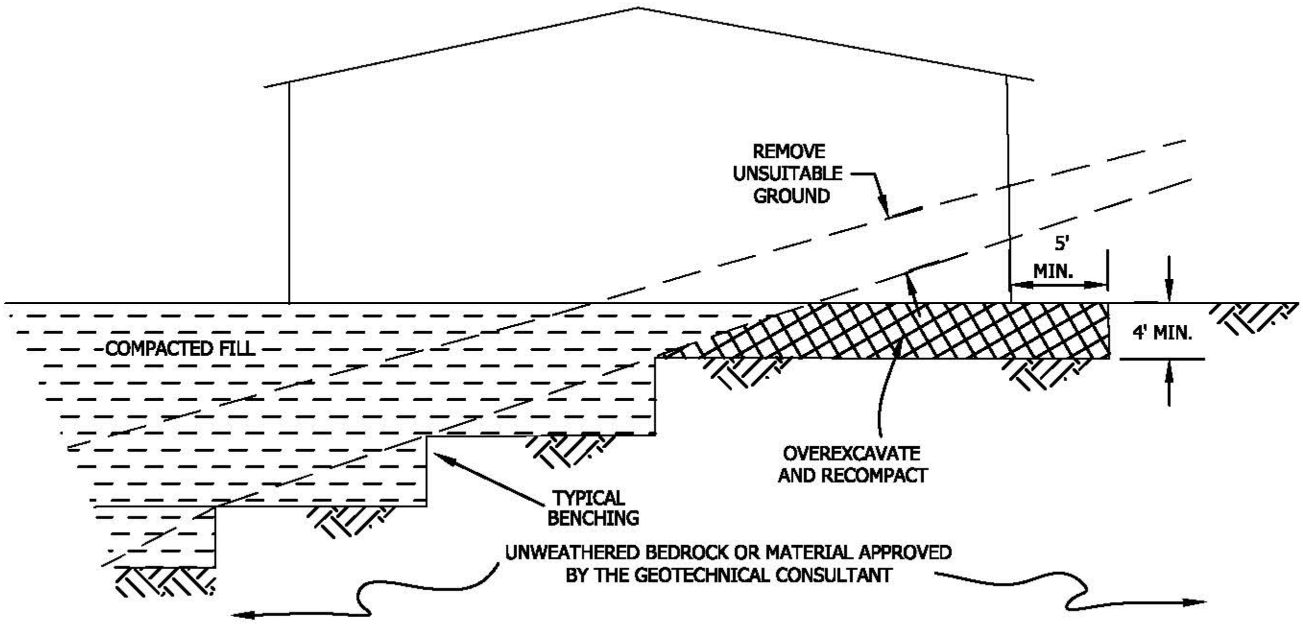
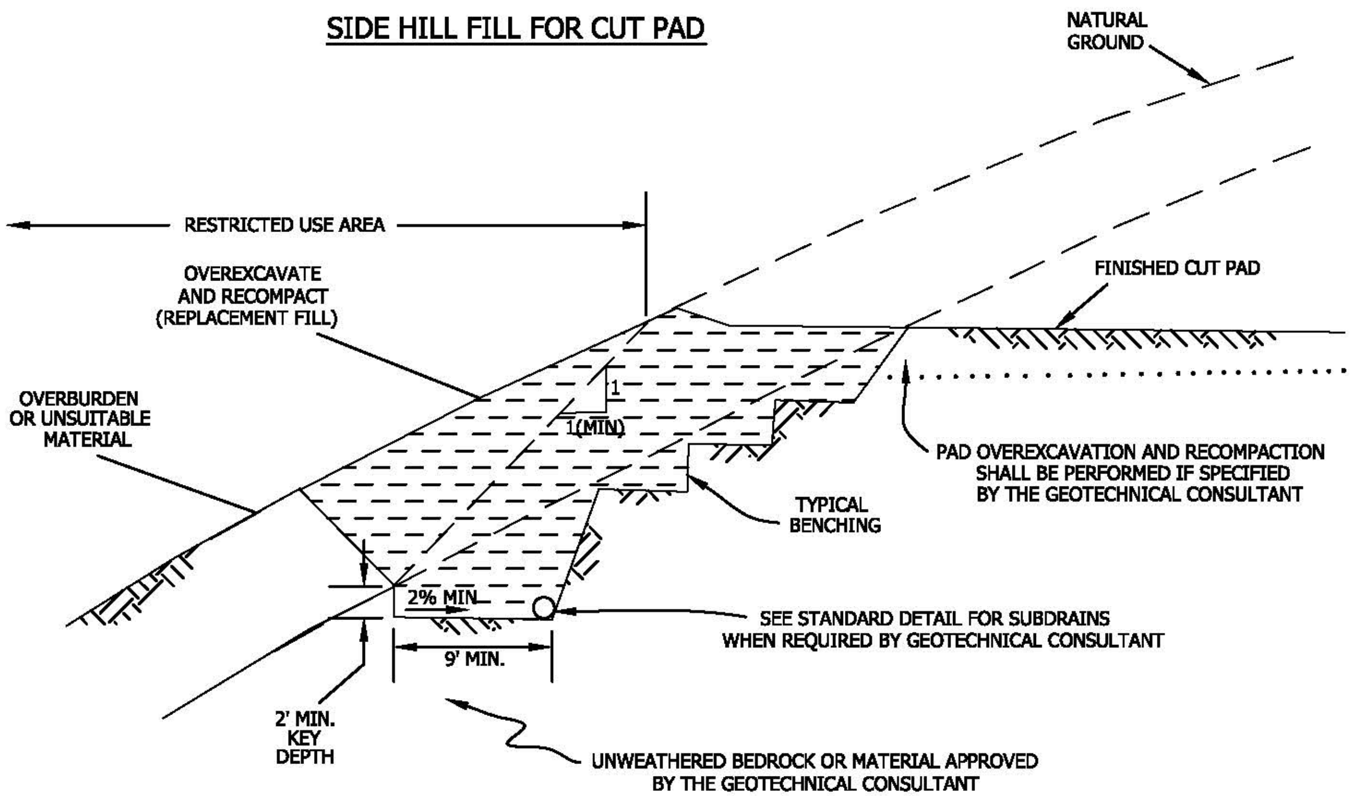
CUT-FILL TRANSITION LOT OVEREXCAVATION

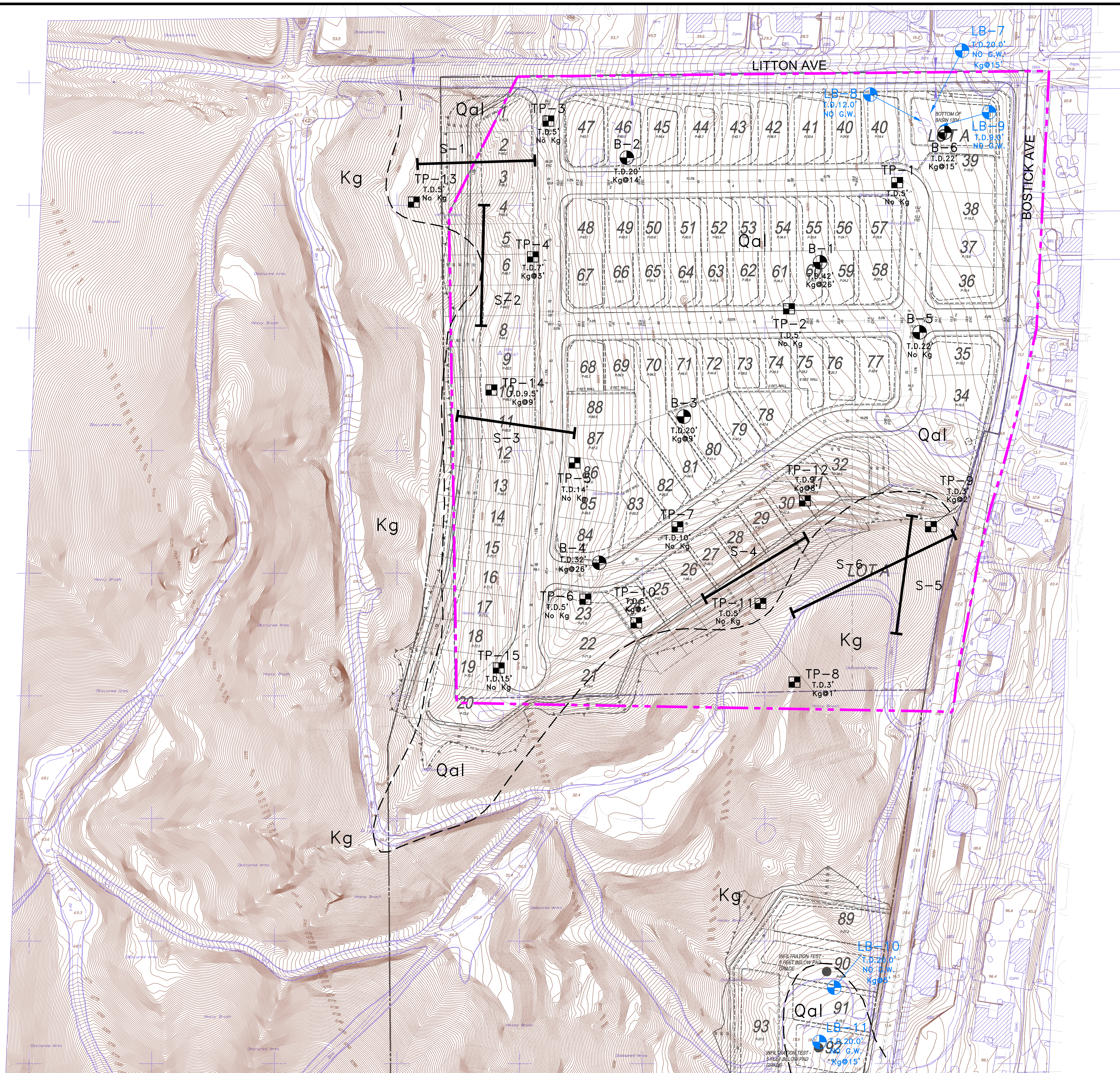


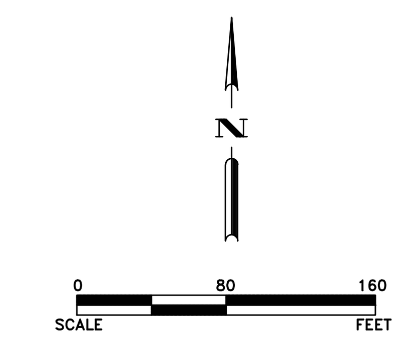
SIDE HILL FILL FOR CUT PAD



TRANSITION LOT FILLS AND SIDE HILL FILLS	GENERAL EARTHWORK AND GRADING SPECIFICATIONS STANDARD DETAILS E	Leighton

P:Drafting\templates\details\transition_fills.dwg (7/00)





LEGEND

Qal	ALLUVIUM
Kg	GRANITIC BEDROCK
B-11 T.D.20.0' NO G.W.	APPROXIMATE LOCATION OF HOLLOW STEM AUGER BORING WITH TOTAL DEPTH (CURRENT INVESTIGATION)
B-6 T.D.22' No Kg	APPROXIMATE LOCATION OF HOLLOW STEM AUGER BORING WITH TOTAL DEPTH AND LITHOLOGIC SUMMARY
TP-15 T.D.15' No Kg	APPROXIMATE LOCATION OF BACKHOE TEST PIT WITH TOTAL DEPTH AND LITHOLOGIC SUMMARY
S-6	APPROXIMATE LOCATION OF SEISMIC LINE
	APPROXIMATE LOCATION OF GEOLOGIC CONTACT; CONTACT BETWEEN Qal and Kg IS EXPECTED TO BE HIGHLY VARIABLE DUE TO THE VARIABLE COLLUVIAL THICKNESS AND WEATHERING OF THE UNDERLYING BEDROCK
	APPROXIMATE LIMITS OF THIS STUDY

GEOTECHNICAL MAP

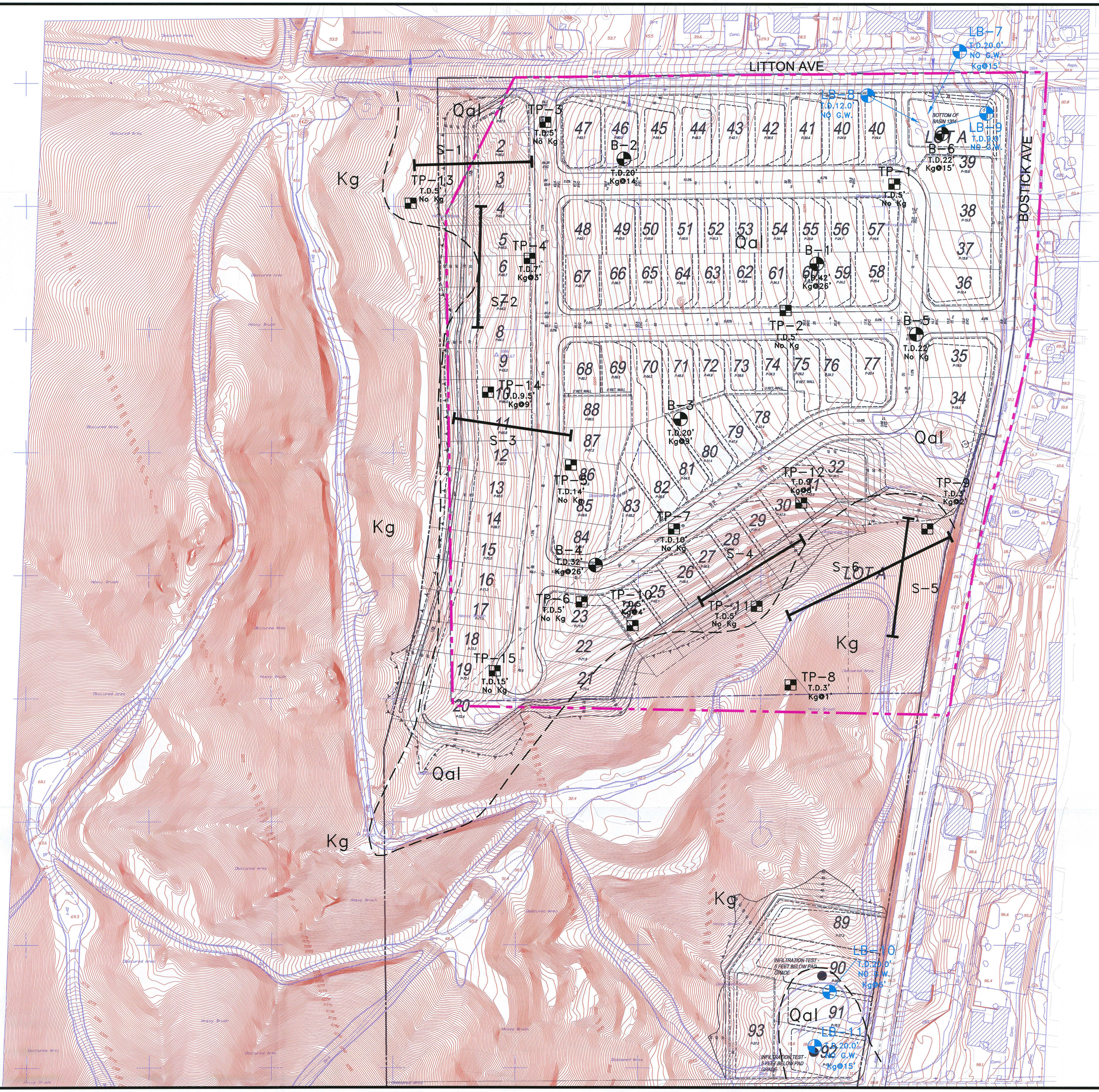
WEST PROPERTY
SOUTHWEST OF LITTON AVENUE AND BOSTICK AVENUE
COLTON, CALIFORNIA

Proj: 10811.001	Eng/Geol: JDH/PB
Scale: 1"=80'	Date: 10/2014

Drafted By: BQT Checked By: BQT

PLATE 1

Leighton



N

0 80 160
SCALE FEET

LEGEND

Qal	ALLUVIUM
Kg	GRANITIC BEDROCK
B-11 T.D.20.0' NO G.W.	APPROXIMATE LOCATION OF HOLLOW STEM AUGER BORING WITH TOTAL DEPTH (CURRENT INVESTIGATION)
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— - - —	APPROXIMATE LIMITS OF THIS STUDY

GEOTECHNICAL MAP

WEST PROPERTY
SOUTHWEST OF LITTON AVENUE AND BOSTICK AVENUE
COLTON, CALIFORNIA

PLATE 1

Proj: 10811.001	Eng/Geol: JDH/PB
Scale: 1"=80'	Date: 10/2014

Leighton

Drafted By: BQT | Checked By: BQT | P:\DRAFTING\10811\001\CAD\OF_2014-10-15\10811-001-PLATE1.DWG (10-21-14 9:30:29AM) Plotted by: btran