Appendix E:
Geotechnical Reports

E.1 - Recharge Basin

GEOTECHNICAL REVIEW OF THE PROPOSED RECHARGE/INFILTRATION BASINS TO BE LOCATED AT THE SOUTHWEST CORNER OF BROOKSIDE AVENUE AND BEAUMONT AVENUE, CITY OF BEAUMONT, CALIFORNIA

Prepared for:

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Project No. 603154-003

November 9, 2012

November 9, 2012

Project No. 603154-003

To: Albert A. Webb Associates
3788 McCray Street
Riverside, California 92506

Attention: Mr. Joseph C. Caldwell, PE, CPSWQ, CPESC

Subject: Geotechnical Review of the Proposed Recharge/Infiltration Basins to be Located at the Southwest Corner of Brookside Avenue and Beaumont Avenue, City of Beaumont, California

In response to your request, Leighton Consulting, Inc. (Leighton) has conducted this review of the proposed infiltration/recharge basins at the southwest corner of Brookside Avenue and Beaumont Avenue in the City of Beaumont, California (APNs 404-010-012 and 404-010-015). Leighton previously conducted subsurface exploration and infiltration testing at the site (Leighton, 2011). The purpose of our current work has been to review the existing data and provide geotechnical recommendations for design and construction of the infiltration/recharge basins.

The plans for the proposed San Gorgonio Pass Water Agency Beaumont Avenue Recharge Facility were prepared by Albert A Webb Associates and are date stamped October 23, 20012. These plans serve as the base for the Geotechnical Map, Figure 2.

Site Description

The 52-acre property is located at the southwest corner of Brookside Avenue and Beaumont Avenue in the City of Beaumont, California. The site is currently vacant and undeveloped with no indication of past improvements. The site is bounded by Beaumont High School across Brookside Avenue to the north, a public park across Beaumont Avenue to the east, Mountain View Middle School to the south, and vacant land to the west. Noble Creek enters the site at the northern boundary, traveling through the site in a southwesterly direction, dividing the rectangular property into two areas, with the majority of the site south of the creek. The portion of the site south of the creek is being designed for the recharge facility. Site topography slopes slightly to the southwest, with site elevations ranging from approximately 2,710 to 2,650 feet above mean sea level at the northeast and southwest corners of the site, respectively (a 2 to 3 percent grade).

Project Description

Based on the plans, the design includes construction of 5 basins in the area between Noble Creek and Beaumont Avenue (see Figure 2, Boring Location Map). The individual basins are to be separated by berms a maximum of 24 feet in height (between Basin 4 and Basin 5). The slopes are designed at an inclination of 3:1 (horizontal to vertical). Minimum 15-foot-wide access roads are planned at the tops of berms.

The bottom of the basins vary in elevation from a low of 2,648 feet above mean sea level (msl) at Basin No 5 at the south end of the property, to a high of 2,690 feet msl at Basin No. 1, adjacent to Brookside Avenue. We understand the project design includes the option to operate the basin is series, with each basin filling to a depth of 3 feet before overflowing a weir structure into the basin below, or in parallel where any single basin can be filled without the adjoining basins containing any water. The maximum depth of water in any basin would be about 6.5 feet, when water would overflow the spillway into the adjacent lower basin.

Spillways, storm drains connecting the basins, access roads and other associated improvements are also planned.

Scope of Work

The scope of work for our study has included:

- Review the Infiltration report previously prepared for the site (Leighton, 2011) as well as published geologic reports and maps covering the site vicinity available from our in-house library (references). Data from the previous report has been considered in our review.

- Transfer of data from the previous report to the current Boring Location Map, Figure 2. Boing logs from the report are included in Appendix B.

- Visit the site to observe the existing conditions.

- Bulk samples of the near surface soils were collected for laboratory testing of maximum dry density, optimum moisture content, grain size distribution, shear strength, sulfate content, chloride content, resistivity and pH. Laboratory Test Results are provided in Appendix C.

- Data from our background review, previous field exploration and geotechnical laboratory testing program were evaluated and analyzed to develop geotechnical conclusions and recommendations for this project.

- Preparation of this report addressing the proposed basin design including recommendations for grading and construction of the proposed improvements.

<u>Geologic Setting</u>

The site is located in the northwestern end of the San Gorgonio Pass area of southern California, near the intersection of the San Bernardino Mountains of the Transverse Range Geomorphic Province, and the San Jacinto Mountains of the Peninsular Ranges Geomorphic Province. The Peninsular Ranges province extends approximately 900 miles southward from the Santa Monica Mountains to the tip of Baja California. The province is characterized by elongate northwest-trending mountain ridges separated by intervening, sediment-floored valleys. However, the most dominant structural features of the province are the northwest-trending fault zones, most of which either die out, merge with, or are terminated by the steep reverse faults at the southern margin of the Transverse Ranges province.

The dominant structural feature within this region is the active San Andreas transform system, which consists of several major northwest-trending, right-lateral, strike-slip faults. The San Andreas Fault Zone (SAFZ) is located approximately 7 miles northeast of the site. The active Banning Fault Zone, considered a branch of the SAFZ, is located approximately 1.5 miles north of the property, and the San Jacinto Fault Zone is located approximately 6 miles southwest of the site.

The site is underlain by alluvial soil eroded from the San Bernardino Mountains and deposited in the site vicinity.

<u>Surface and Subsurface Conditions</u>

A review of regional geologic maps indicate the site is underlain by alluvial soils generally consisting of sand and silty sand with gravel.

Soils encountered within the borings and test pits excavated onsite during the previous study generally consisted of silty sand (SM) and well-graded sand (SW) to the maximum explored depth of 51.5 feet. Isolated sandy silt (ML) layers and poorly graded sand layers (SP) were observed, generally at depths greater than 25 feet. Based on our testing, the fines content of the soils (percent passing a No. 200 sieve) ranged from 14 to 43 percent, with the soils encountered near the southeast corner of the site (Boring LB-4) containing a higher proportion of silt than borings conducted elsewhere onsite, especially at depths greater than 30 feet bgs. Otherwise, the soil profile appeared relatively consistent throughout the site. The soil was generally described as loose near the surface, becoming medium dense to dense with depth. The moisture content of the soil ranged from 2 to 10 percent.

Collapse potential refers to the potential settlement of a soil under existing loads upon being wetted. Based on our experience in the area, the onsite, near surface soil is expected to have a slight collapse potential.

Based on their granular nature, the soils are expected to have a very low expansion potential.

<u>Sulfate Content</u>

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 by weight are considered to have negligible sulfate exposure (2010 California Building Code, CBC).

A near-surface soil sample was tested during this study for soluble sulfate content. The results of this test indicate a soluble sulfate content of 0.041 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.

<u>Resistivity, Chloride and pH</u>

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 greater than 10,000 ohm-cm is considered mildly corrosive. Soil with a chloride content of 500 ppm or more is considered moderately corrosive to ferrous metals.

As a screening for potentially corrosive soil, a representative soil sample was tested during this investigation to determine its minimum resistivity, chloride content, and pH level. The test indicated a chloride content of 41 ppm, a pH of 6.2, and a minimum resistivity of 7,400 ohm-cm. The results indicate that the onsite soil is considered moderately corrosive to buried ferrous metals.

<u>Groundwater</u>

Groundwater was not encountered in any of our borings excavated onsite to a maximum depth of approximately 50 feet below the existing ground surface.

Based on our review of regional maps and groundwater data (WMWD, 2004), groundwater is expected to be deeper than 200 feet below the existing ground surface in the immediate vicinity of the subject site. In addition, the site is mapped in an area with deep groundwater according to the Riverside County Generalized Liquefaction Map (2003). As such, groundwater is not expected to be a constraint to the proposed improvements.

<u>Faulting</u>

The site is not located within a State of California designated Alquist-Priolo Earthquake Fault Zone (CGS, 2000). However, a County of Riverside designated Earthquake Fault Zone for the Beaumont Plains Fault Zone is mapped through the southwest corner of the property (County of Riverside, 2003). This fault zone is mapped as a series of north to northwest trending faults in the general vicinity (Matti et al, 1985; and Treiman, 1994). Leighton and Associates conducted an investigation of this fault at the adjacent property to the west (Leighton and Associates, 2007). Based on our review of available data, there is no indication the fault extends onsite. The eastern limits of the County established Earthquake Fault Zone are shown on the Boring Location Map, Figure 2.

<u>Regional Faulting and Seismicity</u>

The two principal seismic considerations for most sites in southern California are surface rupture along active fault traces and damage to structures due to seismically induced ground shaking. An active fault is one that has moved in the Holocene (last 11,000 years). No known active faults have been mapped onsite and no evidence of faulting has been observed during our study.

The closest mapped, previously known, active fault that has been studied in sufficient detail to evaluate the potential for strong seismic shaking is the San Jacinto-San Jacinto Valley segment fault, located approximately 9 kilometers northeast of the subject site. The San Jacinto-San Jacinto Valley fault is capable of producing a maximum moment magnitude of 6.9 (Mw) with an average slip rate of 12.0 ±6 millimeters per year (Cao et al., 2003). Other known regional active faults that could affect the site include the San Andreas, Banning and Elsinore-Glen Ivy faults. The largest fault in southern California, the San Andreas Fault System, is located approximately 23 kilometers northeast of the site.

The site is likely to be subjected to strong ground shaking during the life of the project (Petersen and Wesnousky, 1994, Petersen et al., 1996). To evaluate the ground motion and a peak level of ground acceleration that the project is likely to experience, we utilized a probabilistic analysis approach, estimating the expected peak ground acceleration level that has a 10 percent probability of exceedance over the approximate lifetime of the project (commonly 50 years). This approach takes into account the historical seismicity of the region, the nature of nearby active faults, their distance to the site, records of previous historical earthquakes, and the site-specific response characteristics (Petersen et al., 1996).

The computer program FRISKSP (Blake, 2000) was used for the analysis. Attenuation relationships used in the computer analysis were developed by Abrahamson and Silva (1997) for soil, Campbell (1997 and 2000) for alluvium, and Sadigh et al. (1997) for deep soil deposits. The analysis indicated an average value for peak horizontal ground acceleration (PHGA) with a 10 percent probability of exceedance in 50 years of 0.61g. Hazard deaggregation indicates that the predominant earthquake magnitude is approximately 6.9 (Mw) at a distance on the order of 9½ kilometers.

PHGA for the site was also estimated using California Geologic Survey (CGS) Probabilistic Seismic Hazards Mapping Ground Motion data (CGS, 2008), 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 0.62g. This correlates well with our PSHA.

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 2010 edition of the California Building Code (CBC). The following data should be considered for the seismic analysis of the subject site:

Table 1. 2010 CBC Site Categorization and Site Coefficients

Description (2010 CBC reference)	Parameter	Design Value
Site Longitude, degrees		-116.980
Site Latitude, degrees		33.958
Site Class Definition (Table 1613.5.2)		D
Mapped MCE Spectral Response Acceleration at 0.2s for Site Class B (Fig 1613.5(3))	S_S	1.5
Mapped MCE Spectral Response Acceleration at 1.0s for Site Class B (Fig 1613.5(4))	S_1	0.6
Short Period Site Coefficient (Table 1613.5.3(1))	F_a	1.0
Long Period Site Coefficient (Table 1613.5.3(2))	F_v	1.5
Adjusted MCE Spectral Response Acceleration at 0.2s Period [$=F_aS_S$] (1613.5.3)	S_{MS}	1.5
Adjusted MCE Spectral Response Acceleration at 1s Period [$=F_vS_1$] (1613.5.3)	S_{M1}	0.9
Design Spectral Response Accel. at 0.2s Period, 5% damped [$=2/3S_{MS}$] (1613.5.4)	S_{DS}	1.0
Design Spectral Response Accel. at 1s Period, 5% damped [$=2/3S_{MS}$] (1613.5.4)	S_{D1}	0.6

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 soil. 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 Generalized Liquefaction Map for Riverside County (2003) indicates the site is located in an area of deep groundwater with sediments considered to have low to very low susceptibility to liquefaction. Regional groundwater data indicates that shallow groundwater conditions do not exist locally, nor have they existed historically. Based on these findings, the potential for liquefaction onsite is considered very low.

Seismically Induced Settlement

Seismically induced settlement consists of dry dynamic settlement (above groundwater) and liquefaction settlement (below groundwater). This settlement occurs primarily within loose to moderately dense, dry or saturated 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 within younger alluvium areas is estimated to be on the order of 3 inches or less. The potential seismically induced differential settlement is estimated to be half of the total settlement over a horizontal distance of 40 feet.

Slope Stability

The onsite slopes for the recharge facility are planned for construction at inclinations of 3:1 (horizontal to vertical) or flatter. With the proposed design the upper portion of the slope will be constructed of compacted fill and the lower portion will be cut into alluvial soils consisting of sand and silty sand with gravel. Based on our analysis, the slopes are expected to be stable as designed under static, pseudo-static and rapid drawdown conditions.

Stability analysis for the slopes is provided in Appendix D.

Conclusions and Preliminary Recommendations

General Conclusion

Based upon this study, we conclude that construction of the proposed development of the site for recharge basins is feasible from a geotechnical standpoint, provided the recommendations presented herein are incorporated into the design and construction of the project. No severe geologic or soil-related hazards or constraints that would preclude development of this project have been found during the course of this study.

General Earthwork and Grading

Compacted fill should be placed in accordance with the General Earthwork and Grading Specifications presented in Appendix E, unless specifically revised or amended below or by future recommendations.

Site Preparation

Prior to construction, the site should be cleared of existing structures, vegetation, trash, 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. Any excavations to remove foundations or other underground structures, such as septic tanks or seepage pits, should be backfilled with compacted fill. In addition, prior to overexcavation and recompaction of the onsite alluvial soil, any clean uncontrolled artificial fill should be removed and may be used as compacted fill for the project.

Overexcavation and Slope Replacement

The near-surface soils are generally loose and potentially compressible. We recommend that these soils be removed and replaced as compacted fill in areas where they support additional fill loads or other improvements. For the berms, in areas of fill or shallow cut, we recommend that the upper 4 feet of existing soil be overexcavated and replaced as compacted fill. In parking areas and access roads outside the area of the berms, we recommend the near surface soils be overexcavated a minimum of 18 inches below existing grade or 12 inches below subgrade, whichever is deeper.

We recommend that the soils beneath the proposed concrete structures, such as the inlet and outlet structures and weir boxes, be overexcavated a minimum depth of 2 feet below the bottom of footings. Where feasible, the overexcavation bottom should extend horizontally beyond the proposed structure a minimum of 2 feet from the outside edges of the footings, or distance equal to the depth of overexcavation below the footings, whichever is farther. These excavations should be observed by Leighton to evaluate the nature of the soil conditions. If loose or soft soils are encountered, additional overexcavation and/or stabilization may be recommended

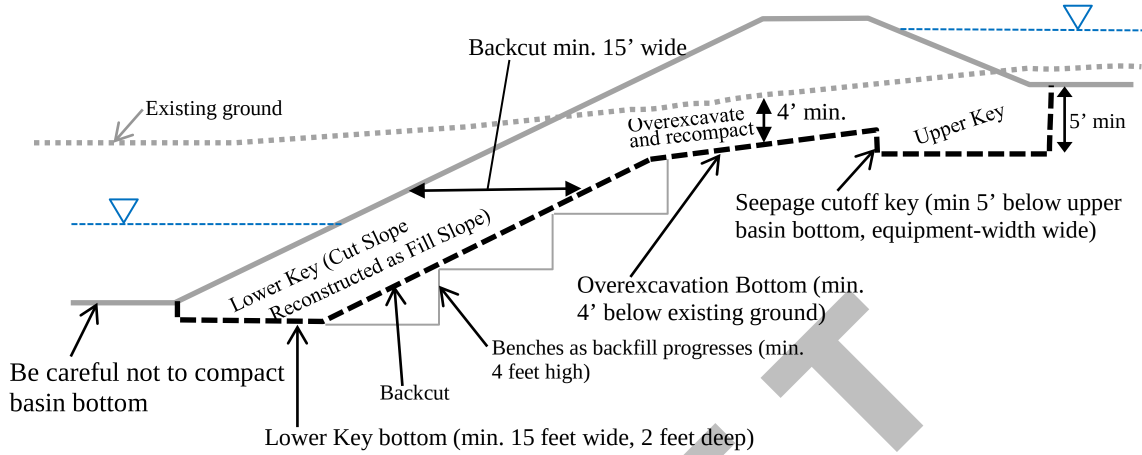
In addition, we recommend that any uncontrolled fill in the area of construction be removed and replaced as compacted fill.

After completion of the overexcavation, and prior to fill placement, the exposed surfaces should be scarified to a minimum depth of 8 inches, moisture conditioned to or slightly above optimum moisture content, and recompacted to a minimum 90 percent relative compaction, relative to the ASTM D 1557 laboratory maximum dry density.

The excavations for the storm drain pipe connecting the basins should extend to the depths as required on the construction plans. These excavations should also be observed by Leighton to evaluate the nature of the soil conditions. If loose or soft soils are encountered, additional overexcavation may be recommended.

Reconstructing Cut Slopes of Berms Between Basins and Trackwalking other Slopes

To limit the potential for seepage through slopes to a lower basin, we recommend that the cut portions of slopes separating basins be reconstructed with compacted fill, as shown on the figure below. We also recommend that a 15-foot wide 5-foot deep seepage cut-of key be constructed below the slope ascending from the upper basins (see Figure 2 and below). This need not be done on other slopes (i.e., slopes that are not on berms separating basins). However, those other slopes should be observed after being cut. Any loose areas should be track walked with heavy equipment and compacted to a minimum of 90 percent relative compaction to improve surficial stability.



Detail of Fill Keys and Overexcavation for Berms Between Basins (not to scale)

RCP Seepage Cutoff

We understand the recharge basins will be connected by reinforced concrete pipe that will allow water to flow from basins to basin. Care should be taken in the design to limit water movement within the bedding and backfill materials surrounding the pipe. We recommend that reinforced-concrete, seepage-cutoff collars or other methods of controlling water movement along the pipe be constructed along the pipes between the basins. The design should be reviewed based on actual soil conditions present in the area. The collar locations should be reviewed during construction and may be adjusted based on the soil conditions observed in the trench excavation.

Provided adequate control of seepage is provided, the remainder of the pipe should be bedded with sand having a Sand Equivalent of 30 or better. Coarse-grained bedding material should not be used. The sand bedding may be jetted in areas below the springline of the pipe, but should otherwise be mechanically compacted. However, care should be taken to limit flooding of the areas between concrete cut-off collars. A sump pump to remove water from jetting may be required. The contractor should submit a sample of the planned bedding material to the project engineer and geotechnical consultant prior to import to the site.

Reconstruction of Berms over RCP

We recommend that the portions of berms that will need to be reconstructed after installation of the reinforced concrete pipes within berms be backfilled in a manner that will limit the potential for groundwater seepage within the fill materials. The areas below the design high water level should be backfilled with selective fine-grained soil. The sides of the excavations should be provided with adequate benches as backfill progresses.

Fill Placement and Compaction

The onsite soil is suitable for use as compacted structural fill, provided it is free of debris 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.

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

Compaction of all fill slopes and stabilized cut slopes (including compaction of the slope face) should be performed in accordance with the General Earthwork and Grading Specifications in Appendix E.

Rippability and Oversized Materials

The alluvial soil materials onsite should be rippable using conventional heavy equipment in good working condition and modern earthmoving methods. Significant amounts of oversized material (greater than 8 inches in dimension) were not encountered during our investigation. However, limited amounts of oversized material may be encountered locally. If encountered, oversized material should be removed, or placed in deeper fill areas in accordance with our recommendations.

Retaining Walls

Areas planned for retaining walls should be overexcavated in accordance with the recommendations provided previously. Retaining walls outside the basins or above design waterlevel of the basins should be backfilled with very low expansive soil and constructed with a backdrain in accordance with the recommendations provided on

Figure 3 (rear of text). Using more expansive soil as retaining wall backfill will result in higher lateral earth pressures exerted on the wall. Based on these recommendations, the following parameters may be used for the design of conventional retaining walls:

Static Equivalent Fluid Weight (pcf)		
Condition	Level Backfill	2:1 Sloping Backfill
Active	35	55
At-Rest	55	80
Passive	350 (Maximum of 3 ksf)	200 (2:1 sloping front)

For upstream or downstream headwalls within the basins, retaining wall backdrains should not be provided below the design water level, since this would allow an open avenue of water seepage into the pipe bedding material. Instead, these walls should be designed to tolerate the hydrostatic forces acting on the walls, assuming soil and water unit weights of 130 pcf and 62.4 pcf respectively

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.

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, an allowable frictional resistance coefficient of 0.30 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 300 psf per foot increase in width or depth to a maximum allowable bearing pressure of 4,000 psf.

Cement Type and Corrosion Protection

Based on the results of our laboratory testing, concrete structures in contact with the onsite soil are generally expected to have negligible exposure to water-soluble sulfates in the soil. Common Type II cement may be used for concrete construction onsite and the concrete should be designed in accordance with the CBC (2010).

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

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.

Cantilever shoring should be designed based on an active fluid pressure of 40 pcf, assuming level ground above the shoring. 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.

<u>Limitations</u>

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, the findings, conclusions, and recommendations presented in this report are based on the assumption that Leighton Consulting will provide geotechnical observation and testing during construction.

This report was prepared for the sole use by the project team for the project specifically described herein.

We appreciate this opportunity to be of service to the San Gorgonio Pass Water Agency. If you have any questions regarding this report, please call our office at your convenience.

Respectfully submitted,

LEIGHTON CONSULTING, INC.

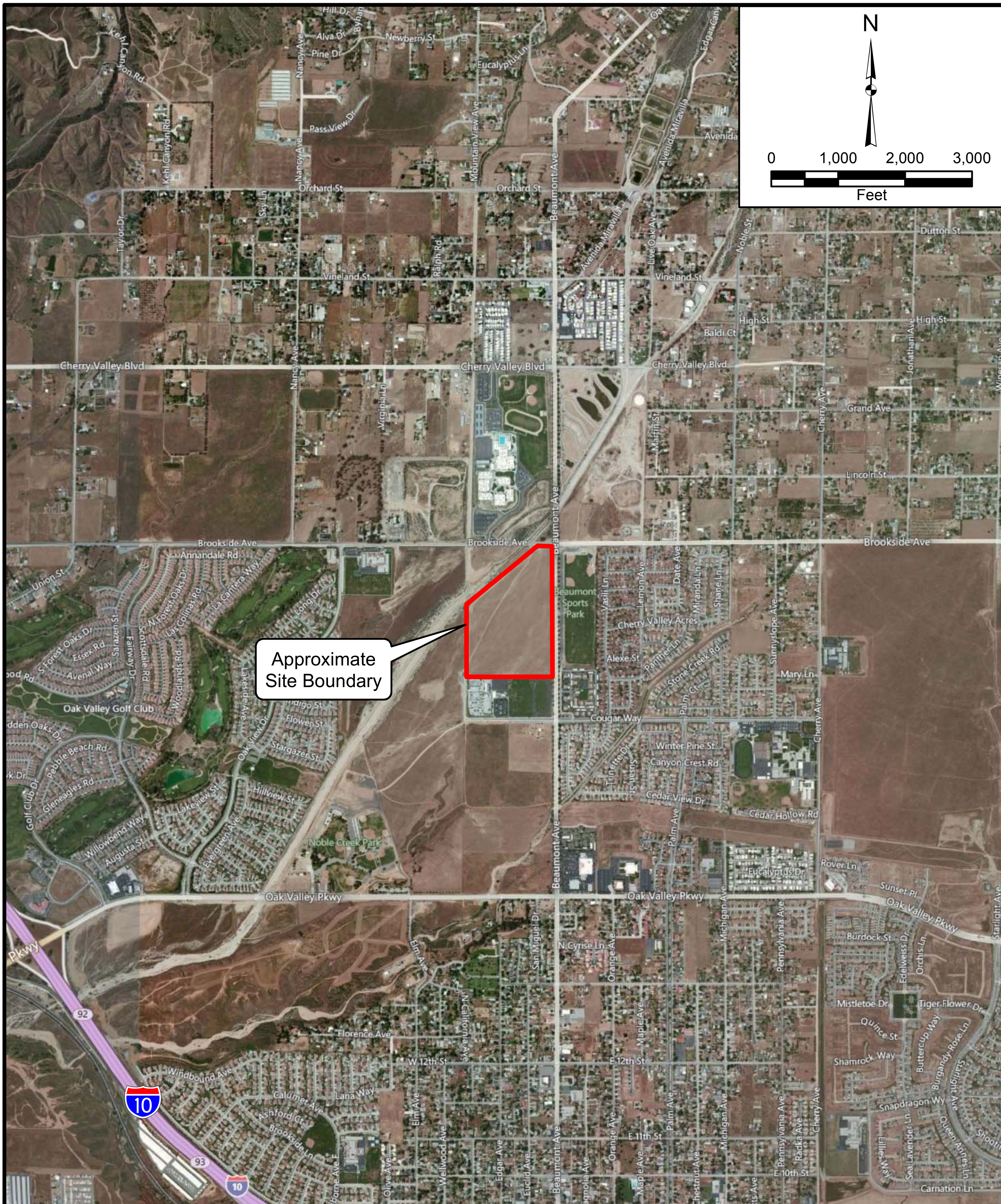
Philip A. Buchiarelli, CEG 1715
Principal Geologist

Jason D. Hertzberg, GE 2711
Associate Engineer

PB/JDH/rsm

Attachments: Figure 1 - Site Location Map
Figure 2 - Boring Location Map
Figure 3 - Retaining Wall Backfill and Subdrain Detail
Appendix A - References
Appendix B - Geotechnical Boring and Trench Logs
Appendix C - Laboratory Test Results
Appendix D - Slope Stability Analysis
Appendix E- General Earthwork and Grading Specifications

Distribution: (3) Addressee



Project: 603154-003	Eng/Geol: JDH/PB
Scale: 1 "=2,000 '	Date: November 2012

Base Map: ESRI Resources, 2010

Thematic Info: Leighton
Author: MAM

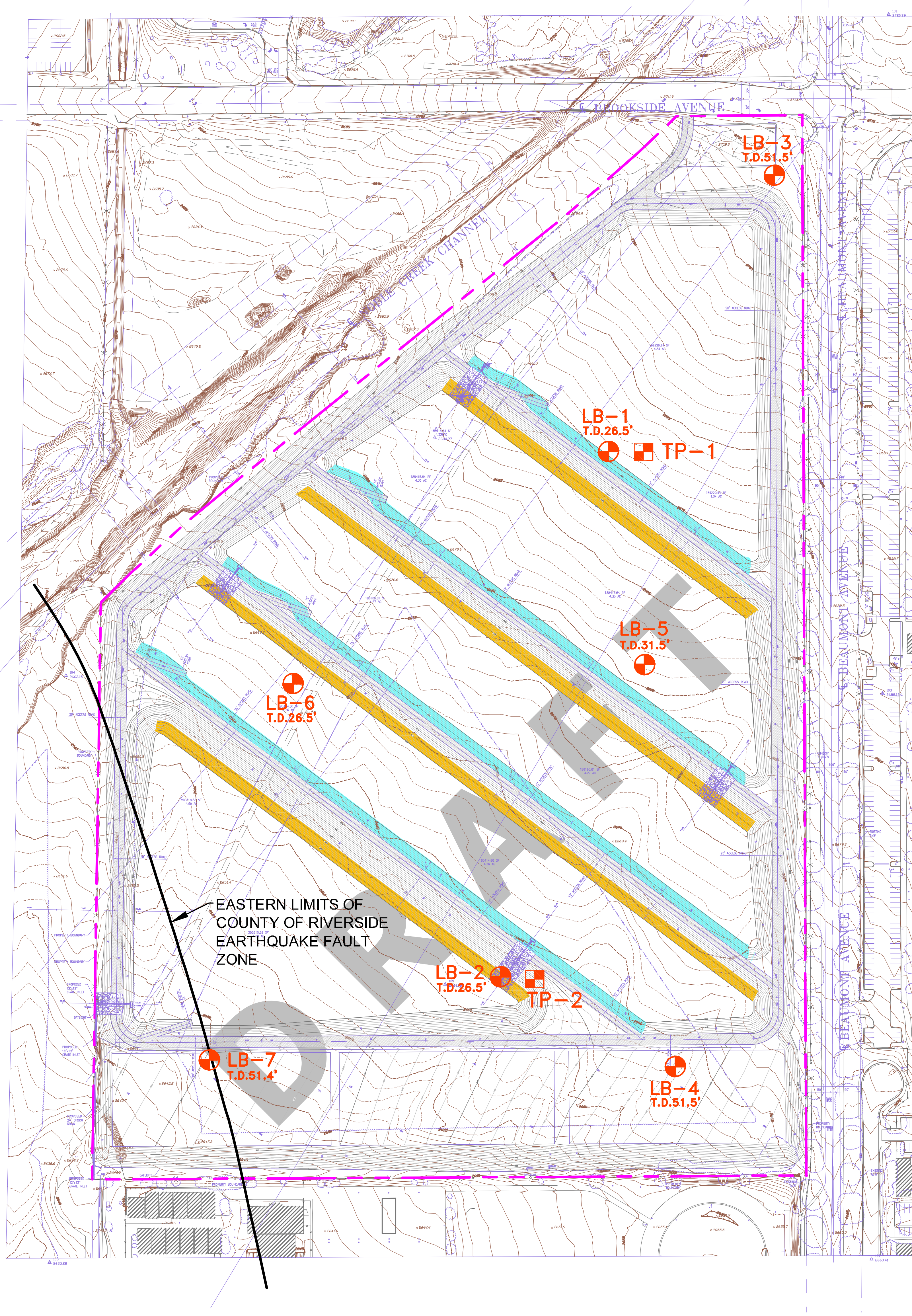
SITE LOCATION MAP

San Gorgonio Pass Water Agency
Southwest Corner of Beaumont Avenue
and Brookside Avenue
Beaumont, California

Figure 1

Leighton

Map Saved: as \\geomatics\data2\Drafting\603024\001\of_2011-05-05\GIS\Figure1.mxd at (12/17/2010 @ 10:55 hrs)



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SCALE FEET

LEGEND

LB-7 T.D.51.4' — APPROXIMATE LOCATION OF BORINGS (LEIGHTON CONSULTING INC., MAY, 2011)

TP-2 — APPROXIMATE LOCATION OF TEST PITS (LEIGHTON CONSULTING INC., MAY, 2011)

RECONSTRUCT CUT SLOPE

15' WIDE 5' DEEP SEEPAGE CUT-OFF KEY

BORING LOCATION MAP

San Gorgonio Pass Water Agency
Southwest Corner of Beaumont Avenue and Brookside Avenue
Beaumont, California

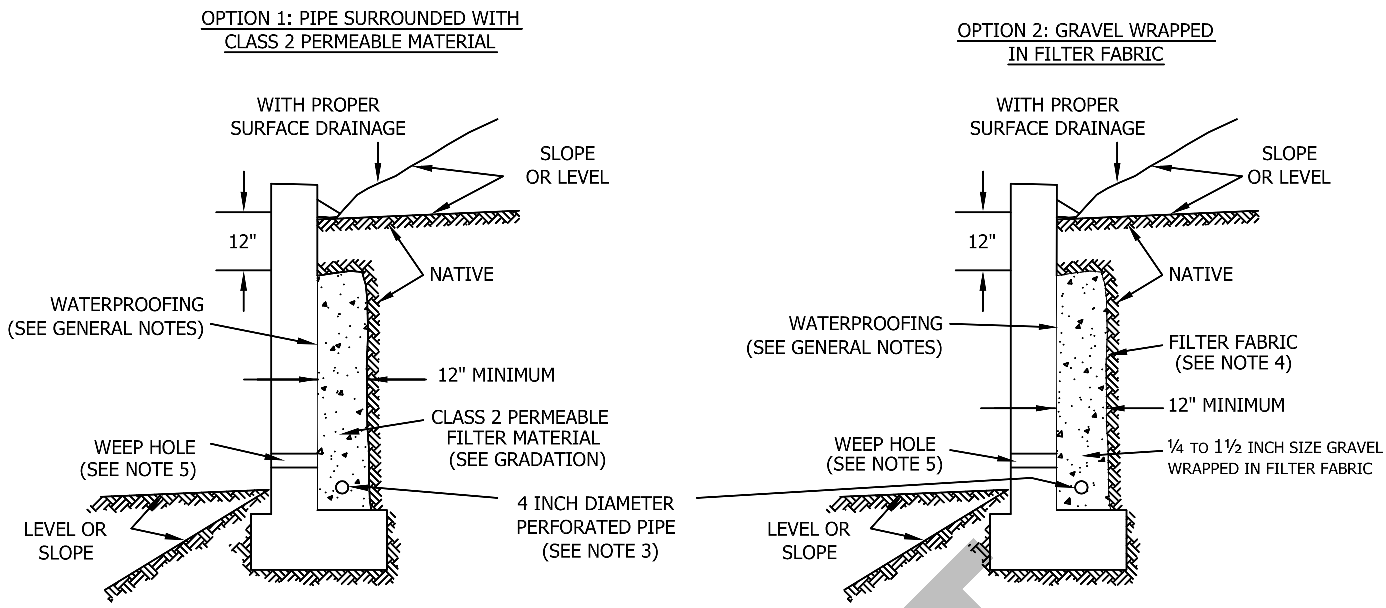
Proj: 603154-003	Eng/Geol: JDH/PB
Scale: 1"=200'	Date: 11/2012

Drafted By: MAM Checked By:

FIGURE 2

Leighton

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

Leighton

Figure 3

P:Drafting\templates\details\retain-wall-backfill-and subdrain.dwg (7/00)

APPENDIX A

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APPENDIX A

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APPENDIX B

GEOTECHNICAL BORING AND TRENCH LOGS

GEOTECHNICAL BORING LOG LB-1

Project No.	603154-002	Date Drilled	4-12-11
Project	SGPWA Infiltration Testing, Beaumont	Logged By	MDH
Drilling Co.	2R Drilling, Inc.	Hole Diameter	8"
Drilling Method	Hollow Stem Auger - 140lb - Autohammer - 30" Drop	Ground Elevation	±2692'
Location	400-ft S/o Brookside Ave, 350-ft W/o Beaumont Ave	Sampled By	MDH

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 *The Soil Description applies only to a location of the exploration at the time of drilling. 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								**Alluvium (Qal)**	
								SM	@Surface: SILTY SAND, loose, very moist, tall grass and brush	
				R-1	8 13 14	114	3	SW	@2.5': Well-graded SAND with gravel, medium dense, brown, moist, fine to coarse sand, fine to coarse gravel	
	5			R-2	7 5 5	104	7	SM	@5': SILTY SAND with gravel, loose, brown, moist, fine to coarse sand, fine to coarse gravel, 15% gravel, 67% sand, 18% fines	SA
				R-3	3 5 7	105	7	SM	@7.5': SILTY SAND, loose, dark brown, moist, fine sand, no gravel	
	10			R-4	4 6 7	112	8	SM	@10': SILTY SAND, loose, dark brown, moist, fine sand, trace coarse angular gravel	
				R-5	6 11 15	112	9	SM	@12.5': SILTY SAND with gravel, medium dense, brown to dark brown, moist, fine sand, coarse angular gravel	
	15			R-6	6 15 21			SM	@15': SILTY SAND with gravel, medium dense, brown to dark brown, moist, fine sand, coarse angular gravel	
								SP-SM	@16.5': Poorly graded SAND with silt and gravel, medium dense, brown, moist	
				R-7	17 23 19			SW	@17.5': Well-graded SAND with gravel, medium dense, light brown to white, slightly moist, fine to coarse gravel, angular	
	20			R-8	10 17 32			SP	@20': Poorly graded SAND, medium dense, brown, slightly moist, medium sand, trace gravel	
				R-9	10 15 25			SP-SM	@22.5': Poorly graded SAND with silt and gravel, medium dense, brown, moist, fine to medium sand, subrounded	
	25			R-10	17 37 43			SW	@25': Well-graded SAND with gravel, very dense, brown, slightly moist, fine to coarse gravel, subrounded	
									Total depth of boring 26.5 feet **Groundwater not encountered** **Backfilled with soil cuttings to 17 feet, bentonite chips to 15 feet** **Percolation pipe installed in hole without gravel, 4/12/2011**	
	30									

SAMPLE TYPES:

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

TYPE OF TESTS:

DS	DIRECT SHEAR	SA	SIEVE ANALYSIS	-200	% FINES PASSING
MD	MAXIMUM DENSITY	SE	SAND EQUIVALENT	AL	ATTERBERG LIMITS
CN	CONSOLIDATION	EI	EXPANSION INDEX	CO	COLLAPSE
CR	CORROSION	RV	R VALUE	PP	POCKET PENETROMETER
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-2

Project No.	603154-002	Date Drilled	4-12-11
Project	SGPWA Infiltration Testing, Beaumont	Logged By	MDH
Drilling Co.	2R Drilling, Inc.	Hole Diameter	8"
Drilling Method	Hollow Stem Auger - 140lb - Autohammer - 30" Drop	Ground Elevation	±2668'
Location	400-ft N/o Mountain View Middle School, 400-ft W/o Beaumont Ave	Sampled By	MDH

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 *The Soil Description applies only to a location of the exploration at the time of drilling. 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								**Alluvium (Qal)**	
								SM	@Surface: SILTY SAND, loose, brown to grayish brown, moist, tall grass and brush	
				R-1	4 4 8	118	4	SW	@2.5': Well-graded SAND with gravel, loose, brown, moist	
	5			R-2	6 7 8	109	3	SW	@5': Well-graded SAND with gravel, loose, brown, moist	
								SM	@6.5': SILTY SAND, loose, brown, moist	
				R-3	6 10 12	118	6	SW-SM	@7.5': Well-graded SAND with silt and gravel, medium dense, brown, moist, fine to coarse gravel	
	10			R-4	8 16 24	113	4	SW-SM	@10': Well-graded SAND with silt and gravel, medium dense, brown, moist, fine to coarse gravel	
				R-5	6 10 13	117	11	SM	@12.5': SILTY SAND, stiff, brown, moist, fine sand, 3% gravel, 63% sand, 34% fines	SA
	15			R-6	7 17 20			SW	@15': Well-graded SAND with gravel, medium dense, light brown, moist, subangular	
				R-7	18 24 32			SW	@17.5': Well-graded SAND with gravel, dense, light brown to dark gray, moist, coarse sand with some fine to medium sand	
	20			R-8	25 50/6"			SW	@20': Well-graded SAND with gravel, very dense, light brown to dark gray, moist	
				R-9	16 24 22			SW	@22.5': Well-graded SAND with gravel, medium dense, brown, moist, subrounded	
								SP	@24': Poorly graded SAND, medium dense, brown, moist, fine to medium sand, trace gravel	
	25			R-10	15 35 50			SW	@25': Well-graded SAND with gravel, very dense, light brown to brown, moist, large white granitic rock in shoe	
									Total depth of boring 26.5 feet **Groundwater not encountered** **Backfilled with soil cuttings to 17 feet, bentonite chips to 15 feet** **Percolation pipe installed in hole without gravel, 4/12/2011**	
	30									

SAMPLE TYPES:

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

TYPE OF TESTS:

DS DIRECT SHEAR
MD MAXIMUM DENSITY
CN CONSOLIDATION
CR CORROSION
UC UNCONFINED COMPRESSIVE STRENGTH
SA SIEVE ANALYSIS
SE SAND EQUIVALENT
EI EXPANSION INDEX
RV R VALUE
-200 % FINES PASSING
AL ATTERBERG LIMITS
CO COLLAPSE
PP POCKET PENETROMETER

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GEOTECHNICAL BORING LOG LB-3

Project No.	603154-002	Date Drilled	5-11-11
Project	SGPWA Infiltration Testing, Beaumont	Logged By	MDH
Drilling Co.	2R Drilling, Inc.	Hole Diameter	8"
Drilling Method	Hollow Stem Auger - 140lb - Autohammer - 30" Drop	Ground Elevation	±2711'
Location	See Boring Location Map, Figure 2	Sampled By	MDH

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								**Alluvium (Qal)**	
				B-1				SM	@Surface: SILTY SAND with gravel, brown, dry	
	5			R-1	3 5 7	114	10	SM	@5': SILTY SAND with gravel, loose, brown, moist, fine to medium sand	
	10			R-2	3 4 6	116	8	SM	@10': SILTY SAND with gravel, loose, brown, moist, fine to medium sand, rootlets, trace gravel	
	15			R-3	12 16 16	118	4	SW	@15': Well-graded SAND with gravel, medium dense, light brown, moist, fine gravel, subangular	
	20			R-4	15 17 22			SW	@20': Well-graded SAND with gravel, medium dense, light brown, moist, fine gravel, subangular	
	25			S-1	5 5 5			SM	@25': SILTY SAND, medium dense, brown, moist, fine sand, trace gravel	
	30									

The Soil Description applies only to a location of the exploration at the time of drilling. 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.

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

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

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

GEOTECHNICAL BORING LOG LB-3

Project No.	603154-002	Date Drilled	5-11-11
Project	SGPWA Infiltration Testing, Beaumont	Logged By	MDH
Drilling Co.	2R Drilling, Inc.	Hole Diameter	8"
Drilling Method	Hollow Stem Auger - 140lb - Autohammer - 30" Drop	Ground Elevation	±2711'
Location	See Boring Location Map, Figure 2	Sampled By	MDH

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 *The Soil Description applies only to a location of the exploration at the time of drilling. 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
	30			S-2	5 7 18			SW	@30': Well-graded SAND with gravel, medium dense, brown, moist, fine to coarse gravel, interbedded silty sand to 1-inch-thick	
	35			S-3	5 5 6			SM	@35': SILTY SAND, medium dense, brown, moist, fine to coarse sand, interbedded well-graded sand	
	40			S-4	9 11 10			SW	@40': Well-graded SAND with gravel, medium dense, brown, moist	
	45			S-5	5 10 16			SM	@45': SILTY SAND, medium dense, brown, moist, fine to coarse sand, interbedded well-graded sand	
	50			S-6	7 6 17			SW	@50': Well-graded SAND with gravel, medium dense, moist, fine to coarse gravel, subangular, interbedded silty sand	
									Total depth of boring 51.5 feet **Groundwater not encountered** **Backfilled with cuttings 5/11/2011**	
	55									
	60									

SAMPLE TYPES:

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

TYPE OF TESTS:

DS	DIRECT SHEAR	SA	SIEVE ANALYSIS	-200	% FINES PASSING
MD	MAXIMUM DENSITY	SE	SAND EQUIVALENT	AL	ATTERBERG LIMITS
CN	CONSOLIDATION	EI	EXPANSION INDEX	CO	COLLAPSE
CR	CORROSION	RV	R VALUE	PP	POCKET PENETROMETER
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-4

Project No.	603154-002	Date Drilled	5-11-11
Project	SGPWA Infiltration Testing, Beaumont	Logged By	MDH
Drilling Co.	2R Drilling, Inc.	Hole Diameter	8"
Drilling Method	Hollow Stem Auger - 140lb - Autohammer - 30" Drop	Ground Elevation	±2667'
Location	See Boring Location Map, Figure 2	Sampled By	MDH

SOIL DESCRIPTION

The Soil Description applies only to a location of the exploration at the time of drilling. 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								**Alluvium (Qal)**	
				B-1					@Surface: SILTY SAND with gravel, brown, dry	MD
	5			R-1	7 11 12			SW	@5': Well-graded SAND with gravel, medium dense, brown, moist, fine to coarse sand, large gravel lodged in sampler shoe, little sample recovery	
	10			R-2	8 16 27	121	3	SW	@10': Well-graded SAND with gravel, medium dense, brown, moist, fine to coarse sand	
	15			R-3	15 31 33	124	3	SW	@15': Well-graded SAND with gravel, dense, brown, moist, fine to coarse sand	
	20			R-4	19 21 23	123	7	SM	@20': SILTY SAND with gravel, medium dense, reddish brown, fine to coarse weathered granitic gravel	
				S-1	16 17 23			SM	@23': SILTY SAND, dense, reddish brown, moist	
	25			S-2	3 5 6			SM	@25': SILTY SAND, medium dense, reddish brown, moist, 1% gravel, 56% sand, 43% fines	SA
				S-3	2 8 15			ML SP	@27.5': SILT, stiff, reddish brown, moist @28': Poorly graded SAND, medium dense, brown, dry, fine sand	
	30									

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

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

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

GEOTECHNICAL BORING LOG LB-4

Project No.	603154-002	Date Drilled	5-11-11
Project	SGPWA Infiltration Testing, Beaumont	Logged By	MDH
Drilling Co.	2R Drilling, Inc.	Hole Diameter	8"
Drilling Method	Hollow Stem Auger - 140lb - Autohammer - 30" Drop	Ground Elevation	±2667'
Location	See Boring Location Map, Figure 2	Sampled By	MDH

SOIL DESCRIPTION

The Soil Description applies only to a location of the exploration at the time of drilling. 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
	30			S-4	3 5 5			ML	@30': SILT, stiff, reddish brown, moist, interbedded layers of brown fine poorly graded sand	
				S-5	5 7 9			ML	@32.5': SILT, very stiff, reddish brown, moist, interbedded layers of brown fine poorly graded sand	
	35			S-6	6 8 8			ML	@35': SANDY SILT, very stiff, reddish brown, moist, fine sand	
	40			S-7	4 8 10			ML	@40': SANDY SILT, very stiff, reddish brown, moist, fine sand, fine poorly graded sand in sampler shoe	
	45			S-8	5 6 9			ML	@45': SANDY SILT, very stiff, reddish brown, moist, fine sand	
	50			S-9	19 22 24			SP	@50': Poorly graded SAND with silt, dense, reddish brown, slightly moist, fine to medium sand	
									Total depth of boring 51.5 feet **Groundwater not encountered** **Backfilled with cuttings 5/11/2011**	
	55									
	60									

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

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

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GEOTECHNICAL BORING LOG LB-5

Project No.	603154-002	Date Drilled	5-11-11
Project	SGPWA Infiltration Testing, Beaumont	Logged By	MDH
Drilling Co.	2R Drilling, Inc.	Hole Diameter	8"
Drilling Method	Hollow Stem Auger - 140lb - Autohammer - 30" Drop	Ground Elevation	±2680'
Location	See Boring Location Map, Figure 2	Sampled By	MDH

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
									The Soil Description applies only to a location of the exploration at the time of drilling. 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.	
	0								**Alluvium (Qal)**	
				B-1				SM	@Surface: SILTY SAND with gravel, brown, dry	
	5			R-1	3 4 4	112	8	SM	@5': SILTY SAND, loose, dark brown, slightly moist, fine sand	
	10			R-2	4 5 7	116	8	SM	@10': SILTY SAND, loose, brown, moist, fine sand, trace fine gravel	
	15			R-3	5 7 10	117	7	SM	@15': SILTY SAND, loose, brown, slightly moist, fine to medium sand, 5% gravel, 72% sand, 23% fines	SA
	20			S-1	8 9 7			SW	@20': Well-graded SAND, medium dense, pale brown, slightly moist	
	25			S-2	15 19 16			SW	@25': Well-graded SAND, dense, pale brown, slightly moist	
	30									

SAMPLE TYPES:

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

TYPE OF TESTS:

DS	DIRECT SHEAR	SA	SIEVE ANALYSIS	-200	% FINES PASSING
MD	MAXIMUM DENSITY	SE	SAND EQUIVALENT	AL	ATTERBERG LIMITS
CN	CONSOLIDATION	EI	EXPANSION INDEX	CO	COLLAPSE
CR	CORROSION	RV	R VALUE	PP	POCKET PENETROMETER
UC	UNCONFINED COMPRESSIVE STRENGTH				

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GEOTECHNICAL BORING LOG LB-5

Project No.	603154-002	Date Drilled	5-11-11
Project	SGPWA Infiltration Testing, Beaumont	Logged By	MDH
Drilling Co.	2R Drilling, Inc.	Hole Diameter	8"
Drilling Method	Hollow Stem Auger - 140lb - Autohammer - 30" Drop	Ground Elevation	±2680'
Location	See Boring Location Map, Figure 2	Sampled By	MDH

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 *The Soil Description applies only to a location of the exploration at the time of drilling. 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
	30			S-3	22 24 14			SW	@30': Well-graded SAND with gravel, dense, pale brown, slightly moist	
									Total depth of boring 31.5 feet **Groundwater not encountered** **Backfilled with cuttings 5/11/2011**	
	35									
	40									
	45									
	50									
	55									
	60									

SAMPLE TYPES:

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

TYPE OF TESTS:

DS	DIRECT SHEAR	SA	SIEVE ANALYSIS	-200	% FINES PASSING
MD	MAXIMUM DENSITY	SE	SAND EQUIVALENT	AL	ATTERBERG LIMITS
CN	CONSOLIDATION	EI	EXPANSION INDEX	CO	COLLAPSE
CR	CORROSION	RV	R VALUE	PP	POCKET PENETROMETER
UC	UNCONFINED COMPRESSIVE STRENGTH				

GEOTECHNICAL BORING LOG LB-6

Project No.	603154-002	Date Drilled	5-11-11
Project	SGPWA Infiltration Testing, Beaumont	Logged By	MDH
Drilling Co.	2R Drilling, Inc.	Hole Diameter	8"
Drilling Method	Hollow Stem Auger - 140lb - Autohammer - 30" Drop	Ground Elevation	±2672'
Location	See Boring Location Map, Figure 2	Sampled By	MDH

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								**Alluvium (Qal)**	
								SM	@Surface: SILTY SAND with gravel, brown, dry	
	5			R-1	4 5 7	117	6	SM	@5': SILTY SAND with gravel, loose, dark brown, moist, fine to medium sand, subrounded	
									@9': Gravel layer, increased drilling resistance	
	10			R-2	4 4 6	105	7	SM	@10': SILTY SAND, loose, brown, slightly moist, fine sand, trace coarse rounded gravel	
	15			R-3	10 18 26	124	2	SW	@15': Well-graded SAND with gravel, medium dense, light brown, slightly moist, fine to coarse angular gravel, near 50% gravel	
	20			S-1	9 14 16			SW	@20': Well-graded SAND with gravel, dense, light brown, slightly moist, fine to coarse angular gravel	
	25			S-2	12 13 13			SW	@25': Well-graded SAND with gravel, medium dense, light brown, slightly moist, fine to coarse angular gravel	
									Total depth of boring 26.5 feet **Groundwater not encountered** **Backfilled with cuttings 5/11/2011**	
	30									

The Soil Description applies only to a location of the exploration at the time of drilling. 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.

SAMPLE TYPES:

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

TYPE OF TESTS:

- DS DIRECT SHEAR
- MD MAXIMUM DENSITY
- CN CONSOLIDATION
- CR CORROSION
- UC UNCONFINED COMPRESSIVE STRENGTH
- SA SIEVE ANALYSIS
- SE SAND EQUIVALENT
- EI EXPANSION INDEX
- RV R VALUE
- -200 % FINES PASSING
- AL ATTERBERG LIMITS
- CO COLLAPSE
- PP POCKET PENETROMETER

GEOTECHNICAL BORING LOG LB-7

Project No.	603154-002	Date Drilled	5-11-11
Project	SGPWA Infiltration Testing, Beaumont	Logged By	MDH
Drilling Co.	2R Drilling, Inc.	Hole Diameter	8"
Drilling Method	Hollow Stem Auger - 140lb - Autohammer - 30" Drop	Ground Elevation	±2651'
Location	See Boring Location Map, Figure 2	Sampled By	MDH

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 *The Soil Description applies only to a location of the exploration at the time of drilling. 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								**Alluvium (Qal)**	
				B-1				SM	@Surface: SILTY SAND with gravel, brown, dry	
	5			R-1	3 5 6	117	8	SM	@5': SILTY SAND, loose, brown, dry, fine sand	
	10			R-2	3 9 13	108	4	SW	@10': Well-graded SAND, medium dense, light brown to brown, slightly moist	
	15			R-3	6 16 23	122	6	SM	@15': SILTY SAND, medium dense, brown, moist, fine sand, interbedded layers of well-graded sand, 9% gravel, 73% sand, 18% fines	SA
	20			R-4	7 11 12			SM	@20': SILTY SAND, medium dense, brown, moist, fine sand, interbedded layers of well-graded sand	
	25			S-1	9 17 12			SW	@25': Well-graded SAND, medium dense, light brown, dry	
	30									

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

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

GEOTECHNICAL BORING LOG LB-7

Project No.	603154-002	Date Drilled	5-11-11
Project	SGPWA Infiltration Testing, Beaumont	Logged By	MDH
Drilling Co.	2R Drilling, Inc.	Hole Diameter	8"
Drilling Method	Hollow Stem Auger - 140lb - Autohammer - 30" Drop	Ground Elevation	±2651'
Location	See Boring Location Map, Figure 2	Sampled By	MDH

SOIL DESCRIPTION

The Soil Description applies only to a location of the exploration at the time of drilling. 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
	30			S-2	6 17 29			SM	@30': SILTY SAND with gravel, dense, brown, slightly moist, fine gravel, interbedded layers of silty sand, 23% gravel, 63% sand, 14% fines	SA
	35			S-3	17 11 8			SW-SM	@35': Well-graded SAND with silt and gravel, medium dense, pale brown, moist	
	40			S-4	7 8 9			SP-SM	@40': Poorly graded SAND with silt, medium dense, brown, slightly moist, fine to medium sand, interbedded layers of well-graded sand	
	45			S-5	24 31 34			SW	@45': Well-graded SAND with gravel, very dense, light brown to white, dry, subangular	
	50			S-6	10 28 50/4"			SP	@50': Poorly graded SAND, very dense, brown, moist, fine sand, well-graded sand at tip of sampler shoe	
									Total depth of boring 51.5 feet **Groundwater not encountered** **Backfilled with cuttings 5/11/2011**	
	55									
	60									

SAMPLE TYPES:

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

TYPE OF TESTS:

DS	DIRECT SHEAR	SA	SIEVE ANALYSIS	-200	% FINES PASSING
MD	MAXIMUM DENSITY	SE	SAND EQUIVALENT	AL	ATTERBERG LIMITS
CN	CONSOLIDATION	EI	EXPANSION INDEX	CO	COLLAPSE
CR	CORROSION	RV	R VALUE	PP	POCKET PENETROMETER
UC	UNCONFINED COMPRESSIVE STRENGTH				

GEOTECHNICAL BORING LOG TP-1

Project No.	603154-002	Date Drilled	4-12-11
Project	SGPWA Infiltration Testing, Beaumont	Logged By	MDH
Drilling Co.	Pipe Line Equipment, Inc.	Hole Diameter	"
Drilling Method	Test Pit	Ground Elevation	±2692'
Location	400-ft S/o Brookside Ave, 330-ft W/o Beaumont Ave	Sampled By	MDH

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 *The Soil Description applies only to a location of the exploration at the time of drilling. 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								**Alluvium (Qal)**	
								SM	@Surface: SILTY SAND with gravel, loose, dark brown to grayish brown, very moist, fine to coarse sand, tall grass, rootlets	
								SW	@1': Well-graded SAND with gravel, light brown to brown, moist, subrounded, caving, low to no cohesion, trace silty sand and sandy silt lenses no more than 2 inches thick and not extending around perimeter of test pit	
				B-1						
	5									
									Total depth of test pit 6.0 feet **Groundwater not encountered** **Double-ring infiltrometer installed at base of test pit, 4/12/2011**	
	10									

SAMPLE TYPES:

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

TYPE OF TESTS:

DS	DIRECT SHEAR	SA	SIEVE ANALYSIS	-200	% FINES PASSING
MD	MAXIMUM DENSITY	SE	SAND EQUIVALENT	AL	ATTERBERG LIMITS
CN	CONSOLIDATION	EI	EXPANSION INDEX	CO	COLLAPSE
CR	CORROSION	RV	R VALUE	PP	POCKET PENETROMETER
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 TP-2

Project No.	603154-002	Date Drilled	4-12-11
Project	SGPWA Infiltration Testing, Beaumont	Logged By	MDH
Drilling Co.	Pipe Line Equipment, Inc.	Hole Diameter	"
Drilling Method	Test Pit	Ground Elevation	±2668'
Location	400-ft N/o Mountain View Middle School, 380-ft W/o Beaumont Ave	Sampled By	MDH

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 *The Soil Description applies only to a location of the exploration at the time of drilling. 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								**Alluvium (Qal)**	
								SM	@Surface: SILTY SAND with gravel, loose, dark brown to grayish brown, very moist, fine to coarse sand, tall grass, rootlets	
								SW	@1': Well-graded SAND with gravel, light brown to brown, moist, subrounded, caving, low to no cohesion, trace silty sand and sandy silt lenses no more than 2 inches thick and not extending around perimeter of test pit	
				B-1						
								SP	@4.5': Poorly graded SAND, brown, moist, fine sand with some medium to coarse sand	
	5									
									Total depth of test pit 5.0 feet **Groundwater not encountered** **Double-ring infiltrometer installed at base of test pit, 4/12/2011**	
	10									

SAMPLE TYPES:

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

TYPE OF TESTS:

DS	DIRECT SHEAR	SA	SIEVE ANALYSIS	-200	% FINES PASSING
MD	MAXIMUM DENSITY	SE	SAND EQUIVALENT	AL	ATTERBERG LIMITS
CN	CONSOLIDATION	EI	EXPANSION INDEX	CO	COLLAPSE
CR	CORROSION	RV	R VALUE	PP	POCKET PENETROMETER
UC	UNCONFINED COMPRESSIVE STRENGTH				

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

APPENDIX C

LABORATORY TEST RESULTS

Leighton

TESTS for SULFATE CONTENT
CHLORIDE CONTENT and pH of SOILS

Project Name:	Webb Beaumont	Tested By :	V. Juliano	Date: 10/26/12
Project No. :	603154-003	Data Input By:	J. Ward	Date: 10/31/12

Boring No.	N/A			
Sample No.	2			
Sample Depth (ft)	N/A			
Soil Identification:	Brown (SM)g			
Wet Weight of Soil + Container (g)	158.17			
Dry Weight of Soil + Container (g)	155.03			
Weight of Container (g)	65.28			
Moisture Content (%)	3.50			
Weight of Soaked Soil (g)	100.13			

SULFATE CONTENT, DOT California Test 417, Part II

Beaker No.	30			
Crucible No.	30			
Furnace Temperature (°C)	830			
Time In / Time Out	7:00/7:45			
Duration of Combustion (min)	45			
Wt. of Crucible + Residue (g)	21.5764			
Wt. of Crucible (g)	21.5746			
Wt. of Residue (g) (A)	0.0018			
PPM of Sulfate (A) x 41150	74.07			
PPM of Sulfate, Dry Weight Basis	**77**			

CHLORIDE CONTENT, DOT California Test 422

ml of Extract For Titration (B)	30			
ml of AgNO3 Soln. Used in Titration (C)	0.6			
PPM of Chloride (C -0.2) * 100 * 30 / B	40			
PPM of Chloride, Dry Wt. Basis	**41**			

pH TEST, DOT California Test 532/643

pH Value	**6.18**			
Temperature °C	20.3			

SOIL RESISTIVITY TEST

DOT CA TEST 532 / 643

Leighton

Project Name:	Webb Beaumont	Tested By :	V. Juliano Date: 10/30/12
Project No. :	603154-003	Data Input By:	J. Ward Date: 10/31/12
Boring No.:	N/A	Depth (ft.) :	N/A
Sample No. :	2		

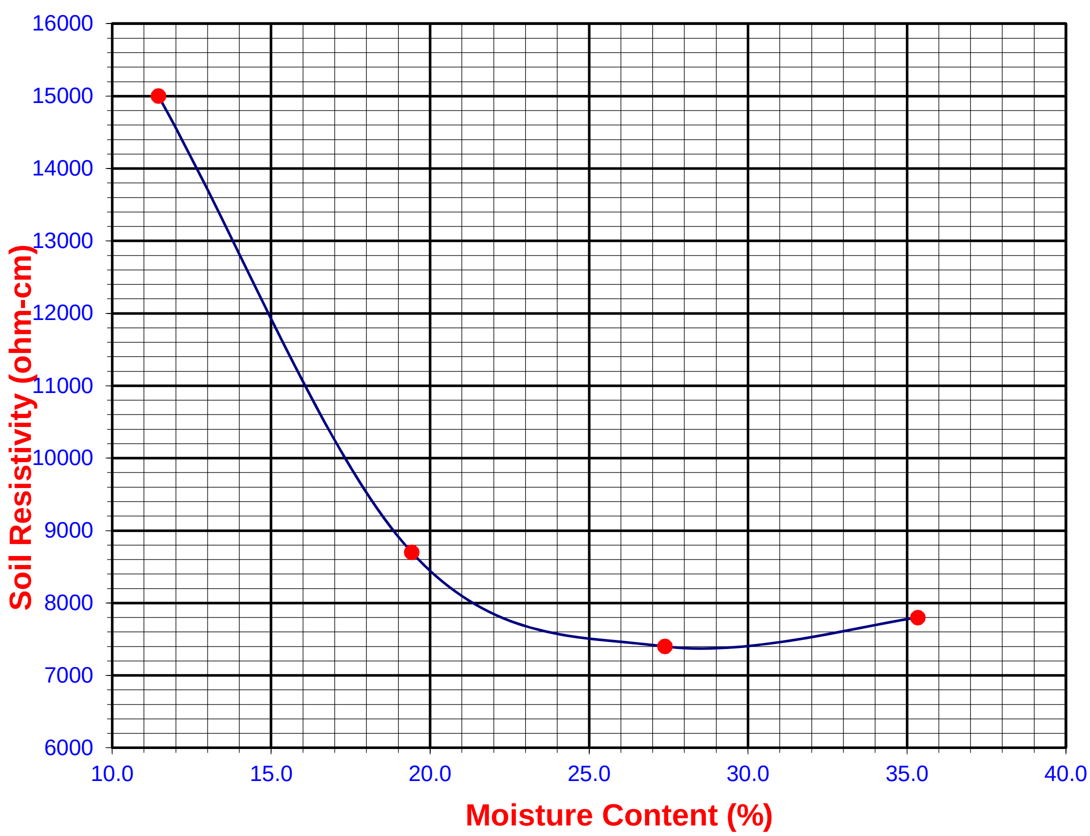
Soil Identification:* Brown (SM)g

*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	10	11.46	15000	15000
2	20	19.42	8700	8700
3	30	27.38	7400	7400
4	40	35.34	7800	7800
5				

Moisture Content (%) (MCi)	3.50
Wet Wt. of Soil + Cont. (g)	158.17
Dry Wt. of Soil + Cont. (g)	155.03
Wt. of Container (g)	65.28
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	
7400	**28.4**	**77**	**41**	**6.18**	**20.3**



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

ASTM D 6913

Leighton

Project Name:	Webb Beaumont	Tested By:	A. Santos	Date:	10/26/12
Project No.:	603154-003	Checked By:	J. Ward	Date:	10/31/12
Exploration No.:	N/A	Depth (feet):	N/A		
Sample No.:	1				
Soil Identification:	Yellowish brown silty sand with gravel (SM)g				

Calculation of Dry Weights	Whole Sample	Sample Passing #4	Moisture Contents	Whole Sample	Sample passing #4
Container No.:	SP04	R-2	Wt. of Air-Dry Soil + Cont.(g)	0.00	0.00
Wt. Air-Dried Soil + Cont.(g)	8830.60	627.90	Wt. of Dry Soil + Cont. (g)	0.00	0.00
Wt. of Container (g)	794.00	108.10	Wt. of Container No.______(g)	1.00	1.00
Dry Wt. of Soil (g)	8036.60	519.80	Moisture Content (%)	0.00	0.00

Passing #4 Material After Wet Sieve	Container No.	R-2
	Wt. of Dry Soil + Container (g)	522.70
	Wt. of Container (g)	108.10
	Dry Wt. of Soil Retained on # 200 Sieve (g)	414.60

U. S. Sieve Size	(mm.)	Cumulative Weight of Dry Soil Retained (g) Whole Sample	Sample Passing #4	Percent Passing (%)
3"	75.000			
1 1/2"	37.500	0.00		100.0
3/4"	19.000	297.20		96.3
3/8"	9.500	780.20		90.3
#4	4.750	1330.00		83.5
#8	2.360		44.60	76.3
#16	1.180		87.10	69.5
#30	0.600		136.20	61.6
#50	0.300		201.30	51.2
#100	0.150		313.30	33.2
#200	0.075		413.80	17.0
PAN				

GRAVEL: **17 %**

SAND: **66 %**

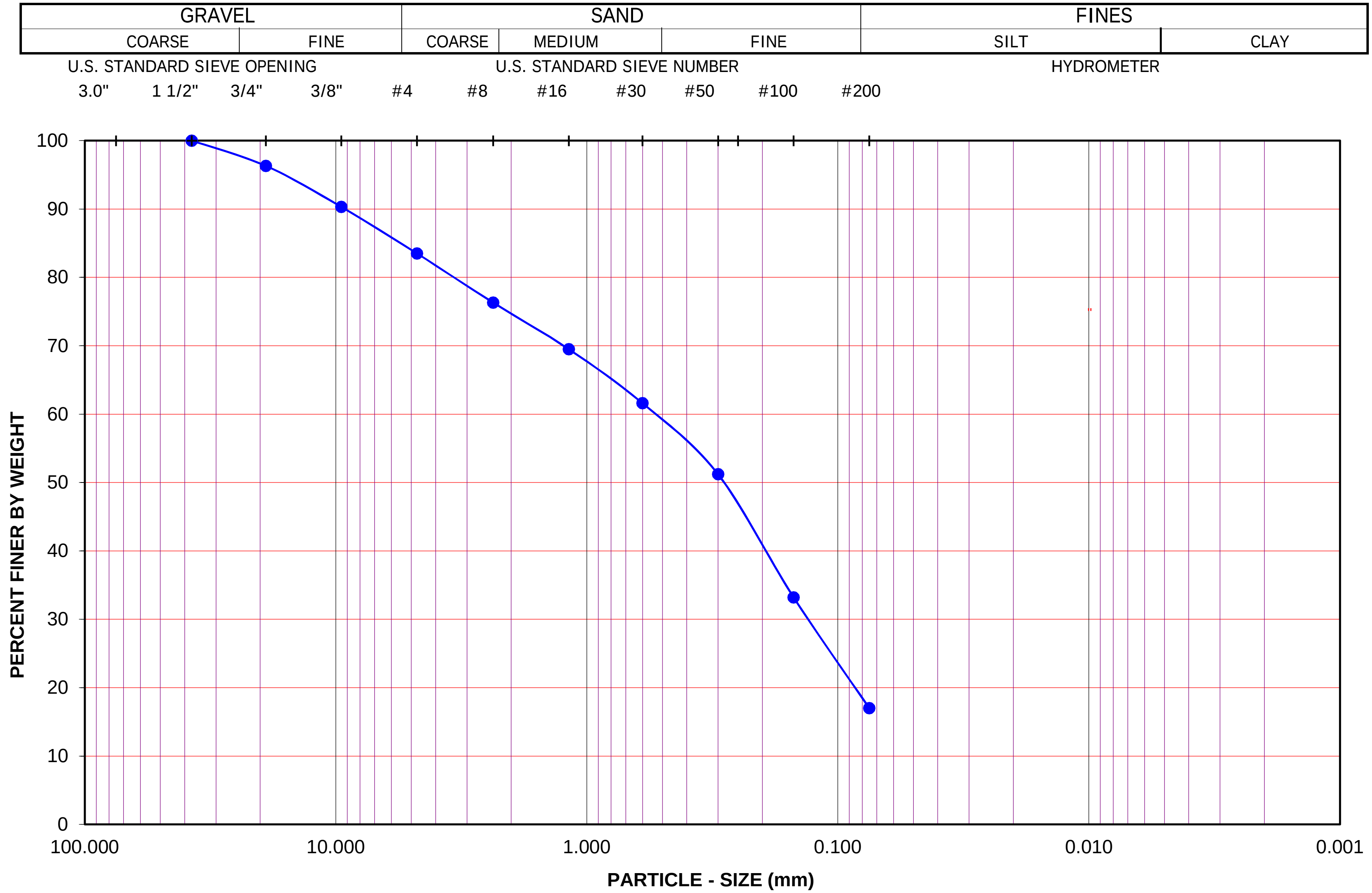
FINES: **17 %**

GROUP SYMBOL: **(SM)g**

Cu = D60/D10 = ______

Cc = (D30)²/(D60*D10) = ______

Remarks: ______



Project Name: Webb Beaumont

Project No.: 603154-003

Exploration No.: N/A

Sample No.: 1

Depth (feet): N/A

Soil Type : (SM)g

Soil Identification: Yellowish brown silty sand with gravel (SM)g

Leighton

PARTICLE - SIZE DISTRIBUTION ASTM D 6913

GR:SA:FI : (%) 17 : 66 : 17

Oct-12

SA 1 (sampled 10-24-12)

MODIFIED PROCTOR COMPACTION TEST

ASTM D 1557

Leighton

Project Name:	Webb Beaumont	Tested By :	G. Berdy	Date:	10/26/12
Project No.:	603154-003	Input By :	J. Ward	Date:	10/29/12
Boring No.:	N/A	Depth (ft.)	N/A	Revised:	10/31/12
Sample No. :	1				
Soil Identification:	Yellowish brown silty sand with gravel (SM)g				

Preparation Method:	X	Moist
		Dry
Compaction Method	X	Mechanical Ram
		Manual Ram

Scalp Fraction (%)	
#3/4	
#3/8	
#4	16.5

Rammer Weight (lb.) = 10.0

Height of Drop (in.) = 18.0

Mold Volume (ft³) 0.03340

TEST NO.	1	2	3	4	5	6
Wt. Compacted Soil + Mold (g)	3816.0	3900.0	3962.0	3964.0		
Weight of Mold (g)	1894.0	1894.0	1894.0	1894.0		
Net Weight of Soil (g)	1922.0	2006.0	2068.0	2070.0		
Wet Weight of Soil + Cont. (g)	498.80	536.40	546.40	493.20		
Dry Weight of Soil + Cont. (g)	484.50	509.50	508.60	450.60		
Weight of Container (g)	50.40	53.10	51.20	51.30		
Moisture Content (%)	3.29	5.89	8.26	10.67		
Wet Density (pcf)	126.9	132.4	136.5	136.6		
Dry Density (pcf)	122.8	125.0	126.1	123.5		

Maximum Dry Density (pcf) 126.0 **Optimum Moisture Content (%)** 8.0

Corrected Dry Density (pcf) 131.5 **Corrected Moisture Content (%)** 7.0

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

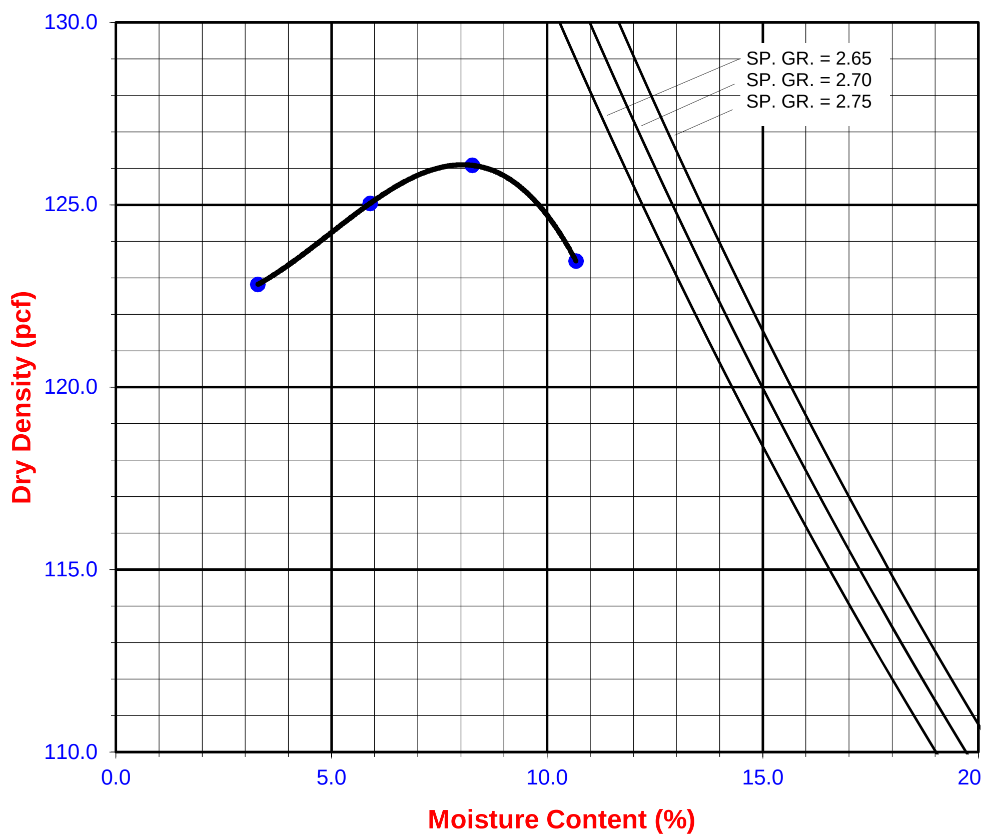
Procedure B
Soil Passing 3/8 in. (9.5 mm) Sieve
Mold : 4 in. (101.6 mm) diameter
Layers : 5 (Five)
Blows per layer : 25 (twenty-five)
Use if +#4 is >20% and +3/8 in. is 20% or less

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:
17:66:17
GR:SA:FI

Atterberg Limits:

LL,PL,PI



MX 1, revised 10-31-12 (sampled 10-24-12)

MODIFIED PROCTOR COMPACTION TEST

ASTM D 1557

Project Name:	Webb Beaumont	Tested By :	G. Berdy	Date:	10/26/12
Project No.:	603154-003	Input By :	J. Ward	Date:	10/29/12
Boring No.:	N/A	Depth (ft.)	N/A		
Sample No. :	1				
Soil Identification:	Yellowish brown silty sand with gravel (SM)g				

Note: Possible correction for oversize material; sieve analysis pending

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

Mold Volume (ft³) 0.03340 *Ram Weight = 10 lb.; Drop = 18 in.*

TEST NO.	1	2	3	4	5	6
Wt. Compacted Soil + Mold (g)	3816.0	3900.0	3962.0	3964.0		
Weight of Mold (g)	1894.0	1894.0	1894.0	1894.0		
Net Weight of Soil (g)	1922.0	2006.0	2068.0	2070.0		
Wet Weight of Soil + Cont. (g)	498.80	536.40	546.40	493.20		
Dry Weight of Soil + Cont. (g)	484.50	509.50	508.60	450.60		
Weight of Container (g)	50.40	53.10	51.20	51.30		
Moisture Content (%)	3.29	5.89	8.26	10.67		
Wet Density (pcf)	126.9	132.4	136.5	136.6		
Dry Density (pcf)	122.8	125.0	126.1	123.5		

Maximum Dry Density (pcf) **126.0** **Optimum Moisture Content (%)** **8.0**

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

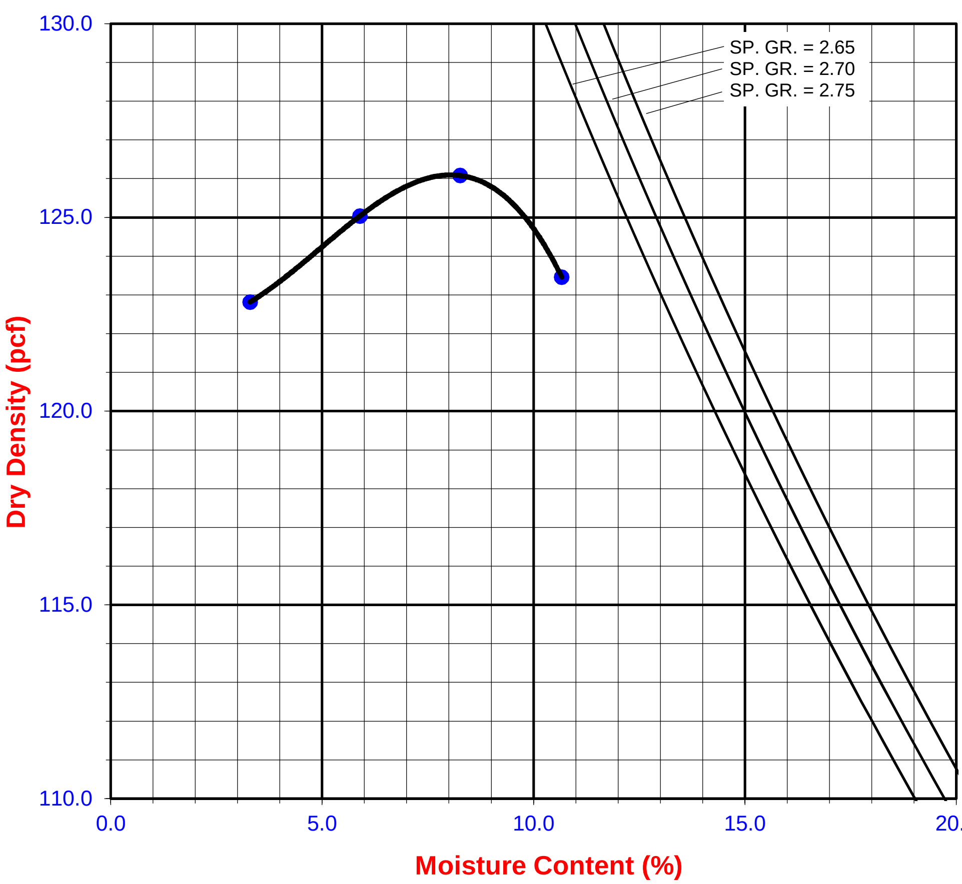
[ ] **Procedure C**
Soil Passing 3/4 in. (19.0 mm) Sieve
Mold : 6 in. (152.4 mm) diameter
Layers : 5 (Five)
Blows per layer : 56 (fifty-six)
Use if +3/8 in. is >20% and +¾ in. is <30%

Particle-Size Distribution:

GR:SA:FI

Atterberg Limits:

LL,PL,PI



MX 1 (sampled 10-24-12)

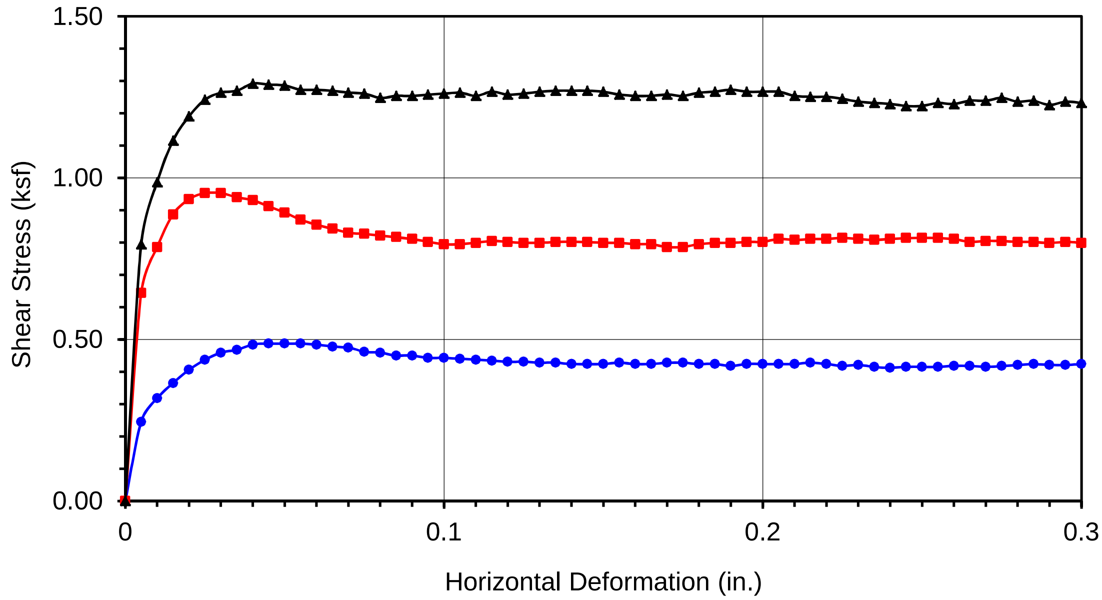
DIRECT SHEAR TEST
Consolidated Undrained

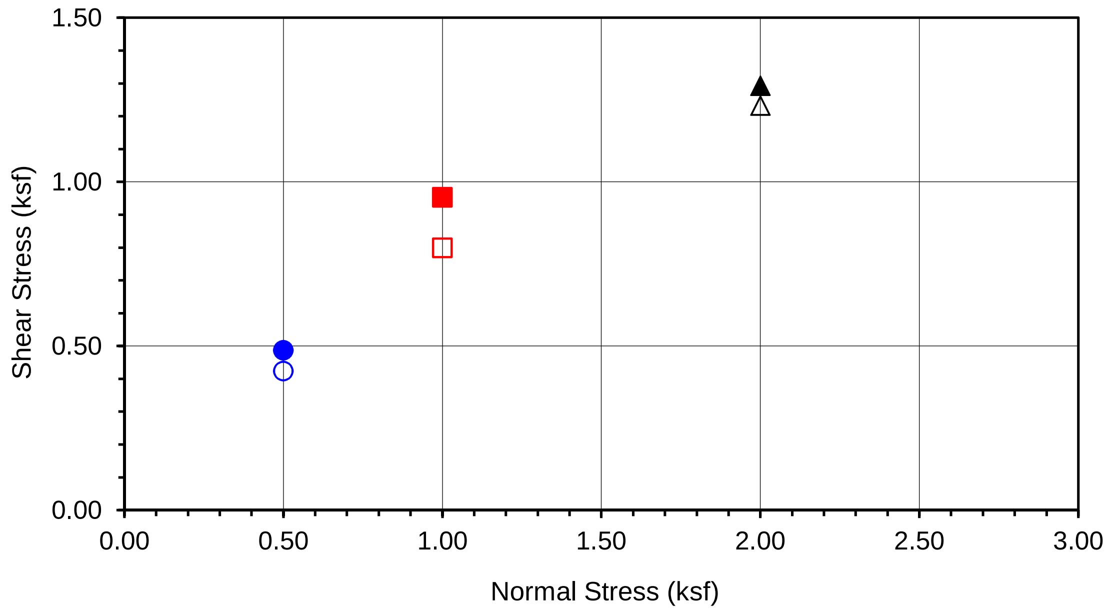
Leighton

Project Name:	Webb Beaumont	Tested By:	F. Tabibkhoei	Date:	10/29/12
Project No.:	603154-003	Checked By:	J. Ward		
Boring No.:	N/A	Sample Type:	90% Remold		
Sample No.:	1	Depth (ft.):	N/A		

Soil Identification: Yellowish brown silty sand with gravel (SM)g

Sample Diameter(in):	2.415	2.415	2.415
Sample Thickness(in.):	1.000	1.000	1.000
Weight of Sample + ring(gm):	190.30	185.69	190.32
Weight of Ring(gm):	42.92	38.31	42.94
Before Shearing			
Weight of Wet Sample+Cont.(gm):	205.15	205.15	205.15
Weight of Dry Sample+Cont.(gm):	192.83	192.83	192.83
Weight of Container(gm):	38.85	38.85	38.85
Vertical Rdg.(in): Initial	0.0000	0.2534	0.2480
Vertical Rdg.(in): Final	-0.0037	0.2585	0.2613
After Shearing			
Weight of Wet Sample+Cont.(gm):	195.60	194.61	194.26
Weight of Dry Sample+Cont.(gm):	172.50	171.80	171.48
Weight of Container(gm):	38.13	37.65	37.32
Specific Gravity (Assumed):	2.70	2.70	2.70
Water Density(pcf):	62.43	62.43	62.43





Boring No.	**N/A**
Sample No.	**1**
Depth (ft)	**N/A**

Sample Type:

90% Remold

Soil Identification:
Yellowish brown silty sand with gravel (SM)g

Normal Stress (kip/ft²)	0.500	1.000	2.000
Peak Shear Stress (kip/ft²)	● 0.487	■ 0.953	▲ 1.292
Shear Stress @ End of Test (ksf)	○ 0.424	□ 0.799	△ 1.232
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)	113.5	113.5	113.5
Saturation (%)	44.5	44.5	44.5
Soil Height Before Shearing (in.)	0.9963	0.9949	0.9867
Final Moisture Content (%)	17.2	17.0	17.0

Leighton

DIRECT SHEAR TEST RESULTS

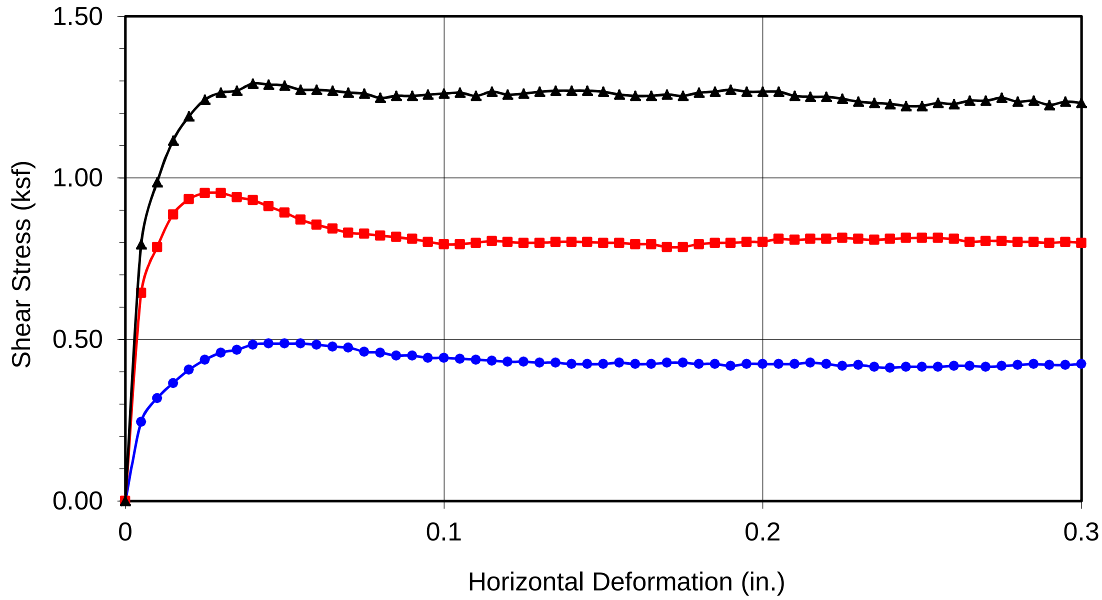
Consolidated Undrained

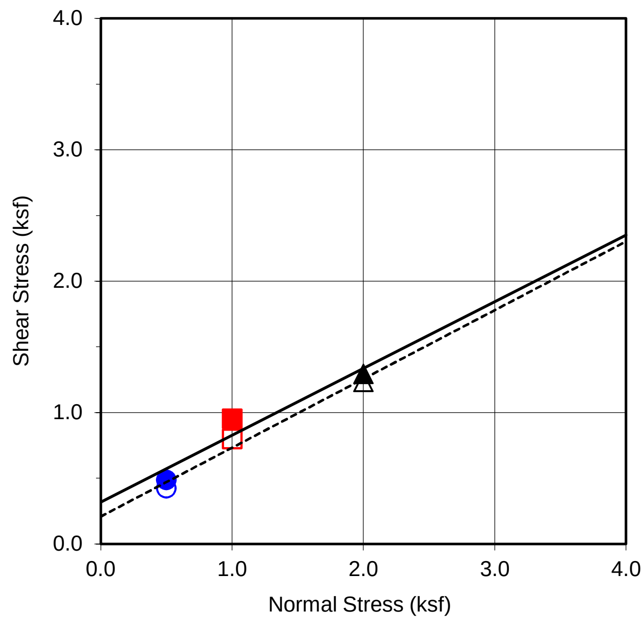
Project No.: 603154-003

Webb Beaumont

10-12

DS 1 (sampled 10-24-12)





Boring No.	**N/A**
Sample No.	**1**
Depth (ft)	**N/A**
Sample Type:	90% Remold
Soil Identification: Yellowish brown silty sand with gravel (SM)g	

Strength Parameters

	C (psf)	ϕ (°)
Peak	317.5	27.0
Ultimate	207.5	27.6

Normal Stress (kip/ft²)	0.500	1.000	2.000
Peak Shear Stress (kip/ft²)	● 0.487	■ 0.953	▲ 1.292
Shear Stress @ End of Test (ksf)	○ 0.424	□ 0.799	△ 1.232
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)	113.5	113.5	113.5
Saturation (%)	44.5	44.5	44.5
Soil Height Before Shearing (in.)	0.9963	0.9949	0.9867
Final Moisture Content (%)	17.2	17.0	17.0

Leighton

DIRECT SHEAR TEST RESULTS

Consolidated Undrained

Project No.: 603154-003

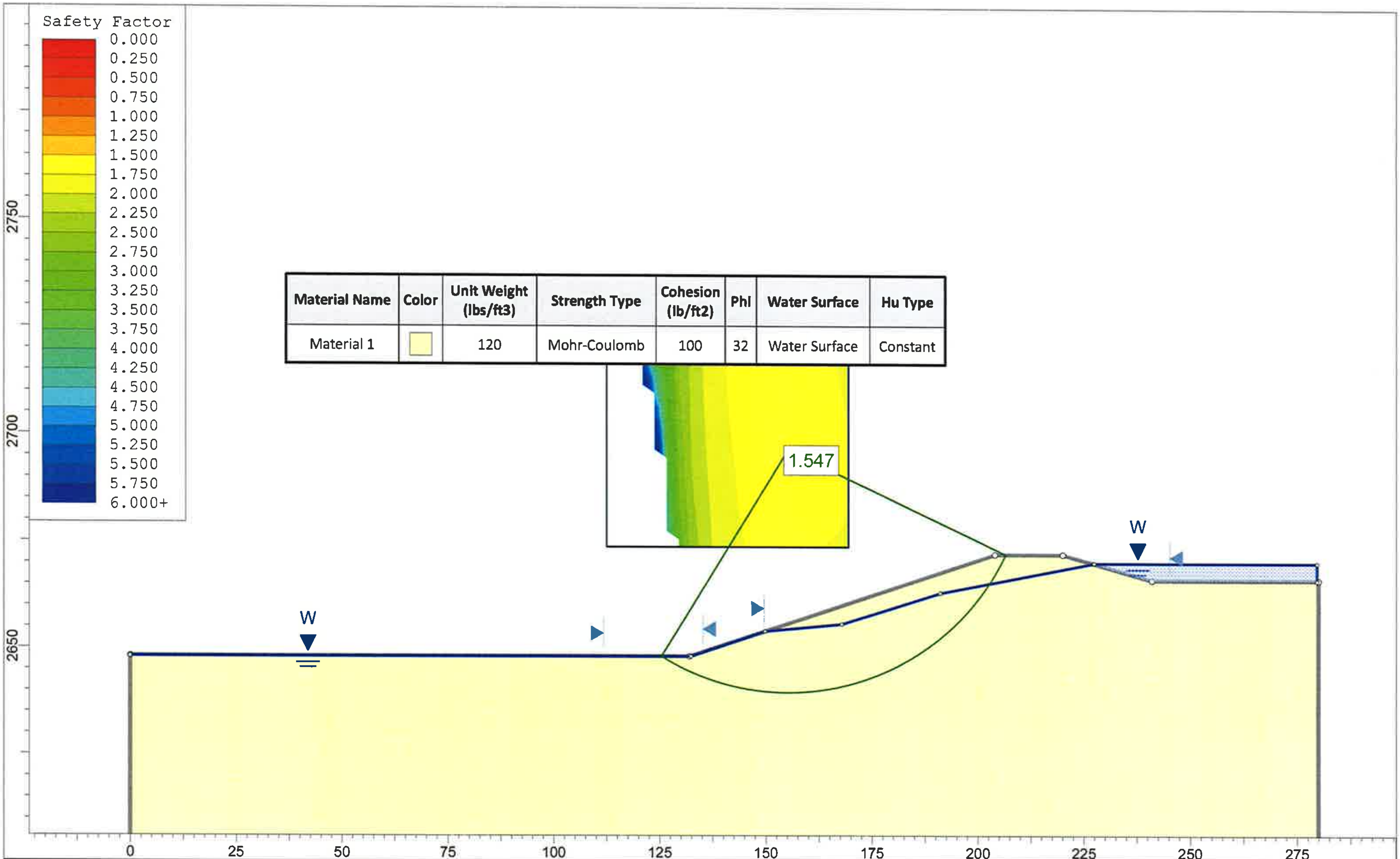
Webb Beaumont

10-12

DS 1 (sampled 10-24-12)

APPENDIX D

SLOPE STABILITY ANALYSIS



Material Name	Color	Unit Weight (lbs/ft3)	Strength Type	Cohesion (lb/ft2)	Phi	Water Surface	Hu Type
Material 1		120	Mohr-Coulomb	100	32	Water Surface	Constant

Project	SGPWA 10028.001		
Analysis Description			
Drawn By		*Scale* 1:375	*Company*
Date	11/5/2012, 4:41:26 PM	*File Name*	Slide1-rrd2.slim

rocscience

SLIDEINTERPRET 6.008

Slide Analysis Information

SGPWA 10028.001

Project Summary

File Name: Slide1-rrd2.slim
Slide Modeler Version: 6.008
Project Title: SGPWA 10028.001
Date Created: 11/5/2012, 4:41:26 PM
Comments:

Static

General Settings

Units of Measurement: Imperial Units
Time Units: days
Permeability Units: feet/second
Failure Direction: Right to Left
Data Output: Standard
Maximum Material Properties: 20
Maximum Support Properties: 20

Analysis Options

Analysis Methods Used

Bishop simplified
Janbu simplified

Number of slices: 25
Tolerance: 0.005
Maximum number of iterations: 50
Check malpha < 0.2: Yes
Initial trial value of FS: 1
Steffensen Iteration: Yes

Groundwater Analysis

Groundwater Method: Water Surfaces
Pore Fluid Unit Weight: 62.4 lbs/ft3
Advanced Groundwater Method: None

Random Numbers

Pseudo-random Seed: 10116

Random Number Generation Method: Park and Miller v.3

Surface Options

Surface Type: Circular
Search Method: Grid Search
Radius Increment: 10
Composite Surfaces: Enabled
Reverse Curvature: Create Tension Crack
Minimum Elevation: Not Defined
Minimum Depth: Not Defined

Material Properties

Property	Material 1
Color	
Strength Type	Mohr-Coulomb
Unit Weight [lbs/ft3]	120
Cohesion [psf]	100
Friction Angle [deg]	32
Water Surface	Water Table
Hu Value	1

Global Minimums

Method: bishop simplified

FS: 1.708760
Center: 152.389, 2711.602
Radius: 69.100
Left Slip Surface Endpoint: 125.379, 2648.000
Right Slip Surface Endpoint: 209.014, 2672.000
Resisting Moment=3.40736e+006 lb-ft
Driving Moment=1.99405e+006 lb-ft

Method: janbu simplified

FS: 1.546660
Center: 155.238, 2696.406
Radius: 56.867
Left Slip Surface Endpoint: 125.393, 2648.000
Right Slip Surface Endpoint: 206.602, 2672.000
Resisting Horizontal Force=49020.5 lb
Driving Horizontal Force=31694.3 lb

Valid / Invalid Surfaces

Method: bishop simplified

Number of Valid Surfaces: 5835
Number of Invalid Surfaces: 1326

Error Codes:

Error Code -113 reported for 17 surfaces
Error Code -1000 reported for 1309 surfaces

Method: janbu simplified

Number of Valid Surfaces: 5835
Number of Invalid Surfaces: 1326

Error Codes:

Error Code -113 reported for 17 surfaces
Error Code -1000 reported for 1309 surfaces

Error Codes

The following errors were encountered during the computation:

-113 = Surface intersects outside slope limits.
-1000 = No valid slip surfaces are generated at a grid center. Unable to draw a surface.

Slice Data

Global Minimum Query (bishop simplified) - Safety Factor: 1.70876

Slice Number	Width [ft]	Weight [lbs]	Base Material	Base Cohesion [psf]	Base Friction Angle [degrees]	Shear Stress [psf]	Shear Strength [psf]	Base Normal Stress [psf]	Pore Pressure [psf]	Effective Normal Stress [psf]
1	3.34541	264.761	Material 1	100	32	84.7218	144.769	112.534	40.8884	71.6459
2	3.34541	755.137	Material 1	100	32	111.998	191.378	263.324	117.089	146.235
3	3.34541	1402.02	Material 1	100	32	149.577	255.592	461.006	212.005	249.001
4	3.34541	2190.46	Material 1	100	32	191.028	326.421	698.141	335.794	362.347
5	3.34541	2908.66	Material 1	100	32	226.977	387.85	909.318	448.663	460.655
6	3.34541	3558.68	Material 1	100	32	257.895	440.68	1096.14	550.935	545.2
7	3.34541	4142.03	Material 1	100	32	284.147	485.539	1259.84	642.844	616.992
8	3.34541	4659.69	Material 1	100	32	309.925	529.588	1401.47	713.988	687.485
9	3.34541	5112.16	Material 1	100	32	346.164	591.511	1520.97	734.384	786.583
10	3.34541	5499.47	Material 1	100	32	377.797	645.564	1617.74	744.652	873.086
11	3.34541	5821.19	Material 1	100	32	404.983	692.018	1692.15	744.725	947.428
12	3.34541	6076.42	Material 1	100	32	427.835	731.067	1744.38	734.463	1009.92
13	3.34541	6263.72	Material 1	100	32	446.424	762.831	1774.4	713.643	1060.75
14	3.34541	6381.13	Material 1	100	32	448.484	766.352	1785.32	718.933	1066.39
15	3.34541	6425.99	Material 1	100	32	443.851	758.435	1775.53	721.816	1053.72
16	3.34541	6394.92	Material 1	100	32	435.49	744.148	1743.75	712.896	1030.85
17	3.34541	6283.58	Material 1	100	32	423.331	723.371	1689.1	691.499	997.603

18	3.34541	6086.51	Material 1	100	32	407.268	695.923	1610.45	656.776	953.675
19	3.34541	5796.74	Material 1	100	32	387.153	661.552	1506.32	607.645	898.672
20	3.34541	5405.35	Material 1	100	32	362.792	619.925	1374.77	542.718	832.055
21	3.34541	4900.72	Material 1	100	32	339.975	580.936	1208.76	439.101	769.659
22	3.34541	4267.38	Material 1	100	32	313.014	534.865	1007.7	311.769	695.931
23	3.34541	3484.07	Material 1	100	32	280.323	479.005	767.663	161.127	606.536
24	3.34541	2464.65	Material 1	100	32	232.382	397.085	475.435	0	475.435
25	3.34541	882.775	Material 1	100	32	104.701	178.909	126.281	0	126.281

Global Minimum Query (janbu simplified) - Safety Factor: 1.54666

Slice Number	Width [ft]	Weight [lbs]	Base Material	Base Cohesion [psf]	Base Friction Angle [degrees]	Shear Stress [psf]	Shear Strength [psf]	Base Normal Stress [psf]	Pore Pressure [psf]	Effective Normal Stress [psf]
1	3.24835	362.148	Material 1	100	32	112.381	173.816	175.838	57.708	118.13
2	3.24835	1033.94	Material 1	100	32	157.682	243.88	395.498	165.243	230.255
3	3.24835	1802.42	Material 1	100	32	209.794	324.48	641.733	282.491	359.242
4	3.24835	2703.7	Material 1	100	32	264.818	409.584	923.444	428.007	495.437
5	3.24835	3519.41	Material 1	100	32	311.218	481.348	1170.07	559.786	610.284
6	3.24835	4254	Material 1	100	32	350.117	541.512	1385.15	678.579	706.567
7	3.24835	4910.96	Material 1	100	32	382.364	591.387	1571.33	784.946	786.382
8	3.24835	5492.78	Material 1	100	32	408.589	631.949	1730.58	879.284	851.296
9	3.24835	6001.1	Material 1	100	32	449.516	695.248	1865.13	912.535	952.598
10	3.24835	6436.77	Material 1	100	32	484.467	749.306	1972.9	933.793	1039.11
11	3.24835	6799.93	Material 1	100	32	513.613	794.384	2054.69	943.443	1111.25
12	3.24835	7090	Material 1	100	32	537.178	830.832	2110.97	941.392	1169.58
13	3.24835	7305.63	Material 1	100	32	555.308	858.873	2141.88	927.425	1214.45
14	3.24835	7444.68	Material 1	100	32	561.035	867.73	2149.03	920.411	1228.62
15	3.24835	7504.05	Material 1	100	32	552.663	854.781	2133.85	925.944	1207.9
16	3.24835	7479.54	Material 1	100	32	539.611	834.594	2093.64	918.047	1175.6
17	3.24835	7365.52	Material 1	100	32	521.754	806.976	2027.22	895.823	1131.4
18	3.24835	7154.58	Material 1	100	32	498.899	771.627	1932.91	858.083	1074.83
19	3.24835	6836.84	Material 1	100	32	470.769	728.119	1808.45	803.247	1005.2
20	3.24835	6398.96	Material 1	100	32	436.98	675.859	1650.75	729.179	921.567
21	3.24835	5822.42	Material 1	100	32	398.944	617.031	1453.89	626.469	827.424
22	3.24835	5080.47	Material 1	100	32	358.919	555.125	1208	479.649	728.351
23	3.24835	4132.2	Material 1	100	32	310.229	479.819	907.636	299.8	607.836
24	3.24835	2909.12	Material 1	100	32	251.088	388.347	537.411	75.9584	461.452
25	3.24835	1142.64	Material 1	100	32	119.158	184.297	134.904	0	134.904

Interslice Data

Global Minimum Query (bishop simplified) - Safety Factor: 1.70876

Slice Number	X coordinate [ft]	Y coordinate - Bottom [ft]	Interslice Normal Force [lbs]	Interslice Shear Force [lbs]	Interslice Force Angle [degrees]
1	125.379	2648	0	0	0

2	128.725	2646.68	431.768	0	0
3	132.07	2645.56	1102.17	0	0
4	135.415	2644.62	2034.74	0	0
5	138.761	2643.86	3204.1	0	0
6	142.106	2643.27	4497.72	0	0
7	145.452	2642.85	5820.84	0	0
8	148.797	2642.6	7093.25	0	0
9	152.143	2642.5	8260.03	0	0
10	155.488	2642.57	9312.62	0	0
11	158.833	2642.8	10201.4	0	0
12	162.179	2643.2	10885.9	0	0
13	165.524	2643.76	11334.8	0	0
14	168.87	2644.5	11525	0	0
15	172.215	2645.41	11398.1	0	0
16	175.56	2646.5	10937.2	0	0
17	178.906	2647.79	10145	0	0
18	182.251	2649.29	9034.79	0	0
19	185.597	2651	7632.24	0	0
20	188.942	2652.96	5978.74	0	0
21	192.287	2655.19	4136.03	0	0
22	195.633	2657.71	2225.21	0	0
23	198.978	2660.57	385.801	0	0
24	202.324	2663.84	-1185.84	0	0
25	205.669	2667.6	-2197.7	0	0
26	209.014	2672	0	0	0

Global Minimum Query (janbu simplified) - Safety Factor: 1.54666

Slice Number	X coordinate [ft]	Y coordinate - Bottom [ft]	Interslice Normal Force [lbs]	Interslice Shear Force [lbs]	Interslice Force Angle [degrees]
1	125.393	2648	0	0	0
2	128.642	2646.14	692.159	0	0
3	131.89	2644.55	1833.24	0	0
4	135.138	2643.21	3377.6	0	0
5	138.387	2642.09	5269.71	0	0
6	141.635	2641.19	7338.52	0	0
7	144.883	2640.49	9446.99	0	0
8	148.132	2639.98	11483.7	0	0
9	151.38	2639.67	13357	0	0
10	154.628	2639.54	15057	0	0
11	157.877	2639.6	16518	0	0
12	161.125	2639.84	17686.2	0	0
13	164.373	2640.28	18518.9	0	0
14	167.622	2640.9	18983.4	0	0
15	170.87	2641.73	19032.7	0	0
16	174.119	2642.76	18621.3	0	0
17	177.367	2644.02	17745.6	0	0

18	180.615	2645.52	16413.1	0	0
19	183.864	2647.27	14645.7	0	0
20	187.112	2649.31	12483.4	0	0
21	190.36	2651.68	9991.9	0	0
22	193.609	2654.43	7285.72	0	0
23	196.857	2657.65	4564.57	0	0
24	200.105	2661.47	2113.25	0	0
25	203.354	2666.09	442.174	0	0
26	206.602	2672	0	0	0

List Of Coordinates

Water Table

X	Y
0	2648
132.297	2648
149.78	2653.93
167.844	2655.63
191.115	2662.93
227.39	2670
279.635	2670
279.635	2665.89

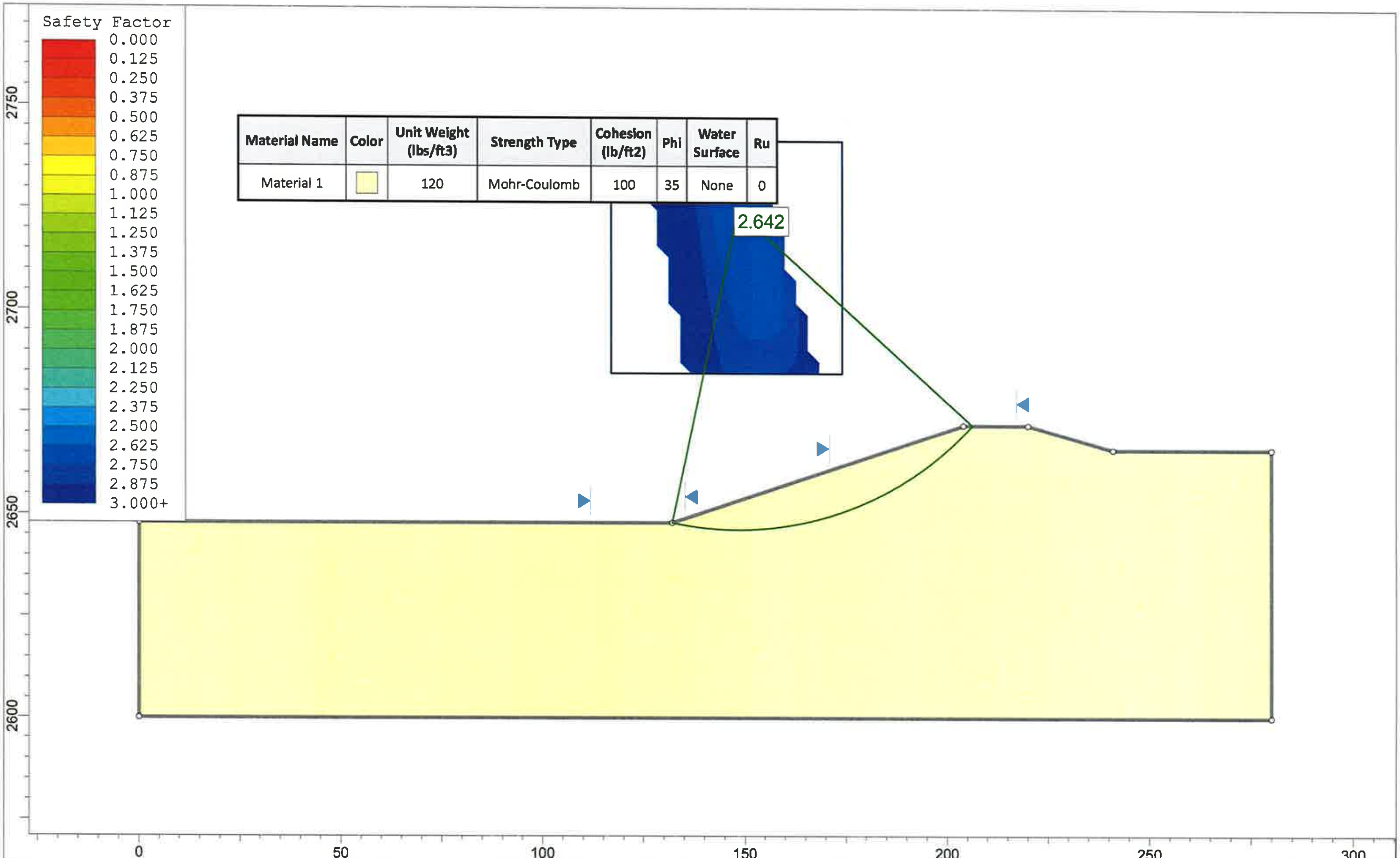
Drawdown Line

X	Y
-0.069	2647.64
132.254	2647.64
150.277	2653.8
168.301	2655.4
192.028	2662.93
227.39	2669.54
280.091	2669.54
280.091	2665.89

External Boundary

X	Y
0	2600
280	2600
280	2666
241	2666
220	2672
204	2672
132	2648

0	2648



Material Name	Color	Unit Weight (lbs/ft3)	Strength Type	Cohesion (lb/ft2)	Phi	Water Surface	Ru
Material 1		120	Mohr-Coulomb	100	35	None	0

rocscience	Project	SGPWA 10028.001		
	Analysis Description			
	Drawn By	Scale 1:393	Company	
SLIDEINTERPRET 6.008	Date	11/5/2012, 4:41:26 PM	File Name	Slide1.slim

Slide Analysis Information

SGPWA 10028.001

Project Summary

File Name: Slide1.slim
Slide Modeler Version: 6.008
Project Title: SGPWA 10028.001
Date Created: 11/5/2012, 4:41:26 PM
Comments:

Static

General Settings

Units of Measurement: Imperial Units
Time Units: days
Permeability Units: feet/second
Failure Direction: Right to Left
Data Output: Standard
Maximum Material Properties: 20
Maximum Support Properties: 20

Analysis Options

Analysis Methods Used

Bishop simplified
Janbu simplified

Number of slices: 25
Tolerance: 0.005
Maximum number of iterations: 50
Check malpha < 0.2: Yes
Initial trial value of FS: 1
Steffensen Iteration: Yes

Groundwater Analysis

Groundwater Method: Water Surfaces
Pore Fluid Unit Weight: 62.4 lbs/ft3
Advanced Groundwater Method: None

Random Numbers

Pseudo-random Seed: 10116

Random Number Generation Method: Park and Miller v.3

Surface Options

Surface Type: Circular
Search Method: Grid Search
Radius Increment: 10
Composite Surfaces: Disabled
Reverse Curvature: Create Tension Crack
Minimum Elevation: Not Defined
Minimum Depth: Not Defined

Material Properties

Property	Material 1
Color	
Strength Type	Mohr-Coulomb
Unit Weight [lbs/ft3]	120
Cohesion [psf]	100
Friction Angle [deg]	35
Water Surface	None
Ru Value	0

Global Minimums

Method: bishop simplified

FS: 2.775900
Center: 145.461, 2735.939
Radius: 88.818
Left Slip Surface Endpoint: 132.303, 2648.101
Right Slip Surface Endpoint: 207.109, 2672.000
Resisting Moment=4.52437e+006 lb-ft
Driving Moment=1.62988e+006 lb-ft

Method: janbu simplified

FS: 2.641730
Center: 148.310, 2724.542
Radius: 78.250
Left Slip Surface Endpoint: 132.019, 2648.006
Right Slip Surface Endpoint: 206.297, 2672.000
Resisting Horizontal Force=51635.8 lb
Driving Horizontal Force=19546.2 lb

Valid / Invalid Surfaces

Method: bishop simplified

Number of Valid Surfaces: 2684
Number of Invalid Surfaces: 2167

Error Codes:

Error Code -1000 reported for 2167 surfaces

Method: janbu simplified

Number of Valid Surfaces: 2684
Number of Invalid Surfaces: 2167

Error Codes:

Error Code -1000 reported for 2167 surfaces

Error Codes

The following errors were encountered during the computation:

-1000 = No valid slip surfaces are generated at a grid center. Unable to draw a surface.

Slice Data

Global Minimum Query (bishop simplified) - Safety Factor: 2.7759

Slice Number	Width [ft]	Weight [lbs]	Base Material	Base Cohesion [psf]	Base Friction Angle [degrees]	Shear Stress [psf]	Shear Strength [psf]	Base Normal Stress [psf]	Pore Pressure [psf]	Effective Normal Stress [psf]
1	2.99222	250.229	Material 1	100	35	59.0913	164.031	91.4464	0	91.4464
2	2.99222	732.222	Material 1	100	35	100.228	278.224	254.531	0	254.531
3	2.99222	1177.47	Material 1	100	35	137.505	381.701	402.311	0	402.311
4	2.99222	1586.27	Material 1	100	35	171.051	474.821	535.301	0	535.301
5	2.99222	1958.8	Material 1	100	35	200.977	557.891	653.936	0	653.936
6	2.99222	2295.1	Material 1	100	35	227.372	631.163	758.578	0	758.578
7	2.99222	2595.11	Material 1	100	35	250.313	694.845	849.526	0	849.526
8	2.99222	2858.62	Material 1	100	35	269.859	749.102	927.014	0	927.014
9	2.99222	3085.31	Material 1	100	35	286.056	794.063	991.224	0	991.224
10	2.99222	3274.71	Material 1	100	35	298.936	829.817	1042.29	0	1042.29
11	2.99222	3426.21	Material 1	100	35	308.52	856.421	1080.28	0	1080.28
12	2.99222	3539.06	Material 1	100	35	314.815	873.895	1105.24	0	1105.24
13	2.99222	3612.33	Material 1	100	35	317.817	882.228	1117.14	0	1117.14
14	2.99222	3644.9	Material 1	100	35	317.509	881.372	1115.91	0	1115.91
15	2.99222	3635.44	Material 1	100	35	313.861	871.246	1101.45	0	1101.45
16	2.99222	3582.39	Material 1	100	35	306.831	851.733	1073.59	0	1073.59
17	2.99222	3483.89	Material 1	100	35	296.364	822.676	1032.09	0	1032.09
18	2.99222	3337.77	Material 1	100	35	282.386	783.875	976.676	0	976.676
19	2.99222	3141.45	Material 1	100	35	264.811	735.088	907.001	0	907.001

20	2.99222	2891.88	Material 1	100	35	243.532	676.02	822.643	0	822.643
21	2.99222	2585.42	Material 1	100	35	218.423	606.32	723.099	0	723.099
22	2.99222	2217.72	Material 1	100	35	189.335	525.574	607.784	0	607.784
23	2.99222	1783.48	Material 1	100	35	156.092	433.297	475.997	0	475.997
24	2.99222	1275.92	Material 1	100	35	118.473	328.869	326.859	0	326.859
25	2.99222	494.723	Material 1	100	35	63.0889	175.129	107.295	0	107.295

Global Minimum Query (janbu simplified) - Safety Factor: 2.64173

Slice Number	Width [ft]	Weight [lbs]	Base Material	Base Cohesion [psf]	Base Friction Angle [degrees]	Shear Stress [psf]	Shear Strength [psf]	Base Normal Stress [psf]	Pore Pressure [psf]	Effective Normal Stress [psf]
1	2.97114	278.638	Material 1	100	35	66.0911	174.595	106.533	0	106.533
2	2.97114	814.887	Material 1	100	35	115.229	304.404	291.919	0	291.919
3	2.97114	1309.45	Material 1	100	35	159.496	421.345	458.929	0	458.929
4	2.97114	1762.96	Material 1	100	35	199.119	526.019	608.419	0	608.419
5	2.97114	2175.87	Material 1	100	35	234.287	618.923	741.099	0	741.099
6	2.97114	2548.44	Material 1	100	35	265.156	700.471	857.562	0	857.562
7	2.97114	2880.75	Material 1	100	35	291.854	770.999	958.286	0	958.286
8	2.97114	3172.7	Material 1	100	35	314.483	830.778	1043.66	0	1043.66
9	2.97114	3424.05	Material 1	100	35	333.121	880.016	1113.98	0	1113.98
10	2.97114	3634.32	Material 1	100	35	347.828	918.868	1169.46	0	1169.46
11	2.97114	3802.88	Material 1	100	35	358.643	947.437	1210.27	0	1210.27
12	2.97114	3928.89	Material 1	100	35	365.585	965.776	1236.46	0	1236.46
13	2.97114	4011.25	Material 1	100	35	368.655	973.888	1248.04	0	1248.04
14	2.97114	4048.62	Material 1	100	35	367.838	971.729	1244.96	0	1244.96
15	2.97114	4039.4	Material 1	100	35	363.097	959.204	1227.07	0	1227.07
16	2.97114	3981.59	Material 1	100	35	354.376	936.167	1194.17	0	1194.17
17	2.97114	3872.84	Material 1	100	35	341.6	902.415	1145.97	0	1145.97
18	2.97114	3710.3	Material 1	100	35	324.668	857.684	1082.08	0	1082.08
19	2.97114	3490.52	Material 1	100	35	303.454	801.644	1002.05	0	1002.05
20	2.97114	3209.32	Material 1	100	35	277.805	733.887	905.286	0	905.286
21	2.97114	2861.59	Material 1	100	35	247.535	653.92	791.08	0	791.08
22	2.97114	2440.98	Material 1	100	35	212.417	561.148	658.589	0	658.589
23	2.97114	1939.5	Material 1	100	35	172.182	454.859	506.792	0	506.792
24	2.97114	1346.91	Material 1	100	35	126.51	334.206	334.482	0	334.482
25	2.97114	544.156	Material 1	100	35	67.6498	178.713	112.413	0	112.413

Interslice Data

Global Minimum Query (bishop simplified) - Safety Factor: 2.7759

Slice Number	X coordinate [ft]	Y coordinate - Bottom [ft]	Interslice Normal Force [lbs]	Interslice Shear Force [lbs]	Interslice Force Angle [degrees]
1	132.303	2648.1	0	0	0
2	135.295	2647.7	212.883	0	0
3	138.288	2647.41	587.196	0	0

4	141.28	2647.22	1075.35	0	0
5	144.272	2647.13	1635.11	0	0
6	147.264	2647.14	2229.1	0	0
7	150.257	2647.25	2824.38	0	0
8	153.249	2647.46	3392.08	0	0
9	156.241	2647.78	3907.17	0	0
10	159.233	2648.2	4348.28	0	0
11	162.225	2648.72	4697.55	0	0
12	165.218	2649.35	4940.6	0	0
13	168.21	2650.08	5066.5	0	0
14	171.202	2650.93	5067.87	0	0
15	174.194	2651.9	4940.99	0	0
16	177.187	2652.98	4686.02	0	0
17	180.179	2654.19	4307.33	0	0
18	183.171	2655.52	3813.94	0	0
19	186.163	2657	3220.07	0	0
20	189.155	2658.61	2545.93	0	0
21	192.148	2660.38	1818.71	0	0
22	195.14	2662.31	1073.99	0	0
23	198.132	2664.42	357.489	0	0
24	201.124	2666.73	-272.369	0	0
25	204.116	2669.24	-740.913	0	0
26	207.109	2672	0	0	0

Global Minimum Query (janbu simplified) - Safety Factor: 2.64173

Slice Number	X coordinate [ft]	Y coordinate - Bottom [ft]	Interslice Normal Force [lbs]	Interslice Shear Force [lbs]	Interslice Force Angle [degrees]
1	132.019	2648.01	0	0	0
2	134.99	2647.43	257.572	0	0
3	137.961	2646.98	733.018	0	0
4	140.932	2646.64	1362.88	0	0
5	143.903	2646.42	2091.64	0	0
6	146.874	2646.3	2870.74	0	0
7	149.845	2646.31	3657.75	0	0
8	152.817	2646.42	4415.76	0	0
9	155.788	2646.65	5112.91	0	0
10	158.759	2646.99	5721.98	0	0
11	161.73	2647.45	6220.19	0	0
12	164.701	2648.03	6589	0	0
13	167.672	2648.72	6814.09	0	0
14	170.644	2649.55	6885.36	0	0
15	173.615	2650.5	6797.1	0	0
16	176.586	2651.58	6548.25	0	0
17	179.557	2652.8	6142.79	0	0
18	182.528	2654.17	5590.34	0	0
19	185.499	2655.69	4906.97	0	0

20	188.47	2657.38	4116.31	0	0
21	191.442	2659.25	3251.11	0	0
22	194.413	2661.31	2355.36	0	0
23	197.384	2663.59	1487.37	0	0
24	200.355	2666.11	724.101	0	0
25	203.326	2668.9	167.715	0	0
26	206.297	2672	0	0	0

List Of Coordinates

External Boundary

X	Y
0	2600
280	2600
280	2666
241	2666
220	2672
204	2672
132	2648
0	2648

APPENDIX E

GENERAL EARTHWORK AND GRADING SPECIFICATIONS

LEIGHTON CONSULTING, INC.

GENERAL EARTHWORK AND GRADING SPECIFICATIONS FOR ROUGH GRADING

Table of Contents

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.

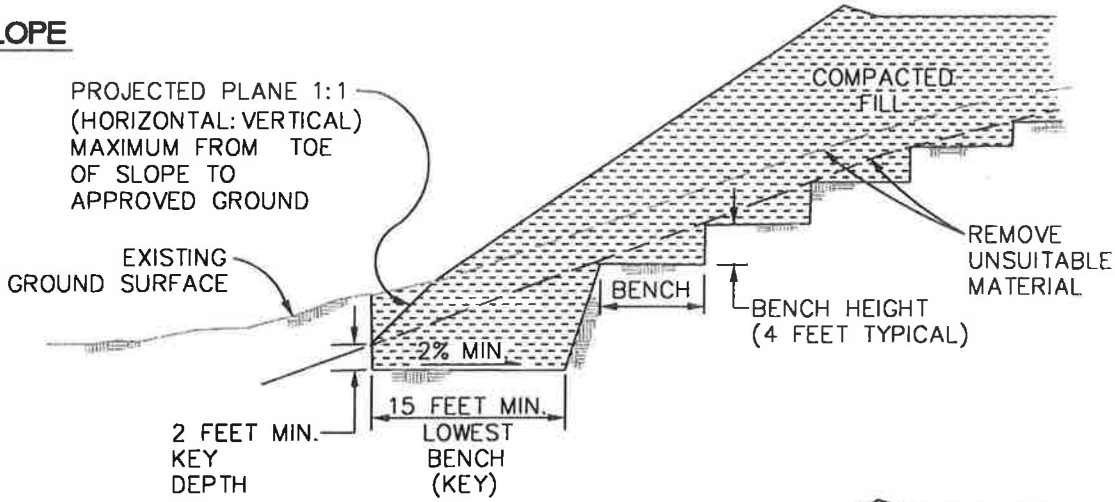
7.2 Bedding and Backfill: All bedding and backfill of utility trenches shall be done in accordance with the applicable provisions of Standard Specifications of Public Works Construction. Bedding material shall have a Sand Equivalent greater than 30 (SE>30). The bedding shall be placed to 1 foot over 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.

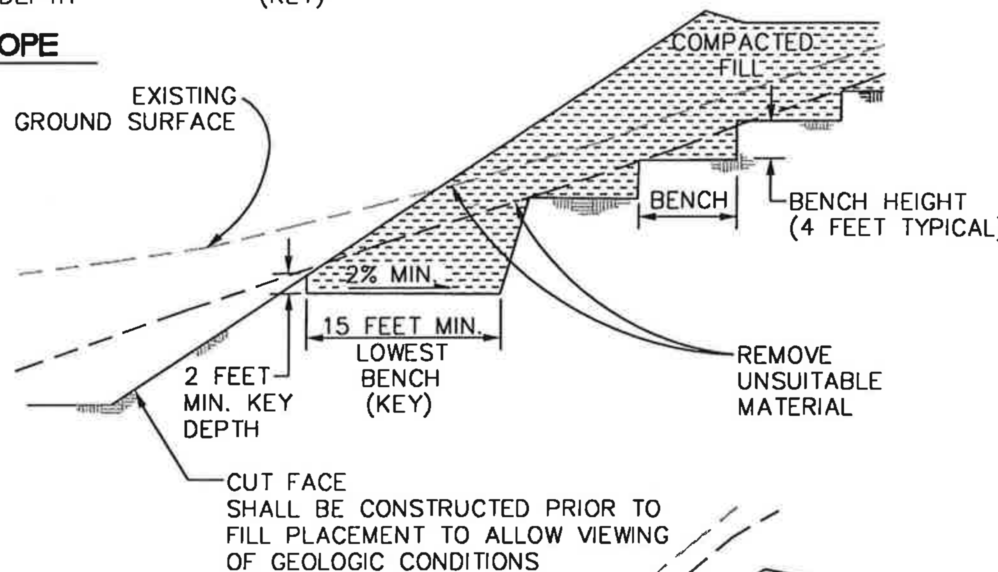
7.3 Lift Thickness: Lift thickness of trench backfill shall not exceed those allowed in the Standard Specifications of Public Works Construction unless the Contractor can demonstrate to the Geotechnical Consultant that the fill lift can be compacted to the minimum relative compaction by his alternative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

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

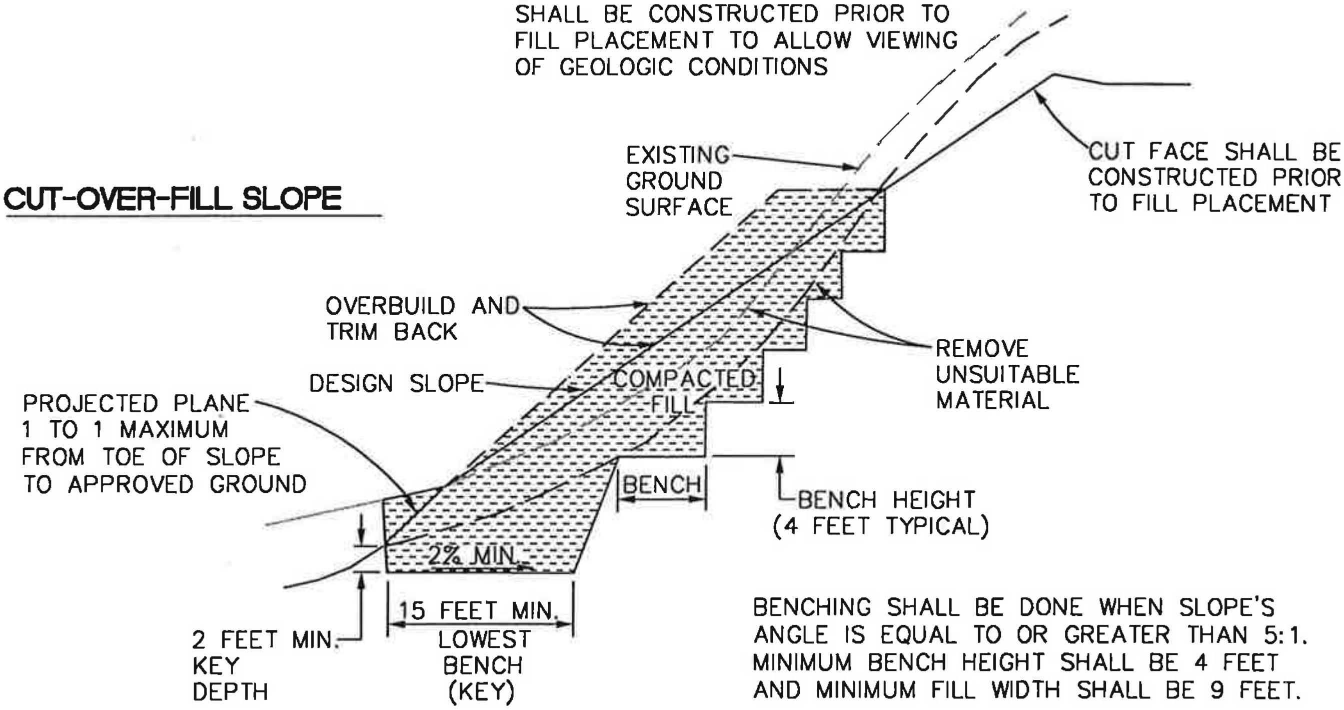
FILL SLOPE



FILL-OVER-CUT SLOPE

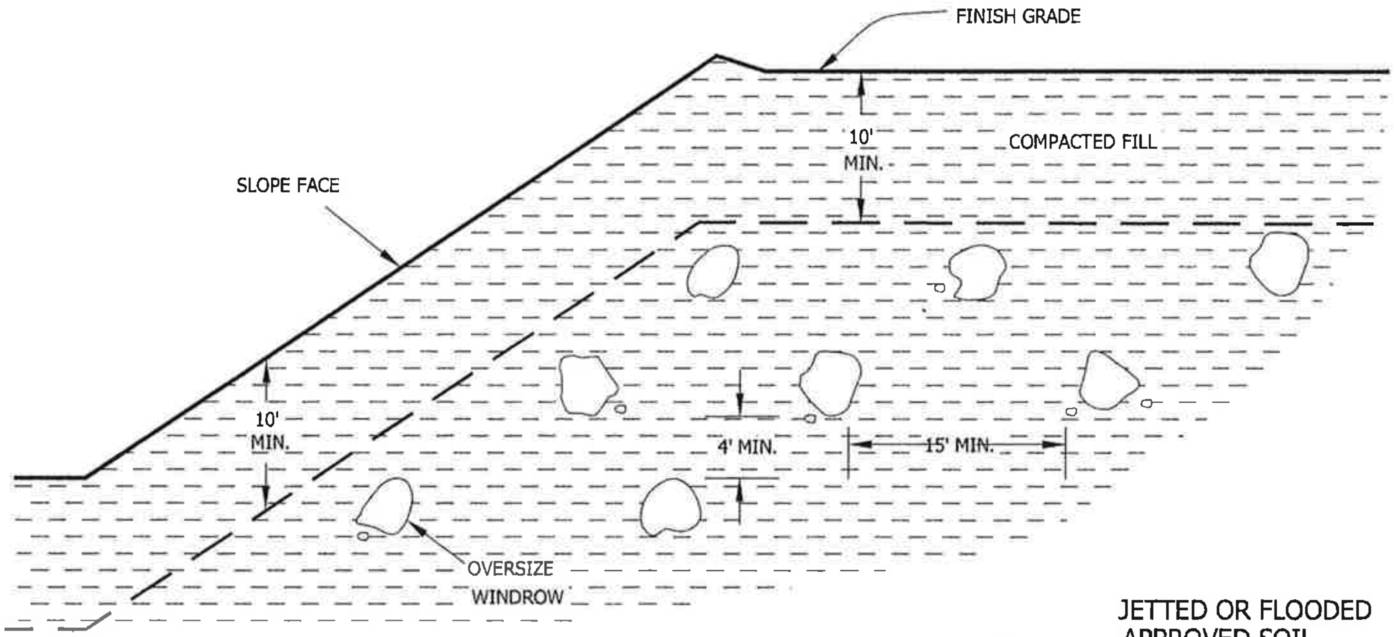


CUT-OVER-FILL SLOPE



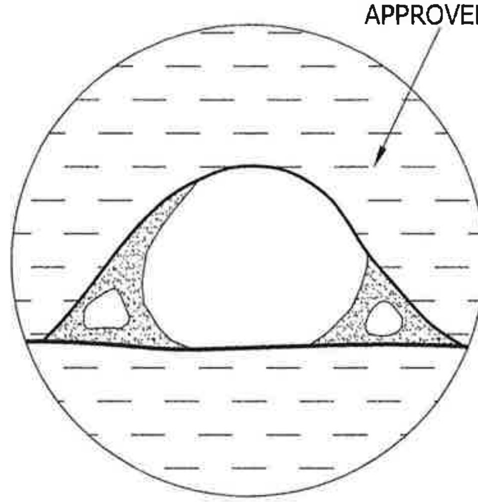
BENCHING SHALL BE DONE WHEN SLOPE'S ANGLE IS EQUAL TO OR GREATER THAN 5:1. MINIMUM BENCH HEIGHT SHALL BE 4 FEET AND MINIMUM FILL WIDTH SHALL BE 9 FEET.

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



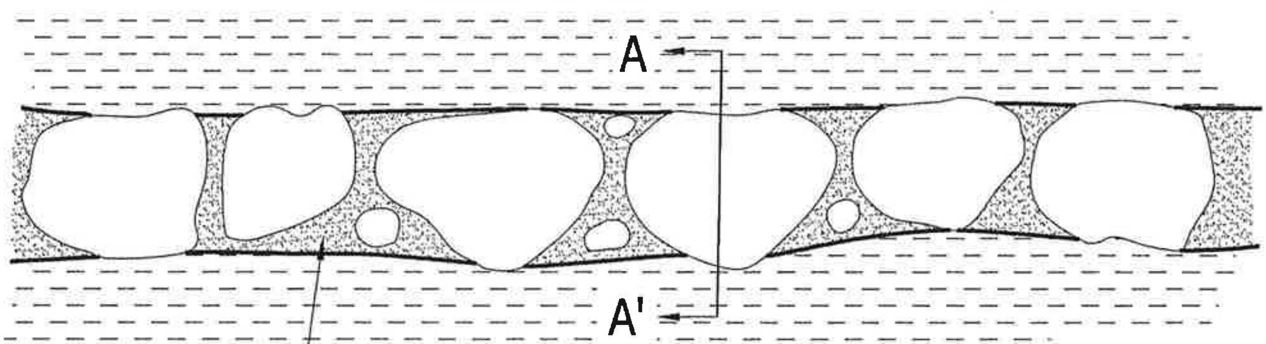
- Oversize rock is larger than 8 inches in largest dimension.
- Backfill with approved soil jetted or flooded in place to fill all the voids.
- Do not bury rock within 10 feet of finish grade.
- Windrow of buried rock shall be parallel to the finished slope face.

JETTED OR FLOODED
APPROVED SOIL



SECTION A-A'

PROFILE ALONG WINDROW



JETTED OR FLOODED
APPROVED SOIL

OVERSIZE ROCK DISPOSAL

GENERAL EARTHWORK AND GRADING
SPECIFICATIONS
STANDARD DETAILS B

A LEIGHTON GROUP COMPANY

Rev. 7/00

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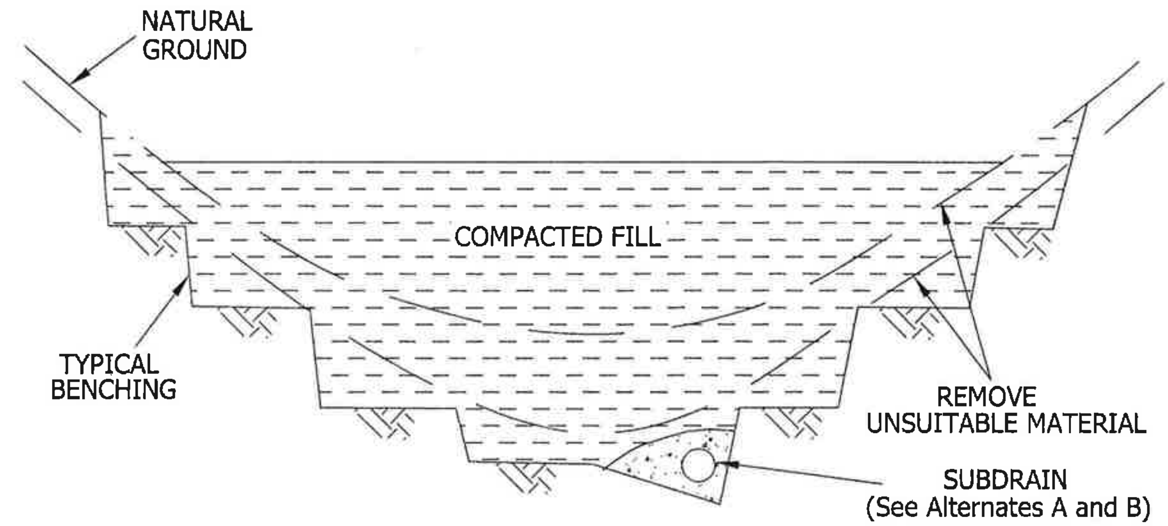
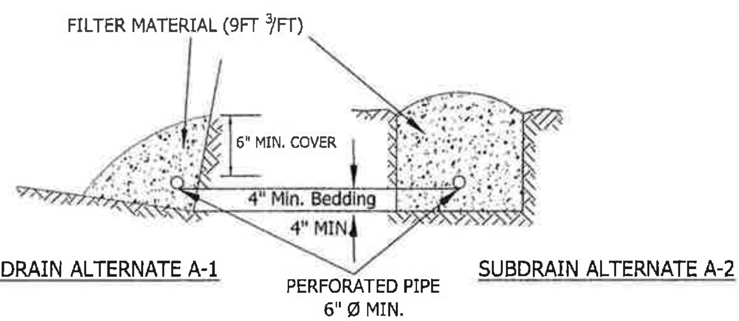


SUBDRAIN ALTERNATE A

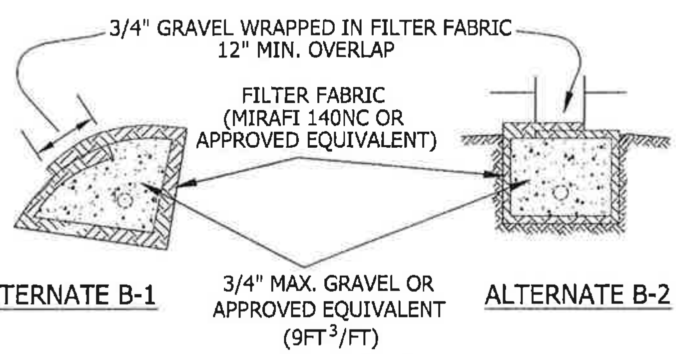


FILTER MATERIAL

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

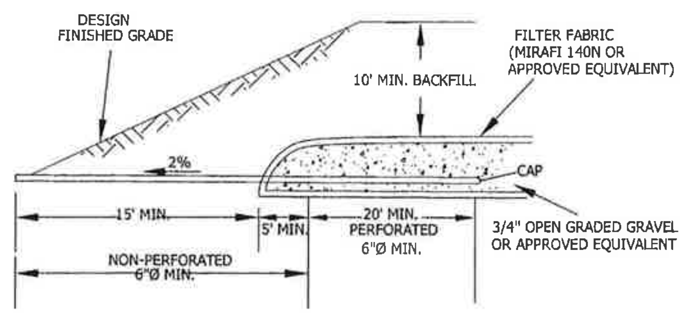
Sieve Size	Percent Passing
1"	100
3/4"	90-100
3/8"	40-100
No. 4	25-40
No. 8	18-33
No. 30	5-15
No. 50	0-7
No. 200	0-3

SUBDRAIN ALTERNATE B



○ PERFORATED PIPE IS OPTIONAL PER GOVERNING AGENCY'S REQUIREMENTS

DETAIL OF CANYON SUBDRAIN TERMINAL



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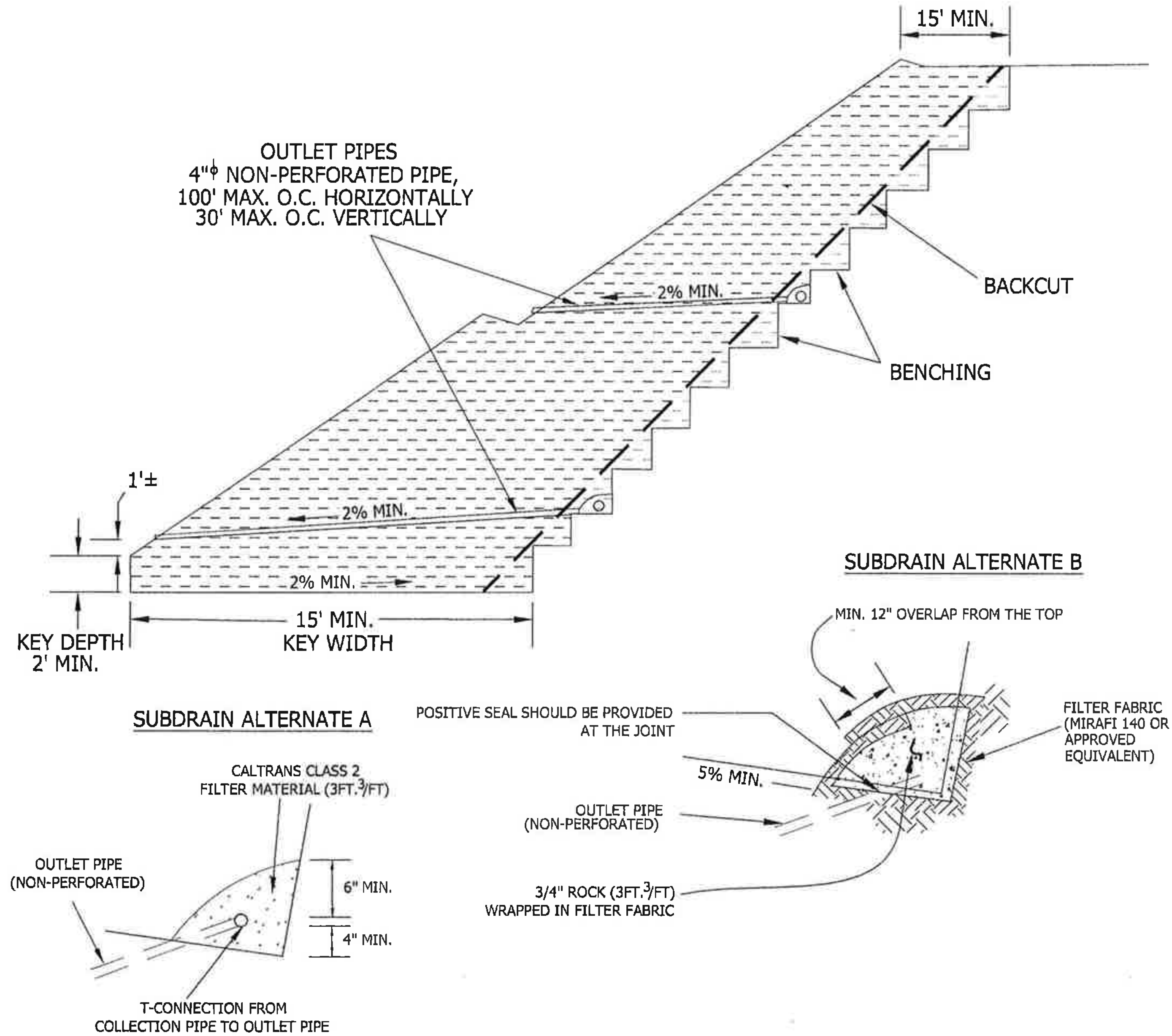
CANYON SUBDRAIN	GENERAL EARTHWORK AND GRADING SPECIFICATIONS STANDARD DETAILS C	A LEIGHTON GROUP COMPANY Rev. 7/00



- 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	A LEIGHTON GROUP COMPANY Rev. 3/01

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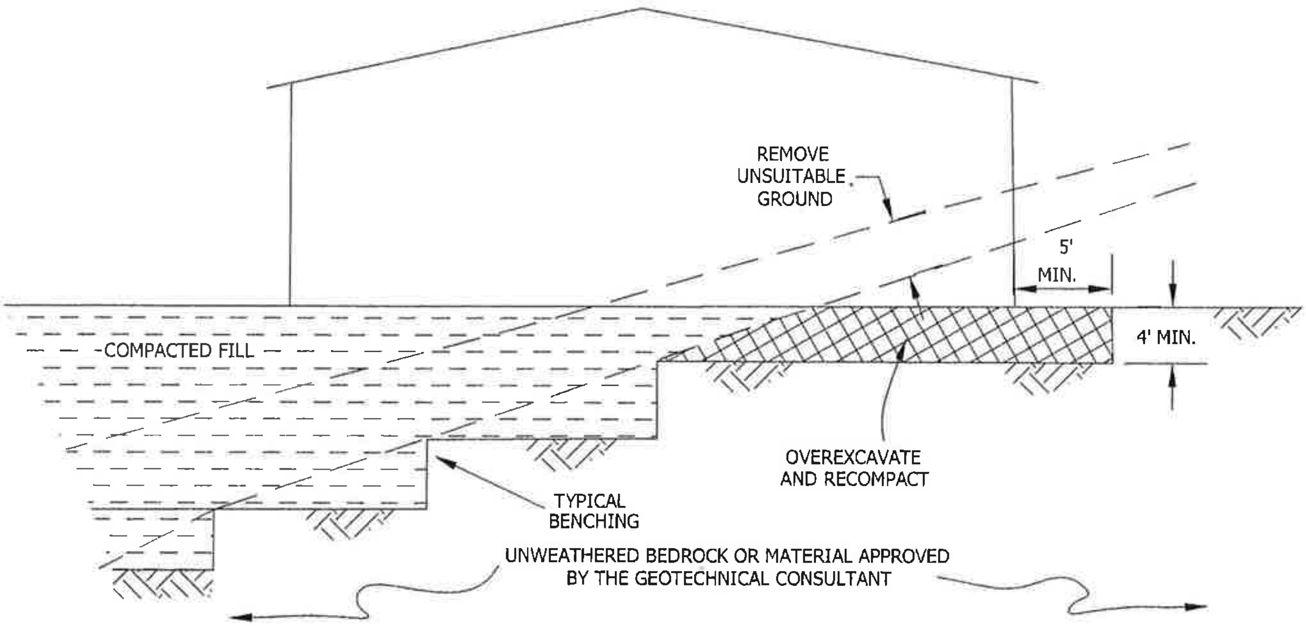
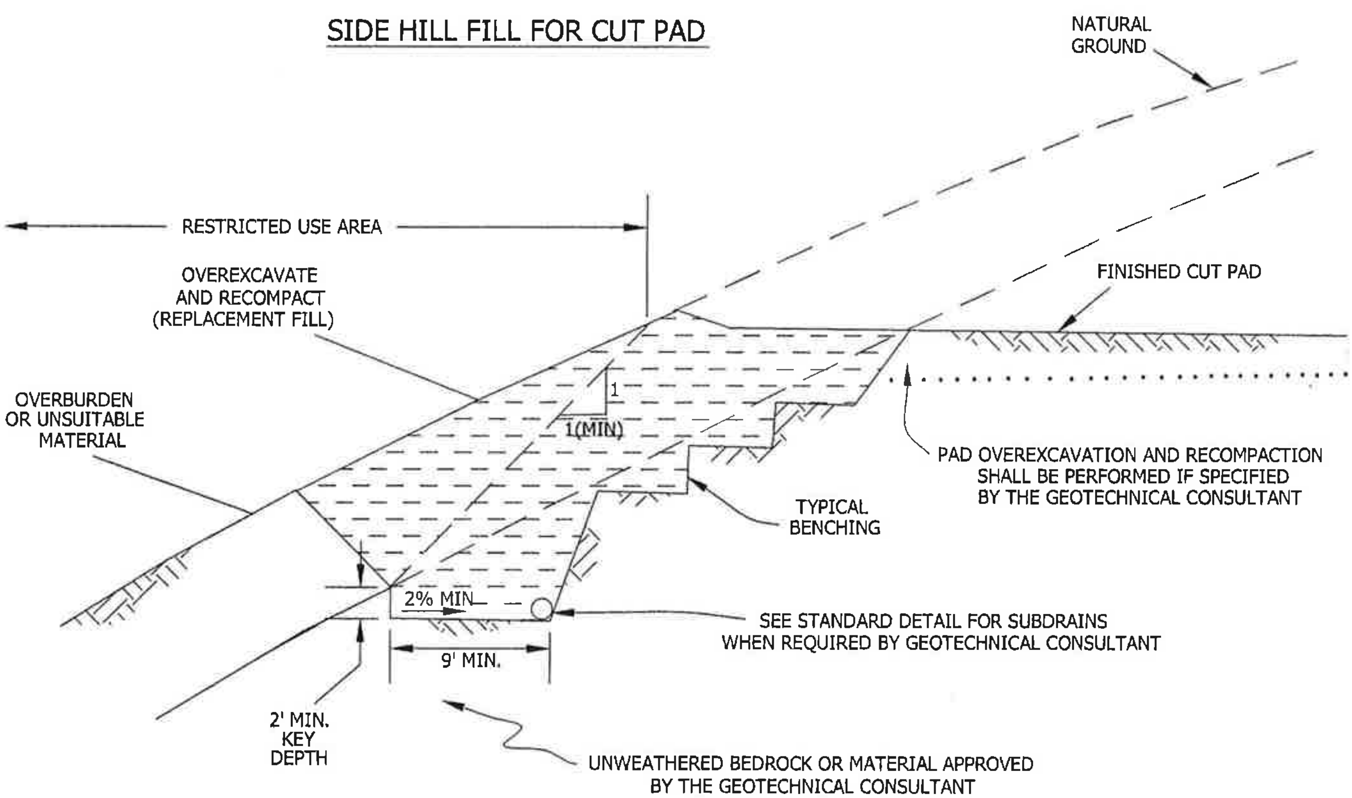
CUT-FILL TRANSITION LOT OVEREXCAVATION



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

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E.2 - Pipeline and Service Connection

GEOTECHNICAL INVESTIGATION REPORT
BROOKSIDE SOUTH STREAMBED RECHARGE PROJECT

San Gorgonio Pass Water Agency
City of Beaumont, Riverside County, California
Converse Project No. 12-81-189-01

February 12, 2013

Prepared For:

Atkins
650 East Hospitality Lane, Suite 450
San Bernardino, CA 92503

Prepared By:

Converse Consultants
10391 Corporate Drive
Redlands, California 92374

Converse Consultants
Geotechnical Engineering, Environmental and Groundwater Science, Inspection and Testing Services

February 12, 2013

Mr. Erik T. Howard, P.E., PLS
Project Manager
Atkins
650 East Hospitality Lane, Suite 450
San Bernardino, CA 92503

Subject: **GEOTECHNICAL INVESTIGATION REPORT**
BROOKSIDE SOUTH STREAMBED RECHARGE PROJECT
San Gorgonio Pass Water Agency
City of Beaumont, Riverside County, California
Converse Project No. 12-81-189-01

Dear Mr. Howard:

Converse Consultants (Converse) is pleased to submit this geotechnical investigation report for the Brookside South Streambed Recharge Project which consists of approximately 6,790 linear feet of 24-inch diameter recharge basin pipeline, located in the City of Beaumont and Riverside County, California. This report was prepared in accordance with our revised proposal dated November 20, 2012, and your Subcontract For Professional Services dated November 28, 2012.

Based on our field investigation, laboratory data and analysis, the proposed pipeline and trenchless crossings are considered feasible from a geotechnical standpoint provided recommendations presented in this report are incorporated in the design and construction.

We appreciate the opportunity to be of continued service to Atkins. If you should have any questions, please do not hesitate to contact us at (909) 796-0544.

CONVERSE CONSULTANTS

Hashmi S. E. Quazi, Ph.D., G. E., P.E.
Regional Manager/Principal Engineer

Dist: 4/Addressee
HS/SM/HSQ/kvg

PROFESSIONAL CERTIFICATION

This report has been prepared by the individual whose seals and signatures appear hereon.

The findings, recommendations, specifications or professional opinions contained in this report were prepared in accordance with generally accepted professional engineering and engineering geologic principles and practice in this area of Southern California. There is no warranty, either expressed or implied.

Harihar Shiwakoti, P.E.
Project Engineer

Scot Mathis, C.E.G
Senior Geologist

EXECUTIVE SUMMARY

The following is a summary of our geotechnical investigation, conclusions and recommendations as presented in the body of this report. Please refer to the appropriate sections of the report for complete conclusions and recommendations. In the event of a conflict between this summary and the report, or an omission in the summary, the report shall prevail.

- The proposed 24 inch diameter and approximately 6790 linear feet of pipeline alignment is located in the City of Beaumont and Riverside County, California. The proposed alignments begin at a parcel of land on the southwest corner of Orchard Street and Mountain View Avenue. The pipeline alignment traverses east along Orchard Street and south along Beaumont Avenue. The proposed alignment ends on Brookside Avenue, west of Beaumont Avenue. The project also consists of a prefabricated building at the southwest corner of Mountain Avenue and Orchard Street for housing pumping equipments.

- Open cut-and-cover technique will be utilized to install the pipe. Bore and jack techniques will be used at the locations crossing two drainage channels. It is anticipated that the invert depth will be approximately ten (10) feet below the existing ground surface (bgs) for most of the pipeline alignment, and the depths of jacking and receiving pits are anticipated to be about fifteen (15) to twenty (20) feet bgs.

- Our scope of work included the following tasks: project set-up, existing document review, subsurface exploration, laboratory testing, engineering analysis, and preparation of this report.

- Thirteen (13) borings (BH-1 through BH-13) were drilled on January 9th and 10th, 2013. The maximum explored depth was 26.5 feet bgs. Boring BH-2 was planned to be drilled to a maximum depth of 50 feet bgs, but was terminated due to auger refusal at 26.5 feet bgs.

- Based on the exploratory borings and laboratory test results, the subsurface materials along the proposed pipeline alignment and bore and jack locations predominantly consisted of sand and silty sand mixtures to the maximum explored depth of 26.5 feet bgs. The upper 10 to 15 feet consists of relatively loose to medium dense, fine to coarse grained sand and silty sand with little gravel up to 2.5 inch in diameter. Below 15 feet the soils is dense to very dense, fine to coarse grained with gravel up to 2 inch in diameter. Although not encountered during boring, based on the augur refusal on some of the borings, we anticipate that cobbles and boulders may be present.

- Groundwater was not encountered in any borings drilled to a maximum depth of 26.5 feet bgs. Based on regional information, the current and historical depths to groundwater in the area of the alignment are greater than 500 feet.

- The northern portion of the project area is within a County of Riverside fault hazard zone. There is a potential for surface fault rupture in this area. The inferred surface trace of the west-northwest-trending Beaumont Fault is located immediately north of the intersection of Orchard Street and Beaumont Avenue. The Beaumont Fault is not zoned as an active fault by the State of California; however, it is zoned as active by the County of Riverside. The County has established a fault hazard zone (Riverside County, 2013) that includes the pump site, the portion of the alignment along Orchard Street, and the portion of the alignment along Beaumont Avenue to approximately 600 feet south of Orchard Street.

- One (1) representative soil sample was tested by HDR/Schiff Associates for corrosivity evaluation with respect to common construction materials such as concrete and steel. The sulfate content of the samples tested indicated that site soils are not deleterious to concrete. Therefore, Type I or Type II Portland Cement may be used for the construction of the concrete structures. The measured values of the minimum electrical resistivity when saturated indicate that the site soil is ‘Moderately’ Corrosive’ for ferrous metals in contact with the soil. A corrosion engineer may be consulted to verify if corrosion mitigation measures for ferrous metals in contact with these soils are required.

- Based on our visual classification and sieve analysis, we anticipate major portion of the soils along the pipeline alignment and at the jacking and receiving pits have “Very Low” expansion potential.

- Based on the results of our field exploration, the subsurface soils at the proposed pipeline alignments and jacking and receiving pits should be excavatable with conventional heavy-duty excavation equipment. Caving of loose sandy soils will likely occur within the upper 10 to 15 feet bgs. Provisions for controlling raveling and running sand soils should be provided during the trenchless operation to minimize ground loss and ground subsidence.

- Earthwork for the project is expected to include sub-grade preparation for prefabricated building, trench excavation, pipe sub-grade preparation, and backfilling of the trench following the placement of the pipe. Earthwork also includes excavation, preparation of sub-grade and backfilling of jacking and receiving pits following the placement of pipe. All backfill material should be compacted to a minimum of 90 percent of the laboratory maximum dry density. The upper 1 foot of backfill beneath the pavement sections should be compacted to at least 95 percent of the laboratory maximum dry density. Moisture content of compacted soils should be kept within $\pm$ 3 percent of optimum moisture content.

- The proposed pipelines may be installed using sloped excavations or vertical excavations supported by shoring. Temporary shoring will be required where sloped excavations are not feasible due to space limitations in existing streets and/or nearby structures. Shoring will be required to hold the side walls of the jacking and receiving pits.

- Allowable net bearing capacity of 2,500 psf for natural soil may be used for anchor and thrust block design. Resistance to lateral loads and lateral bearing capacity may be provided by the passive earth pressures and frictional resistance at the base of the footing. A coefficient of friction of 0.35 between concrete and soil may be used with the dead load forces. An allowable passive earth pressure of 240 psf per foot of footing depth may be used. The passive resistance should be limited to a maximum of 2,500 psf.

- The selection of trenchless pipe crossing methods and equipment depends on pipe material, length of crossing, and anticipated ground conditions, and should be made by the contractor. We recommend that, as a minimum, the guidelines recommended in Section 306-2, "Jacking Operations," of the Standard Specifications for Public Works Construction (SSPWC), be followed.

The results of our investigation indicate that the proposed prefabricated building, and pipeline alignment including trenchless crossing are suitable from a geotechnical standpoint, provided the recommendations presented in the attached report are considered and implemented in the design and construction.

TABLE OF CONTENTS

TABLES

Page No.

FIGURES

Following Page No.

APPENDICES

1.0 INTRODUCTION

This report presents the results of our geotechnical investigation performed for the proposed Brookside South Streambed Recharge Project, located in the City of Beaumont and Riverside County, California. The approximate location of the site is shown on Figure No. 1, *Site Location Map*.

The purposes of this investigation were to determine the nature and engineering properties of the subsurface soils, and to provide design and construction recommendations for the proposed pipeline and trenchless crossing.

This report is prepared for the project described herein and is intended for use solely by Atkins and its authorized agents for design purposes. It should not be used as a bidding document but may be made available to the potential contractors for information on factual data only. For bidding purposes, the contractors should be responsible for making their own interpretation of the data contained in this report.

2.0 PROJECT DESCRIPTION

The proposed pipeline alignment is located in the City of Beaumont and Riverside County, Riverside County, California. The proposed alignment begins at a parcel of land on the southwest corner of Orchard Street and Mountain View Avenue. The pipeline alignment traverses east along Orchard Street and south along Beaumont Avenue. The proposed alignment ends on Brookside Avenue, west of Beaumont Avenue. The table below indicates the locations and lengths of the proposed pipeline segments:

Table No. 1, Street Segments and Pipe Alignment Lengths

Segment	Approximate Alignment Length (feet)
Orchard St. from Mountain View Ave. to Beaumont Ave.	1,250
Beaumont Ave. from Orchard St. to Brookside Avenue	5,350
Brookside Avenue, west of Beaumont Avenue	190
TOTAL	**6,790**

The pipe will be 24-inch in diameter and depth to pipe invert will be within 10 feet below existing ground surface. Cut and cover techniques will be used to install the pipe along most of the alignment, except across two drainage channels where a bore and jack technique will be used.

SITE LOCATION MAP

Beaumont Avenue Recharge Facility Pipeline

City of Beaumont, Riverside County, California

Client: ATKINS

Converse Consultants

Project Number	12-81-189-01
Scale	NTS
Date	February 2013
Figure No.	1

A 12’ x 12’ prefabricated building is planned at the southwest corner of Mountain Avenue and Orchard Street. The building will house pumping equipment.

3.0 SITE DESCRIPTION

Orchard Street from Mountain View Avenue to Beaumont Avenue is a wide residential street oriented in the east-west direction. There are overhead power lines on both sides of the street. Beaumont Avenue is a two-lane road with one lane in each direction. The road is oriented in a north-south direction. The road has dirt shoulders and no curb. From Orchard Street to Cherry Valley Boulevard there are businesses on the east side of the road and overhead power lines on the west side. Beaumont Avenue has tall trees along both sides of the road from Cherry Valley Boulevard to Brookside Avenue. In some areas the trees overhang into the road. There is also a landscaped area beyond the dirt shoulders. Brookside Avenue is also a two lane road, oriented in the east-west direction.

4.0 SCOPE OF WORK

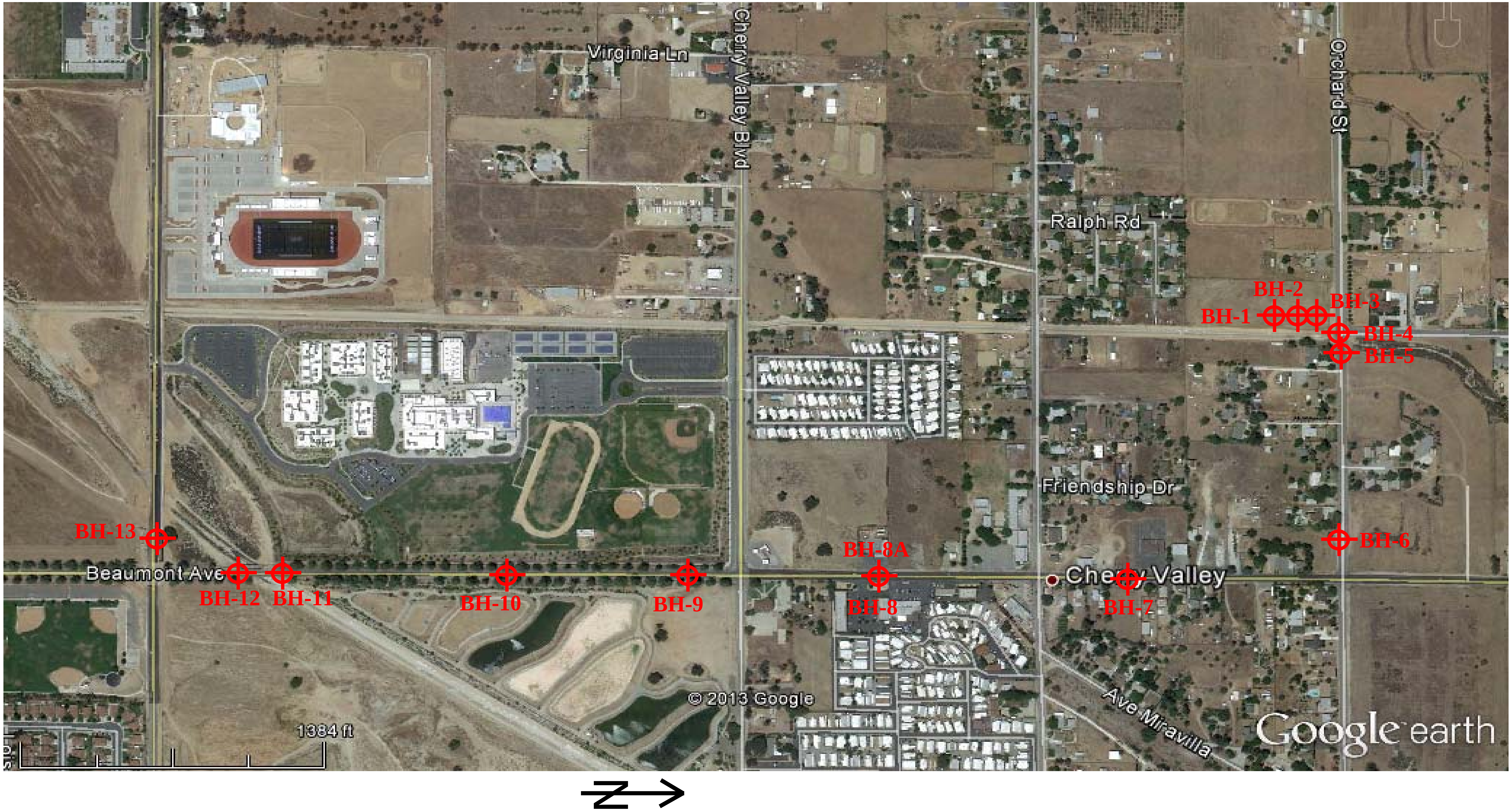
The scope of this investigation included the following tasks:

4.1 Project Set-up

- Conducted a site reconnaissance and staked/marked the boring locations along the pipe alignments, such that drill rig access to all the locations is available
- Obtained encroachment permits from the City of Beaumont Public Works Department, and the County of Riverside Transportation Department.
- Notified underground Service Alert (USA) at least 48 hours prior to drilling to clear the boring locations of any conflict with existing underground utilities

4.2 Subsurface Exploration

Thirteen (13) borings (BH-1 through BH-13) were drilled on January 9 and 10, 2013 at the locations indicated on the Beaumont Avenue Recharge Facility Pipeline 2013 plans and profile prepared by Atkins for the San Gorgonio Pass Water Agency. The approximate location and maximum depths of the borings drilled are presented in Table No. 2, *Approximate Boring Locations and Maximum Depths*. Approximate boring locations are presented in Figure No. 2, *Approximate Boring Location Map.*



APPROXIMATE BORING LOCATION MAP

Explanation	Beaumont Avenue Recharge Facility Pipeline	Project Number 12-81-189-01
Approximate Boring Location	City of Beaumont, Riverside County, California	Scale NTS
	Client: ATKINS	Date Febuary 2013
	Converse Consultants	Figure No. **2**

Borings BH-2 and BH-6 were terminated before reaching the planned depths of 50 feet and 15 feet respectively due to auger refusal. Boring BH-8 was drilled on the street shoulder. Boring BH-8A was re-drilled on the pipeline alignment (on Beaumont Avenue) to verify the soils profile obtained on boring BH-8.

Table No. 2, Boring Locations and Depths

Boring Number/ Depth (feet)	Area/Street Name	Station Number	Remarks
BH-1/16.5	Southwest corner of Orchard Street and Mountain Avenue	10+00	12' by 12' prefabricated building
BH-2/25.0		10+00	12' by 12' prefabricated building
BH-3/16.5		10+00	Pipeline
BH-4/26.4	Orchard Street	10+60	Pipeline + Bore & Jack
BH-5/26.5		10+95	Pipeline + Bore & Jack
BH-6/4.5	Beaumont Avenue	19+90	Pipeline
BH-7/16.5		31+00	Pipeline
BH-8/16.5		41+00 on shoulder	Pipeline
BH-8A/16.5		41+00 on alignment	Pipeline
BH-9/15.5		51+00	Pipeline
BH-10/16.5		61+00	Pipeline
BH-11/26.5		69+25	Pipeline + Bore & Jack
BH-12/26.5		71+00	Pipeline + Bore & Jack
BH-13/16.5		76+00	Pipeline

The borings were drilled using a truck-mounted drill rig equipped with 8-inch diameter hollow-stem augers for soil sampling. The borings were visually logged by our engineer and sampled at regular intervals and at changes in subsurface soils. Relatively undisturbed and bulk soil samples were obtained for laboratory testing. The borings were backfilled with soil cuttings at the completion of drilling. In paved areas, the surface was patched with cold asphalt concrete.

For a description of the field exploration and sampling program see Appendix A, *Field Exploration*.

4.3 Laboratory Testing

Representative samples of the site soils were tested in the laboratory to aid in the soils classification and to evaluate the relevant engineering properties of the site soils. These tests included:

- *In situ* moisture content and dry density (ASTM Standard D2216)
- Collapse (ASTM Standard D5333)
- Sand equivalent (ASTM Standard D2419)
- Soil corrosivity tests (Caltrans 643, 422, 417, and 532)
- Grain size analysis (ASTM Standard D422)
- Maximum dry density and optimum moisture content (ASTM Standard D1557)
- Direct shear (ASTM Standard D3080)

For in situ moisture and dry density data, see the Logs of Borings in Appendix A, Field Exploration. For a description of the laboratory test methods and test results, see Appendix B, *Laboratory Testing Program.*

4.4 Analysis and Report Preparation

Data obtained from the field exploration and laboratory testing program were compiled and evaluated. Geotechnical analyses of the compiled data were performed and this report was prepared to present our findings, conclusions and recommendations for the proposed pipeline and trenchless crossings.

5.0 SUBSURFACE CONDITIONS

A general description of the subsurface conditions and various materials encountered during our field exploration along the alignments are presented in this section.

5.1 Regional Geology

The project alignment is located in the northernmost portion of the Peninsular Ranges Geomorphic Province of Southern California. The Peninsular Ranges Geomorphic Province consists of a series of northwest-trending mountain ranges and valleys bounded on the north by the San Bernardino and San Gabriel Mountains, on the west by the Los Angeles Basin, and on the south by the Pacific Ocean.

The province is a seismically active region characterized by a series of northwest-trending strike-slip faults. The most prominent of the nearby fault zones include the San Jacinto, Cucamonga, and San Andreas Fault Zones, all of which have been known to be active during Quaternary time.

Topography within the province is generally characterized by broad alluvial valleys separated by linear mountain ranges. This northwest-trending linear fabric is created by the regional faulting within the granitic basement rock of the Southern California Batholith. Broad, linear, alluvial valleys have been formed by erosion of these principally granitic mountain ranges.

5.2 Site Geology

The project alignment is located on a south-sloping Pleistocene alluvial fan composed of material derived from the San Bernardino Mountains, located to the north. The fan is formed of weakly indurated sand and gravel (Dibblee, 2003). Relatively thin, unconsolidated deposits of Holocene alluvium, colluvium, or other surficial soils may mantle the denser Pleistocene deposits.

The fan surface has been dissected by active drainage channels, including Little San Gorgonio Creek, which crosses the project alignment south of Vineland Street, and Nobel Creek, which crosses the alignment north of Brookside Avenue.

The northern end of the alignment is adjacent to the surface trace of the Banning Fault. Faulting is discussed in Section 7.0, *Faulting and Seismicity*.

5.3 Existing Pavement Thickness

The thicknesses of the existing asphalt concrete pavement and aggregate base, as observed in the soil borings, are provided in Table No. 3, *Approximate Pavement Thickness*.

Table No. 3, Approximate Pavement Thickness

Boring Number/ Depth (feet)	Area/Street Name	Station Number	Asphalt Concrete (Inches)	Aggregate Base (Inches)
BH-1/15	Southwest corner of Orchard Street and Mountain Avenue	10+00	NA	NA
BH-2/50		10+00	NA	NA
BH-3/15		10+00	NA	NA
BH-4/25	Orchard Street	10+60	4.0	3.0
BH-5/25		10+95	6.0	4.0

Boring Number/ Depth (feet)	Area/Street Name	Station Number	Asphalt Concrete (Inches)	Aggregate Base (Inches)
BH-6/15	Beaumont Avenue	19+90	4.0	0.0
BH-7/15		31+00	10.0	0.0
BH-8/15		41+00 (shoulder)	NA	NA
BH-8A/15		41+00 (street)	7.0	0.0
BH-9/15		51+00	8.0	0.0
BH-10/15		61+00	5.0	0.0
BH-11/25		69+25	6.0	0.0
BH-12/25		71+00	6.0	0.0
BH-13/15		76+00	4.0	11.0

5.4 Subsurface Profile

Based on the exploratory borings and laboratory test results, the subsurface materials along the proposed pipeline alignment and bore and jack locations predominantly consisted of sand and silty sand mixtures to the maximum explored depth of 26.5 feet below ground surface (bgs). The upper 10 to 15 feet bgs consists of relatively loose to medium dense, fine to coarse grained sand and silty sand with scattered gravel up to 2.5 inches in diameter. Below 15 feet bgs the soils is dense to very dense, fine to coarse grained with gravel up to 2 inches in diameter.

Auger refusal in borings BH-2 and BH-6 indicates that cobbles or boulders may be present in some locations. Layers with high percentages of gravel could also cause auger refusal, and may be present locally.

5.5 Groundwater

Groundwater was not encountered in any borings drilled to a maximum depth of 26.5 feet bgs. Regional groundwater data (USGS, 2013) was reviewed to evaluate the current and historical depth to groundwater. A well (USGS 335807116582201) located approximately 0.25 miles east of the central portion of the alignment was monitored from 1991 to 2012. The depth to groundwater during that time ranged from approximately 530 to 610 feet bgs, with the most recent measurements approximately 565 feet bgs.

Several wells located 0.25 to 0.5 miles north of the site contained groundwater as shallow as approximately 50 feet bgs within the past several years. All of the wells with groundwater reporting shallow groundwater are located north of the Beaumont Fault. It is likely that the fault acts as a groundwater barrier, resulting in an accumulation of groundwater on the north side.

Groundwater is not generally expected to be encountered during the construction for the majority of this project. The Orchard Street segment of the project alignment is close to the Beaumont Fault. Although shallow groundwater is not generally anticipated, it is possible that groundwater may be encountered near the intersection of Orchard Street and Beaumont Avenue. Shallow zones of perched groundwater may also be encountered locally.

It should be noted that the depth to groundwater could vary depending upon the season, precipitation, and possible groundwater pumping activity in the vicinity of proposed alignments.

5.6 Excavatability

Based on the results of our field exploration, the subsurface soils at the proposed pipeline alignments and jacking and receiving pits should be excavatable with conventional heavy-duty excavation equipment. Caving of loose sandy soils may occur on the upper 10 to 15 feet bgs.

5.7 Subsurface Variations

Based on results of the subsurface exploration and our experience, some variations in the continuity and nature of subsurface conditions within the project site should be anticipated. Because of the uncertainties involved in the nature and depositional characteristics of the earth material, care should be exercised in interpolating or extrapolating subsurface conditions between or beyond the boring locations.

For a detailed description of the subsurface materials encountered in the exploratory borings, see Drawing Nos. A-2 through A-14, *Logs of Borings,* in Appendix A, *Field Exploration.*

6.0 LABORATORY TEST RESULTS

Physical and chemical tests conducted on this project is discussed below.

6.1 Physical Testing

Laboratory testing was performed to determine the physical characteristics and engineering properties of the subsurface soils. Results of *in-situ* moisture and dry density tests are presented on the Logs of Borings in Appendix A, *Field Exploration.* Tests results are included in Appendix B, *Laboratory Testing Program*. Discussions of the various test results are presented below:

- *In-situ* Moisture and Dry Density – *In-situ* dry density of the soils within upper 15 feet bgs along the pipeline alignment ranged from 93 to 129 pounds per cubic feet (pcf) with the moisture content varying from one (1) to ten (10) percent. *In-situ* dry density of the soils within upper twenty five feet at the jack and bore locations ranged from 99 to 134 pounds per cubic feet (pcf) with the moisture content varying from two (2) to nine (9) percent.

- Swell/Collapse – Two (2) representative soil samples were tested to evaluate collapse potential in accordance to ASTM Standard D5333. The test results indicated negligible collapse potential.

- Sand Equivalent – Two (2) representative bulk soil samples were tested to evaluate Sand Equivalent (SE) in accordance with the ASTM D2419 test method. The measured SE of the soil samples were 20 and 76.

- Grain Size Analysis – Three (3) representative samples were tested to determine the relative grain size distribution in accordance with the ASTM Standard D422. Test results are graphically presented in Drawing No. B-1, *Grain Size Distribution Results.*

- Maximum Dry Density and Optimum Moisture Content – Results of two (2) typical moisture-density relationships of representative soil samples tested according to ASTM Standard D1557 are presented in Drawing No. B-2, *Moisture-Density Relationship Results*, in Appendix B, *Laboratory Testing Program.* The laboratory maximum dry densities were 128.6 and 132.7 pounds per cubic feet (pcf) and optimum moisture contents were 9.4 and 5.6 percent, respectively.

- Direct Shear – Three (3) direct shear tests were performed in accordance with ASTM Standard D3080 on relatively undisturbed ring samples. Result of the direct shear tests are presented in Drawings No. B-3 through B-5, *Direct Shear Test Results* in Appendix B, *Laboratory Testing Program*. Results of direct shear tests indicate that the soils tested had moderate shear strength.

6.2 Chemical Testing - Corrosivity Evaluation

One (1) representative soil sample was tested by HDR/Schiff Associates for corrosivity evaluation with respect to common construction materials such as concrete and steel. The test results are discussed below and are presented in Appendix B, *Laboratory Testing Program.* The test includes pH, sulfate and chloride content, and saturated minimum electrical resistivity.

The sulfate content of the sample tested was 12 mg/kg, which indicated that site soils are not deleterious to concrete. Based on this result, Type I or Type II Portland Cement may be used for the construction of the concrete structures.

The chloride concentration of sample tested ranged was 5.1 mg/kg. The pH value of the site soil was 6.9. The measured value of the minimum electrical resistivity when saturated was 9,600 Ohm-cm. The measured values of the minimum electrical resistivity when saturated indicate that the site soil is 'Moderately' Corrosive' for ferrous metals in contact with the soil. A corrosion engineer may be consulted to verify if corrosion mitigation measures for ferrous metals in contact with these soils are required.

7.0 FAULTING AND SEISMICITY

7.1 Faulting

An active fault is defined as the one that has had surface displacement within Holocene time (about the last 11,000 years). The site is not situated within a currently designated State of California Earthquake Fault Zone (CGS, 1995); however, a portion of the site is within a County of Riverside Fault Zone (Riverside County, 2013).

The inferred surface trace of the west-northwest-trending Beaumont Fault is located immediately north of the intersection of Orchard Street and Beaumont Avenue. The Beaumont Fault is not zoned as an active fault by the State of California; however, it is zoned as active by the County of Riverside. The County has established a fault hazard zone (Riverside County, 2013) that includes the pump site, the portion of the alignment along Orchard Street, and the portion of the alignment along Beaumont Avenue to approximately 600 feet south of Orchard Street.

The County of Riverside has also established several northwest-trending fault hazard zones to the southwest of the site. The closest of these zones is approximately 0.3 miles southwest of the southern end of the alignment. These fault zones do not impact the project alignment.

Based on review of regional geologic mapping (CGS, 1995; Dibblee, 2003; Riverside County, 2013) no other known active faults project toward or extend across the project site.

There are a number of nearby faults, which could produce significant ground shaking at the site during a major earthquake. The closest known active fault is the San Andreas-Southern Segment Fault Zone. The following table summarizes faults considered active by the State of California and located within 100 kilometers of the project alignment (Blake, 2000; Cao, 2003).

Table No. 4, Seismic Characteristics of Nearby Active Faults

Fault Name and Section	Approximate Distance to Site (kilometers)	Max. Moment Magnitude (Mw)
San Andreas – San Bernardino	8.5	7.5
San Jacinto – San Jacinto Valley	13.6	6.9
San Jacinto – San Bernardino	24.5	6.7
Pinto Mountain	25.4	7.2
San Jacinto - Anza	26.3	7.2
North Frontal Fault Zone (West)	33.1	7.2
North Frontal Fault Zone (East)	34.4	6.7
Cleghorn	40.5	6.5
Helendale – S. Lockhardt	45.5	7.3
San Andreas - Coachella	47.1	7.2
Cucamonga	48.4	6.9
Elsinore – Glen Ivy	50.0	6.8
Elsinore - Temecula	50.0	6.8
Lenwood – Lockhardt –Old Woman Springs	53.0	7.5
Burnt Mtn.	53.9	6.5
Landers	55.9	7.3
Eureka Peak	56.4	6.4
Chino-Central Ave. (Elsinore)	56.8	6.7
Whittier	62.2	6.7
Johnson Valley (Northern)	62.3	6.7
San Andreas - Mojave	63.4	7.4
San Andreas – 1857 Rupture	63.4	7.4
Elsinore - Julian	66.1	7.1
San Jose	67.6	6.4
Emerson So. – Copper Mtn.	70.8	7.0
San Jacinto – Coyote Creek	71.7	6.8
Sierra Madre	72.1	7.2

Fault Name and Section	Approximate Distance to Site (kilometers)	Max. Moment Magnitude (Mw)
Calico - Hidalgo	78.7	7.3
Elysian Park Thrust	80.4	6.7
Clamshell - Sawpit	85.5	6.5
Pisgah – Bullion Mtn. – Mesquite Lake	86.4	7.3
Newport – Inglewood (Offshore)	90.9	7.1
Compton Thrust	94.4	6.8
Earthquake Valley	95.0	6.5
Newport – Inglewood (L.A. Basin)	96.3	7.1
Raymond	97.3	6.5

7.2 CBC (2010) Seismic Design Coefficients

Seismic parameters based on the 2010 California Building Code (CBC, 2010) are provided in the following table.

Table No. 5, CBC Seismic Design Parameters

Seismic Parameters	
Site Coordinates	33.9709°N 116.9773°W
Site Class	"D"
Mapped Short period (0.2-sec) Spectral Response Acceleration, S_s	1.500g
Mapped 1-second Spectral Response Acceleration, S_1	0.602g
Site Coefficient (from Table 1613.5.3(1)), F_a	1.0
Site Coefficient (from Table 1613.5.3(2)), F_v	1.5
Design Spectral Response Acceleration for short period, S_{ds}	1.000g
Design Spectral Response Acceleration for 1-second period, S_{d1}	0.602g

7.3 Secondary Effects of Seismic Activity

Buried pipelines are subjected to dynamic stresses due to ground acceleration during earthquake events. A seismic event may also affect buried pipelines from ground surface rupture, soil liquefaction, landslides, lateral spreading, differential settlement due to seismic shaking, and earthquake-induced flooding. A discussion on a site-specific evaluation of each of these seismic effects is presented below:

Surface Fault Rupture: The proposed prefabricated building and pipeline alignment is not located within a currently designated State of California Earthquake Fault Zone (CGS, 2007); however, the pump equipment building and the northern portion of the pipeline alignment are within a County of Riverside fault hazard zone. There is a potential for a surface rupture in this area. The potential for surface fault rupture in the other portions of the alignment cannot be known with certainty, but is considered low.

Soil Liquefaction: Liquefaction is defined as the phenomenon in which a cohesionless soil mass within about the upper 50 feet of the ground surface suffers a substantial reduction in its shear strength, due the development of excess pore pressures. During earthquakes, excess pore pressures in saturated soil deposits may develop as a result of induced cyclic shear stresses, resulting in liquefaction.

Soil liquefaction occurs in submerged granular soils during or after strong ground shaking. There are several requirements for liquefaction to occur. They are as follows:

- Soils must be submerged
- Soils must be primarily granular
- Soils must be loose to medium-dense
- Ground motion must be intense
- Duration of shaking must be sufficient for the soils to lose shear resistance

The site is located within an area designated by the County of Riverside as being susceptible to liquefaction (Riverside County, 2013). Because the current and historical depths to groundwater in the area of the alignment are greater than 50 feet, the potential for liquefaction is considered to be very low.

Landslides: Seismically induced landslides and other slope failures are common occurrences during or soon after earthquakes. The ground along the recycled water pipelines alignments is relatively flat. The project site has a low potential for seismically induced landslides affecting the area.

Lateral Spreading: Seismically induced lateral spreading involves lateral movement of earth materials due to ground shaking. It differs from a slope failure in that ground failure involving a large movement does not occur due to the flatter slope of the initial ground surface. Lateral spreading is characterized by near-vertical cracks with predominantly horizontal movement of the soil mass involved over the liquefied soils. The potential for lateral spreading along the alignments is considered low.

Earthquake-Induced Flooding: The failure of dams or other water-retaining structures as a result of earthquakes may result in downstream flooding. Portions of the site are within flood zones associated with Little San Gorgonio Creek and Noble Creek (Riverside County, 2013). Because these creeks do not contain any dams, the potential for flooding of the alignment due to earthquake-induced dam failure is considered to be low.

Tsunamis: Tsunamis are large waves generated in open bodies of water by fault displacement or major ground movement. Based on the inland location of the site, tsunamis do not pose a hazard.

Seiches: Seiches are large waves generated in enclosed bodies of water in response to ground shaking. Because the site is not located adjacent to significant bodies of water, the potential for seiche-related flooding is considered to be low.

8.0 EARTHWORK RECOMMENDATIONS

8.1 General

Earthwork for the project will include trench excavation, pipe sub-grade preparation and backfilling the trench following the placement of the pipe segments. Earthwork includes excavation, sub-grade preparation and backfilling for the jacking and receiving pits following the placement of the pipe segments. Earthwork also includes sub-grade preparation for the foundation of prefabricated building to house pumping equipments.

Prior to the start of construction, all existing underground utilities should be located along the pipeline alignment and jack and bore locations. Such utilities should either be protected in-place or removed and replaced during construction as required by the project specifications.

Deleterious material, including organics, asphalt, and debris generated during excavation should not be placed as backfill. It is our understanding, after pipe installation, the pavement will be placed to match the existing pavement thickness.

Migration of fines from the surrounding native soils, in the case of water leaks from the pipe, must be considered in selecting the gradation of the materials placed within the trench, including bedding, pipe zone and trench zone backfill, as defined in the following sections. Such migration of fines may deteriorate pipe support and may result in settlement/ground loss at the surface.

Based on our visual classification and sieve analysis, we anticipate that the major portion of the soils along the pipeline alignment and at the jacking and receiving pits have "Very Low" expansion potential.

8.2 Excavation for At-Grade Lightweight Structures

The 12-foot by 12-foot prefabricated building proposed for the southwest corner of Orchard Street and Mountain View Avenue should be founded on dense, stable soils. The upper twelve inches of soils below the bottom of the proposed footing sub-grade should be scarified and recompacted to a minimum of 90 percent of laboratory maximum dry density and within ± three (3) percent of optimum moisture density. Such scarification and recompaction should extend horizontally outside the structure footprint to a distance of at least three (3) feet.

8.3 Jack-and-Bore Recommendations

Recommendations pertaining to the jack-and-bore sections of the alignment are presented in the following subsections.

8.3.1 Ground Classification

The Tunnelman's Ground Classification categorizes predictive soil behaviors for saturated and unsaturated conditions as presented in Table No.6, *Tunnelman's Ground Classification for Soil*. .

Table No. 6, Tunnelman's Ground Classification for Soil

Ground Classifications	Ground Behavior	Typical Soil Types
Hard	Tunnel heading may be advanced without roof support.	Cemented sand and gravel and over-consolidated clay above the ground water table.
Firm	Ground in which a roof section of a tunnel can be left unsupported for several days without inducing a perceptible movement of the ground.	Loess above water table, hard clay, marl, cemented sand and gravel when not highly overstressed.
Raveling	Chunks or flakes of soil begin to drop out of roof at some point during the ground movement period.	Residual soils or soil with clay binder may be fast raveling below ground water table and slow raveling above ground water table. Stiff fissured clays may be slow raveling or fast raveling depending on the degree of overstress.

Ground Classifications	Ground Behavior	Typical Soil Types
Slow Raveling	The time required to excavate 5 feet of tunnel and install a rib set and lagging in a small tunnel is about 6 hours. Therefore, if the stand-up time of raveling ground is more than 6 hours, using ribs and lagging, such as soil would be classified as slow raveling.	
Fast Raveling	If the stand-up time is less than 6 hours, using ribs and lagging, such soil would be classified as fast raveling.	
Squeezing	Ground slowly advances into tunnel without any signs of fracturing. The loss of ground caused by squeeze and the resulting settlement of the ground surface can be substantial.	Ground with low frictional strength. Rate of squeeze depends on degree of overstress. Stiff to hard clay under high cover may move in combination of raveling at execution surface and squeezing at depths.
Swelling	Ground slowly advances into tunnel partly or chiefly because of an increase in the volume of the ground. The volume increase is in response to an increase of water content. In every other respect, swelling ground in a tunnel behaves like a stiff non-squeezing, or slowly squeezing, non-swelling clay.	Highly pre-consolidated clay with plasticity index greater than about 30, generally containing significant percentages of montmorillonite clay.
Running	The removal of lateral support of any surface rising at an angle more than 34 degrees to the horizontal is immediately followed by a running movement of the soil particles.	Clean, dry angular materials.
Cohesive Running	If the running ground has a trace of cohesion, then the run is preceded by a brief period of progressive raveling.	Apparent cohesion in moist sand or weak cementation in any granular soil may allow the material to stand for a brief period of raveling before it breaks down and runs. Such behavior is cohesive running.

Ground Classifications	Ground Behavior	Typical Soil Types
Very Soft Squeezing	Ground advances rapidly into tunnel in a plastic flow.	
Flowing	Ground supporting a tunnel cannot be classified as flowing ground unless water flows or seeps through it toward a tunnel. For this reason, a flowing condition is encountered only in free air tunnels below the water table or under compressed air when the pressure is not high enough in the tunnel to dry the bottom.	Only occurs in inorganic silt, fine silty sand, clean sand or gravel, or sand-and-gravel with some clay binder. Organic silt may behave either as a lowing or as a very soft, squeezing ground.

The results of our subsurface exploration indicate relatively loose sandy soil conditions will likely be encountered on the upper approximately 15 feet bgs. The soil deposits become denser below approximately 15 feet bgs. It is our opinion that trenchless construction at the project site can be accomplished by an experienced contractor using jacking/micro-tunneling equipment. Provisions for controlling raveling and running sand soils should be provided during the trenchless operation to minimize ground loss and ground subsidence.

Site-specific ground conditions and soil classifications pertaining to this project are presented in Table No. 7, *Approximate Site Specific Ground Classifications.*

Table No.7, Approximate Site Specific Ground Classifications

Jack-and-Bore Location	Approximate Depth To Invert (Feet)	Soil Types	Raveling	Running	Cohesive Running
Jack and Bore Locations	15-20	SM, SP, SP-SM	√	√	√

It is the contractor's responsibility to design and select the appropriate tunnel construction method, support system and to follow the requirements of the health and safety rules of the State of California pertaining to tunnel construction and permit requirements of the County of Riverside and other local agencies, if applicable.

8.3.2 Backfill of Jacking and Receiving Pits

We anticipate that the depths of the jacking and receiving pits will be about 15 to 20 feet below the existing grade. The pits should be backfilled following the placement of the pipe crossing.

Pit excavations to receive backfill should be free of trash, debris or other unsatisfactory materials at the time of backfill placement. The bottoms of the excavations should be scarified to a depth of at least 12 inches where possible. The scarified soils should be brought to near-optimum moisture content and compacted to at least 90 percent of the laboratory maximum dry density to produce a firm and unyielding surface. Fill should then be placed on the compacted soils in loose lifts of eight (8) inches or less, moisture conditioned to within ± three (3) percent of optimum, and compacted to at least 90 percent of the laboratory maximum dry density determined by the ASTM D1557 test method. The contractor should select the equipment and processes to be used to achieve the specified density without damage to adjacent ground, various facilities, utilities and completed work.

8.4 Pipe Trenching Recommendations

Recommendations pertaining to the open-cut and cover sections of the alignment are presented in the following subsections.

8.4.1 Pipeline Sub-grade Preparation

For the majority of proposed pipeline alignments, the subsurface materials at the proposed invert depths should be suitable as pipe sub-grade. The final sub-grade surface should be level, firm, uniform, and free of loose materials and properly graded to provide uniform bearing and support to the entire section of the pipe placed on bedding material. Protruding oversize particles, larger than two (2) inches in dimension, if any, should be removed from the trench bottom and replaced with compacted on-site materials.

Any loose, soft and/or unsuitable materials encountered at the pipe sub-grade should be removed and replaced with an adequate bedding material.

During the digging of depressions for proper sealing of the pipe joints, the pipe should rest on a prepared bottom for as near its full length as is practicable.

8.4.2 Pipe Bedding

Bedding is defined as the material supporting and surrounding the pipe, to 12 inches above the pipe. To provide uniform and firm support for the pipe, compacted granular materials such as clean sand may be used as pipe bedding material. The load on the rigid pipes and deflection of flexible pipes and, hence, the pipe design, depends on the type and the amount of bedding placed underneath and around the pipe. The pipe bedding material should be selected by the pipeline designer.

Pipe design generally requires a granular material with a Sand Equivalent (SE) greater than 30. Bedding material for the pipes should be free from oversized particles (greater than 1-inch). Results of SE tests on two (2) representative soil samples were 20 and 76. Selected onsite soils may be suitable to be used as bedding after processing to remove oversize particles larger than 1 inch in maximum dimension.

Migration of fines from the surrounding native and/or fill soils must be considered in selecting the gradation of any imported bedding material. We recommend that the pipe bedding material should satisfy the following criteria:

$$D_{15} < 2.5 \text{ mm (0.098-inch) and } D_{50} < 19.0 \text{ mm (0.75-inch)}$$

Where D_{15} and D_{50} represent particle sizes of the bedding material corresponding to 15 percent and 50 percent passing by weight, respectively.

Care should be taken to densify the bedding material below the springline of the pipe.

8.4.3 Trench Zone Backfill

The trench zone is defined as the portion of the trench above the pipe bedding extending up to the final grade level of the trench surface. Excavated site soils free of deleterious matter may be used to backfill the trench zone. Detail trench backfill recommendations are provided below:

- Trench excavations to receive backfill should be free of trash, debris or other unsatisfactory materials at the time of backfill placement.

- Trench zone backfill shall be compacted to at least 90 percent of the laboratory maximum dry density as per ASTM D1557 test method or as required by the local agency standards. At least the upper one (1) foot of trench backfill underlying pavement should be compacted to at least 95 percent of the laboratory maximum dry density as per ASTM D1557 test method.

- Particles larger than one (1) inch should not be placed within 12 inches of the pavement sub-grade. No more than 30 percent of the backfill volume should be larger than ¾-inch in the largest dimension. Gravel should be well mixed with finer soil. Rocks larger than three (3) inches in the largest dimension should not be placed as trench backfill

- Trench backfill should be compacted by mechanical methods, such as sheepsfoot, vibrating or pneumatic rollers or mechanical tampers to achieve the density specified herein. The backfill materials should be brought to within three ± (3) percent of optimum moisture content then placed in horizontal layers. The thickness of uncompacted layers should not exceed eight (8) inches. Each layer should be evenly spread, moistened or dried as necessary, and then tamped or rolled until the specified density has been achieved.

- The contractor should select the equipment and processes to be used to achieve the specified density without damage to adjacent ground, structures, utilities and completed work.

- The field density of the compacted soil should be measured by the ASTM Standard D1556 or ASTM D2922 test methods or equivalent.

- Observations and field tests should be performed by the project soils consultant to confirm that the required degree of compaction has been obtained. Where compaction is less than that specified, additional compactive effort should be made with adjustment of the moisture content as necessary, until the specified compaction is obtained.

- It should be the responsibility of the contractor to maintain safe working conditions during all phases of construction.

- Trench backfill should not be placed, spread or rolled during unfavorable weather conditions. When the work is interrupted by heavy rain, fill operations should not resume until field tests by the project's geotechnical consultant indicate that the moisture content and density of the fill are in compliance with project specifications.

8.4.4 Imported Backfill Materials

Imported soils, if any, used as compacted trench backfill should be predominantly granular and meet the following criteria:

- Expansion Index less than 20
- Free of all deleterious materials
- Contain no particles larger than 3 inches in the largest dimension
- Contain less than 30 percent by weight retained on ¾-inch sieve
- Contain at least 15 percent fines (passing #200 sieve)
- Have a Plasticity Index of 10 or less

Any imported backfill should be tested and approved by the owner's representative prior to delivery to the site.

9.0 DESIGN RECOMMENDATIONS

9.1 General

The following design recommendations are based on our analysis of the data obtained during field investigation, laboratory testing, and our understanding of the proposed project.

9.2 Seismic Hazards

The prefabricated building location, the pipeline segment along Orchard Street, and the northern approximate 600 feet of the pipeline segment along Beaumont Avenue are within a Riverside County fault hazard zone and may have the potential for surface rupture during a seismic event.

Consideration should be given to flexible couplings, automatic shut-off valves, or other measures to mitigate damage in the event of a fault rupture across the pipeline.

We anticipate that the 12-foot by 12-foot prefabricated building will only be occupied intermittently for equipment maintenance. We do not anticipate that the building will be considered a structure for human occupancy, which generally means more than 2,000 person-hours of occupancy per year. Revisions to the project design or usage of the structure may require further investigation of the potential presence of on-site faulting.

9.3 Foundation Type and Bearing Pressures

Lightweight structures such as the proposed 12-foot by 12-foot prefabricated building may be supported on continuous (strip) and/or isolated spread footings. Continuous and isolated spread footings should be at least 12 inches wide. The depth of embedment below lowest adjacent soil grade should be at least 12 inches. Footings should be founded on at least 12 inches of scarified and compacted soil.

For shallow spread footings founded on scarified and compacted soil, an allowable net bearing capacity of 1,200 pounds per square foot (psf), plus 300 psf for each additional foot of depth, may be used. The maximum allowable bearing capacity should be limited to 2,500 psf.

The allowable net bearing capacity is defined as the maximum allowable net bearing pressure on the ground. It is obtained by dividing the net ultimate bearing capacity by a safety factor. The ultimate bearing capacity is the bearing stress at which ground fails by shear or experiences a limiting amount of settlement at the foundation. The net ultimate bearing capacity is obtained by subtracting the total overburden pressure on a horizontal plane at the foundation level from the ultimate bearing capacity.

The net allowable bearing values indicated above are for the dead loads and frequently applied live loads and are obtained by applying a factor of safety of 3.0 to the net ultimate bearing capacity. If normal code requirements are applied for design, the above vertical bearing value may be increased by 33 percent for short duration loadings, which will include loadings induced by wind or seismic forces.

9.4 Lateral Earth Pressures and Resistance to Lateral Loads

The following subsections outline lateral earth pressures and resistance to lateral loads. Lateral earth pressures and resistance to lateral loads are estimated by using on-site native soils strength parameters obtained from laboratory testing. The following recommendations are considered applicable for all pipeline segments.

9.4.1 Lateral Earth Pressures

The active earth pressure behind any buried wall depends primarily on the allowable movement, type of backfill materials, backfill slopes, wall inclination, surcharges, and any hydrostatic pressures. In general, the lateral earth pressures are presented in the following table.

Table No. 8, Lateral Earth Pressure

Loading Conditions	Equivalent Fluid Pressure (pcf)
Active earth conditions (wall is free to deflect at least 0.001 radian)	37
At-rest (wall is restrained)	56

These pressures assume a level ground surface behind the wall for a distance greater than the wall height, no surcharge, no hydrostatic pressure, and soil expansion index less than 30.

If water pressure is allowed to build-up, the active pressures should be reduced by 50 percent and added to the full hydrostatic pressure to compute the design pressures against the wall.

9.4.2 Passive Earth Pressure

Resistance to lateral loads can be assumed to be provided by friction acting at the base of foundations and by passive earth pressure. A coefficient of friction of 0.35 between concrete and soil may be used with the dead load forces. An allowable passive earth pressure of 240 psf per foot of depth may be used for resistance against recompacted native soils. A factor of safety of 1.5 was applied in calculating passive earth pressure. The maximum value of the passive earth pressure should be limited to 2,500 psf for native soils.

Passive earth resistance values indicated above are for the total dead loads and frequently applied live loads. If normal code requirements are applied for design, the above passive resistance values may be increased by 33 percent for short duration loading, which will include the effect of wind or seismic forces.

Due to the low overburden stress of the soil at shallow depth, the upper one foot of passive resistance should be neglected unless the soil is confined by pavement or slab.

9.5 Soil Parameters for Pipe Design

Structural design of pipeline requires proper evaluation of all possible loads acting on pipes. The stresses and strains induced on the buried pipes depend on many factors, including the type of soil, density, bearing pressure, angle of internal friction, coefficient of passive earth pressure, and coefficient of friction at the interface between the backfill and native soils. The recommended values of the various soil parameters for the pipe design are provided in the following table.

Table No. 9, Soil Parameters for Pipe Design

Soil Parameters	
Average compacted fill unit weight, γ	132.0 pcf
Buoyant weight of backfill, γ_b	70 pcf
Angle of internal friction of soils, ϕ	32 º
Soil cohesion, c	0 pcf
Coefficient of friction between backfill and native soils, fs	0.3
Coefficient of friction between pipe and native soils, fs	0.25
Bearing pressure against native Soils	2,500 psf
Coefficient of passive earth pressure, Kp Coefficient of passive earth pressure Kp	3.12
Modulus of Soil Reaction E' (psi)	1000

9.6 Bearing Pressure for Anchor and Thrust Blocks

Allowable net bearing pressure of 2,500 psf may be used for anchor and thrust block design against site soils. Such thrust blocks should be at least 24 inches wide.

Resistance to lateral forces can be assumed to be provided by friction at the base of thrust blocks and by passive earth pressure. An ultimate value of coefficient of friction of 0.35 may be used between the thrust block and the supporting natural soil or compacted fill. A passive earth pressure of 240 psf per foot of depth may be used for the sides of thrust blocks or anchors poured against undisturbed or recompacted soils. The value of the passive lateral earth pressure should be limited to 2,500 psf. Frictional and passive resistance can be combined for the design of anchors and thrust blocks.

If normal code requirements are applied for design, the above recommended bearing capacity and passive resistances may be increased by 33 percent for short duration loading such as seismic or wind loading.

10.0 CONSTRUCTION RECOMMENDATIONS

10.1 General

Both sloped and vertical braced excavations should be feasible along the pipeline alignments and at the jacking and receiving pit locations. Recommendations pertaining to temporary excavations are presented in this section.

Excavations within existing streets may require vertical side wall excavation. Where the side of the excavation is a vertical cut, it should be adequately supported by temporary shoring to protect workers and any adjacent structures.

All applicable requirements of the California Construction and General Industry Safety Orders, the Occupational Safety and Health Act of 1989, current amendments, and the Construction Safety Act should be met. The soils exposed in cuts should be observed during excavation by the owner's representative. If potentially unstable soil conditions are encountered, modifications of slope ratios for temporary cuts may be required.

10.2 Temporary Sloped Excavations

Temporary open-cut trenches may be constructed with side slopes as recommended in Table No. 10, *Slope Ratios for Temporary Excavations.* Temporary cuts encountering soft and wet fine-grained soils; dry loose, cohesionless soils or loose fill from trench backfill may have to be constructed at a flatter gradient than presented below.

Table No. 10, Slope Ratios for Temporary Excavations

Depth of Cut (feet)	Recommended Maximum Slope (Horizontal:Vertical)[1]
0-4	1.5:1
4-20	2:1

For steeper temporary construction slopes or deeper excavations, or unstable soil encountered during the excavation, shoring or a trench box should be provided by the contractor as necessary, to protect the workers in the excavation. Design recommendations for temporary shoring can be provided if necessary.

Surfaces exposed in slope excavations should be kept moist but not saturated to retard raveling and sloughing during construction. Adequate provisions should be made to protect the slopes from erosion during periods of rainfall. Surcharge loads, including construction materials, should not be placed within five (5) feet of the unsupported slope edge. Stockpiled soils with a height higher than six (6) feet will require greater distance from trench edges.

10.3 Shoring Design

Temporary shoring will be required where open sloped excavations will not be feasible to nearby existing structures or facilities. Temporary shoring may consist of conventional soldier piles and lagging or sheet piles. The shoring for the pipe excavations laterally supported by walers and cross bracing or may be cantilevered. Drilled excavations for soldier piles will require the use of drilling fluids to prevent caving and to maintain an opened hole for pile installation.

Braced shoring should be designed to support a uniform rectangular lateral earth pressure of 27 psf, based on Figure No. 3, *Recommended Lateral Earth Pressure for Braced Excavation*.

Design of cantilever shoring consisting of soldier piles spaced at least two diameters on-center or sheet piles, can be performed based on Figure No. 4, *Lateral Earth Pressure on Cantilever Shoring*.

The contractor should have provisions for soldier pile and sheet pile removal. All voids resulting from removal of shoring should be filled. The method for filling voids should be selected by the contractor, depending on construction conditions, void dimensions and available materials. (The acceptable materials, in general, should be non-deleterious, and able to flow into the voids created by shoring removal, e.g. concrete slurry, “pea” gravel, etc).

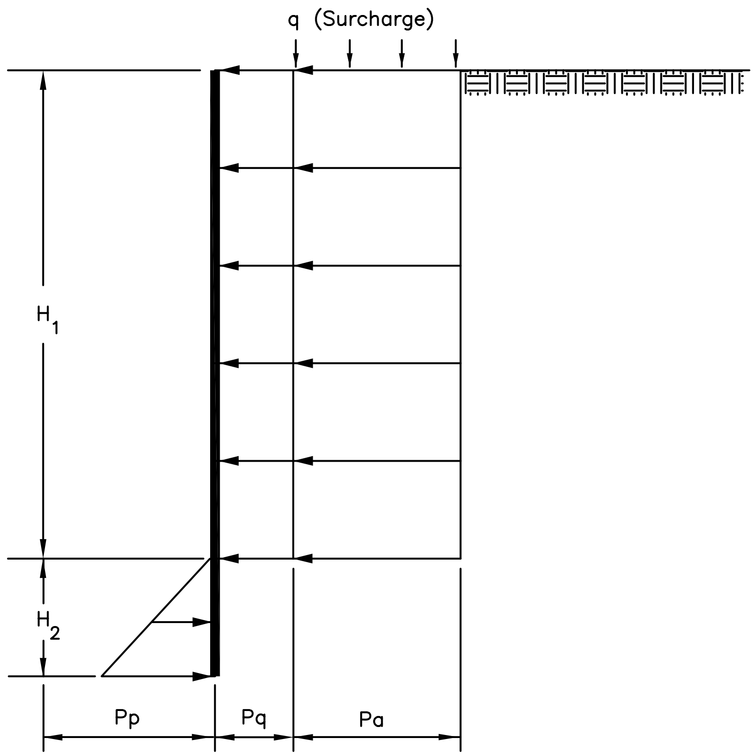
In addition to the lateral earth pressure, surcharge pressures due to miscellaneous loads, such as soil stockpiles, vehicular traffic or construction equipment located adjacent to the shoring, should be included in the design of the shoring. A uniform lateral pressure of 100 psf should be included in the upper 10 feet of the shoring to account for normal vehicular and construction traffic within 10 feet of the trench excavation. As previously mentioned, all shoring should be designed and installed in accordance with state and federal safety regulations.

The lagging between the soldier piles may consist of pressure-treated wood members or solid steel sheets. In our opinion, steel sheeting is expected to be more expedient than wood lagging to install. Although soldier piles and any bracing used should be designed for the full-anticipated earth pressures and surcharge pressures, the pressures on the lagging are less because of the effect of arching between the soldier piles. Accordingly, the lagging between the piles may be designed based on the following guidelines:

- Lagging design load = 0.6 of shoring design load
- Maximum lagging load may be 400 psf without surcharges

Excavations for the proposed pipeline should not extend below a 1:1 horizontal:vertical (H:V) plane extending from the bottom of any existing structures, utility lines or streets. Any proposed excavation should not cause loss of bearing and/or lateral supports of the existing utilities or streets.

TEMPORARY BRACED EXCAVATION
LATERAL EARTH PRESSURE



$P = Pq + Pa$

$\quad = 0.5q + 27H_1$ (350 psf minimum) – active earth pressure

$Pp = 240\ H_2 \leq 2500$ psf – passive earth pressure (on native or compacted soils)

$\mu = 0.35$ – Ultimate friction coefficient

Notes:

1. All values of height (H) in feet, pressure (P) and surcharge (q) in pounds per square foot (psf).
2. Pp and Pa are the passive and active earth pressure respectively; Pq is the incremental surcharge earth pressure; and μ is the allowable friction coefficient, applied to dead normal loads acting on non–pile supported elements.
3. Earth pressures assume no hydrostatic pressures. If hydrostatic pressures are allowed to build up, the incremental earth pressures below the ground–water level should be reduced by 50 percent and added to hydrostatic pressure for total lateral pressure.
4. Pp includes a safety factor of 1.5.
5. Neglect the upper 1 foot for passive pressure unless the surface is confined by a pavement of slab.
6. For traffic surcharge, use a uniform pressure of 100 psf over the top 10 feet.

RECOMMENDED LATERAL EARTH PRESSURE FOR BRACED EXCAVATION

Beaumont Avenue Recharge Basin Pipeline
City of Beaumont, Riverside County, California
For: ATKINS

Project No.
12–81–189–01

Converse Consultants

Drawing No.
3

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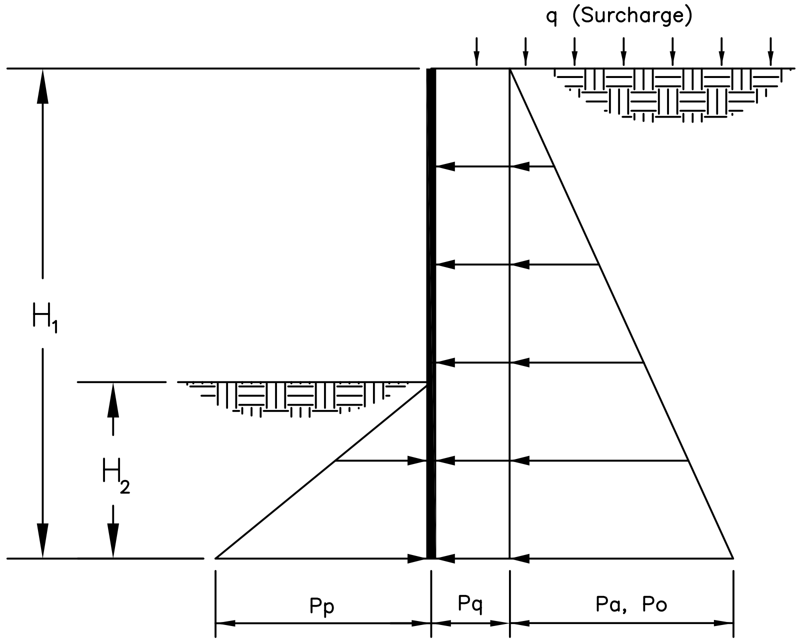
Temporary Cantilever Shoring

Lateral Earth Pressure



$P = Pq + Pa$ (350 psf minimum if less than 350 psf)

$= 0.5q + 37\ H_1$ – active earth pressure (Cantilever walls)

$= 0.5q + 56\ H_1$ – at–rest earth pressure (Restrained walls)

$Pp = 240\ H_2 \leq 2500$ psf – passive earth pressure (on native or compacted soils)

$= 100\ H_2$ psf – passive earth pressure (on upper 10 feet of fill soils)

$\mu = 0.35$ – Ultimate friction coefficient

Notes:

1. All values of height (H) in feet, pressure (P) and surcharge (q) in pounds per square foot (psf).
2. Pp, Pa, and Po are the passive, active, and at–rest earth pressures, respectively; Pe is the incremental seismic earth pressure; Pq is the incremental surcharge earth pressure; and μ is the allowable friction coefficient, applied to dead normal loads acting on non–pile supported elements.
3. Pp include a safety factor μ of 1.5.
4. Neglect the upper 1 foot for passive pressure unless the surface is confined by a pavement or slab.
5. Surcharge load only applies the upper 10 feet.
6. To account for the lateral pressure exerted by groundwater, the active pressures should be reduced by 50% and added to full hydrostatic pressure to compute the design pressures against the walls.
7. For traffic surcharge, assume a 100–psf uniform pressure along the top 10 feet.

RECOMMENDED LATERAL EARTH PRESSURE ON CANTILEVER WALL

Beaumont Avenue Recharge Basin Pipeline
City of Beaumont, Riverside County, California
For: ATKINS

Project No.
12–81–189–01

Converse Consultants

Figure No.
4

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If the excavation extends below a 1:1 (H:V) plane extending from the bottom of the existing structures, utility lines or streets, a maximum of 10 feet in length can be exposed at a time to reduce the potential instability. Backfill should be accomplished in the shortest period of time and in alternating sections.

10.4 Trenchless Pipe Crossing Construction

We anticipate micro-tunneling and/or pipe jacking for the proposed project. Conventional excavation of the boring-and-jacking pits will likely require the use of shoring.

Pipe jacking and micro-tunneling operations involve the initial construction of a jacking/tunneling pit and a receiving pit at each end of the pipe segment to be jacked. Micro-tunneling can be regarded as an extension of pipe jacking where a new pipe is pushed through a hole excavated ahead of the advancing pipe string. Whereas traditional pipe jacking requires a team of workers at the face, micro-tunneling replaces this manual work with a small tunnel boring machine (TBM).

The working/access shafts are utilized to remove the spoil and to transport the construction materials and personnel for a tunnel project. The vertical face of the working shaft may be shored with sheet piles and/or soldier piles and lagging. The face of the shaft also can be supported by ribs and laggings. The design of sheet piling, soldier beam and lagging system may be designed according to the recommendations provided in the Section 10.3, *Shoring Design*. Frequent contact grouting may be necessary to backpack the support during construction to minimize settlement.

The total load that can be developed in the jacking plate would depend on the depth and area of the plate. The jacking equipment should not impose a reaction of more than 2,500 psf on the stabilized soils within the jacking pit. Pipes for use with the micro-tunneling systems must be designed to withstand the high axial jacking forces, and this is likely to be a far more significant design parameter than any post installation loading.

The selection of trenchless pipe crossing methods and equipment depends on pipe material, length of crossing, and anticipated ground conditions, and should be made by the contractor. Grouting through the pipe casing after jacking is recommended to fill any possible voids created by the jacking operation. Jacking operations and tunneling operations should be performed in accordance with the Standard Specifications for Public Works Construction, Sections 306-2 and 306-3 (Public Works Standards, 2012).

Excavation procedures and shoring systems should be properly designed and implemented/installed to minimize the effect of settlement during construction. The contractor is responsible for minimizing impacts of crossing operations. Ground distress potential along a crossing alignment depends on a number of factors, including type of soils, type of face support, internal pressure maintained to support the face, length of unlined zone, if any, and the amount of gap between the shield and the surrounding soils. The potential of any significant ground distress at the surface can be minimized by selecting the proper equipment and construction method. The zone of influence of properly performed pipe crossing should be limited to a distance of about 2D above the crown of the shield, where D is the diameter of the shield. When the depth of crown cover is about 2D or more, maximum ground surface settlement, if any, can be expected to be less than the thickness of the gap around the pipe. Higher ground settlement may occur for less depth of cover and inadequately supported pits can induce significant ground movement or even collapse.

It is the contractor's responsibility to document the existing pre-construction conditions of streets and any facilities, and monitor deformations during construction. We recommend that ground surface above crossing operations be continuously monitored during construction using a surface settlement monument to make sure any vertical and horizontal movements are within allowable limits. Corrective action will be required by the contractor if deformations exceed the allowable limits.

11.0 GEOTECHNICAL SERVICES DURING CONSTRUCTION

The project geotechnical consultant should be present to observe conditions and test the density and moisture of the backfill during the earthwork for this project. The excavations and backfill should be observed and tested as to the compliance with project specifications.

12.0 CLOSURE

This report is prepared for the project described herein and is intended for use solely by Atkins and authorized agents, to assist in the design and construction of the proposed project. Our services have been performed in accordance with applicable state and local ordinances, and generally accepted practices within our profession.

Converse Consultants is not responsible or liable for any claims or damages associated with interpretation of available information provided to others. Site exploration identifies actual soil conditions only at those points where samples are taken, when they are taken. Data derived through sampling and laboratory testing is extrapolated by Converse employees who render an opinion about the overall soil conditions. Actual conditions in areas not sampled may differ. In the event that changes to the project occur, or additional, relevant information about the project is brought to our attention, the recommendations contained in this report may not be valid unless these changes and additional relevant information are reviewed and the recommendations of this report are modified or verified in writing. In addition, the recommendations can only be finalized by observing actual subsurface conditions revealed during construction. Converse cannot be held responsible for misinterpretation or changes to our recommendations made by others during construction.

Our findings and recommendations were obtained in accordance with generally accepted professional principles practiced in geotechnical engineering. We make no other warranty, either expressed or implied. Our conclusions and recommendations are based on the results of the field investigations and laboratory tests, combined with interpolation and extrapolation of soil conditions between and beyond the boring locations.

13.0 REFERENCES

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APPENDIX A

FIELD EXPLORATION

APPENDIX A

FIELD EXPLORATION

Our field investigation included a site reconnaissance and a subsurface exploration program consisting of drilling soil borings. During the site reconnaissance, the surface conditions were noted and the borings were marked at the locations indicated on the Beaumont Avenue Recharge Facility Pipeline 2013 plans and profile prepared by Atkins for the San Gorgonio Pass Water Agency. The borings were located using existing topography and boundary features as a guide and should be considered accurate only to the degree implied by the method. Permits for drilling were obtained from the City of Beaumont and the County of Riverside Transportation Department.

Thirteen (13) borings (BH-1 through BH-13) were drilled on January 9 and 10, 2013. The location and depth of the borings drilled are presented in the table below. Borings BH-2 and BH-6 were terminated before reaching the planned depths of 50 feet and 15 feet due to auger refusal in gravel. Boring BH-8 was drilled on the shoulder of Beaumont Avenue to a depth of 16.5 feet bgs. Boring BH-8A was redrilled on the pipeline alignment in the street to verify the soils profile obtained within boring BH-8.

Table No. A-1, Boring Locations and Depths

Boring Number/ Depth (feet)	Area/Street Name	Station Number	Remarks
BH-1/16.5	Southwest corner of Orchard Street and Mountain Avenue	10+00	12' by 12' prefabricated building
BH-2/25.0		10+00	12' by 12' prefabricated building
BH-3/16.5		10+00	Pipeline
BH-4/26.4	Orchard Street	10+60	Pipeline + Bore & Jack
BH-5/26.5		10+95	Pipeline + Bore & Jack
BH-6/4.5	Beaumont Avenue	19+90	Pipeline
BH-7/16.5		31+00	Pipeline
BH-8/16.5		41+00 (shoulder)	Pipeline
BH-8A/16.5		41+00 (alignment)	Pipeline
BH-9/15.5		51+00	Pipeline
BH-10/16.5		61+00	Pipeline
BH-11/26.5		69+25	Pipeline + Bore & Jack
BH-12/26.5		71+00	Pipeline + Bore & Jack
BH-13/16.5		76+00	Pipeline

The borings were advanced using a truck-mounted drill rig equipped with eight-inch diameter hollow-stem augers and a drive sampler system for soils sampling. Encountered materials were continuously logged by a Converse engineer and visually classified in the field in accordance with the Unified Soil Classification System. Where appropriate, the field descriptions and classifications have been modified to reflect laboratory test results.

Ring samples of the subsurface materials were obtained at frequent intervals in the exploratory borings using a drive sampler (2.4 inches inside diameter and 3.0 inches outside diameter) lined with sample rings. The steel ring sampler was driven into the bottom of the borehole with successive drops of a 140-pound driving weight falling 30 inches. The recorded blow counts for every six (6) inches for a total of 1.5 feet of sampler penetration are shown on the Logs of Borings. Samples were retained in brass rings (2.4 inches inside diameter and 1.0 inch in height). The central portion of the sample was retained and carefully sealed in waterproof plastic containers for shipment to the Converse laboratory. Bulk samples of typical soil types were also obtained. The borings were backfilled with soil cuttings at the completion of drilling and sampling. In paved areas, the surface was patched with cold asphalt concrete.

The exact depths at which material changes occur cannot always be established accurately. Unless a more precise depth can be established by other means, changes in material conditions that occur between drive samples are indicated on the logs at the top of the next drive sample.

For a key to soil symbols and terminology used in the boring logs, refer to Drawing No. A-1, *Unified Soil Classification and Key to Boring Log Symbols*. For logs of borings, see Drawings No. A-2 through A-15, *Logs of Borings*.

SOIL CLASSIFICATION CHART

MAJOR DIVISIONS			SYMBOLS GRAPH	SYMBOLS LETTER	TYPICAL DESCRIPTIONS
COARSE GRAINED SOILS MORE THAN 50% OF MATERIAL IS LARGER THAN NO. 200 SIEVE SIZE	GRAVEL AND GRAVELLY SOILS MORE THAN 50% OF COARSE FRACTION RETAINED ON NO. 4 SIEVE	CLEAN GRAVELS (LITTLE OR NO FINES)		GW	WELL-GRADED GRAVELS, GRAVEL - SAND MIXTURES, LITTLE OR NO FINES
				GP	POORLY-GRADED GRAVELS, GRAVEL - SAND MIXTURES, LITTLE OR NO FINES
		GRAVELS WITH FINES (APPRECIABLE AMOUNT OF FINES)		GM	SILTY GRAVELS, GRAVEL - SAND - SILT MIXTURES
				GC	CLAYEY GRAVELS, GRAVEL - SAND - CLAY MIXTURES
	SAND AND SANDY SOILS MORE THAN 50% OF COARSE FRACTION PASSING ON NO. 4 SIEVE	CLEAN SANDS (LITTLE OR NO FINES)		SW	WELL-GRADED SANDS, GRAVELLY SANDS, LITTLE OR NO FINES
				SP	POORLY-GRADED SANDS, GRAVELLY SAND, LITTLE OR NO FINES
		SANDS WITH FINES (APPRECIABLE AMOUNT OF FINES)		SM	SILTY SANDS, SAND - SILT MIXTURES
				SC	CLAYEY SANDS, SAND - CLAY MIXTURES
FINE GRAINED SOILS MORE THAN 50% OF MATERIAL IS SMALLER THAN NO. 200 SIEVE SIZE	SILTS AND CLAYS	LIQUID LIMIT LESS THAN 50		ML	INORGANIC SILTS AND VERY FINE SANDS, ROCK FLOUR, SILTY OR CLAYEY FINE SANDS OR CLAYEY SILTS WITH SLIGHT PLASTICITY
				CL	INORGANIC CLAYS OF LOW TO MEDIUM PLASTICITY, GRAVELLY CLAYS, SANDY CLAYS, SILTY CLAYS, LEAN CLAYS
				OL	ORGANIC SILTS AND ORGANIC SILTY CLAYS OF LOW PLASTICITY
	SILTS AND CLAYS	LIQUID LIMIT GREATER THAN 50		MH	INORGANIC SILTS, MICACEOUS OR DIATOMACEOUS FINE SAND OR SILTY SOILS
				CH	INORGANIC CLAYS OF HIGH PLASTICITY
				OH	ORGANIC CLAYS OF MEDIUM TO HIGH PLASTICITY, ORGANIC SILTS
HIGHLY ORGANIC SOILS				PT	PEAT, HUMUS, SWAMP SOILS WITH HIGH ORGANIC CONTENTS

NOTE: DUAL SYMBOLS ARE USED TO INDICATE BORDERLINE SOIL CLASSIFICATIONS

BORING LOG SYMBOLS

SAMPLE TYPE

STANDARD PENETRATION TEST
Split barrel sampler in accordance with ASTM D-1586-84 Standard Test Method

DRIVE SAMPLE 2.42" I.D. sampler (CMS).

DRIVE SAMPLE No recovery

BULK SAMPLE

GROUNDWATER WHILE DRILLING

GROUNDWATER AFTER DRILLING

LABORATORY TESTING ABBREVIATIONS

TEST TYPE
(Results shown in Appendix B)

CLASSIFICATION

Plasticity	pi
Grain Size Analysis	ma
Passing No. 200 Sieve	wa
Sand Equivalent	se
Expansion Index	ei
Compaction Curve	max
Hydrometer	h
Disturb	Dist.

STRENGTH

Pocket Penetrometer	p
Direct Shear	ds
Direct Shear (single point)	ds*
Unconfined Compression	uc
Triaxial Compression	tx
Vane Shear	vs
Consolidation	c
Collapse Test	col
Resistance (R) Value	r
Chemical Analysis	ca
Electrical Resistivity	er
Permeability	perm
Soil Cement	sc

Apparent Density	Very Loose	Loose	Medium	Dense	Very Dense
SPT (N)	< 4	4 - 11	11 - 30	31 - 50	> 50
CA Sampler	< 5	5 - 12	13 - 35	36 - 60	> 60
Relative Density (%)	< 20	20 - 40	40 - 60	60 - 80	> 80

Consistency	Very Soft	Soft	Medium	Stiff	Very Stiff	Hard
SPT (N)	< 2	2-4	5-8	9-15	16-30	> 30
CA Sampler	< 3	3-6	7-12	13-25	26-50	> 50

UNIFIED SOIL CLASSIFICATION AND KEY TO BORING LOG SYMBOLS

Converse Consultants

Beaumont Avenue Recharge Basin Pipeline
City of Beaumont, Riverside County, California
For: Atkins

Project No. 12-81-189-01

Drawing No. A-1

Log of Boring No. BH- 1

Dates Drilled: 1/10/2013 Logged by: AM Checked By: SM

Equipment: CME 75/ 8" HSA Driving Weight and Drop: 140 lbs / 30 in

Ground Surface Elevation (ft): 2843 Depth to Water (ft): NOT ENCOUNTERED

Depth (ft)	Graphic Log	SUMMARY OF SUBSURFACE CONDITIONS This log is part of the report prepared by Converse for this project and should be read together with the report. This summary applies only at the location of the boring and at the time of drilling. Subsurface conditions may differ at other locations and may change at this location with the passage of time. The data presented is a simplification of actual conditions encountered.	SAMPLES DRIVE	SAMPLES BULK	BLOWS	MOISTURE	DRY UNIT WT. (pcf)	OTHER
		SILTY SAND (SM): fine to coarse-grained, yellow brown.						
					4/6/7	6	99	ma, col
5					4/5/8	3	110	
					3/8/9	5	113	
10					5/8/13	5	117	
15					3/6/7			
		End of Boring at 16.5 feet. No Groundwater Encountered Borehole backfilled with loose soil cuttings and lightly tampered on 1/10/13.						

Converse Consultants

Beaumont Avenue Recharge Basin Pipeline
City of Beaumont, Riverside County, California
For: Atkins

Project No. **12-81-189-01**

Drawing No. **A-2**

Log of Boring No. BH- 2

Dates Drilled: 1/10/2013 Logged by: AM Checked By: SM

Equipment: CME 75/ 8" HSA Driving Weight and Drop: 140 lbs / 30 in

Ground Surface Elevation (ft): 2847 Depth to Water (ft): NOT ENCOUNTERED

Depth (ft)	Graphic Log	SUMMARY OF SUBSURFACE CONDITIONS This log is part of the report prepared by Converse for this project and should be read together with the report. This summary applies only at the location of the boring and at the time of drilling. Subsurface conditions may differ at other locations and may change at this location with the passage of time. The data presented is a simplification of actual conditions encountered.	SAMPLES DRIVE	SAMPLES BULK	BLOWS	MOISTURE	DRY UNIT WT. (pcf)	OTHER
0		**SILTY SAND (SM):** fine to coarse-grained, brown.		X				
2			X	X	2/4/7	6	106	max
5			X		4/6/6	3	104	
7			X		4/4/4	5	104	
10			X		5/6/8	3	117	
15			X		3/4/6			
20			X		13/31/50-5"	6	125	
25			X		50-2"			
		Boring terminated due to auger refusal at 25'-2". No Groundwater Encountered. Borehole backfilled with loose soil cuttings and lightly tampered on 1/10/13.						

Log of Boring No. BH- 3

Dates Drilled: 1/10/2013 Logged by: AM Checked By: SM

Equipment: CME 75/ 8" HSA Driving Weight and Drop: 140 lbs / 30 in

Ground Surface Elevation (ft): 2847 Depth to Water (ft): NOT ENCOUNTERED

Depth (ft)	Graphic Log	SUMMARY OF SUBSURFACE CONDITIONS This log is part of the report prepared by Converse for this project and should be read together with the report. This summary applies only at the location of the boring and at the time of drilling. Subsurface conditions may differ at other locations and may change at this location with the passage of time. The data presented is a simplification of actual conditions encountered.	SAMPLES DRIVE	SAMPLES BULK	BLOWS	MOISTURE	DRY UNIT WT. (pcf)	OTHER
		SILTY SAND (SM): fine to coarse-grained, light brown.						
					5/7/8	8	116	
5					3/4/5	7	104	ds
					4/7/10	6	115	
10					9/9/11	6	113	
15					5/6/7			
		End of Boring at 16.5 feet. No Groundwater Encountered. Borehole backfilled with loose soil cuttings and lightly tampered on 1/10/13.						

Converse Consultants

Beaumont Avenue Recharge Basin Pipeline
City of Beaumont, Riverside County, California
For: Atkins

Project No. **12-81-189-01**

Drawing No. **A-4**

Log of Boring No. BH- 4

Dates Drilled: 1/10/2013 Logged by: AM Checked By: SM

Equipment: CME 75/ 8" HSA Driving Weight and Drop: 140 lbs / 30 in

Ground Surface Elevation (ft): 2849 Depth to Water (ft): NOT ENCOUNTERED

Depth (ft)	Graphic Log	SUMMARY OF SUBSURFACE CONDITIONS This log is part of the report prepared by Converse for this project and should be read together with the report. This summary applies only at the location of the boring and at the time of drilling. Subsurface conditions may differ at other locations and may change at this location with the passage of time. The data presented is a simplification of actual conditions encountered.	SAMPLES DRIVE	SAMPLES BULK	BLOWS	MOISTURE	DRY UNIT WT. (pcf)	OTHER
		4" ASPHALT CONCRETE / 3" AGGREGATE BASE						
		SILTY SAND (SM): fine to coarse-grained, light brown.						
					15/35/32	3	117	se
5					6/8/8	3	113	
					6/8/11	3	112	
10					5/6/10	4	111	
15					4/4/6			
20					19/35/40	3	134	
25					30/35/50-5"			
		End of Boring at 26.4 feet. No Groundwater encountered. Borehole backfilled with soil cuttings and surface patched with cold asphalt concrete on 1/10/13.						

Converse Consultants

Beaumont Avenue Recharge Basin Pipeline
City of Beaumont, Riverside County, California
For: Atkins

Project No. **12-81-189-01**

Drawing No. **A-5**

Log of Boring No. BH- 5

Dates Drilled: 1/10/2013 Logged by: AM Checked By: SM

Equipment: CME 75/ 8" HSA Driving Weight and Drop: 140 lbs / 30 in

Ground Surface Elevation (ft): 2849 Depth to Water (ft): NOT ENCOUNTERED

Depth (ft)	SUMMARY OF SUBSURFACE CONDITIONS This log is part of the report prepared by Converse for this project and should be read together with the report. This summary applies only at the location of the boring and at the time of drilling. Subsurface conditions may differ at other locations and may change at this location with the passage of time. The data presented is a simplification of actual conditions encountered.	SAMPLES DRIVE	SAMPLES BULK	BLOWS	MOISTURE	DRY UNIT WT. (pcf)	OTHER
	6" ASPHALT CONCRETE / 4" AGGREGATE BASE		X				
	SILTY SAND (SM): fine to coarse grained sand, little gravel up to 1" in largest dimension, brown.	■	X	6/6/6	2	103	max
5		■		4/4/6	5	105	
	SAND WITH SILT (SP-SM): fine to coarse-grained, few gravel up to 1" in largest dimension, brown.	■	X	4/6/8	6	109	ds, ma
10		■	X	6/11/13	4	120	
15		☒		4/4/7			
20	**SILTY SAND (SM):** fine to coarse-grained, brown.	■		13/25/29	8	128	
25		☒		12/40/33			

End of Boring at 26.5 feet.
No Groundwater Encountered.
Borehole backfilled with soil cuttings and surface patched with cold asphalt concrete on 1/10/13.

Converse Consultants

Beaumont Avenue Recharge Basin Pipeline
City of Beaumont, Riverside County, California
For: Atkins

Project No. 12-81-189-01

Drawing No. A-6

Log of Boring No. BH- 6

Dates Drilled: 1/10/2013 Logged by: AM Checked By: SM

Equipment: CME 75/ 8" HSA Driving Weight and Drop: 140 lbs / 30 in

Ground Surface Elevation (ft): 2865 Depth to Water (ft): NOT ENCOUNTERED

Depth (ft)	Graphic Log	SUMMARY OF SUBSURFACE CONDITIONS This log is part of the report prepared by Converse for this project and should be read together with the report. This summary applies only at the location of the boring and at the time of drilling. Subsurface conditions may differ at other locations and may change at this location with the passage of time. The data presented is a simplification of actual conditions encountered.	SAMPLES: DRIVE	SAMPLES: BULK	BLOWS	MOISTURE	DRY UNIT WT. (pcf)	OTHER
		4" ASHPALT CONCRETE / NO AGGREGATE BASE						
		SILTY SAND (SM): fine to coarse-grained, some gravel up to 2" in largest dimension, dark brown.			11/50-6"	7	93	
		Boring terminated due to auger refusal at 4.5' bgs. No Groundwater Encountered. Borehole backfilled with soil cuttings and surface patched with cold asphalt concrete on 1/10/13.						

Converse Consultants

Beaumont Avenue Recharge Basin Pipeline
City of Beaumont, Riverside County, California
For: Atkins

Project No. 12-81-189-01

Drawing No. A-7

Log of Boring No. BH- 7

Dates Drilled: 1/9/2013 Logged by: AM Checked By: SM

Equipment: CME 75/ 8" HSA Driving Weight and Drop: 140 lbs / 30 in

Ground Surface Elevation (ft): 2846 Depth to Water (ft): NOT ENCOUNTERED

Depth (ft)	Graphic Log	SUMMARY OF SUBSURFACE CONDITIONS This log is part of the report prepared by Converse for this project and should be read together with the report. This summary applies only at the location of the boring and at the time of drilling. Subsurface conditions may differ at other locations and may change at this location with the passage of time. The data presented is a simplification of actual conditions encountered.	SAMPLES: DRIVE	SAMPLES: BULK	BLOWS	MOISTURE	DRY UNIT WT. (pcf)	OTHER
		10" ASPHALT CONCRETE / NO AGGREGATE BASE						
		SILTY SAND (SM): fine to coarse-grained, little gravel up to 2.5" in largest dimension, brown.			14/13/16	2	105	
5					7/7/6	4	112	
		- dark brown.			12/20/26	2	111	
10					40/50-4"	2	109	
15					7/25/38			
		End of Boring at 16.5 feet. No Groundwater Encountered. Borehole backfilled with soil cuttings and surface patched with cold asphalt concrete on 1/9/13.						

Converse Consultants

Beaumont Avenue Recharge Basin Pipeline
City of Beaumont, Riverside County, California
For: Atkins

Project No. **12-81-189-01**

Drawing No. **A-8**

Log of Boring No. BH- 8

Dates Drilled: 1/9/2013 Logged by: AM Checked By: SM

Equipment: CME 75/ 8" HSA Driving Weight and Drop: 140 lbs / 30 in

Ground Surface Elevation (ft): 2806 Depth to Water (ft): NOT ENCOUNTERED

Depth (ft)	Graphic Log	SUMMARY OF SUBSURFACE CONDITIONS This log is part of the report prepared by Converse for this project and should be read together with the report. This summary applies only at the location of the boring and at the time of drilling. Subsurface conditions may differ at other locations and may change at this location with the passage of time. The data presented is a simplification of actual conditions encountered.	SAMPLES DRIVE	SAMPLES BULK	BLOWS	MOISTURE	DRY UNIT WT. (pcf)	OTHER
		SILTY SAND (SM): fine to coarse-grained, brown.						
					7/7/9	3	110	
5					50-6"			
		SAND WITH GRAVEL (SP): fine to coarse-grained sand, little gravel up to 2" in largest dimension, brown.			11/12/22	2	111	
10					14/19/26	2	109	
15					8/15/14			
		End of Boring at 16.5 feet. No Groundwater Encountered. Borehole backfilled with loose soil cuttings and lightly tampered on 1/9/13.						

Converse Consultants

Beaumont Avenue Recharge Basin Pipeline
City of Beaumont, Riverside County, California
For: Atkins

Project No. 12-81-189-01

Drawing No. A-9

Log of Boring No. BH- 8A

Dates Drilled: 1/10/2013 Logged by: AM Checked By: SM

Equipment: CME 75/ 8" HSA Driving Weight and Drop: 140 lbs / 30 in

Ground Surface Elevation (ft): 2806 Depth to Water (ft): NOT ENCOUNTERED

Depth (ft)	Graphic Log	SUMMARY OF SUBSURFACE CONDITIONS This log is part of the report prepared by Converse for this project and should be read together with the report. This summary applies only at the location of the boring and at the time of drilling. Subsurface conditions may differ at other locations and may change at this location with the passage of time. The data presented is a simplification of actual conditions encountered.	SAMPLES DRIVE	SAMPLES BULK	BLOWS	MOISTURE	DRY UNIT WT. (pcf)	OTHER
		7" ASPHALT CONCRETE / NO AGGREGATE BASE						
		SAND (SP): fine to coarse-grained, tan.			17/15/18	2	111	se, ca, er
5		**SILTY SAND (SM):** fine to coarse-grained, brown.			4/4/5	10	108	
		SAND (SP): fine to coarse-grained, brown.			11/15/27	2	115	
10					15/30/34	2	129	
15					12/18/20			
		End of Boring at 16.5 feet. No Groundwater Encountered. Borehole backfilled with soil cuttings and surface patched with cold asphalt concrete on 1/10/13.						

Converse Consultants

Beaumont Avenue Recharge Basin Pipeline
City of Beaumont, Riverside County, California
For: Atkins

Project No. 12-81-189-01

Drawing No. A-10

Log of Boring No. BH- 9

Dates Drilled: 1/9/2013 Logged by: AM Checked By: SM

Equipment: CME 75/ 8" HSA Driving Weight and Drop: 140 lbs / 30 in

Ground Surface Elevation (ft): 2776 Depth to Water (ft): NOT ENCOUNTERED

Depth (ft)	Graphic Log	SUMMARY OF SUBSURFACE CONDITIONS This log is part of the report prepared by Converse for this project and should be read together with the report. This summary applies only at the location of the boring and at the time of drilling. Subsurface conditions may differ at other locations and may change at this location with the passage of time. The data presented is a simplification of actual conditions encountered.	SAMPLES DRIVE	SAMPLES BULK	BLOWS	MOISTURE	DRY UNIT WT. (pcf)	OTHER
		8" ASPHALT CONCRETE / NO AGGREGATE BASE						
		SILTY SAND (SM): fine to coarse-grained, dark brown.						
					7/9/10	1	114	
5		- little gravel up to 3/8" in largest dimension, light brown.			7/13/17	2	109	
		SAND WITH GRAVEL (SP): fine to coarse-grained sand, gravel up to 1/2" in largest dimension, tan.			15/19/19	1	112	
10					12/14/19	2	112	
15		- little gravel up to 3/8" in largest dimension.			50-6"			
		End of Boring at 15.5 feet. No Groundwater Encountered. Borehole backfilled with soil cuttings and surface patched with cold asphalt concrete on 1/9/13.						

Converse Consultants

Beaumont Avenue Recharge Basin Pipeline
City of Beaumont, Riverside County, California
For: Atkins

Project No. **12-81-189-01**

Drawing No. **A-11**

Log of Boring No. BH-10

Dates Drilled: 1/9/2013 Logged by: AM Checked By: SM

Equipment: CME 75/ 8" HSA Driving Weight and Drop: 140 lbs / 30 in

Ground Surface Elevation (ft): 2758 Depth to Water (ft): NOT ENCOUNTERED

Depth (ft)	Graphic Log	SUMMARY OF SUBSURFACE CONDITIONS This log is part of the report prepared by Converse for this project and should be read together with the report. This summary applies only at the location of the boring and at the time of drilling. Subsurface conditions may differ at other locations and may change at this location with the passage of time. The data presented is a simplification of actual conditions encountered.	SAMPLES DRIVE	SAMPLES BULK	BLOWS	MOISTURE	DRY UNIT WT. (pcf)	OTHER
		5-1/2" ASPHALT CONCRETE / NO AGGREGATE BASE						
		SILTY SAND (SM): fine to coarse-grained, gravel up to 1/2" in largest dimension, brown.			4/6/8	8	104	col
5		**SAND (SP):** fine to coarse-grained, brown.			5/10/15	2	118	
					9/12/18	1	105	
10					10/14/18			
15					7/10/10			
		End of Boring at 16.5 feet. No Groundwater Encountered. Borehole backfilled with soil cuttings and surface patched with cold asphalt concrete on 1/9/13.						

Log of Boring No. BH-11

Dates Drilled: 1/9/2013 Logged by: AM Checked By: SM

Equipment: CME 75/ 8" HSA Driving Weight and Drop: 140 lbs / 30 in

Ground Surface Elevation (ft): 2728 Depth to Water (ft): NOT ENCOUNTERED

Depth (ft)	Graphic Log	SUMMARY OF SUBSURFACE CONDITIONS This log is part of the report prepared by Converse for this project and should be read together with the report. This summary applies only at the location of the boring and at the time of drilling. Subsurface conditions may differ at other locations and may change at this location with the passage of time. The data presented is a simplification of actual conditions encountered.	SAMPLES: DRIVE	SAMPLES: BULK	BLOWS	MOISTURE	DRY UNIT WT. (pcf)	OTHER
		6" ASPHALT CONCRETE / NO AGGREGATE BASE						
		SILTY SAND (SM): fine to coarse-grained, few gravel up to 1/4" in largest dimension, brown.			9/12/11	3	117	
5					11/10/10	5	104	
					3/5/6	6	104	
10					4/7/11	5	118	
								ma
15		**SAND WITH GRAVEL (SP):** fine to coarse-grained sand, gravel up to 1/4" in largest dimension, tan.			6/9/10			
20		- gravel up to 1/2" in largest dimension.			17/38/43	2	124	
25		- gravel up to 1/4" in largest dimension.			6/6/12			

End of Boring at 26.5 feet.
No Groundwater Encountered.
Borehole backfilled with soil cuttings and surface patched with cold asphalt concrete on 1/9/13.

Converse Consultants

Beaumont Avenue Recharge Basin Pipeline
City of Beaumont, Riverside County, California
For: Atkins

Project No. **12-81-189-01**

Drawing No. **A-13**

Log of Boring No. BH-12

Dates Drilled: 1/9/2013 Logged by: AM Checked By: SM

Equipment: CME 75/ 8" HSA Driving Weight and Drop: 140 lbs / 30 in

Ground Surface Elevation (ft): 2726 Depth to Water (ft): NOT ENCOUNTERED

SUMMARY OF SUBSURFACE CONDITIONS

This log is part of the report prepared by Converse for this project and should be read together with the report. This summary applies only at the location of the boring and at the time of drilling. Subsurface conditions may differ at other locations and may change at this location with the passage of time. The data presented is a simplification of actual conditions encountered.

Depth (ft)	Description	Samples: Drive	Samples: Bulk	Blows	Moisture	Dry Unit Wt. (pcf)	Other
0	**6" ASPHALT CONCRETE / NO AGGREGATE BASE**						
0.5	**SILTY SAND (SM):** fine to coarse-grained, brown.		X				
2		X	X	7/8/5	7	103	
5		X		4/4/4	9	99	
7		X		3/6/8	3	114	
10		X		3/3/6	9	113	ds
15		X		3/4/4			
20		X		10/18/26	4	118	
25		X		7/11/11			

End of Boring at 26.5 feet.
No Groundwater Encountered.
Borehole backfilled with soil cuttings and surface patched with cold asphalt concrete on 1/9/13.

Log of Boring No. BH-13

Dates Drilled: 1/9/2013 Logged by: AM Checked By: SM

Equipment: CME 75/ 8" HSA Driving Weight and Drop: 140 lbs / 30 in

Ground Surface Elevation (ft): 2718 Depth to Water (ft): NOT ENCOUNTERED

Depth (ft)	Graphic Log	SUMMARY OF SUBSURFACE CONDITIONS This log is part of the report prepared by Converse for this project and should be read together with the report. This summary applies only at the location of the boring and at the time of drilling. Subsurface conditions may differ at other locations and may change at this location with the passage of time. The data presented is a simplification of actual conditions encountered.	SAMPLES DRIVE	SAMPLES BULK	BLOWS	MOISTURE	DRY UNIT WT. (pcf)	OTHER
		4" ASPHALT CONCRETE / NO AGGREGATE BASE						
		SILTY SAND (SM): fine to coarse-grained, brown.			8/24/34	3	111	
5					7/8/10	3	93	
					6/7/11	3	109	
10					7/9/12	4	113	
15					8/10/15			
		End of Boring at 16.5 feet. No Groundwater Encountered. Borehole backfilled with soil cuttings and surface patched with cold asphalt concrete on 1/9/13.						

Converse Consultants

Beaumont Avenue Recharge Basin Pipeline
City of Beaumont, Riverside County, California
For: Atkins

Project No. **12-81-189-01**

Drawing No. **A-15**

APPENDIX B

LABORATORY TESTING PROGRAM

APPENDIX B

LABORATORY TESTING PROGRAM

Tests were conducted in our laboratory on representative soil samples for the purpose of classification and evaluation of their physical properties and engineering characteristics. The amount and selection of tests were based on the geotechnical parameters required for this project. Test results are presented herein and on the Logs of Borings, in Appendix A, *Field Exploration*. The following is a summary of the various laboratory tests conducted for this project.

In-Situ **Moisture Content and Dry Density**

Results of these tests performed on relatively undisturbed ring samples were used to aid in the classification and to provide quantitative measure of the *in situ* dry density and moisture content. Data obtained from this test provides qualitative information on strength and compressibility characteristics of the site soils. For test results, see the Logs of Borings in Appendix A, *Field Exploration*.

Collapse Tests

To evaluate the moisture sensitivity (collapse potential) of the encountered soils, two (2) representative ring samples were loaded to approximately two kips per square foot (ksf), allowed to stabilize under load, and then submerged. The test was performed in accordance with ASTM Standard D5333. Results are presented in the following table.

Table No. B-1, Collapse Test Results

Sample Location	Depth (feet)	Soil Type	Collapse (%)
BH-1	2.0-3.5	Silty Sand (SM), yellow brown	2.1
BH-10	2.0-3.5	Silty Sand (SM), brown	2.1

Sand Equivalent

Two (2) representative soil samples were tested in accordance with the ASTM D2419 test method to determine the Sand Equivalent (SE). The test results are presented in the following table.

Table No. B-2, Sand Equivalent Test Results

Boring No.	Depth (feet)	Soil Description	Sand Equivalent
BH-4	0.0-5.0	Silty Sand (SM), light brown	20
BH-8A	0.0-5.0	Sand (SP), brown	76

Soil Corrosivity

One (1) representative soil sample was tested to determine minimum electrical resistivity, pH, and chemical content, including soluble sulfate and chloride concentrations. The purpose of these tests was to determine the corrosion potential of site soils when placed in contact with common pipe materials. These tests were performed by HDR/Schiff Associates. Test results are presented on the following table.

Table No. B-3, Summary of Corrosivity Test Results

Sample Location (Boring/Depth)	pH	Soluble Sulfates (CA 417) (ppm)	Soluble Chlorides (CA 422) (ppm)	Min. Resistivity (CA 643)(Ohm-cm)
BH-8A/ 0.0-5.0	6.9	12	5.1	9,600

Grain-Size Analysis

To assist in classification of soils, mechanical grain-size analyses were performed on selected samples. Testing was performed in general accordance with the ASTM D422 method. Grain-size curves are shown in Drawing No. B-1, *Grain Size Distribution Results*.

Laboratory Maximum Dry Density

Laboratory maximum dry density and optimum moisture content relationship tests were performed on two (2) representative bulk samples. The test was conducted in accordance with ASTM Standard D1557 method. The test results are presented on Drawing No. B-2, *Moisture-Density Relationship Results,* and summarized in the following table.

Table No. B-4, Laboratory Maximum Density Test Results

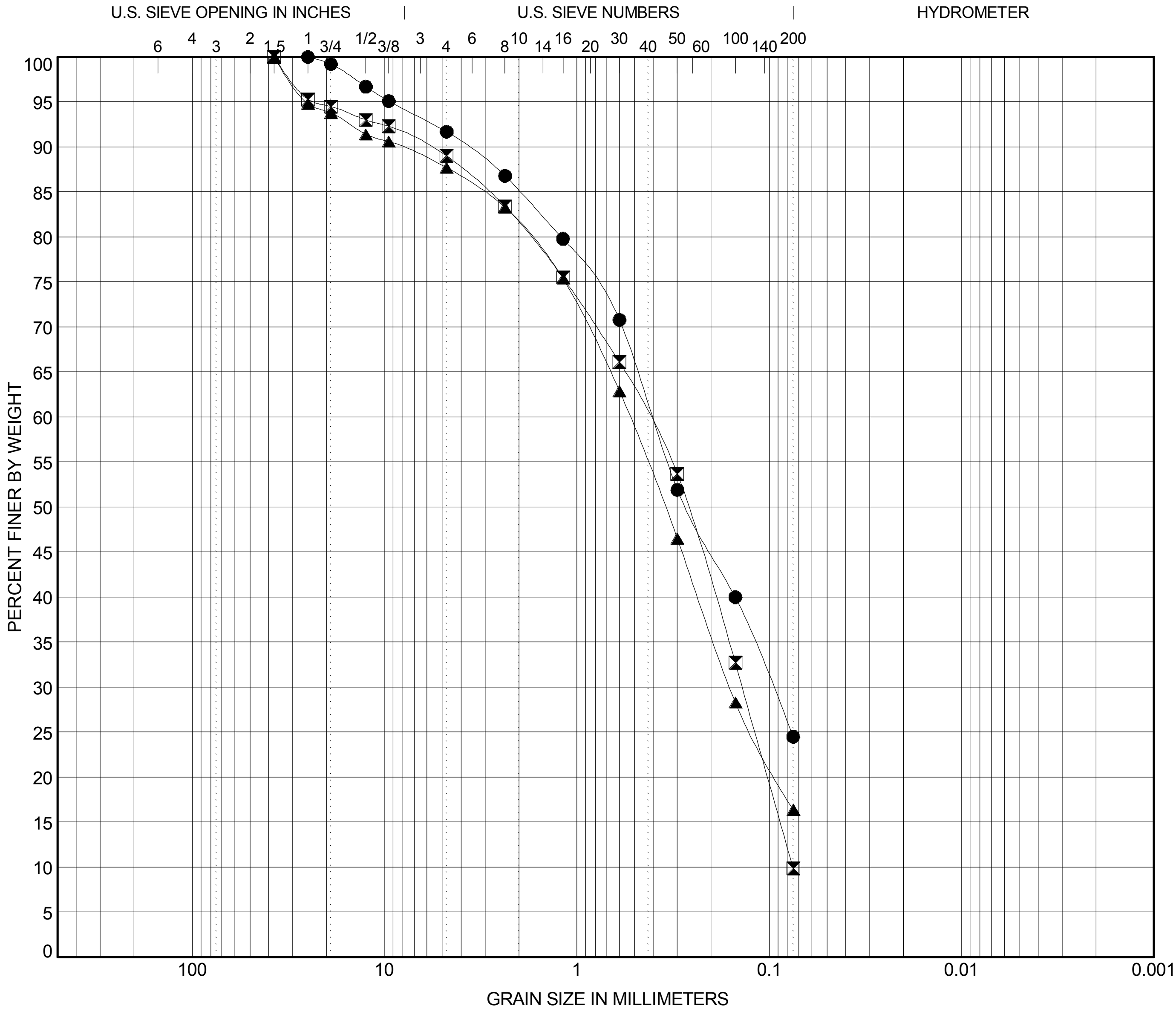
Boring No.	Depth (feet)	Soil Classification	Maximum Dry Density (pcf)	Optimum Moisture (%)
BH-2	0.0-5.0	Silty Sand (SM), dark brown	128.6	9.4
BH-5	0.0-5.0	Silty Sand (SM), brown	126.0	7.0
BH-5	0.0-5.0	Silty Sand (SM), brown, with rock correction	132.7	5.6

Direct Shear

Direct shear tests were performed on relatively undisturbed representative soil samples at soaked moisture conditions per ASTM D3080 method. For each test, three samples contained in a brass sampler ring were placed, one at a time, directly into the test apparatus and subjected to a range of normal loads appropriate for the anticipated conditions. The samples were then sheared at a constant strain rate of 0.05 inch/minute. Shear deformation was recorded until a maximum of about 0.25-inch shear displacement was achieved. Ultimate strength was selected from the shear-stress deformation data and plotted to determine the shear strength parameters. For test results, including sample density and moisture content, see Drawings No. B-3 through B-5*, Direct Shear Test Results,* and the following table.

Table No. B-5, Direct Shear Test Results

Boring No.	Depth (feet)	Soil Classification	Ultimate Strength Parameters	
			Friction Angle (degrees)	Cohesion (psf)
BH-3	5.0-6.5	Silty Sand (SM), yellow brown	31	150
BH-5	7.0-8.5	Sand with Silt (SP-SM), few gravel, yellow brown	31	200
BH-12	10.-11.5	Silty Sand (SM), brown	32	300



COBBLES	GRAVEL		SAND			SILT OR CLAY
	coarse	fine	coarse	medium	fine	

	Boring No.	Depth (ft)	Description	LL	PL	PI	Cc	Cu
●	**BH- 1**	**0-5.0**	**Silty Sand (SM) Yellow-Brown**					
⧗	**BH- 5**	**7.0-11.5**	**Sand with Silt (SP-SM), Yellow-Brown, few gravel**				**0.59**	**5.67**
▲	**BH-11**	**10.0-15.0**	**Silty Sand (SM), Brown, few gravel**					

	Boring No.	Depth (ft)	D100	D60	D30	D10	%Gravel	%Sand	%Silt	%Clay
●	**BH- 1**	**0-5.0**	**25**	**0.404**	**0.096**		**8.3**	**67.2**	**24.5**	
⧗	**BH- 5**	**7.0-11.5**	**37.5**	**0.427**	**0.138**	**0.075**	**11.0**	**79.1**	**9.9**	
▲	**BH-11**	**10.0-15.0**	**37.5**	**0.531**	**0.16**		**12.3**	**71.3**	**16.4**	

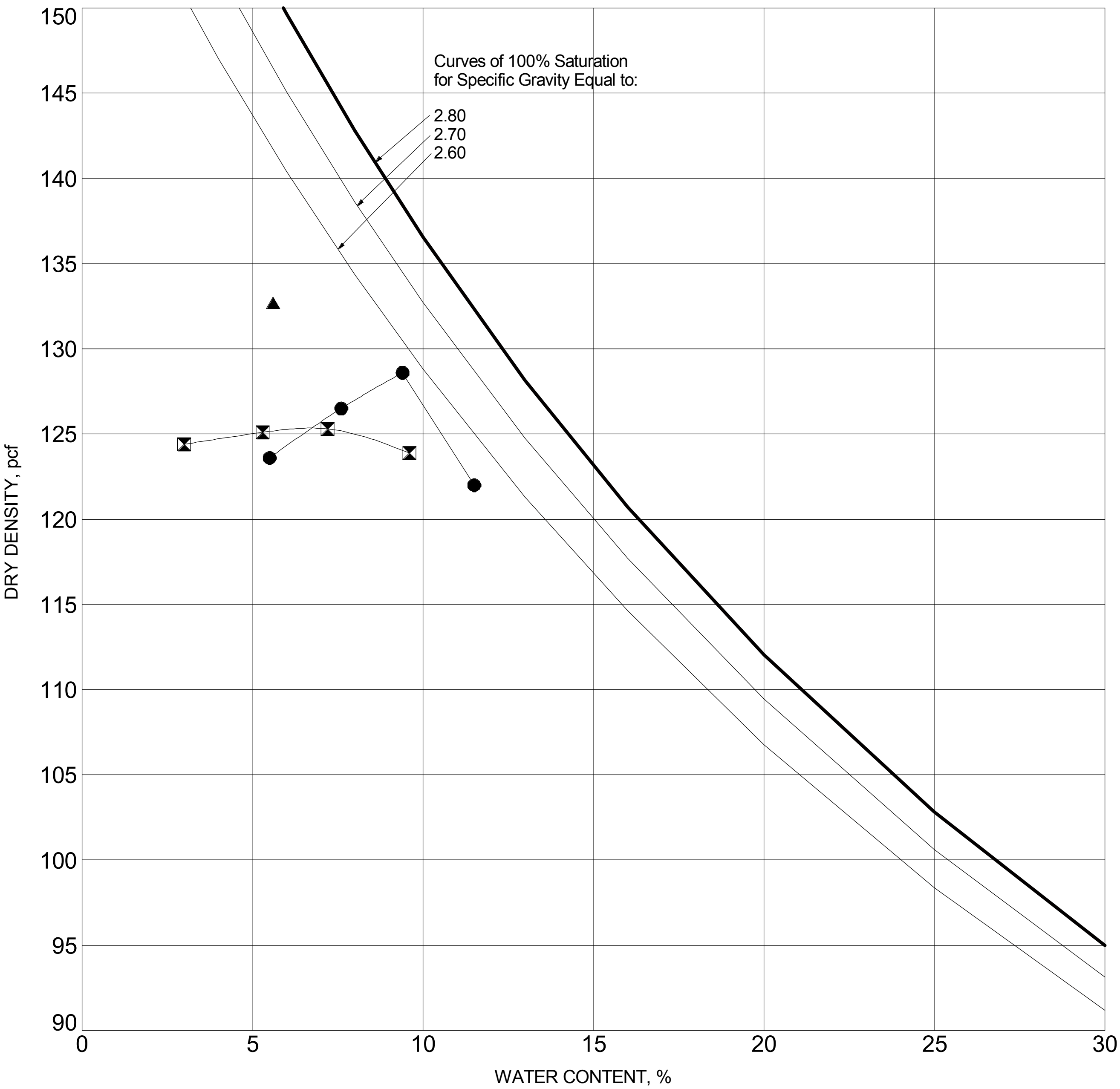
GRAIN SIZE DISTRIBUTION RESULTS

Converse Consultants

Beaumont Avenue Recharge Basin Pipeline
City of Beaumont, Riverside County, California
For: Atkins

Project No. **12-81-189-01**

Drawing No. **B-1**



SYMBOL	BORING NO.	DEPTH (ft)	DESCRIPTION	ASTM TEST METHOD	OPTIMUM WATER, %	MAXIMUM DRY DENSITY, pcf
●	**BH- 2**	**0-5.0**	**Silty Sand (SM) Dark Brown**	**D1557 Method B**	**9.4**	**128.6**
⧗	**BH- 5**	**0-5.0**	**Silty Sand (SM), Brown.**	**D1557 Method B**	**7**	**126**
▲	**BH- 5**	**0-5.0**	**Rock Correction of 20%**	**D1557 Method B**	**5.6**	**132.7**

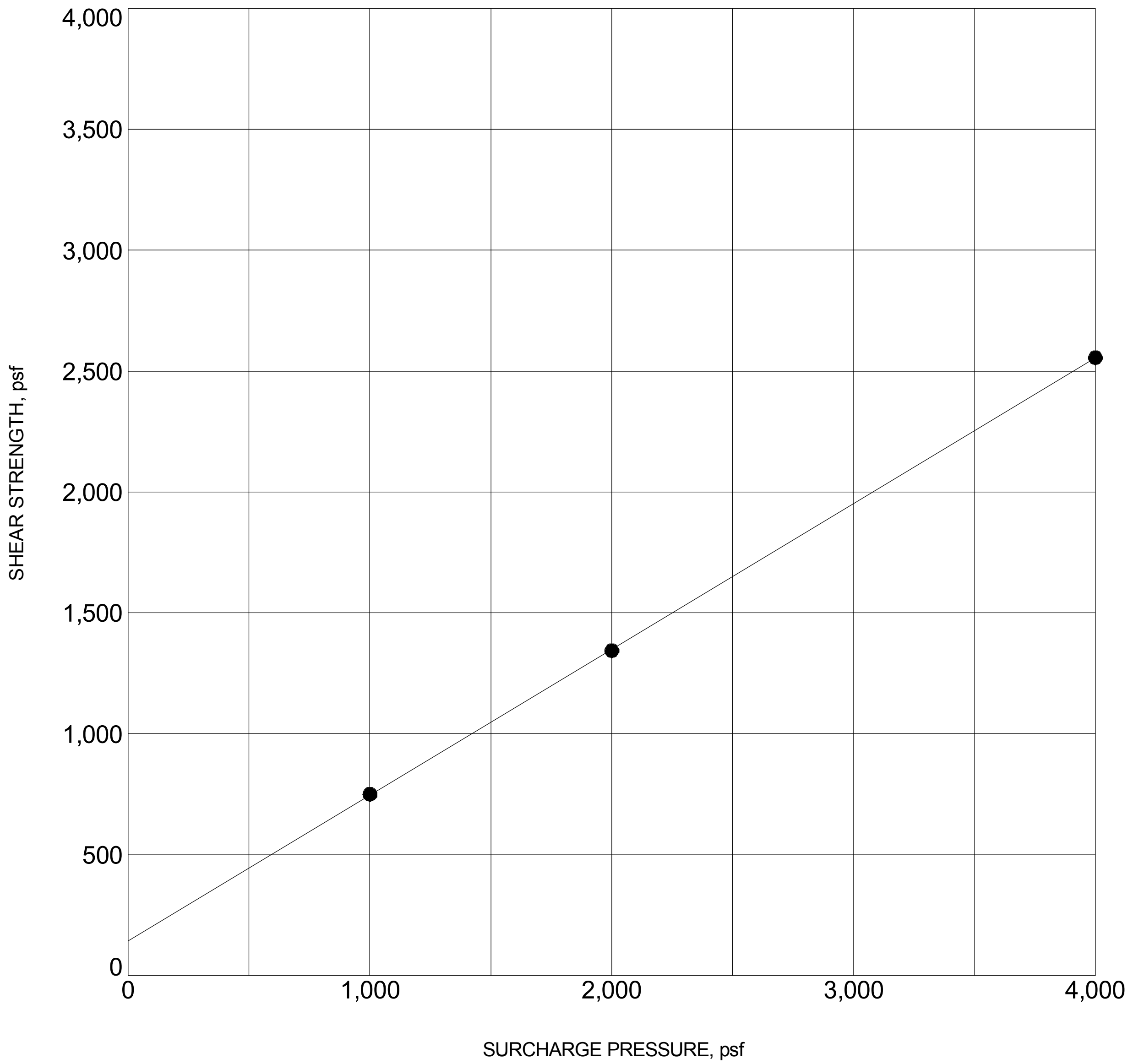
MOISTURE-DENSITY RELATIONSHIP RESULTS

Converse Consultants

Beaumont Avenue Recharge Basin Pipeline
City of Beaumont, Riverside County, California
For: Atkins

Project No.
12-81-189-01

Drawing No.
B-2



BORING NO. :	BH- 3	DEPTH (ft) :	5.0-6.5
DESCRIPTION :	Silty Sand (SM) Yellow Brown		
COHESION (psf) :	150	FRICTION ANGLE (degrees) :	31
MOISTURE CONTENT (%) :	7.0	DRY DENSITY (pcf) :	103.5

NOTE: Ultimate Strength.

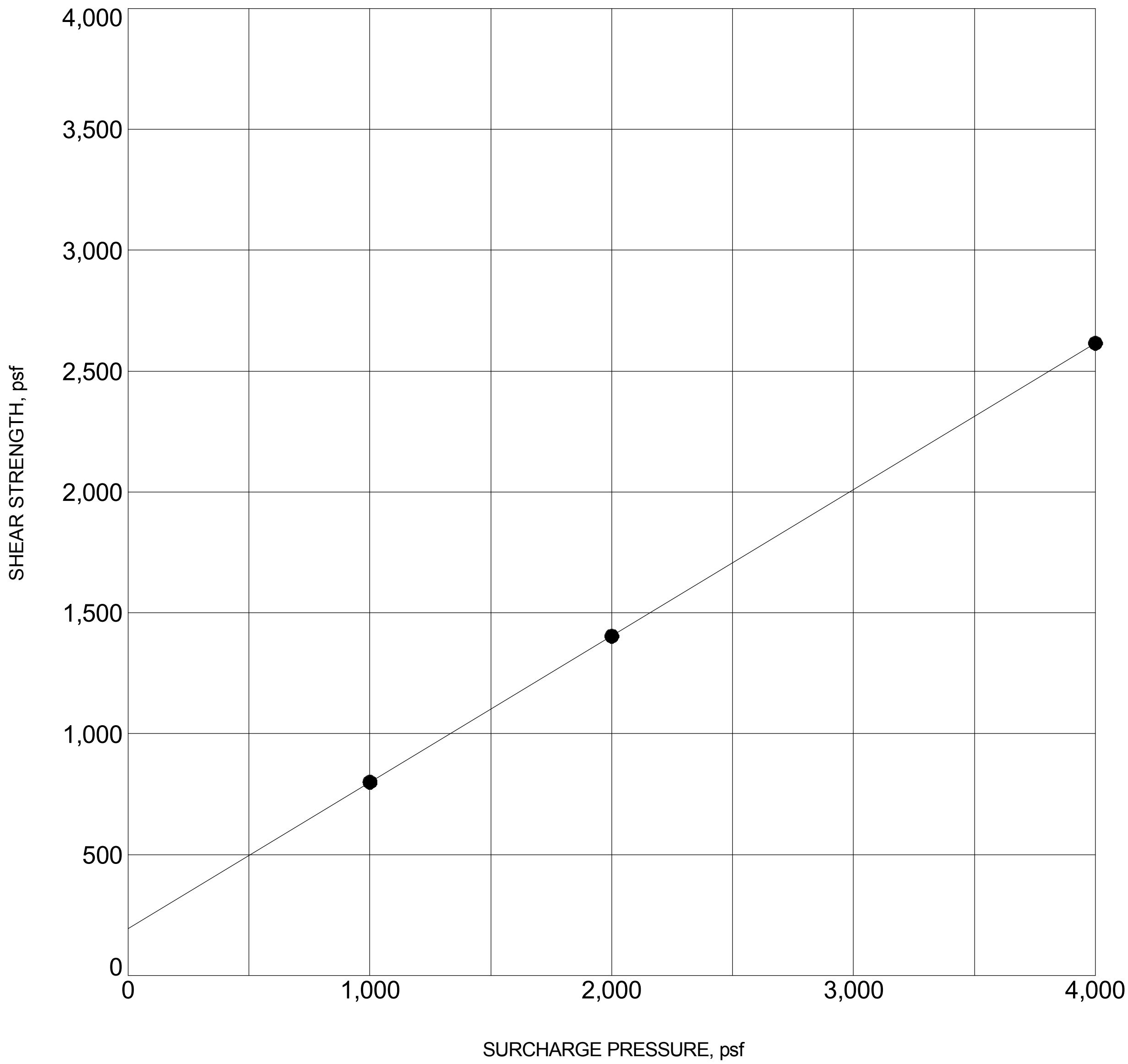
DIRECT SHEAR TEST RESULTS

Converse Consultants

Beaumont Avenue Recharge Basin Pipeline
City of Beaumont, Riverside County, California
For: Atkins

Project No. **12-81-189-01**

Drawing No. **B-3**



BORING NO.	:	**BH- 5**	DEPTH (ft)	:	**7.0-8.5**
DESCRIPTION	:	**Sand with Silt (SP-SM), Yellow-Brown, few gravel**			
COHESION (psf)	:	**200**	FRICTION ANGLE (degrees)	:	**31**
MOISTURE CONTENT (%)	:	**5.7**	DRY DENSITY (pcf)	:	**108.7**

NOTE: Ultimate Strength.

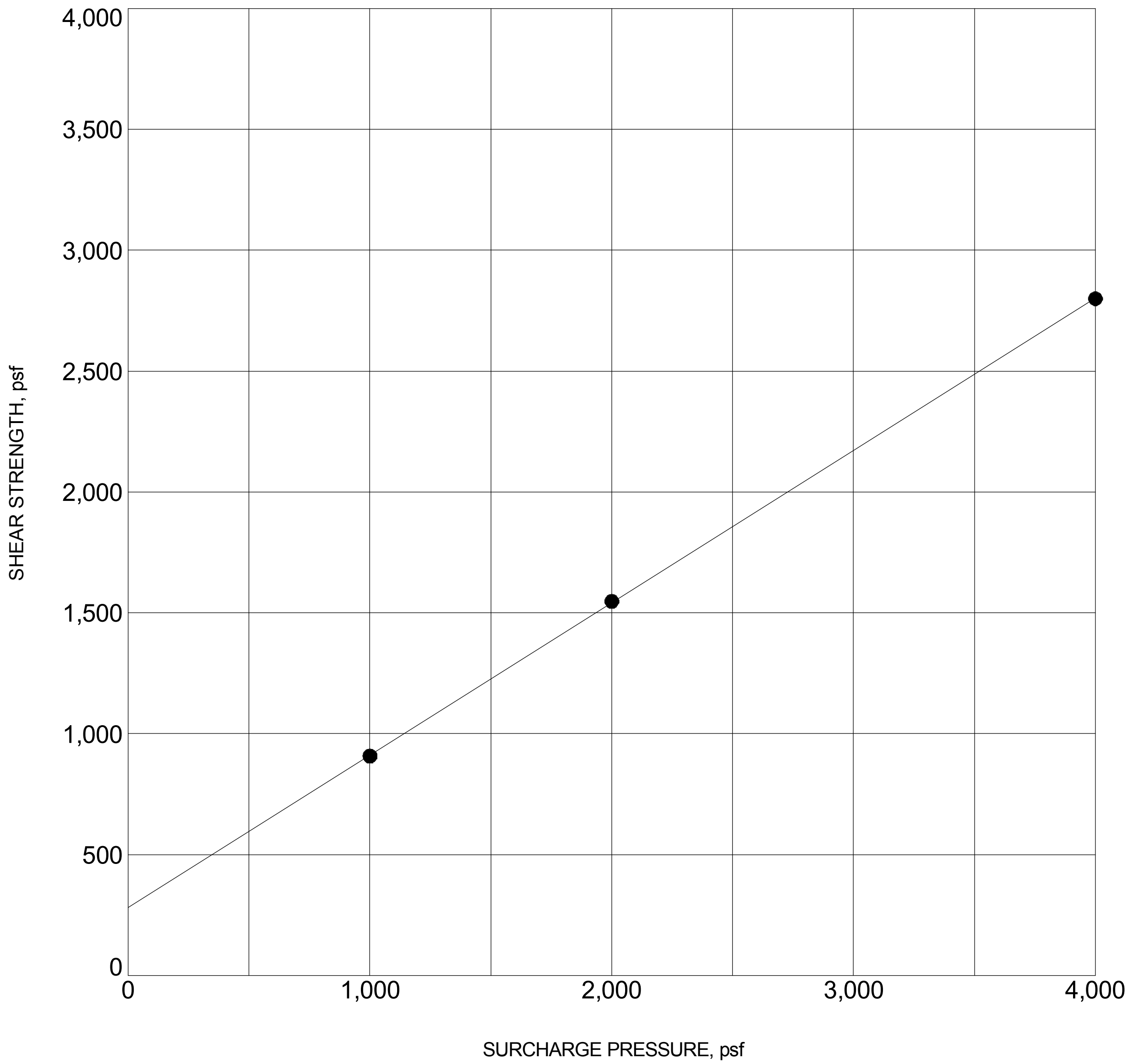
DIRECT SHEAR TEST RESULTS

Converse Consultants

Beaumont Avenue Recharge Basin Pipeline
City of Beaumont, Riverside County, California
For: Atkins

Project No. **12-81-189-01**

Drawing No. **B-4**



BORING NO.	:	**BH-12**	DEPTH (ft)	:	**10.0-11.5**
DESCRIPTION	:	**Silty Sand (SM), Brown**			
COHESION (psf)	:	**300**	FRICTION ANGLE (degrees)	:	**32**
MOISTURE CONTENT (%)	:	**8.5**	DRY DENSITY (pcf)	:	**112.9**

NOTE: Ultimate Strength.

DIRECT SHEAR TEST RESULTS

Converse Consultants

Beaumont Avenue Recharge Basin Pipeline
City of Beaumont, Riverside County, California
For: Atkins

Project No. **12-81-189-01**

Drawing No. **B-5**