APPENDIX H

GEOLOGIC EVALUATIONS AND ESTIMATES

- **H-1 PRELIMINARY STABILITY EVALUATION OF NORTH SLOPE**
- **H-2 EQFAULT ANALYSIS**
- **H-3 REVISED GEOTECHNICAL EVAULATION**

APPENDIX H-1

PRELIMINARY STABILITY EVALUATION OF NORTH SLOPE

April 2, 2009

Project 14828.000.0

Mr. Jerry Canfield
Howard R. Green Company
2550 University Avenue W., Suite 400N
St. Paul, MN 55144

Subject: **Preliminary Stability Evaluation of North Slope**
Sunshine Gas Producers
Landfill Gas to Energy Project
Sunshine Canyon Landfill
Sylmar, California

Dear Mr. Canfield:

At your request, AMEC Geomatrix, Inc. (AMEC) has performed a preliminary stability evaluation of the north slope adjacent to the proposed Landfill Gas to Energy Project (the project) at the Sunshine Canyon Landfill (SCL) in Sylmar, California. The purpose of the preliminary stability evaluation, a summary of known site conditions, and details and results of the evaluation are provided below.

PURPOSE

AMEC is being retained to conduct a geologic and geotechnical investigation for the project. In the process of preparing our original proposal dated February 13, 2009 (and then revised on March 26, 2009), we reviewed available geotechnical reports for the SCL and found that the north-facing slope (down-slope from Flare No. 8, and referred to herein as "the north slope") adjacent to the project site has historically exhibited some slope instability. This could significantly impact the design of the facility layout and the grading required to prepare the site. GeoSyntec (1998) previously evaluated the stability of the north slope as part of their geotechnical report for the construction of Flare No. 8 at the top of the north slope. GeoSyntec indicated the stability of the north slope was slightly below LA County's design criteria and recommended minor grading at the toe of the north slope to increase its stability. Since 1998, two key conditions have changed from those evaluated by GeoSyntec (1998) that impact the stability of the north slope. First, SCL operations re-graded the face and toe of the north slope in 2007-2008. Second, more recent work by A-Mehr (2006, 2008) provides updated geologic and geotechnical models of the SCL site that differ notably from that used by GeoSyntec (1998). The purpose of our preliminary work was to re-evaluate the stability of the north slope using the re-graded condition of the north slope and the updated geologic and geotechnical models by A-Mehr (2006, 2008).

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CURRENT UNDERSTANDING OF SITE CONDITIONS

AMEC visited the site twice in preparation of our proposal. The first visit was with the Sunshine project team and the second visit was to obtain more site information, including reviewing existing reports in the SCL library and meeting with Ms. Susan Jennings of SCL. With the assistance of Howard R. Green Company, AMEC has obtained relevant information from the following existing reports:

- PRA Group (1991) – Geotechnical Study for Proposed Water Tank Pad: This study provided limited subsurface information near and on top of the ridge of the north slope.

- GeoSyntec Consultants (1998) – Geotechnical Recommendations for Flare No. 8 Pad Construction: This primarily evaluated the static and seismic stability of the south and north descending slopes of the ridge and provided grading recommendations for the ridge top. GeoSyntec indicated a portion of the north slope did not meet stability criteria and provided conceptual grading plans for a small soil buttress at the toe of the north slope. It is not known whether the buttress was constructed. GeoSyntec also mapped an inferred landslide on a portion of the north slope.

- GeoSyntec Consultants (2001) – Phase II-C Grading Design: This document included the results of a geologic investigation and static and seismic stability evaluation of the proposed base grade slopes in Phase II-C (located directly south and east of the upper site).

- A-Mehr (2006) – Geologic Report and Slope Stability Analyses for Phases V-VIII: This revised report (revisions were in response to comments from the Los Angeles County Department of Public Works, California Regional Water Quality Control Board, and California Integrated Waste Management Board) provides results of a geologic investigation and stability analyses for proposed cut slopes along the eastern boundary of the County expansion of the SCL, which is approximately 2400 feet east of our project site.

- A-Mehr (2008) – Final Report of Construction Quality Assurance for Phase V-A: This report contains grading information (i.e., overexcavation limits and fill compaction test results) for Phase V-A, which includes a portion of the site.

AMEC has reviewed portions of the above reports and used relevant information in preparing our proposal and completing the preliminary stability evaluation. Based on our current understanding of site conditions, the key issues for re-evaluating the north slope are:

1. The project area and north slope lie within an earthquake-induced landslide zone designated by the California Geological Survey (CGS). As a consequence, LA

County will require our report to address all relevant issues in Special Publication (SP) 117 developed by CGS, including evaluating the stability of the north slope.

2. The SCL operations have re-graded the north slope. Some areas of slope appear flatter and a portion of the slope is steeper where landslide material has been removed (as indicated by Susan Jennings of SCL).

3. There are two important discrepancies between the GeoSyntec (1998) and A-Mehr (2006, 2008) reports, which are the dip of the bedrock bedding in the north slope and the cross-bedded strength of the bedrock materials at the project site. Both may have a significant impact on the stability of the north slope. A-Mehr (2008) indicates the beds are dipping steeper than reported by GeoSyntec (1998). GeoSyntec used significantly higher bedrock strengths than A-Mehr (2006) for the site. Based on our review, we anticipate the stability results for the north slope by GeoSyntec (1998) may be unconservative. The strengths used by A-Mehr (2006) are those recommended by USGS, are the most recently used at the SCL, and would need to be used by AMEC for the project if no bedrock strength testing was performed. We have included, as an optional scope in our proposal, to specifically evaluate the strength of bedrock in the north slope.

PRELIMINARY STABILITY ANALYSIS

A short summary of the methodology, shear strengths, and results of our preliminary stability analysis of the north slope are provided below.

Methodology

Per the project team's request, AMEC performed preliminary stability analyses on the north slope using the A-Mehr (2006) bedrock strengths and varying the dip of the bedrock bedding to encompass the difference in dips reported by GeoSyntec (1998) and A-Mehr (2008). Two-dimensional limit-equilibrium analyses were performed to evaluate the global stability of the north slope and compute a Factor of Safety (FS) against sliding. The computer program Slope/W (Geo-Slope, 2004) was used to perform Spencer's limit-equilibrium analysis method because it satisfies both force and moment equilibrium, and accounts for inter-slice forces. Slope/W is a commercially available computer program with a comprehensive formulation that makes it possible to analyze complex geometric configurations and loading conditions.

In terms of slope stability, the FS against sliding is defined as the ratio of resulting forces (friction and cohesion along a potential failure surface) to driving forces (gravitational forces pulling downslope). A FS of unity (1.0) indicates a delicate balance between the resisting and driving forces and represents incipient failure. Factors of Safety below unity indicate instability. For the limit-equilibrium analyses, the minimum static FS of slope stability was evaluated. The calculated static FS was compared to LA County's criterion of a FS ≥ 1.5 for long-term static stability.

AMEC preliminarily evaluated the static stability of the north slope at two cross sections for several different conditions. These conditions included:

- variation in dipping of bedding,
- variation in location of potential clay seams within the north slope,
- with and without the presence of clay seams, and
- block- and circular-type failure surfaces.

Shear Strength Parameters

The key shear strengths that control the stability of the north slope are associated with interbedded clay seams and across bedding within the Towsley Formation. The strength of the clay seams at the SCL site has been thoroughly evaluated and reported by several consultants. The shear strength parameters used for the clay seams are a cohesion of 400 pounds per square foot (psf) and a friction angle of 14 degrees. These strengths were obtained by back-calculating pre-existing landslides in the SCL area and have been used in several geotechnical investigations at the SCL. It appears these consultants have assumed in their stability analyses that clay seams can be present anywhere within the slopes analyzed. The regulatory agencies (including LA County) are familiar with the clay seam strengths and have approved consultant reports using these strengths. As such, we used the established clay seam strength in our preliminary stability analyses and plan to use them in subsequent analyses.

As discussed above, at least two different cross-bedded bedrock strengths of the Towsley Formation that have been used to evaluate the stability of existing and cut slopes at SCL: one by GeoSyntec (1998, 2001) and one by A-Mehr (2006). GeoSyntec performed unconfined compression and UU triaxial tests on bedrock samples collected from the sedimentation basins in 1997. Results of these tests provided very high bedrock strengths. A-Mehr (2006) used much lower bedrock strengths provided in USGS Open File Report 98-113 for a small study area that includes the SCL site. The USGS publication compiled results from numerous direct shear strength tests and opinions from many experienced professionals in the area. The USGS bedrock strengths represent a broad agreement from many sources and have been considered and reviewed by LA County. For these reasons, we used the same cross-bedded bedrock strength parameters as used in A-Mehr (2006) which were a cohesion of 550 psf and a friction of 34 degrees. We will use these same strengths in subsequent analyses unless we perform bedrock strength testing specific to the north slope (optional scope in our proposal).

Results

AMEC evaluated approximately 10 stability scenarios for our preliminary analysis, which represent combinations of the different conditions described in the methodology section above. Results of our analysis indicate the north slope has a FS ≤ 1.0 for most of the scenarios analyzed, which means portions of the north slope are at incipient failure (as graded) or the current geologic model and/or material strengths may need some small revisions. The landslide on the north slope previously identified by GeoSyntec (1998) supports the instability of this slope. In one scenario, we unconservatively assumed no shallow clay seams exist in the north slope. The FS for this scenario was approximately 1.25, which is also below the design criteria required by LA County. Based on these results, we make the following observations:

- The north slope likely will not meet stability design criteria and will need to be stabilized.
- The lower bedrock strengths by A-Mehr (2006) significantly lower the FS against landsliding.
- The dip of bedding has notable effects on the FS of the slope and the amount of stabilization required. We will use the results of our proposed field exploration program to try to resolve the discrepancy in dip of bedding.
- The presence and locations of clay seams within the north slope can significantly influence the FS and amount of stabilization required. One objective of our field exploration program is to evaluate the quantity and extent of clay seams within the north slope.
- The removal of landslide material by SCL has oversteeped a portion of the north slope and has reduced stability.
- Results of our field exploration program could significantly affect the results of our stability analyses and scope and cost of potential stabilization measures.

As part of our preliminary analysis, we also evaluated a couple of rough-order-magnitude mitigation scenarios to stabilize the north slope using a soil buttress. The soil buttress was configured (in section) to provide sufficient flat area to build the facility as shown in Scheme 6A. These results suggest the existing canyon (where the facility is proposed) may need to be backfilled to near the mid-height of the north slope to achieve adequate FS. These results should be considered preliminary and may change based on the geologic model we develop as part our investigation.

REFERENCES

A-Mehr, Inc., 2006, Geologic Report and Slope Stability Analyses Sunshine Canyon County Extension Landfill Phases V-VII, Report prepared for Browning-Ferris Industries, November.

A-Mehr, Inc., 2008, Final Report of Construction Quality Assurance for Sunshine Canyon County Extension Landfill Phase V-A, Report prepared for Browning-Ferris Industries, July.

Geo-Slope International, Ltd., 2004, Slope/W, GeoStudio 2004: Software for Geotechnical and GeoEnvironmental Modeling, Calgary.

GeoSyntec Consultants, 1998, Geotechnical Recommendations Flare No. 8 Pad Construction, Report prepared for Browning-Ferris Industries of California, Inc., June.

GeoSyntec Consultants, 2001, Phase II-C Grading Design Sunshine Canyon County Extension Landfill, Report prepared for Browning-Ferris Industries, February.

PRA Group Consulting Engineers, 1991, Geotechnical Study Proposed Water Tank Pad Sunshine Canyon Landfill Extension, Report prepared for Browning-Ferris Industries of California, Inc., October 16.

We appreciate the opportunity to provide our services to the project team. If you have any questions regarding the preliminary stability evaluation described herein, please do not hesitate to call the undersigned at (949) 642-0245.

Sincerely,
AMEC Geomatrix, Inc.

Timothy C. Keuscher

Timothy C. Keuscher, PE, GE
Principal Geotechnical Engineer

APPENDIX H-2

EQFAULT ANALYSIS

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                    **********************
                    *                    *
                    *    E Q F A U L T   *
                    *                    *
                    *    Version 3.00    *
                    *                    *
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DETERMINISTIC ESTIMATION OF
PEAK ACCELERATION FROM DIGITIZED FAULTS

JOB NUMBER: 9842-0000

DATE: 04-01-2010

JOB NAME: Test Run

CALCULATION NAME: Test Run Analysis

FAULT-DATA-FILE NAME: C:\Program Files\EQFAULT1\CGSFLTE.DAT

```
SITE COORDINATES:
   SITE LATITUDE:  34.3300
   SITE LONGITUDE:  118.5200

SEARCH RADIUS:   100  mi

ATTENUATION RELATION:  23) Abrahamson & Silva (1995b/1997) Horiz.- Soil
   UNCERTAINTY (M=Median, S=Sigma): M       Number of Sigmas:  0.0
   DISTANCE MEASURE:  clodis
   SCOND:   0
   Basement Depth:  5.00 km     Campbell SSR:        Campbell SHR:
   COMPUTE PEAK HORIZONTAL ACCELERATION

FAULT-DATA FILE USED:  C:\Program Files\EQFAULT1\CGSFLTE.DAT


MINIMUM DEPTH VALUE (km):  0.0
```

EQFAULT SUMMARY

DETERMINISTIC SITE PARAMETERS

Page 1

ABBREVIATED FAULT NAME	APPROXIMATE DISTANCE mi (km)	ESTIMATED MAX. EARTHQUAKE EVENT: MAXIMUM EARTHQUAKE MAG.(Mw)	PEAK SITE ACCEL. g	EST. SITE INTENSITY MOD.MERC.
SANTA SUSANA	0.9(1.5)	6.7	0.622	X
SIERRA MADRE (San Fernando)	2.7(4.3)	6.7	0.526	X
NORTHRIDGE (E. Oak Ridge)	3.1(5.0)	7.0	0.523	X
SAN GABRIEL	5.1(8.2)	7.2	0.353	IX
HOLSER	5.8(9.4)	6.5	0.347	IX
VERDUGO	7.4(11.9)	6.9	0.323	IX
SIMI-SANTA ROSA	9.3(15.0)	7.0	0.282	IX
OAK RIDGE (Onshore)	12.4(20.0)	7.0	0.227	IX
SIERRA MADRE	13.7(22.0)	7.2	0.224	IX
SAN CAYETANO	15.7(25.2)	7.0	0.190	VIII
HOLLYWOOD	17.0(27.3)	6.4	0.145	VIII
SANTA MONICA	18.7(30.1)	6.6	0.143	VIII
UPPER ELYSIAN PARK BLIND THRUST	19.8(31.8)	6.4	0.127	VIII
MALIBU COAST	19.9(32.1)	6.7	0.140	VIII
NEWPORT-INGLEWOOD (L.A.Basin)	21.2(34.1)	7.1	0.125	VII
RAYMOND	21.9(35.3)	6.5	0.120	VII
ANACAPA-DUME	22.3(35.9)	7.5	0.172	VIII
PUENTE HILLS BLIND THRUST	22.4(36.1)	7.1	0.147	VIII
SAN ANDREAS - Mojave M-1c-3	23.5(37.8)	7.4	0.129	VIII
SAN ANDREAS - Whole M-1a	23.5(37.8)	8.0	0.162	VIII
SAN ANDREAS - 1857 Rupture M-2a	23.5(37.8)	7.8	0.150	VIII
SAN ANDREAS - Cho-Moj M-1b-1	23.5(37.8)	7.8	0.150	VIII
SAN ANDREAS - Carrizo M-1c-2	25.5(41.0)	7.4	0.121	VII
PALOS VERDES	26.6(42.8)	7.3	0.112	VII
CLAMSHELL-SAWPIT	26.7(43.0)	6.5	0.100	VII
SANTA YNEZ (East)	28.0(45.0)	7.1	0.099	VII
VENTURA - PITAS POINT	35.5(57.1)	6.9	0.092	VII
WHITTIER	37.1(59.7)	6.8	0.068	VI
M.RIDGE-ARROYO PARIDA-SANTA ANA	38.1(61.3)	7.2	0.099	VII
GARLOCK (West)	39.8(64.0)	7.3	0.081	VII
SAN JOSE	40.7(65.5)	6.4	0.064	VI
PLEITO THRUST	40.9(65.8)	7.0	0.085	VII
CUCAMONGA	43.3(69.7)	6.9	0.077	VII
OAK RIDGE(Blind Thrust Offshore)	43.6(70.2)	7.1	0.085	VII
RED MOUNTAIN	43.6(70.2)	7.0	0.081	VII
OAK RIDGE MID-CHANNEL STRUCTURE	43.6(70.2)	6.6	0.066	VI
CHANNEL IS. THRUST (Eastern)	43.6(70.2)	7.5	0.103	VII
BIG PINE	43.8(70.5)	6.9	0.061	VI
CHINO-CENTRAL AVE. (Elsinore)	47.7(76.7)	6.7	0.064	VI
SAN JOAQUIN HILLS	55.2(88.9)	6.6	0.053	VI

DETERMINISTIC SITE PARAMETERS

Page 2

ABBREVIATED FAULT NAME	APPROXIMATE DISTANCE mi (km)	ESTIMATED MAX. EARTHQUAKE EVENT: MAXIMUM EARTHQUAKE MAG.(Mw)	PEAK SITE ACCEL. g	EST. SITE INTENSITY MOD.MERC.
WHITE WOLF	55.4(89.2)	7.3	0.077	VII
SAN JACINTO-SAN BERNARDINO	58.0(93.4)	6.7	0.043	VI
SAN ANDREAS - SB-Coach. M-1b-2	58.3(93.9)	7.7	0.073	VII
SAN ANDREAS - San Bernardino M-1	58.3(93.9)	7.5	0.065	VI
SAN ANDREAS - SB-Coach. M-2b	58.3(93.9)	7.7	0.073	VII
ELSINORE (GLEN IVY)	60.1(96.8)	6.8	0.044	VI
CLEGHORN	60.3(97.0)	6.5	0.037	V
SANTA CRUZ ISLAND	61.5(98.9)	7.0	0.060	VI
NEWPORT-INGLEWOOD (Offshore)	61.6(99.2)	7.1	0.050	VI
SANTA YNEZ (West)	64.3(103.5)	7.1	0.048	VI
NORTH CHANNEL SLOPE	65.6(105.5)	7.4	0.070	VI
NORTH FRONTAL FAULT ZONE (West)	71.4(114.9)	7.2	0.059	VI
GARLOCK (East)	71.7(115.4)	7.5	0.055	VI
HELENDALE - S. LOCKHARDT	74.1(119.2)	7.3	0.048	VI
LENWOOD-LOCKHART-OLD WOMAN SPRGS	75.2(121.0)	7.5	0.053	VI
SAN JACINTO-SAN JACINTO VALLEY	76.6(123.2)	6.9	0.037	V
CORONADO BANK	80.8(130.0)	7.6	0.053	VI
SANTA ROSA ISLAND	81.9(131.8)	7.1	0.049	VI
ELSINORE (TEMECULA)	82.0(132.0)	6.8	0.032	V
So. SIERRA NEVADA	88.2(141.9)	7.3	0.052	VI
GRAVEL HILLS - HARPER LAKE	88.3(142.1)	7.1	0.036	V
LOS ALAMOS-W. BASELINE	90.5(145.7)	6.9	0.040	V
BLACKWATER	97.1(156.2)	7.1	0.033	V
NORTH FRONTAL FAULT ZONE (East)	98.0(157.7)	6.7	0.032	V
SAN JUAN	99.7 (160.5)	7.1	0.033	V

-END OF SEARCH- 65 FAULTS FOUND WITHIN THE SPECIFIED SEARCH RADIUS.

THE SANTA SUSANA FAULT IS CLOSEST TO THE SITE.
IT IS ABOUT 0.9 MILES (1.5 km) AWAY.

LARGEST MAXIMUM-EARTHQUAKE SITE ACCELERATION: 0.6219 g

APPENDIX H-3

REVISED GEOTECHNICAL EVALUATION

GEOLOGIC AND GEOTECHNICAL INVESTIGATION REPORT

Sunshine Gas Producers

Landfill Gas to Energy Project

Sunshine Canyon Landfill

Sylmar, California

Prepared for:

HR Green, Inc.
2550 University Avenue West,
Suite 400N
St. Paul, Minnesota 55144

Prepared by:

AMEC
510 Superior Avenue, Suite 200
Newport Beach, California 92663
(949) 642-0245

November 15, 2011

Project No. 14828.000.0

amec

GEOLOGIC AND GEOTECHNICAL INVESTIGATION REPORT
Sunshine Gas Producers
Landfill Gas to Energy Project
Sunshine Canyon Landfill
Sylmar, California

November 15, 2011
Project 0148280000

This report was prepared by the staff of AMEC under the supervision of the Engineer(s) and/or Geologist(s) whose seal(s) and signature(s) appear hereon.

The findings, recommendations, specifications, or professional opinions are presented within the limits described by the client, in accordance with generally accepted professional engineering and geologic practice. No warranty is expressed or implied.

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GEOLOGIC AND GEOTECHNICAL INVESTIGATION REPORT
Sunshine Gas Producers
Landfill Gas to Energy Project
Sunshine Canyon Landfill
Sylmar, California

1.0 INTRODUCTION

AMEC Geomatrix, Inc. (AMEC) conducted a geologic and geotechnical investigation for the future Landfill Gas to Energy Project (LGEP) on behalf of HR Green, Inc. (HRGreen) and Sunshine Gas Producers (SGP). The project site is located at the Sunshine Canyon Landfill (SCL) in Sylmar, California. HRGreen is contracted by SGP and is currently designing the LGEP and developing the layout plans. Our investigation incorporated the latest version of the plans (Scheme 10B) available at the time of the investigation. This report presents the results of the geologic and geotechnical investigation performed by AMEC. The location of the site is shown on Figure 1.

AMEC performed this geologic and geotechnical investigation on behalf of HRGreen and SGP and in general accordance with the agreement between AMEC and HRGreen. The key objectives of the geotechnical investigation were to:

1. Characterize the geology, soil, and groundwater conditions within the project site;

2. Evaluate static and seismic stability of the slopes descending into the project site; and

3. Provide geotechnical design recommendations for the proposed facilities.

To accomplish above objectives, AMEC (with the assistance of several subcontractors) performed the following scope of work:

- Compiled and reviewed available geotechnical reports specific to the vicinity of the project site. Reviewed pertinent, available geologic and geotechnical information contained in the files of public agencies such as the California Geological Survey (CGS) and United States Geological Survey (USGS);

- Performed field exploration consisting of geologic mapping of existing bedrock exposures, and logging of bucket auger borings, rock core boring, and hollow-stem auger borings;

- Prepared a geologic map of the project site and surrounding area;
- Performed laboratory testing to characterize the engineering properties of existing and proposed fill materials and of bedrock encountered at the site;
- Performed geologic and geotechnical engineering analyses and developed geotechnical recommendations; and
- Prepared this geologic and geotechnical investigation report.

2.0 PROJECT DESCRIPTION AND BACKGROUND

The proposed project is situated in the northerly portion of the SCL site. SCL is located in the easterly margins of the Santa Susana Mountains immediately north of the community of Granada Hills in Los Angeles County. Primary access to the project site is from the south is by way of the main landfill entrance located at the intersection of San Fernando Road and Sunshine Canyon Road and by an unpaved access road that traverses the perimeter of the active landfill. Northerly access is by unpaved roads that include the Weldon Canyon and Sunshine Motorways, which extend south from Coltrane Avenue and the Golden State Freeway to the north.

2.1 TOPOGRAPHIC SETTING

The SCL is situated within the easterly margins of the Santa Susanna Mountains, a range of roughly east-west trending foothills and mountainous terrain that forms the northerly boundary of the San Fernando Valley. The easterly portion of the mountains is dominated by Oat Mountain, a continuous northwest-southeast trending ridge that descends from elevated terrain to the west within Ventura County to the broad alluvial surface occupied by the community of Sylmar to the east. The proposed project site is at the bottom of a northwest-southeast trending narrow and steep-sided canyon located at the northerly perimeter of the landfill. The ridge immediately to the northeast of the canyon separates the site and the landfill from the Golden State Freeway to the northeast. The south-facing slope of this ridgeline descends into the project area and is designated “the South Slope” in this report. The ridge to the southwest is occupied by Flare Station No.8 and the unpaved access road that descends from the Weldon Canyon-Sunshine Canyon Motorway to the active landfill area. The north-facing slope of this ridge descends into the project area and is designated “the North Slope” in this report. The North and South Slopes are labeled on Figure 2.

2.2 PREVIOUS DEVELOPMENT

Grading operations associated with a previous expansion of the landfill have resulted in cuts and fills within the canyon and on the nose of the ridge line to the southwest that is occupied by Flare Station No.8. Based on our review of available geotechnical reports, grading within these areas included the following:

- removal of unsuitable earth materials along the bottom and slopes of the canyon (primarily on the North Slope);
- placement of subdrains along the bottom of the canyon;
- placement of compacted fill within the canyon bottom and on a portion of the North Slope;
- placement of compacted fill for construction of an access road (the road fill) on the North Slope that descends from the flare station to the bottom of the canyon.
- Cutting of the nose of the southwesterly ridge for grading of the flare station access road.

Grading for the Flare Station No.8 access road and adjacent shoulder resulted in a filled area at the bottom of the canyon approximately 45 to 125-feet wide. The first 500-feet of the access road inclines to the south at a gradient of approximately 9%. The grading for the access road as it ascends from this area up the North Slope has created fill slopes as much as 125 feet high inclined at gradients typically varying from 1.5H:1V (horizontal:vertical) to 2H:1V.

Associated grading operations within the canyon, for the apparent removal of colluvial debris, have left a series of smaller cut slopes along the South Slope. These slopes are as much as 20-feet high and are inclined at a gradient of approximately 1H:1V.

2.3 PROPOSED DEVELOPMENT

Based on the current proposed layout for the LGEP (Scheme 10B) shown in Figure 2, the facility will be sited at the mouth of the canyon. Planned facilities include blowers, aftercoolers, chillers, screw compressors, a regen flare, a plant air system, siloxane removal skids, turbine chillers, several electrical transformers, pumps, combustion turbine generators, a water storage tank, several control, monitoring, and/or maintenance buildings, and two substations. According to HRGreen, these structures will be lightly loaded and supported on either shallow spread footings or mat foundations with bearing pressures typically 500 pounds per square foot (psf) or less. A few transformers will have bearing pressures of less than 1000 psf and the fire water storage tank will impose the highest bearing pressure of approximately 1700 psf.

The majority of the facility will be situated over a main pad graded to approximate elevation 1900 feet by filing the canyon floor with up to 50 feet of additional engineered fill. The main pad will include one substation and a second substation for Southern California Edison (SCE) will be at the eastern toe of the main pad fill on an existing fill pad. An access road will be constructed by placing 10 to 20 feet of fill over the existing east facing slopes of the southwesterly ridge. Construction of the main pad and access road will create an east-facing fill slope on the east side of the main pad. The fill slope will be from approximately 5 to 55 feet high and approximately 700 feet long. Due to site constraints, the proposed fill slope will be inclined at approximately 1.5H:1V. Construction of the main pad will also create an approximately 25-foot high 3H:1V west-facing fill slope at the west end of the main pad. The proposed fill slopes are shown on Figures 2 and 3.

2.4 PREVIOUS GEOTECHNICAL INVESTIGATIONS

With the assistance of HRGreen, AMEC has obtained relevant information for the general site area from the following existing reports:

- PRA Group (1991) – Geotechnical Study for Proposed Water Tank Pad: This study provided limited subsurface information near and on top of the ridge of the North Slope.

- GeoSyntec Consultants (1998) – Geotechnical Recommendations for Flare No. 8 Pad Construction: This primarily evaluated the static and seismic stability of the south and north descending slopes (includes North Slope) of the ridge and provided grading recommendations for the ridge top. GeoSyntec indicated a portion of the North Slope did not meet stability criteria and provided conceptual grading plans for stabilizing the North Slope.

- GeoSyntec Consultants (2001) – Phase II-C Grading Design: This document included the results of a geologic investigation and static and seismic stability evaluation of the proposed base grade slopes in SCL's Phase II-C (located directly south and east of the project site).

- A-Mehr (2006) – Geologic Report and Slope Stability Analyses for Phases V-VIII: This revised report (revisions were in response to comments from the Los Angeles County Department of Public Works, California Regional Water Quality Control Board, and California Integrated Waste Management Board) provides results of a geologic investigation and stability analyses for proposed cut slopes along the eastern boundary of the County expansion of the SCL, which is approximately 2400 feet east of the project site.

- A-Mehr (2008) – Final Report of Construction Quality Assurance for Phase V-A: This report contains grading information (i.e., overexcavation limits and fill compaction test results) for Phase V-A, which includes a portion of the site.

AMEC reviewed the above reports and used relevant information to augment our investigation. From those reports, some key geotechnical issues to be addressed for the project site are:

1. The project area lies within an earthquake-induced landslide zone designated by the California Geological Survey (CGS).

2. The SCL operations have re-graded the North Slope since the GeoSyntec's (1998) report. Some areas of slope appear flatter and a portion of the slope steeper than before grading. Grading of the North Slope included removal of unsuitable material and placement of fill along portions of the slope to create the current access road to the flare station.

3. There are two important discrepancies between the GeoSyntec (1998) and A-Mehr (2006, 2008) reports, which are the dip of the bedrock bedding in the North Slope and the cross-bedded strength of the bedrock materials at the project site. Both significantly influence the stability of the North Slope. A-Mehr (2008) indicates the beds are dipping steeper than reported by GeoSyntec (1998). GeoSyntec used significantly higher bedrock strengths than A-Mehr (2006) for the site.

Based on review of existing information and the proposed plans for the LGEP, AMEC developed a field exploration and laboratory testing program to evaluate the potential for landsliding, the current geologic conditions and stability of the North and South Slopes, and the strength of bedrock at the project site.

3.0 FIELD INVESTIGATION AND LABORATORY TESTING

The field investigation and laboratory testing program consisted of drilling ten exploratory borings, geologic mapping of the terrain in the vicinity of the project area, and laboratory testing of soil samples retrieved during exploratory drilling.

In addition, bulk samples were obtained from stockpiles at the landfill site situated approximately 650 to 1,200 feet south and west of the proposed LGEP. These samples were subsequently tested in the laboratory to evaluate their suitability for use as engineered fill in construction of the proposed main pad and access road.

3.1 PRE-DRILLING ACTIVITIES

Before drilling and geologic field mapping, AMEC staked the proposed locations of the exploratory drilling in the field. AMEC contacted Underground Service Alert (USA) to have member utility companies mark their utilities in the vicinity of the proposed boring locations before commencing the field work. AMEC also requested that the SCL personnel check the exploration locations to confirm that there were no conflicts with any underground utilities or structures owned by the landfill. SCL personnel cleared exploration locations of buried utilities before drilling commenced.

3.2 EXPLORATORY BORINGS

Three bucket-auger borings, six hollow-stem-auger borings, and one rock-core boring were drilled as part of the field exploration. The exploration locations are shown on Figure 3. The details of the exploratory work are described in below subsections.

3.2.1 Bucket Auger Borings

Roy Brothers Drilling of Malibu, California, provided the bucket-auger drilling services. Three borings (BA-1 through BA-3) were drilled on July 12 through July 14, 2010, using an E-Z Bore drill rig equipped with a 24-inch diameter auger. The borings were drilled to depths ranging from 80 to 95 feet below ground surface (bgs). Borings were physically entered and down-hole logged by an AMEC geologist licensed by the State of California as a Professional Geologist and a Certified Engineering Geologist (CEG). The bucket auger borings were drilled near the top of the ridge containing Flare Station No. 8 to further characterize the bedrock lithology and local geologic structure within the North Slope. Upon completion of drilling and sampling, the borings were backfilled and tamped with soil cuttings. Locations of the bucket auger borings are shown on Figure 3 and boring logs are presented in Appendix A.

3.2.2 Hollow Stem Auger Borings

BC^2 Environmental Corporation of Fullerton, California, provided the hollow stem auger drilling services. Six borings (B-1 through B-6) were drilled on July 21 through July 23, 2010, to depths ranging from 31 to 66 feet bgs. The intent of these borings were to: (1) collect samples of the existing fill placed for the access road on the North Slope and at the bottom of the canyon for laboratory testing to evaluate its engineering properties; (2) evaluate the groundwater depth; and (3) help identify the contact between the fill and bedrock on the North Slope. An AMEC field engineer under the direction of a California-licensed Geotechnical Engineer maintained a record of field activities, classified the soils encountered, and prepared a log of the borings. Locations of the hollow stem auger borings are shown on Figure 3 and boring logs are presented in Appendix A.

Relatively undisturbed soil samples were collected from the boring using driven split spoon samplers. Standard Penetration Tests (SPTs) were conducted and the blow counts required to drive the SPT and California-modified split spoon samplers were recorded. Upon completion of drilling and sampling, the borings were backfilled with soil cuttings. Bulk and relatively undisturbed soil samples were delivered to AMEC's laboratory for testing to assist in characterizing engineering properties of subsurface materials. Samples were tested at AMEC's laboratory in Newport Beach, California and at AP Engineering, Inc. in Pomona, California.

3.2.3 Rock-core Boring

BC^2 Environmental Corporation of Fullerton, California, provided the rock-core drilling services. One boring (CH01) was drilled on July 13, 2010 to a depth of 59 feet bgs. The intent of the rock-core boring was to collect high quality bedrock samples for laboratory shear strength testing. An AMEC geologist, licensed and certified by the State of California as a Professional Geologist and as an Engineering Geologist (CEG), maintained a record of field activities, classified the materials encountered, and prepared a log of the boring. Location of the rock-core boring is shown on Figure 3 and the boring log is presented in Appendix A.

The bedrock samples were collected using a core barrel and the samples were carefully packaged and then transported to AMEC's laboratory in cardboard core boxes for detailed examination and laboratory shear strength testing. The boring was backfilled with bentonite chips.

3.3 LABORATORY TESTING

The laboratory testing program was designed to characterize the engineering properties of the soil and bedrock materials encountered. Soil samples were collected by different types of samplers during the field exploration program for laboratory testing including split spoon samplers, California modified split spoon samplers, and bulk samples. Laboratory testing of selected samples included:

- In-situ Dry Density and Moisture Content (ASTM D 2937 & D 2216)
- Grain Size Distribution (ASTM D 422)
- Atterberg Limits (ASTM D 4318)
- Expansion Index (ASTM D 4829)
- Compaction Characteristics (ASTM D 1557)

- Consolidation (ASTM D 2435)
- Direct Shear (ASTM D 3080)
- Unconsolidated Undrained (UU) Triaxial Tests (ASTM D 2850)
- Unconfined Compressive (UC) Strength 1633 (ASTM D 1633)
- Corrosion (CTM 643, CTM 422, CTM 417)

Physical tests were performed in our Newport Beach, California laboratory and AP Engineering, Inc.'s laboratory. The chemical tests related to corrosivity were performed at Schiff Associates laboratory. Laboratory data sheets are provided in Appendix B. A summary of the laboratory testing results is provided in Table 1.

3.4 GEOLOGIC MAPPING

The site specific geologic conditions that include the type and surface distribution of the bedrock, surficial deposits and the spatial orientation of the rock discontinuities that include bedding planes, and joints, were mapped in the field by a California-licensed Engineering Geologist (CEG). The geologic conditions exposed in the large diameter borings, borings BA-1, BA-2 and BA-3, were down-hole logged by the CEG. The geologic conditions mapped in the field along with the proposed grades based on the current site layout (Scheme 10B) are shown on the geologic map (Figure 3). The mapped geology at the site is discussed in Section 4.0 along with the subsurface conditions encountered in the borings.

A total of six aerial photographs dating from December 2, 2004 through November 15, 2009 were utilized to aid in geologic mapping. These photographs were helpful in depicting the evolution of landfill grading operations within the immediate vicinity of the LGEP, Bedrock outcrop patterns and contacts between cut and fill. Rectified images, at a scale of 1 inch equals 40 feet, of the November 11, 2005, March 16, 2006 and November 9, 2009 air photos were also used to aid in mapping of the surficial failures within the natural ascending slopes as shown on the Geologic Map (Figure 3). All of aerial images were obtained from Google Earth and are included in Appendix C

4.0 SUBSURFACE CONDITIONS

The following sections present our findings based on the field exploration and laboratory testing program, geologic mapping, and review of published information on regional geology.

4.1 REGIONAL GEOLOGY

The SCL and the LGEP lie within the western portion of the Transverse Ranges Geomorphic Province, a relatively large area of Southern California characterized by terrain with similar geomorphic and structural geologic features. The Province is comprised of an east-west trending band of rugged and steep mountain ranges and intervening valleys roughly 80 miles in width that extend from Point Arguello on the west to the easterly end of the San Bernardino Mountains, a distance of roughly 250 miles. The terrain within the Province is unique in that it is oblique to the normal northwest trend of the Coast Ranges, Great Valley and Sierra Nevada Provinces situated to the north and the Peninsular Ranges Province to the south. The Transverse Ranges are the result of the middle Miocene and younger tectonism which includes clockwise rotation and compression of the terrain southwest of the San Andreas Fault principally due to crustal movements associated with the North American and Pacific Plate convergence. As a result, east-west trending faults and folds, deep sediment-filled-structural basins and uplifted terrain dominate the geologic structure as well as the topography of the province.

The SCL is situated at the easterly extension of two large northwest-southeast trending plunging folds, the Oat Mountain Syncline and the Pico Anticline, that dominate the geologic structure of the easterly portion of the Santa Susanna Mountains. The bedrock within folds is comprised principally of marine clastic and biogenic sedimentary rocks that vary from the Middle Miocene Topanga Formation to the Pliocene-Pleistocene Saugus Formation.

In addition to the extensive folding of the bedrock, regional tectonism is evidenced by extensive faulting. The north dipping Santa Susana Fault Zone which is considered to be a western extension of the Sierra Madre Fault Zone is positioned along the southerly margin of the Santa Susanna Mountains and is approximately 6,000 feet southeast of the LGEP. Similar south dipping thrust faults situated along the northerly margin of the Santa Susanna Mountains include the Weldon Canyon and the Bacon faults which are situated approximately 3,000 feet and 5,000 feet north of the LGEP respectively. Portions of the mapped traces of the Santa Susana Fault Zone exhibited evidence of displacement during the 1971 San Fernando Earthquake and as such have been included in Alquist Priolo Special Study Zones. The northerly border of the closest AP Zone is approximately 600 feet south of the LGEP. The mapped trace of the fault within this AP Zone is approximately 1,200 feet south of the LGEP.

Other significant faults in the vicinity of the LGEP include the Oak ridge Fault, the Holser Fault and the San Gabriel Fault. The Oak ridge Fault is a roughly east-west trending thrust that dips shallowly toward the south. Its onshore segment extends from the Oxnard Plain to Piru a distance of roughly 30 miles. Evidence of Holocene surface rupture has been observed on this fault in the vicinity of Fillmore. To the west the mapped trace of the Oak ridge Fault appears to be overthrust by the Santa Susana Fault Zone becoming a blind thrust. This blind thrust, the Pico Thrust, is thought to be the source of the 1994 Northridge Earthquake.

The Holser Fault is a south dipping reverse fault that appears to branch from the San Gabriel Fault zone roughly 5 miles north of the LGEP. The most recent rupture on this fault appears to be Late Quaternary.

The San Gabriel Fault Zone is a northwest-southeast trending fault that exhibits right-lateral strike slip movement. Holocene surface rupture has been recognized along this fault in the area between Saugus and Castaic. Other segments of this fault appear to have experienced surface rupture during the Late Quaternary and Quaternary. The San Gabriel Fault Zone is approximately 5 miles north of the LGEP.

The bedrock underlying the immediate vicinity of the proposed LGEP location consists of marine clastic sediments that have been assigned to the Towsley Formation of late Miocene to Pliocene age. The bedrock is composed of fine to coarse-grained sandstone and interbedded micaceous and clayey siltstone with minor amounts of pebbly conglomerate. Typically, the siltstone beds are less prone to development of a thick soil profile and when exposed on steep anti dip slopes form relatively resistant outcrop bands that can be mapped for hundreds of feet. The geologic structure of the bedrock is dominated by its position on the northerly limb of the Pico Anticline. The attitude of bedding is relatively consistent striking to northwest and dipping steeply to the northeast.

4.2 LOCAL GEOLOGIC CONDITIONS

The following is a description of the surficial soil and bedrock units in the immediate vicinity of the site based on review of published data, geologic mapping, and subsurface conditions encountered in the borings. The geology map of the site is presented in Figure 3 and the geologic cross sections that depict the subsurface geologic conditions at the site are presented in Figures 4 and 5. The engineering properties of the geologic units are described in Section 4.3. The geologic units in the vicinity of the site are described below in the order of increasing age.

4.2.1 Artificial Fill (Af):

Artificial fill exists at several locations within the vicinity of the site as a consequence of previous expansion of the landfill and construction of the access road leading to Flare Station No. 8.

The fill materials were encountered in exploratory borings B-1 through B-6 and their surface distribution is based on interpretation of aerial photographs, available topographic maps and field mapping. These materials typically consist of mixtures of residual soil, colluvium and weathered bedrock materials derived from relatively shallow cuts associated with previous nearby grading operations. The fill materials are light olive brown to dark grayish brown in color and consist of a mixture of low plasticity silt and clay, sand, and gravel. The fill materials are classified as sandy lean clay with gravel to clayey sand with gravel. The amount of gravel in the fill material generally varies between 5% and 25% and is typically about 15% by weight of the soil mass. The maximum particle size is typically 1 inch and the gravel particles are generally described as fine gravel (i.e., smaller than ¾ inch). The engineering properties of the fill materials were evaluated based on the laboratory test results and discussed in Section 4.3.

4.2.2 Alluvial Deposits (Qal):

A minor amount of alluvium remains at the head of the canyon that will be occupied by the gas to energy facilities. These deposits consist predominantly of poorly sorted soil and rock fragments interspersed with boulders up to 2 feet in diameter. The alluvial deposits were derived from erosion of upslope colluvial and residual soil deposits and bedrock outcrops. The thickness of the alluvial deposits is expected to vary from 1 to 3 feet.

The floor of the canyon between the alluvial deposits and the hairpin bend in the access road leading to Flare Station No. 8 is occupied by artificial fill. Reportedly the alluvial materials that existed along the bottom of the canyon before grading of the access road were removed before placement of the existing fill. Alluvium was not encountered in the borings below the existing fill that now occupies the floor of the canyon.

4.2.3 Residual Soil (No Map Symbol):

Residual soil deposits consisting of mixtures of sand, silt and, clay interspersed with fragments of weathered sandstone siltstone, form a relatively thin mantle that locally covers the bedrock underlying the natural slopes. Due to their relative thin and scattered nature the distribution of the residual soil is not shown on the geologic map. These materials are, for the most part, the result of in-place weathering, and decomposition of the underlying bedrock and as such are typically composed of similar materials. In general, the thickness of the residual soil deposits varies based upon the parent rock type, the structural orientation of the underlying bedrock and the configuration of the natural slopes. For example, the residual soil development on the North Slope is relatively thick as this slope is underlain by siltstone bedrock that dips roughly parallel to the slope surface. In contrast the steeper southwesterly facing slopes (includes the South Slope), where the bedding dips into slope, the development of a soil mantle is typically absent and sandstone and siltstone outcrops are more typical. Beneath the North Slope the residual soil deposits are anticipated to vary from 2 to 3-feet thick, thickening in the down slope direction. On the steep anti dip slopes soil profile development is generally absent and is not expected to be greater than approximately 2-feet thick.

4.2.4 Colluvium (Qcol):

Colluvial deposits cover the lower portions of the natural slopes merging laterally with the alluvial deposits situated along the canyon floor. These deposits are incoherent accumulations of residual soil and weathered rock debris that have migrated down slope by the process of creep, slope wash and shallow surficial failures. Typically these deposits thicken in the down-slope direction forming broad aprons at the base of the slopes. Colluvial soil deposits are exposed in the existing cut slopes situated along the toe of the slope that ascends from the northeasterly side of the canyon and within shallow swales locally positioned near the bottom of the slopes. The vertical thickness of the colluvial materials on the natural slopes is estimated to vary from 6 to 20 feet or more.

4.2.5 Shallow Slumps (Qs) and Surficial Failure Scars (Qsf)

Numerous small shallow slumps, surficial failure scars and resulting debris flow deposits were mapped in the field and noted in the aerial photographs for the South Slope. Generally, these failures are confined to narrow shallow drainage swales on the slopes. Locally, the resulting failure scares have exposed weathered and jointed bedrock that is prone to minor raveling including the lower portion of the slope that ascends west of the SCE substation.

The small slumps appear to be relatively shallow rotational failures comprised predominately of highly weathered bedrock, residual soil and/or colluvium. The thickness of the slump debris is anticipated to vary from typically 3- to 4–foot thick or less for the smaller failures on the South Slope to likely more for the large slump mapped at the head of the canyon approximately 300 feet west of the LGEP site.

Surficial failures have impacted the steep southwest facing slopes where bedding within the sedimentary rocks dips into slope and the surficial soil cover is minimal. Similar surficial failures were not observed on the North Slope where bedding is generaly inclined steeper than the natural ground surface or within the adjacent fill slopes associated with the access road for Flare Station No. 8. The surficial failures are all comprised of debris derived from the poorly developed soil profile and the underlying weathered bedrock. The resulting failure scars are no more than 6-inches to a foot deep and are generally associated with existing shallow swales. In areas where slope gradients are steep and relatively uniform debris flows typically extend from the failure scars to the bottom of the slope. In contrast, surficial debris flows have a tendency to dissipate before reaching the bottom of the slope areas where colluvial deposition has reduced the gradient of the lower portion of the slope.

Based on review of the air photos and previous evaluation of slope instabilities in the Santa Clarita area and Interstate 5 corridor, most of the surficial failures and debris flows are likely the result of the heavy and prolonged rainfall that occurred during the 2004-2005 rainfall season. In most instances these failures appeared to have occurred in areas that were impacted by previous failures.

The rainfall events that occurred during 2004-2005 rainfall season are significant in that the rainfall season totals were at or near record levels and the duration and intensity of individual storm events in some areas was greater than is typical. Review of aerial photographs that depict the surface conditions of the site from December of 2004 through November of 2009 indicate that the area of the initial failure scars that developed during the 2004-2005 rainfall season enlarged with each subsequent rainfall season. The area of the failure scars and debris flows associated with the 2004-2005 rainfall season comprises a relatively small portion of the surface area of the steep southwest facing slopes. By 2009, the areas involved in surficial failures had increased noticably.

4.2.6 Landslide Debris (Qls):

Review of aerial photographs and mapping of topographic anomalies indicate the presence of two relatively small landslides near the head of the canyon on the North Slope west of the proposed project site (in an ungraded area up the canyon from the project site). These landslides are numbered herein as Landslide No. 1 and No. 2 as shown in Figure 3. The surface expression of these landslides is relatively well defined in aerial photographs and topographic maps that depict the existing site conditions. The lateral margins of the slides are characterized by subtle variations in vegetation patterns and topography in the field.

The largest of the landslides, Landslide No. 1, is located on the slope below the flare station access road within the head of the canyon and is approximately 360 feet long and 220 feet wide. Landslide No.2 is approximately 240 feet long and 180 feet wide and is situated in the natural slope that ascends from the hairpin turn of the access road located in the bottom of the canyon. The toe of Landslide No.2 appears to have been buried by compacted fill placed in the bottom of the canyon during grading of the adjacent fill slope and flare station access road. Landslides No. 1 and 2 are situated in terrain that is comprised of relatively massive siltstone that dips toward the northeast at gradients of approximately 40 to 50 degrees.

Based upon the observations, it appears that these landslides originated from the upper part of the slope and moved downslope toward the bottom of the canyon. They likely are within the highly weathered bedrock that mantles some of the canyon slopes above the less oxidized bedrock. The cause(s) of the landslides has not been determined, but is most likely the result of the lower strength of the highly weathered bedrock and to a lesser degree on structural discontinuities in bedrock (e.g., joints and fractures). The approximate location of the landslides is shown on Figure 3.

4.2.7 Towsley Formation (Ttos):

Sedimentary bedrock in the vicinity of the site has been assigned to the Towsley Formation, a stratigraphically thick sequence of clastic marine sediments of latest Miocene to early Pliocene age. Published references have subdivided the formation into two principal rock units that have broad regional distribution, a thick sequence of light gray to yellow brown, fine to coarse-grained sandstone with minor interbeds of pebbly sandstone and micaceous siltstone (Ttos) and a relatively thin interfingering sequence of dark gray, micaceous, clayey siltstone (Ttoc). Published geologic maps and outcrops mapped in the field indicate the bedrock within the vicinity of the proposed LGEP is comprised of sandstone and siltstone assigned to the (Ttos) unit as shown on Figure 4. Geologic mapping and subsurface exploration for the proposed project have not revealed the presence of rocks assigned to the (Ttoc) unit.

Sandstone (Ttos $_{ss}$) and Siltstone (Ttos $_{slt}$),

The southwesterly portion of the project site which includes the ridge beneath Flare Station No.8, the slopes that descend from the ridge to the northeast and southwest, and the narrow canyon to the northeast, where the proposed LGEP facilities will be located, is underlain by a thick sequence of interbedded siltstone and minor sandstone that grades stratigraphically upward to a massive micaceous siltstone with minor thin beds of fine to very fine-grained sandstone. The interbedded sandstone and siltstone segment of this sequence of rocks is locally exposed in the steep natural slope that descends to the southwest from Flare Station No.8 and within cut slopes along the northerly margin of the landfill. These rocks are typically light gray to light brown in color and micaceous and fine grained.

Beneath the flare station ridge and the North Slope and canyon to the northeast, the bedrock is almost entirely composed of internally massive micaceous siltstone with minor thin beds of fine to very fine sandstone. Natural outcrops are limited in this area; however; the bedrock is exposed in the steep cut slopes associated with the flare station access road and was encountered in the 24-inch diameter bucket auger borings B-1 through B-3 and the core boring CH-01. In the vicinity of the core boring CH-01 the bedrock appears to grade laterally becoming fine-grained silty sandstone. Within 18 to 25 feet of the natural ground surface, the bedrock is generally highly weathered. Near the ground surface the highly weathered segment of the bedrock is marked by relatively intense near vertical fractures filled with caliche. Below these depths the bedrock is moderately weathered to light yellowish brown or a dark reddish brown that grades downward to a dark brown. Beneath a depth of 35 to 45 feet, the siltstone bedrock is typically unoxidized varying from greenish gray to dark gray and finally black. The minor sandstone beds within the unoxidized bedrock are typically blue gray to greenish gray in color.

Bedding within these rocks is typically poorly defined as the contacts between contrasting rock types are generally gradational and the thick siltstone segments are internally massive. Bedding within the massive siltstone is generally defined by micaceous partings. In spite of its poor definition, bedding within the bedrock is relatively uniform, striking to the northwest and dipping toward the northeast at angles varying from 40 to 70 degrees below the horizontal. Minor tectonic deformation within the rock principally associated with regional folding that created the Pico Anticline and the Oat Mountain Syncline is marked by crushed and gouged zones that appear to be roughly parallel bedding. These zones vary from 1-inch to as much as 2-foot thick. Minor clay seams and clayey gouge zones up to 2-inch thick are associated with some of the tectonically crushed bedrock.

Jointing within the bedrock is typically steep and somewhat variable. At most locations, jointing strikes toward the northwest, roughly parallel to the strike of bedding, and dips to the northeast or southwest at angles varying from 40 to 85 degrees. A minor joint set was also noted trending northeast to southwest, roughly perpendicular to the strike of bedding, and dipping steeply to the northwest and southeast.

The bedrock that is exposed in the South Slope that ascends to the northeast from the floor of the canyon is somewhat different than the rock beneath the terrain to the southwest. This slope is comprised of relatively thick beds of sandstone alternating with beds of internally massive siltstone - six beds in total, 3 sandstone beds and 3 siltstone beds have been mapped in the slope. The thick sequence of sandstone and massive siltstone beds strike northwest to southeast and their outcrop patterns roughly parallel the strike of the ridge that separates the landfill from the Golden State Freeway located to the northeast. All of the beds dip into the slope at gradients varying from 35 to 45 degrees. As such, the South Slope has favorable bedding regarding stability and global landsliding. The steep dip of bedding, the continuous outcrop pattern, the mapable contacts between individual rock units over long distances, the lack of deep seated landslide features and the lack or limited development of soil cover suggest the sequence of sandstone and siltstone beneath the South Slope is only moderately weathered, a striking contrast with the highly weathered, soil covered siltstone bedrock situated beneath the portion of North Slope that remains in a natural condition. A brief description of these rock units is provided in the following paragraphs.

The first and lowermost bed in this sequence of rocks, which is positioned roughly 35 to 65 feet above the floor of the canyon, is a massive siltstone that is approximately 40-foot thick. This rock unit forms an outcrop pattern on the slope that can be traced from the road cut in the access road north of the flare station to the area directly above the location of the LGEP, a distance of roughly 900 feet. For the most part this rock outcrop is barren of soil and vegetation.

Immediately above the massive siltstone is a sandstone bed approximately 60-foot thick comprised of light yellow to brown, thickly to thinly bedded, fine to coarse-grained sandstone with minor interbeds of yellow brown sandy siltstone 1/2 to 18-inches thick. This rock unit is generally covered by a thin poorly developed sandy soil profile that supports a relatively light cover of vegetation.

Above the sandstone is the second massive siltstone bed approximately 45 to 50-foot thick, This bed is very similar to the one situated near the bottom of the canyon but appears to be somewhat sandier. This bed is also barren of soil and vegetation and can be traced along the slope for a least 1,200 feet.

Immediately above the siltstone bed described above is the second sandstone bed which is approximately 60 to 70 feet thick. This bed consists of a white fine to very medium-grained sandstone with minor light brown siltstone interbeds up to 1-foot thick. The outcrop of this sandstone bed is also covered by a poorly developed sandy soil profile that supports a relatively light cover of vegetation providing a striking contrast with the barren siltstone beds above and below.

The third and last of the massive siltstone units is at least 60-foot thick. This bed is positioned along the top of the slope and appears to cap the crest of the ridge northeasterly of the LGEP. Like the massive siltstone units below it, this rock unit forms an outcrop pattern on the slope that is relatively barren of soil and vegetation that can be traced for a distance of at least 800 feet.

The third and last sandstone unit is positioned at the crest of the ridge in the vicinity of the intersection of the flare station access road and the road that provides access to existing SCE electrical lines along the crest of the ridge. It is composed of light brown, very fine grained sandstone that is very friable.

4.2.8 Summary of Local Geologic Conditions

Bedrock beneath the proposed LGEP and the SCE substation is comprised of a sequence of internally massive sandstone and siltstone sedimentary rocks that dip steeply toward the northeast. Bedding planes within the bedrock are generally oriented favorably with respect to the natural terrain, the existing graded slopes, and the proposed LGED and the SCE substation. Beneath the North Slope, bedding is inclined at gradients that are steeper than the natural slope. Beneath the South Slope, bedding dips steeply into the natural slope. In addition, the exposed bedrock on the South Slope is generally thought to be moderately weathered due to the steep anti-dip configuration of the slope which promotes removal of the highly weathered debris as they develop. The gross stability of both the North and South slope configurations are generally considered relatively stable from a geologic/kinematic perspective.

Clay seams that appear to be related to tectonic deformation of the bedrock were observed at a depth of roughly 45 feet in boring BA-3 and 80 feet in BA-2. They were not observed in the other borings drilled on the North Slope. These features are potential planes of weakness within the bedrock along which slope movements could occur. However, in both instances the potential for instability along the seams appears to be relatively insignificant for the following reasons. In boring BA-3, the clay seam is a discontinuous feature associated with tightly folded and contorted rock mass that is less than two feet in width, and as such, is interpreted not to have any lateral continuity. Also at this location, slope is covered with existing compacted fill material that is approximately 80 feet wide (see cross section 1-1" in Figure 2). The silty clay seam in boring BA-2 is deep in the slope (encountered at depth of 80 feet) and is inclined at an angle of 44 degrees to the north east. This configuration is steeper than the gradient of the natural slope and slope movement along the clay seam would require a deep seated failure path extending across bedding and through compacted fill for a distance more than 120 feet.

The portion of the North Slope that remains in a natural condition (slope west and up canyon from the proposed LGEP site) is mantled by residual soil, landslide debris, and highly weathered bedrock. The low strength of these shallow materials is likely the cause of the shallow sliding that has occurred in the recent geologic past (Landslide Nos. 1 and 2 on Figure 3). Within the previously graded portion of the North Slope, unsuitable materials and the highly weathered portion of the bedrock appear to have been removed and replaced with compacted fill, which results in a significantly more stable configuration (see cross sections 2-2', 3-3' and 5-5'). The proposed LGEP has been located directly adjacent to the graded portion of the North Slope so that the recognized instabilities in the natural, ungraded portion of the North Slope will not adversely impact the development or use of the LGEP.

Surficial failure scars and resulting debris flow deposits are generally confined to existing narrow and shallow drainage swales on the South Slope that ascends from the proposed LGEP and the SCE substation. Locally, the failure scars have exposed moderately weathered and jointed bedrock that is prone to minor raveling, including the lower portion of the slope that ascends west of the SCE substation. The most obvious surficial failures and debris flows are the result of the heavy and prolonged rainfall that occurred during the 2004-2005 rainfall season and most have enlarged with subsequent rainfall seasons. Surficial failures and resulting debris flows of a similar magnitude should be anticipated during future rainfall events. Future surficial failures are most likely to occur in areas where past events have occurred or where relatively thick surficial debris has accumulated on the ascending slope. The proposed LGEP and the SCE substation should include debris control measures designed to minimize the nuisance of such sumps and minor debris flows. Potential debris control measures are provided in Section 7.1.4.

4.3 SOIL ENGINEERING PARAMETERS

AMEC performed laboratory shear strength tests on relatively undisturbed samples of the existing fill and bedrock materials collected during our field investigation and on remolded samples of potential stockpile materials for use as the proposed engineered fill. The discussion of shear strength evaluation for each material are provided below and the shear strength parameters developed as part of this Study for use in slope stability analyses are presented in Table 2.

4.3.1 Existing Fill

AMEC performed four direct shear strength tests per ASTM D 3080 on relatively undisturbed samples of the existing fill material. The samples were consolidated under various normal loads and saturated before testing. The shear strength test data are included in Appendix B. The shear resistance vs. normal load values at 20% lateral displacement are plotted on Figure 7. Based on the test results, we developed a bilinear shear strength envelope for the existing fill as shown on Figure 7. A bilinear envelope was assigned to better characterize the shear strength characteristics of the fill material at shallow and deeper depths for low and high normal loads, respectively.

4.3.2 Bedrock

AMEC performed unconsolidated uniaxial triaxial (UU) test, unconfined compression (UC) tests and direct shear test on rock core samples collected during field exploration. The shear strength of the Towsley Formation increases with depth with as the level of weathering and oxidation decreases. For the purposes of shear strength evaluation, Towsley Formation was divided into three zones: (i) highly weathered bedrock, (ii) moderately weathered bedrock, and (iii) unoxidized bedrock.

4.3.2.1 Highly Weathered Bedrock

The highly weathered bedrock exhibited shear strength properties similar to a medium stiff clayey soil. AMEC performed one direct shear test and one triaxial UU test on the highly weathered bedrock. The shear strength parameters for this highly weathered material was not used in our stability evaluation of the North and South Slopes because our borings indicate that the highly weathered bedrock was removed before placement of fill on the North Slope, which was also confirmed with our borings performed across the project site. Moderately weathered rock strengths were also used to model the stability of the South Slope. The use of these strengths appears justified based upon the geologic character of the bedrock exposed on the steep anti-dip South Slope as discussed in section 4.2.7 above. The characteristics suggesting moderate weathering include steeply dipping nearly continuous mapable bedrock rock outcrops, the lack of deep seated landslide features and the lack or limited development of soil cover. The steep anti-dip configuration of the slope also promotes the continuous removal of the highly weathered debris as they develop. The shear strength test results are included in Appendix B and summarized in Table 1.

4.3.2.2 Moderately Weathered Bedrock

AMEC performed two triaxial UU tests and one triaxial UC test on the moderately weathered bedrock samples. The test results are summarized in Table 1. The samples tested were assumed to represent intact rock samples with the failure surfaces occurring across bedding and not influenced by any structural discontinuity. Inspection of samples after testing suggested that some may have failed along pre-existing zones of weakness (e.g., stress relief cracks); therefore, assuming the test results represent intact samples may be conservative (i.e., cross bedded strength may actually be higher than the test results).

AMEC used the computer program RocLab (Rocscience Inc., Version 1.031) to develop equivalent Mohr-Coulomb shear strength parameters for the overall oxidized rock mass. RocLab uses triaxial strength test results, the Hoek-Brown failure criterion as described in detail in Hoek and Carranza-Torres (2002) and Hoek and Diederichs (2006), and structural integrity parameters to develop a strength envelope for the rock mass. By using the structural integrity parameters, RocLab accounts for structural discontinuities with the rock mass.

Based on the observations of the rock core samples and the downhole-logged bucket auger holes, AMEC assigned an average geological strength index of 70 to the moderately weathered bedrock for use in RocLab. An average intact rock parameter, mi, of 7 was assigned to the rock mass based on the suggested typical value in the RocLab software for siltstone. This intact rock parameter is likely conservative considering the Towsley Formation in the North and South Slopes is predominately sandy siltstone with occasional sandstone beds, which would increase the intact rock parameter and the rock mass strength. An average compressive strength of 24 kips per square foot (ksf) based on the triaxial strength test results was used in the RocLab software. The estimated rock mass strength envelope for the highly weathered bedrock is plotted on Figure 8.

As discussed in Section 2.4, AMEC reviewed historical reports prepared for projects in the vicinity of the LGEP site. GeoSyntec (1998) previously evaluated the stability of the southwest slopes descending to the north as part of their geotechnical report for the construction of the Flare No. 8. A-Mehr (2006) performed slope stability analyses for the cut slopes along the eastern boundary of the County expansion of the SCL, which is approximately 2,400 feet east of the LGEP site. GeoSyntec used cross bedding bedrock strengths significantly greater than the strength parameters used by A-Mehr (2006). Geosyntec (1998) parameters were based on their laboratory test results on bedrock samples not collected from the North Slope area. A-Mehr (2006) used shear strength parameters recommended by USGS for landslide studies (DCDMG, 1998). For comparison, the cross bedding shear strength envelopes used by A-Mehr (2006) and GeoSyntec (1998) are plotted in Figure 8 along with the shear strength envelope for the moderately weathered bedrock used in this Study.

4.3.2.3 UnOxidized Bedrock

AMEC performed three triaxial UU tests and two triaxial UC test on the unoxidized bedrock samples. The test results are summarized in Table 1. These specimens were assumed to represent intact rock samples with the failure surfaces occurring across bedding and not influenced by any structural discontinuity. The specimens tested showed similar failures along possible predefined zones of weakness as with the oxidized bedrock samples. AMEC used the computer program RocLab (Rocscience Inc., Version 1.031) to develop equivalent Mohr-Coulomb shear strength parameters for the overall unoxidized rock mass.

Based on the observations of the rock core samples and the downhole-logged bucket auger holes, AMEC assigned an average geological strength index of 80 to the unoxidized bedrock for use in RocLab analysis along with an average compressive strength of 31 ksf based on the triaxial strength test results. As with the oxidized bedrock, an average intact rock parameter, mi, of 7 was assigned to the unoxidized bedrock based on the suggested value in RocLab software for siltstone. The average geological strength index and the compressive strength values from the test results from the samples collected at shallower depths were used in an effort to model the upper portion of the unoxidized bedrock. The estimated rock mass strength envelope for the unoxidized bedrock is plotted on Figure 8.

4.3.2.4 Proposed Fill

At the request of HRGreen and SGP, AMEC collected bulk samples from five different soil stockpiles at the SCL site for laboratory testing. The purpose of the sampling and testing was to evaluate their engineering characteristics and whether they are an appropriate source of engineered fill for the project. The suite of laboratory tests performed on each bulk sample included gradation, shear strength, consolidation, corrosion, and expansion index. Test results are provided in Table 1 and Appendix B. The shear strength test results for the five stockpile samples are plotted on Figure 9. Based on these results, a typical shear strength envelope was selected to represent proposed engineered fill in slope stability analyses.

4.4 GROUNDWATER

Groundwater was encountered in the exploratory borings BA-1 and BA-2 at approximately 80 feet bgs at the time of drilling. These borings penetrated the siltstone bedrock beneath the flare station ridge at or near the crest of the North Slope. Groundwater was not encountered in the remaining borings drilled for this investigation.

Historic depth to groundwater data is available for two landfill monitoring wells located in the LGEP site. Landfill monitoring well, CM-10 was located in the bottom of the canyon adjacent to the exploratory boring B-4 at an elevation of approximately 1874 feet above mean sea level (fmsl) and has subsequently been abandoned (see Figure 3). Depth to groundwater measurement data available to AMEC for CM-10 included the monitoring periods during 2004, 2006, and 2007. Based on that available data, groundwater levels in CM-10 have ranged from approximately 29 to 37 feet bgs (approximate elevations 1844 to 1836 fmsl). Groundwater levels are currently being monitored in the LGEP area in landfill monitoring well CM-10R located adjacent to the flare station access road at an elevation of approximately1940 feet as shown on Figure 3. CM-10R was constructed in May 2008. Depth to groundwater data available to AMEC for CM10R included the 2009 monitoring period. Based on that available data, the groundwater level in CM-10R ranged from approximately 85 and 86 feet bgs in 2009 (approximate elevations 1855 to 1854 fmsl). The available depth to groundwater for wells CM-10 and CM-10R are provided in Table 3.

Past and present water levels from the monitoring wells and the groundwater levels encountered in the current exploratory borings were utilized to construct a hypothetical groundwater surface beneath the vicinity of the proposed LGEP facility and within the North and South Slopes.. The hypothetical groundwater surface is illustrated on the geologic cross sections shown on Figures 4 and 5 and was used in the slope stability analyses. Based on the existing data, the depth to groundwater beneath the proposed LGEP is more than 60 to 80 feet. The depth to groundwater beneath the SCE substation is anticipated to be approximately 50 feet. GeoSyntec Consultants (1998) did not include groundwater in their stability analysis of the North Slope.

5.0 SEISMIC CONSIDERATIONS

5.1 REGIONAL TECTONIC SETTING

The project site is located in Southern California, one of the most seismically active regions in the world. According to the National Seismic Hazard Map source model (USGS, 2002), the closest faults to the project site include the following ones:

- The Sierra Madre fault zone, which is approximately two to five kilometers (km) southeast of the site (including the Santa Susana section and the San Fernando section);
- The Northridge fault, which is approximately 5.1 km southwest of the site;
- The San Gabriel fault, which is approximately 7.6 km northeast of the site;
- The Holser fault, which is approximately 8.9 km northwest of the site; and
- Gridded seismic sources that are used by the USGS to represent background seismicity, special seismic zones, and intraslab events. Gridded seismic sources are located within approximately 5 km of the site.

Seismic design of the project is based on the guidelines in 2010 California Building Code (CBC 2010). CBC 2010 guidelines are based on American Society of Civil Engineers publication ASCE 7-05 and require that ground motions for seismic design be based on a maximum considered earthquake (MCE) ground motion. The MCE ground motion is defined in ASCE-7-05 as the most severe ground motion with a 2 percent probability of exceedance within a 50-year period (a return period of approximately 2,475 years) or deterministically as 150 percent of the largest ground motion for characteristic earthquakes on all known active faults within the region.

5.2 SEISMIC DESIGN PARAMETERS

Based on the CBC (2010) Section 1613.5, the following mapped seismic design parameters for the project were developed:

- Mapped spectral accelerations for short periods S_S: 2.36 g
- Mapped spectral accelerations for a 1-s period S_1: 0.81 g
- Site Class: D

- Site Coefficient Fa: 1.0
- Site Coefficient Fv: 1.5
- Adjusted MCE spectral acceleration for short periods SMS= Fa SS = 2.36 g
- Adjusted MCE spectral acceleration for a 1-s period SM1= FvS1 = 1.21 g
- 5 percent damped design spectral response acceleration at short periods SDS: 1.58 g
- 5 percent damped design spectral response acceleration at 1-second period SD1: 0.81 g

5.3 VERTICAL GROUND MOTION RESPONSE SPECTRA

Vertical response spectra are not required by either CBC 2010 or ASCE-7-05. The effects of vertical seismic load effect may be determined in accordance with Section 12.4.2.2 of ASCE-7-05. According to Section 12.4.2.2 of ASCE-7-05, vertical seismic load is essentially equal to $0.2S_{DS}$ times the dead load for all vertical periods where S_{DS} is horizontal five percent damped design spectral response acceleration at short periods as described in Section 5.2.

5.4 SEISMIC HAZARDS

As discussed in Section 4.1, no faults or shear zones have been mapped within the footprint or the immediate vicinity of the project site. Therefore, the potential for fault rupture within the immediate vicinity of the proposed LGEP is considered to be low.

The project site is not within a liquefaction zone as defined by the Seismic Hazard Zone maps for the Oat Mountain Quadrangle (DCDMG, 1998) (Figure 10). Moreover, because the subsurface materials underlying the LGEP site consist of compacted clayey fill materials that are not susceptible to liquefaction and also because of the lack of shallow groundwater, potential for liquefaction at the site is considered remote.

The project site is within an earthquake-induced landslide zone as defined by the Seismic Hazard Zone maps for the Oat Mountain Quadrangle (DCDMG, 1998) (Figure 10). The landslide features around the site have been mapped and discussed in Section 4.2.6. Based on our study and the latest layout of the proposed LGEP shown on Figure 2, there are no existing landslide features within the slopes immediately descending towards the site. The slope stability analyses were performed as part of this Study and discussed in Section 6.0 below.

Seismically induced settlements occur mostly due to the contractive volumetric strains developed in saturated soils during seismic events. Typically, soils with significant amount of fines similar to the existing fill and potential fill materials to be used for the proposed project, experience significantly less settlement than clean granular soils. Based on the lack of groundwater and the significant amount of fines in the existing compacted fill seismically induced settlements are not considered design factors for the LGEP.

6.0 SLOPE STABILITY EVALUATION

Global stability of the existing slopes descending towards the proposed LGEP (i.e., North and South Slopes) and the slopes of the proposed fill pad as part of the LGEP was evaluated by performing two-dimensional limit-equilibrium analyses and calculating a Factor of Safety (FS) against sliding for both static and seismic conditions. The stability of potential shallow failure surfaces were also evaluated using a typical infinite slope formulation. The analysis methods, acceptance criteria, the cross sections analyzed, and the static and seismic slope stability analysis results are discussed in following sections

6.1 LIMIT-EQUILIBRIUM ANALYSIS

The computer program Slope/W (Geo-Slope, 2004) was used to perform Spencer's limit-equilibrium analysis method (Spencer, 1967) because it satisfies both force and moment equilibrium, and accounts for inter-slice forces. Slope/W is a commercially available computer program with a comprehensive formulation that makes it possible to analyze complex geometric configurations and loading conditions.

In terms of slope stability, the FS against sliding is defined as the ratio of resisting forces (friction and cohesion along potential failure surface) to driving forces (gravitational forces pulling downslope). A FS of unity (1.0) indicates a delicate balance between the resisting and driving forces and represents incipient failure. A FS below unity indicates instability.
The seismic stability is evaluated using the pseudo-static analysis method within Slope/W. In this method the earthquake forces are represented by a static lateral force equal to the product of the horizontal seismic coefficient (k) and the weight of the slide mass, and a FS is computed using conventional limit-equilibrium analysis.

The North Slope (graded area), South Slope, and the proposed fill slopes were analyzed for shallow and deep circular failures. The existing and proposed fill slopes were also analyzed for surficial instability. The failure scenarios analyzed are as follows:

- global stability of the proposed fill slopes for static and seismic conditions with and without geogrid reinforcement,

- global stability of North and South Slopes for static and seismic conditions, and
- surficial stability of the existing fill slope on the North Slope and the proposed 1.5H:1V fill slopes (with and without geogrid reinforcement) under saturated conditions.

6.2 INFINITE SLOPE STABILITY ANALYSIS

The existing slopes and proposed fill slopes may be susceptible to shallow surficial failures under dry conditions or during periods of heavy rainfall. These surficial failures, which are generally referred to soil slumps or soil slips, are typically less than about 4 feet in depth, and have small thickness to length ratios. Conventional equations can be used to analyze the surficial stability of these slopes, referred to as infinite slope stability analysis. These equations are based on limit equilibrium methods, and are considered to be valid for slopes that extend a relatively long distance and have consistent subsurface profiles.

An infinite slope formulation by Giroud et al. (1995) which allows the contribution of geosynthetic reinforcement, was used to calculate the FS of the proposed 1.5H:1V reinforced fill slope. This method was also used to analyze the proposed 3H:1V unreinforced fill slopes, and existing unreinforced fill and bedrock slopes. The equation provided in Giroud et al. (1995) is as follows:

$$FS = (\gamma' / \gamma_s) * (\tan\phi' / \tan\beta) + (c' / (\gamma_s * t * \sin\beta)) + (T / (\gamma_s * t * h))$$

where*:*

γ' = buoyant unit weight (pcf)
γ_s = saturated unit weight (pcf)
ϕ' = angle of internal friction (degrees)
β = slope angle (degrees)
c' = cohesion (psf)
t = soil thickness measured perpendicular to ground surface (ft);
t = z cos β,
z= vertical height of the soil column (ft)
T = tensile strength of geogrid (lb/ft)
h = vertical height between geogrid layers (ft)

6.3 ACCEPTANCE CRITERIA

AMEC used the stability criteria provided in the County of Los Angeles Department of Public Works Manual of Preparation of Geotechnical Reports (July 2010) to evaluate the static and seismic performance of the project slopes. These criteria as defined by Los Angeles County are as follows:

Long-term static condition: FS greater than or equal to (≥) 1.5

Pseudo-static: FS ≥ 1.1

Surficial Stability: FS ≥ 1.5

For seismic stability, Los Angeles County generally uses the pseudo-static analysis by the Seed (1979) procedure. In this method, slopes wherein a pseudo-static FS ≥ 1.1 are computed based on a minimum seismic coefficient of 0.15. The minimum depth of saturation was assumed to be 4 feet vertically for the infinite slope analysis per the Los Angeles County Manual.

6.4 CROSS SECTIONS AND SLOPE CONFIGURATIONS

The proposed LGEP consists of an approximately 200 feet wide and 200 feet long pad at an elevation of approximately 1900 feet. The pad construction will consist of placement of up to approximately 50 feet of fill along the bottom of the existing valley and filling a portion of the area in between the North and South slopes. The access to the pad will be achieved by constructing an access road from the south side of the LGEP as shown on Figure 2. The east facing sideslopes of the fill will be constructed at 1.5H:1V because of the space constraints. The west end of the main pad will have a 3H:1V fillslope descending towards the head of the valley.

AMEC developed nine cross sections that depict the geology underlying the project. Seven sections out of nine were used in slope stability analyses. The area and conditions each of the sections analyzed for stability as follows:

- Section 1'-1" represents the bedrock slopes within the South slope descending towards the proposed pad,
- Section 2-2' represents the existing fill slopes within the North slope descending towards the proposed pad.
- Section 5-5' represents the north side of the 1.5H:1V reinforced fill slopes where the fill slope height and width is the highest and the largest, respectively, within the east fill slope,
- Section 6-6' represents the northwest 3H:1V fill slope,
- Section 7-7' represents the south side of the 1.5H:1V reinforced fill slopes where the fill slope width is limited because of the existing slopes and benching is necessary for the geogrid reinforcement,

- Section 8-8’ represents the steepest bedrock slopes within the South slope descending towards the lower pad for the SCE substation, and

- Section 9-9’ represents the bedrock slope along an existing ravine with shallow slumps within the South slope descending towards the lower pad for the future SCE substation.

6.5 SEISMIC COEFFICIENT FOR PSEUDO-STATIC STABILITY

The most commonly used values for the seismic coefficient are based on the recommendations from Seed (1979), which was developed for application to earth dams and for up to 1 meter of displacement. A number of the local regulatory agencies use the Seed (1979) procedure for the seismic coefficient, including the Los Angeles County. The Seed (1979) procedure recommends values of k = 0.10 and 0.15 for M = 6.25 and 8.25 earthquakes respectively. Los Angeles County requires a minimum value of k = 0.15 for pseudo-static analysis, therefore, this value is used in this Study.

6.6 SLOPE STABILITY ANALYSIS RESULTS

Results of the static, pseudo-static, and infinite slope stability analyses are summarized in Table 4. Graphical plots of the Slope/W results are presented in Appendix D. The infinite slope stability analysis calculations, as discussed in Section 6.2, are included in Appendix E. The existing fill and existing bedrock slopes possess a minimum static FS greater than 1.5 and a minimum pseudo-static FS greater than 1.1 under proposed conditions.

As summarized in Section 4.2.8, the highly weathered bedrock within the North slope at the east end of the ridge was removed as part of the access road construction and replaced with compacted fill. As noted in our boring logs, the fill is underlain with moderately weathered bedrock, which is significantly more competent (i.e., higher strength) compared to the highly weathered bedrock. In addition to the overexcavation and removal performed in this area, the proposed LGEP fill will also further increase the FS of this slope against slope instability by further buttressing the toe of the North Slope. As a result, the static FS in Section 2-2’ is greater than 1.5.

As summarized in Section 4.2.8, the bedding pattern on the South Slope is dipping into the slope and the surficial failure scars and resulting debris flow deposits on the South Slope are observed which are generally confined to existing narrow and shallow drainage swales. Although, a transition from moderately weathered to unoxidized bedrock is anticipated within the South Slope, we modeled the entire South Slope in our stability analysis with the moderately weathered bedrock shear strength parameters and the results indicate the Sections 1’-1” and 8-8’ possess minimum static FS of 1.5 and pseudo-static FS of 1.1. This is likely to be conservative based on the weathering profile observed in the North Slope. We also believe the stability results for the South Slope are conservative because we used bedrock strengths derived from siltstone samples only, while the South Slope is comprised of both relatively thick beds of sandstone and beds of massive siltstone.

The proposed 3H:1V west-facing and 1.5H:1V southeast-facing fill slopes were analyzed. The proposed 3H:1V slopes possess a FS of greater than 1.5, however, the proposed 1.5H:1V fill slopes possess an FS of 1.23 as shown in Appendix D. Consequently, the proposed 1.5H:1V southeast-facing fill slope requires placement of geogrid reinforcement to meet the FS criteria described in Section 6.3. The Slope/W software was used to design a uniaxial geogrid type reinforcement arrangement to efficiently stabilize the proposed fill slope to meet the stability criteria. Recommended reinforcement design is discussed in Section 7.1.5 and shown in section and plan view on Figure 11.

The infinite slope stability analysis results presented in Appendix E indicate the reinforced southeast-facing fill slope should include secondary reinforcement with a minimum length of 11 feet in every 4 feet vertically as shown in Figure 11. The infinite slope stability analysis of the proposed 3H:1V unreinforced slopes indicate these slopes possess adequate FS greater than 1.5. The infinite slope stability analysis of the existing 1.5H:1V fill slopes indicate these slopes possess a FS less than the required 1.5 by Los Angeles County based on the required assumption of minimum saturation of 4 feet vertically. Based on our visual observations, the existing fill slope on the North Slope does not currently show signs of slumping. This may be a result of the erosion control measures that are currently in-place (bonded straw mats), which likely prevents the slope from becoming saturated. Additional erosion control measures that may be considered for the existing fill slope are discussed in Section 7.1.4. The infinite slope stability analysis of the existing moderately weathered bedrock slopes as steep as 1H:1V possess adequate FS significantly greater than 1.5.

Based on the slope stability analysis results, the existing slopes descending towards the proposed LGEP and the proposed fill slopes including geogrid reinforcement, where necessary, possess adequate stability.

7.0 DESIGN RECOMMENDATIONS

Provided that all recommendations presented herein are incorporated into design and construction, the proposed construction of the LGEP is feasible from a geotechnical engineering standpoint.

This section presents the design recommendations for earthwork, foundations, retaining walls, and pipelines.

7.1 EARTHWORK

Earthwork for the project is anticipated to consist primarily of fill placement to create the main pad and access road. Minor clearing and subgrade preparation are also anticipated.

7.1.1 Site Preparation

The project site consists primarily of existing road fill. Colluvial deposits are present at the toe of the South Slope. These colluvial deposits should be removed as part of the site preparation before placement of fill.

All construction areas should be cleared of objectionable materials, including grass, weeds, concrete, pavements and any other material that might interfere with the performance or completion of the work. Grubbing should then be performed to remove all roots and other objectionable material. Any holes created by the grubbing process in areas that will receive fill or are at or near final grade should be backfilled with general fill as described in Section 7.2.3. All objectionable material from clearing and grubbing should be removed from the site and disposed of at a suitable off-site disposal area or landfill.

7.1.2 Subgrade Preparation

Areas to receive fill should be scarified to a depth of 6 to 8 inches, moisture conditioned to between 0 and 3 percent above optimum moisture content, and compacted to 90 percent of the maximum dry density as determined by ASTM Test Method D 1557. If the subgrade soil is soft or disturbed, it should be excavated to expose firm soil, with the resulting subgrade scarified and conditioned as above, and the excavated material replaced with compacted fill. Construction of the project will require placement of fill against existing slopes. Fill placed against these existing slopes should be benched into the slopes as shown on Figure 12.

7.1.3 Fill Materials and Compaction Criteria

It is anticipated that three principal fill types will be used at the site. These are (generally from coarsest to finest):

- Crushed Rock
- Aggregate Base
- Engineered Fill
- Backfill Behind Walls

Relative compaction requirements discussed below refer to the percent of the maximum dry density as determined by ASTM Test Method D 1557 (latest edition), the optimum moisture content is also as determined by the same specification as the maximum dry density.

7.1.3.1 Open-Graded Crushed Rock

Open-graded crushed rock may be used to create a firm working surface in areas to receive fill where wet subgrade or other conditions cause difficulty with compaction (e.g., pumping of compaction equipment) or may be used for drainage material. The crushed rock should be an imported material that consists of durable rock and gravel that is free of deleterious material and free from slaking or decomposition under the action of alternate wetting and drying. If used to construct drainage trenches, this material should be surrounded by a filter fabric selected to prevent the migration of fines into the gravel. To create a firm working surface, it is recommended that the working surface consist of 12 to 24 inch thick crushed rock layer over a geosynthetic geogrid. The geogrid will function as a separator and reduce the penetration of the crushed rock into the wet underlying soils. Crushed rock should meet the following gradation requirements.

Standard Sieve Size	Percentage Passing
1 inch	100
¾ inch	90-100
No. 4	0-10
No. 200	0-2

These materials should have a durability index not less than 40. Crushed rock used for building pads should be moistened thoroughly and compacted with a minimum of three passes of plate- or roller-type vibratory compaction equipment, with lifts not thicker than 8 inches before being compacted. Crushed rock does not have a specified relative compaction.

7.1.3.2 Aggregate Base

Imported aggregate base material may be used for pavements or for retaining wall backfill. This material should meet the requirements in the Caltrans Standard Specifications (2006) Section 26, Class 2 Aggregate Base (¾-inch maximum particle size). When placed beneath pavements, aggregate base should be compacted to a relative compaction of at least 95 percent or at 90 percent as backfill adjacent to structures. The moisture content of the material should be between 1 percent below and 3 percent above the optimum moisture content and the material should be placed in horizontal lifts that do not exceed 8 inches before being compacted.

7.1.3.3 Engineered Fill/Backfill Behind Walls

Soil obtained from on-site stock piles and/or excavations may be used as engineered fill/backfill, provided the materials meet the criteria below.

All engineered fill/backfill should be free of organic material, debris, and other deleterious material, contain fragments no larger than 3 inches in maximum dimension, and have an expansion index (EI) less than 40 for general use and an EI less than 20 for backfill behind walls. Engineered fill for general use should also meet the shear strength requirements for Proposed Fill in Table 2. The soils to be used as engineered fill/backfill may be somewhat heterogeneous, therefore, mixing, blending, and moisture conditioning may be required to create a material that can be placed and adequately compacted. All fill/backfill should be scarified, plowed, disked, and/or bladed until it is uniform in consistency and free of large, unbroken clods of soil. The moisture content of the general fill/backfill should be adjusted to between 0 percent and 3 percent above the optimum moisture content.

Before the placement of engineered fill, the subgrade should be prepared in accordance with Section 7.1.2 above. Engineered fill/backfill should be placed in horizontal lifts that do not exceed 8 inches in thickness before compaction, and compacted with suitable equipment to a relative compaction of at least 90 percent. The final surface of the compacted fill/backfill should be graded to promote good surface drainage, as described in Section 7.1.4.

7.1.3.4 Sand Cement Slurry

Sand-cement slurry, also known as controlled density fill (CDF), or controlled low strength material (CLSM), or “Slurry Cement Backfill” in Section 19 of the Caltrans Standard Specifications (July 2002), can be used as an alternative fill/backfill material. Sand-cement slurry consists of a fluid, workable mixture of aggregate, Portland cement, fly ash, and water. Sand-cement slurry can be batched to flow into irregularities in the bottoms and walls of excavations and trenches. It is an ideal backfill material when adequate room is limited or not available for conventional compaction equipment, or when settlement of the backfill must be minimized. No compaction is required to place Sand-cement slurry.

The Caltrans specifications for the gradation of sand-cement slurry aggregate are:

Standard Sieve Size	Percentage Passing
½ inch	100
1 inch	80-100
¾ inch	60-100
⅜ inch	50-100
No. 4	40-80
No. 100	10-40

More restrictive gradation requirements may be desirable to limit the fines content and the size of sand and gravel that may adversely affect (i.e., puncture or tear) the corrosion protection of pipes, for example. We recommend that no more than 15 percent of the aggregate pass through the No. 200 sieve; and the 28-day compressive strength of the CDF be no less than 50 and no more than 110 pounds per square inch (psi).

7.1.4 Drainage and Debris Control

Final site grading should provide surface drainage away from structures and slabs-on-grade. Ponding of surface water should not be allowed adjacent to structures. Where slabs or pavements abut landscaped areas, provisions should be made to protect the base rock layer and subgrade soils against saturation from water in the landscaped areas. If landscape water or surface runoff is allowed to seep into the pavement section, the service life of the pavement may be reduced. Landscape watering adjacent to the structure should be avoided. Where needed, drip irrigation systems should be used.

The proposed grading plan shown on Figure 2 is filling in the northwesterly canyon and creates a basin on the upstream northwest end of the canyon. Proper drainage features should be designed and installed to convey the surface water from the upper parts of the canyon to the east side of the proposed fill.

As discussed in Section 4.2.5, there are existing small slumps and debris flows within drainage swales of the South Slope. It is anticipated that minor slumps and debris flows will continue to occur in these swales in the future during large rainfall events. Consequently, the proposed facilities at the north end of the LGEP and SCE Substation (along the toe of the South Slope) should be protected from these nuisances. Protection could be accomplished by installing debris fencing and/or concrete drainage ditches between the toe of the South Slope and the facilities to contain such slumps and flows if they reached the bottom of the South Slope. Debris fencing and drainage ditches should be consistently monitored after storm events to check their function and integrity, and debris removed from ditches and behind the fencing as needed.

Results of our infinite slope stability analyses indicate that the proposed 1.5H:1V fill slope and the existing fill slope on the North Slope are susceptible to surficial sliding if the upper 4 feet of their slope surface becomes saturated. The surficial instability of the 1.5H:1V fill slope can be mitigated using the conceptual reinforcement layout provided in Section 7.1.5 below. For the existing fill slope, the SGL has placed bonded straw mats on the fill slope to reduce surface erosion. This matting also reduces the potential for the upper 4 feet of the slope to become saturated, and thus, helps to mitigate the potential surficial instability. The straw mats could continue to be used to address surficial instability in the future; however, the straw mats have a relatively short life and require periodic re-application. There are other erosion control products that could be used for the existing slope that can provide longer term solutions for surficial instability. Such products include many types of erosion control blankets (ECBs) and Bonded Fiber Matrix (BFM). A new ECB that reportedly lasts several years is the Marimesh SG by Tencate Marafi , which is a synthetic grass face mat that is applied to the slope for erosion protection and significantly enhances surface water runoff on slopes. Several spray-type products are also available that provide immediate and longer term protection against water penetration by bonding to the soils (short term) and establishing vegetation cover via hydro-seeding (long term) such as a Flexterra® by Profile Products, LLC.

7.1.5 Reinforcement for Proposed Fill Slope

The conceptual reinforcement layout in the proposed southeast-facing fill slope was designed to achieve the FS criteria. Recommended reinforcement of the slope consists of uniaxial geogrid type reinforcement placed every 4 to 8 feet vertically in section. The reinforcement should be placed along the entire length of the southeast face of the proposed fill and extend into the fill the entire width of fill in section with a maximum width of 45 feet. General dimensions and layout of the geogrids area shown on Figure 11. The uniaxial reinforcement geogrid should have a minimum allowable tensile strength of 1,760 lb/ft (e.g., Tensar UX1400HS or equivalent)..

7.2 SHALLOW FOUNDATIONS

Spread footings are an acceptable foundation for many of the at-grade structures planned for the LGEP. Footings should bear directly on properly compacted engineered fill. Excavation bottoms for footings should be checked before construction of the footing. Any loose or soft materials in the footing excavations should be removed and backfilled with engineered fill compacted to at least 90 percent relative compaction per ASTM D1557. Continuous and isolated spread footings may be designed using an allowable (net) bearing capacity of 2000 pounds per square foot (psf). The allowable bearing capacity values apply to combined dead and sustained live loads and may be increased by one-third when considering transient live loads, including seismic and wind forces. Footings should have a minimum width of 2 feet and be embedded at least 2 feet below the lowest adjacent finished grade.

Lateral loads on at grade structures are resisted by the foundation using a combination of friction between structural components and the subgrade soils and the passive resistance in front of the footing. Allowable resistance to lateral loads for footings can be estimated using a coefficient of sliding resistance (μ) of 0.25 (FS of 1.5) between the bottom of concrete footings and soil. A frictional μ value of 0.20 is recommended for slabs underlain by a moisture barrier. Additionally, lateral resistance may be provided by passive pressures acting against the vertical sides of the footings. An allowable equivalent fluid pressure of 200 pounds per cubic foot (pcf) (FS of 2) may be used to calculate passive resistance in compacted fill. The upper one foot of soil below lowest adjacent grade should not be used for calculating passive resistance. The allowable passive pressure may be increased by 33 percent for lateral loading due to wind or seismic forces. The friction coefficient and passive pressure may be used concurrently.

Based on a maximum allowable bearing pressure of 2,000 psf, the total settlement beneath the strip/spread foundations is anticipated to be less than ½ inch.

7.3 MOISTURE BARRIER BELOW FLOOR SLABS

It is recommended that a moisture barrier be installed below floor slabs with moisture-sensitive coverings or equipment. The moisture barrier should consist of 10-mil polyvinyl chloride (PVC) plastic sheeting with joints in the sheeting overlapped by a minimum of 12 inches. The PVC sheeting should be covered with a minimum 2-inch thick clean (i.e., no fines passing No. 200 sieve) sand layer to provide working surface and aid in concrete curing.

7.4 MAT FOUNDATIONS

Mat foundations may be designed for an allowable bearing pressure of 2,000 psf (FS of 3) for dead plus live loads and 3,000 psf for load combinations including transient loads.

Allowable resistance to lateral loads can be estimated using a coefficient of sliding resistance (μ) of 0.25 (FS of 1.5) between the bottom of concrete mat foundation and soil. Lateral resistance may be provided by passive pressures acting against the vertical sides of the mat and an allowable equivalent fluid pressure of 200 pounds per cubic foot (pcf) (FS of 2) may be used to calculate passive resistance in compacted fill. The upper one foot of soil below lowest adjacent grade should not be used for calculating passive resistance. The allowable passive pressure may be increased by 33 percent for lateral loading due to wind or seismic forces. The friction coefficient and passive pressure may be used concurrently. If the design of the mat foundation on fill soils is based on elastic theory, a modulus of subgrade reaction (k) of 100 pounds per cubic inch may used be for design of any size mat supported on compacted fill.

Based on a maximum allowable bearing pressure of 2,000 psf, the total settlement beneath the mat foundations is anticipated to be less than 1 inch.

7.5 RETAINING WALLS AND LATERAL EARTH PRESSURES

Recommendations were developed for design of retaining walls. Retaining walls should be designed to resist both lateral earth pressures (static and seismic) and any additional lateral loads caused by surcharge loads on the adjoining ground surface. The recommended earth pressures for different loading conditions are listed in the following table:

Loading Condition	Equivalent Fluid Weight for Lateral Earth Pressure Calculations
Active Earth Pressure[1]	40 pcf
At-Rest Earth Pressure[1]	60 pcf
Seismic Increment, Active[2]	Uniform 30 H in psf (H in feet)
Seismic Increment, At-Rest[2]	Uniform 20 H in psf (H in feet)
Passive Earth Pressure[3]	400 pcf

Notes:

1. Active pressure is typically used where the wall is unrestrained so that the top of the wall is free to laterally deflect. At-rest pressures should be used where the top of the wall is restrained (e.g. basement walls).

2. The seismic increment is used only for walls taller than 12 feet per the LA County Manual. When considering the seismic load case, the pressure increment should be distributed uniformly against the back of the wall and added to the static lateral earth pressure for active or at-rest conditions. For calculating overall stability, the resultant of the seismic increment should be applied at a point 50 percent of the wall height above the base of the footing.

3. Ignore passive resistance for the upper 12 inches unless pavement or a rigid slab-on-grade covers the ground surface.

If a uniform surcharge load is applied adjacent to the wall, we recommend an additional lateral uniform wall pressure equal to 0.32 times the anticipated vertical surcharge pressure for unrestrained walls and 0.48 times for restrained walls. Transient loads induced, for example, by construction equipment, need not be considered in the design, unless they produce lateral pressures that exceed the pressures produced under earthquake loading conditions.

The above pressures are based on the assumption that sufficient drainage will be provided behind the walls to prevent the build-up of hydrostatic pressures from surface and subsurface water infiltration. Adequate drainage may be provided by a subdrain system consisting of a 4-inch diameter perforated pipe bedded in ¾-inch clean, open-graded rock. The entire rock/pipe unit should be wrapped in filter fabric. The rock and fabric placed behind the wall should be at least one foot in width and should extend to within one foot of finished grade. The upper one foot of backfill should consist of on-site, compacted soils. Alternatively, prefabricated drainage panels may be used instead of drain rock, with the drainage panels connected to a 4-inch-diameter perforated pipe at the base of the wall. In either case, the subdrain pipe should be sloped to drain by gravity and be connected to a system of closed pipes that lead to suitable discharge facilities. In addition, the "high" end and all 90 degree bends of the subdrain pipe should be connected to a riser which extends to the surface and acts as a cleanout.

7.6 PIPELINES

Recommendations for the design of buried pipelines are provided in the following sections.

7.6.1 Nomenclature

The following terminology is used in this report for the purpose of presenting design recommendations for pipe trench excavation and backfill.

1. Pipe Bedding – The pipe bedding includes the full width of the trench from the bottom of the pipe to a horizontal level about 6 inches below the bottom the pipe.

2. Pipe Zone - The pipe zone includes the full width of the trench from the bottom of pipe to a horizontal level about 12 inches above the top of the pipe.

3. Trench Zone - The trench zone is the full width of the trench above the pipe zone to ground surface.

7.6.2 Bearing Capacity and Settlement

It is anticipated that all buried pipelines will be installed within the proposed or existing fill. Based on the results of our investigation, properly compacted engineered fill will provide adequate bearing capacity to support buried pipelines. Generally, the pressure imposed by the pipelines will be less than the existing soil overburden pressure at the proposed invert depths. Therefore, properly compacted fill will be suitable to support the pipelines without settlement being a design factor.

7.6.3 Bedding Material

It is recommended that pipes be bedded on a minimum of 6 inches of crushed rock or select sand meeting the gradation requirements presented in Tables 5 and 6. The select sand should also have a minimum sand equivalent (SE) of 30, as determined by California Test Method 217.

Trench excavations along the pipeline alignments will likely expose saturated interbedded silty and clayey soils. Fill soils that become wet and soft could present difficulties for pipe installation. One way to mitigate this condition is to excavate a minimum of 12 inches below the bottom of planned pipe bedding material, and replace the excavated material with ¾-inch crushed rock. It may be necessary to line the bottom and sidewalls of the overexcavated trench portion with a filter fabric of the type Mirafi 140N or equivalent to prevent the migration of fines into the crushed rock. The fabric may be folded at the top of the crushed rock to completely encapsulate the crushed rock layer.

7.6.4 Pipe Zone Backfill

Backfill to be placed in the pipe zone should consist of crushed rock or select sand conforming to the gradation requirements recommended in Tables 5 and 6 as described above. In addition to the gradation requirements, the select sand should have a minimum Sand Equivalent (SE) of 30, as determined by the California Test Method 217. Based on the results of the laboratory tests, the on-site soils are not suitable for use as pipe-zone backfill, and import of backfill materials will likely be necessary. Alternatively, CLSM may be used.

Further evaluation of trench spoils for use as pipe-zone backfill may be conducted if it is desired to use the excavated material for this purpose. Import of backfill materials will be necessary where excavated materials are deemed unsuitable.

7.6.5 Trench Zone Backfill

The site subsurface materials generated from trench excavations are considered suitable to be used as backfill in the trench zone, provided that they are free of vegetation, debris, organic materials, deleterious materials, and particles greater than 3 inches in largest dimension.
If wet soils are to be reused, they may require an active and diligent drying/mixing operation to reduce the moisture content to a level where adequate compaction can be achieved.

7.6.6 Backfill Placement and Compaction Requirements

Backfill should be compacted by mechanical or vibratory equipment to achieve the required compaction standard. Flooding or jetting should not be used for compaction purposes.

Backfill should be placed on each side of the pipe simultaneously to avoid unbalanced loads on the pipe. All backfill should be moisture-conditioned to, or slightly above, optimum moisture content, placed in lifts not exceeding 8 inches in thickness, and compacted to at least 90 percent of the maximum dry density in the pipe zone and trench zone. In paved areas, the upper 12 inches of subgrade and all overlying aggregate baserock within the trench zone should be compacted to at least 95 percent of maximum dry density. The maximum density and optimum moisture content for each material used should be determined in accordance with ASTM Method D 1557.

7.7 CORROSION AND CHEMICAL ATTACK RESISTANCE

AP Engineering and Testing, Inc. of Pomona, California performed chemical analyses, pH, and minimum resistivity tests on bulk samples of potential fill sources. Corrosion test results are presented in Appendix B.

The soil pH value was determined to range between 6.3 to 7.2, which is considered mildly to severely corrosive. Based on correlations in the Navy Design Manual (NAVFAC DM-5), resistivity results on as-received and saturated soil samples indicate that on-site soils may be heavily to severely corrosive when in contact with ferrous materials. Typical recommendations for mitigation of the corrosive potential of the saturated soil in contact with ferrous materials are the following:

- Below-grade ferrous metals should be given a high quality protective coating, such as an 18-mil plastic tape, extruded polyethylene, coal tar enamel, or Portland cement mortar.

- Below-grade ferrous metals should be electrically insulated (isolated) from above grade ferrous metals and other dissimilar metals, by means of dielectric fittings in utilities and exposed metal structures breaking grade.

- Steel and wire reinforcement within concrete having contact with the site soils should have at least two inches of concrete cover.

If ferrous materials are expected to be placed in contact with site soils, it may be desirable to consult a corrosion specialist regarding chosen construction materials, and/or protection design for the proposed facilities.

The corrosion test results also indicate that potential fill sources have moderate to severe sulfate attack potential on concrete, according to ACI 318-05, Table 4.3.1. Refer to ACI-318 for appropriate concrete mix design. ACI makes no special requirements for cement type or water content when sulfate attack potential is negligible.

7.8 CONFORMANCE WITH SECTION 111 OF THE L.A. COUNTY BUILDING CODE

The proposed grading plan for the LGEP at the SCL has been designed in accordance with generally accepted standards of engineering practice. The design will be safe from the hazards of landslide, settlement, or slippage for structures founded on the main pad. The proposed grading and proposed structure will not adversely impact the property outside the developed area. The design conforms to the requirements of Section 111 of the Los Angeles County Building Code.

8.0 CONSTRUCTION CONSIDERATIONS

The following paragraphs discuss key considerations during construction of the LGEP facility.

8.1 EXCAVATION DIFFICULTY

Based on our field exploration program, earthwork can be performed with conventional construction equipment.

8.2 DEWATERING

Based on current and historical groundwater levels in the vicinity of the site and that minimal excavation is proposed for the project (primarily place of fill), it is anticipated that groundwater will not be encountered during grading. Therefore, the need to dewatering is not anticipated.

8.3 CONSTRUCTION SLOPES

Excavations during construction should be conducted so that slope failure and excessive ground movement will not occur. The short-term stability of excavation depends on many factors, including slope angle, engineering characteristics of the subsoils, height of the excavation and length of time the excavation remains unsupported and exposed to equipment vibrations, rainfall, and desiccation.

If and where spacing permits, and providing that adjacent facilities are adequately supported, open excavations may be considered. In general, unsupported slopes for temporary construction excavations above groundwater should not be expected to stand at an inclination steeper than 1.5H:1V for the proposed and existing fill.

Surcharge loads from vehicle parking and travel lanes or stockpiled materials should be kept away from the top of temporary excavations 10 feet or a horizontal distance equal to at least one-half the depth of excavation, whichever is greater. Surface drainage should be controlled along the top of temporary excavations to preclude wetting of the soils and erosion of the excavation faces. Even with the implementation of the above recommendations, sloughing of the surface of the temporary excavations may still occur, and workmen should be adequately protected from such sloughing.

8.4 POST INVESTIGATION SERVICES

Final project plans and specifications should be reviewed before construction to confirm that the full intent of the recommendations presented herein have been applied to design and construction. Following review of plans and specifications, observation and testing should be performed by the Geotechnical Engineer of Record during soil improvement and grading to document that foundation elements are founded in or penetrate the recommended soils.

9.0 CLOSURE

The conclusions, recommendations, and opinions presented herein are: (1) based upon our evaluation and interpretation of the data obtained from our field and laboratory programs and from previous field explorations; (2) based upon an interpolation of soil conditions between and beyond the borings; (3) are subject to confirmation of the actual conditions encountered during construction; and, (4) are based upon the assumption that sufficient observation and testing will be provided during construction.

If parties other than AMEC are engaged to provide construction geotechnical services, they must be notified that they will be required to assume complete responsibility for the geotechnical phase of the project by concurring with the findings and recommendations in this report or providing alternate recommendations.

If pertinent changes are made in the project plans or conditions are encountered during construction that appear to be different than indicated by this report, please contact this office. Significant variations may necessitate a re-evaluation of the recommendations presented in this report.

10.0 REFERENCES

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TABLES

TABLE 1

SUMMARY OF LABORATORY TEST RESULTS
Landfill Gas to Energy Project
Sunshine Canyon Landfill
Sylmar, California

Boring No.	Sample Type [1]	Sample Depth (feet bgs) [2]		Unified Soil Classification System (USCS)			Moisture Content (ASTM [4] D2216) (%) [5]	Dry Density (ASTM D2937) (pcf) [6]	Sieve Analysis (ASTM D422)			Hydrometer Test (ASTM D422)		Expansion Index (ASTM D4829)	Atterberg Limits (LL:PL:PI) (ASTM D4318)	Modified Compaction (ASTM D1557)		Direct Shear Test (ASTM D3080)				Unconsolidated-Undrained Triaxial Compressive Strength (ASTM D 2850) Maximum Deviator Stress (psf)	Unconfined Compressive Strength (ASTM D2166) Maximum Deviator Stress (psf)
		From (feet)	To (feet)	Formation [3]	Soil Description	USCS Group Symbol			% Gravel	% Sand	% Fines	Silt (%)	Clay (%)			Optimum Moisture Content (%)	Maximum Dry Density (pcf)	Peak Friction Angle (degree)	Peak Cohesion (psf)	Large Displacement [7] Friction Angle (degree)	Large Displacement [7] Cohesion (psf)		
B-1	R	3.0	3.5	af	Sandy Lean Clay with Gravel to Clayey Sand with Gravel	CL-SC	11.3	97.5	-- [8]	--	--	--	--	--	--	--	--	--	--	--	--	--	--
B-1	S	5.0	6.0	af	Sandy Lean Clay with Gravel to Clayey Sand with Grave	CL-SC	--	--	--	--	--	--	--	25	--	--	--	--	--	--	--	--	--
B-1	R	8.0	9.0	af	Sandy Lean Clay with Gravel to Clayey Sand with Grave	CL-SC	10.5	101.2	--	--	--	--	--	--	--	--	--	35	641	34	687	--	--
B-1	R	15.5	16.0	af	Sandy Lean Clay with Gravel to Clayey Sand with Grave	CL-SC	11.9	97.4	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--
B-1	R	25.5	26.0	af	Sandy Lean Clay with Gravel to Clayey Sand with Grave	CL-SC	13.0	107.2	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--
B-1	R	50.5	51.0	af	Sandy Lean Clay with Gravel to Clayey Sand with Grave	CL-SC	18.4	106.5	--	--	--	--	--		--	--	--	--	--	--	--	--	--
B-2	R	3.0	3.5	af	Sandy Lean Clay with Gravel to Clayey Sand with Grave	CL-SC	16.8	103.6	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--
B-2	R	10.5	11.0	af	Sandy Lean Clay with Gravel to Clayey Sand with Grave	CL-SC	15.1	108.2	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--
B-3	R	3.0	3.5	af	Sandy Lean Clay with Gravel to Clayey Sand with Grave	CL-SC	13.2	106.8	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--
B-3	S	5.0	6.5	af	Sandy Lean Clay with Gravel to Clayey Sand with Grave	CL-SC	--	--	--	--	--	--	--	20	--	--	--	--	--	--	--	--	--
B-3	B	5.0	10.0	af	Sandy Lean Clay	CL	--	--	0	38	62	--	--	--	--	11.6	123.3	--	--	--	--	--	--
B-3	R	10.5	11	af	Sandy Lean Clay with Gravel to Clayey Sand with Grave	CL-SC	5.3	118.5	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--
B-3	R	20.5	21.0	af	Sandy Lean Clay with Gravel to Clayey Sand with Grave	CL-SC	18.9	96.2	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--
B-3	R	30.5	31.0	af	Sandy Lean Clay with Gravel to Clayey Sand with Grave	CL-SC	26.4	95.7	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--
B-4	R	3.0	3.5	af	Sandy Lean Clay with Gravel to Clayey Sand with Grave	CL-SC	15.2	109.0	--	--	--	--	--	--	--	--	--	37	400	37	50	--	--
B-4	R	8.0	8.5	af	Sandy Lean Clay with Gravel to Clayey Sand with Grave	CL-SC	14.2	111.1	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--
B-4	S	10.0	11.5	af	Sandy Lean Clay with Gravel to Clayey Sand with Grave	CL-SC	--	--	--	--	--	--	--	--	28:21:7	--	--	--	--	--	--	--	--
B-4	R	13.0	13.5	af	Sandy Lean Clay with Gravel to Clayey Sand with Grave	CL-SC	16.1	105.9	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--
B-4	R	18.0	18.5	af	Sandy Lean Clay with Gravel to Clayey Sand with Grave	CL-SC	15.4	110.3	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--
B-5	R	3.0	3.5	af	Sandy Lean Clay with Gravel to Clayey Sand with Grave	CL-SC	17.2	102.9	--	--	--	--	--	--	--	--	--	41	292	44	99	--	--
B-5	R	10.5	11.0	af	Sandy Lean Clay with Gravel to Clayey Sand with Grave	CL-SC	10.4	101.5	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--
B-5	S	15.0	16.5	af	Sandy Lean Clay with Gravel to Clayey Sand with Grave	CL-SC	--	--	--	--	--	--	--	24	--	--	--	--	--	--	--	--	--
B-5	B	15.0	20.0	af	Sandy Lean Clay with Gravel to Clayey Sand with Grave	CL-SC	--	--	--	--	--	--	--	--	--	10.5	125.0	--	--	--	--	--	--
B-5	R	20.5	21.0	af	Sandy Lean Clay with Gravel to Clayey Sand with Grave	CL-SC	11.9	106.1	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--
B-6	R	3.0	3.5	af	Sandy Lean Clay with Gravel to Clayey Sand with Grave	CL-SC	11.8	105.3	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--
B-6	B	5.0	10.0	af	Sandy Lean Clay	CL	--	--	0	35	65	--	--	--	--	9.9	126.2	--	--	--	--	--	--
B-6	R	8.0	8.5	af	Sandy Lean Clay with Gravel to Clayey Sand with Grave	CL-SC	11.6	103.7	--	--	--	--	--	--	--	--	--	--	--	--	--	--	
B-6	S	10.0	11.5	af	Sandy Lean Clay with Gravel to Clayey Sand with Grave	CL-SC	--	--	--	--	--	--	--	--	28:18:10	--	--	--	--	--	--	--	--
B-6	R	13.0	13.5	af	Sandy Lean Clay with Gravel to Clayey Sand with Grave	CL-SC	12.5	106.3	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--
B-6	R	18.0	18.5	af	Sandy Lean Clay with Gravel to Clayey Sand with Grave	CL-SC	10.6	114.9	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--
B-6	R	23.5	24.0	af	Sandy Lean Clay with Gravel to Clayey Sand with Gravel	CL-SC	12.4	111.5	--	--	--	--	--	--	--	--	--	31	831	31	818	--	--
B-6	R	28.0	28.5	af	Sandy Lean Clay with Gravel to Clayey Sand with Grave	CL-SC	11.7	115.3	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--
B-6	R	33.0	33.5	af	Sandy Lean Clay with Gravel to Clayey Sand with Grave	CL-SC	11.0	104.3	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--
B-6	R	38.0	38.5	af	Sandy Lean Clay with Gravel to Clayey Sand with Grave	CL-SC	12.2	115.9	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--
B-6	R	43.0	43.5	af	Sandy Lean Clay with Gravel to Clayey Sand with Grave	CL-SC	12.8	107.2	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--
B-6	R	48.0	48.5	af	Sandy Lean Clay with Gravel to Clayey Sand with Grave	CL-SC	11.5	100.9	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--
CH-01	C	9.7	10.4	Ttos	Silty Sandstone (highly weathered)		16.8	127.4	--	--	--	--	--	--	37:21:16	20.0	99.0	--	--	--	--	1588	--
CH-01	C	14.0	15.0	Ttos	Silty Sandstone (highly weathered)		18.6	105.6	--	--	--	--	--	--	--	--	--	21	1600	17	650	--	--
CH-01	C	31.4	32.2	Ttos	Silty Sandstone (moderately weathered)		12.7	118.0	--	--	--	--	--	--	--	--	--	--	--	--	--	35663	--
CH-01	C	34.0	34.8	Ttos	Silty Sandstone (moderately weathered)		14.2	117.5	--	--	--	--	--	--	--	--	--	--	--	--	--	16893	--
CH-01	C	36.0	36.5	Ttos	Silty Sandstone (moderately weathered)		13.2	116.6	--	--	--	--	--	--	--	--	--	--	--	--	--	--	20550
CH-01	C	38.0	38.5	Ttos	Silty Sandstone (unoxidized)		13.5	122.4	--	--	--	--	--	--	--	--	--	--	--	--	--	36733	--
CH-01	C	42.0	42.5	Ttos	Silty Sandstone (unoxidized)		10.7	125.3	--	--	--	--	--	--	--	--	--	--	--	--	--	--	19882
CH-01	C	43.4	44.0	Ttos	Silty Sandstone (unoxidized)		11.4	126.2	--	--	--	--	--	--	--	--	--	--	--	--	--	41819	--
CH-01	C	44.0	44.5	Ttos	Silty Sandstone (unoxidized)		12.0	123.4	--	--	--	--	--	--	--	--	--	--	--	--	--	--	26188
CH-01	C	49.8	50.3	Ttos	Silty Sandstone (unoxidized)		13.4	123.9	--	--	--	--	--	--	--	--	--	--	--	--	--	30018	--
BA-2	B	28.8	30.3	Ttos	Fat Clay with Sand	CH	28.8	--	2	19	79	54	25	--	55:28:27	--	--	--	--	--	--	--	--
Stockpile 2	(S-1) B	NA [9]	NA	af	Clayey Sand	SC	--	--	--	--	--	--	--	25	27:19:8	11.5	121.5	34	130	34	129	--	--
Stockpile 2	(S-2) B	NA	NA	af	Sandy Lean Clay	CL	--	--	--	--	--	--	--	39	32:20:12	12.0	121.0	35	53	35	53	--	--
Stockpile 1	(S-3) B	NA	NA	af	Silty Sand	SM	--	--	--	--	--	--	--	2	23:20:3	10.5	122.0	36	0	36	0	--	--
C-1	B	NA	NA	af	Sandy Silt	ML	--	--	--	--	--	--	--	28	--	11.0	119.0	34	196	34	196	--	--
C-2	B	NA	NA	af	Sandy Lean Clay	CL	--	--	--	--	--	--	--	41	--	13.5	118.0	34	161	30	101	--	--

Notes:
1. B = bulk, C= core, R = ring, S = SPT.
2. feet bgs = feet below ground surface.
3. af = artificial fill, Ttos = Towsley.
4. ASTM = American Society for Testing of Materials International.
5. % = percent.
6. pcf = pounds per cubic foot.
7. Large displacement results correspond to the final shear stress at the end of the test which is typically 20% lateral displacement unless otherwise noted.
8. "--" = denotes laboratory test not performed for this sample.
9. NA = not applicable.

TABLE 2

SHEAR STRENGTH PARAMETERS USED IN ANALYSES

Landfill Gas to Energy Project
Sunshine Canyon Landfill
Sylmar, California

Soil / Bedrock			AMEC (this Study)		
			Effective Stress Parameters [2]		
Material	Soil Description	Group Symbol [1]	γ_t (pcf) [3]	ϕ' (degrees) [4]	c' (psf) [5]
1	Existing Fill	CL-SC	120	bilinear [6]	bilinear [6]
2	Proposed Fill	SC-SM	125	31	100
3	Moderately Weathered Bedrock	NA [7]	133	30	1,900
4	Unoxidized Bedrock	NA	140	33	2,900

Notes:
1. Group symbol based upon the Unified Soil Classification System.
2. Shear strength parameters based on direct shear test results at large deformations.
3. γ_t = total unit weight; pcf = pounds per cubic foot.
4. ϕ = angle of friction.
5. c = cohesion.
6. Bilinear strength function is used as shown in Figure 7.
7. NA = not applicable.

TABLE 3

SUMMARY OF GROUNDWATER ELEVATION DATA

Landfill Gas to Energy Project
Sunshine Canyon Landfill
Sylmar, California

Well ID [1]	Reference Elevation (ft msl) [2]	Date	Depth to Groundwater (ft)	Groundwater Elevation (ft msl)
CM-10 [3]	1873.57	3/16/2004 [4]	29.18	1844.39
		5/4/2004 [4]	31.97	1841.60
		8/16/2004 [4]	37.36	1836.21
		11/17/2004 [4]	29.57	1844.00
		9/21/2006 [5]	32.32	1841.25
		10/4/2006 [5]	33.40	1840.17
		12/5/2006 [5]	33.93	1839.64
		12/15/2006 [5]	34.98	1838.59
	1874.74	9/12/2007 [6]	37.88	1836.86
		9/25/2007 [6]	37.95	1836.79
CM-10R	1939.55	3/30/2009 [7]	84.67	1854.88
		4/1/2009 [7]	84.69	1854.86
		6/27/2009 [7]	85.32	1854.23
		8/28/2009 [7]	85.84	1853.71
		9/28/2009 [7]	85.83	1853.72
		11/20/2009 [7]	85.98	1853.57

Notes:

1. ID = Identification.
2. ft msl = feet above mean sea level.
3. Well CM-10 was abandoned.
4. Reference: "Combined Groundwater Monitoring Report, Second Semi-Annual Monitoring Period of 2004, Sunshine Canyon City and County Landfills" by A-Mehr, Inc. dated February 15, 2005.
5. Reference: "Combined Groundwater Monitoring Report, Second Semi-Annual Monitoring Period of 2006, Sunshine Canyon City and County Landfills" by A-Mehr, Inc. dated February, 2007.
6. Reference: "Combined Groundwater Monitoring Report, Second Semi-Annual Monitoring Period of 2007, Sunshine Canyon City and County Landfills" by A-Mehr, Inc. dated February, 2008.
7. Reference: "Combined Groundwater Monitoring Report, Second Semi-Annual Monitoring Period of 2009, Sunshine Canyon City and County Landfills" by A-Mehr, Inc. dated February 15, 2010.

TABLE 4

SUMMARY OF SLOPE STABILITY ANALYSIS RESULTS
Landfill Gas to Energy Project
Sunshine Canyon Landfill
Sylmar, California

CROSS SECTION	CASE	CONDITION ANALYZED	FACTOR OF SAFETY
1'-1" South Slope Rock	1	Static - Global	1.61
	2	Pseudostatic [1] - Global	1.29
2-2' North Slope Existing Fill	3	Static [2] - Surficial with no groundwater seepage	1.50
	4	Static [2] - Surficial with full groundwater seepage	0.72 [3]
	5a	Static - Global (shallow)	1.54
	5b	Static - Global (deep)	1.57
	6a	Pseudostatic [1] - Global (shallow)	1.14
	6b	Pseudostatic [1] - Global (deep)	1.18
5-5' Proposed Fill (East) Reinforced 1.5:1	7	Static [2] - Surficial with full groundwater seepage and with geogrid	1.94
	8	Static with geogrid - Global	1.54
	9	Pseudostatic [1] with geogrid - Global	1.12
6-6' Proposed Fill (West) 3:1	10	Static [2] - Surficial with full groundwater seepage	1.57
	11	Static - Global	2.40
	12	Pseudostatic [1] - Global	1.60
7-7' Proposed Fill (East) Reinforced 1.5:1	7	Static [2] - Surficial with full groundwater seepage and with geogrid	1.94
	13	Static with geogrid - Global	1.58
	14	Pseudostatic [1] with geogrid - Global	1.15
8-8'	15	Static - Global	1.50
	16	Pseudostatic [1] - Global	1.18
9-9'	17	Static - Global	1.65
	18	Pseudostatic [1] - Global	1.29

Notes:
1. Based on the County of Los Angeles (2010) requirement of a "k" coefficient = 0.15 and required Factor of Safety = 1.10.
2. Analysis based on infinite slope equations for the upper 4 feet of soil presented in Appendix E.
3. Factor of safety less than acceptable criterion.

TABLE 5

CRUSHED ROCK GRADATION
Landfill Gas to Energy Project
Sunshine Canyon Landfill
Sylmar, California

U.S. Standard Sieve (ASTM E 11)[1]	Percent Passing by Weight (¾-inch max.)	Percent Passing by Weight (1-inch max.)
1 inch	100	90-100
3/4 inch	90-100	30-60
1/2 inch	30-60	0-20
3/8 inch	0-20	0-20
No. 4	0-5	0-5
No. 8	0	0

Note:
1. ASTM = ASTM International.

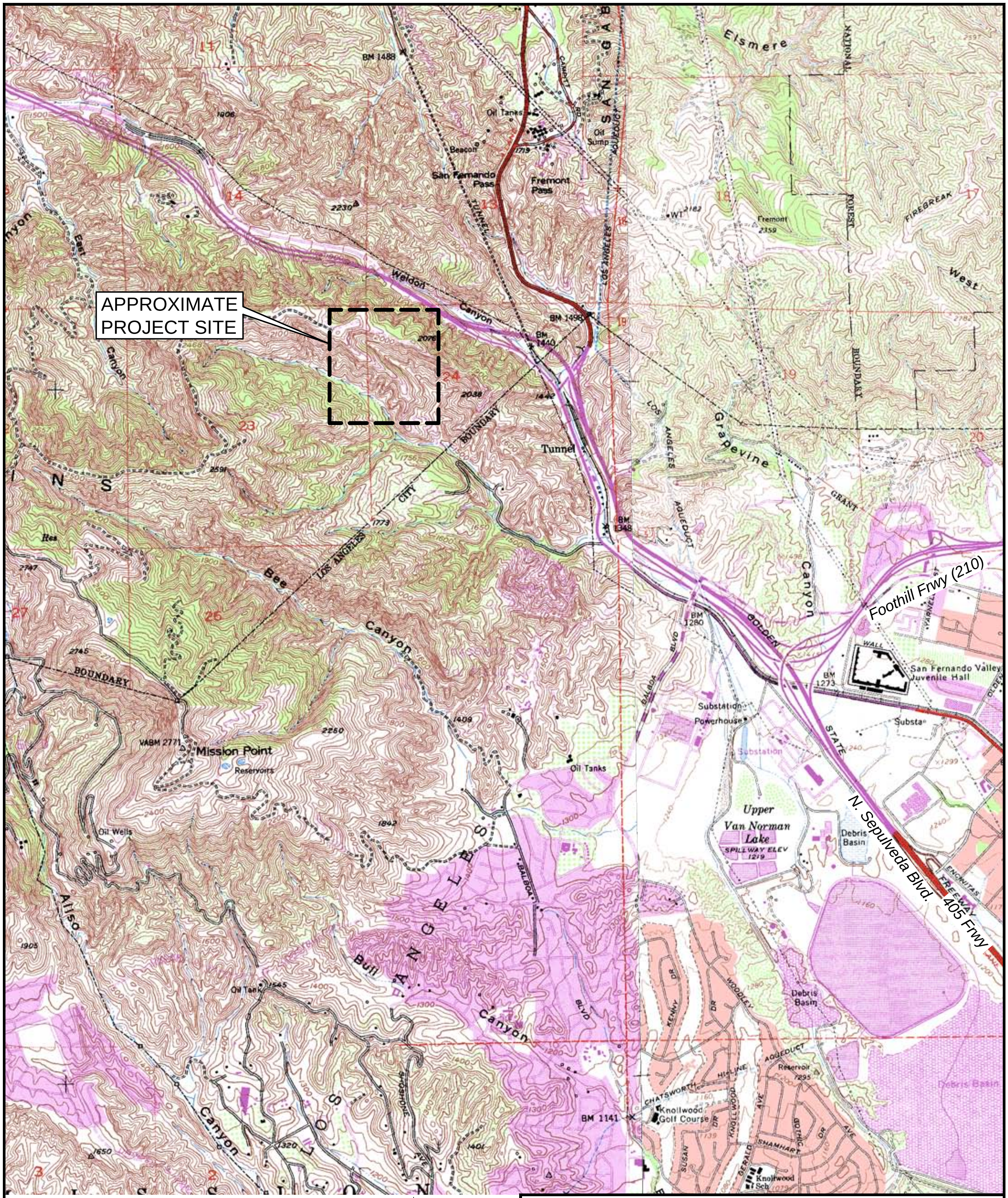
TABLE 6

SELECT SAND GRADATION
Landfill Gas to Energy Project
Sunshine Canyon Landfill
Sylmar, California

U.S. Standard Sieve (ASTM E 11)[1]	Percent Passing by Weight (1-inch max.)
3/8 inch	100
No. 4	75 to 100
No. 40	10 to 50
No. 100	5 to 20
No. 200	0 to 15

Note:
1. ASTM = ASTM International.

FIGURES



Reference: Portions o f Oat Mountain and San Fernando U.S.G.S. 7.5 Minute Topographic Quadrangles, dated 1969 and 1988 respectively.

0 1500 3000
Approximate Scale in Feet

SITE LOCATION MAP
Landfill Gas to Energy Project
Sunshine Canyon Landfill
Sylmar, California

By: jbd	Date: 11/11/11	Project No. 14828.000.0
	amec	Figure 1

Plot Date: 11/11/11 - 2:35pm, Plotted by: pat.herring
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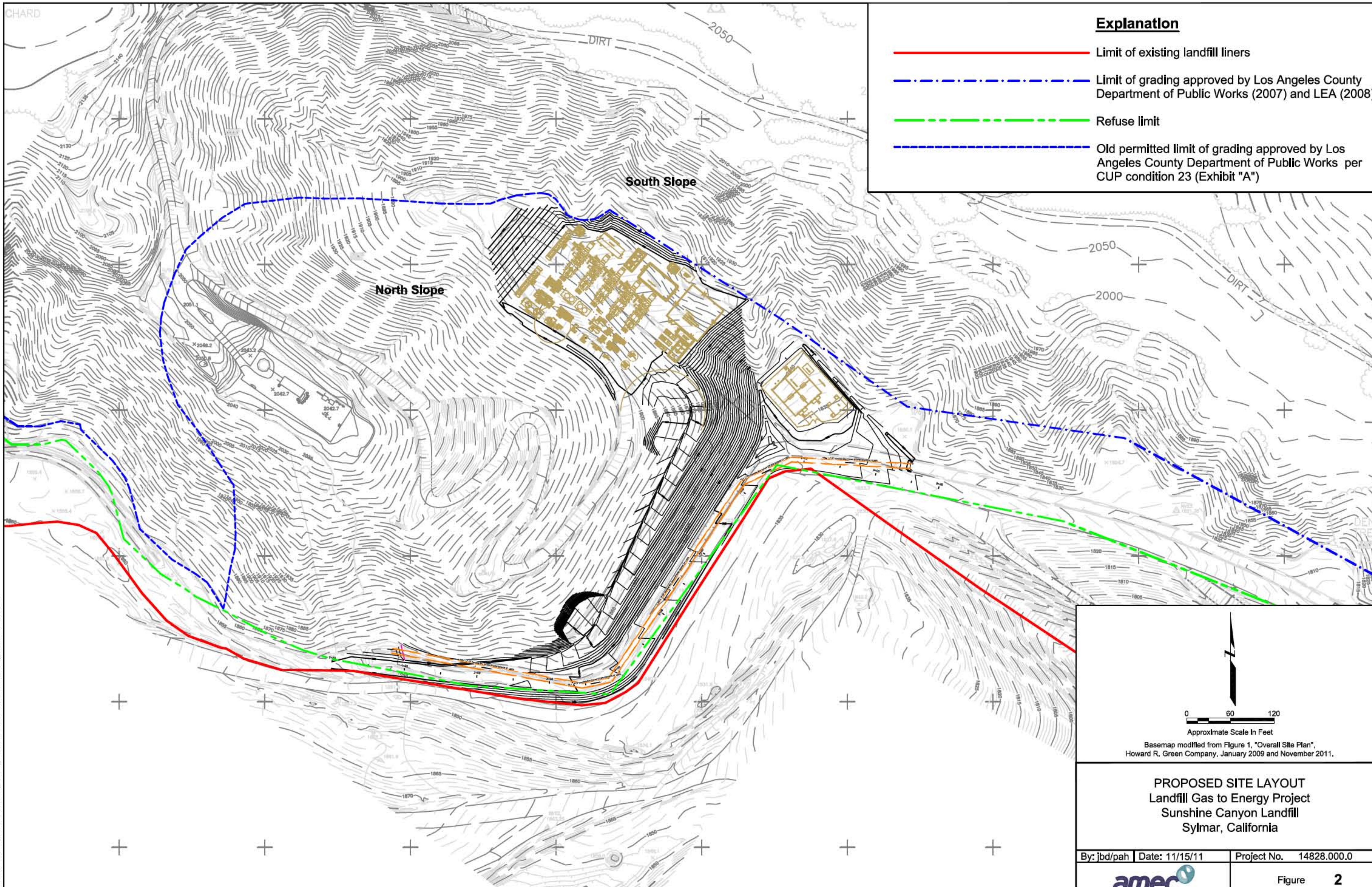
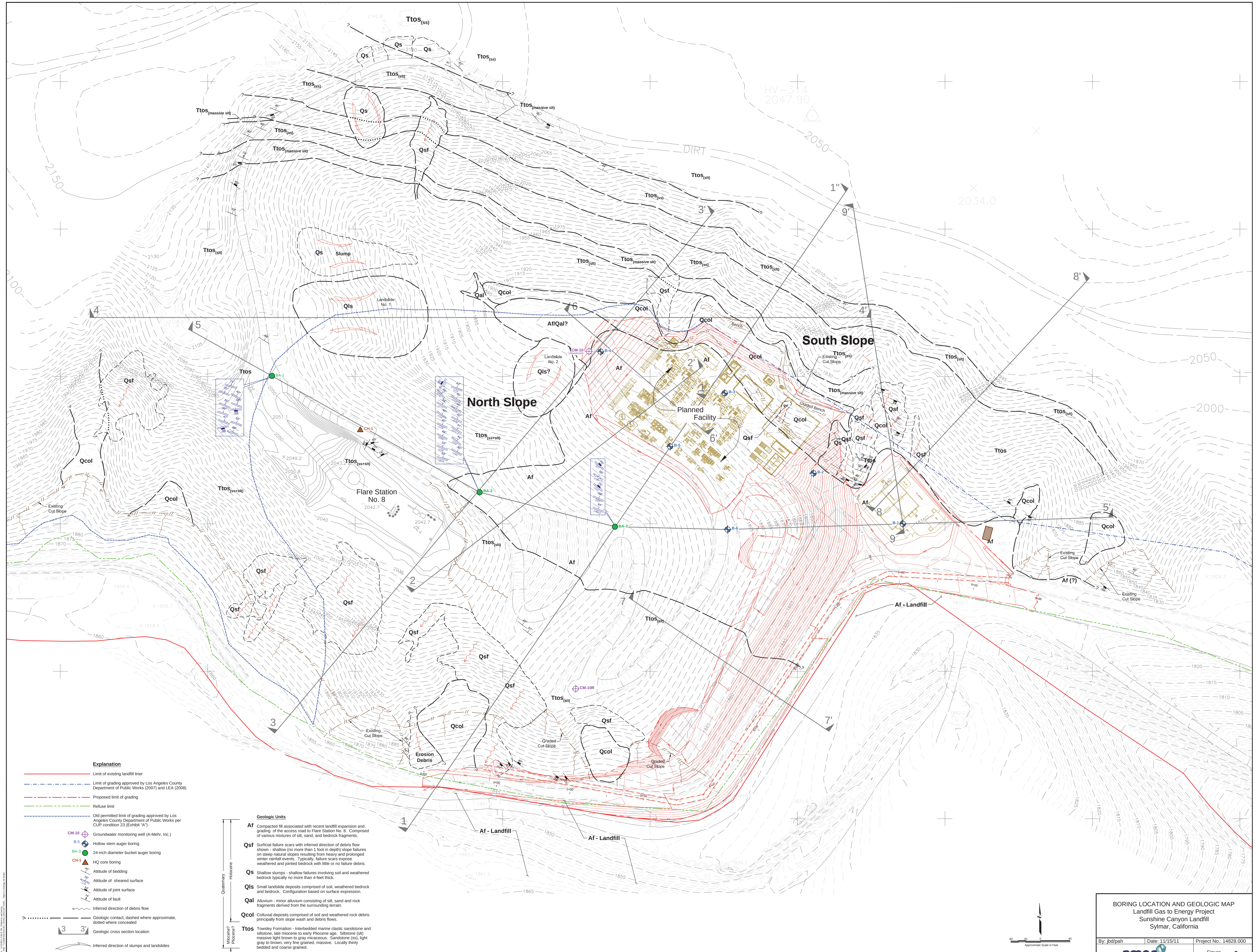


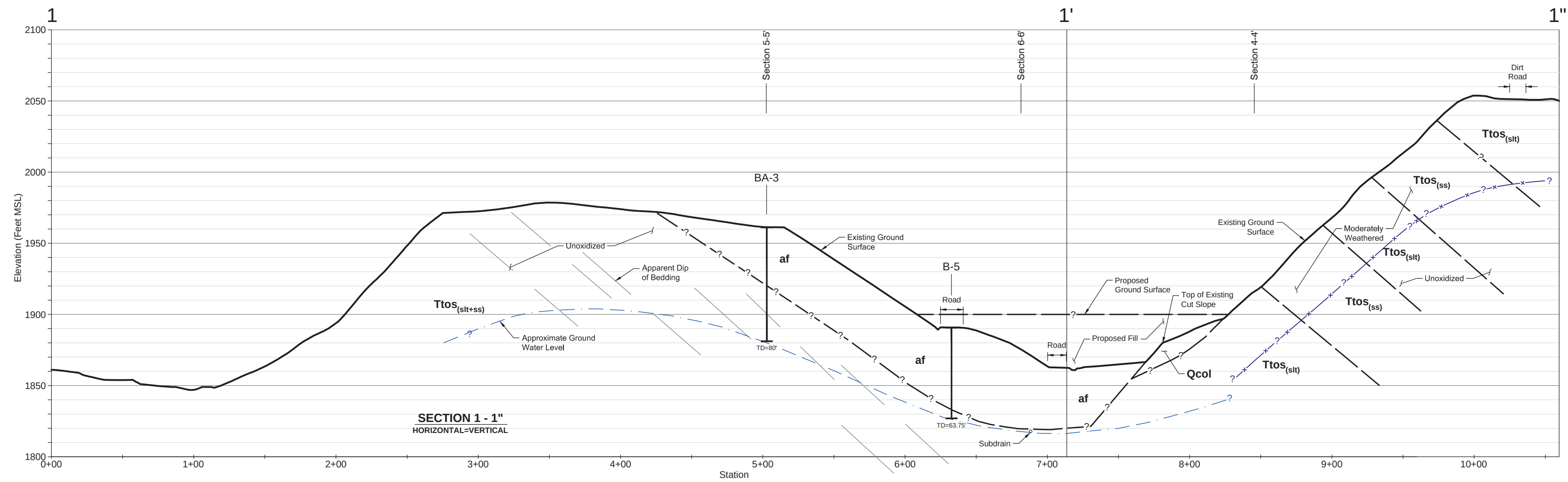
Basemap modified from Figure 1, "Overall Site Plan", Howard R. Green Company, January 2009 and November 2011.

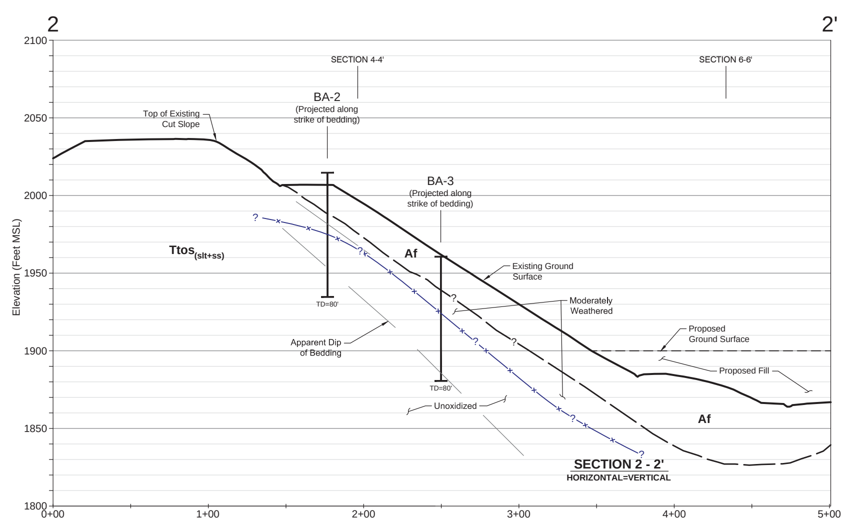
PROPOSED SITE LAYOUT
Landfill Gas to Energy Project
Sunshine Canyon Landfill
Sylmar, California

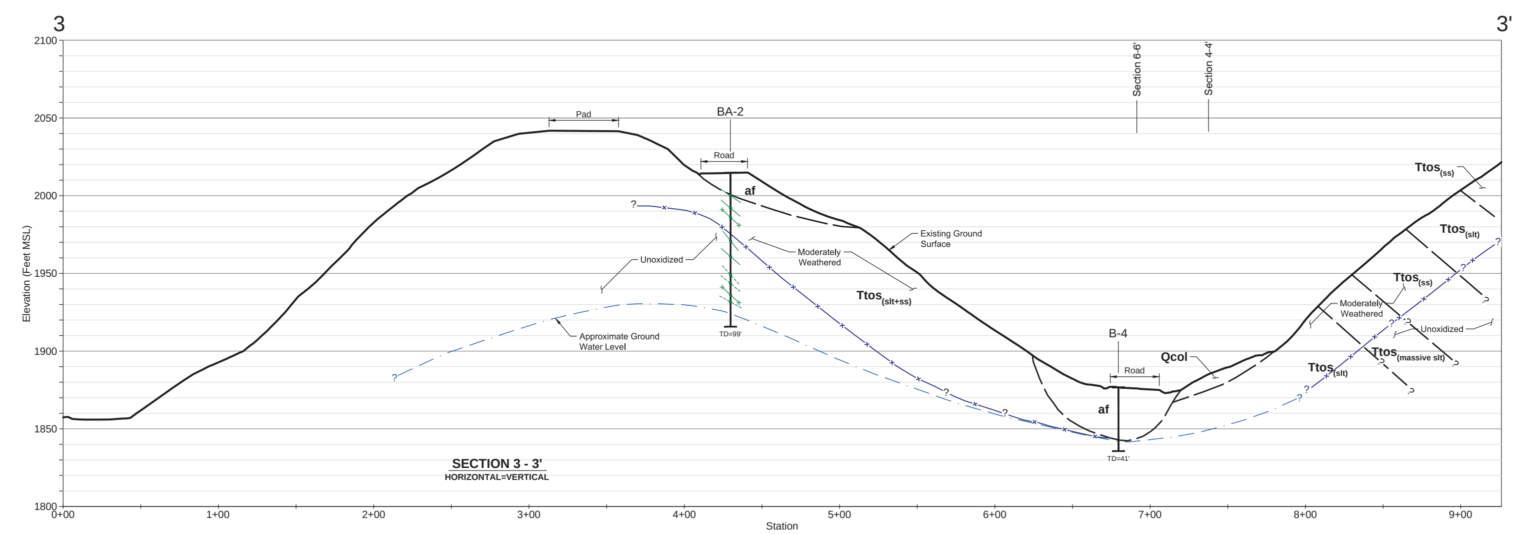
By: jbd/pah	Date: 11/15/11	Project No. 14828.000.0
amec		Figure 2

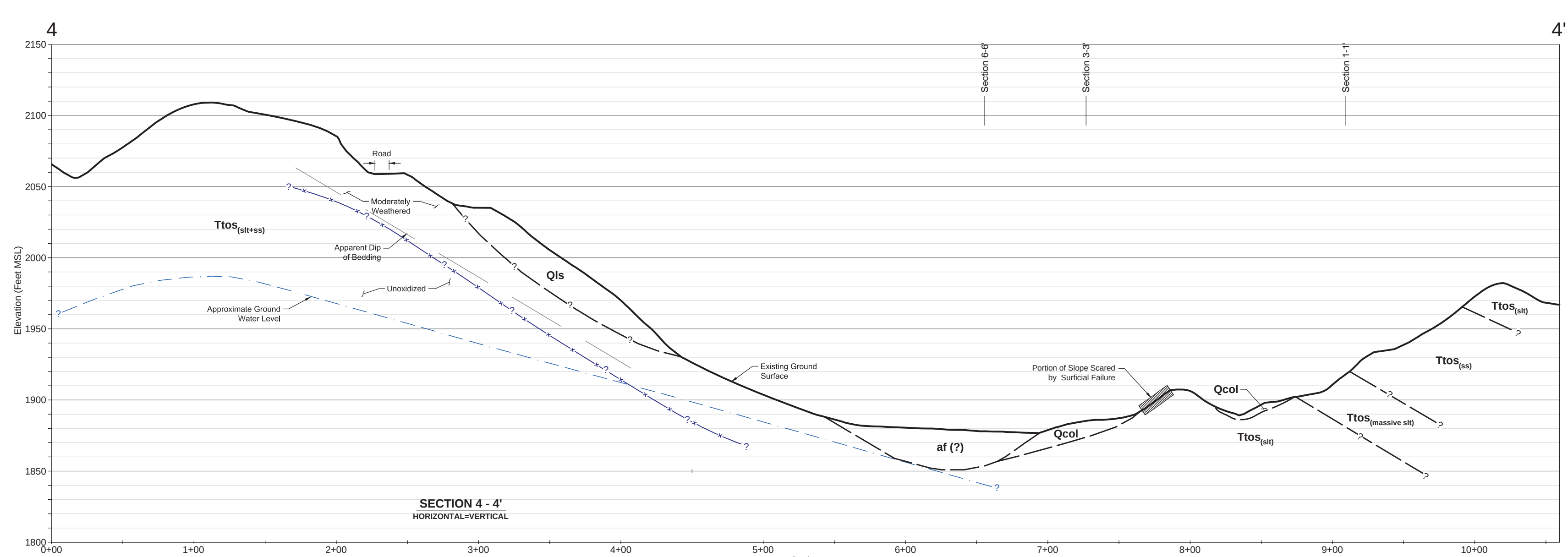
Plot Date: 11/15/2011 10:52:39 AM, Plotted by: pat.herring
Drawing Path: Y:\14828.000.0\ACAD_TB-002_PROP SITE LAYOUT.DWG , Figure 2_Proposed Site Layout


North Slope
South Slope
Planned Facility
Flare Station No. 8
Landslide No. 1
Landslide No. 2
Slump
Graded Bench
Existing Cut Slope
Graded Cut Slope
Erosion Debris
Af - Landfill
Af/Qal?
Qls?
Af (?)
Ttos(ss)
Ttos(slt)
Ttos(massive slt)
Ttos(ss+slt)
Qs
Qsf
Qls
Qal
Qcol
Af
CM-10
CM-10R
CH-1
BA-1
BA-2
BA-3
B-1
B-2
B-3
B-4
B-5
B-6
DIRT
HV-314
2047.90
Explanation
Limit of existing landfill liner
Limit of grading approved by Los Angeles County Department of Public Works (2007) and LEA (2008)
Proposed limit of grading
Refuse limit
Old permitted limit of grading approved by Los Angeles County Department of Public Works per CUP condition 23 (Exhibit "A")
Groundwater monitoring well (A-Mehr, Inc.)
Hollow stem auger boring
24-inch diameter bucket auger boring
HQ core boring
Attitude of bedding
Attitude of sheared surface
Attitude of joint surface
Attitude of fault
Inferred direction of debris flow
Geologic contact, dashed where approximate, dotted where concealed
Geologic cross section location
Inferred direction of slumps and landslides
Geologic Units
Quaternary
Holocene
Miocene? Pliocene?
Af Compacted fill associated with recent landfill expansion and grading of the access road to Flare Station No. 8. Comprised of various mixtures of silt, sand, and bedrock fragments.
Qsf Surficial failure scars with inferred direction of debris flow shown - shallow (no more than 1 foot in depth) slope failures on steep natural slopes resulting from heavy and prolonged winter rainfall events. Typically, failure scars expose weathered and jointed bedrock with little or no failure debris.
Qs Shallow slumps - shallow failures involving soil and weathered bedrock typically no more than 4-feet thick.
Qls Small landslide deposits comprised of soil, weathered bedrock and bedrock. Configuration based on surface expression.
Qal Alluvium - minor alluvium consisting of silt, sand and rock fragments derived from the surrounding terrain.
Qcol Colluvial deposits comprised of soil and weathered rock debris principally from slope wash and debris flows.
Ttos Towsley Formation - Interbedded marine clastic sandstone and siltstone, late miocene to early Pliocene age. Siltstone (slt) massive light brown to gray micaceous. Sandstone (ss), light gray to brown, very fine grained, massive. Locally thinly bedded and coarse grained.
40 20 0 40
Approximate Scale in Feet
Basemap modified from Figure 1, "Overall Site Plan", Howard R. Green Company, January 2009 and November 2011.
BORING LOCATION AND GEOLOGIC MAP
Landfill Gas to Energy Project
Sunshine Canyon Landfill
Sylmar, California
By: jbd/pah
Date: 11/15/11
Project No.: 14828.000
Figure 3









Geologic Units

Quaternary	Holocene	**Af**	Compacted fill associated with recent landfill expansion and grading of the access road to Flare Station No. 8. Comprised of various mixtures of silt, sand, and bedrock fragments.
		Qsf	Surficial failure scars with inferred direction of debris flow shown - shallow (no more than 1 foot in depth) slope failures on steep natural slopes resulting from heavy and prolonged winter rainfall events. Typically, failure scars expose weathered and jointed bedrock with little or no failure debris.
		Qs	Shallow slumps - shallow failures involving soil and weathered bedrock typically no more than 4-feet thick.
		Qls	Small landslide deposits comprised of soil, weathered bedrock and bedrock. Configuration based on surface expression. estimated depth as indicated.
		Qal	Alluvium - minor alluvium consisting of silt, sand and rock fragments derived from the surrounding terrain.
		Qcol	Colluvial deposits comprised of soil and weathered rock debris principally from slope wash and debris flows.
	Miocene? Pliocene?	**Ttos**	Towsley Formation - Interbedded marine clastic sandstone and siltstone, late miocene to early Pliocene age. SIltstone (slt) massive light brown to gray micaceous. Sandstone (ss), light gray to brown, very fine grained, massive. Locally thinly bedded and coarse grained.

Plot Date: 11/15/2011 10:18:19 AM , Plotted by: jim.depoorter
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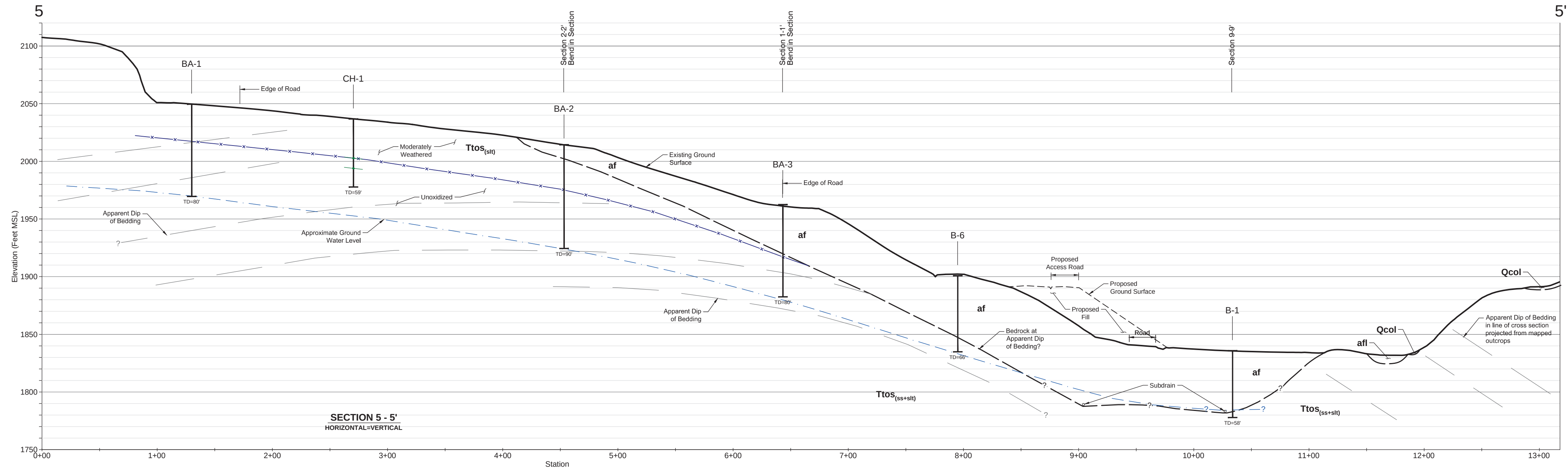
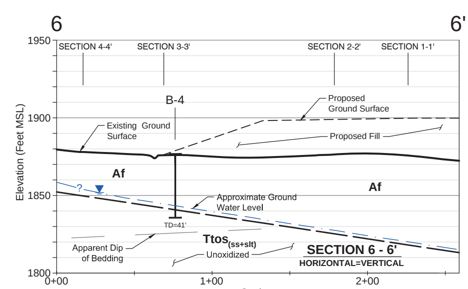
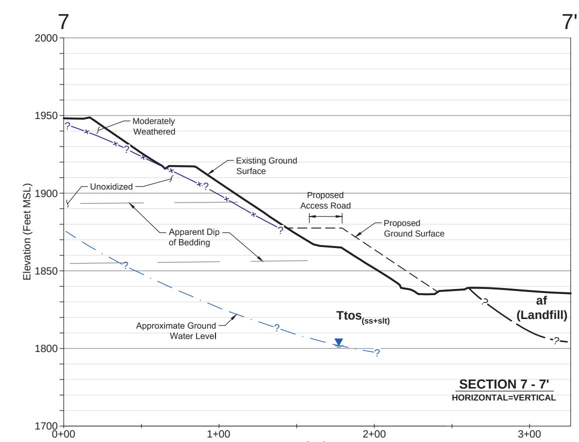
0 20 40
Approximate Scale in Feet
Horizontal = Vertical

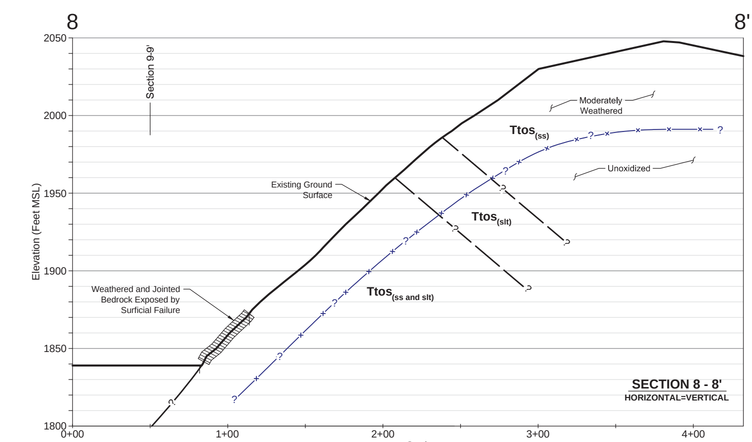
GEOLOGIC CROSS SECTIONS 1, 2, 3 & 4
Landfill Gas to Energy Project
Sunshine Canyon Landfill
Sylmar, California

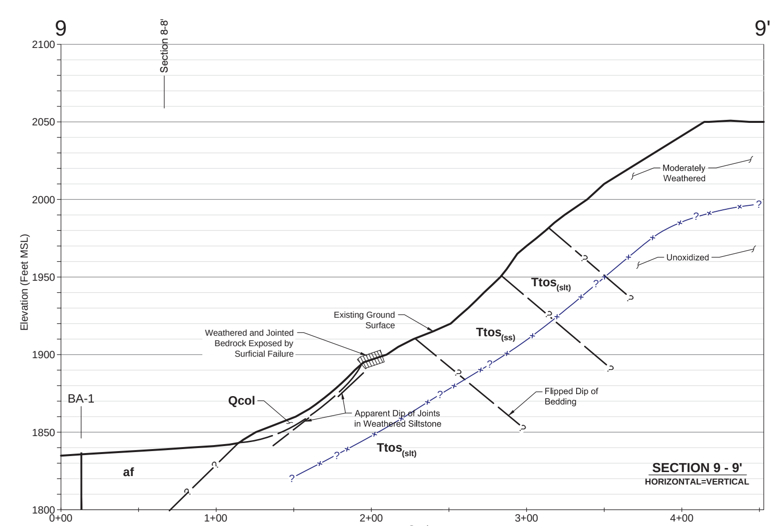
By: jbd	Date: 11/15/11	Project No.: 14828.000
amec		Figure 4











Geologic Units

Quaternary — Holocene

Af Compacted fill associated with recent landfill expansion and grading of the access road to Flare Station No. 8. Comprised of various mixtures of silt, sand, and bedrock fragments.

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Qcol Colluvial deposits comprised of soil and weathered rock debris principally from slope wash and debris flows.

Miocene? Pliocene?

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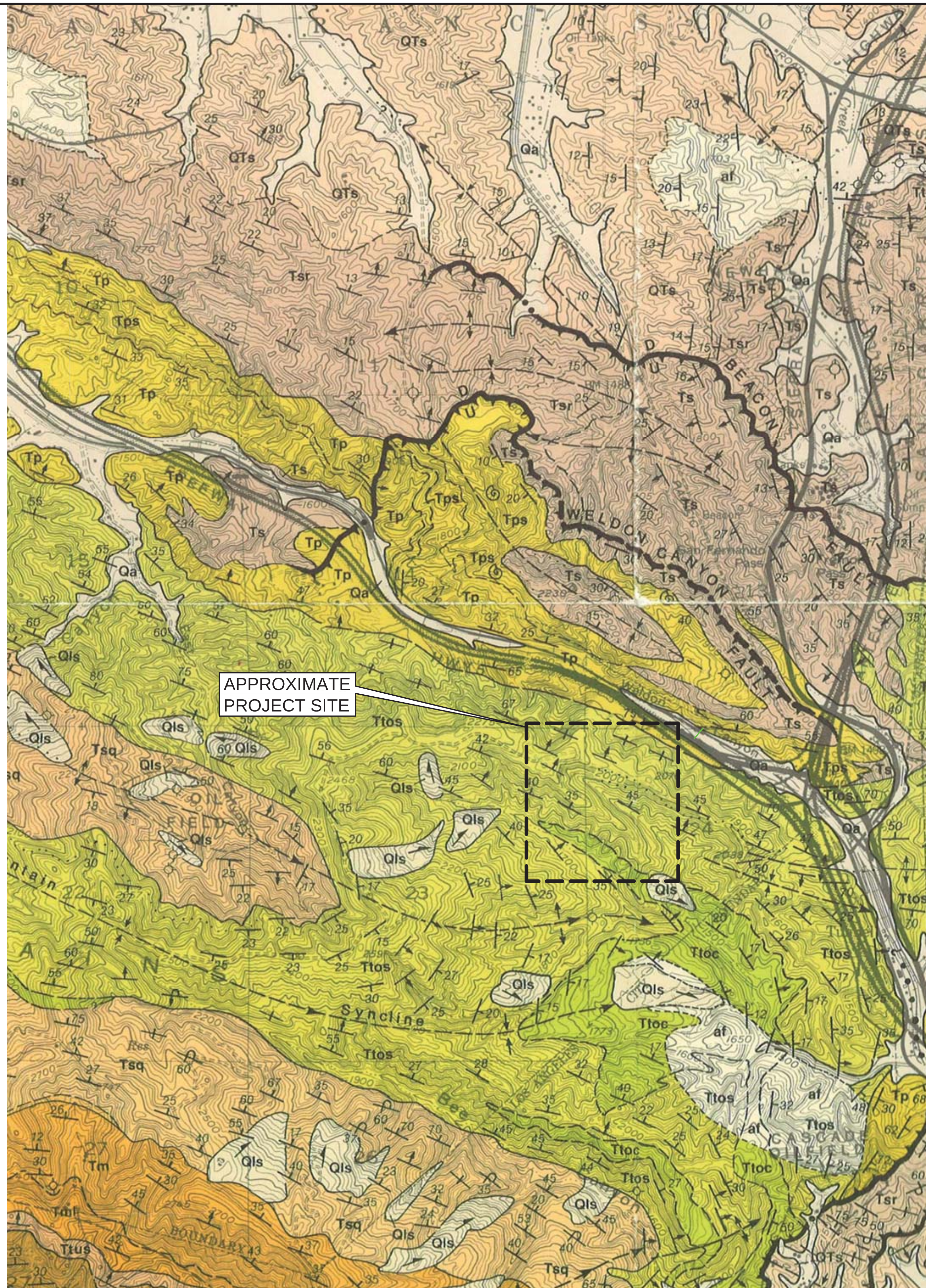
Drawing Path: Y:\14828.000.0\ACAD_TB-002_SECTIONS.DWG , GeoSections 5-9 (Fig 5)

0 20 40
Approximate Scale in Feet
Horizontal = Vertical

GELOGIC CROSS SECTIONS 5, 6, 7, 8 & 9
Landfill Gas to Energy Project
Sunshine Canyon Landfill
Sylmar, California

By: jbd	Date: 11/14/11	Project No.: 14828.000
amec		Figure 5



LEGEND

Holocene | Pleistocene | QUATERNARY

SURFICIAL SEDIMENTS
unconsolidated alluvial deposits; generally undissected
af artificial cut and fill
Qg gravel and sand of major stream channels
Qa alluvial gravel, sand and clay of valley and floodplain areas

LANDSLIDE DEBRIS
Qls

OLDER SURFICIAL SEDIMENTS
dissected, weakly consolidated alluvial deposits
Qos older sandy alluvium, including slope wash, derived from Chatsworth Formation (**Kcs**)
Qoa older alluvium composed largely of angular pebble-size fragments of Miocene shale and some of sandstone (**Tm**, **Tsq**, and **Ttos**) in light gray to tan silty matrix in part indurated by calcareous caliche; crudely bedded to massive; about 200 ft (60m) thick; blends northward upslope in Browns Canyon drainage area into old debris-flow landslides (**Qls**); slightly deformed and much dissected where elevated; but at Horse Flats top surface of deposition preserved; late Pleistocene age; mapped as slope wash, older alluvium, and Saugus Formation (upper member) by Barrows 1975; Evans and Miller 1978; and Saul 1979; probably in places equivalent to Pacoima Formation of Oakeshott 1958; Barrows et al. 1975; and Dibblee 1991

—UNCONFORMITY—

SAUGUS FORMATION
mostly terrestrial, weakly consolidated; Pleistocene and Pliocene age
QTs light gray to brown pebble-cobble conglomerate, sandstone and lesser amounts of grayish to reddish brown, soft siltstone/claystone; conglomerate composed of granitic, gneissic, metavolcanic, quartzitic, gabbroic and anorthositic detritus in sandy matrix; deposited by westward-flowing streams; Pleistocene age; south of Santa Susana fault mapped as middle member of Saugus Formation by Barrows et al. 1974, Evans and Miller 1978, and Saul 1979 [in adjacent San Fernando quadrangle (Dibblee 1991) **QTs** west of Elsmere Canyon should be shown as **Ts**]
Ts (in Newhall area) similar to **QTs**, but correlative in age with units **Tsr** and **Tps** in part; probably Pliocene age
Tsr Sunshine Ranch Member (of Hazzard 1940, in Treiman 1987; Barrows et al. 1974; Evans and Miller 1978; and Saul 1979; type area extends eastward from lower Aliso Canyon to Van Norman Reservoir, Hazzard 1940); terrestrial deposits similar to **QTs**, but south of Santa Susana fault composed largely of more indurated greenish gray claystone, siltstone, and fine grained sandstone, and contains in lower part brackish marine layers of oyster shells; in exposures northwestward from San Fernando Pass, consists mostly of interbedded conglomerate and fine grained sediments that locally contain few thin layers of peat, unit intertongues westward into **Tps**; mostly Pliocene age

Pliocene

PICO FORMATION
marine clastic; mostly Pliocene age
Tps south of Santa Susana fault: Pico sandstone (included in Saugus Formation by Kew 1924; Pico Formation by Butler 1977, Lant 1977, Yeats 1987; Pico and Saugus Formations by Evans and Miller 1978) mostly light gray to nearly white, soft friable sandstone, locally pebbly, contains abundant whole and fragmented bivalve shells west of Browns Canyon; deposited under marine to lagoonal conditions; grades upward into terrestrial Saugus Formation; unconformable on Miocene formations
Tpg conglomerate in lower Limekiln Canyon: gray massive conglomerate of cobbles of granitic and metavolcanic rocks in sandstone matrix; nonmarine (?), unconformable on Monterey Shale (**Tml**), overlain by Saugus Formation
Tp and **Tps** north of Santa Susana fault: Pico Formation of Kew 1924, Winterer and Durham 1958, 1962; **Tps** mostly light gray semi-friable sandstone, locally pebbly; upper beds contain bivalve shell fragments; intertongues into Saugus Formation (**Ts**); **Tp** mostly gray micaceous siltstone-claystone, bedded to massive, includes few thin sandstone layers

TOWSLEY FORMATION
marine clastic; early Pliocene age (Repettian Stage) and possibly latest Miocene ("Delmontian Stage")
Ttos light gray to tan coherent to semi-friable sandstone, medium grained to locally gritty and pebbly, bedded; includes minor micaceous siltstone; grades laterally northward into Saugus Formation (**Ts**) in San Fernando Pass area
Ttoc gray micaceous silty claystone and siltstone; minor sandstone

SISQUOC SHALE
(included in Modelo Formation by Kew 1924; Winterer and Durham 1958, 1962; Saul 1979; same lithologic unit as Sisquoc Formation in northern Ventura basin)
marine clastic; late Miocene age (Mohnian to "Delmontian" Stage)
Tsq dark gray to brownish gray clay shale, bleaches to light gray; crumbly with spheroidal to sub-platy fracture, gypsiferous in fractures, some layers contain large tan dolomitic concretions; includes some thin interbedded semi-siliceous layers; about 1000 ft (300m) thick
Tsqs light gray coherent to semi-friable sandstone

Miocene | TERTIARY

MONTEREY SHALE
(Modelo Formation of Kew 1924; Winterer and Durham 1958, 1962; Barrows et al. 1974; Evans and Miller 1978; Saul 1979; same lithologic unit as Monterey Shale of northern Ventura basin)
marine biogenic; nearly 2000 ft (610m) thick north of Santa Susana fault, very thin south of it; middle and late Miocene age
Tm upper part: thin bedded siliceous shale, dark gray brown but weathers cream-white, hard, platy, brittle, porcelaneous, locally cherty, closely fractured, some layers fissile, about 1500 ft (485m) thick; middle and late Miocene age (mostly Mohnian Stage); south of Santa Susana fault: soft, white weathering thin-bedded, fissile diatomaceous semi-siliceous shale
Tml lower part: thin-bedded, fissile semi-siliceous shale to soft shaly claystone, dark brown, weathers cream white; includes some calcareous shale, and thin tan-weathering hard dolomite strata that are increasingly abundant upward, unit as thick as 500 ft (150m); middle Miocene age (Luisian-Relizian Stage)

DETRITAL SEDIMENTS OF LINDERO CANYON
(included in Topanga Formation of Weber 1984; unconformable on Chatsworth Formation; best exposed in Lindero Canyon, Thousand Oaks quadrangle)
marine transgressive clastic; Miocene age [Luisian (?) Stage]
Tlsc crudely bedded brown to gray conglomerate composed of cobbles of metavolcanic, granitic and quartzitic rocks, and of sandstone derived from Chatsworth Formation; sparsely fossiliferous; contains minor interbeds of light gray calcareous sandstone

TOPANGA FORMATION
(of Yeats 1979; Topanga Group of Saul 1979)
marine transgressive clastic; middle Miocene age
Ttus upper sandstone: light gray to white semi-friable, locally pebbly; massive to vaguely bedded; about 150 ft (45m) thick
Tb basalt flow (or diabase sill?); black, massive fine grained weakly coherent where weathered, present north of Santa Susana fault only; probably correlative with Conejo Volcanics to west
Ttls lower sandstone: light gray to tan, semi-friable to somewhat hard, massive to vaguely bedded; as thick as 600 ft (180m), base unexposed; present north of Santa Susana fault only

—UNCONFORMITY—

Eocene

LLAJAS FORMATION
marine clastic; middle Eocene age
Tll gray micaceous claystone-siltstone and some interbeds of light gray to tan soft sandstone
Tllg basal cobble conglomerate as thick as 50 ft (17m) with cobbles of granitic, metavolcanic and quartzitic rocks

Paleocene

SANTA SUSANA FORMATION
mostly marine clastic; Paleocene age
Tsu dark gray micaceous claystone, crumbly with ellipsoidal fracture; includes few thin sandstone layers
Tsi Simi Conglomerate Member: gray to brown cobble conglomerate with smooth rounded cobbles up to 1 ft (1/3m) in diameter of pinkish-gray quartzite, metavolcanic and granitic rocks in gray sandstone matrix; possibly in part nonmarine, deposited as fan delta

Upper | CRETACEOUS

CHATSWORTH FORMATION
(of Colburn 1981; "Chico" Formation of Kew 1924; Evans and Miller 1978)
marine clastic; late Cretaceous age (Maestrichtian and Campanian Stages
Kcsh gray micaceous shale and siltstone; includes some sandstone strata; included in Martinez Formation (Paleocene) by Evans and Miller 1978
Kcs light gray to light brown, hard, thick bedded sandstone; locally gritty; includes few thin layers of micaceous siltstone; base not exposed

SYMBOLS
(not all symbols present on each map)

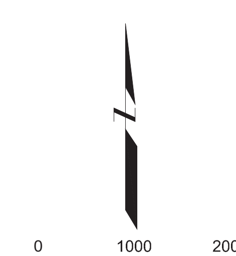
FORMATION CONTACT dashed where inferred or indefinite
MEMBER CONTACT between units of a formation
CONTACT BETWEEN SURFICIAL SEDIMENTS located only approximately in places

FAULT: Dashed where indefinite or inferred, dotted where concealed, queried where existence is doubtful. Parallel arrows indicate inferred relative lateral movement. Relative vertical movement is shown by U/D (U=upthrown side, D=downthrown side). Short arrow indicates dip of fault plane. Sawteeth on upper plate of low angle thrust fault.
U D 25 ?

FOLDS: ANTICLINE SYNCLINE
arrow on axial trace of fold indicates direction of plunge; dotted where concealed by surficial sediments

STRIKE AND DIP OF STRATIFIED ROCKS
18 inclined; horizontal; 20 inclined (approximate); 80 overturned; vertical; 75 inclined; approximate; vertical; metamorphic or igneous rock foliation or flow banding

FOSSIL LOCALITY; DIRECTION OF LANDSLIDE MOVEMENT; SANDSTONE MARKER BED; CONGLOMERATE MARKER BED; ABANDONED EXPLORATORY OIL (OR GAS) WELL; OIL WELLS (ON TOPOGRAPHIC BASE MAP)



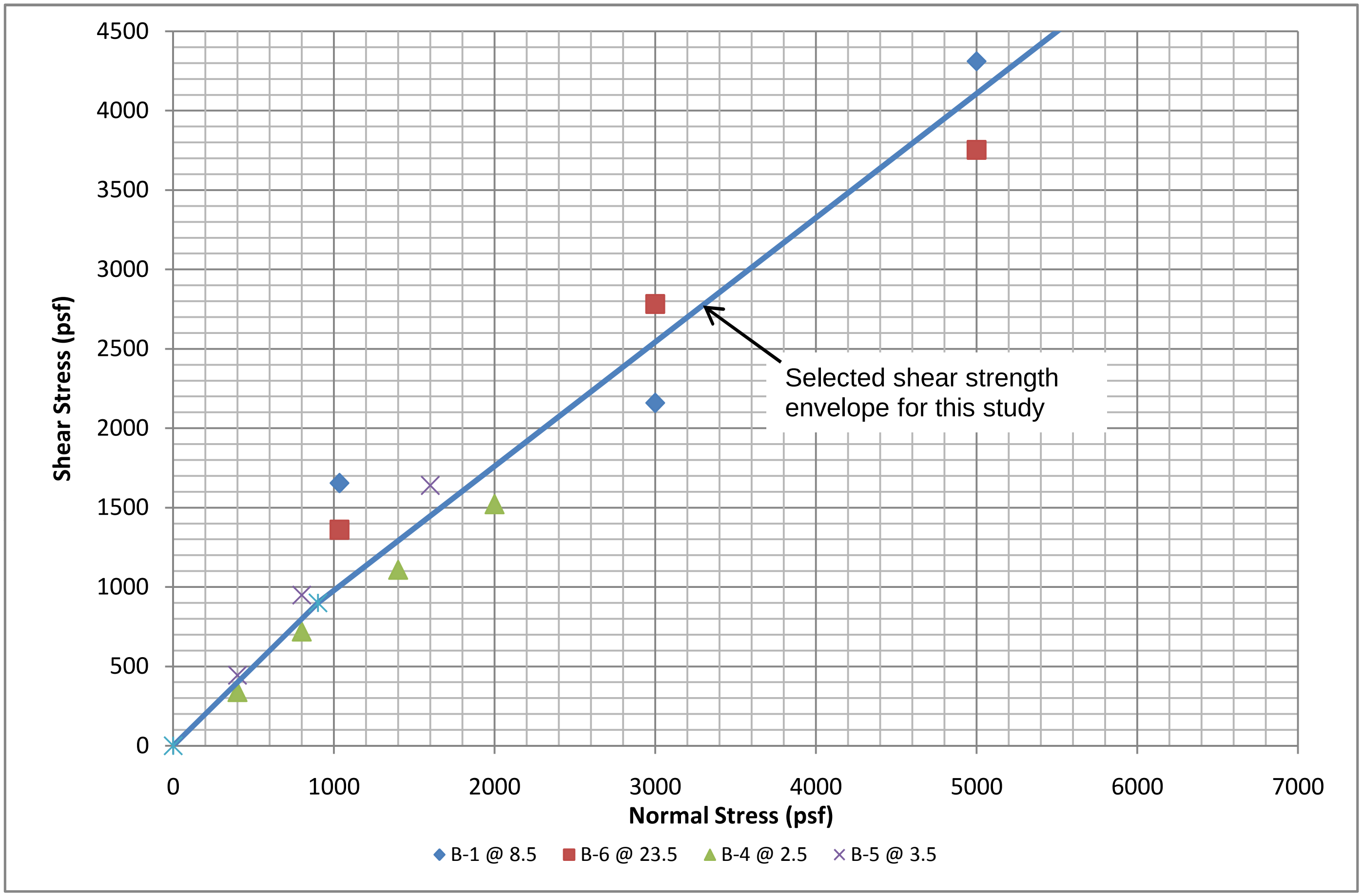
Approximate Scale in Feet

Reference: T.W. and Ehrenspeck, H.E., ed., 1992, Geologic map of the Oat Mountain and Canoga Park (north 1/2) quadrangles, Los Angeles County, California: Dibblee Geological Foundation, Map DF-36, scale 1:24000.

REGIONAL GEOLOGY MAP
Landfill Gas to Energy Project
Sunshine Canyon Landfill
Sylmar, California

By: jbd	Date: 11/11/11	Project No. 14828.000.0
amec		Figure 6

P:\14828.000.0\Docs\Geologic and Geotechnical Investigation Report\Figures\Figures 7-8-9.xls



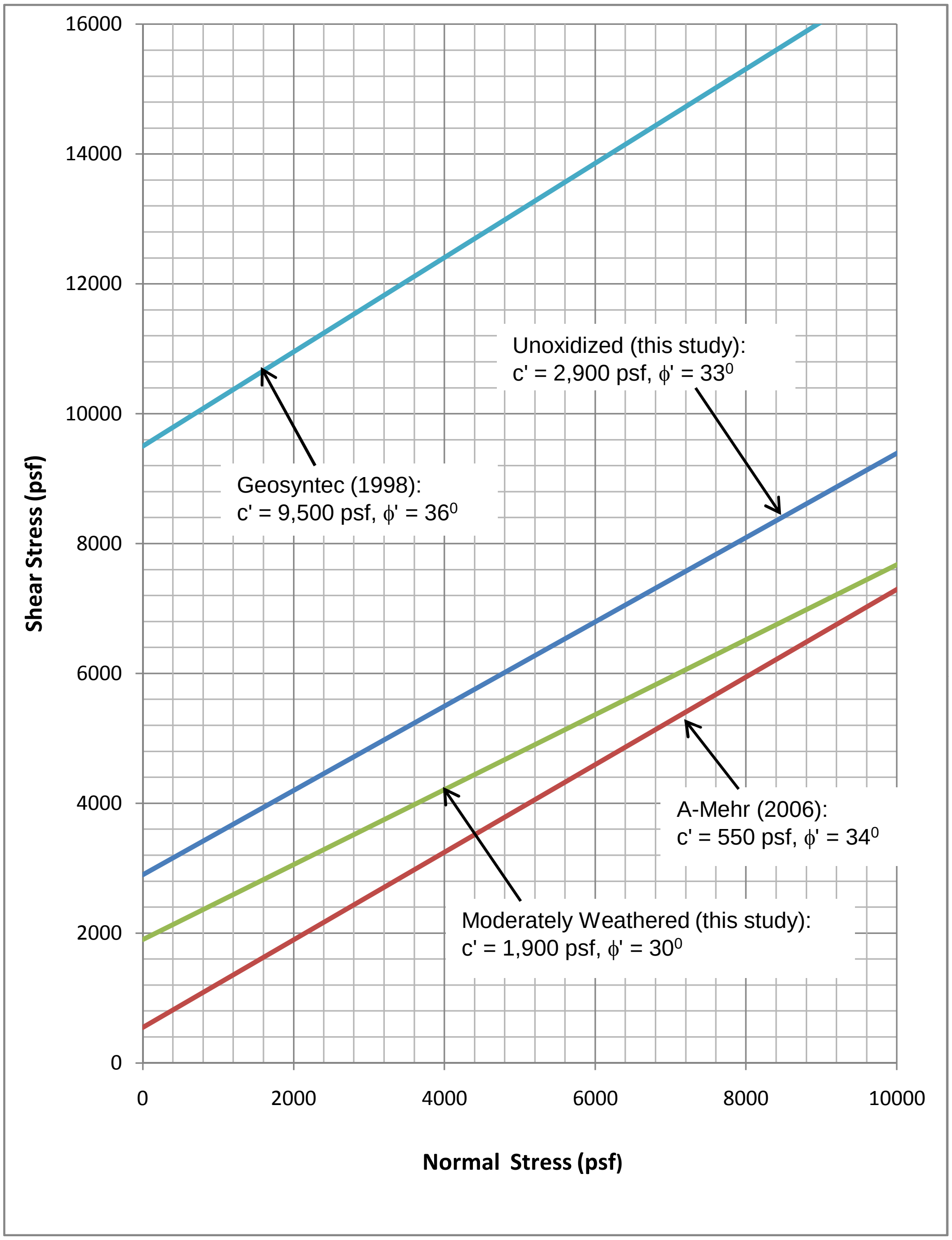
Notes:
1. Direct shear test results based on shear resistance at 20% lateral displacements per ASTM D3080.
2. ϕ = internal friction angle.
3. c = cohesion.
4. psf = pounds per square foot.

Bilinear Shear Strength Envelope

Normal	Shear
0	0
900	900
6000	4888

SHEAR STRENGTH ENVELOPE
EXISTING FILL
Landfill Gas to Energy Project
Sunshine Canyon Landfill
Sylmar, California

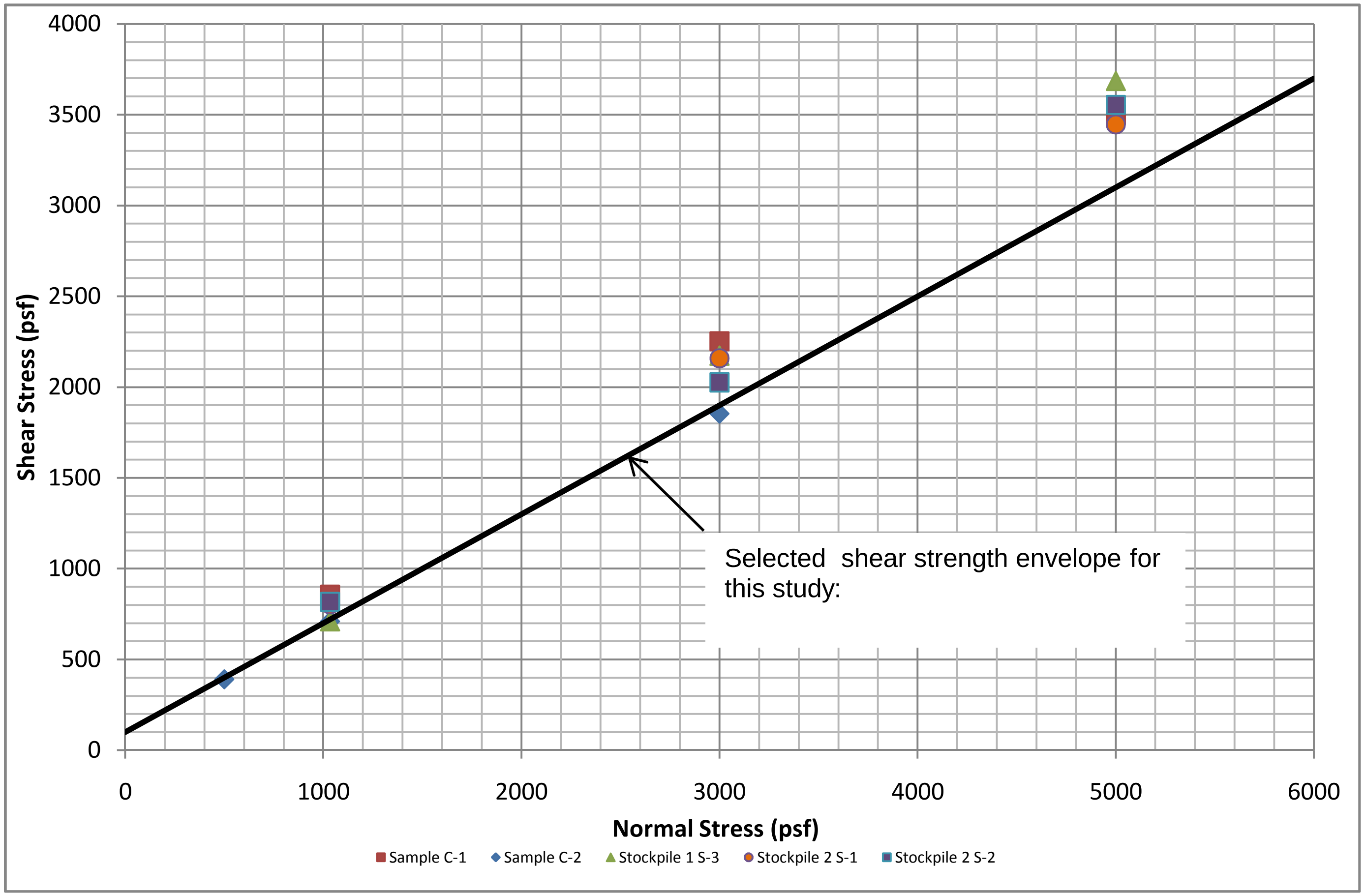
By: ERF	Date: 11/11/11	Project No.: 14828.000
amec		Figure 7



Notes:
1. psf = pounds per square foot.
2. c' = cohesion.
3. ϕ' = internal friction angle.

By: ERF	SHEAR STRENGTH ENVELOPE BEDROCK Landfill Gas to Energy Project Sunshine Canyon Landfill Sylmar, California	Project No.: 14828.000.0
Date: 11/11/11		Figure **8**

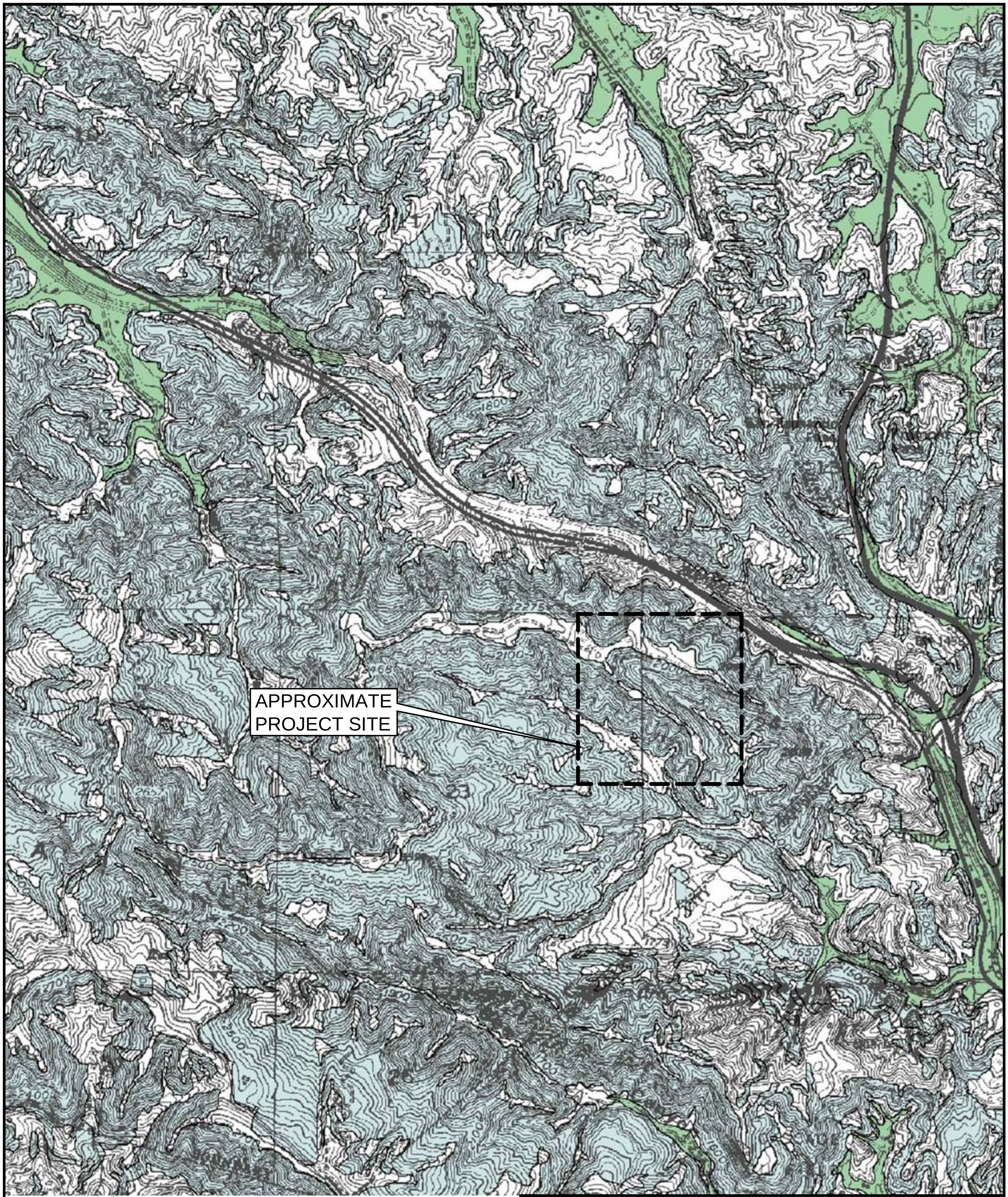
amec



<u>Notes</u>:
1. Direct shear test results based on shear resistance at 20% lateral displacements per ASTM D3080.
2. ϕ = internal friction angle.
3. c = cohesion.
4. psf = pounds per square foot.

SHEAR STRENGTH ENVELOPE
PROPOSED FILL
Landfill Gas to Energy Project
Sunshine Canyon Landfill
Sylmar, California

By: ERF	Date: 11/11/11	Project No.: 14828.000
amec		Figure 9



0 1000 2000

Approximate Scale in Feet

MAP EXPLANATION

Zones of Required Investigation:

Liquefaction

Areas where historic occurrence of liquefaction, or local geological, geotechnical and groundwater conditions indicate a potential for permanent ground displacements such that mitigation as defined in Public Resources Code Section 2693(c) would be required.

Earthquake-Induced Landslides

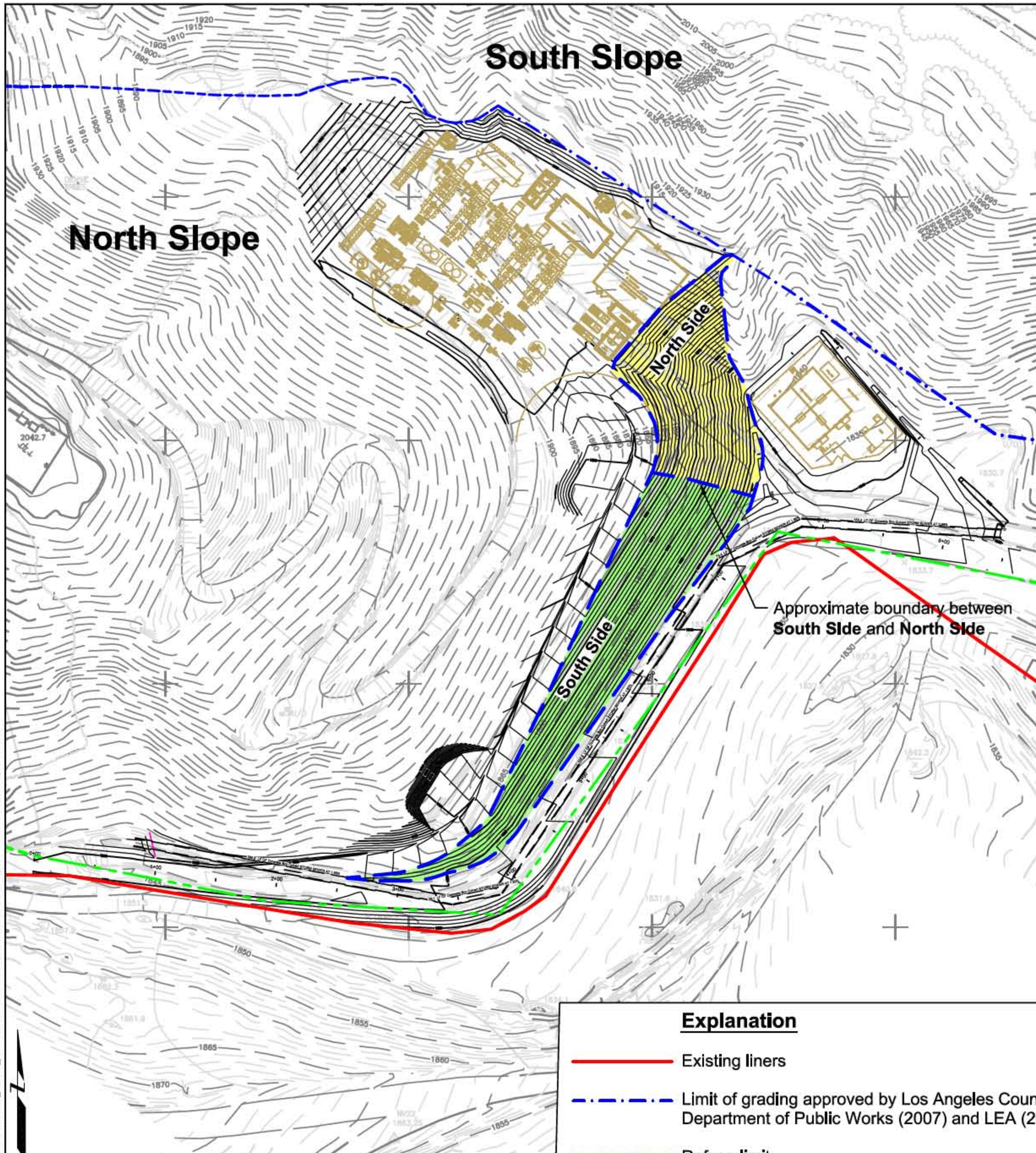
Areas where previous occurrence of landslide movement, or local topographic, geological, geotechnical and subsurface water conditions indicate a potential for permanent ground displacements such that mitigation as defined in Public Resources Code Section 2693(c) would be required.

Seismic Hazard Evaluation of the Oat Mountain 7.5 minute quadrangle, Los Angeles County, California: California Department of Conservation Division of Mines and Geology Open-File Report 97-12.

SEISMIC HAZARD ZONES
Landfill Gas to Energy Project
Sunshine Canyon Landfill
Sylmar, California

By: jbd	Date: 11/11/11	Project No. 14828.000.0
	amec	Figure 10

Plot Date: 11/11/2011 2:31:24 PM, Plotted by: pat.herring
Drawing Path: Y:\14828.000.0\ACAD_TB-002_SEISMIC HAZARD.DWG , Seismic



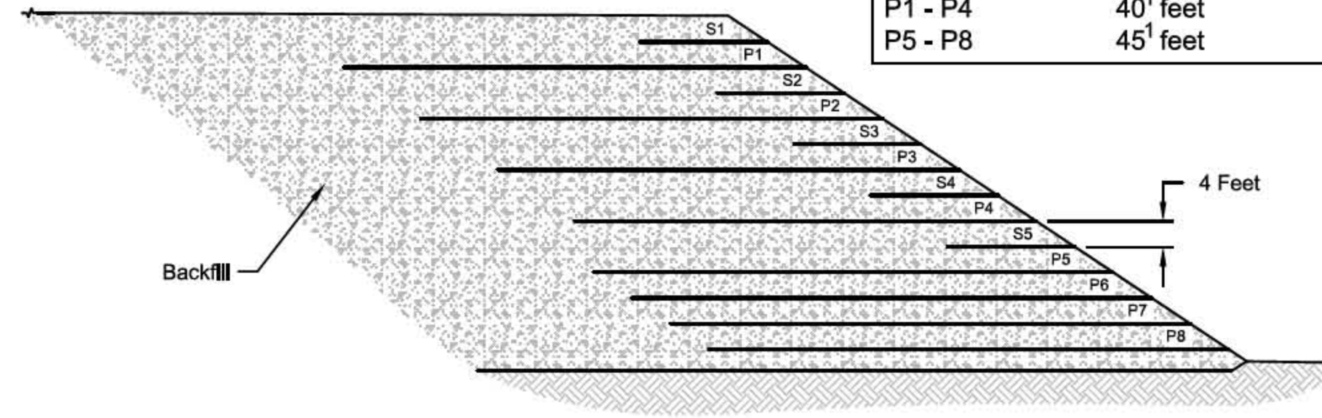
0 50 100
Approximate Scale in Feet

Basemap modified from Figure 1, "Overall Site Plan", Howard R. Green Company, January 2009 and November 2011.

Explanation

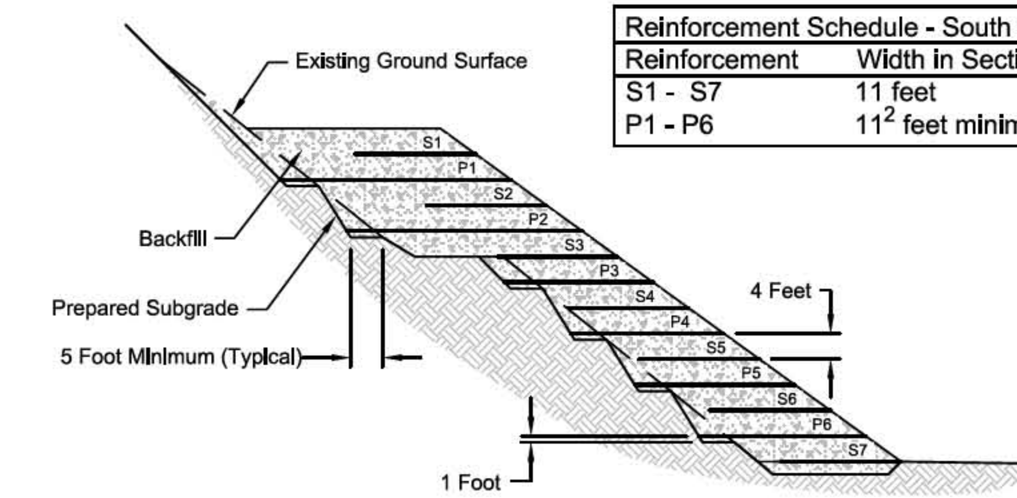
- Existing liners
- Limit of grading approved by Los Angeles County Department of Public Works (2007) and LEA (2008)
- Refuse limit
- Old permitted limit of grading approved by Los Angeles County Department of Public Works per CUP condition 23 (Exhibit "A")

Reinforcement Schedule - North Side	
Reinforcement	Width in Section
S1 - S5	11 feet
P1 - P4	40[1] feet
P5 - P8	45[1] feet



REINFORCEMENT DETAIL
NORTH SIDE
(Not to Scale)

Reinforcement Schedule - South Side	
Reinforcement	Width in Section
S1 - S7	11 feet
P1 - P6	11[2] feet minimum



REINFORCEMENT DETAIL
SOUTH SIDE
(Not to Scale)

Notes:

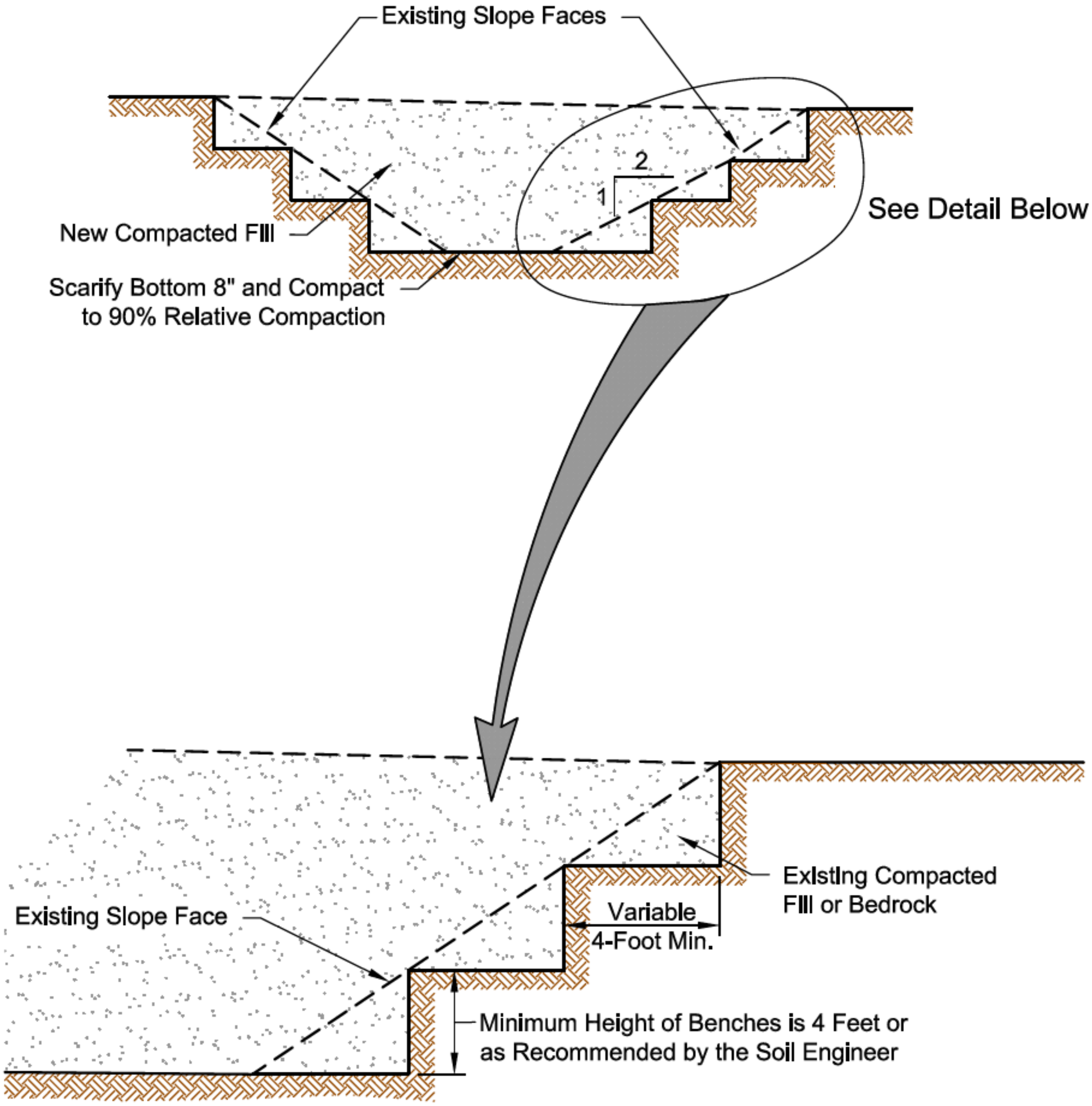
1. The typical width in section of primary reinforcements P1 through P4 should be 40 feet and P5 through P8 should be 45 feet within the north side of the reinforced slope. If the width of the fill is less than these required lengths, then refer to typical section for the south side for benching detail.

2. The minimum width of primary reinforcement should be 11 feet. A minimum of 5-foot wide benches should be excavated into the existing ground surface at an elevation of 1 foot below each primary reinforcement. The primary reinforcement should be placed from the face of the proposed slope to the heel of the bench excavation.

REINFORCEMENT LAYOUT AND DETAILS
Landfill Gas to Energy Project
Sunshine Canyon Landfill
Sylmar, California

By: jbd/pah	Date: 11/15/11	Project No. 14828.000.0
amec		Figure 11

Plot Date: 11/15/2011 5:38:02 PM, Plotted by: pat.herring
Drawing Path: Y:\14828.000.0\ACAD_TB-002_REINFORCEMENT LAYOUT-DETAILS.DWG , Figure 11-Reinforcement Layout and Detail



Not to Scale

TYPICAL BENCHING DETAILS
INTO EXISTING SLOPES
Landfill Gas to Energy Project
Sunshine Canyon Landfill
Sylmar, California

By: pah	Date: 11/15/11	Project No. 14828.000.0
amec		Figure 12

Plot Date: 11/15/2011 5:37:09 PM, Plotted by: pat.herring
Drawing Path: Y:\14828.000.0\ACAD_TB-002_BENCHING_DETAIL.DWG , Figure 12-Typical Bench Detail

APPENDIX A

Boring Logs

EXPLANATION OF BORING LOGS

SUNSHINE CANYON LANDFILL

MAJOR DIVISIONS		LTR	DESCRIPTION
COARSE GRAINED SOILS	GRAVEL	GW	Well-graded gravels or gravel-sand mixtures, little or no fines
		GP	Poorly-graded gravels or gravel-sand mixture, little or no fines
		GM	Silty gravels, gravel-sand-silt mixtures
		GC	Clayey gravels, gravel-sand-clay mixtures
	SAND	SW	Well-graded sands or sand with gravel, little or no fines
		SP	Poorly-graded sands or sand with gravel, little or no fines
		SM	Silty sands, sand-silt mixtures
		SC	Clayey sands, sand-clay mixtures

MAJOR DIVISIONS		LTR	DESCRIPTION
FINE GRAINED SOILS	SILTS AND CLAYS LL<50	ML	Inorganic silts and very fine sand, 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 silt-clays of low plasticity
	SILTS AND CLAYS LL>50	MH	Inorganic silts, micaceous or diatomaceous fine sandy or silty soils, elastic silts
		CH	Inorganic clays of hogh plasticity, fat clays
		OH	Organic clays of medium to high plasticity
HIGHLY ORGANIC SOILS		PT	Peat and other highly organic soils

SAMPLE COLUMN SYMBOLS

Standard penetration test (SPT) sample

Modified California split spoon sample

P Piston sample

Shelby tube sample

Pitcher tube sample

Continuous soil or rock core

NR No recovery

BLOWS/FOOT - Summation of blow counts for deepest 12 inches is sampling interval

RQD% - Rock quality designation in percent

DESCRIPTION COLUMN SYMBOLS

Dashed lines separating soil strata represent inferred boundaries between sampled intervals or no recovery intervals and may be distinct or gradual transitions

Solid lines represent distinct or gradual boundaries observed within sampled intervals

Description right of bracket symbol represents soil conditions within the depth interval defined by the bracket length

Description right of arrow symbol represents soil conditions to the next deeper boundary line unless otherwise noted

Water level at time of drilling

Water level after at least 12 hours from time of drilling

LABORATORY TEST ABBREVIATIONS

ATT	Atterberg Limits	CORR	Corrosion	SE	Sand Equivalent
COLL	Collapse Potential	DS	Direct Shear	SG	Specific Gravity
COMP	Compaction	EI	Expansion Index	TX	Triaxial Test
CON	Consolidation	S	Grain Size Analysis	UC	Unconfined Compression Test
R	R-Value	PERM	Permeability	#200	No. 200 Wash Sieve Analysis

NOTES

1. Soil descriptions are in accordance with the USCS as set forth by ASTM D2488 "Standard Practice for Description and Identification Soil (Visual-Manual Procedure)."
2. Soil color described according to Munsell Soil Color Chart. Rock color described according to Munsell Rock-Color Chart
3. Soil descriptions in these borings are generalized representations and based upon visual classification of cuttings and/or samples during drilling. Descriptions and related information in these borings depict subsurface conditions at the specific location and at the time of drilling only. Soil conditions at other locations may differ from conditions observed at the boring locations. Also, soil and groundwater conditions may change with time at these locations.

EXPLANATION OF BORING LOGS

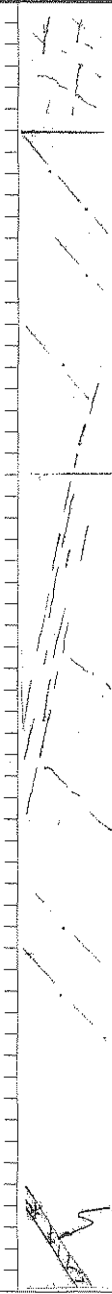
SUNSHINE CANYON LANDFILL

	Sandy Clay and Clayey Sand		Siltstone
	Silty Sandstone		

P:\14828.000.0\GINT\14828.000.0 ROCK LOGS.GPJ

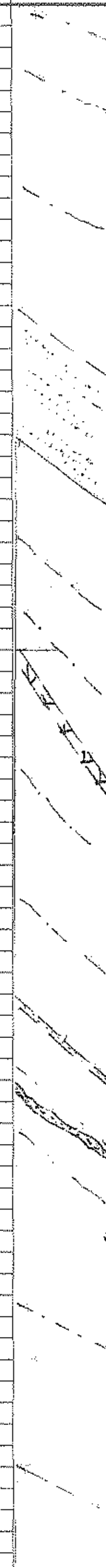
PROJECT: SUNSHINE CANYON LANDFILL Sylmar, California	**Downhole Log of Boring No. BA-1**			
BORING LOCATION: N: 4234600.7 E: 4130287.2	ELEVATION AND DATUM: 2049' above mean sea level			
DRILLING CONTRACTOR: Roy Bros. Drilling	DATE STARTED: 7/14/10		DATE FINISHED: 7/14/10	
DRILLING METHOD: 24" Bucket Auger	TOTAL DEPTH (ft.): 80.0		MEASURING POINT: Ground Surface	
DRILLING EQUIPMENT: E-Z Bore	DEPTH TO WATER	FIRST 79	COMPL. 79	24 HRS. NA
SAMPLING METHOD: NA	LOGGED BY: M. McLarty			
HAMMER WEIGHT: NA DROP: NA	RESPONSIBLE PROFESSIONAL: M. McLarty			REG. NO. CEG 1107

DEPTH (feet)	SAMPLES: Sample No.	Sample	Blows/ 6 inches	DESCRIPTION NAME (USCS): color, moist, % by wt., plast. density, structure, cementation, react. w/HCl, geo. inter. Surface Elevation: 2049' above mean sea level	GRAPHIC LOG	GEOLOGIC DATA
0–3				<u>TOWSLEY FORMATION (Ttos)</u> SILTSTONE: brownish yellow (10YR 6/6) to yellowish brown (5YR 5/4), micaceous, highly weathered, jointed and fractured		
3				@3.0' massive, micaceous, less weathered		
4						Bedding at 4' N70°, 49°NE
10				@10.0' becomes brownish yellow (10YR 6/6) to light gray (7.5YR 7/1)		
18						Attitude of joints at 18' N80°E, 80°SW
19				@19.0' becomes very hard below 19', massive		
22						Bedding at 22' N70°W, 47°, NE
25				@25.0' becomes predominantly light gray (7.5YR 7/1)		
29				@29.0' crushed siltstone, 1" to 2" wide, reddish brown (5YR 5/4) to dark gray (5YR 4/1)		Attitude of crushed zone at 29' N70°W, 55°NE
30						



PROJECT: SUNSHINE CANYON LANDFILL
Sylmar, California

Downhole Log of Boring No. BA-1 (cont'd)



DEPTH (feet)	SAMPLES: Sample No.	Sample	Blows/ 6 inches	DESCRIPTION NAME (USCS): color, moist, % by wt., plast. density, structure, cementation, react. w/HCl, geo. inter.	GEOLOGIC DATA
				<u>TOWSLEY FORMATION (Ttos)</u> SILTSTONE: continued	
31				siltstone becomes dark gray (5YR 4/1) and unoxidized below 29.0'	
32					
33					
34					
35					Bedding at 35' N72°W, 55°NE
36					Attitude of joints at 35' EW85°S
37				@37.0' SANDSTONE: greenish gray (GLEY1 5/1) to bluish gray (GLEY2 5/1), very fine to fine-grained, micaceous, contacts are gradational	
38					
39					
40				@40.0' dark gray (5YR 4/1) to very dark gray (5YR 3/1), massive, slightly micaceous	Contact at 40' N65°W, 35°NE
41					
42					
43					
44					
45				@45.0' 2" wide zone of crushed siltstone	Attitude of crushed siltstone at 45' N70°W, 55°NE
46					
47					
48					
49					
50					Bedding (?) at 50' N75°E, 35°NW
51					
52					
53				@53.0' 1/2" to 1" thick sandstone bed, very fine-grained, blue gray (GLEY2 5/1)	Attitude of sand bed at 53' N70°W, 42°NE
54					
55				@55.0' 2" thick sandstone bed, very fine-grained, blue gray (GLEY2 5/1), contact surfaces are wavey	Attitude of sand bed at 55' N68°W, 35°NE
56					
57					
58				@58.0' becomes very hard and massive	
59					
60					Bedding at 60' N60°W, 40°NE
61					
62					
63					
64					
65					
66					

PROJECT: SUNSHINE CANYON LANDFILL
Sylmar, California

Downhole Log of Boring No. BA-1 (cont'd)

DEPTH (feet)	Sample No.	Sample	Blows/ 6 inches	DESCRIPTION NAME (USCS): color, moist, % by wt., plast. density, structure, cementation, react. w/HCl, geo. inter.	GRAPHIC LOG	GEOLOGIC DATA
67				TOWSLEY FORMATION (Ttos) SILTSTONE: continued		
68						
69						
70						
71						
72				@72.0' crushed siltstone 6" to 7" thick		Attitude of crushed siltstone at 72' N75°W, 75°NE
73						
74						
75						
76						
77						
78						
79						
80				Bottom of boring at 80 feet bgs. Groundwater encountered at 79 feet bgs.		
81						
82						
83						
84						
85						
86						
87						
88						
89						
90						
91						
92						
93						
94						
95						
96						
97						
98						
99						
100						
101						
102						

PROJECT: SUNSHINE CANYON LANDFILL Sylmar, California	**Downhole Log of Boring No. BA-2**

BORING LOCATION: N: 4234442.9 E: 4130568.9	ELEVATION AND DATUM: 2014' above mean sea level			
DRILLING CONTRACTOR: Roy Bros. Drilling	DATE STARTED: 7/13/10		DATE FINISHED: 7/13/10	
DRILLING METHOD: 24" Bucket Auger	TOTAL DEPTH (ft.): 95.0		MEASURING POINT: Ground Surface	
DRILLING EQUIPMENT: E-Z Bore	DEPTH TO WATER	FIRST 85	COMPL. 91	24 HRS. NA
SAMPLING METHOD: NA	LOGGED BY: M. McLarty			
HAMMER WEIGHT: NA DROP: NA	RESPONSIBLE PROFESSIONAL: M. McLarty			REG. NO. CEG 1107

DEPTH (feet)	SAMPLES: Sample No.	Sample	Blows/ 6 inches	DESCRIPTION NAME (USCS): color, moist, % by wt., plast. density, structure, cementation, react. w/HCl, geo. inter. Surface Elevation: 2014' above mean sea level	GRAPHIC LOG	GEOLOGIC DATA
1				ARTIFICIAL FILL (afu) SANDY LEAN CLAY with GRAVEL (CL) to CLAYEY SAND with GRAVEL (SC): dark brown (10YR 3/3)		
2						
3						
4						
5				@4.5' moist, becomes black		
6						
7						
8						
9						
10						
11						
12						
13				TOWSLEY FORMATION (Ttos) SANDSTONE: weathered, very fine grained sandstone, light yellowish brown (10YR 6/4) interbedded with dark reddish brown (5YR 3/3) siltstone, predominantly massive, near vertical joints 1/2" to 2" on center		Contact with underlying bedrock nearly horizontal at 13'
14						
15						Bedding at 15' N40°W, 35°NE
16						Joint at 16' N35°E, 75°NW
17						Bedding at 17' N62°W, 34°NE
18						
19						
20						
21						
22						
23						
24						
25				@24.0' SANDSTONE: silty, dark brown (10YR 3/3) interbedded with some very fine grained sand, brown (10YR 4/3), upper and lower contacts are gradational		Bedding at 24' N55°W, 40°NE
26				@25.0' SILTSTONE: micaceous, dark brown (10YR 3/3) to (10YR 4/3)		
27						Bedding at 28' N65°W, 40°NE
28						Top and bottom contact N60°W, 44°NE
29						
30						

PROJECT: SUNSHINE CANYON LANDFILL
Sylmar, California

Downhole Log of Boring No. BA-2 (cont'd)

DEPTH (feet)	SAMPLES: Sample No.	Sample	Blows/ 6 inches	DESCRIPTION NAME (USCS): color, moist, % by wt., plast. density, structure, cementation, react. w/HCl, geo. inter.	GRAPHIC LOG	GEOLOGIC DATA
				<u>TOWSLEY FORMATION (Ttos)</u> SANDSTONE: continued		
31				@28.95-30.5' SILTSTONE with little clay, broken and crushed with gouged surfaces		S W = 28.8% LL = 55 PI = 27
32				@30.5' micaceous, dark brown (10YR 3/3)		
33						
34						Attitude of jointing at 34' N25°E, 70°SE
35						
36						Bedding at 36' N65°W, 53°NE
37						
38						
39						
40						
41				@40.0' SILTSTONE: micaceous, unoxidized, very dark greenish gray (GLEY2 3/1) to greenish black (GLEY2 2.5/1), micaceous, massive with some very thin (1/16" to 1/8" thick), interbeds of dark gray (5Y 3/1), very fine grained sandstone		Bedding/contact at 40' N44°W, 40°NE
42						
43						
44						Bedding at 44' N70°W, 52°NE
45						
46						
47						
48						
49						
50						
51				becomes massive with near vertical joints and fractures		
52						
53						
54						
55						Bedding at 54' N70°W, 40°NE
56						
57						
58						
59						
60				@59.5' crushed and broken micaceous siltstone, very dark gray (5Y 3/1) to black (5Y 2.5/1)		Attitude of crushed zone at 59.5' N55°W, 44°NE
61				@60.0' micaceous, massive, very dark gray (5Y 3/1) to black (5Y 2.5/1)		
62						
63						
64						
65						
66						Bedding at 66' N58°W, 45°NE

PROJECT: SUNSHINE CANYON LANDFILL
Sylmar, California

Downhole Log of Boring No. BA-2 (cont'd)

DEPTH (feet)	SAMPLES Sample No.	Sample	Blows/ 6 inches	DESCRIPTION NAME (USCS): color, moist, % by wt., plast. density, structure, cementation, react. w/HCl, geo. inter.	GRAPHIC LOG	GEOLOGIC DATA
67				TOWSLEY FORMATION (Ttos) SANDSTONE: continued		
68						
69						
70				@70.0' minor seepage along bedding		Bedding at 70' N60°W, 40°NE
71						
72						
73						
74						
75				@74.5' crushed siltstone, roughly parallel to bedding, 6" to 8" thick		Attitude of crushed zone at 74.5' N55°W, 41°NE
76						
77						
78				@78.0-80.0' crushed siltstone with 2" thick silty clay seam at base		Attitude at base of crushed rock N47°W, 44°NE
79						
80						
81						
82						
83				@83.0' crushed and broken siltstone from 83-85', abundant seepage and "belling" of the boring to roughly 3' in diameter		Bedding at 83' N55°W, 35°NE
84						
85				@85.0' SILSTONE: less massive, micaceous to bottom of boring, dark gray (5Y 3/1) to black (5Y 2.5/1), seepage		
86						
87						
88						
89						
90						
91						
92						
93						
94						
95				Bottom of boring at 95 feet bgs. Down-hole logged to 85 feet bgs. Groundwater 91 feet bgs at completion. Seepage at 85 feet bgs.		
96						
97						
98						
99						
100						
101						
102						

PROJECT: SUNSHINE CANYON LANDFILL Sylmar, California		**Downhole Log of** **Boring No. BA-3**			
BORING LOCATION: N: 4234396.2 E: 4130752.4		ELEVATION AND DATUM: 1960.9' above mean sea level			
DRILLING CONTRACTOR: Roy Bros. Drilling		DATE STARTED: 7/12/10		DATE FINISHED: 7/12/10	
DRILLING METHOD: 24" Bucket Auger		TOTAL DEPTH (ft.): 80.0		MEASURING POINT: Ground Surface	
DRILLING EQUIPMENT: E-Z Bore		DEPTH TO WATER	FIRST NA	COMPL. NA	24 HRS. NA
SAMPLING METHOD: NA		LOGGED BY: M. McLarty			
HAMMER WEIGHT: NA	DROP: NA	RESPONSIBLE PROFESSIONAL: M. McLarty			REG. NO. CEG 1107

DEPTH (feet)	SAMPLES: Sample No.	Sample	Blows/ 6 inches	DESCRIPTION NAME (USCS): color, moist, % by wt., plast. density, structure, cementation, react. w/HCl, geo. inter. Surface Elevation: 1960.9' above mean sea level	GRAPHIC LOG	GEOLOGIC DATA
1				ARTIFICIAL FILL (af) SANDY LEAN CLAY with GRAVEL (CL) to CLAYEY SAND with GRAVEL (SC): weathered subrounded cobbles and angular to subangular rock fragments 1/4" to 8", grayish brown (10YR 5/2), dry to 5', moist below 5'		
22				@22.0' becomes predominantly brown (7.5YR 5/3) to dark brown (7.5YR 3/3)		
35				@35.0' becomes mottled and crudly layered dark brown (7.5YR 3/3), silt with pinkish gray (7.5YR 7/8) very fine silty sand		
40				@40.0' becomes greenish brown (5YR 3/3) to black (5RYR 2.5/1), very fine sand, silty with some clay contains 1/2" to 2" angular rock fragments of dark gray micaceous siltstone		
44				TOWSLEY FORMATION (Ttos) SANDSTONE: weathered, silty, very fine grained streaked with caliche along vertical fractures, brown to reddish brown (5YR 5/4), folded and crushed, dark brown (7.5YR 3/2) to black clay seam 1/4" thick at 44.9' @45.0' becomes unoxidized massive siltstone, very dark greenish gray (GLEY2 3/1) to greenish black (2.5/ 5BG), micaceous		Clay seam at 44.9' N56°W, 59°S Joint at 45' N45°W, 70°NE
56						Clayey joint at 56.5' N50°W, 75°NE Bedding at 57' N66°W, 58°N
60						

PROJECT: SUNSHINE CANYON LANDFILL Sylmar, California	Downhole Log of Boring No. BA-3 (cont'd)

DEPTH (feet)	SAMPLES: Sample No.	Sample	Blows/ 6 inches	DESCRIPTION NAME (USCS): color, moist, % by wt., plast. density, structure, cementation, react. w/HCl, geo. inter.	GRAPHIC LOG	GEOLOGIC DATA
61–80				TOWSLEY FORMATION (Ttos) SANDSTONE: continued @60.6' paper thin polished bedding?, dark gray (7.5YR 3/1) to black siltstone, micaceous, massive, near vertical joints 1/16" to 1/8" wide streaked with caliche		Bedding at 60.6' N70°W, 44°N
80–132				Bottom of boring at 80 feet bgs. Groundwater was not encountered.		

PROJECT: SUNSHINE CANYON LANDFILL Sylmar, California	**Log of Boring No. B-1**

BORING LOCATION: N: 4234400.0 E: 4131142.9			
DATE STARTED: 7/22/10	DATE FINISHED: 7/22/10	NOTES:	
DRILLING METHOD: Hollow Stem Auger		Drilling Contractor: BC^2 Environmental Corp. Drilling Equipment: CME-75 Logged By: E. Forcier	
HAMMER WEIGHT: 140 lb	DROP: 30 in		
SAMPLER: CA Modified & SPT			

ELEV. (feet)	DEPTH (feet)	SAMPLES: Sample No.	SAMPLES: Sample	SAMPLES: Blows/ Foot	MATERIAL DESCRIPTION Surface Elevation: 1835' above mean sea level	PID Reading (ppm)	LABORATORY TESTS: Moisture Content (%)	LABORATORY TESTS: Dry Density (pcf)	LABORATORY TESTS: Other Tests
	0				ARTIFICIAL FILL (af) SANDY LEAN CLAY with GRAVEL (CL) to CLAYEY SAND with GRAVEL (SC): olive brown (2.5Y 4/4), dry to moist, ~55% low plasticity fines, ~30% fine to coarse sand, ~15% fine gravel, siltstone fragments				
	2.5–4	1	NR	2 3 2			11.3	97.5	CON
	5–6.5	2	NR	3 2 2					EI
	7				dark grayish brown (10YR 4/2), moist, ~50% low plasticity fines, ~35% fine to coarse sand, ~15% fine gravel, siltstone fragments				
	7.5–9	3	NR	4 4 4			10.9	95.8	DS
	10–11.5	4	NR	2 2 2					
	15								

ELEV. (feet)	DEPTH (feet)	SAMPLES Sample No.	Sample	Blows/ Foot	MATERIAL DESCRIPTION	PID Reading (ppm)	LABORATORY TESTS Moisture Content (%)	Dry Density (pcf)	Other Tests
		5	NR	5 9 14	ARTIFICIAL FILL (af) SANDY LEAN CLAY with GRAVEL (CL) to CLAYEY SAND with GRAVEL (SC): continued @15.0' ~50% low plasticity fines, ~25% fine to coarse sand, ~25% fine to coarse gravel, siltstone fragments (up to ~1")				
	16								
	17								
	18								
	19								
	20	6	NR	8 9 13	dark greenish gray (GLEY1 4/1 5G)				
	21								
	22								
	23								
	24								
	25	7		7 15 18					
	26								
	27								
	28								
	29								
	30	8	NR	4 6 8					
	31								
	32								

ELEV. (feet)	DEPTH (feet)	Sample No.	Sample	Blows/ Foot	MATERIAL DESCRIPTION	PID Reading (ppm)	Moisture Content (%)	Dry Density (pcf)	Other Tests
					ARTIFICIAL FILL (af) SANDY LEAN CLAY with GRAVEL (CL) to CLAYEY SAND with GRAVEL (SC): continued				
	33								
	34								
	35	9	NR	50/5"	@35.0' poor recovery - likely sampling through cobble sized fragment of siltstone in fill				
	36								
	37								
	38								
	39								
	40	10	NR	4 11 13					
	41								
	42								
	43								
	44								
	45	11	NR	10 10 7	dark greenish brown (10YR 4/2), moist, ~40% fine gravel, ~35% fine to coarse sand, ~25% plasticity fines				
	46								
	47								
	48								
	49								

ELEV. (feet)	DEPTH (feet)	SAMPLES: Sample No.	SAMPLES: Sample	SAMPLES: Blows/ Foot	MATERIAL DESCRIPTION	PID Reading (ppm)	LABORATORY TESTS: Moisture Content (%)	LABORATORY TESTS: Dry Density (pcf)	LABORATORY TESTS: Other Tests
	50	12	NR	2 5 8	ARTIFICIAL FILL (af) SANDY LEAN CLAY with GRAVEL (CL) to CLAYEY SAND with GRAVEL (SC): dark greenish gray (GLEY1 4/1 5G), moist to wet, ~40% fine to coarse sand, ~40% low plasticity fines, ~20% fine gravel, siltstone - pockets of groundwater in sample				
	51								
	52								
	53								
	54				TOWSLEY FORMATION (Ttos) SILTY SANDSTONE: dark greenish gray (GLEY4/1 5G), massive, slightly micaceous, unoxidized				
	55	13		50/3"					
	56								
	57	14		50/6"					
	58				Bottom of boring 58 feet bgs. Groundwater not encountered at time of drilling. Boring backfilled with soil/rock cuttings.				
	59								
	60								
	61								
	62								
	63								
	64								
	65								
	66								

PROJECT: SUNSHINE CANYON LANDFILL Sylmar, California	**Log of Boring No. B-2**

BORING LOCATION: N: 4234468.7 E: 4131021.6

DATE STARTED: 7/23/10	DATE FINISHED: 7/23/10	NOTES: Drilling Contractor: BC^2 Environmental Corp. Drilling Equipment: CME-75 Logged By: E. Forcier
DRILLING METHOD: Hollow Stem Auger		
HAMMER WEIGHT: 140 lb	DROP: 30 in	
SAMPLER: CA Modified & SPT		

ELEV. (feet)	DEPTH (feet)	SAMPLES: Sample No.	Sample	Blows/ Foot	MATERIAL DESCRIPTION Surface Elevation: 1848.1' above mean sea level	PID Reading (ppm)	LABORATORY TESTS: Moisture Content (%)	Dry Density (pcf)	Other Tests
	0–2.5				<u>ARTIFICIAL FILL (af)</u> SANDY LEAN CLAY with GRAVEL (CL) to CLAYEY SAND with GRAVEL (SC): dark grayish brown (10YR 4/2), moist, ~45% fine to coarse sand, ~35% low plasticity fines, ~20% fine to coarse gravel, siltstone fragments				
	2.5–4	1	NR	14 16 20					
	4.5				dark grayish brown (10YR 4/2), moist, ~60% fine to medium sand, ~35% low plasticity fines, ~5% fine gravel				
	5–6.5	2	NR	8 10 10					
	10–11.5	3	NR	10 10 13	mottled olive brown (2.5Y 4/4) and dark grayish brown (2.5Y 4/2), moist, ~40% fine to coarse sand, ~35% low plasticity fines, ~25% fine to coarse gravel, siltstone fragments		15.1	108.2	CON
	15								

ELEV. (feet)	DEPTH (feet)	SAMPLES: Sample No.	SAMPLES: Sample	SAMPLES: Blows/ Foot	MATERIAL DESCRIPTION	PID Reading (ppm)	LABORATORY TESTS: Moisture Content (%)	LABORATORY TESTS: Dry Density (pcf)	LABORATORY TESTS: Other Tests
		4	NR	4 8 11	ARTIFICIAL FILL (af) SANDY LEAN CLAY with GRAVEL (CL) to CLAYEY SAND with GRAVEL (SC): continued				
	16								
	17								
	18				TOWSLEY FORMATION (Ttos) SILTY SANDSTONE: yellowish brown (10YR 5/8) and gray (10YR 5/1) with abundant FeOx staining, jointed and fractured (oxidized), weathered				
	19								
	20	5	NR	7 11 15					
	21								
	22								
	23								
	24								
	25	6		50/8"	light olive brown (2.5Y 5/3), some FeOx staining (transitioning into unoxidized)				
	26								
	27								
	28								
	29				dark greenish gray (GLEY 4/1 5G), massive, slightly micaceous, unoxidized, slightly weathered to no weathered				
	30	7		50/3"					
	31				Bottom of boring at 30.75 feet bgs. Groundwater not encountered at time of boring. Boring backfilled with soil and rock cuttings.				
	32								

PROJECT: SUNSHINE CANYON LANDFILL Sylmar, California	**Log of Boring No. B-3**

BORING LOCATION: N: 4234577.4 E: 4130901.3

DATE STARTED: 7/23/10	DATE FINISHED: 7/23/10	NOTES:
DRILLING METHOD: Hollow Stem Auger		Drilling Contractor: BC^2 Environmental Corp. Drilling Equipment: CME-75 Logged By: E. Forcier
HAMMER WEIGHT: 140 lb	DROP: 30 in	
SAMPLER: CA Modified & SPT		

ELEV. (feet)	DEPTH (feet)	Sample No.	Sample	Blows/Foot	MATERIAL DESCRIPTION Surface Elevation: 1861.8' above mean sea level	PID Reading (ppm)	Moisture Content (%)	Dry Density (pcf)	Other Tests
	0				ARTIFICIAL FILL (af) SANDY LEAN CLAY with GRAVEL (CL) to CLAYEY SAND with GRAVEL (SC): olive brown (2.5Y 4/4), moist, ~45% fine to coarse sand, ~40% low plasticity fines, ~15% fine gravel, siltstone fragments				
	1								
	2								
	3	1		12 21 24					
			NR						
	4				dark greenish gray (GLEY1 4/1 5G)				
	5								
	6	2		6 11 11					EI
			NR						
	7								
	8		B						COMPS
	9								
	10	3		9 18 21	mottled olive brown (2.5Y 4/4) and very dark grayish brown (2.5Y 3/2)				
	11		NR		dark yellowish brown (10YR 4/6), coarse sandstone fragments up to ~1 1/2"				
	12								
	13								
	14								
	15								

PROJECT: SUNSHINE CANYON LANDFILL
Sylmar, California

Log of Boring No. B-3 (cont'd)

ELEV. (feet)	DEPTH (feet)	Sample No.	Sample	Blows/ Foot	MATERIAL DESCRIPTION	PID Reading (ppm)	Moisture Content (%)	Dry Density (pcf)	Other Tests
		4		10 15 15	ARTIFICIAL FILL (af) SANDY LEAN CLAY with GRAVEL (CL) to CLAYEY SAND with GRAVEL (SC): continued				
	16								
	17								
	18								
	19								
	20	5	NR	5 11 13	~40% fine to coarse sand, ~35% low plasticity fines, ~25% fine to coarse gravel, siltstone/sandstone fragments up to ~2"				
	21								
	22								
	23		B						
	24								
	25	6		2 6 8					
	26								
	27								
	28								
	29				TOWSLEY FORMATION (Ttos) SILTSTONE: dark gray (10YR 4/1) with abundant FeOx staining, moist to wet, highly weathered, jointed and fractured, oxidized				
	30	7		4 12 25					
	31								
	32								

ELEV. (feet)	DEPTH (feet)	SAMPLES: Sample No.	SAMPLES: Sample	SAMPLES: Blows/ Foot	MATERIAL DESCRIPTION	PID Reading (ppm)	LABORATORY TESTS: Moisture Content (%)	LABORATORY TESTS: Dry Density (pcf)	LABORATORY TESTS: Other Tests
					TOWSLEY FORMATION (Ttos) SILTSTONE: continued				
	33								
	34								
	35	8		50/3"	dark greenish gray (GLEY4/1 5G), massive, slightly micaceous, unoxidized				
	36				Bottom of boring at 35.75 feet bgs. Groundwater not encountered at time of drilling. Boring backfilled with soil/rock cuttings.				
	37								
	38								
	39								
	40								
	41								
	42								
	43								
	44								
	45								
	46								
	47								
	48								
	49								

PROJECT: SUNSHINE CANYON LANDFILL
Sylmar, California

Log of Boring No. B-4

BORING LOCATION: N: 4234633.6 E: 4130733.0

DATE STARTED: 7/21/10	DATE FINISHED: 7/21/10	NOTES: Drilling Contractor: BC^2 Environmental Corp. Drilling Equipment: CME-75 Logged By: E. Forcier
DRILLING METHOD: Hollow Stem Auger		
HAMMER WEIGHT: 140 lb	DROP: 30 in	
SAMPLER: CA Modified & SPT		

ELEV. (feet)	DEPTH (feet)	SAMPLES: Sample No.	SAMPLES: Sample	SAMPLES: Blows/ Foot	MATERIAL DESCRIPTION Surface Elevation: 1876.7' above mean sea level	PID Reading (ppm)	LABORATORY TESTS: Moisture Content (%)	LABORATORY TESTS: Dry Density (pcf)	LABORATORY TESTS: Other Tests
	0				~3 1/2" asphalt (no aggregate baserock)				
	0.5–2				ARTIFICIAL FILL (af) SANDY LEAN CLAY with GRAVEL (CL) to CLAYEY SAND with GRAVEL (SC): dark greenish gray (GLEY1 4/15G), moist, ~40% fine to coarse sand, ~40% low plasticity fines, ~20% fine gravel, siltstone fragments				
	2.5–4	1		6 9 12			15.2	109.0	DS
	4.5				dark greenish gray (GLEY1 4/15G), moist, ~60% low plasticity fines, ~30% fine sand, ~10% fine gravel, siltstone fragments				
	5–6.5	2		5 8 8					
	7				dark gray (2.5Y 4/1), moist, ~50% low plasticity fines, ~35% fine to coarse sand, ~15% fine gravel, siltstone fragments				
	7.5–9	3	NR	4 7 9			14.2	111.1	CON
	10–11.5	4		7 10 14					ATT
	12.5				fine to coarse gravel, coarse siltstone fragment up to ~2 1/2"				
	12.5–14	5	NR	9 14 17					
	15								

GEO3 PID

ELEV. (feet)	DEPTH (feet)	SAMPLES Sample No.	Sample	Blows/ Foot	MATERIAL DESCRIPTION	PID Reading (ppm)	LABORATORY TESTS Moisture Content (%)	Dry Density (pcf)	Other Tests
	15–16.5	6		7 12 13	ARTIFICIAL FILL (af) SANDY LEAN CLAY with GRAVEL (CL) to CLAYEY SAND with GRAVEL (SC): continued abundant siltstone fragments				
	17.5–19	7	NR	22 29 34	olive brown (2.5Y 4/4), FeOx staining, ~60% medium plasticity fines, ~25% fine to coarse sand, ~15% gravel, siltstone fragments				
	20–21.5	8		10 14 19					
	25–26.5	9		2 5 5					
	30–31.5	10		6 8 10					

PROJECT: SUNSHINE CANYON LANDFILL
Sylmar, California

Log of Boring No. B-4 (cont'd)

ELEV. (feet)	DEPTH (feet)	SAMPLES: Sample No.	SAMPLES: Sample	SAMPLES: Blows/ Foot	MATERIAL DESCRIPTION	PID Reading (ppm)	LABORATORY TESTS: Moisture Content (%)	LABORATORY TESTS: Dry Density (pcf)	LABORATORY TESTS: Other Tests
					ARTIFICIAL FILL (af) SANDY LEAN CLAY with GRAVEL (CL) to CLAYEY SAND with GRAVEL (SC): continued				
	33				TOWSLEY FORMATION (Ttos) SILTY SANDSTONE: greenish gray (GLEY1 5/1 10Y), massive, slightly micaceous, unoxidized				
	34								
	35	11		50/4"					
	36								
	37								
	38								
	39								
	40	12		50/3"	dark greenish gray (GLEY1 4/1 10Y)				
	41				Bottom of boring at 41 feet bgs. Groundwater not encountered at time of drilling. Boring backfilled with soil/rock cuttings.				
	42								
	43								
	44								
	45								
	46								
	47								
	48								
	49								

PROJECT: SUNSHINE CANYON LANDFILL Sylmar, California	**Log of Boring No. B-5**

BORING LOCATION: N: 4234505.0 E: 4130827.2

DATE STARTED: 7/22/10	DATE FINISHED: 7/22/10	NOTES: Drilling Contractor: BC^2 Environmental Corp. Drilling Equipment: CME-75 Logged By: E. Forcier
DRILLING METHOD: Hollow Stem Auger		
HAMMER WEIGHT: 140 lb	DROP: 30 in	
SAMPLER: CA Modified & SPT		

ELEV. (feet)	DEPTH (feet)	SAMPLES: Sample No.	Sample	Blows/ Foot	MATERIAL DESCRIPTION Surface Elevation: 1889.9' above mean sea level	PID Reading (ppm)	LABORATORY TESTS: Moisture Content (%)	Dry Density (pcf)	Other Tests
	0				asphalt				
	0.5–2				ARTIFICIAL FILL (af) SANDY LEAN CLAY with GRAVEL (CL) to CLAYEY SAND with GRAVEL (SC): dark olive brown (2.5Y 3/3) with gray mottling (2.5Y 5/1), moist, ~55% medium plasticity fines, ~30% fine to coarse sand, ~15% fine to coarse gravel, siltstone fragments				
	3	1	NR	6 12 18			17.2	102.9	DS
	5	2	NR	6 7 9					
	6				dark greenish gray (GLEY1 4/1 5G), moist, ~50% low plasticity fines, ~25% fine to coarse sand, ~25% fine to coarse gravel, siltstone fragments up to ~2"				
	8		B						
	10	3	NR	11 14 18					
	15								

Log of Boring No. B-5 (cont'd)

ELEV. (feet)	DEPTH (feet)	Sample No.	Sample	Blows/ Foot	MATERIAL DESCRIPTION	PID Reading (ppm)	Moisture Content (%)	Dry Density (pcf)	Other Tests
	15	4		7, 9, 10	ARTIFICIAL FILL (af) SANDY LEAN CLAY with GRAVEL (CL) to CLAYEY SAND with GRAVEL (SC): continued				
	16								EI
	17								
	18		B						COMP
	19								
	20	5		7, 17, 22					
	21		NR						
	22								
	23								
	24								
	25	6		6, 10, 11					
	26		NR						
	27								
	28								
	29								
	30	7		8, 18, 26					
	31								
	32								

ELEV. (feet)	DEPTH (feet)	Sample No.	Sample	Blows/ Foot	MATERIAL DESCRIPTION	PID Reading (ppm)	Moisture Content (%)	Dry Density (pcf)	Other Tests
					<u>ARTIFICIAL FILL (af)</u> SANDY LEAN CLAY with GRAVEL (CL) to CLAYEY SAND with GRAVEL (SC): continued				
	33				mottled olive brown (2.5Y 4/4) and dark gray (2.5Y 4/1), moist, ~50% medium plasticity fines, ~30% fine to coarse sand, ~20% fine gravel, siltstone fragments				
	34								
	35	8		7 11 12					
	36		NR						
	37				drilling harder				
	38		B						
	39								
	40	9		12 20 20	large rock fragment in sample (granite?)				
	41								
	42								
	43				drilling harder				
	44								
	45	10		17 24 20	olive brown (2.5Y 4/4) with FeOx staining, fragments of gypsum				
	46								
	47								
	48								
	49								

ELEV. (feet)	DEPTH (feet)	SAMPLES Sample No.	Sample	Blows/ Foot	MATERIAL DESCRIPTION	PID Reading (ppm)	LABORATORY TESTS Moisture Content (%)	Dry Density (pcf)	Other Tests
					ARTIFICIAL FILL (af) SANDY LEAN CLAY with GRAVEL (CL) to CLAYEY SAND with GRAVEL (SC): continued				
	50–51.25	11		4 6 8					
	55–56.5	12	NR	5 6 9					
	56				mottled dark gray (2.5Y 4/4) and olive brown (2.5Y 4/1) with FeOx staining				
	58				TOWSLEY FORMATION (Ttos) SILTY SANDSTONE: greenish gray (GLEY1 4/1 5G), massive, slightly micaceous, unoxidized				
	60	13		50/5"					
	63	14		50/3"					
	63.75				Bottom of boring at 63.75 feet bgs. Auger refusal. Groundwater not encountered at time of drilling. Boring backfilled with soil/rock cuttings.				

PROJECT: SUNSHINE CANYON LANDFILL Sylmar, California	**Log of Boring No. B-6**

BORING LOCATION: N: 4234392.4 E: 4130905.1

DATE STARTED: 7/21/10	DATE FINISHED: 7/21/10	NOTES: Drilling Contractor: BC^2 Environmental Corp. Drilling Equipment: CME-75 Logged By: E. Forcier
DRILLING METHOD: Hollow Stem Auger		
HAMMER WEIGHT: 140 lb	DROP: 30 in	
SAMPLER: CA Modified & SPT		

ELEV. (feet)	DEPTH (feet)	SAMPLES: Sample No.	SAMPLES: Sample	SAMPLES: Blows/Foot	MATERIAL DESCRIPTION Surface Elevation: 1901.7' above mean sea level	PID Reading (ppm)	LABORATORY TESTS: Moisture Content (%)	LABORATORY TESTS: Dry Density (pcf)	LABORATORY TESTS: Other Tests
	0–1				ARTIFICIAL FILL (af) SANDY LEAN CLAY with GRAVEL (CL) to CLAYEY SAND with GRAVEL (SC): moist, ~55% low to medium plasticity fines, ~30% fine to medium sand, ~15% fine gravel, fragments of siltstone [FILL]				
	2.5–4	1	SPT	6 8 11					
	5–6.5	2	CA Modified	5 7 9	(bulk sample at 5' to 10')				COMPS
	7.5–9	3	SPT	10 12 18					
	10–11.5	4	CA Modified	5 8 8					ATT
	12.5–14	5	SPT	4 10 12					
	15								

GEO3 PID

ELEV. (feet)	DEPTH (feet)	SAMPLES: Sample No.	SAMPLES: Sample	SAMPLES: Blows/Foot	MATERIAL DESCRIPTION	PID Reading (ppm)	LABORATORY TESTS: Moisture Content (%)	LABORATORY TESTS: Dry Density (pcf)	LABORATORY TESTS: Other Tests
	16	6		7, 12, 14	ARTIFICIAL FILL (af) SANDY LEAN CLAY with GRAVEL (CL) to CLAYEY SAND with GRAVEL (SC): continued olive brown (2.5Y 4/4), ~30% fine to coarse sand, siltstone/sandstone fragments				
	17								
	18	7		7, 15, 15					
	19								
	20	8		6, 9, 9	olive brown (2.5Y 4/4), moist, ~50% fine to coarse sand, ~40% low to medium plasticity fines, ~10% fine gravel and siltstone fragments				
	21								
	22				olive brown (2.5Y 4/4), moist, ~40% fine to coarse sand, ~40% low to medium plasticity fines, ~20% fine to coarse gravel (up to ~ 1 1/2"), fragment of asphalt ~1/2"				
	23	9	NR	8, 14, 17			12.4	111.5	DS
	24								
	25	10	NR	6, 8, 10	mottled gray (2.5Y 5/1) and olive brown (2.5Y 4/4), moist, ~50% low to medium plasticity fines, ~35% fine to coarse sand, ~15% fine gravel [FILL]				
	26								
	27				dark gray (2.5Y 4/1), moist, ~40% fine to coarse sand, ~40% low to medium plasticity fines, ~20% fine to coarse gravel, fragments of siltstone (up to ~2")				
	28	11	NR	10, 15, 21					
	29								
	30	12		6, 9, 12					
	31								
	32								

ELEV. (feet)	DEPTH (feet)	Sample No.	Sample	Blows/ Foot	MATERIAL DESCRIPTION	PID Reading (ppm)	Moisture Content (%)	Dry Density (pcf)	Other Tests
					ARTIFICIAL FILL (af) SANDY LEAN CLAY with GRAVEL (CL) to CLAYEY SAND with GRAVEL (SC): continued				
	33	13		8 14 22	@32.5' ~45% low to medium plasticity fines, ~35% fine to coarse sand, ~20% fine to coarse gravel, siltstone fragments up to ~1"				
	35	14		9 12 15	dark gray (2.5Y 4/1), moist, ~55% low to medium plasticity fines, ~30% fine to coarse sand, ~15% gravel, siltstone fragments				
	38	15		12 17 23	trace root fragment in sample				
	40	16		9 14 19					
	43	17		14 22 26	iron oxide staining				
	45	18	NR	5 6 6					
	48	19		7 9 11	coarse gravel-sized fragment of siltstone up to ~2"				

PROJECT: SUNSHINE CANYON LANDFILL
Sylmar, California

Log of Boring No. B-6 (cont'd)

ELEV. (feet)	DEPTH (feet)	Sample No.	Sample	Blows/ Foot	MATERIAL DESCRIPTION	PID Reading (ppm)	Moisture Content (%)	Dry Density (pcf)	Other Tests
					ARTIFICIAL FILL (af) SANDY LEAN CLAY with GRAVEL (CL) to CLAYEY SAND with GRAVEL (SC): continued				
	50	20	NR	10 17 23					
	52	21	NR	14 50/4"	TOWSLEY FORMATION (Ttos) SILTY SANDSTONE: strong brown (2.5YR 4/6), moist, micaceous, disseminated and joint controlled iron oxide weathering, friable, oxidized				
	55	22	NR	29 50/5"					
	57.5	23	NR	10 12 17	gray mottling (7.5YR 6/1)				
	60	24		22 50/5"	grayish brown (10YR 5/2), very little FeOx staining, transitioning into unoxidized bedrock				
	62.5	25		20 50/5"					
	65	26		30 50/4"	Bottom of boring at 66 feet bgs. Groundwater not encountered at time of drilling. Boring backfilled with soil/rock cuttings.				

PROJECT: SUNSHINE CANYON LANDFILL Sylmar, California	**Log of Boring No. CH01**	
BORING LOCATION: N: 4234527.1 E: 4130406.9	ELEVATION AND DATUM: 2037.8 feet msl	
DRILLING CONTRACTOR: BC2 Drilling, Inc.	DATE STARTED: 7/13/2010	DATE FINISHED: 7/13/2010
DRILLING EQUIPMENT: CME 850	TOTAL DEPTH (feet): 59	MEASURING POINT: ground surface
DRILLING METHOD: Mud Rotary	DEPTH TO FREE WATER FIRST ENCOUNTERED: NA	
SAMPLING METHOD: HQ Core Barrel	DEPTH TO FREE WATER AT COMPLETION: NA	
BOREHOLE/CORE DIAMETER:	LOGGED BY: D. Collins	CHECKED BY: D. Collins

ELEVATION (feet)	DEPTH (feet)	RUN TIME (min)	RUN No	RECOVERY (%)	RQD (%)	FRACTURING	HARDNESS	STRENGTH	WEATHERING	LITHOLOGIC DESCRIPTION	GRAPHIC LOG	DISCONTINUITY DESCRIPTION	REMARKS
	0									ARTIFICIAL FILL (af) SANDY LEAN CLAY with GRAVEL (CL) to CLAYEY SAND with GRAVEL (SC): with cobbles, road fill			7:43, begin drilling using tricone bit to 9'
	2									TOWSLEY FORMATION (Ttos) SILTSTONE			
	5									@5.0' cuttings in mud tank indicates siltstone, olive brown (2.5Y 5/4)			
	9	8:10 8:26	1	60	13	Cl	So	We	Se-Mo	@9.0' SILTY SANDSTONE: gray (2.5Y 6/1) with disseminated and joint controlled iron oxide weathering, strong brown (7.5YR 4/6) to (7.5YR 5/8), micaceous, about 50/50 fine sand and fines, joint surfaces coated with iron oxides and some joints filled with cemented brittle fines		Jo, 80°, Fi (iron oxides), Pl Jo, 45°, Fi (iron oxides), Pl Jo, 20°, Fi (iron oxides), Pl	Drilled using impregnated diamond bit for HQ core recovery LL = 37, PI = 16 TX w = 16.8% dd = 109.0 pcf
	11.4							Fr					
	11.8							We				Jo, 45°, Fi (iron oxides), Pl, Sl	
	14					Mo	So	We	Se-Mo				DS w = 18.6% dd = 105.6 pcf
	15												

FRACTURING: VC-Very Close (<0.01'), Cl-Close (0.1'-0.3'), Mo-Moderate (0.3'-1'), Wi-Wide (1'-3'), and VW-Very Wide (3'-10'). HARDNESS: So-Soft, Lo-Low, Mo-Moderate, Ha-Hard, and VH-Very Hard. STRENGTH: Fr-Friable, We-Weak, Mo-Moderate, St-Strong, and Ex-Extremely Strong. WEATHERING: Fr-Fresh, Sl-Slight, Mo-Moderate, and Se-Severe. DISCONTINUITY: Type (Be-Bedding, Jo-Joint, Fo-Foliation, Sh-Shear, Me-Mechanical Break, and Ve-Vein), Dip Angle, Aperture (Ti-Tight, Op-Open, He-Healed, and Fi-Filled), Surface Shape (Ir-Irregular, Pl-Planar, and Wa-Wavy), Roughness (St-Stepped, Ro-Rough, Mo-Moderately Rough, Sl-Slightly Rough, Sm-Smooth, and Po-Polished).

Project No.
14828.000.0

Figure

AMEC Geomatrix

PROJECT: SUNSHINE CANYON LANDFILL
Sylmar, California

Log of Boring No. CH01 cont.

ELEVATION (feet)	DEPTH (feet)	RUN TIME (min)	RUN NO.	RECOVERY (%)	RQD (%)	FRACTURING	HARDNESS	STRENGTH	WEATHERING
	15–19	8:39 / 8:49	2	95	28	Mo	So	We	Se-Mo
	19–24	9:04 / 9:11	3	100	28		Lo-Mo (from ~20.4)		
	24–29	9:24 / 9:31	4	100	30				Mo (from ~26.4)
	29–33.5	9:42 / 9:49	5	100	27	Mo	Lo-Mo	We	Mo

LITHOLOGIC DESCRIPTION

TOWSLEY FORMATION (Ttos) SILTY SANDSTONE: continued
soft, black carbon commonly disseminated throughout, massive, rare faint layering, no reaction to HCl

@19.0' 1/16" diameter root in joint plane

@24.0' color changing to brown (10YR 5/3) over very broad interval, silty sandstone has abundant thin wisps of reddish iron oxides and yellowish oxide (2.5Y 8/8)

DISCONTINUITY DESCRIPTION

Depth (feet, approx.)	Discontinuity Description
15.9	Be?, 15° to 25°, Ti, Pl
16.7	Jo, 55°, Ti, Pl
17.0	Jo, 45°, Fi, Pl, Sl
17.3	Jo, 55°, Ti, Pl
18.1	Jo, 80° to 90°, Fi (brittle, cemented fines), Wa, Sl
21.1	Jo, 60°, Op, Pl, Sl
21.7	Jo, 80° to 90°, Fi (cemented fines), Wa, St
22.5	Jo, 75°, Fi (cemented fines), Pl, Sl
23.7	Be?, 80°, Ti, Pl
24.5	Jo, 60°, Op, Pl, Mo
24.9	Jo, 70°, Op, Pl, iron oxide coated
25.9	Jo, 70°, Ti, Wa, iron oxide coated
28.1	Jo, 60°, Op, Pl, Mo
29.1	4xJo, 70°, Op, Pl, Mo, iron oxide coated
32.1	Jo, 70°, Fi, Pl, iron oxide and cemented fines filling

REMARKS

At ~31.5–32 ft: TX
w = 12.7%
dd = 118.0 pcf

FRACTURING: VC-Very Close (<0.01'), Cl-Close (0.1'-0.3'), Mo-Moderate (0.3'-1'), Wi-Wide (1'-3'), and VW-Very Wide (3'-10'). HARDNESS: So-Soft, Lo-Low, Mo-Moderate, Ha-Hard, and VH-Very Hard. STRENGTH: Fr-Friable, We-Weak, Mo-Moderate, St-Strong, and Ex-Extremely Strong. WEATHERING: Fr-Fresh, Sl-Slight, Mo-Moderate, and Se-Severe. DISCONTINUITY: Type (Be-Bedding, Jo-Joint, Fo-Foliation, Sh-Shear, Me-Mechanical Break, and Ve-Vein), Dip Angle, Aperture (Ti-Tight, Op-Open, He-Healed, and Fi-Filled), Surface Shape (Ir-Irregular, Pl-Planar, and Wa-Wavy), Roughness (St-Stepped, Ro-Rough, Mo-Moderately Rough, Sl-Slightly Rough, Sm-Smooth, and Po-Polished).

Project No. 14828.000.0

Figure Cont.

PROJECT: SUNSHINE CANYON LANDFILL Sylmar, California	Log of Boring No. CH01 cont.

ELEVATION (feet)	DEPTH (feet)	RUN TIME (min)	RUN NO.	RECOVERY (%)	RQD (%)	FRACTURING	HARDNESS	STRENGTH	WEATHERING	LITHOLOGIC DESCRIPTION	GRAPHIC LOG	DISCONTINUITY DESCRIPTION	REMARKS
	34									TOWSLEY FORMATION (Ttos) SILTY SANDSTONE: continued			TX w = 14.2% dd = 117.5 pcf UC w = 13.2% dd = 116.6 pcf
	35					Cl			Se			Jo, 60°, Ti, Wa, Sl, iron oxide coated	
	36	9:59 10:05	6	100	72								
	37					Wi	Mo	Mo	Sl	@36.5' silty sandstone, color changes to gray (2.5Y 5/1) and dark gray (2.5Y 4/1), weak reaction to HCl			
	38											Jo, 75°, Ti, Fi (1/8" iron oxide, cemented fines), Pl	TX w = 13.5% dd = 122.4 pcf
	39											Jo, 75°, Fi (iron oxide), Pl	
	40											Jo, 65°, Fi (iron oxide, cemented fines), Pl	
	41	10:14 10:20	7	100	100							Jo, 50° to 80°, Fi, St	
	42												UC w = 10.7% dd = 125.3 pcf TX w = 11.4% dd = 126.2 pcf
	43									@42.5' 2" pocket of light gray (2.5Y 7/1), fine sand, friable to loose			
	44					Vw			Fr	@44.0' very dark gray (2.5Y 3/1)			Run 8 lacks joints and bedding UC w = 12.0% dd = 123.4 pcf
	45												
	46	10:30 10:36	8	95	95								
	47												
	48												
	49					Vw	Mo	Mo	Fr	@49.0' abundant disseminated black carbon (1 to 2%)			TX w = 13.4% dd = 123.9 pcf
	50									@50.4' black carbon 1/2" in diameter			
	51	10:44 10:50	9	100	100								
	52												

FRACTURING: VC-Very Close (<0.01'), Cl-Close (0.1'-0.3'), Mo-Moderate (0.3'-1'), Wi-Wide (1'-3'), and VW-Very Wide (3'-10'). HARDNESS: So-Soft, Lo-Low, Mo-Moderate, Ha-Hard, and VH-Very Hard. STRENGTH: Fr-Friable, We-Weak, Mo-Moderate, St-Strong, and Ex-Extremely Strong. WEATHERING: Fr-Fresh, Sl-Slight, Mo-Moderate, and Se-Severe. DISCONTINUITY: Type (Be-Bedding, Jo-Joint, Fo-Foliation, Sh-Shear, Me-Mechanical Break, and Ve-Vein), Dip Angle, Aperture (Ti-Tight, Op-Open, He-Healed, and Fi-Filled), Surface Shape (Ir-Irregular, Pl-Planar, and Wa-Wavy), Roughness (St-Stepped, Ro-Rough, Mo-Moderately Rough, Sl-Slightly Rough, Sm-Smooth, and Po-Polished).

Project No. 14828.000.0

AMEC Geomatrix

PROJECT: SUNSHINE CANYON LANDFILL
Sylmar, California

Log of Boring No. CH01 cont.

ELEVATION (feet)	DEPTH (feet)	RUN TIME (min)	RUN NO.	RECOVERY (%)	RQD (%)	FRACTURING	HARDNESS	STRENGTH	WEATHERING	LITHOLOGIC DESCRIPTION	GRAPHIC LOG	DISCONTINUITY DESCRIPTION	REMARKS
										TOWSLEY FORMATION (Ttos) SILTY SANDSTONE: continued		Be, 60°, Ti, Pl	
	53												
	54									@54.0' dark gray (2.5Y 4/1)			@54-56' mechanical breaks caused by drilling @54' driller reports loss of circulation
	55												
	56	NM / NM	10	97	97								
	57												
	58												
	59									Boring terminated at 59.0 feet bgs			Boring was destroyed after drilling by pouring medium chip bentonite from the surface
	60												
	61												
	62												
	63												
	64												
	65												
	66												
	67												
	68												
	69												
	70												

FRACTURING: VC-Very Close (<0.01'), Cl-Close (0.1'-0.3'), Mo-Moderate (0.3'-1'), Wi-Wide (1'-3'), and VW-Very Wide (3'-10'). HARDNESS: So-Soft, Lo-Low, Mo-Moderate, Ha-Hard, and VH-Very Hard. STRENGTH: Fr-Friable, We-Weak, Mo-Moderate, St-Strong, and Ex-Extremely Strong. WEATHERING: Fr-Fresh, Sl-Slight, Mo-Moderate, and Se-Severe. DISCONTINUITY: Type (Be-Bedding, Jo-Joint, Fo-Foliation, Sh-Shear, Me-Mechanical Break, and Ve-Vein), Dip Angle, Aperture (Ti-Tight, Op-Open, He-Healed, and Fi-Filled), Surface Shape (Ir-Irregular, Pl-Planar, and Wa-Wavy), Roughness (St-Stepped, Ro-Rough, Mo-Moderately Rough, Sl-Slightly Rough, Sm-Smooth, and Po-Polished).

Project No. 14828.000.0

AMEC Geomatrix

APPENDIX B

Laboratory Testing

AMEC Geomatrix

MOISTURE CONTENT AND DRY DENSITY TEST

Project Name: Suanshine Canyon LFGTE Facility　　**Project - Task No.:** 0148280000

Date: 7/27-7/28/10　　**Tested By:** VC, LT

Boring No.	Sample No.	Depth (ft)	Wet Soil and Rings (gr)	Number of Rings	Weight of Rings (gr)	Volume of Sample (cc)	Wet Density (pcf)	Wet Soil and Tare (gr)	Dry Soil and Tare (gr)	Tare No.	Tare Weight (gr)	Moisture Content (%)	Dry Density (pcf)	Soil Description
BA-2	1	28.8-30.3	---	---	---	---	---	216.58	189.82	9	96.89	28.8	---	Fat Clay with Sand (CH)

Ring I.D. (in.) = 2.416　Ring Height (in.) = 1.000

Average weight of 1 ring (gr.):

MOISTURE CONTENT AND DRY DENSITY TEST

Project Name: Suanshine Canyon LFGTE Facility **Project - Task No.:** 0148280000
Date: 7/27-7/28/10 **Tested By:** VC, LT

Boring No.	Sample No.	Depth (ft)	Wet Soil and Rings (gr)	Number of Rings	Weight of Rings (gr)	Volume of Sample (cc)	Wet Density (pcf)	Wet Soil and Tare (gr)	Dry Soil and Tare (gr)	Tare No.	Tare Weight (gr)	Moisture Content (%)	Dry Density (pcf)	Soil Description
B-1	3	8.5-9.0	See Direct Shear Test			---	---	---	---	---	---	10.9	95.8	Sandy Silt (ML)
B-5	1	3.5-4.0	See Direct Shear Test			---	---	---	---	---	---	17.2	102.9	Clayey Sand (SC)
B-6	9	23.5-24.0	See Direct Shear Test			---	---	---	---	---	---	12.4	111.50	Sandy Lean Clay (CL)
BA-2	1	28.8-30.3	---	---	---	---	---	216.58	189.82	9	96.89	28.8	---	Fat Clay with Sand (CH)
CH-01	1	9.7-10.4	See UU Triaxial Compression Test				---	---	---	---	---	16.8	127.40	Bedrock Core-Olive Gray (5Y, 4/2) Sandy Lean Clay (CL)
CH-01	2	31.4-32.2	See UU Triaxial Compression Test				---	---	---	---	---	12.7	118.0	Bedrock Core-Very Dark Grayish Brown (2.5Y, 3/2) Silt with Sand (ML)
CH-01	3	34.0-34.8	See UU Triaxial Compression Test				---	---	---	---	---	14.2	117.5	Bedrock Core-Dark Grayish Brown (2.5Y, 4/2) Sandy Silt (ML)
CH-01	4	38.0-38.5	See UU Triaxial Compression Test				---	---	---	---	---	13.5	122.4	Bedrock Core-Very Dark Gray (5Y, 3/1) Silt (ML)
CH-01	5	43.3-44.0	See UU Triaxial Compression Test				---	---	---	---	---	11.4	126.2	Bedrock Core-Dark Gray (5Y, 4/1) Silt with Sand (ML)
CH-01	6	49.8-50.3	See UU Triaxial Compression Test				---	---	---	---	---	13.4	123.9	Bedrock Core-Dark Gray (5Y, 4/1) Silt (ML)
CH-01	7	36.0-36.5	See Unconfined Compression Test				---	---	---	---	---	13.2	116.6	Bedrock Core: Very Dark Grayish Brown (2.5Y, 3/2) Sandy Silt (ML)
CH-01	8	42.0-42.5	See Unconfined Compression Test				---	---	---	---	---	10.7	125.3	Bedrock Core: Very Dark Gray (2.5Y, 3/1) Silt with Sand (ML)
CH-01	9	44.0-44.5	See Unconfined Compression Test				---	---	---	---	---	12.0	123.4	Bedrock Core: Dark Gray (2.5Y, 4/1) Sandy Silt (ML)

Ring I.D. (in.) = 2.416 Ring Height (in.) = 1.000
Average weight of 1 ring (gr.):

AP Engineering & Testing, Inc.

MOISTURE AND DENSITY TEST RESULTS

Client: AMEC Geomatrix
Project Name: Sunshine Canyon LFGTE Facility
Project Number: 0148280000

Laboratory No.: 10-0802
Date: 08/03/10

Boring No.	Sample No.	Sample Depth (feet)	Moisture Content (%)	Dry Density (pcf)
B-1	-	7.5-9	10.5	101.2
B-1	-	15-16.5	11.9	97.4
B-1	-	25-26.5	13.0	107.2
B-1	-	50-51.5	18.4	106.5
B-2	-	2.5-4	16.8	103.6
B-3	-	2.5-4	13.2	106.8
B-3	-	10-11.5	5.3	118.5
B-3	-	20-21.5	18.9	96.2
B-3	-	30-31.5	26.4	95.7
B-4	-	2.5-4	15.2	108.6
B-4	-	12.5-14	16.1	105.9
B-4	-	17.5-19	15.4	110.3
B-5	-	2.5-4	16.7	105.2
B-5	-	10-11.5	10.4	101.5
B-5	-	20-21.5	11.9	106.1
B-6	-	2.5-4	11.8	105.3
B-6	-	7.5-9	11.6	103.7
B-6	-	12.5-14	12.5	106.3
B-6	-	17.5-19	10.6	114.9
B-6	-	22.5-24	14.8	110.3
B-6	-	27.5-29	11.7	115.3
B-6	-	32.5-34	11.0	104.3
B-6	-	37.5-39	12.2	115.9
B-6	-	42.5-44	12.8	107.2
B-6	-	47.5-49	11.5	100.9

AMEC Geomatrix

PARTICLE SIZE ANALYSIS

ASTM-D422

Project Name:	Sunshine Canyon LFGTE Facility			Project No.:	0148280000	Date:	7/27-7/30/10
Boring No.:	BA-2	Sample No.:	1	Depth:	28.8-30.3 Ft	Tested By: LT, VC	
Soil Description:	Fat Clay with Sand (CH)						

SIEVE ANALYSIS

Sieve Size		Weight Retained		Percent Retained		Percent Passing
Standard	Other	Individual	Cumulative	Individual	Cumulative	(Cumulative)
3"						
2"						
1.5"						
1"						
3/4"			0.00		0.0	100.0
3/8"			1.95		2.1	97.9
#4			2.14		2.3	97.7
#10			6.16		6.6	93.4
#20			9.58		10.3	89.7
#40			11.56		12.4	87.6
#60			12.75		13.7	86.3
#140			15.82		17.0	83.0
#200			19.91		21.4	78.6

Tare No.:	9
Dry wt. and tare, gr.:	189.82
Tare weight, gr.:	96.89
Total dry weight, gr.:	92.93
Tare No., Hydromtr.:	13
Tare No., Hygroscp.:	MC-58
Soaking Container:	H-7
Jar No.:	1

Notes:

HYDROMETER ANALYSIS (152H)

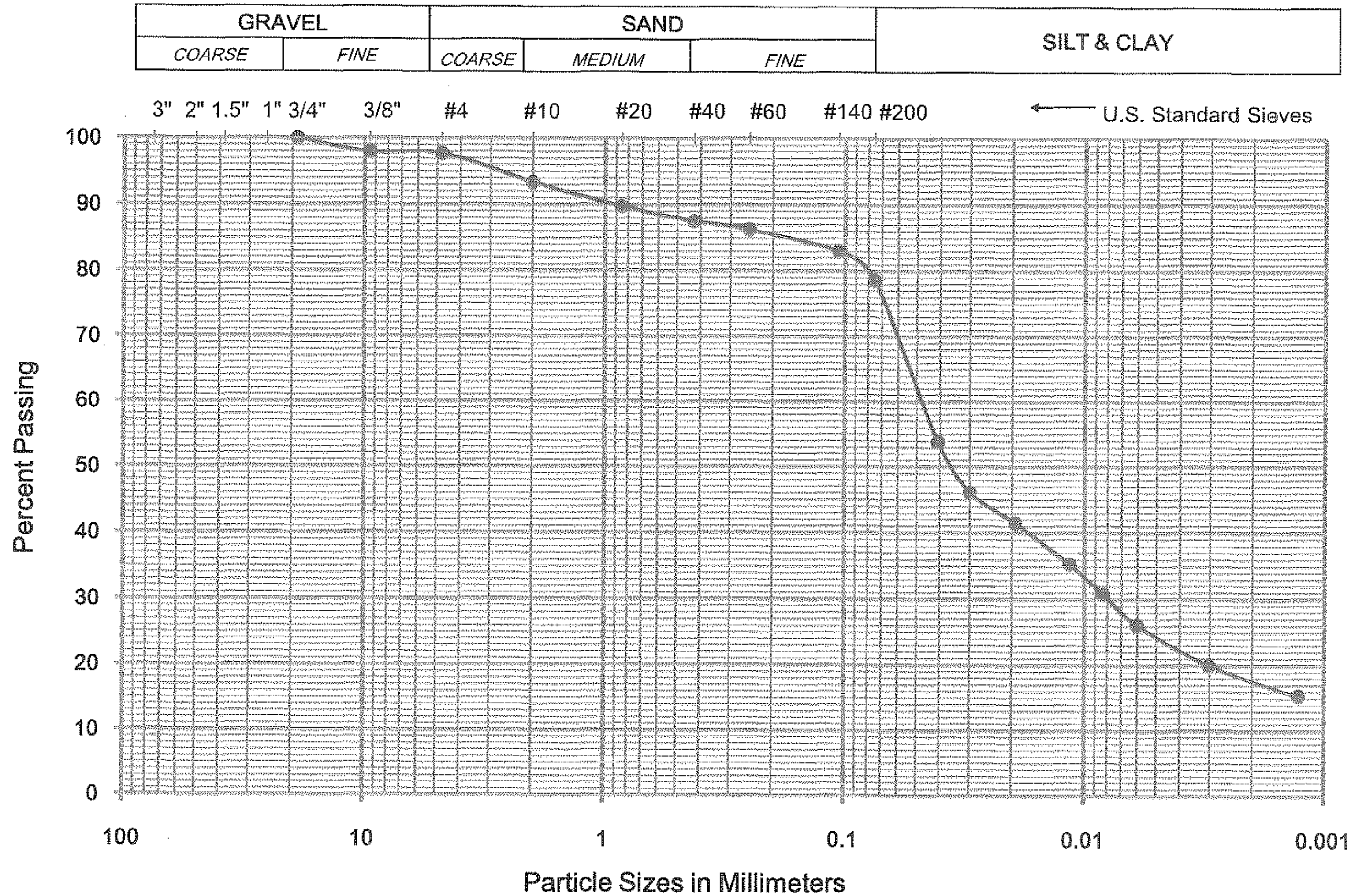
Composite Correction (C_C) =	6 @ 18°C	*Hygroscopic Moisture:*			
Composite Correction (C_C) =	5 @ 20°C	Air-dry Mass, gr.:	25.63	Air-dry Mass in Test (W_1), gr.:	63.21
Composite Correction (C_C) =	3 @ 25°C	Oven-dry Mass, gr.:	24.38	Oven-dry Mass in Test (W_2),gr.:	60.13
Composite Correction (C_C) =	@ °C	Correction Factor (F_C):	0.951		(W_1xF_C)

Specific Gravity:	2.70	Total Mass Represented by the Mass Used in the Hydrometer Test (W), gr.:	64.40
Correction Factor *a*	0.99		(W_2/Percent <#10)

Date	Time	Elapsed Time,min. (T)	Temp. (°C)	Actual Reading (R_1)	Composite Correction (C_C)	Hydrometer Reading (R)	Percent Passing (P) [1]	Value of K	Effective Depth (L)	Diameter of Particle, mm (D) [2]
7/29/10	10:00:58	0.00	---	---	---	---	---	---	---	---
7/29/10	10:01:58	1.00	23	39	4	35	53.8	0.01297	9.9	0.04081
7/29/10	10:02:58	2.00	23	34	4	30	46.1	0.01297	10.7	0.03000
7/29/10	10:05:58	5.00	23	31	4	27	41.5	0.01297	11.2	0.01941
7/29/10	10:15:58	15.00	23	27	4	23	35.4	0.01297	11.9	0.01155
7/29/10	10:30:58	30.00	23	24	4	20	30.7	0.01297	12.4	0.00834
7/29/10	11:00:58	60.00	23	21	4	17	26.1	0.01297	12.9	0.00601
7/29/10	14:10:58	250.00	23	17	4	13	20.0	0.01297	13.5	0.00301
7/30/10	10:00:58	1440.00	23	14	4	10	15.4	0.01297	14.0	0.00128

[1] P = (R*a*/W)*100

[2] D = K*SQRT(L/T)



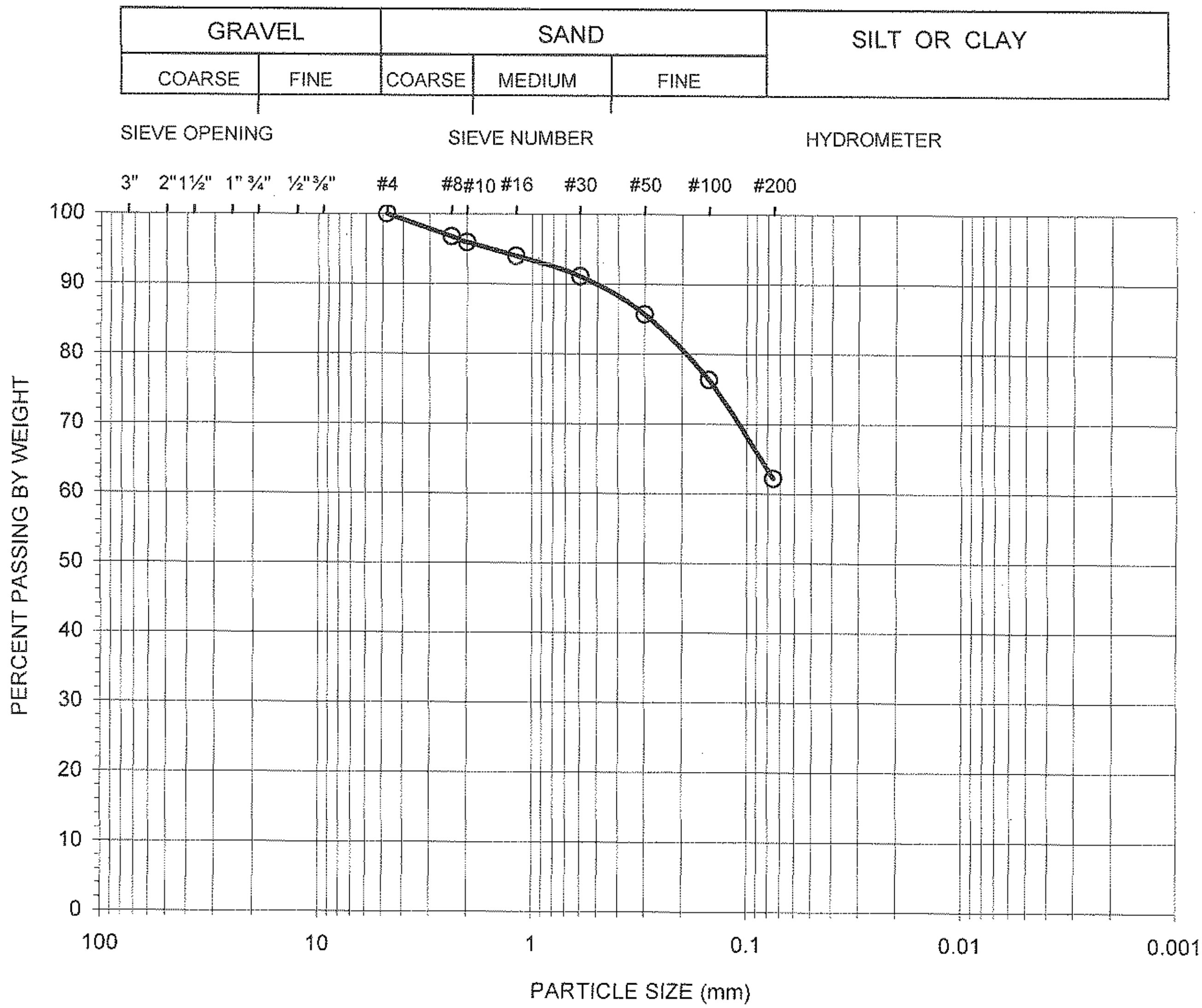
Boring No.:	BA-2	Sample No.:	1	Depth:	28.8-30.3 Ft
Soil Description:	Fat Clay with Sand (CH)				

AMEC Geomatrix	GRAIN SIZE DISTRIBUTION CURVE SUNSHINE CANYON LFGTE FACILITY Sylmar, California	Project No. 0148280000

AP Engineering & Testing, Inc.

GRAIN SIZE DISTRIBUTION CURVE
ASTM D 422

Client Name:	AMEC Geomatrix	Tested by:	ST	Date:	08/11/10
Project Name:	Sunshine Canyon LFGTE Facility	Computed by:	KM	Date:	08/13/10
Project No.:	0148280000	Checked by:	AP	Date:	08/13/10

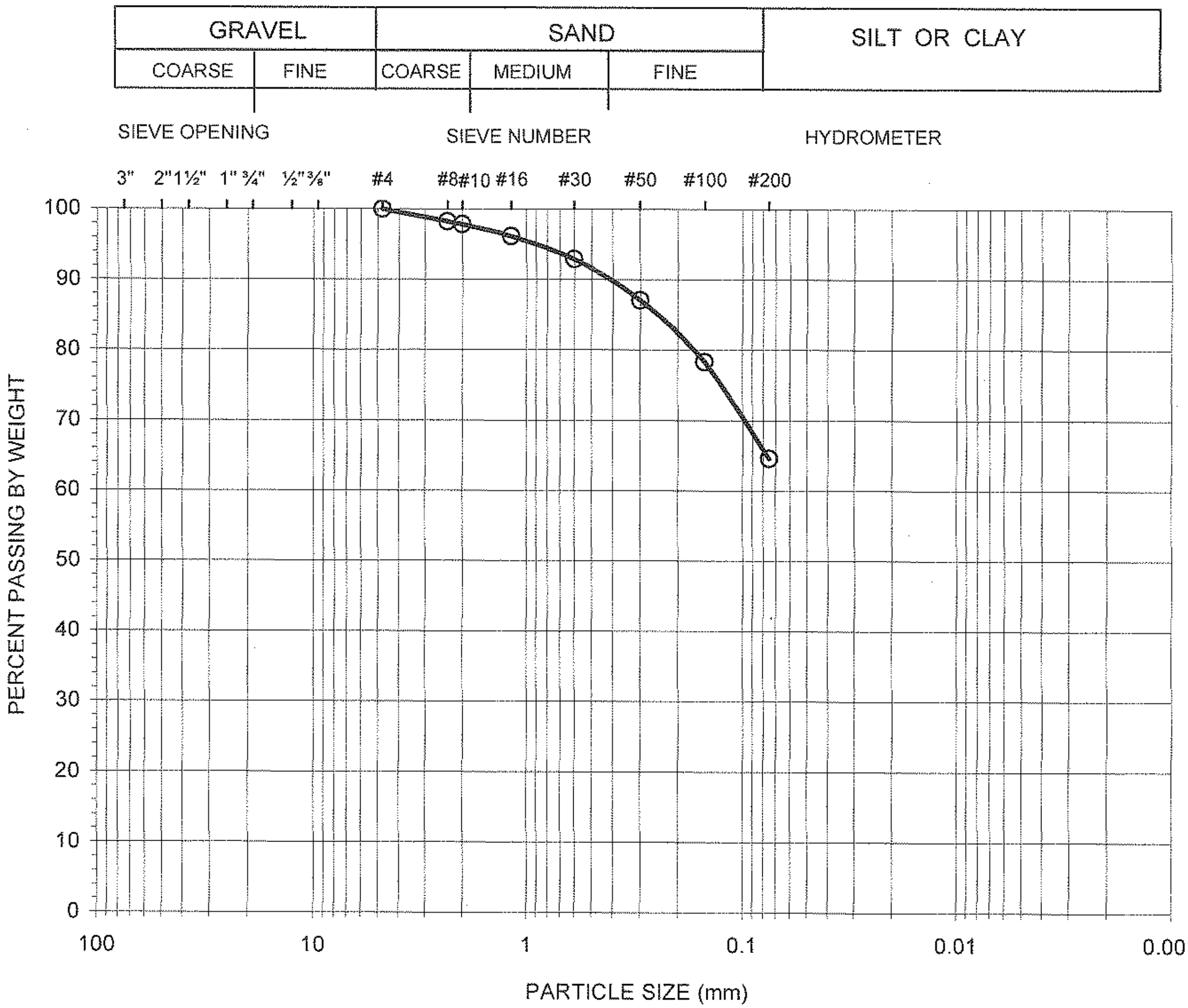


Symbol	Boring No.	Sample No.	Sample Depth (feet)	Percent			Atterberg Limits LL:PL:PI	Soil Symbol ASTM D 2487
				Gravel	Sand	Fines		
○	B-3	-	5-10	0	38	62	N/A	ML

AP Engineering & Testing, Inc.

GRAIN SIZE DISTRIBUTION CURVE
ASTM D 422

Client Name:	AMEC Geomatrix	Tested by:	ST	Date:	08/11/10
Project Name:	Sunshine Canyon LFGTE Facility	Computed by:	KM	Date:	08/13/10
Project No.:	0148280000	Checked by:	AP	Date:	08/13/10



Symbol	Boring No.	Sample No.	Sample Depth (feet)	Percent			Atterberg Limits LL:PL:PI	Soil Symbol ASTM D 2487
				Gravel	Sand	Fines		
O	B-6	-	5-10	0	35	65	N/A	ML

AMEC Geomatrix

PLASTICITY INDEX

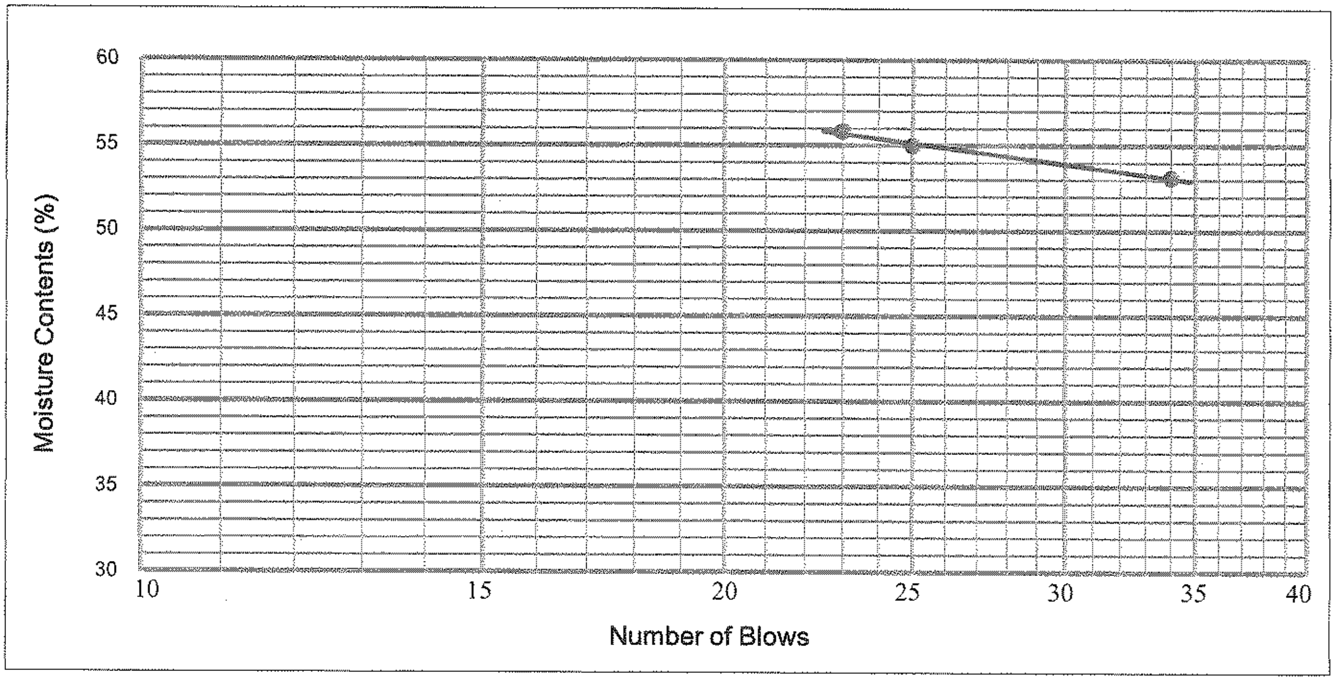
ASTM-D 4318

Project Name:	Sunshine Canyon LFGTE Facility			Project No.:	0148280000
Boring No.:	BA-2	Sample No.:	1	Depth: 28.8-30.3 Ft	Date: 7/27-7/29/10
Soil Description:	Fat Clay with Sand (CH)			Tested by:	VC, LT

DRYING PAN No.: 6 SOAKING DISH No.: A-12

LIQUID LIMIT (LL)

TARE No.:	C-58	C-40	C-8
NUMBER OF BLOWS	23	25	34
WEIGHT OF WET SOIL + TARE, gr.:	32.75	34.03	33.56
WEIGHT OF DRY SOIL + TARE, gr.:	26.62	27.30	27.45
WEIGHT OF TARE, gr.:	15.64	15.06	15.95
MOISTURE CONTENT, %:	**55.8**	**55.0**	**53.1**



LIQUID LIMIT = 55

PLASTIC LIMIT (PL)

TARE No.:	C-48	C-38
WEIGHT OF WET SOIL + TARE, gr.:	24.06	23.88
WEIGHT OF DRY SOIL + TARE, gr.:	22.12	22.11
WEIGHT OF TARE, gr.:	15.07	15.75
MOISTURE CONTENT, %:	**27.5**	**27.8**

PLASTIC LIMIT = 28

PLASTICITY INDEX (PI) = LL - PL = 27

AMEC Geomatrix

PLASTICITY INDEX

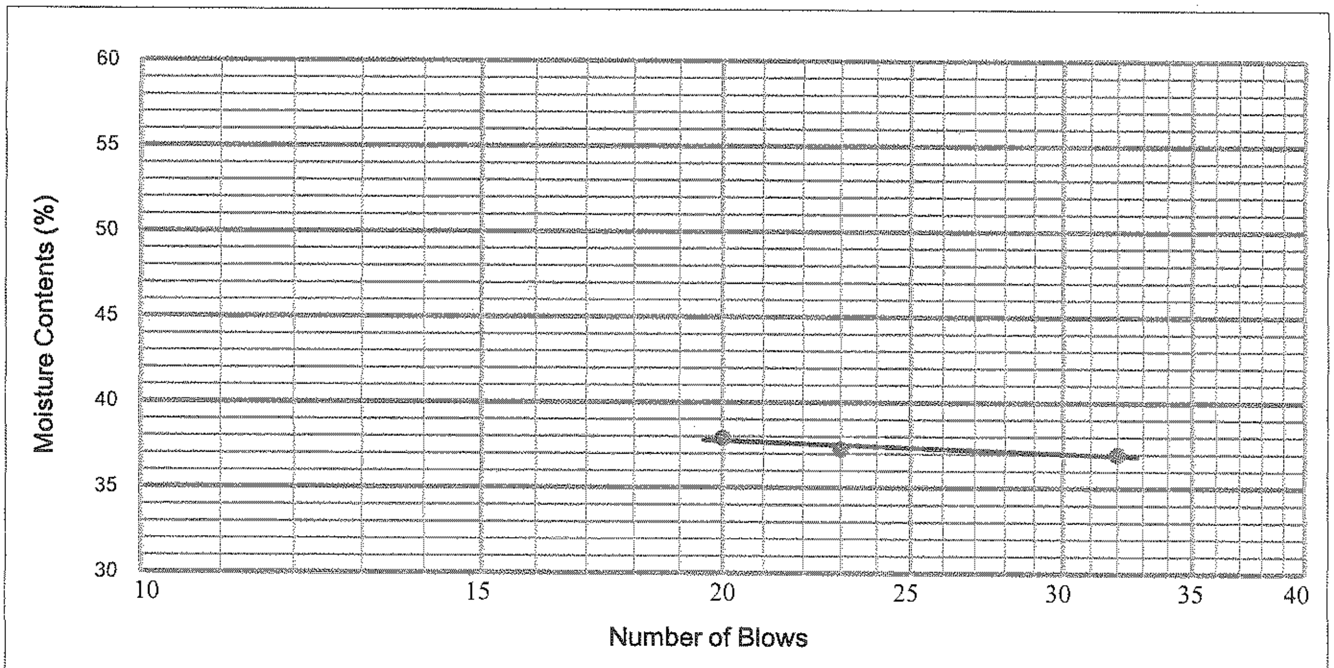
ASTM-D 4318

Project Name:	Sunshine Canyon LFGTE Facility		**Project No.:**	0148280000	
Boring No.:	CH-01	**Sample No.:** 1	**Depth:**	9.7-10.4 Ft	**Date:** 7/27-7/29/10
Soil Description:	Bedrock Core-Olive Gray (5Y, 4/2) Sandy Lean Clay (CL)			**Tested by:**	VC, LT

DRYING PAN No.: 7 SOAKING DISH No.: A-1

LIQUID LIMIT (LL)

TARE No.:	C-1	C-78	C-2
NUMBER OF BLOWS	20	23	32
WEIGHT OF WET SOIL + TARE, gr.:	33.87	37.77	40.14
WEIGHT OF DRY SOIL + TARE, gr.:	28.91	31.74	33.54
WEIGHT OF TARE, gr.:	15.83	15.56	15.69
MOISTURE CONTENT, %:	**37.9**	**37.3**	**37.0**



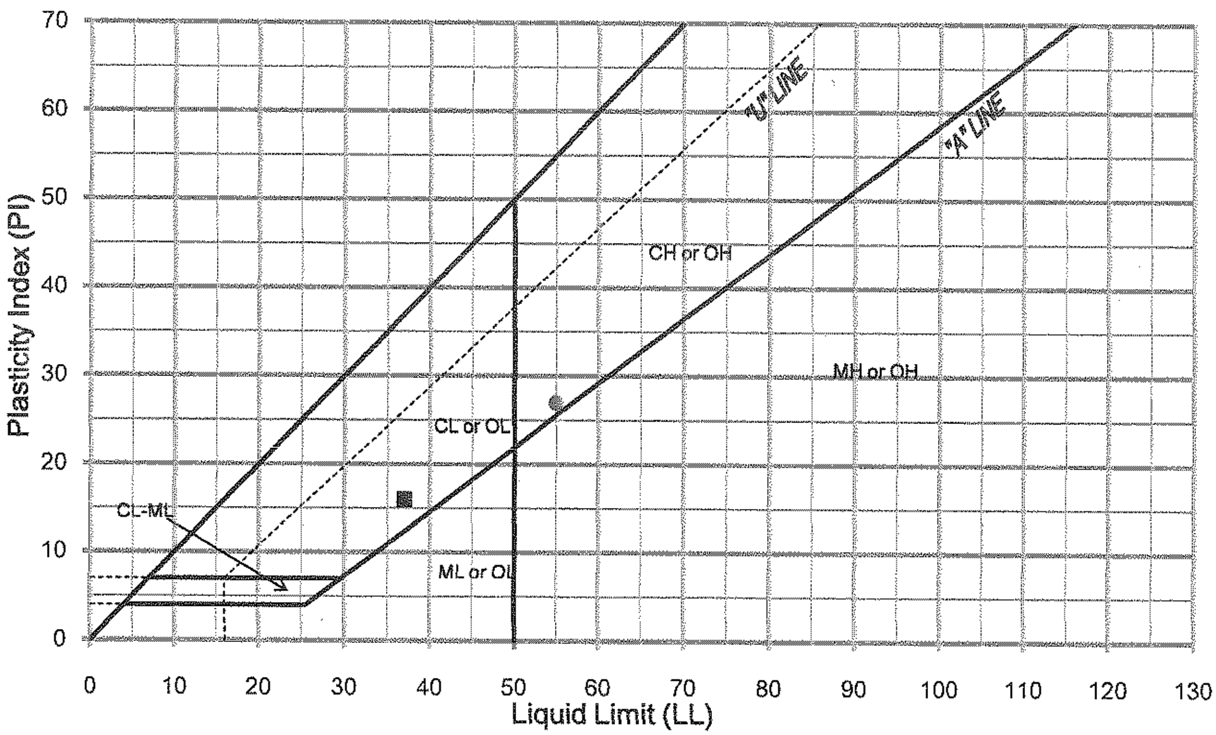
LIQUID LIMIT = 37

PLASTIC LIMIT (PL)

TARE No.:	C-70	C-49
WEIGHT OF WET SOIL + TARE, gr.:	25.72	25.01
WEIGHT OF DRY SOIL + TARE, gr.:	23.89	23.25
WEIGHT OF TARE, gr.:	15.38	14.96
MOISTURE CONTENT, %:	**21.5**	**21.2**

PLASTIC LIMIT = 21

PLASTICITY INDEX (PI) = LL - PL = 16



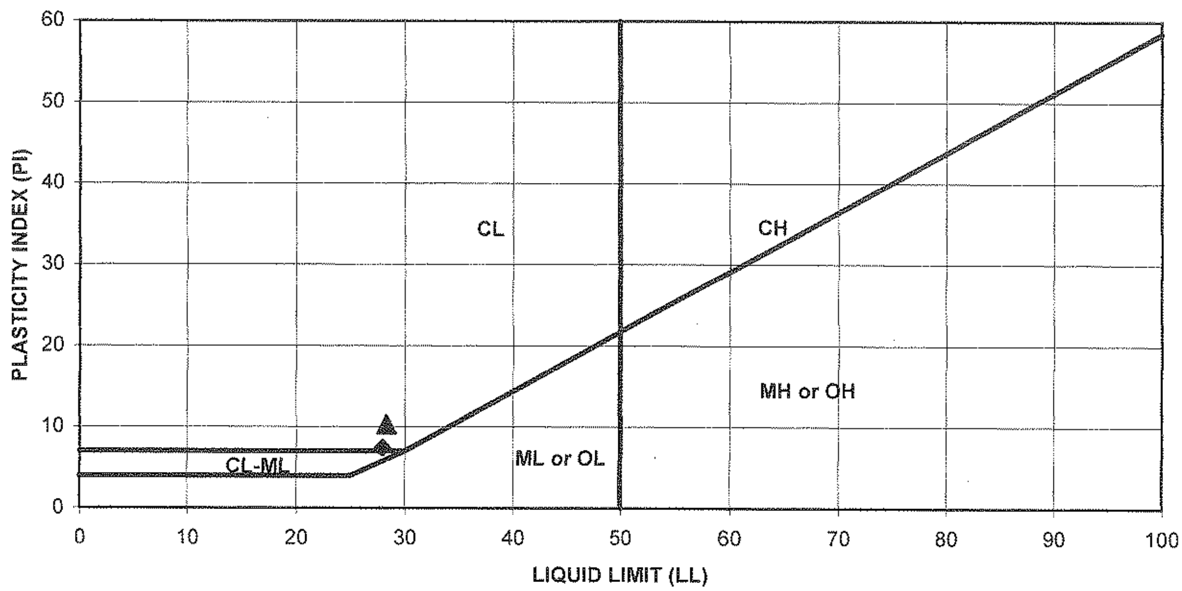
Legend	Boring No.	Sample No.	Sample Depth (Feet)	Liquid Limit (LL)	Plastic Limit (PL)	Plasticity Index (PI)	Group Symbol of Sample
●	BA-2	1	28.8-30.3	55	28	27	CH
■	CH-01	1	9.7-10.4	37	21	16	CL

AMEC Geomatrix	PLASTICITY INDEX (PI) SUNSHINE CANYON LFGTE FACILITY Sylmar, California	Project No. 0148280000

AP Engineering & Testing, Inc.

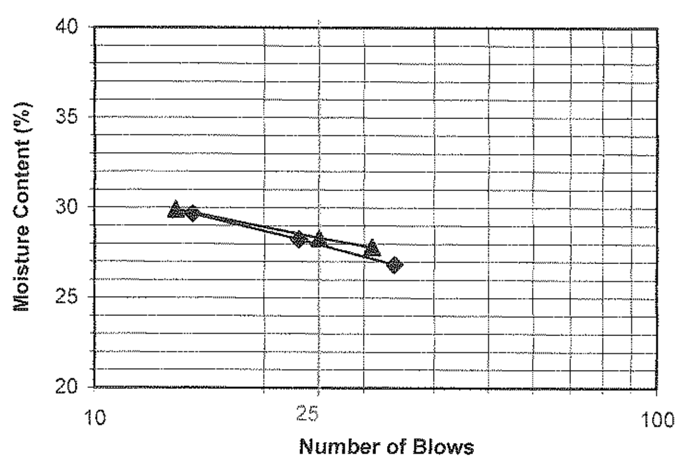
ATTERBERG LIMITS
ASTM D 4318

Project Name: Sunshine Canyon LFGTE Facility Tested By: DK Date: 08/09/10
Project No.: 0148280000 Checked By: AP Date: 08/10/10



PROCEDURE USED

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



Symbol	Boring Number	Sample Number	Depth (feet)	LL	PL	PI	U.S.C.S Symbol
◆	B-4	-	10-11.5	28	21	7	CL-ML
▲	B-6	-	10-11.5	28	18	10	CL

* NP denotes "non-plastic"

AP Engineering & Testing, Inc.

EXPANSION INDEX TEST RESULTS

ASTM D 4829

Client Name: AMEC Geomatrix

Project Name: Sunshine Canyon LFGTE Facility

Project No.: 0148280000

AP Job No.: 10-0802

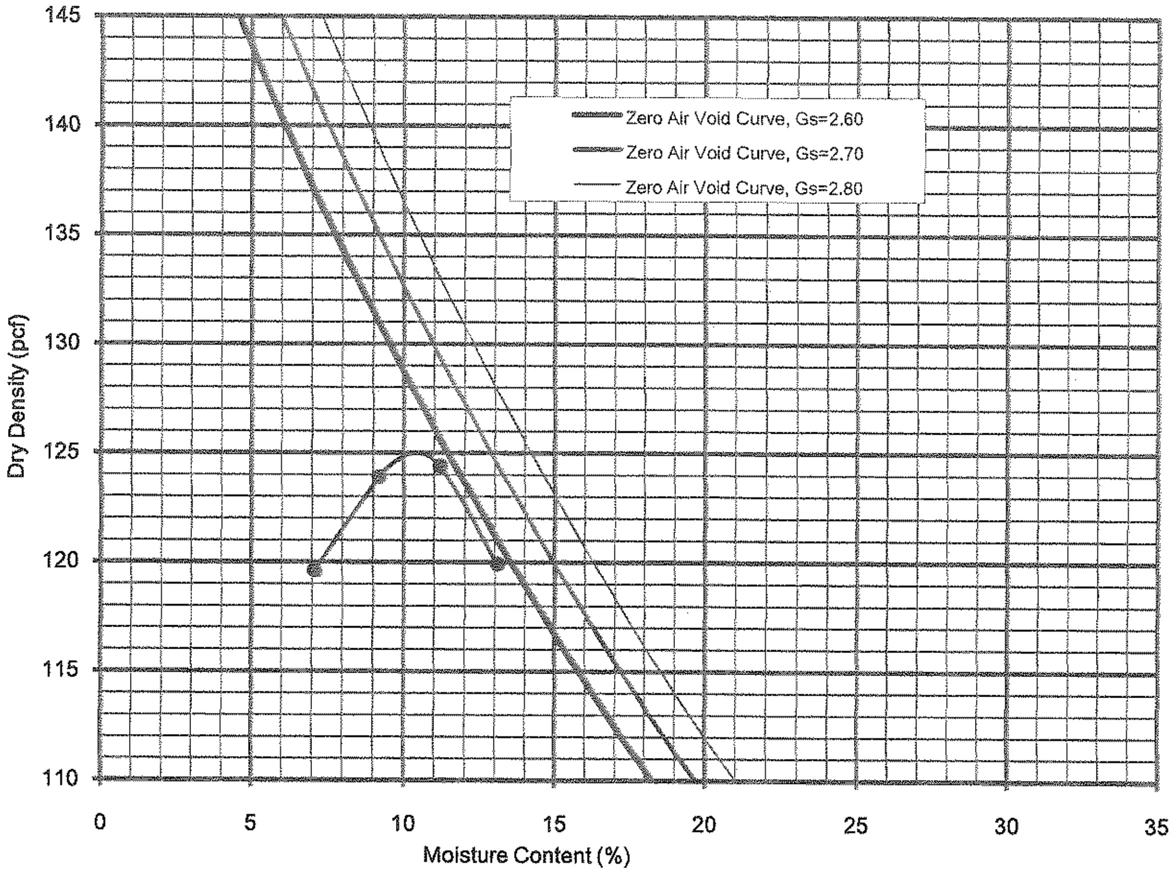
Date: 08/06/10

Boring No.	Sample No.	Depth (ft)	Soil Description	Molded Dry Density (pcf)	Molded Moisture Content (%)	Init. Degree Saturation (%)	Measured Expansion Index	Corrected Expansion Index
B-1	-	5-6.5	Silty Sand	102.6	10.7	45.1	28	25
B-3	-	5-6.5	Silty Sand	104.4	10.7	47.2	22	20
B-5	-	15-16.5	Silty Sand	109.8	9.7	49.1	24	24

ASTM EXPANSION CLASSIFICATION

Expansion Index	Classification
0-20	V. Low
21-50	Low
51-90	Medium
91-130	High
>130	V. High

COMPACTION TEST REPORT



SAMPLE DATA

Boring No.:	B-5	Depth:	15-20 Feet
Soil Description:	Clayey Sand (SC)		
Date:	8/19/2010	Tested by:	L.T.

TEST DATA

Standard: ASTM-D1557-B	Mold Volume (ft^3): 1/30	Hammer Weight (lb.): 10	
Drop Height (Inches): 18	Number of Layers: 5	Blows per Layer: 25	

	1	2	3	4	*Oversize Correction*	
Water Added (gr.)	+47	+94	0	-47	~~Yes~~	No
Weight of Wet Soil & Mold (lb.)	9.08	8.99	8.98	8.74	Coarser (lb.)	Finer (lb.)
Weight of Mold (lb.)	4.47	4.47	4.47	4.47		
Tare No.	36	27	31	32		
Wet Soil & Tare (gr.)	719.63	727.34	653.47	698.11		
Dry Soil & Tare (gr.)	663.51	662.04	612.42	662.19		
Tare Weight (gr.)	162.05	164.22	165.89	156.75		
Moisture Content (%)	11.2	13.1	9.2	7.1		
Dry Density (lb./ft^3)	124.4	119.9	123.9	119.6	------	------

	Uncorrected Value	*Corrected Value*
Maximum Dry Density (lb/ft³)	125.0	N/A
Optimum Moisture Content (%)	10.5	N/A

AMEC Geomatrix	MAXIMUM DRY DENSITY - OPTIMUM MOISTURE CONTENT CURVE SUNSHINE CANYON LFGTE FACILITY Sylmar, California	Project No. 0148280000

AP Engineering & Testing, Inc.

COMPACTION TEST

Client:	AMEC Geomatrix			AP Number:	10-0802
Project Name:	Sunshine Canyon LFGTE Facility	Tested By:	JT	Date:	08/06/10
Project No. :	0148280000	Calculated By:	KM	Date:	08/09/10
Boring No.:	B-3	Checked By:	AP	Date:	08/09/10
Sample No. :	-	Depth (ft.) :	5-10		

Visual Sample Description: Olive Brown Silty Sand

Compaction Method: [X] ASTM D1557 [ ] ASTM D698

Preparation Method: [ ] Moist [X] Dry

METHOD: A

MOLD VOLUME (CU.FT): 0.0333

Trial No.	1	2	3	4	5	6
Wt. Comp. Soil + Mold (gm.)	3603	3719	3794	3763		
Wt. of Mold (gm.)	1752	1752	1752	1752		
Net Wt. of Soil (gm.)	1851	1967	2042	2011		
Container No.						
Wt. of Container (gm.)	155.87	194.12	194.54	220.18		
Wet Wt. of Soil + Cont. (gm.)	481.51	672.25	593.20	694.01		
Dry Wt. of Soil + Cont. (gm.)	460.12	629.58	550.11	636.59		
Moisture Content (%)	7.03	9.80	12.12	13.79		
Wet Density (pcf)	122.42	130.09	135.05	133.02		
Dry Density (pcf)	114.38	118.48	120.46	116.90		

Maximum Dry Density (pcf)	**120.5**	Optimum Moisture Content (%)	**12.0**
Maximum Dry Density w/ Rock Correction (pcf)	**123.3**	Optimum Moisture Content w/ Rock Correction (%)	**11.6**

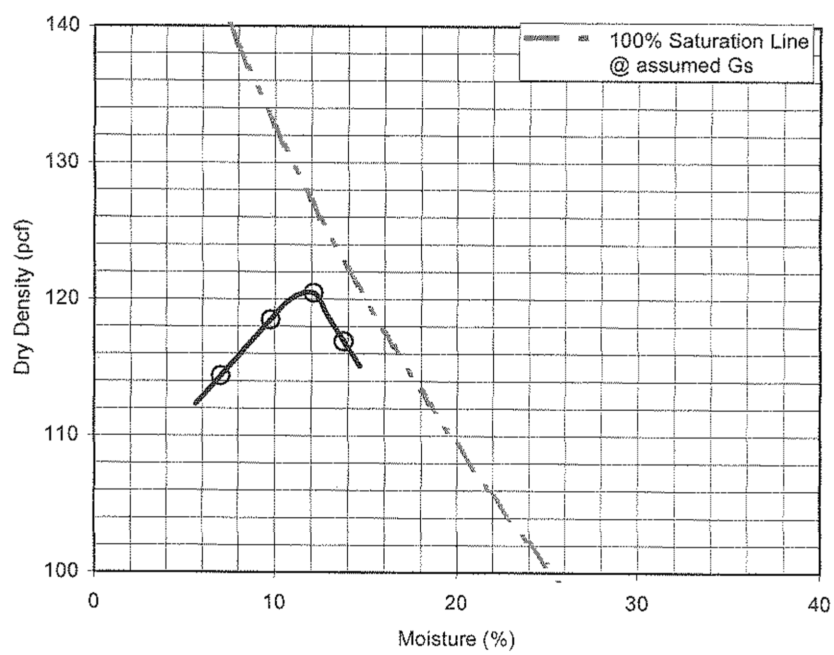
Assumed Specific Gravity = 2.7

PROCEDURE USED

[X] *METHOD A: Percent of Oversize:* *7.8%*
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 No.4 retained < 20%

[ ] *METHOD B: Percent of Oversize:* *NA*
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 + No.4 > 20% and - 3/8 in < 20%

[ ] *METHOD C: Percent of Oversize:* *NA*
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 >20% and +3/4 in <30%



AP Engineering & Testing, Inc.

COMPACTION TEST

Client:	AMEC Geomatrix			AP Number:	10-0802
Project Name:	Sunshine Canyon LFGTE Facility	Tested By:	JT	Date:	08/06/10
Project No. :	0148280000	Calculated By:	KM	Date:	08/09/10
Boring No.:	B-6	Checked By:	AP	Date:	08/09/10
Sample No. :	-	Depth (ft.) :	5-10		

Visual Sample Description: Olive Brown Silty Sand

Compaction Method: [X] ASTM D1557 [ ] ASTM D698

Preparation Method: [ ] Moist [X] Dry

METHOD: A

MOLD VOLUME (CU.FT): 0.0333

Trial No.	1	2	3	4	5	6
Wt. Comp. Soil + Mold (gm.)	3746	3829	3805	3613		
Wt. of Mold (gm.)	1752	1752	1752	1752		
Net Wt. of Soil (gm.)	1994	2077	2053	1861		
Container No.						
Wt. of Container (gm.)	188.56	194.52	195.21	199.41		
Wet Wt. of Soil + Cont. (gm.)	590.86	653.41	801.62	892.72		
Dry Wt. of Soil + Cont. (gm.)	558.08	607.63	729.52	850.41		
Moisture Content (%)	8.87	11.08	13.49	6.50		
Wet Density (pcf)	131.88	137.37	135.78	123.11		
Dry Density (pcf)	121.13	123.66	119.64	115.60		

Maximum Dry Density (pcf)	124.0	Optimum Moisture Content (%)	10.0
Maximum Dry Density w/ Rock Correction (pcf)	126.2	Optimum Moisture Content w/ Rock Correction (%)	9.9

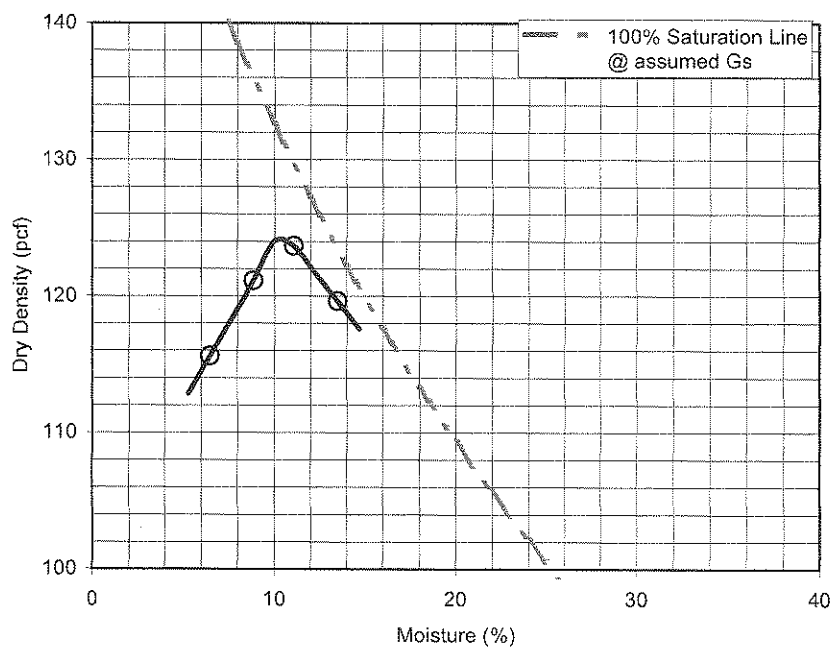
Assumed Specific Gravity = 2.7

PROCEDURE USED

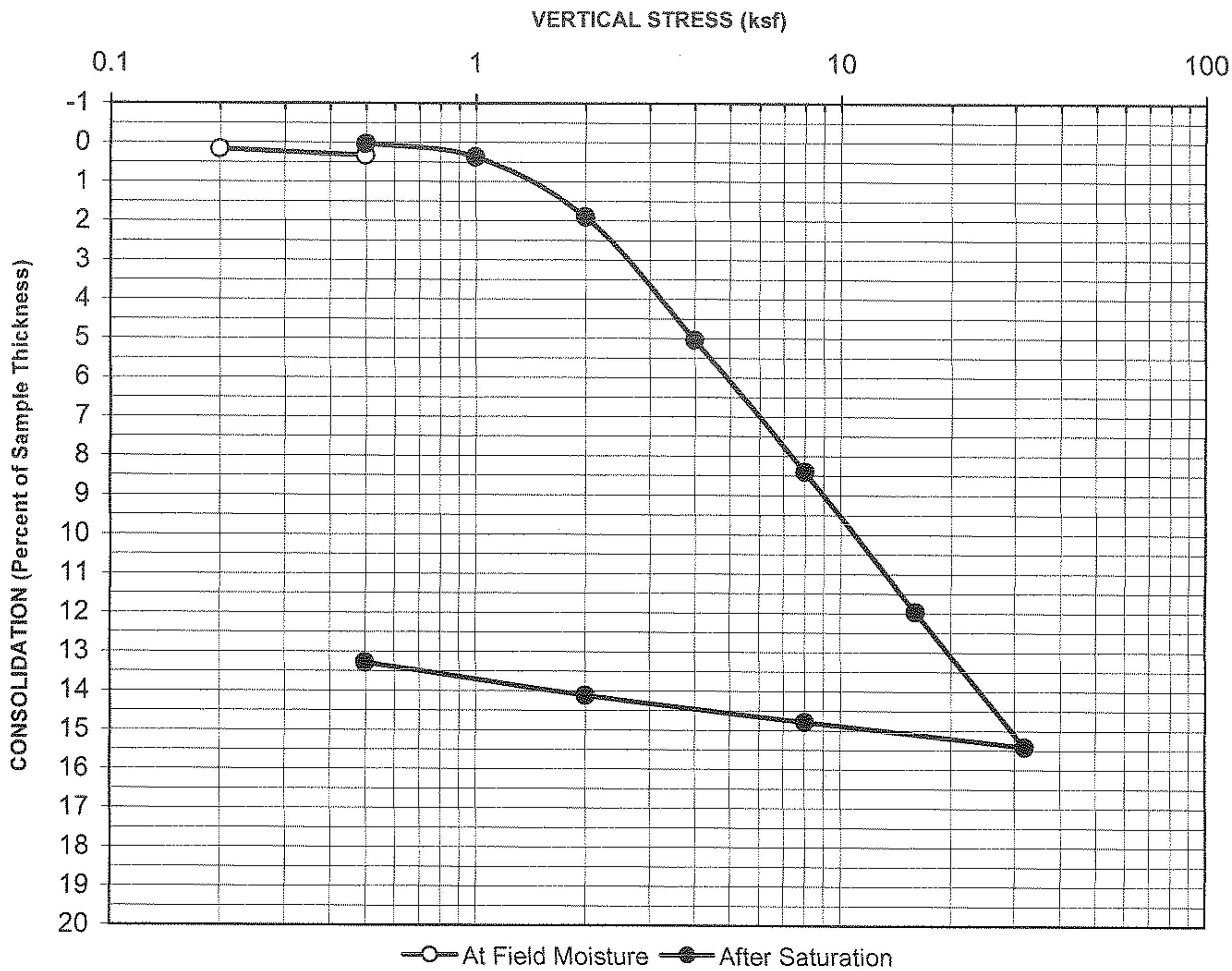
[X] *METHOD A: Percent of Oversize:* 6.7%
Soil Passing No. 4 (4.75 mm) Sieve
Mold : 4 in. (101.6 mm) diameter
Layers : 5 (Five)
Blows per layer : 25 (twenty-five)
May be used if No.4 retained < 20%

[ ] *METHOD B: Percent of Oversize:* NA
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 + No.4 > 20% and - 3/8 in < 20%

[ ] *METHOD C: Percent of Oversize:* NA
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 >20% and +3/4 in <30%



AP Engineering & Testing, Inc.

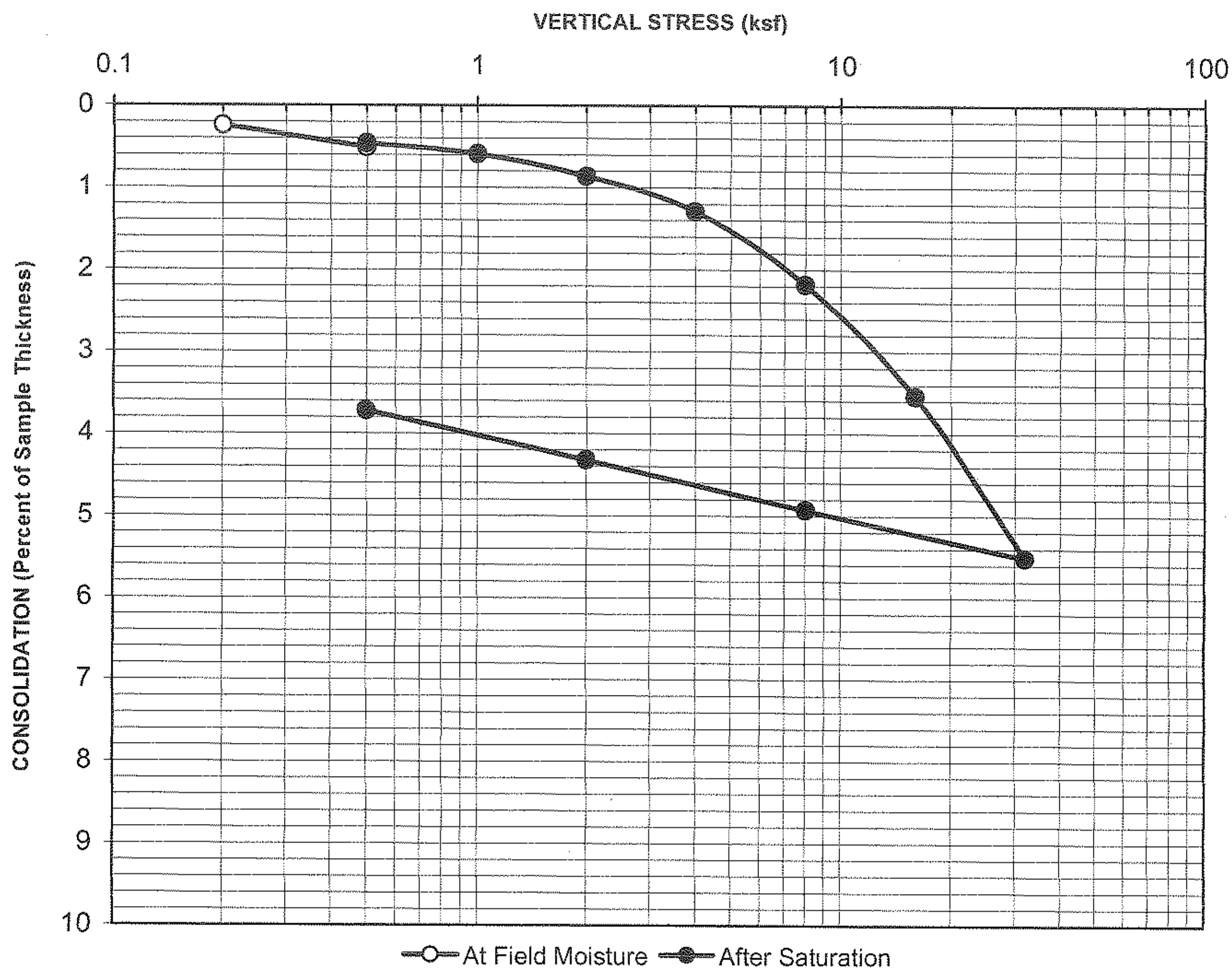


Boring No. :	B-1	Initial Dry Unit Weight (pcf):	97.5
Sample No.:	-	Initial Moisture Content (%):	11.3
Depth (feet):	2.5-4	Final Moisture Content (%):	18.1
Sample Type:	Mod. Cal.	Assumed Specific Gravity:	2.7
Soil Description:	Brown Silt w/siltstone fragments	Initial Void Ratio:	0.73
Remarks:			

CONSOLIDATION CURVE
ASTM D 2435

Project Name: Sunshine Canyon LFGTE Facility
Project No.: 0148280000
Date: 8/2/2010
AP No: 10-0802

AP Engineering & Testing, Inc.

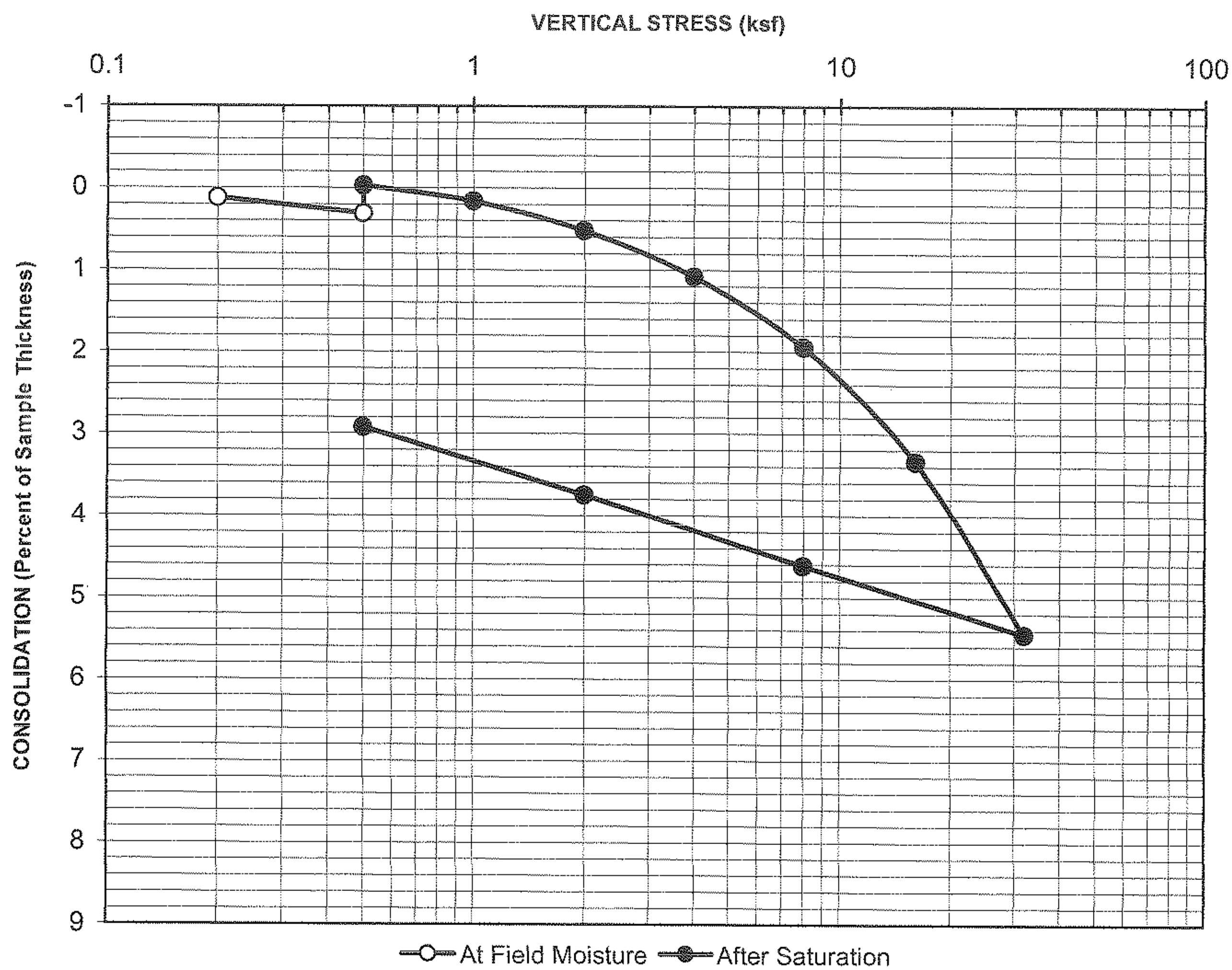


Boring No. :	B-2	Initial Dry Unit Weight (pcf):	108.2
Sample No.:	-	Initial Moisture Content (%):	15.1
Depth (feet):	10-11.5	Final Moisture Content (%):	20.3
Sample Type:	Mod. Cal.	Assumed Specific Gravity:	2.7
Soil Description:	Dark Brown Sandy Silt w/siltstone	Initial Void Ratio:	0.56
Remarks:			

CONSOLIDATION CURVE
ASTM D 2435

Project Name:	Sunshine Canyon LFGTE Facility
Project No.:	0148280000
Date:	8/2/2010
AP No:	10-0802

AP Engineering & Testing, Inc.



Boring No. :	B-4	Initial Dry Unit Weight (pcf):	111.1
Sample No.:	-	Initial Moisture Content (%):	14.2
Depth (feet):	7.5-9	Final Moisture Content (%):	17.5
Sample Type:	Mod. Cal.	Assumed Specific Gravity:	2.7
Soil Description:	Dark Brown Silt w/siltstone	Initial Void Ratio:	0.52
Remarks:			

CONSOLIDATION CURVE
ASTM D 2435

Project Name: Sunshine Canyon LFGTE Facility
Project No.: 0148280000
Date: 8/2/2010
AP No: 10-0802

AMEC Geomatrix

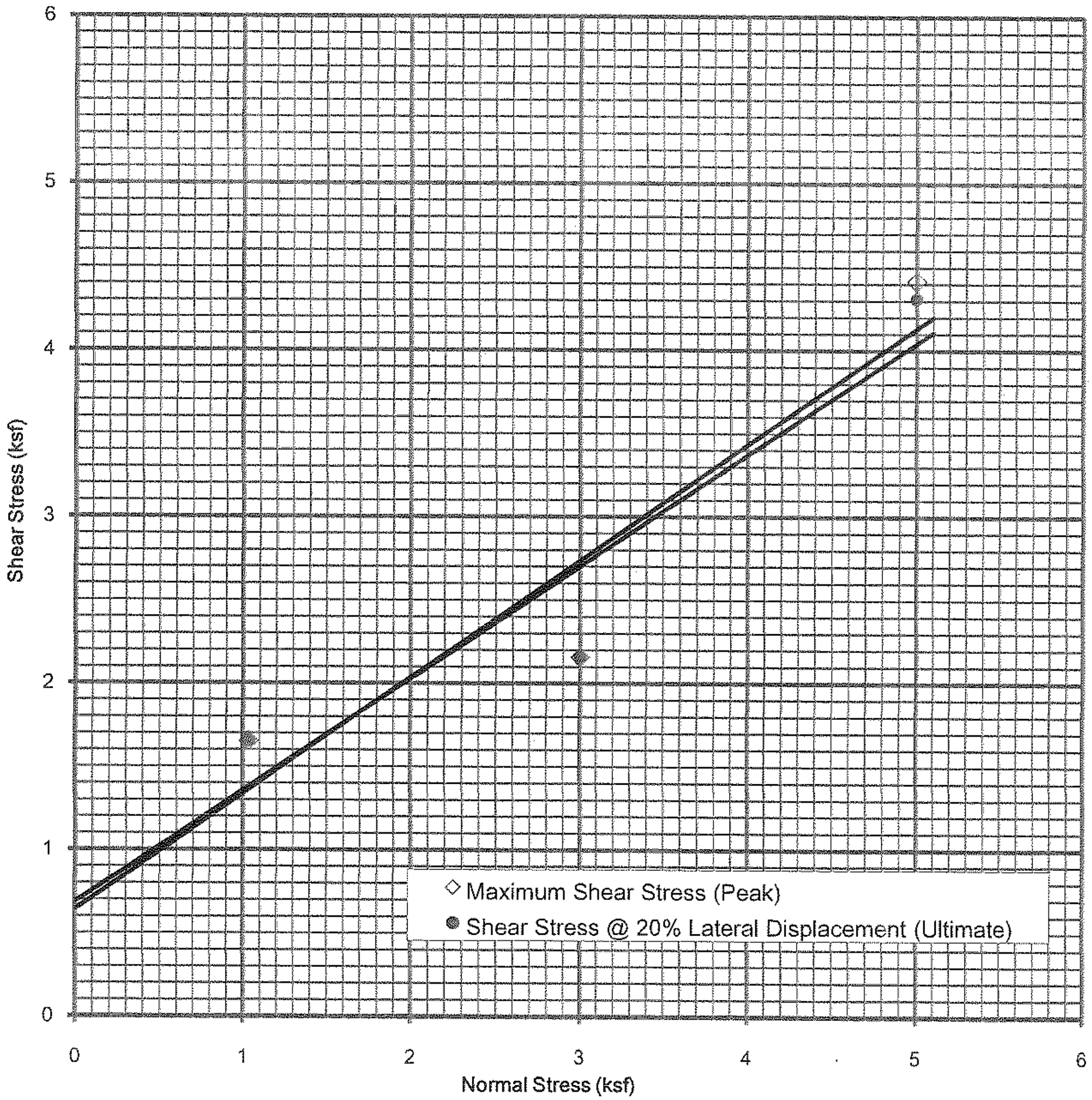
DIRECT SHEAR TEST

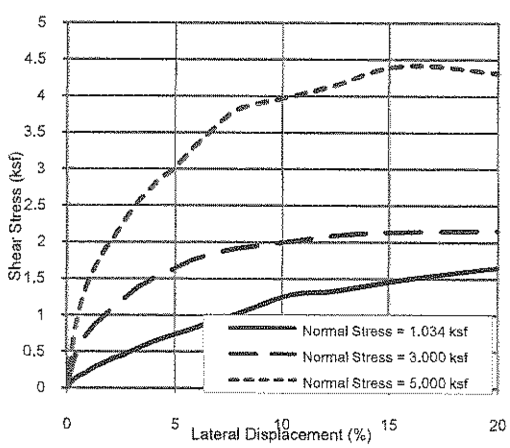
(ASTM-D3080)

Project Name:	Sunshine Canyon LFGTE Facility			**Project No.:**	0148280000		
Boring No.:	B-1	**Sample No.:**	3	**Depth:**	8.5-9.0 Feet	**Date:**	7/26-7/27/10
Soil Description:		Sandy Silt (ML)				**Tested By:**	LT

				Before Test	After Test Load 1	After Test Load 2	After Test Load 3
Sample Diameter, in:	2.416	Weight of Wet Soil & Ring, gr:		513.21	---	---	---
Normal Stress, ksf:	1,3,5	Weight of Ring, gr:		129.76	---	---	---
Over-burdened @, pcf:		Height of Sample, in:		3.00	0.9522	0.9013	0.8779
Shear Rate, in/min:	0.005	Moisture-	Tare No.:	MC-81	---	---	---
Natural Moisture(x):			Wet Weight and Tare, gr:	169.48	142.26	136.57	135.16
Saturated(x):	X		Dry Weight and Tare, gr:	157.82	113.81	112.24	112.50
Intact(x):	X		Tare Weight, gr:	50.44	0.00	0.00	0.00
Remolded to, pcf:			Moisture Content, %:	10.9	25.0	21.7	20.1
@, %:		Wet Density, pcf:		106.2	125.8	129.3	131.1
Notes:		Dry Density, pcf:		95.8	100.6	106.3	109.1
		Saturation %:	S.G. = 2.70 (Assumed)	38.6	100.0	99.9	99.9

Load 1 (KSF): 1.034				Load 2 (KSF): 3.000				Load 3 (KSF): 5.000			
Shear Deflection (in)	Lateral Displacement (%)	Load Ring Reading	Shear Stress (KSF)	Shear Deflection (in)	Lateral Displacement (%)	Load Ring Reading	Shear Stress (KSF)	Shear Deflection (in)	Lateral Displacement (%)	Load Ring Reading	Shear Stress (KSF)
0.0098	0.406	0.0006	0.139	0.0098	0.406	0.0030	0.458	0.0098	0.406	0.0058	0.830
0.0199	0.823	0.0011	0.206	0.0199	0.823	0.0048	0.697	0.0199	0.823	0.0096	1.335
0.0300	1.241	0.0017	0.285	0.0300	1.241	0.0060	0.857	0.0300	1.241	0.0118	1.627
0.0401	1.659	0.0021	0.339	0.0401	1.659	0.0069	0.976	0.0401	1.659	0.0136	1.866
0.0502	2.077	0.0026	0.405	0.0502	2.077	0.0077	1.082	0.0502	2.077	0.0149	2.039
0.0603	2.495	0.0029	0.445	0.0603	2.495	0.0085	1.189	0.0603	2.495	0.0164	2.238
0.0704	2.912	0.0033	0.498	0.0704	2.912	0.0093	1.295	0.0704	2.912	0.0177	2.411
0.0805	3.330	0.0037	0.551	0.0805	3.330	0.0100	1.388	0.0805	3.330	0.0189	2.570
0.0905	3.748	0.0041	0.604	0.0905	3.748	0.0105	1.454	0.0905	3.748	0.0199	2.703
0.1006	4.166	0.0045	0.657	0.1006	4.166	0.0111	1.534	0.1006	4.166	0.0209	2.836
0.1208	5.002	0.0051	0.737	0.1208	5.002	0.0119	1.640	0.1208	5.002	0.0223	3.022
0.1410	5.837	0.0057	0.817	0.1410	5.837	0.0128	1.760	0.1410	5.837	0.0243	3.288
0.1612	6.673	0.0064	0.910	0.1612	6.673	0.0133	1.826	0.1612	6.673	0.0260	3.514
0.1814	7.509	0.0069	0.976	0.1814	7.509	0.0138	1.893	0.1814	7.509	0.0277	3.740
0.2016	8.344	0.0076	1.069	0.2016	8.344	0.0141	1.933	0.2016	8.344	0.0286	3.859
0.2521	10.433	0.0092	1.282	0.2521	10.433	0.0147	2.012	0.2521	10.433	0.0296	3.992
0.3025	12.523	0.0096	1.335	0.3025	12.523	0.0153	2.092	0.3025	12.523	0.0308	4.151
0.3530	14.612	0.0104	1.441	0.3530	14.612	0.0156	2.132	0.3530	14.612	0.0324	4.364
0.4035	16.701	0.0111	1.534	0.4035	16.701	0.0157	2.145	0.4035	16.701	0.0328	4.417
0.4828	19.982	0.0120	1.654	0.4828	19.982	0.0158	2.159	0.4828	19.982	0.0320	4.311
Max. Shear Stress, ksf:			1.654				2.159				4.417
Shear Deflt.@Max Stress,%.:			20.0				20.0				16.7





Boring-Sample No.:	B-1-3
Sample Depth :	8.5-9.0 Feet
Soil Type:	ML
Sample Conditions:	Intact; Saturated
In-Place Dry Density (PCF):	95.8
In-Place Moisture Content (%):	10.9

	◇	●
Cohesion (PSF):	641	687
Friction Angle (Degrees):	35	34

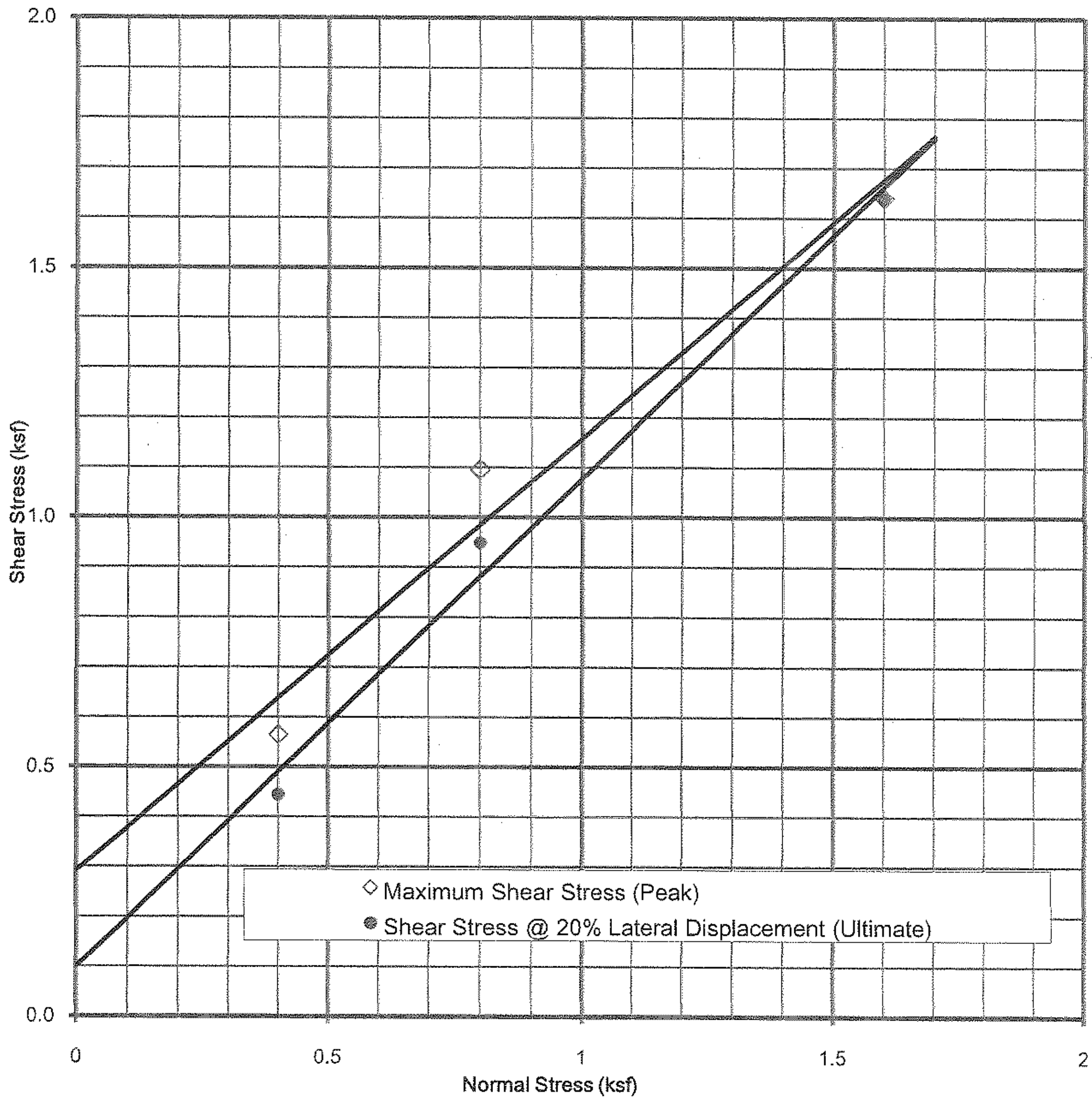
AMEC Geomatrix	DIRECT SHEAR TEST SUNSHINE CANYON LFGTE FACILITY Sylmar, California	Project No. 0148280000

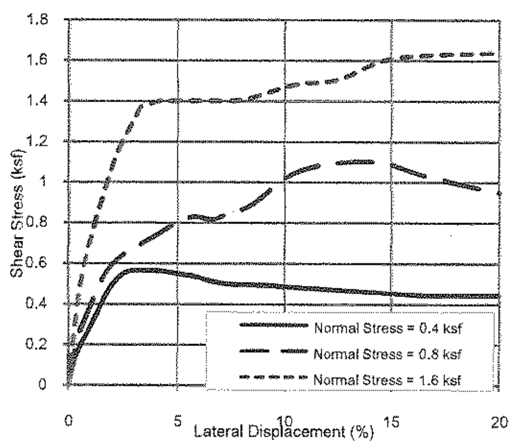
DIRECT SHEAR TEST
(ASTM-D3080)

Project Name:	Sunshine Canyon LFGTE Facility			**Project No.:**	0148280000		
Boring No.:	B-5	**Sample No.:**	1	**Depth:**	3.5-4.0 Feet	**Date:**	8/13-8/17/10
Soil Description:		Clayey Sand (SC)				**Tested By:**	LT

				Before Test	After Test Load 1	After Test Load 2	After Test Load 3
Sample Diameter, in:	2.416	Weight of Wet Soil & Ring, gr:		565.38	---	---	---
Normal Stress, ksf:	0.4,0.8,1.6	Weight of Ring, gr:		130.31	---	---	---
Over-burdened @, pcf:		Height of Sample, in:		3.00	0.9951	0.9876	0.9742
Shear Rate, in/min:	0.005	Moisture-	Tare No.:	MC-75	---	---	---
Natural Moisture(x):			Wet Weight and Tare, gr:	157.91	148.98	149.39	145.32
Saturated(x):	X		Dry Weight and Tare, gr:	142.14	120.79	121.55	119.06
Intact(x):	X		Tare Weight, gr:	50.27	0.00	0.00	0.00
Remolded to, pcf:			Moisture Content, %:	17.2	23.3	22.9	22.1
@, %:		Wet Density, pcf:		120.5	127.5	128.0	128.9
Notes:		Dry Density, pcf:		102.9	103.4	104.1	105.6
		Saturation %:	S.G. = 2.70 (Assumed)	72.6	99.9	100.0	99.8

Load 1 (KSF): 0.4				Load 2 (KSF): 0.8				Load 3 (KSF): 1.6			
Shear Deflec-tion (in)	Lateral Displace-ment (%)	Load Ring Reading	Shear Stress (KSF)	Shear Deflec-tion (in)	Lateral Displace-ment (%)	Load Ring Reading	Shear Stress (KSF)	Shear Deflec-tion (in)	Lateral Displace-ment (%)	Load Ring Reading	Shear Stress (KSF)
0.0098	0.406	0.0008	0.166	0.0098	0.406	0.0013	0.232	0.0098	0.406	0.0026	0.405
0.0199	0.823	0.0015	0.259	0.0199	0.823	0.0022	0.352	0.0199	0.823	0.0044	0.644
0.0300	1.241	0.0022	0.352	0.0300	1.241	0.0030	0.458	0.0300	1.241	0.0058	0.830
0.0401	1.659	0.0029	0.445	0.0401	1.659	0.0037	0.551	0.0401	1.659	0.0069	0.976
0.0502	2.077	0.0034	0.511	0.0502	2.077	0.0041	0.604	0.0502	2.077	0.0079	1.109
0.0603	2.495	0.0037	0.551	0.0603	2.495	0.0044	0.644	0.0603	2.495	0.0087	1.215
0.0704	2.912	0.0038	0.564	0.0704	2.912	0.0045	0.657	0.0704	2.912	0.0093	1.295
0.0805	3.330	0.0038	0.564	0.0805	3.330	0.0048	0.697	0.0805	3.330	0.0099	1.375
0.0905	3.748	0.0038	0.564	0.0905	3.748	0.0050	0.724	0.0905	3.748	0.0100	1.388
0.1006	4.166	0.0038	0.564	0.1006	4.166	0.0052	0.750	0.1006	4.166	0.0101	1.401
0.1208	5.002	0.0037	0.551	0.1208	5.002	0.0056	0.803	0.1208	5.002	0.0101	1.401
0.1410	5.837	0.0036	0.538	0.1410	5.837	0.0058	0.830	0.1410	5.837	0.0101	1.401
0.1612	6.673	0.0034	0.511	0.1612	6.673	0.0057	0.817	0.1612	6.673	0.0101	1.401
0.1814	7.509	0.0033	0.498	0.1814	7.509	0.0060	0.857	0.1814	7.509	0.0101	1.401
0.2016	8.344	0.0033	0.498	0.2016	8.344	0.0062	0.883	0.2016	8.344	0.0102	1.415
0.2521	10.433	0.0032	0.485	0.2521	10.433	0.0074	1.043	0.2521	10.433	0.0107	1.481
0.3025	12.523	0.0031	0.471	0.3025	12.523	0.0078	1.096	0.3025	12.523	0.0109	1.508
0.3530	14.612	0.0030	0.458	0.3530	14.612	0.0078	1.096	0.3530	14.612	0.0116	1.601
0.4035	16.701	0.0029	0.445	0.4035	16.701	0.0073	1.029	0.4035	16.701	0.0118	1.627
0.4828	19.982	0.0029	0.445	0.4828	19.982	0.0067	0.950	0.4828	19.982	0.0119	1.640
Max. Shear Stress, ksf:			0.564				1.096				1.640
Shear Deflt.@Max Stress,%.:			4.2				14.6				20.0





Boring-Sample No.:	B-5-1
Sample Depth :	3.5-4.0 Feet
Soil Type:	SC
Sample Conditions:	Intact; Saturated
In-Place Dry Density (PCF):	102.9
In-Place Moisture Content (%):	17.2

	◇	●
Cohesion (PSF):	292	99
Friction Angle (Degrees):	41	44

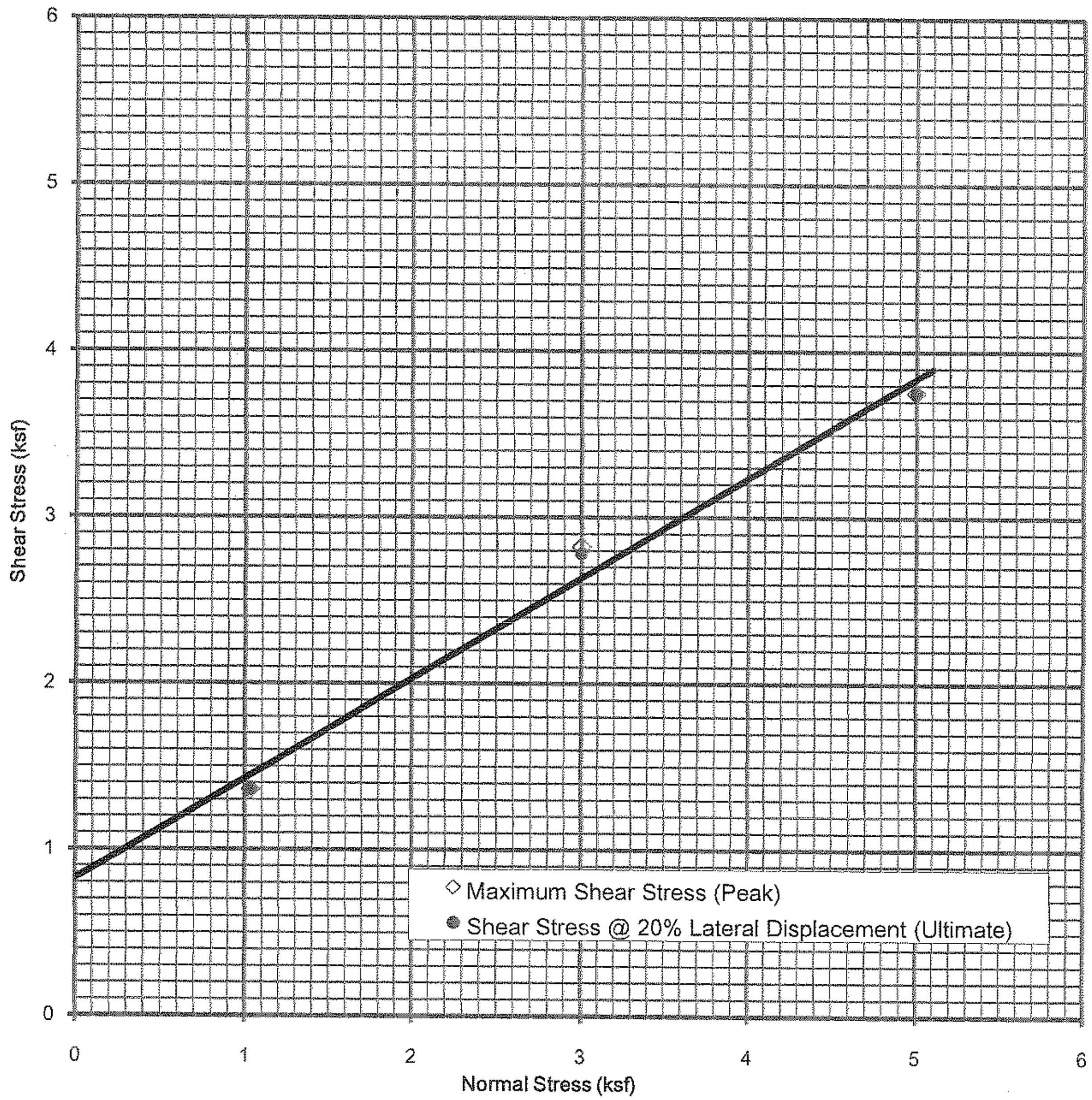
AMEC Geomatrix	DIRECT SHEAR TEST SUNSHINE CANYON LFGTE FACILITY Sylmar, California	Project No. 0148280000

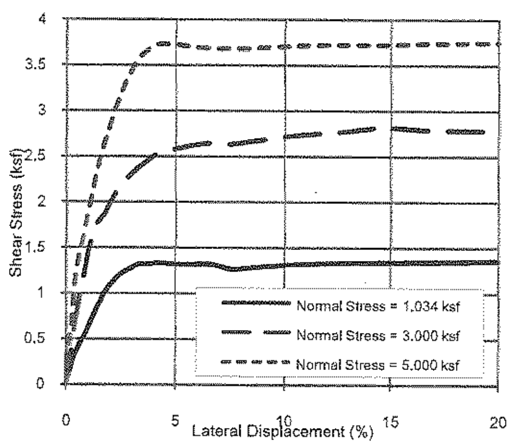
DIRECT SHEAR TEST
(ASTM-D3080)

Project Name:	Sunshine Canyon LFGTE Facility			**Project No.:**	0148280000		
Boring No.:	B-6	**Sample No.:**	9	**Depth:**	23.5-24.0 Feet	**Date:**	7/26-7/28/10
Soil Description:		Sandy Lean Clay (CL)				**Tested By:**	LT

				Before Test	After Test Load 1	After Test Load 2	After Test Load 3
Sample Diameter, in:	2.416	Weight of Wet Soil & Ring, gr:		586.02	---	---	---
Normal Stress, ksf:	1,3,5	Weight of Ring, gr:		133.61	---	---	---
Over-burdened @, pcf:		Height of Sample, in:		3.00	0.9863	0.9797	0.9568
Shear Rate, in/min:	0.005	Moisture-	Tare No.:	MC-52	---	---	---
Natural Moisture(x):			Wet Weight and Tare, gr:	176.99	157.90	155.66	154.32
Saturated(x):	X		Dry Weight and Tare, gr:	163.04	133.62	132.17	132.44
Intact(x):	X		Tare Weight, gr:	50.43	0.00	0.00	0.00
Remolded to, pcf:			Moisture Content, %:	12.4	18.2	17.8	16.5
@, %:		Wet Density, pcf:		125.3	133.6	134.0	135.8
Notes:		Dry Density, pcf:		111.5	113.1	113.8	116.5
		Saturation %:	S.G. = 2.70 (Assumed)	65.4	99.9	99.8	99.9

Load 1 (KSF): 1.034				Load 2 (KSF): 3.000				Load 3 (KSF): 5.000			
Shear Deflec-tion (in)	Lateral Displace-ment (%)	Load Ring Reading	Shear Stress (KSF)	Shear Deflec-tion (in)	Lateral Displace-ment (%)	Load Ring Reading	Shear Stress (KSF)	Shear Deflec-tion (in)	Lateral Displace-ment (%)	Load Ring Reading	Shear Stress (KSF)
0.0098	0.406	0.0022	0.352	0.0098	0.406	0.0048	0.697	0.0098	0.406	0.0081	1.136
0.0199	0.823	0.0037	0.551	0.0199	0.823	0.0089	1.242	0.0199	0.823	0.0124	1.707
0.0300	1.241	0.0054	0.777	0.0300	1.241	0.0124	1.707	0.0300	1.241	0.0160	2.185
0.0401	1.659	0.0069	0.976	0.0401	1.659	0.0135	1.853	0.0401	1.659	0.0191	2.597
0.0502	2.077	0.0080	1.122	0.0502	2.077	0.0149	2.039	0.0502	2.077	0.0216	2.929
0.0603	2.495	0.0087	1.215	0.0603	2.495	0.0160	2.185	0.0603	2.495	0.0237	3.208
0.0704	2.912	0.0091	1.268	0.0704	2.912	0.0168	2.291	0.0704	2.912	0.0252	3.407
0.0805	3.330	0.0095	1.322	0.0805	3.330	0.0175	2.384	0.0805	3.330	0.0265	3.580
0.0905	3.748	0.0095	1.322	0.0905	3.748	0.0181	2.464	0.0905	3.748	0.0272	3.673
0.1006	4.166	0.0096	1.335	0.1006	4.166	0.0186	2.531	0.1006	4.166	0.0276	3.726
0.1208	5.002	0.0095	1.322	0.1208	5.002	0.0190	2.584	0.1208	5.002	0.0276	3.726
0.1410	5.837	0.0095	1.322	0.1410	5.837	0.0193	2.624	0.1410	5.837	0.0274	3.700
0.1612	6.673	0.0095	1.322	0.1612	6.673	0.0195	2.650	0.1612	6.673	0.0273	3.686
0.1814	7.509	0.0091	1.268	0.1814	7.509	0.0194	2.637	0.1814	7.509	0.0273	3.686
0.2016	8.344	0.0092	1.282	0.2016	8.344	0.0196	2.663	0.2016	8.344	0.0273	3.686
0.2521	10.433	0.0095	1.322	0.2521	10.433	0.0201	2.730	0.2521	10.433	0.0275	3.713
0.3025	12.523	0.0096	1.335	0.3025	12.523	0.0204	2.770	0.3025	12.523	0.0276	3.726
0.3530	14.612	0.0097	1.348	0.3530	14.612	0.0208	2.823	0.3530	14.612	0.0276	3.726
0.4035	16.701	0.0097	1.348	0.4035	16.701	0.0206	2.796	0.4035	16.701	0.0277	3.740
0.4828	19.982	0.0098	1.361	0.4828	19.982	0.0205	2.783	0.4828	19.982	0.0278	3.753
Max. Shear Stress, ksf:			1.361				2.823				3.753
Shear Deflt.@Max Stress,%.:			20.0				14.6				20.0





Boring-Sample No.:	B-6-9
Sample Depth :	23.5-24.0 Feet
Soil Type:	CL
Sample Conditions:	Intact; Saturated
In-Place Dry Density (PCF):	111.5
In-Place Moisture Content (%):	12.4

	◇	●
Cohesion (PSF):	831	818
Friction Angle (Degrees):	31	31

AMEC Geomatrix	DIRECT SHEAR TEST SUNSHINE CANYON LFGTE FACILITY Sylmar, California	Project No. 0148280000

AP Engineering & Testing, Inc.

DIRECT SHEAR TEST DATA
ASTM D 3080

Project Name: Sunshine Canyon LFGTE Facility Tested By: KM Date: 8/4/2010
Sample ID: B-4 Checked By: AP Date: 8/9/2010
Sample No.: - Depth (ft): 2.5-4
Description: Brown Sandy Silt w/siltstone
Sample Type: Mod. Cal.
Test Condition: Inundated

Sample Diameter (in)	2.415
Sample Height (in)	1.00
Total Soil+Ring Weight(g)	775.69
Total Ring Weight (g)	171.54
Wet Density (pcf)	125.61
Dry Density (pcf)	109.00

Moisture Determination	Before Test	After Test
Cont. Weight (g)	50.29	104.71
Wet Soil+Cont. (g)	216.34	710.55
Dry Soil+Cont. (g)	194.38	610.32
Moisture Content (%)	15.2	19.8
Degree Saturation	75.3	97.9

METHOD OF SHEARING

[X] Regular Shearing
[ ] Residual Shearing 5 Passes

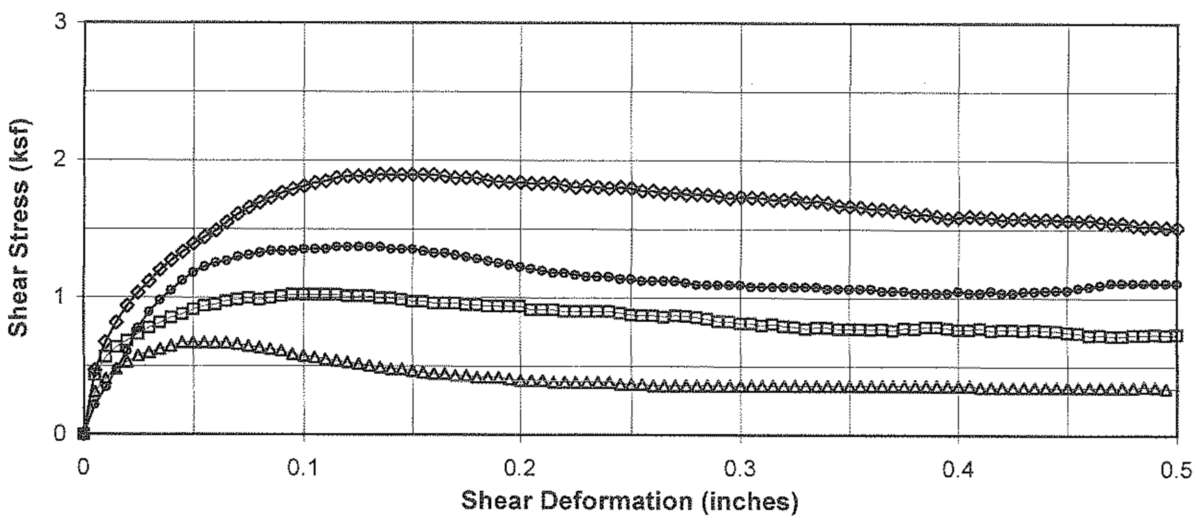
Shear Rate (in/min): 0.01
Shear Distance (in): 0.5

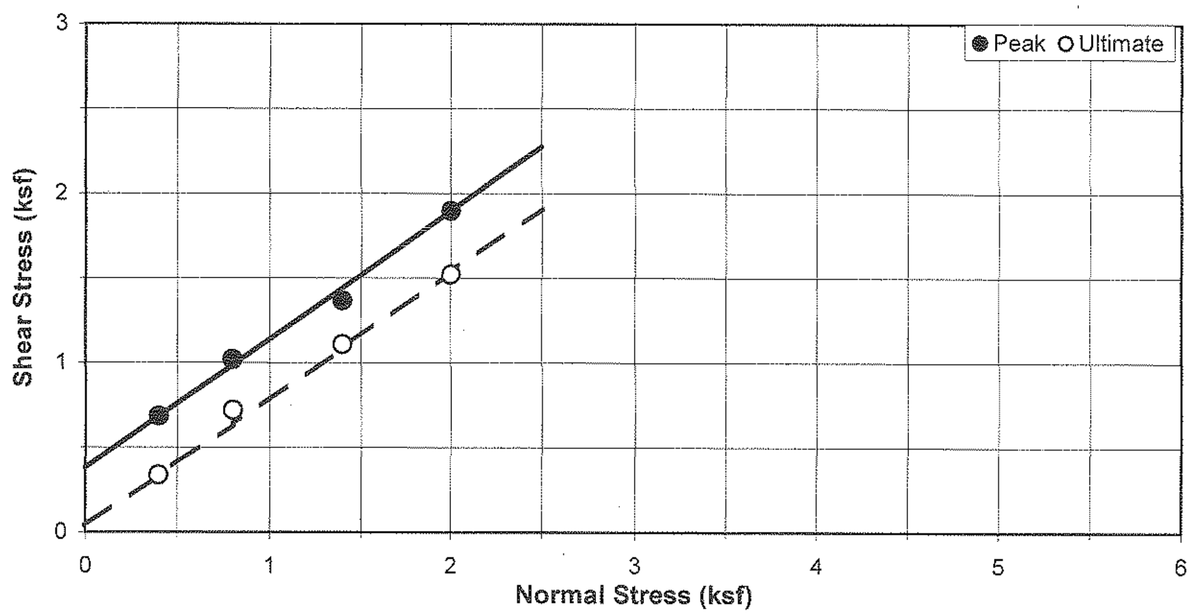
Sample Number	Sample + Ring Wt.	Ring Wt.	Normal Load (ksf)	Max. Shear Reading (psf)	Ultimate Shear Reading (psf)	Remarks
1	193.38	41.98	0.4	684	337	
2	194.50	44.07	0.8	1020	720	
3	193.61	42.74	1.4	1368	1109	
4	194.20	42.75	2.0	1896	1520	

AP Engineering & Testing, Inc.

DIRECT SHEAR TEST RESULTS
ASTM D 3080

Project Name:	Sunshine Canyon LFGTE Facility	Initial Dry Density:	109.0	pcf
Sample ID:	B-4	Moisture Content (before):	15.2	%
Sample No.:	-	Moisture Content (after):	19.8	%
Depth (ft):	2.5-4			
Sample Type:	Mod. Cal.			
Soil Description:	Brown Sandy Silt w/siltstone			
Test Condition:	Inundated			





Strength Parameters	Peak	Ultimate
Cohesion (psf):	400	50
Friction Angle:	37 °	37 °

AP Engineering & Testing, Inc.

DIRECT SHEAR TEST DATA
ASTM D 3080

Project Name: Sunshine Canyon LFGTE Facility Tested By KM Date: 08/03/10

Boring No.: CH01 Checked By AP Date: 08/09/10

Sample No.: - Depth (ft): 14-15

Description: Olive Brown Clay with small siltstones

Sample Type: Mod. Cal.

Test Condition: Inundated

Sample Diameter (in)	2.415
Sample Height (in)	1.00
Total Soil+Ring Weight(g)	578.99
Total Ring Weight (g)	127.03
Wet Density (pcf)	125.29
Dry Density (pcf)	105.62

Moisture Determination	Before Test	After Test
Cont. Weight (g)	50.51	105.27
Wet Soil+Cont. (g)	164.94	567.81
Dry Soil+Cont. (g)	146.98	479.68
Moisture Content (%)	18.6	23.5
Degree Saturation	84.4	105.4

METHOD OF SHEARING

[X] Regular Shearing

[ ] Residual Shearing 5 Passes

Shear Rate (in/min): 0.01

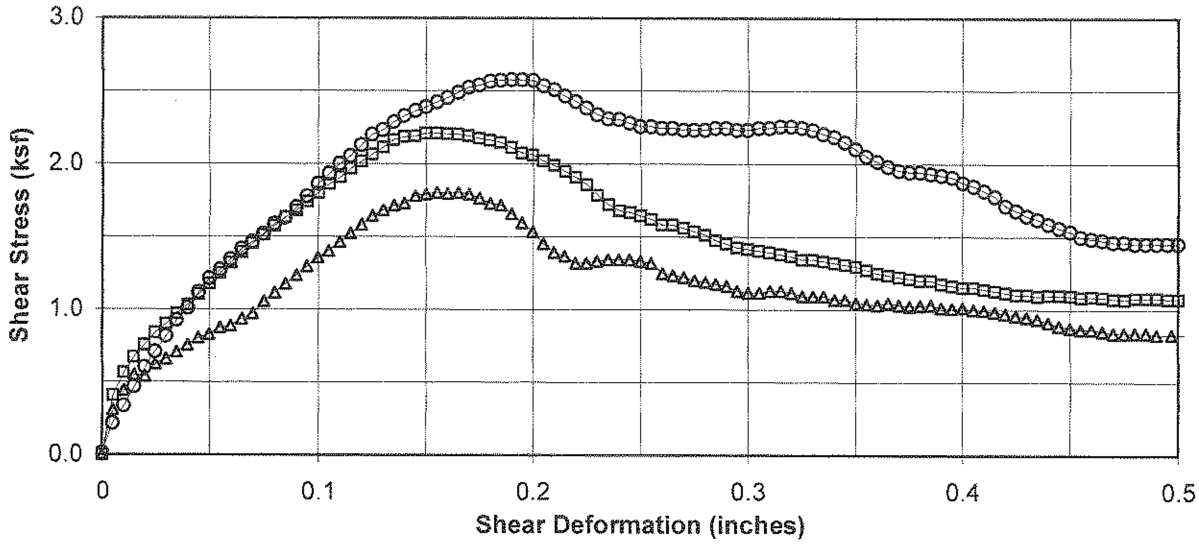
Shear Distance (in): 0.5

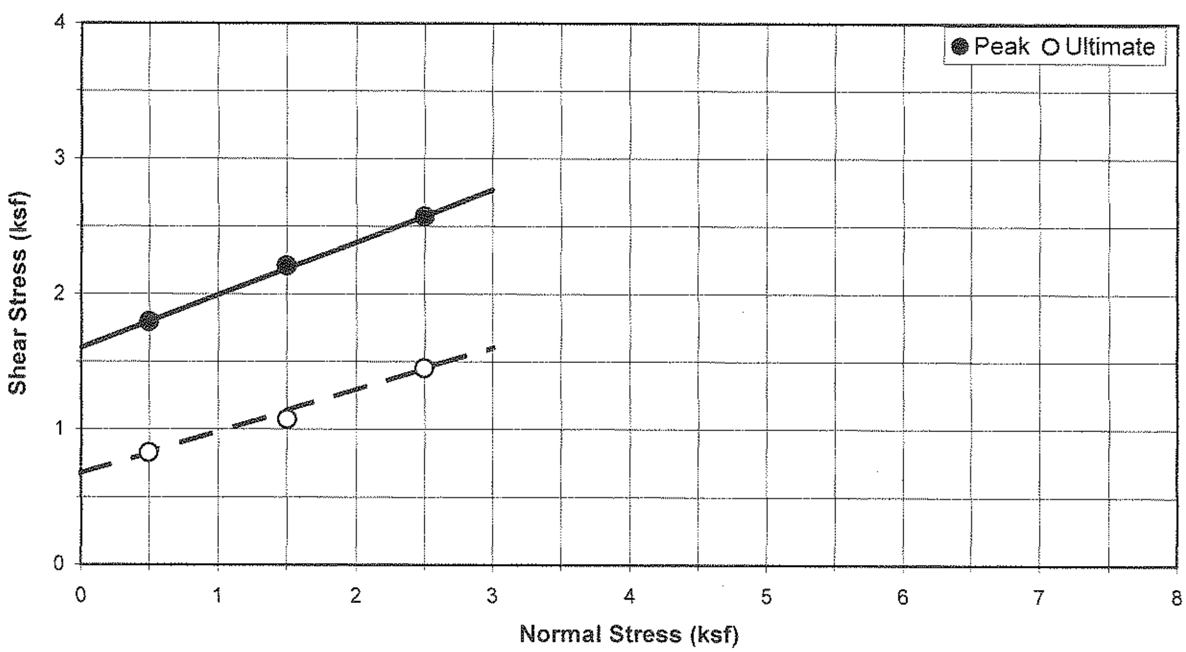
Sample Number	Sample + Ring Wt.	Ring Wt.	Normal Load (ksf)	Max. Shear Reading (psf)	Ultimate Shear Reading (psf)	Remarks
1	193.66	42.19	0.5	1796	828	
2	190.81	41.40	1.5	2209	1074	
3	194.52	43.44	2.5	2575	1452	

AP Engineering & Testing, Inc.

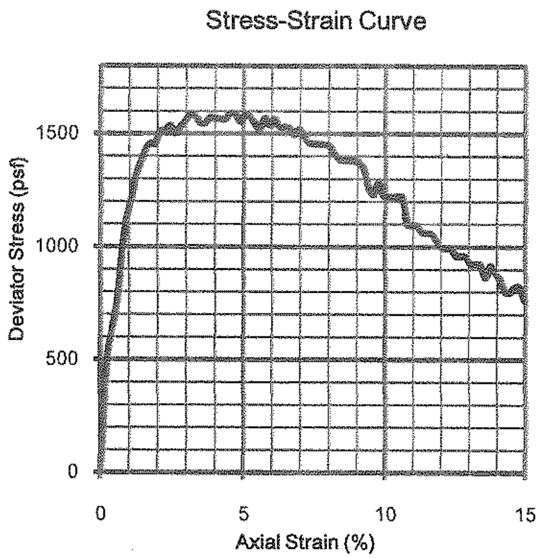
DIRECT SHEAR TEST RESULTS
ASTM D 3080

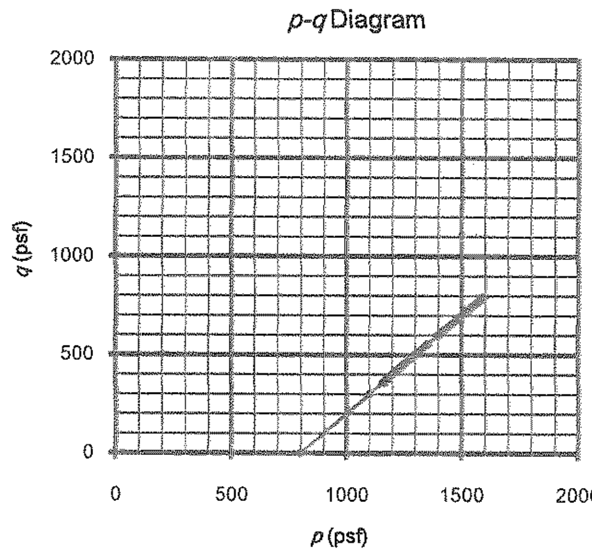
Project Name:	Sunshine Canyon LFGTE Facility	Initial Dry Density:	105.6	pcf
Boring No.:	CH01	Moisture Content (before):	18.6	%
Sample No.:	-	Moisture Content (after):	23.5	%
Depth (ft):	14-15			
Sample Type:	Mod. Cal.			
Soil Description:	Olive Brown Clay with small siltstones			
Test Condition:	Inundated			

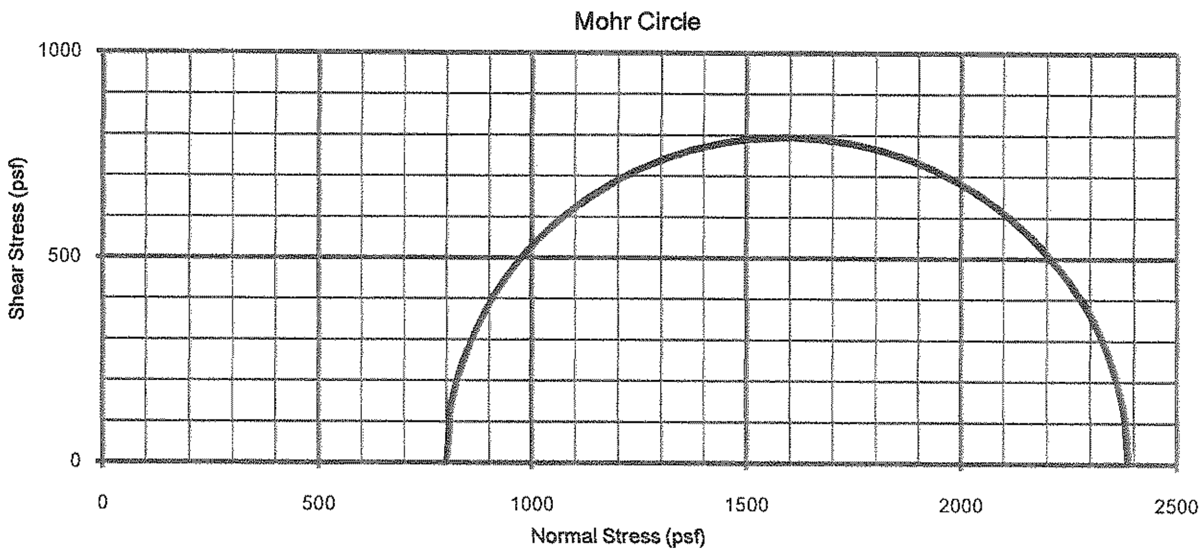




Strength Parameters	Peak	Ultimate
Cohesion (psf):	1600	650
Friction Angle:	21 °	17 °



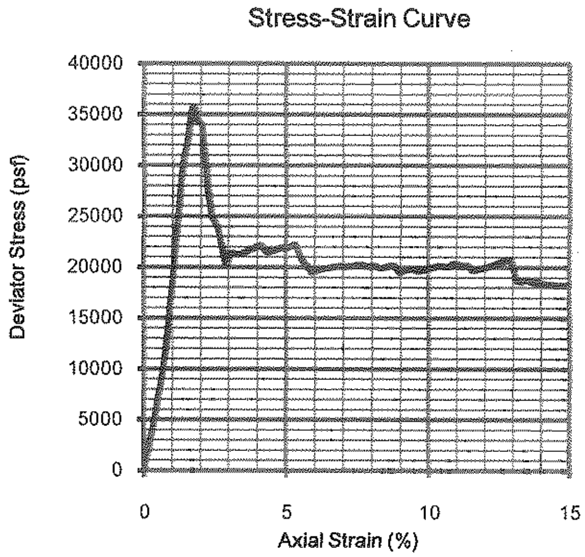


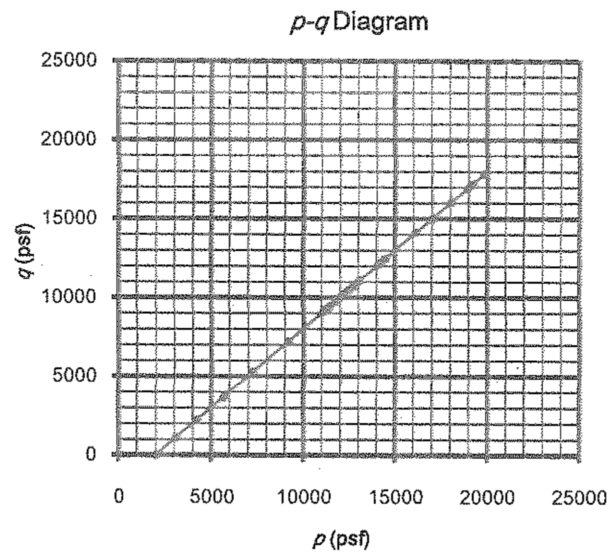


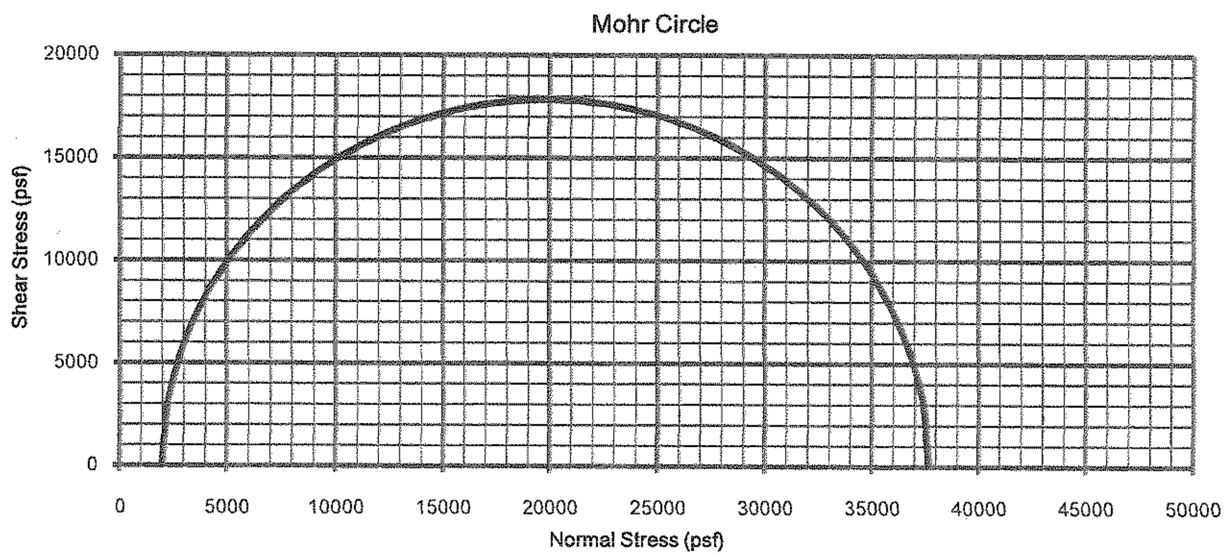
Sample Data	
Boring-Sample No.:	CH-01-1
Depth:	9.7-10.4 Feet
Soil Type:	Bedrock-CL
Height (in)	6.035
Diameter (in)	2.462
Dry Unit Weight (pcf)	109.0
Moisture Content (%)	16.8
Saturation (%)	83.4
Void Ratio	0.55

Test Data	
Cell Pressure (psi)	5.55
Strain Rate (%/min)	0.01811
Maximum Deviator Stress (psi)	11.03
Maximum Deviator Stress (psf)	1,588
Axial Strain @ Max. Deviator Stress (%)	4.52
Principal Stresses @ Failure (psf)	
σ_1 =	2,388
σ_3 =	799
Remarks: Undisturbed; Bulge at Failure	

AMEC Geomatrix	UNCONSOLIDATED-UNDRAINED TRIAXIAL COMPRESSION TEST SUNSHINE CANYON LFGTE FACILITY Sylmar, California	Project No. 0148280000





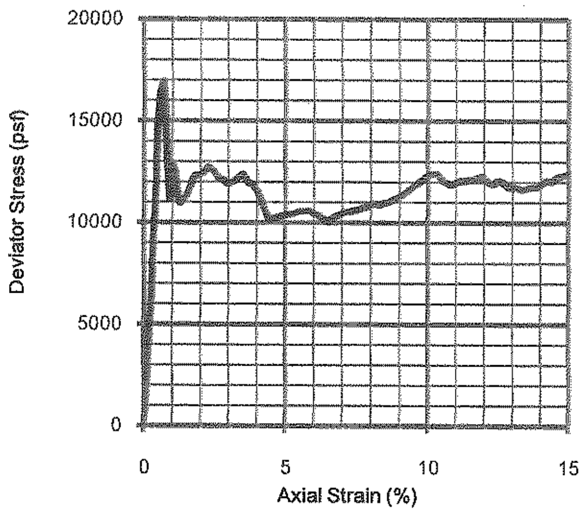


Sample Data	
Boring-Sample No.:	CH-01-2
Depth:	31.4-32.2 Feet
Soil Type:	Bedrock-ML
Height (in)	6.080
Diameter (in)	2.402
Dry Unit Weight (pcf)	118.0
Moisture Content (%)	12.7
Saturation (%)	80.3
Void Ratio	0.43

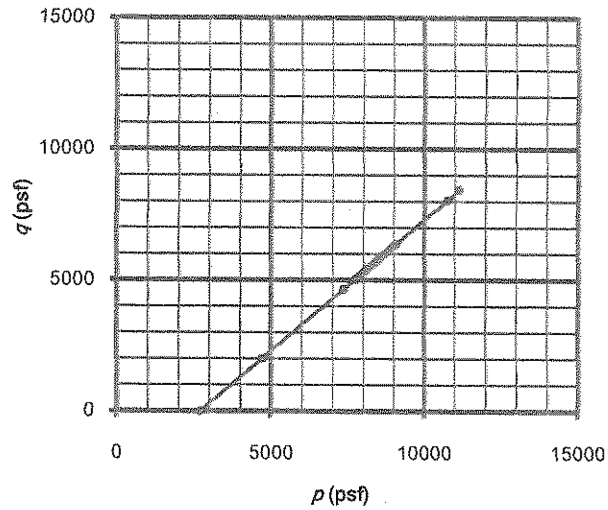
Test Data	
Cell Pressure (psi)	13.92
Strain Rate (%/min)	0.01824
Maximum Deviator Stress (psi)	247.66
Maximum Deviator Stress (psf)	35,663
Axial Strain @ Max. Deviator Stress (%)	1.73
Principal Stresses @ Failure (psf)	
σ_1 =	37,668
σ_3 =	2,004
Remarks: Undisturbed; Diagonal Shear @ ~80°	

AMEC Geomatrix	UNCONSOLIDATED-UNDRAINED TRIAXIAL COMPRESSION TEST SUNSHINE CANYON LFGTE FACILITY Sylmar, California	Project No. 0148280000

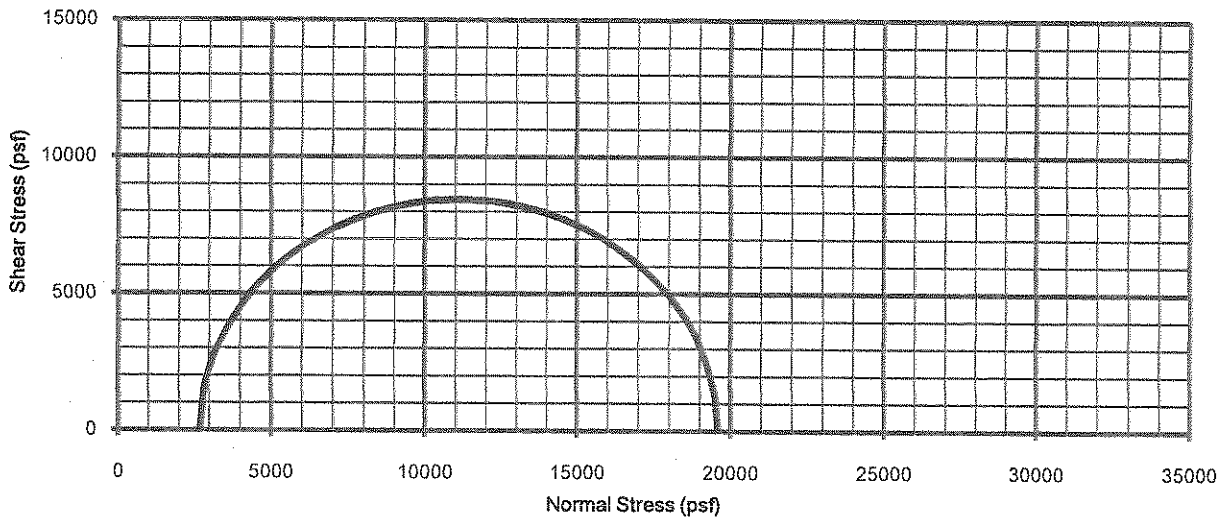
Stress-Strain Curve



p-q Diagram



Mohr Circle

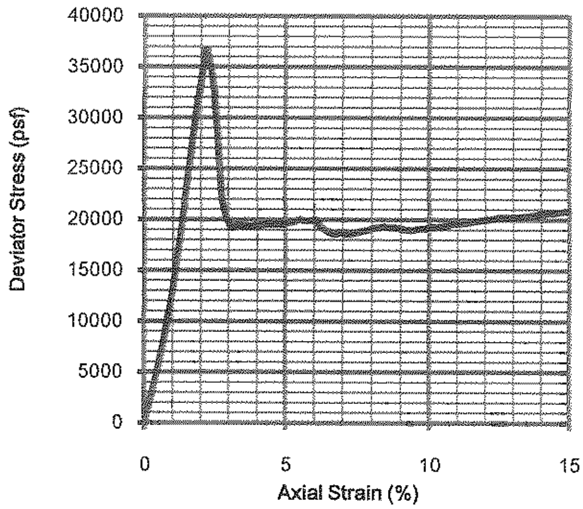


Sample Data	
Boring-Sample No.:	CH-01-3
Depth:	34.0-34.8 Feet
Soil Type:	Bedrock-ML
Height (in)	5.988
Diameter (in)	2.408
Dry Unit Weight (pcf)	117.5
Moisture Content (%)	14.2
Saturation (%)	88.3
Void Ratio	0.43

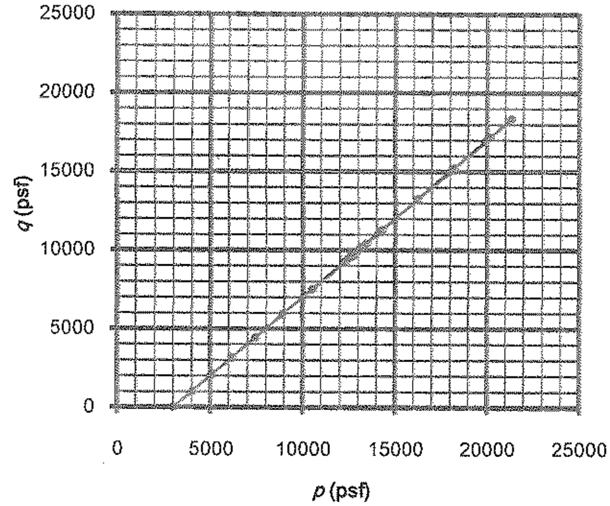
Test Data	
Cell Pressure (psi)	18.73
Strain Rate (%/min)	0.01796
Maximum Deviator Stress (psi)	117.31
Maximum Deviator Stress (psf)	16,893
Axial Strain @ Max. Deviator Stress (%)	0.70
Principal Stresses @ Failure (psf)	
σ_1 =	19,590
σ_3 =	2,697
Remarks: Undisturbed; Diagonal Shear @ ~65°	

AMEC Geomatrix	UNCONSOLIDATED-UNDRAINED TRIAXIAL COMPRESSION TEST SUNSHINE CANYON LFGTE FACILITY Sylmar, California	Project No. 0148280000

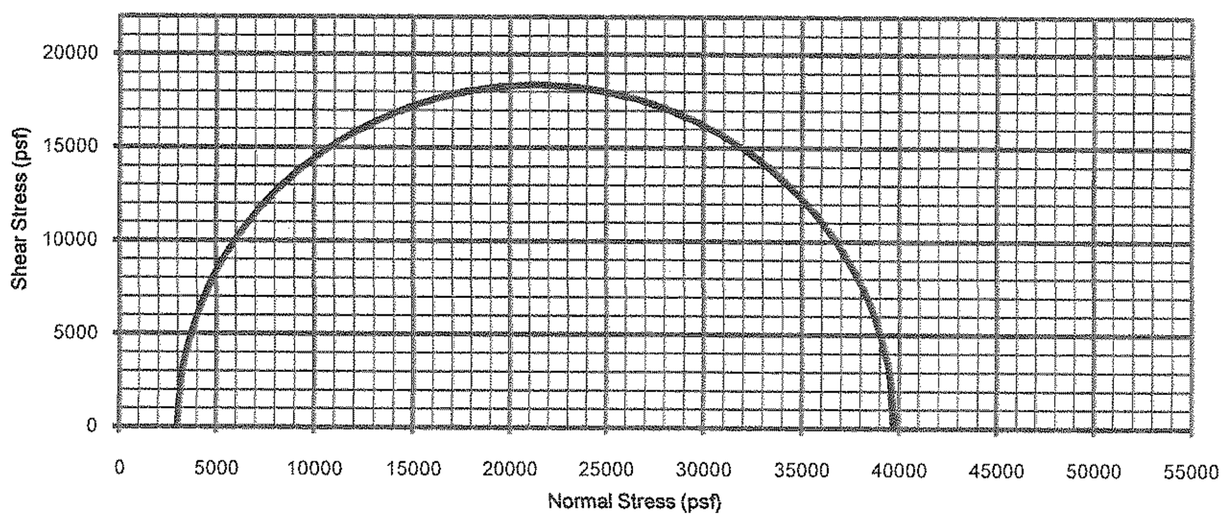
Stress-Strain Curve



p-q Diagram



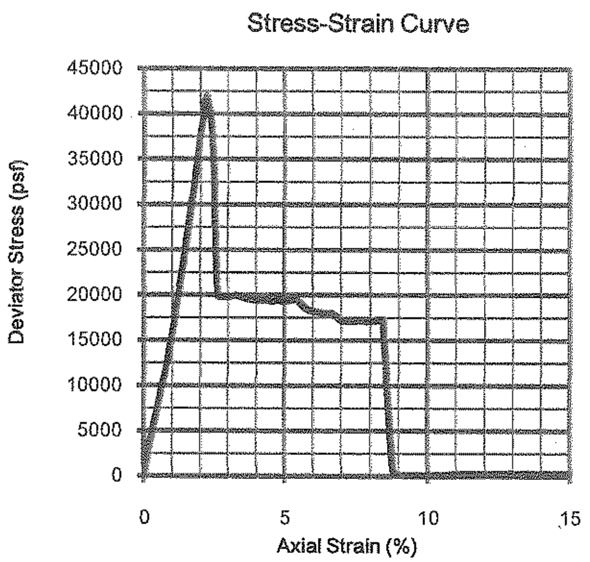
Mohr Circle

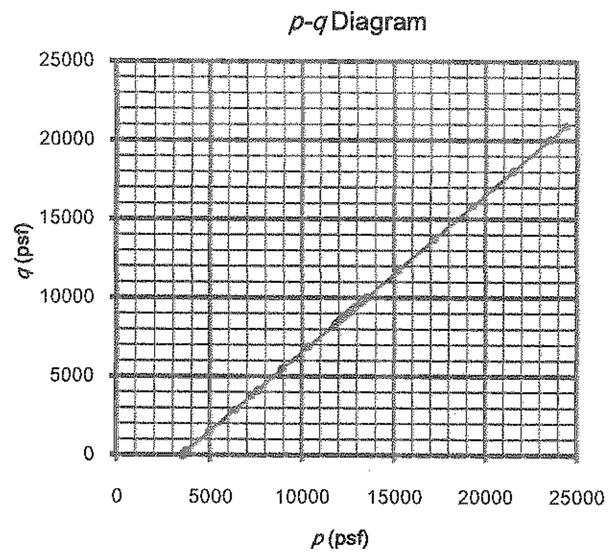


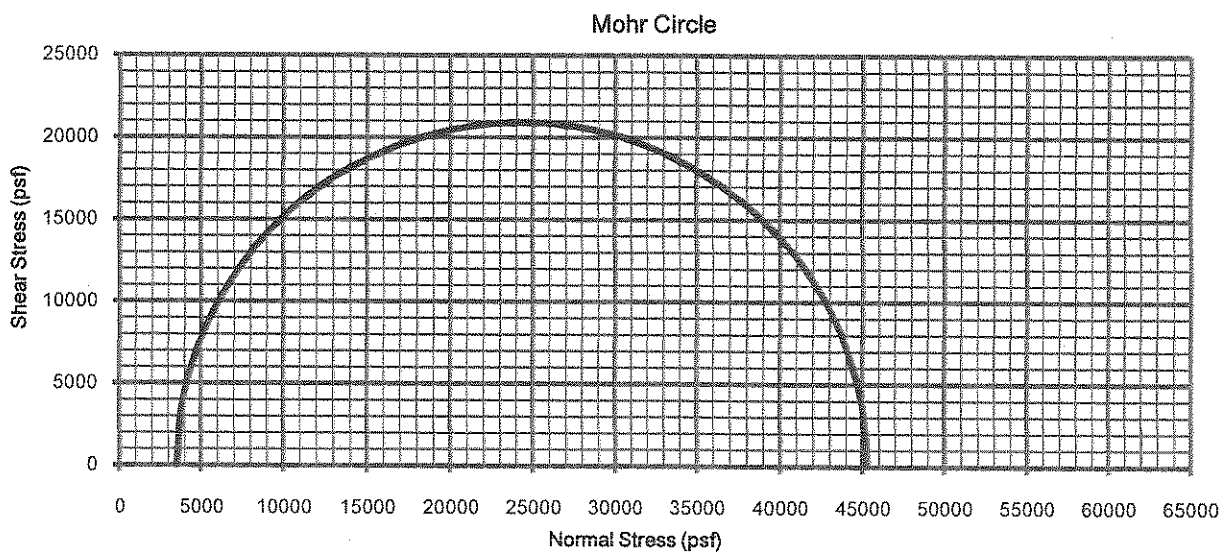
Sample Data	
Boring-Sample No.:	CH-01-4
Depth:	38.0-38.5 Feet
Soil Type:	Bedrock-ML
Height (in)	5.735
Diameter (in)	2.395
Dry Unit Weight (pcf)	122.4
Moisture Content (%)	13.5
Saturation (%)	97.1
Void Ratio	0.38

Test Data	
Cell Pressure (psi)	20.84
Strain Rate (%/min)	0.01721
Maximum Deviator Stress (psi)	255.09
Maximum Deviator Stress (psf)	36,733
Axial Strain @ Max. Deviator Stress (%)	2.20
Principal Stresses @ Failure (psf)	
σ_1 =	39,734
σ_3 =	3,001
Remarks: Undisturbed; Diagonal Shear @ ~70°	

AMEC Geomatrix	UNCONSOLIDATED-UNDRAINED TRIAXIAL COMPRESSION TEST SUNSHINE CANYON LFGTE FACILITY Sylmar, California	Project No. 0148280000



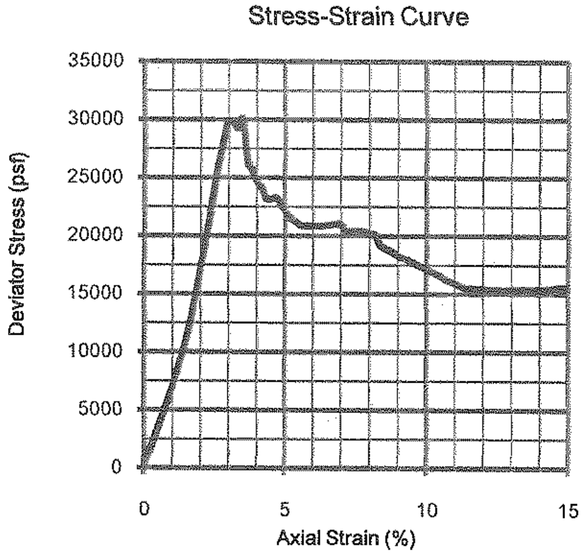


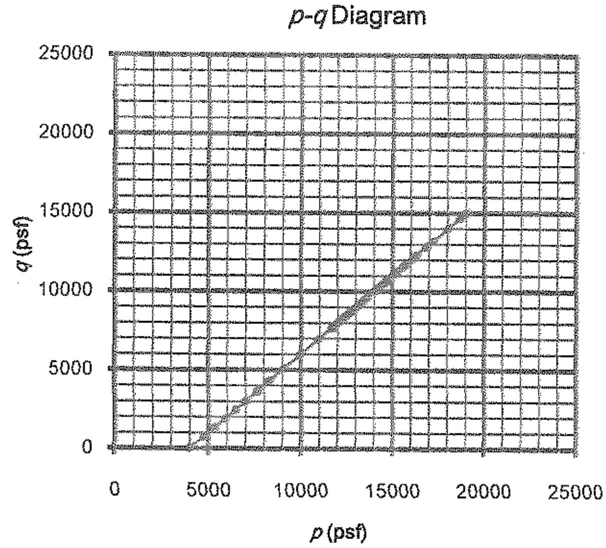


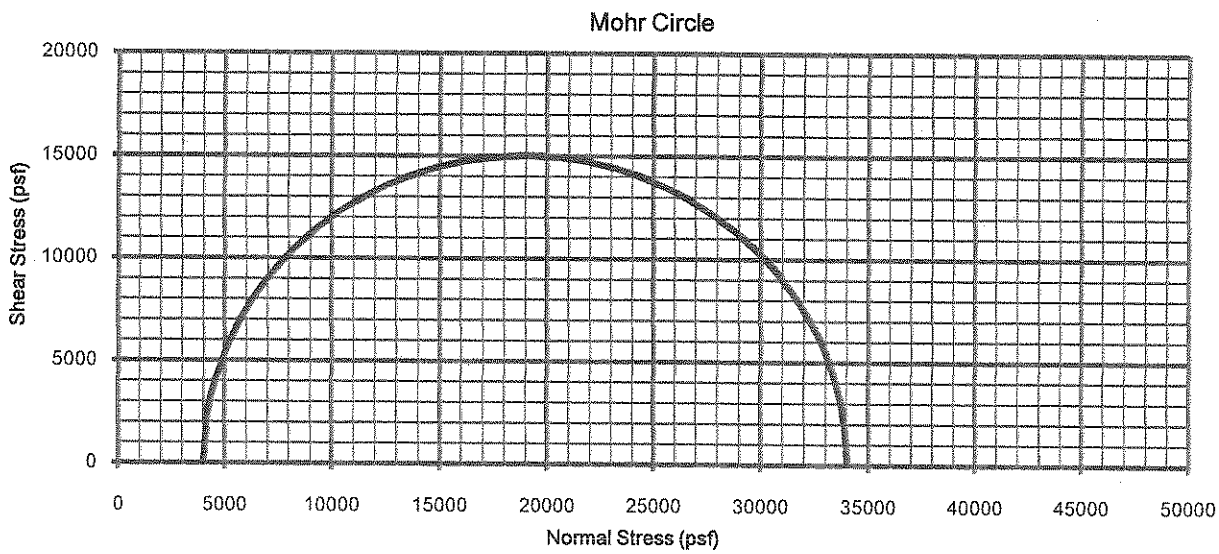
Sample Data	
Boring-Sample No.:	CH-01-5
Depth:	43.3-44.0 Feet
Soil Type:	Bedrock-ML
Height (in)	5.970
Diameter (in)	2.405
Dry Unit Weight (pcf)	126.2
Moisture Content (%)	11.4
Saturation (%)	91.6
Void Ratio	0.34

Test Data	
Cell Pressure (psi)	24.29
Strain Rate (%/min)	0.01791
Maximum Deviator Stress (psi)	290.41
Maximum Deviator Stress (psf)	41,819
Axial Strain @ Max. Deviator Stress (%)	2.28
Principal Stresses @ Failure (psf)	
σ_1 =	45,317
σ_3 =	3,498
Remarks: Undisturbed; Diagonal Shear @ ~65°	

AMEC Geomatrix	UNCONSOLIDATED-UNDRAINED TRIAXIAL COMPRESSION TEST SUNSHINE CANYON LFGTE FACILITY Sylmar, California	Project No. 0148280000







Sample Data	
Boring-Sample No.:	CH-01-6
Depth:	49.8-50.3 Feet
Soil Type:	Bedrock-ML
Height (in)	6.015
Diameter (in)	2.401
Dry Unit Weight (pcf)	123.9
Moisture Content (%)	13.4
Saturation (%)	100.3
Void Ratio	0.36

Test Data	
Cell Pressure (psi)	27.85
Strain Rate (%/min)	0.01805
Maximum Deviator Stress (psi)	208.46
Maximum Deviator Stress (psf)	30,018
Axial Strain @ Max. Deviator Stress (%)	3.49
Principal Stresses @ Failure (psf)	
σ_1 =	34,029
σ_3 =	4,010
Remarks: Undisturbed; Diagonal Shear @ ~55°	

AMEC Geomatrix	UNCONSOLIDATED-UNDRAINED TRIAXIAL COMPRESSION TEST SUNSHINE CANYON LFGTE FACILITY Sylmar, California	Project No. 0148280000

AMEC Geomatrix

UNCONFINED COMPRESSIVE STRENGTH TEST

(ASTM-D2166)

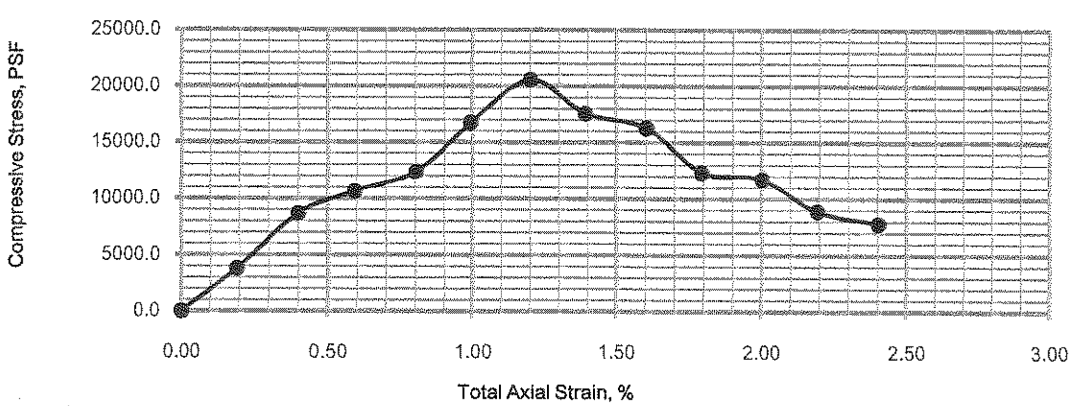
Project Name:	Sunshine Canyon LFGTE Facility			Project No.:	0148280000
Boring No.:	CH-01	Sample No.:	7	Depth:	36.0-36.5 Feet
Soil Description:	Bedrock Core: Very Dark Grayish Brown (2.5Y, 3/2) Sandy Silt (ML)				
Date:	7/30/2010			By:	L.T.

Initial Diameter, in:	2.412
Initial Area, in^2:	4.569
Initial Height, in:	5.243
Height-to-Diameter Ratio:	2.17
Type of Sample:	Undisturbed
Strain Rate, % / minute:	0.5

Note:
Moisture content specimen was obtained after test.

Wet Weight of Sample, grs:	829.42
Moisture Content-	
Tare No.:	MC-24
Wet Weight&Tare, grs:	179.77
Dry Weight & Tare, grs:	164.75
Tare Weight, grs:	50.62
Moisture Content, %:	13.2
Wet Density, pcf:	131.9
Dry Density, pcf:	116.6

Elapsed Time	Axial Load, Pounds	Strain Dial Reading, in	Total Strain, %	Corrected Area, in^2	Compressive Stress, PSF	Remarks
0:00:00	0.0	0.000	0.00	4.569	0.0	
	120.0	0.010	0.19	4.578	3774.6	
	277.0	0.021	0.40	4.588	8694.7	
	340.0	0.031	0.59	4.596	10651.8	
	395.0	0.042	0.80	4.606	12348.7	
	536.0	0.052	0.99	4.615	16724.5	
	660.0	0.063	1.20	4.625	20550.0	
	565.0	0.073	1.39	4.634	17558.1	
	525.0	0.084	1.60	4.644	16280.3	
	398.0	0.094	1.79	4.653	12318.1	Diagonal
	380.0	0.105	2.00	4.663	11735.9	Shear
	289.0	0.115	2.19	4.672	8908.1	at ~67°
	254.0	0.126	2.40	4.682	7812.5	
0:04:48	253.0	0.126	2.40	4.682	7781.7	



Unconfined Compressive Strength, PSF = 20,550

Shear Strength, PSF = 10,275

UNCONFINED COMPRESSIVE STRENGTH TEST
(ASTM-D2166)

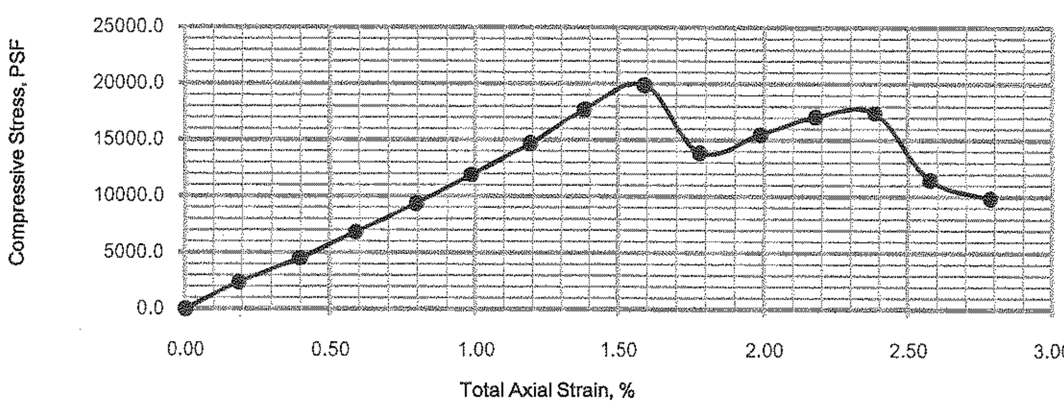
Project Name:	Sunshine Canyon LFGTE Facility	Project No.:	0148280000
Boring No.:	CH-01 Sample No.: 8	Depth:	42.0-42.5 Feet
Soil Description:	Bedrock Core: Very Dark Gray (2.5Y, 3/1) Silt with Sand (ML)		
Date:	7/30/2010	By:	L.T.

Initial Diameter, in:	2.425
Initial Area, in^2:	4.619
Initial Height, in:	5.282
Height-to-Diameter Ratio:	2.18
Type of Sample:	Undisturbed
Strain Rate, % / minute:	0.5

Note:
Moisture content specimen was obtained after test.

Wet Weight of Sample, grs:	888.18
Moisture Content-	
Tare No.:	MC-40
Wet Weight&Tare, grs:	185.62
Dry Weight & Tare, grs:	172.53
Tare Weight, grs:	50.31
Moisture Content, %:	10.7
Wet Density, pcf:	138.7
Dry Density, pcf:	125.3

Elapsed Time	Axial Load, Pounds	Strain Dial Reading, in	Total Strain, %	Corrected Area, in^2	Compressive Stress, PSF	Remarks
0:00:00	0.0	0.000	0.00	4.619	0.0	
	76.0	0.010	0.19	4.627	2365.0	
	145.0	0.021	0.40	4.637	4502.8	
	220.0	0.031	0.59	4.646	6818.9	
	302.0	0.042	0.80	4.656	9340.9	
	386.0	0.052	0.98	4.665	11916.3	
	477.0	0.063	1.19	4.674	14694.6	
	576.0	0.073	1.38	4.683	17710.4	
	648.0	0.084	1.59	4.693	19882.1	
	453.0	0.094	1.78	4.702	13872.3	Diagonal
	507.0	0.105	1.99	4.712	15493.0	Shears
	559.0	0.115	2.18	4.721	17049.1	at ~60°
	572.0	0.126	2.39	4.732	17408.4	and
	378.0	0.136	2.57	4.741	11481.9	~80°
0:05:34	325.0	0.147	2.78	4.751	9850.9	



Unconfined Compressive Strength, PSF = 19,882
Shear Strength, PSF = 9,941

AMEC Geomatrix

UNCONFINED COMPRESSIVE STRENGTH TEST

(ASTM-D2166)

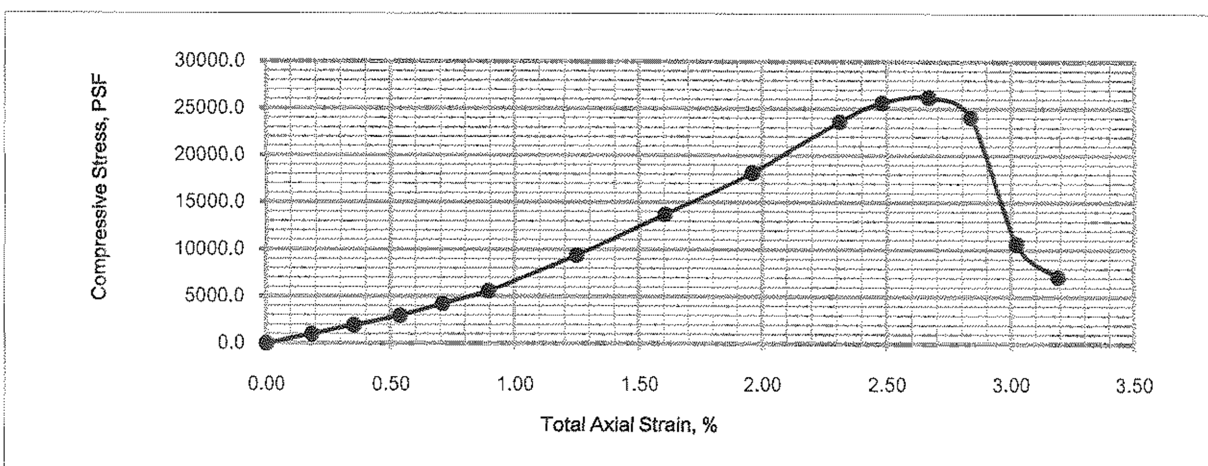
Project Name:	Sunshine Canyon LFGTE Facility	Project No.:	0148280000
Boring No.:	CH-01 Sample No.: 9	Depth:	44.0-44.5 Feet
Soil Description:	Bedrock Core: Dark Gray (2.5Y, 4/1) Sandy Silt (ML)		
Date:	7/30/2010	By:	L.T.

Initial Diameter, in:	2.418
Initial Area, in^2:	4.592
Initial Height, in:	5.922
Height-to-Diameter Ratio:	2.45
Type of Sample:	Undisturbed
Strain Rate, % / minute:	0.5

Note:
Moisture content specimen was obtained after test.

Wet Weight of Sample, grs:	985.94
Moisture Content-	
Tare No.:	MC-30
Wet Weight&Tare, grs:	424.52
Dry Weight & Tare, grs:	384.59
Tare Weight, grs:	50.50
Moisture Content, %:	12.0
Wet Density, pcf:	138.1
Dry Density, pcf:	123.4

Elapsed Time	Axial Load, Pounds	Strain Dial Reading, in	Total Strain, %	Corrected Area, in^2	Compressive Stress, PSF	Remarks
0:00:00	0.0	0.000	0.00	4.592	0.0	
	32.0	0.011	0.19	4.601	1001.6	
	62.0	0.021	0.35	4.608	1937.4	
	95.0	0.032	0.54	4.617	2963.0	
	135.0	0.042	0.71	4.625	4203.4	
	180.0	0.053	0.89	4.633	5594.1	
	302.0	0.074	1.25	4.650	9352.0	
	445.0	0.095	1.60	4.667	13730.8	
	590.0	0.116	1.96	4.684	18139.3	
	771.0	0.137	2.31	4.701	23618.3	Diagonal
	838.0	0.147	2.48	4.709	25626.4	Shear
	858.0	0.158	2.67	4.718	26188.0	at ~70°
	790.0	0.168	2.84	4.726	24070.7	and
	349.0	0.179	3.02	4.735	10613.4	Vertical
0:06:23	234.0	0.189	3.19	4.743	7103.8	Shear



Unconfined Compressive Strength, PSF = 26,188

Shear Strength, PSF = 13,094

AMEC Geomatrix

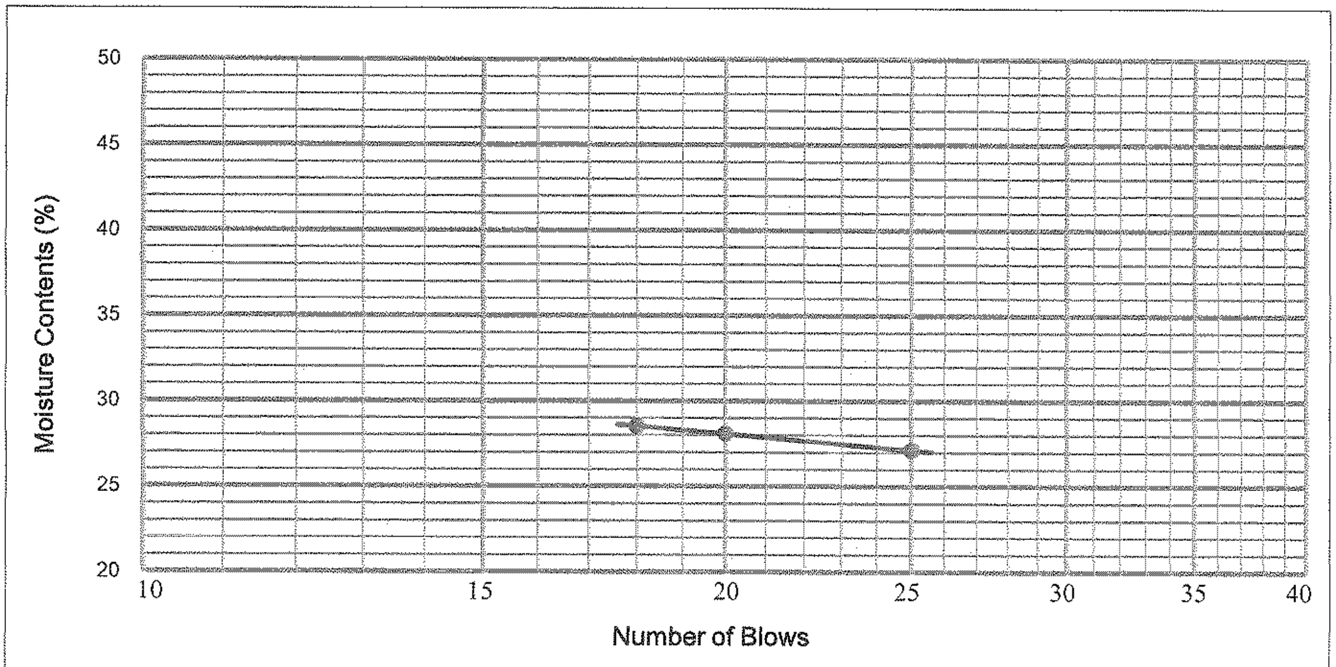
PLASTICITY INDEX
ASTM-D 4318

Project Name:	Sunshine Canyon LFGTE Facility	**Project No.:**	0148280000
Sample No.: 1	**Stockpile No.:** 2		**Date:** 10/28-11/04/10
Soil Description:	Dark Yellowish Brown (10YR, 4/4) Clayey Sand (SC)		**Tested by:** VC, LT

DRYING PAN No.: 30 SOAKING DISH No.: A-8

LIQUID LIMIT (LL)

TARE No.:	C-46	C-72	C-33
NUMBER OF BLOWS	18	20	25
WEIGHT OF WET SOIL + TARE, gr.:	33.48	34.69	36.85
WEIGHT OF DRY SOIL + TARE, gr.:	29.39	30.45	32.32
WEIGHT OF TARE, gr.:	15.05	15.36	15.62
MOISTURE CONTENT, %:	**28.5**	**28.1**	**27.1**



LIQUID LIMIT = 27

PLASTIC LIMIT (PL)

TARE No.:	C-24	C-68
WEIGHT OF WET SOIL + TARE, gr.:	29.00	27.03
WEIGHT OF DRY SOIL + TARE, gr.:	26.83	25.08
WEIGHT OF TARE, gr.:	15.14	15.48
MOISTURE CONTENT, %:	**18.6**	**20.3**

PLASTIC LIMIT = 19

PLASTICITY INDEX (PI) = LL - PL = 8

AMEC Geomatrix

PLASTICITY INDEX

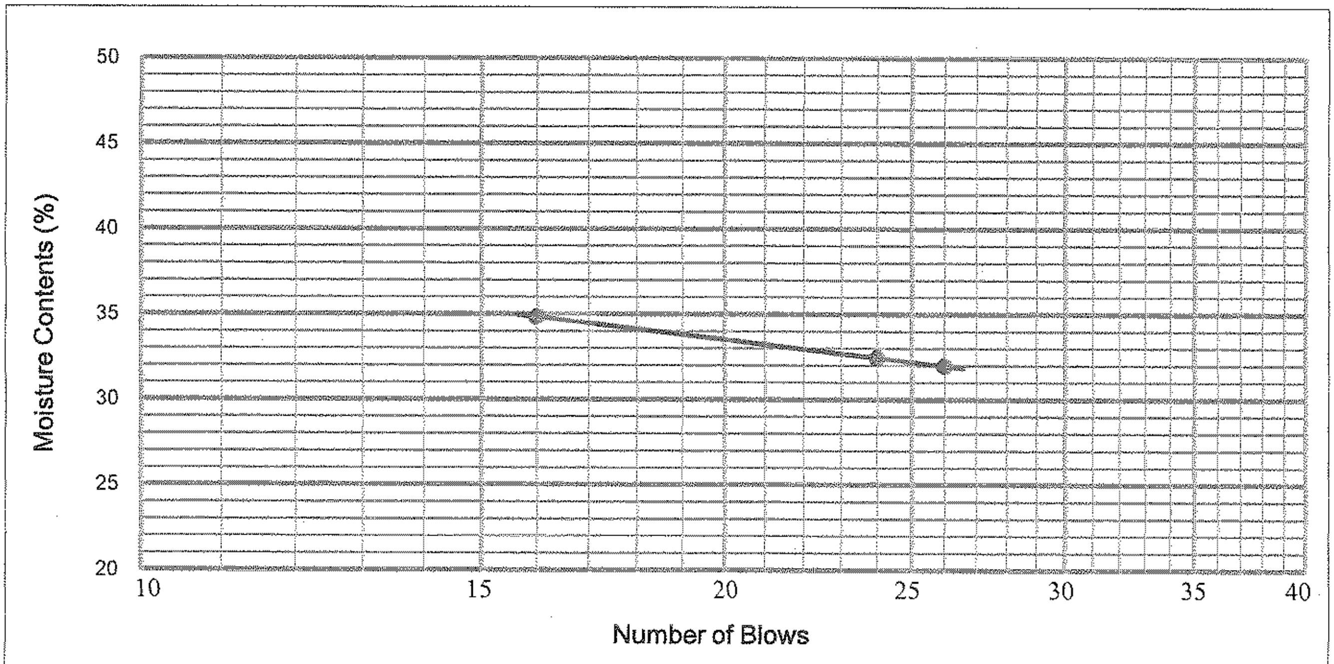
ASTM-D 4318

Project Name:	Sunshine Canyon LFGTE Facility	**Project No.:**	0148280000
Sample No.: 2	**Stockpile No.:** 2		**Date:** 10/28-11/04/10
Soil Description:	Dark Olive Gray (5Y, 3/2) Sandy Lean Clay (CL)		**Tested by:** VC, LT

DRYING PAN No.: 28 SOAKING DISH No.: A-3

LIQUID LIMIT (LL)

TARE No.:	C-41	C-63	C-39
NUMBER OF BLOWS	16	24	26
WEIGHT OF WET SOIL + TARE, gr.:	31.46	34.39	34.77
WEIGHT OF DRY SOIL + TARE, gr.:	27.19	29.80	30.14
WEIGHT OF TARE, gr.:	14.93	15.66	15.66
MOISTURE CONTENT, %:	**34.8**	**32.5**	**32.0**



LIQUID LIMIT = 32

PLASTIC LIMIT (PL)

TARE No.:	C-75	C-21
WEIGHT OF WET SOIL + TARE, gr.:	25.41	28.34
WEIGHT OF DRY SOIL + TARE, gr.:	23.80	26.23
WEIGHT OF TARE, gr.:	15.58	15.64
MOISTURE CONTENT, %:	**19.6**	**19.9**

PLASTIC LIMIT = 20

PLASTICITY INDEX (PI) = LL - PL = 12

PLASTICITY INDEX

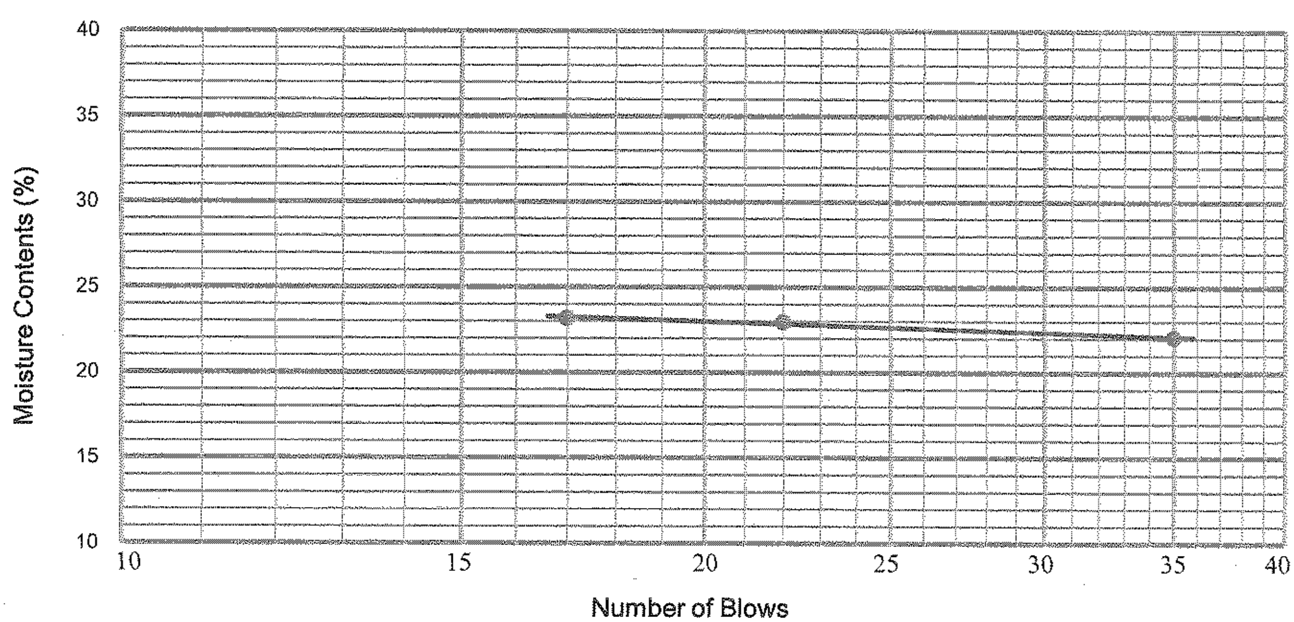
ASTM-D 4318

Project Name:	Sunshine Canyon LFGTE Facility	Project No.:	0148280000
Sample No.: 3	Stockpile No.: 1		Date: 10/28-11/04/10
Soil Description:	Olive Yellow (5Y, 6/8) Silty Sand (SM)		Tested by: VC, LT

DRYING PAN No.: 32 SOAKING DISH No.: A-11

LIQUID LIMIT (LL)

TARE No.:	C-35	C-19	C-32
NUMBER OF BLOWS	17	22	35
WEIGHT OF WET SOIL + TARE, gr.:	38.56	37.00	33.67
WEIGHT OF DRY SOIL + TARE, gr.:	34.29	33.01	30.32
WEIGHT OF TARE, gr.:	15.88	15.63	15.15
MOISTURE CONTENT, %:	**23.2**	**23.0**	**22.1**



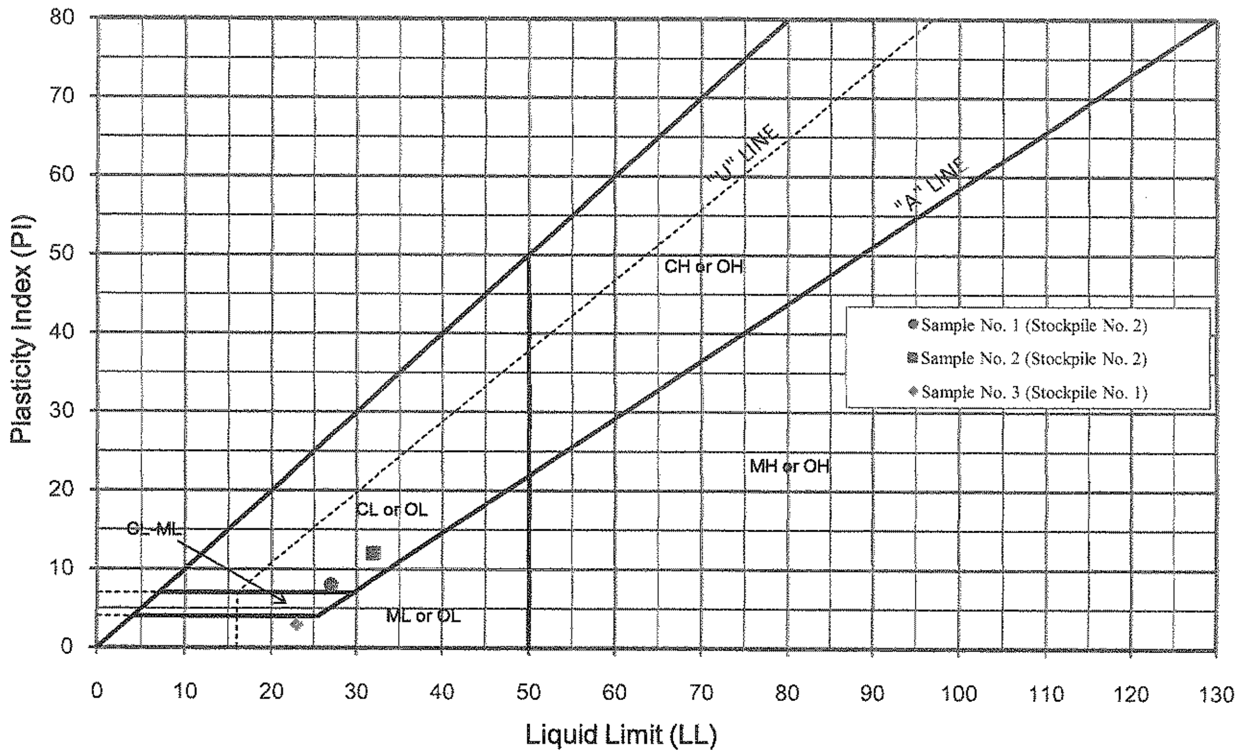
LIQUID LIMIT = 23

PLASTIC LIMIT (PL)

TARE No.:	C-36	C-80
WEIGHT OF WET SOIL + TARE, gr.:	28.6	26.43
WEIGHT OF DRY SOIL + TARE, gr.:	26.44	24.59
WEIGHT OF TARE, gr.:	15.72	15.61
MOISTURE CONTENT, %:	**20.1**	**20.5**

PLASTIC LIMIT = 20

PLASTICITY INDEX (PI) = LL - PL = 3



Sample No.	Stockpile No.	Depth (Feet)	Liquid Limit (LL)	Plastic Limit (PL)	Plasticity Index (PI)	Soil Classification
1	2	---	27	19	8	SC
2	2	---	32	20	12	CL
3	1	---	23	20	3	SM

AMEC Geomatrix	PLASTICITY INDEX (PI) SUNSHINE CANYON LFGTE FACILITY Sylmar, California	Project No. 0148280000

EXPANSION INDEX TEST

(UBC 18-2 (1994)/ASTM-D 4829)

Project Name:	Sunshine Canyon LFGTE Facility	Project No.:	0148280000
Sample No.:	C-1 Source: Excavated Bedrock Material		
Soil Description:	Dark Gray (2.5Y, 4/1) Sandy Silt (ML)		
		Date:	2/16-2/18/2011 By: LT

WET DENSITY CALCULATION	TRIAL 1	TRIAL 2	TRIAL 3	TRIAL 4
RING No.	1			
RING AND WET SOIL, gr.	597.82			
WEIGHT OF RING, gr.	198.58			
WEIGHT OF WET SOIL, gr.	399.24			
WET DENSITY, PCF.	121.0			
MOISTURE CALCULATION				
TARE No.	1			
WET SOIL AND TARE, gr.	407.09			
DRY SOIL AND TARE, gr.	376.36			
TARE WEIGHT, gr.	97.68			
MOISTURE CONTENT, %	11.0			
DRY DENSITY, PCF.	109.0			
SATURATION DEGREE (S_{meas}), % *	54.58			

EXPANSION INDEX (EI) CALCULATION

APPARATUS No.: 1

INITIAL SPECIMEN HEIGHT: 1.0000 inch

		HEIGHT CHANGE, in.	DATE	TIME
INITIAL DIAL READING, in.	0.0500	0.0000	1/17/2011	11:24
PERIODIC DIAL READING, in.	0.0758	0.0258	1/18/2011	13:24

FINAL DIAL READING, in.	0.0758	0.0258	1/18/2011	15:36

EI_{meas} = 26 **

$\boxed{EI_{50} = 28}$ ***

FINAL MOISTURE CONTENT, DRY DENSITY AND SATURATION DEGREE

TARE No.	---	MOISTURE CONTENT, %	21.7
WET SOIL AND TARE, gr.	427.13	FINAL VOLUME, cc.	211.24
DRY SOIL AND TARE, gr.	351.07	FINAL DRY DENSITY, PCF.	106.3
TARE WEIGHT, gr.	0.00	FINAL SATURATION, %	99.9

* SATURATION % = (Moist. Content in % x G_s X Dry Density in PCF) / ((G_s X 62.4)-Dry Density in PCF)

Gs = 2.7 unless it is known to be less than 2.6 or more than 2.8

** EI_{meas} (Measured Expansion Index) = (Height Change/Initial Height) X 1000

*** EI_{50} (Estimated Expansion Index at 50% Saturation) = EI_{meas}-((50-S_{meas})x((65+EI_{meas})/(220-S_{meas})))

AMEC Geomatrix

EXPANSION INDEX TEST

(UBC 18-2 (1994)/ASTM-D 4829)

Project Name:	Sunshine Canyon LFGTE Facility	Project No.:	0148280000
Sample No.:	C-2 Source:	Stockpile South of Gas to Energy Site	
Soil Description:	Very Dark Gray (2.5Y, 3/1) Sandy Lean Clay (CL)		
		Date:	3/09-3/10/2011 By: LT

WET DENSITY CALCULATION	TRIAL 1	TRIAL 2	TRIAL 3	TRIAL 4
RING No.	1			
RING AND WET SOIL, gr.	582.31			
WEIGHT OF RING, gr.	198.58			
WEIGHT OF WET SOIL, gr.	383.73			
WET DENSITY, PCF.	116.3			
MOISTURE CALCULATION				
TARE No.	3			
WET SOIL AND TARE, gr.	401.89			
DRY SOIL AND TARE, gr.	373.38			
TARE WEIGHT, gr.	98.95			
MOISTURE CONTENT, %	10.4			
DRY DENSITY, PCF.	105.4			
SATURATION DEGREE (S_{meas}), % *	46.85			

EXPANSION INDEX (EI) CALCULATION

APPARATUS No.: 1

INITIAL SPECIMEN HEIGHT: 1.0000 inch

		HEIGHT CHANGE, in.	DATE	TIME
INITIAL DIAL READING, in.	0.0500	0.0000	3/9/2011	16:33
PERIODIC DIAL READING, in.	0.0925	0.0425	3/10/2011	10:23

FINAL DIAL READING, in.	0.0925	0.0425	3/10/2011	14:20

EI_{meas} = 43 **

$\boxed{EI_{50} = 41}$ ***

FINAL MOISTURE CONTENT, DRY DENSITY AND SATURATION DEGREE

TARE No.	---	MOISTURE CONTENT, %	24.6
WET SOIL AND TARE, gr.	426.95	FINAL VOLUME, cc.	214.68
DRY SOIL AND TARE, gr.	342.70	FINAL DRY DENSITY, PCF.	101.1
TARE WEIGHT, gr.	0.00	FINAL SATURATION, %	99.6

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* SATURATION % = (Moist. Content in % x G_s X Dry Density in PCF) / ((G_s X 62.4)-Dry Density in PCF)

Gs = 2.7 unless it is known to be less than 2.6 or more than 2.8

** EI_{meas} (Measured Expansion Index) = (Height Change/Initial Height) X 1000

*** EI_{50} (Estimated Expansion Index at 50% Saturation) = EI_{meas}-((50-S_{meas})x((65+EI_{meas})/(220-S_{meas})))

EXPANSION INDEX TEST

(UBC 18-2 (1994)/ASTM-D 4829)

Project Name:	Sunshine Canyon LFGTE Facility	Project No.:	0148280000
Sample No.:	1 Stockpile No.: 2	Depth:	---
Soil Description:	Dark Yellowish Brown (10YR, 4/4) Clayey Sand (SC)		
		Date:	11/08-11/09/10 By: LT

WET DENSITY CALCULATION	TRIAL 1	TRIAL 2	TRIAL 3	TRIAL 4
RING No.	1			
RING AND WET SOIL, gr.	591.42			
WEIGHT OF RING, gr.	198.58			
WEIGHT OF WET SOIL, gr.	392.84			
WET DENSITY, PCF.	119.1			
MOISTURE CALCULATION				
TARE No.	17			
WET SOIL AND TARE, gr.	403.82			
DRY SOIL AND TARE, gr.	373.75			
TARE WEIGHT, gr.	99.59			
MOISTURE CONTENT, %	11.0			
DRY DENSITY, PCF.	107.3			
SATURATION DEGREE (S_{meas}), % *	51.97			

EXPANSION INDEX (EI) CALCULATION

APPARATUS No.:	1
INITIAL SPECIMEN HEIGHT:	1.0000 inch

		HEIGHT CHANGE, in.	DATE	TIME
INITIAL DIAL READING, in.	0.0500	0.0000	11/8/2010	14:35
PERIODIC DIAL READING, in.	0.0737	0.0237	11/9/2010	11:49

FINAL DIAL READING, in.	0.0737	0.0237	11/9/2010	13:25

EI_{meas} = 24 **

EI_{50} = **25** ***

FINAL MOISTURE CONTENT, DRY DENSITY AND SATURATION DEGREE

TARE No.	---	MOISTURE CONTENT, %	22.5
WET SOIL AND TARE, gr.	432.09	FINAL VOLUME, cc.	210.81
DRY SOIL AND TARE, gr.	352.79	FINAL DRY DENSITY, PCF.	104.8
TARE WEIGHT, gr.	0.00	FINAL SATURATION, %	100.0

* SATURATION % = (Moist. Content in % x G_s X Dry Density in PCF) / ((G_s X 62.4)-Dry Density in PCF)

Gs = 2.7 unless it is known to be less than 2.6 or more than 2.8

** EI_{meas} (Measured Expansion Index) = (Height Change/Initial Height) X 1000

*** EI_{50} (Estimated Expansion Index at 50% Saturation) = $EI_{meas}-((50-S_{meas})x((65+EI_{meas})/(220-S_{meas})))$

EXPANSION INDEX TEST

(UBC 18-2 (1994)/ASTM-D 4829)

Project Name:	Sunshine Canyon LFGTE Facility	Project No.:	0148280000
Sample No.:	2 Stockpile No.: 2	Depth:	---
Soil Description:	Dark Olive Gray (5Y, 3/2) Sandy Lean Clay (CL)		
		Date:	11/08-11/09/10 By: LT

WET DENSITY CALCULATION	TRIAL 1	TRIAL 2	TRIAL 3	TRIAL 4
RING No.	2			
RING AND WET SOIL, gr.	591.36			
WEIGHT OF RING, gr.	199.55			
WEIGHT OF WET SOIL, gr.	391.81			
WET DENSITY, PCF.	118.8			
MOISTURE CALCULATION				
TARE No.	2			
WET SOIL AND TARE, gr.	408.53			
DRY SOIL AND TARE, gr.	378.68			
TARE WEIGHT, gr.	96.82			
MOISTURE CONTENT, %	10.6			
DRY DENSITY, PCF.	107.4			
SATURATION DEGREE (S_{meas}), % *	50.29			

EXPANSION INDEX (EI) CALCULATION

APPARATUS No.:	2
INITIAL SPECIMEN HEIGHT:	1.0000 inch

		HEIGHT CHANGE, in.	DATE	TIME
INITIAL DIAL READING, in.	0.0500	0.0000	11/8/2010	15:20
PERIODIC DIAL READING, in.	0.0893	0.0393	11/9/2010	11:49

FINAL DIAL READING, in.	0.0893	0.0393	11/9/2010	13:25

EI_{meas} = 39 **

$\boxed{EI_{50} = 39}$ ***

FINAL MOISTURE CONTENT, DRY DENSITY AND SATURATION DEGREE

TARE No.	---	MOISTURE CONTENT, %	23.3
WET SOIL AND TARE, gr.	433.38	FINAL VOLUME, cc.	214.02
DRY SOIL AND TARE, gr.	351.41	FINAL DRY DENSITY, PCF.	103.3
TARE WEIGHT, gr.	0.00	FINAL SATURATION, %	99.9

* SATURATION % = (Moist. Content in % x G_s X Dry Density in PCF) / ((G_s X 62.4)-Dry Density in PCF)

Gs = 2.7 unless it is known to be less than 2.6 or more than 2.8

** EI_{meas} (Measured Expansion Index) = (Height Change/Initial Height) X 1000

*** EI_{50} (Estimated Expansion Index at 50% Saturation) = $EI_{meas}-((50-S_{meas})x((65+EI_{meas})/(220-S_{meas})))$

EXPANSION INDEX TEST

(UBC 18-2 (1994)/ASTM-D 4829)

Project Name:	Sunshine Canyon LFGTE Facility	Project No.:	0148280000
Sample No.:	3 Stockpile No.: 1	Depth:	---
Soil Description:	Olive Yellow (5Y, 6/8) Silty Sand (SM)		
		Date:	11/08-11/09/10 By: LT

WET DENSITY CALCULATION	TRIAL 1	TRIAL 2	TRIAL 3	TRIAL 4
RING No.	3			
RING AND WET SOIL, gr.	607.06			
WEIGHT OF RING, gr.	199.54			
WEIGHT OF WET SOIL, gr.	407.52			
WET DENSITY, PCF.	123.5			
MOISTURE CALCULATION				
TARE No.	7			
WET SOIL AND TARE, gr.	407.98			
DRY SOIL AND TARE, gr.	378.81			
TARE WEIGHT, gr.	90.28			
MOISTURE CONTENT, %	10.1			
DRY DENSITY, PCF.	112.2			
SATURATION DEGREE (S_{meas}), % *	54.42			

EXPANSION INDEX (EI) CALCULATION

APPARATUS No.:	3
INITIAL SPECIMEN HEIGHT:	1.0000 inch

		HEIGHT CHANGE, in.	DATE	TIME
INITIAL DIAL READING, in.	0.0500	0.0000	11/8/2010	15:45
PERIODIC DIAL READING, in.	0.0501	0.0001	11/9/2010	11:50

FINAL DIAL READING, in.	0.0501	0.0001	11/9/2010	13:25

EI_{meas} = 0 **

$\boxed{EI_{50} = 2}$ ***

FINAL MOISTURE CONTENT, DRY DENSITY AND SATURATION DEGREE

TARE No.	---	MOISTURE CONTENT, %	18.6
WET SOIL AND TARE, gr.	434.16	FINAL VOLUME, cc.	205.95
DRY SOIL AND TARE, gr.	366.13	FINAL DRY DENSITY, PCF.	112.2
TARE WEIGHT, gr.	0.00	FINAL SATURATION, %	100.0

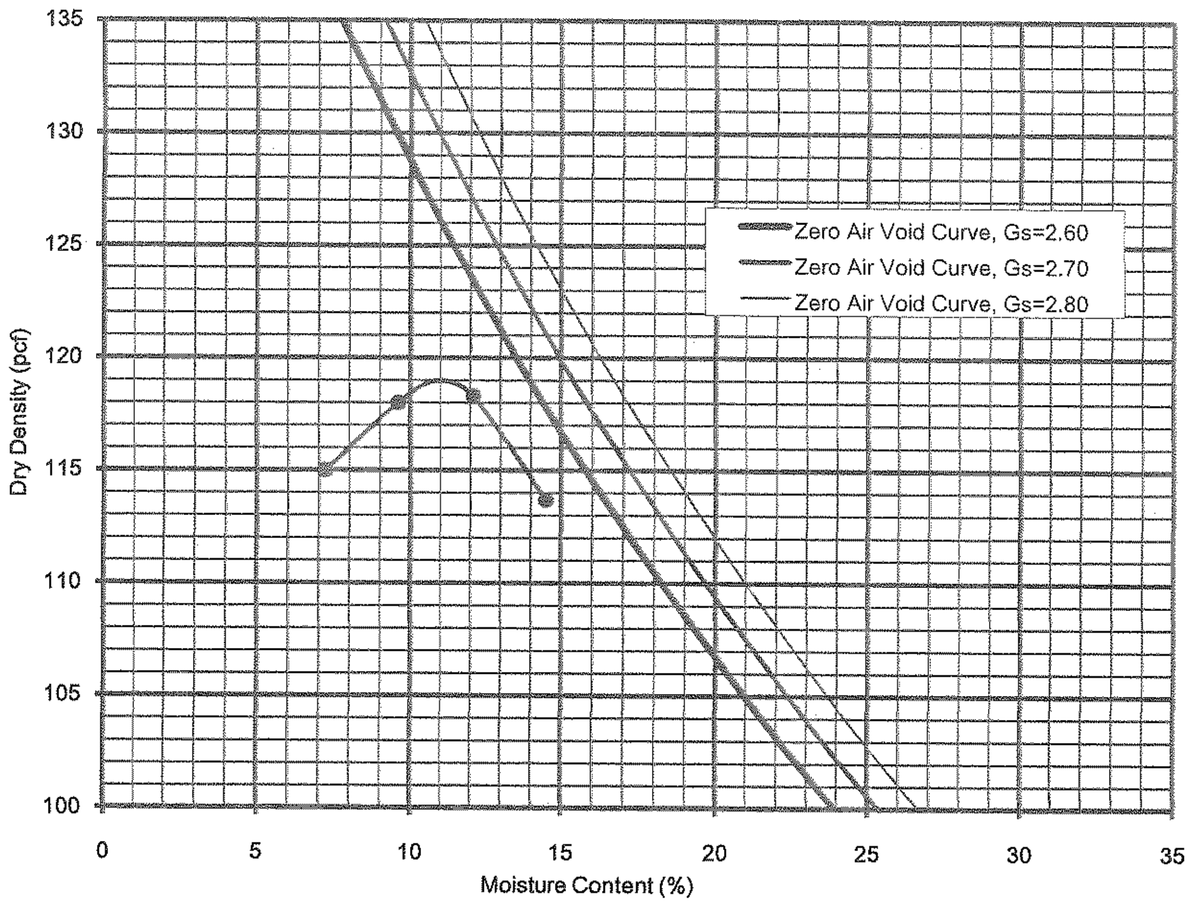
* SATURATION % = (Moist. Content in % x G_s X Dry Density in PCF) / ((G_s X 62.4)-Dry Density in PCF)

Gs = 2.7 unless it is known to be less than 2.6 or more than 2.8

** EI_{meas} (Measured Expansion Index) = (Height Change/Initial Height) X 1000

*** EI_{50} (Estimated Expansion Index at 50% Saturation) = EI_{meas}-((50-S_{meas})x((65+EI_{meas})/(220-S_{meas})))

COMPACTION TEST REPORT



SAMPLE DATA

Sample No.:	C-1	Source:	Excavated Bedrock Material
Soil Description:	Dark Gray (2.5Y, 4/1) Sandy Silt (ML)		
Date:	2/15/2011	Tested by:	L.T.

TEST DATA

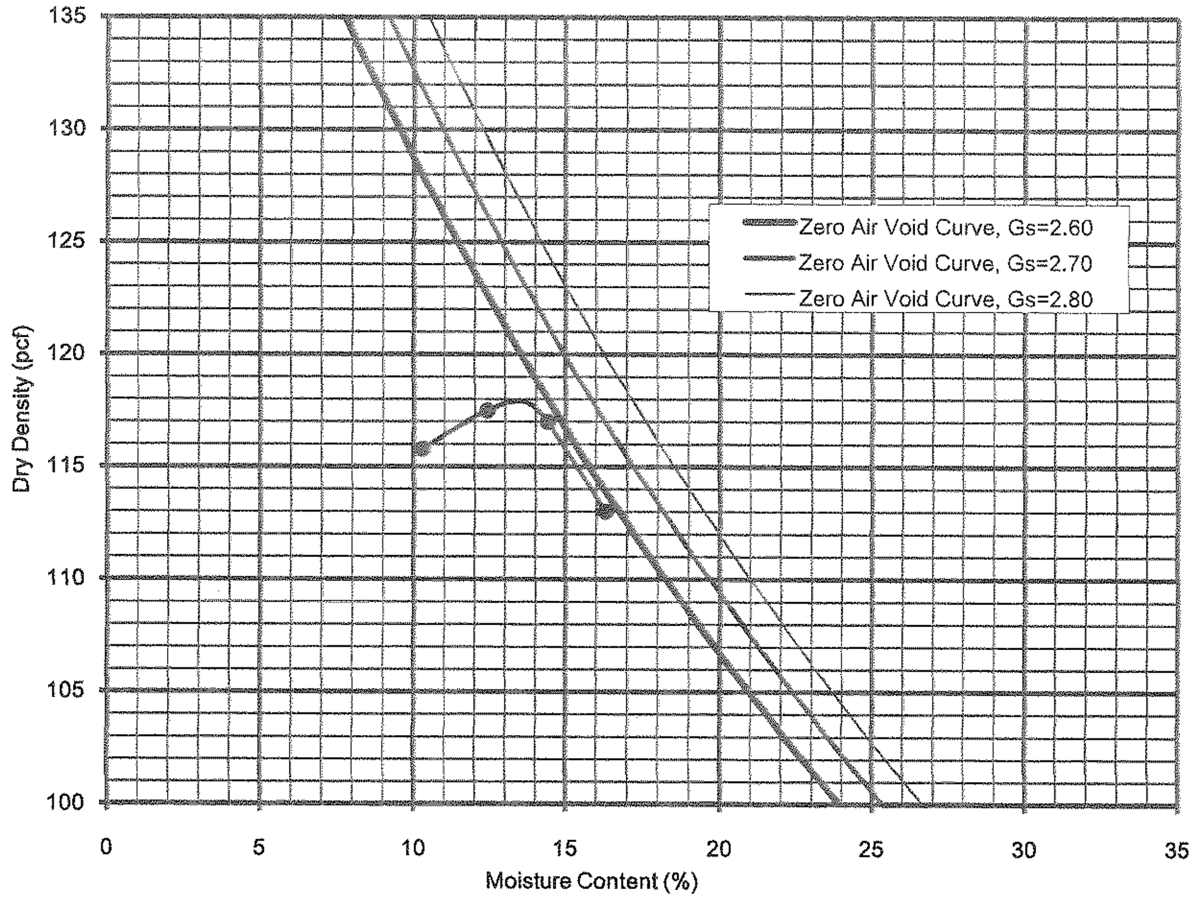
Standard:	ASTM-D1557-B	Mold Volume (ft^3):	1/30	Hammer Weight (lb.):	10
Drop Height (Inches):	18	Number of Layers:	5	Blows per Layer:	25

	1	2	3	4	*Oversize Correction*	
Water Added (gr.)	0	-60	-120	+60	~~Yes~~	No
Weight of Wet Soil & Mold (lb.)	8.89	8.78	8.58	8.81	Coarser (lb.)	Finer (lb.)
Weight of Mold (lb.)	4.47	4.47	4.47	4.47		
Tare No.	15	16	17	14		
Wet Soil & Tare (gr.)	617.33	611.80	587.16	656.71		
Dry Soil & Tare (gr.)	561.13	567.05	554.36	586.14		
Tare Weight (gr.)	97.60	99.89	99.59	98.41		
Moisture Content (%)	12.1	9.6	7.2	14.5		
Dry Density (lb./ft^3)	118.3	118.0	115.0	113.7	------	------

	Uncorrected Value	*Corrected Value*
Maximum Dry Density (lb/ft^3)	119.0	N/A
Optimum Moisture Content (%)	11.0	N/A

AMEC Geomatrix	MAXIMUM DRY DENSITY - OPTIMUM MOISTURE CONTENT CURVE SUNSHINE CANYON LFGTE FACILITY Sylmar, California	Project No. 0148280000

COMPACTION TEST REPORT



SAMPLE DATA

Sample No.:	C-2	Source:	Stockpile South of Gas to Energy Site
Soil Description:	Very Dark Gray (2.5Y, 3/1) Sandy Lean Clay (CL)		
Date:	3/9/2011	Tested by:	L.T.

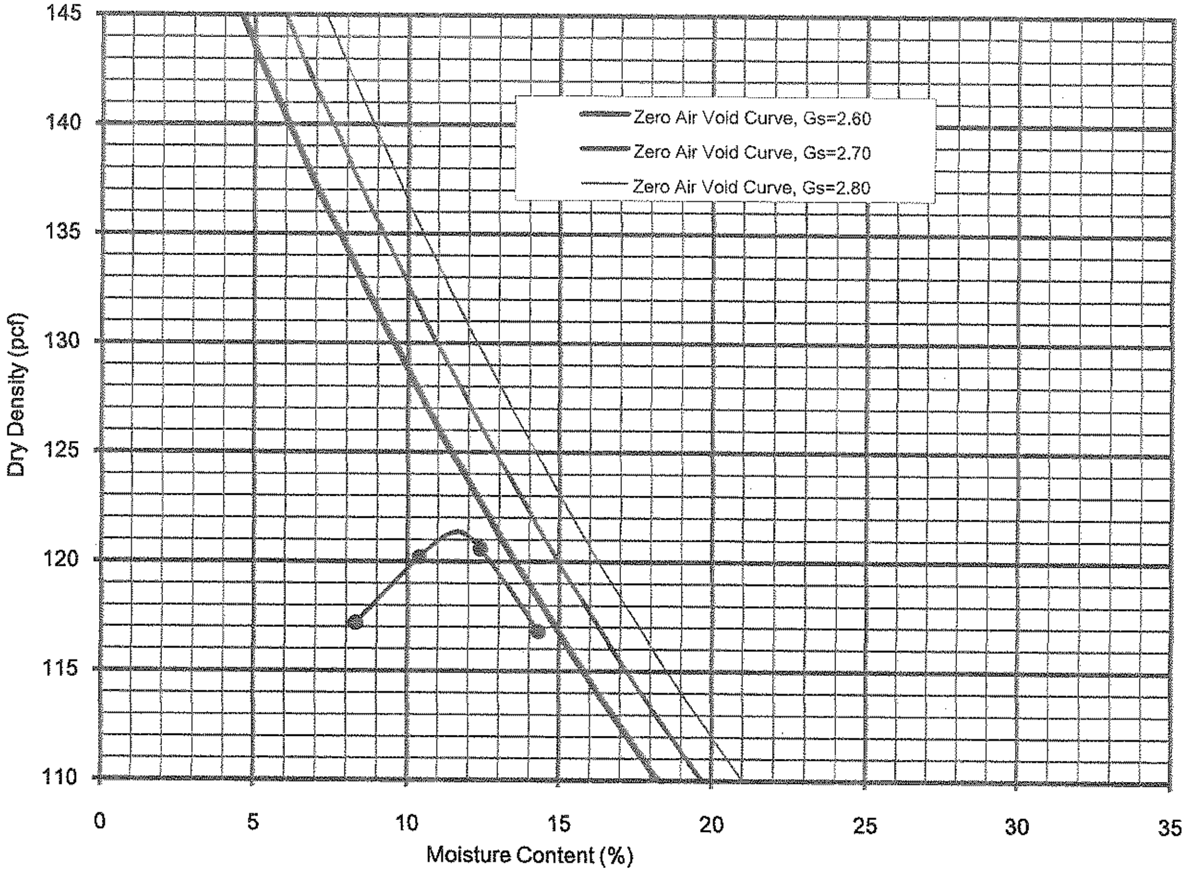
TEST DATA

Standard: ASTM-D1557-B	Mold Volume (ft^3):		1/30	Hammer Weight (lb.):		10
Drop Height (Inches): 18	Number of Layers:		5	Blows per Layer:		25
	1	2	3	4	*Oversize Correction*	
Water Added (gr.)	0	-47	-94	+47	~~Yes~~	No
Weight of Wet Soil & Mold (lb.)	8.93	8.87	8.73	8.85	Coarser (lb.)	Finer (lb.)
Weight of Mold (lb.)	4.47	4.47	4.47	4.47		
Tare No.	10	6	13	7		
Wet Soil & Tare (gr.)	700.88	612.95	597.79	615.67		
Dry Soil & Tare (gr.)	625.04	556.32	550.94	542.19		
Tare Weight (gr.)	98.38	98.21	98.28	90.28		
Moisture Content (%)	14.4	12.4	10.3	16.3		
Dry Density ($lb./ft^3$)	117.0	117.5	115.8	113.0	------	------

	Uncorrected Value	Corrected Value
Maximum Dry Density (lb/ft^3)	**118.0**	N/A
Optimum Moisture Content (%)	**13.5**	N/A

AMEC Geomatrix	MAXIMUM DRY DENSITY - OPTIMUM MOISTURE CONTENT CURVE SUNSHINE CANYON LFGTE FACILITY Sylmar, California	Project No. 0148280000

COMPACTION TEST REPORT



SAMPLE DATA

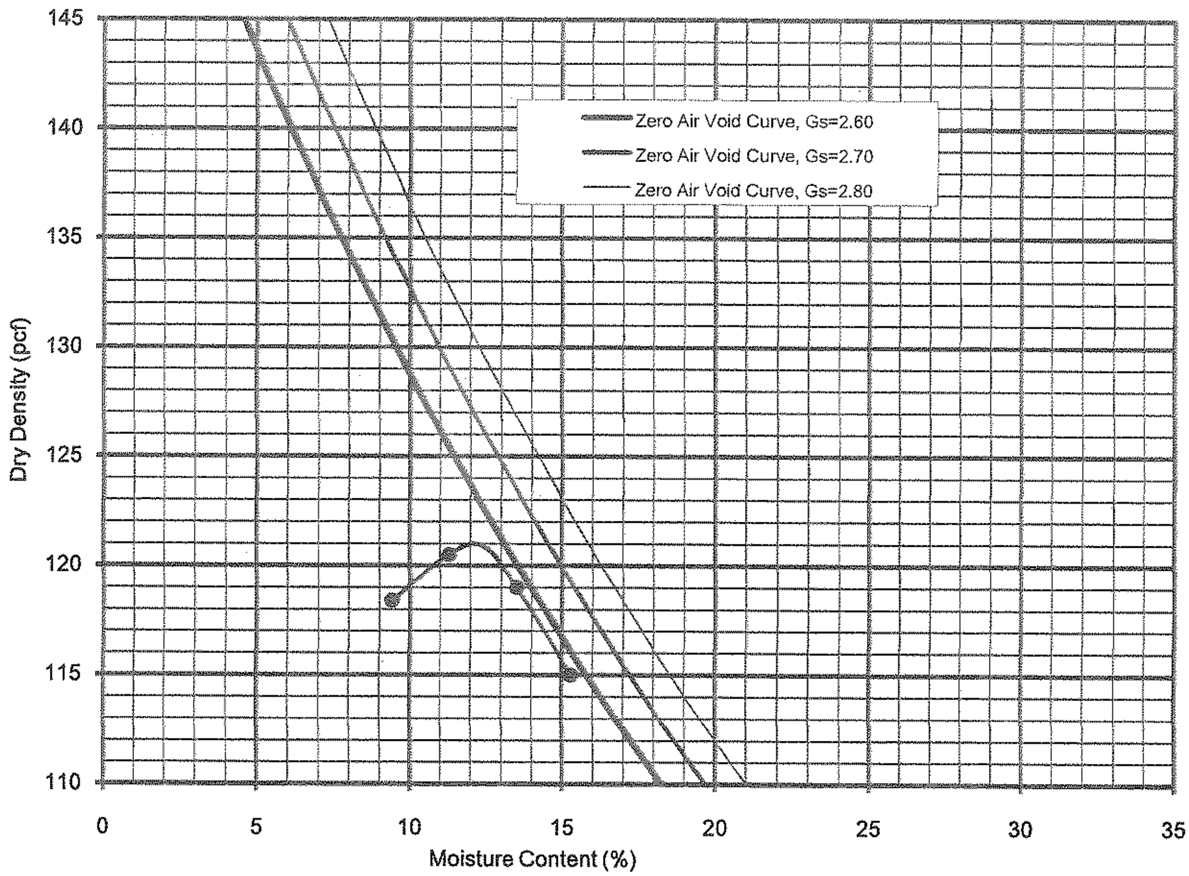
Sample No.:	1	Stockpile No.:	2
Soil Description:	Dark Yellowish Brown (10YR, 4/4) Clayey Sand (SC)		
Date:	10/28/2010	Tested by:	L.T.

TEST DATA

Standard: ASTM-D1557-B	Mold Volume (ft^3):		1/30	Hammer Weight (lb.):		10	
Drop Height (Inches): 18	Number of Layers:		5	Blows per Layer:		25	
	1	2	3	4	*Oversize Correction*		
Water Added (gr.)	-94	-47	0	-141	~~Yes~~	No	
Weight of Wet Soil & Mold (lb.)	8.89	8.99	8.92	8.70	Coarser (lb.)	Finer (lb.)	
Weight of Mold (lb.)	4.47	4.47	4.47	4.47			
Tare No.	20	8	9	16			
Wet Soil & Tare (gr.)	572.40	689.90	640.01	562.94			
Dry Soil & Tare (gr.)	527.08	624.47	571.88	527.51			
Tare Weight (gr.)	89.56	97.39	96.93	99.89			
Moisture Content (%)	10.4	12.4	14.3	8.3			
Dry Density (lb./ft^3)	120.2	120.6	116.8	117.2	------	------	
	Uncorrected Value			*Corrected Value*			
Maximum Dry Density (lb/ft³)	121.5			N/A			
Optimum Moisture Content (%)	11.5			N/A			

AMEC Geomatrix	MAXIMUM DRY DENSITY - OPTIMUM MOISTURE CONTENT CURVE SUNSHINE CANYON LFGTE FACILITY Sylmar, California	Project No. 0148280000

COMPACTION TEST REPORT



SAMPLE DATA

Sample No.:	2	Stockpile No.:	2
Soil Description:	Dark Olive Gray (5Y, 3/2) Sandy Lean Clay (CL)		
Date:	10/28/2010	Tested by:	L.T.

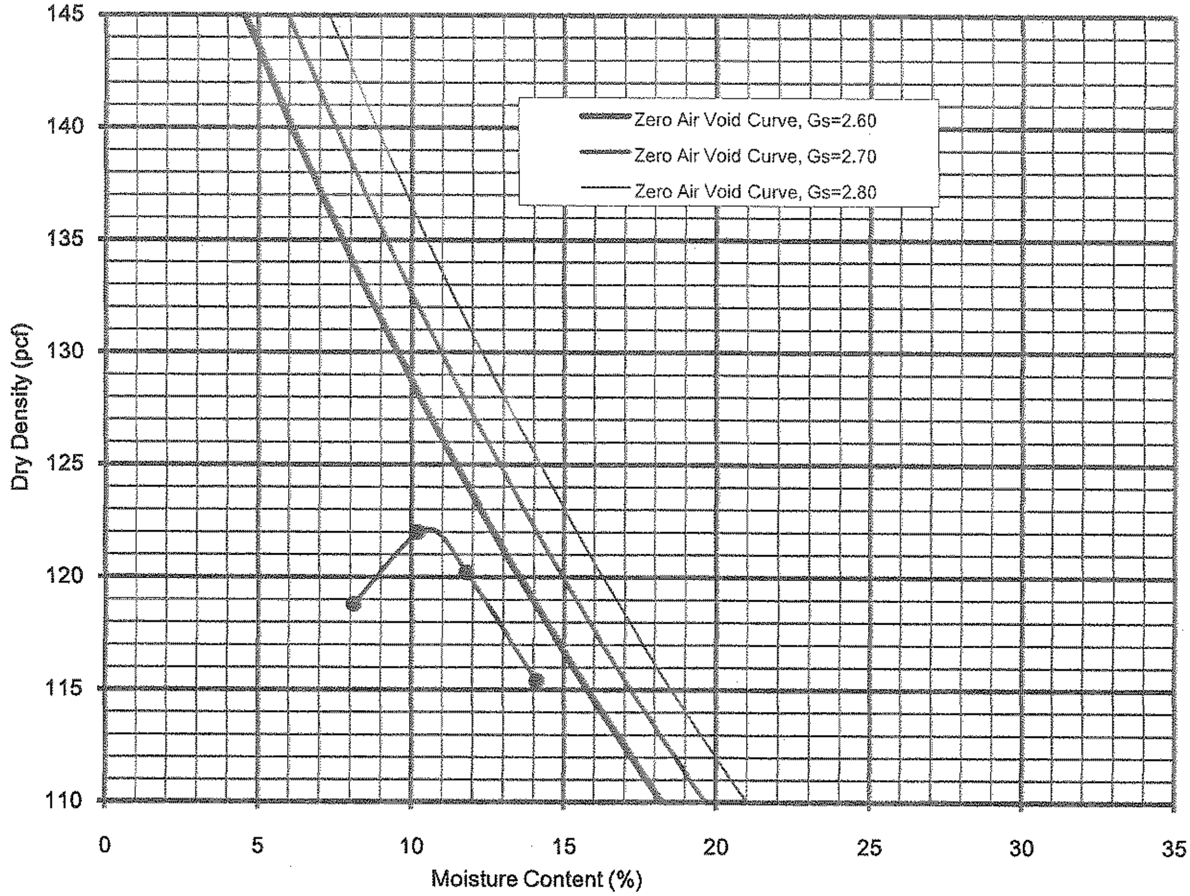
TEST DATA

Standard: ASTM-D1557-B	Mold Volume (ft^3):		1/30	Hammer Weight (lb.):		10
Drop Height (Inches): 18	Number of Layers:		5	Blows per Layer:		25
	1	2	3	4	*Oversize Correction*	
Water Added (gr.)	-94	-47	0	-141	~~Yes~~	No
Weight of Wet Soil & Mold (lb.)	8.94	8.97	8.89	8.79	Coarser (lb.)	Finer (lb.)
Weight of Mold (lb.)	4.47	4.47	4.47	4.47		
Tare No.	1	18	19	4		
Wet Soil & Tare (gr.)	669.46	589.52	675.91	562.73		
Dry Soil & Tare (gr.)	611.63	530.35	598.40	522.66		
Tare Weight (gr.)	97.68	90.72	90.56	97.59		
Moisture Content (%)	11.3	13.5	15.3	9.4		
Dry Density ($lb./ft^3$)	120.5	119.0	115.0	118.4	------	------

	Uncorrected Value	*Corrected Value*
Maximum Dry Density (lb/ft^3)	121.0	N/A
Optimum Moisture Content (%)	12.0	N/A

AMEC Geomatrix	MAXIMUM DRY DENSITY - OPTIMUM MOISTURE CONTENT CURVE SUNSHINE CANYON LFGTE FACILITY Sylmar, California	Project No. 0148280000

COMPACTION TEST REPORT



SAMPLE DATA

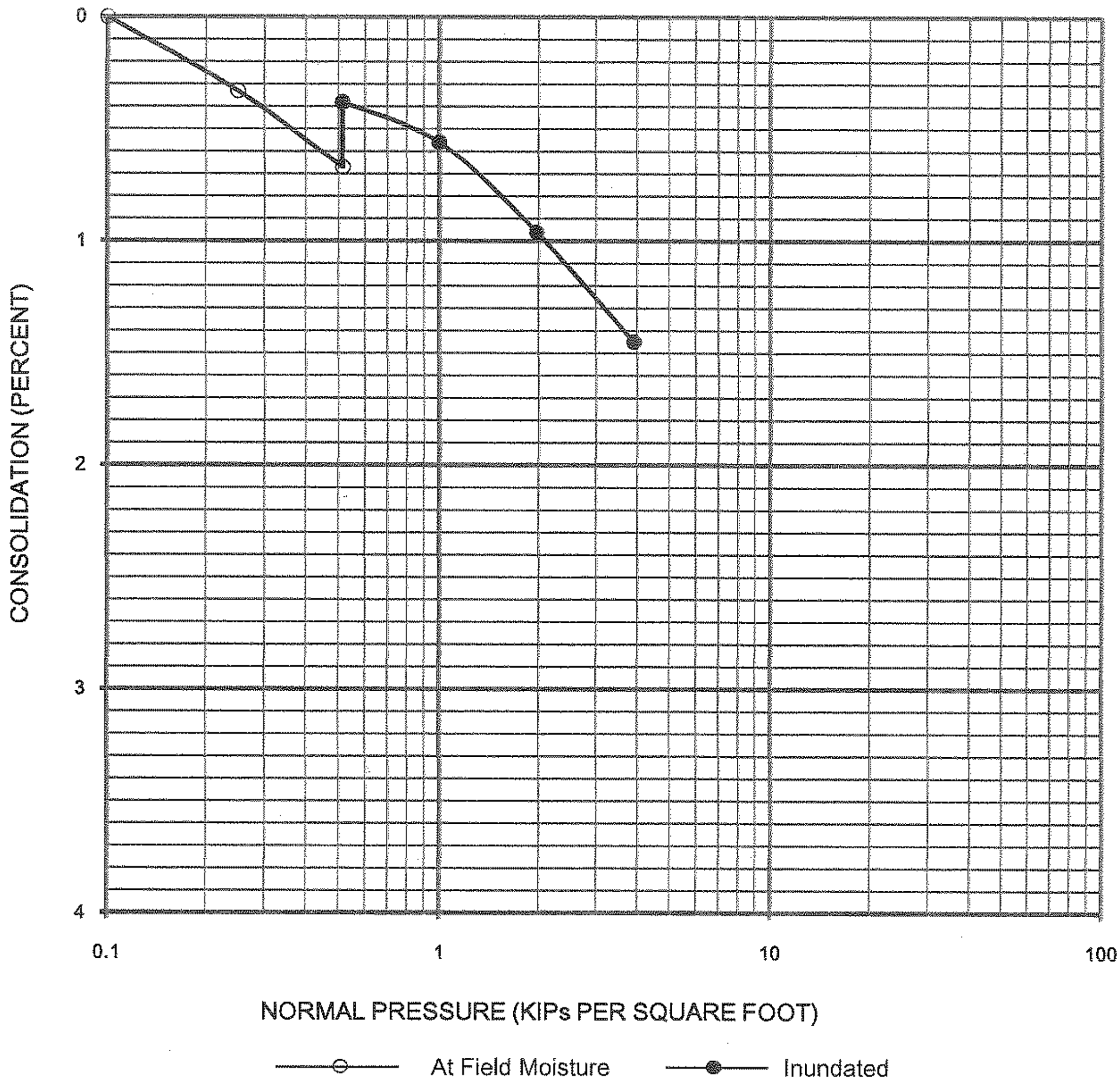
Sample No.:	3	Stockpile No.:	1
Soil Description:	Olive Yellow (5Y, 6/8) Silty Sand (SM)		
Date:	10/28/2010	Tested by:	L.T.

TEST DATA

Standard: ASTM-D1557-B	Mold Volume (ft³):		1/30	Hammer Weight (lb.):	10	
Drop Height (Inches): 18	Number of Layers:		5	Blows per Layer:	25	
	1	2	3	4	*Oversize Correction*	
Water Added (gr.)	-47	0	+47	-94	~~Yes~~	No
Weight of Wet Soil & Mold (lb.)	8.95	8.95	8.86	8.75	Coarser (lb.)	Finer (lb.)
Weight of Mold (lb.)	4.47	4.47	4.47	4.47		
Tare No.	15	14	11	10		
Wet Soil & Tare (gr.)	586.13	631.95	654.29	537.10		
Dry Soil & Tare (gr.)	540.90	575.43	585.38	504.28		
Tare Weight (gr.)	97.60	98.41	96.82	98.38		
Moisture Content (%)	10.2	11.8	14.1	8.1		
Dry Density (lb./ft³)	122.0	120.2	115.4	118.8	------	------

	Uncorrected Value	*Corrected Value*
Maximum Dry Density (lb/ft³)	122.0	N/A
Optimum Moisture Content (%)	10.5	N/A

AMEC Geomatrix	MAXIMUM DRY DENSITY - OPTIMUM MOISTURE CONTENT CURVE SUNSHINE CANYON LFGTE FACILITY Sylmar, California	Project No. 0148280000

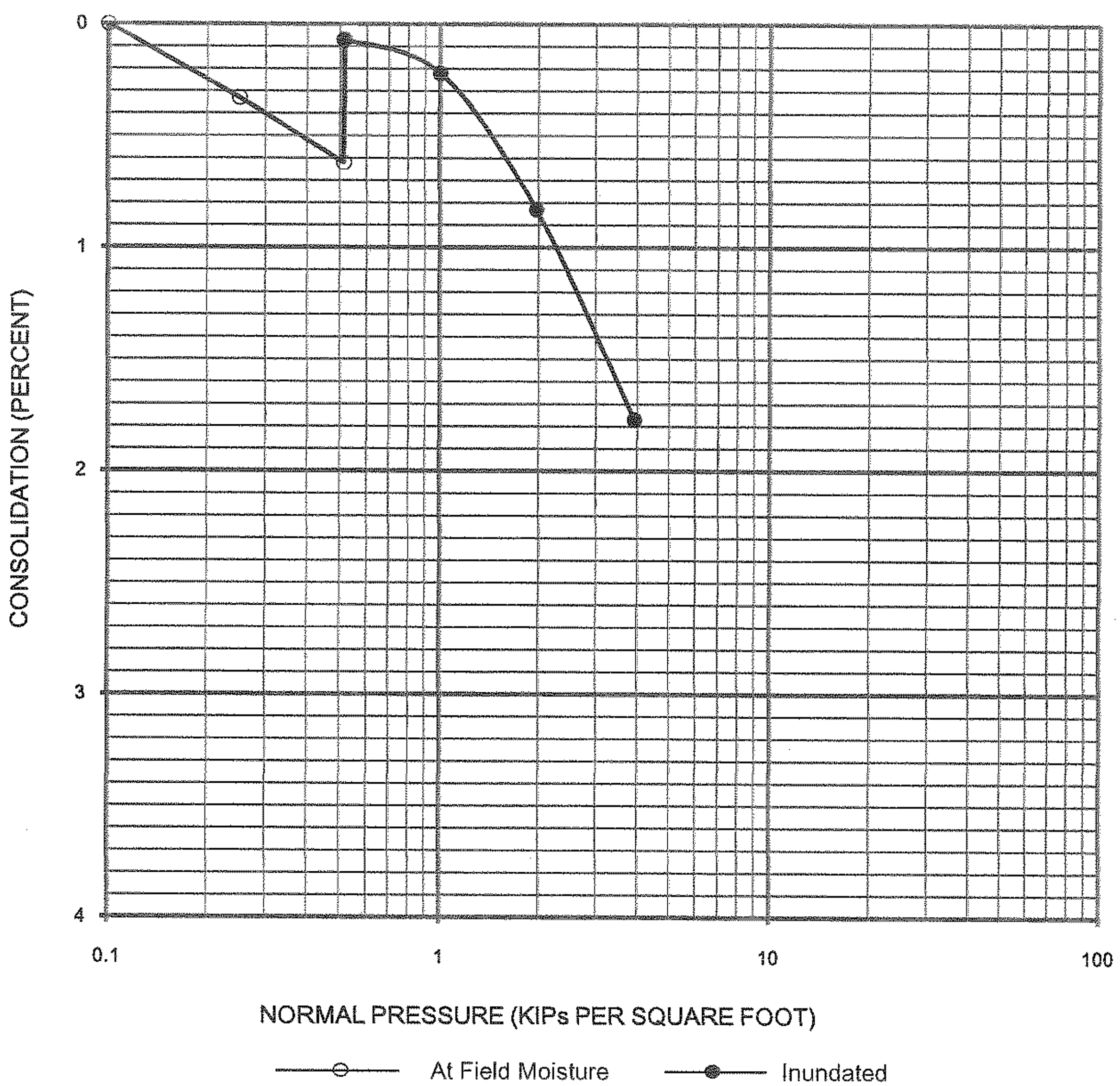


Sample No.:	1	Remold Dry Density (PCF):	109.4 (1)
Stockpile No.:	2	Remold Moisture Content (%):	13.5 (2)
Sample Condition:	Remolded	Soil Type:	SC

(1) *90% maximum dry density*

(2) *2% over optimum moisture*

AMEC Geomatrix	CONSOLIDATION VS. PRESSURE CURVE SUNSHINE CANYON LFGTE FACILITY Sylmar, California	Project No. 0148280000

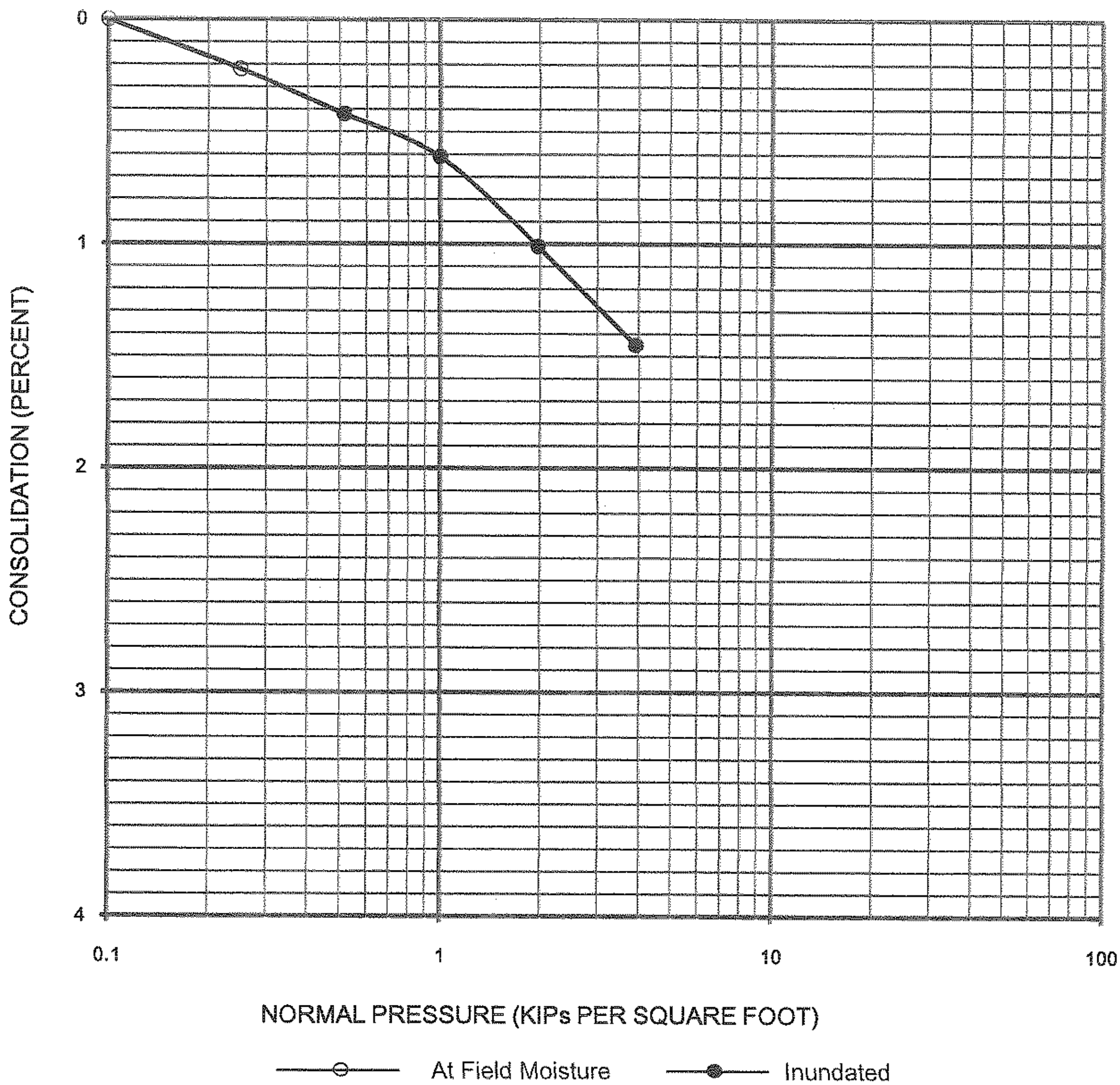


Sample No.:	2	Remold Dry Density (PCF):	108.9 (1)
Stockpile No.:	2	Remold Moisture Content (%):	14.0 (2)
Sample Condition:	Remolded	Soil Type:	CL

(1) *90% maximum dry density*

(2) *2% over optimum moisture*

AMEC Geomatrix	CONSOLIDATION VS. PRESSURE CURVE SUNSHINE CANYON LFGTE FACILITY Sylmar, California	Project No. 0148280000

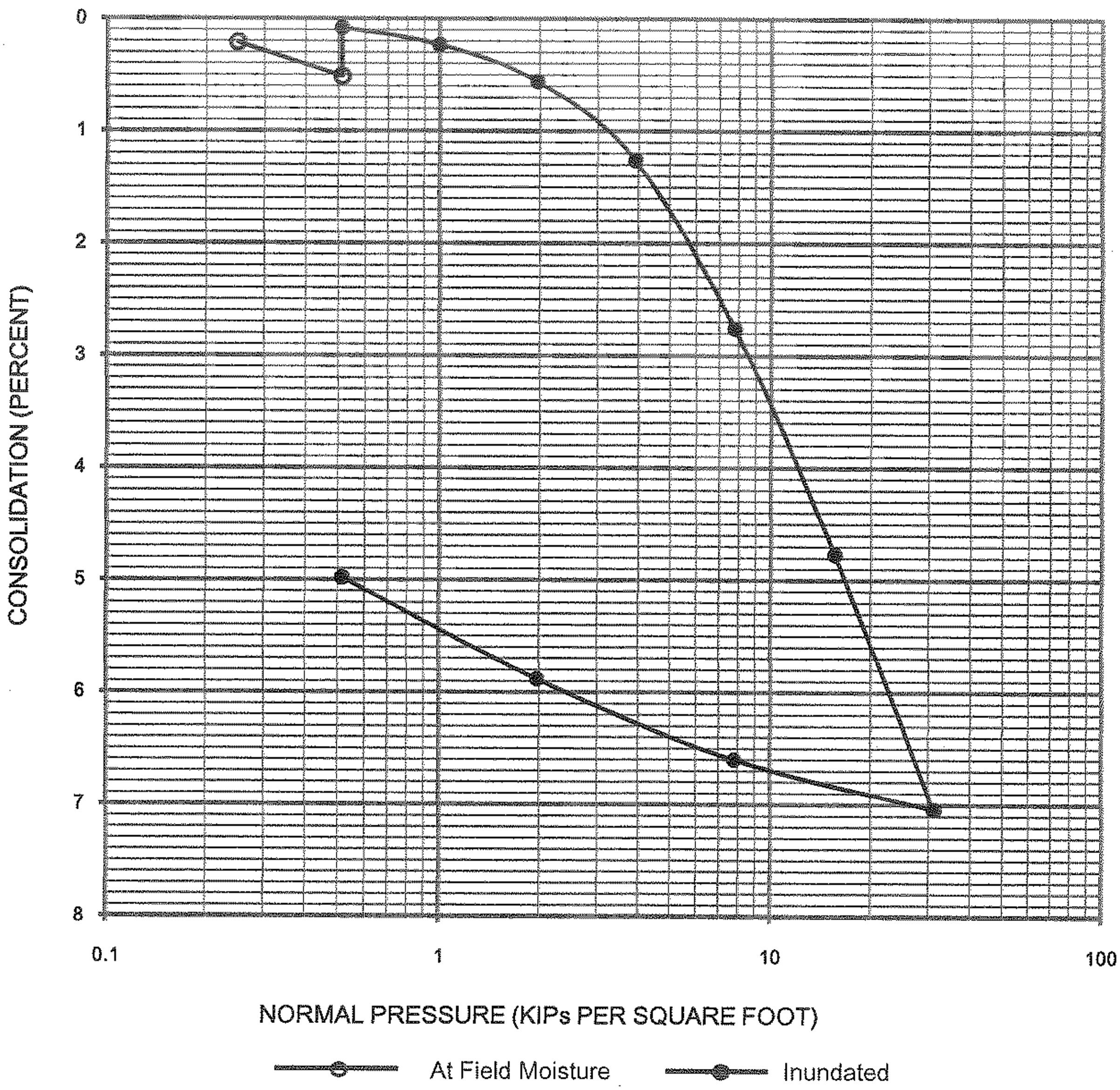


Sample No.:	3	Remold Dry Density (PCF):	109.8 (1)
Stockpile No.:	1	Remold Moisture Content (%):	12.5 (2)
Sample Condition:	Remolded	Soil Type:	SM

(1) *90% maximum dry density*

(2) *2% over optimum moisture*

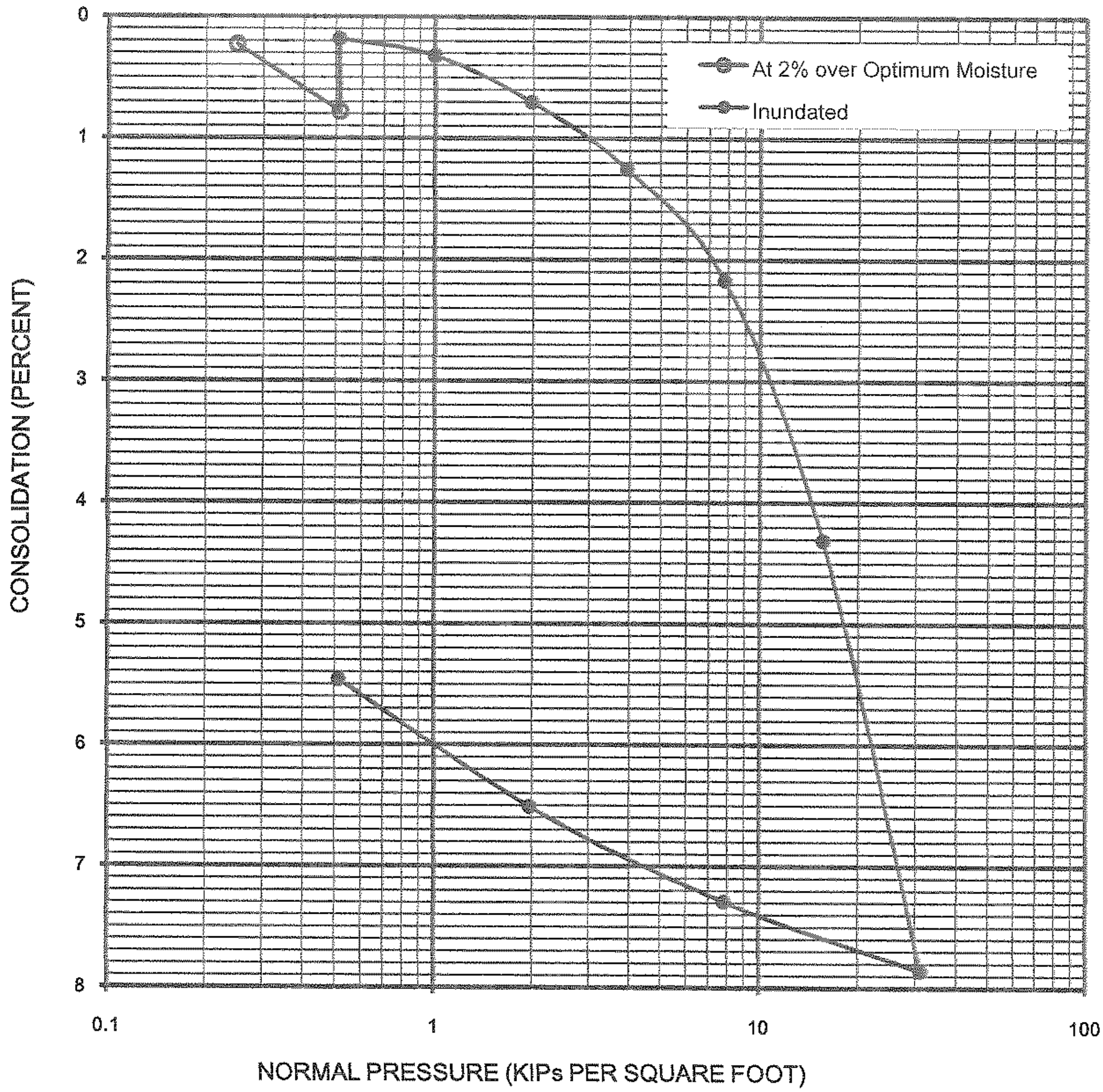
AMEC Geomatrix	CONSOLIDATION VS. PRESSURE CURVE SUNSHINE CANYON LFGTE FACILITY Sylmar, California	Project No. 0148280000



Sample No.:	C-1	Remold Dry Density (PCF):	107.1 [(1)]
Source:	Excavated Bedrock Material	Remold Moisture Content (%):	13.0 [(2)]
Soil Type:	ML		

(1) 90% maximum dry density
(2) 2% over optimum moisture

AMEC Geomatrix	CONSOLIDATION VS. PRESSURE CURVE SUNSHINE CANYON LFGTE FACILITY Sylmar, California	Project No. 0148280000



Sample No.:	C-2	Remold Dry Density (PCF):	106.2 (1)
Source:	Stockpile South of Gas to Energy Site	Remold Moisture Content (%):	15.5 (2)
Soil Type:	CL		

(1) *90% maximum dry density*

(2) *2% over optimum moisture*

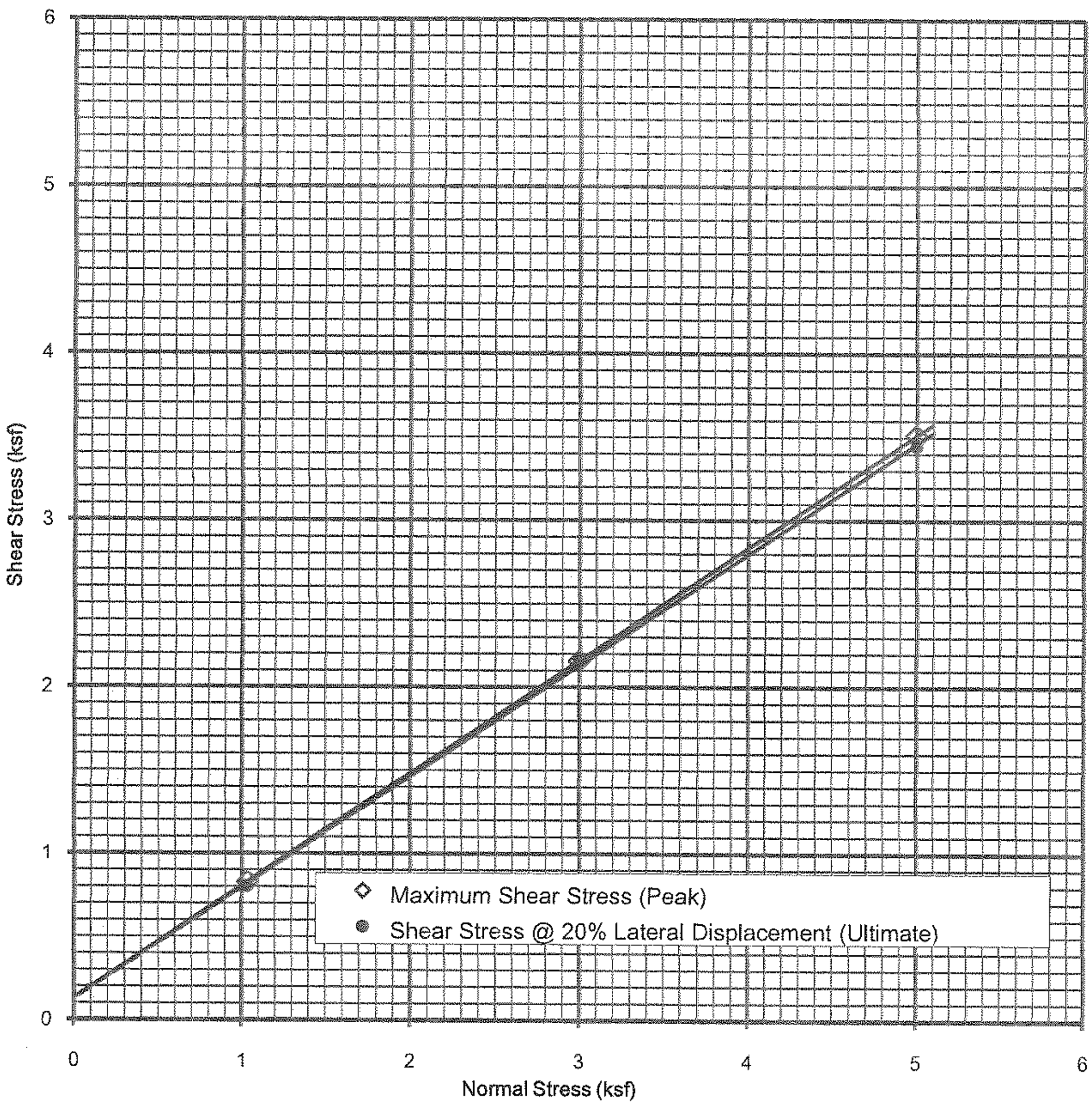
AMEC Geomatrix	CONSOLIDATION VS. PRESSURE CURVE SUNSHINE CANYON LFGTE FACILITY Sylmar, California	Project No. 0148280000

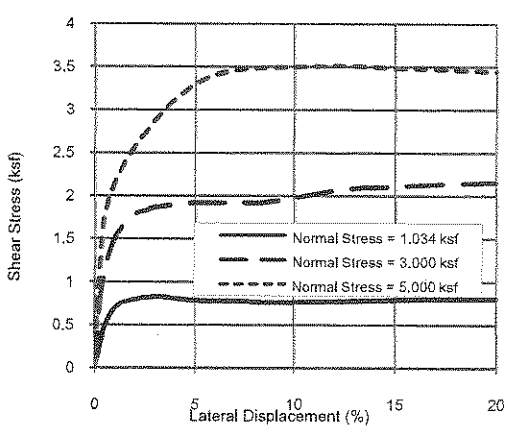
DIRECT SHEAR TEST
(ASTM-D3080)

Project Name:	Sunshine Canyon LFGTE Facility	**Project No.:**	0148280000		
Sample No.: 1	**Stockpile No.:** 2	**Depth:** ---		**Date:**	11/01-11/04/2010
Soil Description:	Dark Yellowish Brown (10YR, 4/4) Clayey Sand (SC)			**Tested By:**	LT

				Before Test	After Test Load 1	After Test Load 2	After Test Load 3
Sample Diameter, in:	2.416	Weight of Wet Soil & Ring, gr:		580.18	---	---	---
Normal Stress, ksf:	1,3,5	Weight of Ring, gr:		132.10	---	---	---
Over-burdened @, pcf:		Height of Sample, in:		3.00	0.9923	0.9802	0.9710
Shear Rate, in/min:	0.005	Moisture-	Tare No.:	---	---	---	---
Natural Moisture(x):			Wet Weight and Tare, gr:		151.86	152.70	151.03
Saturated(x):	X		Dry Weight and Tare, gr:	Remolded	127.05	128.44	127.62
Intact(x):			Tare Weight, gr:		0.00	0.00	0.00
Remolded to 109.4 pcf (90% maximum dry density) at 13.5% moisture content (2% over optimum)			Moisture Content, %:	13.5	19.5	18.9	18.3
		Wet Density, pcf:		124.1	131.7	132.6	133.3
		Dry Density, pcf:		109.4	110.2	111.6	112.6
		Saturation %:	S.G. = 2.70 (Assumed)	67.3	99.6	99.8	99.7

Load 1 (KSF): 1.034				Load 2 (KSF): 3.000				Load 3 (KSF): 5.000			
Shear Deflec-tion (in)	Lateral Displace-ment (%)	Load Ring Reading	Shear Stress (KSF)	Shear Deflec-tion (in)	Lateral Displace-ment (%)	Load Ring Reading	Shear Stress (KSF)	Shear Deflec-tion (in)	Lateral Displace-ment (%)	Load Ring Reading	Shear Stress (KSF)
0.0098	0.406	0.0028	0.431	0.0098	0.406	0.0074	1.043	0.0098	0.406	0.0122	1.680
0.0199	0.823	0.0044	0.644	0.0199	0.823	0.0103	1.428	0.0199	0.823	0.0150	2.052
0.0300	1.241	0.0052	0.750	0.0300	1.241	0.0117	1.614	0.0300	1.241	0.0167	2.278
0.0401	1.659	0.0054	0.777	0.0401	1.659	0.0125	1.720	0.0401	1.659	0.0180	2.451
0.0502	2.077	0.0056	0.803	0.0502	2.077	0.0131	1.800	0.0502	2.077	0.0191	2.597
0.0603	2.495	0.0057	0.817	0.0603	2.495	0.0133	1.826	0.0603	2.495	0.0201	2.730
0.0704	2.912	0.0058	0.830	0.0704	2.912	0.0136	1.866	0.0704	2.912	0.0211	2.863
0.0805	3.330	0.0058	0.830	0.0805	3.330	0.0137	1.880	0.0805	3.330	0.0219	2.969
0.0905	3.748	0.0057	0.817	0.0905	3.748	0.0138	1.893	0.0905	3.748	0.0227	3.075
0.1006	4.166	0.0056	0.803	0.1006	4.166	0.0139	1.906	0.1006	4.166	0.0233	3.155
0.1208	5.002	0.0055	0.790	0.1208	5.002	0.0140	1.919	0.1208	5.002	0.0244	3.301
0.1410	5.837	0.0054	0.777	0.1410	5.837	0.0140	1.919	0.1410	5.837	0.0251	3.394
0.1612	6.673	0.0054	0.777	0.1612	6.673	0.0140	1.919	0.1612	6.673	0.0255	3.447
0.1814	7.509	0.0054	0.777	0.1814	7.509	0.0140	1.919	0.1814	7.509	0.0257	3.474
0.2016	8.344	0.0053	0.764	0.2016	8.344	0.0140	1.919	0.2016	8.344	0.0258	3.487
0.2521	10.433	0.0053	0.764	0.2521	10.433	0.0146	1.999	0.2521	10.433	0.0259	3.500
0.3025	12.523	0.0054	0.777	0.3025	12.523	0.0152	2.079	0.3025	12.523	0.0260	3.514
0.3530	14.612	0.0055	0.790	0.3530	14.612	0.0154	2.105	0.3530	14.612	0.0258	3.487
0.4035	16.701	0.0056	0.803	0.4035	16.701	0.0156	2.132	0.4035	16.701	0.0257	3.474
0.4828	19.982	0.0056	0.803	0.4828	19.982	0.0158	2.159	0.4828	19.982	0.0255	3.447
Max. Shear Stress, ksf:			0.830				2.159				3.514
Shear Deflt.@Max Stress,%.:			3.3				20.0				12.5





Sample No.:	1 (Stockpile No. 2)
Sample Depth :	---
Soil Type:	SC
Sample Conditions:	Remolded; Saturated
Remold Dry Density (PCF):	109.4
Remold Moisture Content (%):	13.5

	◇	●
Cohesion (PSF):	130	129
Friction Angle (Degrees):	34	34

AMEC Geomatrix	DIRECT SHEAR TEST SUNSHINE CANYON LFGTE FACILITY Sylmar, California	Project No. 0148280000

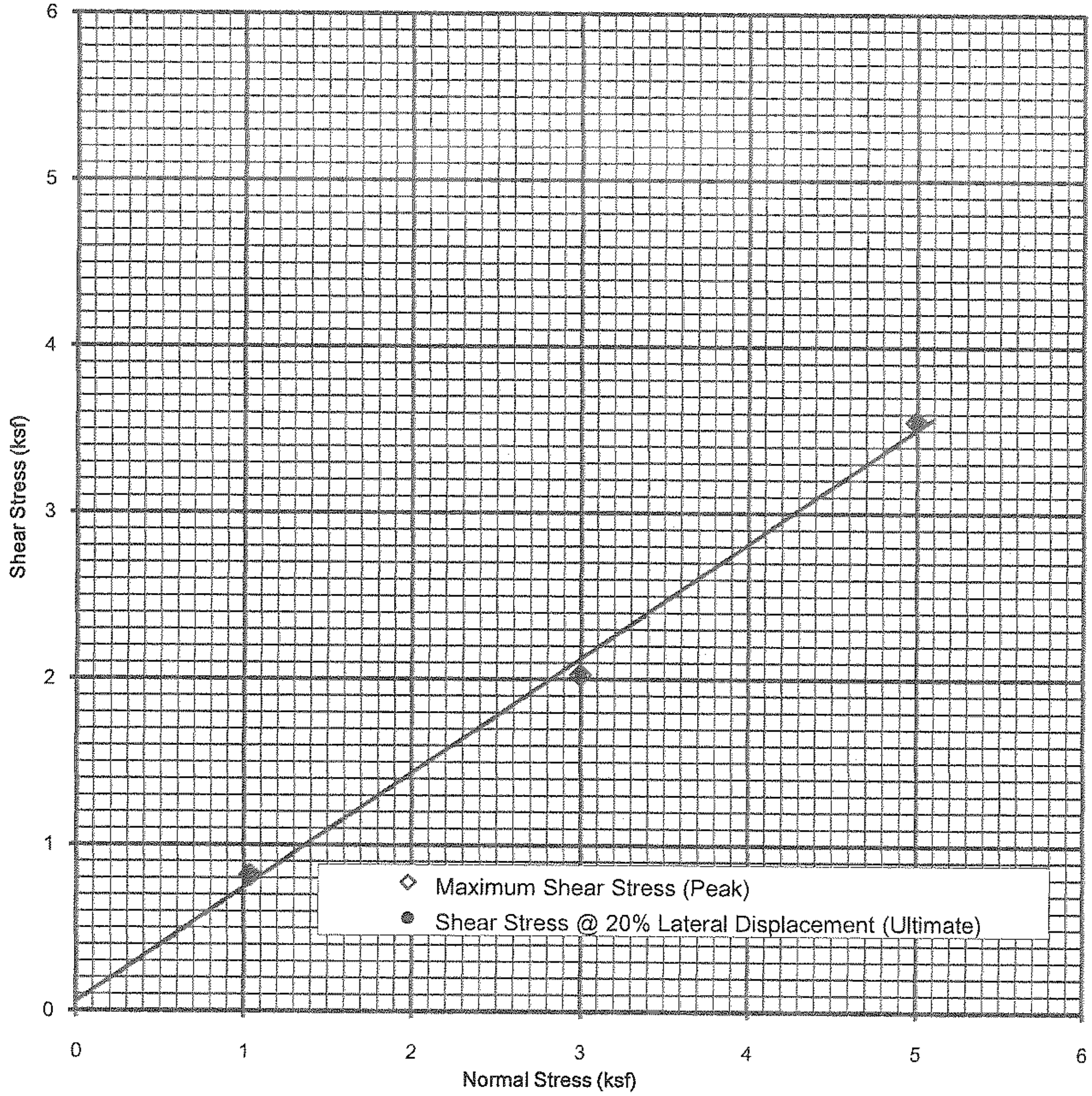
DIRECT SHEAR TEST
(ASTM-D3080)

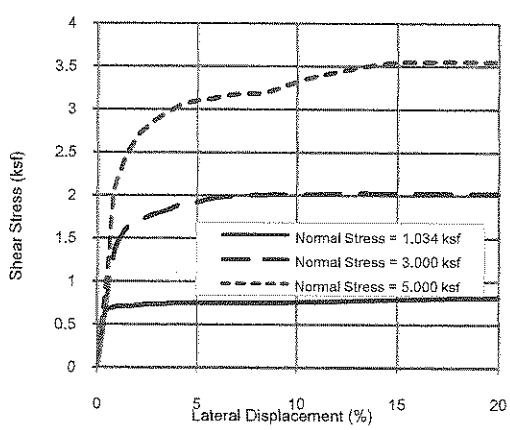
Project Name:	Sunshine Canyon LFGTE Facility		**Project No.:**	0148280000	
Sample No.:	2	**Stockpile No.:** 2	**Depth:** ---	**Date:**	11/01-11/05/2010
Soil Description:		Dark Olive Gray (5Y, 3/2) Sandy Lean Clay (CL)		**Tested By:**	LT

				Before Test	After Test Load 1	After Test Load 2	After Test Load 3
Sample Diameter, in:	2.416	Weight of Wet Soil & Ring, gr:		576.80	---	---	---
Normal Stress, ksf:	1,3,5	Weight of Ring, gr:		128.60	---	---	---
Over-burdened @, pcf:		Height of Sample, in:		3.00	0.9920	0.9796	0.9673
Shear Rate, in/min:	0.005	Moisture-	Tare No.:	---	---	---	---
Natural Moisture(x):			Wet Weight and Tare, gr:		155.54	151.35	150.52
Saturated(x):	X		Dry Weight and Tare, gr:	Remolded	129.85	127.07	127.13
Intact(x):			Tare Weight, gr:		0.00	0.00	0.00
Remolded to 108.9 pcf (90% maximum dry density) at 14.0% moisture content (2% over optimum)			Moisture Content, %:	14.0	19.8	19.1	18.4
		Wet Density, pcf:		124.1	131.5	132.4	133.3
		Dry Density, pcf:		108.9	109.8	111.2	112.6
		Saturation %:	S.G. = 2.70 (Assumed)	69.0	99.8	99.9	99.9

Load 1 (KSF): 1.034				Load 2 (KSF): 3.000				Load 3 (KSF): 5.000			
Shear Deflec-tion (in)	Lateral Displace-ment (%)	Load Ring Reading	Shear Stress (KSF)	Shear Deflec-tion (in)	Lateral Displace-ment (%)	Load Ring Reading	Shear Stress (KSF)	Shear Deflec-tion (in)	Lateral Displace-ment (%)	Load Ring Reading	Shear Stress (KSF)
0.0098	0.406	0.0043	0.631	0.0098	0.406	0.0046	0.671	0.0098	0.406	0.0062	0.883
0.0199	0.823	0.0048	0.697	0.0199	0.823	0.0097	1.348	0.0199	0.823	0.0147	2.012
0.0300	1.241	0.0049	0.710	0.0300	1.241	0.0112	1.547	0.0300	1.241	0.0173	2.358
0.0401	1.659	0.0049	0.710	0.0401	1.659	0.0120	1.654	0.0401	1.659	0.0188	2.557
0.0502	2.077	0.0050	0.724	0.0502	2.077	0.0123	1.694	0.0502	2.077	0.0199	2.703
0.0603	2.495	0.0051	0.737	0.0603	2.495	0.0127	1.747	0.0603	2.495	0.0206	2.796
0.0704	2.912	0.0051	0.737	0.0704	2.912	0.0130	1.787	0.0704	2.912	0.0212	2.876
0.0805	3.330	0.0051	0.737	0.0805	3.330	0.0132	1.813	0.0805	3.330	0.0217	2.942
0.0905	3.748	0.0052	0.750	0.0905	3.748	0.0135	1.853	0.0905	3.748	0.0221	2.996
0.1006	4.166	0.0052	0.750	0.1006	4.166	0.0138	1.893	0.1006	4.166	0.0225	3.049
0.1208	5.002	0.0052	0.750	0.1208	5.002	0.0140	1.919	0.1208	5.002	0.0229	3.102
0.1410	5.837	0.0052	0.750	0.1410	5.837	0.0144	1.973	0.1410	5.837	0.0231	3.128
0.1612	6.673	0.0052	0.750	0.1612	6.673	0.0145	1.986	0.1612	6.673	0.0234	3.168
0.1814	7.509	0.0052	0.750	0.1814	7.509	0.0146	1.999	0.1814	7.509	0.0235	3.182
0.2016	8.344	0.0052	0.750	0.2016	8.344	0.0147	2.012	0.2016	8.344	0.0236	3.195
0.2521	10.433	0.0053	0.764	0.2521	10.433	0.0147	2.012	0.2521	10.433	0.0248	3.354
0.3025	12.523	0.0054	0.777	0.3025	12.523	0.0148	2.026	0.3025	12.523	0.0256	3.461
0.3530	14.612	0.0055	0.790	0.3530	14.612	0.0148	2.026	0.3530	14.612	0.0262	3.540
0.4035	16.701	0.0056	0.803	0.4035	16.701	0.0148	2.026	0.4035	16.701	0.0263	3.554
0.4828	19.982	0.0057	0.817	0.4828	19.982	0.0148	2.026	0.4828	19.982	0.0263	3.554

	Load 1	Load 2	Load 3
Max. Shear Stress, ksf:	0.817	2.026	3.554
Shear Deflt.@Max Stress,%.:	20.0	20.0	20.0





Sample No.:	2 (Stockpile No. 2)
Sample Depth :	---
Soil Type:	CL
Sample Conditions:	Remolded; Saturated
Remold Dry Density (PCF):	108.9
Remold Moisture Content (%):	14.0

	◇	●
Cohesion (PSF):	53	53
Friction Angle (Degrees):	35	35

AMEC Geomatrix	DIRECT SHEAR TEST SUNSHINE CANYON LFGTE FACILITY Sylmar, California	Project No. 0148280000

AMEC Geomatrix

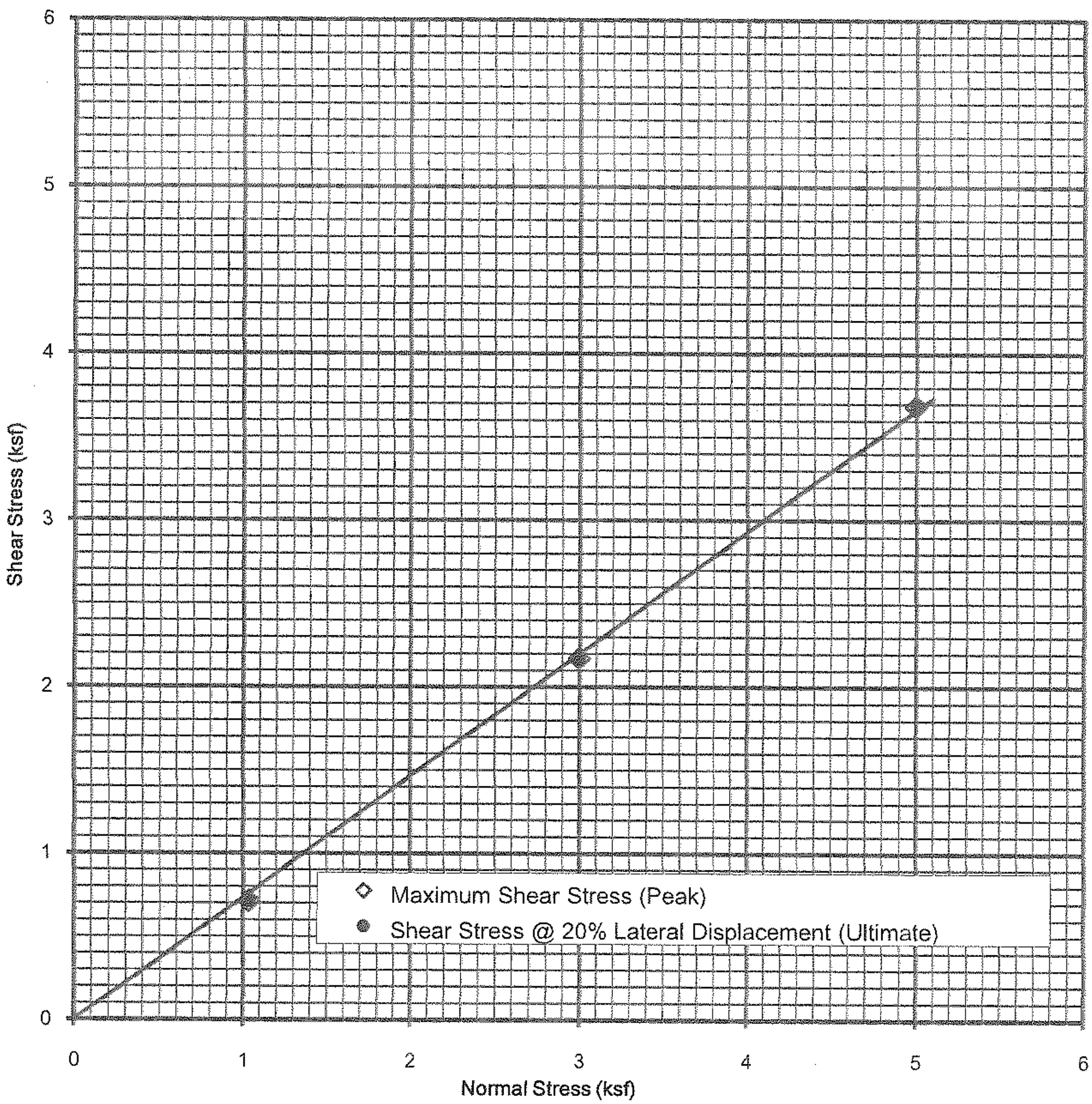
DIRECT SHEAR TEST

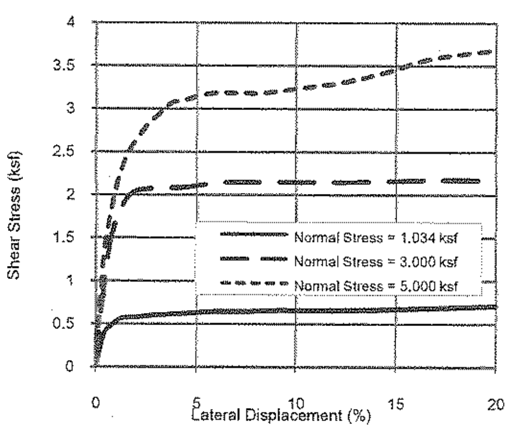
(ASTM-D3080)

Project Name:	Sunshine Canyon LFGTE Facility		**Project No.:**	0148280000	
Sample No.: 3	**Stockpile No.:** 1		**Depth:** ---	**Date:**	11/01-11/02/2010
Soil Description:	Olive Yellow (5Y, 6/8) Silty Sand (SM)			**Tested By:**	LT

				Before Test	After Test Load 1	After Test Load 2	After Test Load 3
Sample Diameter, in:	2.416	Weight of Wet Soil & Ring, gr:		576.38	---	---	---
Normal Stress, ksf:	1,3,5	Weight of Ring, gr:		130.43	---	---	---
Over-burdened @, pcf:		Height of Sample, in:		3.00	0.9958	0.9885	0.9809
Shear Rate, in/min:	0.005	Moisture-	Tare No.:	---	---	---	---
Natural Moisture(x):			Wet Weight and Tare, gr:		155.86	153.36	153.37
Saturated(x):	X		Dry Weight and Tare, gr:	Remolded	130.44	128.76	129.25
Intact(x):			Tare Weight, gr:		0.00	0.00	0.00
Remolded to 109.8 pcf (90% maximum dry density) at 12.5% moisture content (2% over optimum)			Moisture Content, %:	12.5	19.5	19.1	18.7
		Wet Density, pcf:		123.5	131.8	132.3	132.8
		Dry Density, pcf:		109.8	110.3	111.1	111.9
		Saturation %:	S.G. = 2.70 (Assumed)	63.1	99.5	99.7	99.6

Load 1 (KSF): 1.034				Load 2 (KSF): 3.000				Load 3 (KSF): 5.000			
Shear Deflec-tion (in)	Lateral Displace-ment (%)	Load Ring Reading	Shear Stress (KSF)	Shear Deflec-tion (in)	Lateral Displace-ment (%)	Load Ring Reading	Shear Stress (KSF)	Shear Deflec-tion (in)	Lateral Displace-ment (%)	Load Ring Reading	Shear Stress (KSF)
0.0098	0.406	0.0025	0.392	0.0098	0.406	0.0070	0.989	0.0098	0.406	0.0095	1.322
0.0199	0.823	0.0033	0.498	0.0199	0.823	0.0115	1.587	0.0199	0.823	0.0141	1.933
0.0300	1.241	0.0038	0.564	0.0300	1.241	0.0135	1.853	0.0300	1.241	0.0167	2.278
0.0401	1.659	0.0039	0.578	0.0401	1.659	0.0146	1.999	0.0401	1.659	0.0185	2.517
0.0502	2.077	0.0039	0.578	0.0502	2.077	0.0150	2.052	0.0502	2.077	0.0196	2.663
0.0603	2.495	0.0040	0.591	0.0603	2.495	0.0151	2.066	0.0603	2.495	0.0207	2.810
0.0704	2.912	0.0041	0.604	0.0704	2.912	0.0152	2.079	0.0704	2.912	0.0214	2.903
0.0805	3.330	0.0041	0.604	0.0805	3.330	0.0152	2.079	0.0805	3.330	0.0221	2.996
0.0905	3.748	0.0042	0.617	0.0905	3.748	0.0152	2.079	0.0905	3.748	0.0227	3.075
0.1006	4.166	0.0042	0.617	0.1006	4.166	0.0152	2.079	0.1006	4.166	0.0228	3.089
0.1208	5.002	0.0043	0.631	0.1208	5.002	0.0154	2.105	0.1208	5.002	0.0232	3.142
0.1410	5.837	0.0044	0.644	0.1410	5.837	0.0156	2.132	0.1410	5.837	0.0235	3.182
0.1612	6.673	0.0044	0.644	0.1612	6.673	0.0157	2.145	0.1612	6.673	0.0235	3.182
0.1814	7.509	0.0044	0.644	0.1814	7.509	0.0157	2.145	0.1814	7.509	0.0235	3.182
0.2016	8.344	0.0045	0.657	0.2016	8.344	0.0157	2.145	0.2016	8.344	0.0235	3.182
0.2521	10.433	0.0045	0.657	0.2521	10.433	0.0157	2.145	0.2521	10.433	0.0240	3.248
0.3025	12.523	0.0045	0.657	0.3025	12.523	0.0157	2.145	0.3025	12.523	0.0245	3.314
0.3530	14.612	0.0046	0.671	0.3530	14.612	0.0158	2.159	0.3530	14.612	0.0254	3.434
0.4035	16.701	0.0047	0.684	0.4035	16.701	0.0159	2.172	0.4035	16.701	0.0265	3.580
0.4828	19.982	0.0049	0.710	0.4828	19.982	0.0159	2.172	0.4828	19.982	0.0273	3.686
Max. Shear Stress, ksf:			0.710				2.172				3.686
Shear Deflt.@Max Stress,%.:			20.0				20.0				20.0





Sample No.:	3 (Stockpile No. 1)
Sample Depth :	---
Soil Type:	SM
Sample Conditions:	Remolded; Saturated
Remold Dry Density (PCF):	109.8
Remold Moisture Content (%):	12.5

	◇	●
Cohesion (PSF):	0	0
Friction Angle (Degrees):	36	36

AMEC Geomatrix	DIRECT SHEAR TEST SUNSHINE CANYON LFGTE FACILITY Sylmar, California	Project No. 0148280000

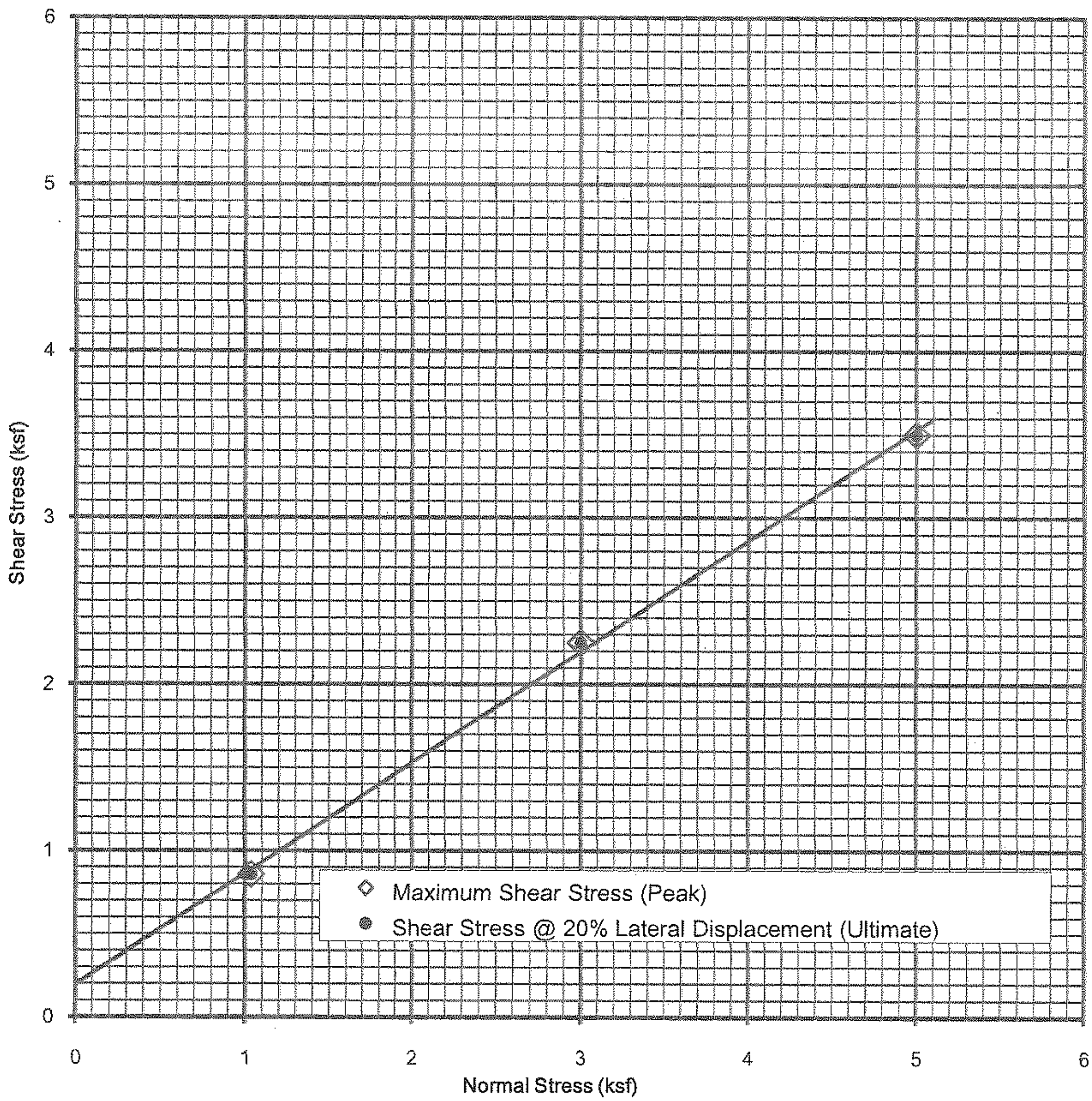
AMEC Geomatrix

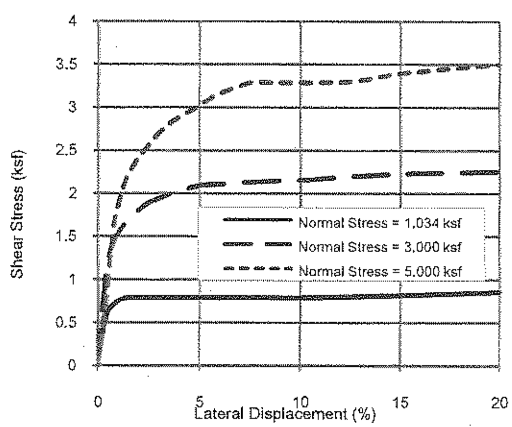
DIRECT SHEAR TEST
(ASTM-D3080)

Project Name:	Sunshine Canyon LFGTE Facility		**Project No.:**	0148280000	
Sample No.:	C-1	**Source:**	Excavated Bedrock Material	**Date:**	2/16-2/17/2011
Soil Description:		Dark Gray (2.5Y, 4/1) Sandy Silt (ML)		**Tested By:**	LT

				Before Test	After Test Load 1	Load 2	Load 3
Sample Diameter, in:	2.416	Weight of Wet Soil & Ring, gr:		567.97	---	---	---
Normal Stress, ksf:	1,3,5	Weight of Ring, gr:		131.05	---	---	---
Over-burdened @, pcf:		Height of Sample, in:		3.00	0.9944	0.9719	0.9683
Shear Rate, in/min:	0.005	Moisture-	Tare No.:	---	---	---	---
Natural Moisture(x):			Wet Weight and Tare, gr:		152.87	149.52	151.28
Saturated(x):	X		Dry Weight and Tare, gr:	Remolded	126.49	125.05	126.72
Intact(x):			Tare Weight, gr:		0.00	0.00	0.00
Remolded to 107.1 pcf (90% maximum dry density) at 13.0% moisture content (2% over optimum)			Moisture Content, %:	13.0	20.9	19.6	19.4
		Wet Density, pcf:		121.0	130.2	131.8	132.0
		Dry Density, pcf:		107.1	107.7	110.2	110.6
		Saturation %:	S.G. = 2.70 (Assumed)	61.2	99.7	99.8	99.9

Load 1 (KSF): 1.034				Load 2 (KSF): 3.000				Load 3 (KSF): 5.000			
Shear Deflec-tion (in)	Lateral Displace-ment (%)	Load Ring Reading	Shear Stress (KSF)	Shear Deflec-tion (in)	Lateral Displace-ment (%)	Load Ring Reading	Shear Stress (KSF)	Shear Deflec-tion (in)	Lateral Displace-ment (%)	Load Ring Reading	Shear Stress (KSF)
0.0098	0.406	0.0040	0.591	0.0098	0.406	0.0079	1.109	0.0098	0.406	0.0055	0.790
0.0199	0.823	0.0050	0.724	0.0199	0.823	0.0105	1.454	0.0199	0.823	0.0121	1.667
0.0300	1.241	0.0054	0.777	0.0300	1.241	0.0117	1.614	0.0300	1.241	0.0152	2.079
0.0401	1.659	0.0055	0.790	0.0401	1.659	0.0126	1.733	0.0401	1.659	0.0168	2.291
0.0502	2.077	0.0055	0.790	0.0502	2.077	0.0132	1.813	0.0502	2.077	0.0179	2.438
0.0603	2.495	0.0055	0.790	0.0603	2.495	0.0138	1.893	0.0603	2.495	0.0188	2.557
0.0704	2.912	0.0055	0.790	0.0704	2.912	0.0141	1.933	0.0704	2.912	0.0197	2.677
0.0805	3.330	0.0055	0.790	0.0805	3.330	0.0144	1.973	0.0805	3.330	0.0204	2.770
0.0905	3.748	0.0055	0.790	0.0905	3.748	0.0147	2.012	0.0905	3.748	0.0210	2.849
0.1006	4.166	0.0055	0.790	0.1006	4.166	0.0149	2.039	0.1006	4.166	0.0214	2.903
0.1208	5.002	0.0055	0.790	0.1208	5.002	0.0153	2.092	0.1208	5.002	0.0223	3.022
0.1410	5.837	0.0055	0.790	0.1410	5.837	0.0154	2.105	0.1410	5.837	0.0232	3.142
0.1612	6.673	0.0055	0.790	0.1612	6.673	0.0155	2.119	0.1612	6.673	0.0238	3.221
0.1814	7.509	0.0055	0.790	0.1814	7.509	0.0156	2.132	0.1814	7.509	0.0243	3.288
0.2016	8.344	0.0055	0.790	0.2016	8.344	0.0157	2.145	0.2016	8.344	0.0243	3.288
0.2521	10.433	0.0055	0.790	0.2521	10.433	0.0158	2.159	0.2521	10.433	0.0243	3.288
0.3025	12.523	0.0056	0.803	0.3025	12.523	0.0161	2.198	0.3025	12.523	0.0244	3.301
0.3530	14.612	0.0057	0.817	0.3530	14.612	0.0163	2.225	0.3530	14.612	0.0250	3.381
0.4035	16.701	0.0058	0.830	0.4035	16.701	0.0164	2.238	0.4035	16.701	0.0254	3.434
0.4828	19.982	0.0060	0.857	0.4828	19.982	0.0165	2.252	0.4828	19.982	0.0259	3.500
Max. Shear Stress, ksf:			0.857				2.252				3.500
Shear Deflt.@Max Stress,%.:			20.0				20.0				20.0





Sample No.:	C-1
Source:	Excavated Bedrock Material
Soil Type:	ML
Sample Conditions:	Remolded; Saturated
Remold Dry Density (PCF)*:	107.1
Remold Moisture Content (%)*:	13.0

	◇	●
Cohesion (PSF):	196	196
Friction Angle (Degrees):	34	34

**Remolded to 90% relative compaction at 2% over optimum moisture*

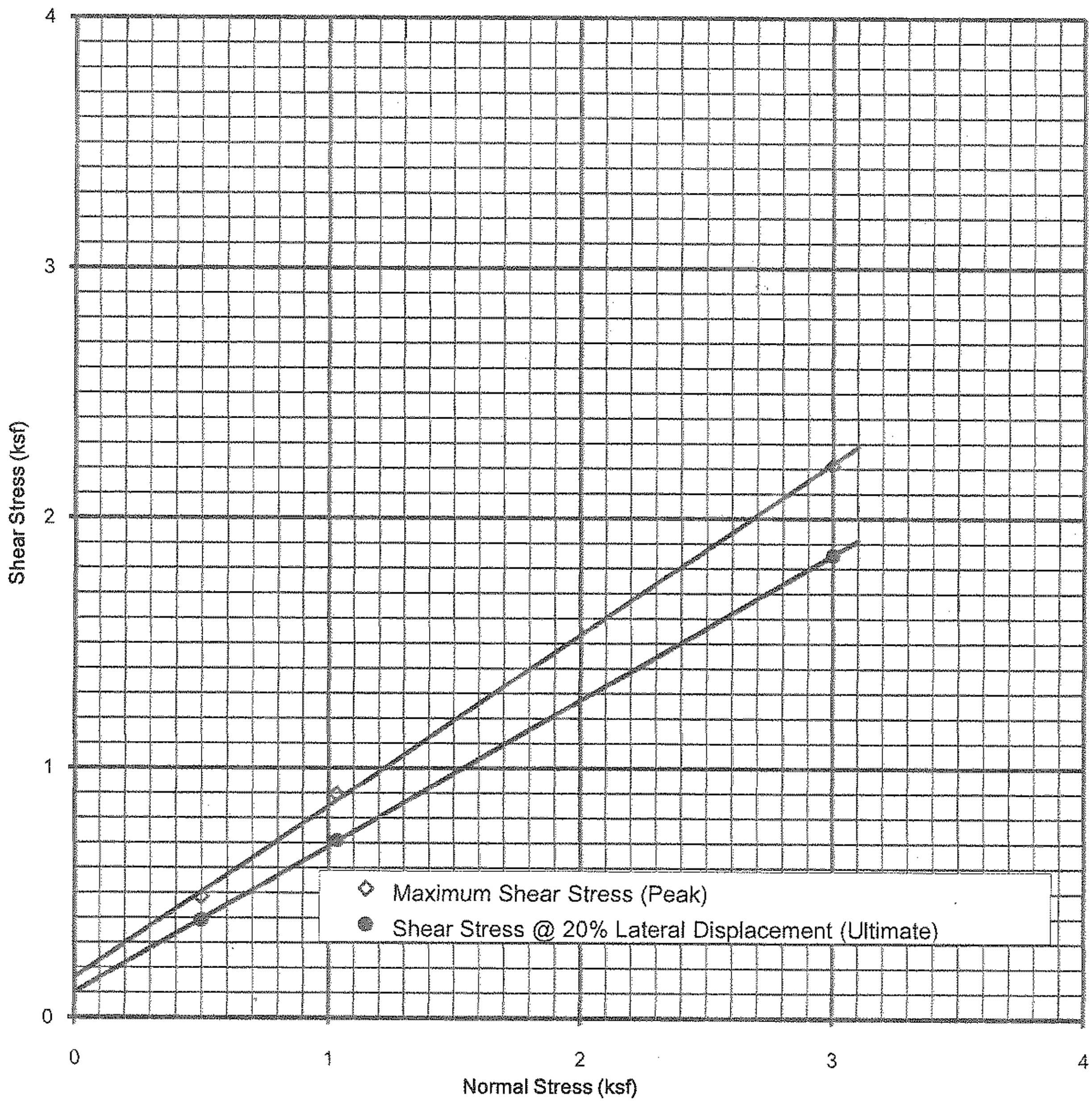
AMEC Geomatrix	DIRECT SHEAR TEST SUNSHINE CANYON LFGTE FACILITY Sylmar, California	Project No. 0148280000

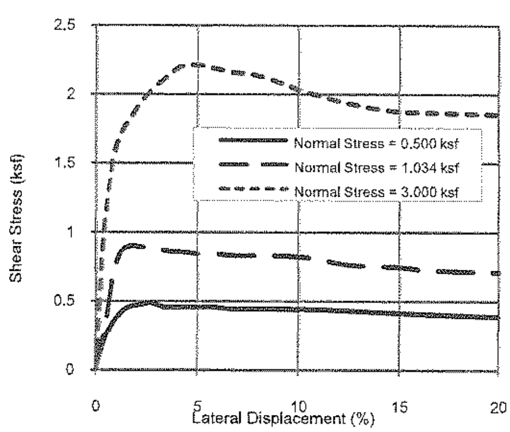
DIRECT SHEAR TEST
(ASTM-D3080)

Project Name:	Sunshine Canyon LFGTE Facility	**Project No.:**	0148280000
Sample No.: C-2	**Source:** Stockpile South of Gas to Energy Site	**Date:**	3/10-3/14/2011
Soil Description:	Very Dark Gray (2.5Y, 3/1) Sandy Lean Clay (CL)	**Tested By:**	LT

					Before Test	After Test Load 1	Load 2	Load 3
Sample Diameter, in:	2.416	Weight of Wet Soil & Ring, gr:			570.74	---	---	---
Normal Stress, ksf:	0.5,1,3	Weight of Ring, gr:			127.91	---	---	---
Over-burdened @, pcf:		Height of Sample, in:			3.00	0.9960	0.9924	0.9781
Shear Rate, in/min:	0.005	Moisture-	Tare No.:		---	---	---	---
Natural Moisture(x):			Wet Weight and Tare, gr:			152.93	153.16	151.09
Saturated(x):	X		Dry Weight and Tare, gr:		Remolded	125.90	126.32	125.45
Intact(x):			Tare Weight, gr:			0.00	0.00	0.00
Remolded to 106.2 pcf (90% maximum dry density) at 15.5% moisture content (2% over optimum)			Moisture Content, %:		15.5	21.5	21.2	20.4
		Wet Density, pcf:			122.7	129.5	129.8	130.8
		Dry Density, pcf:			106.2	106.6	107.0	108.6
		Saturation %:		S.G. = 2.70 (Assumed)	71.3	99.8	99.7	99.9

Load 1 (KSF): 0.500				Load 2 (KSF): 1.034				Load 3 (KSF): 3.000			
Shear Deflec-tion (in)	Lateral Displace-ment (%)	Load Ring Reading	Shear Stress (KSF)	Shear Deflec-tion (in)	Lateral Displace-ment (%)	Load Ring Reading	Shear Stress (KSF)	Shear Deflec-tion (in)	Lateral Displace-ment (%)	Load Ring Reading	Shear Stress (KSF)
0.0098	0.406	0.0013	0.218	0.0104	0.430	0.0018	0.284	0.0098	0.406	0.0071	0.989
0.0199	0.823	0.0022	0.337	0.0205	0.848	0.0048	0.683	0.0199	0.823	0.0110	1.508
0.0300	1.241	0.0028	0.417	0.0306	1.266	0.0061	0.856	0.0300	1.241	0.0125	1.708
0.0401	1.659	0.0031	0.457	0.0407	1.684	0.0064	0.896	0.0401	1.659	0.0133	1.815
0.0502	2.077	0.0032	0.470	0.0508	2.101	0.0064	0.896	0.0502	2.077	0.0140	1.908
0.0603	2.495	0.0033	0.484	0.0609	2.519	0.0063	0.883	0.0603	2.495	0.0146	1.988
0.0704	2.912	0.0033	0.484	0.0710	2.937	0.0062	0.870	0.0704	2.912	0.0151	2.054
0.0805	3.330	0.0031	0.457	0.0811	3.355	0.0062	0.870	0.0805	3.330	0.0155	2.107
0.0905	3.748	0.0031	0.457	0.0912	3.773	0.0061	0.856	0.0905	3.748	0.0159	2.161
0.1006	4.166	0.0031	0.457	0.1013	4.191	0.0061	0.856	0.1006	4.166	0.0162	2.200
0.1208	5.002	0.0031	0.457	0.1214	5.026	0.0060	0.843	0.1208	5.002	0.0163	2.214
0.1410	5.837	0.0031	0.457	0.1416	5.862	0.0060	0.843	0.1410	5.837	0.0161	2.187
0.1612	6.673	0.0030	0.444	0.1618	6.698	0.0059	0.830	0.1612	6.673	0.0159	2.161
0.1814	7.509	0.0030	0.444	0.1820	7.534	0.0059	0.830	0.1814	7.509	0.0158	2.147
0.2016	8.344	0.0030	0.444	0.2022	8.369	0.0059	0.830	0.2016	8.344	0.0156	2.121
0.2521	10.433	0.0030	0.444	0.2527	10.458	0.0058	0.816	0.2521	10.433	0.0148	2.014
0.3025	12.523	0.0029	0.430	0.3031	12.548	0.0054	0.763	0.3025	12.523	0.0142	1.934
0.3530	14.612	0.0028	0.417	0.3536	14.637	0.0053	0.750	0.3530	14.612	0.0138	1.881
0.4035	16.701	0.0027	0.404	0.4041	16.726	0.0051	0.723	0.4035	16.701	0.0137	1.868
0.4828	19.982	0.0026	0.391	0.4834	20.007	0.0050	0.710	0.4828	19.982	0.0136	1.854
Max. Shear Stress, ksf:			0.484				0.896				2.214
Shear Deflt.@Max Stress,%.:			2.9				2.1				5.0





Sample No.:	C-2
Source:	Stockpile South of Gas to Energy Site
Soil Type:	CL
Sample Conditions:	Remolded; Saturated
Remold Dry Density (PCF)*:	106.2
Remold Moisture Content (%)*:	15.5

	◇	●
Cohesion (PSF):	161	101
Friction Angle (Degrees):	34	30

**Remolded to 90% relative compaction at 2% over optimum moisture*

AMEC Geomatrix	DIRECT SHEAR TEST SUNSHINE CANYON LFGTE FACILITY Sylmar, California	Project No. 0148280000

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Table 1 - Laboratory Tests on Soil Sample(s)

AMEC Geomatrix
Sunshine Canyon LFGTE Facility
Your #148280000, SA #11-0165LAB
15-Feb-11

Sample ID

			C-1
Resistivity		**Units**	
as-received		ohm-cm	48,000
saturated		ohm-cm	1,120
pH			7.1
Electrical Conductivity		mS/cm	1.17
Chemical Analyses			
Cations			
calcium	Ca^{2+}	mg/kg	840
magnesium	Mg^{2+}	mg/kg	257
sodium	Na^{1+}	mg/kg	46
potassium	K^{1+}	mg/kg	57
Anions			
carbonate	CO_3^{2-}	mg/kg	ND
bicarbonate	HCO_3^{1-}	mg/kg	64
fluoride	F^{1-}	mg/kg	ND
chloride	Cl^{1-}	mg/kg	1.6
sulfate	SO_4^{2-}	mg/kg	2,371
phosphate	PO_4^{3-}	mg/kg	ND
Other Tests			
ammonium	NH_4^{1+}	mg/kg	22
nitrate	NO_3^{1-}	mg/kg	ND
sulfide	S^{2-}	qual	na
Redox		mV	na

Electrical conductivity in millisiemens/cm and chemical analysis were made on a 1:5 soil-to-water extract.
mg/kg = milligrams per kilogram (parts per million) of dry soil.
Redox = oxidation-reduction potential in millivolts
ND = not detected
na = not analyzed

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Table 1 - Laboratory Tests on Soil Samples

AMEC Geomatrix, Inc.
Sunshine Canyon LFGTE Facility
Your #0148280000, SA #11-0247LAB
10-Mar-11

Sample ID

			C-2
Resistivity		**Units**	
as-received		ohm-cm	6,400
saturated		ohm-cm	640
pH			7.2
Electrical Conductivity		mS/cm	1.32
Chemical Analyses			
Cations			
calcium	Ca^{2+}	mg/kg	838
magnesium	Mg^{2+}	mg/kg	251
sodium	Na^{1+}	mg/kg	159
potassium	K^{1+}	mg/kg	98
Anions			
carbonate	CO_3^{2-}	mg/kg	ND
bicarbonate	HCO_3^{1-}	mg/kg	177
fluoride	F^{1-}	mg/kg	ND
chloride	Cl^{1-}	mg/kg	5.7
sulfate	SO_4^{2-}	mg/kg	2,399
phosphate	PO_4^{3-}	mg/kg	ND
Other Tests			
ammonium	NH_4^{1+}	mg/kg	35
nitrate	NO_3^{1-}	mg/kg	1.9
sulfide	S^{2-}	qual	Positive
Redox		mV	-68

Electrical conductivity in millisiemens/cm and chemical analysis were made on a 1:5 soil-to-water extract.
mg/kg = milligrams per kilogram (parts per million) of dry soil.
Redox = oxidation-reduction potential in millivolts
ND = not detected
na = not analyzed

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Table 1 - Laboratory Tests on Soil Sample(s)

AMEC Geomatrix
Sunshine Canyon Landfill
Your #0148280000, SA #10-1146LAB
4-Nov-10

Sample ID			Sample #1 Stockpile # 2	Sample #2 Stockpile #2	Sample #3 Stockpile #1
Resistivity		**Units**			
as-received		ohm-cm	3,080	2,520	2,160
saturated		ohm-cm	1,000	680	1,000
pH			6.3	6.3	6.4
Electrical Conductivity		mS/cm	0.83	1.30	0.23
Chemical Analyses					
Cations					
calcium	Ca^{2+}	mg/kg	463	915	114
magnesium	Mg^{2+}	mg/kg	203	299	44
sodium	Na^{1+}	mg/kg	107	90	25
potassium	K^{1+}	mg/kg	45	97	13
Anions					
carbonate	CO_3^{2-}	mg/kg	ND	ND	ND
bicarbonate	HCO_3^{1-}	mg/kg	61	49	24
fluoride	F^{1-}	mg/kg	0.9	ND	1.7
chloride	Cl^{1-}	mg/kg	8.9	4.7	3.2
sulfate	SO_4^{2-}	mg/kg	1,720	2,540	462
phosphate	PO_4^{3-}	mg/kg	ND	ND	ND
Other Tests					
ammonium	NH_4^{1+}	mg/kg	3.2	13	ND
nitrate	NO_3^{1-}	mg/kg	1.0	1.0	2.3
sulfide	S^{2-}	qual	na	na	na
Redox		mV	na	na	na

Electrical conductivity in millisiemens/cm and chemical analysis were made on a 1:5 soil-to-water extract.
mg/kg = milligrams per kilogram (parts per million) of dry soil.
Redox = oxidation-reduction potential in millivolts
ND = not detected
na = not analyzed

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APPENDIX C

Aerial Photographs


©2011 Google
Image © 2011 DigitalGlobe
493 ft
Imagery Date: 11/2/2005
34°20'07.08" N 118°31'10.26" W elev 1926 ft
©2010 Google
Eye alt 4014 ft

11/2/2005


© 2011 Google
Image U.S. Geological Survey
©2010 Google
493 ft
Imagery Date: 3/15/2006
34°20'07.08" N 118°31'10.26" W elev 1926 ft
Eye alt 4014 ft

3/15/2006

© 2011 Google
Image © 2011 DigitalGlobe
491 ft
©2010 Google
Imagery Date: 8/25/2006
34°20'07.90" N 118°31'10.64" W elev 1901 ft
Eye alt 4014 ft

8/25/2006


© 2011 Google
Image LAR-IAC
©2010 Google
493 ft
Imagery Date: 1/8/2008
34°20'07.08" N 118°31'10.26" W elev 1926 ft
Eye alt 4014 ft

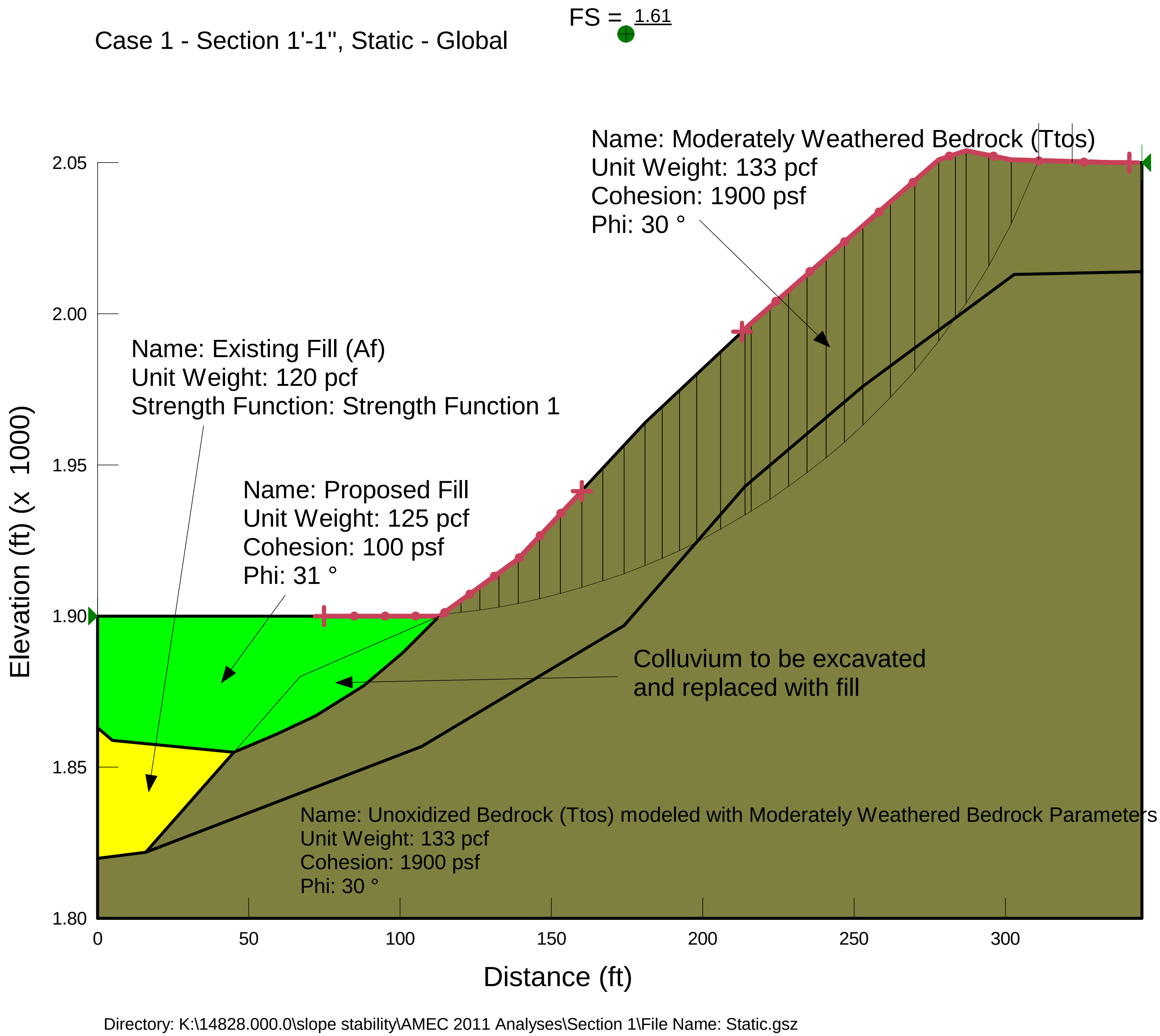
7/27/2008

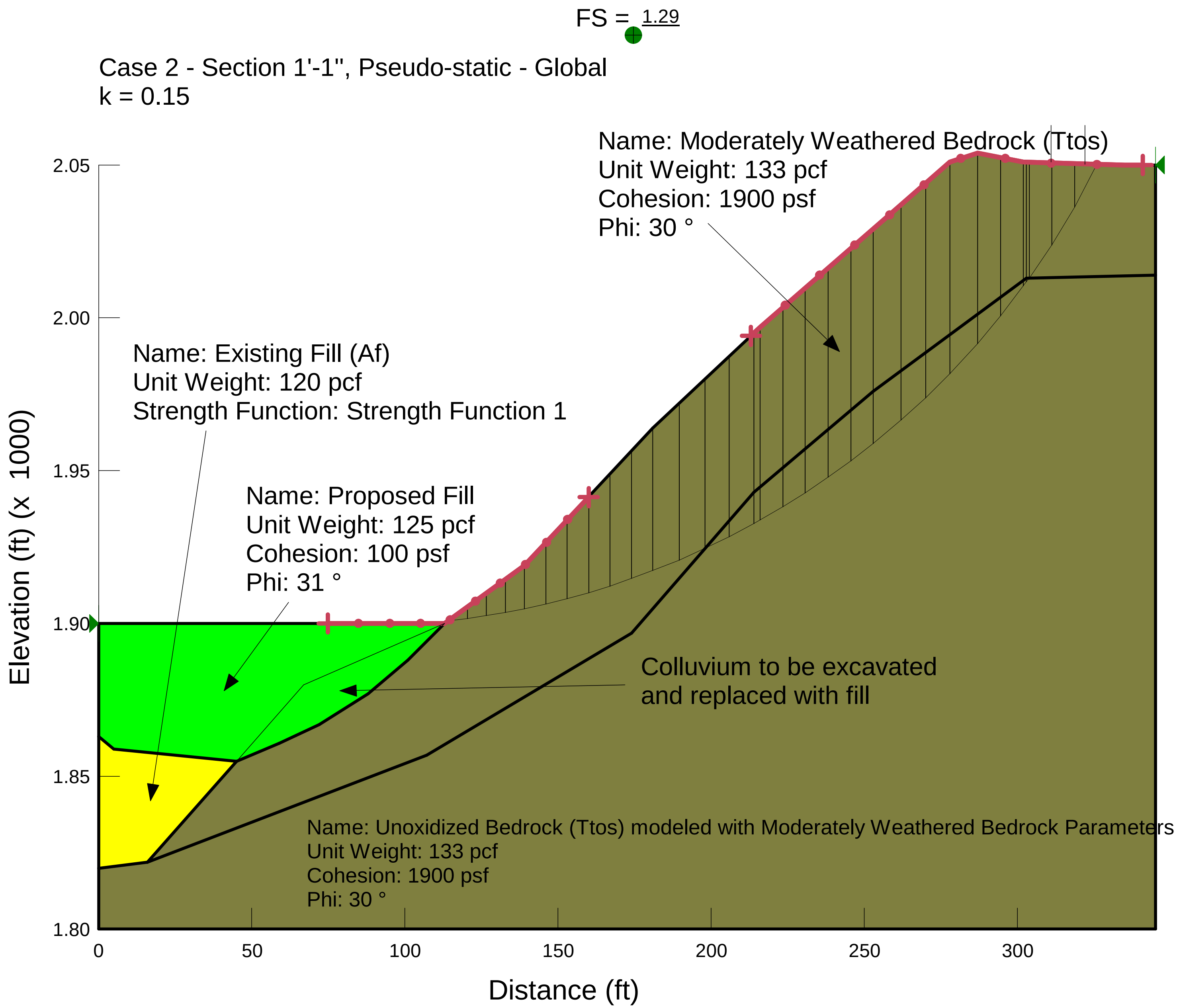

© 2011 Google
493 ft
©2010 Google
Imagery Date: 11/14/2009
34°20'07.08" N 118°31'10.26" W elev 1926 ft
Eye alt 4014 ft

11/14/2009

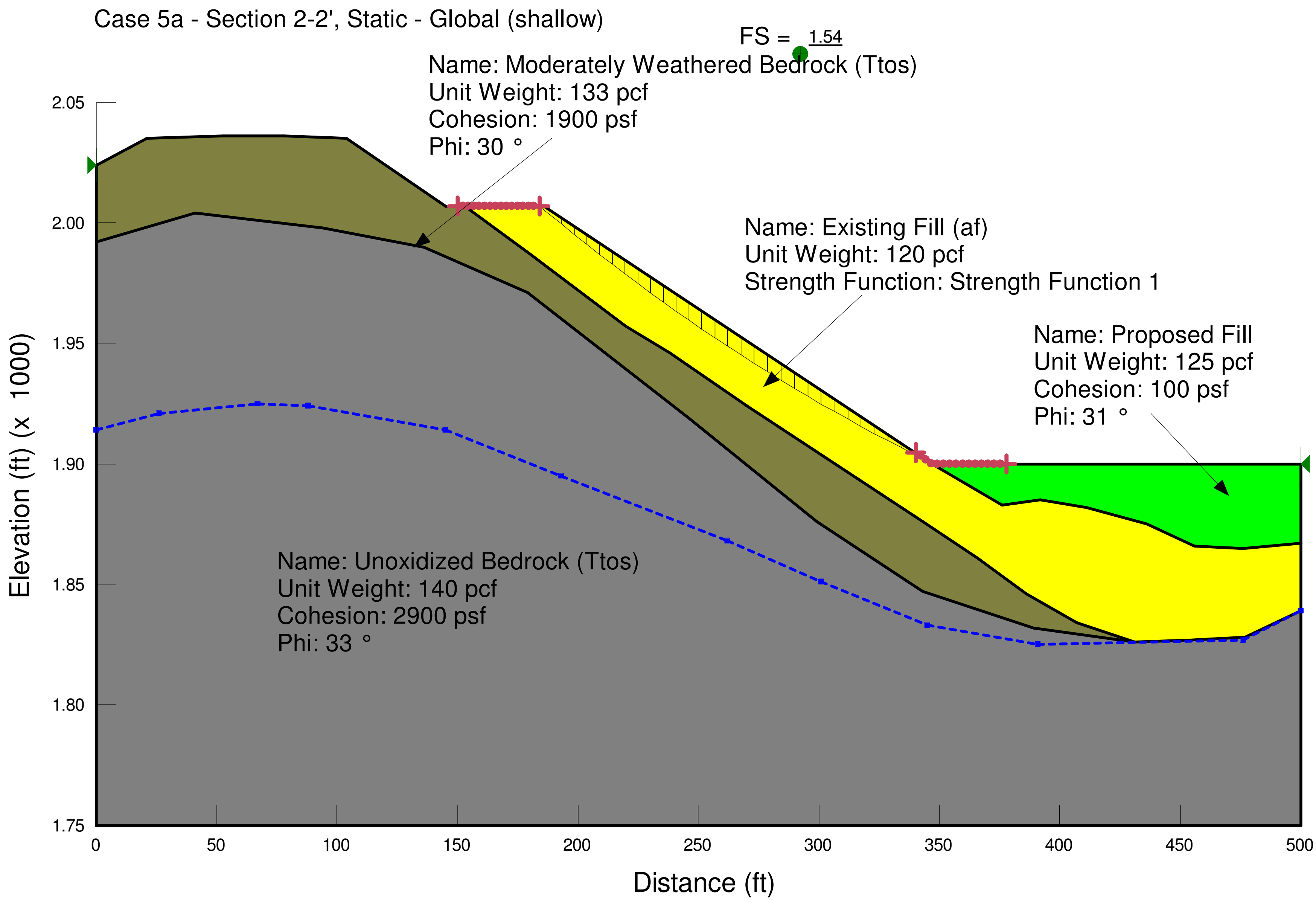
APPENDIX D

Slope Stability Analysis Results

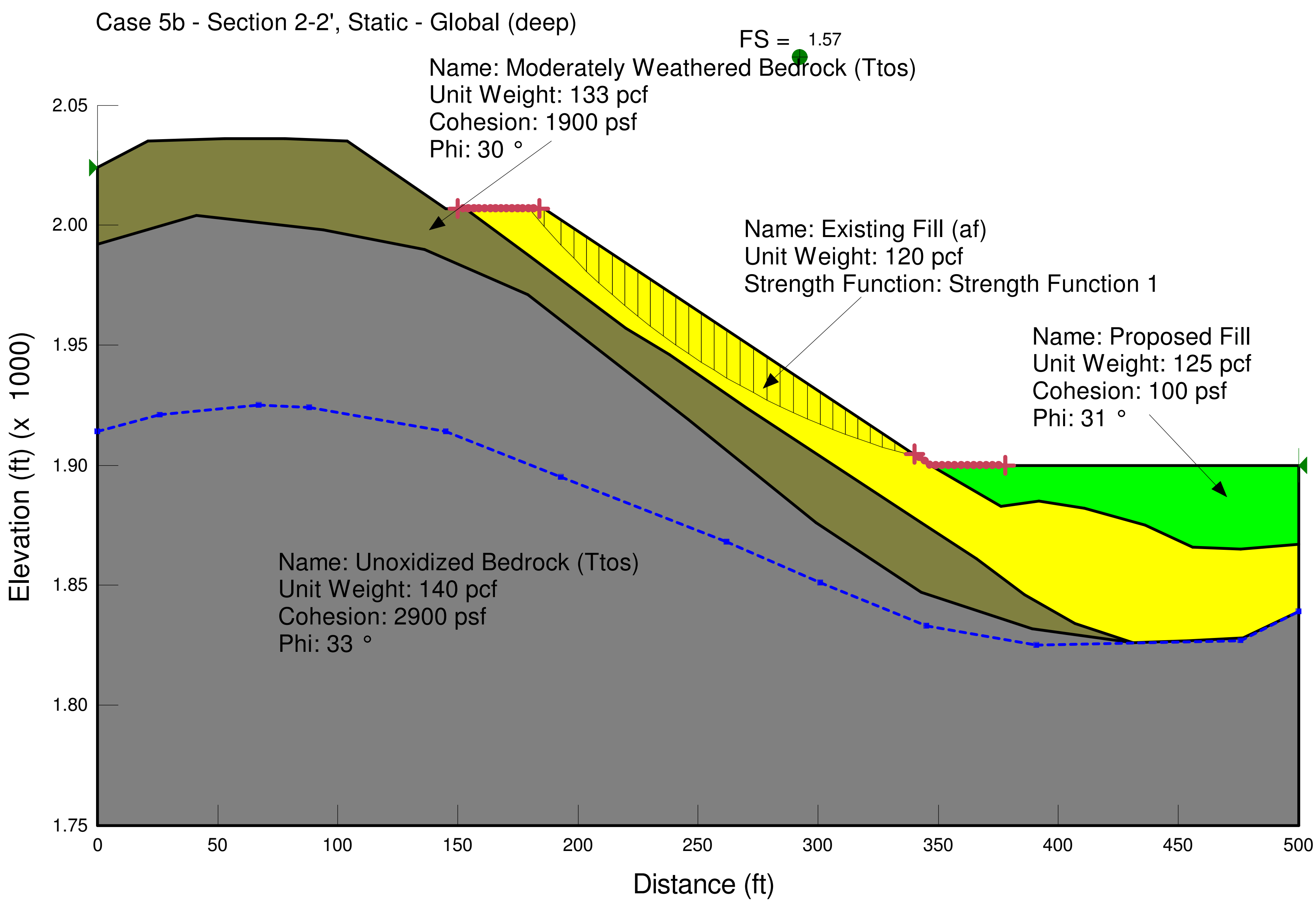

FS = 1.61
Case 1 - Section 1'-1", Static - Global
Name: Moderately Weathered Bedrock (Ttos)
Unit Weight: 133 pcf
Cohesion: 1900 psf
Phi: 30 °
Name: Existing Fill (Af)
Unit Weight: 120 pcf
Strength Function: Strength Function 1
Name: Proposed Fill
Unit Weight: 125 pcf
Cohesion: 100 psf
Phi: 31 °
Colluvium to be excavated
and replaced with fill
Name: Unoxidized Bedrock (Ttos) modeled with Moderately Weathered Bedrock Parameters
Unit Weight: 133 pcf
Cohesion: 1900 psf
Phi: 30 °
Elevation (ft) (x 1000)
2.05
2.00
1.95
1.90
1.85
1.80
0
50
100
150
200
250
300
Distance (ft)
Directory: K:\14828.000.0\slope stability\AMEC 2011 Analyses\Section 1\File Name: Static.gsz



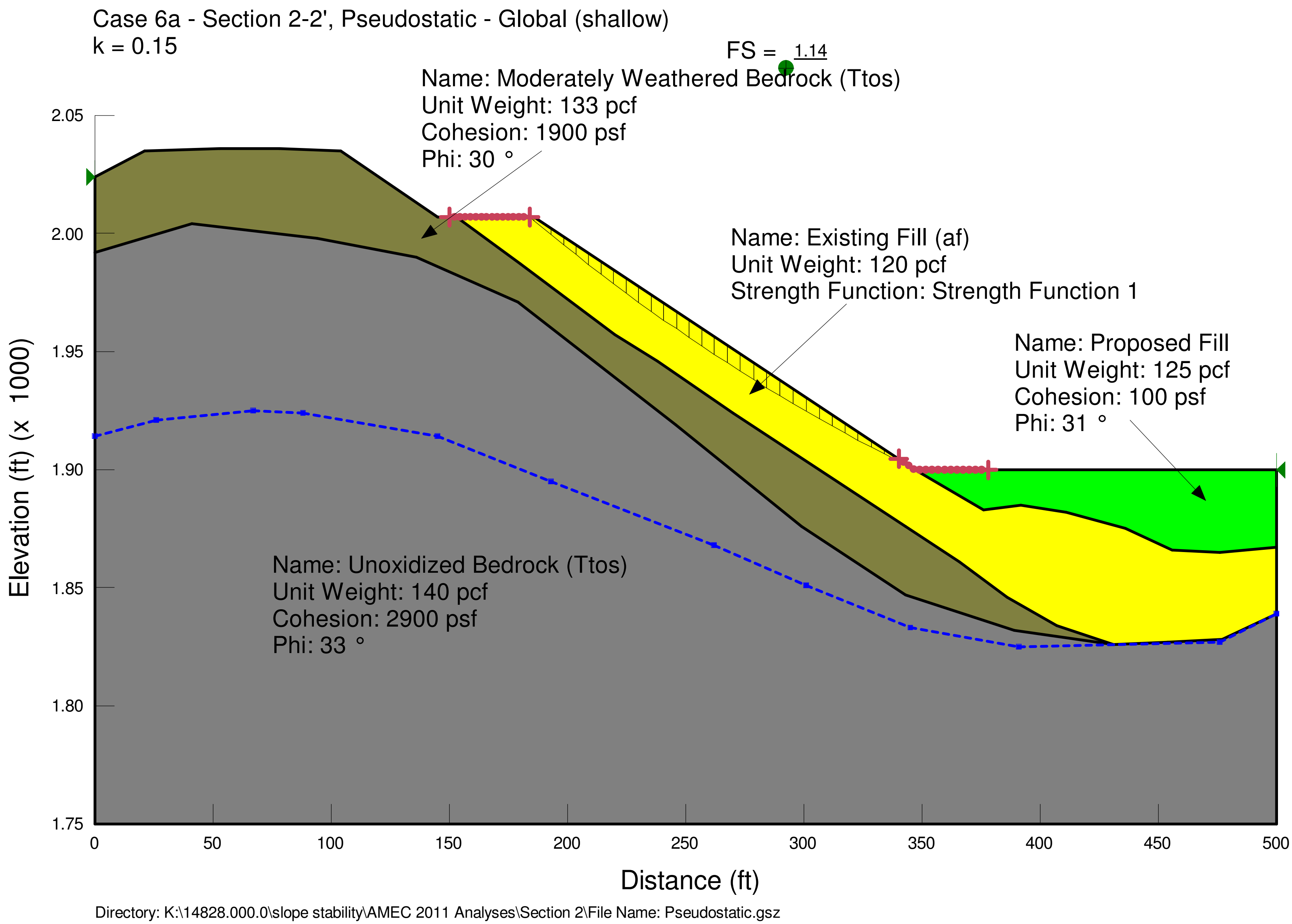
Directory: K:\14828.000.0\slope stability\AMEC 2011 Analyses\Section 1\File Name: Pseudostatic.gsz



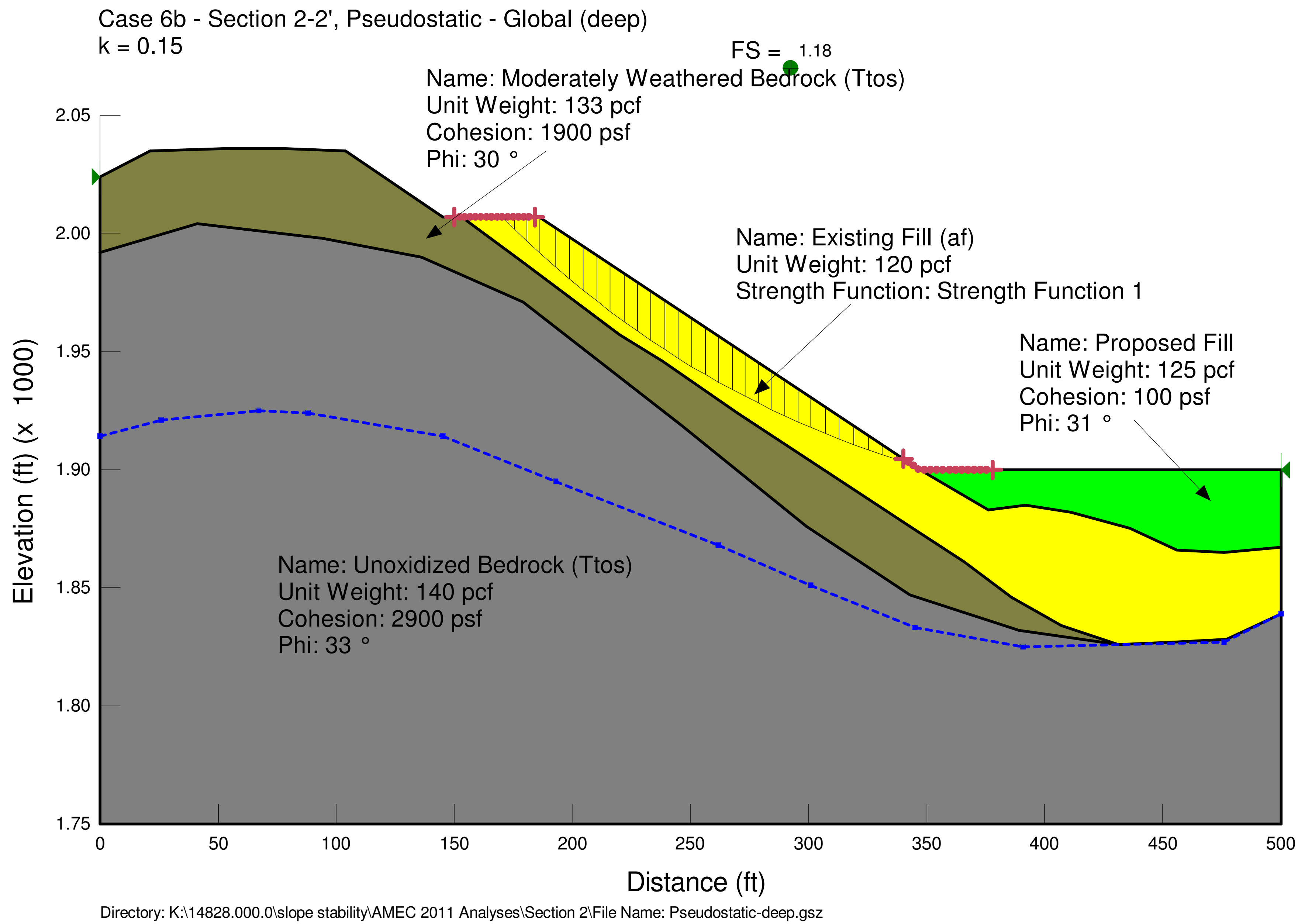
Directory: K:\14828.000.0\slope stability\AMEC 2011 Analyses\Section 2\File Name: Static.gsz

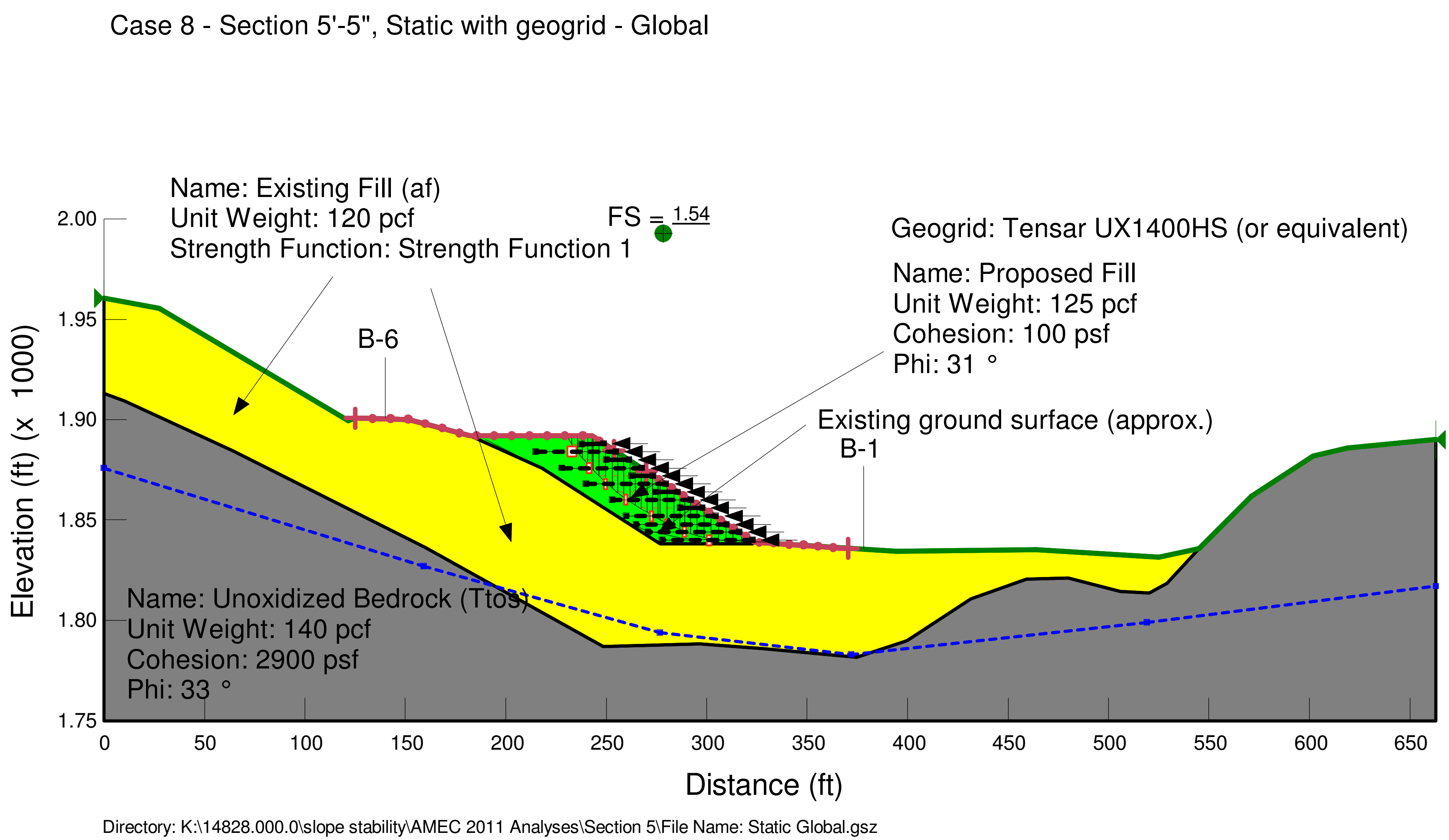


Directory: K:\14828.000.0\slope stability\AMEC 2011 Analyses\Section 2\File Name: Static-deep.gsz



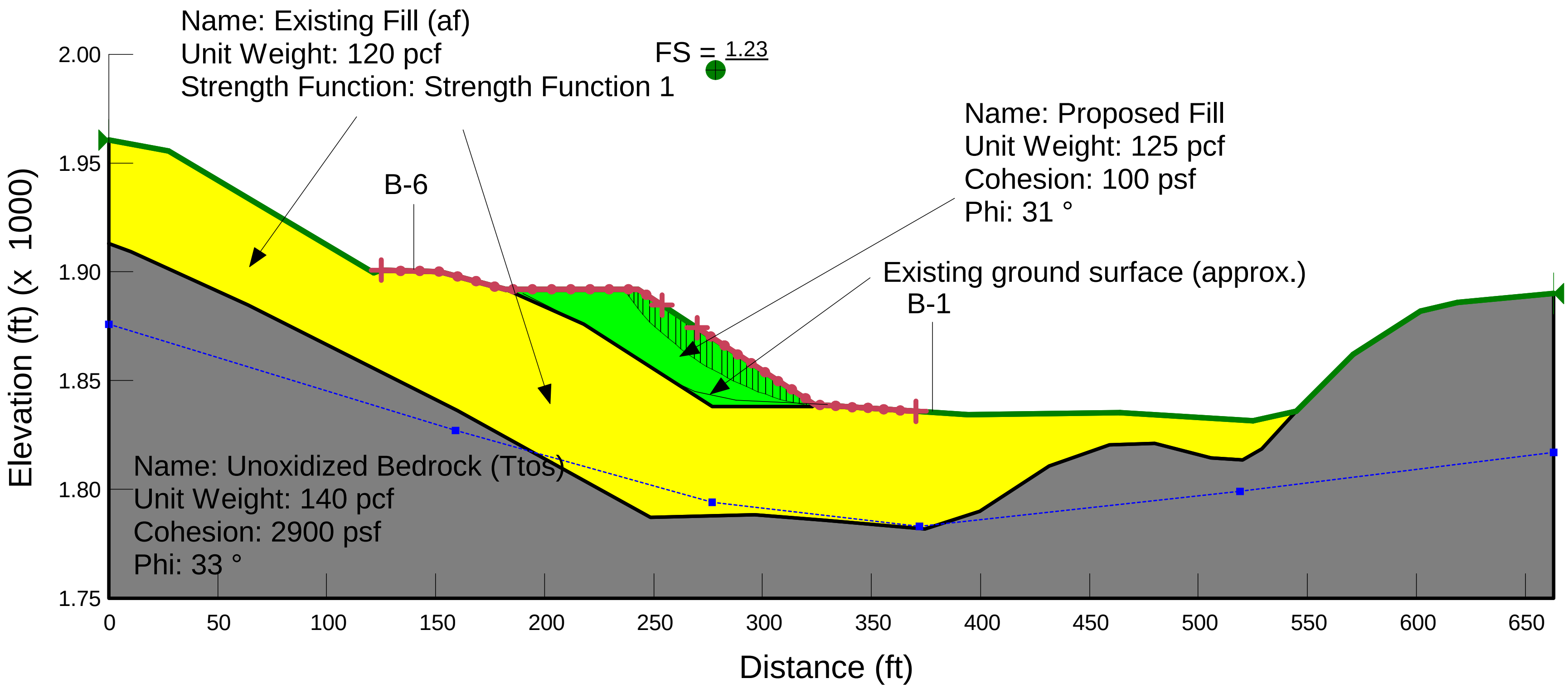
Case 6a - Section 2-2', Pseudostatic - Global (shallow)
k = 0.15
FS = 1.14
Name: Moderately Weathered Bedrock (Ttos)
Unit Weight: 133 pcf
Cohesion: 1900 psf
Phi: 30 °
Name: Existing Fill (af)
Unit Weight: 120 pcf
Strength Function: Strength Function 1
Name: Proposed Fill
Unit Weight: 125 pcf
Cohesion: 100 psf
Phi: 31 °
Name: Unoxidized Bedrock (Ttos)
Unit Weight: 140 pcf
Cohesion: 2900 psf
Phi: 33 °
Elevation (ft) (x 1000)
2.05
2.00
1.95
1.90
1.85
1.80
1.75
0
50
100
150
200
250
300
350
400
450
500
Distance (ft)
Directory: K:\14828.000.0\slope stability\AMEC 2011 Analyses\Section 2\File Name: Pseudostatic.gsz


Case 6b - Section 2-2', Pseudostatic - Global (deep)
k = 0.15
FS = 1.18
Name: Moderately Weathered Bedrock (Ttos)
Unit Weight: 133 pcf
Cohesion: 1900 psf
Phi: 30 °
Name: Existing Fill (af)
Unit Weight: 120 pcf
Strength Function: Strength Function 1
Name: Proposed Fill
Unit Weight: 125 pcf
Cohesion: 100 psf
Phi: 31 °
Name: Unoxidized Bedrock (Ttos)
Unit Weight: 140 pcf
Cohesion: 2900 psf
Phi: 33 °
Elevation (ft) (x 1000)
2.05
2.00
1.95
1.90
1.85
1.80
1.75
0
50
100
150
200
250
300
350
400
450
500
Distance (ft)
Directory: K:\14828.000.0\slope stability\AMEC 2011 Analyses\Section 2\File Name: Pseudostatic-deep.gsz



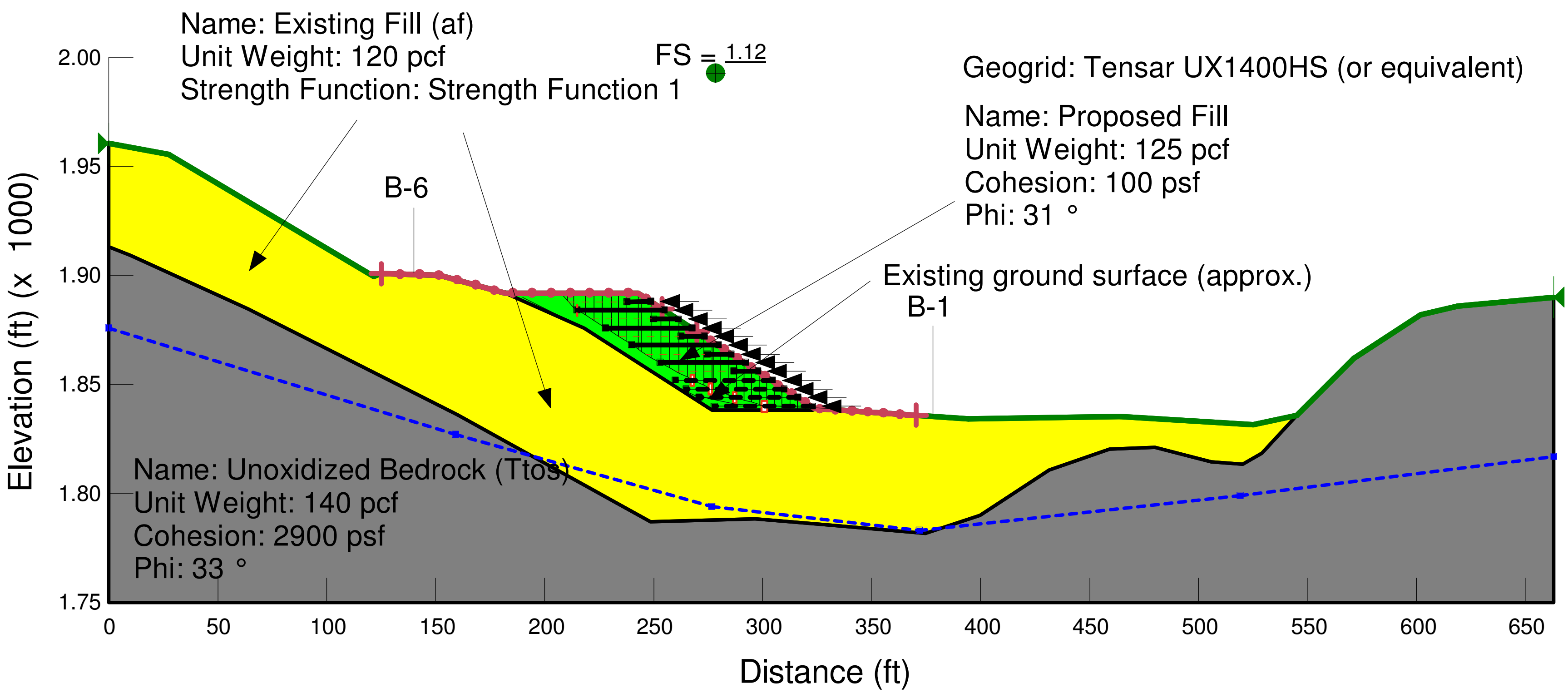
Case 8 - Section 5'-5", Static with geogrid - Global
Name: Existing Fill (af)
Unit Weight: 120 pcf
Strength Function: Strength Function 1
FS = 1.54
Geogrid: Tensar UX1400HS (or equivalent)
Name: Proposed Fill
Unit Weight: 125 pcf
Cohesion: 100 psf
Phi: 31 °
B-6
Existing ground surface (approx.)
B-1
Name: Unoxidized Bedrock (Ttos)
Unit Weight: 140 pcf
Cohesion: 2900 psf
Phi: 33 °
Elevation (ft) (x 1000)
2.00
1.95
1.90
1.85
1.80
1.75
0
50
100
150
200
250
300
350
400
450
500
550
600
650
Distance (ft)
Directory: K:\14828.000.0\slope stability\AMEC 2011 Analyses\Section 5\File Name: Static Global.gsz

Case 8 (NO REINFORCEMENT) - Section 5'-5", Static without geogrid - Global

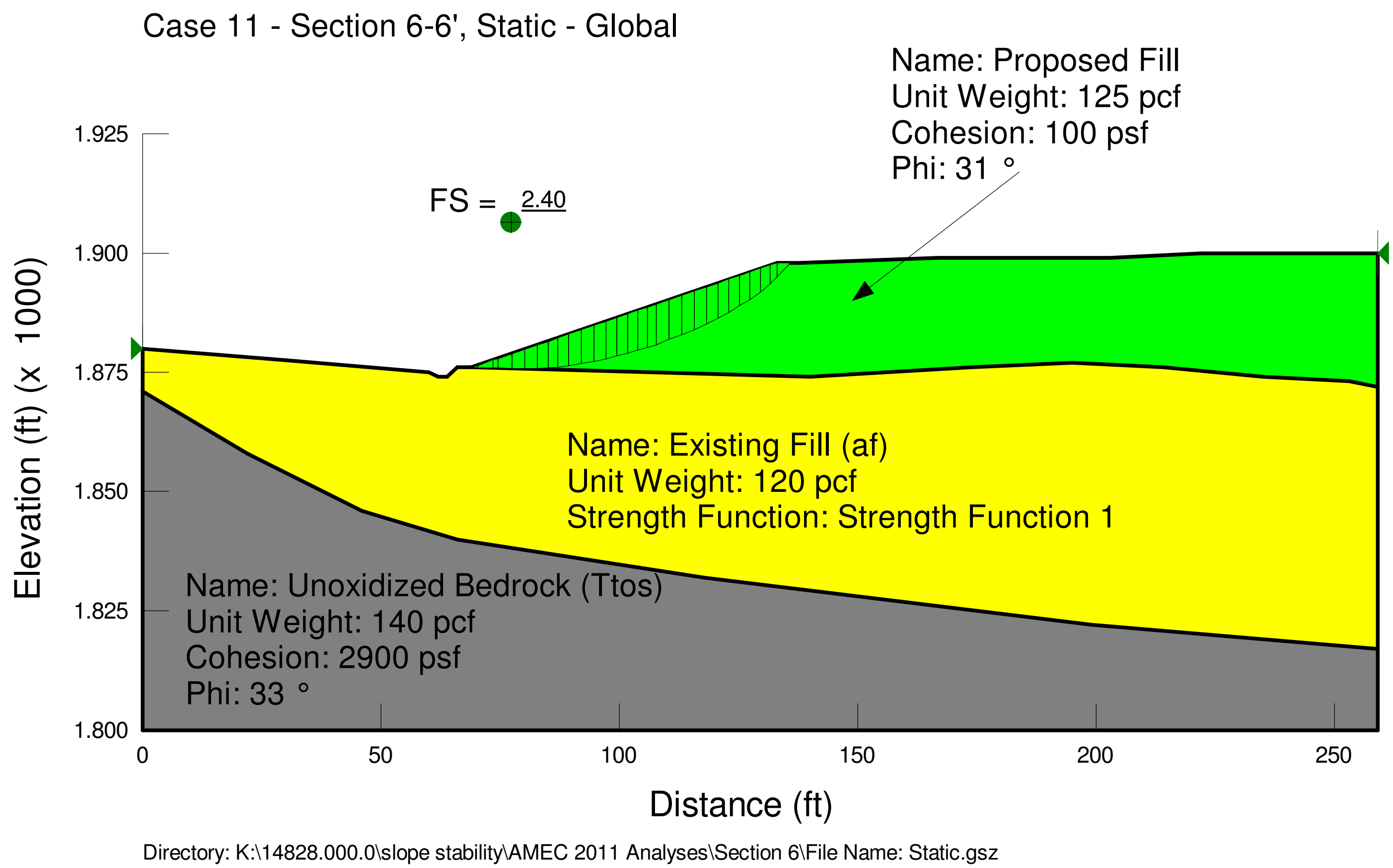


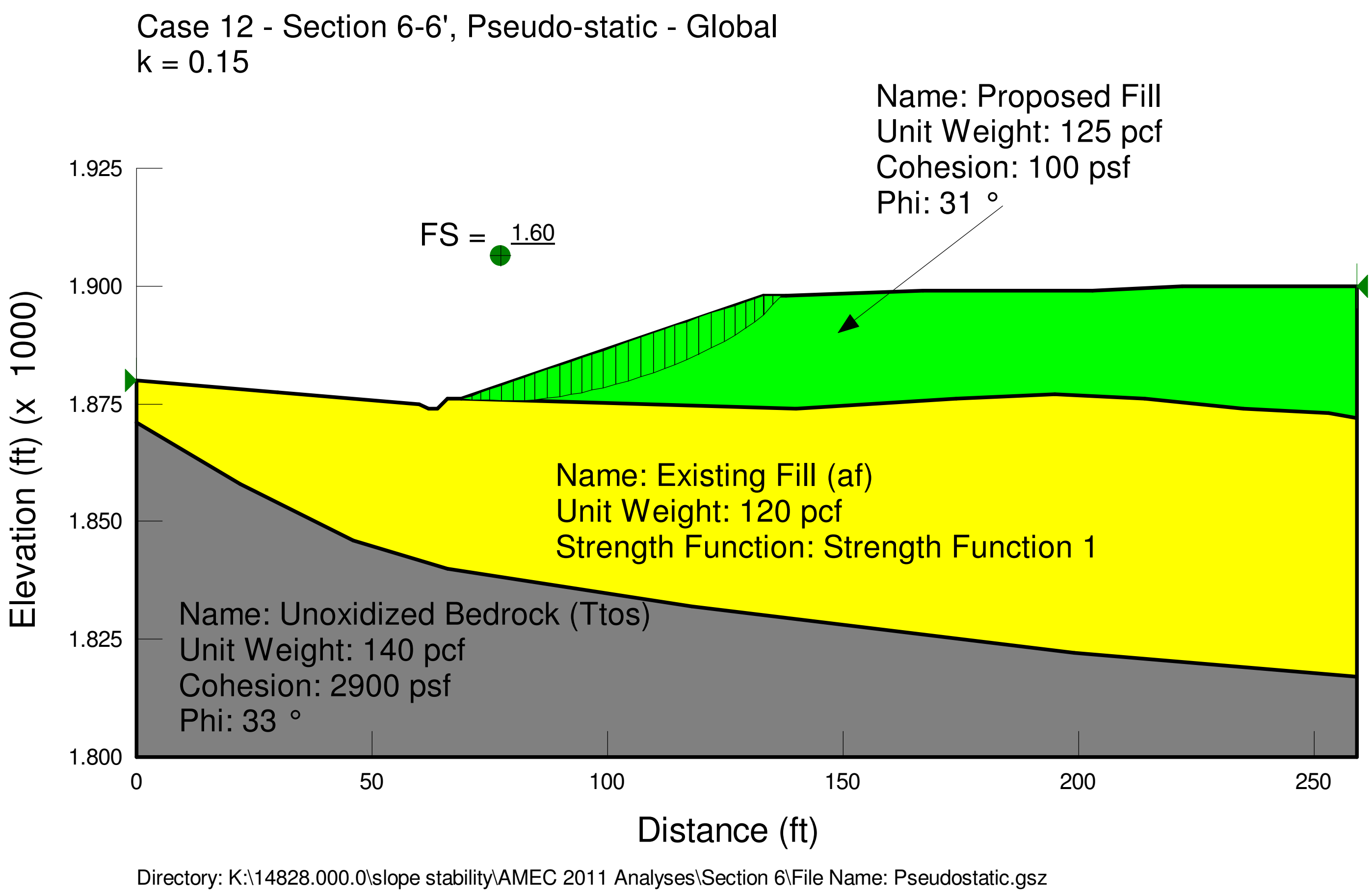
Directory: K:\14828.000.0\slope stability\AMEC 2011 Analyses\Section 5\File Name: Case 8 NO REINFORCEMENT Static Global.gsz

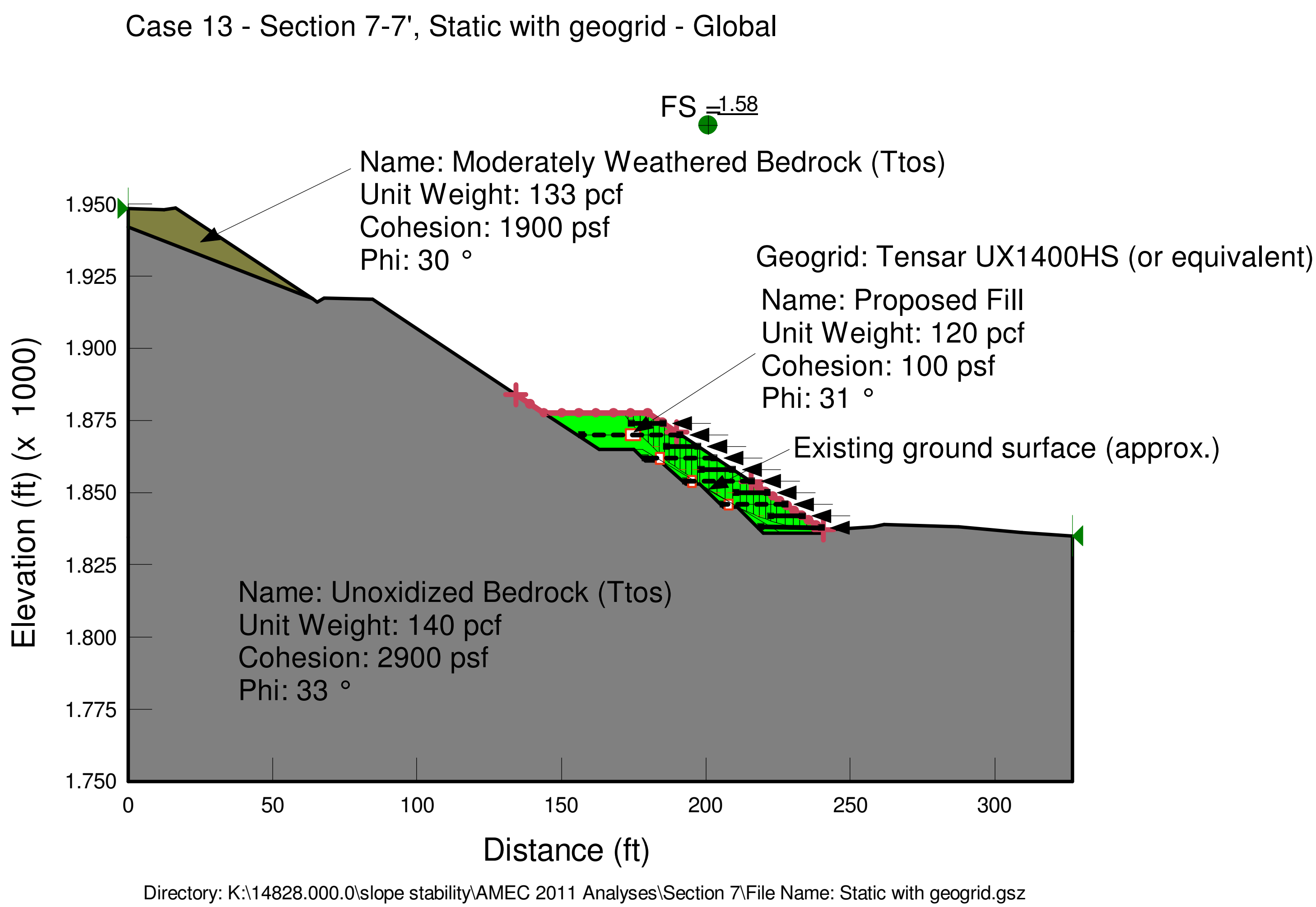
Case 9 - Section 5'-5", Pseudostatic with geogrid - Global
k = 0.15



Directory: K:\14828.000.0\slope stability\AMEC 2011 Analyses\Section 5\File Name: Pseudostatic.gsz

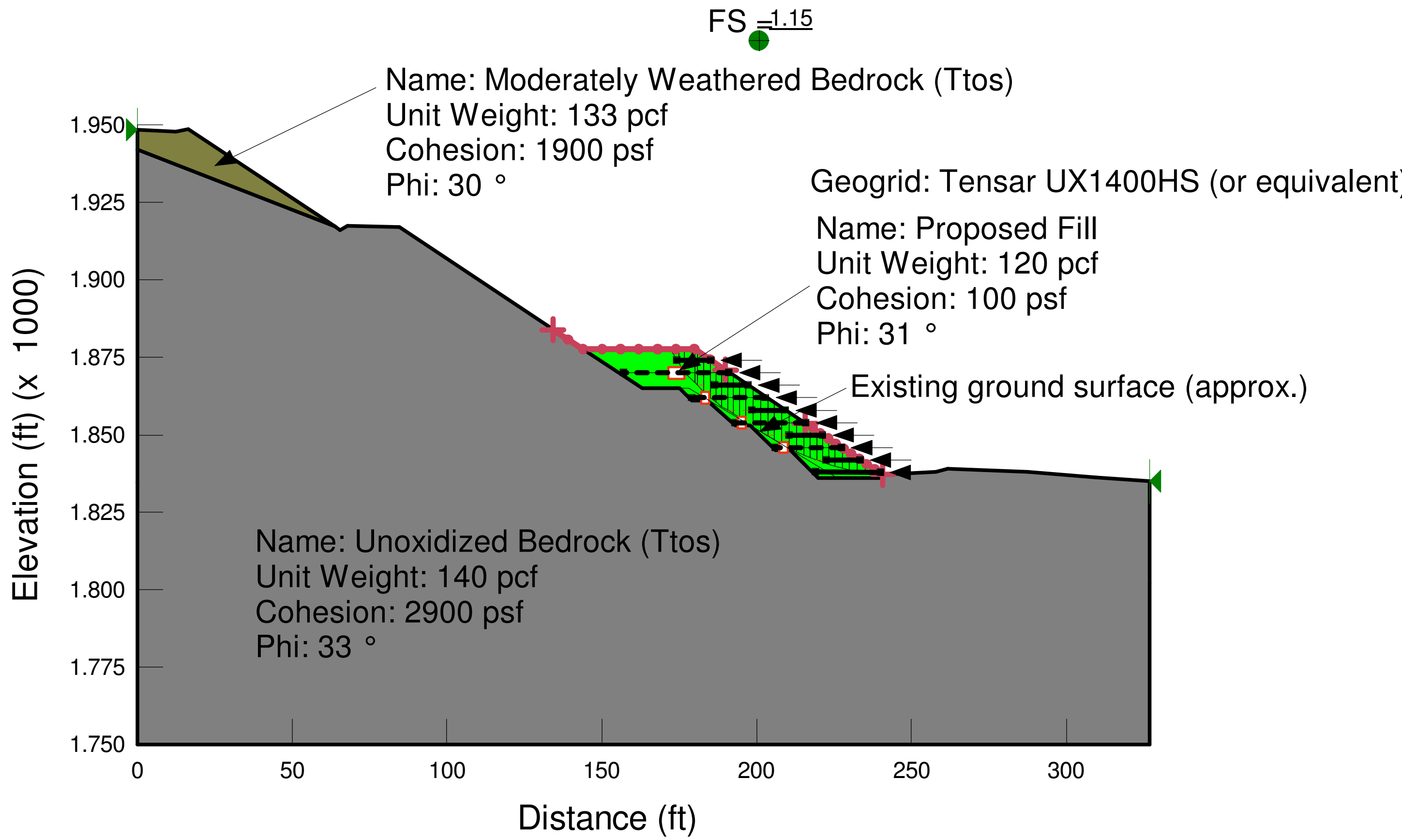

Case 11 - Section 6-6', Static - Global
Name: Proposed Fill
Unit Weight: 125 pcf
Cohesion: 100 psf
Phi: 31 °
FS = 2.40
Name: Existing Fill (af)
Unit Weight: 120 pcf
Strength Function: Strength Function 1
Name: Unoxidized Bedrock (Ttos)
Unit Weight: 140 pcf
Cohesion: 2900 psf
Phi: 33 °
Elevation (ft) (x 1000)
1.925
1.900
1.875
1.850
1.825
1.800
0
50
100
150
200
250
Distance (ft)
Directory: K:\14828.000.0\slope stability\AMEC 2011 Analyses\Section 6\File Name: Static.gsz


Case 12 - Section 6-6', Pseudo-static - Global
k = 0.15
Name: Proposed Fill
Unit Weight: 125 pcf
Cohesion: 100 psf
Phi: 31 °
FS = 1.60
Name: Existing Fill (af)
Unit Weight: 120 pcf
Strength Function: Strength Function 1
Name: Unoxidized Bedrock (Ttos)
Unit Weight: 140 pcf
Cohesion: 2900 psf
Phi: 33 °
1.925
1.900
1.875
1.850
1.825
1.800
Elevation (ft) (x 1000)
0
50
100
150
200
250
Distance (ft)
Directory: K:\14828.000.0\slope stability\AMEC 2011 Analyses\Section 6\File Name: Pseudostatic.gsz

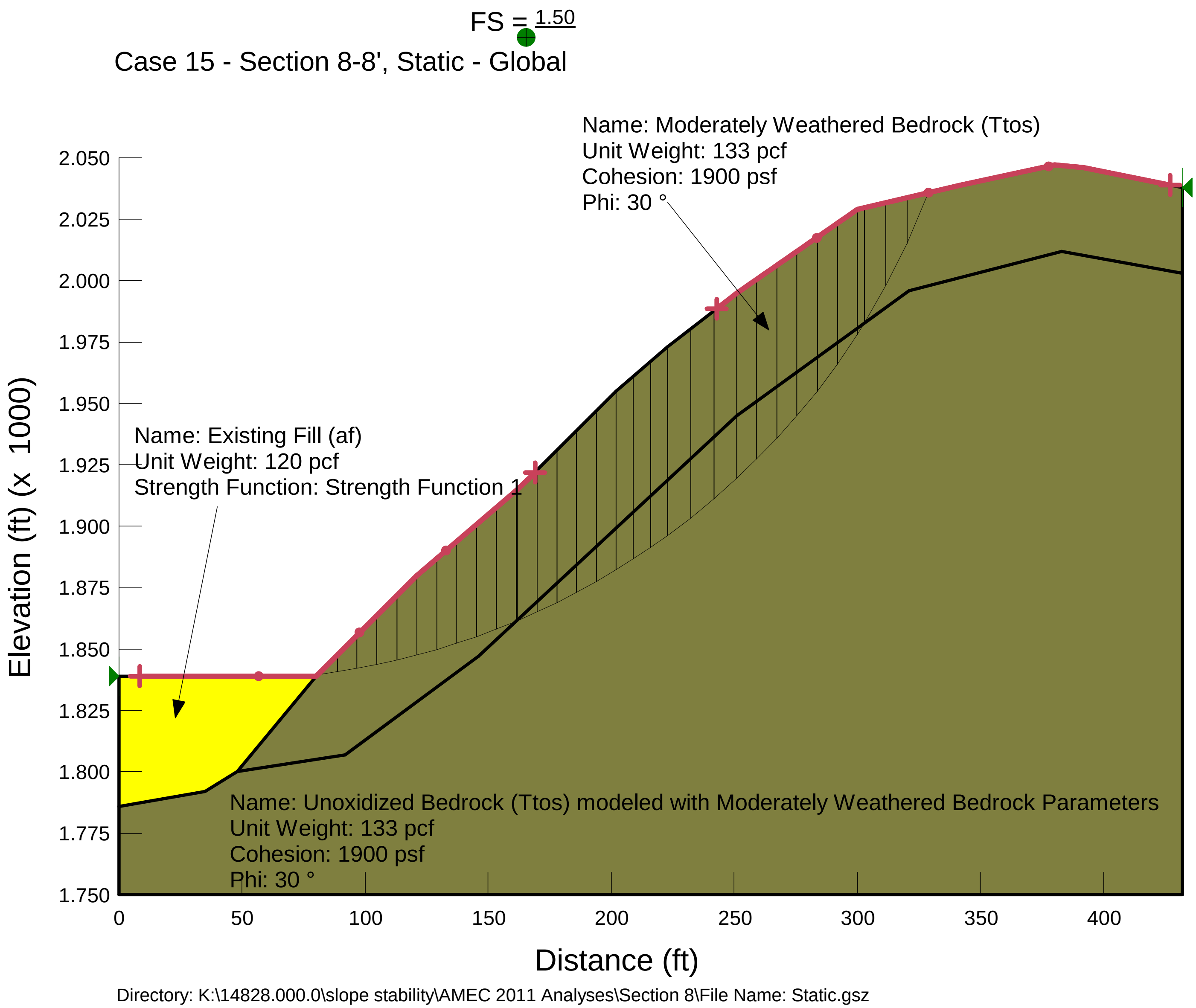


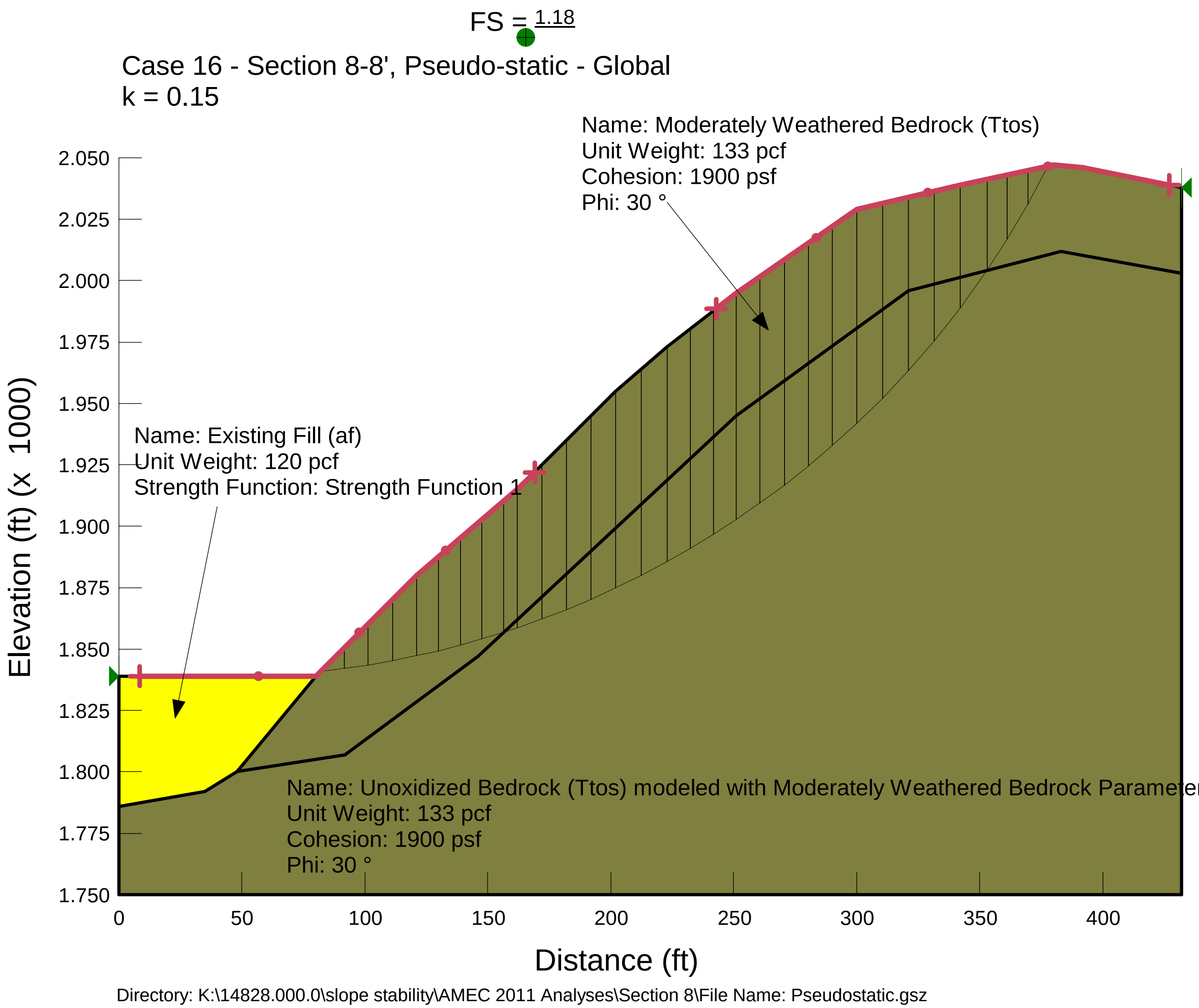
Case 13 - Section 7-7', Static with geogrid - Global
FS = 1.58
Name: Moderately Weathered Bedrock (Ttos)
Unit Weight: 133 pcf
Cohesion: 1900 psf
Phi: 30 °
Geogrid: Tensar UX1400HS (or equivalent)
Name: Proposed Fill
Unit Weight: 120 pcf
Cohesion: 100 psf
Phi: 31 °
Existing ground surface (approx.)
Name: Unoxidized Bedrock (Ttos)
Unit Weight: 140 pcf
Cohesion: 2900 psf
Phi: 33 °
Elevation (ft) (x 1000)
1.950
1.925
1.900
1.875
1.850
1.825
1.800
1.775
1.750
0
50
100
150
200
250
300
Distance (ft)
Directory: K:\14828.000.0\slope stability\AMEC 2011 Analyses\Section 7\File Name: Static with geogrid.gsz

Case 14 - Section 7-7', Pseudostatic with geogrid - Global
k = 0.15

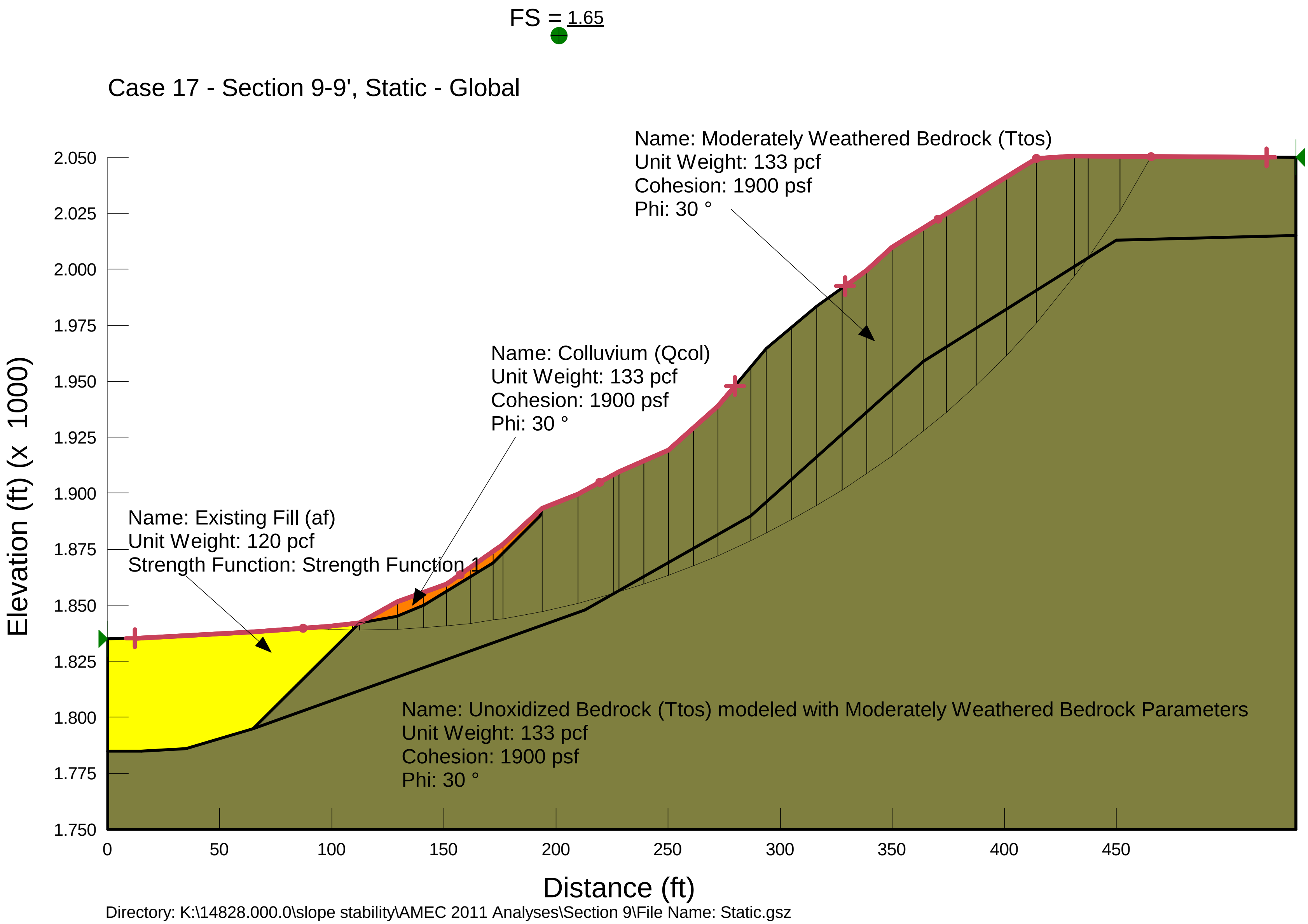


Directory: K:\14828.000.0\slope stability\AMEC 2011 Analyses\Section 7\File Name: Pseudostatic.gsz

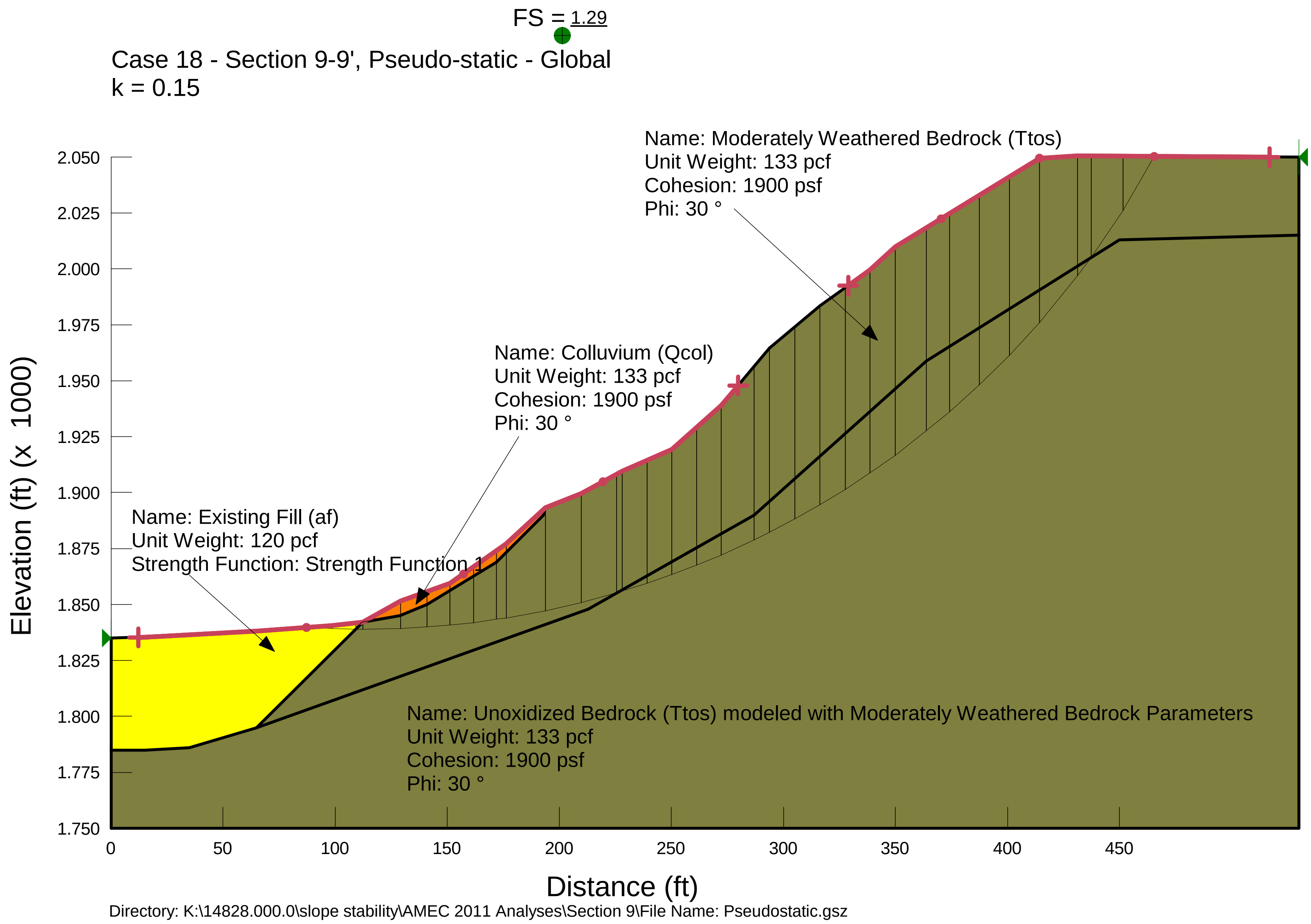

FS = 1.50
Case 15 - Section 8-8', Static - Global
Name: Moderately Weathered Bedrock (Ttos)
Unit Weight: 133 pcf
Cohesion: 1900 psf
Phi: 30 °
Name: Existing Fill (af)
Unit Weight: 120 pcf
Strength Function: Strength Function 1
Name: Unoxidized Bedrock (Ttos) modeled with Moderately Weathered Bedrock Parameters
Unit Weight: 133 pcf
Cohesion: 1900 psf
Phi: 30 °
Elevation (ft) (x 1000)
2.050
2.025
2.000
1.975
1.950
1.925
1.900
1.875
1.850
1.825
1.800
1.775
1.750
0
50
100
150
200
250
300
350
400
Distance (ft)
Directory: K:\14828.000.0\slope stability\AMEC 2011 Analyses\Section 8\File Name: Static.gsz



FS = 1.18
Case 16 - Section 8-8', Pseudo-static - Global
k = 0.15
Name: Moderately Weathered Bedrock (Ttos)
Unit Weight: 133 pcf
Cohesion: 1900 psf
Phi: 30 °
Name: Existing Fill (af)
Unit Weight: 120 pcf
Strength Function: Strength Function 1
Name: Unoxidized Bedrock (Ttos) modeled with Moderately Weathered Bedrock Parameter
Unit Weight: 133 pcf
Cohesion: 1900 psf
Phi: 30 °
Elevation (ft) (x 1000)
2.050
2.025
2.000
1.975
1.950
1.925
1.900
1.875
1.850
1.825
1.800
1.775
1.750
0
50
100
150
200
250
300
350
400
Distance (ft)
Directory: K:\14828.000.0\slope stability\AMEC 2011 Analyses\Section 8\File Name: Pseudostatic.gsz



FS = 1.65
Case 17 - Section 9-9', Static - Global
Name: Moderately Weathered Bedrock (Ttos)
Unit Weight: 133 pcf
Cohesion: 1900 psf
Phi: 30 °
Name: Colluvium (Qcol)
Unit Weight: 133 pcf
Cohesion: 1900 psf
Phi: 30 °
Name: Existing Fill (af)
Unit Weight: 120 pcf
Strength Function: Strength Function 1
Name: Unoxidized Bedrock (Ttos) modeled with Moderately Weathered Bedrock Parameters
Unit Weight: 133 pcf
Cohesion: 1900 psf
Phi: 30 °
Elevation (ft) (x 1000)
2.050
2.025
2.000
1.975
1.950
1.925
1.900
1.875
1.850
1.825
1.800
1.775
1.750
0
50
100
150
200
250
300
350
400
450
Distance (ft)
Directory: K:\14828.000.0\slope stability\AMEC 2011 Analyses\Section 9\File Name: Static.gsz


FS = 1.29
Case 18 - Section 9-9', Pseudo-static - Global
k = 0.15
Name: Moderately Weathered Bedrock (Ttos)
Unit Weight: 133 pcf
Cohesion: 1900 psf
Phi: 30 °
Name: Colluvium (Qcol)
Unit Weight: 133 pcf
Cohesion: 1900 psf
Phi: 30 °
Name: Existing Fill (af)
Unit Weight: 120 pcf
Strength Function: Strength Function 1
Name: Unoxidized Bedrock (Ttos) modeled with Moderately Weathered Bedrock Parameters
Unit Weight: 133 pcf
Cohesion: 1900 psf
Phi: 30 °
Elevation (ft) (x 1000)
2.050
2.025
2.000
1.975
1.950
1.925
1.900
1.875
1.850
1.825
1.800
1.775
1.750
0
50
100
150
200
250
300
350
400
450
Distance (ft)
Directory: K:\14828.000.0\slope stability\AMEC 2011 Analyses\Section 9\File Name: Pseudostatic.gsz

APPENDIX E

Infinite Slope Stability Analysis Package

APPENDIX E
INFINITE SLOPE STABILITY ANALYSIS PACKAGE

Sunshine Gas Producers
Landfill Gas to Energy Project
Sunshine Canyon Landfill
Sylmar, California

The infinite slope formulation by Giroud et al. (1995), which includes the contribution of geosynthetic reinforcement, was used to calculate the factor of safety (FS) of the proposed 1.5:1 reinforced and 3:1 unreinforced fill slopes, and the existing 1.5:1 unreinforced fill slopes and bedrock slopes.

The equation provided in Giroud et al. (1995) for fully saturated condition is as follows:

$$FS = (\gamma' / \gamma_s) * (\tan\phi' / \tan\beta) + (c' / (\gamma_s * t * \sin\beta)) + (T / (\gamma_s * t * h))$$

where*:*

- γ' = buoyant unit weight (pcf)
- γ_s = saturated unit weight (pcf)
- ϕ' = angle of internal friction (degrees)
- β = slope angle (degrees)
- c' = cohesion (psf)
- t = soil thickness measured perpendicular to ground surface (ft);
- t = z cos β,
- z= vertical height of the soil column (ft)
- T = tensile strength of geogrid (lb/ft)
- h = vertical height between geogrid layers (ft)

For proposed reinforced earth fill slopes, the pullout resistance, P_r, is used to calculate the minimum embedment length of geogrid layers required to achieve a minimum FS = 1.5 for a 4-foot thick zone of saturation. The equation for pullout resistance per unit width of reinforcement (lb/ft) is as follows:

$$P_r = C_i \tan\phi' \alpha \sigma_v' L_e C$$

where:

- C_i = pullout coefficient of interaction (dimensionless)
- ϕ' = angle of internal friction (degrees)
- α = scale effect correction factor (dimensionless)
- σ_v' = effective overburden pressure at the soil-reinforcement interfaces (psf)
- L_e = effective geogrid length beyond the slip surface (ft)
- C = reinforcement effective unit perimeter (dimensionless)

UNREINFORCED SLOPES

Existing

<u>1.5:1 Unreinforced Fill Slopes – 4-foot thick no flow condition</u>

γ' = γ_s = 120 (pcf) for unsaturated conditions
ϕ' = 45 (degrees) (See Figure 7 for Shear Strength Envelope at low normal loads)
β = $\tan^{-1}$(1/1.5) = 33.7 (degrees)
c' = 0 (psf) (See Figure 7 for Shear Strength Envelope at low normal loads)
z = 4 (ft)
t = 4 ft * cos (33.7) = 3.33 (ft),

$FS = (120/120)*(\tan 45/\tan 33.7)+0+0 = 1.50$ **≥ 1.5, √OK**

<u>1.5:1 Unreinforced Fill Slopes – 4-foot thick full flow condition</u>

γ' = 120 – 62.4 = 57.6 (pcf)
γ_s = 120 (pcf)
ϕ' = 45 (degrees) (See Figure 7 for Shear Strength Envelope at low normal loads)
β = $\tan^{-1}$(1/1.5) = 33.7 (degrees)
c' = 0 (psf) (See Figure 7 for Shear Strength Envelope at low normal loads)
z = 4 (ft)
t = 4 ft * cos (33.7) = 3.33 (ft),

$FS = (57.6/120)*(\tan 45/\tan 33.7)+0+0 = 0.72$ **< 1.5, NOT OK**

<u>1:1 Moderately Weathered Bedrock Slopes – 4-foot thick full flow condition</u>

γ' = 133 – 62.4 = 70.6 (pcf)
γ_s = 133 (pcf)
ϕ' = 30 (degrees) (See Figure 8 for Shear Strength Envelope)
β = $\tan^{-1}$(1/1) = 45 (degrees)
c' = 1,900 (psf) (See Figure 8 for Shear Strength Envelope)
z = 4 (ft)
t = 4 ft * cos (45) = 2.83 (ft),

$FS = (70.6/133)*(\tan 30/\tan 45)+(1{,}900/(133*2.83*\sin 45))+0 = 7.45$
≥ 1.5, √OK

Proposed

<u>3:1 Fill Slopes - 4-foot thick full flow condition</u>

γ' = 125 – 62.4 = 62.6 (pcf)
γ_s = 125 (pcf)
ϕ' = 31 (degrees) (See Figure 9 for Shear Strength Envelope)
β = $\tan^{-1}$(1/3) = 18.4 (degrees)
c' = 100 (psf) (See Figure 9 for Shear Strength Envelope)
z= 4 (ft)
t = 4 ft * cos (18.4) = 3.79 (ft),

$FS = (62.6/125)*(\tan 31/\tan 18.4)+(100/(125*3.79*\sin 18.4))+0 = 1.57$ **≥ 1.5, √OK**

PROPOSED REINFORCED SLOPES

1.5:1 Reinforced Fill Slopes with 8-foot geogrid spacing - 4-foot thick full flow condition

γ' = 125 – 62.4 = 62.6 (pcf)
γ_s = 125 (pcf)
ϕ' = 31 (degrees) (See Figure 9 for Shear Strength Envelope)
$\beta = \tan^{-1}(1/1.5)$ = 33.7 (degrees)
c' = 100 (psf) (See Figure 9 for Shear Strength Envelope)
z= 4 (ft)
t = 4 ft * cos (33.7) = 3.33 (ft),
T = 1,760 (lb/ft) (Tensar UX1400HS or equivalent)
h = 8 (ft)

$$FS = (62.6/125)*(\tan 31/\tan 33.7) + (100/(125*3.33*\sin 33.7)) + (1{,}760/(125*3.33*8)) = 1.41$$

< 1.5, NOT OK: Therefore, add another layer in between 8-foot spaced primary reinforcements

1.5:1 Reinforced Fill Slopes with 4-foot geogrid spacing - 4-foot thick full flow condition

γ' = 125 – 62.4 = 62.6 (pcf)
γ_s = 125 (pcf)
ϕ' = 31 (degrees) (See Figure 9 for Shear Strength Envelope)
$\beta = \tan^{-1}(1/1.5)$ = 33.7 (degrees)
c' = 100 (psf) (See Figure 9 for Shear Strength Envelope)
z= 4 (ft)
t = 4 ft * cos (33.7) = 3.33 (ft),
T = 1,760 (lb/ft) (Tensar UX1400HS or equivalent)
h = 4 (ft)

$$FS = (62.6/125)*(\tan 31/\tan 33.7) + (100/(125*3.33*\sin 33.7)) + (1{,}760/(125*3.33*4)) = 1.94$$

≥ 1.5, √OK

Calculation of Effective Geogrid Length for Secondary Layers

C_i = 0.5 (manufacturer recommended low-end conservative value for fine grained soils)
ϕ' = 31 (degrees) (See Figure 9 for Shear Strength Envelope)
C = 2 (2 for uniaxial geogrids)
α = 1 (for uniaxial geogrids)
P_r = 1,760 (lb/ft) (Tensar UX1400HS or equivalent)
$\sigma_v' = (L_e/3+4)*\gamma_s$ (psf) & γ_s = 125 (pcf)

$$1760 = 0.5\tan(31)(1)(L_e/3+4)(125)L_e(2)$$

Solve for L_e;
L_e = 4.5'

Total geogrid length = (4 ft vertical saturation x 1.5:1) +4.5' = 10.5' (11')

Minimum geogrid length of 11 feet at 4-foot vertical spacing between geogrid layers over 1.5:1 slope,

Summary of Findings

The infinite slope stability analyses were performed using the equation provided in Giroud et al. (1995). The required FS per the Los Angeles County Manual for Preparation of Geotechnical Reports is 1.5.

The existing fill slopes exhibit a FS of 1.50 for a no-water-flow condition within the fill, and exhibit a FS less than unity (FS=1) for full-flow assumption over a 4-foot thickness.

The bedrock slopes exhibit a high FS against surficial instability.

The FS for the proposed 3:1 unreinforced fill slopes is greater than 1.50. The proposed 1.5:1 reinforced slopes exhibit a FS less than 1.5 for a 8-foot geogrid spacing in between prmary geogrid reinforcement. Including additional geogrid in between the primary reinforcements where the spacing is 8 feet (i.e., every 4 feet vertically), the FS is greater than 1.5. The minimum geogrid length for layers within the 1.5:1 slope should be 11 feet based on the soil shear strength assumed in this Study.

Refer to Report Section 6.2 for the discussion of these results.