SOUTH COAST LANDFILL
REPORT OF WASTE DISCHARGE
DECEMBER 2016

VOLUME II – APPENDIX A: FINAL CLOSURE/POST-CLOSURE MAINTENANCE PLAN
TEXT, TABLES, FIGURES, DRAWINGS, AND APPENDIX A ONLY

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INTRODUCTION

1.0 INTRODUCTION

1.1 Purpose

This Final Closure/Post-Closure Maintenance Plan (FCPCMP) for the South Coast Landfill (SCL) has been prepared for submittal to the California Department of Resources Recycling and Recovery (CalRecycle) (formerly the California Integrated Waste Management Board [CIWMB]), the Mendocino County Department of Public Health (LEA), and the Regional Water Quality Control Board (RWQCB) – North Coast Region on behalf of the County of Mendocino Department of Transportation (DOT), the operator of the landfill, by SWT Engineering (SWT).

The SCL FCPCMP has been prepared in accordance with Title 27 of the California Code of Regulations (27 CCR), Chapters 3 and 4 and the Code of Federal Regulations (CFR) 40, Subpart F. The objectives of this FCPCMP are as follows:

- To provide a basis for the establishment of an accurate detailed cost estimate for closure and post-closure maintenance.
- To provide a detailed plan and schedule for closure implementation.
- To provide a plan and schedule for the inspection, maintenance and monitoring procedures to be implemented during the post-closure maintenance period.
- To allow CalRecycle, the RWQCB, and the LEA to monitor closure and post-closure activities to determine that all landfill closure and post-closure maintenance and monitoring requirements are being followed in accordance with the approved plan.

Volume I consists of the FCPCMP text required under 27 CCR. The sections titled Introduction (Section 1.0) and Existing Environmental Control and Monitoring Systems (Section 2.0) of this document present information which is common to both the Closure Plan (Section 3.0) and Post-Closure Maintenance Plan (Section 4.0). The Post-Closure Emergency Response Plan and Closure/Post-Closure Maintenance Cost Estimate are presented in Sections 5.0 and 6.0, respectively. Professional Certification of Accuracy (Section 7.0), Recordkeeping (Section 8.0) and References (Section 9.0) comprise the remaining sections of this document. Full size drawings and figures are located at the back of Volume I and technical appendices are located in Volume II.

1.2 Setting

1.2.1 Location

The SCL is located in Mendocino County, east of Highway 1 on Fish Rock Road (Figure 1) (Sheet 1 of 13) in the southeast ¼ of the south ½ of Section 4, Township 11 north, Range 15 west, Mount Diablo Base and Meridian. The address of the SCL is 40855 Fish Rock Road, Gualala, California 95445, Assessor's Parcel No. 141-080-26.

1.2.2 Facility Boundaries

The entire property, consisting of 47.65 acres, is owned by the DOT. Approximately six acres were utilized for waste disposal. Figure 2 shows the disposal area footprint and Figure 2A shows the entire property boundary.

1.2.3 Surrounding Land Use

Figure 3 depicts the land use designation and shows 1,000-foot radius from the property boundary. There are no structures within 1,000 feet of the site property boundary. The land surrounding the SCL is zoned Timber Preserve Zone (TPZ), with a minimum parcel size of 160 acres.

No potential adverse land use impacts are foreseeable due to closure of the landfill.

1.2.4 Existing Topography

SCL is located in the foothill region of the Coastal Mountain range in heavily forested rugged mountain terrain. The landfill is situated at an elevation of about 500 feet above mean sea level (msl) and approximately 200 feet northeast of the Little North Fork of the Gualala River. The current topography (February 2012) of the landfill is shown on Figure 2. A pre-landfill topographic map of the area (at the same scale as Figure 2) is not available. Figure 4 shows the refuse limit verification report/trenching area at the SCL.

1.2.5 Geologic Conditions

The SCL is located in the San Andreas Fault Zone in the northern area of the Coast Ranges. The northwest-southeast trending faults and folds have created the ridges and valleys of the fault zone (GeoLogic Associates [GLA], 2003). The site is underlain by the Guinda formation, consisting of marine sandstone and mudstone, which are part of the late Cretaceous unit of the Franciscan assemblage. The rocks have been locally sheared with much clay gouge present in the vicinity of the site (Anderson, 1995).

East of the landfill and east of the Little North Fork, Cretaceous-age marine sandstones and sheared shales of the Coastal Belt Franciscan Formation are the most dominant lithology. West of the site and west of the Little North Fork, marine mudstones and sandstones of the Cretaceous-age Anchor Bay Member of the Gualala Formation and marine sandstones of the Tertiary-age German Rancho Formation crop out. Within the relatively flat-lying central and eastern portions of the property, unconsolidated, well-graded recent-age alluvial terrace deposits of mixed clays, silts, sands and gravel are exposed (GLA, 2003).

Although a number of landslides have been mapped near the site (Davenport, 1984; McKittrick, 1995), no landslide features have been identified on the SCL property. Most of the large-scale landslides in the region have relatively deep-seated failure surfaces with a rotational/transitional mode of movement along planar joints or bedding. In many cases, slope failure appears to be related to erosional processes at the toe of slopes. The fact that landslides are not typically mapped within fault gouge in the area may be related to the nearly vertical textural fabric of shears within the unit. This inference is supported by information presented by McKittrick (1995), which indicates low to moderate landslide susceptibility on most of the SCL property (GLA, 2003).

1.2.6 Hydrogeologic Conditions

Groundwater at the site occurs within fractured gouge zone materials. Data from previous investigation of the site (SHN, 1991) indicates that groundwater is encountered at depths ranging from 8 to 23 feet below ground surface. Along the west side of the property, groundwater may have been encountered in two borings that were excavated to depths of 17 and 12.5 feet, but was not encountered in two other borings that were extended to depths of 12 and 38.5 feet (see 2003 Final Closure Evaluation included in Appendix A) (GLA, 2003).

Figure 3 of Appendix A, which shows groundwater equipotential contours, was developed using groundwater elevation data obtained in August and November 2002. As indicated, groundwater is interpreted to flow from the northeast to the southwest at a hydraulic gradient of approximately 0.08 ft/ft. However, this pattern is expected to be locally interrupted by well-developed shears within the gouge zone matrix with resultant anisotropic flow directed in a more southerly direction (GLA, 2003).

1.3 Facility Information

1.3.1 Historic Background

The SCL is a Class III solid waste disposal facility. The DOT began landfill operations in 1970. The SCL ceased landfill operations in 2000. The SCL is owned and was operated by the DOT. The SCL served the residents of Mendocino County South Coast Area, which is the State Route 1 corridor from the Sonoma County Line to the Navarro River. This area includes the towns of Elk, Irish Beach, Manchester, Point Arena and Gualala. The landfill was constructed in a shallow ravine, using the area fill method to place, compact and cover refuse on a daily basis.

The site operated in accordance with State Minimum Standards for a Class III disposal facility as established by the State Water Resources Control Board (SWRCB) and CalRecycle. The site accepted mixed municipal refuse, classified nonhazardous solid waste and inert waste as defined in the 27 CCR, Sections 20220 and 20230. No liquid or hazardous waste was knowingly accepted at the site. Wastes at the landfill generally consisted of municipal refuse including residential refuse (90 percent), commercial refuse (5 percent), and demolition refuse (5 percent) (SWD, 1996).

1.3.2 Responsible Parties

The following is a listing of the responsible party who will be involved in post-closure maintenance and monitoring activities at the SCL.

Landfill Owner/Operator

County of Mendocino Department of Transportation
Solid Waste Division
340 Lake Mendocino Drive
Ukiah, CA 95482
(707) 463-4078

The DOT Site Representative (i.e., third party representative) will be responsible for the overall monitoring/maintenance of the SCL. Should the title to the site be transferred during closure and/or post-closure maintenance, the new owner shall be notified by the previous owner or his agent of the existence of the 27 CCR standards and of the conditions and agreements assigned to assure compliance. In accordance with 27 CCR, Section 21200, the previous owner shall notify the LEA of the change in writing within 30 days and shall provide the name, firm, mailing address and telephone number of the new owner.

1.3.3 Maximum Extent of Closure

In accordance with 27 CCR, Section 21790, the estimated maximum extent of the landfill that will ever require closure at any given time during the life of the landfill based on the current disposal area footprint is approximately 6 acres.

1.4 Closure Date

Landfill operations ceased at the SCL in 2000. DOT has commenced the final closure process with preparation of submittal of the FCPCMP. The final closure date is contingent upon approval of the Final Closure Plan.

1.5 Post-Closure End Use

The currently proposed post-closure end use for the SCL will be non-vegetated (closure turf) open space. A Medium Volume Transfer Operation (MVTO) is currently located at the site as shown on Figure 2 and is fully permitted. Use of this facility during the post-closure period for the SCL has been approved by the LEA (see approval letter included in Appendix B). The MVTO is not located over the refuse footprint.

Current closure and post-closure requirements limit end use options in order to maintain the integrity of the final cover surface. CalRecycle, the RWQCB, and the LEA in accordance with 27 CCR, Section 21190, must review any future proposed changes to the currently proposed end use that would require construction improvements.

SECTION 2.0
EXISTING ENVIRONMENTAL CONTROL AND MONITORING SYSTEMS

2.0 EXISTING ENVIRONMENTAL CONTROL AND MONITORING SYSTEMS

2.1 Landfill Gas Control/Monitoring System

The SCL currently does not have a landfill gas control system. Two methane probes (LFGW-1 and LFGW-2) were installed in accordance with 27 CCR, Section 20925 at the site at the locations shown on Figures 2 and 12. Perimeter gas monitoring wells were not installed to the south and west of the site, because the steep gradient down to the Gualala River prohibits easy access for construction of the wells. The two gas wells were installed above the low seasonal water table. LFGW-1 consists of a two probe monitoring system and was installed to a depth of 20 feet. LFGW-2 is a one probe system and was installed to a depth of 11 feet. The probes were monitored one time in accordance with the Air Solid Waste Assessment Test (SWAT) sampling and analytical requirements. Specific contaminants were detected in the landfill gas well and in the downwind ambient air samples, and trace amounts of methane were detected in the perimeter probes. However, the results of the Air SWAT investigation indicated negligible levels of organic contaminants in the landfill gas and no hazardous levels of landfill gas are present at the landfill (Anderson, 1995). These landfill gas probes are no longer monitored on a quarterly basis as they are currently not in use and will not be used in the foreseeable future. The probes are covered and locked, but have not been formally decommissioned.

In addition to the two existing landfill gas probes described above, DOT implemented construction of three additional gas probes, completed in June 2012. The probes were installed according to the gas probe design information prepared in the 2003 FCPCMP. The gas probe design from 2003 was also requested to be revisited/confirmed by CalRecycle, who provided verbal approval. Currently, the three additional gas probes are in use as compliance gas wells, which are monitored quarterly as the compliance probes.

Drilling and installation of the three probes (P-1, P-2, and P-3) was performed in June 2012 at the locations shown in Figure 12. The design depth for probe P-1 was 41 feet, however, groundwater was encountered in the boring at 28.5 feet. Therefore, the lower portion of the P-1 boring was backfilled to a depth of 20 feet and probe P-1 was installed above this depth. Borings for P-2 and P-3 were advanced to 52 feet and 21 feet, respectively. Drilling and construction were monitored by a registered geologist. All drilling and construction of the three probes were conducted in accordance with the well construction permit issued by the County Health and Human Services Agency. The gas probe installation report (GLA, 2012), which includes boring and probe construction logs, is included as Appendix C.

Due to the negligible levels of organic contaminants in the landfill gas and no hazardous levels of landfill gas present at the landfill, an active landfill gas control system is not proposed for the site. However, a passive vent landfill gas system will be installed under the final cover for the sole purpose of preventing buildup of landfill gas under the linear low-density polyethylene (LLDPE) geomembrane. In the event that landfill gas production warrants extraction, this passive system can be retrofitted into an active system. This system is further discussed in Section 3.8.

2.2 Groundwater/Surface Water Monitoring System

The groundwater monitoring system at the SCL was initiated in 1987. Five monitoring wells (87-1, 87-2, 87-3, 87-4 and 87-5) were installed in 1987. One additional well (91-1) was installed in 1991 to further delineate the down gradient hydrological conditions at the site. In 1994, monitoring well 87-4 was destroyed and four new monitoring wells (94-1, 94-2, 94-3 and 94-4) were installed in 1994. Monitoring well 94-1 was installed within the reamed borehole of 87-4. The depths of the monitoring wells vary from 15.4 feet to 50 feet.

The groundwater monitoring wells are sampled in accordance with Waste Discharge Requirements (WDR) Order No. 77-23 (included in Appendix B), as amended by Order No. 93-83 which complies with 27 CCR, Subchapter 3, Article 1 requirements. Quarterly water quality monitoring reports are submitted to the RWQCB.

Surface water runoff at the site is sampled seasonally at two locations, SW-1 and SW-2. SW-1 is located along the south side of the landfill where runoff discharges to a surface water detention basin and SW-2 is located near the northwest corner of the landfill where runoff discharges to the Little North Fork of the Gualala River. The groundwater monitoring system at the SCL is reflected in the Storm Water Pollution Prevention Plan (SWPPP) and Second Quarter 2016 Monitoring Report (SHN, 2016).

2.3 Stormwater Monitoring and Reporting

The DOT submitted a Notice of Intent (NOI) to comply with the National Discharge Pollution Elimination System (NPDES) regulations implemented by the RWQCB for monitoring and reporting storm water discharges. Stormwater sampling is performed at the two surface water locations (SW-1 and SW-2) during the first significant storm and at the end of the rainy season in accordance with NPDES requirements.

2.4 Leachate Collection and Removal System

Leachate collection and removal system (LCRS) consists of a leachate infiltration gallery, polyethylene and polyvinyl pipe used to transport leachate that is collected primarily from small surface seeps and transports the leachate to the containment system.

The leachate collection gallery was constructed to completely surround the end of the drainage trench at the edge of refuse. The leachate collection gallery intercepts the drainage trench, captures the leachate and transports the leachate to the leachate containment facility. The leachate collection gallery consists of 2-inch rock which is enveloped in filter fabric and is located under the perimeter road, at the edge of refuse. The leachate drains into a vertical 36-inch CMP riser wrapped in filter fabric, located 6-feet from the edge of the perimeter road. The leachate is gravity fed into two 3-inch polyvinyl chloride (PVC) drains which connects to a 2-inch PVC pipe and then to a 2-inch polyethylene pipe (PEP) pipe and into the leachate containment facility (tank farm). The leachate containment facility is outfitted with a suction coupling for the off-chance the leachate needs to be evacuated from the pipes. During the wet season, leachate is regularly transported by truck and disposed of at the Gualala Community Service District Wastewater Treatment Plant. In 2015, 17, 500 gallons of leachate were hauled to an approved wastewater treatment plant.

The leachate containment facility consists of nine 2,300-gallon plastic tanks, which has the capacity to store 20,700 gallons of leachate. The containment facility is surrounded by an earthen containment structure in the event that a tank(s) developed a leak.

The tanks are periodically pumped by tanker truck and leachate is disposed of at the Gualala Community Service District Sewage Treatment Plant (GCSD) in Gualala, California.

The DOT has entered into a contractual agreement with the GCSD for the disposal of the leachate (SWD, 1996). Pumping of the tanks during the wet season is more frequent.

It should be noted that leachate samples are collected annually in the fourth quarter of each year from the collection tanks located along the south side of the landfill. Results are also included in the quarterly water quality monitoring reports submitted to the RWQCB.

SECTION 3.0
CLOSURE PLAN

3.0 CLOSURE PLAN

3.1 Introduction

Closure of the SCL will be performed in accordance with the applicable regulatory standards included in 27 CCR, Chapters 3 and 4 and 40 CFR, Subpart F. The costs associated with closure of the SCL are described in Section 6.0 of this document. The components and systems required for closure of the SCL include the final cover design, final grading plan, landfill settlement, landfill slope stability, construction quality assurance, drainage and erosion control systems, landfill gas control and monitoring systems, groundwater/surface water monitoring systems, and site security. A description of these closure components, as well as a schedule for construction of the SCL closure improvements, is presented in the following subsections.

Construction drawings developed for the final closure project were utilized as the basis for various figures included in this FCPCMP.

3.2 Final Cover

The purpose of a final cover is to provide long-term minimization of surface water intrusion, to accommodate settlement and subsidence and to isolate wastes from the ground surface. The final cover also provides a base for vegetation which will reduce drainage velocities, erosion and infiltration.

3.2.1 Minimum Design Standards

California Final Cover Regulatory Requirements

The minimum final cover standards for the SCL, as outlined in the closure and post-closure requirements for Class III landfills contained in 27 CCR, Section 21090 include:

Foundation Layer - A minimum two-foot thick layer of approved soil, contaminated soil, incinerator ash, or other waste materials placed immediately over the entire surface of the last lift of refuse. This layer shall have the appropriate engineering properties so as to provide a relatively unyielding surface upon which to place and compact a low-hydraulic conductivity layer.

Low-Hydraulic-Conductivity Layer - A minimum one-foot thick layer of clean low-hydraulic-conductivity soil containing no waste or leachate placed over the foundation layer. The low-hydraulic-conductivity (or low through-flow rate) soils shall be placed on top of the foundation layer and compacted to attain a hydraulic conductivity, which is the lesser of either;

- 1×10^{-6} cm/sec.
- The hydraulic conductivity of any bottom liner system or underlying natural geologic materials.

Erosion Resistant Layer - A minimum one-foot thick layer of soil containing no waste or leachate placed on top of all portions of the low-hydraulic conductivity layer. Vegetation root depths must not exceed the top soil layer thickness. Vegetation is to be replanted, as needed, to provide effective erosion resistance.

The final cover shall be designed to allow minimum maintenance requirements, and final grading should provide for deck/slopes of at least three percent, to prevent ponding and accommodate settlement of the refuse prism.

Federal Final Cover Regulatory Requirements

The minimum final cover standards for the SCL, as outlined in the closure criteria of 40 CFR, Subpart F, Section 258.60, include:

A cover with a permeability less than or equal to the hydraulic-conductivity of any bottom liner system or natural sub-soils present, or a permeability no greater than 1×10^{-5} cm/sec, whichever is less. The infiltration layer shall consist of a minimum 18 inches of earthen material.

A cover which minimizes erosion of the final cover by the use of an erosion resistant layer that contains a minimum six inches of earthen material and is capable of sustaining native plant growth.

3.2.2 Proposed Final Cover Design

Several factors were taken into consideration in establishing the final cover design for the SCL including the geometry of the existing landfill, local climatic conditions, potential landfill settlement, final cover material availability and desired performance criteria, erosion protection, vegetative growth, construction cost and end use at closure. Analyses performed by GLA (GLA, 2012, Appendix A) concluded that an alternative final cover design utilizing the geomembrane system was the most appropriate cover system for the site.

27 CCR and Subtitle D require that landfill final covers be constructed according to identified minimum standards. In California, 27 CCR regulations take precedence because they prescribe more restrictive standards. For unlined Class III landfills, these standards include a two-foot thick foundation layer, a minimum 1-foot-thick low-permeability layer, and a minimum 1-foot thick vegetative layer. Alternatives to these prescribed standards are allowed in 27 CCR, Section 21090 which states that:

> "The RWQCB can allow any alternative final cover design that it finds will continue to isolate the waste in the unit from precipitation and irrigation water at least as well as would a final cover built in accordance with applicable prescriptive standards."

The purpose of the 2012 GLA geotechnical analysis was to evaluate existing and proposed final closure design and construction conditions at the SCL and to re-examine preliminary slope stability analyses completed by an earlier consultant for the DOT. Recognizing that earlier studies of the site employed literature values for material strength properties rather than site-specific data, GLA's work included a subsurface exploration and laboratory testing program to better characterize existing and potential future slope stability conditions. The data collected in this investigative program indicated that the SCL is underlain by fault gouge and alluvial/colluvial soils that have significantly higher shear strengths than were assumed in the earlier studies of the site. Slope stability analyses were then completed to assess the stability of the native western slope abutting the landfill and to evaluate alternative landfill cover configurations. Based on these analyses, it was concluded that adequate slope stability, as well as 27 CCR compliant closure, could be achieved with a minor reconsolidation of wastes away from the western slope, and by using an alternative final cover configuration consisting of (from bottom to top):

- a two-foot thick foundation layer above the existing landfill cover soils, and additional onsite and/or off-site soil or other suitable materials as allowed under 27 CCR (the existing soils will be scarified and recompacted);
- a 60-mil linear low-density polyethylene (LLDPE) Super Grip Net geomembrane; and
- closure turf geotextile with sand infill ballast.

This configuration recognized that only limited borrow soils are available on the property because the landfill cover barrier layer, a geomembrane, will be imported to the site. Since this cover configuration requires only minor refuse reconsolidation and minimizes the volume of import soils, it was considered an optimal approach for closure of the SCL.

A typical cross-section of the proposed closure turf final cover system is shown on Figure 5 (see Details 1 and 2). The proposed final cover section will be placed over all areas within the limits of refuse, at a maximum grade of 2:1 (horizontal to vertical) and minimum grade of 3% in accordance with slope stability analyses completed as required by 27 CCR 21750(f)(5), included in the Final Closure Evaluation, Appendix A. The proposed final grading for the SCL is shown on Figure 6.

Given that the foundation layer is proposed in accordance with the prescriptive standard, the proposed engineered alternative component of the selected final cover design will consist of a LLDPE geomembrane barrier layer (with associated overlying geosynthetics, closure turf geotextile with sand infill ballast material). DOT selected the use of a closure turf material in lieu of the vegetative soil layer as discussed below. In accordance with 27 CCR, Section 20080(b), and 27 CCR 21140(b), the DOT is requesting approval for the proposed engineered alternative based on the LLDPE's higher performance characteristics when compared to the prescriptive standard.

3.2.3 Sources of Cover Material

Foundation layer soils shall be obtained from the on-site borrow area (stormwater basin enlargement), existing deck stockpiles (Figure 4), basin access excavation, and local import soils. The stormwater basin borrow source is located immediately north of the refuse limits and the stockpile is located within the refuse limits, adjacent to the borrow source. Basin access excavation is at the southeast side of the landfill. The total volume from these sources is estimated to be approximately 18,500 cubic yards (cy), which would have necessitated import of additional cover soil from a local borrow source to complete the final cover construction vegetative soil layer. A cost for import soils was estimated using a 50-mile radius to transport the materials. The cost was to include excavation, loading, and transport. The DOT found that a viable source was not available and undertook a design change to alternative final cover utilizing a closure turf in lieu of vegetative and underlying sod.

Soil samples were obtained from the on-site borrow areas and tested. The Final Closure Evaluation prepared by GLA in 2003 (see Appendix A) includes the results of laboratory analyses for material type, grain size analysis, moisture-density relationship, and strength properties (i.e., shear strength, cohesion) of the on-site soils. The GLA report, as well as experience with the existing intermediate cover, verifies that the material is appropriate for use in random compacted fills and for foundation layer soils. Import soils will be required to have similar material properties pursuant to project specifications.

Geosynthetic materials (i.e., geomembrane, (Super Grip Net) geotextile) shall be provided from approved manufacturers as required to meet the performance specifications that will be included in the construction specifications. Appendix D includes typical manufacturer's data for the types of geosynthetics which will be used in construction and meet the design properties required by the slope stability analyses (which have become less stringent due to the reduction of load above the LLDPE Geomembrane to almost zero). As discussed above, a closure turf material in lieu of the vegetative layer component of the final cover will be utilized. Product information on performance and specifications is included in Appendix D-1.

3.2.4 Final Cover Construction

Clearing and Grubbing

Prior to final grading and placement of the final cover, existing vegetative materials will be removed from the surface without disturbing the underlying refuse. The materials removed during clearing and grubbing operations will be used as interim cover for refuse excavation areas, as well as within the refuse reconsolidation area. The balance of this material will be disposed of within the reconsolidation area.

Existing Cover Reconstruction

The thickness of existing interim cover over the refuse area was evaluated by potholing, conducted by GLA in 2002. According to this evaluation, the measured cover thickness at the SCL is an average of 20 inches over most of the refuse fill area, but varies from 6 inches to 96 inches thick. The approximate locations of test pits which penetrated the soil cover together with cover thickness contours are shown on Figure 4 included in Appendix A. Due to the irregularity of the waste placement, the thickness of the vegetation/root systems, it is assumed that more of the interim cover soil will be lost to root zone clearing and grubbing, therefore, additional cover material may need to be placed to achieve the full cover thickness (i.e., 2-foot foundation soil layer) over substantial areas of the refuse prism and to provide proper drainage control.

The final grading plan design assumes no utilization of existing interim cover soils for the construction of the final cover section. Project specifications are written to indicate that the project is a thickness and gradient project and not to elevations shown on the construction drawings. Hike-up stakes or pot holing will be used to verify thickness of foundation layer placement during construction.

The foundation layer construction will be conducted in accordance with 27 CCR, Section 21090(a)(1) and the project specifications. Construction will be verified and documented through the implementation of the Construction Quality Assurance (CQA) Plan (GLA, 2012) included in Appendix E.

On-site borrow and import soils to be utilized for the final cover foundation soil layer shall be placed in loose lifts with a maximum uncompacted thickness of six to eight inches and brought to within one to three percent of dry optimum moisture content and compacted to 90 percent of the maximum dry density as determined by ASTM D1557.

LLDPE Geomembrane/LFG Barrier

The LLDPE geomembrane/LFG barrier for the SCL will consist of a 60-mil LLDPE Super Grip Net geomembrane placed over prepared subgrade (foundation soil layer), which will be overlain by closure turf geotextile. The 60-mil LLDPE Super Grip Net has integrated spikes on the bottom side of the geomembrane with integrated drainage studs on the top of the geomembrane. The bottom spikes provide enhanced interface shear for stability of the geomembrane cover, and the drainage studs provide a path for storm water run-off when overlain by closure turf geotextile. The Super Grip Net and closure turf geotextile (integral geocomposite) will facilitate down slope drainage of any infiltration accummulating over the LLDPE. The LLDPE Closure Turf system flows down slope to perimeter drainage ditches as shown on Figure 5 (Details 3/D1, 5/D1, 6/D1, 7/D1 and 9/D1), Figure 8 Detail 4/D2; or the North Bench on Figure 9 Detail 6/D3. A landfill gas venting system is proposed to be placed below the geomembrane barrier layer as discussed in Section 3.8.

Vegetative/Protective Soil Layer

Closure turf material serves as a separator geotextile to hold the sand infill ballast material on top of the geosynthetic, then completes the drainage geocomposite function of the Super Grip Net geomembrane, and provides an aesthetically pleasing surface as well. Erosion over closure turf is virtually non-existent, therefore stormwater run-off from the closure turf site is much cleaner than a comparable prescriptive landfill closed site. There is no vegetative/protective soil layer in a closure turf final cover system (see Figure 5 Detail 1/D1 and 2/D1 for Final Cover section – Slope and Deck).

North Coast Regional Water Quality Control Board – Closure Turf Information Requests

The following discussion presents information to address RWQCB technical information on the closure turf with utilization of a closure turf material in lieu of a typical vegetative layer. The following outlines the issues voiced by the RWQCB and information proposed by DOT's consultant (SWT Engineering) and the closure turf manufacturer to the acceptability for this material use as the vegetative layer component of the proposed alternative final cover. It should be noted that this material (closure turf) has been utilized successfully at a number of closed non-hazardous solid waste landfills in the U.S. The following outlines the RWQCB issues and provides information addressing the issues from the June 10, 2016 conference call with the RWQCB.

- Drainage and Erosion – see Appendix D-1
- 401 Permitting – 401 permitting is not necessary for use of the cover component.
- Increased Run-off – see Appendix D-1
- Visual Aesthetics – the closure turf is the shade of green which will blend into the surrounding native plant community with ease, creating a pleasant pasture-like setting.
- Sand (Ballast)/Sand Maintenance – see Appendix D-1 and Section 4.6
- Turf bunching/wrinkling - Section 4.6
- Surface Water Ponding - Section 4.6
- Exposed Membrane Potential - Section 4.6
- Fire – See Appendix D-1
- Turf Cover Shelf Life – Manufacturer to provide
- Long Term Maintenance (Post–Closure Maintenance Period) Section 4.6

3.3 Final Grading

3.3.1 Limits of Refuse and Evaluation of Existing Cover

The limits of refuse were determined by a field investigation which involved the use of a backhoe to excavate test pits and a hollow-stem auger drill-rig to determine waste thickness

(GLA, 2003, Appendix A). The limits of waste and approximate existing ground contours are shown on Figure 2. Cross Sections A and B are shown on Figure 2A. As stated in Section 3.2.4, the existing foundation layer is in place over the majority of the landfill footprint.

As discussed above, there are two areas located along the north and east, and two areas along the west and south of the refuse footprint edges where the waste fill will be removed and reconsolidated. These areas are shown on Figure 7.The west and south areas involve removal of 2 feet of existing material to allow construction of the 2 foot thick foundation soil layer for the geosynthetics. Should this material be recoverable, it will be processed and used for foundation soil, placed and compacted. More information related to the refuse removal and reconsolidation is contained in Section 3.13.1.

3.3.2 Finished Surface

This section describes the final grading plan proposed for the SCL as shown on Figure 6. The final landfill footprint is approximately 6 acres. Additional foundation soil materials will be required on the slopes to achieve final slope grades of no steeper than 2:1.

The final grading plan (Figure 6) presents the landfill configuration after closure. The final grading configuration will promote lateral run-off of surface water and accommodate the effects of settlement within the refuse prism. Details for the alternative final cover and the drainage control system are illustrated on Figures 5, 8 and 9. Perimeter maintenance and deck access roads (consisting of 3-inch AC over 95% (per ASTM D1557) compacted native subgrade) will be used to maintain the final cover and environmental control systems throughout the post-closure maintenance period. The most current topographic map (dated February 2012) has been used as the base map for the final grading design.

The maximum elevation of the landfill is the crown of the upper deck that is at an approximate elevation of 537.5 (see Figure 6 - Final Grading Plan) feet above msl. This high point will be located at the south edge of the deck and will provide positive drainage flow to the north drainage control features. The final deck area will have a minimum gradient of three percent to promote drainage and allow for future settlement. Minor filling and shaping of the final surfaces may be conducted to maintain the minimum design gradients.

Final refuse slopes will have a maximum gradient no steeper than 2:1. A perimeter road (bench at north side) will be located around a majority of the perimeter of the landfill. The east, south and north slopes (including the reconsolidation height) are between 30 and 35-feet in height. The maximum vertical height (located at the southeast perimeter of the landfill) from the bottom of the landfill to the top of slope, including reconsolidation area height, is 53 feet. This slope height is part of the alternative final cover design for the southeast slope face.

An intermediary bench on the north perimeter of the landfill to effectuate perimeter access will be constructed. The bench cross slope is 1 foot in 12 feet (8.33 percent maximum) towards the landfill. Collected storm water will be conveyed along the inside of the bench. Downdrains coinciding with bench inlet structures will be constructed along the slope areas of the disposal site to allow for conveyance of stormwater flows from the deck and bench areas to the toe of the slopes. The downdrains will discharge into the stormwater basins located at the southeast and northwest-most portions of the site.

The combined volume and surface area of the two desilting basins will be sufficient to: 1) detain peak site run-off and 2) entrap silt to improve water quality before discharging into the natural drainage course, the North Fork of the Gualala River.

3.4 Landfill Settlement

3.4.1 Settlement Analysis

The landfill appears to be founded on native bedrock materials. Compressible soils (such as colluvial and alluvial soils) appear to have been largely removed for use as daily and interim cover soils over the active life of the landfill. Therefore, no significant settlement of the foundation materials underlying the SCL is anticipated.

The mechanics of refuse settlement are complex due to the extreme heterogeneity of refuse fill. According to Edil et al. (1990), the main mechanisms involved in refuse settlement are:

- Mechanical distortion (bending, crushing, and reorientation)
- Raveling (movement of fines into large voids)
- Physical-chemical changes (corrosion, oxidation, and combustion)
- Biochemical decomposition (fermentation and decay)

The magnitude of refuse settlement can thus be inferred to be a function of: (1) initial refuse density or solid/void ratio, (2) overall density of the refuse prism or ratio of refuse to daily cover soil, (3) content of decomposable materials in the refuse, (4) thickness of refuse lifts and total height of the refuse prism, (5) stress history, (6) time elapsed since each individual lift was placed, and (7) environmental factors such as moisture content, temperature, and gas content.

Based on our experience, the most consistent refuse settlement estimates are obtained by modeling the refuse prism as a 3-dimensional net, calculating the settlement at each node of the net with a time-dependant exponential decay function, and adding the total settlement for each node of the net. Total settlement contours are generated by subtracting total settlement from the final grade at the commencement of fill placement. Based on the work of Huitric (1981), settlement can be modeled as an exponential decay function of the form:

Remaining settlement = $\mathbf{a}Te^{-bt}$

Where **a** and **b** are constants such that total expected settlement is a proportion **a** of the original thickness, T, of a particular lift of refuse, and the rate of settlement decays at an exponential rate of **b**t, where t is the number of years elapsed since the particular lift of refuse was placed. For a municipal landfill with standard compaction equipment, **a** varies between 0.2 and 0.35 and **b** varies between 0.10 and 0.11. For the SCL analysis (Appendix A), intermediate values of 0.3 and 0.105, respectively, were used.

To estimate the historical rates of refuse accumulation, a two-dimensional grid was established over the footprint of the refuse prisms, with a nodal spacing of about 50 feet. The third dimension in the model is the net change in elevation between discrete time intervals, as determined from historical topographic maps and the proposed final fill plan. Inasmuch as development of the landfill footprint and interim fill elevations are not well known, for the purpose of this evaluation, it was assumed that refuse filling occurred evenly over the entire footprint throughout the active life.

Based on the criteria described above, refuse settlement within the SCL was calculated. As expected, the greatest settlement is expected to occur in areas where refuse thicknesses are greatest (i.e., within the center of the SCL). Comparison with final fill grades indicates that post-closure settlement could be as great as 5 feet within the center of the refuse fill. However, considering the elongation properties typical of the LLDPE geomembranes (e.g., >300% [Appendix D]), this long-term settlement will not affect the integrity of the proposed final cover system.

3.4.2 Survey/Settlement Monumentation

In order to monitor the future settlement of the landfill, settlement monuments and survey monuments (Figure 9 [Detail 2]) will be installed on the landfill in accordance with 27 CCR, Section 20950 (d). These monuments are proposed to consist of galvanized pipe, two inches in diameter and six inches in length placed in blocks of concrete, 24-inches in diameter by eight inches in depth. A nail and tag will be placed in the center of each monument for identification.

Two settlement monuments are proposed to be placed on the landfill area as shown on Figure 6. Survey monuments will be placed on undisturbed ground as reference points not subject to settlement as also shown on Figure 6. These points are the horizontal and vertical control points for the site aerial topographic survey and will be maintained or replaced, as needed, throughout the post-closure maintenance period. An aerial photographic survey of the site (with a maximum contour interval of two feet) will be performed and provided to the RWQCB, the LEA, and CalRecycle upon completion of closure activities. The settlement monuments will be surveyed upon completion of all closure construction activities. Additionally, in accordance with 27 CCR requirements, the DOT will update the aerial survey of the entire refuse footprint every five years throughout the post-closure maintenance period. Inspection activities of the survey monuments will be recorded on a form similar to Sample Form C in Appendix F. Included as part of the survey information, an iso-settlement map and report are to be submitted to the RWQCB.

3.5 Landfill Stability

In accordance with 27 CCR, Section 21090 (a)(6), for any portions of the final cover installed after July 18, 1997 for which the RWQCB has not approved a slope and foundation stability report, the discharger must prepare a stability analysis in accordance with the requirements of 27 CCR, Section 21750(f)(5). As shown in Appendix A, slope stability analyses of the landfill and proposed final cover were completed by GLA as part of a geotechnical evaluation of the site.

The analyses integrated the results of a subsurface exploratory and laboratory testing program that was completed by GLA and which included excavation of 4 hollow-stem auger borings around the perimeter of the landfill and direct shear testing of bedrock and soils. The data collected from the borings and laboratory test results indicate that the landfill is underlain by fault gouge and alluvial/colluvial soils that have significantly greater strengths than were assumed in earlier study of the site (EMCON, 1998a).

The slope stability analyses of the landfill and proposed final cover were completed in accordance with CCR Title 27 Section 21090 and indicate that the stability of the landfill and final cover is adequate under both static and seismic loads. Integrating the estimated subgrade geometry beneath the landfill that was determined in earlier subsurface investigation at the site (EMCON, 1998b), the most critical failure geometry for the landfill as a whole is a roughly arcuate surface along Section A-A (Figure 7 of Appendix A) on the west face of the landfill, where a factor of safety of about 1.9 was calculated for cases the landfill is underlain by either gouge or colluvial soils and where the current (low) groundwater elevation condition (Figure 3 of Appendix A) is assumed. When the analyses consider a potentially higher groundwater condition within wastes, the factor of safety was reduced to about 1.84. Since the thickness of saturated wastes will decrease over time after placement of the final cover, landfill stability conditions will also improve.

The potential seismic-induced displacement of the landfill was evaluated using the peak ground acceleration associated with the Maximum Probable Earthquake (MPE) at the site. These analyses indicate that the landfill could deform by 10 inches and that slopes below the landfill could be displaced from 2.5 to 7.0 inches.

The stability of the proposed final cover system was considered addressing both the steepest and highest slopes that will exist on the landfill. These analyses were performed using the limit equilibrium procedures identified by Kramer (1999) and using the interface shear strength of the individual and combined (interface) cover components that are expected to be included in the final cover. The construction specifications for final closure construction will specify the minimum material strength and other performance criteria required to agree with these analyses.

The analyses completed indicate that stability of the proposed final cover configuration is stable under both static and seismic loads. The maximum calculated seismic displacement of the final cover (0.4 to 2 inches) is considered tolerable and could be accommodated by the geomembrane barrier layer of the final cover.

3.6 Construction Quality Assurance

The construction of the final cover system shall be carried out in accordance with the CQA Plan included in Appendix E. The implementation of the CQA Plan will provide documentation that suitable materials and standard construction practices are used to place the final cover system and to document that placement is consistent with the closure plan design specifications in 27 CCR, Section 20323 and 20324. Elements of the CQA Plan include: project description and definitions, qualifications and responsibilities, requirements for the final cover evaluation, inspection standards, testing frequencies, meetings and documentation. This information will be collected during construction of the final closure and incorporated into the project's final closure construction CQA Report which will be submitted to the appropriate agencies for recording and reporting purposes. The design professional who prepares the CQA Plan shall be a registered civil engineer or certified engineering geologist.

3.7 Drainage and Erosion Control

3.7.1 Drainage Control System Design

The primary function of the SCL drainage control system (shown on Figure 6) is to collect and convey storm water in a controlled manner to minimize erosion and potential infiltration of storm water into the refuse prism. The following sections describe the site hydrology, the existing drainage control features, and the proposed drainage control features.

3.7.1.1 Hydrology

A hydrology study for the proposed conditions at the site was conducted in accordance with 27 CCR, Section 20365. The objective of the hydrology study was to calculate storm water run-off for sizing and location information for the site's storm drain facilities at closure.

A rainfall intensity duration frequency curve for the SCL was obtained from the Department of Water Resources (December 1996). A description of the Rational Method for the methods of analyses is included in the introduction to Appendix G. A computer program developed by Advanced Engineering Software was used to compute the run-off. The hydrology study map indicating drainage sub-areas, discharge points and calculations for on-site and off-site flows is presented in Appendix G. A summary of the peak discharge rates is also included in the hydrology study calculations.

3.7.1.2 Existing Drainage Control System

Existing drainage ditches have been in place for several years and have been sized through trial and error, to accommodate maximum flows. A perimeter ditch exists along the toe of the landfill and directs runoff into two desilting basins. Although no formal calculations have been prepared to identify sediment quantities, history indicates that the existing ponds are adequate. Runoff is controlled using culverts and open ditches at the desilting basin outlets. Siltation fences are in place up-stream from the desilting basins to limit the quantity of sediment allowed to enter the desilting basins, consequently, minimizing the quantity of sediment being discharged into and from the basins. Additional erosion control methods include hay bales, silt fences, straw and seed.

3.7.1.3 Proposed Final Drainage Control System

The following describes modifications to the existing drainage structures required for incorporation with the proposed final grades and final cover system. The existing drainage facilities will be either decommissioned or removed and relocated. All drainage structures have been sized to accommodate run-off from a 100-year, 24-hour storm event. Hydraulic calculations completed to size the drainage structures are included in Appendix G. The proposed final drainage system is shown on Figure 6 and the associated details are shown on Figures 5, 8, and 9.

The contributing drainage areas for the SCL are divided into the following drainage areas: South Slope, East Area, Northeast Slope, Top Deck/North Slope, and Landfill Deck Access Road and portion of Top Deck and West Slope.

South Slope

The south slope drainage area originates at the top deck/slope hinge point (see Detail 8/D1 on Figure 5). The slope runoff is not concentrated, but flows evenly through the closure turf and down the Super Grip Net. At the toe of the slope, even collection of slope run-off occurs within a triangular shaped closure turf-lined channel, on the inside of the perimeter access road (see Detail 6/D1 on Figure 5). The drainage channel will collect runoff from the slope above and will then convey the runoff southerly along the inside of the perimeter road to a concrete downdrain. The runoff will then be directed to a riprap dissipater and then to an existing basin. The total south area is 1.27 acres, with a peak run-off of 5.08 CFS developed from nodes 2.00 through 2.15.

East Slope

The east drainage area originates at the hinge point between the top deck and the slope. The runoff is directed down the slope to a closure turf-lined triangular drainage channel on the inside of the perimeter access road (see Detail 6/D1 on Figure 5). The drainage channel will collect runoff from the slope above and will then convey the runoff southerly along the inside of the perimeter road to a concrete downdrain. The runoff will then be directed to a riprap dissipater and then to an existing basin. The total east area is 1.52 acres, with a peak run-off of 6.37 CFS developed from nodes 2.25 through 2.35.

Northeast Slope

The northeast drainage area originates at the beginning of the access road. The runoff is directed along the northeast perimeter road evenly towards the south basin. The level section of the road (see Detail 6/D1 on Figure 5) will collect runoff from the slope above and convey runoff to the south basin access road. The runoff will dissipate at the end of the basin access road into the existing basin. The total northeast area is 1.15 acres, with a peak runoff of 3.19 CFS developed from nodes 2.40 through 2.20.

Top Deck/North Slope

A portion of the North Slope drainage area originates on the northerly portion of the top deck. The runoff flows evenly by grade on the top deck and then down slope via the closure turf to a closure turf-lined bench (see Detail 6/D3). The flow will then be directed along the bench (westerly) to a concrete downdrain at the northwest corner of the expanded north desilting basin. The runoff velocity is dissipated by a riprap pad at the bottom of basin. The total Top Deck/North Slope area is 1.24 acres, with a peak run-off of 4.51CFS originating from nodes 1.40 through 1.50.

Landfill Deck Access Road and Portion of Top Deck

A portion (i.e., two-thirds) of the top deck area flow within the closure turf to two deck swales that direct deck flows to the upstream end of the deck access road closure turf-lined channel (see Detail 9/D1 on Figure 5). Flows are directed northerly along the inside edge of the deck access road to a paved interceptor that directs flow to a concrete inlet and downdrain (see Detail 7/D3 (inlet) and 1/D3 (downdrain) on Figure 9). The runoff will then be directed to a riprap dissipater into the existing basin. The total deck access road/top deck area is 2.30 acres, with a peak runoff of 6.28 CFS originating from nodes 1.00 through 1.35.

West Slope

A portion of the west drainage area originates near the top deck/slope hinge point at the northwesterly end of the landfill. The runoff then is directed by grade to the bottom of the slope to a closure turf-lined drainage channel on the inside of the perimeter access road. The runoff flows northerly and will then confluence with the runoff from the northerly slope bench, and will flow to a concrete inlet and concrete downdrain, and will direct the flow to the north desilting basin. The total west area is 0.49 acres, with a peak run-off of 1.63CFS originating from nodes 1.55 through 1.50

Basins

A hydrology study was completed by Bryan A. Stirrat & Associates (BAS) in 2002 on the SCL to develop the flows of a 100-year, 24-hour storm event, on the 6.56 acre site, as they flow into the north and south retention basins (see Appendix G). A second study was done in 2016 by SWT Engineering for the final closure plans with modified areas and newly designed retention basins. The flows (Q= CFS) for these back up calculations are based on the update design plans prepared by SWT Engineering (included in this document).

The south basin has a new 0.75 acre-foot capacity and accepts all flows from the south and east sides of the landfill (3.94 acres) for a total Q_{100} of 13.59 CFS. This basin has an 18-inch corrugated outlet pipe, assuming it is at a minimum 1% exit slope, which can handle 10.5 CFS with no head pressure. When the basin is filled with 30 inches of head pressure, then the inlet control will take over and the water will have a driving force of up to 14.79 CFS flowing through the pipe. With 30 inches of head pressure, there is still a remaining 18 inches of freeboard prior to spilling over the basin.

The north basin has 2.01 acre-foot capacity and accepts all flows from the deck, north and west sides of the landfill (5.90 acres) for a total Q_{100} of 17.72 CFS. This basin has an 18-inch corrugated outlet pipe, assuming it is at a minimum 1% exit slope, which can handle 10.5 CFS with no pressure head. When the pipe is flowing full, there is 60 inches of freeboard prior to spilling over the basin.

The holding capacity of the retarding basins and the flow rate of the outlet pipes allows peak flows to be slowly released to downstream outlet pipes without the need to consider overflow. Pre-Landfill conditions are not exceeded by Post-Closure/Developed conditions.

Runoff Evaluation of Closure Turf

Based on the analysis, the effects of closure turf on landfill stormwater runoff, for 100 yr – 24 hr event both peak flow (CFS) and stormwater runoff volume, have been analyzed by SWT.

The 100 yr peak flow rate to the south basin is 13.04 CFS with vegetative cover and 13.59 CFS with closure turf which is a nominal increase of 4.2% The 100 yr peak flow rate for the north basin increases by 2.7% from 17.26 CFS with vegetative cover to 17.72 CFS with closure turf. The proposed drainage system basins and outlet pipes exceeded the calculated flow rate to handle the de-minimus increase in peak flow rate. (see Table 1 in Appendix G-1 for hydrology analysis for both peak flow rate comparisons)

The stormwater runoff volumes for the south and north basins add volume from the 100 yr – 24 hr storm event both respectively increased by 10.3% or 0.19 acre-ft (306.53 C.Y.) of additional stormwater runoff volume (see Table 2 in Appendix G-1) from the vegetative cover to the proposed closure turf. Basin size change in runoff volume will easily be accommodated with the proposed basin.

As indicated above the change in design utilizes closure turf rendering a de-minimus increase in surface water volume. However, the additional water generated by using this vegetative layer component is well within the capacity of the basins. Therefore, no additional increase in capacity of the basins is necessary or needed.

3.7.2 Soil Loss Analysis

The proposed final cover has a layer in place for the vegetative layer cover in the form of closure turf as discussed in previous sections of this document. .

3.7.3 Erosion Control

The landfill closure design has erosion control features that will reduce the potential for soil erosion due to water and wind. These features include landfill grading, closure turf final cover system, closure turf-lined channels, and AC paved perimeter roads.

The decks will be graded for sheet flow run-off with a minimum slope of approximately three percent. No soil erosion is anticipated on deck areas or any other area covered with closure turf.

As stated previously in Section 3.3.2, a minimum 10-foot wide bench/perimeter road is located around the perimeter of the landfill. All landfill surface areas not covered by closure turf or AC paving will be vegetated with native grasses. The vegetation will protect the upper soil layer and minimize erosion through the vegetation root masses. The vegetation will consist of primarily native grasses with some shallow root shrubs.

Hydroseed (slurry) components are required to provide an effective germination environment as well as a protective environment for the seed. Wood and/or paper mulches used for slope hydroseeding will provide a short-term growing zone for the new seedlings. In addition to the mulch, a tackifier will be used to help bind or hold the mulch and seed to the slope. An environmentally safe organic tackifier (binder) which will not harm the short-term and long-term growth of grass is recommended. The seed mix will be applied at an approximate rate of 100 lbs. per acre consisting of the following: 60 pounds per acre of Blando Brome, 20 pounds per acre of Zorro Annual Fescue, 10 pounds per acre of “RK” Rose Clover and 10 pounds per acre of “RK” Crimson Clover. To provide a short-term high quality soil environment, fertilizer shall be blended in the hydroseed mix to provide the following coverage: 300 pounds per acre Ureaform (38-0-0) and 215 pounds per acre of Potassium Sulfate (0-0-50). The erosion control plan is shown on Figure 10 and associated details are shown on Figure 11.

3.8 Landfill Gas Control

As discussed in Section 2.1, a landfill gas control system is not in-place at the SCL. A traditional landfill gas control system is not proposed at closure; however, a landfill gas venting system is proposed to be placed below the geomembrane barrier layer. The venting system is designed to prevent potential landfill gas build-up under the LLDPE geomembrane. The system will be comprised of passive landfill gas vents constructed of HDPE pipe which are placed within the limits of the geomembrane cover section in the foundation layer and welded to the geomembrane to provide a water and gas tight seal. The gas will be collected in bilateral, perforated pipes placed in shallow gravel trenches located at high points in the cover system, and vented by the HDPE riser pipes. In addition, a series of passive vertical collection wells will also be installed into the refuse prism at varying depths. Figure 12 shows a layout of the horizontal and vertical collection features and details.

3.9 Landfill Gas Monitoring System

27 CCR, Section 20925 requires that subsurface gas monitoring wells (probes) be installed as part of closure around the perimeter of the landfill within the property limits but outside the limits of refuse with a spacing not to exceed 1,000 feet. As discussed in Section 2.1, two methane wells (probes) (LFGW-1 and LFGW-2) were installed at the site as part of the SWAT testing. The results of the SWAT investigation indicated negligible levels of organic contaminants in the landfill gas and no hazardous levels of landfill gas present at the landfill.

In order to maintain compliance with 27 CCR, Section 20925, three additional multiple depth gas monitoring wells were placed around the perimeter of the SCL in June 2012 as shown on Figures 2 and 12. The three probes (i.e., P-1, P-2, and P-3) were drilled and constructed in accordance with the well construction permit issued by the DOT. The locations of the gas monitoring probes are shown on Figures 2 and 12 and in Appendix C–Figure 1. More information related to the installation of the gas monitoring probes is included in Section 2.1. In addition, boring and construction logs are also included in Appendix C.

3.10 Groundwater/Surface Water Monitoring System

The groundwater/surface water monitoring system discussed in Section 2.2 will remain in-place at closure; therefore, no additional monitoring facilities will be required.

3.11 Site Security

In accordance with 27 CCR, Section 21135, a sign will be posted at the entrance gate to the SCL indicating that the existing on-site transfer station will be the only solid waste management facility at that location, and a number to call in case of emergency.

The SCL ceased landfill operations in 2000, in accordance with 27 CCR, Section 21135, all points of access to the site have been restricted as of the date of the final shipment of waste. Entrance to the site is secured along Fish Rock Road by a 6-foot high chain link fence, equipped with a locking gate at the entrance road to the transfer station/landfill to control site access. A cable gate at the beginning of the deck access ramp will prevent any errant traffic from the transfer station from accessing the landfill deck. Since other sides of the property are surrounded by steep canyon sides and thick forest, unauthorized entry is prevented. The existing security fence alignment and gate locations are shown on Figure 4. A sign will be installed at each access gate to indicate that no unauthorized access is allowed, and a number to call in case of emergency. These measures are intended to reduce incidents of vandalism and illegal disposal of wastes during the post-closure maintenance period.

3.12 Structure Removal/Decommissioning of Environmental Control Systems

Currently, there are no structures at the landfill requiring removal at closure. The transfer station facilities are to remain onsite after closure.

Also, there are no plans to decommission any of the environmental control systems at the SCL during the closure period. If deemed necessary, any decommissioning of boreholes or monitoring wells will be conducted in accordance with the appropriate regulatory agency requirements (including notifications) and in general accordance with post-closure maintenance plan procedures.

3.13 Closure Implementation Schedule

3.13.1 Waste Removal and Reconsolidation

Prior to final closure activities, work to remove and reconsolidate refuse from a portion of the SCL to the existing top deck area will be conducted. It is estimated that approximately 12,320 cy of refuse placed within the recognized footprint and located along the west perimeter (north to south) will be removed and reconsolidated onto the top deck. The capacity of the reconsolidation area is approximately 12,320 cy (refuse and soil). In accordance with 27 CCR, Section 20950 (a)(2)(B), all refuse and contaminated materials will be removed from the area so that it no longer poses a threat to water quality.

In accordance with 27 CCR, Section 21810 (c) and (d), a Work Plan (see Appendix I) was prepared in support of the waste removal and reconsolidation activities at the SCL and contains the following information at a minimum:

- an implementation schedule for refuse removal and reconsolidation activities;
- a characterization of the site conditions;
- a description of the excavation and material management procedures to be followed; and
- a description of health and safety procedures to be followed during refuse removal and reconsolidation activities.

3.13.1.1 Waste Removal and Reconsolidation Activities

Prior to waste removal and reconsolidation activities, the existing cover materials (estimated to vary in thickness from one foot to as much as several feet) will be scrapped off and stockpiled near the area designated for refuse disposal and used for cover soil. Following removal of the existing cover, to a point where approximately six inches remain, refuse and inter-mixed soil will be excavated using conventional excavation equipment. Upon removal, the refuse will be placed into end-dump trucks, or equivalent equipment, and transported to the re-consolidation area on the existing top deck of the landfill (see Figure 7). Removed materials (refuse and inter-mixed soil) will not be stockpiled upon removal and will be covered promptly throughout the day depending on the nature of the removed waste (e.g., highly odorous).

Health and safety procedures will be followed during waste removal and reconsolidation activities. A health and safety plan (HSP), which establishes policies and procedures to be followed during excavation and reconsolidation work, will be provided by the selected Contractor. Procedures outlined in the HSP will be enacted to protect site personnel as well as the public from potential hazards posed as part of the waste excavation and reconsolidation work.

Additional procedures related to dust control, removal and loading of waste, surface water control, spill prevention, and record keeping during the waste removal and reconsolidation are contained in the Work Plan, which is included as Appendix I.

3.13.2 Closure Process

The closure implementation schedule for the SCL (Table 1) delineates the estimated time frame to complete the closure tasks associated with each component of closure. The closure construction process will begin upon completion of final closure design and preparation and approval of the FCPCMP, selection of a qualified contractor, and the subsequent issuance of a Notice to Proceed. This construction schedule may differ from the selected Contractor's schedule based on the Contractor's equipment and personnel resources.

The type of equipment and required personnel expected to be utilized during closure construction includes, but is not limited to, the following:

- Types of Equipment
  - Scrapers
  - Dozers
  - Loaders
  - Compactors
  - Dump Trucks (or End Dump Trailers)
  - Water Trucks
  - Soil Conditioning and Screening Equipment (Grizzlies)
  - Forklift
  - Pickup Trucks
  - Excavator
  - Back Hoe
  - Driller
  - ATV
  - Hydroseeder
  - Concrete Truck
  - Concrete Pump
- Personnel
  - Construction Manager
  - Field Inspector(s)
  - Field Engineer(s)
  - Geotechnical Technician(s)
  - Labor Crews (including qualified geosynthetics welders)
  - Equipment Operators
  - Surveyors
  - Fabricators
  - Mechanics

Some of the closure activities can be conducted concurrently and some require completion of the final cover construction. Upon completion of the tasks described for closure, demobilization will begin. The estimated time frame for completion of closure construction activities for the site is estimated to be approximately 5 months (see Table 1), which is within the 180-day standard time frame required by 27 CCR, Section 21110 (e).

3.13.3 Construction Management

A construction manager will be located on-site during the period of construction. The construction manager, employed, or hired by the DOT, will be responsible for the supervision of construction of the various features included in the closure plan. The construction manager will coordinate the activities of the on-site general contractor and will provide liaison among the design engineers, CQA staff, the DOT, and other contractors.

3.13.3.1 Construction Management Team

The Construction Management team, either employed or hired by the DOT, will be headed by a construction manager. Other key staff will include a design engineer, a health and safety officer, a CQA engineer, and construction inspectors (in some cases it may be the same individual). A survey crew and a geotechnical CQA crew will also be present, as required.

3.13.3.2 Survey Control

The survey crew, under the general direction of the construction manager or superintendent, will be responsible for location of the closure plan facilities and for record drawing information. The crew will be responsible for establishing that the various components of the cover conform to grade and/or thickness requirements of the construction drawings and specifications. The DOT is to provide site survey control to the general contractor for the general contractor’s surveyor to utilize for the construction.

3.13.4 Construction Quality Assurance for Final Cover Placement

The construction specifications will include a final CQA Plan for final cover placement, as part of the final closure plan implementation. The final cover system shall be constructed in accordance with a CQA Plan certified by an appropriately registered engineer or certified engineering geologist. A geotechnical CQA staff, under the direction of the construction manager, will be on-site during the placement of the final cover to monitor compliance with cover design and installation methods included in the CQA Plan. The CQA personnel will have day-to-day responsibility to oversee cover placement and to evaluate whether the cover is constructed according to the project specifications.

3.14 California Environmental Quality Act Documentation

A Negative Declaration (ND) is being prepared for the closure of the SCL by the DOT’s consultant. The ND was certified the third quarter of 2013.

3.15 Labor Transition Plan

In accordance with 27 CCR, Section 21785, a Labor Transition Plan has been developed for the SCL and is included as Appendix K.

3.16 Recording

Upon completion of closure of the SCL, DOT will file a detailed description of the closed sites, including a map, with the Recorder of the County of Mendocino, the LEA, and the local agency that has been selected to maintain the Countywide Integrated Waste Management Plan (CIWMP) in accordance with 27 CCR, Section 21170. The site description, upon completion of closure of the site, will include:

- The date that closure was completed;
- The boundaries including height and depths of the filled area. If the site is closed in increments, the boundaries of each waste management unit;
- The location where the closure and post-closure plans can be obtained; and
- A statement that the future site use is restricted in accordance with the post-closure maintenance plan.

SECTION 4.0
POST-CLOSURE MAINTENANCE PLAN

4.0 POST-CLOSURE MAINTENANCE PLAN

4.1 Introduction

Post-closure maintenance of the closed SCL will be performed in accordance with applicable regulatory standards included in 27 CCR, Chapters 3 and 4, and 40 CFR Section 258.61. Post-closure maintenance activities for the SCL will consist of the following:

- Landfill Gas Migration Monitoring and Maintenance
- Landfill Gas Passive Vent System Monitoring and Maintenance
- Groundwater/Surface Water Systems Monitoring and Maintenance
- Stormwater Monitoring
- Final Cover Inspection and Maintenance
- Landfill Settlement Monitoring and Maintenance
- Access Road/Bench Maintenance
- Drainage Control System Inspection and Maintenance
- Site Security Inspection and Maintenance

4.2 Landfill Gas Migration Monitoring and Maintenance

Perimeter gas probes (LFGW-1 and LFGW-2) were installed around the landfill in 1994. Landfill gas has consistently been detected in LFGW-1 in the past two years, ranging from 5% to 35%. In June 2012, three additional monitoring probes were installed at the SCL. The three probes (P-1, P-2, and P-3) were drilled and constructed in accordance with the well construction permit issued by the County Health and Human Services Agency. More information related to the installation of the landfill gas monitoring probes is included in Section 2.1 and Appendix C. In addition, boring and construction logs are included in Appendix C. Currently, there are no landfill gas collection systems at the SCL. To prevent the potential buildup of landfill gas under the LLDPE geomembrane, a landfill gas venting system is proposed.

4.2.1 Landfill Gas Migration Monitoring Procedures

Monitoring procedures for the gas migration monitoring system will first include inspection of the monitoring probes for visual damage or deficiencies. All probes will monitor total hydrocarbons and TOCs, measured as methane. The monitoring events shall be conducted on a quarterly basis, as required by 27 CCR, Section 20933 (a).

At least one void volume will be evacuated from the probe cavity before gas concentrations are measured. The level of total hydrocarbons measured will be obtained by using a Gastech GT 201. Sample Forms A and B, which are to be used by the landfill gas monitoring personnel, are included in Appendix F. More information related to the landfill gas monitoring system is included in Section 2.1.

4.2.2 Landfill Gas Migration Monitoring Reporting

As required by 27 CCR, Section 20934 (a), the results of the gas migration monitoring program will be submitted to the LEA within 90 days of sampling unless the compliance levels of methane are exceeded. The results will include the concentration of TOCs, measured as methane, in each probe along with information regarding the general conditions under which the sample was obtained (e.g., ambient temperature, rising/falling barometric pressure, time of day). Should the compliance levels be exceeded in any probe, the above-mentioned regulatory agencies shall receive verbal notification of the problem within five working days. In accordance with 27 CCR, Section 20937, a letter will also be submitted to the LEA within ten working days, describing the nature and extent of the problem and the proposed remediation actions.

4.2.3 Maintenance of Landfill Gas Migration Monitoring System

The landfill gas migration monitoring wells will be inspected weekly during wet weather (October - April) and monthly during dry weather (May - September) in conjunction with scheduled monitoring tasks. System components will be repaired and replaced to maintain full system capabilities as intended at initial installation. Should a probe be deemed non-repairable it will be abandoned in-place and a new probe will be placed in the same proximity. In addition, each monitoring probe shall be pressure tested at least once per year to ensure proper functionality.

4.2.4 Structure Monitoring

There are no permanent structures located at the SCL. The transfer station is not confined in a building but located in the open air. A shipping container serves as the office, which is set above ground. Therefore, structure monitoring for migration of landfill gas is unnecessary.

4.3 Landfill Gas Passive Vent System Monitoring and Maintenance

The landfill gas passive vent control system will be regularly inspected in conjunction with scheduled migration monitoring tasks. Above grade system components will be repaired and replaced to maintain full system capabilities as intended at initial installation. Landfill gas flow from the vents will also be measured annually using a pitot tube through a port in the PVC pipe to determine the need for active control.

4.4 Groundwater/Surface Water Systems Monitoring and Maintenance

The purpose of the monitoring program discussed in this section is to detect potential migration of contaminants to the groundwater from the landfill. The groundwater monitoring program for the SCL is performed in accordance with 27 CCR, Chapter 3, Subchapter 3, and WDR Order No. 77-23 as amended by Order No. 93-83. More information related to groundwater monitoring and maintenance is included in Section 2.2.

4.4.1 Groundwater/Surface Water Monitoring

It is expected that modifications to program frequency and protocols will take place depending upon changing conditions, results of monitoring and advancing technology. This plan will be amended to include any changes in the monitoring program or modifications to the system including the installation of any proposed remediation systems. Any amendment to an approved plan will meet the requirements of 27 CCR, Section 21890 and shall, upon concurrence with the LEA, be approved by CalRecycle and the RWQCB.

4.4.1.1 Groundwater/Surface Water Monitoring Procedures

As discussed in Section 2.2, the present groundwater monitoring system consists of nine groundwater monitoring wells and the surface water monitoring program consists of two sampling points. Water samples are collected and analyzed as indicated in WDR Order No. 77-23 (Appendix B) as amended by Order No. 93-83. All monitoring will be performed in accordance with 27 CCR, Chapter 3, Subchapter 3. Sample collection, storage and analysis will be performed in accordance with the most recent version of Standard USEPA Methods and in accordance with WDR Order No. 77-23 and 93-83.

4.4.1.2 Groundwater/Surface Water Monitoring Reporting Procedures

Water quality monitoring quarterly reports will be submitted to the RWQCB 60 days after the end of the quarter. The reports will include all information as outlined in the existing WDR(s) currently in effect. If a release is noted, notification to the RWQCB will be provided by telephone within 48 hours and written within two weeks.

4.4.2 Groundwater Monitoring System Maintenance

Monitoring wells will be regularly inspected weekly during wet weather (October - April) and monthly during dry weather (May - September) in conjunction with scheduled monitoring tasks to determine if the wells have been tampered with or damaged and to verify that the well cover is secure. The wells will be inspected semi-annually for turbidity and total depth of well to determine if the casing is damaged or if the well needs cleaning or redevelopment. All necessary maintenance and/or repairs for wells will be documented on the Site Inspection Record form similar to the one included in Appendix F.

4.5 Stormwater Monitoring

The DOT will continue to collect stormwater samples from locations SW-1 and SW-2 (Figure 3 of Appendix A), and complete testing and reporting as required by the RWQCB. As part of the requirements for a General Industrial Permit, a Stormwater Pollution Prevention Plan (SWPPP) and Stormwater Monitoring Plan (SWMP) were prepared for the landfill in compliance with the NPDES permit requirements. The SWPPP and SWMP will be amended, as necessary, to reflect any future changes in the operation and design of the facility. The current version of these reports is available from the DOT upon request. A copy of the SWPPP is available upon request.

4.6 Final Cover Inspection and Maintenance Plan

The purpose of the completed final cover is to:

- Minimize storm water infiltration into and through the closed landfill,
- Isolate the buried wastes from the surface,
- Promote drainage,
- Minimize erosion or abrasion of the cover, and
- Accommodate settlement and subsidence so that cover integrity is maintained.
- Minimize the venting of landfill gas generated in the facility.

The primary purpose of the final cover maintenance procedures is to maintain the integrity of the completed final cover over the long-term and provide maintenance, scheduling and documentation so that materials and maintenance practices are consistent with the final cover design specifications. Based on the information provided by the turf cover's longevity is at least 30 years or the entire post–closure maintenance period at a minimum. Visual inspections of the final cover will be conducted weekly during wet weather (October - April) and monthly during dry weather (May - September) in conjunction with scheduled monitoring tasks and recorded on the Site Inspection Record form similar to the one included in Appendix F. Inspections will include identification of erosion and settlement problems. The Site Engineer will be responsible for documenting the location and extent of any repairs.

4.6.1 Periodic Leak Search/Identification of Problem Areas

Employees with access to the site will be instructed to report any final cover closure turf damage or uplift, geomembrane issues (puncture, tearing, or movement), erosion, ponding or unusual surface conditions to the Site Engineer, who will record the information in the Site Inspection Record form at the time they are observed. Scheduled, formal inspections will be performed on a quarterly basis to visually observe the following:

- Evidence of erosion (areas of soil above closure turf requiring repair/replanting),
- Visible depressions or ponded water (areas lacking free drainage),
- Exposed refuse (eroded portions of the low-hydraulic conductivity layer, damage to the closure turf and Super Grip Net geomembrane),
- Bunching and/or wrinkling of the closure turf,
- Thinning of the sand ballast placed over the closure turf,
- Areas damaged by equipment operation,
- Differential settlement and subsidence,
- Slope failure,
- Leachate seeps,
- Evidence of odors, and
- Evidence of cracks.

Special occurrences such as vandalism or accident damage will be reported on an Incident Report Form similar to the one in Appendix F. Additionally, the drainage control facilities will also be inspected weekly during wet weather (October - April) and monthly during dry weather (May - September) in conjunction with scheduled monitoring tasks, after a major storm event, seismic events and other natural disasters as described in Section 4.9, for improper operation and resultant effects on the surrounding final cover. A formal report of findings is to be presented to the Site Engineer.

4.6.2 Cover Repair

The proposed final cover design for the SCL consists of a minimum two-foot thick foundation layer, 60-mil LLDPE Super Grip Net geomembrane, and closure turf geotextile (Figure 5).

All final cover repair and/or reconstruction activities shall be conducted in a manner directed at maintaining the integrity of the as-built final cover system. Repair of fill materials should be performed in six to eight-inch layers consistent with the layers and procedures utilized during the original final cover construction. The methods of repair discussed in the subsequent paragraphs are recommended for the following three modes of final cover distress:

- Penetration into or through the final cover associated with any installation or maintenance of gas system components.
- Settlement related sags and drainage interruptions which interfere with the controlled flow and discharge of surface waters from the closed landfill surface.
- Closure turf final cover system damage or surface erosion as a result of intense rains.
- Fire damage results in turf cover and/or geomembrane replacement
- Local surficial slumping of slopes resulting from intense rainfall.
- Vertical and near vertical depressions in closure turf geosynthetics as a result of landfill settlement.

Final cover repair activities will be conducted and documented as specified in the CQA Plan included in Appendix E. A registered engineer or certified engineering geologist should inspect and certify repairs to the final cover.

Elective Penetration

Elective penetration of the final cover should be avoided whenever possible. If intrusion into or through the cover cannot be avoided, it will be initiated only after receiving approval from the Site Engineer. The RWQCB will be notified if the final cover needs elective penetration. All earthwork activities should be completed in accordance with the procedures contained in the specifications for final closure, which would be similar to those in the CQA Plan. All final cover repair and/or reconstruction activities shall be conducted in a manner directed to maintain the integrity of the as-built final cover system. Repair of fill materials should be performed in six to eight inch layers consistent with the layers and procedures utilized during the original final cover construction.

Care should be taken during excavation not to damage the geomembrane barrier layer beyond that which is reasonably necessary. Damaged geomembrane areas will need to be covered with new membrane material placed, overlapped and overlap seams welded in accordance with the specifications included in the CQA Plan.

Sags, Ponding, Drainage Interruptions Sand Ballast Replenishment

Any repair of significant depressions in the final cover will be completed in the landfill area immediately prior to the rainy season (October to April). If significant sags or depressions are identified during other times of the year, the Site Engineer will accurately locate the limits of the depressions both horizontally and vertically. Permanent repairs will be indicated by the need for continued maintenance of a sagging area, and will be left to the judgment of the Site Engineer. If required, the Site Engineer, LEA, CalRecycle and RWQCB will be notified for approval of the proposed modifications to the final cover. Permanent repairs, if needed, will be done to at least the nominal 3% grade, or to the original design slope. Additionally, the overall grades will be maintained annually to preserve the 3% minimum grade and maintain proper drainage.

A channel capable of draining the lowest point of the sag will be constructed or additional soils will be placed such that the intended flow of surface water is unimpeded. The Site Engineer will be responsible for fill placement occurring only in the area of the sag; only that fill which is necessary to facilitate drainage is placed; and that sufficient record of the depths and limits of fill placement are kept. The depth and limit records should always be available so that the appropriate area can be re-excavated and permanently repaired during the annual maintenance period, if necessary, as discussed below.

The permanent repair of sags and/or ponding will be performed by removing the closure turf geotextile, exposing the LLDPE and pulling back the LLDPE to expose the depressed foundation layer. After the limits of the sag are identified, the drainage gradients would be reestablished by excavating to the undisturbed soils and adding select material necessary to maintain a positive gradient and rebuilding to at least a three percent gradient by placing additional foundation soil material at a minimum 90 percent relative compaction. The LLDPE and closure turf would then be re-laid over subgrade, joined and sand ballast placed on closure turf to adjacent material, per CQA Plan requirements. The repair of sags and ponds, when necessary, in the final cover shall be conducted in accordance with the procedures presented in the specifications and CQA Plan developed for final closure. A CQA inspector shall inspect all fill placed in the foundation layer and the LLDPE installation.

Local Surficial Slumping

After the annual rainy season, all surficial slumping shall be repaired in conformance with the recommendations presented below and the construction specifications for final closure.

If the surficial instability extends below the closure turf, the repair shall be completed in such a fashion as to reconstruct the synthetic barrier layer (LLDPE) and the underlying foundation layer soils.

Alternate 1 (Parallel Lifts) - If the repair area is accessible to track-type equipment, the loose soils can be removed and the exposed area track walked to achieve compaction. The removed soils should be dried or watered to the design moisture content, as required, and placed in thin lifts parallel to the angle of the slope. Each lift should be compacted by the equipment to at least 90% of maximum density. When grade is reached, track walking of the final lift should extend beyond the perimeters of the distressed area.

Alternate 2 (Horizontal Lifts) - In lieu of using large construction grading equipment, hand labor for restoration of the slope may be used. The loose or saturated soils should be cleaned out and a level bench cut into competent material at the base of the slump. The removed soils should then be brought to the design moisture content (wetting or drying, as required), placed in horizontal lifts of no more than five inches and compacted by hand operated mechanical tampers. As the fill is raised, it should be keyed into competent material.

Any repair involving removal of the LLDPE must be approved by the Site Engineer and repaired/reinstalled with closure turf and sand infill in accordance with the specifications included in the closure construction documents.

CalRecycle, the LEA, and the RWQCB will be notified in writing of any major repairs involving the final cover

4.6.3 Sand Ballast Maintenance

Sand Ballast

The operator will inspect the surface of the Closure Turf (Engineered Turf) for signs of eroded material (see forms in Appendix F). The primary feature to keeping the Closure Turf in place is a ballast material, generally a fine grained sand or crumb rubber. Due to the potentially harsh weather conditions at the South Coast Landfill, a sand ballast was selected to hold the Closure Turf in place. The Operator will maintain a quantity of sand on site in a stockpile, for placement in the Closure Turf in areas where wind and water erosion have displaced the material. The stockpile will be covered to reduce wind and/or drainage erosional loss.

The sand ballast fill (sand infill) creates a level of protection against severe weather conditions such as high rainfall events and reduces heat absorption during periods of high temperatures. The sand infill is able to support traffic loads (per specs) and a system of non-exposure. In order for the sand fill to keep this level of operation the thickness level must be kept at a minimum standard to be operationally viable.

4.7 Landfill Settlement Monitoring and Maintenance

4.7.1 Survey Record

Regulatory requirements included in 27 CCR, Sections 21090(e) and 21170 dictate that upon completion of closure construction activities, a survey record of the closed landfill be established and recorded with the title of the property, at the County Recorder's office and copies be made available to CalRecycle, the LEA and kept in the site's operating record. The as-built drawings will be certified by a registered engineer or certified engineering geologist. The survey of record will include the following information:

- The date closure construction was completed;
- Boundaries of the disposal area;
- The location and telephone number of where the closure and post-closure plans can be obtained; and
- A statement that future site use is restricted in accordance with the post-closure maintenance plan.

A discussion of the site's operating record requirements is included in Section 8.0.

4.7.2 Survey/Settlement Monuments

In accordance with 27 CCR, Section 20950 (d), at least two permanent monuments are to be installed at the landfill so that facilities constructed during closure can be located, the location and elevation of wastes, containment and drainage structures, environmental control and monitoring facilities can be determined throughout the post-closure maintenance period, and controls can be provided from which to monitor future landfill settlement.

After completion of the final cover, settlement monuments will also be set on the landfill in the disposal area as shown on Figure 6. These monuments will be used to monitor settlement within the closed landfill disposal area and will allow for a determination of the actual settlement that occurs over the post-closure maintenance period.

The disposal area settlement monuments will have been surveyed upon completion of all closure construction activities. The monuments will be surveyed for horizontal and vertical control so that an accurate disposal area topographic map can be developed.

Additionally, 27 CCR, Section 21142 (b) requires operators to produce iso-settlement maps every five years throughout the post-closure maintenance period or until settlement has ceased. The settlement and lateral movement of each monument will be recorded on a form similar to Form A, included in Appendix F. Results of all monitoring data will be maintained as part of the site's operating record kept in the main office of the DOT.

Routine quarterly inspection of the monuments will be performed to ensure that the monuments are intact and usable. The monument will be cleared of all debris and vegetation to allow for visual location of the monument and accurate readings. Should a monument be damaged or missing, a new monument will be placed.

If a monument is within an area requiring regrading and/or other reconstruction, it will be replaced at approximately the same horizontal location and a note will be placed to identify the new elevation. If a settlement monument needs to be moved to a new location, control of the vertical thickness shall be consistent with tolerances of the overall survey record typical of the site.

4.8 Access Road/Bench Maintenance

The site perimeter, deck access roads and bench will require general maintenance. The access roads will be inspected yearly and should be seal coated, if necessary, every five years, or more frequently.

4.9 Drainage Control System Inspection and Maintenance

The following sections delineate the various maintenance activities to be performed on the landfill drainage control facilities for the site.

After the drainage control system has been in service for several years, a more definitive inspection and maintenance schedule can be developed identifying those areas that must be inspected quarterly and those areas that must be inspected prior to and after a storm and those areas that require maintenance before the wet season. In general, the system will be regularly inspected weekly during wet weather (October - April) and monthly during dry weather (May - September) in conjunction with scheduled monitoring tasks, and after a significant storm event, seismic event or natural disaster.

4.9.1 Deck Drainage Control System Features Maintenance

Inspection for proper deck surface drainage will be performed in conjunction with the final cover procedures described in Section 4.6.

4.9.2 Downdrains, Drainage Channels and Ditches

SCL has a series of Closure Turf lined channels. The inspection and maintenance of these channels will be the same as the schedule for the final cover inspection discussed in Section 4.6.

A visual inspection of each open channel and downdrain, exterior to the closure turf area, will be conducted to identify any of the following deficiencies:

- Cracking
- Settlement
- Spalling

The following corrective measures can be taken for deficiencies identified during the inspection.

- Cracking
    - Construction of expansion/control joints.
    - Resurface
    - Replacement
- Settlement (none are anticipated for drainage features outside limits of refuse)
    - Grout injection.
    - Complete replacement with subgrade rework.

- <u>Spalling</u>
    - Sandblast affected area and resurface.
    - Sawcut and remove affected area, dowel into existing undamaged section and resurface.

4.9.3 Desilting Basin Maintenance

Silt and debris will be removed on an as needed basis from the desilting basin to maintain capacity. Silt material may be stockpiled on-site and used for future erosion repairs. The desilting basin drainage structures will also be maintained in accordance with the procedures outlined in this Section.

4.9.4 Overall Drainage Control System Maintenance Schedule

The on-site drainage control facilities must be free of excessive debris and operational at all times. In order to provide the desired protection against flooding and erosion damage, routine (i.e., quarterly and monthly) inspections of the drainage control system will be conducted. Site Inspection Record (Appendix F) forms will be placed in the record library as required by 40 CFR, 258.29.

4.10 <u>Site Security Inspection and Maintenance</u>

Security fencing, access gates and signs will be inspected quarterly to ensure that the integrity of site security has been maintained. All groundwater monitoring wells will have locking well covers. The gates will be inspected to ensure that the locks are intact. Any necessary repairs or replacements will be made during the quarterly inspection.

4.11 <u>Equipment and Labor Requirements</u>

4.11.1 Equipment

Equipment, instruments, and tools expected to be used for post-closure maintenance will be kept onsite. Any required equipment, not kept on site, will be available from other DOT facilities or rented on an "as needed" basis.

4.11.2 Labor

The work force necessary to monitor and maintain the SCL during post-closure will be directed and coordinated by the Site Engineer. Staff will be assigned to each of the following activities:

- Final cover, drainage and general maintenance
- Environmental monitoring and reporting

The maintenance and monitoring personnel will be under the direction of the Site Engineer.

The projected maintenance schedule for each of the post-closure activities is shown on Table 2. The primary purpose of this schedule is to identify the frequency of mandatory inspections for the various systems. The frequency of monitoring the gas migration monitoring system, sampling and analysis for the groundwater/surface water monitoring system and survey of the settlement monuments will be in accordance with the monitoring schedule presented in Table 3.

4.11.3 Materials

Listed below are recommended maintenance materials to be kept on-site for each of the post-closure plan activities.

- Final Cover
  - Stockpiled soil materials
  - Sand
  - Gravel
  - Road base material
  - Ready mix concrete cement
  - Erosion control seed
  - Sheet plastic
  - Sandbags
- Drainage Facilities
  - Cement
  - Asphalt mastic compound
  - Cement grout
  - Backfill material
  - Corrosion resistant paint
- Environmental Monitoring
  - Sampling supplies
  - Spare parts for testing equipment
  - Spare well/probe head parts

SECTION 5.0
POST-CLOSURE EMERGENCY RESPONSE PLAN

5.0 POST-CLOSURE EMERGENCY RESPONSE PLAN

5.1 Purpose and Scope

This Emergency Response Plan (ERP) was prepared in accordance with 27 CCR, Section 21130, as part of the SCL FCPCMP. The ERP identifies occurrences that may exceed the design of the site and endanger public health or the environment. The ERP also sets forth actions which will minimize the effects of these catastrophic events. The provisions of this ERP will be carried out immediately whenever an event occurs such as a fire, explosion, flood, earthquake, vandalism, surface drainage problems or release of any waste product which may threaten public health and/or the environment. 27 CCR, Section 21130 also requires provisions for collapse or failure of artificial or natural dikes, levees or dams. Provisions for this have not been included since such facilities are not located downstream or adjacent to the SCL. The ERP will be kept in the operating record at the main office of the DOT.

5.2 Site Engineer (SE)

A Site Engineer (SE) and an alternate will be designated by the responsible organization, which is DOT. The SE and alternate will be trained to handle all emergency situations. The main responsibility of the SE is to oversee the management of all emergency response procedures implemented at the landfill. The SE is required to be thoroughly familiar with all aspects of the ERP as well as all post-closure maintenance activities, the location and characteristics of buried refuse, the location of facility records and the overall site layout. In addition, the SE shall be given the authority to commit any of the available resources necessary to carry out the ERP.

5.3 Emergency Response Notification Procedure

When any member of the site's maintenance personnel discover or witness an event which constitutes an emergency situation, they shall determine the nature, source, and location of the emergency situation and immediately report the occurrence to the SE. The SE will notify all of the appropriate response agencies to provide assistance to site personnel. If an emergency event occurs when field personnel are not on-site, the general public will be able to use the telephone number posted on a sign to be located at the site entrance to notify the SE. In an emergency when personnel are not on-site, contact 911.

Emergency Contacts

Emergency Coordinator:	Geoffrey Brunet, Site Engineer	
	Work:	(707) 234-2816
Alternate Emergency Coordinator:	Amber Munoz, Deputy Director Dept. of Transportation	
	Work:	(707) 234-2838
	Mobile:	(707) 367-8445

5.4 Emergency Response Procedures

General emergency response procedures for fire, explosions, earthquakes, floods, vandalism, release of waste products to air and soil, or surface drainage problems, are described below.

- Remove all non-essential employees from the vicinity of the incident.
- Remove non-essential equipment, if it can be done safely, from the vicinity of the incident.
- Determine the degree of risk to human health and safety for persons to be working in the vicinity of the emergency prior to sending in people to control the emergency.
- Determine the immediate risk to the environment. If the risk is imminent, it should be determined whether or not the emergency can be safely isolated to minimize damage to the environment.
- Determine the appropriate agencies to call given a particular type of emergency.
- Determine and identify the nearest source of available equipment and supplies for responding to the incident.
- When practicable, the SE may utilize on-site personnel to control the incident.
- The SE or his designee will be responsible for site personnel safety. Site personnel will communicate any damage and/or injury reports to the SE and will coordinate all emergency actions directed by the SE.
- Site personnel will be available for inspection of the landfill after an incident occurs. All crew members will be supplied with appropriate personal protective clothing, as required by the SE, when conducting inspections of the site for possible design failure. All findings will be reported to the SE for action.
- The SE will immediately begin surveillance in those areas of the facility affected by the incident. In addition, monitoring will be conducted to prevent an incident from affecting other areas of the facility or adjacent properties.
- Shut down any control system, such as the gas migration control or treatment systems that have been damaged during an incident.
- The operator will maintain a small stockpile of final cover material for those events which may require immediate cover placement to minimize waste releases, to repair severe cracks, or to fill in large erosion gullies.

The general types of equipment and materials that should be available for emergencies include a first aid kit, CPR mask, fire extinguisher, final cover material, and sandbags. All action steps for the following emergency procedures, should be done in as short a time period as possible to prevent adverse health and safety affects to the public. The first aid kit, CPR mask and fire extinguisher are located in the transfer station attendant building. The assembly area in the event of an evacuation is located adjacent to the front gate. In case of fire or serious medical emergency, the Site Attendant is authorized to close and evacuate the facility.

The existing hazards within the transfer station are:

- Possible small quantities of household hazardous waste from load checking stored inside the Hazardous Materials Storage Building.
- Small biohazardous waste container for discovered sharps is inside the transfer station attendant building.

5.4.1 Fire and/or Explosions

The following procedures will be followed during incidents of fire and/or explosions:

- Contact the South Coast Fire Protection District, even if on-site capabilities are deemed adequate to extinguish fires or control future explosions. On-site personnel will be instructed to follow the fire department’s directions and give their full cooperation.
- In the event of an off-site fire near the landfill, such as a forest fire, the operator will lend its personnel and equipment, if available, to the South Coast Fire Protection District to fight the fire.
- The fire will be extinguished and the effects of the fire or explosion will be mitigated.
- The following are the general telephone numbers for emergency response agencies:

South Coast Fire Protection District	(707) 884-4700
California Department of Forestry, Howard Forest	(707) 459-7408
Pacific Gas & Electric (24-Hour Emergency)	(800) 743-5000
Pacific Gas & Electric (24-Hour Electrical Outages)	(800) 743-5002
Sheriff	(707) 463-4086
Mendocino County Animal Control (contact Sheriff)	(707) 463-4086
Office of Emergency Services (Spills)	(800) 852-7550
Mendocino County Department of Transportation	(707) 463-4363
Mendocino County Environmental Health	(707) 463-4466

5.4.2 Flood

If the site's existing stormwater system is inadequate in diverting flood waters away from the site, the following procedures will be followed:

- Additional berms may be constructed in areas prone to flooding.
- If additional berms are ineffective, the operator may cut a diversion channel to avoid inundation of the refuse cell.
- Sandbags may be used in conjunction with berms or diversion channels.
- If the emergency is due to a flood that is from a 100-year/24-hour storm event or less, this is an indication that the drainage and flood protection design for the facility may be inadequate. A design review shall be performed and, if appropriate, all necessary modifications to drainage and flood protection facilities will be made.

5.4.3 Earthquake

The following procedures will be performed following an earthquake incident:

- Employees driving in the field during an earthquake should stop their vehicle and get out, if it can be done in a safe manner.
- After the earthquake has subsided, site personnel shall report to the site entrance gate. If medical care is required, the procedures in Section 5.7 shall be followed. An inspection of the site shall then be made and a report given to the SE.

- Cracks observed in the final cover after an earthquake should be inspected with a combustible gas analyzer to determine the integrity of the LLDPE and if extensive repairs are required. The location of venting and the gas concentrations will be determined and reported to the SE. Excavation and refill of the smaller surface cracks can be completed immediately. More extensive corrective actions will be authorized by the SE in accordance with the CQA Plan included in Appendix E.
- If the emergency is due to a Holocene magnitude earthquake, this is an indication that the slope or foundation stability design for the facility may be inadequate. A design review shall be performed and, if appropriate, all necessary modifications to the landfill's slope and foundation will be made.

5.4.4 Surface Drainage Problems

In the event of a surface drainage problem, the following procedures shall be followed:

- The DOT will investigate the problem and determine a necessary course of action.
- If a surface inlet is blocked with debris, all necessary labor forces and equipment will be implemented under the direction of the operator to remove the blockage.
- If a storm drain is damaged, a plan will be provided by the SE to repair the problem.
- After the drainage problem is corrected, an assessment of possible damage or erosion will be conducted and all necessary repairs will be made.
- If the emergency is due to a flood that is from a 100-year/24-hour storm event or less, this is an indication that the drainage facilities design for the landfill may be inadequate. A design review shall be performed and, if appropriate, all necessary modifications to drainage facilities will be made.

5.4.5 Vandalism

The following procedures will be followed during incidents of vandalism:

- Repair (i.e., replace, repaint) any portion of the property which has been vandalized.
- Immediately repair any vandalism which affects site security and/or environmental control/monitoring systems.
- A review of the adequacy of site security after each incident shall be performed.
- In case of damage to the front gate, there is a set of replacement gates stored on-site.

5.4.6 Underground Fires

Underground landfill fires occur due to air intrusion and moisture into the refuse cell. Indicators of underground fires are as follows:

- Unusual depression-like settlement with tension cracks.
- Smoke/steam.
- Unusual odor.
- Gas monitoring probe readings.

Should any of the above indicators be noted, the first course of action should be placement of soil to cover the depression and/or cracks. If this measure does not correct the problem, additional measures may be taken.

5.5 Emergency Response Plan Orientation

Contacts should be made with appropriate emergency response agency representatives and the following information should be conveyed:

- Establish understandings between the responding Police/Sheriff and Fire Departments and designate which agency has primary emergency authority during an incident.
- Establish understandings between emergency response teams, emergency response contractors, and equipment suppliers for smooth coordination of emergency response actions.

5.6 Evacuation Procedures

During and/or after an incident, the SE in consultation with other emergency personnel, such as the Fire Department, will assess the potential for injury to any persons located on adjacent properties. If the assessment concludes that an imminent threat to public health is possible, an evacuation of the nearby area will be initiated. Situations which warrant partial or complete evacuation of site personnel and/or local residents are as follows:

- Explosions resulting in airborne debris including particles and large fragments.
- Fires that cannot be readily contained or are spreading to other parts of the facility; or when fire could generate highly toxic fumes, or create a danger of igniting potentially explosive substances which may be stored on-site.

The SE will immediately notify the responding Sheriff Department and all other appropriate emergency response agencies. The SE will check that the entrance gate is unlocked and locked as required.

In case of fire or serious medical emergency, the Site Attendant is authorized to close and evacuate the facility to a designated area.

5.7 Medical Care Procedures

Should an emergency situation result in personal injury, immediate steps will be taken to determine the cause and extent of the injury and to render first aid. The SE will be notified in all cases and the paramedics will be called when required. If further medical attention is necessary, the injured person will be transported to the designated medical facility. Designated facilities for this site are:

Ukiah Valley Medical Center
275 Hospital Drive
Ukiah, California 95482
(707) 462-3111

Redwood Coast Medical Services
46900 Ocean Drive
Gualala, California 95445
(707) 884-4005

5.8 Amendments to the Emergency Response Plan

The ERP will be reviewed and can be amended, in accordance with the criteria listed in 27 CCR, Section 21130 and listed below:

- A failure or release occurs for which the plan did not provide an appropriate response.
- The post-closure use and/or structures on the site change and these changes are not addressed in the existing plan.

- The LEA, the RWQCB or CalRecycle notifies the operator in writing that the current emergency response plan is inadequate under the provisions of this section. The notifying agency shall include within the written notice those items that must be considered for the plan to be in compliance with this section. The operator shall submit an amended ERP to the LEA, the RWQCB and CalRecycle within 30 days of receipt of notification that the plan is inadequate.

Whenever the ERP is amended, a written copy will be submitted to the LEA, the RWQCB and CalRecycle.

SECTION 6.0
CLOSURE/POST-CLOSURE MAINTENANCE COST ESTIMATE

6.0 CLOSURE/POST-CLOSURE MAINTENANCE COST ESTIMATE

6.1 Introduction

In order to establish the basis for the proper level of funding to close and provide post-closure maintenance for the SCL in an environmentally sound manner, a cost estimate was prepared for closure plan improvements and post-closure maintenance procedures presented in Sections 3.0 and 4.0 of this plan. This estimate was then combined with an estimate for construction management and quality assurance services, including contingency, to determine the total closure construction cost for a FCPCMP.

6.2 Closure Cost Estimate

The final closure construction cost estimate is shown on Table 4. The components and systems required for closure of the SCL include the final cover system, final grading, refuse removal and reconsolidation, construction support tasks (i.e., construction management, CQA, etc.), drainage and erosion control systems, installation of survey/settlement monuments, landfill gas control and monitoring systems, groundwater/surface water monitoring systems, and site security. The SCL disposal area that will be closed is approximately 6.6 acres.

6.3 Closure Cost Disbursement Schedule

In accordance with 27 CCR, Section 21800(d), the operator is required to provide a detailed schedule for disbursement of funds for closure for advance payment of closure construction activities to be performed in accordance with this FCPCMP or to obtain reimbursement of costs to be performed in accordance with this FCPCMP. Table 5 presents the disbursement schedule for the SCL closure construction.

6.4 Post-Closure Maintenance Cost Estimate

The post-closure maintenance cost estimate has been prepared utilizing information contained in Section 4.0 and estimates of manpower, materials and equipment to maintain the SCL in compliance with current applicable regulations.

The total annual maintenance and monitoring cost estimate for post-closure is shown on Table 6 (with back-up documentation). These costs are projected in 2016 dollars, assuming no change in the regulatory environment with respect to the SCL. It should be noted that the maintenance and monitoring costs presented have been projected utilizing current regulations and applicable requirements. In the event that changes occur in the regulatory conditions pertaining to the SCL, these estimates will be adjusted accordingly, if necessary, and submitted to CalRecycle, the LEA and the RWQCB.

6.5 Demonstration of Financial Responsibility

In accordance with 27 CCR Sections 22206, 22211, 22225, 22228 and Subtitle D, Subpart G, DOT must demonstrate financial responsibility for closure and post-closure costs. The DOT prepared an Inflation Factor report for closure and post-closure maintenance costs and submitted to CalRecycle. The County of Mendocino maintains an Enterprise Fund for closure of the SCL and post-closure maintenance and corrective action of the SCL is currently financed through a Pledge of Revenue (Resolution No. 05-052 included in Appendix J).

6.6 Estimated Cost for Known or Reasonably Foreseeable Release Mitigation (27 CCR, Section 20380(B))

In accordance with 27 CCR, Section 20380(b), the SCL has established and maintains assurance of financial responsibility for initiating and completing corrective action for all known or reasonably foreseeable releases from the SCL. The cost estimate is intended to provide a basis for the compliance with 27 CCR, Chapter 6, Subchapter 2, Article 4, financial assurance requirements. As discussed in Section 6.5, a Pledge of Revenue Agreement is the financial assurance mechanism for corrective action. A copy of the Financial Assurance Review Report (GLA, 2004) is included in Appendix L.

6.7 Non- Water Release Corrective Action Plan and Cost Estimate (27 CCR, Section 22101)

Based on State regulations established within 27 CCR, Section 22101, effective July 1, 2011, on or before the date of the first permit review or revision plan review, the State now requires preparation of a corrective action plan along with a cost estimate for all known or reasonably foreseeable non-water releases from a solid waste landfill. Under this regulation, the operator may provide the cost and financial assurance for complete replacement of the final cover system, or may submit a site-specific non-water corrective action plan that addresses the potential damage that can be caused by a reasonably foreseeable causal event that exceeds the landfill’s existing design standard. Based on the potential damage, a cost estimate is developed for restoration of the landfill to its design standard. Potential reasonably foreseeable causal event to be considered, as identified by CalRecycle’s Proposed Best Management Practices for Preparing Site-Specific Non-Water Quality Corrective Action Plans (NWQCAP) include earthquakes, precipitation, flooding, fire, or tsunami. A Non-Water Release Corrective Action Plan (NWRCAP) (SHN, May 2015) for SCL was recently submitted and approved by the regulatory agencies (see Appendix L).

SECTION 7.0
PROFESSIONAL CERTIFICATION OF ACCURACY

7.0 PROFESSIONAL CERTIFICATION OF ACCURACY

Current regulations require that a registered civil engineer or a certified engineering geologist prepare and certify the accuracy of closure plans for all Class III landfills. The South Coast Landfill Final Closure/Post-Closure Maintenance Plan (dated November 2016) has been prepared in accordance with 27 CCR, Chapters 3 and 4 and 40 CFR, Part 258 under the supervision of Mr. Michael A. Cullinane, a California Registered Civil Engineer, Registration No. 41981.

Respectfully Submitted:

MICHAEL A. CULLINANE
No. C41981
Exp. 3/31/10
CIVIL

Michael A. Cullinane, P.E.
R.C.E. No. 41981
SWT Engineering

SECTION 8.0
RECORDKEEPING

8.0 RECORDKEEPING

8.1 Operating Record

A complete operating record will be maintained in accordance with 27 CCR, Section 20515 and Federal regulations under 40 CFR, Section 258.29 Recordkeeping Requirements. Closure and post-closure activities will be documented in the operating record and will include, but will not be limited to, the following information:

- Any location restriction demonstration required by 27 CCR, Section 20270;
- Inspection records, training records, and notification procedures as they pertain to hazardous wastes as required by 27 CCR, Section 20870;
- Gas monitoring results from monitoring and any remediation plans required by 27 CCR, Section 20919;
- Closure and post-closure maintenance plans as required by 27 CCR, Section 21780, notice of intent to close the unit as required by 27 CCR, Section 21135, notice of certification of closure as required by 27 CCR, Section 21880, deed notation as required by 27 CCR, Section 21170, and any gas monitoring, testing, or analytical data as required by 40 CFR 258.61;
- Demonstration, certification, finding, monitoring, testing or analytical data required by 40 CFR 258 Subpart E (Sections 258.50 - 258.58);
- Closure and post-closure care plans and any monitoring, testing, or analytical data required by 40 CFR 258 Sections 258.60 - 258.61; and
- Any cost estimates and financial assurance documentation required by 27 CCR, Sections 22221, 22226, 22101, 22103, 21820 and 21840.

Approvals, determinations, and other requirements authorized by the LEA under Chapter 3, Subchapter 4 shall be documented in writing to the operator and placed in the operating record in accordance with 27 CCR, Section 20517.

It should be noted that the operating record also contains information compiled during active operations as required by applicable state and federal regulations.

8.2 Location and Inspection of Operating Records

The operating records are and will be maintained at the Mendocino County Department of Transportation Office in the central files. The street address is 340 Lake Mendocino Drive, Ukiah, California 95482. These records are available during normal business hours for inspection by authorized representatives of those regulatory agencies having jurisdiction over the SCL.

SECTION 9.0
REFERENCES

9.0 REFERENCES

1. California Code of Regulations, Title 22, Division 4, Chapter 30.
2. California Code of Regulations, Title 27.
3. EPA Regulations Title 40 Code of Federal Regulations, Parts 257 and 258 (Subtitle D).
4. California Integrated Waste Management Board, 1995, "Solid Waste Facilities Permit No. 23-AA-0018."
5. California Regional Water Quality Control Board – Central Valley Region, 1995, "Waste Discharge Requirements Order No. 77-23."
6. Bryan A. Stirrat & Associates, 2003, Final Closure and Post-Closure Maintenance Plan for the South Coast Landfill.
7. GeoLogic Associates, 2012, Gas Probe Installation Report for South Coast Landfill.
8. GeoLogic Associates, 2012, Revised Final Cover Analysis, South Coast Landfill.
9. SHN Consulting Engineers & Geologists, Inc., 2016, Second Quarter 2016 Monitoring Report, South Coast Landfill.
10. GeoLogic Associates, 2004, Financial Assurance Review Potential Corrective Action Program and Post-Closure Maintenance Costs, South Coast, Laytonville and Caspar Landfills.
11. SHN Consulting Engineering & Geologists, Inc., 2015, Site-Specific Non-Water Release Corrective Action Plan South Coast Landfill

TABLES

TABLE 1
SOUTH COAST LANDFILL
CLOSURE IMPLEMENTATION SCHEDULE

	MONTH 1	MONTH 2	MONTH 3	MONTH 4	MONTH 5
Clear And Grub	▨				
Stockpile Removal	▨				
Refuse Removal	▨				
Import Soil		▨			
Foundation Layer		▨			
Gas System		▨			
Geosythetics			▨		
Final Cover				▨	
Drainage Features				▨	
Acess Roads					▨
Erosion Control					▨

TABLE 2
SOUTH COAST LANDFILL
POST-CLOSURE MAINTENANCE SCHEDULE

MAINTENANCE ACTIVITY	FREQUENCY
FINAL COVER MAINTENANCE	
A. Inspection	2, 4, 6
B. Repair	5
ACCESS ROADS	
A. Inspection	1
B. Repair	5
C. Regrading	5
DRAINAGE FACILITIES MAINTENANCE	
A. Inspection	2, 4, 6
B. Debris Build-up	4, 5
C. Perimeter Drainage System	5
D. Desilting Basin	5
LANDFILL GAS PASSIVE VENT SYSTEM	
A. Inspection	2, 4, 6
LANDSCAPE MAINTENANCE	
A. Weed Control	5
B. Rodent Control	5
C. Reseeding and Mulching	5
SURVEY MONUMENTATION MAINTENANCE	
A. Inspection	1
B. Disposal Area Monuments	3, 5
PERIMETER FENCE MAINTENANCE	
A. Inspection	1
B. Maintenance and Repair	1, 5
GAS/GROUNDWATER MONITORING WELL MAINTENANCE	
A. Inspection	2, 4, 6
B. Well Maintenance	5
C. Well Replacement	5

LEGEND

1 = Annually
2 = Monthly during dry season (May-Sept.)
3 = Every Five Years
4 = After Each Heavy Rainfall
5 = As Required
6 = Weekly during wet season (Oct.-Apr.)

TABLE 3
SOUTH COAST LANDFILL
POST-CLOSURE MONITORING SCHEDULE

MONITORING ACTIVITY	FREQUENCY
GROUNDWATER MONITORING	
A. Groundwater	1
B. Constituents of Concern Monitoring	3
C. Groundwater Elevation/Flow Rate/Direction	1
D. Turbidity and Total Depth	2
SETTLEMENT	
A. Settlement Monument Monitoring/Aerial Survey	3

LEGEND

1 = Quarterly
2 = Semi-Annually
3 = Every Five Years

TABLE 4
Engineering Cost Opinion
South Coast Landfill Final Closure Construction

Const. Note	Item No.	Description	QTY	Unit	Cost	Total
	1	MOBILIZATION	1	LS	$300,000	$ 300,000.00
	2	TIME AND MATERIAL ALLOCATION	1	T&M	$150,000	$ 150,000.00
	3A	DIESEL FUEL PRICE ADJUSTMENT	1	T&M	$50,000	$ 50,000.00
	3B	CONSTRUCTION SUPPORT TASKS - SWPPP COMPLIANCE - CONSTRUCTION ACTIVITIES STORMWATER MANAGEMENT PLAN (CASMP) - INTERIM EROSION CONTROL AND BMP'S	1	LS	$50,000	$ 50,000.00
	4	PROJECT SURVEY	1	LS	$40,000	$ 40,000.00
BID ITEMS 1 THROUGH 4 SUBTOTAL =						$ 590,000.00
1	5	CLEARING AND GRUBBING	8.50	AC	$2,500.00	$ 21,250.00
5, 12	6	PERFORM REFUSE EXCAVATION & PLACE IN DESIGNATED RECONSOLIDATION AREA SHOWN PER PLAN	12,320	CY	$6.00	$ 73,920.00
7, 8, 13, 14, 17, 18	7A	UNCLASSIFIED FILL/FOUNDATION FILL	450	CY	$1.80	$ 810.00
7, 8, 13, 14, 17, 18	7B	FOUNDATION SOIL LAYER PLACEMENT	275,000	SF	$0.40	$ 110,000.00
7, 8, 13, 14, 17, 18	7C	UNCLASSIFIED EXCAVATION; PERFORM UNCLASSIFIED EXCAVATION AT STOCKPILE AREAS & BORROW AREA FOR FINAL COVER	18,500	CY	$1.80	$ 33,300.00
7, 8, 13, 14, 17, 18	7D	IMPORT SOIL MATERIAL	2,300	CY	$10.00	$ 23,000.00
7	9A	CONSTRUCT FINAL COVER SECTION-SLOPE LLDPE SUPER GRIP NET GEOMEMBRANE (PER DETAIL 1/D1)	210,115	SF	$0.85	$ 178,597.75
8	9B	CONSTRUCT FINAL COVER SECTION-DECK LLDPE SUPER GRIP NET GEOMEMBRANE (PER DETAIL 2/D1)	77,000	SF	$0.85	$ 65,450.00
24	10	CONSTRUCT HDPE BOOT (PER DETAIL 7/D1)	5	EA	$1,000.00	$ 5,000.00
7	11A	CLOSURETURF - SLOPE	210,115	SF	$0.25	$ 52,528.75
8	11B	CLOSURETURF - DECK	77,000	SF	$0.22	$ 16,940.00
9	12A	CONSTRUCT HDPE EDGE DRAIN COLLECTOR - BENCH, SLOPE AND ACCESS ROAD (BELOW LLDPE GEOMEMBRANE) (PER DETAIL 3/D1, 5/D1, 6/D1, 4/D2, AND 6/D3)	2,200	LF	$8.00	$ 17,600.00
26, 27	12B	4-INCH HDPE PIPE SDR 17 OUTLET WITH BENTONITE PLUG AT FINAL COVER LIMIT (PER DETAIL 7/D1)	835	LF	$15.00	$ 12,525.00
10	12C	DRAINAGE COLLECTOR (PER DETAIL 4/D1)	330	LF	$28.00	$ 9,240.00
14	13A	PROTECTIVE SOIL COVER - NORTHEAST ROAD	1,540	CY	$0.50	$ 770.00
17	13B	PROTECTIVE SOIL COVER - SOUTH AND WEST ROAD	665	CY	$0.50	$ 332.50
18	13C	PROTECTIVE SOIL COVER - DECK ACCESS ROAD	140	CY	$0.50	$ 70.00
11, 13	14A	CONSTRUCT ANCHOR TRENCH WITH SOIL BACKFILL (PER DETAIL 5/D1 AND 6/D3)	760	LF	$25.00	$ 19,000.00
17	14B	CONSTRUCT ANCHOR TRENCH WITH CONCRETE BACKFILL/ROCK PLACEMENT (PER DETAIL 11/D1 AND 4/D2)	800	LF	$20.00	$ 16,000.00

TABLE 4

Engineering Cost Opinion

South Coast Landfill Final Closure Construction

Const. Note	Item No.	Description	QTY	Unit	Cost	Total
15	15	CONSTRUCT CONCRETE ACCESS RAMP TO BASIN WITH INTEGRAL CONCRETE CURB (PER DETAIL 4/D3)	2,600	SF	$6.00	$ 15,600.00
22	16A	CONCRETE DOWNDRAIN INLET AT STORMWATER BASIN (PER DETAIL 3/D3)	2	EA	$3,000.00	$ 6,000.00
42	16B	CONCRETE INLET TO DOWN DRAIN (PER DETAIL 7/D3)	4	EA	$4,000.00	$ 16,000.00
20	17	CONCRETE DOWNDRAIN (b=1.0, D=1.5, Z=1) (PER DETAIL 5/D2)	1,400	SF	$6.00	$ 8,400.00
21	18	CONSTRUCT RIP RAP PAD (PER DETAIL 4/D2)	600	SF	$6.00	$ 3,600.00
32	19	CONSTRUCT SPLASHWALL (PER DETAIL 5/D3)	30	LF	$38.00	$ 1,140.00
18, 31	20	CONSTRUCT 3" AC OVER NATIVE SOIL COMPACTED TO 95% OF ASTM D1557	27,450	SF	$2.50	$ 68,625.00
29	21	CONSTRUCT TYPE A AC DIKE PER CALTRANS STD PLAN A87B	970	LF	$8.00	$ 7,760.00
23	22	INSTALL POST-CLOSURE SURVEY/SETTLEMENT MONUMENTS (PER DETAIL 2/D3)	2	EA	PAID PER ITEM 4	
43	23	INSTALL CABLE GATE (PER DETAIL 5/D2)	1	EA	$1,600.00	$ 1,600.00
19	24	INSTALL PIPE BOLLARD (PER DETAIL 8/6)	14	EA	$350.00	$ 4,900.00
25	10	CONSTRUCT BENTONITE PLUG (PER DETAIL 7/D1)	5	EA	PAID PER ITEM 10	
30	20	CONSTRUCT 12" X 12" DEEP LIFT TERMINATION	200	LF	PAID PER ITEM 15 OR 20	
LANDFILL GAS SYSTEM						
33	26	PROTECT/ADJUST EXISTING LFG GAS PROBES CONSTRUCT 3 EACH PIPE BOLLARDS PER PROBE	3	EA	$2,500.00	$ 7,500.00
34	27	CONSTRUCT LFG PASSIVE VERTICAL WELL	350	VF	$200.00	$ 70,000.00
35, 36	28A	CONSTRUCT LFG TRENCH COLLECTION GALLERY/VENT	1,350	LF	$25.00	$ 33,750.00
16, 28	28B	ANCHOR TRENCH/LFG COLLECTION TRENCH	1,190	LF	$27.00	$ 32,130.00
38	28C	LFG/SEEP COLLECTOR	4,500	LF	$2.25	$ 10,125.00
37	29	CONSTRUCT LLDPE SKIRT AND SLEEVE	34	EA	$1,250.00	$ 42,500.00
FINAL EROSION CONTROL						
39	30	HYDROSEEDING	1.24	AC	$4,500.00	$ 5,580.00
40	31	INSTALL FIBER ROLLS (PER DETAIL 3/D8)	2,930	LF	$3.00	$ 8,790.00
41	32	INSTALL GRAVEL BAG CHEVRON (PER DETAIL 1/D4)	135	LF	$25.00	$ 3,375.00
		BID ITEMS 5 THROUGH 32 SUBTOTAL =		$		1,003,709.00
		SUBTOTAL BID ITEMS 1 THROUGH 32 =		$		1,593,709.00
		10% CONTINGENCY =		$		159,370.90
		TOTAL BID AMOUNT =		**$**		**1,753,079.90**

TIME IS OF ESSENCE TO THIS CONTRACT. THE CONTRACTOR SHALL COMPLETE THE FINAL CLOSURE CONSTRUCTION PROJECT AT THE SOUTHCOAST LANDFILL NO LATER THAN TWO HUNDRED SEVENTY (270) CALENDAR DAYS FOLLOWING THE ISSUANCE OF NOTICE TO PROCEED. FAILURE TO MEET THIS TIME LIMIT WILL EXPOSE CONTRACTOR TO LIABILITY FOR LIQUIDATED DAMAGES PURSUANT TO THE AGREEMENT.

NOTES:

TABLE 4

Engineering Cost Opinion

South Coast Landfill Final Closure Construction

Const. Note	Item No.	Description	QTY	Unit	Cost	Total

The project will be bid on a unit price basis. For bidding purposes quantities will be as represented on the bid schedule. Final pay quantities will be based on field measurements, consistent with the project specifications, and as approved by the engineer. Material, or work, completed beyond the limits indicated on the drawings, or as represented on the bid schedule will not be compensated unless the additional work has been authorized, in advance, by the engineer.

Item 2 -Time and Material Allocation and Item 3A-Diesel Fuel Price Adjustment are time-and-materials items payable only upon written authorization by County of Mendocino. The Contractor is advised that actual earthwork quantities (Bid Items 6, 7A, 7B, 7C, 7D, 13A, 13B and 13C) may vary from those indicated. Actual quantities shall be measured for final payment by comparing before and after survey information as defined in these Specifications, or as approved by the Engineer. SSPWC Section 3-2.1 shall not apply to the contract unit price work for these Bid Items performed under this paragraph. The Contractor shall be compensated for the noted Bid Items at the contract unit price regardless of any increase or decrease in final quantities.

Bids in which the total cost calculated for the individual bid item does not correspond to the unit price quoted may be rejected as non-responsive. County of Mendocino reserves the right to waive any non-material defects of irregularities that do not result in any unfair competitive advantage. Any conflict between the unit price and the total cost calculated for an individual item will be resolved by reference to the unit price.

Include all costs for the items set forth above and called for by the Contract Documents. No other line items of cost, prices, or quotes for other activities may be included in this bid. **Bidders are reminded that this Proposal must be signed**.

The Bidder shall hereinafter state that any Subcontractor(s) who will be the Subcontractor(s) on the job for each particular trade or subdivision of the work and will state the firm name and principal location of the mill, shop, or office of each. Failure to list all Subcontractors or if the prime contractor specifies more than one subcontractor for the same portion of work to be performed under the contract in excess of one-half of 1 percent of the prime contractor’s total bid, the prime contractor agrees that he or she is fully qualified to perform that portion himself or herself, and that the prime contractor shall perform that portion himself or herself.

TABLE 5
SOUTH COAST LANDFILL
CLOSURE COST DISBURSEMENT SCHEDULE

Construction Period	Total Monthly Disbursements
Month 1	$462,781
Month 2	$404,734
Month 3	$408,668
Month 4	$201,454
Month 5	$275,443
Total Construction Costs	**$1,753,079.90**

TABLE 6
SOUTH COAST LANDFILL
POST CLOSURE COST ESTIMATE SUMMARY 2016

Item No.	Description	Estimated Quantity		Unit Price	Total Annual Cost (2016 $)
1	**VEGETATIVE COVER MAINTENANCE (EROSION CONTROL) [1.0]**				
	Reseeding (2% of the Hydroseed/Year) [1.1]	0	AC	$ 5,225.00	$ -
	Weed Control/Fire Control (Note: Limit to Areas Around Env. Syst.) [1.2]	16	HRS	$ 125.00	$ 2,000
	Rodent Control [1.3]	6.6	AC	$ 100.00	$ 660
				Item 1 Subtotal	**$ 2,660**
2	**FINAL COVER MAINTENANCE [2.0]**				
	Cover Maintenance Repair	0.22	AC	$ 16,000.00	$ 3,520
	CQA of Turf Install [2.1]	40	HRS	$ 117.67	$ 4,707
				Item 2 Subtotal	**$ 8,227**
3	**LANDFILL GAS REMEDIATION/CONTROL SYSTEM MAINTENANCE [3.0]**				
	Gas Collection/Control System O&M (Note: Includes Piping, Passive Wells, Wellfield Monitoring - Emissions Compliance Only)	6.6	AC	$ 2,226.00	$ 14,692
				Item 3 Subtotal	**$ 14,692**
4	**LANDFILL GAS MIGRATION/VADOSE ZONE SYSTEM MONITORING AND MAINTENANCE [4.0]**				
	Gas Migration Monitoring System Monitoring/Reporting/Maintenance and SCAQMD Rule 1150.1 Monitoring and Sampling	4	YR	$ 750.00	$ 3,000
				Item 4 Subtotal	**$ 3,000**
5	**LEACHATE COLLECTION AND RECOVERY SYSTEM MAINTENANCE [5.0]**				
	Operation and Maintenance (Note: Sumps, Pumps, Tanks, Liquid Disposal)	1	LS	$ 7,300.00	$ 7,300
	Sampling, Laboratory Analysis and Reporting	1	LS	$ 941.00	$ 941
				Item 5 Subtotal	**$ 8,241**
6	**DRAINAGE CONTROL SYSTEM MAINTENANCE [6.0]**				
	Desilting Basin, Trapezoidal Channel, V-ditches Maintenance (Note: Cleaning and Weed Removal).	40	HRS	$ 75.00	$ 3,000
	Concrete Downdrains and Deck Inlets Maintenance	8	HRS	$ 75.00	$ 600
	Desilting Basin, Trapezoidal Channel, V-ditches Replacement (3%/Year)	25	LF	$ 110.00	$ 2,750
	Concrete Downdrains and Deck Inlets Replacement (3%/Year)	20	LF	$ 85.00	$ 1,700
				Item 6 Subtotal	**$ 8,050**
7	**GROUNDWATER SYSTEM MAINTENANCE AND MONITORING [7.0]**				
	Groundwater System Operation and Maintenance (Note: Extraction Wells, Pumps, Tanks)	30	HRS	$ 117.67	$ 3,530
	Monitoring/Sampling/Reporting (Sampling by Eng. Tech II or equivalent. Includes Laboratory Analysis. Semi-annual and Annual Reports Prepared by 3rd Party. Also includes COC testing every 5 years.)	1	YR	$ 4,500.00	$ 4,500
	Groundwater Well Replacement [7.1]	1	YR	$ 500.00	$ 500
				Item 7 Subtotal	**$ 8,030**
8	**SURFACE WATER MONITORING [8.0]**				
	Monitoring/Laboratory Analysis/Reporting (Note: Sampling, Laboratory Analysis, and Reporting)	1	LS	$ 1,948.00	$ 1,948
				Item 8 Subtotal	**$ 1,948**
9	**STORM WATER MONITORING [9.0]**				
	Inspection/Sampling/Reporting	1	LS	$ -	$ -
				Item 9 Subtotal	**$ -**

TABLE 6
SOUTH COAST LANDFILL
POST CLOSURE COST ESTIMATE SUMMARY 2016

Item No.	Description	Estimated Quantity		Unit Price	Total Annual Cost (2016 $)
10	**LANDFILL SETTLEMENT/MONUMENT MAINTENANCE** [10.0]				
	Aerial Survey (Includes Is-Settlement Map) (Once Every 5 Years)	0.2	LS	$ 6,500.00	$ 1,300
	Survey/Settlement Monument Maintenance (Assume One Replacement/10 Years)	0.1	EA	$ 1,750.00	$ 175
				Item 10 Subtotal	**$ 1,475**
11	**SECURITY MAINTENANCE** [11.0]				
	Labor\Materials for 40 LF every year	1	LS	$ 1,282.00	$ 1,282
				Item 11 Subtotal	**$ 1,282**
12	**ACCESS ROAD MAINTENANCE** [13.0]				
	Access Road Maintenance	4	HRS	$ 75.00	$ 300
	Access Road Resurfacing 3" AC over Compactive Native (2% Every Year)	600	SF	$ 8.20	$ 4,920
				Item 12 Subtotal	**$ 5,220**
13	**SITE ADMINISTRATION** [14.0]				
	Site Administration (Note: Civil Eng. Asst 1/16 FT or equivalent for Administrative Oversight. Include permit fees and overhead costs)	1	LS	$ 34,500.00	$ 34,500
	5% Contingency (Note: 5% applies to Site Admin. Additional Unanticipated Overhead or Costs)	1	LS	$ 1,725.00	$ 1,725
				Item 13 Subtotal	**$ 36,225**
				SUBTOTAL ANNUAL POST-CLOSURE MAINTENANCE COST	**$ 99,050**
				TOTAL 30-YEAR POST-CLOSURE MAINTENANCE COST	**$ 2,971,485**

General:

A) DETAILED BREAKDOWNS OF COSTS AND BACK-UP INFORMATION PER ITEM ARE INCLUDED IN APPENDIX M.

B) Table 5 was utilized as the basis for several of Post-Closure Maintenance/Repair Cost Items. Quantities (areas of repair and/or maintenance) were based on assumptions made by SWT Engineering based on our experience with other landfills. Labor rates were obtained from the State of California Industrial Relations; prevailing wage schedule pursuant to California Labor Code Part 7, Chapter 1, Article 2, Sections 1770, 1773, and 1773.1. Equipment rental rates were obtained from the CalTrans Labor Surcharge and Equipment Rental Rates effective April 1, 2016 through March 31, 2017.

C) The annual post-closure maintenance cost estimate includes a cost item for the passive landfill gas venting system. This system is comprised of a network of passive landfill gas vents constructed of HDPE pipe placed within the geomembrane cover section in the foundation layer. The gas is collected in bilateral, perforated pipes placed in shallow gravel trenches and vented by the HDPE riser pipes. The cost to maintain the passive venting system is included in the post-closure maintenance cost estimate. The cost category for the passive landfill gas venting system features utilize a constant annual cost for the minimum 30-year post-closure maintenance period.

D) There are no maintenance and/or monitoring activities associated with the SCL which will occur less frequently than 30 years that are not otherwise accounted for in this post-closure maintenance cost estimate in accordance with 27 CCR 21840(a)(3)(A(i).

Notes:

1.0 Inspection cost included in Site Administration.

1.1 SCL will have a turf cover closure and will not require hydroseeding.

1.2 Weed and Fire Control only for areas around the environmental conctrol system, at 16 hours per year.

1.3 Based on existing annual contract for pest control between South Coast Landfill (SCL) and pest control company.

2.0 Inspection cost included in Site Administration. Cost for full replacement (equipment costs included) of the top turf closure layer over the 30 year span of approximately 0.22 acres per year at $16,000 per acre. Also includes up to $660 per year (or $3,000 per acre) for minor repairs to the super grip net geomembrane under the closure turf. Costs are based on related turf closure projects.

2.1 The cost for CQA is based on Caltrans Labor and Equipment Rates (February 2016) and the Prevailing Wage Rates (April 2016) for inspection, truck, and reporting.

3.0 Based on unit costs from existing contracts and a back-up is included in Appendix M.

TABLE 6
SOUTH COAST LANDFILL
POST CLOSURE COST ESTIMATE SUMMARY 2016

Item No.	Description	Estimated Quantity	Unit Price	Total Annual Cost (2016 $)

4.0 Includes monthly structure monitoring as required by 27 CCR, Section 20931, and SCAQMD Rule 1150.1 Monitoring & Sampling. (Note: ISS, OVA, Ambient Air, Perimeter Probe, Raw Gas, Lab Samples, Structures Monitoring, and Flare Source Testing)

5.0 Based on unit costs from existing contracts and a back-up is included in Appendix M.

6.0 Based on drainage control system closure costs. Assumes 3% replacement/improvement per year (1,400 LF of closure channels v-ditches, down drains, etc. proposed). Inspection cost included in Site Administration.

7.0 For Semi-Annual and Annual Sampling and Lab Testing/Analysis ($2,000/year) and Reporting ($2,500/year) by a 3rd Party with COC Testing every 5 years.

7.1 Assume one well to be replaced over a 30-year period at $15,000/well replacement.

8.0 Based on 2 semi-annual sampling events per year at 2 surface water monitoring locations by an engineering technician/field engineer sampling at 2 hours per semi-annual event at $112/hr. Assume 6 surface water samples per year (4 surface water samples plus 2 quality control samples per year) analyzed in laboratory at $250/sample.

9.0 NPDES will not apply to closed sites as the site is no longer considered an industrial activity. Once closure is completed, a Notice of Termination (NOT) will be filed with the RWQCB, therefore there is no cost associated with Stormwater Monitoring.

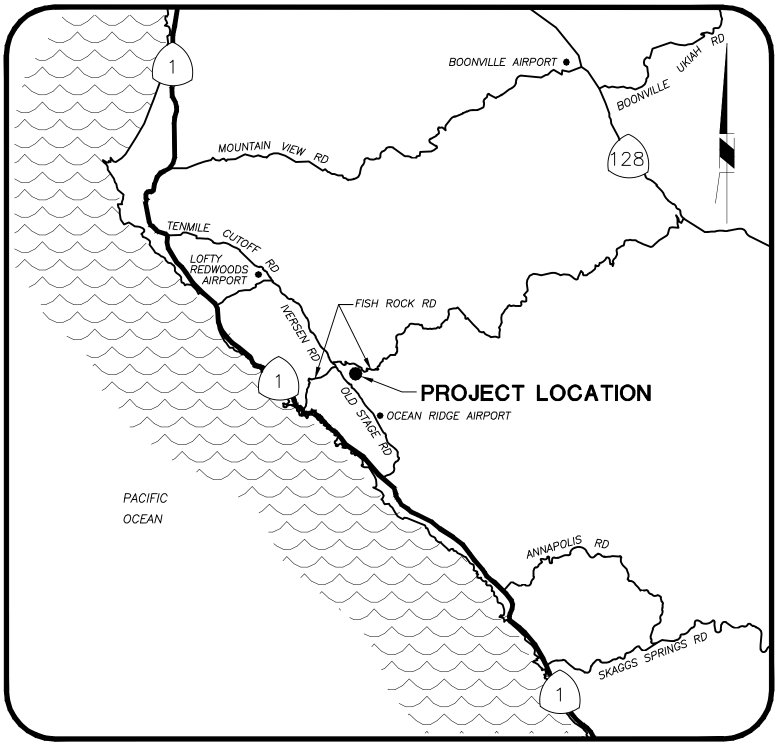
10.0 Aerial Survey to be completed once every 5 years at $6,500/survey and includes an aerial ortho photo. Cost is based on information provided by Analytical Photogrammetric Surveys, Inc. 1 settlement monument to be replaced every 10 years, with the cost based on the closure estimate.

11.0 Cost to replace 40 LF of Fencing per year, one gate and sign every 10 years, back-up costs are included in Appendix M. Task to be performed by on-site, full-time maintenance team.

12.0 Access Roads will be inspected quarterly and after a significant storm event. Inspection cost included in Site Administration. Assumed that 2% of the access roads will need to be resurfaced every year. back-up costs are included in Appendix M.

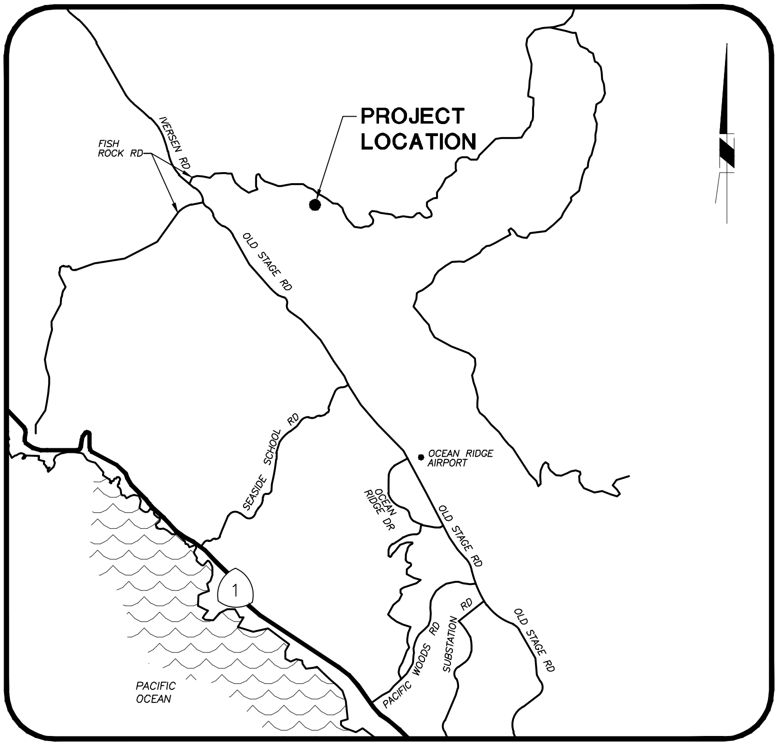
13.0 Cost is based on 1 Site Engineer (2 hrs per week) every year. Permitting review, permits, and overhead costs are included per the back-up documentation included in Appendix M.

FIGURES



VICINITY MAP

SCALE: 1"=8 MI



SITE ADDRESS:
40855 FISH ROCK ROAD
GUALALA, CA 95445
ASSESSOR'S PARCEL NO.
141-080-26

LOCATION MAP

SCALE: 1"=1.5 MI

PREPARED BY:

SWT Civil & Environmental **Engineering**

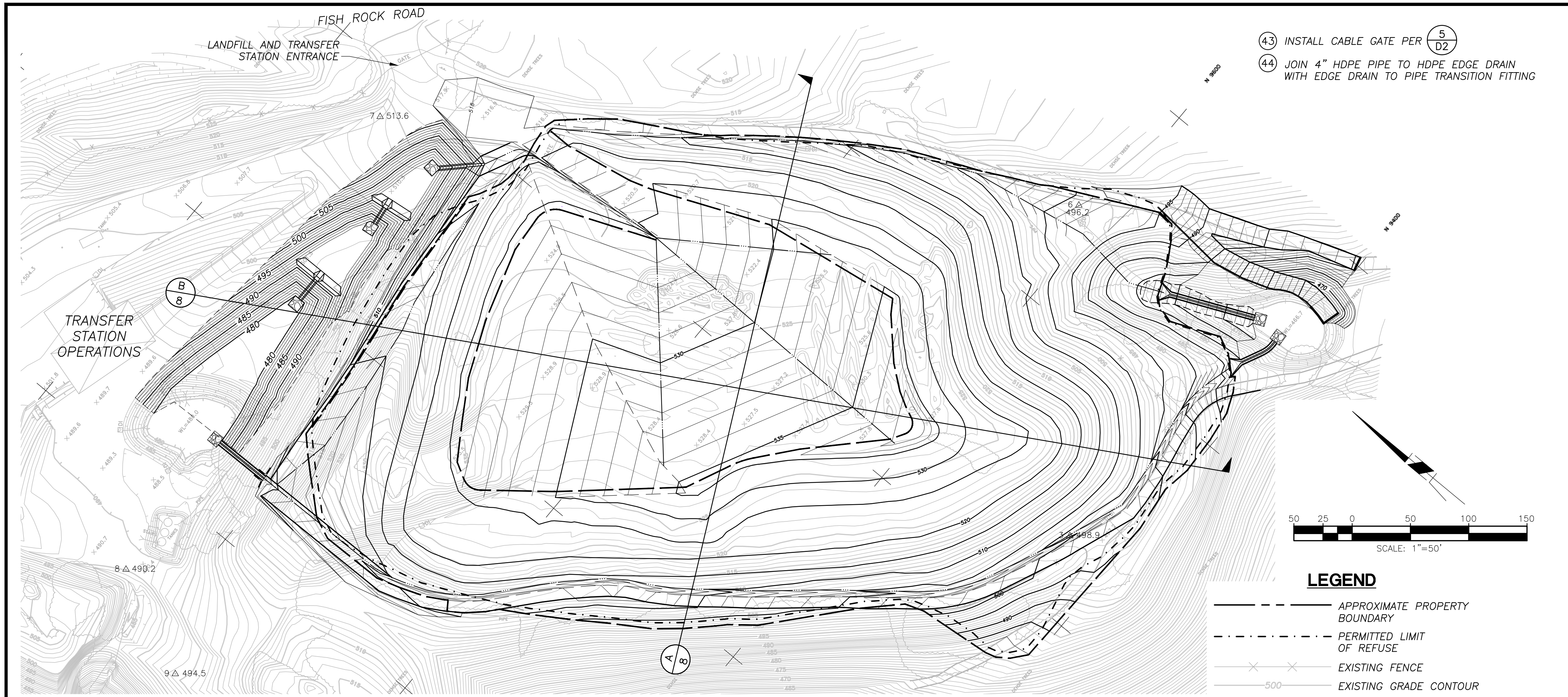
800-C SOUTH ROCHESTER AVENUE
ONTARIO, CALIFORNIA 91761

FIGURE 1

SOUTH COAST LANDFILL

SITE VICINITY AND LOCATION MAP

Z:\PROJECTS\MENDOCINO COUNTY\SOUTH COAST\FCPCMP\ACAD\FIG01-SOC LOCATION MAP



43. INSTALL CABLE GATE PER (5/D2)
44. JOIN 4" HDPE PIPE TO HDPE EDGE DRAIN WITH EDGE DRAIN TO PIPE TRANSITION FITTING

NOTICE TO CONTRACTOR

1. THE EXISTENCE AND LOCATION OF UNDERGROUND UTILITY PIPES OR STRUCTURES SHOWN ON THESE PLANS WERE OBTAINED BY A SEARCH OF AVAILABLE RECORDS. THESE LOCATIONS ARE APPROXIMATE AND SHALL BE CONFIRMED IN THE FIELD BY THE CONTRACTOR SO THAT ANY NECESSARY ADJUSTMENT CAN BE MADE IN ALIGNMENT AND/OR GRADE OF THE PROPOSED IMPROVEMENT. THE CONTRACTOR IS REQUIRED TO TAKE DUE PRECAUTIONARY MEASURES TO PROTECT ANY UTILITY LINES SHOWN AND ANY OTHER LINES NOT OF RECORD OR NOT SHOWN ON THESE PLANS.

2. IT SHALL BE THE RESPONSIBILITY OF THE CONTRACTOR TO MAKE EXACT DETERMINATION AS THE LOCATION OF ALL EXISTING UTILITIES. FORTY-EIGHT (48) HOURS PRIOR TO ANY EXCAVATION, THE CONTRACTOR SHALL NOTIFY UNDERGROUND SERVICE ALERT AT 1-800-422-4133 AND SHALL PROVIDE THE CONSTRUCTION MANAGER WITH VERIFICATION NUMBERS ISSUED.

3. THE FACT THAT ANY UTILITY FACILITY IS SHOWN OR NOT SHOWN UPON THE PLANS SHALL NOT RELIEVE THE CONTRACTOR OF THE RESPONSIBILITY UNDER SECTION 8-1.10 "UTILITY AND NON-HIGHWAY FACILITIES," OF THE STATE STANDARD SPECIFICATIONS. IT SHALL BE THE CONTRACTOR'S RESPONSIBILITY, PURSUANT THERETO, TO ASCERTAIN THE LOCATION OF ANY UTILITY FACILITY WHICH MAY BE SUBJECT TO DAMAGE BY REASON OF THE CONTRACTOR'S OPERATIONS.

4. REGARDING SIGNED DRAWING VERSUS ELECTRONIC COPY: SHOULD THE CONTRACTOR OR CONTRACTOR'S SURVEYOR FIND A CONTOUR LABEL OR ELEVATION DISCREPANCY BETWEEN THE SIGNED DRAWING AND THE ELECTRONIC VERSION (WHICH HAVE BEEN PROVIDED AS A COURTESY FOR EASE OF CONSTRUCTION STAKING), THE CONTRACTOR SHALL NOTIFY THE ENGINEER OF THE DISCREPANCY SO THAT THE DISCREPANCY MAY BE RESOLVED, PRIOR TO CONSTRUCTION. THE SIGNED DRAWING SUPERSEDES THE ELECTRONIC VERSION.

COORDINATE CONTROL POINTS

PT NO	NORTHING	EASTING	ELEVATION
3	9443.63	10064.07	498.9
6	9618.41	10287.41	496.2
7	10128.71	9963.78	513.6
8	10049.25	9525.71	490.2
9	9959.54	9483.59	494.5

NOTES

1. TOPOGRAPHIC DATA COMPILED FROM 15 FEBRUARY 2012 AERIAL PHOTOGRAPHY BY DELTA GEOMATICS CORPORATION.
2. ELEVATIONS ARE IN FEET ABOVE MEAN SEA LEVEL, NATIONAL GEODETIC VERTICAL DATUM OF 1929 (NGVD29).
3. GRID COORDINATES CORRESPOND TO THE CALIFORNIA STATE PLANE COORDINATE SYSTEM, ZONE 3, NAD83.
4. LOCATIONS OF STOCKPILES ARE APPROXIMATE.
5. EQUIPMENT AND MATERIAL STAGING AREA(S) TO BE DETERMINED BY OWNER.
6. ALL EXISTING STRUCTURES SUCH AS BUILDINGS, POLES, FENCES, PIPES, DITCHES, PONDS, ETC. WITHIN LIMIT OF EARTHWORK SHALL BE REMOVED AS REQUIRED BY THE OWNER.

ABBREVIATIONS

FL	FLOWLINE	LFG	LANDFILL GAS
FP	FINISHED PAVEMENT	GB	GRADE BREAK
HP	HIGH POINT	TC	TOP OF CURB
LP	LOW POINT	TS	TOP OF SLOPE
PP	POWER POLE	UD	UNDERDRAIN
INV	INVERT OF PIPE	CHDPE	CORRUGATED HIGH DENSITY POLYETHYLENE

LEGEND

——— — — ———	APPROXIMATE PROPERTY BOUNDARY
— · — · — · —	PERMITTED LIMIT OF REFUSE
—×—×—	EXISTING FENCE
——500——	EXISTING GRADE CONTOUR
——500——	FINISHED GRADE CONTOUR
— — — — — —	TOE OF SLOPE
- - - - - - - -	DAYLIGHT LINE
——·——·——	RIDGE LINE
— - - — C/F — - - —	CUT/FILL TRANSITION
▨	FINISHED BENCH
—— · - · ——	HDPE EDGE DRAIN
——‖——‖——	DRAINAGE COLLECTOR
— · · — · · —	LFG/SEEP COLLECTOR
3:1 →	DIRECTION AND RATE OF SLOPE
1/D1	DETAIL NUMBER SHEET SHOWN ON
7 △ 513.6	HORIZONTAL/VERTICAL CONTROL (HVC)
◆	FINAL COVER SETTLEMENT MONUMENT
⊗P-3	LFG PROBE
	VERTICAL PASSIVE VENT GAS WELL
- - - -○- - - -	LANDFILL GAS PASSIVE VENT RISER LOCATION
- - - - - - - - - -	LANDFILL GAS PASSIVE TRENCH BELOW 60 MIL LLDPE
▚ LFGW-1	GAS MONITORING PROBE

CONSTRUCTION NOTES

1. CLEAR AND GRUB
2. PROTECT IN PLACE
3. REMOVE EXISTING STRUCTURES
4. ADJUST TO FINAL GRADE
5. PERFORM REFUSE EXCAVATION AND PLACE IN DESIGNATED RECONSOLIDATION AREA SHOWN PER PLAN
6. PERFORM UNCLASSIFIED EXCAVATION AT STOCKPILE AREAS AND BORROW AREA FOR FINAL COVER
7. CONSTRUCT FINAL COVER SECTION – SLOPE (1/D1)
8. CONSTRUCT FINAL COVER SECTION – DECK (2/D1)
9. CONSTRUCT HDPE EDGE DRAIN COLLECTOR (BELOW FINAL COVER LLDPE) (3/D1)
10. CONSTRUCT DRAINAGE COLLECTOR (4/D1)
11. CONSTRUCT FINAL COVER SLOPE TERMINATION (5/D1) (WITH SWALE), OR (11/D1) (NO SWALE)
12. PERFORM 2-FOOT (PLUS INTERIM SOIL COVER) REFUSE EXCAVATION PRIOR TO PLACEMENT OF INTERIM COVER AND 2-FOOT FOUNDATION SOIL LAYER; PERFORM 10-FOOT TRANSITION FROM 2-FOOT VERTICAL CUT TO DAYLIGHT WITH EXISTING GRADE (REFUSE EXCAVATION PLUS INTERIM COVER) (4/D2)
13. CONSTRUCT BENCH LINER TERMINATION (6/D3)
14. CONSTRUCT FINAL COVER TERMINATION – NORTHEAST PERIMETER ROAD (6/D1)
15. CONSTRUCT CONCRETE ACCESS RAMP TO BASIN WITH INTEGRAL CONCRETE CURB (4/D3)
16. CONSTRUCT TOP DECK TRANSITION (8/D1)
17. CONSTRUCT FINAL COVER TERMINATION – SOUTH AND WEST PERIMETER ROAD (4/D2)
18. CONSTRUCT DECK ACCESS ROAD (9/D1)
19. INSTALL PIPE BOLLARD (8/6)
20. CONSTRUCT CONCRETE DOWNDRAIN/CHANNEL (1/D3)
21. CONSTRUCT RIP RAP PAD (1/D2)
22. CONSTRUCT BASIN BERM AND DOWNDRAIN INLET AT STORMWATER BASIN (3/D3)
23. RELOCATE/ADJUST SURVEY MONUMENT TO GRADE PER PROJECT SPECIFICATIONS AND (2/D3)
24. CONSTRUCT HDPE BOOT (7/D1)
25. CONSTRUCT BENTONITE PLUG (7/D1)
26. CONSTRUCT 4" SOLID HDPE PIPE (SDR 17)
27. EXTEND PIPE AND JOIN TO EXISTING LEACHATE OUTFALL PIPING/TANK
28. CONSTRUCT ANCHOR TRENCH WITH LFG COLLECTION TRENCH (8/D1)
29. CONSTRUCT TYPE A AC DIKE PER CALTRANS STD PLAN A87B
30. CONSTRUCT 12"x12" DEEP LIFT AC/CONCRETE TERMINATION
31. CONSTRUCT 3" AC OVER NATIVE SOIL COMPACTED TO 95% OF ASTM D1557 (6/D1) OR (9/D1) OR (4/D2)
32. CONSTRUCT SPLASHWALL (5/D3)
33. PROTECT/ADJUST LFG GAS PROBE TO FINAL GRADE; INSTALL 3 EACH PIPE BOLLARDS AT EACH PROBE (8/6)
34. CONSTRUCT VERTICAL LANDFILL GAS WELL PER (3/6)
35. CONSTRUCT LANDFILL GAS VENT PER (2/6)
36. INSTALL LANDFILL GAS TRENCH COLLECTION GALLERY WITH 2 CF/LF TRENCH GRAVEL, 4" SLOTTED HDPE PIPE AND 8 OZ/SY GEOTEXTILE WRAP (2/6)
37. CONSTRUCT LLDPE SKIRT AND SLEEVE (7/6)
38. INSTALL LFG/SEEP COLLECTOR (10/D1)
39. HYDROSEED ON SITE AREA
40. INSTALL FIBER ROLL PER (2/D4) (3/D4)
41. INSTALL GRAVEL BAG CHEVRON PER (1/D4)
42. CONSTRUCT INLET TO CONCRETE DOWNDRAIN (7/D3)

FIGURE 2

NO.	REVISION DESCRIPTION	DATE

PREPARED BY:

SWT Civil & Environmental **Engineering**
800-C SOUTH ROCHESTER AVENUE
ONTARIO, CALIFORNIA 91761

MICHAEL A. CULLINANE
No. 41981
CIVIL

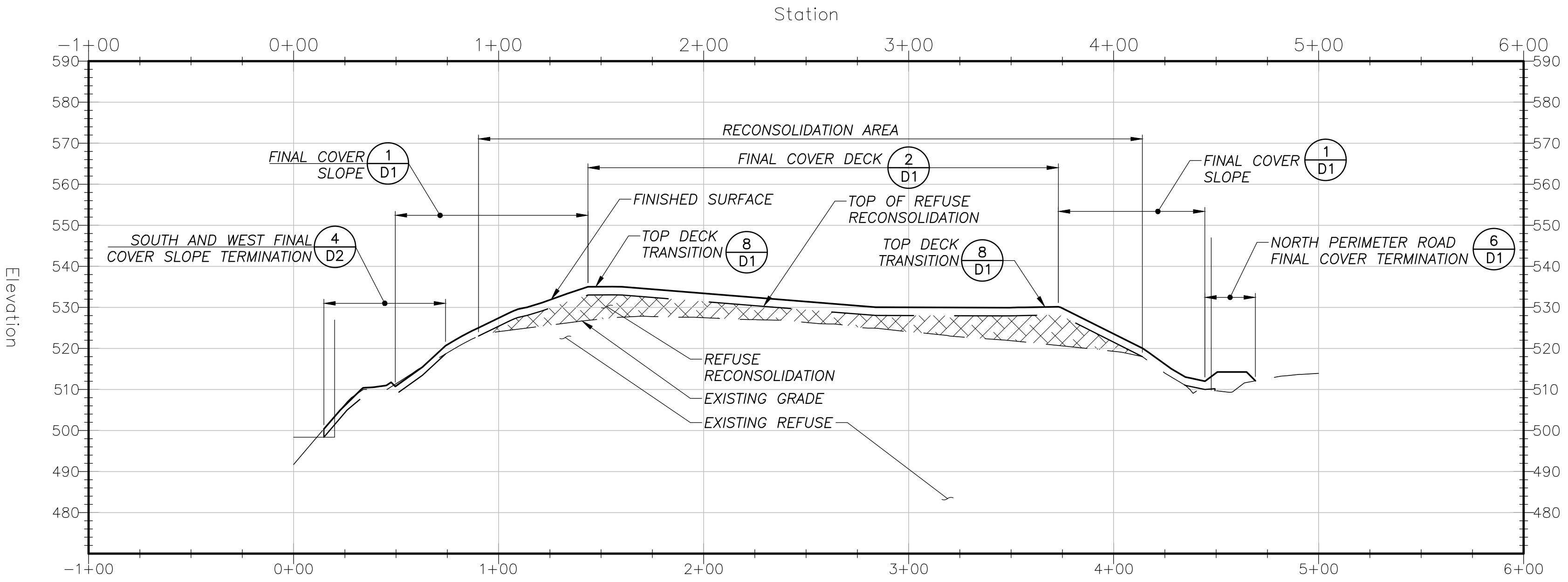
PREPARED UNDER THE SUPERVISION OF ______ DATE

MENDOCINO COUNTY
SOUTH COAST LANDFILL

FINAL CLOSURE CONSTRUCTION PROJECT 2016
SITE PLAN

DESIGNED BY : RMG	SCALE : AS SHOWN	PROJECT NO: –
DRAWN BY : CTD/DMW	DATE : 11-2016	
CHECKED BY :	DATE :	SHEET 2 OF 12
APPROVED BY :	DATE :	

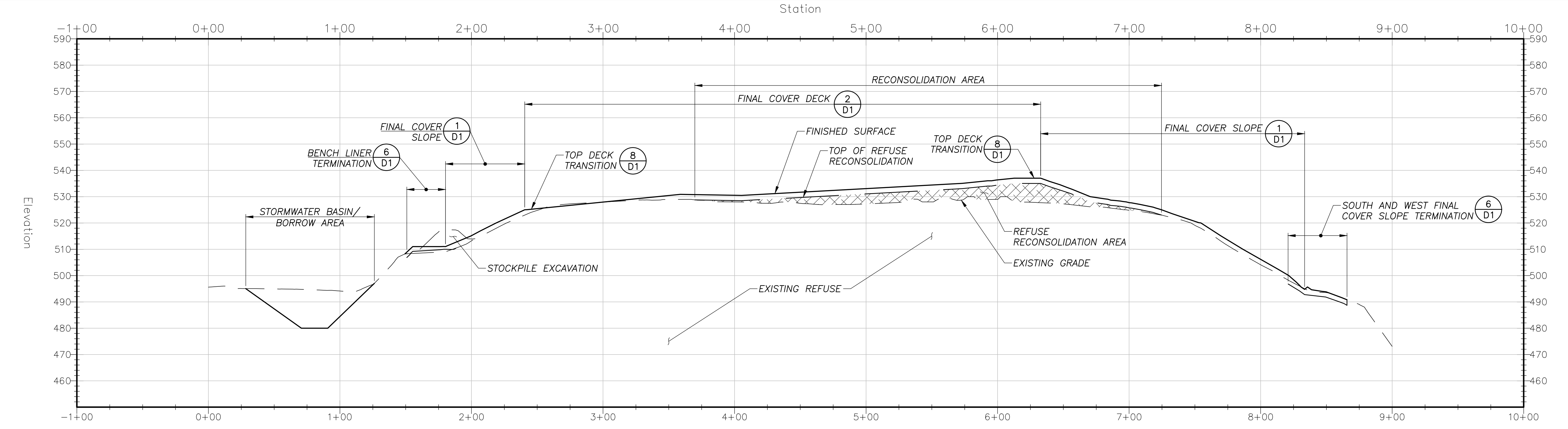
Z:\PROJECTS\MENDOCINO COUNTY\SOUTH COAST\FCPCMP\ACAD\FIG02-SOC SITE PLAN



SECTION A

SCALE: HORIZ. 1"=40'
VERT. 1"=20'

A/8



SECTION B

SCALE: HORIZ. 1"=40'
VERT. 1"=20'

FIGURE 2A

B/8

NO.	REVISION DESCRIPTION	DATE

PREPARED BY:

SWT Civil & Environmental Engineering

800-C SOUTH ROCHESTER AVENUE
ONTARIO, CALIFORNIA 91761

REGISTERED PROFESSIONAL ENGINEER
MICHAEL A. CULLINANE
No. 41981
CIVIL
STATE OF CALIFORNIA

PREPARED UNDER THE SUPERVISION OF

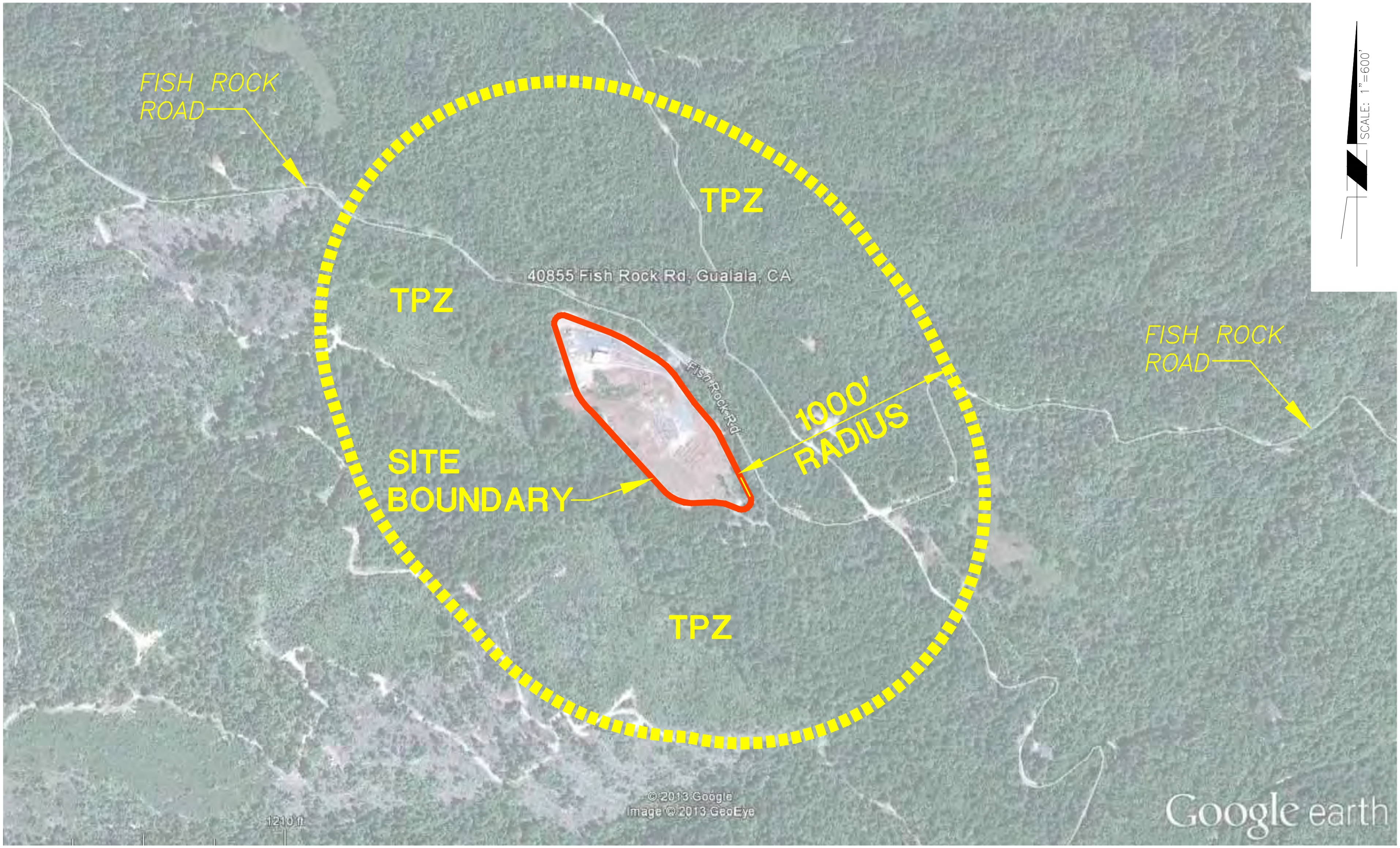
DATE

MENDOCINO COUNTY
SOUTH COAST LANDFILL

FINAL CLOSURE CONSTRUCTION PROJECT 2016
SECTIONS

DESIGNED BY : RMG	SCALE : AS SHOWN	PROJECT NO: –
DRAWN BY : CTD/DMW	DATE : 11-2016	
CHECKED BY :	DATE :	SHEET 7 OF 12
APPROVED BY :	DATE :	

Z:\PROJECTS\MENDOCINO COUNTY\SOUTH COAST\FCPCMP\ACAD\FIG02A-SOC SECTIONS



AERIAL IMAGERY FROM GOOGLE EARTH: AUGUST 15, 2009

TPZ = TIMBER PRESERVE ZONE

PREPARED BY:

SWT Civil & Environmental Engineering

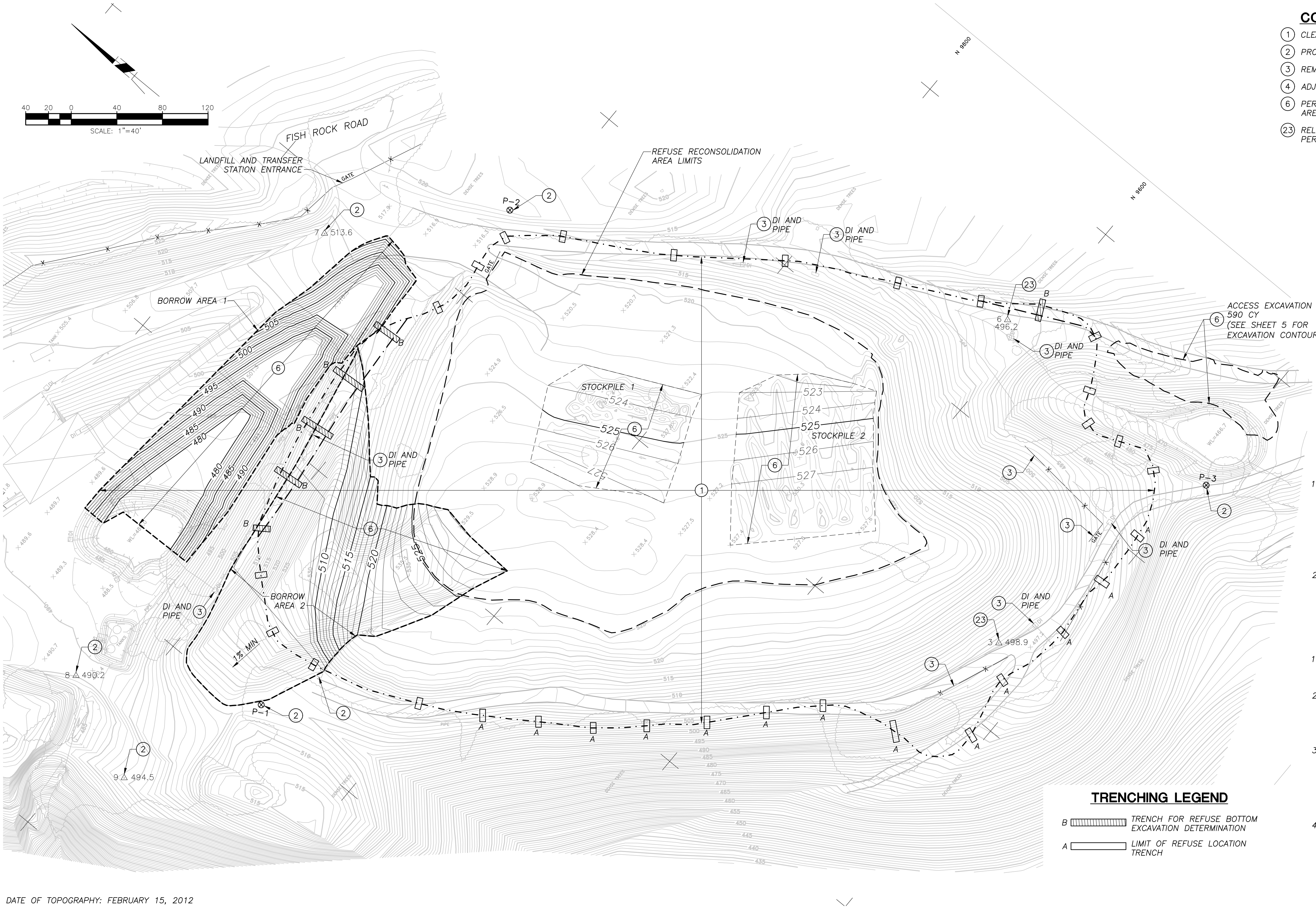
800-C SOUTH ROCHESTER AVENUE
ONTARIO, CALIFORNIA 91761

FIGURE 3

SOUTH COAST LANDFILL

1,000 FOOT RADIUS MAP

Z:\PROJECTS\MENDOCINO COUNTY\SOUTH COAST\FCPCMP\ACAD\FIG03-SOC 1000 FT RADIUS MAP



TRENCHING LEGEND

B ▥ TRENCH FOR REFUSE BOTTOM EXCAVATION DETERMINATION

A ▭ LIMIT OF REFUSE LOCATION TRENCH

DATE OF TOPOGRAPHY: FEBRUARY 15, 2012

CONSTRUCTION NOTES

- ① CLEAR AND GRUB
- ② PROTECT IN PLACE
- ③ REMOVE EXISTING STRUCTURES
- ④ ADJUST TO FINAL GRADE
- ⑥ PERFORM UNCLASSIFIED EXCAVATION AT STOCKPILE AREAS AND BORROW AREA FOR FINAL COVER
- ㉓ RELOCATE/ADJUST SURVEY MONUMENT TO GRADE PER PROJECT SPECIFICATIONS AND 2/D3

BORROW NOTES

1. ORDER OF BORROW AS FOLLOWS:
   - Ⓐ STOCKPILE 1 AND 2
   - Ⓑ BORROW AREA 1
   - Ⓒ ACCESS EXCAVATION
   - Ⓓ BORROW AREA 2 (SEE NOTE 2)
2. IF NOT ALL OF BORROW AREA 2 IS NEEDED, ROUND OFF SLOPE 2:1 MAX AND FLAT AREAS AT 3% OVER REFUSE; 1% OUTSIDE WASTE AREAS.

TRENCHING NOTES

1. CONTRACTOR'S SURVEYOR SHALL STAKE TRENCH LOCATIONS BASED ON A 5-FOOT OFFSET EXTERNAL TO THE APPROXIMATE REFUSE LIMIT.
2. REFUSE LIMIT VERIFICATION TRENCH TO START AT THE 5-FOOT OFFSET AND PROGRESS PERPENDICULAR TO REFUSE LIMIT INWARD UNTIL REFUSE LIMIT IS LOCATED; SURVEY LOCATION OF REFUSE LIMIT AND INTERIM COVER THICKNESS.
3. REFUSE BOTTOM LOCATION TRENCHING SHALL START SIMILAR TO (3) ABOVE; CONTRACTOR SHALL TRENCH INWARD FROM ESTABLISHED REFUSE LIMIT AND SHALL TRENCH THROUGH REFUSE TO 5-FOOT BEYOND INNER (PROPOSED) REFUSE LIMIT SO THAT A BOTTOM PROFILE CAN BE ESTABLISHED.
4. SURVEY OF REFUSE LIMIT AND BOTTOM PROFILE AT THE BEGINNING, END AND AT 10-FOOT MAXIMUM HORIZONTAL SPACING WITHIN THE REFUSE TRENCH SHALL BE PERFORMED.

FIGURE 4

NOTE:
1. SEE SHEET NO. 2 FOR LEGEND

NO.	REVISION DESCRIPTION	DATE

PREPARED BY:

SWT Civil & Environmental Engineering
800-C SOUTH ROCHESTER AVENUE
ONTARIO, CALIFORNIA 91761

MICHAEL A. CULLINANE
No. 41981

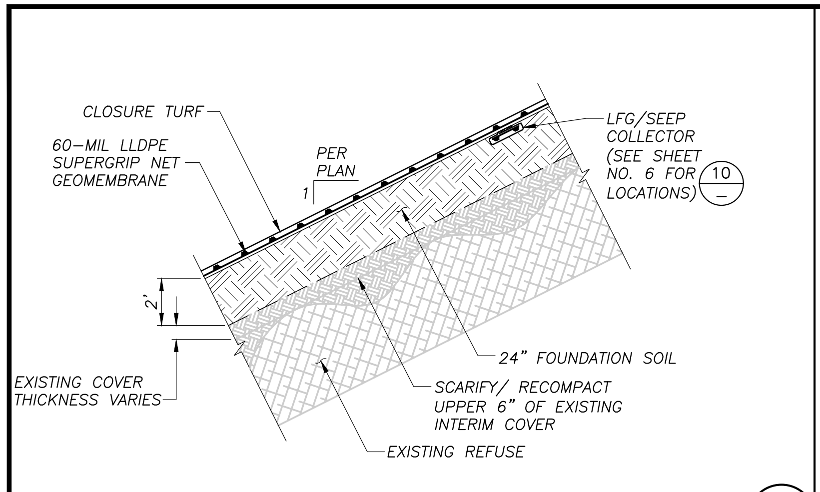
PREPARED UNDER THE SUPERVISION OF ______ DATE

MENDOCINO COUNTY
SOUTH COAST LANDFILL

FINAL CLOSURE CONSTRUCTION PROJECT 2016
REFUSE LIMIT VERIFICATION/TRENCHING/SOIL BORROW PLAN

DESIGNED BY : RMG	SCALE : AS SHOWN	PROJECT NO: –
DRAWN BY : CTD/DMW	DATE : 11-2016	
CHECKED BY :	DATE :	SHEET 3 OF 12
APPROVED BY :	DATE :	

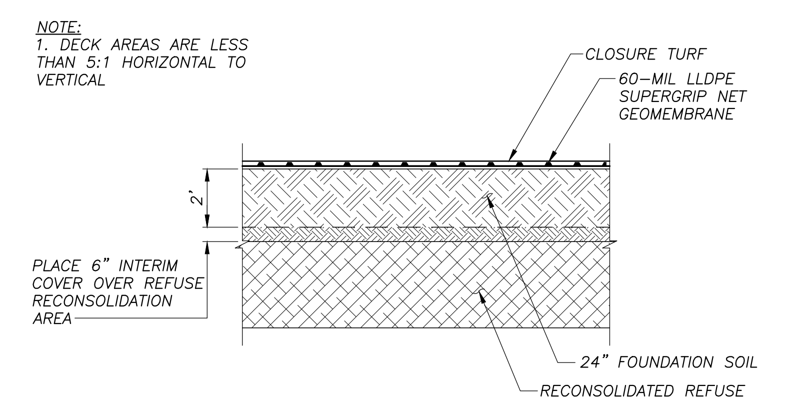
Z:\PROJECTS\MENDOCINO COUNTY\SOUTH COAST\FCPCMP\ACAD\FIG04-SOC STOCKPILE REMOVAL AND BORROW PLAN



FINAL COVER SECTION - SLOPE

NTS

1 D1

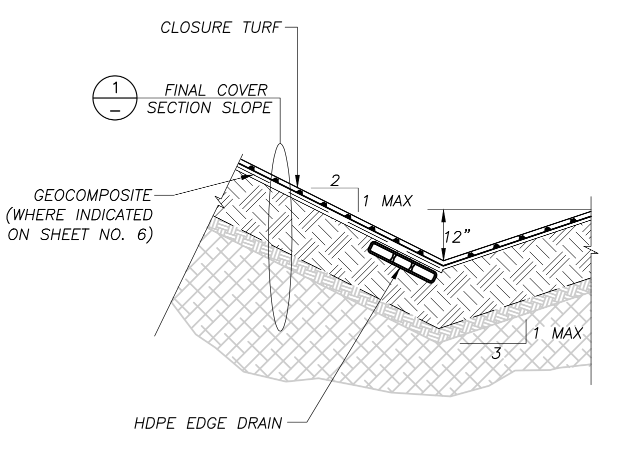


NOTE:
1. DECK AREAS ARE LESS THAN 5:1 HORIZONTAL TO VERTICAL

FINAL COVER SECTION - DECK

NTS

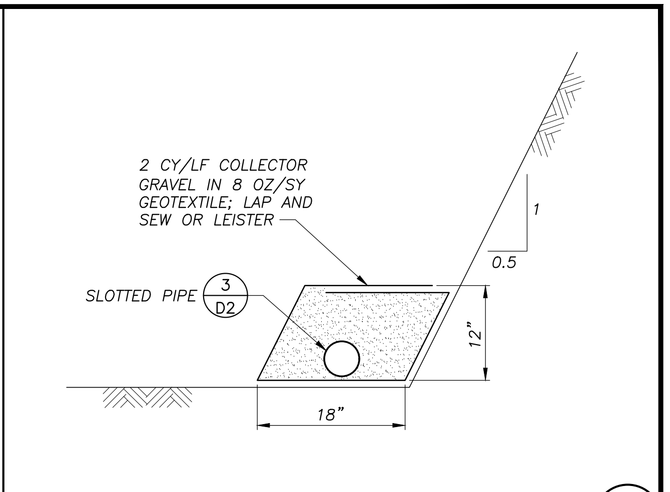
2 D1



HDPE EDGE DRAIN COLLECTOR

NTS

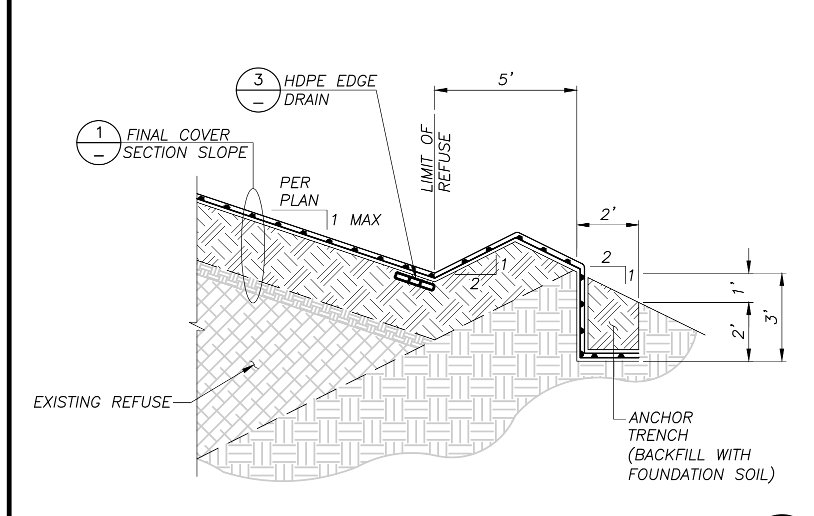
3 D1



DRAINAGE COLLECTOR

NTS

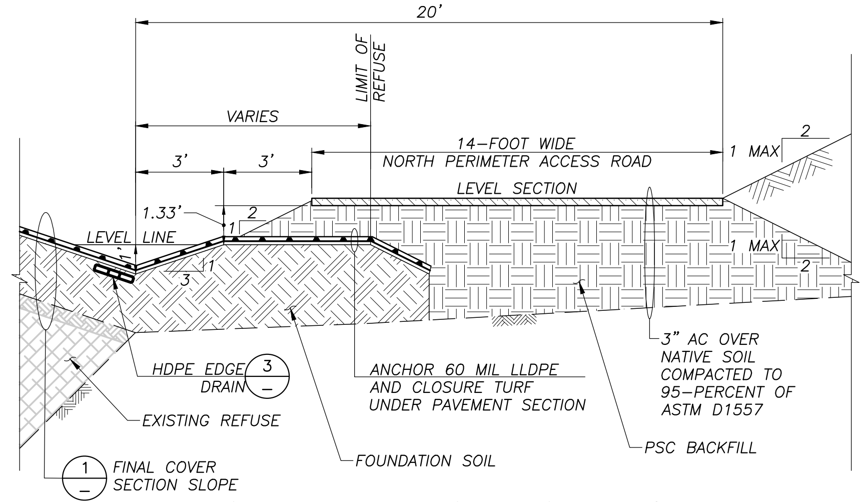
4 D1



FINAL COVER TERMINATION - SLOPE (WITH SWALE)

NTS

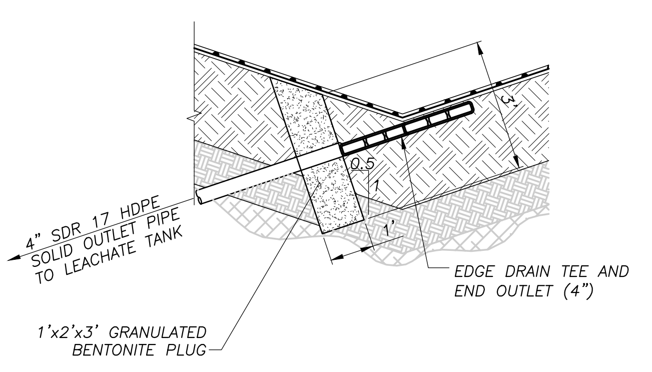
5 D1



FINAL COVER TERMINATION - NORTHEAST PERIMETER ROAD

NTS

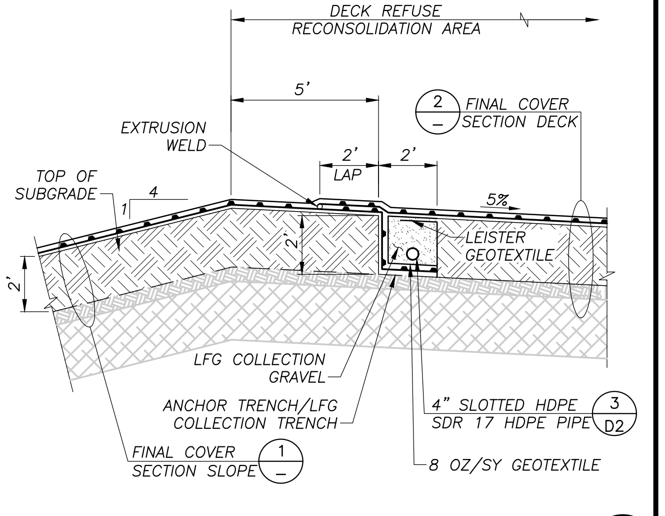
6 D1



BENTONITE PLUG

NTS

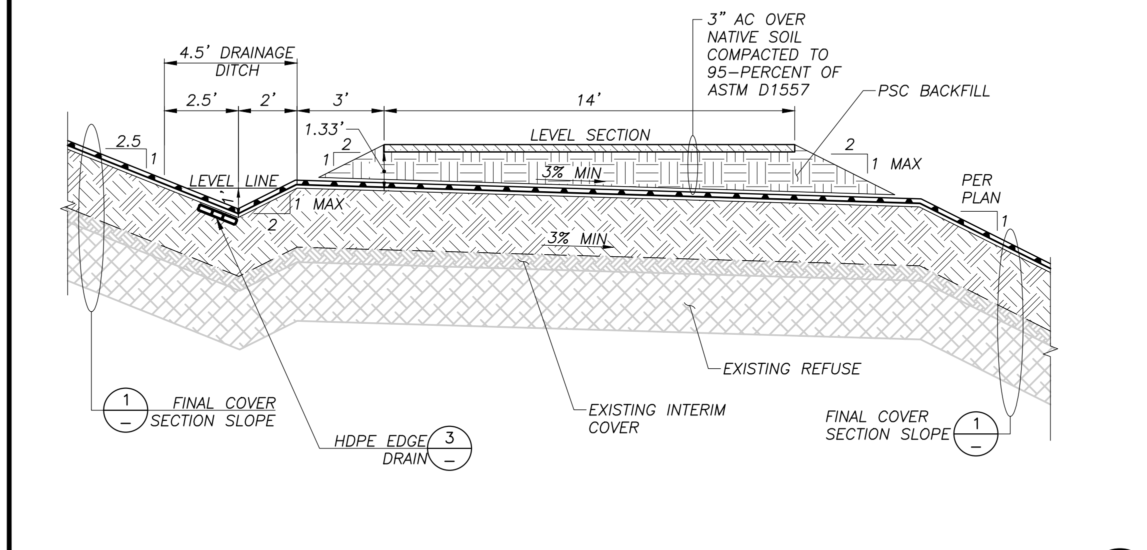
7 D1



TOP DECK TRANSITION

NTS

8 D1

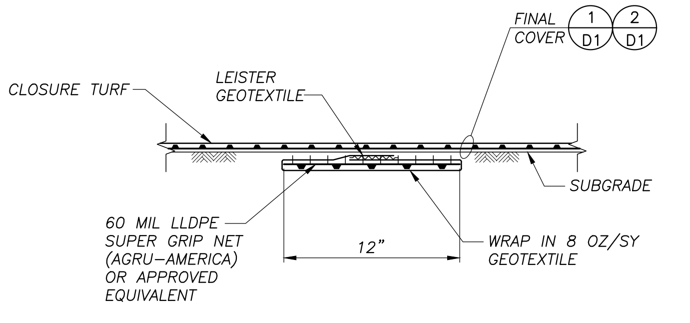


DECK ACCESS ROAD

NTS

9 D1

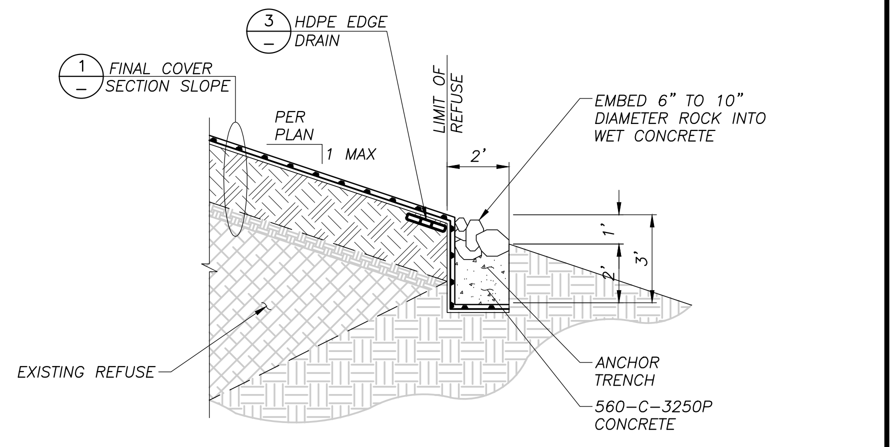
NOTE:
1. PLACE SUPER GRIP NET (12" WIDE STRIPS) AT LOCATION PER SHEET 6
2. EXTEND TO ANCHOR TRENCH AND BELOW GEOMEMBRANE HDPE EDGE DRAIN PER 5/– 8/–



LFG/SEEP COLLECTOR

NTS

10 D1



NOTE: WHERE SHOWN ON PLAN

FIGURE 5

FINAL COVER TERMINATION - SLOPE (NO SWALE)

NTS

11 D1

NO.	REVISION DESCRIPTION	DATE

PREPARED BY:

SWT Civil & Environmental Engineering
800-C SOUTH ROCHESTER AVENUE
ONTARIO, CALIFORNIA 91761

PREPARED UNDER THE SUPERVISION OF ________ DATE

MICHAEL A. CULLINANE No. 41981

MENDOCINO COUNTY
SOUTH COAST LANDFILL
FINAL CLOSURE CONSTRUCTION PROJECT 2016
DETAIL SHEET

DESIGNED BY : RMG	SCALE : AS SHOWN	PROJECT NO: –
DRAWN BY : CTD/DMW	DATE : 11-2016	
CHECKED BY :	DATE :	
APPROVED BY :	DATE :	SHEET 9 OF 12

Z:\PROJECTS\MENDOCINO COUNTY\SOUTH COAST\FCPCMP\ACAD\FIG05-SOC DETAIL SHEET D1

CONSTRUCTION NOTES

① CLEAR AND GRUB

② PROTECT IN PLACE

④ ADJUST TO FINAL GRADE

⑦ CONSTRUCT FINAL COVER SECTION – SLOPE 1/D1

⑧ CONSTRUCT FINAL COVER SECTION – DECK 2/D1

⑨ CONSTRUCT HDPE EDGE DRAIN COLLECTOR (BELOW FINAL COVER LLDPE) 3/D1

⑪ CONSTRUCT FINAL COVER SLOPE TERMINATION 5/D1 (WITH SWALE), OR 11/D1 (NO SWALE)

⑬ CONSTRUCT BENCH LINER TERMINATION 6/D3

⑭ CONSTRUCT FINAL COVER TERMINATION – NORTHEAST PERIMETER ROAD 6/D1

⑯ CONSTRUCT TOP DECK TRANSITION 8/D1

⑰ CONSTRUCT FINAL COVER TERMINATION – SOUTH AND WEST PERIMETER ROAD 4/D2

⑱ CONSTRUCT DECK ACCESS ROAD 9/D1

⑳ CONSTRUCT CONCRETE DOWNDRAIN/CHANNEL 1/D3

㉑ CONSTRUCT RIP RAP PAD 1/D2

㉒ CONSTRUCT BASIN BERM AND DOWNDRAIN INLET AT STORMWATER BASIN 3/D3

㉓ RELOCATE/ADJUST SURVEY MONUMENT TO GRADE PER PROJECT SPECIFICATIONS AND 2/D3

㉔ CONSTRUCT HDPE BOOT 7/D1

㉕ CONSTRUCT BENTONITE PLUG 7/D1

㉖ CONSTRUCT 4" SOLID HDPE PIPE (SDR 17)

㉗ EXTEND PIPE AND JOIN TO EXISTING LEACHATE OUTFALL PIPING/TANK

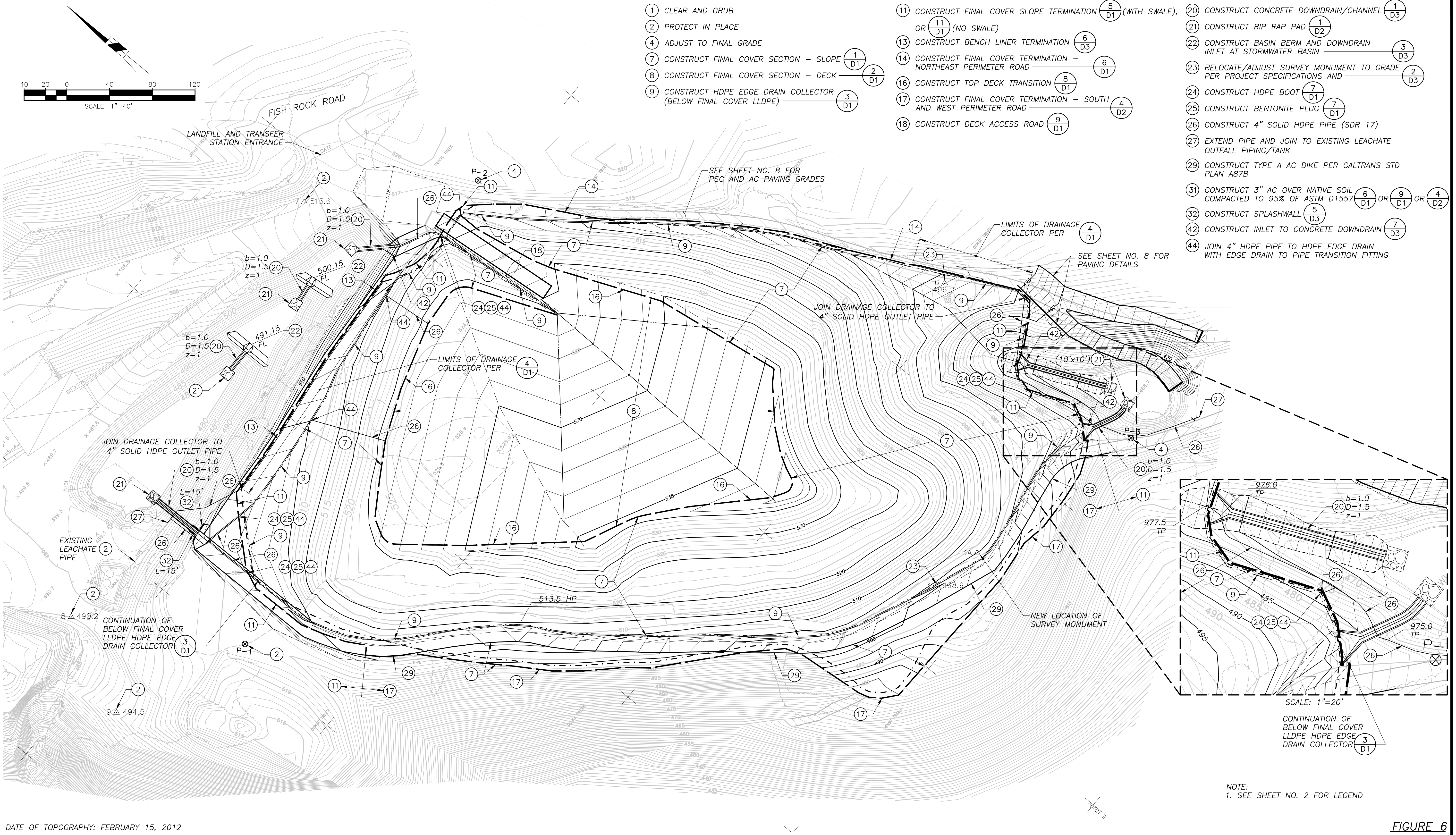
㉙ CONSTRUCT TYPE A AC DIKE PER CALTRANS STD PLAN A87B

㉛ CONSTRUCT 3" AC OVER NATIVE SOIL COMPACTED TO 95% OF ASTM D1557 6/D1 OR 9/D1 OR 4/D2

㉜ CONSTRUCT SPLASHWALL 5/D3

㊷ CONSTRUCT INLET TO CONCRETE DOWNDRAIN 7/D3

㊹ JOIN 4" HDPE PIPE TO HDPE EDGE DRAIN WITH EDGE DRAIN TO PIPE TRANSITION FITTING



DATE OF TOPOGRAPHY: FEBRUARY 15, 2012

NOTE:
1. SEE SHEET NO. 2 FOR LEGEND

FIGURE 6

NO.	REVISION DESCRIPTION	DATE

PREPARED BY:
SWT Civil & Environmental **Engineering**
800–C SOUTH ROCHESTER AVENUE
ONTARIO, CALIFORNIA 91761

PREPARED UNDER THE SUPERVISION OF
MICHAEL A. CULLINANE No. 41981
DATE

MENDOCINO COUNTY
SOUTH COAST LANDFILL

FINAL CLOSURE CONSTRUCTION PROJECT 2016
FINAL GRADING PLAN

DESIGNED BY : RMG	SCALE : AS SHOWN	PROJECT NO: –
DRAWN BY : CTD/DMW	DATE : 11–2016	
CHECKED BY :	DATE :	SHEET **5** OF **12**
APPROVED BY :	DATE :	

Z:\PROJECTS\MENDOCINO COUNTY\SOUTH COAST\FCPCMP\ACAD\FIG06–SOC FINAL GRADING PLAN

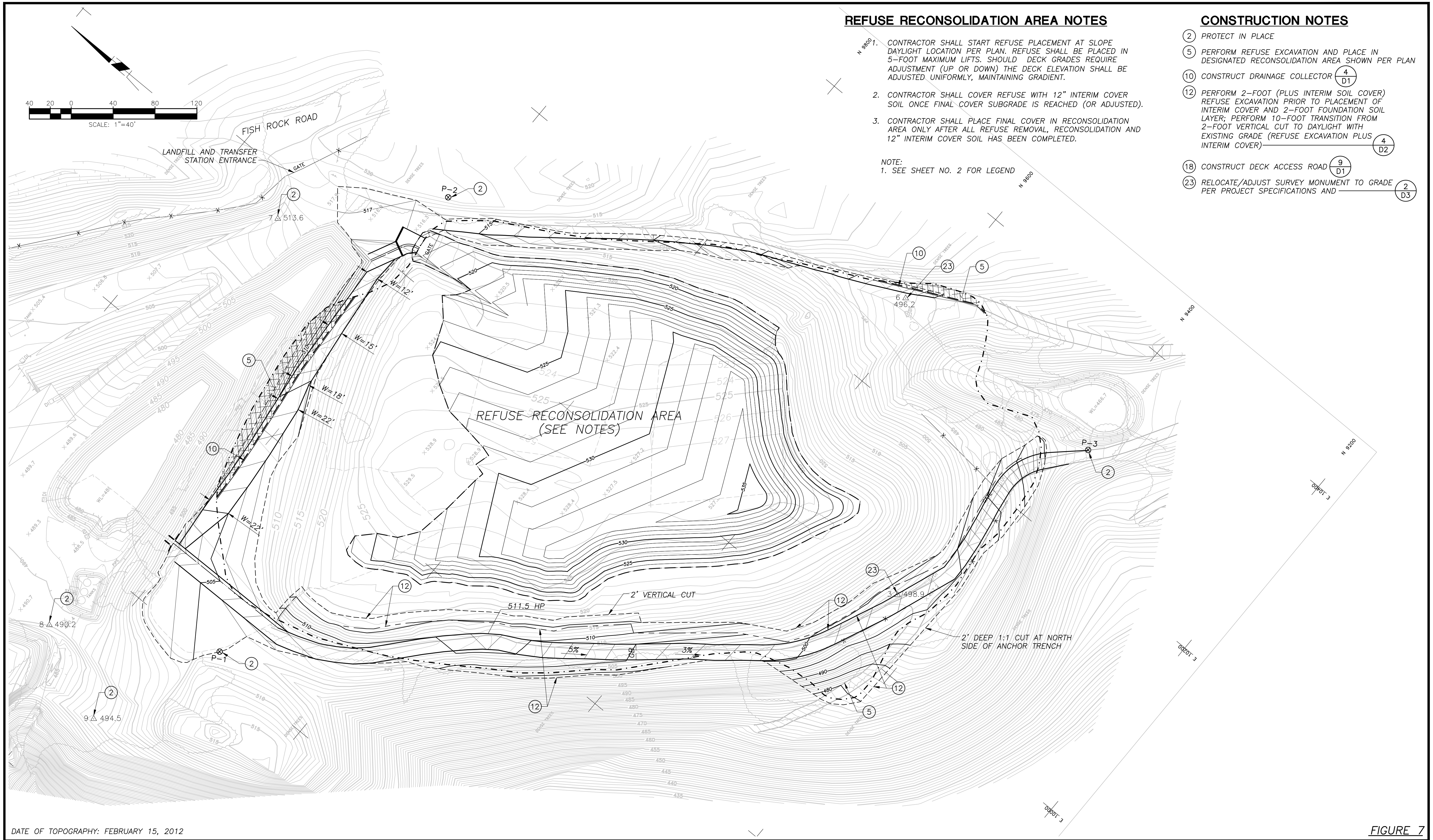
REFUSE RECONSOLIDATION AREA NOTES

1. CONTRACTOR SHALL START REFUSE PLACEMENT AT SLOPE DAYLIGHT LOCATION PER PLAN. REFUSE SHALL BE PLACED IN 5–FOOT MAXIMUM LIFTS. SHOULD DECK GRADES REQUIRE ADJUSTMENT (UP OR DOWN) THE DECK ELEVATION SHALL BE ADJUSTED UNIFORMLY, MAINTAINING GRADIENT.
2. CONTRACTOR SHALL COVER REFUSE WITH 12" INTERIM COVER SOIL ONCE FINAL COVER SUBGRADE IS REACHED (OR ADJUSTED).
3. CONTRACTOR SHALL PLACE FINAL COVER IN RECONSOLIDATION AREA ONLY AFTER ALL REFUSE REMOVAL, RECONSOLIDATION AND 12" INTERIM COVER SOIL HAS BEEN COMPLETED.

NOTE:
1. SEE SHEET NO. 2 FOR LEGEND

CONSTRUCTION NOTES

- ② PROTECT IN PLACE
- ⑤ PERFORM REFUSE EXCAVATION AND PLACE IN DESIGNATED RECONSOLIDATION AREA SHOWN PER PLAN
- ⑩ CONSTRUCT DRAINAGE COLLECTOR 4/D1
- ⑫ PERFORM 2–FOOT (PLUS INTERIM SOIL COVER) REFUSE EXCAVATION PRIOR TO PLACEMENT OF INTERIM COVER AND 2–FOOT FOUNDATION SOIL LAYER; PERFORM 10–FOOT TRANSITION FROM 2–FOOT VERTICAL CUT TO DAYLIGHT WITH EXISTING GRADE (REFUSE EXCAVATION PLUS INTERIM COVER) 4/D2
- ⑱ CONSTRUCT DECK ACCESS ROAD 9/D1
- ㉓ RELOCATE/ADJUST SURVEY MONUMENT TO GRADE PER PROJECT SPECIFICATIONS AND 2/D3



DATE OF TOPOGRAPHY: FEBRUARY 15, 2012

FIGURE 7

NO.	REVISION DESCRIPTION	DATE

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ONTARIO, CALIFORNIA 91761

MICHAEL A. CULLINANE
No. 41981

PREPARED UNDER THE SUPERVISION OF ______ DATE

MENDOCINO COUNTY
SOUTH COAST LANDFILL

FINAL CLOSURE CONSTRUCTION PROJECT 2016
REFUSE RECONSOLIDATION PLAN

DESIGNED BY : RMG	SCALE : AS SHOWN	PROJECT NO: –
DRAWN BY : CTD/DMW	DATE : 11–2016	
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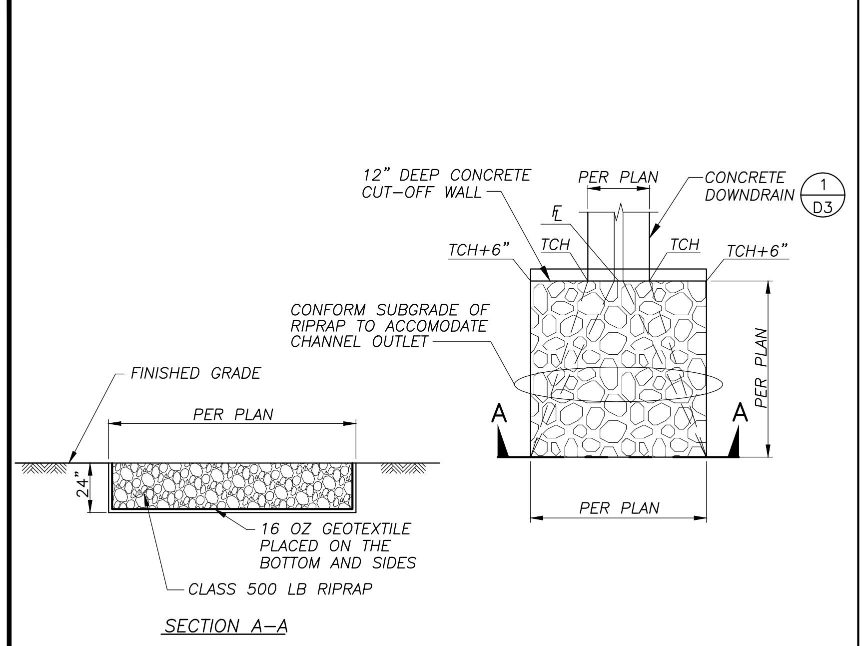
APPROVED BY :	DATE :	

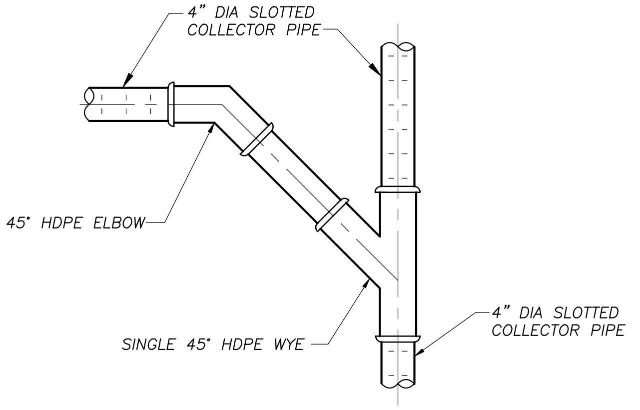
Z:\PROJECTS\MENDOCINO COUNTY\SOUTH COAST\FCPCMP\ACAD\FIG07–SOC REFUSE RECONSOLIDATION PLAN



RIP-RAP PAD

NTS

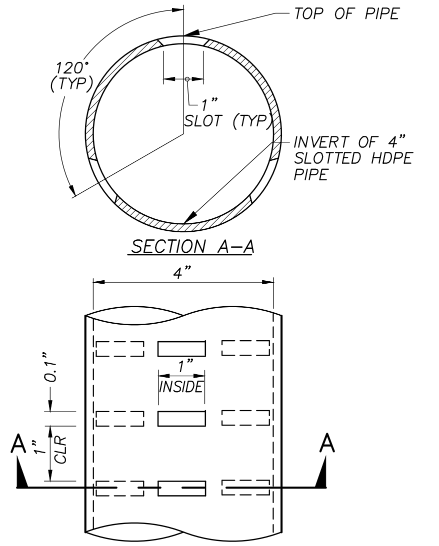
1 D2



SUBDRAIN JUNCTION DETAIL

NTS

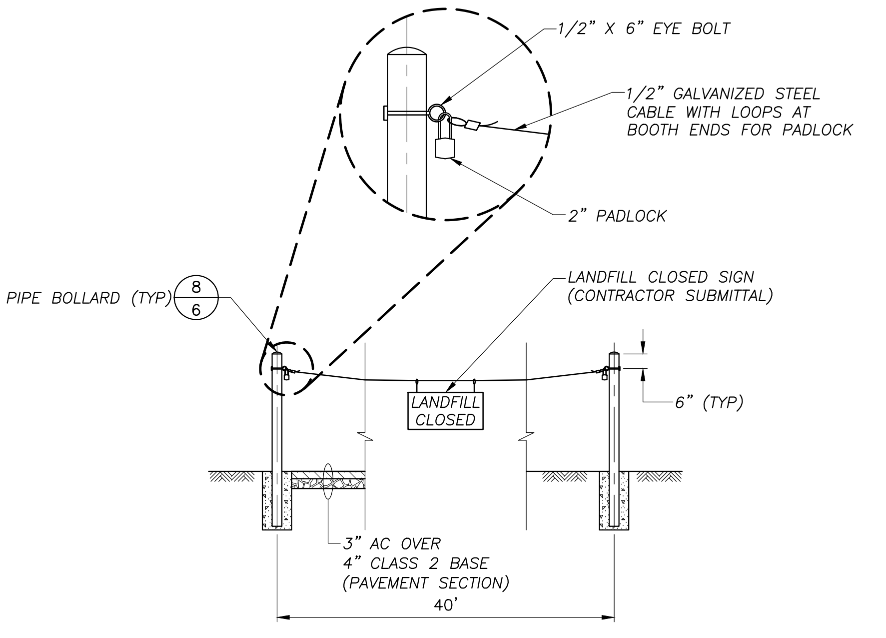
2 D2



SLOTTED HDPE PIPE

NTS

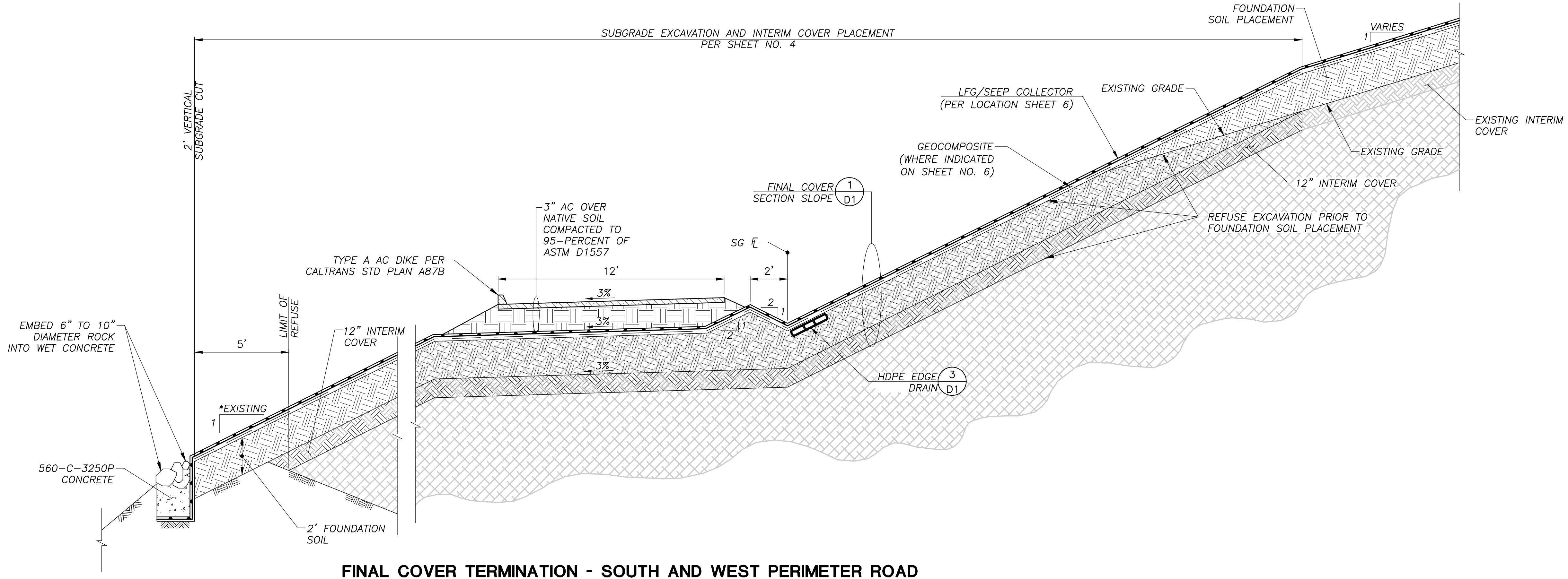
3 D2



CABLE GATE

NTS

5 D2



FINAL COVER TERMINATION - SOUTH AND WEST PERIMETER ROAD

NTS

FIGURE 8

4 D2

NO.	REVISION DESCRIPTION	DATE

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PREPARED UNDER THE SUPERVISION OF ______ DATE

MENDOCINO COUNTY
SOUTH COAST LANDFILL

FINAL CLOSURE CONSTRUCTION PROJECT 2016
DETAIL SHEET

DESIGNED BY : RMG	SCALE : AS SHOWN	PROJECT NO: –
DRAWN BY : CTD/DMW	DATE : 11-2016	
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APPROVED BY :	DATE :	SHEET 10 OF 12

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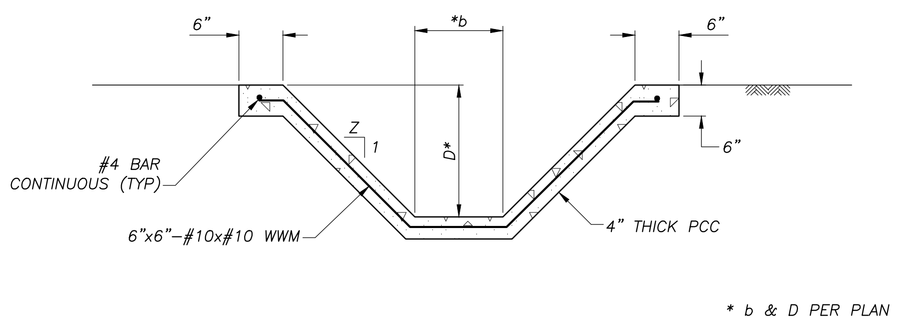
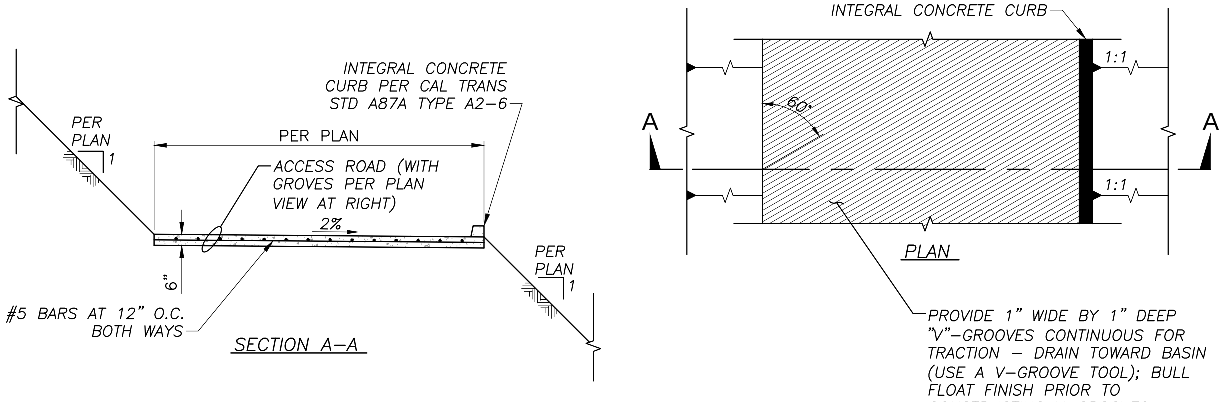


CONCRETE DOWNDRAIN/CHANNEL

NTS

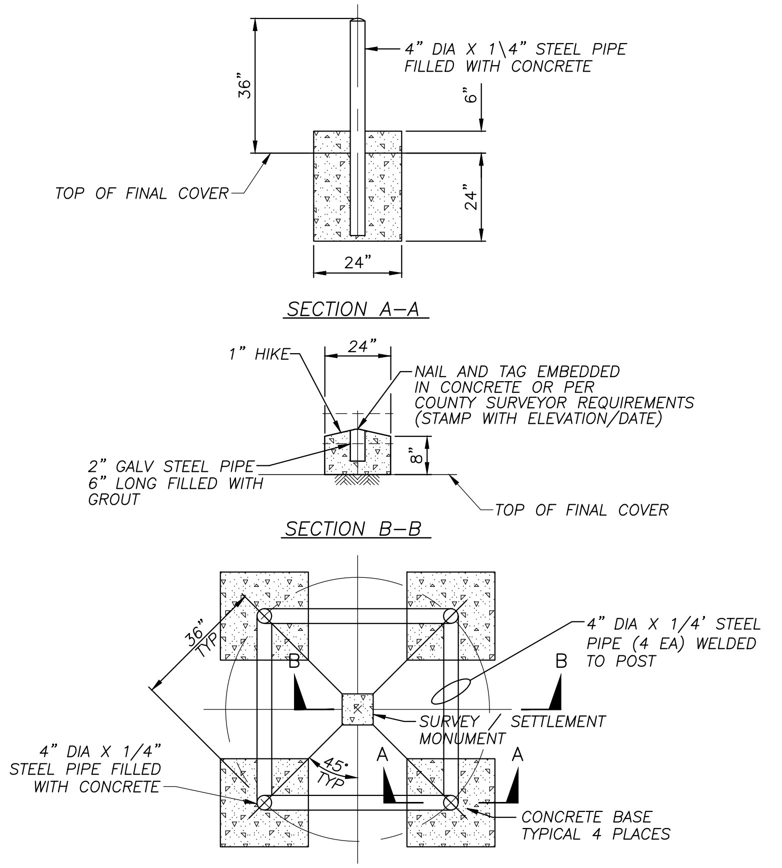
1/D3



CONCRETE ACCESS RAMP TO BASIN

NTS

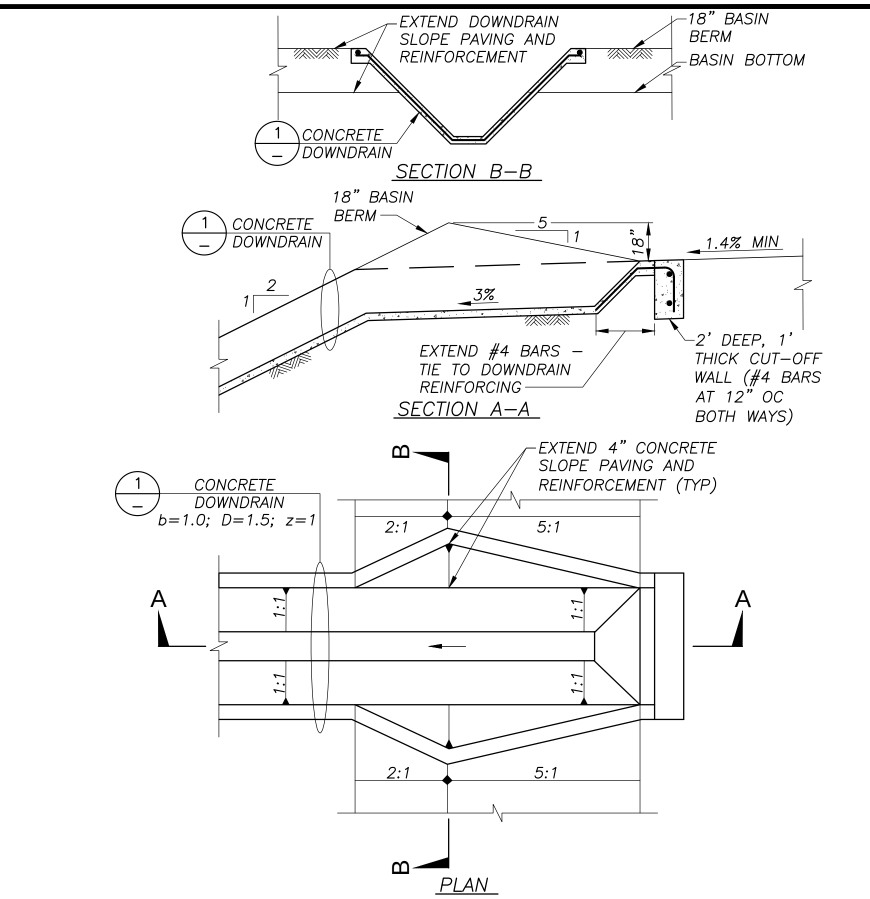
4/D3



POST-CLOSURE SURVEY/ SETTLEMENT MONUMENT

NTS

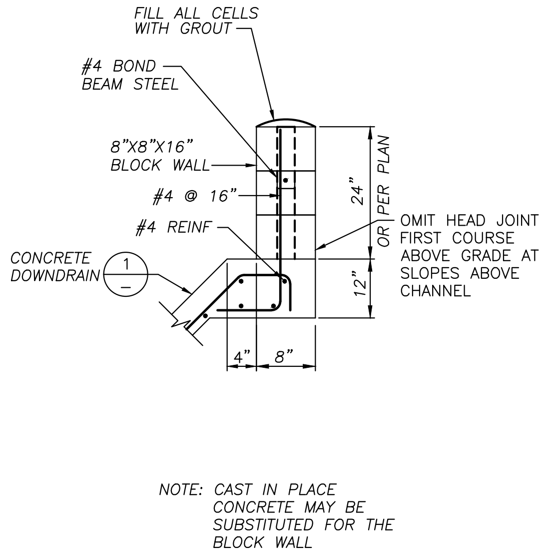
2/D3



BASIN BERM/DOWNDRAIN INLET

NTS

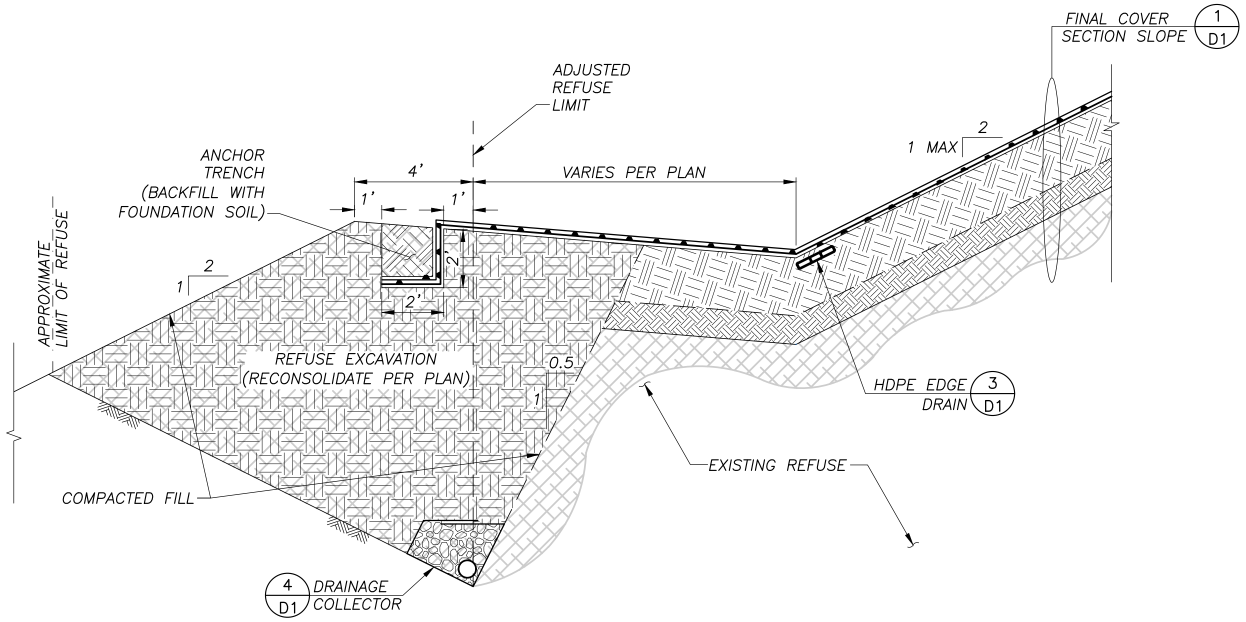
3/D3



SPLASHWALL

NTS

5/D3



BENCH LINER TERMINATION

NTS

6/D3

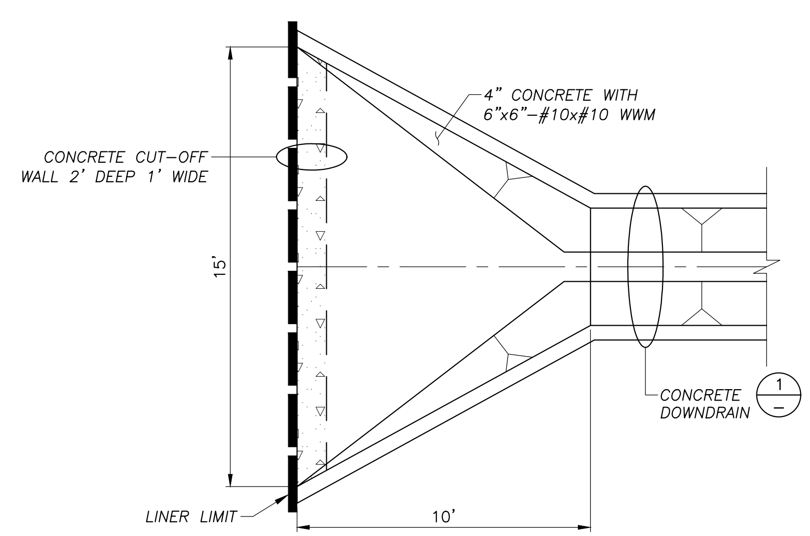


FIGURE 9

CONCRETE INLET TO DOWNDRAIN

NTS

7/D3

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ONTARIO, CALIFORNIA 91761

MICHAEL A. CULLINANE
No. 41981
CIVIL

PREPARED UNDER THE SUPERVISION OF

DATE

MENDOCINO COUNTY
SOUTH COAST LANDFILL

FINAL CLOSURE CONSTRUCTION PROJECT 2016
DETAIL SHEET

DESIGNED BY : RMG	SCALE : AS SHOWN	PROJECT NO: –
DRAWN BY : CTD/DMW	DATE : 11–2016	
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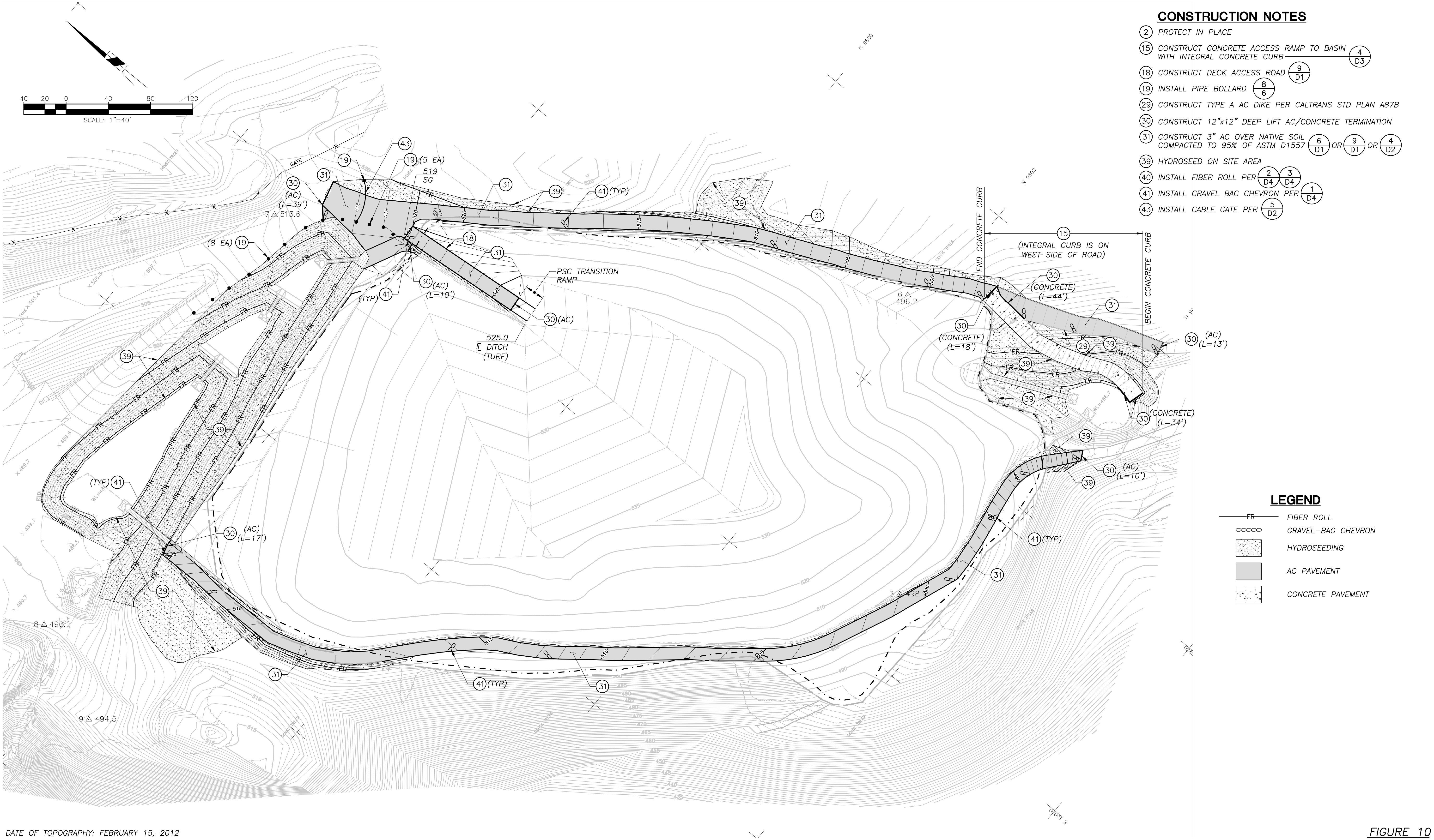
APPROVED BY :	DATE :	SHEET 11 OF 12

Z:\PROJECTS\MENDOCINO COUNTY\SOUTH COAST\FCPCMP\ACAD\FIG09–SOC DETAIL SHEET D3



CONSTRUCTION NOTES

- ② PROTECT IN PLACE
- ⑮ CONSTRUCT CONCRETE ACCESS RAMP TO BASIN WITH INTEGRAL CONCRETE CURB (4/D3)
- ⑱ CONSTRUCT DECK ACCESS ROAD (9/D1)
- ⑲ INSTALL PIPE BOLLARD (8/6)
- ㉙ CONSTRUCT TYPE A AC DIKE PER CALTRANS STD PLAN A87B
- ㉚ CONSTRUCT 12"x12" DEEP LIFT AC/CONCRETE TERMINATION
- ㉛ CONSTRUCT 3" AC OVER NATIVE SOIL COMPACTED TO 95% OF ASTM D1557 (6/D1) OR (9/D1) OR (4/D2)
- ㊴ HYDROSEED ON SITE AREA
- ㊵ INSTALL FIBER ROLL PER (2/D4) (3/D4)
- ㊶ INSTALL GRAVEL BAG CHEVRON PER (1/D4)
- ㊸ INSTALL CABLE GATE PER (5/D2)

LEGEND

—FR—	FIBER ROLL
⊂⊃⊂⊃	GRAVEL-BAG CHEVRON
▨	HYDROSEEDING
■	AC PAVEMENT
⁘	CONCRETE PAVEMENT

FIGURE 10

NO.	REVISION DESCRIPTION	DATE

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MICHAEL A. CULLINANE
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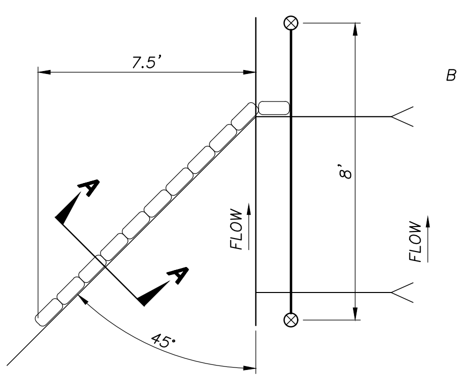
MENDOCINO COUNTY
SOUTH COAST LANDFILL

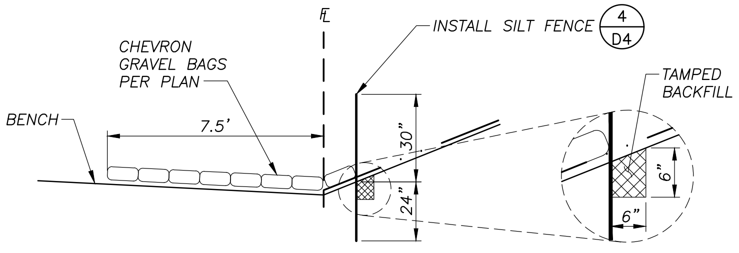
FINAL CLOSURE CONSTRUCTION PROJECT 2016
PAVING AND EROSION CONTROL PLAN

DESIGNED BY : RMG	SCALE : AS SHOWN	PROJECT NO: –
DRAWN BY : CTD/DMW	DATE : 11-2016	
CHECKED BY :	DATE :	SHEET 8 OF 12
APPROVED BY :	DATE :	

Z:\PROJECTS\MENDOCINO COUNTY\SOUTH COAST\FCPCMP\ACAD\FIG10-SOC PAVING AND EROSION CONTROL PLAN



PLAN



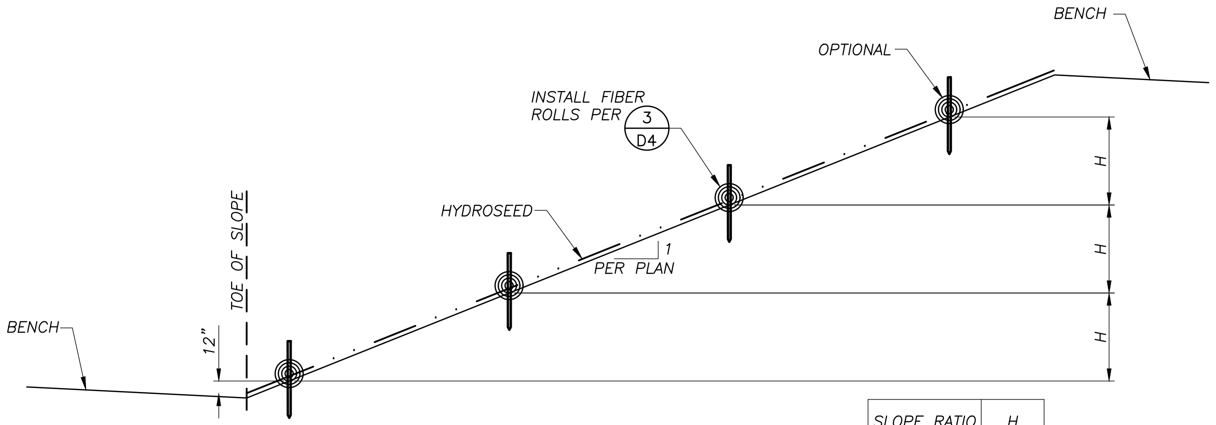
SECTION A–A

GRADIENT	UNPAVED SURFACE	PAVED SURFACE
1%–3%	200	250
3%–5%	150	200
5%–7%	100	150
7%–9%	50	100
9%+	25	50

GRAVEL BAG CHEVRON DETAIL

NTS

1/D4



SLOPE RATIO	H
2:1	7.5'
2.5:1	6.0'
3:1	5.0'

FIBER ROLL SPACING

NTS

2/D4

NOTES:

1) FIBER ROLL INSTALLATION AT BENCH SHALL BE A MINIMUM OF 12" ABOVE BENCH FLOWLINE, UNLESS SILT FENCE IS SPECIFIED AT THAT LOCATION.

2) A 3" DEEP TRENCH, THE WIDTH OF THE ROLL, DUG PER PLAN LOCATION SHALL "NEST" THE STRAW ROLL SO AS NOT TO ALLOW RUNOFF TO FLOW UNDER OR AROUND ROLL.

3) FIBER ROLLS SHALL BE PLACED ON SLOPES AT H–FEET VERTICAL INTERVALS STARTING FROM THE TOE OF RESPECTIVE SLOPE. DECK ROLLS SHALL BE PLACED AT 10–FEET VERTICAL INTERVALS AS SHOWN ON THE PLAN.

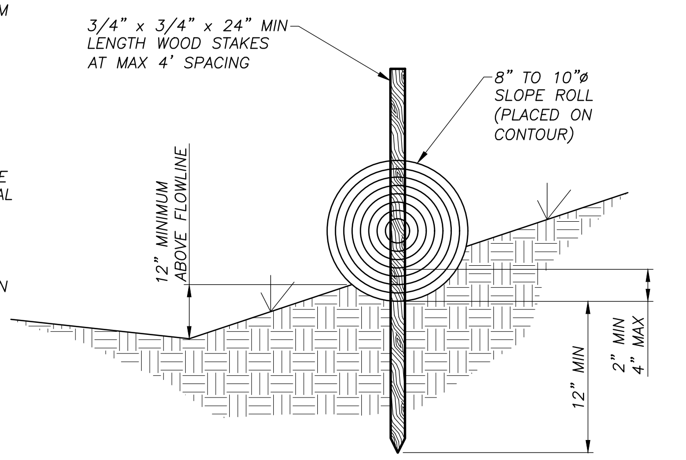
4) INSTALL FIBER ROLLS ALONG A LEVEL CONTOUR.

5) INSTALL FIBER ROLLS UP GRADIENT AND NEAR TRANSITION OF SLOPES.

6) INSTALL NUMEROUS ROLLS IN A ROW BY OVERLAPPING, NOT ABUTTING.

7) TURN ENDS OF ROLLS UP SLOPE TO PREVENT RUNOFF FROM GOING AROUND THE ROLL.

8) SEDIMENT IS TO BE REMOVED WHEN ACCUMULATION REACHES ONE HALF THE DISTANCE BETWEEN THE TOP OF THE ROLL AND ADJACENT GROUND SURFACE.



FIBER ROLLS

NTS

3/D4

NOTES:

1) THE LENGTH OF EACH REACH OF SILT FENCE IN NO CASE SHALL EXCEED 500'. USE GRAVEL BAG CROSS BARRIERS SPACED PER TABLE 1 AT RIGHT.

2) THE LAST 8'–0" OF EACH REACH OF FENCE SHALL BE TURNED UP SLOPE PER DETAIL 4/D4

3) STAKE DIMENSIONS ARE NOMINAL.

4) DIMENSIONS MAY VARY TO FIT FIELD CONDITION.

5) STAKES SHALL BE SPACED AT 8'–0" MAXIMUM AND SHALL BE POSITIONED ON DOWNSTREAM SIDE OF FENCE.

6) JOINING SECTION STAKES TO OVERLAP AND FENCE FABRIC TO FOLD AROUND EACH STAKE ONE FULL TURN. SECURE FABRIC TO STAKE WITH 4 HOG RING TIES.

7) JOINING SECTION STAKES SHALL BE DRIVEN TIGHTLY TOGETHER TO PREVENT POTENTIAL FLOW–THROUGH OF SEDIMENT AT JOINT. THE TOPS OF THE STAKES SHALL BE SECURED TO EACH OTHER WITH WIRE TIES TO PREVENT SEPARATION.

8) FOR END STAKES, FENCE FABRIC SHALL BE FOLDED AROUND TWO STAKES, ONE FULL TURN AND SECURED WITH 4 HOG RING TIES.

9) A MINIMUM OF 4 HOG RING TIES PER STAKE SHALL BE UTILIZED. DIMENSIONS SHOWN ARE TYPICAL.

10) CROSS BARRIERS SHALL BE SPACED PER TABLE 1 AT RIGHT.

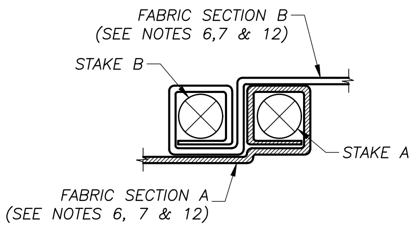
11) MAINTENANCE OPENINGS SHALL BE CONSTRUCTED IN A MANNER TO ENSURE THAT SEDIMENT REMAINS BEHIND THE SILT FENCE.

12) JOINING SECTIONS SHALL NOT BE PLACED AT SUMP LOCATIONS.

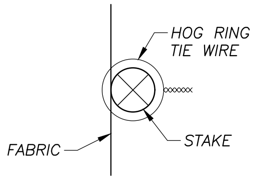
13) GRAVEL BAG ROWS AND LAYERS SHALL BE OFFSET TO ELIMINATE GAPS.

14) SILT FENCE FABRIC SHALL BE BURIED IN A TRENCH 6–INCHES WITH 6–INCH RETURN, AND SHALL BE BACKFILLED PER SECTION B–B, TO PREVENT SILT FROM UNDERMINING/BYPASSING FENCE FABRIC.

15) IN FLATTER SLOPE AREAS ADDITIONAL GRAVEL BAGS MAY BE REQUIRED TO MAINTAIN 12" MIN SILT STORAGE BEHIND CROSS BARRIER.



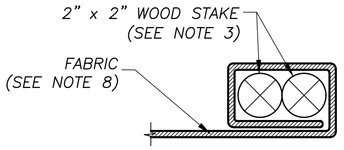
JOINING SECTION DETAIL (TOP VIEW)



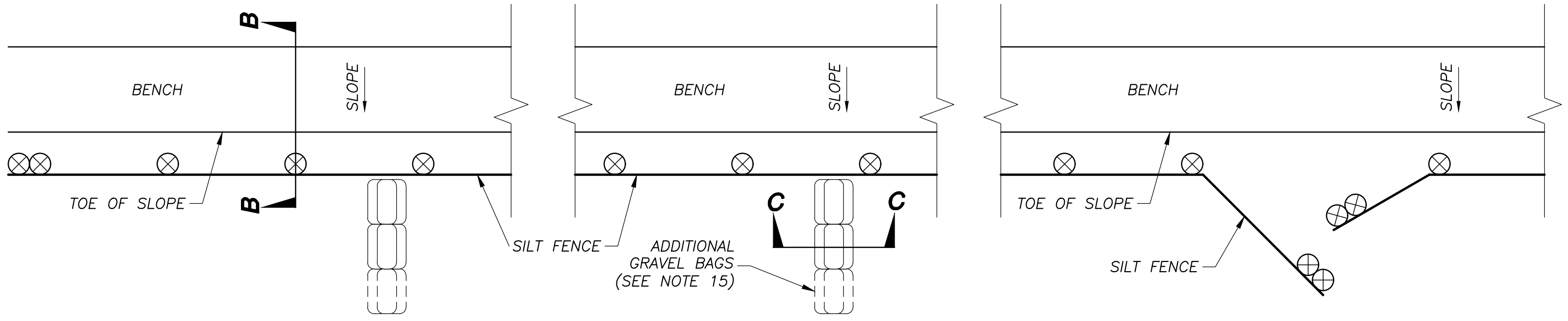
HOG RING TIE DETAIL

TABLE 1

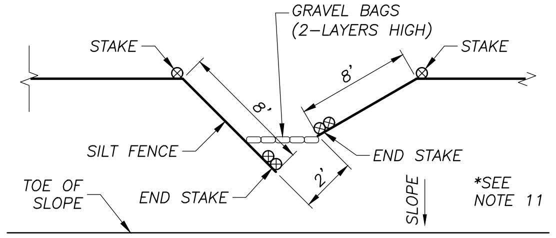
SLOPE OF BENCH/ROAD	CROSS BARRIER SPACING (FEET)
S ≤ 1.0%	100
1.0% < S ≤ 1.5%	75
1.5% < S ≤ 2.0%	50
2.0% < S ≤ 2.5%	50
2.5% < S ≤ 3.0%	40
3.0% < S ≤ 5.0%	30
5.0% < S	25



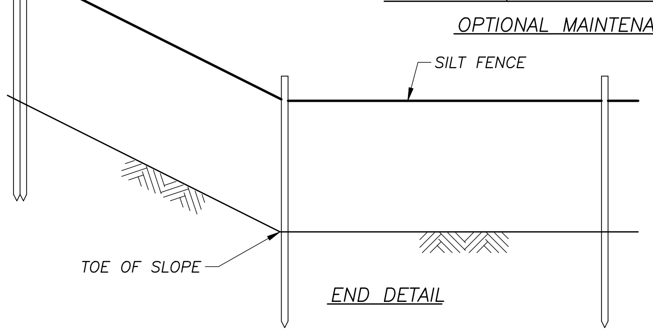
END STAKE SECTION DETAIL (TOP VIEW)



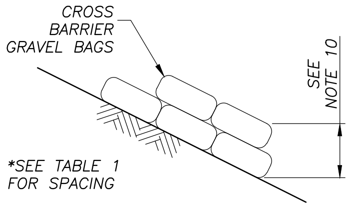
SILT FENCE INSTALLATION DETAILS (STEEL STAKES)



OPTIONAL MAINTENANCE OPENING DETAIL

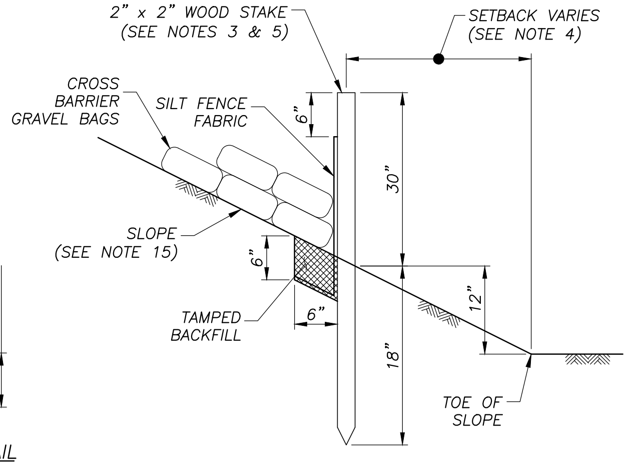


END DETAIL



CROSS BARRIER GRAVEL BAG DETAIL

SECTION C–C



SECTION B–B

SILT FENCE INSTALL

FIGURE 11

NTS

4/D4

NO.	REVISION DESCRIPTION	DATE

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SWT Civil & Environmental **Engineering**

800–C SOUTH ROCHESTER AVENUE
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MICHAEL A. CULLINANE
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PREPARED UNDER THE SUPERVISION OF

DATE

MENDOCINO COUNTY
SOUTH COAST LANDFILL

FINAL CLOSURE CONSTRUCTION PROJECT 2016
EROSION CONTROL DETAIL SHEET

DESIGNED BY : RMG	SCALE : AS SHOWN	PROJECT NO: –
DRAWN BY : CTD/DMW	DATE : 11–2016	
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APPROVED BY :	DATE :	

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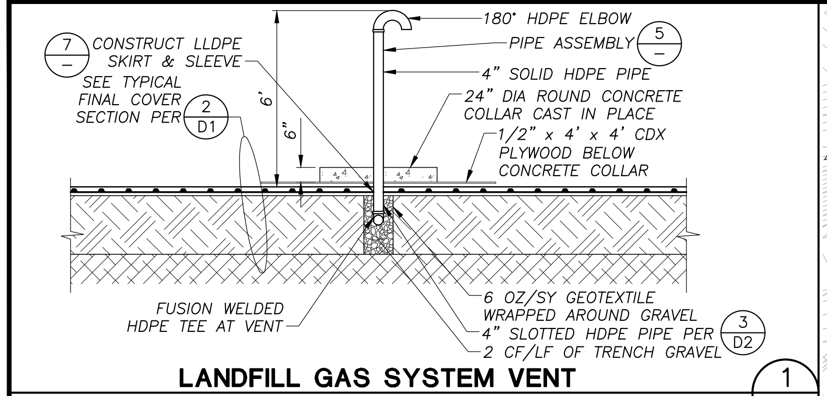
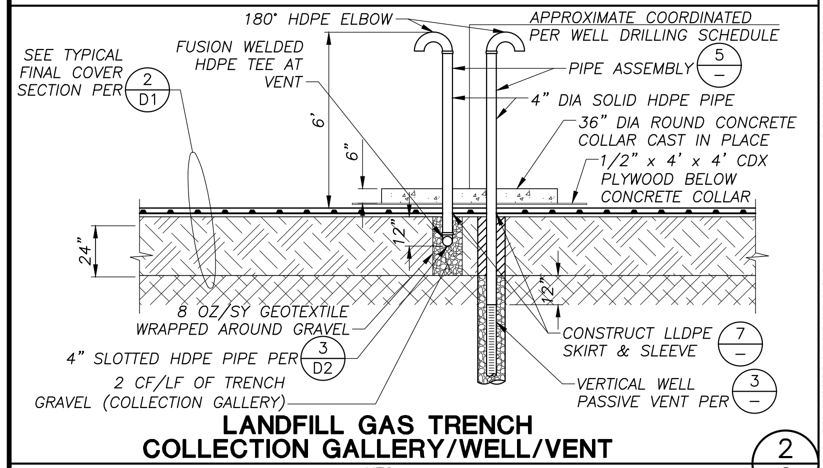


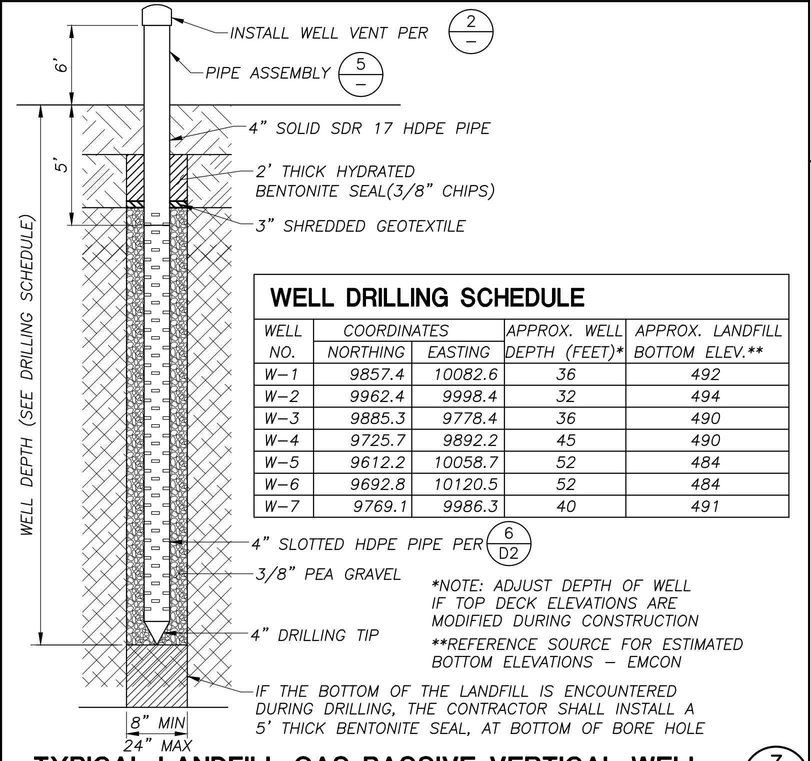
LANDFILL GAS SYSTEM VENT (1/6)

NTS



LANDFILL GAS TRENCH COLLECTION GALLERY/WELL/VENT (2/6)

NTS



WELL DRILLING SCHEDULE

WELL NO.	COORDINATES		APPROX. WELL DEPTH (FEET)*	APPROX. LANDFILL BOTTOM ELEV.**
	NORTHING	EASTING		
W-1	9857.4	10082.6	36	492
W-2	9962.4	9998.4	32	494
W-3	9885.3	9778.4	36	490
W-4	9725.7	9892.2	45	490
W-5	9612.2	10058.7	52	484
W-6	9692.8	10120.5	52	484
W-7	9769.1	9986.3	40	491

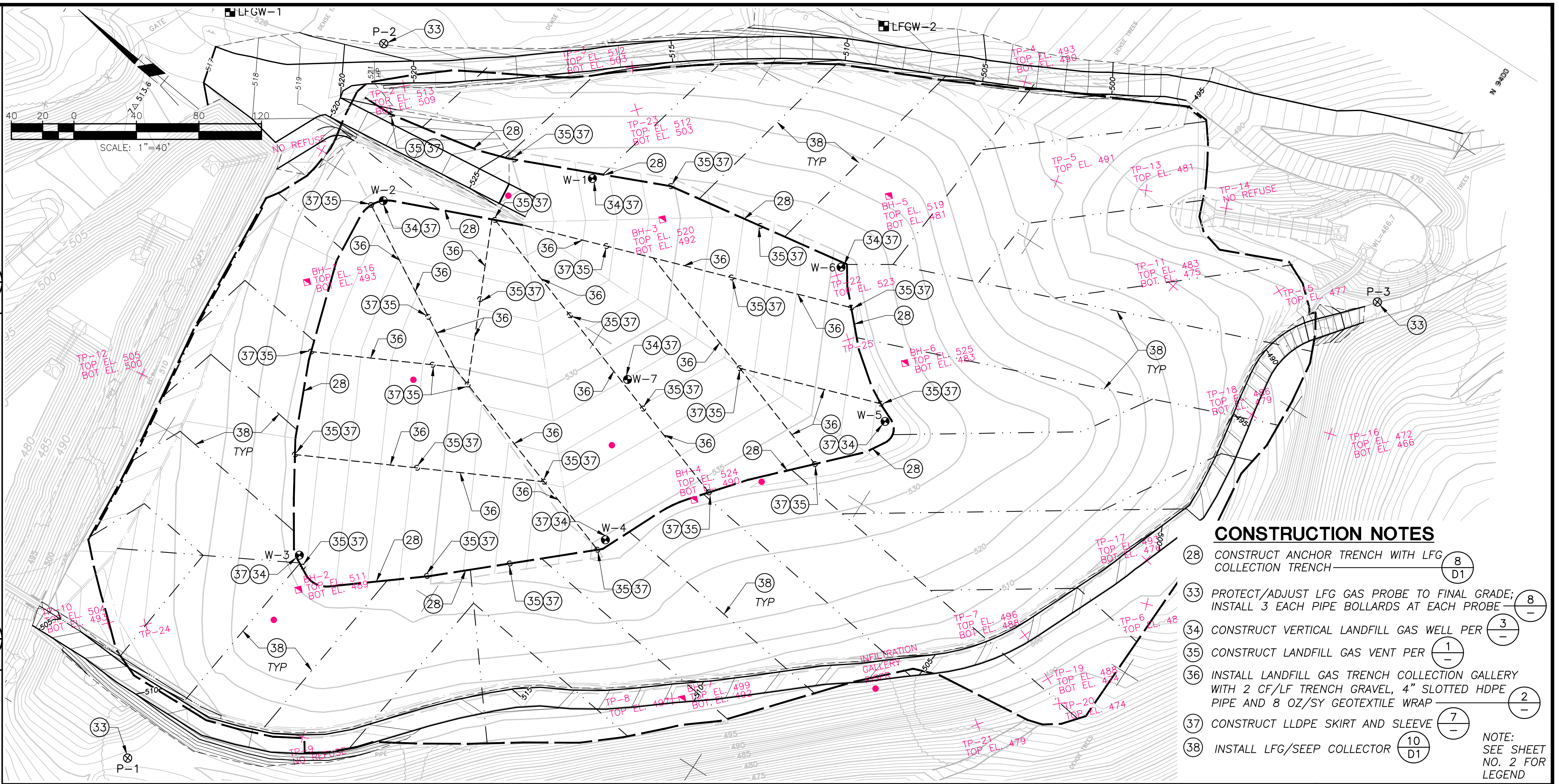
*NOTE: ADJUST DEPTH OF WELL IF TOP DECK ELEVATIONS ARE MODIFIED DURING CONSTRUCTION

**REFERENCE SOURCE FOR ESTIMATED BOTTOM ELEVATIONS – EMCON

IF THE BOTTOM OF THE LANDFILL IS ENCOUNTERED DURING DRILLING, THE CONTRACTOR SHALL INSTALL A 5' THICK BENTONITE SEAL, AT BOTTOM OF BORE HOLE

TYPICAL LANDFILL GAS PASSIVE VERTICAL WELL (3/6)

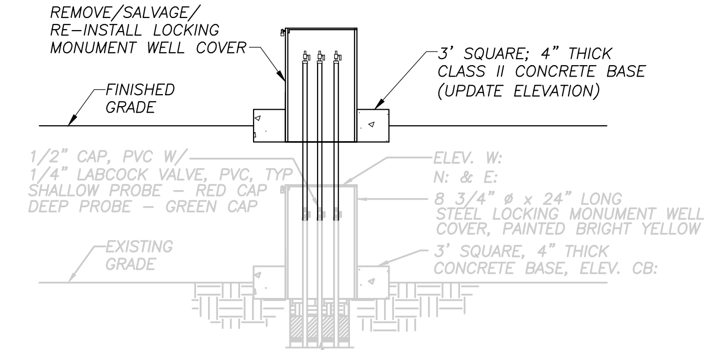
NTS



CONSTRUCTION NOTES

- ㉘ CONSTRUCT ANCHOR TRENCH WITH LFG COLLECTION TRENCH 8/D1
- ㉝ PROTECT/ADJUST LFG GAS PROBE TO FINAL GRADE; INSTALL 3 EACH PIPE BOLLARDS AT EACH PROBE 8/—
- ㉞ CONSTRUCT VERTICAL LANDFILL GAS WELL PER 3/—
- ㉟ CONSTRUCT LANDFILL GAS VENT PER 1/—
- ㊱ INSTALL LANDFILL GAS TRENCH COLLECTION GALLERY WITH 2 CF/LF TRENCH GRAVEL, 4" SLOTTED HDPE PIPE AND 8 OZ/SY GEOTEXTILE WRAP 2/—
- ㊲ CONSTRUCT LLDPE SKIRT AND SLEEVE 7/—
- ㊳ INSTALL LFG/SEEP COLLECTOR 10/D1

NOTE: SEE SHEET NO. 2 FOR LEGEND

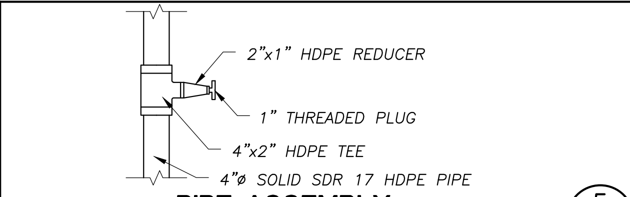


NOTES:

1. IF PROBE ELEVATION MATCHES FINAL GRADE CONSTRUCT PIPE BOLLARDS. 8/—
2. IF PROBE ELEVATION IS BELOW FINAL GRADE, REMOVE AND SALVAGE WELL COVER; PROTECT PROBES; FILL TO GRADE; EXTEND PVC PIPE (NO GLUE); REPLACE LID AND CONCRETE BASE – REVISE ELEVATION ON TAG.
3. INSTALL (3 EA) PIPE BOLLARDS AT EACH PROBE. 8/—

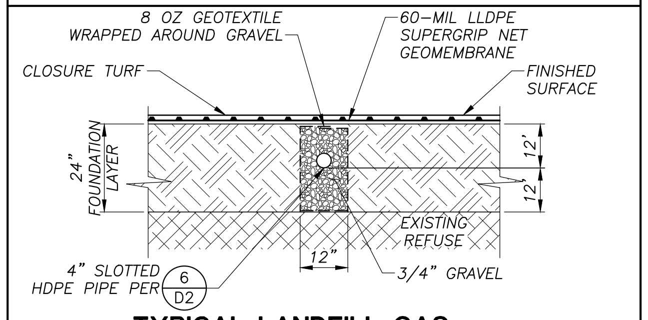
LANDFILL GAS MIGRATION MONITORING PROBE ADJUSTMENT TO GRADE (4/6)

NTS



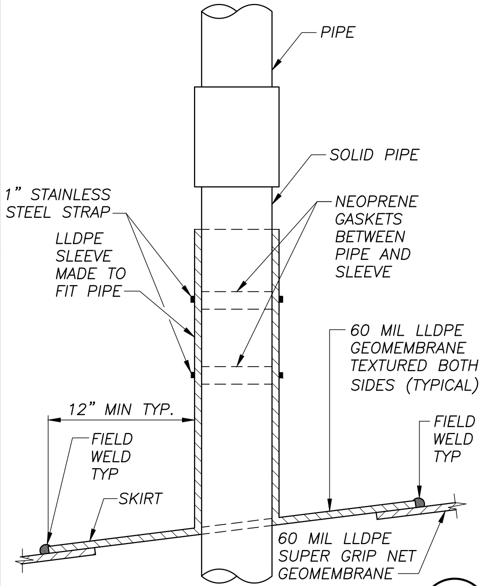
PIPE ASSEMBLY (5/6)

NTS



TYPICAL LANDFILL GAS PASSIVE TRENCH SECTION (6/6)

NTS



LLDPE SKIRT AND SLEEVE (7/6)

NTS

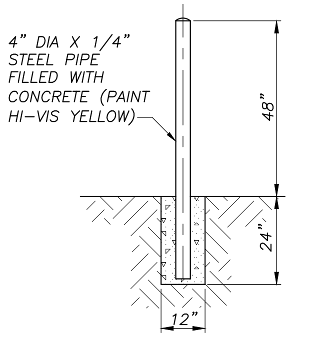


FIGURE 12

PIPE BOLLARD (8/6)

NTS

NO.	REVISION DESCRIPTION	DATE

PREPARED BY:

SWT Civil & Environmental **Engineering**
800-C SOUTH ROCHESTER AVENUE
ONTARIO, CALIFORNIA 91761

PREPARED UNDER THE SUPERVISION OF
MICHAEL A. CULLINANE No. 41981 CIVIL
DATE

MENDOCINO COUNTY
SOUTH COAST LANDFILL

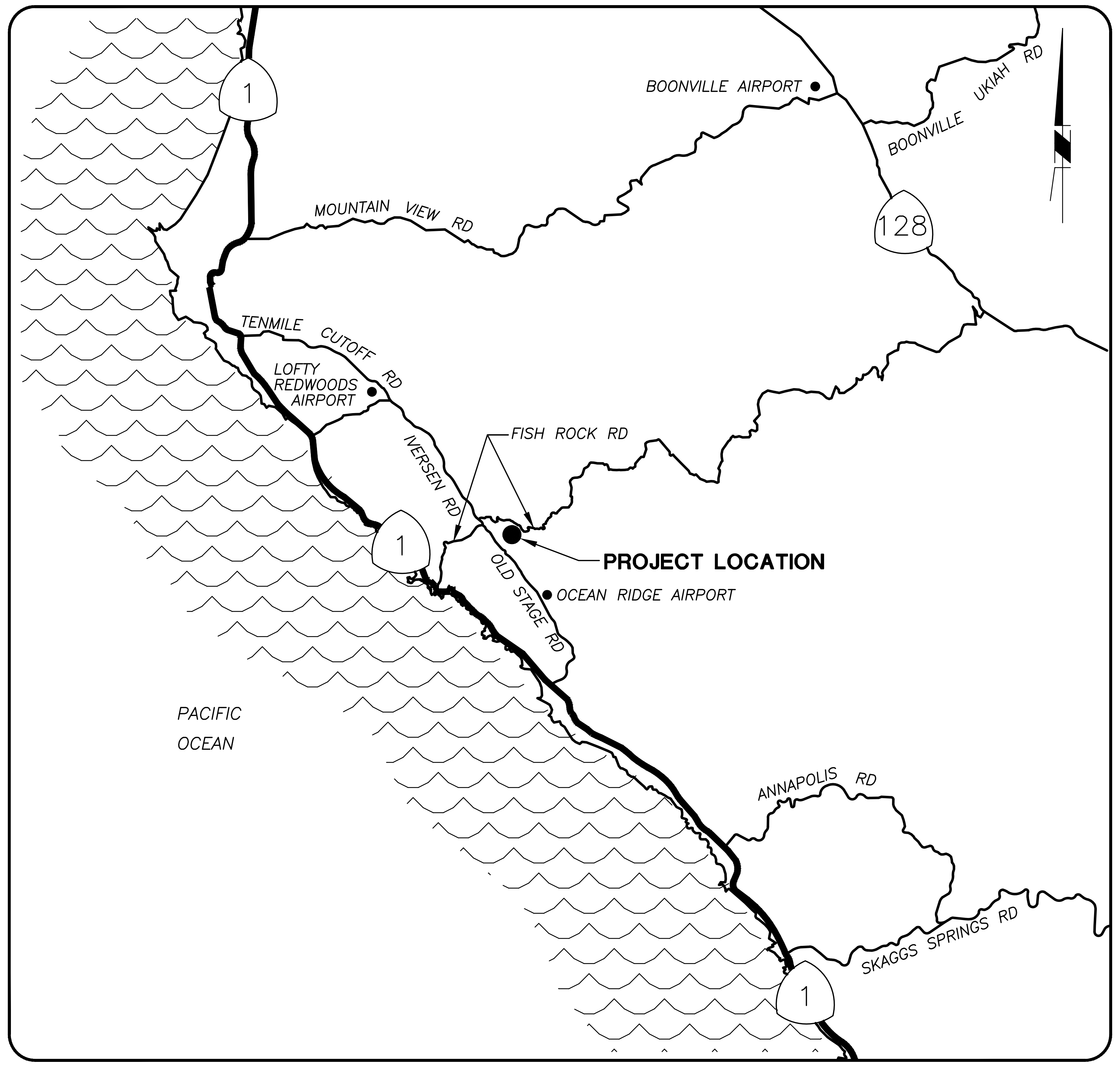
FINAL CLOSURE CONSTRUCTION PROJECT 2016
LANDFILL GAS PLAN

DESIGNED BY : RMG	SCALE : AS SHOWN	PROJECT NO: –
DRAWN BY : CTD/DMW	DATE : 11-2016	
CHECKED BY :	DATE :	SHEET 6 OF 12
APPROVED BY :	DATE :	

Z:\PROJECTS\MENDOCINO COUNTY\SOUTH COAST\FCPCMP\ACAD\FIG12-SOC LANDFILL GAS PLAN

DRAWINGS

SOUTH COAST LANDFILL
FINAL CLOSURE CONSTRUCTION PROJECT
GUALALA, CALIFORNIA
COUNTY OF MENDOCINO
DEPARTMENT OF TRANSPORTATION

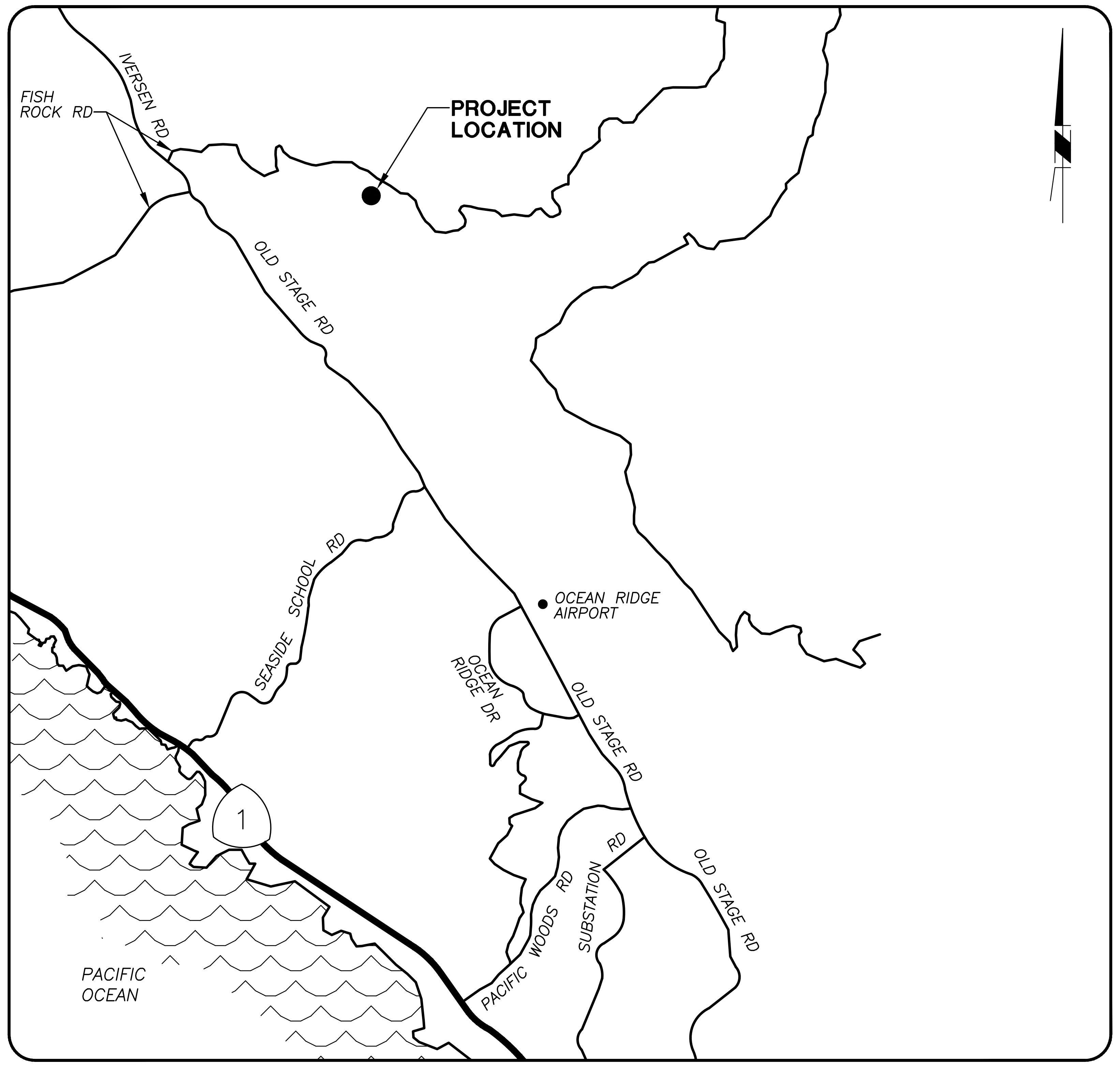


VICINITY MAP

SCALE: 1"=3 MI

LIST OF DRAWINGS

DRAWING NUMBER	TITLE AND DESCRIPTION
1	TITLE SHEET
2	SITE PLAN
3	REFUSE LIMIT VERIFICATION/ TRENCHING/SOIL BORROW PLAN
4	REFUSE RECONSOLIDATION PLAN
5	FINAL GRADING PLAN
6	LFG PLAN
7	SECTIONS
8	PAVING/EROSION CONTROL PLAN
D1	DETAIL SHEET
D2	DETAIL SHEET
D3	DETAIL SHEET
D4	EROSION CONTROL DETAIL SHEET



LOCATION MAP

SCALE: 1"=0.5 MI

DIGALERT
DIAL BEFORE YOU DIG
TWO WORKING DAYS BEFORE YOU DIG
TOLL FREE 1-800-227-2600
A PUBLIC SERVICE BY UNDERGROUND SERVICE ALERT

NO.	REVISION DESCRIPTION	DATE

PREPARED BY:

SWT Civil & Environmental Engineering
800-C SOUTH ROCHESTER AVENUE
ONTARIO, CALIFORNIA 91761

PREPARED UNDER THE SUPERVISION OF

MICHAEL A. CULLINANE
No. 41981

DATE

MENDOCINO COUNTY
SOUTH COAST LANDFILL
FINAL CLOSURE CONSTRUCTION PROJECT 2016
TITLE SHEET

DESIGNED BY : RMG	SCALE : AS SHOWN	PROJECT NO: –
DRAWN BY : CTD/DMW	DATE : 11-2016	
CHECKED BY :	DATE :	SHEET 1 OF 12
APPROVED BY :	DATE :	

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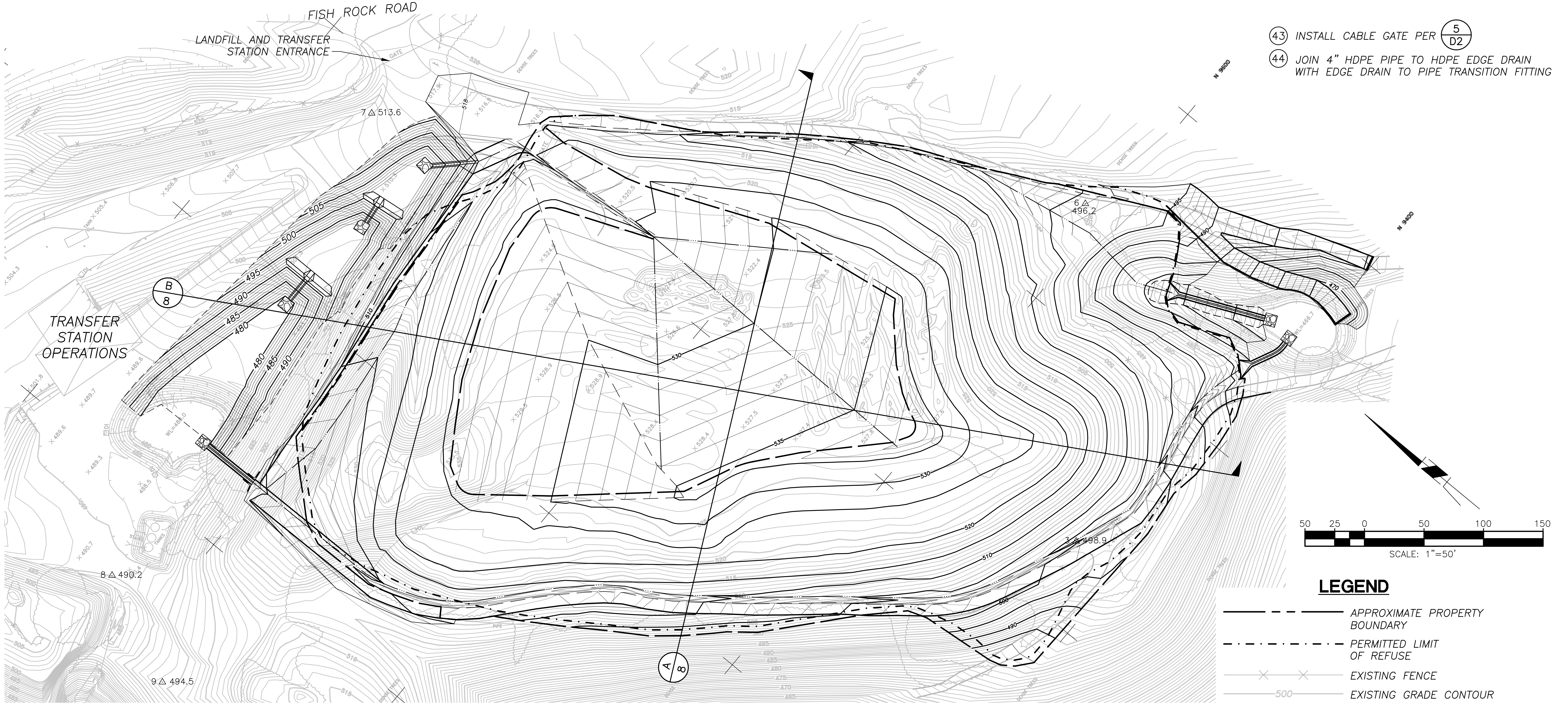


CONSTRUCTION NOTES

1. CLEAR AND GRUB
2. PROTECT IN PLACE
3. REMOVE EXISTING STRUCTURES
4. ADJUST TO FINAL GRADE
5. PERFORM REFUSE EXCAVATION AND PLACE IN DESIGNATED RECONSOLIDATION AREA SHOWN PER PLAN
6. PERFORM UNCLASSIFIED EXCAVATION AT STOCKPILE AREAS AND BORROW AREA FOR FINAL COVER
7. CONSTRUCT FINAL COVER SECTION – SLOPE 1/D1
8. CONSTRUCT FINAL COVER SECTION – DECK 2/D1
9. CONSTRUCT HDPE EDGE DRAIN COLLECTOR (BELOW FINAL COVER LLDPE) 3/D1
10. CONSTRUCT DRAINAGE COLLECTOR 4/D1
11. CONSTRUCT FINAL COVER SLOPE TERMINATION 5/D1 (WITH SWALE), OR 11/D1 (NO SWALE)
12. PERFORM 2-FOOT (PLUS INTERIM SOIL COVER) REFUSE EXCAVATION PRIOR TO PLACEMENT OF INTERIM COVER AND 2-FOOT FOUNDATION SOIL LAYER; PERFORM 10-FOOT TRANSITION FROM 2-FOOT VERTICAL CUT TO DAYLIGHT WITH EXISTING GRADE (REFUSE EXCAVATION PLUS INTERIM COVER) 4/D2
13. CONSTRUCT BENCH LINER TERMINATION 6/D3
14. CONSTRUCT FINAL COVER TERMINATION – NORTHEAST PERIMETER ROAD 6/D1
15. CONSTRUCT CONCRETE ACCESS RAMP TO BASIN WITH INTEGRAL CONCRETE CURB 4/D3
16. CONSTRUCT TOP DECK TRANSITION 8/D1
17. CONSTRUCT FINAL COVER TERMINATION – SOUTH AND WEST PERIMETER ROAD 4/D2
18. CONSTRUCT DECK ACCESS ROAD 9/D1
19. INSTALL PIPE BOLLARD 8/6
20. CONSTRUCT CONCRETE DOWNDRAIN/CHANNEL 1/D3
21. CONSTRUCT RIP RAP PAD 1/D2
22. CONSTRUCT BASIN BERM AND DOWNDRAIN INLET AT STORMWATER BASIN 3/D3
23. RELOCATE/ADJUST SURVEY MONUMENT TO GRADE PER PROJECT SPECIFICATIONS AND 2/D3
24. CONSTRUCT HDPE BOOT 7/D1
25. CONSTRUCT BENTONITE PLUG 7/D1
26. CONSTRUCT 4" SOLID HDPE PIPE (SDR 17)
27. EXTEND PIPE AND JOIN TO EXISTING LEACHATE OUTFALL PIPING/TANK
28. CONSTRUCT ANCHOR TRENCH WITH LFG COLLECTION TRENCH 8/D1
29. CONSTRUCT TYPE A AC DIKE PER CALTRANS STD PLAN A87B
30. CONSTRUCT 12"x12" DEEP LIFT AC/CONCRETE TERMINATION
31. CONSTRUCT 3" AC OVER NATIVE SOIL COMPACTED TO 95% OF ASTM D1557 6/D1 OR 9/D1 OR 4/D2
32. CONSTRUCT SPLASHWALL 5/D3
33. PROTECT/ADJUST LFG GAS PROBE TO FINAL GRADE; INSTALL 3 EACH PIPE BOLLARDS AT EACH PROBE 8/6
34. CONSTRUCT VERTICAL LANDFILL GAS WELL PER 3/6
35. CONSTRUCT LANDFILL GAS VENT PER 2/6
36. INSTALL LANDFILL GAS TRENCH COLLECTION GALLERY WITH 2 CF/LF TRENCH GRAVEL, 4" SLOTTED HDPE PIPE AND 8 OZ/SY GEOTEXTILE WRAP 2/6
37. CONSTRUCT LLDPE SKIRT AND SLEEVE 7/6
38. INSTALL LFG/SEEP COLLECTOR 10/D1
39. HYDROSEED ON SITE AREA
40. INSTALL FIBER ROLL PER 2/D4 3/D4
41. INSTALL GRAVEL BAG CHEVRON PER 1/D4
42. CONSTRUCT INLET TO CONCRETE DOWNDRAIN 7/D3
43. INSTALL CABLE GATE PER 5/D2
44. JOIN 4" HDPE PIPE TO HDPE EDGE DRAIN WITH EDGE DRAIN TO PIPE TRANSITION FITTING

LEGEND

— — - - — —	APPROXIMATE PROPERTY BOUNDARY
— · — · — · —	PERMITTED LIMIT OF REFUSE
——×——×——	EXISTING FENCE
——500——	EXISTING GRADE CONTOUR
——500——	FINISHED GRADE CONTOUR
— — — — — — —	TOE OF SLOPE
- - - - - - - - - - -	DAYLIGHT LINE
——·——·——·——	RIDGE LINE
— - — C/F — - —	CUT/FILL TRANSITION
▨	FINISHED BENCH
—·-·——·-·—	HDPE EDGE DRAIN
——‖——‖——	DRAINAGE COLLECTOR
— ·· — ·· —	LFG/SEEP COLLECTOR
3:1 →	DIRECTION AND RATE OF SLOPE
1/D1	DETAIL NUMBER SHEET SHOWN ON
7 △ 513.6	HORIZONTAL/VERTICAL CONTROL (HVC)
◆	FINAL COVER SETTLEMENT MONUMENT
⊗P-3	LFG PROBE
	VERTICAL PASSIVE VENT GAS WELL
----○----	LANDFILL GAS PASSIVE VENT RISER LOCATION
- - - - - - - - -	LANDFILL GAS PASSIVE TRENCH BELOW 60 MIL LLDPE
▚LFGW-1	GAS MONITORING PROBE

NOTICE TO CONTRACTOR

1. THE EXISTENCE AND LOCATION OF UNDERGROUND UTILITY PIPES OR STRUCTURES SHOWN ON THESE PLANS WERE OBTAINED BY A SEARCH OF AVAILABLE RECORDS. THESE LOCATIONS ARE APPROXIMATE AND SHALL BE CONFIRMED IN THE FIELD BY THE CONTRACTOR SO THAT ANY NECESSARY ADJUSTMENT CAN BE MADE IN ALIGNMENT AND/OR GRADE OF THE PROPOSED IMPROVEMENT. THE CONTRACTOR IS REQUIRED TO TAKE DUE PRECAUTIONARY MEASURES TO PROTECT ANY UTILITY LINES SHOWN AND ANY OTHER LINES NOT OF RECORD OR NOT SHOWN ON THESE PLANS.

2. IT SHALL BE THE RESPONSIBILITY OF THE CONTRACTOR TO MAKE EXACT DETERMINATION AS THE LOCATION OF ALL EXISTING UTILITIES. FORTY-EIGHT (48) HOURS PRIOR TO ANY EXCAVATION, THE CONTRACTOR SHALL NOTIFY UNDERGROUND SERVICE ALERT AT 1-800-422-4133 AND SHALL PROVIDE THE CONSTRUCTION MANAGER WITH VERIFICATION NUMBERS ISSUED.

3. THE FACT THAT ANY UTILITY FACILITY IS SHOWN OR NOT SHOWN UPON THE PLANS SHALL NOT RELIEVE THE CONTRACTOR OF THE RESPONSIBILITY UNDER SECTION 8-1.10 "UTILITY AND NON-HIGHWAY FACILITIES," OF THE STATE STANDARD SPECIFICATIONS. IT SHALL BE THE CONTRACTOR'S RESPONSIBILITY, PURSUANT THERETO, TO ASCERTAIN THE LOCATION OF ANY UTILITY FACILITY WHICH MAY BE SUBJECT TO DAMAGE BY REASON OF THE CONTRACTOR'S OPERATIONS.

4. REGARDING SIGNED DRAWING VERSUS ELECTRONIC COPY: SHOULD THE CONTRACTOR OR CONTRACTOR'S SURVEYOR FIND A CONTOUR LABEL OR ELEVATION DISCREPANCY BETWEEN THE SIGNED DRAWING AND THE ELECTRONIC VERSION (WHICH HAVE BEEN PROVIDED AS A COURTESY FOR EASE OF CONSTRUCTION STAKING), THE CONTRACTOR SHALL NOTIFY THE ENGINEER OF THE DISCREPANCY SO THAT THE DISCREPANCY MAY BE RESOLVED, PRIOR TO CONSTRUCTION. THE SIGNED DRAWING SUPERSEDES THE ELECTRONIC VERSION.

COORDINATE CONTROL POINTS

PT NO	NORTHING	EASTING	ELEVATION
3	9443.63	10064.07	498.9
6	9618.41	10287.41	496.2
7	10128.71	9963.78	513.6
8	10049.25	9525.71	490.2
9	9959.54	9483.59	494.5

NOTES

1. TOPOGRAPHIC DATA COMPILED FROM 15 FEBRUARY 2012 AERIAL PHOTOGRAPHY BY DELTA GEOMATICS CORPORATION.
2. ELEVATIONS ARE IN FEET ABOVE MEAN SEA LEVEL, NATIONAL GEODETIC VERTICAL DATUM OF 1929 (NGVD29).
3. GRID COORDINATES CORRESPOND TO THE CALIFORNIA STATE PLANE COORDINATE SYSTEM, ZONE 3, NAD83.
4. LOCATIONS OF STOCKPILES ARE APPROXIMATE.
5. EQUIPMENT AND MATERIAL STAGING AREA(S) TO BE DETERMINED BY OWNER.
6. ALL EXISTING STRUCTURES SUCH AS BUILDINGS, POLES, FENCES, PIPES, DITCHES, PONDS, ETC. WITHIN LIMIT OF EARTHWORK SHALL BE REMOVED AS REQUIRED BY THE OWNER.

ABBREVIATIONS

FL	FLOWLINE	LFG	LANDFILL GAS
FP	FINISHED PAVEMENT	GB	GRADE BREAK
HP	HIGH POINT	TC	TOP OF CURB
LP	LOW POINT	TS	TOP OF SLOPE
PP	POWER POLE	UD	UNDERDRAIN
INV	INVERT OF PIPE	CHDPE	CORRUGATED HIGH DENSITY POLYETHYLENE

NO.	REVISION DESCRIPTION	DATE

PREPARED BY:

SWT Civil & Environmental **Engineering**
800-C SOUTH ROCHESTER AVENUE
ONTARIO, CALIFORNIA 91761

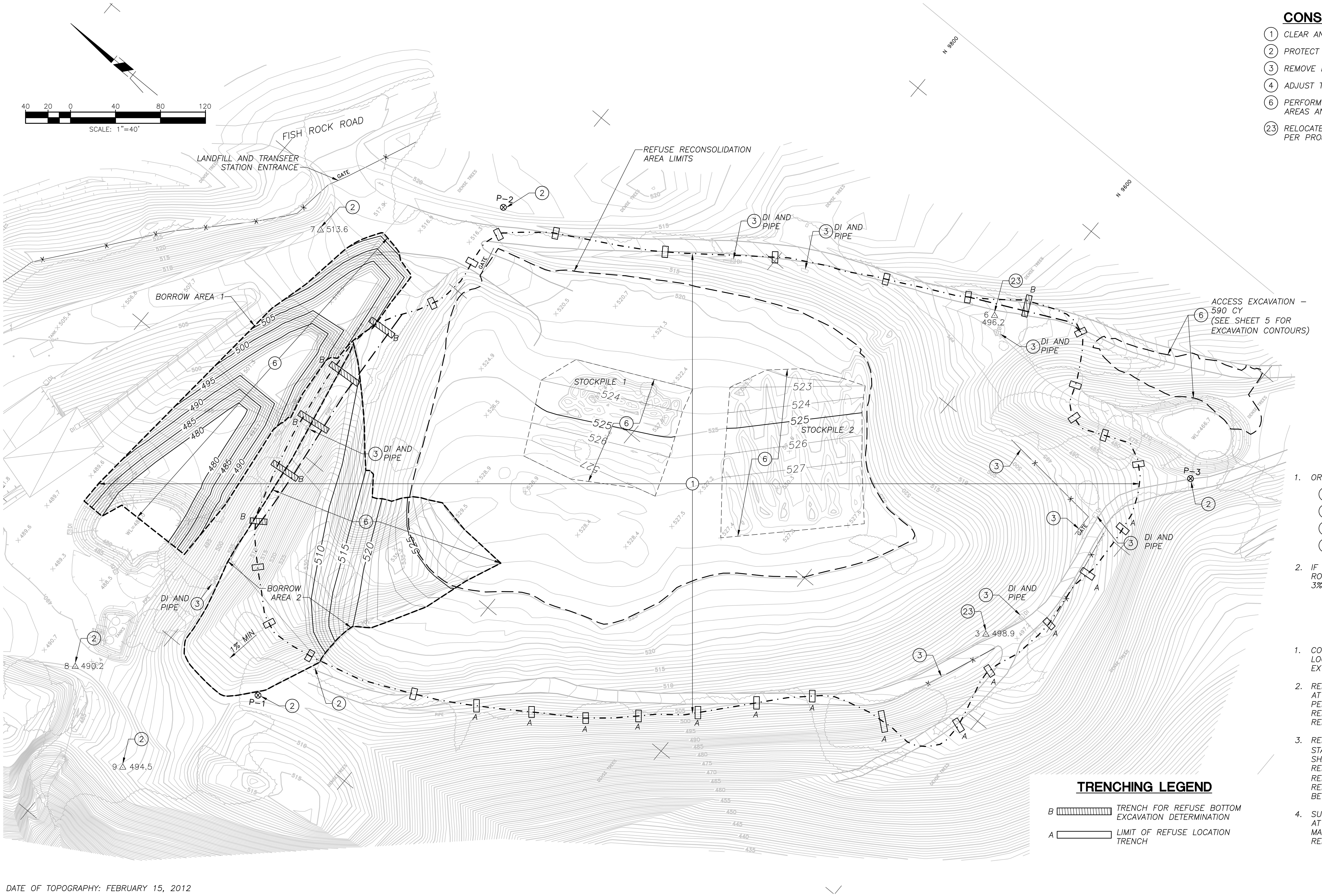
MICHAEL A. CULLINANE
No. 41981

PREPARED UNDER THE SUPERVISION OF ______ DATE

MENDOCINO COUNTY
SOUTH COAST LANDFILL

FINAL CLOSURE CONSTRUCTION PROJECT 2016
SITE PLAN

DESIGNED BY : RMG	SCALE : AS SHOWN	PROJECT NO: –
DRAWN BY : CTD/DMW	DATE : 11-2016	
CHECKED BY :	DATE :	SHEET 2 OF 12
APPROVED BY :	DATE :	



DATE OF TOPOGRAPHY: FEBRUARY 15, 2012

CONSTRUCTION NOTES

① CLEAR AND GRUB

② PROTECT IN PLACE

③ REMOVE EXISTING STRUCTURES

④ ADJUST TO FINAL GRADE

⑥ PERFORM UNCLASSIFIED EXCAVATION AT STOCKPILE AREAS AND BORROW AREA FOR FINAL COVER

㉓ RELOCATE/ADJUST SURVEY MONUMENT TO GRADE PER PROJECT SPECIFICATIONS AND 2/D3

BORROW NOTES

1. ORDER OF BORROW AS FOLLOWS:
   - Ⓐ STOCKPILE 1 AND 2
   - Ⓑ BORROW AREA 1
   - Ⓒ ACCESS EXCAVATION
   - Ⓓ BORROW AREA 2 (SEE NOTE 2)
2. IF NOT ALL OF BORROW AREA 2 IS NEEDED, ROUND OFF SLOPE 2:1 MAX AND FLAT AREAS AT 3% OVER REFUSE; 1% OUTSIDE WASTE AREAS.

TRENCHING NOTES

1. CONTRACTOR'S SURVEYOR SHALL STAKE TRENCH LOCATIONS BASED ON A 5-FOOT OFFSET EXTERNAL TO THE APPROXIMATE REFUSE LIMIT.
2. REFUSE LIMIT VERIFICATION TRENCH TO START AT THE 5-FOOT OFFSET AND PROGRESS PERPENDICULAR TO REFUSE LIMIT INWARD UNTIL REFUSE LIMIT IS LOCATED; SURVEY LOCATION OF REFUSE LIMIT AND INTERIM COVER THICKNESS.
3. REFUSE BOTTOM LOCATION TRENCHING SHALL START SIMILAR TO (3) ABOVE; CONTRACTOR SHALL TRENCH INWARD FROM ESTABLISHED REFUSE LIMIT AND SHALL TRENCH THROUGH REFUSE TO 5-FOOT BEYOND INNER (PROPOSED) REFUSE LIMIT SO THAT A BOTTOM PROFILE CAN BE ESTABLISHED.
4. SURVEY OF REFUSE LIMIT AND BOTTOM PROFILE AT THE BEGINNING, END AND AT 10-FOOT MAXIMUM HORIZONTAL SPACING WITHIN THE REFUSE TRENCH SHALL BE PERFORMED.

TRENCHING LEGEND

B ▨	TRENCH FOR REFUSE BOTTOM EXCAVATION DETERMINATION
A ▭	LIMIT OF REFUSE LOCATION TRENCH

NOTE:
1. SEE SHEET NO. 2 FOR LEGEND

NO.	REVISION DESCRIPTION	DATE

PREPARED BY:

SWT Civil & Environmental **Engineering**
800-C SOUTH ROCHESTER AVENUE
ONTARIO, CALIFORNIA 91761

MICHAEL A. CULLINANE
No. 41981
CIVIL

PREPARED UNDER THE SUPERVISION OF ______ DATE

MENDOCINO COUNTY
SOUTH COAST LANDFILL

FINAL CLOSURE CONSTRUCTION PROJECT 2016
REFUSE LIMIT VERIFICATION/TRENCHING/SOIL BORROW PLAN

DESIGNED BY : RMG	SCALE : AS SHOWN	PROJECT NO: –
DRAWN BY : CTD/DMW	DATE : 11-2016	
CHECKED BY :	DATE :	SHEET 3 OF 12
APPROVED BY :	DATE :	

Z:\PROJECTS\MENDOCINO COUNTY\SOUTH COAST\FINAL CLOSURE CONSTRUCTION 2016\ACAD\SHEET SET\03-SOC STOCKPILE REMOVAL AND BORROW PLAN

REFUSE RECONSOLIDATION AREA NOTES

1. CONTRACTOR SHALL START REFUSE PLACEMENT AT SLOPE DAYLIGHT LOCATION PER PLAN. REFUSE SHALL BE PLACED IN 5-FOOT MAXIMUM LIFTS. SHOULD DECK GRADES REQUIRE ADJUSTMENT (UP OR DOWN) THE DECK ELEVATION SHALL BE ADJUSTED UNIFORMLY, MAINTAINING GRADIENT.
2. CONTRACTOR SHALL COVER REFUSE WITH 12" INTERIM COVER SOIL ONCE FINAL COVER SUBGRADE IS REACHED (OR ADJUSTED).
3. CONTRACTOR SHALL PLACE FINAL COVER IN RECONSOLIDATION AREA ONLY AFTER ALL REFUSE REMOVAL, RECONSOLIDATION AND 12" INTERIM COVER SOIL HAS BEEN COMPLETED.

NOTE:
1. SEE SHEET NO. 2 FOR LEGEND

CONSTRUCTION NOTES

② PROTECT IN PLACE

⑤ PERFORM REFUSE EXCAVATION AND PLACE IN DESIGNATED RECONSOLIDATION AREA SHOWN PER PLAN

⑩ CONSTRUCT DRAINAGE COLLECTOR 4/D1

⑫ PERFORM 2-FOOT (PLUS INTERIM SOIL COVER) REFUSE EXCAVATION PRIOR TO PLACEMENT OF INTERIM COVER AND 2-FOOT FOUNDATION SOIL LAYER; PERFORM 10-FOOT TRANSITION FROM 2-FOOT VERTICAL CUT TO DAYLIGHT WITH EXISTING GRADE (REFUSE EXCAVATION PLUS INTERIM COVER) 4/D2

⑱ CONSTRUCT DECK ACCESS ROAD 9/D1

㉓ RELOCATE/ADJUST SURVEY MONUMENT TO GRADE PER PROJECT SPECIFICATIONS AND 2/D3

DATE OF TOPOGRAPHY: FEBRUARY 15, 2012

NO.	REVISION DESCRIPTION	DATE

PREPARED BY:

SWT Civil & Environmental **Engineering**
800-C SOUTH ROCHESTER AVENUE
ONTARIO, CALIFORNIA 91761

MICHAEL A. CULLINANE No. 41981 CIVIL

PREPARED UNDER THE SUPERVISION OF DATE

MENDOCINO COUNTY
SOUTH COAST LANDFILL

FINAL CLOSURE CONSTRUCTION PROJECT 2016
REFUSE RECONSOLIDATION PLAN

DESIGNED BY : RMG	SCALE : AS SHOWN	PROJECT NO: –
DRAWN BY : CTD/DMW	DATE : 11-2016	
CHECKED BY :	DATE :	SHEET 4 OF 12
APPROVED BY :	DATE :	

Z:\PROJECTS\MENDOCINO COUNTY\SOUTH COAST\FINAL CLOSURE CONSTRUCTION 2016\ACAD\SHEET SET\04-SOC REFUSE RECONSOLIDATION PLAN

CONSTRUCTION NOTES

- ① CLEAR AND GRUB
- ② PROTECT IN PLACE
- ④ ADJUST TO FINAL GRADE
- ⑦ CONSTRUCT FINAL COVER SECTION – SLOPE 1/D1
- ⑧ CONSTRUCT FINAL COVER SECTION – DECK 2/D1
- ⑨ CONSTRUCT HDPE EDGE DRAIN COLLECTOR (BELOW FINAL COVER LLDPE) 3/D1
- ⑪ CONSTRUCT FINAL COVER SLOPE TERMINATION 5/D1 (WITH SWALE), OR 11/D1 (NO SWALE)
- ⑬ CONSTRUCT BENCH LINER TERMINATION 6/D3
- ⑭ CONSTRUCT FINAL COVER TERMINATION – NORTHEAST PERIMETER ROAD 6/D1
- ⑯ CONSTRUCT TOP DECK TRANSITION 8/D1
- ⑰ CONSTRUCT FINAL COVER TERMINATION – SOUTH AND WEST PERIMETER ROAD 4/D2
- ⑱ CONSTRUCT DECK ACCESS ROAD 9/D1
- ⑳ CONSTRUCT CONCRETE DOWNDRAIN/CHANNEL 1/D3
- ㉑ CONSTRUCT RIP RAP PAD 1/D2
- ㉒ CONSTRUCT BASIN BERM AND DOWNDRAIN INLET AT STORMWATER BASIN 3/D3
- ㉓ RELOCATE/ADJUST SURVEY MONUMENT TO GRADE PER PROJECT SPECIFICATIONS AND 2/D3
- ㉔ CONSTRUCT HDPE BOOT 7/D1
- ㉕ CONSTRUCT BENTONITE PLUG 7/D1
- ㉖ CONSTRUCT 4" SOLID HDPE PIPE (SDR 17)
- ㉗ EXTEND PIPE AND JOIN TO EXISTING LEACHATE OUTFALL PIPING/TANK
- ㉙ CONSTRUCT TYPE A AC DIKE PER CALTRANS STD PLAN A87B
- ㉛ CONSTRUCT 3" AC OVER NATIVE SOIL COMPACTED TO 95% OF ASTM D1557 6/D1 OR 9/D1 OR 4/D2
- ㉜ CONSTRUCT SPLASHWALL 5/D3
- ㊷ CONSTRUCT INLET TO CONCRETE DOWNDRAIN 7/D3
- ㊹ JOIN 4" HDPE PIPE TO HDPE EDGE DRAIN WITH EDGE DRAIN TO PIPE TRANSITION FITTING

NOTE:
1. SEE SHEET NO. 2 FOR LEGEND

DATE OF TOPOGRAPHY: FEBRUARY 15, 2012

NO.	REVISION DESCRIPTION	DATE

PREPARED BY:

SWT Civil & Environmental **Engineering**
800–C SOUTH ROCHESTER AVENUE
ONTARIO, CALIFORNIA 91761

REGISTERED PROFESSIONAL ENGINEER MICHAEL A. CULLINANE No. 41981 CIVIL STATE OF CALIFORNIA

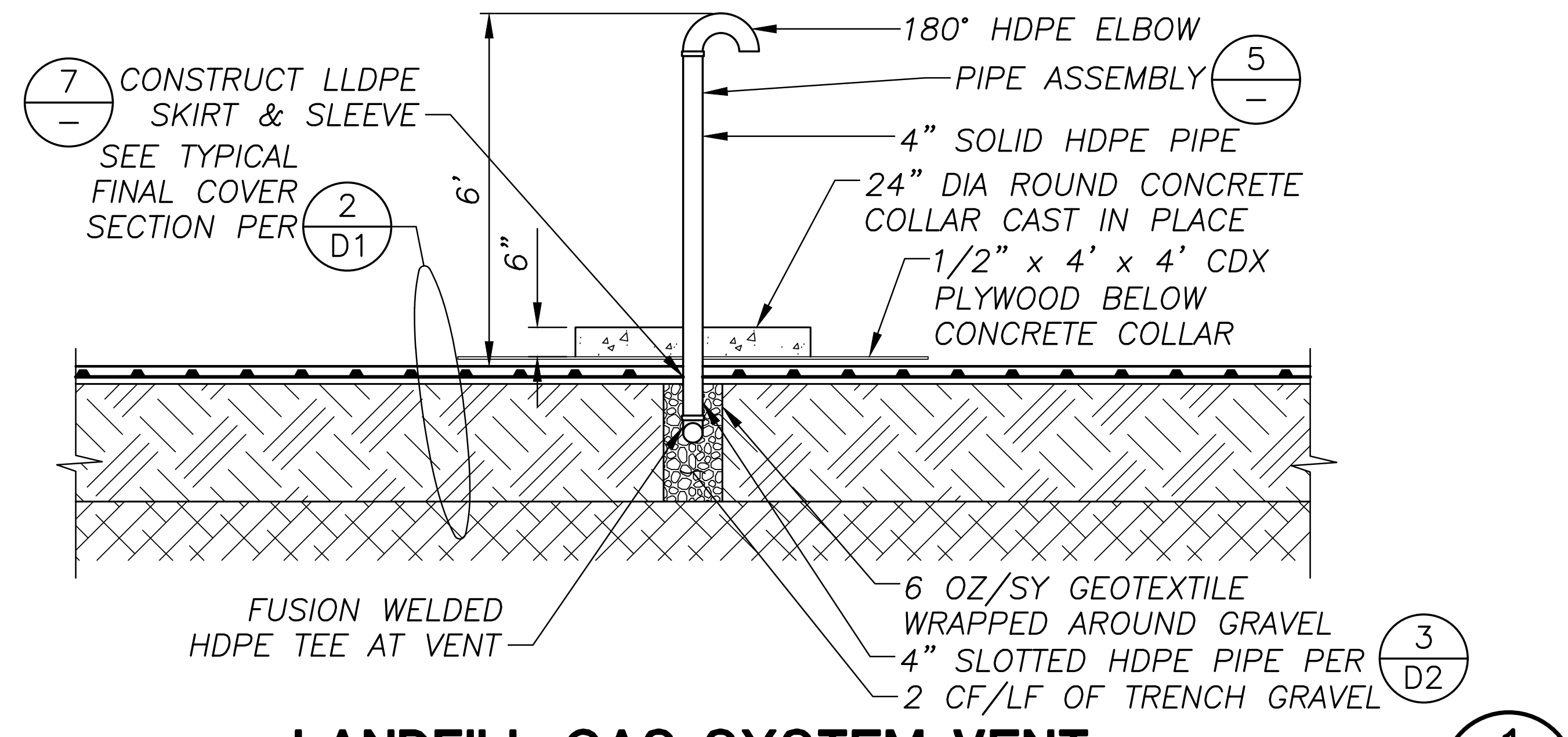
PREPARED UNDER THE SUPERVISION OF ____ DATE

MENDOCINO COUNTY
SOUTH COAST LANDFILL

FINAL CLOSURE CONSTRUCTION PROJECT 2016
FINAL GRADING PLAN

DESIGNED BY : RMG	SCALE : AS SHOWN	PROJECT NO: –
DRAWN BY : CTD/DMW	DATE : 11–2016	
CHECKED BY :	DATE :	SHEET 5 OF 12
APPROVED BY :	DATE :	

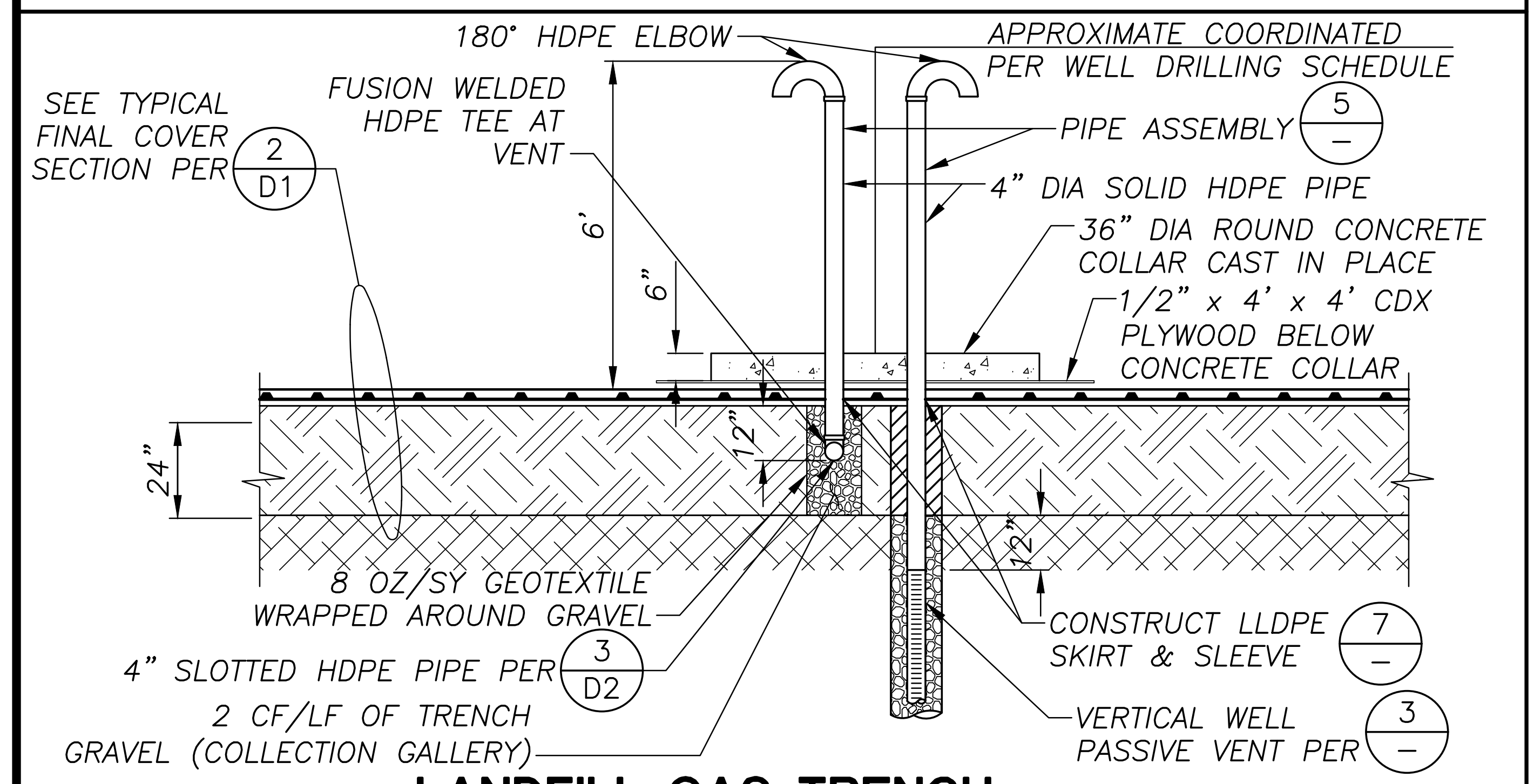
Z:\PROJECTS\MENDOCINO COUNTY\SOUTH COAST\FINAL CLOSURE CONSTRUCTION 2016\ACAD\SHEET SET\05–SOC FINAL GRADING PLAN



LANDFILL GAS SYSTEM VENT

NTS

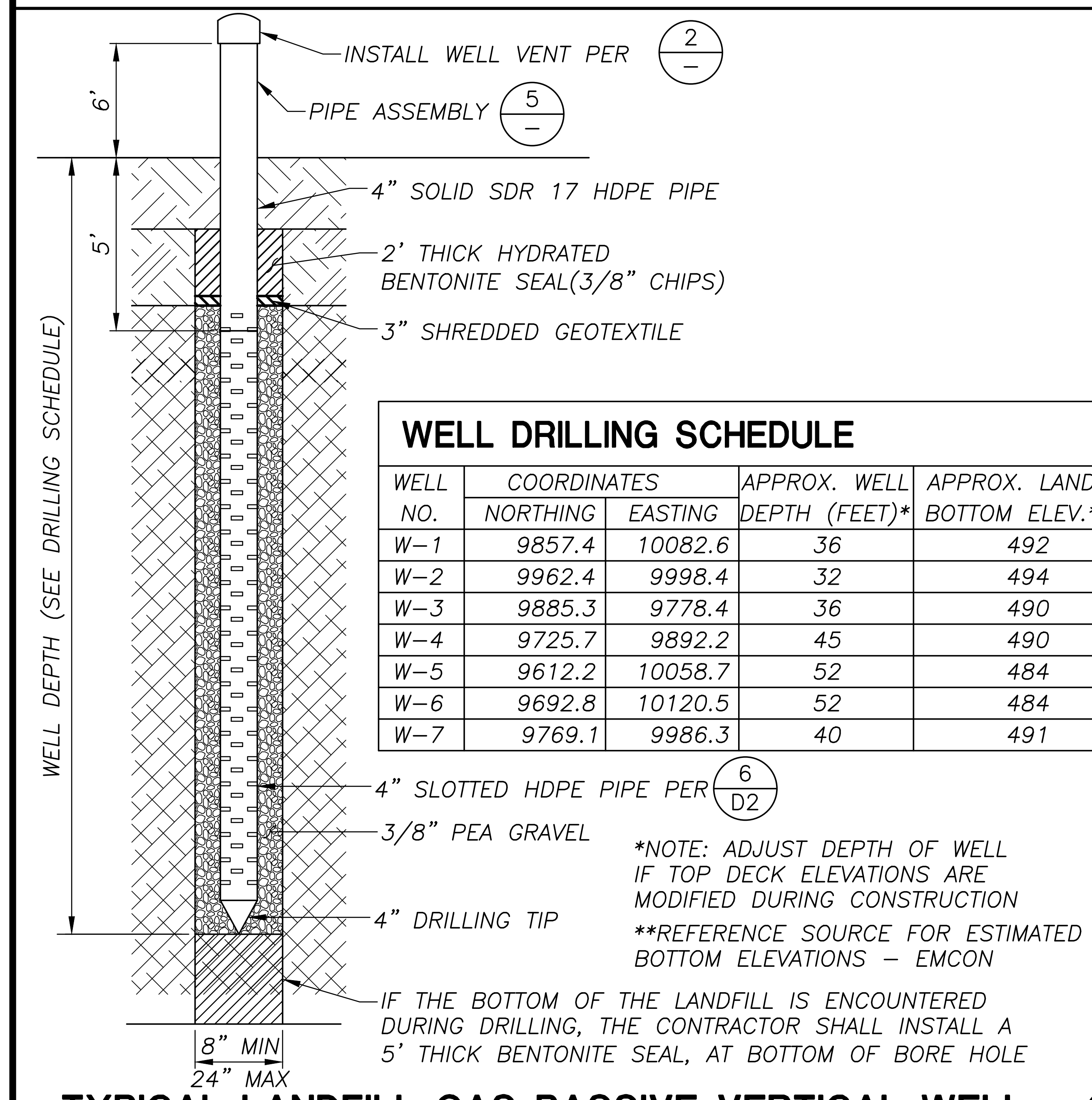
1/6



LANDFILL GAS TRENCH COLLECTION GALLERY/WELL/VENT

NTS

2/6



WELL DRILLING SCHEDULE

WELL NO.	COORDINATES		APPROX. WELL DEPTH (FEET)*	APPROX. LANDFILL BOTTOM ELEV.**
	NORTHING	EASTING		
W−1	9857.4	10082.6	36	492
W−2	9962.4	9998.4	32	494
W−3	9885.3	9778.4	36	490
W−4	9725.7	9892.2	45	490
W−5	9612.2	10058.7	52	484
W−6	9692.8	10120.5	52	484
W−7	9769.1	9986.3	40	491

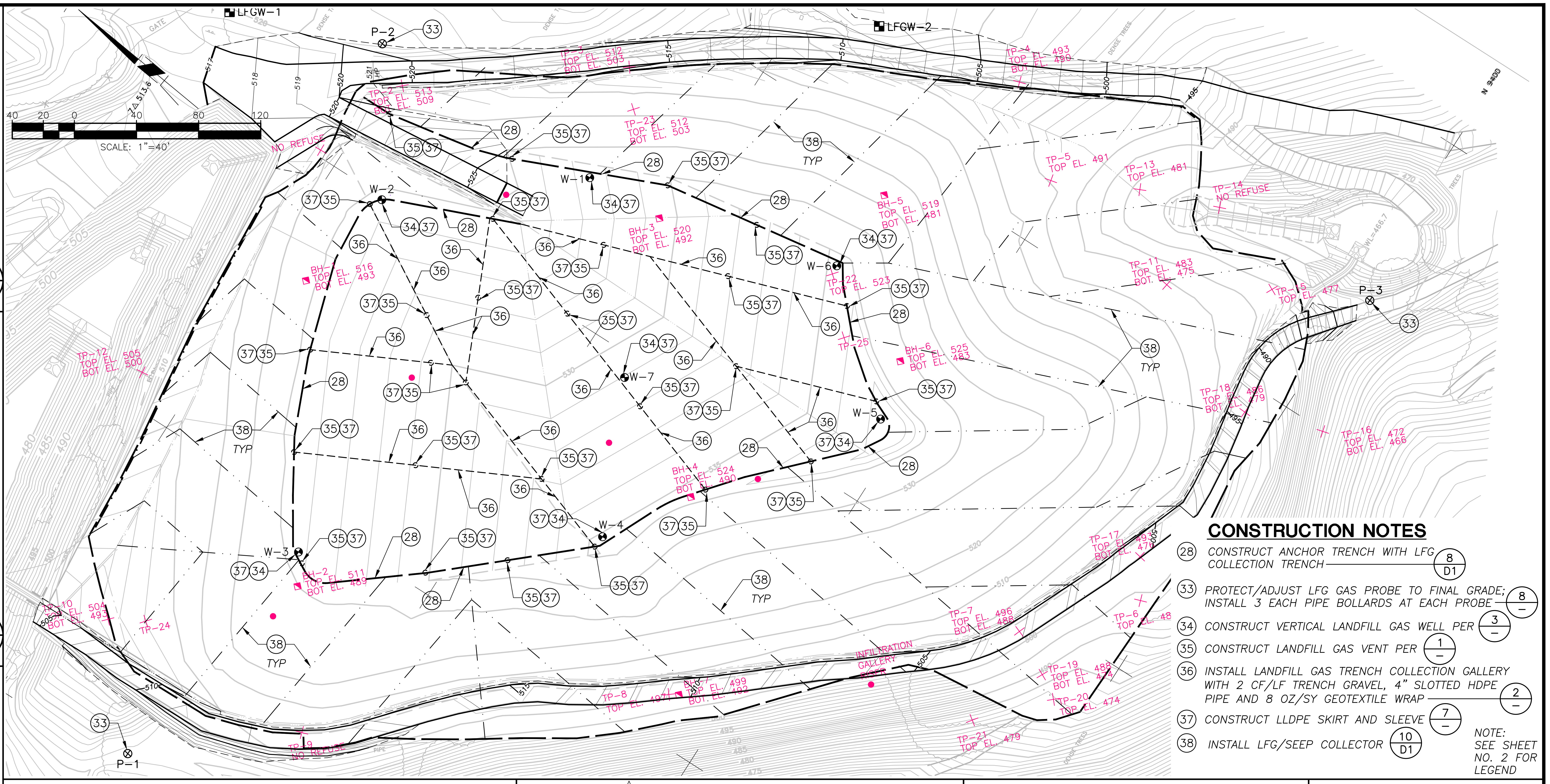
*NOTE: ADJUST DEPTH OF WELL IF TOP DECK ELEVATIONS ARE MODIFIED DURING CONSTRUCTION

**REFERENCE SOURCE FOR ESTIMATED BOTTOM ELEVATIONS – EMCON

TYPICAL LANDFILL GAS PASSIVE VERTICAL WELL

NTS

3/6



CONSTRUCTION NOTES

㉘ CONSTRUCT ANCHOR TRENCH WITH LFG COLLECTION TRENCH 8/D1

㉝ PROTECT/ADJUST LFG GAS PROBE TO FINAL GRADE; INSTALL 3 EACH PIPE BOLLARDS AT EACH PROBE 8/—

㉞ CONSTRUCT VERTICAL LANDFILL GAS WELL PER 3/—

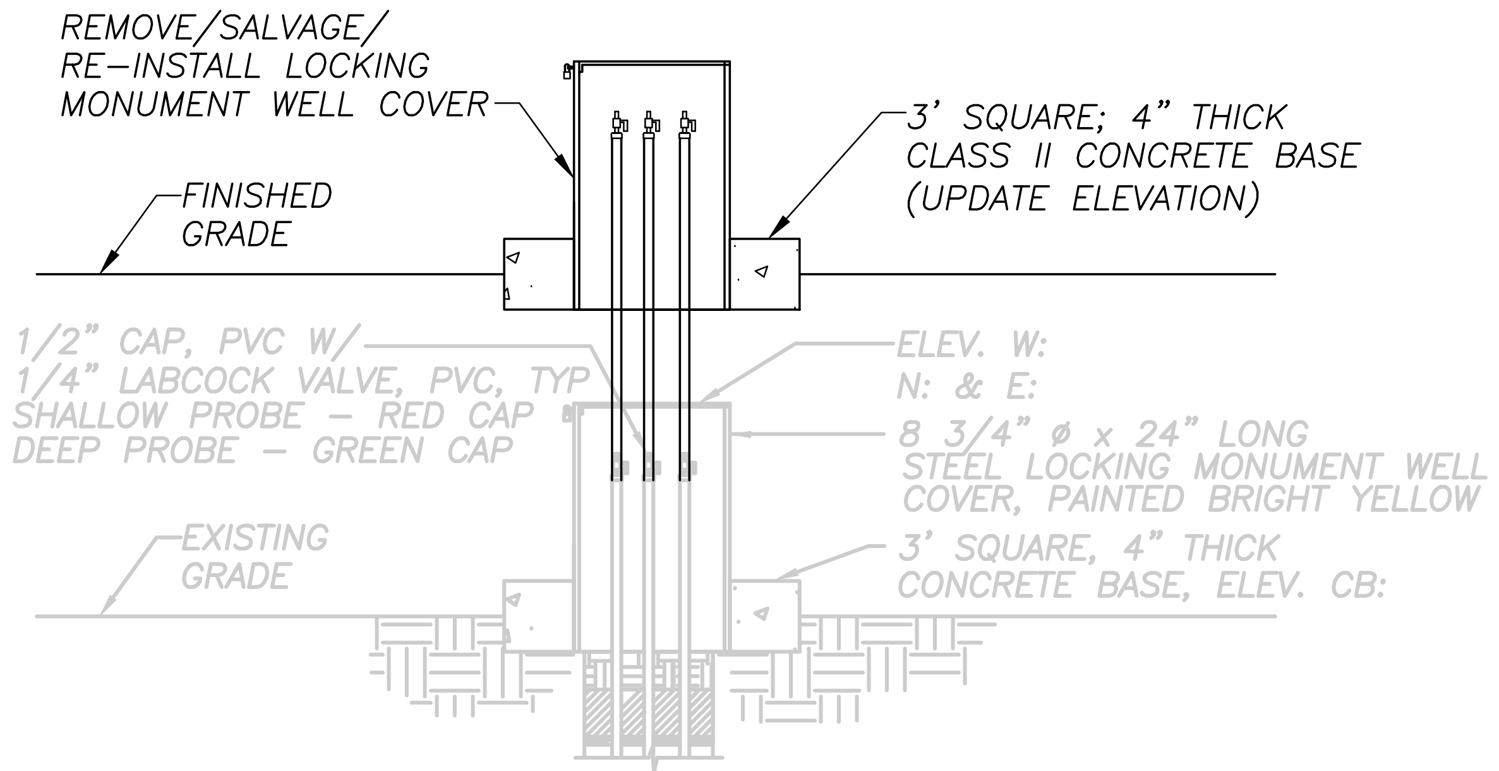
㉟ CONSTRUCT LANDFILL GAS VENT PER 1/—

㊱ INSTALL LANDFILL GAS TRENCH COLLECTION GALLERY WITH 2 CF/LF TRENCH GRAVEL, 4" SLOTTED HDPE PIPE AND 8 OZ/SY GEOTEXTILE WRAP 2/—

㊲ CONSTRUCT LLDPE SKIRT AND SLEEVE 7/—

㊳ INSTALL LFG/SEEP COLLECTOR 10/D1

NOTE:
SEE SHEET NO. 2 FOR LEGEND



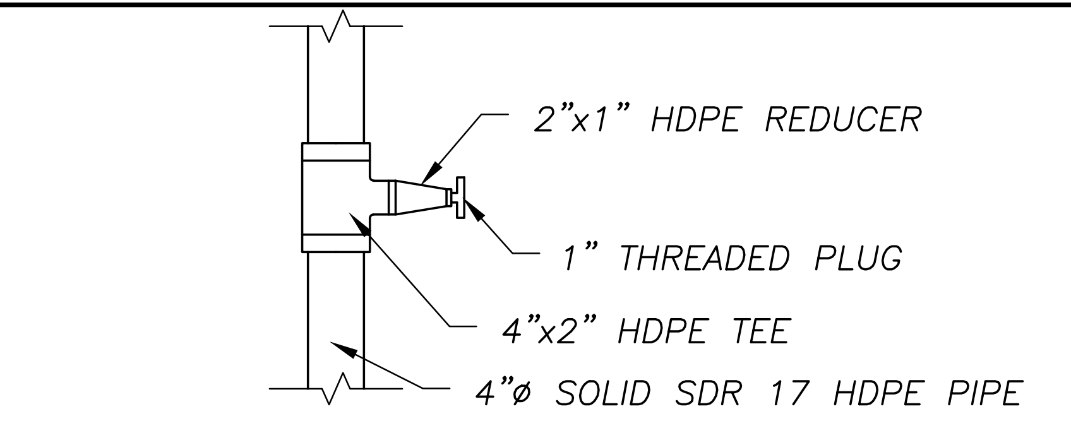
NOTES:

1. IF PROBE ELEVATION MATCHES FINAL GRADE CONSTRUCT PIPE BOLLARDS. 8/—
2. IF PROBE ELEVATION IS BELOW FINAL GRADE, REMOVE AND SALVAGE WELL COVER; PROTECT PROBES; FILL TO GRADE; EXTEND PVC PIPE (NO GLUE); REPLACE LID AND CONCRETE BASE – REVISE ELEVATION ON TAG.
3. INSTALL (3 EA) PIPE BOLLARDS AT EACH PROBE. 8/—

LANDFILL GAS MIGRATION MONITORING PROBE ADJUSTMENT TO GRADE

NTS

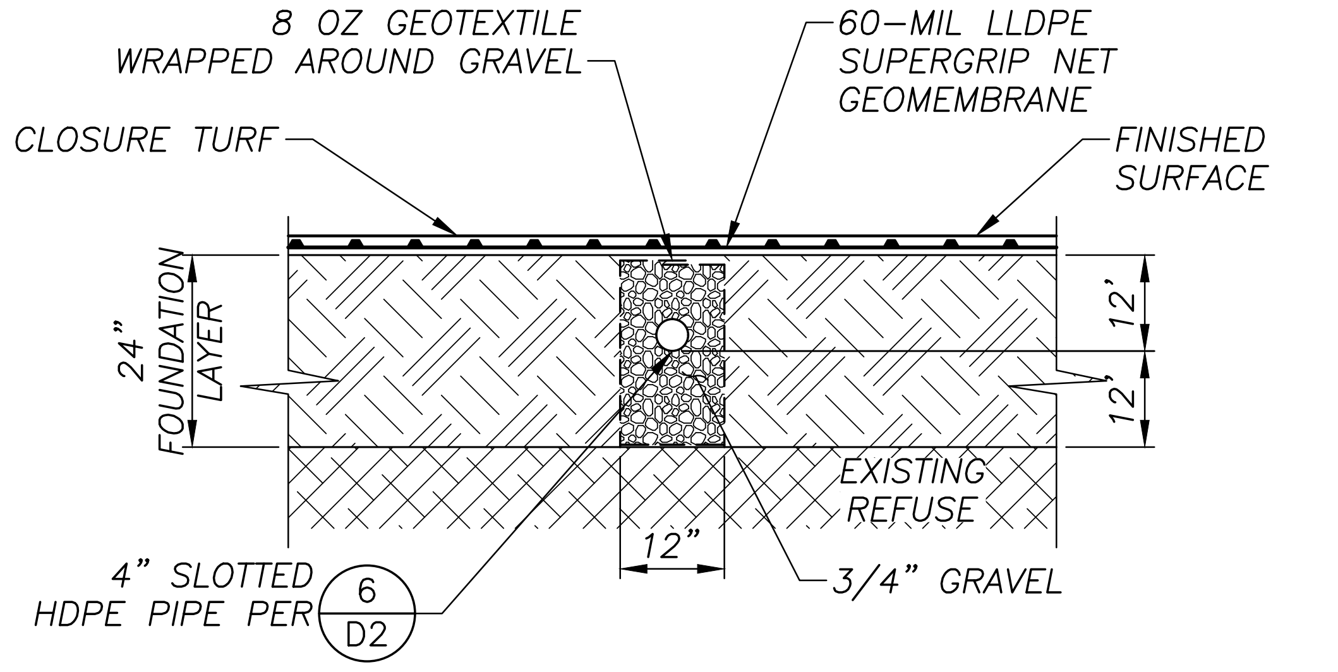
4/6



PIPE ASSEMBLY

NTS

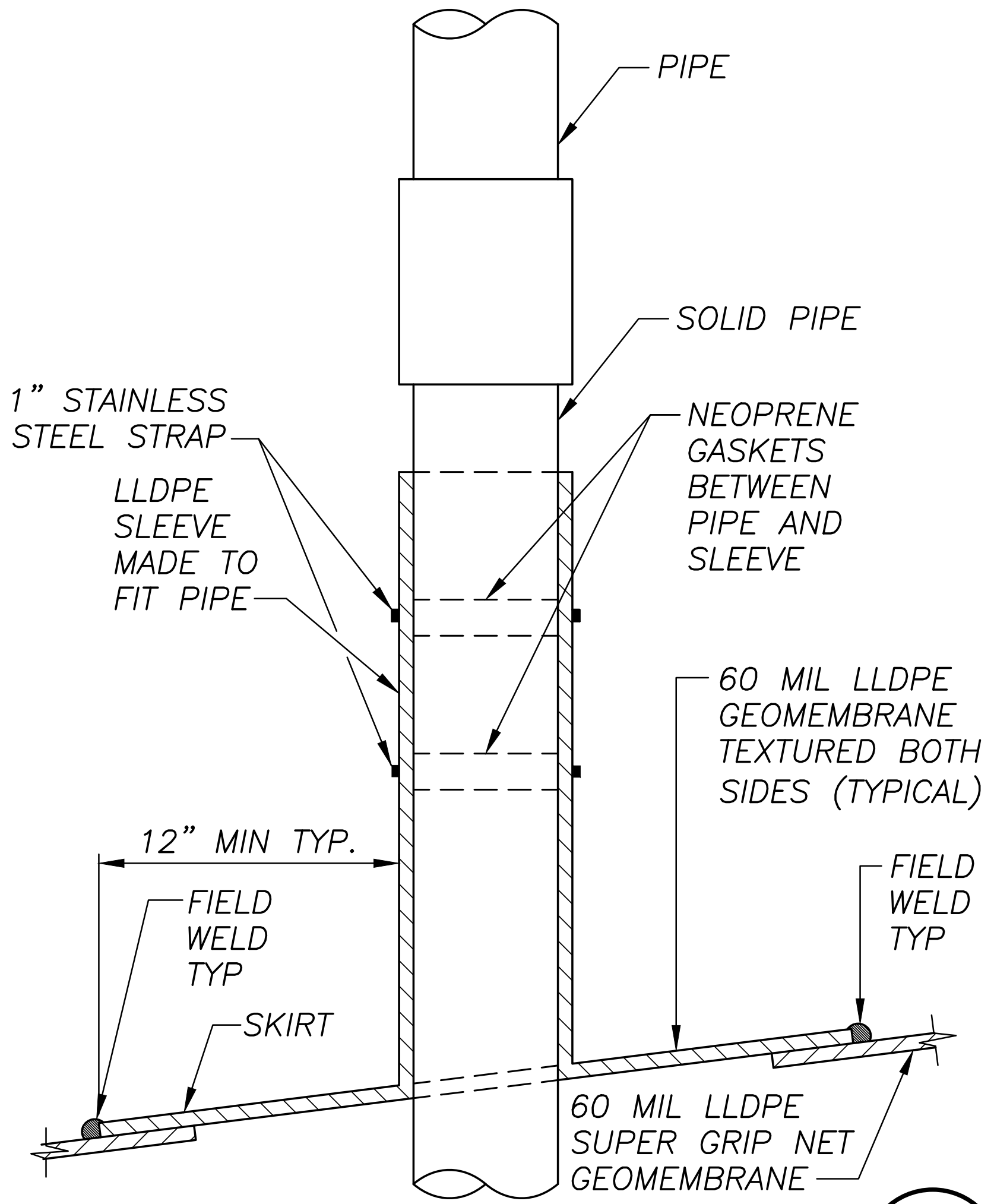
5/6



TYPICAL LANDFILL GAS PASSIVE TRENCH SECTION

NTS

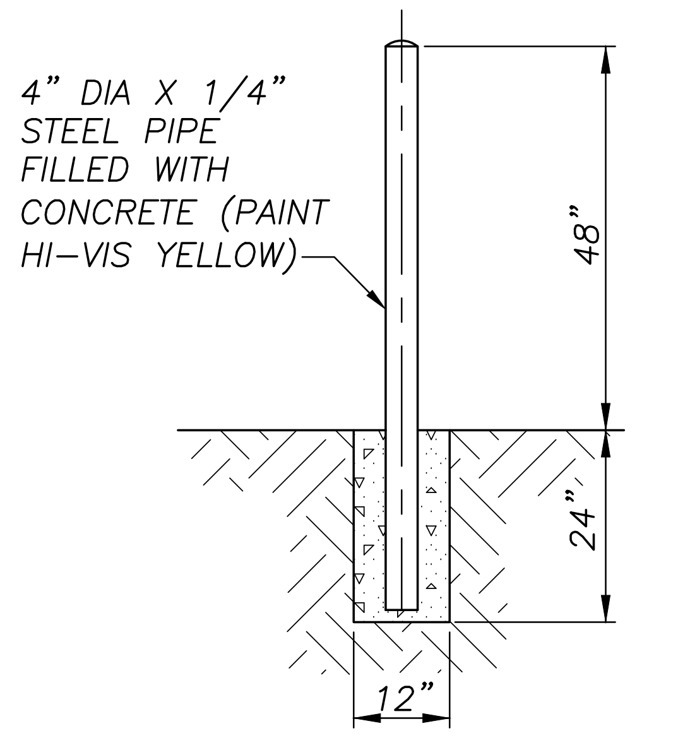
6/6



LLDPE SKIRT AND SLEEVE

NTS

7/6



PIPE BOLLARD

NTS

8/6

NO.	REVISION DESCRIPTION	DATE

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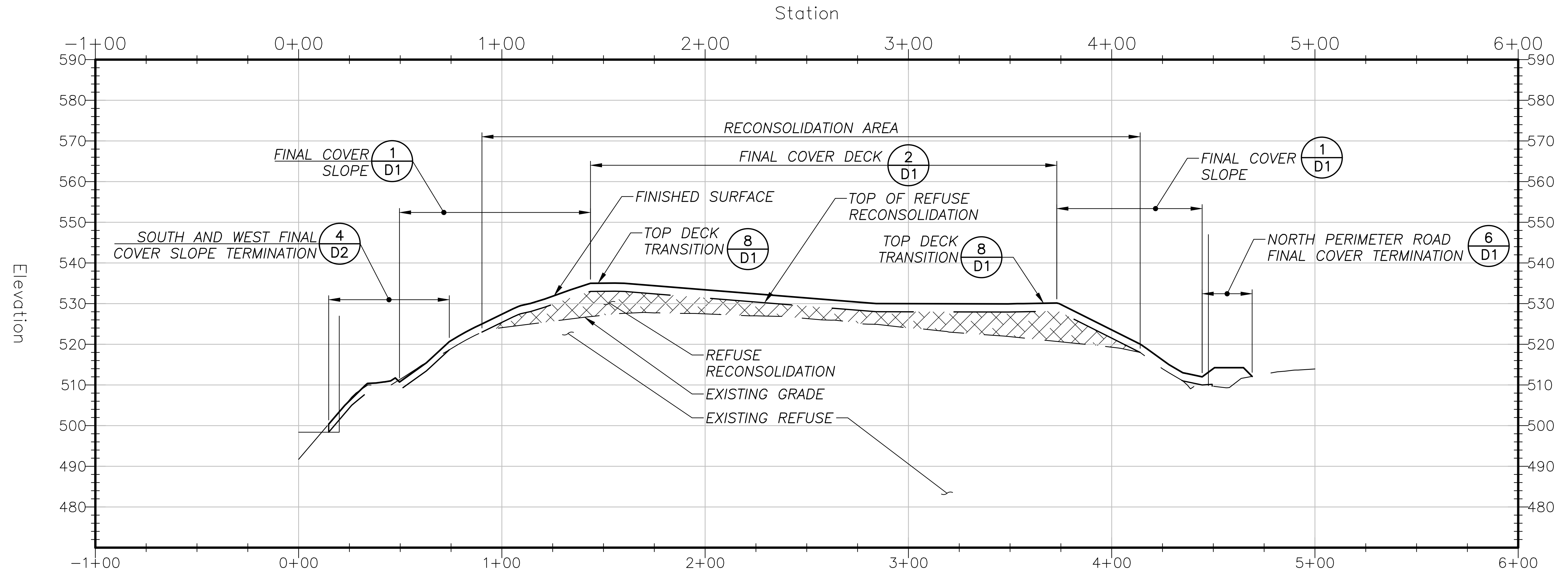
SWT Civil & Environmental Engineering
800−C SOUTH ROCHESTER AVENUE
ONTARIO, CALIFORNIA 91761
PREPARED UNDER THE SUPERVISION OF ________ DATE

MICHAEL A. CULLINANE
No. 41981

MENDOCINO COUNTY
SOUTH COAST LANDFILL

FINAL CLOSURE CONSTRUCTION PROJECT 2016
LANDFILL GAS PLAN

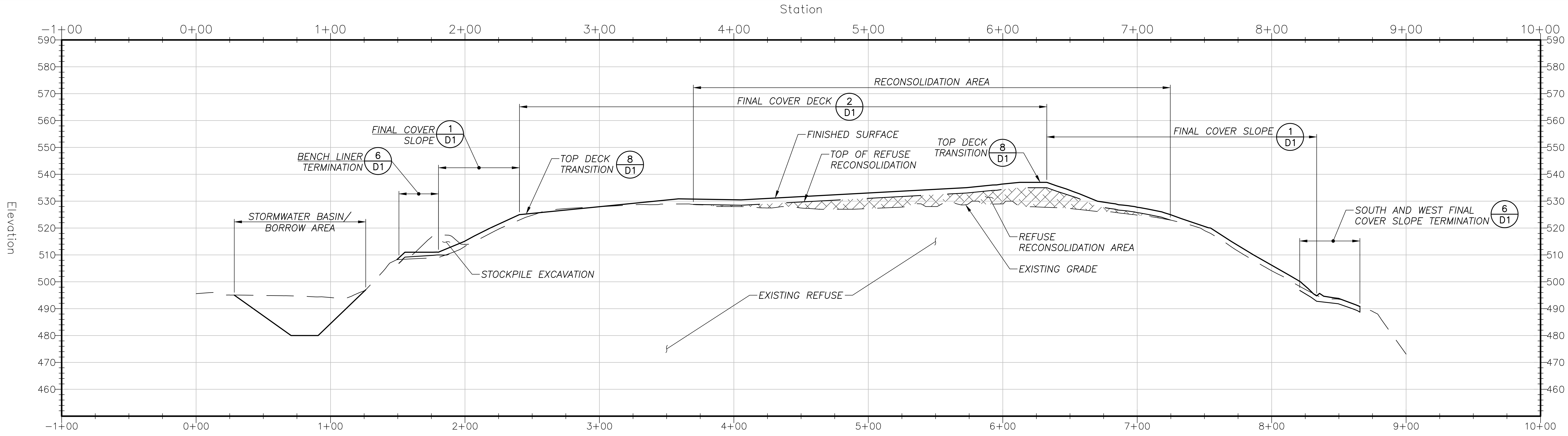
DESIGNED BY : RMG	SCALE : AS SHOWN	PROJECT NO: –
DRAWN BY : CTD/DMW	DATE : 11−2016	
CHECKED BY :	DATE :	SHEET 6 OF 12
APPROVED BY :	DATE :	



SECTION A

SCALE: HORIZ. 1"=40'
VERT. 1"=20'

A
8



SECTION B

SCALE: HORIZ. 1"=40'
VERT. 1"=20'

B
8

NO.	REVISION DESCRIPTION	DATE

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ONTARIO, CALIFORNIA 91761

PREPARED UNDER THE SUPERVISION OF

DATE

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MENDOCINO COUNTY
SOUTH COAST LANDFILL

FINAL CLOSURE CONSTRUCTION PROJECT 2016
SECTIONS

DESIGNED BY : RMG	SCALE : AS SHOWN	PROJECT NO: –
DRAWN BY : CTD/DMW	DATE : 11-2016	
CHECKED BY :	DATE :	SHEET 7 OF 12
APPROVED BY :	DATE :	

Z:\PROJECTS\MENDOCINO COUNTY\SOUTH COAST\FINAL CLOSURE CONSTRUCTION 2016\ACAD\SHEET SET\07-SOC SECTIONS

CONSTRUCTION NOTES

- ② PROTECT IN PLACE
- ⑮ CONSTRUCT CONCRETE ACCESS RAMP TO BASIN WITH INTEGRAL CONCRETE CURB — 4/D3
- ⑱ CONSTRUCT DECK ACCESS ROAD 9/D1
- ⑲ INSTALL PIPE BOLLARD 8/6
- ㉙ CONSTRUCT TYPE A AC DIKE PER CALTRANS STD PLAN A87B
- ㉚ CONSTRUCT 12"x12" DEEP LIFT AC/CONCRETE TERMINATION
- ㉛ CONSTRUCT 3" AC OVER NATIVE SOIL COMPACTED TO 95% OF ASTM D1557 6/D1 OR 9/D1 OR 4/D2
- ㊴ HYDROSEED ON SITE AREA
- ㊵ INSTALL FIBER ROLL PER 2/D4 3/D4
- ㊶ INSTALL GRAVEL BAG CHEVRON PER 1/D4
- ㊸ INSTALL CABLE GATE PER 5/D2

LEGEND

—FR—	FIBER ROLL
○○○○○	GRAVEL-BAG CHEVRON
▨	HYDROSEEDING
▩	AC PAVEMENT
▨	CONCRETE PAVEMENT

NO.	REVISION DESCRIPTION	DATE

PREPARED BY:

SWT Civil & Environmental **Engineering**
800-C SOUTH ROCHESTER AVENUE
ONTARIO, CALIFORNIA 91761

MICHAEL A. CULLINANE No. 41981

PREPARED UNDER THE SUPERVISION OF ________ DATE

MENDOCINO COUNTY
SOUTH COAST LANDFILL

FINAL CLOSURE CONSTRUCTION PROJECT 2016
PAVING AND EROSION CONTROL PLAN

DESIGNED BY : RMG	SCALE : AS SHOWN	PROJECT NO: –
DRAWN BY : CTD/DMW	DATE : 11-2016	
CHECKED BY :	DATE :	SHEET 8 OF 12
APPROVED BY :	DATE :	

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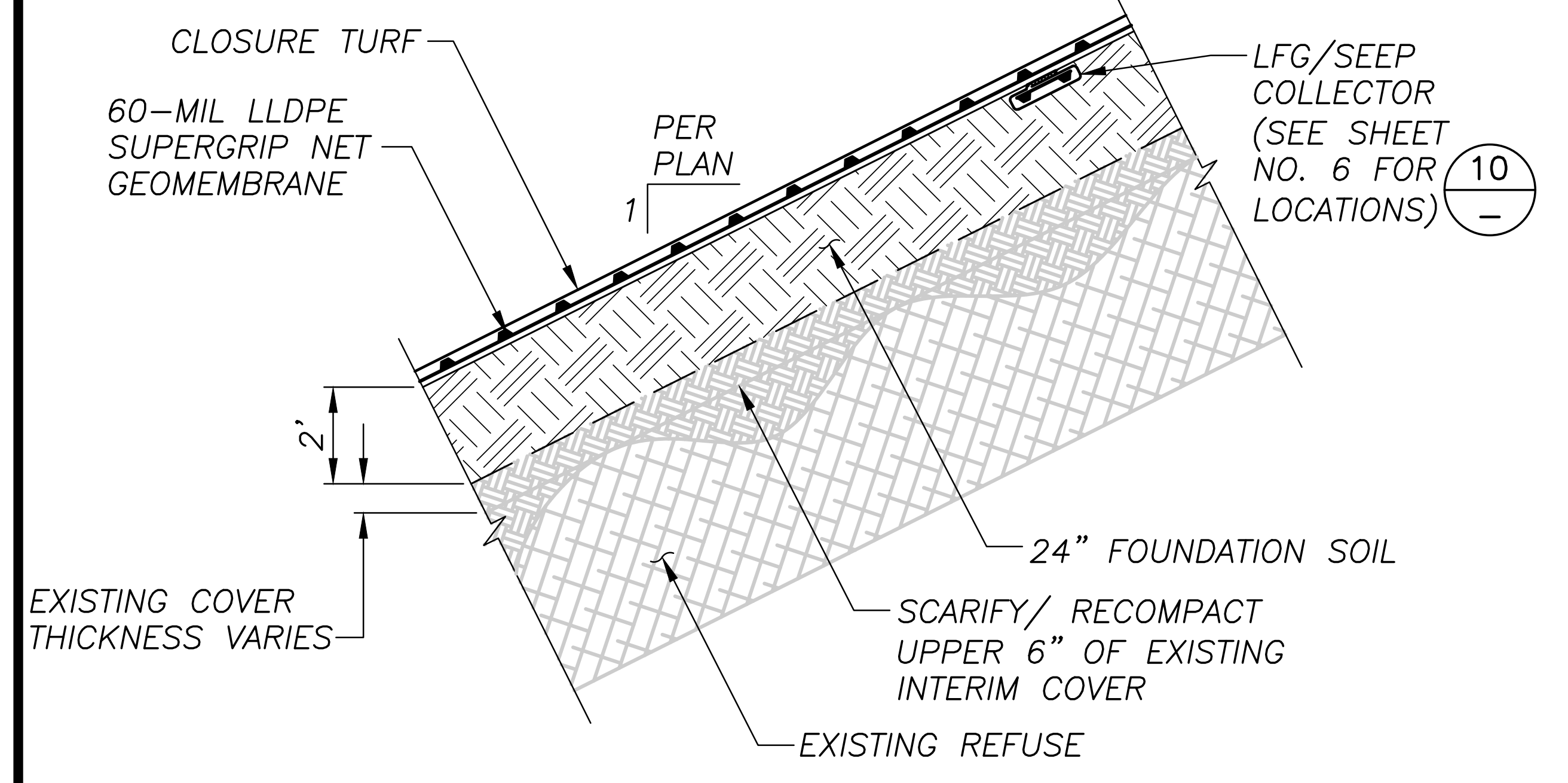
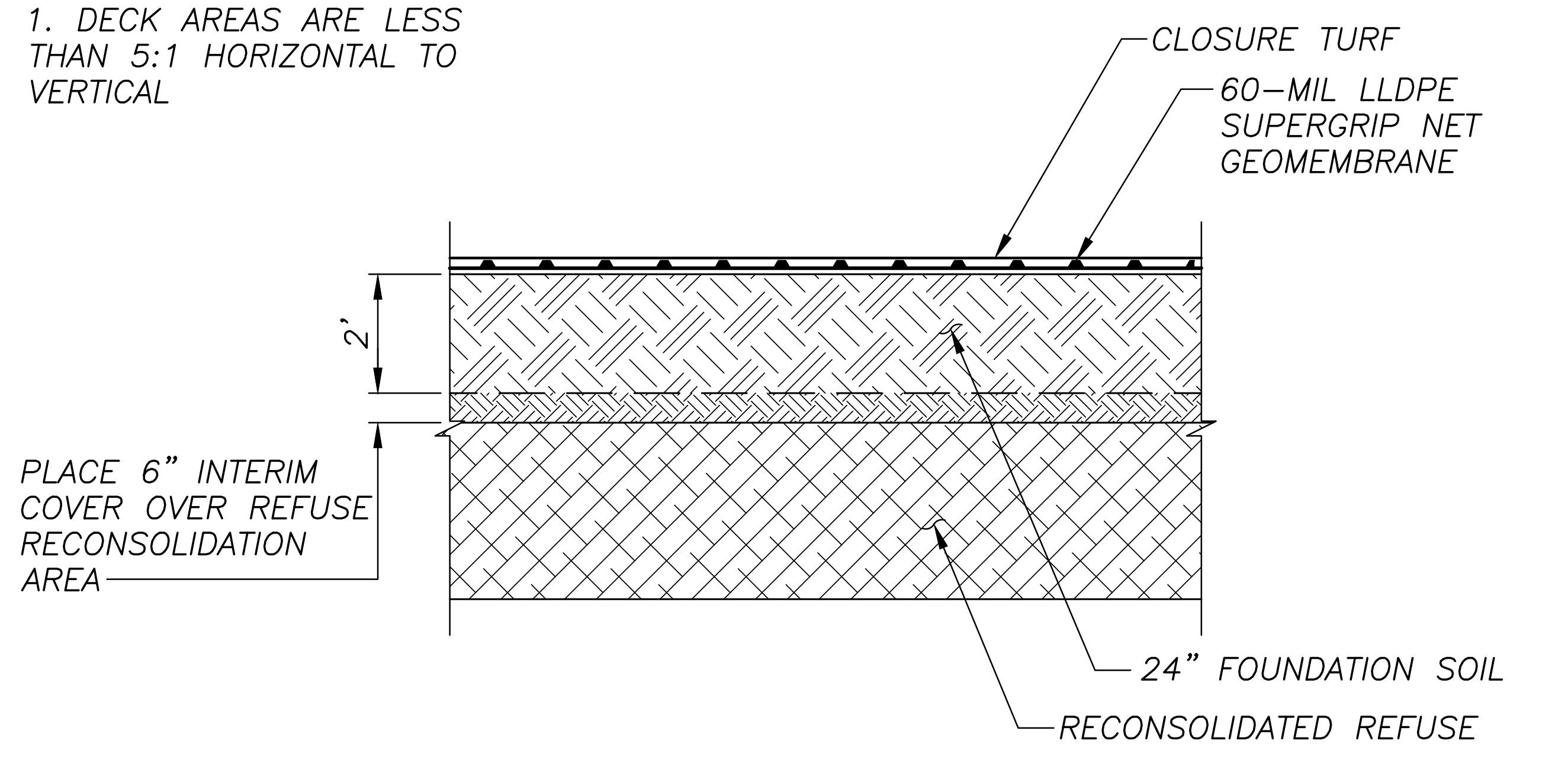


FINAL COVER SECTION - SLOPE

NTS

1/D1

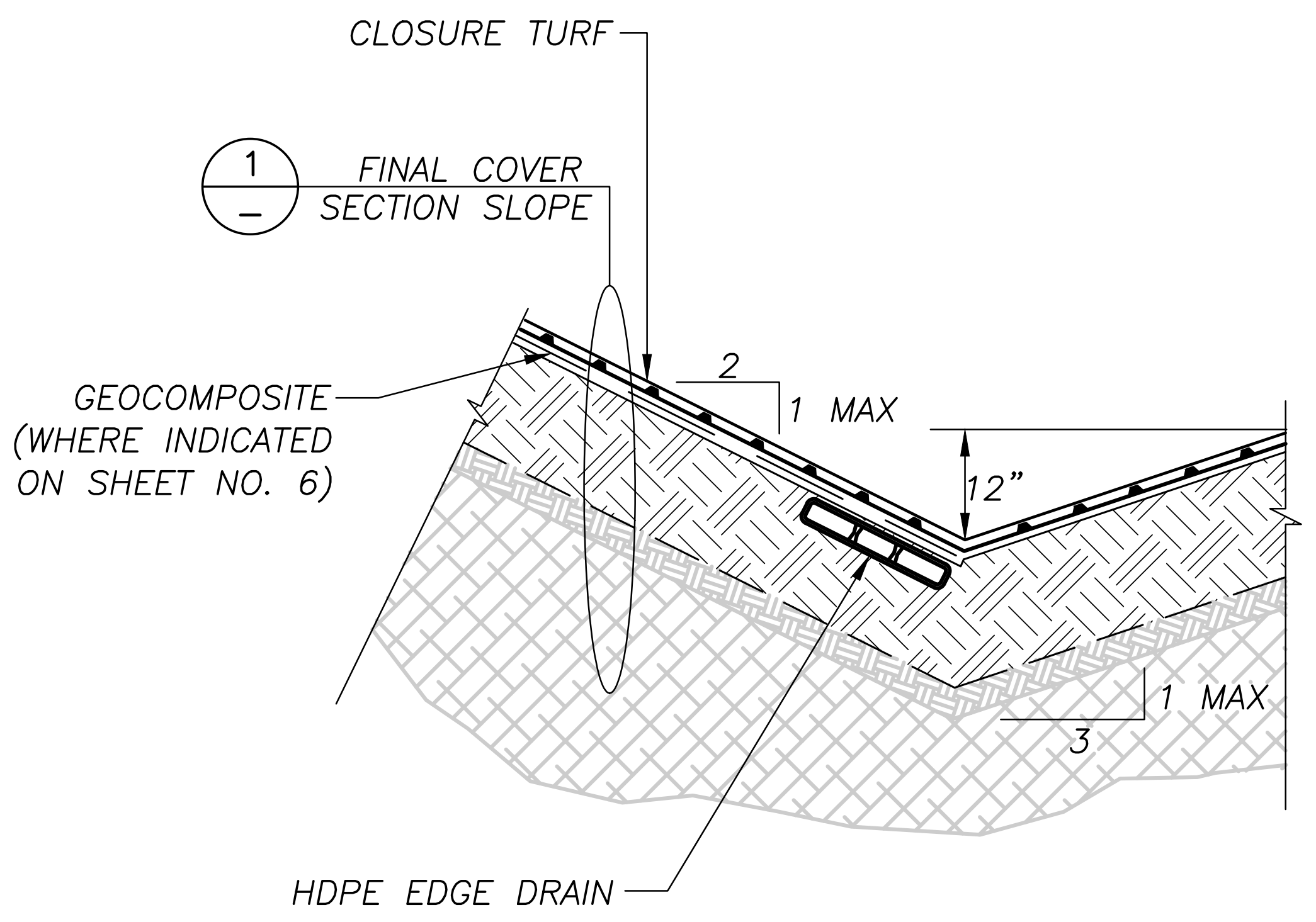
NOTE:
1. DECK AREAS ARE LESS THAN 5:1 HORIZONTAL TO VERTICAL



FINAL COVER SECTION - DECK

NTS

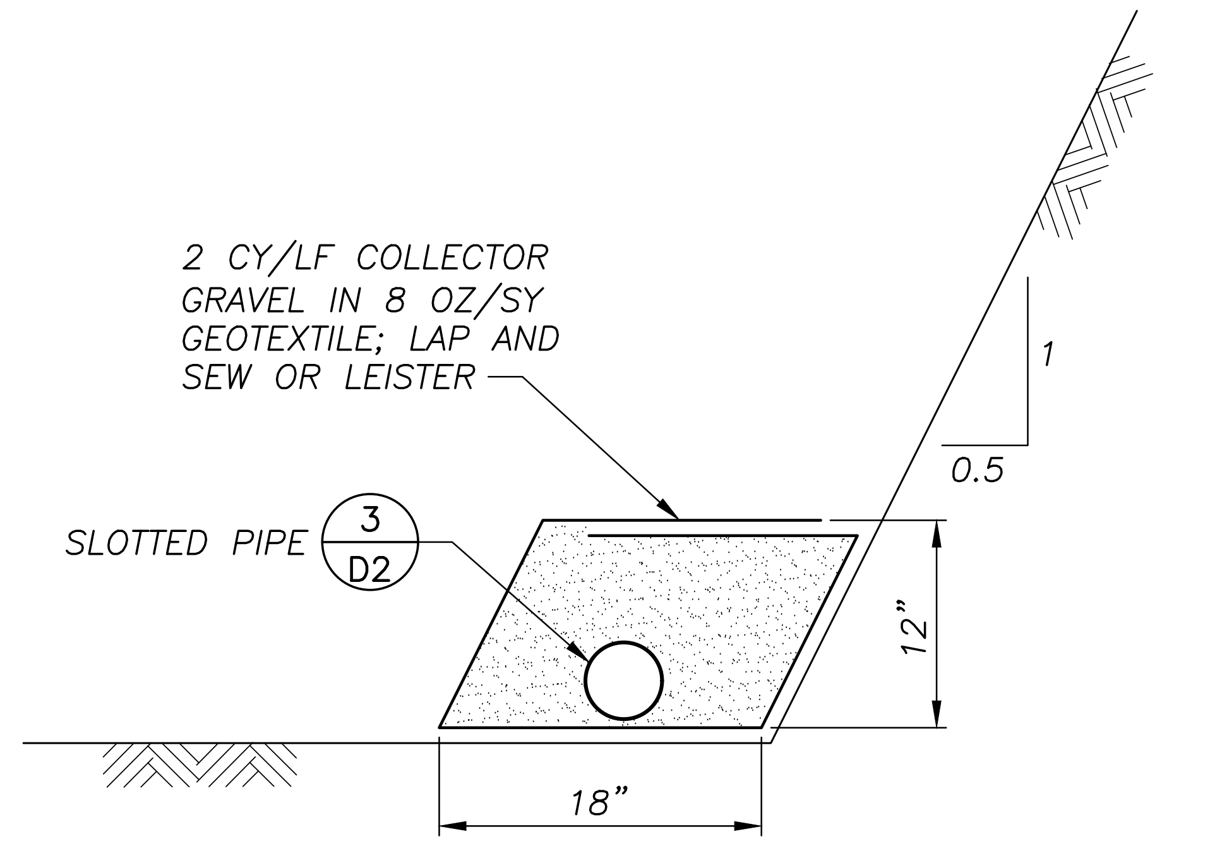
2/D1



HDPE EDGE DRAIN COLLECTOR

NTS

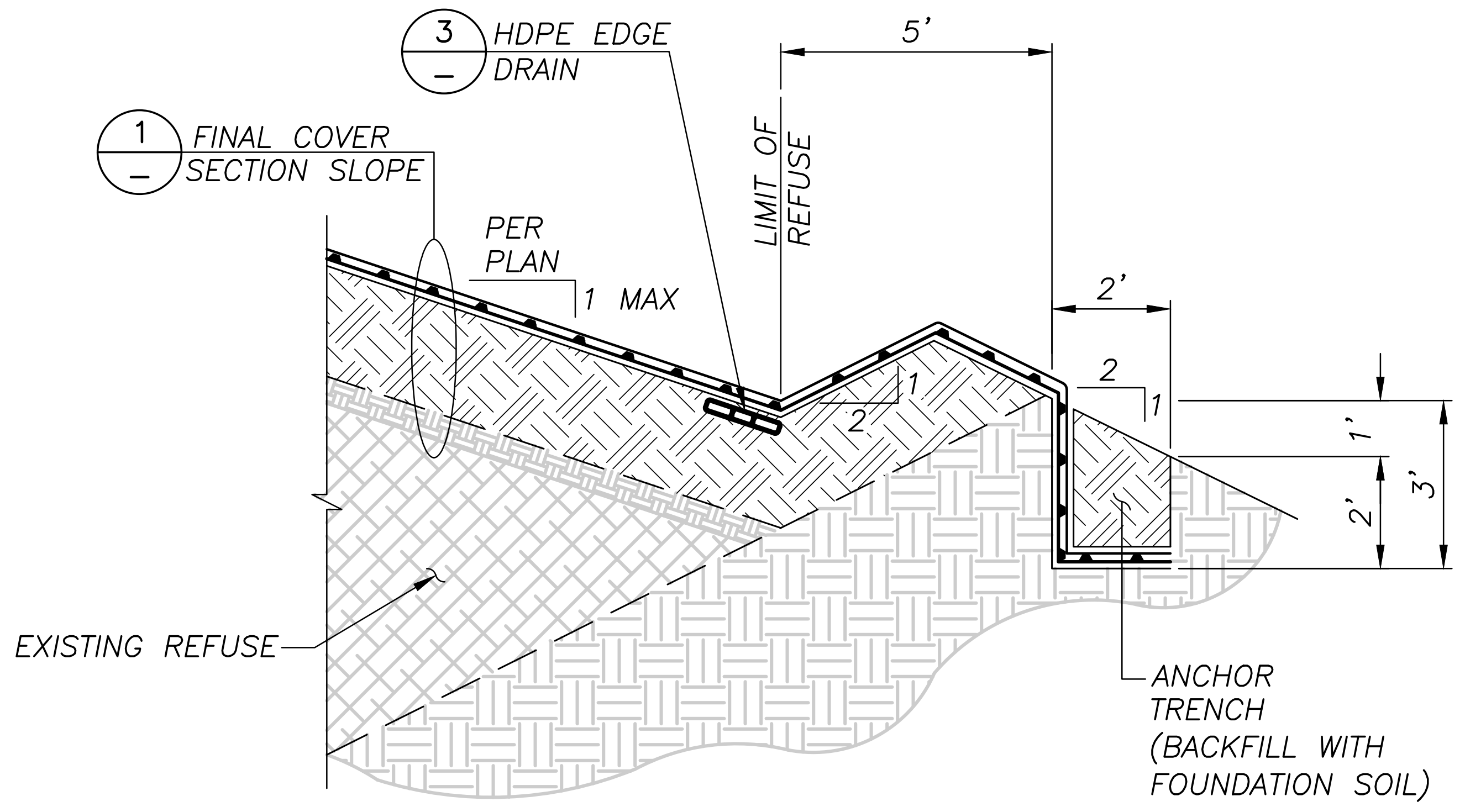
3/D1



DRAINAGE COLLECTOR

NTS

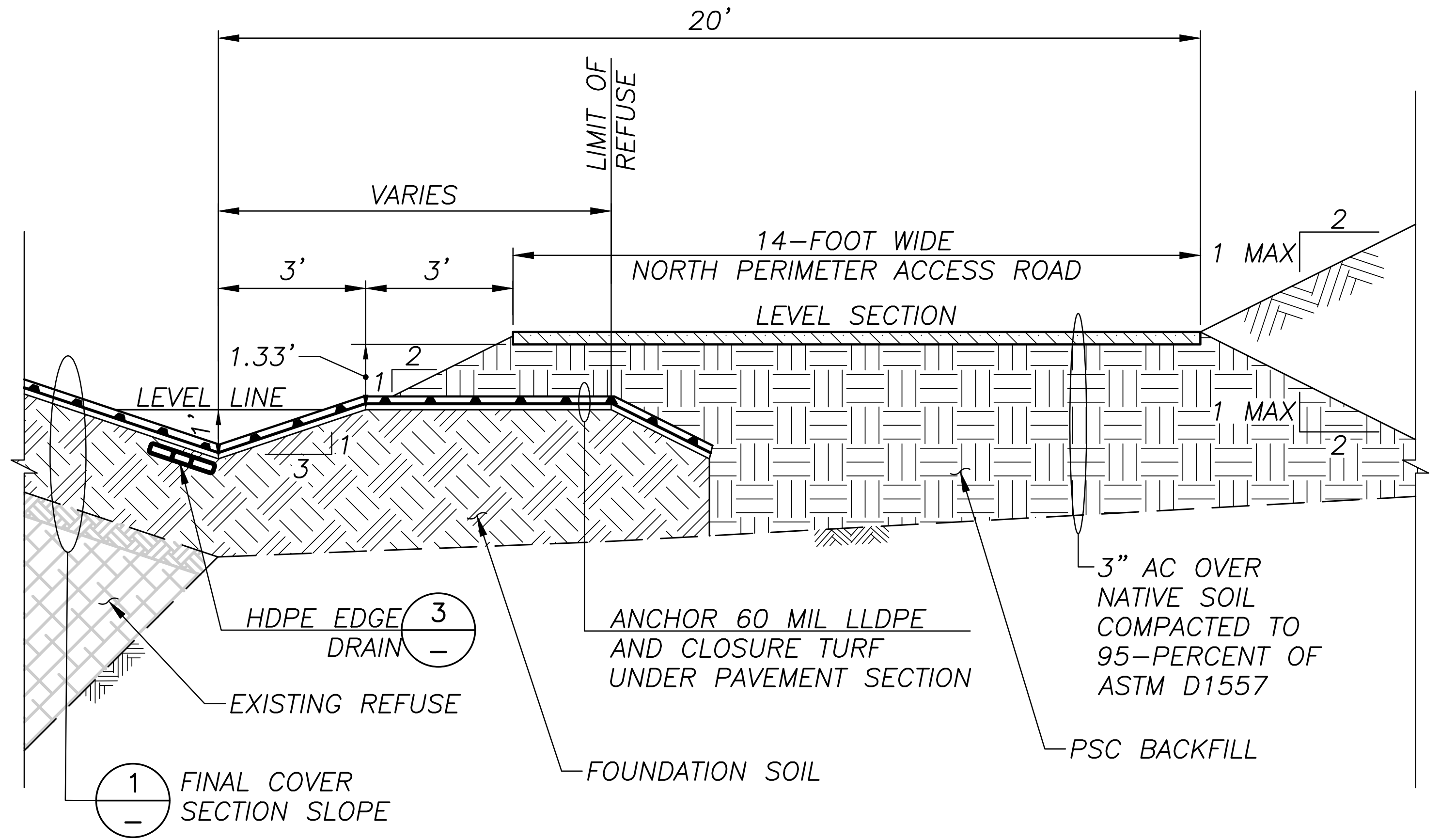
4/D1



FINAL COVER TERMINATION - SLOPE (WITH SWALE)

NTS

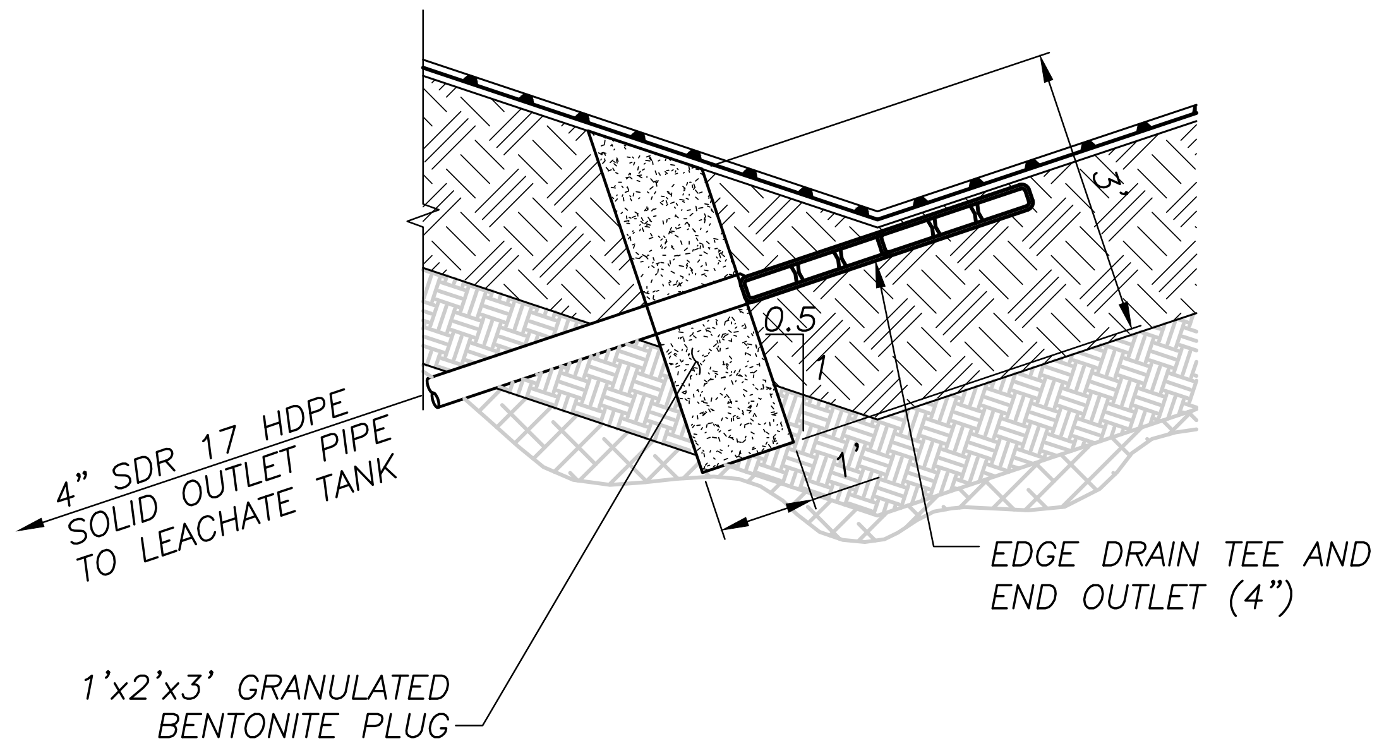
5/D1



FINAL COVER TERMINATION - NORTHEAST PERIMETER ROAD

NTS

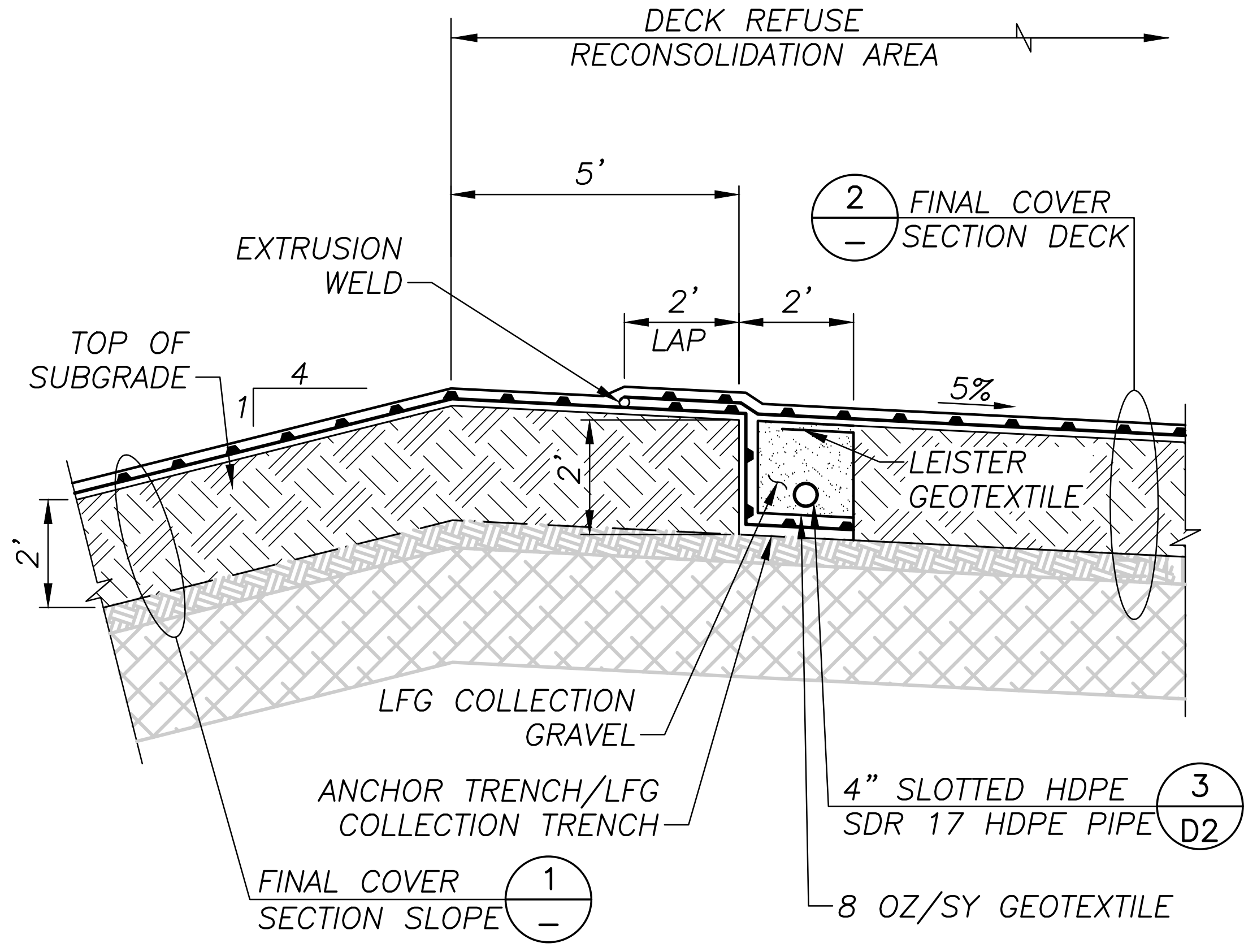
6/D1



BENTONITE PLUG

NTS

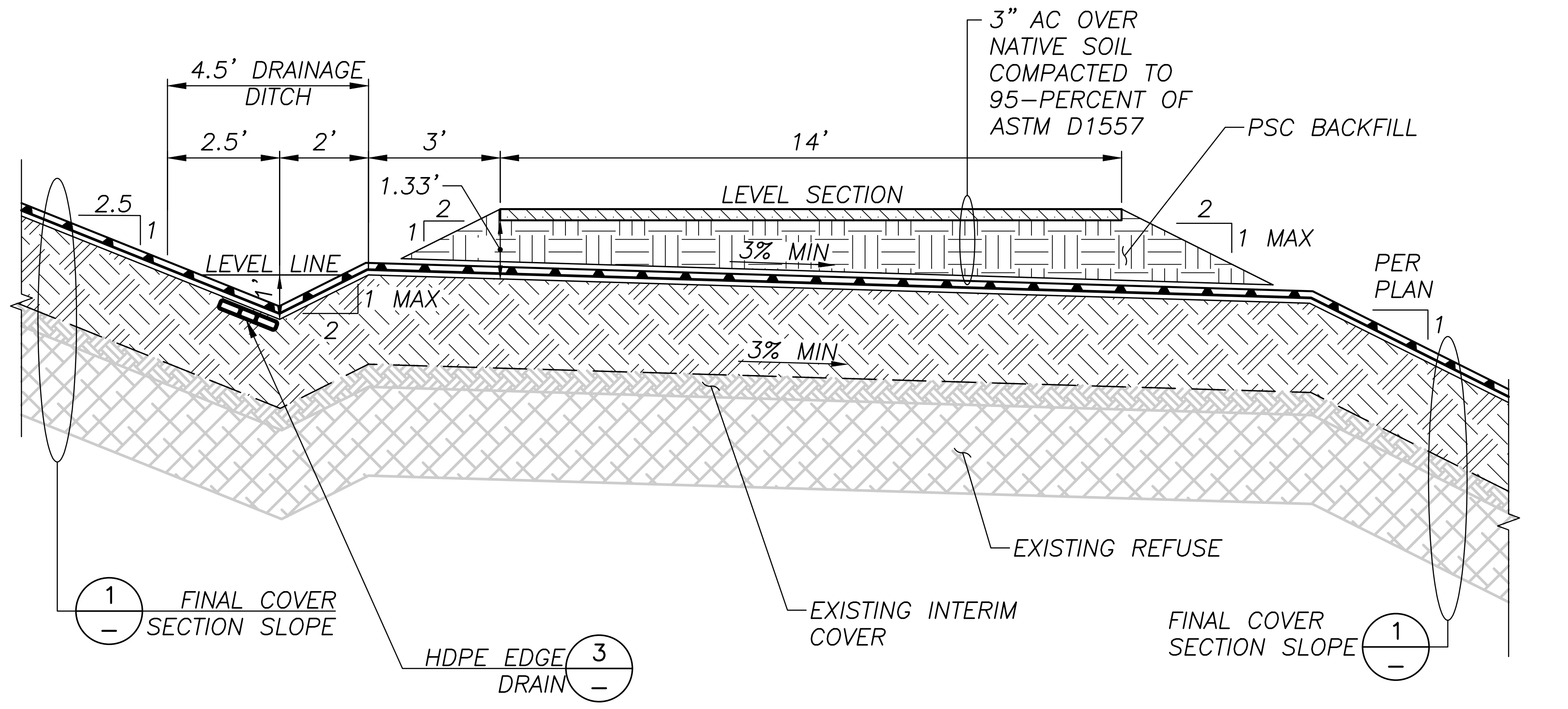
7/D1



TOP DECK TRANSITION

NTS

8/D1

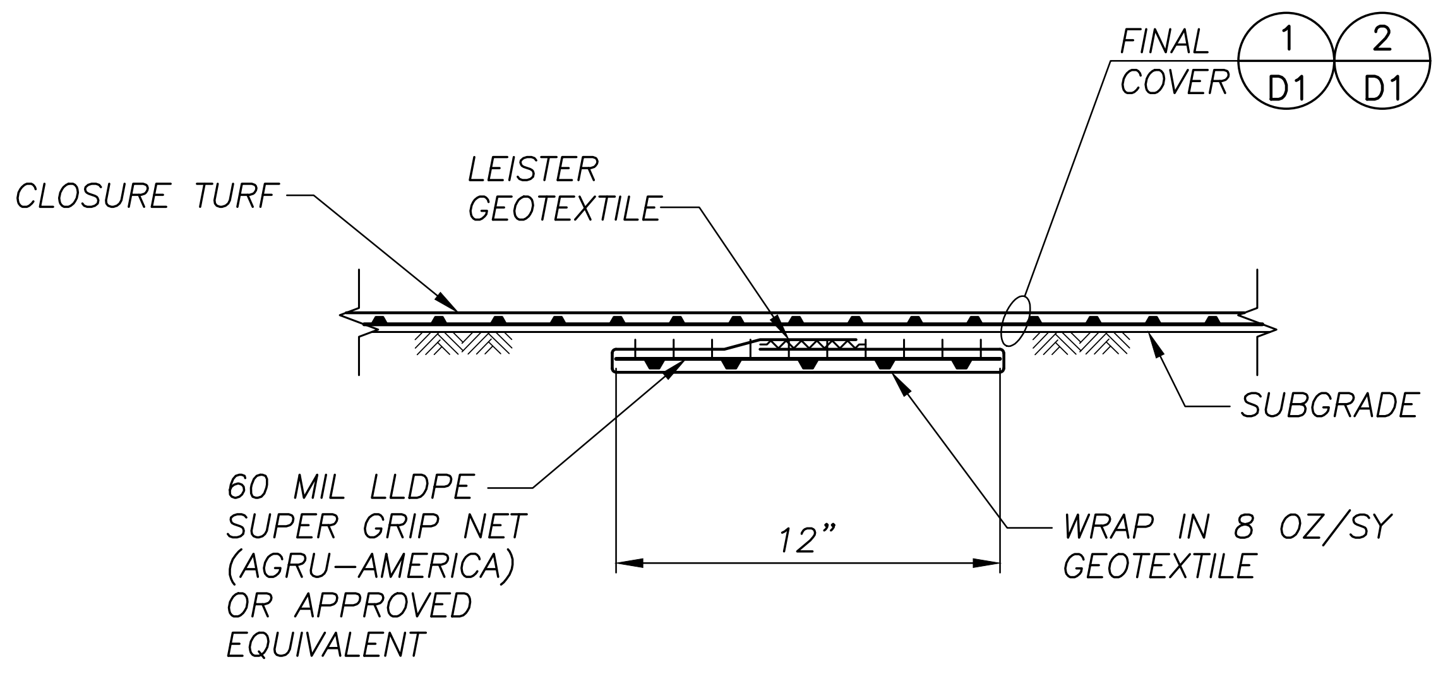


DECK ACCESS ROAD

NTS

9/D1

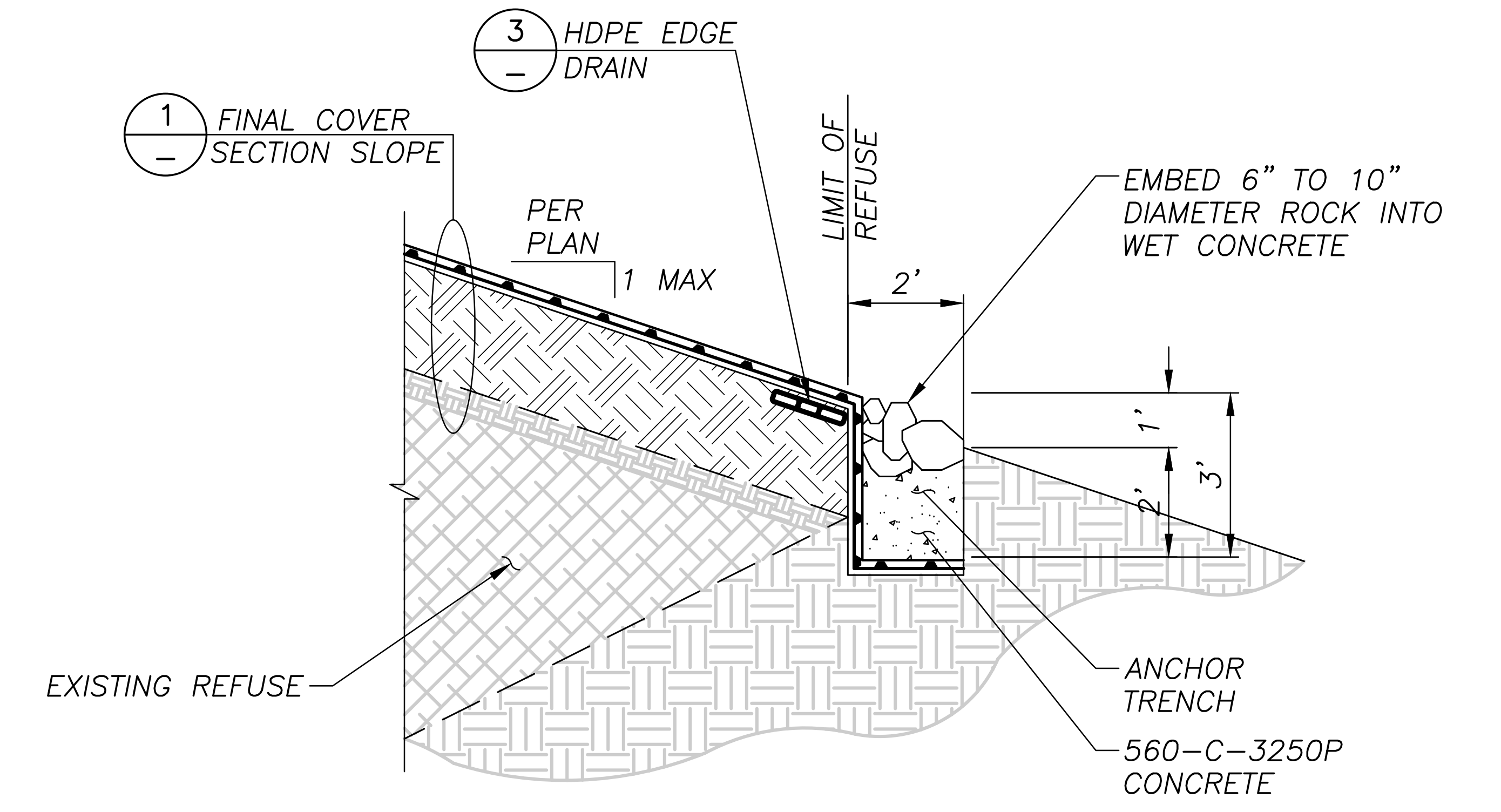
NOTE:
1. PLACE SUPER GRIP NET (12" WIDE STRIPS) AT LOCATION PER SHEET 6
2. EXTEND TO ANCHOR TRENCH AND BELOW GEOMEMBRANE HDPE EDGE DRAIN PER 5/– 8/–



LFG/SEEP COLLECTOR

NTS

10/D1



NOTE: WHERE SHOWN ON PLAN

FINAL COVER TERMINATION - SLOPE (NO SWALE)

NTS

11/D1

NO.	REVISION DESCRIPTION	DATE

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800-C SOUTH ROCHESTER AVENUE
ONTARIO, CALIFORNIA 91761

MICHAEL A. CULLINANE
No. 41981

PREPARED UNDER THE SUPERVISION OF ______ DATE

MENDOCINO COUNTY
SOUTH COAST LANDFILL

FINAL CLOSURE CONSTRUCTION PROJECT 2016
DETAIL SHEET

DESIGNED BY : RMG	SCALE : AS SHOWN	PROJECT NO: –
DRAWN BY : CTD/DMW	DATE : 11-2016	
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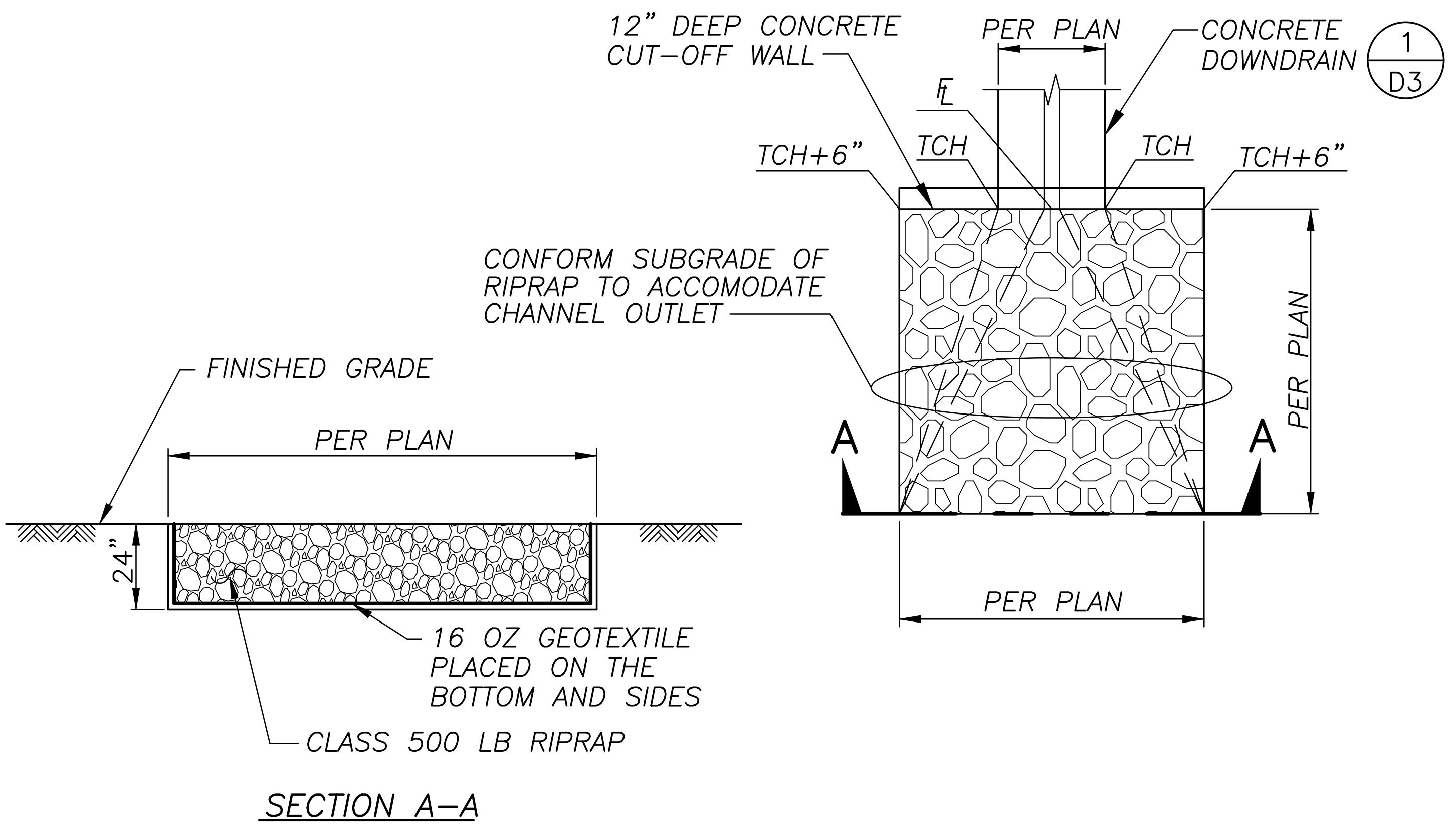
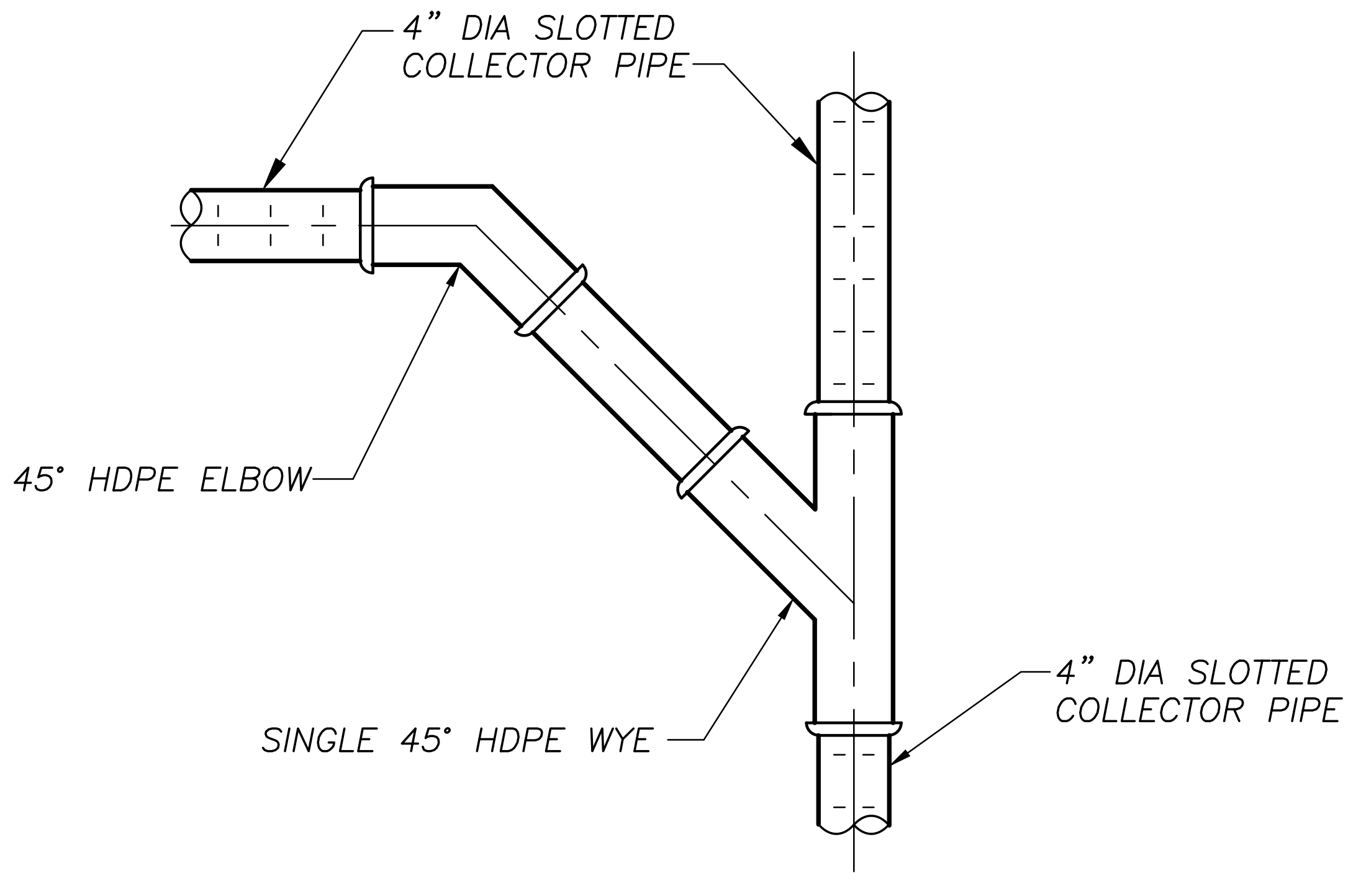


RIP-RAP PAD

NTS

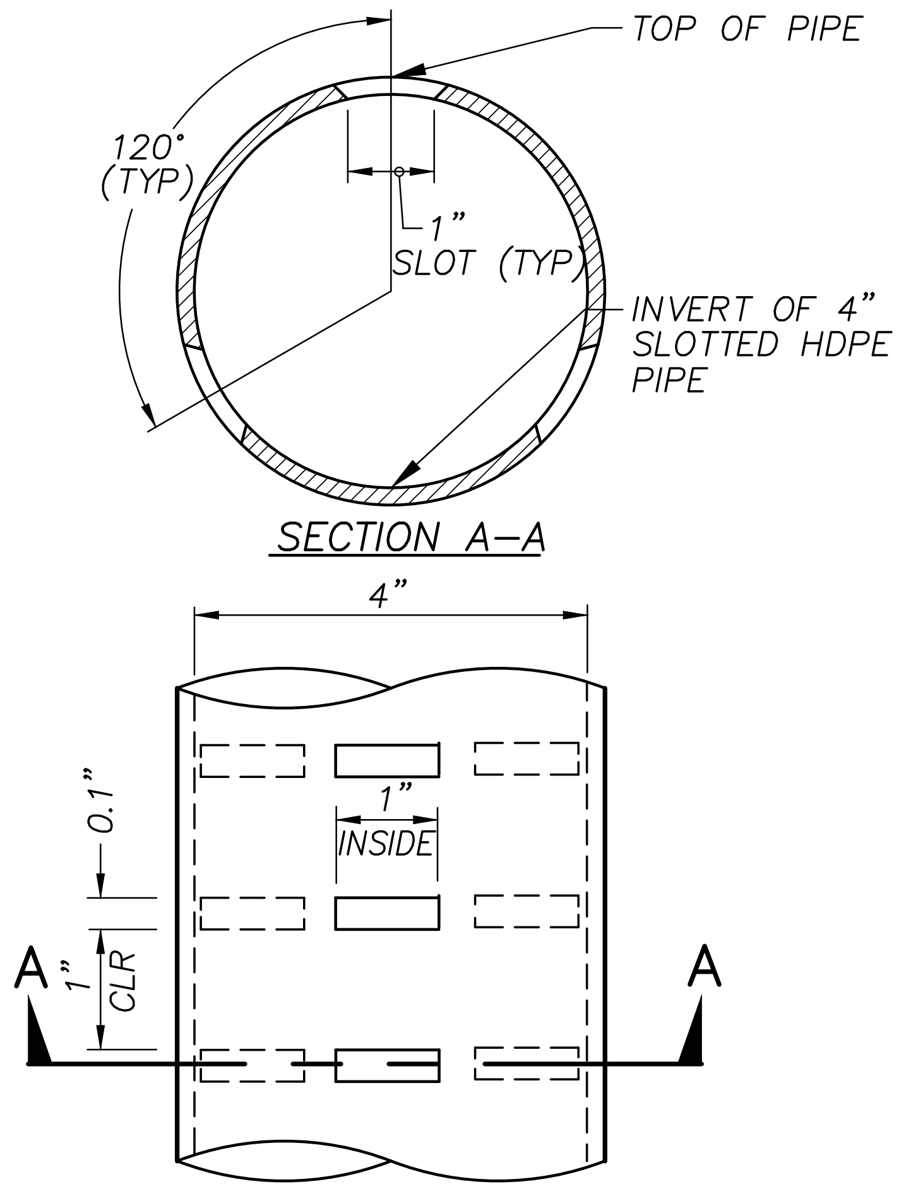
1
D2



SUBDRAIN JUNCTION DETAIL

NTS

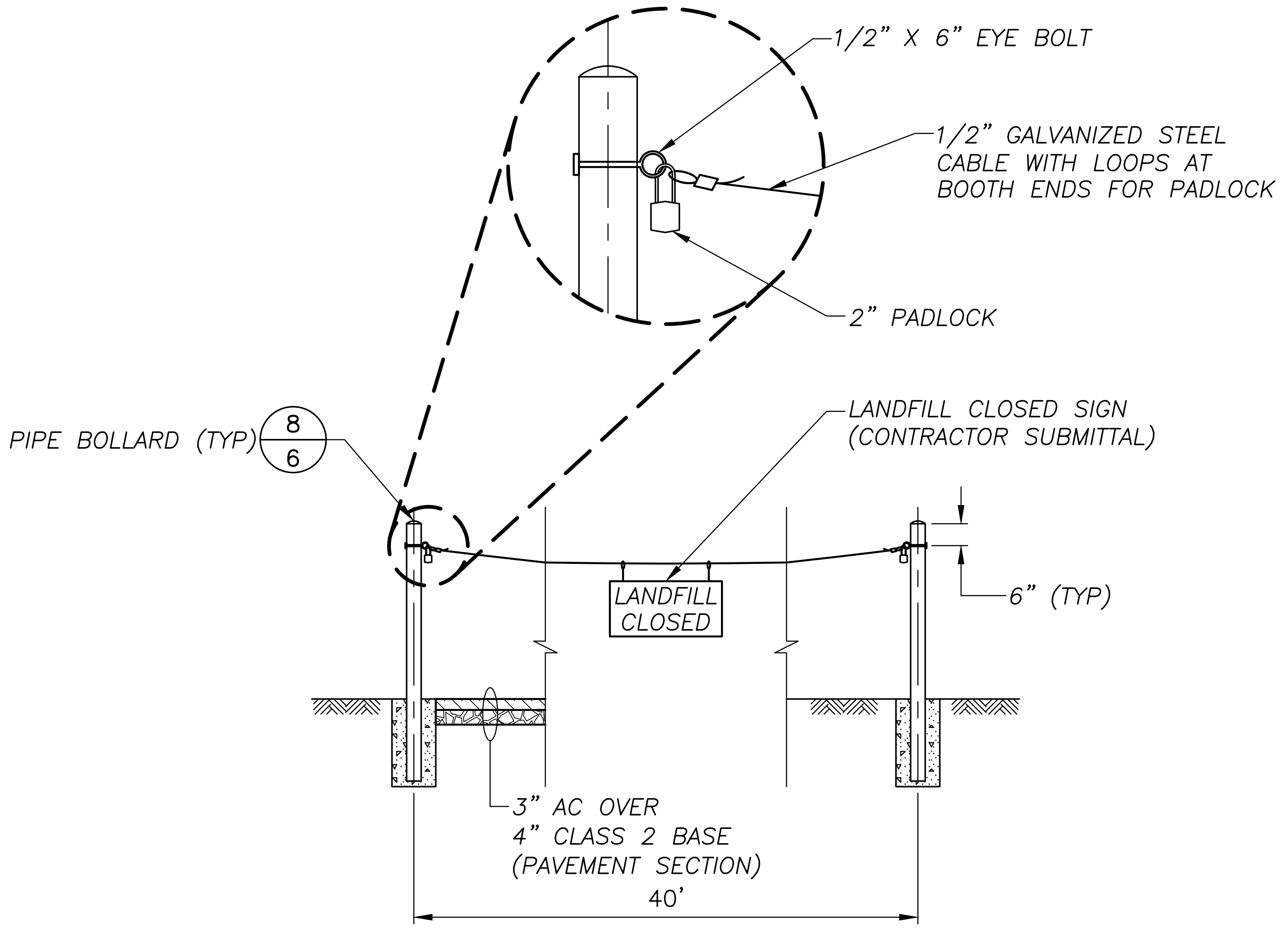
2
D2



SLOTTED HDPE PIPE

NTS

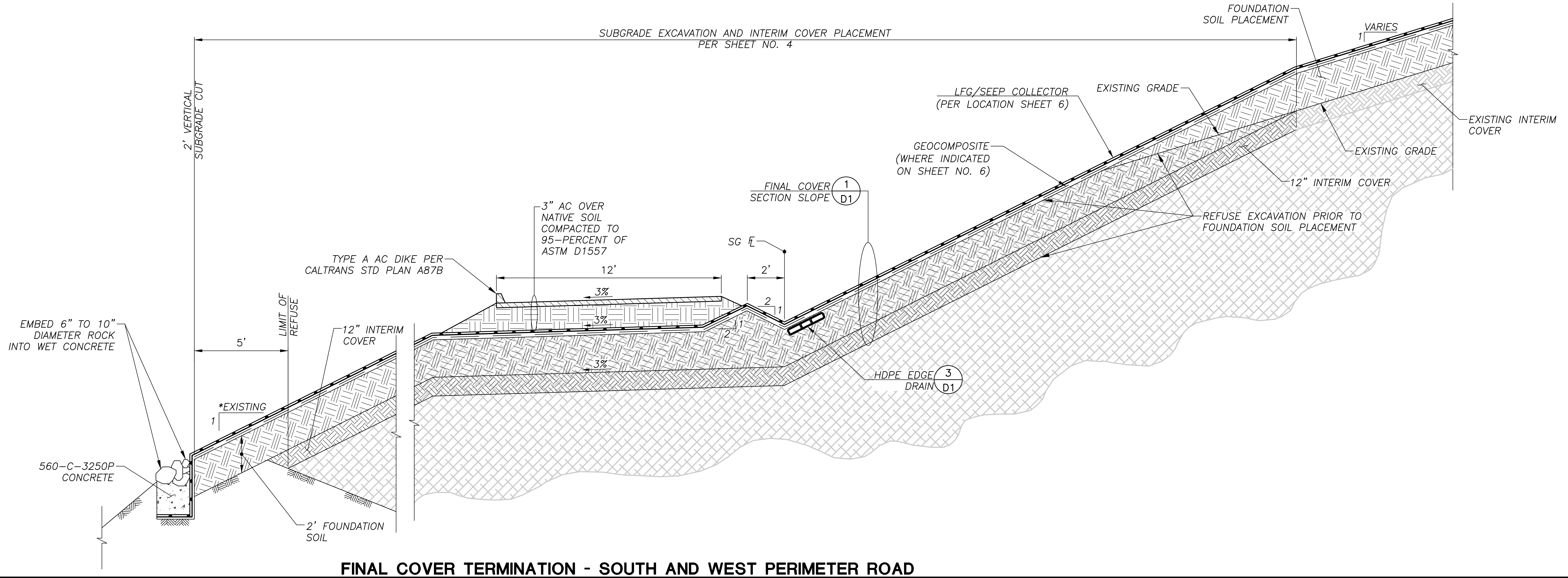
3
D2



CABLE GATE

NTS

5
D2



FINAL COVER TERMINATION - SOUTH AND WEST PERIMETER ROAD

NTS

4
D2

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DATE

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CIVIL

MENDOCINO COUNTY
SOUTH COAST LANDFILL

FINAL CLOSURE CONSTRUCTION PROJECT 2016
DETAIL SHEET

DESIGNED BY : RMG	SCALE : AS SHOWN	PROJECT NO: –
DRAWN BY : CTD/DMW	DATE : 11-2016	
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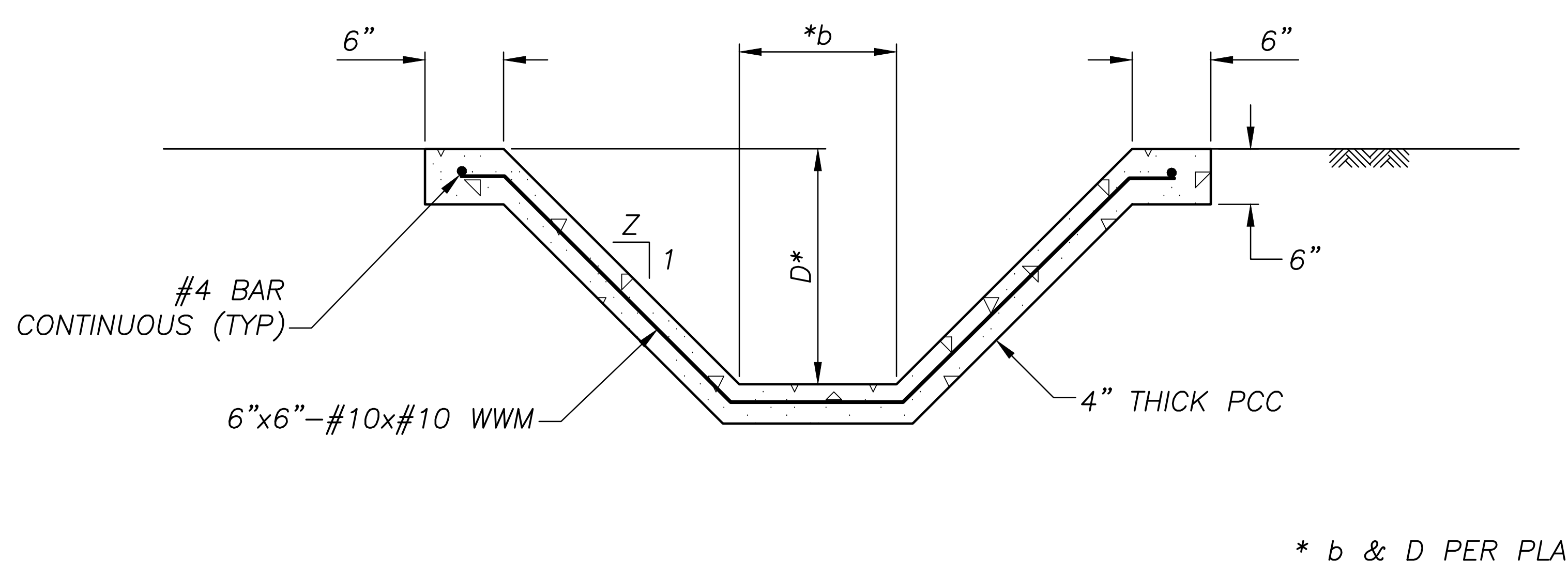
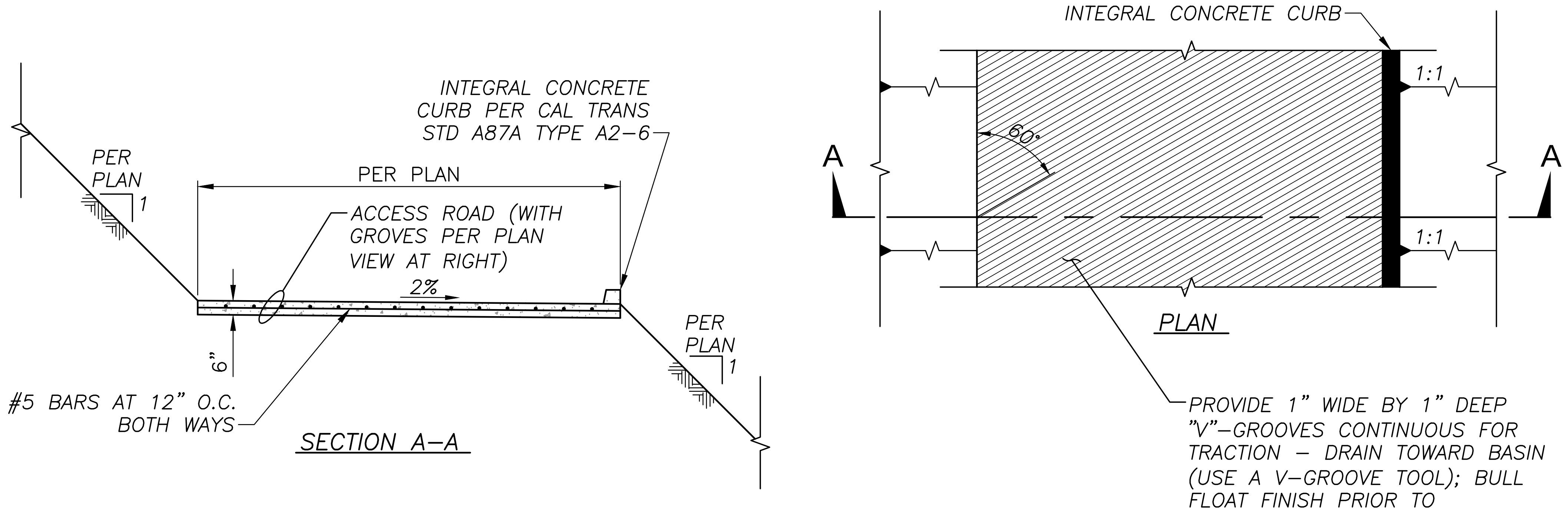


CONCRETE DOWNDRAIN/CHANNEL

NTS

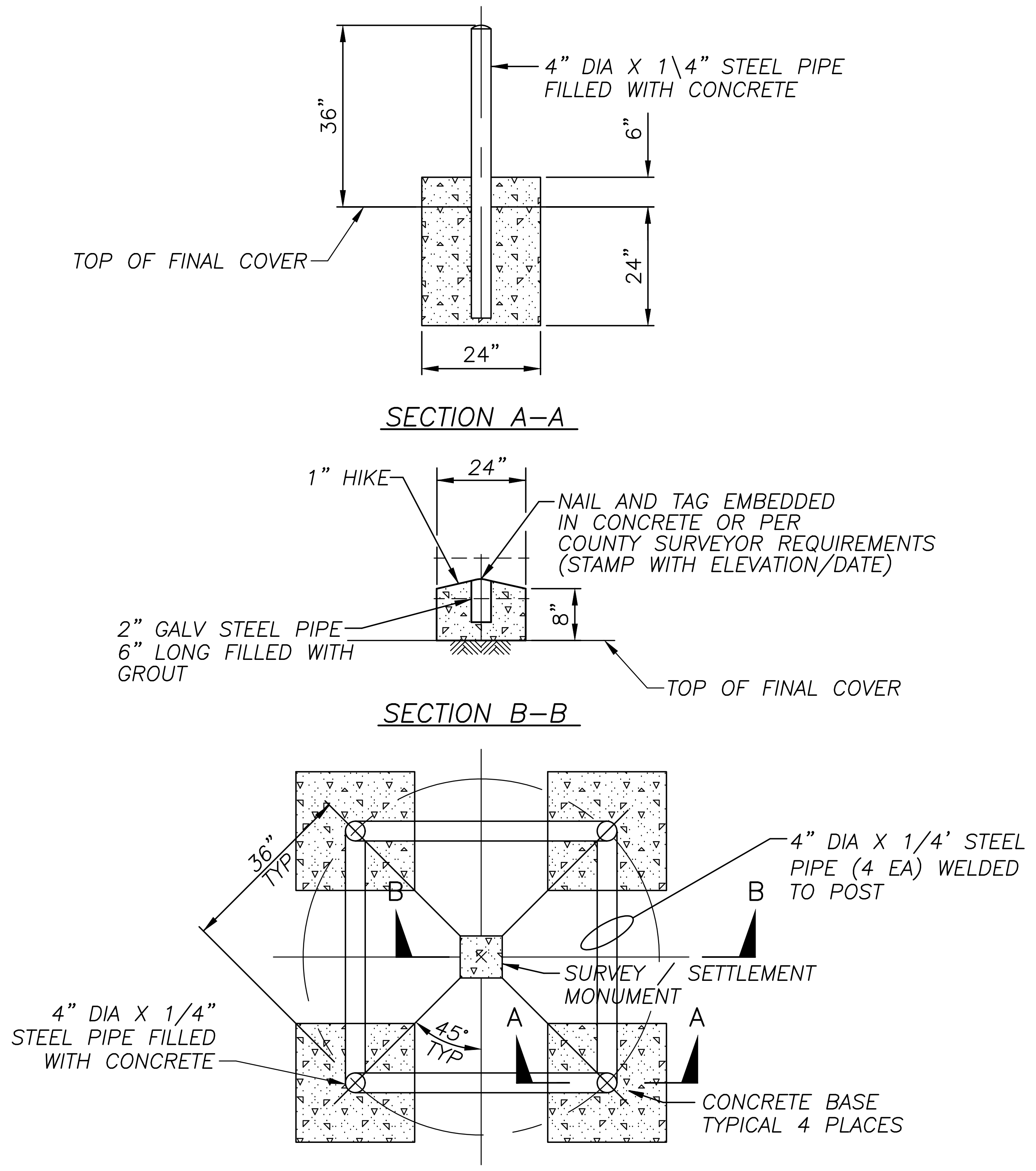
1
D3



CONCRETE ACCESS RAMP TO BASIN

NTS

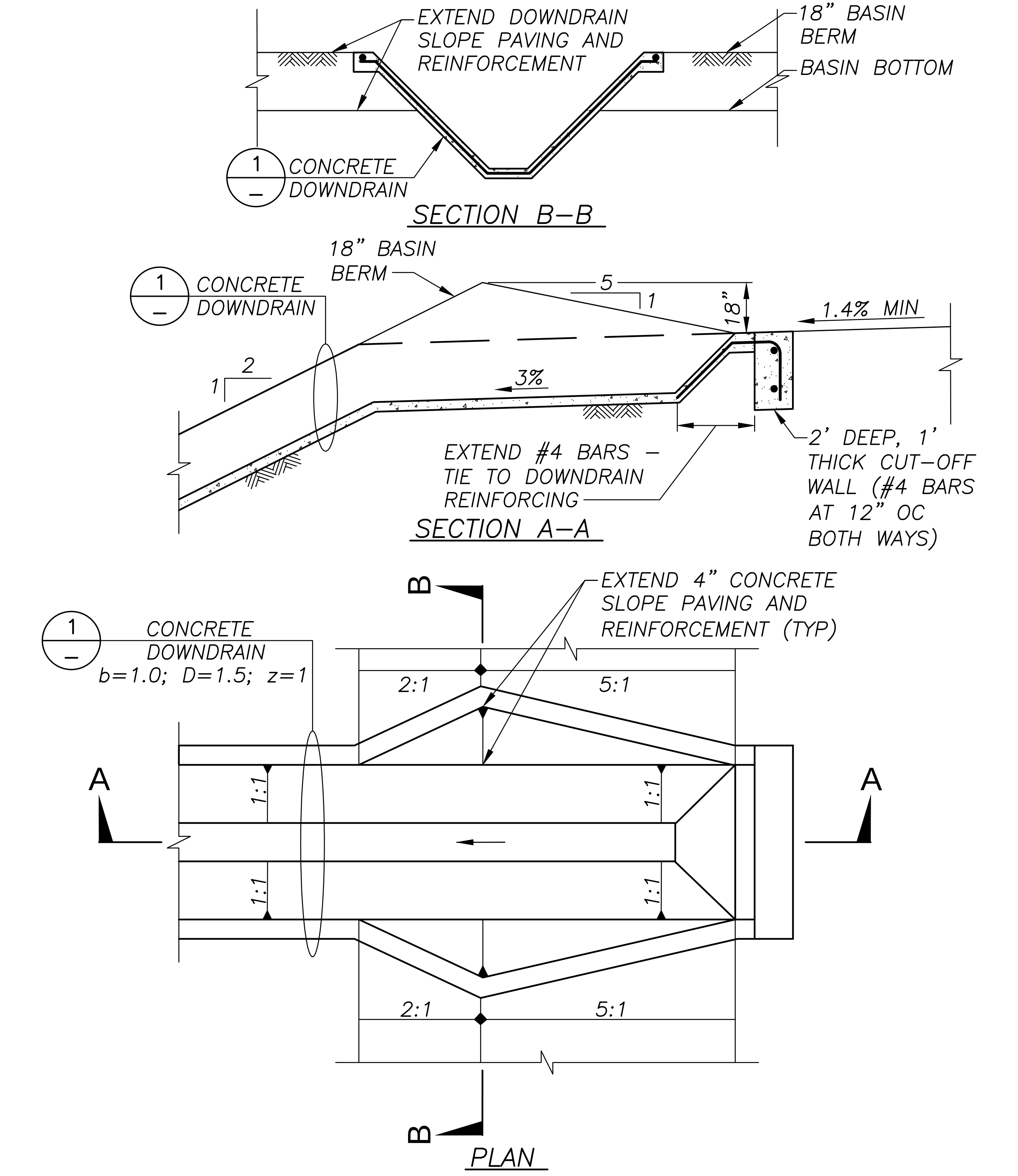
4
D3



POST-CLOSURE SURVEY/ SETTLEMENT MONUMENT

NTS

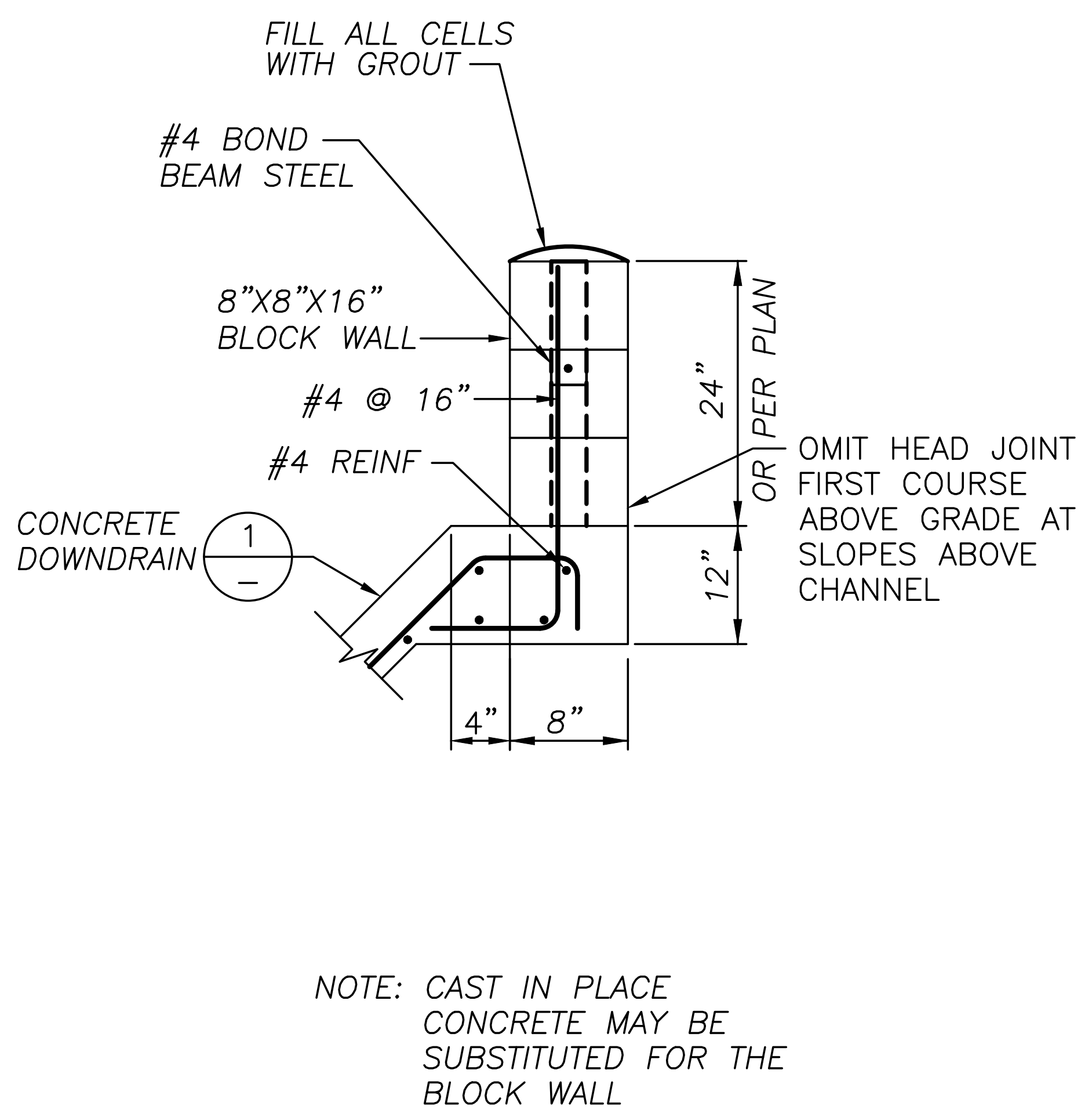
2
D3



BASIN BERM/DOWNDRAIN INLET

NTS

3
D3

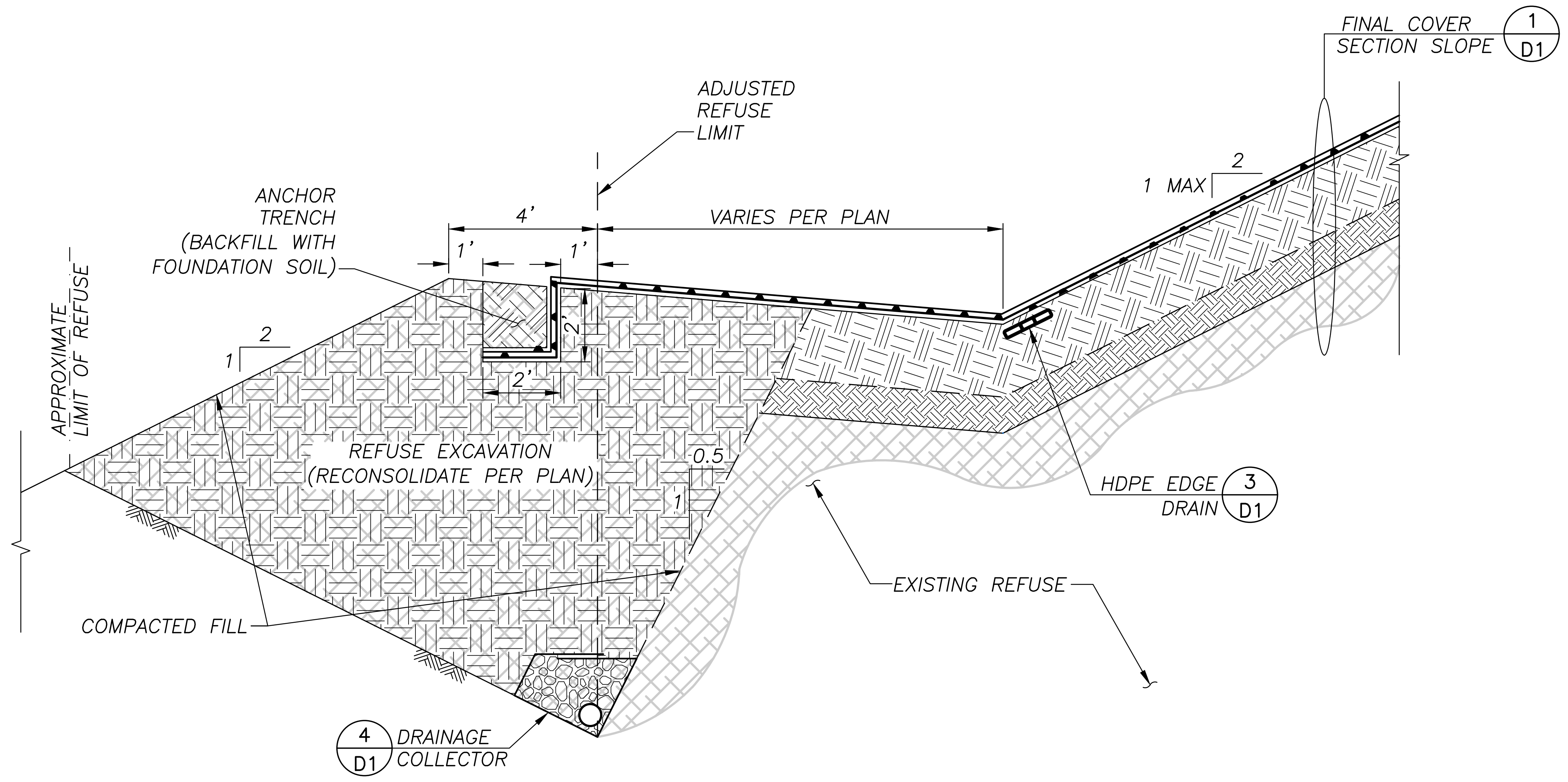


NOTE: CAST IN PLACE CONCRETE MAY BE SUBSTITUTED FOR THE BLOCK WALL

SPLASHWALL

NTS

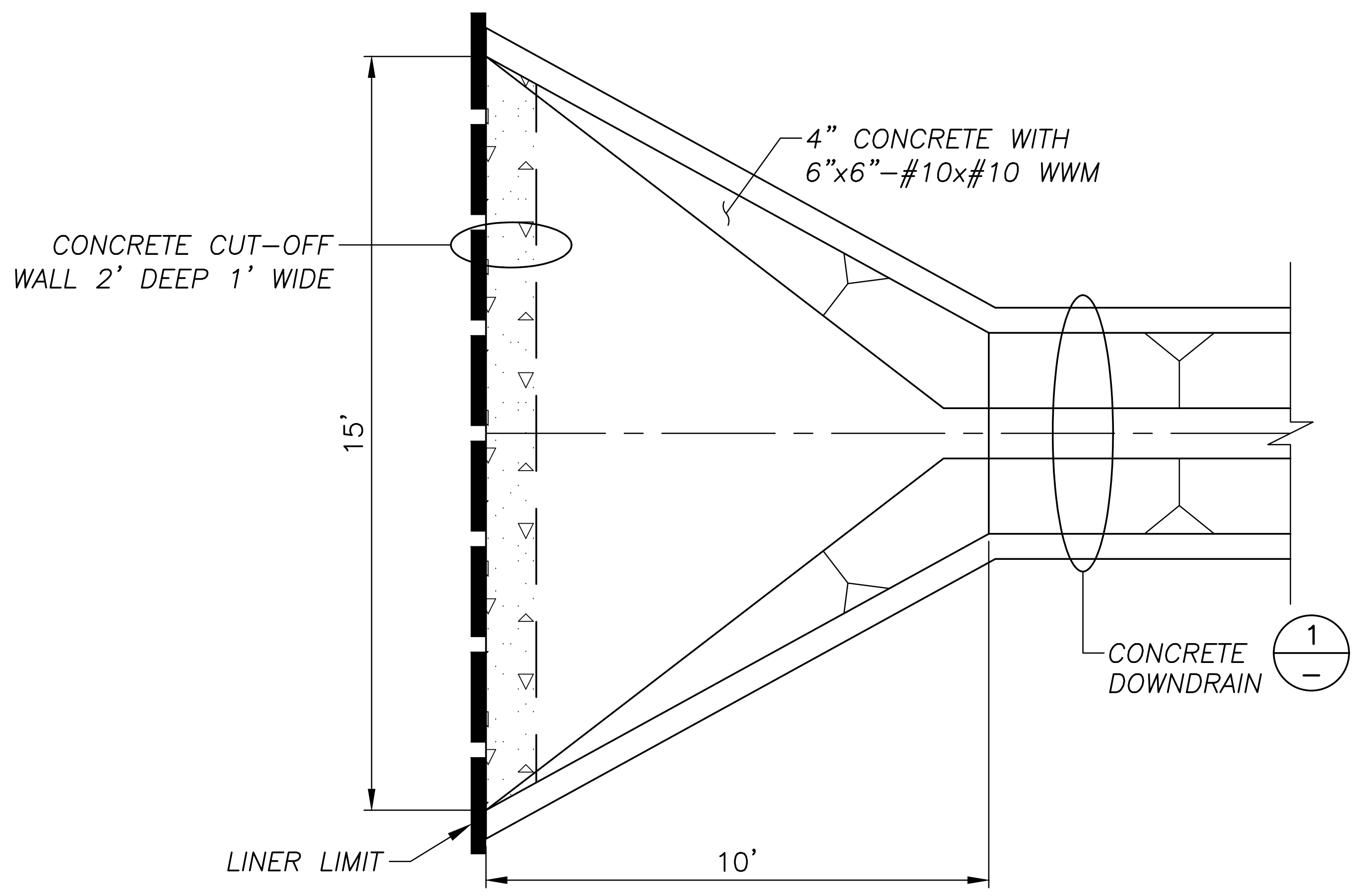
5
D3



BENCH LINER TERMINATION

NTS

6
D3



CONCRETE INLET TO DOWNDRAIN

NTS

7
D3

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ONTARIO, CALIFORNIA 91761

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MENDOCINO COUNTY
SOUTH COAST LANDFILL

FINAL CLOSURE CONSTRUCTION PROJECT 2016
DETAIL SHEET

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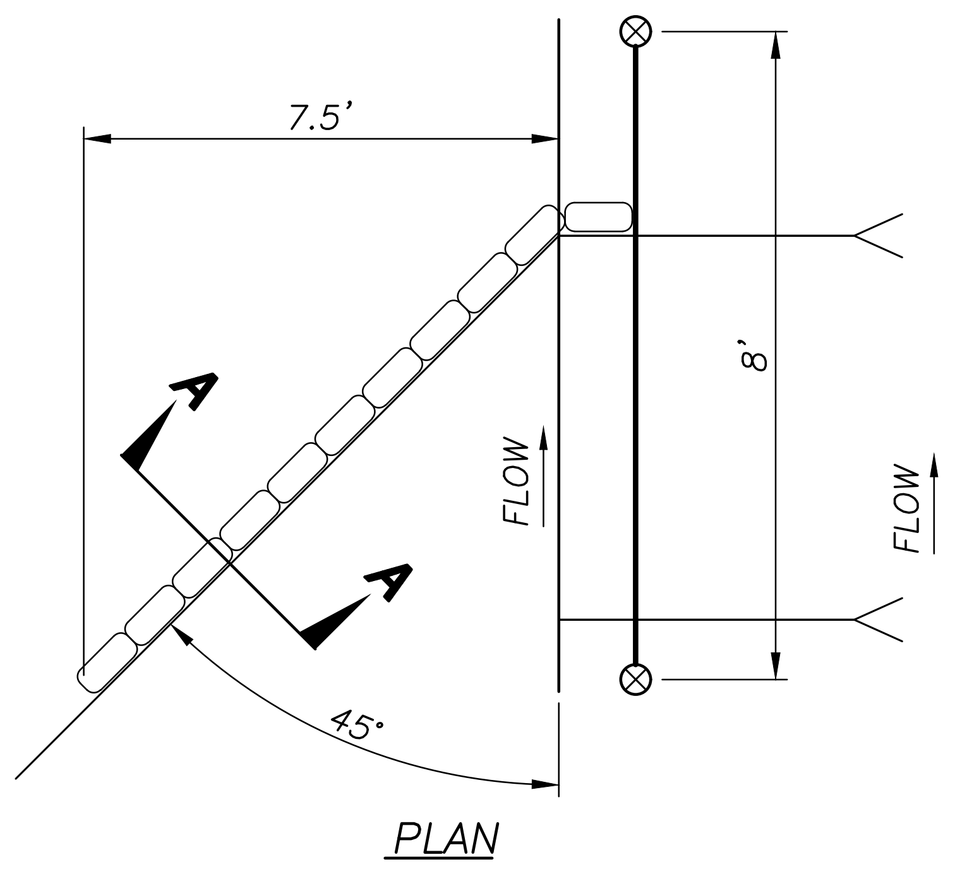
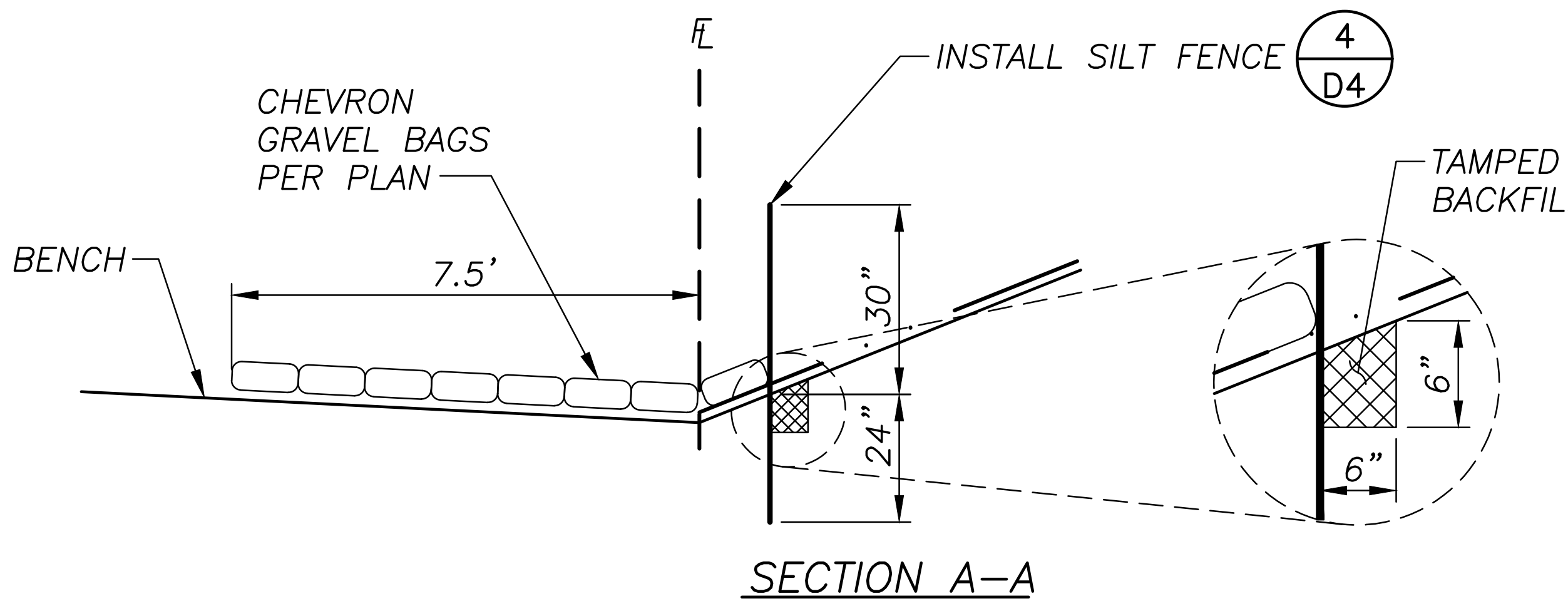


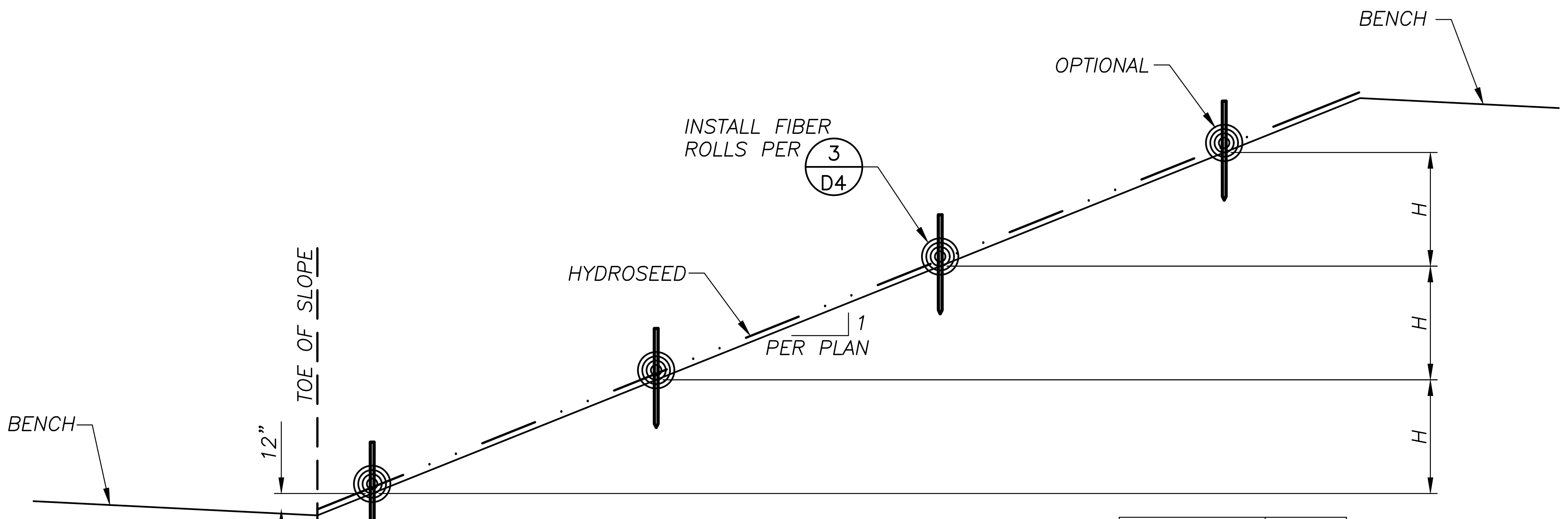


GRADIENT	UNPAVED SURFACE	PAVED SURFACE
1%–3%	200	250
3%–5%	150	200
5%–7%	100	150
7%–9%	50	100
9%+	25	50

GRAVEL BAG CHEVRON DETAIL

NTS

1 D4



SLOPE RATIO	H
2:1	7.5'
2.5:1	6.0'
3:1	5.0'

FIBER ROLL SPACING

NTS

2 D4

NOTES:
1) FIBER ROLL INSTALLATION AT BENCH SHALL BE A MINIMUM OF 12" ABOVE BENCH FLOWLINE, UNLESS SILT FENCE IS SPECIFIED AT THAT LOCATION.

2) A 3" DEEP TRENCH, THE WIDTH OF THE ROLL, DUG PER PLAN LOCATION SHALL "NEST" THE STRAW ROLL SO AS NOT TO ALLOW RUNOFF TO FLOW UNDER OR AROUND ROLL.

3) FIBER ROLLS SHALL BE PLACED ON SLOPES AT H–FEET VERTICAL INTERVALS STARTING FROM THE TOE OF RESPECTIVE SLOPE. DECK ROLLS SHALL BE PLACED AT 10–FEET VERTICAL INTERVALS AS SHOWN ON THE PLAN.

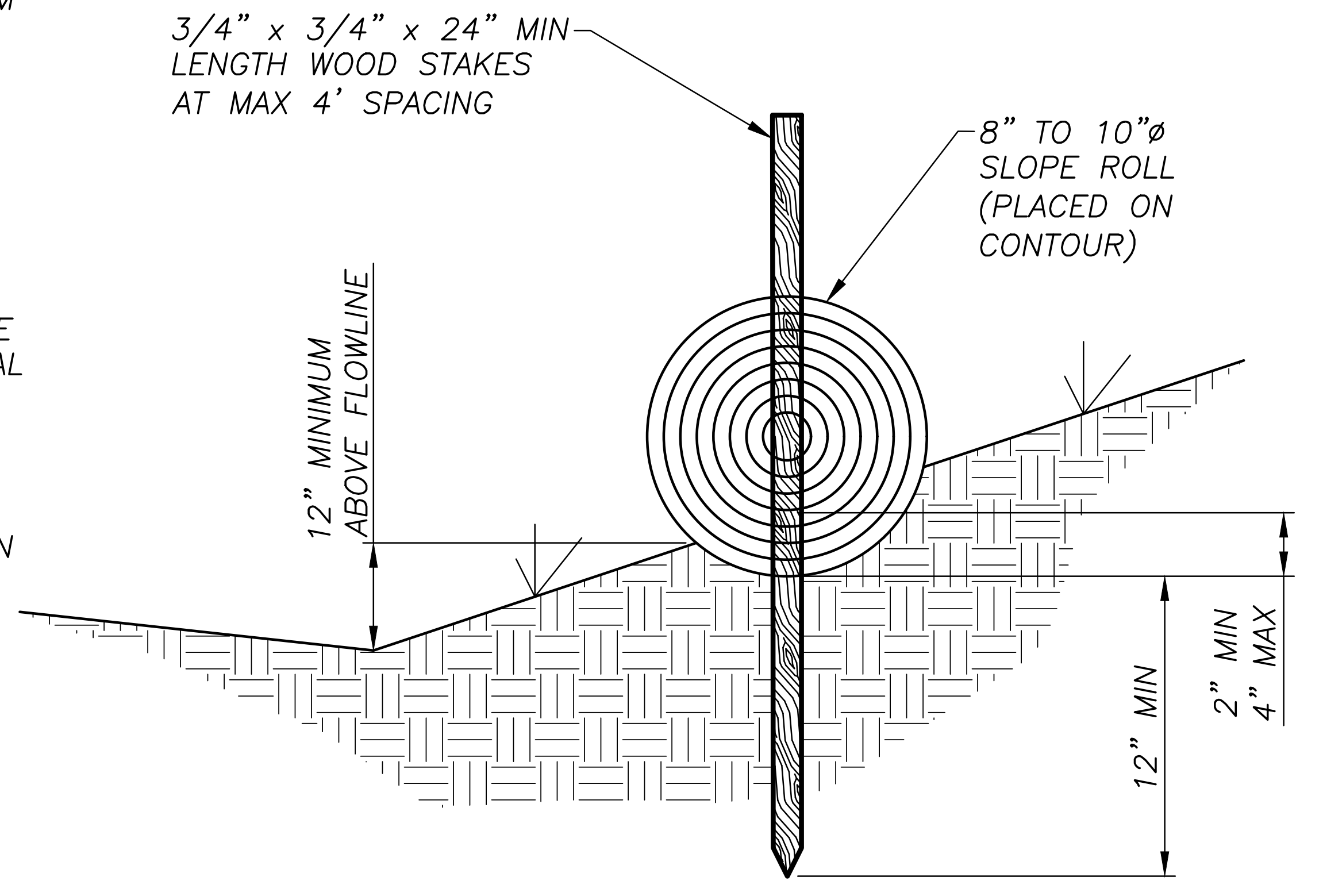
4) INSTALL FIBER ROLLS ALONG A LEVEL CONTOUR.

5) INSTALL FIBER ROLLS UP GRADIENT AND NEAR TRANSITION OF SLOPES.

6) INSTALL NUMEROUS ROLLS IN A ROW BY OVERLAPPING, NOT ABUTTING.

7) TURN ENDS OF ROLLS UP SLOPE TO PREVENT RUNOFF FROM GOING AROUND THE ROLL.

8) SEDIMENT IS TO BE REMOVED WHEN ACCUMULATION REACHES ONE HALF THE DISTANCE BETWEEN THE TOP OF THE ROLL AND ADJACENT GROUND SURFACE.



FIBER ROLLS

NTS

3 D4

NOTES:
1) THE LENGTH OF EACH REACH OF SILT FENCE IN NO CASE SHALL EXCEED 500'. USE GRAVEL BAG CROSS BARRIERS SPACED PER TABLE 1 AT RIGHT.

2) THE LAST 8'–0" OF EACH REACH OF FENCE SHALL BE TURNED UP SLOPE PER DETAIL 4 D4

3) STAKE DIMENSIONS ARE NOMINAL.

4) DIMENSIONS MAY VARY TO FIT FIELD CONDITION.

5) STAKES SHALL BE SPACED AT 8'–0" MAXIMUM AND SHALL BE POSITIONED ON DOWNSTREAM SIDE OF FENCE.

6) JOINING SECTION STAKES TO OVERLAP AND FENCE FABRIC TO FOLD AROUND EACH STAKE ONE FULL TURN. SECURE FABRIC TO STAKE WITH 4 HOG RING TIES.

7) JOINING SECTION STAKES SHALL BE DRIVEN TIGHTLY TOGETHER TO PREVENT POTENTIAL FLOW–THROUGH OF SEDIMENT AT JOINT. THE TOPS OF THE STAKES SHALL BE SECURED TO EACH OTHER WITH WIRE TIES TO PREVENT SEPARATION.

8) FOR END STAKES, FENCE FABRIC SHALL BE FOLDED AROUND TWO STAKES, ONE FULL TURN AND SECURED WITH 4 HOG RING TIES.

9) A MINIMUM OF 4 HOG RING TIES PER STAKE SHALL BE UTILIZED. DIMENSIONS SHOWN ARE TYPICAL.

10) CROSS BARRIERS SHALL BE SPACED PER TABLE 1 AT RIGHT.

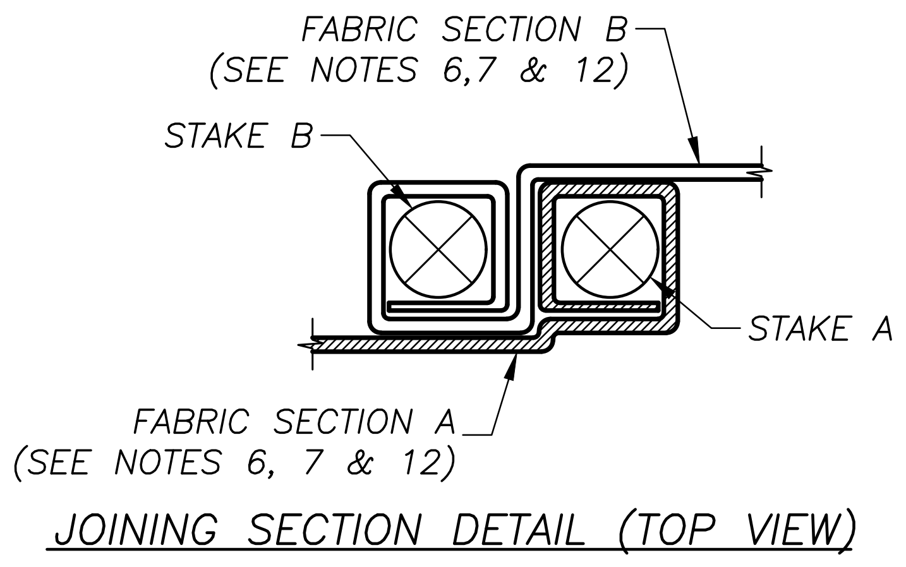
11) MAINTENANCE OPENINGS SHALL BE CONSTRUCTED IN A MANNER TO ENSURE THAT SEDIMENT REMAINS BEHIND THE SILT FENCE.

12) JOINING SECTIONS SHALL NOT BE PLACED AT SUMP LOCATIONS.

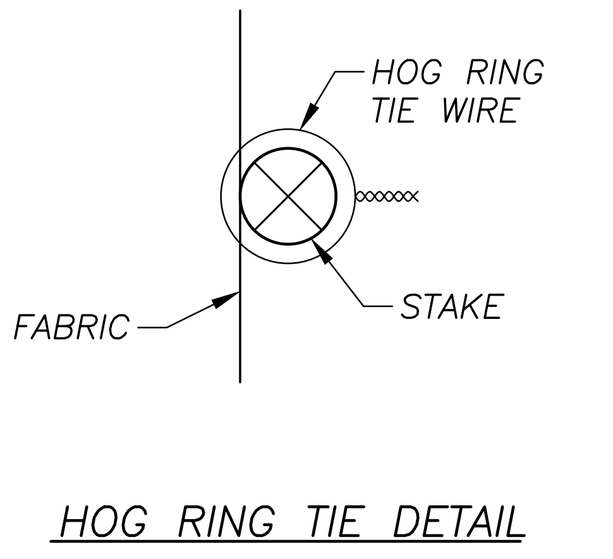
13) GRAVEL BAG ROWS AND LAYERS SHALL BE OFFSET TO ELIMINATE GAPS.

14) SILT FENCE FABRIC SHALL BE BURIED IN A TRENCH 6–INCHES WITH 6–INCH RETURN, AND SHALL BE BACKFILLED PER SECTION B–B, TO PREVENT SILT FROM UNDERMINING/BYPASSING FENCE FABRIC.

15) IN FLATTER SLOPE AREAS ADDITIONAL GRAVEL BAGS MAY BE REQUIRED TO MAINTAIN 12" MIN SILT STORAGE BEHIND CROSS BARRIER.



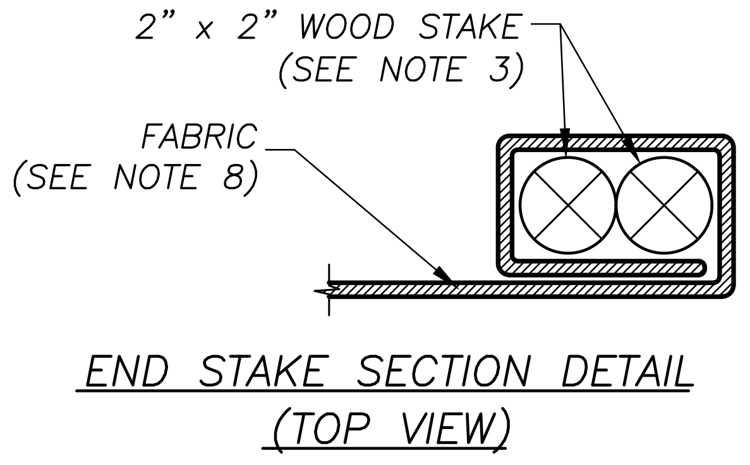
JOINING SECTION DETAIL (TOP VIEW)



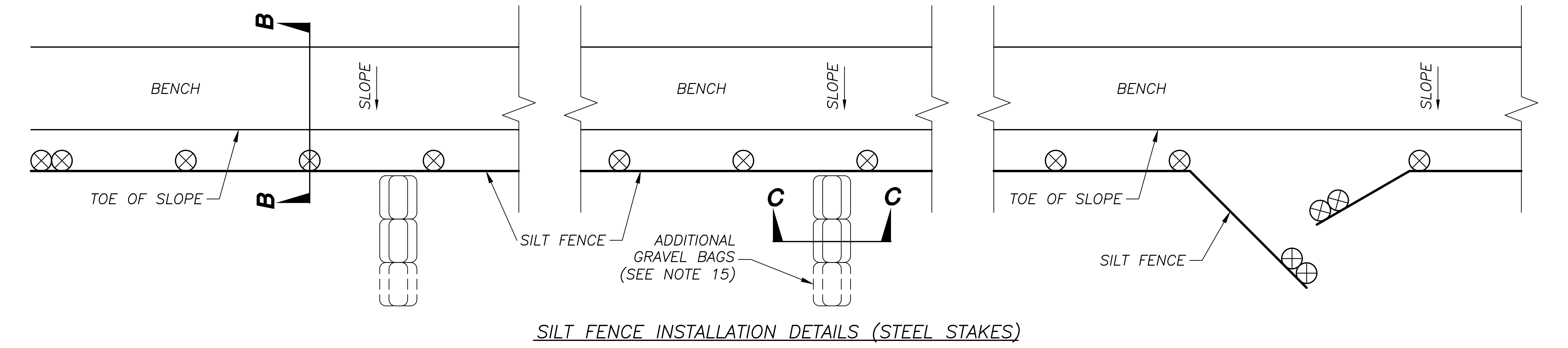
HOG RING TIE DETAIL

TABLE 1

SLOPE OF BENCH/ROAD	CROSS BARRIER SPACING (FEET)
S ≤ 1.0%	100
1.0% < S ≤ 1.5%	75
1.5% < S ≤ 2.0%	50
2.0% < S ≤ 2.5%	50
2.5% < S ≤ 3.0%	40
3.0% < S ≤ 5.0%	30
5.0% < S	25



END STAKE SECTION DETAIL (TOP VIEW)



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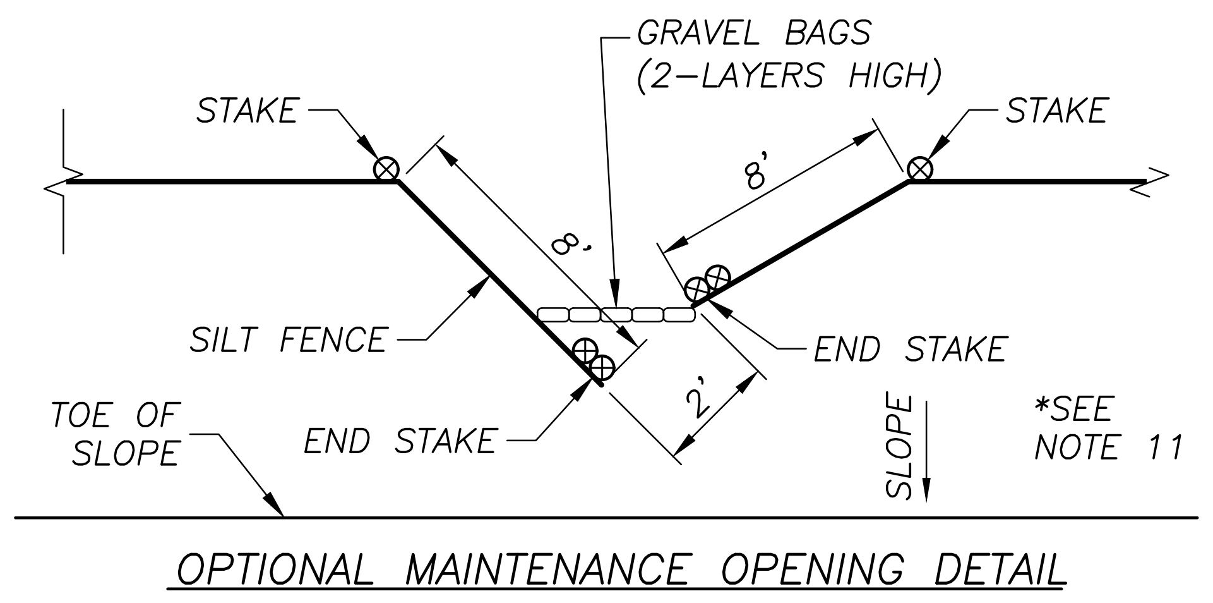

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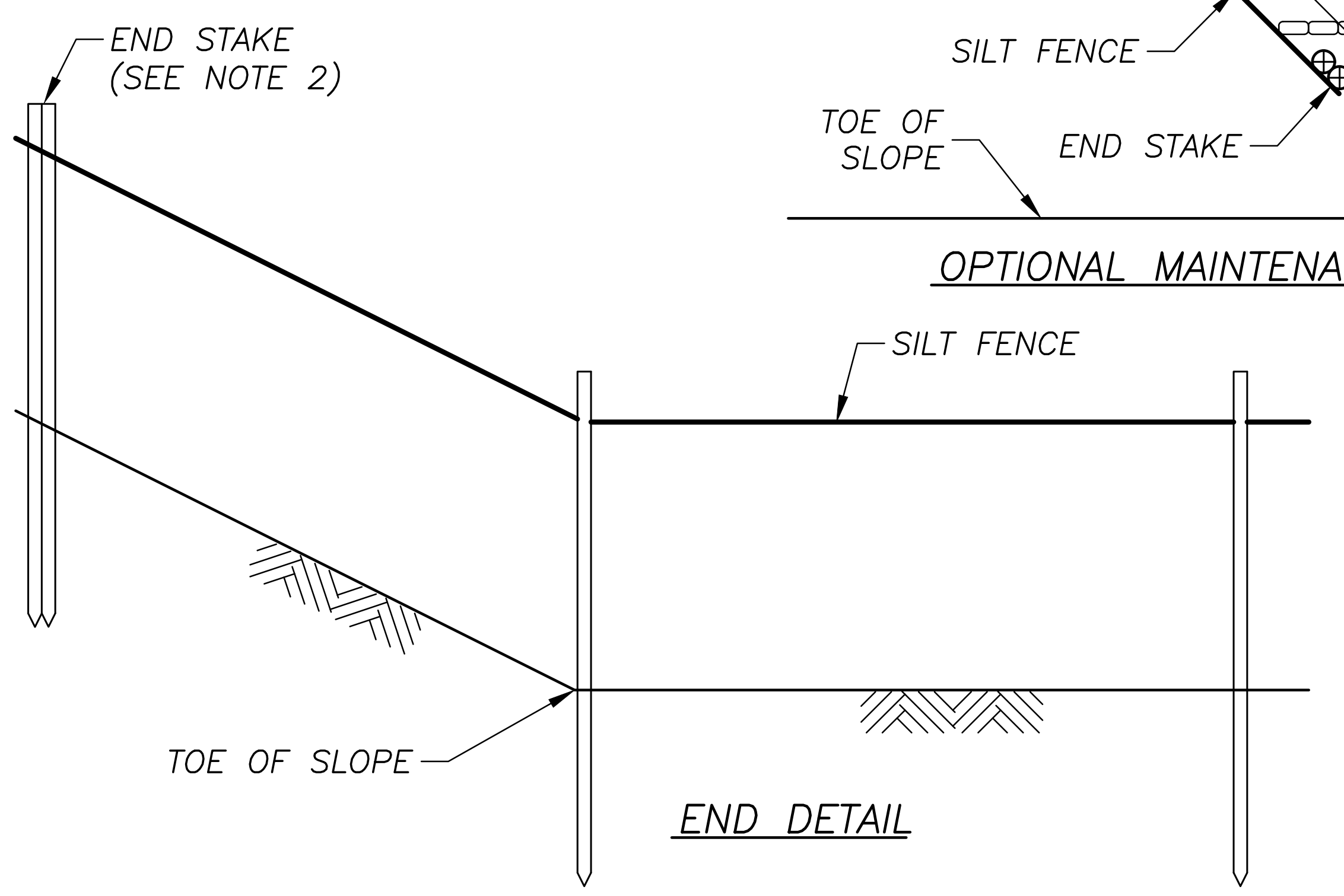

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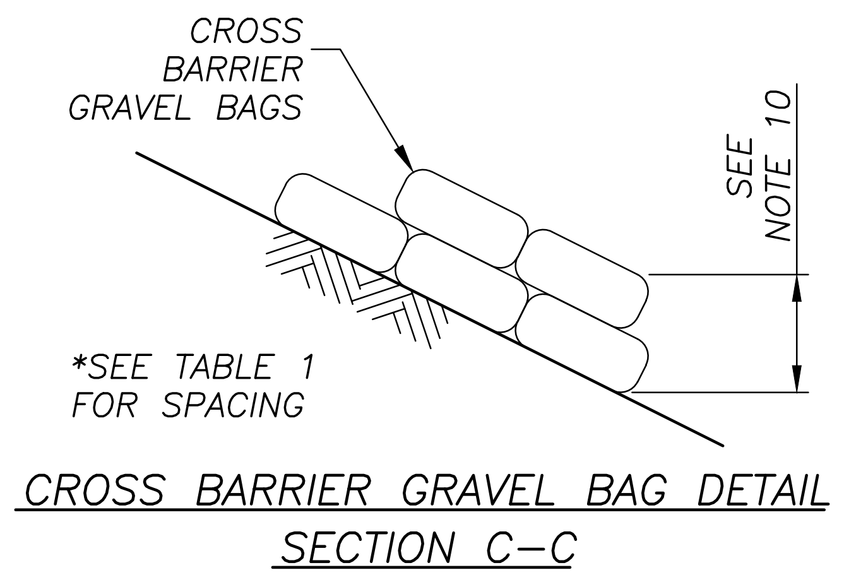

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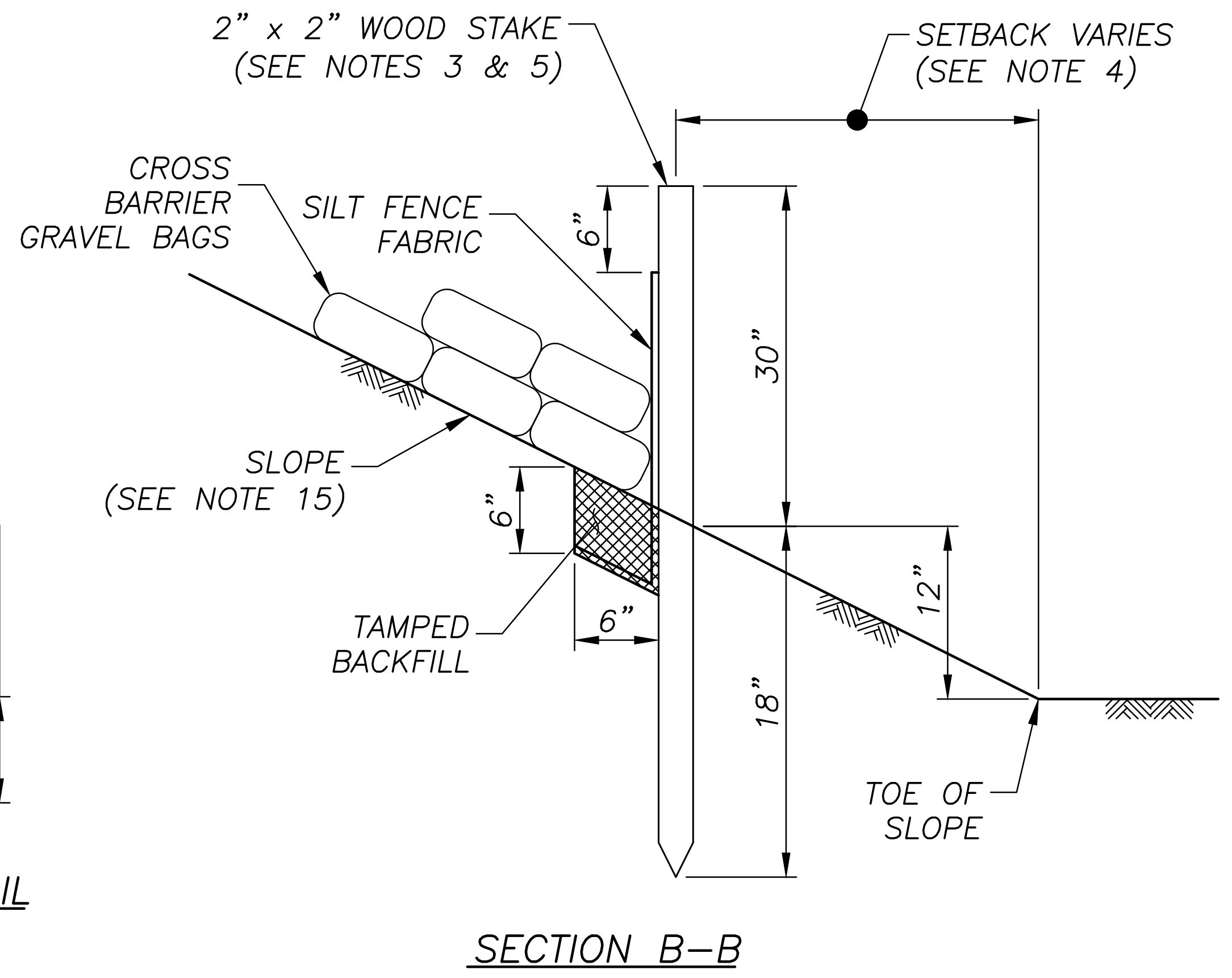
SECTION C–C



SECTION B–B

SILT FENCE INSTALL

NTS

4 D4

NO.	REVISION DESCRIPTION	DATE

PREPARED BY:

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PREPARED UNDER THE SUPERVISION OF

DATE

MENDOCINO COUNTY
SOUTH COAST LANDFILL

FINAL CLOSURE CONSTRUCTION PROJECT 2016
EROSION CONTROL DETAIL SHEET

DESIGNED BY : RMG	SCALE : AS SHOWN	PROJECT NO: –
DRAWN BY : CTD/DMW	DATE : 11–2016	
CHECKED BY :	DATE :	SHEET **12** OF **12**
APPROVED BY :	DATE :	

APPENDICES

APPENDIX A

FINAL CLOSURE EVALUATION
(GLA, SEPTEMBER 2003)

REVISED FINAL CLOSURE ANALYSIS
(GLA, NOVEMBER 2012)

FINAL CLOSURE EVALUATION
(GLA, SEPTEMBER 2003)

FINAL CLOSURE EVALUATION

SOUTH COAST LANDFILL
MENDOCINO COUNTY, CALIFORNIA

SEPTEMBER 2003

SUBMITTED TO:

County of Mendocino
Solid Waste Division
340 Lake Mendocino Drive
Ukiah, California 95482

PREPARED BY:

GeoLogic Associates
1831 Commercenter East
San Bernardino, California 92408
(909) 383-8728

FINAL CLOSURE EVALUATION

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Geologists, Hydrogeologists and Engineers

FINAL CLOSURE EVALUATION
SOUTH COAST LANDFILL
MENDOCINO COUNTY, CALIFORNIA

EXECUTIVE SUMMARY

Geotechnical analyses were completed to evaluate existing and proposed final closure construction conditions at the County of Mendocino's inactive South Coast Landfill (SCLF). Although preliminary analyses of the facility (EMCON Associates, 1998a, 1998b) had indicated potentially problematic slope stability conditions, these results were based on assumptions regarding the distribution and engineering properties of native materials underlying refuse at the site.

Considering these conditions, and to support identification of an appropriate final closure configuration for the SCLF, a subsurface exploratory and laboratory testing program was completed by GeoLogic Associates that included excavation of 4 hollow-stem auger borings around the perimeter of the landfill and direct shear testing of bedrock and soils. The data collected from the borings and laboratory test results indicate that the SCLF is underlain by fault gouge and alluvial/colluvial soils that have significantly greater strengths than were assumed in earlier study of the site.

Considering the high seasonal rainfall totals typical of the area, and in accordance with the engineered alternative configuration allowed by 27 CCR Section 20080(b), from bottom to top, the proposed final cover for the SCLF includes: a foundation layer consisting of the existing 20-inch thick landfill cover, a 60-mil textured LLDPE geomembrane barrier layer, a one-foot thick gravel drainage layer, and an 8-ounce geotextile layer to prevent piping of the overlying 2-foot thick vegetative soil layer.

Slope stability analyses of the landfill and proposed final cover were completed in accordance with CCR Title 27 Section 21090 and indicate that the stability of the landfill is adequate under both static and seismic loads. Displacement analyses of the landfill as a whole indicate dynamic displacement of up to 10 inches could occur under Maximum Probable Earthquake (MPE) loads for the worst case landfill configuration when high groundwater conditions are assumed within refuse. Displacement analyses of the proposed final cover system indicate that dynamic displacement should be less than 7.9 inches. Considering the elongation properties of the proposed low-linear density polyethylene (LLDPE) geomembrane barrier layer that will be employed in the landfill final cover, such displacement is considered acceptable. Since the thickness of saturated wastes will decrease through time after placement of the final cover, landfill stability conditions will also improve.

Settlement analyses completed for this project indicate that up to 5 feet of post-closure refuse settlement may occur. Since settlement will be accommodated across a fairly broad area of the final cover, and considering the elongation properties of the LLDPE geomembrane barrier layer, this condition is considered acceptable.

FINAL CLOSURE EVALUATION
SOUTH COAST LANDFILL
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FINAL CLOSURE EVALUATION
SOUTH COAST LANDFILL
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1.0 INTRODUCTION

1.1 GENERAL

This report and attachments present the results of geotechnical analyses that were completed by GeoLogic Associates (GLA) to evaluate existing slope stability conditions and to identify an appropriate final cover configuration for the County of Mendocino Solid Waste Division's (SWD's) South Coast Landfill (SCLF). Since the landfill is no longer active, it must be closed in accordance with requirements listed in California Code of Regulations (CCR) Title 27, Section 21750 f(5) for Class III landfills. As described herein, project work involved subsurface exploration and laboratory testing to better characterize the distribution and engineering properties of native soils, bedrock, and existing cover and stockpiled soils, geotechnical analyses of existing and proposed final closure conditions, and preparation of this report.

1.2 FACILITY BACKGROUND & LOCATION

The SCLF is an approximately 6-acre Class III landfill that is located on an approximately 47.65-acre parcel off Fish Rock Road in a remote portion of southwestern Mendocino County, California (Figure 1). Refuse disposal operations were conducted at the site from 1970 through November 2001 using cut and cover techniques. Except for its steep southwestern border, the topography of the landfill parcel is relatively flat to gently sloping. As determined in a field study completed by EMCON Associates (1998a), the limits of refuse along the southwestern perimeter of the facility meet the top of an approximately 100-foot high 2:1 (horizontal to vertical) native slope that borders the headwaters of the Little North Fork of the Gualala River (hereafter Little North Fork). The area surrounding the site is vegetated with a moderately dense growth of coniferous trees.

1.3 GEOLOGIC SETTING

The SCLF is located in the northern portion of the Coast Ranges geomorphic province, which is characterized by a series of northwest-southeast trending ridges and valleys that are associated with faults and folds that follow the same trend. Of great significance, the landfill overlies the San Andreas Fault Zone. The fault zone itself consists of fault gouge, a highly sheared and chaotic mix of bedrock units that crop out east and west of the site (Figure 2).

East of the landfill and east of the Little North Fork, Cretaceous-age marine sandstones and sheared shales of the Coastal Belt Franciscan Formation are the most dominant lithology. West of the site and west of the Little North Fork, marine mudstones and sandstones of the Cretaceous-age Anchor Bay Member of the Gualala Formation and marine sandstones of the Tertiary-age German Rancho Formation crop out. Within the relatively flat-lying central and eastern portions of the property, unconsolidated, well-graded Recent-age alluvial terrace deposits of mixed clays, silts, sands and gravel are exposed.

Although a number of landslides have been mapped near the site (Davenport, 1984; McKittrick, 1995), no landslide features have been identified on the SCLF property. Most of the large-scale landslides in the region have relatively deep-seated failure surfaces with a rotational/transitional mode of movement along planar joints or bedding. In many cases, slope failure appears to be related to erosional processes at the toe of slopes. The fact that landslides are not typically mapped within fault gouge in the area may be related to the nearly vertical textural fabric of shears within the unit. This inference is supported by information presented by McKittrick (1995), which indicates low to moderate landslide susceptibility on most of the SCLF property.

1.4 HYDROGEOLOGIC SETTING

Groundwater at the site occurs within fractured gouge zone materials. Data from previous investigation of the site (SHN, 1991) indicates that groundwater is encountered at depths ranging from 8 to 23 feet below ground surface. Along the west side of the property, groundwater may have been encountered in two recent borings that were excavated to depths of 17 and 12.5 feet, but was not encountered in two other borings that were extended to depths of 12 and 38.5 feet (Attachment A).

Figure 3 depicts groundwater equipotential contours developed using groundwater elevation data obtained in August and November 2002. As indicated, groundwater is interpreted to flow from the northeast to the southwest at a hydraulic gradient of approximately 0.08 ft./ft.. However, this pattern is expected to be locally interrupted by well-developed shears within the gouge zone matrix with resultant anisotropic flow directed in a more southerly direction.

1.5 PREVIOUS STUDIES

A Preliminary Seismicity and Slope Stability Analysis of the SCLF was completed by EMCON (1998b) to assist the County of Mendocino in developing a comparison cost estimate for a possible clean closure program. Concurrent with the stability analyses, EMCON (1998a) also completed a field investigation to determine the spatial distribution and limits of refuse at the site.

1.5.1 Landfill Limits

Field investigation of the landfill limits was completed by EMCON (1998a) using a backhoe to excavate test pits to evaluate the thickness of existing landfill cover and using a hollow-stem auger drill-rig to determine the thickness of wastes. The study concluded the following:

- Refuse is typically 25 to 35 feet thick throughout the landfill.

- Wastes have relatively uniform characteristics, typically consisting of mixed paper, wood, plastic, and minor metals.
- The ratio of wastes to soil is typically approximately 3:2, but ranges from about 7:3 to 1:9.
- Interim cover soils consist of gravelly clays and gravelly silts, consistent with borrow soils exposed southwest of the landfill.
- A relatively large mass (10,000 cubic yards [cy]) of soil and wood debris were encountered along the southern perimeter. This mass is essentially free of wastes other than wood and appears to have been derived from clearing and grubbing at the site.
- Leachate was present at the base of the refuse prism. Approximately 12 feet of saturated refuse existed along the northeastern side of the landfill, decreasing to about 6 feet toward the northwest side of the landfill. Refuse was not saturated within the southern areas of the landfill.
- Native materials underlying the refuse prism appear to consist of stiff to very stiff gravelly clays that appear to be consistent with fault gouge materials identified at the ground surface adjacent to the landfill and in GLA's exploratory borings (see Section 2.1).
- The total volume of refuse, associated cover soils, and potentially impacted soils at the base of refuse amounts to approximately 282,000 cy.

1.5.2 Preliminary Slope Stability Analyses

The preliminary slope stability analyses that were completed by EMCON (1998b) were performed using assumed soil/bedrock strength properties (i.e., without site-specific laboratory-determined strength properties). Instead, the analyses considered three "generic" soils/bedrock material types that EMCON considered possibly underlying the site including: 1) cohesionless sands, 2) low-strength clays, and 3) bedrock. Assumed soil strengths were identified based on review of boring log information, and assumed bedrock strengths were based on back-calculations of assumed landslide geometries present west of the landfill on the west-side of the Little North Fork. Since no topographic information was available, the stability analyses did not consider the geometry of the slope descending from the landfill toward the Little North Fork.

Based on the landfill geometry determined in the field investigation and using the assumed soil, bedrock and refuse strengths, EMCON completed two-dimensional static and pseudo-static stability analyses using the PCSTABL5 computer program. Dynamic displacement analyses were completed using the Newmark (1965) method. Given the landfill's location, the dynamic stability analyses integrated seismic loads associated with the maximum probable earthquake (MPE) event on the San Andreas Fault (Mw 8.0), with a peak horizontal ground acceleration of 0.9 g.

The preliminary stability analyses yielded the following results:

- With the exception of soil type 1 (clean sand) which yielded a factor of safety of 1.31 on one of the three cross-sections analyzed, the static of factor of safety for all soil types and cross-sections analyzed was greater than 1.5.
- For configurations that assumed a clean sand subgrade, the calculated dynamic displacement of the landfill ranged from approximately 7.9 to 15.7 feet.
- For configurations that assumed a clay soil subgrade, the calculated dynamic displacement of the landfill ranged from 0.1 to 23.3 feet.
- For configurations that assumed a bedrock subgrade, the calculated dynamic displacement of the landfill ranged from 0 to 1.6 feet.
- Of note, when bedrock was assumed to underlie the landfill, the "failure" surface was quite shallow and did not extend far from the toe of the landfill.

2.0 RECENT FIELD & LABORATORY STUDY

Recognizing that previous stability analyses had indicated that the engineering properties of subsurface soils could strongly affect the performance of the landfill under earthquake loading conditions, a limited field and laboratory investigation was completed by GLA to assess the distribution and engineering properties of native soils, bedrock, and existing cover soils.

2.1 SUBSURFACE INVESTIGATION

GeoLogic Associates' state-certified engineering geologist was mobilized to the site on October 22, 2002 to supervise excavation of backhoe test pits and excavation of exploratory boreholes by hollow-stem auger drill rig. As shown in Attachment A, the geologist continuously observed the excavation work, logged the test pits and borings, and collected both bulk and relatively undisturbed samples of native soils and bedrock for subsequent laboratory analyses.

2.1.1 Existing Cover Evaluation

As shown on Figure 4, twenty (20) test pits were excavated through existing cover soils throughout the landfill footprint to assess the thickness, character, and potential utility of the cover soils. Cover soils were found to be relatively uniform, consisting of gravelly silt with sand and clay, and range from approximately 6 inches to greater than 96 inches thick, with only a small area near the northeast corner of the landfill where cover soils were less than 20 inches thick. Cover soils over most areas are at least 20 inches thick.

2.1.2 Stockpiled Soils

Two (2) test pits were excavated in stockpiled soils that were placed along the northern border of the landfill. These soils were generated through excavation of fault gouge materials during construction of the new refuse transfer station, and appear to represent the types of soils that can be expected if excavation by conventional equipment continues adjacent to the site. As shown in the test pit logs included in Attachment A, the stockpile soils typically consist of gravelly silt and sand with clay, with approximately 10% having grain sizes larger than one-inch.

2.1.3 Exploratory Boreholes

Recognizing the critical location of the landfill adjacent to its westerly slope, four (4) exploratory boreholes were excavated around the north and west sides of the landfill (Figure 4) to better characterize the distribution and engineering properties of native soils underlying and adjacent to the landfill. The boreholes were excavated to depths ranging from 12 to 38.5 feet, and relatively undisturbed samples were retained at 5-foot vertical intervals (Attachment A).

Colluvial/Alluvial soils were identified in only one of the borings (B-2). Well-consolidated fault gouge materials were encountered below a thin veneer of fill soils in two borings, and fault gouge was exposed at the ground surface in the fourth boring (Attachment A). The colluvial/alluvial soils encountered in boring B-2 consist of clayey silt with gravel and are consistent with native soils exposed in borrow soil cut slopes south of the landfill and on eroded slopes on the west side of the property. Similarly, the shaley gouge materials encountered in all the borings are consistent with bedrock materials exposed at the ground surface north of the landfill near the new refuse transfer station.

2.2 LABORATORY TESTING

2.2.1 Soils Test Methods

Laboratory testing was completed by GLA to evaluate the engineering properties of stockpiled borrow soils on the north side of the landfill, and of native soil and bedrock materials underlying and adjacent to the SCLF. As shown in Attachment B, testing included the following:

- Maximum density / optimum moisture content testing (ASTM D1557) was completed to identify appropriate densities and moisture contents to remold select fault gouge and stockpile/cover soils for subsequent permeability and direct shear testing.

- Plasticity Index testing (ASTM D4318) was completed to better characterize site materials. As shown in Attachment B, with results plotting on the "A Line", the plasticity tests indicate that stockpile and cover soils should be considered low-plasticity silty clays and clayey silts.

- Constant-head permeability tests (ASTM D2434) were performed to evaluate the hydraulic conductivity properties of remolded stockpiled soils and existing landfill cover soils. As shown in Attachment B, the results of these analyses indicate that, at 90 percent relative compaction, the existing stock pile soils do not satisfy 27 CCR requirements for final cover (i.e., at about 3×10^{-6} cm/s, they are greater than the standard of 1×10^{-6} cm/s). However, when samples were compacted to 93% relative compaction, the permeability results (6 x 10^{-7} cm/s) did satisfy the prescriptive standard.

- Direct shear testing (ASTM D3080) was performed to identify the shear strength properties of native soils and fault gouge. As shown in Attachment B, the fault gouge tests examined representative, low-strength materials, and addressed their peak, ultimate and residual shear strengths.

2.2.2 Interface Shear Testing

Laboratory interface direct shear testing was completed by Precision Geosynthetic Laboratories to evaluate the interface strengths that can be expected between the low-linear density polyethylene (LLDPE) geomembrane and subgrade foundation layer soils and overlying gravel drainage media planned for use in the final cover. Two samples of foundation layer soils were obtained for testing from the existing soils stockpile on site. One sample of rounded pea gravel that may be available locally for the project was obtained and used for the analyses. The analyses were completed using double-sided textured 60-mil LLDPE obtained from GSE of Houston, Texas. As shown in Attachment B, the interface tests were completed in accordance with ASTM Method D6243-98 and D5321-92 with soils compacted to 90% relative compaction at a wetted condition. The analyses addressed the peak and residual (2.6-inch displacement) interface strengths.

3.0 SEISMICITY

In accordance with 27 CCR Section 20240, seismicity hazard review was completed to verify the earthquake parameters that could affect slope stability conditions at the site in the event of the maximum probable earthquake (MPE) or the largest recorded (historic) earthquake event, whichever is larger.

Using the computer program EQFAULT (Blake, 2000), the MPE of the north coast segment of the San Andreas fault and the 1906 San Francisco event were estimated to be approximately M_w=7.6 to 7.9. Assuming a focal mechanism 2 kilometers distant from the site and attenuation relationships by Abrahamson and Silva (1997), the associated peak ground acceleration associated with the MPE and the 1906 San Francisco Earthquake event is approximately 0.9g. This value is consistent with earlier evaluations of the site (EMCON, 1998b), and is considered the design seismic load for the project.

4.0 SLOPE STABILITY ANALYSES

4.1 MATERIAL STRENGTH PROPERTIES

4.1.1 Gross Stability Parameters

From bottom to top, the materials that govern the gross stability of the existing landfill configuration include: native bedrock (fault gouge), alluvial terrace deposits, and municipal wastes. As determined in the laboratory analyses described here in Section

2.0 and detailed in Attachment B, the engineering properties for these materials are as follows:

Table 1
Gross Stability Parameters

Material	Sat. Unit Weight (lb./cu. Ft.)	Angle Internal Friction (degrees)	Cohesion (lb./sq. ft.)
Fault gouge (1)	135	32	750
Alluvial Terrace soils (2)	130	33	600
Municipal Refuse (3)	80	30	200

(1) – Average direct shear results (Attachment B)
(2) – Average direct shear results (Attachment B)
(3) Shallow refuse fill (Singh & Murphy, 1990)

4.1.2 Cover Stability Parameters

As shown on Figure 5 and listed below, from bottom to top, the materials that are planed for use in construction of the facility's final cover include: a 2-foot thick earth fill foundation layer (existing), a linear low-density polyethylene (LLDPE) geomembrane, a one-foot thick gravel drainage layer, and an 8-ounce non-woven geotextile layer to prevent piping of soils from the overlying one-foot thick earth fill vegetation layer.

Based on manufacturer's specifications, recent testing and published data, the engineering properties that were assumed for these materials and the interfaces between individual elements are as follows:

Table 2
Landfill Cover Parameters

Material	Unit Weight (lb./cu. Ft.)	Angle Internal Friction (degrees)	Cohesion or Adhesion (lb./sq. ft.)
Foundation Layer Soils (1)	130	27	800
Foundation Layer / Textured LLDPE (2)		30.6	100
Textured LLDPE / Gravel (3)		35.9	64
Gravel / Geotextile (4)		35.9	0
Geotextile / Vegetative Layer Soil (5)		30.6	0
Vegetative Layer (1)	130	27	800

(1) - Direct shear results for existing stockpile soils (Attachment B)
(2) – Interface shear results for existing stockpile soils and LLDPE (Attachment B). Adhesion neglected in analyses.
(3) – Interface shear results for gravel and LLDPE (Attachment B). Adhesion neglected in analyses.
(4) – Assumed equal to gravel/LLDPE interface
(5) – Assumed equal to foundation layer/LLDPE

Although not employed in the stability analyses described in Section 4.3, additional strength is provided in the proposed final cover by the tensile strength of the geotextile and LLDPE geomembrane components.

4.2 GROSS LANDFILL STABILITY

4.2.1 Existing Static Condition

The gross stability of the current landfill configuration was analyzed using the computer program SLOPE/W (Geo-Slope, 1995). The primary analytical method used in the analyses was the Spencer and Morgenstern/Price method for general failure modes including circular and non-circular failure surfaces. As shown in Attachment C, the graphical output produced by SLOPE/W indicates critical geometries and material properties included in the analyses.

Both static and pseudo-static (seismic) analyses were completed using limit equilibrium procedures to compute factors of safety against failure. The analyses considered the highest and steepest landfill geometry, which is located near the southwest corner of the property (cross-section B-B'; Figure 6), and the section with the steepest off-site slope (cross-section A-A', Figure 6). Considering that the 2:1 (horizontal:vertical) north-facing slope along the northern side of the landfill is not as high as the configurations evaluated for cross-sections A-A' and B-B", it is not considered a critical geometry. As described in Section 4.3, cross-sections C-C', D-D' and E-E' were also developed to evaluate landfill cover conditions along the western edge of the landfill and to assess the planned road reconstruction and gabion wall construction along the western perimeter of the landfill. Cross-section locationss are indicated on Figure 6. The cross sections are presented in Attachment C.

The subgrade geometry beneath the landfill was estimated based on review of the subsurface investigation results obtained by EMCON (1998a). Groundwater was integrated in the analyses using the data presented on Figure 4 (for one set of calculations) and also using conservatively estimated conditions (in an additional set of calculations) that assume that groundwater exists at a height of 10 feet above the base of refuse within the center of the refuse prism, "daylighting" at the edge of refuse.

While the materials that were encountered within the exploratory borings excavated for this project suggest that the landfill directly overlies fault gouge (and that alluvial / colluvial soils that may once existed on the relatively flat terrace area had been removed for use as refuse daily cover), the stability analyses considered conditions where both fault gouge and alluvial / colluvial soils may underlie the site. Considering the lack of soils encountered in the exploratory borings on the site, the "worst case" foundation condition was conservatively considered to include approximately 15 feet of soils along the outside (west side) edge of refuse with soils thinning toward the center of the landfill (Attachment C).

As shown in Attachment C, the gross stability analyses indicate that the most critical failure geometry is a roughly arcuate surface along Section A-A' on the west face of the landfill where a factor of safety of about 1.9 was calculated for cases where the landfill is underlain by either gouge or colluvial soils and where the current groundwater elevation condition (Figure 3) is assumed. When the analyses consider the potential high groundwater condition within wastes, the factor of safety was reduced to about 1.84.

4.2.2 Potential Dynamic Displacement of Existing Landfill Configuration

Potential earthquake-induced permanent displacement was calculated using the procedure described by Bray et al (1998) for the maximum ground acceleration anticipated at the site during a MPE event. As discussed in Section 3.0, the MPE for the north coast segment of the San Andreas is estimated to be approximately M_w=7.9 with a corresponding peak ground acceleration of 0.9g.

The displacement analyses first identified the yield acceleration (K_y ; the seismic load required to bring the factor of safety to 1.0). Using procedures identified by Bray et al (1998), the yield acceleration is then related to MPE-level seismic shaking, duration and horizontal acceleration to determine the magnitude of displacement along the failure plane (Attachment C).

Due to the period of the waste, the design earthquake acceleration is attenuated and the maximum horizontal equivalent acceleration in the waste is less than the acceleration in the rock of 0.9g. For the critical failure surfaces identified above, dynamic displacement is calculated to be approximately 7 inches for cases where the landfill is underlain by either gouge or colluvial soils and where the current groundwater elevation condition (Figure 3) is assumed.

Dynamic displacement analyses were also performed for the slopes below the landfill. Since attenuation within refuse does not occur outside the landfill, the maximum horizontal ground acceleration that was considered for this case was greater than for the landfill displacement analyses. The calculated displacement of slopes below the landfill ranged from 2.5 to 7.0 inches, with most displacement occurring well downslope of the landfill.

As indicated above, the magnitude of the calculated displacements are relatively minor. Calculated displacement is greatest for the high groundwater level case, where up to 10 feet of refuse may be saturated. This condition may be mitigated by placement of the final cover, which will reduce infiltration of seasonal precipitation to wastes. If needed, additional waters could be removed by pumping the gas collection wells that will be installed through refuse.

4.3 LANDFILL FINAL COVER STABILITY

4.3.1 Final Cover Design

CCR Title 27 Section 21090 requires that "operators shall ensure the integrity of final slopes under both static and dynamic conditions to protect public health and safety and prevent damage to post-closure land uses, . . . prevent public contact with leachate, and prevent exposure of waste." In addition, 27 CCR Section 21750 f(5) requires that "the stability analyses shall ensure the integrity of the Unit, including its foundation, final slopes, and containment systems under both static and dynamic conditions throughout the Unit's life, closure period, and post-closure period." For final closure design, the standard of practice with regard to these requirements has typically meant that landfill slopes should have a static factor of safety of approximately 1.5 and dynamic displacement during the MPE that does not impair the integrity of the final cover.

The prescriptive final cover components listed for unlined landfills in 27 CCR Section 21090 include: a two-foot thick foundation layer, a one-foot thick clay barrier layer whose hydraulic conductivity does not exceed 1 x 10^{-6} centimeters per second (cm/sec), and a one-foot thick vegetative soil layer. Considering the high seasonal rainfall totals typical of the area, and in accordance with the engineered alternative configuration allowed by 27 CCR Section 20080(b), from bottom to top, the final cover for the SCLF was assumed to include: a foundation layer consisting of the existing 20-inch thick landfill cover, a 60-mil textured LLDPE geomembrane barrier layer, a one-foot thick gravel drainage layer, and an 8-ounce geotextile layer to prevent piping of the overlying 2-foot thick vegetative soil layer (Figure 5). In order to intercept leachate at known seep locations, a geonet drainage layer will also be placed below the LLDPE at strategic locations. Since this layer will not be continuous (maximum dimensions of 15 feet high by 30 feet long), its presence below the final cover system will not affect the stability of the final cover.

As detailed below, together, these alternative final cover elements are anticipated to provide enhanced protection against infiltrating rain water, and suitable support to withstand earthquake loads associated with the MPE on the San Andreas fault.

4.3.2 Static Stability of Proposed Final Cover

The stability of the proposed final cover system was considered addressing both the steepest (30-foot high 3.1:1 [Section C-C']) and highest (45-foot high, 3.3:1 [Section E-E']) slopes that currently exist on the landfill. These analyses were performed using the limit equilibrium procedures identified by Kramer (1999) and using the interface shear strength properties (friction only; cohesion/adhesion is ignored) of the individual and combined (interface) cover components listed above in Table 2. Since the final cover vegetative layer will be drained by the underlying gravel drainage layer, the analyses assumed that the vegetative layer soils would have a saturated weight (130 pcf), but without pore pressures.

As shown on Table 2, the lowest interface strength occurs between the LLDPE geomembrane and the subgrade foundation layer soils, and this is considered the critical failure surface within the proposed final cover geometry. As shown in Attachment C, the lowest static factor of safety for the proposed final cover is 1.9. The static factor of safety for the other final cover interfaces is greater than 2.0.

4.3.3 Dynamic Stability of Proposed Final Cover

The potential dynamic displacement of the final cover was evaluated using the procedure described by Bray et al (1998) and taking into account the peak horizontal ground acceleration associated with the MPE on the San Andreas fault (0.9g) and the calculated yield acceleration for the proposed final cover configuration. As shown in Attachment C, dynamic displacement of the final cover configuration would amount to a maximum of about 7.9 inches. Considering the elongation properties of the LLDPE, the integrity of the final cover barrier layer is maintained and such displacement is considered acceptable (Seed and Bonaparte, 1992).

Since the landfill is positioned over the San Andreas fault, in the event of an large earthquake whose focal mechanism is close to the site, ground rupture could occur. The probability of such an occurrence is regarded as considerably smaller than the possibility of an MPE event, and would be largely mitigated by the elastic properties of the refuse and cover materials. While such an event could still result in distress to the final cover, interim use of re-enforced visqueen to prevent rainwater infiltration, and standard soil and geosynthetic cover repair operations could be employed to mitigate this condition.

4.3.4 Stability of Western Perimeter Roadway

In order to install and maintain a maintenance roadway along the western border of the landfill, municipal refuse materials below the existing roadway will be removed and replaced with compacted fill materials. In areas where waste exists at a slope gradient steeper than 3:1, refuse will be excavated to yield a waste slope no steeper than 3:1. Excavated refuse materials are not expected to be greater than about 3000 cubic yards, which will be placed and covered on the landfill deck area.

As shown on cross-sections C-C' and D-D', roadway reconstruction will also involve installation of a rock-filled gabion wall with geogrid-reinforced soil backfill between the wall and refuse to stabilize the soils against erosion and dynamic displacement. The new roadway fill soils will be keyed into native materials and will include horizontally placed geogrid reinforcement materials on approximately 3-foot vertical intervals (see cross-sections C-C' and D-D' in Attachment C). A maximum wall height of approximately 6 feet is anticipated. The geogrid used for construction will have a minimum long term, creep-reduced strength of 1,000 pounds per foot, and will have a minimum width equal to the new compacted fill width. Geogrids will be laid between the gabion baskets with a minimum of 6 inches of geogrid exposed at the front face of wall to monitor for geogrid pull-out.

As shown in Attachment C, the static stability of the gabion wall and roadway is no less than 2.05, and the pseudo-static stability of the fill is no less than 1.53. With the additional tensile strength provided by the geogrid materials, the calculated minimum static stability for potential failure below the roadway is 2.64 and the pseudo-static stability for this configuration is 1.88.

Since the roadway fill will effectively buttress refuse and final cover materials, the gross landfill and final cover slope stability conditions will actually be slightly higher than described above in Sections 4.2 and 4.3.

4.3.5 Anchor Trench Design

Calculations were completed for design of the anchor trench for the liner cover (Attachment C). The calculations were performed assuming a 45-foot high slope configured variously at 2:1 and 3.7: 1. Calculations indicate that a 2 foot wide trench at a depth of 2 feet will provide a minimum factor of safety of 2.2.

5.0 SETTLEMENT ANALYSIS

Settlement analyses were completed to evaluate how landfill grades could change in the future as refuse materials biodegrade and consolidate. While the refuse settlement calculations considered herein are expected to provide an indication of landfill elevation performance, in order to account for the actual refuse placement history at the site, additional calculations will be performed later as part of landfill closure work taking into account the types, volumes, areas, and dates of refuse placement within the WEA.

5.1 FOUNDATION SETTLEMENT

Since the landfill appears to be founded on native bedrock materials and compressible soils (such as colluvial and alluvial soils) appear to have been largely removed for use as daily and interim cover soils, no significant settlement of the foundation materials underlying the SCLF is anticipated.

5.2 REFUSE SETTLEMENT

The mechanics of refuse settlement are complex due to the extreme heterogeneity of refuse fill. According to Edil et al. (1989), the main mechanisms involved in refuse settlement are:

- Mechanical distortion (bending, crushing, and reorientation)
- Raveling (movement of fines into large voids)
- Physical-chemical changes (corrosion, oxidation, and combustion)
- Biochemical decomposition (fermentation and decay)

The magnitude of refuse settlement can thus be inferred to be a function of: (1) initial refuse density or solid/void ratio, (2) overall density of the refuse prism or ratio of refuse to daily cover soil, (3) content of decomposable materials in the refuse, (4) thickness of refuse lifts and total height of the refuse prism, (5) stress history, (6) time elapsed since each individual lift was placed, and (7) environmental factors such as moisture content, temperature, and gas content.

Based on the work of Huitric (1981), settlement can be modeled as an exponential decay function of the form:

$$\text{Remaining settlement} = \mathbf{a}Te^{-\mathbf{b}t}$$

Where **a** and **b** are constants such that total expected settlement is a proportion **a** of the original thickness, T, of a particular lift of refuse, and the rate of settlement decays at an exponential rate of **bt**, where t is the number of years elapsed since the particular lift of refuse was placed. For a municipal landfill with standard compaction equipment **a** varies between 0.2 and 0.35, and **b** varies between 0.10 and 0.11. For the SCLF analysis, intermediate values of 0.3 and 0.105 were used, respectively.

The most consistent refuse settlement estimates are obtained by modeling the refuse prism as a 3-dimensional net, calculating the settlement at each node of the net with the above time-dependant exponential decay function, and adding the total settlement for each node of the net.

To estimate the historical rates of refuse accumulation, a two-dimensional grid was established over the footprint of the proposed SCLF refuse prism, with a nodal spacing of about 50 feet. The third dimension in the model net is the net change in elevation between discrete time intervals. Inasmuch as development of the landfill footprint and fill elevations are not well-understood, for the purpose of this evaluation, it was assumed that refuse filling occurred relatively evenly over the entire footprint.

Based on the criteria described above, refuse settlement within the SCLF was calculated. As expected, the greatest settlement is expected to occur in areas where refuse thicknesses are greatest (i.e., within the center of the SCLF). Figure 6 depicts calculated post-settlement elevation contours. Comparison with final fill grades indicates that post-closure settlement could be as great as 5 feet within the center of the refuse fill. However, considering the elongation properties typical of the LLDPE geomembranes (e.g., >300%), this long-term settlement is not expected to affect the integrity of the landfill cover system.

6.0 CONCLUSIONS AND RECOMMENDATIONS

The geotechnical analyses that were completed for this project indicate that the existing stability of the SCLF is not as problematic as suggested in the preliminary analyses that were completed by EMCON (1998b). EMCON's results were based on assumed soil and bedrock strength parameters and did not take into account actual subsurface conditions. The exploratory borehole and laboratory test data that were obtained for the current project indicate that the landfill is underlain by gouge and soil materials having significantly higher strengths than were integrated in the earlier analyses.

Based on the recently obtained laboratory-determined material strengths, GLA concludes that the gross stability of the landfill is adequate under both static and seismic loads. Dynamic displacement of only up to 10 inches is calculated for the worst case cross-section when high groundwater conditions are assumed within refuse. Such displacement is considered acceptable considering the elongation properties of the final cover geomembrane. This condition will be further mitigated by the fact that the thickness of saturated wastes will decrease through time and associated stability conditions will improve after placement of the final cover.

The analyses that were completed for this project indicate that stability of the proposed final cover configuration is also adequate under both static and seismic loads. The maximum calculated seismic displacement of the final cover (7.9 inches) is considered acceptable and could be accommodated by the geomembrane barrier layer of the final cover.

Finally, the settlement analyses that were completed for this project indicate that up to 5 feet of post-closure refuse settlement may occur. Since settlement will be accommodated across a fairly broad area of the final cover, and considering the elongation properties of the LLDPE geomembrane barrier layer, this condition is considered acceptable.

7.0 CLOSURE

This report is based on the project as described and the limited geotechnical data obtained in this and earlier studies of the South Coast Landfill. Our firm should be notified of any pertinent change in the project plans or if conditions are found that differ from those described in this report, since this may require a revaluation of the conclusions and recommendations presented herein.

This report has not been prepared for use by parties or projects other than those named or described above. It may not contain sufficient information for other parties or other purposes. This report has been prepared in accordance with generally accepted hydrogeologic and geotechnical practices and makes no other warranties, either express or implied, as to the professional advise or data included in it.

GeoLogic Associates

for Joseph G. Franzone, PE, GE
Supervising Geotechnical Engineer

for: Ralph A. Murphy, CEG, CHg.
Principal Engineering Geologist

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FIGURES

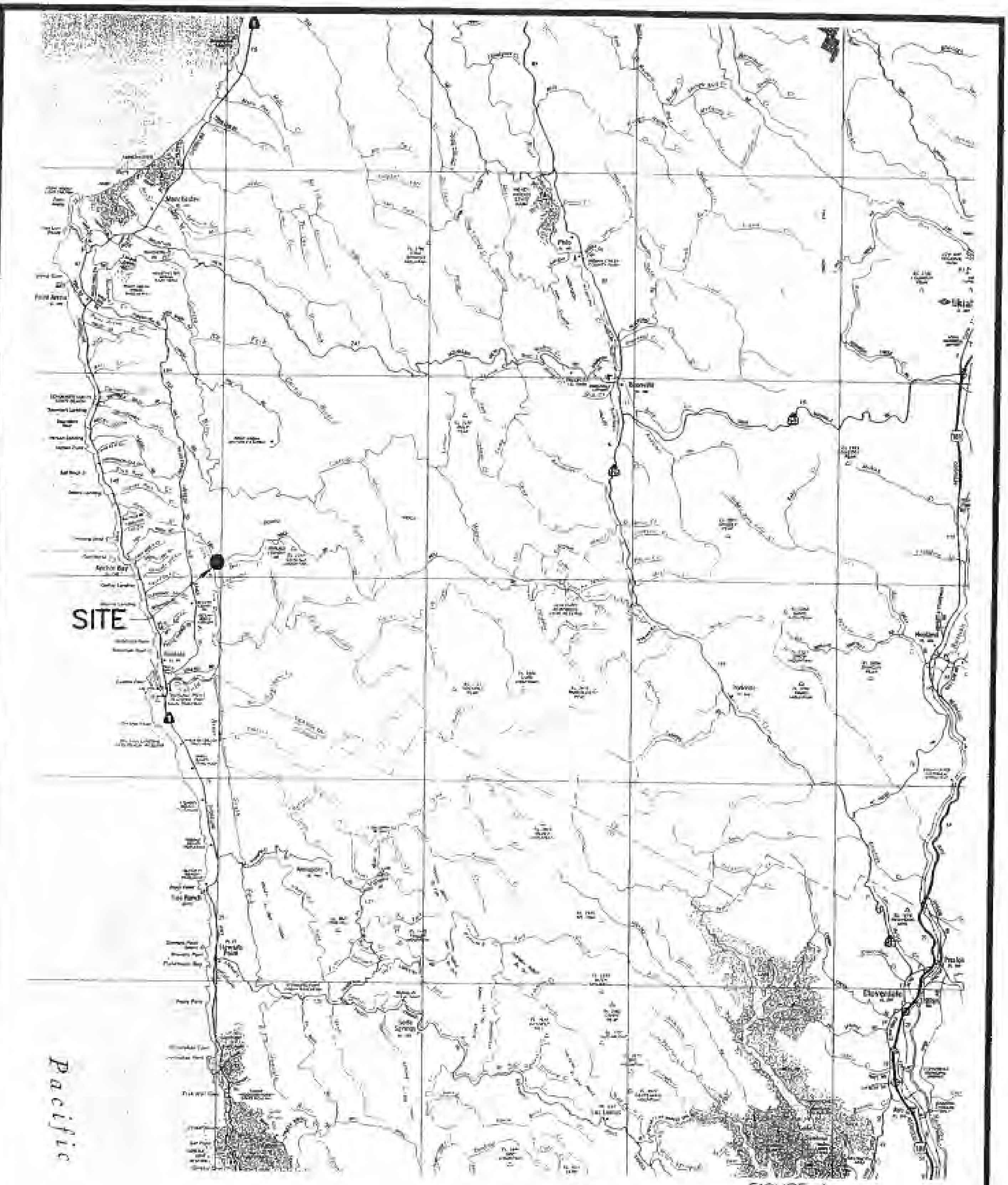


FIGURE 1

APPROXIMATE SCALE 1" = 4.65 MILES

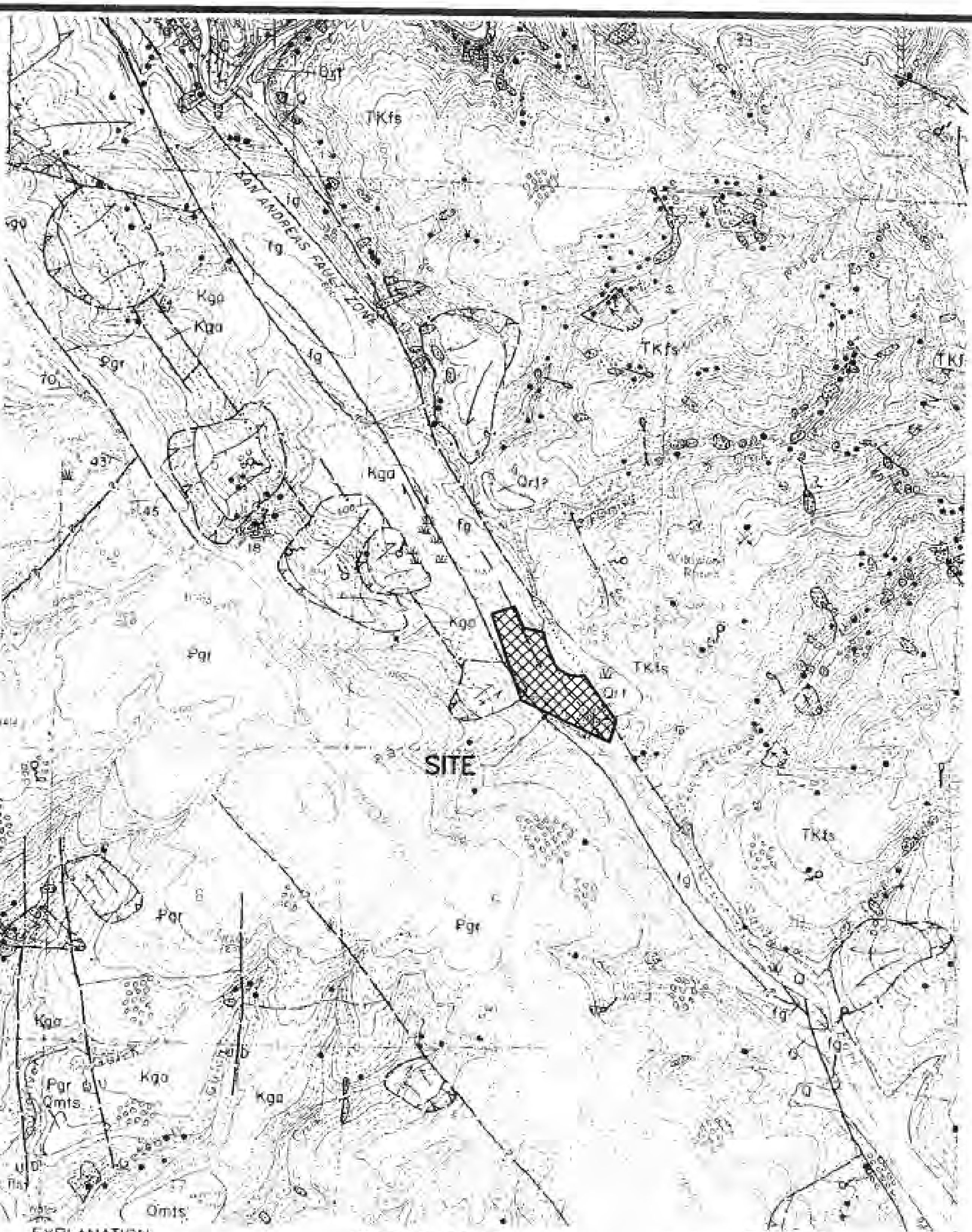
N

REGIONAL LOCATION MAP

FINAL CLOSURE/POST-CLOSURE MAINTENANCE PLAN
SOUTH COAST LANDFILL
COUNTY OF MENDOCINO, CA

GeoLogic Associates
Geologists, Hydrogeologists, and Engineers

DRAWN BY	DATE	JOB NO.
VL	FEBRUARY 2003	2002-072



EXPLANATION:

Qrt - ALLUVIAL TERRACE
fg - FAULT GOUGE
Pgr - GERMAN RANCHO FORMATION
Kga - GUALALA FORMATION
TKfs - COASTAL BELT FRANCISCAN FORMATION

REFERENCE: CDMG, DMG OPEN-FILE REPORT 84-48, GEOLOGY AND GEOMORPHIC FEATURES RELATED TO LANDSLIDING, GUALALA 7.5-MINUTE QUADRANGLE, MENDOCINO COUNTY, CA (1984)

FIGURE 2

GEOLOGIC MAP

FINAL CLOSURE/POST-CLOSURE MAINTENANCE PLAN
SOUTH COAST LANDFILL
COUNTY OF MENDOCINO, CA

GeoLogic Associates
Geologists, Hydrogeologists, and Engineers

DRAWN BY:	DATE:	JOB NO.
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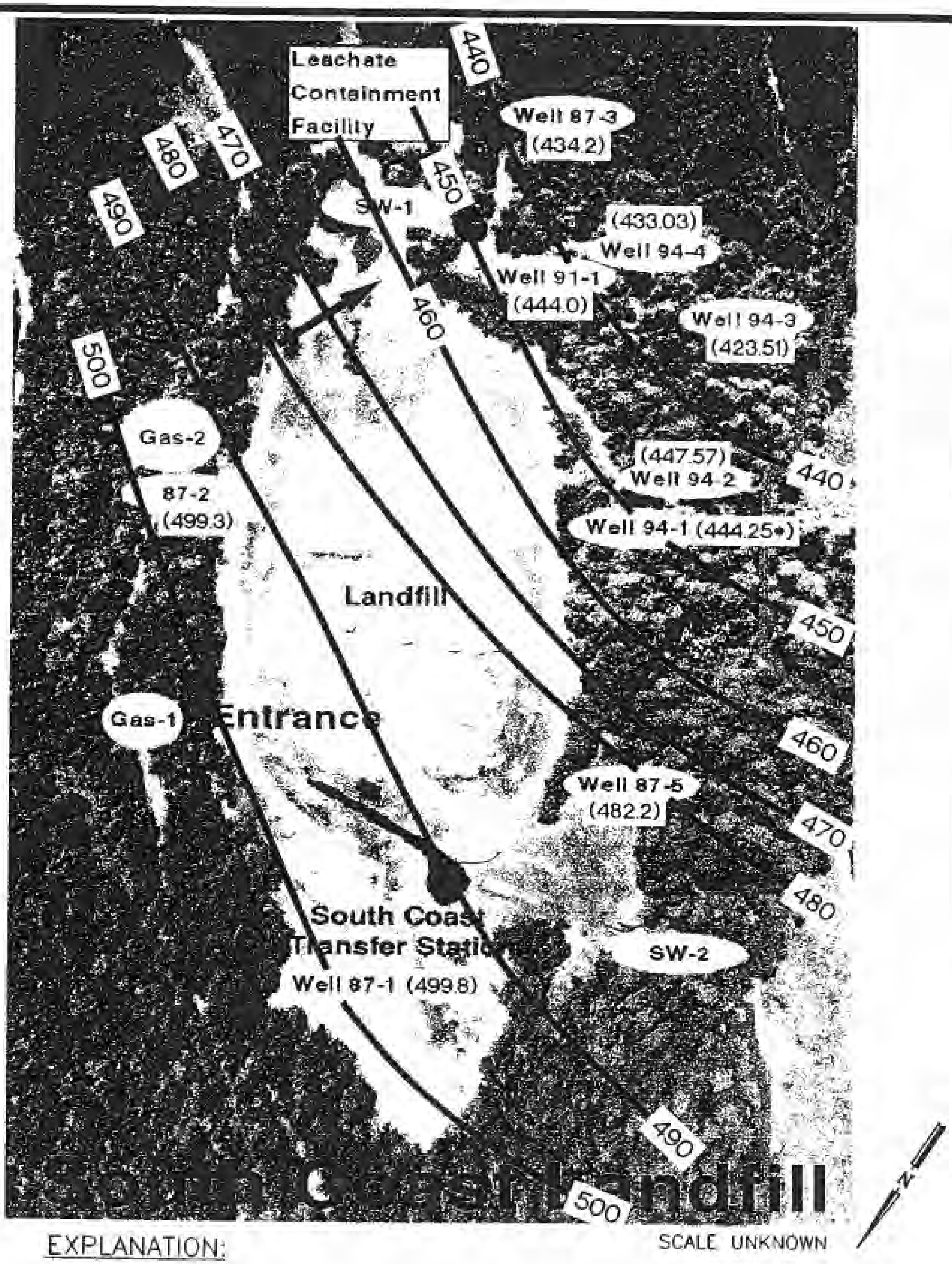

EXPLANATION:

(499.8)	GROUNDWATER ELEVATION (FEET ABOVE MSL)
460	CONTOUR LINE SHOWING GROUNDWATER POTENTIOMETRIC SURFACE ELEVATIONS (CONTOUR INTERVAL = 10 FEET)
→	DIRECTION OF GROUNDWATER FLOW
(444.25*)	DATA NOT USED TO CONSTRUCT CONTOURS

FIGURE 3

SITE MAP AND GROUNDWATER EQUIPOTENTIAL PLAN

FINAL CLOSURE/POST-CLOSURE MAINTENANCE PLAN
SOUTH COAST LANDFILL
COUNTY OF MENDOCINO, CA

GeoLogic Associates
Geologists, Hydrogeologists, and Engineers

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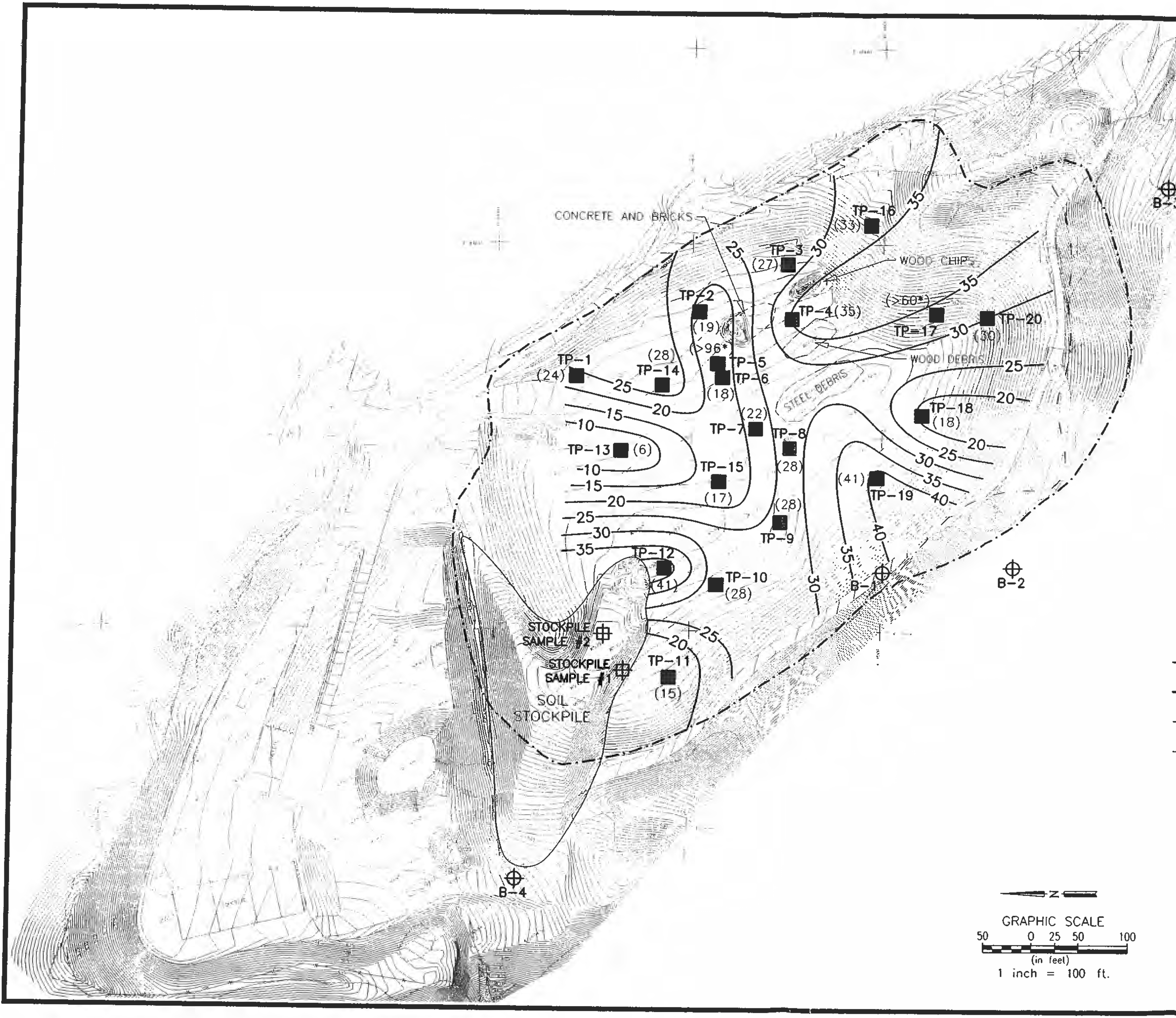

EXPLANATION:

⌖	GEOTECHNICAL BORING LOCATION
■	TEST PIT LOCATION
⊞	STOCKPILE SAMPLE LOCATION
(15)	SOIL COVER THICKNESS (INCHES)
*	DATA NOT USED TO DEVELOP CONTOURS
———	LINE OF EQUAL SOIL COVER THICKNESS (INCHES)
—·—·—	APPROXIMATE LIMIT OF REFUSE
—··—··—	APPROXIMATE LIMIT OF SOIL AND WOOD DEBRIS
———	APPROXIMATE LIMIT OF SOIL STOCKPILE

FIGURE 4

SUBSURFACE INVESTIGATION LOCATION PLAN

FINAL CLOSURE/POST-CLOSURE MAINTENANCE PLAN
SOUTH COAST LANDFILL
COUNTY OF MENDOCINO, CA

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DRAWN BY:	DATE:	JOB NO
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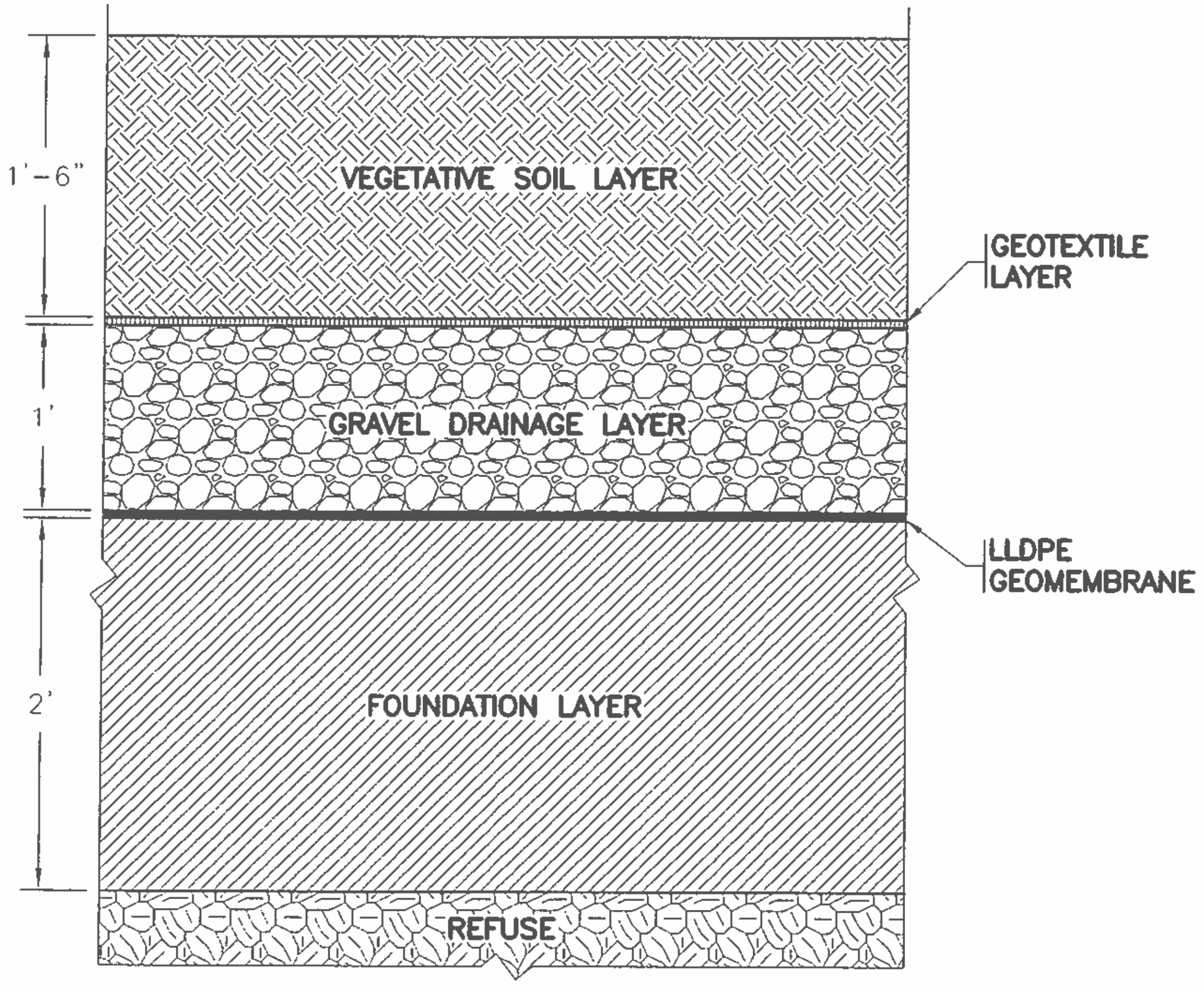

NOT TO SCALE

FIGURE 5

FINAL COVER SECTION

FINAL CLOSURE/POST-CLOSURE MAINTENANCE PLAN
SOUTH COAST LANDFILL
COUNTY OF MENDOCINO, CA

GeoLogic Associates
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DRAWN BY:	DATE:	JOB NO.
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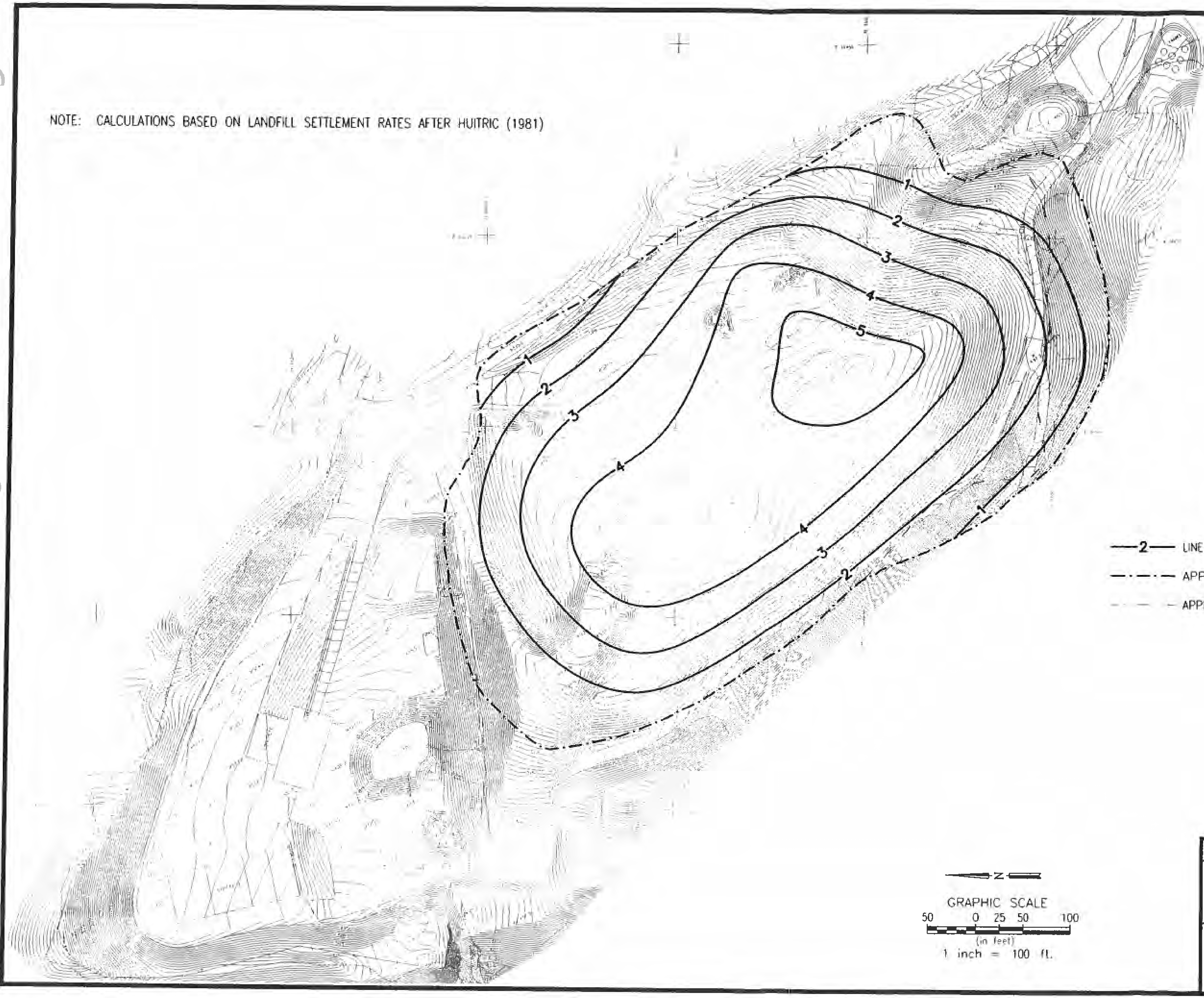
NOTE: CALCULATIONS BASED ON LANDFILL SETTLEMENT RATES AFTER HUITRIC (1981)



EXPLANATION:

—2— LINE OF EQUAL CALCULATED LANDFILL SETTLEMENT (FEET)
—·—·— APPROXIMATE LIMIT OF REFUSE
– – – APPROXIMATE LIMIT OF SOIL AND WOOD DEBRIS

FIGURE 6

CALCULATED LANDFILL SETTLEMENT AT TIME OF CLOSURE

FINAL CLOSURE/POST-CLOSURE MAINTENANCE PLAN
SOUTH COAST LANDFILL
COUNTY OF MENDOCINO, CA

GeoLogic Associates
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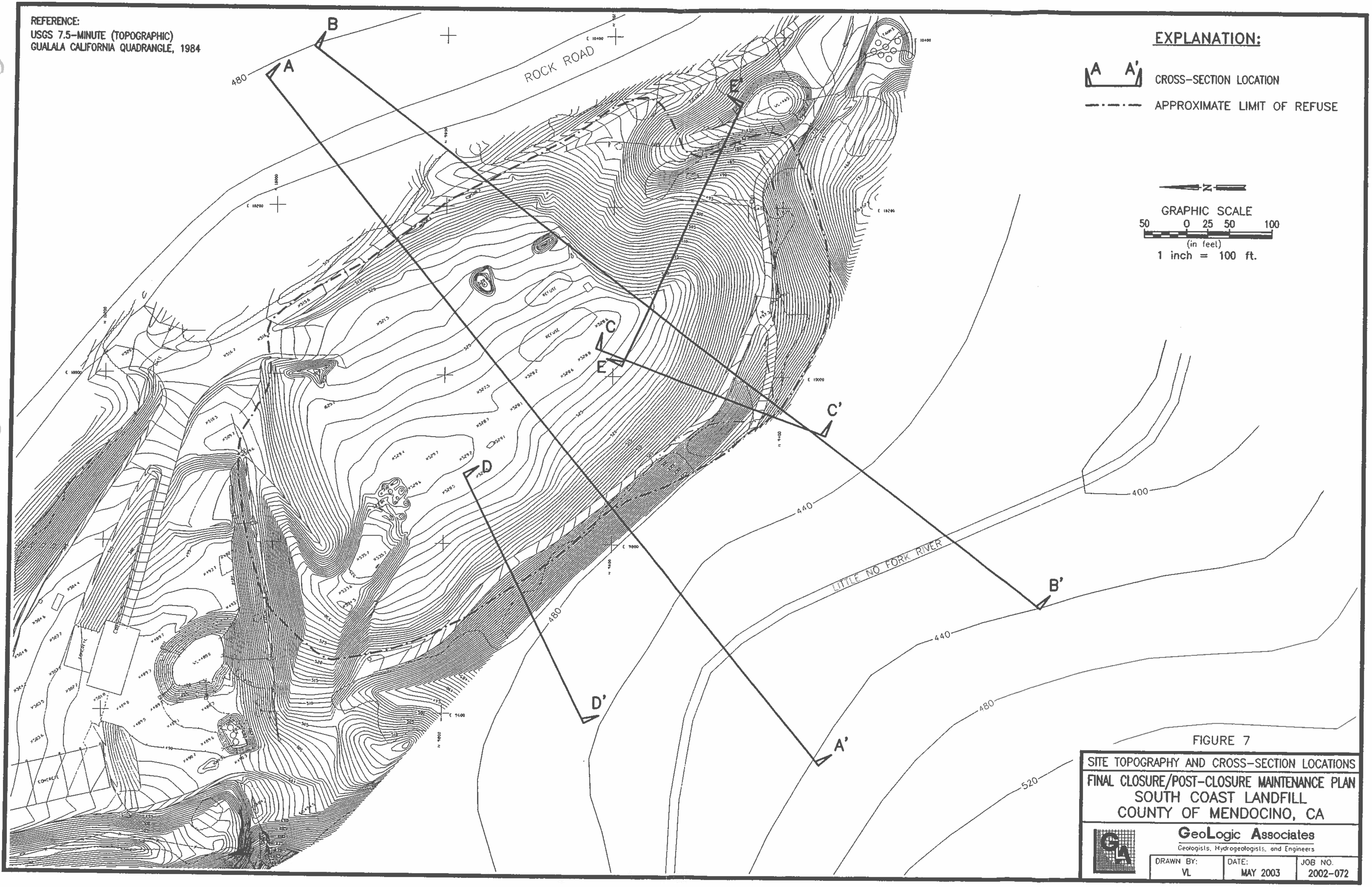
DRAWN BY:	DATE:	JOB NO.
VL	FEBRUARY 2003	2002-072



REFERENCE:
USGS 7.5-MINUTE (TOPOGRAPHIC)
GUALALA CALIFORNIA QUADRANGLE, 1984

EXPLANATION:

A A' CROSS-SECTION LOCATION

—·—·— APPROXIMATE LIMIT OF REFUSE

FIGURE 7

SITE TOPOGRAPHY AND CROSS-SECTION LOCATIONS

FINAL CLOSURE/POST-CLOSURE MAINTENANCE PLAN
SOUTH COAST LANDFILL
COUNTY OF MENDOCINO, CA

GeoLogic Associates
Geologists, Hydrogeologists, and Engineers

DRAWN BY:	DATE:	JOB NO.
VL	MAY 2003	2002-072

ATTACHMENT A

BORING LOGS

SOUTHCOAST LANDFILL
MENDOCINO COUNTY, CALIFORNIA
Thickness of Existing Cover Soil

Test Pit Designation	Cover Soil Thickness	Sample Retained	Remarks
TP-1	24 in.	Yes	Brown to strong brown (7.5YR 4/5) to dark gray (5Y 4/1), poorly sorted GRAVELLY SILT with CLAY.
TP-2	19 in.	No	Brown to strong brown (7.5YR 4/5) to dark gray (5Y 4/1), poorly sorted GRAVELLY SILT with CLAY.
TP-3	27 in.	No	Strong brown (7.5YR 5/7) to dark greenish gray (10Y 3/1), poorly sorted GRAVELLY SILT with CLAY.
TP-4	35 in.	No	Strong brown (7.5YR 5/7) to dark greenish gray (10Y 3/1), poorly sorted GRAVELLY SILT with CLAY.
TP-5	>96 in.	Yes	Strong brown (7.5YR 5/7), poorly sorted GRAVELLY SILT with CLAY.
TP-6	18 in.	No	Strong brown (7.5YR 5/7) to dark greenish gray (10Y 3/1), poorly sorted GRAVELLY SILT with CLAY.
TP-7	22 in.	No	Strong brown (7.5YR 5/7) to dark greenish gray (10Y 3/1), poorly sorted GRAVELLY SILT with CLAY.
TP-8	28 in.	No	Strong brown (7.5YR 5/7) to dark greenish gray (10Y 3/1), poorly sorted GRAVELLY SILT with CLAY.
TP-9	28 in.	No	Strong brown (7.5YR 5/7) to dark greenish gray (10Y 3/1), poorly sorted GRAVELLY SILT with CLAY.
TP-10	28 in.	Yes	Strong brown (7.5YR 5/7) to dark greenish gray (10Y 3/1), poorly sorted GRAVELLY SILT with CLAY.
TP-11	15 in.	No	Brown to strong brown (7.5YR 4/5) to dark gray (5Y 4/1), poorly sorted GRAVELLY SILT with CLAY.
TP-12	41 in.	No	Strong brown (7.5YR 5/7) to dark greenish gray (10Y 3/1), poorly sorted GRAVELLY SILT with CLAY.
TP-13	6 in.	No	Brown to strong brown (7.5YR 4/5) to dark gray (5Y 4/1), poorly sorted GRAVELLY SILT with CLAY.
TP-14	28 in.	No	Strong brown (7.5YR 5/7) to dark greenish gray (10Y 3/1), poorly sorted GRAVELLY SILT with CLAY.
TP-15	17 in.	No	Strong brown (7.5YR 5/7) to dark greenish gray (10Y 3/1), poorly sorted GRAVELLY SILT with CLAY.
TP-16	33 in.	Yes	Strong brown (7.5YR 5/7) to dark greenish gray (10Y 3/1), poorly sorted GRAVELLY SILT with CLAY.
TP-17	>60 in.	No	Strong brown (7.5YR 5/7), poorly sorted GRAVELLY SILT with CLAY.
TP-18	18 in.	No	Strong brown (7.5YR 5/7) to dark greenish gray (10Y 3/1), poorly sorted GRAVELLY SILT with CLAY.
TP-19	41 in.	No	Strong brown (7.5YR 5/7) to dark greenish gray (10Y 3/1), poorly sorted GRAVELLY SILT with CLAY.
TP-20	30 in.	No	Strong brown (7.5YR 5/7) to dark greenish gray (10Y 3/1), poorly sorted GRAVELLY SILT with CLAY.

GeoLogic Associates
Boring Log

BORING NO.: B-1
PAGE: 1 OF 1

JOB NO.: 2002-072
SITE LOCATION: SOUTH COAST LANDFILL
DRILLING METHOD: HOLLOW STEM AUGER
CONTRACTOR: WEEKS DRILLING
LOGGED BY: M. VINCENT, CEG

DATE STARTED: 10/23/02
DATE FINISHED: 10/23/02
ELEVATION: ±511
NORTHING: ND
EASTING: ND

GW DEPTH: NA
TOTAL DEPTH: 38.5 feet

TIME	DRY DENSITY (LBS/CU. FT.)	MOISTURE (%)	BLOWS (COUNT/FT.)	SAMPLE SIZE (INCHES)	SAMPLE NO.	DEPTH IN FEET	ELEVATION IN FEET	MATERIAL SYMBOL	USCS/GEOLOGIC FORMATION	DESCRIPTION	COMMENTS
						0			ML/ CL	FILL: Strong brown (7.5YR 5/7), poorly sorted GRAVELLY SILT with CLAY.	
			24	2.5	1	5					
									ML/ CL	SAN ANDREAS FAULT GOUGE: Dark greenish gray (5BG 4/1 to 10Y 4/1), poorly sorted, highly sheared, fine SANDY SILT with plastic CLAY.	
			27	2.5	2	10					
									ML	Strong brown (7.5YR 4/7), poorly sorted, highly sheared CLAYEY SILT with abundant, subangular GRAVEL consisting of fine SILTY SANDSTONE.	
			16	2.5	3	15				...(14') - color change to mottled strong brown (7.5YR 5/6) with dark greenish gray (10Y 4/1). Contains abundant plant debris.	
			9	2.5	4	20				...(21'-21.5') - black highly plastic CLAY with calcite crystals and subangular GRAVEL composed of well cemented, well undurated, fine SANDY SILTSTONE to SILTY SANDSTONE.	
										Mottled strong brown (7.5YR 4/7) with yellowish red (5YR 4/7) and yellowish brown (10YR 5/6), highly sheared, well indurated, fine SILTY SANDSTONE.	
			65	2.5	5	25					
			50/10"	2.5	6	30					
			55	2.5	7	35					
			50/5"	2.5	NSR						
						40					
						45					
						50					

Notes:

1. Boring terminated at 38.5 feet due to a refusal.
2. No groundwater encountered.
3. Boring backfilled with hydrated bentonite chips, with soil tamped into place in the upper 3 feet.

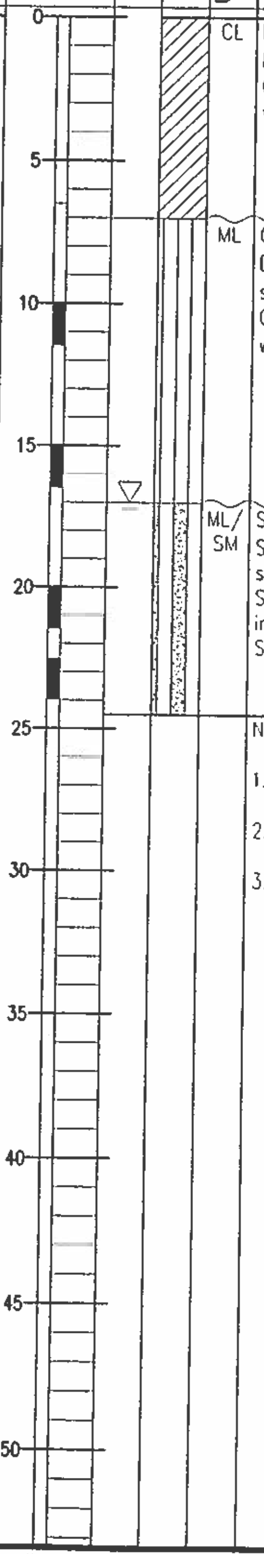
GeoLogic Associates
Boring Log

BORING NO.: B-2
PAGE: 1 OF 1

JOB NO.: 2002-072
SITE LOCATION: SOUTH COAST LANDFILL
DRILLING METHOD: HOLLOW STEM AUGER
CONTRACTOR: WEEKS DRILLING
LOGGED BY: M. VINCENT, CEG

DATE STARTED: 10/23/02
DATE FINISHED: 10/23/02
ELEVATION: ±475
NORTHING: ND
EASTING: ND

GW DEPTH: 17 feet
TOTAL DEPTH: 24.5 feet



TIME	DRY DENSITY (LBS/CU. FT.)	MOISTURE (%)	BLOWS (COUNT/FT.)	SAMPLE SIZE (INCHES)	SAMPLE NO.	DEPTH IN FEET	ELEVATION IN FEET	MATERIAL SYMBOL	USCS/GEOLOGIC FORMATION	DESCRIPTION	COMMENTS
						0			CL	FILL: Dark greenish gray (5BG 4/1 to 10Y 4/1) CLAY with GRAVEL; contains minor amounts of wood chips, bark and scattered refuse.	
			7	2.5	NSR	5					
									ML	COLLUVIUM: Dark greenish gray (5BG 4/1) mottled with strong brown (7.5YR 4/6), highly plastic GRAVELLY SILT with abundant SAND, CLAY and wood debris (stems, bark, chips).	
			26	2.5	1	10					
			25	2.5	2	15					
									ML/SM	SAN ANDREAS FAULT GOUGE: Strong brown (7.5YR 5/6 to 4/7), highly sheared, well indurated GRAVELLY SILT to SILTY SAND with GRAVEL. GRAVEL consists of well indurated and well cemented fragments of fine SILTY SANDSTONE.	...(17') - possible groundwater encountered.
			40	2.5	3	20					
			50	2.5	4						

Notes:

1. Boring terminated at 24.5 feet due to a refusal.
2. Possible groundwater encountered at approximately 17 feet.
3. Boring backfilled with hydrated bentonite chips, with soil tamped into place in the upper 3 feet.

GeoLogic Associates
Boring Log

BORING NO.: B-3
PAGE: 1 OF 1

JOB NO.: 2002-072
SITE LOCATION: SOUTH COAST LANDFILL
DRILLING METHOD: HOLLOW STEM AUGER
CONTRACTOR: WEEKS DRILLING
LOGGED BY: M. VINCENT, CEG

DATE STARTED: 10/23/02
DATE FINISHED: 10/23/02
ELEVATION: ND
NORTHING: ND
EASTING: ND

GW DEPTH: NA
TOTAL DEPTH: 12 feet

TIME	DRY DENSITY (LBS/CU. FT.)	MOISTURE (%)	BLOWS (COUNT/FT.)	SAMPLE SIZE (INCHES)	SAMPLE NO.	DEPTH IN FEET	ELEVATION IN FEET	MATERIAL SYMBOL	USCS/GEOLOGIC FORMATION	DESCRIPTION	COMMENTS
						0		▨	CL/ GW	SAN ANDREAS FAULT GOUGE: Strong brown (7.5YR 5/6) to dark gray (5Y 4/1), highly sheared CLAY with abundant, subangular GRAVEL sized clasts of well cemented, fine SILTY SANDSTONE.	
			70	2.5	1	5		▨			
			60	2.5	2	10		▨			
										Notes: 1. Boring terminated at 12 feet due to a refusal. 2. No groundwater encountered. 3. Boring backfilled with hydrated bentonite chips, with soil tamped into place in the upper 3 feet.	
						15					
						20					
						25					
						30					
						35					
						40					
						45					
						50					

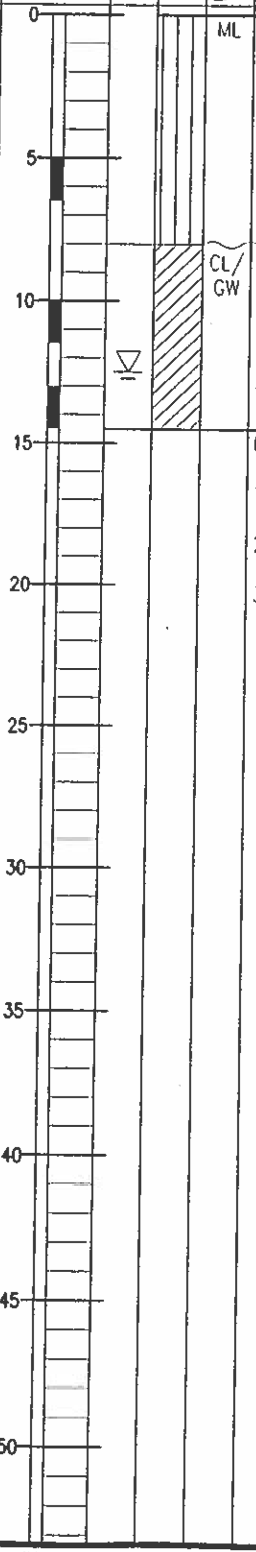
GeoLogic Associates
Boring Log

BORING NO.: B-4
PAGE: 1 OF 1

JOB NO.: 2002-072
SITE LOCATION: SOUTH COAST LANDFILL
DRILLING METHOD: HOLLOW STEM AUGER
CONTRACTOR: WEEKS DRILLING
LOGGED BY: M. VINCENT, CEG

DATE STARTED: 10/23/02
DATE FINISHED: 10/23/02
ELEVATION: ±505
NORTHING: ND
EASTING: ND

GW DEPTH: 12.5 feet
TOTAL DEPTH: 14.5 feet



TIME	DRY DENSITY (LBS/CU. FT.)	MOISTURE (%)	BLOWS (COUNT/FT.)	SAMPLE SIZE (INCHES)	SAMPLE NO.	DEPTH IN FEET	ELEVATION IN FEET	MATERIAL SYMBOL	USCS/GEOLOGIC FORMATION	DESCRIPTION	COMMENTS
						0			ML	STOCKPILE FILL: Strong brown (7.5YR 4/6), poorly sorted SANDY SILT with SILTSTONE GRAVEL. Also contains wood chips and bark.	
			25	2.5	1	5					
									CL/ GW	SAN ANDREAS FAULT GOUGE: Very dark gray (N3), highly sheared CLAY with abundant, subangular GRAVEL. Clasts are composed of well indurated, well cemented fragments of fine SILTY SANDSTONE to fine SANDY SILTSTONE.	
			28	2.5	2	10					
							∇				...(12.5') - possible groundwater encountered.
			50	2.5	3						
						15				Notes:	
						20					
						25					
						30					
						35					
						40					
						45					
						50					

Notes:

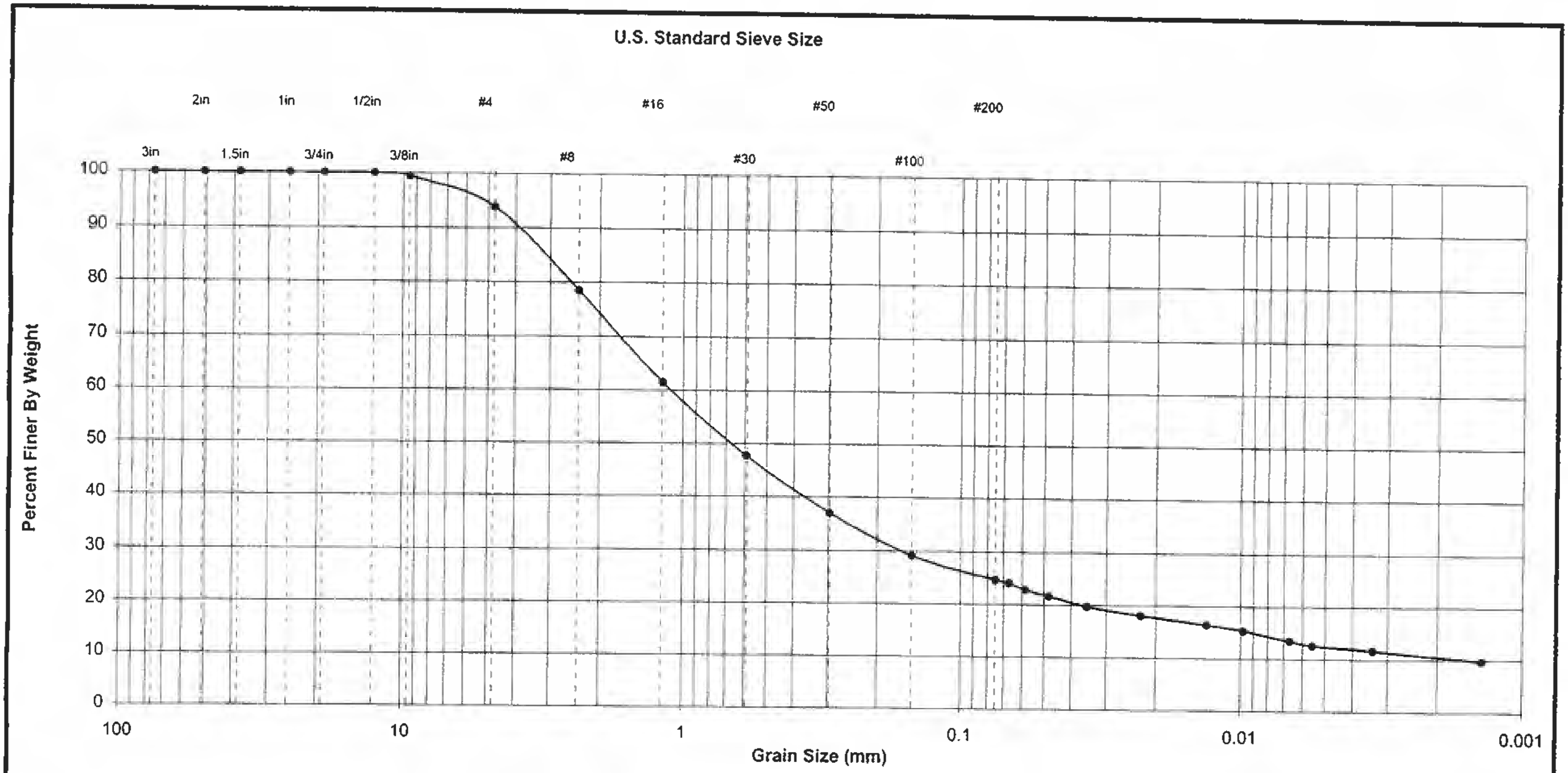
1. Boring terminated at 14.5 feet due to a refusal.
2. Possible groundwater encountered at approximately 12.5 feet.
3. Boring backfilled with hydrated bentonite chips, with soil tamped into place in the upper 3 feet.

ATTACHMENT B

LABORATORY TEST RESULTS

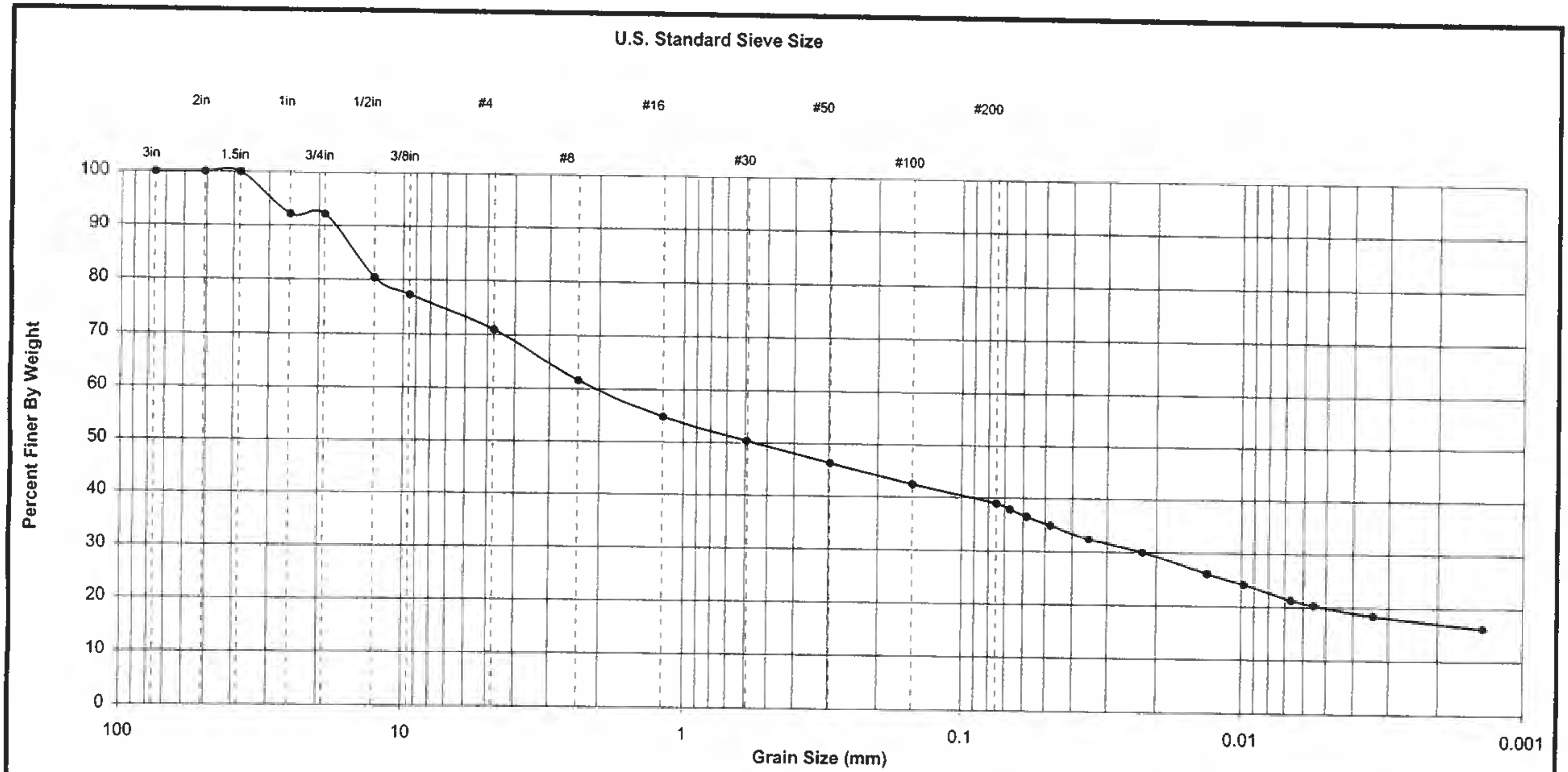
GRAIN SIZE ANALYSIS – ASTM D422

GRAIN SIZE ANALYSIS - ASTM D 422



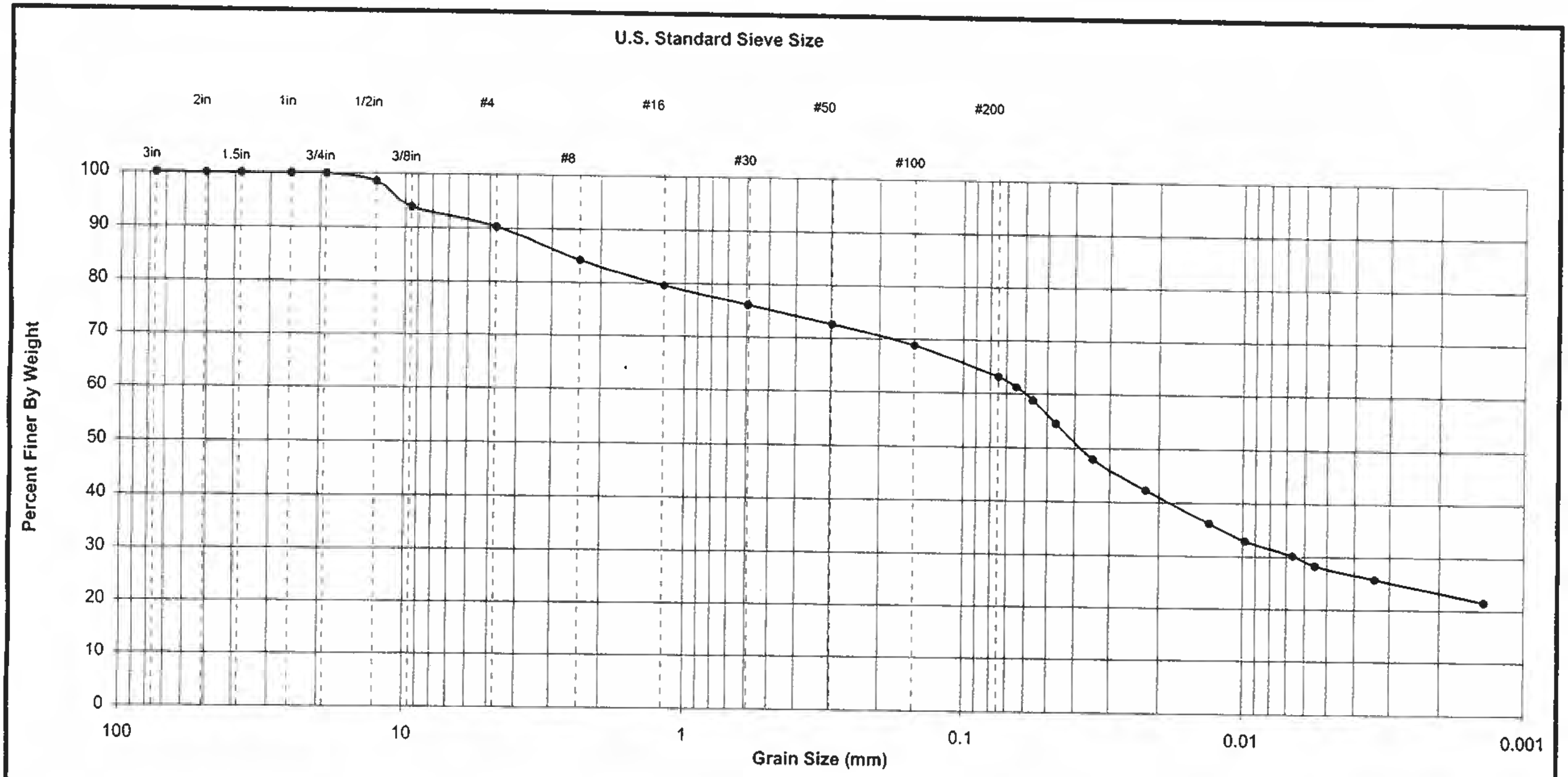
Boring / Sample No.	Initial Dry Density (pcf)	Initial Moist. (%)	Test Dry Density (pcf)	Test Moist. (%)	Permeability, K (cm/sec)	LL	PL	PI	Unified Soil Class.	Description
B-4 #3 @ 13'									SM	SILTY SAND

GRAIN SIZE ANALYSIS - ASTM D 422



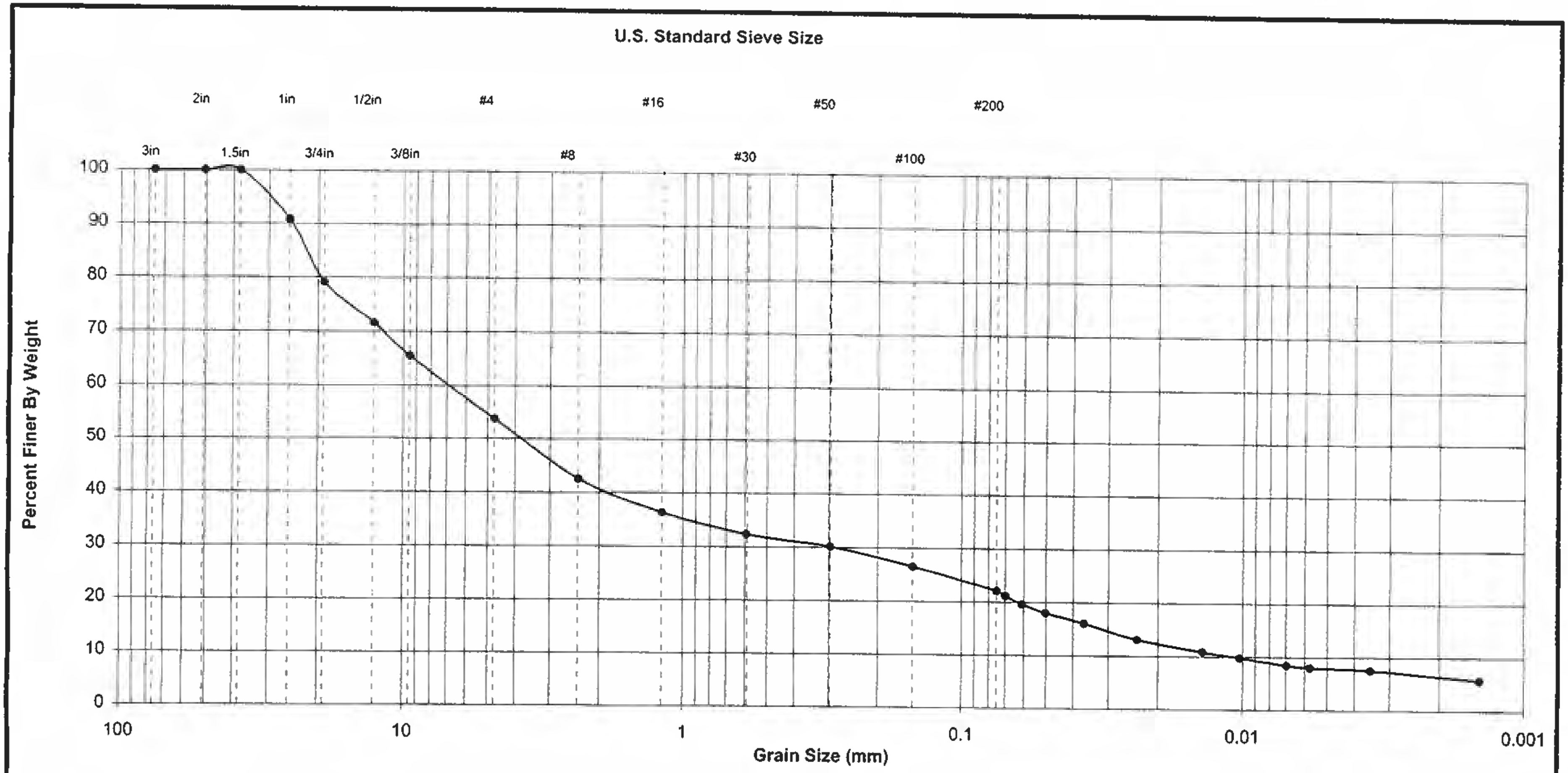
Boring / Sample No.	Initial Dry Density (pcf)	Initial Moist. (%)	Test Dry Density (pcf)	Test Moist. (%)	Permeability, K (cm/sec)	LL	PL	PI	Unified Soil Class.	Description
B-1 #3 @ 15'									ML	Clayey Silt W. Siltstone

GRAIN SIZE ANALYSIS - ASTM D 422



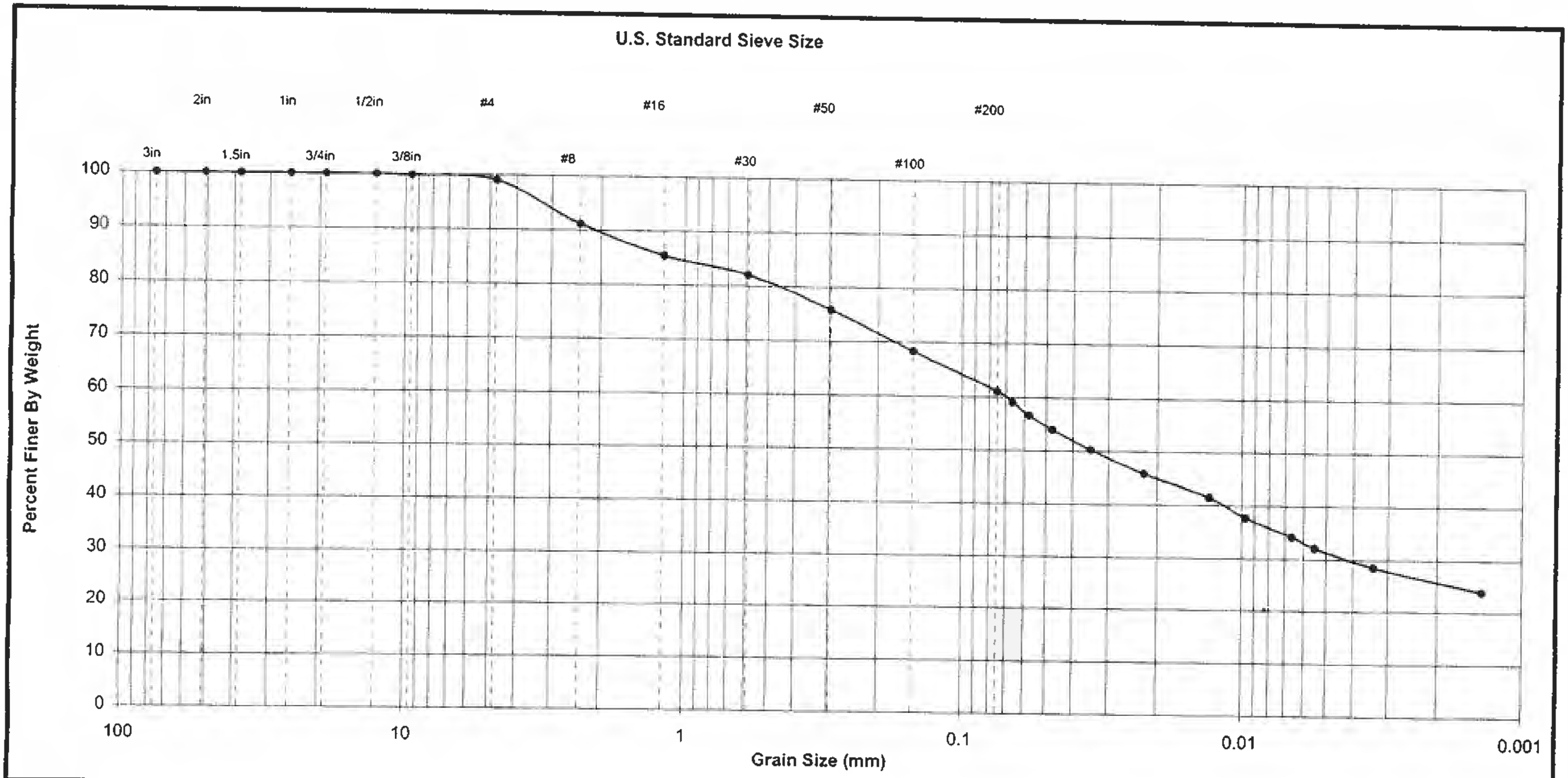
Boring / Sample No.	Initial Dry Density (pcf)	Initial Moist. (%)	Test Dry Density (pcf)	Test Moist. (%)	Permeability, K (cm/sec)	LL	PL	PI	Unified Soil Class.	Description
B-1 #4 @ 20'									ML	Clayey Silt W. Siltstone

GRAIN SIZE ANALYSIS - ASTM D 422



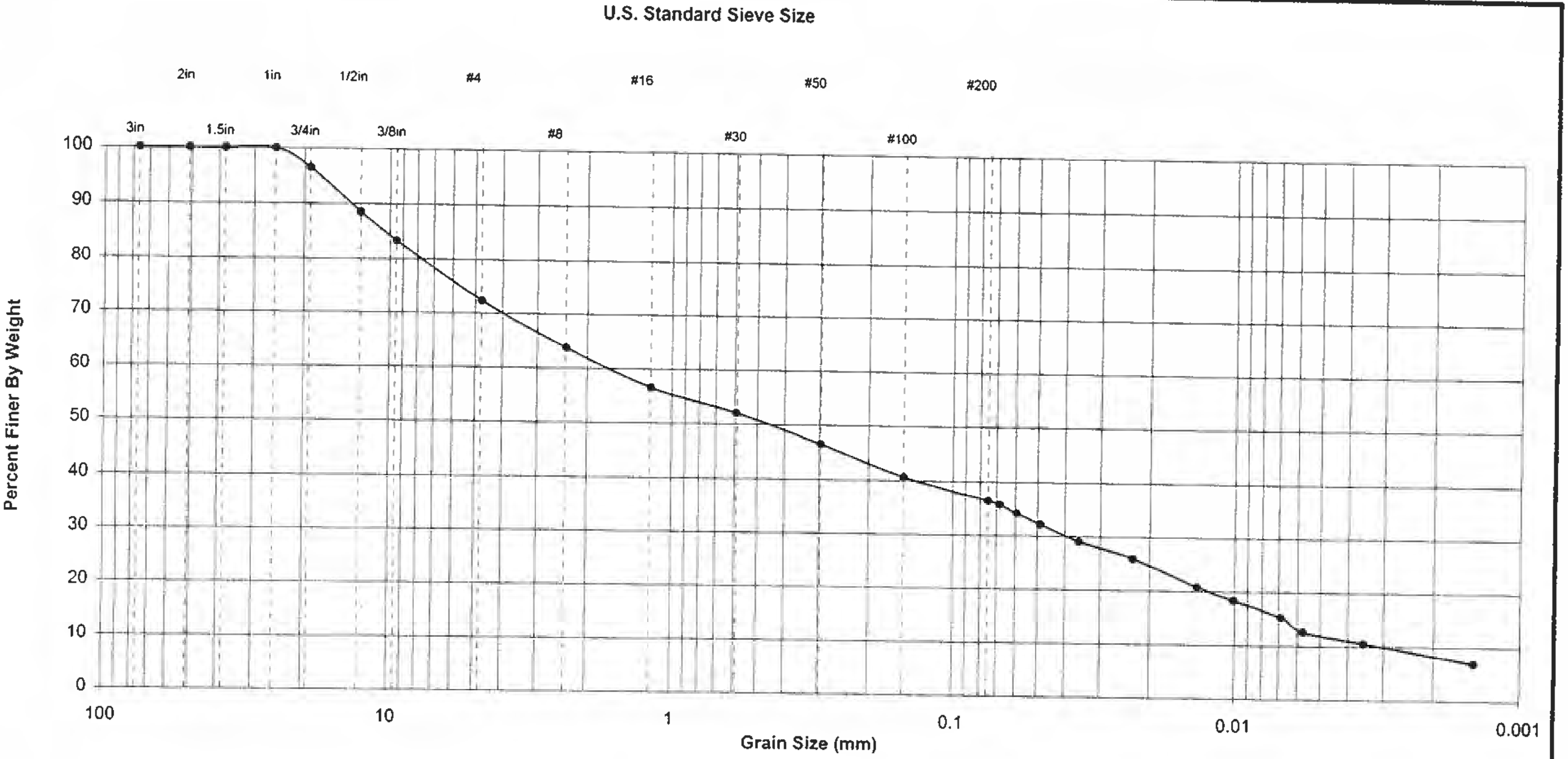
Boring / Sample No.	Initial Dry Density (pcf)	Initial Moist. (%)	Test Dry Density (pcf)	Test Moist. (%)	Permeability, K (cm/sec)	LL	PL	PI	Unified Soil Class.	Description
B-1 #7 @ 35'									ML	CLAYEY SILT W. SILTONE

GRAIN SIZE ANALYSIS - ASTM D 422



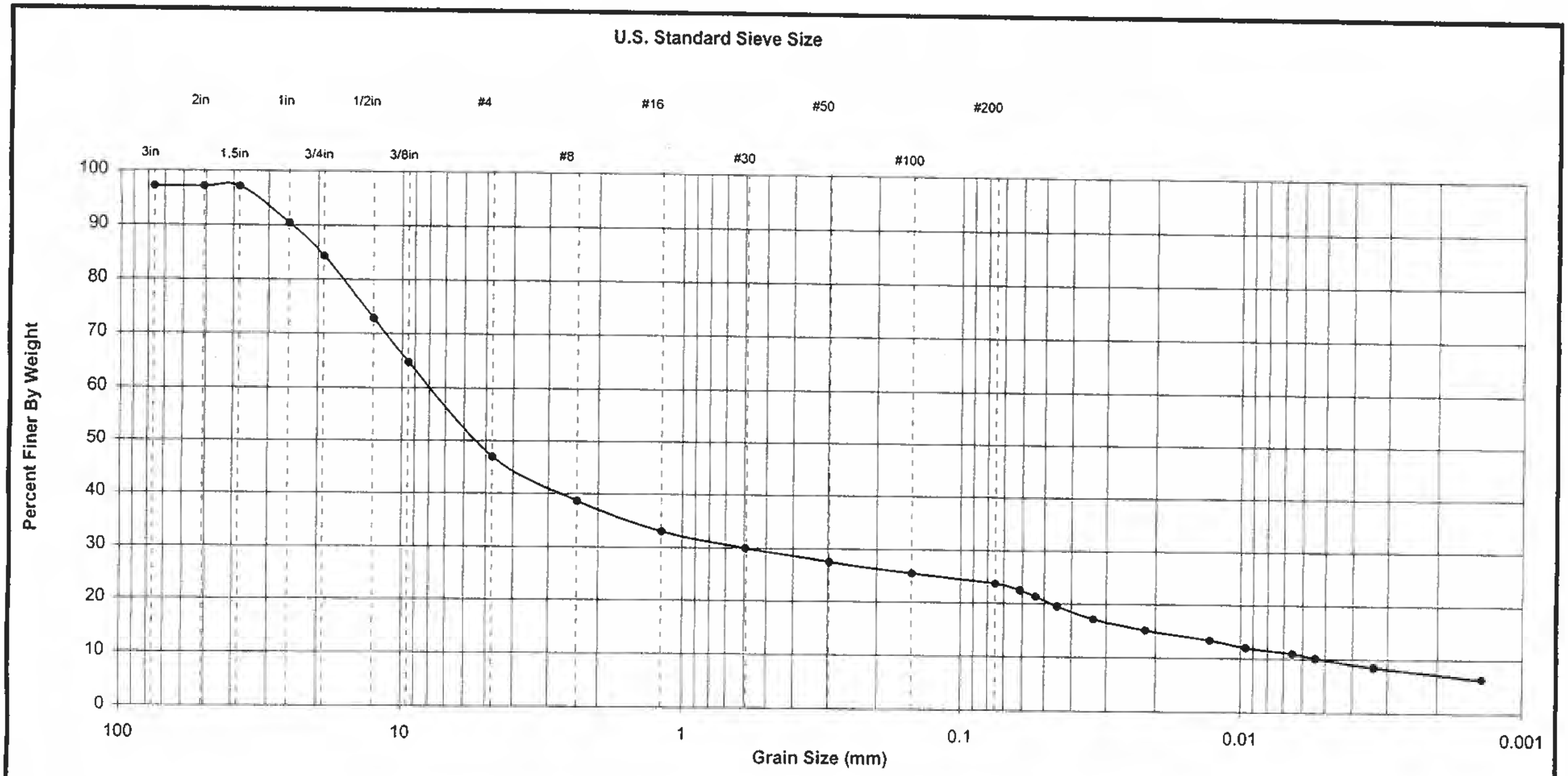
Boring / Sample No.	Initial Dry Density (pcf)	Initial Moist. (%)	Test Dry Density (pcf)	Test Moist. (%)	Permeability, K (cm/sec)	LL	PL	PI	Unified Soil Class.	Description
B-2 #1 @ 10'									ML	Clayey Silt W. Siltstone

GRAIN SIZE ANALYSIS - ASTM D 422



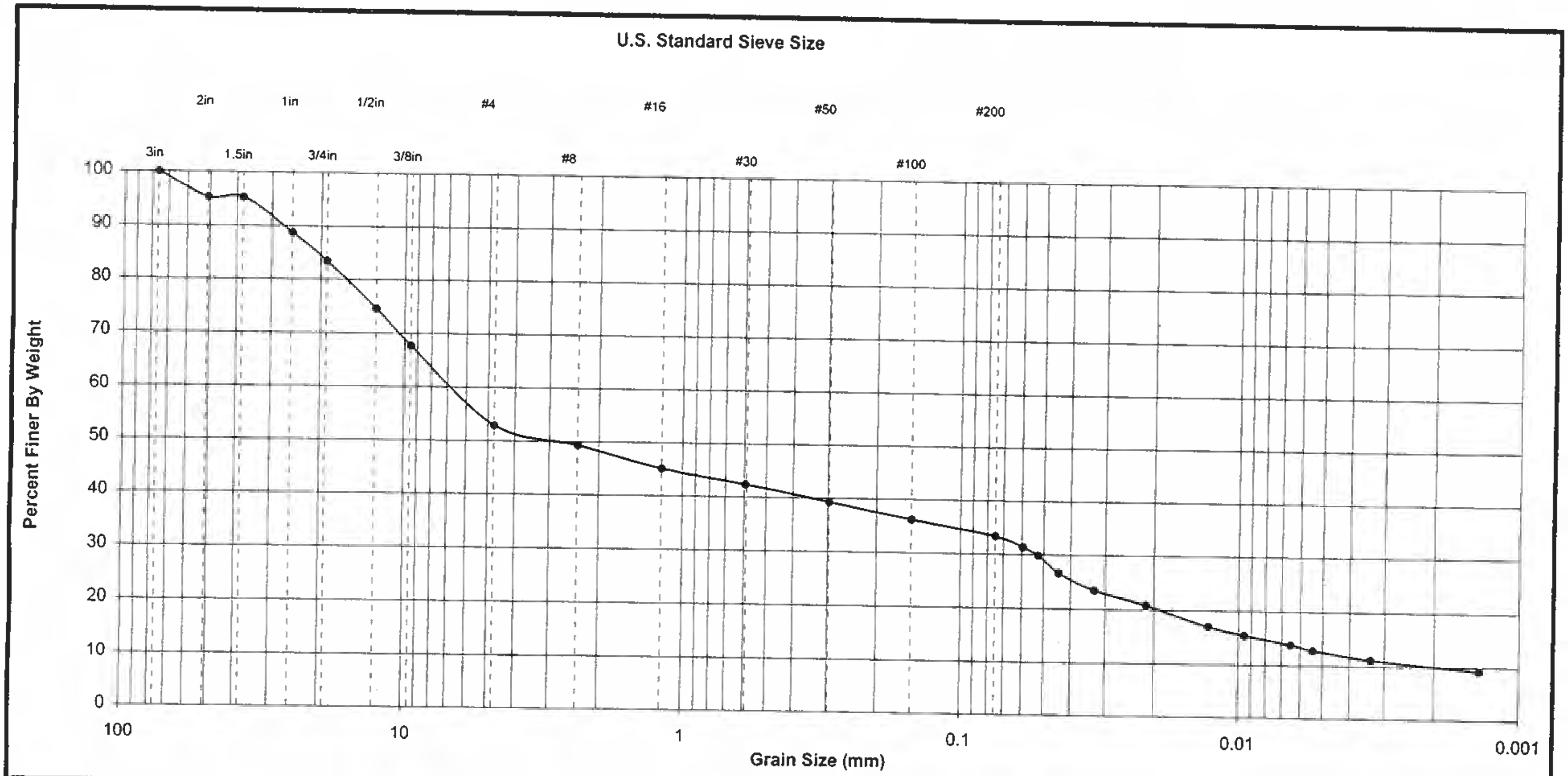
Boring / Sample No.	Initial Dry Density (pcf)	Initial Moist. (%)	Test Dry Density (pcf)	Test Moist. (%)	Permeability, K (cm/sec)	LL	PL	PI	Unified Soil Class.	Description
B-2 #4 @ 22.5'									ML	Clayey Silt W. Siltstone

GRAIN SIZE ANALYSIS - ASTM D 422



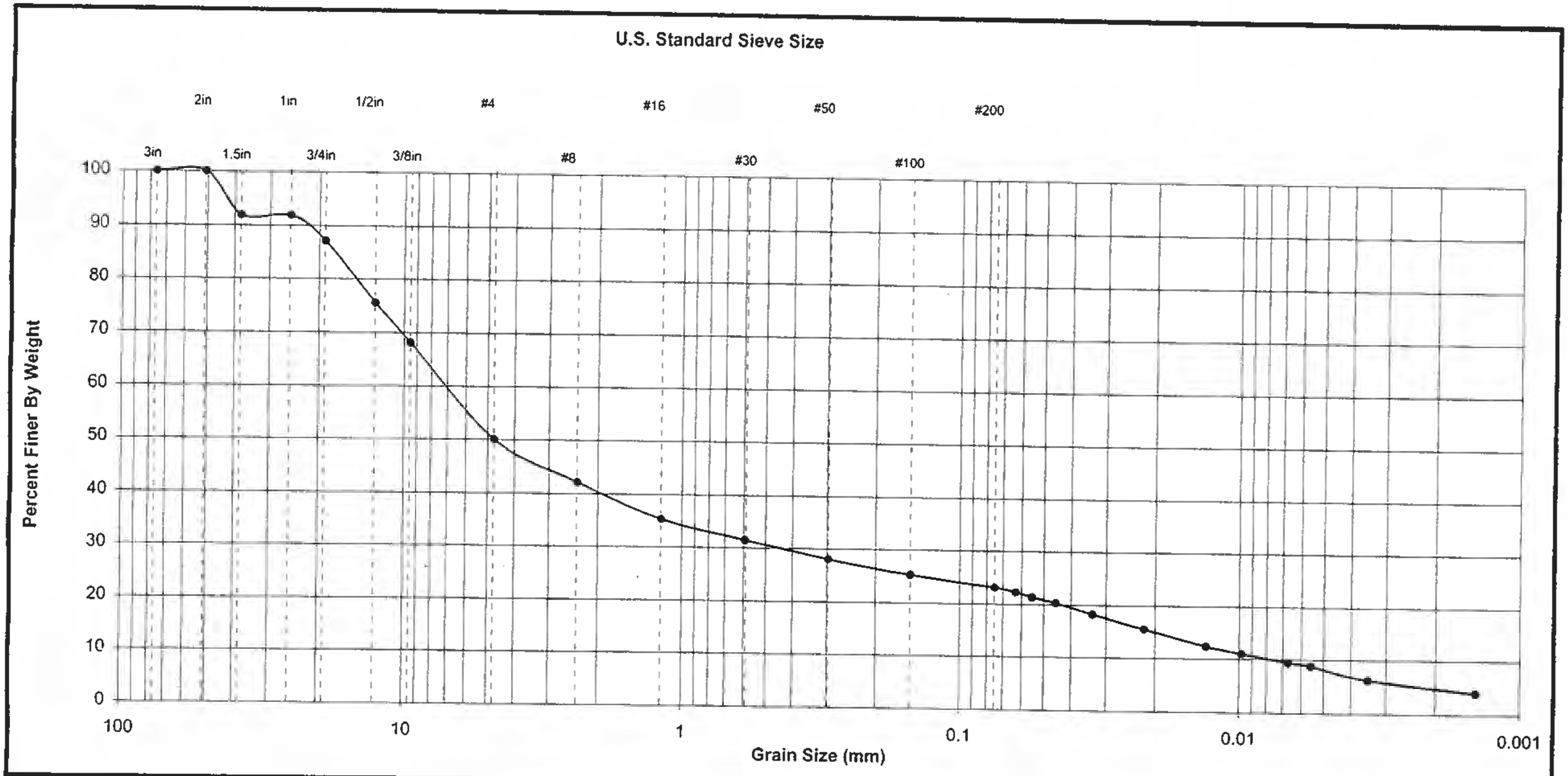
Boring / Sample No.	Initial Dry Density (pcf)	Initial Moist. (%)	Test Dry Density (pcf)	Test Moist. (%)	Permeability, K (cm/sec)	LL	PL	PI	Unified Soil Class.	Description
TP-1, 5, 10, 16									ML	Clayey Silt W. Siltstone

GRAIN SIZE ANALYSIS - ASTM D 422



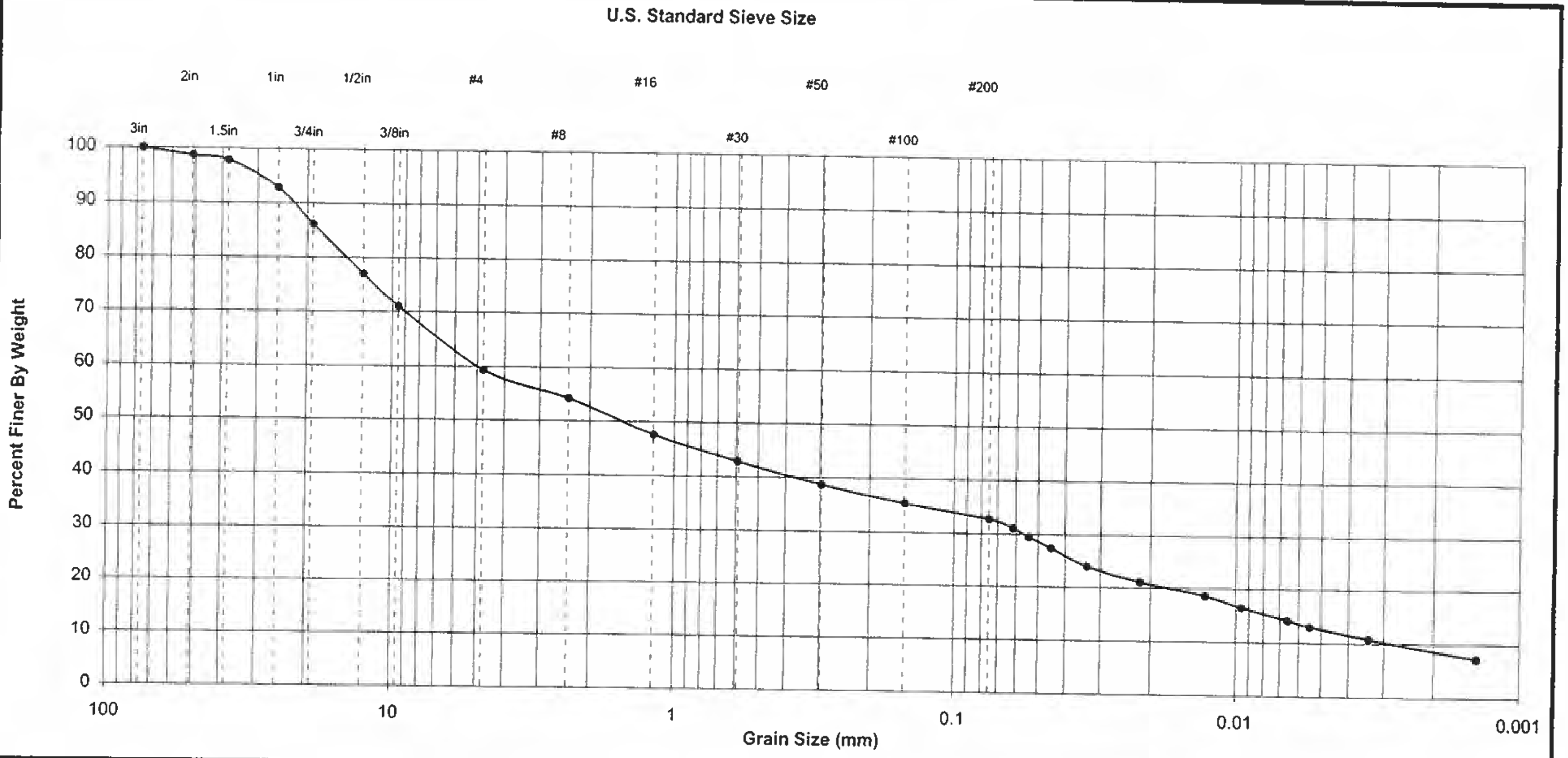
Boring / Sample No.	Initial Dry Density (pcf)	Initial Moist. (%)	Test Dry Density (pcf)	Test Moist. (%)	Permeability, K (cm/sec)	LL	PL	PI	Unified Soil Class.	Description
TP-16									ML	Clayey Silt W. Siltstone

GRAIN SIZE ANALYSIS - ASTM D 422



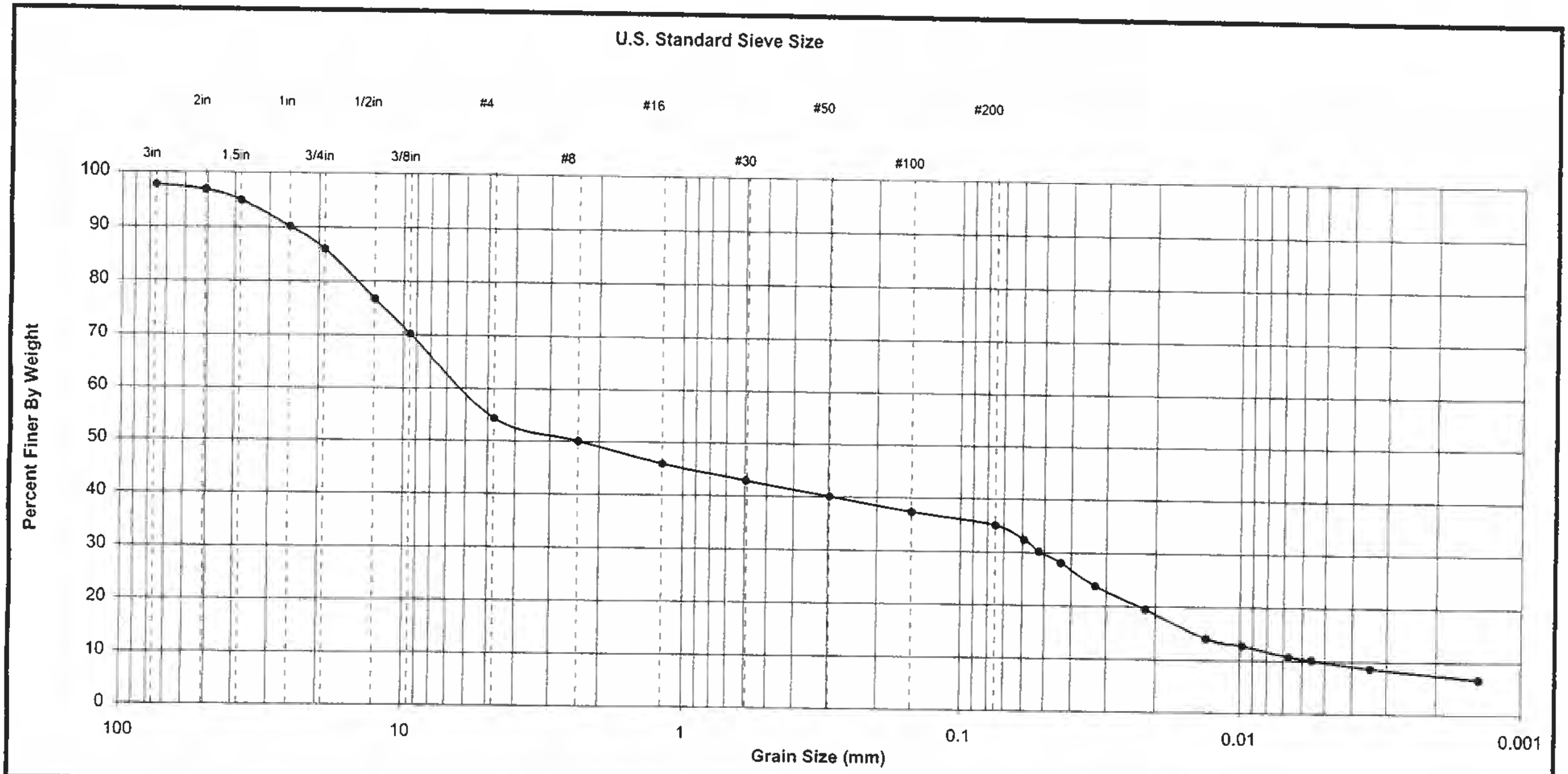
Boring / Sample No.	Initial Dry Density (pcf)	Initial Moist. (%)	Test Dry Density (pcf)	Test Moist. (%)	Permeability, K (cm/sec)	LL	PL	PI	Unified Soil Class.	Description
TP-5									ML	Clayey Silt W. Siltstone

GRAIN SIZE ANALYSIS - ASTM D 422



Boring / Sample No.	Initial Dry Density (pcf)	Initial Moist. (%)	Test Dry Density (pcf)	Test Moist. (%)	Permeability, K (cm/sec)	LL	PL	PI	Unified Soil Class.	Description
Stockpile #2									ML	Clayey Silt W. Siltstone

GRAIN SIZE ANALYSIS - ASTM D 422



Boring / Sample No.	Initial Dry Density (pcf)	Initial Moist. (%)	Test Dry Density (pcf)	Test Moist. (%)	Permeability, K (cm/sec)	LL	PL	PI	Unified Soil Class.	Description
Stockpile #1 & #2									ML	Clayey Silt W. Siltstone

MAXIMUM DENSITY TEST – ASTM D1557

MAXIMUM DENSITY TEST ASTM D1557

Job Name SouthCoast LF

Job No. 2001-082

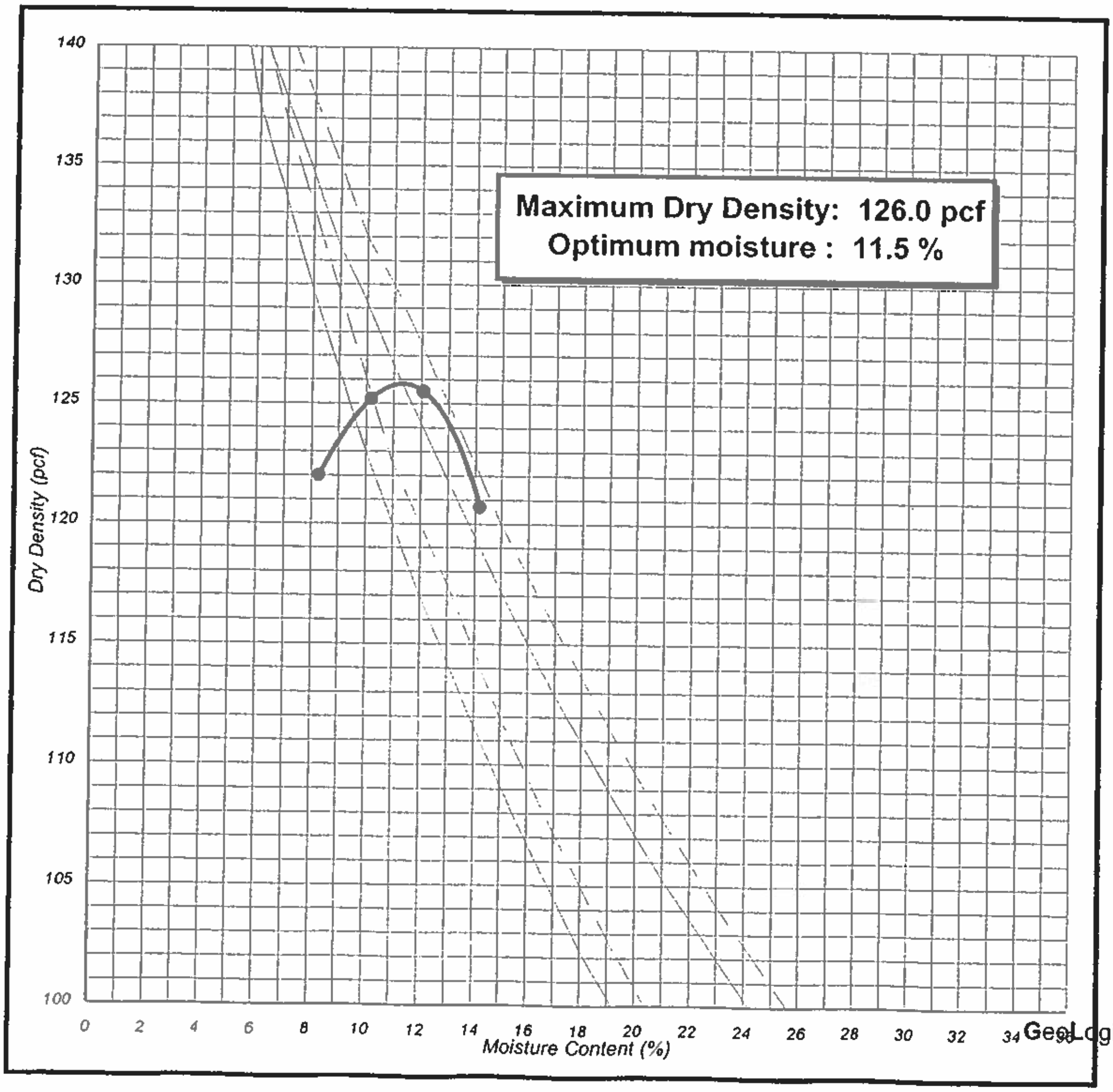
Boring/Sample No. B-1 & B-2 Composite

Description: Rusty Brown, Silty Clay w. Siltstone Frgmts (-3/4")

Date: 11/27/02

By: LD

Method: C	Mold Volume (cf): 0.0750		Blows: 56	Layers: 5	
Specimen	A	B	C	D	E
Total Wet Weight (lbs)	**7679**	**7684**	**7776**	**7485**	
Weight of Mold (lbs)	2990	2990	2990	2990	
Wet Weight of Soil (lbs)	4689	4694	4786	4495	
Wet Density (pcf)	137.8	138.0	140.7	132.1	
Moisture Can No.					
Dry Weight					
Moisture Content (%)	**14.2**	**10.2**	**12.1**	**8.3**	
Dry Density (pcf)	120.7	125.2	125.5	122.0	

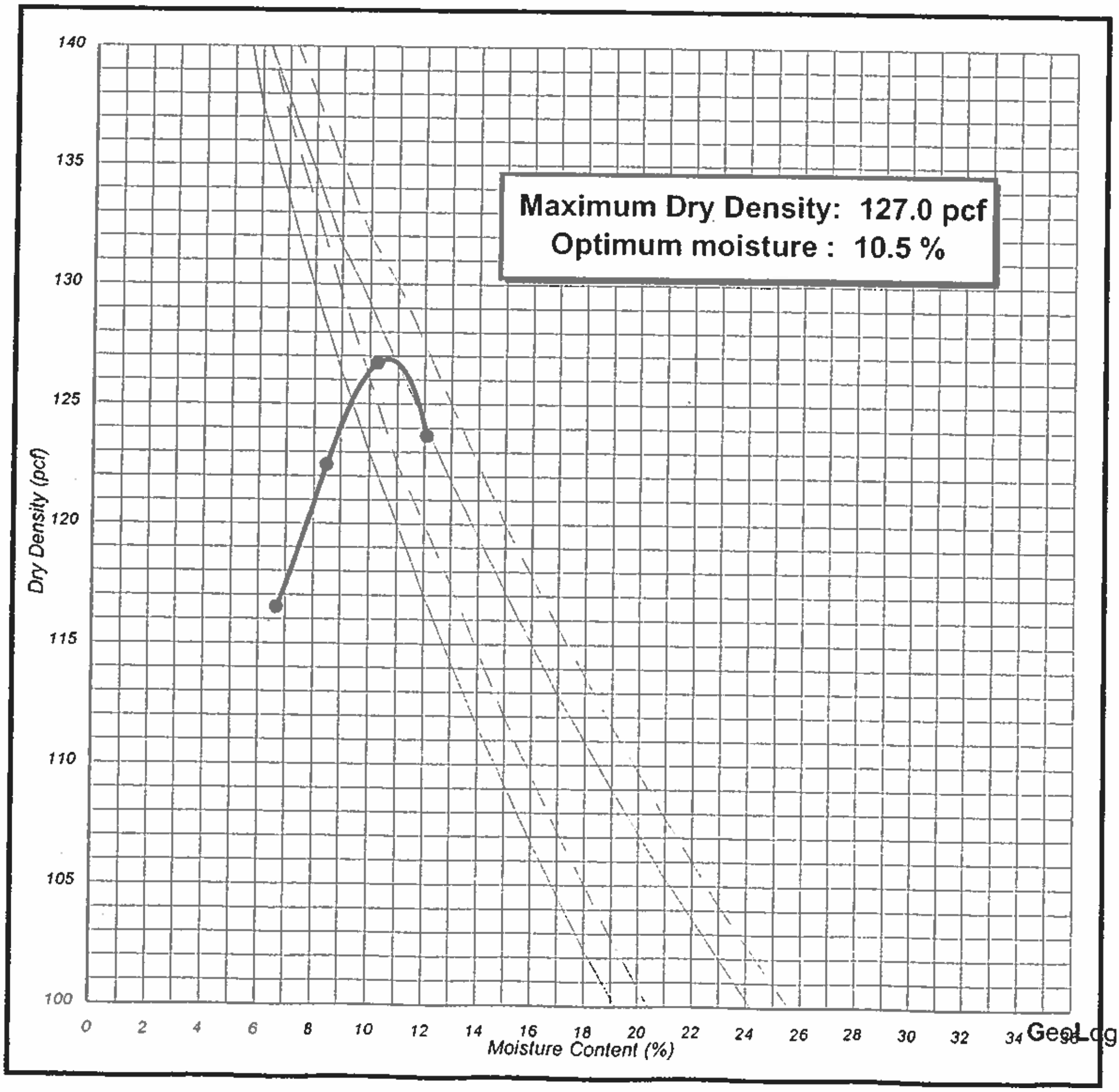


GeoLogic Associates

MAXIMUM DENSITY TEST ASTM D1557

Job Name	SouthCoast LF	**Date:**	11/27/02
Job No.	2001-082	**By:**	LD
Boring/Sample No.	TP-1, 5, 10, 16 Composite		
Description:	D. Brown, Clayey Silt w. Siltstone Frgmts (-3/4")		

Method: **C**	Mold Volume (cf): **0.0750**		Blows: **56**	Layers: **5**	
Specimen	A	B	C	D	E
Total Wet Weight (lbs)	**7744**	**7706**	**7510**	**7219**	
Weight of Mold (lbs)	2990	2990	2990	2990	
Wet Weight of Soil (lbs)	4754	4716	4520	4229	
Wet Density (pcf)	139.8	138.6	132.9	124.3	
Moisture Can No.					
Dry Weight					
Moisture Content (%)	**10.3**	**12.1**	**8.5**	**6.7**	
Dry Density (pcf)	126.7	123.7	122.5	116.5	



Maximum Dry Density: 127.0 pcf
Optimum moisture : 10.5 %

GeoLogic Associates

DENSITY TESTS

PROJECT SouthCoast LF JOB NO. 2001-082 BY LD DATE 12/03/02

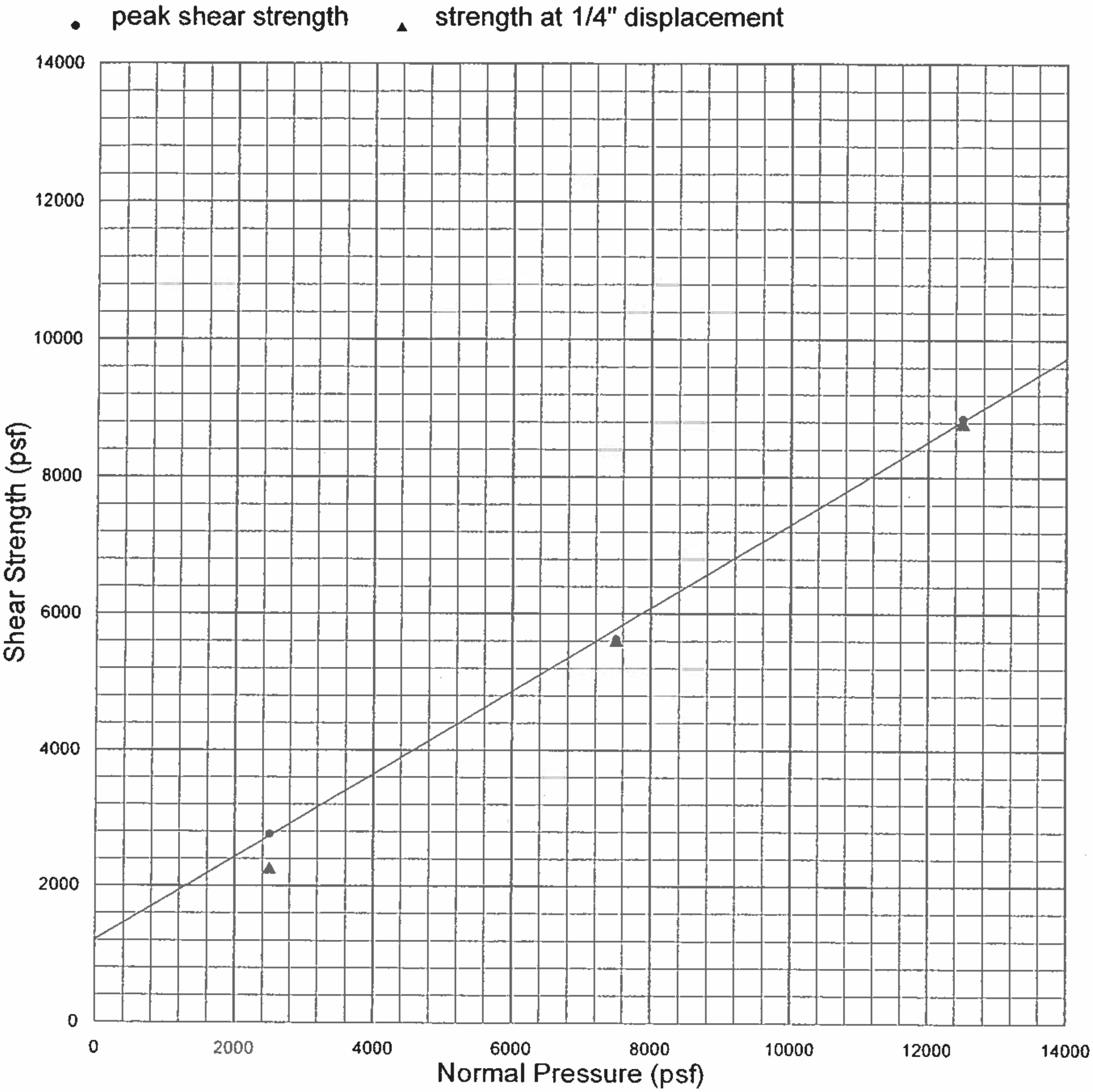
Sample No.	B-1/3	B-1/4	B-1/6	B-1/7	B-2/1	B-2/4	B-4/1	B-4/3
Depth (ft)	15.0	20.0	30.0	35.0	10.0	22.5	5.0	13.0
Testing								
Soil Type	Grqy Brown Mottled, Silty Clay w. Siltstone & rock Frgmts	D. Gray, Silty Clay w. Rock frgmts	Gray Brown Mottled, Clayey Silt w. rock Frmgts	Gray Brown Mottled, Clayey Silt w. rock Frmgts	Gray Brown Mottled, Silty Clay	Gray Brown Mottled, Clayey Silt w. Siltstone	Brown Mottled, Silty Clay	D. Gray, F.C. Sandy Clay
Wet+Tare	572.8	631.0	784.1	820.9	1180.1	767.6	1240.8	861.6
No. Ring	3	4	5	5	5	5	5	5
Can No.								
Dry Weight								
Wet density	121.0	93.3	131.0	137.1	126.3	128.2	126.2	143.9
% Water	17.9	21.0	17.7	14.4	22.0	17.6	13.8	7.2
Dry Density	102.7	77.1	111.3	119.8	103.5	109.0	110.9	134.2
O.B.Press(psf)								
Sample No.								
Depth (ft)								
Testing								
Soil Type								
Wet+Tare								
No. Ring								
Can No.								
Dry Weight								
Wet density								
% Water								
Dry Density								
O.B.Press(psf)								

DIRECT SHEAR TEST – ASTM D3080

Job No. 2001-082

DIRECT SHEAR TEST - ASTM D-3080

SouthCoast LF - Mendocino County

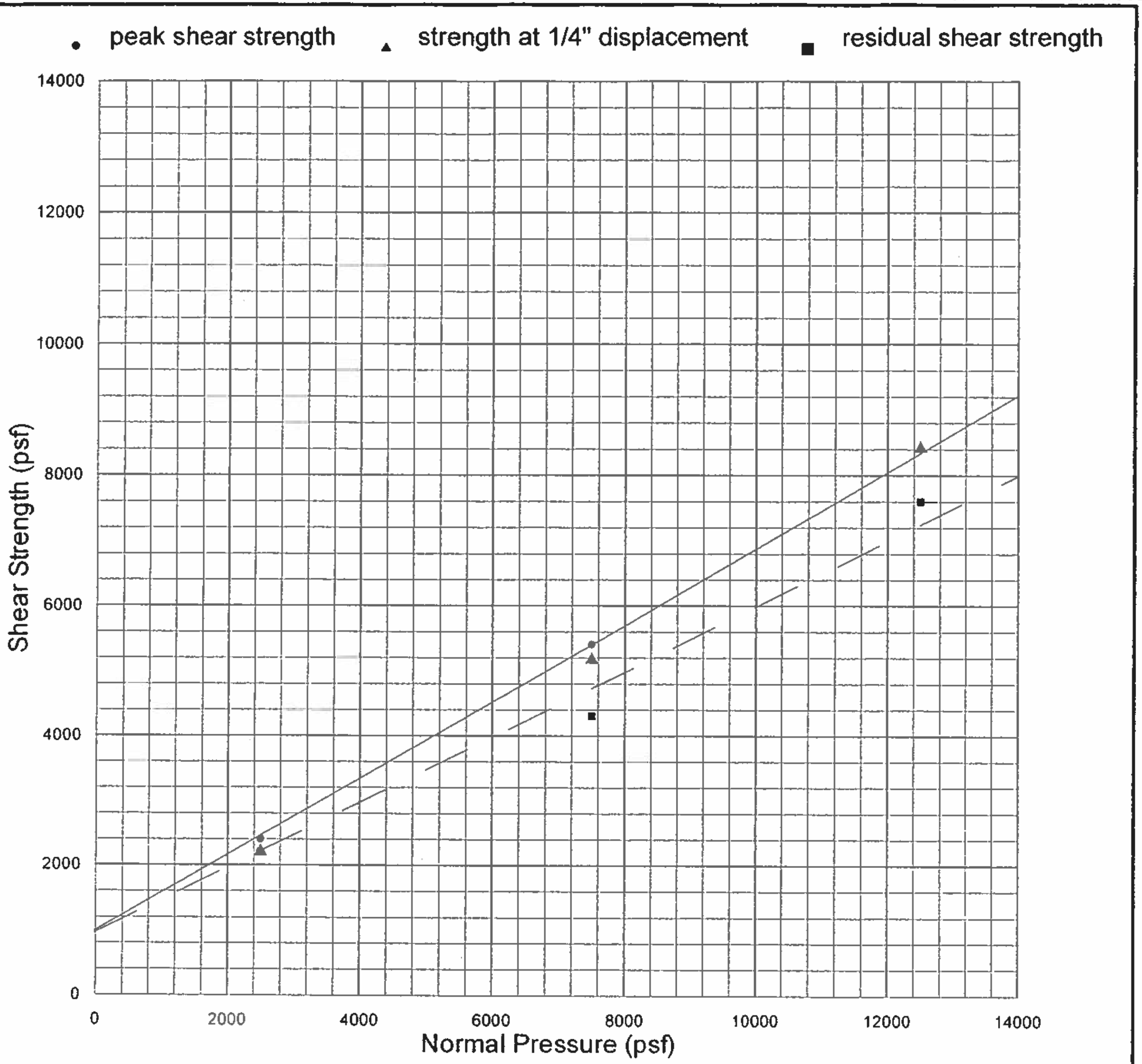


Strain Rate: 0.0042 in. / min.

Sample	Type	Description	Dry Density (pcf)	Initial W.C. (%)
B-1/1 @ 5'	Undisturbed & Saturated	Clayey Silt w. Rock & Siltstone Frgmts	108.2	17.2

Normal Pressure (psf)	Peak Strength (psf)	Ultimate Strength (psf)
2500	2760 @ 0.0865"	2230
7500	5630 @ 0.1270"	5580
12500	8860 @ 0.2130"	8760
	C = 1200 psf	C = 1200 psf
	ϕ = 32 deg.	ϕ = 32 deg.

DIRECT SHEAR TEST - ASTM D-3080



Strain Rate: 0.0042 in. / min.

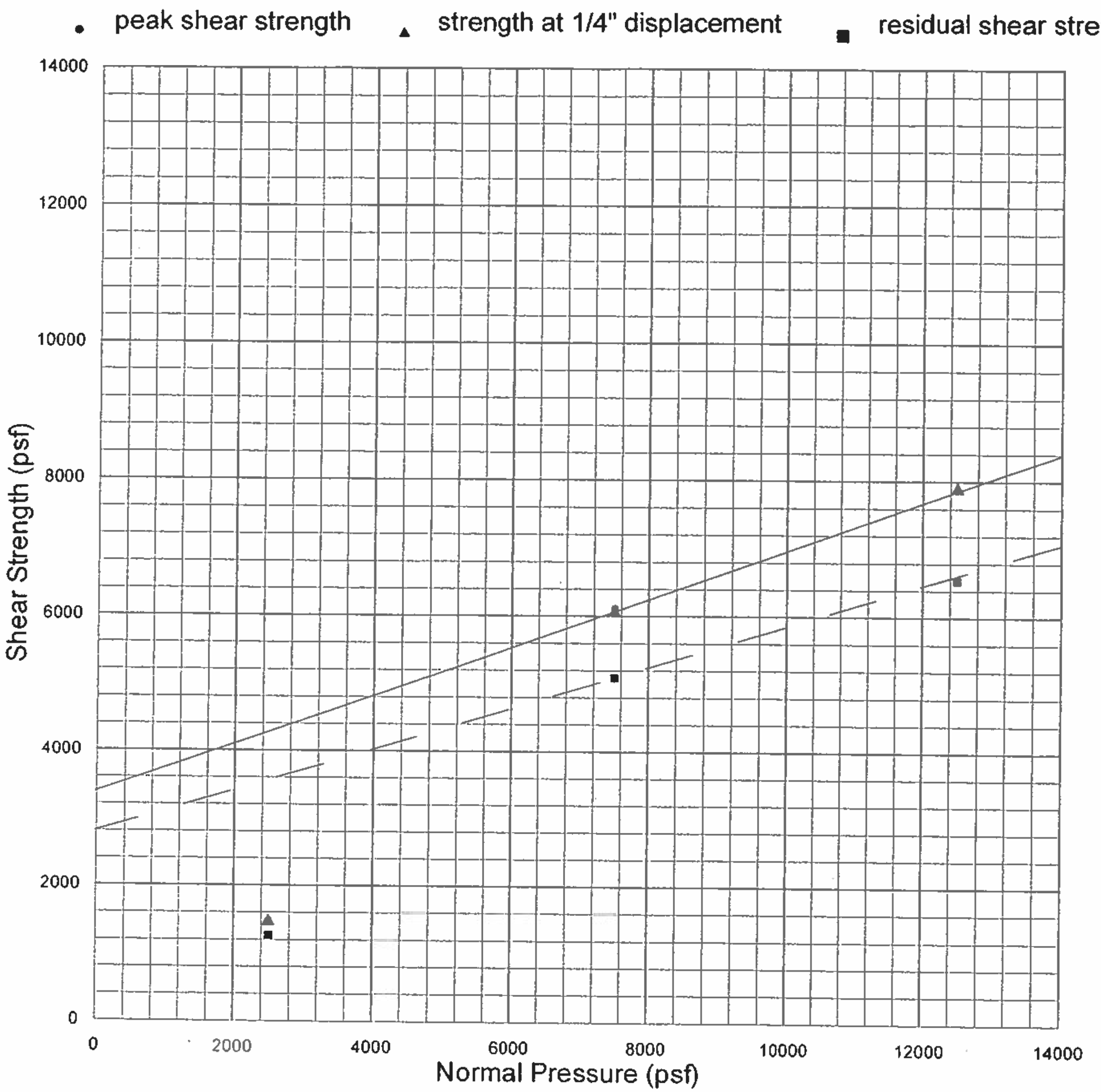
Sample	**Type**	**Description**	**Dry Density (pcf)**	**Initial W.C. (%)**
B-1/3 @ 15'	Undisturbed & Saturated	Silty Clay w. Rock Frgmts	102.7	17.9

Normal Pressure (psf)	Peak Strength (psf)	Ultimate Strength (psf)	Residual Strength (psf)
2500	2400 @ 0.1490"	2200	2200
7500	5410 @ 0.1570"	5170	4300
12500	8420 @ 0.2405"	8420	7600
	C = 1000 psf	C = 1000 psf	C = 1000 psf
	ϕ = 30 deg.	ϕ = 30 deg.	ϕ = 26 deg.

Job No. 2001-082

DIRECT SHEAR TEST - ASTM D-3080

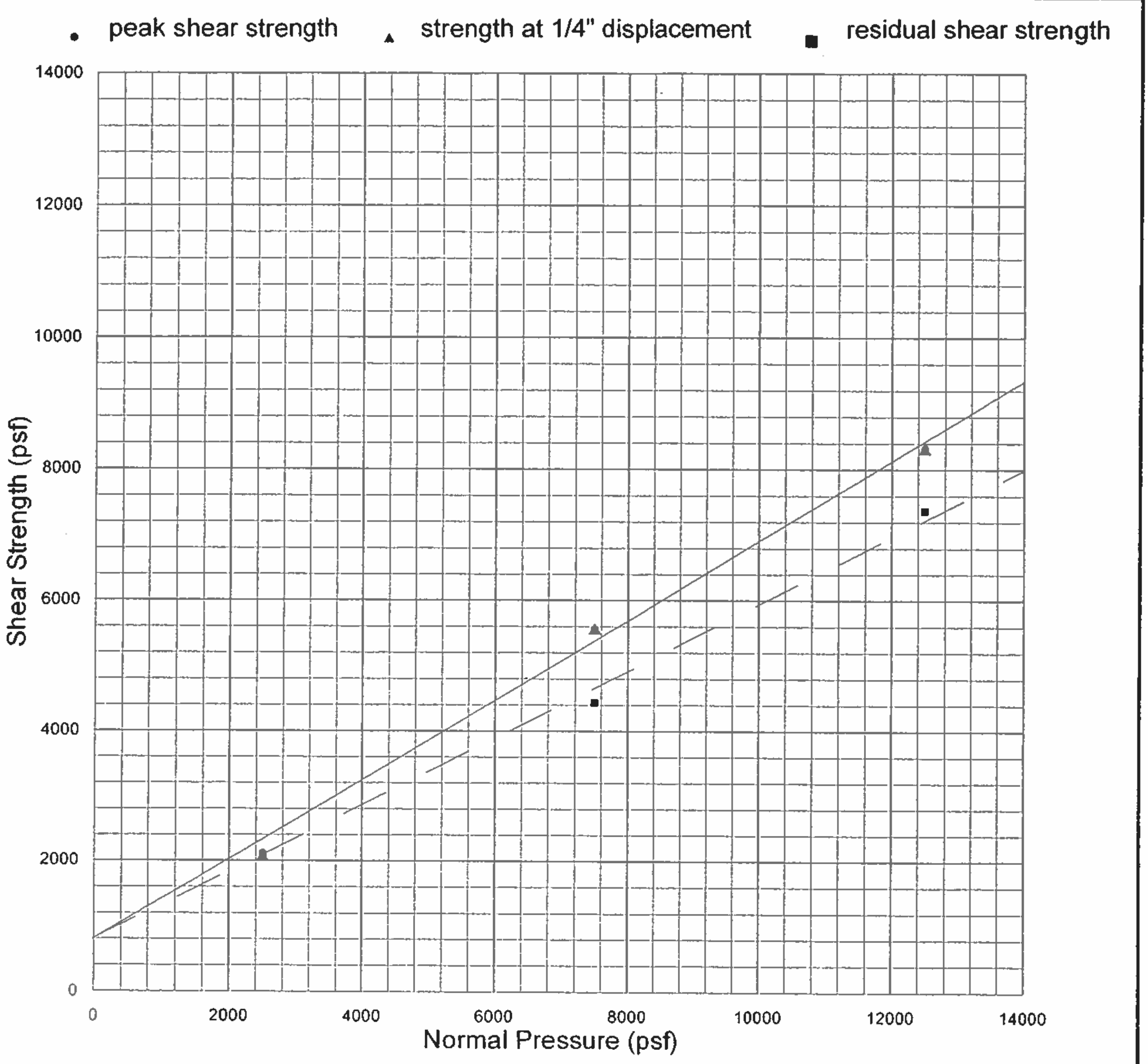
SouthCoast LF - Mendocino County



Strain Rate: 0.0042 in. / min.

Sample	Type	Description	Dry Density (pcf)	Initial W.C. (%)
B-1/4 @ 20'	Undisturbed & Saturated	Silty Clay w. Rock Frgmts	77.1	21.0

Normal Pressure (psf)	Peak Strength (psf)	Ultimate Strength (psf)	Residual Strength (psf)
2500	1480 @ 0.2370"	1460	1260
7500	6110 @ 0.2320"	6050	5090
12500	7880 @ 0.2430"	7880	6540
	C = 3400 psf	C = 3400 psf	C = 2800 psf
	ϕ = 19 deg.	ϕ = 19 deg.	ϕ = 17 deg.

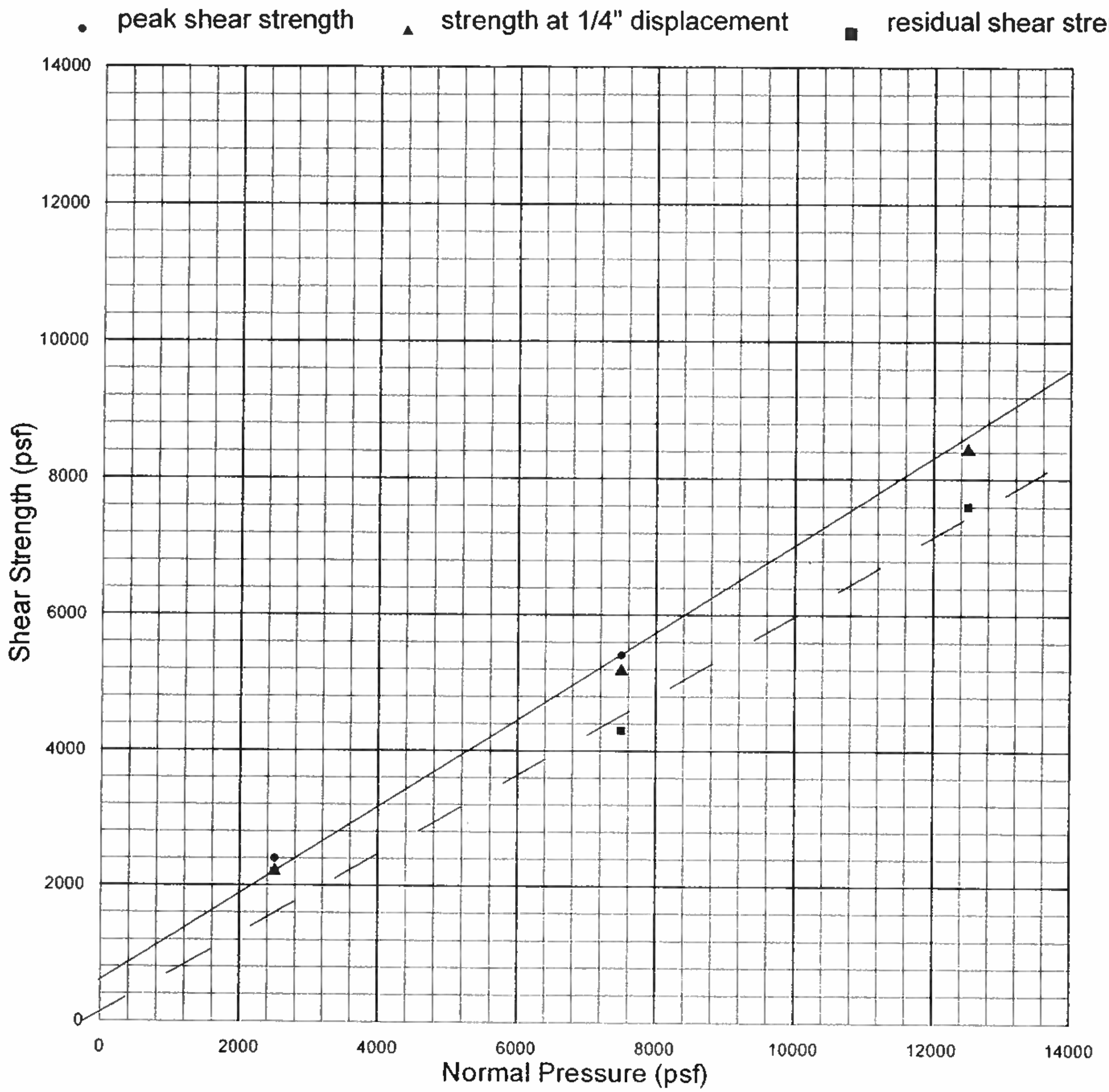


Strain Rate: 0.0042 in. / min.

Sample	Type	Description	Dry Density (pcf)	Initial W.C. (%)
B-4/1 @ 5'	Remolded & Saturated	Silty Clay	110.9	13.8

Normal Pressure (psf)	Peak Strength (psf)	Ultimate Strength (psf)	Residual Strength (psf)
2500	2120 @ 0.1930"	2050	2050
7500	5560 @ 0.2470"	5540	4430
12500	8330 @ 0.2300"	8290	7370
	C = 800 psf	C = 800 psf	C = 800 psf
	ϕ = 31 deg.	ϕ = 31 deg.	ϕ = 27 deg.

DIRECT SHEAR TEST - ASTM D-3080



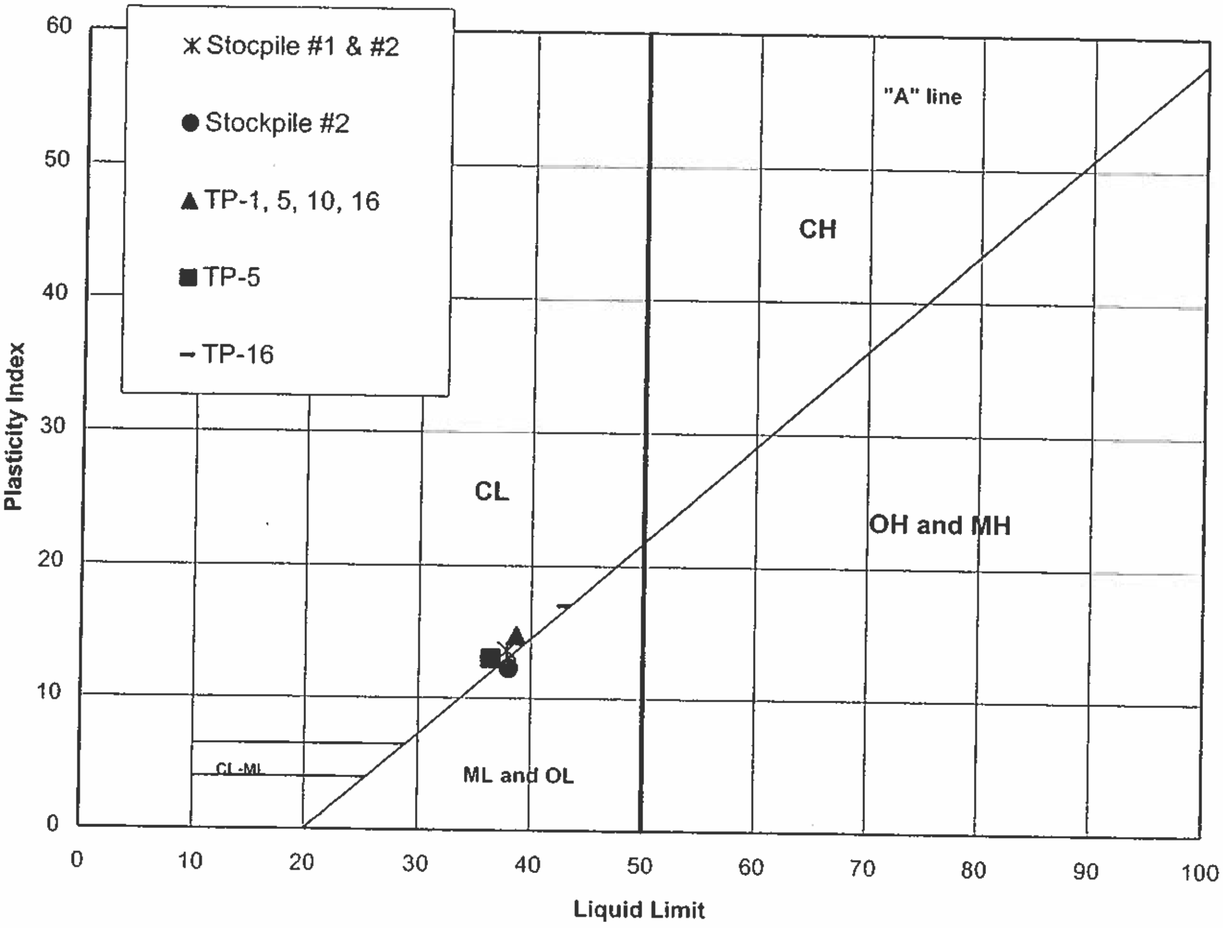
Strain Rate: 0.0042 in. / min.

Sample	Type	Description	Dry Density (pcf)	Initial W.C. (%)
B-2/1 @ 10'	Remolded & Saturated	Silty Clay	103.5	22.0

Normal Pressure (psf)	Peak Strength (psf)	Ultimate Strength (psf)	Residual Strength (psf)
2500	2280 @ 0.1585"	2150	1860
7500	5470 @ 0.1550"	5290	5290
12500	9240 @ 0.2260"	9240	8280
	C = 600 psf	C = 600 psf	C = 150 psf
	ϕ = 33 deg.	ϕ = 33 deg.	ϕ = 30 deg.

PLASTICITY INDEX – ASTM D4318

PLASTICITY INDEX _ ASTM D4318



Sample	Depth	LL	PL	PI	USCS	Material Description
Stocpile #1 & #2		38	24	14	CL	CLAYEY SILT
Stockpile #2		38	26	12	ML	CLAYEY SILT
TP-1, 5, 10, 16		39	24	15	CL	CLAYEY SILT
TP-5		36	23	13	CL	CLAYEY SILT
TP-16		43	26	17	CL	CLAYEY SILT

Job Name SouthCoast LF

Date: 12-9-02

Job No.: 2001-082

HYDRAULIC CONDUCTIVITY – ASTM D5084

HYDRAULIC CONDUCTIVITY - ASTM D5084

Job Name SouthCoast LF

Job No.: 2001-082

Sample: Stockpile #1 & #2
(Remolded @ 93% Max & 120& OMC)

By:LD

Sample Type: ☐ Undisturbed ● Remolded

Soil Type: Brown Mottled, Silty Clay w. Siltstone Frgmts

Ini. Height (in.) :	2	***Final***	
Ini. Diam. (in.) :	2.41	Height (in.) :	2.016
Wet + Tare (grams):	319.2	Diam. (in.) :	2.41
Tare (grams) :	0	Wet Weight (gr.):	333
Water Content (%) :	14	Water Content (%):	18.9
Dry Density (pcf):	116.9	Dry Density (pcf):	116.0

Saturation

Cell Pressure	Back P.	Pore P.	B	Cell Burette	Sample Burette
10	7			-1.7	5.9
20	17			-1	4.3
30	27			-0.1	1.4
40	37	37		0.2	1

Consolidation

Cell Pressure (psi)	Back Pressure (Top)	Back Pressure (Bottom)	Pressure Difference (psi)	Top Burette (cc)	Bottom Burette (cc)	Date&Time Start/Stop	h (cm) h_0/h_1	Q (cc) Top/Bott.	Test Time, t (sec)	Coefficient of Permeability, k (cm/sec)	Remarks (L/A)
40	37.5	36.5	1	0	24	7:12:00	27.48	5.7		7.0E-07	5.12064
				5.7	18.3	11:30:40	14.427	5.7	15520		29.4151573
40	37.5	36.5	1	0	24	11:32:25	27.48	5.9		6.7E-07	
				5.9	18.1	16:12:00	13.969	5.9	16775		
40	37.5	36.5	1	0	24	6:58:00	27.48	2.8		6.4E-07	
				2.8	21.6	9:01:35	21.526	2.4	7415		
40	37.5	36.5	1	2.8	21.6	9:02:00	21.526	7.1		6.6E-07	
				9.9	14.2	15:24:15	4.9235	7.4	22935		
										6.7E-07	$k = \frac{aL}{2\,At} \ln \frac{h_o}{h_i}$
										(Average)	

HYDRAULIC CONDUCTIVITY - ASTM D5084

Job Name SouthCoast LF
Job No.: 2001-082
Sample: Stockpile #1 & #2
By:LD

Sample Type: ☐ Undisturbed ● Remolded

Soil Type: Brown Mottled, Silty Clay w. Siltstone Frgmts

		Final	
Ini. Height (in.) :	2		
Ini. Diam. (in.) :	2.41	Height (in.) :	2.058
Wet + Tare (grams):	388.2	Diam. (in.) :	2.435
Tare (grams) :	84	Wet Weight (gr.):	329.9
Water Content (%) :	12	Water Content (%):	21.5
Dry Density (pcf):	113.4	Dry Density (pcf):	108.0

Saturation

Cell Pressure	Back P.	Pore P.	B	Cell Burette	Sample Burette
10	7			-3.4	9.5
20	17			-0.2	7.2
30	27			-0.7	2.7
40	37	37		0	1.9

Consolidation

Cell Pressure (psi)	Back Pressure (Top)	Back Pressure (Bottom)	Pressure Difference (psi)	Top Burette (cc)	Bottom Burette (cc)	Date&Time Start/Stop	h (cm) h_0/h_1	Q (cc) Top/Bott.	Test Time, t (sec)	Coefficient of Permeability, k (cm/sec)	Remarks (L/A)
40	37.5	36.5	1	0	24	6:41:30	27.48	7.3		3.5E-06	5.22732
				7.3	16.8	7:49:35	10.8775	7.2	4085		30.02859559
40	37.5	36.5	1	0	24	7:51:40	27.48	8.5		3.4E-06	
				8.5	15.5	9:14:04	8.015	8.5	4944		
40	37.5	36.5	1	0	24	9:16:25	27.48	19.8		3.2E-06	
				19.8	4.3	13:21:00	-17.7475	19.7	14675		
40	37.5	36.5	1	0	24	13:23:20	27.48	9.2		3.2E-06	
				9.2	14.9	15:00:15	6.5265	9.1	5815		
40	37.5	36.5	1	0	24	15:03:17	27.48	5.5		3.1E-06	
				5.5	18.6	16:00:00	14.9995	5.4	3403		
										3.3E-06 **(Average)**	$k = \frac{aL}{2At} \ln \frac{h_o}{h_i}$

HYDRAULIC CONDUCTIVITY - ASTM D5084

Job Name SouthCoast LF   Job No.: 2001-082   Sample: TP-1, 5, 10, 16   By:LD

Sample Type: ☐ Undisturbed ● Remolded

Soil Type: Brown Mottled, Clayey Silt w. Siltstone Frgmts

		Final	
Ini. Height (in.) :	2		
Ini. Diam. (in.) :	2.41	Height (in.) :	2.058
Wet + Tare (grams):	388.7	Diam. (in.) :	2.435
Tare (grams) :	84	Wet Weight (gr.):	329.9
Water Content (%) :	11	Water Content (%):	20.2
Dry Density (pcf):	114.6	Dry Density (pcf):	109.2

Saturation

Cell Pressure	Back P.	Pore P.	B	Cell Burette	Sample Burette
10	7			-2.8	10.2
20	17			-0.1	4.4
30	27			-0.6	2.1
40	37	37		-0.6	1.9

Consolidation

Cell Pressure (psi)	Back Pressure (Top)	Back Pressure (Bottom)	Pressure Difference (psi)	Top Burette (cc)	Bottom Burette (cc)	Date&Time Start/Stop	h (cm) h_0/h_1	Q (cc) Top/Bott.	Test Time, t (sec)	Coefficient of Permeability, k (cm/sec)	Remarks (L/A)
40	37.5	36.5	1	0	24	6:41:00	27.48	8.1		3.9E-06	5.22732
				8.1	15.9	7:49:05	8.931	8.1	4085		30.02859559
40	37.5	36.5	1	0	24	7:51:05	27.48	9.1		3.7E-06	
				9.1	14.9	9:13:37	6.641	9.1	4952		
40	37.5	36.5	1	0	24	9:16:13	27.48	20.9		3.5E-06	
				20.9	2.9	13:20:30	-20.61	21.1	14657		
40	37.5	36.5	1	0	24	13:23:03	27.48	10.4		3.7E-06	
				10.4	13.6	14:59:45	3.664	10.4	5802		
40	37.5	36.5	1	0	24	15:03:00	27.48	6.4		3.4E-06	
				6.4	18.3	15:59:30	13.6255	5.7	3390		
										3.7E-06 **(Average)**	$k = \frac{aL}{2At} \ln \frac{h_o}{h_i}$

GeoLogic Associates

INTERFACE SHEAR TEST RESULTS – ASTM D5321

Precision Geosynthetic Laboratories

CLIENT: GEOLOGIC ASSOCIATES
PROJECT: South Coast Landfill / Job# 2002-072
INTERFACE SHEAR TEST RESULTS
(PGL Job No. G030242)

SAMPLE IDENTIFICATIONS:

SAMPLE ID	PRECISION CONTROL NUMBER	DATE RECEIVED	ORIGIN OF MATERIAL
60mil Textured LLDPE Geomembrane (R# 107112879)	86154	4/2/03	GSE, TX.
Soil #1	86180	4/4/03	Geologic Assoc.
Soil #2	86181	4/4/03	Geologic Assoc.
Gravel #3	86182	4/4/03	Geologic Assoc.

TESTS REQUIRED:

TEST METHOD	DESCRIPTION
ASTM D5321	Interface Shear

TEST CONDITIONS: The samples were conditioned for a minimum one hour in the laboratory at 22 ± 2°C (71.6 ± 3.6°F) and at 60 ± 10% relative humidity prior to test.

TEST RESULTS:

The test results are summarized in Tables 1 through 3. The units in which the data are reported are included on these tables.

PRECISION GEOSYNTHETIC LABORATORIES

Cora B. Queja
Vice President

TABLE 1

CLIENT: Geologic Associates
PROJECT: South Coast Landfill / Proj# 2002-072

INTERFACE SHEAR TEST RESULT (ASTM D 5321)
PGL Job No. G030242

QC'd by:
Date: 4/11/03

TEST CONFIGURATION # 1

TOP BOX
SOIL #1 (C#86180)
60mil Text. LLDPE (R# 107112879) C#86154
← BOTTOM BOX

TEST CONDITIONS:

SAMPLE PREPARATION:

1. Specimens were cut along machine direction to 14" x 19" for the upper box, and 14" x 17" for the lower box, with an effective test area of 12" x 12".
2. Soil was compacted to 113.9 pcf @ 11.50% moisture content, forming 2 inch layer in the top box.

HYDRATION:

1. No Hydration

CONSOLIDATION:

1. Each set of specimen was consolidated under Wet/Spray condition for 1 hr @ normal load before shearing.
2. Normal loads were applied using Bladder

SHEAR TEST:

1. Shear test was conducted @ 0.04 in/ min.
2. Sheared @ minimum 2.6 inch horizontal displacement.
3. The test specimens were sheared in Wet/Spray condition
4. Test were performed in general accordance with ASTM D6243-98 / ASTM D5321-92 using Brainard-Kilman LG-112 Direct Shear machine with effective area of 12 in X 12 in.

TEST RESULTS:

Normal Stress (psi)	Normal Stress (psf)	FINAL MC (Soil) (%)	PEAK STRENGTH: SHEAR STRESS (psf)	PEAK STRENGTH: PEAK SECANT ANGLE (degrees)	2.6 " DISPLACEMENT STRENGTH: SHEAR STRESS (psf)	2.6 " DISPLACEMENT STRENGTH: 2.6 " DISPLACEMENT SECANT ANGLE (degrees)
0.69	100	12.99	160	58	146	56
1.74	250	12.24	323	52	284	49
3.47	500	12.11	474	43	469	43

	PEAK STRENGTH	2.6 " DISPLACEMENT STRENGTH
COHESION (psf):	101.82	72.8
COEFFICIENT OF FRICTION:	0.77	0.80
FRICTION ANGLE(degrees):	37.5	38.7

NOTE: The friction angles and cohesion results given here are based on mathematically determined best fit line.

OBSERVATIONS:

See Figure #1 and #2

PGL Job Number: G030242 Config.# 1
Project Name: South Coast Landfill / Proj# 2002-072
SOIL #1 (C#86180) / 60mil Text. LLDPE (R# 107112879) C#86154
QC'd By:

Figure #1
Normal Stress/ Interface Stress

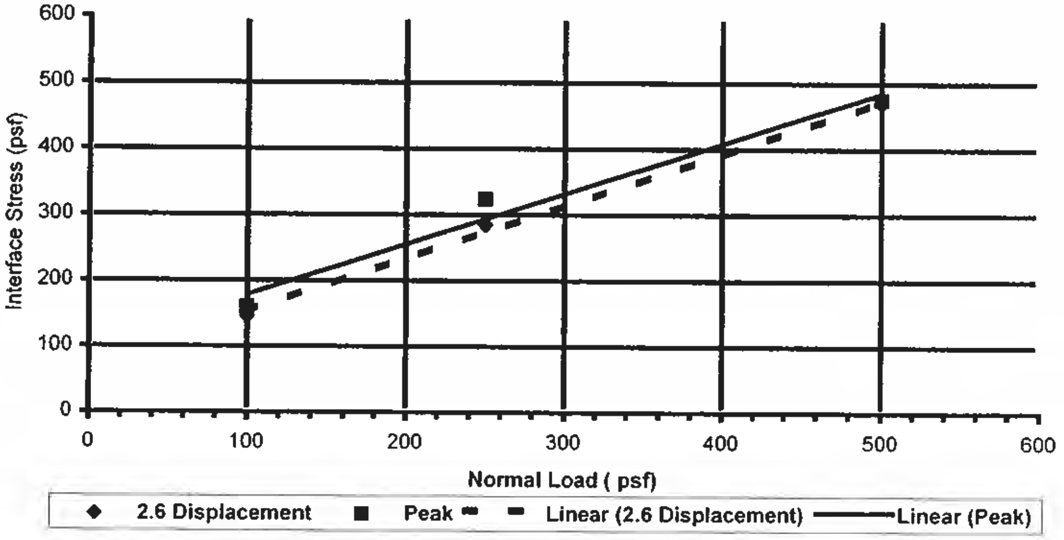
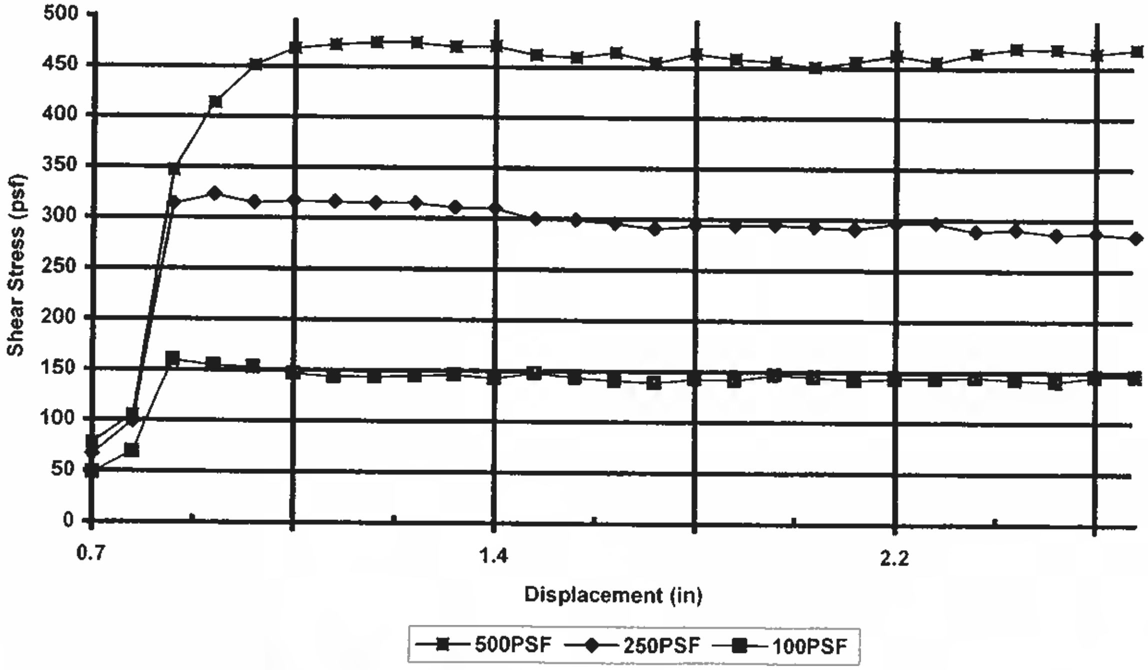


Figure #2
Shear Stress/ Displacement Curve



Precision Geosynthetic Laboratories

ISO 9001 - 2000

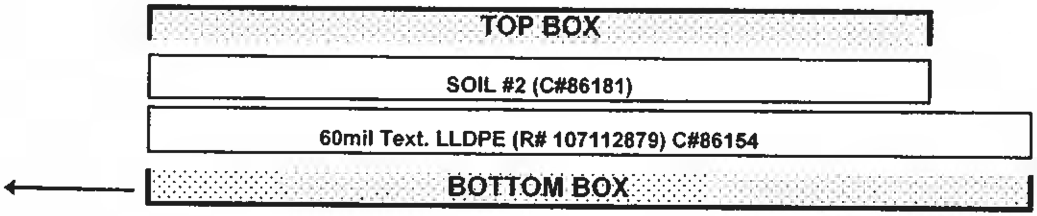
TABLE 2

CLIENT: Geologic Associates
PROJECT: South Coast Landfill / Proj# 2002-072

INTERFACE SHEAR TEST RESULT (ASTM D 5321)
PGL Job No. G030242

QC'd by:
Date: 4/11/03

TEST CONFIGURATION # 2



TEST CONDITIONS:

SAMPLE PREPARATION:

1. Specimens were cut along machine direction to 14" x 19" for the upper box, and 14" x 17" for the lower box, with an effective test area of 12" x 12".
2. Soil was compacted to 111.6 pcf @ 12.50% moisture content, forming 2 inch layer in the top box.

HYDRATION:

1. No Hydration

CONSOLIDATION:

1. Each set of specimen was consolidated under Wet/Spray condition for 1 hr @ normal load before shearing.
2. Normal loads were applied using Bladder

SHEAR TEST:

1. Shear test was conducted @ 0.04 in/ min.
2. Sheared @ minimum 2.6 inch horizontal displacement.
3. The test specimens were sheared in Wet/Spray condition
4. Test were performed in general accordance with ASTM D6243-98 / ASTM D5321-92 using Brainard-Kilman LG-112 Direct Shear machine with effective area of 12 in X 12 in.

TEST RESULTS:

			PEAK STRENGTH		2.6 " DISPLACEMENT STRENGTH	
Normal Stress (psi)	Normal Stress (psf)	FINAL MC (Soil) (%)	SHEAR STRESS (psf)	PEAK SECANT ANGLE (degrees)	SHEAR STRESS (psf)	2.6 " DISPLACEMENT SECANT ANGLE (degrees)
0.69	100	13.61	215	65	209	64
1.74	250	13.6	251	45	239	44
3.47	500	13.05	355	35	263	28

	PEAK STRENGTH	2.6 " DISPLACEMENT STRENGTH
COHESION (psf):	172.59	199.88
COEFFICIENT OF FRICTION:	0.36	0.13
FRICTION ANGLE(degrees):	19.6	7.5

NOTE: The friction angles and cohesion results given here are based on mathematically determined best fit line.

OBSERVATIONS:

See Figure #1 and #2

PGL Job Number: G030242 **Config.#** 2
Project Name: South Coast Landfill / Proj# 2002-072
SOIL #2 (C#86181) / 60mil Text. LLDPE (R# 107112879) C#86154
QC'd By:

Figure #1
Normal Stress/ Interface Stress

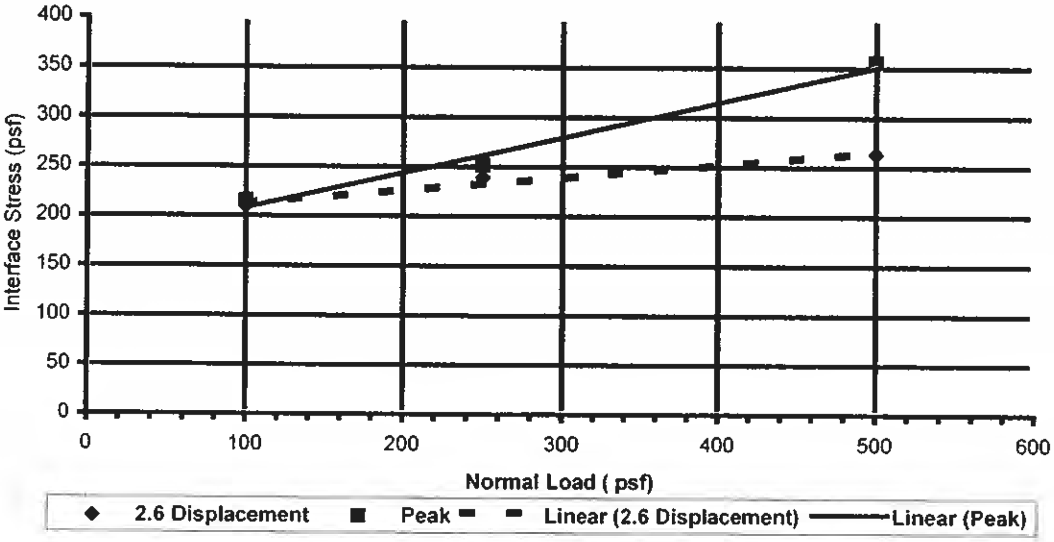
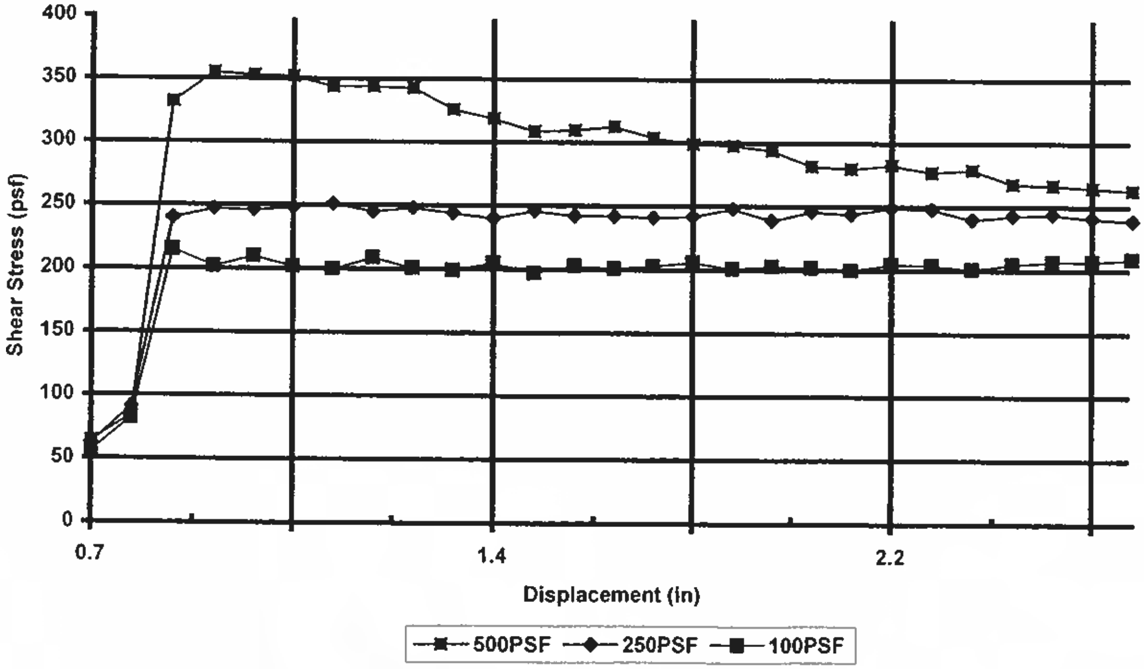


Figure #2
Shear Stress/ Displacement Curve



Precision Geosynthetic Laboratories

ISO 9001 - 2000

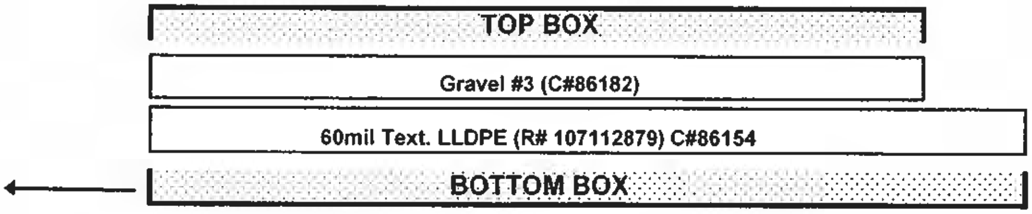
TABLE 3

CLIENT: Geologic Associates
PROJECT: South Coast Landfill / Proj# 2002-072

INTERFACE SHEAR TEST RESULT (ASTM D 5321)
PGL Job No. G030242

QC'd by:
Date: 4/18/03

TEST CONFIGURATION # 3



TEST CONDITIONS:

SAMPLE PREPARATION:

1. Specimens were cut along machine direction to 14" x 19" for the upper box, and 14" x 17" for the lower box, with an effective test area of 12" x 12".
2. Soil was compacted to 86.1 pcf @ 3.60% moisture content, forming 2 inch layer in the top box.

HYDRATION:

1. No Hydration

CONSOLIDATION:

1. Each set of specimen was consolidated under Wet/Spray condition for 2 hrs @ normal load before shearing.
2. Normal loads were applied using Bladder

SHEAR TEST:

1. Shear test was conducted @ 0.04 in/ min.
2. Sheared @ minimum 2.0 inch horizontal displacement.
3. The test specimens were sheared in Wet/Spray condition
4. Test were performed in general accordance with ASTM D6243-98 / ASTM D5321-92 using Brainard-Kilman LG-112 Direct Shear machine with effective area of 12 in X 12 in.

TEST RESULTS:

Normal Stress (psi)	Normal Stress (psf)	PEAK STRENGTH: SHEAR STRESS (psf)	PEAK STRENGTH: PEAK SECANT ANGLE (degrees)	2.0 " DISPLACEMENT STRENGTH: SHEAR STRESS (psf)	2.0 " DISPLACEMENT STRENGTH: 2.0 " DISPLACEMENT SECANT ANGLE (degrees)
0.69	100	163	58	147	56
1.74	250	255	46	228	42
3.47	500	449	42	432	41

	PEAK STRENGTH	2.0 " DISPLACEMENT STRENGTH
COHESION (psf):	85	64
COEFFICIENT OF FRICTION:	0.72	0.72
FRICTION ANGLE(degrees):	35.8	35.9

NOTE: The friction angles and cohesion results given here are based on mathematically determined best fit line.

OBSERVATIONS:

(1) See Figure #1 and #2
(2) Same compacted gravel was used in 3 stresses and consolidated for at least 2 hrs.

PGL Job Number: G030242 **Config.#** 3
Project Name: South Coast Landfill / Proj# 2002-072
Gravel #3 (C#86182) / 60mil Text. LLDPE (R# 107112879) C#86154
QC'd By: ____________

Figure #1
Normal Stress/ Interface Stress

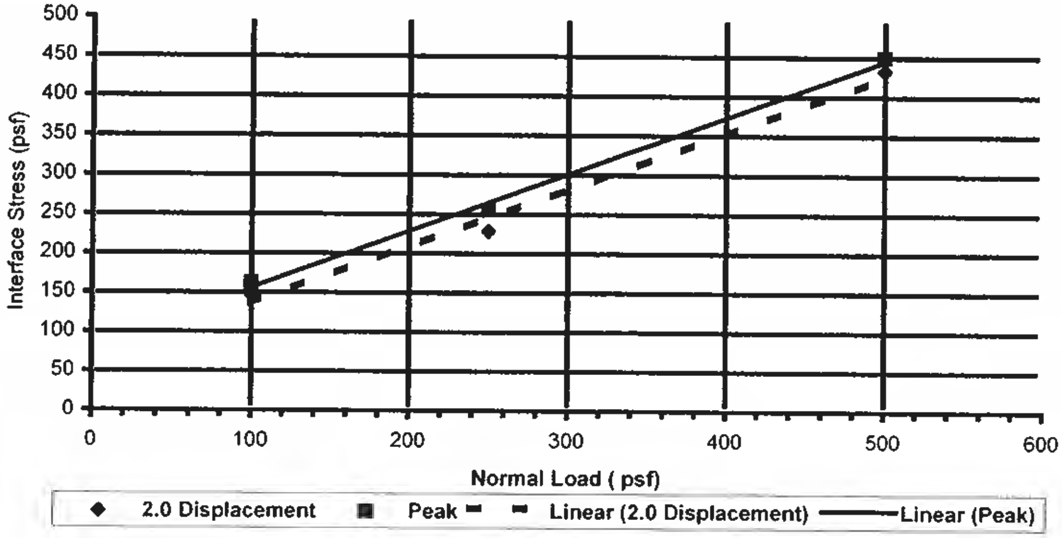
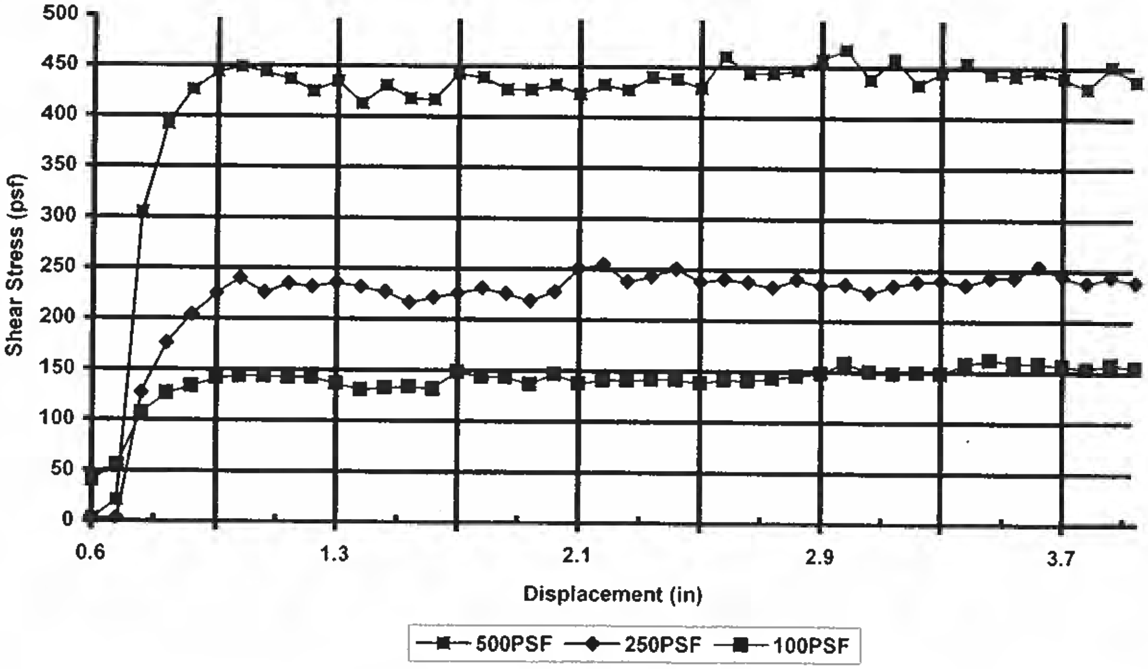


Figure #2
Shear Stress/ Displacement Curve

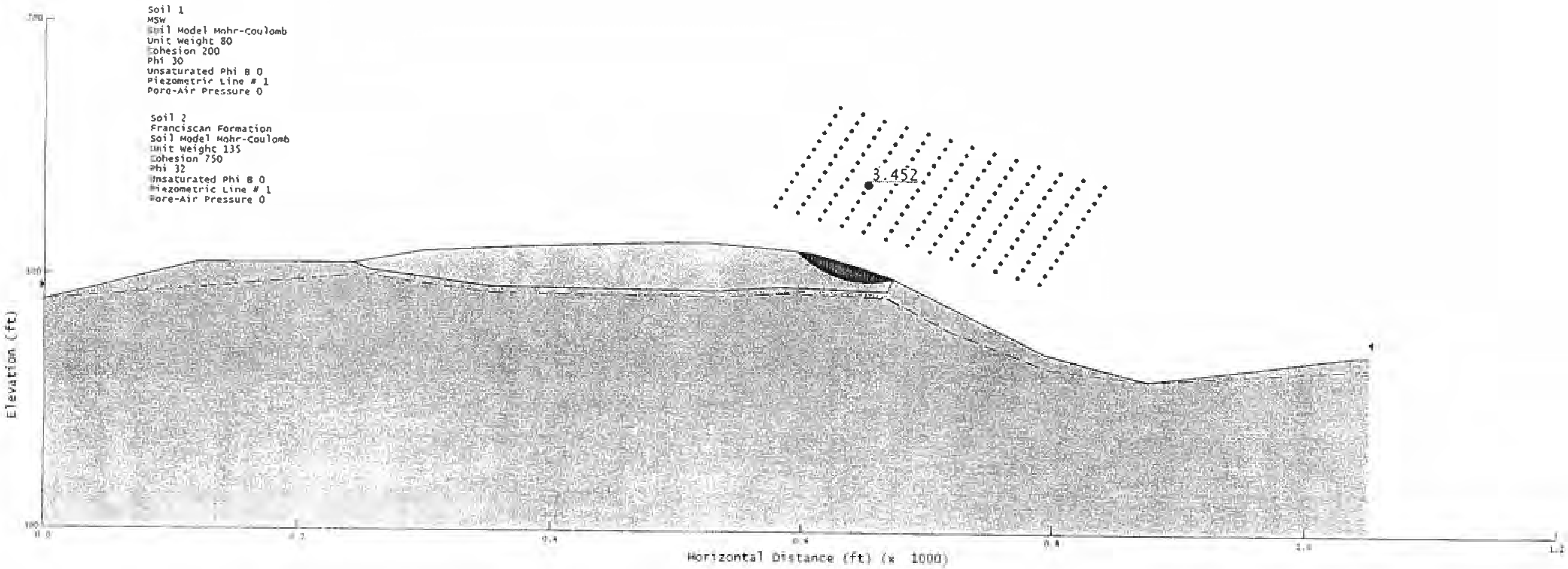


ATTACHMENT C

GEOTECHNICAL ANALYSIS

GROSS LANDFILL STABILITY CALCULATIONS

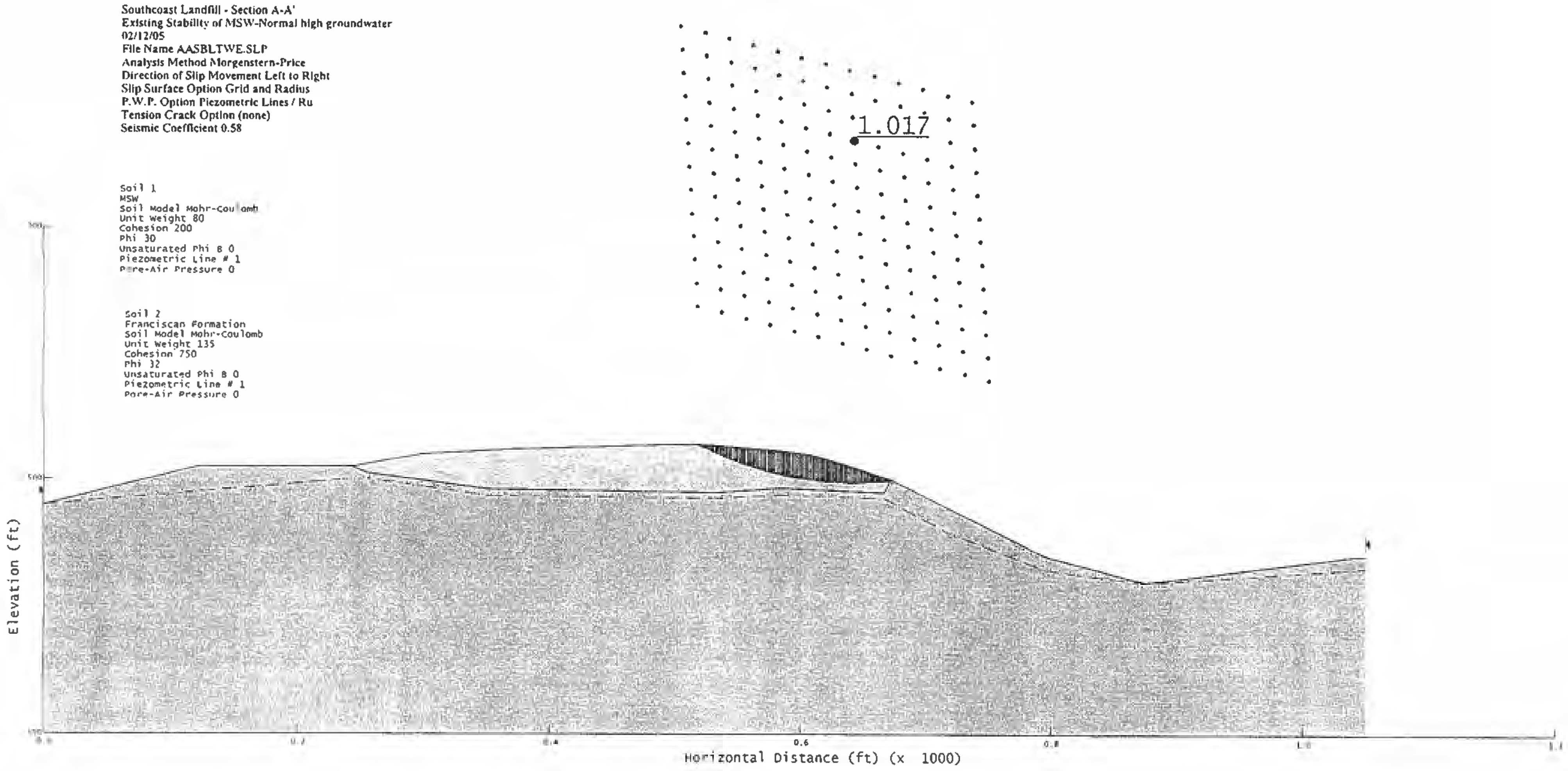
Southcoast Landfill - Section A-A'
Existing Stability of MSW-Normal high groundwater
02/12/05
File Name AAASBLTW.SLP
Analysis Method Morgenstern-Price
Direction of Slip Movement Left to Right
Slip Surface Option Grid and Radius
P.W.P. Option Piezometric Lines / Ru
Tension Crack Option (none)
Seismic Coefficient 0



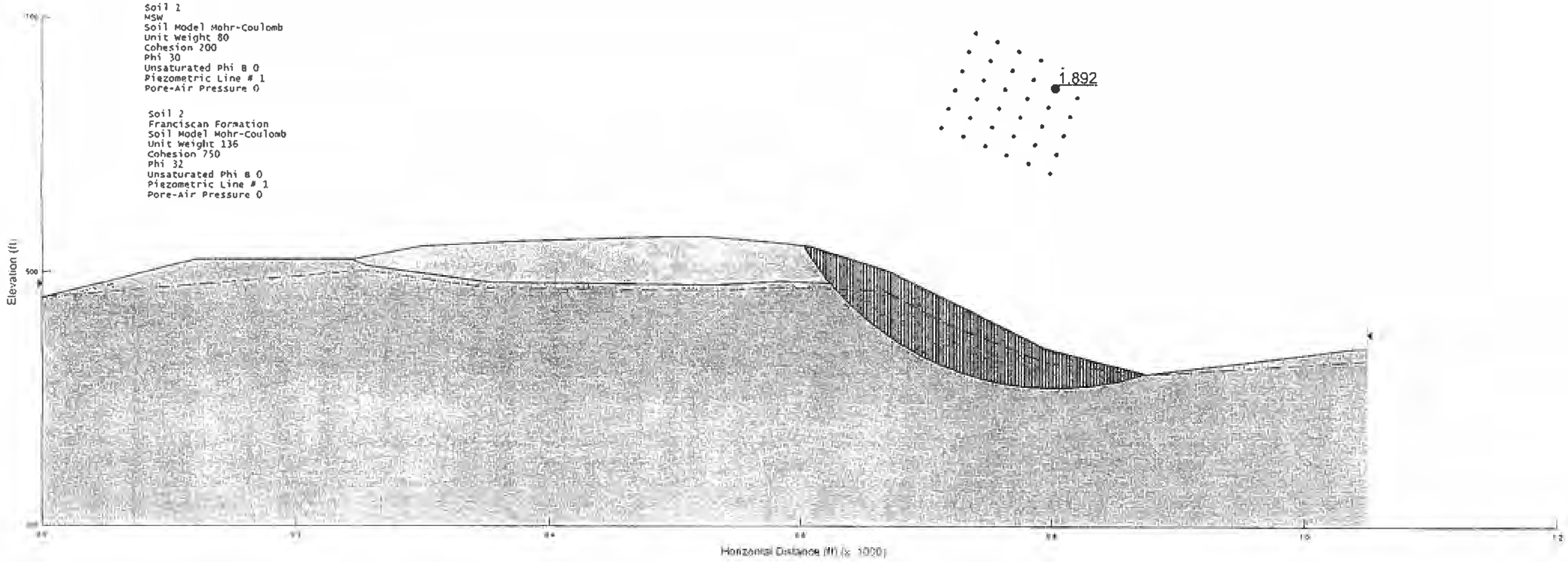
Soil 1
MSW
Soil Model Mohr-Coulomb
Unit Weight 80
Cohesion 200
Phi 30
Unsaturated Phi B 0
Piezometric Line # 1
Pore-Air Pressure 0

Soil 2
Franciscan Formation
Soil Model Mohr-Coulomb
Unit Weight 135
Cohesion 750
Phi 32
Unsaturated Phi B 0
Piezometric Line # 1
Pore-Air Pressure 0

Southcoast Landfill - Section A-A'
Existing Stability of MSW-Normal high groundwater
02/12/05
File Name AASBLTWE.SLP
Analysis Method Morgenstern-Price
Direction of Slip Movement Left to Right
Slip Surface Option Grid and Radius
P.W.P. Option Piezometric Lines / Ru
Tension Crack Option (none)
Seismic Coefficient 0.58



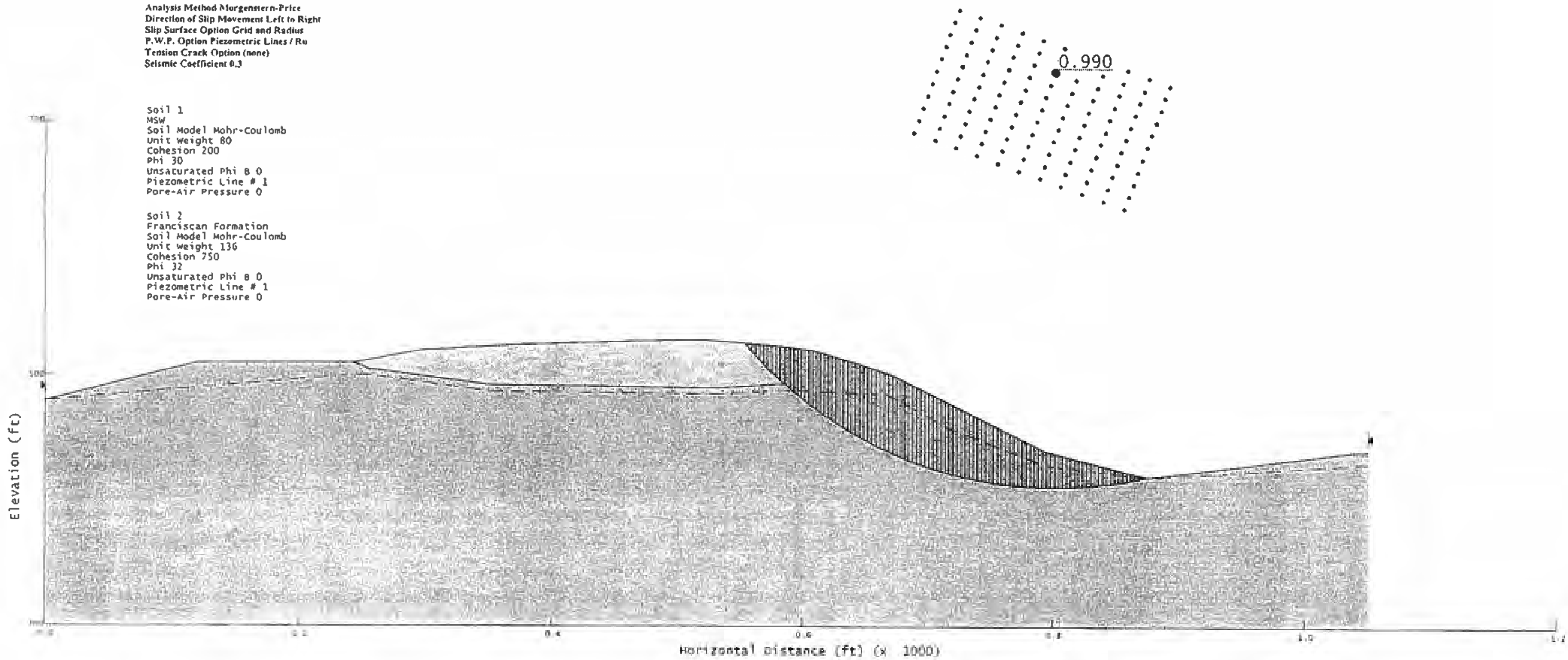
Southcoast Landfill - Section A-A'
Existing Stability of Franciscan Slope and MSW-Normal High groundwater
02/12/07
File Name AAASBLTF.SLP
Analysis Method Morgenstern-Price
Direction of Slip Movement Left to Right
Slip Surface Option Grid and Radius
P.W.P. Option Piezometric Lines / Ru
Tension Crack Option (none)
Seismic Coefficient 0

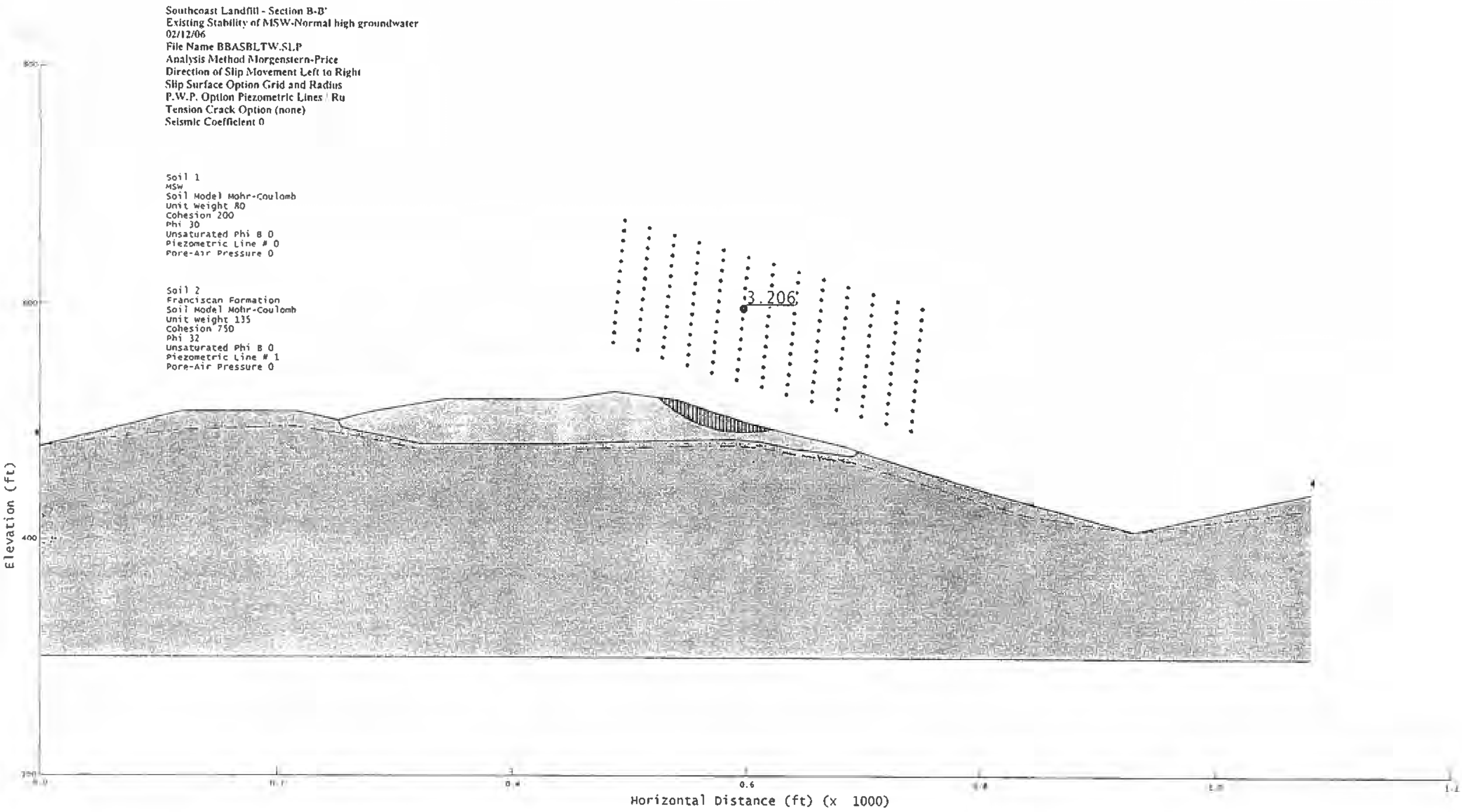


Southeast Landfill - Section A-A'
Existing Dynamic Stability of Franciscan Slope and MSW-Normal high groundwater
02/12/07
File Name AASBLTFE.SLP
Analysis Method Morgenstern-Price
Direction of Slip Movement Left to Right
Slip Surface Option Grid and Radius
P.W.P. Option Piezometric Lines / Ru
Tension Crack Option (none)
Seismic Coefficient 0.3

Soil 1
MSW
Soil Model Mohr-Coulomb
Unit Weight 80
Cohesion 200
Phi 30
Unsaturated Phi B 0
Piezometric Line # 1
Pore-Air Pressure 0

Soil 2
Franciscan Formation
Soil Model Mohr-Coulomb
Unit Weight 136
Cohesion 750
Phi 32
Unsaturated Phi B 0
Piezometric Line # 1
Pore-Air Pressure 0

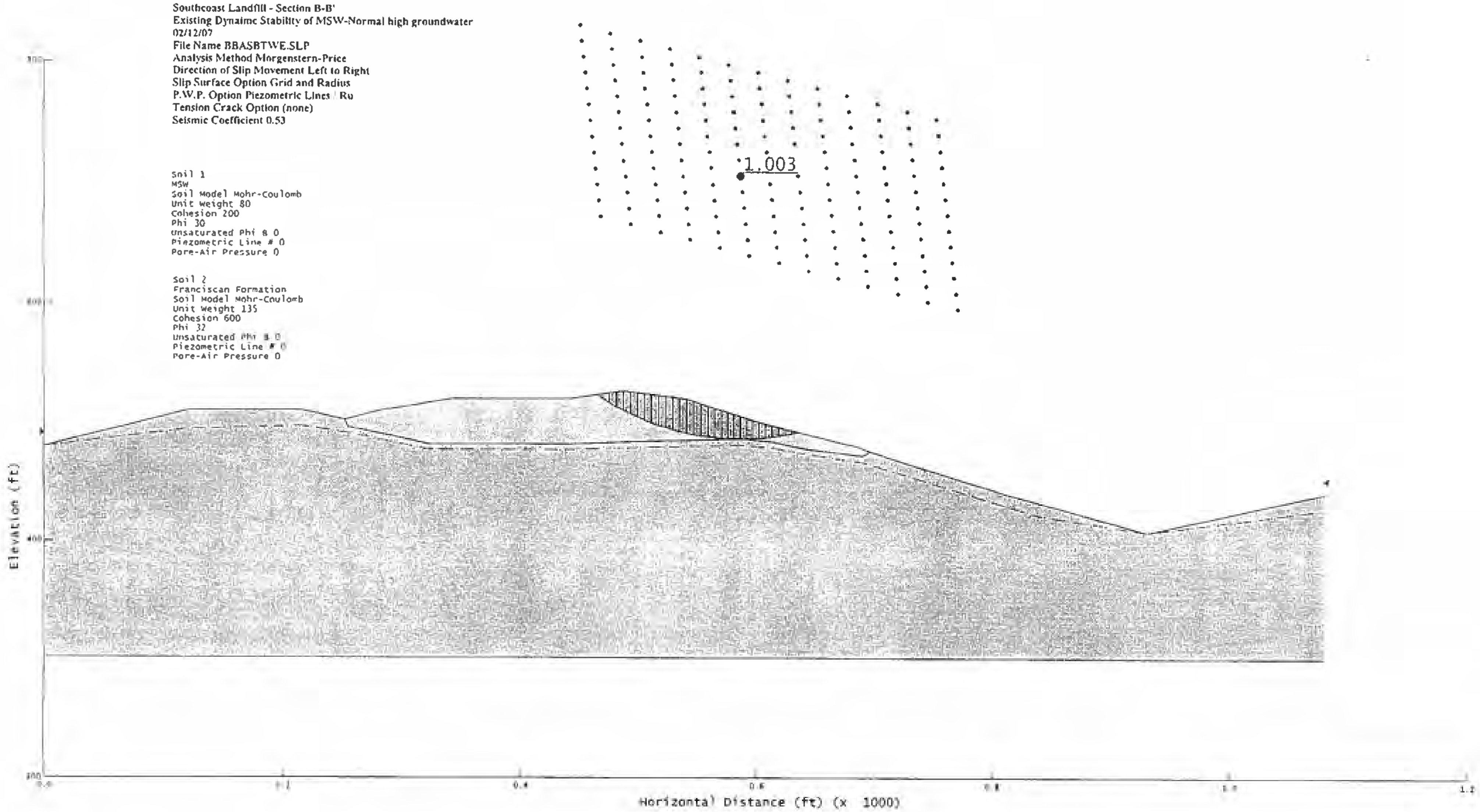




Southcoast Landfill - Section B-B'
Existing Stability of MSW-Normal high groundwater
02/12/06
File Name BBASBLTW.SLP
Analysis Method Morgenstern-Price
Direction of Slip Movement Left to Right
Slip Surface Option Grid and Radius
P.W.P. Option Piezometric Lines / Ru
Tension Crack Option (none)
Seismic Coefficient 0

Soil 1
MSW
Soil Model Mohr-Coulomb
Unit Weight 80
Cohesion 200
Phi 30
Unsaturated Phi B 0
Piezometric Line # 0
Pore-Air Pressure 0

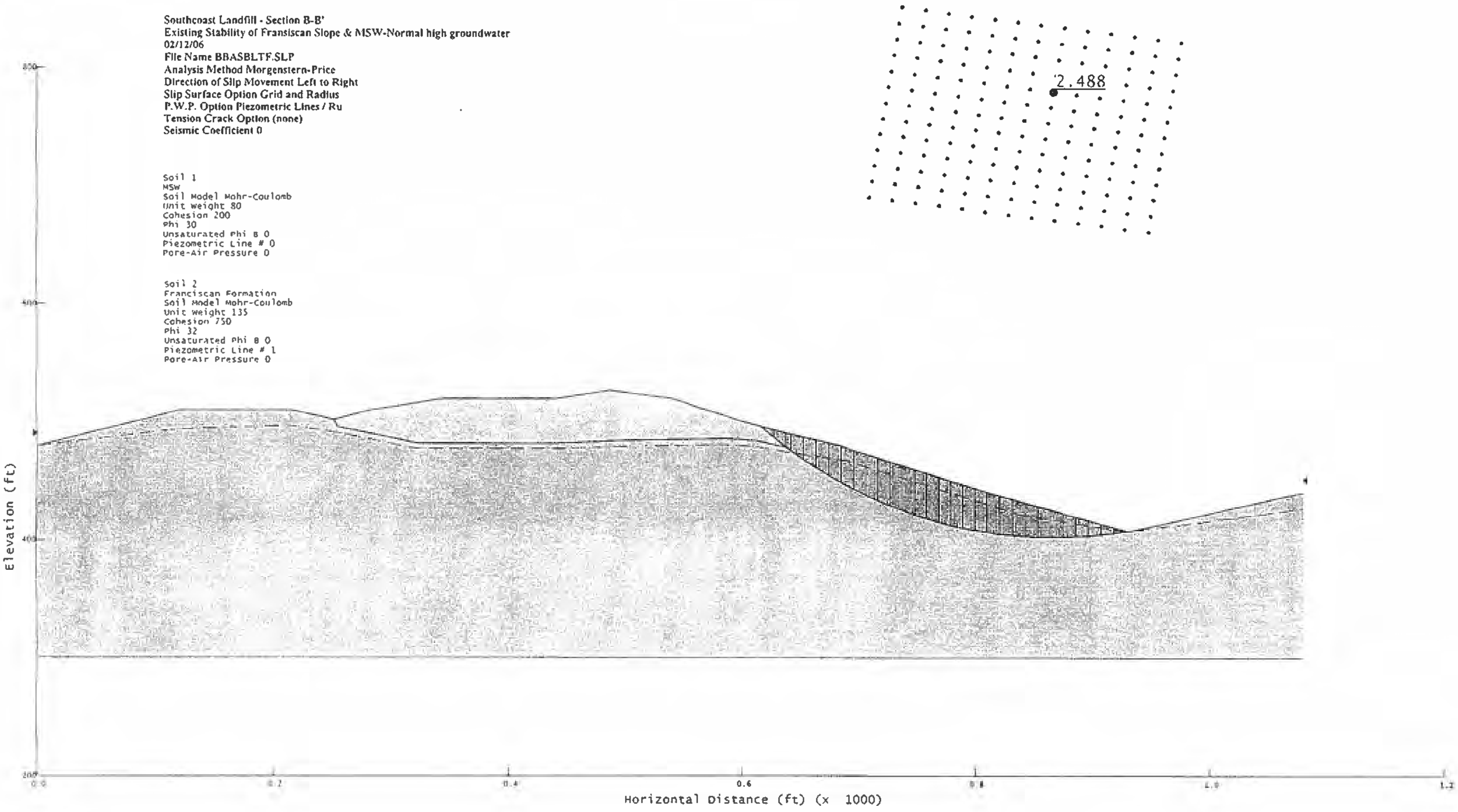
Soil 2
Franciscan Formation
Soil Model Mohr-Coulomb
Unit Weight 135
Cohesion 750
Phi 32
Unsaturated Phi B 0
Piezometric Line # 1
Pore-Air Pressure 0



Southcoast Landfill - Section B-B'
Existing Dynamic Stability of MSW-Normal high groundwater
02/12/07
File Name BBASBTWE.SLP
Analysis Method Morgenstern-Price
Direction of Slip Movement Left to Right
Slip Surface Option Grid and Radius
P.W.P. Option Piezometric Lines / Ru
Tension Crack Option (none)
Seismic Coefficient 0.53

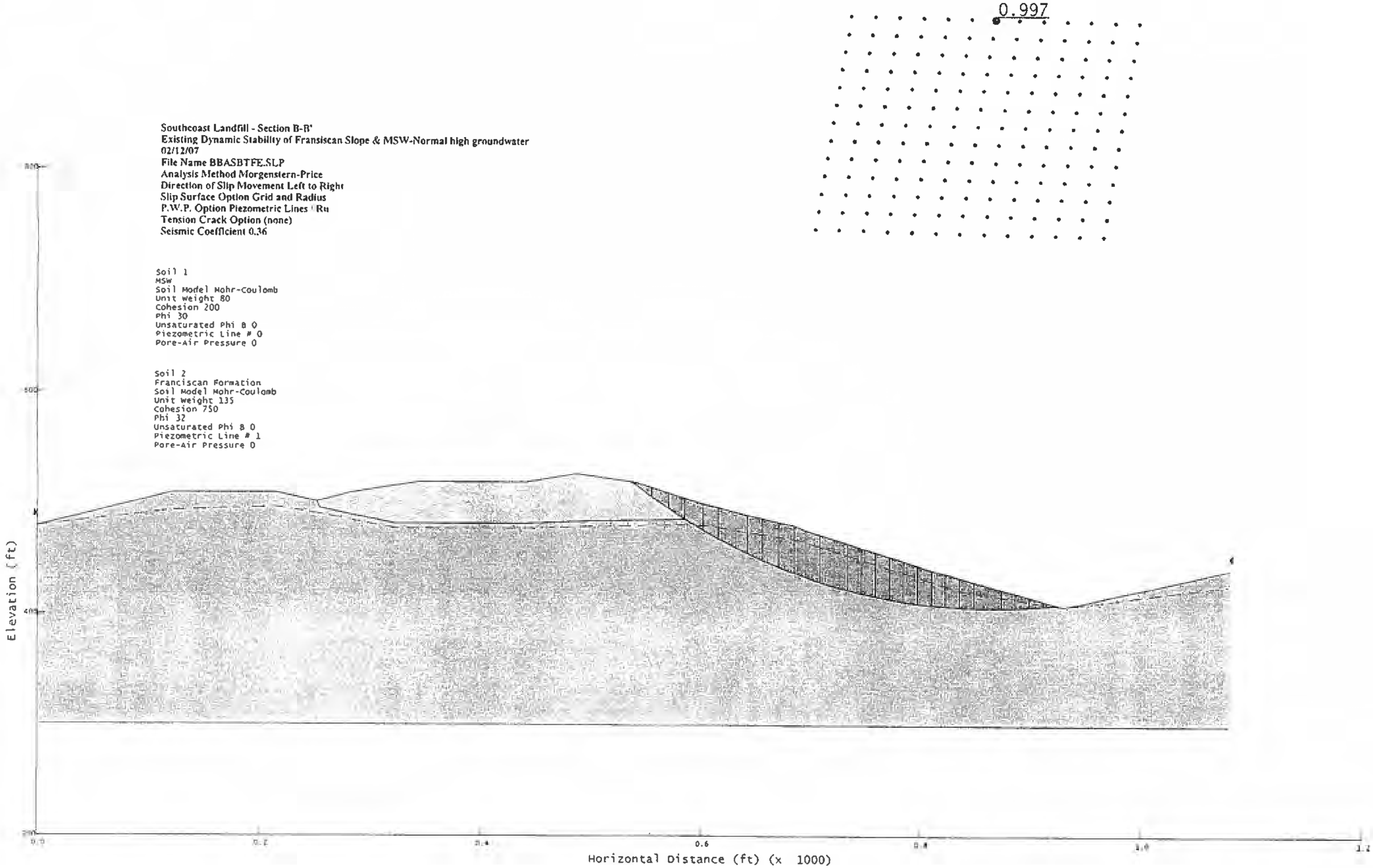
Soil 1
MSW
Soil Model Mohr-Coulomb
Unit Weight 80
Cohesion 200
Phi 30
Unsaturated Phi B 0
Piezometric Line # 0
Pore-Air Pressure 0

Soil 2
Franciscan Formation
Soil Model Mohr-Coulomb
Unit Weight 135
Cohesion 600
Phi 32
Unsaturated Phi B 0
Piezometric Line # 0
Pore-Air Pressure 0


Southcoast Landfill - Section B-B'
Existing Stability of Fransiscan Slope & MSW-Normal high groundwater
02/12/06
File Name BBASBLTF.SLP
Analysis Method Morgenstern-Price
Direction of Slip Movement Left to Right
Slip Surface Option Grid and Radius
P.W.P. Option Piezometric Lines / Ru
Tension Crack Option (none)
Seismic Coefficient 0

Soil 1
MSW
Soil Model Mohr-Coulomb
Unit Weight 80
Cohesion 200
Phi 30
Unsaturated Phi B 0
Piezometric Line # 0
Pore-Air Pressure 0

Soil 2
Franciscan Formation
Soil Model Mohr-Coulomb
Unit Weight 135
Cohesion 750
Phi 32
Unsaturated Phi B 0
Piezometric Line # 1
Pore-Air Pressure 0

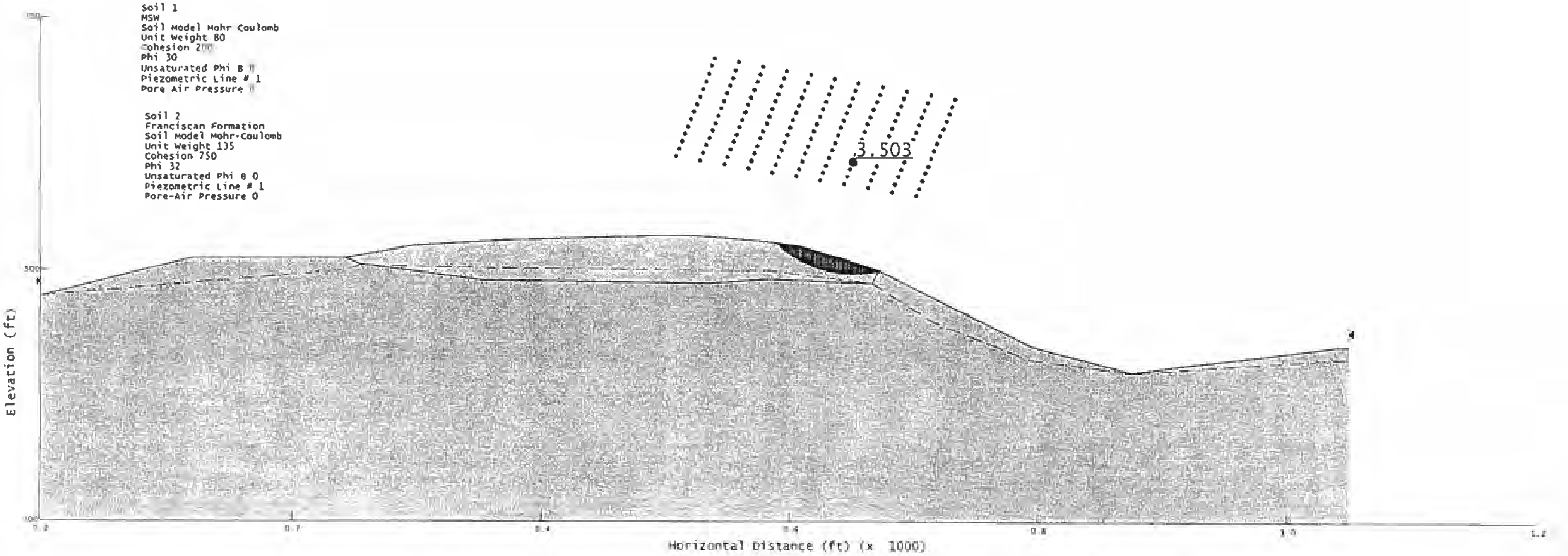


Southcoast Landfill - Section B-B'
Existing Dynamic Stability of Fransiscan Slope & MSW-Normal high groundwater
02/12/07
File Name BBASBTFE.SLP
Analysis Method Morgenstern-Price
Direction of Slip Movement Left to Right
Slip Surface Option Grid and Radius
P.W.P. Option Piezometric Lines / Ru
Tension Crack Option (none)
Seismic Coefficient 0.36

Soil 1
MSW
Soil Model Mohr-Coulomb
Unit Weight 80
Cohesion 200
Phi 30
Unsaturated Phi B 0
Piezometric Line # 0
Pore-Air Pressure 0

Soil 2
Franciscan Formation
Soil Model Mohr-Coulomb
Unit Weight 135
Cohesion 750
Phi 32
Unsaturated Phi B 0
Piezometric Line # 1
Pore-Air Pressure 0

Southcoast Landfill - Section A-A'
Existing Stability of MSW 10' of Saturated MSW
02/12/05
File Name AAASBLTW.SLP
Analysis Method Morgenstern-Price
Direction of Slip Movement Left to Right
Slip Surface Option Grid and Radius
P.W.P. Option Piezometric Lines / Ru
Tension Crack Option (none)
Seismic Coefficient 0



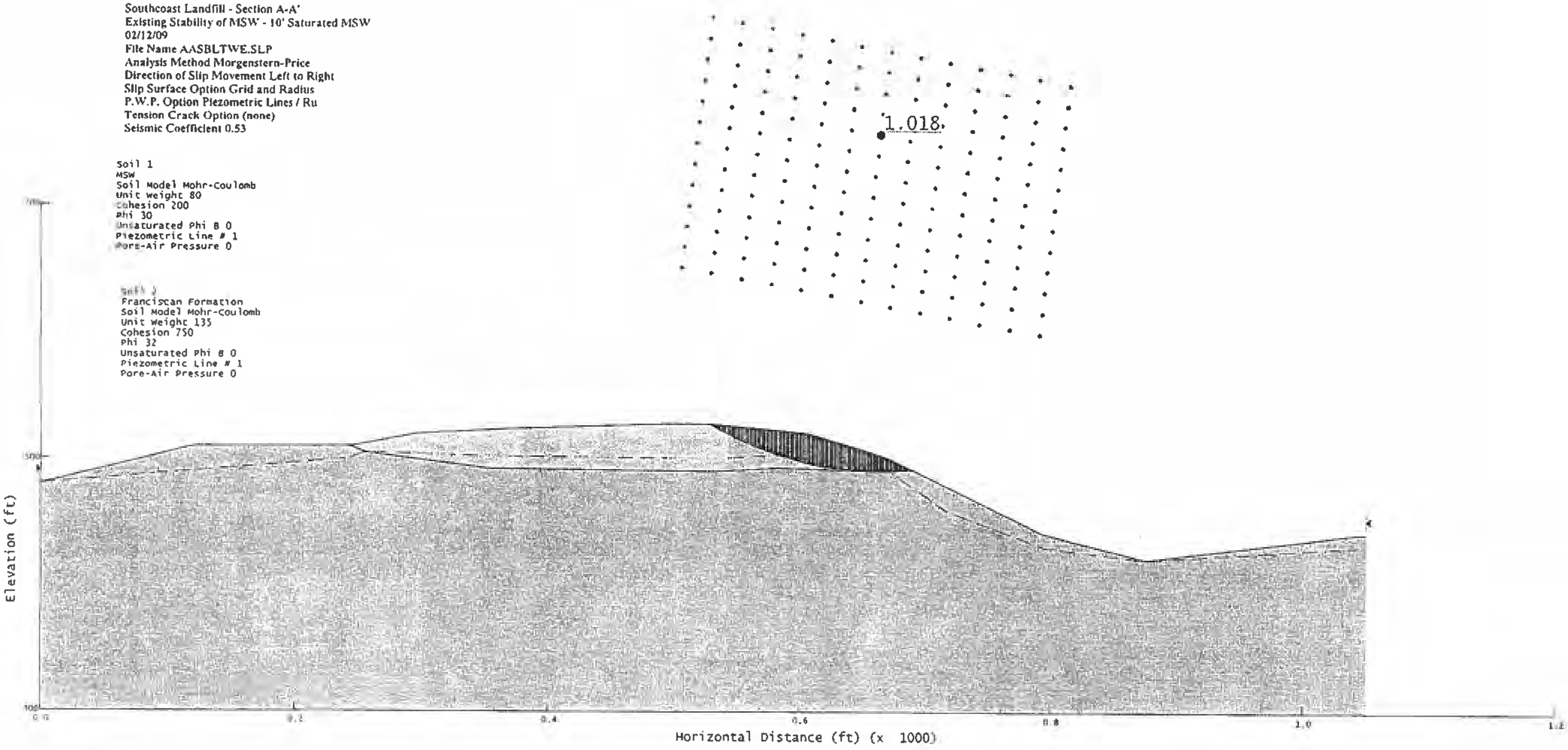
Soil 1
MSW
Soil Model Mohr Coulomb
Unit Weight 80
Cohesion 2[illegible]
Phi 30
Unsaturated Phi B 0
Piezometric Line # 1
Pore Air Pressure 0

Soil 2
Franciscan Formation
Soil Model Mohr-Coulomb
Unit Weight 135
Cohesion 750
Phi 32
Unsaturated Phi B 0
Piezometric Line # 1
Pore-Air Pressure 0

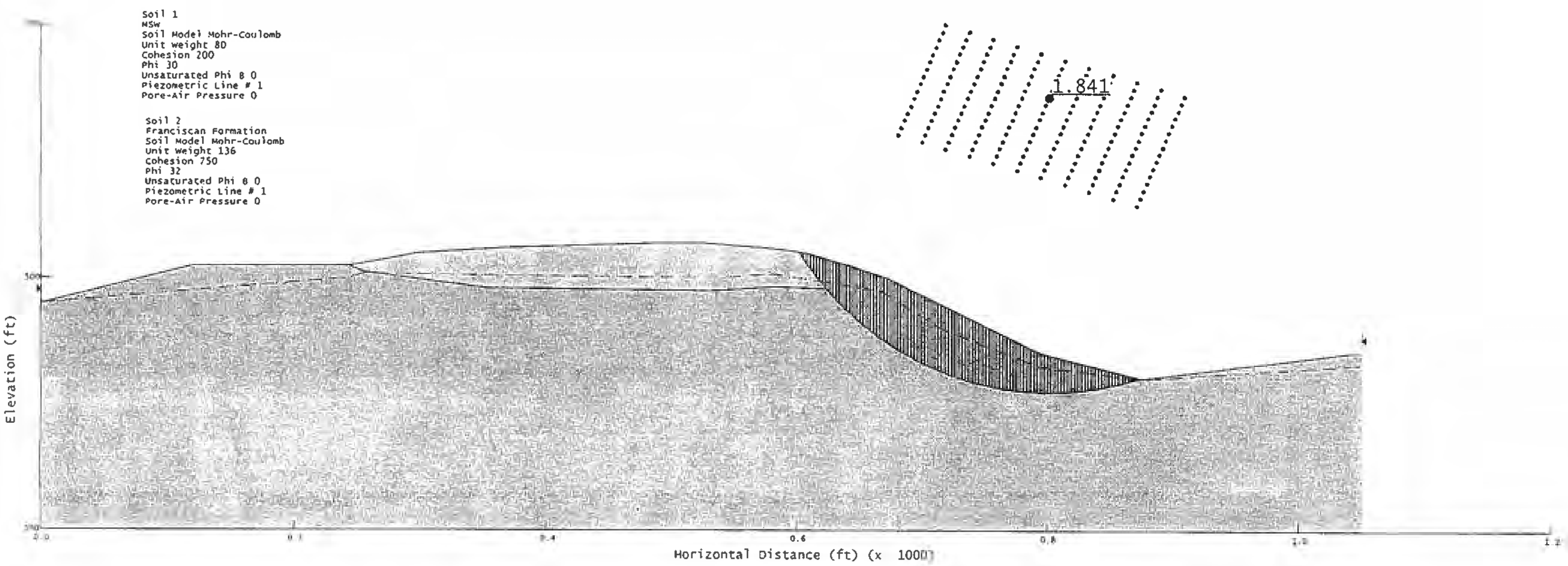
Southcoast Landfill - Section A-A'
Existing Stability of MSW - 10' Saturated MSW
02/12/09
File Name AASBLTWE.SLP
Analysis Method Morgenstern-Price
Direction of Slip Movement Left to Right
Slip Surface Option Grid and Radius
P.W.P. Option Piezometric Lines / Ru
Tension Crack Option (none)
Seismic Coefficient 0.53

Soil 1
MSW
Soil Model Mohr-Coulomb
Unit Weight 80
Cohesion 200
Phi 30
Unsaturated Phi B 0
Piezometric Line # 1
Pore-Air Pressure 0

Soil 2
Franciscan Formation
Soil Model Mohr-Coulomb
Unit Weight 135
Cohesion 750
Phi 32
Unsaturated Phi B 0
Piezometric Line # 1
Pore-Air Pressure 0

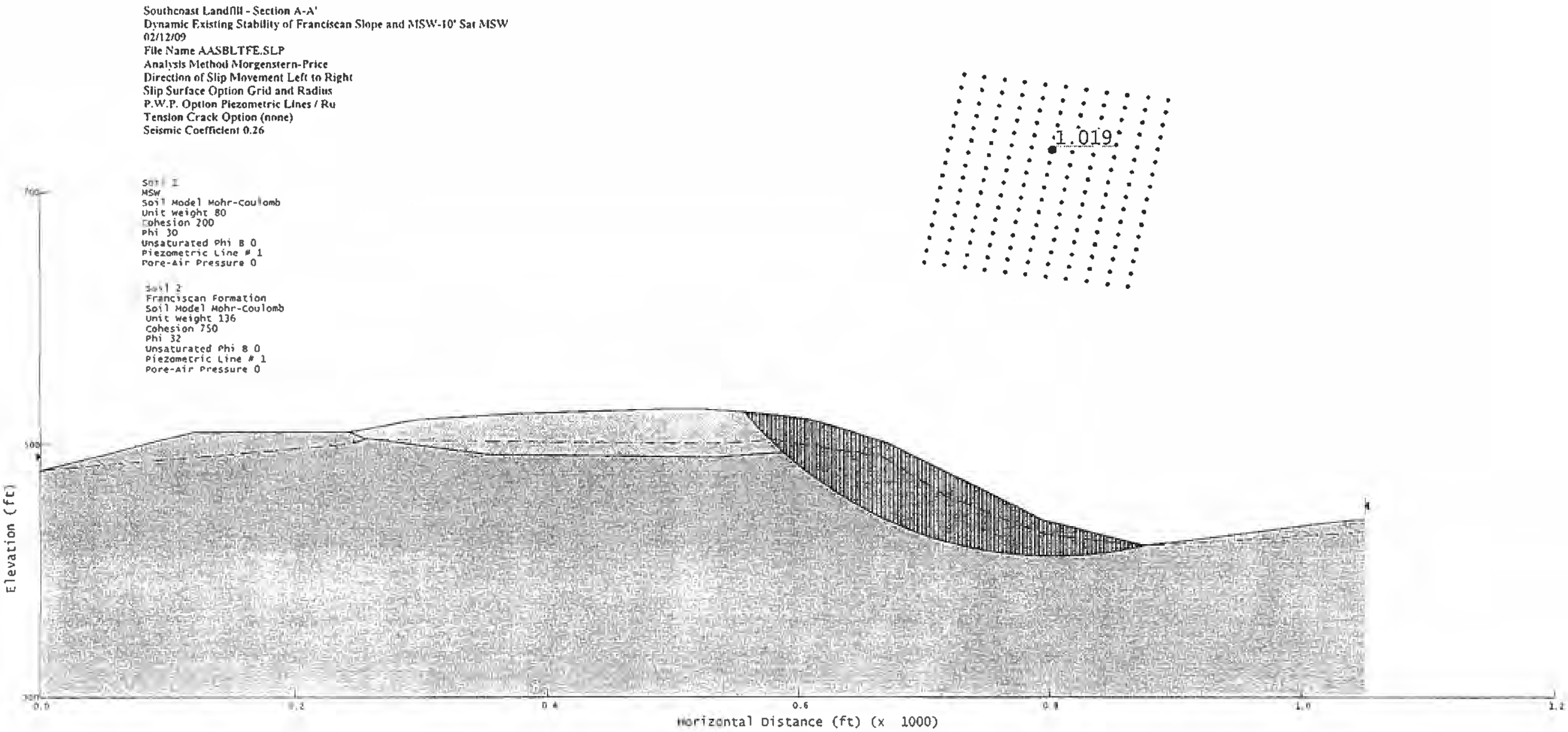


Southcoast Landfill - Section A-A'
Existing Stability of Franciscan Slope and MSW-10' Sat MSW
02/12/07
File Name AAASBLTF.SLP
Analysis Method Morgenstern-Price
Direction of Slip Movement Left to Right
Slip Surface Option Grid and Radius
P.W.P. Option Piezometric Lines / Ru
Tension Crack Option (none)
Seismic Coefficient 0



Soil 1
MSW
Soil Model Mohr-Coulomb
Unit Weight 80
Cohesion 200
Phi 30
Unsaturated Phi B 0
Piezometric Line # 1
Pore-Air Pressure 0

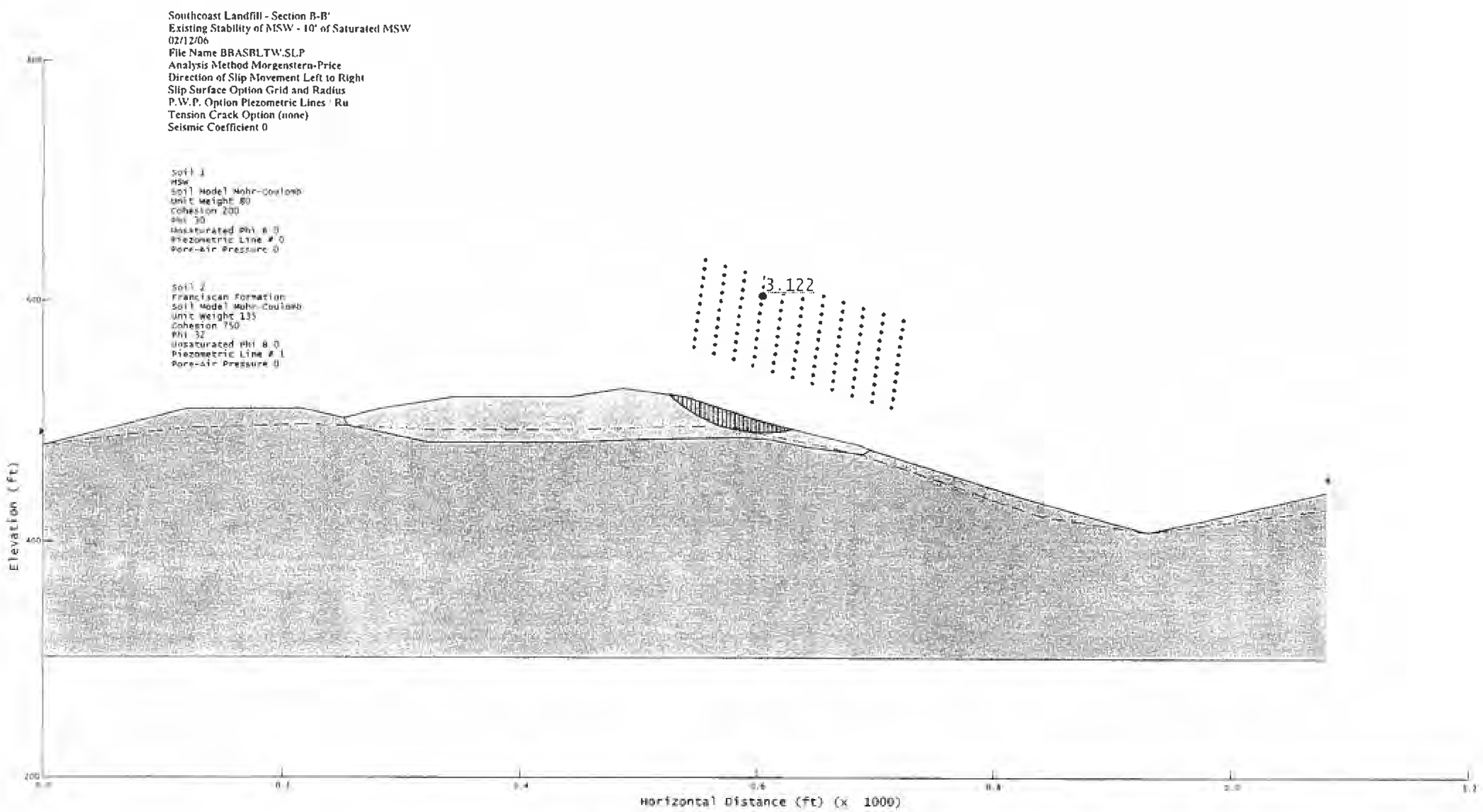
Soil 2
Franciscan Formation
Soil Model Mohr-Coulomb
Unit Weight 136
Cohesion 750
Phi 32
Unsaturated Phi B 0
Piezometric Line # 1
Pore-Air Pressure 0



Southcoast Landfill - Section A-A'
Dynamic Existing Stability of Franciscan Slope and MSW-10' Sat MSW
02/12/09
File Name AASBLTFE.SLP
Analysis Method Morgenstern-Price
Direction of Slip Movement Left to Right
Slip Surface Option Grid and Radius
P.W.P. Option Piezometric Lines / Ru
Tension Crack Option (none)
Seismic Coefficient 0.26

Soil 1
MSW
Soil Model Mohr-Coulomb
Unit Weight 80
Cohesion 200
Phi 30
Unsaturated Phi B 0
Piezometric Line # 1
Pore-Air Pressure 0

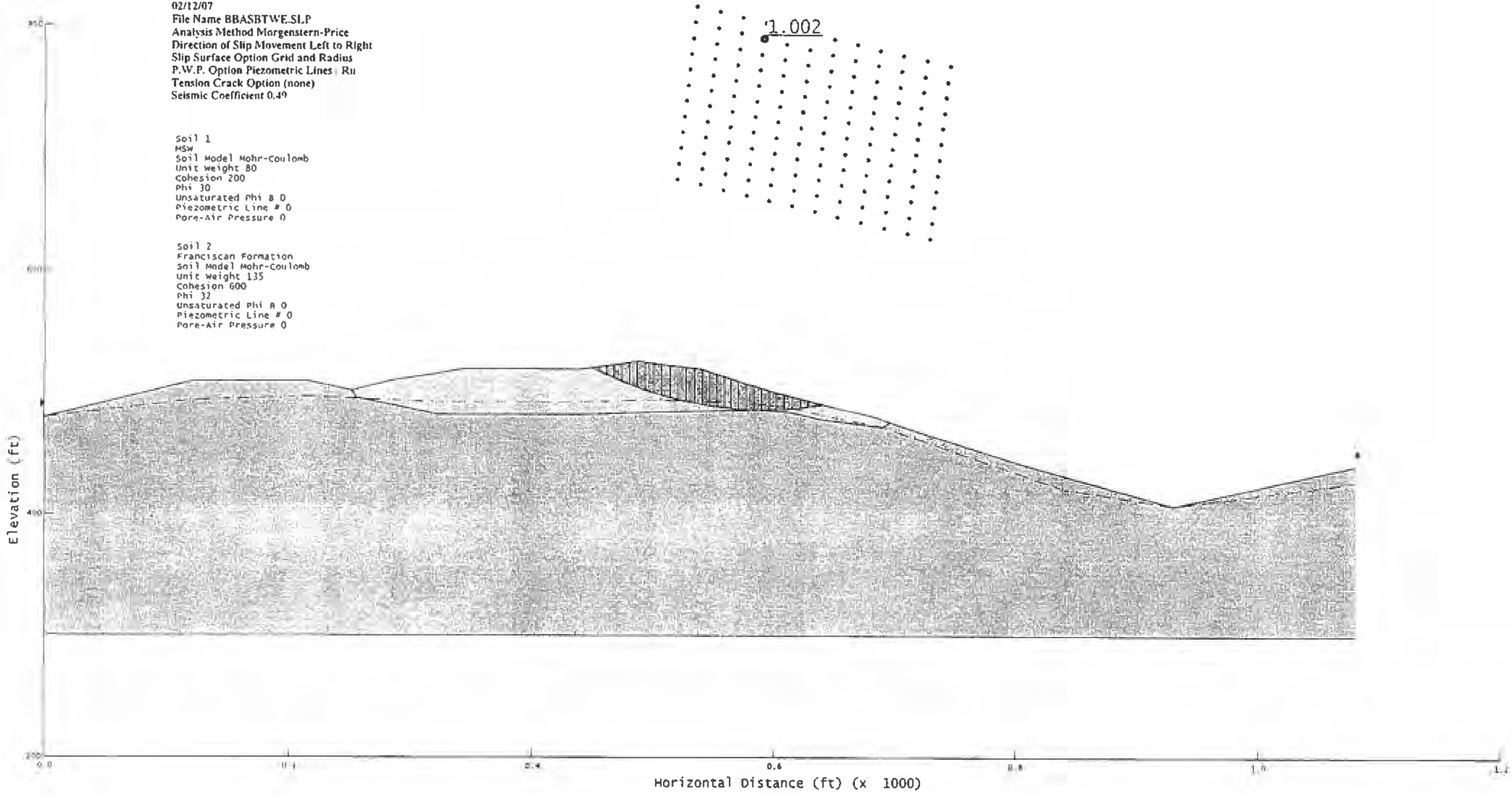
Soil 2
Franciscan Formation
Soil Model Mohr-Coulomb
Unit Weight 136
Cohesion 750
Phi 32
Unsaturated Phi B 0
Piezometric Line # 1
Pore-Air Pressure 0



Southcoast Landfill - Section B-B'
Existing Stability of MSW - 10' of Saturated MSW
02/12/06
File Name BBASBLTW.SLP
Analysis Method Morgenstern-Price
Direction of Slip Movement Left to Right
Slip Surface Option Grid and Radius
P.W.P. Option Piezometric Lines / Ru
Tension Crack Option (none)
Seismic Coefficient 0

Soil 1
MSW
Soil Model Mohr-Coulomb
Unit Weight 80
Cohesion 200
Phi 30
Unsaturated Phi 0.0
Piezometric Line # 0
Pore-Air Pressure 0

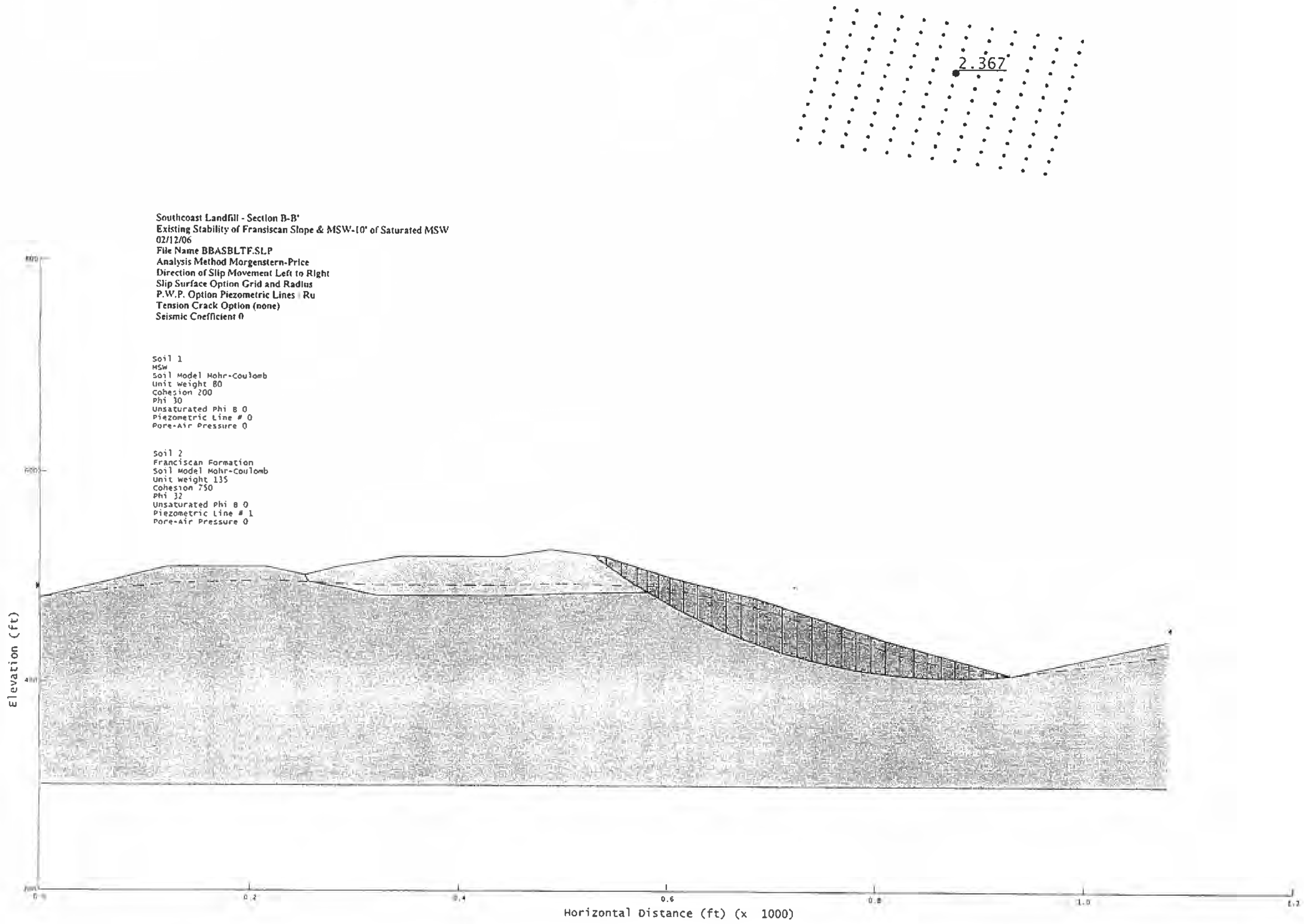
Soil 2
Franciscan Formation
Soil Model Mohr-Coulomb
Unit Weight 135
Cohesion 750
Phi 32
Unsaturated Phi 0.0
Piezometric Line # 1
Pore-Air Pressure 0



Southcoast Landfill - Section B-B'
Existing Dynaimc Stability of MSW-10' of Saturated MSW
02/12/07
File Name BBASBTWE.SLP
Analysis Method Morgenstern-Price
Direction of Slip Movement Left to Right
Slip Surface Option Grid and Radius
P.W.P. Option Piezometric Lines | Ru
Tension Crack Option (none)
Seismic Coefficient 0.49

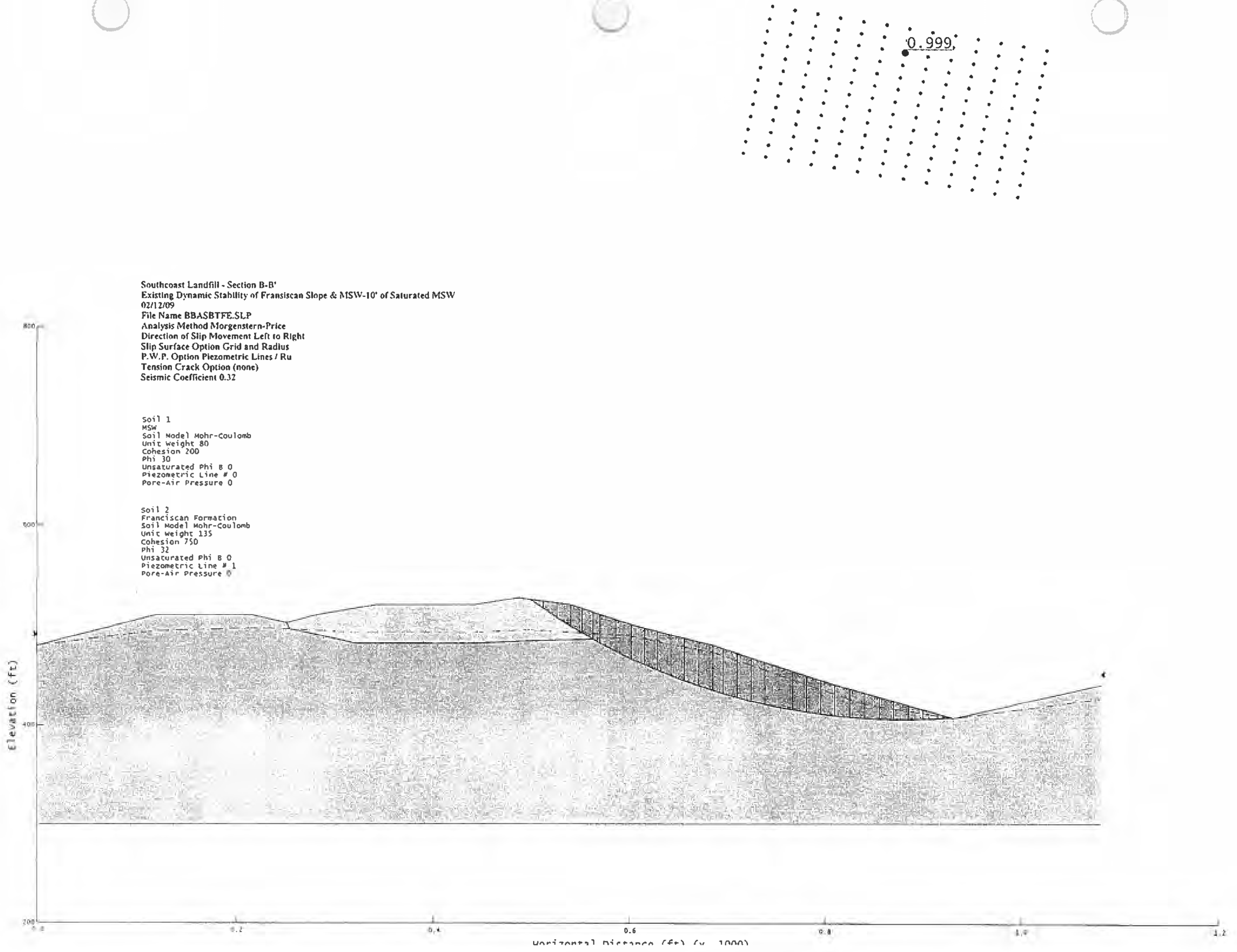
Soil 1
MSW
Soil Model Mohr-Coulomb
Unit Weight 80
Cohesion 200
Phi 30
Unsaturated Phi B 0
Piezometric Line # 0
Pore-Air Pressure 0

Soil 2
Franciscan Formation
Soil Model Mohr-Coulomb
Unit Weight 135
Cohesion 600
Phi 32
Unsaturated Phi B 0
Piezometric Line # 0
Pore-Air Pressure 0


Southcoast Landfill - Section B-B'
Existing Stability of Fransiscan Slope & MSW-10' of Saturated MSW
02/12/06
File Name BBASBLTF.SLP
Analysis Method Morgenstern-Price
Direction of Slip Movement Left to Right
Slip Surface Option Grid and Radius
P.W.P. Option Piezometric Lines | Ru
Tension Crack Option (none)
Seismic Coefficient 0

Soil 1
MSW
Soil Model Mohr-Coulomb
Unit Weight 80
Cohesion 200
Phi 30
Unsaturated Phi B 0
Piezometric Line # 0
Pore-Air Pressure 0

Soil 2
Franciscan Formation
Soil Model Mohr-Coulomb
Unit Weight 135
Cohesion 750
Phi 32
Unsaturated Phi B 0
Piezometric Line # 1
Pore-Air Pressure 0

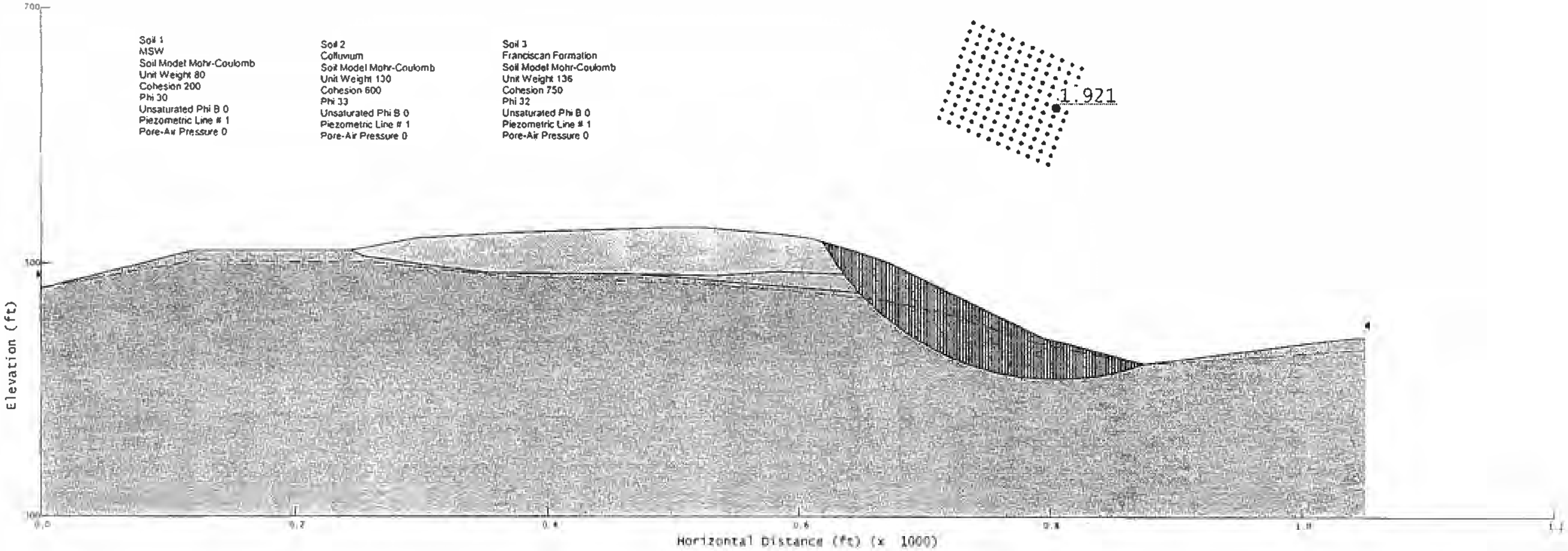


Southcoast Landfill - Section B-B'
Existing Dynamic Stability of Fransiscan Slope & MSW-10' of Saturated MSW
02/12/09
File Name BBASBTFE.SLP
Analysis Method Morgenstern-Price
Direction of Slip Movement Left to Right
Slip Surface Option Grid and Radius
P.W.P. Option Piezometric Lines / Ru
Tension Crack Option (none)
Seismic Coefficient 0.32

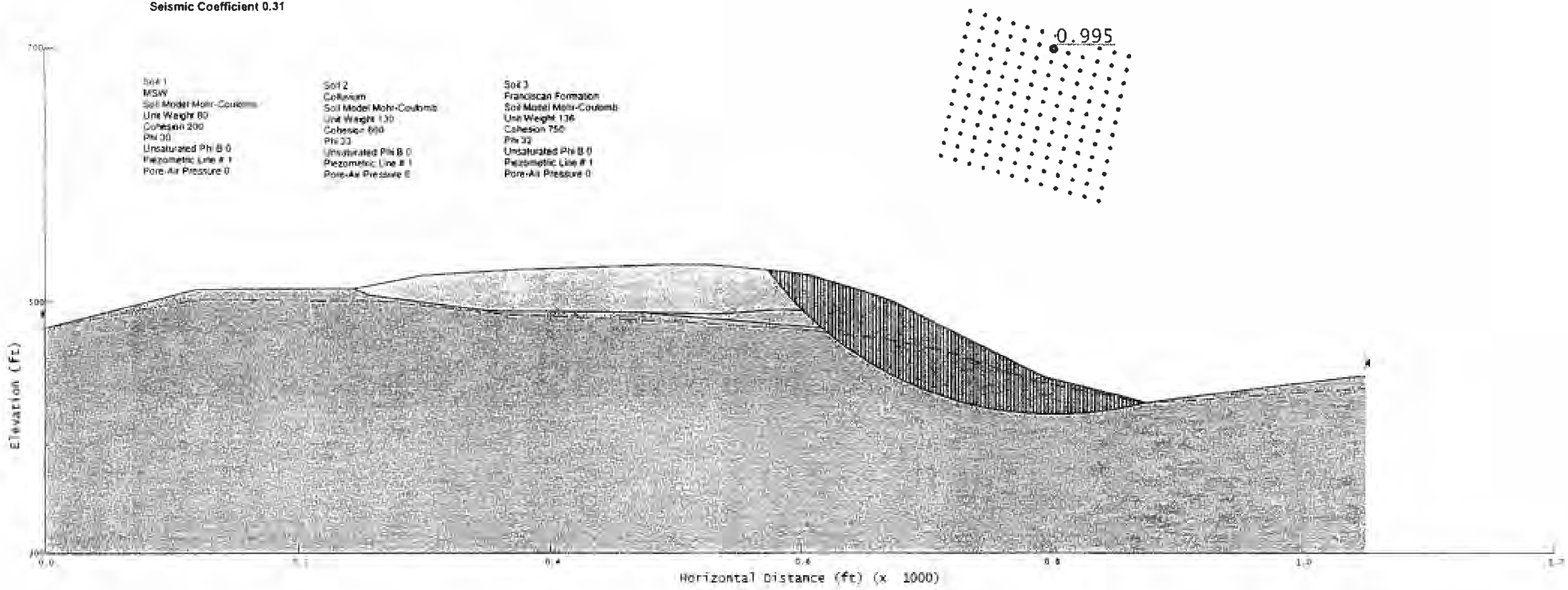
Soil 1
MSW
Soil Model Mohr-Coulomb
Unit Weight 80
Cohesion 200
Phi 30
Unsaturated Phi B 0
Piezometric Line # 0
Pore-Air Pressure 0

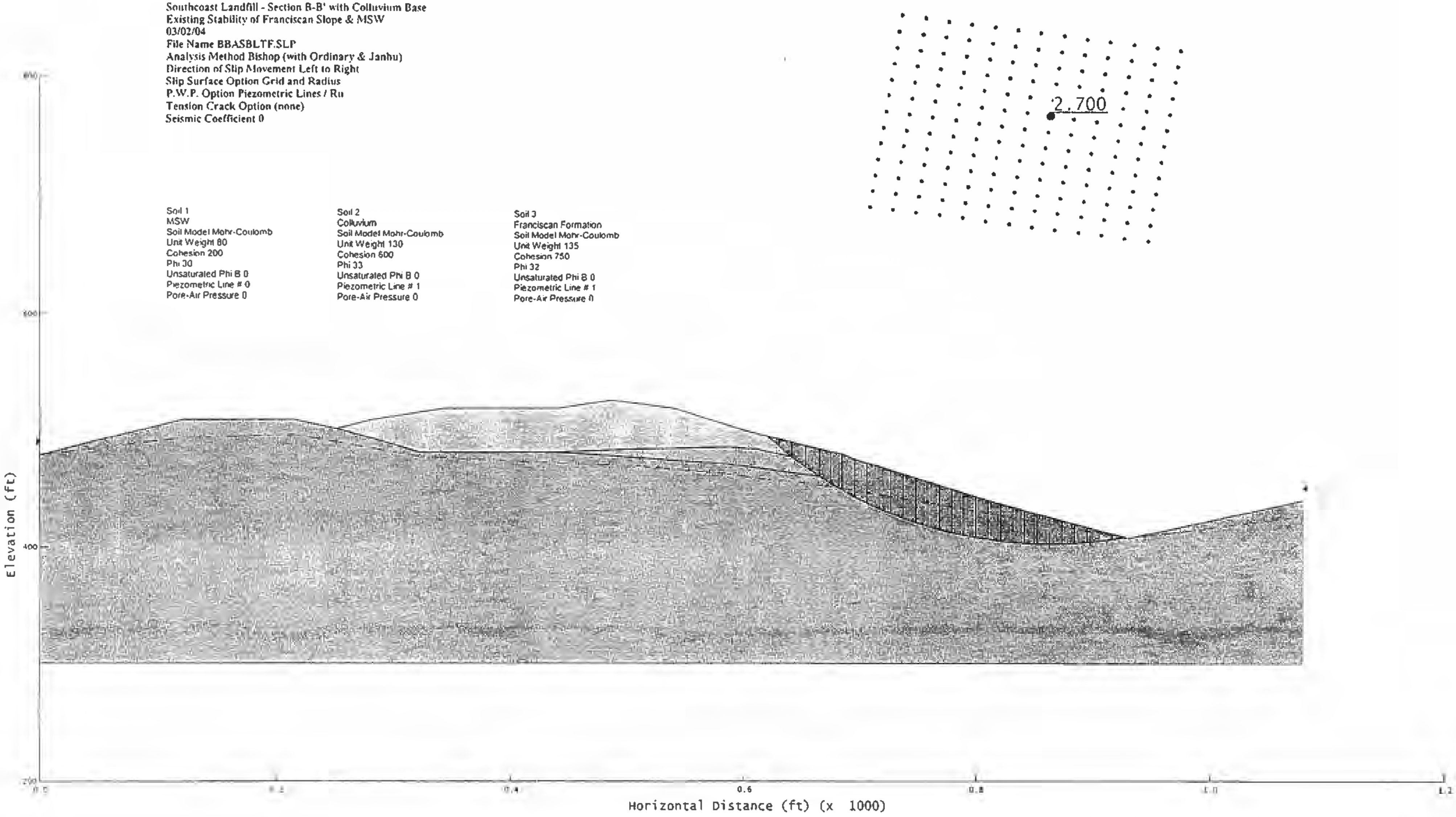
Soil 2
Franciscan Formation
Soil Model Mohr-Coulomb
Unit Weight 135
Cohesion 750
Phi 32
Unsaturated Phi B 0
Piezometric Line # 1
Pore-Air Pressure 0

Southcoast Landfill - Section A-A' with Colluvium
Existing Stability of Franciscan Slope and MSW
03/02/04
File Name AAASBLTF.SLP
Analysis Method Morgenstern-Price
Direction of Slip Movement Left to Right
Slip Surface Option Grid and Radius
P.W.P. Option Piezometric Lines / Ru
Tension Crack Option (none)
Seismic Coefficient 0



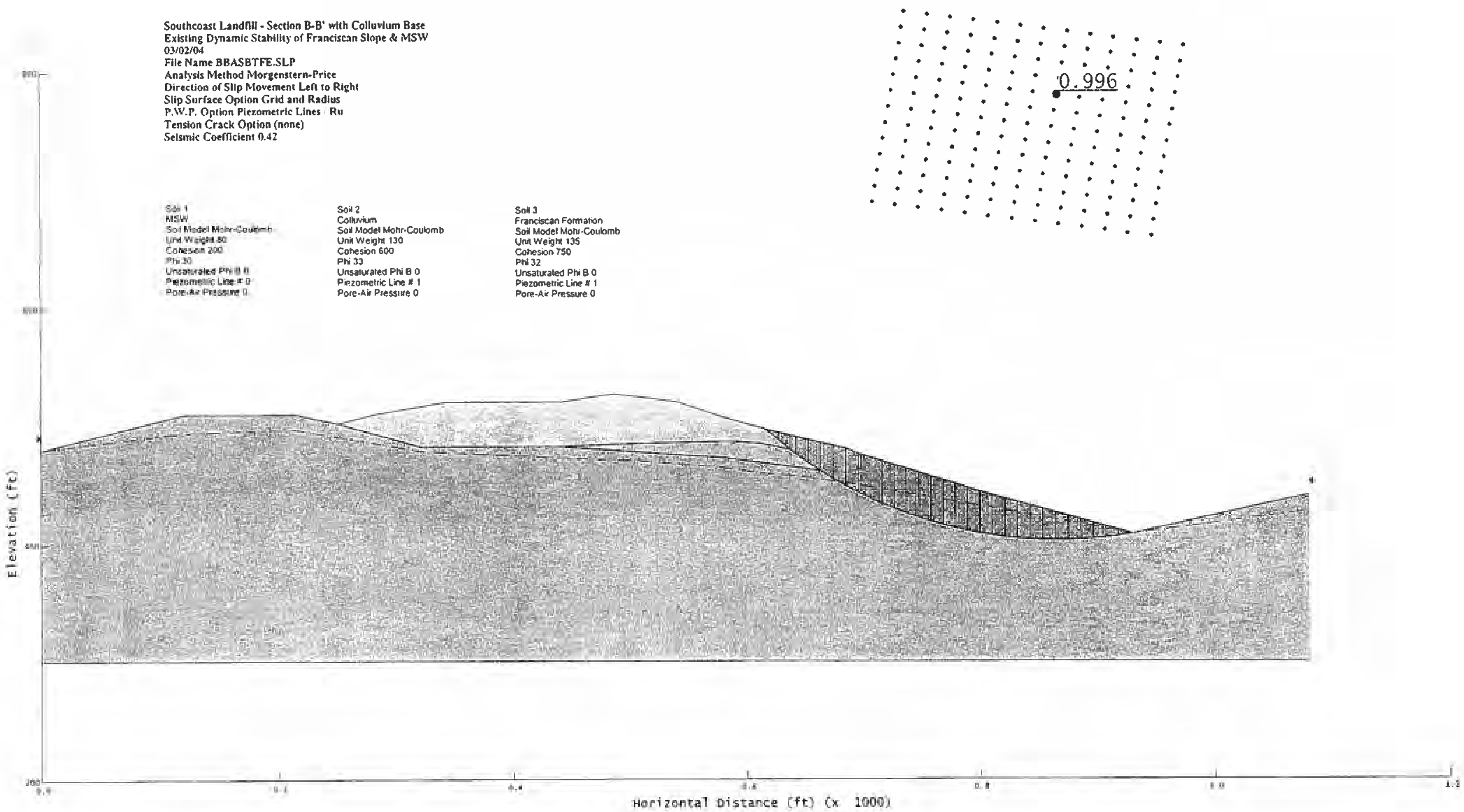
Southcoast Landfill - Section A-A' with Colluvium
Existing Stability of Franciscan Slope and MSW
03/02/04
File Name AASBLTFE.SLP
Analysis Method Morgenstern-Price
Direction of Slip Movement Left to Right
Slip Surface Option Grid and Radius
P.W.P. Option Piezometric Lines / Ru
Tension Crack Option (none)
Seismic Coefficient 0.31





Southcoast Landfill - Section B-B' with Colluvium Base
Existing Stability of Franciscan Slope & MSW
03/02/04
File Name BBASBLTF.SLP
Analysis Method Bishop (with Ordinary & Janhu)
Direction of Slip Movement Left to Right
Slip Surface Option Grid and Radius
P.W.P. Option Piezometric Lines / Ru
Tension Crack Option (none)
Seismic Coefficient 0

Soil 1	Soil 2	Soil 3
MSW	Colluvium	Franciscan Formation
Soil Model Mohr-Coulomb	Soil Model Mohr-Coulomb	Soil Model Mohr-Coulomb
Unit Weight 80	Unit Weight 130	Unit Weight 135
Cohesion 200	Cohesion 600	Cohesion 750
Phi 30	Phi 33	Phi 32
Unsaturated Phi B 0	Unsaturated Phi B 0	Unsaturated Phi B 0
Piezometric Line # 0	Piezometric Line # 1	Piezometric Line # 1
Pore-Air Pressure 0	Pore-Air Pressure 0	Pore-Air Pressure 0



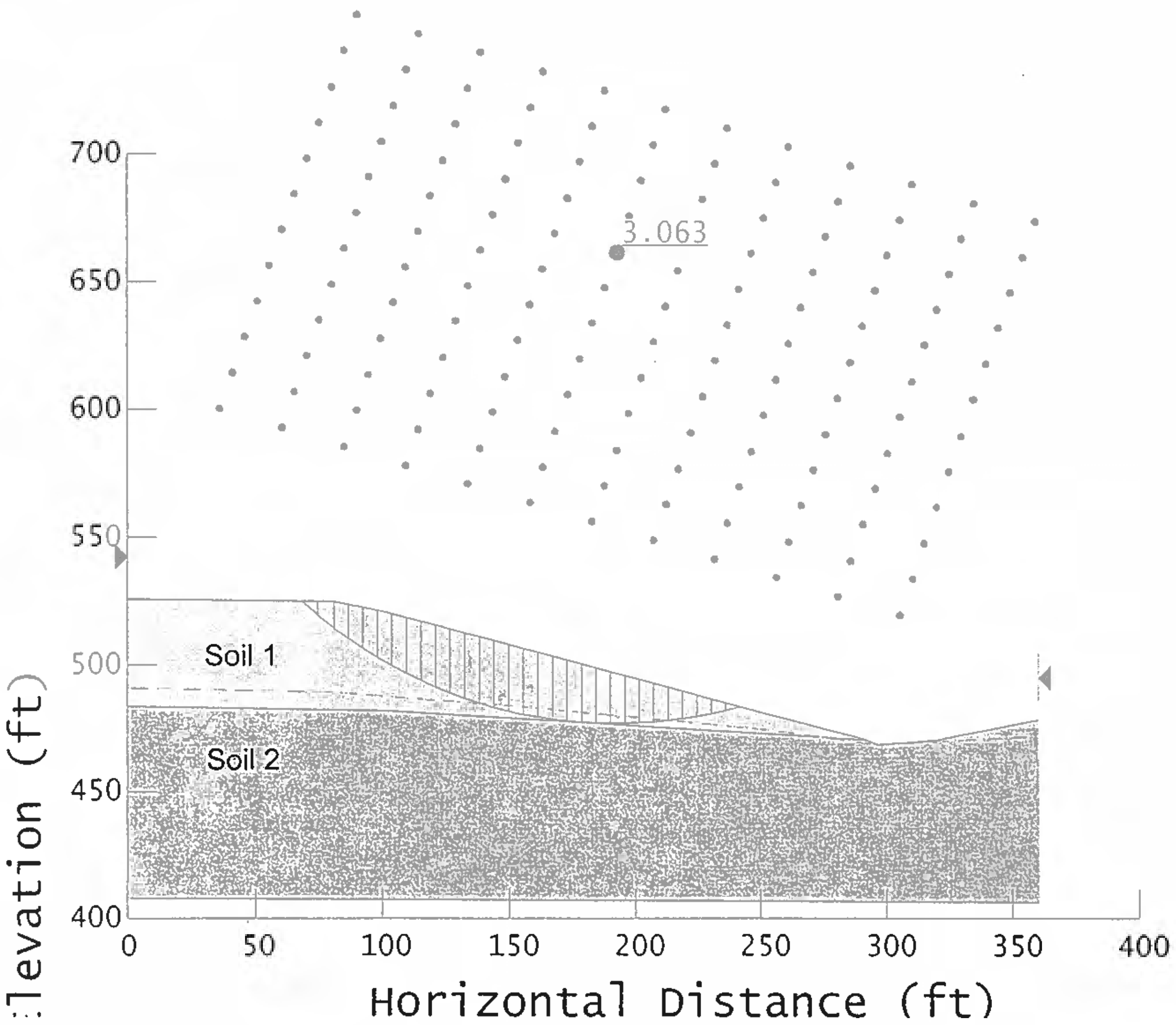
Southcoast Landfill - Section B-B' with Colluvium Base
Existing Dynamic Stability of Franciscan Slope & MSW
03/02/04
File Name BBASBTFE.SLP
Analysis Method Morgenstern-Price
Direction of Slip Movement Left to Right
Slip Surface Option Grid and Radius
P.W.P. Option Piezometric Lines / Ru
Tension Crack Option (none)
Seismic Coefficient 0.42

Soil 1
MSW
Soil Model Mohr-Coulomb
Unit Weight 80
Cohesion 200
Phi 30
Unsaturated Phi B 0
Piezometric Line # 0
Pore-Air Pressure 0

Soil 2
Colluvium
Soil Model Mohr-Coulomb
Unit Weight 130
Cohesion 600
Phi 33
Unsaturated Phi B 0
Piezometric Line # 1
Pore-Air Pressure 0

Soil 3
Franciscan Formation
Soil Model Mohr-Coulomb
Unit Weight 135
Cohesion 750
Phi 32
Unsaturated Phi B 0
Piezometric Line # 1
Pore-Air Pressure 0

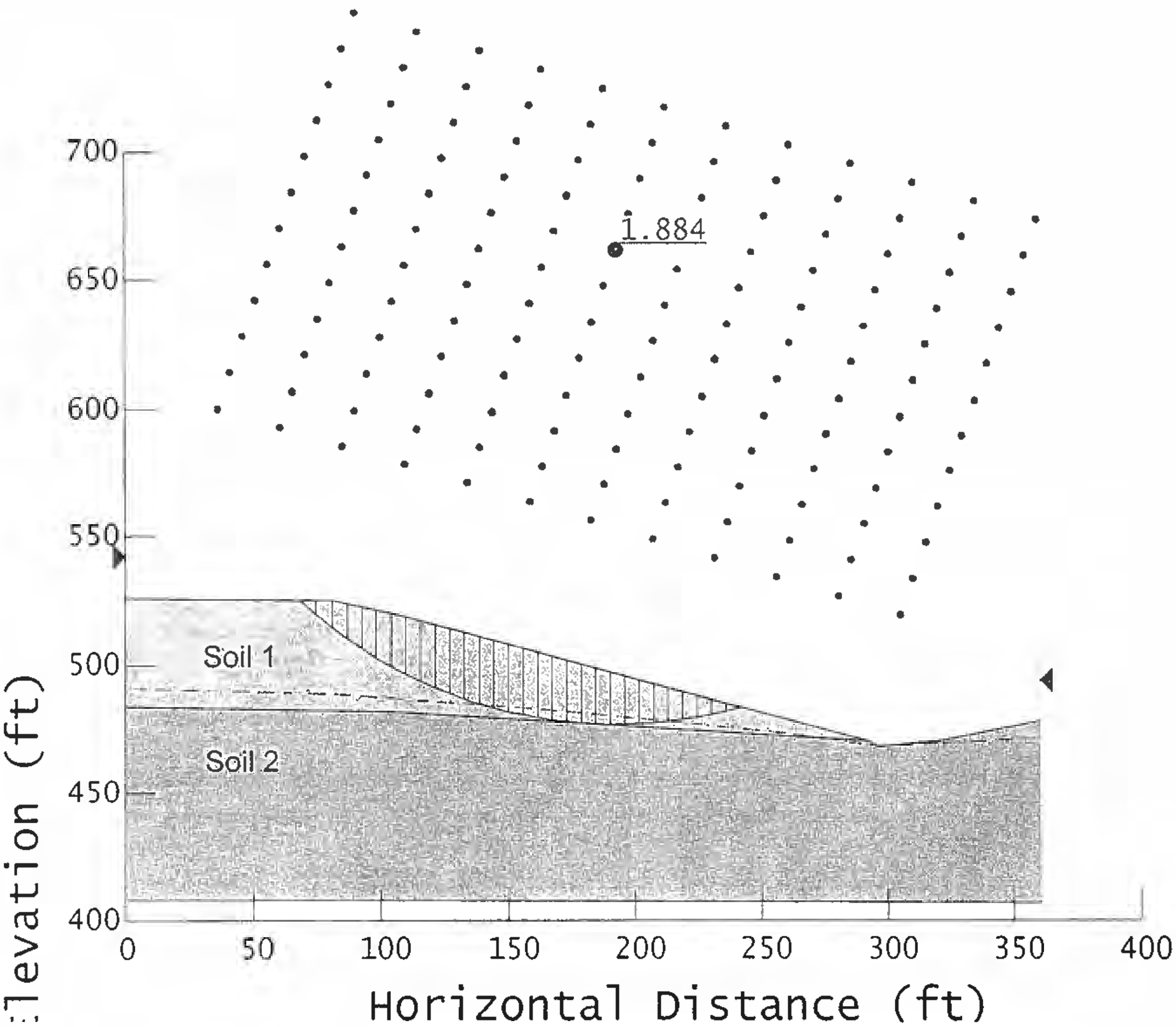
South Coast Landfill Section E-E' Static Stability
Stability of Waste Slope
03/05/15 2
File Name EEMSW.SLP
Analysis Method Morgenstern-Price
Direction of Slip Movement Left to Right
Slip Surface Option Grid and Radius
P.W.P. Option Piezometric Lines / Ru
Tension Crack Option (none)
Seismic Coefficient 0



Soil 1
MSW
Soil Model Mohr-Coulom
Unit Weight 80
Cohesion 200
Phi 30
Unsaturated Phi B 0
Piezometric Line # 0
Pore-Air Pressure 0

Soil 2
Franciscan Fm
Soil Model Mohr-Coulomb
Unit Weight 135
Cohesion 750
Phi 32
Unsaturated Phi B 0
Piezometric Line # 0
Pore-Air Pressure 0

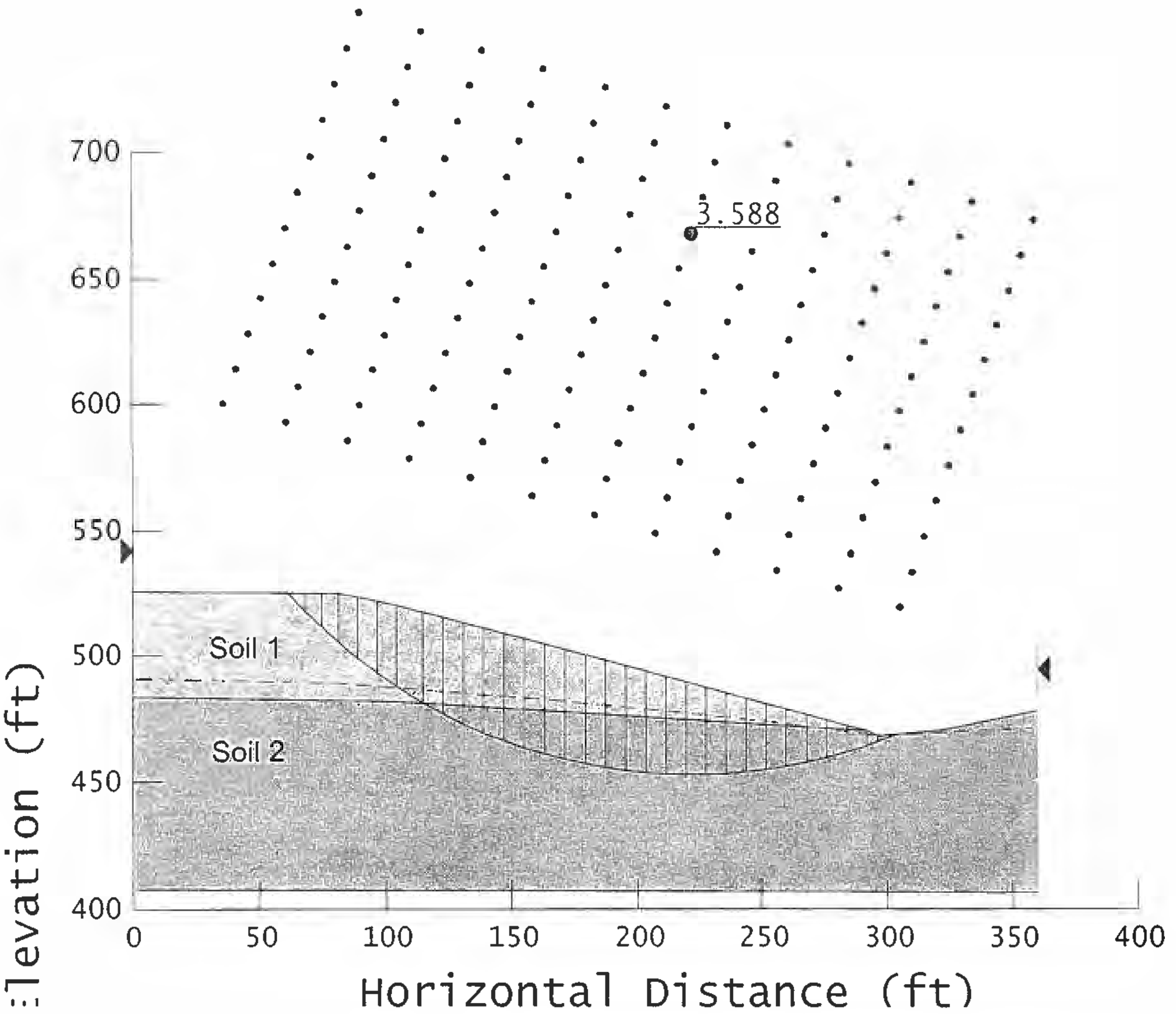
South Coast Landfill Section E-E' Pseudo-Static Stability
Stability of Waste Slope
03/05/15 2
File Name EEMSWEQ.SLP
Analysis Method Morgenstern-Price
Direction of Slip Movement Left to Right
Slip Surface Option Grid and Radius
P.W.P. Option Piezometric Lines / Ru
Tension Crack Option (none)
Seismic Coefficient 0.15



Soil 1
MSW
Soil Model Mohr-Coulomt
Unit Weight 80
Cohesion 200
Phi 30
Unsaturated Phi B 0
Piezometric Line # 0
Pore-Air Pressure 0

Soil 2
Franciscan Fm
Soil Model Mohr-Coulorr
Unit Weight 135
Cohesion 750
Phi 32
Unsaturated Phi B 0
Piezometric Line # 0
Pore-Air Pressure 0

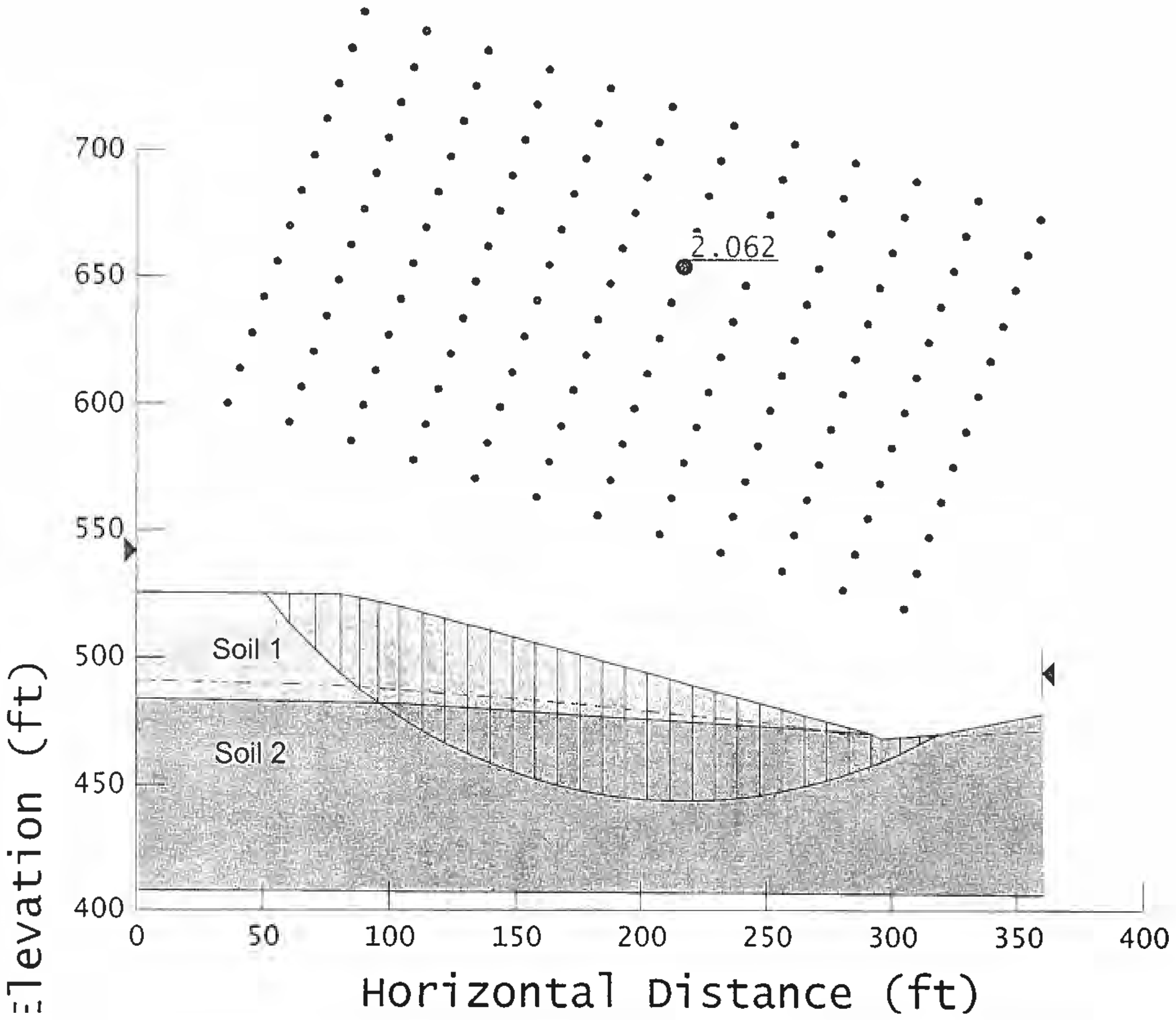
South Coast Landfill Section E-E' Static Stability
Stability of Waste Slope and Franciscan Formation
03/05/15 2
File Name EEW&FR.SLP
Analysis Method Morgenstern-Price
Direction of Slip Movement Left to Right
Slip Surface Option Grid and Radius
P.W.P. Option Piezometric Lines / Ru
Tension Crack Option (none)
Seismic Coefficient 0



Soil 1
MSW
Soil Model Mohr-Coulomb
Unit Weight 80
Cohesion 200
Phi 30
Unsaturated Phi B 0
Piezometric Line # 0
Pore-Air Pressure 0

Soil 2
Franciscan Fm
Soil Model Mohr-Coulomb
Unit Weight 135
Cohesion 750
Phi 32
Unsaturated Phi B 0
Piezometric Line # 0
Pore-Air Pressure 0

South Coast Landfill Section E-E' Pseudo-Static S ty
Stability of Waste Slope and Franciscan Formation
03/05/15 2
File Name EEW&FRE.SLP
Analysis Method Morgenstern-Price
Direction of Slip Movement Left to Right
Slip Surface Option Grid and Radius
P.W.P. Option Piezometric Lines / Ru
Tension Crack Option (none)
Seismic Coefficient 0.15



Soil 1
MSW
Soil Model Mohr-Coulomb
Unit Weight 80
Cohesion 200
Phi 30
Unsaturated Phi B 0
Piezometric Line # 0
Pore-Air Pressure 0

Soil 2
Franciscan Fm
Soil Model Mohr-Coulomb
Unit Weight 135
Cohesion 750
Phi 32
Unsaturated Phi B 0
Piezometric Line # 0
Pore-Air Pressure 0

FINAL COVER STABILITY CALCULATIONS

South Coast Landfill Slope Stability Calculation Summary Sheet

	Normal High GW			10' Saturated MSW			15' of Colluvium at Base of Landfill**		
Section A-A'	FS	ky	Displacement* or Factor of Safety$^{(1)}$	FS	ky	Displacement* or Factor of Safety$^{(1)}$	FS	ky	Displacement* or Factor of Safety$^{(1)}$
Failure in MSW only	3.45	0.58g	<1/2"	3.5	0.53g	<1/2"	-	-	-
Failure in Franciscan & MSW	1.89	0.30g	7"	1.84	0.26g	10"	1.92	0.31g	7-1/2"
Section B-B'									
Failure through MSW only	3.21	0.53g	<1/2"	3.12	0.49g	<1/2"	-	-	-
Failure in Franciscan & MSW	2.49	0.36g	6"	2.37	0.32g	6"	2.7	0.42g	2-1/2"
Section C-C'									
Failure thru Geogrid Buttress	2.45	-	1.69						
Global failure thru buttress and underlying formation	2.64	-	1.88						
Section D-D'									
Failure thru Geogrid Buttress	2.05	-	1.53						
Global failure thru buttress and underlying formation	3.27	-	2.51						
Section E-E'									
Failure thru Waste	3.06	-	1.88						
Failure thru Waste and underlying formation	3.59	-	2.06						

Notes:
* Calculated in accordance with Bray and Rathje, 1998; and Makdisi and Seed, 1978
** Groundwater levels per Figure 3 - Groundwater Equipotential Plan
FS = factor of safety
Fy = Yield acceleration
MSW = Municipal Soild Waste
(1) Pseudo-static factor of safety shown with a horizontal site accararation of 0.15g.

Calculated Factors of Safety
Final Cover Components, 30'high slope @3.1:1
Southcoast Landfill, Mendocino County, California

Shear Strength Parameters				Slope Parameters (1)					Toe Parameters			Computed Forces / Unit Width				Geosynthetics			FACTOR OF SAFETY
Soil							Slope Cover		Soil Thickness			Driving	Resisting						
														Geosynthetic					
Sat. Unit Wt.	Int. Friction Angle	Passive Crit. Failure Angle	Interface Friction Angle	Slope Ratio (H:1V)	Slope Inclin.	Soil Thickness	Vert. Ht.	Slope Length	Total	Ave.	Wedge Fail. Length	Soil Load	Toe Soil Wedge	Interface Friction	Tensile Demand	Ultimate Tensile Strength (2)	Assigned Safety Factor	Allowable Tensile Strength(3)	
γ_s	ϕ	$\phi_{CR} = 45 - \delta/2$	δ	Z	α	d_s	H_v	$L_s = H_v / SIN\,\alpha$	d_T	$d_e = (2/3)\,d_T$	$L_w = d_T / SIN\phi_{CR}$	$D_s = \gamma_s d_s L_S SIN\alpha$	$R_w = \gamma_s d_e L_w COS\phi_{CR} Tan\phi$	$R_F = \gamma_s d_s L_s COS\alpha Tan\delta$	For FS 1.5; $R_T = 1.5^* D_s - R_w - R_f$	T_u	Sfa	$T_A = T_u / SF_A$	$FS = (R_w + R_f + T_A) / D_s$
(pcf)	(deg)	(deg)	(deg)	(ft)	(deg)	(ft)	(ft)	(ft)	(ft)	(ft)	(ft)	(#/ft)	(#/ft)	(#/ft)	(#/ft)	(#/ft)		(#/ft)	
30'high slope @3.1:1																			
Veg. Cover on NW Geotextile																			
130	35.9	27.05	30.6	3.1	17.88	2	30	97.8	2	1.33	4.40	7800	491.4	14298.9	-3090.3	0	3	0	1.90
NW Geotextile on Pea Gravel																			
130	30.6	29.7	35.9	3.1	17.88	2	30	97.8	2	1.33	4.04	7800	359.4	17501.2	-6160.6	0	3	0	2.29
Pea Gravel on Textured LLDPE																			
130	35.9	27.05	35.9	3.1	17.88	3	30	97.8	3	2.00	6.60	11700	1105.7	26251.7	-9807.4	0	3	0	2.34
Textured LLDPE on Foundation																			
130	35.9	27.05	30.6	3.1	17.88	3	30	97.8	3	2.00	6.60	11700	1105.7	21448.4	-5004.0	0	2	0	1.93

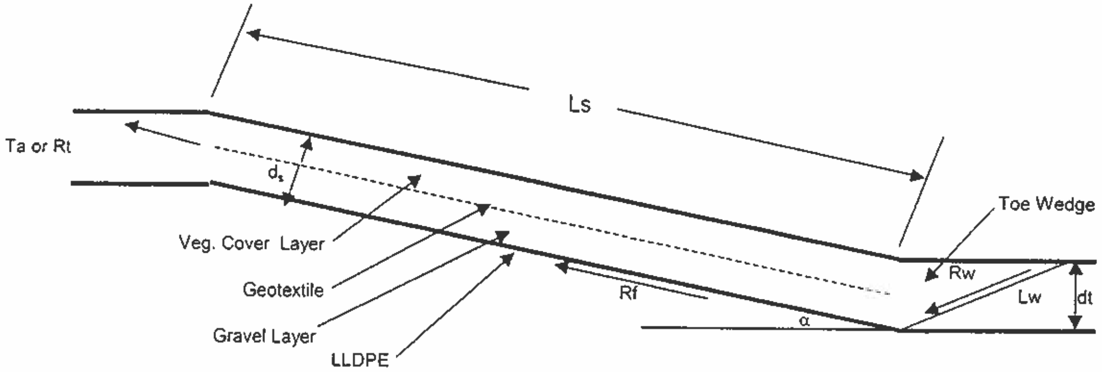
(1)- Assumed configuration, 3.1:1 slope inclination with 30 foot maximum height slope.

30'high slope @3.1:1

<u>Cover Profile</u>

24" of Vegetative Cover
Nonwowen (separation) geotextile
12" of pea gravel
Textured LLDPE
Foundation Layer

<u>Data from 2" displacement tests</u>	<u>c</u>	<u>ϕ</u>
Tested strength of soil on tex LLDPE	100	30.6
Tested strength of pea gavel on tex LLDPE	64	35.9



SEISMIC INDUCED PERMANENT DISPLACEMENT EVALUATION-FINAL COVER
SOUTHCOAST LANDFILL, MENDOCINO COUNTY, CALIFORNIA
30 foot high slope at 3.1:1

PROCEDURE/REFERENCES USED: Bray and Rathje, 1998; Bray, Rathje, Augello, and Merry, 1998; and EMCON, 1998

INPUT DATA:

SITE CONDITION: Rock with fault gouge and weathered rock near-surface.
EFFECTIVE MSW HEIGHT (H): 35 feet
AVERAGE SHEAR WAVE VELOCITY FOR MSW: Vs = 420 ft/sec
(Bray and Rathje, 1998, Figure 2 and EMCON, 1998 Pg. 3-2)
EARTHQUAKE MAGNITUDE FOR MPE: M=8.0 @2km (San Andreas)
MAXIMUM HORIZONTAL SITE ACCELERATION, MHA_{rock}: 0.9g (EMCON, 1998)
SIGNIFICANT DURATION – $D_{5\text{-}95}$ = 33 seconds (Bray, et. al, 1998, Figure 2c and EMCON, 1998, pg. 3-1).
YIELD ACCELERATION, K_y= 0.225

CALCULATIONS:

MEAN PERIOD OF SHAKING FOR MPE: T_m= 0.52 seconds (Bray. et. al., 1998, Figure 2b)

PREDOMINANT PERIOD OF MSW: $T_s = 4H/V_s = T_S$ (@ H=35' = 0.33 sec

RATIO T_s/T_m = 0.63

RATIO MHEA/MHA(NRF) = 1.24 (Bray and Rathje, 1998, Fig. 8b, rock site), top of MSW
NRF for MHA @0.9g = 0.75 (Bray and Rathje, 1998, Figure 6b)

Where: MHEA = Maximum Horizontal Equivalent Acceleration at top of MSW
MHA = Maximum Horizontal Site Acceleration in rock

Thus, MHEA/(MHA_{rock} x NRF) = 1.24, So, MHEA = (1.24)(0.75)(0.9) = K_{max}= 0.84

K_y/K_{max} = 0.225/0.84 = 0.27

From Bray and Rathje, 1998, Figure 13, calculate displacement;

For K_y/K_{max} = 0.27; U = 20 cm = **7.9 inches**

Calculated Yield Accelerations and Seismic Displacements
Final Cover Components
Southcoast Landfill, Mendocino County, California

	Interface Friction Angle δ	Slope Inclination α	Yield Acceleration K_y*	Calculated Displacement*** (inches)
Veg. Cover on NW Geotextile	30.6	17.88	0.225	7.9
NW Geotextile on Pea Gravel	35.9	17.88	0.325	3.5
Pea Gravel on Textured LLDPE	35.9	17.88	0.325	3.5
Textured LLDPE on Foundation	30.6	17.88	0.225	7.9

Notes:

*Solve for T_u and K_y when FS=1.00

$$^{**}FS= \frac{((\gamma_s \delta_s L_s \cos \alpha)-(K_y \gamma_s \delta_s L_\alpha \sin \alpha))(\tan \delta) + T_u}{(\gamma_s \delta_s L_s \sin\alpha + K_y \gamma_s \delta_s L_s \cos \alpha)}$$

**Equation adapted from Kramer, 1996, Geotechnical Earthquake Engineering, pg 434
Variables defined on previous page

*** Bray and Rathje, 1998

Case:
30'high slope @3.1:1

Veg. Cover on NW Geotextile
Ls 97.77
Ky= 0.225 g
FS**= 1.00

NW Geotextile on Pea Gravel
Ls 97.77
Ky= 0.325 g
FS**= 1.00

Pea Gravel on Textured LLDPE
Ls 97.77
Ky= 0.325 g
FS**= 1.00

Textured LLDPE on Foundation
Ls 97.77
Ky= 0.225 g
FS**= 1.00

Calculated Factors of Safety
Final Cover Components, 35'high slope @3.7:1
Southcoast Landfill, Mendocino County, California

Shear Strength Parameters				Slope Parameters (1)					Toe Parameters			Computed Forces / Unit Width				Geosynthetics			FACTOR OF SAFETY
Soil			Interface				Slope Cover		Soil Thickness			Driving	Resisting						
														Geosynthetic					
Sat. Unit Wt.	Int. Friction Angle	Passive Crit. Failure Angle	Friction Angle	Slope Ratio (H:1V)	Slope Inclin.	Soil Thickness	Vert. Ht.	Slope Length	Total	Ave.	Wedge Fail. Length	Soil Load	Toe Soil Wedge	Interface Friction	Tensile Demand	Ultimate Tensile Strength (2)	Assigned Safety Factor	Allowable Tensile Strength(3)	
γ_s	ϕ	$\phi_{CR = 45 - \phi/2}$	δ	Z	α	d_s	H_v	$L_s = H_v / SIN\ \alpha$	d_T	$d_a = (2/3)\ d_T$	$L_w = d_T / SIN\phi_{CR}$	$D_s = \gamma_s d_s L_s SIN\alpha$	$R_w = \gamma_s d_a L_w COS\phi_{CR} Tan\phi$	$R_F = \gamma_s d_s L_s COS\alpha Tan\delta$	For FS 1.5; $R_T = 1.5{}^*D_s - R_w - R_f$	T_u	Sfa	$T_A = T_u / SF_A$	$FS = (R_w + R_f + T_A) / D_s$
(pcf)	(deg)	(deg)	(deg)	(ft)	(deg)	(ft)	(ft)	(ft)	(ft)	(ft)	(ft)	(#/ft)	(#/ft)	(#/ft)	(#/ft)	(#/ft)		(#/ft)	
35' high @3.7:1																			
Veg. Cover on NW Geotextile																			
130	35.9	27.05	30.6	3.7	15.12	2	35	134.2	2	1.33	4.40	9100	491.4	19910.6	-6752.0	0	3	0	2.24
NW Geotextile on Pea Gravel																			
130	30.6	29.7	35.9	3.7	15.12	2	35	134.2	2	1.33	4.04	9100	359.4	24369.6	-11079.1	0	3	0	2.72
Pea Gravel on Textured LLDPE																			
130	35.9	27.05	35.9	3.7	15.12	3	35	134.2	3	2.00	6.60	13650	1105.7	36554.5	-17185.1	0	3	0	2.76
Textured LLDPE on Foundation																			
130	35.9	27.05	30.6	3.7	15.12	3	35	134.2	3	2.00	6.60	13650	1105.7	29866.0	-10496.6	0	2	0	2.27

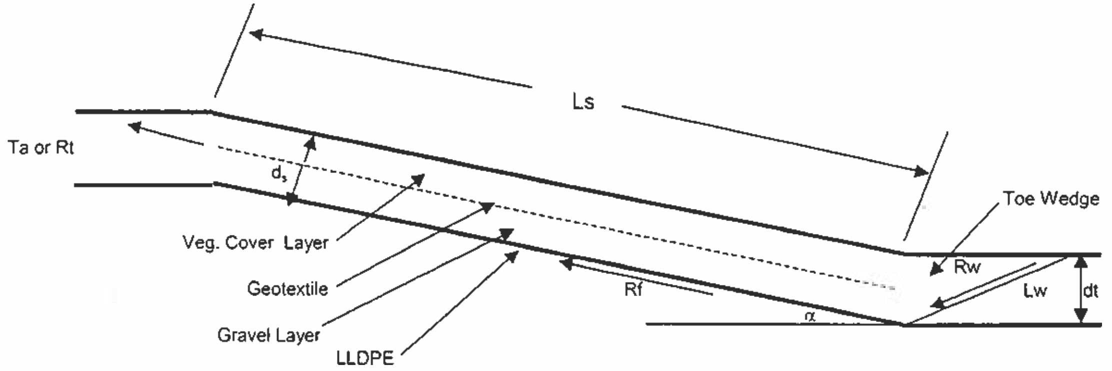
(1)- Assumed configuration, 3.7:1 slope inclination with 35 foot maximum height slope.

35' high @3.7:1

Cover Profile

24" of Vegetative Cover
Nonwowen (separation) geotextile
12" of pea gravel
Textured LLDPE
Foundation Layer

Data from 2" displacement tests	c	ϕ
Tested strength of soil on tex LLDPE	100	30.6
Tested strength of pea gavel on tex LLDPE	64	35.9



SEISMIC INDUCED PERMANENT DISPLACEMENT EVALUATION-FINAL COVER
SOUTHCOAST LANDFILL, MENDOCINO COUNTY, CALIFORNIA
35 foot high slope at 3.7:1

PROCEDURE/REFERENCES USED: Bray and Rathje, 1998; Bray, Rathje, Augello, and Merry, 1998; and EMCON, 1998

INPUT DATA:

SITE CONDITION: Rock with fault gouge and weathered rock near-surface.

EFFECTIVE MSW HEIGHT (H): 35 feet

AVERAGE SHEAR WAVE VELOCITY FOR MSW: $V_s = 420$ ft/sec (Bray and Rathje, 1998, Figure 2 and EMCON, 1998 Pg. 3-2)

EARTHQUAKE MAGNITUDE FOR MPE: M=8.0 @2km (San Andreas)

MAXIMUM HORIZONTAL SITE ACCELERATION, MHA_{rock}: 0.9g (EMCON, 1998)

SIGNIFICANT DURATION – $D_{5\text{-}95} = 33$ seconds (Bray, et. al, 1998, Figure 2c and EMCON, 1998, pg. 3-1).

YIELD ACCELERATION, $K_y = 0.275$

CALCULATIONS:

MEAN PERIOD OF SHAKING FOR MPE: $T_m = 0.52$ seconds (Bray. et. al., 1998, Figure 2b)

PREDOMINANT PERIOD OF MSW: $T_s = 4H/V_s = T_S$ (@ H=35' = 0.33 sec

RATIO $T_s/T_m = 0.63$

RATIO MHEA/MHA(NRF) = 1.24 (Bray and Rathje, 1998, Fig. 8b, rock site), top of MSW
NRF for MHA @0.9g = 0.75 (Bray and Rathje, 1998, Figure 6b)

Where: MHEA = Maximum Horizontal Equivalent Acceleration at top of MSW
MHA = Maximum Horizontal Site Acceleration in rock

Thus, $MHEA/(MHA_{rock} \times NRF) = 1.24$, So, MHEA = $(1.24)(0.75)(0.9) = K_{max} = 0.84$

$K_y/K_{max} = 0.275/0.84 = 0.33$

From Bray and Rathje, 1998, Figure 13, calculate displacement;

For $K_y/K_{max} = 0.33$; U = 14 cm = **5.5 inches**

Calculated Yield Accelerations and Seismic Displacment
Final Cover Components
Southcoast Landfill, Mendocino County, California

	Interface Friction Angle δ	Slope Inclination α	Yield Acceleration K_y*	Calculated Displacement*** (inches)
Veg. Cover on NW Geotextile	30.6	15.12	0.275	5.5
NW Geotextile on Pea Gravel	35.9	15.12	0.38	2.4
Pea Gravel on Textured LLDPE	35.9	15.12	0.38	2.4
Textured LLDPE on Foundation	30.6	15.12	0.275	5.5

Case:
35' high @3.7:1

Veg. Cover on NW Geotextile	
Ls 134.21	
Ky= 0.275	g
FS**= 1.00	
NW Geotextile on Pea Gravel	
Ls 134.21	
Ky= 0.38	g
FS**= 1.00	
Pea Gravel on Textured LLDPE	
Ls 134.21	
Ky= 0.38	g
FS**= 1.00	
Textured LLDPE on Foundation	
Ls 134.21	
Ky= 0.275	g
FS**= 1.00	

Notes:

*Solve for T_u and K_y when FS=1.00

$$**FS= \frac{((\gamma_s \delta_s L_s \cos\alpha)-(K_y \gamma_s \delta_s L_s \sin\alpha))(\tan\delta)+T_u}{(\gamma_s \delta_s L_s \sin\alpha + K_y \gamma_s \delta_s L_s \cos\alpha)}$$

**Equation adapted from Kramer, 1996, Geotechnical Earthquake Engineering, pg 434
Variables defined on previous page

*** Bray and Rathje, 1998

Calculated Factors of Safety
Final Cover Components, 45'high slope @3.3:1
Southcoast Landfill, Mendocino County, California

Shear Strength Parameters				Slope Parameters (1)					Toe Parameters			Computed Forces / Unit Width				Geosynthetics			FACTOR OF SAFETY
Soil			Interface Friction Angle	Slope Ratio (H:1V)	Slope Inclin.	Soil Thickness	Slope Cover		Soil Thickness		Wedge Fail. Length	Driving	Resisting			Ultimate Tensile Strength (2)	Assigned Safety Factor	Allowable Tensile Strength(3)	
Sat. Unit Wt.	Int. Friction Angle	Passive Crit. Failure Angle					Vert. Ht.	Slope Length	Total	Ave.		Soil Load	Toe Soil Wedge	Geosynthetic					
														Interface Friction	Tensile Demand				
γ_s	ϕ	$\phi_{CR} = 45 - \phi/2$	δ	Z	α	d_s	H_v	$L_s = H_v / SIN\ \alpha$	d_T	$d_a = (2/3)\ d_T$	$L_w = d_T / SIN\phi_{CR}$	$D_s = \gamma_s d_s L_S SIN\alpha$	$R_w = \gamma_s d_a L_w COS\phi_{CR} Tan\phi$	$R_F = \gamma_s d_s L_s COS\alpha Tan\delta$	For FS 1.5; $R_T = 1.5{*}D_s - R_w - R_f$	T_u	Sfa	$T_A = T_u / SF_A$	$FS = (R_w + R_f + T_A) / D_s$
(pcf)	(deg)	(deg)	(deg)	(ft)	(deg)	(ft)	(ft)	(ft)	(ft)	(ft)	(ft)	(#/ft)	(#/ft)	(#/ft)	(#/ft)	(#/ft		(#/ft)	
45'high@3.3:1																			
Veg. Cover on NW Geotextile																			
130	35.9	27.05	30.6	3.3	16.86	2	45	155.2	2	1.33	4.40	11700	491.4	22832.0	-5773.4	0	3	0	1.99
NW Geotextile on Pea Gravel																			
130	30.6	29.7	35.9	3.3	16.86	2	45	155.2	2	1.33	4.04	11700	359.4	27945.3	-10754.7	0	3	0	2.42
Pea Gravel on Textured LLDPE																			
130	35.9	27.05	35.9	3.3	16.86	3	45	155.2	3	2.00	6.60	17550	1105.7	41917.9	-16698.6	0	3	0	2.45
Textured LLDPE on Foundation																			
130	35.9	27.05	30.6	3.3	16.86	3	45	155.2	3	2.00	6.60	17550	1105.7	34248.0	-9028.7	0	2	0	2.01

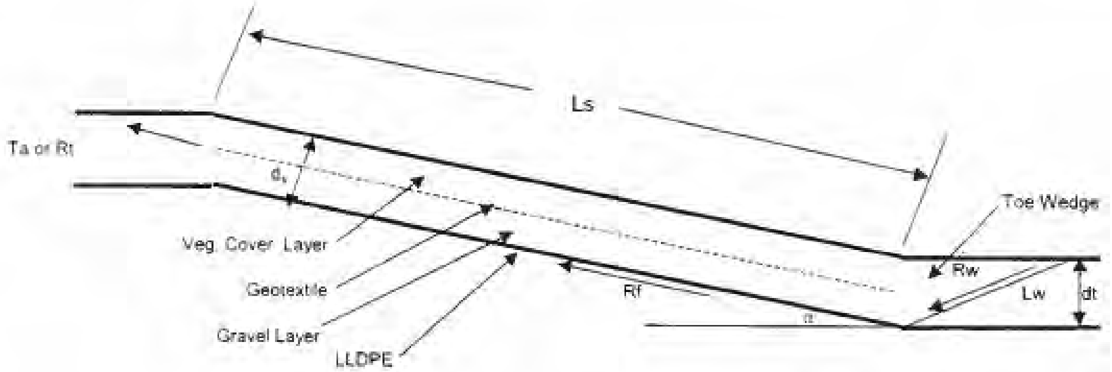
(1)- Assumed configuration, 3.3:1 slope inclination with 45 foot maximum height slope.

45'high@3.3:1

Cover Profile

24" of Vegetative Cover
Nonwowen (separation) geotextile
12" of pea gravel
Textured LLDPE
Foundation Layer

Data from 2" displacement tests	c	φ
Tested strength of soil on tex LLDPE	100	30.6
Tested strength of pea gavel on tex LLDPE	64	35.9



SEISMIC INDUCED PERMANENT DISPLACEMENT EVALUATION-FINAL COVER
SOUTHCOAST LANDFILL, MENDOCINO COUNTY, CALIFORNIA
45 foot high slope at 3.3:1

PROCEDURE/REFERENCES USED: Bray and Rathje, 1998; Bray, Rathje, Augello, and Merry, 1998; and EMCON, 1998

INPUT DATA:

SITE CONDITION: Rock with fault gouge and weathered rock near-surface.

EFFECTIVE MSW HEIGHT (H): 35 feet

AVERAGE SHEAR WAVE VELOCITY FOR MSW: Vs = 420 ft/sec

(Bray and Rathje, 1998, Figure 2 and EMCON, 1998 Pg. 3-2)

EARTHQUAKE MAGNITUDE FOR MPE: M=8.0 @2km (San Andreas)

MAXIMUM HORIZONTAL SITE ACCELERATION, MHA_{rock}: 0.9g (EMCON, 1998)

SIGNIFICANT DURATION – $D_{5\text{-}95}$ = 33 seconds (Bray, et. al, 1998, Figure 2c and EMCON, 1998, pg. 3-1).

YIELD ACCELERATION, K_y= 0.245

CALCULATIONS:

MEAN PERIOD OF SHAKING FOR MPE: T_m= 0.52 seconds (Bray. et. al., 1998, Figure 2b)

PREDOMINANT PERIOD OF MSW: $T_s = 4H/V_s = T_S$ (@ H=35' = 0.33 sec

RATIO T_s/T_m = 0.63

RATIO MHEA/MHA(NRF) = 1.24 (Bray and Rathje, 1998, Fig. 8b, rock site), top of MSW

NRF for MHA @0.9g = 0.75 (Bray and Rathje, 1998, Figure 6b)

Where: MHEA = Maximum Horizontal Equivalent Acceleration at top of MSW
MHA = Maximum Horizontal Site Acceleration in rock

Thus, MHEA/(MHA_{rock} x NRF) = 1.24, So, MHEA = (1.24)(0.75)(0.9) = K_{max}= 0.84

K_y/K_{max} = 0.245/0.84 = 0.29

From Bray and Rathje, 1998, Figure 13, calculate displacement;

For K_y/K_{max} = 0.29; U = 18 cm = **7 inches**

Calculated Yield Accelerations and Seismic Displacements
Final Cover Components
Southcoast Landfill, Mendocino County, California

	Interface Friction Angle δ	Slope Inclination α	Yield Acceleration K_y*	Calculated Displacement*** (inches)
Veg. Cover on NW Geotextile	30.6	16.86	0.245	7
NW Geotextile on Pea Gravel	35.9	16.86	0.345	3
Pea Gravel on Textured LLDPE	35.9	16.86	0.345	3
Textured LLDPE on Foundation	30.6	16.86	0.245	7

Case:
45'high@3.3:1

Veg. Cover on NW Geotextile		
	Ls 155.24	
	Ky= 0.245	g
	FS**= 1.00	
NW Geotextile on Pea Gravel		
	Ls 155.24	
	Ky= 0.345	g
	FS**= 1.00	
Pea Gravel on Textured LLDPE		
	Ls 155.24	
	Ky= 0.345	g
	FS**= 1.00	
Textured LLDPE on Foundation		
	Ls 155.24	
	Ky= 0.245	g
	FS**= 1.00	

Notes:

*Solve for T_u and K_y when FS=1.00

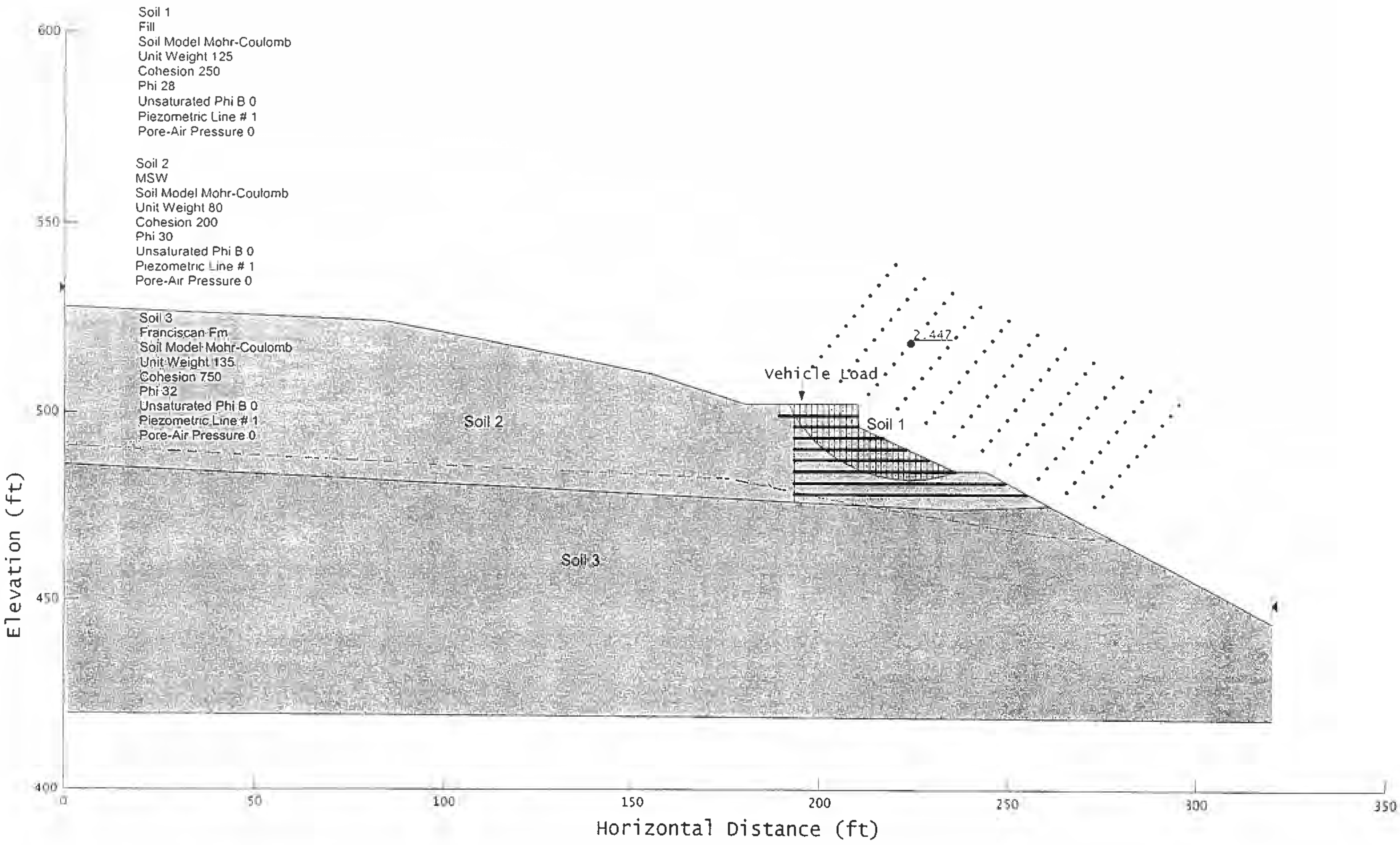
$$**FS= \frac{((\gamma_s\delta_s L_s \cos\alpha)-(K_y \gamma_s\delta_s L_s \sin\alpha))(\tan\delta)+T_u}{(\gamma_s\delta_s L_s \sin\alpha+K_y \gamma_s\delta_s L_s \cos\alpha)}$$

**Equation adapted from Kramer, 1996, Geotechnical Earthquake Engineering, pg 434
Variables defined on previous page

*** Bray and Rathje, 1998

ROADWAY STABILITY CALCULATIONS

outhcoast Landfill-Geogrid Road Buttress Static Stabilitry
Section C-C' Geogrid Road Buttress 6' High
03/05/01 5
File Name SECCC.SLP
Analysis Method Morgenstern-Price
Direction of Slip Movement Left to Right
Slip Surface Option Grid and Radius
P.W.P. Option Piezometric Lines / Ru
Tension Crack Option (none)
Seismic Coefficient 0

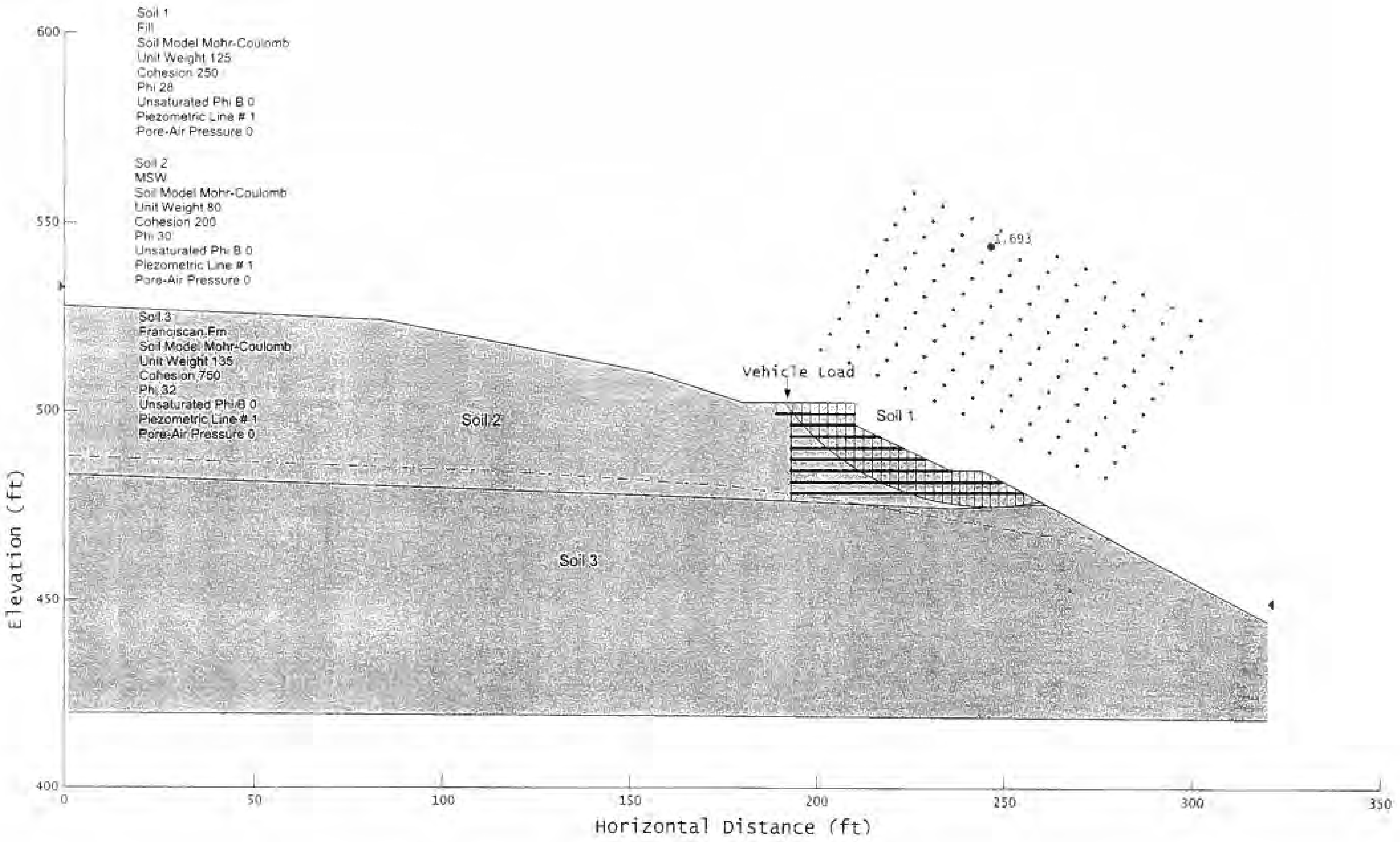


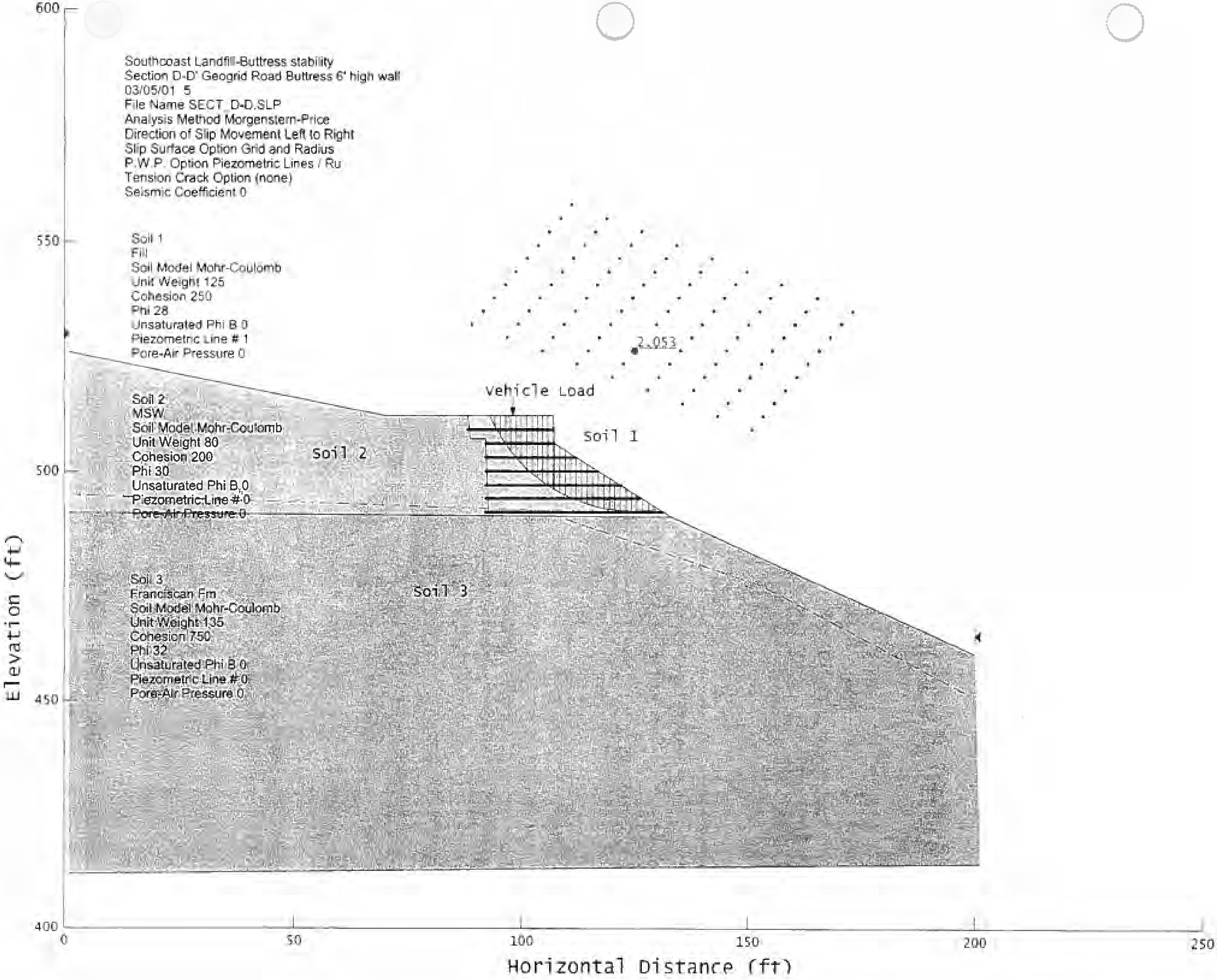
Soil 1
Fill
Soil Model Mohr-Coulomb
Unit Weight 125
Cohesion 250
Phi 28
Unsaturated Phi B 0
Piezometric Line # 1
Pore-Air Pressure 0

Soil 2
MSW
Soil Model Mohr-Coulomb
Unit Weight 80
Cohesion 200
Phi 30
Unsaturated Phi B 0
Piezometric Line # 1
Pore-Air Pressure 0

Soil 3
Franciscan Fm
Soil Model Mohr-Coulomb
Unit Weight 135
Cohesion 750
Phi 32
Unsaturated Phi B 0
Piezometric Line # 1
Pore-Air Pressure 0

uthcoast Landfill-Geogrid Road Buttress Pseudo-Static Stabilitry
ection C-C' Geogrid Road Buttress 6' High
03/05/01 5
File Name SECCCEQ.SLP
Analysis Method Morgenstern-Price
Direction of Slip Movement Left to Right
Slip Surface Option Grid and Radius
P.W.P. Option Piezometric Lines / Ru
Tension Crack Option (none)
Seismic Coefficient 0.15




Southcoast Landfill-Buttress stability
Section D-D' Geogrid Road Buttress 6' high wall
03/05/01 5
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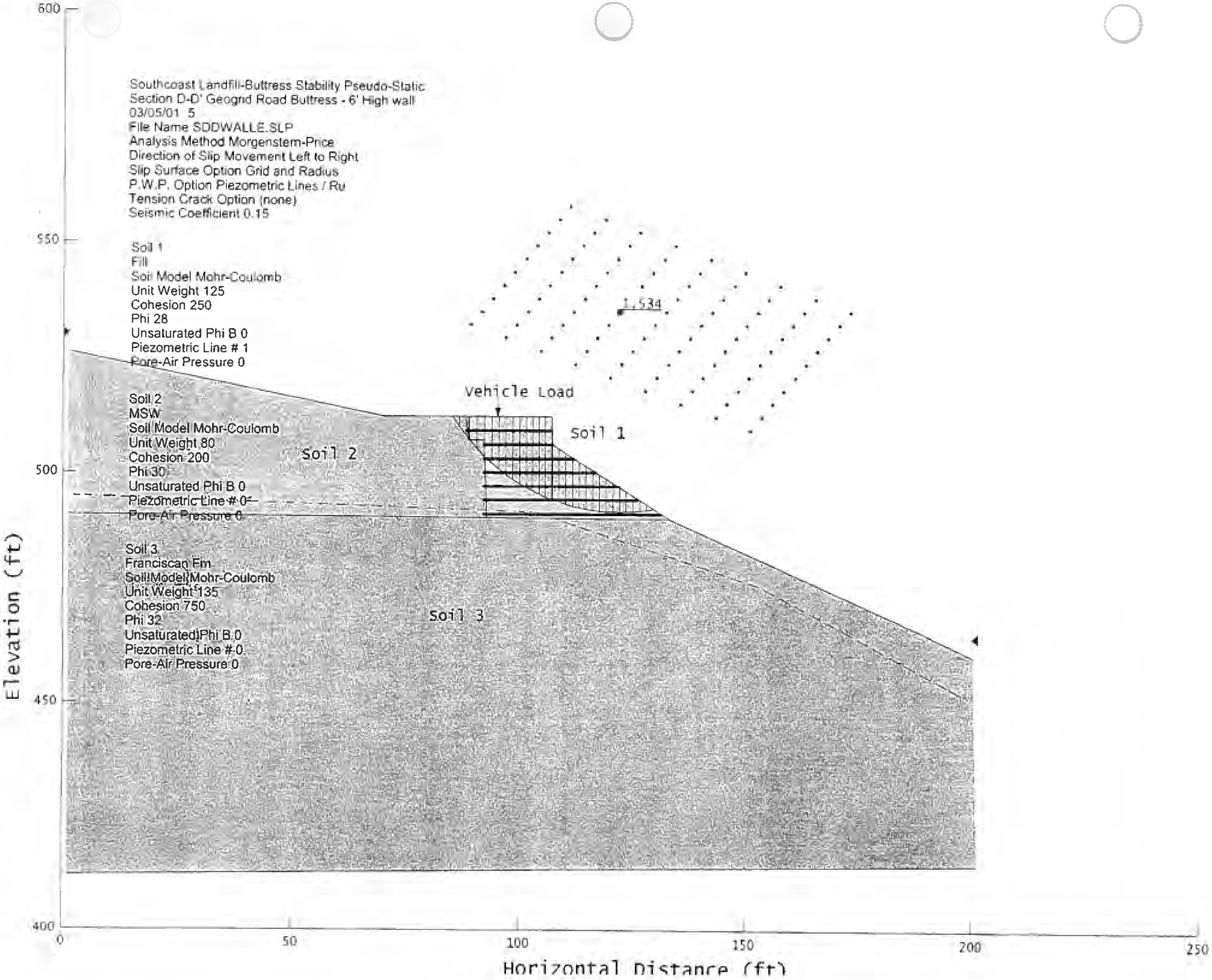
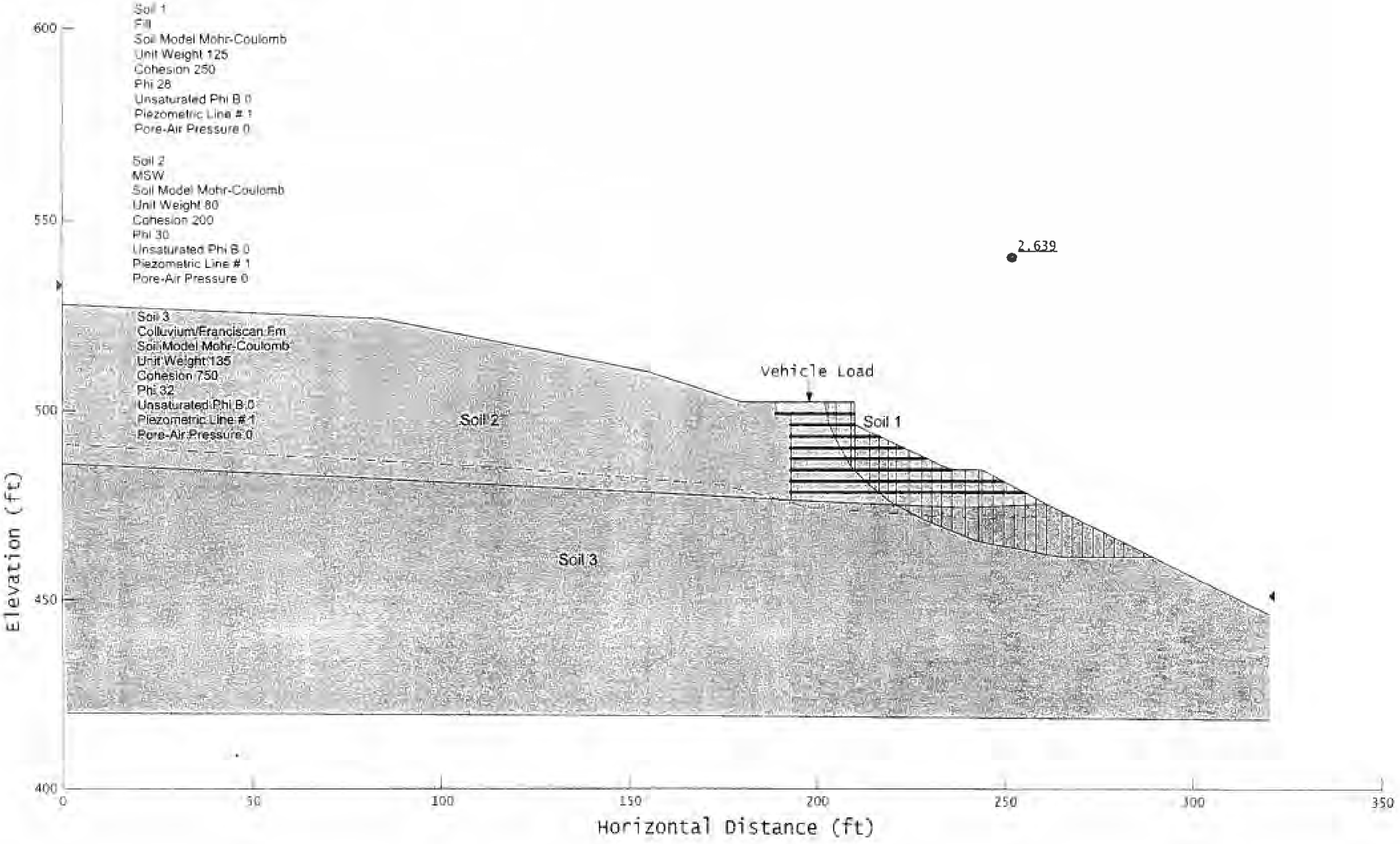
Analysis Method Morgenstern-Price
Direction of Slip Movement Left to Right
Slip Surface Option Grid and Radius
P.W.P. Option Piezometric Lines / Ru
Tension Crack Option (none)
Seismic Coefficient 0
Soil 1
Fill
Soil Model Mohr-Coulomb
Unit Weight 125
Cohesion 250
Phi 28
Unsaturated Phi B 0
Piezometric Line # 1
Pore-Air Pressure 0
Soil 2
MSW
Soil Model Mohr-Coulomb
Unit Weight 80
Cohesion 200
Phi 30
Unsaturated Phi B 0
Piezometric Line # 0
Pore-Air Pressure 0
Soil 3
Franciscan Fm
Soil Model Mohr-Coulomb
Unit Weight 135
Cohesion 750
Phi 32
Unsaturated Phi B 0
Piezometric Line # 0
Pore-Air Pressure 0
2.053
Vehicle Load
Soil 1
Soil 2
Soil 3
600
550
500
450
400
Elevation (ft)
0
50
100
150
200
250
Horizontal Distance (ft)


Southcoast Landfill-Buttress Stability Pseudo-Static
Section D-D' Geogrid Road Buttress - 6' High wall
03/05/01 5
File Name SDDWALLE.SLP
Analysis Method Morgenstern-Price
Direction of Slip Movement Left to Right
Slip Surface Option Grid and Radius
P.W.P. Option Piezometric Lines / Ru
Tension Crack Option (none)
Seismic Coefficient 0.15
Soil 1
Fill
Soil Model Mohr-Coulomb
Unit Weight 125
Cohesion 250
Phi 28
Unsaturated Phi B 0
Piezometric Line # 1
Pore-Air Pressure 0
Soil 2
MSW
Soil Model Mohr-Coulomb
Unit Weight 80
Cohesion 200
Phi 30
Unsaturated Phi B 0
Piezometric Line # 0
Pore-Air Pressure 0
Soil 3
Franciscan Fm
Soil Model Mohr-Coulomb
Unit Weight 135
Cohesion 750
Phi 32
Unsaturated Phi B 0
Piezometric Line # 0
Pore-Air Pressure 0
1.534
Vehicle Load
Soil 1
Soil 2
Soil 3
600
550
500
450
400
Elevation (ft)
0
50
100
150
200
250
Horizontal Distance (ft)

uthcoast Landfill-Geogrid Road Buttress Global Static Stability
Section C-C' Geogrid Road Buttress 6' High
03/05/01 5
File Name SECCC.SLP
Analysis Method Morgenstern-Price
Direction of Slip Movement Left to Right
Slip Surface Option Grid and Radius
P.W.P. Option Piezometric Lines / Ru
Tension Crack Option (none)
Seismic Coefficient 0



Soil 1
Fill
Soil Model Mohr-Coulomb
Unit Weight 125
Cohesion 250
Phi 28
Unsaturated Phi B 0
Piezometric Line # 1
Pore-Air Pressure 0

Soil 2
MSW
Soil Model Mohr-Coulomb
Unit Weight 80
Cohesion 200
Phi 30
Unsaturated Phi B 0
Piezometric Line # 1
Pore-Air Pressure 0

Soil 3
Colluvium/Franciscan Fm
Soil Model Mohr-Coulomb
Unit Weight 135
Cohesion 750
Phi 32
Unsaturated Phi B 0
Piezometric Line # 1
Pore-Air Pressure 0

Southcoast Landfill-Geogrid Road Buttress Pseudo-Static Stability
Section C-C' Geogrid Road Buttress 6' High
03/05/01 5
File Name SECCCDE.SLP
Analysis Method Morgenstern-Price
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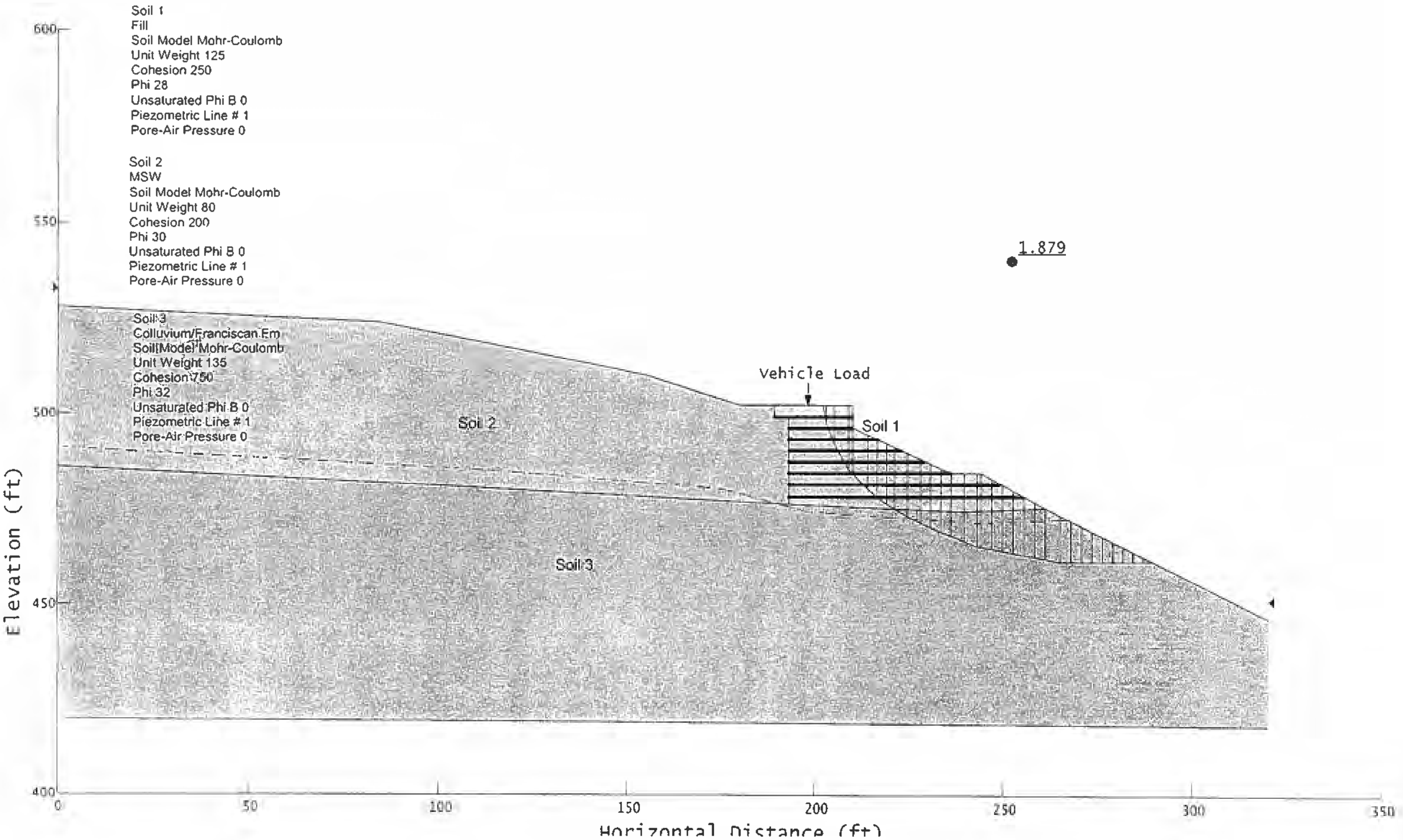
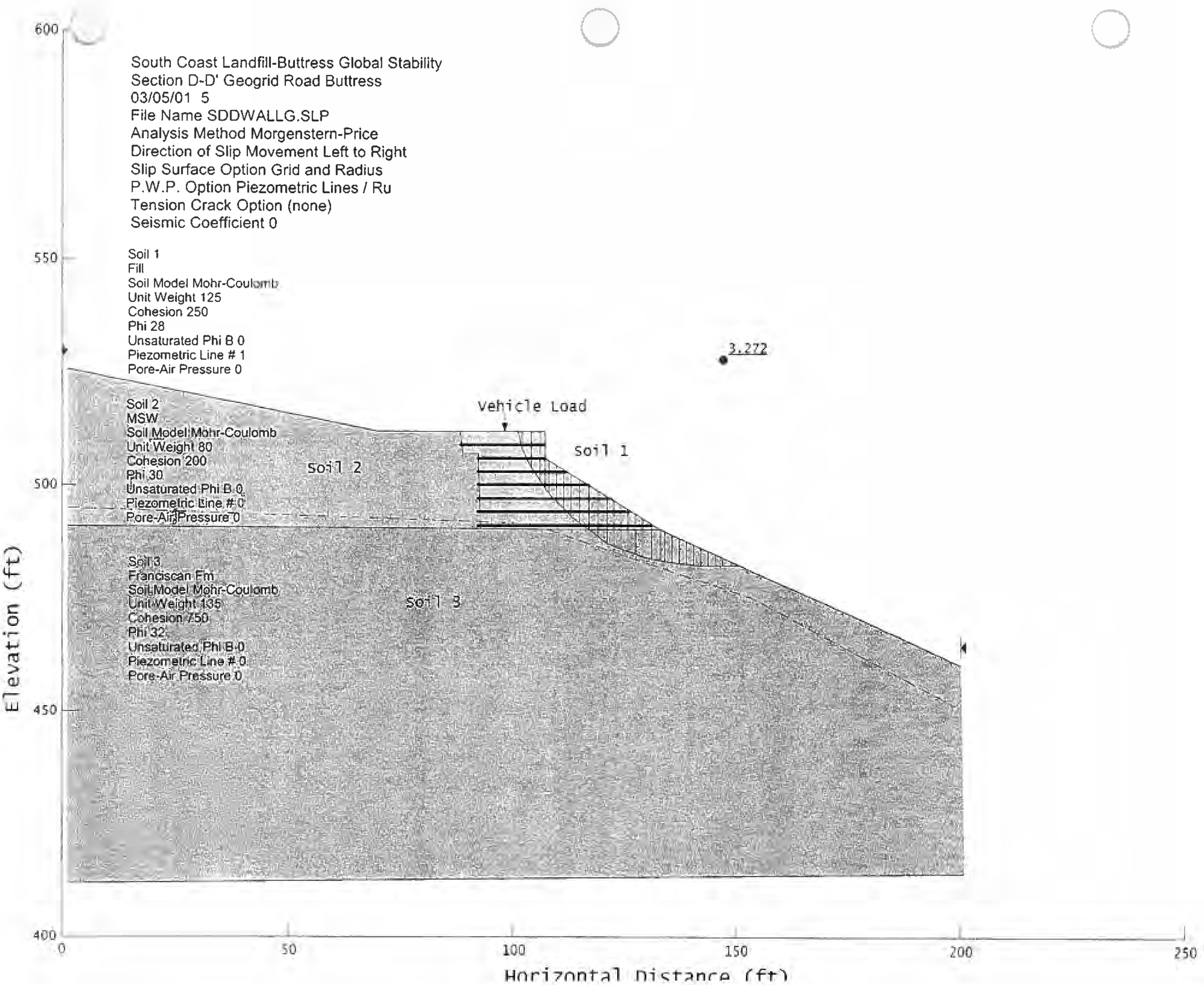
Slip Surface Option Fully Specified
P.W.P. Option Piezometric Lines / Ru
Tension Crack Option (none)
Seismic Coefficient 0.15



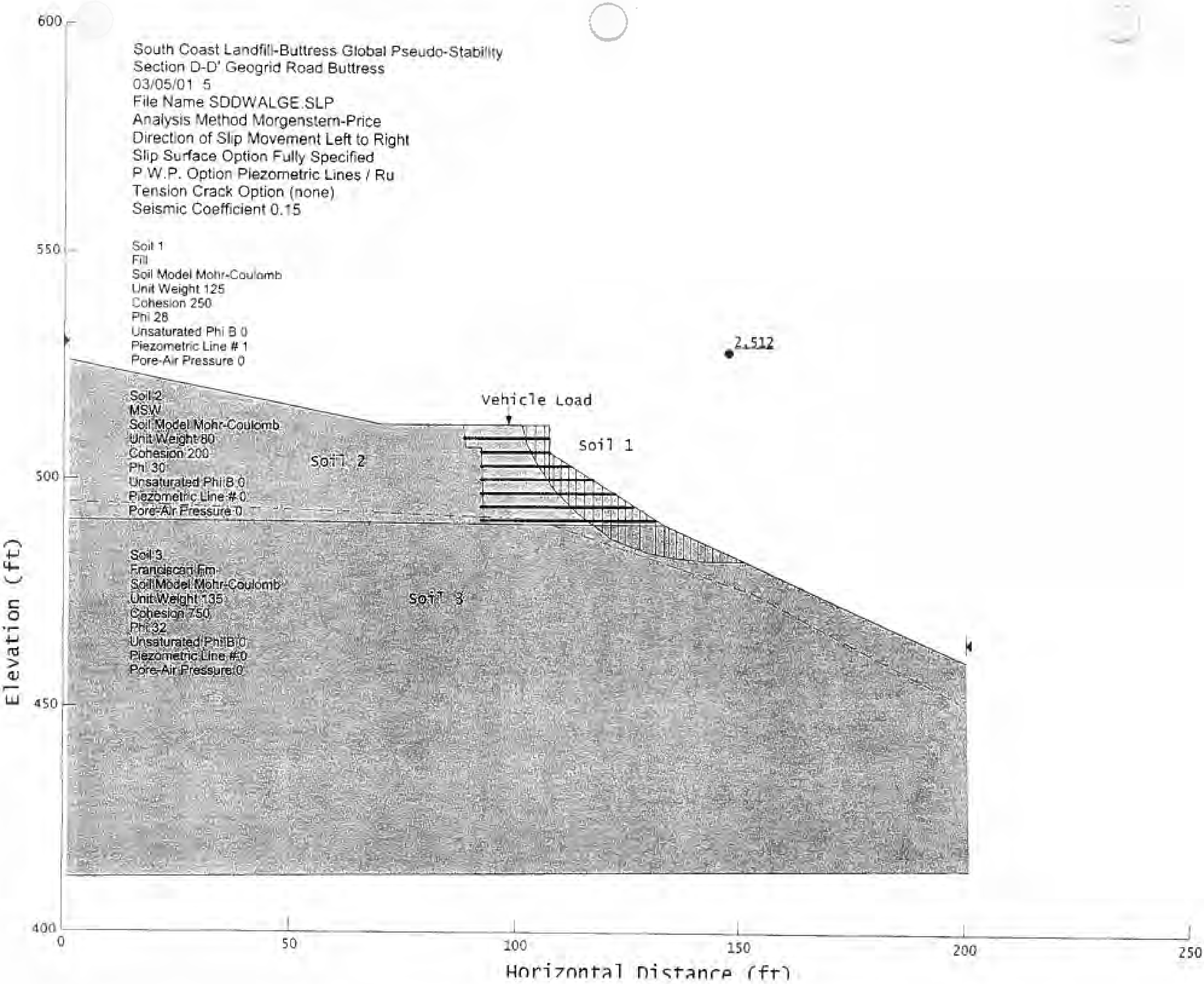


South Coast Landfill-Buttress Global Stability
Section D-D' Geogrid Road Buttress
03/05/01 5
File Name SDDWALLG.SLP
Analysis Method Morgenstern-Price
Direction of Slip Movement Left to Right
Slip Surface Option Grid and Radius
P.W.P. Option Piezometric Lines / Ru
Tension Crack Option (none)
Seismic Coefficient 0

Soil 1
Fill
Soil Model Mohr-Coulomb
Unit Weight 125
Cohesion 250
Phi 28
Unsaturated Phi B 0
Piezometric Line # 1
Pore-Air Pressure 0

Soil 2
MSW
Soil Model Mohr-Coulomb
Unit Weight 80
Cohesion 200
Phi 30
Unsaturated Phi B 0
Piezometric Line # 0
Pore-Air Pressure 0

Soil 3
Franciscan Fm
Soil Model Mohr-Coulomb
Unit Weight 135
Cohesion 750
Phi 32
Unsaturated Phi B 0
Piezometric Line # 0
Pore-Air Pressure 0


South Coast Landfill-Buttress Global Pseudo-Stability
Section D-D' Geogrid Road Buttress
03/05/01 5
File Name SDDWALGE.SLP
Analysis Method Morgenstern-Price
Direction of Slip Movement Left to Right
Slip Surface Option Fully Specified
P.W.P. Option Piezometric Lines / Ru
Tension Crack Option (none)
Seismic Coefficient 0.15
Soil 1
Fill
Soil Model Mohr-Coulomb
Unit Weight 125
Cohesion 250
Phi 28
Unsaturated Phi B 0
Piezometric Line # 1
Pore-Air Pressure 0
Soil 2
MSW
Soil Model Mohr-Coulomb
Unit Weight 80
Cohesion 200
Phi 30
Unsaturated Phi B 0
Piezometric Line # 0
Pore-Air Pressure 0
Soil 3
Franciscan Fm
Soil Model Mohr-Coulomb
Unit Weight 135
Cohesion 750
Phi 32
Unsaturated Phi B 0
Piezometric Line # 0
Pore-Air Pressure 0
2.512
Vehicle Load
Soil 1
Soil 2
Soil 3
Elevation (ft)
600
550
500
450
400
Horizontal Distance (ft)
0
50
100
150
200
250

MATERIAL STRENGTH CALCULATIONS

Choice of Strength Parameters for Tjf (Franciscan Fm).

Previous Work

1) GLA - Laytonville Landfill - Mendocino Co., CA. 2001-082

Tjf $\phi = 30°$ $c = 1500 psf$ $\gamma = 140 pcf$.

2) EMCON - Prelim. Seismicity & Slope Stability Analysis
Gualala, CA - 20122-006.001 - 8/98.

a) Back-calculation

Tjf $\phi = 24° - 36°$ $c = 90 - 600 psf$. $\gamma = 125 pcf$

$\phi_{Ave} = 30°$ $C_{ave} = 345 psf$ $\gamma = 125 pcf$.

b) Based on blow-counts & soil descriptions.

Tjf $\phi = 28°$ $c = 0$ $\gamma = 125$ $\gamma_T = 130$

$\phi = 0°$ $c = 1000$ " "

$\phi = 28°$ $c = 430$ " "

3) GLA - Based on Lab Data.

a) Direct Shear Testing - With 3/8$^+$ rocks removed (conservative)

B-1@15' → Tjf $\phi_{peak} = 30°$ $c = 1000 psf$.

$\phi_{residual} = 26$ $c_r = 1000 psf$

B-2@10' Tjf. $\phi_p = 33°$ $c_p = 600 psf$

$\phi_r = 30°$ $c_r = 150 psf$

B-1@20' Tjf $\phi_p = 32°$ $c_p = 400 psf$

$\phi_r = 28°$ $c_r = 350 psf$

B-4@5' Tjf $\phi_p = 31°$ $c_p = 800 psf$

$\phi_r = 27°$ $c_r = 800 psf$

Average of all 6 points - Direct Shear Testing.

Tjf $\phi_p = 32°$ $c_{peak} = 625 psf \Rightarrow 700 psf$.

$\phi_r = 28°$ $c_{residual} = 580 psf$

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REVIEWED BY:	DATE:	SHEET: 1 OF:

b) Triaxial Testing. – 4 samples tested.

Best fit curves

– $\phi = 44°$	$c = 0$ psf
$\phi = 35°$	$c = 300$ psf
$\phi = 40°$	$c = 0$ psf
$\phi = 33°$	$c = 450$ psf
$\phi_{AVE} = 38°$	$c = 200$ psf.

4) GLA - Sonoma Central Landfill (Shear Testing Ave. Results)

Tjf = $\phi = 30°$ $c = 1480$ psf $\gamma = 136$ psf.

⊳. For MSW use GLA typical value of $\gamma = 80$ $\phi = 30°$ $c = 200$ psf.

Summary: Results of Triaxial & Shear Testing Ave. Best Fit curves indicate friction angle varies from 30° – 40° with an average median value of ~32° – 33°. Cohesion varies from 0 to 1000 psf with an average median value of 450 – 625 psf. (The direct shear values are with rock fragments removed from samples). Rock fabric is generally oriented that bedding is roughly vertical so horizontal strength is typically > than vertical strength. Direct shear testing generally mimics the lower (along bedding) vertical strength.

Approach:

- Use Slope W with an average rock strength and a tension crack through the landfill due to a seismic event, and use $\phi = 32°$ $c = 750$ psf.
  $\gamma = 135$ pcf

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REVIEWED BY:	DATE:	SHEET: 2 OF:

Seismicity: (adopted from EMCON 8/98, 20122-006.001)

San Andreas Fault - M = 8.0
Max Horizontal Acceleration = 0.9g (Bray, R., A. & M, Fig 2a)
Mean Period, T_m = = 0.52 sec (" 2b)
Duration of Shaking = 33 sec (" 2c)

Calculate Seismic Displacement:

Procedure: Bray & Rathje, 1998, Earthquake - Induced Displacements of Solid-Waste Landfills, JGGE, 3/1998.

Data:

Site Condition: Rock w/ Shallow to Moderate Soil / Gouge
MSW Height: 35 feet.
$V_{S\,Ave}$: 420 ft/sec (Bray & Rathje, 1998), Fig 2)

EQ Magnitude: 8.0
MHA: 0.9g
D_{5-95}: 33 sec
Yield Acceleration: see attached calculations: (Slope/W)

Calculations:

Mean Period of Shaking: T_m = 0.52 sec
Predominant Period of MSW = T_s = $4H/V_s$ = 0.33 sec = T_o
T_s/T_m = 0.63 sec.
NRF (Fig. 6, Bray & Rathje, 1998) = 0.75 (Considering Fault Gouge!)

From Bray & Rathje, 1998, Fig. 7b for rock site.
@ T_s/T_m = 0.63 sec ⇒ MHEA / $MHEA_{rock}$ · NRF = 0.8

MHEA = $MHEA_{rock}$ · NRF 0.8

= 0.90g · 0.75 × 0.8 = 0.54g = K_{max}

As a check - calculate K_{max} at specific point in MSW by:

Calculate MHA_{top}. use Fig. 8, Bray & Rathje, 1998

FOR T_s/T_n = 0.64 sec ⇒ MHA / MHA_{rock} · NRF = 1.26g

South Coast

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PREPARED BY:	DATE: 3-3-03	PROJECT NO: 2001-082
REVIEWED BY:	DATE:	SHEET: 3 OF:

$MHA_{top} = MHA_{rock} \cdot NRF \cdot 1.26 = (0.9)(0.75)(1.26) = 0.85g$ @ (top) = K_{MAX}

Calculate K_{MAX} @ failure plane of SLOPE/W runs by inspection.

For SECTION A-A' ⇒ Failure occurs at $z/H = 0.70$.

From Makdisi & Seed, 1978. @ $z/H = 0.70$ ⇒ $a_{max_{ave}} = a_{max_{top}} \cdot 0.50$
OR (Kramer, 1998, Fig 10.18).

So, $K_{MAX_{section A\text{-}A'}} = 0.85 \cdot (0.5) = 0.43g$.

For Section B-B': ⇒ Failure occurs @ $z/H = 1$ (at base of MSW)

For $z/H = 1$; $a_{max_{ave}}$ section B-B' $= a_{max_{top}} \cdot 0.35 = 0.30g$.

So, use $K_{MAX_{MSW}} = 0.54$ (from Bray & Rathje, 1998).

MSW Displacement. (For NORMAL HIGH GROUNDWATER):

	Ky	Kmax	Ky/Kmax	Displacement(1)
Section A-A'	0.58g	0.54g	1.07	< 1/2"
Section B-B'	0.53g	0.54g	0.98	< 1/2"

(1) Bray & Rathje, 1998, Fig. 10.

	10' SATURATED MSW	
	Ky/Kmax	Displacement
Section A-A'	0.98	< 1/2"
Section B-B'	0.90	< 1/2"

Calculate displacement for Tjf Slopes below MSW:

Use Makdisi & Seed, 1978. (Below):

Horizontal ground acceleration at site = $MHEA_{rock} \cdot NRF$
$= (0.90) \cdot (0.75) = 0.68g. = K_{max_{ROCK}}$
in fault gouge.

Reference:

Makdisi & Seed, 1978, Simplified Procedure for estimating dam & embankment earthquake-induced deformations, JGED, ASCE, Vol. 104, GT 7, p849-867.

South Coast

PREPARED BY:	DATE: 3-5-03	PROJECT NO: 2001-082
REVIEWED BY:	DATE:	SHEET: 4 OF:

Actual K_{max} will be somewhere between $K_{max\,MSW} = 0.54g$ and $K_{max\,ROCK} = 0.68g$. So calculate K_{max} based on % of failure plane in each material.

For SECTION A-A':

$K_y = 0.30g$ $K_{max} = 0.1(0.54g) + (0.9)(0.68g) = 0.67g$

$K_y/K_{max} = 0.45$: (from Makdisi & Seed, 1978, Attached Fig. 10.19)

$U/k_{max}T_0 = 0.085$ @ $M \simeq 8.0$

$U = (0.67)\left(\frac{32 ft}{sec}\right)(0.33\ sec).(0.085) = 0.61' \cong 7''$

FOR SECTION B-B'

$K_y = 0.36g$ $K_{max} \simeq (0.15)(0.54g) + (0.85)(0.68g) = 0.66g$

$K_y/K_{max} = 0.53$ (from Fig. 10.19 attached)

$u/a_{max}T_0 = 0.07$

$U = (0.66)(32)(0.33)(0.07) \cong 0.5' \cong \underline{\underline{6''}}$

- Look at case with 15' of Qcol (Colluvium) at WEST. EDGE OF MSW:

	K_y	K_{max}	K_y/K_{max}	Displacement
SECTION A-A'	0.31	0.66	0.47	$(0.68)(32)(0.33)(0.08) = 7\frac{1}{2}''$
SECTION B-B'	0.42	0.68	0.62	$(0.68)(32)(0.33)(0.03) = 2\frac{1}{2}''$

- Look at case with 10' of SATURATED MSW:

	K_y	K_{max}	K_y/K_{max}	Displacement
SECTION A-A'	0.26g	$(0.13)(0.54)+(0.87)(0.68) = 0.66g$	0.40	$(0.66)(32)(0.33)(0.12) = 10''$
SECTION B-B'	0.32g	$(0.15)(.54)+(0.85)(0.68) = 0.66g$	0.48	$(0.66)(32)(0.33)(0.07) = 6''$

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REVIEWED BY:	DATE:	SHEET: 5 OF:

from Kramer, 1996, Geotechnical Earthquake Engineering

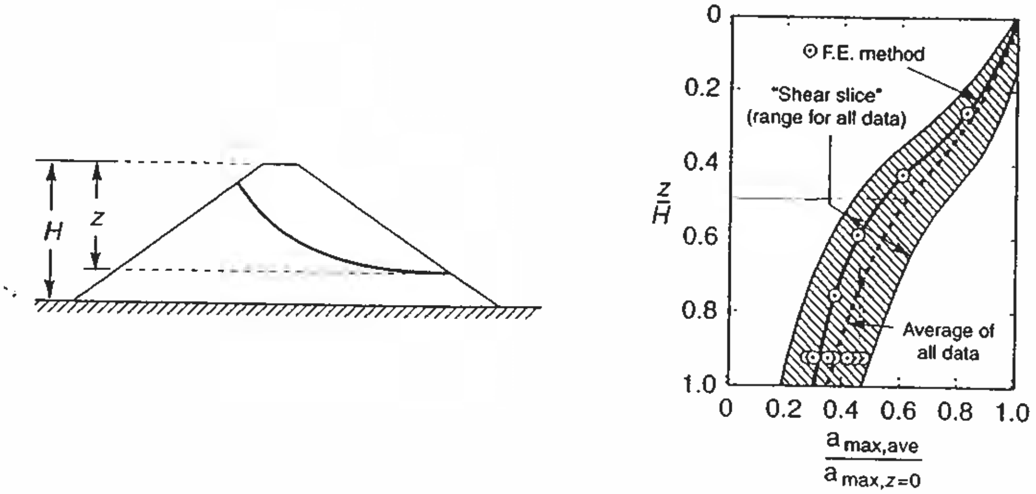


Figure 10.18 Variation of average maximum acceleration with depth of potential failure surface for dams and embankments. (After Makdisi and Seed (1978). Simplified procedure for estimating dam and embankment earthquake-induced deformations, Journal of the Geotechnical Engineering Division, Vol. 104, No. GT7. Reprinted by permission of ASCE.)

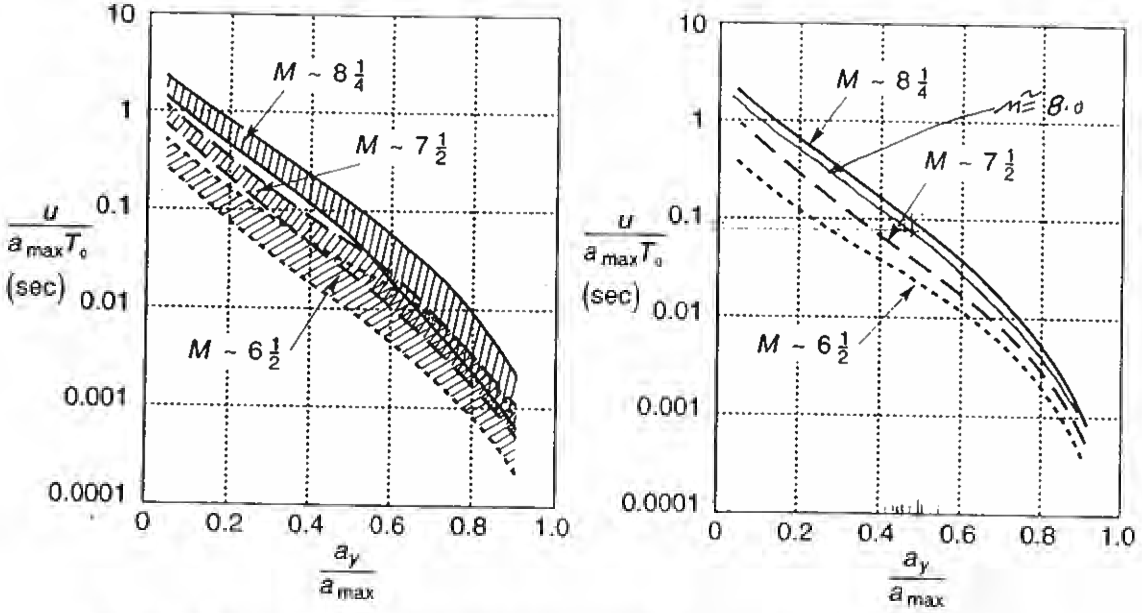


Figure 10.19 Variation of normalized permanent displacement with yield acceleration for earthquakes of different magnitudes: (a) summary for several earthquakes and dams/embankments; (b) average values. (After Makdisi and Seed (1978). Simplified procedure for estimating dam and embankment earthquake-induced deformations, Journal of the Geotechnical Engineering Division, Vol. 104, No. GT7. Reprinted by permission of ASCE.)

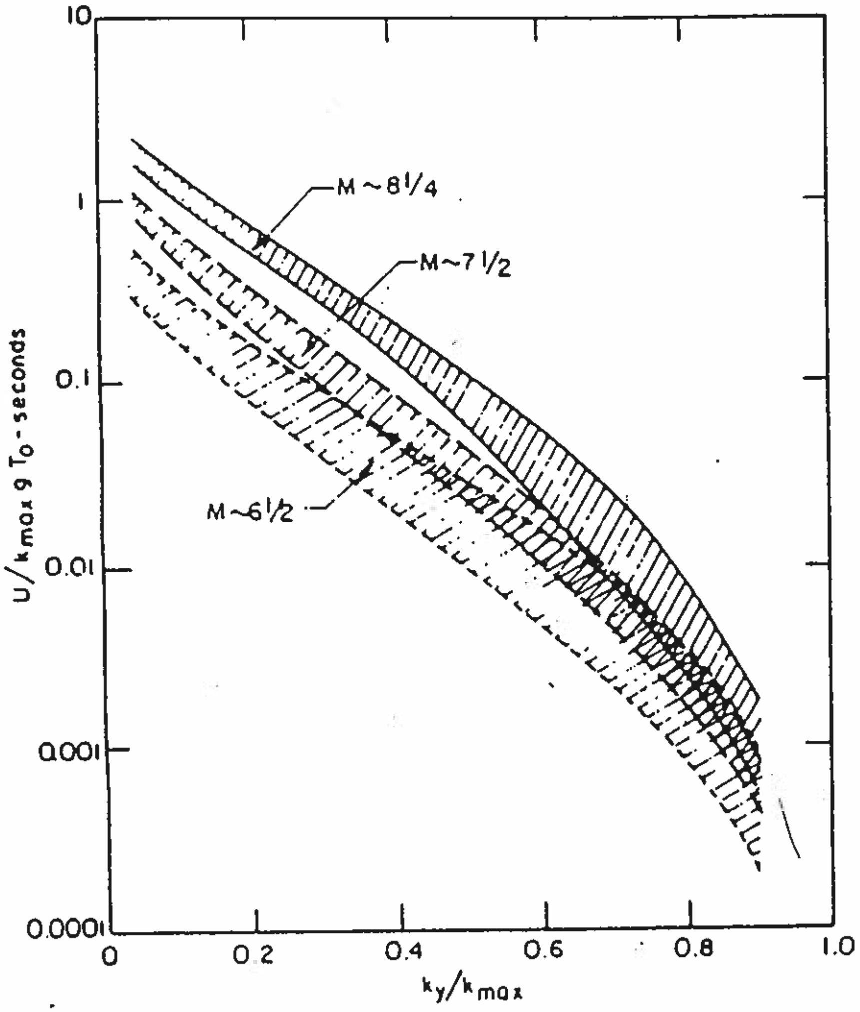


FIG. 15 VARIATION OF NORMALIZED PERMANENT DISPLACEMENT WITH YIELD ACCELERATION - SUMMARY OF ALL DATA

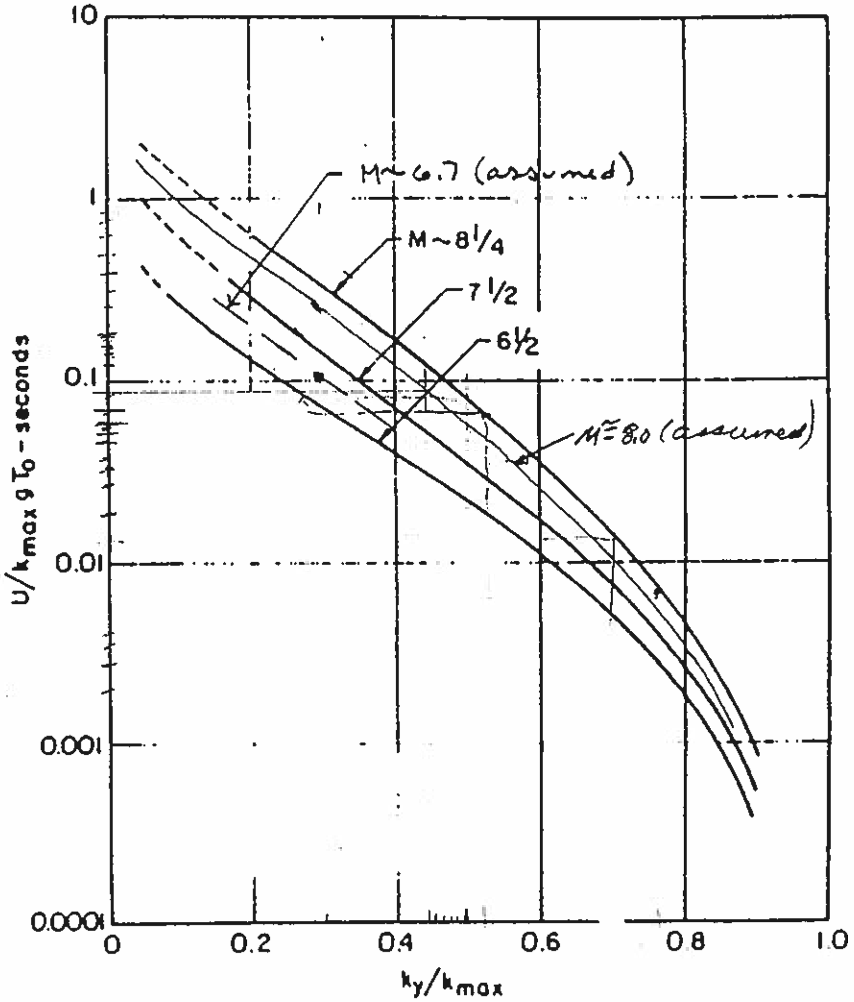
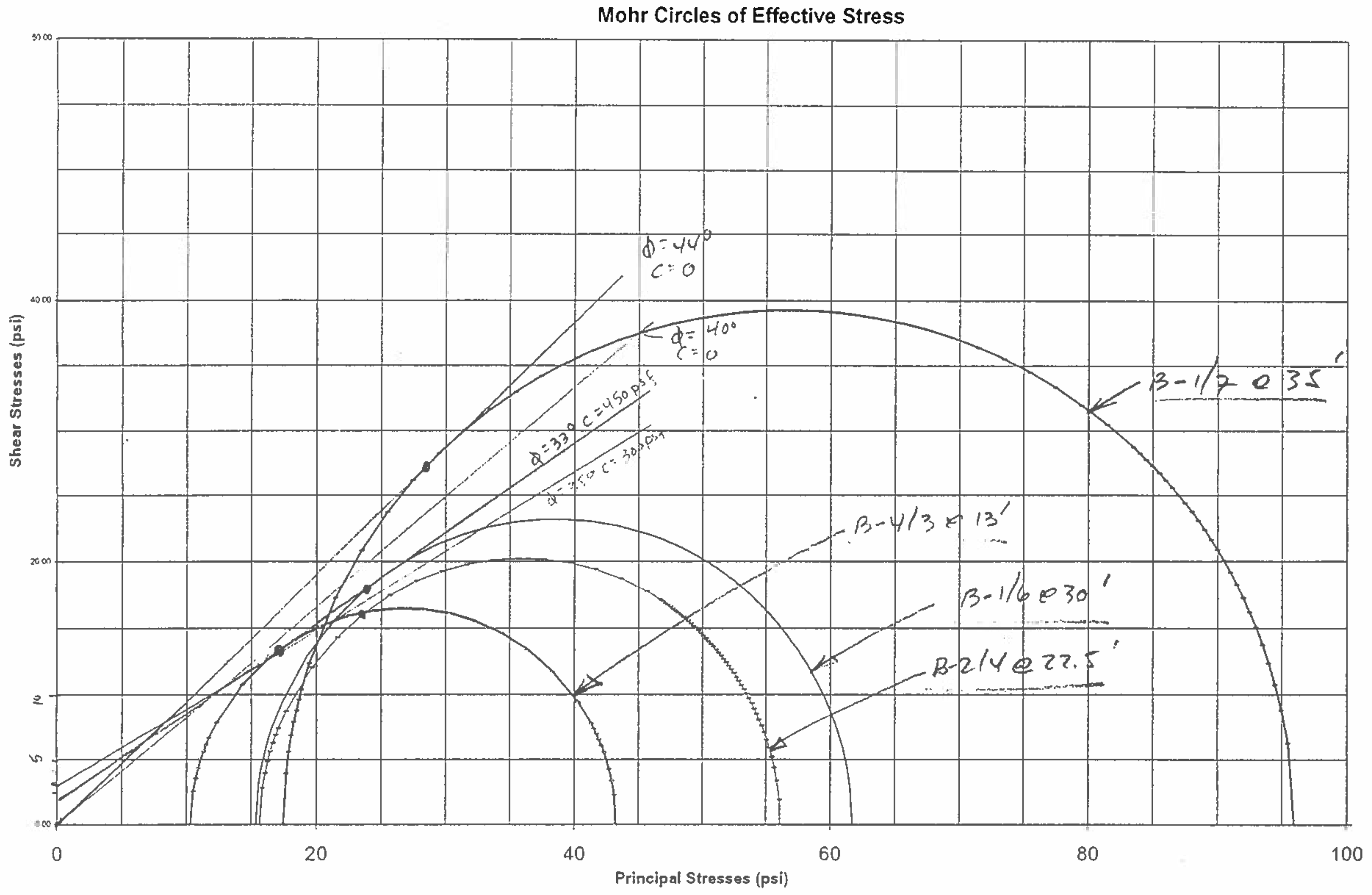
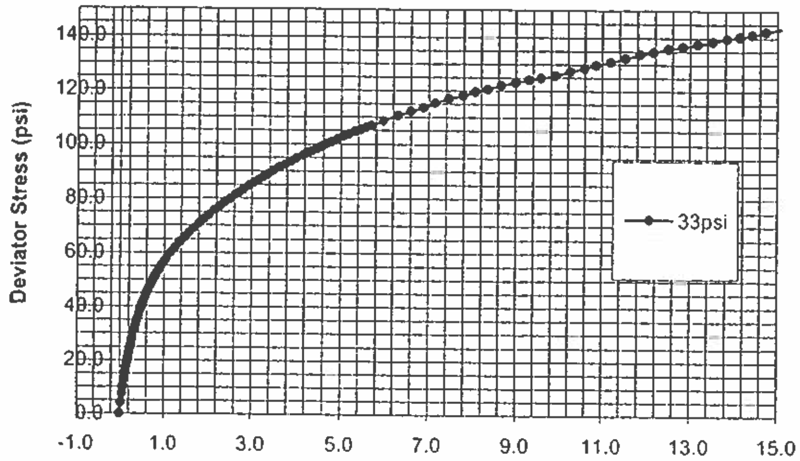


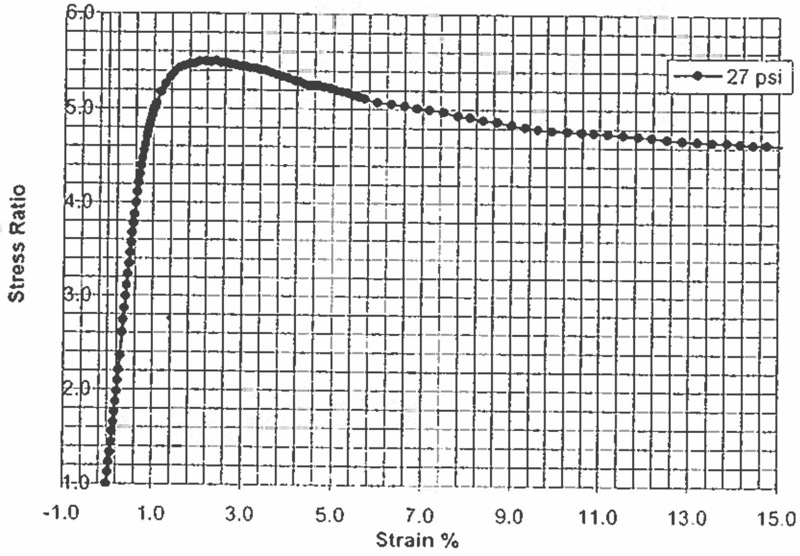
FIG. 16 VARIATION OF AVERAGE NORMALIZED DISPLACEMENT WITH YIELD ACCELERATION

(from Makdisi & Seed, 1978.)

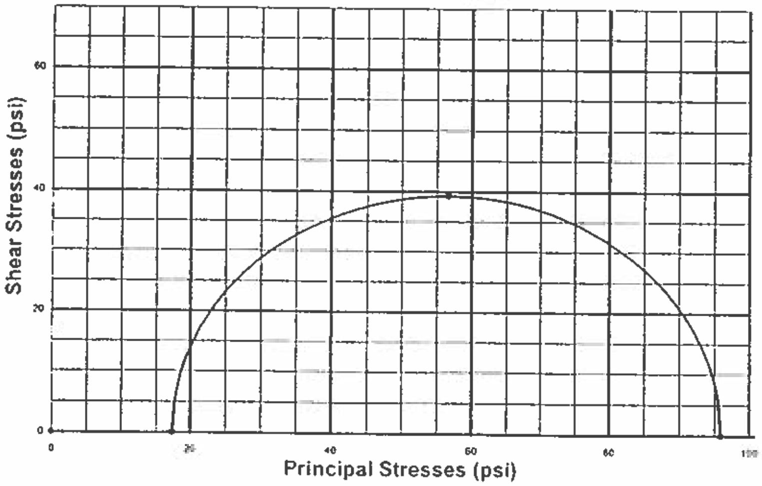

Mohr Circles of Effective Stress
Shear Stresses (psi)
60.00
40.00
20.00
10
5
0.00
Principal Stresses (psi)
0
20
40
60
80
100
ϕ = 44° c = 0
ϕ = 40° c = 0
ϕ = 33° c = 450 psf
ϕ = 35° c = 300 psf
B-1/7 @ 35'
B-4/3 @ 13'
B-1/6 @ 30'
B-2/4 @ 22.5'

3





Mohr Circles of Effective Stress



Soil Description: Brown Mottled, Clayey Silt w. Rock Frgmts

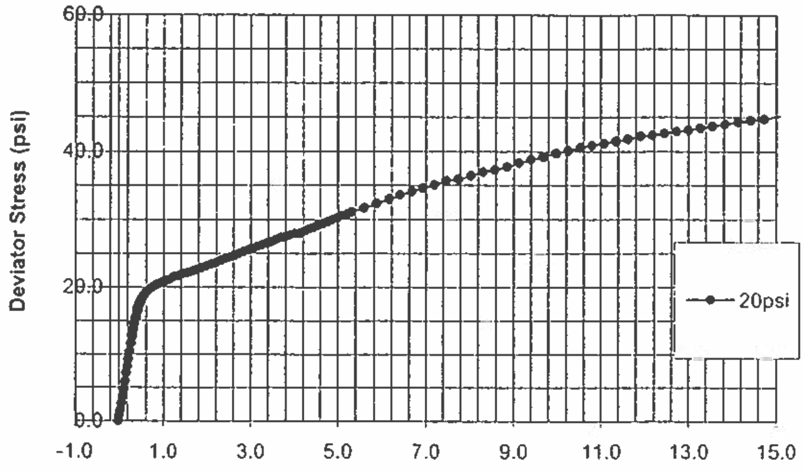
Type of Specimen: *Undisturbed*

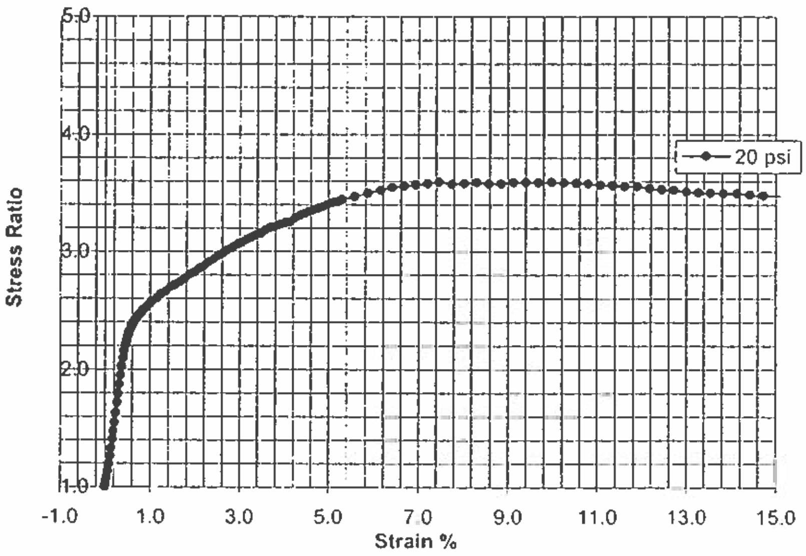
SPECIMEN			A	B	C
INITIAL	Wet Density (pcf)		137.1		
	Water Content (%)		14.4		
	Dry Density (pcf)		119.8		
FINAL	Wet Density (pcf)				
	Water Content (%)		15.3		
	Dry Density (pcf)				
SATURATION STAGE	Initial pwp	psi	0.4		
	Saturated pwp		37.0		
	Final Cell pressure		40.0		
	B value		95.0		
CONSOLIDATIO N STAGE	Cell Pressure	psi	70.0		
	Back Pressure		37.0		
	Initial pwp				
	Final pwp				
COMPRESSION STAGE	Cell Pressure	psi	70.0		
	Initial pwp		37.0		
	Initial σ'_3		33.0		
	Strain Rate (in./min.)		0.0084		
FAILURE CONDITION	Strain %		2.4		
	$(\sigma_1 - \sigma_3)_f$	psi	78.5		
	u_f		15.6		
	σ'_{3f}		17.4		
	σ'_{1f}		95.9		
CONSOLIDATIO N PARAMETERS	C_v				
	m_v				
	k				
SHEAR STRENGTH PARAMETERS			c' =		psf
			ϕ' =		

CONSOLIDATED UNDRAINED TRIAXIAL COMPRESSION TEST

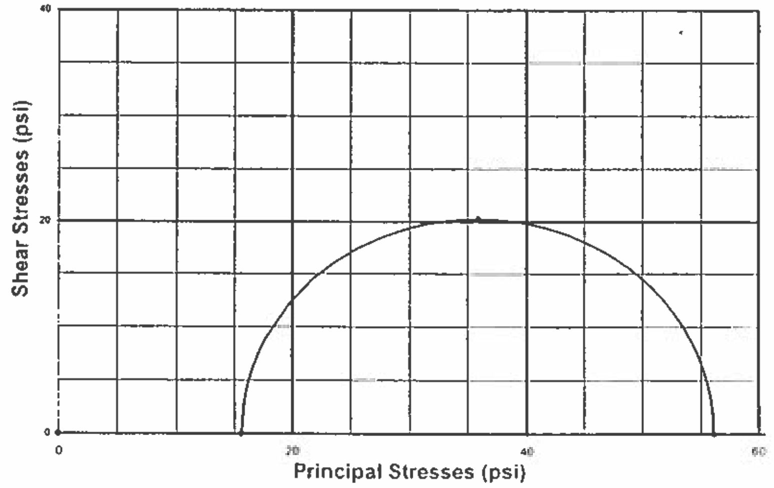
ASTM D 4767

Job Name: SouthCoast LF		Date: 12-3-02
Job No.	2001-084	Sample No.: B-1/7 @ 35'





Mohr Circles of Effective Stress



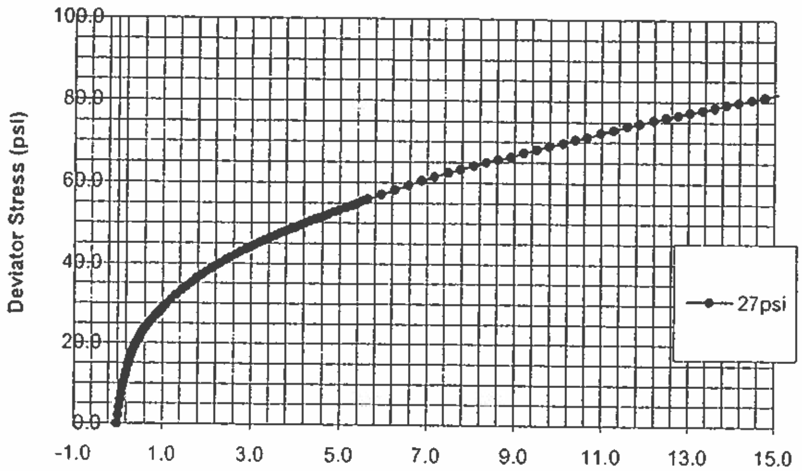
Soil Description: Brown Mottled, Clayey Silt w. Siltstone Frgmts

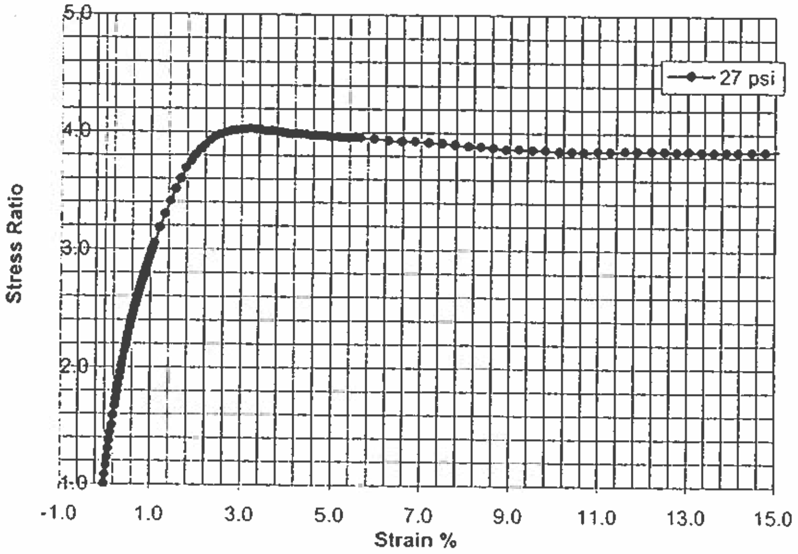
Type of Specimen: *Undisturbed*

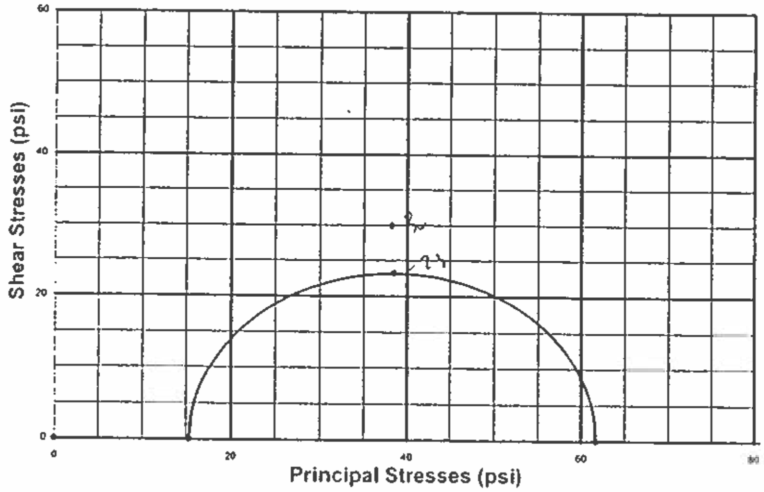
SPECIMEN			A	B	C
INITIAL	Wet Density (pcf)		128.2		
	Water Content (%)		17.6		
	Dry Density (pcf)		109.0		
FINAL	Wet Density (pcf)				
	Water Content (%)		19.0		
	Dry Density (pcf)				
SATURATION STAGE	Initial pwp	psi	0.4		
	Saturated pwp	psi	37.0		
	Final Cell pressure	psi	40.0		
	B value		95.0		
CONSOLIDATION STAGE	Cell Pressure	psi	57.0		
	Back Pressure	psi	37.0		
	Initial pwp	psi			
	Final pwp	psi			
COMPRESSION STAGE	Cell Pressure	psi	57.0		
	Initial pwp	psi	37.0		
	Initial σ'_3	psi	20.0		
	Strain Rate (in./min.)		0.0084		
FAILURE CONDITION	Strain %		10.5		
	$(\sigma_1 - \sigma_3)_f$	psi	40.5		
	u_f	psi	4.4		
	σ'_{3f}	psi	15.6		
	σ'_{1f}	psi	56.1		
CONSOLIDATION PARAMETERS	c_v				
	m_v				
	k				

SHEAR STRENGTH PARAMETERS	c' = psf ϕ' =

CONSOLIDATED UNDRAINED TRIAXIAL COMPRESSION TEST	Job Name: SouthCoast LF	Date: 12-3-02
ASTM D 4767	Job No. 2001-084	Sample No.: B-2/4 @ 22.5'






Mohr Circles of Effective Stress

Soil Description: Brown Mottled, Clayey Silt w. Rock Frgmts

Type of Specimen: *Undisturbed*

SPECIMEN			A	B	C
INITIAL	Wet Density (pcf)		131.0		
	Water Content (%)		17.7		
	Dry Density (pcf)		111.3		
FINAL	Wet Density (pcf)				
	Water Content (%)		19.3		
	Dry Density (pcf)				
SATURATION STAGE	Initial pwp	psi	0.4		
	Saturated pwp		37.0		
	Final Cell pressure		40.0		
	B value		95.0		
CONSOLIDATION STAGE	Cell Pressure	psi	64.0		
	Back Pressure		37.0		
	Initial pwp				
	Final pwp				
COMPRESSION STAGE	Cell Pressure	psi	64.0		
	Initial pwp		37.0		
	Initial σ'_3		27.0		
	Strain Rate (in./min.)		0.0084		
FAILURE CONDITION	Strain %		3.4		
	$(\sigma_1 - \sigma_3)_f$	psi	46.4		
	u_f		11.7		
	σ'_{3f}		15.3		
	σ'_{1f}		61.7		
CONSOLIDATION PARAMETERS	c_v				
	m_v				
	k				

SHEAR STRENGTH PARAMETERS	c' =	psf
	ϕ' =	

CONSOLIDATED UNDRAINED TRIAXIAL COMPRESSION TEST

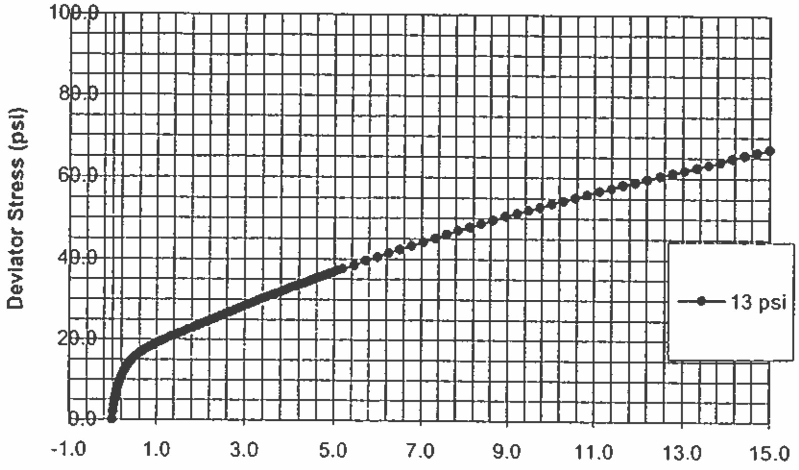
ASTM D 4767

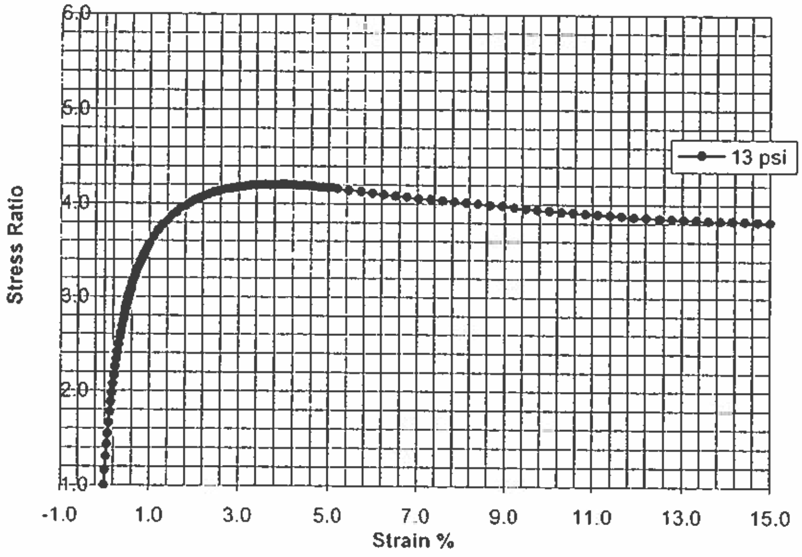
Job Name: SouthCoast LF

Date: 12-3-02

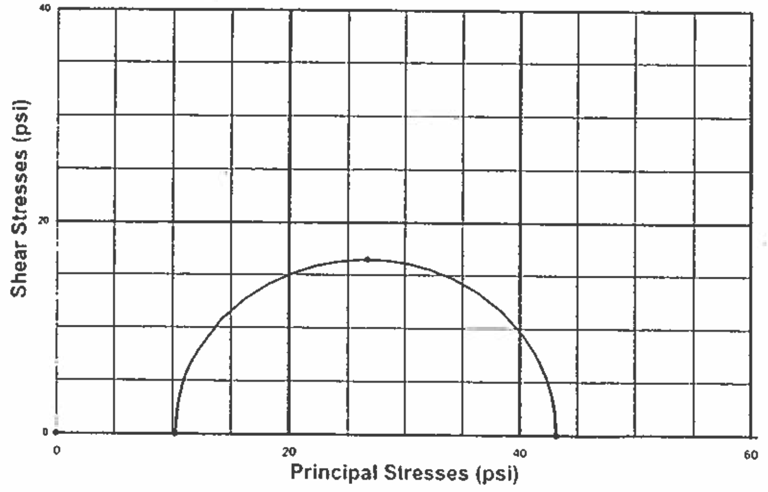
Job No. 2001-084

Sample No.: B-1/6 @ 30'





Mohr Circles of Effective Stress



Soil Description: D. Gray, F.C. Sandy Clay					
Type of Specimen: *Undisturbed*					
SPECIMEN			A	B	C
INITIAL	Wet Density (pcf)		143.9		
	Water Content (%)		7.2		
	Dry Density (pcf)		134.2		
FINAL	Wet Density (pcf)				
	Water Content (%)		10.9		
	Dry Density (pcf)				
SATURATION STAGE	Initial pwp	psi	0.4		
	Saturated pwp		37.0		
	Final Cell pressure		40.0		
	B value		95.0		
CONSOLIDATIO N STAGE	Cell Pressure	psi	50.0		
	Back Pressure		37.0		
	Initial pwp				
	Final pwp				
COMPRESSION STAGE	Cell Pressure	psi	50.0		
	Initial pwp		37.0		
	Initial σ'_3		13.0		
	Strain Rate (in./min.)		0.0084		
FAILURE CONDITION	Strain %		4.0		
	$(\sigma_1 - \sigma_3)_f$	psi	33.0		
	u_f		2.7		
	σ'_{3f}		10.3		
	σ'_{1f}		43.2		
CONSOLIDATIO N PARAMETERS	c_v				
	m_v				
	k				
SHEAR STRENGTH PARAMETERS			c' =		psf
			ϕ' =		

CONSOLIDATED UNDRAINED TRIAXIAL COMPRESSION TEST

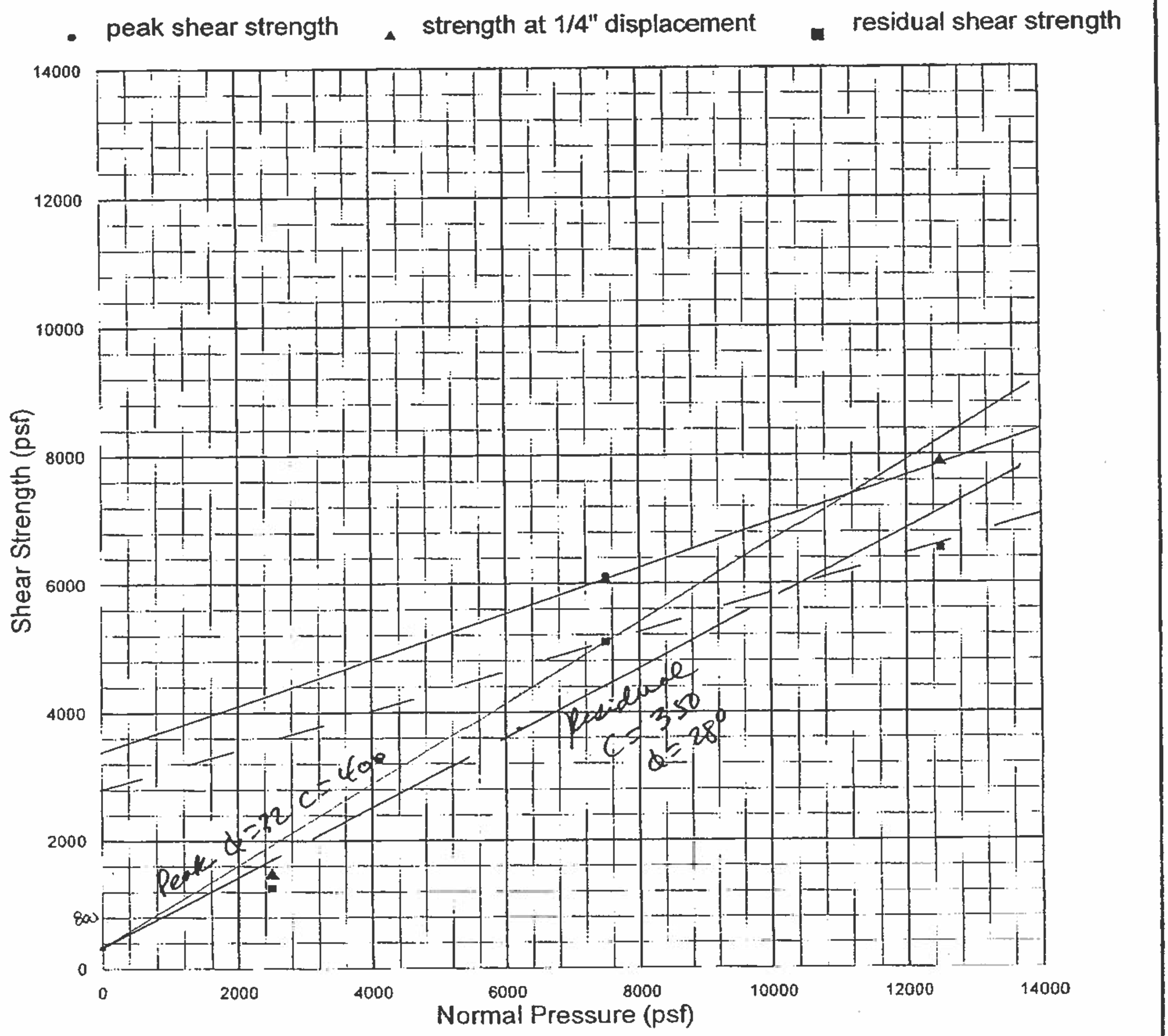
ASTM D 4767

Job Name: SouthCoast LF	Date: 4-05-01
Job No. 2001-084	Sample No.: B-4/3 @ 13'

Job No. 2001-082

DIRECT SHEAR TEST - ASTM D-3080

SouthCoast LF - Mendocino County



Strain Rate: 0.0042 in. / min.

Sample	Type	Description	Dry Density (pcf)	Initial W.C. (%)
B-1/4 @ 20'	Undisturbed & Saturated	Silty Clay w. Rock Frgmts	77.1	21.0

Normal Pressure (psf)	Peak Strength (psf)	Ultimate Strength (psf)	Residual Strength (psf)
2500	1480 @ 0.2370"	1460	1260
7500	6110 @ 0.2320"	6050	5090
12500	7880 @ 0.2430"	7880	6540
	C = 3400 psf	C = 3400 psf	C = 2800 psf
	φ = 19 deg.	φ = 19 deg.	φ = 17 deg.

Peak Best Fit C = 386
φ = 32°

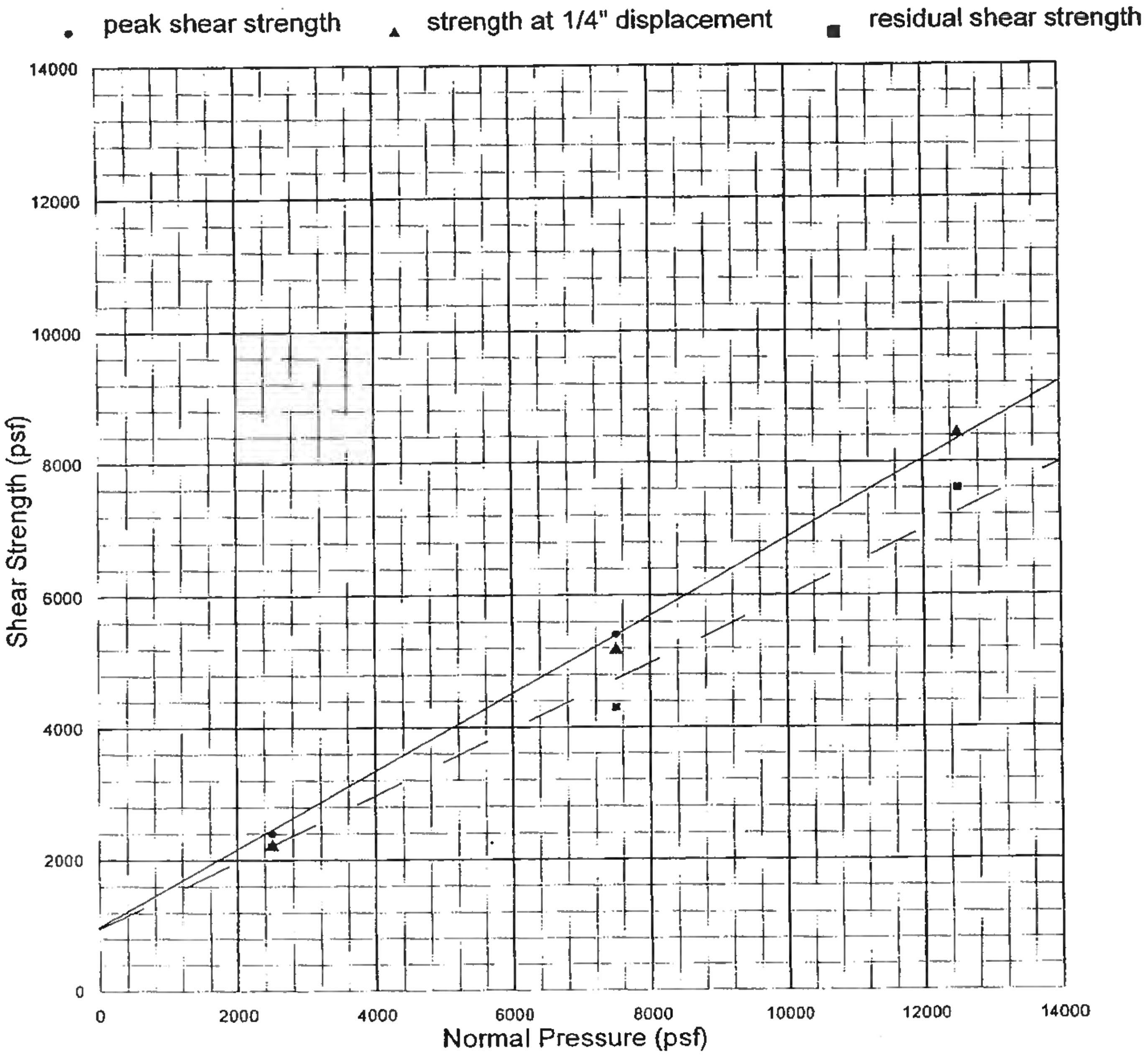
C = 336
φ = 28°

GeoLogic Associates

Job No. 2001-082

DIRECT SHEAR TEST - ASTM D-3080

SouthCoast LF - Mendocino County



Strain Rate: 0.0042 in. / min.

Sample	Type	Description	Dry Density (pcf)	Initial W.C. (%)
B-1/3 @ 15'	Undisturbed & Saturated	Silty Clay w. Rock Frgmts	102.7	17.9

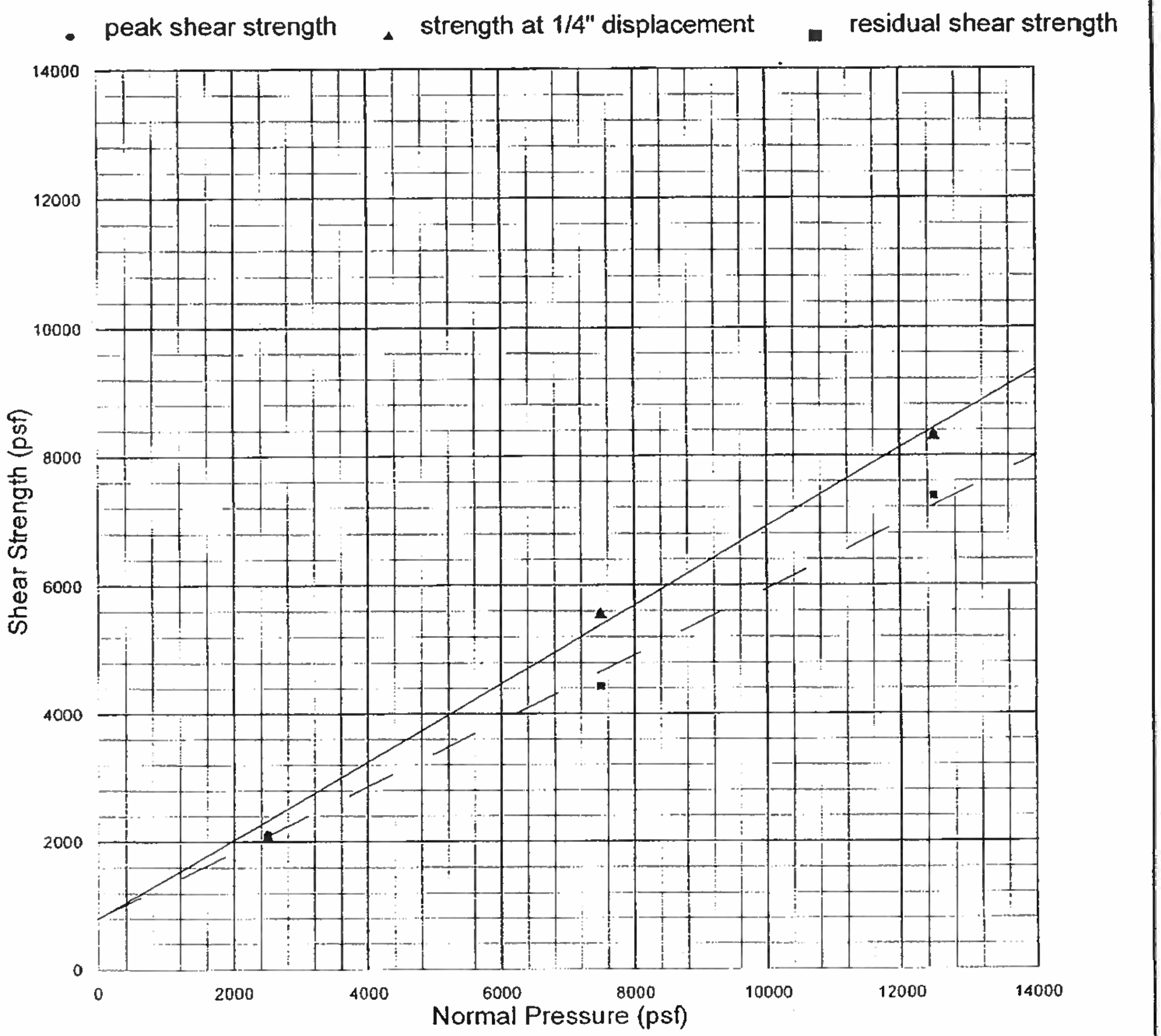
Normal Pressure (psf)	Peak Strength (psf)	Ultimate Strength (psf)	Residual Strength (psf)
2500	2400 @ 0.1490"	2200	2200
7500	5410 @ 0.1570"	5170	4300
12500	8420 @ 0.2405"	8420	7600
	C = 1000 psf	C = 1000 psf	C = 1000 psf
	ϕ = 30 deg.	ϕ = 30 deg.	ϕ = 26 deg.

c = 900
ϕ ≈ 31°

Job No. 2001-082

DIRECT SHEAR TEST - ASTM D-3080

SouthCoast LF - Mendocino County

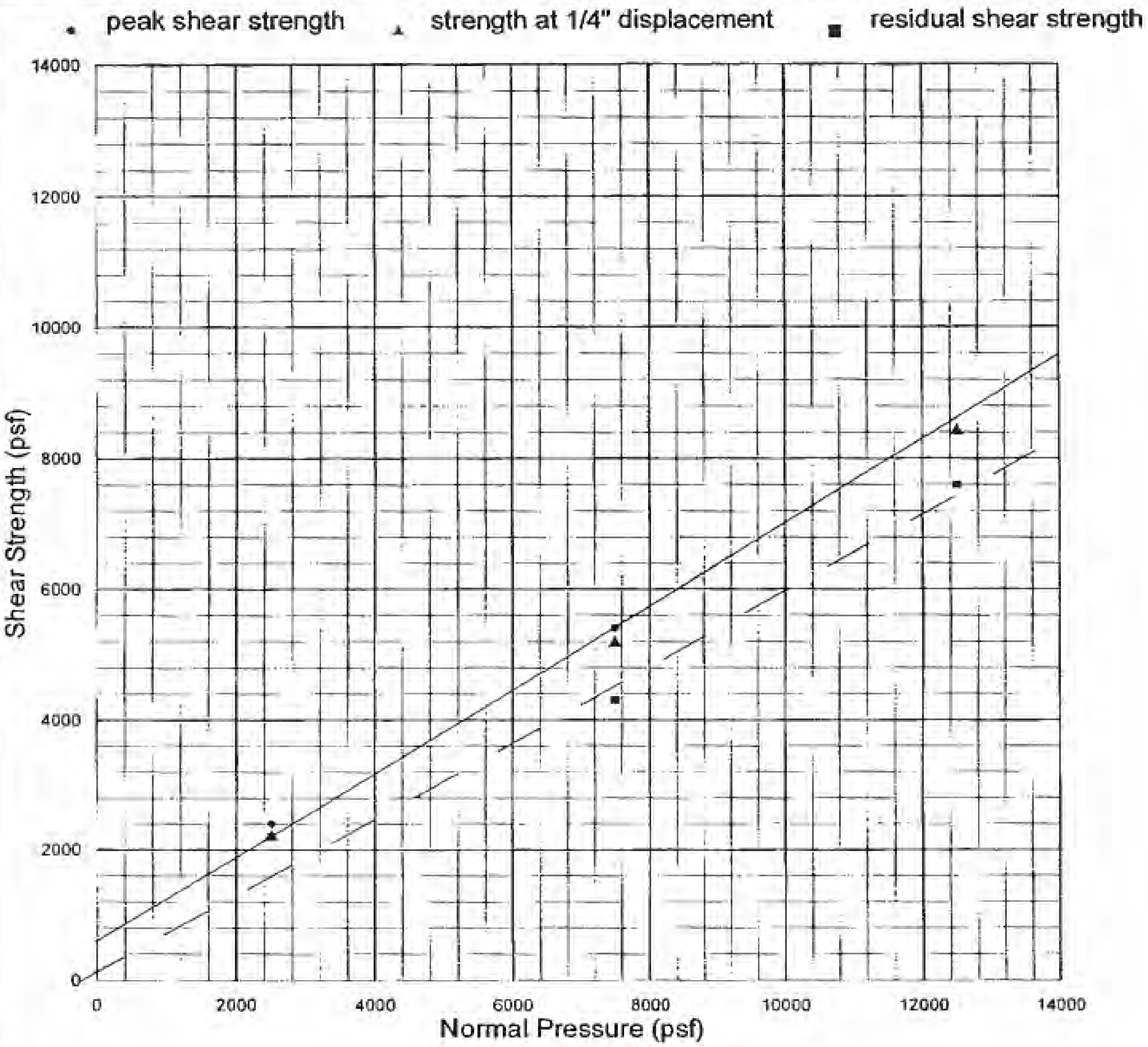


Strain Rate: 0.0042 in. / min.

Sample	Type	Description	Dry Density (pcf)	Initial W.C. (%)
B-4/1 @ 5'	Remolded & Saturated	Silty Clay	110.9	13.8

Normal Pressure (psf)	Peak Strength (psf)	Ultimate Strength (psf)	Residual Strength (psf)
2500	2120 @ 0.1930"	2050	2050
7500	5560 @ 0.2470"	5540	4430
12500	8330 @ 0.2300"	8290	7370
	C = 800 psf	C = 800 psf	C = 800 psf
	ϕ = 31 deg.	ϕ = 31 deg.	ϕ = 27 deg.

DIRECT SHEAR TEST - ASTM D-3080



Strain Rate: 0.0042 in. / min.

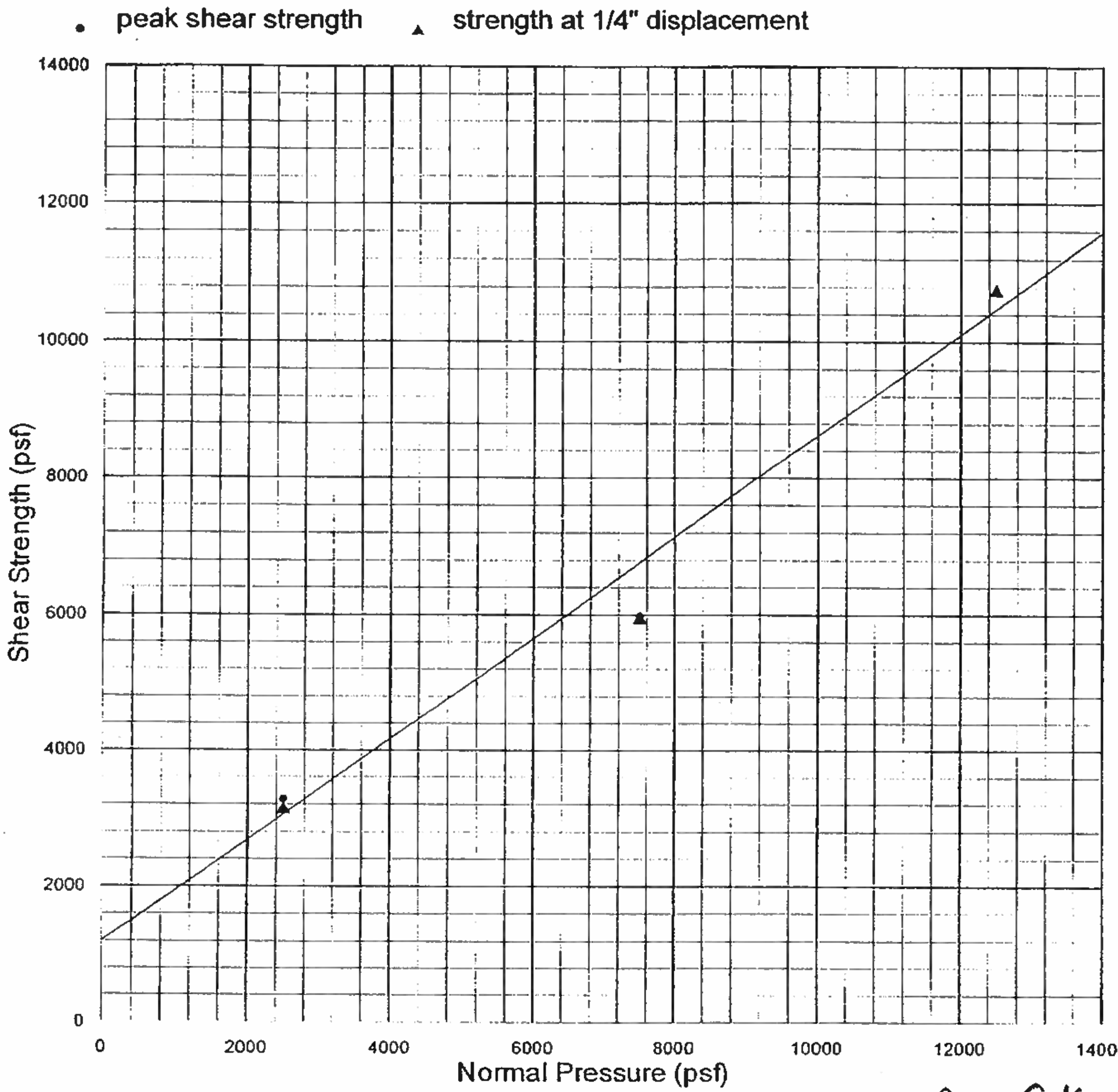
Sample	Type	Description	Dry Density (pcf)	Initial W.C. (%)
B-2/1 @ 10'	Remolded & Saturated	Silty Clay	103.5	22.0

Normal Pressure (psf)	Peak Strength (psf)	Ultimate Strength (psf)	Residual Strength (psf)
2500	2280 @ 0.1585"	2150	1860
7500	5470 @ 0.1550"	5290	5290
12500	9240 @ 0.2260"	9240	8280
	C = 600 psf	C = 600 psf	C = 150 psf
	ϕ = 33 deg.	ϕ = 33 deg.	ϕ = 30 deg.

Job No. 2001-082

DIRECT SHEAR TEST - ASTM D-3080

SouthCoast LF - Mendocino County



Qcol-Colluvium ?

Strain Rate: 0.0042 in. / min.

Sample	Type	Description	Dry Density (pcf)	Initial W.C. (%)
B-2/2	Undisturbed & Saturated	Clayey Silt w. Rock & Siltstone Frgmts	101.9	18.6

Normal Pressure (psf)	Peak Strength (psf)	Ultimate Strength (psf)
2500	3280 @ 0.1285"	3120
7500	5980 @ 0.2235"	5930
12500	10730 @ 0.2130"	10730
	C = 1200 psf	C = 1200 psf
	ϕ = 37 deg.	ϕ = 37 deg.

SEISMICITY

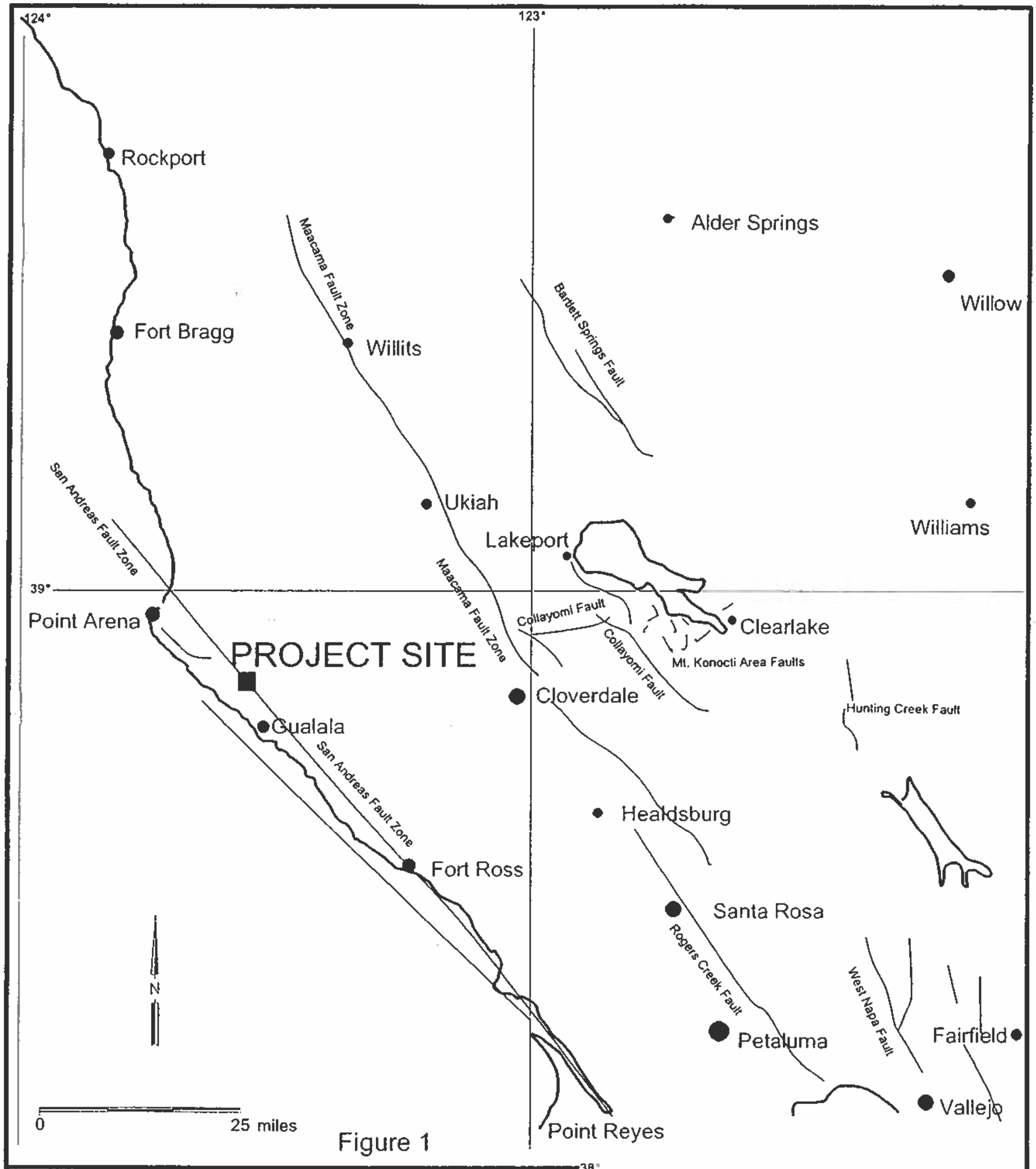


Figure 1

SOUTH COAST LANDFILL
GUALALA, CALIFORNIA

REGIONAL FAULT MAP

GeoLogic Associates
Geologists, Hydrogeologists, and Engineers

DRAFT	DATE	JOB NO.
JGF	1-03	2002-072

Reference: USGS, 2003, Earthquake Hazards Program - Northern California

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                    **********************
                    *                    *
                    *    E Q F A U L T   *
                    *                    *
                    *    Version 3.00    *
                    *                    *
                    **********************

                  DETERMINISTIC ESTIMATION OF
            PEAK ACCELERATION FROM DIGITIZED FAULTS


JOB NUMBER: 2002-072
                                                     DATE: 02-20-2003

JOB NAME: Test Run

CALCULATION NAME: Test Run Analysis

FAULT-DATA-FILE NAME: CDMGFLTE.DAT

SITE COORDINATES:
   SITE LATITUDE:  38.8312
   SITE LONGITUDE:  123.5424

SEARCH RADIUS:   100  mi

ATTENUATION RELATION: 22) Abrahamson & Silva (1995b/1997) Horiz.- Rock
   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:  CDMGFLTE.DAT

MINIMUM DEPTH VALUE (km):  0.0
```

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                         ---------------
                         EQFAULT SUMMARY
                         ---------------




                  -----------------------------
                  DETERMINISTIC SITE PARAMETERS
                  -----------------------------

Page  1
-------------------------------------------------------------------------------
                                |              |ESTIMATED MAX. EARTHQUAKE EVENT
                                | APPROXIMATE  |-------------------------------
          ABBREVIATED           |   DISTANCE   | MAXIMUM  |   PEAK   |EST. SITE
          FAULT  NAME           |   mi    (km) |EARTHQUAKE|   SITE   |INTENSITY
                                |              | MAG.(Mw) | ACCEL. g |MOD.MERC.
================================|==============|==========|==========|=========
SAN ANDREAS (1906)              |   0.0(  0.0) |   7.9    |  0.896   |   XI
SAN ANDREAS (North Coast)       |   0.0(  0.0) |   7.6    |  0.857   |   XI
MAACAMA (Central)               |  27.0( 43.5) |   7.1    |  0.096   |   VII
MAACAMA (South)                 |  29.1( 46.8) |   6.9    |  0.081   |   VII
MAACAMA (North)                 |  37.5( 60.4) |   7.1    |  0.069   |   VI
COLLAYOMI                       |  38.3( 61.6) |   6.5    |  0.048   |   VI
RODGERS CREEK                   |  46.2( 74.4) |   7.0    |  0.053   |   VI
POINT REYES                     |  46.9( 75.4) |   6.8    |  0.060   |   VI
BARTLETT SPRINGS                |  50.3( 80.9) |   7.1    |  0.051   |   VI
HUNTING CREEK - BERRYESSA       |  56.4( 90.7) |   6.9    |  0.040   |   V
ROUND VALLEY                    |  57.2( 92.1) |   6.8    |  0.037   |   V
GREAT VALLEY 2                  |  64.2( 103.3)|   6.4    |  0.033   |   V
GREAT VALLEY 3                  |  65.2( 104.9)|   6.8    |  0.042   |   VI
GREAT VALLEY 1                  |  69.5( 111.9)|   6.7    |  0.037   |   V
WEST NAPA                       |  69.5( 111.9)|   6.5    |  0.025   |   V
GREAT VALLEY 4                  |  76.2( 122.6)|   6.6    |  0.031   |   V
CONCORD - GREEN VALLEY          |  76.9( 123.8)|   6.9    |  0.029   |   V
LAKE MOUNTAIN                   |  79.5( 127.9)|   6.7    |  0.024   |   V
SAN GREGORIO                    |  79.9( 128.6)|   7.3    |  0.037   |   V
GARBERVILLE-BRICELAND           |  80.9( 130.2)|   6.9    |  0.027   |   V
HAYWARD (Total Length)          |  81.4( 131.0)|   7.1    |  0.031   |   V
HAYWARD (North)                 |  81.4( 131.0)|   6.9    |  0.027   |   V
SAN ANDREAS (Peninsula)         |  87.0( 140.0)|   7.1    |  0.029   |   V
GREAT VALLEY 5                  |  92.1( 148.3)|   6.5    |  0.023   |   IV
SIERRA MADRE (San Fernando)     | 100.0( 160.9)|   6.7    |  0.025   |   V
*******************************************************************************

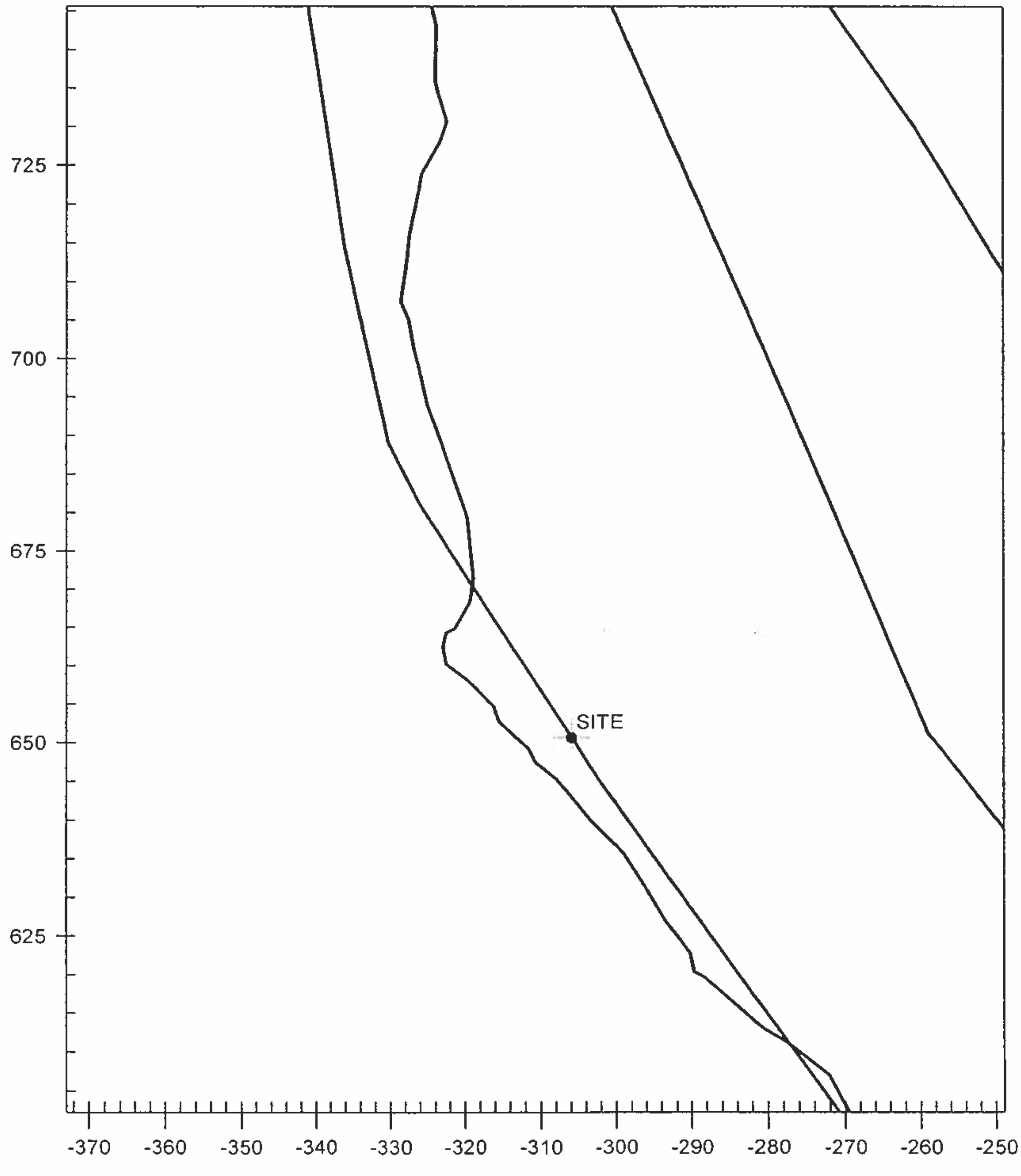


-END OF SEARCH-   25 FAULTS FOUND WITHIN THE SPECIFIED SEARCH RADIUS.

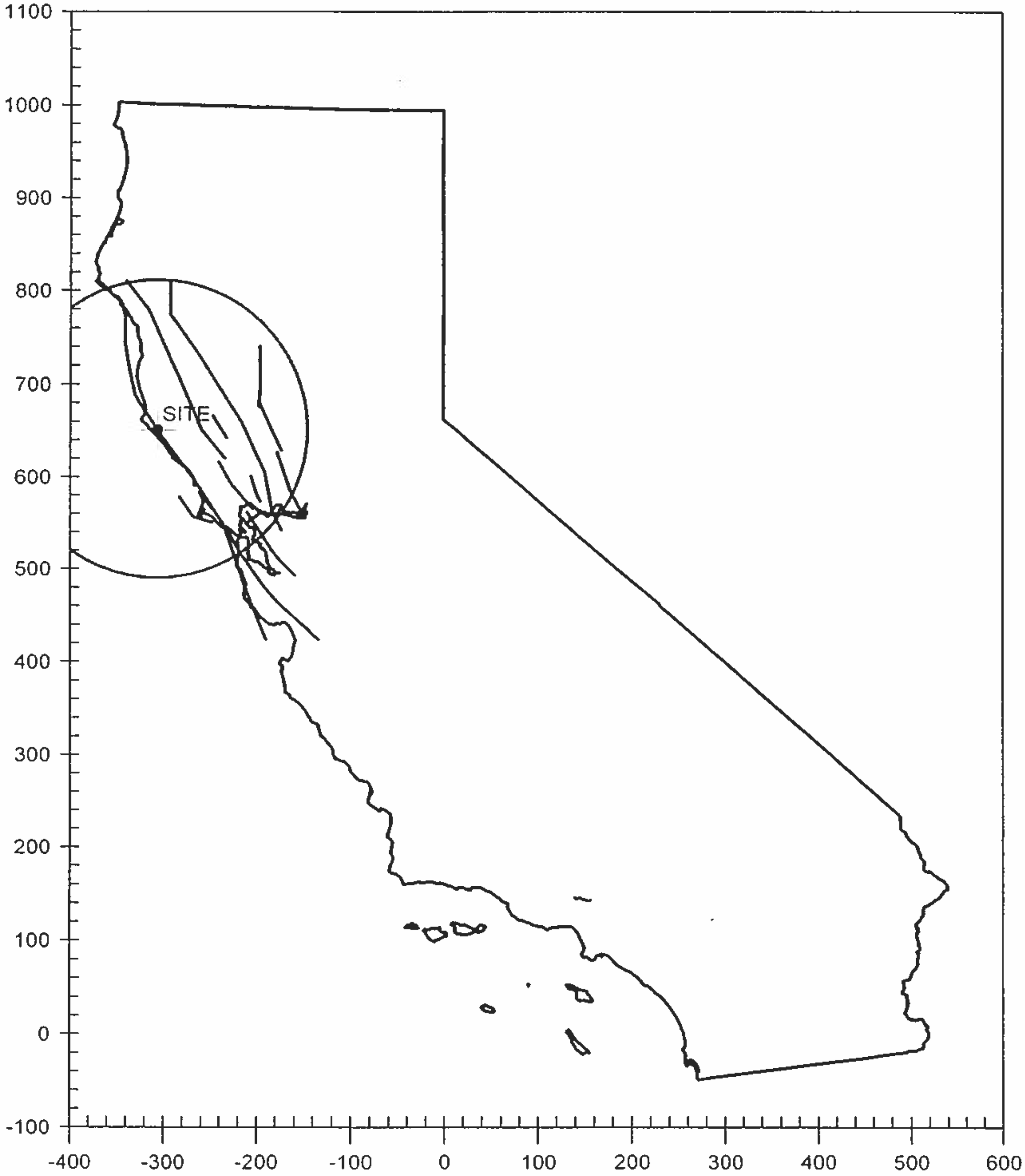
THE SAN ANDREAS (North Coast)        FAULT IS CLOSEST TO THE SITE.
IT IS ABOUT 0.0 MILES (0.0 km) AWAY.

LARGEST MAXIMUM-EARTHQUAKE SITE ACCELERATION: 0.8965 g
```

CALIFORNIA FAULT MAP

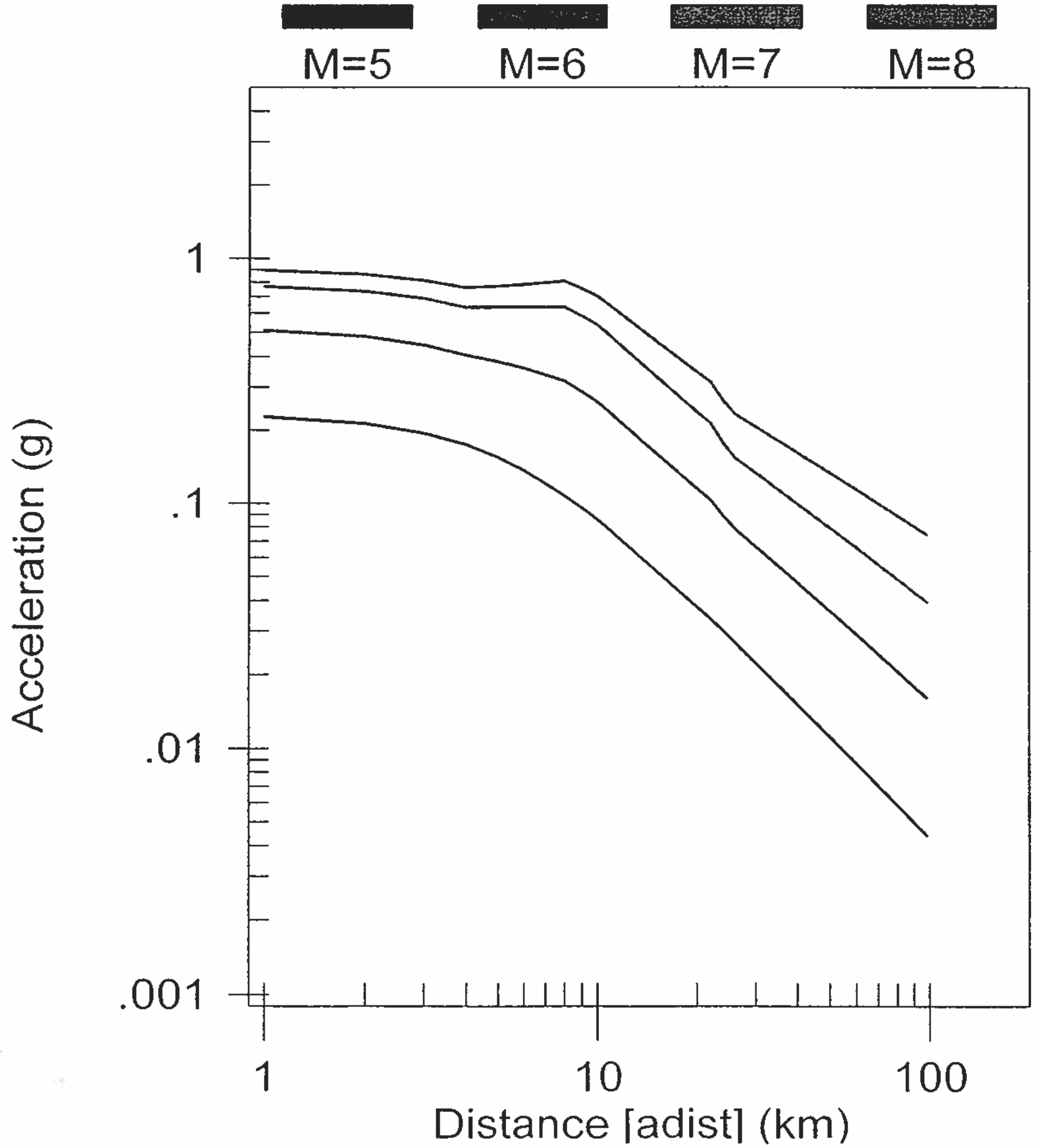


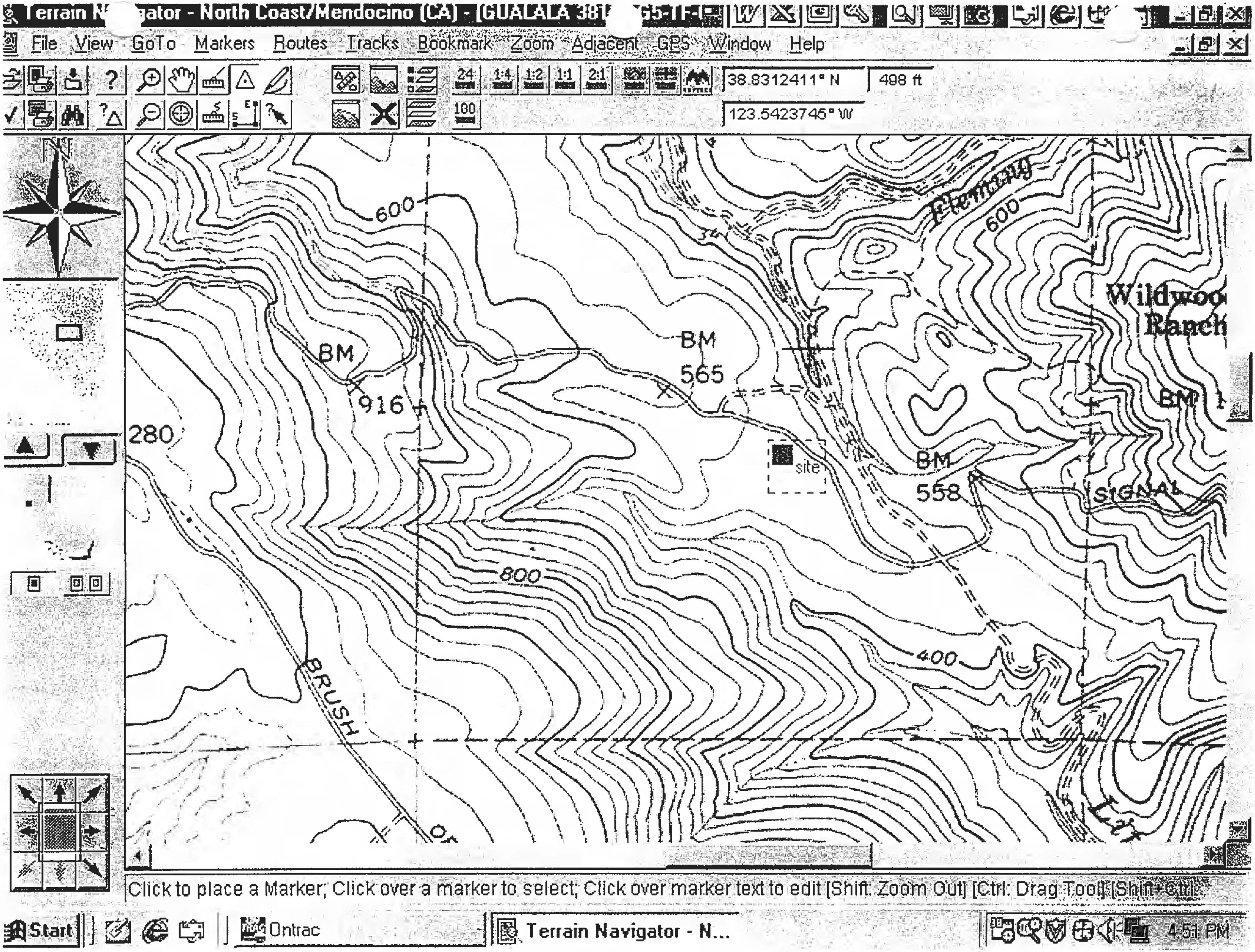
CALIFORNIA FAULT MAP



STRIKE-SLIP FAULTS

22) Abrahamson & Silva (1995b/1997) Horiz.- Rock





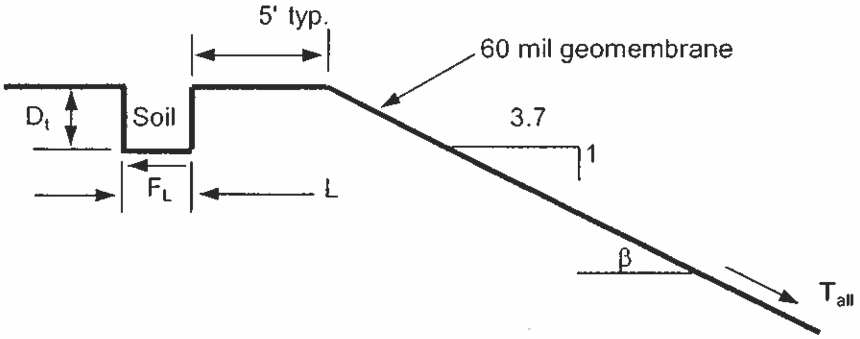
38.8312411° N
123.5423745° W
498 ft

Click to place a Marker; Click over a marker to select; Click over marker text to edit [Shift: Zoom Out] [Ctrl: Drag Tool] [Shift+Ctrl:

ANCHOR TRENCH DESIGN

Anchor Trench Design
South Coast Landfill

Slope Configuration:



Purpose: To calculate the distance L to provide anchorage of the geomembrane up to the allowable tension (T_{all}) of the 60 mil LLDPE

Given:
T_{all}= σ_{all} t
σ_{all}= σ_{yield}/FS= 130psi/2= 65 psi
t= 60 mil= 0.06 inches
d_t= depth of anchor trench = 2 feet
F_L= q tan δ (L) = friction below membrane
q= $D_t \gamma_{soil}$= γ_{soil}= 110 pcf
δ= 14 degrees (min. friction angle between geomembrane & soil)
β= 15.1 degrees (design slope inclination)
H= Height of the slope 45 feet
L_G= Length of the geomembrane 172.5 feet

Assumptions:
-Side slope and 5 foot runout length is neglected in friction calculation (conservative)
-Neglect 12" additional cover on runout length (conservative)
-Frictionless pulley at grade breaks
-Neglect strength of other geofabrics (conservative)

Solve:

T_{all}= cos β = F_L
σ_{all} t cos β = q tan δ L
L= 0.8 feet

For construction, use: **L= 2 feet** **Factor of Safety = 2.4**

Check for pull-out from weight of membrane:

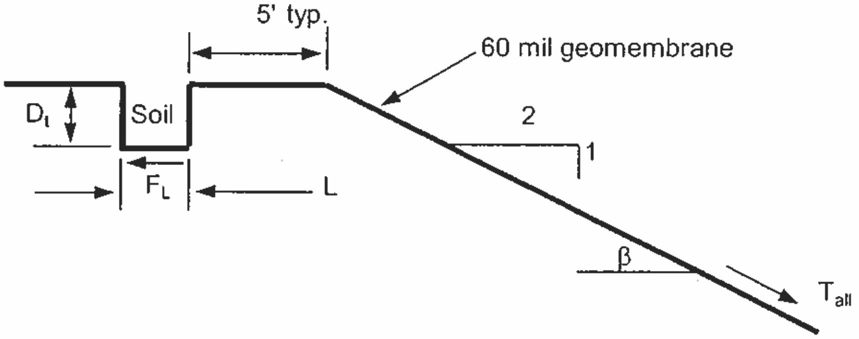
Weight = $\delta_{geomembrane}$ x thickness x length (neglect friction from slope)
Friction = F_L = q tan δ L

Factor of Safety = Friction/Weight= 2.2

Reference: Koerner, 1990, Designing with Geotextiles, Prentice Hall

Anchor Trench Design
South Coast Landfill

Slope Configuration:



Purpose: To calculate the distance L to provide anchorage of the geomembrane up to the allowable tension (T_{all}) of the 60 mil LLDPE

Given:

$T_{all} = \sigma_{all}\, t$

$\sigma_{all} = \sigma_{yield}/FS =$ 130psi/2= 65 psi

$t =$ 60 mil= 0.06 inches

$d_t =$ depth of anchor trench = 2 feet

$F_L = q \tan\delta\,(L) =$ friction below membrane

$q = D_t\, \gamma_{soil} =$ $\gamma_{soil} =$ 110 pcf

$\delta =$ 14 degrees (min. friction angle between geomembrane & soil)

$\beta =$ 26.6 degrees (design slope inclination)

H= Height of the slope 45 feet

$L_G =$ Length of the geomembrane 100.6 feet

Assumptions:

-Side slope and 5 foot runout length is neglected in friction calculation (conservative)

-Neglect 12" additional cover on runout length (conservative)

-Frictionless pulley at grade breaks

-Neglect strength of other geofabrics (conservative)

Solve:

$T_{all} = \cos\beta = F_L$

$\sigma_{all}\, t \cos\beta = q \tan\delta\, L$

L= 0.8 feet

For construction, use: **L= 2 feet** **Factor of Safety = 2.6**

Check for pull-out from weight of membrane:

Weight = $\delta_{geomembrane}$ x thickness x length (neglect friction from slope)

Friction = $F_L = q \tan\delta\, L$

Factor of Safety = Friction/Weight= 3.7

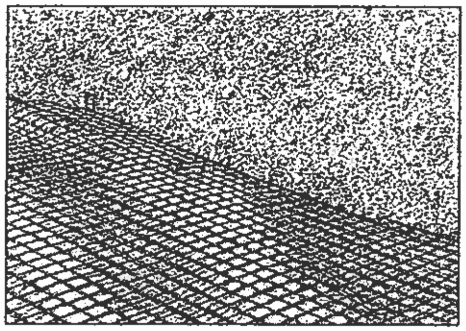
Reference: Koerner, 1990, Designing with Geotextiles, Prentice Hall

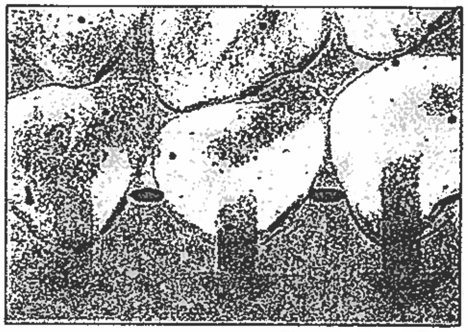
GEOGRID REQUIREMENTS

General Product Description

Fortrac geogrids are tough, rugged woven polyester yarns coated with extremely durable polymer coatings.

The manufacture of Fortrac begins with extremely strong fibers made from a proprietary blend of high-tenacity polyester. These stretch-resistant, creep-resistant fibers, chemically similar to those used in the manufacture of high-performance race car tires, are coated with polyvinyl chloride for protection against mechanical damage during installation, and as a shield against UV rays, damaging micro-organisms and corrosive elements found naturally in soil.





The combination of fiber composition and proprietary manufacturing procedures result in geogrids with extraordinarily high modulus-of-elasticity performance, providing high tensile strength, and low strains to minimize time-dependent creep.

When the grids are in place, the aggregate partially penetrates the grid, and effectively locks into the earthen base below, providing an exceptionally strong and stable interface.

Geogrids are more effective and far less expensive than most geo-engineering alternatives. The inherent flexibility of the coated synthetic yarns protects against stress cracks and provides a firm bond between the subsoil and the aggregate.

Physical Properties of Fortrac Geogrids

	Standard Mesh							Open Mesh		
FORTRAC ITEM	20/9-20	35/20-20	55/30-20	80/30-20	110/30-20	150/30-30	200/30-30	20/20-35	35/35-35	35/35-50
WEIGHT, oz/sq. yard (ASTM D-3776)	5.0	7.0	9.0	14.0	16.0	19.0	21.0	6.0	8.0	8.0
APERTURE SIZE, inch	.8 X .8	.8 X .8	.8 X .8	.8 X .8	.8 X .8	1.2 X 1.2	1.2 X 1.2	1.4 X 1.4	1.4 X 1.4	2.0 X 2.0
OPEN AREA %	75+	75+	70+	65+	60+	60+	60+	85+	85+	85+
WIDE WIDTH TENSILE STRENGTH, lb/ft (ASTM D-4595)										
@ Ultimate, mach. direction	1500	2600	3700	5380	7400	10100	13500	1350	2500	2500
@ Ultimate, cross direction	615	1350	2020	2020	2020	2020	2020	1350	2500	2500
@ 5% strain, mach direction	600	1100	1500	2200	3000	4100	5500	625	900	900
ELONGATION AT BREAK, % (ASTM D-4595)	12.0	12.0	11.0	10.0	10.0	10.0	10.0	12.0	12.0	12.0
LONG TERM ALLOWABLE DESIGN LOAD* MD, lb/ft 100+ Years (Creep FS-1.67)										
• Sand, Silt and Clay	762	1322	1936	2815	3872	5285	7065	686	1270	1270
• .75" minus well graded gravel	728	1261	1881	2735	3761	5135	6862	655	1213	1213
• 2.5" crushed stone and gravel	572	991	1520	2393	3761	5135	6862	514	953	953
ROLL SIZE	12.14 X 656	12.14 X 656	12.14 X 328	12.14 X 328	12.14 X 328	16.41 X 328	16.41 X 328	12.14 X 656	16.41 X 328	12.14 X 328
SQUARE YARDS/ROLL	885	885	443	443	443	598	598	885	598	443

*100 YEAR LONG TERM ALLOWABLE DESIGN LOAD PER GRI GG4 (B)

$$= T_{ALLOWABLE} = \frac{T_{ULTIMATE}}{FS_{ID} \cdot FS_{CR} \cdot FS_{CD} \cdot FS_{BD} \cdot FS_{INT}}$$

Note: Specific Factors of Safety data for TALLOWABLE design calculations are available from Huesker inc. Refer to back page for telephone number and address.

REVISED FINAL CLOSURE ANALYSIS
(GLA, NOVEMBER 2012)

Geo-Logic
ASSOCIATES

November 13, 2012
2012-0007

Mr. John R. Boucher
Vice President
SWT Engineering
800-C South Rochester Avenue
Ontario, CA 91761

REVISED FINAL COVER ANALYSIS
SOUTH COAST LANDFILL
MENDOCINO COUNTY, CALIFORNIA

We are pleased to provide the following final cover evaluation report which supersedes the referenced cover evaluation presented in GLA (2003). This report and attachments present the results of geotechnical analyses that were completed by Geo-Logic Associates (GLA) to evaluate an appropriate final cover configuration for the County of Mendocino Solid Waste Division's (SWD's) South Coast Landfill (SCLF). Since the landfill is no longer active, it must be closed in accordance with requirements listed in California Code of Regulations (CCR) Title 27, Section 21750 f(5) for Class III landfills.

1.0 FACILITY BACKGROUND & LOCATION

The SCLF is an approximately 6-acre Class III landfill that is located on an approximately 47.65-acre parcel off Fish Rock Road in a remote portion of southwestern Mendocino County, California (Figure 1). Refuse disposal operations were conducted at the site from 1970 through November 2001 using cut and cover techniques. Except for its steep southwestern border, the topography of the landfill parcel is relatively flat to gently sloping.

As determined in a field study completed by EMCON Associates (1998a), the limits of refuse along the southwestern perimeter of the facility meet the top of an approximately 100-foot high 2:1 (horizontal to vertical) native slope that borders the headwaters of the Little North Fork of the Gualala River (hereafter Little North Fork). The area surrounding the site is vegetated with a moderately dense growth of coniferous trees.

2.0 SEISMICITY

In accordance with 27 CCR Section 20240, a seismic hazard review was completed to evaluate the earthquake parameters that could affect slope stability conditions at the site in the event of the maximum probable earthquake (MPE) or the largest recorded (historic) earthquake event, whichever is larger.

Using the computer program EQFAULT (Blake, 2004), the MPE of the north coast segment of the San Andreas fault and the 1906 San Francisco event were estimated to be approximately M_w=7.6 to 7.9 (Appendix A). Assuming a focal mechanism 2 kilometers distant from the site and attenuation relationships by Abrahamson and Silva (1997), the associated peak ground acceleration associated with the MPE and the 1906 San Francisco Earthquake event is approximately 0.9g (Appendix A).

Using the newer attenuation relationships of the Next Generation of Attenuation (NGA) Relationship Project in 2008 (Abrahamson, et.al, 2008, Boore, et.al, 2008, Campbell, et.al, 2008, Chiou, et.al, 2008, and Idriss, 2008), a maximum site acceleration of 0.61g is calculated for a fault distance of 0 km (Appendix A).

Since the newer site attenuation relationships produce a site acceleration less than what was historically used at the site (EMCON, 1998b) the larger acceleration value (0.9g) was used in this analysis. The site design seismic parameters used are summarized below:

Table 1
Maximum Probable Earthquake Design Characteristics

Earthquake Magnitude	M=7.9 on the San Andreas Fault: 2 km from the site
Maximum Site Acceleration	0.9g (for the MPE)
Duration of Significant Shaking, $D_{5\text{-}95}$	33 sec. (Bray et. al., 1998)
Mean Period of Shaking, T_m	0.52 sec. (Bray et. al., 1998)

3.0 LABORATORY STUDIES

Recognizing that the chosen cover will have to be stable under high seismic loads and moderate to heavy precipitation, testing was performed on proposed cover soils, geotextiles, and the LLDPE Supergrip Geomembrane proposed for the site. Laboratory test results are presented in Appendix B. As shown, the interface shear tests were completed in accordance with ASTM Method D6243-98 and D5321-92 with soils compacted to 90% relative compaction at a wetted condition. The analyses addressed the peak and residual (large displacement) interface shear strengths.

Two sandwich tests of the proposed cover system were performed with the Super Gripnet LLDPE membrane, overlain by a non-woven geotextile and either sand or onsite clayey cover soils. The minimum interface friction angle for the system was calculated as 32 degrees and a cohesion of 40psf. This value was used as representative of the minimum interface friction for the cover system in this analysis. An additional sandwich test was completed with concrete sand to evaluate the interface shear strength of the geotextile/Super Gripnet. The results of this test yielded a friction angle of 37 degrees and a cohesion of 35 psf (Appendix B).

The Agru Super Gripnet was also tested for transmissivity to insure adequate drainage. The results of the testing are presented in Appendix B and show an adequate drainage for the amount of collected precipitation.

4.0 COVER SLOPE STABILITY ANALYSES

4.1 Cover Material Strength Properties

From top to bottom, the materials that are planned for use in construction of the facility's final cover include: a 2-foot thick compacted final cover, over a non-woven geotextile, over a LLDPE Agru Super Gripnet geomembrane, over 6 inches of compacted existing foundation layer, over existing refuse (Figure 1).

Based on recent testing and published data, the engineering properties that were assumed for these materials and the interfaces between individual elements are as follows:

Table 2
Landfill Cover Strength Parameters

Material or Interface	Unit Weight (pcf)	Angle Internal Friction (degrees)	Cohesion (psf)
Protective Soil Cover*	120	27	800
Protective Soil Cover/Non-woven geotextile**	--	32	40
Non-woven geotextile/Super Gripnet geomembrane***	--	37	35
Super Gripnet geomembrane and existing cover/foundation layer**	--	>32	>40

* Direct shear results for existing stockpile soils (Appendix B)
** Interface shear results for existing stockpile soils and LLDPE (Appendix B).
*** Interface shear results for concrete sand and LLDPE (Appendix B)

Although not employed in the stability analyses, additional strength is provided in the proposed final cover system by the tensile strength of the geotextile and LLDPE geomembrane components and the cohesion/adhesion between the soil and geotextile layers.

4.2 Static Landfill Cover Stability

CCR Title 27 Section 21090 requires that "operators shall ensure the integrity of final slopes under both static and dynamic conditions to protect public health and safety and prevent damage to post-closure land uses, . . . prevent public contact with leachate, and prevent exposure of waste." In addition, 27 CCR Section 21750 f(5) requires that "the stability analyses shall ensure the integrity of the Unit, including its foundation, final slopes, and containment systems under both static and dynamic conditions throughout the Unit's life, closure period, and post-closure period." For final closure design, the standard of practice with regard to these requirements has typically meant that landfill slopes should have a static factor of safety of approximately 1.5 and dynamic displacement during the MPE that does not impair the integrity of the final cover.

The prescriptive final cover components listed for unlined landfills in 27 CCR Section 21090 include: a two-foot thick foundation layer, a one-foot thick clay barrier layer whose hydraulic conductivity does not exceed 1 x 10^{-6} centimeters per second (cm/sec), and a one-foot thick vegetative soil layer. Considering the high seasonal rainfall totals typical of the area, and in accordance with the engineered alternative configuration allowed by 27 CCR Section 20080(b), from top to bottom, the proposed final cover is a 2-foot thick compacted final cover, over a non-woven geotextile, over a LLDPE Ague Super Gripnet geomembrane, over 6 inches of compacted existing foundation layer, over existing refuse (Figure 1). As detailed below, together, these alternative final cover elements are anticipated to provide enhanced protection against infiltrating rain water, and suitable support to withstand earthquake loads associated with the MPE on the San Andreas fault.

The stability of the proposed final cover system was considered by addressing both the steepest portion of the landfill cover (15-foot high at a slope inclination of 3:1 [eastern portion]) and the highest portion (45-foot high slope at an inclination of 3.6:1 [southeastern portion]). These analyses were performed using the limit equilibrium procedures identified by Kramer (1999) and using the interface shear strength properties of the individual and combined (interface) cover components listed above in Table 2.

As shown on Table 2 above, the lowest interface strength occurs between the cover soils and the non-woven geotextile. Accordingly, this interface, as well as the stronger interface between the non-woven geotextile and the Super Gripnet, were analyzed using the proposed final cover geometry. As shown in Table 3, the lowest static factor of safety for the proposed final cover is 1.95 (for the 15-foot high slope at the upper deck of the landfill at an inclination of 3:1 (H:V)). The minimum static factor of safety for the less steep condition (40-foot high slope at 3.6:1) is 2.28 (Table 4).

4.3 Dynamic Stability of Proposed Final Cover

The potential dynamic displacement of the final cover was evaluated using the procedure described by Bray et. al, (1998) and taking into account the peak horizontal ground acceleration associated with the MPE on the San Andreas fault (0.9g) and the calculated yield acceleration for the proposed final cover configuration. As shown in Tables 5 and 6, the dynamic displacement of the final cover configuration is calculated at a maximum of about 9 to 12 inches. Considering the elongation properties of the LLDPE, the integrity of the final cover barrier layer is maintained and such displacement is considered tolerable (Seed and Bonaparte, 1992).

Since the landfill is positioned over the San Andreas fault, in the event of an large earthquake whose focal mechanism is close to the site, ground rupture could occur. The probability of such an occurrence is regarded as considerably smaller than the possibility of an MPE event, and would be largely mitigated by the elastic properties of the refuse and cover materials. While such an event could still result in distress to the final cover, interim use of reinforced visqueen to prevent rainwater infiltration, and standard soil and geosynthetic cover repair operations could be employed to mitigate this condition.

5.0 CONCLUSIONS AND RECOMMENDATIONS

The analyses completed indicate that stability of the proposed final cover configuration is stable under both static and seismic loads. The maximum calculated seismic displacement of the final cover (9 to 12 inches) is considered tolerable and could be accommodated by the geomembrane barrier layer of the final cover.

6.0 CLOSURE

This report is based on the project as described and the limited geotechnical data obtained in this and earlier studies of the South Coast Landfill. Our firm should be notified of any pertinent change in the project plans or if conditions are found that differ from those described in this report, since this may require a revaluation of the conclusions and recommendations presented herein.

This report has not been prepared for use by parties or projects other than those named or described above. It may not contain sufficient information for other parties or other purposes. This report has been prepared in accordance with generally accepted hydrogeologic and geotechnical practices and makes no other warranties, either express or implied, as to the professional advise or data included in it.

Geo**L**ogic **A**ssociates

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Exp. 12-31-13
GEOTECHNICAL
STATE OF CALIFORNIA

Attachments: Table 3 – Infinite Slope Analysis – 15-foot high Slope at 3:1
Table 4 – Infinite Slope Analysis – 40-foot high Slope at 3.6:1
Table 5 – Seismic Induced Permanent Displacement Evaluation-Final Cover
Table 6 – Seismic Induced Permanent Displacement Evaluation-Final Cover
Figure 1 – Alternate Slope Cover Section
Appendix A – Seismic Analysis
Appendix B - Laboratory test results.

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Table 3
Infinite Slope Analysis: 15-Foot High Slope at 3:1 (H:V)
Final Cover Components
Southcoast Landfill, Mendocino County, California

Shear Strength Parameters				Slope Parameters (1)					Toe Parameters			Computed Forces / Unit Width				Geosynthetics			FACTOR OF SAFETY
Soil			Interface	Slope	Slope	Soil	Slope Cover		Soil Thickness		Wedge	Driving	Resisting			Ultimate	Assigned	Allowable	
Sat. Unit Wt.	Int. Friction Angle	Passive Crit. Failure Angle	Friction Angle	Ratio (H:1V)	Inclin.	Thickness	Vert. Ht.	Slope Length	Total	Ave.	Fail. Length	Soil Load	Toe Soil Wedge	Geosynthetic		Tensile Strength (2)	Safety Factor	Tensile Strength(3)	
														Interface Friction	Tensile Demand				
γ_s	ϕ	$\phi_{CR = 45 - \phi/2}$	δ	Z	α	d_s	H_v	$L_s = H_v / \text{SIN}\ \alpha$	d_T	$d_a = (2/3)\ d_T$	$L_w = d_T / \text{SIN}\phi_{CR}$	$D_s = \gamma_s d_s L_S \text{SIN}\alpha$	$R_w = \gamma_s d_a L_w \text{COS}\phi_{CR} \text{Tan}\phi$	$R_F = \gamma_s d_s L_s \text{COS}\alpha \text{Tan}\delta$	For FS 1.5; $R_T = 1.5{*}D_s - R_w - R_f$	T_u	Sfa	$T_A = T_u / SF_A$	$FS = (R_w + R_f + T_A) / D_s$
(pcf)	(deg)	(deg)	(deg)	(ft)	(deg)	(ft)	(ft)	(ft)	(ft)	(ft)	(ft)	(#/ft)	(#/ft)	(#/ft)	(#/ft)	(#/ft)		(#/ft)	
15-foot high slope at 3:1																			
Veg Cover on NW Geotextile																			
130	32	29	32	3	18.43	2	15	47.5	2	1	4.13	3900	293.1	7310.3	-1753.4	0	3	0	1.95
NW Geotextile on Super Gripnet																			
125	37	26.5	37	3	18.43	2	15	47.5	2	1	4.48	3750	377.8	8476.3	-3229.1	0	3	0	2.36

(1)- Assumed configuration

2' (vertical) of Vegetative cover
12 ounce NW Geotextile
AGRU SuperGrip LLDPE
Foundation Layer

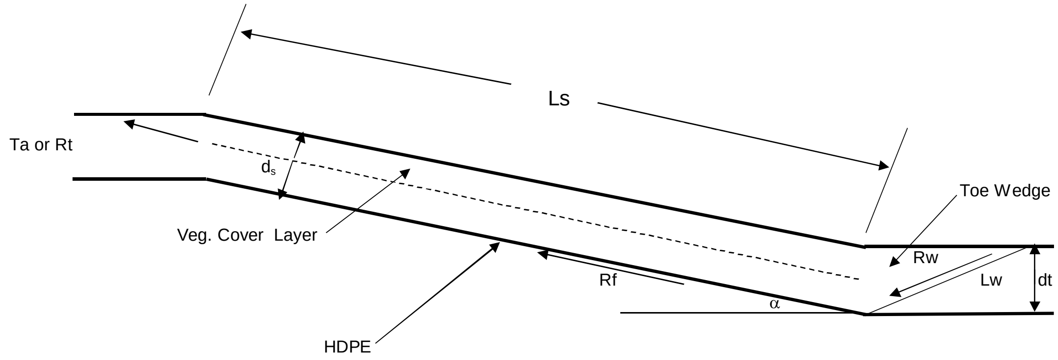


Table 3 (Cont.)
Infinite Slope Analysis: 15-Foot High Slope at 3:1 (H:V)
Final Cover Components
Southcoast Landfill, Mendocino County, California

Soil Weight γ	Interface Friction Angle ϕ	Slope Inclination α	Geogrid/ Geonet Tensile Strength, T_u* lbs/ft	Yield Acceleration K_y*
Veg Cover on NW Geotextile				
130	32	18.43	0	0.24
thickness 2				
NW Geotextile on Super Gripnet				
125	37	18.43	0	0.34
Thickness 2				

Veg Cover on NW Geotextile

Ls 47.46

K= 0.24 g

Geogrid Tensile strength 0 lbs/ft

FS= 1.00

NW Geotextile on Super Gripnet

Ls 47.46

K= 0.335 g

Geogrid Tensile strength 0 lbs/ft

FS= 1.00

$$FS^{**} = \frac{((\gamma_s d_s L_s \cos\alpha) - (K_y \gamma_s \delta_s L_\sigma \sin\alpha))(\tan\delta) + T_u}{(\gamma_s \delta_s L_s \sin\alpha + K_y \gamma_s \delta_s l_s \cos\alpha)}$$

*Solve for T_u and K_y when FS=1.0

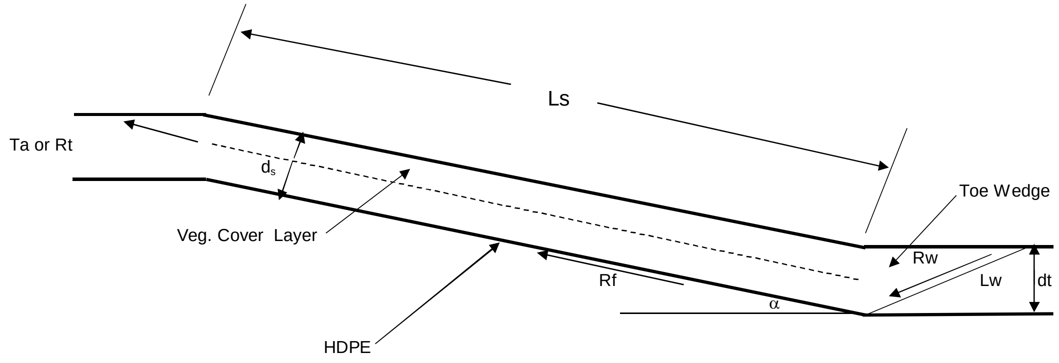
**Equation adapted from Kramer, 1996, Geotechnical Earthquake Engineering, pg 434

Table 4
Infinite Slope Analysis: 40-Foot High Slope at 3.6:1 (H:V)
Final Cover Components
Southcoast Landfill, Mendocino County, California

Shear Strength Parameters				Slope Parameters (1)					Toe Parameters			Computed Forces / Unit Width				Geosynthetics			FACTOR OF SAFETY
Soil			Interface Friction Angle	Slope Ratio (H:1V)	Slope Inclin.	Soil Thickness	Slope Cover		Soil Thickness		Wedge Fail. Length	Driving	Resisting			Ultimate Tensile Strength (2)	Assigned Safety Factor	Allowable Tensile Strength(3)	
Sat. Unit Wt.	Int. Friction Angle	Passive Crit. Failure Angle					Vert. Ht.	Slope Length	Total	Ave.		Soil Load	Toe Soil Wedge	Geosynthetic					
														Interface Friction	Tensile Demand				
γ_s	ϕ	$\phi_{CR = 45 - \phi/2}$	δ	Z	α	d_s	H_v	$L_s = H_v / \text{SIN}\ \alpha$	d_T	$d_a = (2/3)\ d_T$	$L_w = d_T / \text{SIN}\phi_{CR}$	$D_s = \gamma_s d_s L_S \text{SIN}\alpha$	$R_w = \gamma_s d_a L_w \text{COS}\phi_{CR} \text{Tan}\phi$	$R_F = \gamma_s d_s L_s \text{COS}\alpha \text{Tan}\delta$	For FS 1.5; $R_T = 1.5{*}D_s - R_w - R_f$	T_u	Sfa	$T_A = T_u / SF_A$	$FS = (R_w + R_f + T_A) / D_s$
(pcf)	(deg)	(deg)	(deg)	(ft)	(deg)	(ft)	(ft)	(ft)	(ft)	(ft)	(ft)	(#/ft)	(#/ft)	(#/ft)	(#/ft)	(#/ft)		(#/ft)	
40-foot high 3.6 slope																			
Veg Cover on NW Geotextile																			
130	32	29	32	3.6	15.52	2	40	149.5	2	1	4.13	10400	293.1	23392.8	-8085.9	0	3	0	2.28
NW Geotextile on Super Gripnet																			
125	37	26.5	37	3.6	15.52	2	40	149.5	2	1	4.48	10000	377.8	27123.9	-12501.7	0	3	0	2.75

(1)- Assumed configuration

2' of Vegetative cover
12 ounce NW Geotextile
AGRU SuperGrip Net LLDPE
Foundation Layer



C:\2001-082\Copy of South Coast LF Covercalc_Agru Supergrip Lab Data on NW Geotextile with Clay cover_3.6to1.xls\10/30/2012

Table 4 (Cont.)
Infinite Slope Analysis: 40-Foot High Slope at 3.6:1 (H:V)
Final Cover Components
Southcoast Landfill, Mendocino County, California

Soil Weight γ	Interface Friction Angle ϕ	Slope Inclination α	Geogrid/ Geonet Tensile Strength, T_u* lbs/ft	Yield Acceleration K_y*
Veg Cover on NW Geotextile				
130	32	15.52	0	0.30
thickness 2				
NW Geotextile on Super Gripnet				
125	37	15.52	0	0.40
Thickness 2				

Veg Cover on NW Geotextile

Ls 149.53

K= 0.295 g

Geogrid Tensile strength 0 lbs/ft

FS= 1.00

NW Geotextile on Super Gripnet

Ls 149.53

K= 0.395 g

Geogrid Tensile strength 0 lbs/ft

FS= 1.00

$$FS^{**} = \frac{((\gamma_s d_s L_s \cos\alpha) - (K_y \gamma_s \delta_s L_\sigma \sin\alpha))(\tan\delta) + T_u}{(\gamma_s \delta_s L_s \sin\alpha + K_y \gamma_s \delta_s l_s \cos\alpha)}$$

*Solve for T_u and K_y when FS=1.0

**Equation adapted from Kramer, 1996, Geotechnical Earthquake Engineering, pg 434

TABLE 5

SEISMIC INDUCED PERMANENT DISPLACEMENT EVALUATION-FINAL COVER
SOUTHCOAST LANDFILL, MENDOCINO COUNTY, CALIFORNIA
15-foot high slope at 3:1 (H:V)

PROCEDURE/REFERENCES USED: Bray and Rathje, 1998; Bray, Rathje, Augello, and Merry, 1998; and EMCON, 1998

INPUT DATA:

SITE CONDITION: Rock/weathered rock near-surface.
EFFECTIVE MSW HEIGHT (H): 25 feet
AVERAGE SHEAR WAVE VELOCITY FOR MSW: Vs = 581 ft/sec (Bray and Rathje, 1998, Figure 2)
EARTHQUAKE MAGNITUDE FOR MPE: M=8.0 @0-2km (San Andreas)
MAXIMUM HORIZONTAL SITE ACCELERATION, MHA_{rock}: 0.9g (EMCON, 1998)
SIGNIFICANT DURATION – $D_{5\text{-}95}$ = 33 seconds (Bray, et. al, 1998, Figure 2c and EMCON, 1998, pg. 3-1).
YIELD ACCELERATION, K_y= 0.24 (Table 3)

CALCULATIONS:

MEAN PERIOD OF SHAKING FOR MPE: T_m= 0.52 seconds (Bray. et. al., 1998, Figure 2b)

PREDOMINANT PERIOD OF MSW: $T_s = 4H/V_s = T_S$ (@ H=25' = 0.17 sec

RATIO $T_s/T_m = 0.33$

RATIO MHA_{top}/MHA_{rock}(NRF) = 1.5 (Bray and Rathje, 1998, Fig. 8b, rock site), top of MSW
NRF for MHA_{rock} @0.9g = 0.75 (Bray and Rathje, 1998, Figure 6b)

Where: MHA_{top} = Maximum Horizontal Equivalent Acceleration at top of MSW
MHA_{rock} = Maximum Horizontal Site Acceleration in rock

Thus, $MHA_{top}/(MHA_{rock} \times NRF) = 1.5$, So, $MHA_{top} = (1.5)(0.75)(0.9) = K_{max}= 1$

$MHEA_{cover} = 1.25\ MHA_{top} = 1. \times 1.25 = 1.25g = K_{max}$

$K_y/K_{max} = 0.24/1.25 = 0.192$

From Bray and Rathje, 1998, Figure 13, calculate displacement;

CONCLUSION:

For $K_y/K_{max} = 0.192$; U = 30 cm = **12 inches (for 50% probability of exceedance)**

TABLE 6

SEISMIC INDUCED PERMANENT DISPLACEMENT EVALUATION-FINAL COVER
SOUTHCOAST LANDFILL, MENDOCINO COUNTY, CALIFORNIA
40-foot high slope at 3.6:1 (H:V)

PROCEDURE/REFERENCES USED: Bray and Rathje, 1998; Bray, Rathje, Augello, and Merry, 1998; and EMCON, 1998

INPUT DATA:

SITE CONDITION: Rock/weathered rock near-surface.
EFFECTIVE MSW HEIGHT (H): 40 feet
AVERAGE SHEAR WAVE VELOCITY FOR MSW: Vs = 600 ft/sec (Bray and Rathje, 1998, Figure 2)
EARTHQUAKE MAGNITUDE FOR MPE: M=8.0 @2km (San Andreas)
MAXIMUM HORIZONTAL SITE ACCELERATION, MHA_{rock}: 0.9g (EMCON, 1998)
SIGNIFICANT DURATION – $D_{5\text{-}95}$ = 33 seconds (Bray, et. al, 1998, Figure 2c and EMCON, 1998, pg. 3-1).
YIELD ACCELERATION, K_y= 0.295 (Table 4)

CALCULATIONS:

MEAN PERIOD OF SHAKING FOR MPE: T_m= 0.52 seconds (Bray. et. al., 1998, Figure 2b)

PREDOMINANT PERIOD OF MSW: $T_s = 4H/V_s = T_S$ (@ H=40' = 0.27 sec

RATIO T_s/T_m = 0.52

RATIO MHA_{top}/MHA_{rock}(NRF) = 1.4 (Bray and Rathje, 1998, Fig. 8b, rock site), top of MSW
NRF for MHA_{rock} @0.9g = 0.74 (Bray and Rathje, 1998, Figure 6b)

Where: MHA_{top} = Maximum Horizontal Equivalent Acceleration at top of MSW
MHA_{rock} = Maximum Horizontal Site Acceleration in rock

Thus, $MHA_{top}/(MHA_{rock} \times NRF) = 1.4$, So, $MHA_{top} = (1.4)(0.74)(0.9) = K_{max} = 0.93$

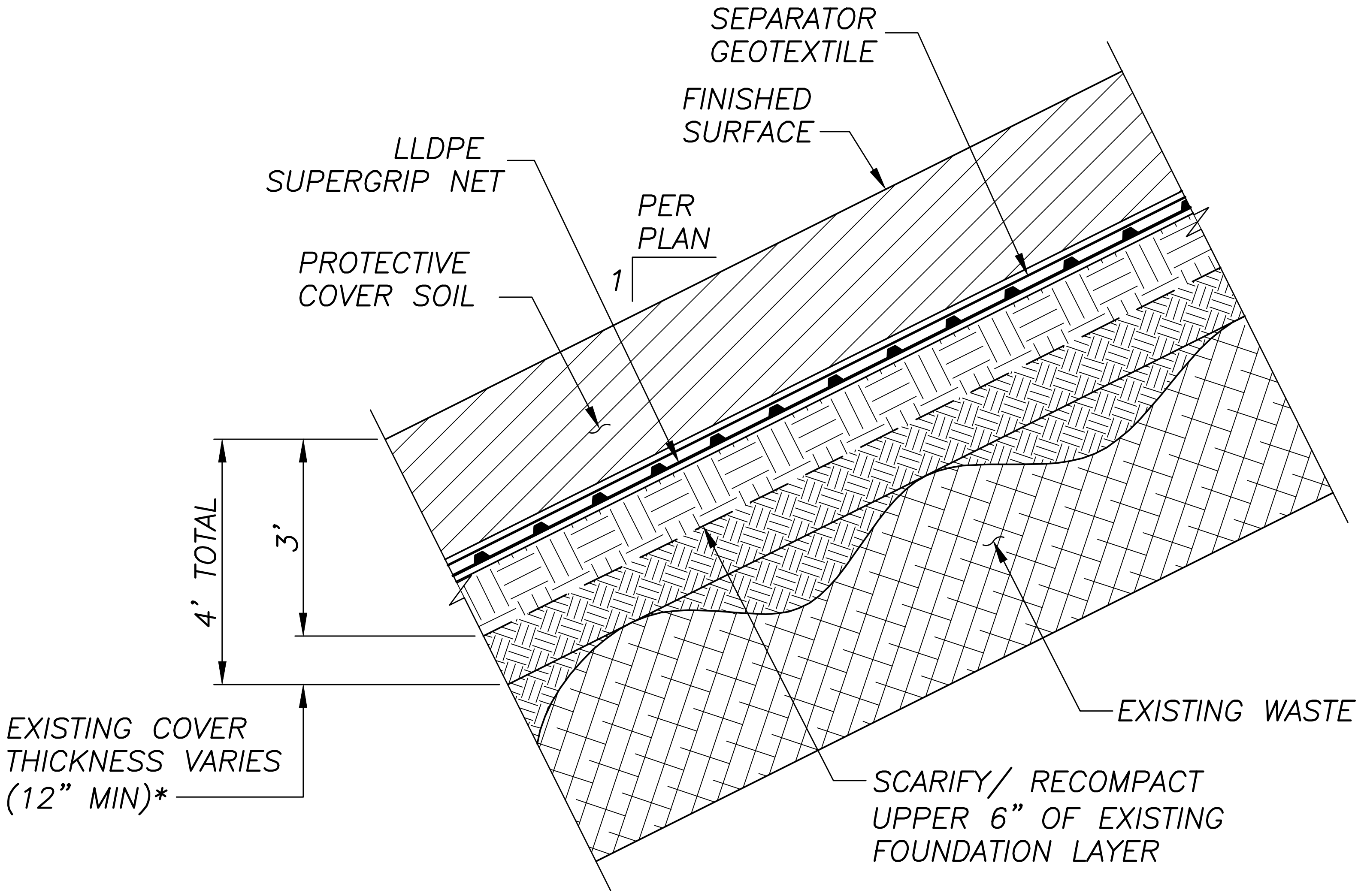
$MHEA_{cover} = 1.25\ MHA_{top} = 0.93 \times 1.25 = 1.16g = K_{max}$

$K_y/K_{max} = 0.295/1.16 = 0.25$

From Bray and Rathje, 1998, Figure 13, calculate displacement;

CONCLUSION:

For $K_y/K_{max} = 0.25$; U = 22 cm = **9 inches (for 50% probability of exceedance)**



PREPARED BY:

SWT Civil & Environmental **Engineering**

800–C SOUTH ROCHESTER AVENUE
ONTARIO, CALIFORNIA 91761

MENDOCINO COUNTY

SOUTH COAST LANDFILL
ALTERNATIVE SLOPE COVER SECTION

Z:\PROJECTS\MENDOCINO COUNTY\SOUTH COAST\ACAD\EXHIBITS\SLOPE COVER

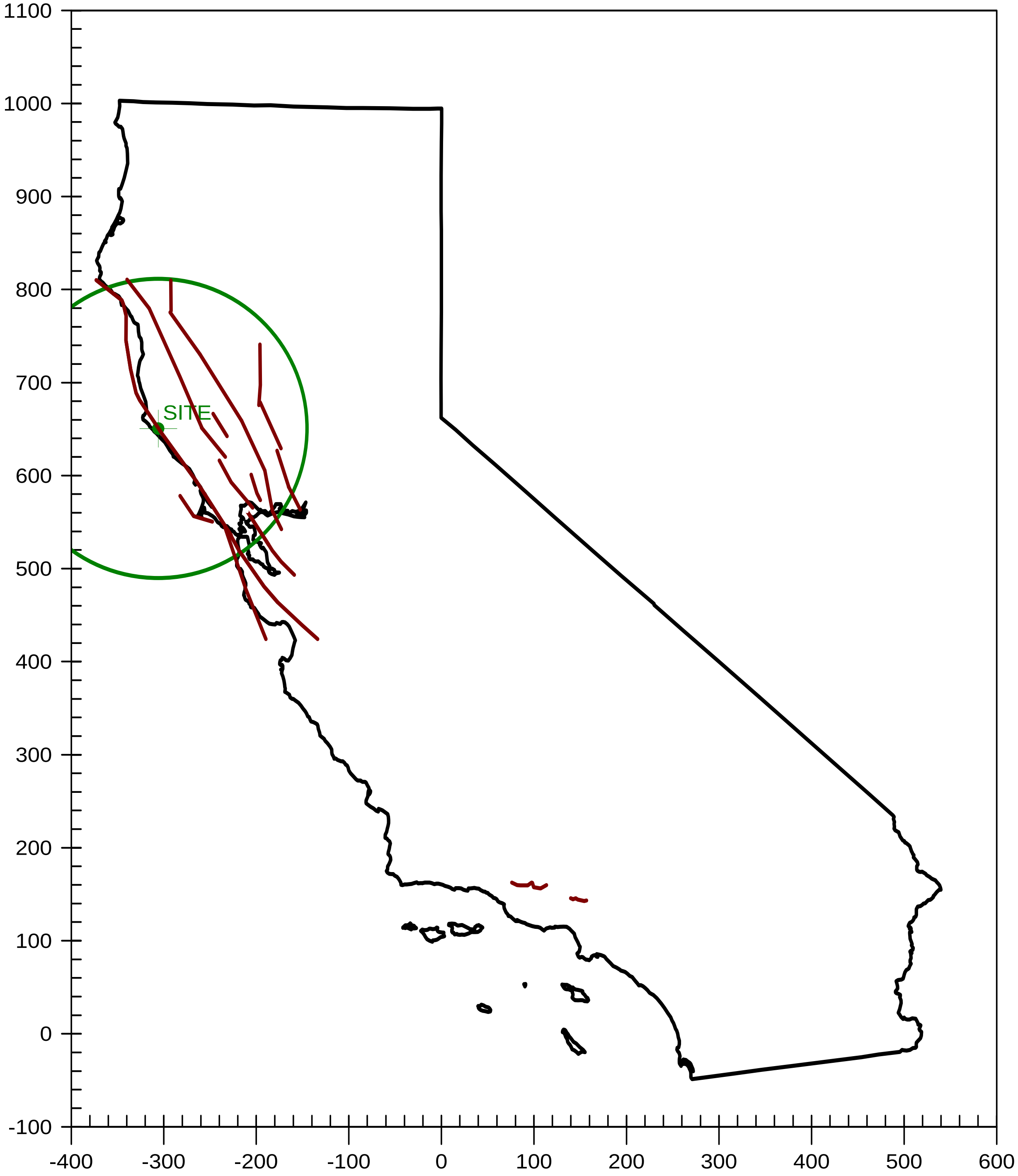
FIGURE 1

APPENDIX A

SEISMIC ANALYSIS

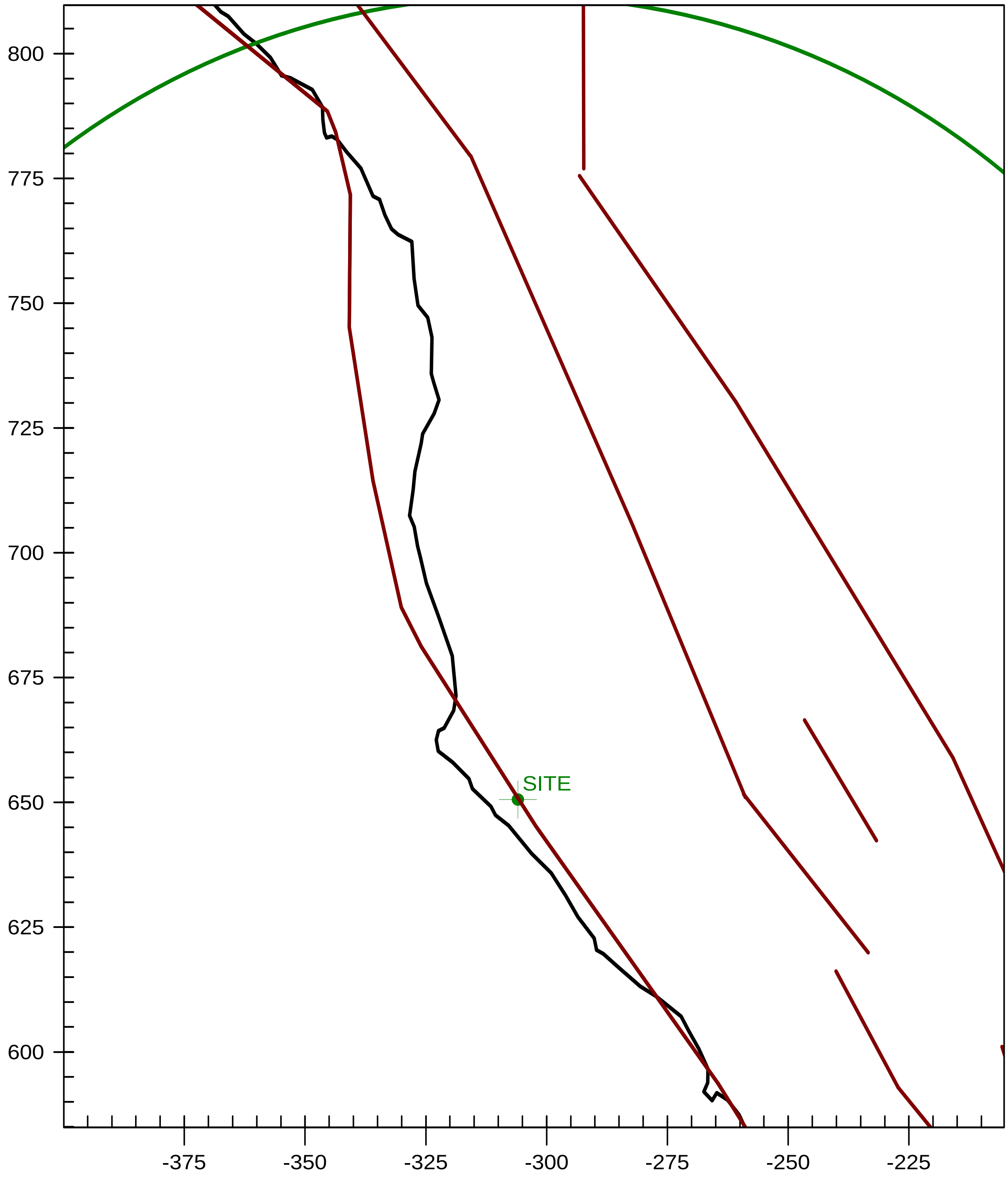
CALIFORNIA FAULT MAP

South Coast Landfill



CALIFORNIA FAULT MAP

South Coast Landfill



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                           DETERMINISTIC ESTIMATION OF
                     PEAK ACCELERATION FROM DIGITIZED FAULTS


JOB NUMBER: 2006-0007
                                                     DATE: 10-29-2012

JOB NAME:               South Coast Landfill

CALCULATION NAME: MPE Analysis

FAULT-DATA-FILE NAME: C:\Program Files\EQFAULT1\CDMG-MPE.dat

SITE COORDINATES:
   SITE LATITUDE:  38.8301
   SITE LONGITUDE:  123.5433

SEARCH RADIUS:   100  mi

ATTENUATION RELATION:  22) Abrahamson & Silva (1995b/1997) Horiz.- Rock
   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\CDMG-MPE.dat

MINIMUM DEPTH VALUE (km):  0.0
```

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                                 ---------------
                                 EQFAULT SUMMARY
                                 ---------------




                          -----------------------------
                          DETERMINISTIC SITE PARAMETERS
                          -----------------------------

Page  1
-------------------------------------------------------------------------------
                                |              |ESTIMATED MAX. EARTHQUAKE EVENT
                                | APPROXIMATE  |-------------------------------
          ABBREVIATED           |   DISTANCE   | MAXIMUM  |   PEAK   |EST. SITE
          FAULT  NAME           |   mi   (km)  |EARTHQUAKE|   SITE   |INTENSITY
                                |              | MAG.(Mw) | ACCEL. g |MOD.MERC.
================================|==============|==========|==========|=========
SAN ANDREAS (1906)              |   0.1(   0.2)|   7.4    |   0.832  |   XI
SAN ANDREAS (North Coast)       |   0.1(   0.2)|   7.4    |   0.832  |   XI
MAACAMA (Central)               |  27.1(  43.6)|   6.5    |   0.071  |   VI
MAACAMA (South)                 |  29.1(  46.8)|   6.4    |   0.063  |   VI
MAACAMA (North)                 |  37.7(  60.6)|   6.6    |   0.052  |   VI
COLLAYOMI                       |  38.3(  61.7)|   5.1    |   0.009  |  III
RODGERS CREEK                   |  46.2(  74.4)|   6.5    |   0.039  |    V
POINT REYES                     |  46.9(  75.4)|   4.9    |   0.010  |  III
BARTLETT SPRINGS                |  50.3(  81.0)|   6.6    |   0.038  |    V
SIERRA MADRE (San Fernando)     |  55.0(  88.5)|   5.6    |   0.020  |   IV
SAN CAYETANO                    |  55.0(  88.5)|   6.4    |   0.039  |    V
HUNTING CREEK - BERRYESSA       |  56.4(  90.7)|   6.4    |   0.030  |    V
ROUND VALLEY                    |  57.3(  92.2)|   6.4    |   0.029  |    V
GREAT VALLEY 2                  |  64.2( 103.4)|   4.2    |   0.003  |    I
GREAT VALLEY 3                  |  65.3( 105.1)|   5.7    |   0.018  |   IV
WEST NAPA                       |  69.5( 111.9)|   5.4    |   0.006  |   II
GREAT VALLEY 1                  |  69.7( 112.1)|   4.3    |   0.003  |    I
GREAT VALLEY 4                  |  76.2( 122.6)|   5.7    |   0.015  |   IV
CONCORD - GREEN VALLEY          |  77.0( 123.9)|   6.5    |   0.022  |   IV
LAKE MOUNTAIN                   |  79.5( 128.0)|   6.4    |   0.020  |   IV
SAN GREGORIO                    |  79.8( 128.5)|   6.5    |   0.021  |   IV
GARBERVILLE-BRICELAND           |  81.0( 130.4)|   6.4    |   0.019  |   IV
HAYWARD (North)                 |  81.4( 131.0)|   6.5    |   0.021  |   IV
HAYWARD (Total Length)          |  81.4( 131.0)|   6.7    |   0.024  |   IV
SAN ANDREAS (Peninsula)         |  86.9( 139.9)|   7.0    |   0.027  |    V
GREAT VALLEY 5                  |  92.1( 148.3)|   5.5    |   0.009  |  III
*******************************************************************************



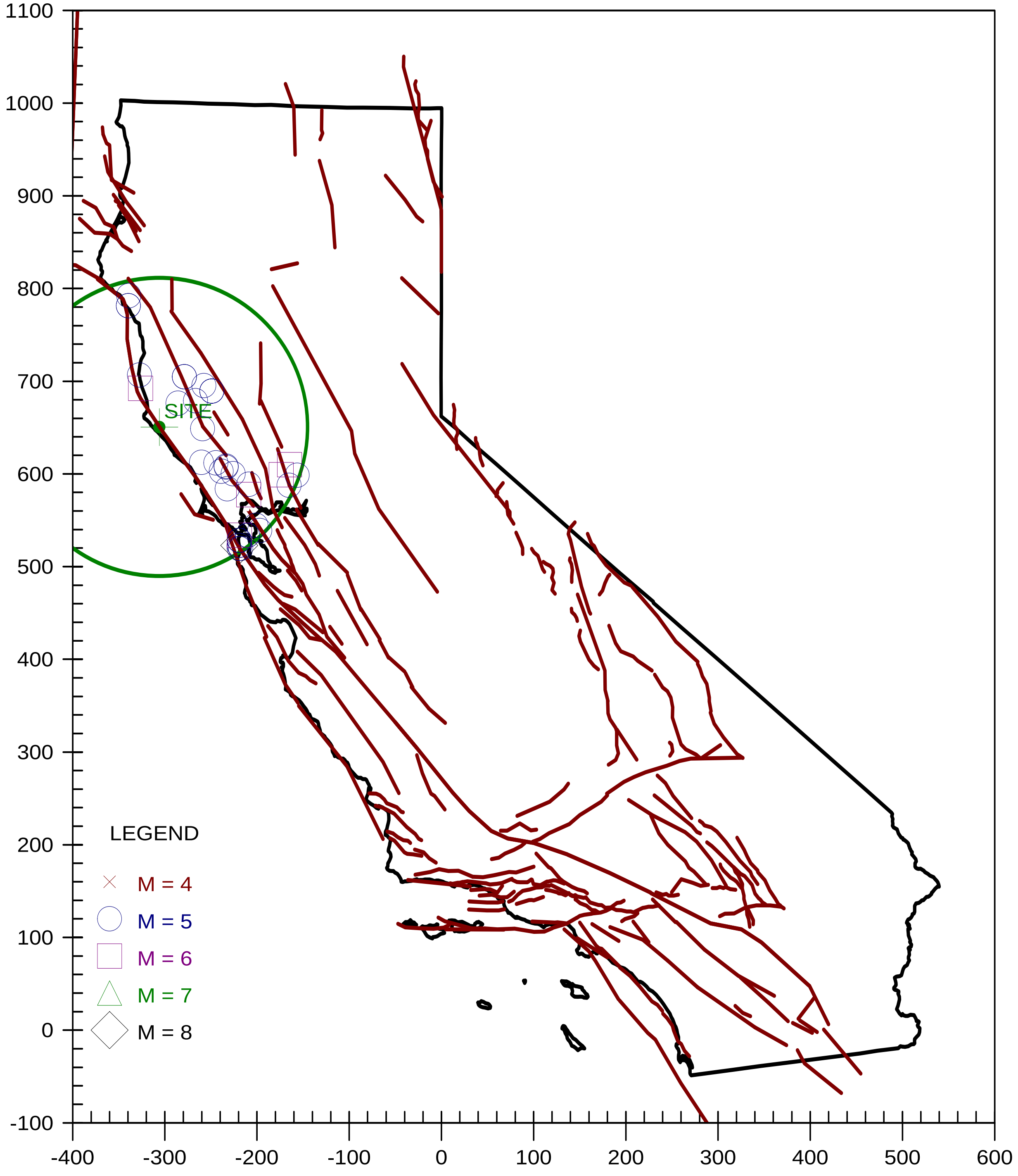
-END OF SEARCH-   26 FAULTS FOUND WITHIN THE SPECIFIED SEARCH RADIUS.

THE SAN ANDREAS (North Coast)        FAULT IS CLOSEST TO THE SITE.
IT IS ABOUT 0.1 MILES (0.2 km) AWAY.

LARGEST MAXIMUM-EARTHQUAKE SITE ACCELERATION: 0.8317 g
```

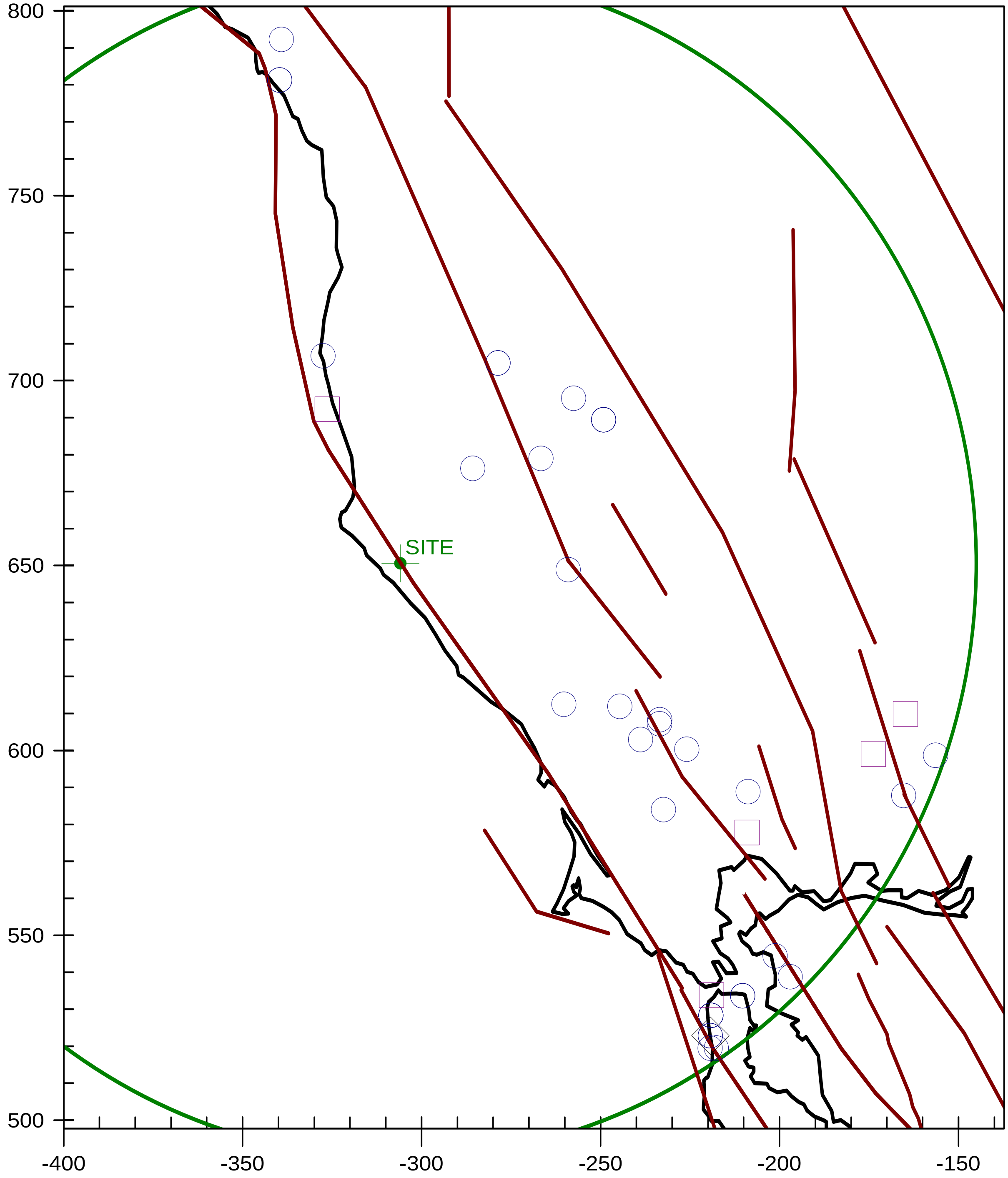
EARTHQUAKE EPICENTER MAP

Southcoast LF



EARTHQUAKE EPICENTER MAP

Southcoast LF



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                                 ESTIMATION OF
                            PEAK ACCELERATION FROM
                        CALIFORNIA EARTHQUAKE CATALOGS


JOB NUMBER: 2012-0007
                                                     DATE: 10-29-2012

JOB NAME:              Southcoast LF

EARTHQUAKE-CATALOG-FILE NAME: C:\Program Files\EQSEARCH\ALLQUAKE_2009.DAT

SITE COORDINATES:
   SITE LATITUDE:  38.8301
   SITE LONGITUDE:  123.5433

SEARCH DATES:
           START DATE:   1800
           END DATE:   1999

SEARCH RADIUS:
           100.0 mi
           160.9 km

ATTENUATION RELATION:  22) Abrahamson & Silva (1995b/1997) Horiz.- Rock
   UNCERTAINTY (M=Median, S=Sigma): M       Number of Sigmas:  0.0
   ASSUMED SOURCE TYPE:  DS [SS=Strike-slip, DS=Reverse-slip, BT=Blind-thrust]
   SCOND:   0  Depth Source:  A
   Basement Depth:  5.00 km     Campbell SSR:        Campbell SHR:
   COMPUTE PEAK HORIZONTAL ACCELERATION

MINIMUM DEPTH VALUE (km):  0.0
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                            -------------------------
                            EARTHQUAKE SEARCH RESULTS
                            -------------------------

Page  1
-------------------------------------------------------------------------------
    |       |        |          |  TIME  |     |     | SITE  |SITE|  APPROX.
FILE|  LAT. |  LONG. |   DATE   |  (UTC) |DEPTH|QUAKE|  ACC. | MM |  DISTANCE
CODE| NORTH |  WEST  |          | H M Sec| (km)| MAG.|   g   |INT.|  mi  [km]
----+-------+--------+----------+--------+-----+-----+-------+----+-----------
DMG |39.0700|123.3200|06/06/1962|1750 6.2|  5.0| 5.20| 0.045 | VI | 20.4( 32.9)
DMG |39.2000|123.8000|04/15/1898| 7 7 0.0|  0.0| 6.40| 0.081 | VII| 29.0( 46.7)
T-A |38.8300|123.0000|08/01/1885| 010 0.0|  0.0| 5.00| 0.022 | IV | 29.2( 47.0)
DMG |39.1000|123.1000|10/08/1869| 930 0.0|  0.0| 5.00| 0.021 | IV | 30.2( 48.6)
MGI |38.5000|123.0000|05/02/1906| 530 0.0|  0.0| 5.00| 0.016 | IV | 37.1( 59.7)
T-A |39.3300|123.8200|12/16/1865| 5 0 0.0|  0.0| 5.00| 0.016 | IV | 37.6( 60.4)
T-A |39.3300|123.2500|01/03/1865| 5 0 0.0|  0.0| 5.00| 0.016 | IV | 37.9( 61.0)
T-A |39.3300|123.2500|09/26/1868| 840 0.0|  0.0| 5.70| 0.036 |  V | 37.9( 61.0)
MGI |39.2500|123.0000|11/15/1938|1348 0.0|  0.0| 5.00| 0.014 | IV | 41.1( 66.1)
MGI |39.2000|122.9000|05/07/1906| 410 0.0|  0.0| 5.30| 0.019 | IV | 42.9( 69.1)
MGI |39.2000|122.9000|05/07/1906| 5 0 0.0|  0.0| 5.30| 0.019 | IV | 42.9( 69.1)
T-A |38.5000|122.8200|01/03/1876|1855 0.0|  0.0| 5.00| 0.012 | III| 45.2( 72.7)
T-A |38.4200|122.7500|10/13/1899| 5 0 0.0|  0.0| 5.70| 0.025 |  V | 51.3( 82.6)
DMG |38.4700|122.6900|10/02/1969| 45646.5|  0.0| 5.60| 0.021 | IV | 52.3( 84.2)
DMG |38.4600|122.6900|10/02/1969| 61957.1|  0.0| 5.70| 0.024 |  V | 52.6( 84.7)
DMG |38.4000|122.6000|08/09/1893| 915 0.0|  0.0| 5.10| 0.010 | III| 58.9( 94.8)
T-A |38.2500|122.6700|08/19/1858| 648 0.0|  0.0| 5.00| 0.008 | III| 61.9( 99.6)
DMG |38.3000|122.4000|10/12/1891| 628 0.0|  0.0| 5.50| 0.013 | III| 71.8(115.5)
DMG |38.2000|122.4000|03/31/1898| 743 0.0|  0.0| 6.20| 0.024 | IV | 75.5(121.6)
DMG |40.0000|124.0000|09/12/1938| 610 0.0|  0.0| 5.50| 0.010 | III| 84.4(135.8)
DMG |40.0000|124.0000|12/20/1940|234042.0|  0.0| 5.50| 0.010 | III| 84.4(135.8)
DMG |38.4000|122.0000|04/19/1892|1050 0.0|  0.0| 6.40| 0.023 | IV | 88.4(142.2)
MGI |37.8000|122.5000|06/21/1808| 0 0 0.0|  0.0| 6.30| 0.021 | IV | 90.8(146.2)
DMG |40.1000|124.0000|05/09/1878| 425 0.0|  0.0| 5.80| 0.014 | IV | 91.0(146.4)
DMG |38.5000|121.9000|04/21/1892|1743 0.0|  0.0| 6.20| 0.019 | IV | 91.5(147.2)
DMG |37.9000|122.3000|04/02/1870|1948 0.0|  0.0| 5.30| 0.007 | II | 93.0(149.7)
T-A |37.7500|122.5000|10/22/1854| 330 0.0|  0.0| 5.00| 0.005 |  I | 93.6(150.6)
T-A |37.7500|122.5000|12/11/1855|12 0 0.0|  0.0| 5.00| 0.005 |  I | 93.6(150.6)
T-A |37.7500|122.5000|02/26/1864| 840 0.0|  0.0| 5.00| 0.005 |  I | 93.6(150.6)
MGI |37.8000|122.4000|05/15/1851|1610 0.0|  0.0| 5.00| 0.004 |  I | 94.3(151.8)
MGI |37.8000|122.4000|10/05/1859|2016 0.0|  0.0| 5.00| 0.004 |  I | 94.3(151.8)
DMG |38.3000|121.9000|05/19/1902|1831 0.0|  0.0| 5.50| 0.009 | III| 96.0(154.4)
DMG |37.7000|122.5000|04/18/1906|131221.0|  0.0| 8.25| 0.078 | VII| 96.4(155.1)
DMG |37.7000|122.5000|06/02/1899| 719 0.0|  0.0| 5.40| 0.007 | II | 96.4(155.1)
MGI |37.7000|122.5000|04/25/1906|2315 0.0|  0.0| 5.30| 0.007 | II | 96.4(155.1)
MGI |37.8500|122.2500|09/10/1935|2355 0.0|  0.0| 5.00| 0.004 |  I | 97.4(156.7)
T-A |37.6700|122.5000|11/23/1852| 7 0 0.0|  0.0| 5.70| 0.011 | III| 98.1(157.8)
DMG |38.4000|121.8000|04/30/1892| 0 9 0.0|  0.0| 5.50| 0.008 | III| 98.6(158.7)
DMG |37.6700|122.4800|03/22/1957|194421.0|  8.5| 5.30| 0.006 | II | 98.7(158.8)

*******************************************************************************
-END OF SEARCH-   39 EARTHQUAKES FOUND WITHIN THE SPECIFIED SEARCH AREA.

TIME PERIOD OF SEARCH:   1800  TO  1999

LENGTH OF SEARCH TIME:   200  years

THE EARTHQUAKE CLOSEST TO THE SITE IS ABOUT 20.4 MILES (32.9 km) AWAY.

LARGEST EARTHQUAKE MAGNITUDE FOUND IN THE SEARCH RADIUS: 8.3

LARGEST EARTHQUAKE SITE ACCELERATION FROM THIS SEARCH: 0.081 g
```

```
COEFFICIENTS FOR GUTENBERG & RICHTER RECURRENCE RELATION:
  a-value=  1.042
  b-value=  0.397
  beta-value=  0.915


------------------------------------
TABLE OF MAGNITUDES AND EXCEEDANCES:
------------------------------------

  Earthquake | Number of Times | Cumulative
   Magnitude |    Exceeded     | No. / Year
  -----------+-----------------+------------
     4.0     |       39        |   0.19598
     4.5     |       39        |   0.19598
     5.0     |       39        |   0.19598
     5.5     |       17        |   0.08543
     6.0     |        6        |   0.03015
     6.5     |        1        |   0.00503
     7.0     |        1        |   0.00503
     7.5     |        1        |   0.00503
     8.0     |        1        |   0.00503
```

CALCULATION OF WEIGHTED AVERAGE 2008 NGA MODELS: ***by Linda Al Atik, PEER - Sep, 2009 - l_atik@berkeley.edu***

This Excel file calculates the weighted average of the natural logarithm of the spectral values from the NGA models

NGA Model:	AS08	BA08	CB08	CY08	I08
	0.2	0.2	0.2	0.2	0.2

AS08: Abrahamson & Silva 2008 NGA Model
BA08: Boore & Atkinson 2008 NGA Model
CB08: Campbell & Bozorgnia 2008 NGA Model
CY08: Chiou & Youngs 2008 NGA Model
I08: Idriss 2008 NGA Model

N	1

Site: **South Coast Landfill**
Fault: **San Andreas Fault**

Explanatory Variables

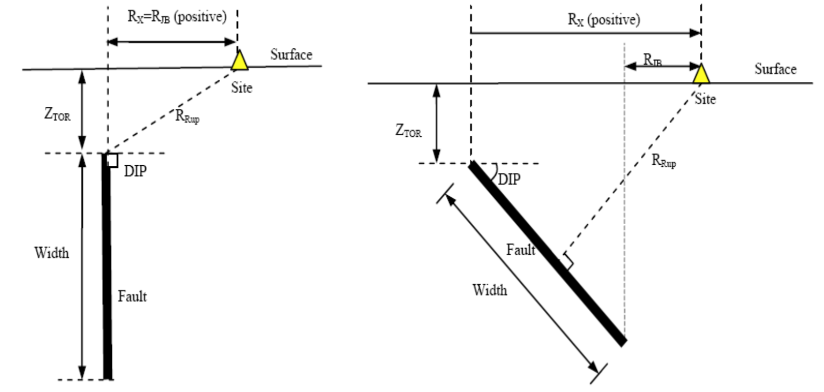
Variable	Value
M	7.90
R_{RUP} (km)	0.00
R_{JB} (km)	0.00
R_X (km)	0.00
U	0
F_{RV}	0
F_{NM}	0
F_{HW}	0
Z_{TOR} (km)	0.00
δ	90
V_{S30} (m/sec)	520
$F_{Measured}$	0
$Z_{1.0}$ (m)	DEFAULT
$Z_{2.5}$ (km)	DEFAULT
W (km)	14
F_{AS}	0
HW Taper	0

Geometric Mean Horizontal Component

GMP	*T* (s)	SA Median	SA Median + N.σ	SA Median - N.σ	SD Median
PSA (g)	0.010	0.612	1.025	0.365	0.002
SD (cm)	0.020	0.622	1.043	0.371	0.006
	0.030	0.663	1.119	0.392	0.015
	0.050	0.755	1.284	0.444	0.047
	0.075	0.864	1.496	0.499	0.121
	0.10	1.050	1.819	0.606	0.261
	0.15	1.248	2.181	0.715	0.697
	0.20	1.361	2.402	0.772	1.352
	0.25	1.393	2.482	0.782	2.162
	0.30	1.350	2.436	0.748	3.016
	0.40	1.246	2.276	0.683	4.950
	0.50	1.106	2.040	0.600	6.866
	0.75	0.828	1.557	0.440	11.556
	1.0	0.676	1.285	0.355	16.773
	1.5	0.471	0.910	0.243	26.295
	2.0	0.340	0.666	0.174	33.774
	3.0	0.221	0.437	0.112	49.360
	4.0	0.155	0.309	0.078	61.578
	5.0	0.117	0.240	0.057	72.814
	7.5	0.078	0.166	0.037	108.825
	10.0	0.039	0.085	0.018	98.012
PGA (g)	0	0.610	1.022	0.364	
PGV (c/s)	-1	85.758	147.304	49.927	

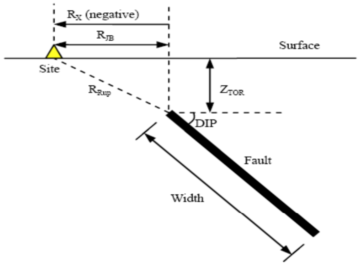
DEFINITION OF PARAMETERS:

N = Number of standard deviations to be considered in the calculations
PSA = Pseudo-absolute acceleration response spectrum (g; 5% damping)
PGA = Peak ground acceleration (g)
PGV = Peak ground velocity (cm/s)
SD = Relative displacement response spectrum (cm; 5% damping)
M = Moment magnitude
R_{RUP} = Closest distance to coseismic rupture (km), used in AS08, CB08 and CY08. See Figures a, b and c for illustration
R_{JB} = Closest distance to surface projection of coseismic rupture (km). See Figures a, b and c for illustration
R_X = Horizontal distance from top of rupture measured perpendicular to fault strike (km), used in AS08 and CY08. See Figures a, b and c for illustation
U = Unspecified-mechanism factor: 1 for unspecified; 0 otherwise, used in BA08
F_{RV} = Reverse-faulting factor: 0 for strike slip, normal, normal-oblique; 1 for reverse, reverse-oblique and thrust
F_{NM} = Normal-faulting factor: 0 for strike slip, reverse, reverse-oblique, thrust and normal-oblique; 1 for normal
F_{HW} = Hanging-wall factor: 1 for site on down-dip side of top of rupture; 0 otherwise, used in AS08 and CY08
Z_{TOR} = Depth to top of coseismic rupture (km), used in AS08, CB08 and CY08
δ = Average dip of rupture plane (degrees), used in AS08, CB08 and CY08
V_{S30} = Average shear-wave velocity in top 30m of site profile
$F_{Measured}$ = Vs30 Factor: 1 if VS30 is measured, 0 if Vs30 is inferred, used in AS08 and CY08
$Z_{1.0}$ = Depth to 1.0 km/sec velocity horizon (m), used in AS08 and CY08. Enter "DEFAULT" in order to use the default values or enter your site specific number
$Z_{2.5}$ = Depth of 2.5 km/s shear-wave velocity horizon (km), used in CB08. Enter "DEFAULT" in order to use the default value or enter your site specific number
W = Fault rupture width (km), used in AS08
F_{AS} = Aftershock factor: 0 for mainshock; 1 for aftershock, used in AS08 and CY08
HW Taper = To choose the hanging wall taper to be used in AS08. Enter 0 to use the hanging wall taper as published in Abrahamson and Silva (2008), or enter 1 to use the revised hanging wall taper suggested by Norm Abrahamson



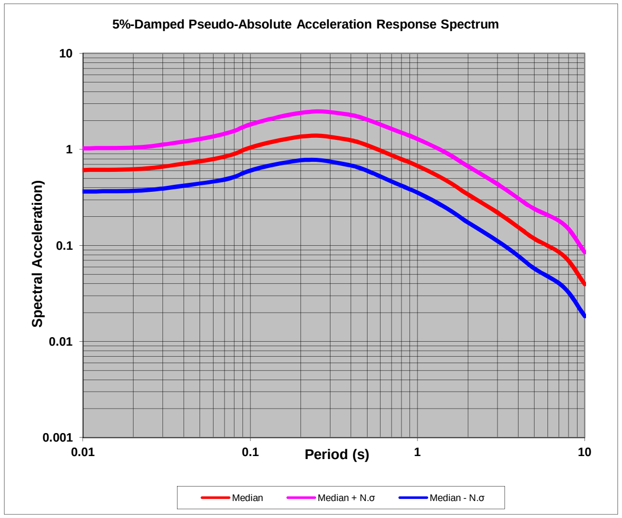
(a) Strike slip faulting

(b) Reverse or normal faulting, hanging-wall site



(c) Reverse or normal faulting, foot-wall site

Figures (a), (b) and (c): Definition of Fault Geometry and Distance Measures

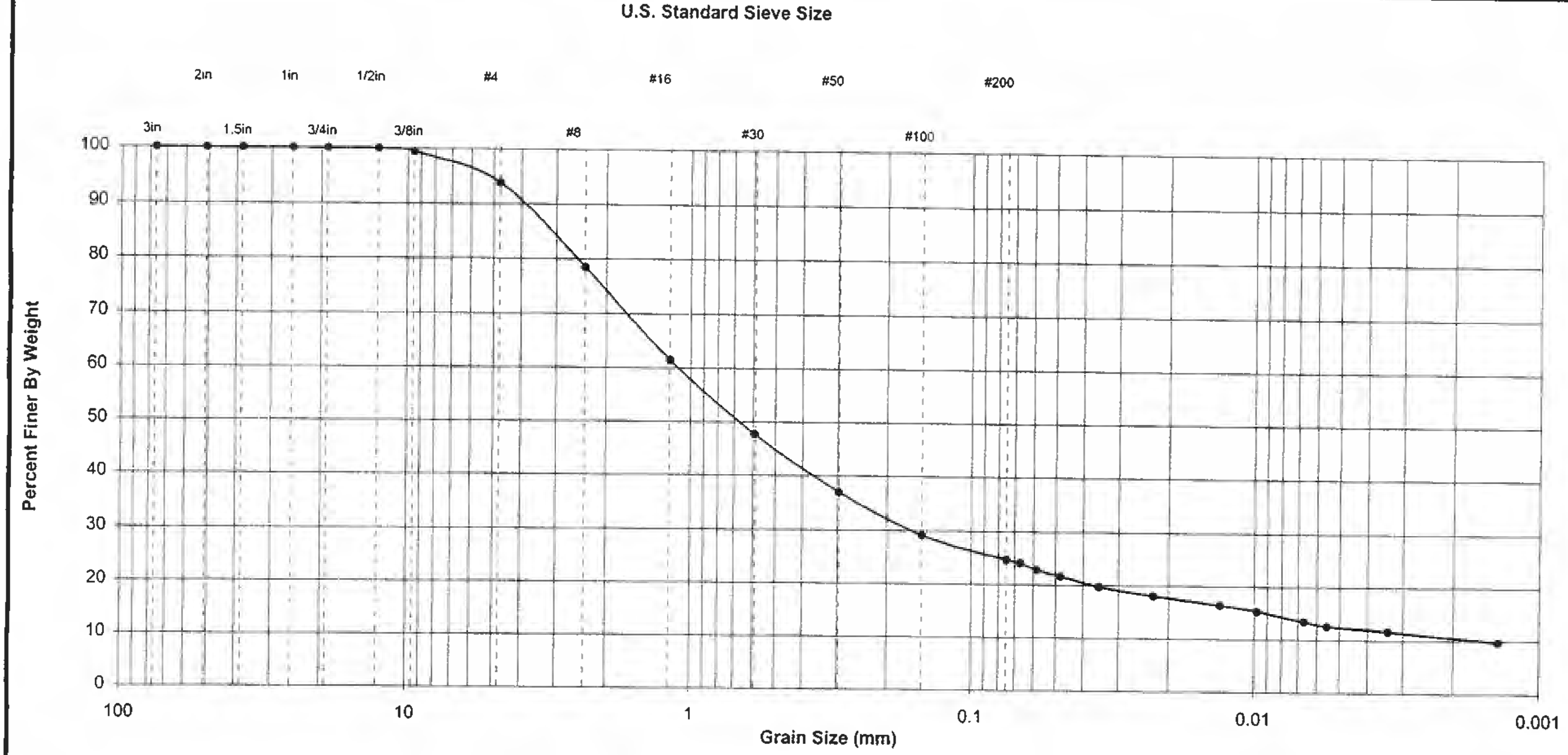


APPENDIX B

LABORATORY TEST RESULTS

GRAIN SIZE ANALYSIS – ASTM D422

GRAIN SIZE ANALYSIS - ASTM D 422



Boring / Sample No.	Initial Dry Density (pcf)	Initial Moist. (%)	Test Dry Density (pcf)	Test Moist. (%)	Permeability, K (cm/sec)	LL	PL	PI	Unified Soil Class.	Description
B-4 #3 @ 13'									SM	SILTY SAND

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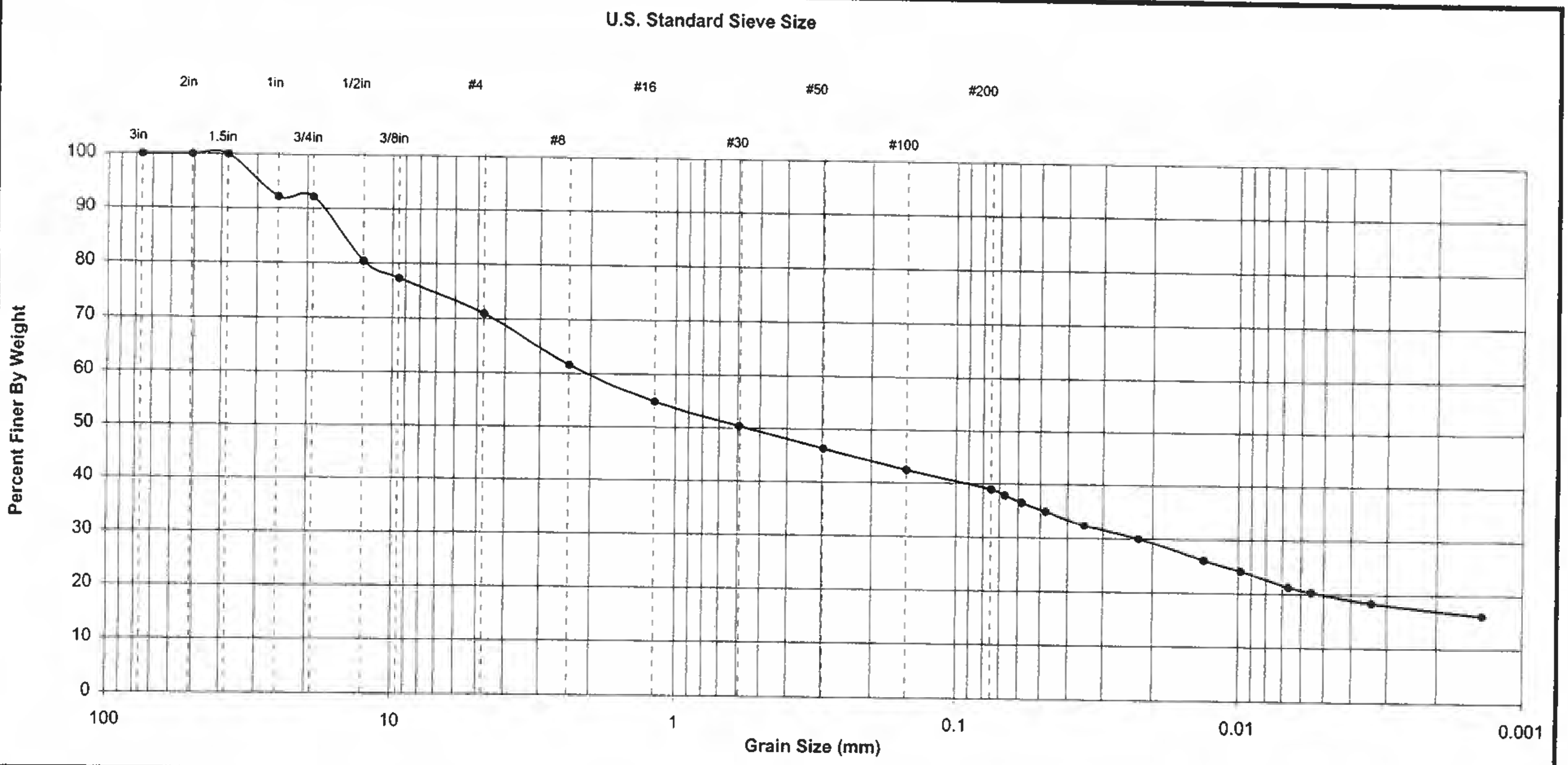
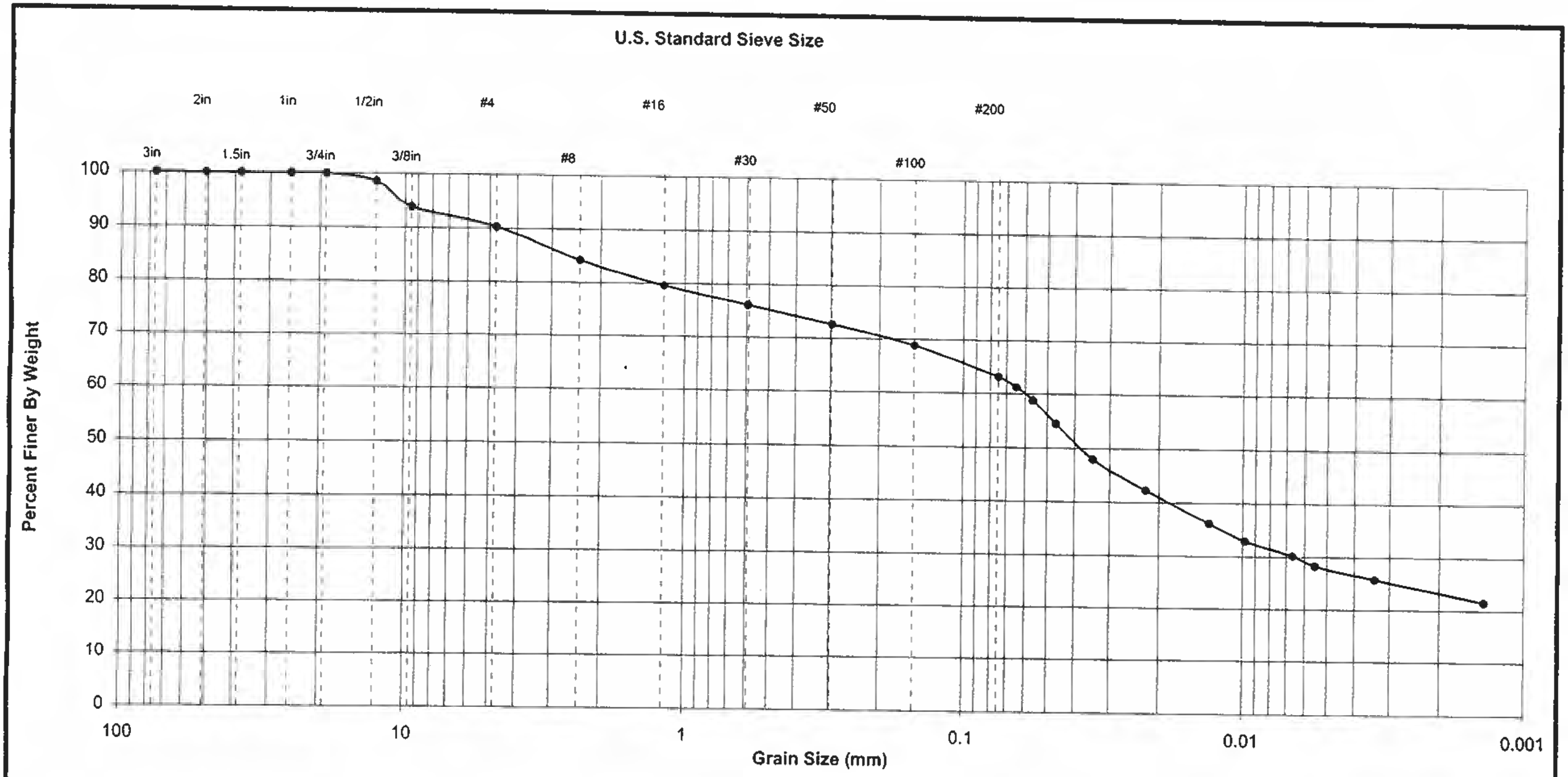
GRAIN SIZE ANALYSIS - ASTM D 422



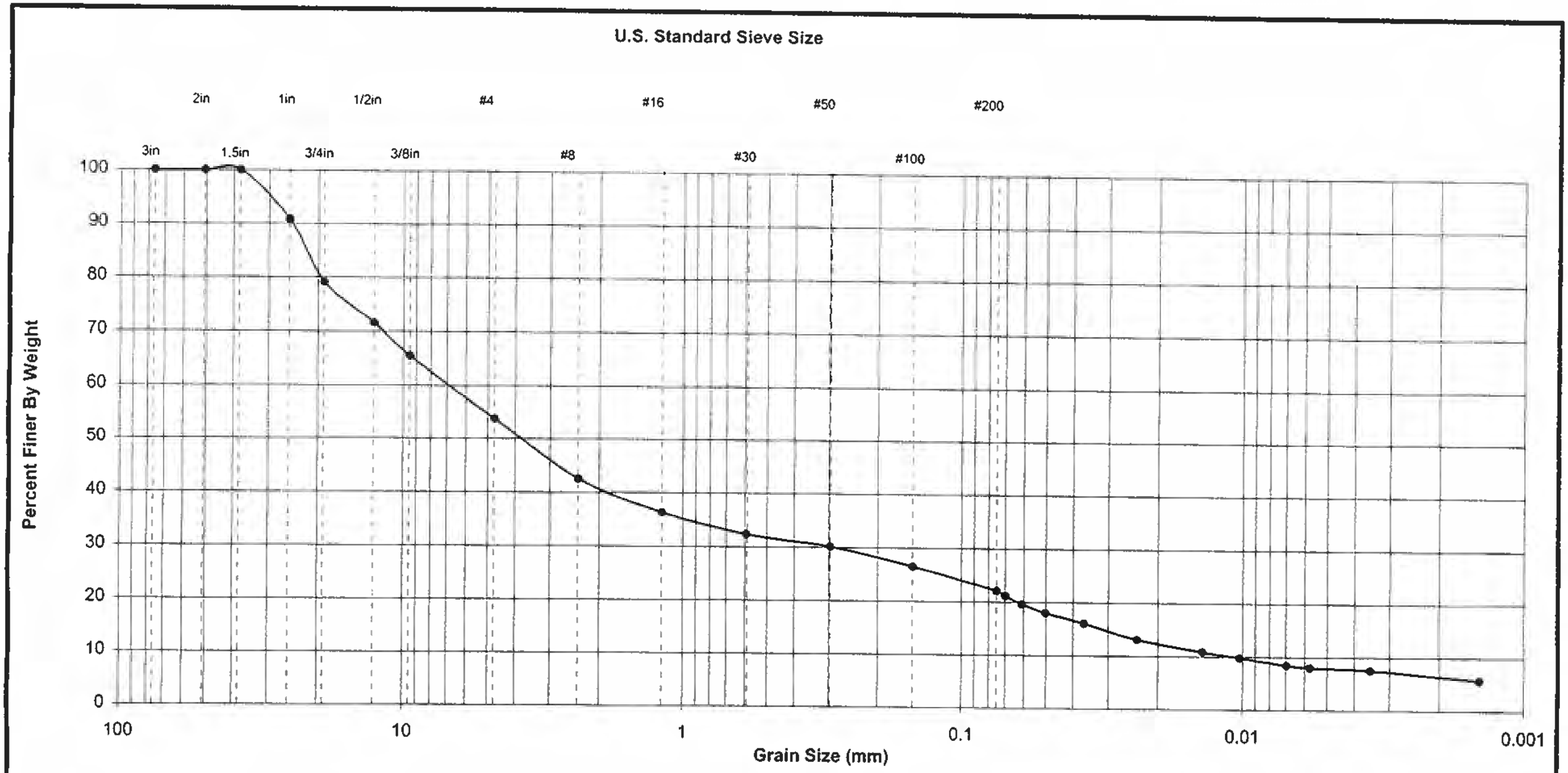
Boring / Sample No.	Initial Dry Density (pcf)	Initial Moist. (%)	Test Dry Density (pcf)	Test Moist. (%)	Permeability, K (cm/sec)	LL	PL	PI	Unified Soil Class.	Description
B-1 #3 @ 15'									ML	Clayey Silt W. Siltstone

GRAIN SIZE ANALYSIS - ASTM D 422



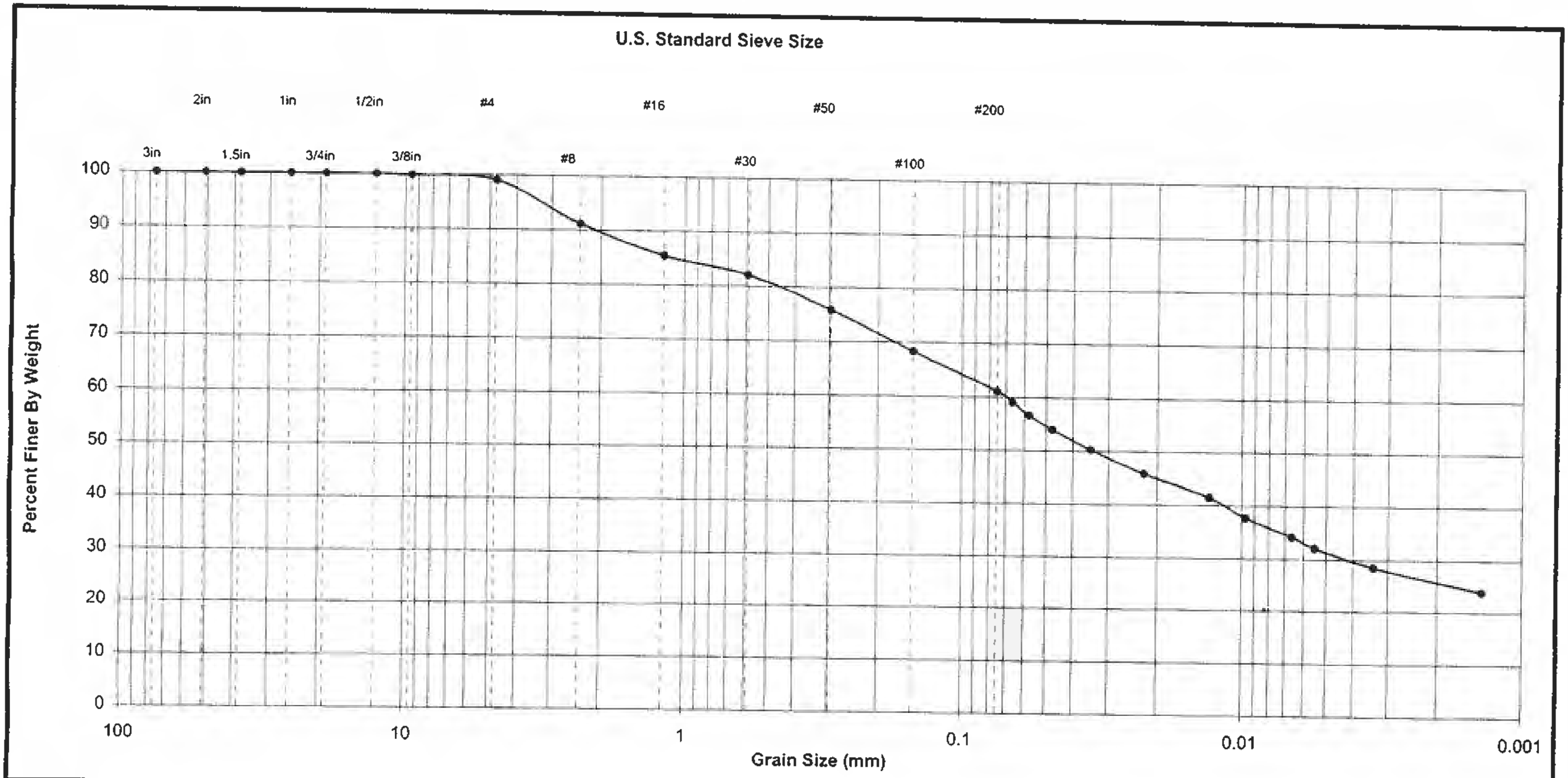
Boring / Sample No.	Initial Dry Density (pcf)	Initial Moist. (%)	Test Dry Density (pcf)	Test Moist. (%)	Permeability, K (cm/sec)	LL	PL	PI	Unified Soil Class.	Description
B-1 #4 @ 20'									ML	Clayey Silt W. Siltstone

GRAIN SIZE ANALYSIS - ASTM D 422



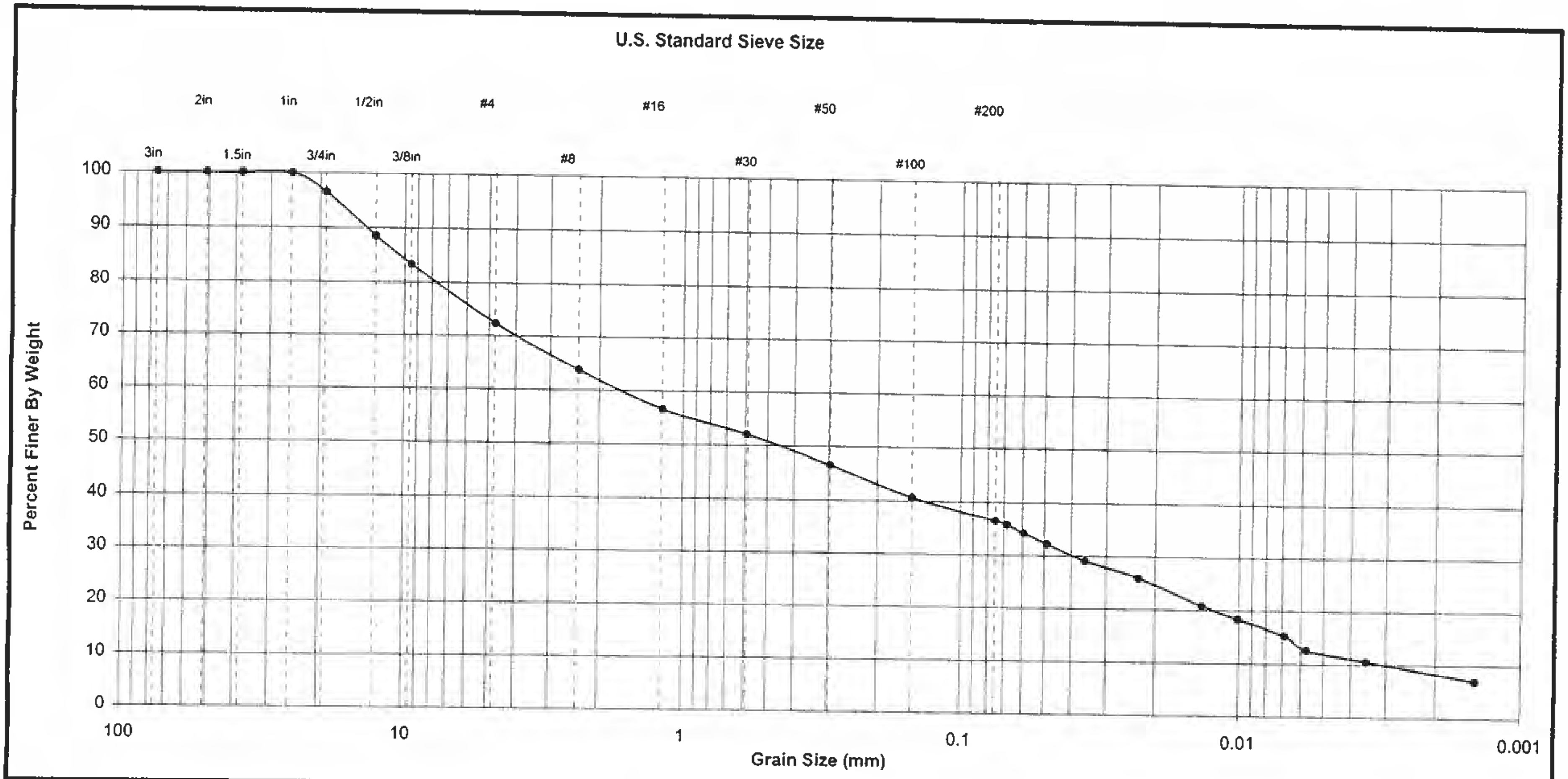
Boring / Sample No.	Initial Dry Density (pcf)	Initial Moist. (%)	Test Dry Density (pcf)	Test Moist. (%)	Permeability, K (cm/sec)	LL	PL	PI	Unified Soil Class.	Description
B-1 #7 @ 35'									ML	CLAYEY SILT W. SILTONE

GRAIN SIZE ANALYSIS - ASTM D 422



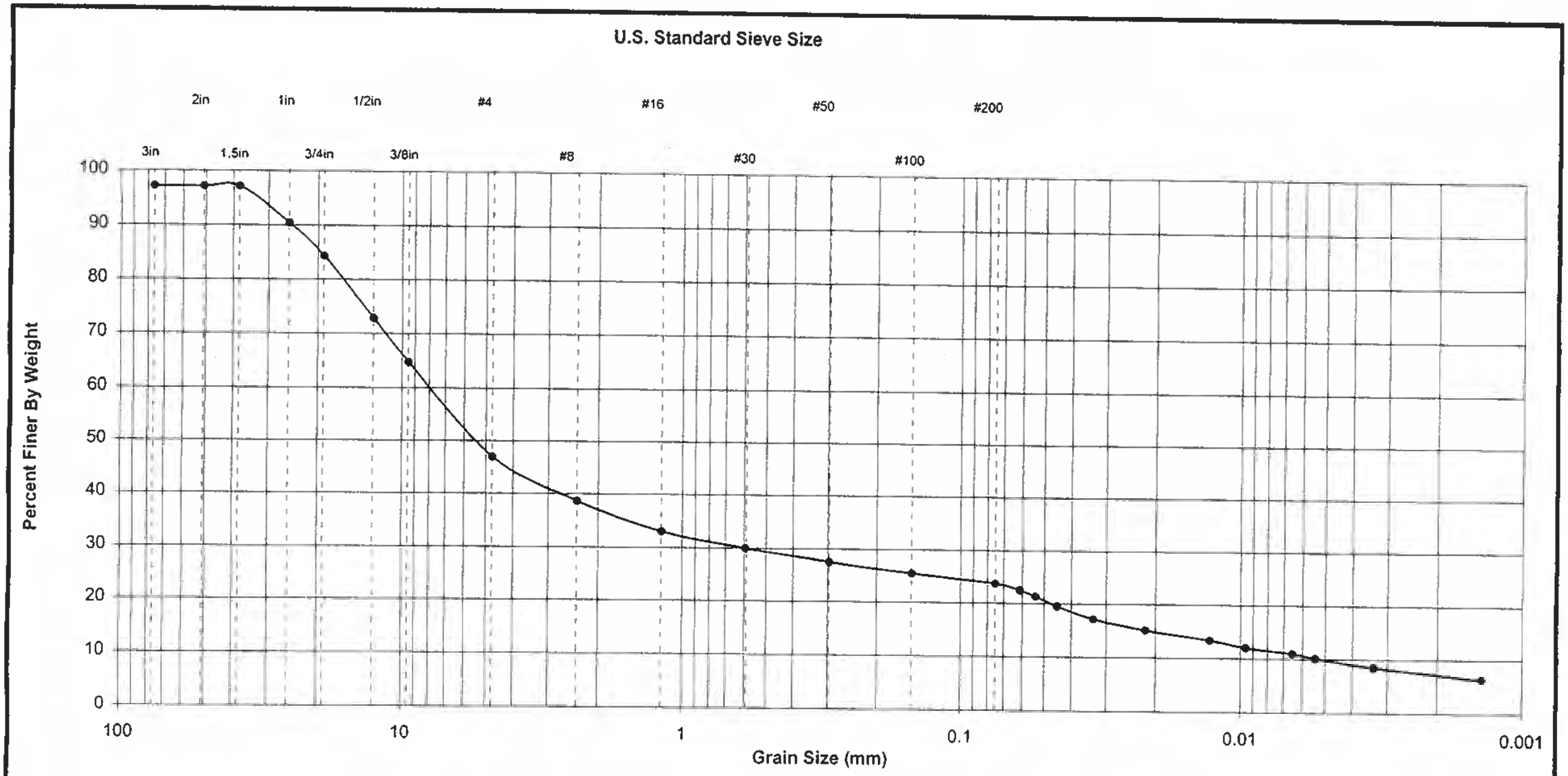
Boring / Sample No.	Initial Dry Density (pcf)	Initial Moist. (%)	Test Dry Density (pcf)	Test Moist. (%)	Permeability, K (cm/sec)	LL	PL	PI	Unified Soil Class.	Description
B-2 #1 @ 10'									ML	Clayey Silt W. Siltstone

GRAIN SIZE ANALYSIS - ASTM D 422



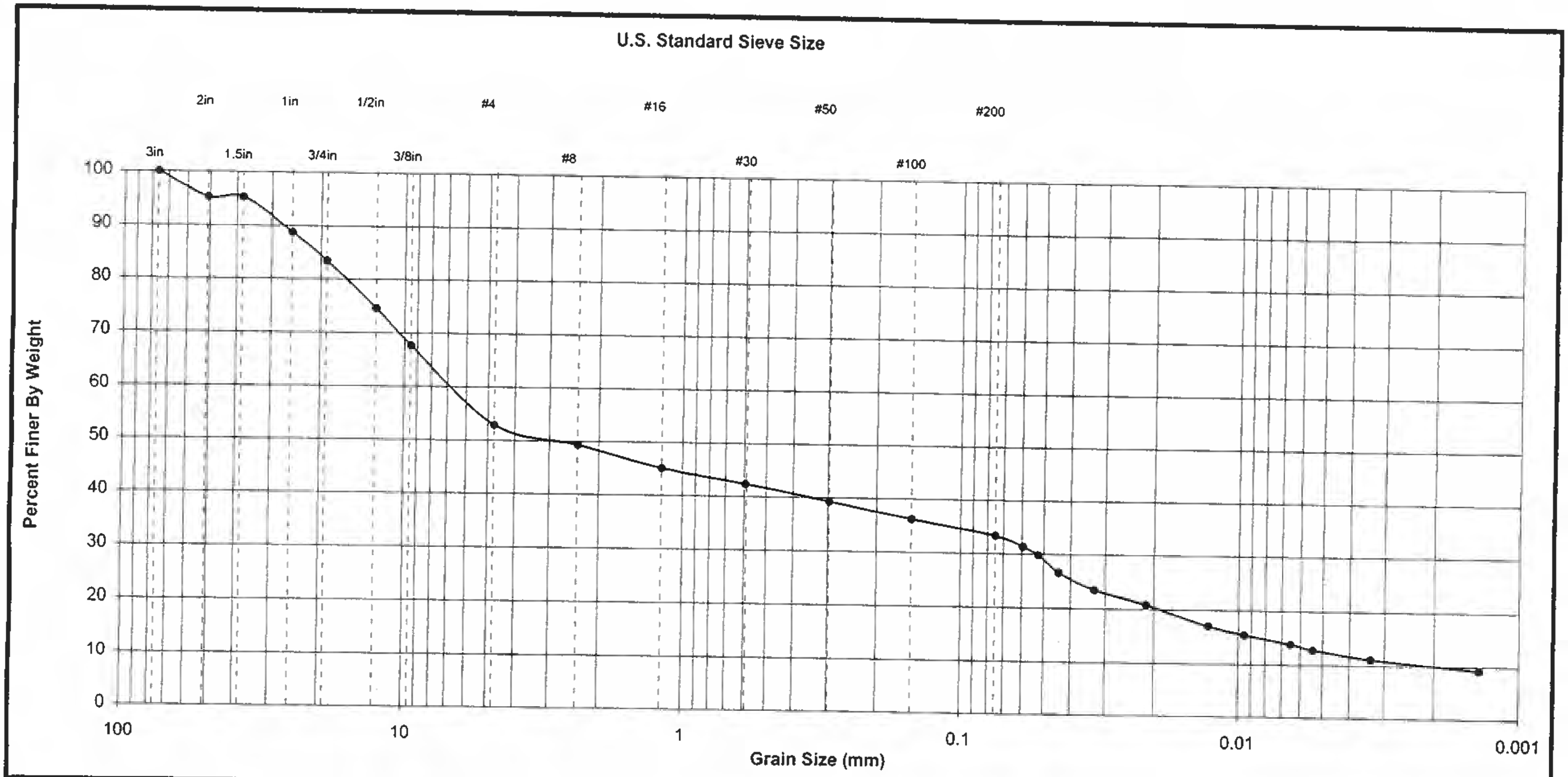
Boring / Sample No.	Initial Dry Density (pcf)	Initial Moist. (%)	Test Dry Density (pcf)	Test Moist. (%)	Permeability, K (cm/sec)	LL	PL	PI	Unified Soil Class.	Description
B-2 #4 @ 22.5'									ML	Clayey Silt W. Siltstone

GRAIN SIZE ANALYSIS - ASTM D 422



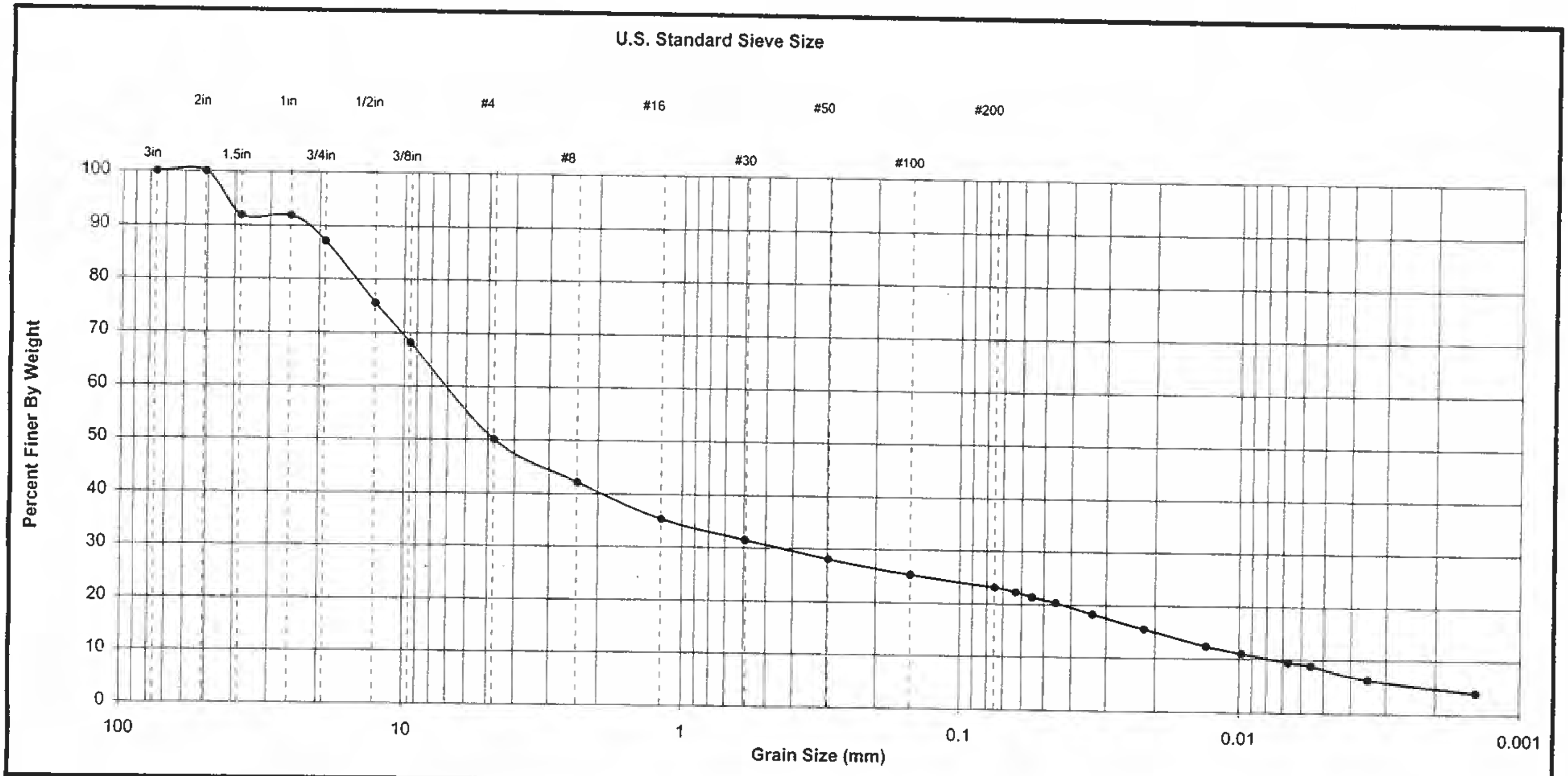
Boring / Sample No.	Initial Dry Density (pcf)	Initial Moist. (%)	Test Dry Density (pcf)	Test Moist. (%)	Permeability, K (cm/sec)	LL	PL	PI	Unified Soil Class.	Description
TP-1, 5, 10, 16									ML	Clayey Silt W. Siltstone

GRAIN SIZE ANALYSIS - ASTM D 422



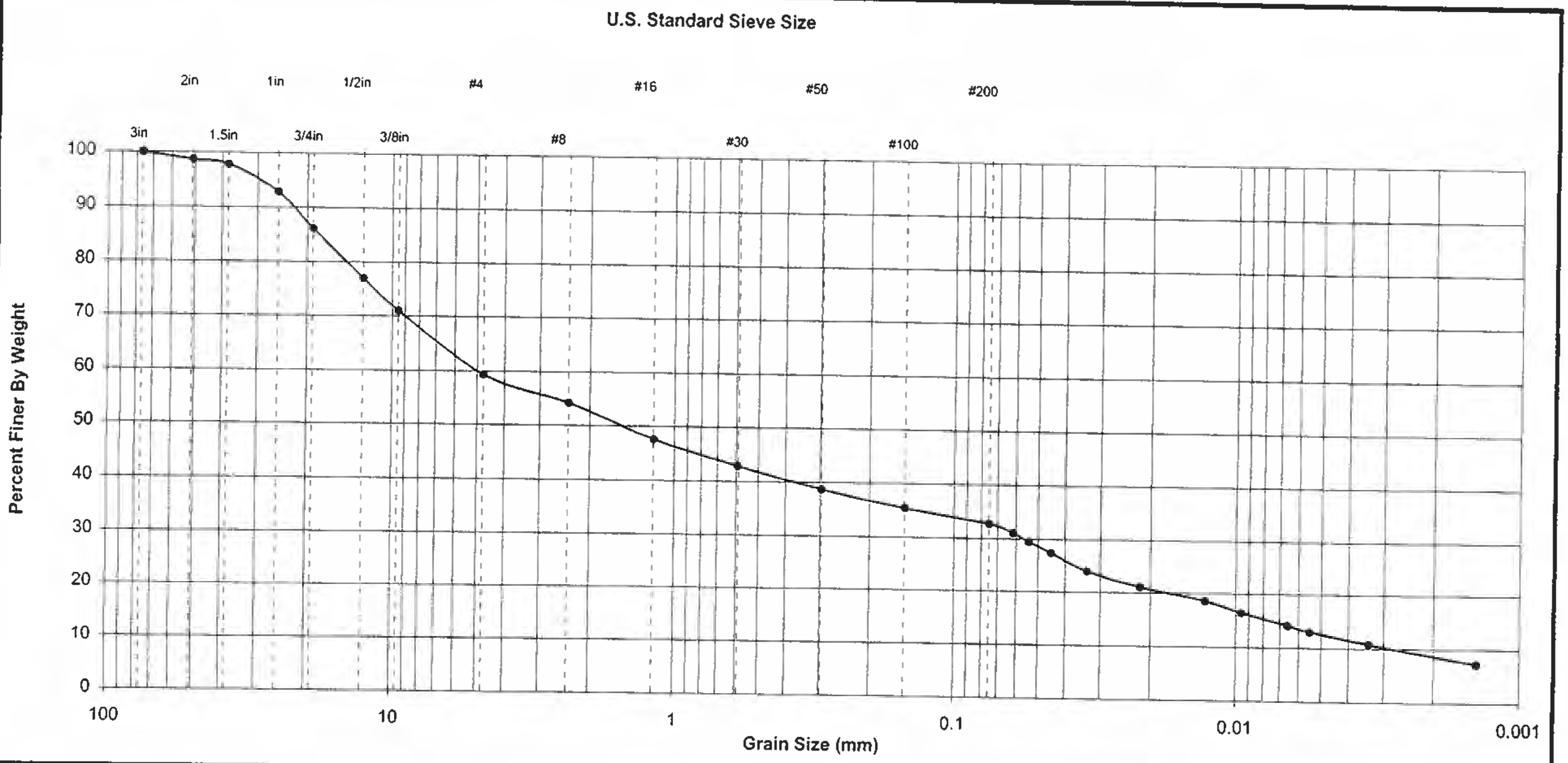
Boring / Sample No.	Initial Dry Density (pcf)	Initial Moist. (%)	Test Dry Density (pcf)	Test Moist. (%)	Permeability, K (cm/sec)	LL	PL	PI	Unified Soil Class.	Description
TP-16									ML	Clayey Silt W. Siltstone

GRAIN SIZE ANALYSIS - ASTM D 422



Boring / Sample No.	Initial Dry Density (pcf)	Initial Moist. (%)	Test Dry Density (pcf)	Test Moist. (%)	Permeability, K (cm/sec)	LL	PL	PI	Unified Soil Class.	Description
TP-5									ML	Clayey Silt W. Siltstone

GRAIN SIZE ANALYSIS - ASTM D 422



Boring / Sample No.	Initial Dry Density (pcf)	Initial Moist. (%)	Test Dry Density (pcf)	Test Moist. (%)	Permeability, K (cm/sec)	LL	PL	PI	Unified Soil Class.	Description
Stockpile #2									ML	Clayey Silt W. Siltstone

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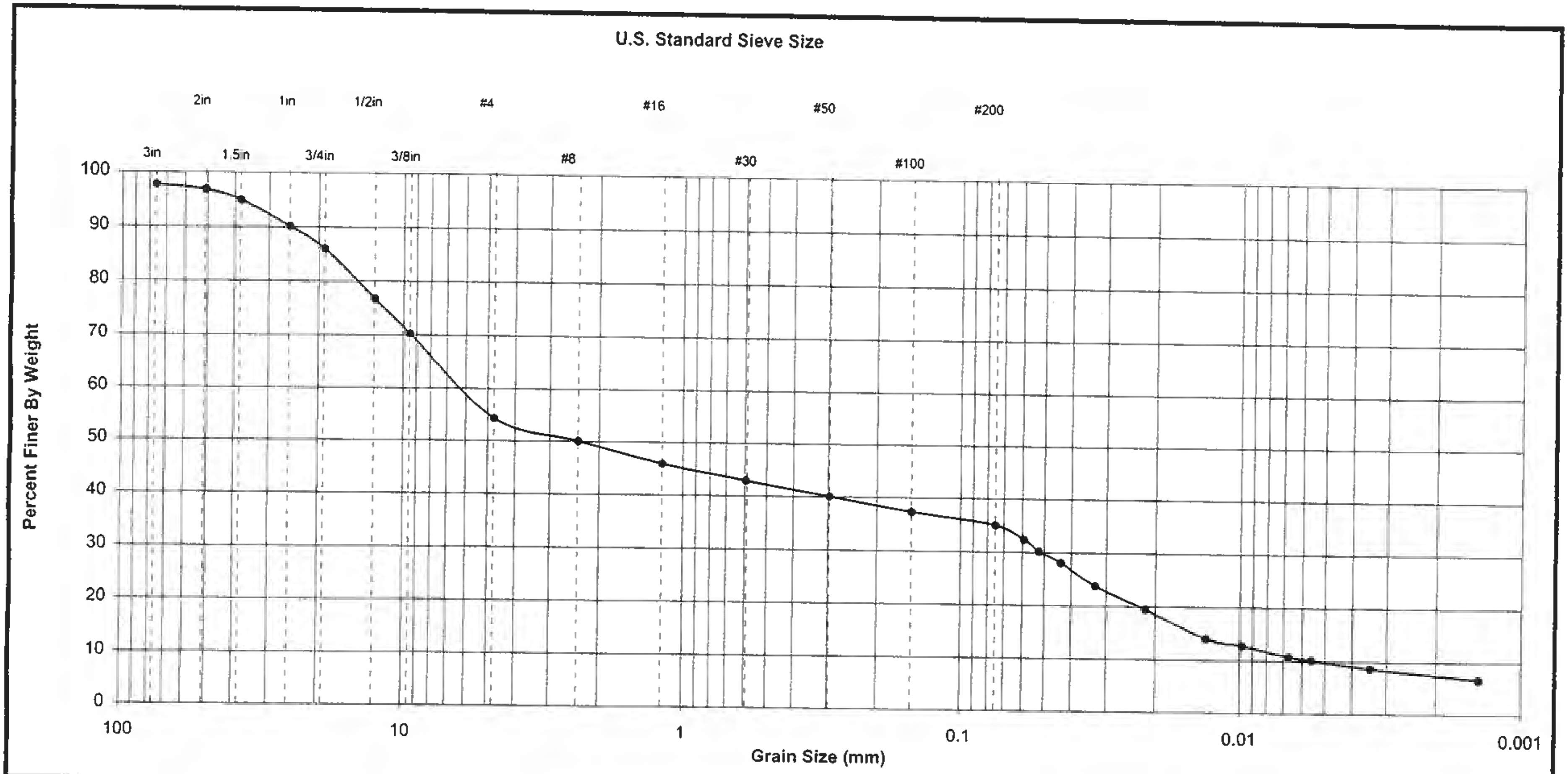
GRAIN SIZE ANALYSIS - ASTM D 422



Boring / Sample No.	Initial Dry Density (pcf)	Initial Moist. (%)	Test Dry Density (pcf)	Test Moist. (%)	Permeability, K (cm/sec)	LL	PL	PI	Unified Soil Class.	Description
Stockpile #1 & #2									ML	Clayey Silt W. Siltstone

GeoLogic Associates

MAXIMUM DENSITY TEST – ASTM D1557

MAXIMUM DENSITY TEST ASTM D1557

Job Name SouthCoast LF

Job No. 2001-082

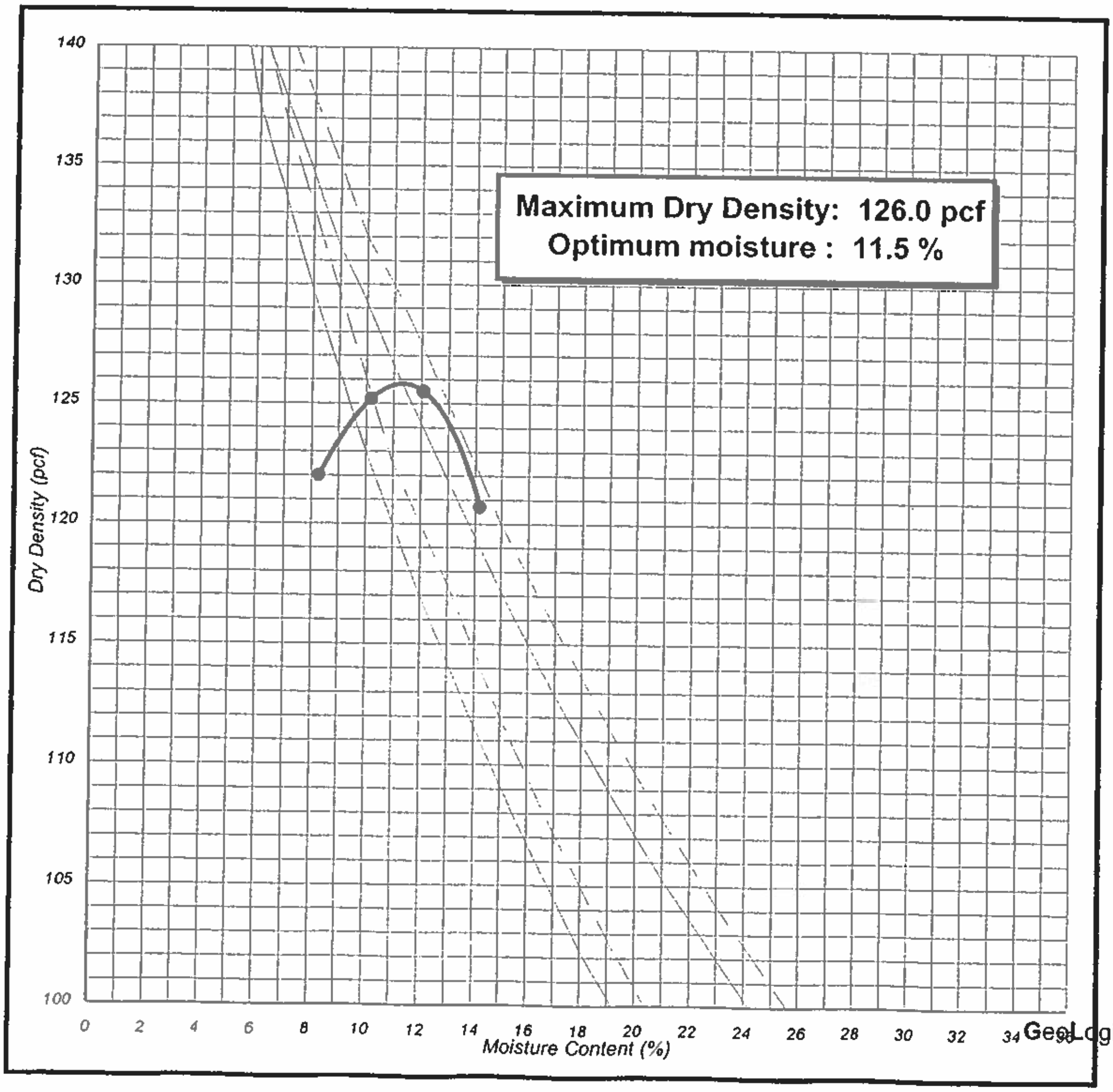
Boring/Sample No. B-1 & B-2 Composite

Description: Rusty Brown, Silty Clay w. Siltstone Frgmts (-3/4")

Date: 11/27/02

By: LD

Method: C	Mold Volume (cf): **0.0750**	Blows: **56**		Layers: **5**	
Specimen	A	B	C	D	E
Total Wet Weight (lbs)	**7679**	**7684**	**7776**	**7485**	
Weight of Mold (lbs)	2990	2990	2990	2990	
Wet Weight of Soil (lbs)	4689	4694	4786	4495	
Wet Density (pcf)	137.8	138.0	140.7	132.1	
Moisture Can No.					
Dry Weight					
Moisture Content (%)	**14.2**	**10.2**	**12.1**	**8.3**	
Dry Density (pcf)	120.7	125.2	125.5	122.0	



Maximum Dry Density: 126.0 pcf
Optimum moisture : 11.5 %

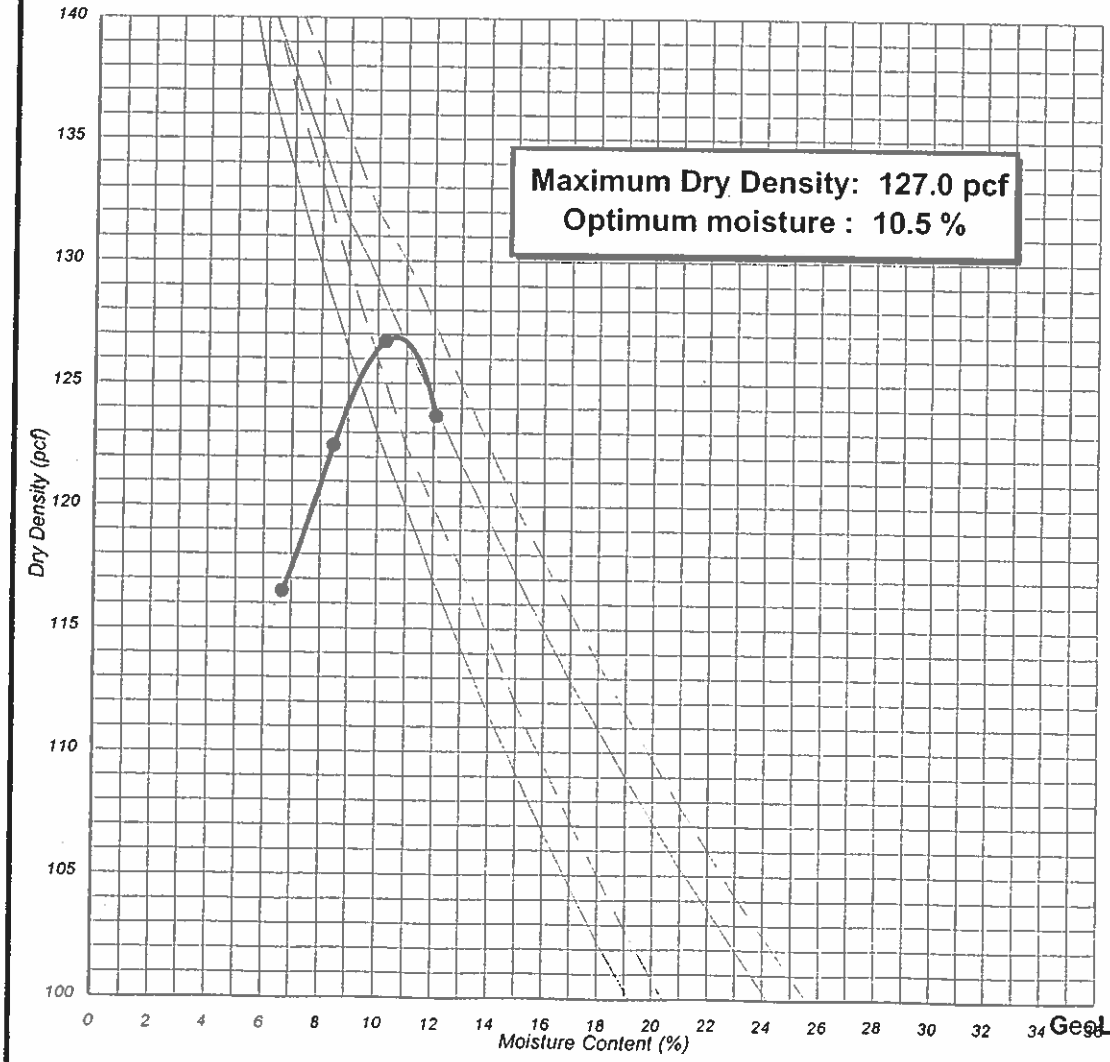
GeoLogic Associates

MAXIMUM DENSITY TEST ASTM D1557

Job Name SouthCoast LF
Job No. 2001-082
Boring/Sample No. TP-1, 5, 10, 16 Composite
Description: D. Brown, Clayey Silt w. Siltstone Frgmts (-3/4")

Date: 11/27/02
By: LD

Method: C	Mold Volume (cf): **0.0750**		Blows: **56**	Layers: **5**	
Specimen	A	B	C	D	E
Total Wet Weight (lbs)	**7744**	**7706**	**7510**	**7219**	
Weight of Mold (lbs)	2990	2990	2990	2990	
Wet Weight of Soil (lbs)	4754	4716	4520	4229	
Wet Density (pcf)	139.8	138.6	132.9	124.3	
Moisture Can No.					
Dry Weight					
Moisture Content (%)	**10.3**	**12.1**	**8.5**	**6.7**	
Dry Density (pcf)	126.7	123.7	122.5	116.5	



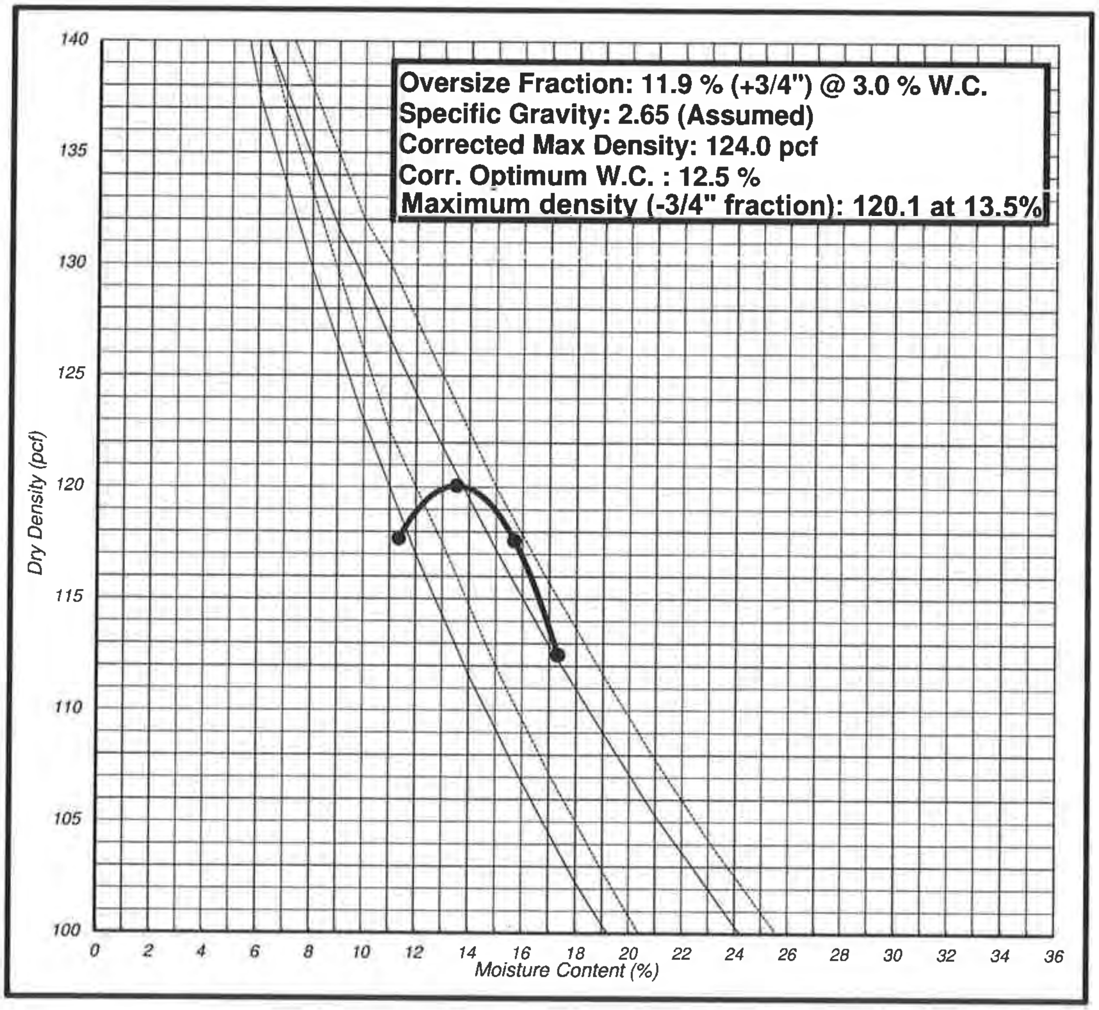
Maximum Dry Density: 127.0 pcf
Optimum moisture : 10.5 %

GeoLogic Associates

MAXIMUM DENSITY TEST ASTM D1557

Job Name	South Coast LF Composite of 3 locations	**Date:**	4/6/2012
Job No.	2012-0007	**By:**	LD
Boring/Sample No.	A (04-05-12)		
Description:	Brown, Silty Clay w. Siltstone & Gravel		

Method: **C**	Mold Volume (cf): **0.0750**	Blows: **56**		Layers: **5**	
Specimen	A	B	C	D	E
Total Wet Weight (lbs)	**16.80**	**16.81**	**16.42**	**16.49**	
Weight of Mold (lbs)	6.59	6.59	6.59	6.59	
Wet Weight of Soil (lbs)	10.20	10.22	9.83	9.90	
Wet Density (pcf)	136.0	136.3	131.1	132.0	
Moisture Can No.					
Dry Weight					
Moisture Content (%)	**15.7**	**13.5**	**11.3**	**17.3**	
Dry Density (pcf)	117.6	120.1	117.7	112.5	

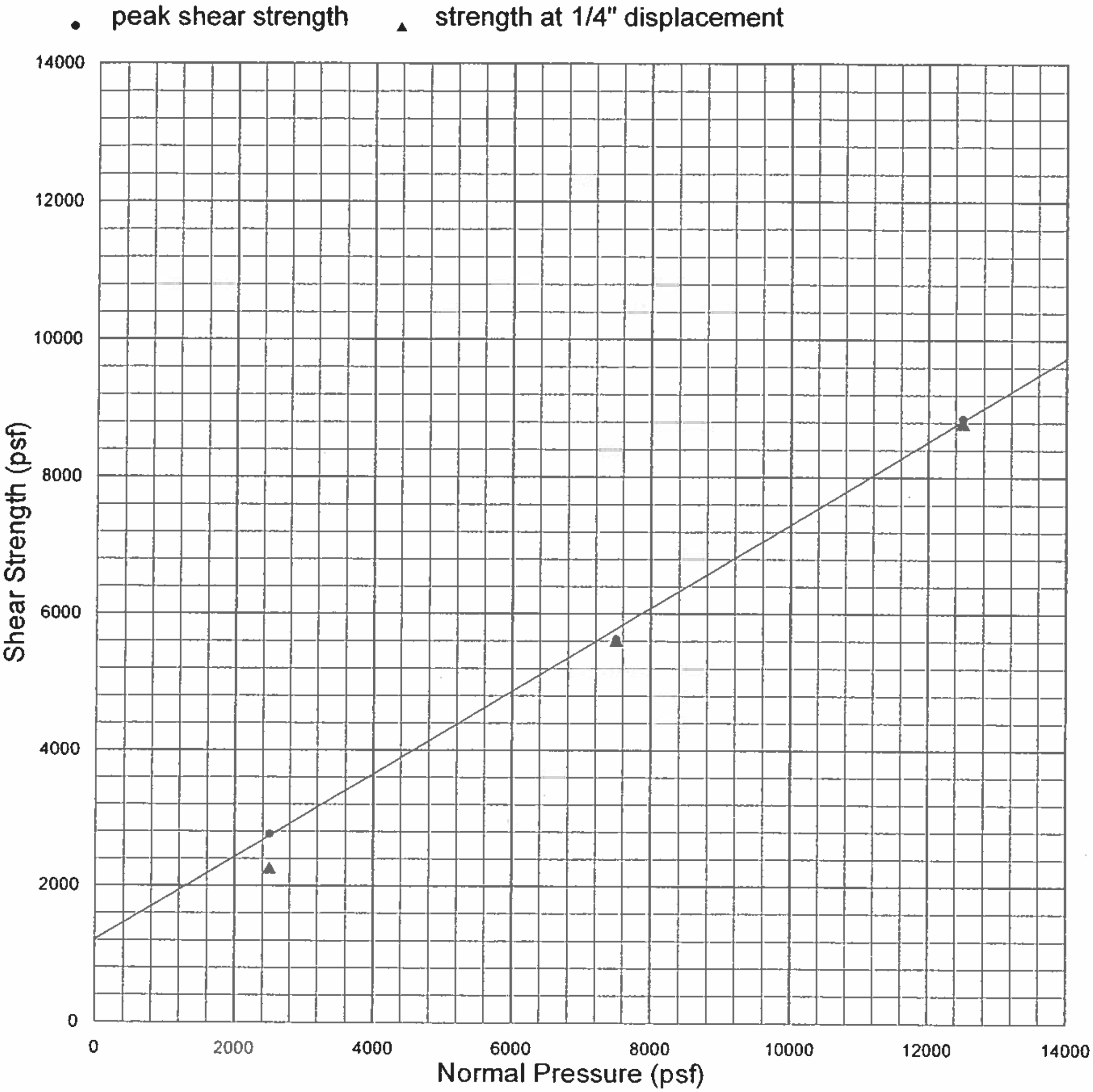


Oversize Fraction: 11.9 % (+3/4") @ 3.0 % W.C.
Specific Gravity: 2.65 (Assumed)
Corrected Max Density: 124.0 pcf
Corr. Optimum W.C. : 12.5 %
Maximum density (-3/4" fraction): 120.1 at 13.5%

Geo-Logic
ASSOCIATES

DIRECT SHEAR TEST – ASTM D3080

DIRECT SHEAR TEST - ASTM D-3080

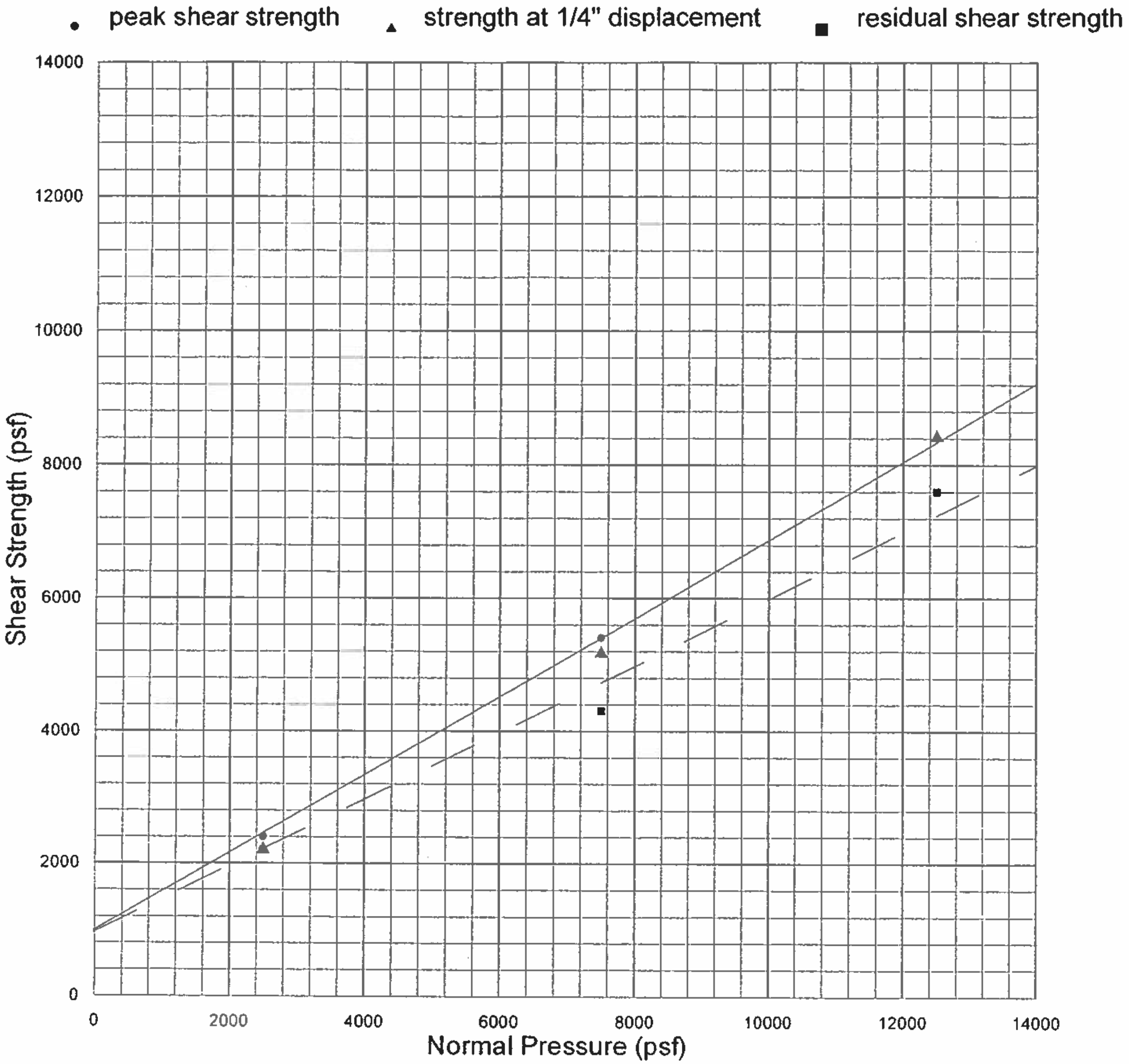


Strain Rate: 0.0042 in. / min.

Sample	**Type**	**Description**	**Dry Density (pcf)**	**Initial W.C. (%)**
B-1/1 @ 5'	Undisturbed & Saturated	Clayey Silt w. Rock & Siltstone Frgmts	108.2	17.2

Normal Pressure (psf)	Peak Strength (psf)	Ultimate Strength (psf)
2500	2760 @ 0.0865"	2230
7500	5630 @ 0.1270"	5580
12500	8860 @ 0.2130"	8760
	C = 1200 psf	C = 1200 psf
	ϕ = 32 deg.	ϕ = 32 deg.

DIRECT SHEAR TEST - ASTM D-3080

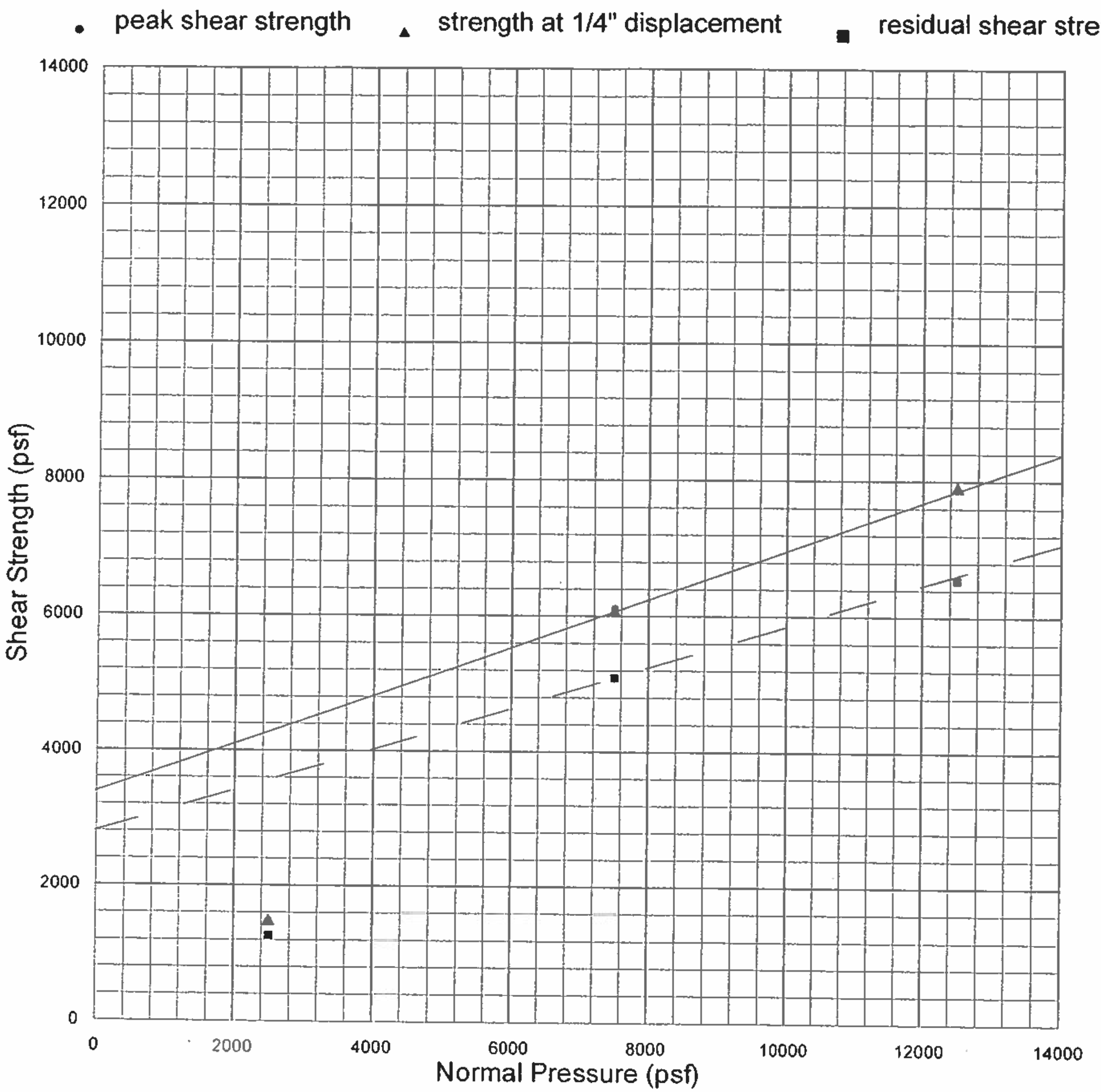


Strain Rate: 0.0042 in. / min.

Sample	**Type**	**Description**	**Dry Density (pcf)**	**Initial W.C. (%)**
B-1/3 @ 15'	Undisturbed & Saturated	Silty Clay w. Rock Frgmts	102.7	17.9

Normal Pressure (psf)	Peak Strength (psf)	Ultimate Strength (psf)	Residual Strength (psf)
2500	2400 @ 0.1490"	2200	2200
7500	5410 @ 0.1570"	5170	4300
12500	8420 @ 0.2405"	8420	7600
	C = 1000 psf	C = 1000 psf	C = 1000 psf
	ϕ = 30 deg.	ϕ = 30 deg.	ϕ = 26 deg.

DIRECT SHEAR TEST - ASTM D-3080

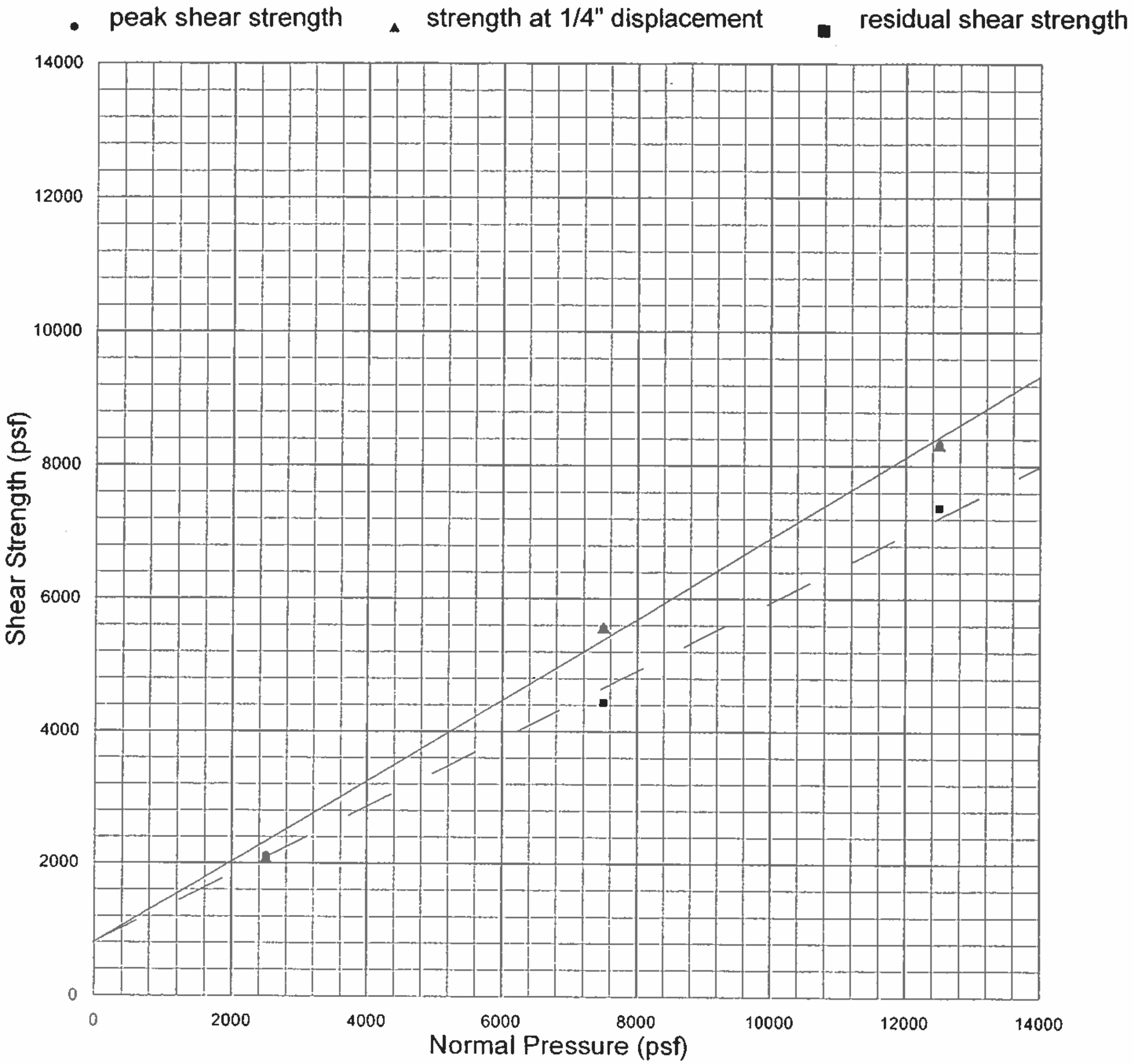


Strain Rate: 0.0042 in. / min.

Sample	**Type**	**Description**	**Dry Density (pcf)**	**Initial W.C. (%)**
B-1/4 @ 20'	Undisturbed & Saturated	Silty Clay w. Rock Frgmts	77.1	21.0

Normal Pressure (psf)	Peak Strength (psf)	Ultimate Strength (psf)	Residual Strength (psf)
2500	1480 @ 0.2370"	1460	1260
7500	6110 @ 0.2320"	6050	5090
12500	7880 @ 0.2430"	7880	6540
	C = 3400 psf	C = 3400 psf	C = 2800 psf
	ϕ = 19 deg.	ϕ = 19 deg.	ϕ = 17 deg.

DIRECT SHEAR TEST - ASTM D-3080



Strain Rate: 0.0042 in. / min.

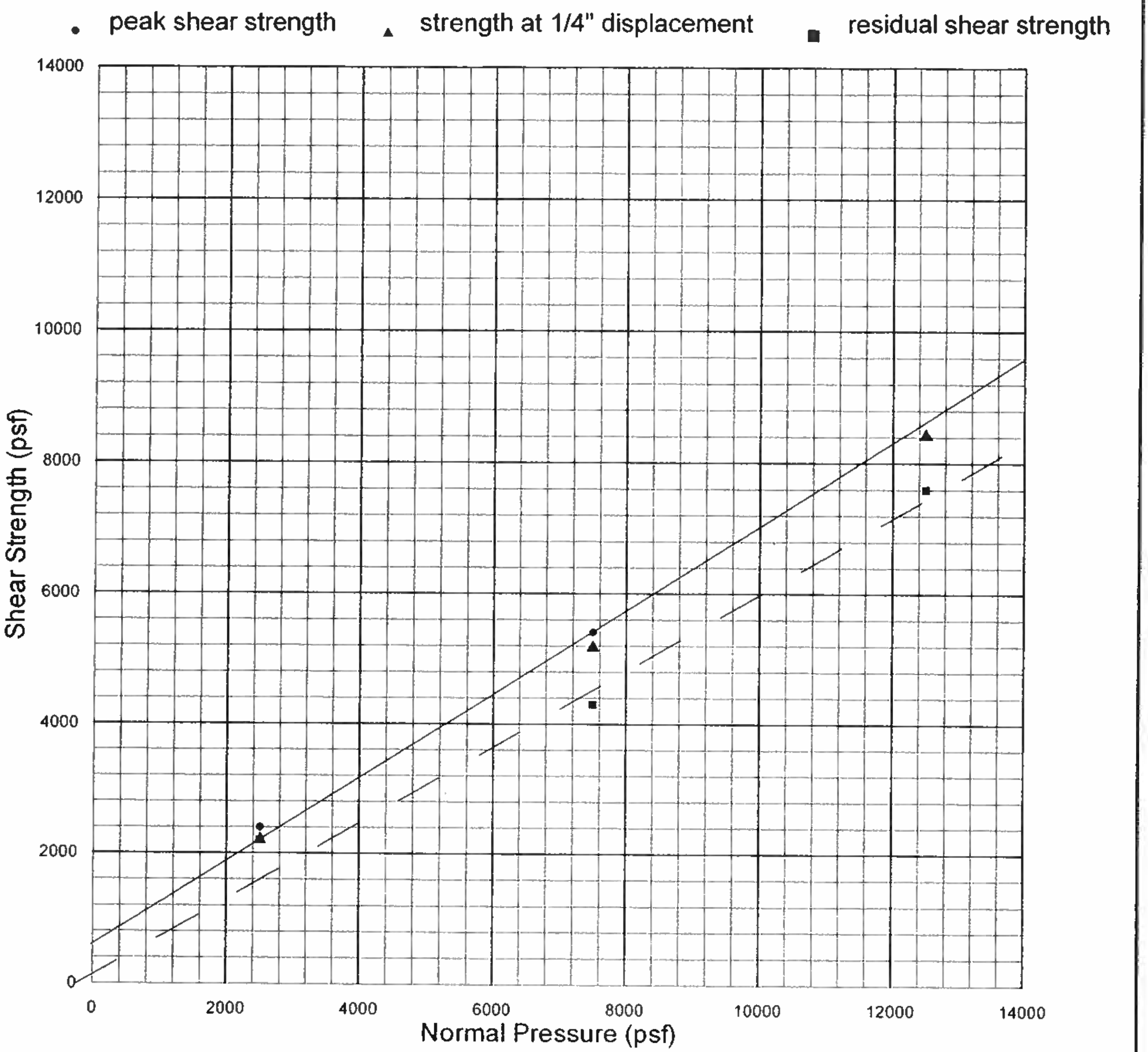
Sample	Type	Description	Dry Density (pcf)	Initial W.C. (%)
B-4/1 @ 5'	Remolded & Saturated	Silty Clay	110.9	13.8

Normal Pressure (psf)	Peak Strength (psf)	Ultimate Strength (psf)	Residual Strength (psf)
2500	2120 @ 0.1930"	2050	2050
7500	5560 @ 0.2470"	5540	4430
12500	8330 @ 0.2300"	8290	7370
	C = 800 psf	C = 800 psf	C = 800 psf
	ϕ = 31 deg.	ϕ = 31 deg.	ϕ = 27 deg.

Job No. 2001-082

DIRECT SHEAR TEST - ASTM D-3080

SouthCoast LF - Mendocino County



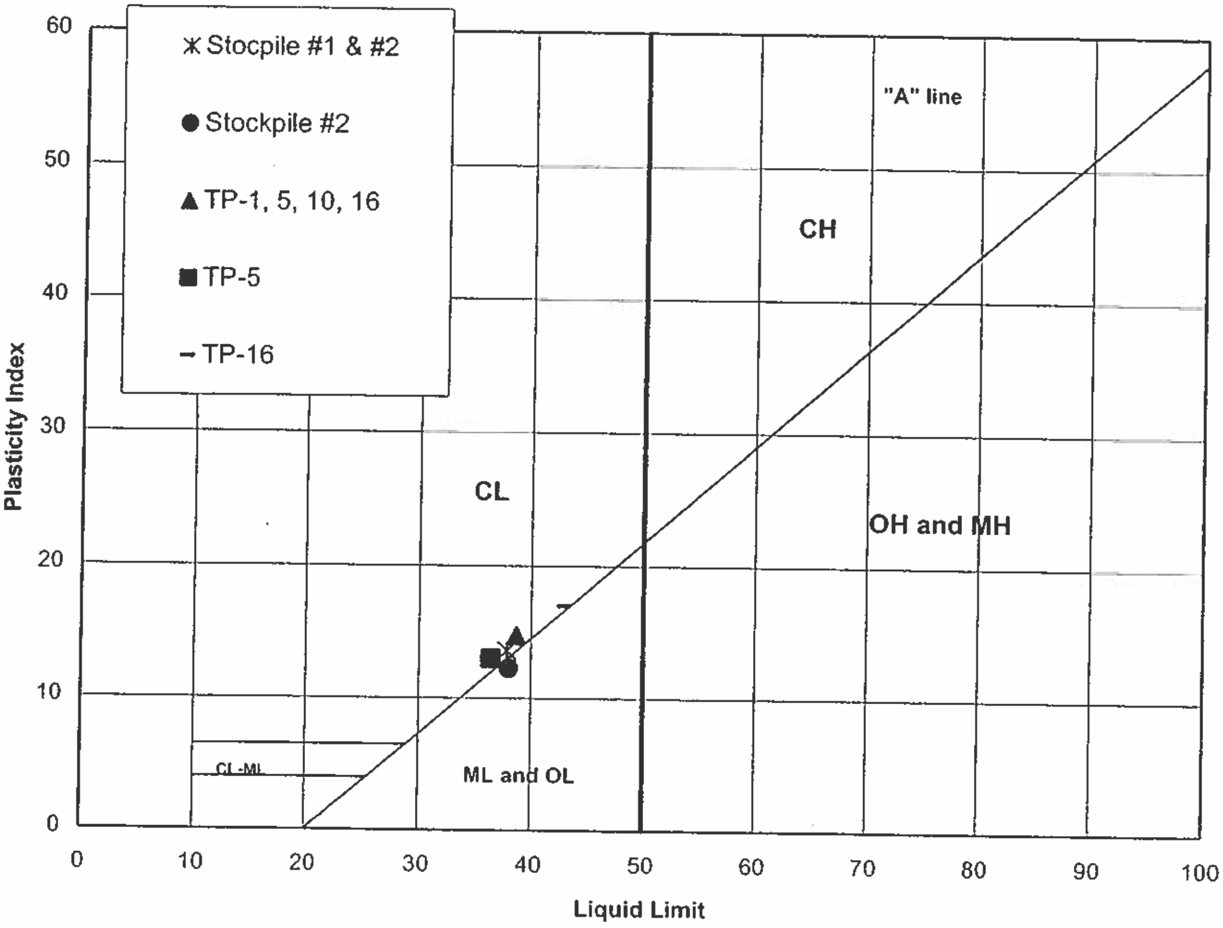
Strain Rate: 0.0042 in. / min.

Sample	Type	Description	Dry Density (pcf)	Initial W.C. (%)
B-2/1 @ 10'	Remolded & Saturated	Silty Clay	103.5	22.0

Normal Pressure (psf)	Peak Strength (psf)	Ultimate Strength (psf)	Residual Strength (psf)
2500	2280 @ 0.1585"	2150	1860
7500	5470 @ 0.1550"	5290	5290
12500	9240 @ 0.2260"	9240	8280
	C = 600 psf	C = 600 psf	C = 150 psf
	ϕ = 33 deg.	ϕ = 33 deg.	ϕ = 30 deg.

PLASTICITY INDEX – ASTM D4318

PLASTICITY INDEX _ ASTM D4318



Sample	Depth	LL	PL	PI	USCS	Material Description
Stocpile #1 & #2		38	24	14	CL	CLAYEY SILT
Stockpile #2		38	26	12	ML	CLAYEY SILT
TP-1, 5, 10, 16		39	24	15	CL	CLAYEY SILT
TP-5		36	23	13	CL	CLAYEY SILT
TP-16		43	26	17	CL	CLAYEY SILT

Job Name SouthCoast LF

Date: 12-9-02

Job No.: 2001-082

HYDRAULIC CONDUCTIVITY – ASTM D5084

HYDRAULIC CONDUCTIVITY - ASTM D5084

Job Name SouthCoast LF　　Job No.: 2001-082　　Sample: Stockpile #1 & #2　　By:LD

(Remolded @ 93% Max & 120& OMC)

Sample Type: ☐ Undisturbed ● Remolded

Soil Type: Brown Mottled, Silty Clay w. Siltstone Frgmts

		Final	
Ini. Height (in.) :	2		
Ini. Diam. (in.) :	2.41	Height (in.) :	2.016
Wet + Tare (grams):	319.2	Diam. (in.) :	2.41
Tare (grams) :	0	Wet Weight (gr.):	333
Water Content (%) :	14	Water Content (%):	18.9
Dry Density (pcf):	116.9	Dry Density (pcf):	116.0

Saturation

Cell Pressure	Back P.	Pore P.	B	Cell Burette	Sample Burette
10	7			-1.7	5.9
20	17			-1	4.3
30	27			-0.1	1.4
40	37	37		0.2	1

Consolidation

Cell Pressure (psi)	Back Pressure (Top)	Back Pressure (Bottom)	Pressure Difference (psi)	Top Burette (cc)	Bottom Burette (cc)	Date&Time Start/Stop	h (cm) h_0/h_1	Q (cc) Top/Bott.	Test Time, t (sec)	Coefficient of Permeability, k (cm/sec)	Remarks (L/A)
40	37.5	36.5	1	0	24	7:12:00	27.48	5.7		7.0E-07	5.12064
				5.7	18.3	11:30:40	14.427	5.7	15520		29.4151573
40	37.5	36.5	1	0	24	11:32:25	27.48	5.9		6.7E-07	
				5.9	18.1	16:12:00	13.969	5.9	16775		
40	37.5	36.5	1	0	24	6:58:00	27.48	2.8		6.4E-07	
				2.8	21.6	9:01:35	21.526	2.4	7415		
40	37.5	36.5	1	2.8	21.6	9:02:00	21.526	7.1		6.6E-07	
				9.9	14.2	15:24:15	4.9235	7.4	22935		
										6.7E-07	$k = \frac{aL}{2At} ln \frac{h_o}{h_i}$
										(Average)	

GeoLogic Associates

HYDRAULIC CONDUCTIVITY - ASTM D5084

Job Name SouthCoast LF

Job No.: 2001-082

Sample: Stockpile #1 & #2

By:LD

Sample Type: ☐ Undisturbed ● Remolded

Soil Type: Brown Mottled, Silty Clay w. Siltstone Frgmts

Ini. Height (in.) :	2	**Final**	
Ini. Diam. (in.) :	2.41	Height (in.) :	2.058
Wet + Tare (grams):	388.2	Diam. (in.) :	2.435
Tare (grams) :	84	Wet Weight (gr.):	329.9
Water Content (%) :	12	Water Content (%):	21.5
Dry Density (pcf):	113.4	Dry Density (pcf):	108.0

Saturation

Cell Pressure	Back P.	Pore P.	B	Cell Burette	Sample Burette
10	7			-3.4	9.5
20	17			-0.2	7.2
30	27			-0.7	2.7
40	37	37		0	1.9

Consolidation

Cell Pressure (psi)	Back Pressure (Top)	Back Pressure (Bottom)	Pressure Difference (psi)	Top Burette (cc)	Bottom Burette (cc)	Date&Time Start/Stop	h (cm) h_0/h_1	Q (cc) Top/Bott.	Test Time, t (sec)	Coefficient of Permeability, k (cm/sec)	Remarks (L/A)
40	37.5	36.5	1	0	24	6:41:30	27.48	7.3		3.5E-06	5.22732
				7.3	16.8	7:49:35	10.8775	7.2	4085		30.02859559
40	37.5	36.5	1	0	24	7:51:40	27.48	8.5		3.4E-06	
				8.5	15.5	9:14:04	8.015	8.5	4944		
40	37.5	36.5	1	0	24	9:16:25	27.48	19.8		3.2E-06	
				19.8	4.3	13:21:00	-17.7475	19.7	14675		
40	37.5	36.5	1	0	24	13:23:20	27.48	9.2		3.2E-06	
				9.2	14.9	15:00:15	6.5265	9.1	5815		
40	37.5	36.5	1	0	24	15:03:17	27.48	5.5		3.1E-06	
				5.5	18.6	16:00:00	14.9995	5.4	3403		
										3.3E-06 **(Average)**	$k = \frac{aL}{2At} ln \frac{h_o}{h_i}$

HYDRAULIC CONDUCTIVITY - ASTM D5084

Job Name SouthCoast LF   Job No.: 2001-082   Sample: TP-1, 5, 10, 16   By:LD

Sample Type: ☐ Undisturbed ● Remolded

Soil Type: Brown Mottled, Clayey Silt w. Siltstone Frgmts

		Final	
Ini. Height (in.) :	2		
Ini. Diam. (in.) :	2.41	Height (in.) :	2.058
Wet + Tare (grams):	388.7	Diam. (in.) :	2.435
Tare (grams) :	84	Wet Weight (gr.):	329.9
Water Content (%) :	11	Water Content (%):	20.2
Dry Density (pcf):	114.6	Dry Density (pcf):	109.2

Saturation

Cell Pressure	Back P.	Pore P.	B	Cell Burette	Sample Burette
10	7			-2.8	10.2
20	17			-0.1	4.4
30	27			-0.6	2.1
40	37	37		-0.6	1.9

Consolidation

Cell Pressure (psi)	Back Pressure (Top)	Back Pressure (Bottom)	Pressure Difference (psi)	Top Burette (cc)	Bottom Burette (cc)	Date&Time Start/Stop	h (cm) h_0/h_1	Q (cc) Top/Bott.	Test Time, t (sec)	Coefficient of Permeability, k (cm/sec)	Remarks (L/A)
40	37.5	36.5	1	0	24	6:41:00	27.48	8.1		3.9E-06	5.22732
				8.1	15.9	7:49:05	8.931	8.1	4085		30.02859559
40	37.5	36.5	1	0	24	7:51:05	27.48	9.1		3.7E-06	
				9.1	14.9	9:13:37	6.641	9.1	4952		
40	37.5	36.5	1	0	24	9:16:13	27.48	20.9		3.5E-06	
				20.9	2.9	13:20:30	-20.61	21.1	14657		
40	37.5	36.5	1	0	24	13:23:03	27.48	10.4		3.7E-06	
				10.4	13.6	14:59:45	3.664	10.4	5802		
40	37.5	36.5	1	0	24	15:03:00	27.48	6.4		3.4E-06	
				6.4	18.3	15:59:30	13.6255	5.7	3390		
										3.7E-06 **(Average)**	$k = \frac{aL}{2At} \ln \frac{h_o}{h_i}$

INTERFACE SHEAR TEST RESULTS – ASTM D5321

Precision Geosynthetic Laboratories

CLIENT: GEOLOGIC ASSOCIATES
PROJECT: South Coast Landfill / Job# 2002-072
INTERFACE SHEAR TEST RESULTS
(PGL Job No. G030242)

SAMPLE IDENTIFICATIONS:

SAMPLE ID	PRECISION CONTROL NUMBER	DATE RECEIVED	ORIGIN OF MATERIAL
60mil Textured LLDPE Geomembrane (R# 107112879)	86154	4/2/03	GSE, TX.
Soil #1	86180	4/4/03	Geologic Assoc.
Soil #2	86181	4/4/03	Geologic Assoc.
Gravel #3	86182	4/4/03	Geologic Assoc.

TESTS REQUIRED:

TEST METHOD	DESCRIPTION
ASTM D5321	Interface Shear

TEST CONDITIONS: The samples were conditioned for a minimum one hour in the laboratory at 22 ± 2°C (71.6 ± 3.6°F) and at 60 ± 10% relative humidity prior to test.

TEST RESULTS:

The test results are summarized in Tables 1 through 3. The units in which the data are reported are included on these tables.

PRECISION GEOSYNTHETIC LABORATORIES

Cora B. Queja
Vice President

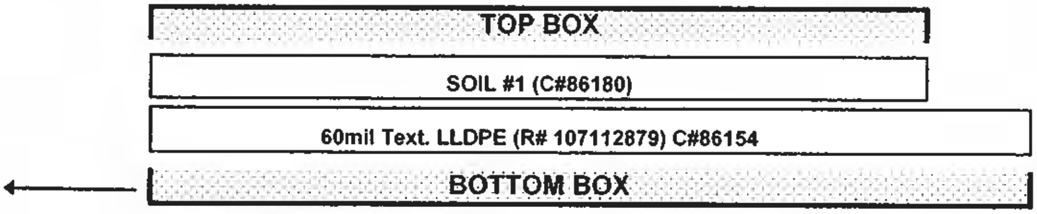
TABLE 1

CLIENT: Geologic Associates
PROJECT: South Coast Landfill / Proj# 2002-072

INTERFACE SHEAR TEST RESULT (ASTM D 5321)
PGL Job No. G030242

QC'd by:
Date: 4/11/03

TEST CONFIGURATION # 1



TEST CONDITIONS:

SAMPLE PREPARATION:

1. Specimens were cut along machine direction to 14" x 19" for the upper box, and 14" x 17" for the lower box, with an effective test area of 12" x 12".
2. Soil was compacted to 113.9 pcf @ 11.50% moisture content, forming 2 inch layer in the top box.

HYDRATION:

1. No Hydration

CONSOLIDATION:

1. Each set of specimen was consolidated under Wet/Spray condition for 1 hr @ normal load before shearing.
2. Normal loads were applied using Bladder

SHEAR TEST:

1. Shear test was conducted @ 0.04 in/ min.
2. Sheared @ minimum 2.6 inch horizontal displacement.
3. The test specimens were sheared in Wet/Spray condition
4. Test were performed in general accordance with ASTM D6243-98 / ASTM D5321-92 using Brainard-Kilman LG-112 Direct Shear machine with effective area of 12 in X 12 in.

TEST RESULTS:

			PEAK STRENGTH		2.6 " DISPLACEMENT STRENGTH	
Normal Stress (psi)	Normal Stress (psf)	FINAL MC (Soil) (%)	SHEAR STRESS (psf)	PEAK SECANT ANGLE (degrees)	SHEAR STRESS (psf)	2.6 " DISPLACEMENT SECANT ANGLE (degrees)
0.69	100	12.99	160	58	146	56
1.74	250	12.24	323	52	284	49
3.47	500	12.11	474	43	469	43

COHESION (psf):	101.82	72.8
COEFFICIENT OF FRICTION:	0.77	0.80
FRICTION ANGLE(degrees):	37.5	38.7

NOTE: The friction angles and cohesion results given here are based on mathematically determined best fit line.

OBSERVATIONS:

See Figure #1 and #2

PGL Job Number: G030242 Config.# 1
Project Name: South Coast Landfill / Proj# 2002-072
SOIL #1 (C#86180) / 60mil Text. LLDPE (R# 107112879) C#86154
QC'd By:

Figure #1
Normal Stress/ Interface Stress

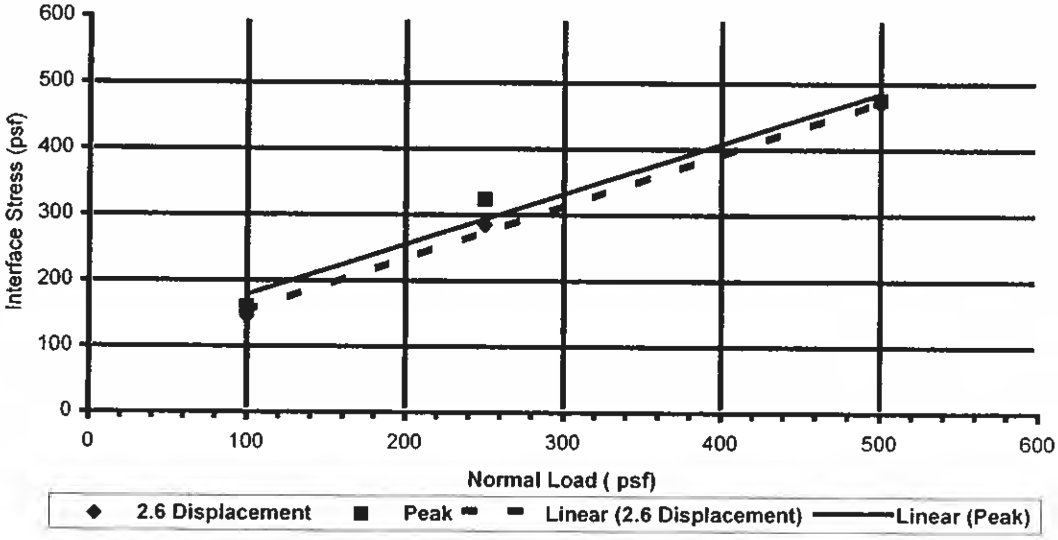


Figure #2
Shear Stress/ Displacement Curve

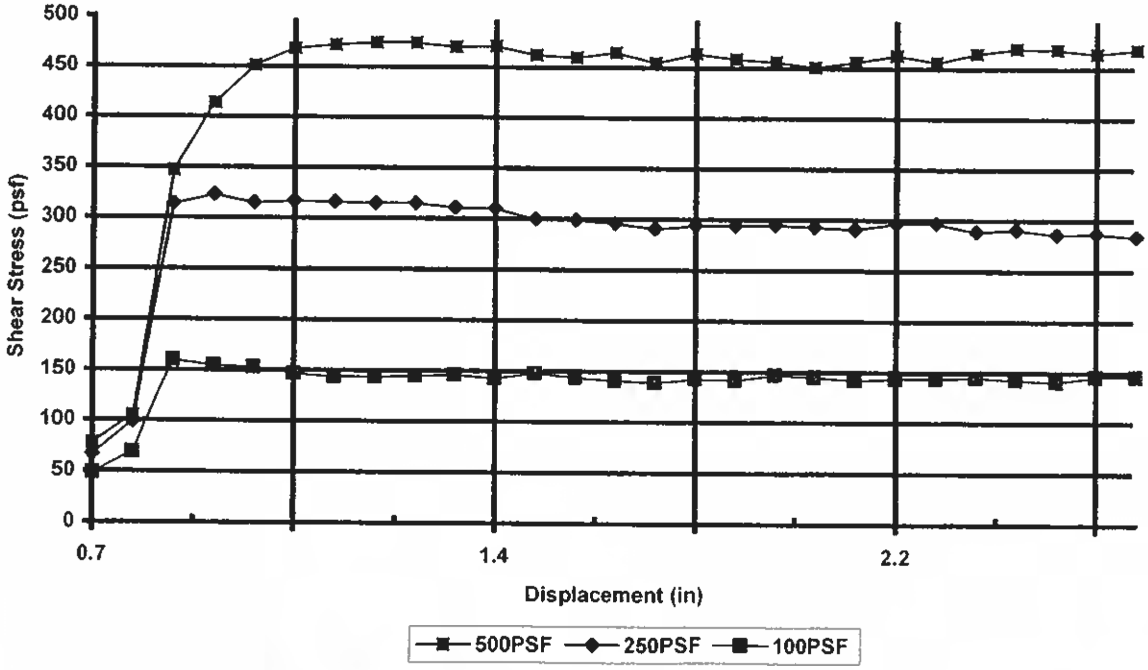


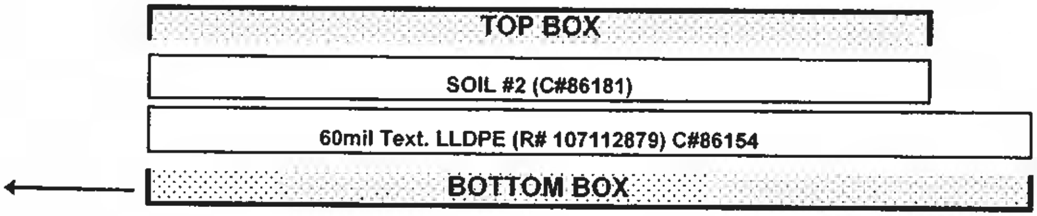
TABLE 2

CLIENT: Geologic Associates
PROJECT: South Coast Landfill / Proj# 2002-072

INTERFACE SHEAR TEST RESULT (ASTM D 5321)
PGL Job No. G030242

QC'd by:
Date: 4/11/03

TEST CONFIGURATION # 2



TEST CONDITIONS:

SAMPLE PREPARATION:

1. Specimens were cut along machine direction to 14" x 19" for the upper box, and 14" x 17" for the lower box, with an effective test area of 12" x 12".
2. Soil was compacted to 111.6 pcf @ 12.50% moisture content, forming 2 inch layer in the top box.

HYDRATION:

1. No Hydration

CONSOLIDATION:

1. Each set of specimen was consolidated under Wet/Spray condition for 1 hr @ normal load before shearing.
2. Normal loads were applied using Bladder

SHEAR TEST:

1. Shear test was conducted @ 0.04 in/ min.
2. Sheared @ minimum 2.6 inch horizontal displacement.
3. The test specimens were sheared in Wet/Spray condition
4. Test were performed in general accordance with ASTM D6243-98 / ASTM D5321-92 using Brainard-Kilman LG-112 Direct Shear machine with effective area of 12 in X 12 in.

TEST RESULTS:

			PEAK STRENGTH		2.6 " DISPLACEMENT STRENGTH	
Normal Stress (psi)	Normal Stress (psf)	FINAL MC (Soil) (%)	SHEAR STRESS (psf)	PEAK SECANT ANGLE (degrees)	SHEAR STRESS (psf)	2.6 " DISPLACEMENT SECANT ANGLE (degrees)
0.69	100	13.61	215	65	209	64
1.74	250	13.6	251	45	239	44
3.47	500	13.05	355	35	263	28

COHESION (psf):	172.59	199.88
COEFFICIENT OF FRICTION:	0.36	0.13
FRICTION ANGLE(degrees):	19.6	7.5

NOTE: The friction angles and cohesion results given here are based on mathematically determined best fit line.

OBSERVATIONS:

See Figure #1 and #2

PGL Job Number: **G030242** **Config.#** **2**
Project Name: **South Coast Landfill / Proj# 2002-072**
SOIL #2 (C#86181) / 60mil Text. LLDPE (R# 107112879) C#86154
QC'd By:

Figure #1
Normal Stress/ Interface Stress

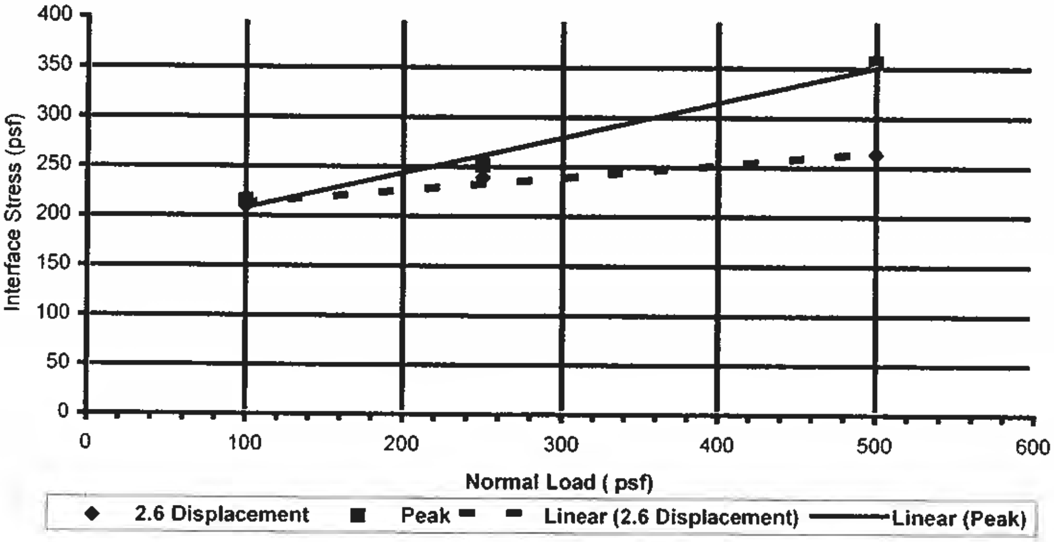
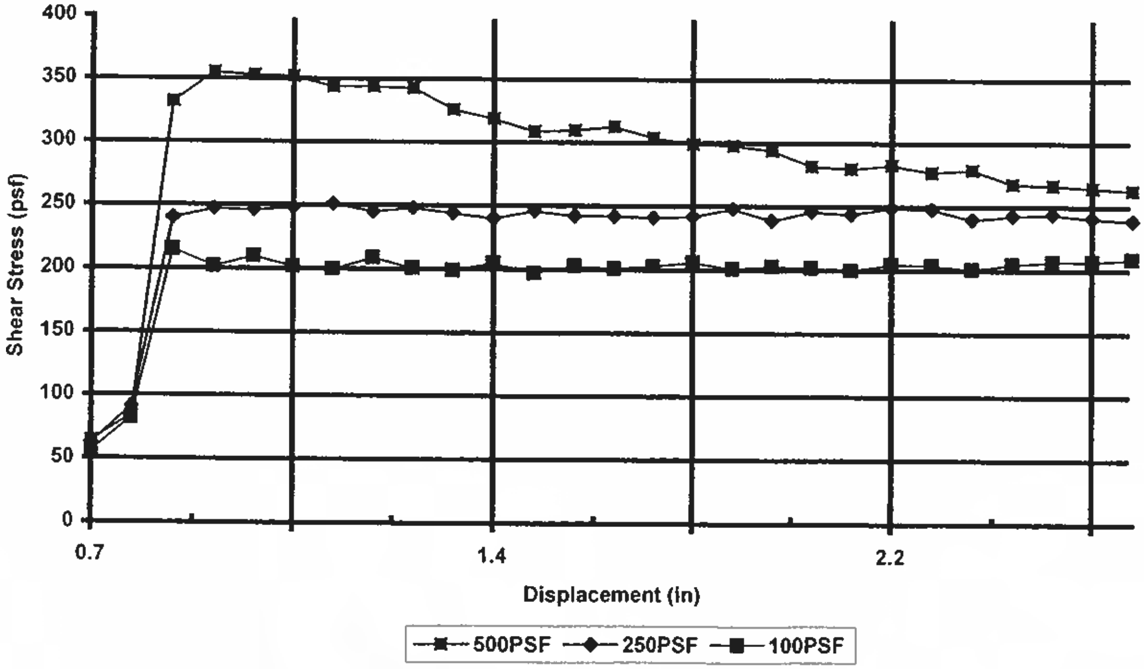


Figure #2
Shear Stress/ Displacement Curve



Precision Geosynthetic Laboratories

ISO 9001 - 2000

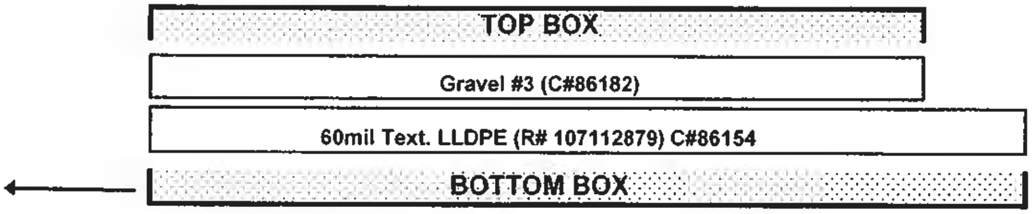
TABLE 3

CLIENT: Geologic Associates
PROJECT: South Coast Landfill / Proj# 2002-072

INTERFACE SHEAR TEST RESULT (ASTM D 5321)
PGL Job No. G030242

QC'd by:
Date: 4/18/03

TEST CONFIGURATION # 3



TEST CONDITIONS:

SAMPLE PREPARATION:

1. Specimens were cut along machine direction to 14" x 19" for the upper box, and 14" x 17" for the lower box, with an effective test area of 12" x 12".
2. Soil was compacted to 86.1 pcf @ 3.60% moisture content, forming 2 inch layer in the top box.

HYDRATION:

1. No Hydration

CONSOLIDATION:

1. Each set of specimen was consolidated under Wet/Spray condition for 2 hrs @ normal load before shearing.
2. Normal loads were applied using Bladder

SHEAR TEST:

1. Shear test was conducted @ 0.04 in/ min.
2. Sheared @ minimum 2.0 inch horizontal displacement.
3. The test specimens were sheared in Wet/Spray condition
4. Test were performed in general accordance with ASTM D6243-98 / ASTM D5321-92 using Brainard-Kilman LG-112 Direct Shear machine with effective area of 12 in X 12 in.

TEST RESULTS:

Normal Stress		PEAK STRENGTH		2.0 " DISPLACEMENT STRENGTH	
(psi)	(psf)	SHEAR STRESS (psf)	PEAK SECANT ANGLE (degrees)	SHEAR STRESS (psf)	2.0 " DISPLACEMENT SECANT ANGLE (degrees)
0.69	100	163	58	147	56
1.74	250	255	46	228	42
3.47	500	449	42	432	41

	PEAK STRENGTH	2.0 " DISPLACEMENT STRENGTH
COHESION (psf):	85	64
COEFFICIENT OF FRICTION:	0.72	0.72
FRICTION ANGLE(degrees):	35.8	35.9

NOTE: The friction angles and cohesion results given here are based on mathematically determined best fit line.

OBSERVATIONS:

(1) See Figure #1 and #2
(2) Same compacted gravel was used in 3 stresses and consolidated for at least 2 hrs.

Precision Geosynthetic Laboratories

ISO 9001 - 2000

PGL Job Number: G030242 **Config.#** 3
Project Name: South Coast Landfill / Proj# 2002-072
Gravel #3 (C#86182) / 60mil Text. LLDPE (R# 107112879) C#86154
QC'd By: ____________

Figure #1
Normal Stress/ Interface Stress

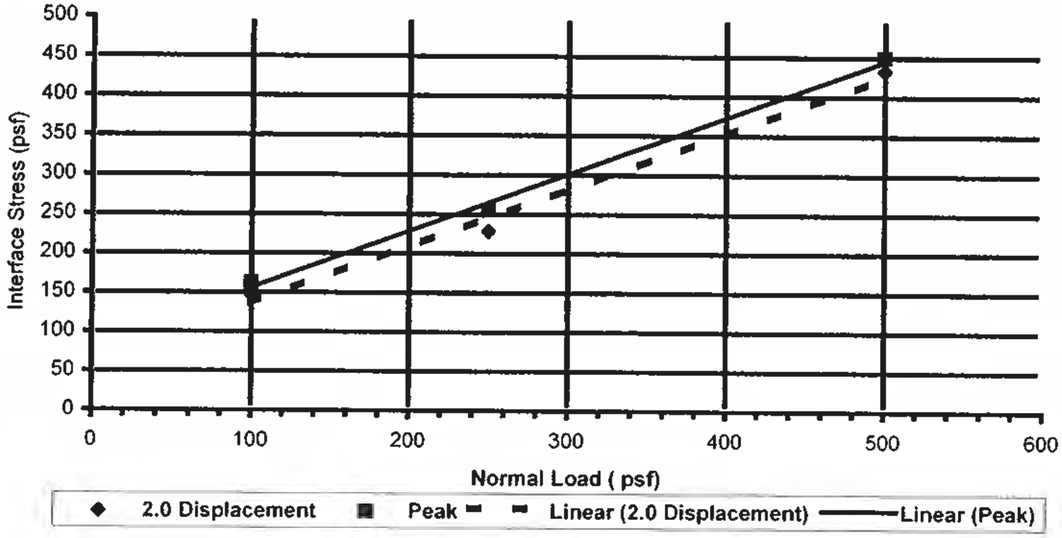
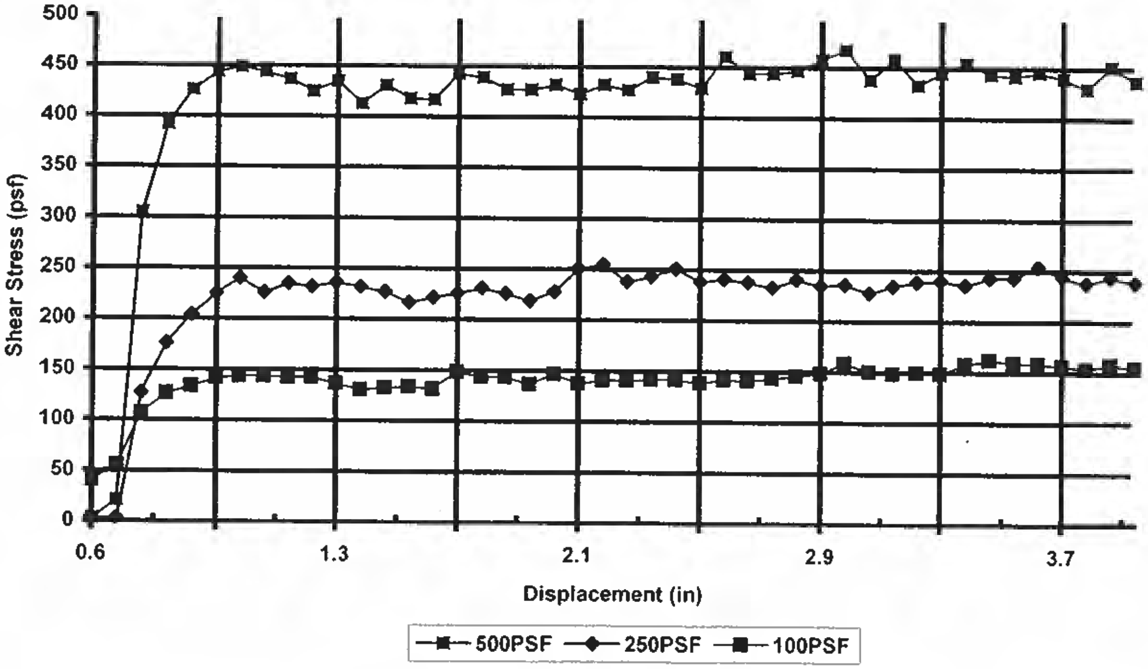
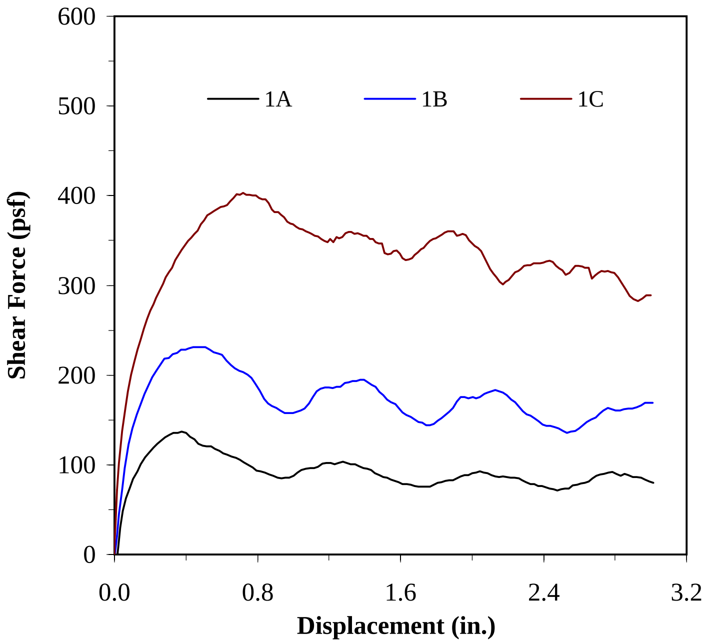


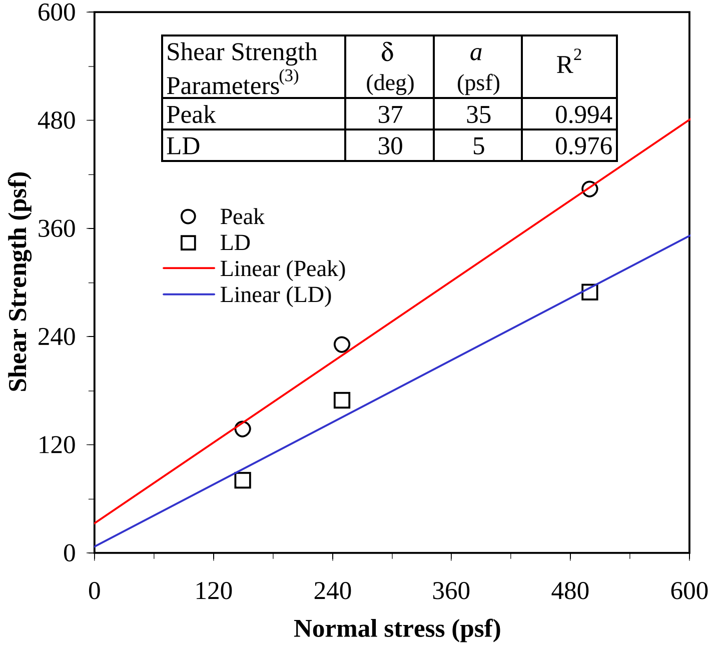
Figure #2
Shear Stress/ Displacement Curve



AGRU AMERICA, INC. - SOUTH COAST LANDFILL
INTERFACE DIRECT SHEAR TESTING (ASTM D 5321)

Upper Shear Box: Concrete sand
Agrutex 121(12 oz) nonwoven geotextile #1116017278 (clamped to the upper shear box) with heart-treated side down /
Agru 50-mil Super Gripnet LLDPE geomembrane # 103553-12 (clamped to the lower shear box) with spike side down
Lower Shear Box: Concrete sand





Shear Strength Parameters$^{(3)}$	δ (deg)	*a* (psf)	R^2
Peak	37	35	0.994
LD	30	5	0.976

Test	Shear	Normal	Shear	Soaking		Consolidation		Drainage Gravel			Brown Clay			GCL		Shear Strength		Failure
No.	Box Size (in. x in.)	Stress (psf)	Rate (in./min)	Stress (psf)	Time (hour)	Stress (psf)	Time (hour)	γ_d (pcf)	ω_i (%)	ω_f (%)	γ_d (pcf)	ω_i (%)	ω_f (%)	ω_i (%)	ω_f (%)	τ_P (psf)	τ_{LD} (psf)	Mode
1A	12 x 12	150	0.04	150	24	-	-	-	-	-	-	-	-	-	-	137	80	(1)
1B	12 x 12	250	0.04	250	24	-	-	-	-	-	-	-	-	-	-	231	169	(1)
1C	12 x 12	500	0.04	500	24	-	-	-	-	-	-	-	-	-	-	403	289	(1)

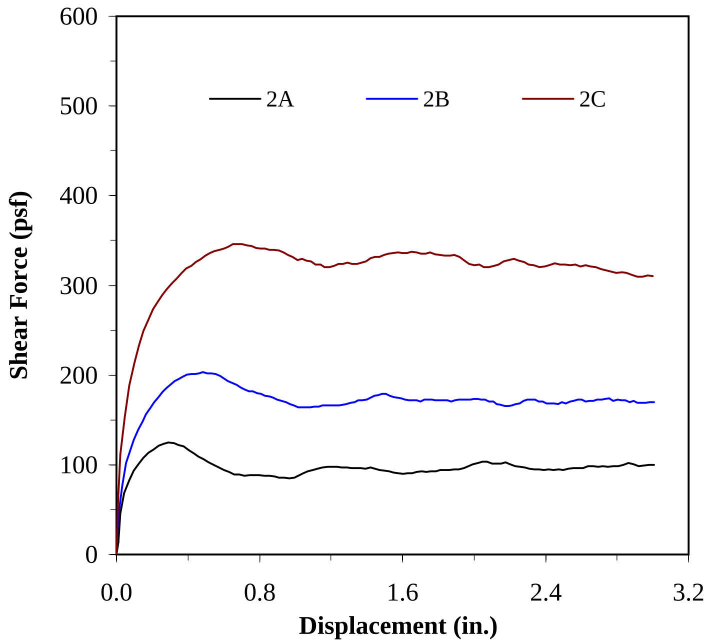
NOTES:
(1) Sliding occurred at the interface between the nonwoven geotextile and stud side of Agru Super Gripnet.
(2) The reported total-stress parameters of friction angle and adhesion were determined from a best-fit line drawn through the test data. Caution should be exercised in using these strength parameters for applications involving normal stresses outside the range of the stresses covered by the test series. The large-displacement (LD) shear strength was calculated using the shear force measured at the end of the test.

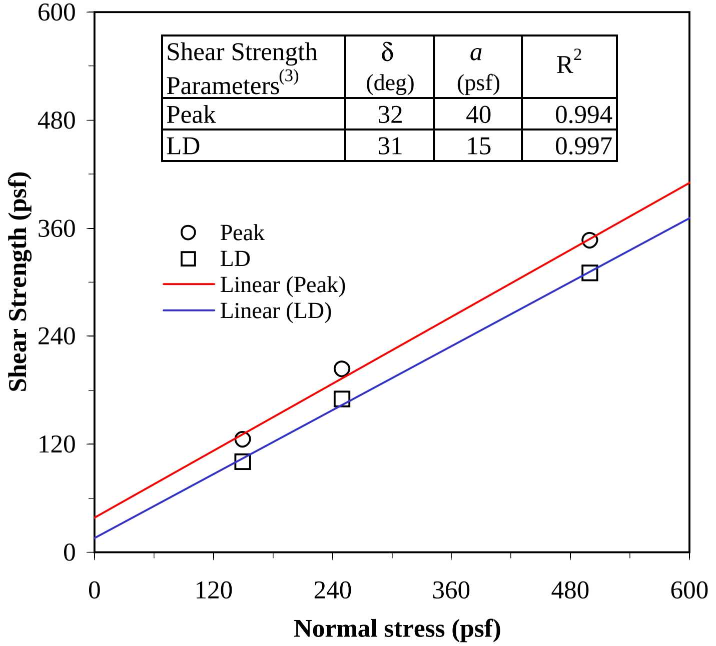
DATE OF REPORT:	5/6/2012
FIGURE NO.	C-1
PROJECT NO.	SGI12009
DOCUMENT NO.	
FILE NO.	

SGI TESTING SERVICES, LLC

AGRU AMERICA, INC. - SOUTH COAST LANDFILL
INTERFACE DIRECT SHEAR TESTING (ASTM D 5321)

Upper Shear Box: Brown clay compacted to approximately 90% of max modified Proctor density at OMC(γ_{dmax} = 120.1 pcf OMC = 13.5%)/
Agrutex 121(12 oz) nonwoven geotextile #1116017278 with heart-treated side down /
Agru 50-mil Super Gripnet LLDPE geomembrane # 103553-12 with spike side down/
Lower Shear Box: Brown clay compacted to approximately 90% of max modified Proctor density at OMC γ_{dmax} = 120.1 pcf OMC = 13.5%)





Shear Strength Parameters[3]	δ (deg)	*a* (psf)	R^2
Peak	32	40	0.994
LD	31	15	0.997

Test No.	Shear Box Size (in. x in.)	Normal Stress (psf)	Shear Rate (in./min)	Soaking Stress (psf)	Soaking Time (hour)	Consolidation Stress (psf)	Consolidation Time (hour)	Drainage Gravel γ_d (pcf)	Drainage Gravel ω_i (%)	Drainage Gravel ω_f (%)	Brown Clay γ_d (pcf)	Brown Clay ω_i (%)	Brown Clay ω_f (%)	GCL ω_i (%)	GCL ω_f (%)	Shear Strength τ_P (psf)	Shear Strength τ_{LD} (psf)	Failure Mode
2A	12 x 12	150	0.04	150	24	-	-	-	-	-	107.5	14.1	-	-	-	125	100	(1)
2B	12 x 12	250	0.04	250	24	-	-	-	-	-	110.1	13.6	-	-	-	203	170	(1)
2C	12 x 12	500	0.04	500	24	-	-	-	-	-	109.9	13.8	-	-	-	346	310	(1)

NOTES:

(1) Sliding occurred at the interface between the the upper clay soil and nonwoven geotextile.

(2) The reported total-stress parameters of friction angle and adhesion were determined from a best-fit line drawn through the test data. Caution should be exercised in using these strength parameters for applications involving normal stresses outside the range of the stresses covered by the test series. The large-displacement (LD) shear strength was calculated using the shear force measured at the end of the test.

DATE OF REPORT:	5/6/2012
FIGURE NO.	C-2
PROJECT NO.	SGI12009
DOCUMENT NO.	
FILE NO.	

SGI TESTING SERVICES, LLC

SGI TESTING SERVICES

A Georgia Limited Liability Company

1 May 2006

Mr. Paul Barker
Agru America, Inc.
700 Rockmead, Suite 150
Kingwood, Texas 77339

Subject: Laboratory Test Results Transmittal
Hydraulic Transmissivity Testing
Landfill Cover System

Dear Mr. Barker,

SGI Testing Services, LLC (SGI$_{sm}$) is pleased to present the attached test results for the above-mentioned project. The note section below addresses sample preparation, sample disposal and a disclosure statement.

SGI$_{sm}$ appreciates the opportunity to provide laboratory testing services to Agru America, Inc. Should you have any questions regarding the attached document(s), or if you require additional information, please do not hesitate to contact the undersigned.

Sincerely,

Zehong Yuan, Ph.D., P.E.
Chief Technical Officer

Attachments

Notes:
(1) Unless otherwise noted in the test results the sample(s)/specimen(s) were prepared in accordance with the applicable test standards or generally accepted sampling procedures.
(2) Contaminated/chemical samples and all related laboratory generated waste (i.e., test liquids, PPE, absorbents, etc.) will be returned to the client or designated representative(s), at the client's cost, within 60 days following the completion of the testing program, unless special arrangements for proper disposal are made with SGI$_{sm}$.
(3) Materials that are not contaminated will be discarded after test specimens and archived specimens are obtained. Archived specimens will be discarded 60 days after the samples are received at the laboratory, unless long-term storage arrangements are specifically made with the laboratory.
(4) The reported results apply only to the materials and test conditions used in the laboratory testing program. The results do not necessarily apply to other materials or test conditions. The test results should not be used in engineering analysis unless the test conditions model the anticipated field conditions. The testing was performed in accordance with general engineering testing standards and requirements. The reported results are submitted for the exclusive use of the client to whom they are addressed.

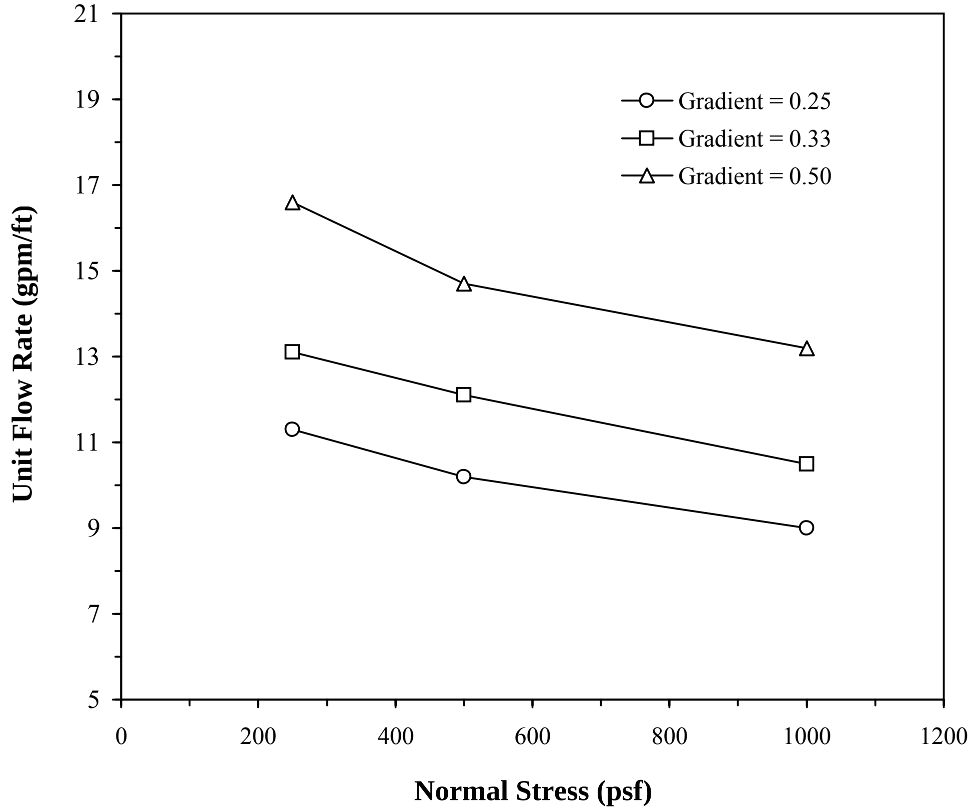
SGI6013/SGI06017

ATTACHMENT A

SHORT-TERM (15-MIN SEATING TIME) TRANSMISSIVITY TEST RESULTS

AGRU AMERICA, INC.
HYDRAULIC TRANSMISSIVITY TESTING (ASTM D 4716)

Test Series No. 1: Test Configuration (from Top to Bottom):
Steel Plate/
Agru 200-mil Geonet (Roll # 503201-06)/
Stud Side of Agru 50-mil LLDPE Supergripnet Geomembrane/
/Steel Plate



Test No.	Flow Direction	Normal Stress (psf)	Seating Time (hour)	Hydraulic Gradient (-)	Transmissivity (m^2/sec)	Unit Flow Rate (gpm/ft)
1	Machine Direction	250	0.25	0.25	9.36E-03	11.3
2		500	0.25	0.25	8.44E-03	10.2
3		1000	0.25	0.25	7.45E-03	9.0
4		250	0.25	0.33	8.22E-03	13.1
5		500	0.25	0.33	7.59E-03	12.1
6		1000	0.25	0.33	6.59E-03	10.5
7		250	0.25	0.50	6.87E-03	16.6
8		500	0.25	0.50	6.08E-03	14.7
9		1000	0.25	0.50	5.46E-03	13.2

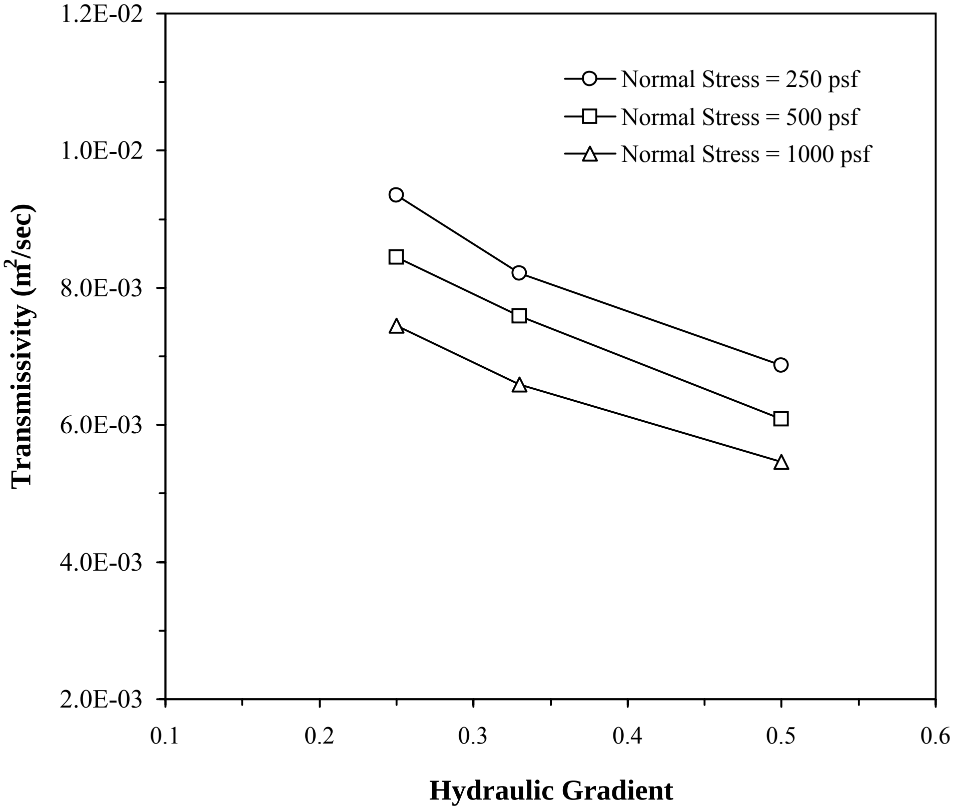
NOTE:
(1) Test Specimen Dimensions: length: 12 in., width = 12.0 in.

DATE REPORTED:	4/3 to 4/5/2006
FIGURE NO.	A-1
PROJECT NO.	SGI6013
DOCUMENT NO.	SGI06017
FILE NO.	

SGI TESTING SERVICES, LLC

AGRU AMERICA, INC.
HYDRAULIC TRANSMISSIVITY TESTING (ASTM D 4716)

Test Series No. 1: Test Configuration (from Top to Bottom):
Steel Plate/
Agru 200-mil Geonet (Roll # 503201-06)/
Stud Side of Agru 50-mil LLDPE Supergripnet Geomembrane/
/Steel Plate



Test No.	Flow Direction	Normal Stress (psf)	Seating Time (hour)	Hydraulic Gradient (-)	Transmissivity (m^2/sec)	Unit Flow Rate (gpm/ft)
1	Machine Direction	250	0.25	0.25	9.36E-03	11.3
2		250	0.25	0.33	8.22E-03	13.1
3		250	0.25	0.50	6.87E-03	16.6
4		500	0.25	0.25	8.44E-03	10.2
5		500	0.25	0.33	7.59E-03	12.1
6		500	0.25	0.50	6.08E-03	14.7
7		1000	0.25	0.25	7.45E-03	9.0
8		1000	0.25	0.33	6.59E-03	10.5
9		1000	0.25	0.50	5.46E-03	13.2

NOTE:
(1) Test Specimen Dimensions: length: 12 in., width = 12.0 in.

DATE REPORTED:	4/3 to 4/5/2006
FIGURE NO.	A-2
PROJECT NO.	SGI6013
DOCUMENT NO.	SGI06017
FILE NO.	

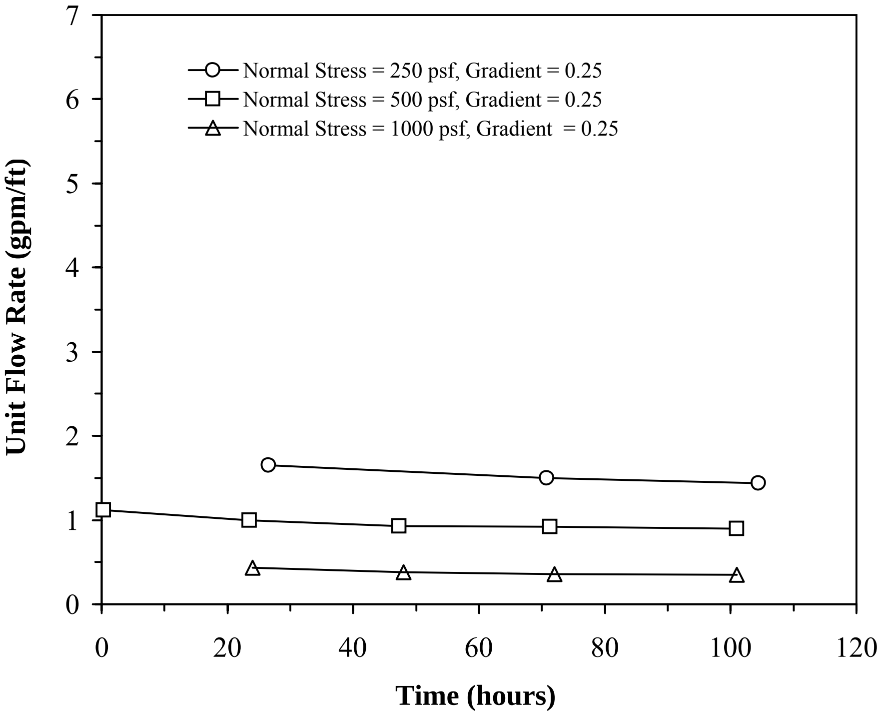
SGI TESTING SERVICES, LLC

ATTACHMENT B

LONG-TERM (100-HOUR) TRANSMISSIVITY TEST RESULTS

AGRU AMERICA, INC.
HYDRAULIC TRANSMISSIVITY TESTING (ASTM D 4716)

Test Series No. 2A: Test Configuration (from Top to Bottom):
Steel Plate/Neoprene Layer/
Landtex 1080H Nonwoven Geotextile with Heat-treated Side Down/
Drain Stud Side of Agru 50-mil LLDPE Supergripnet Geomembrane/
/Steel Plate



Test No.	Flow Direction	Soil Conditions γ_d (pcf)	Soil Conditions ω_i (%)	Normal Stress (psf)	Rime (hour)	Hydraulic Gradient (-)	Transmissivity (m^2/sec)	Unit Flow Rate (gpm/ft)
1	MD	-	-	250	26.5	0.25	1.37E-03	1.65
					70.8	0.25	1.24E-03	1.50
					104.4	0.25	1.19E-03	1.44
2	MD	-	-	500	0.3	0.25	9.27E-04	1.12
					23.5	0.25	8.28E-04	1.00
					47.3	0.25	7.70E-04	0.93
					71.3	0.25	7.62E-04	0.92
					101.0	0.25	7.45E-04	0.90
3	MD	-	-	1000	24.0	0.25	3.56E-04	0.43
					48.0	0.25	3.15E-04	0.38
					72.0	0.25	2.98E-04	0.36
					101.0	0.25	2.90E-04	0.35

NOTES: (1) Test Specimen Dimensions: length: 12 in., width = 12.0 in.
(2) γ_d = dry unit weight; ω_i = moisture content.

DATE TESTED: 4/4 to4/23/2006

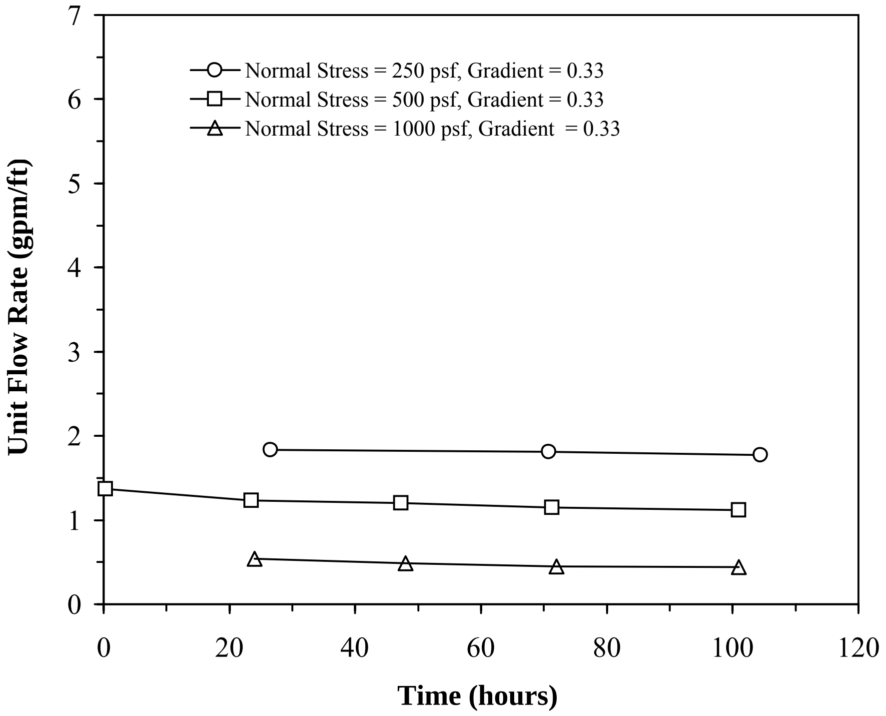
SGI TESTING SERVICES, LLC

FIGURE NO.	B-1
PROJECT NO.	SGI6013
DOCUMENT NO.	SGI06017
FILE NO.	

AGRU AMERICA, INC.

HYDRAULIC TRANSMISSIVITY TESTING (ASTM D 4716)

Test Series No. 2B: Test Configuration (from Top to Bottom):
Steel Plate/Neoprene Layer/
Landtex 1080H Nonwoven Geotextile with Heat-treated Side Down/
Drain Stud Side of Agru 50-mil LLDPE Supergripnet Geomembrane/
/Steel Plate



Test No.	Flow Direction	Soil Conditions		Normal Stress (psf)	Rime (hour)	Hydraulic Gradient (-)	Transmissivity (m^2/sec)	Unit Flow Rate (gpm/ft)
		γ_d (pcf)	ω_i (%)					
1	MD	-	-	250	26.5	0.33	1.15E-03	1.83
					70.8	0.33	1.14E-03	1.81
					104.4	0.33	1.11E-03	1.77
2	MD	-	-	500	0.3	0.33	8.59E-04	1.37
					23.5	0.33	7.71E-04	1.23
					47.3	0.33	7.53E-04	1.20
					71.3	0.33	7.21E-04	1.15
					101.0	0.33	7.02E-04	1.12
3	MD	-	-	1000	24.0	0.33	3.39E-04	0.54
					48.0	0.33	3.07E-04	0.49
					72.0	0.33	2.82E-04	0.45
					101.0	0.33	2.76E-04	0.44

NOTES: (1) Test Specimen Dimensions: length: 12 in., width = 12.0 in.
(2) γ_d = dry unit weight; ω_i = moisture content.

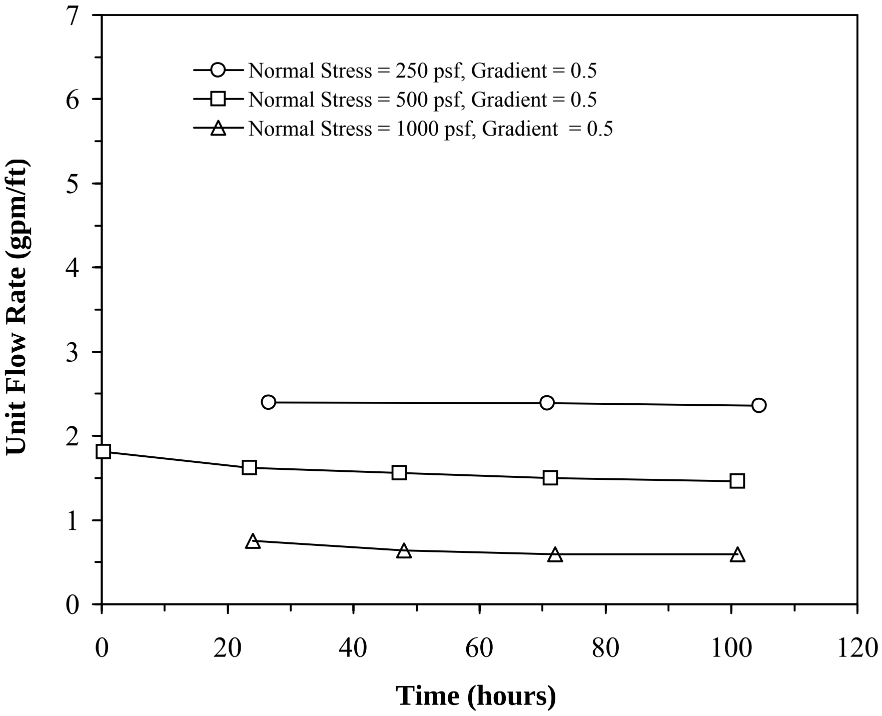
DATE TESTED:	4/4 to4/23/2006
FIGURE NO.	B-2
PROJECT NO.	SGI6013
DOCUMENT NO.	SGI06017
FILE NO.	

SGI TESTING SERVICES, LLC

AGRU AMERICA, INC.

HYDRAULIC TRANSMISSIVITY TESTING (ASTM D 4716)

Test Series No. 2C: Test Configuration (from Top to Bottom):
Steel Plate/Neoprene Layer/
Landtex 1080H Nonwoven Geotextile with Heat-treated Side Down/
Drain Stud Side of Agru 50-mil LLDPE Supergripnet Geomembrane/
/Steel Plate



Test No.	Flow Direction	Soil Conditions		Normal Stress	Rime	Hydraulic Gradient	Transmissivity	Unit Flow Rate
		γ_d (pcf)	ω_i (%)	(psf)	(hour)	(-)	(m^2/sec)	(gpm/ft)
1	MD	-	-	250	26.5	0.50	9.93E-04	2.40
					70.8	0.50	9.89E-04	2.39
					104.4	0.50	9.77E-04	2.36
2	MD	-	-	500	0.3	0.50	7.49E-04	1.81
					23.5	0.50	6.71E-04	1.62
					47.3	0.50	6.46E-04	1.56
					71.3	0.50	6.21E-04	1.50
					101.0	0.50	6.04E-04	1.46
3	MD	-	-	1000	24.0	0.50	3.10E-04	0.75
					48.0	0.50	2.65E-04	0.64
					72.0	0.50	2.47E-04	0.60
					101.0	0.50	2.44E-04	0.59

NOTES: (1) Test Specimen Dimensions: length: 12 in., width = 12.0 in.
(2) γ_d = dry unit weight; ω_i = moisture content.

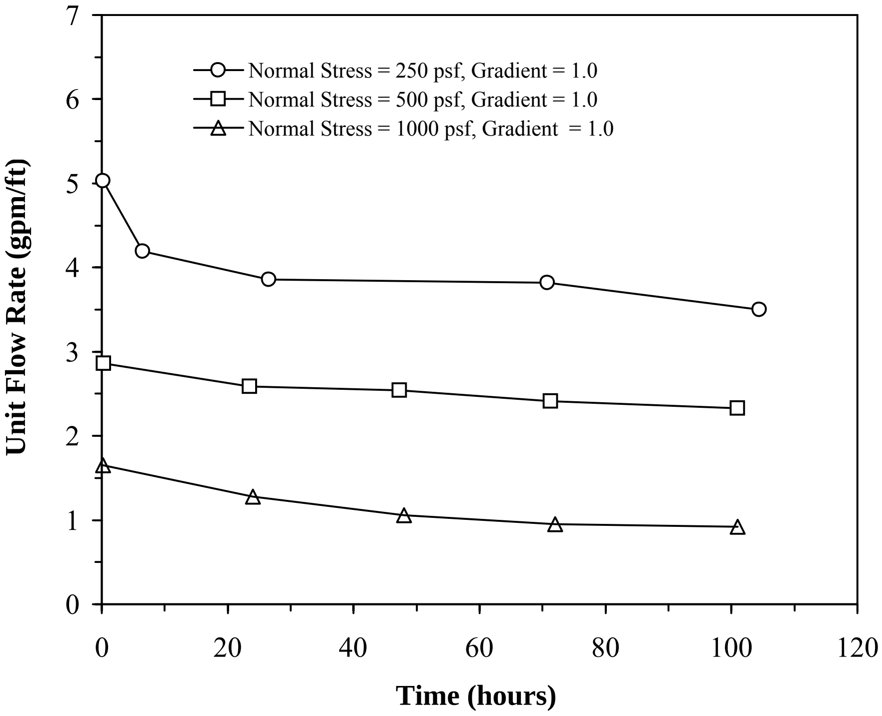
SGI TESTING SERVICES, LLC

DATE TESTED:	4/4 to4/23/2006
FIGURE NO.	B-3
PROJECT NO.	SGI6013
DOCUMENT NO.	SGI06017
FILE NO.	

AGRU AMERICA, INC.

HYDRAULIC TRANSMISSIVITY TESTING (ASTM D 4716)

Test Series No. 2D: Test Configuration (from Top to Bottom):
Steel Plate/Neoprene Layer/
Landtex 1080H Nonwoven Geotextile with Heat-treated Side Down/
Drain Stud Side of Agru 50-mil LLDPE Supergripnet Geomembrane/
/Steel Plate



Test No.	Flow Direction	Soil Conditions		Normal Stress	Rime	Hydraulic Gradient	Transmissivity	Unit Flow Rate
		γ_d (pcf)	ω_i (%)	(psf)	(hour)	(-)	(m^2/sec)	(gpm/ft)
1	MD	-	-	250	0.3	1.00	1.04E-03	5.03
					6.5	1.00	8.67E-04	4.19
					26.5	1.00	7.99E-04	3.86
					70.8	1.00	7.91E-04	3.82
					104.4	1.00	7.24E-04	3.50
2	MD	-	-	500	0.3	1.00	5.92E-04	2.86
					23.5	1.00	5.36E-04	2.59
					47.3	1.00	5.26E-04	2.54
					71.3	1.00	4.99E-04	2.41
					101.0	1.00	4.82E-04	2.33
3	MD	-	-	1000	0.3	1.00	3.42E-04	1.65
					24.0	1.00	2.65E-04	1.28
					48.0	1.00	2.19E-04	1.06
					72.0	1.00	1.97E-04	0.95
					101.0	1.00	1.90E-04	0.92

NOTES: (1) Test Specimen Dimensions: length: 12 in., width = 12.0 in.
(2) γ_d = dry unit weight; ω_i = moisture content.

SGI TESTING SERVICES, LLC

DATE TESTED:	4/4 to4/23/2006
FIGURE NO.	B-4
PROJECT NO.	SGI6013
DOCUMENT NO.	SGI06017
FILE NO.	