APPENDIX G

SOILS ENGINEERING REPORT AND ENGINEERING GEOLOGY INVESTIGATION

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October 7, 2015
Project No. SB00314-3

Attn: Ms. Joddi Leipner
Senior Engineering Environmental Manager
County of Santa Barbara
Public Works Department
Resource Recovery and Waste Management Division
130 East Victoria Street
Santa Barbara, CA 93101

Subject: **Response to Design Modifications - Revised Building Locations and Grading Design**
Tajiguas Resource Recovery Project
Tajiguas Landfill, 14470 Calle Real, Santa Barbara County, California

References: GeoSolutions, Inc., October 4, 2013, Soils Engineering Report and Engineering Geology Investigation, Tajiguas Resource Recovery Project (TRRP), Tajiguas Landfill, Santa Barbara, California, by GeoSolutions, Inc, Project SB00314-1, dated October 4, 2013.

Earth Systems Southern California, December 10, 2014, Engineering Geology and Geotechnical Engineering Report for Tajiguas Landfill Resource Recovery Project, 14770 Calle Real, Santa Barbara County, California, Project VT-24980-01, dated December 10, 2014.

John Kular Consulting, undated, ADF/MRF Grading and Drainage Plan (North)/(South).

Dear Ms. Leipner:

This report presents our geotechnical and geologic conclusions regarding the modifications to the proposed Tajiguas Resource Recovery Project located at Tajiguas Landfill, 14770 Calle Real, in Santa Barbara County, California. Changes to project plans since the time of our report dated October 4, 2013, included a shift to the proposed ADF and MRF building footprints and an increase to the gradient of the proposed cut slope west of the buildings. We have reviewed revisions to the proposed plans (John Kular Consulting, undated) and additional Engineering Geology and Geotechnical Engineering Report (Earth Systems Southern California, 2014) in order to determine whether the project modifications will impact the conclusions and mitigations from the Tajiguas Landfill Resource Recovery Project Internal Draft Subsequent EIR, dated August 2014.

It is our understanding that the proposed ADF and MRF buildings were moved 25 feet south and 20 feet west to minimize the settlement impact associated with the underlying fill on the eastern side of the building footprints (GEO-5) and to minimize visual impacts associated with the proposed development. The revised grading plans also indicated cut slopes in the Rincon Shale west and south of the buildings and operations deck to be approximately 2:1 (horizontal:vertical) with an approximately 15-foot bench cut at about 25 feet (vertical) up the slope. The original cut slopes analyzed in our 2013 report were proposed at 2.5:1, with similar benching. Current plans (John Kular Consulting, undated) show the graded area with cut-slopes extending up about 52 to 67 feet from the easement along the west side of the buildings.

A summary of the original geotechnical and geologic impacts stated in our 2013 report (GeoSolutions, Inc., October 4, 2013) are presented below:

Landslides - The Rincon Shale is generally a weaker unit and prone to landslides when saturated, therefore within the Rincon Shale units there is a moderate potential for landslides. A slope stability analysis was performed on the proposed western cut slope and provides recommendations to maintain the stability of the slope as discussed in Section 6.0. Due to the character of the Vaqueros Sandstone and Sespe Formation, there is a low potential for landslides within these units. **Potential landslide impacts at the Site were identified as less than significant, Class III**.

Severe Erosion – The potential for severe erosion is low considered provided that vegetation and erosion control measures are implemented immediately after the completion of grading. **Therefore, impacts associated with severe erosion can be feasibly mitigated and is interpreted to be a Class II Impact.**

Regional Faulting and Seismicity - The Alquist-Priolo Earthquake Fault Zoning Act of 1972 requires that the California State Geologist establish Earthquake Fault Zones around the surface traces of active faults and to issue appropriate maps. The subject site is not located within an Earthquake Fault Zone (Jennings, 2010). **Potential impacts at the Site due to faulting were identified as less than significant, Class III**. Based on the results of the slope stability analysis, the proposed fill/cut slopes appear to be grossly stable under psuedo-static (seismic) conditions, therefore the potential for seismically induced slope failure at the site is low. **Potential impacts at the Site due to seismically induced slope failure were identified as less than significant, Class III**. Based on the presence of clay in the fill and formational units, there is a low potential for seismically induced settlement at the Site, however there is a high potential within the MSW. The MRF and ADF buildings are proposed in the vicinity of MSW, however foundation recommendations are provided to help mitigate settlement effects. **Therefore, impacts associated with seismically induced settlement can be feasibly mitigated and is interpreted to be a Class II Impact.**

Tsunami/Seiches - As the property is at an elevation of approximately 390 feet, the potential for a tsunami to affect the Site is low. Flooding associated with a seismic event is considered low due to the absence of a body of water upslope of the property. The northern sedimentation basin is located upslope of the current operations deck, however existing 48-inch storm drain inlets are also located upslope which would flow inundated water beneath the operations deck. Therefore, flooding associated with a seismic event (seiches) is considered low. **Potential impacts at the Site due to tsunamis or seiches were identified as less than significant, Class III**.

Liquefaction - Based on the consistency and relative density of the in-situ soils (clay/rock) and the depth to groundwater the potential for seismic liquefaction of soils at the Site is very low. **Potential liquefaction impacts at the Site were identified as less than significant, Class III**.

Expansive Soils – The Rincon Formation was classified as medium expansion from laboratory testing (see Appendix B). Additional fill at the operations deck is proposed to be derived from Rincon Formation located at the west borrow slope. Foundation recommendations will negate the negative impacts of Rincon Formation to the MRF and ADF structures. **Therefore, impacts associated with expansive soils can be feasibly mitigated and is interpreted to be a Class II Impact.**

Slope Stability - **Cut Slope: 2.5:1 - West of the Operations Deck, Fill Slope: 3:1 - South of the Operations Deck**, **Fill Slope: 2:1 - West of the Maintenance Building Pad** - The critical factor of safety results were observed to exceed the minimum design factor of safeties for static and pseudo static. Based on this, if the slopes are constructed to the proposed configurations and in accordance with our recommendations, then it is our opinion that the proposed cut slope should be stable. **Therefore, impacts associated with stability of the modified slopes can be feasibly mitigated and is interpreted to be a Class II Impact.**

Settlement - Significant settlement of the refuse was observed in the analysis of the operations deck. As part of the design of the ADF and MRF buildings, the majority of the buildings are proposed to be constructed on the operations deck underlain by artificial fill or Rincon Shale. Foundation recommendations will negate the negative impacts to the structures for both settlement and differential settlement throughout the pad. **Therefore, impacts associated with the settlement of the operation deck can be feasibly mitigated and is interpreted to be a Class II Impact.** Several hundred feet of refuse and significant settlement is anticipated throughout this area. Recommendations to the compost area pavement section to help mitigate the effects of settlement and improve the structural integrity are provided in Section 8.9. **Therefore, impacts associated with the settlement of the compost area can be feasibly mitigated and is interpreted to be a Class II Impact.**

The project changes and additional recommendations in the Engineering Geology and Geotechnical Engineering Report (Earth Systems, December 10, 2014) do not affect the original landslide, severe erosion, regional faulting and seismicity, tsunami/seiches, liquefaction, expansive soils and settlement impacts. The cut and fill slopes west of the Operations Deck are now proposed to be constructed with slope gradients of 2:1 (horizontal:vertical) between benches which will be located at about every 25 feet (vertical). Based on our understanding of the site conditions, there is potential for surficial failures to occur within areas graded at 2:1 (horizontal:vertical) in the Rincon Shale Formation. **Despite the changes in gradient of the proposed cut-slope, impacts associated with stability of the modified slopes can be feasibly mitigated and are still interpreted to be a Class II Impact.**

Our review was performed in accordance with the usual and current standards of the profession, as they relate to this or similar localities. No other warranty, expressed or implied, is made as to the conclusions and professional advice included in this plan review.

Thank you for the opportunity to have been of service in preparing this report. If you have any questions or require additional assistance, please feel free to contact the undersigned at (805) 543-8539.

Sincerely,
GeoSolutions, Inc.

Patrick B. McNeill

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SOILS ENGINEERING REPORT AND
ENGINEERING GEOLOGY INVESTIGATION
TAJIGUAS RESOURCE RECOVERY PROJECT (TRRP),
TAJIGUAS LANDFILL, SANTA BARBARA COUNTY, CALIFORNIA

PROJECT SB00314-1

Prepared for

Attn: Ms. Joddi Leipner
Senior Engineering Environmental Manager
County of Santa Barbara
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October 4, 2013

October 4, 2013
Project No. SB00314-1

Attn: Ms. Joddi Leipner
Senior Engineering Environmental Manager
County of Santa Barbara
Public Works Department
Resource Recovery and Waste Management Division
130 East Victoria Street
Santa Barbara, CA 93101

Subject: **Soils Engineering Report and Engineering Geology Investigation**
Tajiguas Resource Recovery Project (TRRP)
Tajiguas Landfill, County of Santa Barbara, California

Dear Ms. Leipner:

This Soils Engineering Report and Engineering Geology Investigation has been prepared for the Tajiguas Resource Recovery Project within the Tajiguas Landfill located approximately 26 miles west of the City of Santa Barbara, and 1,600 feet north of U.S. Highway 101, Santa Barbara County, California. Geotechnically and geologically, the site is suitable for the proposed development provided the recommendations in this report for site preparation, earthwork, and foundations are incorporated into the design. The Resource Recovery Project is proposed to be located at the Tajiguas Landfill and would include a Materials Recovery Facility (to recover recyclable materials), a Dry Fermentation Anaerobic Digestion Facility (to process organic waste into biogas and digestate), an Energy Facility that would use the biogas from the Anaerobic Digestion Facility to produce electricity, a composting area to cure the digestate into soil amendments/compost and associated infrastructure (e.g., pipelines, storage tanks).

The operations deck which is to be comprised of a proposed Materials Recovery Facility (MRF), Dry Fermentation Anaerobic Digestion Facility (ADF) and associated loading and parking areas will be constructed on 0-85 feet of fill. It is our understanding final proposed grade elevation of the operation deck will include the addition of up to 10 to 20 feet of fill placed in operations deck area. Due to the potential for large differential settlement (up to 2-3 feet) in this area it is anticipated that a cast-in-place concrete caisson, driven H-Pile or Helical Pier and grade beam type of foundation system will be constructed for the proposed MRF and ADF facilities with all piers founded a minimum of 10.0 feet into uniform competent formational material located approximately 10 to 95 feet below finish grade. (See Plates 2A, 2B, and 2C for fill depths pertaining to facilities footprints.)

The proposed Maintenance Building is to be located along the northeast property line and is anticipated to utilize a mat slab type of foundation system to mitigate the potential of differential settlement associated with varying fill depths within the existing engineering fill pad (Geosyntec, 2009). All foundations are to be excavated into uniform material to limit the potential for distress of the foundation systems due to differential settlement.

It is our understanding the proposed Well Water Storage Tank and Recycled Water Storage Tank are to be located to the northwest of the west borrow slope area, the proposed Composting Area Runoff Collection Runoff Tank located northof the proposed Maintenance Building, and the proposed Percolate Tanks are to be located at the southwest corner of the proposed ADF building. Due to the shallow depths to competent formational material in these four areas, it is anticipated the proposed tank foundation systems will utilize continuous footings founded in uniform competent formational material as observed and approved by a representative of GeoSolutions, Inc. Deepened footings may be required in certain areas to achieve the required embedment depth in uniform competent formational material.

It is our understanding that no structures are proposed for the composting area but a non-permeable hardscape is anticipated to be constructed over the composting area.

Thank you for the opportunity to have been of service in preparing this report. If you have any questions or require additional assistance, please feel free to contact the undersigned at (805) 614-6333.

Sincerely,
GeoSolutions, Inc.

Patrick B. McNeill

Patrick B. McNeill, PE
Principal

Jeffrey A. Pfost, CEG #2493
Project Engineering Geologist

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PROJECT SB00314-1

1.0 INTRODUCTION

This report presents the results of the geotechnical and geologic investigation for the Tajiguas Resource Recovery Project within the Tajiguas Landfill located approximately 26 miles west of the City of Santa Barbara, and 1,600 feet north of U.S. Highway 101, Santa Barbara County, California. See Figure 1: Site Location Map for the general location of the project area. Figure 1: Site Location Map was obtained from the computer program *Topo USA 8.0* (DeLorme, 2009).

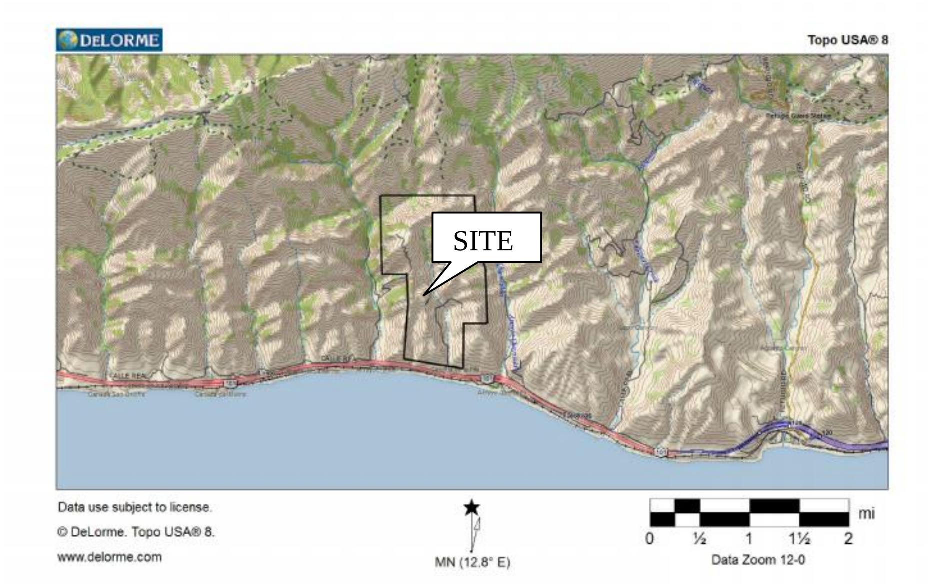


Figure 1: Site Location Map

The Tajiguas Landfill is an existing County-owned and operated municipal solid waste disposal facility located in a coastal canyon known as the Cañada de la Pila, located approximately 26 miles west of the City of Santa Barbara, and 1,600 feet north of U.S. Highway 101, Santa Barbara County. The Santa Barbara County Public Works Department, Resource Recovery and Waste Management Division (RRWMD) is the owner and permitted operator of the landfill.

The County of Santa Barbara proposes to develop a Tajiguas Resource Recovery Project that would process municipal solid waste from the communities currently served by the Tajiguas Landfill. The Tajiguas Resource Recovery Project will be designed and constructed to process various waste streams delivered to the Tajiguas Landfill from unincorporated areas of the South Coast of Santa Barbara, the Cities of Santa Barbara, Goleta, Buellton and Solvang as well as the unincorporated Santa Ynez and New Cuyama Valleys. The waste stream anticipated to be delivered for processing is mixed municipal solid waste. As an optional project element, commingled source separated recyclables could also be brought to the Tajiguas Resource Recovery Project for consolidated processing. The Tajiguas Resource Recovery Project would be located at the Tajiguas Landfill and would include a Materials Recovery Facility (MRF) (to recover recyclable materials), a Dry Fermentation Anaerobic Digestion Facility (ADF) (to process organic waste into biogas and digestate), an Energy Facility that would use the biogas from the Anaerobic Digestion Facility to produce electricity, a composting area where the digestate would be further cured in outdoor windrows at the landfill to create compost and soil amendments, and associated infrastructure including percolate tanks, water storage tanks, wastewater storage tank, runoff collection tank and pipelines. Residual waste (residue) from the processing would be disposed of in the landfill. No change in the landfill's permitted capacity is proposed.

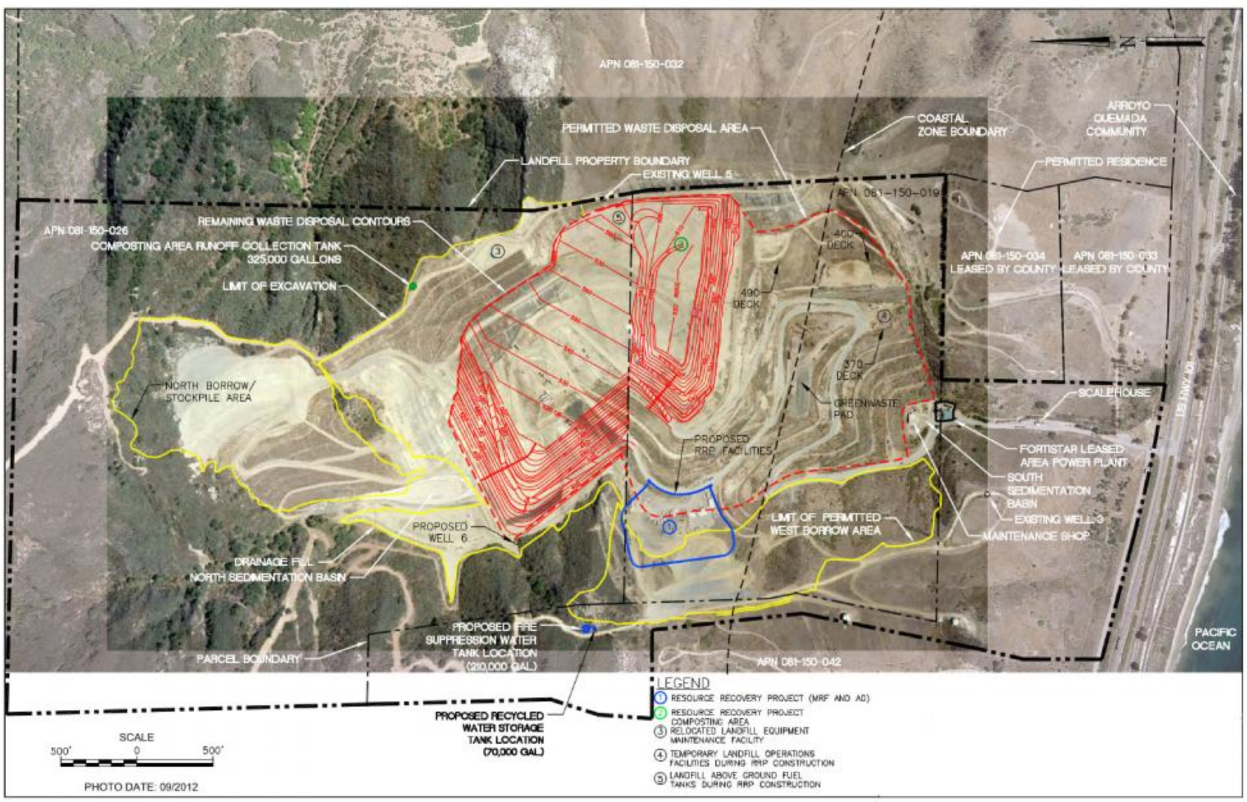


Figure 2: Project Facilities Plan

The Tajiguas Resource Recovery Project would modify current waste management operations at the Tajiguas Landfill by the addition of a Materials Recovery Facility (MRF) and Dry Fermentation Anaerobic Digestion (AD) Facility.

The MRF would be comprised of an approximate 60,000 square foot (sf) facility (70,000 sf if CSSR [optional element] is included as described above) that would sort MSW into three streams:

- Recyclables (i.e., glass, metal, paper, plastic, wood) - recovered and processed for sale;

- Organics – recovered for processing in the Anaerobic Digestion Facility; and
- Residue – materials left over after all recyclables and organics are recovered that would be disposed of at the existing Landfill.

The AD Facility would be housed within an approximate 63,000 sf building, and associated energy facility and percolate storage tanks that would convert all organics recovered from the MSW and SSOW into:

- Bio-gas (primarily composed of methane) – that would be used to power two (2) 1,537 horsepower onsite combined heat and power (CHP) engines driving electric power generators that would generate approximately 1+ net megawatts (MW) of renewable power. The Energy Facility would be located on the south side of the AD Facility; and
- Digestate - that can then be cured into compost and/or soil amendments. The curing would require an approximately 5 acre area on the closed landfill top deck. The compost and/or soil amendments would be marketed for agricultural or landscape use or used for reclamation projects.

The Tajiguas Resource Recovery Project facilities would be located approximately 3,200 feet north of U.S. Highway 101 on the existing Tajiguas Landfill Operations Deck, an approximately 6-acre site that currently houses the Landfill administrative office, two crew trailers, engineering trailer, hazardous material storage, electronic-waste storage, equipment storage and parking, employee parking, maintenance facility and three fuel storage tanks.

The Coastal Zone boundary runs through the southern portion of the Landfill property (see Figure 2). The facilities (MRF, AD facility and composting area) associated with the Tajiguas Resource Recovery Project would be located outside of the Coastal Zone. The composting area is proposed to be located on the top deck of the Landfill. The top deck would be closed and a final landfill cover system installed prior to using it for the Tajiguas Resource Recovery Project composting area. To protect the integrity of the Landfill and protect water quality, closure, post-closure use and post-closure maintenance of the top deck area would be subject to review and approval by CalRecycle, the LEA and the Regional Water Quality Control Board.

In addition to the facilities listed above, a new groundwater well would be constructed to provide water to the project and two new advanced, self-contained commercial wastewater package treatment units would be constructed to treat the project’s domestic wastewater. The treated wastewater would be used for landscape irrigation on the slopes (non-Landfill) adjacent to the MRF and AD Facility. A new 220,000 gallon fire suppression water storage tank would be installed to provide water for the building sprinkler systems, domestic and process/equipment wash down uses, landscape irrigation needs and fire hydrants. Parking would be provided for Tajiguas Resource Recovery Project staff, Landfill operations staff and visitors.

The Tajiguas Resource Recovery Project Site would be located at the existing Landfill operations deck. The easterly portion of the project site overlies the closed Landfill waste footprint and could likely experience continuing future settlement. As a result, the Tajiguas Resource Recovery Project structures would be located on the western portion of the operations deck, west of the waste footprint. A hill slope associated with the permitted Landfill West Borrow Area borders the operations deck to the west. This slope is currently being excavated to provide soil for Landfill operations and Landfill closure activities.

Construction of the Tajiguas Resource Recovery Project facility would require approximately 107,200 cubic yards of cut and 81,200 cubic yards of fill to increase the pad height of the operations deck by up to 20 feet for a maximum finished pad elevation of 394 feet above msl. Grading would be balanced at the Landfill site. Finished slopes in the west borrow area would not exceed 2.5:1 with a bench.

The composting area would also be located on the closed Landfill waste footprint and would likely experience future settlement. No permanent structures are proposed in this area. Asphalt paving, piping, other support facilities in this area and operational procedures would be designed to account for differential settlement and to address Landfill post-closure maintenance and monitoring requirements. A monitoring and maintenance program would be implemented to ensure that Tajiguas Resource Recovery Project facilities located over the closed Landfill would not be damaged by differential settlement and that Tajiguas Resource Recovery Project operations would not damage the Landfill cover system.

2.0 PURPOSE AND SCOPE

The purpose of this study was to explore and evaluate the surface and sub-surface soil conditions at the Site to develop geotechnical information and design criteria, evaluate engineering geologic hazards at the Site and to develop geotechnical and geologic conclusions and recommendations regarding site development. The scope of this study includes the following items:

1. A literature review of available published geotechnical and geologic data pertinent to the project site.

2. A field study consisting of site reconnaissance, exploratory borings, and Cone Penetration Test (CPT) soundings in order to formulate a description of the sub-surface conditions at the Site.

3. Laboratory testing performed on representative soil samples that were collected during our field study.

4. A review of regional faulting and seismicity hazards, landslide potential, surface water and groundwater conditions, and liquefaction hazards.

5. Engineering analysis of the data gathered during our literature review, field study, and laboratory testing.

6. Development of recommendations for site preparation and grading as well as geotechnical design criteria for foundations.

7. Preparation of this report that summarizes our findings, conclusions, and recommendations regarding engineering geology aspects of the project.

3.0 FIELD AND LABORATORY INVESTIGATION

The field investigation was conducted from January 8 through 11, 2013 using a track-mounted CME 55 drill rig and an International Paystar 5000 CPT (Cone Penetration Test) Rig. Four eight-inch diameter exploratory borings were advanced on the operations deck to a maximum depth of 40.0 feet below ground surface (bgs) at the approximate locations indicated on Plate 1A, Site Engineering Geology Map. Two 2.5-inch diameter rock core borings were advanced on the west borrow slope to a maximum depth of 75.0 feet below ground surface at the approximate locations indicated on Plate 1A. Five 1.5-inch diameter CPT soundings were also advanced on the operations deck to a maximum depth of 105.0 feet below ground surface (bgs) at the approximate locations indicated on Plate 1A. Sampling methods included the Standard Penetration Test utilizing standard split-spoon sampler (SPT) without liners, Modified California sampler (CA) with liners, and NQ rock coring barrel with recovery liner. The CME 55 drill rig was equipped with an automatic hammer, which has an efficiency of approximately 80 percent and was used to obtain test blow counts in the form of N-values.

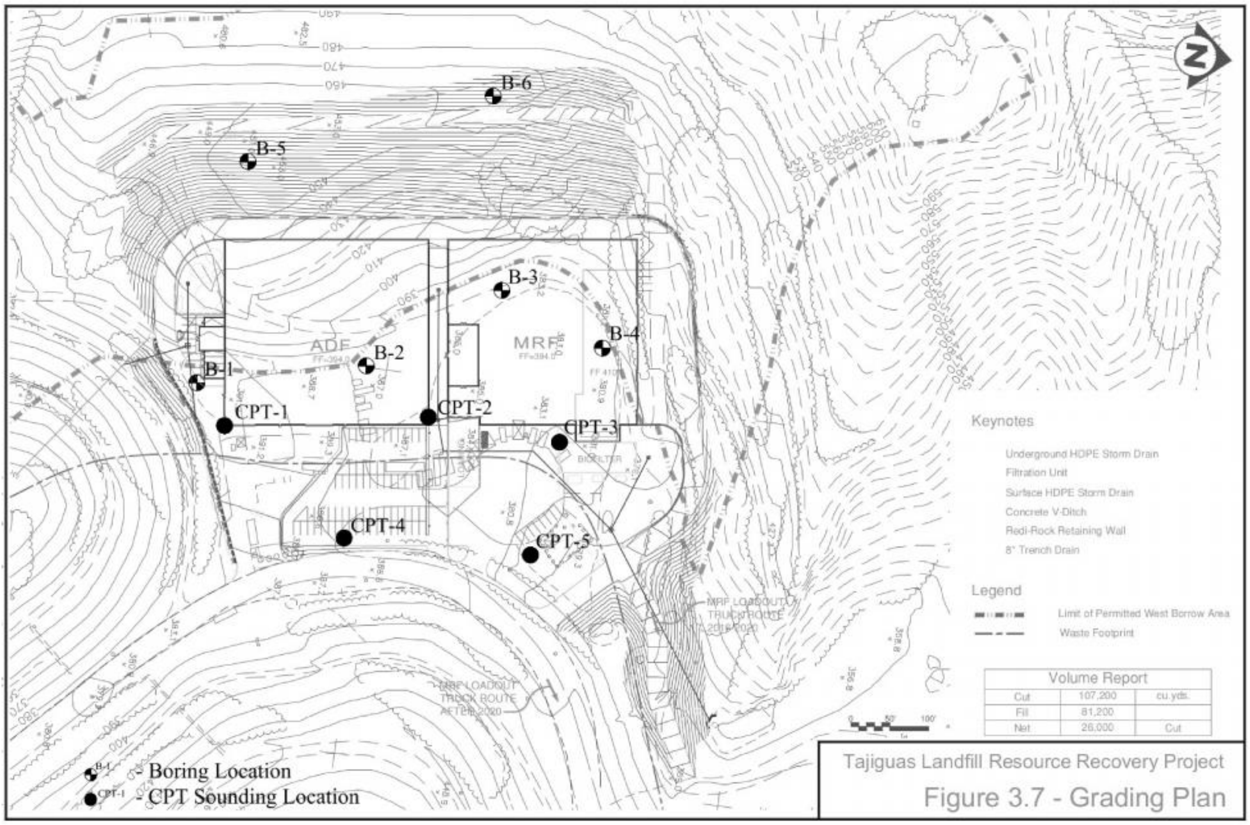


Figure 3: CPT and Boring Locations

Five CPT soundings were performed for this investigation on the operations deck. Two soundings were pushed within the refuse limits and three were pushed along the center of the operations deck. Soundings were pushed to termination depths varying from 33 to 107 feet bgs. Soundings were backfilled with bentonite chips.

Six borings were drilled during the investigation for this field study to determine the depth to formational units at the operations deck. The boring logs are presented in Appendix A. Boring B-1 encountered artificial fill in a slightly moist condition to a depth of 10 feet bgs. Boring B-2 encountered artificial fill to a depth of 25 feet bgs. Methane gas was detected in the four gas monitors while drilling Boring B-2. Boring B-3 encountered artificial fill to a depth of 25 feet bgs. Boring B-4 encountered artificial fill to a depth of 35 feet bgs underlain by siltstone units of the Rincon Formation to a termination depth of 40 feet bgs. Boring B-5 and B-6 were drilled utilizing NQ rock coring equipment on the upper bench of the west borrow slope. Boring B-5 encountered Rincon Shale from the surface to a termination depth of 65 feet bgs. Boring B-6 encountered Rincon Shale from the surface to a termination depth of 75 feet bgs. Borings were backfilled with bentonite powder and native cuttings.

Previous subsurface investigations have been performed on the Operations Deck during the following referenced reports:

- Geo-Logic Associates, 2007, Slope Stability Evaluation, Operations Center Fill Slope, Tajiguas Landfill, Santa Barbara County, California, Project No. 2007-0003, dated April 5, 2007.

- Willdan Geotechnical, 2010, Geotechnical Engineering Investigation Report, Tajiguas Landfill, 2005 Storm Drain Distress Mitigation, Santa Barbara County, California, Project No. 100366, dated November 29, 2010.

A total of 18 CPT Sounding were performed for the referenced Geotechnical Engineering Investigation Report (Willdan, 2010) along the storm drain alignment varying from 19 to 85 feet below ground surface. A total of 5 borings were also drilled throughout the operations deck for the referenced Slope Stability Evaluation (Geo-Logic Associates, 2007) varying from 16 to 26 feet bgs.

Data gathered during the field investigation suggest that soil materials in the vicinity of the Site consist of artificial fill, municipal solid waste (MSW), and Rincon Formation. The surface material at the Site generally consisted of reddish brown to brown silty SAND (SM) with gravel to light brown clayey SAND (SC) with gravel termed artificial fill. The sub-surface materials consisted of gray to light brown CLAYSTONE observed as slightly moist and moderately hard termed Rincon Formation. Groundwater was not encountered in any of the borings.

During the boring/coring operations the soils encountered were continuously examined, visually classified, and sampled for general laboratory testing. A project engineer has reviewed a continuous log of the soils encountered at the time of field investigation. See **Appendix A** for the Boring Logs from the field investigation.

Laboratory tests were performed on soil samples that were obtained from the Site during the field investigation. The results of these tests are listed below in Table 1: Engineering Properties. Laboratory data reports and detailed explanations of the laboratory tests performed during this investigation are provided in **Appendix B**.

Table 1: Engineering Properties

Sample Name	Sample Description	USCS Specification	Plasticity Index	Expansion Index	Expansion Potential	Maximum Dry Density, ¥d (pcf)	Optimum Moisture (%)	Angle of Internal Friction, ¤ (deg.)	Cohesion, c (psf)
A	Dark Yellowish Brown Clayey SAND	SC	24	67	Medium	128.8	10.0	-	-
B	Dark Yellowish Brown Clayey SAND	SC	-	-	-	130.6	8.1	-	-
C	Light Gray CLAYSTONE	CH	26	88	Medium	107.0	17.4	25°	301
B-3 @ 4 ft bgs	Dark Olive Brown Sandy CLAY	CL	-	-	-	-	-	31°	651
B-6 @ 3 ft bgs	Dark Gray CLAYSTONE	CL	-	-	-	-	-	7°	1417

4.0 ENGINEERING GEOLOGY INVESTIGATION

4.1 Regional Geology

The Tajiguas Landfill is located on the south flank of the Santa Ynez Mountains, a component of the Transverse Range Geomorphic Province. This geomorphic province is characterized by generally east-west trending mountain ranges and intervening valleys. Older uplifted bedrock is exposed in the mountains: the valleys are filled with sedimentary rocks and alluvial deposits. The Transverse Ranges are bordered by the

Santa Monica fault to the south and the Santa Ynez fault to the north. The Santa Ynez Range extends from Gaviota Canyon eastward to the Matilija Gorge in Ventura County. The range is composed of a single main crest that is continuous for approximately 50 miles (80 km). The northern flank of the Santa Ynez Range is a steep escarpment created by uplift along the Santa Ynez fault. The southern flank, where the Tajiguas Landfill site is located is characterized by south-plunging ridges that separate incised drainage canyons. These canyons generally include a perennial stream bounded by steep east- and west-facing slopes. The indurated sandstone units typically form prominent, more resistant outcrops and generally support dense chaparral vegetation. The poorly indurated and finer-grained units typically form more gently-sloping, grass-covered hills (Geosyntec, 2008).

4.2 Local Geology

Locally, bedrock underlying the site is the Rincon Shale (Tr), Vaqueros Sandstone (Tvq), and Sespe Formation (Tsp) as depicted on Plate 1A through 1C, Site Engineering Geology Map. Dibblee, 1988 mapped the property as underlain by early Miocene age (23.8-16.4 million years before present {mybp}) Rincon Shale (Tr) and Vaqueros Sandstone (Tvq) and Oligocene age (33.7-23.8 mybp) Sespe Formation (Tsp). Our investigation and surface mapping of the area encountered units of the Rincon Shale, Vaqueros Sandstone, and Sespe Formation underlying artificial fill, municipal solid waste, and landslide deposits. Information derived from subsurface exploration was used to classify subsurface soil and formational units and to supplement geologic mapping.

Plate 1A depicts Rincon Shale (Tr) and Vaqueros Sandstone (Tvq) throughout the operations deck area overlain by artificial fill, municipal solid waste, and landslide deposits. Plate 1B depicts the composting area (also identified as the top deck) underlain by MSW and the composting area runoff collection tank underlain by Sespe Formation (Tsp). Plate 1C depicts Vaqueros Sandstone (Tvq) throughout the Well Water Storage and Recycled Water Storage Tanks. Plate 2A through 2C presents cross sections through the MRF and ADF buildings.

4.2.1 Surficial Units

Artificial Fill

Artificial fill was encountered at the operations deck at various depths. The operations deck was constructed to its current elevation in 2007. The artificial fill encountered during the field investigation of the operation deck consisted of reddish brown to light brown sandy CLAY (CH) with gravel to light brown clayey SAND (SC) with gravel encountered in a slightly moist and medium dense to dense condition. Methane gas was detected in surface monitors during drilling operations.

Municipal Solid Waste with Intermediate Soil Cover

MSW with intermediate soil cover extends within the eastern one-third of the existing operations deck (as observed on Plate 1A) and completely underlies the composting area (Plate 1B). The depth of refuse at the operations deck was observed to be 80-95 feet thick. A previous analysis by SWT Engineering, 2009, estimates the thickness of the MSW under the proposed composting area to vary from 21-313 feet thick. The proposed composting area is currently within an active area of landfilling and it is our understanding that up to an additional 80 feet of MSW will be placed prior to the establishment of the composting area.

4.2.2 Formational Units

Rincon Shale

Dibblee, 1988 describes the early Miocene age (11-1.8 mybp) Rincon Shale as “poorly bedded gray clay shale or claystone.” The Rincon Shale was observed in cut slopes throughout the operations deck including the west borrow cut slope to the west. The Rincon Shale at the site was observed as light gray SHALE and CLAYSTONE in a dry to slightly moist condition. The Rincon Shale is observed to be massive, fresh to slightly weathered (severely weathered at the surface), and moderately soft to moderately hard. The fractures within the Rincon Shale unit varied from friable at the surface to slightly fractured at depth. Bedding was not observed within the existing cut slope, however a continuous layer of light brown inclusions indicated a east-west strike direction, dipping south. Dibblee, 1988 depicts bedding attitudes within the Rincon Shale in the vicinity of the site striking in a slight SW-NE direction, dipping 35 to 69 degrees south.

Based on rock coring borings, Borings B-5 and B-6, the Rincon Shale is observed to be fair to good rock quality based on Rock Quality Determination (RQD) with layers of poor, very poor, and excellent quality. A recovery of 80 to 100 percent was also observed during the coring operations with localized layers ranging from 38 to 70 percent recovery.

Vaqueros Sandstone

Dibblee, 1988 describes the early Miocene age (11-1.8 mybp) Vaqueros Formation as “south of Santa Ynez fault: light gray calcareous sandstone.” The Vaquerous Sandstone at the Site was observed as light brown Sandstone observed in a dry and hard condition. Dibblee, 1988 depicts bedding attitudes within the Vaqueros Sandstone in the vicinity of the site striking in a slight SW-NE direction, dipping 40 to 57 degrees south.

Sespe Formation

Dibblee, 1988 describes the Oligocene age (33.7-23.8 mybp) Sespe Formation as “Gray to tan sandstone and green to red siltstone and claystone; basal part intertongues westward with Alegria Formation south of Santa Ynez fault.” The Sespe Formation at the Site was observed as tan to red to green thinly to thickly bedded siltstone and claystone observed in a dry and hard condition. Dibblee, 1988 depicts bedding attitudes within the Sespe Formation in the vicinity of the site striking in a slight SW-NE direction, dipping 40 to 54 degrees south.

4.3 Surface and Groundwater Conditions

Surface drainage at the proposed location of the MRF and ADF buildings will flow toward the operations deck then into the proposed storm drain system. Surface drainage at the proposed location of the composting area up to a 25-year, 24-hour storm will be collected and stored in the Composting Area Runoff Collection Tank, anything greater will overflow to the storm drain system. Surface drainage in the vicinity of the water tank pads (east and west) will sheet flow off the pad to the west. No springs or seeps were observed at the project site. No evidence of shallow groundwater was observed within the borings at the Site.

4.4 Landslides

Dibblee, 1988 did not map landslides at the property. During site mapping and identified in previous reports (GeoLogic, 2008), two surficial landslides were observed within the cut slope on the west borrow slope (see Figure 4). The northern landslide appears to be a shallow rotational instability within the Rincon shale while the southern landslide appears to be a shallow mud-flow type of instability. The upper portion of the southern landslide was removed during the most recent modification to the west borrow slope. The southern landslide is not observed to effect the proposed development. The northern landslide will be partially removed as part of the modified cut slope configuration although it is recommended that the landslide deposits be completely removed.

The Rincon Shale is generally a weaker unit and prone to landslides when saturated, therefore within the Rincon Shale units there is a moderate potential for landslides. A slope stability analysis was performed on the proposed western cut slope and provides recommendations to maintain the stability of the slope as discussed in Section 6.0. Due to the character of the Vaqueros Sandstone and Sespe Formation, there is a low potential for landslides within these units.

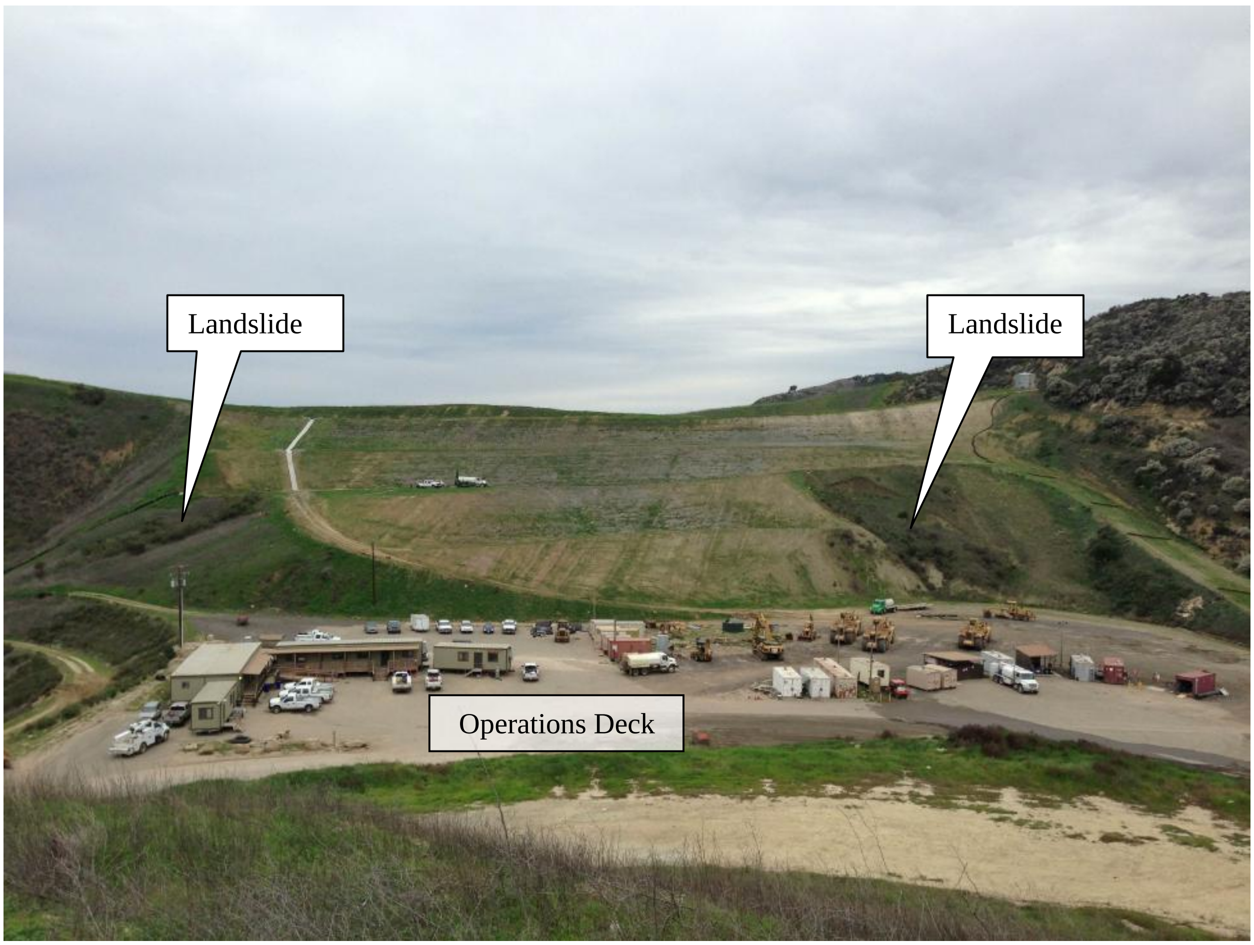


Figure 4: Photograph of the West Borrow Slope

4.5 Slope Stability

A slope stability analysis was performed on the proposed cut and fill slopes associated with proposed development. Fill slopes at the site currently are 2:1 with benches or 3:1 (horizontal:vertical) overall. A slope stability analysis was performed on the following fill slopes: south of the operations deck, which includes an additional 10 to 14 feet of fill and loading from the adjacent ADF building; and west of the maintenance building, which includes loading from the proposed maintenance building but won't have any slope reconfiguration. Based on the slope stability analysis, loading from proposed development does not affect the stability of the slopes and were observed to be grossly stable. The stability of the proposed 2.5:1

(horizontal:vertical) cut slope west of the operations deck was also analyzed and was observed to be grossly stable, provided that recommendations provided in this report are followed. Further discussion of the slope stability analyses are presented in Section 6.0.

4.6 Severe Erosion

The potential for severe erosion is considered low provided that vegetation and erosion control measures are implemented immediately after the completion of grading. It is recommended that the resulting slope face be covered with erosion mat and hydroseeded immediately following construction of the slopes. This will also serve to minimize surficial erosion due to irrigation and/or rainfall. It is recommended that erosion control measure be implemented immediately following the completion of construction for surfaces not being improved as designed by the project civil engineer.

4.7 Regional Faulting and Seismicity

Similar to the surrounding areas, the Site may be affected by moderate to major earthquakes centered on one of the known large, active faults listed in Table 2 below. Moment magnitudes are expressed, although any significant event on these faults could result in moderate to severe ground shaking at the subject site. The potential for ground failure of any portion of the Site during ground shaking is considered low.

Table 2: Active Faults Near the Subject Property

Closest Active Faults to Site	Approximate Distance (miles)	Moment Magnitude (Mw)
Santa Ynez Fault	15.5	7.1
Los Alamos Fault	16.0	6.8
San Andreas Fault	52.0	8.5

The closest known Holocene age fault is the Santa Ynez Fault located approximately 15.5 miles northwest of the Site (Jennings, 2010), however the San Andreas Fault is the most likely active fault to produce ground shaking at the Site. Figure 5 depicts significant historical earthquakes in the region.

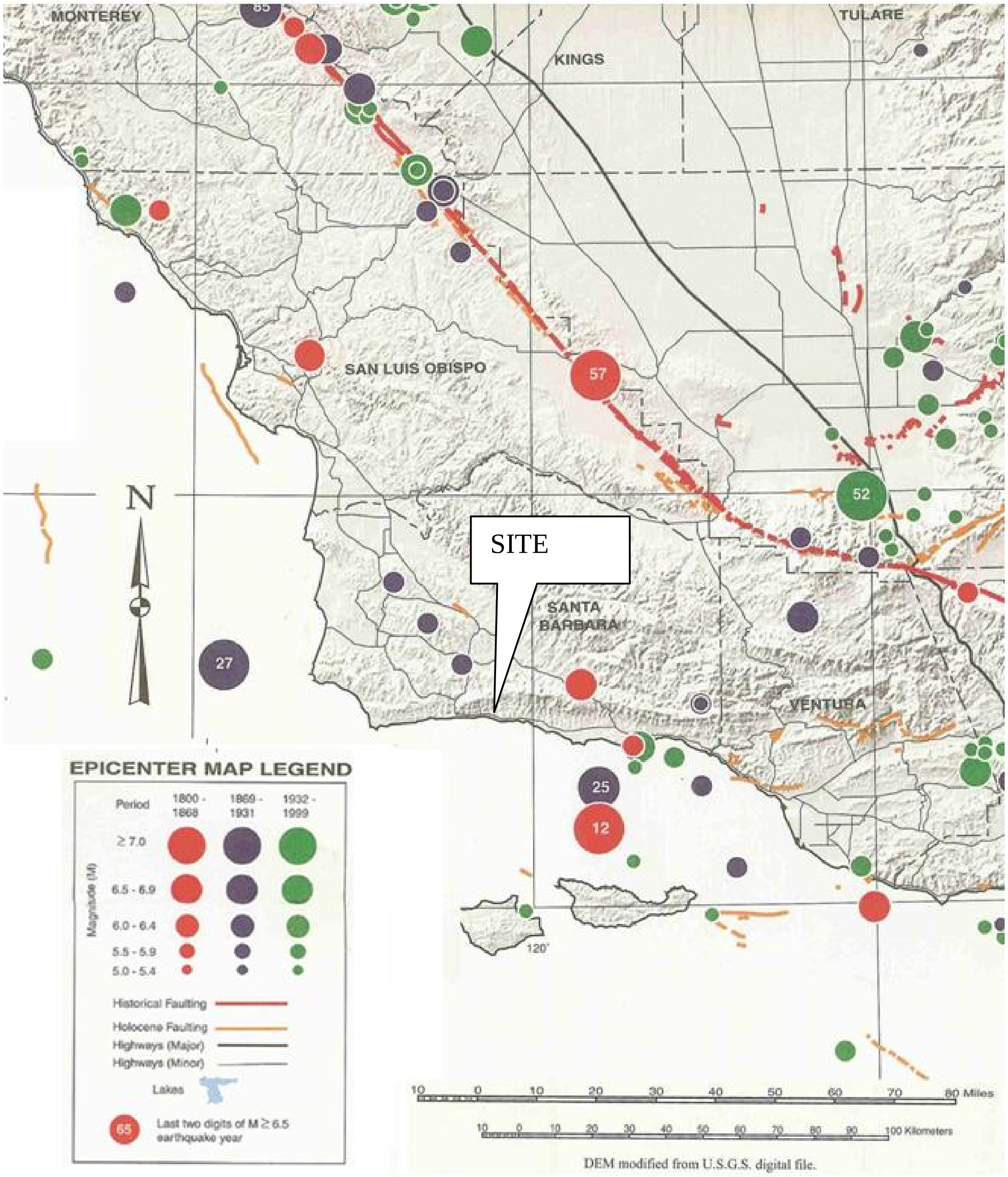


Figure 5: Historical Seismicity Map (Toppozada et al., 2000)

4.7.1 Ground Surface Rupture Due to Faulting

The Alquist-Priolo Earthquake Fault Zoning Act of 1972 requires that the California State Geologist establish Earthquake Fault Zones around the surface traces of active faults and to issue appropriate maps. The subject site is not located within an Earthquake Fault Zone (Jennings, 2010).

4.7.2 Tsunami/Seiches

Tsunamis and seiches are two types of water waves that are generated by earthquake events. Tsunamis are broad-wavelength ocean waves and seiches are standing waves within confined bodies of water, typically reservoirs. As the property is at an elevation of approximately 390 feet, the potential for a tsunami to affect the Site is low. The northern sedimentation basin is located upslope of the current operations deck, however existing 48-inch storm drain inlets are also located upslope which would flow inundated water beneath the operations deck. Therefore, the potential for flooding associated with a seismic event (seiches) is considered low.

4.7.3 Seismically Induced Slope Failure and Settlement

A slope stability analysis of the proposed fill slopes were performed under seismic conditions. A description of the analysis is presented in Section 6.0. Based on the results of the slope stability analysis, the proposed cut/fill slopes appear to be grossly stable under psuedo-static (seismic) conditions, therefore the potential for seismically induced slope failure at the site is low. Seismically induced settlement occurs in loose to medium dense unconsolidated soil above groundwater. These soils compress (settle) when subject to seismic shaking. The settlement can be exacerbated by increased loading, such as from the construction of buildings. Based on the presence of clay in the fill and formational units, there is a low potential for seismically induced settlement at the Site, however there is a high potential within the MSW. The MRF and ADF buildings are proposed in the vicinity of MSW, however foundation recommendations are provided to help mitigate settlement effects.

5.0 SEISMIC DESIGN CONSIDERATIONS

5.1 Seismic Hazard Analysis

According to section 1613 of the 2010 CBC (CBSC, 2010), all structures and portions of structures should be designed to resist the effects of seismic loadings caused by earthquake ground motions in accordance with the *Minimum Design Loads for Buildings and Other Structures* (ASCE7) (ASCE, 2006). ASCE7 considers the most severe earthquake ground motion to be the ground motion caused by the Maximum Considered Earthquake (MCE) (ASCE, 2006), which is defined in Section 1613 of the 2010 CBC to be short period S_{MS} and 1-second period S_{M1}, spectral response accelerations.

The a_{max} of the Site depends on several factors, which include the distance of the Site from known active faults, the expected magnitude of the MCE, and the Site soil profile characteristics.

As per section 1613.5.5 of the 2010 CBC (CBSC, 2010), the Site soil profile classification is determined by the average soil properties in the upper 100 feet of the Site profile. Based on the $(N_1)_{60}$ values calculated for the in-situ tests performed during the field investigation, the Site was defined as Site Class D, Stiff Soil profile per Table 1613.5.2 of the 2010 CBC (CBSC, 2010).

According to section 11.2 of ASCE7 (ASCE, 2006) and section 1613 of the 2010 CBC (CBSC, 2010), buildings and structures should be specifically proportioned to resist Design Earthquake Ground Motions (Design a_{max}). ASCE7 defines the Design a_{max} as "the earthquake ground motions that are two-thirds of the corresponding MCE ground motions" (ASCE, 2006, p. 109). The **Design a_{max} for the Site is equal to S_{D1}=0.730 and S_{DS}=1.264,** which are 1-second period and short period design spectral response accelerations that are equal to two-thirds of the a_{max} or MCE for the Site.

Site coordinates of 34.4853 degrees north latitude and -120.1428 degrees west longitude and a search radius of 100 miles were used in the probabilistic seismic hazard analysis.

5.2 Structural Building Design Parameters

Structural building design parameters within chapter 16 of the 2010 CBC (CBSC, 2010) and sections 11.4.3 and 11.4.4 of ASCE7 (ASCE, 2006) are dependent upon several factors, which include site soil profile characteristics and the locations and characteristics of faults near the Site. As described in section 5.1 of this report, the Site soil profile classification was determined to be Site Class D. This Site soil profile classification and the latitude and longitude coordinates for the Site were used to determine the structural building design parameters.

Spectral Response Accelerations and Site Coefficients were obtained from the U.S. Seismic Design Maps application; this application is available from the United States Geological Survey website (USGS, 2012). This web program utilizes the methods developed in the 2006 and 2009 editions of the International Building Code Recommended Provisions for Seismic Regulations for New Buildings and Other Structures and user-inputted Site latitude and longitude coordinates to calculate seismic design parameters and response spectra (both for period and displacement), for Site Classifications A through E. This data is presented in tabular form in Table 3: 2010 California Building Code, Chapter 16, Structural Design Parameters. Analysis of the Design Spectral Response Acceleration Parameters for the Site and of the Occupancy Category for the proposed structure assign to this project a **Seismic Design Category of D** per Tables 1613.3.5.6(1) and 1613.3.5.6(2) of the 2010 CBC (CBSC, 2010).

Table 3: 2010 California Building Code, Chapter 16, Structural Design Parameters

Site Class - Soil Profile Type	D - Stiff Soil Profile
Mapped Spectral Response Accelerations and Site Coefficients	S_S = 1.896, S_1 = 0.730 F_a = 1.000, F_v = 1.500
Adjusted Maximum Considered Earthquake Spectral Response Accelerations	S_{MS} = 1.896 S_{M1} = 1.095
Design Spectral Response Acceleration Parameters	S_{DS} = 1.264 S_{D1} = 0.730
Occupancy Category (from Table 1604.5, 2010 CBC)	II
Seismic Design Category – Short Period Accel. (from Table 1613.5.6(1), 2010 CBC)	D
Seismic Design Category – Long Period Accel. (from Table 1613.5.6(2), 2010 CBC)	D

5.3 Design Response Spectra – 2010 CBC

According to section 11.4.5 of ASCE7 (ASCE, 2006), a design response spectrum for a site may be required in order to design structures to resist lateral forces caused by ground motions at the Site. The design spectral response acceleration parameters, listed in Table 3: 2010 California Building Code, Chapter 16, Structural Design Parameters, are used to produce the design response spectrum. The U.S. Seismic Design Maps application (USGS, 2012) was used to construct a design response spectrum for the Site, which is shown in Figure 6: Design Response Spectra – 2010 CBC.

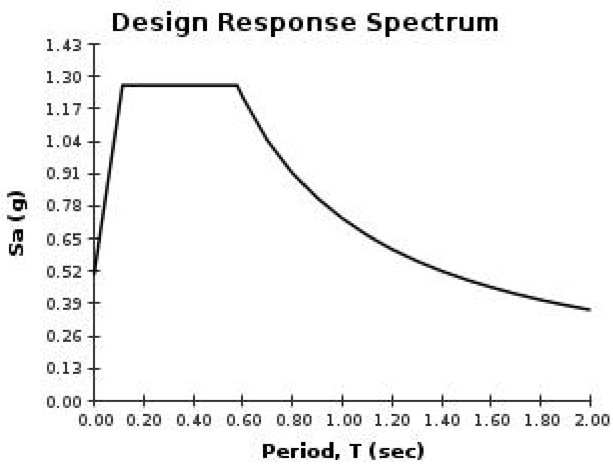


Figure 6: Design Response Spectra – 2010 CBC

5.4 Liquefaction Potential

In the context of soil mechanics, liquefaction is the process that occurs when the dynamic loading of a soil mass causes the shear strength of the soil mass to rapidly decrease. Liquefaction can occur in saturated cohesionless soils.

The most typical liquefaction-induced failures include consolidation of liquefied soils, surface sand boils, lateral spreading of the ground surface, bearing capacity failures of structural foundations, flotation of buried structures, and differential settlement of above-ground structures.

Liquefiable soils must undergo dynamic loading before liquefaction occurs. Ground motion from an earthquake may induce large-amplitude cyclic reversals of shear stresses within a soil mass. Repetitive lateral and vertical loading and unloading usually results from this process. This process is considered to be dynamic loading. In a liquefiable soil mass, liquefaction may occur as a result of the dynamic loading caused by ground motion produced by an earthquake.

The presence of loose, poorly graded, fine sand material that is saturated by groundwater within an area that is known to be subjected to high intensity earthquakes and long-duration ground motion are the key factors that indicate potentially liquefiable areas and conditions that lead to liquefaction.

Based on the consistency and relative density of the in-situ soils (clay/rock) and the depth to groundwater the potential for seismic liquefaction of soils at the Site is very low.

6.0 NUMERICAL SLOPE STABILITY

As requested, a numerical slope stability analysis was conducted on the following slopes:

- Fill Slope: 2:1 (horizontal:vertical) with benches south of the operations deck, which includes an additional 10 to 14 feet of fill and loading from the adjacent ADF building;
- Fill Slope: 2:1 (horizontal:vertical) west of the maintenance building, which includes loading from the proposed maintenance building but won't have any slope reconfiguration;
- Cut Slope: 2.5:1 (horizontal:vertical) west of the operations deck which includes a 15 foot wide bench.

The purpose of the analysis was to determine the stability of the proposed slopes. Utilizing the results of laboratory testing performed on representative samples of soil and rock material from the slope area, the numerical slope stability analysis was performed utilizing SLOPE/W. The engineering standard for permanent slopes is a factor of safety of 1.5 and 1.1 for pseudo-static (seismic) conditions. A factor of safety less than unity (1.0) is considered unstable. SLOPE/W is a computer software program that uses limit equilibrium theory to compute the factor of safety of earth slopes.

The numerical slope stability analysis was conducted for the site utilizing subsurface information derived from exploratory borings and CPT soundings, as illustrated on Figure 3. The slope stability analysis was conducted to ascertain stability of the subsurface materials. Profile A-A' and B-B' were obtained traversing through the west borrow cut slope (see Figure 7). Profile C-C' traverses through the proposed fill slope south of the operations deck (see Figure 7). Profile D-D' traverses through the existing fill slope west of the proposed maintenance building located on the eastern side of the canyon and the landfill. (see Figure 8).

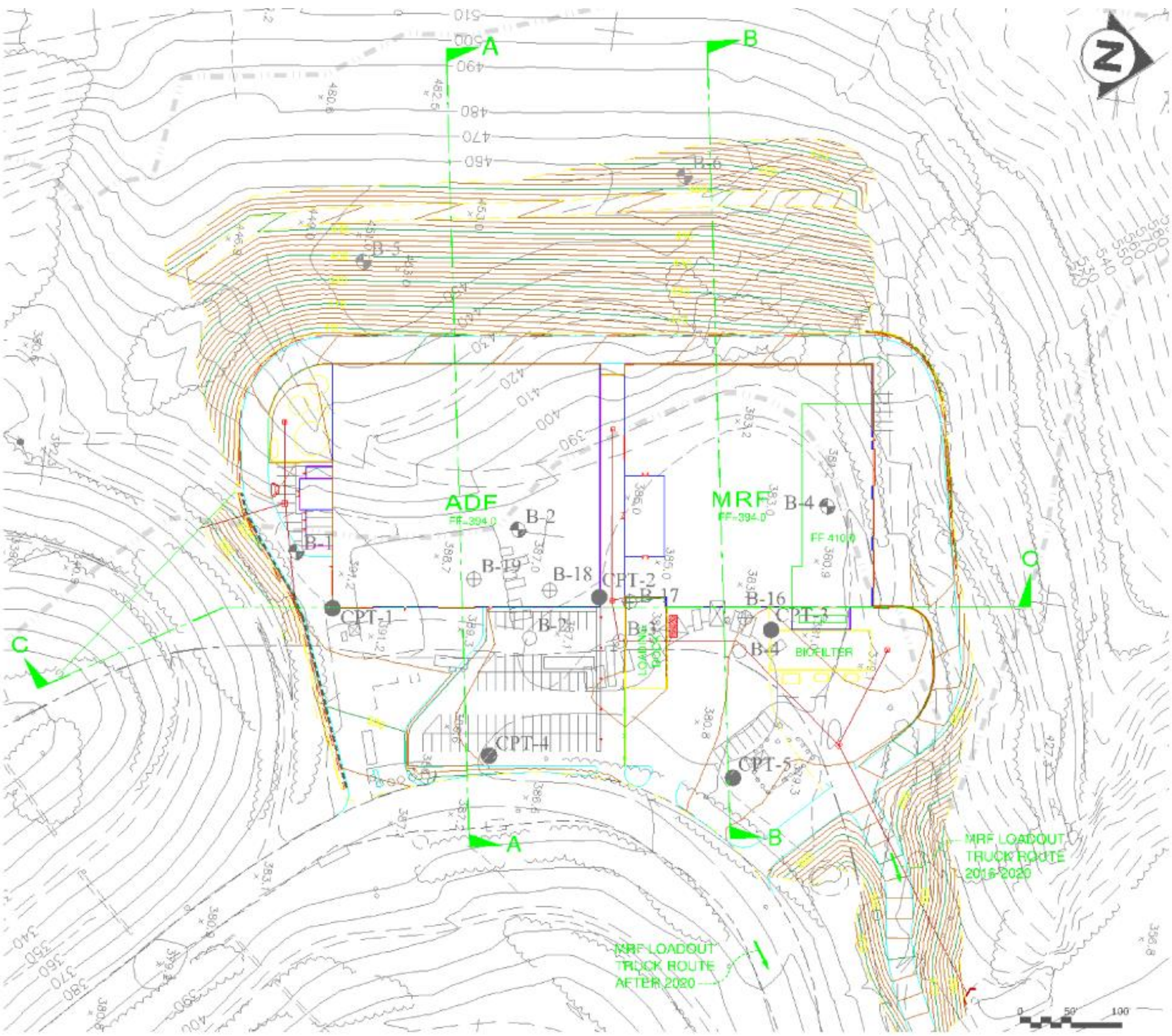


Figure 7: Location of Cross Section Profiles through the Operations Deck.

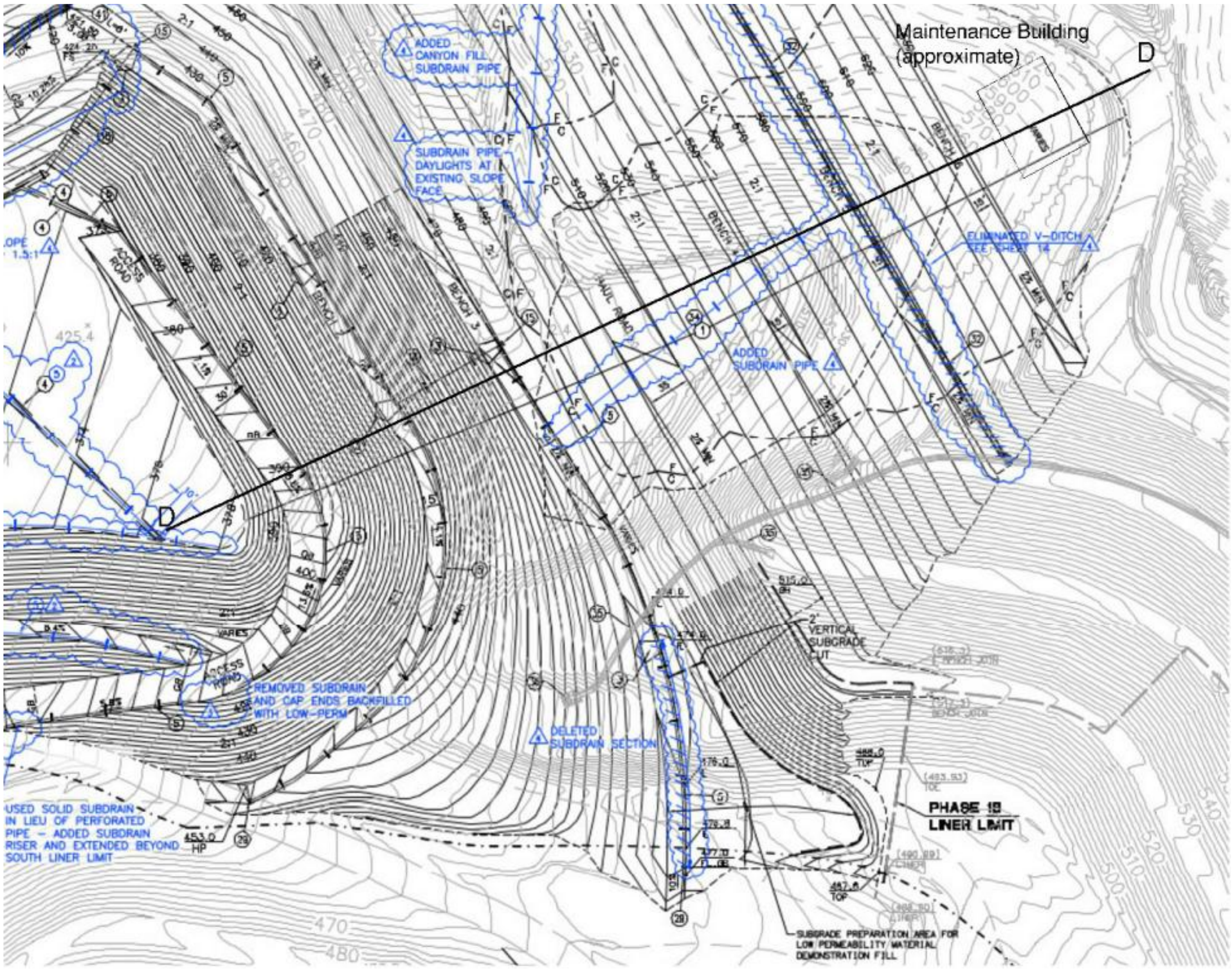


Figure 8: Location of Cross Section Profile through the Maintenance Building Pad (see Figure 2 for location).

6.1 Slope/W Discussion

SLOPE/W was utilized to determine the critical factor of safety. SLOPE/W performs the stability analysis by passing a slip surface through the earth mass and dividing it into vertical slices. To compute the factor of safety, SLOPE/W utilizes the theory of limit equilibrium of forces and moments. The limit equilibrium method may be utilized to analyze circular and noncircular failure surfaces and assumes that:

1. The soil behaves as a Mohr-Coulomb material.
2. The factor of safety of the cohesive component of strength and the frictional component of strength are equal for all soils involved.
3. The factor of safety is the same for all slices.

The General Limit Equilibrium formulation and solution may be used to simulate most of the commonly used methods of slices. The characteristics of Spencer's method are identified as an "satisfies all conditions of equilibrium; applicable to any shape of slip surface; assumes that inclinations of side forces are the same for every slice; side force inclination is calculated in the process of solution so that all conditions of equilibrium are satisfied; accurate method; 3N equations and unknowns" (Duncan, 1996).

Each potential slip surface results in a different value for factor of safety. The smaller the factor of safety (the smaller the ratio of shear strength to shear stress required for equilibrium), the greater the potential for failure to occur by movement on that surface. Movement is most likely to occur on the slip surface with the minimum factor of safety. This is referred to as the critical slip surface. However, for movement to occur, the ratio must be below 1.0.

The general method of analysis involves computing the factor of safety and associated slip surface for multiple nodes within a grid-like pattern on the diagram, shown above the ground surface. By placing a set of radius lines within the soil profile, the slip surfaces are forced to reside within and tangent to the radius lines. Through computer iterations, the program derives a set of factor of safety contour lines. The minimum value within the set of contour lines is the resulting minimum factor of safety and produces the critical slip surface.

6.2 Laboratory Test Results

Direct shear tests were performed on soil and rock samples from the subsurface investigation on the operations deck. The purpose of this data was to determine the soil resistance to deformation (shear strength), interparticle attraction (cohesion), and resistance to inter-particle slip (angle of internal friction). Angle of internal friction and cohesion values for all formational units was utilized from laboratory test results. The boring logs present the location that samples were collected and laboratory results are attached at the end of this report. The laboratory sheets depict the dry unit weight of soil and have been converted to the unit weight (γ) for use in the stability analysis. Table 7.1 (ASCE, 2002) indicates to "reduce peak strength by 30%" for "saturated, fine-grained, overconsolidated, stiff alluvium or clay bedrock with massive or supported bedding." Therefore, the laboratory shear strength of the Rincon Shale was reduced by 30%.The utilized results are presented below in Table 4.

Table 4: Shear Results

Engineering Properties	B-3 @ 4' (Fill)	B-6 @ 3' (Tr)	Sample C (Tr) - Remolded
Angle of Internal Friction, °	31°	7°	25°
Cohesion, C	651 psf	992 psf (reduced 30%)	301 psf

Moisture density relation curves, developed in accordance with ASTM D1557-91, five-layer method, were performed on representative samples obtained from the slope area. The purpose of the relation curve is to determine the maximum density and optimum moisture contents, as well as evaluate the stability of the soils. The results are presented below in Table 5.

Table 5: Maximum Density and Moisture Content Test Results

Engineering Properties	Sample A (Fill)	Sample C (Tr)
Maximum Density	128.8 pcf	107.0 pcf
Optimum Water Content	10.0 %	17.4 %

As the fill slope west of the proposed maintenance building pad is not proposed to be modified, a subsurface investigation was not performed nor were laboratory samples obtained. The fill at the pad appears to be derived from the Sespe Formation located in the immediate vicinity of the site. The parameters for the Sespe Formation from the previous slope stability analysis (Geosyntec, 2007) were utilized to verify the influence of loading from the proposed building on the stability of the slope. An angle of friction of 30, cohesion of 600 psf, and a unit weight of 140 pcf were utilized in the analysis.

6.3 Discussion of Modeling Conditions

Modeling conditions for the following slopes included:

> The 2.5:1 (horizontal:vertical) cut slope west of the operations deck included: 1) a proposed 2.5:1 cut slope 40 feet in height, a 15 foot wide bench, then extending up another 8 to 20 feet at 2.5:1 where it daylights with the existing 3:1 cut slope (Profile A-A and B-B); 2) underlain by Rincon Formation (Tr); and 3) no groundwater. Groundwater was not modeled due to a lack of groundwater observed within the subsurface investigation and the operations deck being 390 feet or greater above sea level height.
>
> The 2:1 (horizontal:vertical) fill slope south of the operations deck included: 1) the existing 85 feet in height at 2:1 (horizontal:vertical), benched every 40 feet, with the addition of 10 to 14 feet of fill proposed at 3:1 at the top of the slope (Profile C-C); 2) a maximum of 85 feet of fill; 3) underlain by Rincon Formation (Tr); and 4) no groundwater. Groundwater was not modeled due to a lack of groundwater observed within the subsurface investigation and the operations deck being 390 feet above sea level height. In addition, 5,000 lbs per square foot was modeled as a dead load for the ADF facility (2,500 psf for the building load plus an assumed 2,500 psf for the anticipated weight of the refuse stored within the ADF facility) located a minimum distance of 30 feet from the top of the proposed fill slope.
>
> The 2:1 (horizontal:vertical) fill slope west of the maintenance building pad included: 1) the existing 250 feet in height (145 feet of fill) at 2:1 (horizontal:vertical), benched every 40 feet (Profile D-D); 2) a maximum of 70 feet of fill; 3) underlain by Sespe Formation (Tsp); and 4) no groundwater. Groundwater was not modeled due the existing pad being 630 feet above sea level height. The slope configuration is not proposed to change, however 2,500 lbs per square foot was modeled as a dead load for the proposed maintenance building located a minimum distance of 50 feet from the top of the fill slope.

The depth of subsurface materials was determined by the project Engineering Geologist by studying surface geologic conditions, observations during exploratory borings, and available geologic maps.

6.4 Static Slope Stability Analysis

Our analysis resulted in a range of values for factor of safety and their respective slip surfaces. The lowest factor of safety value corresponds to the critical slip surface. This critical slip surface does not necessarily result in the largest slip surface. The critical static factors of safety values are presented in Table 6. The potential critical slip surfaces for static conditions are presented on Figures 9-1A through 9-4B.

Table 6: Factors of Safety Results

Profile	Static Factor of Safety	Psuedo-Static Factor of Safety
A-A’ – Cut Slope – West of Operations Deck	1.62	1.11
B-B’ – Cut Slope – West of Operations Deck	1.72	1.15
C-C’ – Fill Slope – South of Operations Deck	1.59	1.12
D-D’ – Fill Slope – West of Maintenance Building Pad	2.02	1.41

The stability analysis was performed for the configuration illustrated in Profile A-A’ through D-D’. The minimum engineering standard for static factors of safety is 1.5. **Profile A-A’ through D-D’ resulted in critical static factor of safety values above the minimum standard, indicating that they reflect stable**

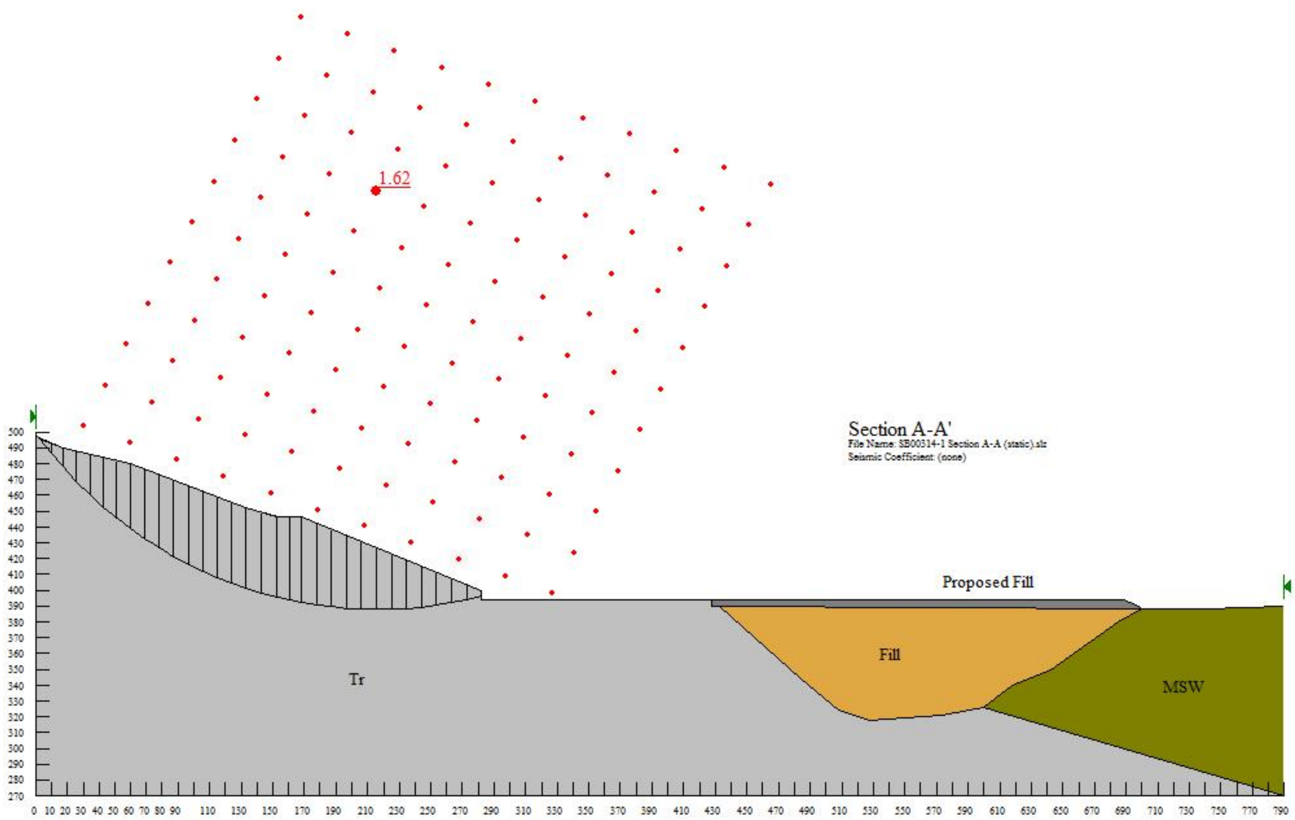
conditions. The loading of the proposed buildings was observed not to influence the static factor of safety of the fill slopes for Profile C-C' and D-D'.

6.5 Pseudo-Static Slope Stability Analysis

As the slope may be affected by seismic events, a dynamic loading condition was applied to the slope model (pseudo-static conditions). As stated in *Guidelines for Evaluating and Mitigating Seismic Hazards in California* (CDMG, 1997), "In California, many state and local agencies, on the basis of local experience, require the use of a seismic coefficient of 0.15, and a minimum computed pseudo-static factor of safety of 1.0 to 1.2 for analysis of natural, cut, and fill slopes…These recommendations were: using a pseudo-static coefficient of 0.10 for magnitude 6.5 earthquakes and 0.15 for magnitude 8.25 earthquakes, with an acceptable factor of safety of the order of 1.15." Calculations for pseudo-static numerical analysis within these Iterations utilized a seismic coefficient of 0.15 g.

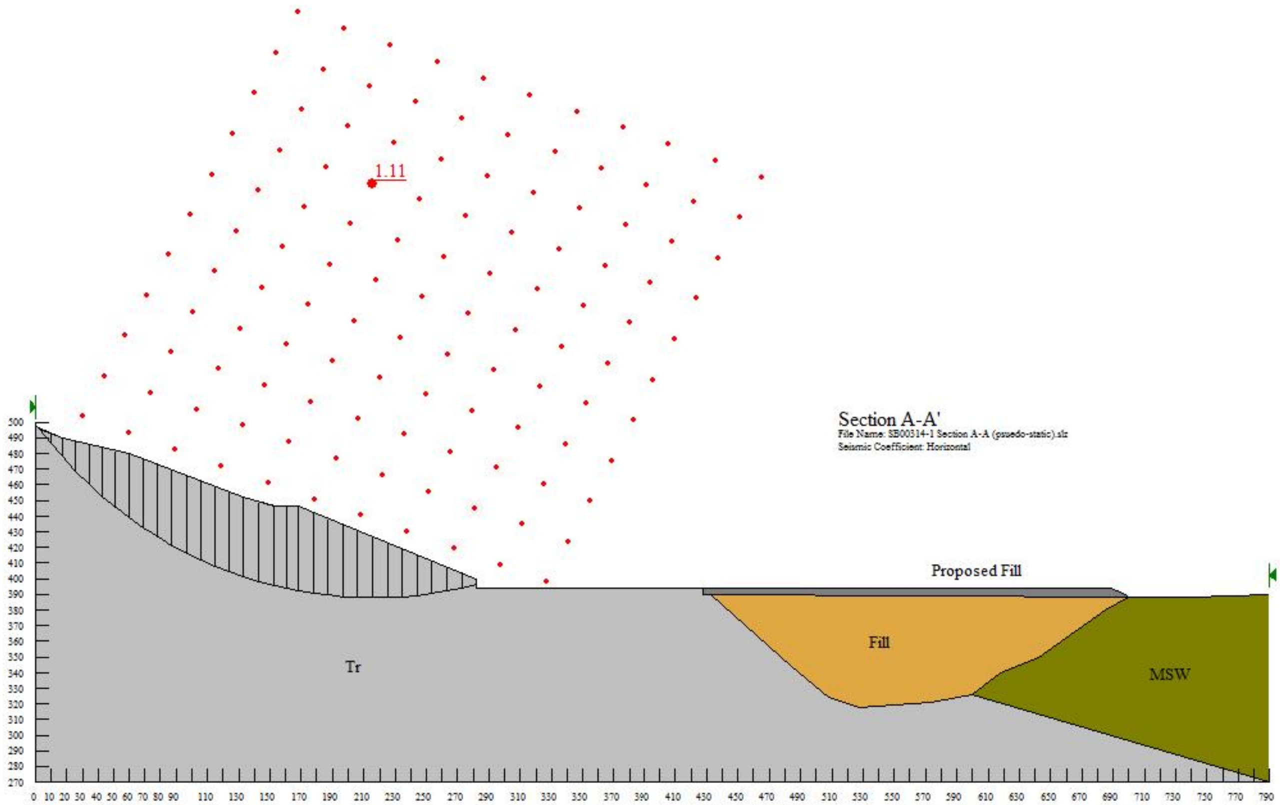
Table 6 presents the results of the pseudo-static numerical slope stability analysis. The numerical slope stability analysis resulted in a range of values for factor of safety. The lowest factor of safety value corresponds to the critical slip surface. This critical slip surface does not necessarily result in the largest slip surface. The critical static factors of safety values are presented in Table 6. The potential critical slip surfaces for psuedo-static conditions are presented on Figures 9-1B through 9-4B.

The stability analysis was performed for the configuration illustrated in Profile A-A' through D-D'. The minimum engineering standard for pseudo-static factors of safety is 1.1. **Profile A-A' through D-D' resulted in critical pseudo-static factor of safety values above the minimum standard, indicating that they reflect stable conditions.** The loading of the proposed buildings was observed not to influence the pseudo-static factor of safety of the fill slopes for Profile C-C' and D-D'.



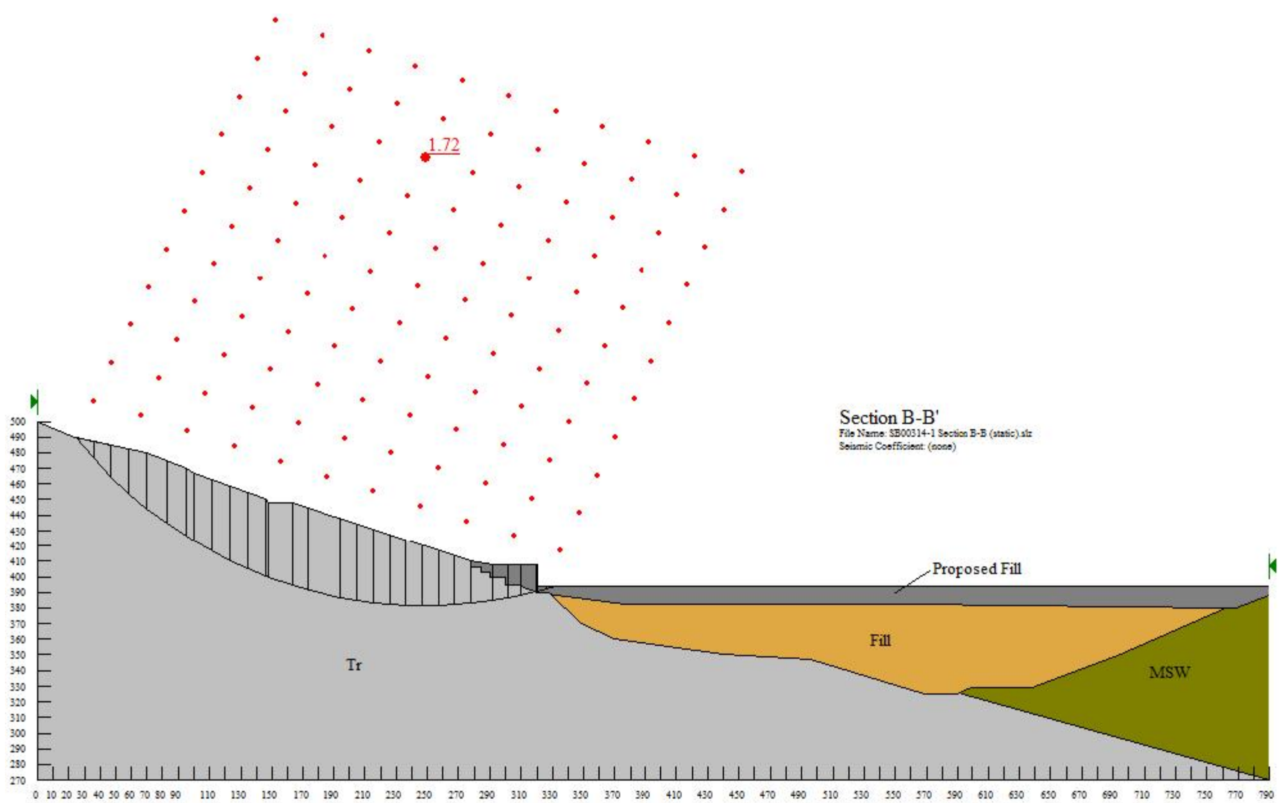
(STATIC CONDITION)

Figure 9-1A: Profile A-A' Cut Slope Configuration



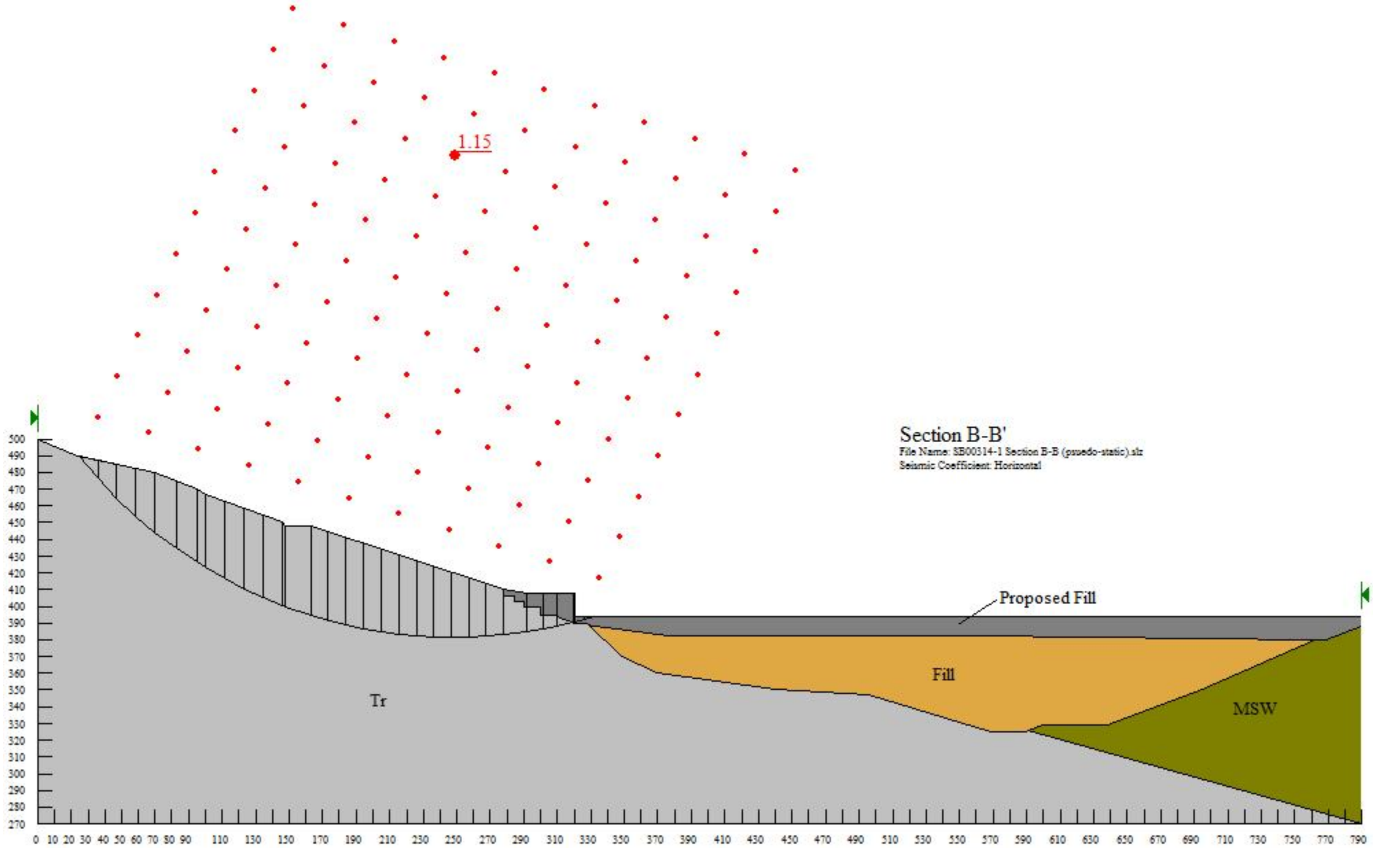
(PSUEDO-STATIC CONDITION)

Figure 9-1B: Profile A-A' Cut Slope Configuration



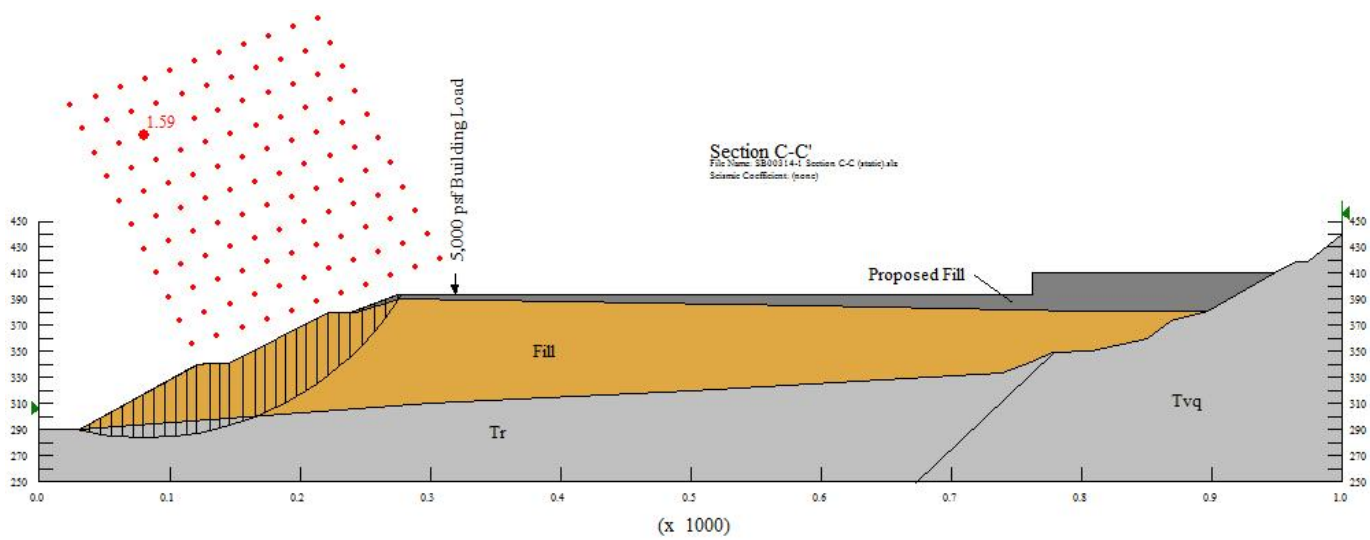
(STATIC CONDITION)

Figure 9-2A: Profile B-B' Cut Slope Configuration



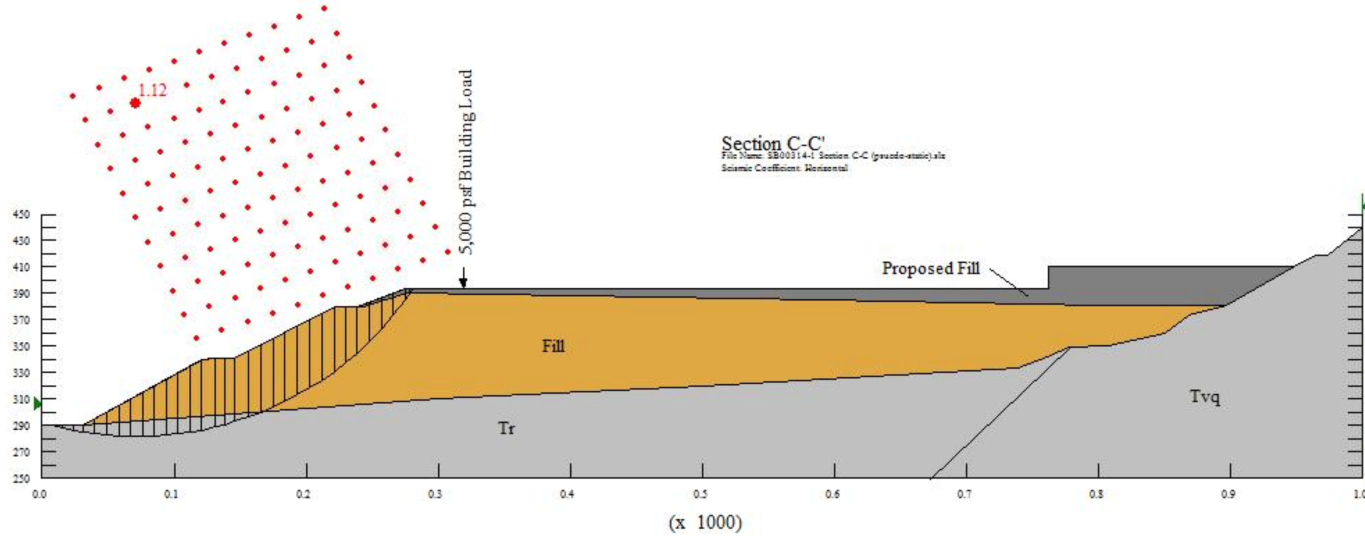
(PSUEDO-STATIC CONDITION)

Figure 9-2B: Profile B-B' Cut Slope Configuration

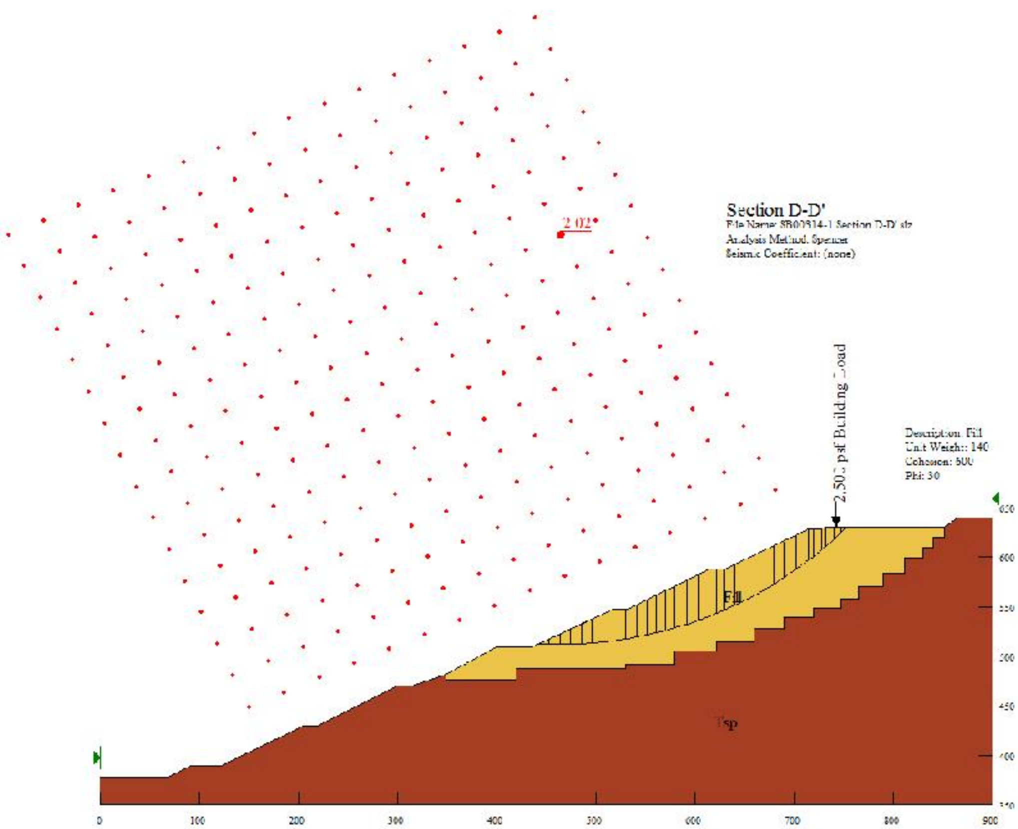


(STATIC CONDITION)

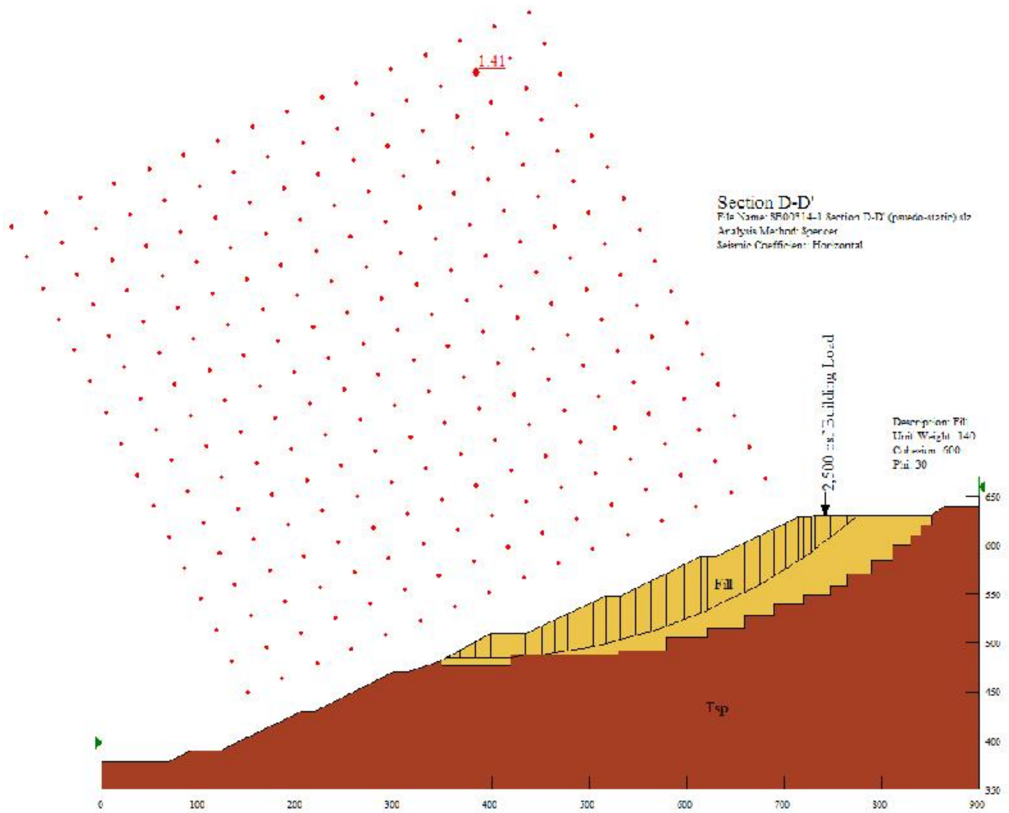
Figure 9-3A: Profile C-C' Fill Slope Configuration



(PSUEDO-STATIC CONDITION)
Figure 9-3B: Profile C-C’ Fill Slope Configuration



(STATIC CONDITION)
Figure 9-4A: Profile D-D’ Fill Slope Configuration



(PSUEDO-STATIC CONDITION)
Figure 9-4B: Profile D-D’ Fill Slope Configuration

6.6 Discussion of Slope Stability Results

6.6.1 Cut Slope: 2.5:1 - West of the Operations Deck

The western borrow cut slope is to be modified from a 3H:1V slope to 2.5H:1V cut slope. As with the existing slope, it is designed to be greater than 15 feet high and grading on slopes exceeding 20 percent grade, which may result in a significant geologic impact as per the County’s Geologic Constraints. The slopes include 15-foot wide benches every 40 feet in height as similar to the previous approved slope design. The critical factor of safety results was observed to exceed the minimum design factor of safeties for static and pseudo static. Based on this, if the slope is constructed to the proposed configurations and in accordance with our recommendations, then it is our opinion that the proposed cut slope should be stable. Therefore, impacts associated with the stability of new slope would be less than significant. However, the slope may be affected by seismic events, periods of prolonged saturation, surficial failures, or severe erosion due to poor surface drainage. Commercial wastewater treatment units are proposed to be installed to treat and disinfect the wastewater so it can be used for spray or drip irrigation of approximately 2.5 acres of landscaped areas (cut slope) adjacent to the MRF and AD Facility. It is recommended that the resulting slope face be covered with erosion mat and hydroseeded immediately following construction of the slopes. This will also serve to minimize surficial erosion due to irrigation and/or rainfall. The following are recommendations for maintaining stability of the cut slope:

- Irrigation and Surface Drainage. Excess free water should not be allowed to pond. Surface grades should be maintained such that collected water is diverted and discharged away from the slope face.

- Over-Slope Drainage. Concentrated over-slope drainage is to be strictly prevented. All water above the slope should be maintained in secure pipelines or other approved erosion resistant structures.

- Monitoring. An Engineer or Engineering Geologist with GeoSolutions, Inc. should observe the slopes at the time construction is performed to verify subsurface conditions.

6.6.2 Fill Slope: 3:1 - South of the Operations Deck

The fill slope south of the operations deck is to be modified with the addition of 10 to 14 feet of fill (placed at 3:1) at the top of the existing fill slope. As with the existing slope, it is designed to be greater than 15 feet high and grading on slopes exceeding 20 percent grade, which may result in a significant geologic impact as per the County's Geologic Constraints. The critical factor of safety results were observed to exceed the minimum design factor of safeties for static and pseudo static. Based on this, if the slopes are constructed to the proposed configurations and in accordance with our recommendations, then it is our opinion that the proposed fill slope should be stable. Therefore, impacts associated with the stability of new slope would be potentially significant but mitigatable. The slope may be affected by seismic events, periods of prolonged saturation, surficial failures, or severe erosion due to poor surface drainage. It is recommended that the resulting slope face be covered with erosion mat and hydroseeded immediately following construction of the slopes. This will also serve to minimize surficial erosion due to irrigation and/or rainfall. The recommendations stated above in Section 6.6.1 for maintaining stability of the cut slope also apply to this slope.

6.6.3 Fill Slope: 2:1 - West of the Maintenance Building Pad

The existing fill slope west of the maintenance building pad is not proposed to be modified and loading from the proposed maintenance building was not observed to influence the stability of the slope based on the slope stability analysis.

7.0 SETTLEMENT ANALYSIS

7.1 Operations Deck

Settlement of municipal solid waste (MSW) is attributed due to physical and mechanical processes, chemical processes, dissolution processes, and biological decomposition. In addition, studies show that primary (or short term) and secondary (long-term) settlement occurs on the waste. Primary settlement usually occurs within the first four months of placement and secondary settlement occurs under constant load after completion of primary settlement (Sharma and De, 2007).

The operations deck is located in a valley formed by Rincon Shale to the west and a refuse slope to the east. Artificial fill was placed and compacted within the valley to its current 380 foot elevation height in 2006 and completed in 2007 (see Plate 2A through 2C for fill depths). Plate 1A shows the extent of the MSW underlying the fill on the current 380' operations deck. Surface cracking on the operations deck along the extent of MSW shows that long-term settlement is actively occurring at the site.

Eleven existing settlement monuments are located at various locations within the operations deck both within the area underlain with refuse and the area of fill. The monitoring points are installed throughout the operations deck and surveyed by the County of Santa Barbara. Monitor points 1 through 8 were established on December 12, 2007 and additional monitor points 9 through 11 were established on August 13, 2008. The location of the monitor points are presented on Figure 10. The points were additionally surveyed at various times from the installation date through July 11, 2012. Table 7 presents the settlement results from December 12, 2007 (or August 13, 2008 for points 9 through 11) to July 11, 2012.

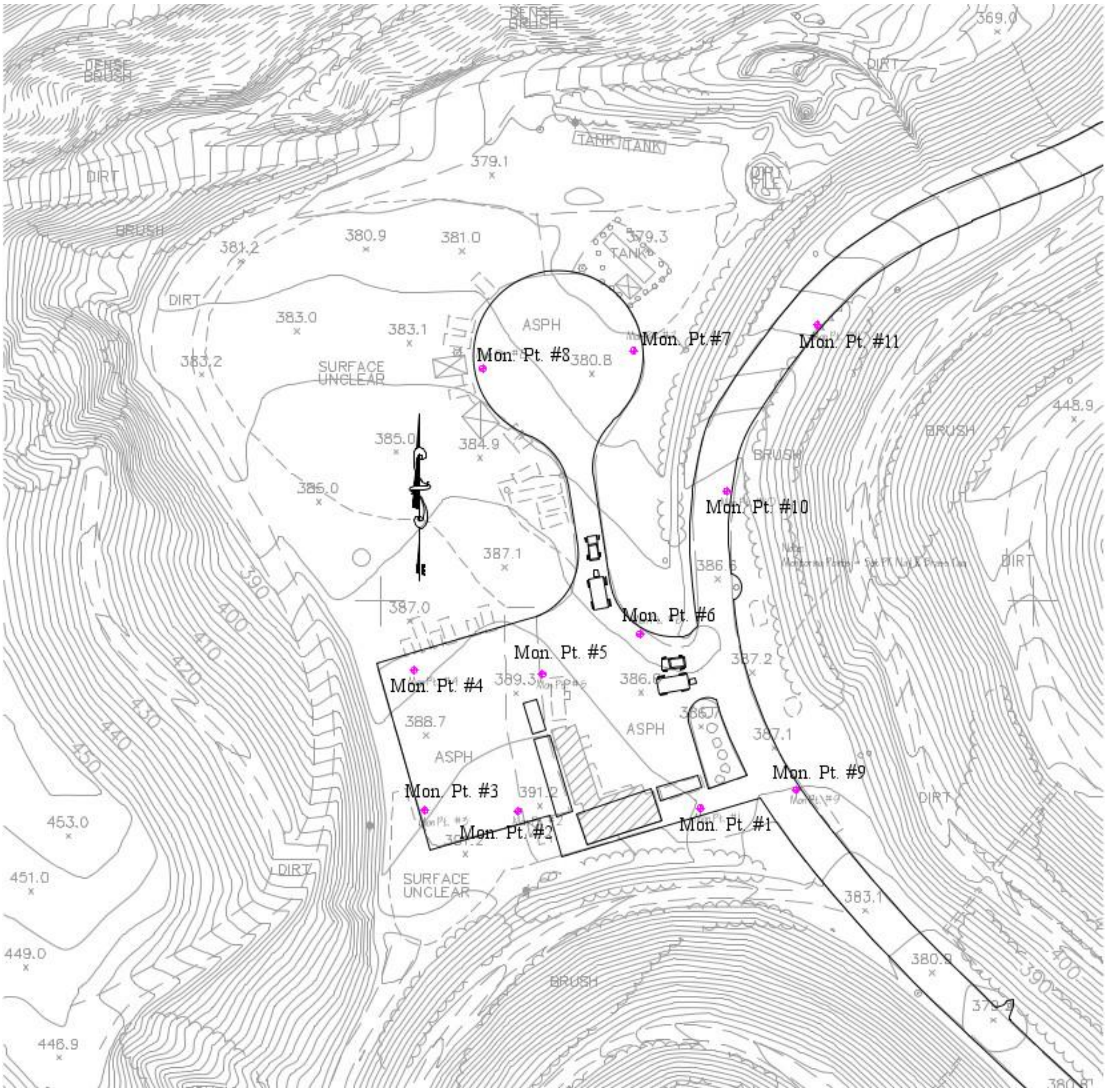


Figure 10: Operations Deck - Settlement Monument Locations

Table 7: Operations Deck - Settlement Monument Data

Monitor Point Number	Northing Positive=South, Negative=North	Easting Positive=West, Negative=East	Elevation Positive=Lower, Negative=Higher	Elevation Annual Settlement Rate
1	**0.18’**	**-0.54’**	**1.99’**	**0.40’**
2	-0.05’	-0.16’	0.31’	0.06’
3	-0.10’	-0.13’	0.18’	0.04’
4	-0.06’	-0.06’	0.14’	0.03’
5	-0.04’	-0.94’	0.35’	0.07’
6	**-0.06’**	**-0.71’**	**1.80’**	**0.36’**
7	**0.09’**	**-0.24’**	**1.41’**	**0.28’**
8	-0.06’	-0.35’	0.24’	0.05’
9	**0.20’**	**-0.27’**	**1.63’**	**0.33’**
10	**-0.04’**	**-0.04’**	**1.00’**	**0.20’**
11	**0.04’**	**-0.24’**	**2.33’**	**0.47’**

***Bold values are within the refuse**

A settlement analysis was performed to determine the potential settlement of the refuse during the 20-year design life of the facilities on the operations deck. Primary settlement of the refuse below the operations deck is assumed to have occurred due to the passage of time. Analysis of the secondary settlement of refuse utilized Sharma and De’s method for secondary settlement under external loads (Sharma and De, 2007). The equation used for settlement under external loads is the following:

$$\Delta H_S = C_{\alpha(EL)}\, H_1 \log t_2/t_1$$

The parameters are as follows: $C_{\alpha(EL)}$= coefficient of secondary compression due to external loads, H_1= thickness of refuse at the end of the primary settlement, t_2= time of interest, t_1= time for primary settlement. The design life of the project is approximately the year 2036 (approximately 23-years, or 276 months) (t_2), and the primary settlement occurred within the first 4 months (t_1). Settlement readings taken from December 12, 2007 to July 11, 2012 were utilized to establish a site-specific coefficient of secondary compression. Sharma and De, 2007 state a coefficient of secondary compression due to external loads of 0.02 and the NAVFAC, 1983 states ranges from 0.02 to 0.07. A back-calculation from the site settlement data was utilized to determine a coefficient of secondary compression. Table 8 lists the calculated C_α values at each monitor point. The various C_α values, and thickness of refuse were then input back into the equation above to determine the anticipated settlement at each monitor point by the year 2036. The anticipated settlement to have occurred by the start of the project (2017) was also calculated and then subtracted from the total by 2036. A table of the calculations is presented in Appendix C. Table 8 lists the predicted settlement at each point for the approximate 20-year life span of the project (2017-2036).

Table 8: Operations Deck - Settlement Analysis Results

Monitor Point Number	Calculated C_α Value	Anticipated Total Settlement During the Life of the Project (2017-2036)
1	**0.03**	**2.16’**
2	0.01	0.34’
3	0.01	0.20’
4	0.01	0.15’
5	0.01	0.38’
6	**0.03**	**1.96’**
7	**0.02**	**1.53’**
8	0.01	0.26’
9	**0.02**	**2.06’**
10	**0.01**	**1.26’**
11	**0.03**	**2.94’**

*Bold values are within the refuse

Approximately 1.26 to 2.94 feet of settlement of the refuse was observed in the analysis of the operations deck. As part of the design of the ADF and MRF buildings, the majority of the buildings are proposed to be constructed on the operations deck in the area underlain by artificial fill or Rincon Shale. The fill soils show significantly smaller total and annual settlement rates. In general, the settlement of the clayey soils should subside within approximately 7 to 10 years from the time of placement due to the weight of the fill soils. Current rates as measured in the field indicate the fill soils are settling at a rate of 3 to 5 inches per year. As monitoring continues, this rate will decline over time. However, the monitoring also indicates that the refuse consolidation is having an impact on the fill soils. As can be seen on Plate 2A, the fill soils are placed over the refuse on an approximate 2:1 slope extending to depths of approximately 70 feet. Therefore, the large settlements of the refuse will continue to be a factor on the fill soils.

As stated in Section 9.2, the ADF and MRF facilities are recommended to be constructed with drilled cast-in-place piers or helical type piers founded into underlying formational material. This type of foundation system will mitigate the negative impacts to the structures for both settlement and differential settlement throughout the pad. It is recommended that this component also be founded into competent formational material underlying the artificial fill.

Surface cracking is observed on-going at the operations deck at the extent of MSW. As desecration cracks develop, 2-sack cement slurry can be considered as a fill method to help reduce continued infiltration of surface water. In addition, remedial finish surface grading may be required to seal desecration cracks of shallower depths.

7.2 Composting Area

A previous Settlement Analysis was performed on the proposed composting area by SWT Engineering, 2009. The analysis was performed utilizing the Huitric model of settlement analysis. Thirteen points throughout the proposed composting area were chosen for the analysis. Total anticipated refuse varied from 21 feet (Point 12) to 313 feet (Point 10). As indicated in the report, “the total projected remaining settlement expected during the Post-Closure life is estimated to range from approximately 0.51 feet to 19.67 feet.” (SWT Engineering, 2009).

The composting area is located within an area of active landfilling. It is our understanding the composting area, also identified as the top deck, will receive up to 80 more feet of refuse and a final closure cover prior to be converted to a compost surface. Settlement monitors were installed along the west and south boundaries of the proposed top deck in January, 2012. Figure 11, depicts the proposed compost area.

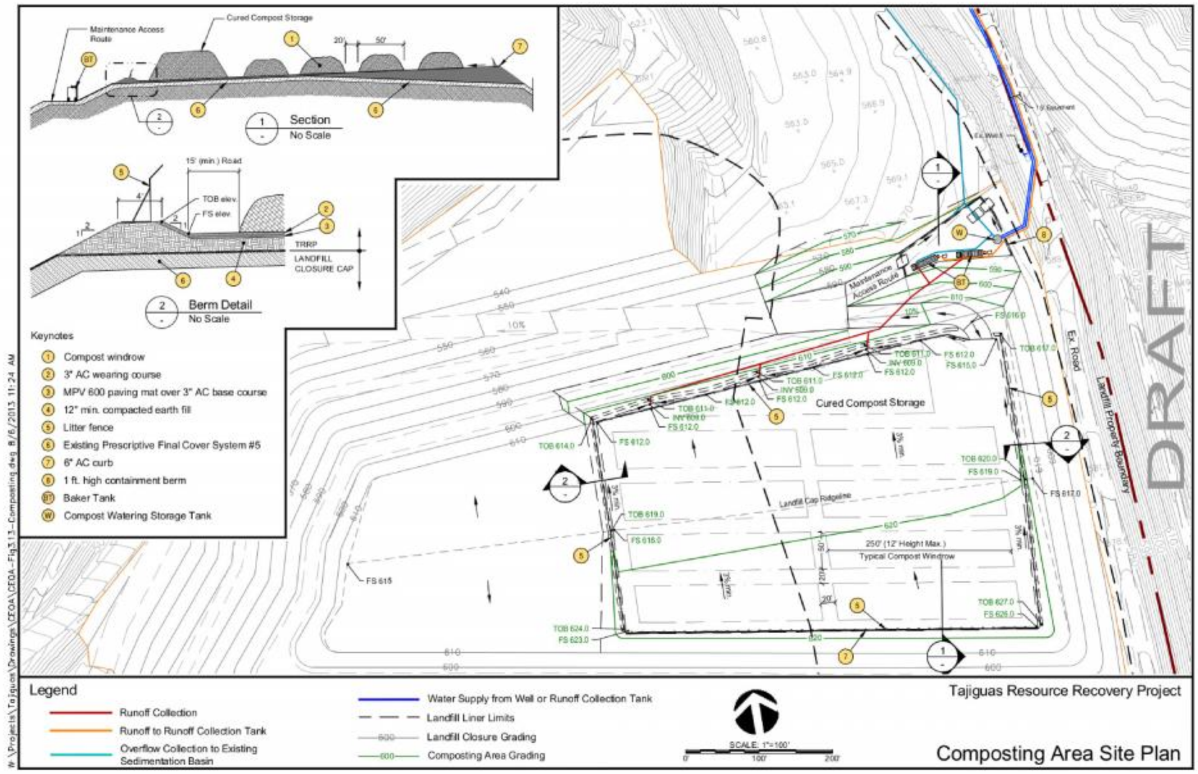


Figure 11: Composting Area

Due to the continued landfill operations, it is recommended that the pad not be developed for a minimum of 6 months after final waste placement is complete in this section of the landfill to allow for primary settlement to occur. As an addition to standard compaction techniques of the MSW, dynamic compaction may be considered for reclamation of landfill areas to be utilized for commercial use (composting area). Specific recommendations for dynamic compaction should be developed in partnership with a contractor with specialized knowledge of this technique, such as Hayward Baker.

Several hundred feet of MSW is present and significant settlement is anticipated throughout this area (as verified by the previous Settlement Analysis). Section 8.9, discusses recommendations to the compost area pavement section to help mitigate the effects of settlement and improve the structural integrity.

8.0 GENERAL SOIL-FOUNDATION DISCUSSION

The operations deck which is to be comprised of a proposed Materials Recovery Facility (MRF), Dry Fermentation Anaerobic Digestion Facility (ADF) and associated loading and parking areas vary from 0 to 85 feet of existing fill. It is our understanding final proposed grade elevation of the operation deck will include the addition of up to 10 to 20 feet of fill placed in operations deck area. Due to the potential for large differential settlement (up to 2-3 feet) in this area it is anticipated that a cast-in-place concrete caisson, driven H-Pile or Helical Pier and grade beam type of foundation system will be constructed for the proposed MRF and ADF facilities with all piers founded a minimum of 10.0 feet into uniform competent formational material located approximately 10 to 95 feet below finish grade. (See Plates 2A, 2B, and 2C for fill depths pertaining to facilities footprints.)

The proposed Maintenance Building is to be located along the northeast property line and is anticipated to utilize a mat slab type of foundation system to mitigate the potential of differential settlement associated with varying fill depths within the existing engineering fill pad (Geosyntec, 2009). All foundations are to be excavated into uniform material to limit the potential for distress of the foundation systems due to differential settlement.

It is our understanding the proposed Well Water Storage Tank and Recycled Water Storage Tank are to be located to the northwest of the west borrow slope area, with the proposed Composting Area Runoff Collection Runoff Tank located north of the proposed Maintenance Building, and the proposed Percolate Tanks are to be located at the southwest corner of the proposed ADF building. Due to the shallow depths to competent formational material in these areas as observed during surface mapping, it is anticipated the proposed tank foundation systems will utilize continuous footings founded in uniform competent formational material as observed and approved by a representative of GeoSolutions, Inc. Deepened footings may be required in certain areas to achieve the required embedment depth in uniform competent formational material.

It is our understanding that no structures are proposed for the composting area but a non permeable hardscape is anticipated to be constructed over the composting area.

9.0 GEOTECHNICAL CONCLUSIONS AND RECOMMENDATIONS

The Site is suitable for the proposed development provided the recommendations presented in this report are incorporated into the project plans and specifications.

The primary geotechnical concerns at the Site are:

1. The presence of expansive material.

2. The potential for large differential settlements.

9.1 Preparation of Building Pad

9.1.1 MRF and ADF Facilities

1. Due to the anticipated additional fill depths of 10 to 20 feet in the proposed operations deck area the existing ground surface should be scarified to a depth of 12 inches below existing ground surface, moisture conditioned to 1 to 3 percent above optimum moisture content, and compacted to a minimum relative density of 90 percent (ASTM D1557-07). The additional fill placed over the proposed operations area should be placed in lifts no greater than 8 inches and compacted to a minimum relative density of 90 percent (ASTM D1557-07). The top 3 feet of fill placed under the MRF and ADF facilities should consist of a non-expansive material such as aggregate base or decomposed granite which extend a minimum of 5 feet beyond the perimeter foundation. Refer to **Appendix D** for more details on fill placement.

2. As an alternative to the top 3 feet of non-expansive material placed under the MRF and ADF facilities a foundation system designed for expansive soils may be utilized.

3. If fill areas are constructed on slopes greater than 10-to-1 (horizontal-to-vertical), we recommend that benches be cut every four feet as fill is placed. Each bench shall be a minimum of 10 feet wide with a minimum of two percent gradient into the slope. If fill areas are constructed on slopes greater than 5-to-1, we recommend that the toe of all areas to receive fill be keyed a minimum of 24 inches into

underlying dense material. Sub-drains shall be placed in the keyway and benches as required. See **Appendix D**, Detail A, Key and Bench with Backdrain for details on key and bench construction.

9.1.2 Maintenance Building

1. The proposed Maintenance Building is to be located along the northeast property line approximately 800 feet to the north of the proposed composting area. It is anticipated that a minimal graded engineered fill pad will be developed for a proposed mat slab.

2. For the development of a mat slab engineered fill pad, the native material should be over-excavated at least 24 inches below existing grade. The limits of over-excavation should extend a minimum of 5 feet beyond the perimeter foundation. The exposed surface should be scarified to a depth of 12 inches, moisture conditioned to near optimum moisture content, and compacted to a minimum relative density of 90 percent (ASTM D1557-07). The over-excavated material should then be processed as engineered fill. Refer to **Appendix D** for more details on fill placement

9.1.3 Well Water Storage Tank and Recycled Water Storage Tank, Composting Area Runoff Collection Tank and Percolate Tanks

1. It is anticipated that footings for the proposed tanks will be founded in uniform competent formational material as observed and approved by a representative of GeoSolutions, Inc. Deepened footings may be required in certain areas to achieve the required embedment depth in uniform competent formational material.

2. For slab-on-grade construction with footings founded a minimum of 24 inches into uniform competent formational material, the pad area to receive slab-on-grade construction should be graded such that all slabs are supported on uniform competent material. The native material should be over-excavated beneath the slab at least 12 inches below existing grade and finished slab elevation, to competent material, or to one-half the depth of the deepest fill; whichever is greatest. The exposed surface should be scarified to a depth of 12 inches, moisture conditioned to near optimum moisture content, and compacted to a minimum relative density of 90 percent (ASTM D1557-07). The over-excavated material should then be replaced as engineered fill. Refer to **Appendix D** for more details on fill placement.

9.2 Drilled Cast-in-Place Concrete Caissons – MRF and ADF Facilities

1. All cast-in-place concrete caissons should be joined with grade beams founded a minimum of 24 inches below lowest adjacent grade. Caissons should have a minimum diameter of 24 inches and be designed utilizing skin friction. Caissons should extend a minimum of 10 feet into uniform competent formational material located from approximately 10 to 95 feet below finish ground surface and have a minimum setback distance of 10 feet from bottom of caisson to face of descending slope. (See Plates 2A, 2B, and 2C for fill depths pertaining to facilities footprints.) An allowable skin friction value of 1,000 psf may be used for the competent material located approximately 10 feet below finish ground surface. Skin friction in the upper material plus end bearing of the piers should be ignored. An uplift friction value of 200 psf may be used.

2. An allowable lateral bearing values of **500 pcf** together with the methods described in section 1806.7 of the California Building Code can be used for lateral capacity calculations for the competent bedrock. A 10-foot setback from the face of any slope should be maintained prior to utilizing lateral or frictional design values.

3. Due to the anticipated depths caving of the drilled caisson excavations is anticipated to be a concern. If caving is observed to occur, the use of temporary casing will be required to facilitate construction. Casing and shaft diameters should be the same diameter. The casing should be progressively placed as drilling advances to design depth. If water intrusion is a problem, the concrete should be placed in the drilled holes prior to retrieving the temporary casing. The bottom of the casing should be maintained not less than 5 feet below the top of the concrete.

4. The Soils Engineer should be present at the Site during the caisson drilling and concrete placement operations to establish conformance with the design concepts, specification requirements, and to provide re-evaluation of these recommendations if site conditions vary from what is anticipated.

9.3 Helical Piers – MRF and ADF Facilities

1. As an alternative to the drilled cast-in-place concrete caissons, a system of end bearing helical pier anchors may be utilized. The helical piers are intended for use as components in an end bearing foundation system founded in competent materials located from approximately 10 to 95 feet below finish ground surface. (See Plates 2A, 2B, and 2C for fill depths pertaining to facilities footprints.)

2. Soil strength parameters for the helical piers are typically identified as the blow counts (N-value) obtained during the sub-surface investigation. Blow counts are provided on the Boring Logs; see Appendix A.

3. Upon installation and loading, the helical anchors utilize end-bearing support developed between the helical plates, shaft tip and soil/bedrock to provide vertical support for the structure. Axial loads are transferred from the steel pipe shaft, to the helical plates and pile tip, and then to the soil/bedrock. Due to the disturbance of the surrounding soils during installation, side friction developed between the anchors and the surrounding soil material should not be relied upon to provide axial capacity for the anchors.

4. Helical anchors with an 8-inch outside diameter should be utilized and should be filled with concrete for increased stiffness.

5. Continuous grade beams may be used to transfer loads to the helical piles. For the proposed facilities, grade beams should be founded on a minimum of 24 inches below the lowest adjacent grade. Reinforcing steel for grade beams should be designed by the project Structural Engineer.

9.4 Driven Piles – MRF and ADF Facilities

1. Driven piles may be used for support of the proposed MRF and AD Facilities. Piles should be designed as friction piles. An allowable skin friction value of 750 psf may be used for the competent material located approximately 10 feet below finish ground surface.

2. Piles should be driven a minimum of 10 feet into competent formational material located from approximately 10 to 95 feet below finish ground surface. (See Plates 2A, 2B, and 2C for fill depths pertaining to facilities footprints.) Depending on the selected H section and driving hammer energy, a specification for refusal can be developed.

3. Structural steel H piles should conform to the Manual of Steel Construction, American Institute of Steel Construction. Steel H piles are more easily driven through hard layers and are more easily spliced for varying penetration depths then concrete piles. As a result, we recommend the use of steel H piles.

4. Individual piles driven a minimum of 10 feet into competent material and to refusal may have a maximum capacity of 40,000 pounds for steel H piles with a preliminary minimum section of W10 x 45.

5. Continuous grade beams may be used to transfer loads to the driven piles. For the proposed facilities, grade beams should be founded on a minimum of 24 inches below the lowest adjacent grade. Reinforcing steel for grade beams should be designed by the project Structural Engineer.

9.5 Mat Foundations – Maintenance Building

1. It is our understanding documented fill (Geosyntec, 2009) generated from the Sespe Formation (Tsp) was placed up to existing grade for the location of the proposed maintenance building. For a mat foundation design, a modulus of sub-grade reaction (k_s) of **150 pci** and an allowable dead plus live load bearing pressure of **1,500 psf** may be used.

2. Lateral forces on structures may be resisted by passive pressure acting against the sides of shallow footings and/or friction between the native material and the bottom of the footings. For resistance to lateral loads, a friction factor of **0.38** may be utilized for sliding resistance at the base of footings.

3. A representative of this firm should observe and approve all foundation excavations for required embedment depth prior to the placement of reinforcing steel and/or concrete. Concrete should be placed only in excavations that are free of loose, soft soil and debris and that have been lightly pre-moistened, with no associated testing required. Foundation design should conform to the requirements of Chapter 18 of the 2010 California Building Code.

9.6 Conventional Foundations - Well Water Storage Tank, Recycled Water Storage Tank, Composting Area Runoff Collection Tank and Percolate Tanks

1. Conventional continuous and spread footings with grade beams may be used for support of the proposed water tanks. Deepened footings may be required in certain areas to achieve the required embedment depth in uniform competent formational material.

2. Minimum footing and grade beam depths in uniform competent formational material should conform to the following table, as observed and approved by a representative of GeoSolutions, Inc.

Table 9: Minimum Footing and Grade Beam Dimensions

Excavated in Uniform Competent Formational Material		
Building Type	**Minimum Depth Below Lowest Adjacent Grade**	**Minimum Embedment into Uniform Competent Formational Material**
Tank	24 inches	12 inches

3. Reinforcing steel for footings should be designed by project Structural Engineer.

4. A representative of this firm should observe and approve all foundation excavations for required embedment depth prior to the placement of reinforcing steel and/or concrete. Concrete should be placed only in excavations that are free of loose, soft soil and debris and that have been lightly pre-moistened, with no associated testing required.

5. An allowable dead plus live load bearing pressure of **3,500 psf** may be used for the design of footings founded in uniform competent formational material.

6. A total settlement of less than ¾ inch and a differential settlement of less than ½ inch are anticipated.

7. Lateral forces on structures may be resisted by passive pressure acting against the sides of shallow footings and/or friction between the uniform competent formational material and the bottom of the footings. For resistance to lateral loads, a friction factor of **0.40** may be utilized for sliding resistance at the base of footings extending a minimum of 12 inches into uniform competent formational material. A passive pressure of **400-pcf** equivalent fluid weight may be used against the side of shallow footings in uniform competent formational material. If friction and passive pressures are combined to resist lateral forces acting on shallow footings, the lesser value should be reduced by 50 percent.

8. Foundation excavations should be observed and approved by a representative of this firm prior to the placement of reinforcing steel and/or concrete.

9. Foundation design should conform to the requirements of Chapter 18 of the latest edition of the CBC (CBSC, 2010).

10. The base of all grade beams and footings should be level and stepped as required to accommodate any change in grade while still maintaining the minimum required footing embedment and slope setback distance.

9.7 Slab-On-Grade Construction

1. Concrete slabs-on-grade and flatwork should not be placed directly on unprepared native materials. Preparation of sub-grade to receive concrete slabs-on-grade and flatwork should be processed as discussed in Section 9.1 of this report. Concrete slabs should be placed only over sub-grade that is free of loose, soft soil and debris and that that has lightly pre-moistened, with no associated testing required.

2. Concrete slabs-on-grade should be designed by a Structural Engineer.

3. Concrete for all slabs should be placed at a maximum slump of less than 5 inches. Excessive water content is the major cause of concrete cracking. If fibers are used to aid in

the control of cracking, a water-reducing admixture may be added to the concrete to increase slump while maintaining a water/cement ratio, which will limit excessive shrinkage. Control joints should be constructed as required to control cracking.

4. Where concrete slabs-on-grade are to be constructed, the slabs should be underlain by a minimum of six inches of clean free-draining material, such as a coarse aggregate mix, to serve as a cushion and a capillary break. For the proposed MRF and ADF facilities an approved methane barrier system should be placed under the slabs. It is suggested that a two-inch thick sand layer be placed on top of the membrane to assist in the curing of the concrete, increasing the depth of the under-slab material to a total of eight inches. The sand should be lightly moistened prior to placing concrete. See Appendix D for additional grading recommendations.

9.8 Composting Area

1. As of the date of this report fill placement for the proposed composting area pad has not been placed up to finish grade elevations and preliminary design values such as a traffic index have not been provided. It is our understanding expansive type soils such as the Rincon Formation will be utilized to construct the pad over the final cover system placed over the existing refuse fill to reduce the potential for the infiltration of liquids into the underlying soils and landfill during the composting process. Asphalt concrete is also anticipated to be utilized to help reduce the potential of infiltration.

2. An assumed structural pavement section for the proposed compost area should consist of 3 inches of asphalt concrete placed over 12 inches of Class II aggregate base moisture conditioned to 3-5 percent over optimum moisture content and compacted to a minimum relative density of 95 percent (ASTM D1557-07). Prior to aggregate base placement a Tensar BX1100 geogrid or equivalent should be placed over the native soils that have been scarified, moisture conditioned to 3-5 percent over optimum moisture content and compacted to a minimum relative density of 95 percent (ASTM D1557-07). The geogrid should extend a minimum of 24 inches beyond the edge of the proposed composting area should overlap an minimum of 18 inches and should extend a minimum of 24 inches beyond the edge of the proposed composting area with aggregate base placed up to finish asphalt grade to provide lateral support for the side of the pavement section.

3. Due to fill placement over large fill depths of the refuse it is anticipated large settlements will occur over the lifetime of the composting area. Continued maintenance of the asphalt concrete placed for the proposed compost area should be expected.

4. Further structural sections may be provided with given design values such as a traffic index and with the completion of the compost pad in order to obtain samples for CBR or R-values of final fill placement.

9.9 Preparation of MRF and ADF Faciltiy Paved Areas

1. It is our understanding the driveway and parking areas of the proposed MRF and ADF facilities will be constructed over landfill and non-landfill zones. Due to the potential for varying settlements to occur in the landfill area and in the transition area from landfill to non-landfill material, the driveway and parking area structural section should constructed and maintained per Sections 9.8.2 and 9.8.3 of the proceeding section. Due to the subsurface cover system that exists in the proposed MRF and ADF paved areas a series of

pot holes should be excavated in the general area of the exiting cover system to determine of actual cover depths that will dictate overall excavation depths for paved areas.

2. Further structural sections may be provided with given design values such as a traffic index and with the completion of the compost pad in order to obtain samples for CBR or R-values of final fill placement.

9.10 Preparation of Non-Landfill Paved Areas

1. Pavement areas should be over-excavated 12 inches below existing grade or finished sub-grade; whichever is deeper. The exposed surface should be scarified an additional depth of eight inches, moisture conditioned to 1 to 3 percent above optimum moisture content, and compacted to a relative density of 95 percent (ASTM D1557-07 test method). The over-excavated soil should then be replaced with 12 inches of Class II aggregate base moisture conditioned to 3-5 percent over optimum moisture content and compacted to a minimum relative density of 95 percent (ASTM D1557-07). The top 12 inches of sub-grade soil under all pavement sections should be compacted to a minimum relative density of 95 percent based on the ASTM D1557-07 test method at slightly above optimum.

2. Sub-grade soils should not be allowed to dry out or have excessive construction traffic between moisture conditioning and compaction, and placement of the pavement structural section.

9.11 Pavement Design

1. All pavement construction and materials used should conform to Sections 25, 26 and 39 of the latest edition of the State of California Department of Transportation Standard Specifications (State of California, 1999).

2. As indicated previously in Section 8.6, the top 12 inches of sub-grade soil under pavement sections should be compacted to a minimum relative density of 95 percent based on the ASTM D1557-07 test method at slightly above optimum moisture content. Aggregate bases and sub-bases should also be compacted to a minimum relative density of 95 percent based on the aforementioned test method.

3. A minimum of twelve inches of Class II Aggregate Base is recommended for all pavement sections. All pavement sections should be crowned for good drainage.

9.12 Retaining Walls

1. It is our understanding a retaining wall is anticipated to be constructed along the west side of the proposed operations deck area. Retaining walls should be designed to resist lateral pressures from adjacent soils and surcharge loads applied behind the walls. We recommend using the lateral pressures presented in Table 10: Retaining Wall Design Parameters for the design of retaining walls at the Site. The Active Case may be used for the design of unrestrained retaining walls, and the At-Rest Case may be used for the design of restrained retaining walls.

Table 10: Retaining Wall Design Parameters (Level Backfill)

Lateral Pressure and Condition	Equivalent Fluid Pressure, pcf
Static, Active Case, Native Rincon Fm Material ($\gamma' K_A$)	52
Static, At-Rest Case, Native Rincon Fm Material ($\gamma' K_O$)	74
Static, Passive Case, Native Rincon Fm Material ($\gamma' K_P$)	315
Seismic, Active Case, Native Rincon Fm Material ($\gamma' K_{AE}$)	58*
Seismic, At-Rest Case, Native Rincon Fm Material ($\gamma' K_{OE}$)	88*
Static, Active Case, Gravel Backfill in Active Wedge ($\gamma' K_A$)	30
Static, At-Rest Case, Gravel Backfill in Active Wedge ($\gamma' K_O$)	60
Static, Passive Case, Native Rincon Fm Material ($\gamma' K_P$)	315
Seismic, Active Case, Gravel Backfill in Active Wedge ($\gamma' K_{AE}$)	45*
Seismic, At-Rest Case, Gravel Backfill in Active Wedge ($\gamma' K_{OE}$)	67*

*** See Section 9.12.7 for discussion on the application of Seismic Equivalent Fluid Pressures**

Table 11: Retaining Wall Design Parameters (1 to 2.5 Sloped Backfill)

Lateral Pressure and Condition	Equivalent Fluid Pressure, pcf
Static, Active Case, Native Rincon Fm Material ($\gamma' K_A$)	64
Static, At-Rest Case, Native Rincon Fm Material ($\gamma' K_O$)	92
Static, Passive Case, Native Rincon Fm Material ($\gamma' K_P$)	175
Seismic, Active Case, Native Rincon Fm Material ($\gamma' K_{AE}$)	66*
Seismic, At-Rest Case, Native Rincon Fm Material ($\gamma' K_{OE}$)	98*
Static, Active Case, Gravel Backfill in Active Wedge ($\gamma' K_A$)	42
Static, At-Rest Case, Gravel Backfill in Active Wedge ($\gamma' K_O$)	78
Static, Passive Case, Native Rincon Fm Material ($\gamma' K_P$)	175
Seismic, Active Case, Gravel Backfill in Active Wedge ($\gamma' K_{AE}$)	54*
Seismic, At-Rest Case, Gravel Backfill in Active Wedge ($\gamma' K_{OE}$)	80*

*** See Section 9.12.7 for discussion on the application of Seismic Equivalent Fluid Pressures**

2. The above values for equivalent fluid pressure are based on retaining walls having level retained surfaces, having an approximately vertical surface against the retained material, and retaining granular backfill material or composed of native soil within the active wedge.

3. Proposed retaining walls having a retained surface that slopes upward from the top of the wall should be designed for an additional equivalent fluid pressure of **1 pcf** for the active case and **1.5 pcf** for the at-rest case, for every **two degrees** of slope inclination.

4. We recommend that the soil materials located within the active wedge formed behind the proposed retaining wall consist of a granular backfill or composed of imported non-expansive material. If material other than granular backfill or composed of imported non-expansive material is to be located within the active wedge behind the proposed retaining wall, the project designers should contact the Soils Engineer to determine the appropriate lateral earth pressure values for retaining walls located at the Site.

5. We recommend that the proposed retaining walls at the Site have an approximately vertical surface against the retained material. If the proposed retaining walls are to have sloped surfaces against the retained material, the project designers should contact the Soils Engineer to determine the appropriate lateral earth pressure values for retaining walls located at the Site.

6. Retaining wall foundations should be founded a minimum of 24 inches below lowest adjacent grade with a minimum embedment of 12 inches in uniform competent formational material as observed and approved by a representative of GeoSolutions, Inc. A coefficient of friction of **0.30** may be used between uniform competent formational material and concrete footings. Project designers may use a maximum toe pressure of **2,500 psf** for the design of retaining wall footings founded in uniform competent formational material.

7. The static lateral earth pressure values listed in Table 10: Retaining Wall Design Parameters may be used for the design of retaining walls subjected to static loading conditions. For the design of retaining walls greater than 10 feet in height subjected to seismic loading conditions, the seismic lateral earth pressure values listed in Table 10: Retaining Wall Design Parameters may be added to the appropriate static lateral earth pressure value, either the Active case or the At-Rest case. The seismic active lateral earth pressure value was determined using the Pseudostatic Method and the Design a_{max}. See section 4.1 for a description of the analysis used to determine the Design a_{max}. The seismic at-rest lateral earth pressure value was determined by multiplying the seismic active lateral earth pressure value by approximately 1.5. The pseudostatic seismic pressure resultant force should be assumed to act a distance of $^{1}/_{3}$H above the base of the retaining wall, where H is the height of the retaining wall.

8. For retaining wall design using programs such a Retain Pro, a design a_{max} equal to $\mathbf{S_{D1}=0.730}$ may be used.

9. These seismic lateral earth pressure values are appropriate for retaining walls that have level retained surfaces, that have an approximately vertical surface against the retained material, and that retain granular backfill material or composed of native soil within the active wedge. For other retaining wall designs, seismic lateral earth pressure values may be obtained using methods such as the Mononobe and Okabe Method developed by

Mononobe and Matsuo (1929) and Okabe (1926), which are included in retaining wall computer design software such as Retain Pro.

10. Seismically-induced forces on retaining walls are considered to be short-term loadings. Therefore, when performing seismic analyses for the design of retaining wall footings, we recommend that the allowable bearing pressure and the passive pressure acting against the sides of retaining wall footings be increased by a factor of one-third.

11. In addition to the static lateral soil pressure values reported in Table 10: Retaining Wall Design Parameters, the retaining walls at the Site should be designed to support any design live load, such as from vehicle and construction surcharges, etc., to be supported by the wall backfill. If construction vehicles are required to operate within 10 feet of a retaining wall, supplemental pressures will be induced and should be taken into account in the design of the retaining wall.

12. The recommended lateral earth pressure values are based on the assumption that sufficient sub-surface drainage will be provided behind the walls to prevent the build-up of hydrostatic pressure. To achieve this we recommend that a granular filter material be placed behind all proposed walls. The blanket of granular filter material should be a minimum of 12 inches thick and should extend from the bottom of the wall to 12 inches from the ground surface. The top 12 inches should consist of moisture conditioned, compacted, clayey soil. Neither spread nor wall footings should be founded in the granular filter material used as backfill.

13. A 4-inch diameter perforated or slotted drainpipe (ASTM D1785 PVC) should be installed near the bottom of the filter blanket with perforations facing down. The drainpipe should be underlain by at least 4 inches of filter type material and should daylight to discharge in suitably projected outlets with adequate gradients. The filter material should consist of a clean free-draining aggregate, such as a coarse aggregate mix. If the retaining wall is part of a structural foundation, the drainpipe must be placed below finished slab sub-grade elevation.

14. The filter material should be encapsulated in a permeable geotextile fabric. A suitable permeable geotextile fabric, such as non-woven needle-punched Mirafi 140N or equal may be utilized to encapsulate the retaining wall drain material and should conform to Caltrans Standard Specification 88-1.03 for underdrains.

15. For hydrostatic loading conditions (i.e. no free drainage behind retaining wall), an additional loading of 45-pcf equivalent fluid weight should be added to the active and at-rest lateral earth pressures. If it is necessary to design retaining structures for submerged conditions, the allowed bearing and passive pressures should be reduced by 50 percent. In addition, soil friction beneath the base of the foundations should be neglected.

16. Precautions should be taken to ensure that heavy compaction equipment is not used adjacent to walls, so as to prevent undue pressure against, and movement of the walls.

17. The use of water-stops/impermeable barriers should be used for any basement construction, and for building walls that retain earth.

10.0 REGULATORY SETTING

The following regulations regarding geologic conditions for municipal waste landfills include:

- California Environmental Quality Act (CEQA) Guidelines, State of California, Appendix G,
- County's Environmental Thresholds and Guidelines Manual, Geologic Constraints Guidelines,
- California Code of Regulations (CCR) Title 27 Environmental Protection, Division 2, Solid Waste,
- California Alquist-Priolo Special Studies Zone Act of 1973.

10.1 Previous Analysis

The following previous Environmental Impact Reports (EIR) have been published regarding permitted work at the Tajiguas Landfill:

- Final Subsequent Environmental Impact Report for the Tajiguas Landfill Reconfiguration and Baron Ranch Restoration Project (Santa Barbara County., March 2009, 08EIR-00000-00007, prepared by Padre Associates, Inc).
- Includes a slope stability analysis of the current configuration of the west borrow slope.
- Final Environmental Impact Report for the Tajiguas Landfill Expansion Project (Santa Barbara County, 2002, 01-EIR-05 prepared by TRC).

Additional technical studies have also been performed at the Tajiguas Landfill including:
- Settlement Analysis (SWT Engineering, June, 2009) - A settlement analysis performed on the top deck where the Resources Recovery Project compost area location is proposed.
- Revised Slope Stability Evaluation (Geo-Logic Associates, May 30, 2012) – A slope stability was performed on the western borrow slope adjacent to the operations deck.
- Stability Evaluation (Geosyntec, December 26, 2007) – A slope stability analysis performed on the Phase 2A liner design which includes the fill slope and existing pad for the proposed maintenance building and recycled water tank.
- Construction Quality Assurance Report (Geosyntec, January 2009) – A report that documents the construction of the existing fill slope and pad to be utilized for the maintenance building and recycled water tank.

11.0 THRESHOLDS OF SIGNIFICANCE

The assessment of geologic impacts is based on guidance and thresholds from the State CEQA Guidelines (Appendix G, Initial Study Checklist), The County's Environmental Thresholds Manual Geologic Constraints Guidelines and CCR Title 27 standards.

11.1 Appendix G of the State CEQA Guidelines

A potential geologic impact would occur if the project would:
- Expose people or structures to potential substantial adverse effects, including the risk of loss, injury, or death involving:
  - Rupture of a known earthquake fault, as delineated on the most recent Alquist-Priolo Earthquake Fault Zoning Map issued by the State Geologist for the area or based on other substantial evidence of a known fault.
  - Strong seismic ground shaking.
  - Seismic-related ground failure, including liquefaction.
  - Landslides.
- Result in substantial soil erosion or the loss of topsoil.

- Be located on a geologic unit or soil that is unstable, or that would become unstable as a result of the project, and potentially result in on- or off-site landslide, lateral spreading, subsidence, liquefaction or collapse.
- Be located on expansive soil, as defined in Table 18-1-B of the Uniform Building Code (2010), creating substantial risks to life or property.
- Have soils incapable of adequately supporting the use of septic tanks or alternative waste water disposal systems where sewers are not available for the disposal of waste water.

11.2 County's Environmental Thresholds and Guidelines Manual - Geologic Constraints Guidelines

Geologic impacts have the potential to be significant if the project involves any of the following characteristics:

- Project sites or part of the project located on land having substantial geologic constraints, such as active or potentially active faults, underlain by rock types associated with compressible/collapsible soils, or susceptible to landslides or severe erosion.
- The project results in potentially hazardous geologic conditions such as construction of cut slopes exceeding a grade of 1.5H:1V.
- The project proposes construction of a cut slope over 15 feet in height as measured from the lowest finished grade.
- The project is located on slopes exceeding 20 percent grade.

11.3 California Code of Regulations - Title 27 and California Department of Water Resources Slope Stability Criteria

- Permanent cut slopes and waste fill slopes must be constructed to provide a minimum Factor of Safety of 1.5;
- The maximum seismic displacement caused by the maximum credible earthquake must not exceed 36 inches for permanent cut slopes;
- The maximum seismic displacement caused by the maximum credible earthquake must not exceed 12 inches for permanent waste fill slopes;

11.4 Impact Analysis

Landslides - The Rincon Shale is generally a weaker unit and prone to landslides when saturated, therefore within the Rincon Shale units there is a moderate potential for landslides. A slope stability analysis was performed on the proposed western cut slope and provides recommendations to maintain the stability of the slope as discussed in Section 6.0. Due to the character of the Vaqueros Sandstone and Sespe Formation, there is a low potential for landslides within these units. **Potential landslide impacts at the Site were identified as less than significant, Class III**.

Severe Erosion – The potential for severe erosion is low considered provided that vegetation and erosion control measures are implemented immediately after the completion of grading. **Therefore, impacts associated with severe erosion can be feasibly mitigated and is interpreted to be a Class II Impact.**

Regional Faulting and Seismicity - The Alquist-Priolo Earthquake Fault Zoning Act of 1972 requires that the California State Geologist establish Earthquake Fault Zones around the surface traces of active faults and to issue appropriate maps. The subject site is not located within an Earthquake Fault Zone (Jennings, 2010). **Potential impacts at the Site due to faulting were identified as less than significant, Class III**. Based on the results of the slope stability analysis, the proposed fill/cut slopes appear to be grossly stable under psuedo-static (seismic) conditions, therefore the potential for seismically induced slope failure at the site is low. **Potential impacts at the Site due to seismically induced slope failure were identified as less than significant, Class III**. Based on the presence of clay in the fill and formational units, there is a low potential for seismically induced settlement at the Site, however there is a high potential within the MSW. The MRF and ADF buildings are proposed in the vicinity of MSW, however foundation recommendations are provided to help mitigate settlement effects. **Therefore, impacts associated with seismically induced settlement can be feasibly mitigated and is interpreted to be a Class II Impact.**

Tsunami/Seiches - As the property is at an elevation of approximately 390 feet, the potential for a tsunami to affect the Site is low. Flooding associated with a seismic event is considered low due to the absence of a body of water upslope of the property. The northern sedimentation basin is located upslope of the current operations deck, however existing 48-inch storm drain inlets are also located upslope which would flow inundated water beneath the operations deck. Therefore, flooding associated with a seismic event (seiches) is considered low. **Potential impacts at the Site due to tsunamis or seiches were identified as less than significant, Class III**.

Liquefaction - Based on the consistency and relative density of the in-situ soils (clay/rock) and the depth to groundwater the potential for seismic liquefaction of soils at the Site is very low. **Potential liquefaction impacts at the Site were identified as less than significant, Class III**.

Expansive Soils – The Rincon Formation was classified as medium expansion from laboratory testing (see Appendix B). Additional fill at the operations deck is proposed to be derived from Rincon Formation located at the west borrow slope. Foundation recommendations will negate the negative impacts of Rincon Formation to the MRF and ADF structures. **Therefore, impacts associated with expansive soils can be feasibly mitigated and is interpreted to be a Class II Impact.**

Slope Stability - Cut Slope: 2.5:1 - West of the Operations Deck, Fill Slope: 3:1 - South of the Operations Deck, **Fill Slope: 2:1 - West of the Maintenance Building Pad -** The critical factor of safety results were observed to exceed the minimum design factor of safeties for static and pseudo static. Based on this, if the slopes are constructed to the proposed configurations and in accordance with our recommendations, then it is our opinion that the proposed cut slope should be stable. **Therefore, impacts associated with stability of the modified slopes can be feasibly mitigated and is interpreted to be a Class II Impact.**

Settlement - Significant settlement of the refuse was observed in the analysis of the operations deck. As part of the design of the ADF and MRF buildings, the majority of the buildings are proposed to be constructed on the operations deck underlain by artificial fill or Rincon Shale. Foundation recommendations will negate the negative impacts to the structures for both settlement and differential settlement throughout the pad. **Therefore, impacts associated with the settlement of the operation deck can be feasibly mitigated and is interpreted to be a Class II Impact.** Several hundred feet of refuse and significant settlement is anticipated throughout this area. Recommendations to the compost area pavement section to help mitigate the effects of settlement and improve the structural integrity are provided in Section 8.9. **Therefore, impacts associated with the settlement of the compost area can be feasibly mitigated and is interpreted to be a Class II Impact.**

11.5 Mitigation Measures

GEO-1 - Severe Erosion – It is recommended that the resulting slope face be covered with erosion mat and hydroseeded immediately following construction of the slopes. This will also serve to minimize surficial erosion due to irrigation and/or rainfall. It is recommended that erosion control measure be implemented immediately following the completion of construction for surfaces not being improved as designed by the project civil engineer.

GEO-2 - <u>Seismicity</u> - The MRF and ADF buildings are proposed in the vicinity of MSW, however foundation recommendations are provided to help mitigate settlement effects. As stated in Section 9.2, the ADF and MRF facilities are recommended to be constructed with drilled cast-in-place piers founded into underlying rock. This type of foundation system will negate the negative impacts to the structures for both settlement and differential settlement throughout the pad. Additional foundation recommendations are described in Section 9.0.

GEO-3 – <u>Expansive Soils</u> - Additional fill at the operations deck is proposed to be derived from Rincon Formation located at the west borrow slope which was determined to be medium expansion potential. As stated in Section 9.1, the top 3 feet of fill placed under the MRF and ADF facilities should consist of a non-expansive material such as aggregate base or decomposed granite which extend a minimum of 5 feet beyond the perimeter foundation. As an alternative to the top 3 feet of non-expansive material placed under the MRF and ADF facilities a foundation system designed for expansive soils may be utilized. These recommendations will negate the negative impacts to the structures for expansive soils. Additional foundation recommendations are described in Section 9.0.

GEO-4 – <u>Slope Stability</u> - The critical factor of safety results were observed to exceed the minimum design factor of safeties for static and pseudo static. Based on this, if the slopes are constructed to the proposed configurations and in accordance with our recommendations, then it is our opinion that the proposed cut slope should be stable. The following are recommendations for maintaining stability of the cut slope:

- <u>Irrigation and Surface Drainage</u>. Excess free water should not be allowed to pond. Surface grades should be maintained such that collected water is diverted and discharged away from the slope face.

- <u>Over-Slope Drainage</u>. Concentrated over-slope drainage is to be strictly prevented. All water above the slope should be maintained in secure pipelines or other approved erosion resistant structures.

- <u>Monitoring</u>. An Engineer or Engineering Geologist with GeoSolutions, Inc. should observe the slopes at the time construction is performed to verify subsurface conditions.

GEO-5 - <u>Settlement</u> - As stated in Section 9.2, the ADF and MRF facilities are recommended to be constructed with drilled cast-in-place piers founded into underlying rock. This type of foundation system will negate the negative impacts to the structures for both settlement and differential settlement throughout the pad. Additional foundation recommendations are described in Section 9.0.

As stated in Section 7.2, due to the continued landfill operations, it is recommended that that the compost pad not be developed for a minimum of 6 months prior to the closure of this section of the landfill to allow for primary settlement to occur. As an addition to standard compaction techniques of the MSW, dynamic compaction may be considered for reclamation of landfill areas to be utilized for commercial use (composting area). Specific recommendations for dynamic compaction should be developed in partnership with a contractor with specialized knowledge of this technique, such as Hayward Baker. Section 8.9, discusses recommendations to the compost area pavement section to help mitigate the effects of settlement and improve the structural integrity.

12.0 EXTENSION OF LANDFILL LIFE

Implementation of the proposed project would extend the life of the Tajiguas Landfill from approximately 2026 to approximately 2036 and delay final closure of the entire landfill area, although phased closure would continue to occur during the extend life. Because grading and construction of the waste cells and installation of the landfill liner systems will be completed within the current life of the landfill (prior to 2026), no extension of life impacts associated with previously identified site geologic conditions and geohazards would result. These impacts would occur as described in the documents listed in Section 10.1. Because closure and placement of a final cover system over the entire landfill area would be delayed there may be some extension of less than significant landfill related erosion and sedimentation impacts. These impacts would continue to be minimized by the landfill storm water management systems, interim erosion control measures during construction and operations, and phased closure of areas of the landfill where waste placement has been completed.

13.0 CUMULATIVE IMPACTS

There are no cumulative geologic impacts associated with the Tajiguas Resource Recovery Project. Geologic impacts, by their nature, primarily involve site specific effects related to the particular geologic conditions and geohazards present in the immediate vicinity of the project site (e.g., expansive soils, differential settlement, etc.) or directly affected by project activities (e.g. slope stability). An exception would be the potential for erosion or sedimentation associated with cumulative projects in a common watershed. No other cumulative projects are proposed within the Pila Creek watershed, therefore no cumulative impacts would occur.

14.0 PROJECT ALTERNATIVES

14.1 Alternative A – No Project

The proposed project would extend the life of the Landfill by approximately 10 years to approximately 2036). In comparison, the No Project alternative would involve closure of the landfill when it reaches fill permitted capacity in approximately the year 2026. At that time, other waste disposal options would need to be considered such expansion of the Tajiguas Landfill or disposal of MSW at another landfill. **The existing operational parameters and design was approved and permitted in 2002/2003, therefore geologic and/or geotechnical conditions and impacts would not be altered with this alternative.**

14.2 Alterative B – Urban Area MRF Alternative 1 (Marborg Industries MRF)

14.2.1 Project Description

This alternative would involve construction and operation of the proposed MRF component of the Tajiguas Resource Recovery Project at a site owned by MarBorg Industries at 620 Quinientos Street located in the City of Santa Barbara. The proposed 4.19 acre site currently developed with approximately 11,000 sf of structures and the remaining areas of the site are paved. The proposed site is located approximately 700 feet southeast of MarBorg Industries Construction and Demolition Materials Recovery and Transfer Facility. Similar to the proposed project, the AD Facility, composting area, and associated water tanks would be located at the Tajiguas Landfill. At this alternative location, the MRF would consist of a 107,000 sf building (net) that would include:

1. Truck scale for weighing incoming MSW and CSSR
2. Tipping floor/waste delivery areas (40,000 sf) to receive an estimated maximum delivery volume of 220,000 tons/year; and 40,000 tons/year of CSSR;
3. MRF waste processing (30,000 sf) and bale storage (10,000 sf);

4. Load-out waste transfer area (23,000 sf) where the non-salvageable residue would be to be transferred to the Tajiguas Landfill for disposal. Transfer trucks would sit at grade and loaded over the top by loaders with extended forks. Two 18-wheel transfer trucks would be able to be loaded simultaneously.
5. Loading dock with dock-high capacity for three container trailers and/or enclosed trucks to receive baled recyclable materials for transport to markets.
6. Office/administration/employee/control room (two stories, 2,000 sf each for total of 4,000 sf);
7. Visitor/education (1,000 sf included as part of the second floor of the office/administration building) and;
8. Parking for 47 employee/visitor vehicles and 7 bicycles. The average building height would be approximately 38 feet with a maximum building height of 40 feet.

Grading quantities are based on the Conceptual Grading Plan by Penfield and Smith. Construction of the site will require import of approximately 13,950 C.Y. of soil or other structural fill to bring the building to the subgrades shown. There is no known contamination of the site.

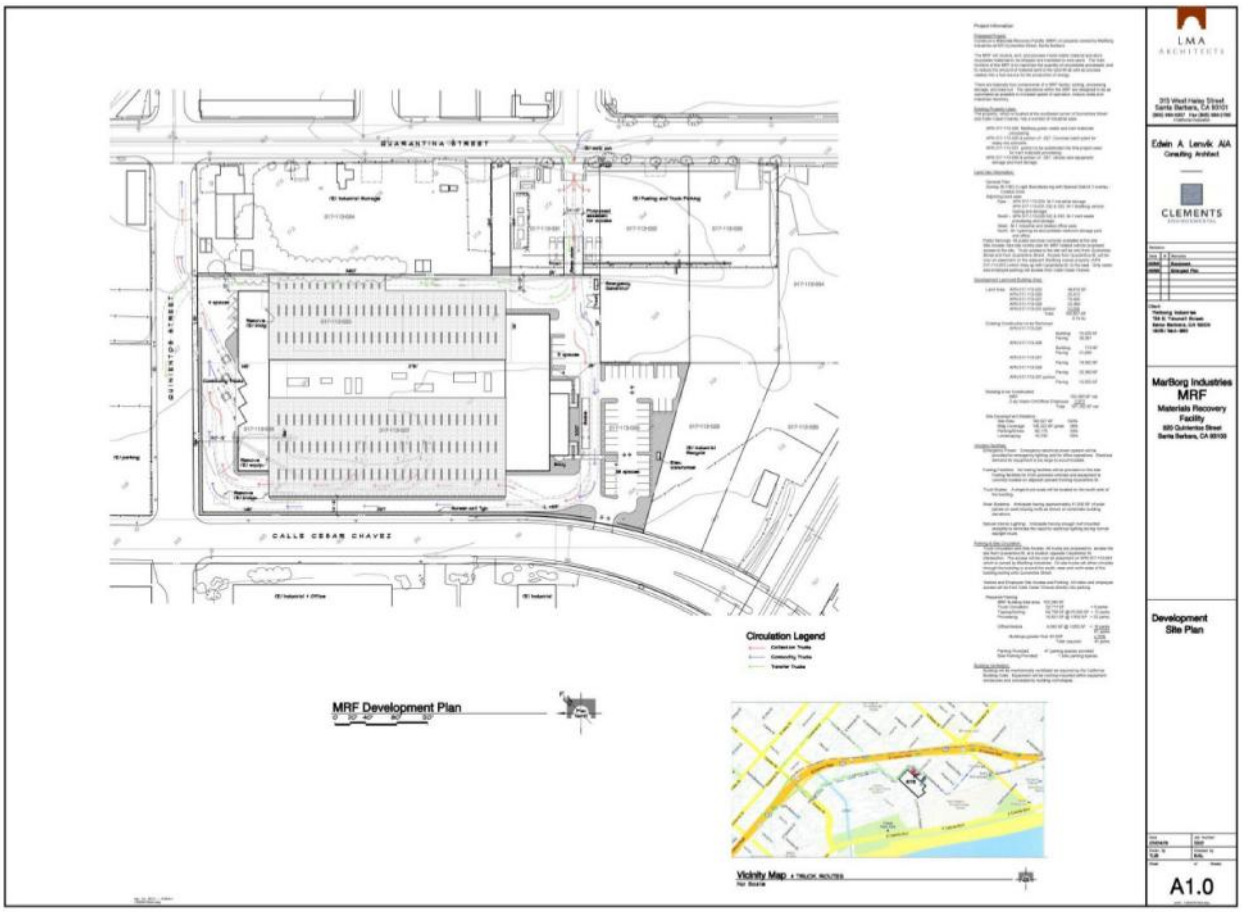


Figure 12: Alternative B – MarBorg Industries Site Plan

14.2.2 Local Geology

Locally, the MarBorg alternative MRF site is located within Alluvium Deposits (Qa). Dibblee, 1986 mapped the property as underlain by Surficial Sediments (Qa) described as "Alluvium: unconsolidated floodplain deposits of silt, sand and gravel." A subsurface investigation was not performed at the site, nor was one requested or proposed. However, a Preliminary Foundation Investigation (Pacific Materials Laboratory, August 13, 2002) at the MarBorg C & D Recycling Facility located at 119 N. Quarantina

Street (located 600 feet northwest of the proposed site, north of U.S. Highway 101) encountered units of tan to black sandy CLAY to brown to gray-brown clayey SAND interpreted to be Alluvium Deposits (Qa). Groundwater was encountered in borings for the above referenced investigation at a depth of 12 to 13 feet below ground surface. Soil types, soil strengths, and groundwater depths are anticipated to be similar to subsurface material at the proposed site.

14.2.3 Landslides

Dibblee, 1986 did not map landslides at the property. Due to a relatively flat topography at the proposed alternative site, the potential for landslides is low. **Potential landslide impacts at the MarBorg alternative site were identified as less than significant, Class III.**

14.2.4 Severe Erosion

Because the site is nearly level and there would be no grading on slopes, therefore **the severe erosion potential impact at the MarBorg alternative MRF site was identified as less than significant, Class III. However, impacts of grading at the Tajiguas landfill for the ADF are still proposed, therefore the Class II, severe erosion potential remains the same for the ADF.**

14.2.5 Regional Faulting and Seismicity

Similar to the surrounding areas, the MarBorg alternative site may be affected by moderate to major earthquakes centered on one of the known large, active faults listed in Table 12 below. Moment magnitudes are expressed, although any significant event on these faults could result in moderate to severe ground shaking at the subject site. The potential for ground failure of any portion of the Site during ground shaking is considered low.

Table 12: Active Faults Near the MarBorg Alternative

Closest Active Faults to Site	Approximate Distance (miles)	Moment Magnitude (Mw)
Santa Ynez Fault	14.5	7.1
Los Alamos Fault	36.0	6.8
San Andreas Fault	38.0	8.5

The closest known Holocene age fault is the Santa Ynez Fault located approximately 14.5 miles northwest of the Site (Jennings, 2010), however the San Andreas Fault is the most likely active fault to produce ground shaking at the Site. Figure 13 depicts significant historical earthquakes in the region.

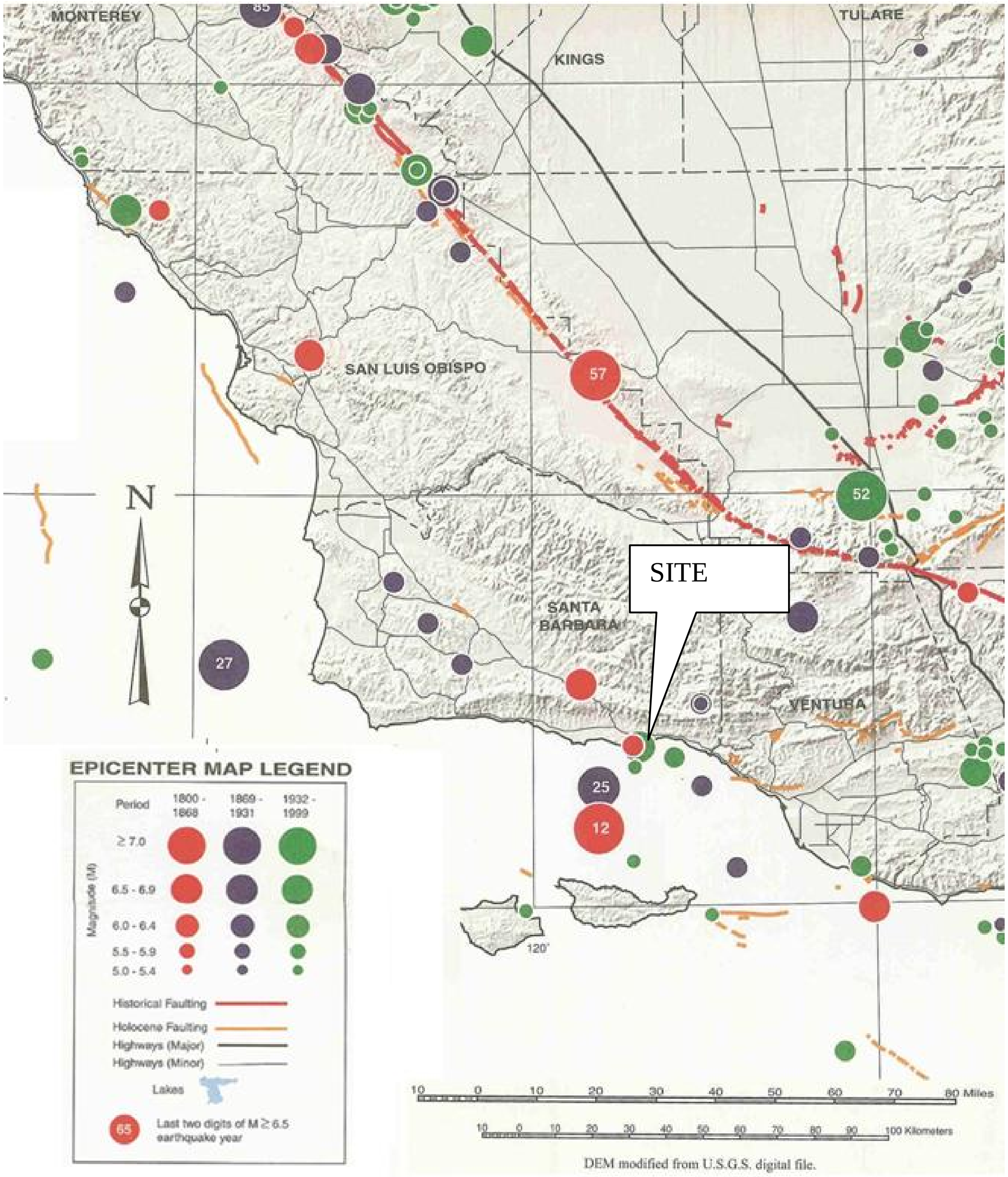


Figure 13: Historical Seismicity Map for MarBorg Alternative (Toppozada et al., 2000)

The Mesa Fault is mapped as inferred approximately 3,200 feet southwest of the MarBorg alternative site (Dibblee, 1986 and Minor et al., 2009). According to the City of Santa Barbara Safety and Public Services Element (City of Santa Barbara, 1979), the Mesa Fault is considered potentially active.

The Lagoon Fault is mapped approximately 0.75 mile northeast of the MarBorg alternative site (Minor et al., 2009). According to the City of Santa Barbara Safety and Public Services Element (City of Santa Barbara, 1979), the Lagoon Fault is considered potentially active since it displaces late Pleistocene fanglomerate.

The Alquist-Priolo Earthquake Fault Zoning Act of 1972 requires that the California State Geologist establish Earthquake Fault Zones around the surface traces of active faults and to issue appropriate maps. The MarBorg alternative site is not located within an Earthquake Fault Zone. **Potential impacts at the MarBorg alternative site due to faulting were identified as less than significant, Class III**.

14.2.6 Tsunami/Seiches

Tsunamis and seiches are two types of water waves that are generated by earthquake events. Tsunamis are broad-wavelength ocean waves and seiches are standing waves within confined bodies of water, typically reservoirs. Cal EMA, 2009 maps the MarBorg alternative site as within a tsunami inundation area. Due to the alternative site being located approximately 0.25 miles from the Pacific Ocean and an elevation of 11 feet, the tsunami potential to affect the site is high. **Potential impacts at the MarBorg alternative site due to tsunamis was identified as significant environment impacts that can be feasibly mitigated, Class II**.

ALT B GEO-1 – The project civil engineer should provide and incorporate recommendations for flooding associated with tsunami if this alternative is considered.

Flooding associated with a seismic event (seiches) is considered low due to the absence of a body of water upslope of the property. **Potential impacts at the Site due to seiches were identified as less than significant, Class III**.

14.2.7 Liquefaction

The presence of loose, poorly graded, fine sand material that is saturated by groundwater within an area that is known to be subjected to high intensity earthquakes and long-duration ground motion are the key factors that indicate potentially liquefiable areas and conditions that lead to liquefaction. Based on the investigation performed for the Preliminary Foundation Investigation (Pacific Materials Laboratory, August 13, 2002) at the MarBorg C & D Recycling Facility, located 600 feet northwest of the site in similar subsurface units, layers of liquefiable zones were identified. Groundwater was observed at 12 to 13 feet below ground surface. **Potential impacts at the MarBorg alternative site due to liquefaction is considered as significant environment impacts that can be feasibly mitigated, Class II**.

ALT B GEO-2 - It is recommended that a Soils Engineering Report be performed at the site to verify subsurface materials, perform a site specific liquefaction analysis and provide foundation recommendations to mitigate liquefaction impacts if this alternative is to be considered. It is anticipated that caissons, helical piers, or h-pile type foundation systems would be utilized for the alternative MRF structure.

14.2.8 Seismically Induced and/or Differential Settlement

Seismically induced settlement occurs in loose to medium dense unconsolidated soil above groundwater. These soils compress (settle) when subject to seismic shaking. The settlement can be exacerbated by increased loading, such as from the construction of buildings. Based on the presence of soft, loose alluvial deposits, there is a high potential for seismically induced settlement at the Site. **Potential impact at the MarBorg alternative site due to seismically induced settlement is considered as significant environment impacts that can be feasibly mitigated, Class II**.

ALT B GEO-3 - It is recommended that a Soils Engineering Report be performed at the site to verify subsurface materials and provide foundation recommendations to mitigate seismically induced settlement impacts if this alternative is to be considered. It is anticipated that caissons, helical piers, or h-pile type foundation systems would be utilized for the alternative MRF structure.

It is anticipated that the proposed MRF will be located entirely within Alluvial Deposits (Qa). Because of the uniformity of the underlying material, **potential impact at the MarBorg alternative site due to differential settlement is considered as less than significant, Class III**.

14.3 Alterative C – Urban Area MRF Alternative 2 (South Coast Recycling and Transfer Station[SCRTS])

14.3.1 Project Description

This Alternative would involve construction and operation of the MRF component of the Tajiguas Resource Recovery Project at the existing County-owned and operated SCRTS site located at 4430 Calle Real in Santa Barbara, California. Under this Alternative the MRF would be integrated with the existing solid waste operations at the SCRTS. Similar to the proposed project, the AD Facility, composting area, and associated water tanks would be located at the Tajiguas Landfill, with disposal of residual waste also at the Tajiguas Landfill.

The solid waste operations area is located on 8.3 acres in the central portion of a larger 143.48 acre publicly owned parcel (APN 059-140-023) containing other public and non-profit uses (e.g., County Road Yard, a Corporation Yard which serves General Services and Flood Control, Growing Solutions Restoration Education Institute, a non-profit native plant nursery, and Hearts Therapeutic Equestrian Center, an non-profit therapeutic riding program).

All existing facilities, excluding the Maintenance Shop, would be demolished in preparation for construction of the proposed MRF and associated facilities. Demolition would include removal of existing asphalt and concrete paving and parking lots, masonry walls, buildings, office trailers and associated materials and solid waste. Approximately 13,200 cy of cut and 7,500 cy of fill (with approximately 5,700 cy of net soil export), would be required over an approximate 6.2 acre area to produce level pads for the MRF building, parking lots and other facilities.

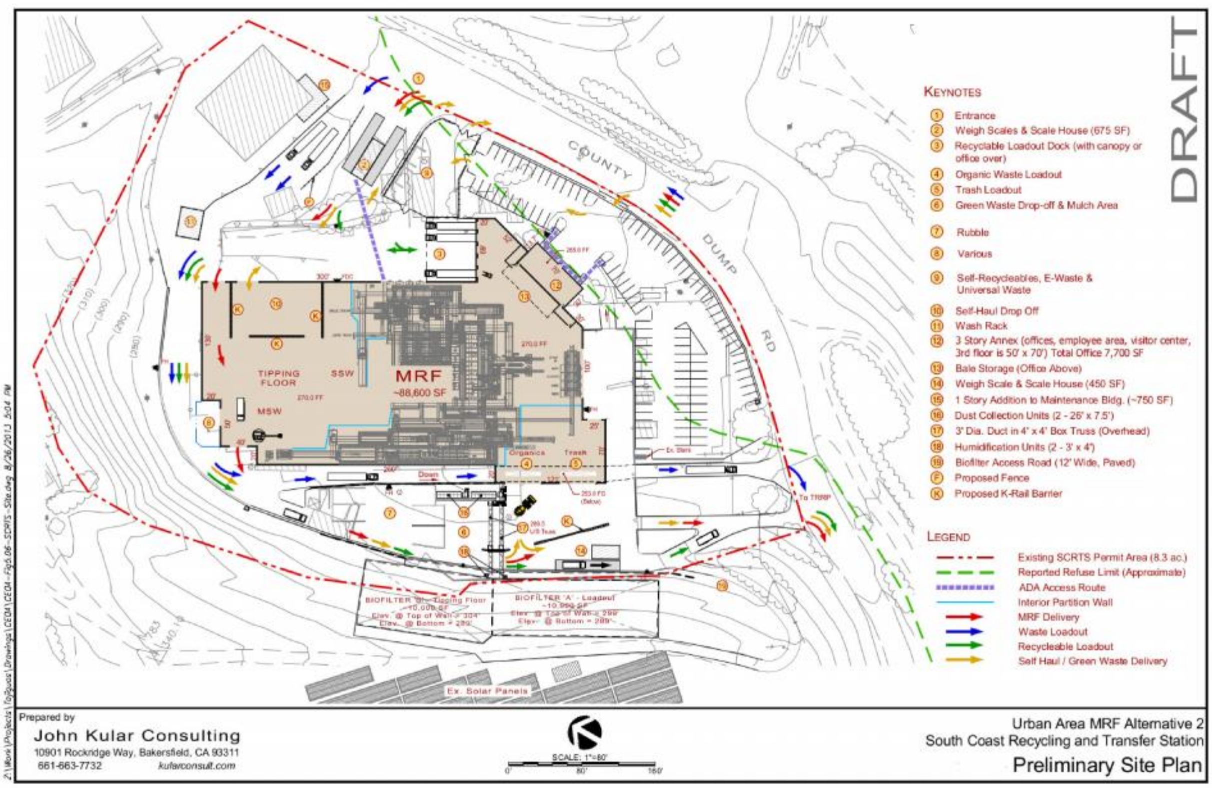


Figure 14: Alternative C – South Coast Recycling and Transfer Station Site Plan

14.3.2 Local Geology

Locally, the site is located within Older Alluvium Deposits (Qog) and Santa Barbara Formation (Qsb). Dibblee, 1986 mapped the property as underlain by Pleistocene age (1.8 mybp to 10,000 ybp) Older Dissected Surficial Sediments (Qog) described as “Former alluvial deposits of silt, sand and gravel, in places weakly consolidated” and Pliocene age (5.3 to 1.8 mybp) Santa Barbara Formation (Qsb) described as “Massive to bedded, poorly consolidated, tan to yellow fossiliferous sand and silt.” Minor et al., 2009 maps and describes the Santa Barbara Formation as “Chiefly marine pale-gray, -buff, and –tan, friable bioturbated and massive sandstone; includes subordinate interbeds and intervals of shale, siltstone, and silty to clayey sandstone.” A subsurface investigation was not performed at the site, nor was one requested or proposed. A Final Negative Declaration (Fugro West, Inc., 1995) also identified “Quaternary-age fanglomerate and Santa Barbara Formation deposits.” at the site.

MSW is located along the eastern boundary of the proposed MRF site as depicted on Figure 14. The MSW is associated with the closed Foothill Landfill that operated between 1940s and 1967. The MRF building is proposed outside of the historic refuse footprint.

14.3.3 Landslides

Dibblee, 1986 and Minor et al., 2009 did not map landslides at the property. The County of Santa Barbara Seismic Safety Element maps the site within a low to moderate potential for landslides and moderate potential for soil creep. The SCRTS alternative site is proposed to be located at the existing transfer station location which is relatively flat with steep cut slopes (0.5H:1V) forming the northern and western pad boundaries. It is our understanding that the proposed MRF building would be located within the existing pad and the slope would not be modified. A previous slope stability analysis performed on the cut slopes (Fugro-McClelland (West), Inc., 1993) were observed to be stable under soil moisture conditions that existed at the time of the investigation. If modification of these slopes is proposed (with the exception of the biofilters), it is recommended that a slope stability analysis be performed to verify stability. The biofilter pads are proposed to be located within the slope surrounded by 10 to 15 foot high retaining wall therefore there is a low potential for slope stability impacts. **Potential landslide impacts at the SCRTS alternative site were identified as less than significant, Class III**.

14.3.4 Severe Erosion

The potential for severe erosion is low considered that grading is proposed to extend within the existing slope (with the exception of the biofilter). As previously stated, the biofilter pads are proposed to be surrounded by retaining walls, therefore **the severe erosion potential impact at the SCRTS alternative site were identified as less than significant, Class III.**

14.3.5 Regional Faulting and Seismicity

Similar to the surrounding areas, the SCRTS alternative site may be affected by moderate to major earthquakes centered on one of the known large, active faults listed in Table 13 below. Moment magnitudes are expressed, although any significant event on these faults could result in moderate to severe ground shaking at the subject site. The potential for ground failure of any portion of the Site during ground shaking is considered low.

Table 13: Active Faults Near the SCRTS Alternative

Closest Active Faults to Site	Approximate Distance (miles)	Moment Magnitude (Mw)
Santa Ynez Fault	9.0	7.1
Los Alamos Fault	31.0	6.8
San Andreas Fault	40.0	8.5

The closest known Holocene age fault is the Santa Ynez Fault located approximately 9.0 miles northwest of the Site (Jennings, 2010), however the San Andreas Fault is the most likely active fault to produce ground shaking at the Site. Figure 15 depicts significant historical earthquakes in the region.

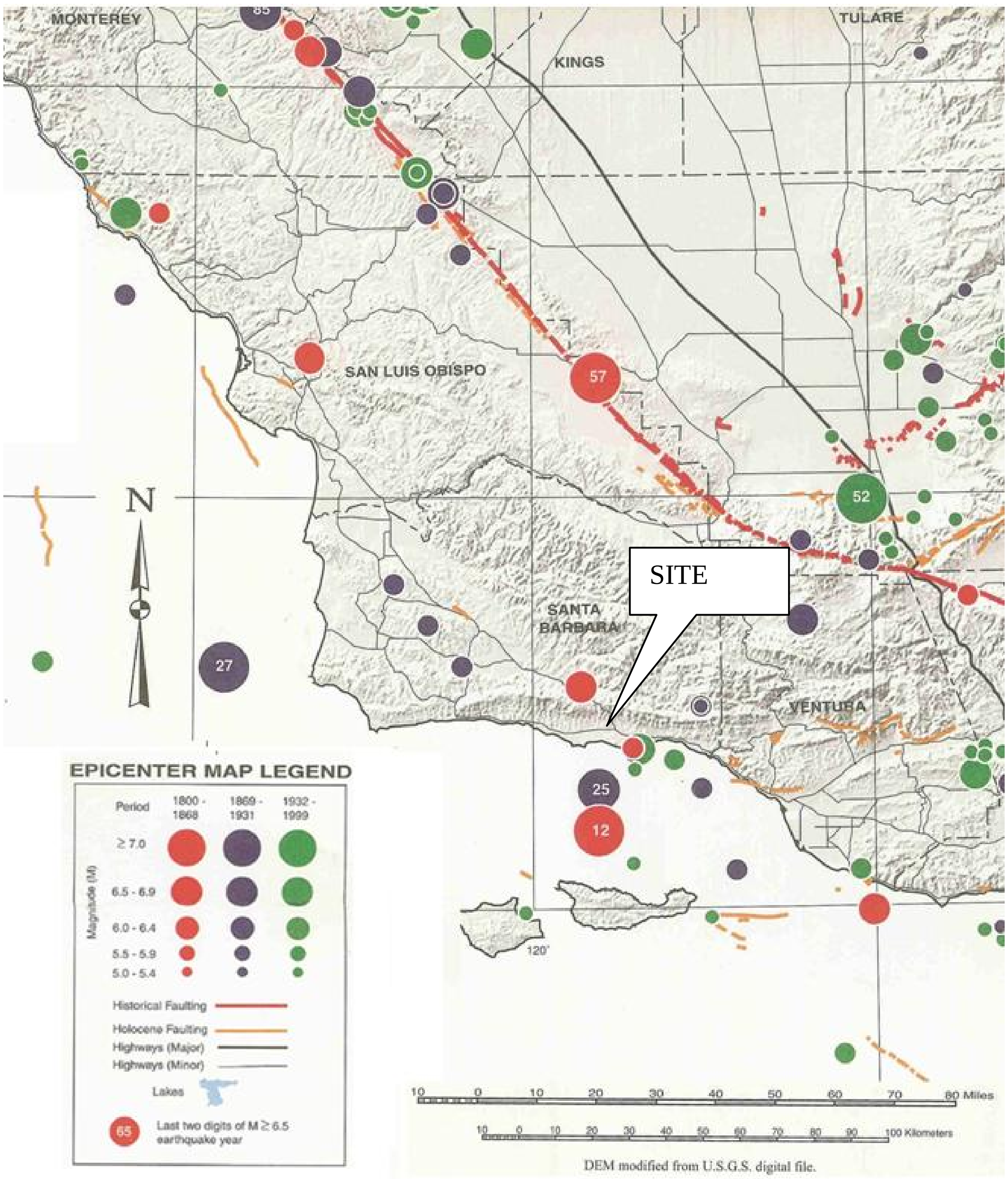


Figure 15: Historical Seismicity Map for the SCRTS Alternative (Toppozada et al., 2000)

The San Jose Fault is mapped approximately 1,000 feet north of the SCRTS alternative site (Dibblee, 1987). According to the County of Santa Barbara Seismic Safety and Safety Element (County of Santa Barbara, 2010), the San Jose Fault is considered potentially active with an estimated maximum credible earthquake magnitude of 5.8. Minor et al., 2009 also maps the Foothill Road Fault in the immediate vicinity of where the San Jose Fault is mapped by Dibblee north of the site.

The More Ranch Fault is mapped approximately 0.75 mile southeast of the SCRTS alternative site (Dibblee, 1987 and Minor et al., 2009). According to the County of Santa Barbara Seismic Safety and Safety Element (County of Santa Barbara, 2010), the More Ranch Fault is considered active as it shows displacement of recent alluvium.

The Final Negative Declaration for the Santa Barbara County Transfer Station (County of Santa Barbara, 1995) also discussed the Modoc fault inferred south of the site based on water level data (Upson, 1951). The location of this fault is unknown, and is not depicted on geologic maps. No evidence of faulting is observed within the existing cut slopes and therefore infers the fault to be south of the site.

The Alquist-Priolo Earthquake Fault Zoning Act of 1972 requires that the California State Geologist establish Earthquake Fault Zones around the surface traces of active faults and to issue appropriate maps. The SCRTS alternative site is not located within an Earthquake Fault Zone. **Potential impacts at the SCRTS alternative site due to faulting were identified as less than significant, Class III**.

14.3.6 Tsunami/Seiches

Tsunamis and seiches are two types of water waves that are generated by earthquake events. Tsunamis are broad-wavelength ocean waves and seiches are standing waves within confined bodies of water, typically reservoirs. Due to the alternative site being located over 2 miles from the Pacific Ocean and have an elevation of 270 feet, the tsunami potential to affect the site is low. Flooding associated with a seismic event (seiches) is considered low due to the absence of a body of water upslope of the property. **Potential impacts at the SCRTS alternative site due to tsunamis or seiches were identified as less than significant, Class III**.

14.3.7 Liquefaction

The presence of near surface Santa Barbara Formation observed within the cut slopes the potential for liquefaction is low. **Potential impacts at the SCRTS alternative site due to liquefaction were identified as less than significant, Class III**.

14.3.8 Seismically Induced and/or Differential Settlement

Seismically induced settlement occurs in loose to medium dense unconsolidated soil above groundwater. These soils compress (settle) when subject to seismic shaking. The settlement can be exacerbated by increased loading, such as from the construction of buildings. Based on the presence of near surface Santa Barbara Formation, there is a low potential for seismically induced settlement at the Site. **Potential impacts at the SCRTS alternative site were identified as less than significant, Class III**.

It is anticipated that the proposed MRF will be located entirely within Santa Barbara Formation and will utilize a conventional foundation system. However the parking lot east of the site will extend over the historical waste footprint and experience settlement. **Potential impacts at the SCRTS alternative site due to differential settlement is considered as significant environment impacts that can be feasibly mitigated, Class II**.

ALT C GEO-1 - It is recommended that a Soils Engineering Report be performed at the site to verify subsurface materials and provide foundation and pavement section recommendations (i.e. geogrid) to mitigate settlement impacts if this alternative is to be considered.

14.4 Alternative D – Off-site Aerobic Composting (Engel and Gray Composting Facility)

14.4.1 Project Description

The Aerobic Composting Alternative would involve processing organic waste recovered in the MRF using open air aerobic composting methods at Engel and Gray's existing composting facility in the City of Santa Maria, instead of enclosed dry fermentation anaerobic digestion at the Tajiguas Landfill. Similar to the proposed project, the MRF and ADF would be located at the Tajiguas Landfill, with disposal of residual waste also at the Tajiguas Landfill. The Engel & Gray facility is comprised of two parcels (APNs 113-120-17, -21) on a 40.15 acre portion of the 161-acre City of Santa Maria Wastewater Treatment Plant (WWTP) facility. The site is located approximately 0.3 miles south of the State Route 166/Ray Road intersection, and about 2.5 miles west of residential areas located at Black Road. The composting facility is situated adjacent to, and immediately west of the developed portion of the WWTP site.

14.4.2 Local Geology

Locally, the alternative compositing site is located within Alluvium Deposits (Qa) described as "valley and floodplain alluvium (Dibblee, 1989)." The City of Santa Maria Safety Element identifies the Santa Maria Valley as alluviul deposits described as "Near-surface deposits consist almost entirely of unconsolidated alluvial gravel, sand, silt, and clay of Holocene age." A subsurface investigation was not performed at the site, nor was one requested or proposed.

14.4.3 Landslides

Dibblee, 1989 did not map landslides at the property. The County of Santa Barbara Seismic Safety Element maps the site within a low potential for landslides. The Off-site Aerobic Composting alternative site is proposed to be located at the existing Engel and Gray composting facility location which is relatively flat. **Potential landslide impacts associated with the Aerobic Composting Alternative was identified as less than significant, Class III**.

14.4.4 Severe Erosion

Because the site is nearly level and there would be no grading on slopes, therefore **the severe erosion potential impact associated with the Aerobic Composting Alternative was identified as less than significant, Class III.**

14.4.5 Regional Faulting and Seismicity

Similar to the surrounding areas, the Aerobic Composting Alternative site may be affected by moderate to major earthquakes centered on one of the known large, active faults listed in Table 14 below. Moment magnitudes are expressed, although any significant event on these faults could result in moderate to severe ground shaking at the subject site. The potential for ground failure of any portion of the Site during ground shaking is considered low.

Table 14: Active Faults Near the Engel and Gray Alternative

Closest Active Faults to Site	Approximate Distance (miles)	Moment Magnitude (Mw)
Hosgri Fault	20.5	7.5
Los Alamos Fault	21.5	6.8
San Andreas Fault	44.5	8.5

The closest known Holocene age fault is the Hosgri Fault located approximately 20.5 miles northwest of the Site (Jennings, 2010), however the San Andreas Fault is the most likely active fault to produce ground shaking at the Site. Figure 16 depicts significant historical earthquakes in the region.

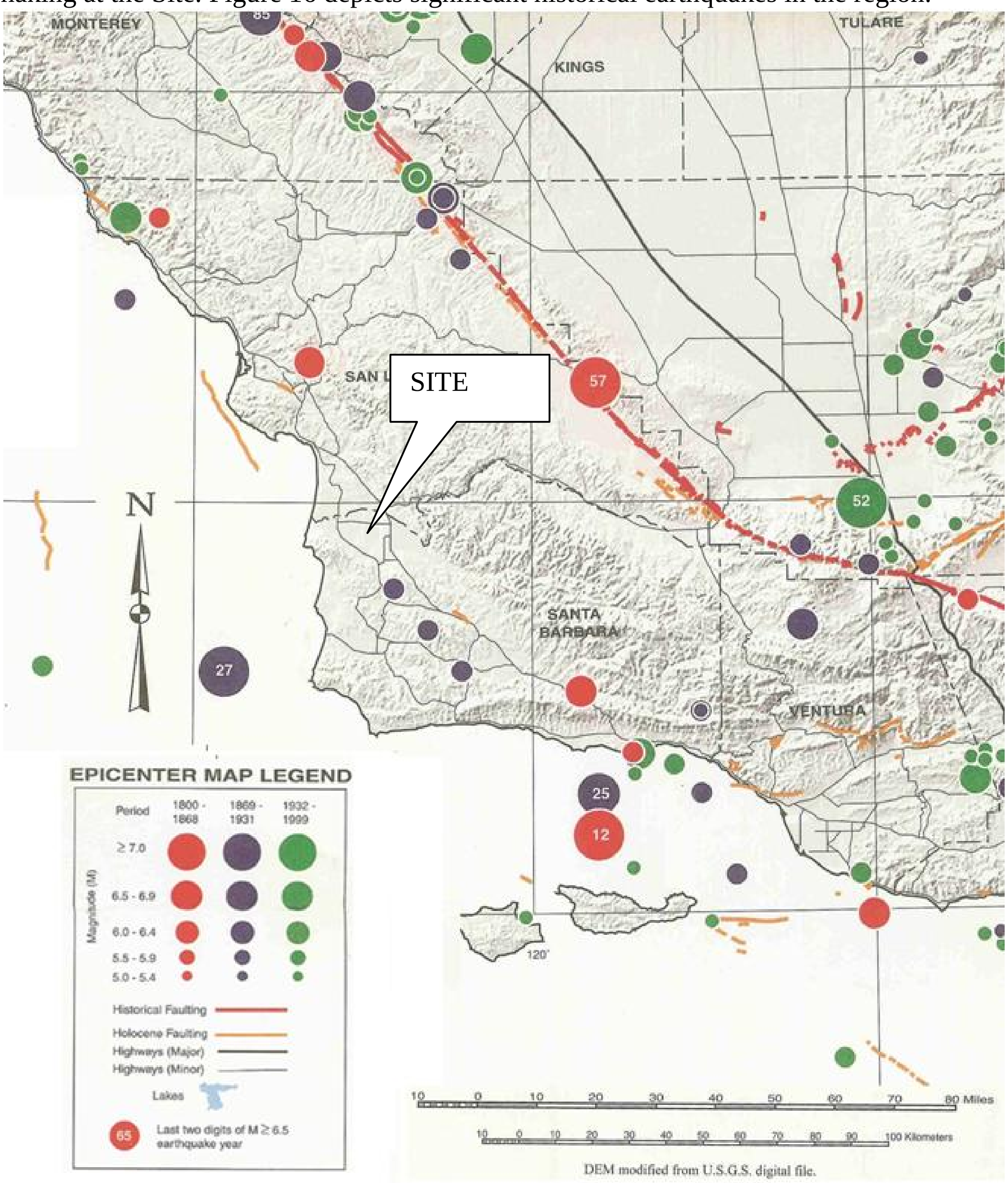


Figure 16: Historical Seismicity Map for the Engel and Gray Alternative (Toppozada et al., 2000)

The Alquist-Priolo Earthquake Fault Zoning Act of 1972 requires that the California State Geologist establish Earthquake Fault Zones around the surface traces of active faults and to issue appropriate maps. The Engel and Gray Composting alternative site is not located within an Earthquake Fault Zone. **Potential impacts at the Aerobic Composting Alternative site due to faulting were identified as less than significant, Class III**.

14.4.6 Tsunami/Seiches

Tsunamis and seiches are two types of water waves that are generated by earthquake events. Tsunamis are broad-wavelength ocean waves and seiches are standing waves within confined bodies of water, typically reservoirs. Due to the alternative site being located over 8 miles from the Pacific Ocean, the tsunami potential to affect the site is low. Flooding associated with a seismic event (seiches) is considered low due to the absence of a body of water upslope of the property. **Potential impacts at the Aerobic Composting Alternative site due to tsunamis or seiches were identified as less than significant, Class III**.

14.4.7 Liquefaction

The presence of loose, poorly graded, fine sand material that is saturated by groundwater within an area that is known to be subjected to high intensity earthquakes and long-duration ground motion are the key factors that indicate potentially liquefiable areas and conditions that lead to liquefaction. Due to the presence of alluvial deposits, there is a high potential for liquefaction, however no structures are proposed at the site as part of this alternative, therefore **potential impacts at the Aerobic Composting Alternative site due to liquefaction is considered as less than significant, Class III**.

14.4.8 Seismically Induced Settlement

Seismically induced settlement occurs in loose to medium dense unconsolidated soil above groundwater. These soils compress (settle) when subject to seismic shaking. The settlement can be exacerbated by increased loading, such as from the construction of buildings. Based on the presence of soft, loose alluvial deposits, there is a high potential for seismically induced settlement at the Site, however no structures are proposed at the site as part of this alternative, therefore **potential impacts at the Aerobic Composting Alternative site due to seismically induced settlement is considered as less than significant, Class III**.

14.5 Alternative E – Tajiguas Landfill Expansion

14.5.1 Project Description

This Alternative would involve expansion of the Tajiguas Landfill to extend its life by at least 10 years (similar to the proposed project) from the currently projected closure in approximately 2026 to approximately 2036. Under the Expansion Alternative, the permitted maximum daily tonnage for the Tajiguas Landfill would remain at its current level of 1,500 tons/day. The existing landfill would be expanded both vertically and horizontally, to provide an additional 3.7 million cubic yards of airspace or 6.9 million tons of waste disposal capacity. The 3.7 million cubic yards of additional capacity would be provided by expanding the Landfill footprint in the back canyon area of the Landfill property in the area of the Landfill reconfiguration project that was approved in 2009.

The overall capacity increase would be achieved by lining and placing additional waste against the existing landfill cut slope and by additional excavations in the back canyon area increasing the waste fill elevations in the back canyon by approximately 60 feet. Approximately 300,000 cubic yards of excavation would be required to create the additional capacity and to facilitate the installation of the composite liner. The fill slopes would be constructed with 15-foot wide benches every 40 vertical feet to create overall fill slopes of 2.4:1. The expansion would be developed in phases.

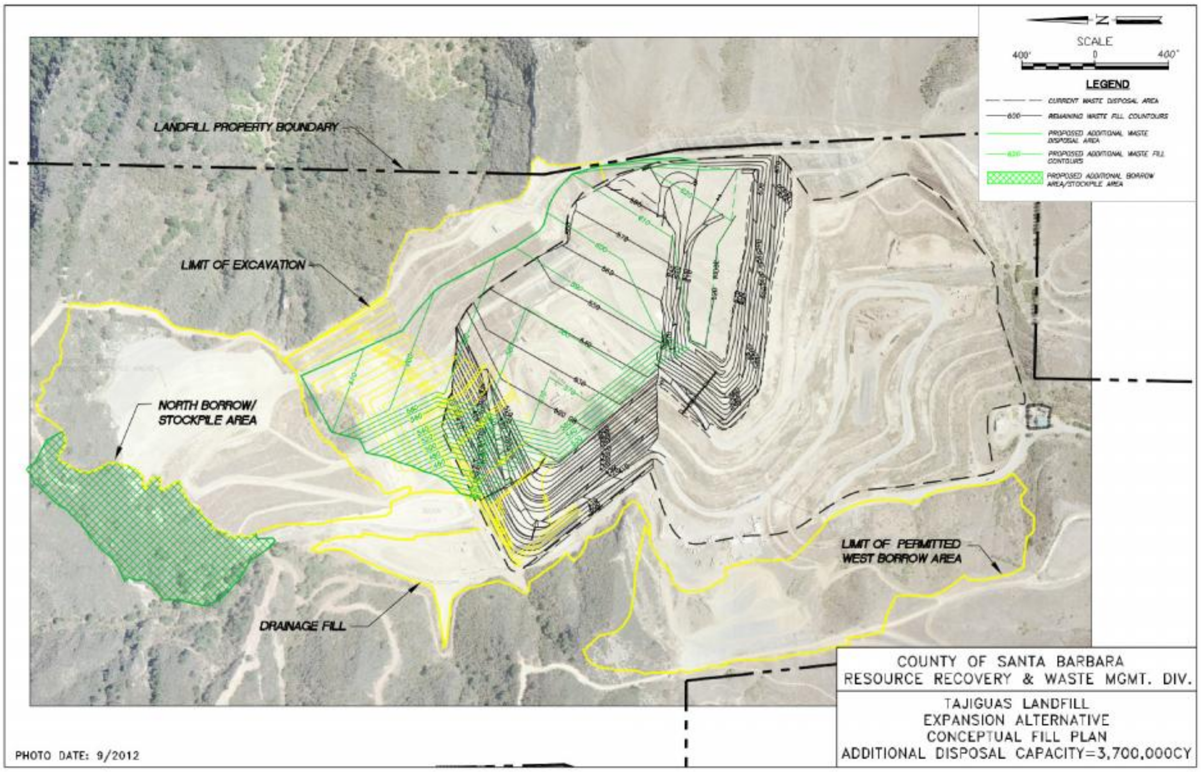


Figure 17: Alternative E – Tajiguas Landfill Expansion Site Plan

14.5.2 <u>Local Geology</u>

As stated in the previous Final Environmental Impact Report for Tajiguas Landfill Expansion Project (01-EIR-05), "units exposed within 1 mile of the landfill include the Cozy Dell Shale, Sacate Formation, Gaviota Formation, Alegria Formation, Sespe Formation, Vaqueros Formation, Rincon Shale and Monterey Shale." Locally, bedrock underlying this alternative site is the Rincon Shale (Tr), Vaqueros Sandstone (Tvq), and Sespe Formation (Tsp). Dibblee, 1988 mapped the property as underlain by early Miocene age (23.8-16.4 million years before present {mybp}) Rincon Shale (Tr) and Vaqueros Sandstone (Tvq), and Oligocene age (33.7-23.8 mybp) Sespe Formation (Tsp) and Alegria Formation (Ta).

14.5.3 <u>Landslides</u>

Dibblee, 1988 did not map landslides at the property. The County of Santa Barbara Seismic Safety Element maps the site within a low potential for landslides. Landslides were not observed in the previous EIR with the exception of the two surficial landslides along the west borrow slope. The modification of the west borrow slope would remain in the current configuration and therefore have a low potential to affect the project. Existing fill and MSW slopes at the Tajiguas landfill have been constructed at a slope gradient of 2.5:1 (horizontal:vertical) with benches or 3:1 with no benches. It is assumed that slopes constructed for this alternative will be in the slope configurations determined to be stable. If slopes are proposed steeper, a slope stability analysis should be performed. **Potential landslide/slope stability impacts for the Tajiguas Landfill expansion alternative site were identified as less than significant, Class III**.

14.5.4 Severe Erosion

The potential for severe erosion is low considered provided that vegetation and erosion control measures are implemented immediately after the completion of proposed additional grading and stockpile. **The severe erosion potential impact for the Tajiguas Landfill expansion alternative site was identified as significant environment impacts that can be feasibly mitigated, Class II..**

14.5.5 Regional Faulting and Seismicity

Similar to the proposed Tajiguas Resource Recovery Project, the Tajiguas Landfill Expansion alternative site may be affected by moderate to major earthquakes centered on one of the known large, active faults listed in Table 15 below. Moment magnitudes are expressed, although any significant event on these faults could result in moderate to severe ground shaking at the subject site. The potential for ground failure of any portion of the Site during ground shaking is considered low.

Table 15: Active Faults Near the Tajiguas Landfill

Closest Active Faults to Site	Approximate Distance (miles)	Moment Magnitude (Mw)
Santa Ynez Fault	15.5	7.1
Los Alamos Fault	16.0	6.8
San Andreas Fault	52.0	8.5

The closest known Holocene age fault is the Santa Ynez Fault located approximately 15.5 miles northwest of the Site (Jennings, 2010), however the San Andreas Fault is the most likely active fault to produce ground shaking at the Site. Figure 18 depicts significant historical earthquakes in the region.

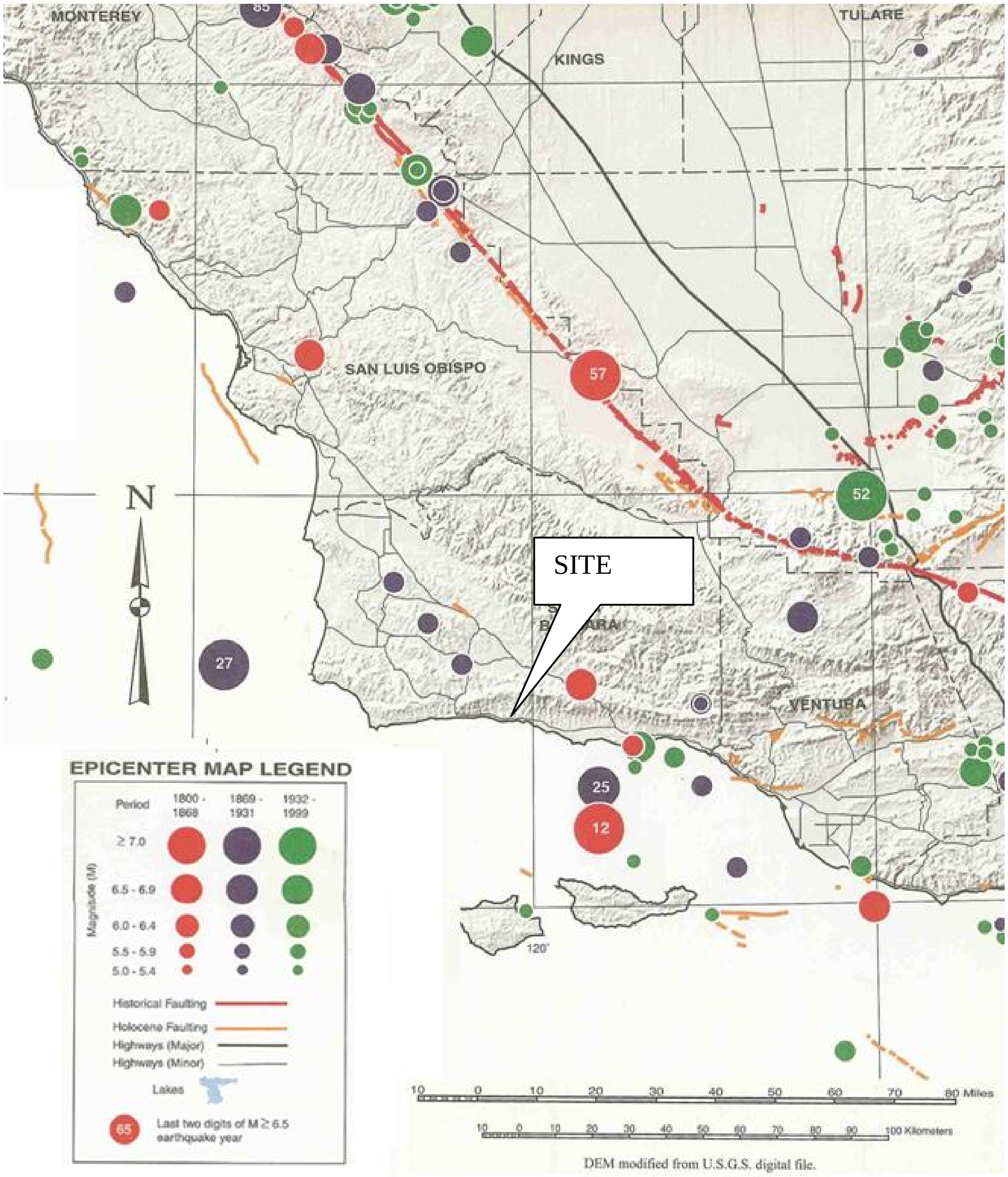


Figure 18: Historical Seismicity Map for the Tajiguas Landfill (Toppozada et al., 2000)

The Alquist-Priolo Earthquake Fault Zoning Act of 1972 requires that the California State Geologist establish Earthquake Fault Zones around the surface traces of active faults and to issue appropriate maps. The Tajiguas Landfill expansion alternative site is not located within an Earthquake Fault Zone. **Potential impacts for the Tajiguas Landfill Expansion alternative site due to seismicity and faulting were identified as less than significant, Class III**.

14.5.6 <u>Tsunami/Seiches</u>

Tsunamis and seiches are two types of water waves that are generated by earthquake events. Tsunamis are broad-wavelength ocean waves and seiches are standing waves within confined bodies of water, typically reservoirs. Due to the alternative site being at an elevation of approximately 390 feet, the potential for a tsunami to affect the site is low. Flooding associated with a seismic event (seiches) is considered low due to the absence of a body of water upslope of the property. **Potential impacts for the Tajiguas Landfill**

Expansion alternative site due to tsunamis or seiches were identified as less than significant, Class III.

14.5.7 Liquefaction

Based on the consistency and relative density of the in-situ soils (clay/rock) and the depth to groundwater the potential for seismic liquefaction of soils at the Site is very low. **Potential liquefaction impacts for the Tajiguas Landfill Expansion alternative Site were identified as less than significant, Class III**.

14.5.8 Seismically Induced Settlement

Seismically induced settlement occurs in loose to medium dense unconsolidated soil above groundwater. These soils compress (settle) when subject to seismic shaking. Based on the presence of MSW, there is a high potential for seismically induced settlement at the Site, however no structures are proposed therefore the impact is less than significant. **Potential impacts for the Tajiguas Landfill Expansion alternative site due to seismically induced settlement were identified as less than significant, Class III**.

14.6 Alternative F – Waste Export To the Simi Valley Landfill and Recycling Center

This Alternative would involve transportation of all MSW generated in the Tajiguas Landfill wasteshed (up to 270,000 tons/year of MSW, maximum of 1,500 tons/day as currently permitted) to the Simi Valley Landfill and Recycling Center (SVLRC), when the Tajiguas Landfill reaches its permitted capacity (approximately 2026). The SVLRC is located at 2801 Madera Road, Simi Valley, California approximately 65 miles from the City of Santa Barbara. The entrance road is located approximately 0.5 miles west of the U.S. 101/Madera Road interchange.

The basis of this Alternative is to provide 10 additional years of MSW disposal capacity, when the Landfill reaches its permitted capacity in 2026. This is equivalent to the 10 year increase in Landfill life provided by the proposed project through reductions in disposal rates associated with increased recycling.

A Final EIR for Expansion of the SVLRC was completed in December 2010, and Major Modification No. 8 to CUP-3142 was approved by Ventura County.

14.6.1 Geologic Setting

The December 2010 Final EIR describes the geology at the SVLRC as “The proposed project site is underlain primarily by the Sespe Formation bedrock. Superimposed on this bedrock are various surficial units, including the upper Pliocene to lower Pleistocene Saugus Formation, which caps subdued ridge tops in the southwest portion of the property; older alluvium, representing erosional remnants of the upper Pleistocene to Holocene alluvial deposits; recent alluvium, occupying the axes of active watercourses; various mass wasting deposits, including translational and rotational landslide masses; and surficial soil and colluvial deposits.”

The Final EIR describes faulting at the SVLRC as:

> *Numerous faults have been identified in the Simi Valley region, including two potentially active faults that traverse the landfill property; however, no active faults are known within the site (Figures 3.7-1 and 3.7-2) (Dibblee and Ehrenspeck 1992; William Lettis & Associates 2004a, 2004b).A potentially active fault shows evidence of movement within the last 1 million years, but not within the last 11,000 years.*
>
> *As illustrated on Figure 3.7-2, the two potentially active faults that cross the landfill property are the Canada de la Brea and Strathearn faults. These faults are roughly east-west trending reverse faults and are upthrown to the north. A recent investigation of the Canada de la Brea Fault completed for the proposed landfill expansion (William Lettis & Associates 2004a) indicated that this fault is likely too short to generate an independent earthquake of sufficient size to produce fault rupture. However, because this fault is located in the hanging wall of the Holocene active Simi Fault, the Canada de la Brea Fault may experience sympathetic (i.e., triggered) slip during large earthquakes on nearby faults. This sympathetic slip, if it occurs, likely would be minor (i.e., on the order of several centimeters).*

14.6.2 Impacts

Under this alternative, MSW sent to the SVLRC would contribute to the following geologic impacts associated with construction and operation of the SVLRC as identified in the referenced December 2010 Final EIR:

Impact GEO-1: Fault Rupture Hazards. Project exists along pre-existing faults or within a State of California designated Alquist-Priolo Special Fault Study Zone; a County of Ventura designated Fault Hazard area; or a County of Ventura designated Potential Fault Hazard Area. **Less than Significant.**

Impact GEO-2: Ground Shaking Hazards. Ground shaking hazards are ubiquitous throughout Ventura County and, ground failure phenomena aside, are accommodated by the Ventura County Building Code. The effects of ground shaking hazard are required to be considered within the existing framework of grading and building code ordinances which apply to all sites and projects. Special threshold criteria for ground shaking hazard are thus not established. **Less than Significant.**

Impact GEO-3: Liquefaction Hazards. A liquefaction hazard is considered to exist based on project location with respect to mapped liquefaction-susceptible areas on the County General Plan maps, on maps contained in Division of Mines and Geology Open-File Report 76-5LA; and whether the project is located in a shallow bedrock area versus and area underlain by recent or older alluvium. **Less than Significant.**

Impact GEO-4: Subsidence. A subsidence hazard is considered to exist on all new water and oil well projects in Ventura County and for all utility and drainage facility projects in the Oxnard Plain. **Less than Significant.**

Impact GEO-5: Expansive Soils. An expansive soil hazard is considered to exist where soil with an expansion index of greater than 20 are present. **Less than Significant.**

Impact GEO-6: Landslides/Mudslides. Location of the site or project in areas with slopes greater than ten percent. **Less than Significant.**

Impact GEO-7: Petroleum Resources. Land use that is proposed to be located in or immediately adjacent to any known petroleum resource area, or adjacent to a principal access road to an existing petroleum Conditional Use Permit (CUP). **Less than Significant.**

Impact GEO-8: Paleontological Resources. Direct impacts to fossil sites including grading and excavation of fossiliferous rock, which can result in the loss of scientifically important fossil specimens and associated geological data. Indirect impact including increased access opportunities and unauthorized collection of fossil materials. **Significant.**

> **Mitigation GEO-1: Paleontological Mitigation Program.** An updated/expanded Paleontological Mitigation Program shall be submitted by Waste Management, Inc. to the County Planning Division for review and approval.

14.7 Alternative G – Waste Export to the Santa Maria Intergrated Waste Management Facility

This Alternative would involve transportation of all MSW generated by the Tajiguas Landfill wasteshed to the Santa Maria Integrated Waste Management Facility (Santa Maria IWMF), when the Tajiguas Landfill reaches its permitted capacity (projected as 2026). The Santa Maria IWMF will be located on a 1,774 acre site, approximately 7 miles south of the Santa Maria city center (approximately 70 miles from the City of Santa Barbara) and one mile east of U.S. 101.

The basis of this Alternative is to provide 10 additional years of MSW disposal capacity, when the Tajiguas Landfill reaches its permitted capacity in approximately 2026. This is equivalent to the 10 year increase in Landfill life provided by the proposed project through reductions in disposal rates associated with increased recycling.

The City of Santa Maria plans to construct a new Class III municipal solid waste landfill (Santa Maria IWMF) to replace the existing Santa Maria Regional Landfill. A Final EIR was completed in April 2010, and the project was approved by City Council.

14.7.1 Geologic Setting

The December 2010 Final EIR describes the geology at the Santa Maria IWMF as "The site is underlain almost entirely by the Pliocene/Pleistocene-age Paso Robles Formation, though the Pleistocene-age Orcutt Sand crops out in the northeastern portion of the property. In addition, artificial fill/drilling mud, recent alluvium and colluvium, landslide debris, and the Careaga Formation have also been encountered (Dibblee, 1994; Tennyson, 1992)."

The Final EIR identifies the following faults at the Santa Maria IWMF:

> *Casmalia-Orcutt Frontal Fault. The Casmalia-Orcutt Frontal Fault is located 1.8 miles east of the site and trends northeast-southwest. This reverse fault juxtaposes Quaternary age rock of the Orcutt Formation against older rocks of the Tertiary Sisquoc and Careaga formations. Because Quaternary rocks are offset, this fault is classified as potentially active. The fault is approximately 17.5 miles long and has a slip rate of 0.01 inch per year. The fault has a Maximum Credible Earthquake (MCE) Magnitude of 6.5 (USGS, 2002) and an estimated Maximum Probable Earthquake (MPE) Magnitude of 5.5.*

> *San Luis Range Fault. The San Luis Range Fault is a northwest-southeast trending fault located approximately 4.7 miles northeast of the site, lying just to the northeast of Santa Maria. This*

thrust fault has a slip rate of 0.01 inch per year and is approximately 38.5 miles long. The fault has an MCE Magnitude of 7.0 to 7.2 (USGS, 2002) and an estimated MPE Magnitude of 5.3.

Los Alamos-West Baseline Fault. The Los Alamos-West Baseline Fault is located approximately 5 miles south of the site and is classified as active. The thrust fault has a slip rate of 0.03 inch per year. The fault has an MCE Magnitude of 6.7 to 6.9 (USGS, 2002) and an estimated MPE Magnitude of 5.5.

Lion's Head Fault. The Lion's Head Fault is an extension of the Casmalia Fault and lies approximately 6.5 miles south of the site. This reverse fault is approximately 24.5 miles long and is identified as potentially active. The fault has an MCE Magnitude of 6.6 (USGS, 2002) and an estimated MPE Magnitude of 5.5.

14.7.2 Impacts

Under this alternative, MSW sent to the Santa Maria IWMF would contribute to the following geologic impacts associated with construction and operation of the Santa Maria IWMF as identified in the referenced December 2010 Final EIR:

Impact G-1 Due to the presence of active faults in the vicinity of the proposed Santa Maria IWMF, the site and surrounding area is subject to moderate to high levels of ground shaking. Design of the proposed facilities and access roads in accordance with the seismic criteria contained in Title 27 and the latest adopted building codes would reduce impacts to a less than significant level.

Impact G-2 Soils on the Santa Maria IWMF site are characterized by high to very high erosion potential, which may result in soil-related hazards to on-site development. This is a significant but mitigatable impact.

Impact G-3 The proposed IWMF would include excavation (subgrade) slopes, waste fill slopes, and final grade (including cover) slopes that could present landsliding hazards. However, proper design and compliance with applicable landfill slope regulations would reduce impacts to a less than significant level.

15.0 ADDITIONAL GEOLOGIC AND GEOTECHNICAL SERVICES

The recommendations contained in this report are based on a limited number of borings and on the continuity of the sub-surface conditions encountered. GeoSolutions, Inc. assumes that it will be retained to provide additional services during future phases of the proposed project. These services would be provided by GeoSolutions, Inc. as required by County of Santa Barbara, the 2010 CBC, and/or industry standard practices. These services would be in addition to those included in this report and would include, but are not limited to, the following services:

1. Consultation during plan development.

2. Plan review of grading and foundation documents prior to construction and a report certifying that the reviewed plans are in conformance with our geotechnical and geologic recommendations.

3. Consultation during selection and placement of a laterally-reinforcing biaxial geogrid product.

4. Construction inspections and testing, as required, during all grading and excavating operations beginning with the stripping of vegetation at the Site, at which time a site meeting or pre-job meeting would be appropriate.

5. Special inspection services during construction of reinforced concrete, structural masonry, high strength bolting, epoxy embedment of threaded rods and reinforcing steel, and welding of structural steel.

6. Preparation of construction reports certifying that building pad preparation and foundation excavations are in conformance with our geotechnical and geologic recommendations.

7. Preparation of special inspection reports as required during construction.

8. In addition to the construction inspections listed above, section 1704.7 of the 2010 CBC (CBSC, 2010) requires the following inspections by the Soils Engineer for controlled fill thicknesses greater than 12 inches as shown in Table 16: Required Verification and Inspections of Soils:

Table 16: Required Verification and Inspections of Soils

Verification and Inspection Task	Continuous During Task Listed	Periodically During Task Listed
1. Verify materials below footings are adequate to achieve the design bearing capacity.	-	X
2. Verify excavations are extended to proper depth and have reached proper material.	-	X
3. Perform classification and testing of controlled fill materials.	-	X
4. Verify use of proper materials, densities and lift thicknesses during placement and compaction of controlled fill.	X	-
5. Prior to placement of controlled fill, observe sub-grade and verify that site has been prepared properly.	-	X

16.0 LIMITATIONS AND UNIFORMITY OF CONDITIONS

1. The recommendations of this report are based upon the assumption that the soil conditions do not deviate from those disclosed during our study. Should any variations or undesirable conditions be encountered during the development of the Site, GeoSolutions, Inc. should be notified immediately and GeoSolutions, Inc. will provide supplemental recommendations as dictated by the field conditions.

2. This report is issued with the understanding that it is the responsibility of the owner or his/her representative to ensure that the information and recommendations contained herein are brought to the attention of the architect and engineer for the project, and incorporated into the project plans and specifications. The owner or his/her representative is responsible to ensure that the necessary steps are taken to see that the contractor and subcontractors carry out such recommendations in the field.

3. As of the present date, the findings of this report are valid for the property studied. With the passage of time, changes in the conditions of a property can occur whether they are due to natural processes or to the works of man on this or adjacent properties. Therefore, this report should not be relied upon after a period of 3 years without our review nor should it be used or is it applicable for any properties other than those studied. However many events such as floods, earthquakes,

grading of the adjacent properties and building and municipal code changes could render sections of this report invalid in less than 3 years.

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REFERENCES

REFERENCES

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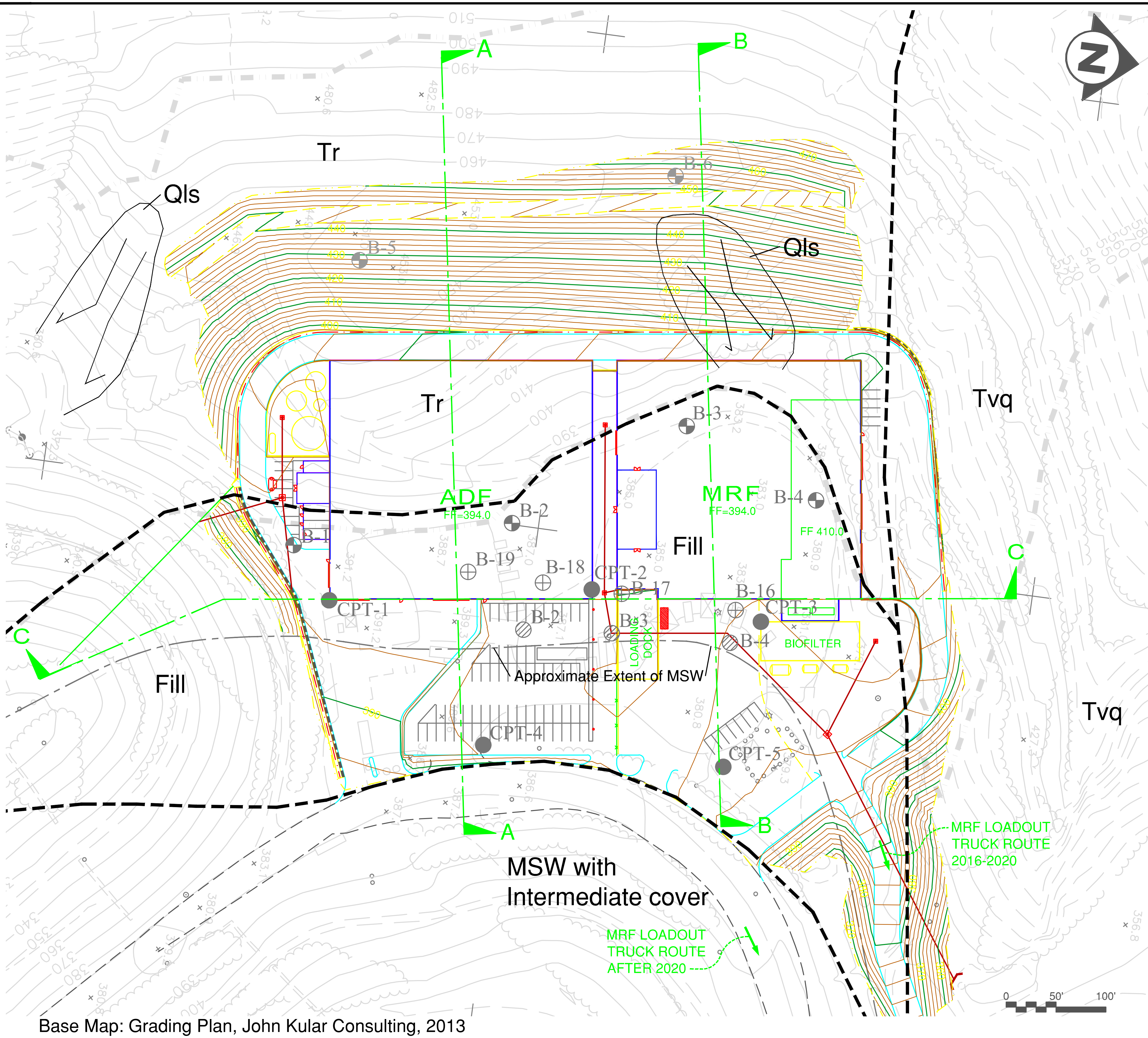
PLATES

Plate 1A, 1B, 1C - Site Engineering Geologic Map

Plate 2A, 2B, 2C – Site Cross Sections

Plate 3A, 3B – Regional Geologic Map, Dibblee, 1988 and Geologic Explanations

Plate 4 – Regional Fault Map, Jennings, 2010



Legend

MSW	Municipal Solid Waste
Fill	Artificial Fill
Qls	Landslide
Tr	Rincon Shale
Tvq	Vaqueros Sandstone
	Contact (dashed where approximate)
B-1	Boring (GeoSolutions, 2013)
CPT-1	CPT Sounding (GeoSolutions, 2013)
B-1	Boring (GeoLogic, 2007)
B-1	Boring (GeoLogic, 2003)
	Limit of Permitted West Borrow Area
	Waste Footprint

PLATE 1A

PROJECT SB00314-1

SITE ENGINEERING GEOLOGY MAP

MRF, ADF AND PERCOLATION TANKS
TAJIGUAS RESOURCE RECOVERY PROJECT,
TAJIGUAS LANDFILL, SANTA BARBARA COUNTY, CALIFORNIA

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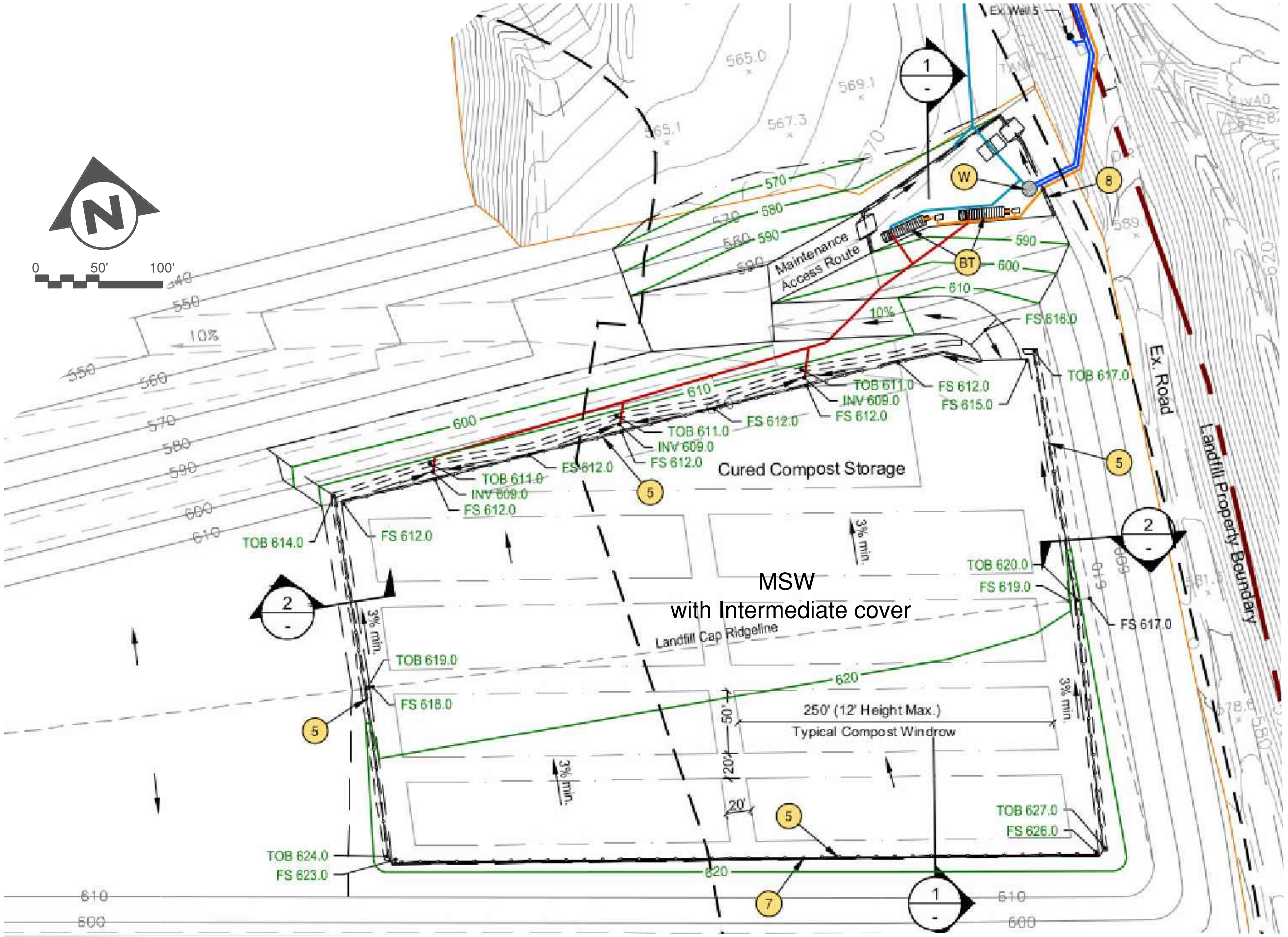
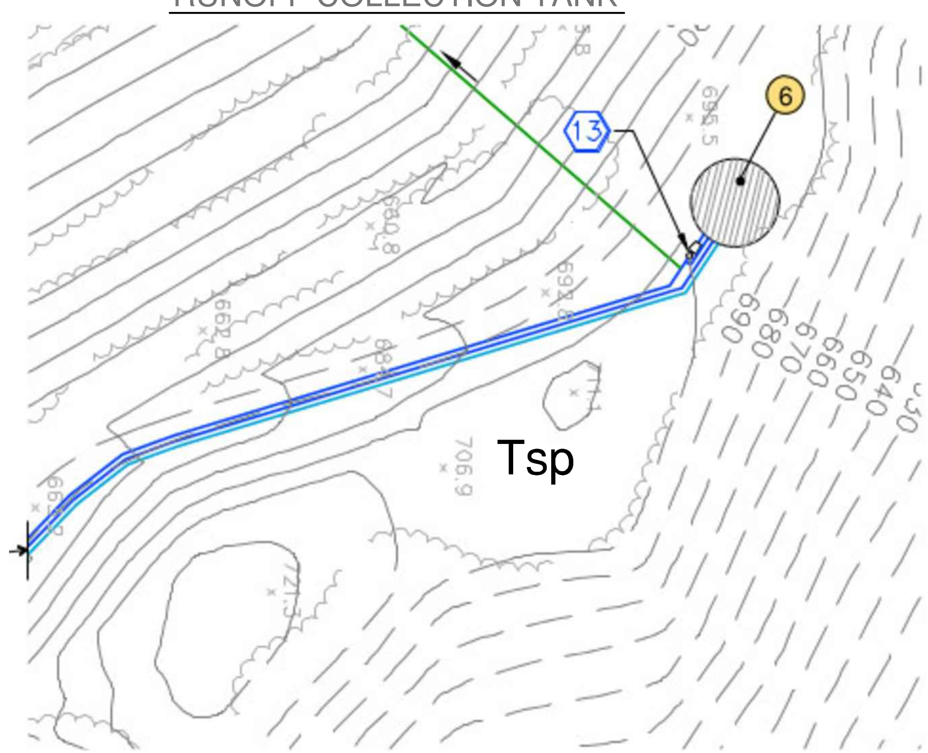


COMPOSTING AREA



RUNOFF COLLECTION TANK

Legend

- Runoff Collection
- Runoff to Runoff Collection Tank
- Overflow Collection to Existing Sedimentation Basin
- Water Supply from Well or Runoff Collection Tank
- Landfill Liner Limits
- 600 Landfill Closure Grading
- 600 Composting Area Grading

Legend - Geology

MSW	Municipal Solid Waste
Tsp	Sespe Formation

Base Map: Grading Plan, John Kular Consulting, 2013

PLATE 1B

PROJECT SB00314-1

SITE ENGINEERING GEOLOGY MAP

COMPOSTING AREA AND RUNOFF COLLECTION TANK
TAJIGUAS RESOURCE RECOVERY PROJECT,
TAJIGUAS LANDFILL, SANTA BARBARA COUNTY, CALIFORNIA

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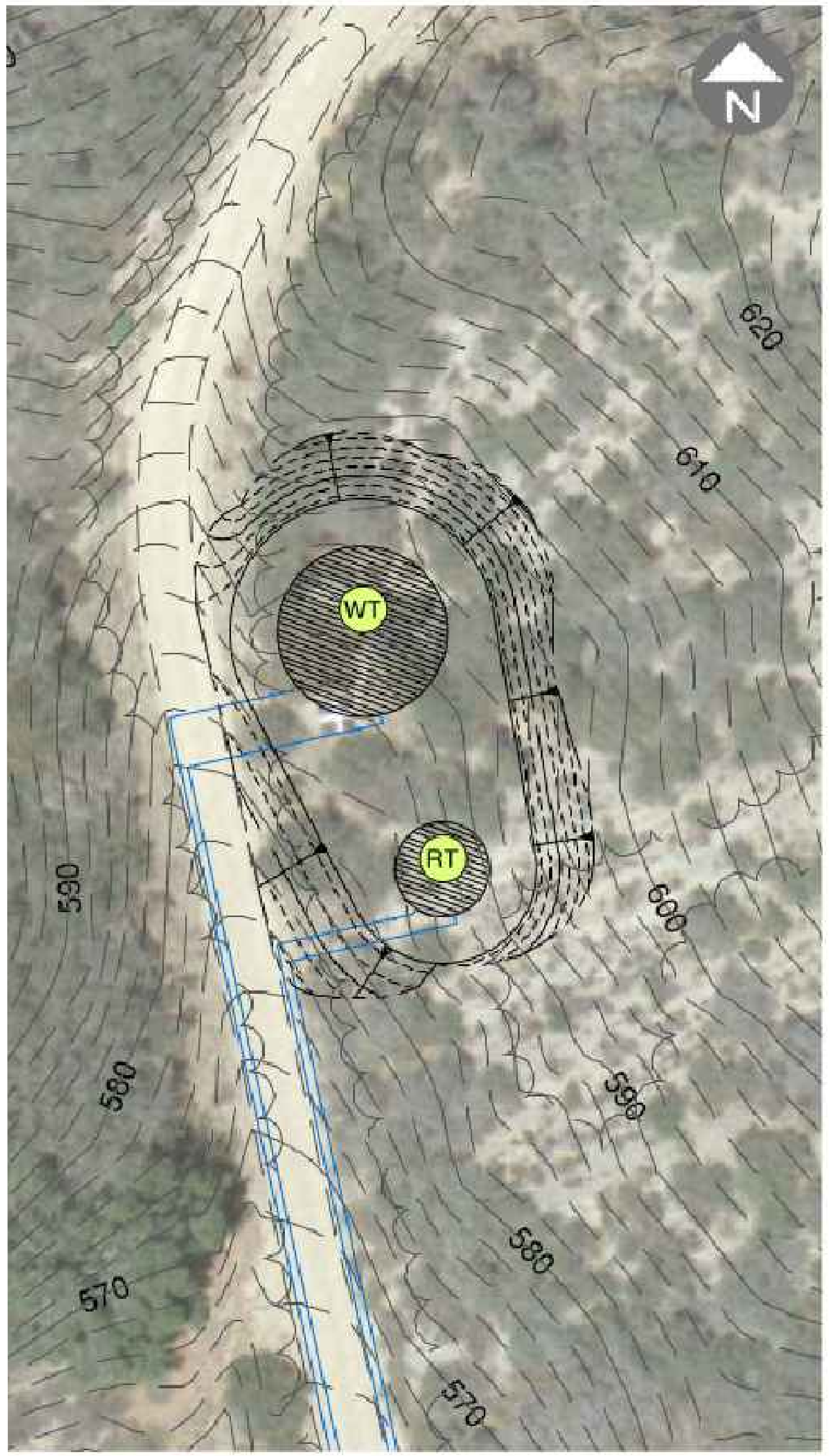
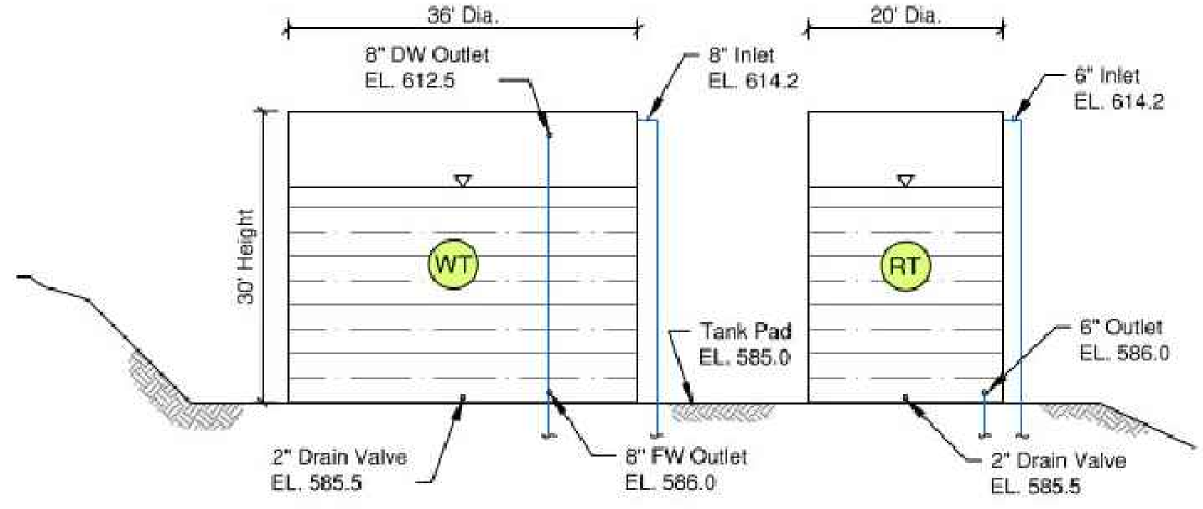


PLAN

SCALE: 1"= 40'
0' 40' 80'



ELEVATION

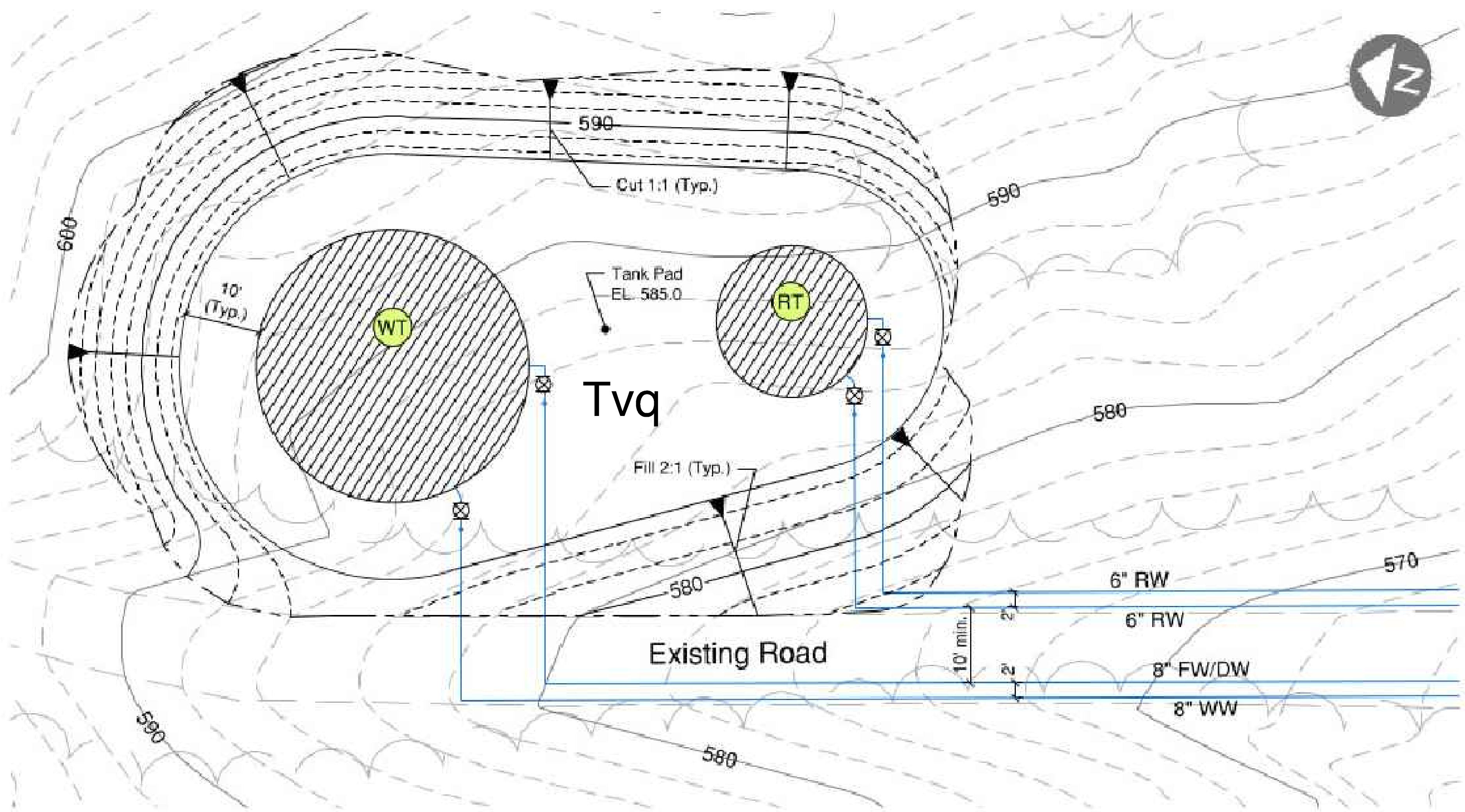
Keynotes

WT Proposed Well Water Storage Tank - 220,000 gallons

RT Proposed Recycled Water Storage Tank - 70,000 gallons

Legend - Geology

Tvq Vaqueros Sandstone



PLAN

SCALE: 1"= 20'
0' 20' 40'

Base Map: Grading Plan, John Kular Consulting, 2013

PLATE 1C

PROJECT SB00314-1

SITE ENGINEERING GEOLOGY MAP

WELL WATER STORAGE TANK AND RECYCLED WATER TANK
TAJIGUAS RESOURCE RECOVERY PROJECT,
TAJIGUAS LANDFILL, SANTA BARBARA COUNTY, CALIFORNIA

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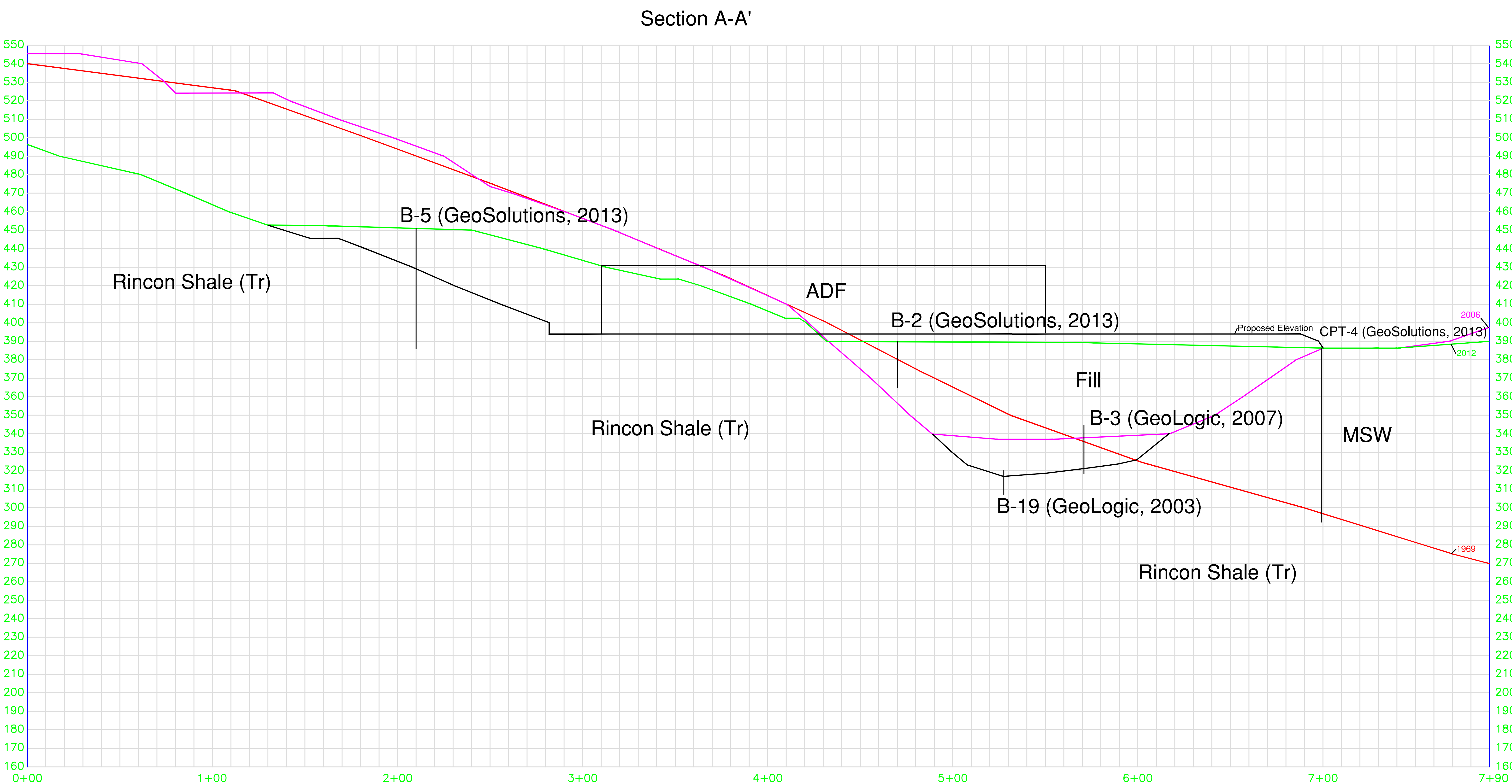

Legend

Fill	Artificial Fill
MSW	Municipal Solid Waste
Tr	Rincon Shale

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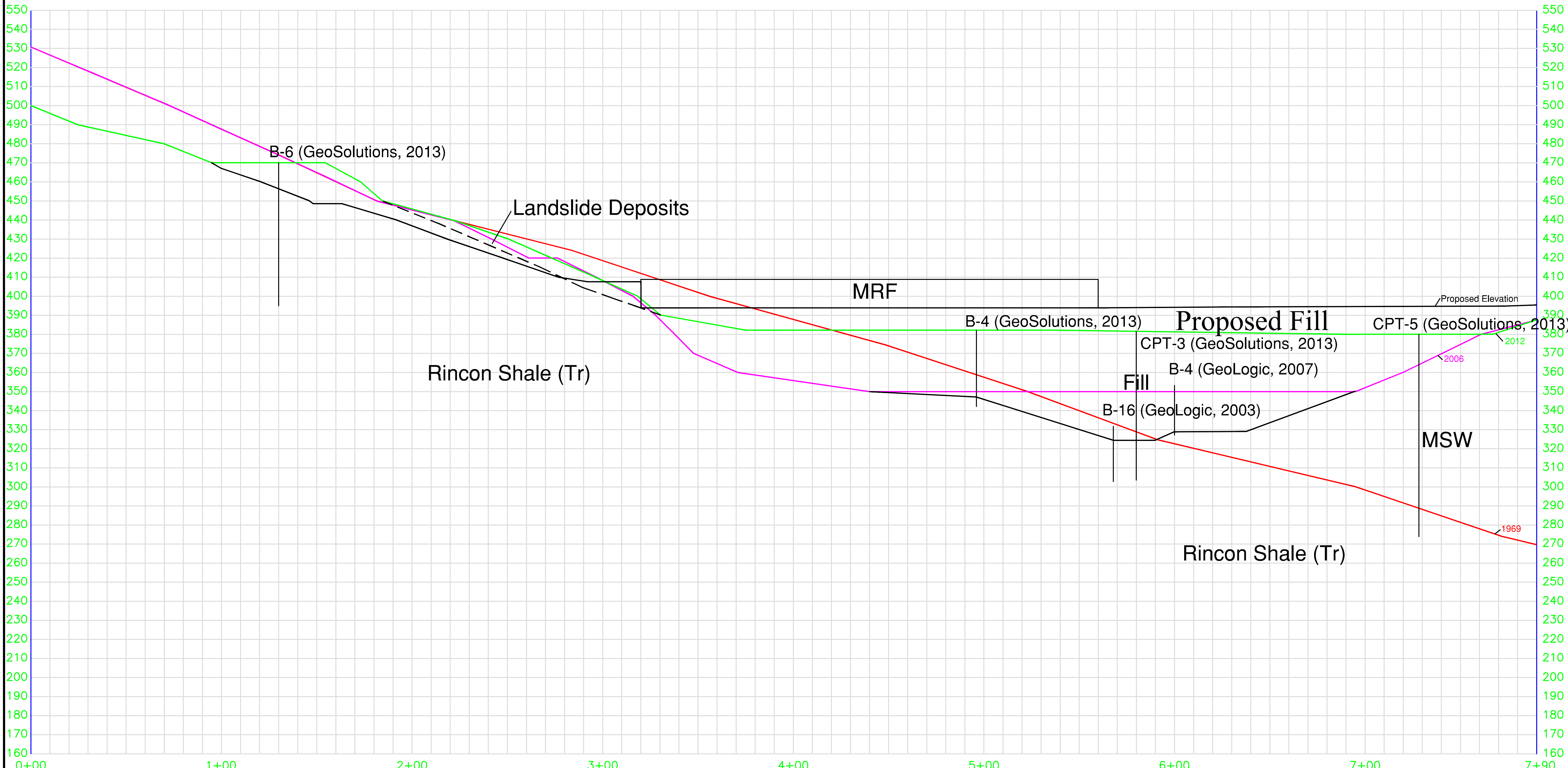
SITE CROSS SECTION
TAJIGUAS RESOURCE RECOVERY PROJECT,
TAJIGUAS LANDFILL, SAN BARBARA COUNTY, CALIFORNIA

PLATE
2A

PROJECT
SB00314-1

Section B-B'



Legend

Fill	Artificial Fill
MSW	Municipal Solid Waste
Tr	Rincon Shale

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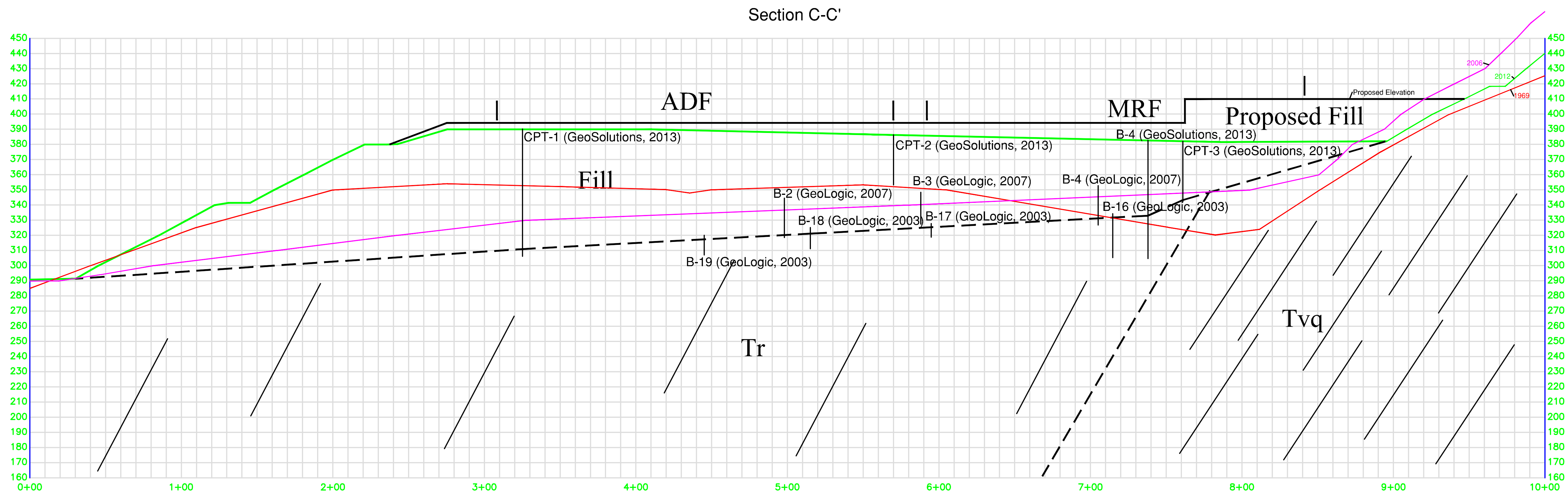
SITE CROSS SECTION
TAJIGUAS RESOURCE RECOVERY PROJECT,
TAJIGUAS LANDFILL, SAN BARBARA COUNTY, CALIFORNIA

PLATE
2B

PROJECT
SB00314-1



Legend

Fill	Artificial Fill
Tr	Rincon Shale
Tvq	Vaqueros Sandstone
— — —	Contact (dashed where approximate)

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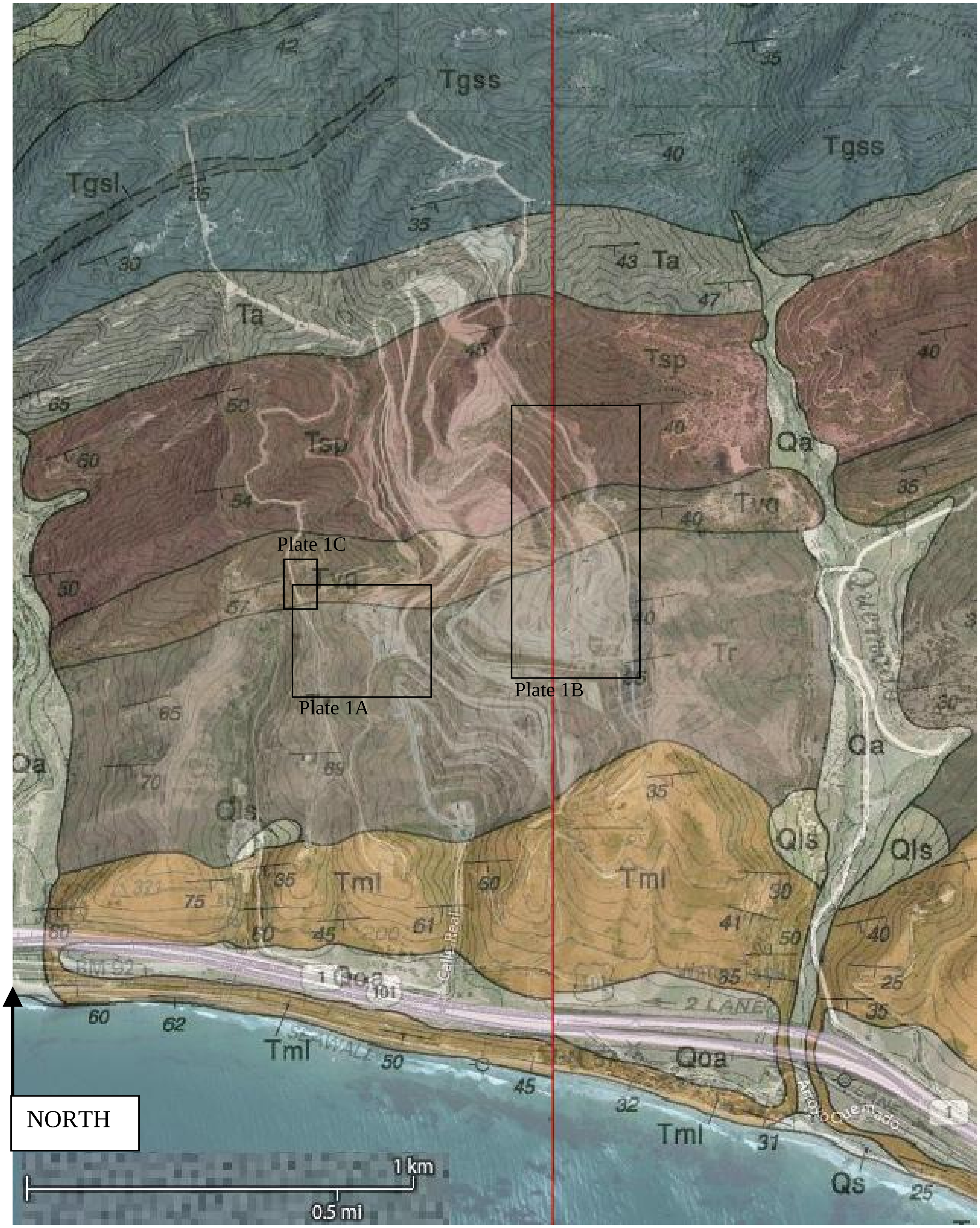
SITE CROSS SECTION

TAJIGUAS RESOURCE RECOVERY PROJECT,
TAJIGUAS LANDFILL, SAN BARBARA COUNTY, CALIFORNIA

PLATE 2C

PROJECT SB00314-1



Solvang and Gaviota Quadrangles, Dibblee, 1988

Santa Ynez and Tajiguas Quadrangles, Dibblee, 1988

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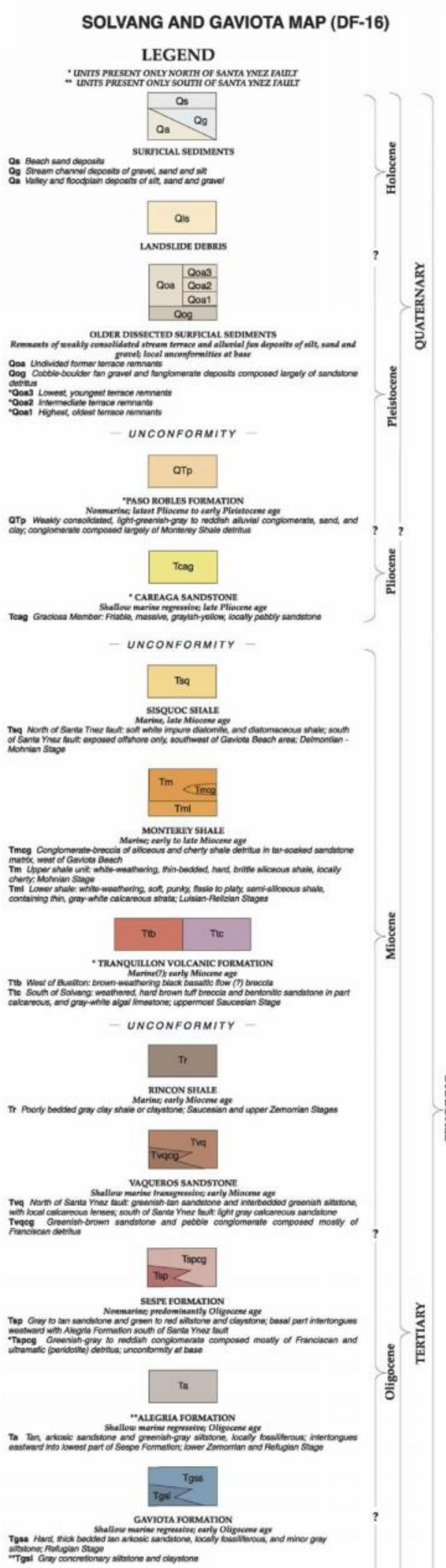
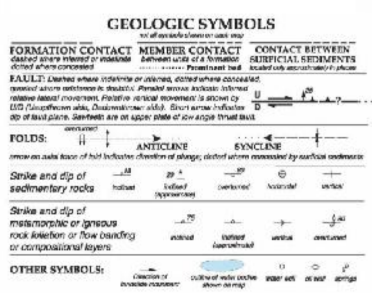
REGIONAL GEOLOGIC MAP

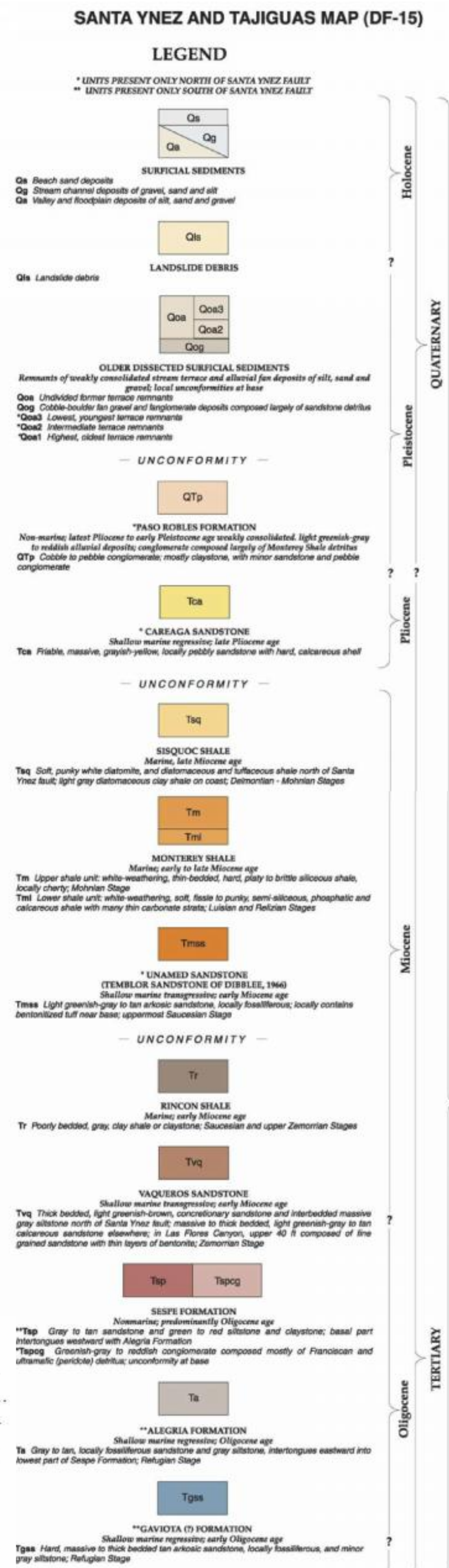
DIBBLEE, 1988
TAJIGUAS RESOURCE RECOVERY PROJECT,
TAJIGUAS LANDFILL,
SANTA BARBARA COUNTY, CALIFORNIA

PLATE
3A

PROJECT NO:
SB00314-1







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GEOLOGIC EXPLANATIONS

DIBBLEE, 1988
TAJIGUAS RESOURCE RECOVERY PROJECT,
TAJIGUAS LANDFILL,
SANTA BARBARA COUNTY, CALIFORNIA

PLATE
3B

PROJECT NO: SB00314-1

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REGIONAL FAULT MAP

TAJIGUAS RESOURCE RECOVERY PROJECT,
TAJIGUAS LANDFILL,
SANTA BARBARA COUNTY, CALIFORNIA

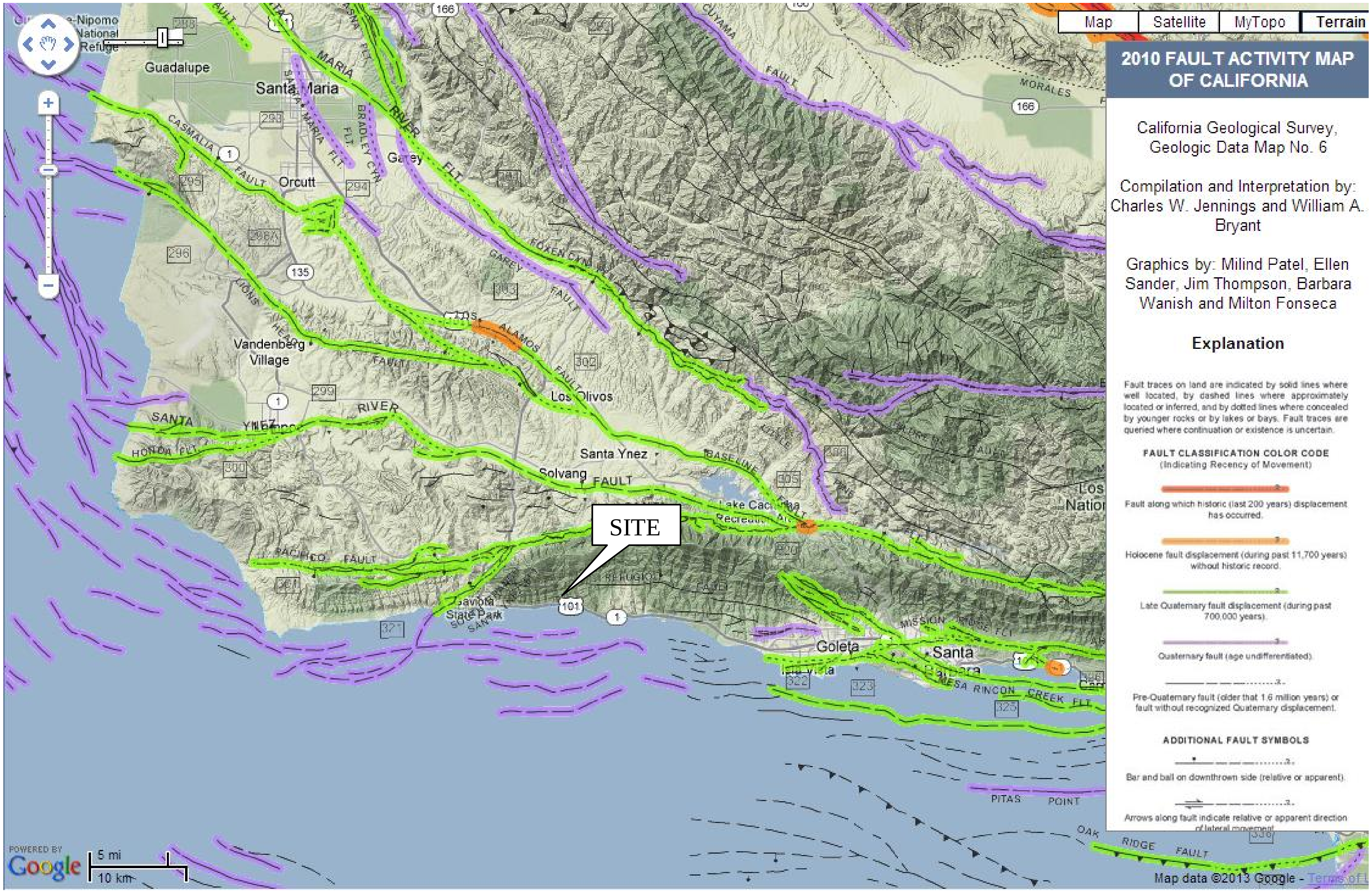


PLATE
4

PROJECT
SB00314-1

APPENDIX A

Field Investigation

Soil Classification Chart

Boring Logs

CPT Soundings

Design Maps Summary Report

FIELD INVESTIGATION

The field investigation was conducted on various dates between January 8 and January 31, 2013 using a CPT Truck provided by Middle Earth Geo Testing, Inc. and our track-mounted CME 55 drill rig. The surface and sub-surface conditions were studied by advancing five CPT soundings and six exploratory borings. This exploration was conducted in accordance with presently accepted geotechnical engineering procedures consistent with the scope of the services authorized to GeoSolutions, Inc.

The CPT sounding with a 20-ton electronic CPT cone is advanced with measurements for cone bearing (q_C), sleeve friction (f_S), and pore water pressure (u) measurements recorded at approximately 5-cm intervals. This provides a near continuous hydro geologic log. All CPT soundings are performed in accordance with ASTM D5778-95 (re-approved 2002) standards.

The CME 55 drill rig with eight-inch diameter hollow-stem continuous flight augers bored six exploratory borings near the approximate locations indicated on Figure 3: CPT and Boring Locations. The drilling and field observation was performed under the direction of the project engineer/engineering geologist. A representative of GeoSolutions, Inc. maintained a log of the soil conditions and obtained soil samples suitable for laboratory testing. The soils were classified in accordance with the Unified Soil Classification System. See the Soil Classification Chart in this appendix.

Standard Penetration Tests with a two-inch outside diameter standard split tube sampler (SPT) without liners (ASTM D1586-99) and a three-inch outside diameter Modified California (CA) split tube sampler with liners (ASTM D3550-01) were performed to obtain field indication of the in-situ density of the soil and to allow visual observation of at least a portion of the soil column. Soil samples obtained with the split spoon sampler are retained for further observation and testing. The split spoon samples are driven by a 140-pound hammer free falling 30 inches. The sampler is initially seated six inches to penetrate any loose cuttings and is then driven an additional 12 inches with the results recorded in the boring logs as N-values, which area the number of blows per foot required to advance the sample the final 12 inches.

The CA sampler is a larger diameter sampler than the standard (SPT) sampler with a two-inch outside diameter and provides additional material for normal geotechnical testing such as in-situ shear and consolidation testing. Either sampler may be used in the field investigation, but the N-values obtained from using the CA sampler will be greater than that of the SPT. The N-values for samples collected using the CA can be roughly correlated to SPT N-values using a conversion factor that may vary from about 0.5 to 0.7. A commonly used conversion factor is 0.67 ($^2/_3$). More information about standardized samplers can be found in ASTM D1586-99 and ASTM D3550-01.

Disturbed bulk samples are obtained from cuttings developed during boring operations. The bulk samples are selected for classification and testing purposes and may represent a mixture of soils within the noted depths. Recovered samples are placed in transport containers and returned to the laboratory for further classification and testing.

Logs of the borings showing the approximate depths and descriptions of the encountered soils, applicable geologic structures, recorded N-values, and the results of laboratory tests are presented in **Appendix B**. The logs represent the interpretation of field logs and field tests as well as the interpolation of soil conditions between samples. The results of laboratory observations and tests are also included in the boring logs. The stratification lines recorded in the boring logs represent the approximate boundaries between the surface soil types. However, the actual transition between soil types may be gradual or varied.

SOIL CLASSIFICATION CHART

MAJOR DIVISIONS		LABORATORY CLASSIFICATION CRITERIA		GROUP SYMBOLS	PRIMARY DIVISIONS
COARSE GRAINED SOILS More than 50% retained on No. 200 sieve	GRAVELS More than 50% of coarse fraction retainined on No. 4 (4.75mm) sieve	Clean gravels (less than 5% fines*)	C_u greater than 4 and C_z between 1 and 3	GW	Well-graded gravels and gravel-sand mixtures, little or no fines
			Not meeting both criteria for GW	GP	Poorly graded gravels and gravel-sand mixtures, little or no fines
		Gravel with fines (more than 12% fines*)	Atterberg limits plot below "A" line or plasticity index less than 4	GM	Silty gravels, gravel-sand-silt mixtures
			Atterberg limits plot below "A" line and plasticity index greater than 7	GC	Clayey gravels, gravel-sand-clay mixtures
	SANDS More than 50% of coarse fraction passes No. 4 (4.75mm) sieve	Clean sand (less than 5% fines*)	C_u greater than 6 and C_z between 1 and 3	SW	Well graded sands, gravely sands, little or no fines
			Not meeting both criteria for SW	SP	Poorly graded sands and gravelly and sands, little or no fines
		Sand with fines (more than 12% fines*)	Atterberg limits plot below "A" line or plasticity index less than 4	SM	Silty sands, sand-silt mixtures
			Atterberg limits plot above "A" line and plasticity index greater than 7	SC	Clayey sands, sand-clay mixtures
FINE GRAINED SOILS 50% or more passes No. 200 sieve	SILTS AND CLAYS (liquid limit less than 50)	Inorganic soil	PI < 4 or plots below "A"-line	ML	Inorganic silts, very fine sands, rock flour, silty or clayey fine sands
		Inorganic soil	PI > 7 and plots on or above "A" line**	CL	Inorganic clays of low to medium plasticity, gravelly clays, sandy clays, silty clays, lean clays
		Organic Soil	LL (oven dried)/LL (not dried) < 0.75	OL	Organic silts and organic silty clays of low plasticity
	SILTS AND CLAYS (liquid limit 50 or more)	Inorganic soil	Plots below "A" line	MH	Inorganic silts, micaceous or diatomaceous fine sands or silts, elastic silts
		Inorganic soil	Plots on or above "A" line	CH	Inorganic clays of high plasticity, fat clays
		Organic Soil	LL (oven dried)/LL (not dried) < 0.75	OH	Organic silts and organic clays of high plasticity
Peat	Highly Organic	Primarily organic matter, dark in color, and organic odor		PT	Peat, muck and other highly organic soils

*Fines are those soil particles that pass the No. 200 sieve. For gravels and sands with between 5 and 12% fines, use of dual symbols is required (I.e. GW-GM, GW-GC, GP-GM, or GP-GC).

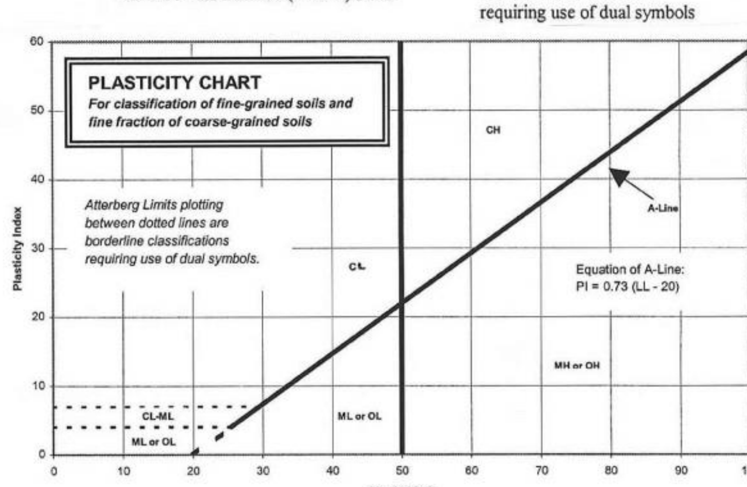
**If the plasticity index is between 4 and 7 and it plots above the "A" line, then dual symbols (I.e. CL-ML) are required.

CLASSIFICATIONS BASED ON PERCENTAGE OF FINES

Less than 5%, Pass No. 200 (75mm)sieve)	GW, GP, SW, SP
More than 12% Pass N. 200 (75 mm) sieve	GM, GC, SM, SC
5%-12% Pass No. 200 (75 mm) sieve	Borderline Classification requiring use of dual symbols

CONSISTENCY

CLAYS AND PLASTIC SILTS	STRENGTH TON/SQ. FT ++	BLOWS/ FOOT +
VERY SOFT	0 - 1/4	0 - 2
SOFT	1/4 - 1/2	2 - 4
FIRM	1/2 - 1	4 - 8
STIFF	1 - 2	8 - 16
VERY STIFF	2 - 4	16 - 32
HARD	Over 4	Over 32



RELATIVE DENSITY

SANDS, GRAVELS AND NON-PLASTIC SILTS	BLOWS/ FOOT +
VERY LOOSE	0 - 4
LOOSE	4 - 10
MEDIUM DENSE	10 - 30
DENSE	30 - 50
VERY DENSE	Over 50

\+ Number of blows of a 140-pound hammer falling 30-inches to drive a 2-inch O.D. (1-3/8-inch I.D.) split spoon (ASTM D1586).

++ Unconfined compressive strength in tons/sq.ft. as determined by laboratory testing or approximated by the standard penetration test (ASTM D1586), pocket penetrometer, torvane, or visual observation.

Drilling Notes:

1. Sampling and blow counts
   a. California Modified – number of blows per foot of a 140 pound hammer falling 30 inches
   b. Standard Penetration Test – number of blows per 12 inches of a 140 pound hammer falling 30 inches

Types of Samples:
X – In-Situ
SPT - Standard Penetration
CA - California Modified
N - Nuclear Gauge
PO – Pocket Penetrometer (tons/sq.ft.)

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Santa Maria, CA 93454

BORING LOG

BORING NO. **B-1**

JOB NO. **SB00314-1**

PROJECT INFORMATION		DRILLING INFORMATION	
PROJECT:	**Tajiguas Landfill**	DRILL RIG:	**CME 55**
DRILLING LOCATION:	**Operations Deck**	HOLE DIAMETER:	**8 Inches**
DATE DRILLED:	**January 10, 2013**	SAMPLING METHOD:	**SPT**
LOGGED BY:	**JAP**	HOLE ELEVATION:	**390 Feet**

Depth of Groundwater: **Not Encountered** Boring Terminated At: **10 Feet**

DEPTH	SOIL DESCRIPTION	USCS	LITHOLOGY	SAMPLE	BLOWS/ 12 IN	$(N_1)_{60}$	FRICTION ANGLE, (degrees)	COHESION, C (psf)	OPTIMUM WATER CONTENT (%)	MAXIMUM DRY DENSITY (pcf)	EXPANSION INDEX (EI)	FINES CONTENT (%)	PLASTICITY INDEX (PI)
0	CLAYEY SAND: light brown, slightly moist, (FILL)	SC											
-5	dense			SPT	14	33							
-10	dense			SPT	15	25							

Depth scale: 0, -1, -2, -3, -4, -5, -6, -7, -8, -9, -10, -11, -12, -13, -14, -15, -16, -17, -18, -19, -20, -21, -22, -23, -24, -25

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BORING LOG

BORING NO. **B-2**

JOB NO. **SB00314-1**

PROJECT INFORMATION		DRILLING INFORMATION	
PROJECT:	**Tajiguas Landfill**	DRILL RIG:	**CME 55**
DRILLING LOCATION:	**Operations Deck**	HOLE DIAMETER:	**8 Inches**
DATE DRILLED:	**January 10, 2013**	SAMPLING METHOD:	**CA/SPT**
LOGGED BY:	**JAP**	HOLE ELEVATION:	**390 Feet**

Depth of Groundwater: **Not Encountered** Boring Terminated At: **25 Feet**

DEPTH	SOIL DESCRIPTION	USCS	LITHOLOGY	SAMPLE	BLOWS/ 12 IN	$(N_1)_{60}$	FRICTION ANGLE, (degrees)	COHESION, C (psf)	OPTIMUM WATER CONTENT (%)	MAXIMUM DRY DENSITY (pcf)	EXPANSION INDEX (EI)	FINES CONTENT (%)	PLASTICITY INDEX (PI)
0	SANDY CLAY: reddish brown to light brown, with gravel, slightly moist, FILL	CH											
-5	very stiff			SPT	15	35							
-10	very stiff			CA	25	23							
-15	very stiff			SPT	19	29							
-18	SANDY CLAY: light brown with gravel of white coarse-grain sandstone, dry to slightly moist, FILL	CH											
-20	hard strong odor, methane detected			SPT	32	42							
-24	very stiff			SPT	17	22							

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BORING LOG

BORING NO. **B-3**

JOB NO. **SB00314-1**

PROJECT INFORMATION		DRILLING INFORMATION	
PROJECT:	**Tajiguas Landfill**	DRILL RIG:	**CME 55**
DRILLING LOCATION:	**Operations Deck**	HOLE DIAMETER:	**8 Inches**
DATE DRILLED:	**January 10, 2013**	SAMPLING METHOD:	**CA/SPT**
LOGGED BY:	**JAP**	HOLE ELEVATION:	**380 Feet**

Depth of Groundwater: **Not Encountered** Boring Terminated At: **25 Feet** Page 1 of 1

DEPTH	SOIL DESCRIPTION	USCS	LITHOLOGY	SAMPLE	BLOWS/ 12 IN	$(N_1)_{60}$	FRICTION ANGLE, (degrees)	COHESION, C (psf)	OPTIMUM WATER CONTENT (%)	MAXIMUM DRY DENSITY (pcf)	EXPANSION INDEX (EI)	FINES CONTENT (%)	PLASTICITY INDEX (PI)
0	SANDY CLAY: reddish brown to light brown, with gravel of claystone, slightly moist, FILL	CH											
-4							31.1	651					
-5	SILTY SAND: light brown to white fine-grained to medium-grained with gravel of sandstone, dry to slightly moist, hard, FILL	SM		SPT	50	65							
-6				B					8.1	130.6			
-10	very dense			CA	20	33							
-15	very dense			SPT	35	53							
-20	very dense			SPT	35	46							
-25	very dense			SPT	40	53							

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BORING LOG

BORING NO. **B-4**

JOB NO. **SB00314-1**

PROJECT INFORMATION	DRILLING INFORMATION
PROJECT: **Tajiguas Landfill**	DRILL RIG: **CME 55**
DRILLING LOCATION: **Operations Deck**	HOLE DIAMETER: **8 Inches**
DATE DRILLED: **January 8, 2013**	SAMPLING METHOD: **CA/SPT**
LOGGED BY: **JAP**	HOLE ELEVATION: **380 Feet**

Depth of Groundwater: **Not Encountered** Boring Terminated At: **40 Feet**

DEPTH	SOIL DESCRIPTION	USCS	LITHOLOGY	SAMPLE	BLOWS/ 12 IN	$(N_1)_{60}$	FRICTION ANGLE, (degrees)	COHESION, C (psf)	OPTIMUM WATER CONTENT (%)	MAXIMUM DRY DENSITY (pcf)	EXPANSION INDEX (EI)	FINES CONTENT (%)	PLASTICITY INDEX (PI)
0	SILTY SAND: reddish brown to brown, with gravel of claystone and sandstone, dry to slightly moist, FILL	SM											
-1				A					10	128.8	67		24
-4	very dense												
-5				CA	52	67							
-8	very dense												
-10				SPT	28	46							
-15	CLAYEY SAND: light brown, with gravel, dry to slightly moist, very dense, FILL	SC		SPT	38	58							
-17	SANDY CLAY: dark brown, with gravel of claystone and sandstone, slightly moist, FILL	CH											

Depth scale: 0, -1, -2, -3, -4, -5, -6, -7, -8, -9, -10, -11, -12, -13, -14, -15, -16, -17, -18

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BORING LOG

BORING NO (cont). **B-4**

JOB NO. **SB00314-1**

Depth of Groundwater: **Not Encountered** Boring Terminated At: **40 Feet**

DEPTH	SOIL DESCRIPTION	USCS	LITHOLOGY	SAMPLE	BLOWS/ 12 IN	$(N_1)_{60}$	FRICTION ANGLE (degrees)	COHESION, C (psf)	OPTIMUM WATER CONTENT (%)	MAXIMUM DRY DENSITY (pcf)	EXPANSION INDEX (EI)	FINE CONTENTS	PLASTICITY INDEX (PI)
-19													
-20	very stiff			SPT	22	29							
-21													
-22													
-23													
-24													
-25	very stiff			SPT	20	26							
-26													
-27													
-28													
-29	stiff												
-30				SPT	13	16							
-31													
-32													
-33													
-34													
-35	CLAYSTONE: blueish gray to light brown, severely to moderately weathered, dry, highly fractured, hard, RINCON FORMATION (Tr)			SPT	30	35							
-36				C			25	301	17.4	107.0	88		26
-37													
-38	hard												
-39													
-40													

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CORING LOG

BORING NO. **B-5**

JOB NO. **SB00314-1**

PROJECT INFORMATION		DRILLING INFORMATION	
PROJECT:	**Tajiguas Landfill**	DRILL RIG:	**CME 55**
DRILLING LOCATION:	**North Slope Bench**	HOLE DIAMETER:	**8 Inches**
DATE DRILLED:	**January 23, 2013**	SAMPLING METHOD:	**CA/Coring**
LOGGED BY:	**JAP**	HOLE ELEVATION:	**451 Feet**

Depth of Groundwater: **Not Encountered** Boring Terminated At: **65 Feet**

DEPTH	SOIL DESCRIPTION	USCS	LITHOLOGY	SAMPLE	BLOWS/ 12 IN	ROCK QUALITY	RQD	PERCENT RECOVERED	FRICTION ANGLE, (degrees)	COHESION, C (psf)
0 to -4	CLAYSTONE: pale yellow (2.5yr 8/4) to dark gray (2.5y 4/1), highly fractured to friable (at the surface), slightly moist, moderately to intensely weathered (at surface), slightly weathered at depth, moderately hard, RINCON FORMATION (Tr)				50/2"					
-5	start of coring					poor	45	90		
-7	highly fractured									
-10						fair	73	100		
-11	moderately to slightly fractured									
-15						fair	73	93		
-20	highly cemented, hard					fair	55	100		
-25	moderately soft					good	81	86		

Depth scale: 0, -1, -2, -3, -4, -5, -6, -7, -8, -9, -10, -11, -12, -13, -14, -15, -16, -17, -18, -19, -20, -21, -22, -23, -24, -25, -26, -27, -28, -29, -30

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CORING LOG

BORING NO (cont).**B-5**

JOB NO. **SB00314-1**

Depth of Groundwater: **Not Encountered** Boring Terminated At: **65 Feet**

DEPTH	SOIL DESCRIPTION	USCS	LITHOLOGY	SAMPLE	BLOWS/ 12 IN	ROCK QUALITY	RQD	PERCENT RECOVERED	FRICTION ANGLE (degrees)	COHESION, C (psf)
-30 to -35	fractures cemented and filled with white calcium carbonate material.					excellent	93	93		
-35 to -40						good	76	95		
-40 to -45						good	76	100		
-45 to -50						good	95	95		
-50 to -55	dry very hard, fractures cemented with white carbonite material					good	85	100		
-55 to -60						good	78	93		
-60 to -65	refused in hard rock					good	81	90		

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CORING LOG

BORING NO. **B-6**

JOB NO. **SB00314-1**

PROJECT INFORMATION		DRILLING INFORMATION	
PROJECT:	**Tajiguas Landfill**	DRILL RIG:	**CME 55**
DRILLING LOCATION:	**North Slope Bench**	HOLE DIAMETER:	**8 Inches**
DATE DRILLED:	**January 23, 2013**	SAMPLING METHOD:	**CA/Coring**
LOGGED BY:	**JAP**	HOLE ELEVATION:	**451 Feet**

Depth of Groundwater: **Not Encountered** Boring Terminated At: **75 Feet**

DEPTH	SOIL DESCRIPTION	USCS	LITHOLOGY	SAMPLE	BLOWS/ 12 IN	ROCK QUALITY	RQD	PERCENT RECOVERED	FRICTION ANGLE, (degrees)	COHESION, C (psf)
0										
-1	CLAYSTONE: pale yellow (2.5yr 8/4) to dark gray (2.5y 4/1), highly fractured to friable (at the surface), slightly moist, moderately to intensely weathered (at surface), slightly weathered at depth, moderately hard, RINCON FORMATION (Tr)									
-3									7	1417
-4	- -				50/5"					
-5	start of coring					very poor	10	71		
-6	moderately fractured, caliche within fractures, soft									
-10						very poor	13	86		
-14	slightly fractured, moderately hard									
-15						fair	70	70		
-20						fair	73	96		
-23	Layer of highly fractured, ~1' thick									
-25						fair	53	53		
-30						fair	65	100		
-31	Layer of highly fractured, ~1' thick									
-34										

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CORING LOG

BORING NO (cont). **B-6**

JOB NO. **SB00314-1**

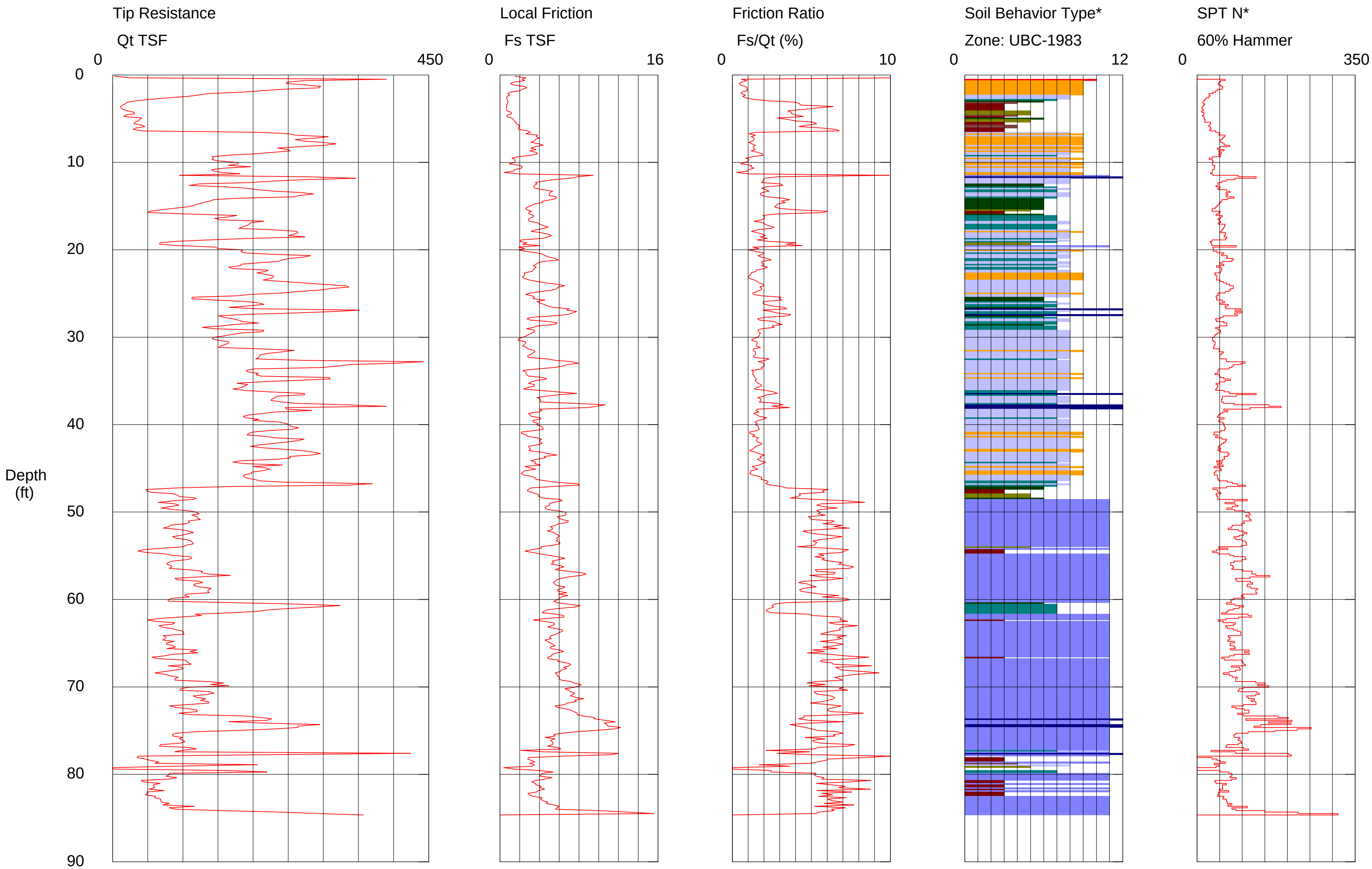
Depth of Groundwater: **Not Encountered** Boring Terminated At: **75 Feet**

DEPTH	SOIL DESCRIPTION	USCS	LITHOLOGY	SAMPLE	BLOWS/ 12 IN	ROCK QUALITY	RQD	PERCENT RECOVERED	FRICTION ANGLE (degrees)	COHESION, C (psf)
-35						good	85	85		
-39	Layer of soft, firable claystone									
-40						good	83	100		
-42	Layer of soft, firable claystone									
-45						good	88	88		
-49	Highly fractured									
-50						fair	50	100		
-52	Slightly fractured									
-55						excellent	100	100		
-60						good	88	88		
-65						very poor	18	38		
-68	Highly fractured									
-70						poor	36	93		
-75										

GeoSolutions

Operator: RA-JC
Sounding: CPT-01
Cone Used: DSG1104

CPT Date/Time: 1/9/2013 10:10:17 AM
Location: Tajiguas Landfill
Job Number: SB00314-1



Maximum Depth = 84.65 feet

Depth Increment = 0.164 feet

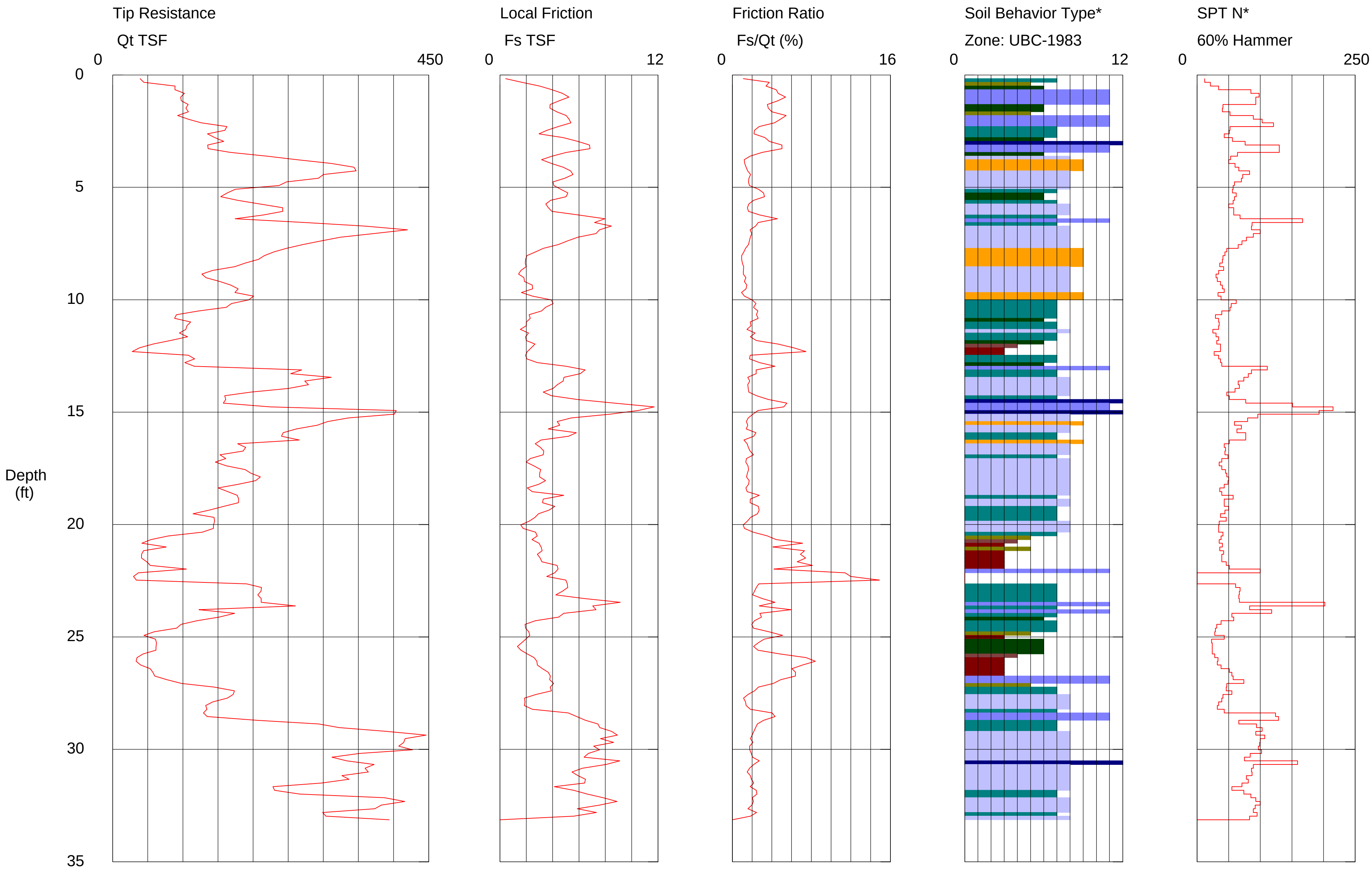
1	sensitive fine grained
2	organic material
3	clay
4	silty clay to clay
5	clayey silt to silty clay
6	sandy silt to clayey silt
7	silty sand to sandy silt
8	sand to silty sand
9	sand
10	gravelly sand to sand
11	very stiff fine grained (*)
12	sand to clayey sand (*)

*Soil behavior type and SPT based on data from UBC-1983

GeoSolutions

Operator: RA-JC
Sounding: CPT-02b
Cone Used: DSG1104

CPT Date/Time: 1/8/2013 1:11:30 PM
Location: Tajiguas Landfill
Job Number: SB00314-1



Maximum Depth = 33.14 feet

Depth Increment = 0.164 feet

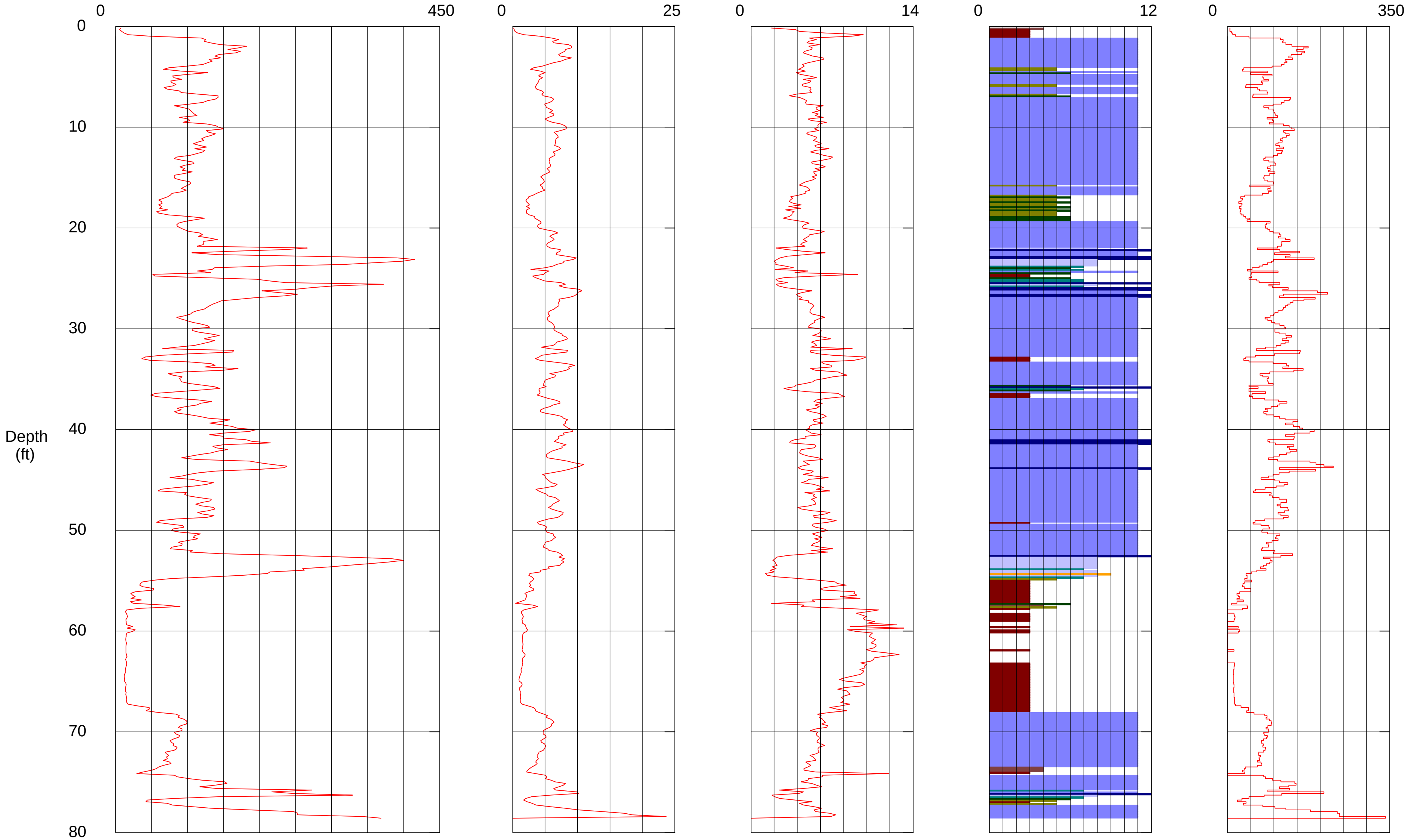
1 sensitive fine grained
2 organic material
3 clay
4 silty clay to clay
5 clayey silt to silty clay
6 sandy silt to clayey silt
7 silty sand to sandy silt
8 sand to silty sand
9 sand
10 gravelly sand to sand
11 very stiff fine grained (*)
12 sand to clayey sand (*)

*Soil behavior type and SPT based on data from UBC-1983

GeoSolutions

Operator: RA-JC
Sounding: CPT-03a
Cone Used: DSG1104

CPT Date/Time: 1/9/2013 12:18:49 PM
Location: Tajiguas Landfill
Job Number: SB00314-1



Maximum Depth = 78.58 feet

Depth Increment = 0.164 feet

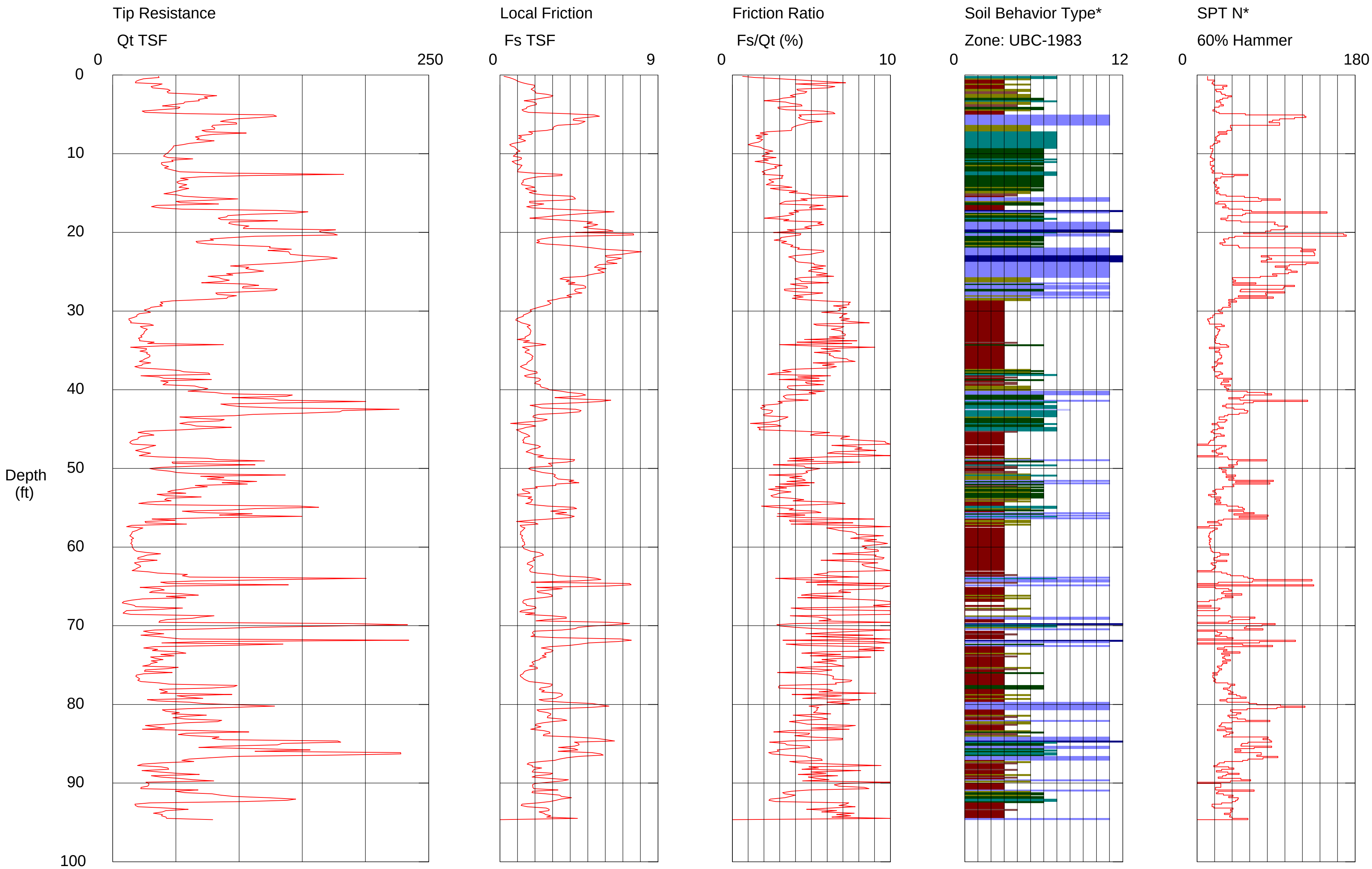
1 sensitive fine grained
2 organic material
3 clay
4 silty clay to clay
5 clayey silt to silty clay
6 sandy silt to clayey silt
7 silty sand to sandy silt
8 sand to silty sand
9 sand
10 gravelly sand to sand
11 very stiff fine grained (*)
12 sand to clayey sand (*)

*Soil behavior type and SPT based on data from UBC-1983

GeoSolutions

Operator: RA-JC
Sounding: CPT-04b
Cone Used: DSG1104

CPT Date/Time: 1/8/2013 2:20:09 PM
Location: Tajiguas Landfill
Job Number: SB00314-1



Maximum Depth = 94.65 feet

Depth Increment = 0.164 feet

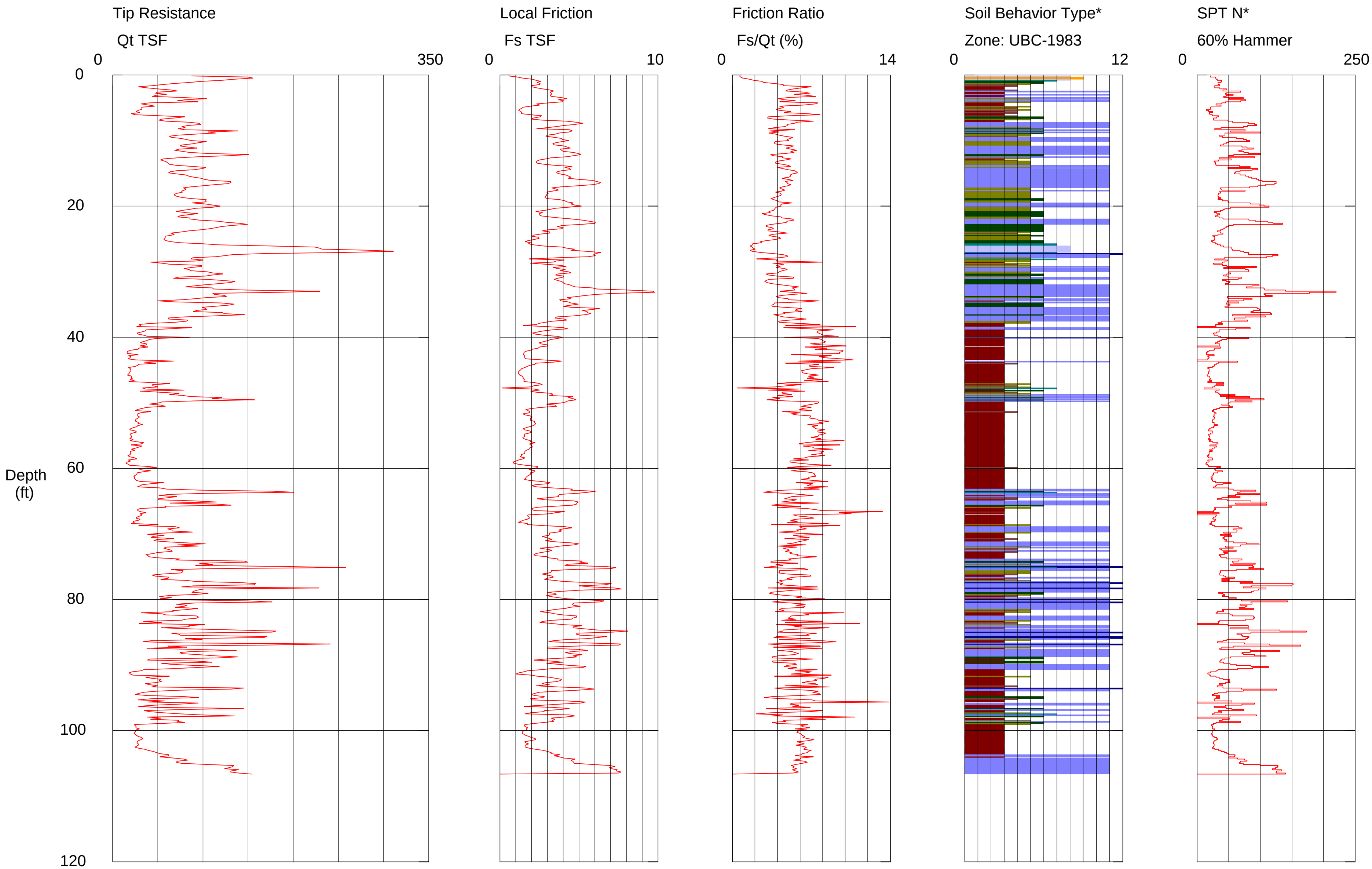
1 sensitive fine grained
2 organic material
3 clay
4 silty clay to clay
5 clayey silt to silty clay
6 sandy silt to clayey silt
7 silty sand to sandy silt
8 sand to silty sand
9 sand
10 gravelly sand to sand
11 very stiff fine grained (*)
12 sand to clayey sand (*)

*Soil behavior type and SPT based on data from UBC-1983

GeoSolutions

Operator: RA-JC
Sounding: CPT-05c
Cone Used: DSG1104

CPT Date/Time: 1/9/2013 8:25:03 AM
Location: Tajiguas Landfill
Job Number: SB00314-1



Maximum Depth = 106.63 feet

Depth Increment = 0.164 feet

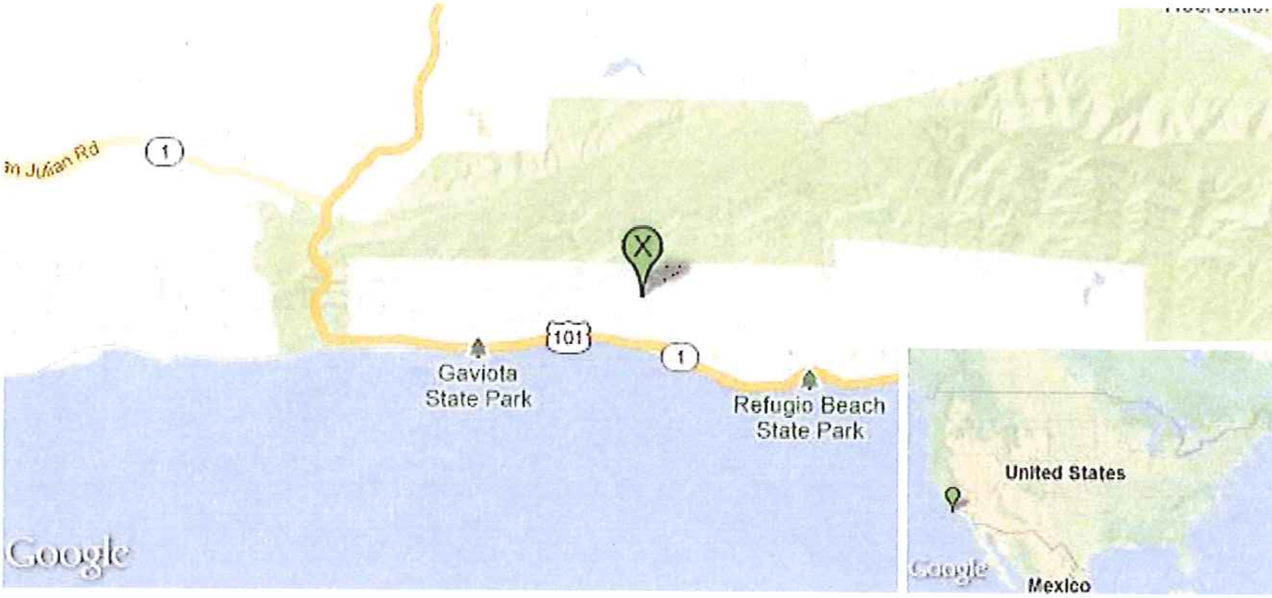
1 sensitive fine grained	4 silty clay to clay	7 silty sand to sandy silt	10 gravelly sand to sand
2 organic material	5 clayey silt to silty clay	8 sand to silty sand	11 very stiff fine grained (*)
3 clay	6 sandy silt to clayey silt	9 sand	12 sand to clayey sand (*)

*Soil behavior type and SPT based on data from UBC-1983

USGS Design Maps Summary Report

User–Specified Input

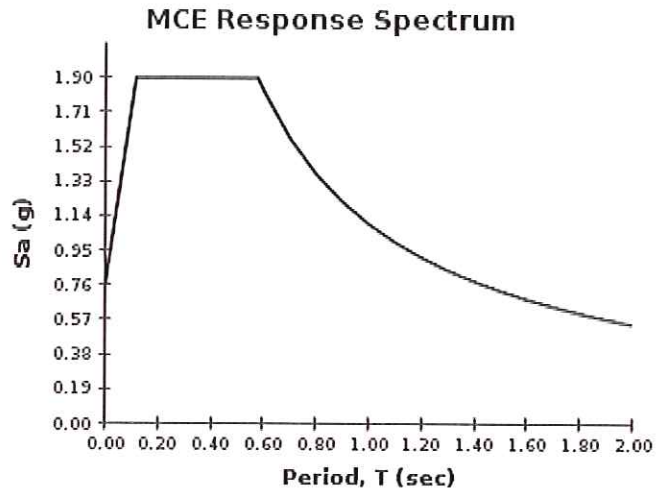
Report Title	Tajiguas Landfill Tue April 9, 2013 21:52:02 UTC
Building Code Reference Document	2006/2009 International Building Code (which makes use of 2002 USGS hazard data)
Site Coordinates	34.4853°N, 120.1248°W
Site Soil Classification	Site Class D – "Stiff Soil"
Occupancy Category	Occupancy Category I

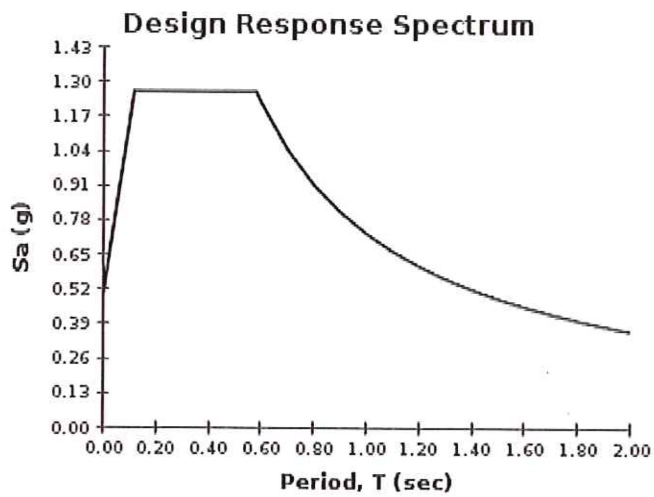


USGS–Provided Output

$S_S =$ 1.896 g $\quad S_{MS} =$ 1.896 g $\quad S_{DS} =$ 1.264 g

$S_1 =$ 0.730 g $\quad S_{M1} =$ 1.095 g $\quad S_{D1} =$ 0.730 g





USGS Design Maps Detailed Report

2006/2009 International Building Code (34.4853°N, 120.1248°W)

Section 1613.5.1 — Mapped acceleration parameters

Note: Maps in the 2006 and 2009 International Building Code are provided for Site Class B. Adjustments for other Site Classes are made, as needed, in Section 1613.5.3.

From Figure 1613.5(1) [1] $S_S = 1.896$ g

From Figure 1613.5(2) [2] $S_1 = 0.730$ g

Section 1613.5.2 — Site class definitions

SITE CLASS	SOIL PROFILE NAME	Soil shear wave velocity, $\overline{v}_S$, (ft/s)	Standard penetration resistance, $\overline{N}$	Soil undrained shear strength, $\overline{s}_u$, (psf)
A	Hard rock	$\overline{v}_S > 5{,}000$	N/A	N/A
B	Rock	$2{,}500 < \overline{v}_S \le 5{,}000$	N/A	N/A
C	Very dense soil and soft rock	$1{,}200 < \overline{v}_S \le 2{,}500$	$\overline{N} > 50$	>2,000 psf
D	Stiff soil profile	$600 \le \overline{v}_S < 1{,}200$	$15 \le \overline{N} \le 50$	1,000 to 2,000 psf
E	Stiff soil profile	$\overline{v}_S < 600$	$\overline{N} < 15$	<1,000 psf
E	—	Any profile with more than 10 ft of soil having the characteristics: 1. Plasticity index $PI > 20$, 2. Moisture content $w \ge 40\%$, and 3. Undrained shear strength $\overline{s}_u < 500$ psf		
F	—	Any profile containing soils having one or more of the following characteristics: 1. Soils vulnerable to potential failure or collapse under seismic loading such as liquefiable soils, quick and highly sensitive clays, collapsible weakly cemented soils. 2. Peats and/or highly organic clays ($H > 10$ feet of peat and/or highly organic clay where H = thickness of soil) 3. Very high plasticity clays ($H > 25$ feet with plasticity index $PI > 75$) 4. Very thick soft/medium stiff clays ($H > 120$ feet)		

For SI: 1ft/s = 0.3048 m/s 1lb/ft² = 0.0479 kN/m²

Section 1613.5.3 — Site coefficients and adjusted maximum considered earthquake spectral response acceleration parameters

TABLE 1613.5.3(1)
VALUES OF SITE COEFFICIENT F_a

Site Class	Mapped Spectral Response Acceleration at Short Period				
	$S_S \le 0.25$	$S_S = 0.50$	$S_S = 0.75$	$S_S = 1.00$	$S_S \ge 1.25$
A	0.8	0.8	0.8	0.8	0.8
B	1.0	1.0	1.0	1.0	1.0
C	1.2	1.2	1.1	1.0	1.0
D	1.6	1.4	1.2	1.1	1.0
E	2.5	1.7	1.2	0.9	0.9
F	See Section 11.4.7 of ASCE 7				

Note: Use straight-line interpolation for intermediate values of S_S

For Site Class = D and S_s = 1.896 g, F_a = 1.000

TABLE 1613.5.3(2)
VALUES OF SITE COEFFICIENT F_v

Site Class	Mapped Spectral Response Acceleration at 1-s Period				
	$S_1 \le 0.10$	$S_1 = 0.20$	$S_1 = 0.30$	$S_1 = 0.40$	$S_1 \ge 0.50$
A	0.8	0.8	0.8	0.8	0.8
B	1.0	1.0	1.0	1.0	1.0
C	1.7	1.6	1.5	1.4	1.3
D	2.4	2.0	1.8	1.6	1.5
E	3.5	3.2	2.8	2.4	2.4
F	See Section 11.4.7 of ASCE 7				

Note: Use straight-line interpolation for intermediate values of S_1

For Site Class = D and S_1 = 0.730 g, F_v = 1.500

In the equations below, the equation number corresponding to the 2006 edition is listed first, and that corresponding to the 2009 edition is listed second.

Equation (16-37; 16-36): $S_{MS} = F_aS_S = 1.000 \times 1.896 = 1.896$ g

---

Equation (16-38; 16-37): $S_{M1} = F_vS_1 = 1.500 \times 0.730 = 1.095$ g

---

Section 1613.5.4 — Design spectral response acceleration parameters

Equation (16-39; 16-38): $S_{DS} = ⅔\ S_{MS} = ⅔ \times 1.896 = 1.264$ g

---

Equation (16-40; 16-39): $S_{D1} = ⅔\ S_{M1} = ⅔ \times 1.095 = 0.730$ g

---

Section 1613.5.6 — Determination of seismic design category

TABLE 1613.5.6(1)
SEISMIC DESIGN CATEGORY BASED ON SHORT-PERIOD RESPONSE ACCELERATION

VALUE OF S_{DS}	OCCUPANCY CATEGORY		
	I or II	III	IV
$S_{DS} < 0.167g$	A	A	A
$0.167g \le S_{DS} < 0.33g$	B	B	C
$0.33g \le S_{DS} < 0.50g$	C	C	D
$0.50g \le S_{DS}$	D	D	D

For Occupancy Category = I and S_{DS} = 1.264 g, Seismic Design Category = D

TABLE 1613.5.6(2)
SEISMIC DESIGN CATEGORY BASED ON 1-SECOND PERIOD RESPONSE ACCELERATION

VALUE OF S_{D1}	OCCUPANCY CATEGORY		
	I or II	III	IV
$S_{D1} < 0.067g$	A	A	A
$0.067g \le S_{D1} < 0.133g$	B	B	C
$0.133g \le S_{D1} < 0.20g$	C	C	D
$0.20g \le S_{D1}$	D	D	D

For Occupancy Category = I and S_{D1} = 0.730 g, Seismic Design Category = D

Note: When S_1 is greater than or equal to 0.75g, the Seismic Design Category is **E** for buildings in Occupancy Categories I, II, and III, and **F** for those in Occupancy Category IV, irrespective of the above.

Seismic Design Category ≡ "the more severe design category in accordance with Table 1613.5.6(1) or 1613.5.6(2)" = D

Note: See Section 1613.5.6.1 for alternative approaches to calculating Seismic Design Category.

References

1. *Figure 1613.5(1)*: http://earthquake.usgs.gov/hazards/designmaps/downloads/pdfs/IBC-2006-Figure1613_5(01).pdf
2. *Figure 1613.5(2)*: http://earthquake.usgs.gov/hazards/designmaps/downloads/pdfs/IBC-2006-Figure1613_5(02).pdf

APPENDIX B

Laboratory Testing

Soil Test Reports

LABORATORY TESTING

This appendix includes a discussion of the test procedures and the laboratory test results performed as part of this investigation. The purpose of the laboratory testing is to assess the engineering properties of the soil materials at the Site. The laboratory tests are performed using the currently accepted test methods, when applicable, of the American Society for Testing and Materials (ASTM).

Undisturbed and disturbed bulk samples used in the laboratory tests are obtained from various locations during the course of the field exploration, as discussed in **Appendix A** of this report. Each sample is identified by sample letter and depth. The Unified Soils Classification System is used to classify soils according to their engineering properties. The various laboratory tests performed are described below:

Expansion Index of Soils (ASTM D4829-03) is conducted in accordance with the ASTM test method and the California Building Code Standard, and are performed on representative bulk and undisturbed soil samples. The purpose of this test is to evaluate expansion potential of the site soils due to fluctuations in moisture content. The sample specimens are placed in a consolidometer, surcharged under a 144-psf vertical confining pressure, and then inundated with water. The amount of expansion is recorded over a 24-hour period with a dial indicator. The expansion index is calculated by determining the difference between final and initial height of the specimen divided by the initial height.

Laboratory Compaction Characteristics of Soil Using Modified Effort (ASTM D1557-07) is performed to determine the relationship between the moisture content and density of soils and soil-aggregate mixtures when compacted in a standard size mold with a 10-lbf hammer from a height of 18 inches. The test is performed on a representative bulk sample of bearing soil near the estimated footing depth. The procedure is repeated on the same soil sample at various moisture contents sufficient to establish a relationship between the maximum dry unit weight and the optimum water content for the soil. The data, when plotted, represents a curvilinear relationship known as the moisture density relations curve. The values of optimum water content and modified maximum dry unit weight can be determined from the plotted curve.

Direct Shear Tests of Soils Under Consolidated Drained Conditions (ASTM D3080-04) is performed on undisturbed and remolded samples representative of the foundation material. The samples are loaded with a predetermined normal stress and submerged in water until saturation is achieved. The samples are then sheared horizontally at a controlled strain rate allowing partial drainage. The shear stress on the sample is recorded at regular strain intervals. This test determines the resistance to deformation, which is shear strength, inter-particle attraction or cohesion c, and resistance to interparticle slip called the angle of internal friction ϕ.

Particle Size Analysis of Soils (ASTM D422-63R02) is used to determine the particle-size distribution of fine and coarse aggregates. In the test method the sample is separated through a series of sieves of progressively smaller openings for determination of particle size distribution. The total percentage passing each sieve is reported and used to determine the distribution of fine and coarse aggregates in the sample.

GeoSolutions, Inc.	SOILS REPORT	(805) 543-8539

Project:	Tajiguas Landfill			Date Tested:	January 10, 2013
Client:				Project #:	SB00314-1
Sample:	A	Depth:	2.0 Feet	Lab #:	286
Location:	B-4			Sample Date:	January 9, 2013
				Sampled By:	JAP

Soil Classification
ASTM D2487-06, D2488-06

Result: Dark Yellowish Brown Clayey SAND

Specification: SC

Sieve Analysis
ASTM D422-63R02

Sieve Size	Percent Passing	Project Specifications
3"		
2"		
1 1/2"		
1"		
3/4"		
No. 4		
No. 8		
No. 16		
No. 30		
No. 50		
No. 100		
No. 200		

Sand Equivalent Cal 217 (06/2011)

		SE
1		
2		
3		
4		

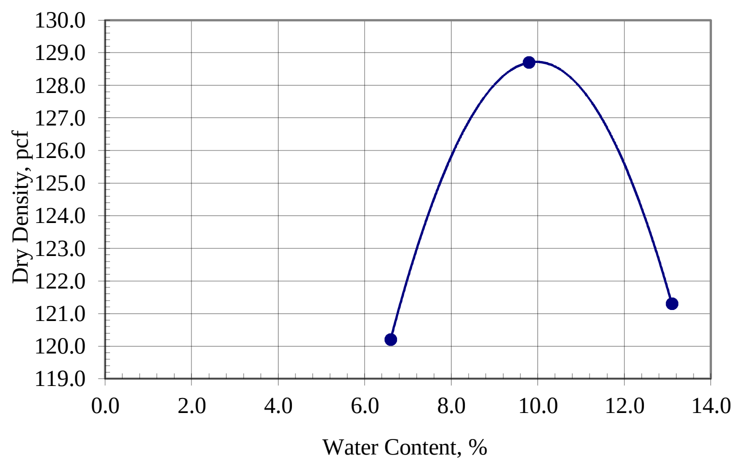
Plasticity Index
ASTM D4318-05

Liquid Limit:	35
Plastic Limit:	11
Plasticity Index:	**24**

Expansion Index
ASTM D4829-08

Expansion Index:	**67**
Expansion Potential:	Medium
Initial Saturation, %:	50

Laboratory Maximum Density
ASTM D1557-07



Mold ID	n/a	Mold Diameter, ins.	4.00
No. of Layers	5	Weight of Rammer, lbs.	10.00
No. of Blows	25		

Estimated Specific Gravity for 100% Saturation Curve = 2.7

Trial #	1	2	3	4
Water Content:	6.6	9.8	13.1	
Dry Density:	120.2	128.7	121.3	
Maximum Dry Density, pcf:		**128.8**		
Optimum Water Content, %:		**10.0**		

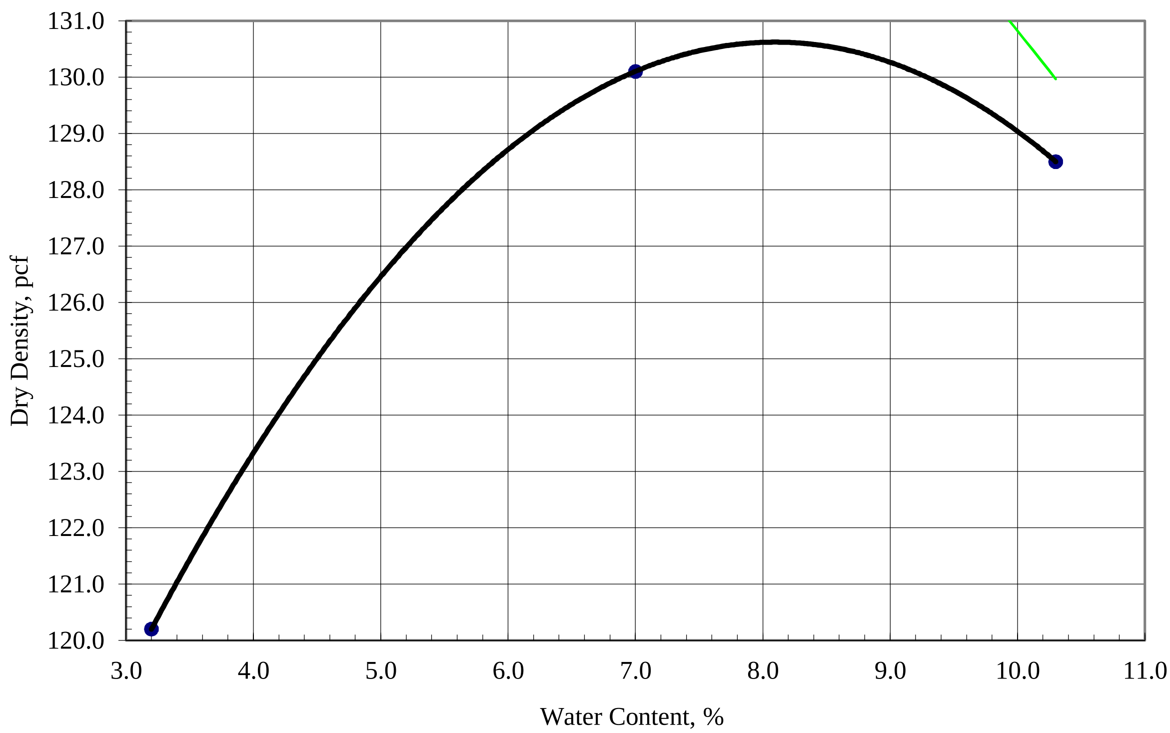
Moisture-Density ASTM D2937-04, **Moisture Content** ASTM D2216-05

Sample	Depth (ft)	Water Content (%)	Dry Density (pcf)	Relative Density	Sample Description

Report By: Aaron Eichman

GeoSolutions, Inc.	LABORATORY COMPACTION REPORT ASTM D1557-07	(805) 543-8539

Project:	Tajiguas Landfill			Date Tested:	January 24, 2013
Client:				Project #:	SB00314-1
Sample #:	B	Depth:	6.0 Feet	Lab #:	286
Source:	B-3			Sample Date:	January 9, 2013
Material:	Dark Yellowish Brown Clayey SAND			Sampled By:	JAP



ASTM Test Designation: [ ] D 698 [x] D 1557 Method: [ ] A [x] B [ ] C

100 % Saturation Curve-Estimated Specific Gravity: 2.65

Laboratory Test Results

Trial #	1	2	3	4
Water Content,%	3.2	7.0	10.3	
Dry Density, pcf	120.2	130.1	128.5	

MAXIMUM DRY DENSITY, pcf:	130.6	OPTIMUM MOISTURE, %:	8.1

Report By: Aaron Eichman

GeoSolutions, Inc.	SOILS REPORT	(805) 543-8539

Project:	Tajiguas Landfill			Date Tested:	January 24, 2013
Client:				Project #:	SB00314-1
Sample:	C	Depth:	36.0 Feet	Lab #:	286
Location:	B-4			Sample Date:	January 9, 2013
				Sampled By:	JAP

Soil Classification
ASTM D2487-06, D2488-06

Result:	Light Gray CLAYSTONE
Specification:	CH

Sieve Analysis
ASTM D422-63R02

Sieve Size	Percent Passing	Project Specifications
3"		
2"		
1 1/2"		
1"		
3/4"		
No. 4		
No. 8		
No. 16		
No. 30		
No. 50		
No. 100		
No. 200		

Sand Equivalent Cal 217 (06/2011)

		SE
1		
2		
3		
4		

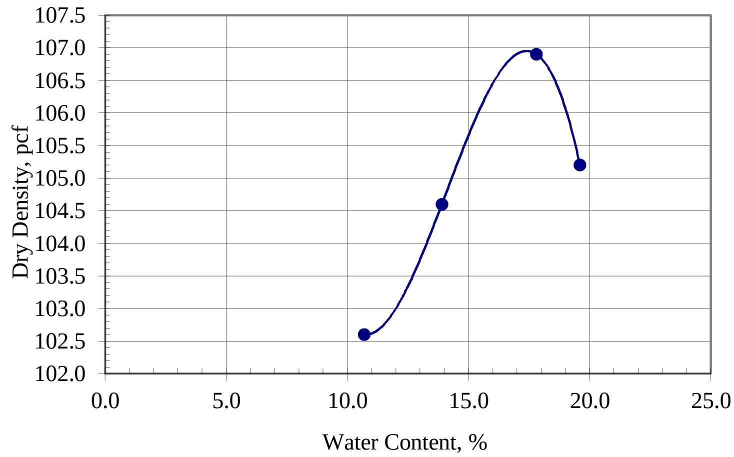
Plasticity Index
ASTM D4318-05

Liquid Limit:	53
Plastic Limit:	27
Plasticity Index:	**26**

Expansion Index
ASTM D4829-08

Expansion Index:	**88**
Expansion Potential:	Medium
Initial Saturation, %:	50

Laboratory Maximum Density
ASTM D1557-07



Mold ID	n/a	Mold Diameter, ins.	4.00
No. of Layers	5	Weight of Rammer, lbs.	10.00
No. of Blows	25		

Estimated Specific Gravity for 100% Saturation Curve = 2.55

Trial #	1	2	3	4
Water Content:	10.7	13.9	17.8	19.6
Dry Density:	102.6	104.6	106.9	105.2

Maximum Dry Density, pcf:	**107.0**
Optimum Water Content, %:	**17.4**

Moisture-Density ASTM D2937-04, **Moisture Content** ASTM D2216-05

Sample	Depth (ft)	Water Content (%)	Dry Density (pcf)	Relative Density	Sample Description

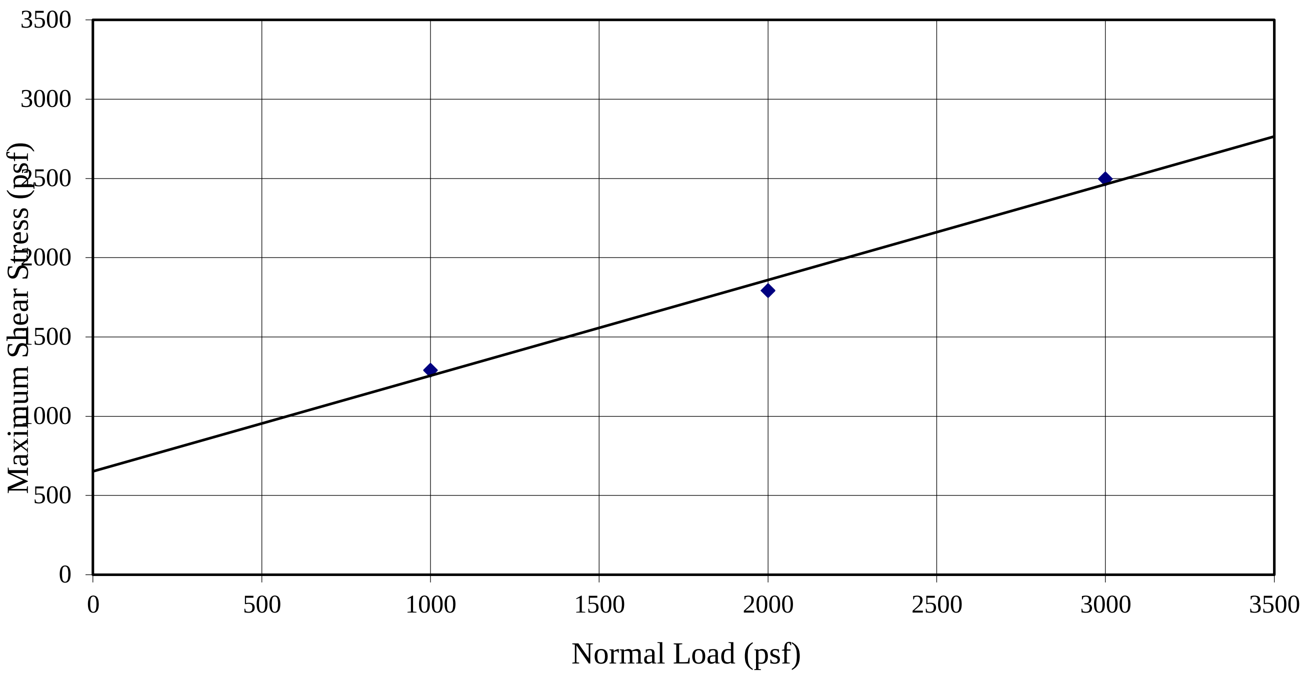
Report By: Aaron Eichman

GeoSolutions, Inc.	DIRECT SHEAR TEST REPORT ASTM D3080-04	(805) 543-8539

Project:	Tajiguas Landfill			Date Tested:	January 16, 2013
Client:				Project #:	SB00314-1
Sample #:	B-3 @ 4'	Depth:	4.0 Feet	Lab #:	286
Location:	B-3			Sample Date:	January 9, 2013
Material:	Dark Olive Brown Sandy CLAY			Sampled By:	JAP

Test Data

Specimen Number	Void Ratio	Saturation, %	Normal Load, psf	Max Shear Stress, psf	Water Content, %	Dry Density, pcf	Relative Density*,%
1	-	-	1000	1288	18.9	110.4	-
2	-	-	2000	1792	18.7	114.6	-
3	-	-	3000	2495	17.1	111.9	-
4							
5							



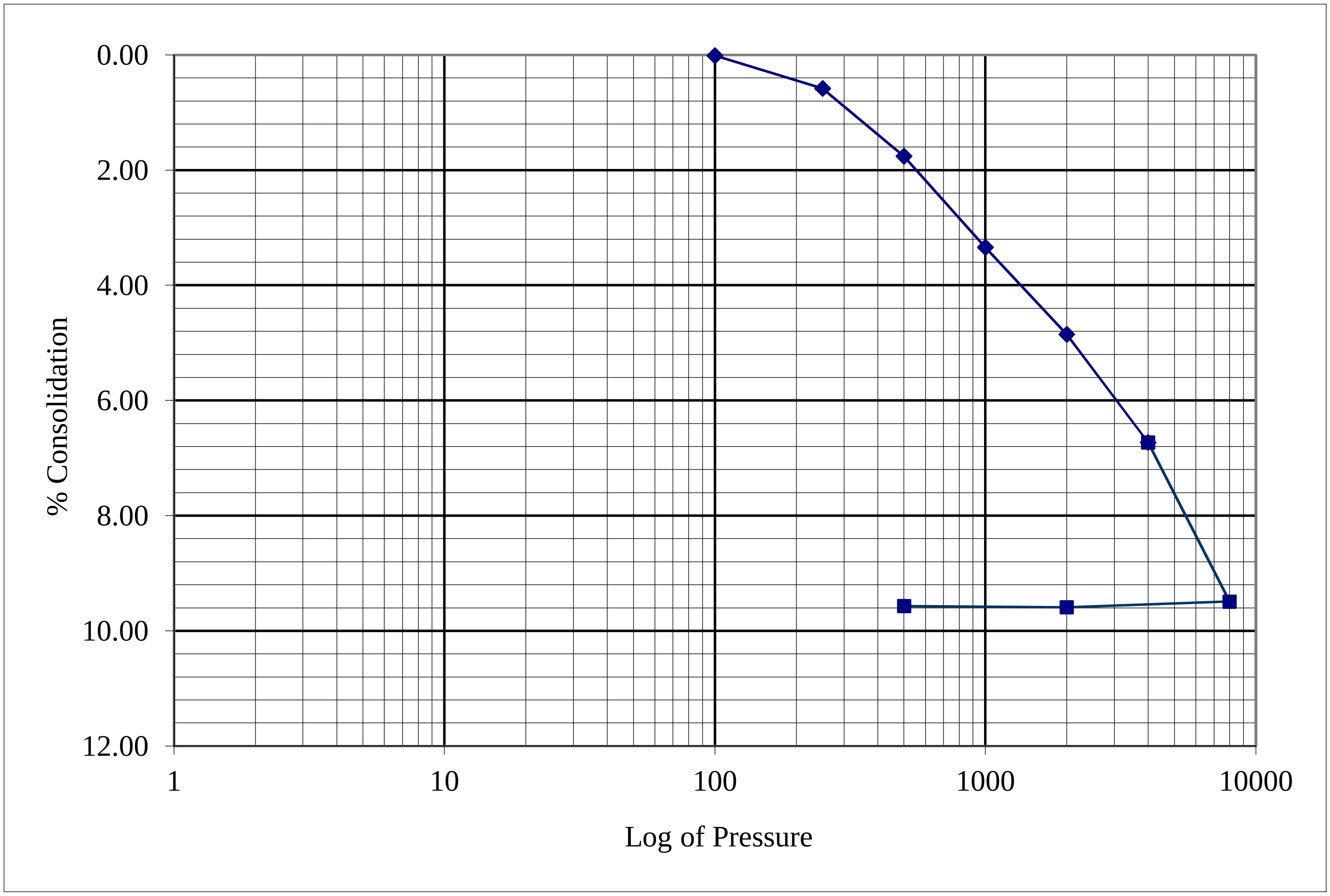
The test specimens were in-situ samples.

Angle of Internal Friction (In-Situ), Phi:	31.1 °
Cohesion (In-Situ), C:	651 psf

Report By: Aaron Eichman

GeoSolutions, Inc.	CONSOLIDATION REPORT D2435-04	(805) 543-8539

Project:	Tajiguas Landfill			Date Tested:	1/29/2013
Client:				Project #:	SB00314-1
Sample:	B-4 @ 4'	Depth:	4.0 Feet	Lab #:	286
Location:	B-4			Sample Date:	1/9/2013
Material:	Very Dark Brown Sandy CLAY			Sampled By:	JAP



Applied Pressure (psf)	% Consolidation
100	0.01
250	0.58
500	1.76
1000	3.34
2000	4.85
4000	6.73
8000	9.49
2000	9.59
500	9.57

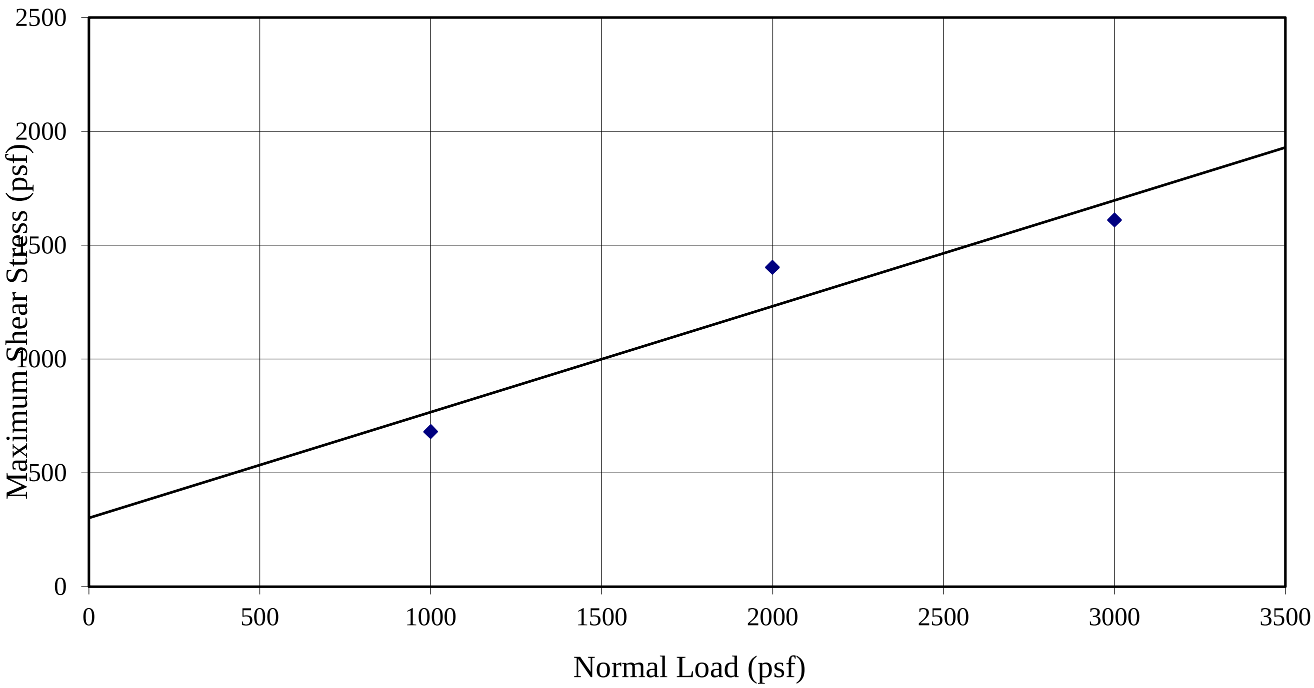
Report By: Aaron Eichman

GeoSolutions, Inc.	DIRECT SHEAR TEST REPORT D3080-04	(805) 543-8539

Project:	Tajiguas Landfill		Date Tested:	April 4, 2013
Client:			Project #:	SB00314-1
Sample #:	C	Depth:	Lab #:	297
Location:			Sample Date:	March 29, 2013
Material:	Very Dark Brown Sandy CLAY		Sampled By:	JAP

Test Data

Specimen Number	Void Ratio	Saturation, %	Normal Load, psf	Max Shear Stress, psf	Water Content, %	Dry Density, pcf	Relative Density*,%
1	-	-	1000	680	27.8	94.8	90
2	-	-	2000	1403	25.1	94.8	90
3	-	-	3000	1610	25.4	94.8	90
4							
5							



*The test specimens were initially remolded at 90% of the maximum dry density (ASTM D1557) and at 2% above the optimum moisture content of the material.

Maximum Dry Density, pcf:	107.0	Optimum Moisture, %:	17.4

Angle of Internal Friction @ 90% Rel. Compaction, Phi:	25 °
Cohesion @ 90% Relative Compaction, C:	301 psf

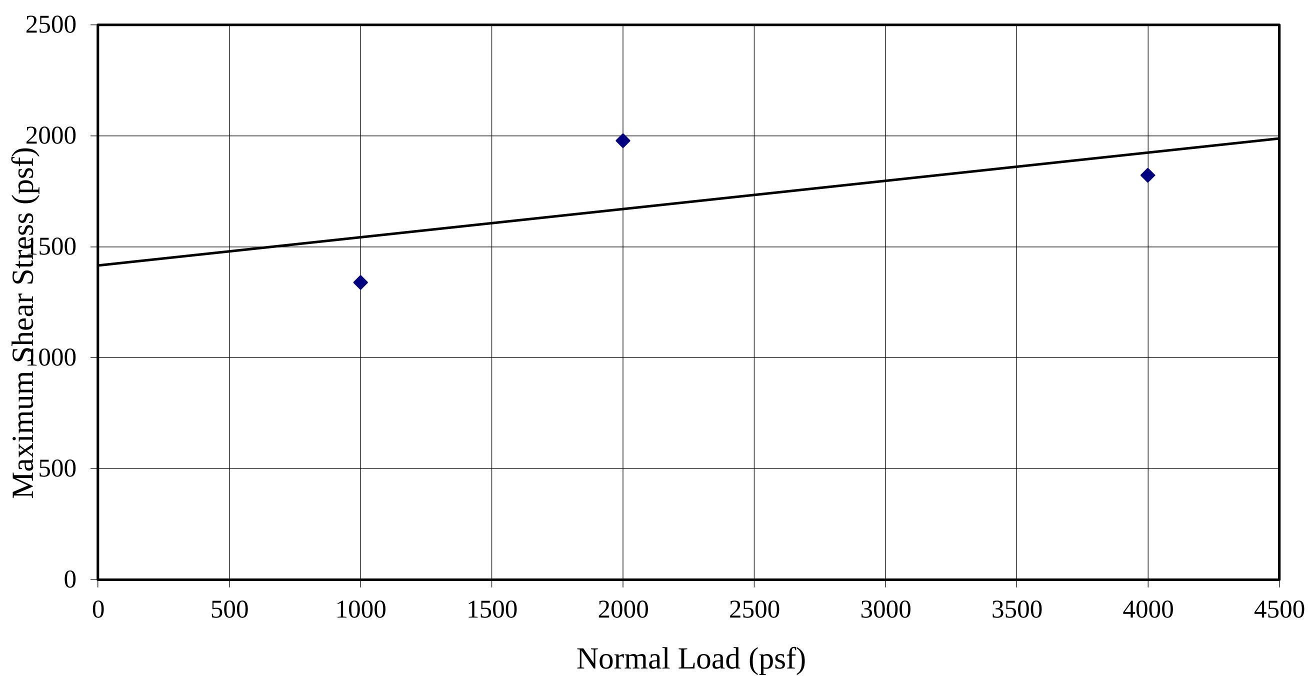
Report By: Aaron Eichman

GeoSolutions, Inc.	DIRECT SHEAR TEST REPORT ASTM D3080-04	(805) 543-8539

Project:	Tajiguas Landfill			Date Tested:	February 6, 2013
Client:				Project #:	SB00314-1
Sample #:	B-6 @ 3'	Depth:	3.0 Feet	Lab #:	286
Location:	B-6			Sample Date:	February 1, 2013
Material:	Gray CLAYSTONE			Sampled By:	JAP

Test Data

Specimen Number	Void Ratio	Saturation, %	Normal Load, psf	Max Shear Stress, psf	Water Content, %	Dry Density, pcf	Relative Density*,%
1	-	-	1000	1339	27.8	102.9	-
2	-	-	2000	1977	26.0	107.3	-
3	-	-	4000	1822	25.2	106.9	-
4							
5							



The test specimens were in-situ samples.

Angle of Internal Friction (In-Situ), Phi:	7 °
Cohesion (In-Situ), C:	1417 psf

Report By: Aaron Eichman

APPENDIX C

Settlement Analysis Data – Operations Deck

Tajiguas Landfill - Operations Deck

Coeffecient of Secondary Compression - Backcalculated from Existing Settlement Data

Point	**1**	2	3	4	5	**6**	**7**	8	**9**	**10**	**11**
Hs	1.99	0.31	0.18	0.14	0.35	1.8	1.41	0.24	1.63	1	2.33
H1	95	84	35	35	80	95	105	78	120	120	120
t2 (yrs)	5	5	5	5	5	5	5	5	4	4	4
t1	0	0	0	0	0	0	0	0	0	0	0
Ca	0.03	0.01	0.01	0.01	0.01	0.03	0.02	0.004	0.02	0.01	0.03

Total Settlement by 2036 (end of project)

Ca	0.03	0.01	0.01	0.01	0.01	0.03	0.02	0.00	0.02	0.01	0.03
H1	95	84	35	35	80	95	105	78	120	120	120
t2	276	276	276	276	276	276	276	276	276	276	276
t1	4	4	4	4	4	4	4	4	4	4	4
Hs-2036	**5.24**	0.82	0.47	0.37	0.92	**4.74**	**3.71**	0.63	**4.98**	**3.05**	**7.12**

Total Settlement by 2017 (start of project)

Ca	0.03	0.01	0.01	0.01	0.01	0.03	0.02	0.00	0.02	0.01	0.03
H1	95	84	35	35	80	95	105	78	120	120	120
t2	48	48	48	48	48	48	48	48	48	48	48
t1	4	4	4	4	4	4	4	4	4	4	4
Hs-2015	**3.07**	0.48	0.28	0.22	0.54	**2.78**	**2.18**	0.37	**2.92**	**1.79**	**4.18**

Settlement Anticipated During the Life of the Project (2017-2036)

Hs-2036	**5.24**	0.82	0.47	0.37	0.92	**4.74**	**3.71**	0.63	**4.98**	**3.05**	**7.12**
Hs-2015	**3.07**	0.48	0.28	0.22	0.54	**2.78**	**2.18**	0.37	**2.92**	**1.79**	**4.18**
Hs-Project	**2.16**	0.34	0.20	0.15	0.38	**1.96**	**1.53**	0.26	**2.06**	**1.26**	**2.94**

Fill	0.21-0.53'
Refuse	**1.26-2.94'**

$\Delta H_S = C_{\alpha(EL)} H_1 \log t_2/t_1$

Equation for Settlement under External Loads (Sharma and De, 2007)

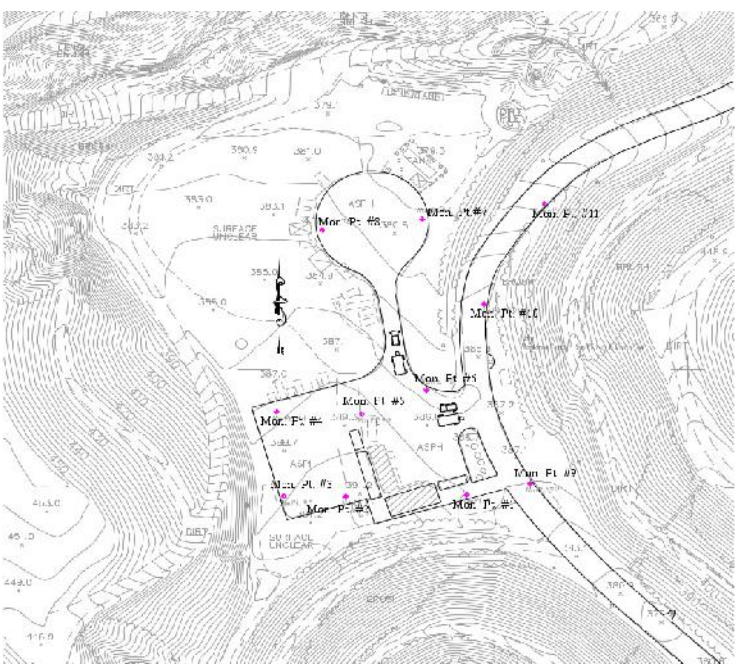
$C_{\alpha(EL)}$= coefficient of secondary compression due to external loads

H_1= thickness of refuse at the end of the primary settlement (feet)

t_2= time of interest (months)

t_1= time for primary settlement (months)

Map provided by the County of Santa Barbara



APPENDIX D

Preliminary Grading Specifications

Key and Bench with Backdrain

PRELIMINARY GRADING SPECIFICATIONS

A. General

i. These preliminary specifications have been prepared for the subject site; GeoSolutions, Inc. should be consulted prior to the commencement of site work associated with site development to ensure compliance with these specifications.

ii. GeoSolutions, Inc. should be notified at least 72 hours prior to site clearing or grading operations on the property in order to observe the stripping of surface materials and to coordinate the work with the grading contractor in the field.

iii. These grading specifications may be modified and/or superseded by recommendations contained in the text of this report and/or subsequent reports.

iv. If disputes arise out of the interpretation of these grading specifications, the Soils Engineer shall provide the governing interpretation.

B. Obligation of Parties

i. The Soils Engineer should provide observation and testing services and should make evaluations to advise the client on geotechnical matters. The Soils Engineer should report the findings and recommendations to the client or the authorized representative.

ii. The client should be chiefly responsible for all aspects of the project. The client or authorized representative has the responsibility of reviewing the findings and recommendations of the Soils Engineer. During grading the client or the authorized representative should remain on-site or should remain reasonably accessible to all concerned parties in order to make decisions necessary to maintain the flow of the project.

iii. The contractor is responsible for the safety of the project and satisfactory completion of all grading and other operations on construction projects, including, but not limited to, earthwork in accordance with project plans, specifications, and controlling agency requirements.

C. Site Preparation

i. The client, prior to any site preparation or grading, should arrange and attend a meeting which includes the grading contractor, the design Structural Engineer, the Soils Engineer, representatives of the local building department, as well as any other concerned parties. All parties should be given at least 72 hours notice.

ii. All surface and sub-surface deleterious materials should be removed from the proposed building and pavement areas and disposed of off-site or as approved by the Soils Engineer. This includes, but is not limited to, any debris, organic materials, construction spoils, buried utility line, septic systems, building materials, and any other surface and subsurface structures within the proposed building areas. Trees designated for removal on the construction plans should be removed and their primary root systems grubbed under the observations of a representative of GeoSolutions, Inc. Voids left from site clearing should be cleaned and backfilled as recommended for structural fill.

iii. Once the Site has been cleared, the exposed ground surface should be stripped to remove surface vegetation and organic soil. A representative of GeoSolutions, Inc. should determine the required depth of stripping at the time of work being completed. Strippings may either be disposed of off-site or stockpiled for future use in landscape areas, if approved by the landscape architect.

D. Site Protection

i. Protection of the Site during the period of grading and construction should be the responsibility of the contractor.

ii. The contractor should be responsible for the stability of all temporary excavations.

iii. During periods of rainfall, plastic sheeting should be kept reasonably accessible to prevent unprotected slopes from becoming saturated. Where necessary during periods of rainfall, the contractor should install check-dams, de-silting basins, sand bags, or other devices or methods necessary to control erosion and provide safe conditions.

E. Excavations

i. Materials that are unsuitable should be excavated under the observation and recommendations of the Soils Engineer. Unsuitable materials include, but may not be limited to: 1) dry, loose, soft, wet, organic, or compressible natural soils; 2) fractured, weathered, or soft bedrock; 3) non-engineered fill; 4) other deleterious materials; and 5) materials identified by the Soils Engineer or Engineering Geologist.

ii. Unless otherwise recommended by the Soils Engineer and approved by the local building official, permanent cut slopes should not be steeper than 2:1 (horizontal to vertical). Final slope configurations should conform to section 1804 of the 2010 California Building Code unless specifically modified by the Soil Engineer/Engineering Geologist.

iii. The Soil Engineer/Engineer Geologist should review cut slopes during excavations. The contractor should notify the Soils Engineer/Engineer Geologist prior to beginning slope excavations.

F. Structural Fill

i. Structural fill should not contain rocks larger than 3 inches in greatest dimension, and should have no more than 15 percent larger than 2.5 inches in greatest dimension.

ii. Imported fill should be free of organic and other deleterious material and should have very low expansion potential, with a plasticity index of 12 or less. Before delivery to the Site, a sample of the proposed import should be tested in our laboratory to determine its suitability for use as structural fill.

G. Compacted Fill

i. Structural fill using approved import or native should be placed in horizontal layers, each approximately 8 inches in thickness before compaction. On-site inorganic soil or approved imported fill should be conditioned with water to produce a soil water content near optimum moisture and compacted to a minimum relative density of 90 percent based on ASTM D1557-07.

ii. Fill slopes should not be constructed at gradients greater than 2-to-1 (horizontal to vertical). The contractor should notify the Soils Engineer/Engineer Geologist prior to beginning slope excavations.

iii. If fill areas are constructed on slopes greater than 10-to-1 (horizontal to vertical), we recommend that benches be cut every 4 feet as fill is placed. Each bench shall be a minimum of 10 feet wide with a minimum of 2 percent gradient into the slope.

iv. If fill areas are constructed on slopes greater than 5-to-1, we recommend that the toe of all areas to receive fill be keyed a minimum of 24 inches into underlying dense material. Key depths are to be observed and approved by a representative of GeoSolutions, Inc. Sub-drains shall be placed in the keyway and benches as required.

H. Drainage

i. During grading, a representative of GeoSolutions, Inc. should evaluate the need for a sub-drain or back-drain system. Areas of observed seepage should be provided with sub-surface drains to release the hydrostatic pressures. Sub-surface drainage facilities may include gravel blankets, rock filled trenches or Multi-Flow systems or equal. The drain system should discharge in a non-erosive manner into an approved drainage area.

ii. All final grades should be provided with a positive drainage gradient away from foundations. Final grades should provide for rapid removal of surface water runoff. Ponding of water should not be allowed on building pads or adjacent to foundations. Final grading should be the responsibility of the contractor, general Civil Engineer, or architect.

iii. Concentrated surface water runoff within or immediately adjacent to the Site should be conveyed in pipes or in lined channels to discharge areas that are relatively level or that are adequately protected against erosion.

iv. Water from roof downspouts should be conveyed in solid pipes that discharge in controlled drainage localities. Surface drainage gradients should be planned to prevent ponding and promote drainage of surface water away from building foundations, edges of pavements and sidewalks. For soil areas we recommend that a minimum of 2 percent gradient be maintained.

v. Attention should be paid by the contractor to erosion protection of soil surfaces adjacent to the edges of roads, curbs and sidewalks, and in other areas where hard edges of structures may cause concentrated flow of surface water runoff. Erosion resistant matting such as Miramat, or other similar products, may be considered for lining drainage channels.

vi. Sub-drains should be placed in established drainage courses and potential seepage areas. The location of sub-drains should be determined after a review of the grading plan. The sub-drain outlets should extend into suitable facilities or connect to the proposed storm drain system or existing drainage control facilities. The outlet pipe should consist of a non-perforated pipe the same diameter as the perforated pipe.

I. Maintenance

i. Maintenance of slopes is important to their long-term performance. Precautions that can be taken include planting with appropriate drought-resistant vegetation as recommended by a landscape architect, and not over-irrigating, a primary source of surficial failures.

ii. Property owners should be made aware that over-watering of slopes is detrimental to long term stability of slopes.

J. Underground Facilities Construction

i. The attention of contractors, particularly the underground contractors, should be drawn to the State of California Construction Safety Orders for "Excavations, Trenches, Earthwork." Trenches or excavations greater than 5 feet in depth should be shored or sloped back in accordance with OSHA Regulations prior to entry.

ii. Bedding is defined as material placed in a trench up to 1 foot above a utility pipe and backfill is all material placed in the trench above the bedding. Unless concrete bedding is required around utility pipes, free-draining sand should be used as bedding. Sand to be used as bedding should be tested in our laboratory to verify its suitability and to measure its compaction characteristics. Sand bedding should be compacted by mechanical means to achieve at least 90 percent relative density based on ASTM D1557-07.

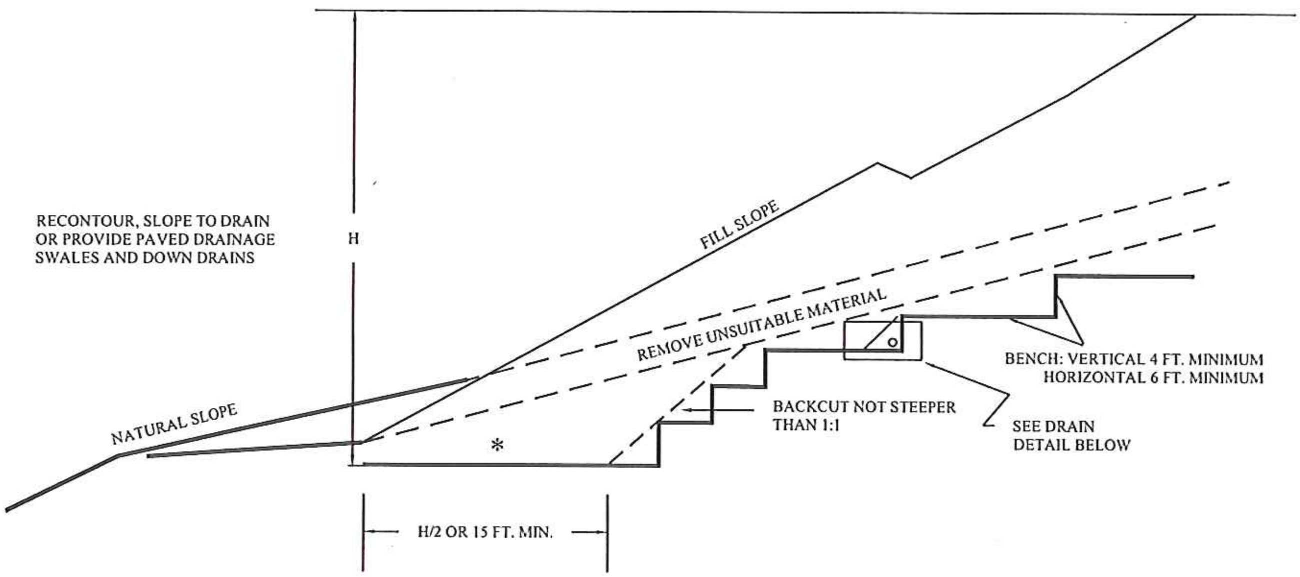
iii. On-site inorganic soils, or approved import, may be used as utility trench backfill. Proper compaction of trench backfill will be necessary under and adjacent to structural fill, building foundations, concrete slabs, and vehicle pavements. In these areas, backfill should be conditioned with water (or allowed to dry), to produce a soil water content of about 2 to 3 percent above the optimum value and placed in horizontal layers, each not exceeding 8 inches in thickness before compaction. Each layer should be compacted to at least 90 percent relative density based on ASTM D1557-07. The top lift of trench backfill under vehicle pavements should be compacted to the requirements given in report under Preparation of Paved Areas for vehicle pavement sub-grades. Trench walls must be kept moist prior to and during backfill placement.

K. Completion of Work

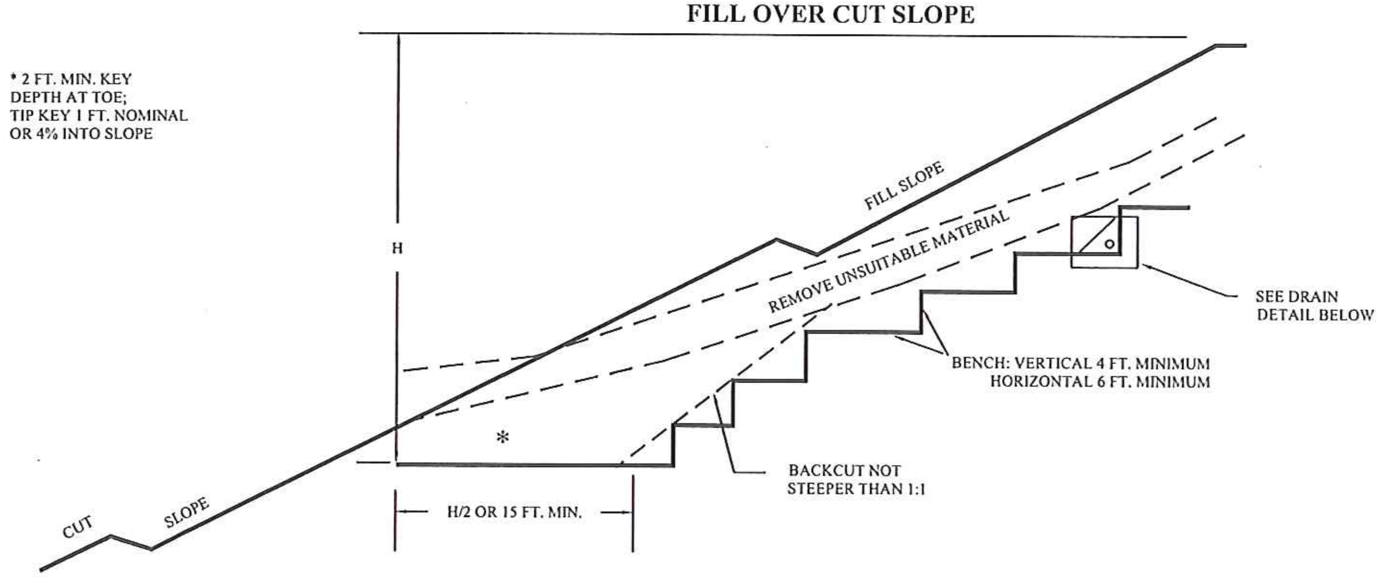
i. After the completion of work, a report should be prepared by the Soils Engineer retained to provide such services in accordance with Section 1804.4 of the 2010 CBC. The report should including locations and elevations of field density tests, summaries of field and laboratory tests, other substantiating data, and comments on any changes made during grading and their effect on the recommendations made in the approved Soils Engineering Report.

ii. Soils Engineers shall submit a statement that, to the best of their knowledge, the work within their area of responsibilities is in accordance with the approved soils engineering report and applicable provisions within Section 1804 of the 2010 CBC.

FILL OVER NATURAL SLOPE



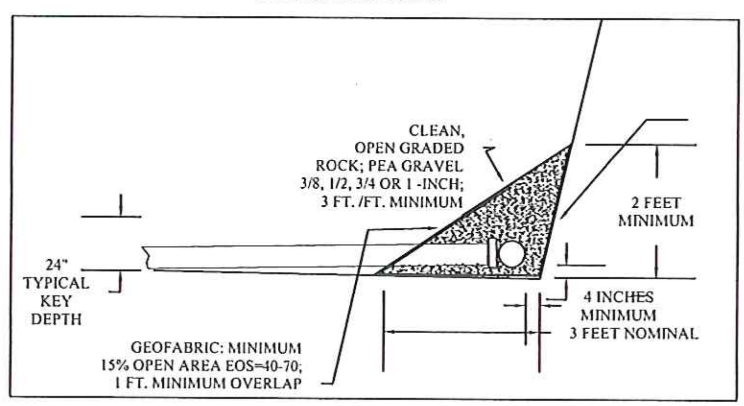
FILL OVER CUT SLOPE



NOTES:
1 - IF OVERFILLING AND CUTTING BACK TO GRADE IS ADOPTED, 15 FT. MIN. FILL WIDTH MAY BE REDUCED TO 12 FT. MIN. IN NO CASE SHOULD THE FILL WIDTH BE LESS THAN 1/2 THE HEIGHT OF FILL REMAINING.

1 - BACKDRAIN AS RECOMMEDNED BY GEOTECHNICAL CONSULTANT PER BUTTRESS BACKDRAIN DETAIL.

DRAIN DETAIL



GeoSolutions, Inc.
220 High Street
San Luis Obispo, CA 93401
(805) 543-8539 Fax: (805) 543-2171

KEY AND BENCH WITH BACKDRAIN

DETAIL
A