GEOTECHNICAL INVESTIGATION, PROPOSED IMPROVEMENTS, SAN ELIJO WATER RECLAMATION FACILITY, ENCINITAS, CALIFORNIA

Prepared For

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Project No. 602855-001

March 24, 2010

Leighton Consulting, Inc.

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Project No. 602835-001

To: Kennedy Jenks Consulting, Inc.
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Attention: Mr. Corey Young, P.E.

Subject: Geotechnical Investigation, Proposed Improvement at San Elijo Water Reclamation Facility, Encinitas, California

In accordance with your request and authorization, we have conducted a geotechnical investigation for the proposed improvements to the San Elijo Water Reclamation Facility in Encinitas, California. Based on the results of our study, it is our opinion that the improvements are feasible from a geotechnical standpoint provided the recommendations provided herein are incorporated into the design and construction of the proposed improvements. The accompanying report presents a summary of our investigation and provides geotechnical conclusions and recommendations relative to the proposed improvements.

If you have any questions regarding our report, please do not hesitate to contact this office. We appreciate this opportunity to be of service.

Respectfully submitted,

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1.0 INTRODUCTION

1.1 Purpose and Scope

This report presents the results of our geotechnical investigation for proposed improvements to the San Elijo Water Reclamation Facility, located in Encinitas, California (Figure 1). The purpose of our investigation was to evaluate the existing geotechnical conditions present at the site and to provide preliminary conclusions and geotechnical recommendations relative to the proposed improvements. Our scope of services for this investigation included:

- Review of available pertinent, published and unpublished geotechnical literature and maps (Appendix A).
- Review of a previous geotechnical investigation for facility improvements, performed by AGRA Earth & Environmental (AGRA, 1995).
- A geotechnical reconnaissance of the site and geologic mapping of site conditions.
- Coordination with Underground Service Alert and San Elijo Joint Powers Authority representatives to locate potential underground utilities on site.
- Obtaining a County of San Diego, Department of Health, Boring Permit.
- Advancement of two cone penetration test (CPT) soundings. The data and interpreted logs from our CPT soundings are presented in Appendix B and their approximate locations are shown on the Exploration Location Map (Figure 2).
- Laboratory testing of representative soil samples obtained from the hand augured exploration at the CPT locations. Results of these tests are presented in Appendix C.
- Compilation and analysis of the geotechnical data obtained from the field investigation and laboratory testing.
- Preparation of this report presenting our geotechnical findings, conclusions, and geotechnical recommendations with respect to the proposed design, site grading, and general construction considerations.

1.2 Site Location and Proposed Improvements

The existing facility is located west of Interstate 5 and north of Manchester Avenue within a relatively narrow tributary canyon. The area of the proposed improvements will be located east of the existing secondary clarifiers. The ground surface elevations of the area range from 32 to 34 feet mean sea level (msl) and is paved with asphalt.

The proposed improvements include construction of a new Advance Water Treatment System which consists of several above ground storage tanks/containers, equipment pads, a chemical storage area, subsurface piping, and an overhead canopy structure. The proposed surface grades of the new improvement appear to be at or near the existing surface elevation or approximately 33 feet msl.

1.3 Previous Investigations

In summary, previous geotechnical studies have been performed for the facility improvements. It appears that the first study was performed by Ninyo & Moore in 1989, and consisted of ten boring of which three boring (B-2, B-8 and B-9) were located in the immediate vicinity of the proposed improvements. The most recent study was performed by AGRA Earth & Environmental in 1995 and consisted of three borings (B-1 through B-3) located immediately south of the subject area. Logs of the applicable borings are presented in Appendix B.

Depth of the borings ranging from 26 to 61.5 feet below the existing ground surface (bgs). The approximate locations of the borings are shown on Figure 2, and the results of this study have been incorporated into the current investigation. Applicable laboratory test data by AGRA Earth & Environmental (AGRA, 1995) has also been included in Appendix C of this report.

San Elijo Water Reclamation Facilities Cardiff by the Sea, California	**SITE LOCATION MAP**	Project No. **602835-001** Date **March 2010** Figure 1

\\GIS\Administration\ArcGISTemplates\NEW_GDT_SiteLocationMap.mxd

2.0 SUBSURFACE EXPLORATION AND LABORATORY TESTING

Our subsurface exploration of the site consisted of the performing two Cone Penetration Test (CPT) soundings. The approximate locations of the soundings are shown on the Exploration Map, Figure 2. In addition, a hand augured boring at each CPT locations was performed to allow evaluation of the soils encountered within near surface of proposed structures, and provided representative samples for laboratory testing. Prior to performing the explorations, Underground Service Alert and representatives of the San Elijo Joint Powers Authority (SEJPA) were contacted to coordinate location and identification of nearby underground utilities.

The CPTs were performed by Kehoe Testing and Engineering and observed by a representative from our firm. Representative bulk samples were obtained from the hand augured borings at the CPT locations for laboratory tests, which are presented in Appendix B. The boreholes created by the CPTs were backfilled with bentonite grout per County of San Diego, Department of Environmental Health requirements.

Laboratory testing was performed on representative bulk samples to evaluate shear strength, expansion potential, and geo-chemical (corrosion) characteristics of the near surface soils. A discussion of the laboratory tests performed and a summary of the laboratory test results are presented in Appendix C.

3.0 SUMMARY OF GEOTECHNICAL CONDITIONS

3.1 Regional Geology

The subject site is situated in the coastal section of the Peninsular Range Province, a California geomorphic province with a long and active geologic history. Throughout the last 54 million years, this area known as the San Diego Embayment has undergone several episodes of marine indunation and subsequent, marine regression. This has resulted in a thick sequence of marine and nonmarine sediments deposited on rocks on Southern California batholith during minor episodic tectonic uplift of the area.

3.2 Site Geology

Based on our subsurface exploration, and review of pertinent geologic literature and maps, the units underlying the site consist of artificial fill and alluvial soils underlain by the Delmar Formation. A brief description of the geologic units as encountered on-site is presented below.

3.2.1 Artificial Fill

Artificial fill was encountered in the previous subsurface explorations, and was on the order of 3 to 4 feet in depth. The fill soil, consisting of brown, loose to dense silty sands, with a trace of gravel, appears to have been placed during the original construction of adjacent facility improvements. Any loose or desiccated fills encountered during the anticipated future grading operations are considered potentially compressible in their present condition and will require removal and recompaction during site grading.

3.2.2 Quaternary Alluvium

Alluvial material encountered consisted of loose to medium dense, gray to light brown silty sand to poorly graded sand with micaceous layers. Thickness of the alluvium beneath the site extended to a depth of approximately 85 feet below the existing ground surface (bgs) in sounding CPT-1. In general, the upper zone of alluvium (i.e., the upper 30 feet) is considered loose and potentially compressible in its present condition, which is consistent with findings of the previous geotechnical investigations (AGRA, 1995).

3.2.3 Tertiary Delmar Formation

The Tertiary Delmar Formation is the bedrock unit underlying the site (Tan, et. al., 1996) and was encounter at a depth of approximately 85 feet. The formation in this area typically consists of interbedded claystone and silty sandstone that are very dense.

3.3 Surface and Ground Water

No indication of surface water or evidence of surface ponding was encountered during our field investigations. In addition, surface water appears to drain as sheet flow from the higher slopes during rainy periods and accumulate in lower elevations.

Ground water encountered in the previous field explorations was at elevation ranging from 20 to 24 feet msl, which is roughly 9 to 13 feet below the existing ground surface (bgs) (AGRA, 1995). Ground water levels may also fluctuate seasonally and rise during rainy periods.

3.4 Landslides

No landslides or indications of deep-seated landsliding were noted at the site during our field exploration or our review of available geologic literature, topographic maps, and stereoscopic aerial photographs. The potential for significant landslides or large-scale slope instability at the site is considered not applicable.

3.5 Flood Hazard

According to a Federal Emergency Management Agency (FEMA) flood insurance rate map (FEMA, 1997); it appears that the facility is not located within a flood.

3.6 Engineering Characteristics of On-site Soils

Based on the results of our laboratory testing of representative on-site soils, and our professional experience on adjacent sites with similar soils, the engineering characteristics of the on-site soils are discussed below.

3.6.1 Expansion Potential

Based on our laboratory testing of the near the surface on-site soils, the expansion potential of the on-site soil is anticipated to range from low. Geotechnical observations and/or laboratory testing upon completion of the grading are recommended to determine the actual expansion potential of finish grade soils on the site.

3.6.2 Soil Corrosivity

A preliminary corrosive soil screening for the on-site materials was completed to evaluate their potential effect on concrete and ferrous metals. The corrosion potential was evaluated using the results of laboratory testing on one representative soil sample obtained during our subsurface evaluation.

Laboratory testing was performed to evaluate pH, minimum electrical resistivity, and chloride and soluble sulfate content. The sample tested had a measured pH of 7.86, and a measured minimum electrical resistivity of 1,275 ohm-cm. Test results also indicated that the sample had a chloride content of 86 ppm, and a soluble sulfate content of 300 ppm (i.e., 0.03%).

3.6.3 Excavation Characteristics

The site is underlain by fill and alluvium, which can be excavated with conventional heavy-duty construction equipment. If oversize material (typically over 8 inches in maximum dimension) is generated, it should be placed in non-structural areas or hauled off-site.

4.0 FAULTING AND SEISMICITY

4.1 Faulting

Our discussion of faults on the site is prefaced with a discussion of California legislation and state policies concerning the classification and land-use criteria associated with faults. By definition of the California Mining and Geology Board, an active fault is a fault which has had surface displacement within Holocene time (about the last 11,000 years). The State Geologist has defined a potentially active fault as any fault considered to have been active during Quaternary time (last 1,600,000 years) but that has not been proven to be active or inactive. This definition is used in delineating Fault-Rupture Hazard Zones as mandated by the Alquist-Priolo Earthquake Fault Zoning Act of 1972 and as most recently revised in 1997. The intent of this act is to assure that unwise urban development does not occur across the traces of active faults. Based on our review of the Fault-Rupture Hazard Zones, the site is not located within any Fault-Rupture Hazard Zone as created by the Alquist-Priolo Act (Hart, 1997).

San Diego, like the rest of Southern California, is seismically active as a result of being located near the active margin between the North American and Pacific tectonic plates. The principal source of seismic activity is movement along the northwest-trending regional fault zones such as the San Andreas, San Jacinto and Elsinore Faults Zones, as well as along less active faults such as the Newport-Inglewood (Offshore) and Rose Canyon Fault Zones.

Our review of geologic literature pertaining to the site and general vicinity indicates that there are no known major or active faults on or in the immediate vicinity of the site (Jennings, 1994). Evidence for faulting was not encountered during our field investigation. The nearest known active regional faults are the Rose Canyon fault located approximately 3.4 mile west of the site, the Newport Inglewood Fault located offshore 13.4 miles west of the site and the Coronado Bank Fault located 17.3 miles west of the site (Blake, 2000).

4.2 Seismicity

The site can be considered to lie within a seismically active region, as can all of Southern California. Table 1 indicates potential seismic events that could be produced by the maximum moment magnitude earthquake. A maximum moment magnitude earthquake is the maximum expectable earthquake given the known tectonic framework. Site-specific seismic parameters for the site are included in Table 1 are the distances to the causative faults, earthquake magnitudes, and postulated ground accelerations as generated by the deterministic fault modeling software EQFAULT (Blake, 2000).

Table 1 Seismic Parameters for Active Faults (Blake, 2000)					
Potential Causative Fault	Distance from Fault to Site (Miles)	Slip Rate* (mm/yr)	Maximum Moment Magnitude	Peak Horizontal Ground Acceleration (g)	One Standard Deviation (g)
Rose Canyon	3.4	1.5	7.2	0.48	0.23
Newport-Inglewood (Offshore)	13.4	1.5	7.1	0.24	0.12
Coronado Bank	17.3	3.0	7.6	0.26	0.12

*CDMG 1996

As indicated in Table 1, the Rose Canyon Fault Zone is the 'active' fault considered having the most significant effect at the site from a design standpoint. A maximum credible earthquake of moment magnitude M7.2 on the fault could produce an estimated peak horizontal ground acceleration of 0.48g at the site (0.23g at one standard deviation confidence interval).

Based on 2007 California Building Code (CBC), we have calculated a Site Class of E for the site based on our experience with similar other sites in the project area, and the results of our subsurface evaluation, which indicate SPT blow counts of generally less than 15 blows/foot within the fill and alluvium underlying the site upper 100 feet of the site.

The effect of seismic shaking may also be mitigated by adhering to the California Building Code or state-of-the-art seismic design parameters of the Structural Engineers Association of California. Provided below are the seismic design parameters for the project determined in accordance with the 2007 CBC (CBSC, 2008) and the USGS Ground Motion Parameter Calculator (Version 5.0.8).

Table 2 2007 CBC Seismic Parameters			
Description	Values		CBC Reference
Site Class	E		Table 1613A.5.2
Short Period Spectral Acceleration	S_s	1.396g	Figure 1613A.5(3)
1-Second Period Spectral Acceleration	S_1	0.529g	Figure 1613A.5(4)
Short Period Site Coefficient	F_a	0.9	Table 1613A.5.3(1)
1-Second Period Site Coefficient	F_v	2.4	Table 1613A.5.3(2)
Modified Short Period Spectral Acceleration	S_{MS}	1.257g	Equation 16A-37
Modified 1-Second Period Acceleration	S_{M1}	1.269g	Equation 16A-38
Design Short Period Spectral Acceleration	S_{DS}	0.838g	Equation 16A-39
Design 1-Second Period Spectral Acceleration	S_{D1}	0.846g	Equation 16A-40

Secondary effects that can be associated with severe ground shaking following a relatively large earthquake include shallow ground rupture, soil liquefaction, and dynamic settlement. These secondary effects of seismic shaking are discussed in the following sections.

4.2.1 Shallow Ground Rupture

Ground rupture because of active faulting is not likely to occur on site due to the absence of known active faults. Cracking due to shaking from distant seismic events is not considered a significant hazard, although it is a possibility at any site.

4.2.2 Liquefaction and Dynamic Settlement

Liquefaction and dynamic settlement of soils can be caused by strong vibratory motion due to earthquakes. Both research and historical data indicate that loose, saturated, granular soils are susceptible to liquefaction and dynamic settlement. Liquefaction is typified by a loss of shear strength in the affected soil layer, thereby causing the soil to behave as a viscous liquid. This effect may be manifested by excessive settlements and sand boils at the ground surface.

Design ground motion considered in our liquefaction triggering analyses was the design earthquake with moment magnitude 6.6 and peak ground acceleration (pga) of 0.34g. In the determination of the design moment magnitude, the USGS Earthquake Hazard Program, GMT, was used, which calculates the moment magnitude based on a probabilistic Seismic Hazard Disaggregation of maximum magnitude earthquake at the site (see Appendix D).

The results of the liquefaction analyses indicate potentially discontinuous layers of the alluvial materials, as encountered in the CPT soundings, are considered susceptible to liquefaction at the design earthquake ground motion. In summary, the potentially liquefiable soil ranges from 10 to 55 feet bgs, which is consistent with the findings of the previous geotechnical report (AGRA, 1995). Summary plots of the analyses using the software LiquefyPro (Civil Tech, 2003) are provided in Appendix D.

Dynamic settlement was evaluated utilizing procedures outlined by Robertson and Wride, 1997 and Tokimatsu and Seed, 1987 and the results of that analysis indicate total liquefaction-induced settlement on the order of 3 to 5 inches can be anticipated as a result of the design earthquake event. Differential settlements due to liquefaction may be on the order of 2 inches. A plot of the liquefaction analysis is provided in Appendix D. In general, flexible connections to accommodate relatively minor vertical and lateral displacement (i.e., 1 to 2 inches) should be considered in the design.

4.2.3 Lateral Spread

Empirical relationships have been derived by Youd and others (Youd, 1993; Bartlett and Youd, 1995; and Youd et. al., 1999) to estimate the magnitude of lateral spread due to liquefaction. These relationships include parameters such as earthquake magnitude, distance of the earthquake from the site, slope height and angle, the thickness of liquefiable soil, and gradation characteristics of the soil.

Based on our analysis, there is a low potential for earthquake-induced lateral spread due to the liquefiable zone in general.

5.0 CONCLUSIONS

Based on the results of our geotechnical investigation of the site, it is our opinion that the proposed improvements are feasible from a geotechnical standpoint, provided the following conclusions and recommendations are incorporated into the project plans and specifications.

The following is a summary of the geotechnical factors that should be considered.

- Based on our subsurface exploration and laboratory testing, the existing fill soils appear to be dense; however, it may be disturbed by site demolition. Therefore, the upper 2 feet of the subject are should be considered potentially compressible and subject to settlement, and should be considered in the design of near surface foundations or placement of additional fill.

- Ground water is anticipated at an elevation ranging from 20 to 24 feet msl, which is roughly 9 to 13 feet below the existing ground surface (bgs) (AGRA, 1995). Ground water levels may also fluctuate seasonally and rise during rainy periods.

- Laboratory test results indicate the soils present on the site have a negligible potential for sulfate attack on concrete. However, onsite soils are considered to have a high potential for corrosion on buried uncoated metal conduits from minimum resistivity testing.

- Active or potentially active faults are not known to exist on or in the immediate vicinity of the site.

- The maximum design earthquake of moment magnitude M6.6 with a peak horizontal ground acceleration of 0.34g.

- Based on our analysis, the saturated granular alluvial soils have a potential for liquefaction due to a design earthquake loading.

- The proposed improvements, as well as the rest of the facility, may be subjected to dynamic differential settlements on the order of 2 inches. It should be noted that in 1994 the San Elijo Joint Powers Authority determined that the original facility was not designed to withstand the current earthquake loading. Therefore, the currently proposed improvements (i.e., 1994 improvements) would not need to consider the effects of strong ground motion (i.e., liquefaction and dynamic settlements) (AGRA, 1995).

- Designer of shoring, if applicable, should note that driven or vibrated installation methods may cause densification of loose granular soil, which may result in the settlement or distress of adjacent structures or other existing improvements, such as piping and manholes.

6.0 RECOMMENDATIONS

6.1 Earthwork

We anticipate that earthwork at the site will consist of site preparation, installation of shoring, excavations, and fill placement. We recommend that earthwork on the site be performed in accordance with the following recommendations and the General Earthwork and Grading Specifications for Rough Grading included in Appendix E. In case of conflict, the following recommendations shall supersede those in Appendix E.

6.1.1 Site Preparation

Prior to grading, all areas to receive structural fill or engineered structures should be cleared of surface and subsurface obstructions, including any existing debris and undocumented or loose fill soils, and stripped of vegetation. Removed vegetation and debris should be properly disposed off site.

The existing fill soils near surface may be potentially compressible and not suitable for support of the proposed improvements. In general, we recommend a removal of at least 1 feet below the proposed foundation bottoms (i.e., spread and continuous footings), and at least 2 feet below the proposed pavement or mat foundations. Note that deeper removals may be needed in localized areas based on field observations by the geotechnical consultant during construction. The removal bottom should be moisture-conditioned and recompacted to a minimum 90 percent relative compaction (based on ASTM Test Method D1557) prior to placing fill. All removal bottoms should be reviewed by the geotechnical consultant prior to fill placement.

6.1.2 Excavations and Shoring

Excavations of the onsite materials may generally be accomplished with conventional heavy-duty earthwork equipment. Temporary sloping gradients should be determined in the field by a "competent person" as defined by OSHA. For preliminary planning, sloping of excavations at 1:1 (horizontal to vertical) to a depth of 5 feet may be assumed. Note that excavations should not extend below a 2:1 plane extending down from existing footings unless properly designed by an engineer.

Excavations greater than 5 feet may need shoring. The shoring, if needed, should be designed by a licensed civil engineer and installed by specialty contractors with knowledge of the specific area soil conditions. We recommend that the following lateral earth pressures be used for designing the shoring. It should be noted that in general, cantilever shoring is not recommended for excavations deeper than 15 to 20 feet based on shoring deflection tolerances.

Cantilever Shoring System

Active pressure = 35H (psf), triangular distribution
Passive Pressure = 200h (psf), below the ground water
H = wall height (active case) or h = embedment (passive case)

Tie-Back or Multi-Braced Shoring System

At-Rest Pressure = 30H (psf), rectangular distribution
Passive Pressure = 200h (psf), below the ground water
H = wall height (at-rest case) or h = embedment (passive case)

General

All pressures are based on dewatered conditions, with the water table at least 4 feet below the base of the excavation. All shoring systems should consider adjacent surcharging loads.

6.1.3 Fill Placement and Compaction

In general, the onsite soils are generally suitable for reuse as compacted fill provided they are free of organic material, debris, and rock fragments larger than 8 inches in maximum dimension. All fill soils should be brought to above-optimum moisture conditions and compacted in uniform lifts to at least 90 percent relative compaction based on laboratory standard ASTM Test Method D1557. The optimum lift thickness required to produce a uniformly compacted fill will depend on the type and size of compaction equipment used. In general, fill should be placed in lifts not exceeding 8 inches in thickness.

Placement and compaction of fill should be performed in general accordance with the current local grading ordinances, sound construction practice, and the General Earthwork and Grading Specifications for Rough Grading presented in Appendix E.

6.1.4 Import Soils

Import soils, if needed, should be granular and tested to have an expansion index of less than 50 (per UBC Standard 18-2). The soils shall be certified (by the soil consultant of the export site) to be free from organic debris and contamination (such as pesticides, hydrocarbons, etc.). The soil engineer shall be notified of the potential borrow source a minimum of 36 hours prior to importing the soils onto the site. The soils engineer shall provide acceptance of these soils prior to trucking of import soils onto the site.

6.2 Foundations

Foundations should be designed in accordance with structural considerations and the following recommendations. These recommendations assume that the soils encountered have a low to medium potential for expansion.

Conventional Footings
For support of near surface grade structures including the proposed retaining wall (i.e., anticipate to be less than 5 feet high), conventional spread and continuous footing may be used. The footing should extend a minimum of 24 inches beneath the lowest adjacent finish grade and may be designed for a maximum allowable bearing pressure of 2,000 pounds per square foot (psf). The allowable pressures may be increased by one-third when considering loads of short duration such as wind or seismic forces. The minimum recommended width of footings is 18 inches for continuous footings and 24 inches for square or round footings, if used.

Mat Foundation
The proposed equipment pads and above ground storage tanks may be supported on a structural mat foundation. A soil modulus of subgrade reaction of 175 pounds per cubic inch is recommended for design of the mat foundation and should be designed in accordance with the structural engineer's requirements.

Settlement
The recommended allowable-bearing capacity for near surface grade structures (i.e. 2,000 psf) is based on a maximum total and differential settlement of 1 inch and 3/4 inch, respectively. Since settlements are a function of footing size and contact bearing pressures, some differential settlement can be expected between adjacent footings where a differential loading condition exists. With increased footing depth/width ratios, differential settlement should be less.

6.3 Retaining Wall Lateral Earth Pressures

For design purposes, the following lateral earth pressure values for level backfill are recommended for retaining walls backfilled with on-site soils or approved granular material of very low to low expansion potential.

Table 3

Retaining Wall Equivalent Fluid Weight (pcf)

Conditions	Level
Active	36
At-Rest	55
Passive	300 (Maximum of 3 ksf)

Unrestrained (yielding) cantilever walls up to 10 feet in height should be designed for an active equivalent pressure value provided above. In the design of walls restrained from movement at the top (nonyielding) such as basement walls, the at-rest pressures should be used. Note that below the water table, the passive pressure should be reduced to 150 psf. If conditions other than those covered herein are anticipated, the equivalent fluid pressure values should be provided on an individual case basis by the geotechnical engineer. A surcharge load for a restrained or unrestrained wall resulting from automobile traffic may be assumed to be equivalent to a uniform pressure of 75 psf which is in addition to the equivalent fluid pressure given above. For other uniform surcharge loads, a uniform pressure equal to 0.35q should be applied to the wall (where q is the surcharge pressure in psf). The wall pressures assume walls are backfilled with free draining materials and water is not allowed to accommodate behind walls. Typical retaining wall drainage design is illustrated in Appendix E. Wall backfill should be compacted by mechanical methods to at least 90 percent relative compaction (based on ASTM D1557). Wall footings should be designed in accordance with the foundation design recommendations and reinforced in accordance with structural considerations. For all retaining walls, we recommend a minimum horizontal distance from the outside base of the footing to daylight of 10 feet.

Lateral soil resistance developed against lateral structural movement can be obtained from the passive pressure value provided above. Further, for sliding resistance, the friction coefficient of 0.35 may be used at the concrete and soil interface. These values may be increased by one-third when considering loads of short duration including wind or seismic loads. The total resistance may be taken as the sum of the frictional and passive resistance provided that the passive portion does not exceed two-thirds of the total resistance.

The geotechnical consultant should approve any backfill materials that will be utilized prior to the backfill placement operations. It is the contractor's responsibility to provide representative samples of the selected backfill material.

6.4 Subterranean Basins

For the design of subterranean basin structures (if applicable), we recommend using the lateral earth pressures presented on Figure 3. To account for potential redistribution of forces during a seismic event, the subterranean walls should also be checked considering an additional seismic pressure distribution equal to $9H_T$ psf, where H_T equals the overall retained height in feet. Uplift pressures due to ground water should also be considered in the design. Resistance to the uplift pressures can be obtained from the weight of the structure, and, if needed, addition of lateral flanges at the base of the tank that utilizes overlying soil weight can be considered.

However, the design of the subsurface structure should neglect sidewalls friction in the evaluation the uplift forces due to potential liquefaction of upper zones during a design earthquake loading condition. In summary, the liquefied soils essentially loose its shear strength and can not provide any frictional restraint.

6.5 Preliminary Pavement Design

The appropriate pavement section depends primarily on the type of subgrade soil, shear strength, traffic load, and planned pavement life. Based on field observations, we are assuming that the on site soil will have a minimum R-Value of 20. Since an evaluation of the characteristics of the actual soils at pavement subgrade cannot be made at this time, we have provided the following pavement sections to be used for planning purposes only. The final subgrade characteristics will be highly dependent on the soils present at finish pavement subgrade.

Table 4 Preliminary Pavement Sections		
Pavement Loading Condition	Traffic Index (20-Year Life)	Anticipated Pavement Sections
Parking & Limited pavement Areas	4.5	3.0 inches AC over 6.0 inches Class 2 Base
Drive Areas	5.0	3.0 inches AC over 8.0 inches Class 2 Base
Truck Drive Areas	6.0	4.0 inches AC over 9.0 inches Class 2 base

For areas subject to unusually heavy truck loading (i.e., pump trucks, delivery trucks, etc.), we recommend a full depth of Portland Cement Concrete (P.C.C.) section of 7 inches with appropriate steel reinforcement and crack-control joints as designed by the project architect. We recommend that sections be as nearly square as possible. A 3,500-psi mix that produces a 600-psi modulus of rupture should be utilized. The actual pavement design should also be in accordance with County of San Diego and ACI design criteria.

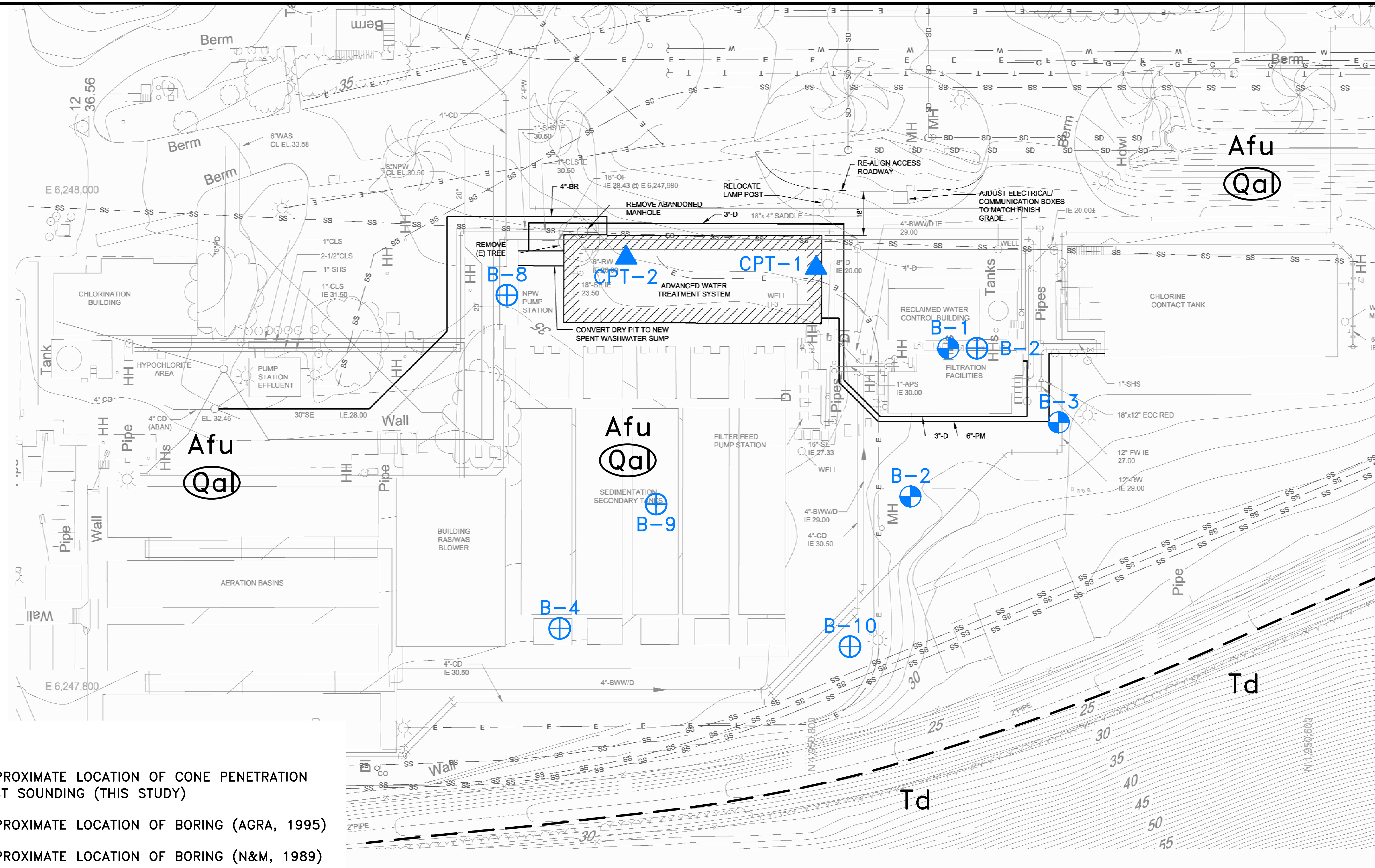
All pavement section materials should conform to and be placed in accordance with the latest revision of the California Department of Transportation Standard Specifications (Caltrans) and American Concrete Institute (ACI) codes. The upper 12 inches of subgrade soil and all aggregate base should be compacted to a relative compaction of at least 95 percent (based on ASTM Test Method D1557).

6.6 Construction Observation

The recommendations provided in this report are based on preliminary design information and subsurface conditions disclosed by widely spaced borings. The interpolated subsurface conditions should be checked in the field during construction. Construction observation of all onsite excavations and field density testing of all compacted fill should be performed by a representative of this office so that construction is in accordance with the recommendations of this report. Final project drawings should be checked by Leighton before grading to see that the recommendations provided in this report are incorporated in project plans.

7.0 LIMITATIONS

The conclusions and recommendations in this report are based in part upon data that were obtained from a limited number of observations, site visits, excavations, samples, and tests. Such information is by necessity incomplete. The nature of many sites is such that differing geotechnical or geological conditions can occur within small distances and under varying climatic conditions. Changes in subsurface conditions can and do occur over time. Therefore, the findings, conclusions, and recommendations presented in this report can be relied upon only if Leighton has the opportunity to observe the subsurface conditions during grading and construction of the project, in order to confirm that our preliminary findings are representative for the site.



LEGEND:

CPT-2 ▲	APPROXIMATE LOCATION OF CONE PENETRATION TEST SOUNDING (THIS STUDY)
B-3	APPROXIMATE LOCATION OF BORING (AGRA, 1995)
B-10	APPROXIMATE LOCATION OF BORING (N&M, 1989)
Afu	UNDOCUMENTED FILL
Qal	ALLUVIUM (CIRCLED WHERE BURIED)
Td	DEL MAR FORMATION
— — —	APPROXIMATE GEOLOGIC CONTACT

N

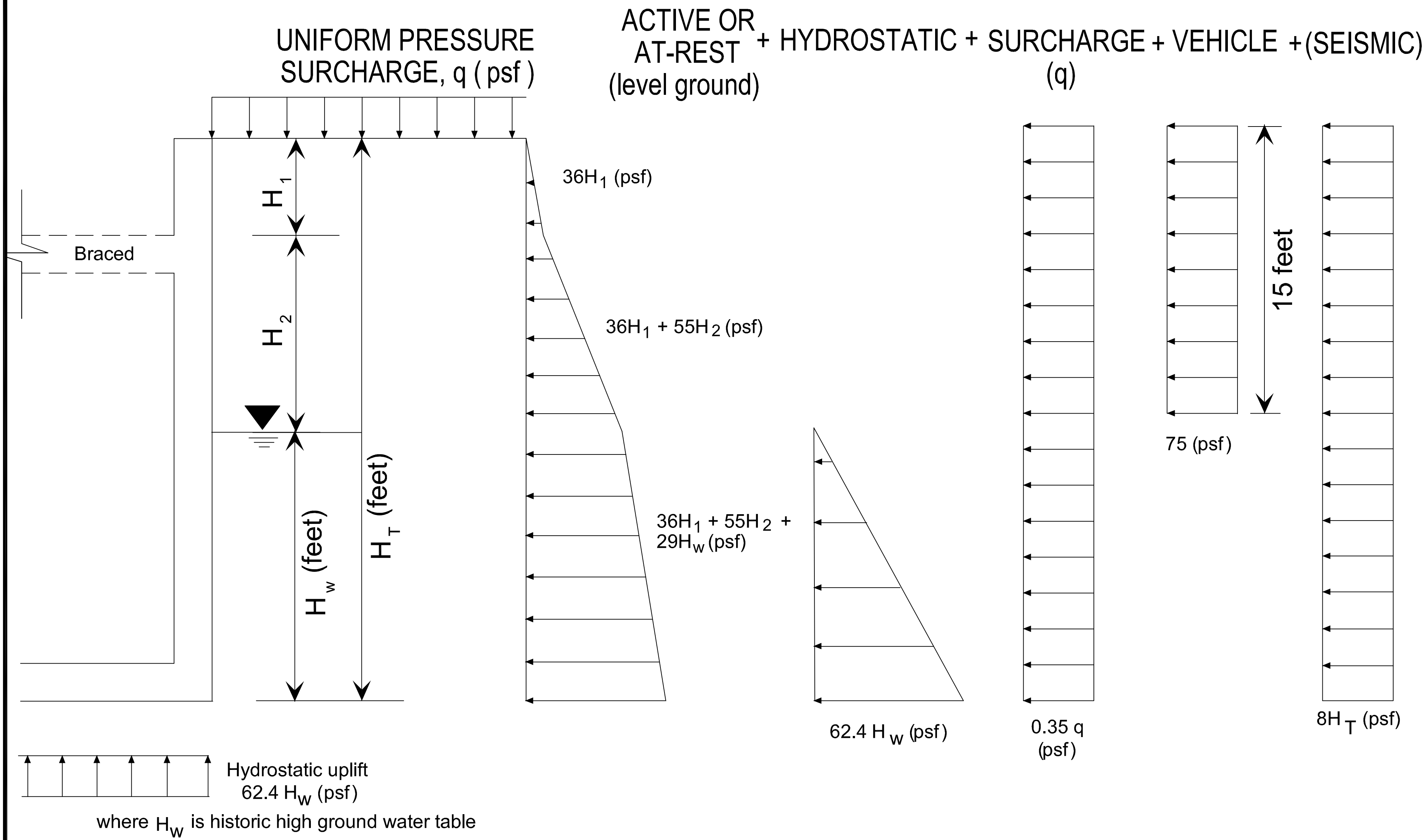
0 40 80
SCALE FEET

EXPLORATION MAP
SAN ELIJO WATER RECLAMATION FACILITIES
CARDIFF BY THE SEA, CALIFORNIA

FIGURE 2

Proj: 602835-001	Eng/Geol: WDO/RCS
Scale: 1"=40'	Date: 03/2010

Leighton



San Elijo Water Reclamation Facility
Cardiff by the Sea, California

LATERAL EARTH AND HYDROSTATIC PRESSURES

Project No.
602835-001

Date
March 2010

Figure 3

APPENDIX A

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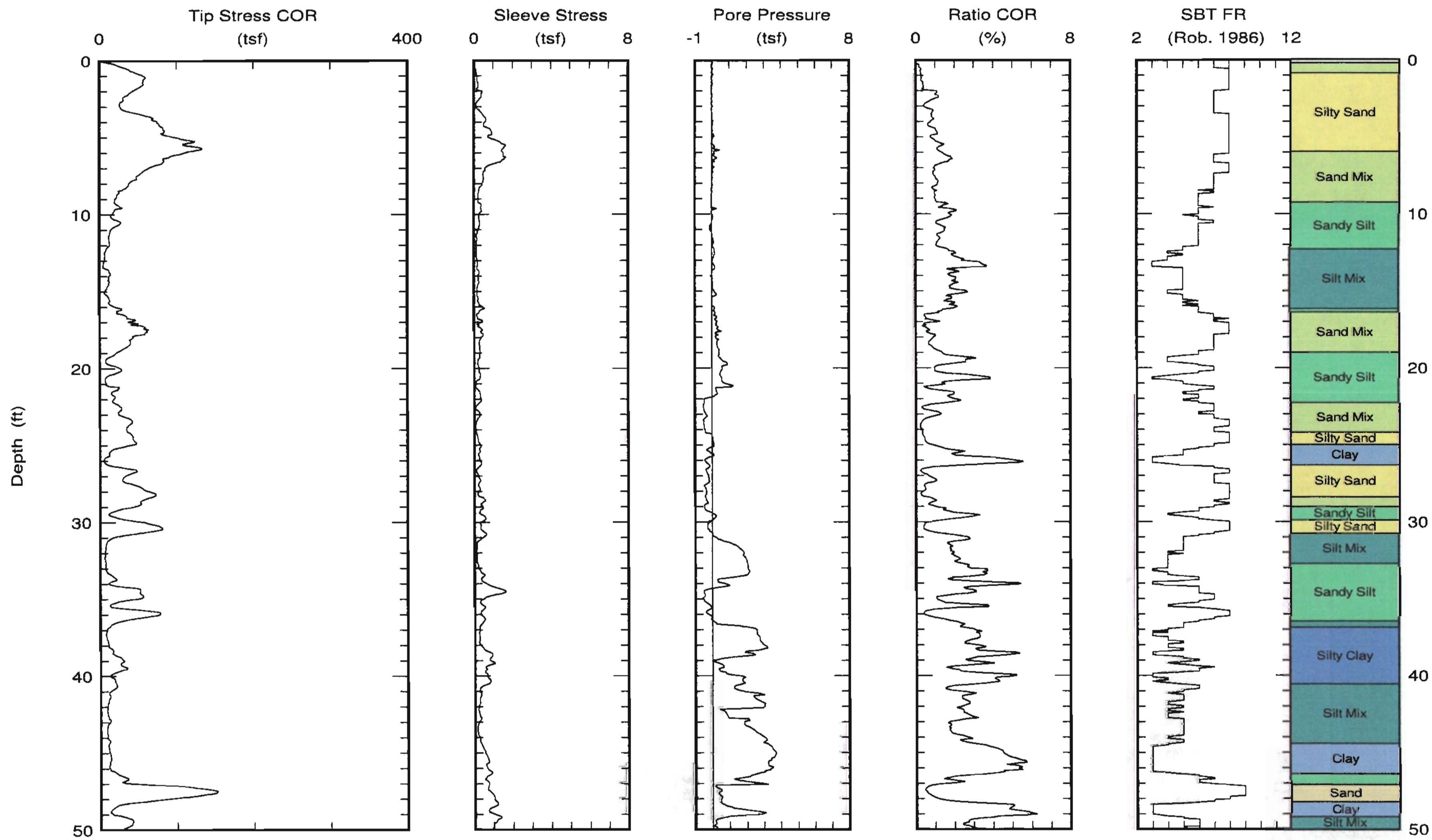
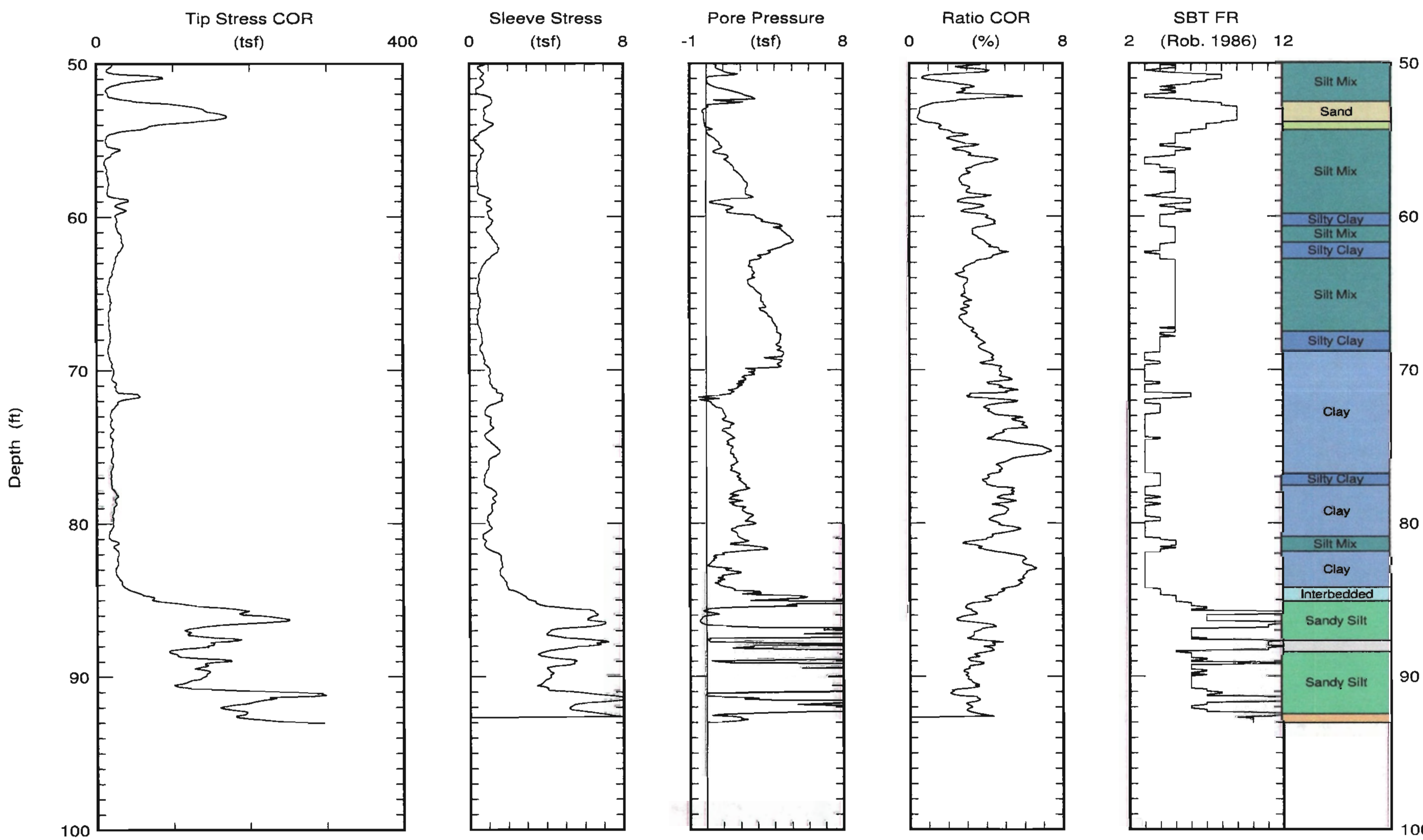
Kehoe Testing & Engineering Office: (714) 901-7270 Fax: (714) 901-7289 rich@kehoetesting.com www.kehoetesting.com	CPT Data 30 ton rig	Date: 23/Feb/2010 Test ID: CPT-1 Project: Encinitas
	Customer: Leighton Consulting Job Site: San Elijo Water Reclamation Project	



Maximum depth: 93.04 (ft)

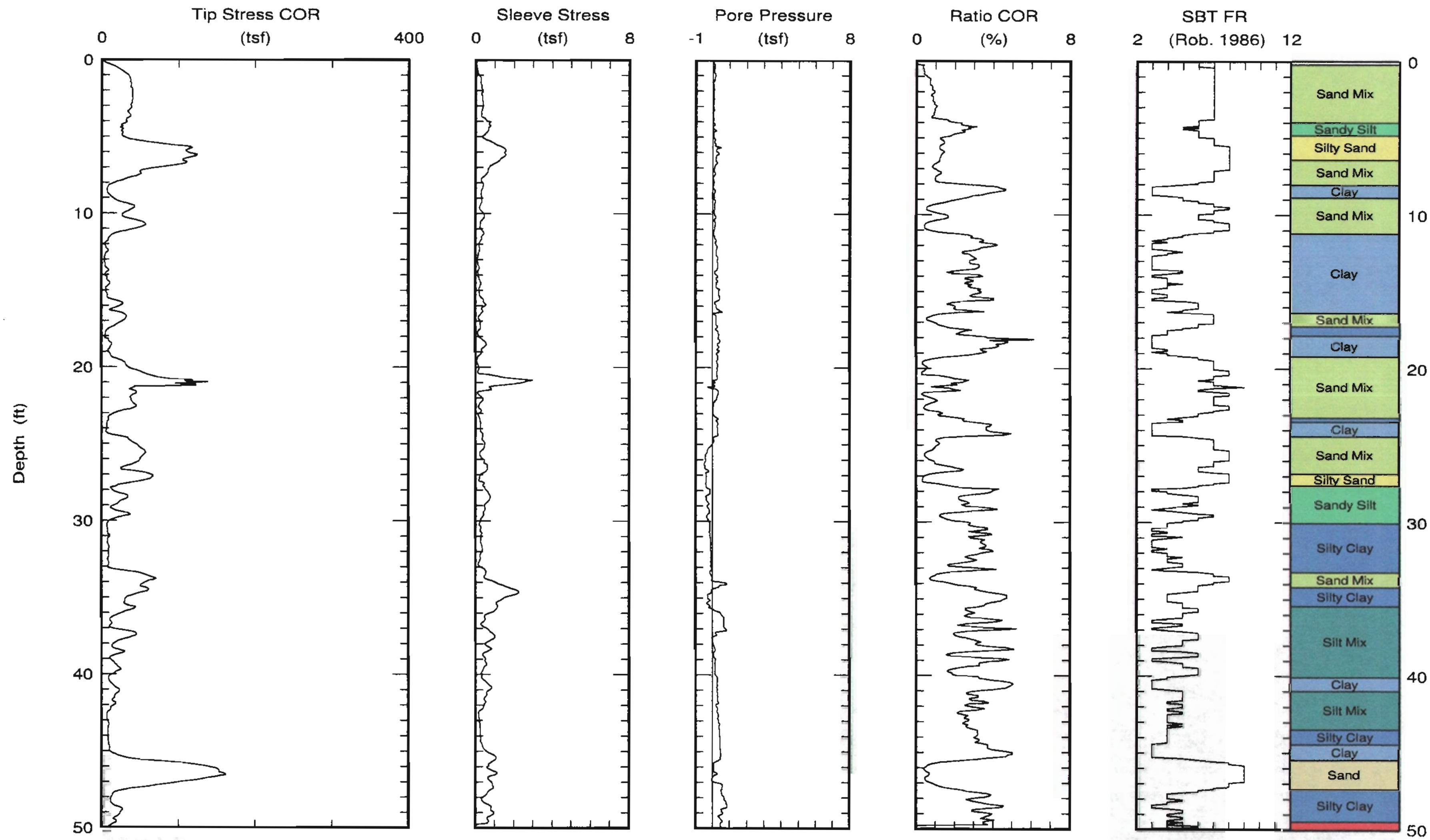
Kehoe Testing & Engineering Office: (714) 901-7270 Fax: (714) 901-7289 rich@kehoetesting.com www.kehoetesting.com	CPT Data 30 ton rig	Date: 23/Feb/2010 Test ID: CPT-1 Project: Encinitas
	Customer: Leighton Consulting Job Site: San Elijo Water Reclamation Project	



Maximum depth: 93.04 (ft)

Page 2 of 2

Kehoe Testing & Engineering Office: (714) 901-7270 Fax: (714) 901-7289 rich@kehoetesting.com www.kehoetesting.com	CPT Data 30 ton rig	Date: 23/Feb/2010 Test ID: CPT-2 Project: Encinitas
	Customer: Leighton Consulting Job Site: San Elijo Water Reclamation Project	

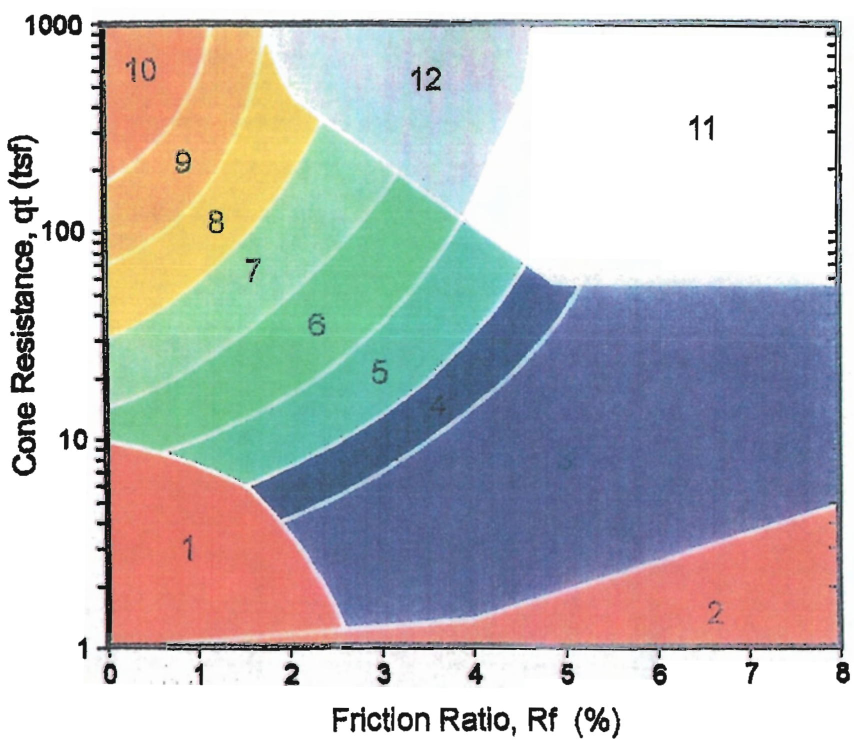


Maximum depth: 50.12 (ft)

KEHOE TESTING & ENGINEERING

CPT Classification Chart

(after Robertson and Campanella, 1988)



Zone	q_t / N	Soil Behavior Type	UCSCS
1	2	sensitive fine grained	OL-OH
2	1	organic material	Pt-OH
3	1	clay	CH
4	1.5	silty clay to clay	CL-CH
5	2	clayey silt to silty clay	ML-CL
6	2.5	sandy silt to clayey silt	MH-ML
7	3	silty sand to sandy silt	SM-ML
8	4	sand to silty sand	SP-SM
9	5	sand	SP
10	6	gravelly sand to sand	SW-SP
11	1	very stiff fine grained *	CL-MH
12	2	sand to clayey sand *	SP-SC

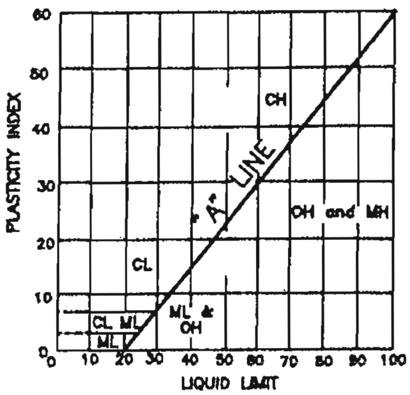
* overconsolidated or cemented

Boring Logs

1995 Geotechnical Investigation Major Structures Associated with the Water Reclamation Treatment and Distribution System for the San Elijo Water Pollution Control Facility, (AGRA, 1995)

UNIFIED SOIL CLASSIFICATION

Pt	OH	CH	MH	OL	CL	ML	SC	SM	SP	SW	GC	GM	GP	GW
Highly organic soils	Silts and clays Liquid limit >50%			Silts and clays Liquid limit <50%			Sands with fines >12% fines		Clean sands <5% fines		Gravels with fines >12% fines		Clean gravels <5% fines	
							Sands—more than 50% of coarse fraction is smaller than No. 4 sieve.				Gravels—more than 50% of coarse fraction is larger than No 4 sieve.			
Fine grained soils (More than 50% is smaller than No. 200 sieve)							Coarse grained soils (More than 50% is larger than No. 200 sieve)							



LABORATORY CLASSIFICATION CRITERIA

GW and SW—$C_u = \frac{D_{60}}{D_{10}}$ greater than 4 for SW & 6 for GW ; $C_c = \frac{(D_{30})^2}{D_{10} \times D_{60}}$ between 1 and 3.

GP and SP – Clean gravel or sand not meeting requirments for GW and SW.
GM and SM – Atterberg limits below "A" line or P.I. less than 4.
GC and SC – Atterberg limits above "A" line with P.I. greater than 7.

FINES (silt or clay)	Fine sand	Medium sand	Coarse sand	Fine gravel	Coarse gravel	Cobbles	Boulders
Sieve sizes: 200	40	10	4	3/4"	3"	12"	

Classification of earth materials shown on this sheet is based on field inspection and should not be construed to imply laboratory anaylsis unless so stated.

MATERIAL SYMBOLS

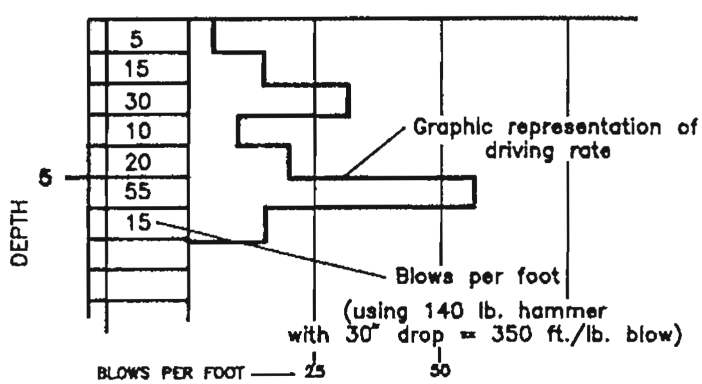
- EXTRUSIVE IGNEOUS ROCK
- ASPHALT
- CLAYSTONE
- CLAYEY SANDSTONE
- CLAYEY SILTSTONE
- METAMORPHIC ROCK
- CONCRETE
- CONGLOMERATE
- INTRUSIVE IGNEOUS ROCK
- INTERBEDDED LIMESTONE AND SHALE
- LIMESTONE
- SANDY CLAYSTONE
- SANDSTONE
- SANDY SILTSTONE
- SILTSTONE
- SILTY CLAYSTONE
- SILTY SANDSTONE

CONSISTENCY CLASSIFICATION FOR SOILS

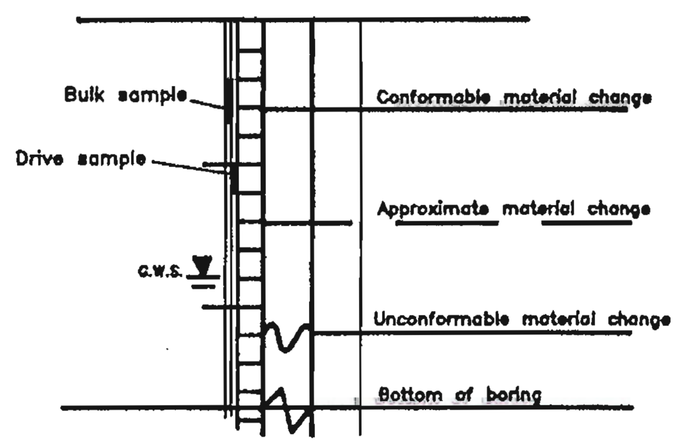
According to the Standard Penetration Test

No. of blows	Granular	Cohesive
0–5	Very loose	Very soft
6–10	Loose	Soft
11–20	Semicompact	Stiff
21–35	compact	Very stiff
36–70	dense	Hard
>70	Very dense	Very hard

LEGEND OF PENETRATION TEST



LEGEND OF BORING



AGRA EARTH & ENVIRONMENTAL, INC.

TEST BORING LOG

TYPE	8" Hollow Stem Auger	ELEVATION	~30.0 feet	BORING	B- 1

STRIKE/DIP and other DEPTH-SPECIFIC NOTES	DRY DENSITY (pcf)	MOISTURE (%)	BLOWS/FOOT	SAMPLE SIZE (inches)	SAMPLE NO.	DEPTH (feet)	UNIFIED SOIL CLASS.	DESCRIPTION
				BAG	1		SM	FILL: Brown, fine to medium SILTY SAND, semicompact, moist, with scattered GRAVEL
	101	5.3	12	2.5	2		SM	ALLUVIUM: Gray, fine to medium SILTY SAND, loose, wet
			10	1.4	3	5		...saturated below 6 feet
							SC	Gray brown, fine CLAYEY SAND, loose, saturated, with minute voids
	103	22.4	3	2.5	4	10		
							SP	Yellow brown, fine to medium SAND, loose
NSR			1	1.4	5	15		
			10	1.4	6	20		
			6	1.4	7	25		
							SC	Gray brown, fine CLAYEY SAND, loose
								... color change to gray green
			10	1.4	8	30		
			9	1.4	9	35		

Notes:

1. Total depth of boring is 36.5 feet.
2. Groundwater encountered at approximately 6 feet.
3. No caving during drilling. Boring caved to 15 feet after removing auger.
4. Boring backfilled with cuttings on 6/16/94.
5. Elevation obtained from plan by Dudek & Associates titled "Proposed Site Plan, Contract No. 3."
6. NSR indicates "no sample recovery."

THIS BORING LOG SUMMARY APPLIES ONLY AT THE TIME AND LOCATION INDICATED. SUBSURFACE CONDITIONS MAY DIFFER AT OTHER LOCATIONS AND TIMES.

LOGGED BY	TMP	DATE	6-16-94

AGRA EARTH & ENVIRONMENTAL, INC.

TEST BORING LOG

TYPE	8" Hollow Stem Auger	ELEVATION	~29.0 feet	BORING	B- 2

STRIKE/DIP and other DEPTH-SPECIFIC NOTES	DRY DENSITY (pcf)	MOISTURE (%)	BLOWS/FOOT	SAMPLE SIZE (inches)	SAMPLE NO.	DEPTH (feet)	UNIFIED SOIL CLASS.	DESCRIPTION
				BAG	1		SM	FILL: Brown, fine to medium SILTY SAND, semicompact, moist, with scattered GRAVEL
	106	3.0	20	2.5	2		SM	ALLUVIUM: Mottled gray brown, fine SILTY SAND, semicompact, moist
	99	17.0	11	2.5	3	5		
				BAG	4			
							SC	Mottled gray brown, fine CLAYEY SAND, loose, saturated
			2	1.4	5	10		
NSR			5	2.5	6	15		
			9	1.4	7	20		
							SM	Yellow brown, fine to medium SILTY SAND, loose
			11	1.4	8	25		
							ML	Mottled gray brown CLAYEY SILT, stiff
							SC	Mottled gray brown, fine CLAYEY SAND, semicompact
			11	1.4	9	30		

Notes:

1. Total depth of boring is 31.5 feet.
2. Groundwater encountered at approximately 8 feet.
3. No caving during drilling. Boring caved to 21 feet after removing auger.
4. Boring backfilled with cuttings on 6/16/94.
5. Elevation obtained from plan by Dudek & Associates titled "Proposed Site Plan, Contract No. 3."
6. NSR indicates "no sample recovery."

MATERIAL SYMBOL

THIS BORING LOG SUMMARY APPLIES ONLY AT THE TIME AND LOCATION INDICATED. SUBSURFACE CONDITIONS MAY DIFFER AT OTHER LOCATIONS AND TIMES.

LOGGED BY TMP | DATE 6-16-94

AGRA EARTH & ENVIRONMENTAL, INC.

TEST BORING LOG

TYPE: 8" Hollow Stem Auger | ELEVATION: ~29.0 feet | BORING: B- 3

STRIKE/DIP and other DEPTH-SPECIFIC NOTES	DRY DENSITY (pcf)	MOISTURE (%)	BLOWS/FOOT	SAMPLE SIZE (inches)	SAMPLE NO.	DEPTH (feet)	UNIFIED SOIL CLASS.	DESCRIPTION
							SM	FILL: Brown, fine to medium SILTY SAND, semicompact, moist, with scattered GRAVEL
	105	5.4	18	2.5	1		SM	ALLUVIUM: Mottled gray brown, fine to medium SILTY SAND, semicompact, wet
			4	1.4	2	5		...4-inch thick lens of fine CLAYEY SAND
				BAG	3			...loose, saturated below 7 feet
	102	20.3	2	2.5	4	10	SP SM	Mottled gray brown, fine SILTY SAND, loose, saturated, with minute voids
NSR			3	1.4	5	15		
							ML	Brown SANDY SILT, soft
			9	1.4	6	20		
							SM	Yellow brown, fine to medium SILTY SAND, loose
			5	1.4	7	25		
							ML	Yellow brown CLAYEY SILT, very soft
							SC	Mottled gray brown, fine to medium CLAYEY SAND, very loose
			5	1.4	8	30		
			8	1.4	9	35		...loose below 35 feet
							ML	Dark gray blue CLAYEY SILT, soft
			10	1.4	10	40		

Notes:

1. Total depth of boring is 41.5 feet.
2. Groundwater encountered at approximately 7 feet.
3. No caving during drilling. Boring caved to 16 feet after removing auger.
4. Boring backfilled with cuttings on 6/16/94.
5. Elevation obtained from plan by Dudek & Associates titled "Proposed Site Plan, Contract No. 3."
6. NSR indicates "no sample recovery."

THIS BORING LOG SUMMARY APPLIES ONLY AT THE TIME AND LOCATION INDICATED. SUBSURFACE CONDITIONS MAY DIFFER AT OTHER LOCATIONS AND TIMES.

LOGGED BY: TMP | DATE: 6-16-94

MATERIAL SYMBOL

DATE DRILLED 11/29/88 BORING NO. B-2

GROUND ELEVATION 26'± (MSL) SHEET 1 OF 4

METHOD OF DRILLING 8" Diameter Hollow Stem Auger

DRIVE WEIGHT 140 lbs. DROP 30"

SAMPLED BY CO LOGGED BY CO

DEPTH (Feet)	SAMPLES Bulk	SAMPLES Driven	BLOWS/FOOT	MOISTURE (%)	DRY DENSITY (PCF)	CLASSIFICATION U.S.C.S.	DESCRIPTION
0							FILL: Brown, moist, loose, silty, fine SAND; concrete, roots and rootlets.
						SM	ALLUVIUM: Gray, wet, loose, silty, fine to medium SAND; occasional rootlets
5			5	25.8			
							@ 7.0': Ground water
10			3	23.5	99.6	CL	Gray, saturated, soft, sandy, silty CLAY; occasional rootlets.
15			3	25.2		SM	Light brown, saturated, very loose, silty, fine to medium SAND. @ 16.0': Clay content increase
20							

Ninyo & Moore

BORING LOG

Malcolm Pirnie, Inc.
San Elijo Water Pollution Control Facility

PROJECT NO.	DATE	
101045-01	3/89	FIGURE B-3

DEPTH (Feet)	SAMPLES Bulk	SAMPLES Driven	BLOWS/FOOT	MOISTURE (%)	DRY DENSITY (PCF)	CLASSIFICATION U.S.C.S.	DATE DRILLED 11/29/88 BORING NO. B-2 SHEET 2 OF 4 GROUND ELEVATION 26'+ (MSL) METHOD OF DRILLING 8" Diameter Hollow Stem Auger DRIVE WEIGHT 140 lbs. DROP 30" SAMPLED BY CO LOGGED BY CO DESCRIPTION
20						CL	Brown, saturated, firm, sandy, silty CLAY.
			16	24.5	100.5	CL-SC	Light brown, saturated, stiff, silty, sandy CLAY, medium dense, silty, clayey SAND; micaceous.
25			10	21.9	104.0	SC-CL	Light brown, saturated, loose, silty, clayey SAND, to firm, silty, sandy CLAY; micaceous.
30						SC-CL	@ 30.0': Color change to gray
			11	23.3	102.4		
35			11	25.8	96.5	CL	Gray, saturated, loose, silty, sandy CLAY.
40							

Ninyo & Moore

BORING LOG

Malcolm Pirnie, Inc.
San Elijo Water Pollution Control Facility

PROJECT NO.	DATE	
101045-01	3/89	FIGURE B-4

DATE DRILLED	11/29/88
BORING NO.	B-2
GROUND ELEVATION	26'± (MSL)
SHEET	3 OF 4
METHOD OF DRILLING	8" Diameter Hollow Stem Auger
DRIVE WEIGHT	140 lbs.
DROP	30"
SAMPLED BY	CO
LOGGED BY	CO

DEPTH (Feet)	SAMPLES Bulk	SAMPLES Driven	BLOWS/FOOT	MOISTURE (%)	DRY DENSITY (PCF)	CLASSIFICATION U.S.C.S.	DESCRIPTION
40			11	28.1		CL	Gray, saturated, stiff, silty, sandy CLAY.
45			27	21.0	107.7	SC	WEATHERED FORMATION: Gray to brown, saturated, medium dense, silty, clayey, fine to coarse SAND; some decayed rootlets.
50			15	25.3		SC-CL	Gray, saturated, medium dense, silty, clayey SAND to firm, silty, sandy CLAY.
55			36	27.6	96.9	CL	Gray, saturated, very stiff, sandy, silty CLAY; caliche, red-brown mottling.
60							

Ninyo & Moore

BORING LOG

Malcolm Pirnie, Inc.
San Elijo Water Pollution Control Facility

PROJECT NO.	DATE	FIGURE
101045-01	3/89	B-5

DEPTH (Feet)	SAMPLES Bulk	SAMPLES Driven	BLOWS/FOOT	MOISTURE (%)	DRY DENSITY (PCF)	CLASSIFICATION U.S.C.S.	DATE DRILLED 11/29/88 BORING NO. B-2 SHEET 4 OF 4 GROUND ELEVATION 26'± (MSL METHOD OF DRILLING 8" Diameter Hollow Stem Auger DRIVE WEIGHT 140 lbs. DROP 30" SAMPLED BY CO LOGGED BY CO DESCRIPTION
60		■				SM	Red-brown, saturated, medium dense, slightly silty SAND.
		■	37	21.9	106.9	CL	Light gray, saturated, very stiff, silty CLAY.
							Total Depth 61.5'. Ground Water at 7.0'. No Caving. Backfilled 11/29/88.
65							
70							
75							
80							

Ninyo & Moore

BORING LOG

Malcolm Pirnie, Inc.
San Elijo Water Pollution Control Facility

PROJECT NO.	DATE	FIGURE
101045-01	3/89	B-6

DATE DRILLED 11/23/88 **BORING NO.** B-4

GROUND ELEVATION 27'± (MSL) SHEET 1 OF 2

METHOD OF DRILLING 8" Diameter Hollow Stem Auger

DRIVE WEIGHT 140 lbs. DROP 30"

SAMPLED BY MAS LOGGED BY MAS

DEPTH (Feet)	SAMPLES Bulk	SAMPLES Driven	BLOWS/FOOT	MOISTURE (%)	DRY DENSITY (PCF)	CLASSIFICATION U.S.C.S.	DESCRIPTION
0							FILL:
						SM	Medium brown, moist, medium dense, silty SAND; occasional pocket of sandy clay, (brown-gray).
							ALLUVIUM:
						SM	Light to medium brown, moist, loose, slightly silty, fine to medium SAND; occasional pocket of brown-gray, sandy clay.
			12	10.0	95.5		
5							
							@ 7.0': Ground water
							@ 8.0': Occasional thin layer (1-2") of brown-gray, sandy clay.
			5	24.5			
10							
			6	No Recovery			
15							
			8	24.8	102.0		
			12	22.9		CL	@ 18.5': A 1' thick layer of medium brown, saturated, stiff, sandy CLAY.
20							

Ninyo & Moore

BORING LOG

Malcolm Pirnie, Inc.
San Elijo Water Pollution Control Facility

PROJECT NO.	DATE	FIGURE B-11
101045-01	3/89	

DEPTH (Feet)	SAMPLES Bulk	SAMPLES Driven	BLOWS/FOOT	MOISTURE (%)	DRY DENSITY (PCF)	CLASSIFICATION U.S.C.S.	DATE DRILLED 11/23/88 BORING NO. B-4; GROUND ELEVATION 27'± (MSL) SHEET 2 OF 2; METHOD OF DRILLING 8" Diameter Stem Auger; DRIVE WEIGHT 140 lbs. DROP 30"; SAMPLED BY MAS LOGGED BY MAS — DESCRIPTION
20							ALLUVIUM: (CONTINUED)
						SM	Light yellow-brown, wet, loose, slightly silty, fine to medium SAND.
		■	16	23.2	101.8		@ 23.5': Silty, fine to coarse-grained SAND; approximately 6" thick.
25							Total Depth 24.5'. Ground Water at 7.0'. Backfilled 11/23/88.
30							
35							
40							

Ninyo & Moore

BORING LOG

Malcolm Pirnie, Inc.
San Elijo Water Pollution Control Facility

PROJECT NO.	DATE	FIGURE
101045-01	3/89	B-12

DEPTH (Feet)	SAMPLES Bulk	SAMPLES Driven	BLOWS/FOOT	MOISTURE (%)	DRY DENSITY (PCF)	CLASSIFICATION U.S.C.S.	DESCRIPTION
0							FILL:
						SM	Light to medium brown, moist, very loose, silty, fine to medium SAND.
			4	24.0	92.3		
5							Brown-gray, wet, sandy CLAY.
							ALLUVIUM:
				▽			@ 7.0': Ground water
						SM	Light to medium brown, wet, loose, slightly silty, fine to medium SAND; some thin layers of sandy clay.
			5	20.7			
10							
			16	21.2	109.7		
						SM	Light yellow-brown, wet, loose, slightly silty, fine to coarse SAND; occasional pocket of brown-gray, sandy clay.
15							
			7	21.7			
20							

DATE DRILLED 11/23/88 BORING NO. B-8

GROUND ELEVATION 30'± (MSL) SHEET 1 OF 2

METHOD OF DRILLING 8" Diameter Hollow Stem Auger

DRIVE WEIGHT 140 lbs. DROP 30"

SAMPLED BY MAS LOGGED BY MAS

Ninyo & Moore

BORING LOG

Malcolm Pirnie, Inc.
San Elijo Water Pollution Control Facility

PROJECT NO.	DATE	FIGURE
101045-01	3/89	B-19

DATE DRILLED 11/23/88 BORING NO. B-9

GROUND ELEVATION 27'± (MSL) SHEET 1 OF 2

METHOD OF DRILLING 8" Diameter Hollow Stem Auger

DRIVE WEIGHT 140 lbs. DROP 30"

SAMPLED BY MAS LOGGED BY MAS

DEPTH (Feet)	SAMPLES Bulk	SAMPLES Driven	BLOWS/FOOT	MOISTURE (%)	DRY DENSITY (PCF)	CLASSIFICATION U.S.C.S.	DESCRIPTION
0							FILL:
						SM	Medium brown, moist, silty, fine to medium SAND; occasional pocket of red-brown, clayey sand.
						ML	Dark gray, very moist to wet, firm, sandy SILT.
		■	12	19.5	97.6		
				▽			ALLUVIUM:
5						SM	Light to medium brown, wet, loose, silty, fine to medium SAND. @ 4.0': Ground water.
			4	25.0			
10						SC-CL	Medium brown, saturated, loose to medium dense, clayey SAND to sandy CLAY; mottled with dark gray, sandy clay.
		■	11	21.2	103.1		
15						SM	Light to medium brown, wet, medium dense, silty SAND; some pockets of clayey sand to sandy clay.
			10	24.6			
20						CL	Medium brown-gray, saturated, firm, sandy CLAY.

Ninyo & Moore

BORING LOG

Malcolm Pirnie, Inc.
San Elijo Water Pollution Control Facility

PROJECT NO.	DATE	FIGURE
101045-01	3/89	B-21

DEPTH (Feet)	SAMPLES Bulk	SAMPLES Driven	BLOWS/FOOT	MOISTURE (%)	DRY DENSITY (PCF)	CLASSIFICATION U.S.C.S.	DESCRIPTION
20							ALLUVIUM: (CONTINUED)
						CL	Medium brown-gray, saturated, firm, sandy CLAY.
		■	11	No recovery		SC	Light to medium brown, wet, loose, clayey SAND; some thin layers of sandy clay.
25							
							Total Depth 26.0'. Ground Water at 4.0'. No Caving. Backfilled and piezometer installed 11/23/88 to 26.0'.
30							
35							
40							

DATE DRILLED 11/23/88 BORING NO. B-9

GROUND ELEVATION 27'± (MSL) SHEET 2 OF 2

METHOD OF DRILLING 8" Diameter Hollow Stem Auger

DRIVE WEIGHT 140 lbs. DROP 30"

SAMPLED BY MAS LOGGED BY MAS

Ninyo & Moore

BORING LOG

Malcolm Pirnie, Inc.
San Elijo Water Pollution Control Facility

PROJECT NO.	DATE	FIGURE
101045-01	3/89	B-22

DATE DRILLED 11/23/88 **BORING NO.** B-10

GROUND ELEVATION 25'± (MSL) SHEET 1 OF 2

METHOD OF DRILLING 8" Diameter Hollow Stem Auger

DRIVE WEIGHT 140 lbs. DROP 30"

SAMPLED BY CO LOGGED BY CO

DEPTH (Feet)	SAMPLES Bulk	SAMPLES Driven	BLOWS/FOOT	MOISTURE (%)	DRY DENSITY (PCF)	CLASSIFICATION U.S.C.S.	DESCRIPTION
0						SM	FILL: Brown, dry to damp, medium dense, silty SAND; concrete and asphalt.
						CL	ALLUVIUM: Gray, moist, stiff, silty CLAY. @ 5.0': Ground water
5			9	26.7	96.1	SM	Gray to light brown, loose, saturated, slightly silty, fine to medium SAND.
10			6	25.0		CL	Light brown, mottled with gray, saturated, firm, sandy, silty CLAY; micaceous.
15			6			SC-CL	Light brown, saturated, very loose, clayey SAND; to stiff, sandy CLAY; micaceous.
20							

Ninyo & Moore

BORING LOG

Malcolm Pirnie, Inc.
San Elijo Water Pollution Control Facility

PROJECT NO.	DATE	FIGURE
101045-01	3/89	B-23

DEPTH (Feet)	SAMPLES Bulk	SAMPLES Driven	BLOWS/FOOT	MOISTURE (%)	DRY DENSITY (PCF)	CLASSIFICATION U.S.C.S.	DATE DRILLED 11/23/88 BORING NO. B-10 GROUND ELEVATION 25'± (MSL) SHEET 2 OF 2 METHOD OF DRILLING 8" Diameter Hollow Stem Auger DRIVE WEIGHT 140 lbs. DROP 30" SAMPLED BY CO LOGGED BY CO DESCRIPTION
20							ALLUVIUM: (CONTINUED)
			6	24.0		SC	Light brown, saturated, loose, silty, clayey, fine to medium SAND; micaceous.
							@ 25.0': A lens of light brown, saturated, medium dense, silty SAND.
25			12	29.6	90.0	SC-CL	Light brown, saturated, loose, silty, clayey SAND; to stiff, sandy, silty CLAY; micaceous.
							Total Depth 26.5'. Ground Water at 5.0'. No Caving. Backfilled 11/23/88.
30							
35							
40							

Ninyo & Moore

BORING LOG

Malcolm Pirnie, Inc.
San Elijo Water Pollution Control Facility

PROJECT NO.	DATE	FIGURE B-24
101045-01	3/89	

APPENDIX C

Laboratory Testing Procedures and Test Results

Direct Shear Tests: Direct shear test was performed on selected remolded samples which were soaked for a minimum of 24 hours under a surcharge equal to the applied normal force during testing. After transfer of the sample to the shear box, and reloading the sample, pore pressures set up in the sample due to the transfer were allowed to dissipate for a period of approximately 1 hour prior to application of shearing force. The sample was tested under various normal loads, a motor-driven, strain-controlled, direct-shear testing apparatus at a strain rate of less than 0.001 to 0.5 inches per minute (depending upon the soil type). The test results are presented in the attached figure.

Expansion Index Tests: The expansion potential of selected materials was evaluated by the Expansion Index Test, ASTM Standard D4829. Specimens are molded under a given compactive energy to approximately the optimum moisture content and approximately 50 percent saturation. The prepared 1-inch thick by 4-inch diameter specimens are loaded to an equivalent 144 psf surcharge and are inundated with water until volumetric equilibrium is reached. The results of these tests are presented in the table below:

Sample Location	Sample Description	Expansion Index	Expansion Potential*
CPT-1, 0-5 Feet	Brown silty Sand (SM)	2	Very Low

* The expansion potential of selected materials was evaluated by the Classification of Expansive Soil, CBC. Table No. 18-I-B.

Soluble Sulfates: The soluble sulfate contents of selected samples were determined by standard geochemical methods. The test results are presented in the table below:

Sample Location	Sulfate Content (Percent by Wt.)	Potential Degree of Sulfate Attack*
CPT-1, 0-5 Feet	0.03	Negligible

* Based on the 1997 edition of the Uniform Building Code, Table No. 19-A-4, prepared by the International Conference of Building Officials (ICBO, 1997).

APPENDIX C (Continued)

Chloride Content: Chloride content was tested in accordance with DOT Test Method No. 422. The results are presented below:

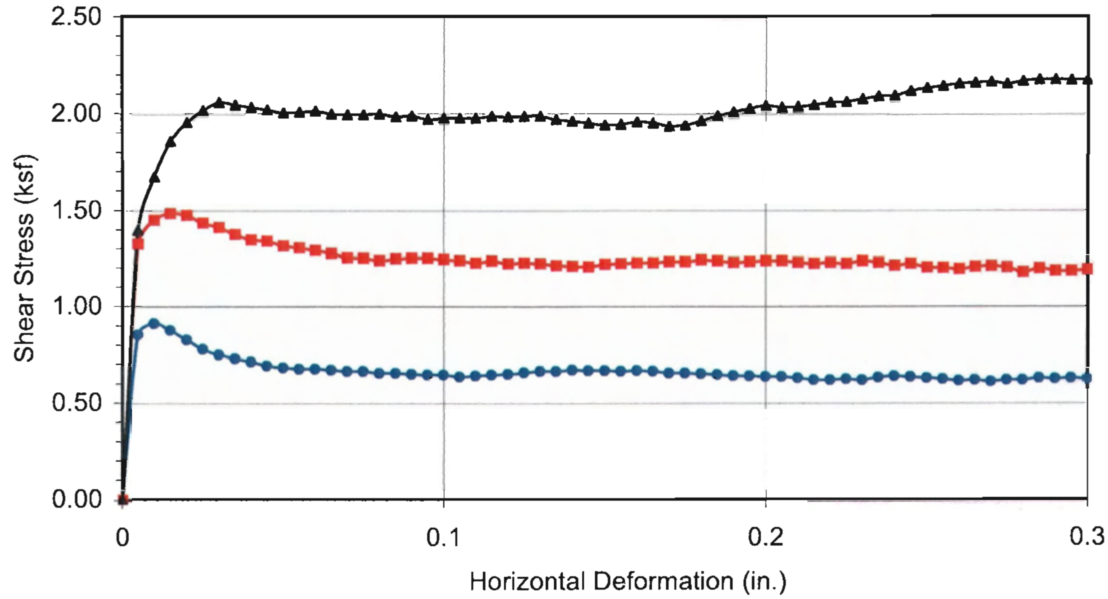
Sample Location	Chloride Content, ppm	Degree of Corrosivity**
CPT-1, 0-5 Feet	86	Threshold

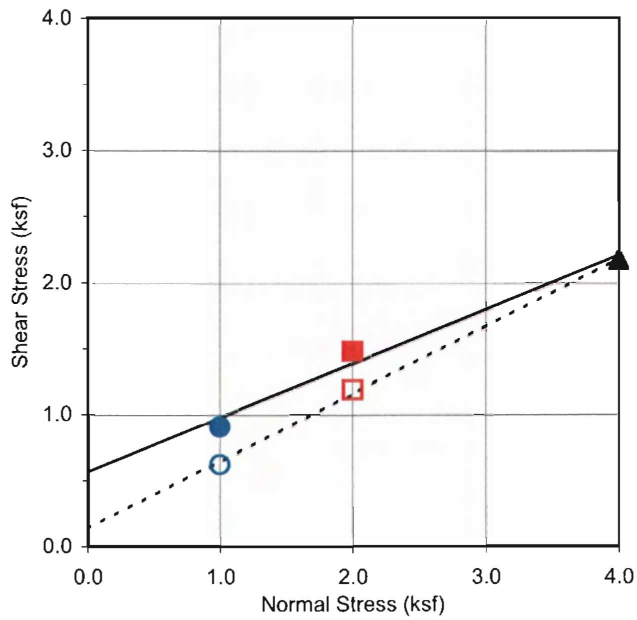
** Based on City of San Diego, Program Guidelines for Design Consultants, CWP, February 1992.

Minimum Resistivity and pH Tests: Minimum resistivity and pH tests were performed in general accordance with California Test Method 643. The results are presented in the table below:

Sample Location	pH	Minimum Resistivity (ohms-cm)	Corrosion Potential**
CPT-1, 0-5 Feet	7.86	1,275	Corrosive

** Based on City of San Diego, Program Guidelines for Design Consultants, CWP, February 1992.





Boring No.	**CPT-2**
Sample No.	**B-1**
Depth (ft)	**0-5**
Sample Type:	90% Remold
Soil Identification:	(SC-SM), BROWN SILTY, CLAYEY SAND WITH TRACE GRAVEL

Strength Parameters

	C (psf)	ϕ (°)
Peak	566.5	22.3
Ultimate	138.5	27.1

Normal Stress (kip/ft²)	1.000	2.000	4.000
Peak Shear Stress (kip/ft²)	● 0.912	■ 1.484	▲ 2.175
Shear Stress @ End of Test (ksf)	○ 0.629	□ 1.191	△ 2.172
Deformation Rate (in./min.)	0.0500	0.0500	0.0500
Initial Sample Height (in.)	1.000	1.000	1.000
Diameter (in.)	2.415	2.415	2.415
Initial Moisture Content (%)	9.50	9.50	9.50
Dry Density (pcf)	114.8	114.8	114.1
Saturation (%)	54.8	54.8	53.7
Soil Height Before Shearing (in.)	0.9978	0.9927	0.9758
Final Moisture Content (%)	16.0	16.2	16.2

Leighton

DIRECT SHEAR TEST RESULTS

Consolidated Undrained

Project No.: 602835-001

KENNEDY & JENKS / SAN ELIJO

02-10

Laboratory Test Results

1995 Geotechnical Investigation Major Structures Associated with the Water Reclamation Treatment and Distribution System for the San Elijo Water Pollution Control Facility, (AGRA, 1995)

APPENDIX B
LABORATORY TESTING

The laboratory test program was designed to fit the specific needs of this project and was limited to testing on-site materials. A brief description of each type of geotechnical test is presented below. Specific results are given on the following pages and on the boring logs in Appendix A.

Moisture contents and dry densities were determined for numerous relatively undisturbed samples. Results are listed on the boring logs in Appendix A adjacent to the sample location.

Strength characteristics of the subsurface soils were determined in the laboratory by direct shear tests performed on relatively undisturbed samples. Three samples were submerged and tested under three different normal loads and 4 samples were tested at natural moisture contents. One sample was tested at natural moisture content under overburden pressure. All samples were tested in a 2.5-inch I.D. circular shear box, using a controlled displacement rate in general accordance with ASTM D 3080.

A laboratory compaction tests were performed on 2 samples to determine maximum dry density and optimum moisture content relationships. The test was performed in general accordance with ASTM D1557.

The weight percent finer than the No. 200 sieve was determined for 5 samples in general accordance with ASTM D 1140.

The grain size distribution was determined for 4 samples in general conformance with ASTM D 422. Results of the tests are plotted in this appendix.

The Expansion Index was determined for 1 sample in accordance with UBC Standard No. 29-2.

The pH and resistivity were determined for 3 samples in general accordance with California Test 643. The soluble sulfate content were determined for 3 samples in general accordance with California Test 417 and the chloride ion content was determined for 3 samples in accordance with California Test 422.

Consolidation characteristics were determined for 3 samples in general conformance with ASTM D 2435. Results of the test are plotted in this appendix.

The remaining soil samples are now stored in our laboratory for future reference and testing if needed. Unless notified to the contrary, all samples will be disposed of 30 days from the date of the final report.

AGRA Earth & Environmental
ENGINEERING GLOBAL SOLUTIONS

Recycled Paper

TABLE B-1
DIRECT SHEAR
TEST RESULTS
(ASTM D 3080)

Boring No./ Sample No.	Normal Stress (psf)	Peak Shear Stress (psf)	Shear Stress at 0.25 Inch Displacement (psf)
1 / 4 (Saturated)	1125 2160 3195	729 1174 1705	729 1174 1705
2 / 2 (Saturated)	1125 2160 3195	966 1951 2510	758 1449 2121
3 / 1 (Saturated)	1125 2160 3195	1013 1553 2370	710 1278 1870
11 / 2 (Natural Moisture Content)	825	777	616
12 / 2 (Natural Moisture Content)	1125 2160 3195	3428 3058 5475	947 1780 2604
12 / 5 (Natural Moisture Content)	1125 2160 3195	1865 2888 4036	928 1648 2566
13 / 2 (Natural Moisture Content)	1125 2160 3195	1951 3002 4059	909 1733 2566

AGRA Earth & Environmental
ENGINEERING GLOBAL SOLUTIONS

Recycled Paper

TABLE B-2
LABORATORY COMPACTION
TEST RESULTS
(ASTM 1557)

Boring No./Sample No.	Maximum Dry Density (pcf)	Optimum Moisture Content (%)
2 / 1	127.5	10.0
3 / 3	108.0	14.0

TABLE 3
-#200 SIEVE WASH
TEST RESULTS
(ASTM D 1140)

Boring No./Sample No.	Percent Finer Than #200 Sieve
1 / 6	5
2 / 5	29
2 / 7	44
2 / 8	12
3 / 6	59

AGRA Earth & Environmental
ENGINEERING GLOBAL SOLUTIONS

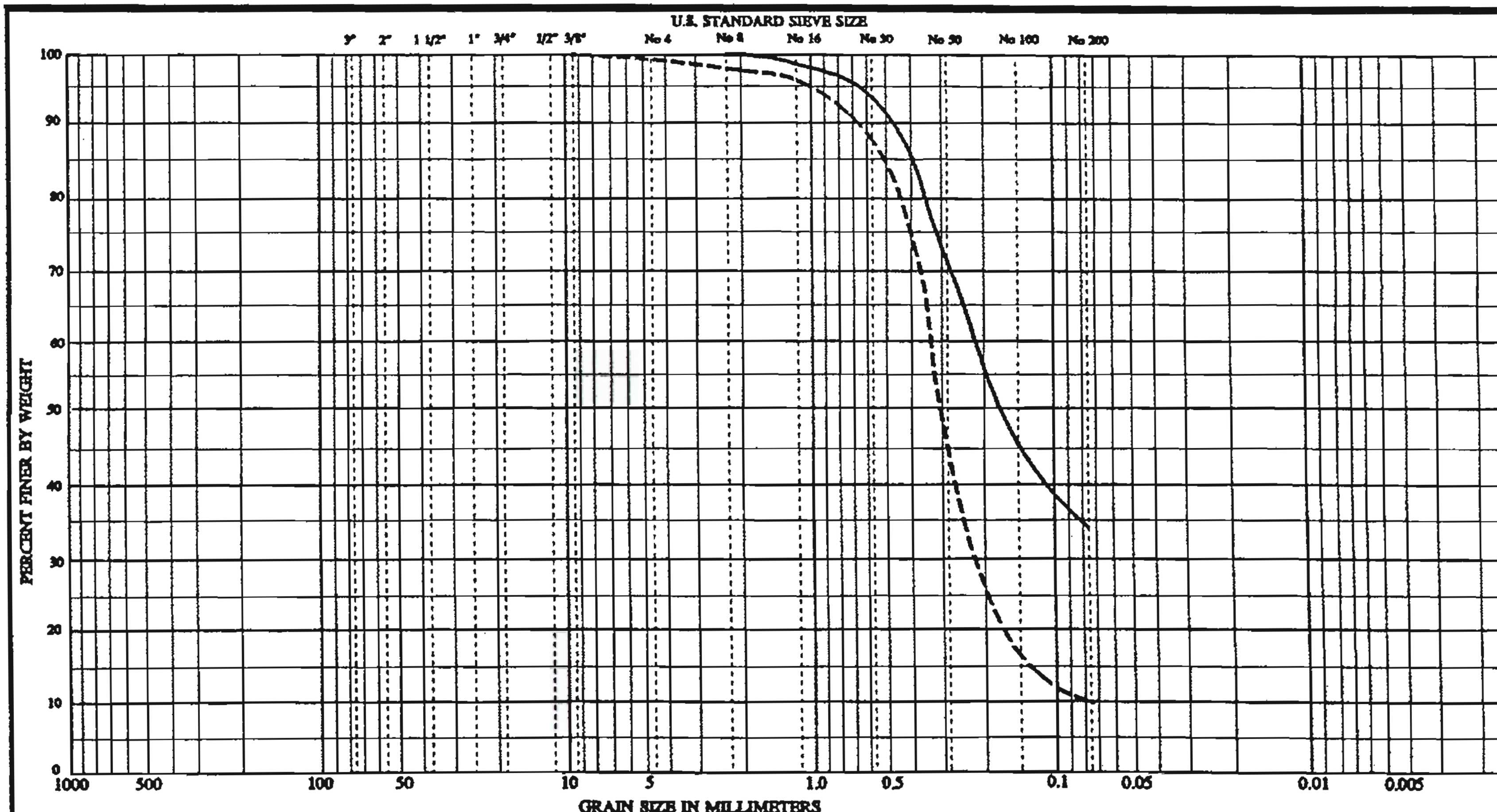
Recycled Paper

TABLE 4
EXPANSION INDEX
TEST RESULTS
(UBC 29-2)

Boring No./Sample No.	Expansion Index	Expansion Potential
12 / 1	0	Low

TABLE 5
CORROSION
TEST RESULTS
(California Test No. 417, 422, 643)

Boring No./Sample No.	pH	Resistivity (ohm-cm)	Chloride Content (ppm)	Soluble Sulphate (ppm)
2 / 4	7.9	2840	51.5	109.5
3 / 3	7.8	4733	40.3	139.2
12 / 4	6.2	296	968.5	157.5



BOULDERS	COBBLES	GRAVEL		SAND			FINES (Silt and Clay)
		Coarse	Fine	Coarse	Medium	Fine	

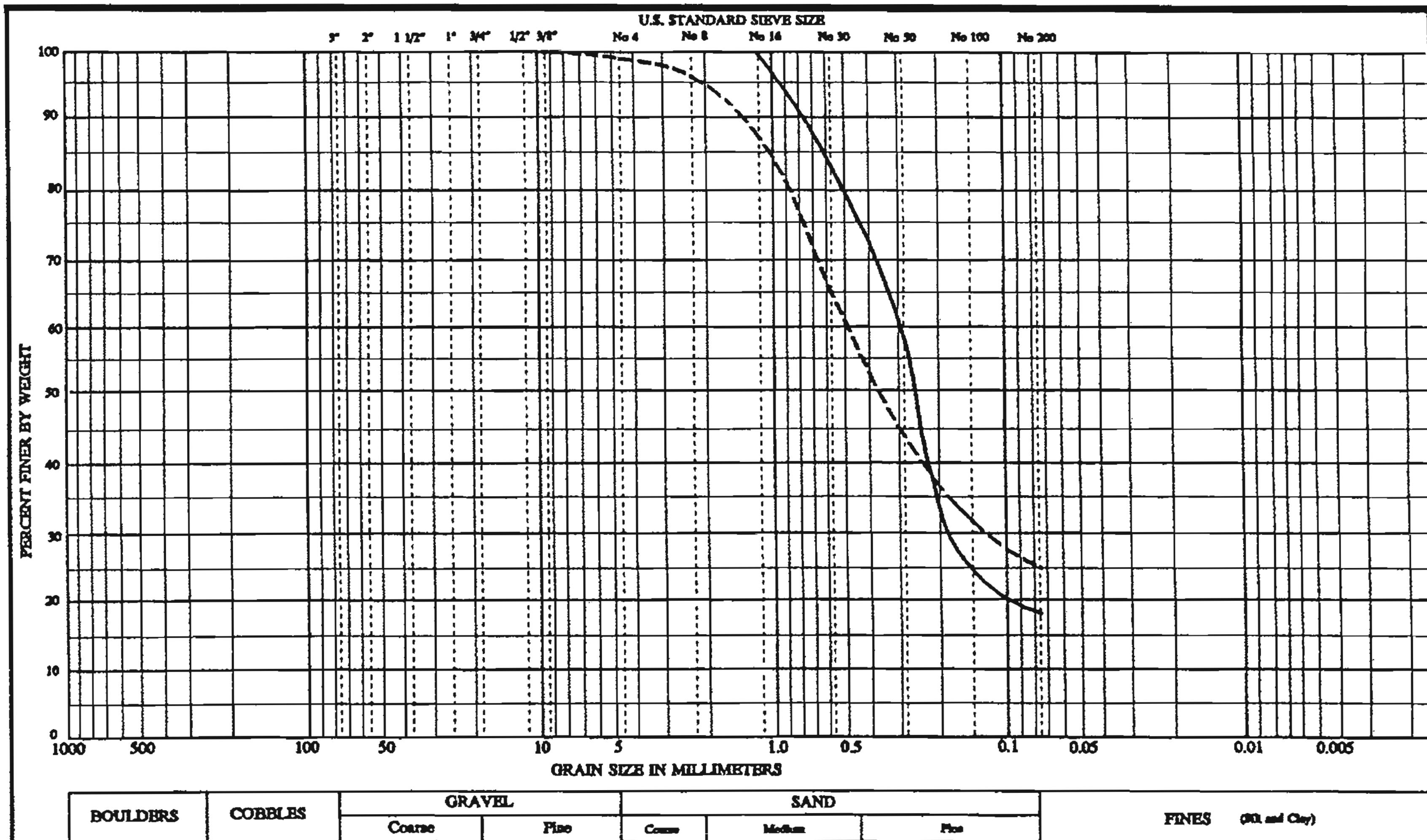
Sample No. or Location		Test Date	Unified Soil Classif.	Description	Natural Water	L L	P L	P I	S E	Max. Dry Density	Optimum Water
1 / 8	——	8/3/94	SC	CLAYEY SAND	22.9						
3 / 3	----	8/3/94	SM	SILTY SAND	5.3						
	....										
	....										

GRAIN SIZE DISTRIBUTION

AGRA E & E

Project: SAN ELIJO WPCF

By: WC | Date: 9 / 12 / 94



Sample No. or Location	Test Date	Unified Soil Classif.	Description	Natural Water	L L	P L	P I	S E	Max. Dry Density	Optimum Water
11 / 1 ———	8/3/94	SM	SILTY SAND	5.8						
12 / 1 - - - -	8/3/94	SM	SILTY SAND	3.9						
....										
....										

GRAIN SIZE DISTRIBUTION

AGRA E & E

Project: SAN ELIJO WPCF

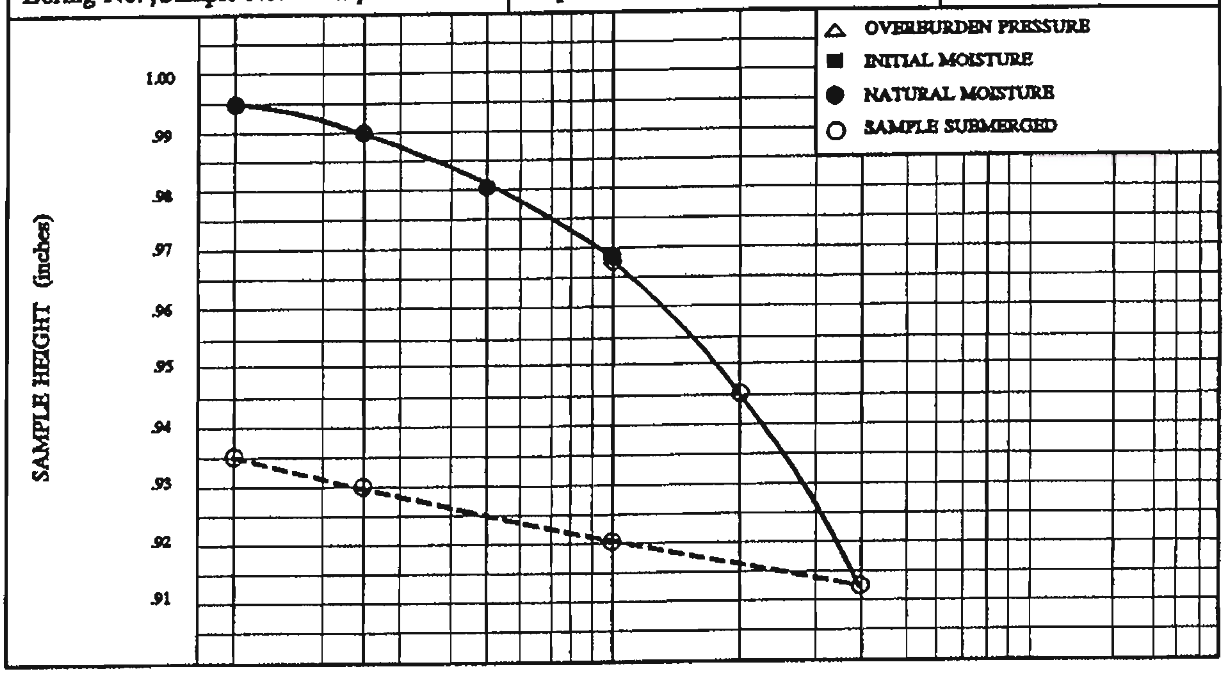
By: WC

Date: 9 / 12 / 94

AGRA Earth & Environmental

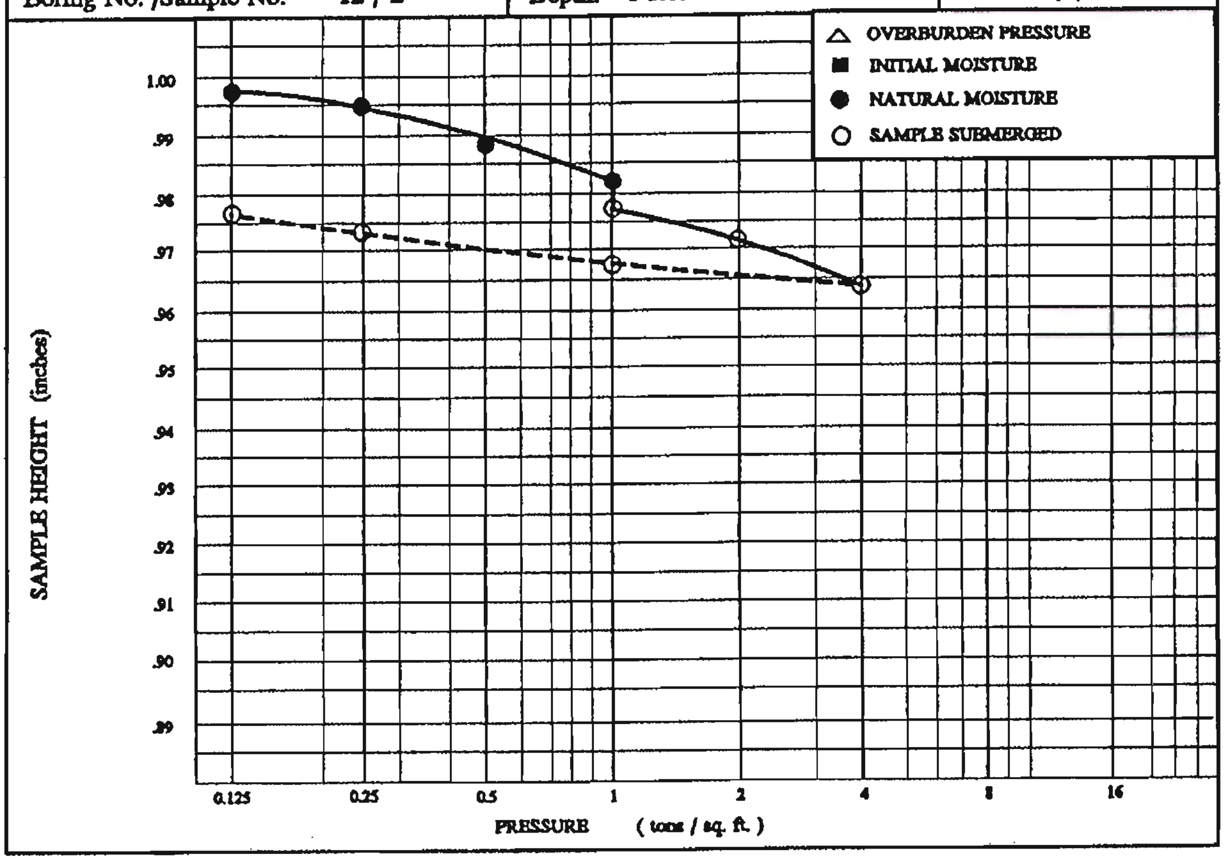
CONSOLIDATION TEST - PRESSURE CURVES

Boring No. /Sample No. 2 / 3	Depth: 5 feet	Date: 8/8/94



CONSOLIDATION TEST - PRESSURE CURVES

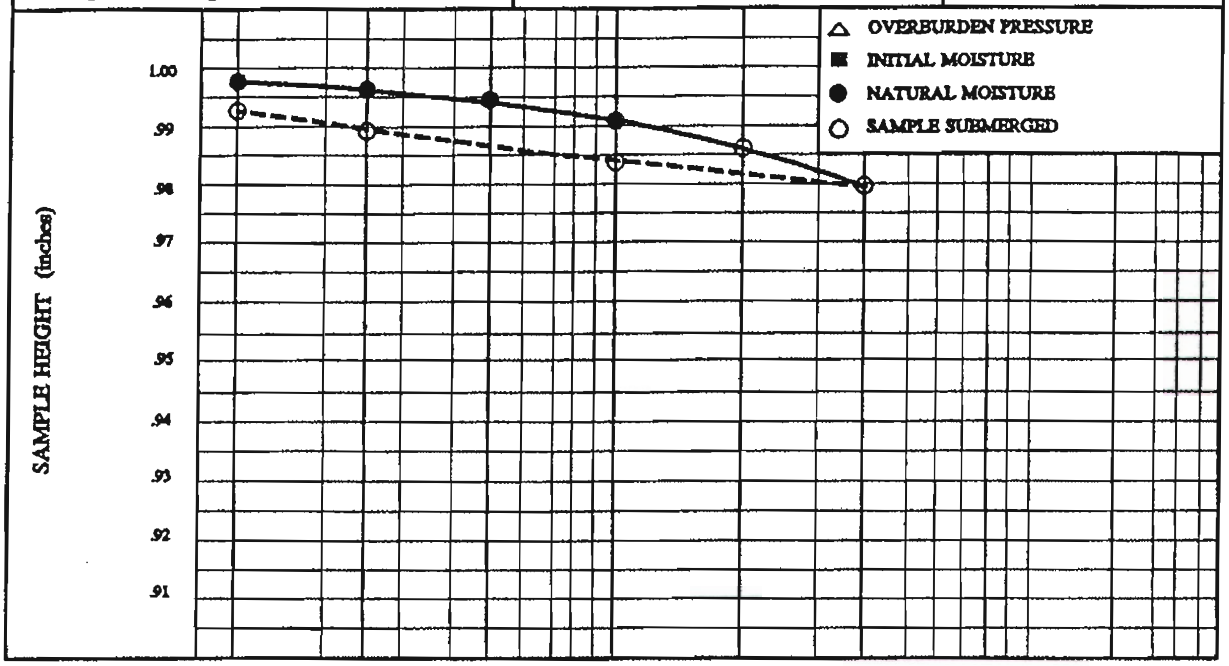
Boring No. /Sample No. 12 / 2	Depth: 5 feet	Date: 8/8/94



AGRA Earth & Environmental

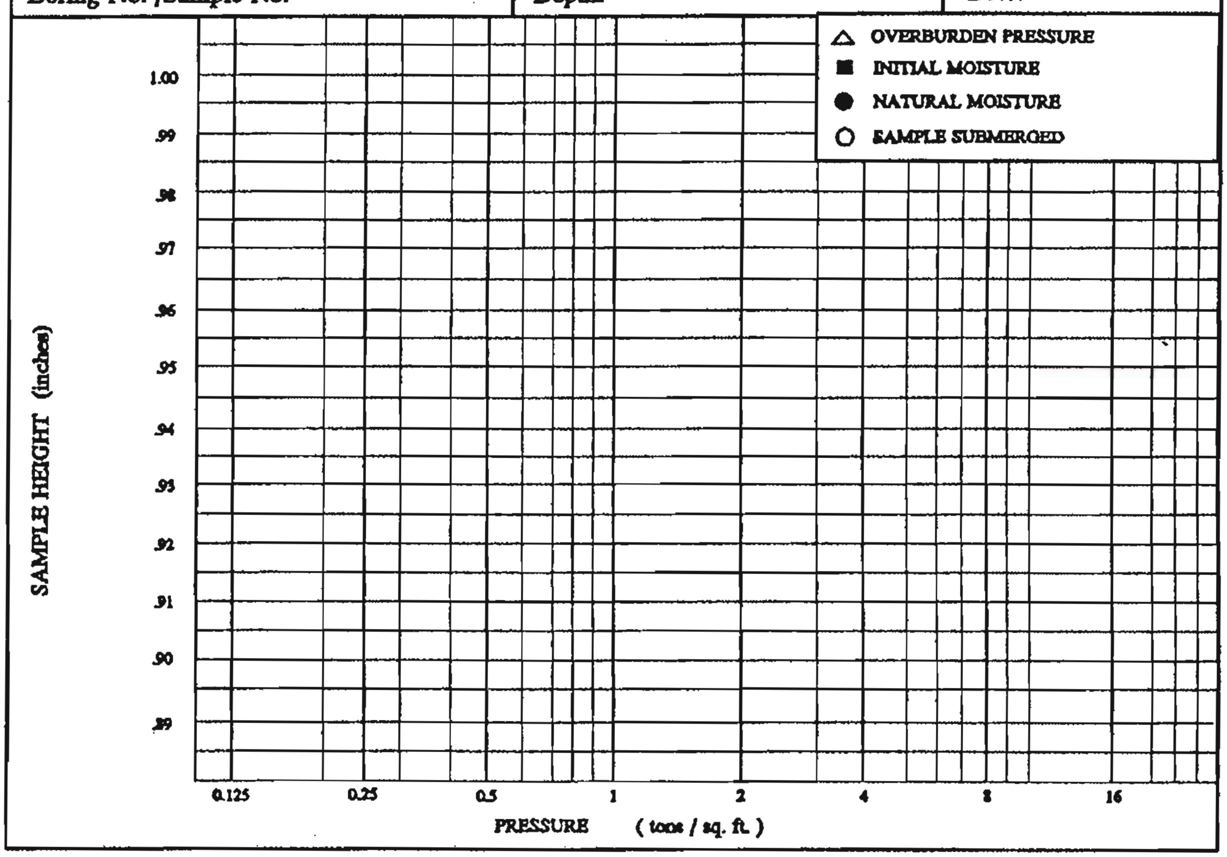
CONSOLIDATION TEST - PRESSURE CURVES

Boring No. /Sample No. 12 / 3	Depth: 10 feet	Date: 8/8/94



CONSOLIDATION TEST - PRESSURE CURVES

Boring No. /Sample No.	Depth:	Date:

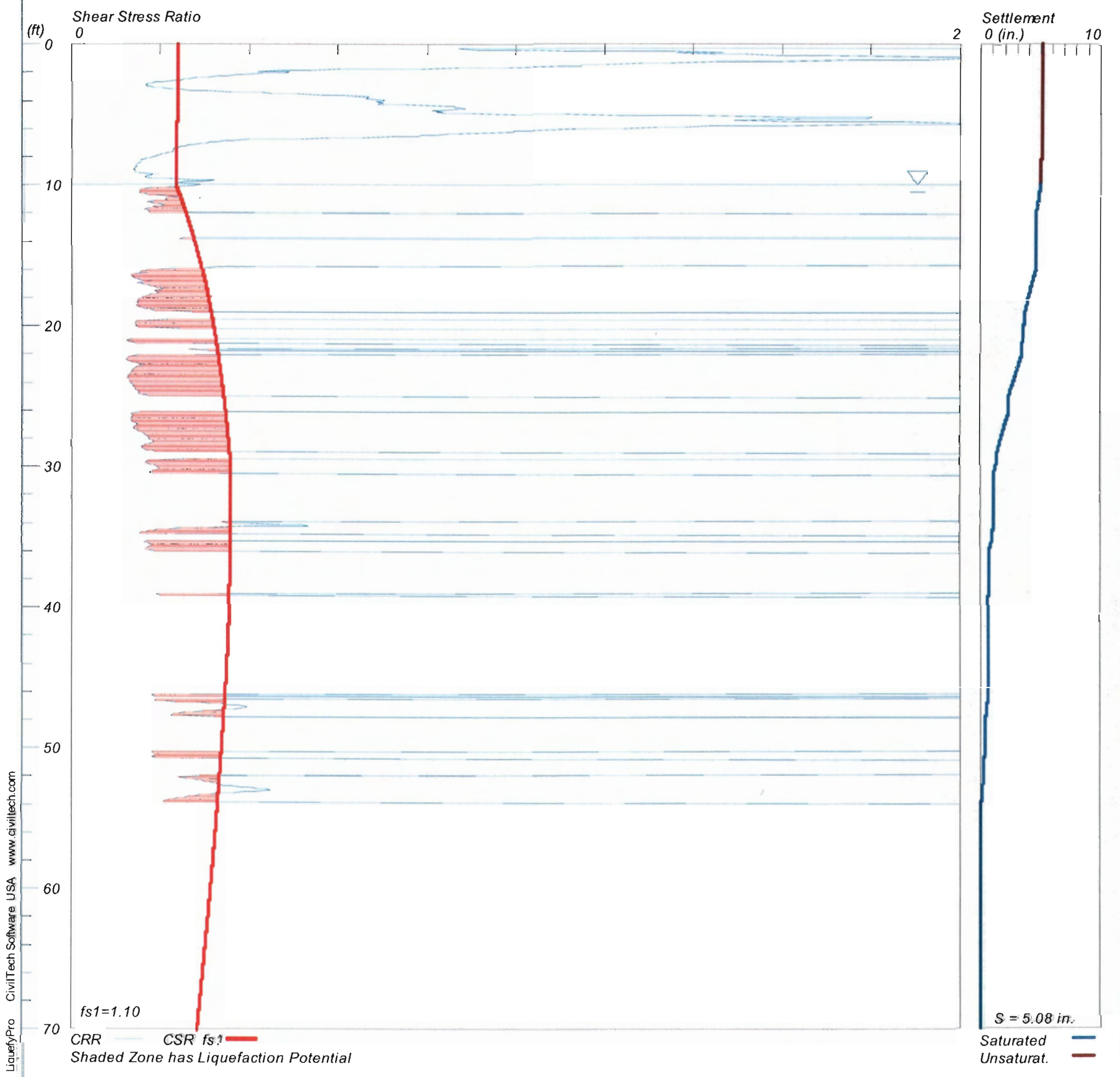


Seismic Induced Settlement

San Elijo Water Reclamation Facility

Hole No.=CPT-1 Water Depth=10 ft Surface Elev.=30

Magnitude=6.6
Acceleration=0.335g



Leighton

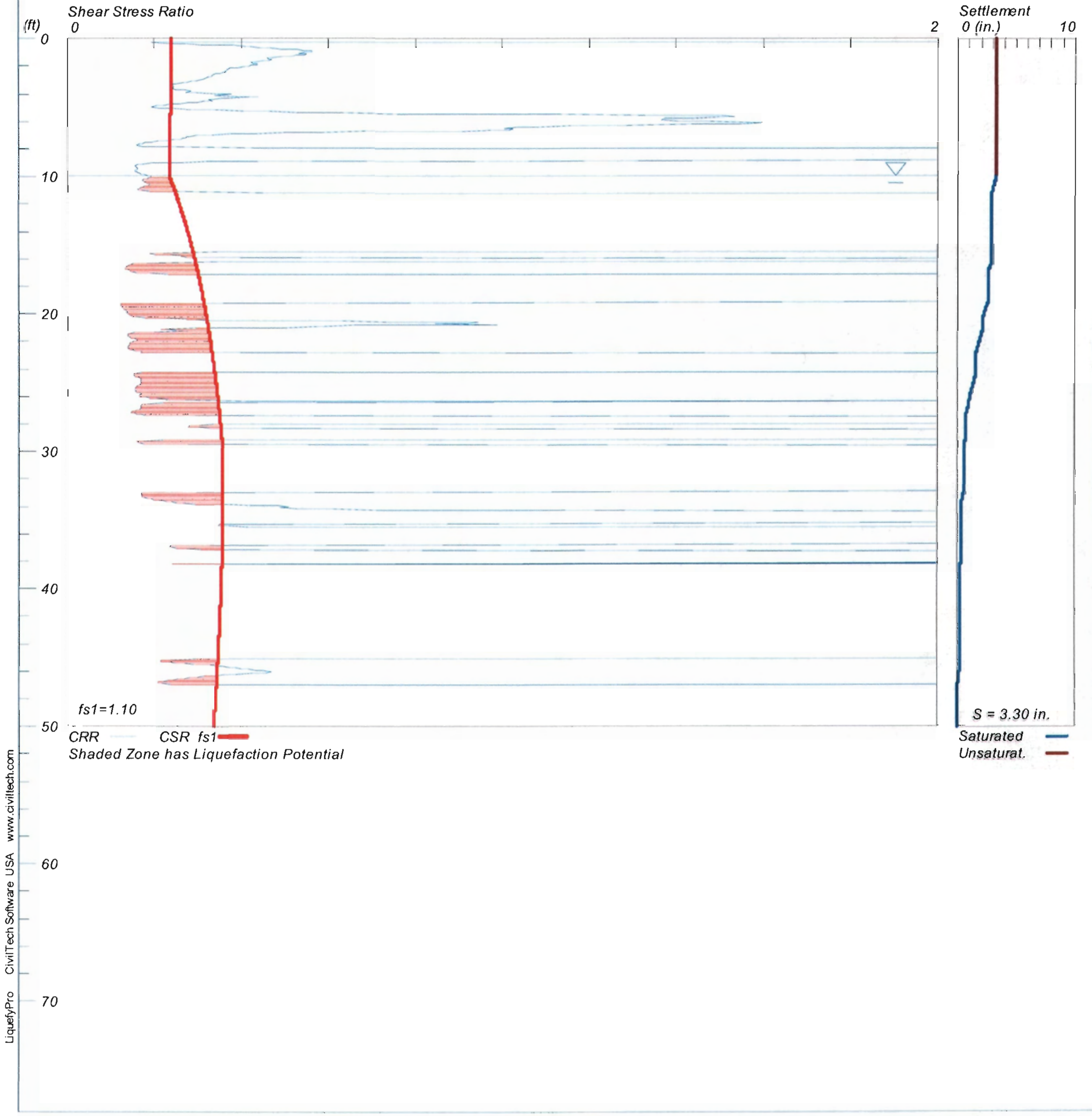
602835-001

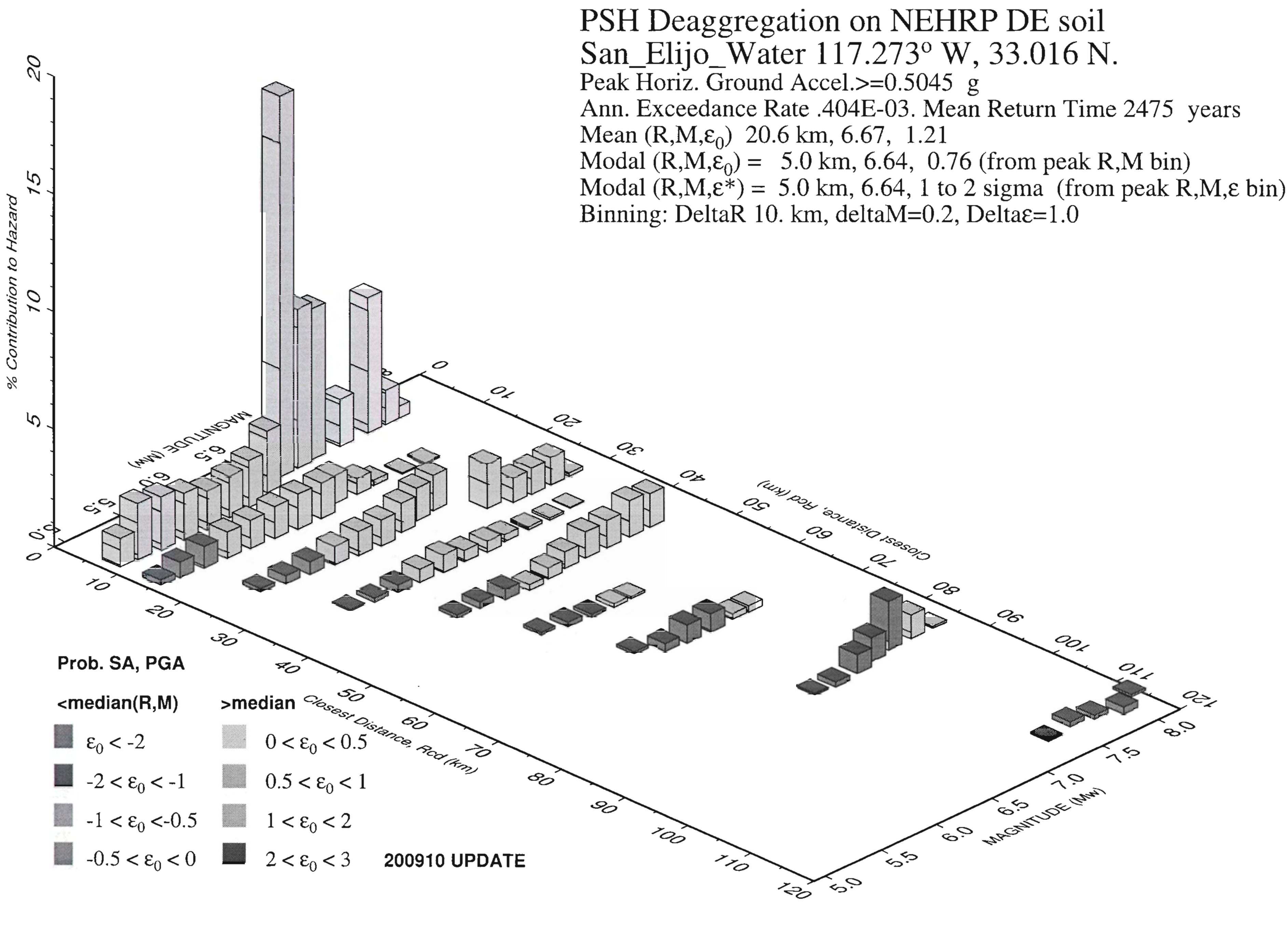
Seismic Induced Settlement

San Elijo Water Reclamation Facility

Hole No.=CPT-2 Water Depth=10 ft Surface Elev.=30

Magnitude=6.6
Acceleration=0.335g





GMT 2010 Mar 2 23:39:49 Distance (R), magnitude (M), epsilon (E0,E) deaggregation for a site on soil with average vs= 180. m/s top 30 m. USGS CGHT PSHA2008 UPDATE Bins with lt 0.05% contrib. omitted

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

JOB NUMBER: 602835-001

DATE: 03-23-2010

JOB NAME: SEWWTP

CALCULATION NAME: Test Run Analysis

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

SITE COORDINATES:
   SITE LATITUDE: 33.0158
   SITE LONGITUDE: 117.2737

SEARCH RADIUS: 100 mi

ATTENUATION RELATION: 14) Campbell & Bozorgnia (1997 Rev.) - Alluvium
   UNCERTAINTY (M=Median, S=Sigma): M   Number of Sigmas: 0.0
   DISTANCE MEASURE: cdist
   SCOND: 0
   Basement Depth: 5.00 km   Campbell SSR: 0   Campbell SHR: 0
   COMPUTE PEAK HORIZONTAL ACCELERATION

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

MINIMUM DEPTH VALUE (km): 3.0

EQFAULT SUMMARY

DETERMINISTIC SITE PARAMETERS

Page 1

ABBREVIATED FAULT NAME	APPROXIMATE DISTANCE mi (km)	ESTIMATED MAX. EARTHQUAKE EVENT MAXIMUM EARTHQUAKE MAG.(Mw)	PEAK SITE ACCEL. g	EST. SITE INTENSITY MOD.MERC.
ROSE CANYON	3.4(5.4)	7.2	0.479	X
NEWPORT-INGLEWOOD (Offshore)	13.4(21.5)	7.1	0.241	IX
CORONADO BANK	17.3(27.8)	7.6	0.261	IX
ELSINORE (JULIAN)	29.1(46.8)	7.1	0.104	VII
ELSINORE (TEMECULA)	29.2(47.0)	6.8	0.080	VII
EARTHQUAKE VALLEY	41.8(67.2)	6.5	0.039	V
PALOS VERDES	42.8(68.9)	7.3	0.076	VII
ELSINORE (GLEN IVY)	43.6(70.1)	6.8	0.048	VI
SAN JOAQUIN HILLS	45.0(72.4)	6.6	0.039	V
SAN JACINTO-ANZA	51.8(83.4)	7.2	0.055	VI
ELSINORE (COYOTE MOUNTAIN)	52.9(85.2)	6.8	0.038	V
SAN JACINTO-COYOTE CREEK	53.9(86.7)	6.6	0.031	V
SAN JACINTO-SAN JACINTO VALLEY	54.2(87.2)	6.9	0.040	V
NEWPORT-INGLEWOOD (L.A.Basin)	55.6(89.4)	7.1	0.046	VI
CHINO-CENTRAL AVE. (Elsinore)	58.1(93.5)	6.7	0.030	V
WHITTIER	62.0(99.8)	6.8	0.031	V
SAN JACINTO - BORREGO	63.9(102.8)	6.6	0.025	V
SAN JACINTO-SAN BERNARDINO	69.2(111.4)	6.7	0.024	V
PUENTE HILLS BLIND THRUST	71.8(115.6)	7.1	0.030	V
SAN ANDREAS - San Bernardino M-1	73.1(117.7)	7.5	0.046	VI
SAN ANDREAS - Whole M-1a	73.1(117.7)	8.0	0.071	VI
SAN ANDREAS - SB-Coach. M-1b-2	73.1(117.7)	7.7	0.055	VI
SAN ANDREAS - SB-Coach. M-2b	73.1(117.7)	7.7	0.055	VI
SAN ANDREAS - Coachella M-1c-5	78.1(125.7)	7.2	0.032	V
SUPERSTITION MTN. (San Jacinto)	78.4(126.1)	6.6	0.019	IV
PINTO MOUNTAIN	78.7(126.6)	7.2	0.032	V
SAN JOSE	79.0(127.1)	6.4	0.015	IV
BURNT MTN.	81.6(131.3)	6.5	0.016	IV
ELMORE RANCH	82.3(132.4)	6.6	0.018	IV
CUCAMONGA	82.3(132.5)	6.9	0.021	IV
SIERRA MADRE	82.6(133.0)	7.2	0.027	V
LAGUNA SALADA	83.1(133.8)	7.0	0.025	V
SUPERSTITION HILLS (San Jacinto)	83.2(133.9)	6.6	0.017	IV
NORTH FRONTAL FAULT ZONE (West)	84.1(135.4)	7.2	0.026	V
EUREKA PEAK	84.8(136.4)	6.4	0.014	IV
CLEGHORN	87.0(140.0)	6.5	0.015	IV
UPPER ELYSIAN PARK BLIND THRUST	87.1(140.1)	6.4	0.013	III
NORTH FRONTAL FAULT ZONE (East)	87.8(141.3)	6.7	0.016	IV
SAN ANDREAS - 1857 Rupture M-2a	89.0(143.2)	7.8	0.046	VI
SAN ANDREAS - Cho-Moj M-1b-1	89.0(143.2)	7.8	0.046	VI

DETERMINISTIC SITE PARAMETERS

Page 2

ABBREVIATED FAULT NAME	APPROXIMATE DISTANCE mi (km)	ESTIMATED MAX. EARTHQUAKE EVENT MAXIMUM EARTHQUAKE MAG.(Mw)	PEAK SITE ACCEL. g	EST. SITE INTENSITY MOD.MERC.
SAN ANDREAS - Mojave M-1c-3	89.0(143.2)	7.4	0.033	V
RAYMOND	90.0(144.8)	6.5	0.014	III
CLAMSHELL-SAWPIT	92.5(148.8)	6.5	0.013	III
LANDERS	93.1(149.8)	7.3	0.028	V
VERDUGO	93.1(149.9)	6.9	0.018	IV
BRAWLEY SEISMIC ZONE	93.6(150.7)	6.4	0.012	III
HOLLYWOOD	94.9(152.7)	6.4	0.012	III
HELENDALE - S. LOCKHARDT	96.5(155.3)	7.3	0.027	V
SANTA MONICA	98.6(158.7)	6.6	0.013	III
IMPERIAL	99.2(159.6)	7.0	0.020	IV
LENWOOD-LOCKHART-OLD WOMAN SPRGS	99.6 (160.3)	7.5	0.031	V

-END OF SEARCH- 51 FAULTS FOUND WITHIN THE SPECIFIED SEARCH RADIUS.

THE ROSE CANYON FAULT IS CLOSEST TO THE SITE.
IT IS ABOUT 3.4 MILES (5.4 km) AWAY.

LARGEST MAXIMUM-EARTHQUAKE SITE ACCELERATION: 0.4793 g

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

JOB NUMBER: 602835-001

DATE: 03-23-2010

JOB NAME: SEWWTP

CALCULATION NAME: Test Run Analysis

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

SITE COORDINATES:
   SITE LATITUDE: 33.0158
   SITE LONGITUDE: 117.2737

SEARCH RADIUS: 100 mi

ATTENUATION RELATION: 14) Campbell & Bozorgnia (1997 Rev.) - Alluvium
   UNCERTAINTY (M=Median, S=Sigma): S   Number of Sigmas: 1.0
   DISTANCE MEASURE: cdist
   SCOND: 0
   Basement Depth: 5.00 km   Campbell SSR: 0   Campbell SHR: 0
   COMPUTE PEAK HORIZONTAL ACCELERATION

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

MINIMUM DEPTH VALUE (km): 3.0

EQFAULT SUMMARY

DETERMINISTIC SITE PARAMETERS

Page 1

ABBREVIATED FAULT NAME	APPROXIMATE DISTANCE mi (km)	ESTIMATED MAX. EARTHQUAKE EVENT: MAXIMUM EARTHQUAKE MAG.(Mw)	PEAK SITE ACCEL. g	EST. SITE INTENSITY MOD.MERC.
ROSE CANYON	3.4(5.4)	7.2	0.708	XI
NEWPORT-INGLEWOOD (Offshore)	13.4(21.5)	7.1	0.356	IX
CORONADO BANK	17.3(27.8)	7.6	0.386	X
ELSINORE (JULIAN)	29.1(46.8)	7.1	0.170	VIII
ELSINORE (TEMECULA)	29.2(47.0)	6.8	0.136	VIII
EARTHQUAKE VALLEY	41.8(67.2)	6.5	0.068	VI
PALOS VERDES	42.8(68.9)	7.3	0.130	VIII
ELSINORE (GLEN IVY)	43.6(70.1)	6.8	0.084	VII
SAN JOAQUIN HILLS	45.0(72.4)	6.6	0.068	VI
SAN JACINTO-ANZA	51.8(83.4)	7.2	0.095	VII
ELSINORE (COYOTE MOUNTAIN)	52.9(85.2)	6.8	0.065	VI
SAN JACINTO-COYOTE CREEK	53.9(86.7)	6.6	0.053	VI
SAN JACINTO-SAN JACINTO VALLEY	54.2(87.2)	6.9	0.069	VI
NEWPORT-INGLEWOOD (L.A.Basin)	55.6(89.4)	7.1	0.080	VII
CHINO-CENTRAL AVE. (Elsinore)	58.1(93.5)	6.7	0.052	VI
WHITTIER	62.0(99.8)	6.8	0.053	VI
SAN JACINTO - BORREGO	63.9(102.8)	6.6	0.043	VI
SAN JACINTO-SAN BERNARDINO	69.2(111.4)	6.7	0.042	VI
PUENTE HILLS BLIND THRUST	71.8(115.6)	7.1	0.052	VI
SAN ANDREAS - San Bernardino M-1	73.1(117.7)	7.5	0.080	VII
SAN ANDREAS - Whole M-1a	73.1(117.7)	8.0	0.122	VII
SAN ANDREAS - SB-Coach. M-1b-2	73.1(117.7)	7.7	0.095	VII
SAN ANDREAS - SB-Coach. M-2b	73.1(117.7)	7.7	0.095	VII
SAN ANDREAS - Coachella M-1c-5	78.1(125.7)	7.2	0.056	VI
SUPERSTITION MTN. (San Jacinto)	78.4(126.1)	6.6	0.033	V
PINTO MOUNTAIN	78.7(126.6)	7.2	0.055	VI
SAN JOSE	79.0(127.1)	6.4	0.026	V
BURNT MTN.	81.6(131.3)	6.5	0.028	V
ELMORE RANCH	82.3(132.4)	6.6	0.031	V
CUCAMONGA	82.3(132.5)	6.9	0.037	V
SIERRA MADRE	82.6(133.0)	7.2	0.046	VI
LAGUNA SALADA	83.1(133.8)	7.0	0.043	VI
SUPERSTITION HILLS (San Jacinto)	83.2(133.9)	6.6	0.030	V
NORTH FRONTAL FAULT ZONE (West)	84.1(135.4)	7.2	0.045	VI
EUREKA PEAK	84.8(136.4)	6.4	0.025	V
CLEGHORN	87.0(140.0)	6.5	0.026	V
UPPER ELYSIAN PARK BLIND THRUST	87.1(140.1)	6.4	0.023	IV
NORTH FRONTAL FAULT ZONE (East)	87.8(141.3)	6.7	0.029	V
SAN ANDREAS - 1857 Rupture M-2a	89.0(143.2)	7.8	0.080	VII
SAN ANDREAS - Cho-Moj M-1b-1	89.0(143.2)	7.8	0.080	VII

DETERMINISTIC SITE PARAMETERS

Page 2

ABBREVIATED FAULT NAME	APPROXIMATE DISTANCE mi (km)	ESTIMATED MAX. EARTHQUAKE EVENT: MAXIMUM EARTHQUAKE MAG.(Mw)	PEAK SITE ACCEL. g	EST. SITE INTENSITY MOD.MERC.
SAN ANDREAS - Mojave M-1c-3	89.0(143.2)	7.4	0.056	VI
RAYMOND	90.0(144.8)	6.5	0.024	IV
CLAMSHELL-SAWPIT	92.5(148.8)	6.5	0.023	IV
LANDERS	93.1(149.8)	7.3	0.049	VI
VERDUGO	93.1(149.9)	6.9	0.031	V
BRAWLEY SEISMIC ZONE	93.6(150.7)	6.4	0.022	IV
HOLLYWOOD	94.9(152.7)	6.4	0.020	IV
HELENDALE - S. LOCKHARDT	96.5(155.3)	7.3	0.046	VI
SANTA MONICA	98.6(158.7)	6.6	0.022	IV
IMPERIAL	99.2(159.6)	7.0	0.034	V
LENWOOD-LOCKHART-OLD WOMAN SPRGS	99.6 (160.3)	7.5	0.053	VI

-END OF SEARCH- 51 FAULTS FOUND WITHIN THE SPECIFIED SEARCH RADIUS.

THE ROSE CANYON FAULT IS CLOSEST TO THE SITE.
IT IS ABOUT 3.4 MILES (5.4 km) AWAY.

LARGEST MAXIMUM-EARTHQUAKE SITE ACCELERATION: 0.7079 g

1.0 General

1.1 Intent

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

1.2 The Geotechnical Consultant of Record

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

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

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

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

1.3 The Earthwork Contractor

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

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

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

2.0 Preparation of Areas to be Filled

2.1 Clearing and Grubbing

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

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

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

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

2.2 Processing

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

2.3 Overexcavation

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

2.4 Benching

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

2.5 Evaluation/Acceptance of Fill Areas

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

3.0 Fill Material

3.1 General

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

3.2 Oversize

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

3.3 Import

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

4.0 Fill Placement and Compaction

4.1 Fill Layers

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

4.2 Fill Moisture Conditioning

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

4.3 Compaction of Fill

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

4.4 Compaction of Fill Slopes

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

4.5 Compaction Testing

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

4.6 Frequency of Compaction Testing

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

4.7 Compaction Test Locations

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

5.0 Subdrain Installation

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

6.0 Excavation

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

7.0 Trench Backfills

7.1 Safety

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

7.2 Bedding and Backfill

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

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

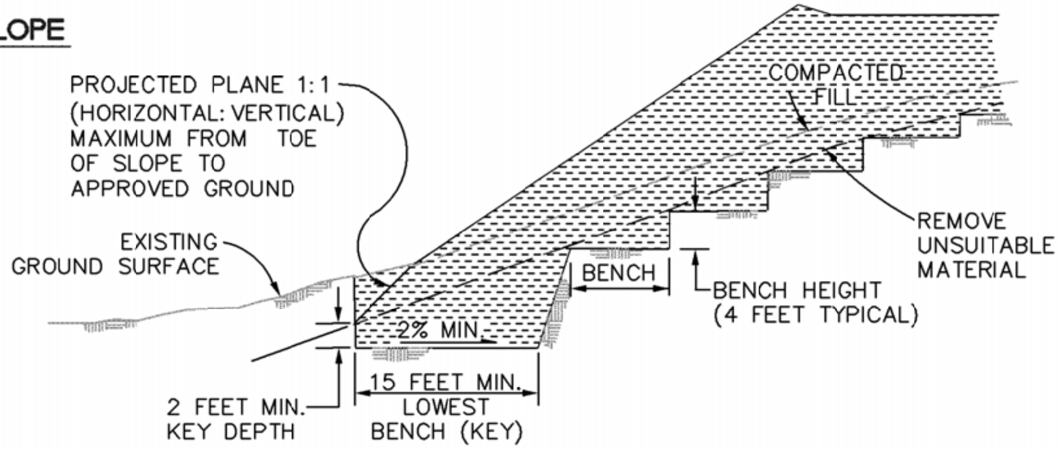
7.3 Lift Thickness

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

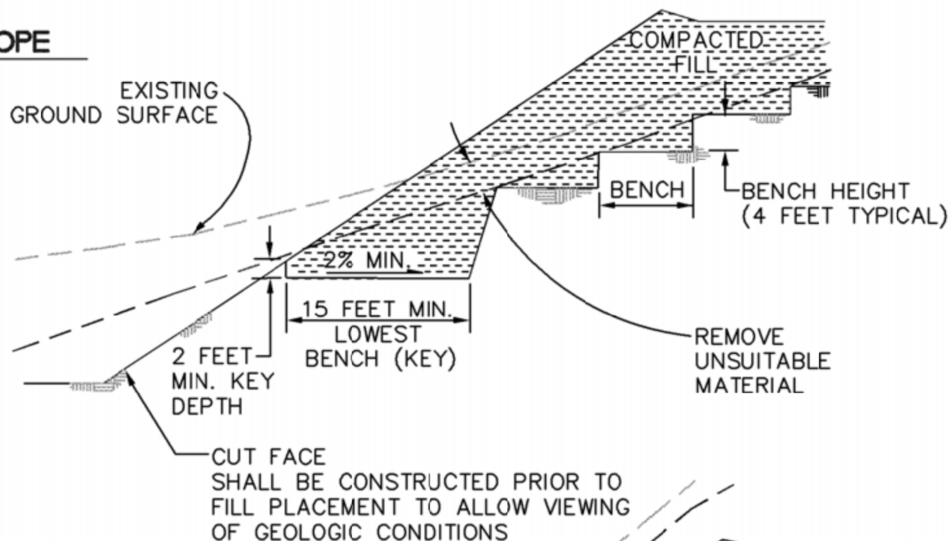
7.4 Observation and Testing

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

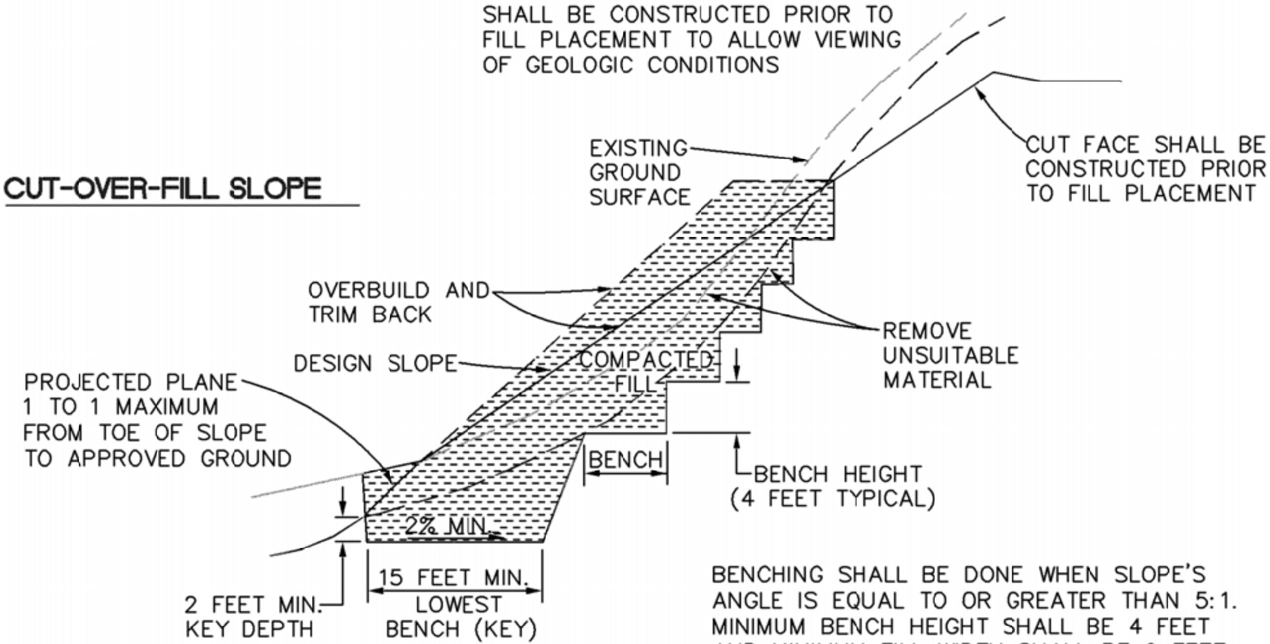
FILL SLOPE



FILL-OVER-CUT SLOPE



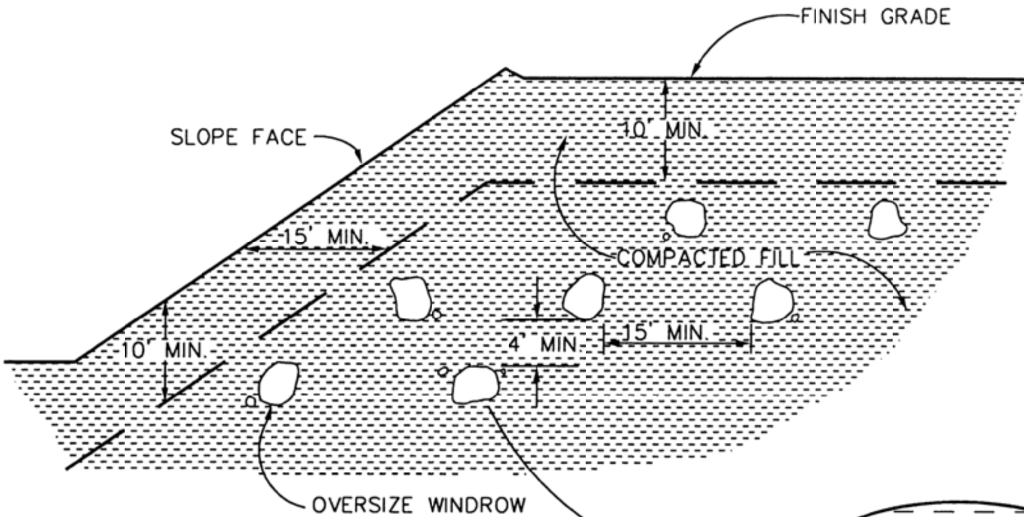
CUT-OVER-FILL SLOPE

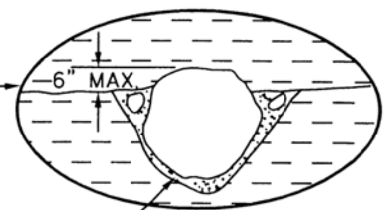


BENCHING SHALL BE DONE WHEN SLOPE'S ANGLE IS EQUAL TO OR GREATER THAN 5:1. MINIMUM BENCH HEIGHT SHALL BE 4 FEET AND MINIMUM FILL WIDTH SHALL BE 9 FEET.

KEYING AND BENCHING

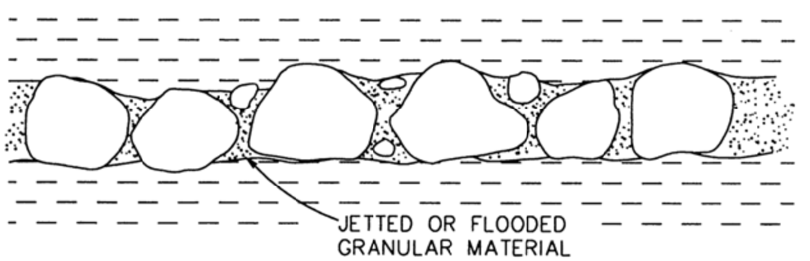
GENERAL EARTHWORK AND GRADING SPECIFICATIONS
STANDARD DETAIL A





DETAIL

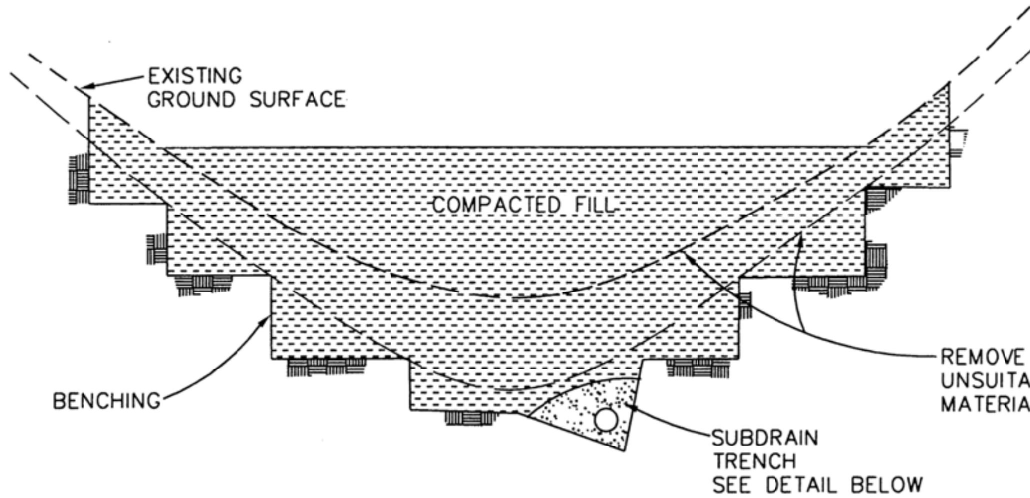
* OVERSIZE ROCK IS LARGER THAN 8 INCHES IN LARGEST DIMENSION.
* EXCAVATE A TRENCH IN THE COMPACTED FILL DEEP ENOUGH TO BURY ALL THE ROCK.
* BACKFILL WITH GRANULAR SOIL JETTED OR FLOODED IN PLACE TO FILL ALL THE VOIDS.
* DO NOT BURY ROCK WITHIN 10 FEET OF FINISH GRADE.
* WINDROW OF BURIED ROCK SHALL BE PARALLEL TO THE FINISHED SLOPE.

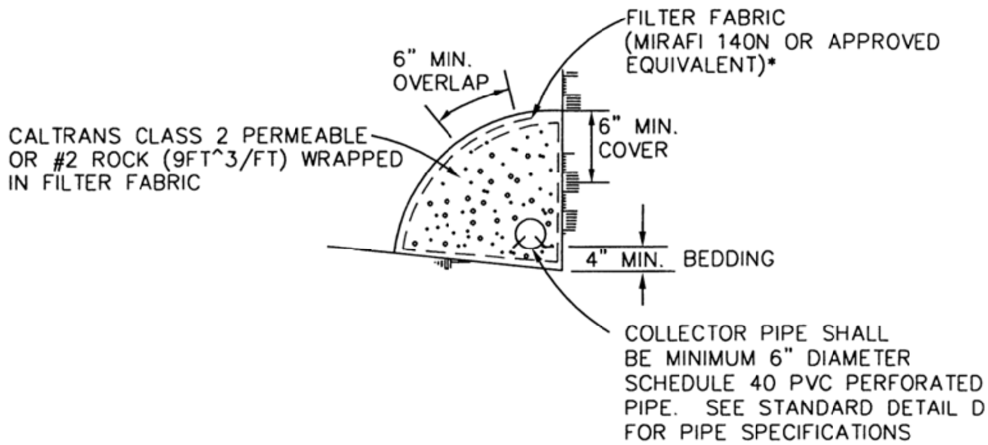


TYPICAL PROFILE ALONG WINDROW

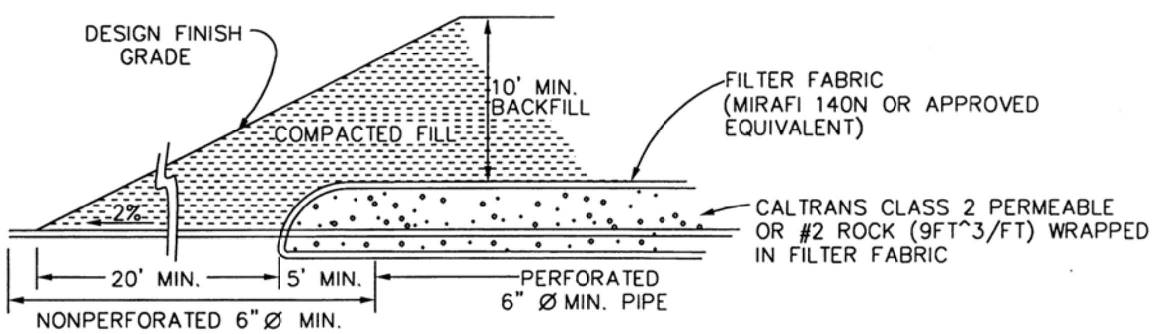
OVERSIZE ROCK DISPOSAL

GENERAL EARTHWORK AND GRADING SPECIFICATIONS
STANDARD DETAIL B





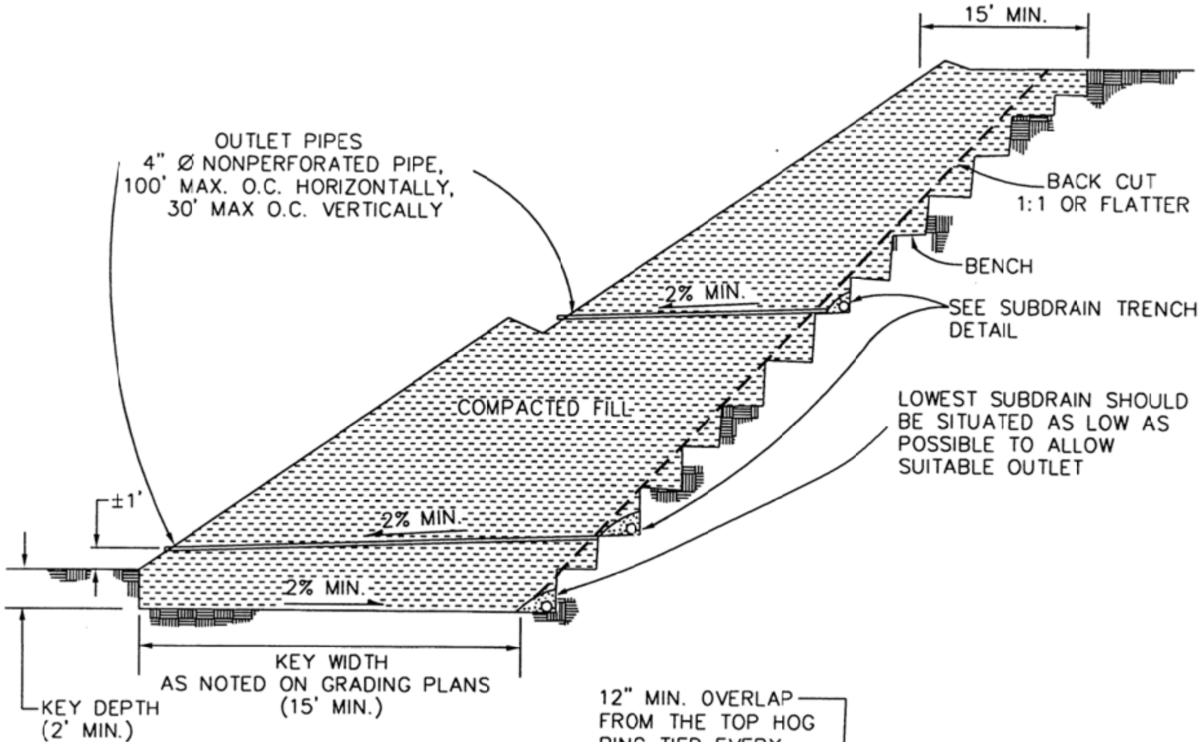
SUBDRAIN DETAIL

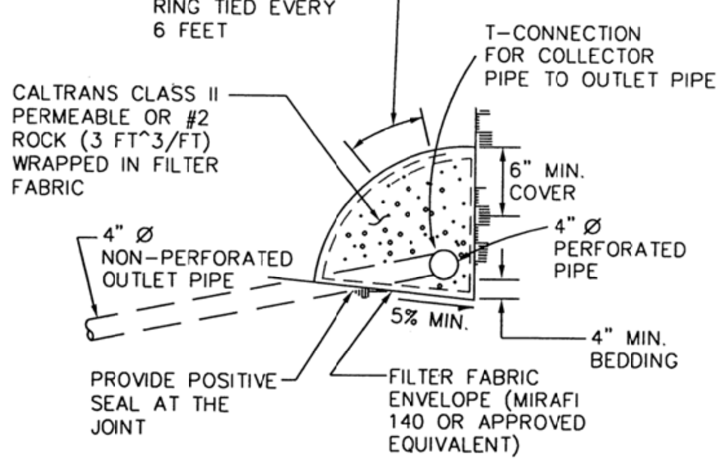


DETAIL OF CANYON SUBDRAIN OUTLET

CANYON SUBDRAINS

GENERAL EARTHWORK AND
GRADING SPECIFICATIONS
STANDARD DETAIL C





SUBDRAIN TRENCH DETAIL

SUBDRAIN INSTALLATION – subdrain collector pipe shall be installed with perforation down or, unless otherwise designated by the geotechnical consultant. Outlet pipes shall be non-perforated pipe. The subdrain pipe shall have at least 8 perforations uniformly spaced per foot. Perforation shall be 1/4" to 1/2" if drill holes are used. All subdrain pipes shall have a gradient of at least 2% towards the outlet.

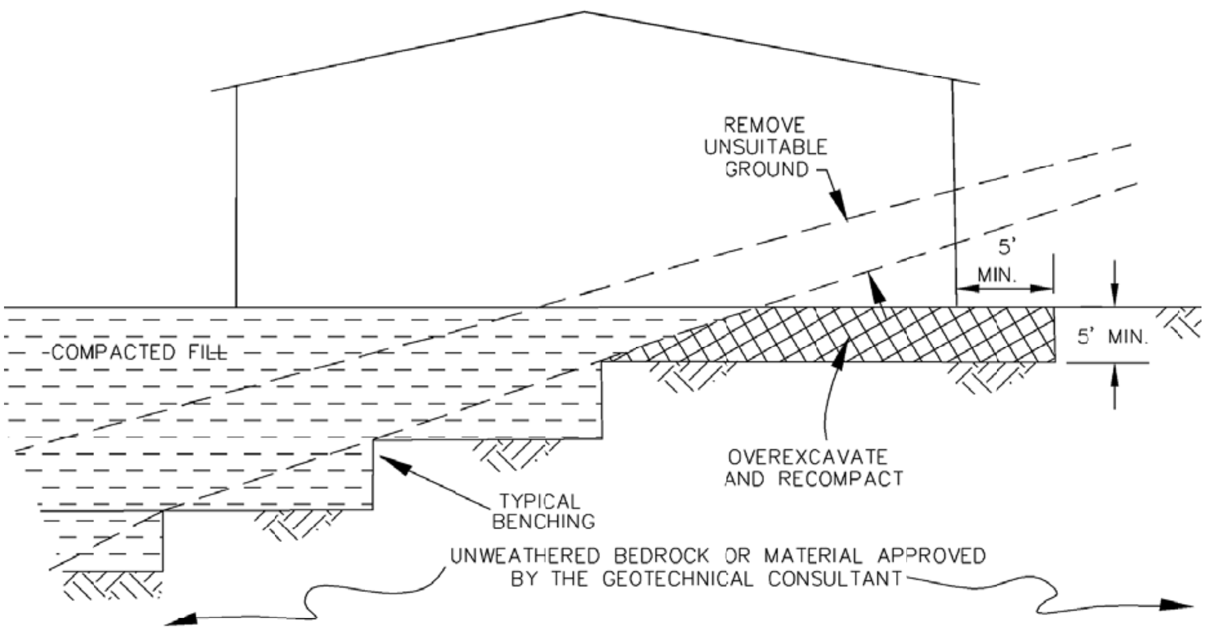
SUBDRAIN PIPE – Subdrain pipe shall be ASTM D2751, SDR 23.5 or ASTM D1527, Schedule 40, or ASTM D3034, SDR 23.5, Schedule 40 Polyvinyl Chloride Plastic (PVC) pipe.

All outlet pipe shall be placed in a trench no wider than twice the subdrain pipe.

BUTTRESS OR REPLACEMENT FILL SUBDRAINS

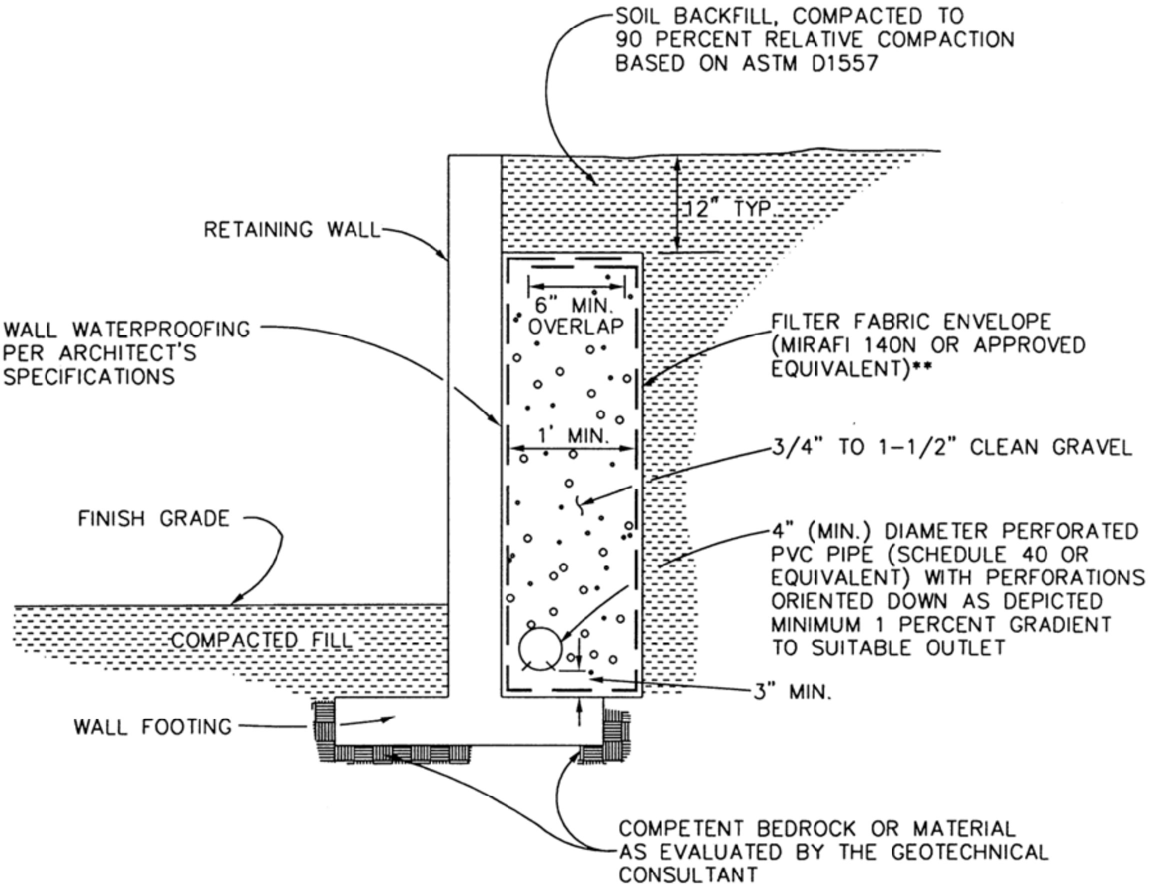
GENERAL EARTHWORK AND
GRADING SPECIFICATIONS
STANDARD DETAIL D

CUT-FILL TRANSITION LOT OVEREXCAVATION



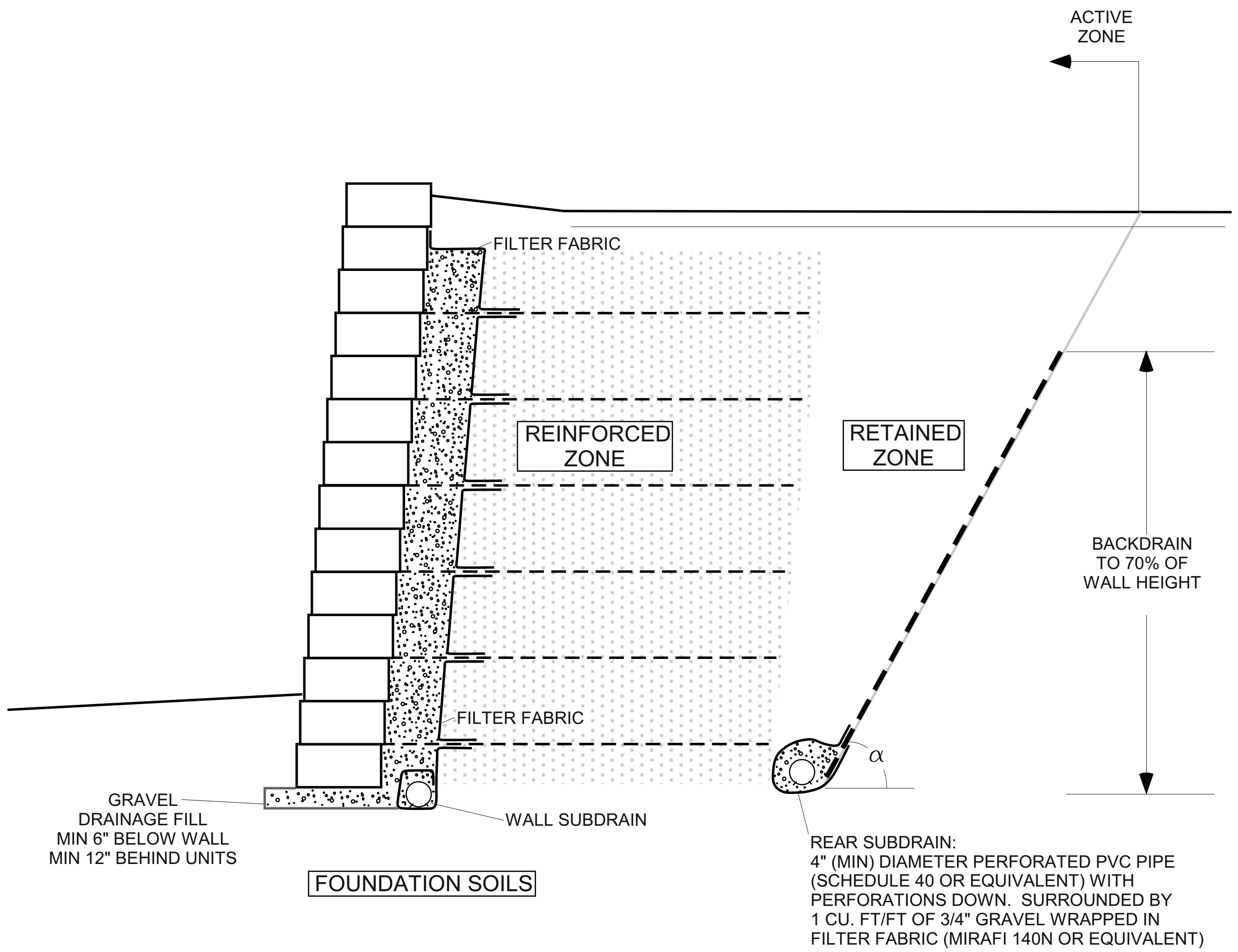
TRANSITION LOT FILLS

GENERAL EARTHWORK AND
GRADING SPECIFICATIONS
STANDARD DETAIL E



NOTE: UPON REVIEW BY THE GEOTECHNICAL CONSULTANT, COMPOSITE DRAINAGE PRODUCTS SUCH AS MIRADRAIN OR J-DRAIN MAY BE USED AS AN ALTERNATIVE TO GRAVEL OR CLASS 2 PERMEABLE MATERIAL. INSTALLATION SHOULD BE PERFORMED IN ACCORDANCE WITH MANUFACTURER'S SPECIFICATIONS.

RETAINING WALL DRAINAGE	GENERAL EARTHWORK AND GRADING SPECIFICATIONS STANDARD DETAIL F



NOTES:

1) MATERIAL GRADATION AND PLASTICITY

REINFORCED ZONE:

SIEVE SIZE	% PASSING
1 INCH	100
NO. 4	20-100
NO. 40	0-60
NO. 200	0-35

FOR WALL HEIGHT < 10 FEET, PLASTICITY INDEX < 20
FOR WALL HEIGHT 10 TO 20 FEET, PLASTICITY INDEX < 10
FOR TIERED WALLS, USE COMBINED WALL HEIGHTS
WALL DESIGNER TO REQUEST SITE-SPECIFIC CRITERIA FOR WALL HEIGHT > 20 FEET

GRAVEL DRAINAGE FILL:

SIEVE SIZE	% PASSING
1 INCH	100
3/4 INCH	75-100
NO. 4	0-60
NO. 40	0-50
NO. 200	0-5

2) CONTRACTOR TO USE SOILS WITHIN THE RETAINED AND REINFORCED ZONES THAT MEET THE STRENGTH REQUIREMENTS OF WALL DESIGN.

3) GEOGRID REINFORCEMENT TO BE DESIGNED BY WALL DESIGNER CONSIDERING INTERNAL, EXTERNAL, AND COMPOUND STABILITY.

3) GEOGRID TO BE PRETENSIONED DURING INSTALLATION.

4) IMPROVEMENTS WITHIN THE ACTIVE ZONE ARE SUSCEPTIBLE TO POST-CONSTRUCTION SETTLEMENT. ANGLE $\alpha = 45+\phi/2$, WHERE ϕ IS THE FRICTION ANGLE OF THE MATERIAL IN THE RETAINED ZONE.

5) BACKDRAIN SHOULD CONSIST OF J-DRAIN 302 (OR EQUIVALENT) OR 6-INCH THICK DRAINAGE FILL WRAPPED IN FILTER FABRIC. PERCENT COVERAGE OF BACKDRAIN TO BE PER GEOTECHNICAL REVIEW.

SEGMENTAL RETAINING WALLS

GENERAL EARTHWORK AND
GRADING SPECIFICATIONS
STANDARD DETAIL G

Important Information about Your

Geotechnical Engineering Report

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

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

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

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

Read the Full Report

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

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

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

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

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

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

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

Subsurface Conditions Can Change

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

Most Geotechnical Findings Are Professional Opinions

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

A Report's Recommendations Are *Not* Final

Do not overrely on the construction recommendations included in your report. *Those recommendations are not final,* because geotechnical engineers develop them principally from judgment and opinion. Geotechnical engineers can finalize their recommendations only by observing actual subsurface conditions revealed during construction. *The geotechnical engineer who developed your report cannot assume responsibility or liability for the report's recommendations if that engineer does not perform construction observation.*

A Geotechnical Engineering Report Is Subject to Misinterpretation

Other design team members' misinterpretation of geotechnical engineering reports has resulted in costly problems. Lower that risk by having your geotechnical engineer confer with appropriate members of the design team after submitting the report. Also retain your geotechnical engineer to review pertinent elements of the design team's plans and specifications. Contractors can also misinterpret a geotechnical engineering report. Reduce that risk by having your geotechnical engineer participate in prebid and preconstruction conferences, and by providing construction observation.

Do Not Redraw the Engineer's Logs

Geotechnical engineers prepare final boring and testing logs based upon their interpretation of field logs and laboratory data. To prevent errors or omissions, the logs included in a geotechnical engineering report should *never* be redrawn for inclusion in architectural or other design drawings. Only photographic or electronic reproduction is acceptable, *but recognize that separating logs from the report can elevate risk.*

Give Contractors a Complete Report and Guidance

Some owners and design professionals mistakenly believe they can make contractors liable for unanticipated subsurface conditions by limiting what they provide for bid preparation. To help prevent costly problems, give contractors the complete geotechnical engineering report, *but* preface it with a clearly written letter of transmittal. In that letter, advise contractors that the report was not prepared for purposes of bid development and that the report's accuracy is limited; encourage them to confer with the geotechnical engineer who prepared the report (a modest fee may be required) and/or to conduct additional study to obtain the specific types of information they need or prefer. A prebid conference can also be valuable. *Be sure contractors have sufficient time* to perform additional study. Only then might you be in a position to give contractors the best information available to you, while requiring them to at least share some of the financial responsibilities stemming from unanticipated conditions.

Read Responsibility Provisions Closely

Some clients, design professionals, and contractors do not recognize that geotechnical engineering is far less exact than other engineering disciplines. This lack of understanding has created unrealistic expectations that have led to disappointments, claims, and disputes. To help reduce the risk of such outcomes, geotechnical engineers commonly include a variety of explanatory provisions in their reports. Sometimes labeled "limitations" many of these provisions indicate where geotechnical engineers' responsibilities begin and end, to help others recognize their own responsibilities and risks. *Read these provisions closely.* Ask questions. Your geotechnical engineer should respond fully and frankly.

Geoenvironmental Concerns Are Not Covered

The equipment, techniques, and personnel used to perform a *geoenvironmental* study differ significantly from those used to perform a *geotechnical* study. For that reason, a geotechnical engineering report does not usually relate any geoenvironmental findings, conclusions, or recommendations; e.g., about the likelihood of encountering underground storage tanks or regulated contaminants. *Unanticipated environmental problems have led to numerous project failures.* If you have not yet obtained your own geoenvironmental information, ask your geotechnical consultant for risk management guidance. *Do not rely on an environmental report prepared for someone else.*

Obtain Professional Assistance To Deal with Mold

Diverse strategies can be applied during building design, construction, operation, and maintenance to prevent significant amounts of mold from growing on indoor surfaces. To be effective, all such strategies should be devised for the *express purpose* of mold prevention, integrated into a comprehensive plan, and executed with diligent oversight by a professional mold prevention consultant. Because just a small amount of water or moisture can lead to the development of severe mold infestations, a number of mold prevention strategies focus on keeping building surfaces dry. While groundwater, water infiltration, and similar issues may have been addressed as part of the geotechnical engineering study whose findings are conveyed in this report, the geotechnical engineer in charge of this project is not a mold prevention consultant; ***none of the services performed in connection with the geotechnical engineer's study were designed or conducted for the purpose of mold prevention. Proper implementation of the recommendations conveyed in this report will not of itself be sufficient to prevent mold from growing in or on the structure involved.***

Rely, on Your ASFE-Member Geotechncial Engineer for Additional Assistance

Membership in ASFE/The Best People on Earth exposes geotechnical engineers to a wide array of risk management techniques that can be of genuine benefit for everyone involved with a construction project. Confer with your ASFE-member geotechnical engineer for more information.

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