GEOTECHNICAL INVESTIGATION
PROPOSED HEADWORKS REHABILITATION AND UPGRADE PROJECT
SAN ELIJO WATER RECLAMATION FACILITY
ENCINITAS, CALIFORNIA

Prepared For:

DUDEK
750 Second Street
Encinitas, CA 92024

Project No. 11196.001

June 28, 2016

Leighton Consulting, Inc.

A LEIGHTON GROUP COMPANY

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Dudek
750 Second Street
Encinitas, CA 92024

Attention: Mr. Michael Hill, PE

Subject: Geotechnical Investigation, Proposed Headworks Rehabilitation and Upgrade Project, Encinitas, California

In accordance with your request and authorization, we have conducted a geotechnical investigation for the proposed Headworks Rehabilitation and Upgrade Project at 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,

LEIGHTON CONSULTING, INC.

William D. Olson, RCE 45283
Associate Engineer

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Senior Project Geologist

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TABLE OF CONTENTS

TABLE OF CONTENTS (Continued)

TABLES

FIGURES

APPENDICES

1.0 INTRODUCTION

1.1 Purpose and Scope

This report presents the results of our geotechnical investigation for proposed Headworks Rehabilitation and Upgrade project at the San Elijo Water Reclamation Facility, located in Cardiff-by-the-Sea (Encinitas), California (Figure 1). The purpose of our investigation was to evaluate the existing geotechnical conditions present at the subject 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- Review of available pertinent, published and unpublished geotechnical literature and maps (Appendix A).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 GCI, 1979, Ninyo & Moore, 1989, and AGRA, 1995, and Leighton in 2010.
- 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.
- Subsurface exploration consisting of the excavation, logging, and sampling of two (2) hollow-stem borings. The logs of the borings 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 borings. Results of these tests are presented in Appendix C.
- Compilation and analysis of the geotechnical data obtained from the field exploration 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 in the northeastern portion of the treatment plant, north of the existing secondary clarifiers. The ground surface elevations of the subject site range from 43 to 46 feet mean sea level (msl), and is currently occupied by existing treatment equipment/facilities surrounded with asphalt paved surfaces.

The proposed improvements and upgrades to the Headworks consists of new screening channels, rehabilitation of the existing channels and grit chamber, new equipment pads, subsurface piping, and associated support structures. The proposed surface grades of the new improvement appear to be at or near the existing surface elevation or approximately 45 feet msl.

Site Latitude and Longitude
33.0169º N
117.2737º W

1.3 Previous Investigations

In summary, previous geotechnical studies have been performed for various facility improvements. The most recently study at the facility was performed by Leighton in 2010 for a new Advance Water Treatment System located just south of the subject site.

Depth of the explorations (borings or CPTs) ranging from 26 to 94 feet below the existing ground surface (bgs). The results of the previous studies have been incorporated into the current investigation.

2.0 SUBSURFACE EXPLORATION AND LABORATORY TESTING

Our subsurface exploration consisted of the excavation of two (2) exploratory borings. The approximate locations of the borings are shown on the Exploration Location Map, Figure 2. The purpose of these excavations was to evaluate the physical characteristics of the onsite soils pertinent to the proposed improvements. The borings allowed evaluation of the soils encountered within proposed improvement area, and to provide 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 exploratory excavations were logged by a representative from our firm. Representative bulk and undisturbed samples were obtained at frequent intervals for laboratory testing, and logs of the borings are presented in Appendix B. The boreholes were backfilled with bentonite grout per County of San Diego, Department of Environmental Health requirements.

Laboratory testing was performed on representative samples to evaluate the moisture, density, shear strength, and geo-chemical (corrosion) characteristics of the subsurface soils. A discussion of the laboratory tests performed and a summary of the laboratory test results are presented in Appendix C. In-situ moisture and density test results are provided on the boring logs (Appendix B).

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. It should be noted that Paralic Estuarine Deposits (Qpe), as shown on Figure 3, Regional Geology Map, were not encountered during this investigation. 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 5 to 8 feet in depth. The fill soil, consisting of brown, loose, poorly graded sands to clayey 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, light brown silty sand to poorly graded sand with micaceous layers. Layers of fat clay, approximately 5 feet thick, were also encountered at depths of 20 and 45 feet below the existing ground surface. Thickness of the alluvium beneath the subject site is anticipated to extend beyond a depth of 55 below the existing ground surface (bgs) based on our current explorations. 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.

3.2.3 Delmar Formation

Although not encountered in our borings, it is anticipated that the entire site is underlain at depth by formational material consisting of Eocene-age Delmar formation. The Delmar Formation generally consists of yellowish-green, sandy, claystone interbedded with medium-gray, coarse-grained, sandstone.

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 recent field explorations was at elevation of approximately 25 feet msl, which is roughly 20 feet below the existing ground surface (bgs). 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, 2012); it appears that the facility is not located within a flood zone.

3.6 Engineering Characteristics of Onsite Soils

Based on the results of our laboratory testing of representative onsite 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 to medium. 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 6.61, and a measured minimum electrical resistivity of 1,177 ohm-cm. Test results also indicated that the sample had a chloride content of 127 ppm, and a soluble sulfate content of 225 ppm (i.e., 0.02%).

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 faulting 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 1,6000,000 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 most recent interim revision in 2007 (Hart, 2007). 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 a Fault-Rupture Hazard Zone as created by the Alquist-Priolo Act (Hart, 2007) and recently modified.

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 effect of seismic shaking may be mitigated by adhering to the California Building Code or state-of-the-art seismic design parameters of the Structural Engineers Association of California. We have provided risk-targeted spectral acceleration parameters per California Building Code (CBSC 2013) for the proposed project site, using the USGS Worldwide Seismic Design Values Tool.

Table 1

CBC Mapped Spectral Acceleration Parameters

Site Class	E		
Site Coefficients	F_a	=	0.900
	F_v	=	2.400
Mapped MCE Spectral Accelerations	S_S	=	1.175g
	S_1	=	0.455g
Site Modified MCE Spectral Accelerations	S_{MS}	=	1.057g
	S_{M1}	=	1.092g
Design Spectral Accelerations	S_{DS}	=	0.705g
	S_{D1}	=	0.728g

Utilizing ASCE Standard 7-10, in accordance with Section 11.8.3, the following additional parameters for the peak horizontal ground acceleration are associated with the Geometric Mean Maximum Considered Earthquake (MCE_G). The mapped MCE_G peak ground acceleration (PGA) is 0.492g for the site. For a Site Class D, the F_{PGA} is 0.900 and the mapped peak ground acceleration adjusted for Site Class effects (PGA_M) is 0.442g for the site.

Secondary effects that can be associated with severe ground shaking following a relatively large earthquake include shallow ground rupture, soil liquefaction and dynamic settlement, seiches and tsunamis. 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 0.49g. 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 borings, are considered susceptible to liquefaction at the design earthquake ground motion. In summary, the potentially liquefiable soil ranges from 20 to 55 feet bgs, which is consistent with the findings of the previous geotechnical reports (Leighton, 2010 and 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 6.5 to 7 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. Remedial grading may be required within the upper 2 feet.

- Ground water is anticipated at an elevation ranging 25 feet msl, which is roughly 20 feet below the existing ground surface (bgs) (Leighton, 2010 and 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 0.49g.

- 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.

- Designers 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.

<u>Cantilever Shoring System</u>

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

<u>Tie-Back or Multi-Braced Shoring System</u>

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)

<u>General</u>

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 <u>Fill Placement and Compaction</u>

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. 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 2

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 4. 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 $8H_T$ 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 basin structure 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 cannot 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 onsite soil will have a minimum R-Value of 12. 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 3
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 7.0 inches Class 2 Base
Drive Areas	5.0	3.0 inches AC over 9.0 inches Class 2 Base
Truck Drive Areas	6.0	4.0 inches AC over 10.0 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 City of Encinitas, 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.

FIGURES

Project: 11196.001	Eng/Geol: WDO/MDJ
Scale: 1 " = 2,000 '	Date: June 2016

Base Map: ESRI ArcGIS Online 2016
Thematic Information: Leighton
Author: (mmurphy)

SITE LOCATION MAP

San Elijo Water Reclamation Facility
Encinitas, California

Figure 1

Leighton

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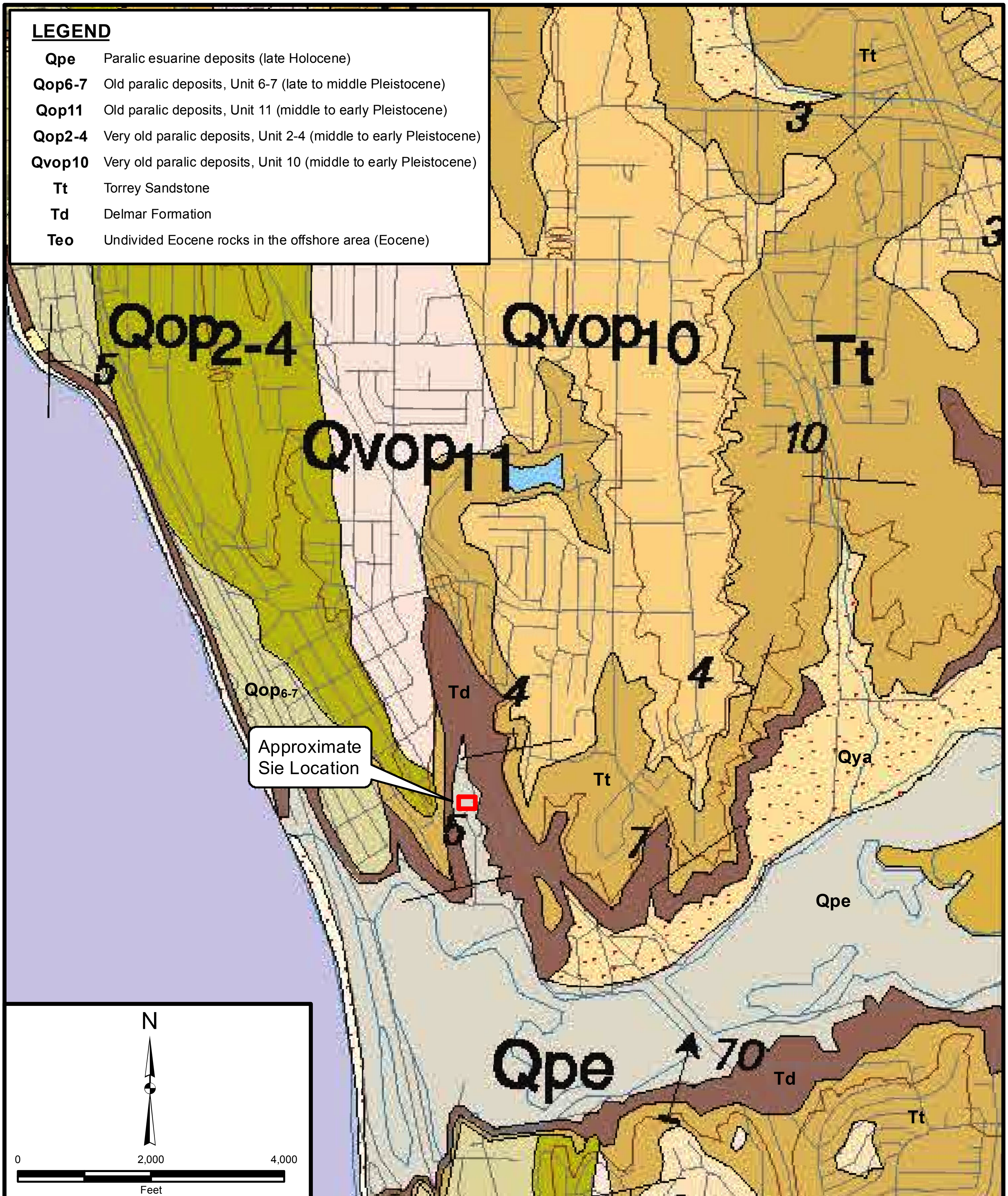

Project: 11196.001	Eng/Geol: WDO/MDJ
Scale: 1 " = 2,000 '	Date: June 2016

Reference: Geologic Map of the Oceanside 30'x60' Quadrangle, California, Compiled by Michael P. Kennedy and Siang S. Tan, 2005.
Author: (mmurphy)

REGIONAL GEOLOGY MAP

San Elijo Water Reclamation Facility
Encinitas, California

Figure 3

Leighton

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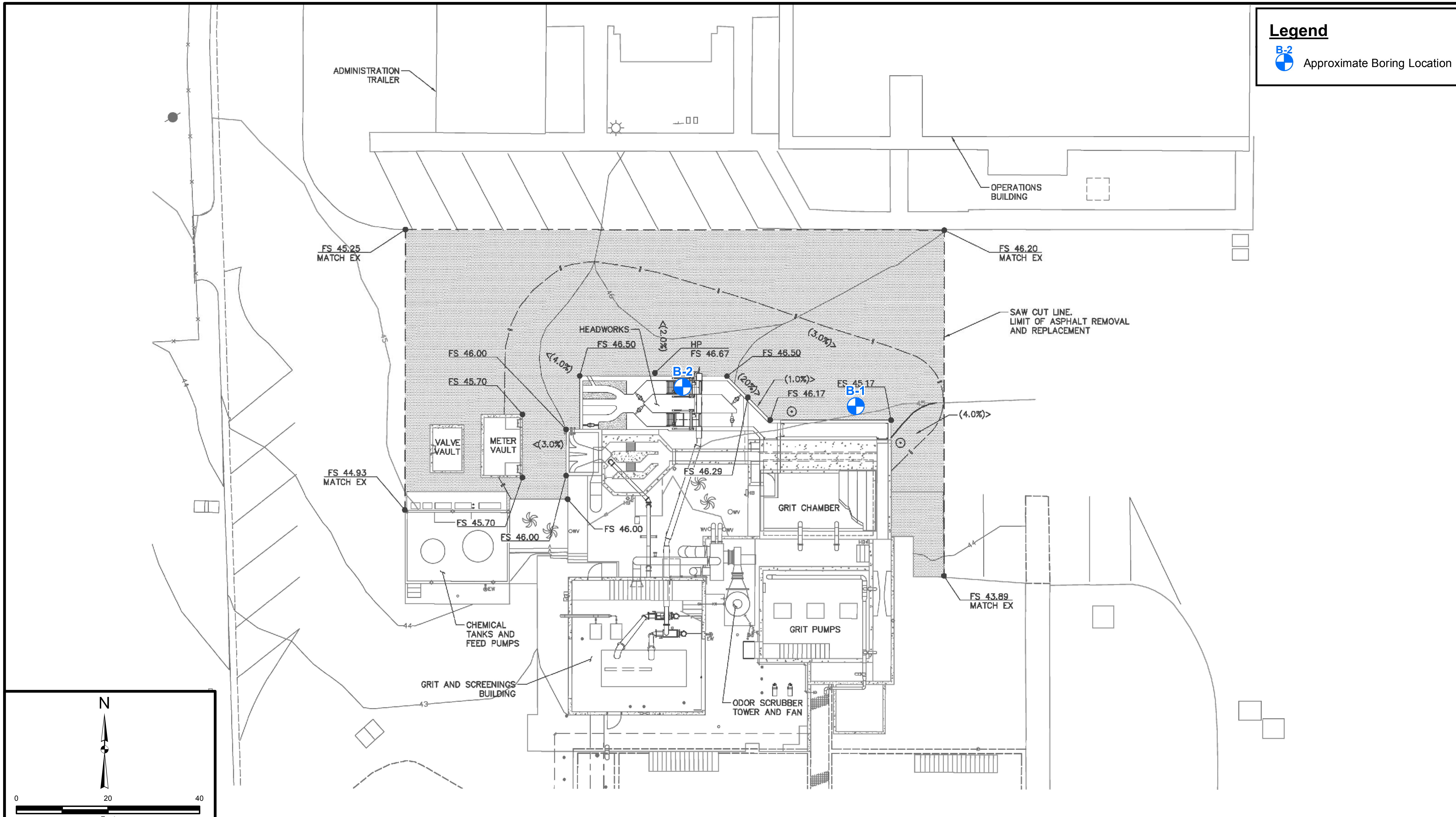

Project: 11196.001	Eng/Geol: WDO/MDJ
Scale: 1 " = 20 '	Date: June 2016

Base Map: Sheet C-1, Civil Site Plan by Dudek, dated 3/14/2016.

Author: Leighton Geomatics (mmurphy)

EXPLORATION LOCATION MAP

San Elijo Water Reclamation Facility

Encinitas, California

Figure 2

Leighton

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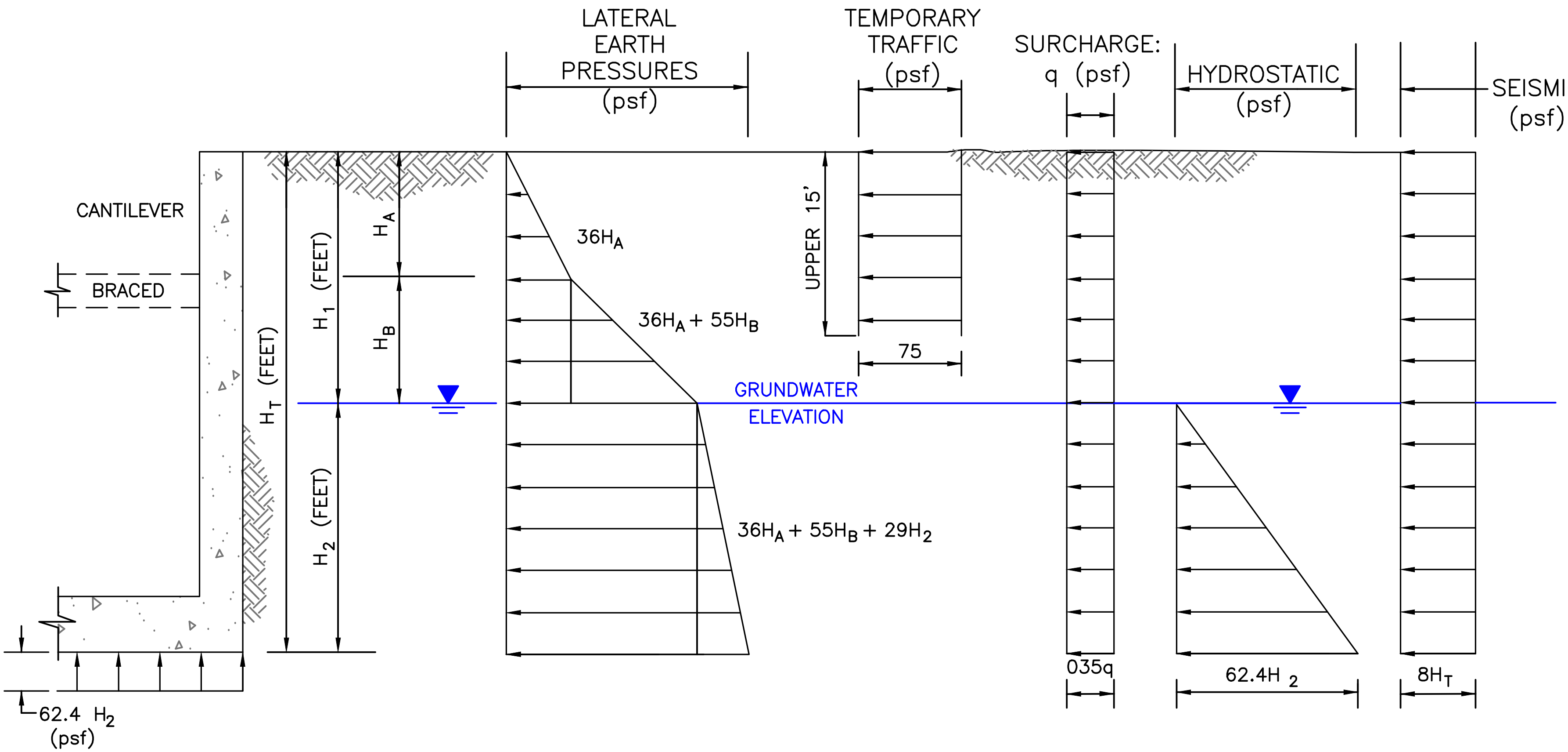

Project: 11196.001	Eng/Geol: WDO/MDJ
Scale: NTS	Date: June 2016
Reference:	
Author: MAM	

LATERAL EARTH AND HYDROSTATIC PRESSURES

San Elijo Water Reclamation Facility
Encinitas, Califonia

Figure 4

Leighton

P:\DRAFTING\11196\001\CAD\11196-001_F04_LEP_2016-05-31.DWG (06-17-16 11:55:14AM) Plotted by: mmurphy

APPENDIX A

REFERENCES

APPENDIX A

REFERENCES

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APPENDIX B

BORING LOGS

GEOTECHNICAL BORING LOG KEY

Project No.		**Date Drilled**	
Project	KEY TO BORING LOG GRAPHICS	**Logged By**	
Drilling Co.		**Hole Diameter**	
Drilling Method		**Ground Elevation**	
Location		**Sampled By**	

Elevation Feet	Depth Feet	Graphic Log (N S)	Attitudes	Sample No.	Blows Per 6 Inches	Dry Density pcf	Moisture Content, %	Soil Class. (U.S.C.S.)	SOIL DESCRIPTION	Type of Tests
									This Soil Description applies only to a location of the exploration at the time of sampling. Subsurface conditions may differ at other locations and may change with time. The description is a simplification of the actual conditions encountered. Transitions between soil types may be gradual.	
	0								Asphaltic concrete	
									Portland cement concrete	
								CL	Inorganic clay of low to medium plasticity; gravelly clay; sandy clay; silty clay; lean clay	
								CH	Inorganic clay; high plasticity, fat clays	
								OL	Organic clay; medium to plasticity, organic silts	
	5							ML	Inorganic silt; clayey silt with low plasticity	
								MH	Inorganic silt; diatomaceous fine sandy or silty soils; elastic silt	
								ML-CL	Clayey silt to silty clay	
								GW	Well-graded gravel; gravel-sand mixture, little or no fines	
								GP	Poorly graded gravel; gravel-sand mixture, little or no fines	
	10							GM	Silty gravel; gravel-sand-silt mixtures	
								GC	Clayey gravel; gravel-sand-clay mixtures	
								SW	Well-graded sand; gravelly sand, little or no fines	
								SP	Poorly graded sand; gravelly sand, little or no fines	
								SM	Silty sand; poorly graded sand-silt mixtures	
	15							SC	Clayey sand; sand-clay mixtures	
									Bedrock	
									Ground water encountered at time of drilling	
				B-1					Bulk Sample	
	20			C-1					Core Sample	
				G-1					Grab Sample	
				R-1					Modified California Sampler (3" O.D., 2.5 I.D.)	
				SH-1					Shelby Tube Sampler (3" O.D.)	
				S-1					Standard Penetration Test SPT (Sampler (2" O.D., 1.4" I.D.)	
	25			PUSH					Sampler Penetrates without Hammer Blow	
	30									

SAMPLE TYPES:

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

TYPE OF TESTS:

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

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

GEOTECHNICAL BORING LOG B-1

Project No.	11196.001	**Start Date Drilled**	6-7-16	**End Date Drilled**	6-7-16
Project	Dudek/San Elijo			**Hole Diameter**	8"
Drilling Co.	Baja Exploration			**Ground Elevation**	45'
Drilling Method	CME-75 - 140lb - Autohammer - 30" Drop			**Logged By**	CDL
Location	See Figure 2			**Sampled By**	CDL

Elevation Feet	Depth Feet	Graphic Log (N S)	Attitudes	Sample No.	Blows Per 6 Inches	Dry Density pcf	Moisture Content, %	Soil Class. (U.S.C.S.)	SOIL DESCRIPTION *This Soil Description applies only to a location of the exploration at the time of sampling. Subsurface conditions may differ at other locations and may change with time. The description is a simplification of the actual conditions encountered. Transitions between soil types may be gradual.*	Type of Tests
45	0								ASPHALT/CONCRETE, approximately 4" thick over AGGREGATE BASE 12" over	
				B-1 1'-5'				GP SP	ARTIFICIAL FILL (Afu) @ 1': SAND, loose, brown, moist, fine to medium SAND	
40	5			R-1	2 3 5	115	5	SC	@ 5': Clayey SAND, loose, dark brown, moist, fine SAND, trace asphalt concrete debris, fill mottling, trace GRAVEL	
								SM	QUATERNARY ALLUVIUM (Qal)	
35	10			R-2	3 5 9	99	9		@ 10': Silty SAND, loose, brown to dark brown, moist, fine SAND, deformed bedding	DS
								SP	@ 11': SAND, loose, light brown, moist, fine to medium SAND	
30	15			R-3	5 9 14	106	6		@ 15': SAND, medium dense, light brown, moist, fine to medium SAND, trace SILT in dark brown mottling	
25	20			S-1	2 1 3			CH	@ 20': Fat CLAY, soft, brown, wet, high plasticity	
20	25			S-2	3 4 3			SC	@ 25': Clayey SAND, loose, light orange-brown, wet, fine SAND, approximately 25% to 30% fines	
15	30									

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

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

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

GEOTECHNICAL BORING LOG B-1

Project No. 11196.001
Start Date Drilled 6-7-16
End Date Drilled 6-7-16
Project Dudek/San Elijo
Hole Diameter 8"
Drilling Co. Baja Exploration
Ground Elevation 45'
Drilling Method CME-75 - 140lb - Autohammer - 30" Drop
Logged By CDL
Location See Figure 2
Sampled By CDL

SOIL DESCRIPTION

This Soil Description applies only to a location of the exploration at the time of sampling. Subsurface conditions may differ at other locations and may change with time. The description is a simplification of the actual conditions encountered. Transitions between soil types may be gradual.

Elevation Feet	Depth Feet	Graphic Log (N S)	Attitudes	Sample No.	Blows Per 6 Inches	Dry Density pcf	Moisture Content, %	Soil Class. (U.S.C.S.)	SOIL DESCRIPTION	Type of Tests
15	30			S-3	2 1 3			SC	QUATERNARY ALLUVIUM (Qal) continued @ 30': Clayey SAND, loose, light orange-brown, wet, trace amount of light gray mottling	
10	35			S-4	3 5 7				@ 35': Clayey SAND, medium dense, dark brown, fine SAND	
								SM	@ 36': Grades with depth to a silty SAND, medium dense, light to orange-brown, wet, fine SAND, approximately 15% fines	
5	40			S-5	3 3 7			SC	@ 40': Clayey SAND, medium dense, brown to orange-brown, wet, fine SAND, pockets of medium SAND, decaying roots, approximately 15% fines	
								CL	@ 40.5': Sandy CLAY, stiff, brown, wet, fines SAND, approximately 30% fines SAND, decaying roots	
0	45			S-6	5 5 4			CH	@ 45': Fat CLAY with SAND, stiff, brown, very moist, fine SAND, high plasticity	
-5	50			S-7	3 2 1				@ 50': No recovery	
-10	55									
-15	60									

Total Depth = 51.5 Feet
Groundwater encountered at 20 feet at time of drilling
Backfilled with Bentonite grout on 6/7/13

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

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

GEOTECHNICAL BORING LOG B-2

Project No.	11196.001	**Start Date Drilled**	6-7-16	**End Date Drilled**	6-7-16
Project	Dudek/San Elijo			**Hole Diameter**	8"
Drilling Co.	Baja Exploration			**Ground Elevation**	45'
Drilling Method	CME-75 - 140lb - Autohammer - 30" Drop			**Logged By**	CDL
Location	See Figure 2			**Sampled By**	CDL

SOIL DESCRIPTION

This Soil Description applies only to a location of the exploration at the time of sampling. Subsurface conditions may differ at other locations and may change with time. The description is a simplification of the actual conditions encountered. Transitions between soil types may be gradual.

Elevation Feet	Depth Feet	Graphic Log (N S)	Attitudes	Sample No.	Blows Per 6 Inches	Dry Density pcf	Moisture Content, %	Soil Class. (U.S.C.S.)	SOIL DESCRIPTION	Type of Tests
45	0							GP	ASPHALT/CONCRETE, approximately 4" over AGGREGATE BASE, approximately 4" over	
				B-1 1'-5'				SP	ARTIFICIAL FILL @ 1': Poorly-graded SAND, loose, brown, moist, fine SAND	CR
40	5			R-1	4 5 5	111	8	SP	QUATERNARY ALLUVIUM (Qal) @ 5': Poorly-graded SAND, loose, light brown, moist, fine to medium SAND	
								SM	@ 6.5': Silty SAND, loose, dark brown, moist, fine to medium SAND, faint laminated bedding	
35	10			R-2	3 5 9	100	7		@ 10': Silty SAND, loose, dark brown, moist, fine to medium SAND, faint laminated bedding	
								SP	@ 11': Poorly-graded SAND, loose, brown, moist, fine to medium SAND	
30	15			R-3	5 3 6	108	12		@ 15': Poorly-graded SAND, loose, light brown to dark brown with depth Drilling mud added to auger	DS
25	20			S-1	1 1 2			CH	@ 20': Sandy fat CLAY, soft, light brown, wet, fine to medium SAND, soft, brown, wet, high plasticity, approximately 20% fines	
20	25			S-2	4 8 9				@ 25': Lean sandy CLAY, very stiff, light brown, wet, fine SAND, approximately 30% to 40% SAND	
								SC	@ 26': Clayey SAND, medium dense, light brown, wet, fine to medium SAND	
15	30									

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

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

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

GEOTECHNICAL BORING LOG B-2

Project No.	11196.001	**Start Date Drilled**	6-7-16	**End Date Drilled**	6-7-16
Project	Dudek/San Elijo			**Hole Diameter**	8"
Drilling Co.	Baja Exploration			**Ground Elevation**	45'
Drilling Method	CME-75 - 140lb - Autohammer - 30" Drop			**Logged By**	CDL
Location	See Figure 2			**Sampled By**	CDL

SOIL DESCRIPTION

This Soil Description applies only to a location of the exploration at the time of sampling. Subsurface conditions may differ at other locations and may change with time. The description is a simplification of the actual conditions encountered. Transitions between soil types may be gradual.

Elevation Feet	Depth Feet	Graphic Log (N S)	Attitudes	Sample No.	Blows Per 6 Inches	Dry Density pcf	Moisture Content, %	Soil Class. (U.S.C.S.)	SOIL DESCRIPTION	Type of Tests
15	30			S-3	3 5 5			SC	@ 30': Fat CLAY, stiff, brown, wet, 3" thick grades abruptly with depth to clayey SAND, medium dense, light brown, wet, fine to medium SAND, fines decrease with depth from approximately 30% to 15% fines	
10	35			S-4	push 18"				@ 35': Clayey SAND, very loose, brown, wet, fine to medium SAND, approximately 30% CLAY	
5	40			S-5	push 5"				@ 40': Clayey SAND, very loose, brown, wet, fine to medium SAND, approximately 30% CLAY	
								CH	@ 41': Fat CLAY, stiff, brown, mottled, moist, high plasticity, some fine SAND, charcoal fragments	
0	45			S-6	3 6 7				@ 45': Fat CLAY, stiff, brown, mottled, moist, high plasticity, decomposed rootlets	
								SC		
-5	50			S-7	3 2 3				@ 50': Clayey SAND, loose, brown, wet, fine to medium SAND, approximately 30% fines	
								CH	@ 51': Fat CLAY, firm, brown, moist, high plasticity	
									Total Depth = 51.5 Feet **Groundwater encountered at 20 feet at time of drilling** **Backfilled with Bentonite grout patch with concrete on 6/7/16**	
-10	55									
-15	60									

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

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

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

APPENDIX C

LABORATORY TESTING

APPENDIX C

Laboratory Testing Procedures and Test Results

Direct Shear Tests: Direct shear tests were performed on selected undisturbed 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 figures.

Chloride Content (DOT Test Method No. 422): Chloride content contained within selected samples was tested in accordance with DOT Test Method No. 422. The results are presented in the table below:

Sample Location	Chloride Content, ppm
B-2 @ 1-5’	127

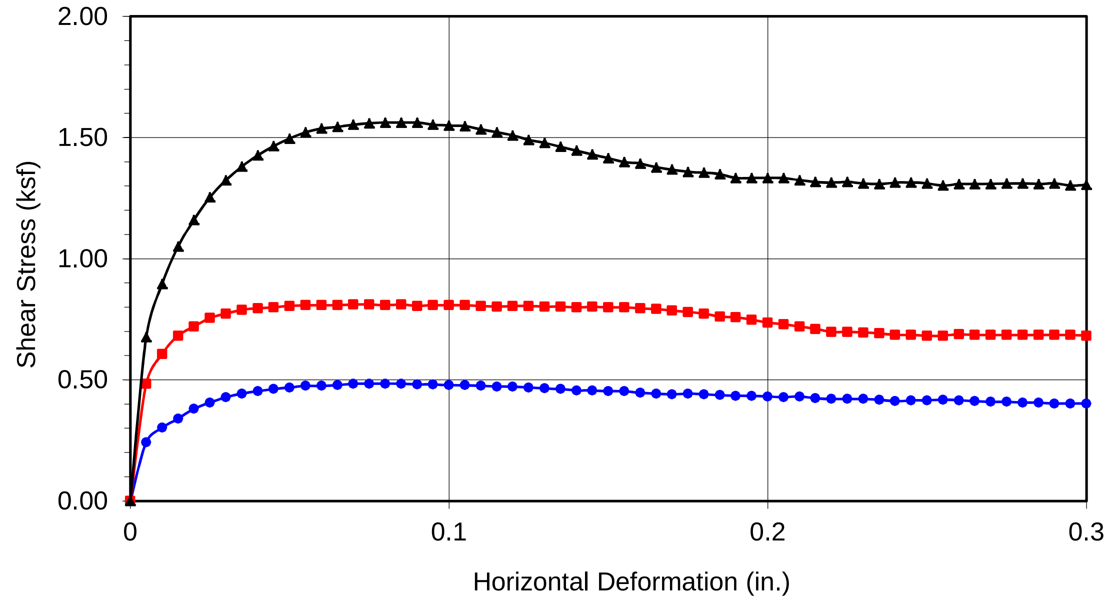
pH and Resistivity (California Test No. 643): Minimum resistivity and pH tests were performed in general accordance with California Test Method 643. The results are presented in the table below:

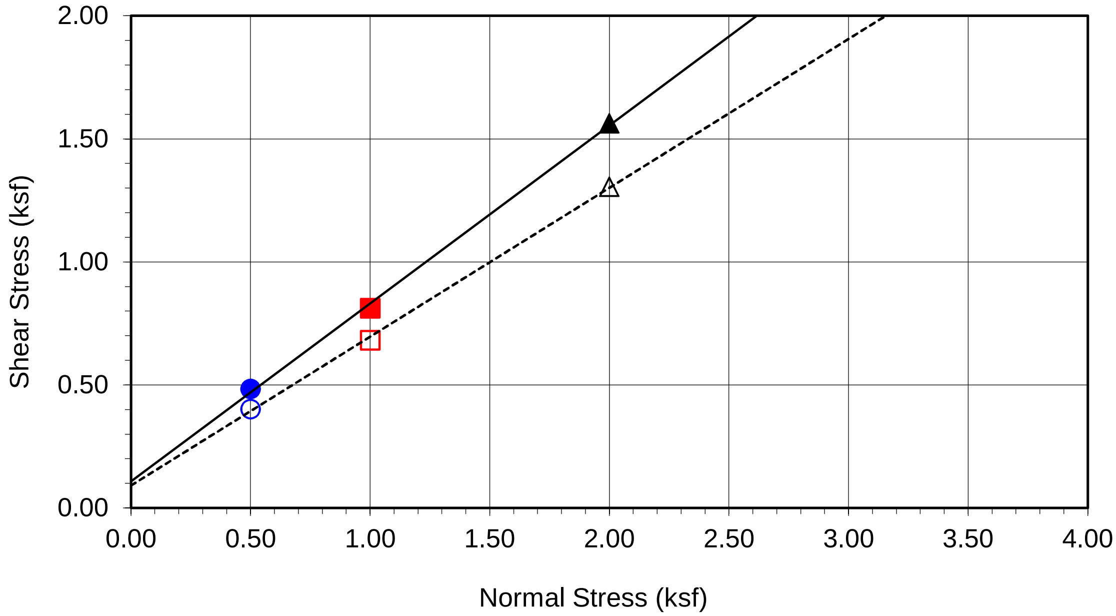
Sample Location	pH	Minimum Resistivity
B-2 @ 1-5’	6.61	1,177

APPENDIX C (Continued)

Soluble Sulfate (California Test No. 417): The soluble sulfate contents contained within selected samples of soil were determined by California Test Method 417. The test results are presented in the table below:

Sample Location	Soluble Sulfates (ppm)
B-2 @ 1-5’	225





Boring No.	**B-1**
Sample No.	**R-2**
Depth (ft)	**10**
Sample Type:	Ring

Soil Identification:
Light olive brown silty sand
(SM)

Strength Parameters

	C (psf)	ϕ (°)
Peak	109	36
Ultimate	90	31

Normal Stress (kip/ft²)	0.500	1.000	2.000
Peak Shear Stress (kip/ft²)	● 0.484	■ 0.811	▲ 1.562
Shear Stress @ End of Test (ksf)	○ 0.402	□ 0.682	△ 1.305
Deformation Rate (in./min.)	0.0033	0.0033	0.0033
Initial Sample Height (in.)	1.000	1.000	1.000
Diameter (in.)	2.415	2.415	2.415
Initial Moisture Content (%)	8.55	8.55	8.55
Dry Density (pcf)	97.8	99.0	100.0
Saturation (%)	31.9	32.9	33.7
Soil Height Before Shearing (in.)	0.9972	0.9951	0.9893
Final Moisture Content (%)	19.8	19.1	18.8

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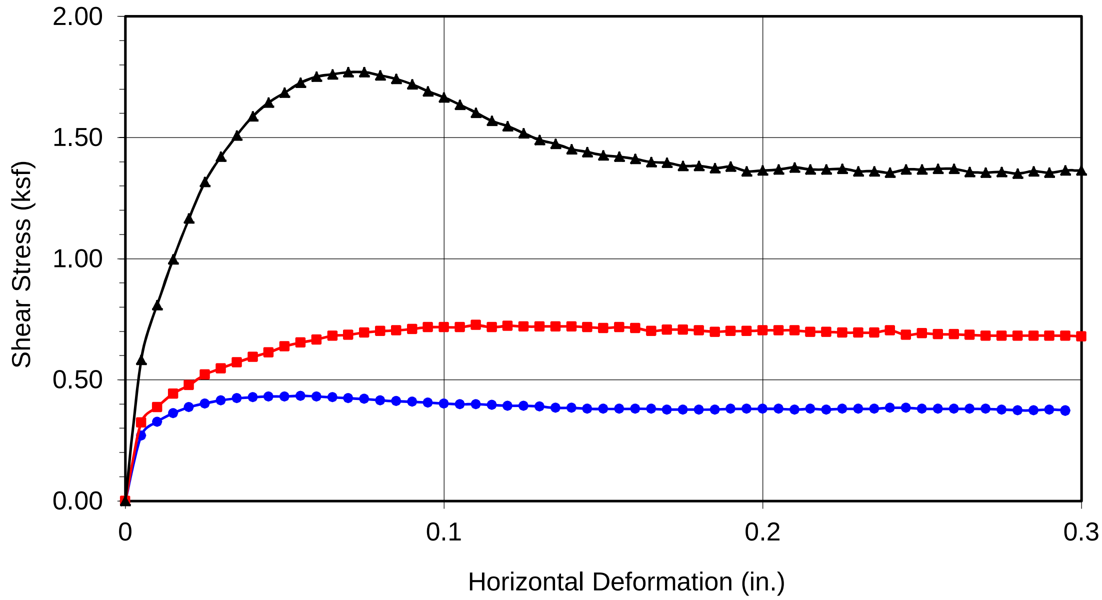
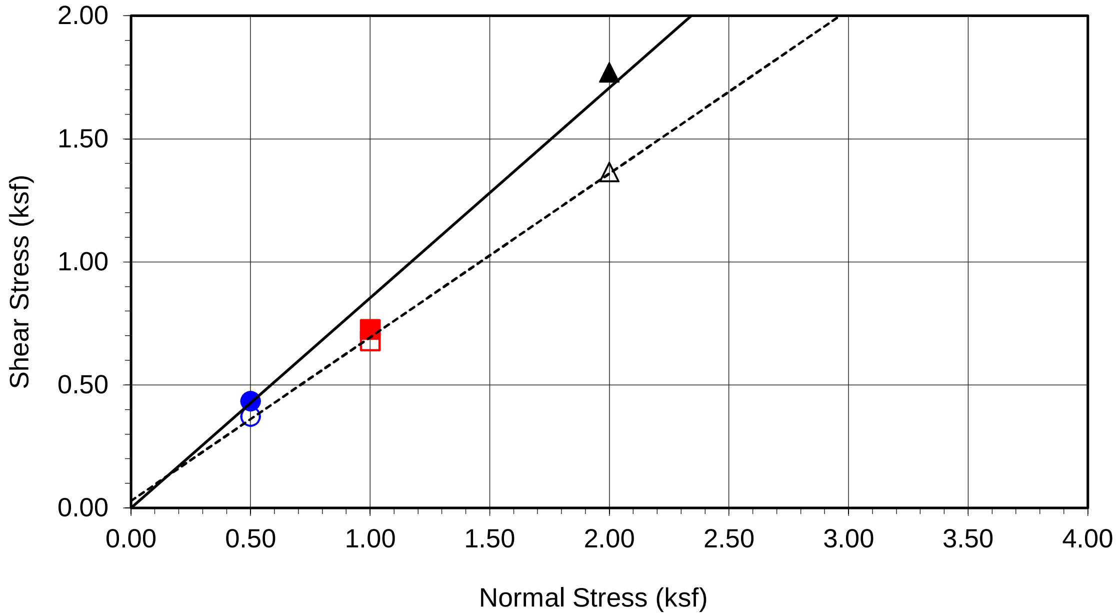
DIRECT SHEAR TEST RESULTS
Consolidated Drained - ASTM D 3080

Project No.: 11196.001

Dudek/San Elijo

06-16





Boring No.	**B-2**
Sample No.	**R-3**
Depth (ft)	**15**
Sample Type:	Ring
Soil Identification: Dark yellowish brown silty sand (SM)	

Strength Parameters

	C (psf)	ϕ (°)
Peak	0	40
Ultimate	29	34

Normal Stress (kip/ft²)	0.500	1.000	2.000
Peak Shear Stress (kip/ft²)	● 0.434	■ 0.726	▲ 1.770
Shear Stress @ End of Test (ksf)	○ 0.371	□ 0.679	△ 1.364
Deformation Rate (in./min.)	0.0033	0.0033	0.0033
Initial Sample Height (in.)	1.000	1.000	1.000
Diameter (in.)	2.415	2.415	2.415
Initial Moisture Content (%)	11.75	11.75	11.75
Dry Density (pcf)	101.1	107.8	109.0
Saturation (%)	47.5	56.3	58.1
Soil Height Before Shearing (in.)	0.9956	0.9728	0.9893
Final Moisture Content (%)	17.5	25.7	17.6

Leighton

DIRECT SHEAR TEST RESULTS

Consolidated Drained - ASTM D 3080

Project No.: 11196.001

Dudek/San Elijo

06-16

SOIL RESISTIVITY TEST
DOT CA TEST 643

Project Name: DUDEK / SAN ELIJO   Tested By : BCC   Date: 6/19/16
Project No. : 11196.001   Data Input By: BCC   Date: 6/20/16
Boring No.: B-2   Checked By: BCC   Date: 6/20/16
Sample No. : B-1   Depth (ft.) : 1.0-5.0
Visual Soil Identification: SP-SM

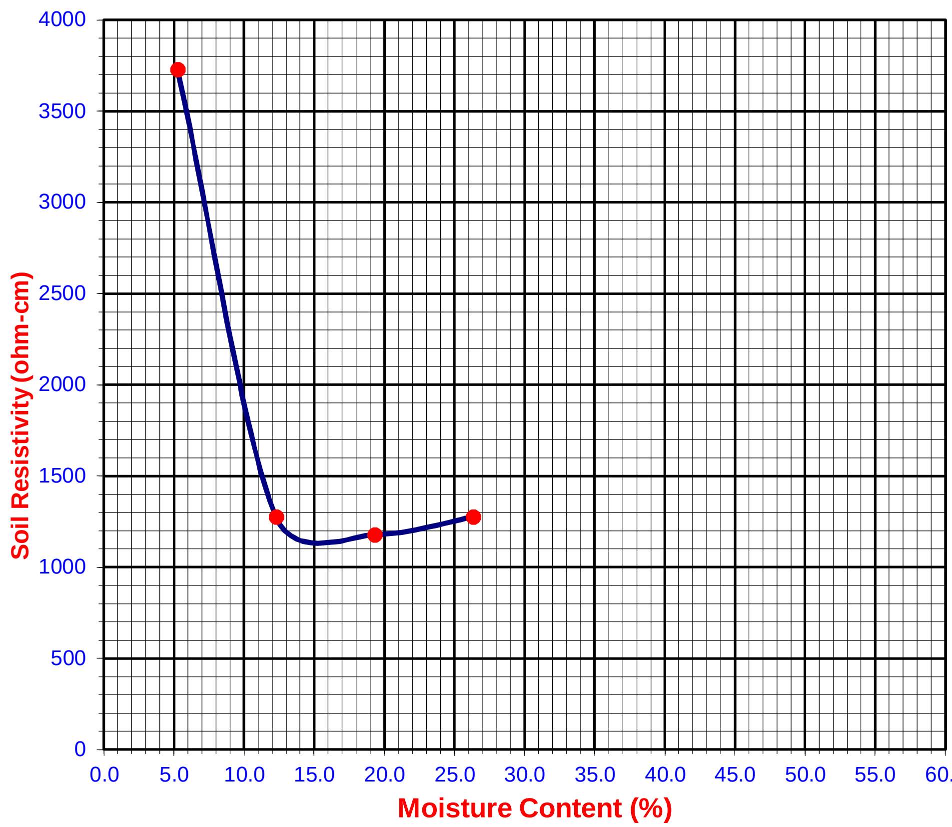
** NOTE: ASTM G-187 REQUIRES SOIL SPECIMENS TO PASS THROUGH NO.8 SIEVE PRIOR TO TESTING. THEREFORE, THIS TEST METHOD MAY NOT BE REPRESENTATIVE FOR COARSER MATERIALS.

Initial Moisture Content (%)

Wet Wt. of Soil + Cont. (g)	100.00
Dry Wt. of Soil + Cont. (g)	95.00
Wt. of Container (g)	0.00
Moisture Content (%)	5.26

Initial Soil Weight (g)(Wt)	150.0
Box Constant:	0.981

Remolded Specimen	Moisture Adjustments				
Water Added (ml)	0	10	20	30	
Adj. Moisture Content	5.26	12.28	19.30	26.32	
Resistance Rdg. (ohm)	3800	1300	1200	1300	
Soil Resistivity (ohm-cm)	3728	1275	1177	1275	

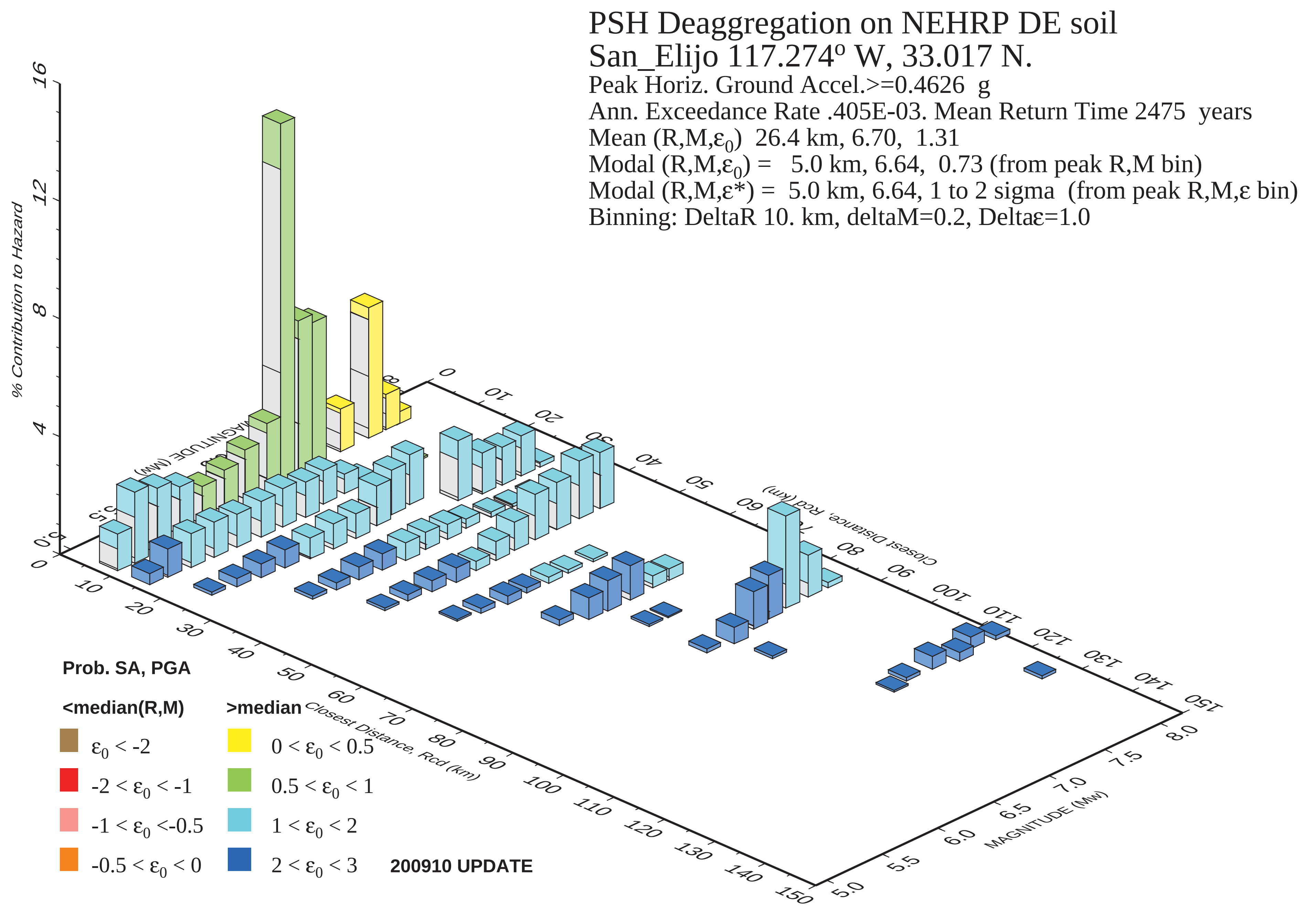


Minimum Resistivity (ohm-cm)	Moisture Content (%)	Sulfate Content (ppm)	Chloride Content	Soil pH
AASHTO T-288, DOT CA Test 643		DOT CA Test 417 Part II	DOT CA Test 422	AASHTO T-288, DOT CA Test 643
1177	19.30	225	126.7	6.61

Rev. 12-04

APPENDIX D

SEISMIC ANALYSIS



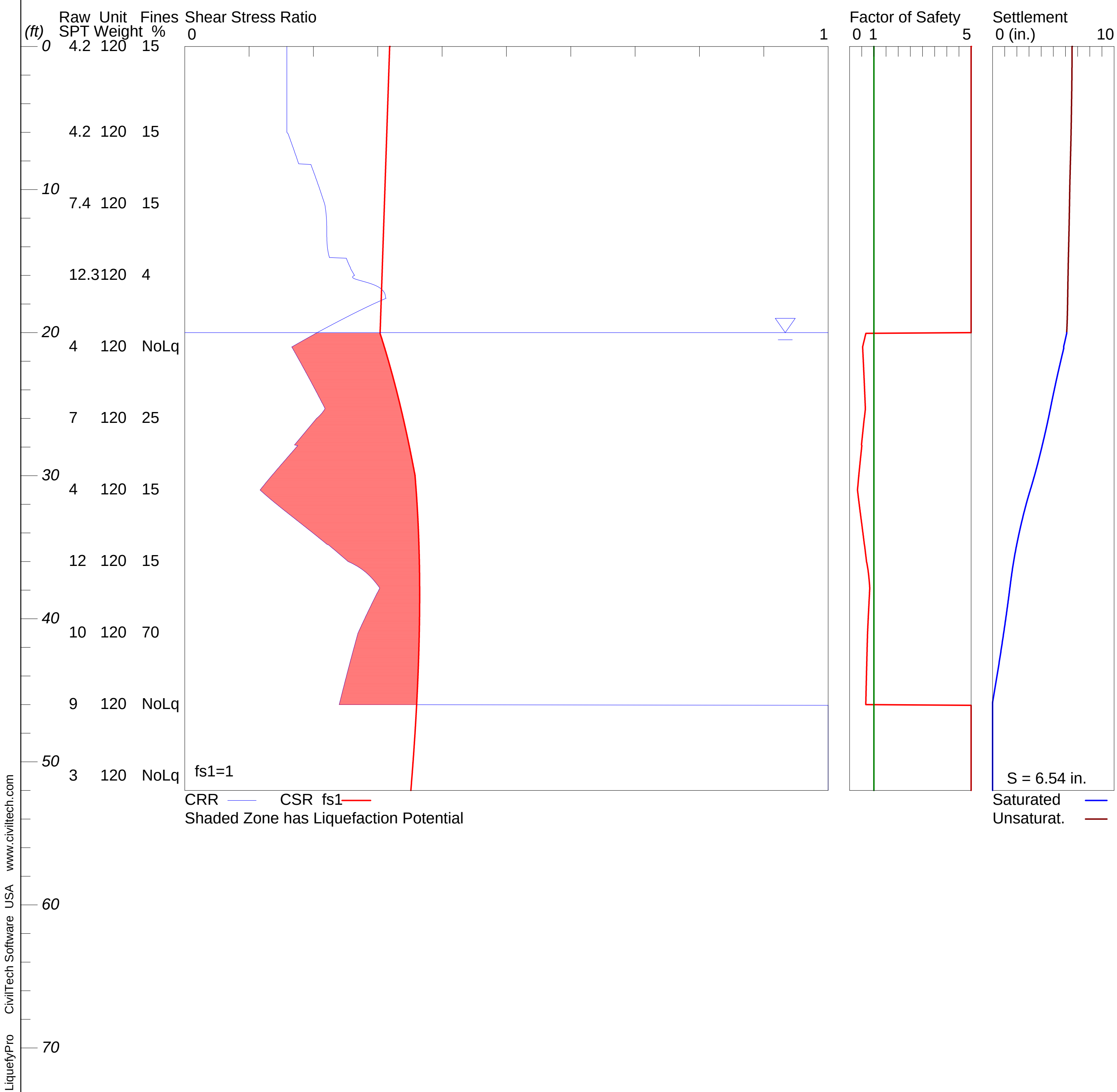
GMT 2016 Jun 21 21:07:31 Distance (R), magnitude (M), epsilon (E0,E) deaggregation for a site on soil with average vs= 180. m/s top 30 m. USGS CGHT PSHA2008 UPDATE Bins with lt 0.05% contrib. omitted

LIQUEFACTION ANALYSIS

San Elijo Reclamation Facility

Hole No.=B-1 Water Depth=20 ft Surface Elev.=45

Magnitude=6.6
Acceleration=0.49g



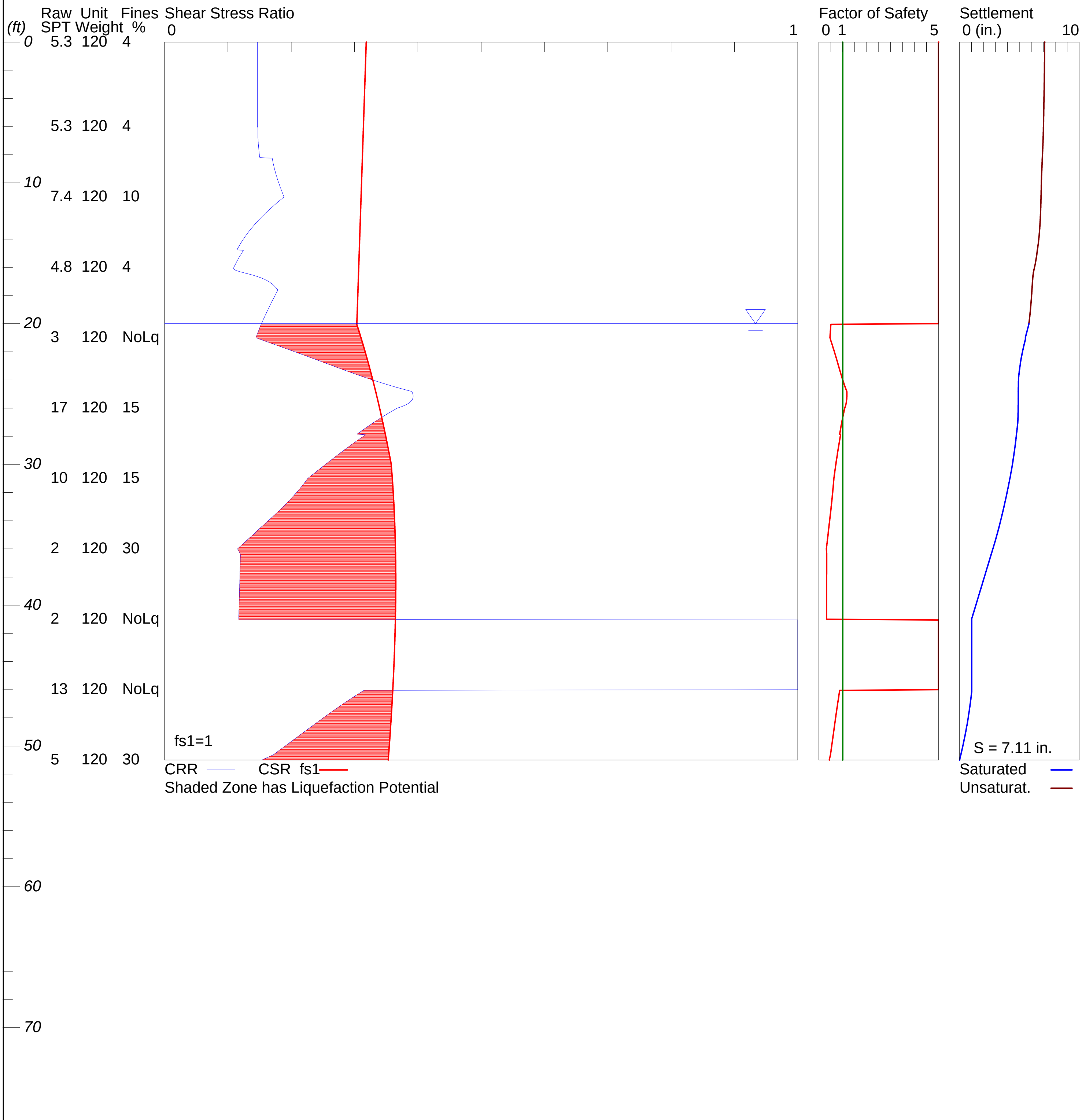
Encinitas, CA

LIQUEFACTION ANALYSIS

San Elijo Reclamation Facility

Hole No.=B-2 Water Depth=20 ft Surface Elev.=45

Magnitude=6.6
Acceleration=0.49g



Encinitas, CA

APPENDIX E

GENERAL EARTHWORK AND GRADING SPECIFICATIONS

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.

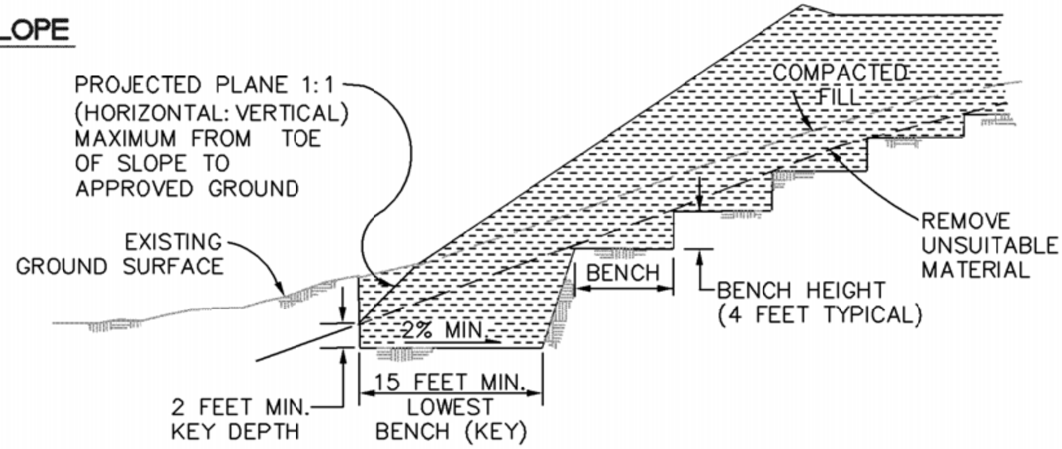
7.3 Lift Thickness

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.

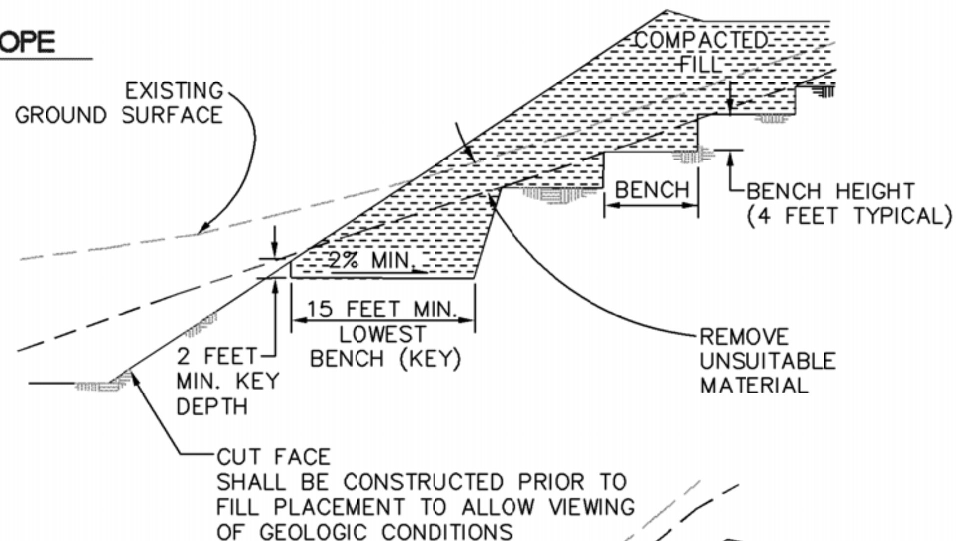
7.4 Observation and Testing

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

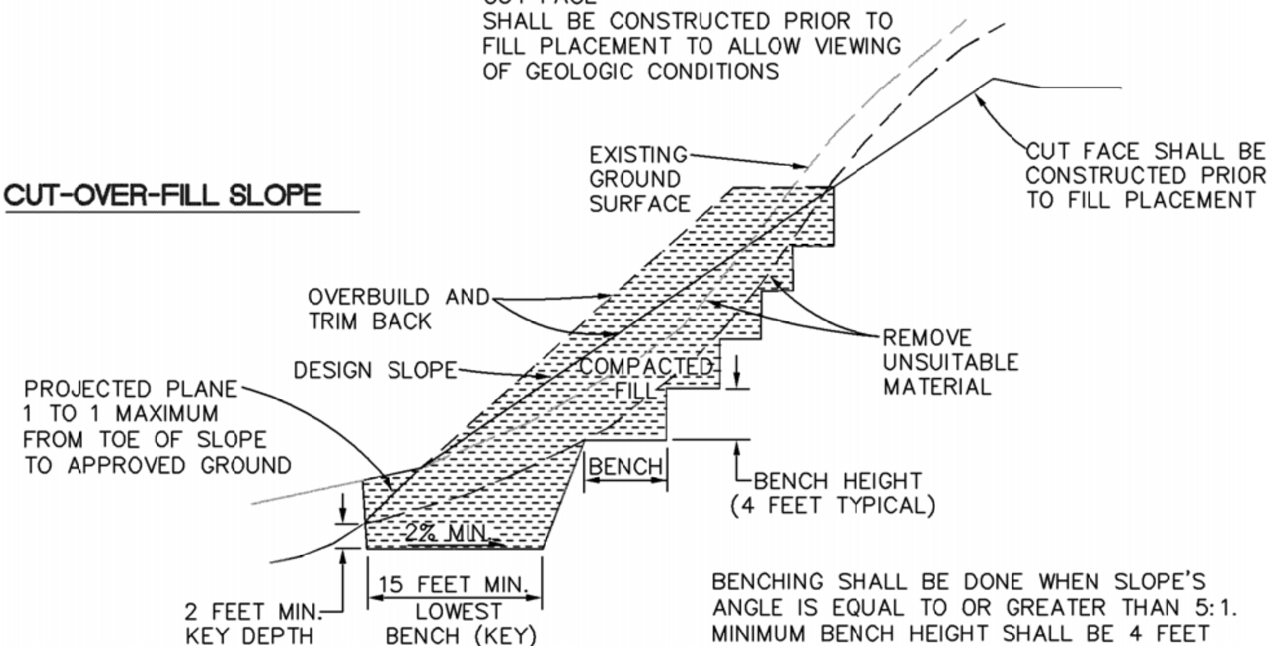
FILL SLOPE



FILL-OVER-CUT SLOPE

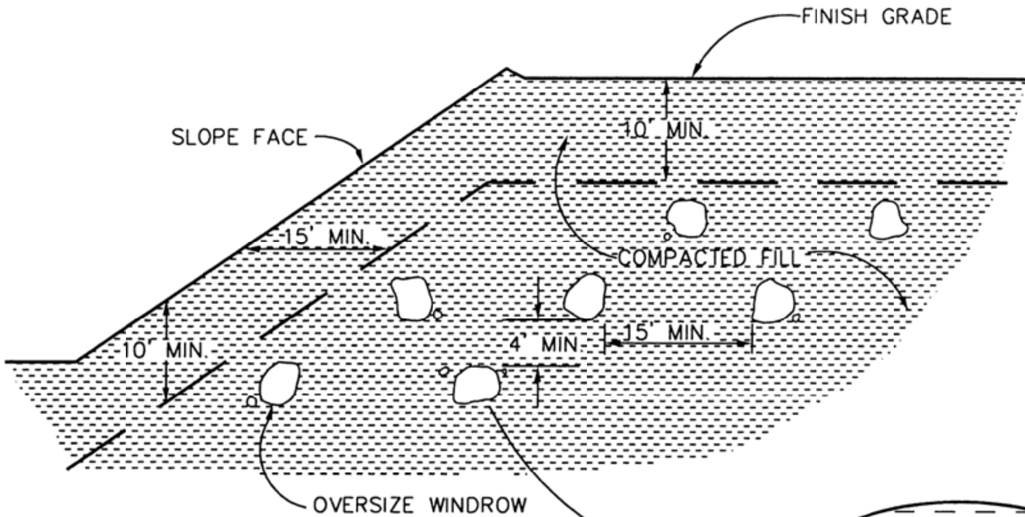


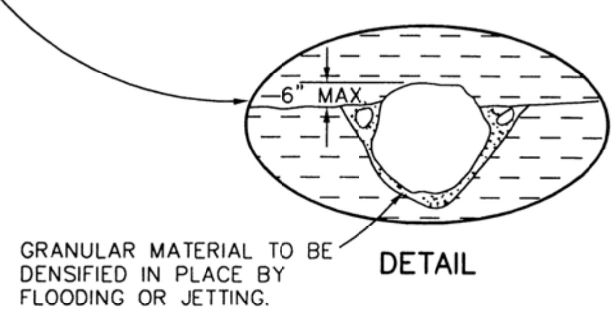
CUT-OVER-FILL SLOPE



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.

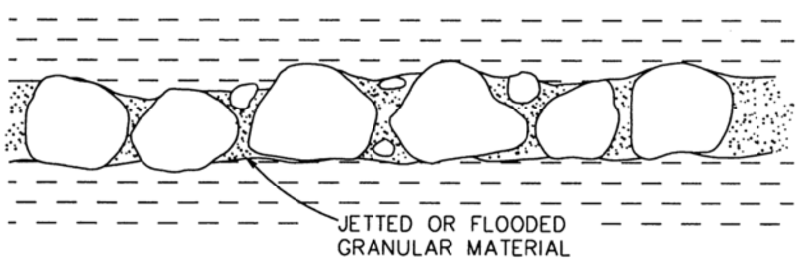
KEYING AND BENCHING	GENERAL EARTHWORK AND GRADING SPECIFICATIONS STANDARD DETAIL A





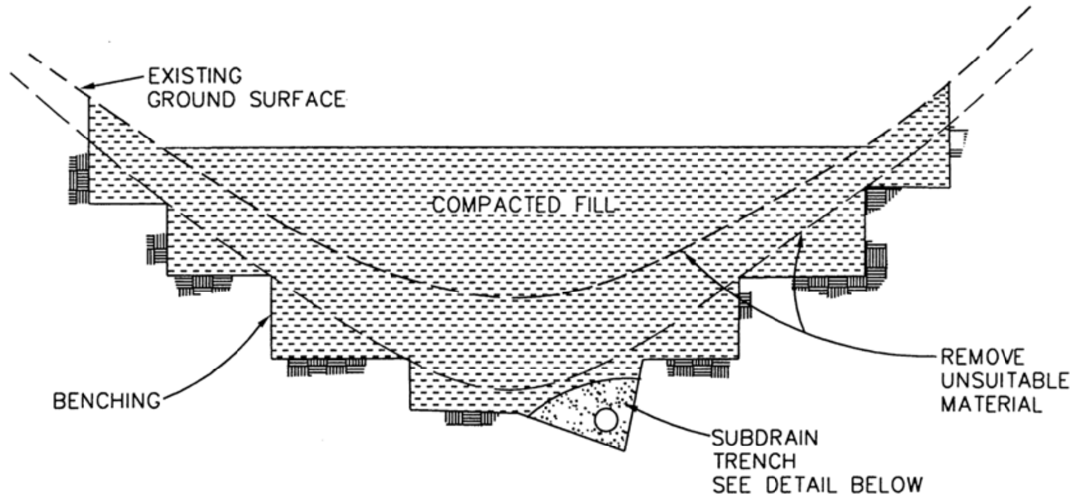
DETAIL

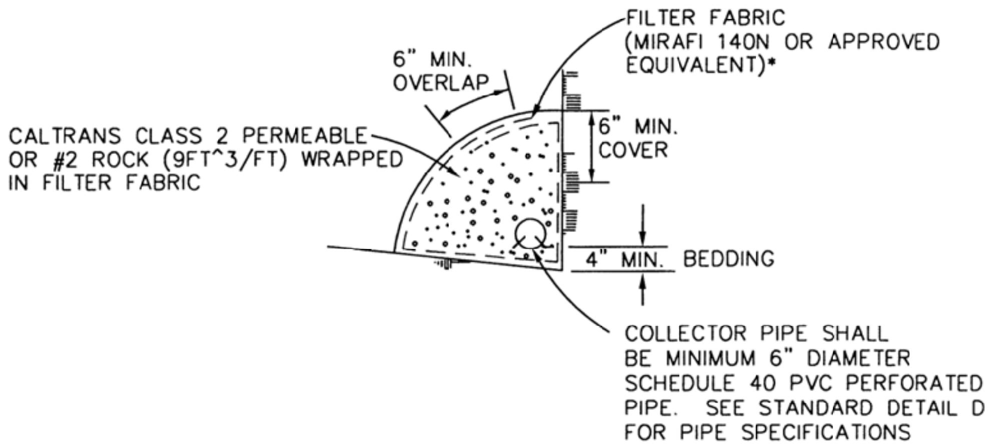
* OVERSIZE ROCK IS LARGER THAN 8 INCHES IN LARGEST DIMENSION.
* EXCAVATE A TRENCH IN THE COMPACTED FILL DEEP ENOUGH TO BURY ALL THE ROCK.
* BACKFILL WITH GRANULAR SOIL JETTED OR FLOODED IN PLACE TO FILL ALL THE VOIDS.
* DO NOT BURY ROCK WITHIN 10 FEET OF FINISH GRADE.
* WINDROW OF BURIED ROCK SHALL BE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



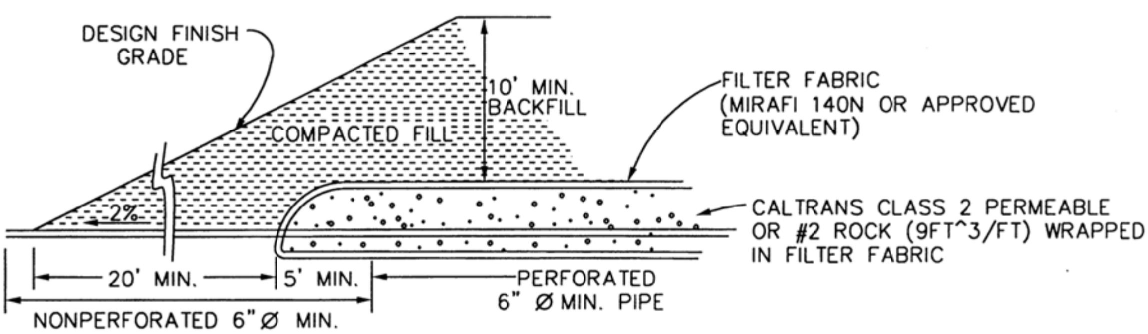
TYPICAL PROFILE ALONG WINDROW

OVERSIZE ROCK DISPOSAL	GENERAL EARTHWORK AND GRADING SPECIFICATIONS STANDARD DETAIL B



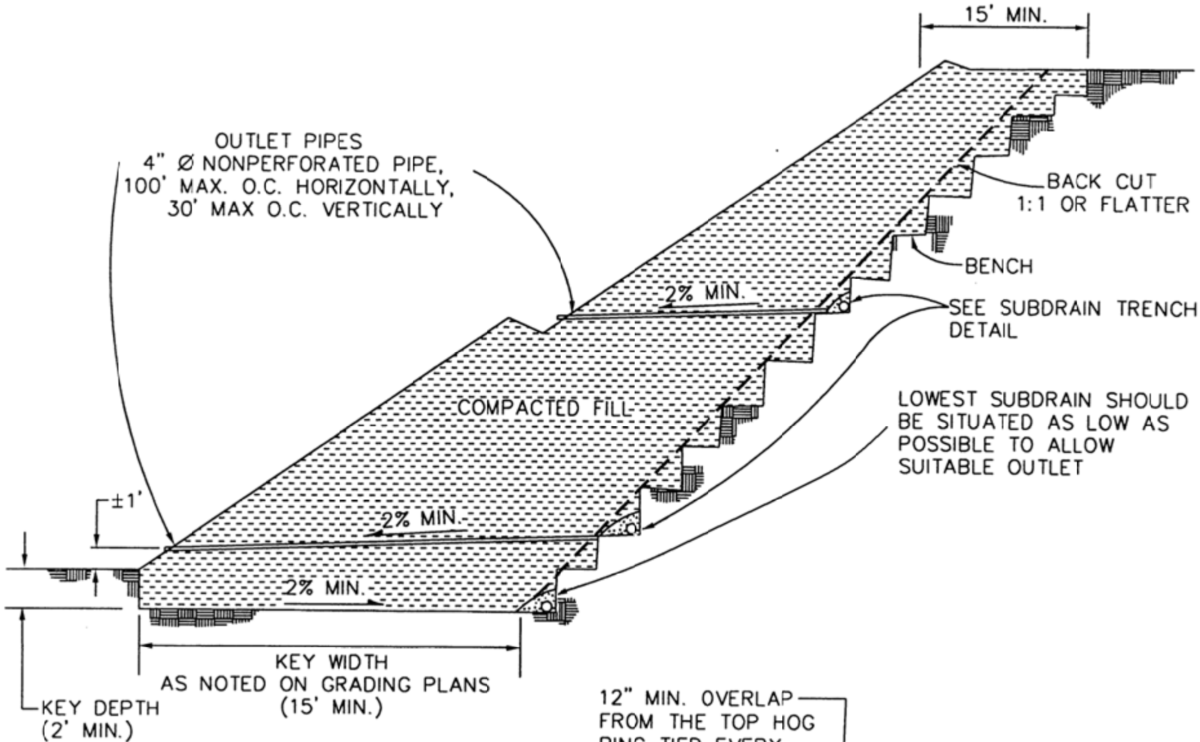


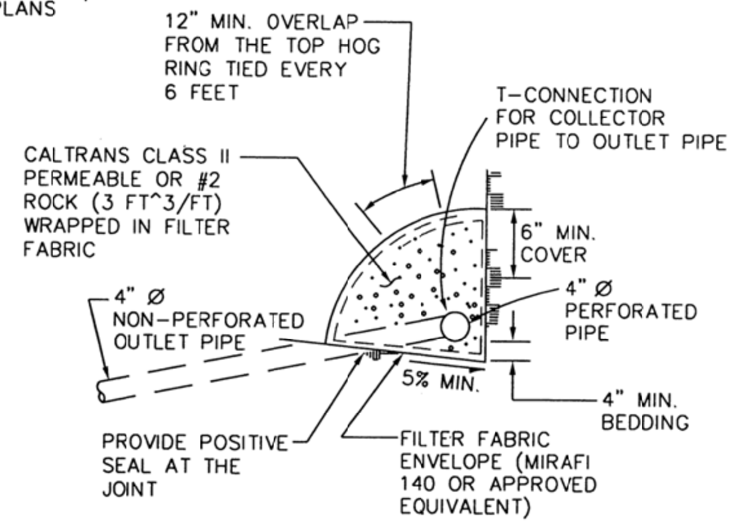
SUBDRAIN DETAIL



DETAIL OF CANYON SUBDRAIN OUTLET

CANYON SUBDRAINS	GENERAL EARTHWORK AND GRADING SPECIFICATIONS STANDARD DETAIL C





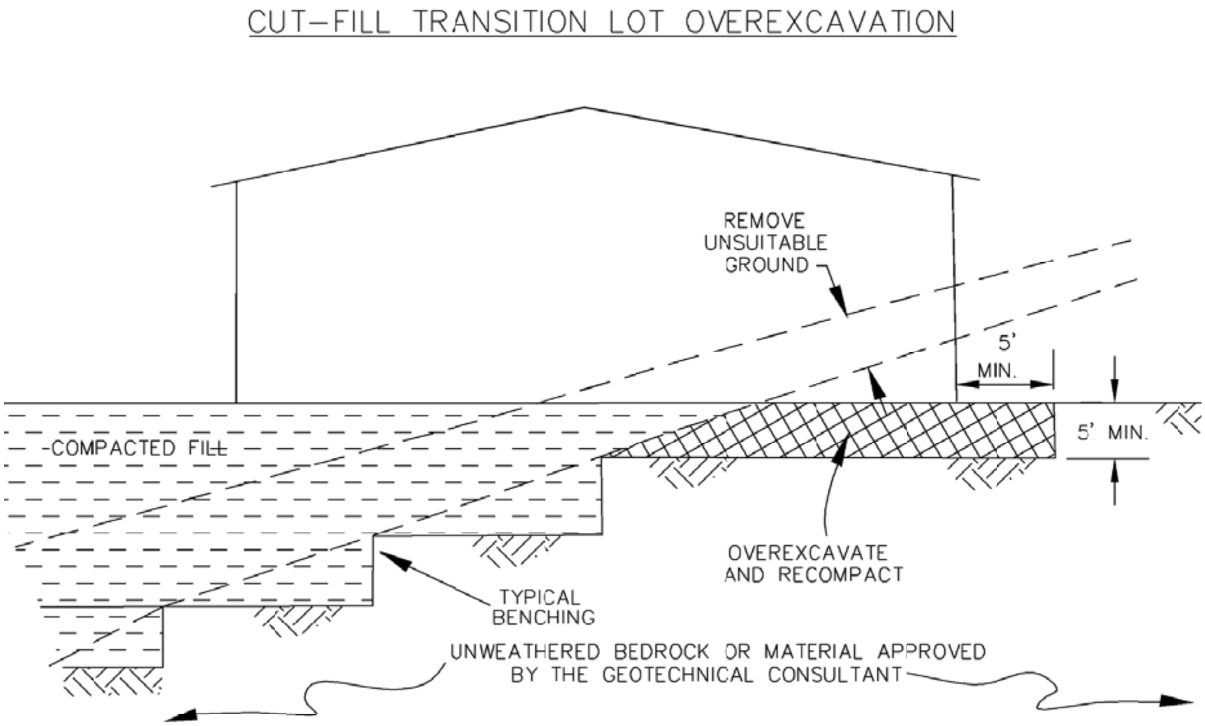
SUBDRAIN TRENCH DETAIL

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.

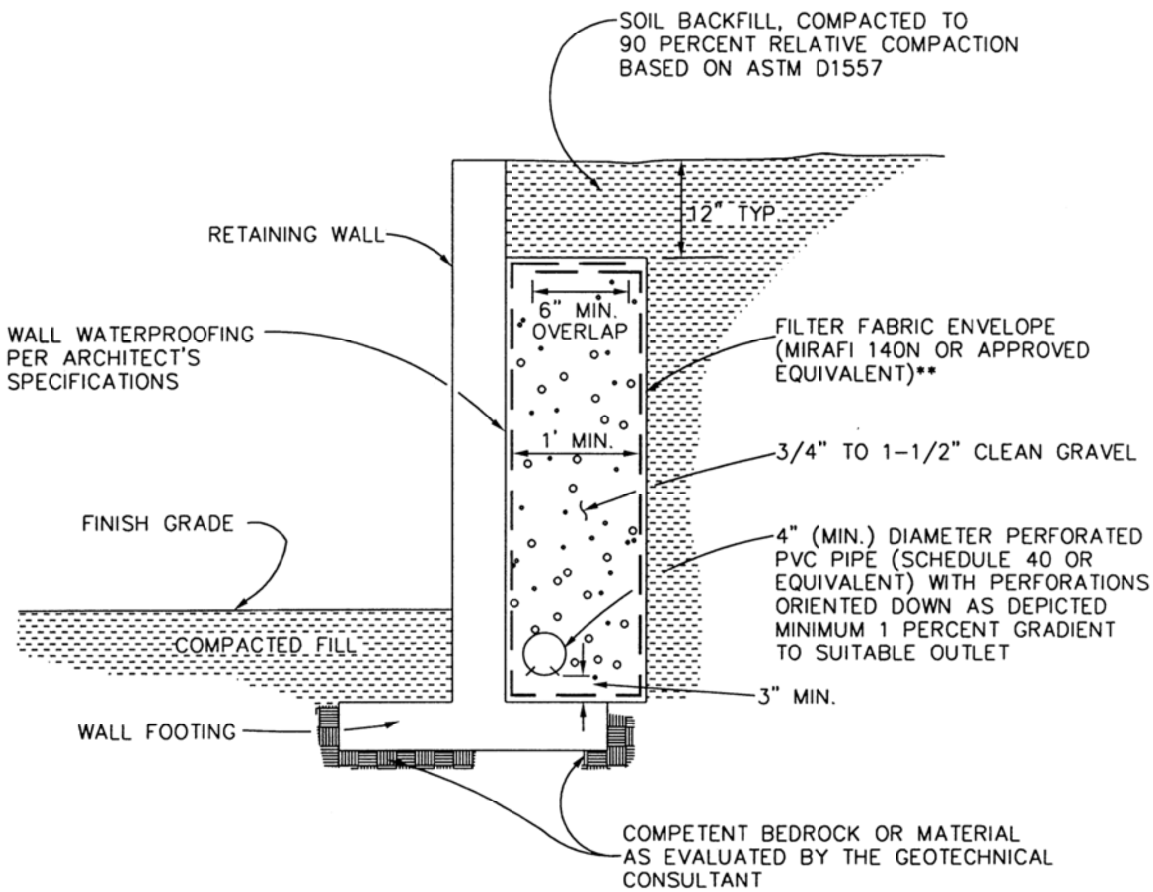
All outlet pipe shall be placed in a trench no wider than twice the subdrain pipe.

BUTTRESS OR REPLACEMENT FILL SUBDRAINS	GENERAL EARTHWORK AND GRADING SPECIFICATIONS STANDARD DETAIL D



TRANSITION LOT FILLS

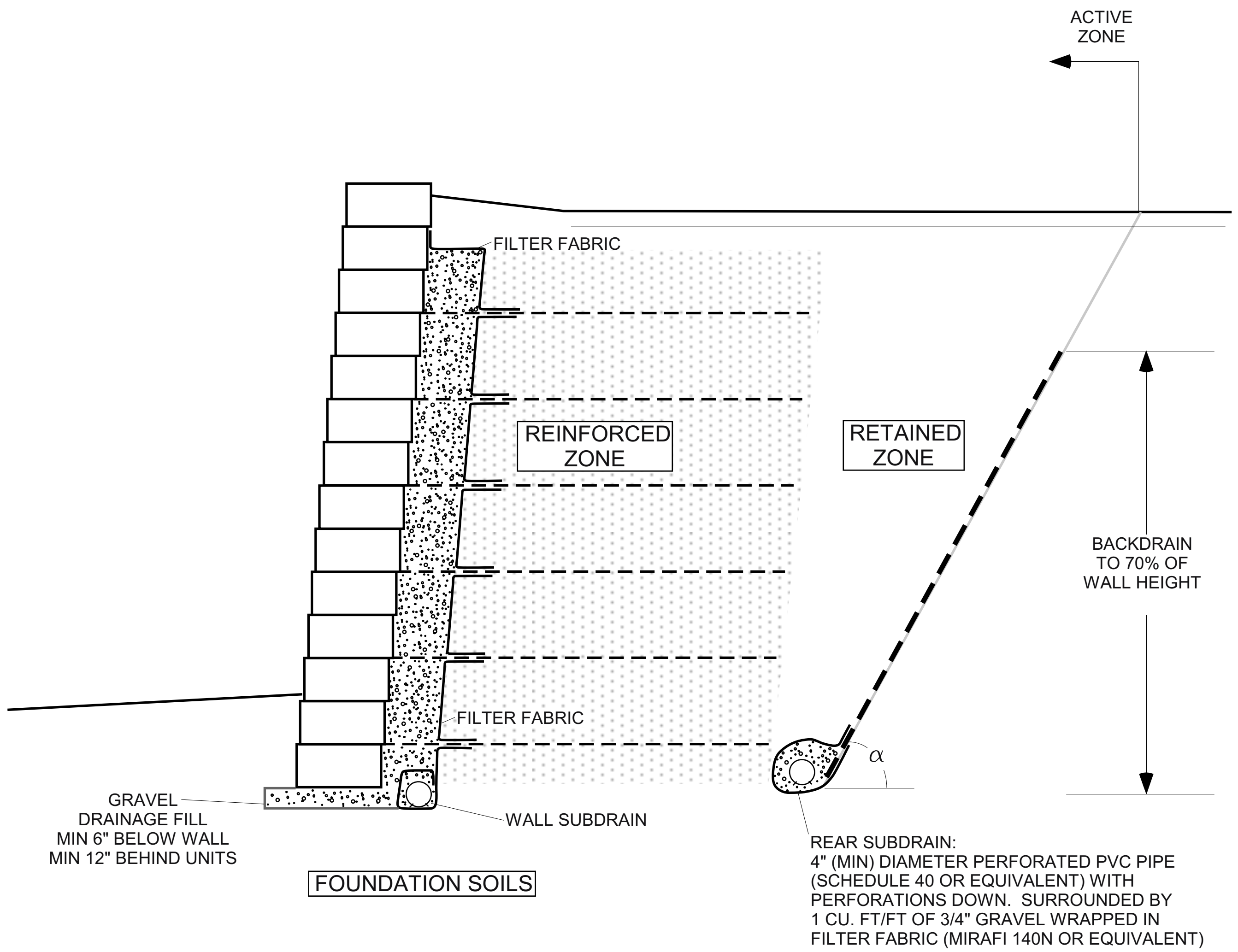
GENERAL EARTHWORK AND
GRADING SPECIFICATIONS
STANDARD DETAIL E



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



OUTLET SUBDRAINS EVERY 100 FEET, OR CLOSER, BY TIGHTLINE TO SUITABLE PROTECTED OUTLET

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

LIQUEFACTION ANALYSIS

San Elijo Reclamation Facility

Hole No.=B-2 Water Depth=20 ft Surface Elev.=45

Magnitude=6.6
Acceleration=0.49g