APPENDIX 4.6-A

GEOTECHNICAL INVESTIGATION FOR SALT CREEK SUBSTATION PROPONENT'S ENVIRONMENTAL ASSESSMENT (PEA)

Prepared by
KLEINFELDER WEST, INC.
5015 Shoreham Place
San Diego, CA 92122

March 7, 2008

GEOTECHNICAL INVESTIGATION
PROPOSED SDG&E OTAY RANCH SUBSTATION
CHULA VISTA, CALIFORNIA

March 7, 2007

A report prepared for:

San Diego Gas & Electric
8313 Century Park Court
San Diego, California 92123

Attention: Mr. Ron Brunton

GEOTECHNICAL INVESTIGATION
PROPOSED SDG&E OTAY RANCH SUBSTATION
CHULA VISTA, CALIFORNIA

Kleinfelder Project No. 67735

Prepared by:

KLEINFELDER WEST, INC.

Kevin M. Crennan, GE 2511
Senior Geotechnical Engineer

REGISTERED PROFESSIONAL ENGINEER
KEVIN M. CRENNAN
No. 2511
Exp. 12/31/09
GEOTECHNICAL
STATE OF CALIFORNIA

Scott Rugg, CEG 1651
Project Engineering Geologist

REGISTERED GEOLOGIST
SCOTT H. RUGG
NO. 1651
CERTIFIED ENGINEERING GEOLOGIST
STATE OF CALIFORNIA

KLEINFELDER WEST, INC.
5015 Shoreham Place
San Diego, California 92122
(858) 320-2000

March 7, 2008

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EXECUTIVE SUMMARY

Based on data collected during our investigation, it is our opinion that the proposed development is feasible from a geotechnical perspective. We developed the following summary of key geotechnical and geologic items relating to development of the proposed substation facility improvements.

- There are no known faults crossing the site or known ancient deep-seated landslides lying beneath the site that would affect the proposed construction. The site is located in the seismically active southern California area. The potential for liquefaction and dynamic compaction appears to be relatively low over the majority of this site.
- Topsoil/colluvial materials were encountered to approximate depths of 2 to 4 feet across the majority of the natural slope areas with localized depths up to about 8 to 10 feet. The thicker deposits are located near the base of slopes. Remedial removals of these materials should occur in areas to receive engineered fill or other proposed improvements. These soils are generally fine grained and moderately expansive and should not be placed as engineered fill within 3 feet of finished grade.
- Excavation of the topsoil/colluvium and formational sandstone can likely be achieved with heavy-duty excavation equipment. The on-site topsoil/colluvium and formation may be used as backfill or fill material provided it is prepared and placed in accordance with the recommendations contained in this report. Given the quantity of coarse grained material from cuts into formational materials, it is preferable not to place the fine grained colluvium below structural areas.
- Fill materials underlying the proposed retaining wall along the western portion of the existing access road consist of clay which exhibits lower strength parameters and higher compressibility than the granular formational materials. Granular fill materials generated from cuts into formational materials should be used as fill within the reinforced zone of the proposed segmented retaining walls
- Groundwater at the project site is located well below the depth of the proposed construction activities.

1.0 INTRODUCTION

This report presents the results of our geotechnical investigation for the SDG&E Otay Ranch Substation facility to be located in the Otay Ranch area of Chula Vista, California. Specifically the proposed substation is to be located just southeast of the recent extension of Hunte Parkway, and just west of the SDG&E easement. The results of our preliminary geotechnical siting study for the site were presented in our July 20, 2006 report and are included as Appendix D of this report.

The latitude and longitude coordinates of the substation are:

Latitude: 32.6189°N

Longitude: 116.9486°W

The site location is shown on the Vicinity Map, Figure 1. A site plan showing approximate site limits, proposed improvements and field exploration locations is presented as Figure 2, Site Plan/Geologic Map. Figure 2 is based on civil drawings prepared by SDG&E. An aerial photo of the site and surrounding area is presented on Figure 3.

1.1 PROJECT DESCRIPTION

We understand the proposed project will consist of the construction of a new 69/12kV distribution substation. Although we have not been provided with details of the proposed substation, we anticipate that equipment will consist of transformers, switch stands, circuit breakers, capacitor banks, switchgear, and a single story concrete masonry control building. Based on our review of the preliminary site plans and our discussions with SDG&E, the substation area would be about 250 feet by 270 feet and be accessed by a driveway from Hunte Parkway to the north. The substation pad will consist of 12 inches of compacted class II base course on top of compacted select fill subgrade in accordance with SDG&E Standard EW-6. The current site generally consists of gentle to moderately sloping hillsides which descend downward to the west, south and east to a natural drainage system below the site. Preliminary pad elevations are about 486 to 490 feet MSL. Grading for the substation pad will consist of performing cuts on the north and fills on the south, with cut and fill slope heights up to about 35 and 25 feet, respectively.

The majority of the southern side of the main access road will require construction of a geogrid-reinforced segmental block retaining wall (Keystone Wall) to accommodate road widening, with heights up to about 17 feet on the western end and decreasing to the east. The geogrid-reinforced wall is considered the most feasible and economical wall type at this location due to settlement considerations of placing new fill of variable height over existing fill of variable depth. The entire substation pad area will be secured with a perimeter masonry block wall.

1.2 PURPOSE AND SCOPE OF SERVICES

The purpose of our current geotechnical and geologic engineering services was to evaluate the soil and geologic conditions at the site and provide conclusions and recommendations for preliminary design of the proposed development. The scope of our investigation consisted of a literature review, subsurface investigation, geotechnical laboratory testing, engineering evaluation and analysis, and the preparation of this report.

The following geotechnical information and recommendations are presented in our report:

- Vicinity map and site plan showing approximate locations of soil borings and test pits;
- Logs of soil borings, test pits and laboratory test results;
- Discussion of field exploration methods and laboratory test procedures;
- Discussion of the site and subsurface conditions;
- Discussion of faulting and seismicity in the region;
- Discussion of potential geologic hazards, which may impact the site;
- A map showing faults and historical earthquakes in the region;
- Review of anticipated excavation conditions;
- Guidelines for earthwork construction, including recommendations for site preparation, fill placement, and compaction;
- Lateral earth pressures and recommendations for design of retaining walls;
- Discussion of the possible foundation types;
- Discussion of the possible retaining wall types;
- Soil parameters for design of structural mat foundations or conventional spread footings;

- Soil parameters for drilled pier design using the EPRI – MFAD computer program; and
- Preliminary screening of the soil properties affecting corrosion of concrete and steel.

The recommendations contained within this report are subject to the limitations presented in Section 6.0. An information sheet prepared by ASFE (the Association of Engineering Firms Practicing in the Geosciences) is also included as Appendix E. We recommend that all individuals utilizing this report read the limitations along with the attached document.

2.0 INVESTIGATIVE METHODS

2.1 GEOLOGIC EVALUATION

Our geologic evaluation consisted of reviewing aerial photographs, researching geologic reports and maps reasonably available to our office, and observation of the geotechnical conditions in the field at the time of our subsurface investigation. The site geology is shown on the Site Plan/Geologic Map, Figure 2 and the geology of the site area is shown Local Geologic Map, Figure 4.

We have reviewed the following documents in preparation of this report::

1. Grading Plan for Otay Ranch – Village 11, Phase III subdivision, prepared by Hunsaker & Associates, undated.
2. Drawings C-1 and C-2, Conceptual Grading Plan and Grading Profiles for the Otay Ranch Substation, prepared by San Diego Gas and Electric, dated January 30, 2007.
3. Stereoscopic Aerial Photographic Plates 210-32F-5 & 4, on file at the County of San Diego Cartographic Services, dated November 29, 1978.
4. Aerial Photographic Plate 78-E11, on file at the County of San Diego Cartographic Services, dated 1928.
5. Geology of National City, Imperial Beach and Otay Mesa Quadragles, Southern San Diego Metropolitan Area, California, Map Sheet 29, Michael P. Kennedy and Siang S. Tan, CDMG, 1977.
6. Geotechnical Investigation, Otay Ranch Village 11 Subdivision, Chula Vista, California, prepared by GeoCon, dated February 2000.

2.2 SUBSURFACE INVESTIGATION

The subsurface investigation included four borings and eight test pits. All activities were approved by a biologist and erosion control measures were implemented. Three small diameter borings were excavated on January 18, 2007 to depths between 36 and 40 feet. The 8-inch diameter borings were excavated with a limited access hollow stem auger by Tri-County Drilling. An additional boring was drilled to a depth of 91.5 feet on March 8, 2007 for the proposed retaining wall on the access road off of Hunte Parkway. An engineer from our office supervised the field operations and logged the borings.

Selected bulk, disturbed, and intact samples were retrieved from the borings, sealed, and transported to our laboratory for further evaluation. Our typical vertical sampling interval was five feet. We recorded the number of blows necessary to drive either a Standard Penetration Test (SPT) sampler or a California sampler at each sampling location. The borings were backfilled using bentonite chips and any remaining soil cuttings were spread out in the vicinity of the boring location. The approximate location of each boring is shown on Figure 2 and logs of borings are included in Appendix A.

Eight backhoe test pits were excavated on December 20, 2006. The depth ranged from about 4 to 12 feet. An engineering geologist from our office supervised the field operations and logged the pits. Selected bulk samples were retrieved from the borings and transported to our laboratory for further evaluation. The pits were backfilled with nominal effort.

2.3 LABORATORY TESTING

A limited laboratory testing program was conducted to substantiate field classifications and evaluate selected physical characteristics and engineering properties of the soils encountered. Moisture content, unit weight, plasticity index, sieve analyses, R-value, direct shear, expansion index and corrosion tests were performed in general accordance with the applicable ASTM or Caltrans test methods. Details of the laboratory testing program are presented in Appendix B.

3.0 SITE CONDITIONS

3.1 GEOLOGIC SETTING

The project area is situated in the Peninsular Ranges Geomorphic Province. This geomorphic province encompasses an area that extends approximately 900 miles from the Transverse Ranges and the Los Angeles Basin south to the southern tip of Baja California, and varies in width from approximately 30 to 100 miles (Norris and Webb, 1990). The province is characterized by mountainous terrain on the east composed mostly of Mesozoic igneous and metamorphic rocks, and relatively low-lying coastal terraces to the west underlain by late Cretaceous, Tertiary, and Quaternary age sedimentary rocks. Most of the coastal region of the County of San Diego, including the general site area, occur within this coastal region and are underlain by sedimentary rock. Specifically, the project site in this portion of the Province is underlain at depth by Quaternary-age and Tertiary-age (Eocene) sediments.

3.2 TECTONIC SETTING

The Peninsular Ranges are traversed by several major active faults (Figure 6). The Whittier-Elsinore, San Jacinto, and the San Andreas faults are major active fault systems located northeast of the site and the Rose Canyon, Newport-Inglewood (offshore), Coronado Bank, and San Diego Trough are active faults located to the west-southwest. Major tectonic activity associated with these and other faults within this regional tectonic framework is right-lateral strike-slip movement. These faults, as well as other faults in the region, have the potential for generating strong ground motions at the project site. Further discussion of faulting relative to the site is provided in the Faulting and Seismicity section of this report.

3.3 SITE DESCRIPTION

Site access is provided via a recently constructed access road from the south side of Hunte Parkway. This access road and the recently constructed extension of Hunte Parkway are part of a significant grading project for the Otay Ranch – Village 11 residential subdivision. This earthwork has included grading work on the north side of the proposed substation site and consisted primarily of cut depths up to about 30 feet on the east and fill depths up to about 100 feet below the access road. The earthwork resulted in the creation of a v-shaped cut made for the access road with slopes up to 30 feet in height and gradients of approximately 2:1 (horizontal to vertical).

An existing 96-inch RCP storm drain is located within the existing canyon fill below the western end of the access road and discharges into the drainage at the base of the slope. The pipe has a flow line elevation of about 490 feet at the cleanout near Hunte Parkway and slopes down at about 4 horizontal to 1 vertical to an elevation of about 421 at the energy dissipater. At the location where the pipe crosses the proposed retaining wall, the top of pipe is at about 480 feet, existing grade is about 508 feet and the proposed top of wall is 523 feet.

The south side of the site has remained relatively undisturbed and consists of gentle to moderately sloping hillsides which descend downward to the west, south and east to a natural drainage system below the site. The undeveloped portions of the site are covered with grasses which appear to have been previously used for cattle grazing.

3.4 SITE GEOLOGY AND SUBSURFACE CONDITIONS

Geologic units encountered in our borings, or mapped during our field evaluation included fill, topsoil/colluvium, and the Otay Formation. The areal extent of fill and Otay Formation is depicted on our Site Plan/Geologic Map (Figure 2). Note that the overlying topsoil/colluvium is not depicted although it generally increases in thickness downslope. Detailed descriptions of these units are provided in Appendix A (Boring Logs and Test Pits), and generalized descriptions are provided in the subsequent sections below.

Several tonal bands are apparent on aerial photography on hillslopes nearby the site that closely follow the surface topography. These bands are due to slight color variations between adjacent stratagraphic subunits and indicate that the geologic structure is generally horizontal. Regionally, the Kennedy and Tan geologic map (1977) indicate the structure has very low dips to the southwest.

3.4.1 Fill

Fill materials present along portions of the access roads are primarily associated with the construction of Hunte Parkway. Our review of previous topography indicates that the fill may be up to about 100 feet in depth. Boring B-4 in the access road near the intersection with Hunte Parkway encountered 91 feet of fill. Compaction of this fill was observed and tested by Geocon and was reported to a minimum 90 percent relative compaction (of ASTM D1557 modified proctor. The material consisted of lean clay with some fat clay. This material was not observed on the proposed site and may have been imported from some distance. Penetration resistance blow counts ranged from 32 to 74 blows per foot which indicates that the material appears adequately compacted.

3.4.2 Topsoil/Colluvium

Topsoil/Colluvium was encountered in all of our borings and test pits with the exception of Boring B4, which was performed in the existing access road. This material is related to natural soil development processes (pedogenesis) and movement downslope by precipitation and gravity. The topsoil/colluvium materials were generally encountered from the ground surface to depths of approximately 2 to 4 feet. However, colluvium depths of about 6, 8 and 10 feet were observed in Test Pits 2, 4 and 7, respectively. These locations are further downslope than the other explorations and likely represent greater accumulations of colluvium. As encountered, the topsoil/colluvium consisted of light brown to dark brown, dry to moist, soft to firm, sandy silt, sandy clay and clayey sand with some organics and pinhole porosity. It is compressible in its current conditions and will require removal and recompaction within areas of planned grading.

3.4.3 Otay Formation

The Pliocene-age Otay Formation has been mapped underlying the subject site (Kennedy and Tan, 1977), and was encountered in all of our explorations performed during our subsurface evaluation. Depending on site location and elevation, the Otay Formation typically consists of arkosic sandstone or claystone. As encountered in the borings and test pits for this site, the Otay Formation consisted of light brown and light gray, friable to weakly cemented, coarse-grained sandstone. Due to the low cementation, this material may also be considered an intermediate geomaterial that classifies as very dense sand. Recorded blow counts for the Otay Formation are considered relatively high, having a range of penetration of 2 to 5 inches for 50 blows using a 140 pound hammer dropped 30 inches with an attached modified California or split-spoon sampler.

We did not observe any significant clay beds within our explorations or on the full height of cut slope exposures. We also reviewed a geologic map of this area prepared by Geocon for the adjacent subdivision. Their map shows the site to be underlain by the coarse “gritstone” granular facies of the lower Otay Formation.

3.4.4 Groundwater

Groundwater was not encountered in any of our borings and is anticipated well below the proposed construction elevations. Perched groundwater in the filled drainage to the west of the site may be on the order of 225 to 230 feet in elevation. It should be noted that groundwater levels could fluctuate due to seasonal variations, irrigation, and other factors. Groundwater or seepage is not expected to be a constraint to the construction of the project or to be a design consideration.

4.0 DISCUSSIONS, ANALYSIS, AND RECOMMENDATIONS

4.1 POTENTIAL GEOLOGIC HAZARDS

Potential geologic hazards considered in our study include, surface rupture, seismic shaking, landslides, liquefaction, seismically induced settlement, tsunamis, seiches, flooding, and expansive soils. The following sections discuss these hazards and their potential at this site in more detail:

4.1.1 Faulting and Seismicity

The project vicinity is considered to be seismically active, as is most of southern California. Our review of the referenced geologic maps do not show any mapped fault traces extending through or nearby the site. We also reviewed stereoscopic aerial photographs and specifically looked for indications of faulting during our recent geologic reconnaissance. Based on these surface interpretive methods, we did not observe indication of faulting on or nearby the site.

The Rose Canyon fault zone is the closest active fault system to the site and is located approximately 11.3 miles (18.1 km) to the west. Studies indicate that the most recent earthquake on the Rose Canyon fault in San Diego occurred after A.D. 1523 but before the Spanish arrived in 1769. Two additional later earthquakes may have occurred, on offshore segments of the Rose Canyon fault in the 1800s.

The Rose Canyon fault zone consists of predominantly right-lateral strike-slip faults that extend south-southeast from La Jolla bisecting the San Diego metropolitan area. Various fault strands display strike-slip, normal, oblique, or reverse components of displacement which is typical of faults that have variations in strike and dip along their length. The fault zone extends offshore at La Jolla and continues north-northwest subparallel to the coastline. South of downtown San Diego, the fault zone splits into several splays that underlie San Diego Bay, Coronado, and the ocean floor south of Coronado. Portions of the fault zone in the Mount Soledad, Rose Canyon, and downtown San Diego areas have been designated by the State of California (CDMG, 1991, 2003) as being Earthquake Fault Zones.

A major strand of the potentially active La Nacion fault has been mapped approximately 3.8 miles (6.1 km) west of the site. The La Nacion fault zone is composed of several parallel to subparallel, west dipping normal faults that displace Tertiary and Quaternary deposits. Radiocarbon dates of unfaulted Holocene alluvium overlying the fault range from approximately 6,800 years to 13,400 years old (Hart, 1974). In addition, geomorphic features commonly associated with Holocene faulting, such as sag ponds and well-defined scarps, have not been observed along the La Nacion fault zone (Elliott and Hart, 1977). Furthermore, the California Geological Survey (CGS) does not consider the La Nacion fault zone to be an active or independent seismogenic source. Based on this data, we consider the seismic parameters associated with the closest known active fault, the Rose Canyon fault, more appropriate for design purposes. Based on the above information, the hazard with respects to ground rupture at the site is considered low. The locations of faults and earthquake epicenters are shown on Figure 5.

4.1.2 Surface Rupture

As previously discussed, the subject site is not underlain by a known active or potentially active fault. Therefore, the potential for ground rupture due to faulting at the site is considered low. Ground lurching is defined as movement of low density materials on a bluff, steep slope, or embankment due to earthquake shaking. Since there are slopes located on and adjacent to some of the project site, lurching or cracking of the ground surface as a result of nearby or distant seismic events is considered possible.

4.1.3 Seismic Shaking and CBC Seismic Design Parameters

The most significant seismic event likely to affect the project site would be a maximum moment magnitude 7.2 earthquake (Cao et al., 2003) resulting from the Rose Canyon fault zone (CDMG, 1998), located approximately 6.3 kilometers northeast of the project site.

This section presents our recommendations for seismic design parameters in accordance with the 2007 California Building Code (CBC) (CBSC 2007), which is based on the 2006 International Building Code (IBC). Based on our field investigation and using the 2007 CBC Table 1613.5.2, we classify the site as Site Profile C. This site is defined as very dense soil and soft rock with average shear wave velocities within the upper 100 feet between 1,200 ft/s (360 m/s) and 2,500 ft/s (760 m/s), average SPT N>50, or average su greater than or equal to 2,000 psf.

Based on the Site Class C designation and on the site location with respect to mapped spectral acceleration parameters SS and S1, Kleinfelder developed 2007 CBC seismic design parameters. The recommended seismic design parameters are summarized in Table 1 below.

Table 1
Recommended 2007 CBC Seismic Design Parameters

Design Parameter	Symbol	Recommended Value	2007 CBC / (ASCE 7) Reference(s)
Site Class	--	C	Section 1613.5.5
Mapped spectral acceleration for short periods (Site Class B)	S_s	0.94g	Section 1613.5.1
Mapped spectral acceleration for a 1-second period (Site Class B)	S_1	0.34g	Section 1613.5.1
Site Coefficient	F_a	1.026	Table 1613.5.3(1)
Site Coefficient	F_v	1.460	Table 1613.5.3(2)
MCE(1) Peak Ground Acceleration (S_M at T=0)	PGA_M	0.38g	n/a
MCE(1) spectral response acceleration for short periods	S_{MS}	0.96g	Section 1613.5.3 / (Section 11.4.3)
MCE* spectral response acceleration at 1-second period	S_{M1}	0.50g	Section 1613.5.3 / (Section 11.4.3)
Design Peak Ground Acceleration (S_D at T=0)	PGA_D	0.26g	(Section 11.4.5)
Design spectral response acceleration (5% damped) at short periods	$S_{DS} = 2/3 \cdot S_{MS}$	0.64g	Section 1613.5.4 / (Section 11.4.4)
Design spectral response acceleration (5% damped) at 1-second period	$S_{D1} = 2/3 \cdot S_{M1}$	0.33g	Section 1613.5.4 / (Section 11.4.4)

Table 1 Notes:

1. MCE: Maximum Considered Earthquake (2% probability of exceedance in 50 years).

4.1.4 Landslides

Landslides are deep-seated ground failures (several tens to hundreds of feet deep) in which a large arcuate shaped section of a slope detaches and slides downhill.

Landslides are not to be confused with minor slope failures (slumps), which are usually limited to the topsoil zone and can occur on slopes composed of almost any geologic material. Landslides can cause damage to structures both above and below the slide mass. Structures above the slide area are typically damaged by undermining of foundations. Areas below a slide mass can be damaged by being overridden and crushed by the failed slope material.

Several formations within the San Diego region are particularly prone to landsliding. These formations generally have high clay content and mobilize when they become saturated with water. Other factors, such as steeply dipping bedding that project out of the face of the slope and/or the presence of fracture planes, will also increase the potential for landsliding.

No 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. It is our professional opinion that the potential for landsliding is low.

4.1.5 Liquefaction and Seismic Settlement

The term liquefaction describes a phenomenon in which saturated, cohesionless soils temporarily lose shear strength (liquefy) due to increased pore water pressures induced by strong, cyclic ground motions during an earthquake. Structures founded on or above potentially liquefiable soils may experience bearing capacity failures due to the temporary loss of foundation support, vertical settlements (both total and differential), and undergo lateral spreading. The factors known to influence liquefaction potential include soil type, relative density, grain size, confining pressure, depth to groundwater, and the intensity and duration of the seismic ground shaking. The cohesionless soils most susceptible to liquefaction are loose, saturated sands and some silts.

The majority of the subject site is underlain at depth by weakly to moderately cemented sandstones or by compacted fill. Based on the dense nature of the on-site formational deposits as well as the absence of a shallow groundwater in those areas, it is our opinion that the potential for liquefaction and seismic related settlement across the majority of the site is low.

4.1.6 Flood Hazard

According to a Federal Emergency Management Agency (FEMA) flood insurance map overlay 2177F on the SANGIS database, the site is outside of a 100-year and 500-year floodplains and subject to minimal flooding. Based on review of topographic maps, the site is not located downstream of a dam or within a dam inundation area. In addition, based on our document review there are no dams or facilities upstream of the site that could cause inundation of the subject site. Based on this review and our site reconnaissance, the potential for flooding of the site is considered low.

4.1.7 Expansive Soils

Expansive soils are characterized by their ability to undergo significant volume changes (shrink or swell) due to variations in moisture content. Changes in soil moisture content can result from precipitation, landscape irrigation, utility leakage, roof drainage, perched groundwater, drought, or other factors and may result in unacceptable settlement or heave of structures or concrete slabs supported on grade.

A sample of the topsoil was tested for expansion index (UBC Standard 18-2). These test results indicated an expansion index (EI) of 46. Based on this result and our visual evaluation of topsoil and colluvial soil variability through the site, these materials may be classified in the medium expansion range (<50 EI) with the potential for high expansion in some areas. The granular materials of the Otay Formation will be present over the majority of the substation pad and will comprise the majority of cut materials to be used as compacted fill. These granular materials were not tested but are considered to have a very low to low expansion potential.

4.2 SLOPE STABILITY

The majority of existing and proposed site slopes are considered grossly stable without rigorous analyses due to their planned inclinations, strength of subsurface materials and lack of adverse bedding. However, Kleinfelder performed static and seismic slope stability analyses for the proposed retaining wall overlying the existing fill slope at the north end of the proposed access road. The proposed segmented wall reaches an exposed maximum height of about 17 feet near the intersection with Hunte Parkway. We used limit equilibrium computer program SLOPE/W (Geo-Slope, 2001) with Spencer's method of slices considering circular and block slip surfaces. Shear strength parameters were developed from the results of our field investigation, direct shear testing and engineering judgment. The results of the stability analysis for static and pseudo-static conditions are included as Figures 6 and 7, respectively.

The permanent fill slopes should be constructed no steeper than 2:1 (horizontal to vertical) and keyed and benched into approved materials. Fill materials should be placed and compacted in accordance with the Section 5.1.4 of this report. Fill keys, where required, should be a minimum of 15 feet wide and should extend a minimum of 3 feet into the competent formation material. For our analyses, we assumed the MSE wall will have geogrids every 3 feet vertically and the grids are at least 80 percent of the wall height. The grid length was increased to obtain the required minimum safety factor. The base of the wall should have a minimum horizontal distance of 7 feet from the finished slope surface and the height of the wall is measured from the base of the lowest block.

The external static and seismic factors of safety calculated in our slope stability analyses were above the generally accepted minimum factors of safety of 1.5 and 1.1, respectively. Based on the results of our field investigation and engineering evaluations, it is our opinion that the proposed wall geometry is stable provided that the internal geogrids are of sufficient length.

4.3 SITE GRADING

4.3.1 General

Based on our understanding of the project and the results of our investigation, we anticipate that grading for the substation pad will generally consist of making cuts up to approximately 30 to 35 feet in the northern and eastern portions of the site and placing fills up to about 20 to 25 feet in the southern and western portion of the site. Preliminary plans also indicate fill up 17 feet for a segmental geogrid-reinforced wall for widening the entrance of the access road from Hunte Parkway.

All site preparation and earthwork operations should be performed in accordance with applicable codes, including Chapter 15 of the City of Chula Vista Municipal Code. All reference to maximum dry density is established in accordance with American Society for Testing and Materials (ASTM) ASTM D 1557. We recommend that site earthwork and construction be performed in accordance with the following recommendations and the guidelines presented in the Guidelines for Earthwork Construction included in

Appendix D. In case of conflict, the following specific recommendations supersede those outlined in Appendix D.

In general, these earthwork requirements should be applied to the structure foundations, including an area extending at least 5 feet beyond their foundation perimeters. Also, these earthwork recommendations should be applied to flatwork such as driveways or walkways, except that the additional area being recompacted need only extend 1 foot beyond their perimeters.

4.3.2 Pre-construction Conference

We recommend that a pre-construction conference be held. Owner representatives, the civil engineer, geotechnical consultant, and contractor should be in attendance to discuss the plans and construction requirements of the project.

4.3.3 Construction Observation

The recommendations presented in this report are based on our understanding of the proposed project and on our evaluation of the data collected. The interpolated subsurface conditions should be evaluated in the field during construction. Final project drawings and specifications should be reviewed by the project geotechnical consultant prior to the commencement of construction.

A representative from our firm should be present during construction to evaluate the suitability of the various soils types exposed during excavation at the site for use as engineered fill. Also, all site preparation and fill placement should be observed and tested by a representative of our firm. This is especially true during the remedial removal and scarification process so that we can observe whether any undesirable material or conditions are encountered in the construction area.

4.3.4 Excavation Characteristics

The explorations completed at the site indicate the subsurface materials consist of loose to soft topsoil/colluvium, over friable to weakly cemented sandstones (or very dense soil) of the Otay Formation. Excavation into the on-site materials can likely be achieved with moderate to heavy effort with conventional heavy-duty excavation equipment. Segregation of the fine grained topsoil and colluvium from the granular formational materials should be anticipated. Depending on grading quantities, the fine grained materials should be exported or used in non-structural fills on site, including slopes.

4.3.5 Site Preparation

Prior to site grading, existing trees and shrubs will require removal. Existing underground structures and utilities (if any) should be completely removed as required to accommodate the proposed improvements. Excavations for removal of the above items should be dish-shaped and backfilled with properly compacted engineered fill. The actual locations of sanitary sewers, storm drains, water mains, and other utilities should be verified in the field at the time of construction. Abandoned utilities should be completely removed, and the loose backfill removed and replaced. The trenches created by relocating any existing utilities should be backfilled with properly compacted fill.

All deleterious, organic, and inert materials exposed at the surface should be stripped and isolated. The stripping work should include the removal of soil that, in the judgment of the geotechnical engineer or geologist, is uncertified, compressible, collapsible, or contains significant voids. The stripping operation should expose a firm, non-yielding subgrade that is free of voids, organics, and deleterious materials. The subgrade exposed at the bottom of each excavation should be observed by a qualified representative from our office prior to the placement of any fill to observe that potentially unsuitable soils have been removed. Additional removals may be required as a result of observation and testing of the exposed subgrade soils.

Based on our review of the preliminary project site plan, and anticipated remedial grading, cuts and fills up to 35 and 25 feet, respectively, in depth may be performed. To avoid potential differential settlement at cut/fill transitions under structural areas, we recommend that remedial grading be performed so that a minimum of 3 feet be undercut and replaced with properly compacted fill. As an alternative, the cut portion of the site may be sloped at about 4 horizontal to 1 vertical to transition deeper fill to cut. Details of cut / fill transitions are presented on Figure 8. Final recommendations should be provided upon review of the substation layout and evaluation of differential settlement for specific improvements. The excavated soil should be moisture conditioned, replaced and compacted, as recommended below. We recommend that foundation components of the proposed structures be founded either entirely in undisturbed Otay Formation or entirely in engineered fill materials; foundations of any given structure should not transition between native and fill support. This may be achieved by either overexciting the cut area and replacing with a similar depth of compacted fill or by deepening foundation excavations in fill to formational materials and placing a minimum 2-sack sand cement slurry back up foundation elevation.

We anticipate that on-site native materials will be used to complete the grading for the project. The formational materials of the Otay Formation will generally break down fairly well under compactive effort, but some oversize cemented sandstone may remain. Oversize material greater than 6 inches in diameter should be placed a minimum of 8 feet below finish grade in areas outside the substation pad, a minimum of 8 feet from the face of fill slopes, and not in areas where underground construction is planned such as tower foundations or trenches for ducts.

4.3.6 Recommendations for Treatment of Compressible / Potentially Expansive Soils

The site is covered with a variable thickness of potentially compressible and expansive topsoil/colluvium. The thickness of the potentially compressible soil in each exploration location is estimated as follows:

Table 2
Depth of Topsoil and Colluvium

Boring / Test Pit	Approximate Depth (feet)
TP1	5.5
TP2	6
TP3	4
TP4	8
TP5	2
TP6	2
TP7	10
TP8	3
B1	2
B2	2
B3	2

We recommend that existing potentially compressible soils within the limits of site grading be removed to native formation prior to the placement of engineered fill materials. Soils with an expansion index over 50 may be blended with other granular soils and used as embankment fill. The expansive soils may also be used as deeper compacted fill in non-structural areas but not placed in the outer portion of fill slopes. The outer portion is defined as the outer 15 feet from slope faces or the height of the slope, whichever is less.

4.3.7 Engineered Fill

Fill materials generated from the on-site formational soils are generally suitable for placement as compacted fill provided they are free of oversized rock, expansive clay, organic materials, and deleterious debris. Based on the medium expansion potential (EI=46) of one laboratory test and our geologic logging, the topsoil / colluvium is potentially expansive. Additional testing may be performed to further characterize these soils. Based on the anticipated quantities from cuts in granular formation, it should be feasible to either export the fine grained colluviuim or place in non-structural areas. Rocks or cemented formation greater than 3 inches in diameter should not be placed within 2 feet of finished grade. Oversize material in excess of 6 inches in diameter should not be used in structural fill within 8 feet of finished grade. Fill soil placed within the upper 4 feet of finished grade in structural areas should consist of granular material with a very low to low expansion index (expansion index of 30 or less) as evaluated by UBC Standard 18-2 (Expansion Index Test). Selective grading may achieve the recommended 4-foot zone of very low to low expansive soils in structural areas.

Fill should be moisture conditioned to or above optimum and be compacted to 90 percent or more relative compaction in accordance with ASTM D 1557. Expansive soils with an expansion index greater than 30 should be similarly compacted, but at a moisture content over 2 to 3 percent above optimum. Although the optimum lift thickness for fill soils will be dependent on the type of compaction equipment used, fill should generally be placed in uniform lifts not exceeding approximately 8 inches in loose thickness. Although not anticipated, oversized material, rocks, or hard lumps greater than 6 inches in dimension should not be used in compacted fills within 8 feet of finished grade.

In pavement areas, the upper 12 inches of subgrade soils should be moisture conditioned to a moisture content of at least optimum and compacted to 95 percent or more of the maximum laboratory dry density, as evaluated by ASTM D 1557.

4.3.8 Import Materials

Although not anticipated for this project other than aggregate base, we recommend that general import material consist of granular, very low to low expansive material (expansion index of 30 or less) as evaluated by UBC Standard 18-2 (Expansion Index Test) and with low corrosivity characteristics. Low corrosivity material is defined as having a minimum resistivity of more than 2,000 ohm-cm when tested in accordance with California Test 643, unless defined otherwise by the corrosion consultant. Import material should be evaluated by the geotechnical consultant at the borrow site for its suitability as fill prior to importation to the project site.

4.3.9 Temporary Slopes

Temporary cut slopes are primarily anticipated for the area behind the retaining walls between the north side of the proposed access road and Hunte Parkway, and into the slope to accommodate potential geogrid lengths for the walls on the southern side of the access road. Care should be taken to identify the location and protect all subsurface structures including the cleanout structure for the existing storm drain. Except as discussed with regard to utility trench excavation, temporary cut slopes in topsoil/colluvium or granular fill materials should not be steeper than 1.5:1. Cut slopes in clayey fill or underlying formational materials to overall excavation depths of 20 feet can be as steep as 1:1. If steeper side slopes should be necessary due to construction restrictions, or excavations are deeper than 20 feet, shoring and bracing should be considered and a specific geotechnical analysis performed. OSHA and Cal-OSHA requirements should be observed for all excavations. If excavations deeper than 20 feet below existing site grades will be made that are not going to be shored or braced, then slopes should be cut at a gradient of 1.5H:1V.

The contractor is responsible for the stability of temporary excavations and his "competent person" should perform regular inspections of any temporary excavations. The contractor should retain a competent geotechnical engineer to develop systems to mitigate the effects of settlement induced by excavations. On a case-by-case basis, the contractor should protect structures which fall on a wedge formed by a 2H:1V slope extending from the bottom of excavation, and on settlement-sensitive structures falling on a wedge 4 horizontal to 1 vertical slope extending from the bottom of the excavation. The protection systems proposed by the contractor should be reviewed by the client's geotechnical engineer prior to constructing these protective systems.

4.3.10 Permanent Slopes

Preliminary plans indicate that cut and fill slopes will have maximum height of 25 to 35 feet. In general, both cut and fill slopes up to a maximum height of 50 feet can be as steep as 2H:1V. If fill slopes higher than 50 feet will be constructed, we should review each situation on a case-by-case basis. Flatter side slopes or benching may be needed for fill slopes higher than 50 feet.

New fill slopes should not be constructed above existing topsoil or colluvial soils. Where new fill slopes will be built, the existing topsoil or colluvial soil should be excavated and a keyway constructed into the underlying formational materials. The dimensions and depth of the keyway will depend on final slope configurations and heights. For fill slopes constructed at 2H:1V up to 40 feet high, a keyway having a minimum width of 15 feet and a minimum depth of 3 feet into formational material would be appropriate. Figure 8 shows a typical keyway and benching detail.

New fill placed on existing slopes that are steeper than 5H:1V should be keyed and benched into the existing hillside. Keyway recommendations are presented in the preceding paragraph. Benches should be a minimum of 10 feet in width and spaced at no more than 4-foot vertical height intervals.

Subsurface drainage of slopes is not anticipated, but may be needed depending on slope configurations, facility locations, and the conditions encountered in the field during construction. We should review project plans prior to final design and prepare recommendations for subsurface drains, if needed. If any zones of specific seepage, are encountered during construction, they should be addressed as recommended by the geotechnical engineer in the field at that time.

4.3.11 Bulking and Shrinkage Factors

Estimates of engineered fill bulking and shrinkage factors are typically based on comparing laboratory compaction tests with the in-place density of the soil material as encountered during the subsurface evaluation. Due to limited lab testing due to high resistance of the sampler, and variations in existing and compacted soil densities, the bulking and shrinkage factors are to be considered very approximate. Based on the results of our laboratory testing and experience, it is our opinion that the topsoil/colluvium materials will have an approximate shrinkage factor on the order of 7 to 12 percent when excavated from their existing state and placed as compacted fills.

A bulking factor of approximately 5 to 10 percent is anticipated for materials of the Otay Formation.

4.4 UTILITY TRENCH EXCAVATIONS

4.4.1 Temporary Trench Excavations

We recommend that trenches and excavations be designed and constructed in accordance with OSHA regulations. These regulations provide trench sloping and shoring design parameters for trenches up to 20 feet deep based on a description of the soil types encountered. Trenches over 20 feet deep should be designed by the Contractor's engineer based on site-specific geotechnical analyses. For planning purposes, we recommend the following OSHA soil classifications be used in the table below:

Table 3
OSHA Soil Classifications

Fill, Topsoil/Colluvium	Type C
Otay Formation	Type B

The classification of Otay Formation considers the minimal cementation in the sands. A Type A classification is unlikely but possible if more cohesive or cemented zones are encountered. Temporary excavations should be constructed in accordance with OSHA recommendations. Excavations deeper than 5 feet should be shored or laid back on a slope no steeper than 1.5H:1V (horizontal:vertical) above the Otay Formation and 1H:1V within the Otay Formation. In the case of trench excavations, OSHA requirements regarding personnel safety should be met using appropriate shoring (including trench boxes), or by laying back the slopes in accordance with OSHA requirements. Temporary excavations that encounter seepage may require shoring or may be stabilized by placing sandbags or gravel along the base of the seepage zone. Excavations encountering seepage should be evaluated on a case-by-case basis. On-site safety of personnel is the responsibility of the contractor, and their designated "competent person" should perform regular inspections of all temporary excavations.

4.4.2 Pipe Bedding and Trench Backfill

Pipe bedding should consist of sand or similar granular material having a sand equivalent value of 30 or less. The sand should be placed in a zone that extends a minimum of 4 inches below and 12 inches above the pipe for the full trench width. The bedding material should be compacted to a minimum of 90 percent of the maximum dry density. Trench backfill above pipe bedding may consist of approved, on-site or import soils placed in lifts no greater than 8 inches loose thickness and compacted to 90 percent of the maximum dry density. Sand cement slurry is also acceptable.

It will be necessary to keep vibrations away from the immediate excavation area and provide adequate setback of stockpiled materials and construction equipment for a stable condition. It is recommended that the setback distance be one-half the excavation depth. Some minor sloughing may occur as the moisture content of the soils in the excavation walls dry out. Shoring and/or bracing of trenches may be required where construction personnel are working within excavations. Applicable governmental safety codes should be applied for safety of personnel.

4.5 SETTLEMENT OF DEEP FILLS

Settlement of deep fills occurs from self weight of the fill. This occurs slowly, even when subsurface and surface drainage is provided, and is a function of a number of variables including soil type, age of fill, degree of wetting and depth. Experience has shown that this consolidation may approach from 0.2 percent (for granular soils) to 0.5 percent (for clayey soils) of the fill thickness. We estimate that the long-term total fill settlement for granular fill from the Otay Formation would be on the order of 0.75 to 1.0 inches for fill depths of 30 to 40 feet, respectively. The settlement may be larger if the fine grained soil from topsoil and colluvium is incorporated into the fill thickness. This settlement is in addition to the static settlements due to loading from structures or new fill loading as discussed in Section 4.6.2. Specific settlement estimates can be provided once the locations of the proposed substation improvements are known.

Based on the approximate depth of 90 feet of existing fill below the proposed access road and a consolidation factor of about 0.4 percent, we estimate about 4 ½ inches of potential long-term settlement following completion of fill placement in about 2001. Although the actual magnitude and rate of settlement is dependent on several variables, experience has shown this can take about 10 to 20 years to occur with about half in the first 5 years. Based on these approximations, we estimate the fill may settle an additional 1 to 2 inches in the next 10 to 15 years. This settlement is in addition to the static settlements due to loading from the proposed retaining wall, as discussed in Section 4.6.2.

4.6 FOUNDATIONS AND SLABS FOR STRUCTURES

4.6.1 General

The proposed substation structures and walls may be supported on shallow spread and continuous footings and shallow and deep drilled piers founded on either engineered fill soils or undisturbed formational materials. Foundations for each individual structure should be supported on the same type of material, that is, either entirely supported by engineered fill or undisturbed Otay Formation. Foundations should not be supported on a combination of both materials such as may occur where there is a transition between fill and formational material. The fill soils below the footprint of each improvement should be prepared as stated in Section 4.3.7. All footing excavations should be observed by a representative of the geotechnical engineer prior to placing reinforcing or concrete to verify proper subgrade conditions.

Spread and continuous footings for the substation structures that will be founded on engineered fill soils can be designed using an allowable soil bearing pressure of 2,500 psf, for dead loads plus long-term live loads. Footings that are founded on undisturbed formational materials can be designed using an allowable soil bearing pressure of 5,000 psf, for dead loads plus long-term live loads. These values are based on a minimum width of 12 inches and may be increased by 500 psf for each additional foot of depth up to maximum of 4,000 psf for fill and 7,000 psf for undisturbed formational. Mat foundations that will be founded on engineered fill soils can be designed using an allowable soil bearing pressure of 4,000 psf, for dead loads plus long-term live loads and mat foundations that are founded on undisturbed formational materials can be designed using an allowable soil bearing pressure of 7,000 psf, for dead loads plus long-term live loads. These values can be increased by one-third for short term loads such as those due to wind and seismic forces.

All footings should be extended in depth as necessary so that no existing or proposed utility trenches will extend below a plane having a downward slope of 2H:1V from a line 9 inches above the bottom edge of the closest footing. In addition, no parallel trenches should be within 18 inches from the closest edge of the footing. New footings should not be excavated below the bottom of adjacently located existing building foundations.

Foundations should have a minimum width of 15 inches and have an embedment at least 12 inches below the lowest adjacent grade. Structural reinforcement should be provided by the project structural engineer for load carrying purposes.

4.6.2 Estimated Settlements

Estimated total settlements for the proposed improvements, constructed in accordance with the recommendations contained herein, are anticipated to be less than 1/2 inch. Estimated differential settlement between points 40 feet apart on continuous footings and/or isolated spread footings are anticipated to be less than 1/4 inch. These settlements are in addition to long-term settlement of the deep engineered fills discussed previously in Section 4.5.

4.6.3 Lateral Resistance

For passive resistance, we recommend using an equivalent fluid weight of 325 pcf for footings or grade beams poured neat against properly compacted select fill or Otay Formation. This lateral pressure assumes a horizontal surface for the soil mass extending at least 10 feet from the face of the footing, or three times the height of the surface generating passive pressure, whichever is greater. The upper 12 inches of material in areas not protected by concrete slabs should not be included in design for passive resistance to lateral loads. The coefficient of friction between the bottom of the footings or grade beams and the prepared soil can be assumed as 0.40. Passive and frictional resistance may be combined without reduction.

4.6.4 Concrete Slabs-On-Grade

Concrete slabs-on-grade can be used for light equipment pads. These pads should be supported by a minimum of 6 inches of compacted Caltrans Class II aggregate base over undisturbed Otay Formation or compacted fills. The aggregate base should be compacted to at least 90 percent of ASTM D 1557. As a minimum, these slabs should have a thickness of 6 inches and should be reinforced with No. 4 steel rebar placed mid-height and spaced at 12 inches on center in both directions. Additional reinforcement should be placed as required by the structural engineer.

Slab-on-grade floors for the substation facility, if any, should be underlain by formation or engineered fill compacted as described in Section 4.2.7. To provide uniform subgrade support, a 6 inch layer of clean free-draining sand, gravel or crushed rock conforming to Section 7.1 of Appendix C should be placed between the finished subgrade and the bottom of the concrete. The aggregate base should be compacted to at least 90 percent of ASTM D 1557.

4.7 DRILLED PIER FOUNDATIONS

Drilled pier lengths should satisfy criteria for downward, uplift and lateral loading. We understand that SDG&E will utilize computer program Moment Foundation Analysis Design (MFAD) to design deep foundations such as drilled piers. We recommend utilizing the following soil parameters for compacted fill from onsite materials. These values are intended for use in computer program MFAD only, values for other design analyses may be provided upon request.

Table 4
Recommended Soil Parameters For MFAD Analysis

Soil Type	Unit Cohesion (psf)	Friction Angle (degrees)	Total Moist Unit Weight (pcf)	Moisture Content (%)	Total Saturated Unit Weight (pcf)	Deformation Modulus Ep (ksi)	Strength Reduction Factor
FILL Silty Sand (SM)	0	32	120	12	144	1.0	1.0
OTAY FORM Sand (SM)	0	38	120	10	144	4.0	1.0

The upper 1-foot of material should be ignored in design.

For downward loading, we have considered end bearing and skin friction along the shaft embedment into formational materials. We recommend neglecting downward friction in fill soils due to the potential for settlement. Skin friction along the entire shaft length is considered for uplift. The depth of fill can be estimated from grading plans and should be verified at pier locations following earthwork operations. Unanticipated deviations from estimated fill depth prior to grading could result in incorrect lengths of reinforcing cages and potential delays to adjust length during construction.

We have performed preliminary analyses for a pier diameter of 3 feet using computer program SHAFT by Ensoft, Inc. The piers should have a minimum total depth of 10 feet, or a minimum embedment depth of 2 pile diameters into formational materials, whichever is deeper. Allowable shaft capacity for downward loading were calculated with a factor of safety of 3 on end bearing and 2 on skin friction. The capacity curves are presented in Figures 10 and 11 for compression and uplift, respectively. Pile design should consider the effects of pile spacing if the spacing is less than 3 diameters.

We estimate that total post-construction settlement of the CIDH pile foundations resulting from structural loads should not exceed 1 inch.

4.8 SELECTION OF RETAINING WALL TYPE

The selection of type of retaining wall for different site conditions and applications may be based consideration of numerous factors. For discussion purposes, wall types considered are conventional gravity walls (concrete cast-in-place, masonry block, etc), segmental mechanically stabilized earth with geogrid reinforcing (MSE), soldier beam and lagging (with or without tieback anchors), soil nail, secant pile, and sheetpile walls. The issues listed below are general considerations with a discussion of site specific conditions following.

- The relative cost of a wall is a function of whether the wall is primarily located within a cut condition into an existing slope, primarily a new fill condition, or both cut and fill. This condition affects the amount of soil that is excavated, stockpile and replaced. For cut conditions, soldier beam, soil nail or sheetpile walls do not require excavation beyond the wall limits. Although it is possible to fill about the cut portion of soil nail or soldier beam walls, extending the height only practical in short elnghts or heights. MSE walls require the most excavation to accommodate the geogrid reinforcing length for internal and external stability.
- The proximity to property lines can impact wall type. Permanent tieback anchors or soil nails require a permanent easement from property owners or municipalities. Other wall types may require a temporary construction easement for excavation slopes.
- Future land use behind wall should be anticipated. Geogrids for MSE walls are required for long term support and cannot be cut to accommodate future improvements such as utility trenches or foundations. Similarly, permanent tieback anchors or soil nails restrict future excavations. Gravity walls are generally the most accommodating.

- MSE walls typically require select granular fill in the reinforced zone, whereas other wall types can be designed for a lower quality material.
- Potential for wall settlement. MSE walls can tolerate significantly more total and differential settlement and deformation than concrete gravity walls, and can be constructed on a wider spectrum of ground conditions. This flexibility can significantly reduce the extent of subgrade improvement or the need for deep foundations.
- Global stability of walls constructed on slopes. For new wall geometries that result in unstable slopes, longer geogrid reinforcing in MSE walls has the ability to improve the safety factor.
- Visual aesthetics and desire for consistency of wall type. Site location may dictate wall type or facing. The use of several wall types may be most cost effective for sites with variable conditions, however, aesthetics may limit the umber of wall types or facings. MSE walls have a wider capability for plantable units or landscape terraces.
- MSE walls have a higher resistance to seismic loading than rigid walls and have typically performed better during seismic events.
- Long-term performance. Corrosive soils can impact long-term performance of ground anchors and contaminated soil can impact synthetic geogrids.

Retaining walls for widening the existing access road at the subject site can be generalized to three areas; 1) short walls (less than about 5 to 8 feet) on the north side of the access road which are cut into the existing slope, 2) the higher walls (over about 5 to 8 feet) on the western approximate 250 feet of the downslope (south) side of the access road off of Hunte Parkway, and 3) the lower walls (less than about 5 feet) on the eastern approximate 460 feet of the downslope (south) side of the access road.

We recommend that conventional concrete gravity walls be used for the upslope (north) side of the access road. This is primarily due to the cut condition into the existing slope and low wall height. Soldier beam and lagging or soil nail walls may also be considered to avoid excavation and stockpiling of wall backfill.

We recommend that MSE walls be used for the higher walls on the western downslope side of the access road. This is primarily due to the ability of MSE walls to tolerate settlement of the underlying clay fill and ability of the geogrid reinforcing to mitigate global instability caused by the new loading on a descending fill slope. The top of the temporary backcut ascending from the lowest geogrid may impact the width of the access road (particularly at the eastern end of this section) and should be addressed in project planning and design. For example, access may be restricted or blocked until the wall height is of sufficient height. The temporary backcut may also impact the existing storm drain or other utilities. Temporary shoring or utility relocation may be utilized in this case.

For aesthetic and continuity reasons, one continuous wall type of MSE may be desired for the eastern 450 feet on the downslope side of the road. However, different wall types should be considered due to the significant length of the wall and other considerations. Global stability and settlement are acceptable for gravity walls due to the low wall height. Excavation depths for foundation setback from the slope face will be similar (8 feet) for both wall types, but the MSE wall will likely result in slightly larger excavations when foundation widths and geogrid lengths are compared. The temporary backcut of the MSE excavation will encroach further into the access road and have a higher temporary impact on construction traffic. Distribution ducts and possible future subsurface utilities within the road would need to be located north of the geogrid zone.

4.9 CAST-IN-PLACE AND MASONRY BLOCK RETAINING WALLS

Lateral pressures acting against retaining walls can be calculated assuming that the backfill soils act as a fluid. The equivalent fluid weight (efw) value would depend on allowable wall movement. Walls which are free to rotate at least 0.5 percent of the wall height can be designed for the active efw. Walls which are restrained at the top or are sensitive to movement and tilting should be designed for the at-rest efw. Specific information for segmented retaining walls is provided in Section 4.9.

Our site specific study indicates that potential fill materials generated from cuts into the Otay Formation are suitable for use as retaining wall backfill, but that fill, topsoil and colluvium may not be suitable due to their expansive and slow-draining properties. Therefore, the following values assume that non- to low-expansive sandy soils (SP, SM, SC) will be used as backfill. Values given in the table below are in terms of equivalent fluid weight and assume a triangular distribution.

Table 5
Equivalent Fluid Weights (efw)
For Calculating Lateral Earth Pressures

Condition	Slope Inclination	Equivalent Fluid Weight (pcf)
Active	Level	35
	2:1	65
At-Rest	Level	55
	2:1	90

Fifty and thirty percent of any uniform areal surcharge placed at the top of the wall may be assumed to act as a uniform horizontal pressure over the entire wall for the at-rest and active cases, respectively. As a minimum, we recommend that a traffic surcharge equivalent to 2 feet of soil backfill be assumed as a surcharge for the at-rest condition. For this condition a pressure of 120 psf may be assumed to act as a uniform horizontal pressure over the entire height of the wall, H. Seismic induced lateral loading may be neglected for the site walls but can be provided if requested. We should be contacted where other point or line loads are expected so we can provide recommendations for additional wall stresses.

Walls should be provided with drains to reduce the potential for build-up of hydrostatic pressure. A typical drainage system could consist of either a prefabricated drainage board or a one- to two-foot-wide zone of Caltrans Class 2 permeable material immediately adjacent to the wall, with a perforated pipe at the base. The pipe should be discharged to an appropriate outlet, which is protected against erosion and becoming covered or plugged. For the prefabricated drainage board option, the geotextile manufacturer's recommendations should be followed for installation of a drainage fabric system.

Allowable foundation bearing pressure values described in previous sections of this report can be increased by one-third when calculating resistance caused by loads of short duration, such as earthquake loads. Restraining passive pressure and friction values should not be increased by this amount, but a lower factor of safety that is normally applied to static loads could be used. This factor of safety for dynamic load conditions should not be less than 1.2. Backfill for retaining walls should consist of predominately granular materials from on-site excavations or imported fill. All backfill should be placed in 8-inch loose lifts, moisture-conditions to 2 percentage points above optimum moisture content, and compacted to at least 90 percent relative compaction.

Wall backfill should be compacted by mechanical methods to at least 90 percent relative compaction in accordance with ASTM D 1557. For all retaining walls, we recommend a minimum horizontal distance from the outside base of the footing to daylight of 7 feet for slopes of less than 20 feet in height, and 10 feet for slopes of greater heights.

4.10 MECHANICALLY STABILIZED EARTH (MSE) RETAINING WALLS

The following are our geotechnical recommendations for the wall designer. Wall design and construction should be performed in general accordance with the current standards "Mechanically Stabilized Earth Walls and Reinforced Soil Slopes Design and Construction Guidelines," Publication No. FWHA-NHI-00-043, dated March 2001.

1. Recommended total stress and unit weight properties of the retained and foundation soils for segmented wall designs are summarized below. These values are for the existing clay fill along the access road. Other values may be provided if walls are used in other areas such as the base of fill slopes.

Soil Zone	Soil Unit Weight (pcf)	Internal Friction Angle	Cohesion (psf)
Retained	125	24°	300
Foundation	125	24°	300

2. Recommended ranges of total stress and unit weight properties of typical soil types for segmented retaining wall drainage fill and the reinforced (infill) soil zone are summarized below.

Soil Type	Soil Unit Weight (pcf)	Internal Friction Angle	Cohesion (psf)
Sand, Silty Sands (SW, SP, SM)	125	32°	0

The fill used within the reinforced zone should have a minimum friction angle of 32 degrees when tested in accordance with ASTM D-300, a maximum plasticity Index of 6, no rocks greater than 3 inches in maximum dimension, contain at least 40 percent of material smaller than ¼ inch in size, and have a maximum of 20 percent passing the #200 seive.

3. The allowable bearing capacity of the retaining wall foundation soils can be assumed as 3,000 pounds per square foot on engineered fill. This value is based on a safety factor of 2.0 for wall design.

4. A foundation leveling pad is recommended to provide a firm level surface on which to place the base course units at the design elevations. A minimum of 6 inches of compacted aggregate base is recommended. The compacted aggregate base should meet the following grading requirements:

Sieve Size	Percent Passing
1 inch	100
No. 4	35-80
No. 10	20-65
No. 40	10-35
No. 200	0-10

The top of the foundation leveling pad should be compacted by a minimum of three passes of a vibrating base plate compactor. The vibrating compactor should have a minimum weight of 200 pounds and a minimum vibration frequency of 1600 cycles per minute.

5. The minimum embedment of retaining wall foundation should be either the exposed wall height in feet multiplied by a factor of 0.20, or 1.5 feet, whichever is greater. In addition, the wall foundation should have a minimum 8-foot setback from the surface of the finished slope.

6. The dry-stack mortarless construction method for segmented retaining walls bearing on an aggregate bearing pad is generally considered a flexible structure that can tolerate total settlements on the order of 3 to 6 inches with differential settlements on the order of 1 percent of the total wall height. Anticipated settlements for the maximum foundation pressures are not expected to exceed one inch for a properly compacted fill and subgrade. This settlement is in addition to potential long term consolidation of the fill due to self weight and increase in moisture content.

7. Walls should be designed for any surcharge loading due to slopes and traffic. For walls that may be subjected to light surcharge loading only, we recommend that a surcharge loading of at least 100 pounds per square foot be incorporated into the design. The access road wall should be designed for the trucks that deliver the transformers or other large equipment for the substation. These surcharge loads should be provided when identified.

8. Critical segmented retaining walls should also be designed for seismic loading. These may be walls that support slopes below the substation or along the access road to the substation. Based on a horizontal acceleration of 0.23g, the resultant seismic force (in pounds) for each linear foot of wall can be estimated as $5H^2$ where H is the height of the wall (in feet) above its base. The resultant seismic force acts at 0.6H above the wall base.

9. The wall designer should incorporate a drainage system into all walls. A minimum of 12 inches of drainage fill should extend behind each wall to within 1 foot of final grade. Drainage fill should consist of free-draining, sound, durable particles of 1-inch minus or No. 57 crushed stone conforming to the following gradation:

Sieve size	Percent Passing
1 inch	100
3/4 inch	75-100
No. 4	0-10
No. 50	0-5

A perforated PVC pipe should be placed near the bottom of the wall and sloped a minimum of 0.5%. The drain outlets should be protected and the locations documented. In addition, a prefabricated drainage composite, such as Miradrain® 6000, should be used to collect water in the areas of the backcut with the potential for seepage, such as a fill/formation interface. This may be waived for the majority of the site with backcut located within fill but may be required in cut areas on the eastern end of the wall.

10. We recommend that the wall designer incorporate the following minimum design criteria into the segmented retaining wall design:

External Stability	Minimum Factor of Safety
Sliding	1.5
Overturning	1.5
Bearing Capacity	2.0
Local Stability	1.5
Shear Facing Units	1.5
Global Stability	1.3

11. Geogrid Reinforcement Installation

   a. The geogrid should be installed at the wall height, horizontal location, and to the extent as shown on the project construction plans, or as directed by the design engineer.

   b. The geogrid should be laid horizontally on compacted infill and connected to the concrete wall units. Embedment details should be consistent with details utilized in evaluation of connection strength.

c. Correct orientation (roll direction) of the geogrid should be verified by the contractor.

d. The geogrid should be pulled taut and free of wrinkles prior to placement of soil fill. The geogrid may be secured in placed with staples, pins, sandbags, or fill as required by fill properties, fill placement procedures, or weather conditions, or as directed by the design engineer.

e. The procedure for tensioning the geosynthetic geogrid should be uniform throughout the wall length and height.

f. Overlaps:

   1. Overlap of the geogrid in the design strength direction should not be permitted. The design strength direction is that length of geogrid perpendicular to the wall face and should be one continuous piece of material.

   2. If required, overlaps of adjacent rolls should be in accordance with manufacturer's recommendations. Geogrid should be continuous throughout wall length, except for curves.

12. Fill Placement Over Geogrid

a. Reinforced wall fill material should comply with the specified soil parameters in 2 above and be placed in maximum 8-inch compacted lifts on the geogrid or as directed by the design engineer. Each lift should be compacted to a minimum density of 90 percent of ASTM D 1557.

b. The geogrid should be pretensioned by hand to remove wrinkles. Tensioning is usually facilitated by the use of steel rakes. Constant tension should be applied to each section of geogrid until soil fill has been placed. Soil fill should be placed, spread, and compacted in such a manner that prevents the development of wrinkles and/or movement of the geogrid.

c. Only hand-operated compaction equipment should be allowed within three feet of the front of wall face.

d. If possible, fill should be placed from the wall face outward to ensure that the geogrid remains taut. Soil should be placed in uniform lifts.

e. Tracked construction equipment should not be operated directly on the geogrid. A minimum fill thickness of 8 inches is required prior to operation of tracked vehicles over the geogrid. Turning of tracked vehicles should be kept to a minimum to prevent tracks from displacing the fill and damaging the geogrid.

f. If, in accordance with manufacturer's recommendations, rubber-tired equipment may pass over the geogrid at slow speeds (less than 10 mph), sudden braking and sharp turning should be avoided.

g. Surface draining during, and after, construction of the wall should be provided to minimize water infiltration in the reinforced soil zone.

13. Common soil backfill placed behind the reinforced wall fill (infill) material should be placed as engineered fill.

4.11 PAVEMENT SECTIONS

For purposes of preliminary analysis of pavements, we performed an R-value test on a soil sample considered representative of potential subgrade materials on-site. This should be considered preliminary due to the length and change in elevation of the sloping access road. Our limited test result indicates an R-value on the order of 21. In general, the topsoil and colluvial soils will have low R-values and the granular sands of the Otay Formation will have higher R-values. Based on the quantity of material generated from cuts into formation, a more economical pavement section would result from using granular material from the Otay Formation in the upper two feet. However, it may not be economical to remove the clayey fill on the access road. For preliminary design we have assumed an R-Value of 15.

Actual pavement recommendations should be based on R-value tests performed on bulk samples of the soils that are exposed at the finished subgrade elevations across the site at the completion of the grading operations.

4.11.1 Flexible Pavements

Flexible pavement sections have been evaluated in general accordance with the Caltrans method for flexible pavement design. Traffic indices of 4.5, 5.0, and 6.0 were used to calculate the design thickness. Recommendations for other traffic indices can be provided upon request. Recommended flexible pavement sections for these conditions are given in the following table:

Table 6
Flexible Pavement Sections

Traffic Index	Asphalt Concrete (inches)	Aggregate Base (inches)
4.5	3	6.5
5.0	3	8
6.0	4	9.5

Flexible pavements should be constructed in accordance with Section 302-5 of the Standard Specifications for Public Works Construction (Greenbook), 2000 edition. Aggregate base should comply with the specifications in Section 26 of Caltrans Standard Specifications. Aggregate base should be compacted to a minimum of 95 percent relative compaction (ASTM D 1557).

4.11.2 Rigid Pavement

Portland cement concrete pavement (PCCP) may be desirable at entry points and other locations where tight-turning heavy vehicles are expected. For moderate commercial usage, we recommend a 7-inch thick rigid concrete pavement over 4 inches of aggregate base compacted to at least 95 percent relative compaction (ASTM D 1557). The subgrade beneath the aggregate base should be compacted to at least 95 percent relative compaction (ASTM D 1557). Aggregate base should comply with the specifications in Section 26 of Caltrans Standard Specifications.

According to the 2001 UBC, Section 1701.5, non-structural slabs-on-grade and site work concrete fully supported on earth and concrete are exempt from inspection by a special inspector. Therefore, pavements can be designed with a higher compressive strength and are exempt from special inspection. We recommend a 28-day compressive strength of at least 4,000 pounds per square inch for the pavement concrete mix design. The concrete mix should also be designed for a slump not exceeding 4 inches. Thickened edges should be used along outside edges of concrete pavements. Edge thickness should be at least 2 inches greater than the concrete pavement thickness and taper to the actual concrete pavement thickness 36 inches inward from the edge. Integral curbs may be used in lieu of thickened edges.

Continuous sections of concrete pavement should have construction joints spaced on an approximate 12-foot square grid system or less. All longitudinal or transverse control joints should be constructed by saw-cutting, hand forming, or placing pre-molded filler such as zip strips. Longitudinal or transverse construction joints should be keyed or doweled to mitigate differential movement. In general, longitudinal or transverse construction joints should be keyed or doweled to mitigate differential movement.

4.12 FLATWORK

To reduce the potential manifestation of distress to exterior concrete flatwork due to movement of the underlying soil, we recommend that such flatwork be constructed with crack-control joints at appropriate spacing as designed by the structural engineer. Subgrade should be prepared in accordance with the earthwork recommendations presented earlier in this report. Positive drainage should be established and maintained adjacent to flatwork.

4.13 PRELIMINARY CORROSIVE SOIL SCREENING

A preliminary corrosive soil screening for representative on-site soil 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 a composite soil sample of the Otay Formation obtained during our subsurface evaluation. We do not anticipate that concrete and ferrous metals will be in contact with the more corrosive topsoil and colluvium.

Table 7
Corrosion Test Results

Boring	Depth (ft)	pH	Sulfate (ppm)	Chloride (ppm)	Minimum Resistivity (ohm-cm)
B-3	3-10	9.4	<10	10	1,400

Concrete in contact with soil or water that contains high concentrations of soluble sulfates can be subject to chemical deterioration. Based on the UBC criteria (UBC, 2001), the potential for sulfate attack is negligible for water-soluble sulfate contents in soil ranging from 0.00 to 0.10 percent by weight (0 to 1,000 ppm), and moderate for water-soluble sulfate contents ranging from 0.10 to 0.20 percent by weight (1,000 to 2,000 ppm). The potential for sulfate attack is severe for water-soluble sulfate contents ranging from 0.20 to 2.00 percent by weight (2,000 to 20,000 ppm) and very severe for water-soluble sulfate contents over 2.00 percent by weight (greater than 20,000 ppm). Based on the corrosion test results, the sulfate content is less than 10 ppm, therefore the potential for sulfate attack is considered negligible.

Our corrosion screening tests are preliminary in nature. Additional sampling and testing may be warranted after completion of grading if improvements will be in contact with soils other than the granular Otay Formation.

4.14 SURFACE DRAINAGE

Foundation performance depends greatly on how well the runoff waters drain from the site. This drainage should be maintained both during construction and over the entire life of the project. Final elevations at the site should be planned so that positive drainage is established around structures. Positive drainage is defined as a slope of 2 percent or more for a distance of 5 feet or more away from structure foundations.

4.15 SLOPE PROTECTION AND MAINTENANCE

Although graded slopes on this site are anticipated to be grossly stable, the surficial soils may be somewhat erodible due to low cohesion of the sands. For this reason, the finished slopes should be planted as soon as practical after the end of construction. Cut slopes into the Otay Formation may be difficult to plant. Preferably, deep-rooted plants adapted to semi-arid climates should be used. Due to the close proximity to a natural drainageway, we anticipate that aggressive erosion control measures should be implemented. In general, runoff water should not be permitted to drain over the edges of slopes unless that water is confined to properly designed and constructed drainage facilities.

5.0 ADDITIONAL STUDIES

The review of plans and specifications, and the observation and testing by Kleinfelder of earthwork related construction activities, are an integral part of the conclusions and recommendations made in this report. If Kleinfelder is not retained for these services, the client will be assuming our responsibility for any potential claims that may arise during or after construction. The required tests, observations, and consultation by Kleinfelder during construction includes, but is not limited to:

- A review of plans and specifications;
- Observation of site clearing;
- Construction observation and density testing of fill material placement, trench backfill and subgrade preparation; and
- Observation of foundation excavations and foundation construction.

6.0 LIMITATIONS

Our firm has prepared this geotechnical investigation report for the exclusive use of our client. Kleinfelder offers various levels of investigative and engineering services to suit the varying needs of different clients. Although risk can never be eliminated, more detailed and extensive evaluations yield more information, which may help understand and manage the level of risk. Since detailed evaluation and analysis involve greater expense, our clients participate in determining levels of service, which provide adequate information for their purposes as acceptable levels of risk. SDG&E has reviewed our scope of work and determined that it does not need or want a greater level of service than that being provided for this design study phase. A brochure prepared by ASFE (Association of Firms Practicing in the Geoscience) has been included in Appendix D of this report. All individuals reading this report should also read the attached brochure.

The services provided under this contract as described in this report include professional opinions and judgments based on the data collected. These services have been performed according to our agreed scope of services at the time the report was written. No warranty is expressed or implied. This report is issued with the understanding the owner chooses the risk he wishes to bear by the expenditures involved with the construction alternatives and scheduling that is chosen.

Regulations and professional standards applicable to Kleinfelder's services are continually evolving. Techniques are, by necessity, often new and relatively untried. Different professionals may reasonably adopt different approaches to similar problems.

The conclusions and recommendations presented in this report are based on information obtained from the review of documents, nine borings, observations of our engineer and geologist, our laboratory testing program, and our experience. It is the client's responsibility to see that all parties to the project, including the designer, contractor, subcontractors, etc., are made aware of this report in its entirety.

7.0 REFERENCES

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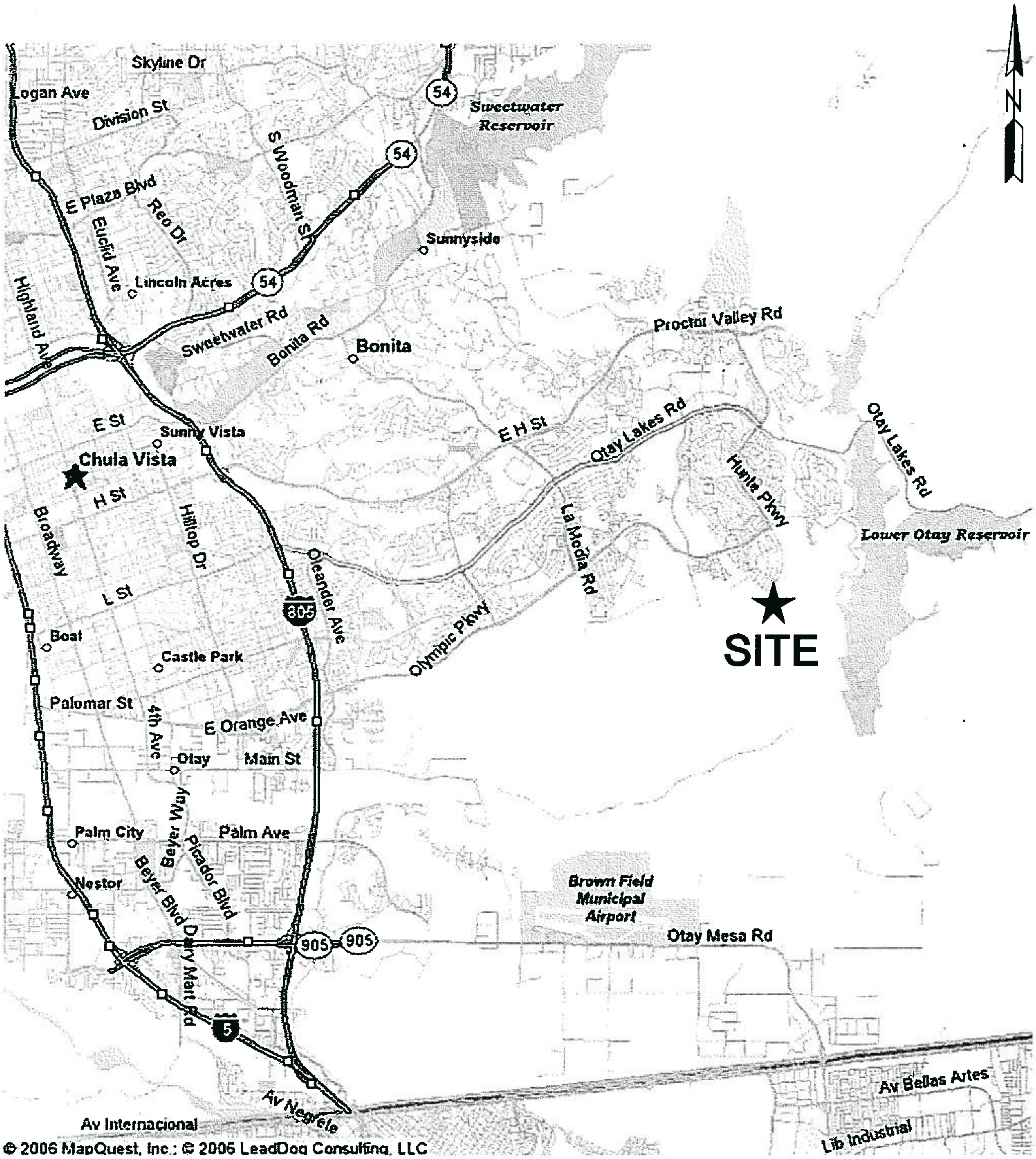
FIGURES

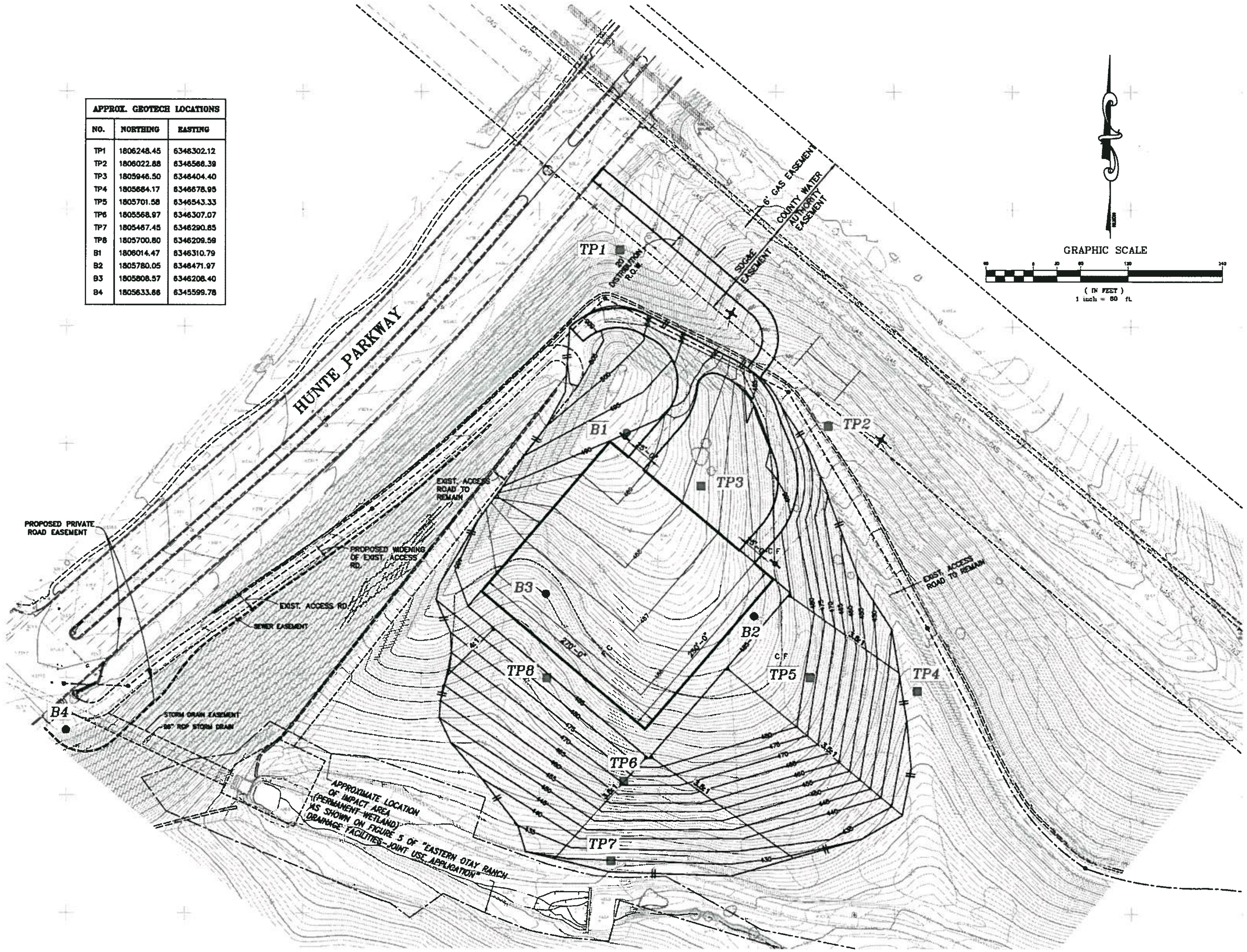


NOT TO SCALE

KLEINFELDER 5015 SHOREHAM PLACE SAN DIEGO, CALIFORNIA 92122		VICINITY MAP OTAY RANCH SUBSTATION SITE OTAY, CALIFORNIA	FIGURE 1
CHECKED BY: SHR	FN: 67735SITE		
PROJECT NO. 67735	DATE: 04/2006		

PRELIMINARY
FOR PLANNING PURPOSES ONLY

APPROX. GEOTECH LOCATIONS		
NO.	NORTHING	EASTING
TP1	1806248.45	6346302.12
TP2	1806022.88	6346566.39
TP3	1805946.50	6346404.40
TP4	1805684.17	6346678.95
TP5	1805701.58	6346543.33
TP6	1805568.97	6346307.07
TP7	1805467.45	6346290.85
TP8	1805700.80	6346209.59
B1	1806014.47	6346310.79
B2	1805780.05	6346471.97
B3	1805808.57	6346208.40
B4	1805633.86	6345599.78

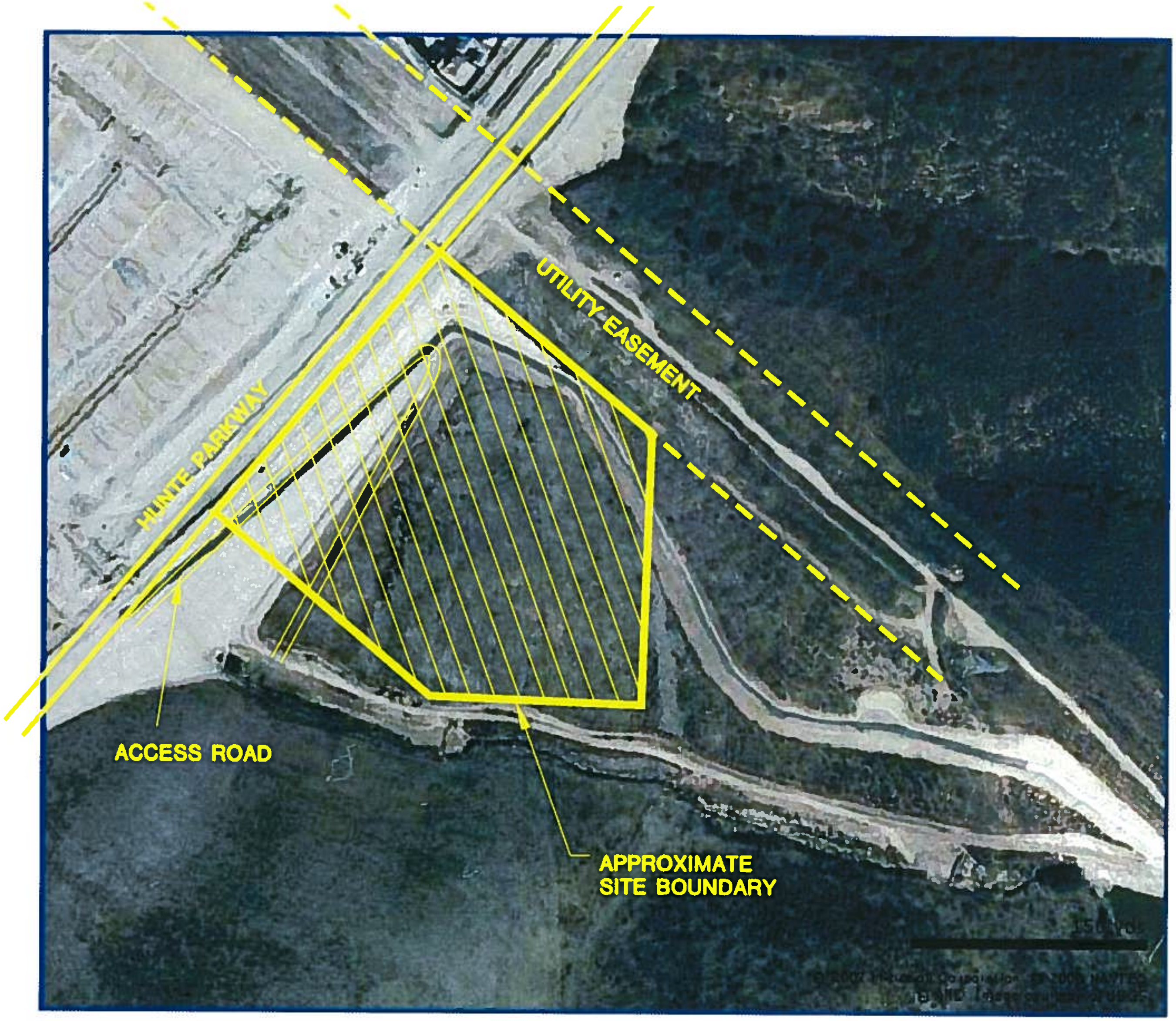


LEGEND

- RETAINING WALL
- SEWER MANHOLE
- SEWER LINE
- CUT/FILL LINE
- ELEC. TRANSMISSION LINE
- FINISHED CONTOUR LINES
- EXISTING COUNTER LINES
- GAS TRANSMISSION LINE
- WATER
- DRAINAGE SWALE
- NEW TRANSMISSION STEEL CABLE POLE
- NEW TRANSMISSION STEEL POLE
- SOIL BORING — Bx
- TEST PIT — TPx

PRELIMINARY

KLEINFELDER 5015 SHOREHAM PLACE SAN DIEGO, CALIFORNIA 92122		SITE PLAN OTAY RANCH SUBSTATION SITE OTAY, CALIFORNIA	FIGURE 2
CHECKED BY: SHR	FN: 67735SITE		
PROJECT NO. 67735	DATE: 10/2007		



KLEINFELDER
5015 SHOREHAM PLACE
SAN DIEGO, CALIFORNIA 92122

CHECKED BY: SHR	FN: 67735SITE
PROJECT NO. 67735	DATE: 10/2007

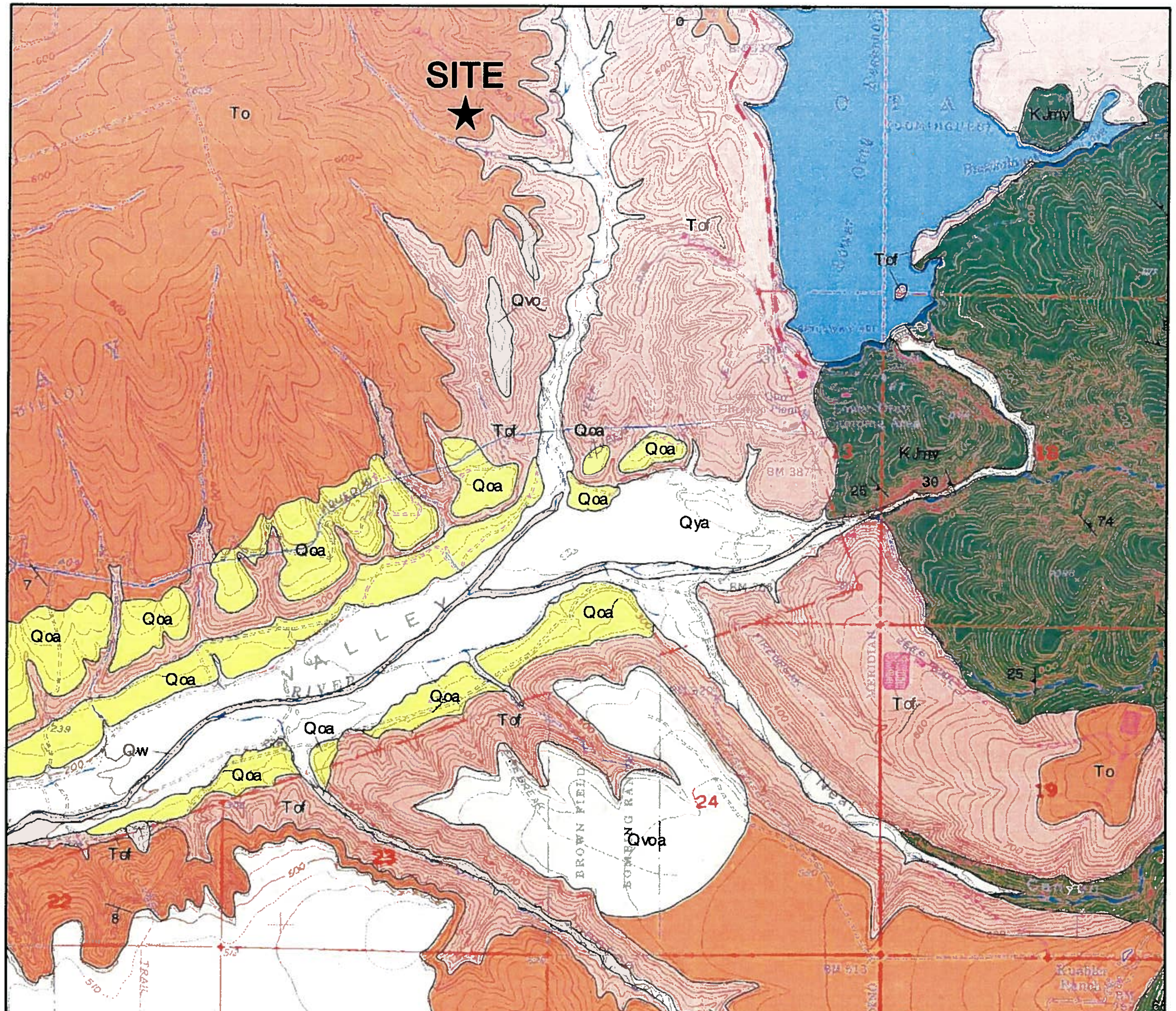
AERIAL PHOTOGRAPH OF SITE

OTAY RANCH SUBSTATION SITE

OTAY, CALIFORNIA

FIGURE

3



DESCRIPTION OF MAP UNITS

Qw	Late Holocene active channel and wash deposits; unconsolidated sand, silt, gravel and clay. Deposits along smaller drainage channels are included in Qya.
Qya	Holocene alluvial deposits; unconsolidated to poorly consolidated silt, clay, sand and gravel. Includes modern active sediments along small drainage channels.
Qoa	Alluvial deposits (late to middle Pleistocene); moderately consolidated, poorly sorted flood plain deposits consisting of gravely sandy silt and clay.
Qvoa	Alluvial deposits (middle to early Pleistocene); well consolidated, poorly sorted flood plain deposits consisting of gravel, sand, silt and clay.
To	Otay Formation (Oligocene to Miocene); poorly indurated massive light colored sandstone, siltstone and claystone, interbedded with bentonite lenses.
Tof	Otay Formation-fanglomerate facies (Oligocene to Miocene); poorly cemented bouldery conglomerate and coarse-grained sandstone. Interfingered with overlying To.
KJmv	Metavolcanic rocks (Jurassic and Cretaceous); mildly metamorphosed volcanic, volcaniclastic and sedimentary rocks. Volcanic rocks range from basalt to rhyolite, but are predominately andesite and dacite. In general, metavolcaniclastic rocks are most abundant.

N

NOT TO SCALE

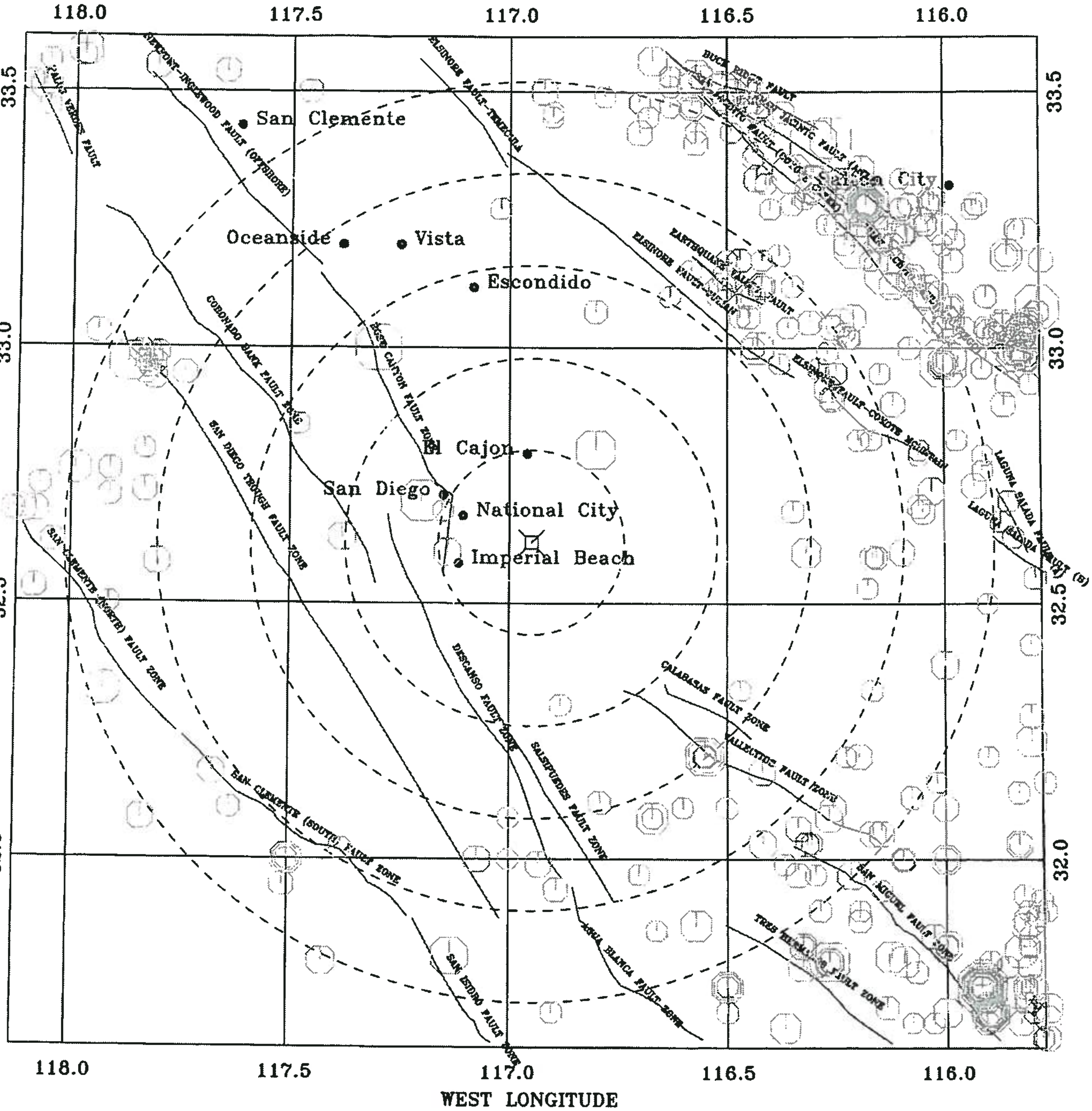
KLEINFELDER
5015 SHOREHAM PLACE
SAN DIEGO, CALIFORNIA 92122

CHECKED BY: SHR	FN: 67735SITE
PROJECT NO. 67735	DATE: 10/2007

LOCAL GEOLOGIC MAP

OTAY RANCH SUBSTATION SITE
OTAY, CALIFORNIA

FIGURE
4



MAGNITUDE	:	4	5	6	7	8
SYMBOL	:					

FAULT MAP AND EPICENTERS OF EARTHQUAKES

OTAY RANCH SUBSTATION, SAN DIEGO, CALIFORNIA

RADIUS OF LARGEST CIRCLE IS 100 KM

KLEINFELDER 5015 SHOREHAM PLACE SAN DIEGO, CALIFORNIA 92122		FAULT MAP	FIGURE
CHECKED BY: SHR	FN: 67735SITE	OTAY RANCH SUBSTATION SITE	5
PROJECT NO. 67735	DATE: 10/2007	OTAY, CALIFORNIA	

KLEINFELDER, Inc.
SLOPE STABILITY ANALYSIS

Project No.: 67735
Project : SDG&E Otay Substation
Driveway Extension
(Static Condition)

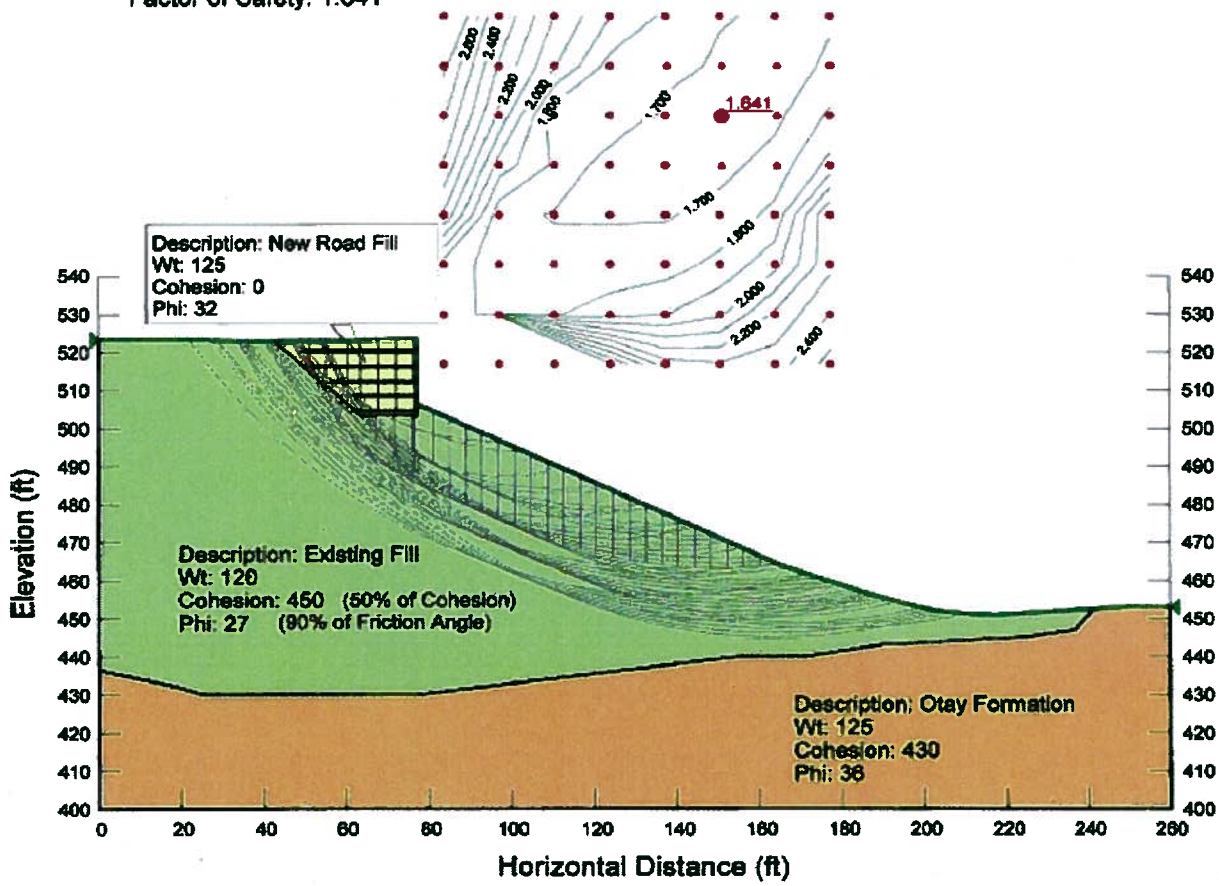
File Name: MSE Reinforce II.gsz
Date: 3/20/2007
Analysis Method: Spencer
Horz Seismic Load: 0g

Factor of Safety: 1.641

REINFORCEMENT INFORMATION

Reinf. Type: Fabric
Reinf. Capacity: 3080 lb/ft
Bar Safety Factor: 1

Contact Cohesion: 0 psf
Contact Phi: 24 degree
Interface Factor: 2



KLEINFELDER 5015 SHOREHAM PLACE SAN DIEGO, CALIFORNIA 92122		STATIC SLOPE STABILITY ANALYSIS OTAY RANCH SUBSTATION SITE OTAY, CALIFORNIA	FIGURE 6
CHECKED BY: KMC	FN: 67735SLOPESTA		
PROJECT NO. 67735	DATE: 03/2007		

KLEINFELDER, Inc.
SLOPE STABILITY ANALYSIS

Project No.: 67735
Project : SDG&E Otay Substation
Driveway Extension
(Seismic Condition)

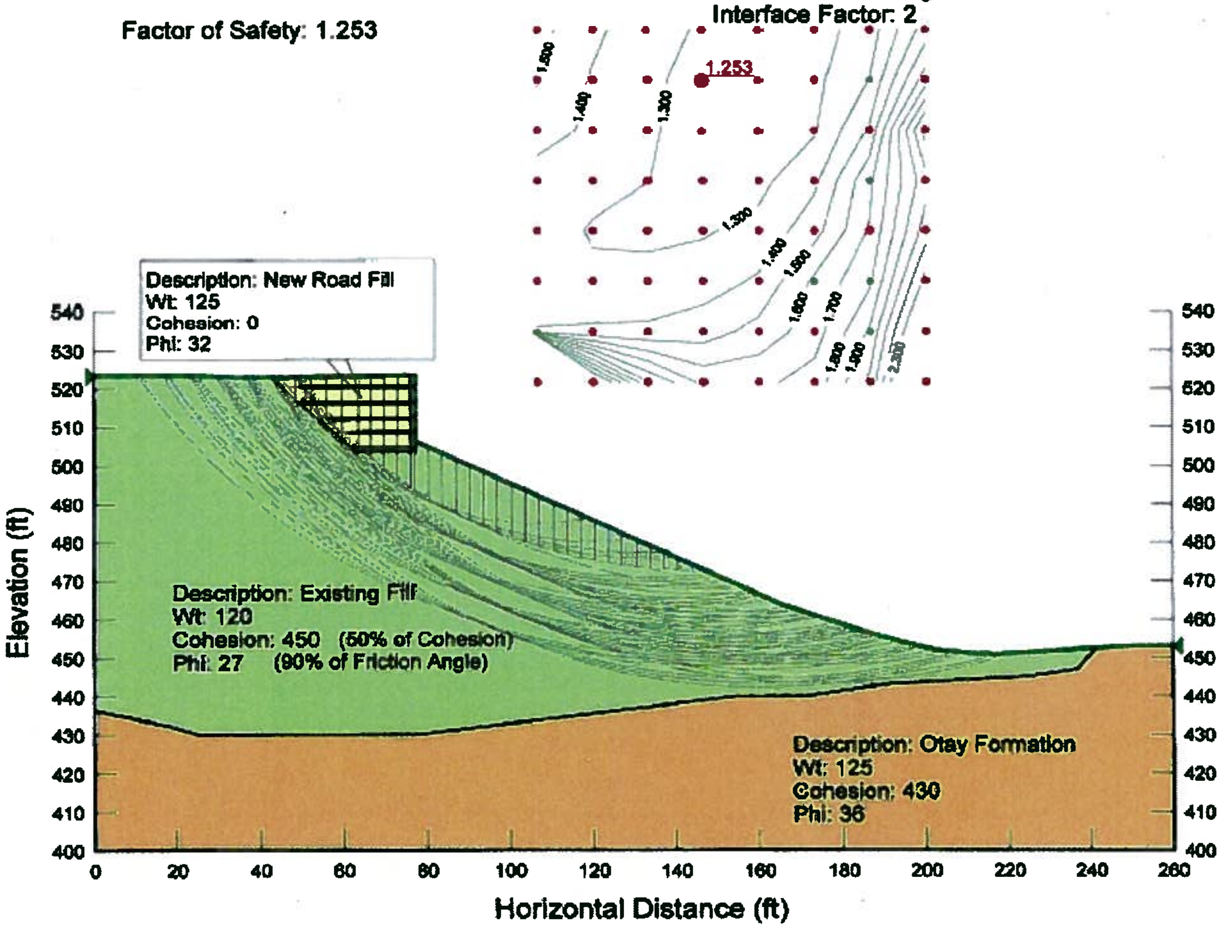
File Name: MSE Reinforce IIs.gsz
Date: 3/20/2007
Analysis Method: Spencer
Horz Seismic Load: 0.15g

Factor of Safety: 1.253

REINFORCEMENT INFORMATION

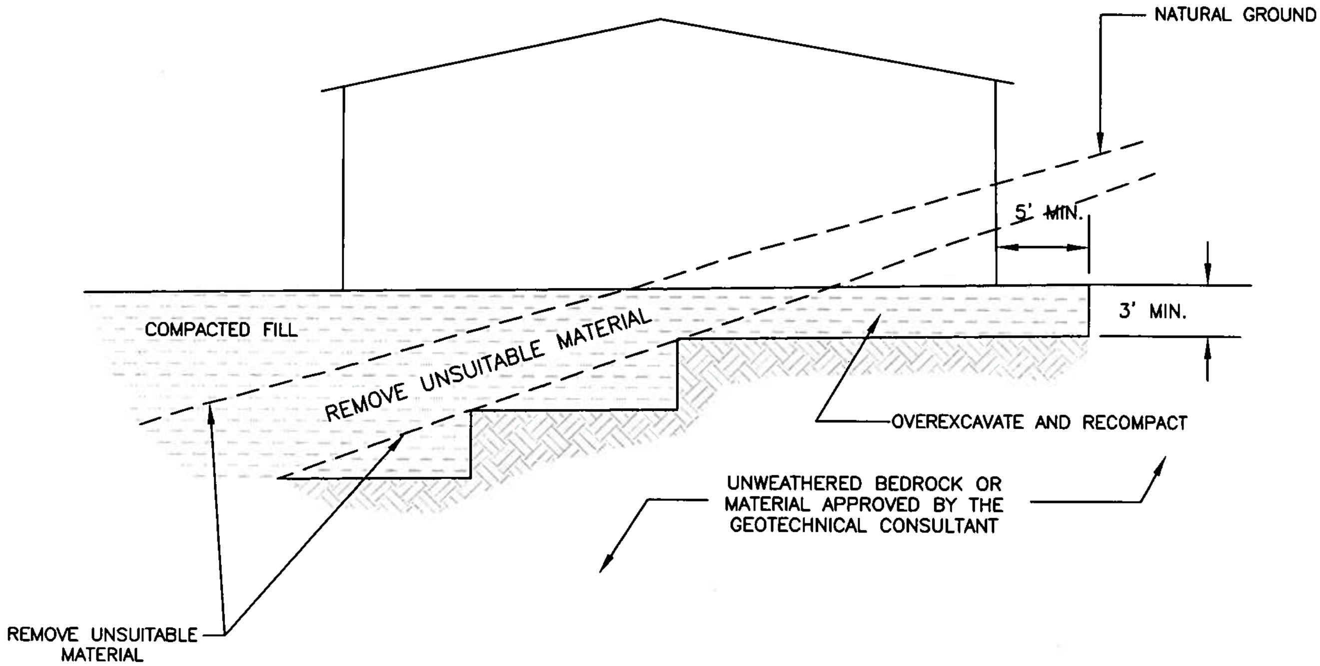
Reinf. Type: Fabric
Reinf. Capacity: 3080 lb/ft
Bar Safety Factor: 1

Contact Cohesion: 0 psf
Contact Phi: 24 degree
Interface Factor: 2



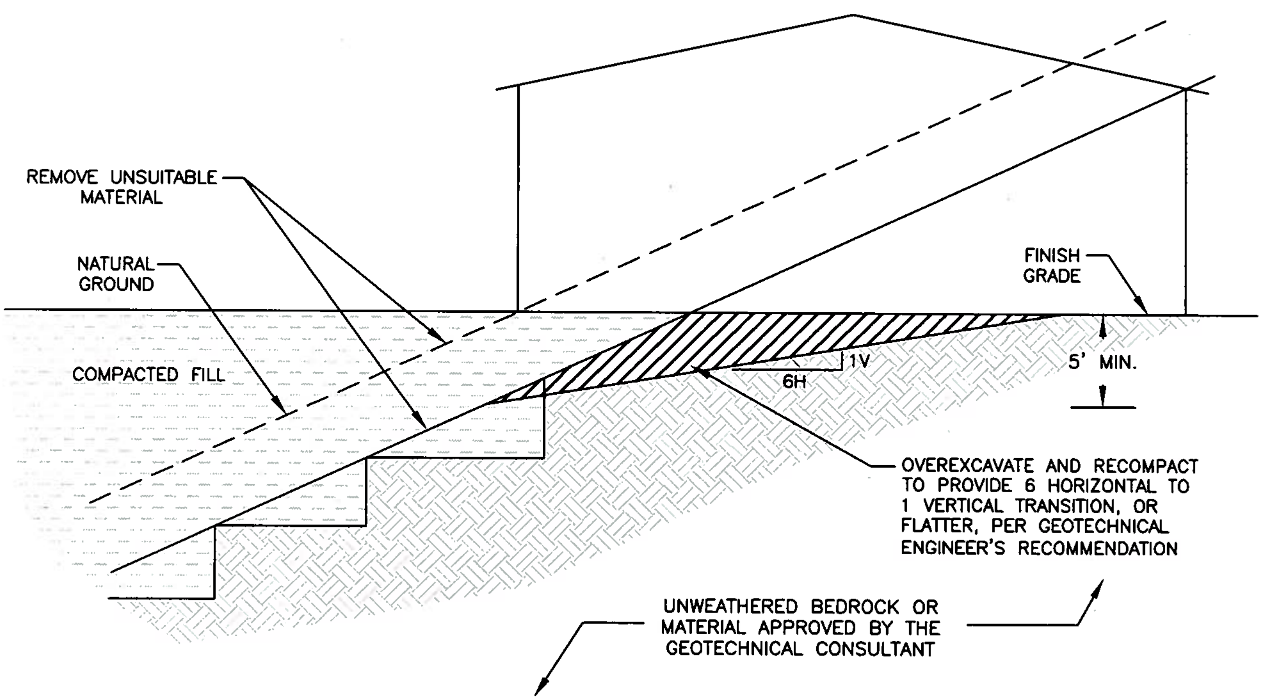
KLEINFELDER 5015 SHOREHAM PLACE SAN DIEGO, CALIFORNIA 92122		SEISMIC SLOPE STABILITY ANALYSIS OTAY RANCH SUBSTATION SITE OTAY, CALIFORNIA	FIGURE 7
CHECKED BY: KMC	FN: 67735SLOPESEIS		
PROJECT NO. 67735	DATE: 03/2007		

CUT-FILL LOT



PRIMARILY CUT LOT

(OPTION)



KLEINFELDER

5015 SHOREHAM PLACE
SAN DIEGO, CALIFORNIA 92122

CHECKED BY: KC	FN: 67735_CUT
PROJECT NO. 67735	DATE: 10/2007

CUT-FILL TRANSITION DETAILS

OTAY RANCH SUBSTATION
CHULA VISTA, CALIFORNIA

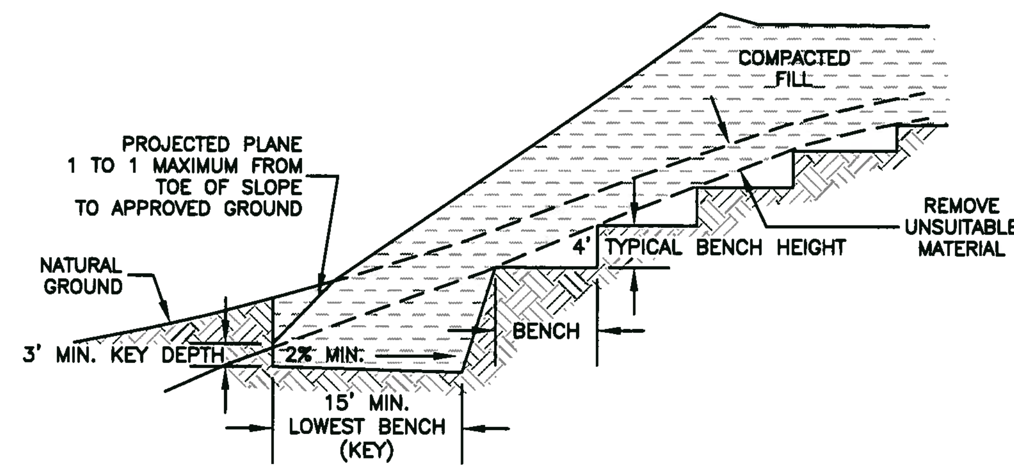
FIGURE

8

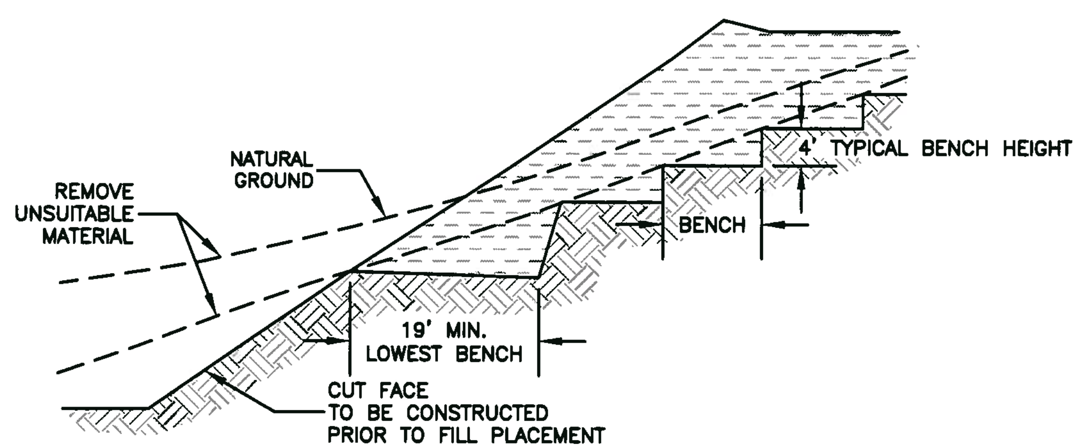
BENCHING DETAILS

NOTES:
LOWEST BENCH: DEPTH AND WIDTH SUBJECT TO FIELD CHANGE BASED ON CONSULTANT'S INSPECTION.

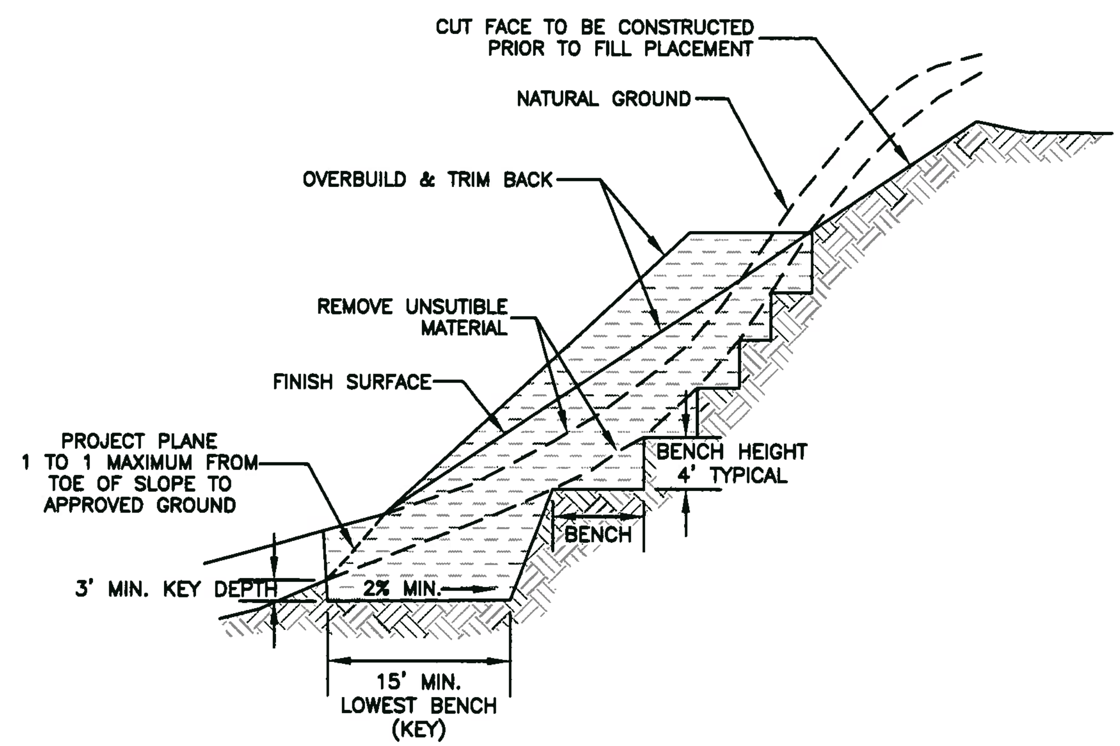
SUBDRAINAGE: BACK DRAINS MAY BE REQUIRED AT THE DISCRETION OF THE GEOTECHNICAL CONSULTANT.



FILL SLOPE

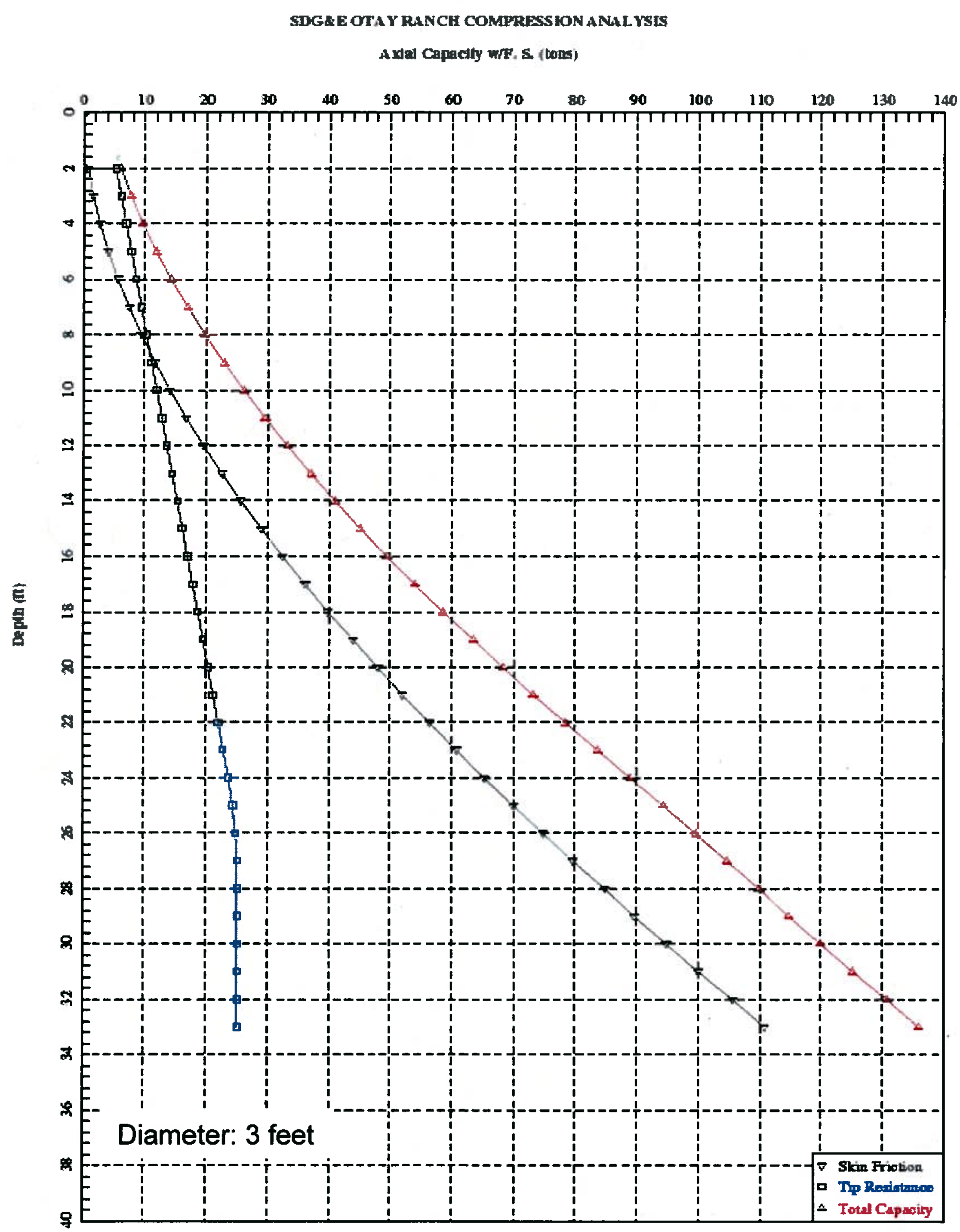


FILL-OVER-CUT SLOPE



CUT-OVER-FILL SLOPE

KLEINFELDER 5015 SHOREHAM PLACE SAN DIEGO, CALIFORNIA 92122	BENCHING DETAILS OTAY RANCH SUBSTATION SITE OTAY, CALIFORNIA	FIGURE 9
CHECKED BY: SHR — FN: 67735DETAILS PROJECT NO. 67735 — DATE: 10/2007		



KLEINFELDER

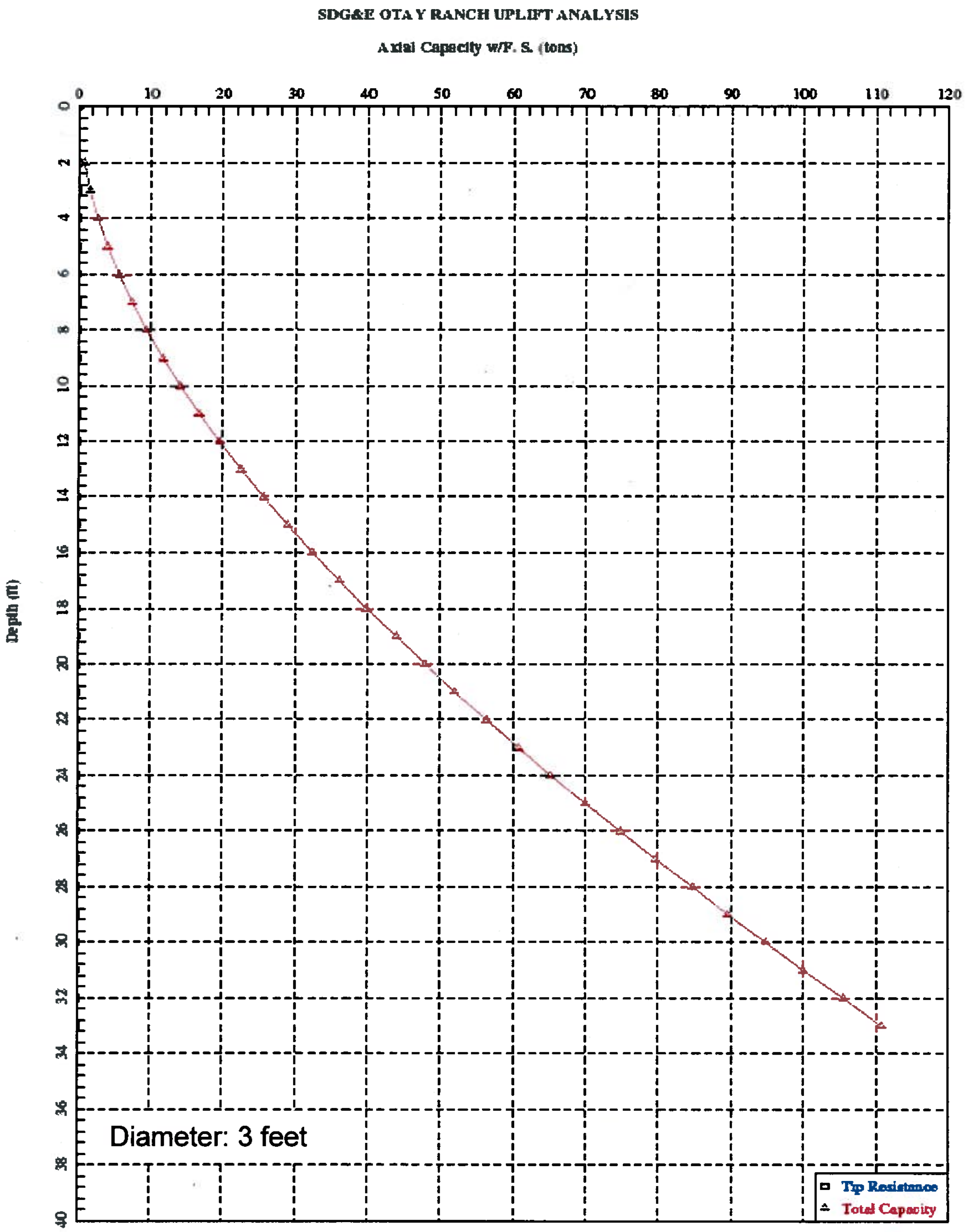
5015 SHOREHAM PLACE
SAN DIEGO, CALIFORNIA 92122

CHECKED BY: KMC	FN: 67735COMPRESS
PROJECT NO. 67735	DATE: 03/2007

ALLOWABLE AXIAL CAPACITY CURVES
3 – FOOT DIAMETER DRILLED PIERS

OTAY RANCH SUBSTATION SITE
OTAY, CALIFORNIA

FIGURE
10



KLEINFELDER

5015 SHOREHAM PLACE
SAN DIEGO, CALIFORNIA 92122

CHECKED BY: KMC	FN: 67735UPLIFT
PROJECT NO. 67735	DATE: 03/2007

ALLOWABLE UPLIFT CAPACITY CURVE
3 – FOOT DIAMETER DRILLED PIERS

OTAY RANCH SUBSTATION SITE
OTAY, CALIFORNIA

FIGURE
11