GEOCON
INCORPORATED

GEOTECHNICAL
ENVIRONMENTAL
MATERIALS

PRELIMINARY GEOTECHNICAL INVESTIGATION

HAWANO EAST OTAY PROPERTY SAN DIEGO COUNTY, CALIFORNIA

PREPARED FOR

HAWANO CORPORATION, N.V.
C/O PARAGON MANAGEMENT COMPANY, LLC
LA JOLLA, CALIFORNIA

JULY 7, 2010
PROJECT NO. G1223-52-01

GEOCON
INCORPORATED

GEOTECHNICAL ■ ENVIRONMENTAL ■ MATERIALS

Project No. G1223-52-01
July 7, 2010

Hawano Corporation, N.V.
c/o Paragon Management Company, LLC
4225 Executive Square, Suite 920
La Jolla, California 92037

Attention: Mr. Dan Berkus

Subject: HAWANO – EAST OTAY PROPERTY
SAN DIEGO COUNTY, CALIFORNIA
PRELIMINARY GEOTECHNICAL INVESTIGATION

Dear Mr. Berkus:

In accordance with the authorization of our Proposal No. LG-10102 dated April 6, 2010, we herein submit the results of our preliminary geotechnical investigation for the subject site. The accompanying report presents our findings, conclusions and preliminary recommendations pertaining to the geotechnical aspects of the proposed development. The study also includes an evaluation of the geologic units, geologic hazards, and soil characteristics. Based on the results of this study, it is our opinion the site is considered suitable for development provided the recommendations of this report are followed.

Should you have any questions regarding this report, or if we may be of further service, please contact the undersigned at your convenience.

Very truly yours,

GEOCON INCORPORATED

John Hoobs
CEG 1524

PROFESSIONAL GEOLOGIST ★ JOHN HOOBS No. 1524 CERTIFIED ENGINEERING GEOLOGIST ★ STATE OF CALIFORNIA

Shawn Foy Weedon
GE 2714

REGISTERED PROFESSIONAL ENGINEER ★ SHAWN WEEDON No. 2714 Exp.06/30/11 GEOTECHNICAL ★ STATE OF CALIFORNIA

JH:SFW:dmc

(2/del) Addressee
(1/del) Kimley-Horn and Associates, Inc.
Attention: Mr. Adam Corral
(1/del) T&B Planning
Attention: Mr. Jeramey Harding

6960 Flanders Drive ■ San Diego, California 92121-2974 ■ Telephone 858.558.6900 ■ Fax 858.558.6159

TABLE OF CONTENTS

TABLE OF CONTENTS (Continued)

PRELIMINARY GEOTECHNICAL INVESTIGATION

1. PURPOSE AND SCOPE

This report presents the results of our preliminary geotechnical investigation for the proposed Hawano East Otay property located in San Diego County, California (see Vicinity Map, Figure 1). The purpose of the investigation is to evaluate subsurface soil and geologic conditions at the site and, based on the conditions encountered, to provide preliminary recommendations pertaining to the geotechnical aspects of developing the property. The proposed development will include the construction of an industrial business park with roadway and infrastructure improvements. Plans for development, as presently proposed, are presented on the Geologic Map, Figure 2 (map pocket).

The scope of our investigation included geologic mapping, subsurface exploration, laboratory testing, engineering analyses, and the preparation of this report. As a part of our investigation, we have reviewed stereoscopic aerial photographs, published geologic maps, published geologic reports, and previous geotechnical reports related to the property. A summary of the background information reviewed for this study is presented in the *List of References*.

The field investigation performed for this report included geologic mapping and the excavation of five large-diameter borings and 25 backhoe trenches. A discussion of the field investigation and logs of the large-diameter borings and backhoe trenches are presented in Appendix A. The approximate locations of the exploratory excavations are presented on the Geologic Map (Figure 2). We performed laboratory tests on soil samples obtained from the exploratory excavations to evaluate pertinent physical and chemical properties for engineering analysis. The results of the laboratory testing are presented in Appendix B.

Kimley-Horn and Associates, Inc. provided the topographic information and proposed grading and development plans used during our field investigation and preparation of the Geologic Map. References to elevations presented in this report are based on the referenced topographic information. Geocon does not practice in the field of land surveying and is not responsible for the accuracy of such topographic information.

2. SITE AND PROJECT DESCRIPTION

The Hawano East Otay property is located just north of the United States/Mexico border and east of the Otay Mesa border crossing in south San Diego County. Access to the site will be provided via Airway Road on the northern portion of the property and by extending Siempre Viva Road and Enrico Fermi Place east along the central portion of the site. Alta Road will also be constructed along the east boundary of the property and Via De La Amistad will be constructed along the south boundary.

The approximate 80-acre property consists of a broad mesa that slopes approximately 2 percent to the south with minor drainages radiating to the south from a central ridge on the northern portion of the site. Site elevations within the area of development range from approximately 494 feet above mean sea level (MSL) at the southeast corner of the property to approximately 556 feet MSL at the northern property boundary. Site vegetation consists of native grasses and vegetation. A water main and pump station exists along the eastern boundary of the property and an SDG&E gas main exists along the southern boundary. A truck parking storage facility is adjacent to the western portion of the property north of Siempre Viva Road.

Grading of the site will consist of maximum cuts and fills of approximately 12 feet and 20 feet, respectively, with cut and fill slopes having a combined maximum height of about 40 feet and a maximum slope inclination of 2:1 (horizontal to vertical). An existing fill slope with a maximum height of 25 feet and an inclination of 2:1 (horizontal to vertical) exists along the western property boundary associated with the truck parking storage facility. A drainage swale exists along the base of the existing fill slope with a storm drain pipe originating from the truck storage facility outletting into the drainage swale located at the eastern terminus of existing Siempre Viva Road.

The planned development includes 23 industrial pads varying from approximately 1 to 6 acres. An approximate 3-acre water quality basin is proposed on the southern portion of the property that will drain south of proposed Via De La Amistad. A one-acre sewer lift station pad is proposed on the southeast corner of the property, east of Alta Road. Each pad will sheet-flow to individual desilting basins. The basins located on pads north of Siempre Viva Road flow into storm drains that outlet south of Via De La Amistad. Basins located on pads south of Siempre Viva Road flow into storm drains that empty into the water quality basin and then outlet south of Via De La Amistad.

The locations and descriptions provided herein are based on a site reconnaissance, and review of the referenced plans and project information provided by Kimley-Horn and Associates, Inc.

3. GEOLOGIC SETTING

The site is located in the coastal plain of the Peninsular Ranges province of southern California. The Peninsular Ranges are a geologic and geomorphic province that extends from the Imperial Valley to the Pacific Ocean and from the Transverse Ranges to the north and into Baja California to the south. The coastal plain of San Diego County is underlain by a thick sequence of relatively undisturbed and nonconformable marine sedimentary rocks that range in age from Upper Cretaceous through the Pleistocene with intermittent deposition. Geomorphically, the coastal plain is characterized by a stair stepped series of marine terraces which young to the west that have been dissected by west flowing rivers that drain the Peninsular Ranges to the east. The coastal plain is a relatively stable geologic block that is dissected by relatively few faults consisting of the potentially active La Nacion Fault zone and the active Rose Canyon Fault Zone located to the west of the site. The site is located on the eastern edge of the coastal plain and is just west of the Metavolcanic Rocks of the San Ysidro Mountains. A marine sedimentary unit consisting of the Tertiary age Otay Formation makes up the northern and central portions of the site. The Otay Formation typically consists of three lithostratagraphic members composed of a basal conglomerate member, a middle gritstone member and an upper sandstone to claystone member with a maximum reported regional thickness of roughly 400 feet. The site generally contains the upper two members of this unit. The southern portion of the site consists of a marine terrace deposit named Very Old Paralic Deposit Undivided. This unit is not shown in this area of the site on the state geologic maps prepared by Kennedy (2005) and Todd (2004).

4. GEOLOGIC MATERIALS

4.1 General

During our field investigation, we encountered two surficial deposits consisting of previously placed fill and topsoil. Two geologic formations exist at the site consisting of the Tertiary-age Otay Formation and early to middle Pleistocene-age Very Old Paralic Deposits Undivided. The lateral extent of the materials encountered is shown on the Geologic Map, Figure 2 (map pocket). Figure 3 present a Geologic Cross-Section providing an interpretation of the subsurface geologic conditions. The descriptions of the soil and geologic conditions are shown on the boring and trench logs located in Appendix A and described herein in order of increasing age.

4.2 Previously Placed Fill (Qpf)

Previously placed fill associated with the construction of the Airway Truck Parking Storage facility consisting of an approximately 25-foot-high fill slope exists on the western boundary north of Siempre Viva Road. The mass grading of the adjacent site was completed in 2004 with testing and observation of the grading operations provided by Geocon Incorporated and reported in our final report of grading dated September 15, 2004. In general, the fill slope materials consist of clays and sands. In its present condition, the fill soil is suitable for support of additional fill or structures; however, the outer surface of the fill slope will require remedial grading. The existing fill is generally suitable for reuse as compacted fill, provided it is substantially free of organics and debris.

4.3 Topsoil (unmapped)

Holocene-age topsoil is present as a relatively thin veneer locally overlying formational materials across the site. The topsoil has a maximum encountered thickness of 4 feet and can be characterized as soft to stiff, moist, dark brown, silty clay. The topsoil is typically very highly expansive and compressible when overlying the Otay Formation. Removal of the topsoil and placement in the deeper fill areas will be necessary in areas to support proposed fill or structures. Due to the relatively thin thickness of these deposits, topsoil is not shown on the Geologic Map or Cross-Section.

4.4 Very Old Paralic Deposits Undivided (Qvop)

Early to middle Pleistocene-age Very Old Paralic Deposits Undivided is located on the southern portion of the site with a thickness in excess of 20 feet. This unit typically consists of dense to very dense, damp, grayish to reddish brown, silty, fine to medium sandstone with interbeds of cohesionless fine to coarse sand and localized layers of silt and clay. These deposits are generally suitable for the support of proposed fill and structural loads. However, the cohesionless layers of this unit may be susceptible to excess erosion and may require slope stabilization measures if exposed in cut slopes. The cohesionless sands should not be used to construct fill slopes; however, the sand could be mined and used as slab underlayment or bedding sand in utility trenches. This unit is suitable for use as compacted fill on the site.

4.5 Otay Formation (To)

Tertiary-age Otay Formation is located in the northern and central portions of the site. This unit consists of dense to very dense and hard, slightly and moderately cemented, clayey sandstone, sandy siltstone, and sandy claystone that are locally thinly laminated. Excavations within the unit will generally be possible with heavy-duty grading equipment with moderate to heavy effort; however, moderately cemented zones are expected and will create very difficult ripping and generate oversize cemented material. The Otay Formation is suitable for the support of proposed fill and structural loads. Cut slopes composed of claystone and siltstone layers will require slope stabilization during grading operations. Slope drains may be necessary subsequent to development to intercept potential seepage created by landscape irrigation.

5. GEOLOGIC STRUCTURE

The geologic structure within the sedimentary units at the site is characterized by a regional gentle southwesterly dip. Local dip directions vary from the southwest to southeast with a maximum dip of 5 degrees. The contact between the Very Old Paralic Deposits Undivided and the underlying Otay Formation is unconformable and generally slopes down to the west and south. This unit is generally massive and devoid of structure within the cohesionless sand layers and poorly bedded within the silty sandstone layers. The Otay Formation contain layers of thinly laminated siltstone beds that do not appear to have undergone structural folding that potentially could create bedding plane shearing.

6. GROUNDWATER

We did not encounter a static groundwater table or seepage in the exploratory excavations performed for this study. It is not uncommon for seepage conditions to develop where none previously existed due to the permeability characteristics of the geologic units encountered. During the rainy season, perched water conditions are likely to develop within the drainage swales that may require special consideration during grading operations. Groundwater elevations are dependent on seasonal precipitation, irrigation, and land use, among other factors, and vary as a result.

7. GEOLOGIC HAZARDS

7.1 Seismic Hazard Analysis

It is our opinion, based on a review of published geologic maps and reports, that the site is not located on known active, potentially active, or inactive fault traces. An active fault is defined by the California Geological Survey (CGS) as a fault showing evidence for activity within the last 11,000 years. The site is not located within a State of California Earthquake Special Study Zone.

According to the computer program *EZ-FRISK (Version 7.40),* five known active faults are located within a search radius of 50 miles from the property. The Rose Canyon Fault, located approximately 13 miles northwest of the site, is the nearest known active fault and is the dominant source of potential ground motion. Earthquakes that might occur on the Rose Canyon Fault Zone or other faults within the southern California and northern Baja California area are potential generators of significant ground motion at the site. The estimated maximum earthquake magnitude and peak ground acceleration for the Rose Canyon Fault are 7.2 and 0.22g, respectively. Table 7.1.1 lists the estimated maximum earthquake magnitude and peak ground acceleration for the most dominant faults in relation to the site location. We calculated peak ground acceleration (PGA) using Boore-Atkinson (2008) NGA USGS2008, Campbell-Bozorgnia (2008) NGA USGS 2008, and Chiou-Youngs (2008) NGA acceleration-attenuation relationships.

TABLE 7.1.1
DETERMINISTIC SPECTRA SITE PARAMETERS

Fault Name	Distance from Site (miles)	Maximum Earthquake Magnitude (Mw)	Peak Ground Acceleration		
			Boore-Atkinson 2008 (g)	Campbell-Bozorgnia 2008 (g)	Chiou-Youngs 2008 (g)
Rose Canyon	13	7.2	0.22	0.17	0.20
Coronado Bank Fault Zone	20	7.6	0.20	0.14	0.18
Elsinore-Julian	42	7.1	0.10	0.07	0.07
Elsinore-Coyote Mountain	43	6.8	0.09	0.06	0.05
Earthquake Valley	45	6.5	0.07	0.05	0.04

In the event of a major earthquake on the referenced faults or other significant faults in the southern California and northern Baja California area, the site could be subjected to moderate to severe ground shaking. With respect to this hazard, the site is considered comparable to others in the general vicinity.

We performed a site-specific probabilistic seismic hazard analysis using the computer program *EZ-FRISK*. Geologic parameters not addressed in the deterministic analysis are included in this analysis. The program operates under the assumption that the occurrence rate of earthquakes on each mapped Quaternary fault is proportional to the faults slip rate. The program accounts for fault rupture length as a function of earthquake magnitude, and site acceleration estimates are made using the earthquake magnitude and distance from the site to the rupture zone. The program also accounts for uncertainty in each of following: (1) earthquake magnitude, (2) rupture length for a given magnitude, (3) location of the rupture zone, (4) maximum possible magnitude of a given earthquake, and (5) acceleration at the site from a given earthquake along each fault. By calculating the expected accelerations from considered earthquake sources, the program calculates the total average annual expected number of occurrences of site acceleration greater than a specified value. We utilized acceleration-attenuation relationships suggested by Boore-Atkinson (2008), Campbell-Bozorgnia (2008) and Chiou-Youngs (2008) in the analysis. Table 7.1.2 presents the site-specific probabilistic seismic hazard parameters including acceleration-attenuation relationships and the probability of exceedence.

TABLE 7.1.2
PROBABILISTIC SEISMIC HAZARD PARAMETERS

Probability of Exceedence	Peak Ground Acceleration		
	Boore-Atkinson, 2008 (g)	Campbell-Bozorgnia, 2008 (g)	Chiou-Youngs, 2008 (g)
2% in a 50 Year Period	0.46	0.38	0.45
5% in a 50 Year Period	0.35	0.29	0.33
10% in a 50 Year Period	0.27	0.23	0.25

The California Geologic Survey (CGS) has a program that calculates the ground motion for a 10 percent of probability of exceedence in a 50-year period based on an average of several attenuation relationships. Table 7.1.3 presents the calculated results from the Probabilistic Seismic Hazards Mapping Ground Motion Page from the CGS website.

TABLE 7.1.3
PROBABILISTIC SITE PARAMETERS FOR SELECTED FAULTS
CALIFORNIA GEOLOGIC SURVEY

Calculated Acceleration (g) Firm Rock	Calculated Acceleration (g) Soft Rock	Calculated Acceleration (g) Alluvium
0.21	0.23	0.27

While listing peak accelerations is useful for comparison of potential effects of fault activity in a region, other considerations are important in seismic design, including the frequency and duration of motion and the soil conditions underlying the site. Seismic design of the structures should be performed in accordance with the 2007 California Building Code (CBC) guidelines currently adopted by the County of San Diego.

7.2 Liquefaction

Liquefaction typically occurs when a site is located in a zone with seismic activity, onsite soils are cohesionless, static groundwater is encountered within 50 feet of the surface, and soil relative densities are less than about 70 percent. If the four previous criteria are met, a seismic event could result in a rapid pore-water pressure increase from the earthquake-generated ground accelerations. Seismically induced settlement may occur whether the potential for liquefaction exists or not. The potential for liquefaction and seismically induced settlement occurring within the site soil is considered to be very low due to the dense nature of proposed fill and the very dense nature of the formational materials.

7.3 Expansive Soil

The majority of the geologic units will likely possess a “very low” to “medium” expansion potential (Expansion Index of 90 or less). However, some of the geologic units may contain a “high” to “very high” expansive potential (Expansion Index of 91 to greater than 130). These units can include topsoil and the claystone beds within the Otay Formation. We expect the proposed grading will expose claystone within cut slopes and near finish grade within lots and public rights-of-ways. Consequently, undercutting of lots, streets, curb and gutter, and sidewalk subgrade will be required where highly expansive clay is exposed or located near finish grade.

7.4 Landslides

Examination of stereoscopic aerial photographs in our files, our geologic reconnaissance, and review of available geotechnical and geologic reports for the site vicinity indicate that landslides are not present at the property or at a location that could impact the site. We do not consider landsliding to be a geologic hazard to the project.

7.5 Slope Stability

We evaluated the proposed slope configurations, as depicted on the Geologic Map, to evaluate both surficial and global stability based on the current geologic information. The portions of the site planned for development are generally underlain by Quaternary-age topsoil, Pleistocene-aged Very Old Paralic Deposits Undivided, and Tertiary-age Otay Formation. The unit most likely to be subject to slope instability is the claystone and siltstone portions of the Otay Formation encountered at several locations throughout the site. Therefore, cut slopes will require the construction of stability fills as detailed on Figure 4 to achieve acceptable factors of safety.

The proposed slopes should be stable from shallow sloughing conditions provided the recommendations for grading and drainage are incorporated into the design and construction of the proposed slopes. In general, it is our opinion that permanent, graded fill slopes or cut slopes that are constructed with stability fills with gradients of 2:1 (horizontal to vertical) or flatter would possess Factors of Safety of 1.5 or greater. Figures 5, 6, and 7 present the slope stability analysis for cut and fill slopes and surficial slope stability analysis, respectively.

7.6 Tsunamis and Seiches

A tsunami is a series of long period waves generated in the ocean by a sudden displacement of large volumes of water. Causes of tsunamis include underwater earthquakes, volcanic eruptions, or offshore slope failures. The first order driving force for locally generated tsunamis offshore southern California is expected to be tectonic deformation from large earthquakes (Legg, *et al.*, 2002). The County of San Diego Hazard Mitigation Plan (2004) maps zones of high risk for tsunami run-up for coastal areas throughout the county. The site is not included within one of these high-risk hazard areas. The site is approximately 12 miles from the Pacific Coast and ranges between approximately 494 feet and 556 feet above MSL. Therefore, we consider the risk associated with tsunamis to be negligible.

A seiche is a run-up of water within a lake or embayment triggered by fault- or landslide-induced ground displacement. The site is not located near or downstream from a lake or embayment, therefore, it is our opinion that the potential of seiches affecting the site is considered negligible.

7.7 Hydroconsolidation

Hydroconsolidation is the tendency of unsaturated soil structure to collapse upon saturation resulting in the overall settlement of the affected soil and overlying foundations or improvements supported thereon. Potentially compressible surficial soil underlying the proposed structures and fill is typically removed and recompacted during remedial site grading. However, if compressible soil is left in-place, a potential for settlement due to hydroconsolidation of the soil exists. The potential for hydroconsolidation can be mitigated, if necessary, by performing remedial grading and the use of stiffer foundation systems. Based on the laboratory test results, the potential for hydroconsolidation ranges from 0 percent to 1.3 percent with an average of about 0.8 percent within the sandy portion of the Old Paralic Deposits Undivided. We expect the amount of settlement due to hydroconsolidation is approximately 1 inch for the sandy material left in place on the southern portion of the property.

8. CONCLUSIONS AND RECOMMENDATIONS

8.1 General

8.1.1 It is our opinion that no soil or geologic conditions were encountered during the investigation that would preclude the proposed development of the Hawano East Otay project provided the recommendations presented herein are followed and implemented during construction.

8.1.2 Potential geologic hazards at the site include seismic shaking, and expansive and compressible soil. Based on our investigation and available geologic information, active or potentially active faults are not present underlying or trending toward the site.

8.1.3 The existing topsoil materials are highly expansive and potentially compressible and therefore unsuitable in their present condition for the support of compacted fill or settlement-sensitive improvements. Remedial grading of the topsoil will be required and recommendations for remedial grading are provided herein. The formational units are suitable for the support of proposed fill and structural loads.

8.1.4 We did not encounter groundwater or seepage during our subsurface exploration. However, seepage may be a constraint during construction of cut slopes and should be mitigated with the use of stability fills. Seepage within formational materials and perched groundwater conditions within the drainage swales may be encountered during the grading operations, especially during the rainy seasons.

8.1.5 The rippability of the topsoil is expected to range from easy to moderate. We expect the formational units to be rippable with moderate to heavy effort to proposed finish grades. Cemented zones should be expected within portions of the Otay Formation and will generate oversized material and will require special handling techniques.

8.1.6 In general, fill slopes and cut slopes composed of the sandstone portions of the Otay Formation and Very Old Paralic Deposits Undivided should possess Factors of Safety of at least 1.5 at inclinations of 2:1 (horizontal to vertical), or flatter. Surficial slope stability analysis indicates a factor of safety of at least 1.5.

8.1.7 Proposed cut slopes that expose the Otay Formation will require slope stabilization. Recommendations for slope stabilization are provide within the grading section of this report.

8.1.8 The proposed industrial buildings and retaining walls may be supported on conventional foundations bearing in either competent formational materials or properly compacted fill. Geocon Incorporated should evaluate the building foundation systems when the locations of these structures have been finalized. Transitioning foundations and slabs from formational material to compacted fill should be evaluated. Formational over-excavations may be required where engineered fill is to be utilized for foundation support. This will require future evaluation once the building locations have been finalized. General recommendations for the design of shallow foundations are provided herein.

8.1.9 Proper drainage should be maintained in order to preserve the engineering properties of the fill in both the building pads and slope areas. Recommendations for site drainage are provided herein.

8.1.10 Geocon Incorporated should be contacted to provide additional recommendations when the project team has developed updated plans depicting the locations of the proposed improvements.

8.2 Soil Characteristics

8.2.1 The soil encountered in the field investigation is considered to be “expansive” (Expansion Index [EI] greater than 20) as defined by 2007 California Building Code (CBC) Section 1802.3.2. Table 8.2.1 presents soil classifications based on the expansion index.

TABLE 8.2.1
SOIL CLASSIFICATION BASED ON EXPANSION INDEX

Expansion Index (EI)	Soil Classification
0 – 20	Very Low
21 – 50	Low
51 – 90	Medium
91 – 130	High
Greater Than 130	Very High

8.2.2 Based on laboratory tests of representative samples of the materials expected at proposed grades presented in Appendix B (Table B-III), the on-site material is expected to possess a “very low” to “very high” expansion potential (Expansion Index greater than 130). We expect the topsoil and claystone layers within the Otay Formation will likely possess a “high” to “very high” expansion potential (Expansion Index of 91 to greater than 130). The siltstone layers within the Otay Formation are expected to have a “medium” to “high” expansion potential (Expansion Index of 51 to less than 130). The sandstone portions of the Otay Formation and the Very Old Paralic Deposits Undivided will likely possess a "very low" to "low" expansion potential (Expansion Index of 50 or less). Additional testing for expansion potential should be performed once final grades are achieved.

8.2.3 We performed laboratory tests on samples of the site materials to evaluate the percentage of water-soluble sulfate content. Results from the laboratory water-soluble sulfate content tests are presented in Appendix B and indicate that the on-site materials at the locations tested possess "negligible" to "moderate" sulfate exposure to concrete structures as defined by 2007 CBC Section 1904.3 and ACI 318. Table 8.2.2 presents a summary of concrete requirements set forth by 2007 CBC Section 1904.3 and ACI 318. The presence of water-soluble sulfates is not a visually discernible characteristic; therefore, other soil samples from the site could yield different concentrations. Additionally, over time landscaping activities (i.e., addition of fertilizers and other soil nutrients) may affect the concentration. Additional corrosion testing of the finish grade soils should be performed during grading.

TABLE 8.2.2
REQUIREMENTS FOR CONCRETE EXPOSED TO SULFATE-CONTAINING SOLUTIONS

Sulfate Exposure	Exposure Class	Water-Soluble Sulfate Percent by Weight	Cement Type	Maximum Water to Cement Ratio by Weight	Minimum Compressive Strength (psi)
Negligible	S0	0.00-0.10	--	--	2,500
Moderate	S1	0.10-0.20	II	0.50	4,000
Severe	S2	0.20-2.00	V	0.45	4,500
Very Severe	S3	> 2.00	V+Pozzolan or Slag	0.45	4,500

8.2.4 We performed laboratory tests on a sample of the site materials encountered to check the corrosion potential to subsurface metal structures. We performed the laboratory tests in accordance with California Test Method No. 643. The laboratory test results are presented in Appendix B.

8.2.5 Geocon Incorporated does not practice in the field of corrosion engineering. Therefore, further evaluation by a corrosion engineer may be performed if improvements that could be susceptible to corrosion are planned.

8.3 Seismic Design Criteria

8.3.1 We used the computer program *Seismic Hazard Curves and Uniform Hazard Response Spectra,* provided by the USGS to calculate the seismic design criteria. Table 8.3 summarizes site-specific design criteria obtained from the 2007 CBC, Chapter 16 *Structural Design,* Section 1613 *Earthquake Loads.* Soil values C and D will be present on the site depending on the thickness of fill soil beneath a particular proposed building. The short spectral response has a period of 0.2 second.

TABLE 8.3
2007 CBC SEISMIC DESIGN PARAMETERS

Parameter	Value		IBC-06 Reference
Site Class	C	D	Table 1613.5.2
Fill Thickness, T	T<20 feet	T≥20 feet	--
Spectral Response – Class B (short), S_S	0.921g	0.921g	Figure 1613.5(3)
Spectral Response – Class B (1 sec), S_1	0.335g	0.335g	Figure 1613.5(4)
Site Coefficient, F_a	1.031	1.131	Table 1613.5.3(1)
Site Coefficient, F_v	1.465	1.730	Table 1613.5.3(2)
Maximum Considered Earthquake Spectral Response Acceleration (short), S_{MS}	0.950g	1.042g	Section 1613.5.3 (Eqn 16-37)
Maximum Considered Earthquake Spectral Response Acceleration (1 sec), S_{M1}	0.490g	0.579g	Section 1613.5.3 (Eqn 16-38)
5% Damped Design Spectral Response Acceleration (short), S_{DS}	0.633g	0.695g	Section 1613.5.4 (Eqn 16-39)
5% Damped Design Spectral Response Acceleration (1 sec), S_{D1}	0.327g	0.386g	Section 1613.5.4 (Eqn 16-40)

8.3.2 Conformance to the criteria in Table 8.3 for seismic design does not constitute any kind of guarantee or assurance that significant structural damage or ground failure will not occur if a large earthquake occurs. The primary goal of seismic design is to protect life, not to avoid all damage, since such design may be economically prohibitive.

8.4 Grading

8.4.1 Grading should be performed in accordance with the *Recommended Grading Specifications* contained in Appendix C and the County of San Diego Grading Ordinance.

8.4.2 Prior to commencing grading, a preconstruction conference should be held at the site with the county inspector, owner or developer, grading contractor, civil engineer, environmental consultant, and geotechnical engineer in attendance. Special soil handling and/or the grading plans can be discussed at that time.

8.4.3 Site preparation should begin with the removal of deleterious material, debris and vegetation. The depth of removal should be such that material exposed in cut areas or soil to be used as fill is relatively free of organic matter. Material generated during stripping and/or site demolition should be exported from the site.

8.4.4 Abandoned buried utilities (if encountered) should be removed and the resultant depressions and/or trenches should be filled with properly compacted material as part of the remedial grading.

8.4.5 Topsoil within the limits of grading should be removed to expose firm formational materials. The actual depth of removal should be evaluated by the geotechnical engineering consultant during the grading operations. The topsoil and soil with an Expansion Index greater than 90 should be placed in deeper fill areas at least 6 feet from finish sheet-grade elevations. The bottom of the excavations should be scarified to a depth of at least 1 foot, moisture conditioned as necessary, and properly compacted. The outer portion of the existing fill slopes will require benching to remove the loose upper portion during grading operations.

8.4.6 We expect the sandy portion of the Old Paralic Deposits would be encountered within the bottom of the planned basin at the southern portion of the property. Water that enters the basin may infiltrate into the cohesionless sand layers and could cause distress down gradient. The upper three feet of the basin and the outer five feet of the sidewalls of the basin should be removed and replaced with properly compacted finer grained soils. The existing finer grained soils within the Otay Formation should be used for the fill within the basin to prevent water from infiltrating into the cohesionless sand layers.

8.4.7 The geotechnical engineering consultant should observe the removal bottoms to check the exposure of the formational materials. Deeper excavations may be required if highly weathered formational material is present at the base of the removals.

8.4.8 The site should be brought to final finish grade elevations with fill compacted in layers. Layers of fill should be no thicker than will allow for adequate bonding and compaction. Fill, including backfill and scarified ground surfaces, should be compacted to a dry density of at least 90 percent of the laboratory maximum dry density near to slightly above optimum moisture content in accordance with ASTM D 1557. Fill materials placed below optimum moisture content may require additional moisture conditioning prior to placing additional fill.

8.4.9 Import fill (if necessary) should consist of granular materials with a “very low” to “medium” expansion potential (EI of 90 or less) generally free of deleterious material and rock fragments larger than 3 inches if used for capping and should be compacted as recommended herein. Geocon Incorporated should be notified of the import soil source and should perform laboratory testing of import soil prior to its arrival at the site to evaluate its suitability as fill material.

8.4.10 Cut slopes located within weak and/or sheared claystone and/or siltstone beds of the Otay Formation will require stability fills. In addition, cut slopes exposing cohesionless sands within the Very Old Paralic Deposits Undivided may also require stability fills. In general, the Typical Stability Fill Detail presented on Figure 4 should be used for design and construction of stability fills, where required. The backcut for the stability fills should commence at least 10 feet from the top of the proposed finish-graded slope and should extend at least 3 feet into formational material. The drains and outlets should be surveyed for proper line and gradient to check flow and to evaluate future outlet or drain tie-in locations by the project civil engineer.

8.4.11 Cut slope excavations including fill slope shear keys and stability fills should be observed during grading operations to check that soil and geologic conditions do not differ significantly from those expected.

8.4.12 The outer 15 feet (or a distance equal to the height of the slope, whichever is less) of fill slopes should be composed of properly compacted granular “soil” fill to reduce the potential for surficial sloughing. In general, soil with an Expansion Index of 90 or less and at least 35 percent sand-size particles should be acceptable as granular “soil” fill. Soil of questionable strength to satisfy surficial stability should be tested in the laboratory for acceptable drained shear strength. The use of cohesionless sand in the outer portion of fill slopes should be avoided. Fill slopes should be overbuilt at least 2 feet and cut back or be compacted by backrolling with a loaded sheepsfoot roller at vertical intervals not to exceed 4 feet to maintain the moisture content of the fill. The slopes should be track-walked at the completion of each slope such that the fill is compacted to a dry density of at least 90 percent of the laboratory maximum dry density near to slightly above optimum moisture content to the face of the finished slope.

8.4.13 Finished slopes should be landscaped with drought-tolerant vegetation having variable root depths and requiring minimal landscape irrigation. In addition, the slopes should be drained and properly maintained to reduce erosion.

8.4.14 Geocon Incorporated should provide additional grading recommendations when the project team has determined the locations of the planned improvements and prepared updated plans.

8.5 Earthwork Grading Factors

8.5.1 Estimates of bulking and shrinkage factors are based on empirical judgments comparing the material in its natural state as encountered in the exploratory excavations to a compacted state. Variations in natural soil density and in compacted fill density render shrinkage value estimates very approximate. As an example, the contractor can compact the fill to a dry density of 90 percent or higher of the laboratory maximum dry density. Thus, the contractor has an approximately 10 percent range of control over the fill volume. Bulking of rock units is a function of rock density, structure, overburden pressure, and the physical behavior of blasted material. Based on our experience, the shrinkage and bulking factors presented in Table 8.5 can be used as a basis for estimating how much the on-site soil may shrink or swell (bulk) when excavated from their natural state and placed as compacted fill. Please note that these estimates are for preliminary quantity estimates only. Due to the variations in the actual shrinkage/bulking factors, a balance area that can also accommodate rock should be provided to accommodate these variations.

TABLE 8.5
SHRINKAGE AND BULK FACTORS

Soil Unit	Shrink/Bulk Factor
Topsoil (unmapped)	10-15 % shrink
Otay Formation (To)	2-4 % bulk
Very old Paralic Deposits Undivided	2 % shrink to 2 % bulk

8.6 Conventional Shallow Foundations

8.6.1 The proposed industrial buildings can be supported on a conventional shallow foundation system bearing on compacted fill. The recommendations provided herein are applicable for soils with an expansion index of 90 or less within the upper 4 feet of finish grade. Foundation for the structure should consist of continuous strip footings and/or isolated spread footings. Continuous footings should be at least 12 inches wide and extend at least 24 inches below lowest adjacent pad grade. Isolated spread footings should have a minimum width and depth of 24 inches. For building pads with finish grade soil with an expansion index between 90 and 130, the depth of the foundations should be extended to at least 36 inches below lowest adjacent pad grade.

8.6.2 Steel reinforcement for continuous footings should consist of at least four No. 5 steel reinforcing bars placed horizontally in the footings; two near the top and two near the bottom. Steel reinforcement for the spread footings should be designed by the project structural engineer. A typical wall/column footing dimension detail is presented on Figure 8.

8.6.3 The recommended allowable bearing capacity for foundations with minimum dimensions described herein is 2,500 pounds per square foot (psf) for footings bearing in compacted fill soil. The allowable soil bearing pressure may be increased by an additional 500 psf for each additional foot of depth and 300 psf for each additional foot of width, to a maximum allowable bearing capacity of 4,000 psf for footings founded in compacted fill soil. The values presented above are for dead plus live loads and may be increased by one-third when considering transient loads due to wind or seismic forces.

8.6.4 We expect the total and differential settlements under the imposed allowable loads would be ½ inch.

8.6.5 Foundation excavations should be observed by the geotechnical engineer (a representative of Geocon Incorporated) prior to the placement of reinforcing steel to check that the exposed soil conditions are similar to those expected and that they have been extended to the appropriate bearing strata. If unexpected soil conditions are encountered, foundation modifications may be required.

8.7 Concrete Slabs-on-Grade

8.7.1 Interior concrete slabs-on-grade for the buildings should be at least 5 inches thick. As a minimum, reinforcement for slabs-on-grade should consist of No. 4 steel reinforcing bars placed at 18 inches on center in both horizontal directions. The slab thickness may need to be increased if forklift loads are imposed. The structural engineer should be consulted to determine the proper slab thickness.

8.7.2 The concrete slab-on-grade recommendations are based on soil support characteristics only. The project structural engineer should evaluate the structural requirements of the concrete slabs for supporting equipment and storage loads.

8.7.3 Concrete slabs on grade should be underlain by 4 inches of clean sand to reduce the potential for differential curing, slab curl, and cracking. Slabs that may receive moisture-sensitive floor coverings or may be used to store moisture-sensitive materials should be underlain by a vapor retarder placed near the middle of the sand bedding. The vapor retarder used should be specified by the project architect or developer based on the type of floor covering that will be installed. The vapor retarder design should be consistent with the guidelines presented in Section 9.3 of the American Concrete Institute's (ACI) *Guide for Concrete Slabs that Receive Moisture-Sensitive Flooring Materials* (ACI 302.2R-06).

8.7.4 To control the location and spread of concrete shrinkage cracks, crack control joints should be provided. The crack control joints should be created while the concrete is still fresh using a grooving tool, or shortly thereafter using saw cuts. The structural engineer should take into consideration criteria of the American Concrete Institute when establishing crack control spacing patterns.

8.7.5 Special subgrade presaturation is not deemed necessary prior to placing concrete; however, the exposed foundation and slab subgrade soil should be moisture conditioned, as necessary, to maintain a moist condition as would be expected in any such concrete placement.

8.7.6 Where buildings or other improvements are planned near the top of a slope steeper than 3:1 (horizontal:vertical), special foundations and/or design considerations are recommended due to the tendency for lateral soil movement to occur.

- For fill slopes less than 20 feet high, building footings should be deepened such that the bottom outside edge of the footing is at least 7 feet horizontally from the face of the slope.
- When located next to a descending 3:1 (horizontal to vertical) fill slope or steeper, the foundations should be extended to a depth where the minimum horizontal distance is equal to H/3 (where H equals the vertical distance from the top of the fill slope to the base of the fill soil) with a minimum of 7 feet but need not exceed 40 feet. The horizontal distance is measured from the outer, deepest edge of the footing to the face of the slope.
- Although other improvements, which are relatively rigid or brittle, such as concrete flatwork or masonry walls, may experience some distress if located near the top of a slope, it is generally not economical to mitigate this potential. It may be possible, however, to incorporate design measures that would permit some lateral soil movement without causing extensive distress. Geocon Incorporated should be consulted for specific recommendations.

8.7.7 The recommendations presented herein are intended to reduce the potential for cracking of slabs and foundations as a result of differential movement. However, even with the incorporation of the recommendations presented herein, foundations and slabs-on-grade will still crack. The occurrence of concrete shrinkage cracks is independent of the soil supporting characteristics. Their occurrence may be reduced and/or controlled by limiting the slump of the concrete, the use of crack control joints and proper concrete placement and curing. Crack control joints should be spaced at intervals no greater than 12 feet. Literature provided by the Portland Concrete Association (PCA) and American Concrete Institute (ACI) present recommendations for proper concrete mix, construction, and curing practices, and should be incorporated into project construction.

8.8 Concrete Flatwork

8.8.1 Exterior concrete flatwork not subject to vehicular traffic should be constructed in accordance with the recommendations herein. Slab panels should be a minimum of 4 inches thick and, when in excess of 8 feet square, should be reinforced with No. 3 reinforcing bars spaced 18 inches on center in both directions or 4 x 4 – W4.0/W4.0 (4 x 4 - 4/4) welded wire mesh to reduce the potential for cracking for subgrade soil with an Expansion Index of 90 or less. In addition, concrete flatwork should be provided with crack control joints to reduce and/or control shrinkage cracking. Crack control spacing should be determined by the project structural engineer based upon the slab thickness and intended usage. Criteria of the American Concrete Institute (ACI) should be taken into consideration when establishing crack control spacing. Subgrade soil for exterior slabs not subjected to vehicle loads should be compacted in accordance with criteria presented in the grading section prior to concrete placement. Subgrade soil should be properly compacted and the moisture content of subgrade soil should be evaluated prior to placing concrete.

8.8.2 Even with the incorporation of the recommendations within this report, the exterior concrete flatwork has a likelihood of experiencing some uplift due to expansive soil beneath grade; therefore, the reinforcement should overlap continuously in flatwork to reduce the potential for vertical offsets within flatwork. Additionally, flatwork should be structurally connected to the curbs, where possible, to reduce the potential for offsets between the curbs and the flatwork.

8.8.3 Where exterior flatwork abuts the structure at entry or exit points, the exterior slab should be dowelled into the structure’s foundation stemwall. This recommendation is intended to reduce the potential for differential elevations that could result from differential settlement or minor heave of the flatwork. Dowelling details should be designed by the project structural engineer.

8.9 Conventional Retaining Walls

8.9.1 Retaining walls not restrained at the top and having a level backfill surface should be designed for an active soil pressure equivalent to the pressure exerted by a fluid density of 40 pounds per cubic foot (pcf). Where the backfill will be inclined at no steeper than 2:1 (horizontal to vertical), an active soil pressure of 55 pcf is recommended. These soil pressures assume that the backfill materials within an area bounded by the wall and a 1:1 plane extending upward from the base of the wall possess an EI of 90 or less. For those lots with finish grade soils having an EI greater than 90 and/or where backfill materials do not conform to the criteria herein, Geocon Incorporated should be consulted for additional recommendations.

8.9.2 Unrestrained walls are those that are allowed to rotate more than 0.001H (where H equals the height of the retaining portion of the wall in feet) at the top of the wall. Where walls are restrained from movement at the top, an additional uniform pressure of 7H psf should be added to the above active soil pressure.

8.9.3 The structural engineer should determine the seismic design category for the project. If the project possesses a seismic design category of D, E, or F, the proposed retaining walls should be designed with seismic lateral pressure added to the active pressure. The seismic load exerted on the wall should be a triangular distribution with a pressure of 23H (where H is the height of the wall, in feet, resulting in pounds per square foot [psf]) exerted at the top of the wall and zero at the base of the wall. We used a peak site acceleration of 0.28g calculated form the 2007 California Building Code (S_{DS}/2.5) and applying a pseudo-static coefficient of 0.5.

8.9.4 Unrestrained walls will move laterally when backfilled and loading is applied. The amount of lateral deflection is dependant on the wall height, the type of soil used for backfill, and loads acting on the wall. The retaining walls and improvements above the retaining walls should be designed to incorporate an appropriate amount of lateral deflection as determined by the structural engineer.

8.9.5 Retaining walls should be provided with a drainage system adequate to prevent the buildup of hydrostatic forces and waterproofed as required by the project architect. The soil immediately adjacent to the backfilled retaining wall should be composed of free draining material completely wrapped in Mirafi 140 (or equivalent) filter fabric for a lateral distance of 1 foot for the bottom two-thirds of the height of the retaining wall. The upper one-third should be backfilled with less permeable compacted fill to reduce water infiltration. The use of drainage openings through the base of the wall (weep holes) is not recommended where the seepage could be a nuisance or otherwise adversely affect the property adjacent to the base of the wall. The recommendations herein assume a properly compacted granular (EI of 50 or less) free-draining backfill material with no hydrostatic forces or imposed surcharge load. Figure 9 presents a typical retaining wall drainage detail. If conditions different than those described are expected or if specific drainage details are desired, Geocon Incorporated should be contacted for additional recommendations.

8.9.6 In general, wall foundations having a minimum depth and width of 1 foot may be designed for an allowable soil bearing pressure of 2,000 psf, provided the soil within 4 feet below the base of the wall has an Expansion Index of 90 or less. The proximity of the foundation to the top of a slope steeper than 3:1 could impact the allowable soil bearing pressure. Therefore, Geocon Incorporated should be consulted where such a condition is expected.

8.9.7 The recommendations presented herein are generally applicable to the design of rigid concrete or masonry retaining walls having a maximum height of 8 feet. In the event that walls higher than 8 feet or other types of walls are planned, Geocon Incorporated should be consulted for additional recommendations.

8.10 Lateral Loads

8.10.1 For resistance to lateral loads, an allowable passive earth pressure equivalent to a fluid density of 350 pcf is recommended for footings or shear keys poured neat against properly compacted granular fill or undisturbed formational materials. The allowable passive pressure assumes a horizontal surface extending away from the base of the wall at least 5 feet or three times the height of the surface generating the passive pressure, whichever is greater. The upper 12 inches of material not protected by floor slabs or pavement should not be included in the design for lateral resistance.

8.10.2 An allowable friction coefficient of 0.35 may be used for resistance to sliding between soil and concrete. This friction coefficient may be combined with the allowable passive earth pressure when determining resistance to lateral loads.

8.11 Preliminary Pavement Recommendations

8.11.1 The final pavement sections for parking lots and roadways should be based on the R-Value of the subgrade soils encountered at final subgrade elevation. Streets should be designed in accordance with the County of San Diego specifications when final Traffic Indices and R-value test results of subgrade soil are completed. We calculated the flexible pavement sections in general conformance with the Caltrans Method of Flexible Pavement Design (Highway Design Manual, Section 608.4) Based on the results of our laboratory R-Value testing, we have assumed an R-Value of 5 for the subgrade soil for the purposes of this preliminary analysis. Preliminary flexible pavement sections are presented in Table 8.11.1.

TABLE 8.11.1
PRELIMINARY FLEXIBLE PAVEMENT SECTIONS

Location	Assumed Traffic Index	Assumed Subgrade R-Value	Asphalt Concrete (inches)	Class 2 Aggregate Base (inches)
Parking stalls for automobiles and light-duty vehicles	5.0	5	3	10
Driveway areas within industrial pads	6.0	5	4	12
Roadways	7.0	5	5	14
Major Roadways	8.0	5	5	18

8.11.2 The upper 12 inches of the subgrade soil should be compacted to a dry density of at least 95 percent of the laboratory maximum dry density near to slightly above optimum moisture content beneath pavement sections.

8.11.3 Base materials should conform to Section 26-1.028 of the *Standard Specifications for The State of California Department of Transportation (Caltrans)* with a ¾-inch maximum size aggregate. Base materials should be compacted to a dry density of at least 95 percent of the laboratory maximum dry density near to slightly above optimum moisture content. The asphalt concrete should conform to Section 203-6 of the *Standard Specifications for Public Works Construction (Greenbook)*. Asphalt concrete should be compacted to a density of at least 95 percent of the laboratory Hveem density in accordance with ASTM D 2726.

8.11.4 A rigid Portland cement concrete (PCC) pavement section should be placed in driveway entrance aprons and trash bin loading/storage areas. The concrete pad for trash truck areas should be large enough such that the truck wheels will be positioned on the concrete during loading. We calculated the rigid pavement section in general conformance with the procedure recommended by the American Concrete Institute report ACI 330R-01 Guide for Design and Construction of Concrete Parking Lots using the parameters presented in Table 8.11.2.

TABLE 8.11.2
RIGID PAVEMENT DESIGN PARAMETERS

Design Parameter	Design Value
Modulus of subgrade reaction, k	100 pci
Modulus of rupture for concrete, M_R	500 psi
Traffic Category, TC	A-1 and C
Average daily truck traffic, ADTT	10 and 100

8.11.5 Based on the criteria presented herein, the PCC pavement sections should have a minimum thickness as presented in Table 8.11.3.

TABLE 8.11.3
PRELIMINARY RIGID PAVEMENT RECOMMENDATIONS

Location	Portland Cement Concrete (inches)
Automobile Parking Areas	6
Trash and Heavy Truck and Fire Lane Areas	7

8.11.6 The PCC pavement should be placed over subgrade soil that is compacted to a dry density of at least 95 percent of the laboratory maximum dry density near to slightly above optimum moisture content. This pavement section is based on a minimum concrete compressive strength of approximately 3,000 psi (pounds per square inch).

8.11.7 A thickened edge or integral curb should be constructed on the outside of concrete slabs subjected to wheel loads. The thickened edge should be 1.2 times the slab thickness or a minimum thickness of 2 inches, whichever results in a thicker edge, at the slab edge and taper back to the recommended slab thickness 3 feet behind the face of the slab (e.g., a 7-inch-thick slab would have a 9-inch-thick edge). Reinforcing steel will not be necessary within the concrete for geotechnical purposes with the possible exception of dowels at construction joints as discussed below.

8.11.8 To control the location and spread of concrete shrinkage cracks, crack-control joints (weakened plane joints) should be included in the design of the concrete pavement slab. Crack-control joints should not exceed 30 times the slab thickness with a maximum spacing of 15 feet (e.g., a 7-inch-thick slab would have a 15-foot spacing pattern) and should be sealed with an appropriate sealant to prevent the migration of water through the control joint to the subgrade materials. The depth of the crack-control joints should be determined by the referenced ACI report.

8.11.9 To provide load transfer between adjacent pavement slab sections, a trapezoidal-keyed construction joint is recommended. As an alternative to the keyed joint, dowelling is recommended between construction joints. As discussed in the referenced ACI guide, dowels should consist of smooth, ⅞-inch-diameter reinforcing steel 14 inches long embedded a minimum of 6 inches into the slab on either side of the construction joint. Dowels should be located at the midpoint of the slab, spaced at 12 inches on center and lubricated to allow joint movement while still transferring loads. Other alternative recommendations for load transfer should be provided by the project structural engineer.

8.11.10 The performance of asphalt concrete pavement is highly dependent on providing positive surface drainage away from the edge of the pavement. The ponding of water on or adjacent to pavement areas should not be allowed as it will likely result in pavement distress and subgrade failure. Drainage from landscaped areas should be directed to controlled drainage structures. Landscape areas adjacent to the edge of asphalt pavements are not recommended due to the potential for surface or irrigation water to infiltrate the underlying permeable aggregate base and cause distress. Where such a condition cannot be avoided, consideration should be given to incorporating measures that will significantly reduce the potential for subsurface water migration into the aggregate base. If planter islands are planned, the perimeter curb should extend at least 6 inches below the level of the base materials.

8.12 Site Drainage and Moisture Protection

8.12.1 Adequate site drainage is critical to reduce the potential for differential soil movement, erosion and subsurface seepage. Under no circumstances should water be allowed to pond adjacent to footings. The site should be graded and maintained such that surface drainage is directed away from structures in accordance with 2007 CBC 1803.3 or other applicable standards. In addition, surface drainage should be directed away from the top of slopes into swales or other controlled drainage devices. Roof and pavement drainage should be directed into conduits that carry runoff away from the proposed structure.

8.12.2 In the case of basement walls or building walls retaining landscaping areas, a water-proofing system should be used on the wall and joints, and a Miradrain drainage panel (or similar) should be placed over the waterproofing. A perforated drainpipe of schedule 40 or better should be installed at the base of the wall below the floor slab and drained to an appropriate discharge area. Accordion-type pipe is not acceptable. The project architect or civil engineer should provide detailed specifications on the plans for all waterproofing and drainage.

8.12.3 Underground utilities should be leak free. Utility and irrigation lines should be checked periodically for leaks, and detected leaks should be repaired promptly. Detrimental soil movement could occur if water is allowed to infiltrate the soil for prolonged periods of time.

8.12.4 Landscaping planters adjacent to paved areas are not recommended due to the potential for surface or irrigation water to infiltrate the pavement's subgrade and base course. We recommend that area drains to collect excess irrigation water and transmit it to drainage structures or impervious above-grade planter boxes be used. In addition, where landscaping is planned adjacent to the pavement, we recommend construction of a cutoff wall along the edge of the pavement that extends at least 6 inches below the bottom of the base material.

8.12.5 If detention basins, bioswales, retention basins, or water infiltration devices are being considered, Geocon Incorporated should be retained to provide recommendations pertaining to the geotechnical aspects of possible impacts and design. Distress may be caused to planned improvements and properties located hydrologically downstream. The distress depends on the amount of water to be detained, its residence time, soil permeability, and other factors. We have not performed a hydrogeology study at the site. Downstream properties may be subjected to seeps, springs, slope instability, raised groundwater, movement of foundations and slabs, or other impacts as a result of water infiltration.

8.13 Grading and Foundation Plan Review

8.13.1 Geocon Incorporated should review the grading plans and foundation plans for the project prior to final design submittal to evaluate whether additional analysis and/or recommendations are required.

LIMITATIONS AND UNIFORMITY OF CONDITIONS

1. The recommendations of this report pertain only to the site investigated and are based upon the assumption that the soil conditions do not deviate from those disclosed in the investigation. If any variations or undesirable conditions are encountered during construction, or if the proposed construction will differ from that anticipated herein, Geocon Incorporated should be notified so that supplemental recommendations can be given. The evaluation or identification of the potential presence of hazardous or corrosive materials was not part of the scope of services provided by Geocon Incorporated.

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

3. The findings of this report are valid as of the present date. However, changes in the conditions of a property can occur with the passage of time, whether they are due to natural processes or the works of man on this or adjacent properties. In addition, changes in applicable or appropriate standards may occur, whether they result from legislation or the broadening of knowledge. Accordingly, the findings of this report may be invalidated wholly or partially by changes outside our control. Therefore, this report is subject to review and should not be relied upon after a period of three years.

4. The firm that performed the geotechnical investigation for the project should be retained to provide testing and observation services during construction to provide continuity of geotechnical interpretation and to check that the recommendations presented for geotechnical aspects of site development are incorporated during site grading, construction of improvements, and excavation of foundations. If another geotechnical firm is selected to perform the testing and observation services during construction operations, that firm should prepare a letter indicating their intent to assume the responsibilities of project geotechnical engineer of record. A copy of the letter should be provided to the regulatory agency for their records. In addition, that firm should provide revised recommendations concerning the geotechnical aspects of the proposed development, or a written acknowledgement of their concurrence with the recommendations presented in our report. They should also perform additional analyses deemed necessary to assume the role of Geotechnical Engineer of Record.

N

NO SCALE

GEOCON INCORPORATED

GEOTECHNICAL CONSULTANTS
6960 FLANDERS DRIVE - SAN DIEGO, CALIFORNIA 92121 - 2974
PHONE 858 558-6900 - FAX 858 558-6159

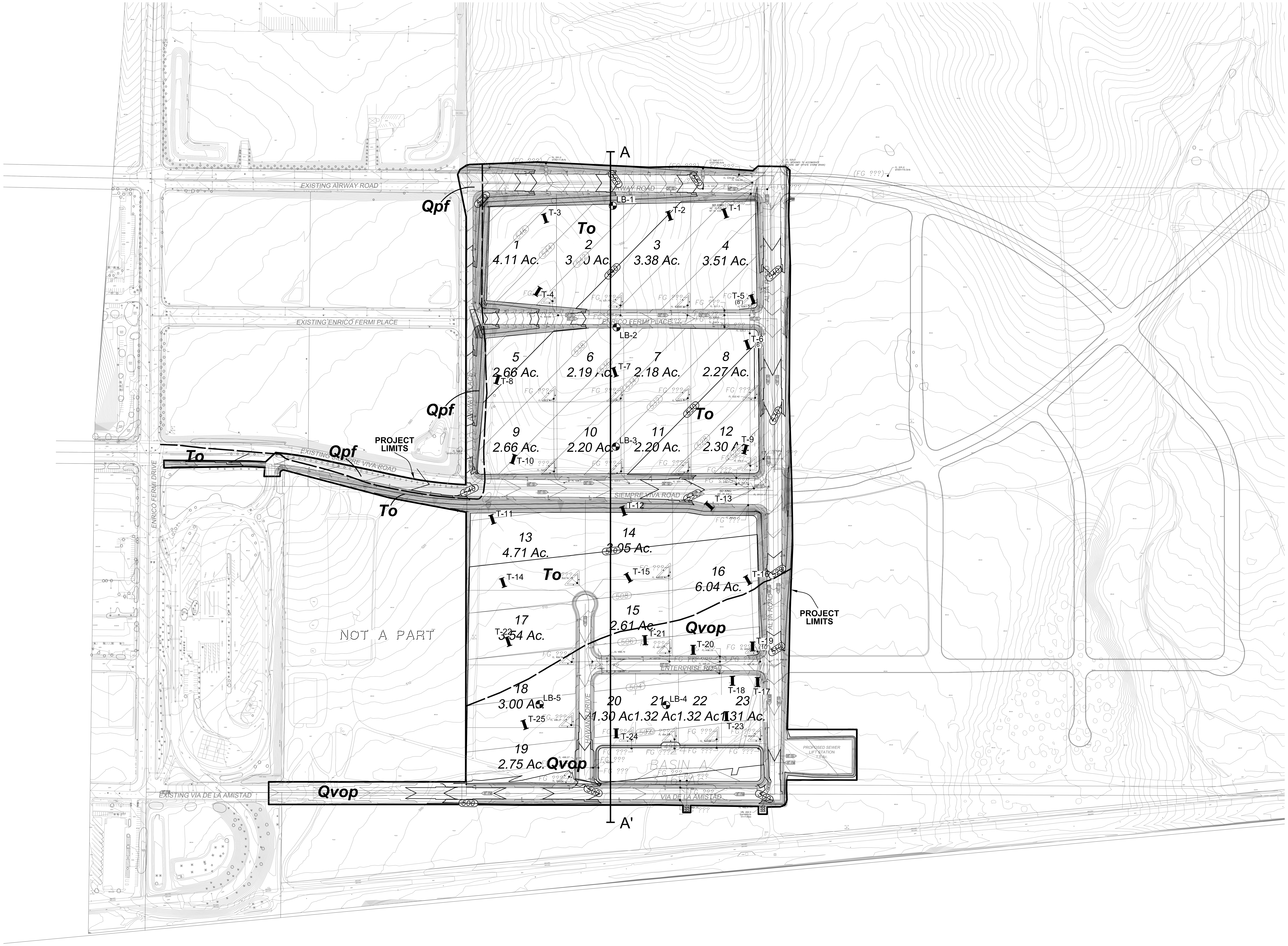
VICINITY MAP

HAWANO
EAST OTAY PROPERTY
SAN DIEGO COUNTY, CALIFORNIA

ME / RS		DSK/GTYPD	DATE 07 - 07 - 2010	PROJECT NO. G1223 - 52 - 01	FIG. 1

G1223-42-01_VICMAP_FIG1_ME..DWG

Y:\R14TEMP\1_GEOTECH\1_GEOTECH_RUBEN\GEOCON 2010\G1223-52-01 HAWANO\DETAILS\G1223-52-01_VICMAP_FIG1_ME.dwg



GEOCON LEGEND

Qpf	PREVIOUSLY PLACED FILL (2004)
Qvop	VERY OLD PARALIC DEPOSITS UNDIVIDED
To	OTAY FORMATION
T-25	APPROX. LOCATION OF EXPLORATORY TRENCH
LB-5	APPROX. LOCATION OF LARGE DIAMETER BORING
	APPROX. LOCATION OF GEOLOGIC CONTACT
A — A'	GEOLOGIC CROSS-SECTION

GEOLOGIC MAP

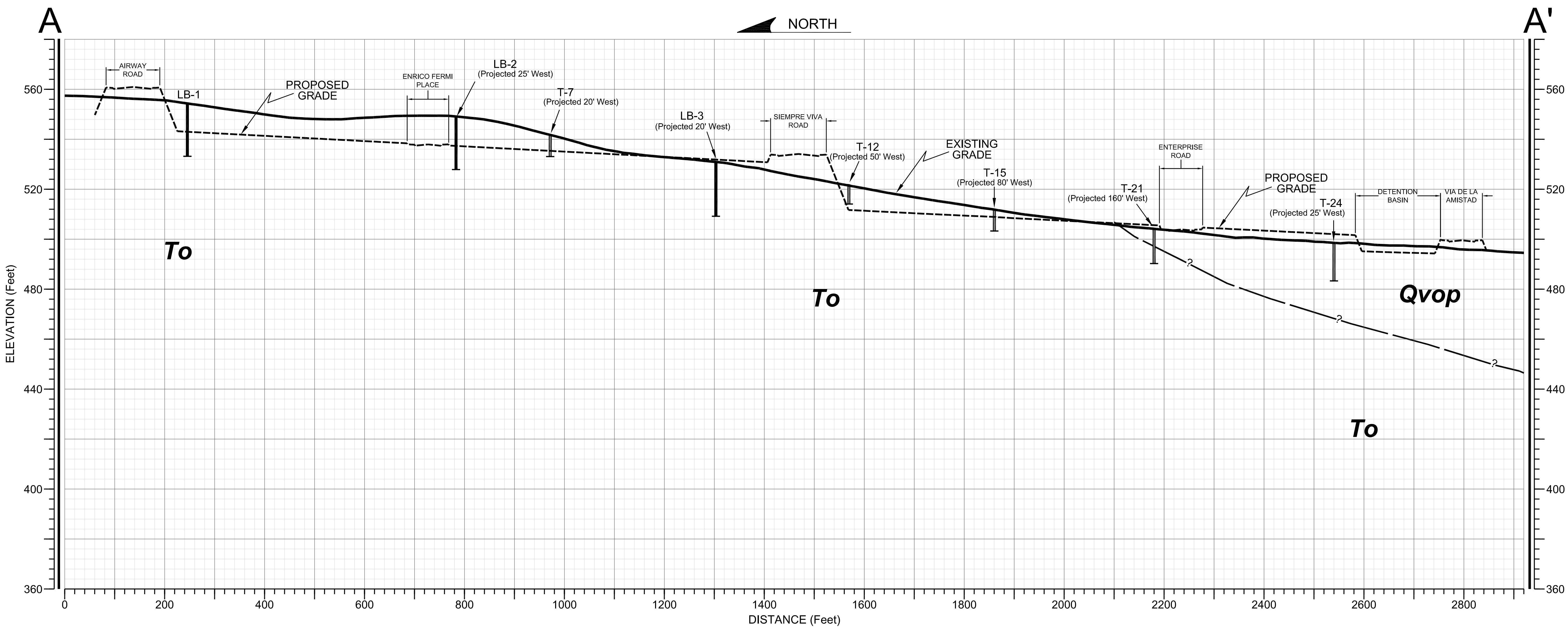
HAWANO
EAST OTAY PROPERTY
SAN DIEGO COUNTY, CALIFORNIA

GEOCON INCORPORATED
GEOTECHNICAL CONSULTANTS
6960 FLANDERS DRIVE - SAN DIEGO, CALIFORNIA 92121 - 2974
PHONE 858 558-6900 - FAX 858 558-6159

SCALE 1" = 200'	DATE 07 - 07 - 2010	
PROJECT NO. G1223 - 52 - 01		FIGURE 2
SHEET 1 OF 1		

Y:\R14TEMP\1_GEOTECH\1_GEOTECH_RUBEN\GEOCON 2010\G1223-52-01 HAWANO\JH_G1223 GEO-MAP.dwg

HAWANO
EAST OTAY PROPERTY
SAN DIEGO COUNTY, CALIFORNIA



GEOLOGIC CROSS - SECTION A-A'

SCALE: 1" = 200' (Horiz.); 1" = 40' (Vert.)

GEOCON LEGEND

Qvop ....VERY OLD PARALIC DEPOSITS UNDIVIDED

To ....OTAY FORMATION

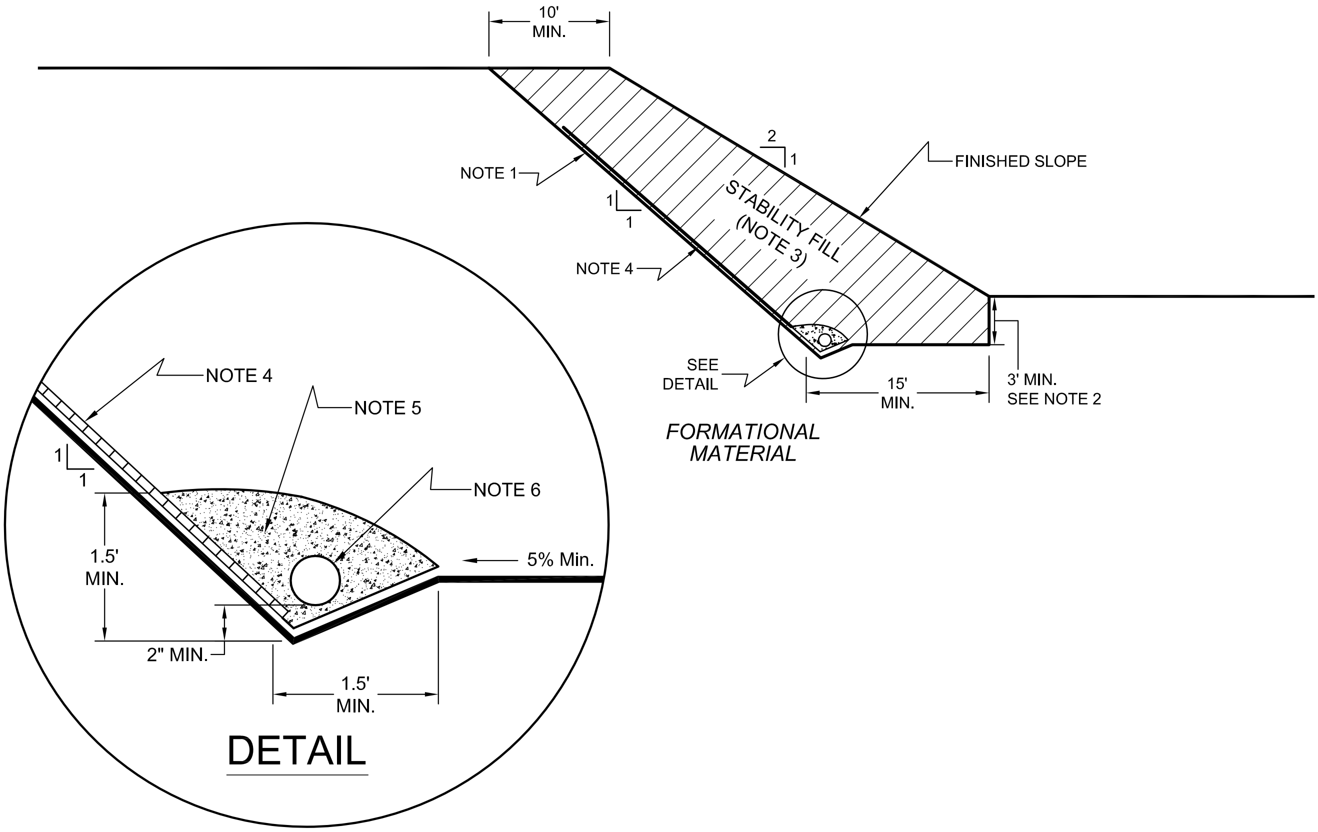
LB-3 ....APPROX. LOCATION OF LARGE DIAMETER BORING

T-24 ....APPROX. LOCATION OF TRENCH

?........APPROX. LOCATION OF GEOLOGIC CONTACT
(Queried Where Uncertain)

GEOCON
INCORPORATED
GEOTECHNICAL CONSULTANTS
6960 FLANDERS DRIVE - SAN DIEGO, CALIFORNIA 92121 - 2974
PHONE 858 558-6900 - FAX 858 558-6159

PROJECT NO. G1223 - 52 - 01
FIGURE 3
DATE 07 - 07 - 2010



NOTES:

1.....EXCAVATE BACKCUT AT 1:1 INCLINATION (UNLESS OTHERWISE NOTED).

2.....BASE OF STABILITY FILL TO BE 3 FEET INTO FORMATIONAL MATERIAL, SLOPING A MINIMUM 5% INTO SLOPE.

3.....STABILITY FILL TO BE COMPOSED OF PROPERLY COMPACTED GRANULAR SOIL.

4.....CHIMNEY DRAINS TO BE APPROVED PREFABRICATED CHIMNEY DRAIN PANELS (MIRADRAIN G200N OR EQUIVALENT) SPACED APPROXIMATELY 20 FEET CENTER TO CENTER AND 4 FEET WIDE. CLOSER SPACING MAY BE REQUIRED IF SEEPAGE IS ENCOUNTERED.

5.....FILTER MATERIAL TO BE 3/4-INCH, OPEN-GRADED CRUSHED ROCK ENCLOSED IN APPROVED FILTER FABRIC (MIRAFI 140NC).

6.....COLLECTOR PIPE TO BE 4-INCH MINIMUM DIAMETER, PERFORATED, THICK-WALLED PVC SCHEDULE 40 OR EQUIVALENT, AND SLOPED TO DRAIN AT 1 PERCENT MINIMUM TO APPROVED OUTLET.

NO SCALE

TYPICAL STABILITY FILL DETAIL

GEOCON
INCORPORATED
GEOTECHNICAL CONSULTANTS
6960 FLANDERS DRIVE - SAN DIEGO, CALIFORNIA 92121 - 2974
PHONE 858 558-6900 - FAX 858 558-6159

HAWANO
EAST OTAY PROPERTY
SAN DIEGO COUNTY, CALIFORNIA

ME / RS		DSK/GTYPD	DATE 07 - 07 - 2010	PROJECT NO. G1223 - 52 - 01	FIG. 4

ASSUMED CONDITIONS :

SLOPE HEIGHT	H	= 30 feet
SLOPE INCLINATION	2 : 1	(Horizontal : Vertical)
TOTAL UNIT WEIGHT OF SOIL	γ_t	= 130 pounds per cubic foot
ANGLE OF INTERNAL FRICTION	ϕ	= 27 degrees
APPARENT COHESION	C	= 435 pounds per square foot
NO SEEPAGE FORCES		

ANALYSIS :

$\gamma_{c\phi} = \dfrac{\gamma H \tan\phi}{C}$		EQUATION (3-3), REFERENCE 1
$FS = \dfrac{N_{cf} C}{\gamma H}$		EQUATION (3-2), REFERENCE 1
$\gamma_{c\phi}$	= 4.4	CALCULATED USING EQ. (3-3)
N_{cf}	= 19	DETERMINED USING FIGURE 10, REFERENCE 2
FS	= 2.2	FACTOR OF SAFETY CALCULATED USING EQ. (3-2)

REFERENCES :

1......Janbu, N., Stability Analysis of Slopes with Dimensionless Parameters, Harvard Soil Mechanics, Series No. 46, 1954

2......Janbu, N., Discussion of J.M. Bell, Dimensionless Parameters for Homogeneous Earth Slopes, Journal of Soil Mechanics and Foundation Design, No. SM6, November 1967.

SLOPE STABILITY ANALYSIS - CUT SLOPES

GEOCON INCORPORATED

GEOTECHNICAL CONSULTANTS
6960 FLANDERS DRIVE - SAN DIEGO, CALIFORNIA 92121 - 2974
PHONE 858 558-6900 - FAX 858 558-6159

HAWANO
EAST OTAY PROPERTY
SAN DIEGO COUNTY, CALIFORNIA

JH / RS		DSK/GTYPD	DATE 07 - 07 - 2010	PROJECT NO. G1223 - 52 - 01	FIG. 5

ASSUMED CONDITIONS :

SLOPE HEIGHT	H	= 40 feet
SLOPE INCLINATION	2 : 1 (Horizontal : Vertical)	
TOTAL UNIT WEIGHT OF SOIL	γ_t	= 130 pounds per cubic foot
ANGLE OF INTERNAL FRICTION	ϕ	= 27 degrees
APPARENT COHESION	C	= 250 pounds per square foot
NO SEEPAGE FORCES		

ANALYSIS :

$$\gamma_{c\phi} = \frac{\gamma H \tan\phi}{C}$$ EQUATION (3-3), REFERENCE 1

$$FS = \frac{N_{cf} C}{\gamma H}$$ EQUATION (3-2), REFERENCE 1

$\gamma_{c\phi}$	=	10.2	CALCULATED USING EQ. (3-3)
N_{cf}	=	35	DETERMINED USING FIGURE 10, REFERENCE 2
FS	=	1.8	FACTOR OF SAFETY CALCULATED USING EQ. (3-2)

REFERENCES :

1......Janbu, N., Stability Analysis of Slopes with Dimensionless Parameters, Harvard Soil Mechanics, Series No. 46, 1954

2......Janbu, N., Discussion of J.M. Bell, Dimensionless Parameters for Homogeneous Earth Slopes, Journal of Soil Mechanics and Foundation Design, No. SM6, November 1967.

SLOPE STABILITY ANALYSIS - FILL SLOPES

GEOCON INCORPORATED

GEOTECHNICAL CONSULTANTS
6960 FLANDERS DRIVE - SAN DIEGO, CALIFORNIA 92121 - 2974
PHONE 858 558-6900 - FAX 858 558-6159

HAWANO
EAST OTAY PROPERTY
SAN DIEGO COUNTY, CALIFORNIA

JH / RS		DSK/GTYPD	DATE 07 - 07 - 2010	PROJECT NO. G1223 - 52 - 01	FIG. 6

ASSUMED CONDITIONS :

SLOPE HEIGHT	H	= Infinite
DEPTH OF SATURATION	Z	= 3 feet
SLOPE INCLINATION	2 : 1 (Horizontal : Vertical)	
SLOPE ANGLE	i	= 26.5 degrees
UNIT WEIGHT OF WATER	γ_w	= 62.4 pounds per cubic foot
TOTAL UNIT WEIGHT OF SOIL	γ_t	= 125 pounds per cubic foot
ANGLE OF INTERNAL FRICTION	ϕ	= 27 degrees
APPARENT COHESION	C	= 300 pounds per square foot

SLOPE SATURATED TO VERTICAL DEPTH Z BELOW SLOPE FACE

SEEPAGE FORCES PARALLEL TO SLOPE FACE

ANALYSIS :

$$FS = \frac{C + (\gamma_t - \gamma_w)\ Z \cos^2 i \tan \phi}{\gamma_t\ Z \sin i \cos i} \qquad 2.1$$

REFERENCES :

1......Haefeli, R. *The Stability of Slopes Acted Upon by Parallel Seepage*, Proc. Second International Conference, SMFE, Rotterdam, 1948, 1, 57-62

2......Skempton, A. W., and F.A. Delory, *Stability of Natural Slopes in London Clay*, Proc. Fourth International Conference, SMFE, London, 1957, 2, 378-81

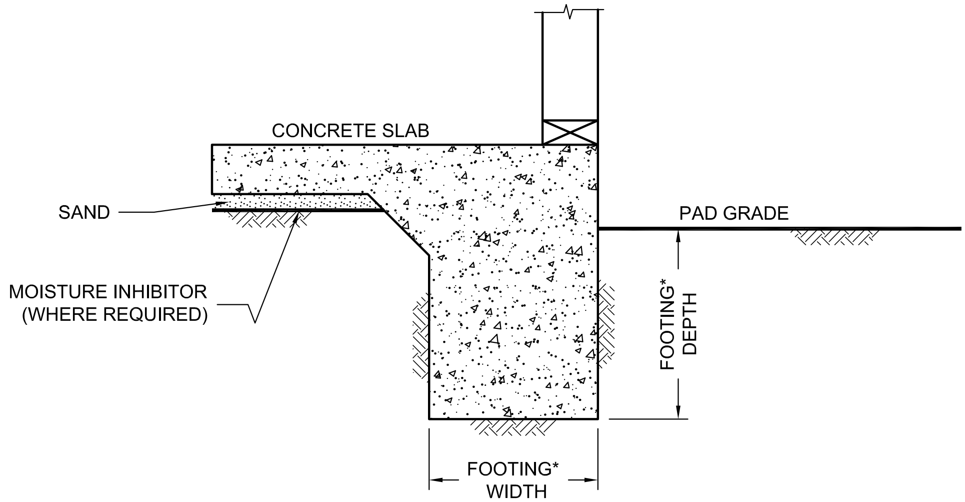
SURFICIAL SLOPE STABILITY ANALYSIS

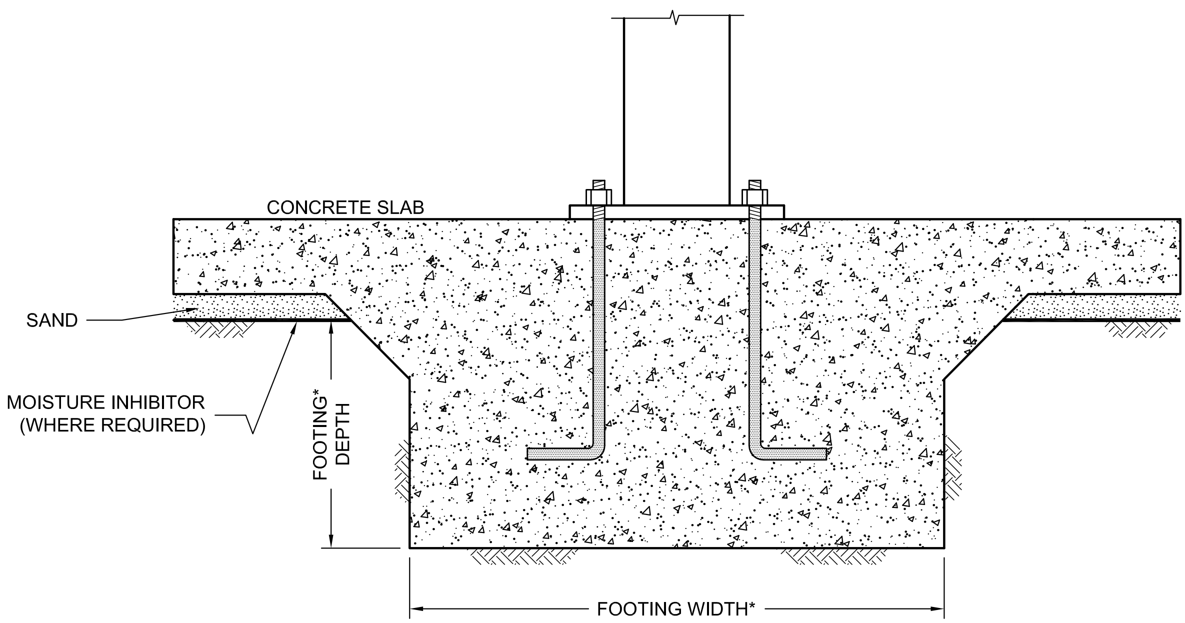
GEOCON INCORPORATED

GEOTECHNICAL CONSULTANTS
6960 FLANDERS DRIVE - SAN DIEGO, CALIFORNIA 92121 - 2974
PHONE 858 558-6900 - FAX 858 558-6159

HAWANO
EAST OTAY PROPERTY
SAN DIEGO COUNTY, CALIFORNIA

JH / RS		DSK/GTYPD	DATE 07 - 07 - 2010	PROJECT NO. G1223 - 52 - 01	FIG. 7





*....SEE REPORT FOR FOUNDATION WITDH AND DEPTH RECOMMENDATION

NO SCALE

WALL / COLUMN FOOTING DIMENSION DETAIL

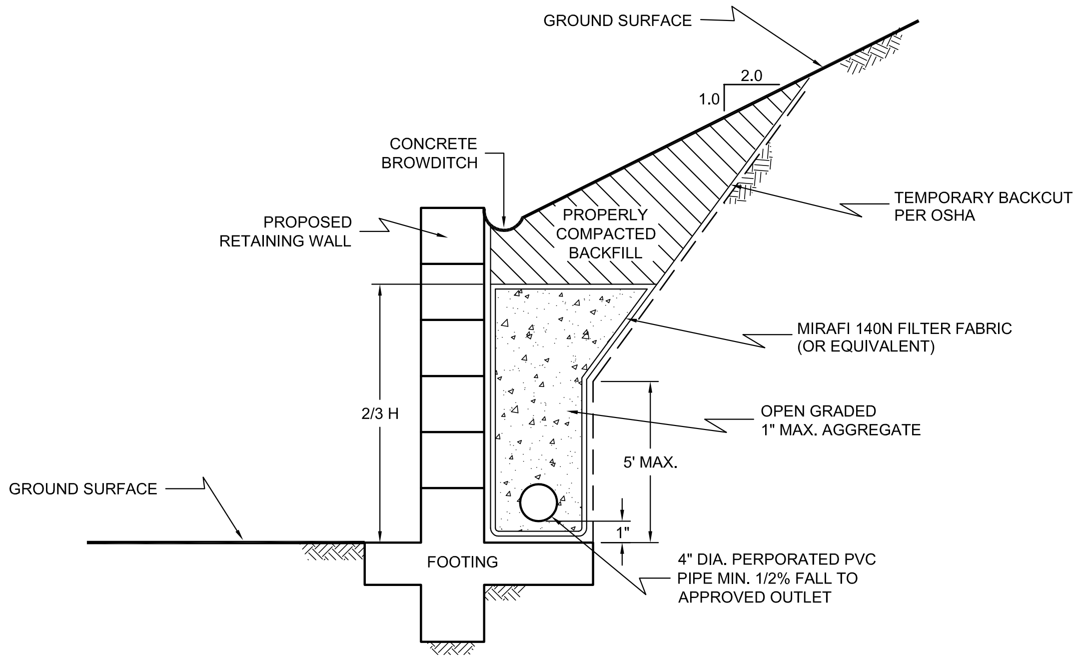
GEOCON
INCORPORATED
GEOTECHNICAL CONSULTANTS
6960 FLANDERS DRIVE - SAN DIEGO, CALIFORNIA 92121 - 2974
PHONE 858 558-6900 - FAX 858 558-6159

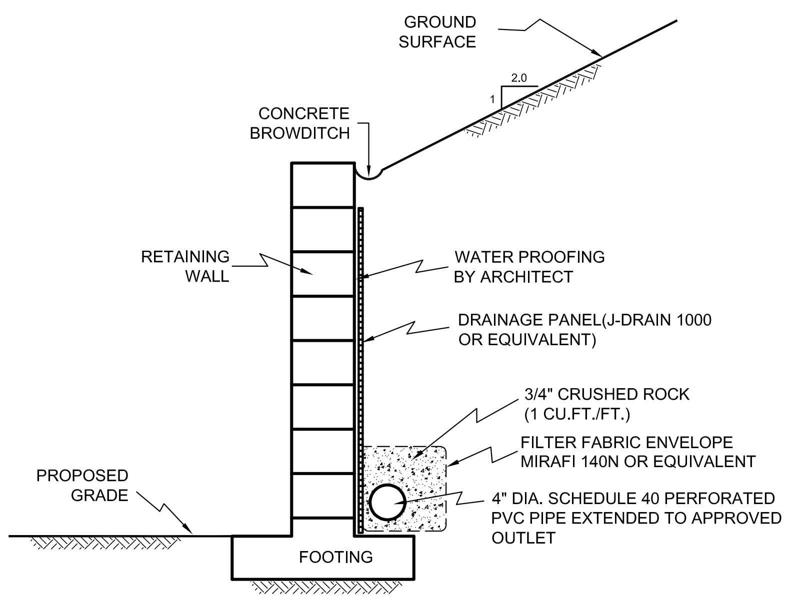
HAWANO
EAST OTAY PROPERTY
SAN DIEGO COUNTY, CALIFORNIA

ME / RS		DSK/GTYPD	DATE 07 - 07 - 2010	PROJECT NO. G1223 - 52 - 01	FIG. 8

G1223-05-01_COLFOOT2_FIG3_ME..DWG

Y:\R14TEMP\1_GEOTECH\1_GEOTECH_RUBEN\GEOCON 2010\G1223-52-01 HAWANO\DETAILS\G1223-52-01_COLFOOT2_FIG8_ME.dwg





NOTE :

DRAIN SHOULBE UNIFORMLY SLOPED AT 1/2% OR GREATER TO GRAVITY OUTLET OR TO A SUMP WHERE WATER CAN BE REMOVED BY PUMPING

NO SCALE

TYPICAL RETAINING WALL DRAIN DETAIL

GEOCON INCORPORATED
GEOTECHNICAL CONSULTANTS
6960 FLANDERS DRIVE - SAN DIEGO, CALIFORNIA 92121 - 2974
PHONE 858 558-6900 - FAX 858 558-6159

HAWANO
EAST OTAY PROPERTY
SAN DIEGO COUNTY, CALIFORNIA

ME / RS		DSK/GTYPD	DATE 07 - 07 - 2010	PROJECT NO. G1223 - 52 - 01	FIG. 9

APPENDIX A

EXPLORATORY EXCAVATIONS

Our subsurface exploration consisted of drilling 5 large diameter borings and excavating 25 backhoe trenches. We performed the field investigation on April 26 through 30, 2010. The locations of the exploratory borings and trenches were determined in the field using the topographic map provided by the project civil engineer and using compass and tape. The large diameter borings were excavated to a maximum depth of 19.9 feet with a truck-mounted drill rig equipped with a 30-inch-diameter bucket-auger. The backhoe trenches were excavated using a John Deere 310 backhoe equipped with a 24-inch-wide bucket and extended to a maximum depth of 17 feet. The approximate boring and trench locations are shown on the Geologic Map (Figure 2).

We obtained samples during our subsurface exploration in the borings using a Modified California sampler. The sampler is composed of steel and is driven to obtain ring samples. The Modified California sampler has an inside diameter of 2.5 inches and an outside diameter of 3 inches. Up to 18 rings are placed inside the sampler that is 2.375 inches in diameter and 1 inch in height. We placed the ring samples in moisture-tight containers and transported them to the laboratory for testing. We also obtained bulk samples for laboratory testing.

The sampler was driven 12 to 18 inches into the bottom of the excavation with the use of a telescoping Kelly bar. The weight of the Kelly bar (4,500 pounds maximum) drives the sampler and varies in weight with depth. The height of drop is usually 18 inches. Blow counts are recorded for every 12 inches the sampler is driven. The penetration resistance values shown on the boring logs are shown in terms of blows per foot. These values are not to be taken as N-values and adjustments have not been applied.

We estimated elevations shown on the boring and trench logs using the topographic map. We visually examined, classified, and logged the soil conditions encountered in the borings and trenches in general conformance with the American Society for Testing and Materials (ASTM) Practice for Description and Identification of Soils (Visual-Manual Procedure D 2844). The logs of the exploratory borings and trenches are presented on Figures A-1 through A-29 and included herein. The logs depict the various soil types encountered and indicate the depths at which samples were obtained.

APPENDIX B

LABORATORY TESTING

We performed laboratory tests in accordance with generally accepted test methods of the American Society for Testing and Materials (ASTM) or other suggested procedures. Selected soil samples were analyzed for in-situ dry density and moisture content, maximum dry density and optimum moisture content, direct shear strength, expansion potential, water-soluble sulfate, water-soluble chloride ion, pH and resistivity, R-Value, sand equivalent, gradation, and consolidation. The results of the laboratory tests are presented on Tables B-I through B-VIII and Figures B-1 through B-4. The in-place dry density and moisture content of the samples tested are presented on the boring logs in Appendix A.

TABLE B-I
SUMMARY OF LABORATORY MAXIMUM DRY DENSITY AND OPTIMUM MOISTURE CONTENT TEST RESULTS
ASTM D 1557

Sample No.	Description	Maximum Dry Density (pcf)	Optimum Moisture Content (% dry wt.)
T1-1	Very dark brown, fine to medium, Sandy CLAY with trace gravel	108.6	15.2
T8-2	Light grayish brown, Clayey SILT with little fine sand	102.3	19.7
T23-1	Reddish brown, Clayey, fine to coarse SAND with trace gravel	126.5	9.4

TABLE B-II
SUMMARY OF LABORATORY DIRECT SHEAR STRENGTH TEST RESULTS
ASTM D 3080

Sample No.	Dry Density (pcf)	Moisture Content (%)		Peak [Ultimate] Cohesion (psf)	Peak [Ultimate] Angle of Shear Resistance (degrees)
		Initial	After Test		
LB1-1	97.4	17.3	26.2	1655[1200]	36[27]
LB1-2	105.7	10.9	21.3	1085[575]	33[36]
LB3-3	93.4	26.5	33.6	855[435]	28[27]

TABLE B-III
SUMMARY OF LABORATORY EXPANSION INDEX TEST RESULTS
ASTM D 4829

Sample No.	Moisture Content (%)		Dry Density (pcf)	Expansion Index	Expansion Classification
	Before Test	After Test			
T1-1	15.9	39.8	89.6	149	Very High
T8-2	14.9	36.0	90.8	82	Medium
T23-1	9.8	21.2	108.5	66	Medium

TABLE B-IV
SUMMARY OF LABORATORY WATER-SOLUBLE SULFATE TEST RESULTS
CALIFORNIA TEST NO. 417

Sample No.	Water-Soluble Sulfate (%)	Water-Soluble Sulfate (ppm)	Sulfate Exposure*
T8-2	0.004	40	Negligible
T23-1	0.105	1,050	Moderate

*Reference: 2007 California Building Code.

TABLE B-V
SUMMARY OF LABORATORY WATER-SOLUBLE CHLORIDE ION CONTENT TEST RESULTS
AASHTO TEST NO. T 291

Sample No.	Chloride Ion Content (%)	Chloride Ion Content (ppm)
T8-2	0.005	45

TABLE B-VI
SUMMARY OF LABORATORY POTENTIAL OF HYDROGEN (pH) AND RESISTIVITY TEST RESULTS
CALIFORNIA TEST METHOD 643

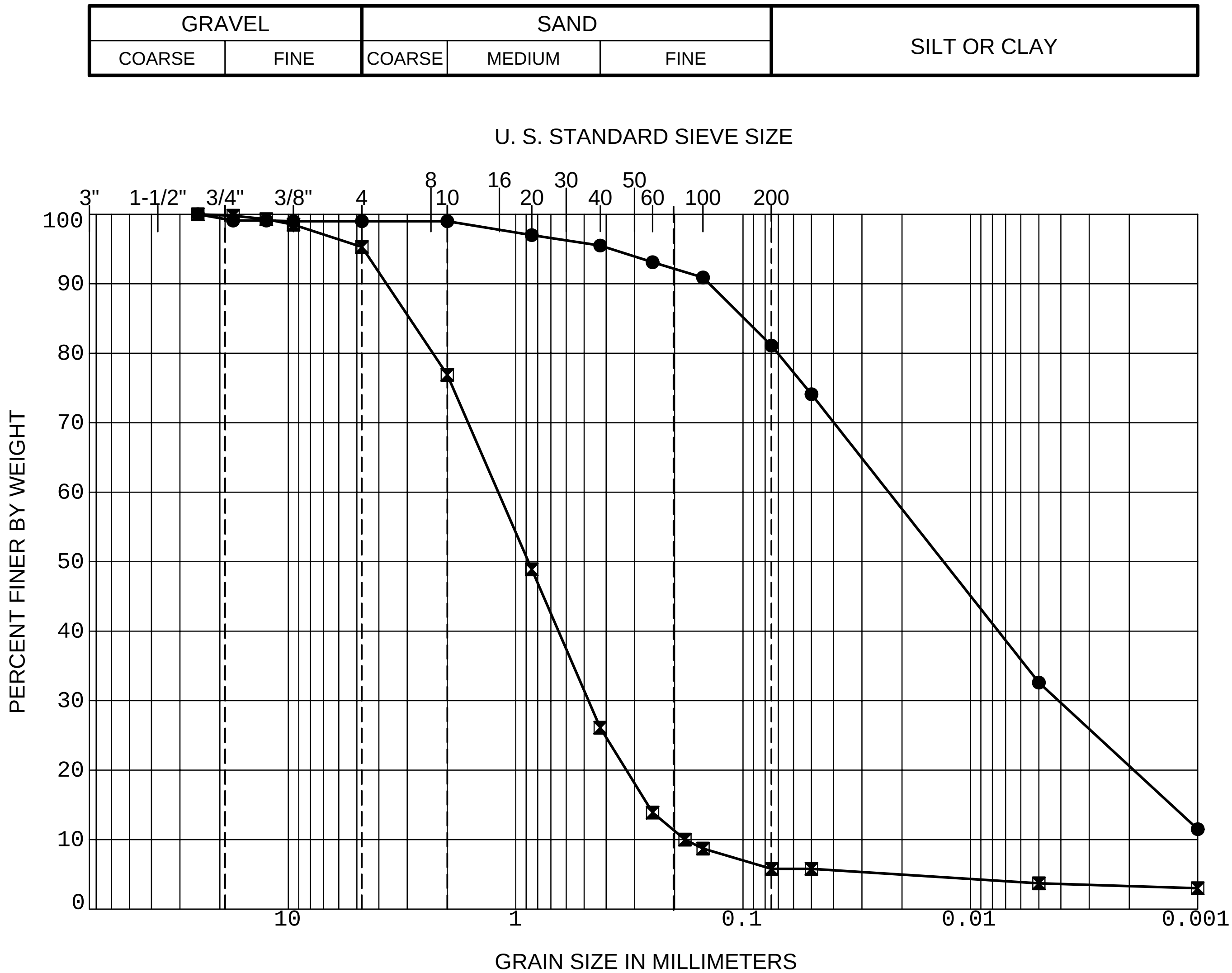
Sample No.	pH	Minimum Resistivity (ohm-centimeters)
T8-2	9.2	1030

TABLE B-VII
SUMMARY OF LABORATORY RESISTANCE VALUE (R-VALUE) TEST RESULTS
ASTM D 2844

Sample No.	R-Value
T8-2	26
T23-1	8

TABLE B-VIII
SUMMARY OF LABORATORY SAND EQUIVALENT TEST RESULTS
ASTM D 2419

Sample Number	Sand Equivalent
T23-3	44



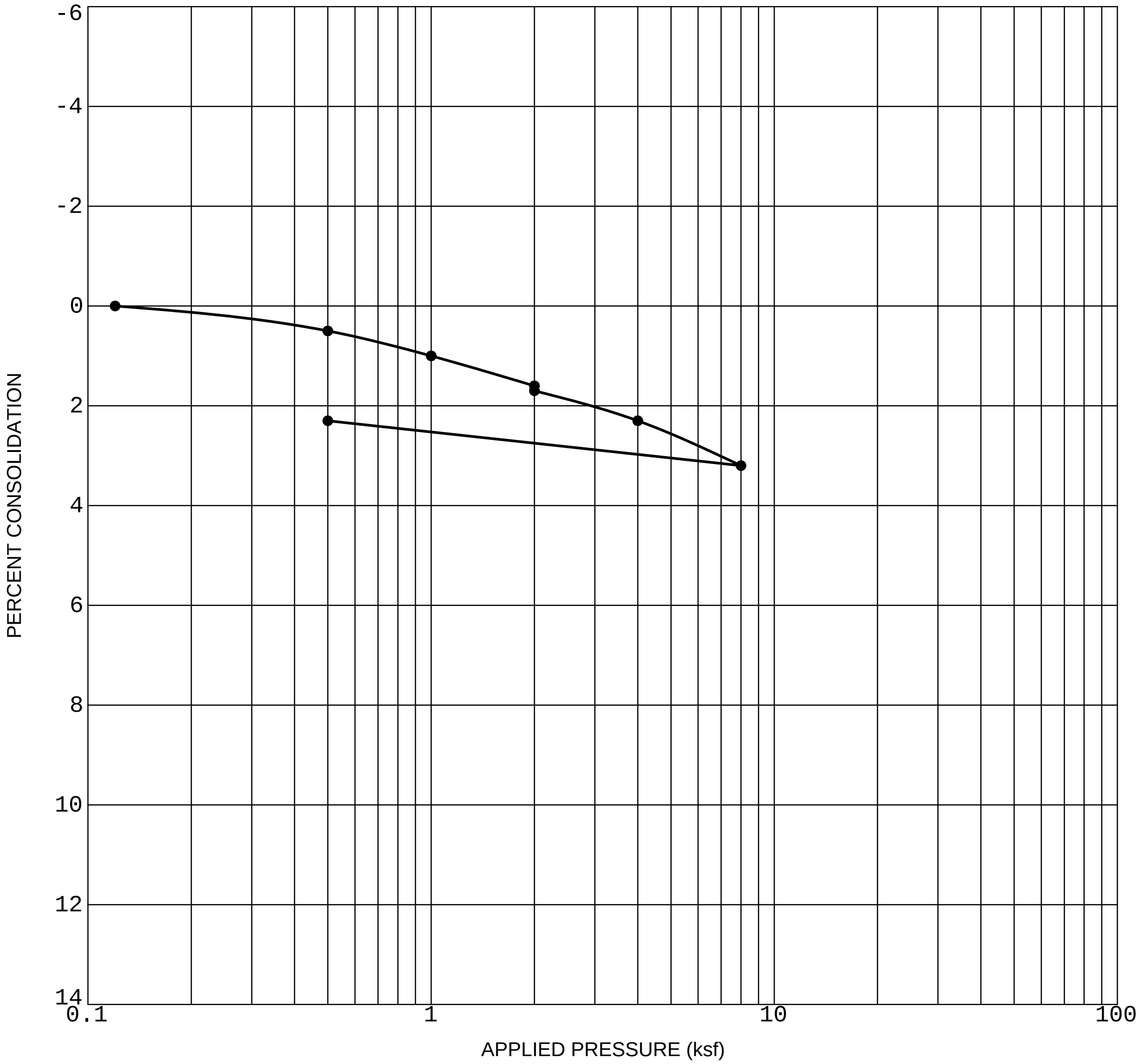
	SAMPLE	DEPTH (ft)	CLASSIFICATION	NAT WC	LL	PL	PI
●	T8-2	4.0	(CL) Silty CLAY				
⊠	T23-3	6.0	(SP/SM) SAND with silt				

GRADATION CURVE

HAWANO
EAST OTAY PROPERTY
SAN DIEGO COUNTY, CALIFORNIA

PROJECT NO. G1223-52-01

SAMPLE NO. LB4-2



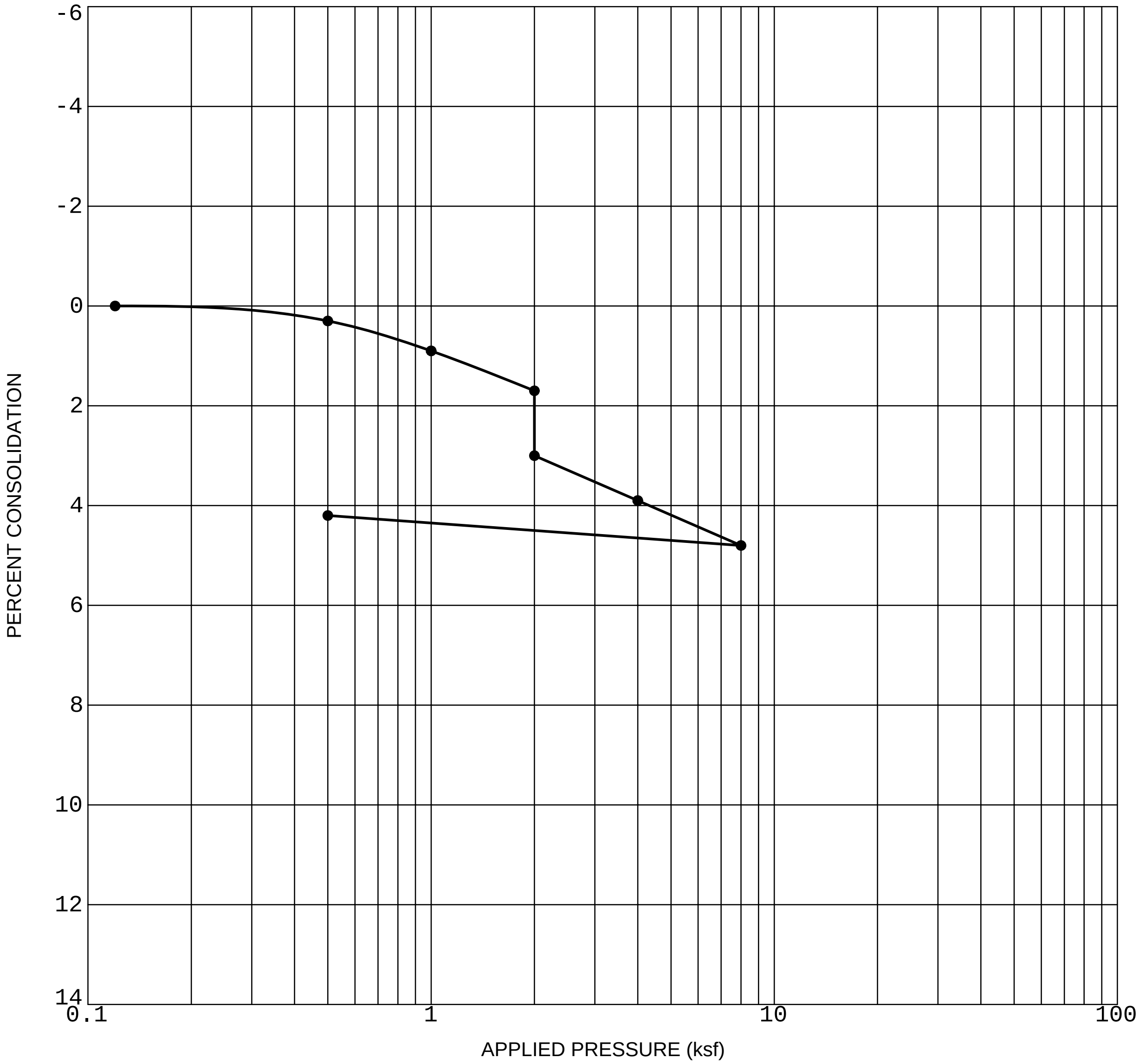
Initial Dry Density (pcf)	109.6		**Initial Saturation (%)**	70.4
Initial Water Content (%)	13.6		**Sample Saturated at (ksf)**	2.0

CONSOLIDATION CURVE

HAWANO
EAST OTAY PROPERTY
SAN DIEGO COUNTY, CALIFORNIA

PROJECT NO. G1223-52-01

SAMPLE NO. LB4-3



Initial Dry Density (pcf)	114.3		**Initial Saturation (%)**	12.0
Initial Water Content (%)	2.0		**Sample Saturated at (ksf)**	2.0

CONSOLIDATION CURVE

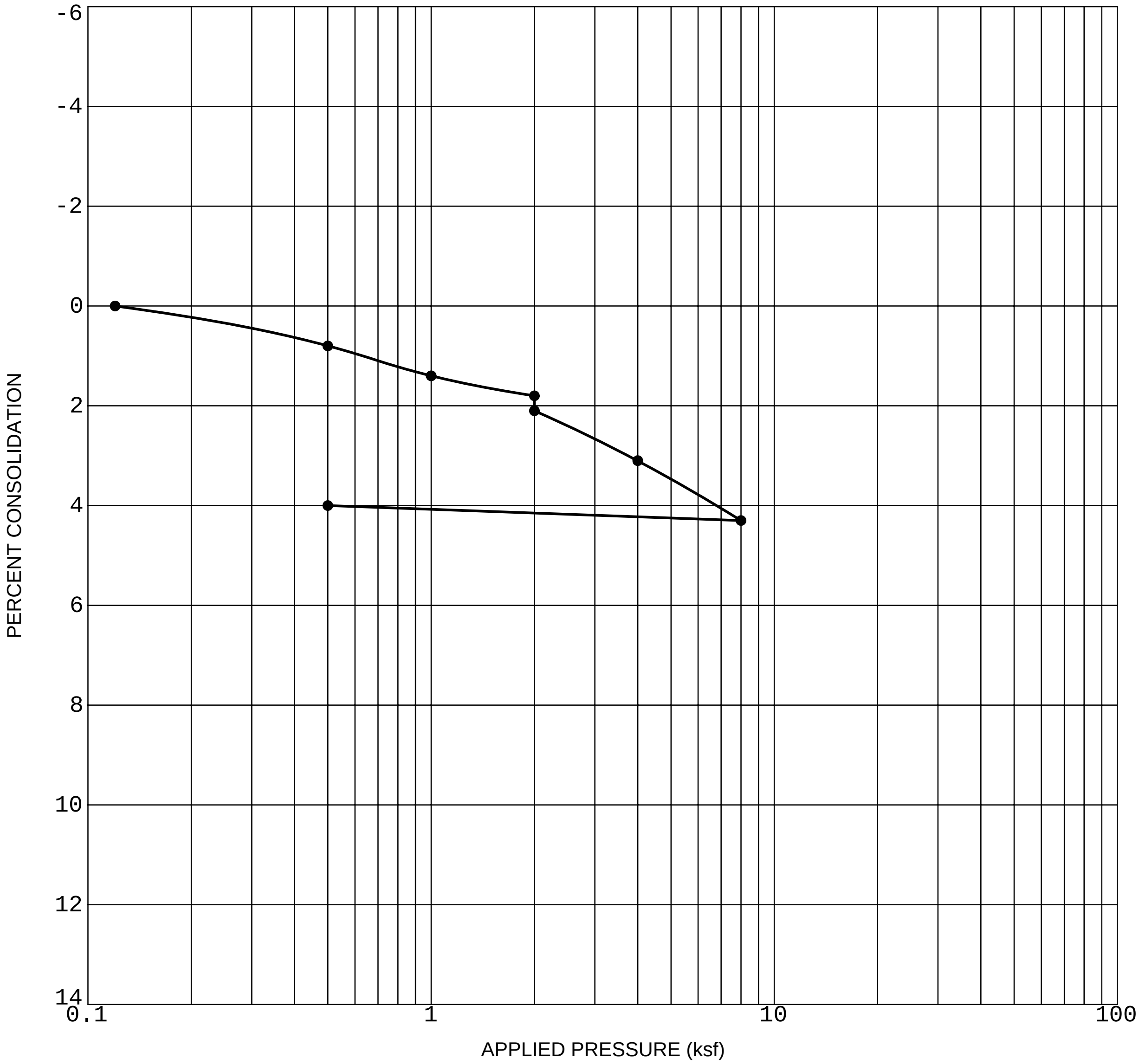
HAWANO
EAST OTAY PROPERTY
SAN DIEGO COUNTY, CALIFORNIA

G1223-52-01.GPJ

Figure B-3

GEOCON

SAMPLE NO. T20-2



Initial Dry Density (pcf)	94.0	**Initial Saturation (%)**	16.5
Initial Water Content (%)	4.8	**Sample Saturated at (ksf)**	2.0

CONSOLIDATION CURVE

HAWANO
EAST OTAY PROPERTY
SAN DIEGO COUNTY, CALIFORNIA

APPENDIX C

RECOMMENDED GRADING SPECIFICATIONS

FOR

HAWANO EAST OTAY PROPERTY
SAN DIEGO COUNTY, CALIFORNIA

PROJECT NO. G1223-52-01

RECOMMENDED GRADING SPECIFICATIONS

1. GENERAL

1.1 These Recommended Grading Specifications shall be used in conjunction with the Geotechnical Report for the project prepared by Geocon Incorporated. The recommendations contained in the text of the Geotechnical Report are a part of the earthwork and grading specifications and shall supersede the provisions contained hereinafter in the case of conflict.

1.2 Prior to the commencement of grading, a geotechnical consultant (Consultant) shall be employed for the purpose of observing earthwork procedures and testing the fills for substantial conformance with the recommendations of the Geotechnical Report and these specifications. The Consultant should provide adequate testing and observation services so that they may assess whether, in their opinion, the work was performed in substantial conformance with these specifications. It shall be the responsibility of the Contractor to assist the Consultant and keep them apprised of work schedules and changes so that personnel may be scheduled accordingly.

1.3 It shall be the sole responsibility of the Contractor to provide adequate equipment and methods to accomplish the work in accordance with applicable grading codes or agency ordinances, these specifications and the approved grading plans. If, in the opinion of the Consultant, unsatisfactory conditions such as questionable soil materials, poor moisture condition, inadequate compaction, adverse weather, result in a quality of work not in conformance with these specifications, the Consultant will be empowered to reject the work and recommend to the Owner that grading be stopped until the unacceptable conditions are corrected.

2. DEFINITIONS

2.1 **Owner** shall refer to the owner of the property or the entity on whose behalf the grading work is being performed and who has contracted with the Contractor to have grading performed.

2.2 **Contractor** shall refer to the Contractor performing the site grading work.

2.3 **Civil Engineer** or **Engineer of Work** shall refer to the California licensed Civil Engineer or consulting firm responsible for preparation of the grading plans, surveying and verifying as-graded topography.

2.4 **Consultant** shall refer to the soil engineering and engineering geology consulting firm retained to provide geotechnical services for the project.

2.5 **Soil Engineer** shall refer to a California licensed Civil Engineer retained by the Owner, who is experienced in the practice of geotechnical engineering. The Soil Engineer shall be responsible for having qualified representatives on-site to observe and test the Contractor's work for conformance with these specifications.

2.6 **Engineering Geologist** shall refer to a California licensed Engineering Geologist retained by the Owner to provide geologic observations and recommendations during the site grading.

2.7 **Geotechnical Report** shall refer to a soil report (including all addenda) which may include a geologic reconnaissance or geologic investigation that was prepared specifically for the development of the project for which these Recommended Grading Specifications are intended to apply.

3. MATERIALS

3.1 Materials for compacted fill shall consist of any soil excavated from the cut areas or imported to the site that, in the opinion of the Consultant, is suitable for use in construction of fills. In general, fill materials can be classified as *soil* fills, *soil-rock* fills or *rock* fills, as defined below.

3.1.1 **Soil fills** are defined as fills containing no rocks or hard lumps greater than 12 inches in maximum dimension and containing at least 40 percent by weight of material smaller than ¾ inch in size.

3.1.2 **Soil-rock fills** are defined as fills containing no rocks or hard lumps larger than 4 feet in maximum dimension and containing a sufficient matrix of soil fill to allow for proper compaction of soil fill around the rock fragments or hard lumps as specified in Paragraph 6.2. **Oversize rock** is defined as material greater than 12 inches.

3.1.3 **Rock fills** are defined as fills containing no rocks or hard lumps larger than 3 feet in maximum dimension and containing little or no fines. Fines are defined as material smaller than ¾ inch in maximum dimension. The quantity of fines shall be less than approximately 20 percent of the rock fill quantity.

3.2 Material of a perishable, spongy, or otherwise unsuitable nature as determined by the Consultant shall not be used in fills.

3.3 Materials used for fill, either imported or on-site, shall not contain hazardous materials as defined by the California Code of Regulations, Title 22, Division 4, Chapter 30, Articles 9 and 10; 40CFR; and any other applicable local, state or federal laws. The Consultant shall not be responsible for the identification or analysis of the potential presence of hazardous materials. However, if observations, odors or soil discoloration cause Consultant to suspect the presence of hazardous materials, the Consultant may request from the Owner the termination of grading operations within the affected area. Prior to resuming grading operations, the Owner shall provide a written report to the Consultant indicating that the suspected materials are not hazardous as defined by applicable laws and regulations.

3.4 The outer 15 feet of *soil-rock* fill slopes, measured horizontally, should be composed of properly compacted *soil* fill materials approved by the Consultant. *Rock* fill may extend to the slope face, provided that the slope is not steeper than 2:1 (horizontal:vertical) and a soil layer no thicker than 12 inches is track-walked onto the face for landscaping purposes. This procedure may be utilized provided it is acceptable to the governing agency, Owner and Consultant.

3.5 Samples of soil materials to be used for fill should be tested in the laboratory by the Consultant to determine the maximum density, optimum moisture content, and, where appropriate, shear strength, expansion, and gradation characteristics of the soil.

3.6 During grading, soil or groundwater conditions other than those identified in the Geotechnical Report may be encountered by the Contractor. The Consultant shall be notified immediately to evaluate the significance of the unanticipated condition

4. CLEARING AND PREPARING AREAS TO BE FILLED

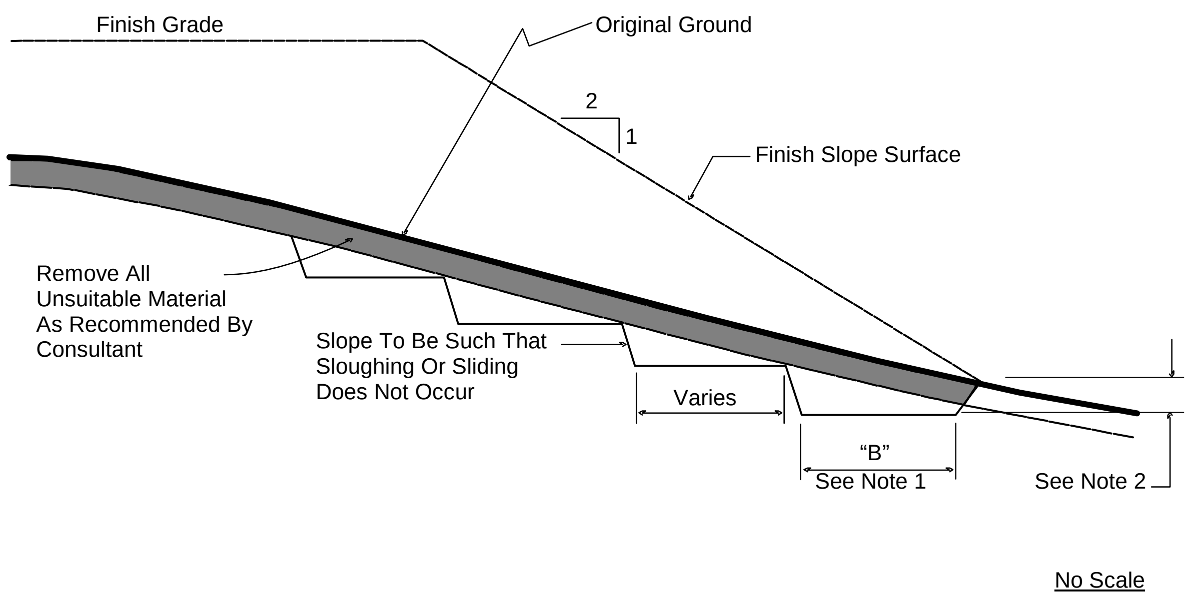
4.1 Areas to be excavated and filled shall be cleared and grubbed. Clearing shall consist of complete removal above the ground surface of trees, stumps, brush, vegetation, man-made structures, and similar debris. Grubbing shall consist of removal of stumps, roots, buried logs and other unsuitable material and shall be performed in areas to be graded. Roots and other projections exceeding 1½ inches in diameter shall be removed to a depth of 3 feet below the surface of the ground. Borrow areas shall be grubbed to the extent necessary to provide suitable fill materials.

4.2 Any asphalt pavement material removed during clearing operations should be properly disposed at an approved off-site facility. Concrete fragments that are free of reinforcing steel may be placed in fills, provided they are placed in accordance with Section 6.2 or 6.3 of this document.

4.3 After clearing and grubbing of organic matter and other unsuitable material, loose or porous soils shall be removed to the depth recommended in the Geotechnical Report. The depth of removal and compaction should be observed and approved by a representative of the Consultant. The exposed surface shall then be plowed or scarified to a minimum depth of 6 inches and until the surface is free from uneven features that would tend to prevent uniform compaction by the equipment to be used.

4.4 Where the slope ratio of the original ground is steeper than 5:1 (horizontal:vertical), or where recommended by the Consultant, the original ground should be benched in accordance with the following illustration.

TYPICAL BENCHING DETAIL



DETAIL NOTES: (1) Key width "B" should be a minimum of 10 feet, or sufficiently wide to permit complete coverage with the compaction equipment used. The base of the key should be graded horizontal, or inclined slightly into the natural slope.

(2) The outside of the key should be below the topsoil or unsuitable surficial material and at least 2 feet into dense formational material. Where hard rock is exposed in the bottom of the key, the depth and configuration of the key may be modified as approved by the Consultant.

4.5 After areas to receive fill have been cleared and scarified, the surface should be moisture conditioned to achieve the proper moisture content, and compacted as recommended in Section 6 of these specifications.

5. COMPACTION EQUIPMENT

5.1 Compaction of *soil* or *soil-rock* fill shall be accomplished by sheepsfoot or segmented-steel wheeled rollers, vibratory rollers, multiple-wheel pneumatic-tired rollers, or other types of acceptable compaction equipment. Equipment shall be of such a design that it will be capable of compacting the *soil* or *soil-rock* fill to the specified relative compaction at the specified moisture content.

5.2 Compaction of *rock* fills shall be performed in accordance with Section 6.3.

6. PLACING, SPREADING AND COMPACTION OF FILL MATERIAL

6.1 *Soil* fill, as defined in Paragraph 3.1.1, shall be placed by the Contractor in accordance with the following recommendations:

6.1.1 *Soil* fill shall be placed by the Contractor in layers that, when compacted, should generally not exceed 8 inches. Each layer shall be spread evenly and shall be thoroughly mixed during spreading to obtain uniformity of material and moisture in each layer. The entire fill shall be constructed as a unit in nearly level lifts. Rock materials greater than 12 inches in maximum dimension shall be placed in accordance with Section 6.2 or 6.3 of these specifications.

6.1.2 In general, the *soil* fill shall be compacted at a moisture content at or above the optimum moisture content as determined by ASTM D 1557-02.

6.1.3 When the moisture content of *soil* fill is below that specified by the Consultant, water shall be added by the Contractor until the moisture content is in the range specified.

6.1.4 When the moisture content of the *soil* fill is above the range specified by the Consultant or too wet to achieve proper compaction, the *soil* fill shall be aerated by the Contractor by blading/mixing, or other satisfactory methods until the moisture content is within the range specified.

6.1.5 After each layer has been placed, mixed, and spread evenly, it shall be thoroughly compacted by the Contractor to a relative compaction of at least 90 percent. Relative compaction is defined as the ratio (expressed in percent) of the in-place dry density of the compacted fill to the maximum laboratory dry density as determined in accordance with ASTM D 1557-02. Compaction shall be continuous over the entire area, and compaction equipment shall make sufficient passes so that the specified minimum relative compaction has been achieved throughout the entire fill.

6.1.6 Where practical, soils having an Expansion Index greater than 50 should be placed at least 3 feet below finish pad grade and should be compacted at a moisture content generally 2 to 4 percent greater than the optimum moisture content for the material.

6.1.7 Properly compacted *soil* fill shall extend to the design surface of fill slopes. To achieve proper compaction, it is recommended that fill slopes be over-built by at least 3 feet and then cut to the design grade. This procedure is considered preferable to track-walking of slopes, as described in the following paragraph.

6.1.8 As an alternative to over-building of slopes, slope faces may be back-rolled with a heavy-duty loaded sheepsfoot or vibratory roller at maximum 4-foot fill height intervals. Upon completion, slopes should then be track-walked with a D-8 dozer or similar equipment, such that a dozer track covers all slope surfaces at least twice.

6.2 *Soil-rock* fill, as defined in Paragraph 3.1.2, shall be placed by the Contractor in accordance with the following recommendations:

6.2.1 Rocks larger than 12 inches but less than 4 feet in maximum dimension may be incorporated into the compacted *soil* fill, but shall be limited to the area measured 15 feet minimum horizontally from the slope face and 5 feet below finish grade or 3 feet below the deepest utility, whichever is deeper.

6.2.2 Rocks or rock fragments up to 4 feet in maximum dimension may either be individually placed or placed in windrows. Under certain conditions, rocks or rock fragments up to 10 feet in maximum dimension may be placed using similar methods. The acceptability of placing rock materials greater than 4 feet in maximum dimension shall be evaluated during grading as specific cases arise and shall be approved by the Consultant prior to placement.

6.2.3 For individual placement, sufficient space shall be provided between rocks to allow for passage of compaction equipment.

6.2.4 For windrow placement, the rocks should be placed in trenches excavated in properly compacted *soil* fill. Trenches should be approximately 5 feet wide and 4 feet deep in maximum dimension. The voids around and beneath rocks should be filled with approved granular soil having a Sand Equivalent of 30 or greater and should be compacted by flooding. Windrows may also be placed utilizing an "open-face" method in lieu of the trench procedure, however, this method should first be approved by the Consultant.

6.2.5 Windrows should generally be parallel to each other and may be placed either parallel to or perpendicular to the face of the slope depending on the site geometry. The minimum horizontal spacing for windrows shall be 12 feet center-to-center with a 5-foot stagger or offset from lower courses to next overlying course. The minimum vertical spacing between windrow courses shall be 2 feet from the top of a lower windrow to the bottom of the next higher windrow.

6.2.6 Rock placement, fill placement and flooding of approved granular soil in the windrows should be continuously observed by the Consultant.

6.3 *Rock* fills, as defined in Section 3.1.3, shall be placed by the Contractor in accordance with the following recommendations:

6.3.1 The base of the *rock* fill shall be placed on a sloping surface (minimum slope of 2 percent). The surface shall slope toward suitable subdrainage outlet facilities. The *rock* fills shall be provided with subdrains during construction so that a hydrostatic pressure buildup does not develop. The subdrains shall be permanently connected to controlled drainage facilities to control post-construction infiltration of water.

6.3.2 *Rock* fills shall be placed in lifts not exceeding 3 feet. Placement shall be by rock trucks traversing previously placed lifts and dumping at the edge of the currently placed lift. Spreading of the *rock* fill shall be by dozer to facilitate *seating* of the rock. The *rock* fill shall be watered heavily during placement. Watering shall consist of water trucks traversing in front of the current rock lift face and spraying water continuously during rock placement. Compaction equipment with compactive energy comparable to or greater than that of a 20-ton steel vibratory roller or other compaction equipment providing suitable energy to achieve the required compaction or deflection as recommended in Paragraph 6.3.3 shall be utilized. The number of passes to be made should be determined as described in Paragraph 6.3.3. Once a *rock* fill lift has been covered with *soil* fill, no additional *rock* fill lifts will be permitted over the *soil* fill.

6.3.3 Plate bearing tests, in accordance with ASTM D 1196-93, may be performed in both the compacted *soil* fill and in the *rock* fill to aid in determining the required minimum number of passes of the compaction equipment. If performed, a minimum of three plate bearing tests should be performed in the properly compacted *soil* fill (minimum relative compaction of 90 percent). Plate bearing tests shall then be performed on areas of *rock* fill having two passes, four passes and six passes of the compaction equipment, respectively. The number of passes required for the *rock* fill shall be determined by comparing the results of the plate bearing tests for the *soil* fill and the *rock* fill and by evaluating the deflection variation with number of passes. The required number of passes of the compaction equipment will be performed as necessary until the plate bearing deflections are equal to or less than that determined for the properly compacted *soil* fill. In no case will the required number of passes be less than two.

6.3.4 A representative of the Consultant should be present during *rock* fill operations to observe that the minimum number of "passes" have been obtained, that water is being properly applied and that specified procedures are being followed. The actual number of plate bearing tests will be determined by the Consultant during grading.

6.3.5 Test pits shall be excavated by the Contractor so that the Consultant can state that, in their opinion, sufficient water is present and that voids between large rocks are properly filled with smaller rock material. In-place density testing will not be required in the *rock* fills.

6.3.6 To reduce the potential for "piping" of fines into the *rock* fill from overlying *soil* fill material, a 2-foot layer of graded filter material shall be placed above the uppermost lift of *rock* fill. The need to place graded filter material below the *rock* should be determined by the Consultant prior to commencing grading. The gradation of the graded filter material will be determined at the time the *rock* fill is being excavated. Materials typical of the *rock* fill should be submitted to the Consultant in a timely manner, to allow design of the graded filter prior to the commencement of *rock* fill placement.

6.3.7 *Rock* fill placement should be continuously observed during placement by the Consultant.

7. OBSERVATION AND TESTING

7.1 The Consultant shall be the Owner's representative to observe and perform tests during clearing, grubbing, filling, and compaction operations. In general, no more than 2 feet in vertical elevation of *soil* or *soil-rock* fill should be placed without at least one field density test being performed within that interval. In addition, a minimum of one field density test should be performed for every 2,000 cubic yards of *soil* or *soil-rock* fill placed and compacted.

7.2 The Consultant should perform a sufficient distribution of field density tests of the compacted *soil* or *soil-rock* fill to provide a basis for expressing an opinion whether the fill material is compacted as specified. Density tests shall be performed in the compacted materials below any disturbed surface. When these tests indicate that the density of any layer of fill or portion thereof is below that specified, the particular layer or areas represented by the test shall be reworked until the specified density has been achieved.

7.3 During placement of *rock* fill, the Consultant should observe that the minimum number of passes have been obtained per the criteria discussed in Section 6.3.3. The Consultant should request the excavation of observation pits and may perform plate bearing tests on the placed *rock* fills. The observation pits will be excavated to provide a basis for expressing an opinion as to whether the *rock* fill is properly seated and sufficient moisture has been applied to the material. When observations indicate that a layer of *rock* fill or any portion thereof is below that specified, the affected layer or area shall be reworked until the *rock* fill has been adequately seated and sufficient moisture applied.

7.4 A settlement monitoring program designed by the Consultant may be conducted in areas of *rock* fill placement. The specific design of the monitoring program shall be as recommended in the Conclusions and Recommendations section of the project Geotechnical Report or in the final report of testing and observation services performed during grading.

7.5 The Consultant should observe the placement of subdrains, to verify that the drainage devices have been placed and constructed in substantial conformance with project specifications.

7.6 Testing procedures shall conform to the following Standards as appropriate:

7.6.1 Soil and Soil-Rock Fills:

7.6.1.1 Field Density Test, ASTM D 1556-02, *Density of Soil In-Place By the Sand-Cone Method.*

7.6.1.2 Field Density Test, Nuclear Method, ASTM D 6938-08A, *Density of Soil and Soil-Aggregate In-Place by Nuclear Methods (Shallow Depth).*

7.6.1.3 Laboratory Compaction Test, ASTM D 1557-02, *Moisture-Density Relations of Soils and Soil-Aggregate Mixtures Using 10-Pound Hammer and 18-Inch Drop*.

7.6.1.4. Expansion Index Test, ASTM D 4829-03, *Expansion Index Test*.

7.6.2 Rock Fills

7.6.2.1 Field Plate Bearing Test, ASTM D 1196-93 (Reapproved 1997) *Standard Method for Nonreparative Static Plate Load Tests of Soils and Flexible Pavement Components, For Use in Evaluation and Design of Airport and Highway Pavements.*

8. PROTECTION OF WORK

8.1 During construction, the Contractor shall properly grade all excavated surfaces to provide positive drainage and prevent ponding of water. Drainage of surface water shall be controlled to avoid damage to adjoining properties or to finished work on the site. The Contractor shall take remedial measures to prevent erosion of freshly graded areas until such time as permanent drainage and erosion control features have been installed. Areas subjected to erosion or sedimentation shall be properly prepared in accordance with the Specifications prior to placing additional fill or structures.

8.2 After completion of grading as observed and tested by the Consultant, no further excavation or filling shall be conducted except in conjunction with the services of the Consultant.

9. CERTIFICATIONS AND FINAL REPORTS

9.1 Upon completion of the work, Contractor shall furnish Owner a certification by the Civil Engineer stating that the lots and/or building pads are graded to within 0.1 foot vertically of elevations shown on the grading plan and that all tops and toes of slopes are within 0.5 foot horizontally of the positions shown on the grading plans. After installation of a section of subdrain, the project Civil Engineer should survey its location and prepare an *as-built* plan of the subdrain location. The project Civil Engineer should verify the proper outlet for the subdrains and the Contractor should ensure that the drain system is free of obstructions.

9.2 The Owner is responsible for furnishing a final as-graded soil and geologic report satisfactory to the appropriate governing or accepting agencies. The as-graded report should be prepared and signed by a California licensed Civil Engineer experienced in geotechnical engineering and by a California Certified Engineering Geologist, indicating that the geotechnical aspects of the grading were performed in substantial conformance with the Specifications or approved changes to the Specifications.

LIST OF REFERENCES

1. American Geological Institute, 1982, *AGI Data Sheets for Geology in the Field, Laboratory, and Office*, second edition, Data Sheet 58.2.

2. Boore, D. M., and G. M Atkinson (2008), *Ground-Motion Prediction for the Average Horizontal Component of PGA, PGV, and 5%-Damped PSA at Spectral Periods Between 0.01 and 10.0 S*, Eartquake Spectra, Volume 24, Issue 1, pages 99-138, February 2008.

3. California Geologic Survey (CGS), Todd, 2004, *Geologic Map of the El Cajon 7.5′ Quadrangle, San Diego County, California: A Digital Database,* scale 1:24,000.

4. California Geological Survey (CGS), 2003, *Earthquake Shaking Potential for California*, from USGS/CGS Seismic Hazards Model, CSSC No. 03-02.

5. California Geological Survey (CGS), 2003, *Probabilistic Seismic Hazards Mapping-Ground Motion Page*, CGS Website: www.conserv.ca.gov/cgs/rghm/pshamap.

6. California Geological Survey, *Seismic Shaking Hazards in California,* Based on the USGS/*CGS Probabilistic* Seismic Hazards Assessment (PSHA) Model, 2002 (revised April 2003). 10% probability of being exceeded in 50 years; http://redirect.conservation.ca.gov/cgs/rghm/pshamap/pshamain.html.

7. Campbell, K. W. and Y. Bozorgnia, *NGA Ground Motion Model for the Geometric Mean Horizontal Component of PGA, PGV, PGD and 5% Damped Linear Elastic Response Spectra for Periods Ranging from 0.01 to 10 s*, Preprint of version submitted for publication in the NGA Special Volume of Earthquake Spectra, Volume 24, Issue 1, pages 139-171, February 2008.

8. Chiou, Brian S. J. and Robert R. Youngs, *A NGA Model for the Average Horizontal Component of Peak Ground Motion and Response Spectra*, preprint for article to be published in NGA Special Edition for Earthquake Spectra, Spring 2008.

9. Geocon Incorporated, September 15, 2004, *Final Report of Testing and Observation Services During Site Grading for Airway Truck Parking and Storage, San Diego County, California* (Project No. 07201-22-02).

10. Kennedy, M. P. and S. S. Tan, 2005, *Geologic Map of the San Diego 30’ x 60’ Quadrangle, California*, California Geologic Survey.

11. Ishihara, K. (1985), *Stability of Natural Deposits During Earthquakes*, Proceedings, 11th International Conference on Soil Mechanics and Foundation Engineering, San Francisco, pp. 321-376.

12. Jennings, C. W., revised 2010, *Fault Activity Map of California and Adjacent Areas*, California Geologic Data Map Series, Map No. 6.

13. Legg, M. R., J. C. Borrero, and C. E. Synolakis, 2002, *Evaluation of Tsunami Risk to Southern California Coastal Cities*, 2002 NEHRP Professional Fellowship Report, dated January.

14. Risk Engineering Company, *EZ-FRISK*, 2010.

LIST OF REFERENCES (Continued)

15. Treiman, Jerome A., 1993, *The Rose Canyon Fault Zone Southern California,* California Division of Mines and Geology Publication.

16. URS, 2004, *San Diego County Multi-Jurisdictional Hazard Mitigation Plan, San Diego County, California*, dated March 15, (URS Project No. 27653042.00500).

17. USGS computer program, Seismic Hazard Curves and Uniform Hazard Response Spectra, http://earthquake.usgs.gov/research/hazmaps/design/.

18. Unpublished reports, aerial photographs and maps on file with Geocon Incorporated.

19. Walsh, Stephen L. and Demere, Thomas A., 1991, *Age and Stratigraphy of the Sweetwater and Otay Formations, San Diego County California,* In Abbott,P. L. and May, J. A., eds., 1991, Eocene Geologic history San Diego Region, Pacific section SEPM, Vol. 68,p. 131-148.