

**GEO TECHNICAL INVESTIGATION
AND
GEOLOGIC FAULT INVESTIGATION**

**CEDAR/KETTNER
PARKING/RESIDENTIAL STRUCTURE
SAN DIEGO, CALIFORNIA**



GEOCON
INCORPORATED

GEOTECHNICAL
ENVIRONMENTAL
MATERIALS

PREPARED FOR

**COUNTY OF SAN DIEGO GENERAL SERVICES
SAN DIEGO, CALIFORNIA**

**SEPTEMBER 18, 2003
REVISED OCTOBER 14, 2003**



Project No. 06851-22-02
September 18, 2003
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County of San Diego General Services
5555 Overland Drive, Suite 2600
San Diego, California 92123

Attention: Mr. Jeff Redlitz

Subject: CEDAR/KETTNER PARKING/RESIDENTIAL STRUCTURE
SAN DIEGO, CALIFORNIA
GEOTECHNICAL INVESTIGATION
AND GEOLOGIC FAULT INVESTIGATION

Gentlemen:

In accordance with your authorization of our Proposal No. LG-03217 dated April 30, 2003 and revised July 11, 2003, we herewith submit the results of our geotechnical investigation and geologic fault investigation for the subject site. The accompanying report presents the findings from our study and our conclusions and recommendations pertaining to the geotechnical aspects of the proposed new development. The findings of this study indicate that no active faults traverse the property, and the site is suitable for development provided the recommendations of this report are followed.

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

Very truly yours,

GEOCON INCORPORATED

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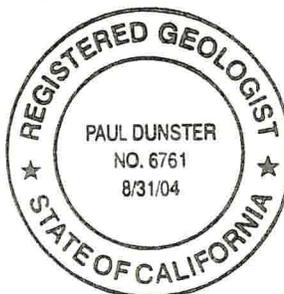


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GEOTECHNICAL INVESTIGATION AND GEOLOGIC FAULT INVESTIGATION

1. PURPOSE AND SCOPE

This report presents the results of a geotechnical investigation for the combined car parking and office structure that is proposed at the southwest corner of Cedar Street and Kettner Boulevard in San Diego, California. A vicinity map is included as Figure 1. The purpose of the geotechnical investigation was to evaluate the site's soil conditions and general site geology, and to identify geotechnical constraints (if any) that may impact development of the property. The faulting evaluation was performed to assess whether active faults traverse the property. The site is situated within the City of San Diego Downtown Special Fault Zone and required a detailed fault evaluation to satisfy the City of San Diego Building Department requirements.

The scope of this investigation included a review of stereoscopic aerial photographs and readily available published and unpublished geologic literature (see *List of References*). A field investigation was conducted that included drilling five borings to a maximum depth of 91 feet and excavating two trenches to a maximum depth of 14½ feet. The trenches were excavated to assess whether active faults traverse the property.

Laboratory tests were performed on selected soil samples obtained during the field investigation to determine pertinent physical properties for engineering analyses and to assist in providing recommendations for site grading and foundation design criteria. Details of the laboratory testing and a summary of the test results are presented in Appendix B.

Boring logs and the fault trench log are presented in Appendix A. Results of laboratory tests are presented on the boring logs in Appendix A and in tabular form in Appendix B. Recommended grading specifications are presented in Appendix C.

Figure 2, the Site Plan, depicts the approximate configuration of the property and the approximate locations of the borings and trenches.

2. SITE AND PROJECT DESCRIPTION

The site encompasses approximately 80 percent (approximately 48,750 square feet) of the block located at the southwest corner of the intersection of Cedar Street and Kettner Boulevard in the downtown area of San Diego, California. The property is used mainly as an at-grade parking lot. A building located in the southeast corner of the block will be demolished as part of the subject project.

Another building in the southwest corner of the block will remain and that portion of the block was not included in the investigation. We understand that a seven-story parking structure with three levels of subterranean parking is proposed for the site. A three-story office will wrap around the parking structure along the south, west and north sides.

Based upon the results of our geotechnical investigation and fault study, the site is underlain by fill and alluvial soils, which are in turn underlain by the Bay Point Formation and the San Diego Formation.

The above locations, site descriptions, and our understanding of the proposed development are based on a site reconnaissance, review of published geologic literature, our field investigations, and discussions with you. If development plans differ from those described herein, Geocon Incorporated should be contacted for review of the plans and possible revisions to this report.

3. SOIL AND GEOLOGIC CONDITIONS

Our field investigation encountered four geologic units: fill, alluvium, Bay Point Formation, and San Diego Formation. The occurrence and distribution of each unit, including a description of the unit, are shown on the boring logs in Appendix A and on the Site Plan, Figure 2. A geologic cross-section is presented on Figure 3. Each geologic unit is also described below.

3.1 Fill (Qaf)

Fill was encountered in two of the borings and both of the trenches. The fills encountered were up to 10 feet deep and consisted of loose to dense, dry to moist, silty and clayey sand with varying amounts of gravel and debris consisting of pieces of brick, glass and wood. Portions of the fill had a hydrocarbon odor. During the excavation of Trench 2 an accumulation of partially burned household refuse was encountered that included bottles, ash, wood, wire, and ceramics. The refuse was encapsulated in a cylindrical concrete structure. We expect that the fill will be removed during excavation for the proposed improvements. A pocket of buried refuse was also encountered in Trench 1 between approximate Stations 40+00 to 55+00. The location is not indicated on the trench log as we logged the north wall of the trench. The materials were only exposed on the south side of the trench.

3.2 Alluvium (Qal)

Alluvium was encountered in both trenches and consisted of loose, damp to moist, silty sand. Portions of this deposit may actually be highly weathered sections of the Bay Point Formation or

residual soil derived from the Bay Point Formation. It is expected that the alluvium will be removed during excavation for the proposed improvements.

3.3 Bay Point Formation (Qbp)

Pleistocene-age Bay Point Formation was observed in all of the borings and in the fault trenches. The Bay Point Formation typically consists of loose to dense silty and clayey sand that is partially cemented in places. Interbeds and lenses of rounded fine to coarse gravel and clay were also observed in the formation. Portions of this formation had a hydrocarbon odor. The Bay Point Formation is considered suitable for the support of the proposed structure.

3.4 San Diego Formation (Tsd)

Tertiary-age San Diego Formation was encountered in all of the borings at depths of between approximately 23 and 36 feet. The San Diego Formation typically consists of moist to saturated dense to very dense silty and clayey sand interbedded with stiff to hard clay, sandy clay, sandy silt, silt, and clay. Interbeds of gravel were also encountered in this formation.

4. GROUNDWATER

Groundwater was encountered in all of the borings at depths of between approximately 27½ and 34 feet below the existing ground surface. Dewatering will be required during construction of the subterranean levels. Waterproofing will also be necessary for that portion of the basement walls below groundwater levels. Uplift pressures on the structure and hydrostatic forces on the basement walls should be included in the design of the structures. Proper surface drainage of irrigation and rainwater will also be important to future performance of the project.

5. GEOLOGIC STRUCTURE

The geologic units described herein have nearly horizontal bedding attitudes with a slight apparent dip to the west as observed in the trenches. The regional dip is estimated to be approximately 3 to 5 degrees toward the south and west but can vary locally several degrees in other directions.

Review of the *City of San Diego Seismic Safety Study, Geologic Hazards and Faults*, 1995 edition, indicates that the site has a Geologic Hazard Category of 13 and is approximately ½ mile northwest of an area designated by the State of California as an *Alquist-Priolo Earthquake Fault Zone*. Category 13 is described as the Downtown Special Fault Zone and, as such, a fault evaluation was required for the property to assess the presence or absence of faults on the site.

6. GEOLOGIC HAZARDS

6.1 Faulting and Seismicity

The site is located near the southern onshore portion of the Rose Canyon Fault Zone in an area that is transitional between the predominately right-lateral faulting characteristic of the faults north of the downtown area and the predominantly dip slip faulting characteristic of faults making up the southern portion of the Rose Canyon Fault Zone (Treiman, 1993). South of the downtown area, the major faults that compose the southern end of the Rose Canyon Fault Zone are the Spanish Bight, Coronado and Silver Strand Faults. The east side of this zone is represented by the La Nacion Fault (Treiman, 1993). Together, these faults define a wide and complexly faulted basin occupied by San Diego Bay and a narrow section of the continental shelf west of the Silver Strand.

Trenching by Lindvall and others (1990) on the Rose Canyon Fault in Rose Canyon several miles north of the site by numerous consultants between approximately Twelfth Avenue and Seventeenth Street and by numerous consultants at sites near First Avenue in the downtown area have shown that Holocene soils (soils less than 11,000 years old or less) have been displaced by faulting within the Rose Canyon Fault Zone. The fault hazard study conducted for the subject property did not encounter evidence of faulting.

The historic seismicity or instrumental seismic record in the San Diego area indicates that there have been numerous minor earthquakes in the San Diego Bay area, including events in 1964 and 1985 between M3 and 4+ (Treiman, 1993). No surface rupture has been recorded with any of the seismic activity. Anderson and others (1989) indicate that the greatest peak acceleration recorded in the downtown area (at San Diego Light and Power) was 34 cm/sec² (0.03g) produced by an offshore earthquake in 1964 (M 5.6).

Anderson and others (1989) have also estimated recurrence times for major earthquakes that may affect the San Diego Region. By combining geologic data with their model for ground motion attenuation for each earthquake event, they have provided an estimation for the recurrence rate of various levels of peak ground acceleration in the San Diego area. The results of their work indicate that peak accelerations of 10 to 20 percent gravity (g) are expected approximately once every 100 years (Anderson and others, 1989). Higher peak accelerations will also occur but with a lower probability of occurrence or higher return period.

Lindvall and others (1991) have postulated a maximum likely slip rate of about 2 mm/yr and a best estimate of about 1.5 mm/yr, based on recent three-dimensional trenching on the Rose Canyon Fault in Rose Canyon several miles north of the site. They found stratigraphic evidence of at least three

events during the past 8,100 years. The most recent surface rupture displaces the modern A horizon (topsoil), suggesting that this event probably occurred within the past 500 years.

The nearest known active fault to the site is a strand of the Rose Canyon Fault Zone located approximately ½ mile southeast of the property. Several strands of the Rose Canyon Fault are located within Alquist-Priolo Earthquake Fault Zones located in the downtown area. Historically, the Rose Canyon Fault has exhibited low seismicity with respect to earthquakes in excess of magnitude 5.0 or greater. Earthquakes on the Rose Canyon Fault having a maximum magnitude of 6.9 are considered representative of the potential for seismic ground shaking within the property. The "maximum magnitude earthquake" is defined as the maximum earthquake that appears capable of occurring under the presently known tectonic framework.

Table 6.1 below presents a list of significant active faults, their distance from the site, and a summary of potential ground shaking effects. The information presented on Table 6.1 is derived from an analysis using *EQFAULT*, a computer program that performs deterministic analyses based upon distances from the site to known earthquake faults that have been digitized into an earthquake catalog. Attenuation relationships by Sadigh (1997) were used to estimate the maximum peak site accelerations.

**TABLE 6.1
DETERMINISTIC SITE PARAMETERS**

Fault Name	Approximate Distance From Site (miles)	Estimated Maximum Earthquake Magnitude	Estimated Peak Site Acceleration (g)
Rose Canyon Fault Zone	0.5	6.9	0.53
Coronado Bank	13	7.4	0.24
Newport Inglewood (Offshore)	34	6.9	0.07
Elsinore-Julian	42	7.1	0.06
Elsinore-Temecula	46	6.8	0.04
Earthquake Valley	47	6.5	0.03
Elsinore-Coyote Mountain	50	6.8	0.04
Palos Verdes	59	7.1	0.04

6.2 Probabilistic Seismic Hazard Analysis

The computer program FRISKSP (Blake, 1995, updated 1998) was used to perform a site-specific probabilistic seismic hazard analysis. The program is a modified version of FRISK (McGuire, 1978)

that models faults as lines to evaluate site-specific probabilities of exceedence of given horizontal accelerations for each line source. 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 mappable Quaternary fault is proportional to the fault's 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 closest 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 all considered earthquake sources, the program calculates the total average annual expected number of occurrences of a site acceleration greater than a specified value. Attenuation relationships suggested by Sadigh *et al.* (1997) were utilized in the analysis. The results of the analysis indicate that for a 10 percent probability of exceedence in 50 years, a mean site acceleration of 0.33 g may be generated. This value corresponds to a return period of approximately 475 years. For a 10 percent probability of exceedence in 100 years (949-year return period), a mean site acceleration of 0.45 g may be generated.

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. We recommend that the seismic design of the structures be performed in accordance with the California Building Code (CBC) guidelines currently adopted by the City of San Diego.

6.3 On-Site Faulting Evaluation

Prior to our field investigation, we reviewed aerial photographs to assist in our evaluation of geomorphic features that could be indicative of faulting at the property prior to site development. It was found that the earliest available aerial photos (1929) did not pre-date site development. Therefore, the photos were not useful in our evaluation.

A 5,400-foot-long sewer trench varying from 6 to 22 feet deep was excavated by a private contractor in 1980 and traversed the downtown area from east to west. In general, the alignment extended from Kettner Boulevard eastward to Twelfth Avenue along Broadway and E Street. The trench was logged and the results submitted to the San Diego Association of Geologists Field Trip Guide in April 1982 (Strieff, Elder-Mills, Artim, 1982). This trench did not encounter faults that would extend through the site, assuming typical fault trends in the downtown area.

Trenching to assess whether faults traverse the property was performed to a maximum depth of 14.5 feet at the two locations indicated on Figure 2. The logs of the trenches are included as Figures A-6 through A-9. Faulting in the southern portion of the Rose Canyon Fault Zone, which includes the downtown area, is predominately dip slip (Treiman, 1993). Therefore, relatively large offsets and discordance in the stratigraphy would be expected if active faulting is present. No Holocene deposits were encountered in the borings for use in correlation at the site. Stratigraphic correlation within the trenches indicates that the relative position of the three units within the Bay Point Formation appear to be continuous. In addition, the two clay beds observed in Trench 2 and the gravel bed observed in Trench 1 appear to be continuous. No indications of faulting, such as discordant bedding, clay gouge, shearing, or slickensides, were observed in the fault trenches or samples obtained from our borings.

A report entitled *Fault Hazard Investigation Proposed Park at Little Italy Project, San Diego, California*, prepared by Construction Testing & Engineering, Inc. (CTE), dated November 7, 2001 discussed and described a fault that was encountered in foundation excavations for the building located immediately north of the subject site. CTE concluded that the fault can be considered “potentially active”. We deepened our trenches in the area where faulting was expected, but no evidence of faulting was observed.

6.4 Liquefaction

The potential for liquefaction of the site soils during a strong earthquake is limited to those soils that are in a relatively loose, unconsolidated condition and are located below the groundwater table. Since the underlying formations are very dense, the potential for liquefaction occurring at the site is considered to be very low.

6.5 Landslides

Examination of aerial photographs in our files and review of available geotechnical reports for the site vicinity indicate that no landslides are present at the property or at a location that could impact the subject site.

7. CONCLUSIONS AND RECOMMENDATIONS

7.1 General

- 7.1.1 The site is not located within a currently established Alquist-Priolo Earthquake Fault Zone but is located within a fault study zone established by the City of San Diego. Our investigation was performed in compliance with the City of San Diego Building Department and the *City of San Diego Seismic Safety Study, Geologic Hazards and Faults*, 1995.
- 7.1.2 No evidence of faulting was observed in the Pleistocene-age Bay Point Formation encountered during our field investigation. Accordingly, the potential for surface rupture due to faulting in the area of the proposed development is considered very low.
- 7.1.3 With the exception of possible strong seismic shaking, no significant geologic hazards were observed or are known to exist on the site that would adversely affect the proposed project. No special seismic design considerations, other than those recommended herein, are required.
- 7.1.4 From a geotechnical standpoint, it is our opinion that the site is suitable for the proposed development provided the recommendations presented herein are implemented in design and construction of the project.
- 7.1.5 Our field investigation indicates that the site is underlain by fill, alluvium, Bay Point Formation, and San Diego Formation. The fill and alluvium are expected to be completely removed during excavations for the proposed structure. However, any existing fill soils encountered beyond the planned excavation limits will not be considered suitable in their present condition for support of settlement-sensitive structures.
- 7.1.6 Groundwater was encountered in the borings at depths of 27½ to 34 feet during this investigation. Groundwater levels in the vicinity of San Diego Bay will typically be relatively constant at an elevation of approximately 3 to 4 feet MSL. Uplift forces and hydrostatic pressure should be included in design of the structure slab and basement walls.
- 7.1.7 The proposed structure can be supported on conventional shallow footings or a mat foundation founded in formational materials.

7.1.8 Excavations for subterranean parking are estimated to be on the order of 35 feet deep. Parameters for temporary excavations and temporary shoring are presented in subsequent sections of this report.

7.2 Excavation and Soil Characteristics

7.2.1 The majority of the soils that will likely be encountered are considered to have a “very low” to “high” expansion potential (Expansion Index [EI] of between 0 and 130) as defined by Uniform Building Code (UBC) Table No. 18-I-B. A “high” expansion potential layer was encountered at the elevation of the bottom of the proposed structure, but no moisture variation is expected in this layer.

7.2.2 The majority of in situ soils can be excavated with moderate to heavy effort using conventional heavy-duty equipment. Cobbles and concretions are not uncommon within the formational soils and when encountered can create excavation difficulties. Additionally, chunks of concrete and other debris should be anticipated within the artificial fill.

7.2.3 It is the responsibility of the contractor to ensure that all excavations and trenches are properly shored and maintained in accordance with applicable OSHA rules and regulations in order to maintain safety and maintain the stability of adjacent existing improvements.

7.3 Corrosive Potential

7.3.1 Potential of Hydrogen (pH) and resistivity tests were performed on a representative sample of the site materials to evaluate the corrosion potential to subsurface structures. The tests were performed in accordance with California Test Method No. 643 and indicate that a “corrosive” condition may exist with respect to buried metals. The results are presented in Appendix B and should be considered for the design of underground structures.

7.3.2. Laboratory tests were performed on a representative sample of the site materials to determine the percentage of soluble sulfate. Results from the laboratory soluble-sulfate tests are presented in Appendix B and indicate that the on-site materials possess “negligible” sulfate exposure to concrete structures as defined by CBC Table 19-A-3. Based on the laboratory data, concrete design requirements are not set forth in CBC when concrete is exposed to a “negligible” amount of sulfate.

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

7.4 Seismic Design Criteria

7.4.1 Table 7.4 summarizes site-specific design criteria obtained from the 2000 CBC. The values listed are for the Rose Canyon Fault, which is identified as the nearest Type B fault and is more dominant than the nearest Type A fault due to its proximity to the site. A strand of the Rose Canyon Fault is located approximately 0.2 miles from the site.

**TABLE 7.4
SEISMIC DESIGN PARAMETERS**

Parameter	Value	UBC Reference
Seismic Zone Factor	0.40	Table 16-I
Soil Profile Type	S_D	Table 16-J
Seismic Coefficient, C_a	0.57	Table 16-Q
Seismic Coefficient C_v	1.02	Table 16-R
Near Source Factor, N_a	1.3	Table 16-S
Near Source Factor N_v	1.6	Table 16T
Seismic Source	B	Table 16-U

7.4.2 Several site-specific response spectra are presented on Figure 4, including the response spectrum generated by the UBC code, two deterministic response spectra for mean and mean plus one standard deviation, and two probabilistic design response spectra for a return period of 475 years and 949 years. The appropriate spectrum for structural design should be selected by the project structural engineer.

7.5 Grading

7.5.1 Grading should be performed in accordance with the *Recommended Grading Specifications* in Appendix C. Where the recommendations of this report conflict with Appendix C, the recommendations of this section take precedence.

7.5.2 Earthwork should be observed and compacted fill tested by representatives of Geocon Incorporated.

- 7.5.3 A pre-construction conference with the owner, contractor, civil engineer, and soil engineer in attendance should be held at the site prior to the beginning of export or shoring operations. Special soil handling requirements can be discussed at that time.
- 7.5.4 Grading of the site should commence with the removal of all existing improvements from the areas to be graded. Deleterious debris and contaminated soils should be exported from the site and should not be mixed with the fill soils. All existing underground improvements within the proposed building areas should be removed and the resulting depressions properly backfilled in accordance with the procedures described herein.
- 7.5.5 We anticipate that all existing fill and alluvium will be removed during excavation for the subterranean parking structure. If fill soils remain within areas to receive settlement-sensitive structures or structural fill, these soils should be removed to expose the underlying formational materials.
- 7.5.6 In areas to receive fill the ground surface should be scarified to a depth of at least 6 inches, moisture conditioned, and compacted to at least 90 percent of the laboratory maximum dry density at near optimum moisture content, as determined by ASTM Test Method D 1557-00.
- 7.5.7 Excavated soils free of deleterious debris can be placed as fill and compacted in layers to the design finish grade elevations. All fill and backfill soils should be placed in horizontal loose layers approximately 8 inches thick, moisture conditioned to a moisture content at or slightly above optimum, and compacted to at least 90 percent relative compaction, as determined by ASTM Test Method D 1557-00. The upper 12 inches of fill beneath pavement should be moisture conditioned and compacted to at least 95 percent relative compaction.
- 7.5.8 Import fill soil should consist of granular materials with a “low” expansion potential (EI less than 50) free of deleterious material or stones larger than 3 inches and should be compacted as recommended above. 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 determine its suitability as fill material.

7.6 Construction Dewatering

- 7.6.1 As indicated previously, because of the presence of groundwater at a depth of approximately 30 feet, we anticipate that dewatering will be performed during

construction. The dewatering scheme likely will include pumping of the groundwater from wellpoints. The wellpoint system design should be evaluated by the specialty dewatering contractor.

7.6.2 Discharge of water from excavations will require securing NPDES or other applicable permits. Compliance with the permit requirements may require testing and treatment of the water prior to discharge to sewers or storm drains.

7.6.3 Dewatering for construction will affect the water level outside of the excavation. This will result in an increase of effective stresses and could induce settlement of soils underlying adjacent areas. Lateral movement of shoring can also induce settlement. Due to the dense nature of the underlying soils, settlement of adjacent and nearby structures as a result of dewatering and shoring is expected to be limited. However, we recommend that the existing condition of adjacent improvements be documented with photography, video recordings and/or survey prior to construction, and also monitored during construction.

7.7 Excavation Slopes, Shoring, and Tiebacks

7.7.1 Deep excavations and cuts can often result in settlement of the surrounding ground surface. This settlement may be sufficient to cause damage or distress to buildings, retaining walls, utilities, services, or other structures located near the excavation.

7.7.2 Temporary slopes should be made in conformance with OSHA requirements. The formational units can be considered a Type A soil. Fill soils and alluvium should be considered Type B soil. In general, no special shoring requirement will be necessary if temporary excavations will be less than 5 feet in height. Temporary excavations greater than 5 feet in height, however, should be laid back with a $\frac{3}{4}$:1 (horizontal:vertical) gradient in formational material or 1:1 in fill soils. These excavations should not become saturated or allowed to dry out. Surcharge loads should not be permitted within a distance equal to the height of the excavation from the top of the excavation. The top of the excavation should be a minimum of 15 feet from the edge of existing improvements. Excavations steeper than those recommended or closer than 15 feet from an existing surface improvement should be temporarily shored in accordance with applicable OSHA codes and regulations. Alternatively, a permanent or temporary soil nail wall can be constructed as the excavation proceeds. Recommendations for a soil nail wall are presented in subsequent sections of this report.

- 7.7.3 The design of temporary shoring is governed by soil and groundwater conditions, and by the depth and width of the excavated area. Continuous support of the excavation face can be provided by a system of soldier piles and wood lagging. Excavations exceeding 15 feet may require tieback anchors to provide additional wall restraint.
- 7.7.4 Temporary cantilevered shoring can be designed for an active soil pressure equivalent to the pressure exerted by a fluid density of 30 pcf. Temporary tied-back shoring should be designed using a lateral pressure envelope acting uniformly on the back of the shoring and applying a pressure equal to $19H$, where H is the height of the shoring in feet (resulting pressure in pounds per square foot). Also, lateral earth pressure due to the surcharging effects of adjacent structures or traffic loads should be considered where appropriate during design of the shoring system (see Figure 4). Lateral loads due to adjacent footings should also be included (see Figure 5).
- 7.7.5 Passive soil pressure resistance for embedded portions of soldier piles can be based upon an equivalent passive soil fluid weight of $375+400D$, where D is the depth of embedment in feet (resulting in pounds per square foot), as shown in Figure 6. The passive resistance can be assumed to act over a width of three pile diameters. We recommend that cantilevered soldier piles without tiebacks be embedded a minimum of 0.5 times the maximum height of the excavation (this depth is to include footing excavations). The project structural engineer should determine the actual embedment depth.
- 7.7.6 Lateral movement of shoring is associated with vertical ground settlement outside of the excavation. Therefore, it is essential that the soldier pile and tieback system allow very limited amounts of lateral displacement. Earth pressures acting on a lagging wall can result in the movement of the shoring toward the excavation and result in ground subsidence outside of the excavation. Therefore, we recommend that horizontal movements of the shoring wall be accurately monitored and recorded during excavation and anchor construction. Survey points should be established at both the top and at least one intermediate point between the top of the pile and the base of the excavation on 20 percent of the soldier piles. These points should be monitored on a regular basis during excavation. The shoring system should be designed to limit horizontal soldier pile movement where adjacent improvement could be affected.
- 7.7.7 Tieback anchors employed in shoring should be designed such that anchors fully penetrate the active zone behind the shoring. The active zone can be considered the wedge of soil from the face of the shoring to a plane extending upward from the base of the excavation at a 28 degree angle from vertical, as shown on Figure 7. Normally, tieback anchors are

contractor-designed and installed, and there are numerous anchor construction methods available. Experience has shown that the use of pressure grouting during formation of the bonded portion of the anchor will decrease the probability of anchor failure.

- 7.7.8 Anchor capacity is a function of construction method, workmanship, depth of anchor, batter, diameter of the bonded section, and the length of the bonded section. A factor of safety of 2.5 to 3 is common for the design of a tieback system. The following soil strength parameters can be used in design of the temporary shoring:

Formational Materials

Cohesion = 350 psf

Friction Angle = 34 degrees

Surficial Soils

Cohesion = 100 psf

Friction Angle = 30 degrees

- 7.7.9 All anchors should be proof tested to at least 130 percent of the anchor's design working load. Following a successful proof test, it is recommended that anchors be locked off at 80 percent of the anchor's allowable working load. Anchor test failure criteria should be established in project plans and specifications. Any anchor test failure criteria should be based upon a maximum allowable displacement at 130 percent of the anchor's working load (anchor creep) and a maximum residual displacement within the anchor following stressing. Anchor stressing should only be conducted after sufficient hydration has occurred within the anchor grout. Anchors that fail to meet project-specified test criteria should be replaced.

- 7.7.10 Lagging should keep pace with excavation and anchor construction. We recommend that the excavation not be advanced deeper than 3 feet below the bottom of lagging at any time. These unlagged gaps of up to 3 feet should only be allowed to stand for short periods of time in order to decrease the probability of soil sloughing and caving. Backfilling should be conducted when necessary between the back of lagging and excavation sidewalls to reduce sloughing in this zone. Furthermore, the excavation should not be advanced further than 4 feet below a row of tiebacks prior to those tiebacks being proof tested and locked off.

- 7.7.11 If tieback anchors are employed, we recommend that an accurate survey of existing utilities (and other underground structures) adjacent to the shoring wall be conducted. The survey should include both locations and depths of existing utilities. Locations of anchors should be adjusted as necessary during the design and construction process to accommodate existing and proposed utilities.

7.7.12 The condition of existing buildings, streets, sidewalks, and other structures around the perimeter of the planned excavation should be documented prior to the start of shoring and excavation work. Special attention should be given to documenting existing cracks or other indications of differential settlement within these adjacent structures, pavements and other improvements. Any underground utilities sensitive to settlement should be videotaped prior to construction to verify integrity of pipes. In addition, monitoring points should be established indicating location and elevation around the excavation and upon existing buildings. These points should be monitored on a regular basis during construction. Inclometers should be installed and monitored behind any shoring sections that will be advanced deeper than 30 feet below the existing ground surface.

7.7.13 Tiebacks should be abandoned in accordance with the requirements of the City of San Diego.

7.8 Soil Nail Wall

7.8.1 As an alternative to temporary shoring followed by construction of a permanent basement wall, a permanent or temporary soil nail wall can be used. Soil nail walls consist of installing closely spaced steel bars (nails) into a slope or excavation in a top-down construction sequence. For permanent application following installation of a horizontal row of nails, drains, waterproofing, and wall reinforcing steel are placed and shotcrete applied to create a final wall.

7.8.2 The wall should be designed by an engineer familiar with the design of soil nail walls.

7.8.3 In general, ground conditions are moderately suited to soil nail construction techniques. However, clean sands may be encountered within the San Diego Formation that may result in some raveling of the unsupported excavation. No caving of boreholes was observed during our investigation because drilling mud prevented caving.

7.8.4 If the soil nail wall will be a permanent structure, a wall drain system should be incorporated into the design.

7.8.5 The existing soils were found to be corrosive. Corrosion protection should be provided for the nails in a permanent application.

7.8.6 Verification testing should be performed to confirm design assumptions. Testing should include pullout tests to obtain actual bond stress values. Approximately five percent of the

soil nails should also be proof tested. Testing and observation of nail installation, grout strength, shotcrete strength, and nail testing should be performed by a representative of Geocon Incorporated.

7.8.7 The following soil strength parameters can be used in design of the soil nails:

Formational Materials

Cohesion = 350 psf

Friction Angle = 34 degrees

Ultimate Bond Stress = 10 psi

7.9 Foundations

7.9.1 We expect that the buildings will be supported on conventional spread footings or a mat foundation located approximately 35 feet below the existing ground surface. Minor structures outside of the building may also be supported on a conventional foundation system founded in properly compacted fill or formational material. Conventional foundations consisting of continuous strip footings and/or isolated spread footings should be designed in accordance with recommendations below.

7.9.2 Continuous footings should be at least 12 inches wide and should extend at least 18 inches below lowest adjacent pad grade. Isolated spread footings should be at least 24 inches wide and should extend at least 18 inches below lowest adjacent pad grade. Steel reinforcement for continuous footings should consist of at least four No. 4 steel reinforcing bars placed horizontally in the footings, two near the top and two near the bottom. The steel reinforcement for spread footings should be designed by the project structural engineer. A footing dimension detail is presented on Figure 8.

7.9.3 The recommended allowable bearing capacity for foundations founded in formational materials is 6,000 psf. This allowable soil bearing pressure may be further increased by an additional 800 psf for each additional foot of depth and 400 psf for each additional foot of width to a maximum bearing capacity of 9,000 psf. The allowable bearing capacity for footings founded in properly compacted fill is 2,500 psf. 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.

7.9.4 Foundation excavations should be observed by the geotechnical engineer (a representative of Geocon Incorporated) prior to the placement of reinforcing steel and concrete to verify

that the exposed soil conditions are consistent with those anticipated and that they have been extended to the appropriate bearing strata. If unanticipated soil conditions are encountered, foundation modifications may be required.

7.10 Mat Foundation Recommendations

- 7.10.1 The core of the proposed structure may be supported on a mat foundation. A mat foundation consists of a thick rigid concrete mat that allows the entire footprint of the structure to carry building loads. In addition, the mat can tolerate significantly greater differential movements such as those associated with very large loads.
- 7.10.2 The allowable bearing capacity can be taken as 9,000 pounds per square foot (psf). This bearing capacity includes a factor of safety of at least 3. The modulus of subgrade reaction for design of the mat can be taken as 125 to 175 pounds per cubic inch (pci) for the formational soils. Anticipated total and differential settlements are estimated to be ½ inch and ¼ inch, respectively, under static loads.
- 7.10.3 Foundation excavations should be observed by the Geotechnical Engineer (a representative of Geocon Incorporated) prior to the placement of reinforcing steel and concrete to verify that the exposed soil conditions are consistent with those anticipated and have been extended to appropriate bearing strata. If unanticipated soil conditions are encountered, foundation modifications may be required.

7.11 Concrete Slabs

- 7.11.1 All exterior concrete flatwork not subject to vehicular traffic should be constructed in accordance with the following recommendations. Slab panels should be a minimum of 4 inches thick and, when in excess of 8-foot square, should be reinforced with 6 x 6 - W2.9/W2.9 (6 x 6 - 6/6) welded wire mesh to reduce the potential for cracking. In addition, all 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 soils 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 soils should be properly compacted and the moisture content of surficial soils should be verified prior to placing concrete.

7.11.2 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 and construction and curing practices and should be incorporated into project construction.

7.12 Lateral Loading

7.12.1 To resist lateral loads, a passive pressure exerted by an equivalent fluid weight of 300 pounds per cubic foot (pcf) and 400 pcf should be used for design of footings or shear keys poured neat against properly compacted granular fill soils or formational materials, respectively. The upper 12 inches of material in areas not protected by floor slabs or pavement should not be included in design for passive resistance.

7.12.2 If friction is to be used to resist lateral loads, an allowable coefficient of friction between soil and concrete of 0.35 should be used for design.

7.13 Retaining Walls

7.13.1 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 35 pcf. For walls supporting formational materials, the value can be reduced to 30 pcf. Where the backfill will be inclined at 2:1 (horizontal:vertical), an active soil pressure of 50 pcf is recommended. Highly expansive soils should not be used as backfill material behind retaining walls. All soil placed for retaining wall backfill should have an Expansion Index less than 50.

7.13.2 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) at the top of the wall. Where walls are restrained from movement at the top, an additional uniform pressure of $10H$ psf should be added to the above active soil pressure. For retaining walls subject to vehicular loads within a horizontal distance equal to two-thirds the wall height, a surcharge equivalent to

2 feet of fill soil should be added. Hydrostatic pressure should be added for the portion of basement walls below elevation 3 feet MSL.

- 7.13.3 Retaining walls above groundwater should be provided with a drainage system adequate to prevent the buildup of hydrostatic forces and should be waterproofed as required by the project architect. 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 above recommendations assume a properly compacted granular (EI less than 50) 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 anticipated, or if specific drainage details are desired, Geocon Incorporated should be contacted for additional recommendations.

7.14 Site Drainage and Moisture Protection

- 7.14.1 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 and 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.

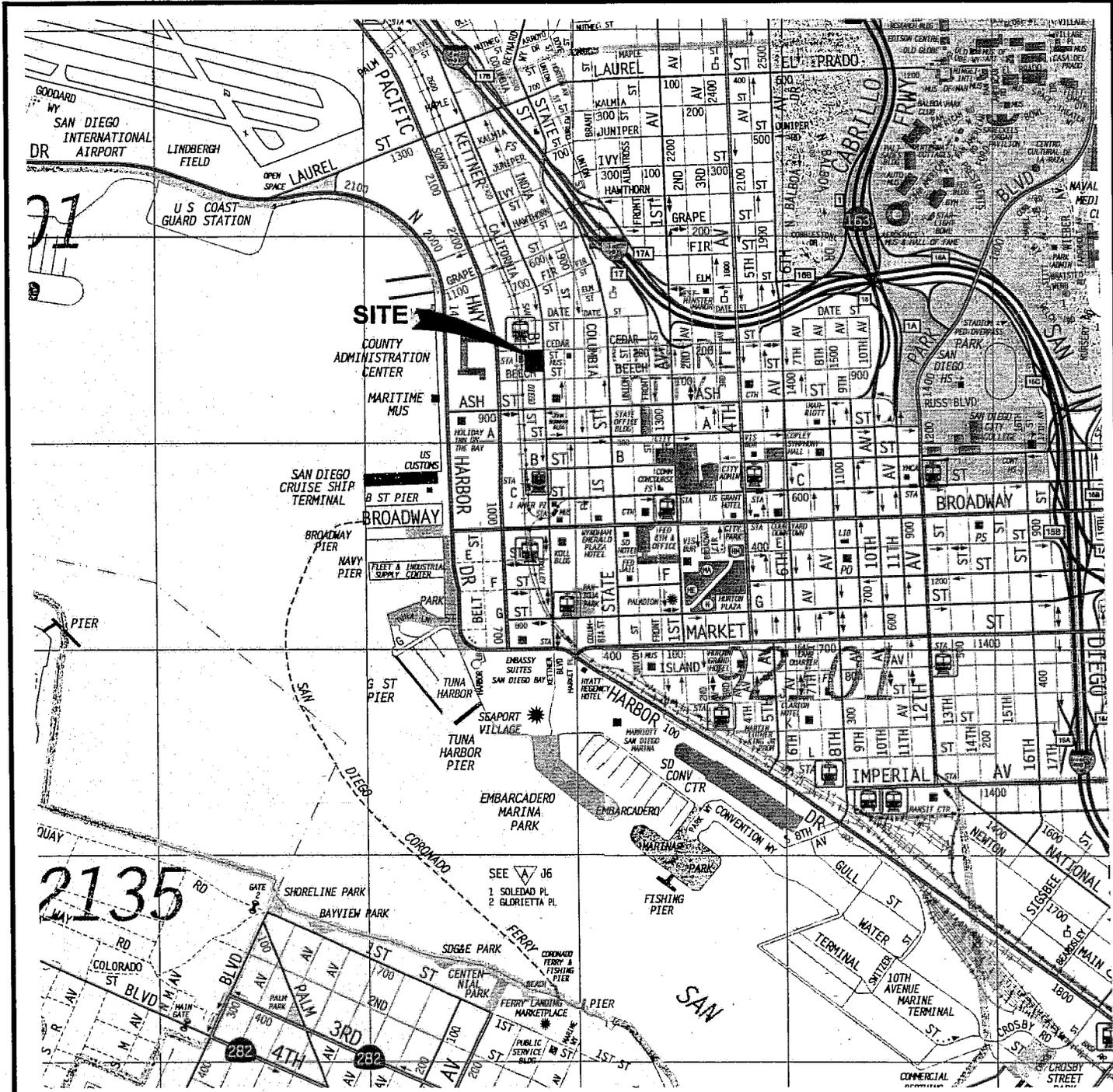
- 7.14.2 Landscaping planters adjacent to paved areas are not recommended due to the potential for surface or irrigation water to infiltrate the pavement's subgrade and base course. We recommend that subdrains to collect excess irrigation water and transmit it to drainage structures or impervious above-grade 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.

7.15 Foundation Plan Review

- 7.15.1 Geocon Incorporated should review the grading plans and foundation plans for the project prior to final design submittal to determine 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 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 that 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 be 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.



SOURCE : 2003 THOMAS BROTHERS MAP
SAN DIEGO COUNTY, CALIFORNIA

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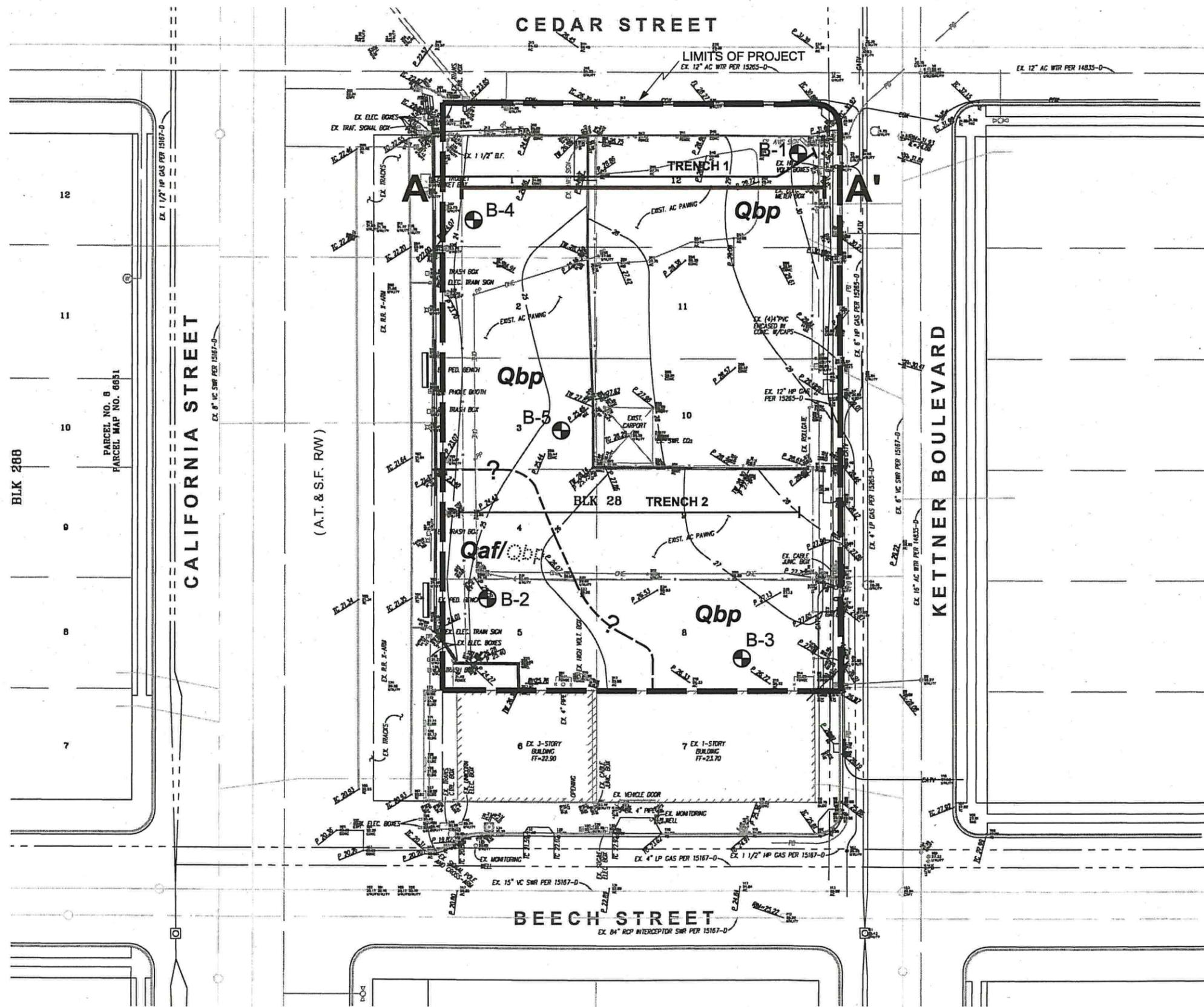
VICINITY MAP

CEDAR/KETTNER PARKING/RESIDENTIAL STRUCTURE
SAN DIEGO, CALIFORNIA

PD / RSS	DSK / G000
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DATE 10-14-2003	PROJECT NO. 06851 - 22 - 02	FIG. 1
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CEDAR / KETTNER
PARKING / RESIDENTIAL STRUCTURE
SAN DIEGO, CALIFORNIA



SCALE : 1" = 50'

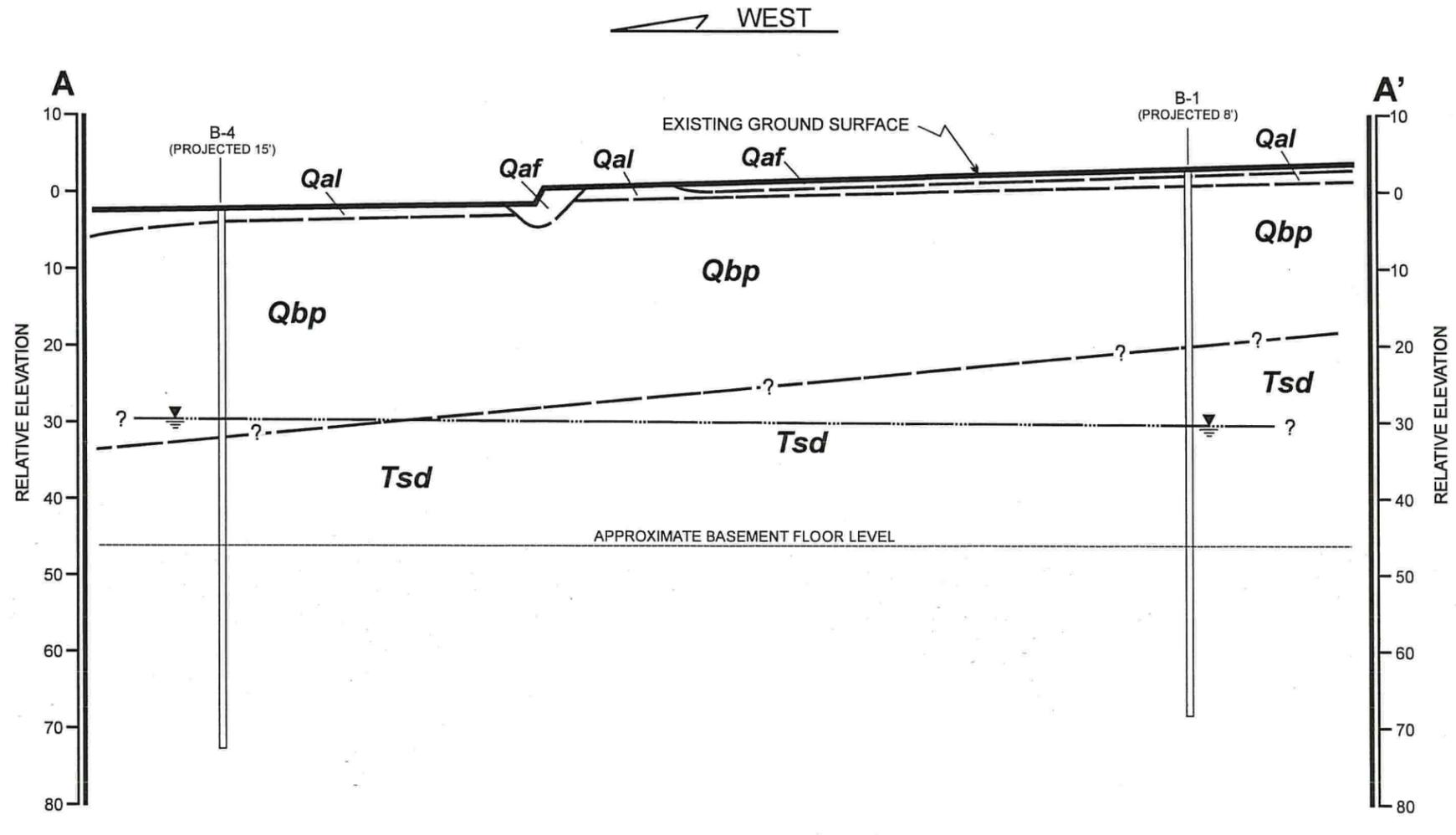
GEOCON LEGEND

- B-5APPROX. LOCATION OF BORING
- Qaf**FILL
- Qbp**BAY POINT FORMATION (Dotted Where buried)
- A A'APPROX. LOCATION OF CROSS - SECTION
-APPROX. LOCATION OF TRENCH

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FIGURE 2
DATE 10 - 14 - 2003

SITE PLAN

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 PARKING / RESIDENTIAL STRUCTURE
 SAN DIEGO, CALIFORNIA



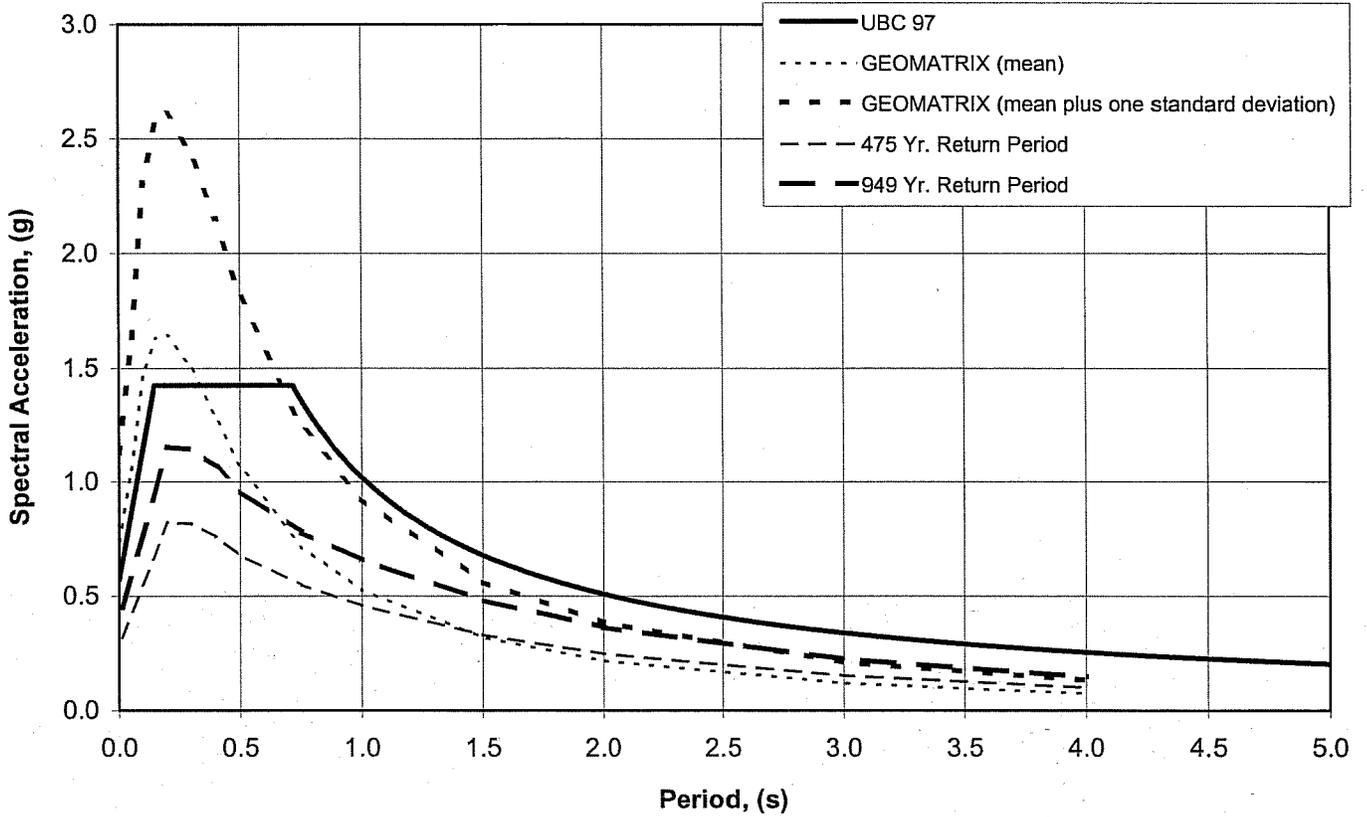
GEOLOGIC CROSS-SECTION

SCALE : 1" = 20' (HORIZ. = VERT.)

- LEGEND
- Qaf*.....FILL
 - Qal*.....ALLUVIUM
 - Qbp*.....BAY POINT FORMATION
 - Tsd*.....SAN DIEGO FORMATION
 -APPROX. LOCATION OF GEOLOGIC CONTACT
 -APPROX. GROUNDWATER LEVEL

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 FIGURE 3
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KC / KC

DESIGN RESPONSE SPECTRA

CEDAR / KETTNER

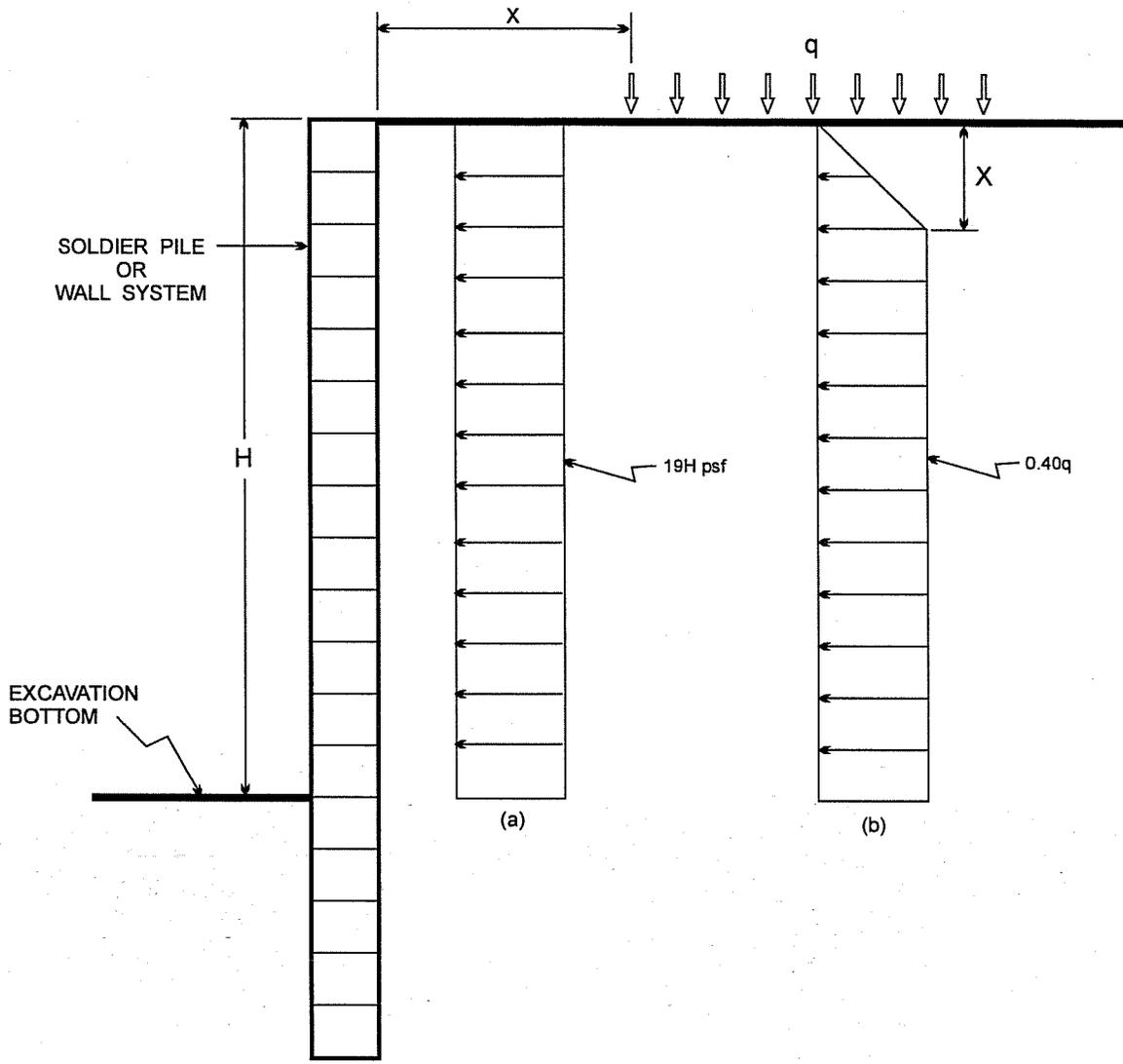
PARKING / RESIDENTIAL STRUCTURE

SAN DIEGO, CALIFORNIA

DATE 10-14-2003

PROJECT NO. 06851-22-02

FIG. 4



(a).....SOIL PRESSURE
 (b).....UNIFORM SURCHARGE

NO SCALE

LATERAL ACTIVE PRESSURES FOR VERTICAL EXCAVATIONS

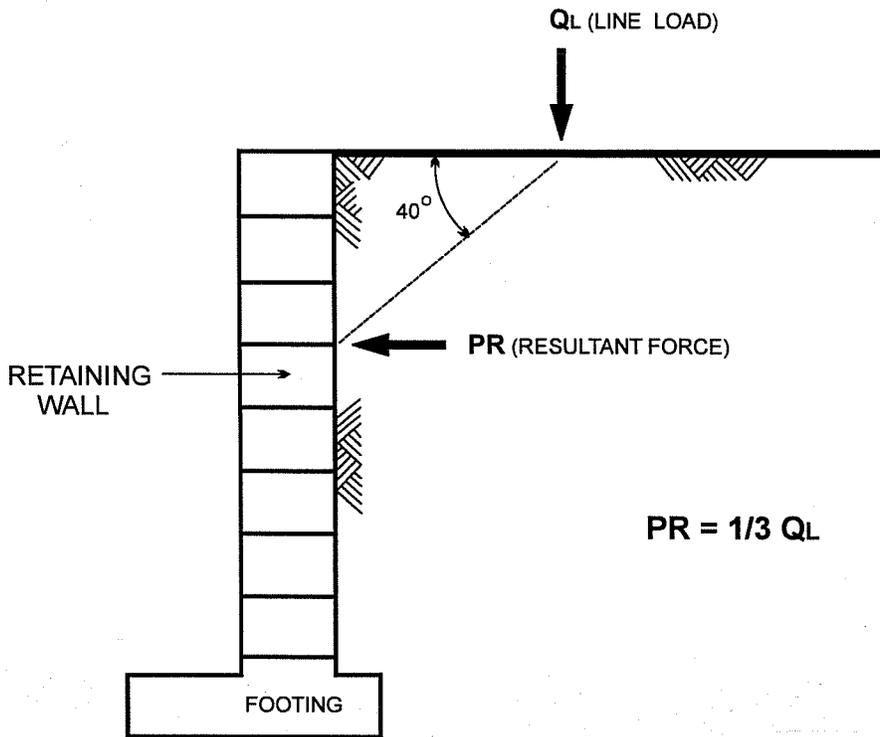
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PD / RSS		DSK / E0000	DATE 10-14-2003	PROJECT NO. 06851 - 22 - 02	FIG. 5
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NO SCALE

RETAINING WALL LOADING DIAGRAM

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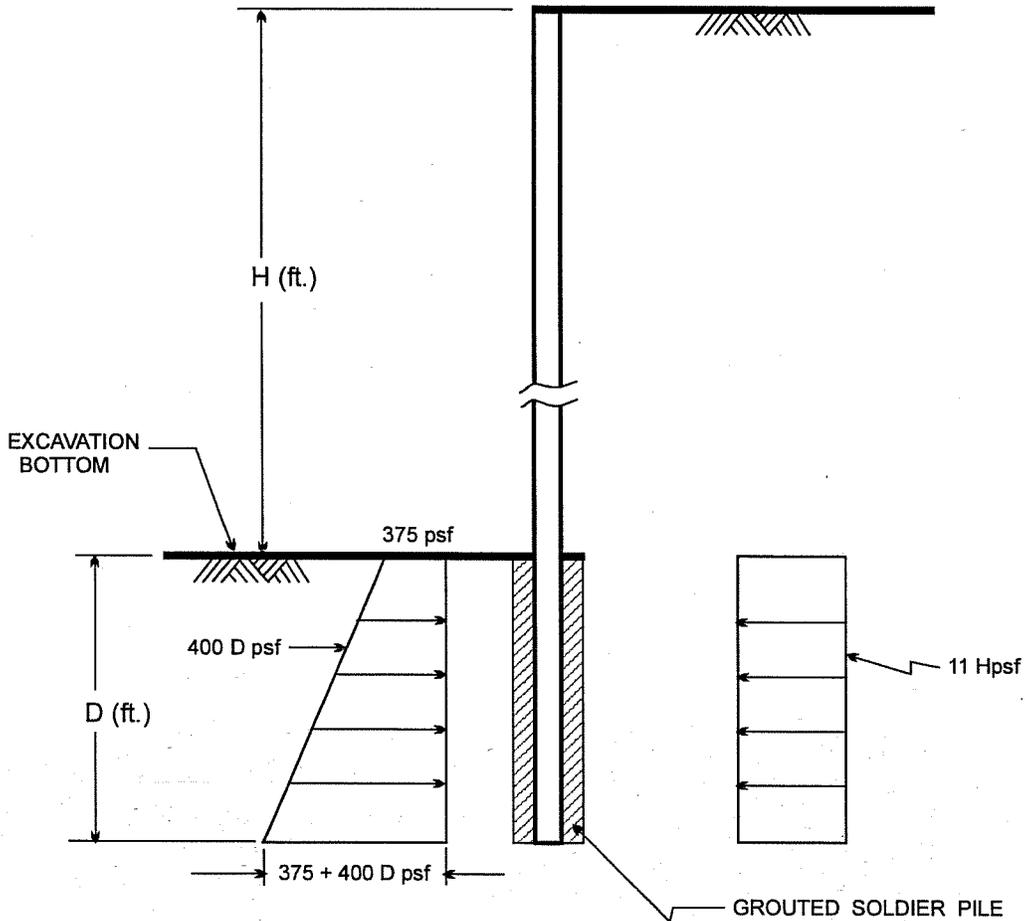
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DATE 10-14-2003

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FIG. 6



NO SCALE

RECOMMENDED GROUDED SOLDIER PILE PRESSURE DISTRIBUTION

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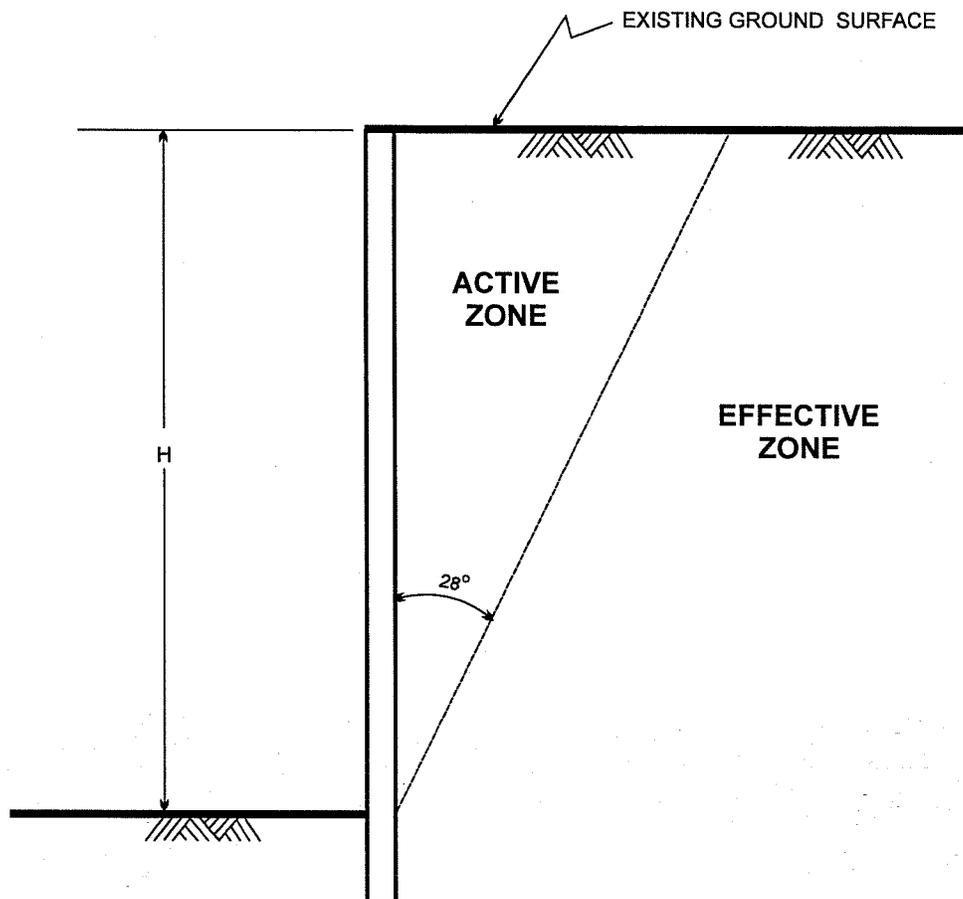
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DATE 10-14-2003

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FIG. 7



NO SCALE

RECOMMENDED EFFECTIVE ZONE FOR TIEBACK ANCHORS

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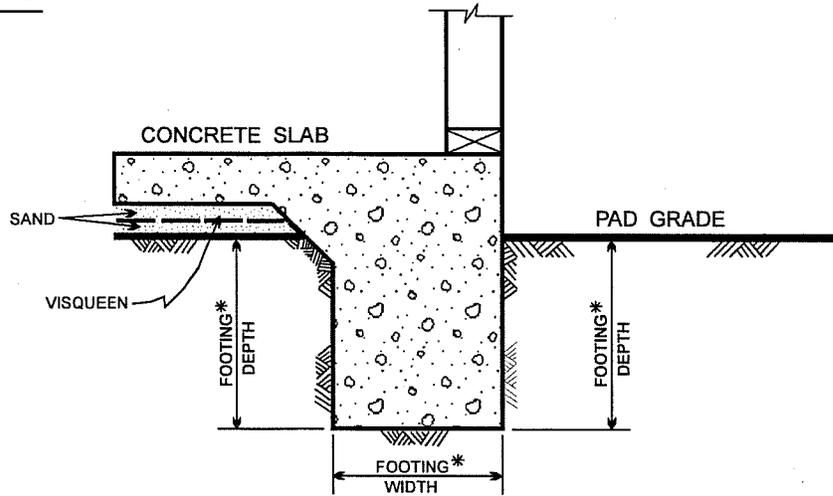
DATE 10-14-2003

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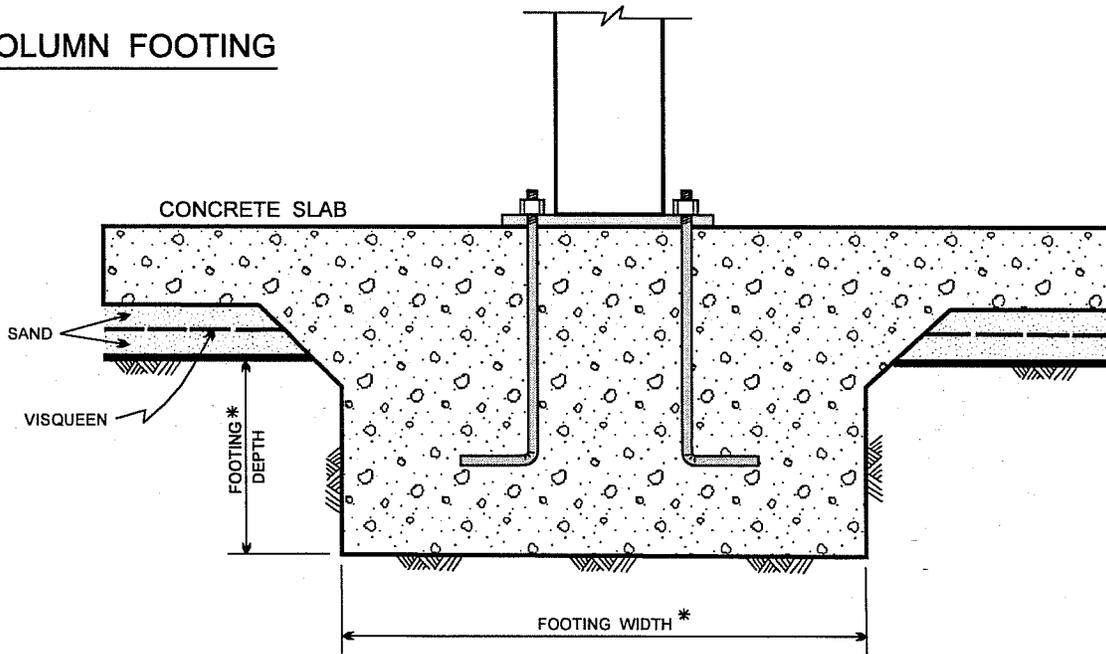
FIG. 8

REZTA / RSS

WALL FOOTING



COLUMN FOOTING



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

NO SCALE

WALL / COLUMN FOOTING DIMENSION DETAIL

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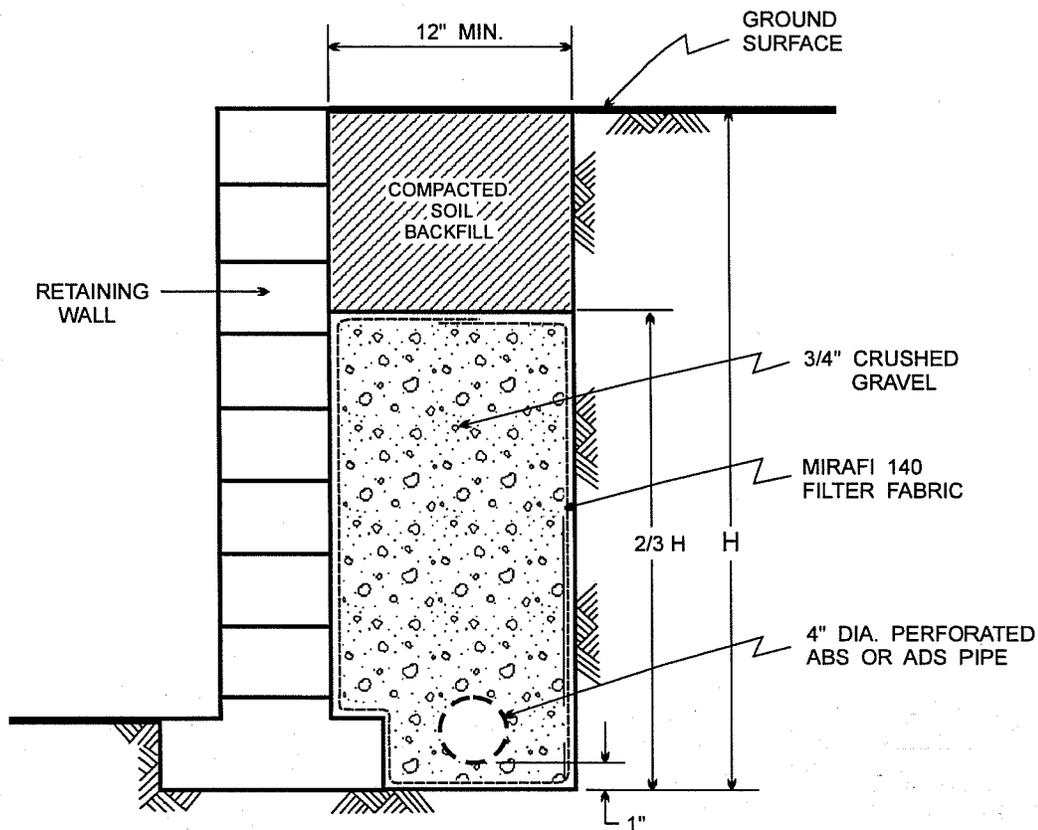
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DATE 10-14-2003

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FIG 9



NO SCALE

RETAINING WALL DRAINAGE DETAIL

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DSK / GTYPD

DATE 10-14-2003

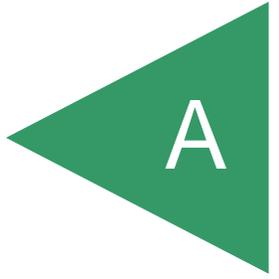
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FIG.10

RETWALL / RSS / VII-IXV

APPENDIX

A



APPENDIX A

FIELD INVESTIGATION

Fieldwork for our investigations was performed between July 28 and August 1, 2003 and consisted of the excavation and detailed logging of two exploratory trenches and five small-diameter borings. The locations of the exploratory trenches and borings are shown on the Site Plan, Figure 2. Trench logs, boring logs, and an explanation of the geologic units encountered are presented on Figures A-1 through A-9.

The small-diameter borings were drilled to depths of between 71 and 91 feet below the existing ground surface using a truck-mounted drill rig equipped with mud rotary drilling equipment. Relatively undisturbed samples were obtained from the small-diameter borings by driving a 3-inch O.D. split-tube sampler 12 inches into the undisturbed soil mass with blows from a 140-pound hammer falling a distance of 30 inches. The sampler was equipped with 1-inch-high by 2 $\frac{3}{8}$ -inch-diameter brass sampler rings to facilitate removal and testing.

The trenches were oriented in a generally east-west direction at close to right angles to the regional and local trend of splays within the Rose Canyon Fault Zone. A total of 314 lineal feet of trench was logged by our engineering geologist during the investigation. The trenches were excavated to a maximum depth of 14.5 feet below the existing ground surface with a rubber-tired John Deere 310 backhoe.

Trench widths were generally 2 feet with locally wider areas where sloughing occurred. Detailed logging of the trench walls was performed at a scale of 1 inch equals 5 feet (1" = 5'). Stationing along the trench surfaces was established during logging for accurate location of features and for ease of description. Also, a horizontal string line was established within the trenches for use as an internal reference. The entire surface of the formations exposed along the respective north and south sides of each trench was cleaned and examined for indications of faulting. These indications could include offset units, contacts, laminations, tectonically disturbed or deformed clay layers, clay gouge, fissures, or slickensides.

The soils encountered in the borings and trenches were visually examined, classified and logged in general accordance with American Society for Testing and Materials (ASTM) practice for Description and Identification of Soils (Visual-Manual Procedure D2488). The logs depict the soil and geologic conditions observed and the depth at which samples were obtained.

Asphalt concrete was used to repair the parking lot's surface on August 1, 2003.

DEPTH IN FEET	SAMPLE NO.	LITHOLOGY	GROUNDWATER	SOIL CLASS (USCS)	BORING B 1		PENETRATION RESISTANCE (BLOWS/FT.)	DRY DENSITY (P.C.F.)	MOISTURE CONTENT (%)
					ELEV. (MSL.) _____	DATE COMPLETED <u>07-30-2003</u>			
					EQUIPMENT <u>MUD ROTARY</u>				
MATERIAL DESCRIPTION									
0					BAY POINT FORMATION Medium dense, moist, red-brown, Silty, fine to coarse SAND and gravel				
2	B1-1			SM			21	123.5	8.7
4									
6	B1-2						24		
8									
10	B1-3						24		
12									
14									
16	B1-4				-Becomes dense at 15 feet		42		
18									
20	B1-5						60	104.3	19.8
22									
24				SC	SAN DIEGO FORMATION Very dense, moist, olive green, Clayey, fine to coarse SAND				
26	B1-6						43		
28				CL	Hard, moist, olive green, Sandy CLAY -6" gravel at 27 feet				

Figure A-1,
Log of Boring B 1, Page 1 of 3

06851-22-02.GPJ

SAMPLE SYMBOLS		
	... SAMPLING UNSUCCESSFUL	
	... DISTURBED OR BAG SAMPLE	
	... STANDARD PENETRATION TEST	
	... CHUNK SAMPLE	
		... WATER TABLE OR SEEPAGE

NOTE: THE LOG OF SUBSURFACE CONDITIONS SHOWN HEREON APPLIES ONLY AT THE SPECIFIC BORING OR TRENCH LOCATION AND AT THE DATE INDICATED. IT IS NOT WARRANTED TO BE REPRESENTATIVE OF SUBSURFACE CONDITIONS AT OTHER LOCATIONS AND TIMES.

DEPTH IN FEET	SAMPLE NO.	LITHOLOGY	GROUNDWATER	SOIL CLASS (USCS)	BORING B 1		PENETRATION RESISTANCE (BLOWS/FT.)	DRY DENSITY (P.C.F.)	MOISTURE CONTENT (%)	
					ELEV. (MSL.) _____	DATE COMPLETED <u>07-30-2003</u>				
					EQUIPMENT <u>MUD ROTARY</u>					
MATERIAL DESCRIPTION										
30	B1-7		▼	SC	Very dense, moist, olive-green, Clayey, fine to coarse SAND		42			
32										
34	B1-8								52	
36										
38										
40	B1-9			SC	Very stiff, saturated, mottled, tan and orange, Sandy SILT, micaceous		31	103.3	23.5	
42										
44										
46	B1-10			ML	Very stiff, saturated, gray brown, Sandy CLAY		21			
48										
50	B1-11			SC	Dense, saturated, green-brown, Clayey SAND		26	116.6	15.7	
52										
54					-6" layer of gravel at 52 feet					
56	B1-12			CL	Very stiff, saturated, gray, Sandy CLAY		18			
58										

Figure A-1,
Log of Boring B 1, Page 2 of 3

06851-22-02.GPJ

SAMPLE SYMBOLS		
	... SAMPLING UNSUCCESSFUL	
	... DISTURBED OR BAG SAMPLE	
	... STANDARD PENETRATION TEST	
	... CHUNK SAMPLE	
		... DRIVE SAMPLE (UNDISTURBED)
		... WATER TABLE OR SEEPAGE

NOTE: THE LOG OF SUBSURFACE CONDITIONS SHOWN HEREON APPLIES ONLY AT THE SPECIFIC BORING OR TRENCH LOCATION AND AT THE DATE INDICATED. IT IS NOT WARRANTED TO BE REPRESENTATIVE OF SUBSURFACE CONDITIONS AT OTHER LOCATIONS AND TIMES.

DEPTH IN FEET	SAMPLE NO.	LITHOLOGY	GROUNDWATER	SOIL CLASS (USCS)	BORING B 1		PENETRATION RESISTANCE (BLOWS/FT.)	DRY DENSITY (P.C.F.)	MOISTURE CONTENT (%)
					ELEV. (MSL.) _____	DATE COMPLETED <u>07-30-2003</u>			
					EQUIPMENT <u>MUD ROTARY</u>				
MATERIAL DESCRIPTION									
60	B1-13			SM	Very dense, saturated, gray-brown, Silty, fine to coarse SAND		81/11"	107.3	21.1
62				GC	Very dense, saturated, gray-brown, Sandy GRAVEL				
64				CL	Hard, saturated, gray brown, Sandy CLAY				
66	B1-14						34		
68				SC	Dense, saturated, orange-brown, Clayey, fine to coarse SAND				
70	B1-15						22	111.8	17.3
					BORING TERMINATED AT 71 FEET Groundwater at 34 feet Hole filled with 1 x 50lb sack of cement grout				

Figure A-1,
Log of Boring B 1, Page 3 of 3

06851-22-02.GPJ

SAMPLE SYMBOLS	... SAMPLING UNSUCCESSFUL	... STANDARD PENETRATION TEST	... DRIVE SAMPLE (UNDISTURBED)
	... DISTURBED OR BAG SAMPLE	... CHUNK SAMPLE	... WATER TABLE OR SEEPAGE

NOTE: THE LOG OF SUBSURFACE CONDITIONS SHOWN HEREON APPLIES ONLY AT THE SPECIFIC BORING OR TRENCH LOCATION AND AT THE DATE INDICATED. IT IS NOT WARRANTED TO BE REPRESENTATIVE OF SUBSURFACE CONDITIONS AT OTHER LOCATIONS AND TIMES.

DEPTH IN FEET	SAMPLE NO.	LITHOLOGY	GROUNDWATER	SOIL CLASS (USCS)	BORING B 2		PENETRATION RESISTANCE (BLOWS/FT.)	DRY DENSITY (P.C.F.)	MOISTURE CONTENT (%)
					ELEV. (MSL.) _____	DATE COMPLETED 07-30-2003			
					EQUIPMENT MUD ROTARY				
MATERIAL DESCRIPTION									
0					UNDOCUMENTED FILL Dense, moist, green-gray, Silty, fine to medium SAND				
2	B2-1			SM			13		
4									
6	B2-2						25		
8									
10	B2-3						18	126.2	10.0
12					BAY POINT FORMATION Medium dense, moist, mottled orange-brown, Silty, fine to coarse SAND				
14									
16	B2-4			SM			9		
18									
20	B2-5				-Becomes green-gray at 20 feet		75		
22									
24									
26	B2-6						66		
28									

**Figure A-2,
Log of Boring B 2, Page 1 of 3**

06851-22-02.GPJ

SAMPLE SYMBOLS	<input type="checkbox"/> ... SAMPLING UNSUCCESSFUL	<input type="checkbox"/> ... STANDARD PENETRATION TEST	<input type="checkbox"/> ... DRIVE SAMPLE (UNDISTURBED)
	<input checked="" type="checkbox"/> ... DISTURBED OR BAG SAMPLE	<input type="checkbox"/> ... CHUNK SAMPLE	<input type="checkbox"/> ... WATER TABLE OR SEEPAGE

NOTE: THE LOG OF SUBSURFACE CONDITIONS SHOWN HEREON APPLIES ONLY AT THE SPECIFIC BORING OR TRENCH LOCATION AND AT THE DATE INDICATED. IT IS NOT WARRANTED TO BE REPRESENTATIVE OF SUBSURFACE CONDITIONS AT OTHER LOCATIONS AND TIMES.

DEPTH IN FEET	SAMPLE NO.	LITHOLOGY	GROUNDWATER	SOIL CLASS (USCS)	BORING B 2		PENETRATION RESISTANCE (BLOWS/FT.)	DRY DENSITY (P.C.F.)	MOISTURE CONTENT (%)
					ELEV. (MSL.) _____	DATE COMPLETED <u>07-30-2003</u>			
					EQUIPMENT				
					MUD ROTARY				
MATERIAL DESCRIPTION									
30	B2-7					-No recovery	60		
32			▼	SM					
34									
36	B2-8					-6" gravel layer at 35 feet SAN DIEGO FORMATION Stiff, saturated, mottled orange-tan, Sandy SILT, micaceous	15		
38				ML					
40	B2-9					-10" gravel layer at 39 feet -No recovery	37		
42						Stiff, saturated, mottled orange and gray brown, Sandy CLAY			
44				CL					
46	B2-10					Very stiff, saturated, mottled orange and gray brown, Sandy SILT	36		
48				ML					
50	B2-11						34	101.9	24.1
52									
54						Hard, saturated, mottled orange and tan, CLAY			
56	B2-12			CL			32		
58									

Figure A-2,
Log of Boring B 2, Page 2 of 3

06851-22-02.GPJ

SAMPLE SYMBOLS					
	... SAMPLING UNSUCCESSFUL		... STANDARD PENETRATION TEST		... DRIVE SAMPLE (UNDISTURBED)
	... DISTURBED OR BAG SAMPLE		... CHUNK SAMPLE		... WATER TABLE OR SEEPAGE

NOTE: THE LOG OF SUBSURFACE CONDITIONS SHOWN HEREON APPLIES ONLY AT THE SPECIFIC BORING OR TRENCH LOCATION AND AT THE DATE INDICATED. IT IS NOT WARRANTED TO BE REPRESENTATIVE OF SUBSURFACE CONDITIONS AT OTHER LOCATIONS AND TIMES.

DEPTH IN FEET	SAMPLE NO.	LITHOLOGY	GROUNDWATER	BORING B 2		PENETRATION RESISTANCE (BLOWS/FT.)	DRY DENSITY (P.C.F.)	MOISTURE CONTENT (%)
				ELEV. (MSL.) _____	DATE COMPLETED <u>07-30-2003</u>			
					EQUIPMENT <u>MUD ROTARY</u>			
					MATERIAL DESCRIPTION			
60	B2-13			CL		71		
62								
64					-4" Layer of gravel at 64 feet			
66	B2-14				-6" layer of gravel at 65.5 feet	50/2"		
68				SM	Very dense, saturated, mottled orange and gray, Silty, fine to coarse SAND			
70	B2-15					73	114.5	16.3
					BORING TERMINATED AT 71 FEET Groundwater at 31.8 feet Hole filled with 1 x 50lb sack of cement slurry			

Figure A-2,
Log of Boring B 2, Page 3 of 3

06851-22-02.GPJ

SAMPLE SYMBOLS	 ... SAMPLING UNSUCCESSFUL	 ... STANDARD PENETRATION TEST	 ... DRIVE SAMPLE (UNDISTURBED)
	 ... DISTURBED OR BAG SAMPLE	 ... CHUNK SAMPLE	 ... WATER TABLE OR SEEPAGE

NOTE: THE LOG OF SUBSURFACE CONDITIONS SHOWN HEREON APPLIES ONLY AT THE SPECIFIC BORING OR TRENCH LOCATION AND AT THE DATE INDICATED. IT IS NOT WARRANTED TO BE REPRESENTATIVE OF SUBSURFACE CONDITIONS AT OTHER LOCATIONS AND TIMES.

DEPTH IN FEET	SAMPLE NO.	LITHOLOGY	GROUNDWATER	SOIL CLASS (USCS)	BORING B 3		PENETRATION RESISTANCE (BLOWS/FT.)	DRY DENSITY (P.C.F.)	MOISTURE CONTENT (%)
					ELEV. (MSL.) _____	DATE COMPLETED <u>07-31-2003</u>			
					EQUIPMENT <u>MUD ROTARY</u>				
MATERIAL DESCRIPTION									
0					FILL Medium dense, moist, red-brown, Silty, fine to medium SAND				
2	B3-1			SM			6	116.1	8.0
4					BAY POINT FORMATION Medium dense, moist, red-brown, Silty, fine to medium SAND				
6	B3-2						25		
8									
10	B3-3			SM			36		
12									
14									
16	B3-4				-Becomes mottled, orange-tan at 15 feet		20		
18					-12" thick gravel layer at 18 feet				
20	B3-5				-Becomes silty fine sand		70	109.4	13.9
22									
24									
26	B3-6				-No recovery		59		
28									

Figure A-3,
Log of Boring B 3, Page 1 of 3

06851-22-02.GPJ

SAMPLE SYMBOLS		
	... SAMPLING UNSUCCESSFUL	
	... DISTURBED OR BAG SAMPLE	
	... STANDARD PENETRATION TEST	
	... CHUNK SAMPLE	
		... DRIVE SAMPLE (UNDISTURBED)
		... WATER TABLE OR SEEPAGE

NOTE: THE LOG OF SUBSURFACE CONDITIONS SHOWN HEREON APPLIES ONLY AT THE SPECIFIC BORING OR TRENCH LOCATION AND AT THE DATE INDICATED. IT IS NOT WARRANTED TO BE REPRESENTATIVE OF SUBSURFACE CONDITIONS AT OTHER LOCATIONS AND TIMES.

DEPTH IN FEET	SAMPLE NO.	LITHOLOGY	GROUNDWATER	SOIL CLASS (USCS)	BORING B 3		PENETRATION RESISTANCE (BLOWS/FT.)	DRY DENSITY (P.C.F.)	MOISTURE CONTENT (%)
					ELEV. (MSL.) _____	DATE COMPLETED 07-31-2003			
					EQUIPMENT MUD ROTARY				
					MATERIAL DESCRIPTION				
30	B3-7				Very dense, moist, tan, Clayey, fine SAND		20		
32			▼	SC					
34					SAN DIEGO FORMATION Hard, saturated, tan-gray, Sandy SILT, micaceous				
36	B3-8						36		
38				ML					
40	B3-9						34	106.8	21.7
42									
44									
46	B3-10				Very stiff, saturated, mottled orange and gray, Sandy CLAY		20		
48									
50	B3-11						39	104.1	11.5
52				CL					
54									
56	B3-12						50/3-1/2"		
58					-6" gravel layer at 59 feet				

**Figure A-3,
Log of Boring B 3, Page 2 of 3**

06851-22-02.GPJ

SAMPLE SYMBOLS		
	... SAMPLING UNSUCCESSFUL	
	... DISTURBED OR BAG SAMPLE	
	... STANDARD PENETRATION TEST	
	... CHUNK SAMPLE	
		▼ ... WATER TABLE OR SEEPAGE

NOTE: THE LOG OF SUBSURFACE CONDITIONS SHOWN HEREON APPLIES ONLY AT THE SPECIFIC BORING OR TRENCH LOCATION AND AT THE DATE INDICATED. IT IS NOT WARRANTED TO BE REPRESENTATIVE OF SUBSURFACE CONDITIONS AT OTHER LOCATIONS AND TIMES.

DEPTH IN FEET	SAMPLE NO.	LITHOLOGY	GROUNDWATER	SOIL CLASS (USCS)	BORING B 3		PENETRATION RESISTANCE (BLOWS/FT.)	DRY DENSITY (P.C.F.)	MOISTURE CONTENT (%)		
					ELEV. (MSL.) _____	DATE COMPLETED <u>07-31-2003</u>					
						EQUIPMENT					
						MUD ROTARY					
MATERIAL DESCRIPTION											
60	B3-13			SC	Dense, saturated, mottled, olive green and orange, Clayey, fine to medium SAND		53	112.7	17.3		
62											
64							-6" Layer of gravel at 63 feet				
66	B3-14						-Becomes silty fine sand at 65 feet				81
68											
70	B3-15						70				
BORING TERMINATED AT 71 FEET Groundwater at 32.5 feet Hole filled with 1 x 50lb sack of cement slurry											

**Figure A-3,
Log of Boring B 3, Page 3 of 3**

06851-22-02.GPJ

SAMPLE SYMBOLS	 ... SAMPLING UNSUCCESSFUL	 ... STANDARD PENETRATION TEST	 ... DRIVE SAMPLE (UNDISTURBED)
	 ... DISTURBED OR BAG SAMPLE	 ... CHUNK SAMPLE	 ... WATER TABLE OR SEEPAGE

NOTE: THE LOG OF SUBSURFACE CONDITIONS SHOWN HEREON APPLIES ONLY AT THE SPECIFIC BORING OR TRENCH LOCATION AND AT THE DATE INDICATED. IT IS NOT WARRANTED TO BE REPRESENTATIVE OF SUBSURFACE CONDITIONS AT OTHER LOCATIONS AND TIMES.

DEPTH IN FEET	SAMPLE NO.	LITHOLOGY	GROUNDWATER	SOIL CLASS (USCS)	BORING B 4		PENETRATION RESISTANCE (BLOWS/FT.)	DRY DENSITY (P.C.F.)	MOISTURE CONTENT (%)
					ELEV. (MSL.) _____	DATE COMPLETED <u>07-31-2003</u>			
					EQUIPMENT <u>MUD ROTARY</u>				
MATERIAL DESCRIPTION									
0					BAY POINT FORMATION Medium dense, moist, red-brown, Silty, fine to medium SAND				
2	B4-1			SM			9		
4									
6	B4-2				-Becomes dense at 5 feet		32		
8									
10	B4-3						30	119.7	11.9
12									
14									
16	B4-4						36		
18									
20	B4-5				-Becomes micaceous at 20 feet		48		
22									
24									
26	B4-6						53		
28									

Figure A-4,
Log of Boring B 4, Page 1 of 3

06851-22-02.GPJ

SAMPLE SYMBOLS	...		
		SAMPLING UNSUCCESSFUL	
	DISTURBED OR BAG SAMPLE		CHUNK SAMPLE
			DRIVE SAMPLE (UNDISTURBED)
			WATER TABLE OR SEEPAGE

NOTE: THE LOG OF SUBSURFACE CONDITIONS SHOWN HEREON APPLIES ONLY AT THE SPECIFIC BORING OR TRENCH LOCATION AND AT THE DATE INDICATED. IT IS NOT WARRANTED TO BE REPRESENTATIVE OF SUBSURFACE CONDITIONS AT OTHER LOCATIONS AND TIMES.

DEPTH IN FEET	SAMPLE NO.	LITHOLOGY	GROUNDWATER	SOIL CLASS (USCS)	BORING B 4		PENETRATION RESISTANCE (BLOWS/FT.)	DRY DENSITY (P.C.F.)	MOISTURE CONTENT (%)
					ELEV. (MSL.) _____	DATE COMPLETED 07-31-2003			
					EQUIPMENT MUD ROTARY				
					MATERIAL DESCRIPTION				
30	B4-7				SAN DIEGO FORMATION Dense, moist, mottled orange and olive green, Clayey, fine to medium SAND		18	105.3	21.8
32									
34									
36	B4-8			SC			43		
38									
40	B4-9				Very stiff, saturated, olive-green, SILT, micaceous		32		
42				ML					
44									
46	B4-10						28		
48					Very stiff, saturated, olive-gray, CLAY				
50	B4-11			CL			34	109.3	19.9
52									
54									
56	B4-12				Dense, saturated, green-tan, fine to coarse SAND, trace clay		41		
58				SP					

Figure A-4,
Log of Boring B 4, Page 2 of 3

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SAMPLE SYMBOLS	... SAMPLING UNSUCCESSFUL	... STANDARD PENETRATION TEST	... DRIVE SAMPLE (UNDISTURBED)
	... DISTURBED OR BAG SAMPLE	... CHUNK SAMPLE	... WATER TABLE OR SEEPAGE

NOTE: THE LOG OF SUBSURFACE CONDITIONS SHOWN HEREON APPLIES ONLY AT THE SPECIFIC BORING OR TRENCH LOCATION AND AT THE DATE INDICATED. IT IS NOT WARRANTED TO BE REPRESENTATIVE OF SUBSURFACE CONDITIONS AT OTHER LOCATIONS AND TIMES.

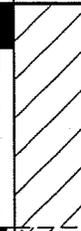
DEPTH IN FEET	SAMPLE NO.	LITHOLOGY	GROUNDWATER	SOIL CLASS (USCS)	BORING B 4		PENETRATION RESISTANCE (BLOWS/FT.)	DRY DENSITY (P.C.F.)	MOISTURE CONTENT (%)
					ELEV. (MSL.) _____	DATE COMPLETED <u>07-31-2003</u>			
					EQUIPMENT <u>MUD ROTARY</u>				
MATERIAL DESCRIPTION									
60	B4-13			CL	Very stiff, saturated, olive-green, CLAY		27		
62									
64									
66	B4-14			SC	Very dense, saturated, tan, Clayey, fine to coarse SAND		50/5"		
68									
70	B4-15							108.6	19.2
					BORING TERMINATED AT 71 FEET Groundwater at 27.5 feet Hole filled with 1 x 50lb sack of cement slurry				

Figure A-4,
Log of Boring B 4, Page 3 of 3

06851-22-02.GPJ

SAMPLE SYMBOLS	 ... SAMPLING UNSUCCESSFUL	 ... STANDARD PENETRATION TEST	 ... DRIVE SAMPLE (UNDISTURBED)
	 ... DISTURBED OR BAG SAMPLE	 ... CHUNK SAMPLE	 ... WATER TABLE OR SEEPAGE

NOTE: THE LOG OF SUBSURFACE CONDITIONS SHOWN HEREON APPLIES ONLY AT THE SPECIFIC BORING OR TRENCH LOCATION AND AT THE DATE INDICATED. IT IS NOT WARRANTED TO BE REPRESENTATIVE OF SUBSURFACE CONDITIONS AT OTHER LOCATIONS AND TIMES.

DEPTH IN FEET	SAMPLE NO.	LITHOLOGY	GROUNDWATER	SOIL CLASS (USCS)	BORING B 5		PENETRATION RESISTANCE (BLOWS/FT.)	DRY DENSITY (P.C.F.)	MOISTURE CONTENT (%)	
					ELEV. (MSL.) _____	DATE COMPLETED <u>07-30-2003</u>				
					EQUIPMENT <u>MUD ROTARY</u>					
MATERIAL DESCRIPTION										
0										
2				SM	BAY POINT FORMATION Very dense, moist, red-brown, Silty, fine to medium SAND					
4										
6										
8										
10	B5-1									69/11"
12										
14										
16										
18										
20	B5-2									75
22										
24										
26										
28										

Figure A-5,
Log of Boring B 5, Page 1 of 4

06851-22-02.GPJ

SAMPLE SYMBOLS	...		
	<input type="checkbox"/>	SAMPLING UNSUCCESSFUL	<input type="checkbox"/>
<input checked="" type="checkbox"/>	DISTURBED OR BAG SAMPLE	<input checked="" type="checkbox"/>	DRIVE SAMPLE (UNDISTURBED)
<input type="checkbox"/>	CHUNK SAMPLE	<input type="checkbox"/>	WATER TABLE OR SEEPAGE

NOTE: THE LOG OF SUBSURFACE CONDITIONS SHOWN HEREON APPLIES ONLY AT THE SPECIFIC BORING OR TRENCH LOCATION AND AT THE DATE INDICATED. IT IS NOT WARRANTED TO BE REPRESENTATIVE OF SUBSURFACE CONDITIONS AT OTHER LOCATIONS AND TIMES.

DEPTH IN FEET	SAMPLE NO.	LITHOLOGY	GROUNDWATER	SOIL CLASS (USCS)	BORING B 5		PENETRATION RESISTANCE (BLOWS/FT.)	DRY DENSITY (P.C.F.)	MOISTURE CONTENT (%)		
					ELEV. (MSL.) _____	DATE COMPLETED <u>07-30-2003</u>					
					EQUIPMENT <u>MUD ROTARY</u>						
					MATERIAL DESCRIPTION						
30	B5-3	[Lithology pattern: vertical lines with dots]	▼	SM	-Becomes saturated at 31.3 feet		67				
32											
34											
36					SAN DIEGO FORMATION Very stiff, saturated, mottled green-gray and orange, Sandy SILT						
38											
40	B5-4			ML			25				
42											
44											
46											
48											
50	B5-5						31	98.4	26.7		
52											
54											
56											
58											

Figure A-5,
Log of Boring B 5, Page 2 of 4

06851-22-02.GPJ

SAMPLE SYMBOLS	□ ... SAMPLING UNSUCCESSFUL	▣ ... STANDARD PENETRATION TEST	■ ... DRIVE SAMPLE (UNDISTURBED)
	▨ ... DISTURBED OR BAG SAMPLE	◼ ... CHUNK SAMPLE	▼ ... WATER TABLE OR SEEPAGE

NOTE: THE LOG OF SUBSURFACE CONDITIONS SHOWN HEREON APPLIES ONLY AT THE SPECIFIC BORING OR TRENCH LOCATION AND AT THE DATE INDICATED. IT IS NOT WARRANTED TO BE REPRESENTATIVE OF SUBSURFACE CONDITIONS AT OTHER LOCATIONS AND TIMES.

DEPTH IN FEET	SAMPLE NO.	LITHOLOGY	GROUNDWATER	SOIL CLASS (USCS)	BORING B 5		PENETRATION RESISTANCE (BLOWS/FT.)	DRY DENSITY (P.C.F.)	MOISTURE CONTENT (%)
					ELEV. (MSL.) _____	DATE COMPLETED <u>07-30-2003</u>			
					EQUIPMENT <u>MUD ROTARY</u>				
					MATERIAL DESCRIPTION				
60	B5-6	[Lithology pattern]		SM	Very dense, saturated, brown, Silty, fine to coarse SAND		77		
62			-6" Layer of gravel at 61.5 feet						
64									
66									
68									
70	B5-7						40	111.0	18.8
72									
74									
76									
78									
80	B5-8						80	120	13.1
82									
84									
86									
88									

Figure A-5,
Log of Boring B 5, Page 3 of 4

06851-22-02.GPJ

SAMPLE SYMBOLS	<input type="checkbox"/> ... SAMPLING UNSUCCESSFUL	<input type="checkbox"/> ... STANDARD PENETRATION TEST	<input checked="" type="checkbox"/> ... DRIVE SAMPLE (UNDISTURBED)
	<input checked="" type="checkbox"/> ... DISTURBED OR BAG SAMPLE	<input type="checkbox"/> ... CHUNK SAMPLE	<input type="checkbox"/> ... WATER TABLE OR SEEPAGE

NOTE: THE LOG OF SUBSURFACE CONDITIONS SHOWN HEREON APPLIES ONLY AT THE SPECIFIC BORING OR TRENCH LOCATION AND AT THE DATE INDICATED. IT IS NOT WARRANTED TO BE REPRESENTATIVE OF SUBSURFACE CONDITIONS AT OTHER LOCATIONS AND TIMES.

DEPTH IN FEET	SAMPLE NO.	LITHOLOGY	GROUNDWATER	SOIL CLASS (USCS)	BORING B 5		PENETRATION RESISTANCE (BLOWS/FT.)	DRY DENSITY (P.C.F.)	MOISTURE CONTENT (%)
					ELEV. (MSL.) _____	DATE COMPLETED <u>07-30-2003</u>			
					EQUIPMENT <u>MUD ROTARY</u>				
					MATERIAL DESCRIPTION				
90	B5-9						55	108.7	20.6
					BORING TERMINATED AT 91 FEET Groundwater at 31.3 feet Hole filed with 2 x 50lb sacks of cement slurry				

Figure A-5,
Log of Boring B 5, Page 4 of 4

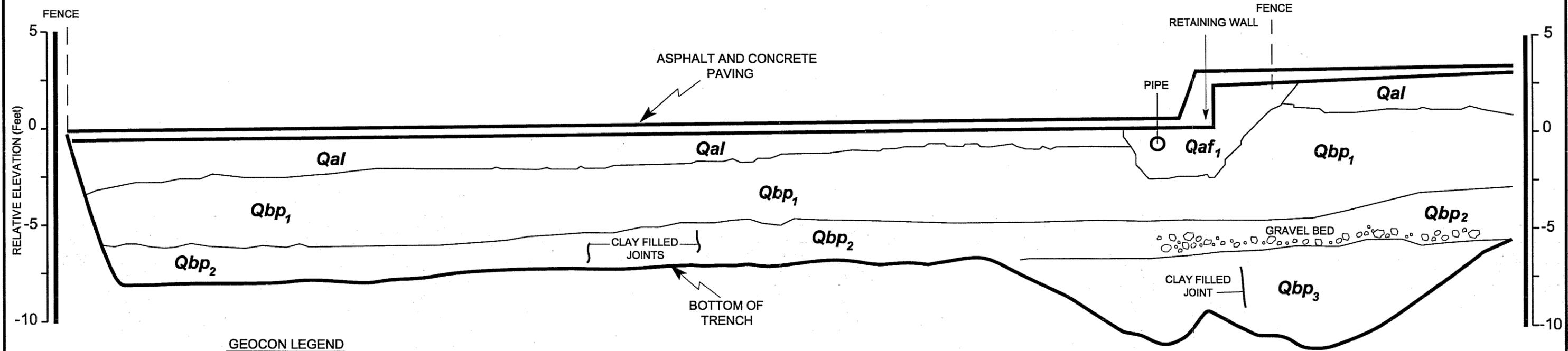
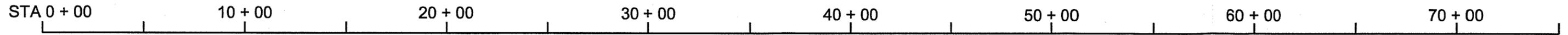
06851-22-02.GPJ

SAMPLE SYMBOLS	<input type="checkbox"/> ... SAMPLING UNSUCCESSFUL	<input type="checkbox"/> ... STANDARD PENETRATION TEST	<input checked="" type="checkbox"/> ... DRIVE SAMPLE (UNDISTURBED)
	<input checked="" type="checkbox"/> ... DISTURBED OR BAG SAMPLE	<input checked="" type="checkbox"/> ... CHUNK SAMPLE	<input checked="" type="checkbox"/> ... WATER TABLE OR SEEPAGE

NOTE: THE LOG OF SUBSURFACE CONDITIONS SHOWN HEREON APPLIES ONLY AT THE SPECIFIC BORING OR TRENCH LOCATION AND AT THE DATE INDICATED. IT IS NOT WARRANTED TO BE REPRESENTATIVE OF SUBSURFACE CONDITIONS AT OTHER LOCATIONS AND TIMES.

CEDAR / KETTNER
PARKING / RESIDENTIAL STRUCTURE
SAN DIEGO, CALIFORNIA

WEST



GEOCON LEGEND

- Qaf**FILL
- Qaf₁**Loose, damp, dark, yellowish - brown, Silty, fine to coarse SAND(SM), with gravel scattered debris, including brick, glass and wood
- Qaf₂**Loose, to medium dense, damp, olive - gray, Clayey SAND(SC), scattered debris
- Qaf₃**Debris, bottles, ash, wood, wire, ceramics enclosed in a cylindrical structure lined with 1" thick concrete walls
- Qaf₄**Dense, dry, pale yellowish - brown, Silty SAND(SM) with gravel
- Qaf₅**Medium dense, moist, greenish -gray to olive - grey, Silty SAND(SM), strong organic odor
- Qal**ALLUVIUM
- Qal**Loose, damp to moist, olive - gray, Silty SAND(SM)
- Qbp**BAY POINT FORMATION
- Qbp₁**Loose, damp to moist, moderate brown to moderate yellowish - brown, Silty SAND(SM), becomes medium dense below approximately 2 to 3 feet
- Qbp₂**Medium dense, moist, mottled moderate brown, light brown, moderate and pale yellowish - brown, and light olive - gray, Clayey SAND(SC); in Trench 2, 2 approximately horizontal, parallel, non continuous, lenticular clay beds were observed. These interbeds are between 1 and 6 inches thick. In Trench 1 a discontinuous, lenticular gravel interbed was observed.
- Qbp₃**Dense, damp, mottled yellows and browns, Silty SAND(SM), partially cemented

SCALE 1" = 5' (VERT. = HORIZ.)

LOG OF FAULT TRENCH 1

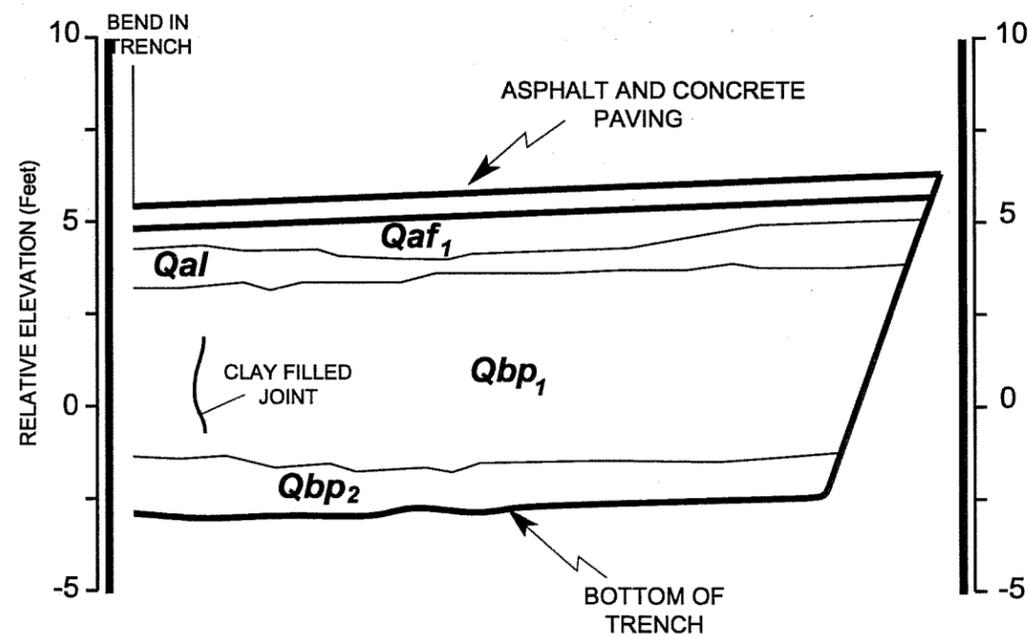
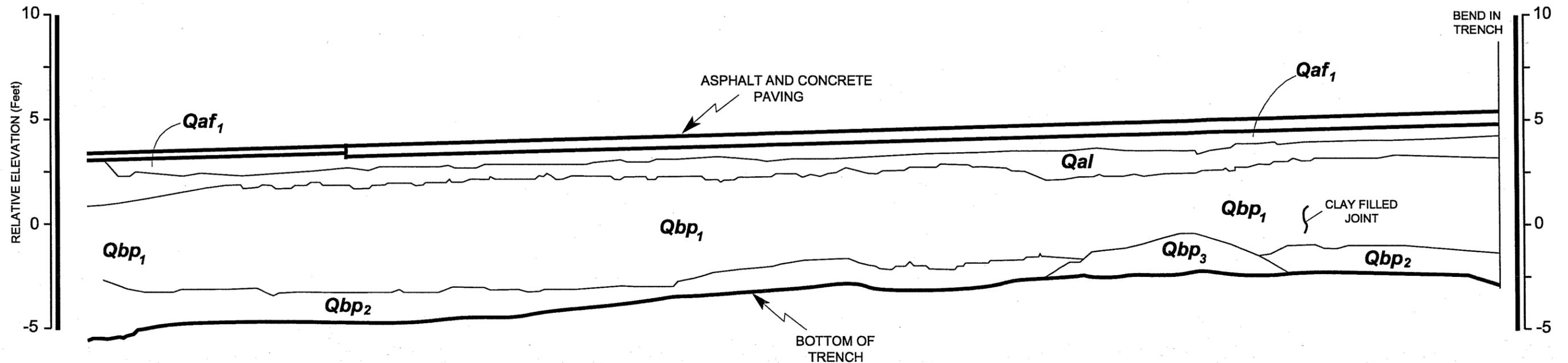
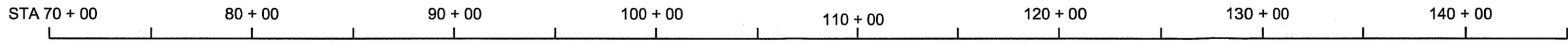
GEOCON
INCORPORATED

GEOTECHNICAL CONSULTANTS
6960 FLANDERS DRIVE - SAN DIEGO, CALIFORNIA 92121 - 2974
PHONE 858 558-6900 - FAX 858 558-6159
PROJECT NO. 06851 - 22 - 02
FIGURE A - 6
DATE 10-14-2003



CEDAR / KETTNER
PARKING / RESIDENTIAL STRUCTURE
SAN DIEGO, CALIFORNIA

← WEST



GEOCON LEGEND

- QafFILL
- Qaf₁Loose, damp, dark, yellowish - brown, Silty, fine to coarse SAND(SM), with gravel scattered debris, including brick, glass and wood
- Qaf₂Loose, to medium dense, damp, olive - gray, Clayey SAND(SC), scattered debris
- Qaf₃Debris, bottles, ash, wood, wire, ceramics enclosed in a cylindrical structure lined with 1" thick concrete walls
- Qaf₄Dense, dry, pale yellowish - brown, Silty SAND(SM) with gravel
- Qaf₅Medium dense, moist, greenish - gray to olive - grey, Silty SAND(SM), strong organic odor
- QalALLUVIUM
- QalLoose, damp to moist, olive - gray, Silty SAND(SM)
- QbpBAY POINT FORMATION
- Qbp₁Loose, damp to moist, moderate brown to moderate yellowish - brown, Silty SAND(SM), becomes medium dense below approximately 2 to 3 feet
- Qbp₂Medium dense, moist, mottled moderate brown, light brown, moderate and pale yellowish - brown, and light olive - gray, Clayey SAND(SC); in Trench 2, 2 approximately horizontal, parallel, non continuous, lenticular clay beds were observed. These interbeds are between 1 and 6 inches thick. In Trench 1 a discontinuous, lenticular gravel interbed was observed.
- Qbp₃Dense, damp, mottled yellows and browns, Silty SAND(SM), partially cemented

SCALE 1" = 5' (VERT. = HORIZ.)

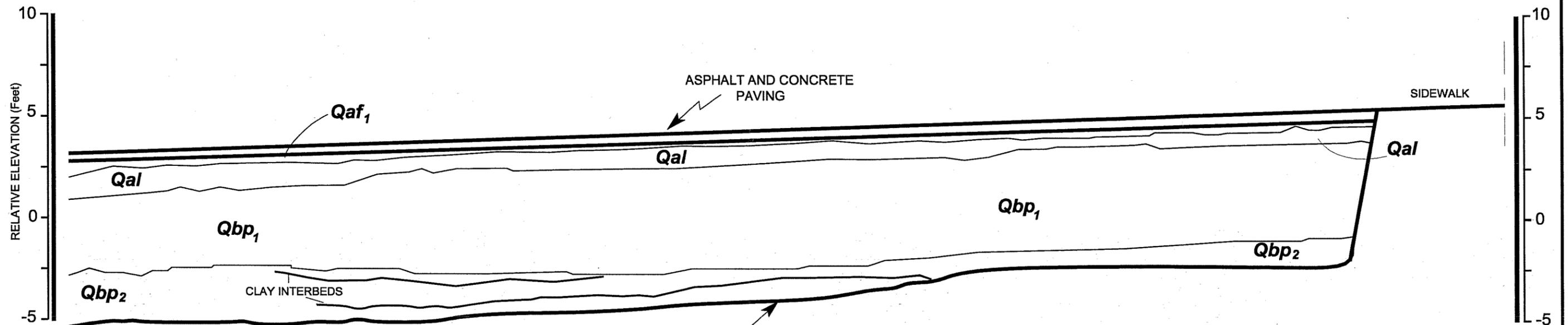
LOG OF FAULT TRENCH 1

GEOCON
INCORPORATED
GEOTECHNICAL CONSULTANTS
6960 FLANDERS DRIVE - SAN DIEGO, CALIFORNIA 92121 - 2974
PHONE 858 558-6900 - FAX 858 558-6159
PROJECT NO. 06851 - 22 - 02
FIGURE A - 7
DATE 10-14-2003

CEDAR / KETTNER
PARKING / RESIDENTIAL STRUCTURE
SAN DIEGO, CALIFORNIA

EAST

STA 70 + 00 60 + 00 50 + 00 40 + 00 30 + 00 20 + 00 10 + 00 0 + 00



GEOCON LEGEND

- Qaf**FILL
- Qaf₁**Loose, damp, dark, yellowish - brown, Silty, fine to coarse SAND(SM), with gravel scattered debris, including brick, glass and wood
- Qaf₂**Loose, to medium dense, damp, olive - gray, Clayey SAND(SC), scattered debris
- Qaf₃**Debris, bottles, ash, wood, wire, ceramics enclosed in a cylindrical structure lined with 1" thick concrete walls
- Qaf₄**Dense, dry, pale yellowish - brown, Silty SAND(SM) with gravel
- Qaf₅**Medium dense, moist, greenish -gray to olive - gray, Silty SAND(SM), strong organic odor
- Qal**ALLUVIUM
- Qal**Loose, damp to moist, olive - gray, Silty SAND(SM)
- Qbp**BAY POINT FORMATION
- Qbp₁**Loose, damp to moist, moderate brown to moderate yellowish - brown, Silty SAND(SM), becomes medium dense below approximately 2 to 3 feet
- Qbp₂**Medium dense, moist, mottled moderate brown, light brown, moderate and pale yellowish - brown, and light olive - gray, Clayey SAND(SC); in Trench 2, 2 approximately horizontal, parallel, non continuous, lenticular clay beds were observed. These interbeds are between 1 and 6 inches thick. In Trench 1 a discontinuous, lenticular gravel interbed was observed.
- Qbp₃**Dense, damp, mottled yellows and browns, Silty SAND(SM), partially cemented

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PROJECT NO. 06851 - 22 - 02
FIGURE A - 8
DATE 10-14-2003

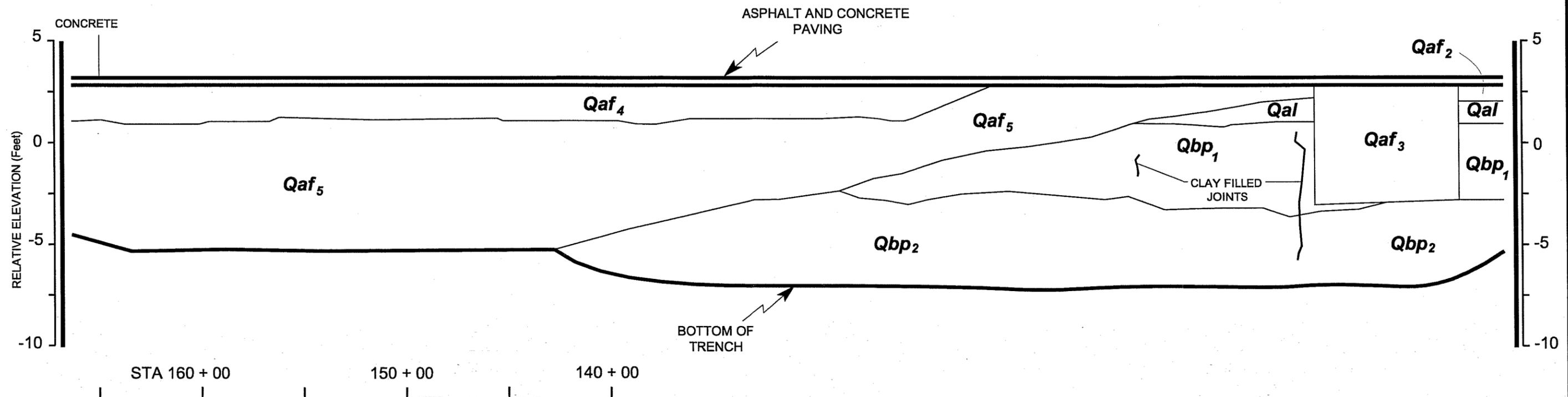
SCALE 1" = 5' (VERT. = HORIZ.)

LOG OF FAULT TRENCH 2

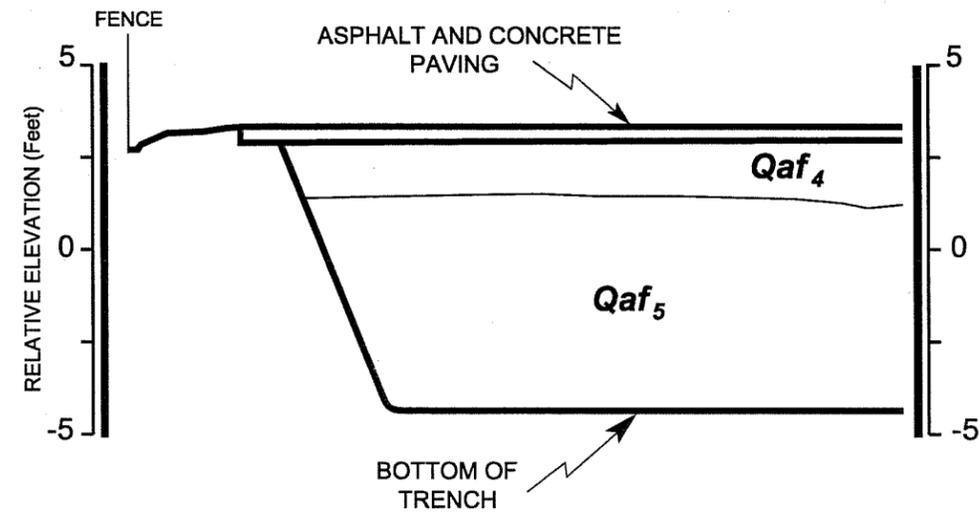
CEDAR / KETTNER
PARKING / RESIDENTIAL STRUCTURE
SAN DIEGO, CALIFORNIA

EAST

STA 140 + 00 130 + 00 120 + 00 110 + 00 100 + 00 90 + 00 80 + 00 70 + 00



STA 160 + 00 150 + 00 140 + 00



GEOCON LEGEND

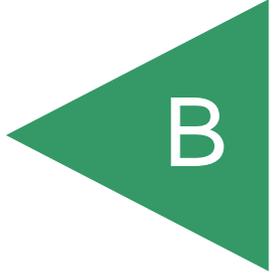
- Qaf**FILL
- Qaf₁**Loose, damp, dark, yellowish - brown, Silty, fine to coarse SAND(SM), with gravel scattered debris, including brick, glass and wood
- Qaf₂**Loose, to medium dense, damp, olive - gray, Clayey SAND(SC), scattered debris
- Qaf₃**Debris, bottles, ash, wood, wire, ceramics enclosed in a cylindrical structure lined with 1" thick concrete walls
- Qaf₄**Dense, dry, pale yellowish - brown, Silty SAND(SM) with gravel
- Qaf₅**Medium dense, moist, greenish -gray to olive - grey, Silty SAND(SM), strong organic odor
- Qal**ALLUVIUM
- Qal**Loose, damp to moist, olive - gray, Silty SAND(SM)
- Qbp**BAY POINT FORMATION
- Qbp₁**Loose, damp to moist, moderate brown to moderate yellowish - brown, Silty SAND(SM), becomes medium dense below approximately 2 to 3 feet
- Qbp₂**Medium dense, moist, mottled moderate brown, light brown, moderate and pale yellowish - brown, and light olive - gray, Clayey SAND(SC); in Trench 2, 2 approximately horizontal, parallel, non continuous, lenticular clay beds were observed. These interbeds are between 1 and 6 inches thick. In Trench 1 a discontinuous, lenticular gravel interbed was observed.
- Qbp₃**Dense, damp, mottled yellows and browns, Silty SAND(SM), partially cemented

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PROJECT NO. 06851 - 22 - 02
FIGURE A - 9
DATE 10-14-2003

SCALE 1" = 5' (VERT. = HORIZ.)

LOG OF FAULT TRENCH 2

APPENDIX



APPENDIX B

LABORATORY TESTING

Laboratory tests were performed in accordance with generally accepted test methods of the American Society for Testing and Materials (ASTM) or other suggested procedures. Selected soil samples were tested for their in-place dry density and moisture content, consolidation, shear strength, expansion, compaction, "R" Value, water-soluble sulfate, pH, and resistivity characteristics.

The results of our laboratory tests are presented on Tables B-I through B-VI and on Figure B-1. The in-place dry density and moisture content results are indicated on the exploratory boring logs.

**TABLE B-I
SUMMARY OF LABORATORY DIRECT SHEAR TEST RESULTS
ASTM D3080-98**

Sample No.	Dry Density (pcf)	Moisture Content (%)	Unit Cohesion (psf)	Angle of Shear Resistance (degrees)
B3-5	109.4	13.9	640	41
B3-9	106.8	21.7	669	34
B5-5	98.4	26.7	1213	27

**TABLE B-II
SUMMARY OF LABORATORY EXPANSION INDEX TEST RESULTS
ASTM D4829-95**

Sample No.	Moisture Content		Dry Density (pcf)	Expansion Index
	Before Test (%)	After Test (%)		
B3-11	11.5	33.3	104.1	115
T2-1	9.3	18.6	115.8	4

**TABLE B-III
SUMMARY OF LABORATORY pH AND RESISTIVITY TEST RESULTS
CALIFORNIA TEST NO. 643**

Sample No.	pH	Minimum Resistivity (ohm-centimeters)
B2-4	7.9	630

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

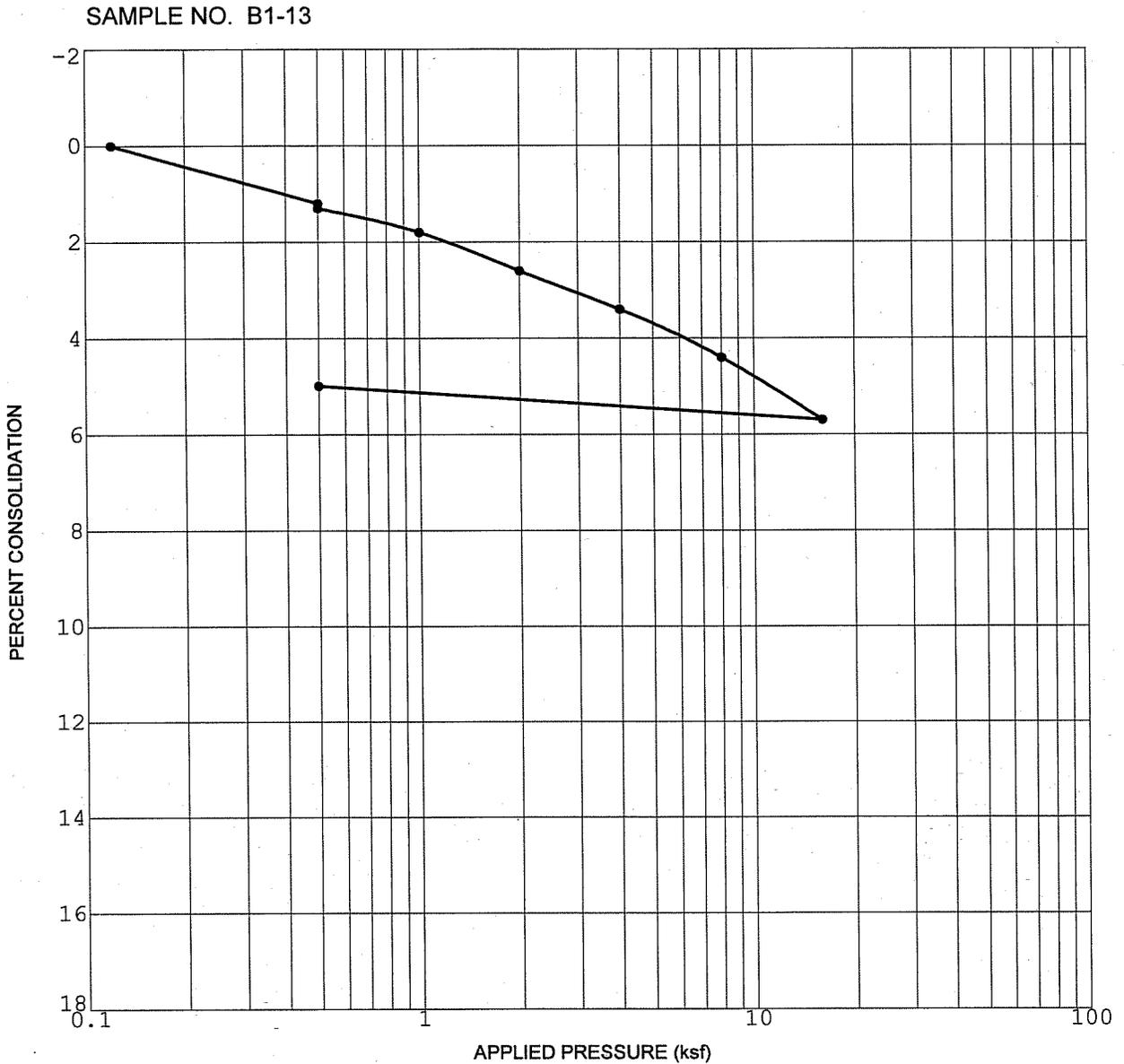
Sample No.	Water-Soluble Sulfate (%)
B2-4	0.036

**TABLE B-V
SUMMARY OF LABORATORY MAXIMUM DRY DENSITY
AND OPTIMUM MOISTURE CONTENT TEST RESULTS
ASTM D 1557-00**

Sample No.	Description	Maximum Dry Density (pcf)	Optimum Moisture Content (% dry wt.)
T2-1	Moderate brown, Silty SAND	133.0	8.2

**TABLE B-VI
SUMMARY OF LABORATORY R-VALUE TEST RESULTS
ASTM D 2844-94**

Sample No.	R-Value
T2-1	47



Initial Dry Density (kg/m3)	107.3
Initial Water Content (%)	21.1

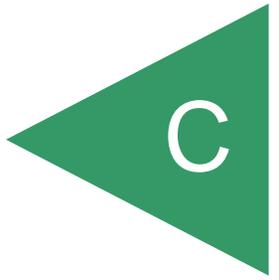
Initial Saturation (%)	100+
Sample Saturated at (ksf)	0.5

CONSOLIDATION CURVE

CEDAD/KETTNER PARKING/RESIDENTIAL STRUCTURE

SAN DIEGO, CALIFORNIA

APPENDIX



APPENDIX C

RECOMMENDED GRADING SPECIFICATIONS

FOR

CEDAR/KETTNER
PARKING/RESIDENTIAL STRUCTURE
SAN DIEGO, CALIFORNIA

PROJECT NO. 06851-22-02

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. It will be necessary that the Consultant provide adequate testing and observation services so that he may determine that, in his 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 him 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, and so forth, 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 construction 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 3/4 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 3/4 inch in maximum dimension. The quantity of fines shall be less than approximately 20 percent of the rock fill quantity.

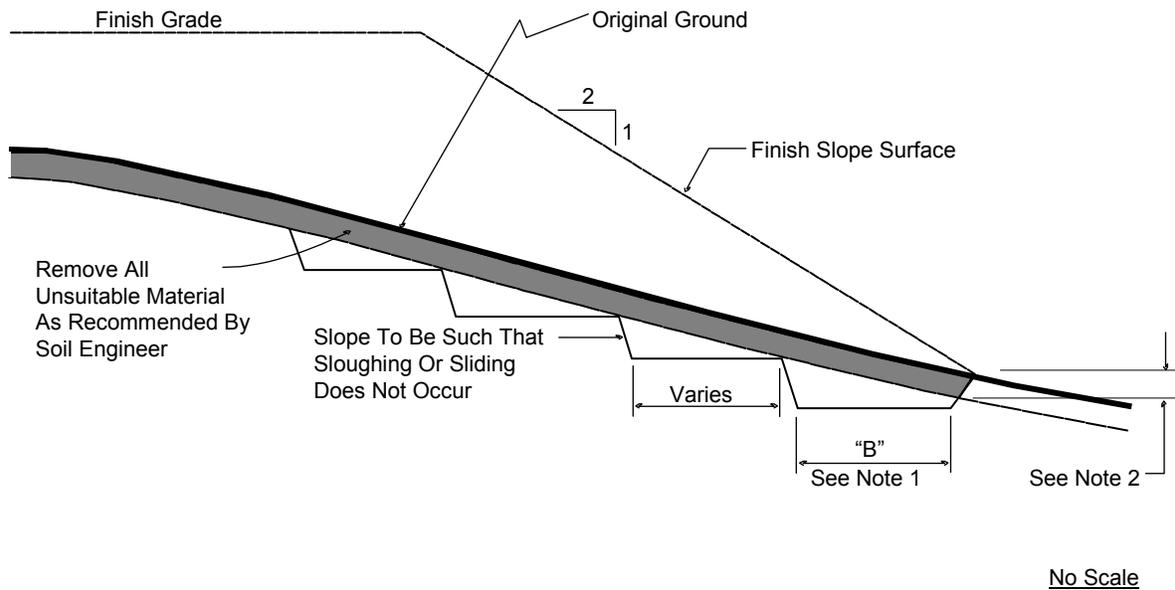
- 3.2. Material of a perishable, spongy, or otherwise unsuitable nature as determined by the Consultant shall not be used in fills.
- 3.3. Materials used for fill, either imported or on-site, shall not contain hazardous materials as defined by the California Code of Regulations, Title 22, Division 4, 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. Representative samples of soil materials to be used for fill shall 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 1-1/2 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 which 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 or other unsuitable material, loose or porous soils shall be removed to the depth recommended in the Geotechnical Report. The depth of removal and compaction shall 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 6:1 (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 wide, 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 bottom 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, plowed or scarified, the surface should be disced or bladed by the Contractor until it is uniform and free from large clods. The area should then be moisture conditioned to achieve the proper moisture content, and compacted as recommended in Section 6.0 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 D1557-00.
- 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 D1557-00. 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. Soils having an Expansion Index of greater than 50 may be used in fills if 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. All rock placement, fill placement and flooding of approved granular soil in the windrows must be continuously observed by the Consultant or his representative.
- 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, maximum slope of 5 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 will 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 D1196-93, may be performed in both the compacted *soil* fill and in the *rock* fill to aid in determining the number of passes of the compaction equipment to be performed. If performed, a minimum of three plate bearing tests shall 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 shall be present during *rock* fill operations to verify 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. In general, at least one test should be performed for each approximately 5,000 to 10,000 cubic yards of *rock* fill placed.
- 6.3.5. Test pits shall be excavated by the Contractor so that the Consultant can state that, in his 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. All *rock* fill placement shall be continuously observed during placement by representatives of the Consultant.

7. OBSERVATION AND TESTING

- 7.1. The Consultant shall be the Owners 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 shall be placed without at least one field density test being performed within that interval. In addition, a minimum of one field density test shall be performed for every 2,000 cubic yards of *soil* or *soil-rock* fill placed and compacted.
- 7.2. The Consultant shall perform random field density tests of the compacted *soil* or *soil-rock* fill to provide a basis for expressing an opinion as to 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 shall verify that the minimum number of passes have been obtained per the criteria discussed in Section 6.3.3. The Consultant shall 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. If performed, plate bearing tests will be performed randomly on the surface of the most-recently placed lift. Plate bearing tests will be performed to provide a basis for expressing an opinion as to whether the *rock* fill is adequately seated. The maximum deflection in the *rock* fill determined in Section 6.3.3 shall be less than the maximum deflection of the properly compacted *soil* fill. When any of the above criteria 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 shall 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 D1556-00, *Density of Soil In-Place By the Sand-Cone Method.*

7.6.1.2. Field Density Test, Nuclear Method, ASTM D2922-96, *Density of Soil and Soil-Aggregate In-Place by Nuclear Methods (Shallow Depth).*

7.6.1.3. Laboratory Compaction Test, ASTM D1557-00, *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 D4829-95, *Expansion Index Test.*

7.6.2. Rock Fills

7.6.2.1. Field Plate Bearing Test, ASTM D1196-93 (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.

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