GEOTECHNICAL INVESTIGATION
ISLANDER MOTEL UPGRADE
2428 CENTRAL AVENUE
Alameda, California

Resources for Community Development
Alameda, California

16 June 2011
Project No. 750604801

Treadwell&Rollo
A LANGAN COMPANY

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Mr. Brian Saliman
Resources for Community Development
2220 Oxford Street
Berkeley, CA 94704

Subject: Geotechnical Investigation
Islander Motel Upgrade
2428 Central Avenue
Alameda, California

Dear Mr. Saliman:

Treadwell & Rollo, A Langan Company, is pleased to present our geotechnical investigation report for the proposed upgrade and conversion of the Islander Motel in Alameda, California. Our work was performed in accordance with our proposal dated 4 May 2011. Report copies have been distributed as indicated on the distribution page at the end of this report.

Based on the results of our investigation we judge that the project is feasible as planned from a geotechnical standpoint. The project includes seismically retrofitting and converting an existing four-story motel building into apartments and adding two one-story buildings at the site. Additional improvements include new courtyard, patio, garden, and parking spaces.

Based on the results of our subsurface investigation, the existing ground surface is generally underlain by native Dune sand. However, the site was formerly a gas station and four fuel storage tanks were removed from the site in 1971. The fill placed as backfill within the tank excavations consists of loose and potentially liquefiable sand. The existing footings for the Motel Building consist of shallow isolated interior and a continuous perimeter footing. The new footings will likely also consist of combination of shallow continuous perimeter footings and isolated interior foundations for the proposed improvements. All foundations should bear on the native Dune sand encountered at the site. Accordingly, footings currently bearing above the potentially liquefiable fill will need to be underpinned to support the building loads during a seismic event. Details regarding these conditions and foundation alternatives for the project are included in this report.

The conclusions and recommendations contained in this report are based on a limited subsurface exploration program. Consequently, variations between expected and actual soil conditions may be found at localized areas during construction. We should be retained to observe footing excavation, installation of foundation elements, and testing of backfill, if any, during which time we may make changes in our recommendations, if deemed necessary.

We appreciate the opportunity provide our services to you on this project. If you have any questions, please call.

Sincerely yours,
TREADWELL & ROLLO, A LANGAN COMPANY

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1.0 INTRODUCTION

This report presents the results of our geotechnical investigation for the proposed Islander Motel Upgrade project in Alameda, California. The existing Islander Motel site is bordered by Central Avenue on the north, Park Avenue on the west, and residential structures on the south and east. The project site is approximately rectangular, with plan dimensions of approximately 120 by 183 feet. The location of the site is shown on the Site Location Map, Figure 1. The site is occupied by an existing motel in the northern portion of the site, with an accessory building in the southeast corner and concrete pavement throughout the rest of the site.

We understand the proposed project consists of converting the existing 62-unit motel into apartments, replacing an existing elevator and staircase, removing the existing accessory building, and adding a new office building and a new community building, as well as providing new courtyard, patio, garden, and parking spaces. We understand the new office space and community building will be architecturally integrated into the existing motel building. The accessory building will be demolished as part of this project. The locations of the proposed office space and community building are shown on the Site Plan, Figure 2. In addition, the motel building will undergo a voluntary seismic upgrade that will generally consist of adding shear walls and associated foundation elements.

Prior to construction of the Islander Motel, the project site was occupied by a Chevron service station. Gettler-Ryan, Inc. prepared a groundwater monitoring assessment report, dated 9 October 1998, regarding the former service station and documented the location of four former underground storage tanks (USTs). This work was conducted in accordance with the environmental closure of the site. These tanks were removed and subsequently backfilled prior to construction of the motel building. Based on the site plan provided by Gettler-Ryan, Inc. it appears that the four tanks were located in the vicinity of the southwestern corner of the existing motel. The general location of the four tanks is also shown on the Site Plan, Figure 2.

2.0 SCOPE OF SERVICES

Our geotechnical investigation was performed in general accordance with the scope of services included in our proposal dated 4 May 2011. Our scope of services consisted of a subsurface exploration program, which included drilling soil borings, advancing cone penetration tests (CPTs), and performing dynamic cone penetration tests (DPTs). On the basis of evaluation of the subsurface conditions at the site we performed engineering studies to develop conclusions and recommendations regarding:

- soil and groundwater conditions at the site
- appropriate foundation types to support the proposed new buildings and new foundation elements for the existing building
- design parameters for the recommended foundation type
- improvement of existing fill, as appropriate
- site seismicity and seismic hazards, including liquefaction potential
- estimated foundation settlement and settlement of surrounding improvements
- earthwork, including fill placement and utility trench backfill
- concrete flatwork and flexible pavement design
- 2010 California Building Code (CBC) soil profile type and factors
- construction considerations.

3.0 FIELD INVESTIGATION AND LABORATORY TESTING

To explore the subsurface conditions at the project site we drilled two borings and advanced two CPTs and four DPTs. The approximate locations of the borings, CPTs, and DPTs are presented in the Site Plan, Figure 2. Prior to performing the field investigation, we:

- obtained a soil boring permit from the Alameda County Department of Public Works (ACDPW)
- obtained a Right-Of-Way permit from the City of Alameda
- notified Underground Service Alert (USA)
- checked the boring and CPT locations for underground utilities using an independent utility locating contractor.

3.1 Borings

As part of our investigation, two borings, designated as B-1 and B-2, and were drilled on 23 May 2011. Borings B-1 and B-2 were drilled to depths of 41.5 and 36.5 feet, respectively, below the existing ground surface (bgs). The boring were drilled by HEW drilling of San Jose using a truck-mounted hollow-stem auger drill rig. Our field engineer logged the soil conditions encountered in the borings and obtained representative samples of the soil encountered for visual classification and laboratory testing. Logs of Borings B-1 and B-2 are presented in Appendix A on Figures A-1 thru A-2. The materials encountered were classified in accordance with the soil classification system shown on Figure A-3.

Soil samples were obtained using the Standard Penetration Test (SPT) sampler, with a 2.0-inch outside and 1.5-inch inside diameter, without liners. The SPT sampler was driven using a 140-pound, above-ground, automatic-trip safety hammer, falling about 30 inches. The samplers were driven up to 18 inches and the hammer blows required to drive the samplers every six inches of penetration were recorded and are presented on the boring logs. A "blow count" is defined as the number of hammer blows per six inches of penetration. The blow counts required to drive the SPT sampler was converted to approximate SPT N-values using a factors of 1.2 to account for the hammer energy and are shown on the boring logs. The blow counts used for this conversion were the last two blow counts of an 18 inch drive.

The borings were backfilled with cement grout in accordance with the requirements of ACDPW. Soil cuttings generated during drilling of the borings was temporarily stored on site, tested, and subsequently disposed.

3.2 Cone Penetration Tests

John Sarmiento & Associates of Orinda, California, advanced two CPTs, designated CPT-1 and CPT-2, on 25 May 2011. CPTs were performed by hydraulically pushing a 1.4-inch-diameter, cone-tipped probe into the ground. Electrical strain gauges within the cone continuously measured soil parameters during the entire depth of each probing. The accumulated data was processed by computer to provide engineering information such as the type and approximate strength characteristics of the soil encountered and used to estimate strength parameters throughout the depths explored. Both CPTs were extended to 50 feet bgs. The CPT logs showing tip resistance, friction ratio, equivalent SPT N-value, shear strength, internal friction angle, and soil behavior type are presented in Appendix B. The CPTs were backfilled in accordance with the permit obtained from ACDPH with cement grout to the ground surface.

3.3 Dynamic Cone Penetration Tests

Four dynamic cone penetration tests were advanced at the site by our field engineer on 23 May 2011. We advanced the four DPTs at the locations of the former USTs. The location of the four DPTs are presented on the Site Plan, Figure 2. The purpose of the DPTs was to evaluate the condition of the tank backfill material.

The DPTs consisted of driving a 1.4-inch-diameter, cone-tipped probe into the ground with a 35-pound hammer falling 15 inches. The blows used to drive the probe were converted to Standard Penetration Test N-values to quantitatively evaluate the strength of the backfill soil in the vicinity of the old fuel storage tanks, and are included in Appendix C.

No soil samples are obtained during the advancement of the DPT. Therefore, following advancing of the DPTs, we hand augured to a depth of about four feet at each DPT location to obtain samples of the backfill soil at each DPT location.

3.4 Laboratory Testing

Soil samples were re-examined in our office to check field classifications and to select representative samples for laboratory testing. Soil samples were tested to measure fines content and the laboratory test results are included on the boring logs. In addition, one soil sample from boring B-2 was tested for soil corrosivity. The results of this test are presented in Appendix D.

3.5 Former Borings/Wells

Following our field investigation we received a copy of the Remedial Action Completion Certification prepared by Alameda County Health Care Services dated 27 December 2011 in regards to the site closure for the removal of the former USTs. This document included the logs of monitoring wells that had been installed at the site. These logs were used as part of our evaluation of the site, where appropriate, and are included in Appendix D.

4.0 SITE AND SUBSURFACE CONDITIONS

4.1 Site Conditions

The Islander Motel site is relatively level, with ground surface elevations ranging from 27 to 28 feet[1]. The main motel building occupies the northern portion of the site and consists of a four-story structure constructed at grade in about 1971. The structure consists of a one-story concrete podium (used for at-grade parking) and three upper residential floors of timber framing. The existing motel complex also includes an L-shaped one-story accessory building located in the southeastern corner of the site.

Original foundation drawings for the motel building, prepared by Ross P. Shoaf and dated March 1971, indicate the existing motel building is supported on shallow isolated interior footings and a continuous perimeter footing. The existing perimeter footings are about 36 inches wide and 22 inches deep (beneath the existing ground surface), and the interior footings are 9½ to 10½ feet wide and 28 inches deep.

Based on the information provided in the Gettler-Ryan Inc. reports, there were four USTs tanks located in the northern portion of the site, generally located beneath the northwestern corner of the existing motel building. We understand these tanks were removed and the subsequent excavations were backfilled with soil. No information regarding compaction of backfill was available.

4.2 Subsurface Conditions

Based on the results of our investigation, the site is underlain by native loose to very dense, fine grained, sand, locally referred to as Dune sand, with the exception of the location of the former USTs. At all of the points explored, the Dune sand is loose in the upper six to eight feet and generally becomes dense below this depth. On the southern side of the site the Dune sand extended down to the maximum depth explored in our borings, 41.5 feet. However, on the northern part of the site the Dune sand was underlain by a five-foot-thick stiff clay layer at a depth of 40 feet bgs. This clay layer was in-turn underlain by dense to very dense sand to the maximum depth explored of 50 feet.

The DPTs and hand-augers were advanced at or near the approximate documented locations of the former USTs. We encountered fill in all of the DPTs. Within the upper few feet of each DPT, the fill

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1 All elevations based on "Topographic Survey of 2428 Central Avenue" by Kier & Wright Civil Engineers & Surveyors, Inc. dated May 2011; assumed datum is Mean Sea Level (NGVD 1929).

generally consisted of loose to medium dense silty sand with occasional gravel; a four-inch-thick layer of ash was encountered in DPT-2 at a depth of two feet. Based on the results of the DPTs, we judge that the fill extends up to 11½ feet beneath the existing ground surface in the former UST area.

Groundwater was encountered at depths of 13½ to 18 feet bgs in our borings. The groundwater levels measured may not represent stabilized long term levels and groundwater may fluctuate several feet due to seasonal rainfall. However, the Gettler-Ryan Inc. report presents those groundwater measurements that were taken over the course of several years, with groundwater generally fluctuating between 6.7 and 8.2 feet beneath the existing ground surface. We judge a high groundwater level of about 6.5 feet bgs (Elevation 21 feet MSL) is appropriate for design.

Given the depth of the fill in the location of the former fuel storage tanks, which is up to 11½ feet deep, several feet of the loose sandy fill encountered in our DPTs lies beneath the groundwater table.

5.0 REGIONAL SEISMICITY AND FAULTING

The major active faults in the site vicinity include the Hayward, Calaveras, San Andreas, the Mount Diablo Thrust Faults. These and other faults of the region are shown on Figure 3. For each of the active faults within 50 kilometers (km) of the site, the distance from the site and estimated Mean Characteristic Moment Magnitude[2] [2007 Working Group on California Earthquake Probabilities (WGCEP) (2007) and Cao et al. (2003)] are summarized in Table 1.

[2] Moment magnitude is an energy-based scale and provides a physically meaningful measure of the size of a faulting event. Moment magnitude is directly related to average slip and fault rupture area.

TABLE 1

Regional Faults and Seismicity

Fault Segment	Approximate Distance from Site (km)	Direction from Site	Mean Characteristic Moment Magnitude
Total Hayward	6.1	Northeast	7.00
Total Hayward-Rodgers Creek	6.1	Northeast	7.33
Total Calaveras	21	East	7.03
Mount Diablo Thrust	22	Northeast	6.70
N. San Andreas - Peninsula	23	West	7.23
N. San Andreas (1906 event)	23	West	8.05
Green Valley Connected	27	Northeast	6.80
N. San Andreas - North Coast	29	West	7.51
San Gregorio Connected	31	West	7.50
Monte Vista-Shannon	36	South	6.50
Greenville Connected	38	East	7.00
Rodgers Creek	40	Northwest	7.07
West Napa	45	North	6.70
Great Valley 5, Pittsburg Kirby Hills	45	Northeast	6.70

Figure 3 also shows the earthquake epicenters for events with magnitude greater than 5.0 from January 1800 through December 2000. Since 1800, four major earthquakes have been recorded on the San Andreas Fault. In 1836 an earthquake with an estimated maximum intensity of VII on the Modified Mercalli (MM) scale (Figure 4) occurred east of Monterey Bay on the San Andreas Fault (Toppozada and Borchardt 1998). The estimated Moment magnitude, Mw, for this earthquake is about 6.25. In 1838, an earthquake occurred with an estimated intensity of about VIII-IX (MM), corresponding to an Mw of about 7.5. The San Francisco Earthquake of 1906 caused the most significant damage in the history of the Bay Area in terms of loss of lives and property damage. This earthquake created a surface rupture along the San Andreas Fault from Shelter Cove to San Juan Bautista, approximately 470 kilometers in length. It had a maximum intensity of XI (MM), a Mw of about 7.9, and was felt 560 kilometers away in Oregon, Nevada, and Los Angeles. The most recent earthquake to affect the Bay Area was the Loma Prieta

Earthquake of 17 October 1989, in the Santa Cruz Mountains with a Mw of 6.9, approximately 87 km south of the site.

In 1868 an earthquake with an estimated maximum intensity of X on the MM scale occurred on the southern segment (between San Leandro and Fremont) of the Hayward Fault. The estimated Mw for the earthquake is 7.0. In 1861, an earthquake of unknown magnitude (probably a Mw of about 6.5) was reported on the Calaveras Fault. The most recent significant earthquake on this fault was the 1984 Morgan Hill earthquake (Mw = 6.2).

The 2007 WGCEP at the U.S. Geologic Survey (USGS) predicted a 63 percent chance of a magnitude 6.7 or greater earthquake occurring in the San Francisco Bay Area in 30 years. More specific estimates of the probabilities for different faults in the Bay Area are presented in Table 2.

TABLE 2

WGCEP (2007) Estimates of 30-Year Probability of a Magnitude 6.7 or Greater Earthquake

Fault	Probability (percent)
Hayward-Rodgers Creek	31
N. San Andreas	23
Calaveras	7
San Gregorio	6
Concord-Green Valley	3
Greenville	3
Mount Diablo Thrust	1

6.0 SEISMIC HAZARDS

Historically, ground surface ruptures closely follow traces of geologically young faults. The site is not within an Earthquake Fault Zone, as defined by the Alquist-Priolo Earthquake Fault Zoning Act, and no known active or potentially active faults exist on the site. In a seismically active area, the remote possibility exists for future faulting in areas where no faults previously existed; however, we conclude the risk of surface faulting and consequent secondary ground failure is low.

During a major earthquake on a segment of one of the nearby faults, strong to very strong shaking is expected to occur at the site. Strong ground shaking during an earthquake can result in ground failure such as that associated with soil liquefaction[3], lateral spreading[4], and cyclic densification[5]. We used the results of our exploration to evaluate the potential of these phenomena occurring at the project site. These are discussed in the following subsections.

6.1 Liquefaction

When a saturated, cohesionless soil liquefies during a major earthquake, it experiences a temporary loss of shear strength due to a transient rise in excess pore water pressure generated by strong ground motion. Flow failure, lateral spreading, differential settlement, loss of bearing strength, ground fissures, and sand boils are evidence of excess pore pressure generation and liquefaction. We used the results from our investigation to evaluate the potential for liquefaction and subsequent settlement using the methodology outlined in the Proceedings of the NCEER Workshop on the Evaluation of Liquefaction of Soils (Youd et al. 2001).

In our analyses, a peak ground acceleration (PGA) of 0.40 times gravity was used. This PGA was calculated using the procedures specified in Section 1613 of the 2010 CBC for the Design Earthquake, using Site Class D. An earthquake magnitude of 7.5 was also assumed in our analyses. In addition, we used the shallowest groundwater depths observed, 6.7 feet bgs, in the monitoring wells that were previously installed at the site in our liquefaction analyses.

Across the majority of the site, native Dune sand was encountered beneath the groundwater table. Generally speaking the Dune sand was sufficiently dense to resist liquefaction during an earthquake beneath a depth of about 6 to 7.5 feet beneath the existing ground surface. However, isolated thin layers of medium dense clayey sand were encountered at depth that could potentially liquefy during a major seismic event. Specifically, in Boring B-1 it appears that a 10-inch-thick layer of Dune sand is potentially liquefiable between the depths of 6.5 and 7.5 feet, bgs. In CPT-1 we encountered a

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3 Liquefaction is a transformation of soil from a solid to a liquefied state during which saturated soil temporarily loses strength resulting from the buildup of excess pore water pressure, especially during earthquake-induced cyclic loading. Soil susceptible to liquefaction includes loose to medium dense sand and gravel, low-plasticity silt, and some low-plasticity clay deposits.

4 Lateral spreading is a phenomenon in which surficial soil displaces along a shear zone that has formed within an underlying liquefied layer. Upon reaching mobilization, the surficial blocks are transported downslope or in the direction of a free face by earthquake and gravitational forces.

5 Cyclic Densification (also referred to as Differential Compaction) is a phenomenon in which non-saturated, cohesionless soil is densified by earthquake vibrations, causing ground surface settlement.

nine-inch-thick potentially liquefiable sand at a depth of 49.5 feet bgs. Lastly, in CPT-2 we encountered three thin potentially liquefiable sand layers at depth: a 10-inch-thick layer at a depth of 26 feet bgs, a 12-inch-thick layer at a depth of 42½ feet bgs, and an 18 inch thick layer at 48½ feet bgs. In Boring B-2 all of the material encountered below a depth of 6.5 feet bgs was sufficiently dense to resist liquefaction during a major seismic event. Because these potentially liquefiable layers are thin and capped by non-liquefiable soils, we judge the potential for loss of bearing of footings is low where the improvements are underlain by native Dune sand.

In the area of the site underlain by fill placed following the removal of the USTs, up to five feet of potentially liquefiable soil was encountered, ranging from the groundwater surface to 11.5 feet bgs. This layer is about 4½ feet below the existing and proposed footings of the building. Considering the thickness and close proximity to the bottom of new and existing footing, we judge footings bearing above this layer could experience bearing failure during a major earthquake. Special foundation provisions should be implemented, as discussed in Section 7.1.2, to limit the risk of bearing failure of footings in this area.

The liquefaction-induced settlement of the potentially liquefiable layers was estimated in accordance with the method developed by Tokimatsu and Seed (1984). Where the existing and proposed improvements bear on native Dune sand, we estimate liquefaction-induced settlement could be on the order of zero to ½ inch. However, within the portion of the site underlain by the UST backfill, we estimate ground surface settlements could be greater than one inch.

6.2 Lateral Spreading

Lateral spreading is a phenomenon in which a surficial soil displaces along a shear zone that has formed within an underlying liquefied layer. The surficial blocks are transported downslope or in the direction a free face, such as a channel, by earthquake and gravitational forces. The site and surrounding area are flat. In addition, on the basis of the results of our investigation, we did not encounter any continuous potentially liquefiable layers at the site. We therefore conclude the potential for lateral spreading beneath the site is low.

6.3 Cyclic Densification

Cyclic densification is a phenomenon in which non-saturated, cohesionless soil is compacted by earthquake vibrations, causing settlement of the ground surface. Across the site we encountered loose to medium dense Dune sand above the groundwater table. These layers could densify during a major earthquake; however, they are relatively thin. The calculated seismically induced settlement caused by seismic densification based on the data from the borings and CPTs is less than ¼ inch.

7.0 DISCUSSION AND CONCLUSIONS

From a geotechnical standpoint, we conclude the proposed project is feasible from a geotechnical standpoint, provided the recommendations presented in this report are incorporated into the project plans and specifications, and are implemented during construction.

7.1 Foundations and Settlement

We evaluated the existing foundations and have also developed criteria for the proposed foundations at the site. A discussion regarding foundation types is discussed in the following sections.

7.1.1 Motel Foundations in Native Sand

The existing foundations of the motel building are mostly bearing in loose to medium dense Dune sand about 22 to 28 inches below the existing ground surface. This material is relatively weak and will compress slightly during loading. New foundations associated with the proposed shear walls within the building may also be supported on continuous shallow foundations. The new footings will generally be at locations immediately adjacent to existing footings. To limit the potential for undermining the existing footings during construction, the new shear wall foundations should bear at the same elevation as the existing footings. When these new foundations are loaded (i.e. during an earthquake) these foundations will settle. In Section 8.4.1 we provide two bearing pressures for design of new shear wall footings; one is to limit the settlement of the foundation and the other is a maximum allowable bearing pressure. For these two pressures, we estimate total settlements on the order of ½ and 1 inch, respectively. Differential settlement between columns could be on the order of half the total settlement.

As discussed in Section 6, the site may also experience about ¼ to ¾ of an inch of settlement due to cyclic densification and liquefaction induced settlement (greater in locations underlain by the USTs backfill).

7.1.2 Motel Foundations Bearing on Existing Backfill

As discussed in Section 6.1, the existing fill within the area of the former USTs is potentially liquefiable. The exact aerial limits of this fill are not currently known; however, Figure 2 presents our estimate of the portion of the site underlain by fill associated with tank backfill. Beneath a depth of 6.7 feet, we judge portions of this fill are potentially liquefiable. The top of the potentially liquefiable fill is about 4 ½ feet from the bottom of the existing footings.

Because of the close proximity of the potentially liquefiable soils to the foundation elements, we judge the existing footings will be susceptible to significant settlement and/or loss of bearing during a major earthquake. Accordingly, the building loads within this fill zone should be transferred to the native soil below the potentially liquefiable fill material. For planning purposes, we judge the bottom of the potentially liquefiable soil could be as deep as about 11½ feet bgs.

There are several ways to effectively underpin the existing footings and transfer the vertical building loads to the deeper and non-liquefiable soil. Considering the localized area where this will be required and the limited headroom inside the existing first floor of the motel, we anticipate either hand-dug underpinning piers or helical piers will be the most cost effective. Either of these systems can also be installed beneath new shallow foundations within the portion of the project where the fill is anticipated.

Hand-dug underpinning piers are usually 30- by 48-inches in plan dimension (or larger), are excavated beneath existing foundation elements, and are shored using pressure-treated lagging as the excavation proceeds. These pits are kept relatively small and constructed in a staggered format such that only a small portion of the existing footings are unsupported at any given time. The pits will be excavated until they encounter the medium dense to dense Dune sand beneath the fill. The open piers are reinforced with steel and are filled with concrete; the top of the pier is then jacked and dry-packed to fit tightly with the base of the existing foundations. The piers should act in end bearing in the bearing strata below the existing fill. Underpinning piers will likely encounter groundwater below a depth of about 6.5 feet bgs; dewatering will be required.

Helical piers generally consist of a square or circular bar, 3 to 5 inches in diameter, with a single 12-inch-diameter circular helix near the bottom of the bar. These piers will be installed adjacent to the existing footings. The pier/bar is then advanced with torque into the ground until the pier has extended into the zone beneath the potentially liquefiable and an appropriate torque is recorded during installation.

The installation can include several sections of steel bar if required to reach the appropriate bearing strata. The steel bar is then cut off at the foundation depth and the bar is secured to the existing foundation with a steel bracket that is bolted to the foundation. The connection between the helical pier and the existing footing is then typically encapsulated in concrete.

7.1.3 Foundations for New Improvements

The new one-story office building, community building, staircase, and elevator are relatively lightly loaded. We judge the most cost effective and appropriate foundation for these structures is a combination of shallow isolated interior footings and continuous perimeter footings. Where these improvements lie with the area of the former underground storage tanks, the criteria discussed above should be implemented such that no new foundation elements rely on the existing fill for foundation support.

For footings designed in accordance with the recommendations presented Section 8.4.2 of this report, we estimate total and differential settlements due to static dead plus live loads will be less than 1- and ½-inch, respectively. The majority of this settlement will likely occur during construction. As discussed above in Section 6, during an earthquake additional settlement about ¼ to ¾ inch may occur across the site during a significant seismic event.

7.2 Ground Improvement

As discussed above, the backfill in the former UST locations is potentially liquefiable and foundations should not bear above this stratum. We evaluated several types of ground improvement techniques which could be used improve the strength of the existing fill material to limit the risk of liquefaction. Because the existing motel building bears on this stratum, techniques to vibrate and densify the fill (such as Rapid Impact Compaction or stone columns) were discounted because of the potential for settlement of the existing motel building. The potentially liquefiable soil does not have sufficient overburden (i.e. depth) for compaction grouting to be viable. Lastly, permeation grouting was discounted because the fill material is likely too variable to have the permeation grout evenly spread out through the soil matrix prior to solidification. We therefore conclude that ground improvement techniques to improve the existing fill are not economically viable for this project.

7.3 Construction Considerations

In general native Dune sand and/or sandy fill will likely be encountered during construction activities, which can be readily excavated with conventional excavation equipment. However, in DPT-2 we encountered a thin zone where we had very high blow counts within the fill. This could be indicative of some debris or rubble in the fill matrix. The presence of debris or rubble may cause difficulty during construction, including during helical pier installation.

In addition, if excavations deeper than 6.5 feet are planned (i.e. for the new elevator pit or hand-dug underpinning piers) groundwater will likely be encountered. Pits can be locally dewatered using sumps and pumps but significant groundwater inflow into excavations should be anticipated.

7.4 Soil Corrosivity

A soil corrosivity test was performed on a sample of near-surface soil obtained from Boring B-2. The results of this test are presented in Appendix D. The architect and structural engineer should review the results of the test and make structural design changes as appropriate for the project to account for the soil corrosivity.

8.0 RECOMENDATIONS

Our recommendations regarding site preparation, grading, foundation design, seismic design, and other geotechnical aspects of this project are presented in the remainder of this section.

8.1 Site Preparation and Grading

Demolition of the existing accessory building and other improvements for the new development will include the removal of existing concrete pavements, utility lines, and existing foundations. All voids resulting from demolition activities should be properly backfilled with engineered fill as described later in this section. Existing foundations should be completely removed, and the resulting void space should be backfilled with engineered fill.

Where utilities that are removed extend off site, they should be capped or plugged with grout at the property line. It may be feasible to abandoned utilities in-place by filling them with grout, provided they will not impact future utilities or building foundations. The utility lines, if encountered, should be addressed on a case-by-case basis.

From a geotechnical standpoint, asphalt and concrete generated by demolition activities may be crushed and reused provided it is free of organic material and rocks or lumps greater than four inches in greatest dimension. The acceptability of using crushed asphalt (from an environmental standpoint) at the site should be verified by the property owner and architect. Where crushed concrete or asphalt pavement materials are used, particles between 1-1/2 and 4 inches in greatest dimension should comprise no more than 30 percent of the fill by weight. The criteria for acceptable fill are discussed in Section 7.1.2.

We anticipate the fill placement for this project will generally be localized and will consist of:

- backfill following the removal of existing utilities or foundation elements
- backfill around new foundations, as required
- utility trench backfill.

All fill should consist of soil and/or crushed asphalt and concrete that is non-corrosive, non-hazardous, free of organic matter or other deleterious material, contains no rocks or lumps larger than four inches in greatest dimension, has a low expansion potential (defined by a liquid limit of less than 40 and a plasticity index lower than 12), and is approved by the Geotechnical Engineer. Soil excavated during construction will generally be acceptable for use as general fill and backfill. Fill should be placed in horizontal lifts not exceeding eight inches in loose or uncompacted thickness, moisture-conditioned to near optimum moisture content, and compacted to at least 90 percent relative compaction[6]. Clean sand or gravel (defined as soil with less than 10 percent fines by weight) used as fill should be compacted to at least 95 percent relative compaction. If fill is located in pavement areas that will receive vehicular traffic, the upper six inches of the subgrade should be compacted to at least 95 percent relative compaction to achieve a firm, non-yielding subgrade. Jetting of backfill should not be permitted.

Bulk samples of proposed fill material, including on-site fill, should be submitted to the geotechnical engineer for approval at least three working days before it is used on site. For imported fill, the grading subcontractor should provide analytical test results or other suitable environmental documentation indicating the proposed fill material is free of hazardous materials at least three days before use at the site. If this data is not provided, up to two weeks may be required to perform any required analytical testing on proposed import soil.

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6 Relative compaction refers to the in-place dry density of soil expressed as a percentage of the maximum dry density of the same material, as determined by the ASTM D1557-07 laboratory compaction procedure.

8.2 Utilities and Utility Trench Backfill

Utility trenches should be excavated a minimum of four inches below the bottom of pipes or conduits and have clearances of at least six inches on both sides. Excavations for utility trenches can be readily made with a backhoe. All trenches should conform to the current CAL-OSHA requirements.

To provide uniform support, pipes or conduits should be bedded on a minimum of four inches of sand or fine gravel. After pipes and conduits are tested, inspected (if required), and approved, they should be covered to a depth of six inches with moisture conditioned sand or fine gravel, which should then be mechanically tamped.

Backfill for utility trenches and other excavations is also considered fill, and should be placed and compacted according to the recommendations previously presented in Section 8.1. Jetting of trench backfill should not be permitted. Special care should be taken when backfilling utility trenches within the building footprint and pavement areas. Special care should be taken when backfilling utility trenches in pavement areas. Poor compaction may cause excessive settlements, resulting in damage to the pavement section. Backfill for utilities or tie-ins that extend beneath tidy sidewalk and streets should be performed per City of Alameda Specifications.

Temporary dewatering may be required during construction if excavations extend below the groundwater table.

8.3 Excavation and Cut Slopes and Temporary Cut Slopes

Installation of the new foundation elements will require excavation in sandy soil. Unsupported temporary cut slopes in fill or native Dune sand should be no steeper than 1½:1 (horizontal to vertical).
The contractor should be familiar with applicable local, state, and federal regulations for unshored excavations, including the current OSHA Excavation and Trench Safety Standards. The contractor should be solely responsible for the maintaining a safe working environment inside and around all excavations made on-site. Where space does not permit or where excavations extend beneath the groundwater table shoring and dewatering will be required.

With the exception of underpinning pits, we do not anticipate any shoring will be required. If needed, we can provide detailed recommendations for temporary shoring at a later date.

8.4 Foundations

The proposed structures (office building, community room, staircase, and elevator) may be supported on shallow isolated interior and continuous perimeter footings bearing in native Dune sand. New and existing footings within the portion of the site underlain by UST backfill (see Figure 2) should be underpinned or otherwise gain support in the dense native Dune sand beneath the fill. Underpinning is discussed further in the following section.

8.4.1 Existing Motel Footings and New Shear Wall Footings

The existing footings consist of isolated footings beneath isolated columns and a continuous footing beneath the perimeter walls. The existing footings are bearing in loose to medium dense Dune sand about 22 to 28 inches below the existing ground surface. New shear wall footings are planned immediately adjacent to these existing footings. New shear wall footings should be bear at the same elevation as the adjacent existing footings.

The new shear wall footings will be continuous footings and should have a minimum width of 24 inches. Continuous foundations may be designed using an allowable bearing pressure of 3,000 psf for dead plus live loads and may be increased by 1/3 for total loads, including wind or seismic loads. However, as discussed in Section 7.1.1 the new footings will likely settle on the order of one inch to mobilize this bearing capacity. If desired reduce the amount of anticipated settlement to less than ½ inch, a bearing pressure of 1,500 psf should be used for design of the new shear wall footings.

Installation of the new shear wall footings will require the excavation of overburden soil currently confining the existing foundations. Where this condition exists, a temporary reduction of the bearing capacity of the soil beneath the existing footings may occur during the excavation of adjacent overburden soil. Therefore, where excavations are planned adjacent to existing foundations, the existing footings will need to be evaluated for this 'construction condition' using a reduced bearing capacity of 1,500 psf. Excavations should not be permitted below the depth of nearby footings with the exception of shored underpinning, which should only be performed in narrow segments.

8.4.2 New Foundations for New Improvements

The proposed new improvements, including the new staircase, elevator, and new buildings may also be supported on continuous perimeter and isolated interior spread footings bearing on native Dune sand. We recommend new footings be designed using an allowable bearing pressure of 3,000 psf for dead plus live load conditions. This value can be increased by one-third for total loads, including wind or seismic forces. Continuous and isolated footings should be at least 18 inches wide and 24 inches square, respectively. Continuous and isolated footings should be bottomed at least 30 and 36 inches, respectively, below the lowest adjacent soil subgrade. Where new footings for the proposed improvements will abut existing footings of the motel building, the depth of the new footing should match the existing. As space permits, the new footing should then be stepped down to the minimum depths presented above at an overall inclination of about 30 degrees, or flatter, from the horizontal. Footing designed using an allowable bearing pressure of 3,000 psf are estimated to settle one inch. If desired to reduce the settlement to about ½ inch the allowable bearing pressure should be reduced to 1,500 psf.

Footings adjacent to utility trenches or other footings should bear below an imaginary 30-degree line projected upward from the bottom edge of the utility trench or adjacent footings. Where excavations are planned adjacent to existing foundations, the existing footings will need to be evaluated using a reduced bearing capacity of 1,500 psf.

8.4.3 Existing and New Foundations in Existing UST Backfill

As discussed above in Section 7.1.2, the existing fill is potentially liquefiable and not capable of supporting the vertical loads of existing or new foundations during an earthquake. For the static condition, footings may be designed in accordance with the recommendations above for existing footings and the new shear wall footings in section 8.2.1. However, during an earthquake all of the vertical load from these footings should be transferred to the dense sand strata beneath the potentially liquefiable fill. Recommendations for hand-dug and helical piers are presented following subsections. The existing or new footings cast on either of these types of piers should be capable of spanning between pier elements.

<u>8.4.3.1 Hand-Dug Piers</u>

For vertical resistance, hand-dug underpinning piers should act in end-bearing at a depth of at least one foot into the dense Dune sand beneath the fill. For planning purposes, a total depth of 12½ feet should be estimated for all piers within the portion of the site underlain by fill. End bearing hand-excavated piers extending into dense to Dune sand may be designed using an allowable bearing pressure of 7,000 psf for dead plus live loads. This allowable bearing capacity can be used provided the underpinning pits are clean of all loose soil and the pits can be physically inspected. We should observe the subgrade of the underpinning pier excavations to check that it is properly cleaned and can support the design pressure. In addition, if dense Dune sand is encountered at higher elevations than planned, underpinning pits can be terminated at shallower depths.

The width of the underpinning piers should be determined by the structural engineer or underpinning designer based on the ability of the existing foundation to span an area of non-support. However, typically pits are smaller than 5 feet wide. To reduce the potential movement of the existing motel building and provide adequate foundation support during installation of the underpinning piers, only a small percentage of the existing footings should be unsupported at any given time. For underpinning of the continuous footing, the installation should be staggered/staged such that adjacent piers are not excavated until the first stage of piers have cured. In addition, no more than 25 percent of the footing length should be unsupported at any given time.

For isolated footings requiring underpinning, the piers should be installed in four sections, one under each corner of the existing footings. At no time should more than 25 percent of the isolated footings be unsupported during construction.

All underpinning piers also be preloaded (jacked) prior to dry packing to reduce settlement as the foundation load is transferred to the piers.

8.4.3.2 Helical Piers

Helical Piers are typically designed and installed by specialty contractors. The piers should be advanced at least two feet into the dense Dune sand beneath the fill. For planning purposes, we estimate a minimum helix diameter of 12 inches. For a 12-inch-diameter helical pier embedded at least two feet into dense sand, we estimate an allowable capacity in end-bearing of about 9 kips. Based on literature provided by Chance Engineering, a manufacture of helical piers, the helical pier should be also have a minimum of 1,800 foot-pounds of torque at the end of the pier installation to achieve this capacity.
If this torque is not achieved, the pier should be deepened or additional helices may be added to the bond zone beneath the fill.

During the installation of the helical piers we should be on site to observe the installation process and confirm the appropriate depth and torque is achieved during pier installation.

8.2.4 Lateral Resistance

Lateral loads on footings bearing on native soil can be resisted by a combination of passive pressure on the vertical faces of the footings friction along their bases. Passive resistance may be calculated using an equivalent fluid weight of 240 pounds per cubic foot (pcf) in either engineered fill or native Dune sand. The upper one foot of soil should be ignored unless it is confined by a slab or pavement. Friction at the base of the footings may be calculated using a base fiction of 0.3 times the dead load of the structure. The passive pressure and frictional coefficient include a factor of safety of at least 1.5 and can be used in combination without reduction.

Within the portion of the site underlain by fill, lateral loads may be resisted using a reduced passive pressure acting on the vertical faces of the footings. Passive resistance in this portion of the site may be calculated using an equivalent fluid weight of 180 pcf. The upper one-foot of soil should be ignored unless it is confined by a slab or pavement. Because the hand-dug or helical piers extend into the potentially liquefiable soil, they should not be relied upon to provide lateral support for the building. If necessary, the lateral loads from this portion of the building may be transmitted to other foundation elements outside the limits of the existing fill.

8.4.5 Foundation Installation

We should observe the bottom of new footing excavations during foundation installation to check that the soil is as anticipated in this report. If loose or weak soil is encountered in the foundation excavations, it should be over excavated and replaced with engineered fill or lean concrete, except where underpinning is provided. Following excavation, the footing excavations should be kept moist. The footing excavations should be free of standing water, debris, and disturbed materials prior to placing concrete. We should check footing excavations prior to placement of reinforcing steel and again before concrete is placed.

8.5 Floor Slabs

Floor slabs within the UST backfill area will likely experience significant settlement and may potentially be damaged during an earthquake.

Moisture is likely to condense on the underside of the floor slabs, even though it will be above the groundwater table. In areas with sensitive flooring or where water vapor would be problematic, we recommend installing a capillary moisture break and a water vapor retarder beneath the slab-on-grade floors. A capillary moisture break consists of at least four inches of clean, free-draining gravel or crushed rock. The vapor retarder should meet the requirements for Class C vapor retarders stated in ASTM E1745-97 and should be placed in accordance with the requirements of ASTM E1643-98. These requirements include overlapping seams by six inches, taping seams, and sealing penetrations in the vapor retarder. The vapor retarder should be covered with two inches of sand to aid in curing the concrete and protect the vapor retarder during slab construction. The particle size of the gravel/crushed rock and sand should meet the gradation requirements presented in Table 3.

TABLE 3

Gradation Requirements for Capillary Moisture Break

Sieve Size	Percentage Passing Sieve
Gravel or Crushed Rock	
1 inch	100
3/4 inch	30-75
1/2 inch	5–10
3/8 inch	0-2
Sand	
No. 4	100
No. 200	0-5

The sand overlying the membrane should be dry at the time concrete is placed. Excess water trapped in the sand could eventually be transmitted as vapor through the slab. If rain is forecast prior to pouring the slab, the sand should be covered with plastic sheeting to avoid wetting. If the sand becomes wet, concrete should not be placed until the sand has been dried or replaced.

Concrete mixes with high water/cement ratios result in excess water in the concrete, which increases the cure time and result in excessive vapor transmission through the slab. Therefore, concrete for the floor slab should have a low water/cement ratio of less than 0.50. If approved by the project structural engineer, the sand can be eliminated and the concrete can be placed directly over the vapor retarder, provided the water/cement ratio of the concrete does not exceed 0.45 and water is not added in the field. If necessary, workability should be increased by adding plasticizers. In addition, the slab should be properly cured.

Before the floor covering is placed, the contractor should check that the concrete surface and the moisture emission level (if emission testing is required) meet the manufacturer's requirements.

8.6 Flexible Pavement Design

The State of California flexible pavement design method was used to develop the recommended asphalt concrete pavement sections. We expect the final soil subgrade in asphalt-paved areas will generally consist native sand and/or silty or clayey sand. Based on our experience with similar soil, we selected an R-value of 25 for design. If fill is placed in the area that will lie beneath paved areas (i.e. where the existing accessory building will be demolished), the fill material should have an R-value of at least 25. Additional tests may be performed during construction to confirm the use of a higher R-value, if deemed appropriate. Depending on the results of the tests, the pavement design can be revised.

We understand the proposed pavement at the site will consist of a drive isle and parking spaces. Recommended pavement sections for several traffic indices are presented in Table 4. The appropriate traffic indices (TIs) for the new pavement should be selected by the project civil engineer.

TABLE 4

Asphaltic Concrete Pavement Section Design

TI	Asphaltic Concrete (inches)[1]	Class 2 Aggregate Base[2] (inches)[3]
4.5	2.5	6.5
5.0	3.0	7.0
5.5	3.0	8.0

Notes:
1. Asphaltic Concrete should have a minimum thickness of 2.5 inches
2. Class 2 Aggregate Base material should have a minimum R-Value of 78
3. Class 2 Aggregate Base should have a minimum thickness of 6 inches

Pavement components should conform to the current Caltrans Standard Specifications. The upper six inches of the soil subgrade in pavement areas should be moisture-conditioned to above optimum and compacted to at least 95 percent relative compaction and rolled to provide a smooth non-yielding surface. Aggregate base should be compacted to at least 95 percent relative compaction.

8.7 2010 California Building Code Mapped Seismic Values

For seismic design in accordance with the provisions of 2010 California Building Code, we judge the site is Site Class D. Although we encountered some localized potentially liquefiable material within the fill and deep native sand, the layers appeared to be thin, isolated, and discontinuous. We judge there is insufficient liquefaction potential at the site to designate it Site Class F. Accordingly, we for seismic design in accordance with the 2010 San Francisco Building Code (CBC) we recommend the following:

- Maximum Considered Earthquake (MCE) S_S and S_1 of 1.50g and 0.60g, respectively
- Site Class D
- Site Coefficients F_A and F_V of 1.0 and 1.5
- Maximum Considered Earthquake (MCE) spectral response acceleration parameters at short periods, SMS, and at one-second period, SM_1, of 1.50g and 0.90g, respectively
- Design Earthquake (DE) spectral response acceleration parameters at short period, SD_S, and at one-second period, SD_1, of 1.00g and 0.60g, respectively.

9.0 GEOTECHNICAL SERVICES DURING CONSTRUCTION

Prior to construction, Treadwell & Rollo, a Langan Company should review the project plans and specifications to check that they conform with the intent of our recommendations. During construction, our field engineer should provide on-site observation and testing during installation of building foundations, underpinning, earthwork, and preparation of pavement subgrade and placement of aggregate base. These observations will allow us to compare actual with anticipated soil conditions and to check that the contractor's work conforms with the geotechnical aspects of the foundation drawings.

10.0 LIMITATIONS

The conclusions and recommendations presented in this report result from limited subsurface investigation. Actual subsurface conditions may vary. If any variations or undesirable conditions are encountered during construction, or if the proposed construction will differ from that described in this report, Treadwell & Rollo, A Langan Company should be notified so that supplemental recommendations can be made.

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FIGURES

Base map: The Thomas Guide
Alameda County
2002

0 2000 Feet
Approximate scale

ISLANDER MOTEL UPGRADE
Alameda, California

Treadwell&Rollo
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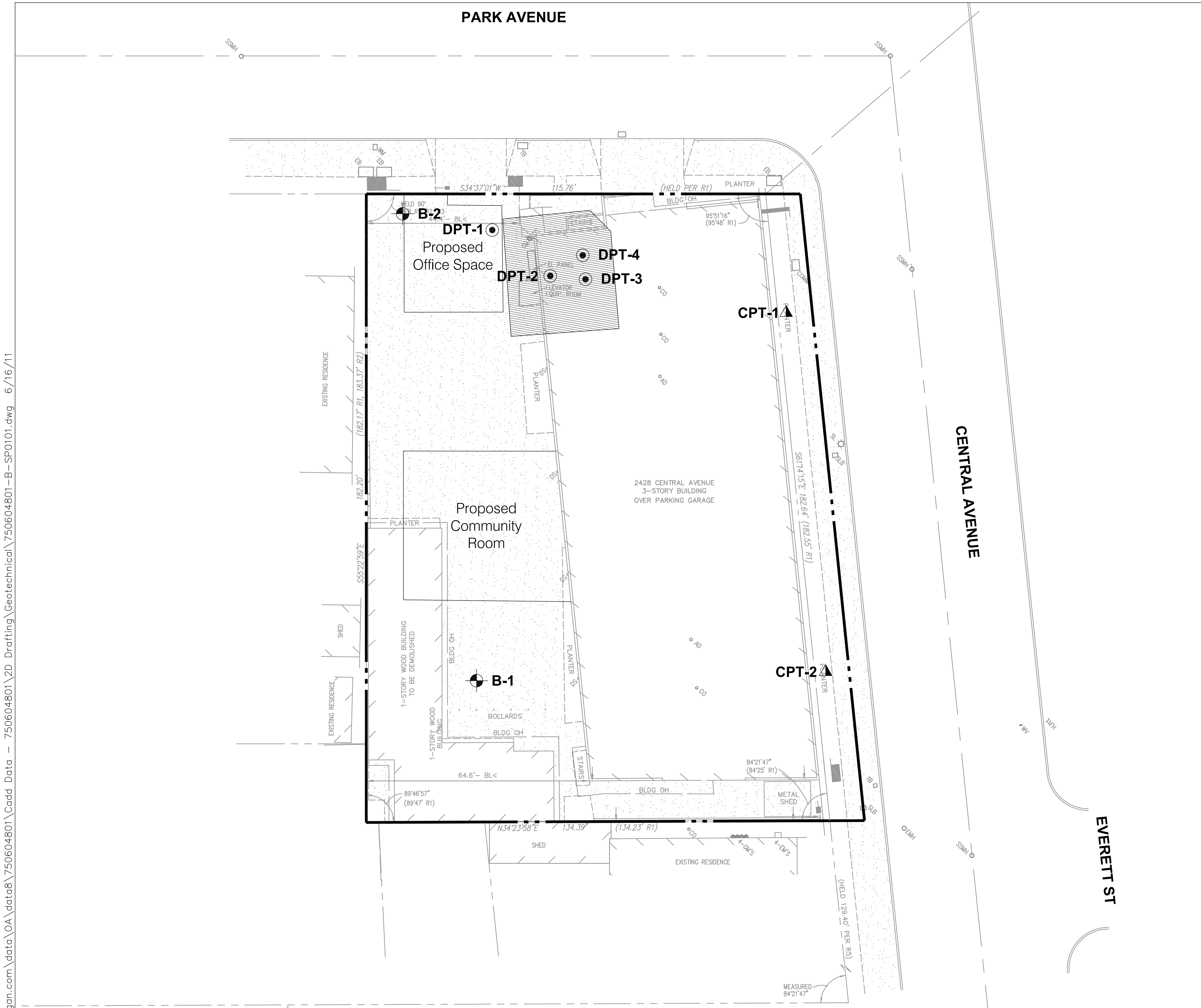
SITE LOCATION MAP

Date 06/08/11	Project No. 750604801	Figure 1



EXPLANATION

B-1 — Approximate location of boring by Treadwell & Rollo, Inc., May 2011

CPT-1 — Approximate location cone penetration test by Treadwell & Rollo, May 2011

DPT-1 — Approximate location of dynamic penetration test by Treadwell & Rollo, May 2011

Approximate location of former underground storage tanks

Property boundary

0 30 Feet

Approximate scale

Reference: Base Map from a drawing titled "Alta/ Acsm Land Title Survey of 2428 Central Avenue for the Housing Authority of the City of Alameda,", by Kier & Wright, dated 05/11.

ISLANDER MOTEL UPGRADE
Alameda, California

SITE PLAN

Date 05/26/11	Project No. 750604801	Figure 2

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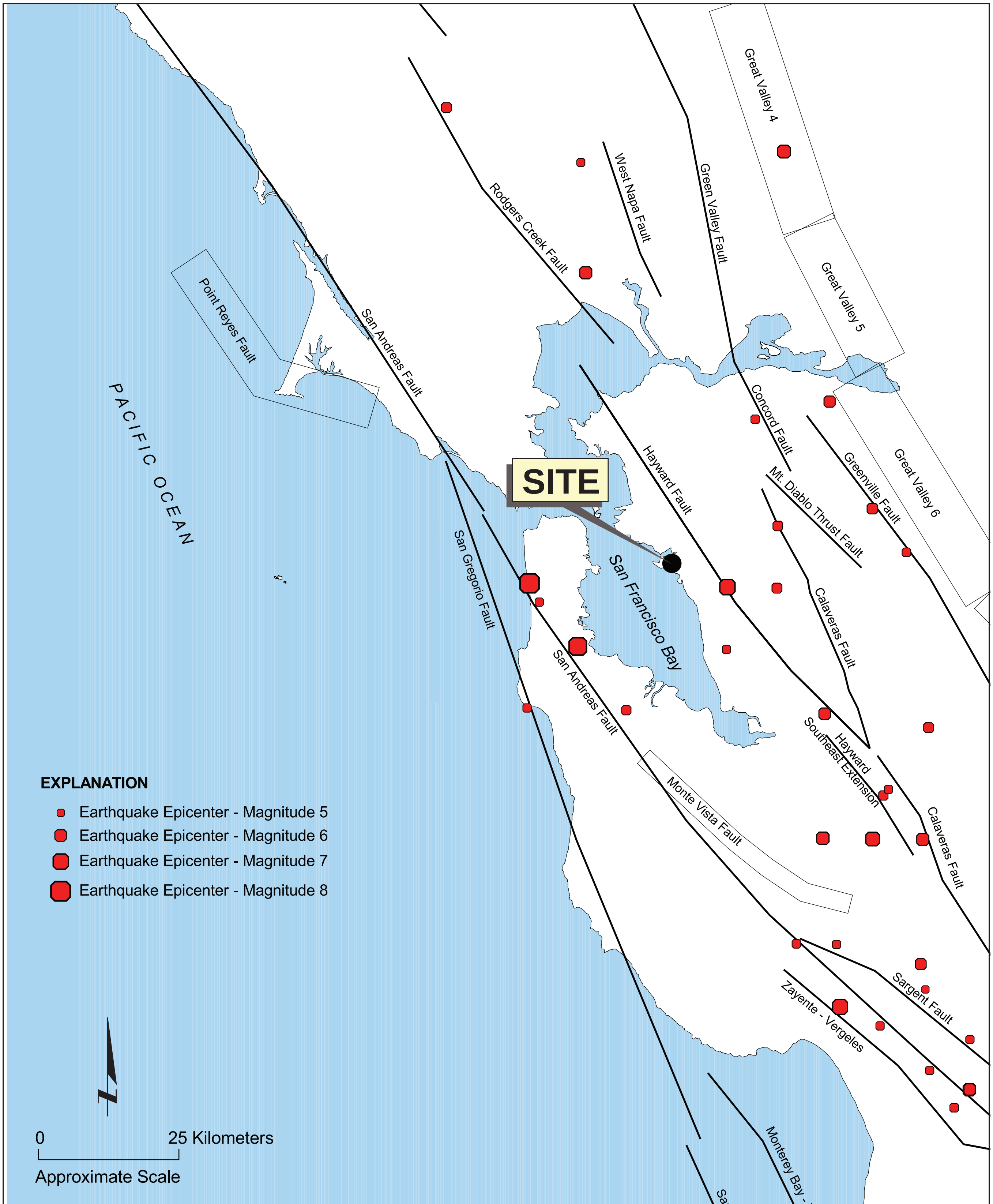

NOTES:
Digitized data for fault coordinates and earthquake catalog was developed by the California Department of Conservation Division of Mines and Geology. The historic earthquake catalog includes events from January 1800 to December 2000.

ISLANDER MOTEL UPGRADE
Alameda, California

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MAP OF MAJOR FAULTS AND EARTHQUAKE EPICENTERS IN THE SAN FRANCISCO BAY AREA

Date 06/15/11 | Project No. 750604801 | Figure 3

I **Not felt by people, except under especially favorable circumstances. However, dizziness or nausea may be experienced.**
Sometimes birds and animals are uneasy or disturbed. Trees, structures, liquids, bodies of water may sway gently, and doors may swing very slowly.

II **Felt indoors by a few people, especially on upper floors of multi-story buildings, and by sensitive or nervous persons.**
As in Grade I, birds and animals are disturbed, and trees, structures, liquids and bodies of water may sway. Hanging objects swing, especially if they are delicately suspended.

III **Felt indoors by several people, usually as a rapid vibration that may not be recognized as an earthquake at first. Vibration is similar to that of a light, or lightly loaded trucks, or heavy trucks some distance away. Duration may be estimated in some cases.**
Movements may be appreciable on upper levels of tall structures. Standing motor cars may rock slightly.

IV **Felt indoors by many, outdoors by a few. Awakens a few individuals, particularly light sleepers, but frightens no one except those apprehensive from previous experience. Vibration like that due to passing of heavy, or heavily loaded trucks. Sensation like a heavy body striking building, or the falling of heavy objects inside.**
Dishes, windows and doors rattle; glassware and crockery clink and clash. Walls and house frames creak, especially if intensity is in the upper range of this grade. Hanging objects often swing. Liquids in open vessels are disturbed slightly. Stationary automobiles rock noticeably.

V **Felt indoors by practically everyone, outdoors by most people. Direction can often be estimated by those outdoors. Awakens many, or most sleepers. Frightens a few people, with slight excitement; some persons run outdoors.**
Buildings tremble throughout. Dishes and glassware break to some extent. Windows crack in some cases, but not generally. Vases and small or unstable objects overturn in many instances, and a few fall. Hanging objects and doors swing generally or considerably. Pictures knock against walls, or swing out of place. Doors and shutters open or close abruptly. Pendulum clocks stop, or run fast or slow. Small objects move, and furnishings may shift to a slight extent. Small amounts of liquids spill from well-filled open containers. Trees and bushes shake slightly.

VI **Felt by everyone, indoors and outdoors. Awakens all sleepers. Frightens many people; general excitement, and some persons run outdoors.**
Persons move unsteadily. Trees and bushes shake slightly to moderately. Liquids are set in strong motion. Small bells in churches and schools ring. Poorly built buildings may be damaged. Plaster falls in small amounts. Other plaster cracks somewhat. Many dishes and glasses, and a few windows break. Knickknacks, books and pictures fall. Furniture overturns in many instances. Heavy furnishings move.

VII **Frightens everyone. General alarm, and everyone runs outdoors.**
People find it difficult to stand. Persons driving cars notice shaking. Trees and bushes shake moderately to strongly. Waves form on ponds, lakes and streams. Water is muddied. Gravel or sand stream banks cave in. Large church bells ring. Suspended objects quiver. Damage is negligible in buildings of good design and construction; slight to moderate in well-built ordinary buildings; considerable in poorly built or badly designed buildings, adobe houses, old walls (especially where laid up without mortar), spires, etc. Plaster and some stucco fall. Many windows and some furniture break. Loosened brickwork and tiles shake down. Weak chimneys break at the roofline. Cornices fall from towers and high buildings. Bricks and stones are dislodged. Heavy furniture overturns. Concrete irrigation ditches are considerably damaged.

VIII **General fright, and alarm approaches panic.**
Persons driving cars are disturbed. Trees shake strongly, and branches and trunks break off (especially palm trees). Sand and mud erupts in small amounts. Flow of springs and wells is temporarily and sometimes permanently changed. Dry wells renew flow. Temperatures of spring and well waters varies. Damage slight in brick structures built especially to withstand earthquakes; considerable in ordinary substantial buildings, with some partial collapse; heavy in some wooden houses, with some tumbling down. Panel walls break away in frame structures. Decayed pilings break off. Walls fall. Solid stone walls crack and break seriously. Wet grounds and steep slopes crack to some extent. Chimneys, columns, monuments and factory stacks and towers twist and fall. Very heavy furniture moves conspicuously or overturns.

IX **Panic is general.**
Ground cracks conspicuously. Damage is considerable in masonry structures built especially to withstand earthquakes; great in other masonry buildings - some collapse in large part. Some wood frame houses built especially to withstand earthquakes are thrown out of plumb, others are shifted wholly off foundations. Reservoirs are seriously damaged and underground pipes sometimes break.

X **Panic is general.**
Ground, especially when loose and wet, cracks up to widths of several inches; fissures up to a yard in width run parallel to canal and stream banks. Landsliding is considerable from river banks and steep coasts. Sand and mud shifts horizontally on beaches and flat land. Water level changes in wells. Water is thrown on banks of canals, lakes, rivers, etc. Dams, dikes, embankments are seriously damaged. Well-built wooden structures and bridges are severely damaged, and some collapse. Dangerous cracks develop in excellent brick walls. Most masonry and frame structures, and their foundations are destroyed. Railroad rails bend slightly. Pipe lines buried in earth tear apart or are crushed endwise. Open cracks and broad wavy folds open in cement pavements and asphalt road surfaces.

XI **Panic is general.**
Disturbances in ground are many and widespread, varying with the ground material. Broad fissures, earth slumps, and land slips develop in soft, wet ground. Water charged with sand and mud is ejected in large amounts. Sea waves of significant magnitude may develop. Damage is severe to wood frame structures, especially near shock centers, great to dams, dikes and embankments, even at long distances. Few if any masonry structures remain standing. Supporting piers or pillars of large, well-built bridges are wrecked. Wooden bridges that "give" are less affected. Railroad rails bend greatly and some thrust endwise. Pipe lines buried in earth are put completely out of service.

XII **Panic is general.**
Damage is total, and practically all works of construction are damaged greatly or destroyed. Disturbances in the ground are great and varied, and numerous shearing cracks develop. Landslides, rock falls, and slumps in river banks are numerous and extensive. Large rock masses are wrenched loose and torn off. Fault slips develop in firm rock, and horizontal and vertical offset displacements are notable. Water channels, both surface and underground, are disturbed and modified greatly. Lakes are dammed, new waterfalls are produced, rivers are deflected, etc. Surface waves are seen on ground surfaces. Lines of sight and level are distorted. Objects are thrown upward into the air.

ISLANDER MOTEL UPGRADE Alameda, California	**MODIFIED MERCALLI INTENSITY SCALE**
Treadwell&Rollo A LANGAN COMPANY	Date 06/15/11 | Project No. 750604801 | Figure 4

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APPENDIX A
Logs of Borings

PROJECT:	**ISLANDER MOTEL UPGRADE** Alameda, California	**Log of Boring B-1** PAGE 1 OF 2

Boring location: See Site Plan, Figure 2

Logged by: W. Stegerstrom

Date started: 5/23/11 | Date finished: 5/23/11

Drilling method: Hollow Stem Auger

Hammer weight/drop: 140 lbs./30 inches | Hammer type: Automatic

Sampler: Standard Penetration Test (SPT)

Ground Surface Elevation: ~27 foot[2]

DEPTH (feet)	Sampler Type	Blows/ 6"	SPT N-Value[1]	LITHOLOGY	MATERIAL DESCRIPTION	Type of Strength Test	Confining Pressure Lbs/Sq Ft	Shear Strength Lbs/Sq Ft	Fines %	Natural Moisture Content, %	Dry Density Lbs/Cu Ft
0					5 inches Concrete						
1				SP	SILTY SAND (SP) trace fines, trace gravel						
2–3	SPT	3 3 3	7	SM	SILTY SAND (SM) yellow-brown to brown, loose, dry to moist, trace rootlets, trace silt, medium grained				14.1		
5–6	SPT	3 3 3	7	SM					16.0		
6.5				SM	with clayey interbeds						
7–8	SPT	7 9 16	30	SC	CLAYEY SAND (SC) yellow-brown to olive-brown, dense, moist, trace fines				19.6		
9.5–11	SPT	11 14 15	35	SP-SC	SAND with CLAY (SP-SC) light olive-brown, dense, wet, trace clay						
15–16	SPT	9 12 19	33	SP-SC							
17				SP-SC	▽ (05/23/11, 11:55 a.m.)						
20–21	SPT	13 21 28	59	SP-SC	very dense						
25–26	SPT	18 20 24	53	SP-SC							
30											

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Project No.: 750604801 | Figure: A-1a

PROJECT:	ISLANDER MOTEL UPGRADE Alameda, California	Log of Boring B-1 PAGE 2 OF 2

DEPTH (feet)	Sampler Type	Sample	Blows/ 6"	SPT N-Value[1]	LITHOLOGY	MATERIAL DESCRIPTION	Type of Strength Test	Confining Pressure Lbs/Sq Ft	Shear Strength Lbs/Sq Ft	Fines %	Natural Moisture Content, %	Dry Density Lbs/Cu Ft
31	SPT		9 18 22	48	SP-SC	SAND with CLAY (SP-SC) (continued) dense						
32												
33												
34					SP	SAND (SP) light olive, very dense, wet, with trace silt						
35												
36	SPT		10 15 28	52								
37												
38												
39												
40						olive-gray						
41	SPT		18 32 45	92								
42												
43												
44												
45												
46												
47												
48												
49												
50												
51												
52												
53												
54												
55												
56												
57												
58												
59												
60												

Boring terminated at a depth of 41.5 feet below ground surface.
Boring backfilled with cement grout.
Groundwater encountered at 17 feet below ground surface during drilling.

[1] SPT blow counts for the last two increments were converted to SPT N-Values using a factor of 1.2 to account for hammer energy.
[2] Elevations based on Topographic Survey by Kier & Wright, May 2011, assumed to be similar to Mean Sea Level.

Treadwell&Rollo
A LANGAN COMPANY

Project No.:	Figure:
750604801	A-1b

TEST GEOTECH LOG 750604801.GPJ TR.GDT 6/16/11

PROJECT:	**ISLANDER MOTEL UPGRADE** Alameda, California	**Log of Boring B-2**

Boring location: See Site Plan, Figure 2
Logged by: W. Stegerstrom
Date started: 5/23/11 | Date finished: 5/23/11
Drilling method: Hollow Stem Auger
Hammer weight/drop: 140 lbs./30 inches | Hammer type: Automatic
Sampler: Standard Penetration Test (SPT)

DEPTH (feet)	SAMPLES: Sampler Type	Sample	Blows/6"	SPT N-Value[1]	LITHOLOGY	MATERIAL DESCRIPTION Ground Surface Elevation: ~27.5 foot[2]	Type of Strength Test	Confining Pressure Lbs/Sq Ft	Shear Strength Lbs/Sq Ft	Fines %	Natural Moisture Content, %	Dry Density Lbs/Cu Ft
0						6 inches Concrete						
1–4	SPT		3, 3, 3	8	SP	SAND (SP) red-brown, loose, dry to moist, fine-grained, with silt						
5–7	SPT		2, 2, 7	11	SC	CLAYEY SAND (SC) yellow-brown, medium dense, moist				17.8		
7–9	SPT		11, 15, 21	43	SC	CLAYEY SAND (SC) mottled olive-brown and brown, dense, wet, with trace fine gravel				17.9		
10–11	SPT		12, 11, 13	29	SP-SC	SAND with CLAY (SP-SC) yellow-brown to olive-brown, medium dense to dense, moist, with trace silt				11.3		
13						∇ (05/23/11, 9:33 a.m.)						
15–16	SPT		9, 14, 18	38		light olive-brown, dense						
20–21	SPT		10, 17, 20	44								
25–26	SPT		9, 18, 21	47								
30												

Treadwell&Rollo
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Project No.: 750604801 | Figure: A-2a

PROJECT: **ISLANDER MOTEL UPGRADE**
Alameda, California

Log of Boring B-2

DEPTH (feet)	Sampler Type	Sample	Blows/ 6"	SPT N-Value[1]	LITHOLOGY	MATERIAL DESCRIPTION	Type of Strength Test	Confining Pressure Lbs/Sq Ft	Shear Strength Lbs/Sq Ft	Fines %	Natural Moisture Content, %	Dry Density Lbs/Cu Ft
31	SPT		15 24 33	68	SP-SC	SAND with CLAY (SP-SC) (continued) very dense						
32												
33												
34					SP	SAND (SP) light olive, very dense, wet, with trace silt						
35												
36	SPT		8 14 30	53								
37												
38												
39												
40												
41												
42												
43												
44												
45												
46												
47												
48												
49												
50												
51												
52												
53												
54												
55												
56												
57												
58												
59												
60												

Boring terminated at a depth of 36.5 feet below ground surface.
Boring backfilled with cement grout.
Groundwater encountered at 13.8 feet below ground surface during drilling.

[1] SPT blow counts for the last two increments were converted to SPT N-Values using a factor of 1.2 to account for hammer energy.
[2] Elevations based on Topographic Survey by Kier & Wright, May 2011, assumed to be similar to Mean Sea Level.

Treadwell&Rollo
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Project No.: 750604801
Figure: A-2b

UNIFIED SOIL CLASSIFICATION SYSTEM

Major Divisions		Symbols	Typical Names
Coarse-Grained Soils (more than half of soil > no. 200 sieve size	**Gravels** (More than half of coarse fraction > no. 4 sieve size)	**GW**	Well-graded gravels or gravel-sand mixtures, little or no fines
		GP	Poorly-graded gravels or gravel-sand mixtures, little or no fines
		GM	Silty gravels, gravel-sand-silt mixtures
		GC	Clayey gravels, gravel-sand-clay mixtures
	Sands (More than half of coarse fraction < no. 4 sieve size)	**SW**	Well-graded sands or gravelly sands, little or no fines
		SP	Poorly-graded sands or gravelly sands, little or no fines
		SM	Silty sands, sand-silt mixtures
		SC	Clayey sands, sand-clay mixtures
Fine -Grained Soils (more than half of soil < no. 200 sieve size)	**Silts and Clays** LL = < 50	**ML**	Inorganic silts and clayey silts of low plasticity, sandy silts, gravelly silts
		CL	Inorganic clays of low to medium plasticity, gravelly clays, sandy clays, lean clays
		OL	Organic silts and organic silt-clays of low plasticity
	Silts and Clays LL = > 50	**MH**	Inorganic silts of high plasticity
		CH	Inorganic clays of high plasticity, fat clays
		OH	Organic silts and clays of high plasticity
Highly Organic Soils		**PT**	Peat and other highly organic soils

GRAIN SIZE CHART

Classification	Range of Grain Sizes: U.S. Standard Sieve Size	Range of Grain Sizes: Grain Size in Millimeters
Boulders	Above 12"	Above 305
Cobbles	12" to 3"	305 to 76.2
Gravel	3" to No. 4	76.2 to 4.76
coarse	3" to 3/4"	76.2 to 19.1
fine	3/4" to No. 4	19.1 to 4.76
Sand	No. 4 to No. 200	4.76 to 0.075
coarse	No. 4 to No. 10	4.76 to 2.00
medium	No. 10 to No. 40	2.00 to 0.420
fine	No. 40 to No. 200	0.420 to 0.075
Silt and Clay	Below No. 200	Below 0.075

Unstabilized groundwater level

Stabilized groundwater level

SAMPLE DESIGNATIONS/SYMBOLS

Sample taken with Sprague & Henwood split-barrel sampler with a 3.0-inch outside diameter and a 2.43-inch inside diameter. Darkened area indicates soil recovered

Classification sample taken with Standard Penetration Test sampler

Undisturbed sample taken with thin-walled tube

Disturbed sample, hand auger

Sampling attempted with no recovery

Core sample

Analytical laboratory sample

Sample taken with Direct Push sampler

SAMPLER TYPE

C — Core barrel

CA — California split-barrel sampler with 2.5-inch outside diameter and a 1.93-inch inside diameter

D&M — Dames & Moore piston sampler using 2.5-inch outside diameter, thin-walled tube

O — Osterberg piston sampler using 3.0-inch outside diameter, thin-walled Shelby tube

PT — Pitcher tube sampler using 3.0-inch outside diameter, thin-walled Shelby tube

S&H — Sprague & Henwood split-barrel sampler with a 3.0-inch outside diameter and a 2.43-inch inside diameter

SPT — Standard Penetration Test (SPT) split-barrel sampler with a 2.0-inch outside diameter and a 1.5-inch inside diameter

ST — Shelby Tube (3.0-inch outside diameter, thin-walled tube) advanced with hydraulic pressure

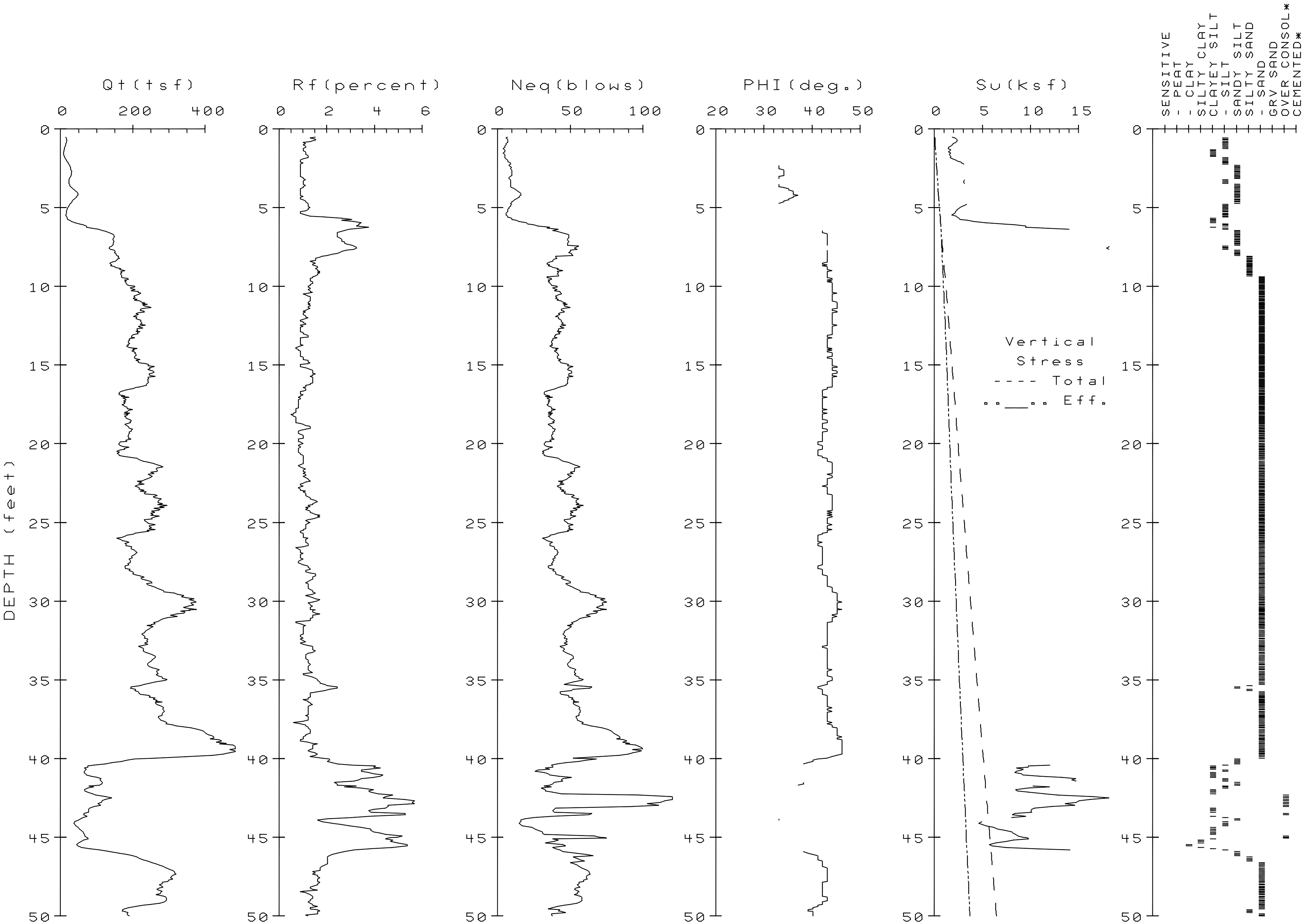
ISLANDER MOTEL UPGRADE
Alameda, California

Treadwell&Rollo
A LANGAN COMPANY

CLASSIFICATION CHART

Date 06/08/11 | Project No. 750604801 | Figure A-3

APPENDIX B
Logs of Cone Penetration Tests



Terminated at 50.0 feet

Groundwater measured at 6.3 feet

PROJECT: ISLANDER MOTEL REMODEL
LOCATION: Alameda CA
PROJ. NO.: 750604801(T&R-487)

CPT NO.: CPT-1
DATE : 05-25-2011

Treadwell & Rollo / Langan

cpts by John Sarmiento & Associates

PROJECT: ISLANDER MOTEL REMODEL
LOCATION: Alameda CA
PROJ. NO.: 750604801(T&R-487)
Terminated at 50.0 feet

CPT NO.: CPT-1
DATE : 05-25-2011
TIME : 11:58:00
Groundwater measured at 6.3 feet

TREADWELL & ROLLO / LANGAN
cpts by John Sarmiento & Associates

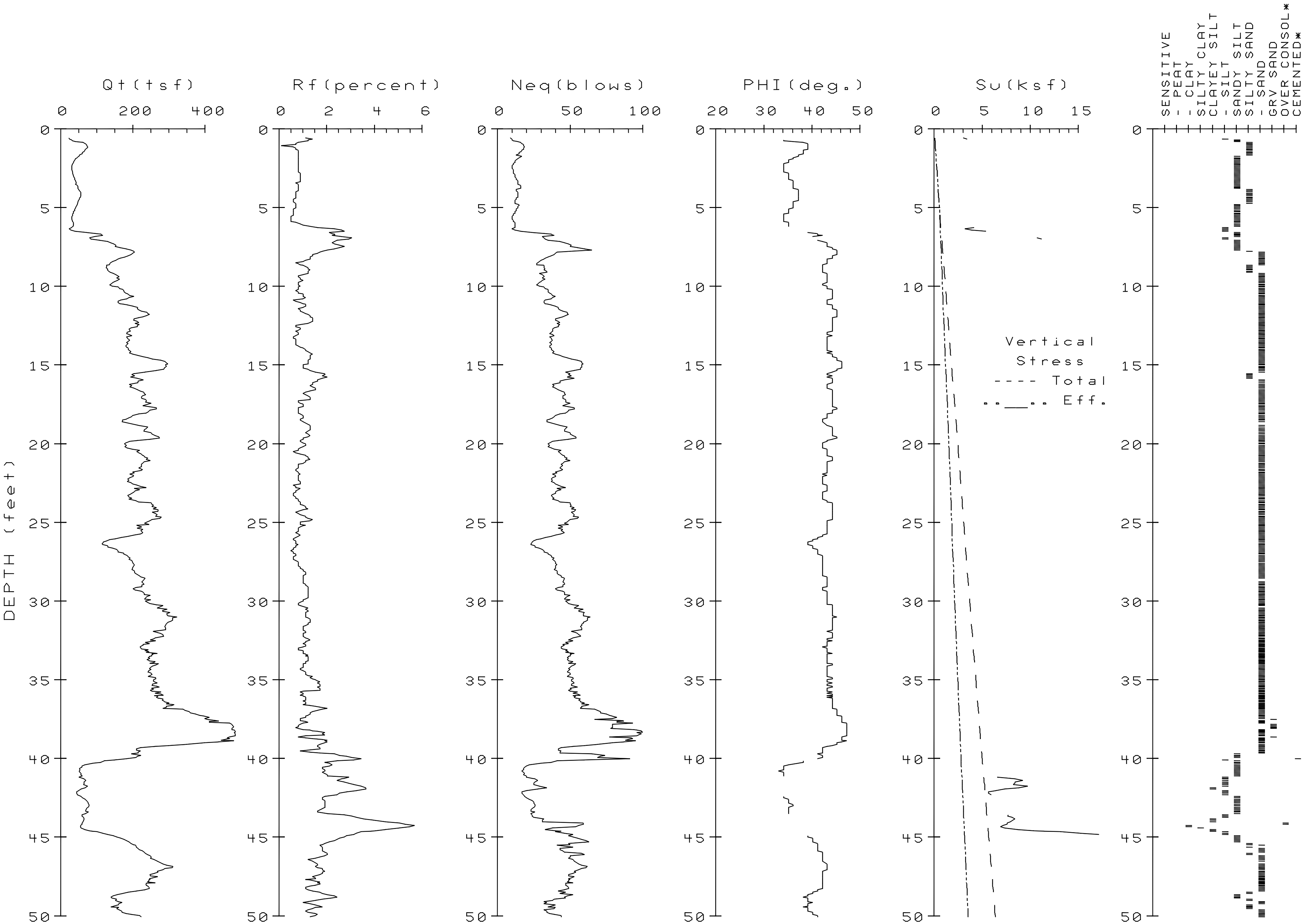
DEPTH (feet)	Qt (tsf)	Qt' (tsf)	Fs (tsf)	Rf (%)	SPT (N)	SPT' (N')	EffVtStr (ksf)	PHI (deg.)	SU (ksf)	SOIL BEHAVIOR TYPE	DENSITY RANGE (pcf)
0,51	13,9	22,16	0,21	1,5	7	11	0,06	----	1,84	Clayey SILT to Silty CLAY	100-110
1,02	14,2	22,66	0,15	1,0	6	9	0,11	----	1,88	Sandy SILT to Clayey SILT	"
1,53	8,9	14,29	0,10	1,1	4	7	0,16	----	1,77	Clayey SILT to Silty CLAY	90-100
2,04	18,3	29,30	0,17	0,9	7	12	0,21	----	2,43	Sandy SILT to Clayey SILT	100-110
2,53	28,3	45,34	0,25	0,9	9	15	0,27	33	----	Silty SAND to Sandy SILT	"
3,02	27,6	44,11	0,26	0,9	9	15	0,32	33	----	"	"
3,52	25,6	40,94	0,25	1,0	9	14	0,37	33	----	"	"
4,01	45,9	73,42	0,41	0,9	15	24	0,43	36	----	"	110-120
4,50	35,5	56,77	0,35	1,0	12	19	0,48	35	----	"	"
5,06	19,8	31,65	0,24	1,2	8	13	0,55	----	2,60	Sandy SILT to Clayey SILT	"
5,53	17,4	27,81	0,23	1,3	7	11	0,60	----	2,28	"	"
6,03	53,1	84,90	1,74	3,3	21	34	0,67	----	7,03	"	130-140
6,51	124,1	198,58	3,18	2,6	41	66	0,70	42	----	Silty SAND to Sandy SILT	"
7,06	144,4	230,99	3,81	2,6	48	77	0,74	43	----	"	"
7,52	135,0	215,94	4,34	3,2	54	86	0,78	----	17,94	Sandy SILT to Clayey SILT	"
8,04	155,2	246,24	3,07	2,0	52	82	0,81	43	----	Silty SAND to Sandy SILT	"
8,53	135,5	210,14	2,01	1,5	34	53	0,85	42	----	SAND to Silty SAND	"
9,04	166,0	251,24	2,84	1,7	41	63	0,89	43	----	"	"
9,55	187,3	276,63	2,42	1,3	37	55	0,92	44	----	SAND	"
10,04	203,8	293,65	2,44	1,2	41	59	0,96	44	----	"	"
10,54	211,4	297,94	2,30	1,1	42	60	0,99	44	----	"	120-130
11,04	233,1	323,69	2,71	1,2	47	65	1,02	45	----	"	"
11,52	218,5	300,08	2,50	1,1	44	60	1,05	44	----	"	"
12,05	207,5	281,43	2,37	1,1	42	56	1,08	44	----	"	"
12,53	214,7	287,77	2,09	1,0	43	58	1,11	44	----	"	"
13,01	213,1	282,34	2,11	1,0	43	56	1,14	44	----	"	"
13,52	185,9	243,12	1,71	0,9	37	49	1,18	43	----	"	"
14,01	200,0	258,89	1,55	0,8	40	52	1,20	43	----	"	110-120
14,54	203,3	259,68	2,08	1,0	41	52	1,24	44	----	"	120-130
15,03	240,1	303,10	2,08	0,9	48	61	1,27	44	----	"	"
15,55	243,4	302,82	3,56	1,5	49	61	1,30	44	----	"	130-140
16,02	241,1	295,95	3,40	1,4	48	59	1,34	44	----	"	"
16,53	192,7	233,57	2,10	1,1	39	47	1,37	43	----	"	120-130
17,00	167,3	200,41	1,49	0,9	33	40	1,40	42	----	"	"
17,52	169,2	200,08	1,43	0,8	34	40	1,43	42	----	"	"
18,04	191,2	223,54	1,10	0,6	38	45	1,46	43	----	"	110-120
18,51	193,8	224,28	1,36	0,7	39	45	1,48	43	----	"	"
19,01	190,7	217,80	2,48	1,3	38	44	1,52	42	----	"	130-140
19,52	185,4	209,66	1,91	1,0	37	42	1,55	42	----	"	120-130
20,04	161,3	180,51	1,39	0,9	32	36	1,59	41	----	"	"
20,53	162,4	179,98	1,35	0,8	32	36	1,62	41	----	"	"
21,00	216,2	237,29	2,22	1,0	43	47	1,65	43	----	"	"
21,53	265,3	288,01	2,69	1,0	53	58	1,68	44	----	"	"
22,01	246,8	265,32	2,66	1,1	49	53	1,71	44	----	"	"
22,55	229,7	244,15	2,01	0,9	46	49	1,74	43	----	"	"
23,06	236,7	249,54	2,44	1,0	47	50	1,77	43	----	"	"
23,54	273,7	286,18	3,86	1,4	55	57	1,81	44	----	"	130-140
24,04	285,4	296,33	3,21	1,1	57	59	1,84	44	----	"	120-130
24,53	248,6	255,99	4,22	1,7	50	51	1,88	43	----	"	130-140
25,03	241,2	246,28	3,01	1,2	48	49	1,91	43	----	"	"
25,56	243,7	246,88	2,15	0,9	49	49	1,95	43	----	"	120-130
26,00	154,8	155,74	1,79	1,2	31	31	1,97	41	----	"	"
26,54	196,8	196,74	1,88	1,0	39	39	2,01	42	----	"	"
27,03	204,5	204,39	1,89	0,9	41	41	2,04	42	----	"	"

PROJECT: ISLANDER MOTEL REMODEL   CPT NO.: CPT-1
LOCATION: Alameda CA   DATE : 05-25-2011
PROJ. NO.: 750604801(T&R-487)   TIME : 11:58:00
Terminated at 50.0 feet   Groundwater measured at 6.3 feet

TREADWELL & ROLLO / LANGAN
cpts by John Sarmiento & Associates

DEPTH (feet)	Qt (tsf)	Qt' (tsf)	Fs (tsf)	Rf (%)	SPT (N)	SPT' (N')	EffVtStr (ksf)	PHI (deg.)	SU (ksf)	SOIL BEHAVIOR TYPE	DENSITY RANGE (pcf)
27,54	190,9	190,59	1,59	0,8	38	38	2,07	42	----	SAND	120-130
28,06	188,2	187,77	1,99	1,1	38	38	2,10	42	----	"	"
28,52	222,2	221,57	3,13	1,4	44	44	2,14	43	----	"	130-140
29,04	252,4	251,51	3,08	1,2	50	50	2,17	43	----	"	"
29,55	321,1	319,76	4,61	1,4	64	64	2,21	45	----	"	"
30,04	373,3	371,48	5,41	1,4	75	74	2,25	46	----	"	"
30,51	374,5	372,35	4,95	1,3	75	74	2,28	46	----	"	"
31,03	295,0	293,16	4,12	1,4	59	59	2,32	44	----	"	"
31,51	252,6	250,79	2,50	1,0	51	50	2,35	43	----	"	120-130
32,00	229,9	228,15	2,19	1,0	46	46	2,38	43	----	"	"
32,57	240,4	238,38	2,61	1,1	48	48	2,41	43	----	"	"
33,08	228,5	226,44	3,26	1,4	46	45	2,45	43	----	"	130-140
33,57	260,3	257,77	2,91	1,1	52	52	2,48	43	----	"	120-130
34,05	239,9	236,16	3,03	1,3	48	47	2,52	43	----	"	130-140
34,52	266,7	259,59	3,64	1,4	53	52	2,55	44	----	"	"
35,03	289,1	277,76	4,62	1,6	58	56	2,59	44	----	"	"
35,52	192,4	182,61	4,59	2,4	64	61	2,62	41	----	Silty SAND to Sandy SILT	"
36,03	246,7	231,21	3,16	1,3	49	46	2,66	43	----	SAND	"
36,54	266,1	246,10	3,60	1,4	53	49	2,70	43	----	"	"
37,06	284,0	259,10	3,33	1,2	57	52	2,73	43	----	"	"
37,56	275,3	247,87	3,24	1,2	55	50	2,77	43	----	"	"
38,06	360,8	320,56	4,49	1,2	72	64	2,81	45	----	"	"
38,54	415,6	365,09	4,55	1,1	83	73	2,84	45	----	"	120-130
39,02	456,0	395,40	6,47	1,4	91	79	2,87	46	----	"	130-140
39,56	485,1	414,29	6,47	1,3	97	83	2,91	46	----	"	"
40,02	202,9	171,07	4,36	2,1	68	57	2,95	41	----	Silty SAND to Sandy SILT	"
40,55	75,2	62,46	2,90	3,9	38	31	2,98	----	9,69	Clayey SILT to Silty CLAY	"
41,03	71,4	58,74	3,09	4,3	36	29	3,02	----	9,17	"	"
41,50	115,9	94,87	2,62	2,3	39	32	3,05	38	----	Silty SAND to Sandy SILT	"
42,05	66,8	54,37	2,49	3,7	33	27	3,09	----	8,55	Clayey SILT to Silty CLAY	"
42,58	122,4	99,11	6,37	5,2	122	99	3,13	----	15,96	Very Stiff Fine Grained *	>140
43,04	93,4	75,26	4,22	4,5	93	75	3,17	----	12,09	"	130-140
43,57	63,3	50,71	3,28	5,2	63	50	3,21	----	8,07	"	"
44,09	37,2	29,67	0,83	2,2	15	12	3,24	----	4,59	Sandy SILT to Clayey SILT	"
44,55	57,4	45,52	2,19	3,8	28	22	3,28	----	7,27	Clayey SILT to Silty CLAY	"
45,02	74,5	58,77	3,45	4,6	74	58	3,31	----	9,55	Very Stiff Fine Grained *	"
45,54	47,7	37,40	2,50	5,3	47	37	3,35	----	5,97	CLAY	"
46,07	182,7	142,61	4,08	2,2	61	48	3,39	40	----	Silty SAND to Sandy SILT	"
46,50	239,7	186,24	4,87	2,0	60	47	3,42	42	----	SAND to Silty SAND	"
47,01	299,1	231,10	5,04	1,7	60	46	3,46	43	----	SAND	"
47,56	302,6	232,38	5,06	1,7	61	46	3,50	43	----	"	"
48,02	272,6	208,30	4,26	1,6	55	42	3,53	42	----	"	"
48,51	275,3	209,41	2,96	1,1	55	42	3,56	42	----	"	120-130
49,02	290,1	219,39	3,73	1,3	58	44	3,60	43	----	"	130-140
49,53	176,4	132,70	2,32	1,3	35	27	3,63	40	----	"	"
50,00	189,8	142,03	2,30	1,2	38	28	3,67	40	----	"	"

DEPTH = Sampling interval (~0.1 feet)
Qc = Tip bearing uncorrected   Qt = Tip bearing corrected   Fs = Sleeve friction resistance   Rf = Qt / Fs
SPT = Equivalent Standard Penetration Test   Qt' and SPT' = Qt and SPT corrected for overburden
EffVtStr = Effective Vertical Stress using est. density**   Phi = Soil friction angle*
Su = Undrained Soil Strength* (see classification chart)
References: * Robertson and Campanella, 1988   **Olsen, 1989   *** Durgunoglu & Mitchell, 1975



Terminated at 50.0 feet

Groundwater measured at 5.2 feet

PROJECT: ISLANDER MOTEL REMODEL
LOCATION: Alameda CA
PROJ. NO.: 750604801(T&R-487)

CPT NO.: CPT-2
DATE : 05-25-2011

Treadwell & Rollo / Langan

cpts by John Sarmiento & Associates

PROJECT: ISLANDER MOTEL REMODEL
LOCATION: Alameda CA
PROJ. NO.: 750604801(T&R-487)
Terminated at 50.0 feet

CPT NO.: CPT-2
DATE : 05-25-2011
TIME : 10:54:00
Groundwater measured at 5.2 feet

TREADWELL & ROLLO / LANGAN
cpts by John Sarmiento & Associates

DEPTH (feet)	Qt (tsf)	Qt' (tsf)	Fs (tsf)	Rf (%)	SPT (N)	SPT' (N')	EffVtStr (ksf)	PHI (deg.)	SU (ksf)	SOIL BEHAVIOR TYPE	DENSITY RANGE (pcf)
0,58	22,6	36,11	0,26	1,1	9	14	0,07	----	3,00	Sandy SILT to Clayey SILT	110-120
1,00	72,6	116,16	0,54	0,7	18	29	0,12	39	----	SAND to Silty SAND	"
1,52	60,5	96,80	0,50	0,8	15	24	0,17	38	----	"	"
2,05	36,8	58,90	0,29	0,8	12	20	0,23	35	----	Silty SAND to Sandy SILT	100-110
2,50	31,1	49,74	0,25	0,8	10	17	0,28	34	----	"	"
3,04	36,8	58,91	0,32	0,9	12	20	0,34	35	----	"	110-120
3,52	40,9	65,49	0,33	0,8	14	22	0,40	36	----	"	"
4,02	55,4	88,70	0,41	0,7	14	22	0,45	37	----	SAND to Silty SAND	"
4,56	49,3	78,93	0,31	0,6	12	20	0,51	37	----	"	100-110
5,03	40,9	65,47	0,27	0,7	14	22	0,56	36	----	Silty SAND to Sandy SILT	"
5,57	31,5	50,35	0,17	0,5	10	17	0,58	34	----	"	90-100
6,04	36,3	58,11	0,31	0,9	12	19	0,60	35	----	"	110-120
6,51	40,5	64,80	1,08	2,7	16	25	0,64	----	5,35	Sandy SILT to Clayey SILT	130-140
7,00	83,6	133,70	2,43	2,9	33	52	0,67	----	11,09	"	"
7,56	164,6	263,28	4,15	2,5	55	88	0,71	44	----	Silty SAND to Sandy SILT	"
8,06	186,1	297,68	2,57	1,4	37	60	0,75	44	----	SAND	"
8,51	137,6	220,16	0,97	0,7	28	44	0,77	43	----	"	110-120
9,04	134,3	214,34	1,63	1,2	34	54	0,80	42	----	SAND to Silty SAND	120-130
9,55	163,2	255,16	1,49	0,9	33	51	0,84	43	----	SAND	"
10,06	159,4	244,11	1,33	0,8	32	49	0,87	43	----	"	"
10,50	185,5	278,95	1,77	1,0	37	56	0,90	44	----	"	"
11,03	157,8	232,19	1,36	0,9	32	46	0,93	43	----	"	"
11,57	222,6	319,82	2,10	0,9	45	64	0,96	45	----	"	"
12,07	202,4	283,41	2,88	1,4	40	57	1,00	44	----	"	130-140
12,57	210,9	292,50	1,57	0,7	42	59	1,03	44	----	"	110-120
13,00	182,2	250,54	1,45	0,8	36	50	1,05	43	----	"	"
13,51	186,2	253,44	1,39	0,7	37	51	1,07	43	----	"	"
14,01	186,5	250,77	2,04	1,1	37	50	1,11	43	----	"	120-130
14,54	242,1	320,74	3,16	1,3	48	64	1,14	45	----	"	130-140
15,00	288,6	377,99	2,95	1,0	58	76	1,17	46	----	"	120-130
15,54	201,1	259,31	3,74	1,9	50	65	1,21	43	----	SAND to Silty SAND	130-140
16,03	213,4	271,20	3,32	1,6	43	54	1,25	44	----	SAND	"
16,57	212,1	265,53	3,12	1,5	42	53	1,29	44	----	"	"
17,04	222,6	275,10	2,82	1,3	45	55	1,32	44	----	"	"
17,50	242,1	295,76	2,31	1,0	48	59	1,35	44	----	"	120-130
18,04	236,5	285,13	1,96	0,8	47	57	1,38	44	----	"	"
18,51	171,2	203,99	1,51	0,9	34	41	1,41	42	----	"	"
19,03	236,7	277,68	3,14	1,3	47	56	1,45	44	----	"	130-140
19,56	271,0	313,02	3,23	1,2	54	63	1,49	45	----	"	"
20,01	176,7	202,05	1,91	1,1	35	40	1,52	42	----	"	120-130
20,57	215,9	244,57	1,56	0,7	43	49	1,55	43	----	"	110-120
21,00	236,3	265,06	2,96	1,3	47	53	1,58	44	----	"	130-140
21,55	216,4	240,47	1,64	0,8	43	48	1,61	43	----	"	110-120
22,02	201,5	221,85	1,65	0,8	40	44	1,64	43	----	"	120-130
22,55	199,1	216,69	1,83	0,9	40	43	1,67	42	----	"	"
23,07	201,7	217,63	1,41	0,7	40	44	1,70	42	----	"	110-120
23,53	189,4	202,64	1,48	0,8	38	41	1,72	42	----	"	"
24,06	263,9	279,40	2,86	1,1	53	56	1,75	44	----	"	120-130
24,52	266,1	279,90	2,33	0,9	53	56	1,78	44	----	"	"
25,05	231,6	241,82	2,19	0,9	46	48	1,82	43	----	"	"
25,51	221,3	229,76	1,69	0,8	44	46	1,84	43	----	"	110-120
26,04	151,6	156,36	0,96	0,6	30	31	1,87	41	----	"	"
26,56	130,4	133,62	0,93	0,7	26	27	1,90	40	----	"	"
27,02	174,7	177,98	1,16	0,7	35	36	1,92	41	----	"	"

PROJECT: ISLANDER MOTEL REMODEL
LOCATION: Alameda CA
PROJ. NO.: 750604801(T&R-487)
Terminated at 50.0 feet

CPT NO.: CPT-2
DATE : 05-25-2011
TIME : 10:54:00
Groundwater measured at 5.2 feet

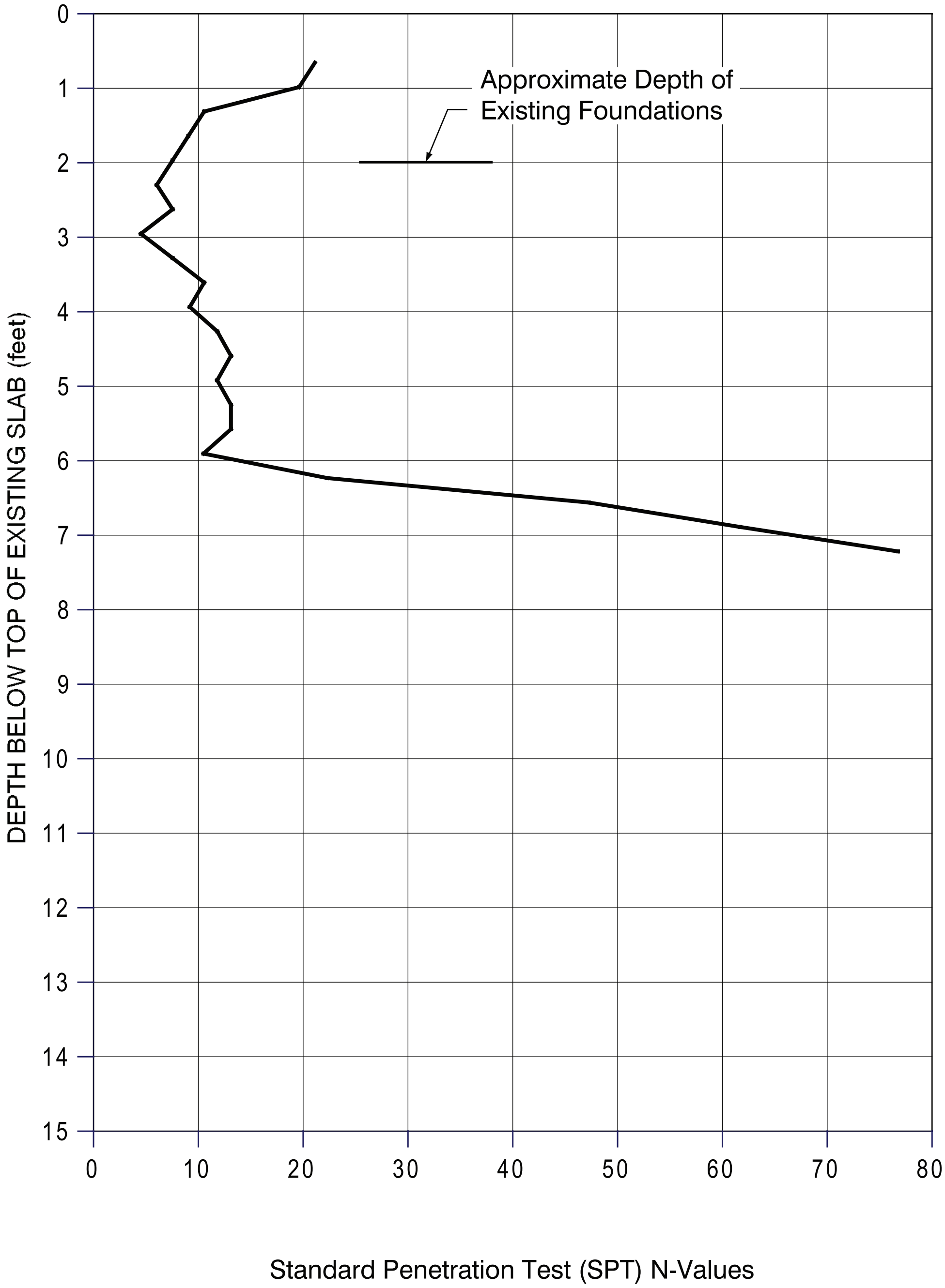
TREADWELL & ROLLO / LANGAN
cpts by John Sarmiento & Associates

DEPTH (feet)	Qt (tsf)	Qt' (tsf)	Fs (tsf)	Rf (%)	SPT (N)	SPT' (N')	EffVtStr (ksf)	PHI (deg.)	SU (ksf)	SOIL BEHAVIOR TYPE	DENSITY RANGE (pcf)
27,51	197,8	200,38	1,50	0,8	40	40	1,95	42	----	SAND	110-120
28,08	203,4	204,34	2,06	1,0	41	41	1,98	42	----	"	120-130
28,55	232,4	232,33	2,33	1,0	46	46	2,01	43	----	"	"
29,00	224,0	223,82	2,44	1,1	45	45	2,04	43	----	"	"
29,51	231,0	230,63	2,75	1,2	46	46	2,08	43	----	"	130-140
30,07	239,8	239,28	2,16	0,9	48	48	2,11	43	----	"	120-130
30,51	293,2	292,37	3,19	1,1	59	58	2,14	44	----	"	"
31,01	320,3	319,23	3,62	1,1	64	64	2,17	45	----	"	"
31,55	287,6	286,39	3,23	1,1	58	57	2,20	44	----	"	"
32,05	275,8	274,53	2,88	1,0	55	55	2,24	44	----	"	"
32,51	261,8	260,38	3,14	1,2	52	52	2,27	44	----	"	130-140
33,04	223,7	222,36	1,88	0,8	45	44	2,30	43	----	"	120-130
33,52	245,5	243,88	2,95	1,2	49	49	2,34	43	----	"	130-140
34,05	262,6	260,63	2,61	1,0	53	52	2,37	44	----	"	120-130
34,58	249,7	247,63	2,78	1,1	50	50	2,40	43	----	"	"
35,00	256,7	254,50	3,96	1,5	51	51	2,43	43	----	"	130-140
35,53	250,4	248,06	4,30	1,7	50	50	2,47	43	----	"	"
36,02	259,2	256,32	2,69	1,0	52	51	2,50	43	----	"	120-130
36,53	290,6	284,24	3,33	1,1	58	57	2,54	44	----	"	"
37,01	353,6	341,85	5,03	1,4	71	68	2,57	45	----	"	130-140
37,51	404,3	386,76	3,45	0,9	67	64	2,60	46	----	Gravelly SAND to SAND	120-130
38,04	474,5	448,66	3,56	0,7	79	75	2,63	47	----	"	"
38,56	480,4	449,02	5,28	1,1	96	90	2,67	47	----	SAND	"
39,01	395,2	365,17	7,42	1,9	79	73	2,70	45	----	"	130-140
39,53	220,4	201,27	1,90	0,9	44	40	2,73	42	----	"	120-130
40,01	181,7	163,69	6,23	3,4	91	82	2,77	41	----	SAND to Clayey SAND *	>140
40,56	58,1	51,54	1,07	1,8	19	17	2,81	34	----	Silty SAND to Sandy SILT	130-140
41,03	56,2	49,28	1,15	2,0	18	16	2,84	34	----	"	"
41,58	64,9	56,00	1,87	2,9	25	22	2,88	----	8,31	Sandy SILT to Clayey SILT	"
42,03	50,4	42,98	1,62	3,2	20	17	2,92	----	6,37	"	"
42,58	68,3	57,30	1,22	1,8	22	19	2,96	35	----	Silty SAND to Sandy SILT	"
43,00	79,1	65,62	1,50	1,9	26	22	2,99	36	----	"	"
43,58	59,9	49,23	1,61	2,7	24	19	3,03	----	7,63	Sandy SILT to Clayey SILT	"
44,01	62,1	50,76	2,75	4,4	31	25	3,06	----	7,91	Clayey SILT to Silty CLAY	"
44,51	67,5	54,96	3,00	4,4	33	27	3,10	----	8,63	"	"
45,07	165,9	134,25	4,19	2,5	55	45	3,14	40	----	Silty SAND to Sandy SILT	"
45,50	206,8	166,63	3,60	1,7	41	33	3,17	41	----	SAND	"
46,03	233,8	187,34	4,66	2,0	58	47	3,21	42	----	SAND to Silty SAND	"
46,51	265,7	211,84	3,67	1,4	53	42	3,24	42	----	SAND	"
47,00	287,7	228,15	5,04	1,8	58	46	3,28	43	----	"	"
47,51	267,4	210,98	3,77	1,4	53	42	3,31	42	----	"	"
48,04	236,7	185,64	3,59	1,5	47	37	3,35	42	----	"	"
48,57	171,8	133,99	3,09	1,8	43	33	3,39	40	----	SAND to Silty SAND	"
49,01	151,7	117,77	2,26	1,5	38	29	3,42	39	----	"	"
49,54	155,0	119,67	2,15	1,4	39	30	3,46	39	----	"	"
50,06	221,9	170,35	2,90	1,3	44	34	3,50	41	----	SAND	"

DEPTH = Sampling interval (~0.1 feet)
Qc = Tip bearing uncorrected   Qt = Tip bearing corrected   Fs = Sleeve friction resistance   Rf = Qt / Fs
SPT = Equivalent Standard Penetration Test   Qt' and SPT' = Qt and SPT corrected for overburden
EffVtStr = Effective Vertical Stress using est. density**   Phi = Soil friction angle*
Su = Undrained Soil Strength* (see classification chart)
References: * Robertson and Campanella, 1988   **Olsen, 1989   *** Durgunoglu & Mitchell, 1975

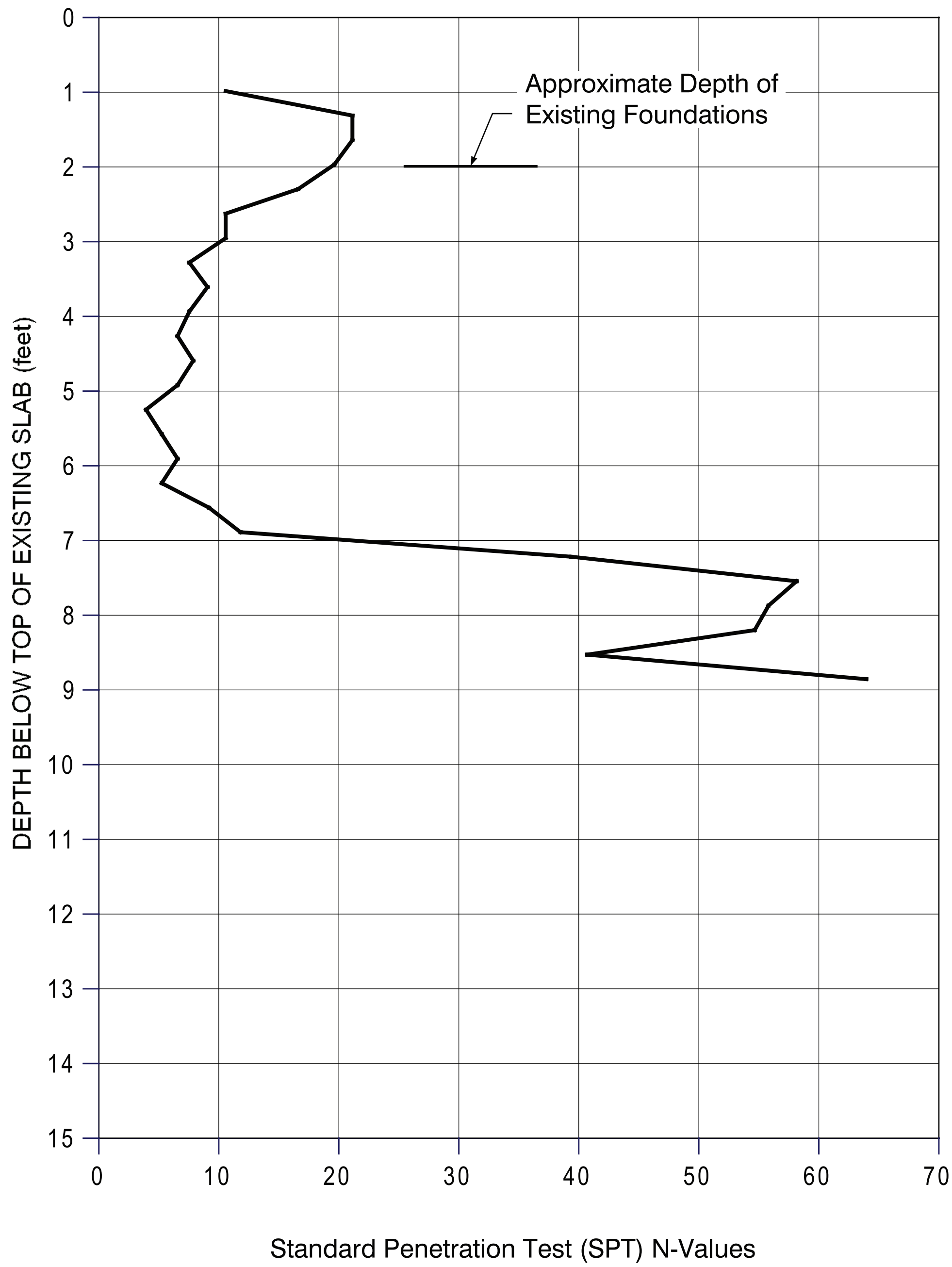
Treadwell&Rollo
A LANGAN COMPANY

APPENDIX C
Logs of Dynamic Cone Penetration Tests



Note: 1. Depth of zero feet on graph corresponds to top of slab.

ISLANDER MOTEL UPGRADE Alameda, California	**DYNAMIC PENETROMETER TEST** **DPT-1**
Treadwell&Rollo A LANGAN COMPANY	Date 06/13/11 | Project No. 750604801 | Figure C-1



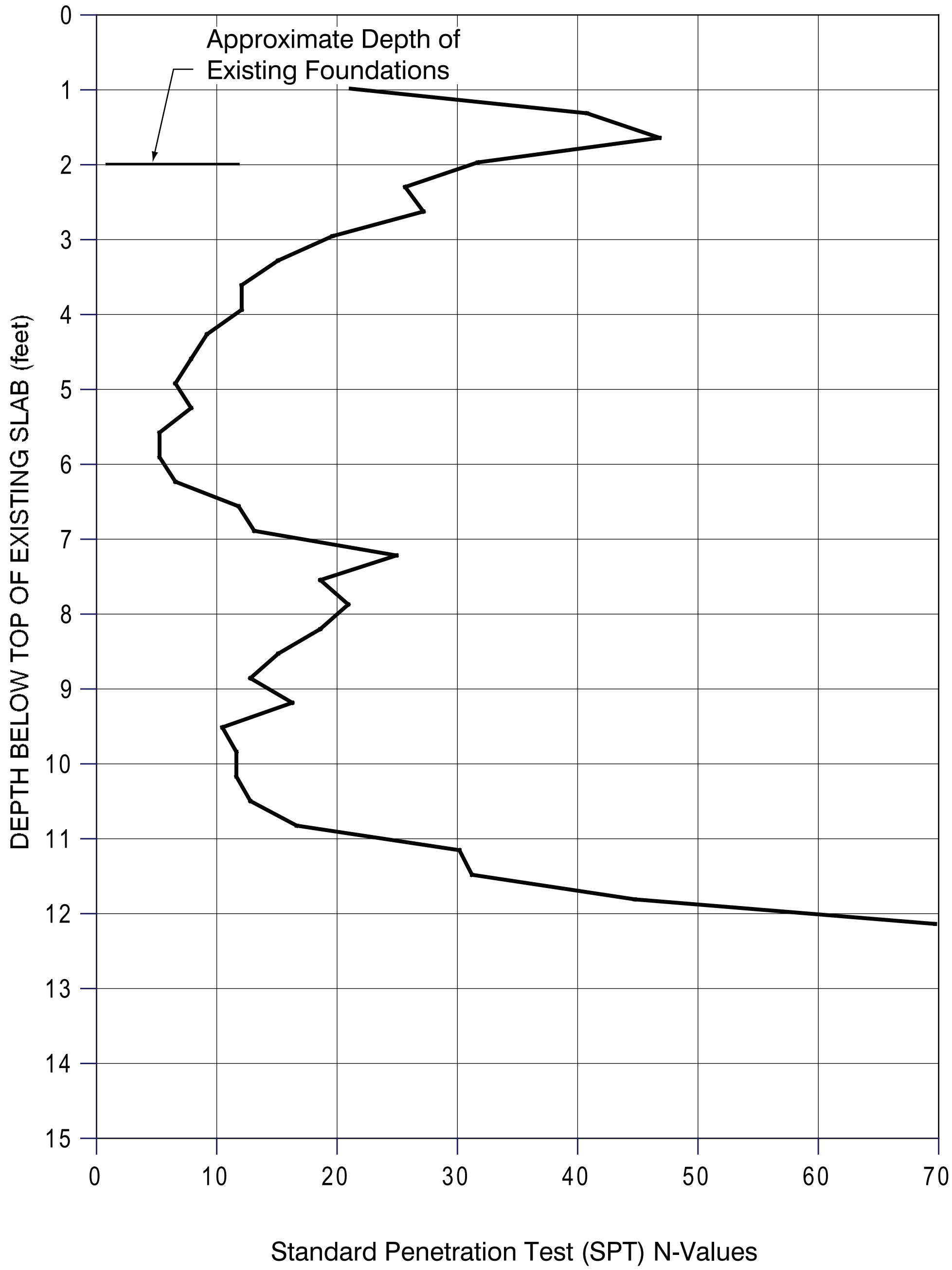
Note: 1. Depth of zero feet on graph corresponds to top of slab.

ISLANDER MOTEL UPGRADE
Alameda, California

Treadwell&Rollo
A LANGAN COMPANY

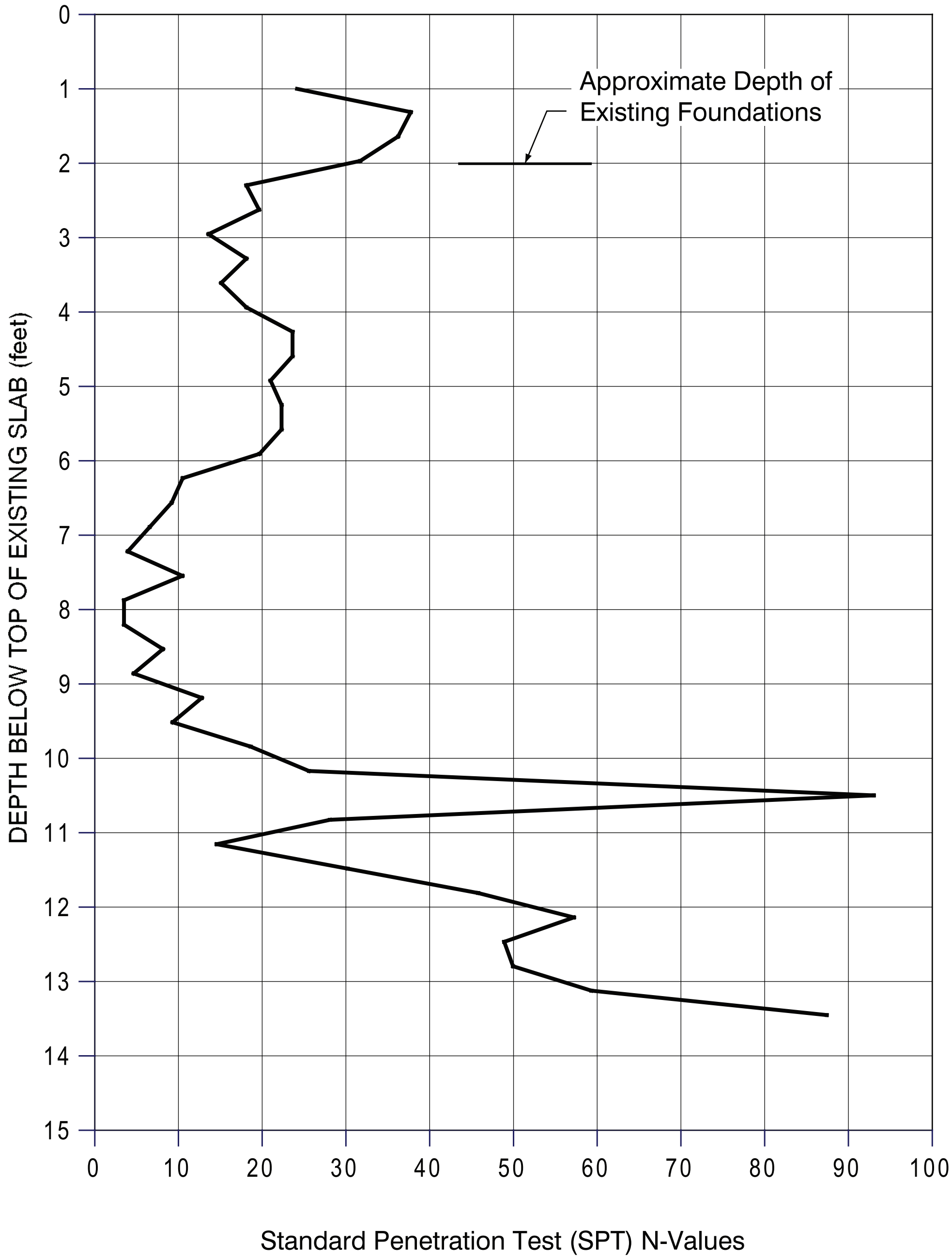
DYNAMIC PENETROMETER TEST
DPT-2

Date 06/13/11	Project No. 750604801	Figure C-2



Note: 1. Depth of zero feet on graph corresponds to top of slab.

ISLANDER MOTEL UPGRADE Alameda, California	**DYNAMIC PENETROMETER TEST** **DPT-3**
Treadwell&Rollo A LANGAN COMPANY	Date 06/13/11 | Project No. 750604801 | Figure C-3



Note: 1. Depth of zero feet on graph corresponds to top of slab.

ISLANDER MOTEL UPGRADE
Alameda, California

Treadwell&Rollo
A LANGAN COMPANY

DYNAMIC PENETROMETER TEST
DPT-4

Date 06/13/11	Project No. 750604801	Figure C-4

APPENDIX D
Results of Corrosivity Testing

E T S

975 Transport Way, Suite 2
Petaluma, CA 94954
(707) 778-9605/FAX 778-9612
e-mail: entech@pacbell.net

Environmental Technical Services

-Soil, Water & Air Testing & Monitoring
-Analytical Labs
-Technical Support

Serving people and the environment so that both benefit.

COMPANY:	Treadwell & Rollo, 501 14th Street, 3rd Floor, Oakland, CA 94612			ANALYST(S)	SUPERVISOR
ATTN:	Scott Walker		DATE of	D. Salinas	D. Jacobson
JOB SITE:	Islander Motel uprgade, bay area, California	DATE RECEIVED	COMPLETION	S. Santos	LAB DIRECTOR
PROJ NUM #:	750604801	6/7/2011	3/15/2011		G.S. Conrad PhD

LAB SAMPLE NUMBER	SAMPLE ID	DESCRIPTION of SOIL and/or SEDIMENT	SOIL pH -log[H+]	NOMINAL RESISTIVITY ohm-cm	ELECTRICAL CONDUCTIVITY µmhos/cm	SULFATE SO4 ppm	CHLORIDE Cl ppm
04447-1	IM1/BA	B-2-1 @ 1.5'	**6.42**	**8,650**	**[115]**	**21**	**39**
Method	Detection	Limits --->	---	1	0.1	1	1

LAB SAMPLE NUMBER	SAMPLE ID	DESCRIPTION of SOIL and/or SEDIMENT	SALINITY ECe mmhos/cm	SOLUBLE SULFIDES (S=) ppm	SOLUBLE CYANIDES (CN=) ppm	REDOX mV	PERCENT MOISTURE %
Method	Detection	Limits --->	---	0.1	0.1	1	0.1

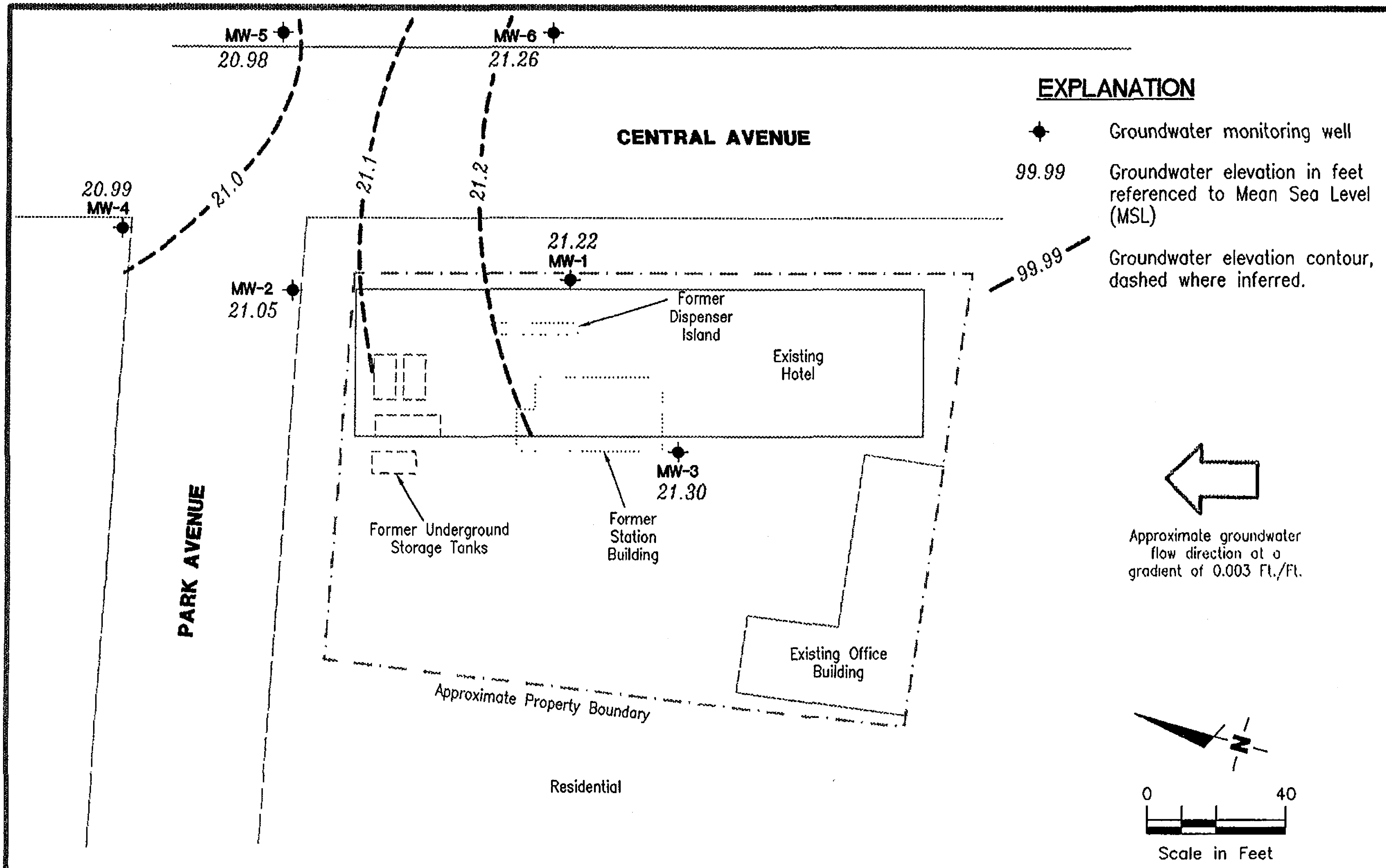
** COMMENTS **

Resistivity is well over 5,000 ohm-cm which is good, but soil reaction (i.e., pH) is mildly acidic which does not help; sulfate and chloride are very low. The standard CalTrans times to perforation are as follows: for 18 ga steel the time is >24 yrs, and for 12 ga it goes up to 53.5 yrs. For gray/ductile steel/iron the calculated average pitting rate is ≈0.07 mm/yr, so pitting to a 2 mm depth is at ≈28.5 yrs, and to a 4 mm depth is at >57 yrs. Chloride level is so low that this anion should not have any measurable corrosion impact on steel reinforcement; likewise, sulfate is also so low that there would be no measureable adverse impact on concrete, cements, mortars or grouts. In principle, this soil could benefit significantly from alkaline treatment such that raising its pH to the 7.5-8.5 range would increase the 18 ga time to perf to >60 yrs (i.e., more than double the untreated time); and the pitting rate would drop to <0.02 mm/yr putting the 2 mm depth time up to >100 yrs. However, keep in mind that lime treatment does not last more than a few years in an unprotected setting, but has much greater longevity in protected environments (i.e., underneath slabs, buildings, etc.). Otherwise, to increase metals longevity in this soil would require upgrading (e.g. increased gauge or more resistant steels, etc.); and/or cathodic protection along with coating or wrapping the steel could be utilized (the number and size of sacrificial anodes would be modest and no impressed current should be needed); other alternatives do include more engineering fill, or employing the use of plastic, fiberglass or concrete pipe, etc. Last, standard concrete mixes should be fine in this soil based on the results.

\\\\NOTES: Methods are from following sources: extractions by Cal Trans protocols as per Cal Test 417 (SO4), 422 (Cl), and 532/643 (pH & resistivity); &/or by ASTM Vol. 4.08 & ASTM Vol. 11.01 (=EPA Methods of Chemical Analysis, or Standard Methods); pH - ASTM G 51; Spec. Cond. - ASTM D 1125; resistivity - ASTM G 57; redox - Pt probe/ISE; sulfate - extraction Title 22, detection ASTM D 516 (=EPA 375.4); chloride - extraction Title 22, detection ASTM D 512 (=EPA 325.3); sulfides - extraction by Title 22, and detection EPA 376.2 (= SMEWW 4500-S D); cyanides - extraction by Title 22, and detection by ASTM D 4374 (=EPA 335.2)

Treadwell&Rollo
A LANGAN COMPANY

APPENDIX E
Logs of Previous Borings/Wells by Others



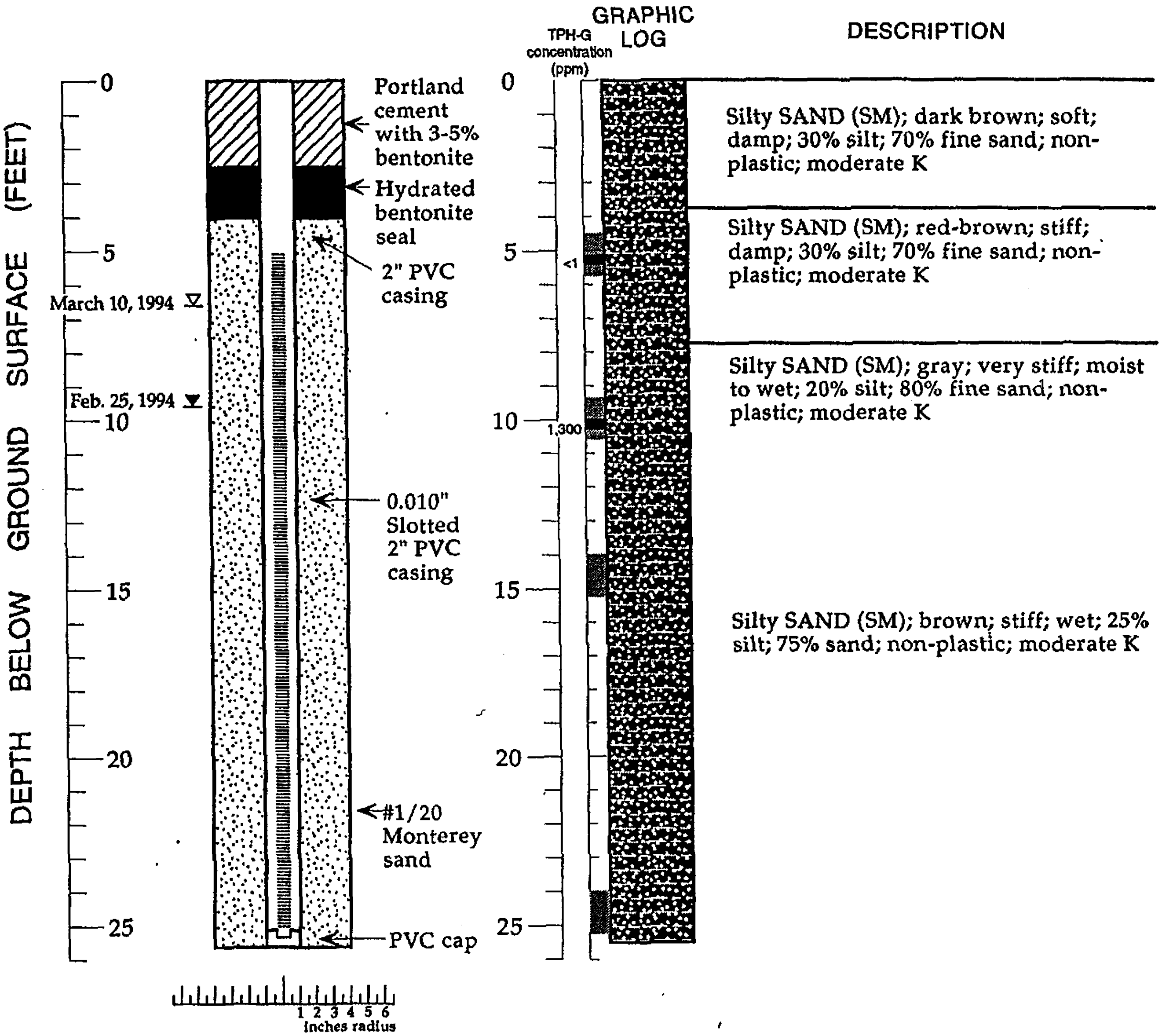
Gettler - Ryan Inc.
6747 Sierra Ct., Suite J (510) 551-7555
Dublin, CA 94568

POTENTIOMETRIC MAP
Former Chevron Service Station No. 9-0100
2428 Central Avenue
Alameda, California

FIGURE
2

JOB NUMBER	REVIEWED BY	DATE	REVISED DATE
5178.02		August 30, 1996	

WELL MW-1



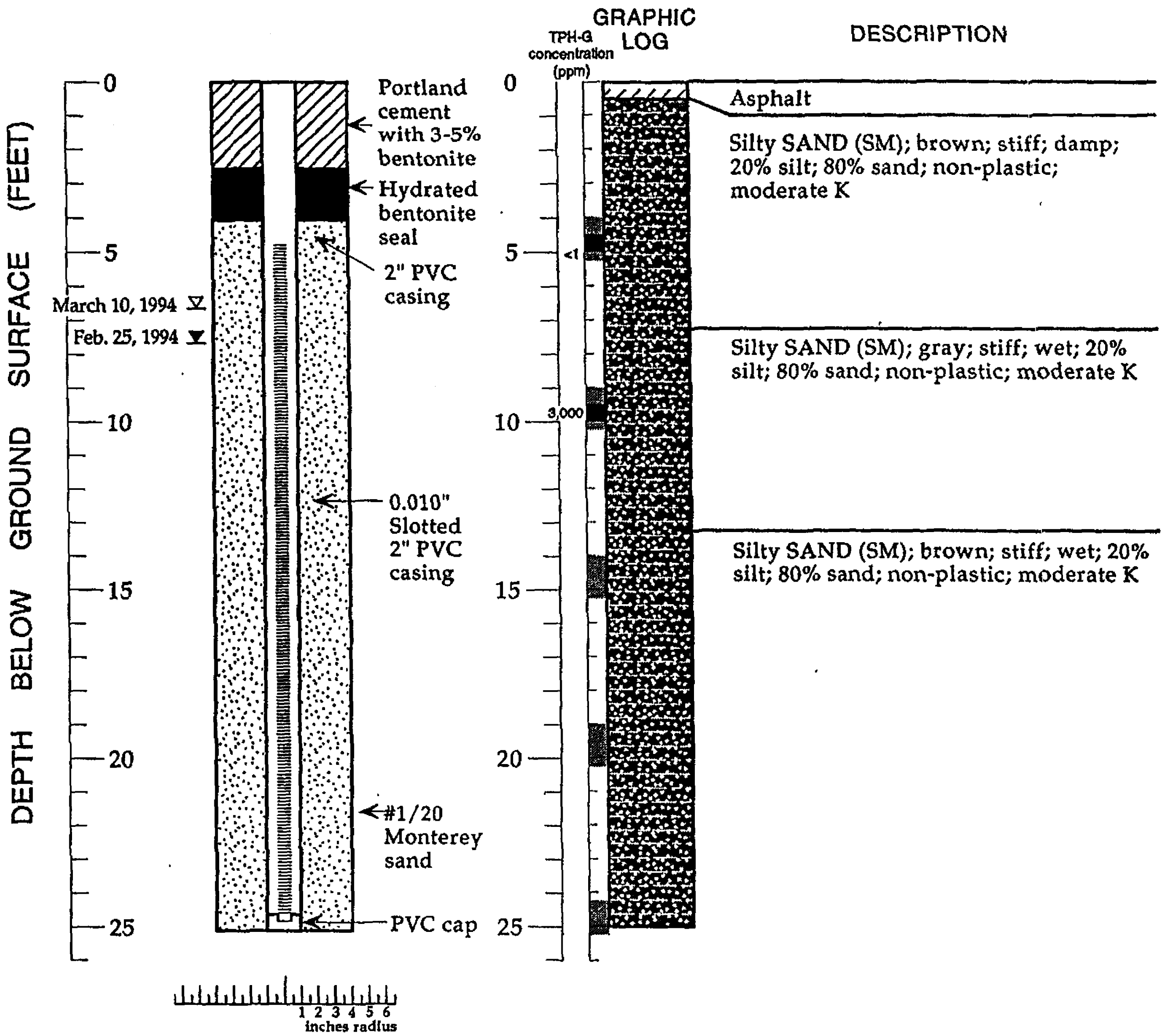
EXPLANATION

- Water level during drilling (date)
- Water level (date)
- Contact (dotted where approximate)
- —?—?— Uncertain contact
- Gradational contact
- Location of recovered drive sample
- Location of drive sample sealed for chemical analysis
- Cutting sample
- K = Estimated hydraulic conductivity

Logged By: Joyce Adams
Supervisor: James W. Carmody; CEG 1576
Drilling Company: Soils Exploration Services, Vacaville, CA
License Number: #C57-582696
Driller: Tim Dunne
Drilling Method: Hollow-stem auger
Date Drilled: February 24, 1994
Well Head Completion: 2" locking well-plug, traffic-rated vault
Type of Sampler: Split barrel (2" ID)
Ground Surface Elevation: 29.54 feet above mean sea level
TPH-G: Total petroleum hydrocarbon as gasoline in soil by modified EPA Method 8015

Boring Log and Well Construction Details - Well MW-1 - Former Chevron Service Station #9-0100, 2428 Central Street, Alameda, California

WELL MW-2



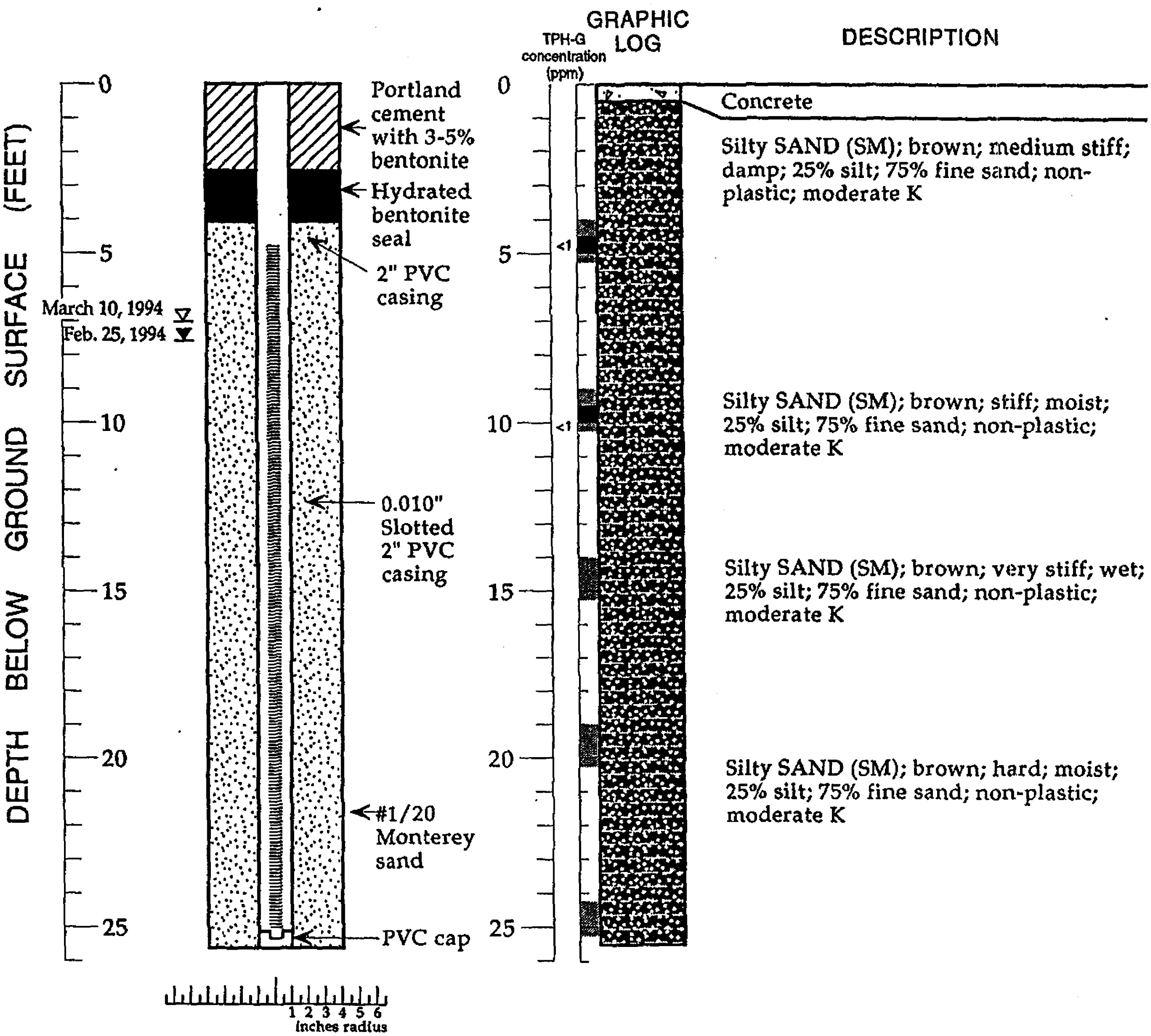
EXPLANATION

- Water level during drilling (date)
- Water level (date)
- Contact (dotted where approximate)
- —?—?— Uncertain contact
- Gradational contact
- Location of recovered drive sample
- Location of drive sample sealed for chemical analysis
- Cutting sample
- K = Estimated hydraulic conductivity

Logged By: Joyce Adams
Supervisor: James W. Carmody; CEG 1576
Drilling Company: Soils Exploration Services, Vacaville, CA
License Number: C57-582696
Driller: Tim Dunne
Drilling Method: Hollow-stem auger
Date Drilled: February 25, 1994
Well Head Completion: 2" locking well-plug, traffic-rated vault
Type of Sampler: Split barrel (2" ID)
Ground Surface Elevation: 29.44 feet above mean sea level
TPH-G: Total petroleum hydrocarbon as gasoline in soil by modified EPA Method 8015

Boring Log and Well Construction Details - Well MW-2 - Former Chevron Service Station #9-0100, 2428 Central Street, Alameda, California

WELL MW-3



EXPLANATION

- Water level during drilling (date)
- Water level (date)
- Contact (dotted where approximate)
- —?—?— Uncertain contact
- Gradational contact
- Location of recovered drive sample
- Location of drive sample sealed for chemical analysis
- Cutting sample
- K = Estimated hydraulic conductivity

Logged By: Joyce Adams
Supervisor: James W. Carmody; CEG 1576
Drilling Company: Soils Exploration Services, Vacaville, CA
License Number: #C57-582696
Driller: Tim Dunne
Drilling Method: Hollow-stem auger
Date Drilled: February 25, 1994
Well Head Completion: 2" locking well-plug, traffic-rated vault
Type of Sampler: Split barrel (2" ID)
Ground Surface Elevation: 30.36 feet above mean sea level
TPH-G: Total petroleum hydrocarbon as gasoline in soil by modified EPA Method 8015

Boring Log and Well Construction Details - Well MW-3 - Former Chevron Service Station #9-0100, 2428 Central Street, Alameda, California

c782-004.ai

Gettler-Ryan, Inc.

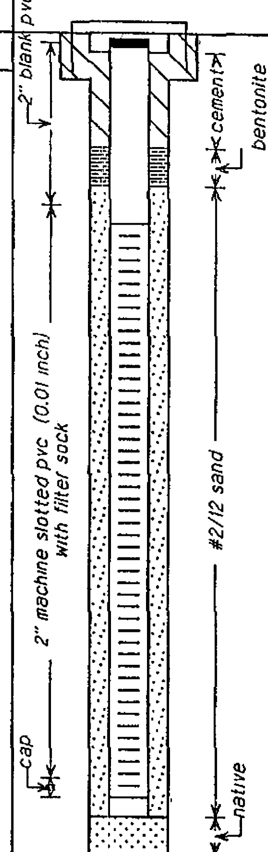
Log of Boring MW-4

PROJECT: *Former Chevron SS# 9-0100*	LOCATION: *2428 Central Avenue, Alameda, CA*
G-R PROJECT NO.: *5178.02*	SURFACE ELEVATION: *29.31 feet MSL*
DATE STARTED: *08/26/96*	WL (ft. bgs): *8.0* DATE: *08/26/96* TIME: *10:10*
DATE FINISHED: *08/26/96*	WL (ft. bgs): *8.0* DATE: *08/26/96* TIME: *12:00*
DRILLING METHOD: *8 in. Hollow Stem Auger*	TOTAL DEPTH: *21.5 Feet*
DRILLING COMPANY: *Bay Area Exploration, Inc.*	GEOLOGIST: *B. Sieminski*

DEPTH feet	PID (ppm)	BLOWS/FT. *	SAMPLE NUMBER	SOIL CLASS	GEOLOGIC DESCRIPTION
0					PAVEMENT - concrete over baserock.
				SP	SAND (SP) - yellowish brown (10YR 5/6), damp, medium dense; 100% fine sand.
4			MW4-4		Becomes moist; with up to 5% silt.
6	42	16	MW4-6		
7.5	3.5	26	MW4-7.5		Becomes saturated.
10		26			
16	1.1	34	MW4-16		Becomes dense; color change to light olive brown (2.5Y 5/6); flowing sand.
21	0	38	MW4-21		

(* = converted to equivalent standard penetration blows/ft.)

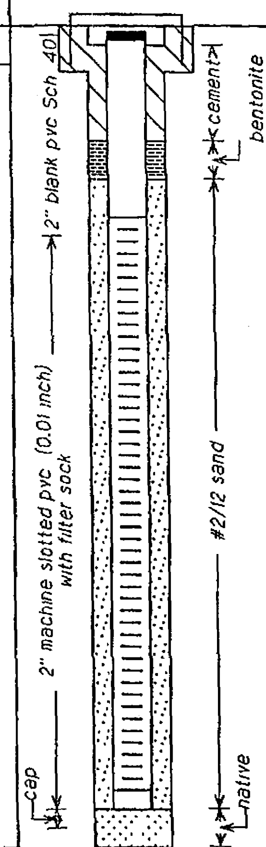
WELL DIAGRAM



Gettler-Ryan, Inc.	Log of Boring MW-5
PROJECT: *Former Chevron SS# 9-0100*	LOCATION: *2428 Central Avenue, Alameda, CA*
G-R PROJECT NO. : *5178.02*	SURFACE ELEVATION: *28.88 feet MSL*
DATE STARTED: *08/26/96*	WL (ft. bgs): *7.5* DATE: *08/26/96* TIME: *15:10*
DATE FINISHED: *08/26/96*	WL (ft. bgs): *7.5* DATE: *08/26/96* TIME: *16:30*
DRILLING METHOD: *8 in. Hollow Stem Auger*	TOTAL DEPTH: *21.5 Feet*
DRILLING COMPANY: *Bay Area Exploration, Inc.*	GEOLOGIST: *B. Sieminski*

DEPTH feet	PID (ppm)	BLOWS/FT. *	SAMPLE NUMBER	SOIL CLASS	GEOLOGIC DESCRIPTION
0					PAVEMENT - concrete over baserock.
				SP	SAND (SP) - yellowish brown (10YR 5/6), damp, medium dense; 95% fine sand, 5% silt.
5			MW5-5.5		Becomes moist.
	25	13	MW5-6		
	111	25	MW5-7		Becomes saturated.
10	8.3	26	MW5-11		
15	9.7	26	MW5-16		Color change to light olive brown (2.5Y 5/4); 100% fine to medium sand; flowing sand.
20	0	36	MW5-21		Becomes dense.
25					(* = converted to equivalent standard penetration blows/ft.)
30					
35					

WELL DIAGRAM



JOB NUMBER: *5178.02*

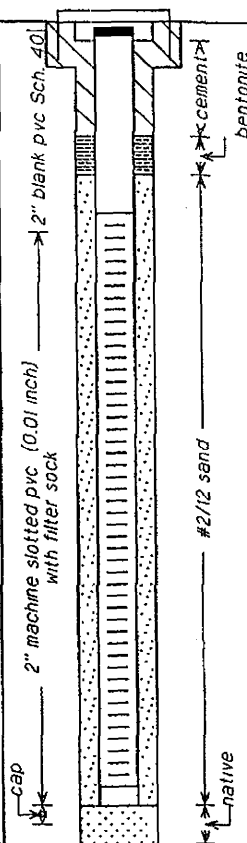
Gettler-Ryan, Inc.

Log of Boring MW-6

PROJECT: *Former Chevron SS# 9-0100*	LOCATION: *2428 Central Avenue, Alameda, CA*
G-R PROJECT NO.: *5178.02*	SURFACE ELEVATION: *29.24 feet MSL*
DATE STARTED: *08/26/96*	WL (ft. bgs): *7.9* DATE: *08/26/96* TIME: *12:30*
DATE FINISHED: *08/26/96*	WL (ft. bgs): *7.9* DATE: *08/26/96* TIME: *14:55*
DRILLING METHOD: *8 in. Hollow Stem Auger*	TOTAL DEPTH: *21.5 Feet*
DRILLING COMPANY: *Bay Area Exploration, Inc.*	GEOLOGIST: *B. Sieminski*

DEPTH feet	PID (ppm)	BLOWS/FT. *	SAMPLE NUMBER	SAMPLE INT.	GRAPHIC LOG	SOIL CLASS	GEOLOGIC DESCRIPTION
							PAVEMENT - concrete over baserock.
						SP	SAND (SP) - yellowish brown (10YR 5/6), damp, medium dense; 95% fine sand, 5% silt.
5	45	10	MW6-5.5 MW6-6				Becomes moist.
	48	20	MW6-7				
							▽▼ Becomes saturated.
10	35	36	MW6-11				Becomes dense.
15	25	38	MW6-16				Color changes to light olive brown (2.5Y 5/4); 100% fine to medium sand; flowing sand.
20	0	34	MW6-21				
25							(* = converted to equivalent standard penetration blows/ft.)
30							
35							

WELL DIAGRAM



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QUALITY CONTROL REVIEWER:

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