GEOTECHNICAL STUDY

PARKING STRUCTURE
MOORPARK COLLEGE

Prepared by

Geotechniques

February 2011

GEOTECHNIQUES
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February 25, 2011
Project No. 1003.026

Ventura County Community College District
c/o Heery International
103 Durley Avenue
Camarillo, California 93010

Attention: Mr. Dick Jones, Sr. Project Director

Subject: Geotechnical Study, Parking Structure, Moorpark College, Moorpark, California

Dear Mr. Jones:

This geotechnical study has been prepared for the proposed parking structure at Moorpark College. The study was conducted in general conformance with our proposal, dated June 8, 2010.

This report describes our field exploration, laboratory testing, interpretations of subsurface conditions, and recommendations for subgrade preparation and foundation design.

We appreciate the opportunity to work with you on this project. Please do not hesitate to contact us if you require further assistance on this or future projects.

Sincerely,

GEOTECHNIQUES

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1.0 INTRODUCTION

This geotechnical study is for the proposed parking structure to be constructed at Moorpark College, in Moorpark, California. The structure site is located in the lower campus area, south of the gymnasium and immediately southeast of the tennis courts. Our services for this project were authorized by execution of Ventura County Community College District's Standard Consultant Agreement by Chancellor James Meznek, on September 14, 2010.

1.1 PURPOSE AND SCOPE

The purpose of our geotechnical study was to explore and evaluate the geotechnical conditions at the site and to provide geotechnical opinions and recommendations concerning the proposed parking structure and foundation subgrade preparation and design.

Our understanding of the proposed project and the general scope of geotechnical services provided for this study are based on communications with Mssrs. Shahin Azmoudeh, Architect, and Edward Tan, Structural Engineer, with International Parking Design ("IPD"), and Mssrs. Glen Pace and Tristan Santos, Civil Engineers from Penfield and Smith. Information from our meeting of June 3, 2010, and subsequent telephone and email communications, preliminary schematic layouts and plans, field exploration, and laboratory testing have been used in preparing this report.

1.2 WORK PERFORMED

Our scope of services for the proposed project was described in our proposal, dated June 8, 2010. A summary of the proposed scope of work is outlined below:

- Review of existing geotechnical reports prepared for nearby campus facilities,
- Review of available schematic and preliminary plans for the proposed parking structure and adjacent improvements;
- Drilling and sampling of four hollow-stem auger borings to depths between about 30 and 50½ feet below the ground surface in footprint of the proposed parking structure;
- Advancing six cone penetrometer test soundings to depths between about 21 and 31½ feet below the ground surface in the footprint of the proposed parking structure,
- Laboratory testing of selected soil samples obtained from the exploratory borings;
- Evaluating field and laboratory test data, assessing and organizing data, and performing engineering analyses; and
- Preparing this written report with graphics.

On the basis of the data obtained for the site, we have provided our geotechnical opinions and recommendations regarding:

- Soil and groundwater conditions at the parking structure site;
- Potential for geologic hazards consisting of fault rupture, liquefaction, seismically induced settlement, and expansive/collapsible soils to impact the site,
- Estimates of peak horizontal ground accelerations (pga) and response spectra for probabilistic, deterministic, and design Maximum Considered Earthquake ("MCE")
- Seismic design parameters derived from the 2010 California Building Code.
- Site subgrade preparation, grading, and compaction recommendations for fill placement;
- Recommendations for material selection and evaluating the suitability of onsite soils for use as fill;
- Suitability of site soils for stormwater infiltration;
- Temporary excavations;
- Requirements for imported soil and fill materials placed below building improvement areas;
- Design of shallow foundations, including allowable bearing pressure and estimated total and differential settlements, and subgrade modulus;
- Where appropriate, design of deep foundations, consisting of allowable compressive and uplift capacities, lateral capacities, pile embedment lengths, and drilled pile construction considerations;
- Resistance to lateral loads, passive earth pressures, and frictional coefficients;
- Static and dynamic lateral earth pressures for restrained retaining walls,
- Design considerations for slabs-on-grade,
- Soil corrosivity considerations;
- Asphalt concrete and concrete pavement sections; and
- Utility trench backfill.

Subsurface geologic mapping of the parking structure site was not included in the scope of work for this geotechnical study. Our discussion on the general geology of the campus area was based on published geologic maps and other published documents as described in Section 4.0.

1.3 PROJECT PERSONNEL

The following key design personnel are associated with this project under the Project Director, Mr. Dick Jones, of Heery International, on behalf of Ventura County Community College District:

- Mr. Shahin Azmoudeh, Architect, International Parking Design ("IPD")
- Mr. Edward Tan, Structural Engineer, International Parking Design
- Mr. Glen Pace, Civil Engineer, Penfield & Smith,
- Mr. Tristan Santos, Civil Engineer, Penfield & Smith

2.0 LOCATION AND DESCRIPTION OF SITE

2.1 LOCATION

The location of the proposed parking structure site is shown on Plate 1 - Vicinity Map. The subject site is situated between the northwestern third of Parking Lot G and the campus tennis courts to the east and west, respectively, and is bounded on the north by the south gymnasium road, and on the south by a soccer field. Plate 1 shows the site location relative to nearby facilities and improvements. The site is currently used as an overflow car parking area.

2.2 TOPOGRAPHY

Original campus grading was performed in the mid-1960s. According to site topography on Plate 2 – Site Layout and Exploration Plan, the ground surface at the site slopes gently down to the southwest at less than 3 percent. The site perimeter to the east and north slopes up about 15 feet to Parking Lot G and the south gymnasium road, respectively, at a gradient between about 2h:1v and 1½h:1v. The site slopes up between about 5 and 10 feet to the southwest and northeast edges of the tennis courts, respectively, at a gradient of about 2h:1v. In general, level areas such as Parking Lot G and the subject site were established during original campus development through 1960s-era cut/fill grading operations, whereby the southmost edges of those level-graded areas received between about 5 and 15 feet of fill from the west to the east, respectively.

3.0 PROJECT DESCRIPTION

Plate 2 shows the location of the proposed parking structure, which shall incorporate a campus police station at the lower two levels of the northwest end of the structure.

3.1 Proposed Site Setting

The parking structure will be constructed below-grade into the eastern and northern flanks of the existing slopes along the westerly quarter of Parking Lot G and along the southerly shoulder of the south gymnasium road. Access from the south gymnasium road will tie into a northeastern entrance on the second parking level at about El. 651 feet. The lower parking level will be accessed at the southeasterly corner of the proposed structure at the overflow parking field grade, or about El. 640 feet.

3.2 Proposed Structure

The footprint of the rectangular parking structure is anticipated to be about 180 feet by 275 feet, with the long axis of the structure parallel to the south gymnasium road According to the Structural Engineer, Mr. Edward Tan of IPD, maximum anticipated concentrated loads for the four-level, poured-in-place moment frame parking structure with post tension slab consist of the following:

- Interior girder columns with dead load of 600 kips and live load of 150 kips,
- Typical interior column loads with 400 kip dead and 100 kip live load, and
- Typical exterior column loads with 240 kip dead and 60 kip live loads.

4.0 GENERAL SUMMARY OF SITE CONDITIONS

Our geotechnical study for the proposed parking structure consisted of reviewing geotechnical studies, aerial photographs, and published geology maps and reports relevant to the Moorpark College campus area. Subsequent to our literature review, we performed field exploration and laboratory testing together with geotechnical evaluation of the resulting data. The field exploration program consisted of advancing six cone penetrometer test (CPT) soundings until refusal was met at each location, at depths ranging between about 21 and 31½ feet below the ground surface, followed by drilling and sampling four exploratory borings to depths between about 30½ feet and 50½ feet below the existing ground surface. Logs of the borings and CPTs are presented in Appendix A. Plates 3.1 and 3.2 – Cross Section A-A' and Cross Section B-B', show existing and proposed grades relative to the proposed parking structure and subsurface strata encountered at several exploration locations along orthogonal projections of the long and short axes of the building envelope.

Laboratory testing was conducted on selected soil samples obtained from the borings to characterize general geotechnical engineering properties of the soil. The field and laboratory data for this study are presented in Appendices A and B, respectively.

4.1 GEOLOGIC SETTING

Plates 4, 5, and 6 - Regional Geologic Maps, present the surficial geology of the Moorpark College campus area mapped by Dibblee (1992), Irvine (1990), and Weber et al. (1973). According to those maps, the parking structure site is underlain by older alluvium of late-Pleistocene to early-Holocene age. The older alluvium unconformably overlies Pleistocene-age Saugus Formation and Oligocene-age Sespe Formation.

Oil wells Nos. 8 and 12 located within about 2,500 feet of the proposed parking structure site are part of the "Oak Park Field" located immediately east of the campus (California Division of Oil and Gas, 1973) and penetrate reservoir beds of the Sespe

Formation. The approximate location of the Oak Park Field is shown on Plates 4 and 6, and the locations of the individual wells within the oil field are shown on Plate 7 – Canada de la Brea Fault Projections.

Hazard maps by CGS (2001) and Ventura County (2007) do not show landslides or areas susceptible to earthquake-induced landslides on the Ventura College campus. Landslides are not mapped on or immediately adjacent to the campus according to Dibblee (1992), Irvine (1990), and Weber et al. (1973).

4.2 EARTH MATERIALS

Descriptions of soil conditions presented herein are based on visual classification of samples obtained from our field exploration and the results of subsequent geotechnical laboratory testing.

In general, subsurface conditions encountered in the borings consist predominantly of Quaternary-age clayey and sandy older alluvium. Artificial fill placed during original campus development in the mid-1960s generally was encountered along the southerly and westerly margins of the site, both at Parking Lot G and in the overflow parking field.

4.2.1 Artificial Fill

Artificial fill (Qaf) was encountered in the upper several feet of Borings Nos. 1 and 2 located at the north-and southwesterly corners of the field area and in Boring No. 10, located at the top of the southerly fill slope of Parking Lot G, near the southeastern corner of the site. Fill thicknesses generally appear to decrease to the northeast, where the toe of the graded slope along the gymnasium road appears to be underlain by native clayey materials.

Artificial fill materials encountered in the borings typically were very dense to very firm or hard, as suggested by standard penetration test (SPT) blow count data which ranged from about 35 to 80 blows per foot (bpf) after corrections for overburden pressure, sampler type, sampler rod length, and driving hammer type were applied to raw field data.

4.2.2 Older Alluvium (Qoa)

Quaternary older alluvial deposits (Qoa) underlie artificial fill materials, where encountered. Older alluvium generally consisted of clayey sand (SC) and sandy lean clay (CL), with silty sand (SM) to sand with varying amounts of gravel and cobbles (SP) encountered below about El. 613 feet, or about 29 feet below the ground surface in Boring B-1, below about 18 feet below the ground surface in Boring B-2 (about El. 621 feet), and below about 10 feet, or El. 631 feet in Boring B-8 and El. 639 feet in Boring B-10. The sand and gravel materials likely are derived from the Saugus Formation.

4.2.3 Engineering Properties of Alluvium

Dry densities of samples of clayey sand and lean clay earth materials ranged from about 102 to 124 pounds per cubic foot (pcf) with moisture contents of those samples ranging from about 9 to 20 percent. Dry densities of samples of sandy earth materials ranged from about 102 to 119 pounds per cubic foot (pcf) with moisture contents of those samples ranging from about 6 to 24 percent.

The expansion index (EI) measured from a sample of very clayey sand encountered at about 5 feet in Boring B-1 was 2.

Driving resistance of the soil samplers used during the field exploration, in terms of field blowcounts, is shown on the boring logs in Appendix A. Field blowcounts were corrected for hammer energy, depth, sampler type, and rod length. Corrected blowcounts in sandy layers ranged from about 33 to well over 100 bpf, with average blowcounts in excess of 50 bpf.

Normalized (to 1 tsf) cone tip resistance values for older alluvial materials between depths of about 10 and 20 feet typically ranged from about 50 to 75 tons per square foot (tsf) for clays and 75 to 100 tsf for clayey sand/sandy clay. Below about El. 620 to 625 feet, normalized tip resistance values in the Saugus formation sands typically averaged between about 200 and 300 psf, with refusal met below about El. 615 and 618 feet.

Consolidation test results presented on Plates B-4.1 and B-4.2 in Appendix B suggest that sandy clay older alluvium at a depth of 15 feet in Boring B-1 has a recompression ratio, $C_{\varepsilon r}$, of about 0.01; and clayey fine sand older alluvium in Boring B-2 at a depth of 20 feet has a recompression ratio, $C_{\varepsilon r}$, of about 0.009. The compression ratio, $C_{\varepsilon c}$, for the sandy clay from Boring B-1 was about 0.13 and for the clayey sand from Boring B-2 was about 0.04. Consolidation test results suggest that the clayey older alluvial materials are overconsolidated, with an overconsolidation ratio (OCR) of about 8 in the sample of sandy clay from Boring B-1 at a depth of 15 feet, and an overconsolidation ratio of about 2 in the sample of clayey sand from Boring B-2 at a depth of 20 feet. Hydroconsolidation tests also were performed on the liner samples. The tests were performed by loading the samples to the approximate overburden stress and subsequently inundating the samples. Test results suggest a collapse strain of about 1 percent in the sandy clay sample from Boring B-1 at a depth of 15 feet. No hydroconsolidation potential was observed in the sample of clayey sand from Boring B-2 at a depth of 20 feet.

4.3 GROUNDWATER

Groundwater was not encountered during drilling in the borings to the maximum exploration depth of 50½ feet below existing grade in the lower level area. Further, groundwater was not encountered to a depth of about 51.5 feet (El. 606 feet) in an earlier exploration performed for the nearby Fitness Center site located immediately east/southeast of the gymnasium. At the lower end of the campus, groundwater was not encountered in borings performed at the athletic field to a maximum depth of about 21 feet (corresponds to El. 613 feet), but was encountered at the maintenance facility site located approximately 1,000 feet southwest of the site, between the athletic field and Collins Drive, at a depth of about 34 feet (corresponds to El. 593 feet).

In general, older alluvial materials encountered were typically slightly moist to moist, with the exception of a layer of very dense sandy older alluvium with calcium carbonate inclusions encountered below a depth of about 34 feet in Boring B-1, with moisture contents of about 23 and 24 percent.

Variations in soil moisture may occur and localized perched water conditions or seeps, while not encountered, may develop as a result of rainfall, irrigation, runoff, and other factors.

4.4 SEISMIC CONSIDERATIONS

The seismicity evaluation for the project site and presented in this report consisted of the assessment of earthquake hazards such as strong ground motion, liquefaction, liquefaction and seismically induced settlements, lateral movements, and fault rupture.

4.4.1 General

The project site location is in the immediate vicinity of 34.2988º N latitude and 118.8332º W longitude, and is located in the seismically active southern California area. Several active or potentially active faults are known or postulated to exist within about 10 miles of the campus site including the Simi-Santa Rosa, Oak Ridge, Santa Susana, and San Cayetano faults.

Table 1 presents a summary of the approximate distances from the site coordinates and the maximum magnitudes for some of the nearby fault sources that may cause future shaking at the parking structure site.

Table 1. Significant Faults

Fault Name	Approximate Distance from Site (Miles)*	Estimated Maximum Magnitude, M_w
Simi-Santa Rosa	0	6.7
Oak Ridge (Onshore)	1.5	6.9
Santa Susana	5	6.6
San Cayetano	7	6.8
Holser	7	6.5
Anacapa-Dume	11	7.3
Malibu Coast	16	6.7
San Gabriel	17	7.0
Ventura-Pitas Point	18	6.8
Santa Ynez (East)	18	7.0
San Andreas	32	7.8

* Distance from site to closest point on the vertical projection of the fault plane to the ground surface.

4.4.2 Historical Seismicity

The historical record indicates that the Moorpark College campus area has experienced shaking from a number of seismic events over the course of the last two centuries. Some of the seismic events that likely resulted in varying degrees of ground motion at the site are the earthquakes of 1812, 1827, 1852, 1855, 1857, 1893, 1936, 1952, 1956, 1965, 1971, 1974, 1977, 1991, and 1994. The 1812 and 1857 events are thought to have occurred along the Mojave Segment of the San Andreas fault and caused significant damage to developed areas of southern and central California. Those earthquakes were estimated to have had moment magnitudes of approximately M7.1 and M7.8, respectively. The 1952 Tehachapi earthquake had an estimated moment magnitude of M7.7. Strong historical ground motion dislodged ceiling tiles in campus facilities during the M6.4, 1971 San Fernando earthquake. The campus is located about 30 miles northwest of the epicenter of the M6.7, 1994 Northridge earthquake, and the pga from that earthquake was estimated to have been about 0.3 to 0.4g or less (Stewart et al., 1994). A strong motion instrument located at the fire station in downtown Moorpark recorded a peak horizontal acceleration of 0.3g from the Northridge earthquake (CDMG, 1994). Both of those earthquakes produced over one dozen significant aftershocks.

4.4.3 Potential Seismicity and Faults

Potential Seismicity. Ventura County is the only county in southern California that has not directly experienced the effects of a devastating historical earthquake on a fault within its boundaries (Weber et al., 1973). That dormancy is inconsistent with the active tectonic framework of the county. There are numerous regional and local active faults within and extending through the county that pose a seismic risk to the region.

Geodetic surveys suggest that the Ventura basin is experiencing crustal shortening in a north-south direction at a rate of about 16 to 38 millimeters per year (mm/y) over the past 200,000 to 300,000 years (Huftile, 1995). Based on that rate, the Ventura region should have experienced the equivalent of two moment-magnitude 7.5 earthquakes during the last 200 years. However, other than the San Andreas fault, which crosses the northeastern corner of Ventura County, no large-magnitude earthquakes have occurred over that period along the Simi-Santa Rosa, Oak Ridge, San Cayetano, or other faults in the county. Portions of Ventura County have been affected by earthquakes occurring in other geographic regions, such as the damage in Fillmore and Simi Valley due to the January 17, 1994, Northridge earthquake (magnitude 6.7).

That relative earthquake quiescence in Ventura County is concerning because portions of Ventura County exhibit some of the greatest Quaternary deformation rates in California and the world. For instance, the Ventura anticline, located about 20 miles west of the site, has exhibited uplift rates of about 6 to 7 millimeters per year (mm/yr) for the last 40,000 to 100,000 years (Lajoie et al., 1982). That rate compares with typical coastal terrace uplift rates in other areas of California of about 0.3 to 0.5 mm/yr. The high deformation rate implies a high tectonic activity rate for the region, which has not historically been experienced.

Faults. The site is located about 1¼ miles north of the main surface trace of the Simi-Santa Rosa (reverse) fault; however, the parking structure site is located on the hanging wall of that thrust fault. The Simi-Santa Rosa fault is included in the Alquist-Priolo Earthquake Zone (CDMG, 1999). The Simi-Santa Rosa fault has a recurrence interval of about 1,000 years and an average displacement of about 1 meter per event.

The trace of the south-dipping Oak Ridge (Onshore) thrust fault is located about 2 miles north of the site. Studies of the Oak Ridge fault estimate an average slip-rate of about 5 mm/yr (Yeats and Huftile, 1995). Petersen et al. (1996) estimates the recurrence interval for earthquakes along the Oak Ridge fault at about 291 years. Because of that rate, and the large average slip-rate and length, the Oak Ridge fault poses a seismic hazard for the subject site.

4.4.4 Ground Rupture Potential

Mapped Faults. As shown on Plate 8 – Portion of California Earthquake Fault Zones, Simi Valley West Quadrangle, the parking structure site is located about 1¼ miles north of the inferred surface trace of the Simi-Santa Rosa fault and the designated Alquist-Priolo Earthquake Fault Zone (CDMG, 1999). Plate 5 – Regional Geologic Map from Irvine (1990), shows the inferred buried trace of the west/southwest-east/northeast striking Canada de la Brea (CDLB) fault about 300 feet south of the parking structure site (Irvine, 1990). Additionally, using oil well data from the California Division of Oil and Gas (CDOG, 1973) for the Oak Park Field, Bright (CDOG, 1973) presents an east-west striking buried trace of the CDLB fault that projects toward the general vicinity of the Child Development and Academic Centers, between about 400 and 800 feet northeast of the parking structure site. Geologic interpretation of the oil well data for the Oak Park Field by the CDOG suggests that the CDLB fault is a steeply (80 degrees) northward dipping reverse fault that offsets strata of the Sespe Formation by about 800 feet. Work by Bright suggests that the Sespe Formation strata in the Oak Park oil field is Oligocene age (approximately 23 to 37 million years old). Plate 7 – Canada de la Brea Fault Projections, shows the projected CDOG fault trace and the Irvine (1990) inferred buried fault trace relative to the proposed parking structure.

Aerial Photograph Review. A southwest-northeast trending lineament was observed east of the Moorpark College campus area within the Sespe Formation bedrock outcroppings on the 1953 USDA stereographic aerial pairs. The lineament appears to coincide with the mapped location of the CDLB fault by Irvine, but is not visible within the southern portion of the Moorpark College campus, which is covered by older alluvium. No lineaments were observed to traverse through the parking structure site.

Previous Fault Studies. Geotechnical Associates (GA, 1999) performed a geologic study of several faults including the CDLB for Simi Valley Tract No. 5182. Their site is located in Sand Canyon, north of Erringer Road/SR 118, and about 3 miles east of Moorpark College. Trenching by GA located the CDLB fault within the Sespe Formation bedrock near the location shown on regional geologic maps. Supplemental trenching by GA identified unfaulted alluvial sediments greater than 11,000 years overlying the trace of the CDLB in the Sespe Formation strata. On the basis of their trenching, GA concluded that the CDLB fault was not active in the Sand Canyon area of Simi Valley.

In the immediate area of the proposed parking structure, a fault study was performed for the Child Development Center (CDC) site (Fugro, 2003). The fault study consisted of the excavation of a backhoe trench (location shown on Plate 7) adjacent to the proposed building footprint and oriented roughly perpendicular to the CDOG-projected strike of the CDLB fault. The fault trench exposed primarily granular older alluvial deposits that were thinly to thickly bedded. Geologic features suggestive of faulting were not observed in the backhoe trench. As depicted on Plate 7, the backhoe trench excavated for the CDC site appears to cover the westward projection of the CDOG-mapped CDLB fault toward the southern half of the east campus area. Additionally, features suggestive of faulting were not observed during mapping of an approximately 200 foot-long, 15- to 20-foot variable height excavation slope (location shown on Plate 7) for the Academic Center building pad (Geotechniques, 2008).

Conclusions. Lineaments were not observed on aerial photographs traversing the southern half of the Moorpark College campus. Evidence of faulting was not observed on grading slope excavations at the Academic Center site and on Child Development Center fault trench slopes excavated roughly perpendicular to the westward projection of the Canada de la Brea fault mapped by Bright (CDOG, 1973). The fault study performed by GA about 3 miles east of the site indicates that the mapped trace of the CDLB fault does not offset sediments younger than 11,000 years. On the basis of the foregoing, the CDLB fault would be classified as potentially active. Per State of California guidelines (Alquist-Priolo Act), trenching does not have to be performed for potentially active faults. However, because evidence of the CDLB fault has not been observed in 1) a fault trench excavated at its westward projection, 2) subsequent cut slopes during grading, and 3) aerial photographs of the southern half of the campus, the potential for fault rupture associated with the CDLB fault at the parking structure site appears to be low.

Slope excavations, particularly the approximately 200-foot-long cut that is anticipated to range in height from about 24 feet at the southeastern end to almost 30 feet at the northeastern end of the eastern pad area along the eastern slope exposed during foundation area grading should be observed and mapped by the Engineering Geologist concurrent with earthmoving operations.

4.4.5 Strong Ground Motion Estimates

Site-specific probabilistic seismic hazard analyses were performed to estimate strong ground shaking parameters at the project site due to a seismic event having a 2 percent probability of exceedance in 50 years. This event also is known as the Maximum Considered Earthquake (MCE). The computer program OpenSHA, version 1.02, was used to calculate the Probabilistic MCE. An average across three different ground motion prediction equations (GMPEs) was used: Boore et al. (1997) for V_{s30} = 365 meters/sec, Sadigh et al. (1997) for soil, and Campbell and Bozorgnia [(2003) for alluvium. The probabilistic MCE was compared to the ASCE 7-05 Design Spectrum, the ASCE 7-05 Deterministic MCE, and 150% of the deterministic MCE envelope calculated using the fault magnitudes and distances shown in the Table 1. The final design MCE was constructed from the probabilistic MCE based upon the guidelines provided in ASCE 7-05. The MCE Uniform Hazard Spectrum and data for probabilistic, deterministic, and design MCE are presented on Plate - 9 - Uniform Hazard Spectra and Data for Probabilistic MCE, Deterministic MCE, and Design MCE.

For purposes of calculating liquefaction and seismically induced settlement potential, a pga on the order of 0.49g is appropriate for use with a magnitude 7.5 earthquake according CGS Note 48 (2011).

4.4.6 2010 California Building Code Design Criteria

Utilizing California Building Code (CBC) (ICBO, 2010) descriptions, the Site Class can be considered type D, which is characterized by undrained shear strengths typically between about 1,000 and 2,000 pounds per square foot (psf) and average (uncorrected) blow counts between 15 and 50. Relevant site seismic coefficients consistent with Chapter 16 of the 2010 CBC are summarized below:

- S_S 1.825
- S_1….0.702
- F_a.....1.0
- F_v.....1.5
- S_{MS} ..1.825
- S_{M1}…1.053
- S_{DS} ...1.217
- S_{D1}….0.702

4.4.7 Vertical Motions

Although specific analyses were not performed for vertical peak ground acceleration, we suggest that vertical components of motion be taken as equal to the horizontal component, consistent with the results of a study by Bozorgnia et al. (1999).

4.4.8 Liquefaction

General. Soil liquefaction is generally defined as the temporary buildup of excess pore water pressure resulting in a condition of near zero effective stress and the temporary loss of strength induced by earthquake ground shaking. Loose saturated sands and non-plastic silts are considered susceptible to liquefaction. Clayey soils and bedrock are typically considered non-liquefiable.

According to Seed (1979), typical subsurface conditions observed at most sites where liquefaction has occurred include: 1) groundwater is shallower than a depth of about 15 feet, and 2) liquefied layers are shallower than a depth of about 45 feet. However, Seed (1979) states that those conditions should not be construed to suggest that liquefaction cannot be induced at greater depths in response to earthquake shaking.

Depth to Groundwater. Groundwater was not encountered in the borings to the maximum exploration depth, or to El. 606 feet. Groundwater was encountered at the maintenance facility site, located approximately 1,000 feet southwest of the proposed parking structure and about 15 feet lower in elevation than the parking structure site, at a depth of about 34 feet below the ground surface, corresponding to El. 593 feet. Although free water was not encountered in the borings for this project site, very moist conditions were encountered in a very dense silty sand layer in Boring B-1 below a depth of about 34 feet.

Additionally, according to the Seismic Hazards Zone Report (CDMG, 2001),the subject site at Moorpark College campus is located well outside the approximate boundary of areas within which historic groundwater levels have been shallower than 40 feet.

Grain-Size Characteristics and Consistency. Soils in the upper 30 to 50 feet are predominantly clayey sand to sandy lean clay, typically underlain by very dense silty sands to sands with gravel. Clayey soils generally are not considered susceptible to liquefaction. The average corrected blowcounts for samples of sandy layers common below a depth of about 30 feet was over 50 bpf, suggesting very dense conditions.

Liquefaction Potential. Because present and historical groundwater data do not suggest groundwater above about El. 593 feet at the campus, and also because sandy soils encountered below El. 620 feet for this study are very dense, liquefaction potential at the parking structure site is considered low.

4.4.9 Lateral Movements

Lateral movement may occur when a soil mass "rides" on liquefied soil layers, carried downslope or toward a free face. Procedures for estimating large-scale lateral movements have been developed by Bartlett and Youd (1995). Their empirically derived procedures for estimating lateral movements depend on earthquake magnitude, distance between the site and the seismic event, ground slope or ratio of free-face height to distance between free face and structure, thickness of liquefied layer, fines content, average particle size of the material comprising the liquefied layer, and N-value.

However, because of the low liquefaction potential from absence of groundwater in the upper 34 feet (i.e., above El. 606 feet) and the very dense slightly cemented sandy materials encountered below the upper 10 to 30 feet, lateral spreading is unlikely at the parking structure site.

4.4.10 Seismically Induced Settlement of Dry Sands

Seismically induced settlement can occur in unsaturated (i.e., above the groundwater table) sandy soils that are loose to medium dense. The procedure typically used to estimate seismically induced settlement of unsaturated granular materials (Tokimatsu and Seed, 1987) generally applies to non-plastic soil materials with less than 10 to 15 percent clay-sized particles. The extension of that procedure to fine-grained soils, in our opinion, is over-conservative. In unsaturated clayey sand or sandy clays, capillary stresses result in negative pore pressures and large effective stresses. That condition in fine-grained soils would tend to limit slip between grains, which is needed for volumetric strain to occur. Hence, for unsaturated sandy clay or clayey sand, volumetric strain (which would produce settlement) resulting from cyclic strain should be minor.

Despite the unlikelihood of the clayey (i.e., plastic) alluvial materials to experience volumetric strain, sandy materials, including the stiff, cohesive, clayey sand materials were evaluated for seismically induced settlement potential using procedures presented in Pradel (1998). Seismically induced settlement was estimated at up to about ½ inch. Calculations for seismically induced settlement are presented in Appendix C.

4.4.11 Tsunami, Seiche and Flooding Hazard

According to Ventura County (Ventura County Board of Supervisors, 2007), the maximum tsunami run up elevation for most of Ventura County is about +50 feet msl. The project site is located above elevation +630 feet MSL datum and at least 30 miles north of the Pacific Ocean. Additionally, no enclosed bodies of water, such as lakes or reservoirs, are located near the parking structure site. Therefore, tsunamis and seiches are unlikely to impact the site.

Additionally, the campus is located at least ¼ mile north of the closest 100-year flood zone (for Arroyo Simi) and is situated on a slope with an overall gradient of about 10 percent down to the southwest. The potential for flooding impacts to the site from 100-year events on nearby rivers and streams is negligible. The campus also lies above the inundation path from a breach of the closest reservoir, the Wood Ranch Reservoir (City of Moorpark, 2001).

5.0 OVERVIEW OF SITE GEOTECHNICAL CONSIDERATIONS

This section briefly describes geotechnical considerations affecting site development and performance that should be mitigated and/or accommodated in foundation subgrade preparation and overall project design. Geotechnical mitigative measures are presented subsequently.

5.1 HYDROCONSOLIDATION

Hydroconsolidation is a phenomenon whereby soil structure collapses (or settles) when wetted. Natural deposits susceptible to hydroconsolidation typically are aeolian (wind-blown), alluvial, or colluvial materials, deposited with a meta-stable structure, which may exhibit a high apparent strength when dry. That dry strength may be attributed to the clay and silt constituency of the soil, and the presence of salts. Additionally, capillary tension may act to "bond" soil grains. Once those soils are wetted, the constituency, including soluble salts or "bonding" agents, is weakened or dissolved, capillary tensions are reduced, and collapse of the soil structure occurs.

Hydroconsolidation can occur to considerable depths if water infiltrates the subsurface over an extended area for a sustained period. Homogeneous soil (with a uniform collapse potential) subjected to a laterally- and vertically-uniform introduction of water would settle somewhat uniformly. However, because soils typically are not homogeneous, potential collapse settlement likely will vary across a site and with depth. More significantly, the degree of saturation likely will vary across a site, because water introduction into the subsurface can vary greatly, particularly if a pipe leaks undetected over a long period of time or if a pipe breakage occurs.

Settlement from hydroconsolidation occurs gradually and cumulatively, as soils along the advancing plume of water are wetted. Therefore, total estimated collapse settlement occurs no more “suddenly” than the time required for water to infiltrate the full depth of vulnerable soils. This progression allows for intervention and repair of the water leak, as structure distress resulting from earlier stages of infiltration and collapse settlement instigates troubleshooting and expeditious leak repair.

Further, once soils are saturated, hydroconsolidation potential is fully realized. Subsequent episodes of wetting of previously saturated soil do not result in additional hydroconsolidation settlement or collapse, as this phenomenon occurs only once. Therefore, soils within depths typically saturated from seasonal or irrigation infiltration are no longer anticipated to undergo hydroconsolidation.

A hydroconsolidation test on sample from Boring B-1 at a depth of 15 feet suggests a potential for collapse settlement in the clayey older alluvium, with strain on the order of about 1 percent. Essentially no hydroconsolidation was observed in a sample of sandy older alluvium from Boring B-2 at a depth of 20 feet. Moreover, hydroconsolidation potential is not likely in the sandy soils encountered below a depth of about 13 and 22 feet because of the typically very dense consistency of those materials. Hence, we estimate that potential collapse settlement could be on the order of up to about 2 inches were clayey soils to become wetted to a depth as great as about 20 feet below existing grade.

Localized events such as leaks would result in differential settlement of native soils between wetted and adjacent unaffected (i.e., unwetted) areas.

Differential settlement, rather than uniform areal settlement, more likely results in structure distress. Therefore, the potential for concentrated water seepage into the subsurface soils should be minimized by controlling and maintaining onsite drainage, avoiding or minimizing the extent of underground wet utilities beneath of near foundations, and providing a relatively impermeable cap of clayey soils in the compacted fill zone beneath the foundation. .

Options to reduce the potential for collapse settlement impacts on the foundation consist of overexcavation and recompaction of soils exhibiting the collapse potential, stiffening the foundation or designing the foundation as a mat to better accommodate differential settlement, and/or implementation of measures to prevent subsurface water infiltration and protect foundation soils from the effects of external water sources, such as from irrigation, leaking water lines, sewer lines, and storm drains. For example, overexcavation and recompaction of the upper 10 to 12 feet of earth materials should mitigate the potential for seismically induced settlement and hydroconsolidation in those materials, thereby reducing the potential for hydroconsolidation by at least 1 inch. Moreover, facilities personnel should routinely be apprised of the importance of maintaining site drainage and preventing water infiltration into the subsurface.

5.2 ARTIFICIAL FILL

Three of the four borings advanced for this study encountered artificial fill in the upper 5 to 12 feet below existing grade. The fill most likely was placed during original campus development in the mid-1960s. Fill placed at that time in greenbelt areas, outside building pad areas, was, according to the original report (LeRoy Crandall, 1965, 1966), compacted to a lesser standard than that for building areas, or 85 percent relative compaction instead of 90 percent relative compaction. Moreover, compaction at that time was based on a lower compactive energy (Standard Proctor versus Modified Proctor) and procedural standard than today (three layer instead of current five layer curve). Old artificial fill materials placed with procedures and standards from the 1960s likely would be more susceptible to settlement than those placed under more stringent standards today.

Hence, previously-placed artificial fill materials should be removed in the building area, and any original topsoil materials that may be encountered below the original fill contact also should be removed and replaced with suitable fill materials compacted consistent with the recommendations presented subsequently.

6.0 RECOMMENDATIONS

6.1 SITE DEVELOPMENT AND GRADING

To reduce the potential for differential settlement from previously-placed fill materials and hydroconsolidation of undisturbed native materials, we recommend overexcavation and recompaction of soils in the building footprint to a distance to 10 feet beyond the perimeter. The parking structure loads may be applied to the subsurface with spread footings to accommodate concentrated loads, or, to reduce differential settlements further, a shallow mat. Overexcavation of the structure foundation footprint should extend a minimum of 7 feet below the elevation of the deepest footing bottom to a distance of 10 feet beyond the outside edge of perimeter footings. Removals should be deepened as necessary to remove all previously-placed artificial fill materials and any underlying original topsoil or organic material. Fill materials placed in the building area should be compacted to a minimum of 95 percent of the maximum dry density as determined by ASTM D1557.

6.1.1 Site Preparation

Prior to commencing earthmoving operations, underground utilities in conflict with overexcavation and recompaction activities in the building area should be removed and rerouted. Pertinent areas should be cleared of below-grade structures, debris, organics, pavement, abandoned utilities, or other unsuitable materials. Those unsuitable materials should be stripped from improvement areas and discarded offsite.

Depressions or disturbed areas left from the removal of such material should be replaced with compacted fill, in accordance with Section 6.1.8.

6.1.2 Excavation Considerations

Grading and excavation can be performed with conventional heavy-duty and, where applicable, limited-access and/or manual earthmoving equipment in good working order.

Temporary Excavation Slopes. Temporary slopes and excavations should conform to federal Occupational Safety and Health Administration (OSHA) and/or California Division of Occupational Safety and Health (DOSH) regulations and other applicable local ordinances and building codes, as required. The contractor should be made responsible for all safety issues affecting open excavations. Temporary slopes should be continuously monitored by the contractor and loose or unstable soil masses should be removed immediately. Caving and running of cut slopes in sandy alluvial materials should be anticipated. Stockpiled material or equipment should not be placed within a distance from the slope crest equal to the height of the slope.

Soils disturbed or loosened during the installation and/or removal of temporary excavation support systems, such as shoring, should be restored to original or higher in-place densities through overexcavation and recompaction along the affected area using an excavator of adequate reach with a sheepsfoot compacting wheel, or through an equivalent operation approved in advance by the Engineer during the submittal process.

Surface water runoff should be directed away from temporary slopes and should not be allowed to flow across slope faces and excavations.

Protection of Constructed Works. Temporary lateral and axial support should be maintained by the contractor throughout the construction of proposed improvements to prevent lateral or vertical movements of earth materials and existing improvements.

Temporary Excavation Slope Observation. As noted in Section 4.4.4, an Engineering Geologist should observe temporary excavation slopes during site grading for evidence of faulting. This effort will require cooperation with the contractor and integrating the observation/logging effort with the earthmoving operation and the installation of temporary support measures, where needed.

6.1.3 Subgrade Preparation in Building Area

The parking structure area to a distance of 10 feet beyond the outermost edge of footings should be overexcavated to a minimum depth of 7 feet below the lowest footing elevation. The bottom of the overexcavation should be observed by Geotechniques. The excavation bottom should be deepened, as needed, to remove any remaining artificial fill or soft or loose soil, if encountered. General fill meeting the requirements of Section 6.2.2 should be used as fill in the foundation area. Select fill meeting the requirements of Section 6.2.4 should be used as fill within 12 inches of the bottom of floor slabs.

6.1.4 Grading for Pavement or Exterior Slab-on-Grade Areas, Areas to Receive Artificial Fill

To provide relatively uniform support for asphalt concrete and Portland cement concrete pavements or exterior slabs-on-grade, or as subgrade preparation in areas outside the building area to receive artificial fill, we recommend that existing soils be overexcavated to a depth of 1 foot below the bottom of the structural section, or below existing grade, or below unsuitable existing artificial fill materials, whichever is deeper. The exposed surface should be observed by Geotechniques. After observation of the excavation bottom, the exposed surface should be scarified to depth of 12 inches, processed to pea-sized consistently at optimum moisture content, and compacted to a minimum of 95 percent relative compaction.

Select fill should be placed within 12 inches of the bottom of the pavement structural section or slab and should be compacted to at least 95 percent relative compaction.

6.1.5 Grading for Lightly-Loaded Foundations Outside Building Area

Subgrade preparation for lightly-loaded ancillary unoccupied structures separate from and outside the parking structure should be overexcavated to a depth of 1 foot below the bottom of the footing or foundation, or below existing grade, or entirely through existing artificial fill, whichever is deeper. The exposed surface should be observed by Geotechniques. After observation of the excavation bottom, the exposed surface should be scarified to depth of 12 inches, processed to pea-sized consistently at optimum moisture content, and compacted to a minimum of 95 percent relative compaction.

6.1.6 Special Subgrade Stabilization Measures

Special stabilization measures may be required if soft or pumping subgrade is encountered during construction. These measures may be required to provide a firm and unyielding subgrade surface. Special subgrade stabilization measures that have successfully been used nearby consist of:

- Deepening the excavation bottom by about 1 to 2 feet, followed by laying a geotextile such as Mirafi 600X, or equivalent on the excavation bottom, followed by the placement of about 1 to 2 feet of Class 2 base materials over the fabric; or
- Thoroughly mixing at least 4 percent cement (by weight) into the upper 1 foot subgrade according to Section 301-3 of the “Greenbook” (Standard Specifications for Public Works Construction, latest edition) .

Whether these measures are required and the extent to which used will depend on the condition of the subgrade at the time of construction, the moisture content of the subgrade materials, and the nature of the construction activities (e.g., vibratory compaction equipment, number of equipment passes).

Past experience with wet subgrade soils suggests that aggregate base fill materials between 1- and 2-feet-thick may be required to provide a suitable subgrade surface (i.e., firm and unyielding) upon which fill materials may be placed and compacted. Such special measures should be considered if soft or pumping subgrade becomes a nuisance during construction. We suggest that contract documents include contingency items for procurement and placement of geosynthetics, aggregate base materials, or cement.

6.1.7 Drainage

Positive drainage should be developed and maintained away from the parking structure foundations. Hardscape areas should be maximized adjacent to building additions to reduce the potential for water infiltration. Planter areas and landscape areas adjacent to foundations should be avoided. Similarly, irrigation systems also should be avoided near building areas. Roof and surface runoff should be collected and conveyed away from the building and on-grade improvement areas. Water should not be allowed to accumulate or pond near structure foundations or on-grade improvements.

6.1.8 Fill Placement and Compaction

Fill placement and grading operations should be performed according to the grading recommendations of this report. We recommend that, unless otherwise noted, all fill materials be compacted to at least 95 percent relative compaction, based on the maximum dry density determined from ASTM D1557.

Onsite soils used as compacted general fill and imported select fill materials should be placed and compacted at a moisture content of between 0 and +3 percent of optimum moisture content. Each layer should be spread evenly and should be thoroughly blade-mixed during the spreading to provide relative uniformity of material within each layer. Soft or yielding materials should be removed and be replaced with properly compacted fill material, prior to placing the next layer.

Rock, gravel and other oversized material greater than 4 inches in diameter, should be removed from the fill material being placed. Rock less than 4 inches in diameter should not be nested and voids caused by inclusion of rock in the fill should be filled with sand or other approved material. All roots larger than ¾-inch diameter should be removed and discarded.

All fill materials, including scarified materials, should be thoroughly processed to pea-sized or finer consistency or finer prior to applying compactive effort. When the moisture content of the fill material is below that sufficient to achieve the recommended compaction, water should be added to the fill during processing. While water is being added, the soil should be bladed and mixed to provide relatively uniform moisture content throughout the material. When the moisture content of the fill material is excessive, the fill material should be aerated by blading or other methods. Fill should be spread in loose lifts no thicker than approximately 8 inches prior to being compacted. Fill and backfill materials may need to be placed in thinner lifts to achieve the recommended compaction with the equipment being used.

6.2 MATERIALS

6.2.1 Onsite Soils

Based on the logs of the boring and laboratory test results soils within the recommended overexcavation depths generally consist of clayey fine sand to sandy lean clay. Sandy materials that meet the requirements for select fill should be used as fill in the upper 1 foot of pavement, slabs-on-grade and sidewalks. Sandy materials should be selectively separated from clayey excavated materials and stockpiled for use as select fill.

There is a potential that clayey onsite soils could be sensitive to changes in moisture content. Control of moisture content and compaction layer thickness will likely be necessary to achieve the recommended compaction.

6.2.2 General Fill

General fill should be free of organics, oversize material (e.g., greater than 4 inches in diameter), trash and debris, and other deleterious material. General fill materials should have an expansion index of less than 20. The expansion index of imported materials or clayey onsite materials used as general fill within the upper 4 feet of foundation subgrade (but below the upper 1 foot of foundation subgrade that should consist of select fill) should be tested during grading to verify that the expansion index of the material is less than 20. General fill may be used as backfill in foundation overexcavation areas, except within a 1h:1v envelope behind below-grade walls, where select backfill should be used. General fill may be used below the upper 1 foot of subgrade (i.e., below the structural section of pavement or slab-on-grade) in asphalt concrete pavement areas and in exterior slab-on-grade areas.

6.2.3 Imported Fill

Imported fill to be used as general should meet the requirements of general fill and designated stockpiles should be observed and tested by Geotechniques prior to being brought to the site.

6.2.4 Select Fill

Backfill materials for retaining walls and below-grade walls, and fill materials within placed within the upper 18 inches of floor slab subgrade should consist of granular material with the following properties, and should be placed in accordance with Sections 6.1.8 and 6.5.7:

- Sand equivalent of at least 20,
- 100 percent passing the 3-inch sieve,
- Expansion index less than 10,
- 50 to 100 percent of the material passing the No. 4 sieve, and
- At least 15 percent of the material passing the No. 30 sieve.

6.2.5 Granular Material under Floor Slabs-on-Grade

Granular material for vapor retarder should consist of imported material conforming to Caltrans Standard Specifications (2006) for sand bedding, Section 19-3.025B.

6.2.6 Drainage Materials

Drainage material should conform to Caltrans Standard Specifications (2006) for Class 2 Permeable Material, Section 68-1.025. As an alternative, drainage material can consist of Caltrans Class 1 permeable material, or ¾-inch uniformly graded crushed clean gravel. All drainage materials should be enclosed in or protected with a filter fabric.

6.2.7 Aggregate Base

Aggregate base should consist of imported material conforming to Caltrans Standard Specifications for Class 2 aggregate base, Section 26-1.02A [Caltrans, 2006] or Section 200-2.4 of the "Greenbook" (International Conference of Building Officials [ICBO], 2009, or latest edition) for Crushed Miscellaneous Base (CMB).

6.3 FOUNDATION RECOMMENDATIONS

6.3.1 Foundation System

The proposed parking structure can be supported on shallow spread footings or a mat foundation founded on compacted fill. The following recommendations for foundation design are based on the loading summary presented in Section 3.2.

To reduce the potential for settlement from high concentrated loads, and to reduce differential settlement between adjacent load bearing members, the building area should be overexcavated and recompacted to a depth of 7 feet below the lowest foundation member, as described in Section 6.1.3.

6.3.2 Allowable Bearing Pressure

The allowable bearing values may be increased by one-third for wind and design earthquake loads.

Spread Footings. The recommended net (i.e., ignore weight of concrete footing) allowable bearing pressure for spread footings (with a minimum width of 24 inches) is 3,500 psf. We understand that footings will be founded a minimum of 36 inches below the lowest adjacent finish grade. No increases for additional footing embedment or width are allowed. The factor of safety for the allowable bearing pressure is not less than 3.0.

Mat Foundation. The recommended allowable bearing pressure for mat foundations is 2,000 psf.

6.3.3 Subgrade Modulus

A modulus of subgrade reaction of 170 pci is recommended for design of mat-type foundations. That value is for a 1-foot square plate, assumes a sandy subgrade, and must be corrected for mat size and shape.

6.3.4 Reinforcement

Continuous and spread footings should be reinforced per the design engineer's recommendations.

6.3.5 Settlement

On the basis of the structural loads provided above, estimated total static settlement for 15-foot-square spread footings (at column locations) bearing on a minimum of 7 feet of compacted fill is less than 1 inch. For those conditions, foundations should be designed to accommodate static differential settlement between adjacent columns of about ½ inch over a distance of 30 feet (i.e., a distortion ratio on the order of about 1/720). Mat foundations (that would apply a reduced bearing pressure on subgrade soils), should be designed to accommodate a differential settlement of about ¼ inch over a distance of 30 feet.

For design purposes, an estimated additional 1 inch of settlement from seismically induced settlement of dry sands or collapse settlement should be added to the static settlement from structural loads. Differential settlement estimated at about ½ inch over a distance of 30 feet should be added to the differential settlement above for static loads.

6.3.6 Lateral Resistance

Sliding. Ultimate sliding resistance generated through a granular soil/concrete interface (i.e., sand or base layer beneath concrete slabs-on-grade) can be estimated by multiplying the total dead weight structural loads by a coefficient of 0.4. For foundation members, ultimate sliding resistance generated through a clayey/concrete interface can be estimated by multiplying the total dead weight structural loads by a coefficient of 0.25.

Passive Resistance. For building foundations, ultimate passive earth resistance may be estimated using an equivalent fluid weight of 350 pcf.

Factors of Safety. Sliding and passive resistance may be used together without reduction when used with the factors of safety recommended herein. Minimum factors of safety of 1.5 and 2.0 are recommended for foundation overturning and sliding, respectively, where sliding resistance and passive resistance are combined. The factor of safety for sliding can be reduced to 1.5, if passive resistance is neglected. For seismic conditions, the factors of safety for overturning and sliding may be reduced to 1.1.

6.3.7 Premoistening of Footing Areas

Foundation soils, including slab-on-grade subgrade, should be lightly moistened just prior to concrete placement.

6.4 SLABS-ON-GRADE

Interior slabs-on-grade shall be underlain by a minimum of 1 foot of select fill (Section 6.1.3) placed in accordance with Section 6.1.8.

6.4.1 Slab Thickness

Interior slabs-on-grade should be designed by the structural engineer to support the anticipated floor loads. We recommend that the slab be a minimum of 5 inches thick for automotive use (no trucks). Crack control joints should be spaced at a maximum spacing of 12 feet in both directions.

6.4.2 Reinforcement

Slab-on-grade reinforcement should be according to the design engineer's recommendations. Reinforcement should be placed at mid-height of the slab with a means to keep it positioned during concrete placement.

6.4.3 Slab Subgrade

The following slab subgrade materials are recommended to help inhibit and/or reduce moisture and vapor transmission underneath interior floor slabs-on-grade.

Gravel Layer. To provide a capillary break inhibiting moisture transmission to the floor slab, 4 inches of clean, crushed, angular ¾-inch gravel may be placed over the compacted fill and lightly vibrated with three to four passes of a vibroplate compactor or smooth-drum vibratory roller. A gradation conforming to that for Grade 67 (maximum particle size of 1 inch) per ASTM C33 is acceptable for the gravel layer.

Vapor Barrier and Sand Layer. In order to reduce the risk of distress to moisture-sensitive flooring due to moisture vapor penetration of the floor slab, a continuous impermeable membrane such as 15 mil Stego Wrap, or equivalent, should be laid over the gravel.

If the capillary break, or gravel layer, is not used, the vapor barrier should be placed mid-height of a 4-inch layer consisting of clean, poorly graded sand with less than 5 percent passing the No. 200 sieve. A gradation consistent with ASTM C33 for fine aggregate is recommended for the sand. The sand layer should be located directly below the slab. Each 2-inch layer of sand (above and below vapor barrier) should be lightly moistened and compacted with a vibroplate or smooth-drum vibratory roller.

If a gravel layer is used as a capillary break, the vapor barrier should be placed over the gravel layer, and covered with 2-inches of sand (ASTM C33) that is lightly moistened and compacted with a vibroplate or smooth-drum vibratory roller.

The 2-inch-thick layer of sand above the vapor barrier is intended to promote uniform curing of the slab. The sand is not coarse enough to provide a capillary break;

however, the gravel layer placed on top of the compacted fill should provide a capillary break, if needed.

6.5 RETAINING WALLS

6.5.1 Retaining Wall Footings

Footings for below-grade building walls should be bottomed a minimum depth of 3 feet below lowest adjacent grade and should be underlain by at least 7 feet of compacted fill.

6.5.2 Backfill Materials

Backfill materials should consist of select fill material according to Section 6.2.4.

6.5.3 Static Lateral Earth Pressures

For static conditions, below-grade restrained retaining walls should be designed to resist an at-rest pressure of 55 pounds per cubic foot, equivalent fluid weight, for level, drained select backfill materials, described previously in Section 6.2.4. The equivalent fluid weight is based on the assumption that retaining walls will be provided with drainage provisions such that the buildup of hydrostatic pressures are precluded. Lateral pressure distributions should be applied along a vertical line through the heel of the wall between the intersection of the line with the ground surface above the wall and a point defined by the elevation of the lowest structural member of the wall (e.g., footing or shear key bottom).

6.5.4 Surcharge Loads

Surcharge loads on the ground adjacent to the wall induce additional pressures on earth retaining structures and should be considered in the wall design. Uniform area surcharge pressures for retaining walls may be assumed equal to 0.5 of the applied surcharge pressure for surcharges within 15 feet of the wall.

6.5.5 Dynamic Earth Pressures

For restrained walls, the increase in lateral earth pressure due to earthquake loading can be estimated using the Mononobe-Okabe theory, as described by Seed and Whitman (1970). That theory is based on the assumption that sufficient wall movement occurs during seismic shaking to allow active earth pressure conditions to develop. The theory is not directly applicable to restrained walls; however, there is a supporting reference (Nadim and Whitman, 1992) that suggests the Mononobe-Okabe method can be used to estimate dynamic forces for such walls.

In the Mononobe-Okabe approach, the total dynamic pressure can be divided into static and dynamic components. The estimated dynamic lateral force increase (due to seismic loading conditions) for either unrestrained or restrained walls may be taken as 10H pounds per square foot of wall assuming little or no movement of the wall.

The centroid of the dynamic lateral force increase should be applied at a distance of 0.6 x H above the base of the wall. The distribution of the resultant dynamic lateral force can be assumed to be an inverted triangle (base of the triangle at top of the wall).

To estimate the total dynamic lateral force, the dynamic lateral force increase should be added to the static earth pressure force computed using an active (not at-rest) lateral earth pressure of 35 pcf, equivalent fluid weight.

6.5.6 Retaining Wall Construction

Drainage Measures. Drainage measures should be provided behind below-grade walls to preclude the buildup of hydrostatic pressures. The drain should consist of a 2-foot-wide zone of granular free-draining material meeting the recommendations for drainage material provided in Section 6.2.6. The free-draining material should be placed in layers by the methods recommended for Section 6.1.8, and lightly vibrated with a small, manually-operated vibratory compactor.

In lieu of free-draining backdrain material recommended above, manufactured drainage structures (e.g., Miradrain, manufactured by Mirafi, Inc., or similar) can be used behind retaining walls. Manufacturer recommendations for the installation of any of those products should generally be followed, although they should be reviewed by the geotechnical engineer. Manufactured drainage structures should be attached to the back of the retaining wall rather than on the excavated face. This implies that safe access will be provided along the exterior of the retaining wall to attach the drainage structure and required waterproofing materials.

The drainage material behind retaining walls should be hydraulically connected to a backdrain system located at the base of the wall and consisting of a perforated pipe surrounded by clean gravel which in turn is surrounded by filter fabric such as Mirafi 180N. The entire drainage system should be tied to an exterior drainage exit.

Water stops should be installed in both expansion and/or construction joints along below-grade walls and foundation slabs. The backside of retaining walls should be waterproofed to mitigate the potential for efflorescence.

6.5.7 Retaining Wall Backfill Placement and Compaction

Fill Placement. Retaining wall backfill should consist of select fill described in Section 6.2.4 and placed within a 1h:1v envelope projected upward from the heel of the wall footing to within about 1 to 1½ feet of the ground surface, above which, in non-paved areas, more cohesive materials such as general fill is recommended to help reduce surface runoff infiltration behind the wall. Select backfill should be compacted to at least 95 percent relative compaction.

Compaction Adjacent to Walls. Backfill within 5 feet, measured horizontally, behind the retaining structure should be compacted with lightweight hand-operated compaction equipment to reduce the potential for induction of large compaction-induced stresses. If large or heavy compaction equipment is used, compaction-induced stresses can result in increased lateral earth pressures on the retaining walls. If anything but lightweight, hand-operated compaction equipment is to be used, further evaluation of the potential for compaction-induced stresses may be warranted.

6.5.8 Excavation Adjacent to Elevator Pit Walls

Drilled shafts excavated adjacent to elevator pit retaining walls, such as drilled shafts for elevator hydraulic rams at the bottom of elevator pits, or any excavation performed adjacent to and below retaining wall foundations, should be supported during construction to prevent lateral movements and caving of foundation soils.

6.6 UTILITY TRENCHES

Utility trenches should be braced or sloped in accordance with the requirements of (Cal) OSHA. Utility trench backfill should be governed by the provisions of this report relating to minimum compaction recommendations. Backfill should be moisture conditioned between 0 and 3 percent over optimum moisture content prior to placing in trench. Backfill should be compacted to a minimum of 95 percent relative compaction as determined from ASTM D1557.

Rock larger than 4 inches in maximum dimension should be excluded from backfill. Jetting of trench backfill materials should not be permitted.

Backfill materials and compaction should meet the requirements of the local governing agency or the recommendations of this report, whichever are more stringent.

6.7 CORROSIVITY

Corrosivity tests were performed on a bulk sample collected between depths of about 1 and 5 feet in boring B-1. Results of pH, soluble chloride, soluble sulfate, and resistivity tests are presented as follows:

Table 2. Summary of Chemical Test Results

Boring	Depth (feet)	Material Description	Sulfates %	Chlorides (%)	Resistivity (ohm-cm)	pH
B- 1	1 - 5	Very clayey SAND (SC)	0.02	0.01	1,360	7.84

Recommendations regarding corrosion are provided by the corrosion engineer, Atlantic Consultants, on Plate B-5, in Appendix B.

6.8 PAVEMENT SECTIONS

Flexible pavement such as asphalt concrete will be used in parking areas and driveways. Flexible pavement sections consisting of asphalt concrete and aggregate base were evaluated using the methods outlined by Caltrans (1995). Concrete pavement will be used adjacent to structure entrances and exits or in driveways.

6.8.1 Design Basis

Asphalt concrete and concrete pavement sections were estimated on the basis of an assumed R-value of 26 for subgrade soil (which was the measured R-value for subgrade materials at the athletic field located west/southwest of the parking structure site [Fugro, 2005]. A Traffic Index (TI) of 6.0 was assumed for driveway areas to receive auto and low volume truck traffic. Those values do not allow for construction traffic after the pavement is placed.

If the design TI value is different from the assumed value, Geotechniques should be notified accordingly for reevaluation of pavement section thickness. Alternately, the projected daily truck traffic (including number of axles and weight per axle) would need to be furnished to Geotechniques so that the TI could be estimated per Caltrans procedures.

R-value tests should be performed on subgrade materials near the completion of rough grading in order to verify pavement design sections, particularly if subgrade materials are more granular than typical near-surface sandy clay, which likely would result in a reduction of pavement section design thickness.

6.8.2 Asphalt Concrete Pavement Sections

The following minimum pavement sections, consisting of asphalt concrete over aggregate base, are based on subgrade materials with an assumed R-value of 26 and the assumed TI value of 6.0:

Table 3. Recommended Flexible Pavement Sections

Traffic Index	Asphalt Concrete Thickness (inches)	Aggregate Base Thickness (inches)
6.0	3	9
6.0	4	7

To maintain aggregate base section integrity, the base section should be underlain by a geotextile such as Mirafi 600X, or equivalent. The geotextile should be placed in accordance with the manufacturer's recommendations. The geotextile should not be trafficked with construction or compacting equipment until the full, loose lift thickness of aggregate base has been spread over the geotextile. This may be achieved by pushing the base onto the geotextile ahead of the spreading equipment and driving over the base only after full thickness has been placed in front of the advancing equipment.

6.8.3 Concrete Pavement Sections

Assuming a TI of 6.0 and a minimum subgrade R-value of 26, concrete pavement should consist of a minimum thickness of 7 inches of Portland Cement Concrete (PCC) constructed over 6 inches of aggregate base.

The concrete should have a minimum 28-day compressive strength of 3,500 pounds per square inch (psi). Minimum concrete pavement reinforcement should consist of No. 3 bars spaced at 18 inches each way. Reinforcement should not be less than structural requirements for shrinkage and temperature. Load transfer at cold joints should be transferred using minimum 5/8-inch smooth dowels, with one end treated (i.e., greased) to slip. The dowels should be at least 18 inches long and spaced no less than 12 inches on center.

Crack control joints are recommended every 16 to 18 feet in each direction.

6.8.4 Base Materials

Aggregate base materials used in pavements should conform to Section 6.2.7, and should be compacted to a minimum of 95 percent of the maximum dry density determined by ASTM D1557, latest edition.

6.8.5 Pavement Subgrade

Pavement subgrade areas should be prepared in accordance with the recommendations in Section 6.14, and should have a minimum R-value of 26 in the upper 1 foot.

6.8.6 Drainage

Proper site drainage is essential in pavement areas. Grades should be established to expedite runoff away from pavements and reduce moisture infiltration into the base and subgrade. Drainage on pavement surfaces should be achieved by sheetflow, and concentrated runoff should be avoided except where accommodated by concrete drainage improvements such as ribbon gutters or swales.

6.9 ADDITIONAL SERVICES

The design and construction phases benefit from and require a continuum of geotechnical input, evaluation, and observation of site conditions in support of the intent and recommendations of this report. The responsible geotechnical engineer should, in order to provide this continued service, render interpretations, respond to additional information, and observe the contractor's implementation of design during construction.

6.9.1 Construction Documents Review

We recommend reviewing final site improvement and foundation plans and specifications prior to submittal to the reviewing agency and/or prior to bidding. The purpose of the review is to assess general compliance with the earthwork and foundation recommendations of this report, and to confirm that the recommendations given in this report are incorporated in the project design plans and specifications.

6.9.2 Construction Observation and Testing

We further recommend providing geotechnical field services during site grading, excavation, foundation construction, and utility trench backfilling phases of earthwork. The purpose of those services is to observe compliance with construction drawings, specifications, and the geotechnical recommendations in this report. The observation and testing services can help the contractor avoid or manage adverse field conditions that may otherwise lead to cost overruns or change orders, and to allow for changes in the recommendations in the event that subsurface conditions differ from those anticipated prior to construction.

7.0 LIMITATIONS

7.1 LOCAL PRACTICE

The conclusions and professional opinions presented in this report were derived according to generally accepted geotechnical engineering principles and practices at the time and location this report was prepared. This statement is in lieu of all warranties, express or implied. Standards of practice are subject to change with time. Recommendations presented are based on the scope of work performed; they are professional opinions that are limited to the extent of the available data.

7.1.1 Report Use

Geotechniques prepared this report concerning the parking structure site for the exclusive use of Ventura County Community College District and their authorized agents and this report shall not be considered transferable. It may not contain sufficient information for other parties or other uses. If any changes are made to the project that differs from those described in this report, the conclusions and recommendations contained in this report may be rendered invalid. Geotechniques should review any changes in the project, and provide revisions as necessary to the recommendations presented in this report. The recommendations, data, information, and drawings presented in this report are intended as design-input purposes and not intended to serve as construction drawings or specifications.

Although this report may comment or discuss construction techniques or procedures for the design engineer's or contractor's guidance, this report shall not be interpreted to prescribe or dictate construction procedures or to relieve the contractor in any way of their responsibility for the construction.

This report may be subject to review by controlling agencies, and any modifications they deem necessary shall be made a part thereof, subject to our technical acceptance of such modifications. All submissions of this report shall be in its entirety. Under no circumstances shall this report be summarized and synthesized or quoted out of context for any purpose.

7.1.2 Design Changes

If any changes in the nature and design (including structural loadings different from those anticipated), or other improvements are planned, the conclusions and recommendations contained in this report shall not be considered valid unless the changes are reviewed and conclusions of this report modified or verified in writing.

7.1.3 Potential Variation in Subsurface Conditions

Soil and rock deposits vary in type, strength, and other geotechnical properties between points of observations and exploration. Additionally, groundwater, soil moisture, and soil behavior also can vary seasonally or for other reasons. Therefore, we do not and cannot have a complete knowledge of the subsurface conditions underlying the site. The conclusions and recommendations presented in this report are based upon the findings at the points of exploration, interpolation and extrapolation of information between and beyond the points of observation, and are subject to confirmation based on the conditions revealed by construction.

7.1.4 Hazardous Materials

An investigation and discussion of potential subsurface contamination is beyond the scope of this geotechnical study, as are environmental assessments for the presence or absence of hazardous/toxic materials in the soil, surface water, groundwater, or atmosphere. Any statements or absence of statements in this report or data presented herein regarding odors, unusual or suspicious items, or conditions observed are strictly for descriptive purposes and are not intended to convey engineering judgment regarding hazardous/toxic assessment.

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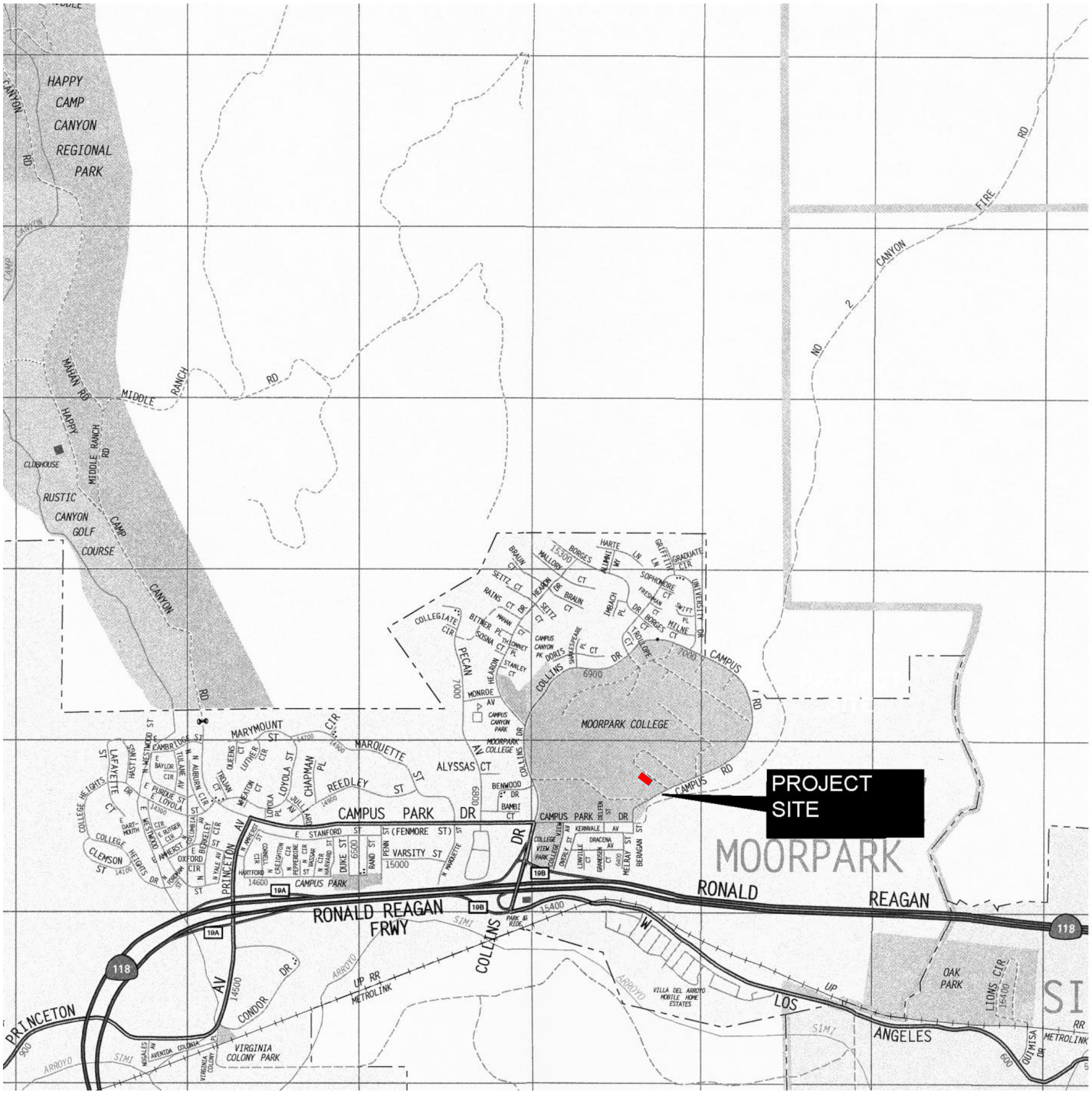
PLATES



VICINITY MAP

PARKING STRUCTURE
MOORPARK COLLEGE
Moorpark, California

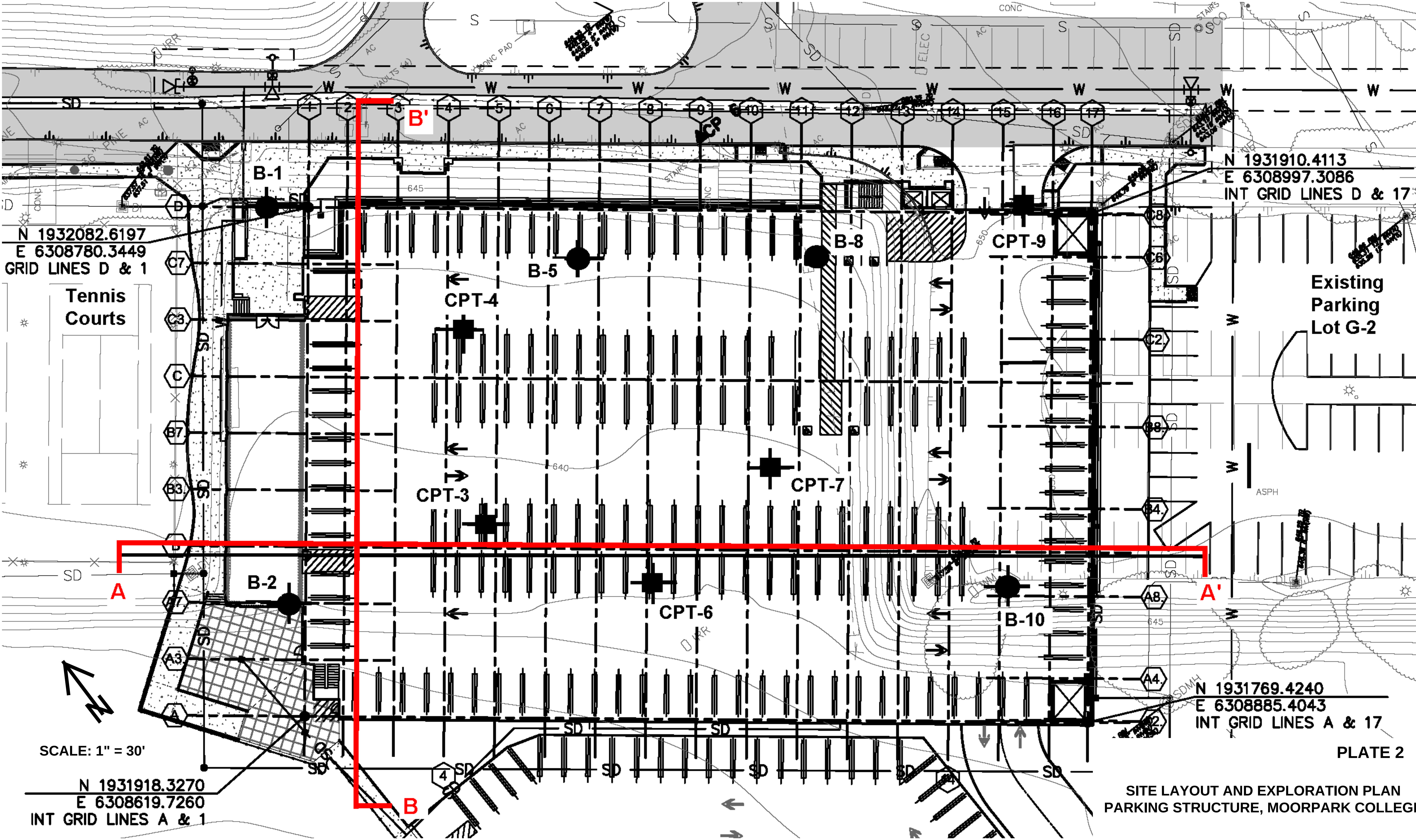
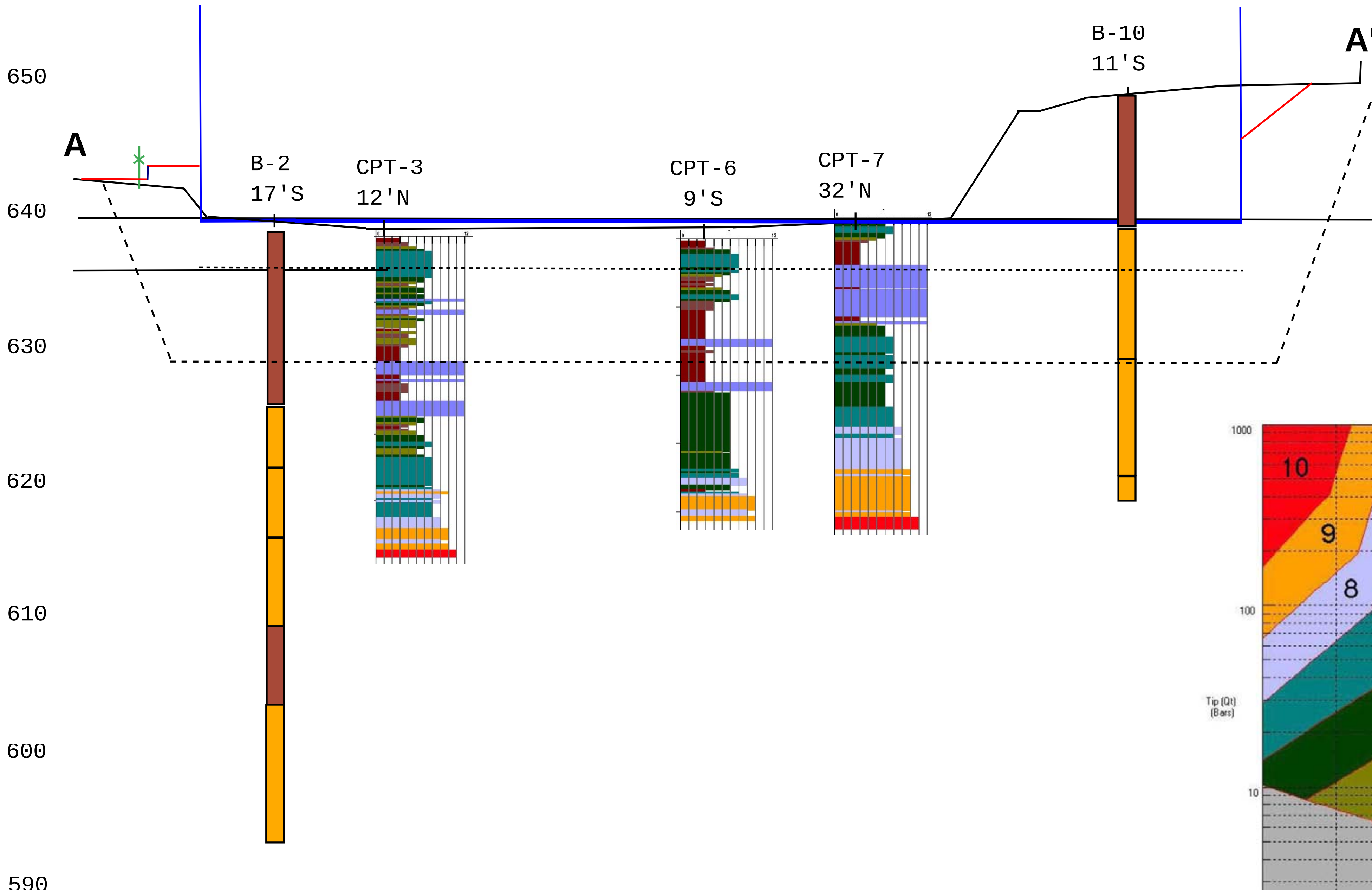
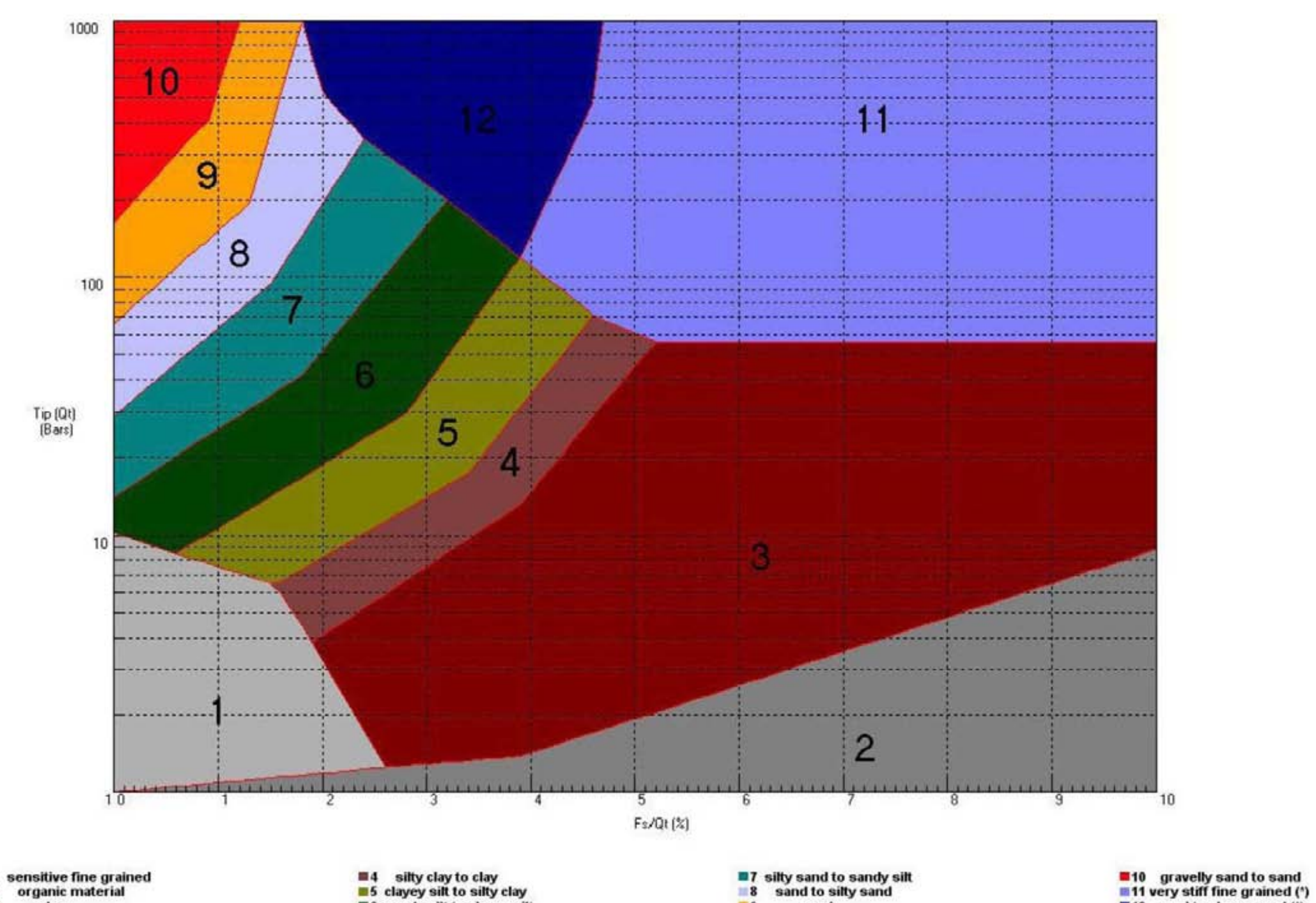


PLATE 2

SITE LAYOUT AND EXPLORATION PLAN
PARKING STRUCTURE, MOORPARK COLLEGE





KEY

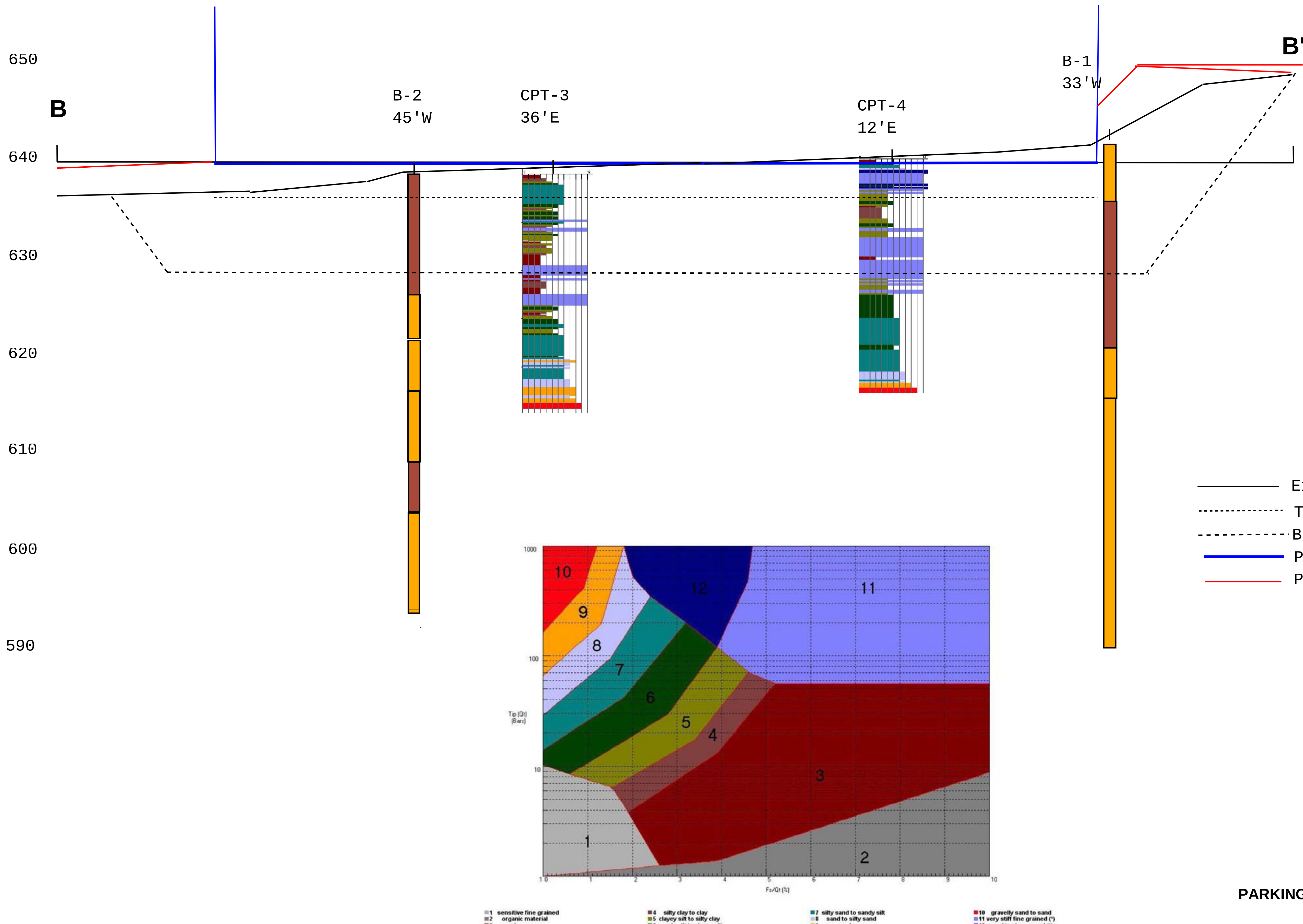
Existing Ground Surface
Typical footing bottom elevation
Bottom of Overexcavation
Parking Structure Floor
Proposed Grade
Curb Wall
Existing Fence

SCALE

Horizontal: 1" = 40'
Vertical: 1" = 10'

PLATE 3.1

CROSS-SECTION A - A'
PARKING STRUCTURE, MOORPARK COLLEGE



SCALE

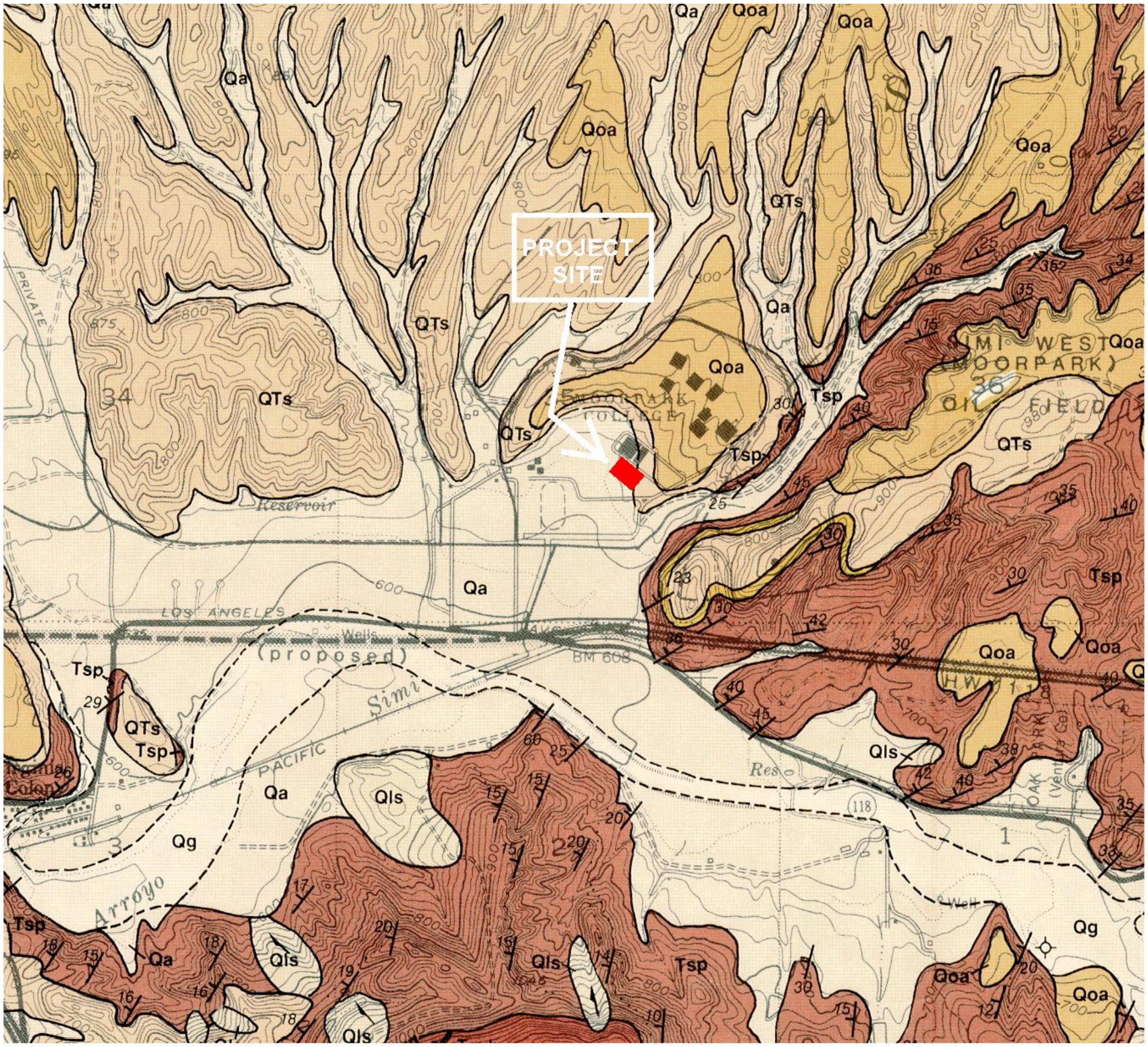
Horizontal: 1" = 20'
Vertical: 1" = 10'

KEY

Existing Ground Surface
Typical footing bottom elevation
Bottom of Overexcavation
Parking Structure Floor
Proposed Grade

PLATE 3.2

CROSS-SECTION B - B'
PARKING STRUCTURE, MOORPARK COLLEGE



Base map source: Geologic Map of the Simi Quadrangle (Dibblee, 1992).

LEGEND

Qg	Stream Channel Gravel and Sand
Qa	Alluvium
Qls	Landslide Debris
Qoa	Older Alluvium
QTs	Saugus Formation
Tcvb	Conejo Volcanics
Tsp	Sespe Formation

— — — Formation Contact

– – – – – Member Contact

---------- Contact Between Surficial Sediments

Strike and Dip of Stratified Rocks:

20 inclined

Abandoned Exploratory Oil (or Gas) Well

NORTH

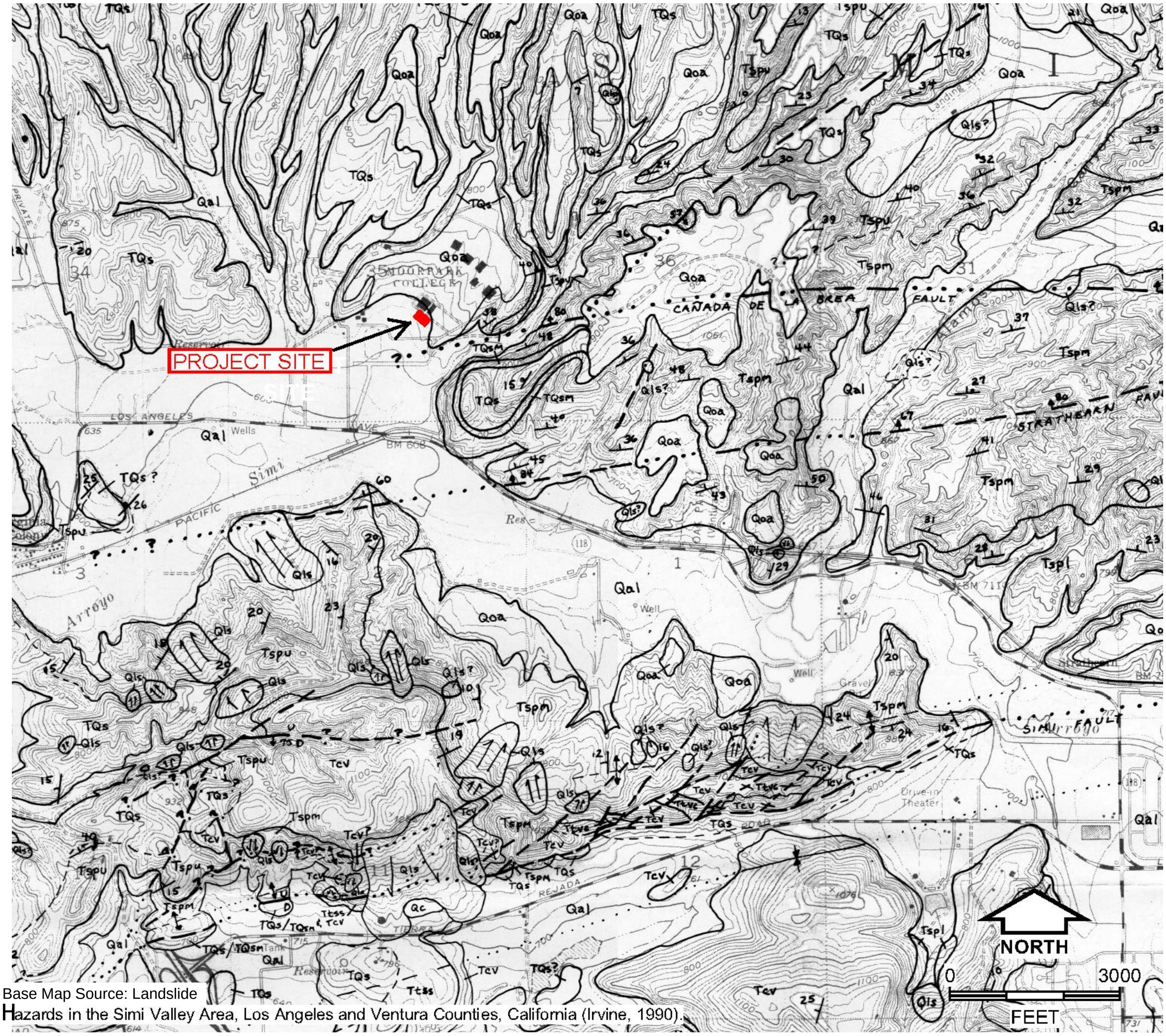
0 1000 2000

FEET

REGIONAL GEOLOGIC MAP FROM DIBBLEE (1992)

PARKING STRUCTURE
MOORPARK COLLEGE
Moorpark, California

GEOTECHNIQUES



Base Map Source: Landslide Hazards in the Simi Valley Area, Los Angeles and Ventura Counties, California (Irvine, 1990).

LEGEND

Qal	Alluvium
Qoa	Older Alluvium
Qls	Landslide Deposits
TQs	Saugus Formation
TQsm	Saugus Formation (lower, marine member)
Ttss	Calabasas and Topanga Canyon Formation
Tcv	Conejo Volcanics
Tspu	Sespe Formation (upper member)
Tspm	Sespe Formation (middle member)
Tspl	Sespe Formation (lower member)

Contact, dashed where approximate, dotted where concealed

Axis of anticlinal fold, dotted where concealed

Axis of synclinal fold, dotted where concealed

75 U D ? Fault - dashed where approximate, dotted where concealed, queried where uncertain, U = upthrown side, D = downthrown side, number and arrow indicate attitude of fault

35 Strike and dip of bedding

15 Approximate strike and dip of bedding

Landslide - arrows show general direction of movement

REGIONAL GEOLOGIC MAP FROM IRVINE (1990)

PARKING STRUCTURE

Moorpark College, Moorpark, California

Project Site

BASE MAP SOURCE: Geologic Map of Southern Ventura County, California, Showing Mineral Deposits, compiled by California Division of Mines and Geology, Preliminary Report 14 (Weber et al., 1973).

0 5000 10,000
FEET

LEGEND

Qal	Younger Alluvium	TQsp	San Pedro Formation
Qsc	Stream Channel Deposits	TQs	Saugus Formation
Qfp	Flood Plain Deposits	Tp	Pico Formation
Qvf	Valley Fill deposits	Tto	Towsley Formation
Qco	Colluvium and Slopewash	Tm	Monterey Formation
Qf	Alluvial Fan Deposits	Tt	Topanga Formation
Qoal	Older Alluvium	Tcvb	Conejo Volcanics, basalt to andesite
Qt	Terrace Deposits	Tcvi	Conejo Volcanics, basalt to andesite; intrusive
Qof	Older Alluvial Fan Deposits	Tv	Vaqueros Formation
Qov	Older Valley and Floodplain Deposits	Ts	Sespe Formation
Qod	Older Lacustrine and Deltaic Deposits	Tl	Llajas Formation
Qls	Landslide Deposits		

Geologic contact, dashed where approximate

Anticline, showing plunge, dotted where concealed

Syncline, showing plunge, dotted where concealed

80 U D •••? Fault, dashed where approximately located, dotted where concealed, conjectural where queried, arrow indicates dip of fault, U = up, D = down

Suggested fault trace as identified on aerial photos, but not verified

25 Strike and dip of bedding

25 Approximate strike and dip of bedding

Horizontal bedding

LS Denotes that landsliding is widespread in area where symbol is located

FCA Indicates that Fox Canyon aquifer is exposed in area; this aquifer consists of lower part of San Pedro-Saugus Formations.

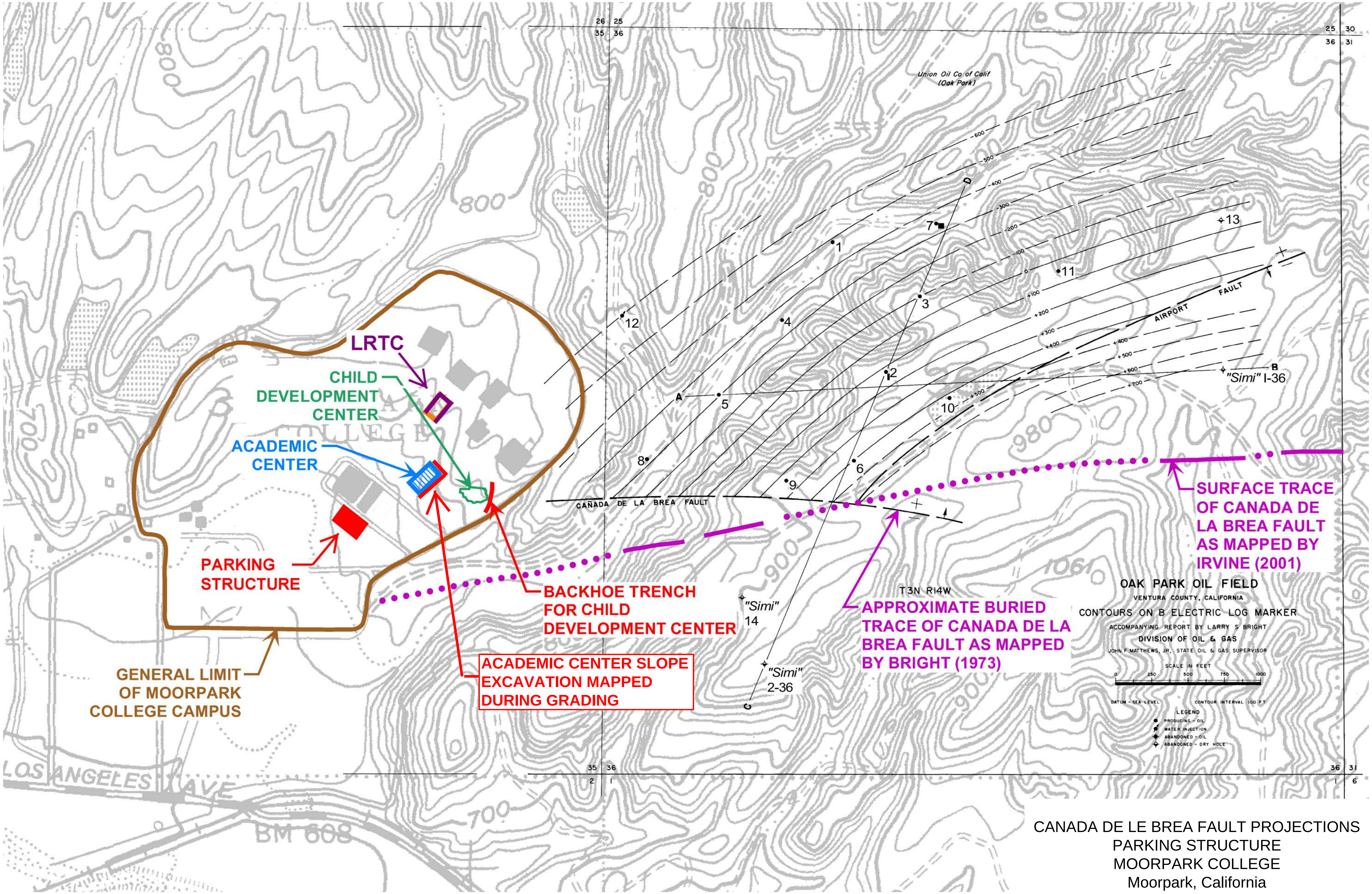
GCA Indicates that Grimes Canyon aquifer is exposed in area; this aquifer consists of upper part of Santa Barbara Formation in Grimes Canyon area; where the formation is included with the San Pedro Formation.

Petroleum, natural gas and natural gas liquid

30 (M) Mineral deposits

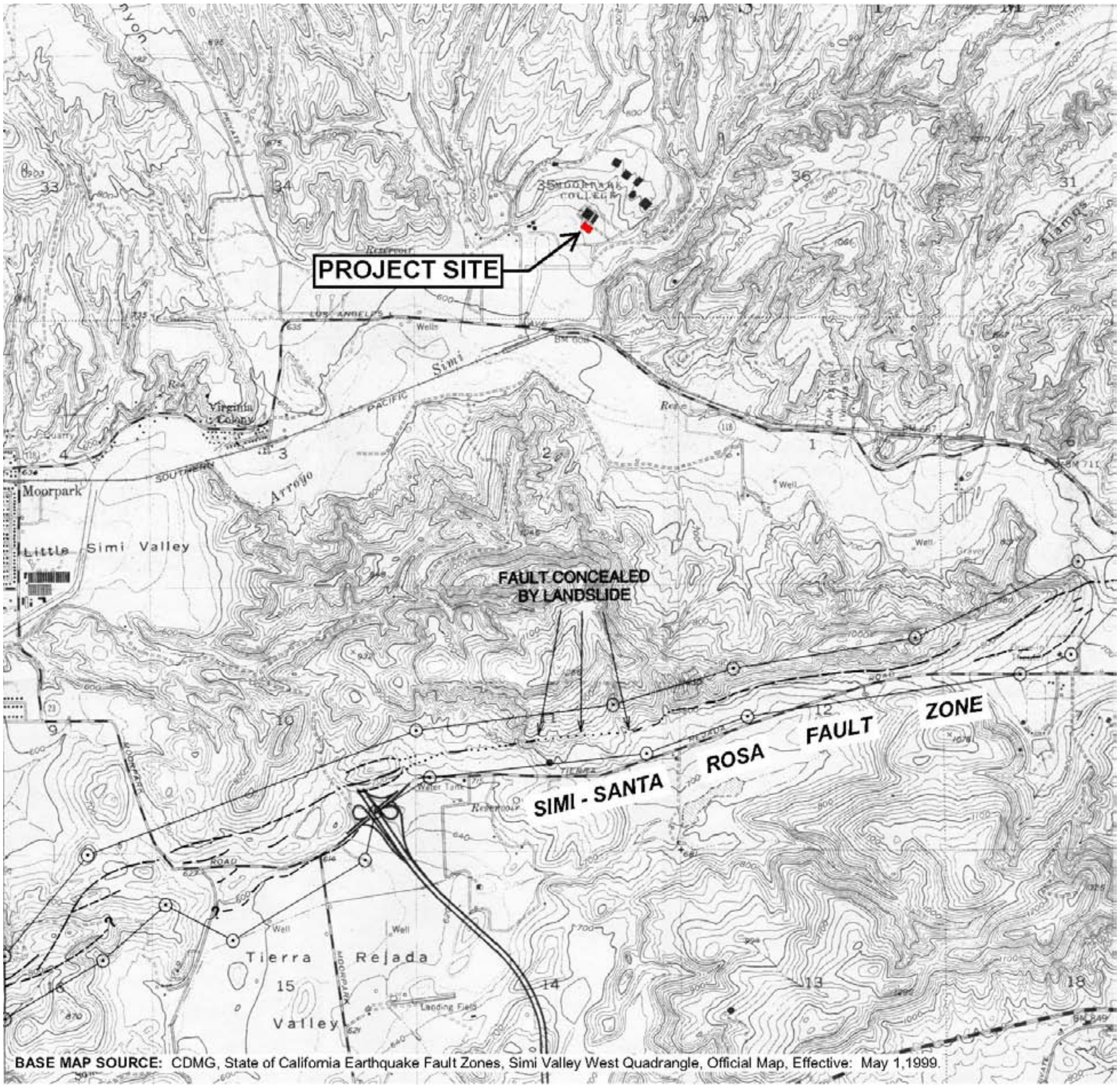
REGIONAL GEOLOGIC MAP FROM WEBER ET AL. (1973)

PARKING STRUCTURE
MOORPARK COLLEGE
Moorpark, California

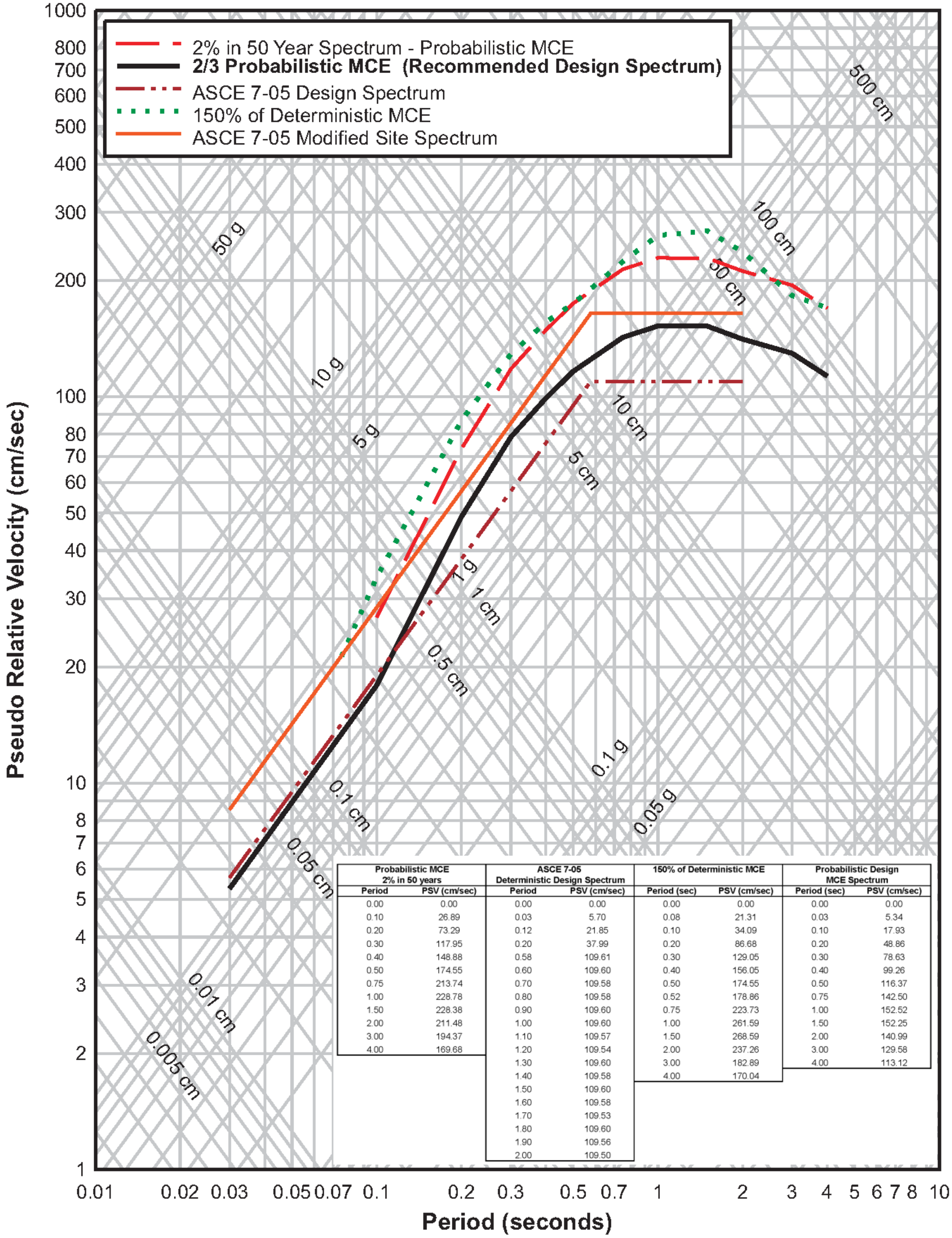


CANADA DE LE BREA FAULT PROJECTIONS
PARKING STRUCTURE
MOORPARK COLLEGE
Moorpark, California

PLATE 7



PORTION OF STATE OF CALIFORNIA EARTHQUAKE FAULT ZONES MAP
SIMI VALLEY WEST QUADRANGLE
PARKING STRUCTURE
MOORPARK COLLEGE
Moorpark, California



Probabilistic MCE 2% in 50 years

Period	PSV (cm/sec)
0.00	0.00
0.10	26.89
0.20	73.29
0.30	117.95
0.40	148.88
0.50	174.55
0.75	213.74
1.00	228.78
1.50	228.38
2.00	211.48
3.00	194.37
4.00	169.68

ASCE 7-05 Deterministic Design Spectrum

Period	PSV (cm/sec)
0.00	0.00
0.03	5.70
0.12	21.85
0.20	37.99
0.58	109.61
0.60	109.60
0.70	109.58
0.80	109.58
0.90	109.60
1.00	109.60
1.10	109.57
1.20	109.54
1.30	109.60
1.40	109.58
1.50	109.60
1.60	109.58
1.70	109.53
1.80	109.60
1.90	109.56
2.00	109.50

150% of Deterministic MCE

Period (sec)	PSV (cm/sec)
0.00	0.00
0.08	21.31
0.10	34.09
0.20	86.68
0.30	129.05
0.40	156.05
0.50	174.55
0.52	178.86
0.75	223.73
1.00	261.59
1.50	268.59
2.00	237.26
3.00	182.89
4.00	170.04

Probabilistic Design MCE Spectrum

Period (sec)	PSV (cm/sec)
0.00	0.00
0.03	5.34
0.10	17.93
0.20	48.86
0.30	78.63
0.40	99.26
0.50	116.37
0.75	142.50
1.00	152.52
1.50	152.25
2.00	140.99
3.00	129.58
4.00	113.12

UNIFORM HAZARD SPECTRA

5% DAMPING

PARKING STRUCTURE
MOORPARK COLLEGE
Moorpark, California

APPENDIX A

SUBSURFACE EXPLORATION

APPENDIX A
SUBSURFACE EXPLORATION

INTRODUCTION

The contents of this appendix shall be integrated with the geotechnical study of which it is a part. They shall not be used in whole or in part as a sole source for information or recommendations regarding the subject site.

FIELD EXPLORATION

Subsurface conditions at the Moorpark College Parking Structure site were explored by the excavation and sampling of four hollow-stem-auger borings and by advancing six Cone Penetrometer Test (CPT) soundings. The locations of the borings and CPTs are shown on Plate 2. Exploration locations were located in the field by siting, pacing, and measuring from existing improvements. Their locations should be considered accurate only to the degree implied by the method used.

Cone Penetrometer Test Soundings

Six Cone Penetrometer Test (CPT) soundings were performed by Middle Earth Geo Testing, Inc., of Orange, California, on October 21, 2010. The cone penetrometer is mounted on a 20-ton truck and consists of a 36-millimeter-diameter rod with a 10-square-centimeter, 60-degree-apex-angle cone at the base. The cone is equipped with electronic load cells that measure both point resistance and frictional resistance between the soils and the cylinder side of the cone. The primary purpose of performing CPTs was to provide a nearly continuous log of the earth materials and soil stratigraphy between boring locations and sample intervals.

Although many factors influence CPT profiles, including: physical cone properties, vertical effective stress, pore pressure, soil compressibility and fabric, and depositional characteristics, the classifications are generally consistent with the laboratory classification data and with the visual descriptions made during the soil borings (Plate A-11 presents one example of soil classification using CPT data).

CPT soundings were advanced until refusal was met on gravelly layers at depths ranging from about 20½ to 30 feet below the ground surface. Upon completion, the CPT soundings were backfilled with bentonite pellets. Data results of CPT soundings consisting of plots of sleeve friction, tip resistance, and friction ratio versus depth are presented on Plates A-3, A-4, A-5, A-6, A-7 and A-9. A soil classification chart is presented on Plate A-11.

Drilling and Sampling. Four borings were advanced to depths between about 30 and 50.5 feet below the existing grade on November 12, 2011. The borings were excavated with a truck-mounted CME 75 drilling rig operated by Martini Drilling Corporation of Los Alamitos, California. After completion of drilling, the borings were backfilled with excavated materials. Backfill materials are anticipated to settle, and should be backfilled level with adjacent grade in the future, as needed.

The borings were logged and sampled at approximately 5-foot intervals. Samples were extracted from the subsurface using a 2-⅜-inch-inside-diameter (ID) Modified California liner sampler and with a 1½-inch-ID standard penetration test (SPT) split-spoon sampler. The samplers were driven by a 140-pound automatic-trip hammer free falling from a height of 30 inches.

The logs of the borings describe the earth materials encountered, sampling method used, and field and laboratory tests performed. The logs also show the excavation date, groundwater levels encountered during drilling, and the name of the logger and drilling subcontractor. The borings were logged by an Engineer using ASTM D2487 for visual classification of soils. The boundaries between soil types shown on the logs are approximate because the transition between different soil layers may be gradational. The logs of the borings are presented as Plates A-1, A-2, A-8, and A-10.

GEOTECHNIQUES
LOG OF EXPLORATORY BORING

PROJECT NO.:	1003.026	**DRILLER:**	Martini Drilling	**LOGGED BY:**	CW
PROJECT NAME:	Parking Structure	**DRILL METHOD:**	8-inch Hollow Stem Auger	**OPERATOR:**	Gene/Brandon
LOCATION:	South Campus, Moorpark College	**HAMMER:**	140 pound auto-trip	**RIG TYPE:**	CME 75
ELEVATION:	642 feet (approx.)			**DATE:**	11/12/2010

BORING NO.: B-1

Depth (ft)	Samples: Blows/ 6"	Samples: SPT N-value	Samples: Sample Number	USCS Symbol	MATERIAL DESCRIPTION AND COMMENTS	Water Content (%)	Dry Density (pcf)	% Finer #200 Sieve/ Atterberg Limits
				SC:	**ARTIFICIAL FILL (Qaf):** Very clayey SAND (SC): Medium red-brown, very dense, moist			
5	10 15 25	40	1		- mottled black, brown, orange-brown in sampler nose, at 6'	9	111.6	EI = 2
				CL	**OLDER ALLUVIUM (Qoa):** Fine sandy lean CLAY (CL): Medium brown, stiff, moist			
10	3 4 7	11	2		- dark red-brown in sampler nose, at 11'	12	102	
15	4 5 8	13	3		- medium red-brown, medium stiff, at 15'	13	112	20/ LL ~ 26 PI ~ 10
20	3 3 2	5	4		- cuttings color change to medium brown, at 21'			
25	6 10 19	29	5	SC	Clayey fine SAND (SC): Medium brown, dense, moist	9	113	
					- drilling difficulty encountered, at 26'			
				SM	Very silty fine SAND (SM): Medium red-brown, very dense, with gravel, moist, occasional clay pods			

Legend: -Ring -Disturbed Ring - SPT - Bulk - No Recovery - Groundwater

check: cw 01/03/11

PLATE A-1.1

GEOTECHNIQUES
LOG OF EXPLORATORY BORING

PROJECT NO.:	1003.026	**DRILLER:**	Martini Drilling	**LOGGED BY:**	CW
PROJECT NAME:	Parking Structure	**DRILL METHOD:**	8-inch Hollow Stem Auger	**OPERATOR:**	Gene/Brandon
LOCATION:	South Campus, Moorpark College	**HAMMER:**	140 pound auto-trip	**RIG TYPE:**	CME 75
ELEVATION:	642 feet (approx.)			**DATE:**	11/12/2010

BORING NO.: B-1 (continued)

Depth (ft)	SAMPLES: Sample Type	Blows/ 6"	N-value	Sample Number	Graphical Log	USCS Symbol	MATERIAL DESCRIPTION AND COMMENTS	Laboratory Testing: Water Content (%)	Dry Density (pcf)	Others
						SM	Very silty fine SAND (SM): Medium red-brown, very dense, with angular gravel, moist, occasional clay pods			
30	Disturbed Ring / Bulk	3 13 16	29	6						
35	Ring	18 18 51	69	7			- light red-brown to pink, very dense, slightly cemented, $CaCO_3$ and clay pods, at 35'	24	102	
40	Disturbed Ring / Bulk	9 15 27	42	8			- very dense, at 40'			
45	Ring	12 29 38	67	9			- very dense, at 45'	23	108	
50	Disturbed Ring / Bulk	25 51		10			- pink to tan, very dense, cemented, with fine to medium gravel-sized shards, at 45'			
							TOTAL DEPTH 50.5 FEET **GROUNDWATER NOT ENCOUNTERED**			
							BORING BACKFILLED WITH CUTTINGS UPON COMPLETION.			
55										

Legend: -Ring ' - Disturbed Ring '-Bulk -No Recovery -Groundwater

check: cw 01/03/11

GEOTECHNIQUES
LOG OF EXPLORATORY BORING

PROJECT NO.: 1003.026
PROJECT NAME: Parking Structure
LOCATION: South Campus, Moorpark College
ELEVATION: 639 feet (approx.)

DRILLER: Martini Drilling
DRILL METHOD: 8-inch Hollow Stem Auger
HAMMER: 140 pound auto-trip

LOGGED BY: CW
OPERATOR: Gene/Brandon
RIG TYPE: CME 75
DATE: 11/12/2010

BORING NO.: B-2

Depth (ft)	Blows/ 6"	SPT N-value	Sample Number	USCS Symbol	MATERIAL DESCRIPTION AND COMMENTS	Water Content (%)	Dry Density (pcf)	% Passing #200 Sieve
				CL	**ARTIFICIAL FILL (Qaf):** Fine sandy lean CLAY (CL): Mottled medium - brown, light tan, and black , very dense, very moist			
5	8 16 36	52	11			18	115	
10	7 6 13	19	12		- mottled dark brown and black, at 10'	17	114	
15	4 7 10	17	13	SC	**OLDER ALLUVIUM (Qoa):** Clayey fine SAND (SC): Medium to dark red brown, medium dense, moist	14	--	16
20	6 11 12	23	14	SC/ SM	Fine SAND with silt and clay (SC/SM): Medium orange-brown, dense, moist	12	113	
					- drilling chatter from gravel, at ~22.5 - ~23.5'			
25	7 30 19	49	15	SP	Fine to medium SAND with gravel and cobbles (SP): Medium light brown, very dense, moist - with clay and fine gravel in nose, at 26' - drilling chatter, between ~26' and ~29'			

Legend: -Ring -Disturbed Ring - SPT - Bulk - No Recovery - Groundwater

check: cw 01/03/11

GEOTECHNIQUES
LOG OF EXPLORATORY BORING

PROJECT NO.:	1003.026	**DRILLER:**	Martini Drilling	**LOGGED BY:**	CW
PROJECT NAME:	Parking Structure	**DRILL METHOD:**	8-inch Hollow Stem Auger	**OPERATOR:**	Gene/Brandon
LOCATION:	South Campus, Moorpark College	**HAMMER:**	140 pound auto-trip	**RIG TYPE:**	CME 75
ELEVATION:	639 feet (approx.)			**DATE:**	11/12/2010

BORING NO.: B-2 (continued)

Depth (ft)	SAMPLES: Sample Type	SAMPLES: Blows/ 6"	SAMPLES: N-value	SAMPLES: Sample Number	Graphical Log	USCS Symbol	MATERIAL DESCRIPTION AND COMMENTS	Laboratory Testing: Water Content (%)	Laboratory Testing: Dry Density (pcf)	Laboratory Testing: % Finer #200 Sieve/ Atterberg Limits
						SP	Fine to medium SAND with gravel and cobbles (SP): Medium light brown, very dense, moist			
30		9 4 2	6	16		**CL**	Fine sandy lean CLAY (CL): Light yellow-brown, medium stiff, very moist	20	--	33/ LL ~ 27 PI ~ 11
35		14 17 25	42	17		**SM**	Silty fine SAND (SM): Light yellow brown, very dense, trace fine gravel, moist			
40		10 30 37	67	18			-with fine to medium rounded gravel, at 40'			
45		>50		19			Fine SAND with clay (SC): Red-brown, very dense, moist			
							- Refusal, at 45' **TOTAL DEPTH 45 FEET** **GROUNDWATER NOT ENCOUNTERED** BORING BACKFILLED WITH CUTTINGS UPON COMPLETION			
50										
55										

Legend: -Ring ' - Disturbed Ring '-Bulk -No Recovery -Groundwater

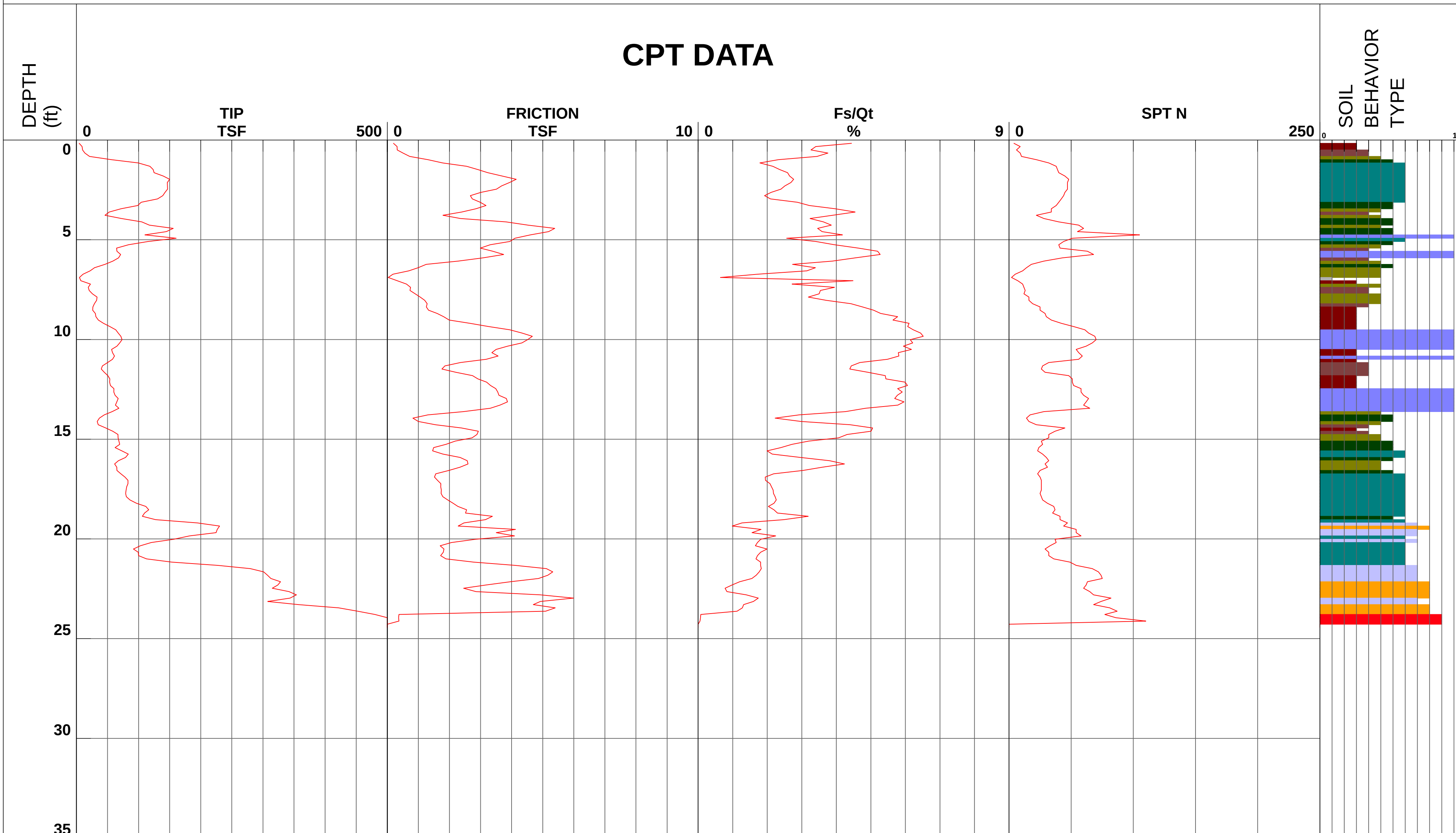
check: cw 01/03/11

PLATE A-2.2

Geotechniques

Location	Moorpark College Parking Structure	Operator	ML/DH	Filename	SDF(864).cpt
Job Number	1003.026	Cone Number	DSG0906	GPS	
Hole Number	CPT-03	Date and Time	10/21/2010 12:34:43 PM	Maximum Depth	24.44 ft
Water Table Depth	NE				
Surface El. (ft.)	640				

CPT DATA



1 - sensitive fine grained	4 - silty clay to clay	7 - silty sand to sandy silt	10 - gravelly sand to sand
2 - organic material	5 - clayey silt to silty clay	8 - sand to silty sand	11 - very stiff fine grained (*)
3 - clay	6 - sandy silt to clayey silt	9 - sand	12 - sand to clayey sand (*)

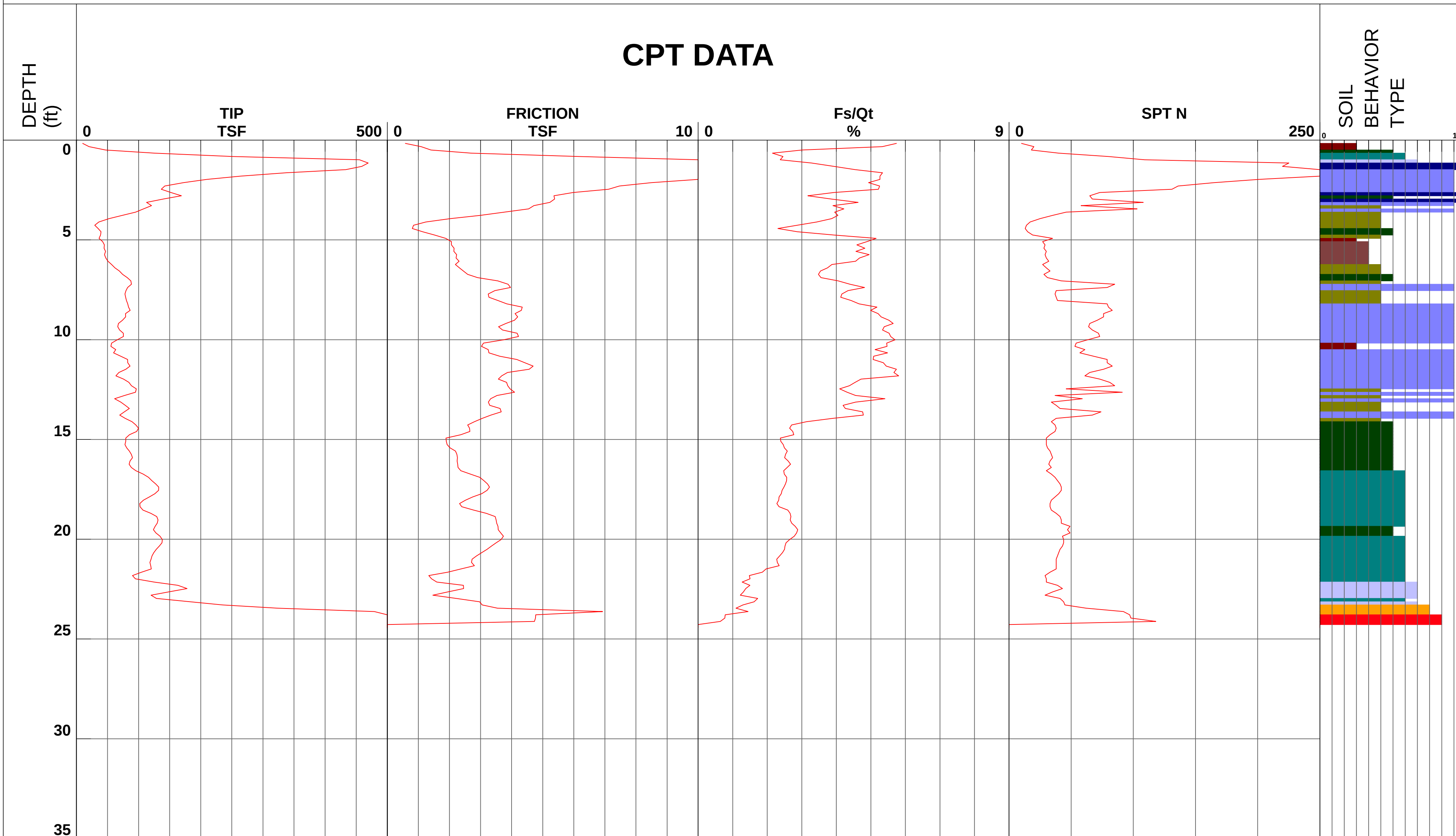
PLATE A-3

Depth Increment

*Soil behavior type and SPT based on data from UBC-1983

Geotechniques

Location	Moorpark College Parking Structure	Operator	ML/DH	Filename	SDF(865).cpt
Job Number	1003.026	Cone Number	DSG0906	GPS	
Hole Number	CPT-04	Date and Time	10/21/2010 1:08:58 PM	Maximum Depth	24.44 ft
Water Table Depth		NE			
Surface El. (ft.) 640					



1 - sensitive fine grained
2 - organic material
3 - clay
4 - silty clay to clay
5 - clayey silt to silty clay
6 - sandy silt to clayey silt
7 - silty sand to sandy silt
8 - sand to silty sand
9 - sand
10 - gravelly sand to sand
11 - very stiff fine grained (*)
12 - sand to clayey sand (*)

PLATE A-4

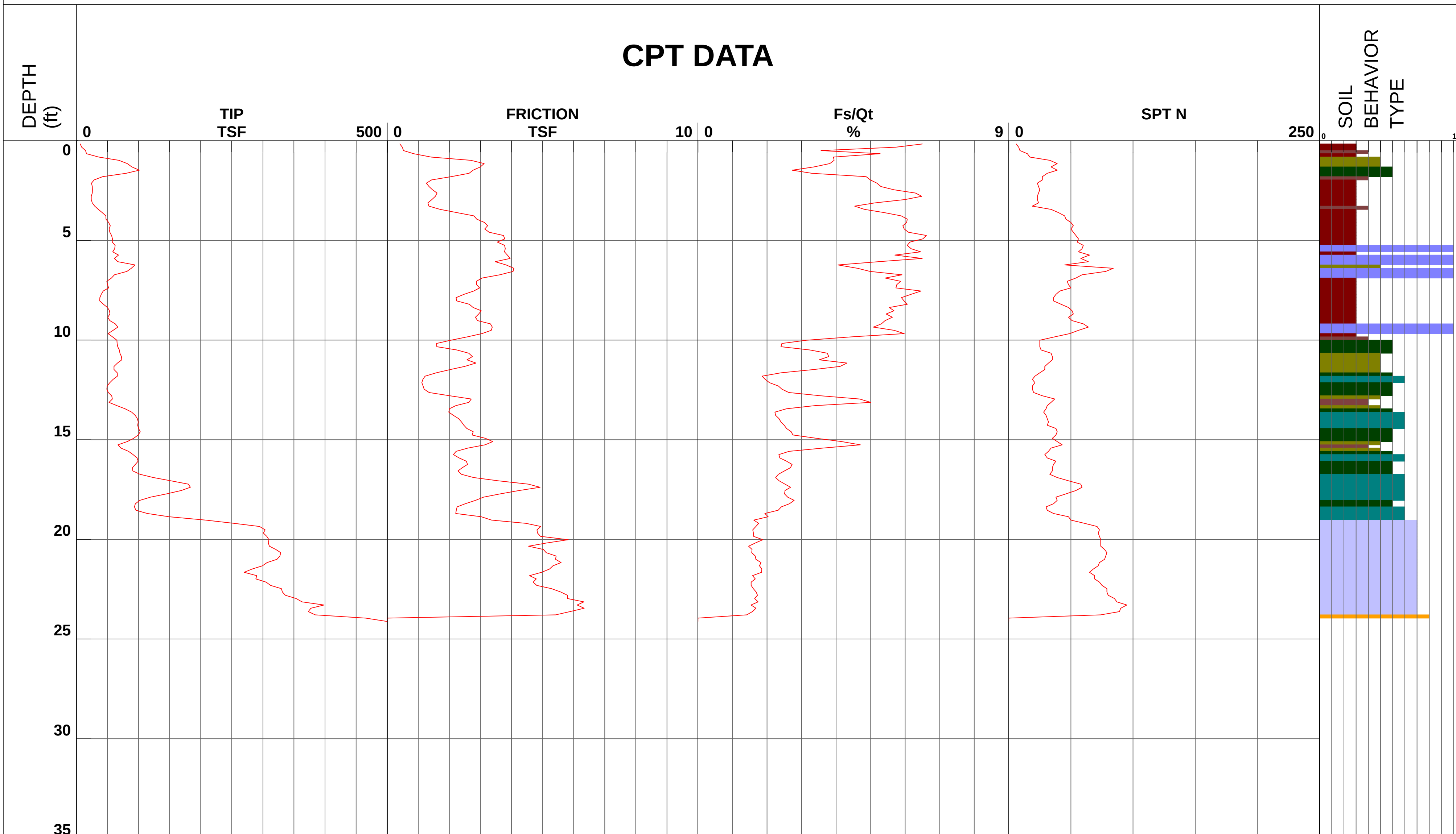
Depth Increment

*Soil behavior type and SPT based on data from UBC-1983

Geotechniques

Location	Moorpark College Parking Structure	Operator	ML/DH	Filename	SDF(866).cpt
Job Number	1003.026	Cone Number	DSG0906	GPS	
Hole Number	CPT-05	Date and Time	10/21/2010 1:55:45 PM	Maximum Depth	24.11 ft
Water Table Depth		NE			
Surface El. (ft.) 640					

CPT DATA



1 - sensitive fine grained
2 - organic material
3 - clay
4 - silty clay to clay
5 - clayey silt to silty clay
6 - sandy silt to clayey silt
7 - silty sand to sandy silt
8 - sand to silty sand
9 - sand
10 - gravelly sand to sand
11 - very stiff fine grained (*)
12 - sand to clayey sand (*)

PLATE A-5

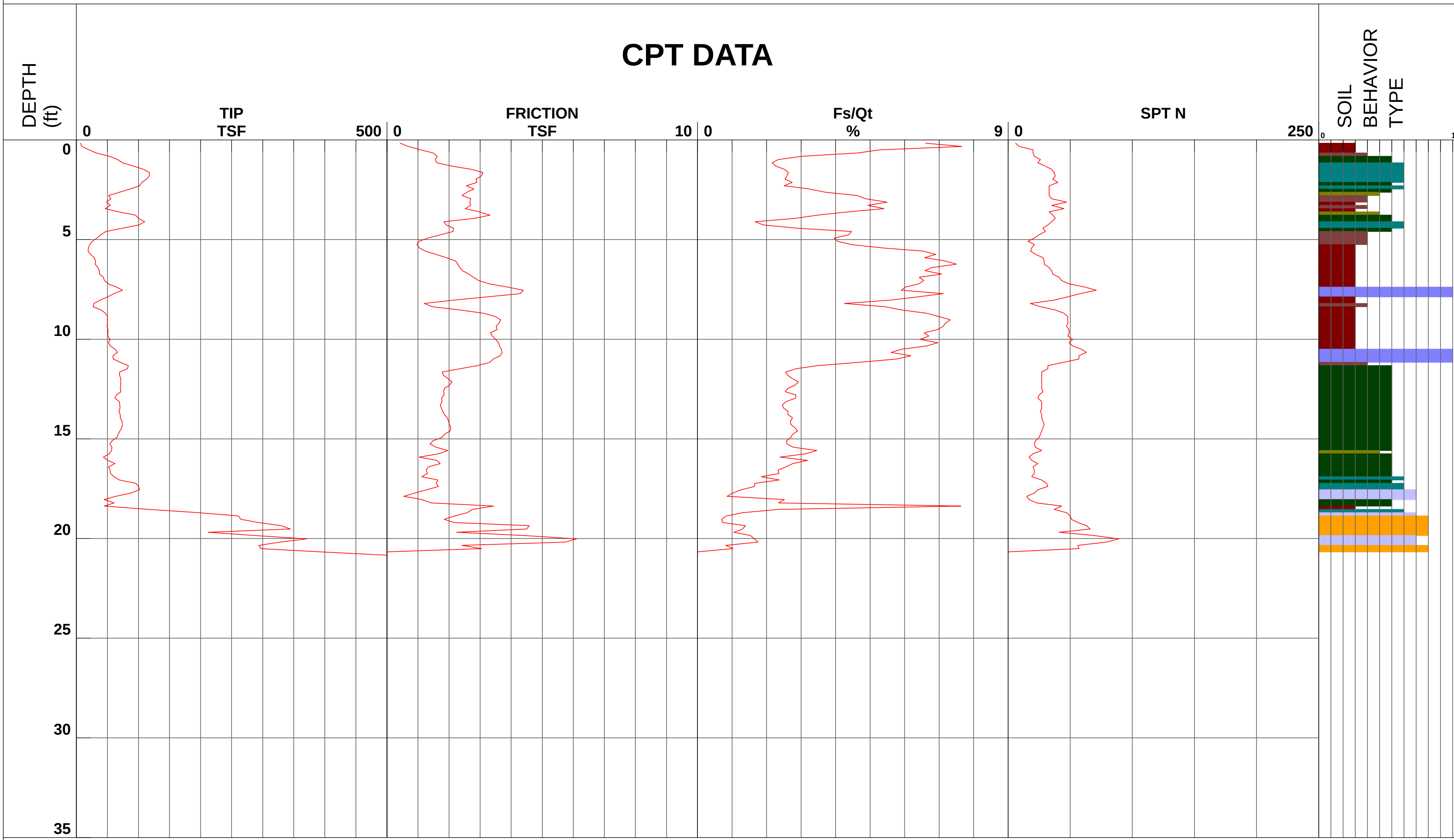
Depth Increment

*Soil behavior type and SPT based on data from UBC-1983

Geotechniques

Location	Moorpark College Parking Structure	Operator	ML/DH	Filename	SDF(863).cpt
Job Number	1003.026	Cone Number	DSG0906	GPS	
Hole Number	CPT-06	Date and Time	10/21/2010 12:10:56 PM	Maximum Depth	20.83 ft
Water Table Depth		NE			
Surface El. (ft.)	640				

CPT DATA



1 - sensitive fine grained	4 - silty clay to clay	7 - silty sand to sandy silt	10 - gravelly sand to sand
2 - organic material	5 - clayey silt to silty clay	8 - sand to silty sand	11 - very stiff fine grained (*)
3 - clay	6 - sandy silt to clayey silt	9 - sand	12 - sand to clayey sand (*)

PLATE A-6

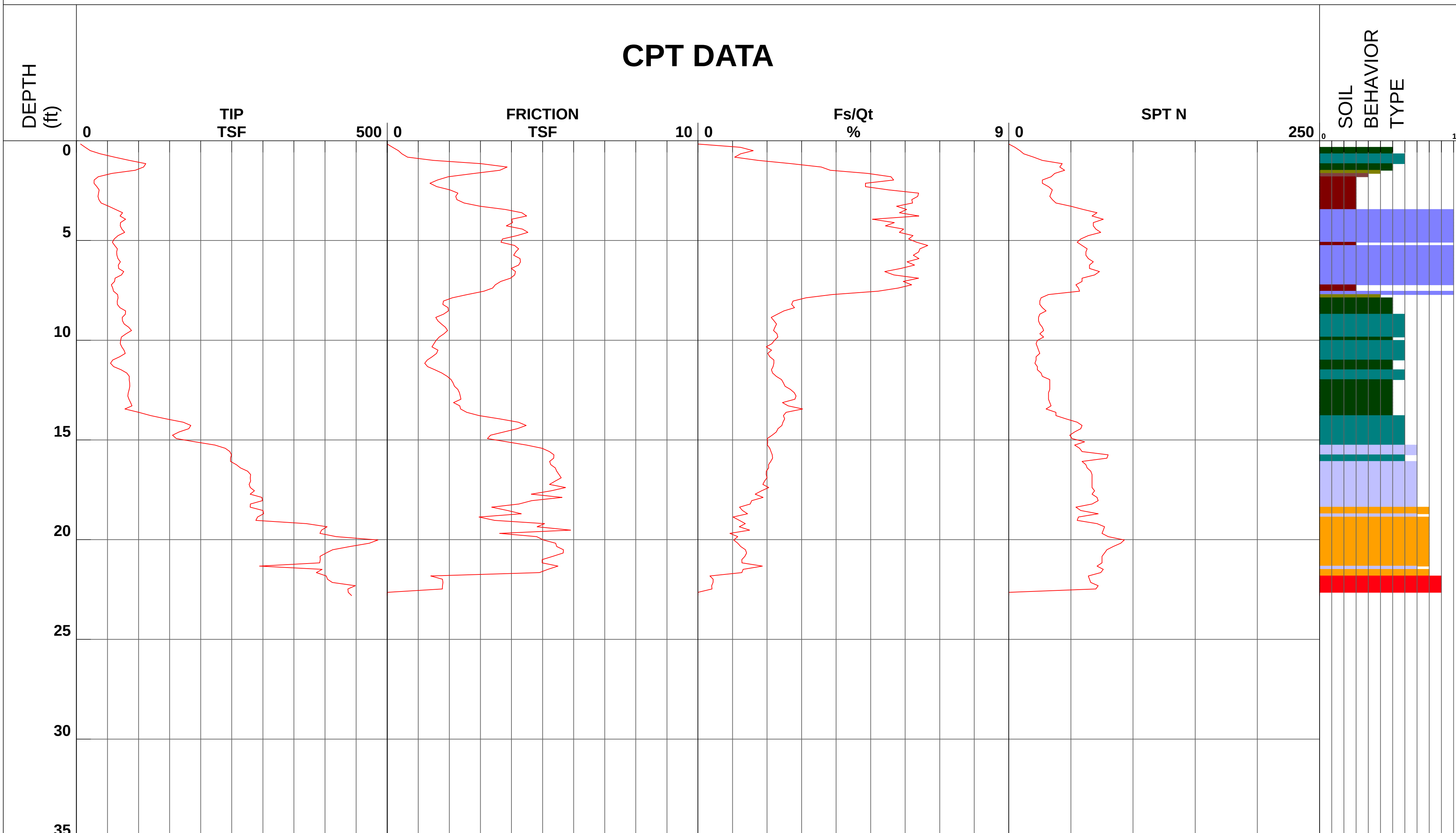
Depth Increment

*Soil behavior type and SPT based on data from UBC-1983

Geotechniques

Location	Moorpark College Parking Structure	Operator	ML/DH	Filename	SDF(867).cpt
Job Number	1003.026	Cone Number	DSG0906	GPS	
Hole Number	CPT-07	Date and Time	10/21/2010 2:43:08 PM	Maximum Depth	22.80 ft
Water Table Depth		NE			
Surface El. (ft.) 640					

CPT DATA



- 1 - sensitive fine grained
- 2 - organic material
- 3 - clay
- 4 - silty clay to clay
- 5 - clayey silt to silty clay
- 6 - sandy silt to clayey silt
- 7 - silty sand to sandy silt
- 8 - sand to silty sand
- 9 - sand
- 10 - gravelly sand to sand
- 11 - very stiff fine grained (*)
- 12 - sand to clayey sand (*)

PLATE A-7

Depth Increment

*Soil behavior type and SPT based on data from UBC-1983

GEOTECHNIQUES
LOG OF EXPLORATORY BORING

PROJECT NO.:	1003.026	**DRILLER:**	Martini Drilling	**LOGGED BY:**	CW
PROJECT NAME:	Parking Structure	**DRILL METHOD:**	8-inch Hollow Stem Auger	**OPERATOR:**	Gene/Brandon
LOCATION:	South Campus, Moorpark College	**HAMMER:**	140 pound auto-trip	**RIG TYPE:**	CME 75
ELEVATION:	641 feet (approx.)			**DATE:**	11/12/2010

BORING NO.: B-8

Depth (ft)	SAMPLES: Blows/ 6"	SPT N-value	Sample Number	USCS Symbol	MATERIAL DESCRIPTION AND COMMENTS	Laboratory Testing: Water Content (%)	Dry Density (pcf)	% Passing #200 Sieve
				CL	**OLDER ALLUVIUM (Qoa):** Fine sandy lean CLAY (CL): Red-brown, very dense, very moist			
5	9 15 18	33	20			14	124	
10	4 9 12	21	21	SP/ SC	Fine SAND with silt and lean clay (SP/SC)	13	119	
					- cobbles encountered, at 13'			
15	50		22					
20	9 11 25	36	23	GP	Gravel with fine to medium sand (GP): Tan to light brown, very dense, moist			
25	11 14 23	37	24	CL	Fine sandy lean CLAY (CL): Red-brown, hard, moist			

Legend: -Ring -Disturbed Ring - SPT - Bulk - No Recovery - Groundwater

check: cw 01/03/11

PLATE A-8.1

GEOTECHNIQUES
LOG OF EXPLORATORY BORING

PROJECT NO.:	1003.026	**DRILLER:**	Martini Drilling	**LOGGED BY:**	CW
PROJECT NAME:	Parking Structure	**DRILL METHOD:**	8-inch Hollow Stem Auger	**OPERATOR:**	Gene/Brandon
LOCATION:	South Campus, Moorpark College	**HAMMER:**	140 pound auto-trip	**RIG TYPE:**	CME 75
ELEVATION:	641 feet (approx.)			**DATE:**	11/12/2010

BORING NO.: B-8 (continued)

Depth (ft)	SAMPLES: Sample Type	Blows/ 6"	N-value	Sample Number	Graphical Log	USCS Symbol	MATERIAL DESCRIPTION AND COMMENTS	Laboratory Testing: Water Content (%)	Dry Density (pcf)	% Finer #200 Sieve/ Atterberg Limits
						CL	Fine sandy lean CLAY (CL): Red-brown, hard, moist			
30	Ring / Bulk	5 11 15	26	25						
35	Ring / Disturbed Ring	8 27 60	87	26		**SP**	Fine to medium SAND with gravel and cobbles (SP): Tan to gray, very dense, slightly moist	6	109	
40	Ring / Bulk	25 23 27	50	27			- orange-brown and tan, angular weathered gravel, at 40' tan, very dense, slightly moist			
							- gravel/cobble chatter between 42' and 44'			
	Bulk	60		28			- gravel-sized chips/shards weathered sandstone			
45							- Refusal, at 44' **TOTAL DEPTH 44 FEET** **GROUNDWATER NOT ENCOUNTERED** BORING BACKFILLED WITH CUTTINGS UPON COMPLETION			
50										
55										

Legend: -Ring ' - Disturbed Ring '-Bulk -No Recovery -Groundwater

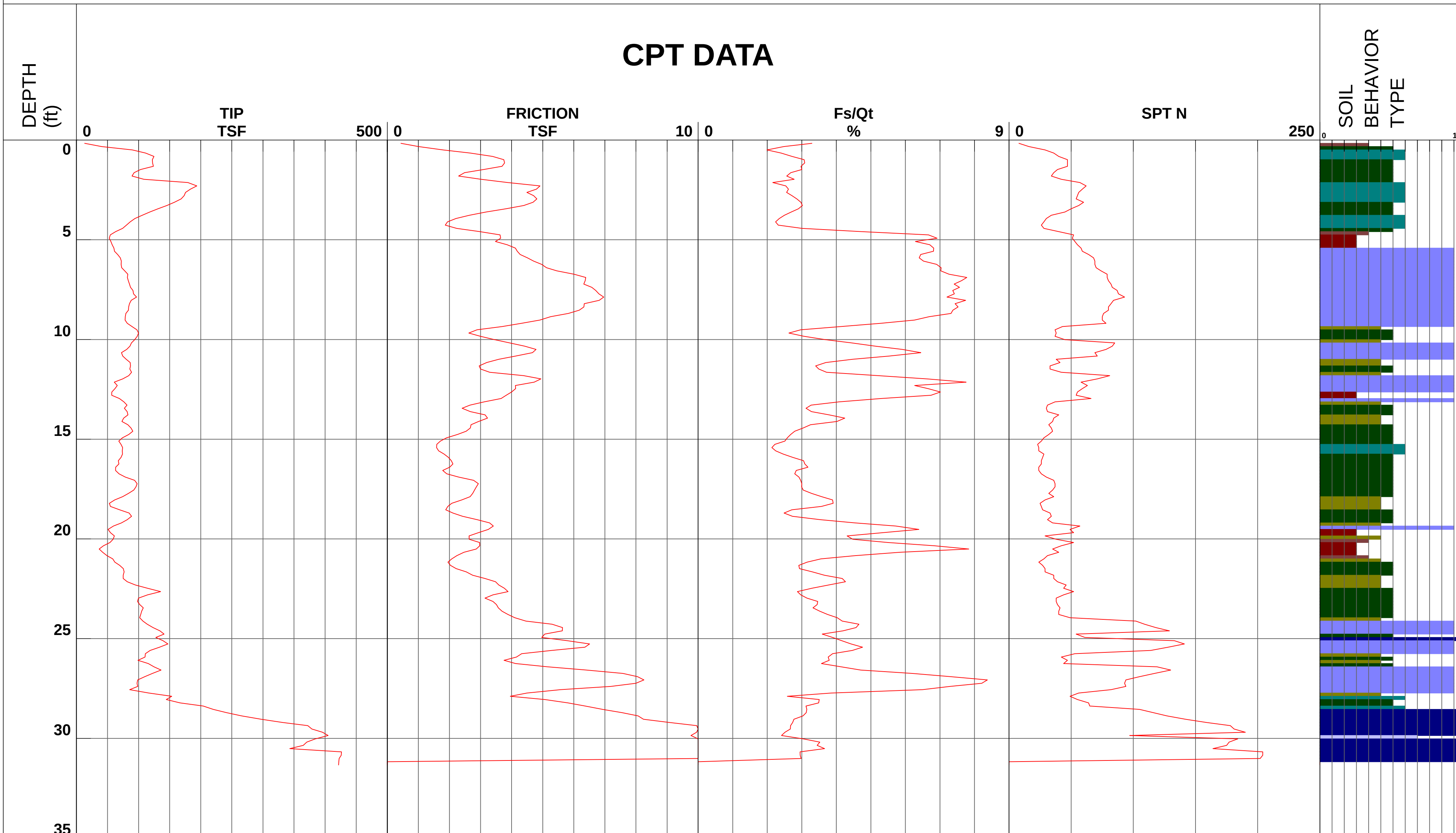
check: cw 01/03/11

PLATE A-8.2

Geotechniques

Location	Moorpark College Parking Structure	Operator	ML/DH	Filename	SDF(862).cpt
Job Number	1003.026	Cone Number	DSG0906	GPS	
Hole Number	CPT-09	Date and Time	10/21/2010 11:12:51 AM	Maximum Depth	31.33 ft
Water Table Depth		NE			
Surface El. (ft.) 650					

CPT DATA



1 - sensitive fine grained	4 - silty clay to clay	7 - silty sand to sandy silt	10 - gravelly sand to sand
2 - organic material	5 - clayey silt to silty clay	8 - sand to silty sand	11 - very stiff fine grained (*)
3 - clay	6 - sandy silt to clayey silt	9 - sand	12 - sand to clayey sand (*)

PLATE A-9

Depth Increment

*Soil behavior type and SPT based on data from UBC-1983

GEOTECHNIQUES
LOG OF EXPLORATORY BORING

PROJECT NO.:	1003.026	**DRILLER:**	Martini Drilling	**LOGGED BY:**	CW
PROJECT NAME:	Parking Structure	**DRILL METHOD:**	8-inch Hollow Stem Auger	**OPERATOR:**	Gene/Brandon
LOCATION:	South Campus, Moorpark College	**HAMMER:**	140 pound auto-trip	**RIG TYPE:**	CME 75
ELEVATION:	649 feet (approx.)			**DATE:**	11/12/2010

BORING NO.: B-10

Depth (ft)	SAMPLES: Sample Type	SAMPLES: Blows/ 6"	SAMPLES: SPT N-value	SAMPLES: Sample Number	Graphical Log	USCS Symbol	MATERIAL DESCRIPTION AND COMMENTS	Laboratory Testing: Water Content (%)	Laboratory Testing: Dry Density (pcf)	Laboratory Testing: % Finer #200 Sieve/ Atterberg Limits
1–8	Bulk					CL	**ARTIFICIAL FILL (Qaf):** Fine sandy CLAY (CL): Red-brown, very firm, moist			
10	Ring	8 12 17	29	29		CL	**Older Alluvium (Qoa):** Fine sandy lean CLAY (CL): Medium reddish-brown, dense, trace gravel, moist	11	121	
15	SPT	8 15 15	30	30				6		LL~26 PI~14
20	Ring	7 14 28	42	31		SM	Silty fine to medium sand with clay (SM/SC): Medium light brown, very dense, moist	7	113	
25	SPT	9 15 17	32	32						11

Legend: -Ring -Disturbed Ring - SPT - Bulk - No Recovery - Groundwater

check: cw 01/03/11

GEOTECHNIQUES
LOG OF EXPLORATORY BORING

PROJECT NO.:	1003.026	**DRILLER:**	Martini Drilling	**LOGGED BY:**	CW
PROJECT NAME:	Parking Structure	**DRILL METHOD:**	8-inch Hollow Stem Auger	**OPERATOR:**	Gene/Brandon
LOCATION:	South Campus, Moorpark College	**HAMMER:**	140 pound auto-trip	**RIG TYPE:**	CME 75
ELEVATION:	649 feet (approx.)			**DATE:**	11/12/2010

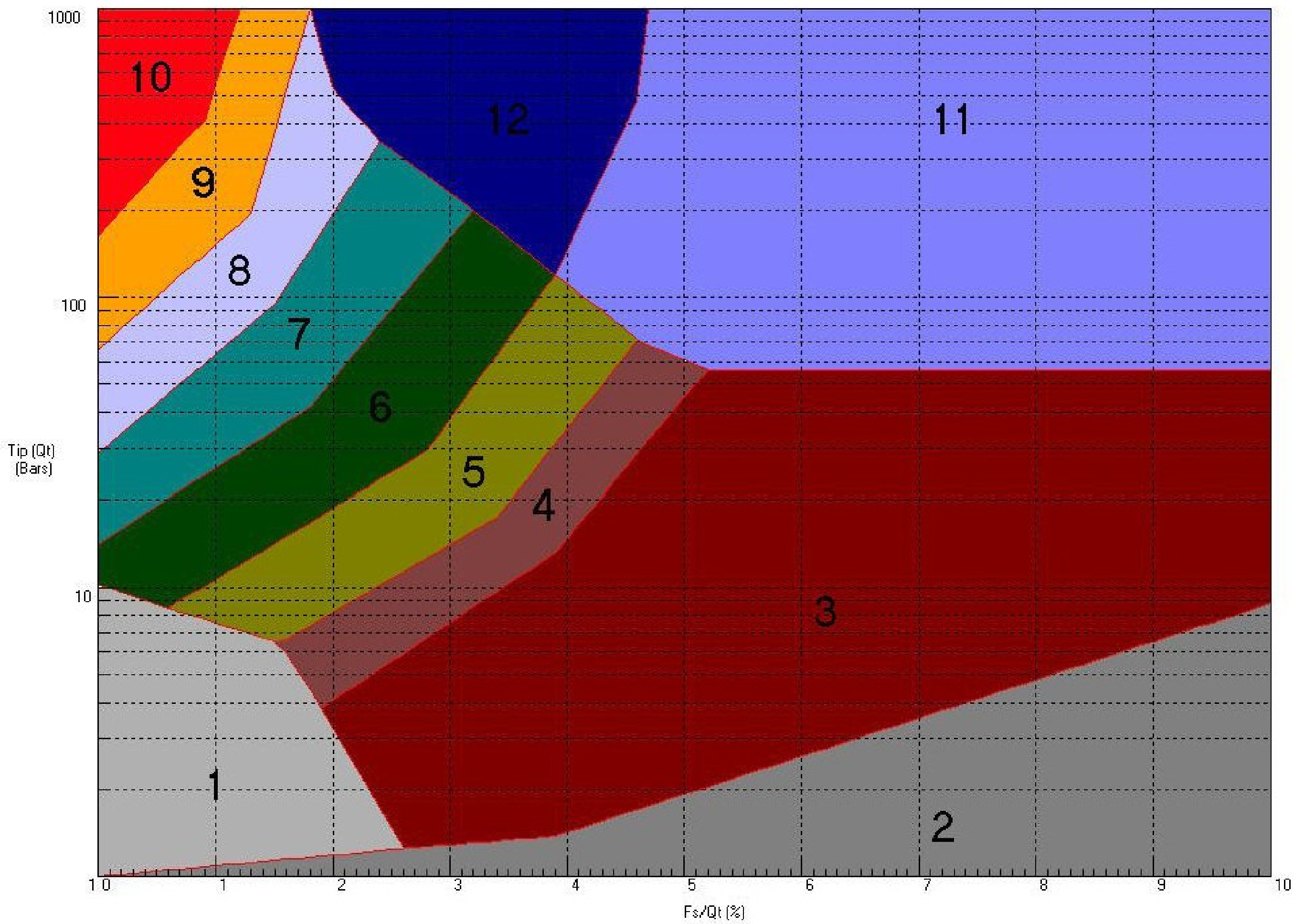
BORING NO.: B-10 (continued)

Depth (ft)	SAMPLES: Sample Type	Blows/ 6"	N-value	Sample Number	Graphical Log	USCS Symbol	MATERIAL DESCRIPTION AND COMMENTS	Laboratory Testing: Water Content (%)	Dry Density (pcf)	% Finer #200 Sieve/ Atterberg Limits
						SP	Fine to medium SAND (SP): Tan, very dense			
30	Ring	60		33			- cobble-sized rock obstructed sampler			
							- Refusal, at 30'			
							TOTAL DEPTH 30 FEET **GROUNDWATER NOT ENCOUNTERED**			
							BORING BACKFILLED WITH CUTTINGS UPON COMPLETION			
35										
40										
45										
50										
55										

Legend: -Ring; ' - Disturbed Ring; '-Bulk; -No Recovery; -Groundwater

check: cw 01/03/11

PLATE A-10.2



1 sensitive fine grained	4 silty clay to clay	7 silty sand to sandy silt	10 gravelly sand to sand
2 organic material	5 clayey silt to silty clay	8 sand to silty sand	11 very stiff fine grained (*)
3 clay	6 sandy silt to clayey silt	9 sand	12 sand to clayey sand (*)

COLOR KEY FOR CPT LOGS

PARKING STRUCTURE
MOORPARK COLLEGE
Moorpark, California

APPENDIX B
LABORATORY TESTING

APPENDIX B
LABORATORY TESTING

Laboratory tests were performed on selected relatively undisturbed and bulk soil samples to estimate engineering characteristics of the various earth materials encountered. Testing was performed in general accordance with ASTM Standards for Soil Testing, latest revision. Laboratory test results are summarized on Plates B-1 through B-5 of this appendix, on the boring logs, and in the report text.

LABORATORY MOISTURE AND DENSITY DETERMINATIONS

Moisture content and dry density determinations were performed on selected samples collected to evaluate the natural water content and dry density of the various soils encountered. The results are presented on the boring logs.

EXPANSION INDEX

An expansion index test was performed on a selected sample of near-surface soil from the upper 5 feet of boring B-2 to estimate expansion characteristics. The test was performed in general accordance with ASTM D4329. The results of that test indicate an Expansion Index of 2.

GRAIN-SIZE TESTS

Grain size distribution was determined in general accordance with standard test method ASTM D422. In addition, we performed tests to determine the amount of material in soils finer than the No. 200 Sieve in accordance with ASTM test method D1140. The grain-size curve is presented on Plate B-1 - Grain Size Curve, and the results of percent passing the No. 200 sieve (or fines content) are presented on the boring logs.

ATTERBERG LIMITS

Liquid and plastic limits were determined in general accordance with standard test method ASTM D4318. The test results are shown on Plate B-2 – Atterberg Limits.

COMPACTION CURVE

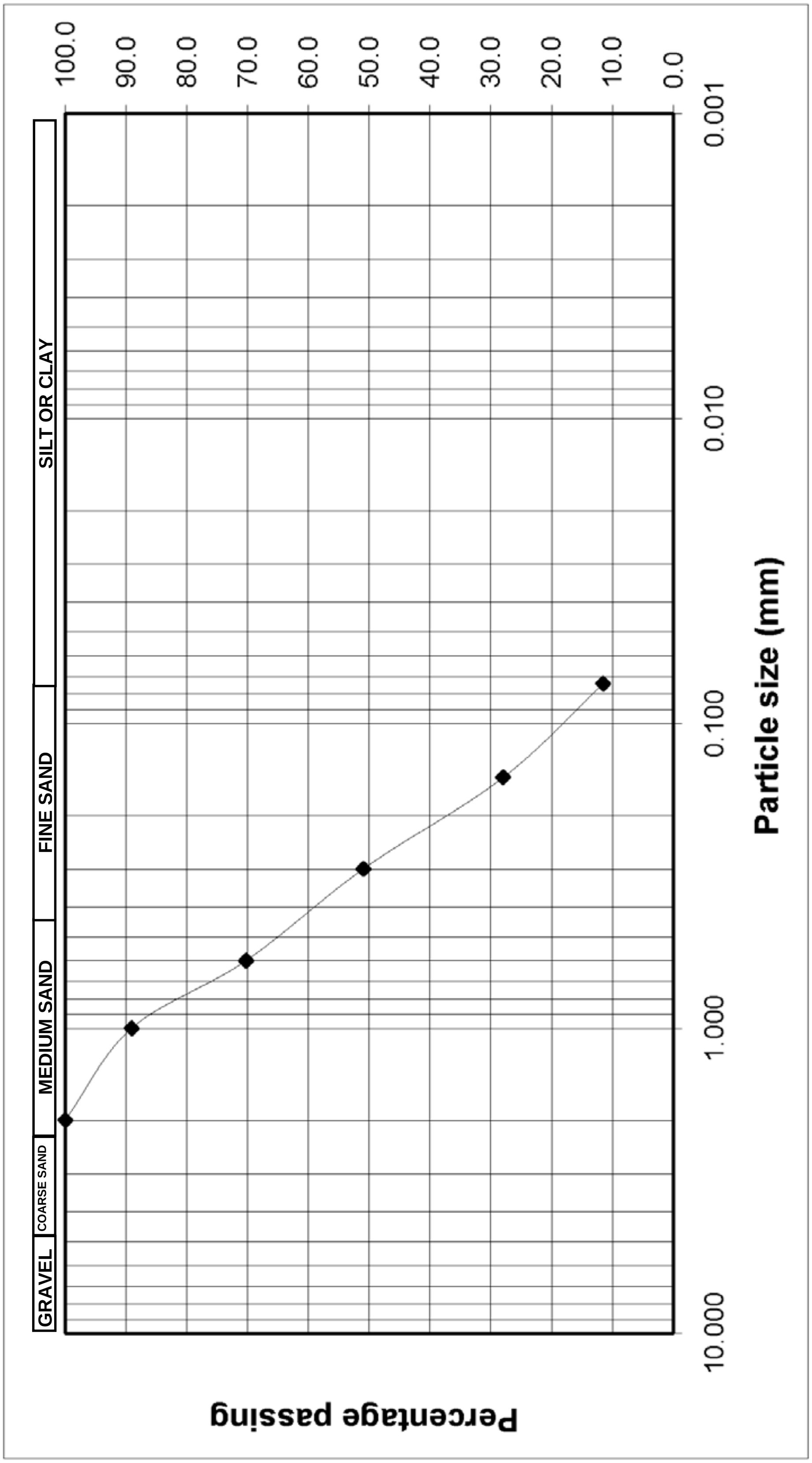
A compaction test was performed on a selected bulk sample of near-surface clay soil to assess compaction characteristics. The test was performed in general accordance with ASTM D1557, and results are presented on Plate B-3 – Compaction Curve.

CONSOLIDATION TESTS

One-dimensional consolidation tests were performed on driven-ring samples in general accordance with ASTM Test Method 2435. The samples were incrementally loaded up to the approximate overburden pressure, flooded with water, and incrementally loaded to 32 kips per square foot (ksf), and allowed to consolidate under each load increment. Displacement measurements were recorded at the end of each load increment and after equilibrium was achieved after flooding the samples with water. The samples were unloaded and allowed to rebound. Displacements also were measured and recorded during the unloading cycle. The results of the consolidation tests, in the form of percent consolidation versus log of pressure curves, are presented on Plate B-4 - Consolidation Test Results.

SOIL CHEMISTRY TESTS/CORROSION TESTS

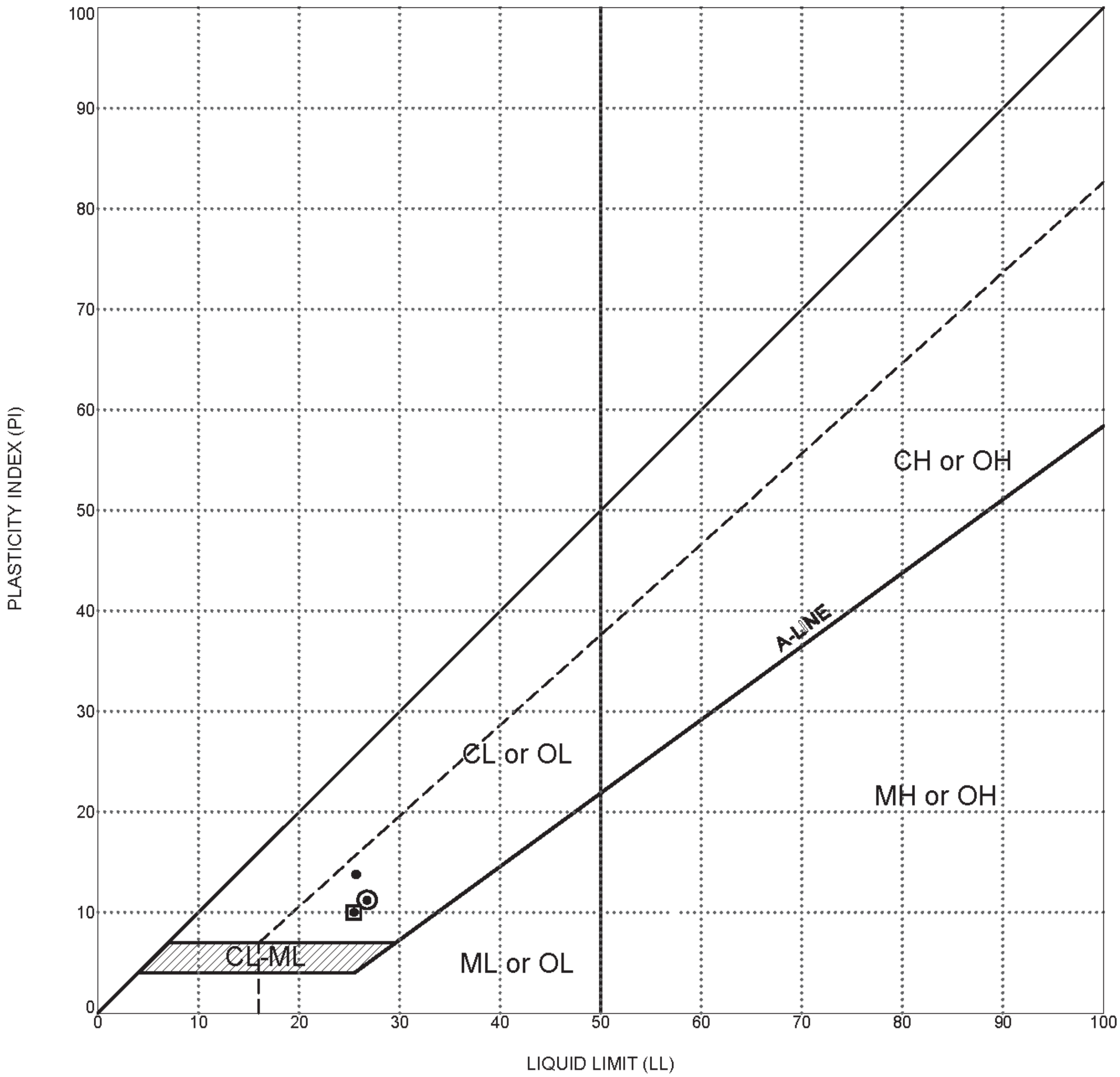
One sample of the near-surface soil was tested for resistivity, pH, sulfate, and chloride content to assess corrosion potential by Atlantic Consultants, Inc., of Folsom, California. The results of the tests and corresponding recommendations are presented on Plate B-5 - Laboratory Report (Atlantic Consultants' report), and summarized in tabular form in the report text.



BORING/DEPTH	**CLASSIFICATION**
B-10/26'	Silty fine to medium sand with clay (**SM/SC**)

GRAIN SIZE CURVE

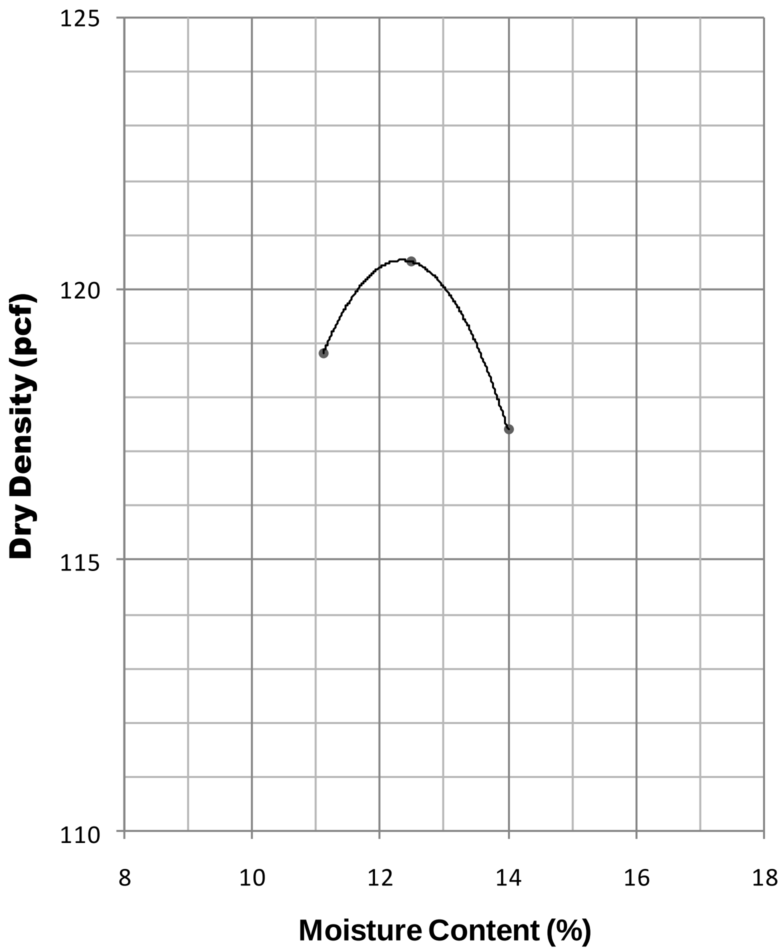
PARKING STRUCTURE
MOORPARK COLLEGE
Moorpark, California



SYMBOL	SAMPLE	SOIL DESCRIPTION	ATTERBERG LIMITS Liquid Limit (LL)	Plastic Limit (PL)	Plasticity Index (PI)
⊡	B-1 @ 15'	Fine sandy lean CLAY (**CL**)	**26**	**16**	**10**
⊙	B-2 @ 30'	Fine sandy lean CLAY (**CL**)	**27**	**16**	**11**
●	B-10 @ 15'	Fine sandy lean CLAY (**CL**)	**26**	**12**	**14**

ATTERBERG LIMITS
PARKING STRUCTURE
MOORPARK COLLEGE
Moorpark, California

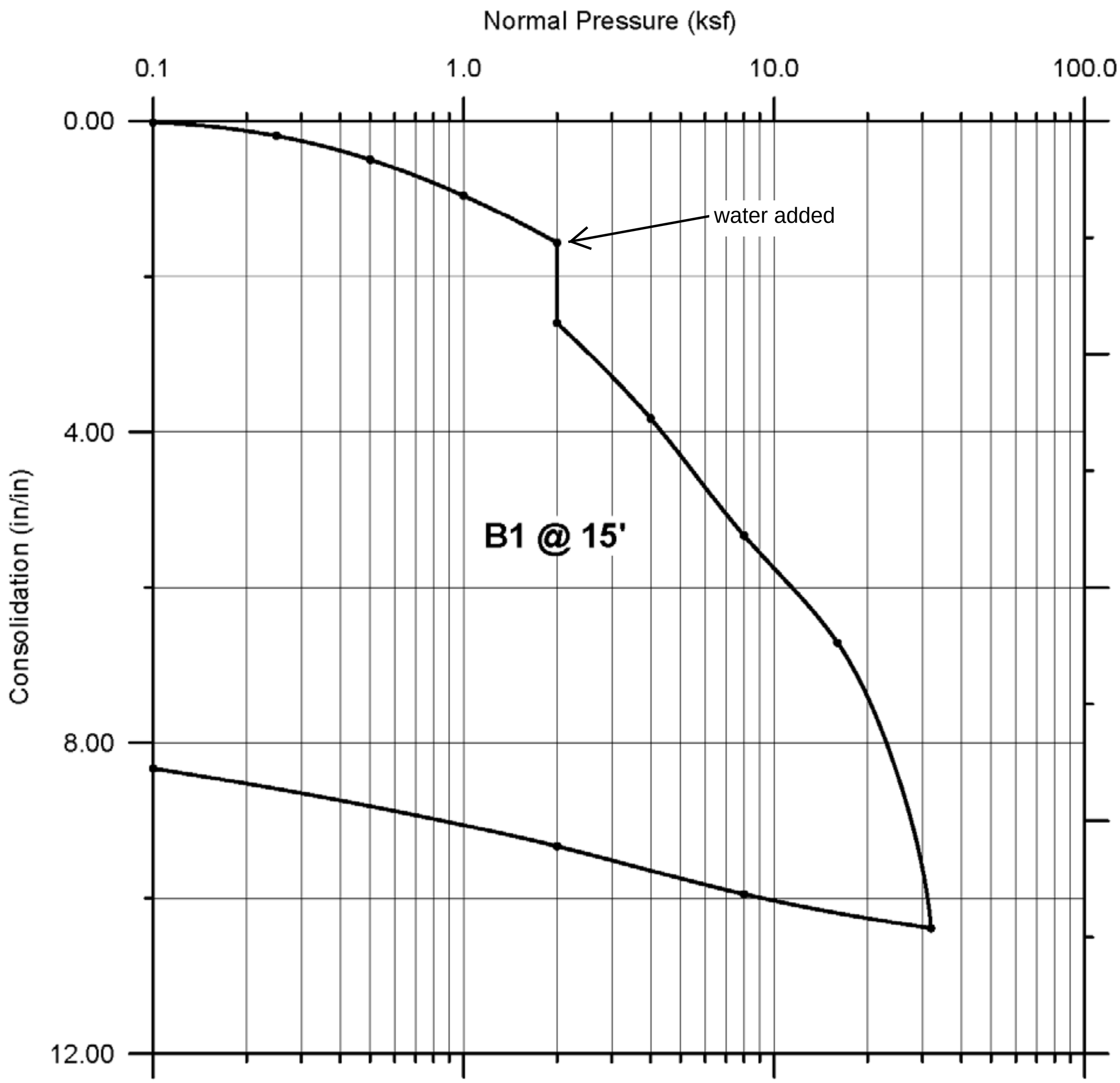
PLATE B-2



Soil Description	Maximum Dry Density	Optimum Moisture Content
Red-brown clayey sand (**SC**)	120.5 pcf	12.5%

COMPACTION CURVE

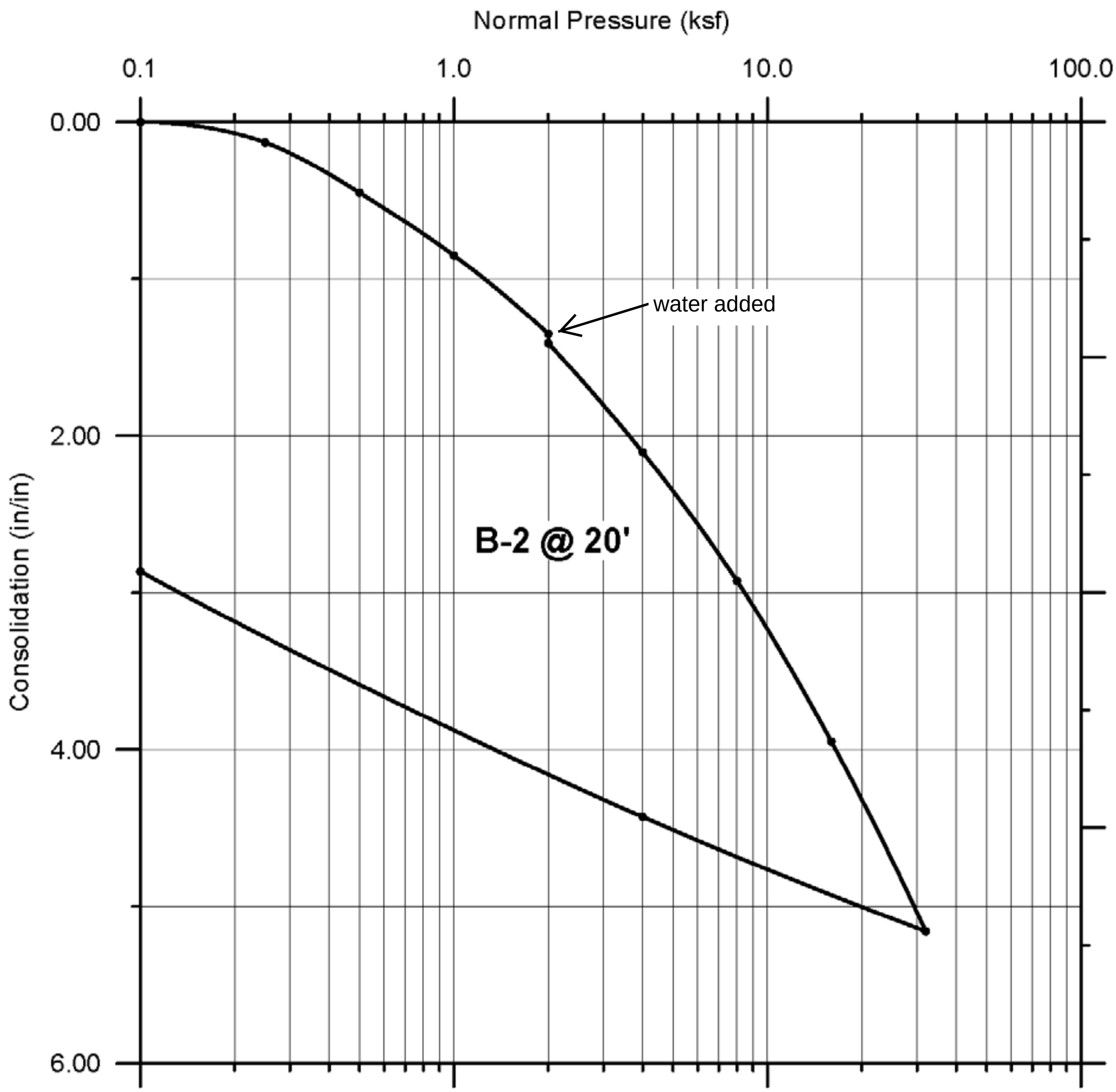
PARKING STRUCTURE
MOORPARK COLLEGE
Moorpark, California



Soil Description	**Unit Dry Weight**	**Moisture Content**
Red-brown fine sandy lean CLAY (**CL**)	110.4 pcf	11.9%

CONSOLIDATION TEST RESULTS

PARKING STRUCTURE
MOORPARK COLLEGE
Moorpark, California



Soil Description	**Unit Dry Weight**	**Moisture Content**
Fine SAND with silt and clay (**SC/SM**)	115.3 pcf	13.3%

CONSOLIDATION TEST RESULTS

PARKING STRUCTURE
MOORPARK COLLEGE
Moorpark, California

Atlantic Consultants

112 Bunker Court
Folsom, CA 95630
(ph) 916.849.6420 (fax) 916.983.1838
Kerri@AtlanticCorrosionEngineers.com
corrprincess@ardennet.com
www.AtlanticCorrosionEngineers.com

December 3, 2010

Geotechniques
Attention: Carole Wockner
1645 Donlon Street, Suite 107
Ventura, CA 93003

Atlantic Job No.: 2010-049

Subject: Soil Chemistry Analysis for Geotechniques- Job # 1003.025
1 Sample: Moorpark College Parking Structure (B-1 @ 1-5')

Sample Number	As Rec'd Resistivity (ohm-cm)	[1]Minimum Resistivity (ohm-cm)	[2]pH	[3]Sulfate %	[3]Chloride %	As Rec'd Description
B-1	7,200	1,360	7.84	0.0200	0.0100	Med. Brn Sandy clay.

NOTE: SAMPLES WERE ANALYZED IN ACCORDANCE WITH THE FOLLOWING METHODS.
1. MINIMUM RESISTIVITY DETERMINED BY SOIL BOX METHOD, (PER ASTM G-57)
2. PH MEASURED BY POTENTIOMETRIC METHOD USING STANDARD ELECTRODES. (PER CAL TRANS. #643)
3. CHLORIDE AND SULFATE WERE ANALYZED IN ACCORDANCE WITH EPA METHODS FOR CHEMICAL ANALYSIS FOR WATER AND WASTE, NO. 300 EPA-600/4-79-020. CONCENTRATION BY WEIGHT OF DRY SOIL.

CONCLUSIONS:

Material	Corrosion Class	Recommendation
Concrete	Negligible for sulfate exposure and negligible for chloride exposure, pH is neutral to slightly basic. (ACI 318)	-Type II Portland cement for concrete with a maximum water cement ratio of 0.50 and a minimum of 3 inches of cover over steel reinforcement. It is suggested that a 6 mil polyethylene barrier be placed between concrete slabs and soil to reduce intrusion of moisture into the concrete slabs.
Steel Cast/Ductile Iron Mortar Coated Steel	Moderately Corrosive	- Install corrosion monitoring and cathodic protection for buried ferrous metal piping. - Provide electrical continuity along steel and ductile iron piping, to facilitate the installation of corrosion monitoring and cathodic protection. - Electrically isolate underground metal piping from above grade piping and other metallic structures. - Use separate ground rods for grounding interior piping.
Copper Piping	Corrosive Not tested for Ammonia NOTE: The soils were not tested for ammonium. Even trace amounts of ammonium can cause failure of copper piping.	- Overhead plumbing is the most effective method of corrosion control. - Copper pipes should not be installed in soils, which may contain ammonia without cathodic protection. - If Copper pipes are installed below ground, the soils should be tested for ammonia and Keldahl nitrogen. - Electrical isolation between hot and cold water lines and between buried copper and steel piping and structural steel should be maintained. - If ammonia is present, coat and cathodically protect any buried copper piping.

112 Bunker Court
Folsom, CA 95630
(ph) 916.849.6420 (fax) 916.983.1838
Kerri@AtlanticCorrosionEngineers.com
corrprincess@ardennet.com
www.AtlanticCorrosionEngineers.com

The test results and recommendations are based on the sample submitted, which may not be representative of overall site conditions. Additional sampling may be required to more fully characterize soil conditions.

Sincerely,
ATLANTIC CONSULTANTS, INC.

Kerri M. Howell, P.E.
President

APPENDIX C

SEISMICALLY INDUCED SETTLEMENT CALCULATIONS

1/3/2011
Proj. No 1003.03

Moorpark College Parking Structure
Blowcount Corrections and Calculation of Seismically Induced Settlement
Ref: Pradel (1998)

pga,g 0.64
Mag. 7.5

B-1

Borehole Dia. In. 8

Ef
Rope and cathead 60
Auto-trip hammer 90
Ratio 1.5

Corrections of Blowcount Data

po, psf 2116.8

Ncal/Nspt 1.60

Layer No.	Depth to Layer Bot. ft	Layer Thick. ft	Unit Weight pcf	sigma @ Bot. Lyr. psf	Depth @ Mid. Pt. ft	Sigma @ Mid. Pt. psf	Cn	Energy Corr.	Borehole Dia. Corr.	Liner Corr.	Rod Corr.	N-cal-fld bls/ft	N-spt-fld bls/ft	Est. fld. N-spt bls/ft	Est. N60 bls/ft	N1/60 bls/ft	Material	Fines %	Clay %	Correction for Triggering Analysis del N-fc	N-eff bls/ft	Correction for Residual Strength del N-fc	N-eff bls/ft	Layer
1	6	6	122	732.0	3.0	366.0	2.00	1.5	1	1.0	0.75	40		25	28	56		50	20		56		56	1
2	14.5	8.5	114	1701.0	10.3	1216.5	1.32	1.5	1	1.0	0.75	11		7	8	10		75	20		10		10	2
3	21	6.5	120	2481.0	17.8	2091.0	1.01	1.5	1	1.2	0.90	13		8	13	13		50	20		13		13	3
4	26	5	120	3081.0	23.5	2781.0	0.87	1.5	1	1.2	0.96	29		18	32	28		50	10		28		28	4
5	31	5	120	3681.0	28.5	3381.0	0.79	1.5	1	1.0	1.00		29	29	44	34		25	5		34		34	5
6	39.5	8.5	120	4701.0	35.3	4191.0	0.71	1.5	1	1.2	1.00	69		43	78	56		5	0		56		56	6
7	50.5	11	120	6021.0	45.0	5361.0	0.63	1.5	1	1.2	1.00	67		42	76	48		80	15		48		48	7

Seismically Induced Settlement
Ko 0.475

zo, ft 100 | po, psf 2116.8 | Nc 15.2 | 1.005 | Liquefaction-Related Settlement

Layer No.	rd	Avg. Shear Stress psf	sigma-mean psf	Gmax psf	a	b	Gamma	strain-15	strain-Nc	del S in	Interpr. del S in	Cum Dry sett. in	Layer	N1/60eff bls/ft	liq-strain %	Inc. sett. in	Cum. Liq. Sett. in	Inc. Sett. in	Cum. S in	Depth ft
1	0.999	152.1	237.9	1.22E+06	0.128	23755	3.89E-04	1.13E-04	1.13E-04	0.016	0.000	0.233	1	56			0.00	0.000	0.2	0.0
2	0.990	500.8	790.7	1.25E+06	0.139	11555	5.25E-03	1.18E-02	1.18E-02	2.413	0.000	0.233	2	10			0.00	0.000	0.2	6.0
3	0.969	843.3	1359.2	1.80E+06	0.149	8349	3.45E-03	5.61E-03	5.63E-03	0.879	0.000	0.233	3	13			0.00	0.000	0.2	14.5
4	0.948	1096.4	1807.7	2.64E+06	0.157	7036	1.41E-03	9.57E-04	9.61E-04	0.115	0.115	0.233	4	28			0.00	0.115	0.2	21.0
5	0.925	1300.8	2197.7	3.14E+06	0.164	6258	1.14E-03	5.95E-04	5.98E-04	0.072	0.072	0.118	5	34			0.00	0.072	0.1	26.0
6	0.889	1550.8	2724.2	4.10E+06	0.174	5501	7.72E-04	2.26E-04	2.27E-04	0.046	0.046	0.046	6	56			0.00	0.046	0.0	31.0
7	0.832	1854.6	3484.7	4.40E+06	0.188	4746	8.46E-04	2.98E-04	2.99E-04	0.079	0.000	0.000	7	48			0.00	0.000	0.0	39.5

Total, inches 0.233

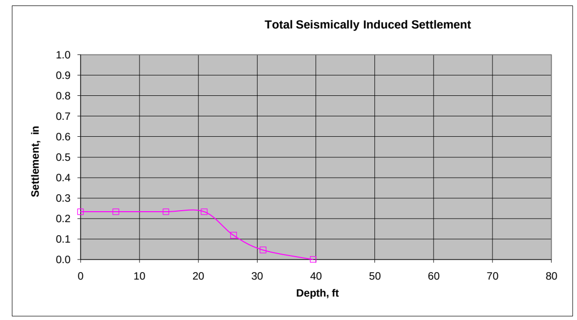


PLATE C-1

1/3/2011
Proj. No 1003.03

Moorpark College Parking Structure
Blowcount Corrections and Calculation of Seismically Induced Settlement
Ref: Pradel (1998)

pga,g 0.64
Mag. 7.5

B-2

Borehole Dia. In. 8 | Ef
Rope and cathead 60
Auto-trip hammer 90
Ratio 1.5

Corrections of Blowcount Data — po, psf 2116.8

Ncal/Nspt 1.60

Layer No.	Depth to Layer Bot. ft	Layer Thick. ft	Unit Weight pcf	sigma @ Bot. Lyr. psf	Depth @ Mid. Pt. ft	Sigma @ Mid. Pt. psf	Cn	Energy Corr.	Borehole Dia. Corr.	Liner Corr.	Rod Corr.	N-cal-fld bls/ft	N-spt-fld bls/ft	Est. fld. N-spt bls/ft	Est. N60 bls/ft	N1/60 bls/ft	Material	Fines %	Clay %	Correction for Triggering Analysis del N-fc	N-eff bls/ft	Correction for Residual Strength del N-fc	N-eff bls/ft	Layer
1	6	6	122	732.0	3.0	366.0	2.00	1.5	1	1.0	0.75	52		33	37	73		75	20		73		73	1
2	12	6	114	1416.0	9.0	1074.0	1.40	1.5	1	1.0	0.75	19		12	13	19		75	20		19		19	2
3	18	6	120	2136.0	15.0	1776.0	1.09	1.5	1	1.2	0.90		17	17	28	30		50	10		30		30	3
4	22.5	4.5	120	2676.0	20.3	2406.0	0.94	1.5	1	1.0	0.96	23		14	21	19		50	10		19		19	4
5	30	7.5	120	3576.0	26.3	3126.0	0.82	1.5	1	1.0	1.00	49		31	46	38		25	5		38		38	5
6	31	1	120	3696.0	30.5	3636.0	0.76	1.5	1	1.2	1.00		6	6	11	8		20	15		8		8	6
7	45	14	120	5376.0	38.0	4536.0	0.68	1.5	1	1.2	1.00		42	42	76	52		10	0		52		52	7

Seismically Induced Settlement
Ko 0.475

zo, ft 100 | po, psf 2116.8 | Nc 15.2 | 1.005 | Liquefaction-Related Settlement

Layer No.	rd	Avg. Shear Stress psf	sigma-mean psf	Gmax psf	a	b	Gamma	strain-15	strain-Nc	del S in	Interpr. del S in	Cum Dry sett. in	Layer	N1/60eff bls/ft	liq-strain %	Inc. sett. in	Cum. Liq. Sett. in	Inc. Sett. in	Cum. S in	Depth ft
1	0.999	152.1	237.9	1.33E+06	0.128	23755	3.01E-04	6.34E-05	6.37E-05	0.009	0.000	0.513	1	73			0.00	0.000	0.5	0.0
2	0.992	443.2	698.1	1.44E+06	0.137	12452	1.96E-03	2.12E-03	2.13E-03	0.306	0.000	0.513	2	19			0.00	0.000	0.5	6.0
3	0.978	722.6	1154.4	2.18E+06	0.145	9208	1.18E-03	7.17E-04	7.20E-04	0.104	0.104	0.513	3	30			0.00	0.104	0.5	12.0
4	0.961	961.5	1563.9	2.19E+06	0.153	7675	2.09E-03	2.16E-03	2.17E-03	0.235	0.235	0.409	4	19			0.00	0.235	0.4	18.0
5	0.936	1216.6	2031.9	3.11E+06	0.161	6559	1.04E-03	4.86E-04	4.88E-04	0.088	0.088	0.175	5	38			0.00	0.088	0.2	22.5
6	0.915	1383.8	2363.4	2.03E+06	0.167	5991	6.46E-03	1.85E-02	1.86E-02	0.447	0.000	0.087	6	8			0.00	0.000	0.1	30.0
7	0.874	1648.9	2948.4	4.17E+06	0.178	5246	8.12E-04	2.57E-04	2.59E-04	0.087	0.087	0.087	7	52			0.00	0.087	0.1	31.0

Total, inches 0.513

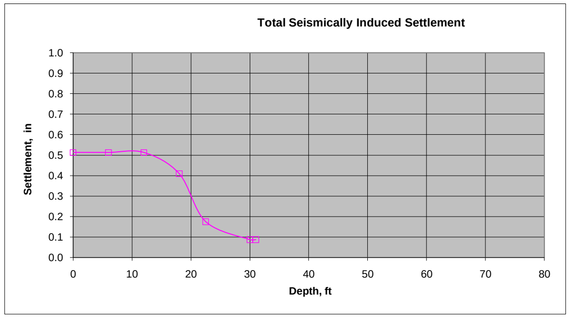


PLATE C-2

1/3/2011
Proj. No 1003.03

Moorpark College Parking Structure
Blowcount Corrections and Calculation of Seismically Induced Settlement
Ref: Pradel (1998)

pga,g 0.64
Mag. 7.5

B-8

Borehole Dia. In. 8

Ef
Rope and cathead 60
Auto-trip hammer 90
Ratio 1.5

Corrections of Blowcount Data

po, psf 2116.8

Ncal/Nspt 1.60

Layer No.	Depth to Layer Bot. ft	Layer Thick. ft	Unit Weight pcf	sigma @ Bot. Lyr. psf	Depth @ Mid. Pt. ft	Sigma @ Mid. Pt. psf	Cn	Energy Corr.	Borehole Dia. Corr.	Liner Corr.	Rod Corr.	N-cal-fld bls/ft	N-spt-fld bls/ft	Est. fld. N-spt bls/ft	Est. N60 bls/ft	N1/60 bls/ft	Material	Fines %	Clay %	Correction for Triggering Analysis del N-fc	N-eff bls/ft	Correction for Residual Strength del N-fc	N-eff bls/ft	Layer
1	9.5	9.5	125	1187.5	4.8	593.8	1.89	1.5	1	1.0	0.75	52		33	37	69		70	20		69		69	1
2	12	2.5	125	1500.0	10.8	1343.8	1.26	1.5	1	1.0	0.75	19		12	13	17		70	20		17		17	2
3	18	6	125	2250.0	15.0	1875.0	1.06	1.5	1	1.2	0.82		17	17	25	27		18	10		27		27	3
4	24.5	6.5	120	3030.0	21.3	2640.0	0.90	1.5	1	1.2	0.96		36	36	63	56		50	10		56		56	4
5	29.5	5	120	3630.0	27.0	3330.0	0.80	1.5	1	1.0	1.00		37	37	56	44		25	5		44		44	5
6	34.5	5	120	4230.0	32.0	3930.0	0.73	1.5	1	1.2	1.00		26	26	47	35		5	0		35		35	6
7	45	10.5	120	5490.0	39.8	4860.0	0.66	1.5	1	1.2	1.00		87	87	158	104		80	15		104		104	7

Seismically Induced Settlement
Ko 0.475

zo, ft 100 po, psf 2116.8 Nc 15.2 1.005 Liquefaction-Related Settlement

Layer No.	rd	Avg. Shear Stress psf	sigma-mean psf	Gmax psf	a	b	Gamma	strain-15	strain-Nc	del S in	Interpr. del S in	Cum Dry sett. in	Layer	N1/60eff bls/ft	liq-strain %	Inc. sett. in	Cum. Liq. Sett. in	Inc. Sett. in	Cum. S in	Depth ft
1	0.998	246.4	385.9	1.66E+06	0.131	17770	3.73E-04	8.45E-05	8.49E-05	0.019	0.000	0.288	1	69			0.00	0.000	0.3	0.0
2	0.989	552.6	873.4	1.56E+06	0.140	10886	2.40E-03	2.96E-03	2.98E-03	0.179	0.000	0.288	2	17			0.00	0.000	0.3	9.5
3	0.978	762.8	1218.8	2.15E+06	0.146	8913	1.38E-03	9.67E-04	9.71E-04	0.140	0.140	0.288	3	27			0.00	0.140	0.3	12.0
4	0.957	1050.8	1716.0	3.26E+06	0.156	7259	7.28E-04	2.11E-04	2.12E-04	0.033	0.033	0.148	4	56			0.00	0.033	0.1	18.0
5	0.932	1291.2	2164.5	3.38E+06	0.164	6315	9.25E-04	3.57E-04	3.58E-04	0.043	0.043	0.115	5	44			0.00	0.043	0.1	24.5
6	0.907	1483.0	2554.5	3.39E+06	0.171	5717	1.15E-03	5.97E-04	6.00E-04	0.072	0.072	0.072	6	35			0.00	0.072	0.1	29.5
7	0.864	1745.9	3159.0	5.44E+06	0.182	5033	5.20E-04	7.18E-05	7.21E-05	0.018	0.000	0.000	7	104			0.00	0.000	0.0	34.5

Total, inches 0.288

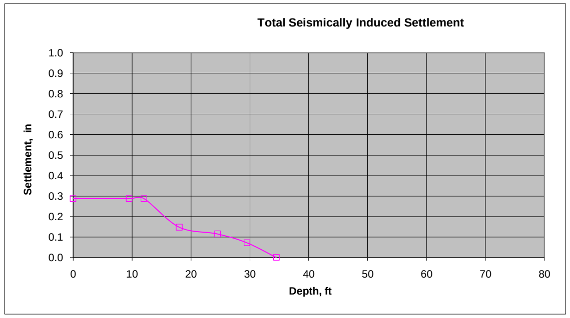


PLATE C-3

1/3/2011
Proj. No 1003.03

Moorpark College Parking Structure
Blowcount Corrections and Calculation of Seismically Induced Settlement
Ref: Pradel (1998)

pga,g 0.64
Mag. 7.5

B-10

Borehole Dia. In. 8

	Ef
Rope and cathead	60
Auto-trip hammer	90
Ratio	1.5

Corrections of Blowcount Data   po, psf 2116.8

Ncal/Nspt 1.60

Layer No.	Depth to Layer Bot. ft	Layer Thick. ft	Unit Weight pcf	sigma @ Bot. Lyr. psf	Depth @ Mid. Pt. ft	Sigma @ Mid. Pt. psf	Cn	Energy Corr.	Borehole Dia. Corr.	Liner Corr.	Rod Corr.	N-cal-fld bls/ft	N-spt-fld bls/ft	Est. fld. N-spt bls/ft	Est. N60 bls/ft	N1/60 bls/ft	Material	Fines %	Clay %	Correction for Triggering Analysis del N-fc	N-eff bls/ft	Correction for Residual Strength del N-fc	N-eff bls/ft	Layer
1	12	12	125	1500.0	6.0	750.0	1.68	1.5	1	1.0	0.75	50		31	35	59		70	20		59		59	1
2	14.5	2.5	125	1812.5	13.3	1656.3	1.13	1.5	1	1.0	0.75	29		18	20	23		70	20		23		23	2
3	19.5	5	125	2437.5	17.0	2125.0	1.00	1.5	1	1.2	0.82		30	30	45	45		18	10		45		45	3
4	24.5	5	125	3062.5	22.0	2750.0	0.88	1.5	1	1.2	0.96	42		26	46	40		50	10		40		40	4
5	29.5	5	125	3687.5	27.0	3375.0	0.79	1.5	1	1.0	1.00		32	32	48	38		25	5		38		38	5
6	34.5	5	125	4312.5	32.0	4000.0	0.73	1.5	1	1.2	1.00	120		75	136	99		5	0		99		99	6
7	39.5	5	125	4937.5	37.0	4625.0	0.68	1.5	1	1.2	1.00	120		75	136	92		80	15		92		92	7

Seismically Induced Settlement
Ko 0.475

zo, ft 100   po, psf 2116.8   Nc 15.2   1.005   Liquefaction-Related Settlement

Layer No.	rd	Avg. Shear Stress psf	sigma-mean psf	Gmax psf	a	b	Gamma	strain-15	strain-Nc	del S in	Interpr. del S in	Cum Dry sett. in	Layer	N1/60eff bls/ft	liq-strain %	Inc. sett. in	Cum. Liq. Sett. in	Inc. Sett. in	Cum. S in	Depth ft
1	0.996	310.9	487.5	1.77E+06	0.133	15446	4.67E-04	1.27E-04	1.28E-04	0.037	0.000	0.159	1	59			0.00	0.000	0.2	0.0
2	0.983	677.1	1076.6	1.92E+06	0.144	9602	1.62E-03	1.36E-03	1.37E-03	0.082	0.000	0.159	2	23			0.00	0.000	0.2	12.0
3	0.972	859.2	1381.3	2.71E+06	0.149	8268	8.43E-04	3.22E-04	3.24E-04	0.039	0.039	0.159	3	45			0.00	0.039	0.2	14.5
4	0.954	1091.2	1787.5	2.98E+06	0.157	7083	9.84E-04	4.27E-04	4.29E-04	0.051	0.051	0.120	4	40			0.00	0.051	0.1	19.5
5	0.932	1308.6	2193.8	3.24E+06	0.164	6264	1.06E-03	4.92E-04	4.94E-04	0.059	0.059	0.069	5	38			0.00	0.059	0.1	24.5
6	0.907	1509.4	2600.0	4.85E+06	0.172	5657	5.31E-04	7.78E-05	7.82E-05	0.009	0.009	0.009	6	99			0.00	0.009	0.0	29.5
7	0.880	1692.3	3006.3	5.09E+06	0.179	5185	5.65E-04	9.04E-05	9.08E-05	0.011	0.000	0.000	7	92			0.00	0.000	0.0	34.5

Total, inches 0.159

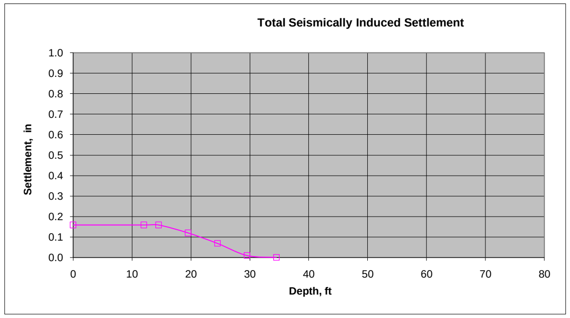


PLATE C-4