Initial Study/ Mitigated Negative Declaration

Appendix B Geotechnical Design Report

Milton Street Park Project

Prepared for

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July 2012

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TO	Ms. Ana Petrlic	DATE	7/22/10
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	Los Angeles, CA 90065	JOB NO.	51-1-10014-001
SUBJECT	**MILTON STREET PARK GEOTECHNICAL REPORT**		

THE FOLLOWING ITEMS ARE TRANSMITTED:

DATE	NO. COPIES	DESCRIPTION
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☒ Per your request ☐ For your approval ☐ For your information ☐ For your files
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Comments: Ana: Enclosed are six copies of the geotechnical Report for Milton Street Park project. Please let me know if you have any questions or comments. Feel free to call me anytime at 8181-539-8400 or 8181-237-6604. Thank you again for the work.

Dean Francuch

By: Dean Francuch c:

Title: Senior Principal Geologist

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MILTON STREET PARK
MOUNTAINS RECREATION AND CONSERVATION AUTHORITY
DEL REY DISTRICT, LOS ANGELES, CALIFORNIA

1.0 INTRODUCTION

1.1 Summary

This report presents the results of our geotechnical design services for the proposed Milton Street Park project for Mountain Recreation & Conservation Authority (MRCA). The location of the site is shown on the Vicinity Map, Figure 1. The project includes a linear park, a viewing deck extending over the edge of the Ballona Creek drainage slope, bike path realignment, a potential parking area, storm drain line and associated treatment storage for surface water, and other parking lot improvements. The existing conditions are shown in the Site and Exploration Plan, Figure 2.

We explored the subsurface conditions by drilling five borings as described in Appendix A. Fill soils, up to about 12 feet thick, were encountered in the borings. The underlying native soil consists of alluvial deposits. Geotechnical laboratory testing, including an R Value and corrosion testing, is presented in Appendix B.

The liquefaction potential of the alluvial deposits were judged to be low. Based on our current understanding of the overlook platform, foundation support should be with a deep foundation system, likely drilled shafts. However, alternative deep foundation systems, such as helical piers or micropiles or other foundation types, could also be feasible once a specific design has been defined.

From field percolation testing at four of the five soil borings, we recommend an infiltration rate on the site of 25 feet per day. We have also provided recommendations for pavement design of the parking lot, assuming asphalt-concrete (AC) surfacing.

1.2 Project Description

The proposed project will consist of the following items:

- Construction of a linear park approximately 1.6 acres in area and roughly 1,150 feet long by 60 feet wide;

- Construction of a viewing deck extending over the edge of the Ballona Creek drainage slope;
- Bike path realignment;
- A potential parking area at the northeast end of the park;
- Storm drain line and associated treatment storage for surface water; and
- Various park amenities.

2.0 SCOPE OF SERVICES

Our services were authorized by Ms. Ana Petrlic of MRCA, on June 4, 2010, in support for their design for the new facility. The purpose of our services is to determine the geotechnical conditions beneath the site and to provide data for design of foundations, paving and grading. Our services consisted of the following main tasks:

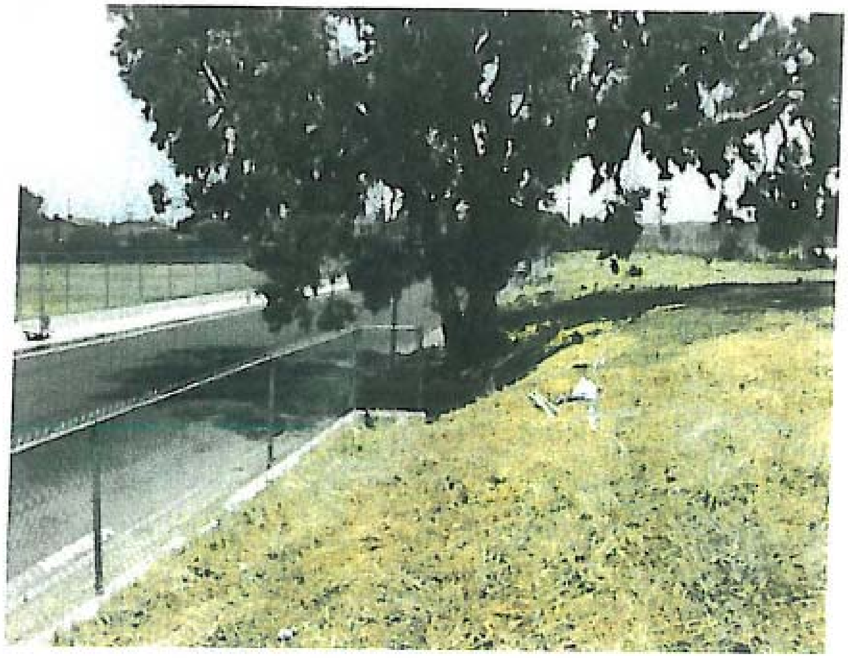
- Subsurface explorations to determine the nature and stratigraphy of the subsurface soils and to obtain soil samples for laboratory testing.
- Laboratory testing of soil samples to determine the static physical soil properties.
- Engineering evaluation of the geotechnical data to develop recommendations for design of foundations, retaining walls, for floor slab support, and for earthwork for the proposed Milton Street Park.

A more complete description of our services is provided in our proposal dated March 25, 2010.

3.0 SITE CONDITIONS

The proposed park is on an existing levee on the northwest side of the Ballona Creek channel and is bounded by a high school soccer court and Milton Street to the northwest and the Ballona Creek bike path to the southeast. The Ballona Creek channel is concrete-lined to reduce the potential for erosion and has an approximate slope inclination of 2 horizontal to 1 vertical (2H:1V). The water level of Ballona Creek fluctuates with tidal conditions and seasonal rainfall, but is approximately 20 feet below the levee crest.

The levee crest is approximately 15 feet wide and accommodates the existing bike path. The elevation of the levee crest is between +24 and +25 feet above mean sea level (MSL). The slope down to Milton Street is approximately 7H:1V. Existing retaining walls of approximately 4 feet high were located along the land side of the levee. A chain-link fence is located near the slope toe above portions of the retaining walls. Ground surface elevation along Milton Street is approximately +16 feet above MSL. Vegetation on the landward slope of the levee consists of scattered trees and short grass as shown in Photograph 1.



Photograph 1 – Landward Side of Levee Slope

4.0 GEOLOGIC SETTING

4.1 Regional Geology

The site is located within the southwestern block of the Los Angeles basin. The southwestern block, which is roughly rectangular in shape, is bounded by the Santa Monica Mountains to the northwest, Long Beach to the southeast, the Palos Verdes Peninsula to the southwest and the Newport-Inglewood fault zone to the northeast (Yerkes and others, 1965). The northwest-southeast trending Newport-Inglewood fault zone forms the major tectonic structure in the area, and is responsible for uplift along the nearby Baldwin Hills. The southwestern block is underlain by basement rocks at depths generally ranging from approximately 5,000 to 14,000 feet, although outcrop near the Palos Verdes Peninsula. In the area of the project site, the near surface geologic units consist primarily of recent alluvium, with uplifted older Quaternary-age marine terraces to the northwest and Pleistocene-age marine sediments exposed in the Baldwin Hills to the northeast.

4.2 Geologic Units

4.2.1 General

The site is located in an alluvial area northwest and adjacent to Ballona Creek, which drains the western portion of the Los Angeles basin. The creek was channelized by the U.S. Army Corps of Engineers in 1935 to reduce flooding in the area. Creek channelization consisted of excavating and straightening the original creek meanders, placing the excavated material on the adjacent banks for levee protection, and lining the creek-side levee face with concrete to reduce erosion.

We reviewed geologic mapping of the site, which shows floodplain deposits (geologic symbol: Qya2), underlying the site (State of California, 1998). Geologic units underlying the site were correlated with mapped geologic symbols in the project vicinity as shown below and in the boring logs. These units are divided into separate and discrete deposits of differing engineering characteristics. These units are variable in composition and origin and are described in more detail in the following sections.

4.2.2 Artificial Fill (af)

Fill is mapped in the Marina Del Rey area and on the north side of Ballona Creek (State of California, 1998). Though not mapped at the site, fill was placed during construction of the levee and channelization of Ballona Creek. Fill is typically made up of a variety of soil types, but likely consists of dredged material from the creek channel. Organic material is sometimes present, and in typical fill of this age, the soil typically has low strength.

4.2.3 Floodplain Deposits (Qya2)

Younger alluvium is mapped in the Ballona Creek area, with floodplain deposits a subset of these deposits (State of California, 1998). This unit was deposited from flooding of the creek prior to channelization. This unit contains soft clay and silt deposits near the surface, with loose to medium dense, fine- to medium-grained sand below. Sand and gravel deposits are common below a depth of 40 to 50 feet. According to published reports, this unit has a high potential for liquefaction.

5.0 FIELD EXPLORATIONS

5.1 Soil Borings

The subsurface conditions at the site were explored with five soil borings, designated B-1 through B-5, to depths between 11 to 51.5 feet below the existing grades. The borings were drilled on June 21, 2010, using limited access track-mounted drilling equipment subcontracted to us. The soils encountered in the borings were logged by our staff engineer, who also obtained bulk samples for laboratory testing. At the completion of Borings B-1 through B-4, 4-inch-diameter casings were installed for percolation testing described below. Depths of Borings B-1 through B-4 were terminated at 10 feet above MSL as prescribed in the Request for Proposal prepared by Psomas and verified with Mr. Drew Beck of Psomas prior to drilling. Details of the explorations performed and the logs of the borings are presented in Appendix A. The location of the borings is shown on Figure 2.

5.2 Field Percolation Testing

We performed percolation tests in four borings (B-1 through B-4) on June 22, 2010. The percolation tests utilized a 10 to 14-foot-long, 4-inch-diameter Schedule 40 polyvinyl chloride pipe with a 5-foot screened section at the bottom. The test setup was filled with clean water and pre-soaked approximately for one day to saturate the surrounding soil. Each test was performed by filling each pipe with water to the top and measuring the drop in water level at various intervals to assess the rate of water infiltration into the soil. The results of the percolation tests are presented in Figure 3. Recommendations for infiltration are described in Section 9.6 of this report.

6.0 LABORATORY TESTING

Laboratory tests were performed on selected samples obtained from the borings to aid in the classification of the soils and to determine the pertinent engineering properties of the foundation soils. The following tests were performed:

- Moisture content and dry density determinations
- Atterberg limits
- Sieve analyses
- Direct shear
- Stabilometer (R-Value)
- Soil corrosivity

All testing was performed in general accordance with applicable ASTM International (ASTM) specifications. Details of the laboratory testing program and test results are presented in Appendix B.

7.0 SUBSURFACE CONDITIONS

7.1 General

The following sections present brief description of the soil stratigraphy and groundwater encountered in the field explorations. The subsurface conditions encountered were consistent with the mapped geologic conditions described in Section 4.2.

7.2 Subsurface Stratigraphy

Fill soils, approximately 5 to 12 feet in thickness, were encountered at the surface in all borings. The artificial fill consists of very loose to medium dense, silty, slightly gravelly to gravelly, fine to medium sand and fine to medium sandy silt and does not appear to be well compacted.

Underlying the fill is floodplain deposits to the bottom of the borings. These deposits consist of interbedded layers of soft to stiff, silty, fine sandy clay and fine sandy silt and very loose to medium dense, silty, fine to medium sand. At Boring B-5, the floodplain deposits grades less clayey below about 15 feet. We observed dense, slightly silty, fine to medium sand at 48 feet below the ground surface to the bottom of boring B-5.

7.3 Groundwater

Groundwater was encountered in our deepest boring, Boring B-5, at a depth of approximately 21 feet below ground surface (Elevation +2 feet above MSL). Historic-high groundwater levels are mapped at about 5 to 10 feet below the ground surface, which is assumed to be the surface of Milton Street as approximately the elevation at +16 feet above MSL. Therefore, the historic-high groundwater elevation is approximately +11 to 6 feet MSL for the site. Groundwater levels will fluctuate in response to water levels in Ballona Creek, recent rainfall, and other factors.

8.0 GEOLOGIC HAZARDS

8.1 General

This section identifies potential geologic hazards at the site, the significant adverse impacts of the geologic hazards, and recommended measures to mitigate adverse impacts. This discussion includes the impact of the hazards of landsliding, flooding and subsidence to meet the requirements of the City of Los Angeles, Department of Building and Safety. The primary geotechnical issue from these hazards is seismic ground shaking.

8.2 Seismic Hazards

8.2.1 Surface Fault Rupture

The numerous faults in southern California include active, potentially active, and inactive faults. Classification for these major groups is based upon criteria developed by the California Division of Mines and Geology (CDMG, now known as the California Geologic Survey) for the Alquist-Priolo (AP) Zone Act program. By definition, an active fault has ruptured within Holocene geologic time (about the last 11,000 years). Known active faults do not underlie the site and surface rupture from fault plane displacement propagating to the surface is, therefore, considered remote. The closest established AP Zone to the site is the Newport-Inglewood Fault System, which is located approximately 3 miles to the east-northeast.

8.2.2 Ground Shaking

The site will likely experience seismic ground shaking similar to other areas in the seismically active Los Angeles Area. Earthquakes along several active faults in the region can cause moderate to strong ground shaking at the site. The intensity of earthquake motion will depend on the characteristics of the generating fault, distance to the earthquake fault, earthquake magnitude, earthquake duration, and site-specific geologic conditions.

Ground motions at the site, in the form of peak ground acceleration (PGA), were estimated from probabilistic seismic hazard analyses (PSHA) performed by the U.S. Geological Survey (USGS) (Petersen, 2008). The PSHA is a method for estimating ground motions that takes into account uncertainties and randomness in potential earthquake source, size, location, recurrence, and source-to-site attenuation. Results of the PSHA for the site indicate that a design PGA of 0.35 g (g = acceleration of gravity) has a 10 percent chance of being exceeded in 50 years (475-year return period), and a design PGA of 0.63 g has a 2 percent chance of being exceeded in 50 years (2,475-year return period). The most likely sources for these ground motions are the Hollywood Fault Zone to the north, the Palos Verdes Fault Zone offshore to the west, and the Newport-Inglewood Fault Zone to the east. The ground motions are based on bedrock conditions. Ground motions may be amplified or attenuated for the softer fill and alluvial/estuarine deposits at the site depending on the level of ground shaking on the underlying bedrock, underlying soil type, depth to bedrock, and other factors.

To mitigate the potential for future damage from strong seismic events, we recommend that the new structure be analyzed using seismic designs from the latest building codes. We understand seismic design of the new building will be in accordance with the 2008 City of Los Angeles Building Code (LABC – International Code Council, 2008). Relevant geotechnical parameters for the 2008 LABC are provided in Section 9.2.

8.2.3 Liquefaction

Soil liquefaction is a phenomenon that occurs during seismic loading in loose, saturated, cohesionless soils. During this phenomenon, the pore pressure of the soil increases while the initial effective stress decreases. When the two approach equal states, the result is a reduction in shear strength of the soil. This resulting reduction in strength can further lead to ground settlement and lateral spreading.

Underlying the site, there is a slight potential for liquefaction in the loose to medium dense, slightly clayey, silty sand and below the groundwater table. Liquefaction analyses were performed using the soil properties encountered in the explorations for the design PGA previously described in the "Ground Shaking" section, as plots of factor of safety (FS) against liquefaction versus depth. For the design PGA, we used two-thirds of the 2,475-year return period in accordance with the 2008 LABC. The design PGA was corrected for site conditions (Site Class D described in Section 9.2), which results in a PGA of 0.27 g.

The liquefaction potential was evaluated using blow counts derived from the Standard Penetration Test shown in boring B-5. The blow counts were corrected for depth, equipment variations, and fines content (percent passing the No. 200 sieve) where applicable. Using the corrected blow counts, an estimation of the liquefaction potential can be analyzed with correlations between the blow counts and the Cyclic Resistance Ratio (CRR). The CRR indicates the shaking threshold for liquefaction. The CRR is compared with the Cyclic Stress Ratio (CSR), which is the stress induced by the design earthquake described previously. The ratio of the CRR to CSR gives the FS for liquefaction potential. An FS below 1.0 indicates that liquefaction is probable for the design earthquake (Youd and Idriss, 2001).

Based on our analyses, the lowest FS is about 1.4 at approximately 35 feet below the ground surface at Boring B-5. It is our opinion that the potential for liquefaction to adversely affect the proposed structure and improvements is low.

8.2.4 Seismically-Induced Ground Settlement

Ground shaking could induce settlement of loose granular soils above the water table. We used a simplified procedure to review the potential for this settlement during ground shaking from the design PGA (Tokimatsu and Seed, 1987). Based on this method, the potential for settlement in the fill is low during design seismic events.

8.2.5 Tsunami

Tsunamis are short-duration, earthquake-generated water waves. The extent and severity of a tsunami generated within the Pacific Ocean basin depends on location of the earthquake event, ground motions, underwater landslides, and fault offset. A tsunami wave at the site generated from near-source events or in the Pacific basin can be large, also depending highly on the geometry of the adjacent shoreline.

We reviewed the "Tsunami Inundation Map for Emergency Planning" prepared by the CGS (State of California, 2009). The map shows the potential for inundation to reach the western edge of the project site, gradually dropping in elevation as the run-up dissipates in the Ballona Creek channel. The tsunami runup would likely manifest itself on the site as a rapid increase in the channel water level, but is not anticipated to overtop the levee.

8.3 Slope Stability

From our reconnaissance and field explorations, we did not observe significant slope instability along the levee on both sides of the crest. Based on the lack of field distress, levee slope geometry, and low potential for liquefaction, it is our opinion that slope instability at the site under static and seismic conditions is low.

8.4 Flooding and Inundation

The site is located adjacent to Ballona Creek and the channel is noted as being a "100 year flood area" by the County of Los Angeles flood zone. Review of the Federal Emergency Management Act (FEMA) Flood Insurance Rate Maps (FIRM) maps depicts that the channel up to the levee crest is located in a "Special Flood Hazard Area, Zone A" which is subject to inundation by the 1 percent annual Chance Flood (100 Year Flood). The Zone A designation indicates that the base flood elevation has not been determined. On the land side of the levee crest, the site is located within an area of "Zone X," indicating an area in which flood hazards are determined to be less than 0.2 percent annual Chance Flood.

8.5 Erosion

Typically, sandy soils on steep slopes subject to high velocity water flow or non-vegetated areas are susceptible to erosion. The creek-side levee slope is lined with concrete to reduce the potential for erosion during high flow events in Ballona Creek. Therefore, the potential for significant erosion at the project site is low. However, cracking or other damage to the concrete lining could result in increased erosion of the levee slope.

8.6 Expansive Soils

Expansive soil occurs when clay particles interact with water particles, causing volume changes in the clay soil. The clay soil may swell when saturated with water and contract when dried. This phenomenon generally decreases in magnitude with increasing confinement pressure at depth. These volume changes may damage lightly loaded foundations and shallow improvements.

The levee fill underlying the site consists has a low to moderate expansive potential based on the Atterberg Limits testing (Appendix B). Also, the fines content of the surficial soils indicate the clay soils contain a significant portion of sand. Based on this data, the sandy clay soils are considered to have low expansive potential.

8.7 Oil Wells

According to maps prepared by the State of California Department of Conservation, Division of Oil, Gas and Geothermal Resources, abandoned or active oil wells are not located within the subject site (State of California, 2010). The site is located about one mile east of the Playa Del Rey oil field.

8.8 Methane Gas

The site is located within the City of Los Angeles Methane Hazard Zone. Methane is likely associated with the nearby Playa Del Rey oil field. Methane is considered dangerous when trapped in a poorly ventilated crawl space, basement, or other below grade structure where gas could accumulate. There are no structures of this type planned for the site.

8.9 Subsidence

The site is not located within an area of known subsidence associated with fluid withdrawal (groundwater or petroleum) or peat oxidation. Regional subsidence is, therefore, not considered a significant impact to the proposed development.

8.10 Soil Corrosion

The corrosion studies indicate that the on-site soils are moderately to mildly corrosive to ferrous metals, and the sulfate attack to portland cement concrete is negligible. The report of soil corrosion study presented in Appendix B should be referred to for a discussion of the corrosion potential of the soils and for potential mitigation measures.

9.0 ENGINEERING RECOMMENDATIONS

9.1 General

Based on our site review, subsurface exploration, and engineering analyses, it is our opinion that the proposed park development is suitable for construction provided the recommendations in this report are incorporated into the design. Geologic hazards described in the previous section that will affect design of structures include seismic ground shaking. A summary of the various project aspects is provided below. Subsequent sections present more details of our recommendations. Preliminary considerations for construction are provided in Section 10.0 of this report.

Weak levee fill soils were encountered in all borings. The fill soils are not uniformly well compacted and are not suitable for foundation support of the viewing deck. The underlying floodplain deposits are medium dense to dense with significant fines content below the historic-high groundwater table resulting in a low potential for liquefaction.

The observation platform foundations, proposed on the creek-side slope of the levee, could be susceptible to erosion from scouring during high creek flows if the concrete lining is damaged or deteriorated. We did not observe damage to the concrete liner during our field reconnaissance, but we recommend the observation platform be founded on a deep foundation system, such as drilled shafts, to mitigate this potential and also to transfer deck loads to the medium dense floodplain deposits below the weaker fill.

Some minor retaining walls (less than 4 feet in height) are proposed for realignment of the bike path. If the site is properly graded in accordance with the recommendations of this report, these lightly-loaded short retaining walls could be supported on spread footings in the underlying levee fill.

9.2 Seismic Design

Parameters for the 2008 LABC are given in Table 1. For seismic design of structures using this code, mapped short-period and 1-second-period spectral accelerations, S_s and S_1, respectively, are required. S_s and S_1 are for a maximum considered earthquake, which corresponds to ground motions with a 2 percent probability of exceedance in 50 years (with a deterministic maximum cap in some regions). As previously discussed, the USGS completed PSHA for the entire country in November 1996, which were updated and republished in May 2008. The PSHA ground motion results can be obtained for the USGS website. The results of the updated USGS PSHA were referenced to determine S_s and S_1 for this site.

TABLE 1
CITY OF LOS ANGELES BUILDING CODE 2008
SEISMIC PARAMETERS FOR DESIGN OF NEW STRUCTURES
(SITE CLASS D)

Spectral Response Acceleration (SRA) and Site Coefficients	Short Period	1-Second Period
Mapped SRA	$S_s = 1.53$	$S_1 = 0.60$
Site Coefficients	$F_a = 1.00$	$F_v = 1.50$
Maximum Considered Earthquake SRA	$S_{MS} = 1.53$	$S_{M1} = 0.90$
Design SRA	$S_{DS} = 1.02$	$S_{D1} = 0.60$

9.3 Foundations

9.3.1 General

Based on our subsurface explorations and analyses, we conclude that a lightly loaded overlook platform could be supported on traditional foundation elements such as drilled cast-in-place concrete piles (drilled shafts). To mitigate the costs and reduce equipment associated with drilled shafts, we suggest an alternate deep foundation system described in the following sections. Options for an alternate deep foundation system include helical piers or micropiles. The advantages of these systems over a drilled shaft are that they are generally cost competitive and typically require smaller construction equipment and less material to be mobilized to the site. Shallow foundations, consisting of spread footings, are suitable to support the short retaining walls and other lightly loaded structures at the site.

The following sections present details for spread footings, drilled shafts, and alternative deep foundations. We have selected helical piers and micropiles as possible deep foundation alternatives to the shallow foundations and drilled shafts.

9.3.2 Spread Footings

Foundations for the short retaining walls (less than 4 feet in height) are assumed to be continuous spread footings. The retaining wall footings would likely bear on the relatively weak levee fill. Therefore, we recommend an allowable bearing capacity of 1,000 pounds per square foot (psf). The allowable values could be increased by one-third to account for wind and seismic loading conditions.

Resistance to lateral forces caused by seismic, unbalanced earth pressures, and/or other forces could be provided by both passive earth pressures acting against the embedded portion of foundations and frictional resistance against the base of foundations. We recommend a coefficient of friction of 0.25 be used between cast-in-place concrete and soil for calculating the resistance to sliding at the base of the footings. A FS of 1.5 is used to estimate the resistance to lateral movement.

Passive resistance should be ignored if a possibility exists that soil providing the resistance could be removed in the future. In our opinion, passive earth pressures in soil around the footing excavation could be estimated using an equivalent fluid weight of 150 pounds per cubic foot (pcf). These values include a FS of 1.5 to limit lateral movements.

9.3.3 Drilled Shafts

We recommend the drilled shafts be designed using a combination of the dead load and 100 percent of the live load. The loads will be resisted by the skin friction and end bearing of the drilled shaft. For design of a drilled shaft, we recommend applying an allowable, static skin friction to the shaft perimeter as shown in Table 2 below. We recommend a minimum shaft diameter of 12 inches. The allowable, static skin friction value for drilled shafts is obtained by applying a FS of 2.0 to the estimated ultimate value. For seismic loading, the FS could be reduced to 1.1 or greater. We recommend neglecting end bearing of the shafts for the design based on potential construction issues due to the high groundwater and loose to medium dense floodplain deposits. End bearing could be considered depending on the method of installation of the drilled shafts, such as using auger cast piles.

TABLE 2
DRILLED SHAFT AXIAL CAPACITY

Soil Layer	Depth Below Existing Grade	Allowable Skin Friction
Fine Sandy, Clayey SILT (ML - Fill)	0 to 8 feet	Ignore
Silty, Fine Gravelly Sand (SC/SM)	8 to 48 feet	400 psf
Slightly Silty Sand (SP-SM)	Below 48 feet	700 psf

Note:
psf = pounds per square foot

Permanent casing may be necessary if loose sand is encountered in the drilled shaft. The floodplain deposits could have layers of loose sand underlying the fill. If permanent casing is used, frictional resistance along the shafts will be reduced by about 25 percent. The reduction is due to the smoother surface of the steel casing against the soil as opposed to the rougher contact between the concrete and soil.

Based on the subsurface conditions encountered in the borings and the anticipated design loads, we estimate that total settlements for the drilled shafts would be on the order of ½-inch, with differential settlements of about ¼-inch. Since the deep foundations would be installed into the granular soils, these settlements would be primarily elastic and would occur rapidly as the load is applied.

Lateral capacity of the drilled shaft would depend on the lateral loading, drilled shaft diameter, and connection details. As these details are developed for the drilled shaft foundation design, we could provide geotechnical input parameters for lateral pile software (e.g., LPILE) or perform the lateral analyses during our supplemental consultation phase (Section 11.2).

9.3.4 Alternative Deep Foundations

Helical piers consist of a square steel shaft (1.5-inch-square is commonly used) with an 8- to 10-inch-diameter, steel helix located at the leading edge. The helix is a round steel plate formed into a ramped spiral attached to the shaft. The helix is similar to a drill auger, so the anchor penetrates and screws into the soil. The plate diameter and thickness is dependent on the load and supporting material. Axial capacity is transferred through the shaft to the helix.

Depending on the capacity required, one or more additional helices are located along the shaft at about 3-foot intervals. The smallest helix is the lowest one installed into the ground, with the sizes progressively becoming larger for a multiple-helix installation. The helical anchors are installed using a rotary-type torque motor, either electrically or hydraulically powered. After the installation and attachment to the footing, the bolts on the anchors should be tightened to take slack out of the system.

We recommend helical piers be spaced such that a minimum distance of three times the largest helix diameter is maintained between the helices of adjacent piers. Torque-monitoring equipment is also recommended for all installations to allow estimating of pier capacities. The helical pier should be screwed into the alluvium.

As an alternative to helical piers, micropiles (also referred to as mini-piles, pin piles, or pipe piles) are small-diameter piles (typically less than 12 inches) that can be pushed, driven, or drilled into the ground. Micropiles are steel pipes with a wall thickness typically less than ½ inch. Micropiles could also be injected with pressurized grout to increase capacity. The installation of the micropile and use of grout injection depends on the soil stratigraphy and load demands.

For this project, we recommend a driven micropile into the underlying alluvium. The micropiles are typically installed by pneumatic hammer or vibrations. The micropiles and equipment used for installation are portable and could be transported and set up at the site with minimal disturbance. Given the relatively light loads, we do not anticipate the need for pressurized grout with the micropiles.

We recommend a performance specification be developed for final design and bids solicited from specialty contractors qualified in helical pier installation. The contractor should be responsible for final design of the helical piers based on input from the structural engineer and us. Load testing of the helical piers should be part of the specifications to confirm capacity and to satisfy likely County requirements. Lateral capacity of the helical piers is typically small and should be analyzed for resistance to the earthquake loads.

If an alternative deep foundation system is under consideration, we could contact the governing agency (i.e., City of Los Angeles) about approval and use of an alternate deep foundation system. It is our experience that these foundation systems could require load testing, but could be approved by the City assuming proper design detailing and observation during installation by a geotechnical engineer.

9.4 Retaining Walls

We understand retaining walls are planned to accommodate the improvements on the levee, and will typically be less than 3 feet tall. These walls will be located in the levee fill and will retain the bike path and/or sloping ground surfaces behind the wall. Refer to "Spread Footings" section for design of retaining walls supported on spread footings.

For design of cantilevered retaining walls, where the surface of the backfill is level, it can be assumed that drained soils will exert a lateral pressure equal to that developed by a fluid with a density of 30 pcf. In addition to the recommended earth pressure, walls adjacent to streets or other areas subject to vehicular traffic should be designed to resist a uniform lateral pressure of 100 psf, acting as a result of an assumed 300 psf surcharge behind the walls due to normal vehicular traffic.

Retaining walls should be designed to resist hydrostatic pressures or be provided with a drain pipe or weepholes. The drain could consist of a 4-inch-diameter perforated pipe placed with perforations down at the base of the wall. The pipe should be sloped at least 2 inches in 100 feet and surrounded by filter gravel. The filter gravel should meet the requirements of Class 2 Permeable Material as defined in the current State of California, Department of Transportation, Standard Specifications.

If Class 2 Permeable Material is not available, ¾-inch crushed rock or gravel separated from the on-site soils by an appropriate filter fabric can be used. The crushed rock or gravel should have less than 5 percent passing a No. 200 sieve.

9.5 Pavements

In general, for new pavements, a proper subbase and subgrade will need to be prepared and proof-loaded. Pavement subgrade should be proof-rolled to identify any remaining soft or unsuitable soils. The proof-rolling operations should consist of several passes of a heavy (10-ton or heavier static weight) vibratory roller to compact the surface to a dense, unyielding condition. If loose and/or wet, spongy soil zones are identified by the proof-rolling process, the soils should be removed and replaced with compacted structural fill.

We used Caltrans design methodology to estimate pavement sections. The Caltrans design calculates a pavement section, composed of aggregate base (AB) and AC, from demand using a Traffic Index (TI) and resistance of the soil (R value). From laboratory testing of a bulk sample collected at the site, we used an R value of 10 (Appendix B).

The required paving and base thicknesses will depend on the expected wheel loads and volume of traffic (Traffic Index or TI). Assuming that the paving subgrade will consist of the on-site or comparable soils compacted as recommended in the "Structural Fill" section to at least 90 percent relative compaction per ASTM D-1557, the minimum recommended paving thicknesses are presented in Table 3.

TABLE 3
RECOMMENDED PAVING THICKNESSES

Traffic Use	Traffic Index	Asphalt Concrete (inches)	Caltrans Class 2 AB (inches)
Parking Stalls	4	3.0	6
	5	3.0	8
Driveways	6	3.5	12

Notes: R Value = 10 (laboratory testing)
AB = aggregate base

We can estimate the recommended AC and AB thicknesses for other TI values, if required. The AB should conform to requirements of Class 2 as described in Section 26 of State of California Department of Transportation Standard Specifications, latest edition. The base course should be compacted to at least 95 percent relative compaction (ASTM D 1557).

9.6 Infiltration

Based on the result of the percolation testing (Figure 3) at borings B-1 to B-4, we measured percolation rates ranging from 7 to 32 feet per day after four hours at all locations, which is reasonable given the materials encountered in the borings. Using a median value of 14 feet per day and applying a factor of safety of 2, we recommend an allowable rate of 7 feet per day. Therefore, we recommend this rate be used to design the stormwater discharge system. We also recommend that the discharge system include a controlled overflow system to allow runoff water to drain to a suitable location during periods of intense rainfall.

10.0 CONSTRUCTION CONSIDERATIONS

10.1 General

The applicability of the design parameters recommended in Section 9.0 depends on quality construction practices. The following sections present general recommendations that should be considered.

10.2 Site Preparation

After the site is cleared for the bike path and observation platform, the exposed levee fill should be carefully observed for the removal of all unsuitable surficial deposits, including organic debris, concrete and other hard debris larger than 4 inches in diameter. We also recommend that any existing fill soils observed within the paving area of the proposed parking lot be excavated to a depth of 2 feet. Next, the exposed soils should be scarified to a depth of 6 inches, brought to near-optimum moisture content and rolled with heavy compaction equipment. At least the upper 6 inches of the exposed soils should be compacted to at least 90 percent of the maximum dry density obtainable by the ASTM Designation: D 1557 method of compaction (ASTM, 2009).

10.3 Excavations and Temporary Slopes

Temporary excavation slopes required to construct the retaining walls and install underground utilities should be the responsibility of the Contractor. The Contractor is present at the site continuously and is best able to observe changes in site and soil conditions and to monitor the performance of excavations. All temporary slopes, shoring, and temporary walls should conform to applicable local, state, and federal safety regulations.

For preliminary cost-estimating purposes only, the recommended temporary, unsupported excavation slopes are 1H:1V for slope heights greater than 4 feet. Steeper slopes may be achievable depending on site conditions and construction time. Flatter slopes or slope protection could be required where seepage is present or during wet weather conditions. Plastic sheeting could be necessary to protect the slopes from erosion and raveling in wet weather. It should be expected that the cut face could experience some sloughing and raveling.

For fill embankments constructed using the requirements for structural fill placement and compaction outlined in the "Structural Fill" section below, we recommend that permanent side slopes of no steeper than 2H:1V be used. Fill should be carefully compacted on the slope face in a series of horizontal benches, or the fill embankment could be overbuilt and cut back to a 2H:1V configuration.

10.4 Structural Fill

All fill soil placed beneath pavements, walkways, or areas where settlements are to be minimized should be structural fill. Backfill behind walls should also be structural fill, although a lesser degree of compaction could be allowed if settlements are not of concern. Common fill could be placed in landscaped areas provided it is properly moisture conditioned.

Structural fill should consist of a well-graded mixture of on-site or imported granular soil that is free of organics, contaminants, debris, and rock fragments larger than 4 inches. The suitability of soil for use as structural fill would depend on its gradation and moisture content. As the amount of fines (portion of soil particles passing a U.S. Standard No. 200 sieve, based on the minus ¾-inch fraction) increases, soil becomes more sensitive to small changes in moisture content, and adequate compaction becomes more difficult to achieve. Structural fill placed during wet weather or on wet subgrade soils should contain no more than 5 percent fines. During dry weather, the fines content may be higher, provided the fill is at suitable moisture content, or could be moisture conditioned and compacted to the specified degree. The fines should be non-plastic, and the moisture content of the soil should be within ±2 percent of the optimum moisture content as determined by ASTM D 1557. Additional information on wet weather construction is described in Section 10.5.

10.4.1 Placement

Prior to the placement of structural fill, all foundation, pavement, and walkway areas should be stripped of unsuitable soils, and any remaining soil containing organic matter or debris, or soil disturbed by the contractor's operations should be removed. Structural fill should be placed in uniform lifts and compacted to a dense and unyielding condition. In accordance with the City of Los Angeles requirements for compaction of fills, where cohesionless soil having less than 15 percent finer than 0.005 millimeter (mm) is used for fill, the fill shall be compacted to at least 95 percent; if the soils have more than 15 percent finer than 0.005 mm, the fill shall be compacted to at least 90 percent. The on-site soils should be compacted to 90 percent; depending on the gradation, imported soils will probably require 95 percent compaction (ASTM D 1557). All fills should be placed in uniform, horizontal layers not exceeding 8 inches in loose thickness for heavy compactors or 4 inches for hand-operated mechanical compactors. The appropriate lift thickness will depend on the contractor's equipment and the moisture content and quality of the fill material.

If subgrade fill soils become loosened or disturbed, additional excavation to expose competent, undisturbed soils and replacement with properly compacted structural fill will be required. We recommend that a representative from our firm be present during structural fill placement to observe the work and perform in-place density tests to evaluate whether or not the specified compaction is being achieved.

10.4.2 Suitability of On-site Soils

The levee fill and native, on-site soils generally consist of silty sand and sandy silt. For the levee fill, cobbles and concrete debris larger than 4 inches in diameter should not be used in the fill. These soils may be moisture-sensitive, depending on the silt content and susceptible to disturbance by construction equipment during wet weather. Based on experience, the optimum moisture content of the native soil is in the range of 10 to 14 percent, depending on the silt content. The laboratory results indicate that most of the native soil has a moisture content near this range and, therefore, the specified compaction criteria will likely be achieved during dry weather. Isolated areas of wetter material may be encountered and will require extensive drying by aeration before it can be used. To expedite construction, this wetter material can be placed in landscaped areas or transported offsite.

10.4.3 Import Soil

We recommend that imported material used for structural fill consist of select, granular material. This material should consist of a well-graded sand and gravel with a maximum particle size smaller than 3 inches, at least 40 percent retained on the U.S. No. 4 sieve, and less than 5 percent passing the U.S. No. 200 sieve, based on that fraction passing the ¾-inch sieve. This material should conform to Section 19-3.06 (Type E Backfill) of the Caltrans Standard Specifications (Caltrans, 2006) with the exception of the grading requirements described above. We recommend that select import also be used during wet weather or placement on wet subgrades.

A higher fines content for import fill could be considered assuming earthwork occurs during periods of dry weather (see Section 10.6). We recommend that the fines content not exceed 30 percent. Import material should consist of relatively non-expansive soils with an expansion index of less than 35.

10.5 Wet Weather Conditions

In Southern California, it is advisable to schedule earthwork in dry weather conditions, which is typically April through October (with wet weather likely in January and February). Most of the soil that contains sufficient fines may become difficult or impossible to proof-roll and properly compact if the moisture content significantly exceeds the optimum. Performing earthwork during dry weather would reduce these problems and costs associated with rainwater, trafficability, and handling of wet soil. However, should wet weather/wet condition earthwork be unavoidable, the following recommendations are provided:

- The ground surface in and surrounding the construction area should be sloped as much as possible and sealed with a smooth-drum roller to promote runoff of precipitation away from work areas and to prevent ponding of water.
- Work areas or slopes should be covered with plastic. The use of sloping, ditching, sumps, dewatering, and other measures should be employed as necessary to permit proper completion of the work.
- Earthwork should be accomplished in small sections to minimize exposure to wet conditions. That is, each section should be small enough so that the removal of unsuitable soils and placement and compaction of clean structural fill can be accomplished on the same day. The size of construction equipment may have to be limited to prevent soil disturbance. It may be necessary to excavate soils with a backhoe, or equivalent, and locate them so that equipment does not pass over the excavated area. Thus, subgrade disturbance caused by equipment traffic will be significantly reduced.
- General fill material should consist of clean, well-graded, sand and gravel soils, of which not more than 5 percent fines by dry weight passes the No. 200 mesh sieve, based on wet-sieving the fraction passing the ¾-inch mesh sieve, in case wet weather condition is expected. In-place soil or fill soil that becomes wet and unstable and/or too wet to suitably compact should be removed and replaced with clean, granular soil with approved gradation.
- No soil should be left uncompacted and exposed to moisture. A smooth-drum vibratory roller, or equivalent, should roll the surface to seal out as much water as possible.
- Excavation and placement of structural fill material should be observed on a full-time basis by Shannon & Wilson, Inc. to determine that all work is being accomplished in accordance with the project specifications and our recommendations.
- Grading and earthwork should not be accomplished during periods of heavy, continuous rainfall.

The above recommendations apply for all weather conditions, but are most important for wet-weather earthwork. They should be incorporated into the contract specifications for foundation and pavement construction.

11.0 ADDITIONAL GEOTECHNICAL SERVICES

11.1 General

This report concludes our geotechnical design services for the project. We recommend additional geotechnical services as described below be considered for final design and construction of the project. At your request, we would prepare separate proposals with a detailed scope of services for your review and authorization.

11.2 Supplemental Consultation and Plan Review

We will be available to discuss our recommendations with the project team. We can also provide recommendations for alternative foundation and pavement designs and assist in permitting issues, as requested. This could include final design of alternative deep foundations for the overlook platform. As the improvement plans are completed, we should review the documents to confirm that the intent of our recommendations has been incorporated.

11.3 Construction Observation and Testing

The purpose of our construction observation and testing services will be to monitor compliance of the site grading, earthwork, and foundation installations with the project plans and specifications. This includes observing site preparation, placement and compaction of new fills, and preparation of retaining wall footings, deep foundation installations, and pavement subgrades. In particular, we should review subgrade conditions of the bike path and pavement areas to identify areas of very loose to loose, silty sand requiring overexcavation and replacement with compacted fill.

12.0 LIMITATIONS

This report was prepared for the exclusive use of MRCA and other members of the design team for specific application to this project. This report should be provided to prospective Contractors for information on factual data only and not as a warranty of subsurface conditions, such as those interpreted from the exploration logs and discussions of subsurface conditions included in this report.

The analyses, conclusions, and recommendations contained in this report are based on site conditions as they presently exist. We assume that the exploratory borings made for this project are representative of the subsurface conditions throughout the project alignment (i.e., the subsurface conditions everywhere are not significantly different from those disclosed by the explorations). If conditions different from those described in this report are observed or appear to be present during construction, we should be advised at once so that we can review these conditions and reconsider our recommendations, where necessary. If there is a substantial lapse of time between submission of our report and the start of work at the site, or if conditions have changed because of natural forces or construction operations at or near the site, it is recommended that this report be reviewed to determine the applicability of the conclusions and recommendations considering the changed conditions and time lapse.

Within the limitations of the scope, schedule, and budget, the analyses, conclusions, and recommendations presented in this report were prepared in accordance with generally accepted professional geotechnical engineering principles and practices in this area at the time this report was prepared. We make no other warranty, either express or implied. These conclusions and recommendations were based on our understanding of the project as described in this report and the site conditions as interpreted from the current explorations.

Unanticipated soil conditions are commonly encountered and cannot be fully determined by merely taking soil samples or completing test borings. Such unexpected conditions frequently require that additional expenditures be made to attain a properly constructed project. Therefore, some contingency fund is recommended to accommodate such potential extra costs.

The scope of our geotechnical services did not include any environmental assessment or evaluation regarding the presence or absence of wetlands or hazardous or toxic materials in the soil, surface water, groundwater, or air at the subject site. Shannon & Wilson, Inc. can provide these services at your request.

Shannon & Wilson, Inc. has prepared the document, "Important Information About Your Geotechnical Report," in Appendix C to assist you and others in understanding the use and limitations of this report.

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13.0 REFERENCES

ASTM International, 2009, Annual book of standards, construction, v. 4.08, soil and rock (I): D 420 - D 5876: West Conshohocken, Penn., ASTM International, 1 v.

Caltrans, 2006, Standard Specifications, State of California Department of Transportation, July.

International Code Council, Inc., 2008, 2008 City of Los Angeles Building Code (based on the 2007 CBC and 2006 IBC), January.

Petersen, M.D.; Frankel, A.D.; Harmsen, S.C.; and others, 2008, Documentation for the 2008 update of the national seismic hazard maps: U.S. Geological Survey Open-File Report 08-118, available: http://pubs.usgs.gov/of/2008/1128.

State of California, 2010, Oil and gas maps: Department of Conservation, Division of Oil, Gas, and Geothermal Resources: http://www.conservation.ca.gov/dog/maps/Pages/index_map.aspx.

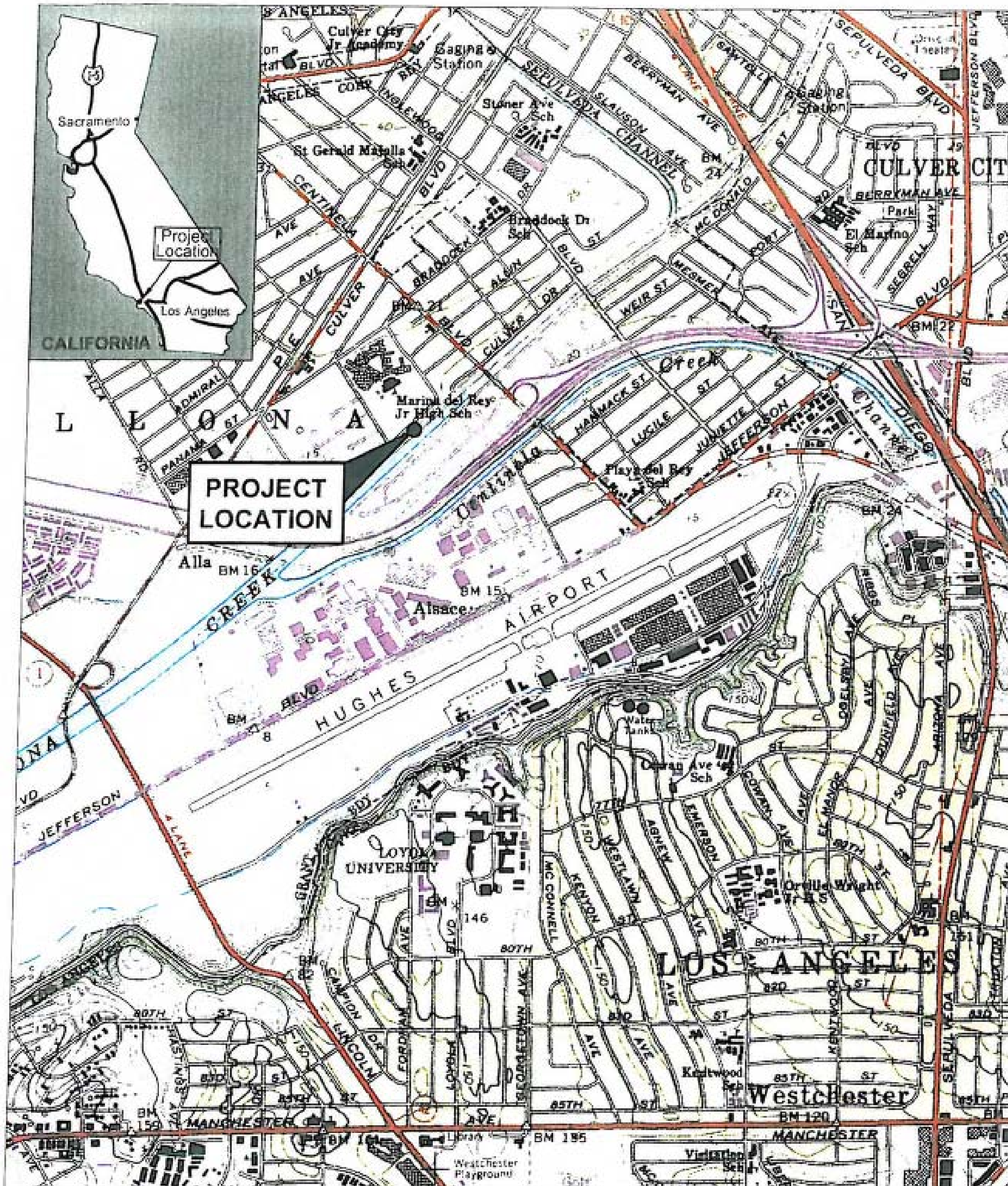
State of California, 2009, Tsunami inundation map for emergency planning, Venice Quadrangle, Los Angeles County: Produced by California Emergency Management Agency, California Geological Survey, and University of Southern California – Tsunami Research Center; dated March 1, 2009, mapped at 1:24,000 scale.

State of California, 1998, Seismic hazard zone report for the Venice 7.5-Minute Quadrangle, Los Angeles County, California: Department of Conservation, Division of Mines and Geology, Seismic Hazard Zone Report 036.

Tokimatsu, Kohji and Seed, H.B., 1987, Evaluation of Settlements in Sands Due to Earthquake Shaking: Journal of Geotechnical Engineering, v. 113, no.8, p. 861-878.

Yerkes, R.F., McCulloh, T.H., Schoellhamer, J.E. and Vedder, J.G., 1965, Geology of the Los Angeles Basin, California-An Introduction: in Geology of the Eastern Los Angeles Basin Southern California; United States Geological Survey Professional Paper 420-A.

Youd, T.L., and Idriss, I.M., 2001, Liquefaction resistance of soils: summary report from the 1996 NCEER and 1998 NCEER/NSF workshops on evaluation of liquefaction resistance of soils: Journal of Geotechnical and Geoenvironmental Engineering, v. 127, no. 4, p. 297-313.



0 1000 2000 4000

Scale: 1 inch=2,000 feet

NOTE

Map adapted from 1:24,000 USGS topographic map of Venice, CA quadrangle, dated 1998.

Milton Street Park MRCA Del Rey District, Los Angeles, California	
VICINITY MAP	
July 2010	51-1-10014-001
SHANNON & WILSON, INC. GEOTECHNICAL AND ENVIRONMENTAL CONSULTANTS	**FIG. 1**

Filename: I:\PROJECTS\10014 Milton St Park\Graphics\Vicinity Map.dwg Date: 07-20-2010 Login: Louis Larios

PERCOLATION TEST
BORING B-1

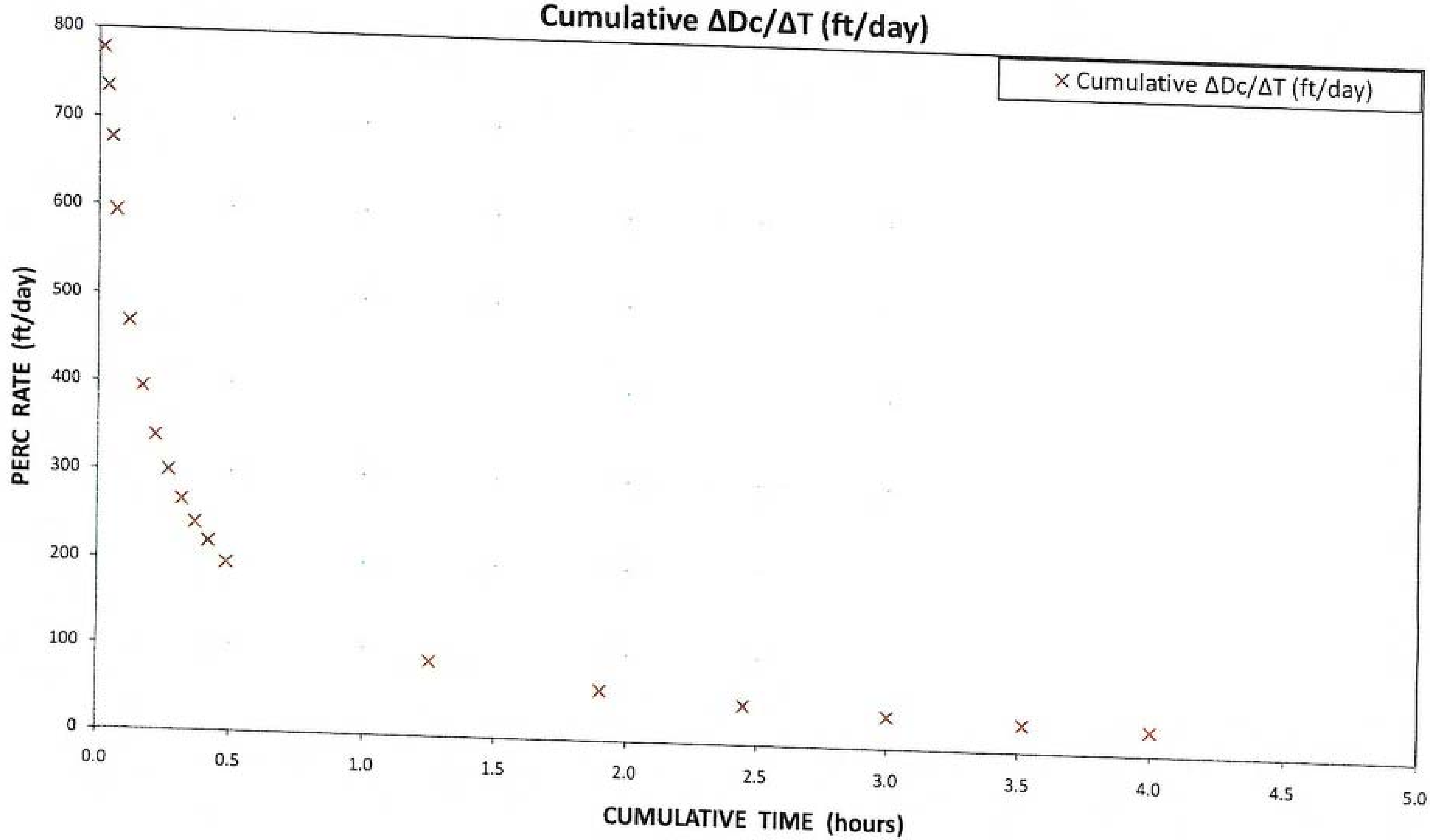


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FIG. 3
Sheet 1 of 4

PERCOLATION TEST
BORING B-2

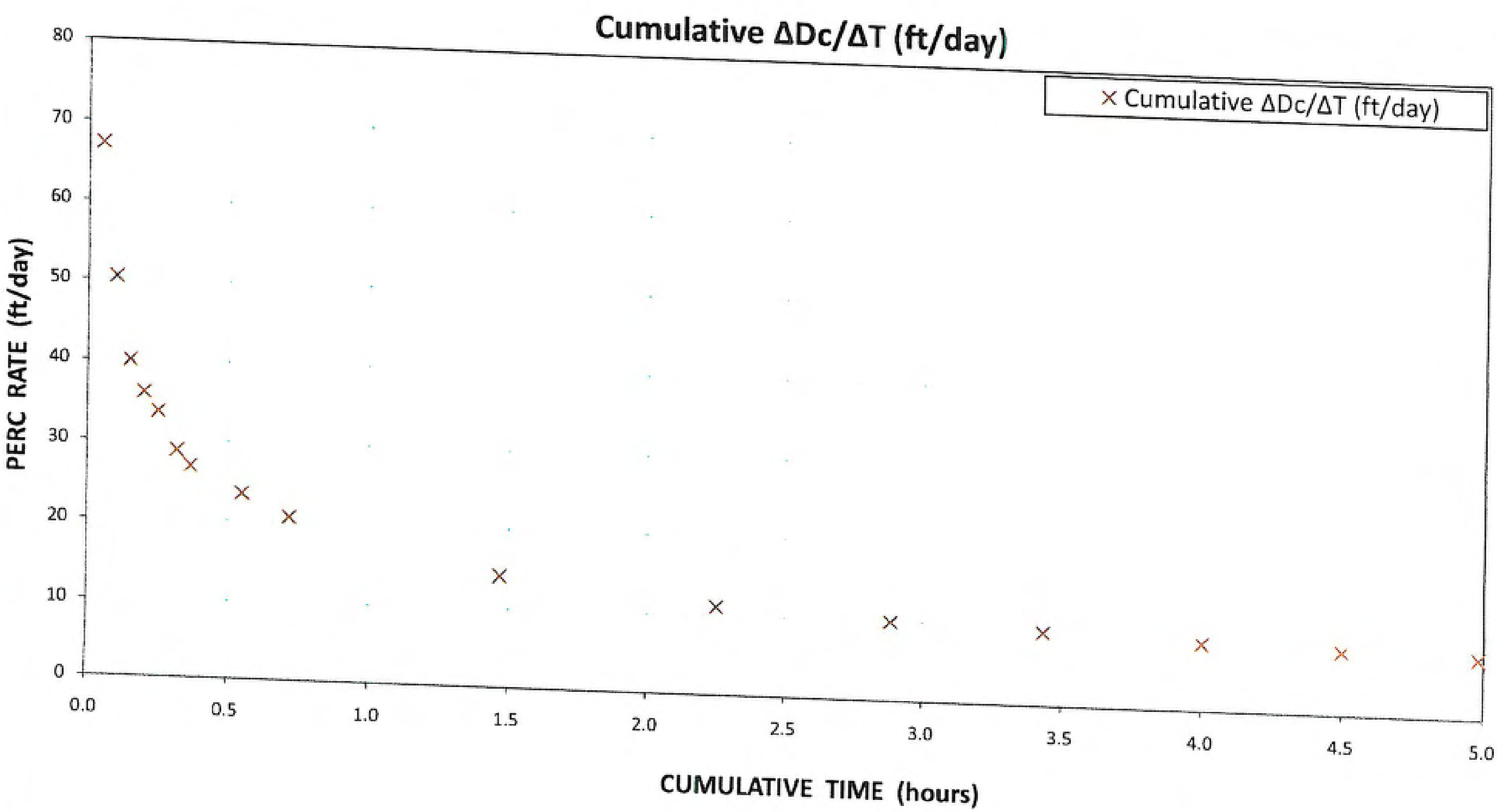

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FIG. 3
Sheet 2 of 4

PERCOLATION TEST
BORING B-3

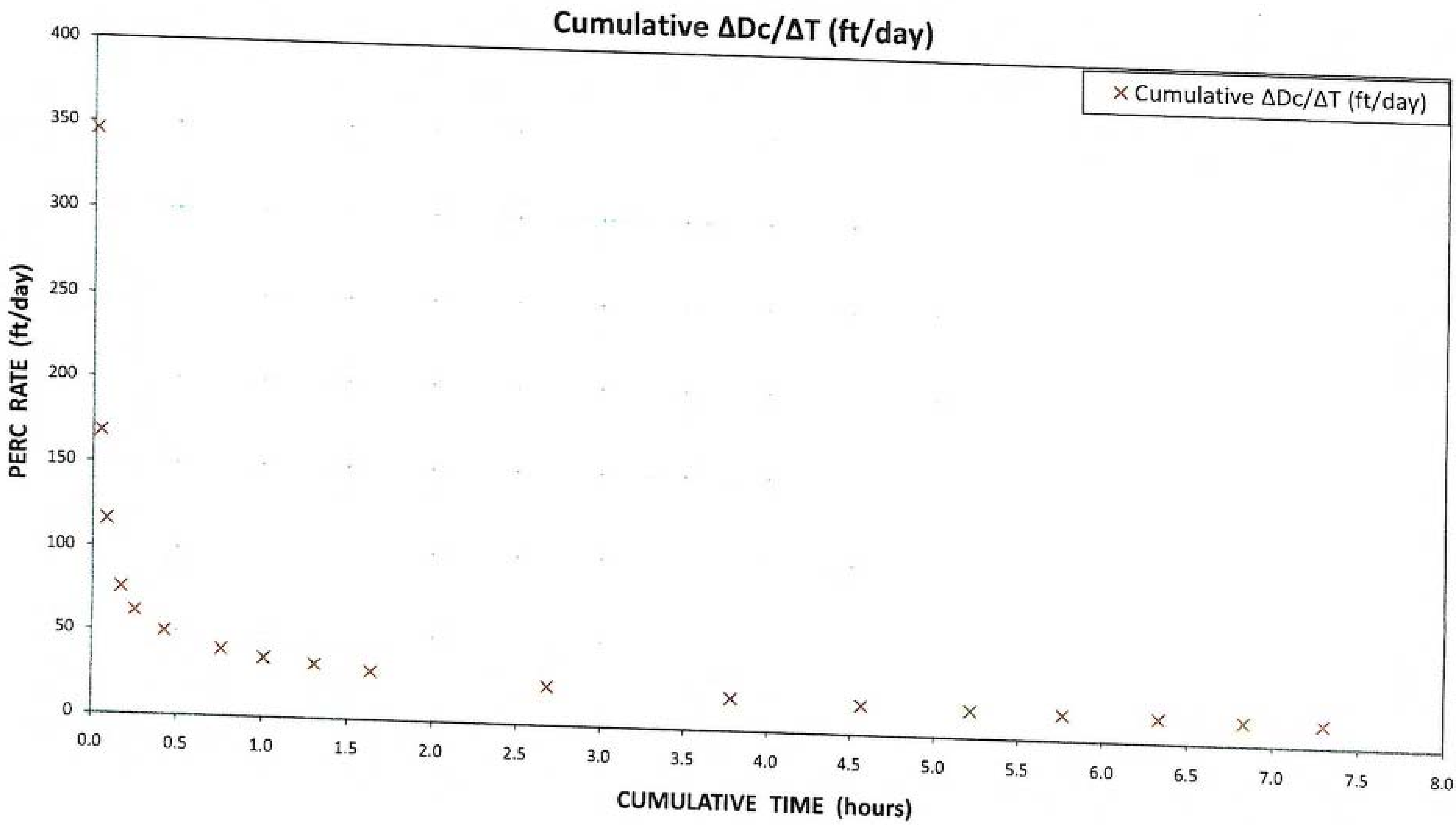

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FIG. 3
Sheet 3 of 4

PERCOLATION TEST
BORING B- 4

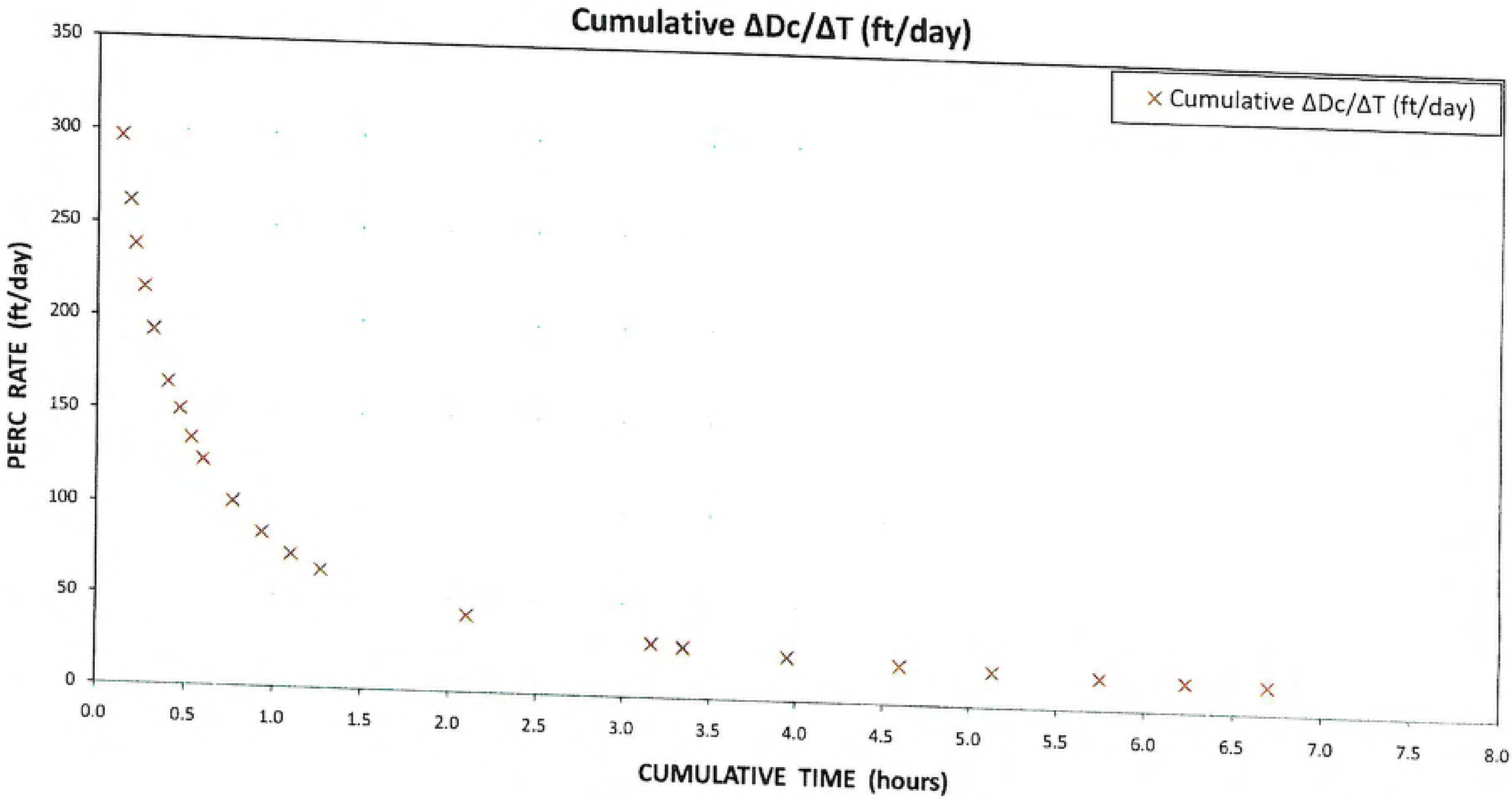

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FIG. 3
Sheet 4 of 4

APPENDIX A

SUBSURFACE EXPLORATIONS

APPENDIX A

SUBSURFACE EXPLORATIONS

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FIGURES

APPENDIX A

SUBSURFACE EXPLORATIONS

A.1 GENERAL

A portion of the field exploration program for the Milton Street Park project consisted of drilling and soil sampling of five borings (designated B-1 through B-5) and installing four wells at B-1 through B-4. The locations of the borings were determined based on the preliminary construction design and measured by taping and/or pacing from mapped features. The elevations of the borings were determined by the elevation contours on the base map shown in Figure 2. All the boring locations and elevations should be considered accurate to the degree implied by the method used.

A representative from Shannon & Wilson, Inc. was present throughout the field exploration period to observe the sampling operations, retrieve representative soil samples for laboratory testing, and prepare descriptive field logs for the explorations. Soils were classified in general accordance with ASTM International (ASTM) Designation: D 2488, Standard Recommended Practice for Description of Soils (Visual-Manual Procedure). Figure A-1 presents the key to our classification of the materials encountered. The exploration logs completed by Shannon & Wilson, Inc. are presented in Figures A-2 through A-6. Refer to Section 4.2 of the report for information on the geologic symbols used in the soil descriptions on the boring logs.

A.2 SOIL BORINGS

The subsurface explorations performed for this project consisted of drilling and sampling using a limited-access rig. The borings were advanced to depths ranging between 11 to 51.5 feet below the ground surface.

A.2.1 Drilling Procedures

The five borings were completed by 2R Drilling, Inc., California, under subcontract to Shannon and Wilson, Inc. All borings were drilled on June 22, 2010.

An 8-inch-diameter, continuous-flight auger technique was used to complete the borings. After completion of drilling and sampling, the driller sealed the boring where water was encountered using grout in boring B-5. Borings B-1 through B-4 were replaced by wells and presoaked it overnight to run a percolation test the next day.

A.2.2 Standard Penetration Test (SPT)

The SPT method was performed in general accordance with ASTM Designation: D 1586, Standard Method for Penetration Testing and Split-barrel Sampling of Soils. SPTs were generally performed at 2.5-foot intervals to 10 feet and at 5-foot intervals thereafter. The SPT consists of driving a 2-inch outside-diameter (O.D.) split-spoon sampler a distance of 18 inches into the bottom of the borehole with a 140-pound hammer falling 30 inches. The number of blows required for the last 12 inches of penetration is termed the Standard Penetration Resistance (N-value). When the resistance exceeded 50 blows for 6 inches or less penetration, the test was terminated and the number of blows and corresponding penetration were recorded. The value is an empirical parameter that provides a means for evaluating the relative density, or compactness, of granular soils and the consistency, or stiffness, of cohesive soils. The N-values are plotted on the boring logs.

The split-spoon sampler used during the penetration testing recovered a disturbed sample of the soil. The samples were field classified and recorded on the logs by our field representative, sealed in jars, and returned to our laboratory for testing.

A.2.3 Modified California Sampler (MCS)

The MCS is similar in concept to the SPT sampler. The MCS is driven 12 inches using a 140-pound hammer falling 30 inches similar to the SPT sampler. The MCS blow counts are recorded on the boring logs. The MCS blow counts are not used for analyses given the typically poor correlation with the SPT N-values. The MCS blow count should be used as a relative measurement of density or consistency with other MCS samples.

The MCS sampler barrel has a larger O.D. (3.25-inch) and is usually lined with 2.5-inch-diameter metal tubes or 1-inch-high rings to contain samples. Samples from the MCS are considered disturbed due to the large area ratio of the sampler. The MCS samples were field classified and recorded on the logs by our field representative, sealed with plastic end caps and/or plastic liners, and returned to our laboratory for testing.

A.2.4 Groundwater Observations

Where encountered during drilling, groundwater was observed at depth of approximately 21 feet below the ground surface. Groundwater measurements were generally taken upon completion of the boring prior to backfilling.

A.3 REFFERENCE

ASTM International (ASTM), 2006, Annual Book of Standards-Construction, v. 4.08, soil and rock, (I): D 420 – D 5611: West Conshohocken, Pa.

Shannon & Wilson, Inc. (S&W), uses a soil classification system modified from the Unified Soil Classification System (USCS). Elements of the USCS and other definitions are provided on this and the following page. Soil descriptions are based on visual-manual procedures (ASTM D 2488-93) unless otherwise noted.

S&W CLASSIFICATION OF SOIL CONSTITUENTS

- MAJOR constituents compose more than 50 percent, by weight, of the soil. Major consituents are capitalized (i.e., SAND).
- Minor constituents compose 12 to 50 percent of the soil and precede the major constituents (i.e., silty SAND). Minor constituents preceded by "slightly" compose 5 to 12 percent of the soil (i.e., slightly silty SAND).
- Trace constituents compose 0 to 5 percent of the soil (i.e., slightly silty SAND, trace of gravel).

MOISTURE CONTENT DEFINITIONS

Dry	Absence of moisture, dusty, dry to the touch
Moist	Damp but no visible water
Wet	Visible free water, from below water table

ABBREVIATIONS

ATD	At Time of Drilling
Elev.	Elevation
ft	feet
FeO	Iron Oxide
MgO	Magnesium Oxide
HSA	Hollow Stem Auger
ID	Inside Diameter
in	inches
lbs	pounds
Mon.	Monument cover
N	Blows for last two 6-inch increments
NA	Not applicable or not available
NP	Non plastic
OD	Outside diameter
OVA	Organic vapor analyzer
PID	Photo-ionization detector
ppm	parts per million
PVC	Polyvinyl Chloride
SS	Split spoon sampler
SPT	Standard penetration test
USC	Unified soil classification
WOH	Weight of hammer
WOR	Weight of drill rods
WLI	Water level indicator

GRAIN SIZE DEFINITION

DESCRIPTION	SIEVE NUMBER AND/OR SIZE
FINES	< #200 (0.08 mm)
SAND*	
- Fine	#200 to #40 (0.08 to 0.4 mm)
- Medium	#40 to #10 (0.4 to 2 mm)
- Coarse	#10 to #4 (2 to 5 mm)
GRAVEL*	
- Fine	#4 to 3/4 inch (5 to 19 mm)
- Coarse	3/4 to 3 inches (19 to 76 mm)
COBBLES	3 to 12 inches (76 to 305 mm)
BOULDERS	> 12 inches (305 mm)

* Unless otherwise noted, sand and gravel, when present, range from fine to coarse in grain size.

RELATIVE DENSITY / CONSISTENCY

COARSE-GRAINED SOILS		FINE-GRAINED SOILS	
N, SPT, BLOWS/FT.	RELATIVE DENSITY	N, SPT, BLOWS/FT.	RELATIVE CONSISTENCY
0 - 4	Very loose	Under 2	Very soft
4 - 10	Loose	2 - 4	Soft
10 - 30	Medium dense	4 - 8	Medium stiff
30 - 50	Dense	8 - 15	Stiff
Over 50	Very dense	15 - 30	Very stiff
		Over 30	Hard

WELL AND OTHER SYMBOLS

Bent. Cement Grout	Surface Cement Seal
Bentonite Grout	Asphalt or Cap
Bentonite Chips	Slough
Silica Sand	Bedrock
PVC Screen	
Vibrating Wire	

Milton Street Park
MRCA
Del Rey District, Los Angeles, California

SOIL CLASSIFICATION AND LOG KEY

July 2010 51-1-10014-001

SHANNON & WILSON, INC.
Geotechnical and Environmental Consultants

FIG. A-1
Sheet 1 of 2

UNIFIED SOIL CLASSIFICATION SYSTEM (USCS)
(From ASTM D 2487-98 & 2488-93)

MAJOR DIVISIONS			GROUP/GRAPHIC SYMBOL	TYPICAL DESCRIPTION
COARSE-GRAINED SOILS *(more than 50% retained on No. 200 sieve)*	Gravels *(more than 50% of coarse fraction retained on No. 4 sieve)*	Clean Gravels *(less than 5% fines)*	GW	Well-graded gravels, gravels, gravel/sand mixtures, little or no fines.
			GP	Poorly graded gravels, gravel-sand mixtures, little or no fines
		Gravels with Fines *(more than 12% fines)*	GM	Silty gravels, gravel-sand-silt mixtures
			GC	Clayey gravels, gravel-sand-clay mixtures
	Sands *(50% or more of coarse fraction passes the No. 4 sieve)*	Clean Sands *(less than 5% fines)*	SW	Well-graded sands, gravelly sands, little or no fines
			SP	Poorly graded sand, gravelly sands, little or no fines
		Sands with Fines *(more than 12% fines)*	SM	Silty sands, sand-silt mixtures
			SC	Clayey sands, sand-clay mixtures
FINE-GRAINED SOILS *(50% or more passes the No. 200 sieve)*	Silts and Clays *(liquid limit less than 50)*	Inorganic	ML	Inorganic silts of low to medium plasticity, rock flour, sandy silts, gravelly silts, or clayey silts with slight plasticity
			CL	Inorganic clays of low to medium plasticity, gravelly clays, sandy clays, silty clays, lean clays
		Organic	OL	Organic silts and organic silty clays of low plasticity
	Silts and Clays *(liquid limit 50 or more)*	Inorganic	MH	Inorganic silts, micaceous or diatomaceous fine sands or silty soils, elastic silt
			CH	Inorganic clays or medium to high plasticity, sandy fat clay, or gravelly fat clay
		Organic	OH	Organic clays of medium to high plasticity, organic silts
HIGHLY-ORGANIC SOILS	Primarily organic matter, dark in color, and organic odor		PT	Peat, humus, swamp soils with high organic content (see ASTM D 4427)

NOTE: No. 4 size = 5 mm; No. 200 size = 0.075 mm

<u>NOTES</u>

1. Dual symbols *(symbols separated by a hyphen, i.e., SP-SM, slightly silty fine SAND)* are used for soils with between 5% and 12% fines or when the liquid limit and plasticity index values plot in the CL-ML area of the plasticity chart.
2. Borderline symbols *(symbols separated by a slash, i.e., CL/ML, silty CLAY/clayey SILT; GW/SW, sandy GRAVEL/gravelly SAND)* indicate that the soil may fall into one of two possible basic groups.

Milton Street Park
MRCA
Del Rey District, Los Angeles, California

SOIL CLASSIFICATION AND LOG KEY

July 2010 — 51-1-10014-001

SHANNON & WILSON, INC.
Geotechnical and Environmental Consultants

FIG. A-1
Sheet 2 of 2

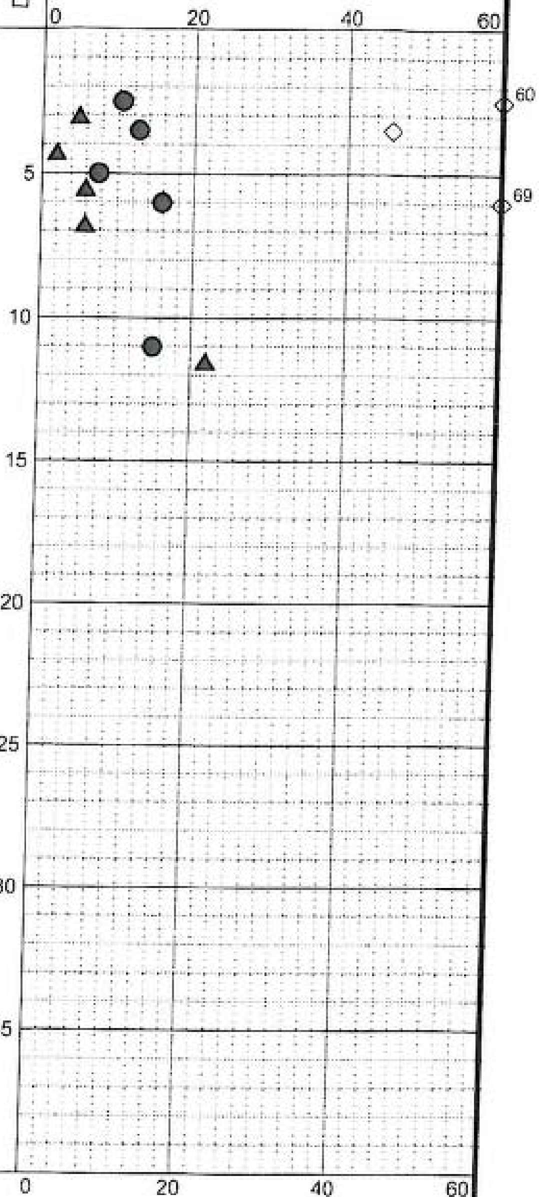
BORING_CLASS2 51-1-10014-001.GPJ SWNEW.GDT 7/8/10

Total Depth:	*12 ft.*	Northing:	*~*	Drilling Method:	*Hollow Stem Auger*	Hole Diam.:	*10 in.*
Top Elevation:	*~ 21 ft.*	Easting:	*~*	Drilling Company:	*2R Drilling*	Rod Diam.:	*AWG*
Vert. Datum:	*MSL*	Station:	*~ N/A ft.*	Drill Rig Equipment:	*CME 55*	Hammer Type:	*Automatic*
Horiz. Datum:	*N/A*	Offset:	*~ N/A*	Other Comments:			

SOIL DESCRIPTION *Refer to the report text for a proper understanding of the subsurface materials and drilling methods. The stratification lines indicated below represent the approximate boundaries between material types, and the transition may be gradual.*	Depth, ft.	Symbol	Samples	Ground Water
Very loose, mottled brown and yellow-brown, slightly fine gravelly, fine to medium sandy, SILT; dry to moist; (FILL) ML.			1	
Very loose, mottled yellow-brown and gray-brown, silty SAND, trace of gravel; dry to moist; rootlets; (FILL) SM.	3.5		2, 3, 4	Water Not Observed
Soft to stiff, dark gray and brown, fine sandy CLAY; moist to wet; slightly porous; (Qya2) CL.	6.0		5	
BOTTOM OF BORING COMPLETED 6/21/2010	12.0			

PENETRATION RESISTANCE (blows/foot)

▲ Hammer Wt. & Drop: *140 lbs/30 inches*



LEGEND

- * Sample Not Recovered
- Modified California Sampler
- Standard Penetration Test
- Piezometer Screen and Sand Filter
- Bentonite-Cement Grout
- Bentonite Chips/Pellets
- Bentonite Grout

NOTES

1. Refer to KEY for explanation of symbols, codes, abbreviations and definitions.
2. The stratification lines represent the approximate boundaries between soil types, and the transition may be gradual.
3. The discussion in the text of this report is necessary for a proper understanding of the nature of the subsurface materials.
4. Groundwater level, if indicated above, is for the date specified and may vary.
5. USCS designation is based on visual-manual classification and selected lab testing.
6. The hole location was measured from existing site features and should be considered approximate.

Milton Street Park
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LOG OF BORING B-3

July 2010 51-1-10014-001

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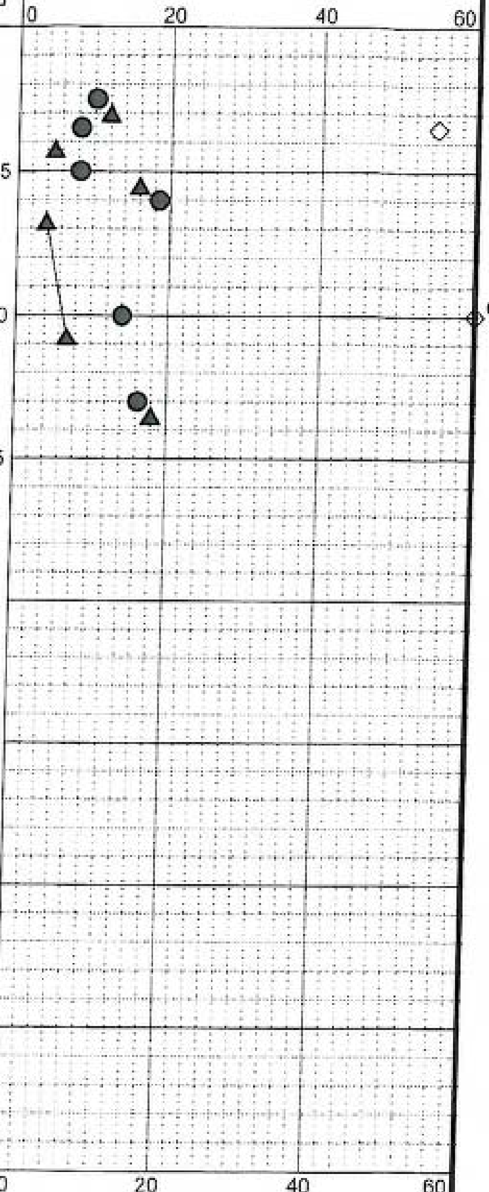
FIG. A-4

Total Depth:	14 ft.	Northing:	~	Drilling Method:	Hollow Stem Auger	Hole Diam.:	10 in.
Top Elevation:	~ 23.5 ft.	Easting:	~	Drilling Company:	2R Drilling	Rod Diam.:	AWG
Vert. Datum:	MSL	Station:	~ N/A ft.	Drill Rig Equipment:	CME 55	Hammer Type:	Automatic
Horiz. Datum:	N/A	Offset:	~ N/A	Other Comments:			

SOIL DESCRIPTION *Refer to the report text for a proper understanding of the subsurface materials and drilling methods. The stratification lines indicated below represent the approximate boundaries between material types, and the transition may be gradual.*	Depth, ft.	Symbol	Samples	Ground Water	Depth, ft.
Medium dense, yellow-brown, silty fine SAND and fine sandy SILT, trace of gravel up to 1" in diameter; asphalt pieces; dry to moist; (FILL) SM/ML.	3.5		1, 2	Water Not Observed	
Loose to medium dense, yellow-brown, fine sandy SILT, trace clay; dry to moist; pieces of glass, (FILL) ML.	12.0		3, 4, 5		5, 10
Very loose, yellow-brown, silty fine SAND; moist to wet; trace clay; (Qya2) SM.	14.0		6		
BOTTOM OF BORING COMPLETED 6/21/2010					15, 20, 25, 30, 35

PENETRATION RESISTANCE (blows/foot)

▲ Hammer Wt. & Drop: 140 lbs/30 inches



LEGEND

- * Sample Not Recovered
- Modified California Sampler
- Standard Penetration Test
- Piezometer Screen and Sand Filter
- Bentonite-Cement Grout
- Bentonite Chips/Pellets
- Bentonite Grout

◇ % Fines (<0.075mm)
● % Water Content
Plastic Limit ├──●──┤ Liquid Limit
Natural Water Content

NOTES

1. Refer to KEY for explanation of symbols, codes, abbreviations and definitions.
2. The stratification lines represent the approximate boundaries between soil types, and the transition may be gradual.
3. The discussion in the text of this report is necessary for a proper understanding of the nature of the subsurface materials.
4. Groundwater level, if indicated above, is for the date specified and may vary.
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LOG OF BORING B-4

July 2010 51-1-10014-001

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FIG. A-5

MASTER LOG E 51-1-10014-001.GPJ SHAN WIL.GDT 7/15/10 Log: MAZ Rev: Typ: LOL

Total Depth:	51.5 ft.	Northing:	-	Drilling Method:	Hollow Stem Auger
Top Elevation:	~ 23 ft.	Easting:	-	Drilling Company:	2R Drilling
Vert. Datum:	MSL	Station:	~ N/A ft.	Drill Rig Equipment:	CME 55
Horiz. Datum:	N/A	Offset:	~ N/A	Other Comments:	

Hole Diam.:	8 in.
Rod Diam.:	AWG
Hammer Type:	Automatic

SOIL DESCRIPTION

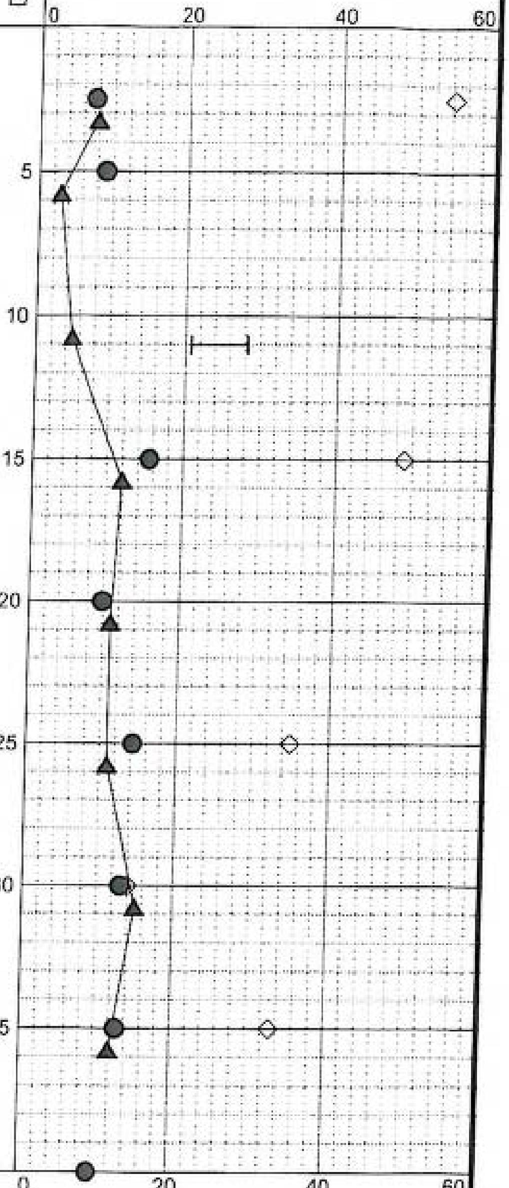
Refer to the report text for a proper understanding of the subsurface materials and drilling methods. The stratification lines indicated below represent the approximate boundaries between material types, and the transition may be gradual.

Soil Description	Depth, ft.	Samples
Medium stiff to stiff, mottled yellow-brown and gray-brown, fine sandy, clayey SILT; dry to moist; rootlets, (FILL) ML.		1, 2
Loose to dense, yellow-brown and gray-brown, silty, clayey, fine gravelly, SAND, moist to wet; (Qya2) SC/SM.	8.0	3
Grades slightly clayey		4, 5, 6, 7, 8

Ground Water: During Drilling ∇

PENETRATION RESISTANCE (blows/foot)

▲ Hammer Wt. & Drop: 140 lbs/30 inches



CONTINUED NEXT SHEET

LEGEND

- • Sample Not Recovered
- Ⅰ Standard Penetration Test
- ⋈ Modified California Sampler
- ∇ Ground Water Level ATD

◇ % Fines (<0.075mm)

● % Water Content

Plastic Limit ├─●─┤ Liquid Limit

Natural Water Content

NOTES

1. Refer to KEY for explanation of symbols, codes, abbreviations and definitions.
2. The stratification lines represent the approximate boundaries between soil types, and the transition may be gradual.
3. The discussion in the text of this report is necessary for a proper understanding of the nature of the subsurface materials.
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LOG OF BORING B-5

July 2010 51-1-10014-001

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FIG. A-6
Sheet 1 of 2

MASTER LOG E 51-1-10014-001.GPJ SHAN WIL.GDT 7/15/10 Log: MAZ Rev: Typ: LOL

Total Depth:	51.5 ft.	Northing:	~	Drilling Method:	Hollow Stem Auger
Top Elevation:	~ 23 ft.	Easting:	~	Drilling Company:	2R Drilling
Vert. Datum:	MSL	Station:	~ N/A ft.	Drill Rig Equipment:	CME 55
Horiz. Datum:	N/A	Offset:	~ N/A	Other Comments:	

Hole Diam.:	8 in.
Rod Diam.:	AWG
Hammer Type:	Automatic

SOIL DESCRIPTION

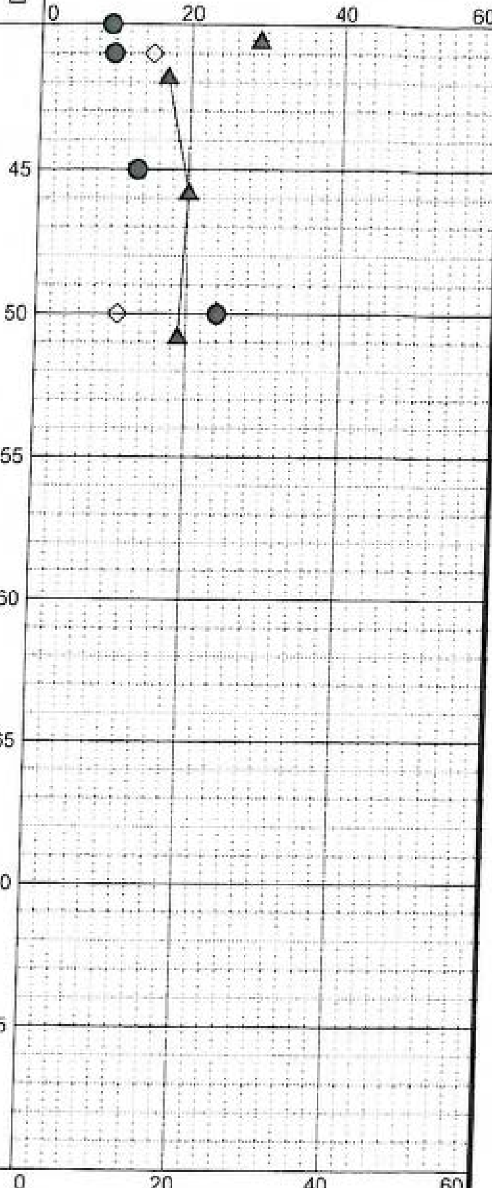
Refer to the report text for a proper understanding of the subsurface materials and drilling methods. The stratification lines indicated below represent the approximate boundaries between material types, and the transition may be gradual.

SOIL DESCRIPTION	Depth, ft.	Samples
		9, 10, 11
Dense, yellow-brown, slightly silty, fine to medium SAND; wet; (Qya2) SP-SM.	48.0	12
BOTTOM OF BORING COMPLETED 6/21/2010	51.5	

Symbol | Samples | Ground Water | Depth, ft.

PENETRATION RESISTANCE (blows/foot)

▲ Hammer Wt. & Drop: 140 lbs/30 inches



LEGEND

* Sample Not Recovered

Standard Penetration Test

Modified California Sampler

Ground Water Level ATD

◇ % Fines (<0.075mm)

● % Water Content

Plastic Limit ├─●─┤ Liquid Limit

Natural Water Content

NOTES

1. Refer to KEY for explanation of symbols, codes, abbreviations and definitions.
2. The stratification lines represent the approximate boundaries between soil types, and the transition may be gradual.
3. The discussion in the text of this report is necessary for a proper understanding of the nature of the subsurface materials.
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LOG OF BORING B-5

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FIG. A-6
Sheet 2 of 2

MASTER_LOG_E 51-1-10014-001.GPJ SHAN_WIL.GDT 7/15/10 Log: MAZ Rev: Typ: LOL

APPENDIX B

GEOTECHNICAL LABORATORY TESTING PROCEDURES AND RESULTS

APPENDIX B

GEOTECHNICAL LABORATORY TESTING PROCEDURES AND RESULTS

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TABLE

FIGURES

APPENDIX B

GEOTECHNICAL LABORATORY TESTING PROCEDURES AND RESULTS

B.1 GENERAL

This appendix contains descriptions of the procedures and results of geotechnical laboratory tests performed for our study of the Milton Street Park project. Samples recovered from borings were tested to evaluate the basic index, strength, and engineering properties of the subsurface soils. Geotechnical laboratory testing of recovered soils included visual classifications, water content determinations, grain size analyses, Atterberg Limits, compaction, corrosion, and direct shear tests. All laboratory testing was performed in general accordance with ASTM International (ASTM) standard test procedures. The geotechnical laboratory testing was conducted at the Shannon & Wilson, Inc. laboratories in Los Angeles, California. A summary of laboratory testing is presented in Table B-1.

B.2 JAR SAMPLES

Standard Penetration Test samples were stored in 16-ounce, clear plastic jars. Jar samples were stored in cardboard boxes (up to 12 jars per box) and logged in to our laboratory for tracking and testing. If potential environmental impacts were noted during drilling, material was transferred to glass jar containers and the jar was set aside and not selected for further geotechnical testing. For the project, we did not observe samples with potential environmental impacts.

Our field representative examined and classified the soil samples. Our engineer and/or geologist reviewed the samples in the office and assigned laboratory testing in accordance with our scope of services.

B.3 MODIFIED CALIFORNIA SAMPLER (MCS)

The MCS barrel has a 2.5-inch inner diameter and is usually lined with 1-inch rings to contain samples. The MCS samples were extruded from the rings for laboratory testing. The laboratory assignments are similar to the procedures outlined above for the jar samples.

B.4 WATER CONTENT DETERMINATION

The natural water contents of all the soil samples recovered from the borings were determined in general accordance with ASTM D 2216-98, Standard Method of Laboratory Determination of water (Moisture) Content of Soil, Rock, and Soil-Aggregate Mixtures. Comparison of natural water content of a soil with its index properties can be useful in characterizing soil unit weight, consistency, compressibility, and strength. Water contents are included with test results presented in this appendix, in Table B-1, and in the boring logs presented in Appendix A, respectively. For samples where no other testing was conducted, water content results are presented only in the boring logs.

B.5 GRAIN SIZE ANALYSIS

The grain size distribution of selected samples was determined in general accordance with the ASTM D 422, Standard Test Method for Particle-Size Analysis of Soils. Two general procedures were used to determine the grain size distribution of soil, including sieve analysis and combined analysis (sieve analysis and hydrometer analysis). These tests are useful for classifying soils, for providing correlation with soil properties, and for evaluating liquefaction potential.

The results are presented as grain size distribution curves in Figure B-1. Each gradation sheet provides the Unified Soil Classification System (USCS) group symbol, the sample description, water content, and the Atterberg limits (if performed). The USCS for samples with fewer than 50 percent fines (smaller than 0.075 millimeter [mm]) were classified in general accordance with ASTM D 2488, Standard Practice for Description and Identification of Soils (Visual-Manual Procedure). The percent passing the No. 200 sieve (0.075 mm) is also shown in the exploration logs included in Appendix A.

B.6 ATTERBERG LIMITS

Soil plasticity was determined by performing Atterberg Limits tests on selected fine-grained samples, or samples with greater than 50 percent passing the No. 200 sieve. The tests were performed in general accordance with ASTM D 4318, Standard Test Method for Liquid Limit, Plastic Limit, and Plasticity Index of Soils. The Atterberg Limits results include Liquid Limit (LL), Plastic Limit (PL), and Plasticity Index (PI=LL-PL). These limits are generally used to assist in classification of soils, to indicate soil consistency (when compared to natural water content), to provide correlation to soil properties, to evaluate clogging potential, and to estimate liquefaction potential.

The LL, PL, and PI values determined from the Atterberg Limits tests are shown in plasticity charts included in Figure B-2. The plasticity charts provide the USCS group symbol, the sample description, water content, and percent passing the No. 200 sieve (if a grain size analysis was performed). The results of the Atterberg Limits determinations are also shown graphically on the exploration logs presented in Appendix B.

B.7 DIRECT SHEAR TESTS

The direct shear tests were performed on selected soil samples obtained from the MCS in general accordance with ASTM D 3080, Standard Test Method for Direct Shear Test of Soils under Consolidated Drained Conditions. A sample is placed in a test box that is split into two halves horizontally. The lower half is held stationary while the upper half is pushed such that the soil sample will shear along a horizontal surface. The normal load and shear stress are recorded during the testing. The tests were performed at field moisture content and after soaking to near-saturated moisture content and at various surcharge pressures. Three tests at variable normal loads were completed from adjacent 1-inch samples retrieved from the MCS.

Results of the tests are plotted with normal stress versus shear stress. The results from the direct shear tests are presented in Figure B-3. A best-fitting straight line is plotted between the points to estimate the internal friction angle and cohesion values.

B.8 COMPACTION TESTS

A compaction test was performed on soil samples to determine the moisture-density relationship of the subgrade soil. The samples were selected to represent the material anticipated in the pavement and track areas. The compaction test was performed in general accordance with ASTM Designation: D 1557 (AASHTO Designation T180), Moisture-Density Relations of Soils and Soil-Aggregate Mixtures Using 10-lb. (4.54-kg) Rammer and 18-inch (457-mm) Drop. In this test, several specimens at different moisture content are prepared from each sample. Each specimen is then compacted into a 4-inch-diameter, 4.6-inch-high mold using a compactive effort. The dry density of the specimen is then determined from the weight and moisture content of the specimen. The results of the test are presented in Figure B-4.

B.9 R-VALUE

To provide information for paving design, a stabilometer test ("R" value test) was performed on a sample of the upper soils from boring 4. The test was performed for us by Labelle.Marvin Professional Pavement Engineering. The results of the test are presented in Figure B-5.

B.10 CORROSION TESTS

Soil samples for corrosion and salinity testing were collected in selected samples as shown in Figure B-6. Soil samples for corrosion testing were submitted to Atlantic Consultants, Inc., which tested the samples for a variety of corrosion parameters, including pH, resistivity, and chloride and sulfate concentrations. Soil measurements were determined by the U.S.

Environmental Protection Agency (EPA)- or ASTM-approved analytical methods. The following parameters were tested:

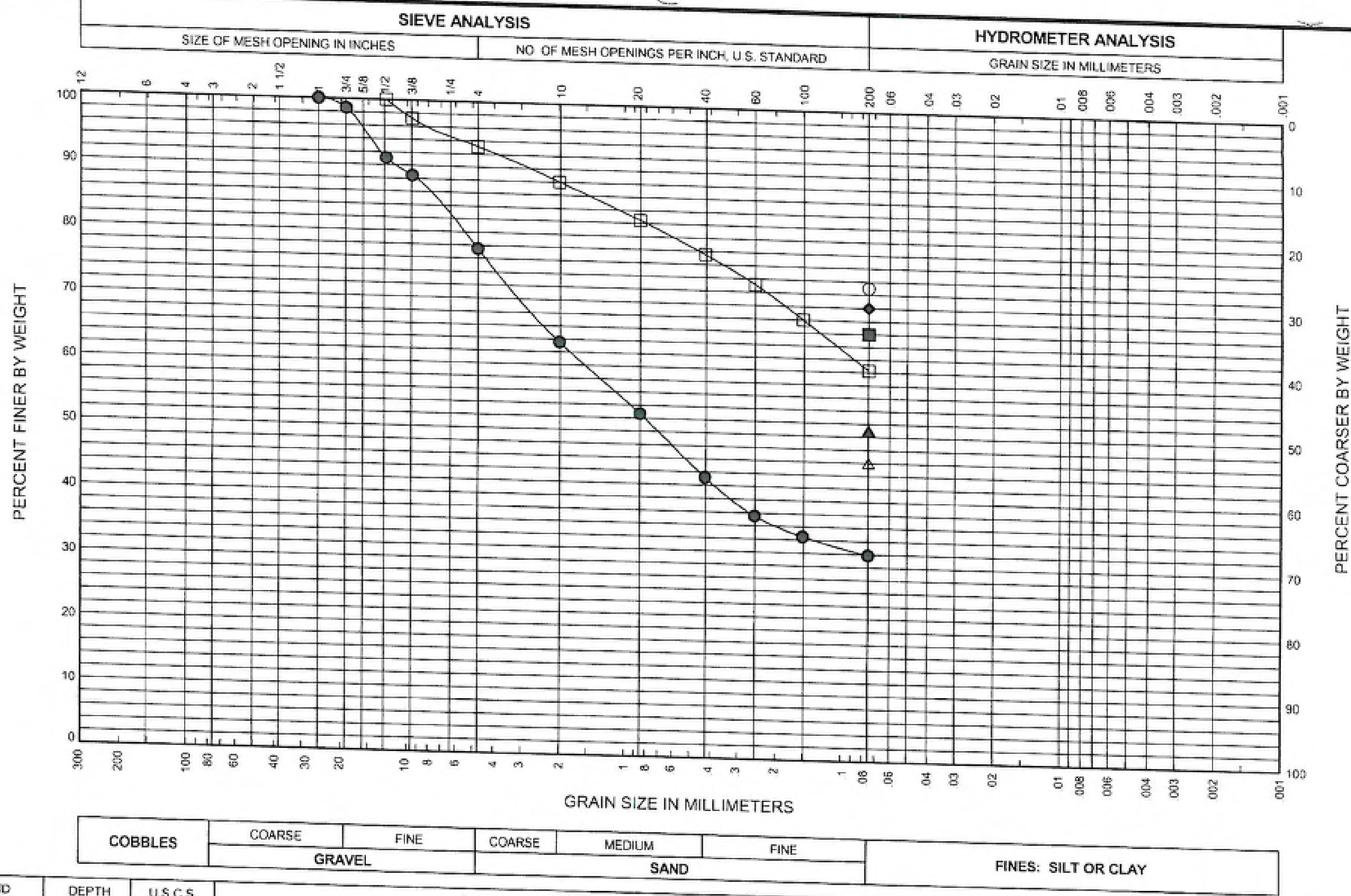
- **Sulfate and Chloride Concentration**: Sulfate is an ion that can lead to damage to metallic and concrete structures. Chloride is an ion that converts to hydrochloric acid, which can cause corrosion of metals. Also, its presence tends to decrease the soil resistivity. Chlorides may be found naturally in soils as a result of brackish groundwater and historical geological sea beds or from external from high organic content or the presence of pollutants.
- **Resistivity:** Soil resistivity is a measure of the tendency for electrical currents produced during the corrosion process to flow freely through the electrolyte. A decrease in resistivity relates to an increase in potential corrosion activity. In general, for gravelly soils with little fine matrix, typical resistivity values range from about 50,000 to 100,000 ohm centimeters (ohm-cm). For soils that are silty or clayey, the resistivity decreases to range from about 1,000 to 20,000 ohm-cm.
- **PH:** Soil pH is an indication of the acidity or alkalinity of soil and is measured in pH units. Soil pH is defined as the negative logarithm of the hydrogen ion concentration. The pH scale goes from 0 to 14 with a pH of 7 as the neutral point. As the amount of hydrogen ions in the soil increases, the soil pH decreases, thus becoming more acidic. From a pH of 7 to 0, the soil is increasingly acidic; from a pH of 7 to 14, the soil is increasingly alkaline or basic. Soils commonly have a pH range of about 5 to 8. The pH test methods used included the EPA 9045 method and the ASTM D 4972 method.

B.11 REFERENCE

ASTM International (ASTM), 2007, Annual book of ASTM standards: soil and rock, building stone; geosynthetics: Philadelphia, Pa., ASTM International, v. 04.08 and 4.09.

Table B-1
SUMMARY OF LABORATORY TESTING

Exploration SW -	Test Depth (feet)	Sample Type (SPT, MCS, ST)	Blow Count (blows/foot)	USCS	Lab Water Content (%)	Dry Density (PCF)	Grain Size Analyses			Plasticity		
							Gravel (%)	Sand (%)	Fines (%)	Liquid Limit	Plastic Limit	Plasticity Index (PI)
B-1	2.5	MC	5	SM	4	-	22	46	31	-	-	-
B-1	3.5	SPT	4	SM	4.5	-	-	-	-	-	-	-
B-1	5	MC	5	SM	13.8	-	-	-	-	-	-	-
B-1	6	SPT	12	CL	20.1	-	-	-	66	42	21	21
B-1	10	MC	15	ML	16	-	-	-	51	-	-	-
B-2	2.5	MC	16	SM	5.6	-	-	-			-	-
B-2	3.5	SPT	10	ML	16.6	-	-	-	70	-	-	-
B-2	5	MC	9	CL	18.7	106	-	-	-	-	-	-
B-2	6	SPT	8	CL	19.5	-	-	-	73	-	-	-
B-3	2.5	MC	5	ML	10.5	-	6.7	33	60.3	-	-	-
B-3	3.5	SPT	4	SM	12.7	-	-	-	46	-	-	-
B-3	5	MC	6	SM	7.6	-	-	-	-	-	-	-
B-3	6	SPT	10	CL	15.9		-	-	69	-	-	-
B-3	11	MC	22	CL	16.8	113	-	-	-	-	-	-
B-4	2.5	MC	12	SM	10.2	116	-	-	-	-	-	-
B-4	3.5	SPT	12	ML	8.3	-	-	-	54.7	-	-	
B-4	5	MC	16	ML	8.3	-	-	-	-	-	-	-
B-4	6	SPT	7	ML	18.5	-	-	-	-	-	-	-
B-4	10	SPT	13	ML	14	-	-	-	65	-	-	-
B-4	13	MC	18	SM	17.2	110	-	-	-	-	-	-
B-5	2.5	SPT	22	ML	7.6	-	-	-	54.3	-	-	-
B-5	5	SPT	7	ML	9.1	-	-	-	-	-	-	-
B-5	11	SPT	8	SC/SM	-	-	-	-	-	28	21	7
B-5	15	SPT	20	SM	15.5	-	-	-	49	-	-	-
B-5	20	SPT	19	SM	9.9	-	-	-	-	-	-	-
B-5	25	SPT	14	SM	14.3	-	20	45	35	-	-	-
B-5	30	SPT	29	SM	13	-	-	-	14	-	-	-
B-5	35	SPT	17	SM	12.8	-	24	43	33	-	-	-
B-5	40	MC	27	SM	9.5	-	-	-	-	-	-	-
B-5	41	SPT	33	SM	9.9	-	-	-	15	-	-	-
B-5	45	SPT	30	SM	13.2	-	-	-	-	-	-	-
B-5	50	SPT	34	SP-SM	24.1	-	-	-	11.1	-	-	-



BORING AND SAMPLE NO.	DEPTH (feet)	U.S.C.S. SYMBOL	SAMPLE DESCRIPTION	FINES %	NAT. W.C. %	LL %	PL %	PI %
● B-1, S-1	2.5	SM	Mottled yellow-brown and brown, silty, gravelly, fine to medium SAND	31.9	4.0			
■ B-1, S-4	6.0	CL	Gray, slightly sandy, silty CLAY	66.0	20.1	42	21	21
▲ B-1, S-5	10.0	ML	Mottled light brown, reddish brown, and gray, slightly gravelly, fine sandy SILT	51.0	16.0			
◆ B-2, S-2	3.5	ML	Mottled yellow-brown and gray-brown, fine sandy SILT	70.0	16.6			
○ B-2, S-4	6.0	SM	Yellow-brown, silty, fine to medium SAND	73.0	19.5			
□ B-3, S-1	2.5	ML	Mottled brown and yellow-brown, slightly fine gravelly, fine to medium sandy, SILT	60.3	10.5			
△ B-3, S-2	3.5	SM	Mottled yellow-brown and gray-brown, silty SAND	46.0	12.7			

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GRAIN SIZE DISTRIBUTION

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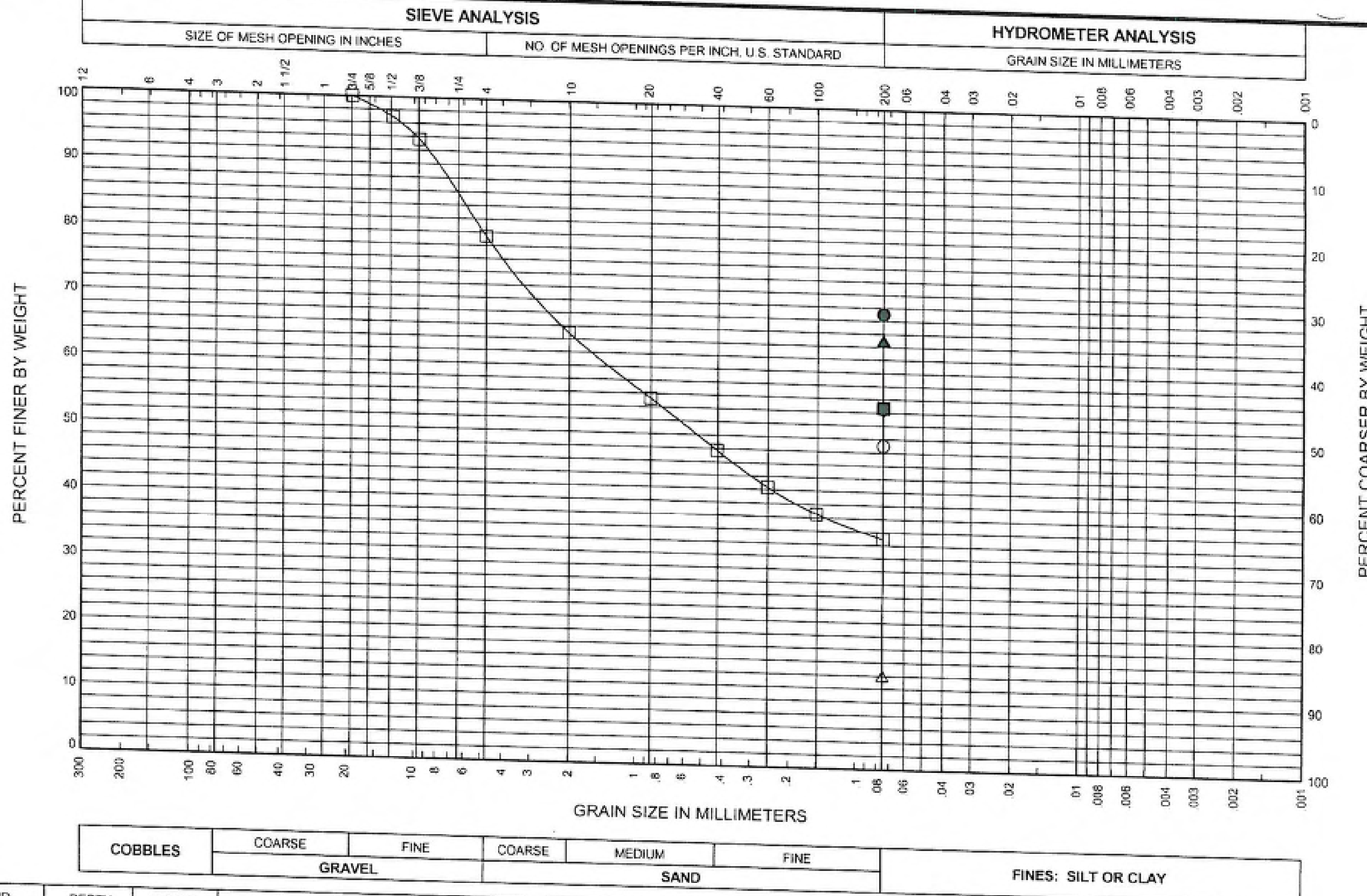
FIG. B-1
Sheet 1 of 3

MRCA



BORING AND SAMPLE NO.	DEPTH (feet)	U.S.C.S SYMBOL	SAMPLE DESCRIPTION	FINES %	NAT. W.C. %	LL %	PL %	PI %
● B-3, S-4	6.0	CL	Dark gray and brown, fine sandy CLAY	69.0	15.9			
■ B-4, S-2	3.5	ML	Yellow-brown, fine sandy SILT	54.7	8.3			
▲ B-4, S-5	10.0	ML	Yellow-brown, fine sandy SILT	65.0	14.0			
◆ B-5, S-1	2.5	ML	Mottled yellow-brown and gray-brown, fine sandy, clayey SILT	54.3	7.6			
○ B-5, S-4	15.0	SC/SM	Mottled yellow-brown and gray-brown, silty, clayey, fine gravelly, SAND	49.0	15.5			
□ B-5, S-6	25.0	SC/SM	Mottled yellow-brown and gray-brown, silty, clayey, fine gravelly, SAND	35.0	14.3			
△ B-5, S-7	30.0	SC/SM	Mottled yellow-brown and gray-brown, silty, clayey, fine gravelly, SAND	14.0	13.0			

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FIG. B-1
Sheet 2 of 3

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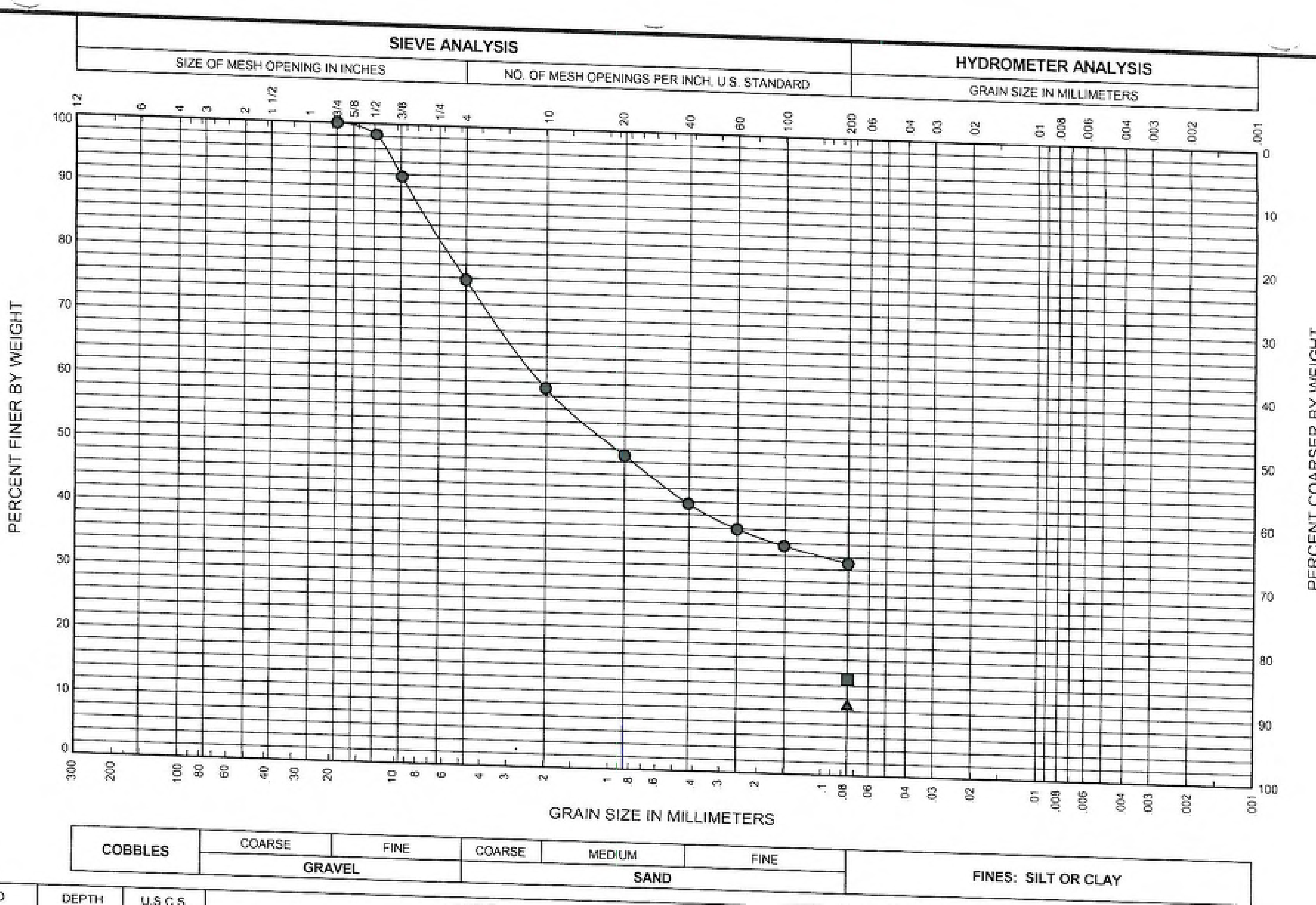
FIG. B-1



BORING AND SAMPLE NO.	DEPTH (feet)	U.S.C.S. SYMBOL	SAMPLE DESCRIPTION	FINES %	NAT. W.C. %	LL %	PL %	PI %
● B-5, S-8	35.0	SC/SM	Mottled yellow-brown and gray-brown, silty, clayey, fine gravelly, SAND	33.1	12.8			
■ B-5, S-10	41.0	SC/SM	Mottled yellow-brown and gray-brown, silty, clayey, fine gravelly, SAND	15.0	9.9			
▲ B-5, S-12	50.0	SP-SM	Yellow-brown, slightly silty, fine to medium SAND	11.1	24.1			

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FIG. B-1
Sheet 3 of 3

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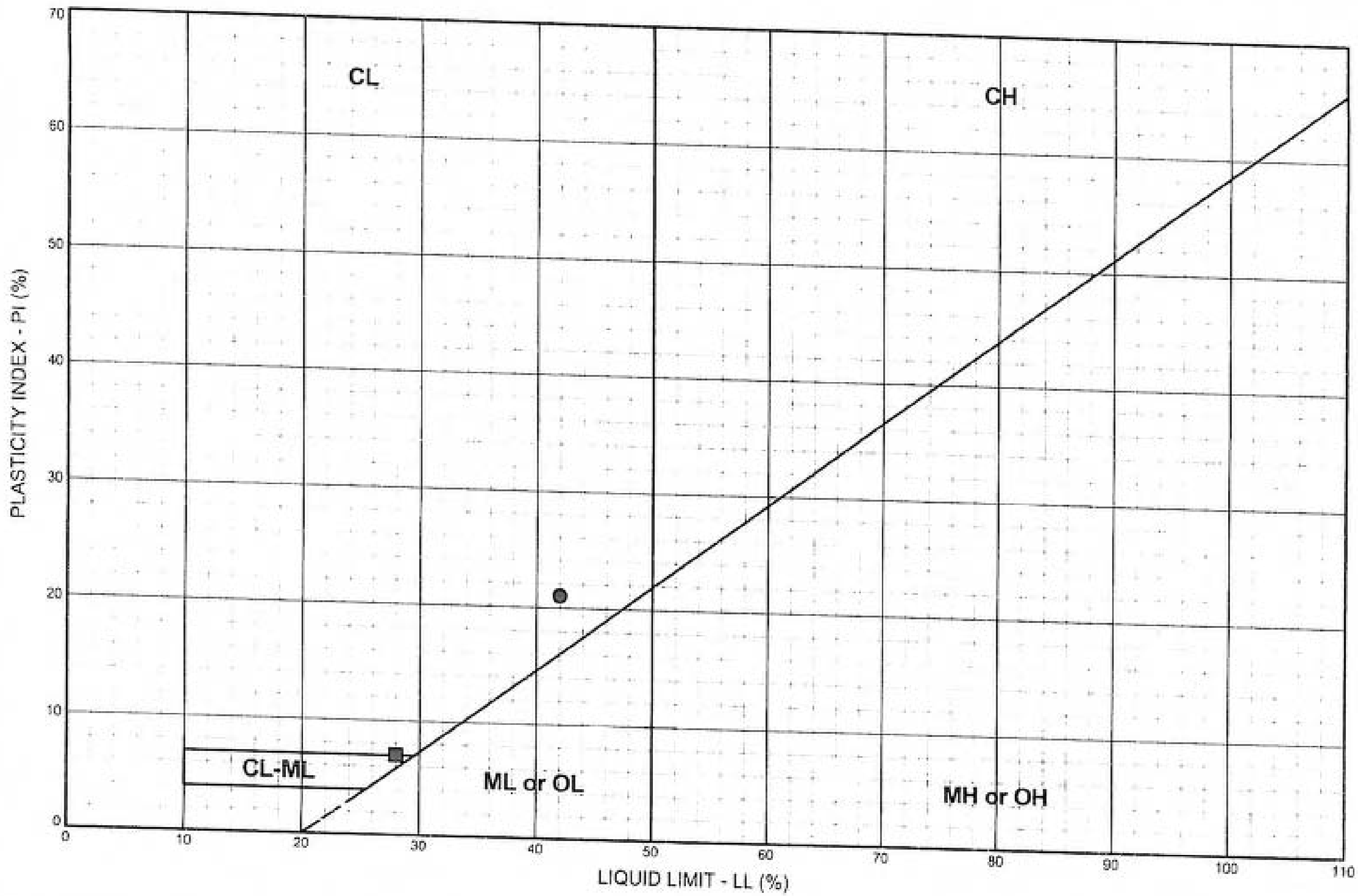
FIG. B-1



LEGEND

CL: Low plasticity inorganic clays; sandy and silty clays

CH: High plasticity inorganic clays

ML or OL: Inorganic and organic silts and clayey silts of low plasticity

MH or OH: Inorganic and organic silts and clayey silts of high plasticity

CL-ML: Silty clays and clayey silts

BORING AND SAMPLE NO.	DEPTH (feet)	U.S.C.S. SYMBOL	SOIL CLASSIFICATION	LL %	PL %	PI %	NAT W.C. %	PASS #200. %
● B-1, S-4	6.0	CL	Gray, slightly sandy, silty CLAY	42	21	21	20.1	66.0
■ B-5, S-3	11.0	SC/SM	Mottled yellow-brown and gray-brown, silty, clayey, fine gravelly, SAND	28	21	7		

Milton Street Park
MRCA
Del Rey District, Los Angeles, California

PLASTICITY CHART

July 2010

51-1-10014-001

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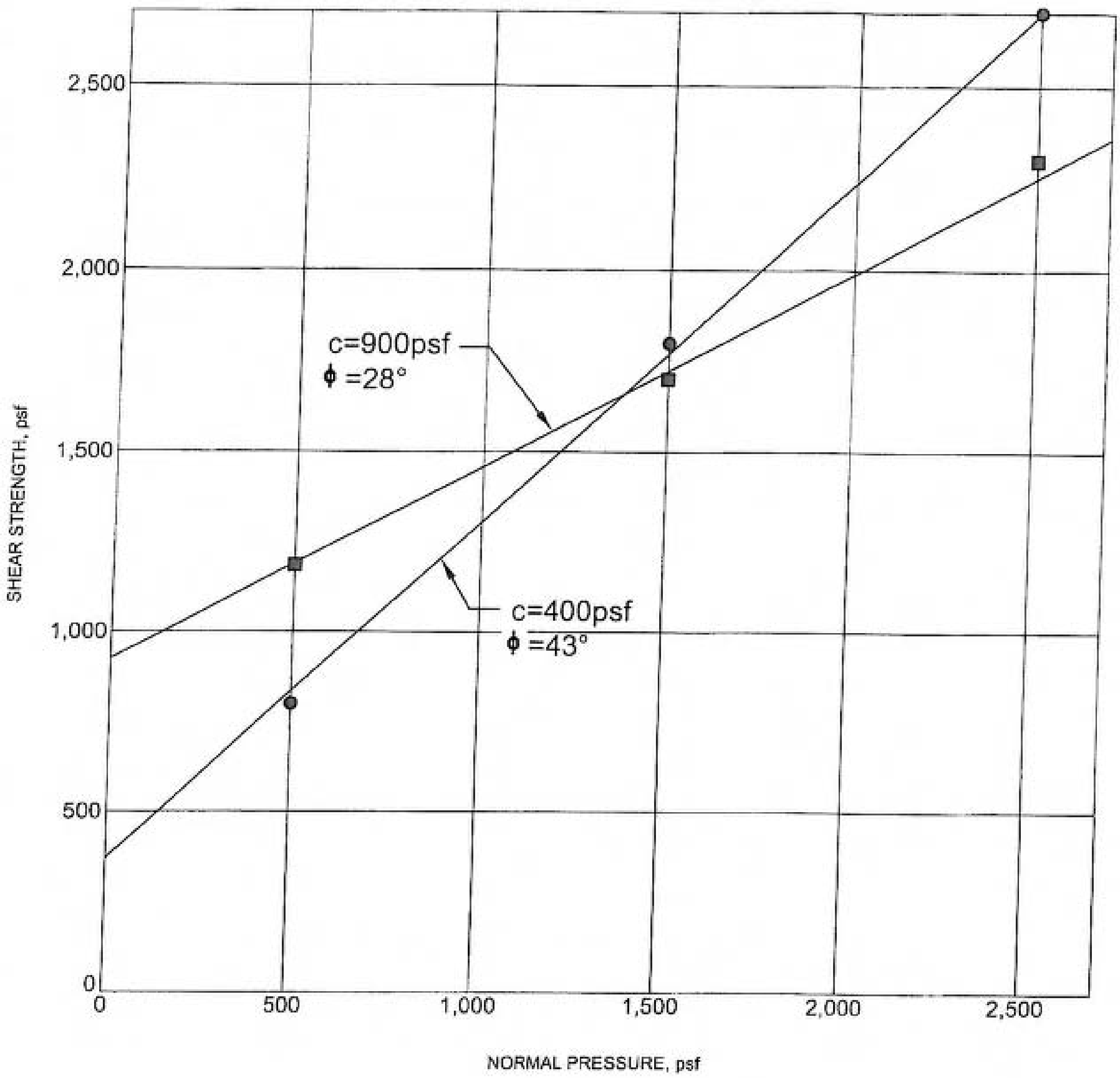
FIG. B-2

MRCA

FIG. B-2

DIRECT SHEAR TEST



B-

BOREHOLE	DEPTH	Classification	γ_d	MC%	c	ϕ
● 3	11.0	Dark gray and brown, fine sandy CLAY; CL.	113	16.8	400	43
■ 4	13.0	Yellow-brown, silty fine SAND, trace clay; SM.	110	17.2	900	28

Filename: G:\2010 Projects\10014 Milton St Park\Graphics\DIRECT SHEAR.dwg Date: 07-07-2010 Login: Louis Larios

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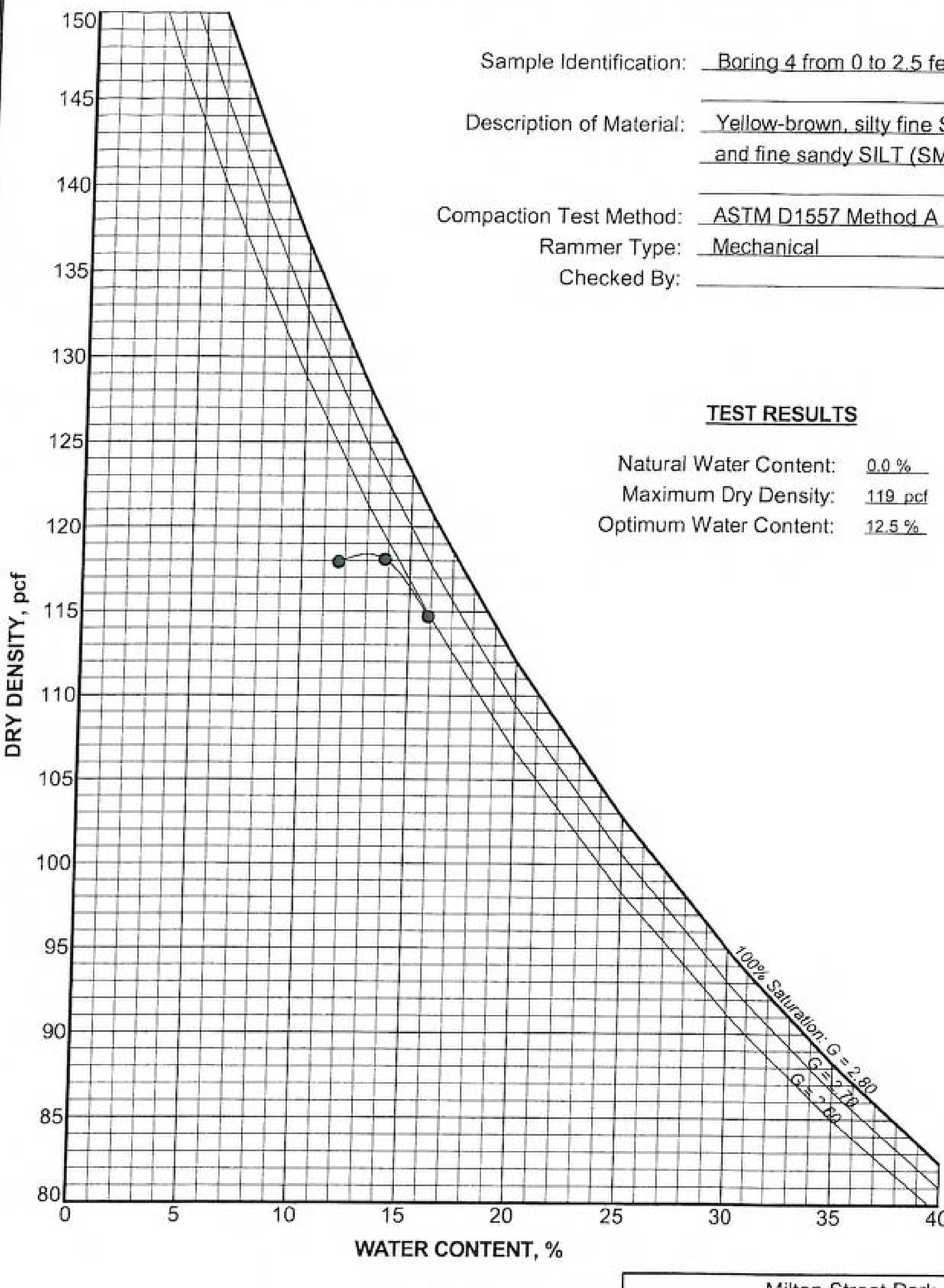
FIG. B-3



Sample Identification: Boring 4 from 0 to 2.5 feet.

Description of Material: Yellow-brown, silty fine SAND and fine sandy SILT (SM)

Compaction Test Method: ASTM D1557 Method A

Rammer Type: Mechanical

Checked By:

TEST RESULTS

Natural Water Content: 0.0 %

Maximum Dry Density: 119 pcf

Optimum Water Content: 12.5 %

Milton Street Park
MRCA
Del Rey District, Los Angeles, California

COMPACTION TEST DATA

July 2010 51-1-10014-001

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Geotechnical and Environmental Consultants

FIG. B-4

• ANALYSIS
• DESIGN

LaBelle • Marvin

• SOILS, ASPHALT TECHNOLOGY

PROFESSIONAL PAVEMENT ENGINEERING
A CALIFORNIA CORPORATION

June 29, 2010

RECEIVED

JUL 0 1 2010

SHANNON & WILSON, INC.

Mr. Travis Deane
Shannon & Wilson, Inc.
706 West Broadway., Ste. 201
Glendale, California 91204

Project No. 37005

Dear Mr. Deane:

Testing of the bulk soil sample delivered to our laboratory on 6/28/2010 has been completed.

Job Number:	***51-1-10014-001***
Samples:	***Boring No. 4 Composite***

R-Value data sheets are attached for your use and file. The opportunity to be of service is sincerely appreciated and should you have any questions, kindly call.

Respectfully Submitted,

REGISTERED PROFESSIONAL ENGINEER
STEVEN R. MARVIN
NO. 30659
EXP. 3/31/12
CIVIL
STATE OF CALIFORNIA

Steven R. Marvin
RCE 30659

SRM:tw

2700 S. GRAND AVENUE • SANTA ANA, CA 92705-5404 • (714) 546-3468 • FAX (714) 546-5841
INFO@LABELLEMARVIN.COM

R - VALUE DATA SHEET

J.N. 51-1-10014-001

PROJECT NUMBER 37005 BORING NUMBER: Boring 4 Composite

SAMPLE DESCRIPTION: Brown Clayey Silt

Item	SPECIMEN		
	a	b	c
Mold Number	1	2	3
Water added, grams	100	70	152
Initial Test Water, %	18.9	16.2	23.7
Compact Gage Pressure,psi	40	80	30
Exudation Pressure, psi	282	425	182
Height Sample, Inches	2.58	2.44	2.70
Gross Weight Mold, grams	3039	3029	3052
Tare Weight Mold, grams	1965	1969	1977
Sample Wet Weight, grams	1074	1060	1075
Expansion, Inches x 10exp-4	13	45	0
Stability 2,000 lbs (160psi)	57 / 136	41 / 104	68 / 148
Turns Displacement	4.21	3.62	4.26
R-Value Uncorrected	9	27	5
R-Value Corrected	9	26	5
Dry Density, pcf	106.0	113.3	97.5
	DESIGN CALCULATION DATA		
Traffic Index Assumed:	4.0	4.0	4.0
G.E. by Stability	0.93	0.76	0.97
G. E. by Expansion	0.43	1.50	0.00

Equilibrium R-Value 10 by EXUDATION

Examined & Checked: 6 /29/ 10

REMARKS: Gf = 1.25
0.0% Ret. On the 3/4" Sieve.

REGISTERED PROFESSIONAL ENGINEER STEVEN R. MARVIN C 30659 EXP. STATE OF CALIFORNIA

Steven R. Marvin, RCE 30659

The data above is based upon processing and testing samples as received from the field. Test procedures in accordance with latest revisions to Department of Transportation, State of California, Materials & Research Test Method No. 301.

LaBelle • Marvin

R-VALUE GRAPHICAL PRESENTATION

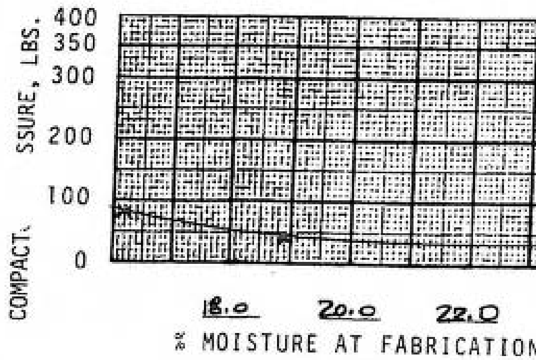
PROJECT NO. 37005
J.N. 51-1-10014-001
BORING NO. Boring 4 Composite

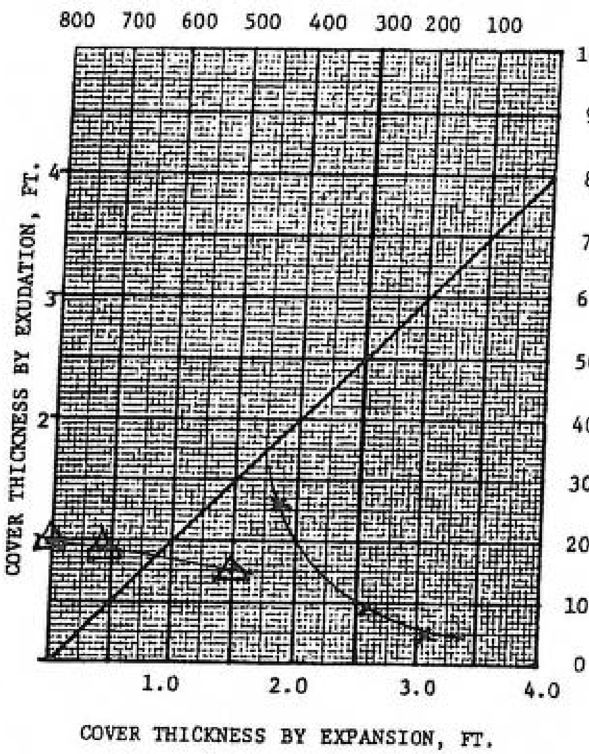
DATE 6-29-10

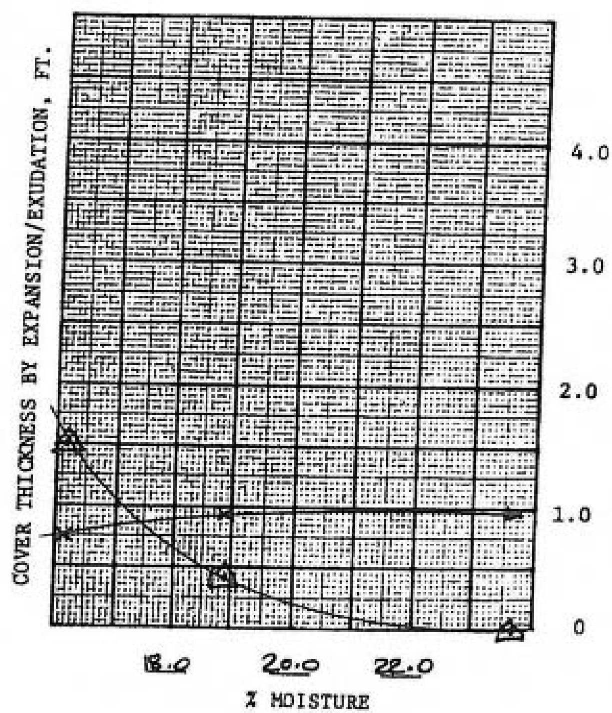
TRAFFIC INDEX Assume 4.0

R-VALUE BY EXUDATION 10

R-VALUE BY EXPANSION 2







R-VALUE vs. EXUD. PRES.

T by EXUDATION

EXUD. T vs. EXPAN. T

T by EXPANSION

REMARKS

GF=1.25

LaBelle • Marvin
PROFESSIONAL PAVEMENT ENGINEERING

SHANNON & WILSON, INC.
GEOTECHNICAL AND ENVIRONMENTAL CONSULTANTS

FIG. B-5
Sheet 3 of 3

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

July 9, 2010

Shannon and Wilson
Attention: Dean Francuch
706 W. Broadway, Suite 201
Glendale, CA 91204

Atlantic Job No.: 2010-028

Subject: Soil Chemistry Analysis for Shannon and Wilson, Job # 51-1-10014-001
2 Samples: B-2 @ 2-5' and B-5 @ 11-31' – Milton Street Park, Del Rey Dist., Los Angeles, CA.

Sample Number	As Rec'd Resistivity (ohm-cm)	[1]Minimum Resistivity (ohm-cm)	[2]pH	[3]Sulfate %	[3]Chloride %	(As Rec'd) Description
B-2	9,600	2,240	7.22	0.0036	0.0033	Medium Brown, Moist
B-5	3,600	1,920	7.17	0.0085	0.0060	Dark Brown, Sandy, Moist

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. (UBC Table 19-A-4)	-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 concrete slabs.
Steel Cast/Ductile Iron Mortar Coated Steel	Moderately to Mildly 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.

Atlantic Consultants

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Folsom, CA 95630
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Kerri@AtlanticCorrosionEngineers.com
corrprincess@ardennet.com
www.AtlanticCorrosionEngineers.com

The test results and recommendations are based on the samples 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

Kerri M. Howell, P.E.
President

APPENDIX C

IMPORTANT INFORMATION ABOUT YOUR GEOTECHNICAL/ENVIRONMENTAL REPORT

SHANNON & WILSON, INC.
Geotechnical and Environmental Consultants

Attachment to and part of Report 51-1-10014-001

Date: July 20, 2010
To: Ms. Ana Petrlic
Mountains Recreation & Conservation Authority

IMPORTANT INFORMATION ABOUT YOUR GEOTECHNICAL/ENVIRONMENTAL REPORT

CONSULTING SERVICES ARE PERFORMED FOR SPECIFIC PURPOSES AND FOR SPECIFIC CLIENTS.

Consultants prepare reports to meet the specific needs of specific individuals. A report prepared for a civil engineer may not be adequate for a construction contractor or even another civil engineer. Unless indicated otherwise, your consultant prepared your report expressly for you and expressly for the purposes you indicated. No one other than you should apply this report for its intended purpose without first conferring with the consultant. No party should apply this report for any purpose other than that originally contemplated without first conferring with the consultant.

THE CONSULTANT'S REPORT IS BASED ON PROJECT-SPECIFIC FACTORS.

A geotechnical/environmental report is based on a subsurface exploration plan designed to consider a unique set of project-specific factors. Depending on the project, these may include: the general nature of the structure and property involved; its size and configuration; its historical use and practice; the location of the structure on the site and its orientation; other improvements such as access roads, parking lots, and underground utilities; and the additional risk created by scope-of-service limitations imposed by the client. To help avoid costly problems, ask the consultant to evaluate how any factors that change subsequent to the date of the report may affect the recommendations. Unless your consultant indicates otherwise, your report should not be used: (1) when the nature of the proposed project is changed (for example, if an office building will be erected instead of a parking garage, or if a refrigerated warehouse will be built instead of an unrefrigerated one, or chemicals are discovered on or near the site); (2) when the size, elevation, or configuration of the proposed project is altered; (3) when the location or orientation of the proposed project is modified; (4) when there is a change of ownership; or (5) for application to an adjacent site. Consultants cannot accept responsibility for problems that may occur if they are not consulted after factors which were considered in the development of the report have changed.

SUBSURFACE CONDITIONS CAN CHANGE.

Subsurface conditions may be affected as a result of natural processes or human activity. Because a geotechnical/environmental report is based on conditions that existed at the time of subsurface exploration, construction decisions should not be based on a report whose adequacy may have been affected by time. Ask the consultant to advise if additional tests are desirable before construction starts; for example, groundwater conditions commonly vary seasonally.

Construction operations at or adjacent to the site and natural events such as floods, earthquakes, or groundwater fluctuations may also affect subsurface conditions and, thus, the continuing adequacy of a geotechnical/environmental report. The consultant should be kept apprised of any such events, and should be consulted to determine if additional tests are necessary.

MOST RECOMMENDATIONS ARE PROFESSIONAL JUDGMENTS.

Site exploration and testing identifies actual surface and subsurface conditions only at those points where samples are taken. The data were extrapolated by your consultant, who then applied judgment to render an opinion about overall subsurface conditions. The actual interface between materials may be far more gradual or abrupt than your report indicates. Actual conditions in areas not sampled may differ from those predicted in your report. While nothing can be done to prevent such situations, you and your consultant can work together to help reduce their impacts. Retaining your consultant to observe subsurface construction operations can be particularly beneficial in this respect.

A REPORT'S CONCLUSIONS ARE PRELIMINARY.

The conclusions contained in your consultant's report are preliminary because they must be based on the assumption that conditions revealed through selective exploratory sampling are indicative of actual conditions throughout a site. Actual subsurface conditions can be discerned only during earthwork; therefore, you should retain your consultant to observe actual conditions and to provide conclusions. Only the consultant who prepared the report is fully familiar with the background information needed to determine whether or not the report's recommendations based on those conclusions are valid and whether or not the contractor is abiding by applicable recommendations. The consultant who developed your report cannot assume responsibility or liability for the adequacy of the report's recommendations if another party is retained to observe construction.

THE CONSULTANT'S REPORT IS SUBJECT TO MISINTERPRETATION.

Costly problems can occur when other design professionals develop their plans based on misinterpretation of a geotechnical/environmental report. To help avoid these problems, the consultant should be retained to work with other project design professionals to explain relevant geotechnical, geological, hydrogeological, and environmental findings, and to review the adequacy of their plans and specifications relative to these issues.

BORING LOGS AND/OR MONITORING WELL DATA SHOULD NOT BE SEPARATED FROM THE REPORT.

Final boring logs developed by the consultant are based upon interpretation of field logs (assembled by site personnel), field test results, and laboratory and/or office evaluation of field samples and data. Only final boring logs and data are customarily included in geotechnical/environmental reports. These final logs should not, under any circumstances, be redrawn for inclusion in architectural or other design drawings, because drafters may commit errors or omissions in the transfer process.

To reduce the likelihood of boring log or monitoring well misinterpretation, contractors should be given ready access to the complete geotechnical engineering/environmental report prepared or authorized for their use. If access is provided only to the report prepared for you, you should advise contractors of the report's limitations, assuming that a contractor was not one of the specific persons for whom the report was prepared, and that developing construction cost estimates was not one of the specific purposes for which it was prepared. While a contractor may gain important knowledge from a report prepared for another party, the contractor should discuss the report with your consultant and perform the additional or alternative work believed necessary to obtain the data specifically appropriate for construction cost estimating purposes. Some clients hold the mistaken impression that simply disclaiming responsibility for the accuracy of subsurface information always insulates them from attendant liability. Providing the best available information to contractors helps prevent costly construction problems and the adversarial attitudes that aggravate them to a disproportionate scale.

READ RESPONSIBILITY CLAUSES CLOSELY.

Because geotechnical/environmental engineering is based extensively on judgment and opinion, it is far less exact than other design disciplines. This situation has resulted in wholly unwarranted claims being lodged against consultants. To help prevent this problem, consultants have developed a number of clauses for use in their contracts, reports and other documents. These responsibility clauses are not exculpatory clauses designed to transfer the consultant's liabilities to other parties; rather, they are definitive clauses that identify where the consultant's responsibilities begin and end. Their use helps all parties involved recognize their individual responsibilities and take appropriate action. Some of these definitive clauses are likely to appear in your report, and you are encouraged to read them closely. Your consultant will be pleased to give full and frank answers to your questions.

The preceding paragraphs are based on information provided by the
ASFE/Association of Engineering Firms Practicing in the Geosciences, Silver Spring, Maryland