A Report Prepared for:

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GEOTECHNICAL INVESTIGATION
SUNSET RESERVOIR PERCHLORATE TREATMENT FACILITY
PASADENA, CALIFORNIA

Project No. 2005-017

by

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1.0 INTRODUCTION

This report presents the results of the geotechnical investigation performed by Diaz•Yourman & Associates (DYA) for the proposed ion-exchange treatment system in Pasadena, California. Stetson Engineers, Inc. (Stetson) authorized this work on April 7, 2005. However, the project was then put on hold. Subsequently, Stetson provided a notice to proceed on November 11, 2008, to continue the work. A contact was also provided on November 18, 2008.

The ion-exchange treatment system for perchlorate removal from the Sunset Reservoir will be located at Sunset Reservoir site in Pasadena, California, as shown on the Vicinity Map, Figure 1. The City of Pasadena Water and Power Department (PWP) shut down some of the Sunset Reservoir wells because of perchlorate contamination. The loss of these wells has severely impacted PWP's ability to meet the City of Pasadena's water demand. Therefore, PWP requires a perchlorate treatment system for its Sunset reservoir wells in order to recover some of its production capacity. The proposed perchlorate treatment project is summarized in Table 1.

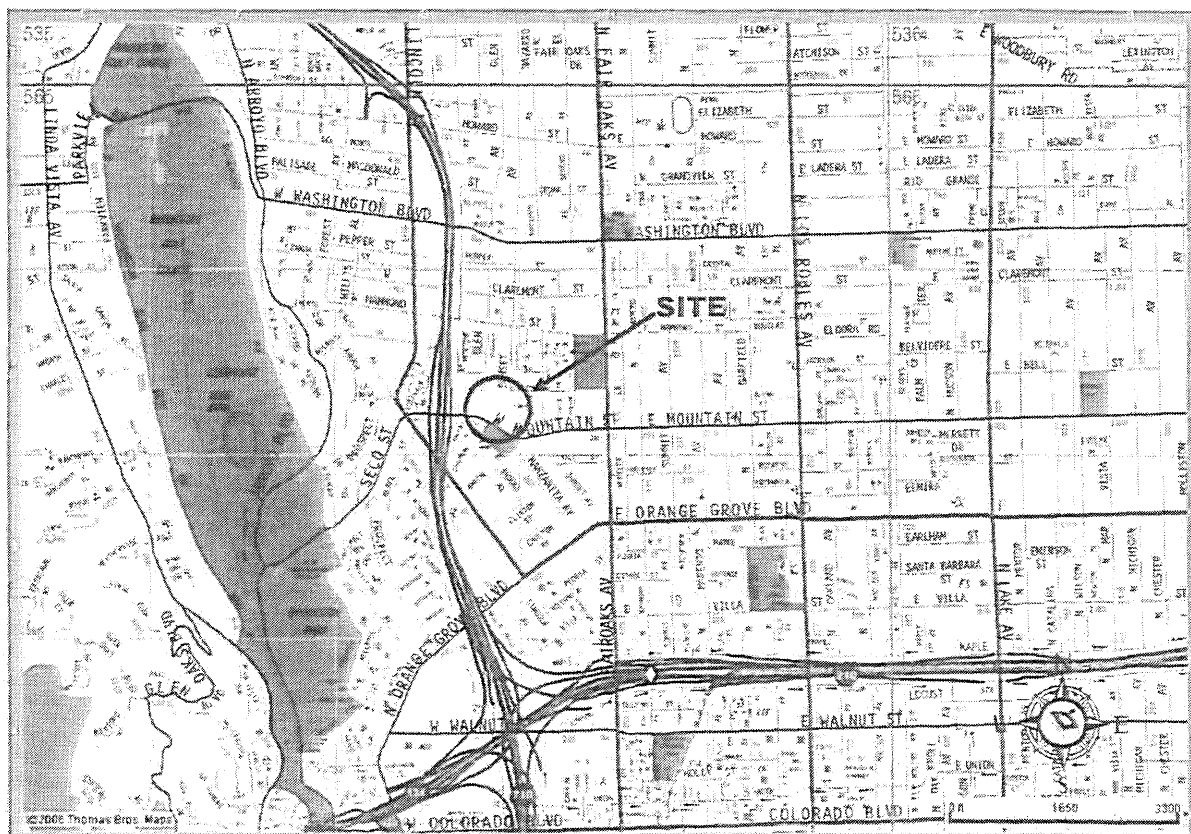


Figure 1 - VICINITY MAP

Table 1 - SUMMARY OF PROPOSED FEATURES

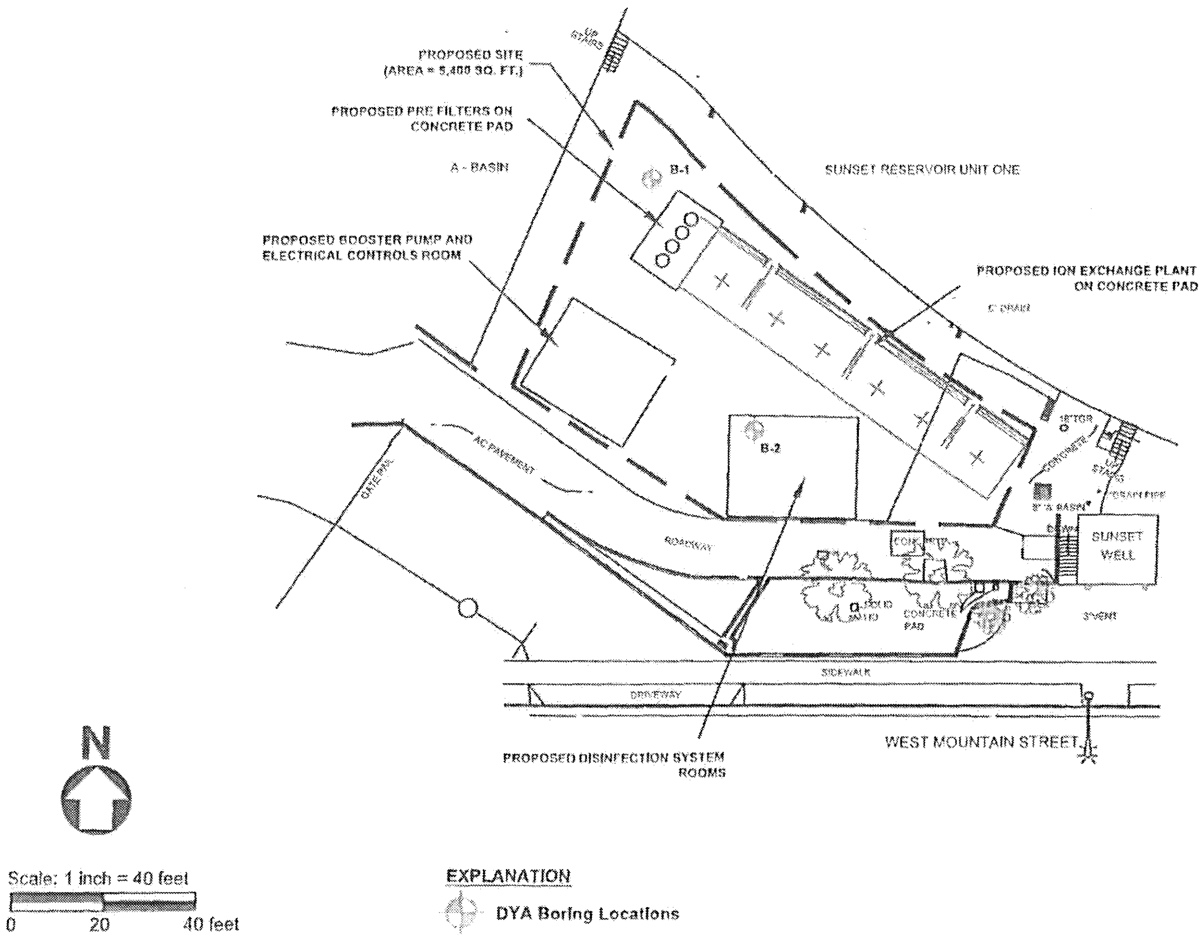
FEATURE	DESCRIPTION	ANTICIPATED STRUCTURAL LOAD
Ion exchange plant	6 units on an 81- by 16-foot mat foundation	150 psf
Pre filters	4 units on an 18- by 12-foot mat foundation	<150 psf
Booster pump station, electrical control room and a wet well	3 pump columns, 10-foot-deep, 16- by 16-foot (plan area) wet well with a control room on top of the wet well	Not known
Disinfection system room	22- by 28-foot building	Not known
Note: • psf = pounds per square foot.		

The approximate layout of the proposed project is shown on the Site Plan, Figure 2. The foundation loads of the proposed structures are summarized in Table 1. The proposed grades will be within 1 foot of existing grades, except a 10-foot-deep wet well will be constructed below the booster pump station.

The purpose of DYA's investigation was to provide geotechnical input for the design of the proposed project. The scope of our services consisted of the following tasks:

- Reviewing geotechnical data.
- Conducting a field investigation.
- Performing laboratory tests on selected samples.
- Performing engineering analyses to develop conclusions and recommendations regarding the following:
  - Site preparation and grading
  - Shallow foundation bearing capacity
  - Estimated total and differential foundation settlements
  - Resistance to lateral loads
  - Temporary shoring and lateral earth pressures
  - Slab-on-grade support
  - Asphalt concrete (AC) and Portland cement concrete (PCC) pavement thickness design
  - Soil corrosion potential
- Preparing this report.

2005/2005-017/CADD/Figure 2 - SIT PLAN 12-17-2008.dwg



Reference: Electronic base map provided by Stetson Engineers (2008)

Figure 2 - SITE PLAN

2.0 DATA REVIEW, FIELD INVESTIGATION, AND LABORATORY TESTING

Geotechnical data from the project vicinity presented in previous reports were reviewed to supplement site data collected during this investigation. A list of the documents reviewed is presented in the bibliography (Section 7.0).

The field investigation, conducted on May 12, 2005, consisted of drilling two soil borings at the locations shown on Figure 2. The boring locations were chosen to provide areal coverage of the project site for grading and data for foundation and pavement design. The boring depths, ranging from approximately 41 to 44 feet, were selected to extend to the depth of significant influence of the proposed loads. Details of the field investigation, including sampling procedures and boring logs, are presented in Appendix A.

Soil samples collected from the borings were re-examined in the laboratory to substantiate field classifications. Selected soil samples were tested for moisture content, dry density, grain-size distribution, percent passing the No. 200 sieve, Atterberg limits, shear strength, compaction characteristics, pavement-supporting capacity, and corrosion potential (pH, electrical resistivity, soluble chlorides, and soluble sulfates). Expansion index tests were not performed because the surface soils were visually classified as sands and silty sand. The soil samples tested are identified on the boring logs. Laboratory test data are summarized on the boring logs in Appendix A and presented on individual test reports in Appendix B.

3.0 SITE CONDITIONS

3.1 SURFACE CONDITIONS

The project site included two small single-story buildings that were being used as storage buildings for PWP during our field investigation. The project site was paved with PCC. The thickness of the PCC varied from 5 to 6 inches. Several types of distresses were observed in the pavement surface during the time of our investigation. The existing ground surface elevation was approximately 940 feet above mean sea level (MSL).

3.2 SUBSURFACE CONDITIONS

The subsurface soils encountered during our geotechnical field investigation can be summarized as follows:

- The upper approximately 10 feet of soil consisted of dark/grayish brown, moist, loose to medium dense, fine- to coarse-grained sand with little gravel. The uncorrected standard penetration test (SPT) numbers ranged from 8 to 16 blows per foot (bpf). The average dry density of the insitu soils was approximately 105 pounds per cubic foot (pcf). The insitu moisture content ranged from 9 to 12 percent. Based on the compaction test (ASTM D 1557) performed on soil sample from Boring B-2, the relative compaction[1] of the subgrade soils underneath the PCC slab ranged from 72 to 77 percent. The determination of the dry density and relative compaction is affected by the sample disturbance during the sampling process, transportation, and laboratory testing. Therefore, the dry density and the relative compaction values are considered approximate.
- The deeper soils consisted of medium dense to very dense sands. The uncorrected SPT numbers ranged from 15 to greater than 50 bpf. The average dry density of the insitu soils was 120 pcf and the insitu moisture content ranged from 3 to 10 percent.

Groundwater was encountered at Boring B-1 at 35 feet below the ground surface (bgs) but not in Boring B-2. However, the depth to groundwater near the project has been reported as deep as 150 feet bgs (California Geological Survey [CGS], Open file report 98-05). Therefore, we judge the water encountered at 35 feet bgs in Boring B-1 could be perched water.

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[1] Relative compaction refers to the in-place dry density of soil expressed as a percentage of the maximum dry density of the same material, as determined by the American Society for Testing Materials (ASTM) D1557-91 test method. Optimum moisture content is the moisture content corresponding to the maximum dry density, as determined by the ASTM D1557-91 test method.

4.0 CONCLUSIONS AND RECOMMENDATIONS

Based on geotechnical considerations, the site is suitable for the proposed project. The primary geotechnical consideration is the looseness and variable density of the upper soils, which would provide uneven foundation support and settle once subjected to the proposed loads. The upper 3 feet of soils should, therefore, be removed and re-compacted. If any fill material is encountered during grading, fill should be removed and re-compacted. We recommend that a registered geologist or geotechnical engineer observe the grading operation. The proposed facilities can be supported on shallow foundations such as mat and/or strip foundation placed on a layer of compacted fill. Excavation for wet well requires shoring. Our recommendations are presented in the following sections.

4.1 SEISMIC/GEOLOGIC HAZARDS

The site, like most of Southern California, will be subject to strong ground shaking during major earthquakes. Seismic design can be performed in accordance with the criteria listed in Table 2.

Table 2 - SEISMIC DESIGN CRITERIA

CHARACTERISTIC	CRITERIA
Site Class	D
S_s - mapped maximum considered earthquake spectral acceleration at short periods (g)	2.43[1]
S_1 - mapped maximum considered earthquake spectral acceleration at 1-second period (g)	0.85[1]
F_a - site coefficient	1.0
F_v - site coefficient	1.5
S_{MS} - adjusted maximum considered earthquake spectral acceleration at short periods (g)	2.43
S_{M1} - adjusted maximum considered earthquake spectral acceleration at 1-second period	1.27
California Seismic Hazards Mapping Act, Liquefaction Zone	Site outside the mapped
California Seismic Hazards Mapping Act, Landslide Zone	Site outside the mapped
Alquist-Priolo Special Study Zone	Site outside special study zones
Peak ground acceleration,[2] (g)	0.65

Notes:
1. ASCE 7-05 mapped values based on Earthquake Ground Motion Tool computer program (U.S. Geological Survey, 2007).
2. California Building Code (CBC) Section 1802.2.7 (S_{DC} /2.5).

Based on the depth to historically highest groundwater levels at the proposed site, we judge that subsurface soils subject to liquefaction will be very low to remote. However, the potential seismic settlement was estimated using procedures presented by Tokimatsu and Seed (1987). Based on our analyses, the seismic settlement is estimated to be less than 1 inch for the existing site conditions (i.e., without excavation to remove existing loose soils) for a peak ground acceleration (PGA) of 0.65g and moment magnitude of 6.9. Static foundation settlements are noted in Section 4.3 and are in addition to the seismic settlements noted here. The recommended removal and recompaction of the upper 3 feet of existing soils should reduce the potential for seismic settlement by approximately ½ inch.

4.2 EARTHWORK

4.2.1 Site Preparation and Grading

Prior to the start of construction, the following should be performed:

- All utilities should be located in the field and either rerouted, removed, abandoned, or protected.
- PCC should be separated for recycling.

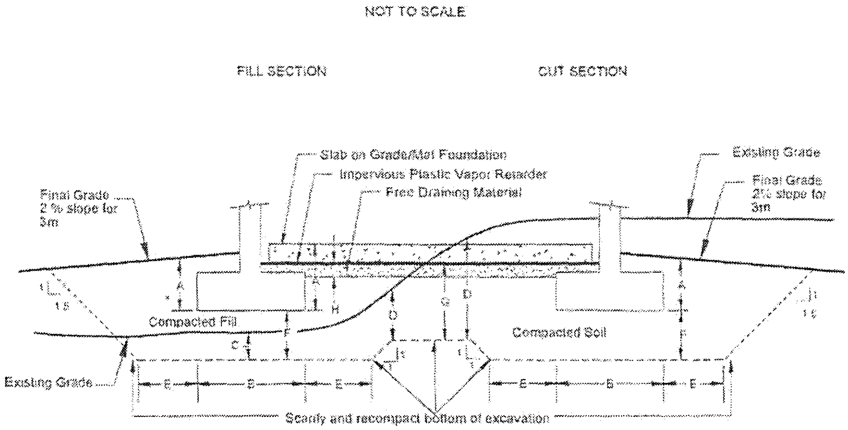
The upper soil should be excavated and replaced with compacted fill as shown on Figure 3. The bottom of the excavation should be

- Scarified to a depth of 8 inches.
- Moisture-conditioned to above-optimum moisture content.
- Compacted to at least 95 percent relative compaction.

Fill and backfill should be compacted by:

- Placing in loose layers less than 8 inches thick.
- Moisture-conditioning to above-optimum moisture content.
- Compacting to at least 95 percent relative compaction.

The wet well mat foundation can be supported on medium dense to dense natural soils. However, if the medium dense to dense natural soils are disturbed during construction (by mechanical means), the disturbed natural soils should be compacted to at least 95 percent relative compaction.



LOCATION	MINIMUM DIMENSIONS (feet)		
	Strip Foundation	Mat Foundation	Wet Well
A. Footing Embedment Below Subgrade	1.5	1	See Table 1
B. Footing Width	1	See Table 1	See Table 1
C. Excavation Below Existing Grade (Footing)	3	--	0
D. Excavation Below Existing Grade (Slab)	3	3	0
E. Excavation Beyond Footing	5	5	0
F. Compacted Soil/Fill Below Footing	2	2	--
G. Compacted Soil/Fill Below Slab	--		0
H. Free-Draining Material	--	0.5	--
PRESSURE (psf)			
Static (net) Allowable Bearing Capacity (FS≥3)	2,000	500	2,000
Increase per Foot of Depth	300	--	300
Increase per Foot of Width	200	--	200
Maximum Static Bearing Capacity (FS≥3)	3,000	500	3,000
Maximum Transient Bearing Capacity (FS≥2)	4,000	650	4,000

Figure 3 - GRADING/FOUNDATION DETAILS

Concrete flatwork (i.e., slabs-on-grade, sidewalks, hardscape, curbs, and gutters) should be underlain by a minimum of 12 inches of compacted engineered soil compacted to at least 95 percent relative compaction.

Import materials for fill and select backfill should meet the criteria in Table 3. Select backfill is material placed within a horizontal distance of 5 feet or one-half of the wall height, whichever is greater, behind retaining/basement walls.

Table 3 - IMPORT FILL AND SELECT BACKFILL CRITERIA

CRITERIA	IMPORT FILL	SELECT BACKFILL
Maximum particle size (inches)	4	1
Maximum liquid limit (%)	10	5
Maximum plasticity index (%)	5	0
Maximum percentage passing the No. 200 sieve (%)	40	30
Minimum sand equivalent	20	20

The soils encountered in the borings are expected to meet the above criteria for select backfill. However, during construction, we recommend to perform laboratory testing to confirm the suitability to use as select backfill criteria.

Site grading may be accomplished with conventional heavy-duty construction equipment. The fill should be compacted using soil compactors or such as a vibratory padded drum roller, as defined by the Caterpillar Performance Handbook (2001), or equivalent. However, to avoid overstressing basement or retaining walls, backfill should be compacted using lightweight compaction equipment or the walls should be braced.

4.2.2 Excavations and Temporary and Permanent Slopes

Temporary excavations as deep as 12 feet will be required for construction of a wet well. Temporary excavations should be sloped or shored. Temporary shoring will be required for the proposed structures adjacent to the existing structures or near property lines.

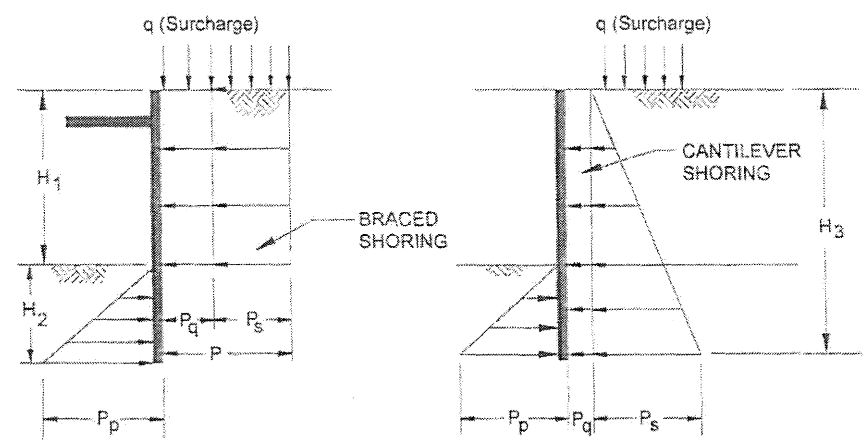
Stability of temporary excavations is a function of several factors, including the total time the excavation is exposed, moisture condition, soil type and consistency, and contractor's operations. The contractor is responsible for excavation safety. As a guideline, temporary construction excavations greater than 3 feet but less than 12 feet deep should be planned with slopes no steeper than 1.5H:1V (horizontal to vertical). For steeper temporary construction slopes or deeper excavations, shoring should be provided for stability and protection. The soil encountered in the borings indicates that the subsurface materials are highly susceptible to caving.

The support of temporary excavations is the responsibility of the contractor. Shoring is usually designed as either cantilever (unbraced) or braced. Cantilever shoring is commonly constructed by either using soldier piles with lagging placed between piles or using sheet piles. If soldier piles and lagging are used, continuous lagging is required. Difficulty in installing the lagging due to caving cohessionless soils should be anticipated.

Soils encountered in the borings were mostly silty sands and sands. Based on the Occupational Safety and Health Administration (OSHA) procedures, the soils can be classified as Soil Type C. Therefore, Soil Type C should be considered when selecting shoring in accordance with OSHA criteria.

In addition, the contractor should strictly adhere to grading requirements of the City of Pasadena and applicable federal and state health and safety regulations such as those of OSHA.

Suggested lateral earth pressures for use in shoring design are presented on Figure 4, which also includes the effect of surcharge and traffic. Temporary shoring design should incorporate the expected construction procedures, sequence, and loads. In particular, the stockpiling of excavated materials should be considered in design, as well as steel plates for cross traffic and the presence of heavy construction equipment or spoil piles next to the trenches. The shoring design is the responsibility of the Contractor and should be designed by a registered engineer retained by the Contractor. We recommend that the design of temporary shoring be performed using shoring pressures equal to or greater than those shown on Figure 4, and passive resistance equal to or less that that shown on Figure 4. The passive pressures shown on Figure 4 assume natural soils.



$P_P = 330\ H_2 < 3{,}500$ psf	BRACED SHORING $P = P_q + P_s$ $P = 0.5q + 35\ H_1$ (300 psf minimum)	CANTILEVER SHORING $P = P_q + P_s$ $= 0.35q + 37\ H_3$

Notes:

- All values of height (H) in feet, and pressure (P) and surcharge (q) in pounds per square foot (psf).
- Values for temporary excavation using flexible walls.
- For traffic surcharge, assume no less than a 100 psf uniform horizontal pressure along the top 10 feet.
- Earth pressure assumes no hydrostatic pressures. If hydrostatic pressures are slowed to building up, the incremental earth pressures below groundwater level should be reduced by 50 percent and added to hydrostatic pressure for total lateral pressure.

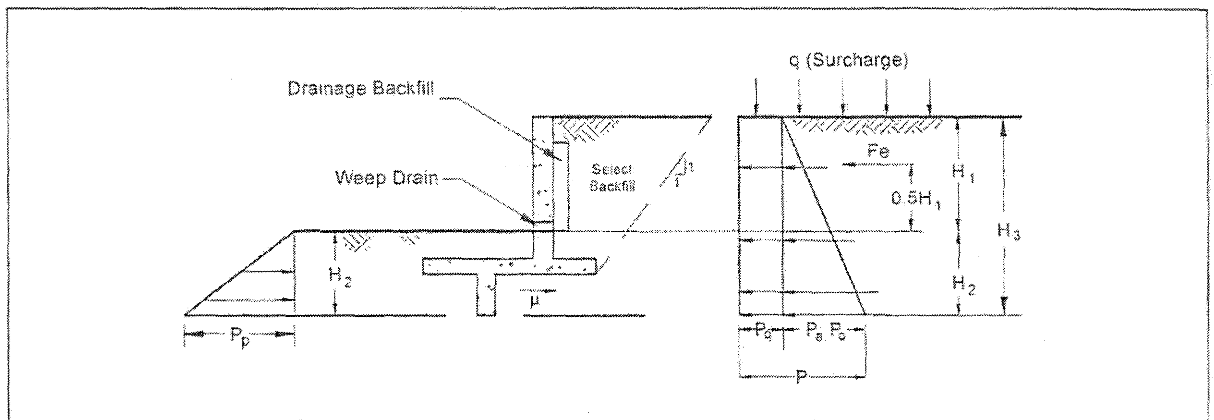
Figure 4 - LATERAL EARTH PRESSURE - TEMPORARY SHORING

4.3 FOUNDATION DESIGN

The proposed structures can be supported on shallow foundations placed on a layer of compacted fill or on natural medium dense sandy soils as shown on Figure 3. The static and temporary allowable bearing capacities include factors of safety of at least 3 and 2, respectively, against shear failure. For properly constructed foundations supported on compacted fill, total static settlement due to the proposed structural loads is estimated to be less than ½ inch. Differential static settlements between similarly loaded footings are expected to be less than ½ inch. Most of the static settlements are expected to occur as the loads are applied or shortly thereafter. The static settlements noted above are in addition to the seismic settlements noted in Section 4.1.

4.4 RESISTANCE TO LATERAL LOADS AND LATERAL EARTH PRESSURES

The lateral resistance may be calculated using 50 percent of passive resistance plus 50 percent of base friction, 100 percent passive resistance only, or 100 percent base friction only. Lateral loads can be resisted by an allowable passive soil pressure and base friction, as outlined on Figure 5 for compacted fill, applied against below-grade walls and foundation elements. Retaining and subterranean walls should be designed to resist lateral earth pressures with the equivalent fluid pressures as illustrated on Figure 5. Lateral earth pressures are presented for walls free to rotate and restrained walls. At-rest earth pressures (restrained walls) should be used for basement walls and where the top of the wall is not expected to move laterally more than 0.001 H_1 (see Figure 5). The lateral earth pressures on Figure 5 are based on site compacted soils. See Figure 6 for typical sections of wall drains.

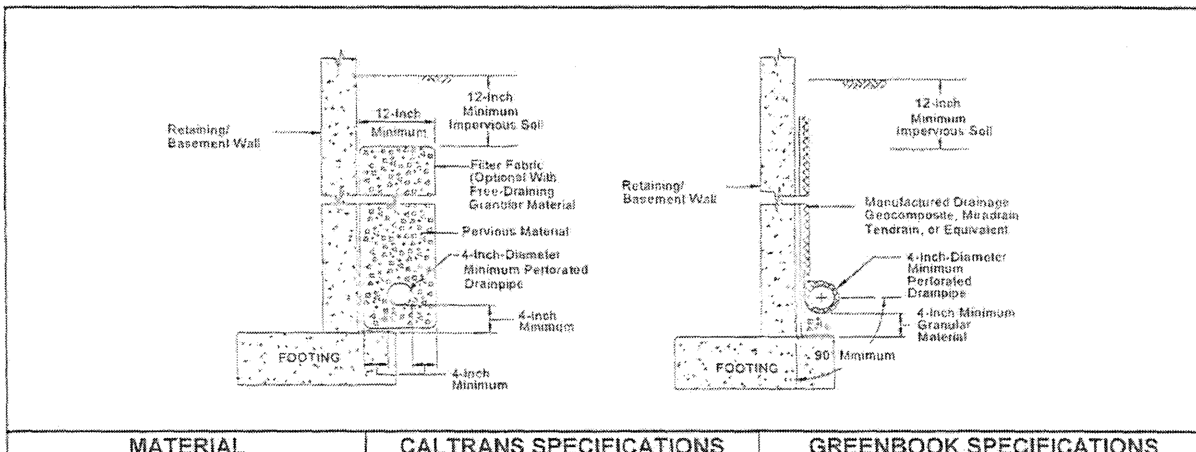


$P_p = 330\ H_2 \leq 3{,}500$ psf $\mu = 0.4$	Cantilever Walls $P = P_a + P_q = 37\ H_3 + 0.35q$ $Fe = 15\ H_1^2$	Restrained Walls $P = P_o + P_q = 55\ H_3 + 0.5q$ $Fe = 29\ H_1^2$

Notes:

- All values of height (H) in feet, pressure (P), and surcharge (q) in pounds per square foot (psf) and force (F) in pounds.
- P_p, P_a, and P_o are the passive, active, and at-rest earth pressures, respectively; Fe is the incremental seismic force.
- P_q is the incremental surcharge pressure, and μ is the allowable friction coefficient applied to dead normal (buoyant) loads. Fe is in addition to the active and at-rest pressures. Below groundwater, active and at-rest pressure should be reduced by 50 percent and hydrostatic pressure should be added to active and at-rest pressures. P_p should be reduced by 50 percent below the groundwater.
- For 2H:1V slopes above the wall, increase the active and at-rest pressures by 50 percent; for 1.5H:1V slope, increase the active and at-rest pressures by 100 percent.
- Neglect the upper 1 foot for passive pressure unless the surface is contained by a pavement or slab.
- Peak ground acceleration (PGA) was used to calculate Fe, 50 percent of the PGA for cantilever walls and 75 percent of PGA for restrained walls was used.
- See Figure 6 for drainage details.

Figure 5 - LATERAL EARTH PRESSURES - PERMANENT



MATERIAL	CALTRANS SPECIFICATIONS	GREENBOOK SPECIFICATIONS
Free-Draining Granular Material	68-1.025 (Class 2)	300-3.5.2
Geotextile Filter Fabric	68-1.03	300-8
Perforated Pipe	68-1.02	207-13.4

Notes:

- Drainpipe should drain to an outlet.
- Filter fabric wraps completely around perforated drainpipe and pervious materials.

Figure 6 - RETAINING OR BASEMENT WALL DRAINAGE

4.5 SLABS-ON-GRADE AND CONCRETE FLATWORK/MAT FOUNDATION

Slabs-on-grade should be underlain by compacted free-draining granular materials as outlined on Figure 3. The free-draining granular material should contain less than 5 percent fines (passing the No. 200 sieve) and should be placed immediately below the slab-on-grade. A modulus of subgrade (compacted subgrade) reaction of 100 pounds per cubic inch (pci) can be used for mat foundation design.

Moisture vapor will tend to migrate through the slab-on-grade. A waterproofing specialist should be consulted. To reduce vapor migration through the floor building slab, the following should be considered:

- Minimum 10-millimeter-thick plastic vapor barrier with joints overlapped by at least 6 inches and taped.
- Sealing the plastic vapor barrier around plumbing, electrical, and other conducts.
- No sand above the plastic vapor barrier.
- Minimum 7-day wet cure with no curing compounds.
- Two-month drying period before floor coverings are placed.
- Concrete mix design, materials, placement, curing, and finishing in conformance with the Greenbook and the American Concrete Institute (ACI; 1996, 1997).

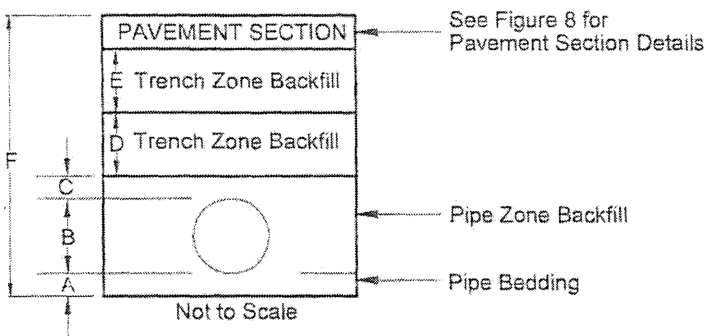
The plastic vapor barrier should satisfy the requirements of ASTM E 1745 (Class "A"). ACI 302.1R-96 defines a vapor barrier as having a water vapor transmission rate (WVTR) of 0.00, plus a testing tolerance generally of a WVTR of 0.008 or less when tested in accordance with ASTM E 96. Note that commonly used "poly" or "visqueen" does not meet ASTM E 1745 requirements. Vapor barriers should be installed in accordance to ASTM E 1643. Care should be taken to seal the plastic vapor barrier and avoid puncturing the plastic vapor barrier during construction.

4.6 UTILITY TRENCHES

Utility trenches (either open or backfilled) that parallel structures, pavement, or flatwork should be planned so that they do not extend below a plane with a downward slope of 1.5H:1V from the bottom edge of footings, pavement, or flatwork. Temporary shoring to provide footing, pavement, flatwork, or utility support is recommended unless localized settlements on the order of 1 percent of the trench depth can be tolerated.

All excavations should comply with appropriate safety standards outlined in Section 4.2.2.

Utility pipes should be placed on the bottom of a neatly cut trench on a layer of bedding as outlined on Figure 7 or according to the manufacturer's recommendations, whichever is greater. Jetting should not be allowed for compaction purposes. We anticipate that the near-surface soils will be suitable for use as bedding materials.



MATERIAL	MINIMUM THICKNESS (feet)	MINIMUM RELATIVE COMPACTION[1] (%)	BACKFILL SPECIFICATIONS	
			Caltrans Specifications	Greenbook Specifications
Pipe Bedding	A = 0.33 or B/4	--	19-3.025B	306-1.2.1
Pipe Zone	C = 1	--	19-3.025B	306-1.2.1
Trench Zone	D varies	90[2]	--	--
Trench Zone[3]	E = 2	95	--	--

Notes:
1. Based on ASTM D1557.
2. To reduce settlement, use 95 percent relative compaction.
3. E = 0 if no pavement or settlement-sensitive structures at surface.
- Minimum values; use manufacturer's recommendations if greater.

Figure 7 - PIPELINE BACKFILL SCHEMATIC

Settlement can affect the post-construction performance of trenching projects. Ground behind the shoring can settle (ground loss), yield laterally, and/or the trench backfill can settle. The contractor's operations will significantly affect both types of settlement. Some ground loss is likely with any shoring system. In general, sandy soils will experience more ground loss than clayey soils because of the tendency of sand to ravel or slough immediately upon trench excavation. If there are settlement sensitive structures within approximately 1.5 F (F = the depth of the excavation, see Figure 7), these structures should each be evaluated independently.

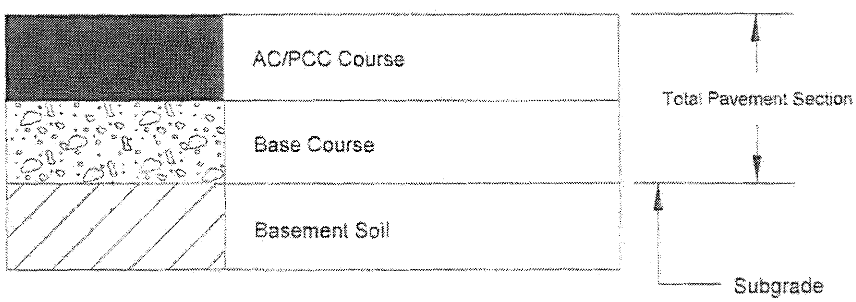
Some settlement of the backfill above the pipelines should be expected even when the backfill is properly compacted. Therefore, there should be provisions for periodic resurfacing of the trench area. Post-construction settlement can be reduced if the backfill is compacted wet of the optimum moisture content (typically 1 to 2 percent above). To reduce the potential for future settlement, it is critical that the propoer field quality control (QC) and quality assurance (QA) procedures are implemented to check that the full depth of trench backfill is compacted to at least the required relative compaction of 95 percent.

Using cement slurry backfill can also mitigate settlement. However, significant amounts of slurry are required and it will be necessary to remove and dispose of the trench excavation spoils.

4.7 PAVEMENT THICKNESS DESIGN

Recommended minimum AC and PCC pavement sections are presented on Figure 8. The recommended minimum pavement sections are based on the following:

- R-value of 50.
- Caltrans and American Association of State Highway and Transportation Officials (AASHTO) design methods for AC and PCC, respectively.
- Traffic index (TI) of 5, provided by Stetson.



COURSE	MINIMUM THICKNESS (inches)		
	AC Over Base	Full Depth AC	PCC Over Base
	TI[1] = 5	TI = 5	TI = 5
AC/PCC[2]	3	5	6
Base[3]	5	--	8
Basement Soil[4]	12	12	12

Notes:
1. TI = Traffic Index.
2. Asphalt concrete (AC) and Portland cement concrete (PCC) should satisfy the requirements of Caltrans Standard Specifications Sections 39 and 40, respectively, or Greenbook Sections 203, and 201 and 302, respectively.
3. Base course = Crushed aggregate base or crushed miscellaneous base, in accordance with Caltrans Standard Specifications Section 26 or Greenbook Sections 200-2.2 and 200-2.4, respectively. The minimum relative compaction is 95 percent.
4. Compacted in-place natural soil or fill; the minimum is 95 percent relative compaction.

Figure 8 - PAVEMENT THICKNESS

Generally, rigid PCC pavement costs more for initial construction but requires less maintenance than that for flexible AC pavements. For heavy wheel loads along limited alignments, turning areas, dolly pads, and refuse pickup areas, PCC pavement is preferred. For PCC pavements, the following should be considered:

- Construct pavements in a 15-foot square grid or smaller (20-foot rectangular areas if a square is not practical).
- Expansion joints should extend the full depth of the pavement.
- Potential joints depth of ¼ of the pavement thickness.
- Cure for a minimum of 7 days.
- No traffic until the compressive strength exceeds 2,000 pounds per square inch (psi).
- Minimum compressive strength of 3,000 psi.
- Dowels to strengthen joints.
- Minimum slope of 1 percent.

The minimum thickness of compacted subgrade is outlined on Figure 8. The subgrade soils should be firm, unyielding and not "pumping." Aggregate base requirements and specifications are outlined on Figure 8. The basement soil (subgrade) and aggregate base should be compacted to at least 95 percent relative compaction as shown on Figure 8.

4.8 SOIL CORROSION POTENTIAL

Analytical chemical test results from two tests performed during this investigation indicated 50 to 57 parts per million (ppm) soluble sulfate concentrations in the near-surface soils. Based on these test results, Type II cement should be used in accordance with the section 1904.3 of the California Building Code (CBC) 2007.

Two soil samples were tested for pH, soluble chloride and soluble sulfate, and soil electrical resistivity to check for corrosion potential. The test values are summarized in Table 4. Also presented in Table 4 are Caltrans (2003) corrosion criteria. The corrosion potential test results are presented in Appendix B. Based on Caltrans guidelines, the onsite soils are classified as noncorrosive to buried metal pipes.

Table 4 - CORROSION POTENTIAL

	TEST VALUES	CALTRANS CRITERIA FOR CORROSIVE MATERIALS
pH	7.27 and 7.29	<5.5
Soluble sulfate content (ppm)	50 and 57	>2,000
Soluble chloride content (ppm)	86 and 121	>500
Electrical resistivity (ohm-cm)	8,000 and 11,450	<1,000

5.0 PLAN REVIEW, CONSTRUCTION OBSERVATION, AND TESTING

DYA should be retained to review the finished grading earthwork and foundation plans and specifications for conformance with the intent of our recommendations. The review will enable DYA to modify the recommendations if final design conditions are different than presently understood.

During construction, DYA should provide field observation and testing to check that the site preparation, excavation, foundation installation, and finished grading conform to the intent of these recommendations, project plans, and specifications. This would allow DYA to develop supplemental recommendations as appropriate for the actual soil conditions encountered and the specific construction techniques used by the contractor.

As needed during construction, DYA should be retained to consult on geotechnical questions, construction problems, and unanticipated site conditions.

6.0 LIMITATIONS

This report has been prepared for this project in accordance with generally accepted geotechnical engineering practices common to the local area. No other warranty, expressed or implied, is made.

The analyses and recommendations contained in this report are based on the literature review, field investigation, and laboratory testing conducted in the area. The results of the field investigation indicate subsurface conditions only at the specific locations and times, and only to the depths penetrated. They do not necessarily reflect strata variations that may exist between such locations. Although subsurface conditions have been explored as part of the investigation, we have not conducted chemical laboratory testing on samples obtained or evaluated the site with respect to the presence or potential presence of contaminated soil or groundwater conditions.

The validity of our recommendations is based in part on assumptions about the stratigraphy. Observations during construction can help confirm such assumptions. If subsurface conditions different from those described are noted during construction, recommendations in this report must be re-evaluated. DYA should be retained to observe earthwork construction in order to help confirm that our assumptions and recommendations are valid or to modify them accordingly. In accordance with CBC Chapter 17 Section 1704, DYA cannot assume responsibility or liability for the adequacy of recommendations if we do not observe construction.

This report is intended for use only for the project described. In the event that any changes in the nature, design, or location of the facilities are planned, the conclusions and recommendations contained in this report should not be considered valid unless the changes are reviewed and conclusions of this report modified or verified in writing by DYA. We are not responsible for any claims, damages, or liability associated with the interpretation of subsurface data or reuse of the subsurface data or engineering analyses without our express written authorization.

7.0 BIBLIOGRAPHY

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APPENDIX A

FIELD INVESTIGATION

APPENDIX A - FIELD INVESTIGATION

The field investigation for the proposed project consisted of drilling two borings (B-1 and B-2) to depths ranging from approximately 41 to 44 feet. The approximate boring locations are shown on Figure 2.

Borings were drilled by Layne Christensen Company on May 12, 2005, with a truck-mounted CME-75 drill rig using hollow-stem auger drilling techniques. Our field engineer observed the drilling operations and collected drive samples for visual examination and subsequent laboratory testing. Drive samples were collected with a 2.4-inch-inside-diameter (3.0-inch-outside-diameter) modified California split-barrel sample lined with brass tubes and a standard split-spoon penetrometer with dimensions in accordance with ASTM 3550 and 1586, respectively. Both samplers were driven with a 140-pound hammer falling 30 inches. An automatic trip hammer was used. The blows required to drive the modified California sampler were converted to equivalent standard penetration test (SPT) N-values by multiplying by 0.65 (N = 0.65 x modified California blows per foot). The sampler blow counts were recorded in 6-inch increments. Penetration for both the modified California and SPT samplers was terminated if 50 blows were recorded with less than a 6-inch penetration. An approximate equivalent blow count was then calculated by linear extrapolation. For the modified California sampler, the extrapolated blow count was then further modified as noted above to be an equivalent SPT blow count.

Soils encountered in the borings were classified in general accordance with the ASTM Soil Classification System (ASTM D2487 and 2488), which is summarized on Plate A1. Boring logs presented on Plates A2 through A5 were prepared from visual examination of the samples, cuttings obtained during drilling operations, and results of laboratory tests.

Groundwater was encountered during the field investigation at Boring B-1 at 35 feet below the ground surface but not in Boring B-2; we suspect it could be perched water. Borings were backfilled with soil cuttings.

Boring locations were identified in the field by measuring from known locations using a measuring wheel. The geographic coordinates of the boring locations were recorded using a hand-held differential global positioning system (gps) unit with a 6-foot horizontal accuracy.

SOIL CLASSIFICATION SYSTEM-ASTM D2487

MAJOR DIVISIONS			SYMBOLS GRAPH	SYMBOLS LETTER	TYPICAL DESCRIPTIONS
COARSE-GRAINED SOILS MORE THAN 50% OF MATERIAL IS LARGER THAN NO. 200 SIEVE SIZE	GRAVEL AND GRAVELLY SOILS MORE THAN 50% OF COARSE FRACTION RETAINED ON NO. 4 SIEVE	CLEAN GRAVELS (LITTLE OR NO FINES)		GW	WELL-GRADED GRAVELS, GRAVEL - SAND MIXTURES LITTLE OR NO FINES
				GP	POORLY GRADED GRAVELS, GRAVEL - SAND MIXTURES, LITTLE OR NO FINES
		GRAVELS WITH FINES (APPRECIABLE AMOUNT OF FINES)		GM	SILTY GRAVELS, GRAVEL - SAND - SILT MIXTURES
				GC	CLAYEY GRAVELS, GRAVEL - SAND - CLAY MIXTURES
	SAND AND SANDY SOILS MORE THAN 50% OF COARSE FRACTION PASSING ON NO. 4 SIEVE	CLEAN SANDS (LITTLE OR NO FINES)		SW	WELL-GRADED SANDS, GRAVELLY SANDS, LITTLE OR NO FINES
				SP	POORLY GRADED SANDS, GRAVELLY SAND, LITTLE OR NO FINES
		SANDS WITH FINES (APPRECIABLE AMOUNT OF FINES)		SM	SILTY SANDS, SAND - SILT MIXTURES
				SC	CLAYEY SANDS, SAND - CLAY MIXTURES
FINE-GRAINED SOILS MORE THAN 50% OF MATERIAL IS SMALLER THAN NO. 200 SIEVE SIZE	SILTS AND CLAYS	LIQUID LIMIT LESS THAN 50		ML	INORGANIC SILTS AND VERY FINE SANDS, ROCK FLOUR, SILTY OR CLAYEY FINE SANDS OR CLAYEY SILTS WITH SLIGHT PLASTICITY
				CL	INORGANIC CLAYS OF LOW TO MEDIUM PLASTICITY, GRAVELLY CLAYS, SANDY CLAYS, SILTY CLAYS, LEAN CLAYS
				OL	ORGANIC SILTS AND ORGANIC SILTY CLAYS OF LOW PLASTICITY
	SILTS AND CLAYS	LIQUID LIMIT GREATER THAN 50		MH	INORGANIC SILTS, MICACEOUS OR DIATOMACEOUS FINE SAND OR SILTY SOILS
				CH	INORGANIC CLAYS OF HIGH PLASTICITY
				OH	ORGANIC CLAYS OF MEDIUM TO HIGH PLASTICITY, ORGANIC SILTS
HIGHLY ORGANIC SOILS				PT	PEAT, HUMUS, SWAMP SOILS WITH HIGH ORGANIC CONTENTS

NOTE: DUAL SYMBOLS ARE USED TO INDICATE BORDERLINE SOIL CLASSIFICATIONS

"Push" Sampler

Split Barrel "Drive" Sampler With Liner

Standard Penetration Test (SPT) Sampler

Bag Sample

Concrete/Rock Core

Groundwater Surface

SP

NP = Nonplastic
EIT = Expansion Index Test
SG = Specific Gravity
SE = Sand Equivalent
UC = Unconfined Comp.
CD = Consol. Drained Comp.
CU = Consol. Undrained Comp.
UU = Undrained, Unconsol. Comp.
RV/CBR = R-Value/California Bearing Ratio
CHEM = Chemical Analysis
DS = Direct Shear
CON = Consolidation
SA = Grain size; HYD = Hydrometer
COMP= Compaction Test
[PID] Reading in ppm above background

KEY TO LOG OF BORINGS

Pasadena Ion Exchange Treatment System for Perchlorate Removal
Project No. 2005-017

PLATE
A1

BORING LOCATION:	See Figure 2	ELEVATION AND DATUM (feet):	920 MSL
LATITUDE:	34° 9' 25.7" N	LONGITUDE:	118° 9' 11.8" W
DRILLING EQUIPMENT:	CME-75	DRILLING METHOD:	Hollow Stem Auger
BORING DIAMETER (inches):	6	BORING DEPTH (feet):	42
DATE STARTED:	5/12/05	DATE COMPLETED:	5/12/05
SPT HAMMER DROP:	30 inches WT: 140 lbs	DRIVE HAMMER DROP:	30 inches WT: 140 lbs
LOGGED BY: RSZ	CHECKED BY: SN	DRIVE SAMPLER DIAMETER (inches)	ID: 2.4 OD: 3

Elevation (feet)	Depth (feet)	Sampler	Symbol	Blows per 6 Inches	SPT N Blows per Foot	Field Unc. Comp. Str. (tsf)	DESCRIPTION	Dry Density (pcf)	Moisture Content (%)	Liquid Limit (%)	Plasticity Index (%)	Percent Passing #200 Sieve	Other Tests [PID]
							PORTLAND CEMENT CONCRETE - 5 inches						
							SILTY SAND (SM); dark brown, moist, medium dense, fine-to coarse-grained sand, fine to coarse gravel						
				7 12 10	14								CHEM
915	5			6 6 7	8		loose	100	12			24	
910	10			5 6 8	14		dark brown, moist, medium dense, fine-to coarse-grained sand, fine gravel						
905	15			21 19 12	20		olive brown	122	10				DS
900	20			3 6 21	27		decreased fines					17	
							POORLY GRADED SAND with SILT (SP-SM); olive brown, moist, very dense, fine- to coarse-grained sand, trace fine gravel						
895	25			14 38 50/5 inches	64			128	9				
							SILTY SAND (SM); brown, moist, medium dense, fine-grained sand						

LOG OF BORING B-1

Pasadena Ion Exchange Treatment System for Perchlorate Removal

Project No. 2005-017

PLATE

A2

Template: DYLG1; Prj ID: 2005-017.GPJ

Elevation (feet)	Depth (feet)	Sampler	Symbol	Blows per 6 Inches	SPT N Blows per Foot	Field Unc. Comp. Str. (tsf)	DESCRIPTION	Dry Density (pcf)	Moisture Content (%)	Liquid Limit (%)	Plasticity Index (%)	Percent Passing #200 Sieve	Other Tests [PID]
				12 8 7	15								
885	35			9 14 50/5 inches	48		▽ dense, wet, decreased fines	109	18				
							POORLY GRADED SAND (SP): brown, wet, dense, fine- to coarse-grained sand, trace fine gravel						
880	40			19 24 24	48								
							Bottom of boring at 41.5 feet. Groundwater encountered at 35 feet. Boring backfilled with cement bentonite slurry and cutting. Surface patched with rapid set concrete.						
875	45												
870	50												
865	55												
860	60												
855	65												

LOG OF BORING B-1

Pasadena Ion Exchange Treatment System for Perchlorate Removal

Project No. 2005-017

PLATE

A3

Template: DYLG1; Prj ID: 2005 017.GPJ

BORING LOCATION:	See Figure 2	ELEVATION AND DATUM (feet):	920 MSL
LATITUDE:	34° 9' 25.3" N	LONGITUDE:	118° 9' 12.0" W
DRILLING EQUIPMENT:	CME-75	DRILLING METHOD:	Hollow Stem Auger
BORING DIAMETER (inches):	6	BORING DEPTH (feet):	45
DATE STARTED:	5/12/05	DATE COMPLETED:	5/12/05
SPT HAMMER DROP:	30 inches WT: 140 lbs	DRIVE HAMMER DROP:	30 inches WT: 140 lbs
LOGGED BY: RSZ	CHECKED BY: SN	DRIVE SAMPLER DIAMETER (inches)	ID: 2.4 OD: 3

Elevation (feet)	Depth (feet)	Sampler	Symbol	Blows per 6 Inches	SPT N Blows per Foot	Field Unc. Comp. Str. (tsf)	DESCRIPTION	Dry Density (pcf)	Moisture Content (%)	Liquid Limit (%)	Plasticity Index (%)	Percent Passing #200 Sieve	Other Tests [PID]
							PORTLAND CEMENT CONCRETE - 6 inches						
							SILTY SAND (SM); dark brown, moist, loose, fine- to medium-grained sand, trace fine gravel						
				3 5 7	8			108	9	19	2	28	COMP CHEM RV
915	5			6 9 14	15		dark gray, medium dense, fine- to coarse-grained sand, trace fine to coarse gravel	95	9				
				7 7 9	16		WELL GRADED GRAVEL with SAND and SILT (GW-GM); grayish brown, medium dense, fine to coarse gravel, fine- to coarse-grained sand						
910	10			12 25 17	27			115	3			5	SA
				11 17 23	40		light olive brown, dense, fine- to coarse-grained sand, trace fine gravel						
905	15			19 36 39	49		SILTY SAND (SM); olive brown, dense, moist, fine- to coarse-grained sand, trace fine gravel	120	4				
				4 7 7	14		light brown, medium dense					21	
900	20			15 29 31	39		dense						
				9 17 17	34								
895	25			14 18 46	42		olive brown	108	6				
				8 20 26	46		dark brown, increased fines						

LOG OF BORING B-2

Pasadena Ion Exchange Treatment System for Perchlorate Removal

Project No. 2005-017

PLATE

A4

Elevation (feet)	Depth (feet)	Sampler	Symbol	Blows per 6 Inches	SPT N Blows per Foot	Field Unc. Comp. Str. (tsf)	DESCRIPTION	Dry Density (pcf)	Moisture Content (%)	Liquid Limit (%)	Plasticity Index (%)	Percent Passing #200 Sieve	Other Tests [PID]
				17 33 50/5 inches	60		very desne						
885	35						POORLY GRADED SAND with SILT (SP-SM): olive brown, moist, very dense, fine- to coarse-grained sand, trace fine gravel						
				16 23 28	51								
880	40												
				27 60/6 inches	78								
							Boring terminated at 44 feet due to refusal. Groundwater not encountered. Boring backfilled with cuttings. Surface patched with rapid set concrete.						
875	45												
870	50												
865	55												
860	60												
855	65												

LOG OF BORING B-2

Pasadena Ion Exchange Treatment System for Perchlorate Removal

Project No. 2005-017

PLATE

A5

APPENDIX B - LABORATORY TESTING

Diaz•Yourman & Associates (DYA) selected soil samples to be tested and the tests to be performed on the selected samples. Laboratory testing was performed by Leighton Consulting. Laboratory data are summarized on the boring logs in Appendix A and presented on Plates B1 through B6. We have reviewed and concur with the test results and accept full responsibility for their use in our analysis. A summary of the geotechnical laboratory testing is presented in Table B1. Corrosion potential test results are summarized in Table B2.

Table B1 - LABORATORY TESTING SUMMARY

TEST NAME	PROCEDURE	PURPOSE	LOCATION
Percent Passing the No. 200 Sieve	ASTM D1140-92	Classification, index properties	Boring Logs
Moisture Content, Dry Density	ASTM D2216-92	Classification, index properties	Boring Logs
Grain-Size Distribution	ASTM D422-63	Classification, index properties	Plate B1
Atterberg Limits	ASTM D-4318-93	Expansion potential, classification, index properties	Plate B2
Direct Shear	ASTM D3080-90	Shear strength	Plate B3
Compaction	ASTM D1557-91	Earthwork	Plate B4
Resistance (R-) Value	ASTM D2844-69 CTM 301	Pavement thickness design	Plate B5
pH	CTM 532	Corrosion potential	Table B2 and Plate B6
Resistivity	CTM 532	Corrosion potential	Table B2 and Plate B6
Soluble Sulfates	CTM 417-B	Corrosion potential	Table B2 and Plate B6
Soluble Chlorides	CTM 422	Corrosion potential	Table B2 and Plate B6

Notes:

- ASTM = American Society for Testing and Materials
- CTM = Caltrans Test Method

Table B2 - LABORATORY TESTING SUMMARY

Boring No.	B-1	B-2
Depth (feet)	0 to 5	0 to 5
pH	7.27	7.29
Water Soluble Sulfate Content (ppm)	57	50
Water Soluble Chloride Content (ppm)	86	121
Minimum Resistivity (ohms-cm)	8,000	11,450

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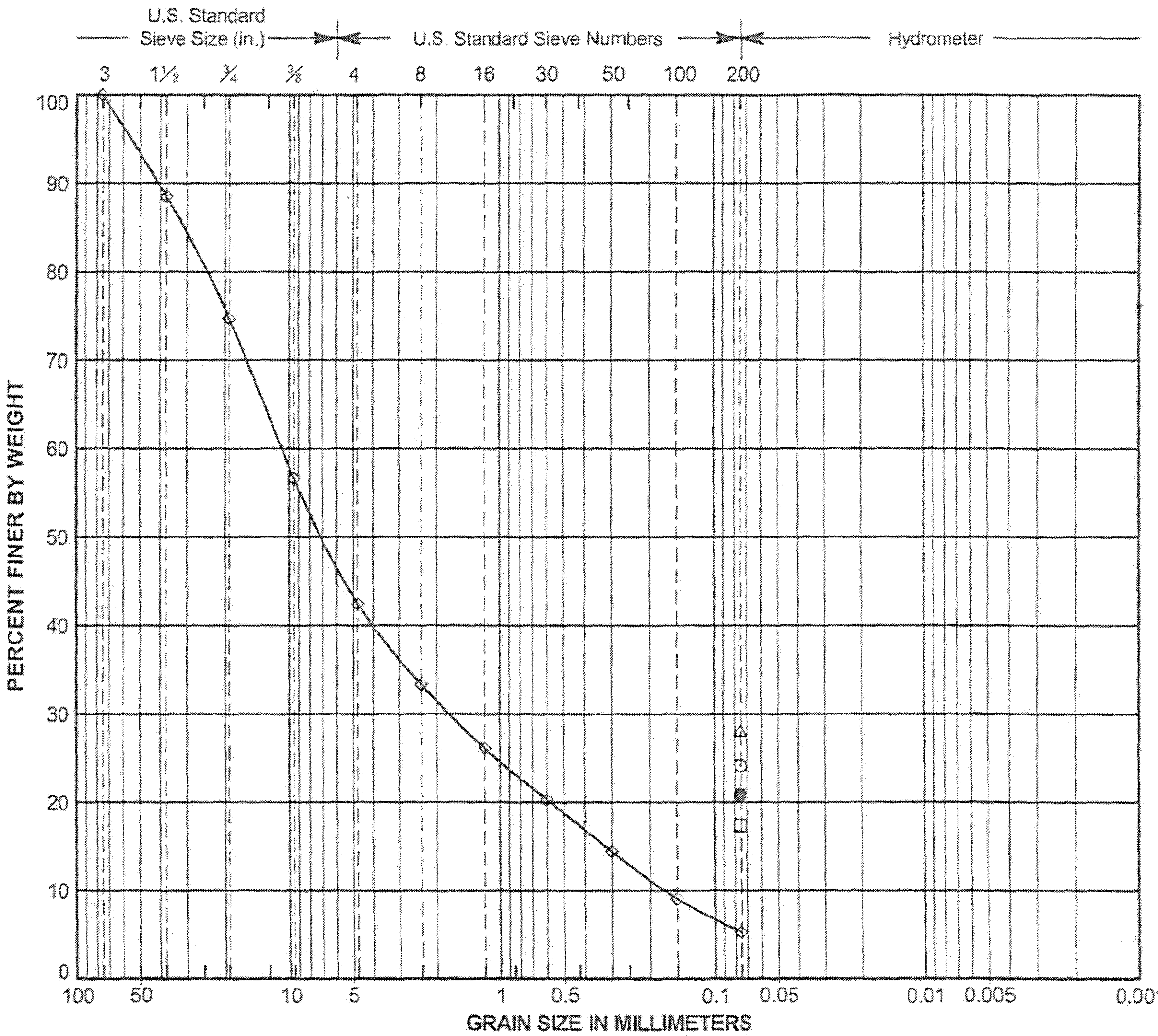


COBBLES	GRAVEL		SAND			SILT or CLAY
	Coarse	Fine	Coarse	Medium	Fine	

Laboratory Testing by: Leighton Consulting, Inc.

Symbol	Source	Depth (feet)	Classification	Natural M. C. (%)	Liquid Limit (%)	Plasticity Index (%)	% Passing #200 Sieve
⊙	B- 1	5.0	SILTY SAND (SM)	12			24
□	B- 1	20.0	SILTY SAND (SM)				17
△	B- 2	2.5	SILTY SAND (SM)	9	19	2	28
◇	B- 2	10.0	WELL-GRADED GRAVEL with SILT (GW-GM)	3			5
●	B- 2	17.5	SILTY SAND (SM)				21

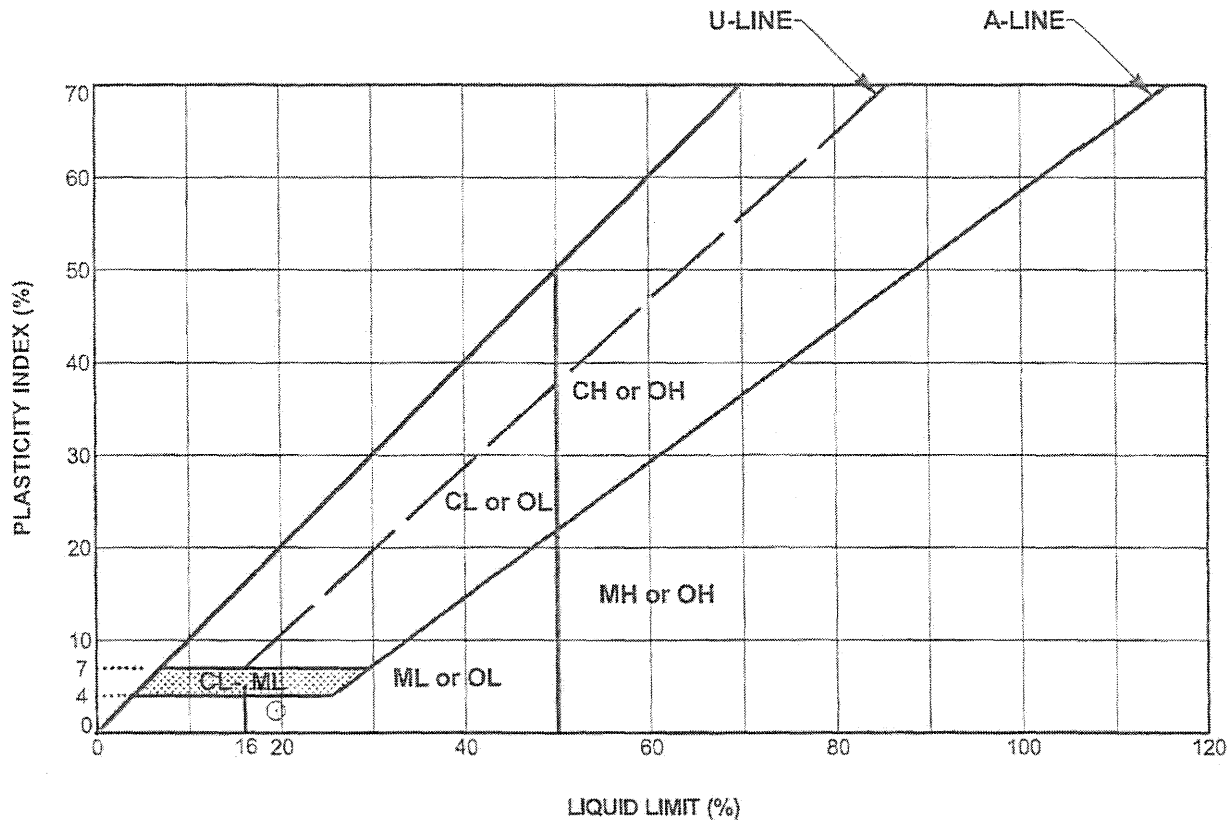
PARTICLE SIZE ANALYSIS

Pasadena Ion Exchange Treatment System for Perchlorate Removal

Project No. 2005-017

PLATE

B1



Laboratory Testing by: Leighton Consulting, Inc.

Symbol	Source	Depth (feet)	Classification	Natural M. C. (%)	Liquid Limit (%)	Plastic Limit (%)	Plasticity Index (%)	% Passing #200 Sieve
⊙	B- 2	2.5	SILTY SAND (SM)	9	19	17	2	28

PLASTICITY CHART

Pasadena Ion Exchange Treatment System for Perchlorate Removal
Project No. 2005-017

PLATE
B2

MODIFIED PROCTOR COMPACTION TEST

ASTM D 1557

Leighton

Project Name:	Pasadena IX Treatment	Tested By :	PP	Date:	05/21/05
Project No.:	2005-017	Input By :	LF	Date:	05/25/05
Boring No.:	B-2	Depth (ft.)	3		
Sample No. :	N/A				
Soil Identification:	Brown Silty Sand with Gravel (SM)g				

Preparation Method:	X	Moist		
		Dry		
Compaction Method	X	Mechanical Ram		
		Manual Ram		

Scalp Fraction (%)	
#3/4	14.7
#3/8	
#4	

Rammer Weight (lb.) = 10.0

Height of Drop (in.) = 18.0

Mold Volume (ft³) 0.07513

TEST NO.	1	2	3	4	5	6
Wt. Compacted Soil + Mold (g)	6849.7	7074.8	7278.6	7223.5		
Weight of Mold (g)	2319.0	2319.0	2319.0	2319.0		
Net Weight of Soil (g)	4530.7	4755.8	4959.6	4904.5		
Wet Weight of Soil + Cont. (g)	1238.00	1213.30	1186.90	1061.60		
Dry Weight of Soil + Cont. (g)	1208.10	1161.40	1111.10	980.40		
Weight of Container (g)	72.90	79.40	82.60	74.60		
Moisture Content (%)	2.63	4.80	7.37	8.96		
Wet Density (pcf)	132.9	139.6	145.5	143.9		
Dry Density (pcf)	129.5	133.2	135.5	132.1		

Maximum Dry Density (pcf) 135.5

Optimum Moisture Content (%) 7.0

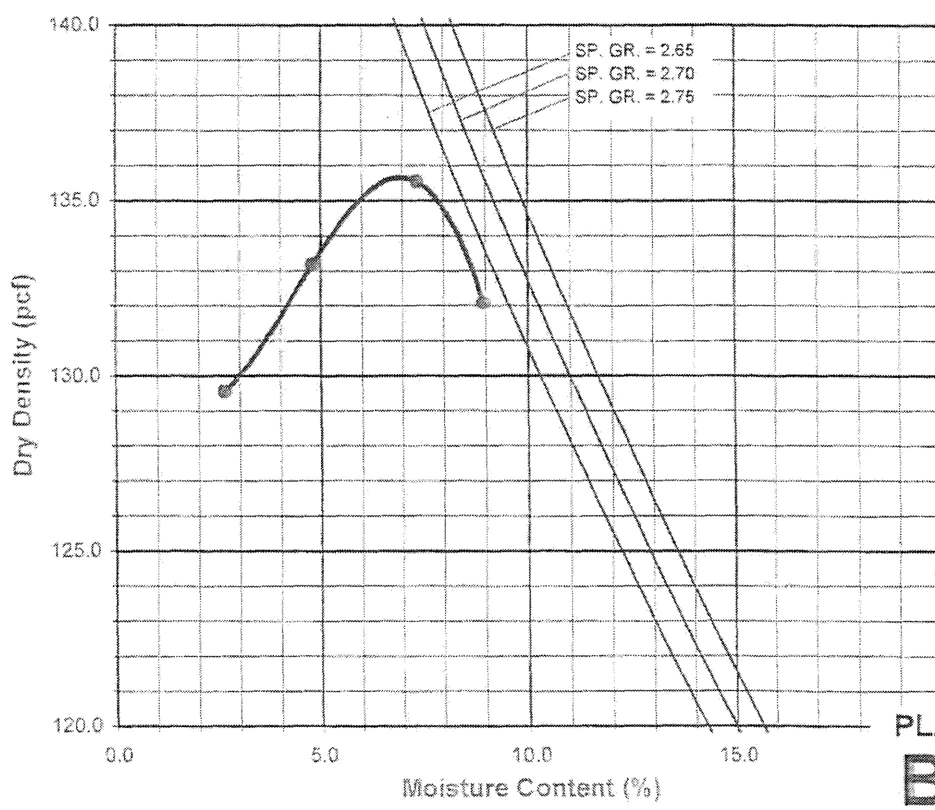
Corrected Dry Density (pcf) 139.5

Corrected Moisture Content (%) 6.0

☐ **Procedure A**
Soil Passing No. 4 (4.75 mm) Sieve
Mold : 4 in. (101.6 mm) diameter
Layers : 5 (Five)
Blows per layer : 25 (twenty-five)
May be used if +#4 is 20% or less

☐ **Procedure B**
Soil Passing 3/8 in. (9.5 mm) Sieve
Mold : 4 in. (101.6 mm) diameter
Layers : 5 (Five)
Blows per layer : 25 (twenty-five)
Use if +#4 is >20% and +3/8 in. is 20% or less

☒ **Procedure C**
Soil Passing 3/4 in. (19.0 mm) Sieve
Mold : 6 in. (152.4 mm) diameter
Layers : 5 (Five)
Blows per layer : 56 (fifty-six)
Use if +3/8 in. is >20% and +¾ in. is <30%



Particle-Size Distribution:

GR:SA:FI

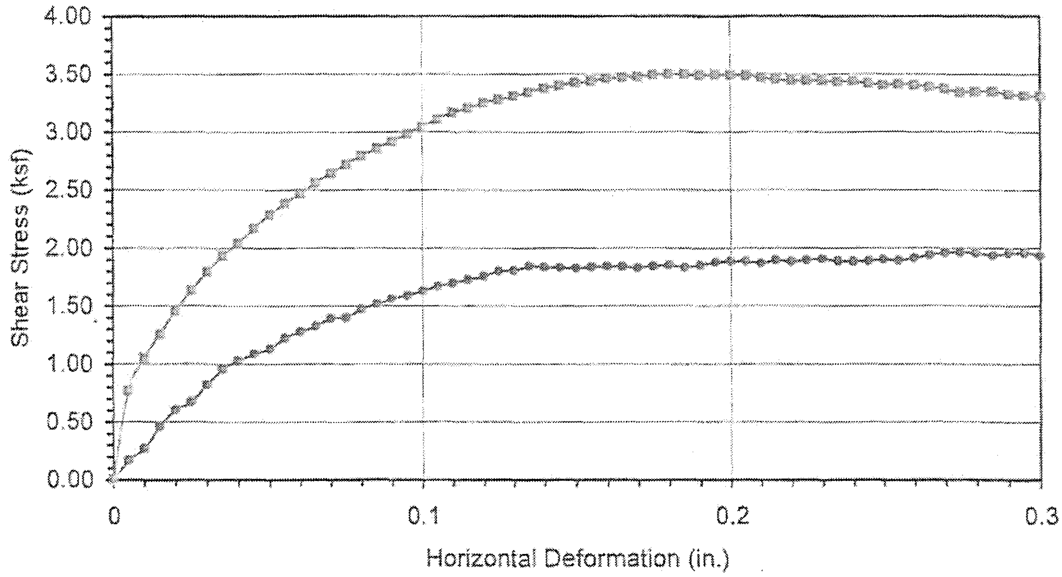
Atterberg Limits:

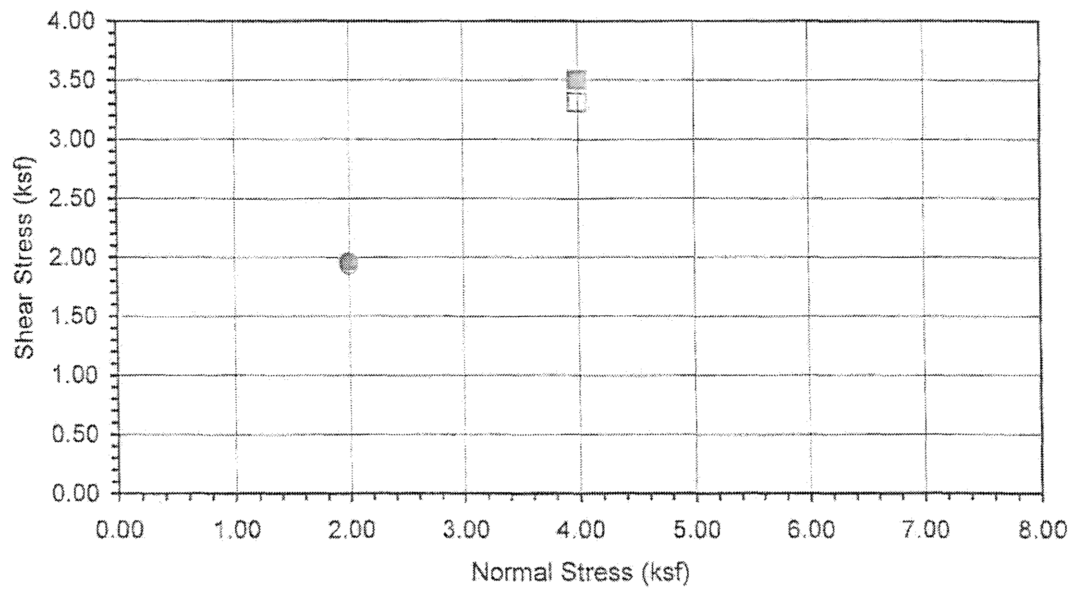
LL,PL,PI

PLATE

B3

MX w Rc Cor b-2 @ 3'





Boring No.	B-1
Sample No.	4
Depth (ft)	15
Sample Type: Drive	
Soil Identification: Olive Silty Sand (SM) / Large Gravel Removed	

Normal Stress (kip/ft²)	2.000	4.000	
Peak Shear Stress (kip/ft²)	● 1.961	■ 3.501	▲
Shear Stress @ End of Test (ksf)	○ 1.930	□ 3.311	△
Deformation Rate (in./min.)	0.0033	0.0033	
Initial Sample Height (in.)	1.000	1.000	
Diameter (in.)	2.415	2.415	
Initial Moisture Content (%)	10.29	10.29	
Dry Density (pcf)	107.6	107.8	
Saturation (%)	49.0	49.4	
Soil Height Before Shearing (in.)	0.9830	0.9715	
Final Moisture Content (%)	18.2	17.8	

Leighton

DIRECT SHEAR TEST RESULTS
Consolidated Drained - ASTM D 3080

Project No.: 2005-017

Pasadena IX Treatment

PLATE
B4

Leighton

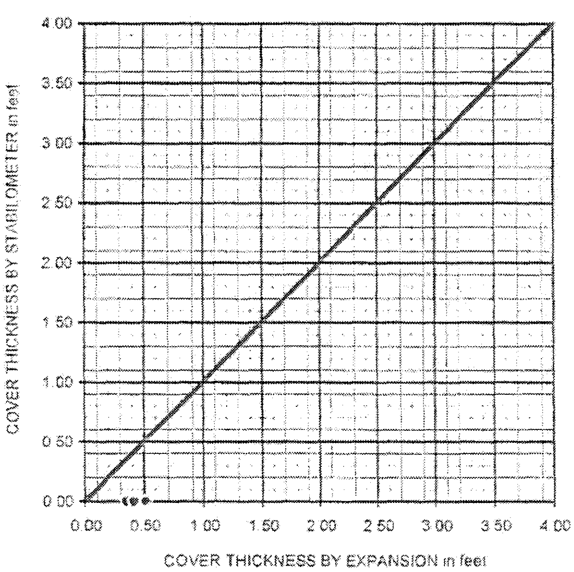
R-VALUE TEST RESULTS

PROJECT NAME:	Pasadena IX Treatment	PROJECT NUMBER:	2005-017
SAMPLE NUMBER:	N/A	SAMPLE LOCATION:	B-2 @ 3
SAMPLE DESCRIPTION:	(SM)g	TECHNICIAN:	SCF
		DATE SAMPLED	5/18/2005

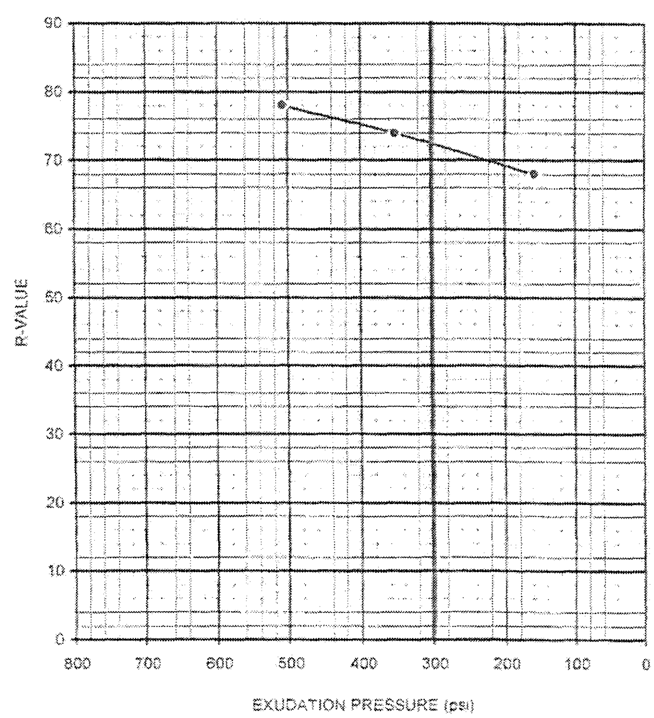
TEST SPECIMEN	a	b	c
MOISTURE AT COMPACTION %	8.1	8.3	8.5
HEIGHT OF SAMPLE, Inches	2.44	2.46	2.45
DRY DENSITY, pcf	132.9	132.5	131.8
COMPACTOR PRESSURE, psi	300	260	175
EXUDATION PRESSURE, psi	507	352	157
EXPANSION, Inches x 10exp-4	0	0	0
STABILITY Ph 2,000 lbs (160 psi)	21	26	32
TURNS DISPLACEMENT	4.40	4.62	4.80
R-VALUE UNCORRECTED	79	74	68
R-VALUE CORRECTED	78	74	68

DESIGN CALCULATION DATA	a	b	c
GRAVEL EQUIVALENT FACTOR	1.0	1.0	1.0
TRAFFIC INDEX	5.0	5.0	5.0
STABILOMETER THICKNESS, ft.	0.35	0.42	0.51
EXPANSION PRESSURE THICKNESS, ft.	0.00	0.00	0.00

EXPANSION PRESSURE CHART



EXUDATION PRESSURE CHART



R-VALUE BY EXPANSION:	100
R-VALUE BY EXUDATION:	72
EQUILIBRIUM R-VALUE:	72

PLATE
B5

Leighton

SOIL RESISTIVITY TEST

DOT CA TEST 532 / 643

Project Name:	Pasadena IX Treatment	Tested By :	GB
Project No. :	2005-017	Data Input By:	LF
Boring No.:	B-1	Depth (ft.) :	3.0
Sample No. :	N/A		
Soil Identification:	(SM)g		

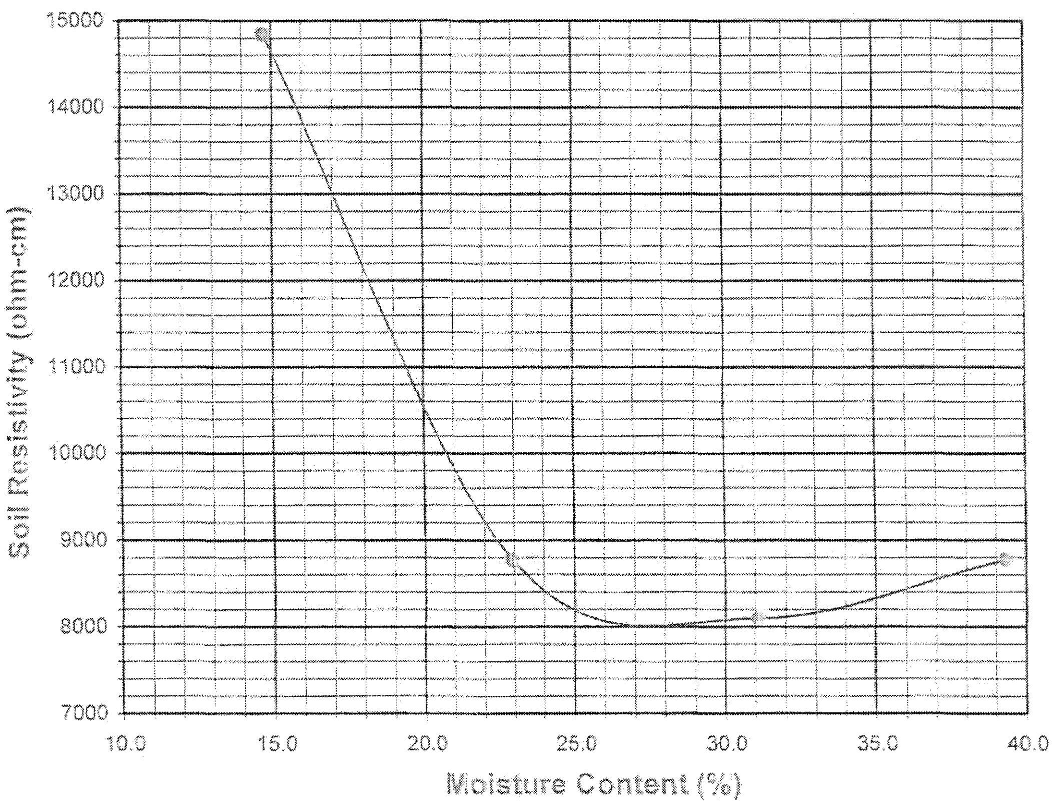
Date: 05/21/05

Date: 05/24/05

Specimen No.	Water Added (ml) (Wa)	Adjusted Moisture Content (MC)	Resistance Reading (ohm)	Soil Resistivity (ohm-cm)
1	100	14.74	2200	14841
2	200	22.93	1300	8770
3	300	31.13	1200	8095
4	400	39.32	1300	8770
5				

Moisture Content (%) (MCi)	6.54
Wet Wt. of Soil + Cont. (g)	226.45
Dry Wt. of Soil + Cont. (g)	216.19
Wt. of Container (g)	59.35
Container No.	
Initial Soil Wt. (g) (Wt)	1300.00
Box Constant	6.746
MC =(((1+Mci/100)x(Wa/Wt+1))-1)x100	

Min. Resistivity (ohm-cm)	Moisture Content (%)	Sulfate Content (ppm)	Chloride Content (ppm)	Soil pH	
				pH	Temp. (°C)
DOT CA Test 532 / 643		DOT CA Test 417 Part II	DOT CA Test 422	DOT CA Test 532 / 643	
8000	**28.0**	**57**	**86**	**7.27**	**21.6**



PLATE

B6

Leighton

SOIL RESISTIVITY TEST

DOT CA TEST 532 / 643

Project Name:	Pasadena IX Treatment	Tested By :	GB	Date:	05/21/05
Project No. :	2005-017	Data Input By:	LF	Date:	05/24/05
Boring No.:	B-2	Depth (ft.) :	3.0		
Sample No. :	N/A				
Soil Identification:	(SM)g				

Specimen No.	Water Added (ml) (Wa)	Adjusted Moisture Content (MC)	Resistance Reading (ohm)	Soil Resistivity (ohm-cm)
1	100	17.21	1900	12817
2	200	25.58	1700	11468
3	300	33.95	1800	12143
4				
5				

Moisture Content (%) (MCi)	8.83
Wet Wt. of Soil + Cont. (g)	217.11
Dry Wt. of Soil + Cont. (g)	204.17
Wt. of Container (g)	57.68
Container No.	
Initial Soil Wt. (g) (Wt)	1300.00
Box Constant	6.746
MC =(((1+Mci/100)x(Wa/Wt+1))-1)x100	

Min. Resistivity (ohm-cm)	Moisture Content (%)	Sulfate Content (ppm)	Chloride Content (ppm)	Soil pH: pH	Soil pH: Temp. (°C)
DOT CA Test 532 / 643		DOT CA Test 417 Part II	DOT CA Test 422	DOT CA Test 532 / 643	
11450	**26.3**	**50**	**121**	**7.29**	**21.7**

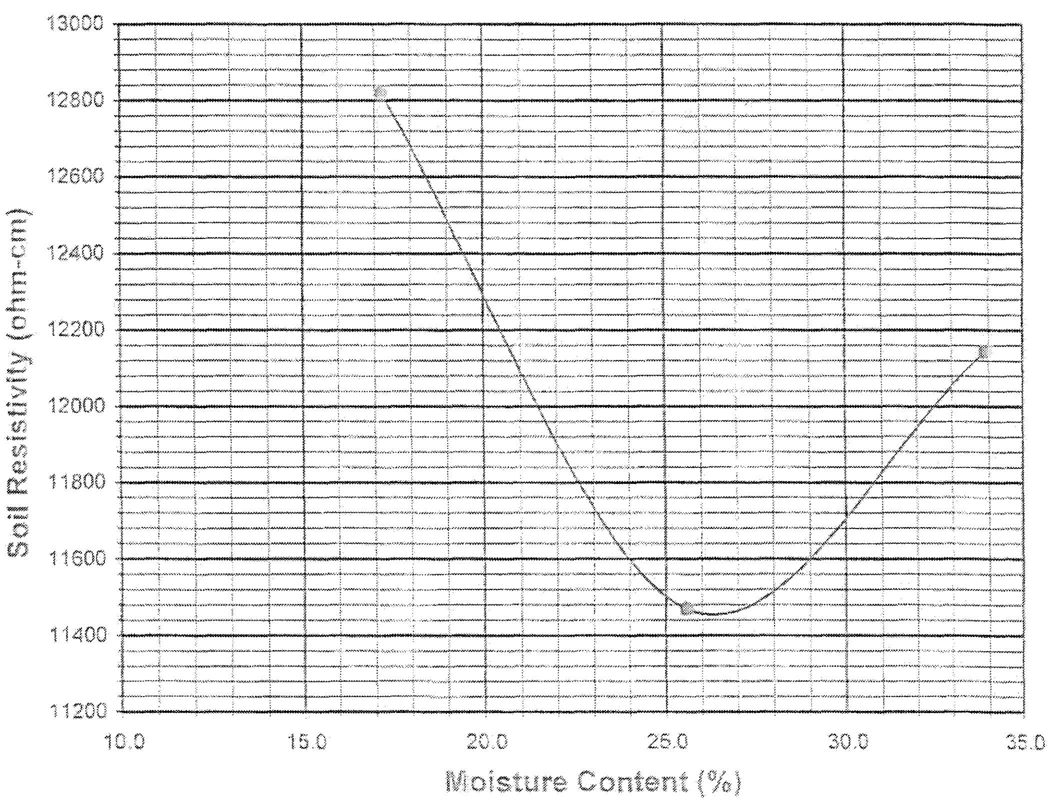


PLATE
B7

DISTRIBUTION

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QUALITY CONTROL REVIEWER

Allen M. Yourman, Jr., P.E., G.E.
Principal

AMY/SN:cfp