FOUNDATION REPORT

DEL AMO BOULEVARD EXTENSION AND GRADE SEPARATION OVER BNSF RAILROAD CITY OF TORRANCE, CALIFORNIA

Submitted to:

Moffatt & Nichol Engineers
220 Commerce
Suite 250
Irvine, California 92602

Submitted by:

AMEC Earth & Environmental, Inc.
1290 N. Hancock Street
Suite 102
Anaheim, California 92807

July 11, 2006

Job No. 5-212-100600

AMEC Earth & Environmental, Inc.
1290 North Hancock Street, Suite 102
Anaheim, California
USA 92807-1924
Tel (714) 779-2591
Fax (714) 779-8377

TABLE OF CONTENTS

Page

TABLE OF CONTENTS

Page

LIST OF APPENDICES

PLATES

1 INTRODUCTION

This Foundation Report presents the results and recommendations of the geotechnical study for the proposed Del Amo Boulevard Extension in the City of Torrance, California. The purpose of this study was to evaluate the general soil conditions in the area of the proposed extension and provide design and construction recommendations to aid the design team in the preparation of project plans and specifications.

1.1 EXISTING INFORMATION

The following references were used in preparation of this report.

- Del Amo Boulevard Extension Project Recirculated Draft EIR/EA, City of Torrance, Public Works Department, Engineering Division, August 13, 2003.

- Geotechnical Study, Dry Ice Plant, Airco Facility, 2535 Del Amo Boulevard, Torrance, California, T.W. Cooper, Inc., January 22, 1987.

- Foundation Investigation, Proposed CO2 Plant, Mobil Oil Corporation Refinery, Torrance, California, Dames & Moore Job 095-201-02, April 29, 1977.

- Seismic Hazard Evaluation of The Torrance 7.5-Minute Quadrangle, Los Angeles, California, 1998, Department of Conservation, Division of Mines and Geology. Open File Report 98-26.

- Project Study Report for extension of Del Amo Boulevard, Holmes & Narver, September 20, 1999.

1.2 PROPOSED CONSTRUCTION

Historically, Del Amo Boulevard has not been continuous between Crenshaw Boulevard and Maple Avenue. This discontinuous portion is currently being utilized as right-of-way access for the adjacent ExxonMobil Refinery and the Dow Chemical Manufacturing plant. Current plans call for new road and bridge construction within the existing right-of-way access in order to complete Del Amo Boulevard as a continuous roadway.

Present plans calls for extending Del Amo Boulevard about ¾ mile from Crenshaw Boulevard to Maple Avenue. In addition, an approximately ½ mile segment of the existing Del Amo Boulevard between Maple Avenue and Prairie Avenue is to be widened. As part of the extension and widening, the following construction is proposed:

- Construction of a single span bridge over the existing BNSF railroad. The bridge is to be about 116 feet in length and 84 feet in width with tall abutment walls. Approach fills are to be confined with MSE wall type structures at the abutments.

- Realignment of the existing railroad spur located at the southern boundary of the ExxonMobil refinery.

- Mechanically Stabilized Embankment (MSE) type retaining walls are proposed in order to support portions of the fill embankment required for the bridge approach. The MSE walls will range from about 5 to 30 feet in height and will be located as shown on the plans.

- Embankment slopes are proposed for additional locations of approach fill.

- Construction of new structural pavement for a four lane roadway with a total right-of-way width of about 100 feet.

1.3 SCOPE OF STUDY

The geotechnical study performed by AMEC Earth & Environmental, Inc. (AMEC) included subsurface exploration, soil sampling, laboratory testing, engineering analyses, development of design recommendations and preparation of this report. The scope of work included performance of the following tasks:

1. Site reconnaissance and lay out of the test boring locations.

2. Review of available data for project vicinity. The available data is listed in Section 1.1 of this report.

3. Drilling of eight (8) test borings in the proximity of the proposed improvements.

4. Classification and logging of substrata encountered in each test boring at the time of drilling.

5. Performance of standard penetration tests in each test boring.

6. Obtaining in-situ samples of substrata from the test borings.

7. Observation of groundwater conditions in the test borings at the time of drilling.

8. Performance of laboratory tests on selected soil samples.

9. Analysis and interpretation of field and laboratory test results.

10. Evaluation of regional geology and engineering seismology.

11. Development of seismic design parameters for ground motion at this site.

12. Engineering analyses and development of design recommendations for the support of vertical and lateral loads for the proposed bridge and retaining walls.

13. Evaluation of corrosion potential of subsurface soils.

14. Preparation of comments and recommendations for construction specifications and considerations.

15. Preparation of a written report, documenting the work performed, physical data acquired, and geotechnical design recommendations.

2 FIELD AND LABORATORY STUDY

The field and laboratory studies included site reconnaissance, subsurface exploration and performance of laboratory tests on selected samples. This phase of the study was completed in July 2005.

2.1 SUBSURFACE EXPLORATION

The field study consisted of drilling six (6), 6-inch diameter, hollow stem auger borings and two (2), 5-inch diameter, mud rotary wash borings to depths ranging from 13.5 to 101.5 feet. The borings were advanced in order to obtain soil samples and enable evaluation of subsurface conditions. The boring locations advanced for the geotechnical evaluation of the bridge structure (Borings B-1 and B-2) are shown on the Log of Test Borings presented at the rear of this report. The locations of the remaining borings are shown on the Boring Location Plan, Plate II. The location and ground elevation were estimated from the layout plan prepared by Moffatt & Nichol. Details concerning the drilling operations are presented on the Test Boring Logs in Appendix A.

Both bulk and relatively undisturbed in-situ samples of encountered soil types were obtained from the borings for laboratory testing and examination. The in-situ samples were obtained by driving a 2.5-inch I.D. ring sampler with a 300-lb. weight dropping about 12 inches in the rotary wash borings and with a 140-lb. hammer dropping about 30 inches in the hollow stem auger borings. Standard penetration tests were conducted using a 1.4 inch I.D. standard penetration sampler driven with a 140 lb. hammer dropping 30 inches in general conformance with ASTM D1586 procedures. Sample size, depth and penetration rate are shown on the Log of Test Borings.

The drilling and sampling operations were performed under the supervision of an experienced engineer who logged the borings and prepared the samples for subsequent examination and laboratory testing. Earth materials were visually classified in the field in general accordance with the Unified Soil Classification System by observation of the samples and boring returns. A description of this classification system is presented in Appendix A.

2.2 LABORATORY TESTING

Laboratory tests were performed to provide a basis for design recommendations. Selected samples retrieved from the borings were tested to evaluate in-situ moisture content and dry density, shear strength, sulphate content, chloride content, pH, minimum electrical resistivity, expansion potential, R-value, consolidation, grain size distribution, and maximum density. Test results and descriptions of the laboratory testing procedures are presented in Appendix B. The results of the in-situ moisture-density tests are shown on the Log of Test Borings, Plate I and the Test Boring Logs in Appendix A.

3 SITE CONDITIONS

3.1 SURFACE CONDITIONS

The proposed extension of Del Amo Boulevard will primarily traverse an existing right-of-way access road from Crenshaw Boulevard to the BNSF Railroad. This right-of-way is semi-improved with asphaltic concrete pavement, a railroad spur line, sparse landscaping, as well as exposed subgrade soils. The area has been previously graded such that a drainage swale is located at the approximate central portion of the right-of-way. This swale is vegetated and deepens to the east in order to keep a positive flow as the flow transfers to a culvert to pass below an access road to the ExxonMobil refinery. Ground elevations within the right-of-way generally range from about 70 to 75 feet. Vegetation consists of grasses and mature trees.

The north side of the right-of-way consists of improved property utilized as an oil refinery by ExxonMobil. Improvements located immediately adjacent to the right-of-way are generally minor. However, a large diameter storage tank and containment berm is located at the northwest portion of the property near the BNSF railroad. The southerly side of the right-of-way consists of property operated by the Dow Chemical Company.

Improvements located in close proximity of the right-of-way generally consist of storage tanks and vehicle access/packing facilities. A grass area is located at the westerly end of the property.

The proposed alignment of the extension between the BNSF railroad and Maple Avenue currently consists of unimproved property owned by the City of Torrance. The area is vegetated with grasses and mature trees. Ground elevations vary significantly from about elevation 75 feet near the BNSF railroad up to about 110 feet near Maple Avenue. A soil stockpile about 20 feet in height is present near the central portion of the property.

3.2 SUBSURFACE CONDITIONS

Subsurface exploration at the site generally encountered granular alluvial soils intermixed with layers of fine-grained material. Materials encountered adjacent to the Dow Chemical Plant (Borings B-5 to B-8) consisted of brown fine to medium grain sand intermixed with gravel. Granular soils encountered ranged in consistency from loose to dense. Clay layers were encountered at Boring B-5 at 42.5 to 47.5-feet, Boring B-6 at 42.5 to 46.5-feet, Boring B-7 at 32.5 to depth of exploration (41.5-feet), and Boring B-8 at 2.5 to 12.5-feet.

Boring B-1 to B-3 in the city lot west of the BNSF Railway, encountered primarily brownish tan fine grain sand intermixed with gravel. Clay soils were encountered at Boring B-1 at 72.5-feet to depth of exploration (101.5-feet). Granular soils encountered ranges in consistency from loose to dense. Material encountered at Boring B-1 had detectable gas odors from 70-feet to 101.5-feet. Clay soils encountered ranged in consistency from stiff to very stiff. Boring B-4 was excavated south of Del Amo Boulevard west of Maple Avenue; subsurface excavation encountered relatively loose tan silty sand.

Soil immediately east of the BNSF Railway (Boring B-2) encountered primarily granular soils in the form of fine grain sand intermixed with gravel and silt, ranging in consistency from medium dense to very dense. Clay layer were encountered at 25.5 to 29-feet and 43 to 63-feet. Clay soils encountered had a consistency ranging from very stiff to hard at depth.

3.3 GROUNDWATER

Groundwater depths as shown on the Test Boring Logs ranged from about 30 to 37 feet below the existing ground surface. Those depths correspond to elevations ranging from 36 to 44 feet. A Phase II Soil Investigation Report dated July of 2001 was prepared for the site by EDAW, Inc. Groundwater contours presented in that report indicated that groundwater elevations generally vary from 30 to 40 feet. Towards Van Ness Avenue, the groundwater elevations deepened to an approximate elevation of 0 feet. Geotechnical recommendations presented in this report have assumed a groundwater elevation of 40 feet.

4 ENGINEERING SEISMOLOGY

The project site is located in close proximity of several active and potentially active faults. The Southern California region is known to be seismically active and much geologic and seismologic evidence is available. The engineering seismology study included the examination of local and regional faulting and a review of existing historic earthquake data.

4.1 REGIONAL FAULTING

Earthquakes occurring within roughly 60 miles of the subject site are capable of generating ground shaking of engineering significance to the proposed construction. The California Seismic Hazard Map (1996) shows that many active and potentially active faults meet this criteria.

The faults of most concern to the project are the active faults. Active faults are basically those which are known to have surface displacement during the past 11,000 years. In our opinion, the Palos Verde and Newport-Inglewood Faults are the most important to the seismic design of the proposed structure.

4.2 LOCAL FAULTING

The site is not located within a currently established Earthquake Fault Zone. Neither our field observations nor literature search disclosed an active fault trace through the project site. In our opinion, it is not very likely that any ground or fault rupture will occur at the site during the design life of the proposed construction. As indicated on the California Seismic Hazard Map (1996), the nearest known faults include the Palos Verde (M = 7), the Newport-Inglewood (M=7), and the Redondo Canyon (offshore) (M = 6.25).

4.3 MAXIMUM CREDIBLE BEDROCK MOTION

Based upon the fault and subsurface soil conditions, a peak bedrock acceleration of 0.5g is considered applicable for this site with an associated moment magnitude (Mw) of 7. As indicated by the soil profile shown on the Log of Test Borings, the subsurface conditions are indicative of Type D soils as defined by Caltrans Seismic Design Criteria (December 2001) Version 1.2. In addition, since the fault is located less than 15 km from the bridge site, an increase in the ARS values was allotted. This evaluation is shown graphically on Figure 1.

4.4 LIQUEFACTION POTENTIAL

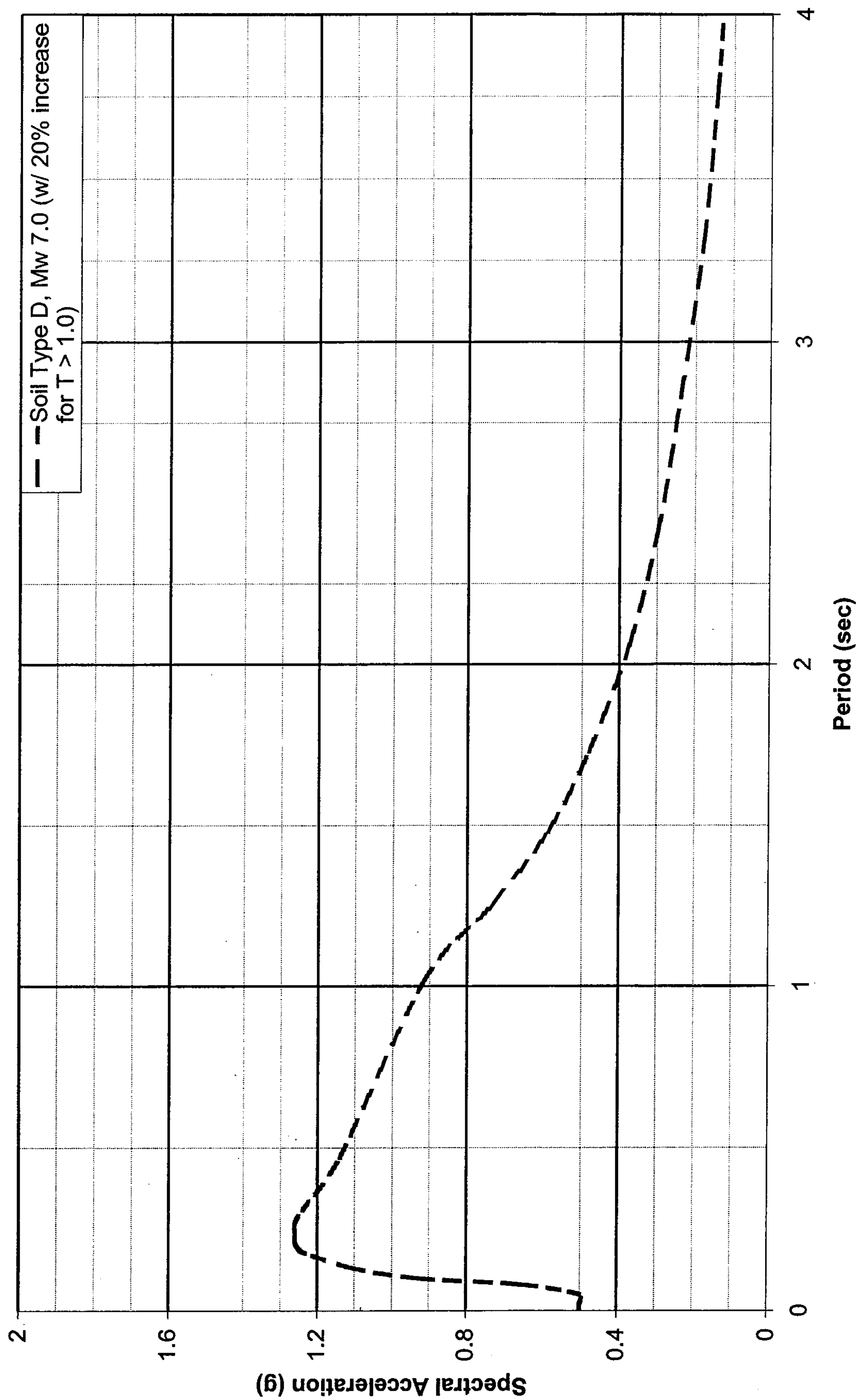
Essentially there are three conditions, which must be present at a site for liquefaction to occur. Relatively loose fine-grained sandy soils, shallow groundwater (within about 50-feet of the ground surface), and potential for strong ground motion. While the potential for strong ground motion and shallow groundwater exists onsite, the lack of loose fine-grained sandy soils below the groundwater table indicates that the potential for liquefaction is low.

5 CONCLUSIONS AND RECOMMENDATIONS

During the course of the geotechnical evaluation of this project, AMEC has identified several factors which could potentially affect design and construction of the project. Those issues include the following:

1. Due to the compressibility of the alluvial soils and in order to control total and differential settlements at the proposed bridge structure, it is recommended that deep foundations be utilized for support of the proposed structure. The use of spread footings is not recommended.

Figure 1
ARS Curve (0.5g, Soil Type D)



4-212-1029

ASR CURVES.xls

2. The soil and groundwater conditions at the bridge site favor using pile foundations to support the bridge. Cast-in-drilled hole (CIDH) piles could be used for this project, however, the relatively shallow groundwater and occasional sand layers will require casing and/or drilling slurry for installation of the CIDH piles. Since hard driving conditions are expected during pile installation, precast concrete piles are not considered to be the most viable driven pile option. Therefore, steel H-pile sections are recommended if the driven pile option is chosen for this project.

3. The upper soil materials were found to be subject to volume loss with increasing moisture content (hydrocompression). The depth of those materials is estimated to vary from 5 to 25 feet. Recommendations are presented in this report for the remediation of the potential effects of these materials.

4. The placement of fill soils for the MSE walls and embankments will result in settlement of the underlying soils beneath the wall and also some distance away from the wall. Estimated settlements due to MSE wall construction are presented within this report to aid in the evaluation of settlement effects upon adjacent properties and utilities.

5.1 BRIDGE STRUCTURE

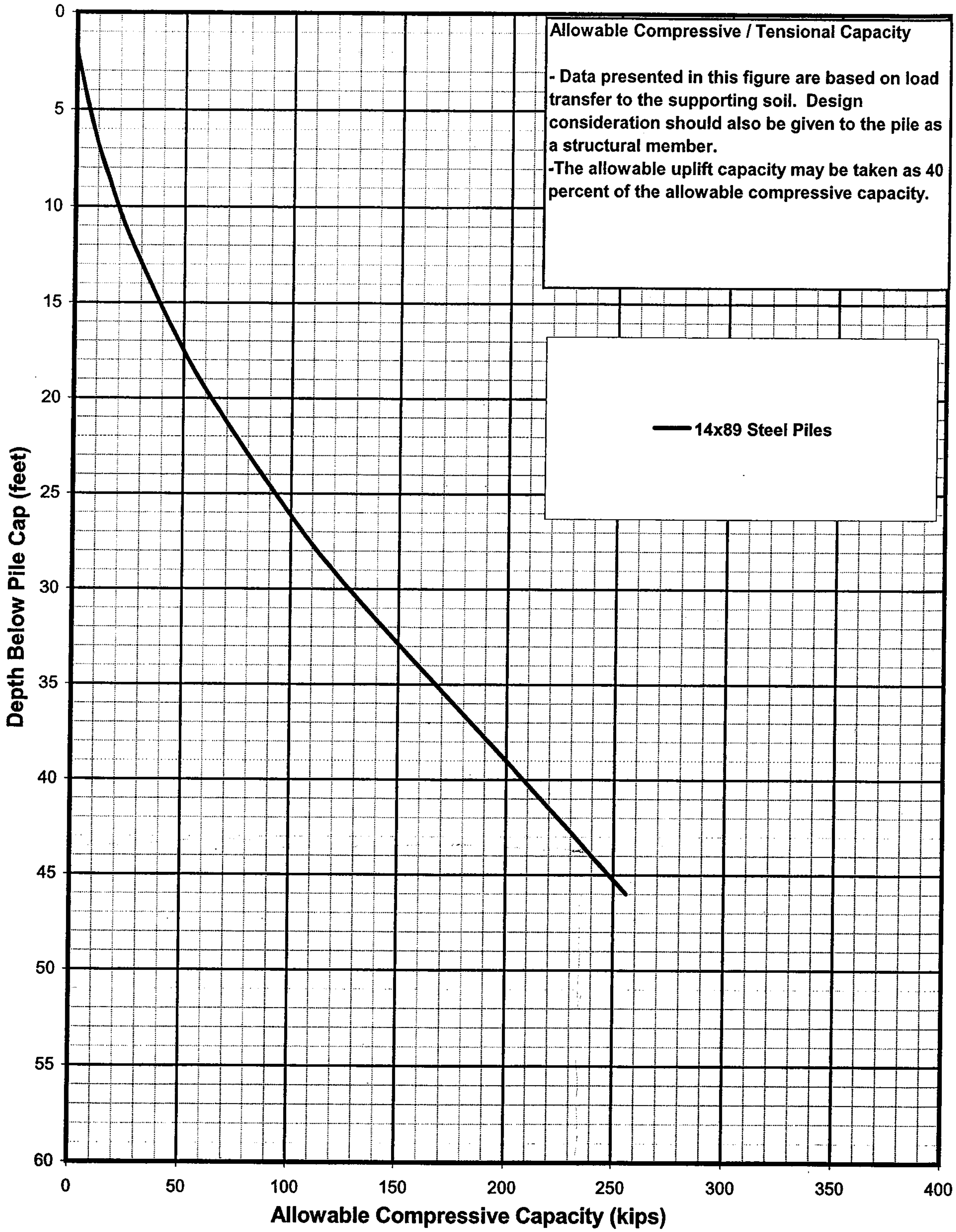
Geotechnical recommendations were developed for the subject site to meet the specific project needs. Current construction practices have been considered in the formulation of these recommendations. Due to the potential for intolerable total and differential settlements, the use of spread footings are not recommended. The soil and groundwater conditions at the bridge site favor using deep foundations to support the bridge. Cast-in-drilled hole (CIDH) piles may be considered as a foundation alternative; however, the relatively shallow groundwater and occasional sand layers will require casing and/or drilling slurry for installation of the CIDH piles. Driven piles may also be considered as a foundation alternative for this project. Since hard driving conditions are expected during pile installation, precast concrete piles are not considered to be the most viable driven pile option. Therefore, steel H-pile sections are recommended for this project. AMEC has provided foundation recommendations for 14x89 steel piles.

5.1.1 Axial Pile Capacity

The recommended pile tip elevations for 14x89 steel H-piles are presented on Figure 2. Those axial capacities are based on the allowable pile capacity. The ultimate compressive capacity may be taken as twice the allowable compressive capacity. The allowable uplift capacity may be taken as 40 percent of the allowable compressive capacity. Ultimate uplift capacity may be taken as twice the allowable tensional capacity. Piles should be spaced at a minimum of 2 ½ pile diameters center-to-center. If closer spacing is required, some reduction for group action may be required. The bridge plans indicate that abutment pile caps will be founded at elevations of 71.5 and 74.5 feet.

AMEC Earth & Environmental, Inc.

ALLOWABLE COMPRESSIVE CAPACITY



Job No. 5-212-100800

Figure 2

5.1.2 Lateral Pile Capacity

The design of 14x89 steel H-piles for lateral loading may be based on the data presented in Table 1. The estimated lateral load/deflective characteristics of the piles may be taken as presented on Figure 3. The criteria apply to isolated piles spaced no closed than 2 1/2 pile diameters on center. If structural design indicates that closer spacing may be required, the group action can be evaluated after the design loads and geometric parameters are determined.

TABLE 1
LATERAL LOAD CAPACITY

Condition of Fixity	Piles			
	Free	Fixed	Free	Fixed
Lateral Load (kips)	See Figure 3			
Pile Diameter	14x89 Strong		14x89 Weak	
Maximum Positive Moment (kips-ft.)	6.9P	1.7P	8.0P	1.8P
Maximum Negative Moment (kips-ft.)	-	-5.0P	-	-4.8P
Depth to Maximum Positive Moment (ft.)	9	10	9	9
Depth to Point of Inflection (ft.)	-	5.5	-	5.5
Depth to Zero Moment (ft.)	20.5	20.5	18.5	18.5

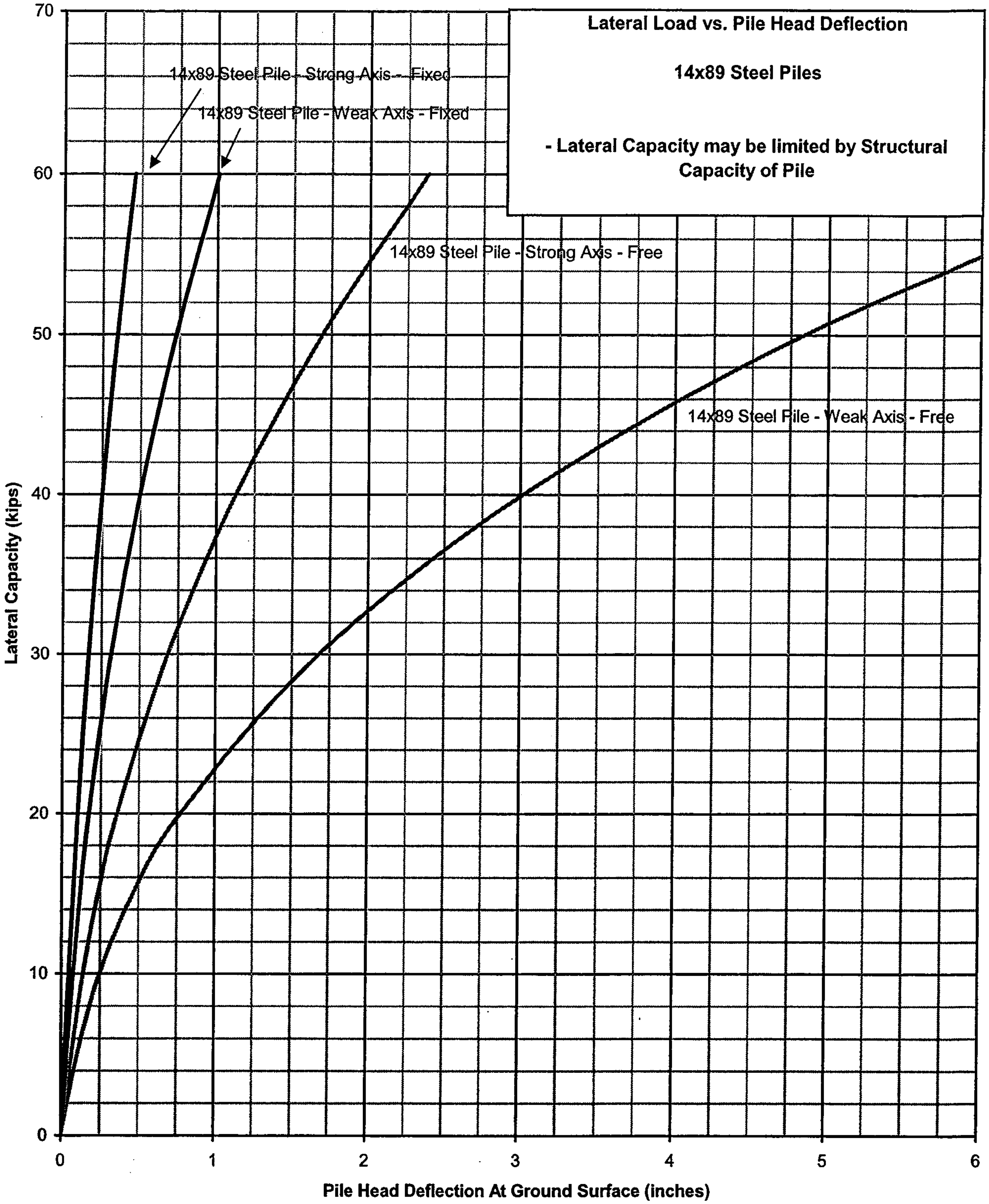
P = Lateral load (kips)

5.1.3 Abutments And Wingwalls

5.1.3.1 Wall Backfill

Structure backfill should conform to Section 19-3.06 of the Caltrans Standard Specifications. The material should have an expansion index no greater than 50 (low expansion potential) as established by ASTM D4829. Additionally, the material should possess a minimum resistivity greater than 1,000 ohm-cm (determined by California Test Method 643), a maximum chloride concentration of less than 500 ppm (determined by California Test Method 422), and a maximum sulfate content of less than 2,000 ppm (determined by California Test Method 417). It is expected that a majority of on-site materials will meet these requirements.

AMEC Earth & Environmental, Inc.
FIGURE 3



Job No. 5-212-100600

14x89LateralCHART.xls

5.1.3.2 Lateral Earth Pressure

Abutment walls should be backfilled in accordance with the previous section and be provided with drainage to minimize the buildup of hydrostatic pressures. For level backfill conditions and static loading the following parameters are considered applicable.

Friction Angle (Φ)	=	32
Active Earth Pressure Coefficient	=	0.30
Unit Weight	=	120 pcf
Active Lateral Earth Pressure	=	36 psf/ft
At-Rest Earth Pressure Coefficient	=	0.47
At-Rest Lateral Earth Pressure	=	56 psf/ft

The value of 36 psf/ft assumes that the wall is free to rotate at least .001 radian. For walls that are not capable of this movement, the pressure distribution for the at-rest condition should be used. The ultimate passive pressure resisting forces into the approach backfill during seismic loading may be taken as 5.0 (H/5.5) ksf, where H is the height of the abutment in feet. The maximum passive pressure is 5.0 ksf. A wall movement on the order of 0.025H would be necessary to develop this ultimate resistance.

It is understood that the wall designer is considering applying seismic earth pressure loading on the abutment walls. If seismic earth pressures are incorporated into the design of the abutment walls, then a seismic earth load of 22 H^2 lb/ft of wall length should be used for lateral earth loads against the abutment walls. This load distribution should be assumed to act at 1/3 down from the top of the wall.

5.1.4 Approach Fills

Based on preliminary bridge plans, significant approach fills will be required at both abutment locations to a maximum height of approximately 35 feet. The foundation materials below the proposed embankments generally consist of granular soils. Total settlements are anticipated to be on the order of 2 3/4-inches. Settlements due to the additional fill loads are expected to occur fairly rapidly after fill placement. Therefore, a minimum settlement period of one-month after fill placement and prior to pile construction is considered appropriate.

5.2 MSE WALL STRUCTURES

MSE walls are proposed along both sides of the Del Amo Boulevard extension with wall heights ranging up to 35 feet at the bridge abutments. The width of the walls is expected to be about 100 feet.

5.2.1 Foundation Design

The following recommendations should be incorporated into the MSE wall design and construction.

5.2.1.1 Settlements

Analysis performed by AMEC indicates that moderate settlements of the underlying soils can be expected over an approximately 5 month time period due to loads imposed by construction of the MSE walls. Table 2 provides estimated settlements at various locations within and away from the MSE wall for various wall heights. These estimated settlements may be interpolated and extrapolated for various wall heights and specific locations.

TABLE 2
ANTICIPATED MSE WALL SETTLEMENTS

Wall Height (ft)	Estimated Settlement (inches)			
	Center of MSE Structure	Outside Edge of MSE	10 Feet From Outside Edge	20 Feet From Outside Edge
35	2 ¾	2 ¼	1 ¼	½
30	2 ½	2	1	< ½
25	2 ¼	1 ¾	1	< ½
20	2	1 ½	1	< ½
10	1 ½	1	½	< ½

Due to the flexible nature of MSE construction, it is expected that these settlements can be tolerated by the proposed wall structures. However, the influence of MSE wall loads on adjacent structures and underground utilities will need to be evaluated. Expected settlements provided in Table 2 may be utilized for this evaluation. It is recommended that construction of the MSE walls be staged uniformly over a period of two months in order to reduce the potential for excessively high rates of settlement. Additionally, where existing improvements are in close proximity to the proposed walls, it may be prudent to initiate a survey monitoring program of adjacent ground and structures to provide early warning of any unacceptable settlement and/or distress levels.

5.2.1.2 Foundation Embedment

In order to provide adequate bearing capacity for the support of the MSE walls, it is recommended that the base of the MSE walls extended a minimum distance of two feet below the lowest adjacent finished grade.

5.2.1.3 Bearing Capacity

The allowable bearing capacity for MSE walls may be taken as q_{all} = 350 (B-2e) psf, where B is the base of the MSE wall in feet and e is the eccentricity of the resultant forces at the base of the wall.

5.2.1.4 Lateral Capacity

Lateral loads may be resisted by friction between the supporting soils and the bottom of the MSE wall and/or by lateral passive resistance acting against the sides of the MSE wall. An allowable coefficient of friction of 0.27 is considered applicable. The recommended lateral passive resistance may be taken at 195 psf/foot of depth. The upper two feet of soil in front of the MSE wall should not be utilized to calculate passive resistance unless the ground surface is covered with asphalt or concrete.

5.2.1.5 Earth Pressures

Earth pressures behind the MSE walls may be calculated using an equivalent fluid pressure of 36 psf/foot of depth. This value is based on level, well drained backfill. The effect of any surcharge (dead or live load) should be added to the lateral earth pressure using an active coefficient of 0.30. A maximum credible ground acceleration of 0.5g may be used for the evaluation of seismic earth pressures.

5.2.2 Wall Subgrade Requirements

Prior to placement of backfill below or within proposed MSE walls, the existing grade should be compacted to a minimum relative compaction of 95 percent to a depth of 3 feet. All fill placed beneath MSE walls should be compacted to a minimum of 95 percent relative compaction.

5.2.3 Wall Backfill Requirements

Design procedures and materials used for MSE wall construction should conform to Caltrans Bridge Standard Details Sheets, Numbers 1 through 9 (Case 2) and Bridge Design Aids pages 3-8.1 to 3-8.6 or an approved alternative. Backfill within MSE construction should be Structure Backfill conforming to section 19-3.06 of the Caltrans Standard specifications as modified by the requirements in Section 6.2 of this report or to the recommendations of the MSE wall manufacturer. In addition, this material should be non-corrosive to buried metals.

5.2.4 Drainage

It is understood that the roadway right-of-way between the MSE walls will consist of pavement, either asphaltic concrete for the wearing surface and concrete for the hardscape. There are no plans for landscaping or irrigation. Therefore, drainage behind the MSE walls may consist of a 12 inch thickness of Caltrans Class 2 permeable drainage material behind the MSE wall panels. The drain should connect to a drainage gallery for collection of subsurface water prior to disposal. The drainage gallery should consist of 4-inch diameter Schedule 40 PVC with ¼-inch perforations placed down within Caltrans Class II permeable material. Outlets should be provided every 150 feet, or less. These recommended drainage details are presented in Figure 4.

5.3 PAVEMENT DESIGN

5.3.1 Structural Pavement

It is anticipated that the proposed roadway section will be supported on imported soil material. A design R-value of 50 was used to evaluate the proposed pavement section. Any fill material imported to the site in order to establish proposed pavement subgrade should have an R-value of at least 50 in the upper 4 feet of roadway embankment. Based upon California Department of Transportation Design Procedures and a Traffic Index of 11 as supplied by Moffatt & Nichol, the recommended section for new pavement may be taken as presented in Table 3.

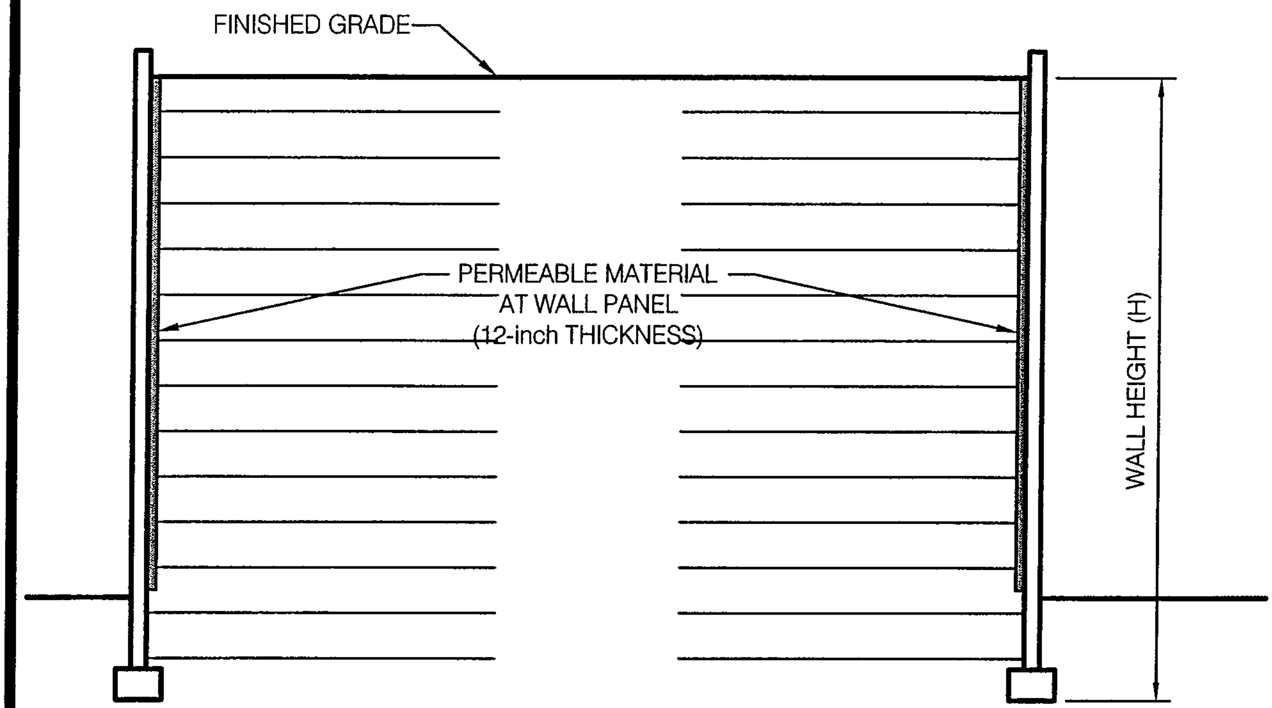
TABLE 3
STRUCTURAL PAVEMENT SECTION

Traffic Index	Asphalt Layer Thickness (inches)	Aggregate Base Layer Thickness (inches)
11	7.5	8

Legend of Terms:

AC – Asphalt concrete
AB – Aggregate Base

The upper 12-inches of subgrade should be removed in cut or at grade areas, then uniformly moisture conditioned to 110 percent of optimum moisture content and compacted to 95 percent of maximum dry density as determined by ASTM D1557. Unless otherwise specified by others, Asphaltic Concrete (AC) and aggregate base should conform to the current Standard Specifications for Public Works Construction. Aggregate base should be compacted to at least 95 percent of the maximum density (ASTM D1557).



DRAINAGE DETAILS
MSE RETAINING WALL

FILE: 5-212-100600-005 PLOT DATE: 7/8/2006

ANAHEIM R:\Drawings\2005\Dept1\5-212-1006005-212-100600-005.dwg, Drainage Detail Rev (Figure 4) 7/08/2006 8:52:51 AM

NOT TO SCALE

FIGURE 4

DEL AMO BOULEVARD EXTENSION
CITY OF TORRANCE, CALIFORNIA

DRAINAGE DETAILS

AMEC Earth & Environmental, Inc.

FIELD	DRAFT	APPROV.	DATE	JOB No.
--	--		July 2006	5-212-1006

5.3.2 Structure Approach Pavement System

Requirements for the use of a structure approach pavement system are provided within Section 610.3 of the Caltrans Highway Design Manual and the Caltrans Memorandum to Designers 5-3. Geotechnical conditions do not dictate the need for the use of an approach. However, a structure approach system may be preferred if the roadway is required for access to hospitals and/or emergency shelters. Caltrans Memorandum to Designers 5-3 offers further guidance for the necessity of approach systems.

5.4 CORROSION

Selected soil samples were tested to evaluate the pH, resistivity, and soluble sulfate content of the on-site soils. The results of these tests are provided in Appendix A with an overview of the test results given in Table 4.

TABLE 4
Chemical Test Results

Laboratory Test	Range of Test Results
Minimum Resistivity (ohm-cm)	2390 - >14400
pH	6.4 - 7.4
Soluble Sulfates (%)	0.0165 - 0.0463
Chloride Content (ppm)	40 - 113

5.4.1 Concrete Corrosion

Laboratory tests generally indicate negligible concentrations of soluble sulphate. Therefore, no special cement or concrete design is required for concrete coming into contact with onsite materials. Imported soil should be tested to confirm low sulphate content.

5.4.2 Metallic Corrosion

The on-site soils indicate that the on-site soils may present a corrosion potential for buried bare metal conduit. Use of Section 854 of the Highway Design Manual and California Test Method No. 643 should incorporate a pH value of 6.4 and a minimum electrical resistivity of 2390 ohm-cm.

5.5 EARTHWORK RECOMMENDATIONS

The grading for this project will primarily consist of site preparation, backfill of abutment and MSE walls, and changes to existing grades to accommodate approach fills.

Grading and earthwork should be performed in general accordance with the latest edition of the Caltrans Standard Specifications (CSS) and the following recommendations. The optimum moisture content and relative compaction, as used in the following sections, should be evaluated based on using ASTM D1557. Special attention should be given to Sections 19-3.06, 19-5.02 and 19-6.01 regarding material size used in fill placement.

5.5.1 Site Preparation

Clearing and grubbing within the area of proposed road work should conform to Section 16 of the CSS. Stripping of surface organics outside of the existing roadway is expected to generally require removals on the order of 2 to 4 inches of surface materials and should be performed prior to grading operations. Deeper removals may be required in areas of the heavy foliage/shrubs and trees. This organic material will not be suitable for use within fills. Any existing AC pavement removed during construction of improvements within the roadways may be broken up (maximum particle size of 3 inches) and mixed into general embankment fill materials.

After clearing and grubbing has been completed, the proposed fill and/or pavement areas should be prepared in accordance with Section 19 of the CSS. The upper soil materials were found to be subject to volume loss with increases in moisture content (hydrocompression). In order to minimize the potential for future distress to the proposed improvements, some remedial grading will be required. Within the area of proposed MSE wall structures, the upper 36-inches of subgrade below the proposed foundation elevation should be removed. Those removals should extend a distance of 3 feet outside the MSE wall limits. Within the areas of proposed embankments exceeding a height of 5 feet, the upper 3-feet of existing subgrade soils should be removed. Those removals should extend a distance of 3 feet beyond the toe of the embankments. Removals in remaining areas should extend to a depth of at least 12-inches below proposed or existing grade, whichever is deeper.

After making the above removals, the upper 6-inches of exposed subgrade should be uniformly moisture conditioned to 110 percent of optimum moisture content and compacted to 95 percent of maximum dry density (ASTM D1557).

5.5.2 Fill Placement And Embankment Construction

Placement of roadway fill should conform to Section 19 of the CSS, particularly Sections 19-5 and 19-6, and be compacted at or above optimum moisture content. Relative compaction should be in accordance with Sections 19-5.03 and 19-5.04 of the CSS. Based on the field investigation and laboratory testing, subgrade soils requiring over excavation and recompaction tend to undergo a volume loss of approximately 10 to 15 percent when placed as engineered fill. Thus, the earthwork factor associated with recompacting native soils is anticipated to range from approximately 0.85 to 0.90.

Embankment slopes should be no steeper than 1V:2H (vertical:horizontal) along the sides of new roadway improvements. Where new embankments will be constructed next to existing embankments, benching into the existing embankments should be performed as specified in the Section 19-6.01 of the CSS.

Fill material should be uniformly moisture conditioned to 110 percent of optimum moisture content and compacted to 95 percent of maximum dry density (ASTM D1557) to desired grade. Any soil materials imported to be used within the upper 4 feet of roadway fill should have a minimum R-value of 50.

5.5.3 General Site Grading

Site grading operations should conform with the applicable local building and safety codes and to the rules and regulations of those governmental agencies having jurisdiction over the subject construction.

The grading contractor is responsible to notify governmental agencies, as required, and the geotechnical engineer at the start of site clean-up, the initiation of grading and any time that grading operations are resumed after an interruption. Each step of the grading should be approved in a specific area by the geotechnical engineer and, where required, by the applicable governmental agencies before proceeding with subsequent work.

The following site grading recommendations should be regarded as minimal. All site grading recommendations should be incorporated within the project plans and specifications.

1. Prior to any grading, all vegetation, trash, surface structures and debris should be removed and is disposed off-site. Any existing utility lines, underground storage tanks, or other subsurface structures which are not to be utilized should be removed, destroyed or abandoned in compliance with current governmental regulations and with approval from the geotechnical engineer.

2. Subsequent to clean-up operations, and prior to initial grading, a reasonable search should be made for subsurface obstructions and/or possible loose fill or detrimental soil types. This search should be conducted by the contractor with advice from and under the observation of the geotechnical engineer.

3. Prior to the construction of pavement sections and retaining walls, subgrade preparation shall be made. Grading recommendations for specific areas of construction are outlined in this report.

4. Where new embankments will be constructed next to existing embankments, benching into the existing embankments should be performed as specified in Section 19-6.01 of the CSS.

5. Observation and field tests shall be performed during grading by the geotechnical engineer to assist the contractor in obtaining the proper moisture content and required degree of compaction. Where less than the required dry density is indicated, additional compactive effort and any necessary adjustments in moisture content shall be made to obtain the required compaction.

6. Wherever, in the opinion of the geotechnical engineer, an unsatisfactory condition is being created, whether by cutting or filling, the work should not proceed in that area until the condition has been corrected.

5.6 SHORING

Due to the proximity of the existing BNSF railroad, temporary slopes may not be practical for construction of the abutment foundations. It is therefore expected that temporary shoring will be required to support the excavation while the bridge structure is being constructed. Shoring design should be in accordance with the Caltrans Shoring manual. Geotechnical recommendations for braced, anchored, and cantilever shoring in general accordance with the design manual are provided below

5.6.1 Lateral Earth Pressure

The recommended lateral earth pressure against cantilevered, braced, or anchored shoring can be determined from Figure 5. The diagrams are based on average soil conditions and are applicable for excavations up to 25 feet in depth. In addition, any surcharge should be added to the indicated earth pressures utilizing an earth pressure coefficient of 0.35.

5.6.2 Resistance to Lateral Loads

The necessary depth of penetration of isolated soldier piles or continuous sheet piles to resist the lateral loading on sheeting can be determined from the expressions for ultimate passive lateral bearing pressure presented below.

Type of Pile	Ultimate Passive Lateral Bearing Pressure (psf) Native Soil
Isolated Soldier	650Z *
Continuous Sheet	550Z

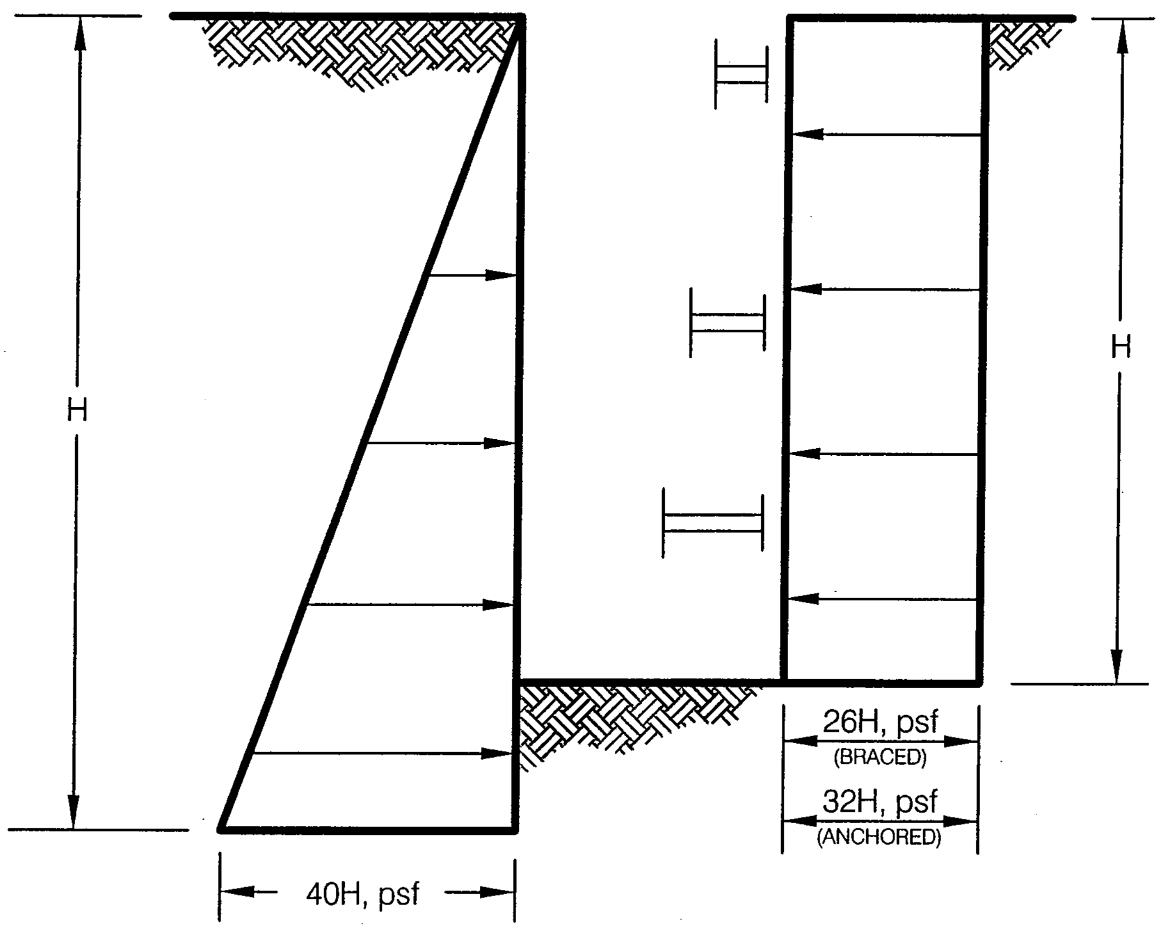
Where Z = depth in feet below bottom of excavation

*NOTE: The value for soldier piles incorporates a factor to account for soil arching and is applicable only to piles spaced at least four diameters apart.

Since the above expressions are based on the ultimate shear strength of the soil, an appropriate safety factor (minimum 1.5) should be incorporated into the design. Shoring systems should meet applicable provisions of all Federal and State of California Health and Safety regulations, the requirements of this report or the local building and safety codes, whichever is the more stringent. All components of shoring system should be designed by a Registered Civil Engineer.

FIGURE 5

RECOMMENDED EARTH PRESSURE AGAINST CANTILEVERED AND BRACED SHORING



ANAHEIM R:\Drawings\2005\Dept1\5-212-100600\5-212-100600-005.dwg, Earth Pressure (Figure 5) 3/27/2006 10:43:44 AM

E-PRESS

5.7 RAILROAD SPUR CONSTRUCTION

A relocation of the existing railroad spur is proposed to re-route railway traffic after construction of the grade separation. Present plans call for the tracks to be placed on 8 inches of ballast, which in turn is to be placed over 12 inches of sub-ballast. The spur line will be located within the unimproved right-of-way between the existing tracks and the ExxonMobil property and within the area of the existing maintenance road. It is recommended that the upper 12-inches of soil within the unimproved zone be scarified and recompacted to at least optimum moisture content and 95 percent relative compaction (ASTM D1557). Provided that the area of the existing maintenance road remains undisturbed during grading operations, remedial grading is not considered warranted for that area. Under these conditions a maximum allowable subgrade or top of roadbed bearing capacity of 3,000 psf may be used for design of the railroad spur.

5.8 SLOPE STABILITY

The existing topography west of the proposed bridge structure is comprised of an embankment. Sloping ground conditions are located to the north, south and east. These slopes are generally variable in gradient and height with an overall average gradient of approximately 2:1 (horizontal:vertical). The 2:1 gradients are generally located to the north and south; however, there are localized areas within those slopes where the gradients are steeper. The descending north facing slope located to the northeast, in the area of the proposed bridge abutment, has an overall gradient of about 1.8:1. Slope heights generally are on the order of 20 to 25 feet to the south and over 30 feet to the north. Several stockpile locations are present at the top of the embankment with heights up to 25 feet. Existing improvements in this area consist of the railroad alignment to the northeast and a retaining structure to the south.

Proposed improvements in this area consist of MSE walls, a roadway, and a bridge structure. Several embankments are proposed to accommodate the roadway alignment. These include a fill slope along the north side of the roadway between approximate Stations 24+00 to 26+50. The proposed gradient is at 2:1 and the embankment height is up to 10 feet. This slope will be located above a gently sloping ground surface. From about Station 27+00 to 31+00 a fill embankment up to about 10 feet in height is proposed along the north side of the roadway. The toe of this embankment will be offset from 10 to 40 feet from the descending slope further north except at about Station 31+00. At Station 31+00, the embankment slope toe is located at the top of the existing 1.8:1 slope. A fill slope is to be located from about Station 27+00 to 30+00 south of the roadway. This slope will be a natural/fill transition slope. No drainage improvements to the slope were noted on the plan set.

The existing soil materials in this area are primarily comprised of sands with silts. Engineering evaluation of the proposed improvements in this area indicate that overall gross slope stability of existing and proposed slopes is expected to possess static factors of safety greater than 1.5. However, surifical stability analysis indicates that the surficial stability of existing and proposed slopes is expected to be less than 1.5, particularly if significant saturation of slopes occur. In addition, the majority of the existing embankment and the proposed fill embankment may be susceptible to erosion and gullying. It is recommended that serious consideration be given to landscape and drainage design to minimize infiltration of surface water and to provide for stabilizing vegetation on slope faces.

6 CONSTRUCTION SPECIFICATIONS AND CONSIDERATIONS

6.1 DRIVEN PILE INSTALLATION

Driven precast concrete piles should be installed in accordance with Section 49 of the Caltrans Standard Specifications (CSS) and the recommendations provided herein. Oversize predrilling is not considered necessary for this site. Small diameter predrilling per Caltrans Standard Specification 49-1.05 should not be performed without authorization from the Geotechnical Engineer. Due to the presence of dense to very dense sandy soils, the pile driving resistance is expected to increase accordingly. Piles will need to penetrate the dense material to reach the proposed tip elevations and it is expected that approved manufactured driving tips will be required. It should be noted that the elevation at which the dense sands are encountered may vary across the site. The use of water jets should not be allowed to install piles.

6.2 SITE GRADING

In general, required filling and backfilling should be in conformance with applicable portions of Sections 6 and 19 of the Caltrans Standard Specifications. We suggest the project special provisions include the following amendments to the Standard Specifications.

1. Section 19-3.06 - Ponding and jetting of backfill shall not be permitted.

2. Section 19-3.065 - Pervious backfill shall have a gradation which provides an adequate piping ratio with regard to adjacent soil, or a non-woven geotextile shall be placed between pervious backfill and adjacent soil. A geocomposite drain (e.g., Miradrain, Eljen Drain, Hitek Drain or Tensar DC1100 or approved alternative) may be placed behind abutments, wingwalls and retaining walls in lieu of pervious backfill.

3. Fill material for constructing embankments, placed behind walls, or used as subgrade for structural pavement sections should meet the following requirements:

Maximum Particle Size	3-inch
Percent Passing No. 200 Sieve	20 to 40
Percent Passing No. 4 Sieve	40 to 100
Maximum Expansion Index	51

Minimum R-value of 50 for material placed within 4 feet of finished grade.
Minimum Resistivity greater than 1,000 ohm-cm (CTM 643)
Chloride content less than 500 ppm (CTM 422)
Soluble sulphate content less than 2,000 ppm (CTM 417)

4. The optimum moisture content and relative compaction should be evaluated based on ASTM D1557.

7 GEOTECHNICAL REVIEW

Geotechnical review is of paramount importance in engineering practice. The poor performance of many improvements has been attributed to inadequate construction review.

7.1 PLANS AND SPECIFICATIONS

The Geotechnical Consultant should be retained to review the project plans and specifications prior to bidding and construction. This review is considered necessary to confirm that the recommendations in the geotechnical report have been effectively implemented.

This office is best qualified to make professional interpretations based on the work already performed. Such a review should be reported in writing by the Geotechnical Consultant.

7.2 CONSTRUCTION REVIEW

Observations and testing should be performed by the Geotechnical Consultant during construction. It should be anticipated that the substrata exposed by the construction may differ from that encountered during the subsurface exploration performed as part of this study. Reasonably continuous construction review during site grading and foundation excavation allows for evaluation of the "as-built" soil conditions, and allows for recommendations and appropriate revisions where required during construction.

Site preparation, removal of unsuitable soils, imported fill materials, fill placement, and other site grading operations should be observed and tested by the Geotechnical Consultant. If a firm other than AMEC is engaged to provide these services, it should be notified that it will be required to assume complete responsibility for all phases (design and construction) of the project within the purview of the geotechnical engineer. The firm should notify the owner, designer, appropriate governmental agencies and AMEC in writing that it concurs with the recommendations in this report and any subsequent addenda or that it will provide alternative recommendations.

8 CLOSURE

This report is based on the project as described and the information obtained from the exploration borings at the locations indicated on the Log of Test Borings. The findings are based on the results of the field, laboratory, and office studies, combined with an interpolation and extrapolation of soil conditions between and beyond the borings. The results reflect an interpretation of the limited direct evidence obtained. This firm should be notified of any pertinent change in the project plans. If subsurface conditions are found to differ from those described herein, it may require a revaluation of the recommendations.

This report has not been prepared for use by parties or projects other than those named or described above. It may not contain sufficient information for other parties or other purposes. It has been prepared in accordance with generally accepted geotechnical practices and makes no other warranties, either expressed or implied, as to the professional advice or data included in it.

Respectfully submitted,

REGISTERED PROFESSIONAL ENGINEER
DOUGLAS DAHNCKE
No. GE 2279
Exp. 12/31/07
GEOTECHNICAL
STATE OF CALIFORNIA

Douglas Dahncke
Senior Engineer
GE 2279 (Expires December 31, 2007)
DD/dc

c: Mr. Arun Jain, Addressee (8)

APPENDIX A

AMEC Earth & Environmental, Inc.

TEST BORING LOG

TYPE	5" DIA. ROTARY WASH	ELEVATION	95 FEET	BORING	B-1

STRIKE / DIP	RELATIVE COMPACTION	DRY DENSITY (lbs/cu.ft.)	MOISTURE (%)	BLOWS/FOOT	SAMPLE SIZE (INCHES)	SAMPLE NO.	DEPTH IN FEET	MATERIAL SYMBOL	UNIFIED SOIL CLASS.	DESCRIPTION
									SP	Brownish tan, fine grain SAND
				25	2.5	1	5			. . . (5 feet) brown, fine SAND with minor GRAVEL
			11.9	45	1.4	2	10			. . . (10 feet) no GRAVEL encountered, dense
		102.5	11.2	61	2.5	3	15			
				31	1.4	4	20			. . . (20 feet) dense
		100.3	6.3	60	2.5	5	25			
				33	1.4	6	30			. . . (30 feet) some white coloring encountered, dense
		104.4	12.4	77/11"	2.5	7	35			
				47	1.4	8	40			. . . (40 feet) dense
		103.9	13.4	65	2.5	9	45			
				41	1.4	10	50			. . . (50 feet) dense
		98.3	15.0	87/9"	2.5	11	55			
							60			

Continued

THIS BORING LOG SUMMARY APPLIES ONLY AT THE TIME AND LOCATION INDICATED. SUBSURFACE CONDITIONS MAY DIFFER AT OTHER LOCATIONS AND TIMES.

LOGGED BY	W.L.	DATE	6-21-05

AMEC Earth & Environmental, Inc.

TEST BORING LOG

TYPE 5" DIA. ROTARY WASH | ELEVATION 95 FEET | BORING B-1

STRIKE / DIP	RELATIVE COMPACTION	DRY DENSITY (lbs/cu.ft.)	MOISTURE (%)	BLOWS/FOOT	SAMPLE SIZE (INCHES)	SAMPLE NO.	DEPTH IN FEET	MATERIAL SYMBOL	UNIFIED SOIL CLASS.	DESCRIPTION
				62	1.4	12				. . . (60 feet) very dense
		103.2	15.6	54	2.5	13	65			
				55	1.4	14	70			. . . (70 feet) gas odor detected, very dense
									CL	Brown CLAY with gas odor
		87	39.4	57	2.5	15	75			
				21	1.4	16	80			. . . (80 feet) brown CLAY, very stiff
		101	25.3	45	2.5	17	85			. . . (85 feet) brown SANDY CLAY
				15	1.4	18	90			. . . (90 feet) stiff
		80.1	45.9	16	2.5	19	95			. . . (95 feet) gray CLAY
				28	1.4	20	100			. . . (100 feet) very stiff

NOTES:

1. Total depth of boring 101.5 feet.
2. Boring elevation based on site plan prepared by Moffatt & Nicol.
3. Boring backfilled from bottom with bentonite cement grout on June 21, 2005.

THIS BORING LOG SUMMARY APPLIES ONLY AT THE TIME AND LOCATION INDICATED. SUBSURFACE CONDITIONS MAY DIFFER AT OTHER LOCATIONS AND TIMES.

LOGGED BY W.L. | DATE 6-21-05

AMEC Earth & Environmental, Inc.

TEST BORING LOG

TYPE	5" DIA. ROTARY WASH	ELEVATION	79 FEET	BORING	B-2

STRIKE / DIP	RELATIVE COMPACTION	DRY DENSITY (lbs/cu.ft.)	MOISTURE (%)	BLOWS/FOOT	SAMPLE SIZE (INCHES)	SAMPLE NO.	DEPTH IN FEET	UNIFIED SOIL CLASS.	DESCRIPTION
								SP	Brown, fine grain SAND
		99.5	9.9	7	2.5	1	5		
				20	1.4	2	10		. . . (10 feet) medium dense
		107.1	18.6	19	2.5	3	15		
				47	1.4	4	20		. . . (20 feet) tanish coloring encountered, dense
		103.5	20.3	16	2.5	5	25	CL	Brown SANDY CLAY
								SP	Whitish gray, fine grain SAND, very dense
				56	1.4	6	30		
								SP	Mottled brown, medium to coarse grain SAND
		107.8	16.7	78/11"	2.5	7	▼ 35		
								SP SM	Fine grain SAND with SILT, very dense
				70	1.4	8	40		
								CL	Brown with tan streaks, fine grain CLAY
		86	38.0	32	2.5	9	45		
				23	1.4	10	50		. . . (50 feet) medium dense
		89.4	31.3	40	2.5	11	55		
							60		

Continued

THIS BORING LOG SUMMARY APPLIES ONLY AT THE TIME AND LOCATION INDICATED. SUBSURFACE CONDITIONS MAY DIFFER AT OTHER LOCATIONS AND TIMES.

LOGGED BY	W.L.	DATE	6-20-05

AMEC Earth & Environmental, Inc.

TEST BORING LOG

TYPE	5" DIA. ROTARY WASH	ELEVATION	79 FEET	BORING	B-2

STRIKE / DIP	RELATIVE COMPACTION	DRY DENSITY (lbs/cu.ft.)	MOISTURE (%)	BLOWS/FOOT	SAMPLE SIZE (INCHES)	SAMPLE NO.	DEPTH IN FEET	MATERIAL SYMBOL	UNIFIED SOIL CLASS.	DESCRIPTION
				44	1.4	12				. . . (60 feet) dark gray, dense
									SP	Brown, fine grain SAND
		100.7	15.8	77/9"	2.5	13	65			
				78	1.4	14	70			. . . (70 feet) gray coloring, very dense

NOTES:

1. Total depth of boring 71.5 feet.
2. Groundwater encountered at 35 feet.
3. Caving to 38.5 feet when boring bailed.
4. Boring elevation based on site plan prepared by Moffatt & Nichol.
5. Boring backfilled from bottom with bentonite cement grout on June 20, 2005.

THIS BORING LOG SUMMARY APPLIES ONLY AT THE TIME AND LOCATION INDICATED. SUBSURFACE CONDITIONS MAY DIFFER AT OTHER LOCATIONS AND TIMES.

LOGGED BY	W.L.	DATE	6-20-05

AMEC Earth & Environmental, Inc.

TEST BORING LOG

TYPE	6" DIA. HOLLOW STEM	ELEVATION	108 FEET	BORING	B-3

STRIKE / DIP	RELATIVE COMPACTION	DRY DENSITY (lbs/cu.ft.)	MOISTURE (%)	BLOWS/FOOT	SAMPLE SIZE (INCHES)	SAMPLE NO.	DEPTH IN FEET	UNIFIED SOIL CLASS.	DESCRIPTION
					Bulk	1		SP	Tan, fine grained SAND with minor GRAVEL
		110	7.3	83	2.5	2			. . . (2.5 feet) brown, fine grain
			8.1	21	1.4	3	5		. . . (5 feet) medium dense
		96.5	6.0	9	2.5	4	10		
			9.6	9	1.4	5	15		. . . (15 feet) tan, fine grain SAND, loose
		98.2	11.1	20	2.5	6	20		. . . (20 feet) orange to tan, fine grain GRAVEL
			9.0	26	1.4	7	25		. . . (25 feet) medium dense
		93.2	4.8	33	2.5	8	30		
			7.3	39	1.4	9	35		. . . (35 feet) whitish tan coloring encountered, dense
		98.9	7.4	84/11"	2.5	10	40		. . . (40 feet) whitish tan coloring encountered
			4.6	64	1.4	11	45		. . . (45 feet) very dense
		97.9	11.1	57	2.5	12	50		

NOTES:

1. Total depth of boring 51.5 feet.
2. No groundwater encountered.
3. Boring elevation based on site plan prepared by Moffatt & Nichol.

Continued

Column headings: STRIKE / DIP; RELATIVE COMPACTION; DRY DENSITY (lbs/cu.ft.); MOISTURE (%); BLOWS/FOOT; SAMPLE SIZE (INCHES); SAMPLE NO.; DEPTH IN FEET; MATERIAL SYMBOL; UNIFIED SOIL CLASS.

THIS BORING LOG SUMMARY APPLIES ONLY AT THE TIME AND LOCATION INDICATED. SUBSURFACE CONDITIONS MAY DIFFER AT OTHER LOCATIONS AND TIMES.

LOGGED BY	W.L.	DATE	6-16-05

AMEC Earth & Environmental, Inc.

TEST BORING LOG

TYPE	6" DIA. HOLLOW STEM	ELEVATION	108 FEET	BORING	B-3

4. Boring backfilled from bottom with bentonite cement grout on June 16, 2005.

STRIKE / DIP	RELATIVE COMPACTION	DRY DENSITY (lbs/cu.ft.)	MOISTURE (%)	BLOWS/FOOT	SAMPLE SIZE (INCHES)	SAMPLE NO.	DEPTH IN FEET	MATERIAL SYMBOL	UNIFIED SOIL CLASS.

THIS BORING LOG SUMMARY APPLIES ONLY AT THE TIME AND LOCATION INDICATED. SUBSURFACE CONDITIONS MAY DIFFER AT OTHER LOCATIONS AND TIMES.

LOGGED BY	W.L.	DATE	6-16-05

AMEC Earth & Environmental, Inc.

TEST BORING LOG

TYPE	6" DIA. HOLLOW STEM AUGER	ELEVATION	102 FEET	BORING	B-4

STRIKE / DIP	RELATIVE COMPACTION	DRY DENSITY (lbs/cu.ft.)	MOISTURE (%)	BLOWS/FOOT	SAMPLE SIZE (INCHES)	SAMPLE NO.	DEPTH IN FEET	MATERIAL SYMBOL	UNIFIED SOIL CLASS.	DESCRIPTION
					Bulk	1			SP SM	Tan SILTY SAND
				28	2.5	2				
			12.2	4	1.4	3	5			. . . (5 feet) very loose
		94.8	8.2	23	2.5	4	10			

NOTES:

1. Total depth of boring 11.5 feet.
2. No groundwater encountered.
3. Boring elevation based on site plan prepared by Moffatt & Nichol.
4. Boring backfilled from bottom with bentonite cement grout on June 16, 2005.

THIS BORING LOG SUMMARY APPLIES ONLY AT THE TIME AND LOCATION INDICATED. SUBSURFACE CONDITIONS MAY DIFFER AT OTHER LOCATIONS AND TIMES.

LOGGED BY	W.L.	DATE	6-16-05

AMEC Earth & Environmental, Inc.

TEST BORING LOG

TYPE **6" DIA. HOLLOW STEM AUGER** | ELEVATION **74 FEET** | BORING **B-5**

STRIKE / DIP	RELATIVE COMPACTION	DRY DENSITY (lbs/cu.ft.)	MOISTURE (%)	BLOWS/FOOT	SAMPLE SIZE (INCHES)	SAMPLE NO.	DEPTH IN FEET	UNIFIED SOIL CLASS.	DESCRIPTION
									One inch GRAVEL on surface
					Bulk	1		SP	Brown, fine grain SAND with minor GRAVEL
			11.9	7	1.4	2			. . . (2.5 feet) loose
		99.5	15.7	18	2.5	3	5		. . . (5 feet) wet zone encountered
			16.5	10	1.4	4	10		. . . (10 feet) orangish tan coloring encountered
				27	2.5	5	15		. . . (15 feet) orange to white coloring encountered
								SP SM	Brown SAND with SILT, medium dense
			11.1	12	1.4	6	20		
		93.9	14.1	39	2.5	7	25		
			27.9	28	1.4	8	30		. . . (30 feet) brown coloring encountered, medium dense
					Bulk	9			
		89.2	26.6	31	2.5	10	35		
								CL	Brown CLAY, stiff
			33.0	15	1.4	11	40		
								SP	Brown fine grain SAND
		91.1	24.6	38	2.5	12	45		
								SC	Grayish brown CLAYEY SAND, medium dense
			37.3	17	1.4	13	50		

NOTES:

1. Total depth of boring 51.5 feet.
2. Groundwater encountered at 34.5 feet.
3. Boring elevation based on site plan prepared by Moffatt & Nichol.

Continued

THIS BORING LOG SUMMARY APPLIES ONLY AT THE TIME AND LOCATION INDICATED. SUBSURFACE CONDITIONS MAY DIFFER AT OTHER LOCATIONS AND TIMES.

LOGGED BY **W.L.** DATE **6-15-05**

MATERIAL SYMBOL

AMEC Earth & Environmental, Inc.

TEST BORING LOG

TYPE 6" DIA. HOLLOW STEM AUGER	ELEVATION 74 FEET	BORING B-5

4. Boring backfilled from bottom with bentonite cement grout on June 15, 2005.

STRIKE / DIP	RELATIVE COMPACTION	DRY DENSITY (lbs/cu.ft.)	MOISTURE (%)	BLOWS/FOOT	SAMPLE SIZE (INCHES)	SAMPLE NO.	DEPTH IN FEET	MATERIAL SYMBOL	UNIFIED SOIL CLASS.

THIS BORING LOG SUMMARY APPLIES ONLY AT THE TIME AND LOCATION INDICATED. SUBSURFACE CONDITIONS MAY DIFFER AT OTHER LOCATIONS AND TIMES.

LOGGED BY W.L. DATE 6-15-05

AMEC Earth & Environmental, Inc.

TEST BORING LOG

TYPE	6" DIA. HOLLOW STEM AUGER	ELEVATION	73 FEET	BORING	B-6

STRIKE / DIP	RELATIVE COMPACTION	DRY DENSITY (lbs/cu.ft.)	MOISTURE (%)	BLOWS/FOOT	SAMPLE SIZE (INCHES)	SAMPLE NO.	DEPTH IN FEET	UNIFIED SOIL CLASS.	DESCRIPTION
								SP	Brown, fine grain SAND with minor GRAVEL and SILT
					Bulk	1		SM	
			9.9	6	1.4	2			. . . (2.5 feet) brown, fine grain SAND with SILT, loose
		113.5	13.6	31	2.5	3	5		. . . (5 feet) brown, fine grain SILTY SAND
			17.7	22	1.4	4	10		. . . (10 feet) medium dense
				38	2.5	5	15		. . . (15 feet) fine grain SILTY SAND
			6.3	21	1.4	6	20		. . . (20 feet) tan, fine grain SILTY SAND, medium dense
		91.8	25.2	68	2.5	7	25		
			25.8	14	1.4	8	30		. . . (30 feet) medium dense
								SP	Tan medium grain SAND with thin layers of quartz shells
				18	2.5	9	35		
							▼	SC	Brownish gray CLAYEY SAND, loose
			28.1	10	1.4	10	40		
									Brown CLAY with some quartz
		85.5	27.1	40	2.5	11	45		
									Brown, medium grain SAND, medium dense
			23.4	29	1.4	12	50		

NOTES:

1. Total depth of boring 53.5 feet.
2. Groundwater encountered at 37 feet.
3. Boring elevation based on site plan prepared by Moffatt & Nichol.

Continued

THIS BORING LOG SUMMARY APPLIES ONLY AT THE TIME AND LOCATION INDICATED. SUBSURFACE CONDITIONS MAY DIFFER AT OTHER LOCATIONS AND TIMES.

LOGGED BY	W.L.	DATE	6-15-05

AMEC Earth & Environmental, Inc.

TEST BORING LOG

TYPE 6" DIA. HOLLOW STEM AUGER	ELEVATION 73 FEET	BORING B-6

4. Boring backfilled from bottom with bentonite cement grout on June 15, 2005.

STRIKE / DIP	RELATIVE COMPACTION	DRY DENSITY (lbs/cu.ft.)	MOISTURE (%)	BLOWS/FOOT	SAMPLE SIZE (INCHES)	SAMPLE NO.	DEPTH IN FEET	MATERIAL SYMBOL	UNIFIED SOIL CLASS.

THIS BORING LOG SUMMARY APPLIES ONLY AT THE TIME AND LOCATION INDICATED. SUBSURFACE CONDITIONS MAY DIFFER AT OTHER LOCATIONS AND TIMES.

LOGGED BY W.L. DATE 6-15-05

AMEC Earth & Environmental, Inc.

TEST BORING LOG

TYPE	6" DIA. HOLLOW STEM AUGER	ELEVATION	71 FEET	BORING	B-7

STRIKE / DIP	RELATIVE COMPACTION	DRY DENSITY (lbs/cu.ft.)	MOISTURE (%)	BLOWS/FOOT	SAMPLE SIZE (INCHES)	SAMPLE NO.	DEPTH IN FEET	UNIFIED SOIL CLASS.	Description
								SP SM	ASPHALT: 2 inches, 2.5 inch BASE
					Bulk	1			Brown, fine SILTY SAND, some CLAY encountered
		116.6	13.7	29	1.4	2			. . . (2.5 feet) brown SILTY SAND
			16.7	17	2.5	3	5		. . . (5 feet) brown, slightly CLAYEY SAND, medium dense
		110.7	16.2	39	1.4	4	10		. . . (10 feet) more CLAY encountered
								SP	Brown, fine grain SAND, medium dense
			16.2	19	2.5	5	15		
		98.2	7.1	79	1.4	6	20		. . . (20 feet) tan, fine grain SAND
			6.2	28	2.5	7	25		. . . (25 feet) medium dense
		83.4	29.6	27	1.4	8	▼ 30		
								CL	Grades to brown CLAY with fine SAND, stiff
			30.6	9	2.5	9	35		
		73.6	36.1	15	1.4	10	40		. . . (40 feet) brown, fine grain SANDY CLAY

NOTES:

1. Total depth of boring 41.5 feet.
2. Groundwater encountered at 30.5 feet.
3. Boring elevation based on site plan prepared by Moffatt & Nichol.
4. Boring backfilled from bottom with bentonite cement grout on June 15, 2005.

MATERIAL SYMBOL

THIS BORING LOG SUMMARY APPLIES ONLY AT THE TIME AND LOCATION INDICATED. SUBSURFACE CONDITIONS MAY DIFFER AT OTHER LOCATIONS AND TIMES.

LOGGED BY	W.L.	DATE	6-15-05

AMEC Earth & Environmental, Inc.

TEST BORING LOG

TYPE	6" DIA. HOLLOW STEM AUGER	ELEVATION	72 FEET	BORING	B-8

STRIKE / DIP	RELATIVE COMPACTION	DRY DENSITY (lbs/cu.ft.)	MOISTURE (%)	BLOWS/FOOT	SAMPLE SIZE (INCHES)	SAMPLE NO.	DEPTH IN FEET	UNIFIED SOIL CLASS.	DESCRIPTION
									ASPHALT: 3 inches, 5 inch BASE
					Bulk	1		SP SM	Brown SILTY SAND
		111.5	7.9	16	1.4	2		CL	Brown CLAY with minor SAND
				20	2.5	3	5		
		110.7	12.0	46	1.4	4	10		. . . (10 feet) less CLAY encountered
								SP	Orange to tan, fine grain SAND, dense
			4.8	49	2.5	5	15		
		95.8	3.9	58	1.4	6	20		

NOTES:

1. Total depth of boring 21.5 feet.
2. No groundwater encountered.
3. Boring elevation based on site plan prepared by Moffatt & Nichol.
4. Boring backfilled from bottom with bentonite cement grout on June 15, 2005.

THIS BORING LOG SUMMARY APPLIES ONLY AT THE TIME AND LOCATION INDICATED. SUBSURFACE CONDITIONS MAY DIFFER AT OTHER LOCATIONS AND TIMES.

LOGGED BY	W.L.	DATE	6-15-05

APPENDIX B

LABORATORY TESTING

The laboratory testing was designed to fit the specific needs of this project and was limited to testing on-site materials. A brief description of each type of test is presented below. Specific results are given on pages B-2 through B-13.

In addition to the in-situ field tests, strength characteristics of the subsurface soils were determined in the laboratory by direct shear tests performed on nine (9) undisturbed samples. The samples were submerged and tested under three different normal loads in a 2.5 inch I.D. circular shear box, using a controlled displacement rate of 0.0083 inches per minute in general accordance with ASTM D3080.

The settlement characteristics of eight (8) soil sample were evaluated by means of laboratory consolidation tests. The samples were tested in a floating ring consolidometer using a dead weight lever system for load application in general accordance with ASTM D2435.

Atterberg Limit tests (Plasticity Index and Liquid Limit) were determined for five (5) soil samples in general accordance with ASTM D4318.

The concentration of soluble sulphate was determined for eight (8) soil samples in general accordance with California Test Method No. 417.

The chloride content of eight (8) soil samples was determined in general accordance with California Test Method No. 422.

Corrosivity tests were performed on five (5) soil samples to determine the pH and minimum electrical resistance of the soils. These tests were conducted in general accordance with California Test Method No. 643.

The grain size distribution for material passing the 200 sieve was determined for eight (8) samples in general accordance with ASTM D1140.

A Resistance (R-value) test was performed on a typical subgrade sample from the proposed paving areas. Testing was conducted in general accordance with California Test Method No. 301.

An expansion test was performed on one (1) soil sample in general accordance with the standard procedure for the Expansion Index Test (UBC Standard 29-2). In this testing procedure, the remolded sample is compacted with an energy input of 11,300 ft-lbs. Per cubic foot at 50 percent saturation. After remolding, the sample is confined under a pressure of 144 psf and allowed to soak for twenty-four hours. The resulting volume change due to increase in moisture content is recorded together with initial moisture content and dry density.

The remaining soil samples are now stored in our laboratory for future reference and analysis if desired. Unless notified to the contrary, all samples will be disposed of 30 days from the date of this report.

amec

DIRECT SHEAR TEST RESULTS
(ASTM D3080)

Boring No./ Sample No.	Normal Stress (psf)	Peak Shear		Shear Stress At 0.25 inch Displacement (psf)
		Stress (psf)	Displacement (inch)	
1/5	1000	730	0.125	650
	2000	1310	0.195	1240
	3000	2040	0.130	1860
1/9	4000	3144	0.093	2616
	5000	3803	0.105	2964
	6000	4307	0.110	3575
1/13	4000	3000	0.08	2600
	5000	3920	0.065	3140
	6000	4310	0.13	4010
1/15	5000	3359	0.096	2472
	6000	3995	0.104	3455
	7000	4679	0.174	4451
2/3	1000	980	0.07	660
	2000	1720	0.095	1260
	3000	2690	0.10	2110
2/5	2000	1584	0.094	1296
	3000	2160	0.092	2016
	4000	2712	0.071	2304
2/7	3000	2940	0.06	2110
	4000	3820	0.065	2520
	5000	3740	0.105	3220
2/9	4000	2712	0.127	2448
	5000	3431	0.141	3132
	6000	3719	0.100	3611
3/4	1000	684	0.055	636
	2000	1260	0.106	1224
	3000	1824	0.228	1704

amec

DIRECT SHEAR TEST RESULTS
(ASTM D3080)

Boring No./ Sample No.	Normal Stress (psf)	Peak Shear		Shear Stress At 0.25 inch Displacement (psf)
		Stress (psf)	Displacement (inch)	
4/2	1000	1080	0.086	708
	2000	1656	0.061	1308
	3000	2196	0.092	1824
5/5	1000	816	0.085	684
	2000	1644	0.101	1284
	3000	2280	0.109	1920
6/5	1000	840	0.061	696
	2000	1596	0.089	1368
	3000	2304	0.127	2004
7/8	1000	696	0.131	648
	2000	1404	0.134	1272
	3000	2004	0.233	1980

ATTERBERG LIMITS
(ASTM D4318)

Boring No./ Sample No.	Liquid Limit	Plastic Limit	Plasticity Index
2/9	**40**	**26**	**14**
5/6	**--**	**NP**	**--**
6/6	**--**	**NP**	**--**
6/7	**--**	**NP**	**--**
7/9	**29**	**19**	**10**

amec

#200 SIEVE WASH
(ASTM D1140)

Boring No./Sample No.	% Passing
2/8	18
4/1	23
5/6	12
5/6	15
6/6	13
6/7	1
7/9	49
8/1	45

SOLUBLE SULPHATE AND CHLORIDE CONTENT
(CALIF. 417 AND CALIF. 422)

Boring No./Sample No.	Soluble Sulphate (%)	Chloride Content (ppm)
1/4	0.0165	70
1/16	--	113
2/4	0.0206	40
2/10	0.0370	60
3/1	--	50
4/1	--	78
5/1	0.0463	40
6/1	0.0370	40

CORROSIVITY TEST
(CALIF. 643)

Boring No./Sample No.	Minimum Resistivity (ohm-cm)	pH
2/4	2390	7.0
3/1	7180	7.4
4/1	>14,400	6.4
5/1	>14,400	6.4
6/1	>14,400	7.2

R-VALUE
(CALIF. 301)

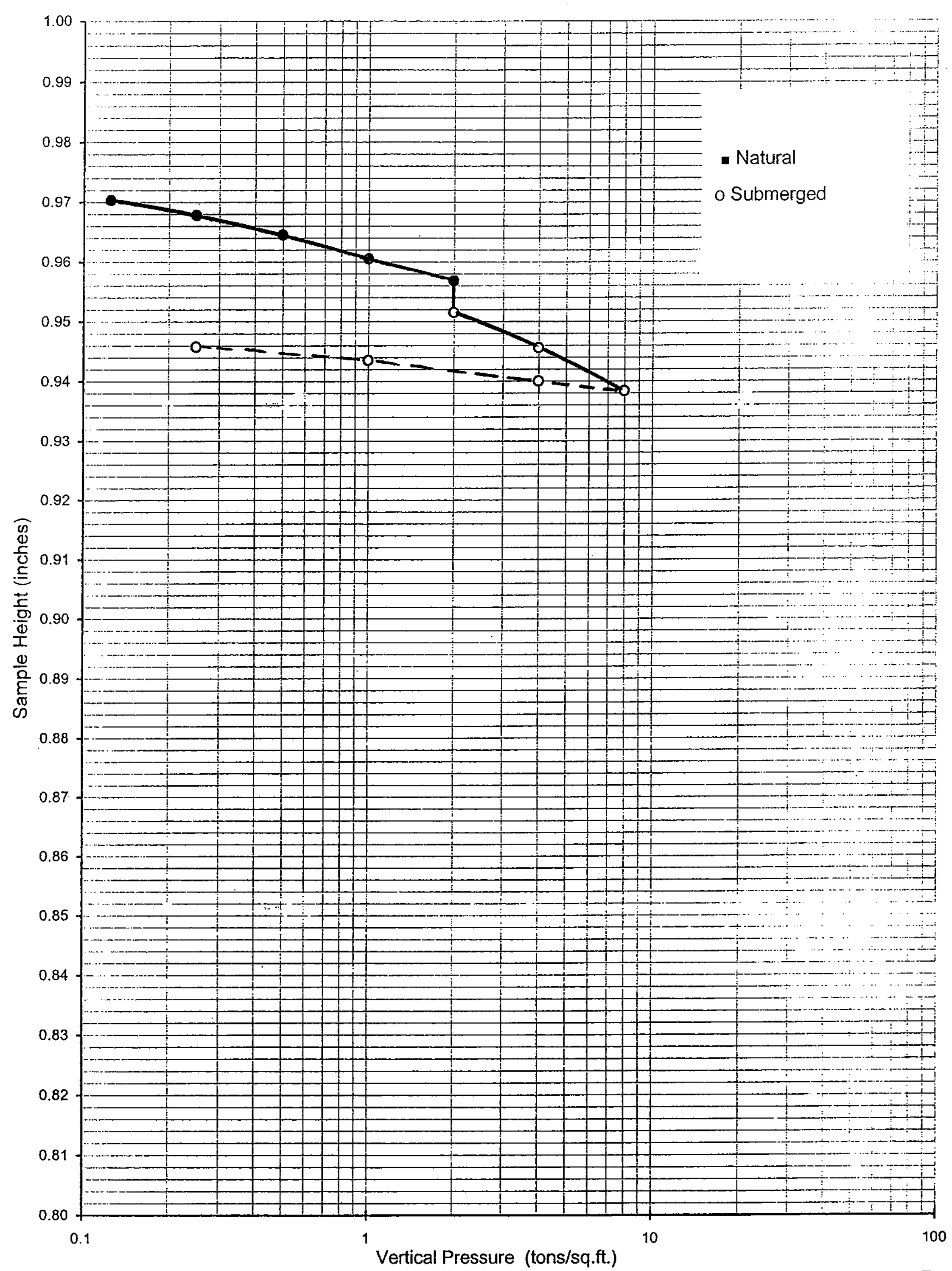
Boring No./Sample No.	Test Moisture (%)	Dry Density (pcf)	Expansion Dial Reading (inch)	Exudation Pressure (psi)	Corrected R-Value
4/1A	9.2	111.3	0	799	72
4/1B	10.9	108.9	0	527	71
4/1C	13.4	109.1	0	185	68
7/1A	12.8	112.0	0	105	64
7/1B	11.9	111.8	0	204	75
7/1C	10.3	113.1	0	443	79

EXPANSION INDEX
(UBC 29-2)

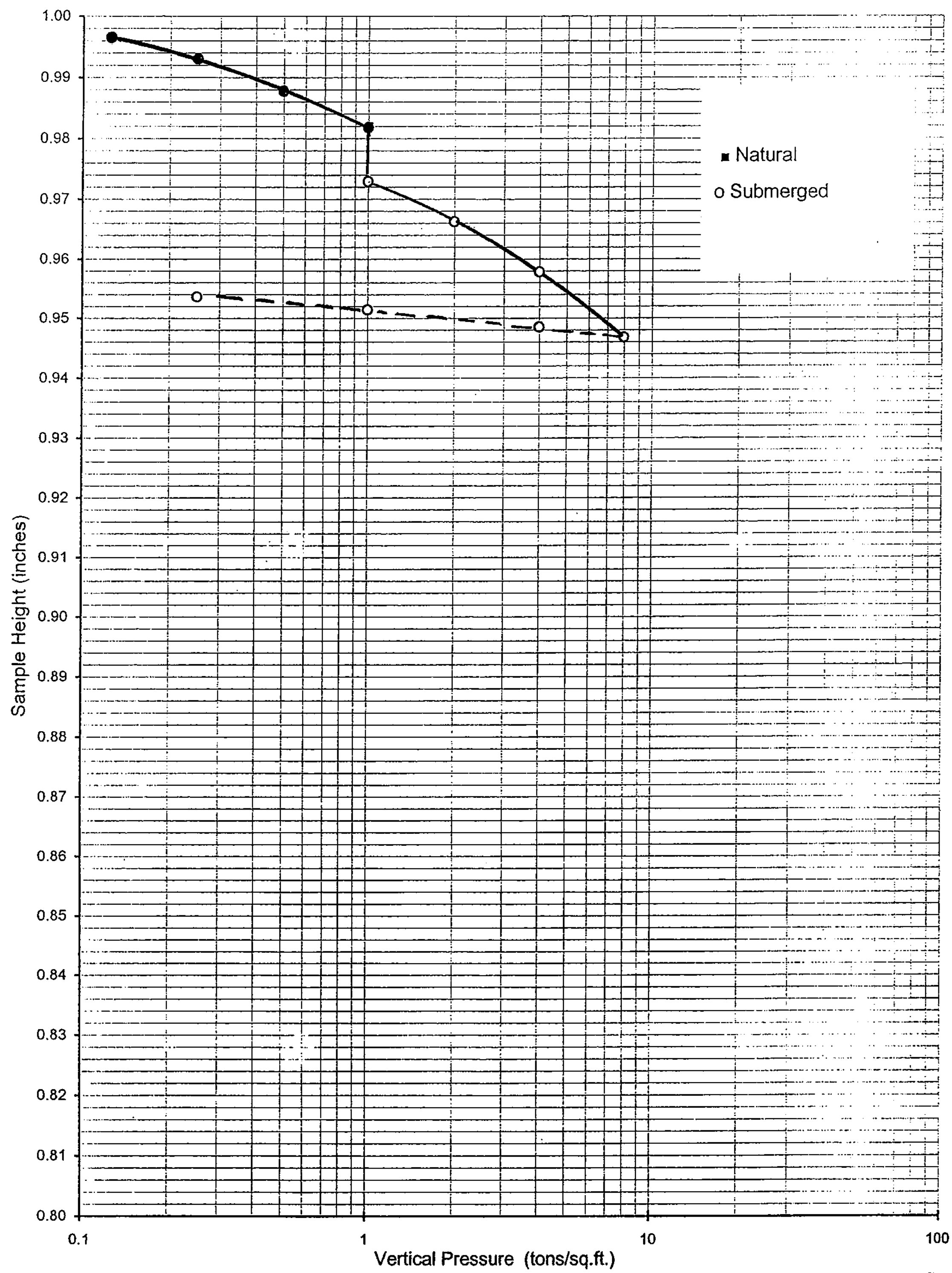
Boring No./Sample No.	Expansion Index	Swell (%)
3/1	0	0

CONSOLIDATION TESTS

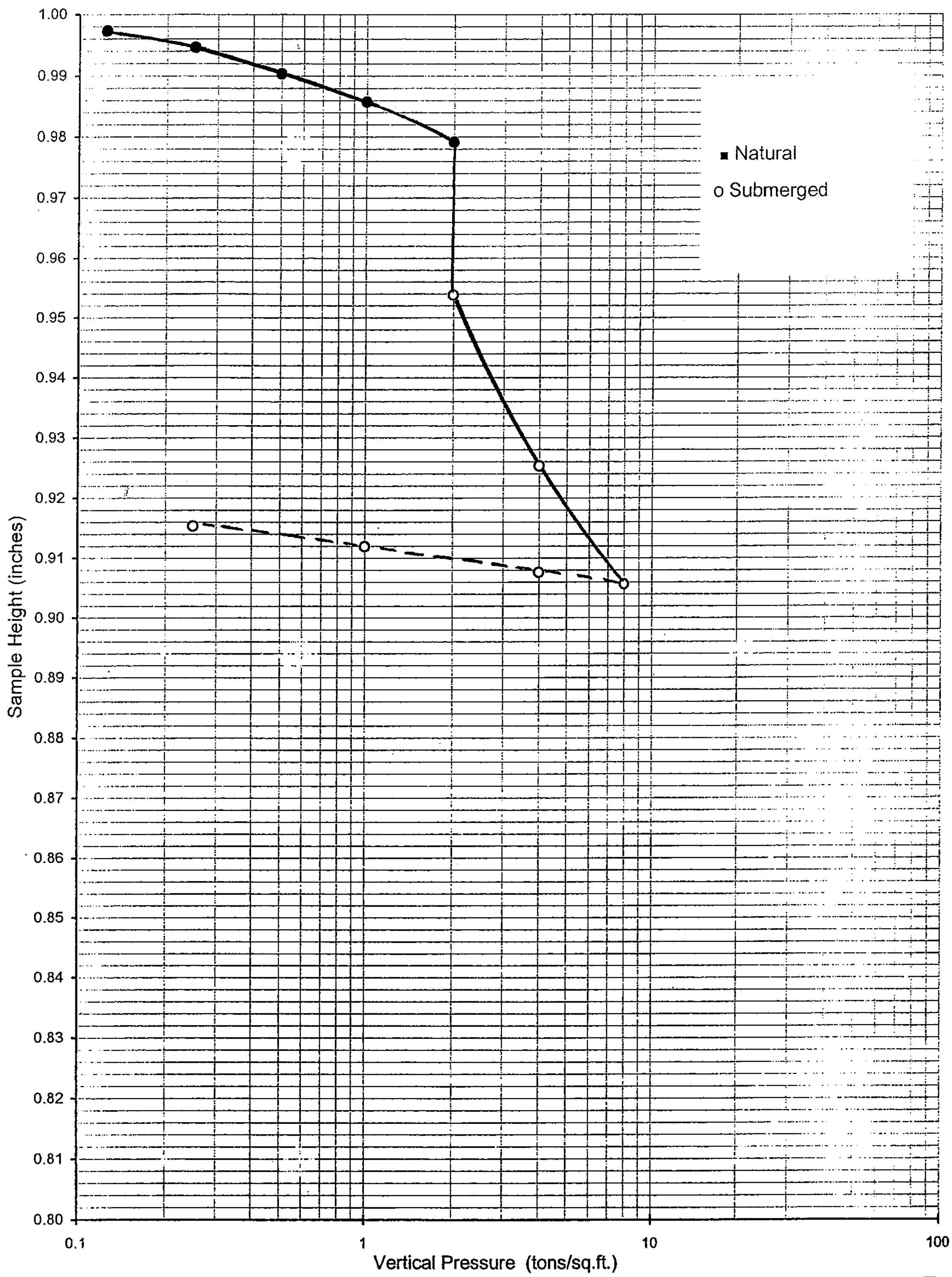
5-212-100600	B1/5	Depth 25'	Date 07-18-05



5-212-100600	B2/1	Depth 5'	Date 07-18-05

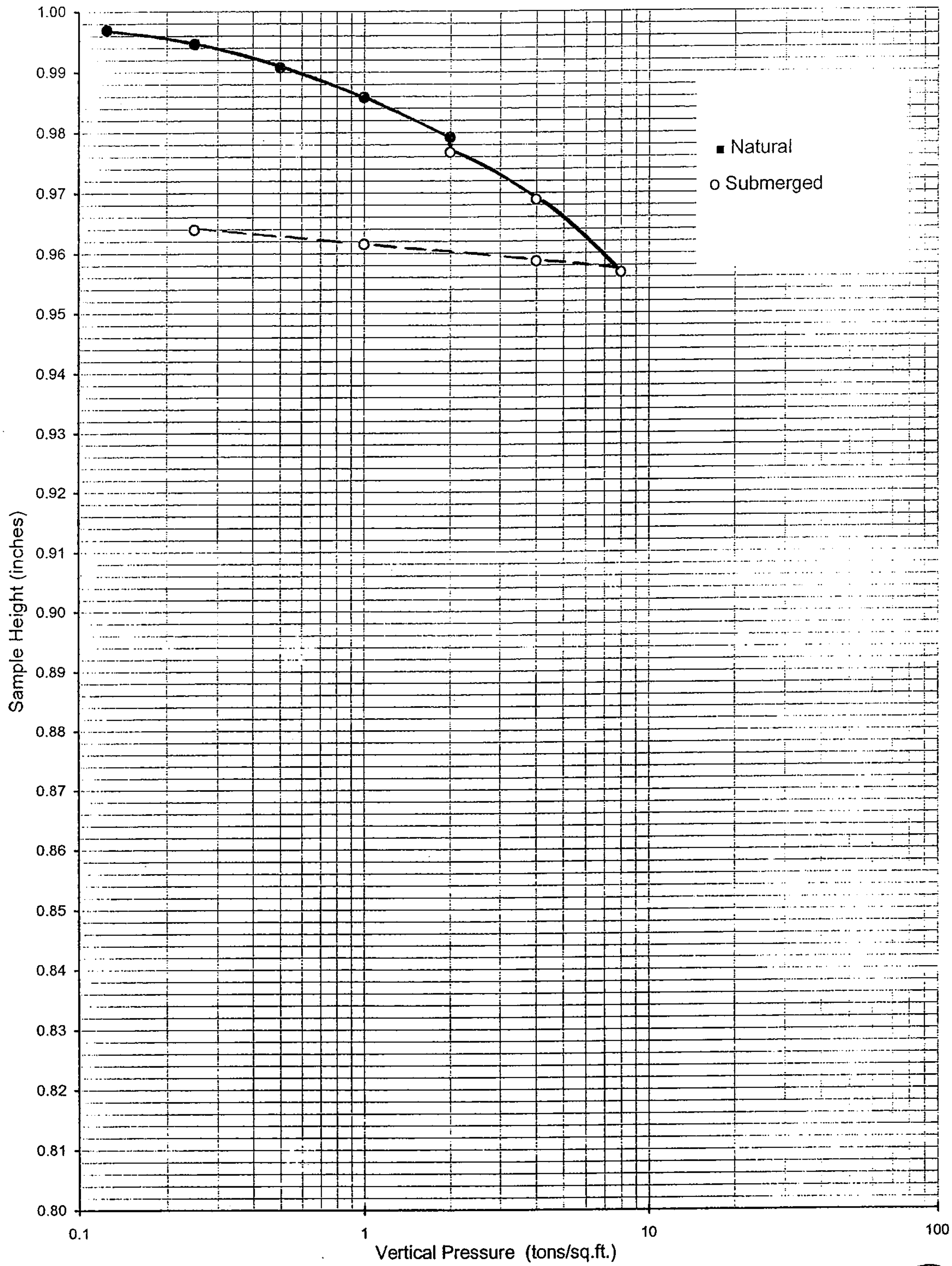


5-212-100600	B3/6	Depth 20'	Date 07-18-05

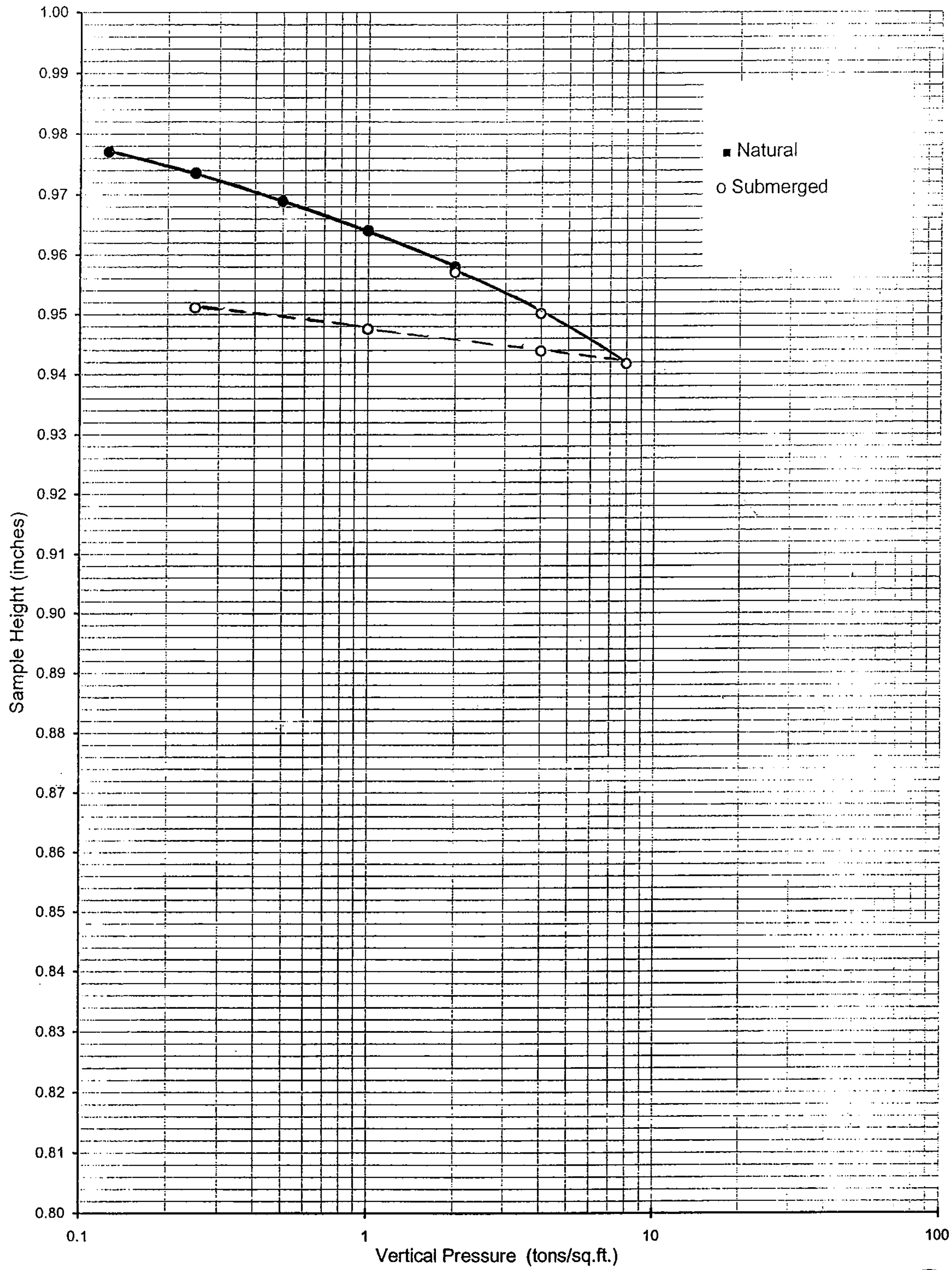


amec

5-212-100600	B5/3	Depth 5'	Date 07-18-05



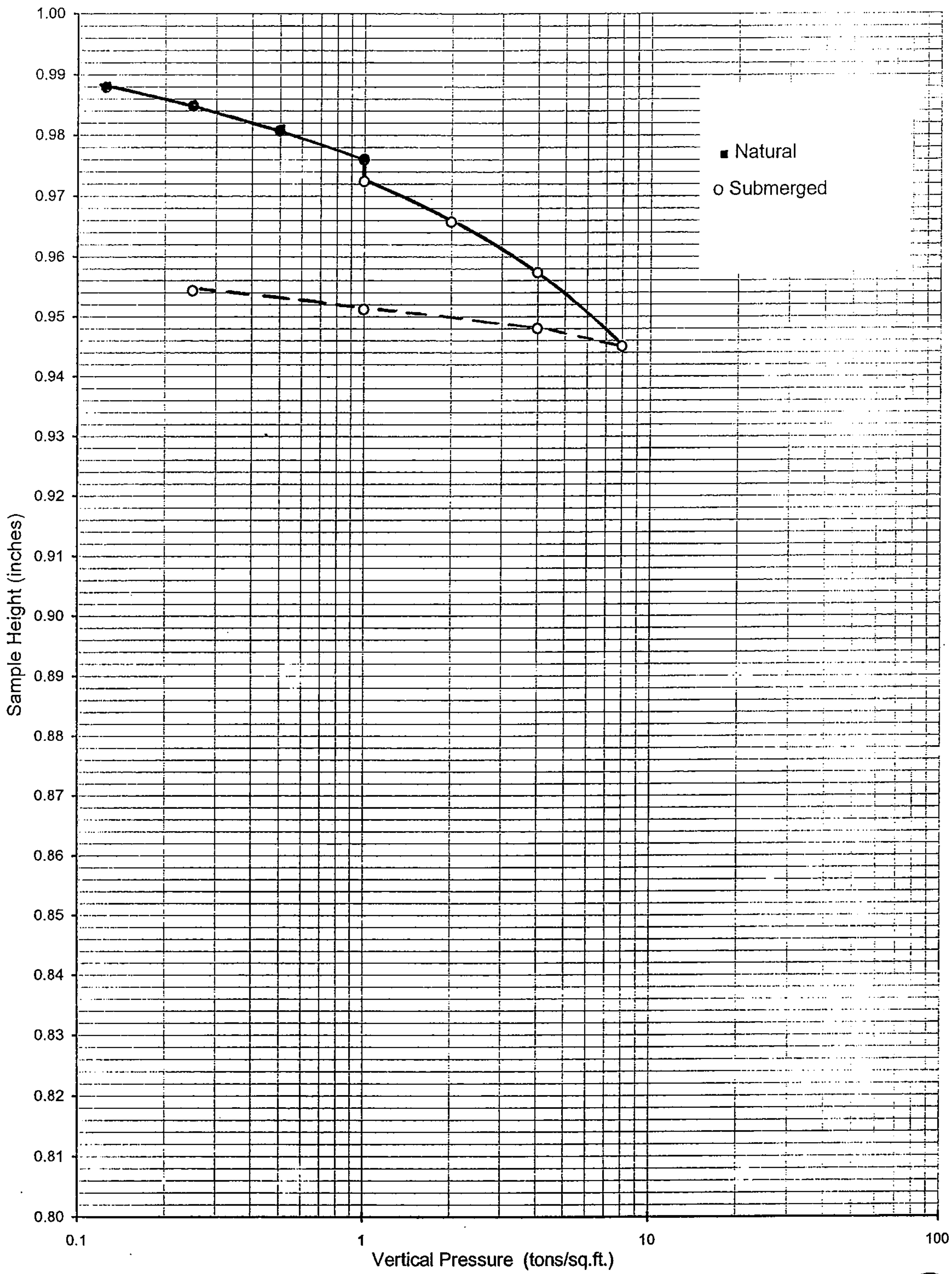
5-212-100600	B5/7	Depth 25'	Date 07-18-05



amec

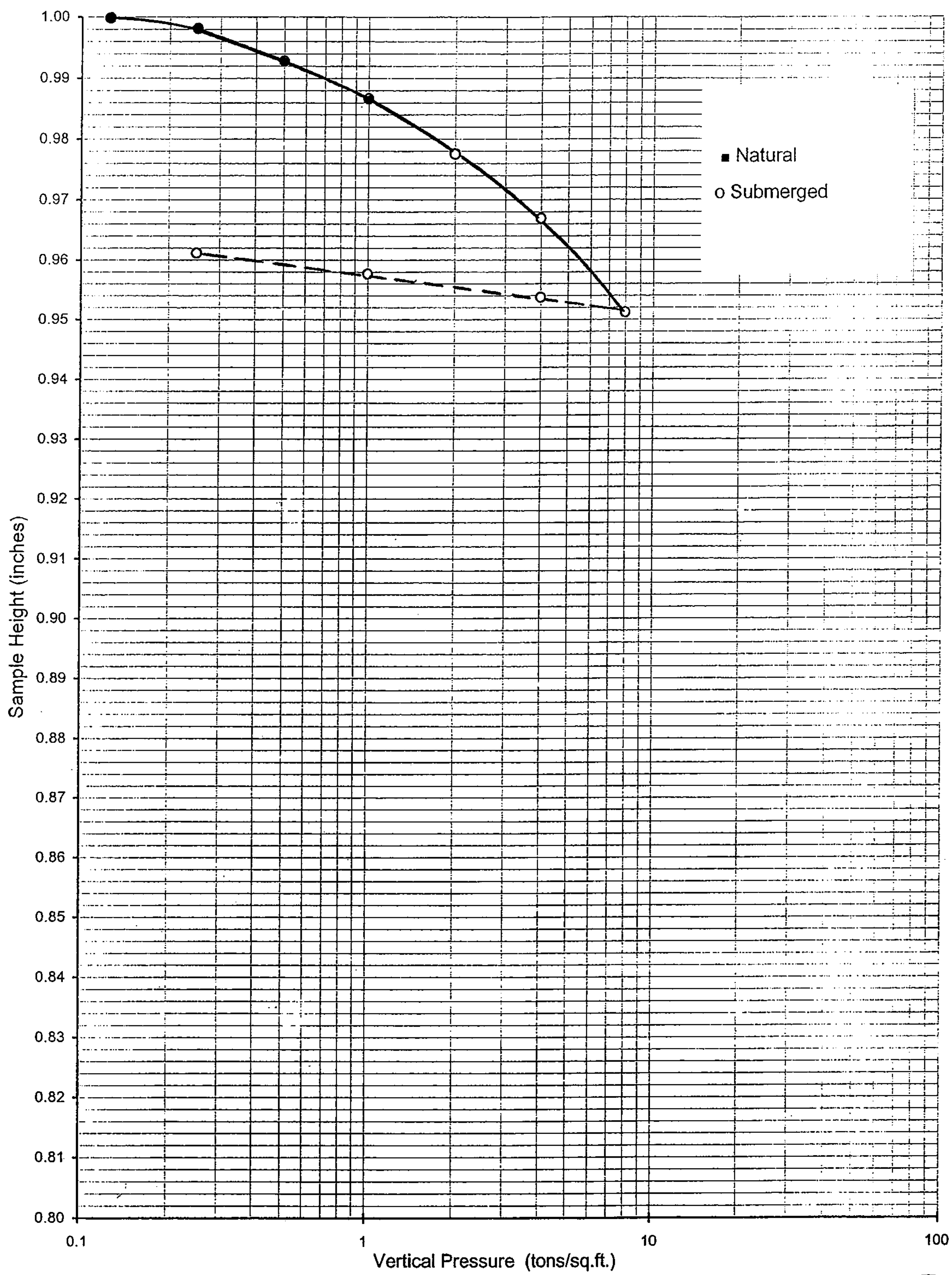
CONSOLIDATION TESTS

5-212-100600	B6/3	Depth 5'	Date 07-18-05

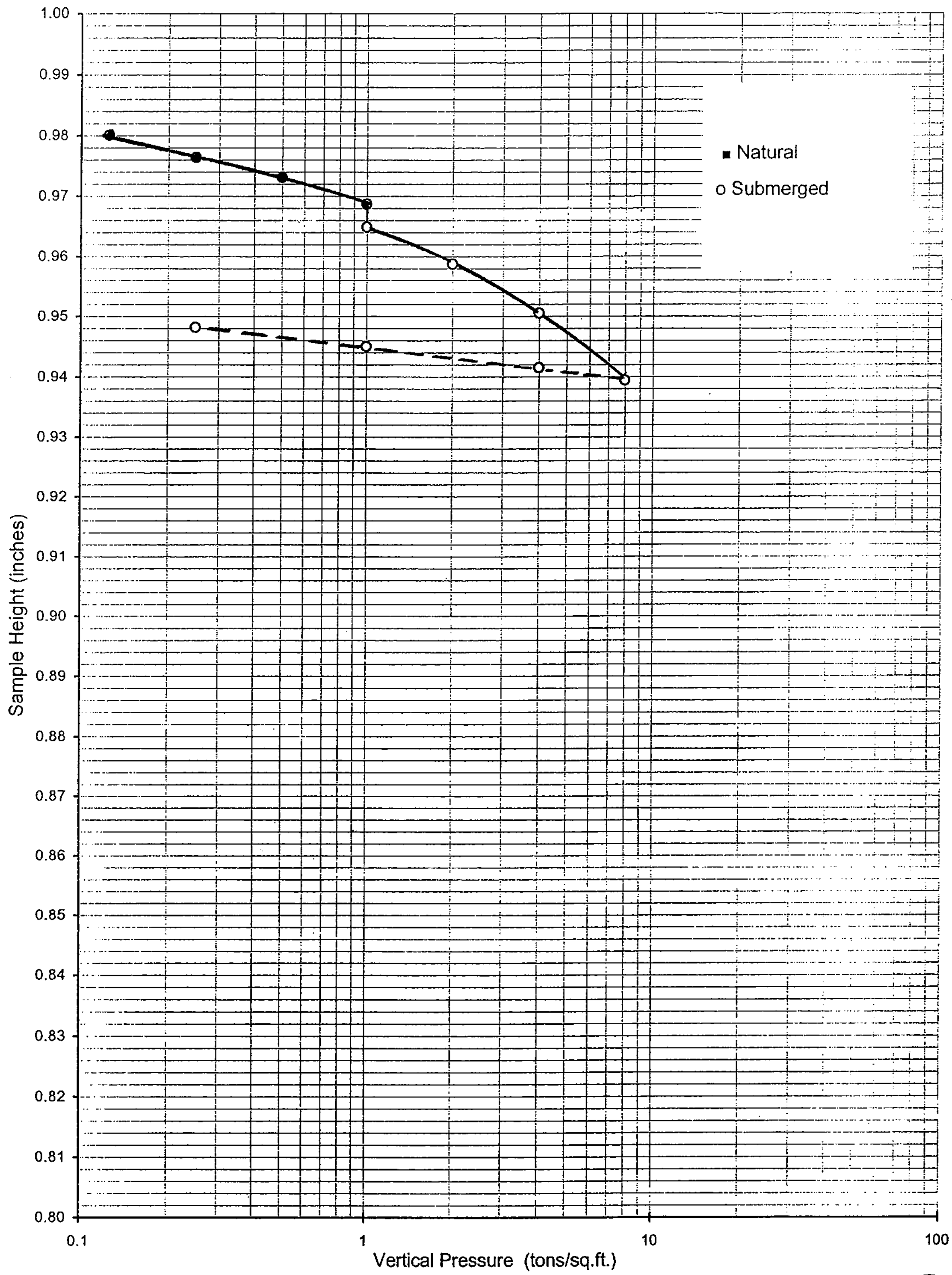


CONSOLIDATION TESTS

5-212-100600	B6/8	Depth 35'	Date 07-18-05

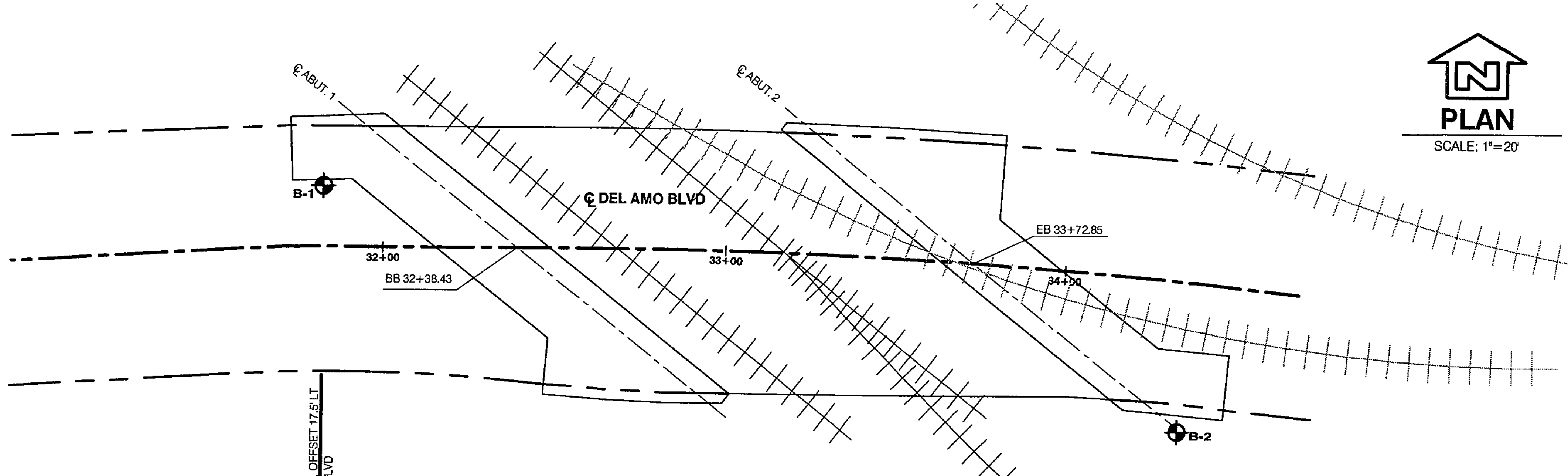


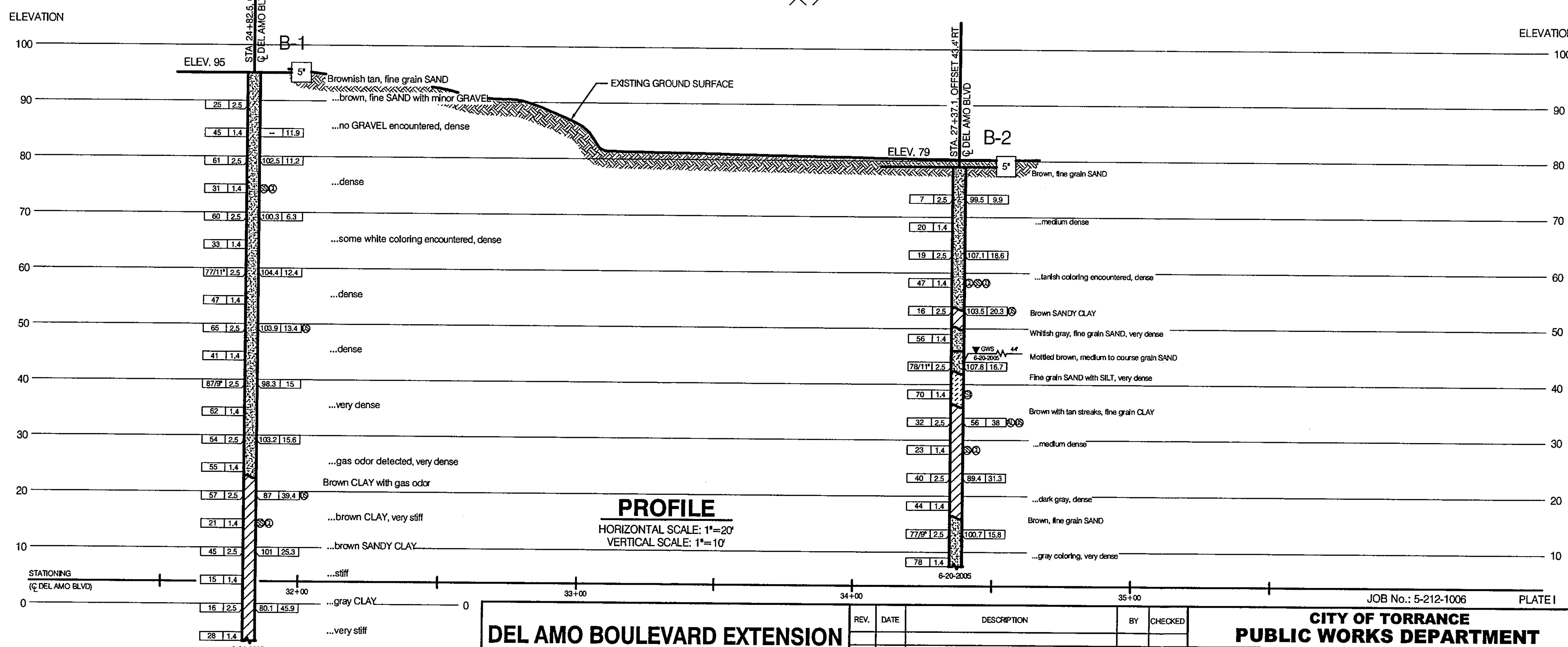
5-212-100600	B 7/4	Depth 10'	Date 07-18-05



amec

PLATES





NOTES:

1.) BENCHMARK:

2.) SAMPLER SIZES AND HAMMER WEIGHTS:
1.4 INCH I.D./2 INCH O.D. - 140 POUNDS; 30-INCH DROP
2.4 INCH I.D./2.75 INCH O.D. - 300 POUNDS; 12-INCH DROP

2.) GROUNDWATER ENCOUNTERED AT APPROXIMATE ELEVATION OF 44 FEET IN BORING B-2. GROUNDWATER ELEVATION NOT MEASURED IN BORING B-1.

JOB No.: 5-212-1006 PLATE I

DEL AMO BOULEVARD EXTENSION LOG OF TEST BORINGS

AMEC Earth & Environmental, Inc.
1290 North Hancock Street Anaheim, Ca 92807

REV.	DATE	DESCRIPTION	BY	CHECKED

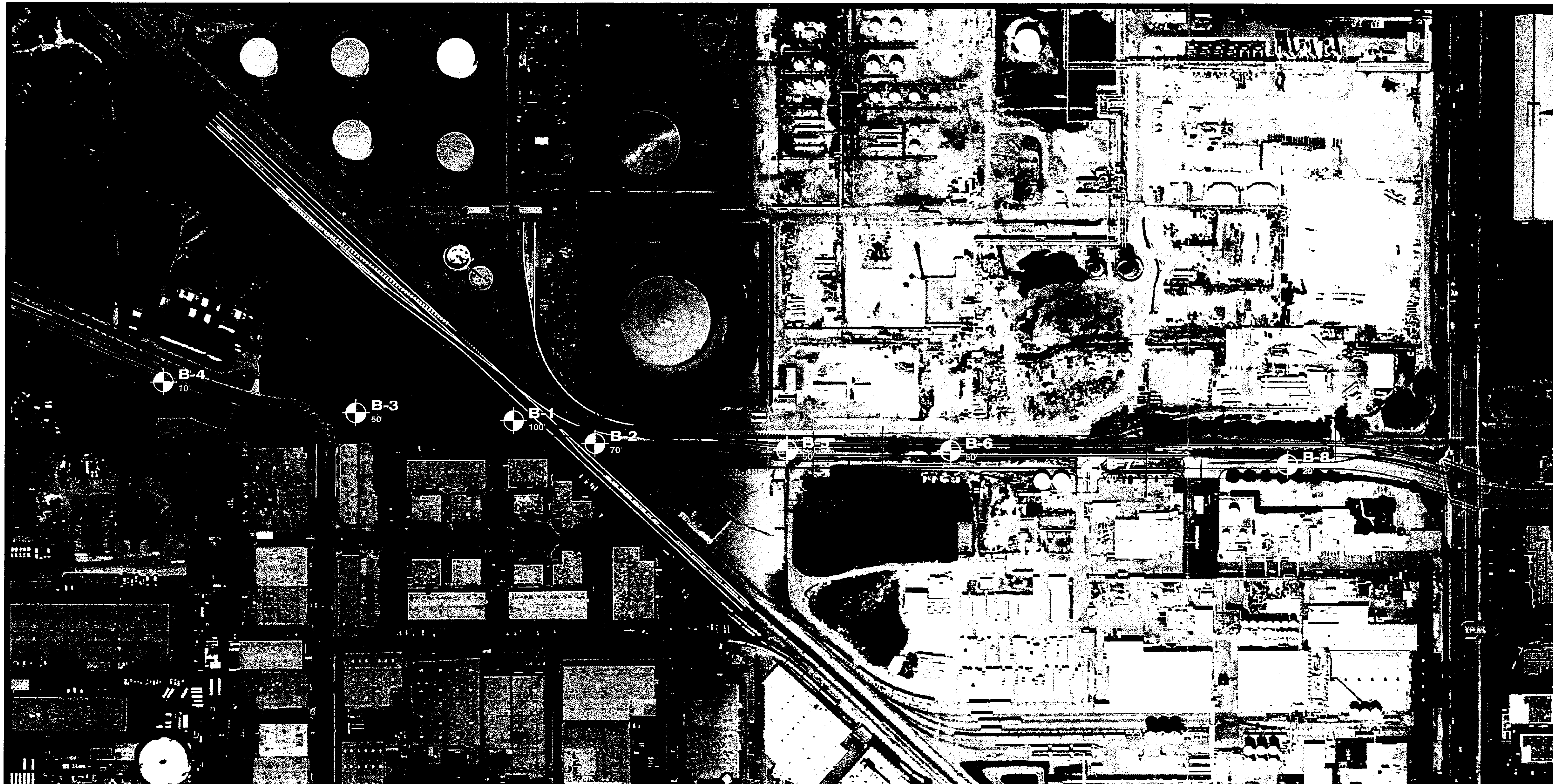
CITY OF TORRANCE PUBLIC WORKS DEPARTMENT

DRAWN: JAMES B. DePOORTER
DESIGNED: WILLIAM LU
PROJECT MANAGER: DOUGLAS DAHNCKE
APPROVED:
CRAIG BILEZERIAN CITY ENGINEER R.C.E. NO. 55339 EXP. 12/31/06 DATE

SCALE: AS SHOWN SHEET 98 OF 129

SHEET S-300

PLAN NO.: ST-1013



LEGEND

APPROXIMATE BORING LOCATION

N

SCALE: 1"=300'

DEL AMO BOULEVARD EXTENSION PROJECT CRENSHAW BOULEVARD TO PRAIRIE AVENUE CITY OF TORRANCE, CALIFORNIA				
BORING LOCATIONS				
AMEC Earth & Environmental, Inc.				
FIELD	DRAFT	APPROV	DATE	JOB No.
DD	JBD		Dec. 2005	5-212-1006

PLATE II

FILE: 5-212-100600-002 PLOT DATE: 12/14/2005