ORANGE COUNTY WATER DISTRICT
18700 Ward Street
Fountain Valley, California 92708-8300

ADDENDUM NO. 1
TO
CONTRACT DOCUMENTS
FOR
BOND BASIN SLOPE REPAIR
CONTRACT NO. SB-2025-1

THE BIDDER SHALL EXECUTE THE CERTIFICATION AT THE END OF THIS ADDENDUM AND SHALL ATTACH IT TO THE PROPOSAL SUBMITTED.

The following corrections, clarifications, additions, and/or deletions are made to the Contract Documents:

1. Replace Bid Document language for Insurance Conditions on Page IC-1 Number 1. as follows:

   General Liability – Commercial General Liability (CGL) – Insurance Services Office (ISO) Commercial General Liability Coverage (Occurrence Form CG 00 01) including products and completed operations, property damage, bodily injury, personal and advertising injury with limit of at least ten million dollars ($10,000,000) per occurrence or the full per occurrence limits of the policies available, whichever is greater. If a general aggregate limit applies, either the general aggregate limit shall apply separately to this project/location (coverage as broad as the ISO CG 25 03, or ISO CG 25 04 endorsement provided to the OCWD) or the general aggregate limit shall be twice the required occurrence limit.

2. Within Technical Specification Section 1.09 (Summary of Work), **DELETE** the following:

   "The District will lower the level of the basin to elevation 200 between June 1st to September 30th."

   REPLACE with the following:

   "The District will lower the level of the basin to elevation 200 between July 14th to September 30th."

3. The attached Questions and Answer Table to the Contract Documents are hereby incorporated in the Contract Documents for Contract No. SB-2025-1, which is referenced as Attachment A-1.

4. The following are provided as reference documents for clarification purposes:

   a. Attachment B-1 – Site Photos Water Surface Elev 220
      i. 11 photos of slope failure

   b. Attachment B-2 – Site Photos Water Surface Elev 243
      i. 7 photos of slope failure

   c. Attachment B-3 – Investigations and Reports
      i. Reports named in Special Provisions Section 100.G

   d. For existing topography files and the final grading plan please submit an email request to procurement@ocwd.com, and include SB-2025-1 in the email subject header.

THE BIDDER SHALL EXECUTE THE CERTIFICATION AT THE END OF THIS ADDENDUM AND SHALL ATTACH IT TO THE PROPOSAL SUBMITTED.

ORANGE COUNTY WATER DISTRICT

Signed by:
john kennedy
1D6D7D3D69BE464...

John C. Kennedy
General Manager

3/7/2025
Date

END OF ADDENDUM

MANDATORY FORM

ACKNOWLEDGE RECEIPT OF ADDENDUM NO. 1

CONTRACT DOCUMENTS
FOR
BOND BASIN SLOPE REPAIR
CONTRACT NO. SB-2025-1

March 10, 2025

THE BIDDER SHALL EXECUTE AND ATTACH THE FOLLOWING CERTIFICATION TO THE PROPOSAL.

BIDDER'S CERTIFICATION

I acknowledge receipt of the foregoing Addendum No. 1 to Contract No. SB-2025-1 and accept all conditions contained therein:

Date: ______________________________

Bidder: ______________________________

By: ______________________________

ATTACHMENT A-1

SB-2025-1 Bond Basin Slope Repair
Q&A Table

No.	QUESTIONS	ANSWERS
1	Does general liability insurance need to cover $5,000,000 per occurrence, or $10,000,000 per occurrence?	General Liability Insurance limit shall be at least $10,000,000 per occurance
2	Is builder's risk coverage required?	Yes, Builders All Risk Isurance puresuant to Public Contracting Code Section 7105 is required.

ATTACHMENT B-1

ATTACHMENT B-2

ATTACHMENT B-3

WILLDAN Geotechnical | extending your reach

May 11, 2009

Ms. Sandy Scott
Orange County Water District
18700 Ward Street
Fountain Valley, California 92708

Subject: **Report of Geotechnical Investigation**
Santiago Pits Intertie Project
Orange County, California
Willdan Geotechnical Project No. 17637-2000

Dear Mr. Jones:

Willdan Geotechnical Inc. (Willdan) is pleased to present this geotechnical investigation report for the Santiago Pits Intertie project in Orange County, California. This report presents the results of our investigation and recommendations for your use in preparing the project plans and specifications.

The Santiago Basins Intertie Project consists of construction of a connection between the north and south basins to allow the District greater flexibility in managing recharge water. This study will also address the temporary and long term stability of the west facing embankment slope below north Hewes Street. The work for this investigation has been performed in accordance with our proposal dated September 8, 2008.

We appreciate the opportunity to be of service for this project and look forward to assisting your in future projects. If you have any questions, please do not hesitate to contact us.

Respectfully submitted,

WILLDAN GEOTECHNICAL

REGISTERED PROFESSIONAL ENGINEER
ROSS KHIABANI
No. 2202
EXP. 6/30/10
GEOTECHNICAL
STATE OF CALIFORNIA

Ross Khiabani, PE, GE
Principal Engineer

Sean A. Mantegh, Ph.D.
Senior Staff Engineer

Distribution: (4) Addressee

REPORT

GEOTECHNICAL INVESTIGATION
FOR
THE SANTIAGO PITS INTERTIE PROJECT
ORANGE COUNTY, CALIFORNIA

Prepared for

Orange County Water District
18700 Ward Street
Fountain Valley, California 92708

Prepared by

Willdan Geotechnical
1515 South Sunkist Street, Suite E
Anaheim, California 92806
Willdan Geotechnical Project No. 17637-2000

May 11, 2009

TABLE OF CONTENTS

APPENDICES

1.0 INTRODUCTION

1.1 EXISTING SITE CONDITION

The project site is located at Santiago Open Pits Intertie, Orange County, California (Figure 1). The latitude and longitude of the site are 33.8043° N, 117.8059° W, respectively. The site elevation adjacent to N. Hewes road is approximately 330 feet above mean sea level (MSL).

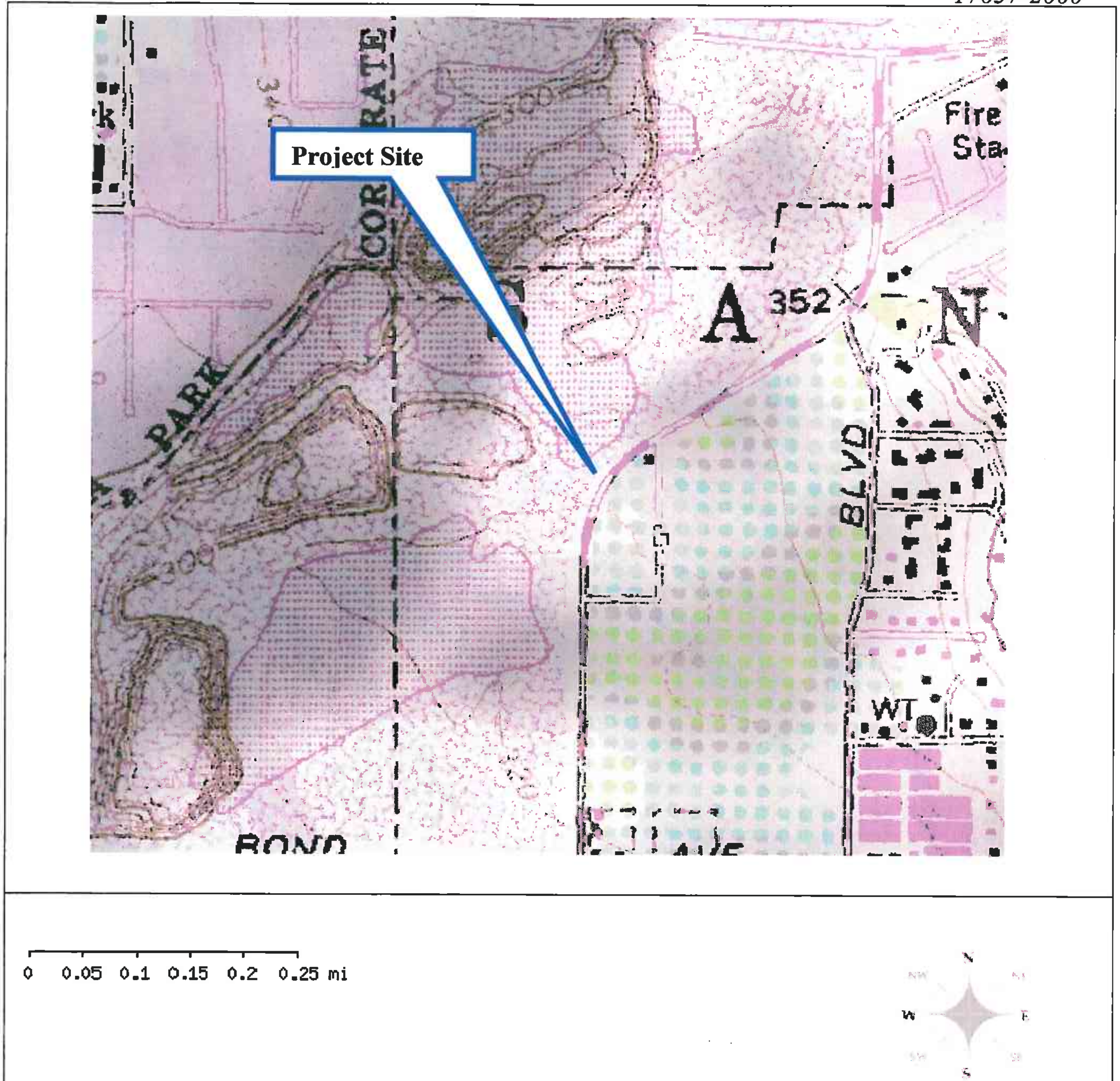
1.2 PROJECT DESCRIPTION

The Santiago Basins Intertie project consists of construction of a pipe connection between the north and south basins to allow the District greater flexibility in managing recharge water. This study will also address the stability of the west facing embankment slope below north Hewes Street.

1.3 PURPOSE AND SCOPE

The purpose of this investigation is to evaluate the subsurface soil conditions and to provide recommendations to aid in preparation of the project plans and specifications to accomplish the proposed construction. Based on the current project plan, the scope of work for this investigation included the following:

- Review of published and unpublished geologic and geotechnical documents;
- Drilling and sampling of one exploratory boring to the depths 136.5 feet;
- Laboratory testing of representative bulk and relatively undisturbed soil samples;
- Geotechnical analyses to develop recommendations for design and construction of the planned ;
- Presenting our findings, conclusions, and recommendations in this report.



WILLDAN Geotechnical

SITE LOCATION MAP

Figure 1

Santiago Pits, Orange County, California	Project No. 17637-2000	Date: 5-11-2009

2.0 FIELD EXPLORATION AND LABORATORY TESTING

2.1 FIELD EXPLORATION

Field exploration for this investigation consisted of drilling and sampling one soil boring to the depths of 136.5 feet below existing site grade. Approximate boring location is shown on Figure 2, Boring Location Map.

Soil borings were advanced using a trucked-mounted hollow-stem rig (CME 95) equipped with an 8-inch diameter flight auger. Both bulk and split-spoon samples, using 3-inch diameter rings (Modified California Sampler) were collected during drilling. Bulk samples were collected from the soil cutting generated as the auger advanced. The split-spoon samples were collected by driving the sampler at selected depths with a 140-pound hammer dropping at 30 inches. Blow counts records for every 6 inches of penetration of the sampler and description of the soil materials encountered during drilling were entered into the boring logs in accordance with the Unified Soil Classification System (USCS). The boring logs are included in Appendix A.

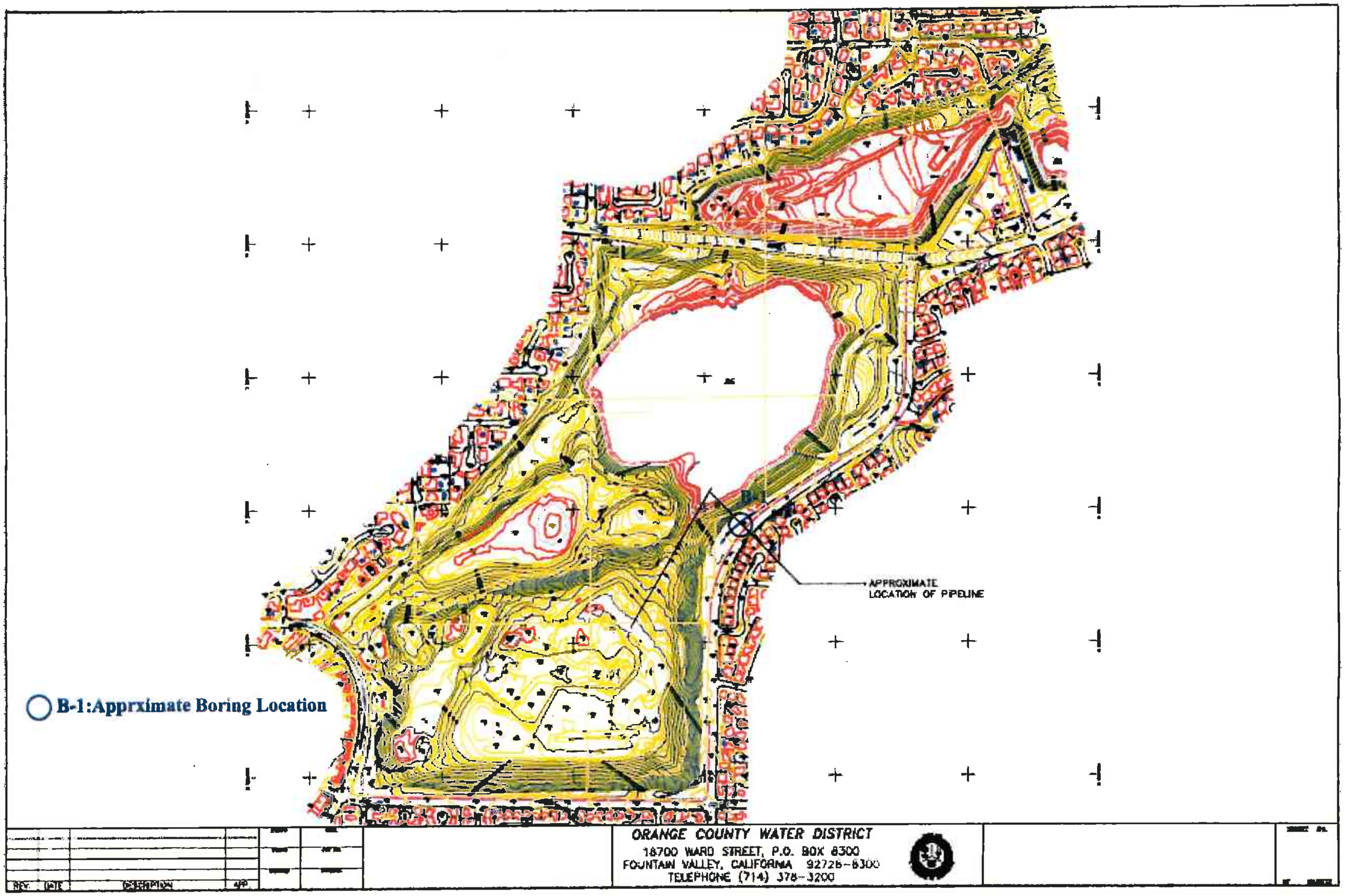
Upon completion of drilling, the borings were backfilled with soil cuttings. Soil samples collected from the field were delivered to Willdan's Geotechnical laboratory for testing.

2.2 LABORATORY TESTING

Laboratory tests were conducted on selected samples of the earth materials to determine their physical properties and engineering characteristics. The following laboratory tests were performed for this project:

- In-situ Moisture Content and Dry Density;
- Sieve Analysis;
- Direct Shear; and
- Soil Corrosivity (Minimum Resistivity, pH, Sulfate Content and Chloride Content).

The laboratory tests were conducted in general accordance with American Society for Testing of Materials (ASTM) Standards or California Test Methods. The laboratory test results are provided in Appendix B. The in situ-moisture content and dry density test results are shown on the boring logs.



WILLDAN Geotechnical

Boring Location Map	
Date: 5/11/2009	Figure : 2
Willdan Project: 17637-2000	

3.0 SUBSURFACE CONDITIONS

3.1 SOIL CONDITIONS

Based on the material encountered during drilling, the subsurface soils consist of predominately poorly graded gravel and silty sand. Based on blow counts records during exploration, the consistency of the soil material is generally medium dense to very dense. A more detailed description of the subsurface soil stratigraphy is presented in the boring logs included in Appendix B.

3.2 GROUNDWATER

Groundwater was not encountered during drilling performed on October 29, 2008. The historical highest groundwater was reported more than 40 feet below ground surface in the general areas (CDMG, 1997).

4.0 CONCLUSIONS AND RECOMMENDATIONS

4.1 GENERAL CONCLUSIONS

Based on result of our investigation, the proposed construction of intertie pipe at the site for its intended use is feasible. Presented below are recommendations for pipeline installation, and other geotechnical aspect of the project for incorporating in the project plan and specifications. Two main approaches of trenchless technology techniques and open cut are suggested in the following sections:

4.2 TRENCHLESS TECHNOLOGY

Trenchless techniques are defined as the techniques used for new installation, as well as renewal and inspection of pipes, with minimum excavation from the surface (NASTT, 2000).
The increase in public awareness of social costs (e.g., restoration costs, the need to dig around existing utilities) and the need to protect the environment have led to growth in utilization of trenchless technologies in comparison to open cut excavation methods.

Considering ground condition and expected diameter of the proposed pipe, microtunneling, pipe jacking and auger boring are the most feasible trenchless methods in this project

Microtunneling: is a form of pipe jacking using remotely controlled machines, operated and steered from the surface. Based on the soil removal system, microtunneling methods are divided into slurry and auger. The presence of gravel and boulder as the predominant stratigraphy in the project site suggests slurry microtunneling as the most favorable method. The following provides more detailed information on microtunneling methods:

Auger Boring: consists of simultaneously jacking the pipe through the earth while removing the spoil inside the pipe by means of a rotating cutting head attached to a flight of augers. The auger transmits torque to the cutting head and transfers spoil back to the machine. Table 1 provides typical auger boring parameters.

Pipe jacking is a technique of installing pipes behind a shield machine by hydraulic jacks from a jacking pit (NASTT, 2000). While the pipeline is pushed, soil is excavated in a shield forward of the leading pipe by hand or machine depending on the ground conditions. After the first pipe is jacked forward, the jacks are retracted and another pipe is placed to be jacked behind the first one. The process continues until it is completed. Typical pipe jacking parameters are provided in Table 1

In the microtunneling process, a remotely controlled tunneling machine is driven from the entry point to the reception point. The tunneling machine is launched through a steel ring, which isolates the ground surface by auger or slurry. As the machine advances, jacking pipe is added at the entry point. A laser and hydraulic steering jacks inside the tunneling machine are applied to control the position of machine. The process is mostly used in deep pipe installations. The two main cutting removal systems in microtunneling consist of slurry and auger.

TABLE 1. TYPICAL INSTALLATION PRAMETERS (AFTER MANTEGH, 2001)

Method	Pipe Length (ft)	Pipe diameter (inches)	Pipe material	Area requirement
Auger Boring	39 - 492	7.8 -35	Steel Casing	Length : 16 -36 ft Width: 8.2 – 11.8 ft
Pipe Jacking	Up to 1607	41 - 119	RCP, steel	Jacking pit is a function of pipe size. Pit sizes vary from 10 to 29.5 ft
Microtunneling (Slurry)	~ 490	5 - 98	(RCP), (GRP), (DIP), (VCP), Steel, (PVC), (PCP)	Length : 175 ft Width: 25 ft
Microtunneling (Auger)	~340	<5	RCP, GIP, DIP, VCP, Steel, PVC, PCP	Length : 175 ft Width: 25 ft

Reinforced Concrete pipe (RCP)
Glass fiber Reinforced Polyester (GRP)
Ductile Iron Pipe (DIP)
Vitrified Clay Pipe (VCP)
Steel, Poly-Vinyl Chloride (PVC)
Polymer Concrete Pipe (PCP)

4.3 OPEN CUT

Temporary excavations must be properly shored or appropriately laidback. Based on the earth materials encountered in our borings and slope stability analysis discussed in the following section, an excavation of 75 feet or less in depth, below existing berm (EL. 245) may be performed with 1:1 (H:V) side slope maximum. This option is preferred to be excavated in two sections.

Prolonged and permanent excavation of 20 feet or less in depth with a maximum 1.5:1 (H:V) layback is also feasible.

The contractor is responsible for worker safety in the field during construction. The contractor shall conform to all applicable occupational safety and health standards, rules, regulations, and orders established by the State of California.

4.3.1 Slope Stability

Slope stability analyses were performed to evaluate the stability of the existing slope and slope under temporary and permanent conditions. Cross sections B-B' was considered to address the stability of the slopes between the Bond and Blue Diamond basins. The cross section location is provided on Figure 3.

The computer program GSTAB7 was used in the analyses. The Modified Bishop method option of GSTAB7 designed to analyze circular failure surfaces on different slope angles was used for these analyses

Test results show soil ultimate strengths of cohesion (C) ranged between 0 to 80 psf and friction angle (φ) ranged between 26 and 37.5 degrees. For the purpose of this project soil strengths of (C=100 pcf and φ= 37.5 degrees) developed by WCC (1985) were utilized to address the stability of the existing and permanent excavation of 1.5:1 (H:V) slope in the proposed location of the berm and pipeline connecting Bond and Blue Diamond basins (cross section B-B'). These strengths represent a reasonable approximation of the upper-bound strength of gravelly materials encountered in our boring. The stability analyses show that the slope exceed the required safety factors of 1.5 and 1.1 for static and pseudostatic conditions respectively .

The temporary stability of the open trench excavation for options of 25, 50 and 75-foot trench excavations below the existing berm (EL 245) were also analyzed using soil strengths of (C=100 pcf and φ= 45 degrees). The maximum slope for trench excavations of 1:1 (H:V) is proposed without any external stabilizing forces.

The above temporary excavations may also be accomplished with vertical side walls provided the trench is shored. The shoring should be designed for external forces of 1.8, 21.5 and 59.2 kip/ft, for 25 and 50 and 75 feet deep trenches, respectively, to maintain a safety factor of 1.2 under temporary condition.

The results illustrate that the slope meet the required safety factors (fs$\geq$ 1.2) for all the above depths. Table 2 shows the results obtained from slope stability analyses for Cross Section B-B' under existing, permanent and temporary conditions. Appendix C also shows the plots and computer output for the stability analyses.

TABLE 2. SLOPE STABILITY ANALYSES FOR CROSS SECTION B-B'

<table>
<tr><th rowspan="2">Condition</th><th rowspan="2">Total Unit Weight (pcf)</th><th colspan="2">Shear Strength Parameters</th><th rowspan="2">Depth (ft)</th><th rowspan="2">Cut Slope Angle (H:V)</th><th rowspan="2">External Force (Kips/ Ft)</th><th rowspan="2">Fs (Static)</th><th rowspan="2">Fs (Pseudo-Static)</th></tr>
<tr><th>(C) (psf)</th><th>(φ) (Degree)</th></tr>
<tr><td>Existing</td><td>120</td><td>100</td><td>37.5</td><td>NA</td><td>NA</td><td>NA</td><td>2.04</td><td>1.35</td></tr>
<tr><td>Permanent Excavation</td><td>120</td><td>100</td><td>37.5</td><td>20</td><td>1.5:1</td><td>NA</td><td>1.55</td><td>1.17</td></tr>
<tr><td rowspan="6">Temporary Excavation</td><td rowspan="6">120</td><td rowspan="2">100</td><td rowspan="2">45</td><td rowspan="2">25</td><td>1:1</td><td>NA</td><td>1.62</td><td rowspan="2">NA</td></tr>
<tr><td>0:1 (Vertical)</td><td>1.8</td><td>1.21</td></tr>
<tr><td rowspan="2">100</td><td rowspan="2">45</td><td rowspan="2">50</td><td>1:1</td><td>NA</td><td>1.36</td><td rowspan="2">NA</td></tr>
<tr><td>0:1 (Vertical)</td><td>21.5</td><td>1.20</td></tr>
<tr><td rowspan="2">100</td><td rowspan="2">45</td><td rowspan="2">75</td><td>1:1</td><td>NA</td><td>1.26</td><td rowspan="2">NA</td></tr>
<tr><td>0:1 (Vertical)</td><td>59.2</td><td>1.20</td></tr>
</table>

4.4 PIPE TRENCH BEDDING AND BACKFILL

Bedding materials consisting of sand, gravel, or crushed aggregate should be used to backfill around pipe and trench backfill. The fill should be placed in loose lifts not to exceed 8 inches, moisture-conditioned to 2 to 4 percent above optimum, and mechanically compacted to at least 90 percent relative compaction in accordance with ASTM D1557. Use of onsite native material will be acceptable as backfill material.

4.5 REVIEW OF CONSTRUCTION PLANS

Recommendations contained in this report are based on preliminary plans. The geotechnical consultant should review the final construction plans and specifications in order to confirm that the general intent of the recommendations contained in this report have been incorporated in the documents. Recommendations contained in this report may require modification or additional recommendations may be necessary based on the final design.

4.6 GEOTECHNICAL OBSERVATION AND TESTING

It is recommended that all excavation and installation of pipeline be performed under the inspection and testing of the geotechnical consultant during the following stages of construction:

- Excavations and backfilling for pipe trenches; and
- When any unusual subsurface conditions are encountered.

5.0 CLOSURE

This report is intended for the use by Orange County Water District and it's consultants for the design of the proposed Santiago Pits Intertie Project in the City of Orange, Orange County, California.

The findings and recommendations contained in this report are based on the results of the field investigation, laboratory tests, and engineering analyses.

Services performed by Willdan Geotechnical have been conducted in accordance with generally accepted professional geotechnical engineering principles and practices at this time. No other representation, express or implied, and no warranty or guarantee is included or intended.

6.0 REFERENCES

American Society for Testing and Materials (ASTM, 2000). Annual Book of Standards. Soil and Rock; Dimension Stone; Geosynthetics. Vol. 04.08.

Green Book (Standard Specifications for Public Works Construction

State of California, Division of Mines and Geology (CDMG), (1997) Seismic Hazard Zones, Orange 7.5 Minute Quadrangle, Orange County.

Mantegh, A. 2001. Evaluation of Site Investigation Techniques for Geological Hazard Assessment for new Pipes Installed Using Trenchless Technology Techniques. M.Sc Thesis. University of Waterloo, Canada.

North American Society for Trenchless Technology (NASTT 2000. Glossary of Terms. http://www.nastt.org/glossary/ghtml

Woodward-Clyde Consultants (WCC), August 1985. Geotechnical Investigation Santiago Creek Replenishment Project Site Improvements. Orange County, California

APPENDIX A. BORING LOG

MAJOR DIVISIONS			SYMBOLS		TYPICAL NAMES
COARSE GRAINED SOILS Half is larger than no. 200 sieve	GRAVELS More than half coarse fraction is larger than no. 4 sieve	Clean gravels with little or no fines	GW		Well graded gravels, gravel-sand mixtures
			GP		Poorly graded gravels, gravel-sand mixtures
		Gravels with over 12% fines	GM		Silty gravels, poorly graded gravel-sand-silt mixtures
			GC		Clayey gravels, poorly graded gravel-sand-clay mixtures
	GRAVELS More than half coarse fraction is smaller than no. 4 sieve	Clean sands with little or no fines	SW		Well graded sands, gravelly sands
			SP		Poorly graded sands, gravelly sands
		Sands with over 12% fines	SM		Silty sands, poorly graded sand-silt mixtures
			SC		Clayey sands, poorly graded sand-clay mixtures
FINE GRAINED SOILS Half is smaller than no.	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 clays and organic silty clays of low plasticity
	SILTS AND CLAYS Liquid limit greater than 50		MH		Inorganic silts, micaceous or diatomaceous fine, sandy or silty soils, elastic silts
			CH		Inorganic clays of high plasticity, fat clays
			OH		Organic clays of medium to high plasticity, organic silts
HIGHLY ORGANIC SOILS			Pt		Peat and other highly organic soils

SAND AND GRAVEL	BLOWS/FOOT*
VERY LOOSE	0 - 4
LOOSE	5 - 10
MEDIUM DENSE	11 - 30
DENSE	31 - 50
VERY DENSE	OVER 50

RELATIVE DENSITY

SILTS & CLAYS	STRENGTH (ksf)	BLOWS/FOOT*
VERY SOFT	0 - 1/4	< 2
SOFT	1/4 - 1/2	2 - 4
FIRM	1/2 - 1	5 - 8
STIFF	1 - 1.5	9- 15
VERY STIFF	1.5 - 2	16 - 30
HARD	OVER 2	OVER 31

CONSISTENCY

* Applicable only for Standard Penetration Tests (ASTM D-1586)

STANDARD PENETRATION TEST
Split Barrel sampler in accordance with ASTM D 1586-84

DRIVE SAMPLE
2.42" inside diameter, 140# weight, 30" drop (unless otherwise specified on boring log)

NO SAMPLE RECOVERY

BULK SAMPLE
Loose cuttings from exploration

WATER TABLE

TEST TYPE
Results shown in Appendix B

	OTHER
Chemical Analysis	CA
Sieve Analysis	SA
Unconfined Compression	UC
Hydrometer Analysis	HA
Expansion Index	EI
Compaction	Max
% Passing #200 Sieve	W
Pocket Penetrometer	p
Direct Shear	DS
Direct Shear (Remolded)	DS_R
Atterberg Limits	AL
Consolidation	CN
R-Value	R

EXPLORATION LOG KEY

WILLDAN Geotechnical

Project No. 17637-2000

Figure No. A-1

LOG OF BORING B-1

Borehole Location: **See Figure 2**	Approx. Elevation:		Sheet 1 of 3
Borehole Coordinates:	Date Started: **10/29/08**	Date Finished: **10/29/08**	
Drilling Equipment: **CME 75**	Total Depth: **136.5 ft**	Depth to Groundwater: **GW Not Encountered**	
Drilling Method: **Hollow Stem Auger**	Borehole Diameter: **8 inches**		
Driller: **Redman Drilling Inc.**	Logged By: **SM**	Checked By: **RK**	
Hammer Information: **Weight-140 lbs and Drop-30 inches**			

Elevation (ft)	Depth (ft)	Lithology	Description	Remarks	Sampler	Number	Blows/6"	Moisture Content (%)	Dry Density (pcf)	Additional Tests
	0		Clayey Sand with Gravel (SC), very dense, brown, moist							
	5–10					Bulk 1		5.6		SA
	10		Clayey Gravel with Sand (GC), very dense, brown, moist 3 rings recovered			R-1	9/12/22	8.7	108.4	
	20					R-2	50/6"	5.5	87.9	
	30		Clayey Sand with Gravel (SC No recovery at 30 feet			R-3	50/4"			DS
						Bulk 2				SA
	35		Clayey Gravel with Sand (GC), very dense, brown, moist			R-4	50/6"	4.2	91.3	DS
	40					R-5	50/6"	13.5	84.0	

WILLDAN Geotechnical

OCWD Santiago Pits

Project Number: 17637-2000

FIGURE A-2a

LOG OF BORING B-1

Borehole Location: See Figure 2	Approx. Elevation:		Sheet 2 of 3
Borehole Coordinates:	Date Started: 10/29/08	Date Finished: 10/29/08	
Drilling Equipment: CME 75	Total Depth: 136.5 ft	Depth to Groundwater: GW Not Encountered	
Drilling Method: Hollow Stem Auger	Borehole Diameter: 8 inches		
Driller: Redman Drilling Inc.	Logged By: SM	Checked By: RK	
Hammer Information: Weight-140 lbs and Drop-30 inches			

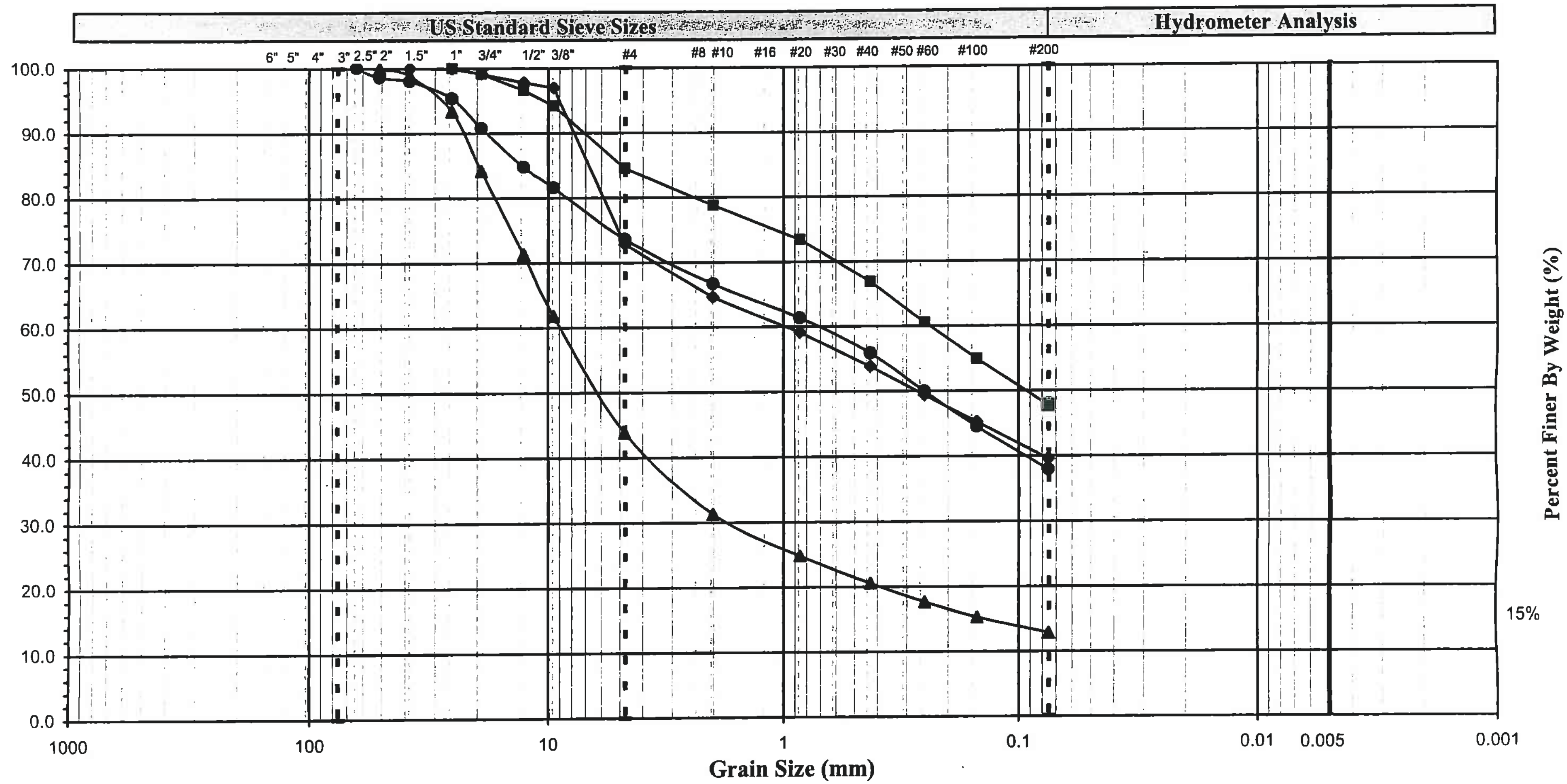
Elevation (ft)	Depth (ft)	Lithology	Description	Remarks	Sampler	Number	Blows/6"	Moisture Content (%)	Dry Density (pcf)	Additional Tests
	50	GC	Clayey Gravel with Sand (GC), very dense, brown, moist 2 rings recovered			R-6	50/6"	3.6	110.7	DS
	55									
	60		3 rings recovered			R-7	50/6"	3.8	95.8	
	65									
	70		4 rings recovered			R-8	50/6"	6.3	58.1	
	75		More sand appesrs							
	80	SC	Clayey Sand with Gravel (SC)			R-9	6/10/24	20	111.2	
	85	GC	Clayey Gravel with Sand (GC), very dense, brown, moist			Bulk 3				SA
	90		No recovery at 90 feet			R-10	50/6"			
	95		No recovery at 95 feet			R-11	50/6"			

LOG OF BORING B-1

Borehole Location: **See Figure 2**		Approx. Elevation:	Sheet 3 of 3
Borehole Coordinates:		Date Started: **10/29/08**	Date Finished: **10/29/08**
Drilling Equipment: **CME 75**		Total Depth: **136.5 ft**	Depth to Groundwater: **GW Not Encountered**
Drilling Method: **Hollow Stem Auger**		Borehole Diameter: **8 inches**	
Driller: **Redman Drilling Inc.**		Logged By: **SM**	Checked By: **RK**
Hammer Information: **Weight-140 lbs and Drop-30 inches**			

Elevation (ft)	Depth (ft)	Lithology	Description	Remarks	Sampler	Number	Blows/6"	Moisture Content (%)	Dry Density (pcf)	Additional Tests
	100		Clayey Gravel with Sand (GC), very dense, brown, moist 2 rings recovered			R-12	50/6"	6.3	81.7	DS
	110					R-13	50/6"	7.7	89.7	
						Bulk 4				SA
	120		Sandy Silt (ML), very stiff, light brown, moist			R-14	8/14/20	22.1	105.3	
	125		Poorly Graded Gravel (GP)/ Silty Gravel (GM), very dense, light brown, moist							
	130					R-15	50/6"			
	135					R-16	50/6"	5.6	91.9	
	136.5		Total Depth 136.5 ft GW Not Encountered Backfilled with Native Soil.							
	140									
	145									

APPENDIX B. LABORATORY TEST RESULTS



Cobbles	Gravel		Sand			Silt	Clay
	Coarse	Fine	Coarse	Medium	Fine		

Symbol	Boring Number	Sample Number	Depth (ft)		Depth (m)		Soil Color	Soil Description	U.S.C.S.
●	B - 1	B - 1	5.0	10.0	1.53	3.05	Brown	Clayey Sand with Gravel	SC
▲	B - 1	B - 2	30.0	35.0	9.15	10.68	Brown	Clayey Gravel with Sand	GC
■	B - 1	B - 4	85.0	89.0	25.93	27.15	Brown	Clayey Sand with Gravel	SC
◆	B - 1	B - 5	110.0	115.0	33.55	35.08	Brown	Clayey Sand with Gravel	SC
○									
△									
□									
Remark									

WILLDAN Geotechnical

OCWD Satiago Pits Intertie

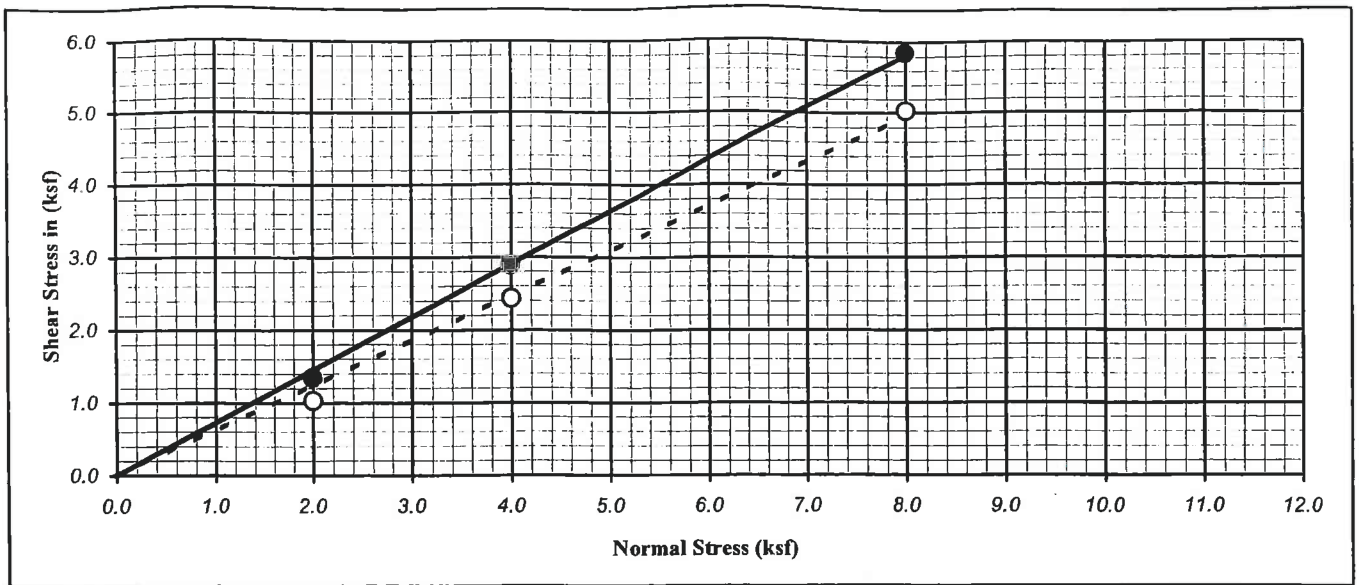
Project No. : 17637-2000 | Date : 11/25/08

GRAIN SIZE ANALYSIS

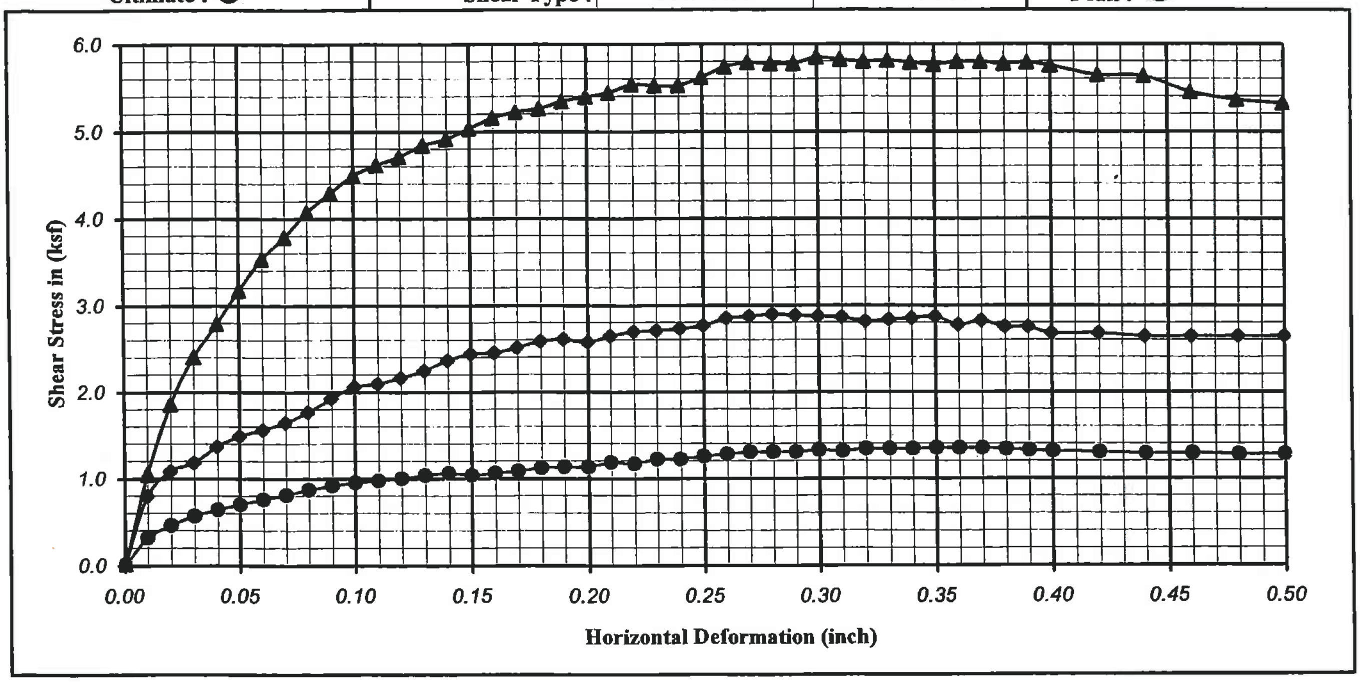
(ASTM D-422-63)

P.F. Chan (10-01-00)

Figure No. : B-1



Ultimate : ○	Shear Type :	Saturated	Remolded	Peak : ●



Boring No. :	B - 1			**Strength Intercept (C) :**	0.00	(ksf)	**Peak**	0.00 (ksf)	**Ultimate**
Sample No. :	R - 3				0.00	(kPa)		0.00 (kPa)	
Depth (ft/m) :	35.0	36.0	10.7 / 11.0	**Friction Angle (Ø) :**	35.99	Degree		31.78 Degree	
Description :	Brown, Clayey Gravel (GC)							**Shear Rate (inch/min.):**	0.01

SYMBOL	MOISTURE CONTENT (%)	DRY DENSITY (pcf)	DRY DENSITY (kN/m^3)	VOID RATIO	NORMAL STRESS (ksf)	NORMAL STRESS (kPa)	PEAK STRESS (ksf)	PEAK STRESS (kPa)	ULTIMATE STRESS (ksf)	ULTIMATE STRESS (kPa)
●	18.68	113.61	17.88	0.48	2.00	95.76	1.34	64.35	1.03	49.41
◆	18.68	113.61	17.88	0.48	4.00	191.52	2.89	138.47	2.44	116.64
▲	18.68	113.61	17.88	0.48	8.00	383.04	5.84	279.81	5.03	240.74
Remark	Sample -No. 4 Sieve									

OCWD Satiago Pits Intertie

DIRECT SHEAR TEST

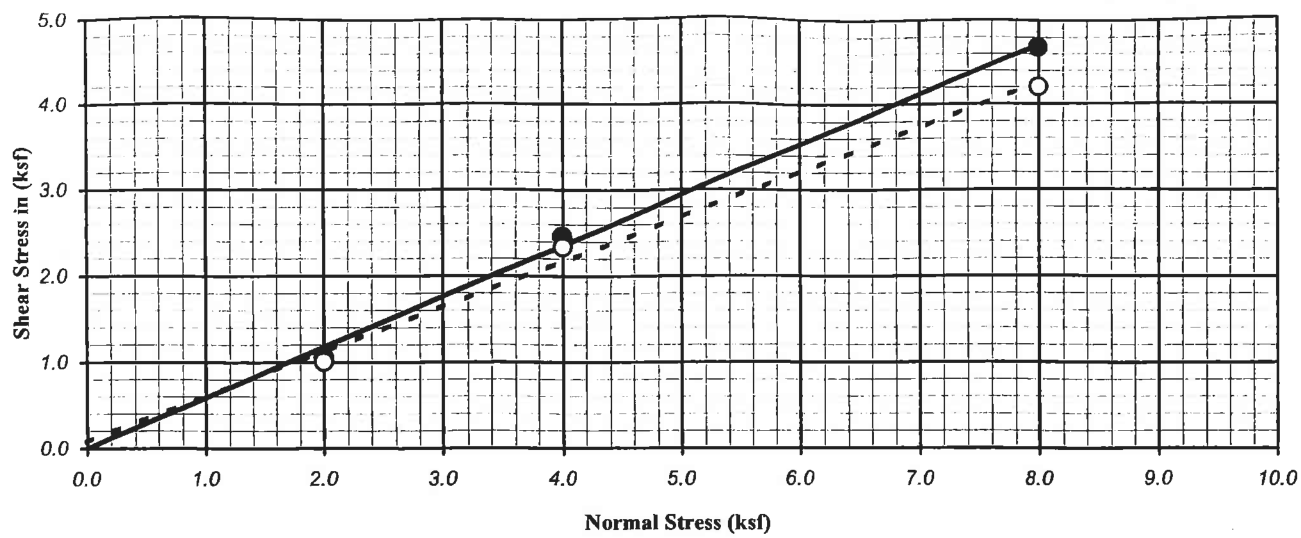
(ASTM D-3080 / T-236)

WILLDAN Geotechnical

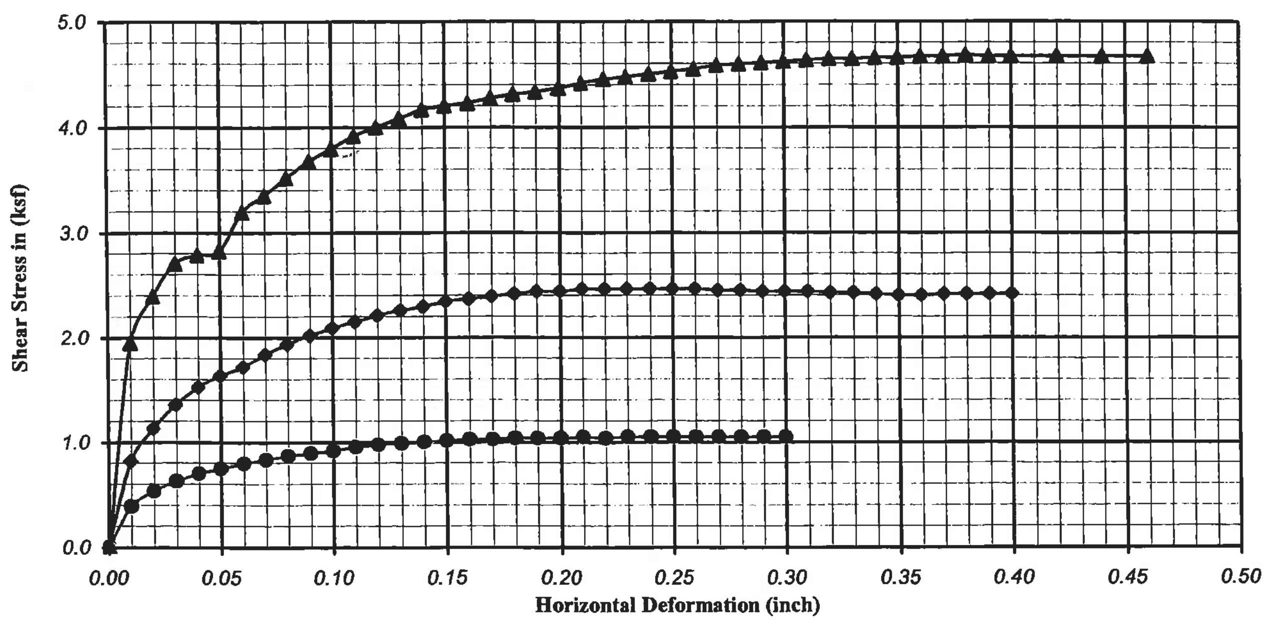
Project No. : 17637-2000

Date : 11/19/08

Figure No. : B-2



Ultimate : ○	Shear Type :		Peak : ●



Boring No. :	B - 1	Strength Intercept (C) :	0.00	(ksf)	Peak	0.08	(ksf)	Ultimate
Sample No. :	R - 4		0.00	(kPa)		3.73	(kPa)	
Depth (ft/m) :	40.0 / 41.0 / 12.2 / 12.5	Friction Angle (Ø) :	30.44	Degree		27.58	Degree	
Description :	Brown, Lean Clay with Sand & Gravel (CL)					Shear Rate (inch/min.):	0.005	

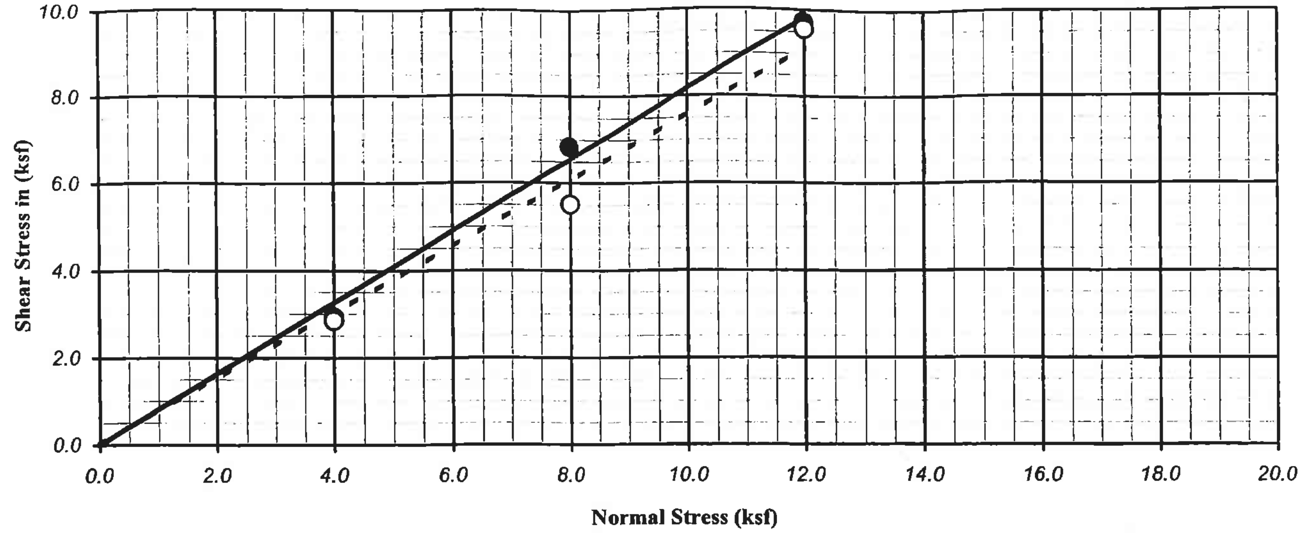
SYMBOL	MOISTURE CONTENT (%)	DRY DENSITY (pcf)	DRY DENSITY (kN/m^3)	VOID RATIO	NORMAL STRESS (ksf)	NORMAL STRESS (kPa)	PEAK STRESS (ksf)	PEAK STRESS (kPa)	ULTIMATE STRESS (ksf)	ULTIMATE STRESS (kPa)
●	20.84	106.88	16.82	0.58	2.00	95.76	1.04	49.99	1.01	48.26
◆	19.59	112.94	17.78	0.49	4.00	191.52	2.46	117.78	2.34	112.04
▲	20.09	115.92	18.25	0.45	8.00	383.04	4.68	224.08	4.20	201.10
Remark										

OCWD Satiago Pits Intertie

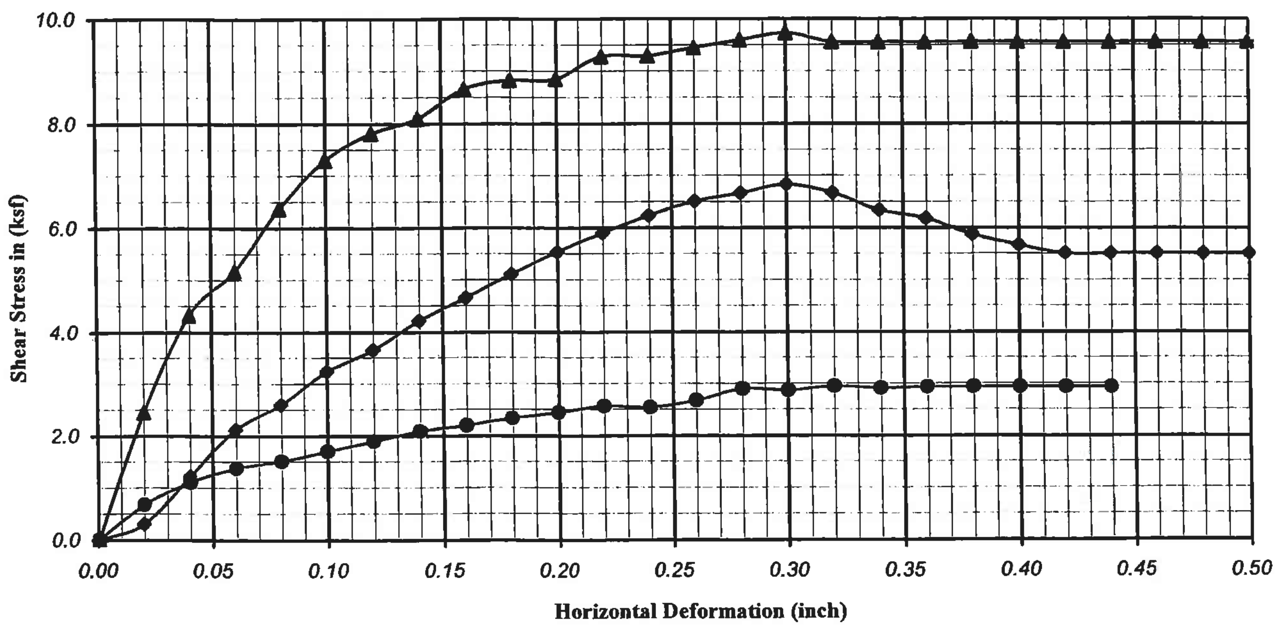
DIRECT SHEAR TEST
(ASTM D-3080 / T-236)

WILLDAN Geotechnical | Project No. : 17637-2000 | Date : 11/19/08

Figure No. : B-3



Ultimate : ○	Shear Type :	Saturated	Undisturbed	Peak : ●



Boring No. :	B - 1			Strength Intercept (C) :	0.00	(ksf)	Peak	0.00	(ksf)	Ultimate
Sample No. :	R - 6				0.00	(kPa)		0.00	(kPa)	
Depth (ft/m) :	60.0	61.0	18.3	18.6 Friction Angle (Ø) :	39.25	Degree		37.21	Degree	
Description :	Brown, Clayey Gravel with Sand (GC)							Shear Rate (inch/min.):	0.02	

SYMBOL	MOISTURE CONTENT (%)	DRY DENSITY		VOID RATIO	NORMAL STRESS		PEAK STRESS		ULTIMATE STRESS	
		(pcf)	(kN/m³)		(ksf)	(kPa)	(ksf)	(kPa)	(ksf)	(kPa)
●	22.27	105.03	16.53	0.60	4.00	191.52	2.94	140.77	2.86	136.75
◆	22.27	105.03	16.53	0.60	8.00	383.04	6.83	326.92	5.51	263.72
▲	22.27	105.03	16.53	0.60	12.00	574.56	9.72	465.35	9.55	457.30
Remark										

OCWD Satiago Pits Intertie

DIRECT SHEAR TEST

(ASTM D-3080 / T-236)

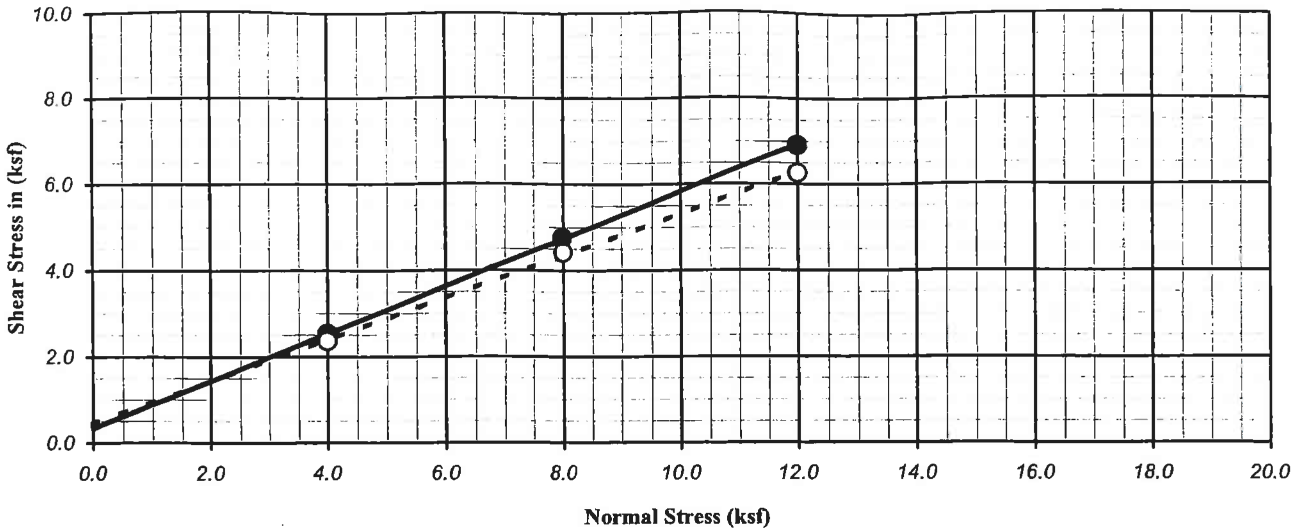
WILLDAN Geotechnical

Project No. : 17637-2000

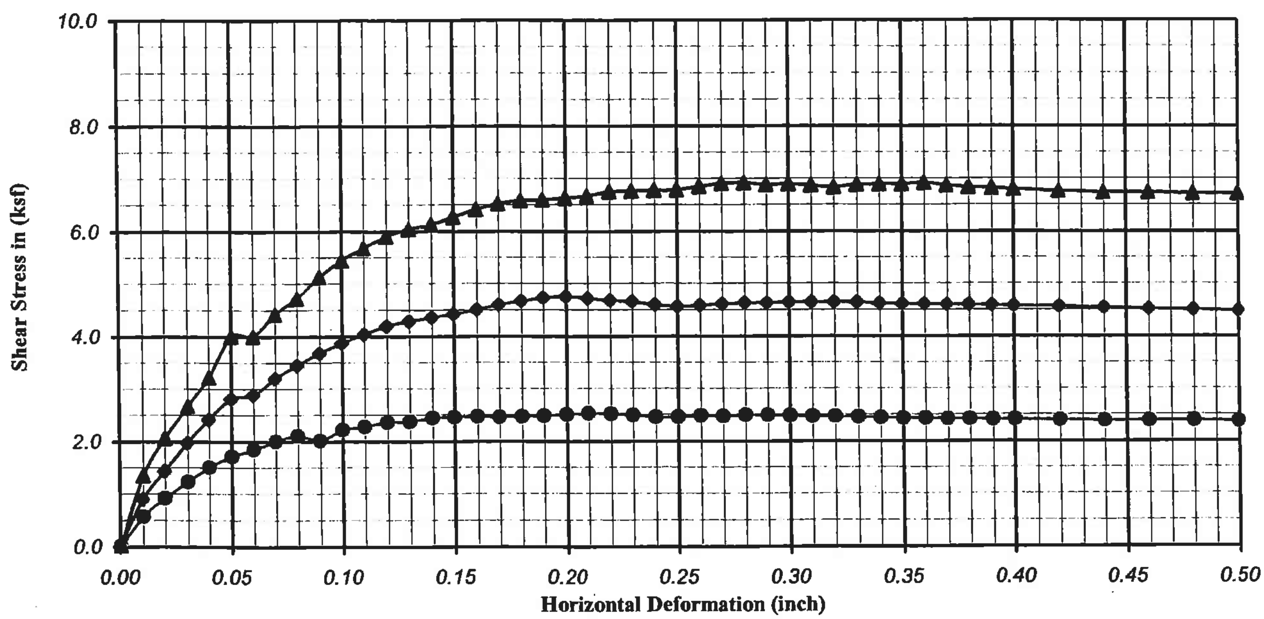
Date : 11/21/08

PF Chart (10-01-00)

Figure No. : B-4



Ultimate : ○	Shear Type :	Peak : ●



Boring No. :	B - 1			Strength Intercept (C) :	0.34	(ksf)	Peak	0.45	(ksf)	Ultimate
Sample No. :	R - 12				16.09	(kPa)		21.45	(kPa)	
Depth (ft/m) :	120	121	37	37 Friction Angle (Ø) :	28.77	Degree		25.99	Degree	
Description :	Brown, Lean Clay with Sand (CL)						Shear Rate (inch/min.):	0.005		

SYMBOL	MOISTURE CONTENT (%)	DRY DENSITY		VOID RATIO	NORMAL STRESS		PEAK STRESS		ULTIMATE STRESS	
		(pcf)	(kN/m^3)		(ksf)	(kPa)	(ksf)	(kPa)	(ksf)	(kPa)
●	22.12	107.29	16.89	0.57	4.00	191.52	2.52	120.66	2.36	113.19
◆	21.96	112.33	17.68	0.50	8.00	383.04	4.75	227.53	4.42	211.44
▲	20.11	116.46	18.33	0.45	12.00	574.56	6.91	330.95	6.26	299.92
Remark										

OCWD Satiago Pits Intertie

DIRECT SHEAR TEST

(ASTM D-3080 / T-236)

WILLDAN Geotechnical

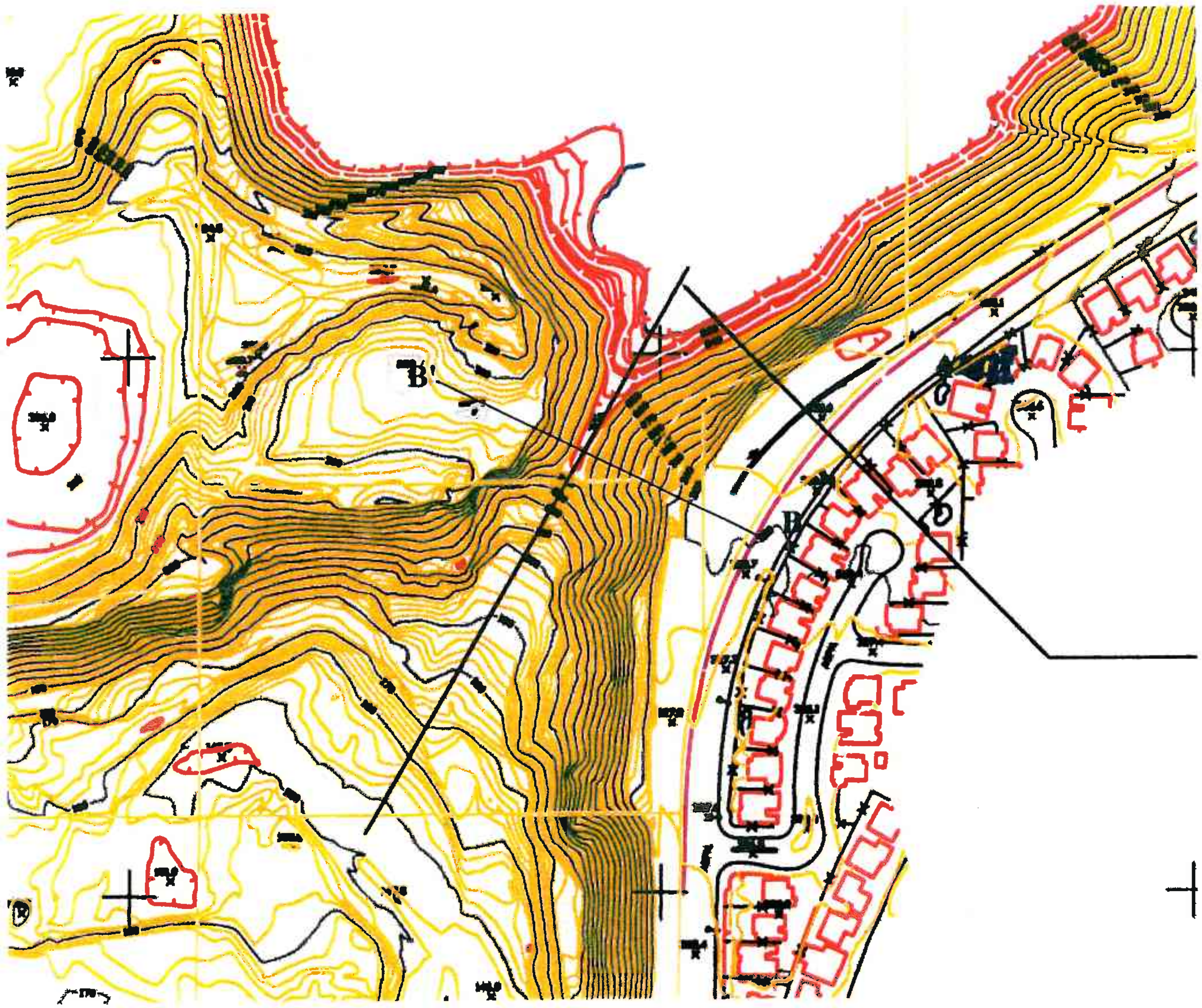
Project No. : 17637-2000

Date : 11/24/08

PF Chan (10-01-00)

Figure No. : B-5

APPENDIX C. SLOPE STABILITY ANALYSIS

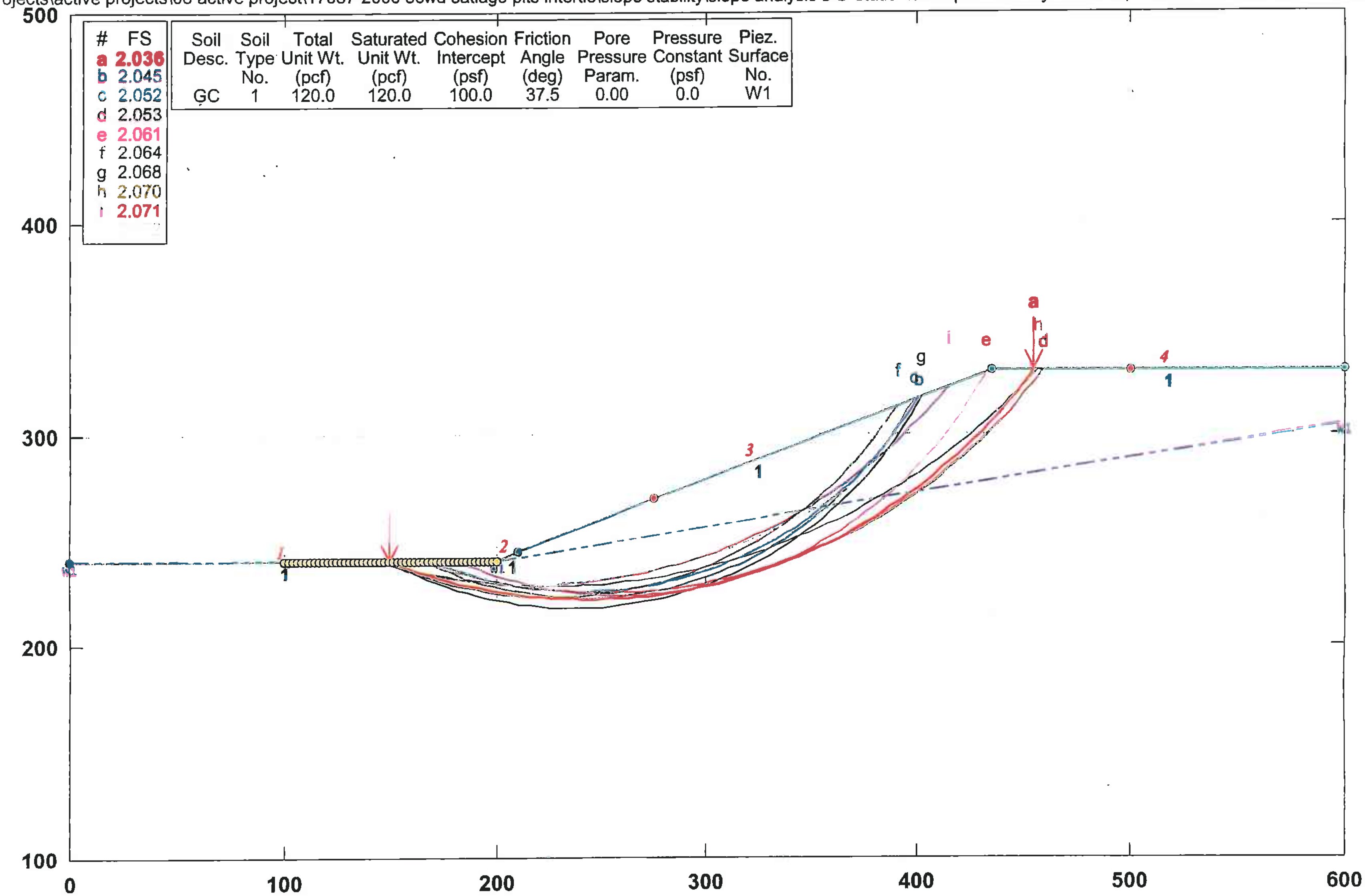


WILLDAN Geotechnical

Location of Cross Section B-B'	
Date:2/92009	Figure :3
Willdan Project: 17637-2000	

Cross Section B-B' Static Case (No Cut)

q:\all projects\active projects\08 active project\17637-2000 ocwd satiago pits intertie\slope stability\slope analysis b-b' static no cut.pl2 Run By: Sean. M, Willdan Geotechnical 3/3/2009 02:09PM



GSTABL7 v.2 FSmin=2.036

Safety Factors Are Calculated By The Modified Bishop Method

GSTABL7

***** GSTABL7 *****

** GSTABL7 by Garry H. Gregory, P.E. **
** Original Version 1.0, January 1996; Current Version 2.005, Sept. 2006 **

****

SLOPE STABILITY ANALYSIS SYSTEM
Modified Bishop, Simplified Janbu, or GLE Method of Slices.
(Includes Spencer & Morgenstern-Price Type Analysis)
Including Pier/Pile, Reinforcement, Soil Nail, Tieback,
Nonlinear Undrained Shear Strength, Curved Phi Envelope,
Anisotropic Soil, Fiber-Reinforced Soil, Boundary Loads, Water
Surfaces, Pseudo-Static & Newmark Earthquake, and Applied Forces.

****

Analysis Run Date: 3/3/2009
Time of Run: 11:27AM
Run By: Sean. M, Willdan Geotechnical
Input Data Filename: q:\All Projects\Active Projects\08 Active Project\17637-2000 OCWD Satiago Pits Intertie\Slope Stability\slope analysis b-b' static no cut.in
Output Filename: q:\All Projects\Active Projects\08 Active Project\17637-2000 OCWD Satiago Pits Intertie\Slope Stability\slope analysis b-b' static no cut.OUT
Unit System: English
Plotted Output Filename: q:\All Projects\Active Projects\08 Active Project\17637-2000 OCWD Satiago Pits Intertie\Slope Stability\slope analysis b-b' static no cut.PLT
PROBLEM DESCRIPTION: Cross Section B-B'
Static Case (No Cut)

BOUNDARY COORDINATES
4 Top Boundaries
4 Total Boundaries

Boundary No.	X-Left (ft)	Y-Left (ft)	X-Right (ft)	Y-Right (ft)	Soil Type Below Bnd
1	0.00	240.00	200.00	240.00	1
2	200.00	240.00	210.00	245.00	1
3	210.00	245.00	435.00	330.00	1
4	435.00	330.00	600.00	330.00	1

User Specified Y-Origin = 100.00(ft)
Default X-Plus Value = 0.00(ft)
Default Y-Plus Value = 0.00(ft)

ISOTROPIC SOIL PARAMETERS
1 Type(s) of Soil

Soil Type No.	Total Unit Wt. (pcf)	Saturated Unit Wt. (pcf)	Cohesion Intercept (psf)	Friction Angle (deg)	Pore Pressure Param.	Pressure Constant (psf)	Piez. Surface No.
1	120.0	120.0	100.0	37.5	0.00	0.0	1

1 PIEZOMETRIC SURFACE(S) SPECIFIED
Unit Weight of Water = 62.40 (pcf)
Piezometric Surface No. 1 Specified by 3 Coordinate Points
Pore Pressure Inclination Factor = 0.50

Point No.	X-Water (ft)	Y-Water (ft)
1	0.00	240.00
2	200.00	240.00
3	600.00	305.00

EARTHQUAKE DATA HAS BEEN SUPPRESSED
A Critical Failure Surface Searching Method, Using A Random
Technique For Generating Circular Surfaces, Has Been Specified.
250 Trial Surfaces Have Been Generated.
5 Surface(s) Initiate(s) From Each Of 50 Points Equally Spaced
Along The Ground Surface Between X = 100.00(ft)
and X = 200.00(ft)
Each Surface Terminates Between X = 275.00(ft)
and X = 500.00(ft)
Unless Further Limitations Were Imposed, The Minimum Elevation
At Which A Surface Extends Is Y = 0.00(ft)
5.00(ft) Line Segments Define Each Trial Failure Surface.
Following Are Displayed The Ten Most Critical Of The Trial
Failure Surfaces Evaluated. They Are
Ordered - Most Critical First.
* * Safety Factors Are Calculated By The Modified Bishop Method * *

Total Number of Trial Surfaces Attempted = 250
Number of Trial Surfaces With Valid FS = 250
Statistical Data On All Valid FS Values:
FS Max = 3.630 FS Min = 2.036 FS Ave = 2.377
Standard Deviation = 0.260 Coefficient of Variation = 10.95 %
Failure Surface Specified By 70 Coordinate Points

Point No.	X-Surf (ft)	Y-Surf (ft)
1	148.980	240.000
2	153.647	238.207
3	158.348	236.504
4	163.081	234.893
5	167.845	233.373
6	172.636	231.945
7	177.455	230.610
8	182.298	229.368
9	187.165	228.219
10	192.052	227.165
11	196.959	226.206
12	201.884	225.341
13	206.824	224.571
14	211.779	223.897
15	216.745	223.319
16	221.722	222.837
17	226.707	222.450
18	231.698	222.160
19	236.695	221.966
20	241.694	221.869
21	246.694	221.868
22	251.693	221.964
23	256.689	222.156
24	261.681	222.445
25	266.666	222.829
26	271.643	223.310
27	276.609	223.887
28	281.564	224.559
29	286.505	225.327
30	291.430	226.191
31	296.337	227.149
32	301.225	228.201
33	306.092	229.348
34	310.935	230.588
35	315.754	231.922
36	320.546	233.348
37	325.310	234.867
38	330.044	236.477
39	334.746	238.178
40	339.414	239.969
41	344.046	241.851
42	348.642	243.821
43	353.198	245.880
44	357.714	248.026
45	362.188	250.259
46	366.618	252.578
47	371.002	254.981
48	375.339	257.469
49	379.627	260.041
50	383.865	262.694
51	388.051	265.429
52	392.183	268.244
53	396.260	271.138
54	400.281	274.111
55	404.243	277.160
56	408.146	280.285
57	411.988	283.485
58	415.768	286.759
59	419.483	290.104

60	423.134	293.521
61	426.717	297.008
62	430.233	300.563
63	433.680	304.186
64	437.056	307.874
65	440.360	311.626
66	443.591	315.442
67	446.748	319.319
68	449.829	323.257
69	452.834	327.253
70	454.818	330.000

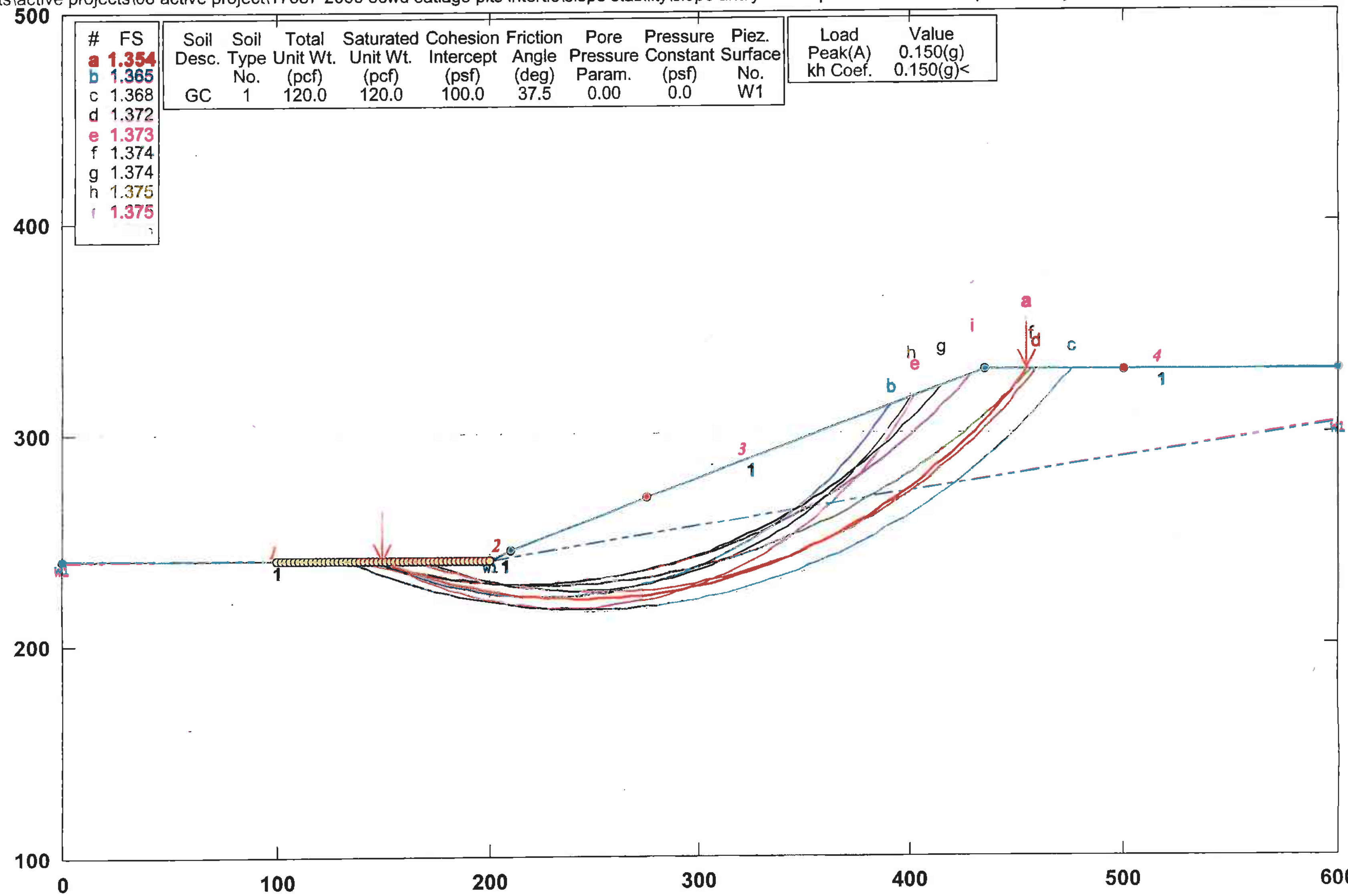
Circle Center At X = 244.234 ; Y = 480.957 ; and Radius = 259.102

Factor of Safety

*** 2.036 ***

Cross Section B-B' PseudoStatic Case (No Cut)

q:\all projects\active projects\08 active project\17637-2000 ocwd satiago pits intertie\slope stability\slope analysis b-b' pseudostatic no cut.pl2 Run By: Sean. M, Willdan Geotechnical 3/3/2009 11:32



GSTABL7 v.2 FSmin=1.354

Safety Factors Are Calculated By The Modified Bishop Method

GSTABL7

*** **GSTABL7** ***
** GSTABL7 by Garry H. Gregory, P.E. **
** Original Version 1.0, January 1996; Current Version 2.005, Sept. 2006 **

****
SLOPE STABILITY ANALYSIS SYSTEM
Modified Bishop, Simplified Janbu, or GLE Method of Slices.
(Includes Spencer & Morgenstern-Price Type Analysis)
Including Pier/Pile, Reinforcement, Soil Nail, Tieback,
Nonlinear Undrained Shear Strength, Curved Phi Envelope,
Anisotropic Soil, Fiber-Reinforced Soil, Boundary Loads, Water
Surfaces, Pseudo-Static & Newmark Earthquake, and Applied Forces.
****

Analysis Run Date: 3/3/2009
Time of Run: 11:32AM
Run By: Sean. M, Willdan Geotechnical
Input Data Filename: q:\All Projects\Active Projects\08 Active Project\17637-2000 OCWD Satiago Pits Intertie\Slope Stability\slope analysis b-b' pseudostatic no cut.in
Output Filename: q:\All Projects\Active Projects\08 Active Project\17637-2000 OCWD Satiago Pits Intertie\Slope Stability\slope analysis b-b' pseudostatic no cut.OUT
Unit System: English
Plotted Output Filename: q:\All Projects\Active Projects\08 Active Project\17637-2000 OCWD Satiago Pits Intertie\Slope Stability\slope analysis b-b' pseudostatic no cut.PLT
PROBLEM DESCRIPTION: Cross Section B-B'
PseudoStatic Case (No Cut)

BOUNDARY COORDINATES
4 Top Boundaries
4 Total Boundaries

Boundary No.	X-Left (ft)	Y-Left (ft)	X-Right (ft)	Y-Right (ft)	Soil Type Below Bnd
1	0.00	240.00	200.00	240.00	1
2	200.00	240.00	210.00	245.00	1
3	210.00	245.00	435.00	330.00	1
4	435.00	330.00	600.00	330.00	1

User Specified Y-Origin = 100.00(ft)
Default X-Plus Value = 0.00(ft)
Default Y-Plus Value = 0.00(ft)

ISOTROPIC SOIL PARAMETERS
1 Type(s) of Soil

Soil Type No.	Total Unit Wt. (pcf)	Saturated Unit Wt. (pcf)	Cohesion Intercept (psf)	Friction Angle (deg)	Pore Pressure Param.	Pressure Constant (psf)	Piez. Surface No.
1	120.0	120.0	100.0	37.5	0.00	0.0	1

1 PIEZOMETRIC SURFACE(S) SPECIFIED
Unit Weight of Water = 62.40 (pcf)
Piezometric Surface No. 1 Specified by 3 Coordinate Points
Pore Pressure Inclination Factor = 0.50

Point No.	X-Water (ft)	Y-Water (ft)
1	0.00	240.00
2	200.00	240.00
3	600.00	305.00

Specified Peak Ground Acceleration Coefficient (A) = 0.150(g)
Specified Horizontal Earthquake Coefficient (kh) = 0.150(g)
Specified Vertical Earthquake Coefficient (kv) = 0.000(g)
Specified Seismic Pore-Pressure Factor = 0.000
A Critical Failure Surface Searching Method, Using A Random Technique For Generating Circular Surfaces, Has Been Specified.
250 Trial Surfaces Have Been Generated.
5 Surface(s) Initiate(s) From Each Of 50 Points Equally Spaced
Along The Ground Surface Between X = 100.00(ft)
and X = 200.00(ft)
Each Surface Terminates Between X = 275.00(ft)
and X = 500.00(ft)
Unless Further Limitations Were Imposed, The Minimum Elevation
At Which A Surface Extends Is Y = 0.00(ft)
5.00(ft) Line Segments Define Each Trial Failure Surface.
Following Are Displayed The Ten Most Critical Of The Trial

Failure Surfaces Evaluated. They Are
Ordered - Most Critical First.
* * Safety Factors Are Calculated By The Modified Bishop Method * *
Total Number of Trial Surfaces Attempted =   250
Number of Trial Surfaces With Valid FS = 250
Statistical Data On All Valid FS Values:
FS Max =   2.052   FS Min =   1.354   FS Ave =   1.552
Standard Deviation =   0.131   Coefficient of Variation =   8.42 %
Failure Surface Specified By 70 Coordinate Points

Point No.	X-Surf (ft)	Y-Surf (ft)
1	148.980	240.000
2	153.647	238.207
3	158.348	236.504
4	163.081	234.893
5	167.845	233.373
6	172.636	231.945
7	177.455	230.610
8	182.298	229.368
9	187.165	228.219
10	192.052	227.165
11	196.959	226.206
12	201.884	225.341
13	206.824	224.571
14	211.779	223.897
15	216.745	223.319
16	221.722	222.837
17	226.707	222.450
18	231.698	222.160
19	236.695	221.966
20	241.694	221.869
21	246.694	221.868
22	251.693	221.964
23	256.689	222.156
24	261.681	222.445
25	266.666	222.829
26	271.643	223.310
27	276.609	223.887
28	281.564	224.559
29	286.505	225.327
30	291.430	226.191
31	296.337	227.149
32	301.225	228.201
33	306.092	229.348
34	310.935	230.588
35	315.754	231.922
36	320.546	233.348
37	325.310	234.867
38	330.044	236.477
39	334.746	238.178
40	339.414	239.969
41	344.046	241.851
42	348.642	243.821
43	353.198	245.880
44	357.714	248.026
45	362.188	250.259
46	366.618	252.578
47	371.002	254.981
48	375.339	257.469
49	379.627	260.041
50	383.865	262.694
51	388.051	265.429
52	392.183	268.244
53	396.260	271.138
54	400.281	274.111
55	404.243	277.160
56	408.146	280.285

57	411.988	283.485
58	415.768	286.759
59	419.483	290.104
60	423.134	293.521
61	426.717	297.008
62	430.233	300.563
63	433.680	304.186
64	437.056	307.874
65	440.360	311.626
66	443.591	315.442
67	446.748	319.319
68	449.829	323.257
69	452.834	327.253
70	454.818	330.000

Circle Center At X = 244.234 ; Y = 480.957 ; and Radius = 259.102

Factor of Safety

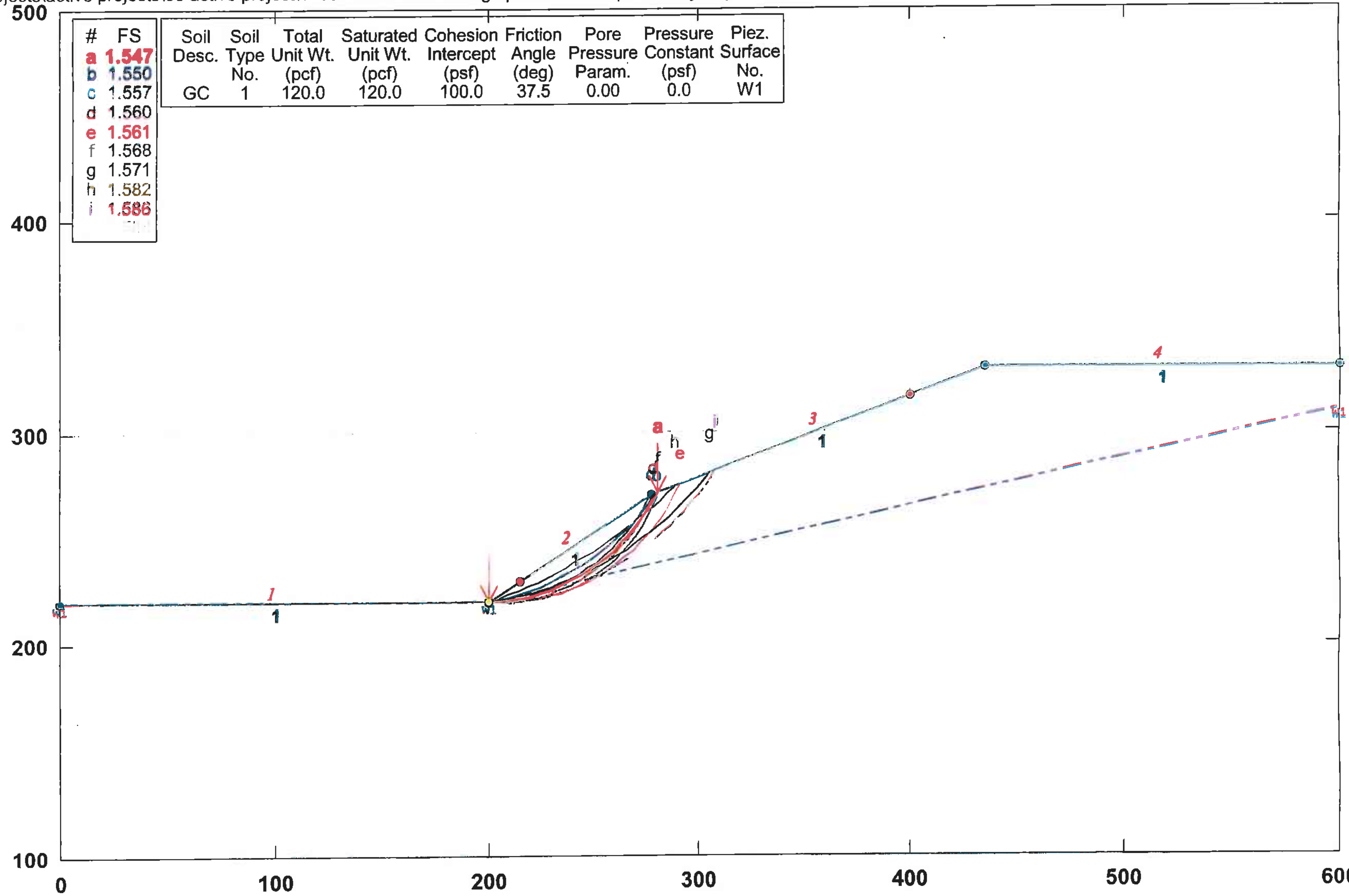
*** 1.354 ***

Cross Section B-B' Static Case (20' permanent)

q:\all projects\active projects\08 active project\17637-2000 ocwd satiago pits intertie\slope stability\slope analysis b-b' 20'perm3-3.pl2 Run By: Sean. M, Willdan Geotechnical 3/3/2009 01:41PM

#	FS
a	1.547
b	1.550
c	1.557
d	1.560
e	1.561
f	1.568
g	1.571
h	1.582
i	1.586

Soil Desc.	Soil Type No.	Total Unit Wt. (pcf)	Saturated Unit Wt. (pcf)	Cohesion Intercept (psf)	Friction Angle (deg)	Pore Pressure Param.	Pressure Constant (psf)	Piez. Surface No.
GC	1	120.0	120.0	100.0	37.5	0.00	0.0	W1



GSTABL7 v.2 FSmin=1.547

Safety Factors Are Calculated By The Modified Bishop Method

GSTABL7

```
*** GSTABL7 ***
** GSTABL7 by Garry H. Gregory, P.E. **
** Original Version 1.0, January 1996; Current Version 2.005, Sept. 2006 **
(All Rights Reserved-Unauthorized Use Prohibited)
*********************************************************************************
SLOPE STABILITY ANALYSIS SYSTEM
Modified Bishop, Simplified Janbu, or GLE Method of Slices.
(Includes Spencer & Morgenstern-Price Type Analysis)
Including Pier/Pile, Reinforcement, Soil Nail, Tieback,
Nonlinear Undrained Shear Strength, Curved Phi Envelope,
Anisotropic Soil, Fiber-Reinforced Soil, Boundary Loads, Water
Surfaces, Pseudo-Static & Newmark Earthquake, and Applied Forces.
*********************************************************************************
Analysis Run Date:        3/3/2009
Time of Run:              01:41PM
Run By:                   Sean. M, Willdan Geotechnical
Input Data Filename:      Q:\All Projects\Active Projects\08 Active Project\17637-2000
OCWD Satiago Pits Intertie\Slope Stability\slope analysis b-b' 20'perm3-3.in
Output Filename:          Q:\All Projects\Active Projects\08 Active Project\17637-2000
OCWD Satiago Pits Intertie\Slope Stability\slope analysis b-b' 20'perm3-3.OUT
Unit System:              English
Plotted Output Filename:  Q:\All Projects\Active Projects\08 Active Project\17637-2000
OCWD Satiago Pits Intertie\Slope Stability\slope analysis b-b' 20'perm3-3.PLT
PROBLEM DESCRIPTION:  Cross Section B-B'
                      Static Case (20' permanent)
BOUNDARY COORDINATES
    4 Top   Boundaries
    4 Total Boundaries
Boundary   X-Left    Y-Left    X-Right   Y-Right   Soil Type
  No.       (ft)      (ft)      (ft)      (ft)     Below Bnd
   1         0.00    220.00    200.00    220.00        1
   2       200.00    220.00    277.50    270.00        1
   3       277.50    270.00    435.00    330.00        1
   4       435.00    330.00    600.00    330.00        1
User Specified Y-Origin =      100.00(ft)
Default X-Plus Value = 0.00(ft)
Default Y-Plus Value = 0.00(ft)
ISOTROPIC SOIL PARAMETERS
 1 Type(s) of Soil
Soil  Total  Saturated  Cohesion Friction   Pore   Pressure  Piez.
Type Unit Wt. Unit Wt. Intercept  Angle  Pressure Constant Surface
 No.  (pcf)    (pcf)    (psf)     (deg)   Param.   (psf)     No.
  1   120.0    120.0    100.0     37.5    0.00      0.0      1
1 PIEZOMETRIC SURFACE(S) SPECIFIED
Unit Weight of Water =  62.40 (pcf)
Piezometric Surface No.  1 Specified by  3 Coordinate Points
Pore Pressure Inclination Factor =  0.50
  Point      X-Water     Y-Water
   No.        (ft)        (ft)
    1          0.00      220.00
    2        200.00      220.00
    3        600.00      310.00
EARTHQUAKE DATA HAS BEEN SUPPRESSED
A Critical Failure Surface Searching Method, Using A Random
Technique For Generating Circular Surfaces, Has Been Specified.
 250 Trial Surfaces Have Been Generated.
   5 Surface(s) Initiate(s) From Each Of    50 Points Equally Spaced
Along The Ground Surface Between  X = 200.00(ft)
                             and  X = 200.10(ft)
Each Surface Terminates Between   X = 215.00(ft)
                             and  X = 400.00(ft)
Unless Further Limitations Were Imposed, The Minimum Elevation
At Which A Surface Extends Is  Y =      0.00(ft)
 5.00(ft) Line Segments Define Each Trial Failure Surface.
Following Are Displayed The Ten Most Critical Of The Trial
      Failure Surfaces Evaluated. They Are
      Ordered - Most Critical First.
      * * Safety Factors Are Calculated By The Modified Bishop Method * *
```

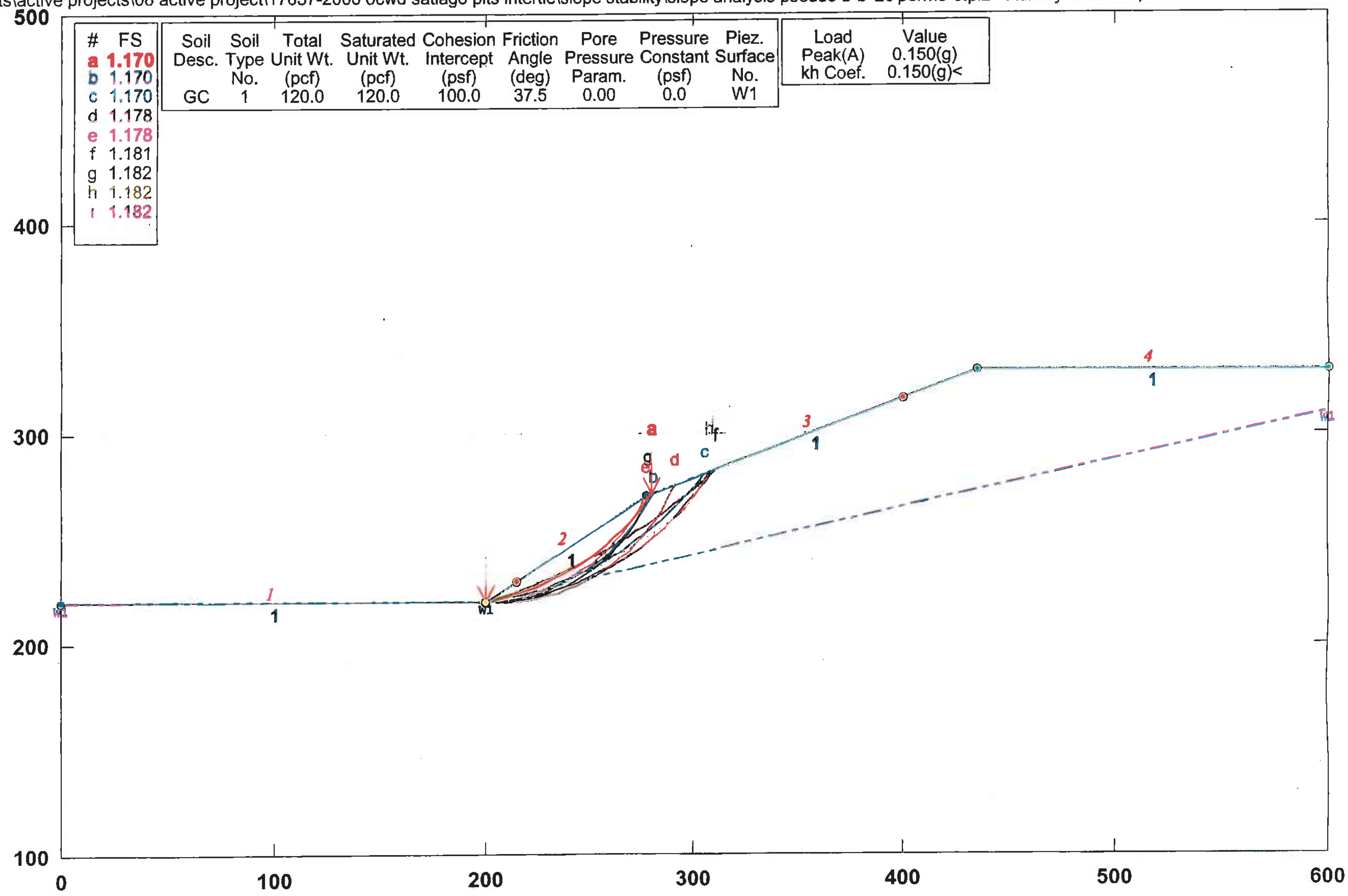
Total Number of Trial Surfaces Attempted = 250
Number of Trial Surfaces With Valid FS = 250
Statistical Data On All Valid FS Values:
FS Max = 5.382 FS Min = 1.547 FS Ave = 1.998
Standard Deviation = 0.423 Coefficient of Variation = 21.15 %
Failure Surface Specified By 21 Coordinate Points

Point No.	X-Surf (ft)	Y-Surf (ft)
1	200.004	220.003
2	204.998	220.240
3	209.972	220.751
4	214.911	221.533
5	219.799	222.583
6	224.623	223.900
7	229.367	225.478
8	234.018	227.314
9	238.562	229.401
10	242.984	231.733
11	247.273	234.304
12	251.414	237.106
13	255.396	240.129
14	259.207	243.366
15	262.835	246.807
16	266.269	250.441
17	269.500	254.257
18	272.517	258.245
19	275.311	262.391
20	277.874	266.684
21	280.145	271.008

Circle Center At X = 198.157 ; Y = 311.373 ; and Radius = 91.389
Factor of Safety
*** 1.547 ***

Cross Section B-B' Pseudo Static Case (20' permanent)

q:\all projects\active projects\08 active project\17637-2000 ocwd satiago pits intertie\slope stability\slope analysis pseudo b-b' 20'perm3-3.pl2 Run By: Sean. M, Willdan Geotechnical 3/3/2009 01:4



GSTABL7 v.2 FSmin=1.170
Safety Factors Are Calculated By The Modified Bishop Method

GSTABL7

```
                          ***  GSTABL7  ***
               ** GSTABL7 by Garry H. Gregory, P.E. **
  ** Original Version 1.0, January 1996; Current Version 2.005, Sept. 2006 **
            (All Rights Reserved-Unauthorized Use Prohibited)
*********************************************************************************
                  SLOPE STABILITY ANALYSIS SYSTEM
      Modified Bishop, Simplified Janbu, or GLE Method of Slices.
      (Includes Spencer & Morgenstern-Price Type Analysis)
      Including Pier/Pile, Reinforcement, Soil Nail, Tieback,
      Nonlinear Undrained Shear Strength, Curved Phi Envelope,
      Anisotropic Soil, Fiber-Reinforced Soil, Boundary Loads, Water
      Surfaces, Pseudo-Static & Newmark Earthquake, and Applied Forces.
*********************************************************************************
 Analysis Run Date:        3/3/2009
 Time of Run:              01:47PM
 Run By:                   Sean. M, Willdan Geotechnical
 Input Data Filename:      Q:\All Projects\Active Projects\08 Active Project\17637-2000
OCWD Satiago Pits Intertie\Slope Stability\slope analysis pseudo b-b' 20'perm3-3.in
 Output Filename:          Q:\All Projects\Active Projects\08 Active Project\17637-2000
OCWD Satiago Pits Intertie\Slope Stability\slope analysis pseudo b-b' 20'perm3-3.OUT
 Unit System:              English
 Plotted Output Filename:  Q:\All Projects\Active Projects\08 Active Project\17637-2000
OCWD Satiago Pits Intertie\Slope Stability\slope analysis pseudo b-b' 20'perm3-3.PLT
 PROBLEM DESCRIPTION:  Cross Section B-B'
                        Pseudo Static Case (20' permanent)
 BOUNDARY COORDINATES
     4 Top   Boundaries
     4 Total Boundaries
 Boundary     X-Left     Y-Left    X-Right    Y-Right    Soil Type
    No.        (ft)       (ft)       (ft)       (ft)     Below Bnd
     1          0.00     220.00     200.00     220.00        1
     2        200.00     220.00     277.50     270.00        1
     3        277.50     270.00     435.00     330.00        1
     4        435.00     330.00     600.00     330.00        1
 User Specified Y-Origin =       100.00(ft)
 Default X-Plus Value = 0.00(ft)
 Default Y-Plus Value = 0.00(ft)
 ISOTROPIC SOIL PARAMETERS
  1 Type(s) of Soil
 Soil  Total  Saturated  Cohesion Friction   Pore   Pressure   Piez.
 Type Unit Wt. Unit Wt.  Intercept  Angle  Pressure Constant Surface
  No.  (pcf)    (pcf)     (psf)    (deg)    Param.   (psf)     No.
   1   120.0    120.0     100.0     37.5    0.00      0.0       1
 1 PIEZOMETRIC SURFACE(S) SPECIFIED
 Unit Weight of Water =  62.40 (pcf)
 Piezometric Surface No.  1 Specified by  3 Coordinate Points
 Pore Pressure Inclination Factor =  0.50
   Point      X-Water     Y-Water
    No.        (ft)        (ft)
     1          0.00      220.00
     2        200.00      220.00
     3        600.00      310.00
 Specified Peak Ground Acceleration Coefficient (A) =   0.150(g)
 Specified Horizontal Earthquake Coefficient (kh) =   0.150(g)
 Specified Vertical Earthquake Coefficient (kv) =   0.000(g)
 Specified Seismic Pore-Pressure Factor =   0.000
 A Critical Failure Surface Searching Method, Using A Random
 Technique For Generating Circular Surfaces, Has Been Specified.
  250 Trial Surfaces Have Been Generated.
    5 Surface(s) Initiate(s) From Each Of    50 Points Equally Spaced
 Along The Ground Surface Between  X = 200.00(ft)
                              and  X = 200.10(ft)
 Each Surface Terminates Between   X = 215.00(ft)
                              and  X = 400.00(ft)
 Unless Further Limitations Were Imposed, The Minimum Elevation
 At Which A Surface Extends Is  Y =      0.00(ft)
  5.00(ft) Line Segments Define Each Trial Failure Surface.
 Following Are Displayed The Ten Most Critical Of The Trial
```

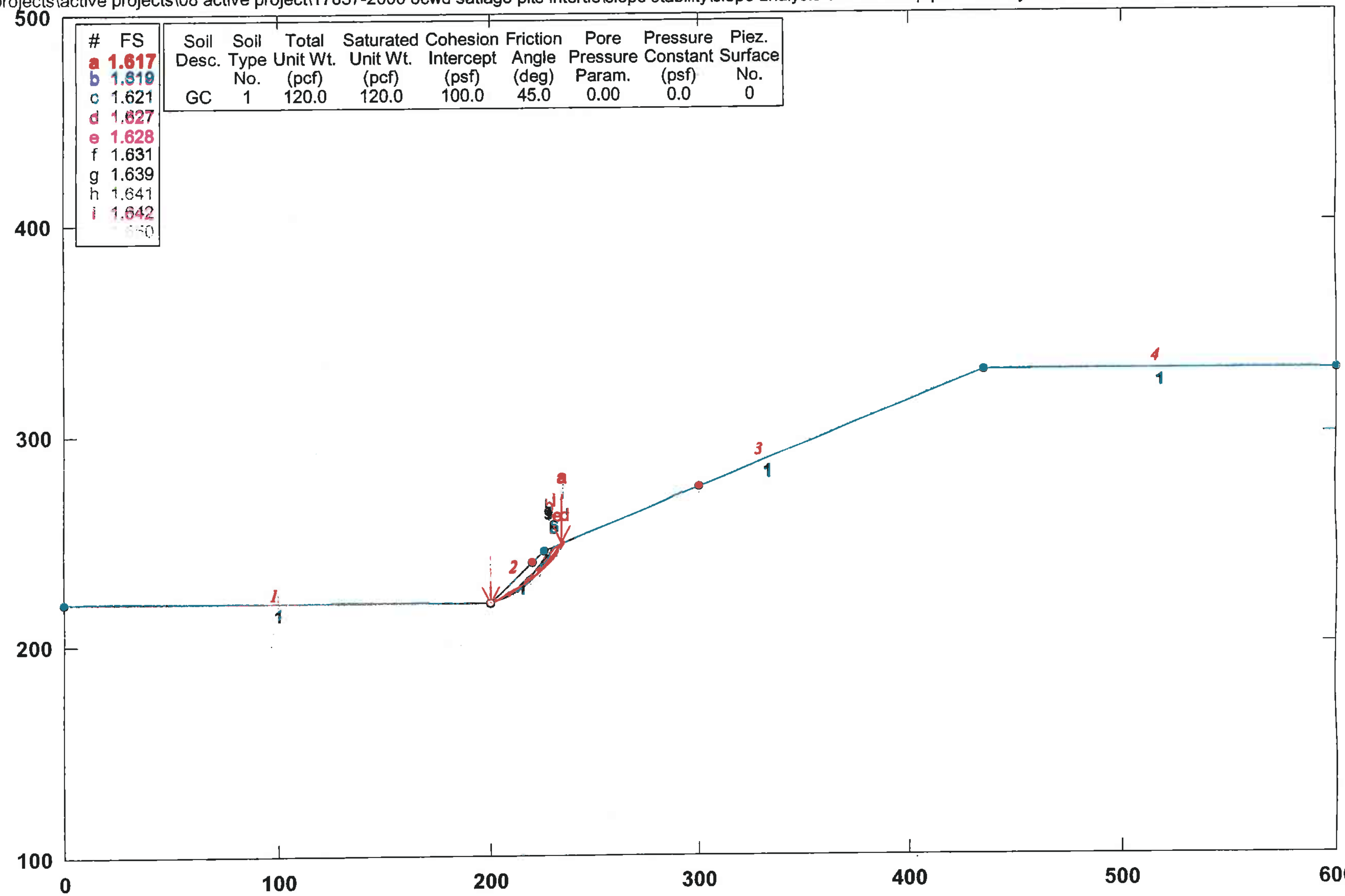
Failure Surfaces Evaluated. They Are
Ordered - Most Critical First.
* * Safety Factors Are Calculated By The Modified Bishop Method * *
Total Number of Trial Surfaces Attempted = 250
Number of Trial Surfaces With Valid FS = 250
Statistical Data On All Valid FS Values:
FS Max = 4.273 FS Min = 1.170 FS Ave = 1.502
Standard Deviation = 0.344 Coefficient of Variation = 22.91 %
Failure Surface Specified By 21 Coordinate Points

Point No.	X-Surf (ft)	Y-Surf (ft)
1	200.067	220.044
2	204.972	221.014
3	209.834	222.183
4	214.644	223.547
5	219.395	225.104
6	224.080	226.853
7	228.689	228.790
8	233.217	230.912
9	237.655	233.215
10	241.996	235.696
11	246.233	238.351
12	250.359	241.175
13	254.367	244.164
14	258.251	247.312
15	262.005	250.615
16	265.622	254.067
17	269.096	257.663
18	272.422	261.397
19	275.594	265.262
20	278.607	269.252
21	279.709	270.842

Circle Center At X = 178.546 ; Y = 341.674 ; and Radius = 123.520
Factor of Safety
*** 1.170 ***

Cross Section B-B' Static Case (25' temporary)

q:\all projects\active projects\08 active project\17637-2000 ocwd satiago pits intertie\slope stability\slope analysis b-b' 25' temp.pl2 Run By: Sean. M, Willdan Geotechnical 2/6/2009 03:32PM



GSTABL7 v.2 FSmin=1.617

Safety Factors Are Calculated By The Modified Bishop Method

*** **GSTABL7** ***

** GSTABL7 by Garry H. Gregory, P.E. **

** Original Version 1.0, January 1996; Current Version 2.005, Sept. 2006 **

****

SLOPE STABILITY ANALYSIS SYSTEM

Modified Bishop, Simplified Janbu, or GLE Method of Slices.
(Includes Spencer & Morgenstern-Price Type Analysis)
Including Pier/Pile, Reinforcement, Soil Nail, Tieback,
Nonlinear Undrained Shear Strength, Curved Phi Envelope,
Anisotropic Soil, Fiber-Reinforced Soil, Boundary Loads, Water
Surfaces, Pseudo-Static & Newmark Earthquake, and Applied Forces.

****

Analysis Run Date: 3/3/2009
Time of Run: 11:41AM
Run By: Sean. M, Willdan Geotechnical
Input Data Filename: q:\All Projects\Active Projects\08 Active Project\17637-2000 OCWD Satiago Pits Intertie\Slope Stability\slope analysis b-b' 25' temp.in
Output Filename: q:\All Projects\Active Projects\08 Active Project\17637-2000 OCWD Satiago Pits Intertie\Slope Stability\slope analysis b-b' 25' temp.OUT
Unit System: English
Plotted Output Filename: q:\All Projects\Active Projects\08 Active Project\17637-2000 OCWD Satiago Pits Intertie\Slope Stability\slope analysis b-b' 25' temp.PLT
PROBLEM DESCRIPTION: Cross Section B-B'
Static Case (25' temporary)

BOUNDARY COORDINATES
4 Top Boundaries
4 Total Boundaries

Boundary No.	X-Left (ft)	Y-Left (ft)	X-Right (ft)	Y-Right (ft)	Soil Type Below Bnd
1	0.00	220.00	200.00	220.00	1
2	200.00	220.00	226.00	245.00	1
3	226.00	245.00	435.00	330.00	1
4	435.00	330.00	600.00	330.00	1

User Specified Y-Origin = 100.00(ft)
Default X-Plus Value = 0.00(ft)
Default Y-Plus Value = 0.00(ft)

ISOTROPIC SOIL PARAMETERS
1 Type(s) of Soil

Soil Type No.	Total Unit Wt. (pcf)	Saturated Unit Wt. (pcf)	Cohesion Intercept (psf)	Friction Angle (deg)	Pore Pressure Param.	Pressure Constant (psf)	Piez. Surface No.
1	120.0	120.0	100.0	45.0	0.00	0.0	0

EARTHQUAKE DATA HAS BEEN SUPPRESSED
A Critical Failure Surface Searching Method, Using A Random Technique For Generating Circular Surfaces, Has Been Specified.
250 Trial Surfaces Have Been Generated.
5 Surface(s) Initiate(s) From Each Of 50 Points Equally Spaced
Along The Ground Surface Between X = 200.00(ft)
and X = 200.10(ft)
Each Surface Terminates Between X = 220.00(ft)
and X = 300.00(ft)
Unless Further Limitations Were Imposed, The Minimum Elevation
At Which A Surface Extends Is Y = 0.00(ft)
5.00(ft) Line Segments Define Each Trial Failure Surface.
Following Are Displayed The Ten Most Critical Of The Trial
Failure Surfaces Evaluated. They Are
Ordered - Most Critical First.
* * Safety Factors Are Calculated By The Modified Bishop Method * *
Total Number of Trial Surfaces Attempted = 250
Number of Trial Surfaces With Valid FS = 250
Statistical Data On All Valid FS Values:
FS Max = 3.700 FS Min = 1.617 FS Ave = 2.459
Standard Deviation = 0.606 Coefficient of Variation = 24.63 %
Failure Surface Specified By 10 Coordinate Points

Point No.	X-Surf (ft)	Y-Surf (ft)

1	200.014	220.014
2	204.596	222.015
3	209.025	224.336
4	213.278	226.965
5	217.334	229.889
6	221.172	233.093
7	224.773	236.562
8	228.120	240.277
9	231.194	244.220
10	233.866	248.199

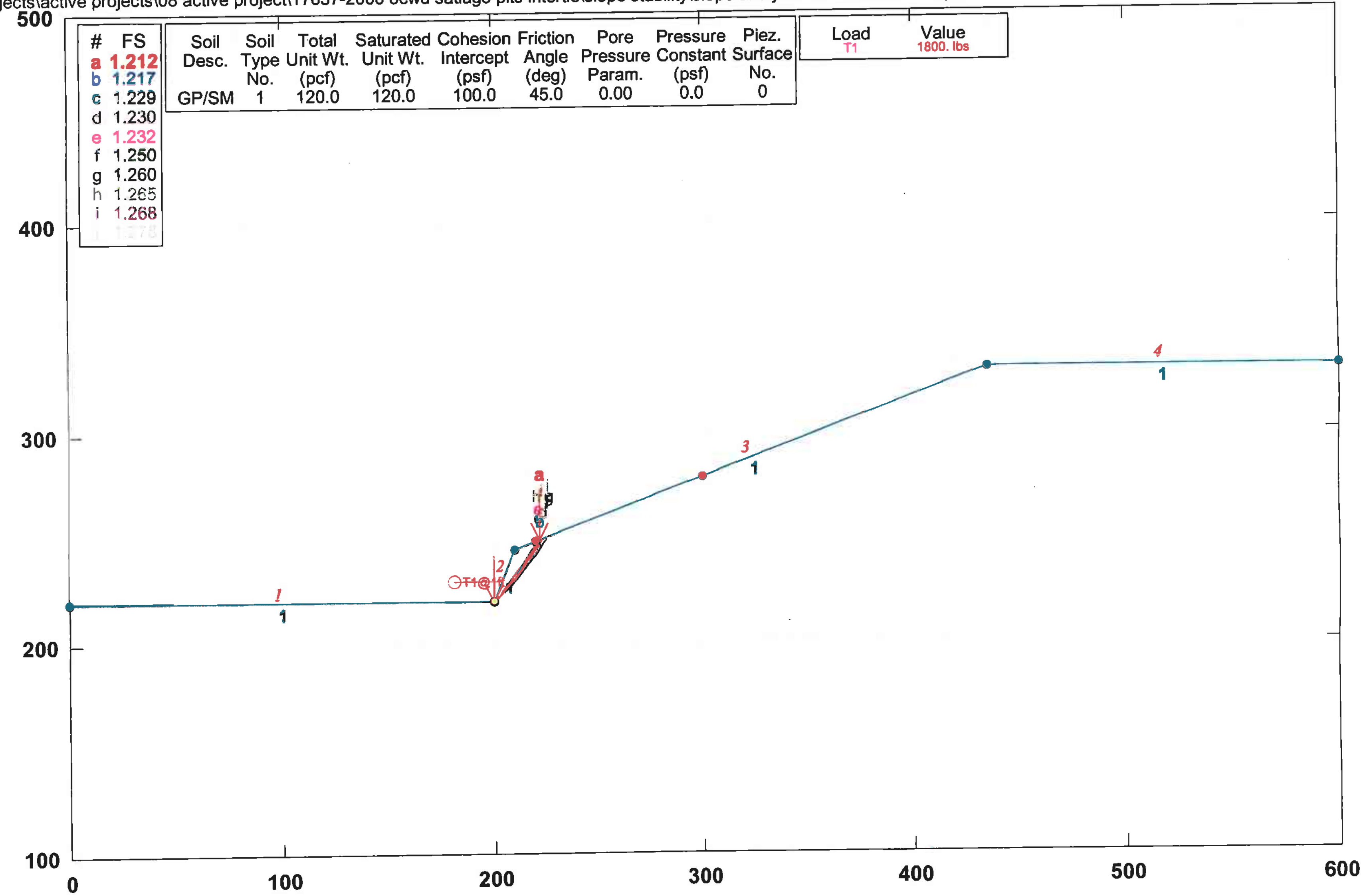
Circle Center At X = 174.152 ; Y = 285.496 ; and Radius = 70.405

Factor of Safety

*** 1.617 ***

Cross Section B-B' Static Case (25 feet Vertical)

q:\all projects\active projects\08 active project\17637-2000 ocwd satiago pits intertie\slope stability\slope analysis b-b' static 25 feet.pl2 Run By: Sean. M, Willdan Geotechnical 5/12/2009 09:12AM



GSTABL7 v.2 FSmin=1.212

Safety Factors Are Calculated By The Modified Bishop Method

GSTABL7

*** **GSTABL7** ***

** GSTABL7 by Garry H. Gregory, P.E. **

** Original Version 1.0, January 1996; Current Version 2.005, Sept. 2006 **

****

SLOPE STABILITY ANALYSIS SYSTEM

Modified Bishop, Simplified Janbu, or GLE Method of Slices.
(Includes Spencer & Morgenstern-Price Type Analysis)
Including Pier/Pile, Reinforcement, Soil Nail, Tieback,
Nonlinear Undrained Shear Strength, Curved Phi Envelope,
Anisotropic Soil, Fiber-Reinforced Soil, Boundary Loads, Water
Surfaces, Pseudo-Static & Newmark Earthquake, and Applied Forces.

****

Analysis Run Date: 5/12/2009
Time of Run: 09:01AM
Run By: Sean. M, Willdan Geotechnical
Input Data Filename: Q:\All Projects\Active Projects\08 Active Project\17637-2000 OCWD Satiago Pits Intertie\Slope Stability\slope analysis b-b' static 25 feet.in
Output Filename: Q:\All Projects\Active Projects\08 Active Project\17637-2000 OCWD Satiago Pits Intertie\Slope Stability\slope analysis b-b' static 25 feet.OUT
Unit System: English
Plotted Output Filename: Q:\All Projects\Active Projects\08 Active Project\17637-2000 OCWD Satiago Pits Intertie\Slope Stability\slope analysis b-b' static 25 feet.PLT
PROBLEM DESCRIPTION: Cross Section B-B'
Static Case (25 feet trench height)

BOUNDARY COORDINATES
4 Top Boundaries
4 Total Boundaries

Boundary No.	X-Left (ft)	Y-Left (ft)	X-Right (ft)	Y-Right (ft)	Soil Type Below Bnd
1	0.00	220.00	200.00	220.00	1
2	200.00	220.00	210.00	245.00	1
3	210.00	245.00	435.00	330.00	1
4	435.00	330.00	600.00	330.00	1

User Specified Y-Origin = 100.00(ft)
Default X-Plus Value = 0.00(ft)
Default Y-Plus Value = 0.00(ft)

ISOTROPIC SOIL PARAMETERS
1 Type(s) of Soil

Soil Type No.	Total Unit Wt. (pcf)	Saturated Unit Wt. (pcf)	Cohesion Intercept (psf)	Friction Angle (deg)	Pore Pressure Param.	Pressure Constant (psf)	Piez. Surface No.
1	120.0	120.0	100.0	45.0	0.00	0.0	0

EARTHQUAKE DATA HAS BEEN SUPPRESSED

TIEBACK LOAD(S)
1 Tieback Load(s) Specified

Tieback No.	X-Pos (ft)	Y-Pos (ft)	Load (lbs)	Spacing (ft)	Inclination (deg)	Length (ft)	Force Method
1	204.00	230.00	1800.0	1.0	0.00	0.0	2

NOTE - An Equivalent Line Load Is Calculated For Each Row Of Tiebacks Assuming A Uniform Distribution Of Load Horizontally Between Individual Tiebacks. Force Method 1 Considers Only Tangential Tieback Forces. Force Method 2 Considers Both Tangential and Normal Tieback Forces. Force Method 3 Considers Only Normal Tieback Forces. Force Method 4 Limits Normal and Tangential Tieback-Force Distribution to 1.5 Times the Tieback Inclination, or to 30 Degrees Below (Left of) the Tieback-Failure Surface Intersection, Whichever is Greater.

A Critical Failure Surface Searching Method, Using A Random Technique For Generating Circular Surfaces, Has Been Specified.
250 Trial Surfaces Have Been Generated.
5 Surface(s) Initiate(s) From Each Of 50 Points Equally Spaced
Along The Ground Surface Between X = 200.00(ft)
and X = 200.10(ft)
Each Surface Terminates Between X = 220.00(ft)
and X = 300.00(ft)
Unless Further Limitations Were Imposed, The Minimum Elevation
At Which A Surface Extends Is Y = 0.00(ft)

5.00(ft) Line Segments Define Each Trial Failure Surface.
Following Are Displayed The Ten Most Critical Of The Trial
Failure Surfaces Evaluated. They Are
Ordered - Most Critical First.
* * Safety Factors Are Calculated By The Modified Bishop Method * *
Total Number of Trial Surfaces Attempted = 250
Number of Trial Surfaces With Valid FS = 250
Statistical Data On All Valid FS Values:
FS Max = 3.882 FS Min = 1.212 FS Ave = 2.333
Standard Deviation = 0.806 Coefficient of Variation = 34.53 %
Failure Surface Specified By 9 Coordinate Points

Point No.	X-Surf (ft)	Y-Surf (ft)
1	200.082	220.204
2	203.772	223.579
3	207.248	227.172
4	210.499	230.971
5	213.512	234.961
6	216.275	239.128
7	218.779	243.457
8	221.013	247.929
9	221.637	249.396

Circle Center At X = 146.741 ; Y = 282.238 ; and Radius = 81.813
Factor of Safety
*** 1.212 ***
Individual data on the 9 slices

Slice No.	Width (ft)	Weight (lbs)	Water Force Top (lbs)	Water Force Bot (lbs)	Tie Force Norm (lbs)	Tie Force Tan (lbs)	Earthquake Force Hor (lbs)	Earthquake Force Ver (lbs)	Surcharge Load (lbs)
1	3.7	1295.2	0.0	0.0	0.	0.	0.0	0.0	0.0
2	3.5	3504.4	0.0	0.0	237.	0.	0.0	0.0	0.0
3	2.8	4220.1	0.0	0.0	647.	312.	0.0	0.0	0.0
4	0.5	863.5	0.0	0.0	70.	74.	0.0	0.0	0.0
5	3.0	4624.4	0.0	0.0	249.	388.	0.0	0.0	0.0
6	2.8	3250.8	0.0	0.0	109.	249.	0.0	0.0	0.0
7	2.5	1968.1	0.0	0.0	63.	172.	0.0	0.0	0.0
8	2.2	816.6	0.0	0.0	43.	127.	0.0	0.0	0.0
9	0.6	46.1	0.0	0.0	12.	34.	0.0	0.0	0.0

Failure Surface Specified By 9 Coordinate Points

Point No.	X-Surf (ft)	Y-Surf (ft)
1	200.088	220.220
2	203.851	223.513
3	207.392	227.042
4	210.697	230.794
5	213.752	234.753
6	216.543	238.901
7	219.060	243.221
8	221.292	247.695
9	222.077	249.562

Circle Center At X = 151.233 ; Y = 279.844 ; and Radius = 77.084
Factor of Safety
*** 1.217 ***
Failure Surface Specified By 9 Coordinate Points

Point No.	X-Surf (ft)	Y-Surf (ft)
1	200.051	220.128
2	203.415	223.827
3	206.649	227.641
4	209.749	231.563
5	212.712	235.591
6	215.535	239.718
7	218.213	243.940
8	220.743	248.252
9	221.292	249.266

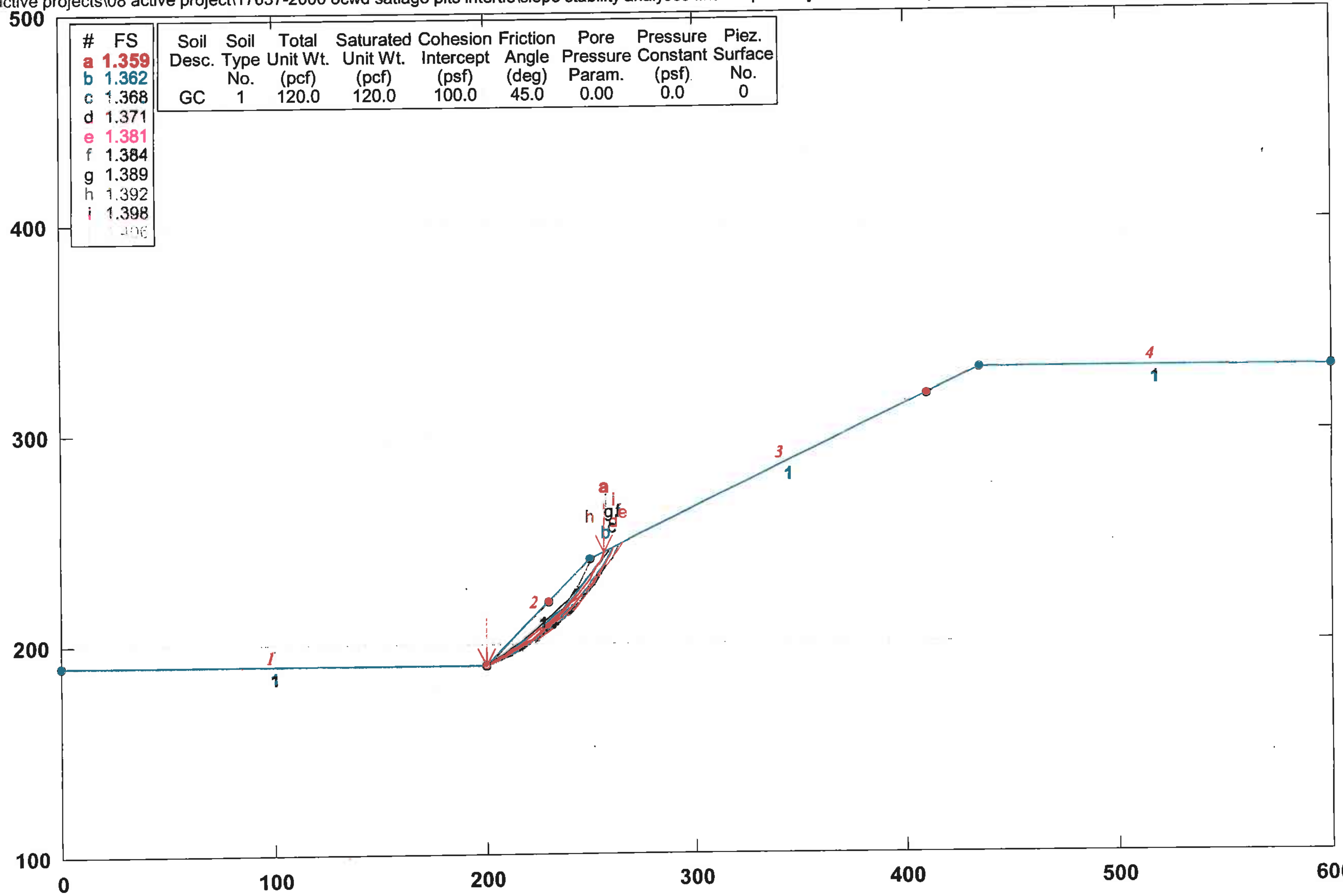
Circle Center At X = 94.604 ; Y = 319.384 ; and Radius = 144.813

```
        Factor of Safety
       ***    1.229    ***
Failure Surface Specified By  9 Coordinate Points
  Point      X-Surf       Y-Surf
   No.        (ft)         (ft)
    1        200.041      220.102
    2        203.594      223.620
    3        207.009      227.272
    4        210.283      231.052
    5        213.409      234.954
    6        216.384      238.972
    7        219.203      243.102
    8        221.862      247.336
    9        223.440      250.077
Circle Center At X =   109.881 ; Y =   314.699 ; and Radius =   130.681
        Factor of Safety
       ***    1.230    ***
Failure Surface Specified By  9 Coordinate Points
  Point      X-Surf       Y-Surf
   No.        (ft)         (ft)
    1        200.002      220.005
    2        203.282      223.779
    3        206.449      227.648
    4        209.499      231.610
    5        212.430      235.661
    6        215.239      239.798
    7        217.923      244.016
    8        220.481      248.312
    9        220.937      249.132
Circle Center At X =    74.835 ; Y =   332.120 ; and Radius =   168.037
        Factor of Safety
       ***    1.232    ***
Failure Surface Specified By 10 Coordinate Points
  Point      X-Surf       Y-Surf
   No.        (ft)         (ft)
    1        200.020      220.051
    2        203.623      223.518
    3        207.112      227.100
    4        210.482      230.794
    5        213.730      234.595
    6        216.853      238.500
    7        219.847      242.504
    8        222.709      246.604
    9        225.436      250.795
   10        225.465      250.842
Circle Center At X =    95.319 ; Y =   332.480 ; and Radius =   153.631
        Factor of Safety
       ***    1.250    ***
Failure Surface Specified By 10 Coordinate Points
  Point      X-Surf       Y-Surf
   No.        (ft)         (ft)
    1        200.012      220.031
    2        203.640      223.471
    3        207.162      227.020
    4        210.576      230.674
    5        213.876      234.429
    6        217.062      238.283
    7        220.130      242.232
    8        223.077      246.271
    9        225.900      250.398
   10        226.414      251.201
Circle Center At X =    88.725 ; Y =   341.060 ; and Radius =   164.417
        Factor of Safety
       ***    1.260    ***
Failure Surface Specified By  9 Coordinate Points
  Point      X-Surf       Y-Surf
   No.        (ft)         (ft)
```

```
     1         200.051      220.128
     2         203.192      224.018
     3         206.260      227.967
     4         209.251      231.973
     5         212.166      236.035
     6         215.004      240.152
     7         217.763      244.322
     8         220.442      248.543
     9         220.759      249.065
Circle Center At X =    -3.489 ; Y =   387.712 ; and Radius =   263.653
       Factor of Safety
      ***    1.265    ***
Failure Surface Specified By  9 Coordinate Points
  Point       X-Surf       Y-Surf
   No.         (ft)         (ft)
     1         200.039      220.097
     2         203.236      223.941
     3         206.387      227.824
     4         209.489      231.745
     5         212.542      235.704
     6         215.547      239.701
     7         218.503      243.734
     8         221.408      247.803
     9         222.830      249.847
Circle Center At X =  -110.893 ; Y =   482.011 ; and Radius =   406.544
       Factor of Safety
      ***    1.268    ***
Failure Surface Specified By  9 Coordinate Points
  Point       X-Surf       Y-Surf
   No.         (ft)         (ft)
     1         200.020      220.051
     2         203.318      223.810
     3         206.582      227.597
     4         209.813      231.413
     5         213.010      235.257
     6         216.174      239.129
     7         219.303      243.029
     8         222.398      246.956
     9         225.381      250.811
Circle Center At X =  -226.360 ; Y =   597.439 ; and Radius =   569.405
       Factor of Safety
      ***    1.278    ***
          **** END OF GSTABL7 OUTPUT ****
```

Cross Section B-B' Static Case (50' temporary)

q:\all projects\active projects\08 active project\17637-2000 ocwd satiago pits intertie\slope stability analyses final\slope analysis b-b' 50' temporary1.pl2 Run By: Sean. M, Willdan Geotechnical 5/12/2009 12:



GSTABL7 v.2 FSmin=1.359

Safety Factors Are Calculated By The Modified Bishop Method

GSTABL7

*** **GSTABL7** ***
** GSTABL7 by Garry H. Gregory, P.E. **
** Original Version 1.0, January 1996; Current Version 2.005, Sept. 2006 **

****
SLOPE STABILITY ANALYSIS SYSTEM
Modified Bishop, Simplified Janbu, or GLE Method of Slices.
(Includes Spencer & Morgenstern-Price Type Analysis)
Including Pier/Pile, Reinforcement, Soil Nail, Tieback,
Nonlinear Undrained Shear Strength, Curved Phi Envelope,
Anisotropic Soil, Fiber-Reinforced Soil, Boundary Loads, Water
Surfaces, Pseudo-Static & Newmark Earthquake, and Applied Forces.
****

Analysis Run Date: 5/12/2009
Time of Run: 12:41PM
Run By: Sean. M, Willdan Geotechnical
Input Data Filename: Q:\All Projects\Active Projects\08 Active Project\17637-2000 OCWD Satiago Pits Intertie\Slope Stability Analyses final\slope analysis b-b' 50' temporary1.ir
Output Filename: Q:\All Projects\Active Projects\08 Active Project\17637-2000 OCWD Satiago Pits Intertie\Slope Stability Analyses final\slope analysis b-b' 50' temporary1.OU
Unit System: English
Plotted Output Filename: Q:\All Projects\Active Projects\08 Active Project\17637-2000 OCWD Satiago Pits Intertie\Slope Stability Analyses final\slope analysis b-b' 50' temporary1.PI
PROBLEM DESCRIPTION: Cross Section B-B'
Static Case (50' temporary)

BOUNDARY COORDINATES
4 Top Boundaries
4 Total Boundaries

Boundary No.	X-Left (ft)	Y-Left (ft)	X-Right (ft)	Y-Right (ft)	Soil Type Below Bnd
1	0.00	190.00	200.00	190.00	1
2	200.00	190.00	250.00	240.00	1
3	250.00	240.00	435.00	330.00	1
4	435.00	330.00	600.00	330.00	1

User Specified Y-Origin = 100.00(ft)
Default X-Plus Value = 0.00(ft)
Default Y-Plus Value = 0.00(ft)

ISOTROPIC SOIL PARAMETERS
1 Type(s) of Soil

Soil Type No.	Total Unit Wt. (pcf)	Saturated Unit Wt. (pcf)	Cohesion Intercept (psf)	Friction Angle (deg)	Pore Pressure Param.	Pressure Constant (psf)	Piez. Surface No.
1	120.0	120.0	100.0	45.0	0.00	0.0	0

EARTHQUAKE DATA HAS BEEN SUPPRESSED
A Critical Failure Surface Searching Method, Using A Random Technique For Generating Circular Surfaces, Has Been Specified.
250 Trial Surfaces Have Been Generated.
5 Surface(s) Initiate(s) From Each Of 50 Points Equally Spaced
Along The Ground Surface Between X = 200.00(ft)
and X = 200.10(ft)
Each Surface Terminates Between X = 230.00(ft)
and X = 410.00(ft)
Unless Further Limitations Were Imposed, The Minimum Elevation
At Which A Surface Extends Is Y = 0.00(ft)
5.00(ft) Line Segments Define Each Trial Failure Surface.
Following Are Displayed The Ten Most Critical Of The Trial
Failure Surfaces Evaluated. They Are
Ordered - Most Critical First.
* * Safety Factors Are Calculated By The Modified Bishop Method * *
Total Number of Trial Surfaces Attempted = 250
Number of Trial Surfaces With Valid FS = 250
Statistical Data On All Valid FS Values:
FS Max = 3.142 FS Min = 1.359 FS Ave = 2.068
Standard Deviation = 0.507 Coefficient of Variation = 24.51 %
Failure Surface Specified By 17 Coordinate Points

Point No.	X-Surf (ft)	Y-Surf (ft)

1	200.012	190.012
2	204.562	192.087
3	209.016	194.357
4	213.367	196.821
5	217.607	199.471
6	221.727	202.305
7	225.719	205.315
8	229.576	208.497
9	233.290	211.844
10	236.855	215.350
11	240.263	219.009
12	243.509	222.812
13	246.585	226.754
14	249.486	230.826
15	252.207	235.021
16	254.742	239.331
17	256.872	243.343

Circle Center At X = 154.789 ; Y = 295.224 ; and Radius = 114.520

Factor of Safety

*** 1.359 ***

Individual data on the 17 slices

Slice No.	Width (ft)	Weight (lbs)	Water Force Top (lbs)	Water Force Bot (lbs)	Tie Force Norm (lbs)	Tie Force Tan (lbs)	Earthquake Force Hor (lbs)	Earthquake Force Ver (lbs)	Surcharge Load (lbs)
1	4.5	675.6	0.0	0.0	0.	0.	0.0	0.0	0.0
2	4.5	1906.7	0.0	0.0	0.	0.	0.0	0.0	0.0
3	4.4	2925.4	0.0	0.0	0.	0.	0.0	0.0	0.0
4	4.2	3734.7	0.0	0.0	0.	0.	0.0	0.0	0.0
5	4.1	4339.9	0.0	0.0	0.	0.	0.0	0.0	0.0
6	4.0	4748.8	0.0	0.0	0.	0.	0.0	0.0	0.0
7	3.9	4971.4	0.0	0.0	0.	0.	0.0	0.0	0.0
8	3.7	5020.0	0.0	0.0	0.	0.	0.0	0.0	0.0
9	3.6	4908.8	0.0	0.0	0.	0.	0.0	0.0	0.0
10	3.4	4654.3	0.0	0.0	0.	0.	0.0	0.0	0.0
11	3.2	4274.4	0.0	0.0	0.	0.	0.0	0.0	0.0
12	3.1	3788.8	0.0	0.0	0.	0.	0.0	0.0	0.0
13	2.9	3218.8	0.0	0.0	0.	0.	0.0	0.0	0.0
14	0.5	525.5	0.0	0.0	0.	0.	0.0	0.0	0.0
15	2.2	1911.2	0.0	0.0	0.	0.	0.0	0.0	0.0
16	2.5	1373.3	0.0	0.0	0.	0.	0.0	0.0	0.0
17	2.1	380.4	0.0	0.0	0.	0.	0.0	0.0	0.0

Failure Surface Specified By 17 Coordinate Points

Point No.	X-Surf (ft)	Y-Surf (ft)
1	200.018	190.018
2	204.425	192.381
3	208.748	194.894
4	212.981	197.555
5	217.119	200.361
6	221.158	203.308
7	225.093	206.394
8	228.918	209.613
9	232.630	212.963
10	236.224	216.440
11	239.695	220.038
12	243.040	223.755
13	246.254	227.585
14	249.334	231.524
15	252.275	235.567
16	255.076	239.709
17	257.564	243.680

Circle Center At X = 133.949 ; Y = 318.570 ; and Radius = 144.536

Factor of Safety

*** 1.362 ***

Failure Surface Specified By 18 Coordinate Points

Point X-Surf Y-Surf

No.	(ft)	(ft)
1	200.004	190.004
2	204.599	191.977
3	209.105	194.144
4	213.514	196.501
5	217.818	199.045
6	222.010	201.771
7	226.082	204.673
8	230.026	207.746
9	233.835	210.985
10	237.502	214.384
11	241.020	217.937
12	244.383	221.636
13	247.586	225.476
14	250.621	229.450
15	253.484	233.549
16	256.168	237.767
17	258.671	242.096
18	260.157	244.941

Circle Center At X = 156.263 ; Y = 298.298 ; and Radius = 116.794

Factor of Safety

*** 1.368 ***

Failure Surface Specified By 18 Coordinate Points

Point No.	X-Surf (ft)	Y-Surf (ft)
1	200.076	190.076
2	204.673	192.041
3	209.184	194.198
4	213.601	196.541
5	217.916	199.067
6	222.122	201.771
7	226.210	204.650
8	230.175	207.696
9	234.008	210.906
10	237.704	214.274
11	241.256	217.793
12	244.657	221.458
13	247.902	225.262
14	250.985	229.198
15	253.901	233.260
16	256.644	237.441
17	259.209	241.732
18	261.234	245.465

Circle Center At X = 155.480 ; Y = 300.774 ; and Radius = 119.344

Factor of Safety

*** 1.371 ***

Failure Surface Specified By 19 Coordinate Points

Point No.	X-Surf (ft)	Y-Surf (ft)
1	200.096	190.096
2	204.481	192.499
3	208.800	195.018
4	213.051	197.651
5	217.230	200.395
6	221.335	203.249
7	225.363	206.212
8	229.311	209.280
9	233.176	212.452
10	236.956	215.725
11	240.647	219.098
12	244.248	222.567
13	247.755	226.131
14	251.166	229.787
15	254.479	233.531
16	257.692	237.363
17	260.801	241.278
18	263.806	245.275

19	265.374	247.479

Circle Center At X = 111.761 ; Y = 356.459 ; and Radius = 188.361
Factor of Safety
*** 1.381 ***
Failure Surface Specified By 19 Coordinate Points

Point No.	X-Surf (ft)	Y-Surf (ft)
1	200.031	190.031
2	204.677	191.877
3	209.240	193.921
4	213.711	196.159
5	218.082	198.587
6	222.345	201.200
7	226.492	203.994
8	230.515	206.963
9	234.407	210.102
10	238.160	213.405
11	241.769	216.866
12	245.225	220.479
13	248.523	224.237
14	251.657	228.133
15	254.621	232.160
16	257.410	236.310
17	260.017	240.576
18	262.440	244.950
19	263.165	246.405

Circle Center At X = 159.348 ; Y = 299.180 ; and Radius = 116.485
Factor of Safety
*** 1.384 ***
Failure Surface Specified By 18 Coordinate Points

Point No.	X-Surf (ft)	Y-Surf (ft)
1	200.039	190.039
2	204.295	192.663
3	208.482	195.396
4	212.597	198.236
5	216.638	201.181
6	220.601	204.229
7	224.485	207.378
8	228.287	210.626
9	232.003	213.971
10	235.632	217.410
11	239.172	220.942
12	242.619	224.563
13	245.972	228.272
14	249.228	232.067
15	252.386	235.944
16	255.442	239.900
17	258.396	243.935
18	258.553	244.161

Circle Center At X = 100.309 ; Y = 356.553 ; and Radius = 194.096
Factor of Safety
*** 1.389 ***
Failure Surface Specified By 16 Coordinate Points

Point No.	X-Surf (ft)	Y-Surf (ft)
1	200.016	190.016
2	204.619	191.970
3	209.102	194.184
4	213.452	196.649
5	217.654	199.358
6	221.695	202.303
7	225.561	205.474
8	229.240	208.860
9	232.720	212.450
10	235.990	216.233
11	239.038	220.196

12	241.855	224.327
13	244.433	228.612
14	246.761	233.036
15	248.833	237.587
16	249.624	239.624

Circle Center At X = 168.205 ; Y = 271.435 ; and Radius = 87.413

Factor of Safety

*** 1.392 ***

Failure Surface Specified By 18 Coordinate Points

Point No.	X-Surf (ft)	Y-Surf (ft)
1	200.000	190.000
2	204.734	191.608
3	209.380	193.457
4	213.924	195.543
5	218.355	197.859
6	222.661	200.400
7	226.831	203.159
8	230.854	206.129
9	234.718	209.302
10	238.415	212.669
11	241.933	216.221
12	245.264	219.950
13	248.399	223.845
14	251.330	227.896
15	254.048	232.093
16	256.548	236.423
17	258.821	240.876
18	260.773	245.241

Circle Center At X = 171.118 ; Y = 282.825 ; and Radius = 97.214

Factor of Safety

*** 1.398 ***

Failure Surface Specified By 18 Coordinate Points

Point No.	X-Surf (ft)	Y-Surf (ft)
1	200.000	190.000
2	204.776	191.481
3	209.456	193.241
4	214.024	195.274
5	218.464	197.573
6	222.760	200.131
7	226.898	202.937
8	230.863	205.983
9	234.641	209.258
10	238.220	212.750
11	241.586	216.447
12	244.728	220.337
13	247.635	224.405
14	250.296	228.638
15	252.703	233.020
16	254.847	237.537
17	256.721	242.173
18	257.163	243.485

Circle Center At X = 177.393 ; Y = 271.452 ; and Radius = 84.531

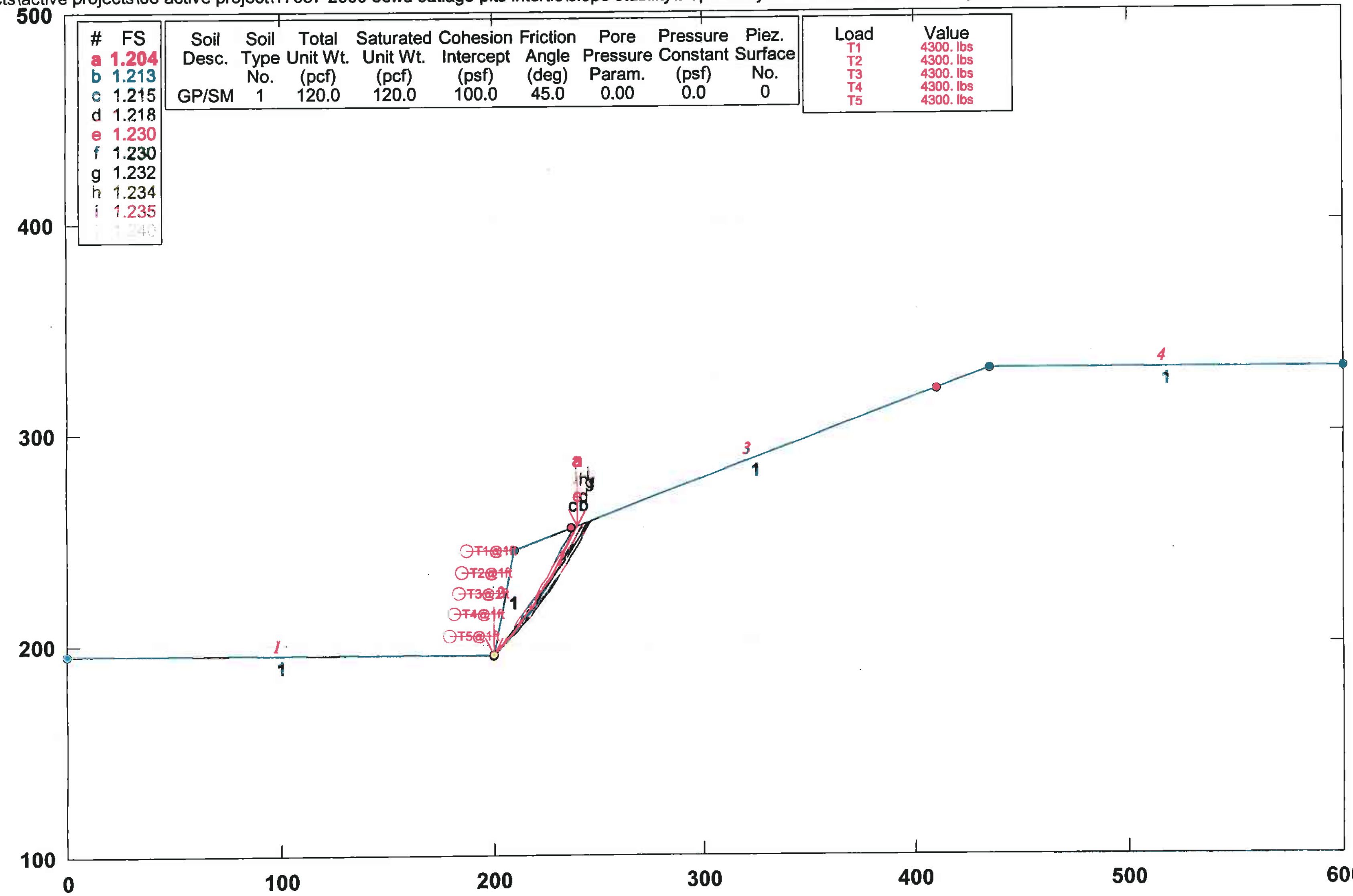
Factor of Safety

*** 1.406 ***

**** END OF GSTABL7 OUTPUT ****

Cross Section B-B' Static Case (50 feet Vertical)

q:\all projects\active projects\08 active project\17637-2000 ocwd satiago pits intertie\slope stability\slope analysis b-b' static 195' bottom.pl2 Run By: Sean. M, Willdan Geotechnical 5/12/2009 09:10



GSTABL7 v.2 FSmin=1.204

Safety Factors Are Calculated By The Modified Bishop Method

GSTABL7

*** **GSTABL7** ***

** GSTABL7 by Garry H. Gregory, P.E. **

** Original Version 1.0, January 1996; Current Version 2.005, Sept. 2006 **

****

SLOPE STABILITY ANALYSIS SYSTEM

Modified Bishop, Simplified Janbu, or GLE Method of Slices.
(Includes Spencer & Morgenstern-Price Type Analysis)
Including Pier/Pile, Reinforcement, Soil Nail, Tieback,
Nonlinear Undrained Shear Strength, Curved Phi Envelope,
Anisotropic Soil, Fiber-Reinforced Soil, Boundary Loads, Water
Surfaces, Pseudo-Static & Newmark Earthquake, and Applied Forces.

****

Analysis Run Date: 5/12/2009
Time of Run: 09:10AM
Run By: Sean. M, Willdan Geotechnical
Input Data Filename: Q:\All Projects\Active Projects\08 Active Project\17637-2000 OCWD Satiago Pits Intertie\Slope Stability\slope analysis b-b' static 195' bottom.in
Output Filename: Q:\All Projects\Active Projects\08 Active Project\17637-2000 OCWD Satiago Pits Intertie\Slope Stability\slope analysis b-b' static 195' bottom.OUT
Unit System: English
Plotted Output Filename: Q:\All Projects\Active Projects\08 Active Project\17637-2000 OCWD Satiago Pits Intertie\Slope Stability\slope analysis b-b' static 195' bottom.PLT
PROBLEM DESCRIPTION: Cross Section B-B'
Static Case (50 feet Vertical)

BOUNDARY COORDINATES
4 Top Boundaries
4 Total Boundaries

Boundary No.	X-Left (ft)	Y-Left (ft)	X-Right (ft)	Y-Right (ft)	Soil Type Below Bnd
1	0.00	195.00	200.00	195.00	1
2	200.00	195.00	210.00	245.00	1
3	210.00	245.00	435.00	330.00	1
4	435.00	330.00	600.00	330.00	1

User Specified Y-Origin = 100.00(ft)
Default X-Plus Value = 0.00(ft)
Default Y-Plus Value = 0.00(ft)

ISOTROPIC SOIL PARAMETERS
1 Type(s) of Soil

Soil Type No.	Total Unit Wt. (pcf)	Saturated Unit Wt. (pcf)	Cohesion Intercept (psf)	Friction Angle (deg)	Pore Pressure Param.	Pressure Constant (psf)	Piez. Surface No.
1	120.0	120.0	100.0	45.0	0.00	0.0	0

EARTHQUAKE DATA HAS BEEN SUPPRESSED

TIEBACK LOAD(S)
5 Tieback Load(s) Specified

Tieback No.	X-Pos (ft)	Y-Pos (ft)	Load (lbs)	Spacing (ft)	Inclination (deg)	Length (ft)	Force Method
1	209.90	244.50	4300.0	1.0	0.00	0.0	2
2	207.90	234.50	4300.0	1.0	0.00	0.0	2
3	205.90	224.50	4300.0	1.0	0.00	0.0	2
4	203.90	214.50	4300.0	1.0	0.00	0.0	2
5	201.90	204.50	4300.0	1.0	0.00	0.0	2

NOTE - An Equivalent Line Load Is Calculated For Each Row Of Tiebacks Assuming A Uniform Distribution Of Load Horizontally Between Individual Tiebacks. Force Method 1 Considers Only Tangential Tieback Forces. Force Method 2 Considers Both Tangential and Normal Tieback Forces. Force Method 3 Considers Only Normal Tieback Forces. Force Method 4 Limits Normal and Tangential Tieback-Force Distribution to 1.5 Times the Tieback Inclination, or to 30 Degrees Below (Left of) the Tieback-Failure Surface Intersection, Whichever is Greater.

A Critical Failure Surface Searching Method, Using A Random Technique For Generating Circular Surfaces, Has Been Specified.
250 Trial Surfaces Have Been Generated.
5 Surface(s) Initiate(s) From Each Of 50 Points Equally Spaced
Along The Ground Surface Between X = 200.00(ft)
and X = 200.10(ft)
Each Surface Terminates Between X = 237.00(ft)

and X = 410.00(ft)
Unless Further Limitations Were Imposed, The Minimum Elevation
At Which A Surface Extends Is Y = 0.00(ft)
5.00(ft) Line Segments Define Each Trial Failure Surface.
Following Are Displayed The Ten Most Critical Of The Trial
Failure Surfaces Evaluated. They Are
Ordered - Most Critical First.
* * Safety Factors Are Calculated By The Modified Bishop Method * *
Total Number of Trial Surfaces Attempted = 250
Number of Trial Surfaces With Valid FS = 250
Statistical Data On All Valid FS Values:
FS Max = 4.076 FS Min = 1.204 FS Ave = 2.432
Standard Deviation = 0.912 Coefficient of Variation = 37.49 %
Failure Surface Specified By 16 Coordinate Points

Point No.	X-Surf (ft)	Y-Surf (ft)
1	200.084	195.419
2	203.425	199.139
3	206.680	202.934
4	209.848	206.803
5	212.927	210.742
6	215.915	214.751
7	218.811	218.827
8	221.614	222.968
9	224.321	227.171
10	226.932	231.435
11	229.446	235.758
12	231.860	240.136
13	234.173	244.569
14	236.386	249.053
15	238.495	253.586
16	239.638	256.197

Circle Center At X = 38.887 ; Y = 343.587 ; and Radius = 218.947
Factor of Safety
*** 1.204 ***
Individual data on the 16 slices

Slice No.	Width (ft)	Weight (lbs)	Water Force Top (lbs)	Water Force Bot (lbs)	Tie Force Norm (lbs)	Tie Force Tan (lbs)	Earthquake Force Hor (lbs)	Earthquake Force Ver (lbs)	Surcharge Load (lbs)
1	3.3	2602.6	0.0	0.0	0.	0.	0.0	0.0	0.0
2	3.3	7509.4	0.0	0.0	1246.	0.	0.0	0.0	0.0
3	3.2	11955.6	0.0	0.0	1374.	893.	0.0	0.0	0.0
4	0.2	689.4	0.0	0.0	65.	39.	0.0	0.0	0.0
5	2.9	12882.6	0.0	0.0	1378.	634.	0.0	0.0	0.0
6	3.0	12164.9	0.0	0.0	1619.	865.	0.0	0.0	0.0
7	2.9	10771.4	0.0	0.0	1700.	850.	0.0	0.0	0.0
8	2.8	9403.6	0.0	0.0	1781.	945.	0.0	0.0	0.0
9	2.7	8067.3	0.0	0.0	1809.	947.	0.0	0.0	0.0
10	2.6	6768.0	0.0	0.0	1779.	1029.	0.0	0.0	0.0
11	2.5	5511.6	0.0	0.0	1669.	1039.	0.0	0.0	0.0
12	2.4	4303.5	0.0	0.0	1482.	1125.	0.0	0.0	0.0
13	2.3	3149.3	0.0	0.0	1245.	1159.	0.0	0.0	0.0
14	2.2	2054.5	0.0	0.0	1006.	1128.	0.0	0.0	0.0
15	2.1	1024.4	0.0	0.0	800.	1053.	0.0	0.0	0.0
16	1.1	149.4	0.0	0.0	384.	557.	0.0	0.0	0.0

Failure Surface Specified By 17 Coordinate Points

Point No.	X-Surf (ft)	Y-Surf (ft)
1	200.014	195.072
2	203.331	198.813
3	206.582	202.612
4	209.765	206.468
5	212.881	210.378
6	215.928	214.343
7	218.905	218.360
8	221.811	222.428

9	224.646	226.547
10	227.408	230.715
11	230.096	234.931
12	232.711	239.193
13	235.251	243.500
14	237.714	247.851
15	240.102	252.244
16	242.412	256.678
17	242.760	257.376

Circle Center At X = -12.126 ; Y = 386.457 ; and Radius = 285.713

Factor of Safety

*** 1.213 ***

Failure Surface Specified By 16 Coordinate Points

Point No.	X-Surf (ft)	Y-Surf (ft)
1	200.049	195.245
2	203.109	199.199
3	206.112	203.198
4	209.056	207.239
5	211.941	211.322
6	214.767	215.447
7	217.533	219.613
8	220.238	223.818
9	222.881	228.062
10	225.463	232.344
11	227.982	236.663
12	230.439	241.018
13	232.832	245.408
14	235.161	249.832
15	237.426	254.290
16	238.069	255.604

Circle Center At X = -70.425 ; Y = 407.772 ; and Radius = 343.983

Factor of Safety

*** 1.215 ***

Failure Surface Specified By 17 Coordinate Points

Point No.	X-Surf (ft)	Y-Surf (ft)
1	200.051	195.256
2	203.346	199.016
3	206.579	202.830
4	209.749	206.697
5	212.855	210.615
6	215.896	214.584
7	218.871	218.603
8	221.780	222.669
9	224.622	226.783
10	227.396	230.944
11	230.100	235.149
12	232.736	239.398
13	235.301	243.690
14	237.795	248.023
15	240.218	252.397
16	242.568	256.810
17	242.882	257.422

Circle Center At X = -27.041 ; Y = 397.575 ; and Radius = 304.144

Factor of Safety

*** 1.218 ***

Failure Surface Specified By 16 Coordinate Points

Point No.	X-Surf (ft)	Y-Surf (ft)
1	200.041	195.204
2	203.021	199.220
3	205.965	203.260
4	208.875	207.326
5	211.750	211.417
6	214.589	215.533
7	217.393	219.673

```
     8        220.161       223.837
     9        222.892       228.025
    10        225.587       232.236
    11        228.246       236.471
    12        230.869       240.728
    13        233.454       245.008
    14        236.002       249.310
    15        238.513       253.634
    16        240.064       256.357
 Circle Center At X =  -262.298 ; Y =   541.465 ; and Radius =   577.627
        Factor of Safety
       ***    1.230   ***
 Failure Surface Specified By 17 Coordinate Points
   Point      X-Surf       Y-Surf
    No.        (ft)         (ft)
     1        200.016       195.082
     2        203.501       198.668
     3        206.918       202.318
     4        210.266       206.031
     5        213.545       209.806
     6        216.753       213.641
     7        219.889       217.536
     8        222.951       221.488
     9        225.940       225.496
    10        228.853       229.560
    11        231.690       233.677
    12        234.450       237.847
    13        237.132       242.067
    14        239.734       246.336
    15        242.257       250.653
    16        244.699       255.016
    17        246.774       258.892
 Circle Center At X =     9.340 ; Y =   383.857 ; and Radius =   268.316
        Factor of Safety
       ***    1.230   ***
 Failure Surface Specified By 17 Coordinate Points
   Point      X-Surf       Y-Surf
    No.        (ft)         (ft)
     1        200.004       195.020
     2        203.199       198.866
     3        206.359       202.742
     4        209.483       206.646
     5        212.570       210.578
     6        215.621       214.540
     7        218.636       218.529
     8        221.614       222.545
     9        224.554       226.589
    10        227.457       230.660
    11        230.322       234.758
    12        233.150       238.881
    13        235.939       243.031
    14        238.690       247.206
    15        241.403       251.407
    16        244.076       255.632
    17        245.890       258.559
 Circle Center At X =  -215.471 ; Y =   543.451 ; and Radius =   542.239
        Factor of Safety
       ***    1.232   ***
 Failure Surface Specified By 17 Coordinate Points
   Point      X-Surf       Y-Surf
    No.        (ft)         (ft)
     1        200.012       195.061
     2        203.069       199.018
     3        206.096       202.998
     4        209.094       206.999
     5        212.063       211.023
     6        215.002       215.067
```

7	217.912	219.134
8	220.791	223.221
9	223.641	227.329
10	226.461	231.459
11	229.250	235.608
12	232.009	239.778
13	234.737	243.968
14	237.435	248.178
15	240.102	252.407
16	242.738	256.656
17	243.302	257.581

Circle Center At X = -339.432 ; Y = 614.862 ; and Radius = 683.545

Factor of Safety

*** 1.234 ***

Failure Surface Specified By 17 Coordinate Points

Point No.	X-Surf (ft)	Y-Surf (ft)
1	200.084	195.419
2	203.686	198.887
3	207.203	202.441
4	210.631	206.081
5	213.969	209.803
6	217.215	213.607
7	220.366	217.488
8	223.422	221.446
9	226.379	225.478
10	229.237	229.581
11	231.993	233.752
12	234.647	237.990
13	237.196	242.292
14	239.638	246.654
15	241.973	251.076
16	244.199	255.553
17	245.540	258.426

Circle Center At X = 60.428 ; Y = 344.121 ; and Radius = 204.000

Factor of Safety

*** 1.235 ***

Failure Surface Specified By 16 Coordinate Points

Point No.	X-Surf (ft)	Y-Surf (ft)
1	200.025	195.123
2	203.989	198.169
3	207.796	201.411
4	211.435	204.840
5	214.898	208.446
6	218.176	212.222
7	221.261	216.157
8	224.145	220.241
9	226.820	224.466
10	229.280	228.819
11	231.519	233.289
12	233.531	237.867
13	235.311	242.539
14	236.854	247.295
15	238.157	252.122
16	238.988	255.951

Circle Center At X = 141.366 ; Y = 275.589 ; and Radius = 99.578

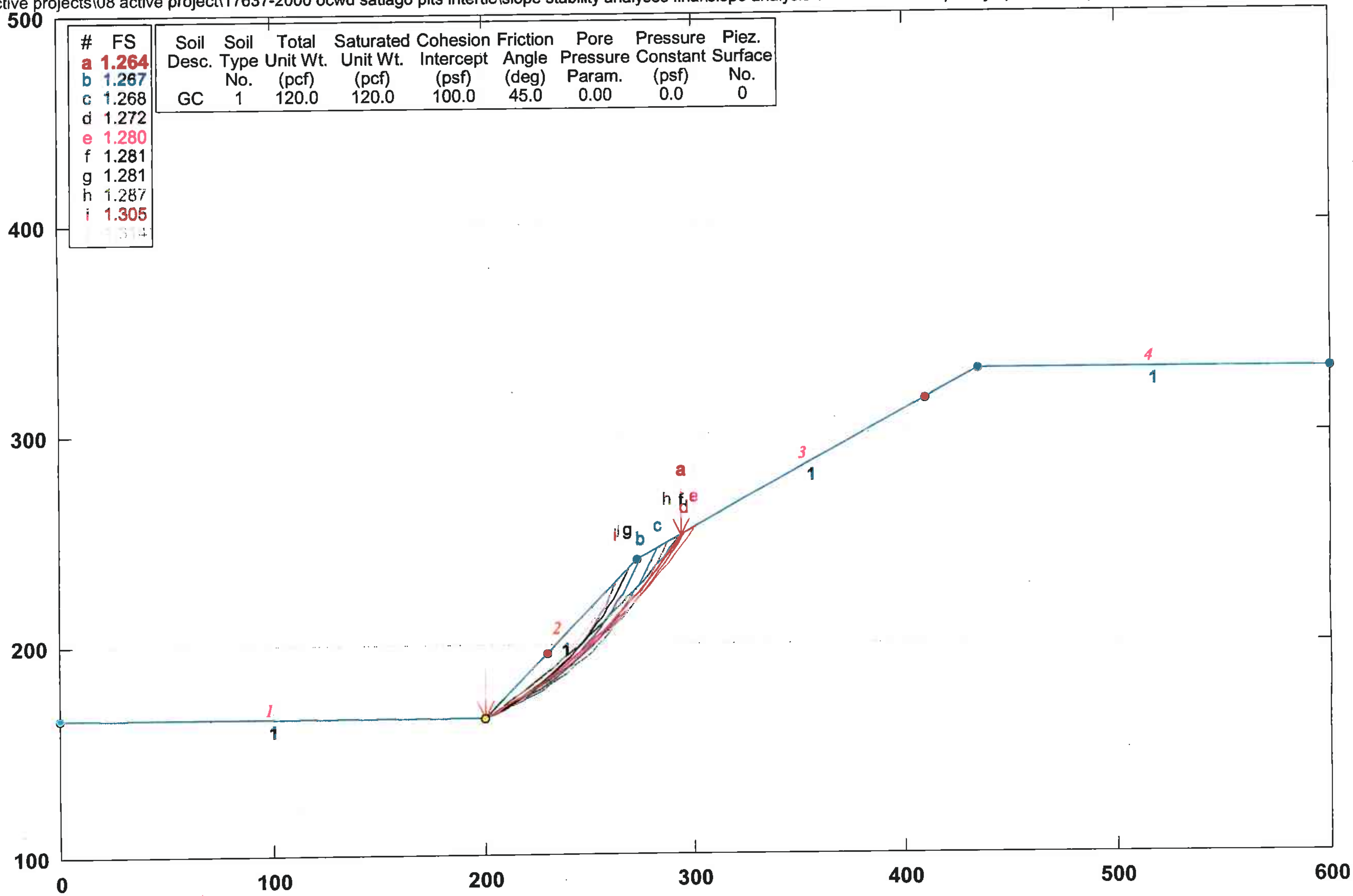
Factor of Safety

*** 1.240 ***

**** END OF GSTABL7 OUTPUT ****

Cross Section B-B' Static Case (75' temporary)

q:\all projects\active projects\08 active project\17637-2000 ocwd satiago pits intertie\slope stability analyses final\slope analysis b-b' static 75' temporary1.pl2 Run By: Sean. M, Willdan Geotechnical 5/12/2009 12



GSTABL7 v.2 FSmin=1.264
Safety Factors Are Calculated By The Modified Bishop Method

GSTABL7

```
                           ***  GSTABL7  ***
                 ** GSTABL7 by Garry H. Gregory, P.E. **
   ** Original Version 1.0, January 1996; Current Version 2.005, Sept. 2006 **
              (All Rights Reserved-Unauthorized Use Prohibited)
*********************************************************************************
                    SLOPE STABILITY ANALYSIS SYSTEM
      Modified Bishop, Simplified Janbu, or GLE Method of Slices.
      (Includes Spencer & Morgenstern-Price Type Analysis)
      Including Pier/Pile, Reinforcement, Soil Nail, Tieback,
      Nonlinear Undrained Shear Strength, Curved Phi Envelope,
      Anisotropic Soil, Fiber-Reinforced Soil, Boundary Loads, Water
      Surfaces, Pseudo-Static & Newmark Earthquake, and Applied Forces.
*********************************************************************************
Analysis Run Date:        5/12/2009
Time of Run:              12:48PM
Run By:                   Sean. M, Willdan Geotechnical
Input Data Filename:      Q:\All Projects\Active Projects\08 Active Project\17637-2000
OCWD Satiago Pits Intertie\Slope Stability Analyses final\slope analysis b-b' static 75' tempor
Output Filename:          Q:\All Projects\Active Projects\08 Active Project\17637-2000
OCWD Satiago Pits Intertie\Slope Stability Analyses final\slope analysis b-b' static 75' tempor
Unit System:              English
Plotted Output Filename:  Q:\All Projects\Active Projects\08 Active Project\17637-2000
OCWD Satiago Pits Intertie\Slope Stability Analyses final\slope analysis b-b' static 75' tempor
PROBLEM DESCRIPTION:  Cross Section B-B'
                      Static Case (75' temporary)
BOUNDARY COORDINATES
    4 Top   Boundaries
    4 Total Boundaries
Boundary    X-Left     Y-Left    X-Right    Y-Right    Soil Type
  No.        (ft)       (ft)       (ft)       (ft)     Below Bnd
   1         0.00     165.00     200.00     165.00        1
   2       200.00     165.00     273.00     240.00        1
   3       273.00     240.00     435.00     330.00        1
   4       435.00     330.00     600.00     330.00        1
User Specified Y-Origin =        100.00(ft)
Default X-Plus Value = 0.00(ft)
Default Y-Plus Value = 0.00(ft)
ISOTROPIC SOIL PARAMETERS
  1 Type(s) of Soil
Soil  Total  Saturated  Cohesion Friction   Pore    Pressure   Piez.
Type Unit Wt. Unit Wt. Intercept  Angle   Pressure Constant  Surface
 No.  (pcf)    (pcf)     (psf)    (deg)    Param.    (psf)     No.
  1   120.0    120.0     100.0     45.0     0.00      0.0       0
EARTHQUAKE DATA HAS BEEN SUPPRESSED
A Critical Failure Surface Searching Method, Using A Random
Technique For Generating Circular Surfaces, Has Been Specified.
 100 Trial Surfaces Have Been Generated.
   2 Surface(s) Initiate(s) From Each Of    50 Points Equally Spaced
Along The Ground Surface Between  X = 200.00(ft)
                             and  X = 200.10(ft)
Each Surface Terminates Between   X = 230.00(ft)
                             and  X = 410.00(ft)
Unless Further Limitations Were Imposed, The Minimum Elevation
At Which A Surface Extends Is  Y =      0.00(ft)
 5.00(ft) Line Segments Define Each Trial Failure Surface.
Following Are Displayed The Ten Most Critical Of The Trial
      Failure Surfaces Evaluated. They Are
      Ordered - Most Critical First.
      * * Safety Factors Are Calculated By The Modified Bishop Method * *
      Total Number of Trial Surfaces Attempted =   100
      Number of Trial Surfaces With Valid FS =  100
      Statistical Data On All Valid FS Values:
         FS Max =   2.378   FS Min =   1.264   FS Ave =   1.705
         Standard Deviation =    0.356   Coefficient of Variation =   20.90 %
      Failure Surface Specified By 27 Coordinate Points
        Point       X-Surf       Y-Surf
         No.         (ft)         (ft)
          1        200.004      165.004
```

2	204.396	167.394
3	208.739	169.871
4	213.033	172.433
5	217.275	175.081
6	221.463	177.811
7	225.596	180.625
8	229.673	183.520
9	233.691	186.495
10	237.650	189.550
11	241.547	192.682
12	245.381	195.891
13	249.151	199.176
14	252.855	202.535
15	256.491	205.966
16	260.059	209.469
17	263.556	213.043
18	266.982	216.685
19	270.335	220.394
20	273.613	224.169
21	276.816	228.009
22	279.942	231.911
23	282.990	235.875
24	285.959	239.898
25	288.847	243.979
26	291.654	248.117
27	293.937	251.632

Circle Center At X = 81.802 ; Y = 387.436 ; and Radius = 251.888

Factor of Safety

*** 1.264 ***

Individual data on the 27 slices

Slice No.	Width (ft)	Weight (lbs)	Water Force Top (lbs)	Water Force Bot (lbs)	Tie Force Norm (lbs)	Tie Force Tan (lbs)	Earthquake Force Hor (lbs)	Earthquake Force Ver (lbs)	Surcharge Load (lbs)
1	4.4	559.2	0.0	0.0	0.	0.	0.0	0.0	0.0
2	4.3	1623.6	0.0	0.0	0.	0.	0.0	0.0	0.0
3	4.3	2592.6	0.0	0.0	0.	0.	0.0	0.0	0.0
4	4.2	3467.3	0.0	0.0	0.	0.	0.0	0.0	0.0
5	4.2	4248.7	0.0	0.0	0.	0.	0.0	0.0	0.0
6	4.1	4938.2	0.0	0.0	0.	0.	0.0	0.0	0.0
7	4.1	5537.5	0.0	0.0	0.	0.	0.0	0.0	0.0
8	4.0	6048.2	0.0	0.0	0.	0.	0.0	0.0	0.0
9	4.0	6472.6	0.0	0.0	0.	0.	0.0	0.0	0.0
10	3.9	6812.7	0.0	0.0	0.	0.	0.0	0.0	0.0
11	3.8	7071.1	0.0	0.0	0.	0.	0.0	0.0	0.0
12	3.8	7250.5	0.0	0.0	0.	0.	0.0	0.0	0.0
13	3.7	7353.6	0.0	0.0	0.	0.	0.0	0.0	0.0
14	3.6	7383.6	0.0	0.0	0.	0.	0.0	0.0	0.0
15	3.6	7343.7	0.0	0.0	0.	0.	0.0	0.0	0.0
16	3.5	7237.3	0.0	0.0	0.	0.	0.0	0.0	0.0
17	3.4	7068.1	0.0	0.0	0.	0.	0.0	0.0	0.0
18	3.4	6839.7	0.0	0.0	0.	0.	0.0	0.0	0.0
19	2.7	5341.8	0.0	0.0	0.	0.	0.0	0.0	0.0
20	0.6	1203.6	0.0	0.0	0.	0.	0.0	0.0	0.0
21	3.2	5819.6	0.0	0.0	0.	0.	0.0	0.0	0.0
22	3.1	4887.4	0.0	0.0	0.	0.	0.0	0.0	0.0
23	3.0	3954.1	0.0	0.0	0.	0.	0.0	0.0	0.0
24	3.0	3024.1	0.0	0.0	0.	0.	0.0	0.0	0.0
25	2.9	2101.5	0.0	0.0	0.	0.	0.0	0.0	0.0
26	2.8	1190.8	0.0	0.0	0.	0.	0.0	0.0	0.0
27	2.3	307.8	0.0	0.0	0.	0.	0.0	0.0	0.0

Failure Surface Specified By 23 Coordinate Points

Point No.	X-Surf (ft)	Y-Surf (ft)
1	200.074	165.076
2	204.612	167.175
3	209.074	169.430

4	213.456	171.838
5	217.752	174.396
6	221.957	177.102
7	226.066	179.951
8	230.073	182.941
9	233.975	186.067
10	237.766	189.327
11	241.442	192.716
12	244.999	196.230
13	248.432	199.866
14	251.737	203.617
15	254.911	207.481
16	257.948	211.453
17	260.847	215.527
18	263.603	219.699
19	266.212	223.964
20	268.673	228.317
21	270.982	232.752
22	273.135	237.264
23	274.750	240.972

Circle Center At X = 141.695 ; Y = 297.227 ; and Radius = 144.472

Factor of Safety

*** 1.267 ***

Failure Surface Specified By 25 Coordinate Points

Point No.	X-Surf (ft)	Y-Surf (ft)
1	200.086	165.088
2	204.685	167.049
3	209.218	169.159
4	213.680	171.415
5	218.066	173.816
6	222.372	176.358
7	226.592	179.039
8	230.723	181.856
9	234.760	184.806
10	238.699	187.886
11	242.535	191.093
12	246.265	194.423
13	249.884	197.873
14	253.389	201.439
15	256.776	205.117
16	260.041	208.904
17	263.181	212.795
18	266.193	216.786
19	269.073	220.873
20	271.818	225.052
21	274.426	229.318
22	276.893	233.667
23	279.218	238.094
24	281.397	242.594
25	282.619	245.344

Circle Center At X = 142.210 ; Y = 307.210 ; and Radius = 153.454

Factor of Safety

*** 1.268 ***

Failure Surface Specified By 28 Coordinate Points

Point No.	X-Surf (ft)	Y-Surf (ft)
1	200.090	165.092
2	204.562	167.328
3	208.984	169.662
4	213.354	172.093
5	217.669	174.619
6	221.927	177.239
7	226.126	179.953
8	230.265	182.758
9	234.341	185.654
10	238.352	188.639

11	242.297	191.712
12	246.173	194.870
13	249.978	198.114
14	253.711	201.440
15	257.371	204.847
16	260.954	208.335
17	264.459	211.900
18	267.885	215.542
19	271.230	219.258
20	274.493	223.047
21	277.671	226.907
22	280.764	230.836
23	283.769	234.832
24	286.685	238.893
25	289.512	243.017
26	292.247	247.203
27	294.889	251.448
28	295.518	252.510

Circle Center At X = 100.854 ; Y = 369.217 ; and Radius = 226.968

Factor of Safety

*** 1.272 ***

Failure Surface Specified By 29 Coordinate Points

Point No.	X-Surf (ft)	Y-Surf (ft)
1	200.053	165.055
2	204.440	167.454
3	208.784	169.930
4	213.083	172.483
5	217.336	175.111
6	221.542	177.815
7	225.700	180.592
8	229.808	183.443
9	233.865	186.366
10	237.869	189.360
11	241.820	192.424
12	245.715	195.559
13	249.555	198.761
14	253.337	202.031
15	257.061	205.368
16	260.725	208.770
17	264.329	212.236
18	267.870	215.766
19	271.349	219.358
20	274.763	223.010
21	278.112	226.723
22	281.395	230.494
23	284.611	234.323
24	287.758	238.208
25	290.836	242.149
26	293.844	246.143
27	296.780	250.190
28	299.645	254.288
29	300.198	255.110

Circle Center At X = 66.758 ; Y = 414.015 ; and Radius = 282.399

Factor of Safety

*** 1.280 ***

Failure Surface Specified By 27 Coordinate Points

Point No.	X-Surf (ft)	Y-Surf (ft)
1	200.041	165.042
2	204.193	167.828
3	208.309	170.666
4	212.390	173.555
5	216.434	176.495
6	220.441	179.486
7	224.411	182.526
8	228.342	185.616

9	232.233	188.755
10	236.086	191.943
11	239.898	195.178
12	243.669	198.461
13	247.398	201.792
14	251.086	205.168
15	254.731	208.591
16	258.333	212.058
17	261.891	215.571
18	265.405	219.128
19	268.874	222.729
20	272.298	226.373
21	275.676	230.060
22	279.007	233.788
23	282.291	237.558
24	285.528	241.369
25	288.717	245.221
26	291.857	249.111
27	293.763	251.535

Circle Center At X = -20.218 ; Y = 497.769 ; and Radius = 399.026

Factor of Safety

*** 1.281 ***

Failure Surface Specified By 22 Coordinate Points

Point No.	X-Surf (ft)	Y-Surf (ft)
1	200.051	165.053
2	204.524	167.287
3	208.916	169.677
4	213.221	172.219
5	217.435	174.911
6	221.551	177.750
7	225.565	180.731
8	229.472	183.851
9	233.267	187.107
10	236.945	190.494
11	240.502	194.007
12	243.934	197.644
13	247.236	201.399
14	250.404	205.267
15	253.434	209.244
16	256.322	213.325
17	259.066	217.505
18	261.661	221.779
19	264.105	226.141
20	266.394	230.586
21	268.525	235.109
22	268.750	235.634

Circle Center At X = 138.710 ; Y = 293.482 ; and Radius = 142.327

Factor of Safety

*** 1.281 ***

Failure Surface Specified By 26 Coordinate Points

Point No.	X-Surf (ft)	Y-Surf (ft)
1	200.086	165.088
2	204.750	166.890
3	209.352	168.845
4	213.887	170.951
5	218.349	173.207
6	222.735	175.608
7	227.038	178.154
8	231.255	180.840
9	235.381	183.665
10	239.411	186.624
11	243.341	189.716
12	247.166	192.936
13	250.883	196.280
14	254.487	199.746

15	257.974	203.329
16	261.341	207.025
17	264.583	210.831
18	267.698	214.743
19	270.682	218.755
20	273.531	222.863
21	276.243	227.064
22	278.815	231.352
23	281.243	235.723
24	283.525	240.172
25	285.658	244.694
26	286.987	247.771

Circle Center At X = 148.050 ; Y = 306.788 ; and Radius = 150.953
Factor of Safety
*** 1.287 ***

Failure Surface Specified By 20 Coordinate Points

Point No.	X-Surf (ft)	Y-Surf (ft)
1	200.010	165.010
2	204.589	167.020
3	209.074	169.230
4	213.457	171.635
5	217.731	174.231
6	221.885	177.013
7	225.913	179.975
8	229.807	183.112
9	233.558	186.418
10	237.161	189.885
11	240.607	193.508
12	243.889	197.279
13	247.003	201.192
14	249.941	205.237
15	252.698	209.409
16	255.268	213.697
17	257.647	218.095
18	259.830	222.594
19	261.812	227.184
20	262.638	229.354

Circle Center At X = 156.812 ; Y = 269.707 ; and Radius = 113.259
Factor of Safety
*** 1.305 ***

Failure Surface Specified By 20 Coordinate Points

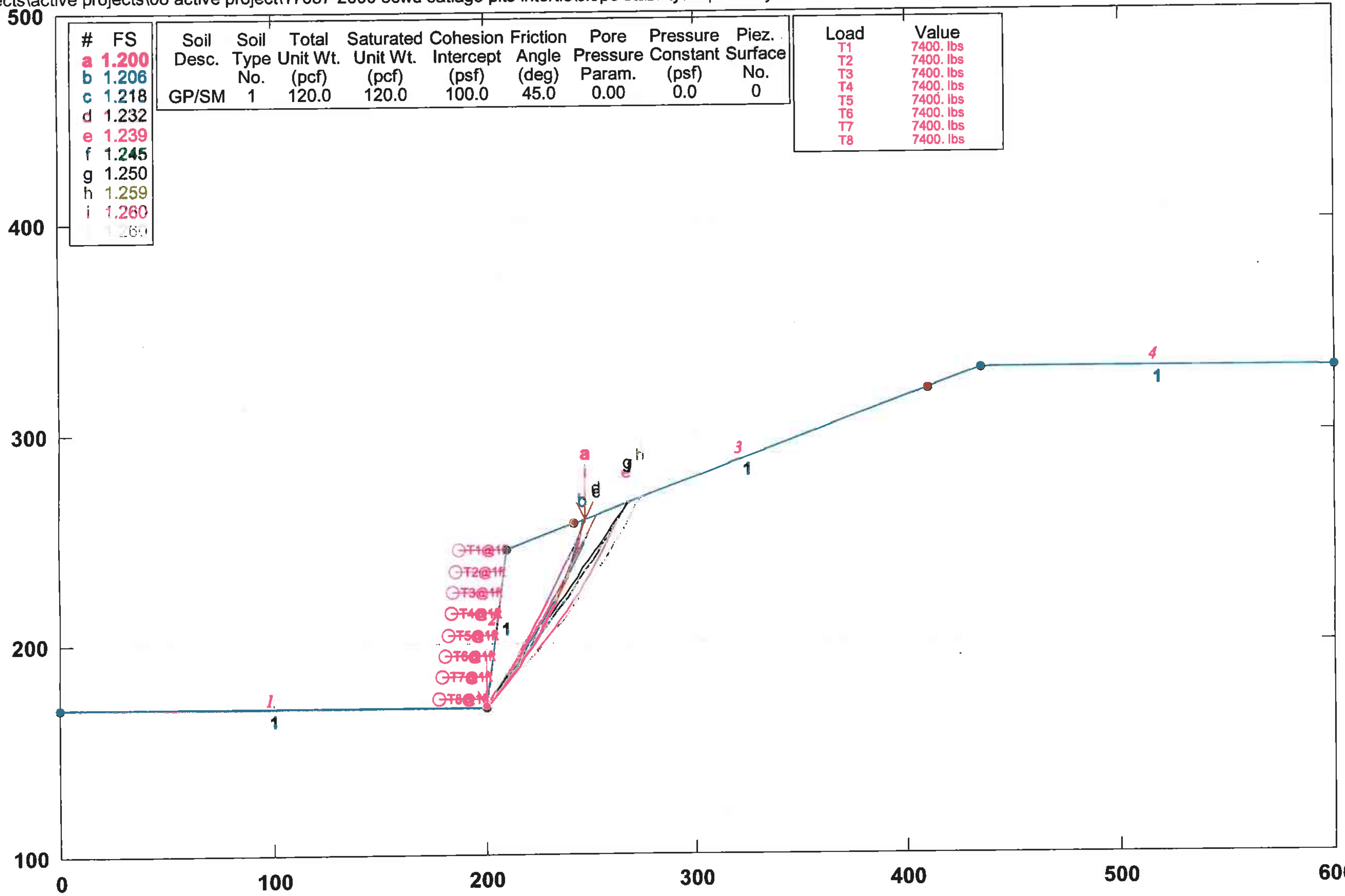
Point No.	X-Surf (ft)	Y-Surf (ft)
1	200.088	165.090
2	204.396	167.628
3	208.625	170.296
4	212.770	173.091
5	216.829	176.011
6	220.797	179.053
7	224.671	182.215
8	228.447	185.493
9	232.121	188.884
10	235.691	192.385
11	239.152	195.993
12	242.502	199.705
13	245.738	203.517
14	248.856	207.425
15	251.854	211.427
16	254.728	215.518
17	257.477	219.694
18	260.098	223.952
19	262.588	228.288
20	263.791	230.538

Circle Center At X = 118.957 ; Y = 307.784 ; and Radius = 164.145
Factor of Safety
*** 1.314 ***

**** END OF GSTABL7 OUTPUT ****

Cross Section B-B' Static Case (75 feet Vertical)

q:\all projects\active projects\08 active project\17637-2000 ocwd satiago pits intertie\slope stability\slope analysis b-b' static 75 feet.pl2 Run By: Sean. M, Willdan Geotechnical 5/12/2009 09:19AM



GSTABL7 v.2 FSmin=1.200

Safety Factors Are Calculated By The Modified Bishop Method

GSTABL7

*** **GSTABL7** ***

** GSTABL7 by Garry H. Gregory, P.E. **

** Original Version 1.0, January 1996; Current Version 2.005, Sept. 2006 **

****

SLOPE STABILITY ANALYSIS SYSTEM

Modified Bishop, Simplified Janbu, or GLE Method of Slices.
(Includes Spencer & Morgenstern-Price Type Analysis)
Including Pier/Pile, Reinforcement, Soil Nail, Tieback,
Nonlinear Undrained Shear Strength, Curved Phi Envelope,
Anisotropic Soil, Fiber-Reinforced Soil, Boundary Loads, Water
Surfaces, Pseudo-Static & Newmark Earthquake, and Applied Forces.

****

Analysis Run Date: 5/12/2009
Time of Run: 09:19AM
Run By: Sean. M, Willdan Geotechnical
Input Data Filename: Q:\All Projects\Active Projects\08 Active Project\17637-2000 OCWD Satiago Pits Intertie\Slope Stability\slope analysis b-b' static 75 feet.in
Output Filename: Q:\All Projects\Active Projects\08 Active Project\17637-2000 OCWD Satiago Pits Intertie\Slope Stability\slope analysis b-b' static 75 feet.OUT
Unit System: English
Plotted Output Filename: Q:\All Projects\Active Projects\08 Active Project\17637-2000 OCWD Satiago Pits Intertie\Slope Stability\slope analysis b-b' static 75 feet.PLT
PROBLEM DESCRIPTION: Cross Section B-B'
Static Case (75 feet Vertical)

BOUNDARY COORDINATES
4 Top Boundaries
4 Total Boundaries

Boundary No.	X-Left (ft)	Y-Left (ft)	X-Right (ft)	Y-Right (ft)	Soil Type Below Bnd
1	0.00	170.00	200.00	170.00	1
2	200.00	170.00	210.00	245.00	1
3	210.00	245.00	435.00	330.00	1
4	435.00	330.00	600.00	330.00	1

User Specified Y-Origin = 100.00(ft)
Default X-Plus Value = 0.00(ft)
Default Y-Plus Value = 0.00(ft)

ISOTROPIC SOIL PARAMETERS
1 Type(s) of Soil

Soil Type No.	Total Unit Wt. (pcf)	Saturated Unit Wt. (pcf)	Cohesion Intercept (psf)	Friction Angle (deg)	Pore Pressure Param.	Pressure Constant (psf)	Piez. Surface No.
1	120.0	120.0	100.0	45.0	0.00	0.0	0

EARTHQUAKE DATA HAS BEEN SUPPRESSED

TIEBACK LOAD(S)
8 Tieback Load(s) Specified

Tieback No.	X-Pos (ft)	Y-Pos (ft)	Load (lbs)	Spacing (ft)	Inclination (deg)	Length (ft)	Force Method
1	209.93	244.50	7400.0	1.0	0.00	0.0	2
2	208.60	234.50	7400.0	1.0	0.00	0.0	2
3	207.27	224.50	7400.0	1.0	0.00	0.0	2
4	205.93	214.50	7400.0	1.0	0.00	0.0	2
5	204.60	204.50	7400.0	1.0	0.00	0.0	2
6	203.27	194.50	7400.0	1.0	0.00	0.0	2
7	201.93	184.50	7400.0	1.0	0.00	0.0	2
8	200.60	174.50	7400.0	1.0	0.00	0.0	2

NOTE - An Equivalent Line Load Is Calculated For Each Row Of Tiebacks Assuming A Uniform Distribution Of Load Horizontally Between Individual Tiebacks. Force Method 1 Considers Only Tangential Tieback Forces. Force Method 2 Considers Both Tangential and Normal Tieback Forces. Force Method 3 Considers Only Normal Tieback Forces. Force Method 4 Limits Normal and Tangential Tieback-Force Distribution to 1.5 Times the Tieback Inclination, or to 30 Degrees Below (Left of) the Tieback-Failure Surface Intersection, Whichever is Greater.

A Critical Failure Surface Searching Method, Using A Random Technique For Generating Circular Surfaces, Has Been Specified.
100 Trial Surfaces Have Been Generated.
2 Surface(s) Initiate(s) From Each Of 50 Points Equally Spaced

Along The Ground Surface Between X = 200.00(ft)
and X = 200.10(ft)
Each Surface Terminates Between X = 242.00(ft)
and X = 410.00(ft)
Unless Further Limitations Were Imposed, The Minimum Elevation
At Which A Surface Extends Is Y = 0.00(ft)
5.00(ft) Line Segments Define Each Trial Failure Surface.
Following Are Displayed The Ten Most Critical Of The Trial
Failure Surfaces Evaluated. They Are
Ordered - Most Critical First.
* * Safety Factors Are Calculated By The Modified Bishop Method * *
Total Number of Trial Surfaces Attempted = 100
Number of Trial Surfaces With Valid FS = 100
Statistical Data On All Valid FS Values:
FS Max = 3.442 FS Min = 1.200 FS Ave = 1.957
Standard Deviation = 0.692 Coefficient of Variation = 35.38 %
Failure Surface Specified By 22 Coordinate Points

Point No.	X-Surf (ft)	Y-Surf (ft)
1	200.004	170.031
2	203.335	173.759
3	206.573	177.570
4	209.715	181.459
5	212.759	185.426
6	215.703	189.467
7	218.546	193.580
8	221.286	197.762
9	223.921	202.011
10	226.450	206.325
11	228.870	210.700
12	231.181	215.134
13	233.381	219.624
14	235.469	224.167
15	237.443	228.761
16	239.302	233.403
17	241.045	238.089
18	242.671	242.817
19	244.179	247.585
20	245.568	252.388
21	246.836	257.224
22	247.275	259.082

Circle Center At X = 51.960 ; Y = 305.688 ; and Radius = 200.799
Factor of Safety
*** 1.200 ***
Individual data on the 22 slices

Slice No.	Width (ft)	Weight (lbs)	Water Force Top (lbs)	Water Force Bot (lbs)	Tie Force Norm (lbs)	Tie Force Tan (lbs)	Earthquake Force Hor (lbs)	Earthquake Force Ver (lbs)	Surcharge Load (lbs)
1	3.3	4248.7	0.0	0.0	1724.	0.	0.0	0.0	0.0
2	3.2	12234.9	0.0	0.0	2257.	2504.	0.0	0.0	0.0
3	3.1	19440.8	0.0	0.0	2304.	1251.	0.0	0.0	0.0
4	0.3	2131.8	0.0	0.0	245.	136.	0.0	0.0	0.0
5	2.8	20490.5	0.0	0.0	2399.	1465.	0.0	0.0	0.0
6	2.9	20900.8	0.0	0.0	2754.	1510.	0.0	0.0	0.0
7	2.8	19162.9	0.0	0.0	2920.	1595.	0.0	0.0	0.0
8	2.7	17450.5	0.0	0.0	3043.	1576.	0.0	0.0	0.0
9	2.6	15770.6	0.0	0.0	3151.	1635.	0.0	0.0	0.0
10	2.5	14130.0	0.0	0.0	3237.	1626.	0.0	0.0	0.0
11	2.4	12535.8	0.0	0.0	3296.	1679.	0.0	0.0	0.0
12	2.3	10994.8	0.0	0.0	3324.	1668.	0.0	0.0	0.0
13	2.2	9513.8	0.0	0.0	3312.	1725.	0.0	0.0	0.0
14	2.1	8099.3	0.0	0.0	3250.	1724.	0.0	0.0	0.0
15	2.0	6758.0	0.0	0.0	3131.	1773.	0.0	0.0	0.0
16	1.9	5496.1	0.0	0.0	2950.	1793.	0.0	0.0	0.0
17	1.7	4319.9	0.0	0.0	2712.	1830.	0.0	0.0	0.0
18	1.6	3235.4	0.0	0.0	2434.	1876.	0.0	0.0	0.0

19	1.5	2248.5	0.0	0.0	2138.	1873.	0.0	0.0	0.0
20	1.4	1364.6	0.0	0.0	1849.	1822.	0.0	0.0	0.0
21	1.3	589.4	0.0	0.0	1584.	1736.	0.0	0.0	0.0
22	0.4	44.5	0.0	0.0	547.	630.	0.0	0.0	0.0

Failure Surface Specified By 22 Coordinate Points

Point No.	X-Surf (ft)	Y-Surf (ft)
1	200.045	170.337
2	203.367	174.074
3	206.593	177.895
4	209.720	181.796
5	212.747	185.775
6	215.671	189.831
7	218.492	193.960
8	221.206	198.159
9	223.812	202.426
10	226.308	206.758
11	228.694	211.153
12	230.966	215.606
13	233.124	220.117
14	235.167	224.680
15	237.093	229.295
16	238.900	233.956
17	240.588	238.663
18	242.156	243.411
19	243.602	248.197
20	244.925	253.019
21	246.126	257.873
22	246.312	258.718

Circle Center At X = 55.219 ; Y = 302.454 ; and Radius = 196.034

Factor of Safety

*** 1.206 ***

Failure Surface Specified By 23 Coordinate Points

Point No.	X-Surf (ft)	Y-Surf (ft)
1	200.039	170.291
2	203.053	174.281
3	206.018	178.307
4	208.934	182.368
5	211.801	186.465
6	214.617	190.596
7	217.383	194.762
8	220.098	198.960
9	222.762	203.192
10	225.374	207.455
11	227.934	211.750
12	230.441	216.076
13	232.896	220.432
14	235.297	224.817
15	237.645	229.232
16	239.940	233.674
17	242.179	238.145
18	244.365	242.642
19	246.495	247.165
20	248.570	251.714
21	250.590	256.288
22	252.554	260.886
23	252.647	261.111

Circle Center At X = -126.321 ; Y = 419.991 ; and Radius = 410.927

Factor of Safety

*** 1.218 ***

Failure Surface Specified By 23 Coordinate Points

Point No.	X-Surf (ft)	Y-Surf (ft)
1	200.027	170.199
2	202.887	174.300
3	205.714	178.424

4	208.508	182.571
5	211.267	186.741
6	213.992	190.933
7	216.682	195.148
8	219.338	199.384
9	221.959	203.642
10	224.545	207.921
11	227.096	212.222
12	229.611	216.543
13	232.091	220.885
14	234.535	225.246
15	236.944	229.628
16	239.317	234.029
17	241.653	238.450
18	243.953	242.889
19	246.217	247.348
20	248.444	251.824
21	250.634	256.319
22	252.787	260.831
23	252.976	261.235

Circle Center At X = -298.987 ; Y = 521.370 ; and Radius = 610.193

Factor of Safety

*** 1.232 ***

Failure Surface Specified By 25 Coordinate Points

Point No.	X-Surf (ft)	Y-Surf (ft)
1	200.000	170.000
2	203.560	173.511
3	207.061	177.081
4	210.503	180.708
5	213.884	184.391
6	217.205	188.129
7	220.463	191.921
8	223.659	195.767
9	226.790	199.665
10	229.857	203.614
11	232.858	207.613
12	235.793	211.661
13	238.661	215.757
14	241.461	219.899
15	244.192	224.087
16	246.854	228.320
17	249.446	232.596
18	251.966	236.914
19	254.416	241.273
20	256.793	245.672
21	259.097	250.109
22	261.328	254.584
23	263.484	259.095
24	265.566	263.641
25	266.801	266.458

Circle Center At X = -11.167 ; Y = 387.632 ; and Radius = 303.241

Factor of Safety

*** 1.239 ***

Failure Surface Specified By 25 Coordinate Points

Point No.	X-Surf (ft)	Y-Surf (ft)
1	200.090	170.675
2	203.233	174.563
3	206.355	178.469
4	209.454	182.393
5	212.530	186.334
6	215.584	190.293
7	218.616	194.269
8	221.625	198.263
9	224.611	202.273
10	227.574	206.300

11	230.514	210.345
12	233.431	214.405
13	236.325	218.483
14	239.196	222.577
15	242.044	226.687
16	244.867	230.813
17	247.668	234.955
18	250.445	239.113
19	253.198	243.287
20	255.927	247.476
21	258.633	251.681
22	261.314	255.901
23	263.971	260.137
24	266.604	264.387
25	268.193	266.984

Circle Center At X = -481.478 ; Y = 724.896 ; and Radius = 878.462

Factor of Safety

*** 1.245 ***

Failure Surface Specified By 25 Coordinate Points

Point No.	X-Surf (ft)	Y-Surf (ft)
1	200.029	170.215
2	202.953	174.271
3	205.872	178.330
4	208.787	182.392
5	211.698	186.457
6	214.604	190.526
7	217.506	194.598
8	220.403	198.673
9	223.296	202.751
10	226.184	206.833
11	229.068	210.917
12	231.947	215.005
13	234.822	219.096
14	237.693	223.190
15	240.558	227.287
16	243.420	231.387
17	246.277	235.491
18	249.129	239.597
19	251.977	243.707
20	254.820	247.820
21	257.659	251.936
22	260.494	256.055
23	263.323	260.177
24	266.149	264.302
25	267.907	266.876

Circle Center At X = -3496.030 ; Y = 2837.900 ; and Radius = 4558.223

Factor of Safety

*** 1.250 ***

Failure Surface Specified By 26 Coordinate Points

Point No.	X-Surf (ft)	Y-Surf (ft)
1	200.025	170.184
2	203.593	173.686
3	207.115	177.235
4	210.591	180.830
5	214.019	184.469
6	217.400	188.153
7	220.732	191.881
8	224.014	195.653
9	227.248	199.467
10	230.431	203.322
11	233.563	207.220
12	236.644	211.157
13	239.674	215.135
14	242.651	219.152
15	245.575	223.208

16	248.447	227.301
17	251.264	231.432
18	254.027	235.599
19	256.735	239.802
20	259.388	244.040
21	261.986	248.313
22	264.527	252.619
23	267.012	256.957
24	269.440	261.328
25	271.810	265.731
26	273.513	268.994

Circle Center At X = -65.806 ; Y = 444.623 ; and Radius = 382.077

Factor of Safety

*** 1.259 ***

Failure Surface Specified By 22 Coordinate Points

Point No.	X-Surf (ft)	Y-Surf (ft)
1	200.094	170.705
2	202.854	174.875
3	205.573	179.070
4	208.253	183.292
5	210.893	187.538
6	213.492	191.810
7	216.050	196.106
8	218.567	200.426
9	221.043	204.770
10	223.477	209.137
11	225.870	213.527
12	228.221	217.940
13	230.530	222.375
14	232.796	226.832
15	235.020	231.310
16	237.202	235.809
17	239.340	240.329
18	241.435	244.869
19	243.487	249.428
20	245.496	254.007
21	247.461	258.605
22	247.731	259.254

Circle Center At X = -236.389 ; Y = 462.609 ; and Radius = 525.096

Factor of Safety

*** 1.260 ***

Failure Surface Specified By 26 Coordinate Points

Point No.	X-Surf (ft)	Y-Surf (ft)
1	200.027	170.199
2	203.558	173.739
3	207.048	177.319
4	210.497	180.939
5	213.904	184.599
6	217.267	188.299
7	220.588	192.037
8	223.865	195.813
9	227.099	199.627
10	230.287	203.478
11	233.431	207.366
12	236.530	211.290
13	239.583	215.249
14	242.590	219.244
15	245.551	223.273
16	248.465	227.337
17	251.331	231.434
18	254.150	235.563
19	256.920	239.725
20	259.643	243.919
21	262.316	248.144
22	264.941	252.400

```
   23         267.516      256.686
   24         270.041      261.002
   25         272.516      265.346
   26         274.810      269.484
Circle Center At X =  -103.686 ; Y =   476.769 ; and Radius =   431.539
       Factor of Safety
      ***    1.260   ***
          **** END OF GSTABL7 OUTPUT ****
```

BOND PIT SLOPE STABILIZATION
SANTIAGO BASIN SLOPE REMEDIATION PROJECT
ORANGE COUNTY, CALIFORNIA

Prepared for
ORANGE COUNTY WATER DISTRICT

Project No. 1664
February 5, 1996

GROUP DELTA CONSULTANTS

Geotechnical Engineering
Geology
Hydrogeology
Coastal Engineering
Hydrology
Hydraulics
Environmental Engineering

Project No. 1664
February 5, 1996

Mr. Steven R. Conklin
ORANGE COUNTY WATER DISTRICT
P.O. Box 8300
Fountain Valley, California 92728-8300

BOND PIT SLOPE STABILIZATION
SANTIAGO BASIN SLOPE REMEDIATION PROJECT
ORANGE COUNTY, CALIFORNIA

Dear Mr. Conklin:

Group Delta Consultants, Inc. (GDC) is pleased to submit the results of our preliminary design studies for stabilizing the southwesterly slope within Bond Pit adjacent to Bond Avenue in the City of Orange, California. The accompanying report provides the basis of design, along with preliminary cost estimates for the construction of a 15-foot-high tied-back free-form structural shotcrete wall intended for stabilization of Bond Avenue.

We appreciate the opportunity to be of service and trust this information meets your needs. If you have any questions or require additional information, please give us a call.

for GROUP DELTA CONSULTANTS, INC.

Very truly yours,

Walter F. Crampton, Principal Engineer
R.C.E. 23792, R.G.E. 245

WFC/JMK/jc
Attachments

(4) Addressee
(1) Mr. Allan Flowers, Orange County Water District
(1) Dr. David Huntley

SAN DIEGO 4455 Murphy Canyon Road, Suite 100 ▲ San Diego, California 92123-4379 ▲ (619) 573-1777 *voice* (619) 573-0069 *fax*
IRVINE 3347 Michelson Drive, Suite 390 ▲ Irvine, California 92715-1692 ▲ (714) 975-7474 *voice* (714) 975-7390 *fax*

ORANGE COUNTY WATER DISTRICT
Project No. 1664

February 5, 1996

TABLE OF CONTENTS

BOND PIT SLOPE STABILIZATION
SANTIAGO BASIN SLOPE REMEDIATION PROJECT
ORANGE COUNTY, CALIFORNIA

1 INTRODUCTION

This report presents our preliminary design for stabilizing the southwesterly slope within Bond Pit adjacent to Bond Avenue in the City of Orange, California. In addition to increasing the stability of Bond Avenue, the design concept is intended to avoid the potential for degradation of the hydraulic conductivity of the pit slopes.

As a result of our last meeting with Orange County Water District (OCWD) staff on November 28, 1995, the design concept for stabilizing the southwesterly slope has changed substantially from that discussed in our September 1, 1995, concept design report, and more recently in our October 25, 1995, proposal for a scoping program optimizing slope stabilization for Bond Pit. In summary, it was agreed to pursue a concept design for a tied-back wall constructed above elevation 290 feet. Since the majority of the wall would be above the working pool elevation, there would be little to no impact on the basin's recharge characteristics. It was further agreed that the tied-back wall would be constructed along the OCWD's existing fence alignment adjacent to Bond Avenue, and that the elevation of the perimeter roadway in this area would be lowered to the base of the wall to facilitate its construction and further improve embankment stability. An added feature of this concept is that, once again, a rather large sacrificial perimeter roadway/buffer will exist adjacent to Bond Avenue, and any ongoing surficial sloughage will continue to naturally remove any caking that might otherwise be degrading the hydraulic conductivity of the aquifer.

This report also addresses, at the preliminary level, seismic design considerations, along with the reduction in embankment stability due to rapid drawdown associated with the U.S. Army Corps of Engineers (USCOE) plan for use of the Santiago Basin as a flood detention facility.

2 BACKGROUND INFORMATION

Numerous geotechnical studies have been performed to address the stability of the existing pit slopes. These studies date back to 1985 when Woodward-Clyde Consultants (WCC) addressed both static and dynamic stability, and recommended various remedial measures to improve the stability of the existing slopes. Previous geotechnical studies have addressed variations in the lithology and strength parameters of the materials that are exposed in the slopes, and the effects of infilling and drawdown on static and dynamic stability. Considerable information also exists regarding both site-specific and regional geology, along with the hydrogeology of the Santiago Creek watershed. A list of the various documents reviewed as part of this project is presented in the References section at the end of this report. A summary of the soil strengths used in previous stability studies is presented in Table 1.

Associated with previous geotechnical investigative work, collateral studies have been done to determine the potential for colloidal materials to clog the aquifer under various proposed slope stabilization alternatives. As we understand, concern by OCWD staff over aquifer degradation and post-construction colloidal clogging has been a key factor in assessing the viability of the various stabilization alternatives, and, to date, there has been a hesitation on the part of OCWD staff to initiate any significant slope stabilization program to improve embankment stability along Bond Avenue. In our discussions with OCWD staff on November 28, 1995, it was agreed that GDC's scope of work would be modified from that generally described in our October 25, 1995, proposal to specifically developing the preliminary design for a tied-back wall constructed above the working pool elevation. This scope of work modification was adopted principally to avoid the potential for degradation of the hydraulic conductivity of the pit slopes while effecting an immediate stabilization of Bond Avenue.

3 GENERAL DESIGN CRITERIA

The principal issues controlling the design of this project relate to the stability of the relatively steep slopes within Bond Pit adjacent to Bond Avenue, tempered by the concern for degradation of the aquifer. Prior to OCWD acquiring the abandoned gravel pits, slopes adjacent the central portion of Bond Avenue were in excess of 100 feet in height, with an average inclination on the order of 0.8:1.0, or approximately 50 degrees. In view of the very real concern over seismic stability, WCC recommended the construction of a seismic stability buttress (Woodward-Clyde 1985), which was constructed in 1989, extending up to elevation 230 feet and placed at an inclination of 2:1. After completion of the buttress, a 75±-foot-tall, 0.8:1.0 slope remained, and the pits were subsequently filled with water in early 1990.

WCC continued to monitor failures along the Bond Pit slope through April 1994, recording 23 feet of total slope-top retreat during the measurement period (Woodward-Clyde 1995). Slope-top retreat reported by WCC locally ranged from 2½ feet per year to in excess of 5 feet per year.

During our field investigative work in August 1995, limited bathymetric surveys were conducted northerly of Bond Avenue between 500 and 700 feet easterly of Prospect Street. These bathymetric surveys indicated that surficial sloughing had overrun and buried the seismic stability buttress placed in 1989, resulting in a submerged talus deposit having an inclination on the order of 38± degrees, with the above-water slopes ranging from 50 degrees to locally approaching 70 degrees. The water table at the time of our bathymetric surveys was approximately 274 feet.

WCC concluded that the surficial slope failures, and the associated slope-top retreat, were the result of both rapid drawdown and erosion from wind-driven waves generated on the basin. In reviewing the available data, we concur with WCC's conclusions, recognizing that water contained within the aquifer has a significant impact on the surficial stability of these relatively steep slopes when the water level in the pond recedes. The true angle of repose for these alluvial deposits is likely on the order of 38 degrees, which we measured during our field investigative work below the static pond level. The in-situ soil strength without water previously sustained a 50 degree slope; however, this is likely due to the presence of fine-grained and otherwise cemented zones within the aquifer, and not indicative of the effective angle of internal friction of the alluvial deposits. Various design criteria are discussed in greater detail in the following paragraphs.

3.1 Soil Strength Parameters

Relatively high soil strengths ($\phi = 45$ degrees, $c = 0$ psf, and $\gamma_t = 140$ pcf) were utilized in all of our stability analyses. These soil strengths were developed by WCC (1985), in part by back-calculating the soil strengths necessary to sustain the existing relatively steep slopes. These values represent a reasonable approximation of the upper-bound strength of gravelly materials. In general, this is a reasonable and prudent approach, and the likelihood of obtaining much higher soil strengths through a costly field investigation program is not likely. WCC used even higher soil strengths ($\phi = 50$ degrees, $c = 0$ psf, and $\gamma_t = 140$ pcf) for surficial stability calculations. However, again, this was predicated on back-calculating the stability of the existing slopes, which locally are quite steep. In our previous discussions with OCWD staff, it was agreed that the cost of an extensive geotechnical field investigation program to refine soil strength parameters may not yield substantially better data, especially in view of the relatively high assumed strengths necessary to numerically demonstrate stability of the relatively high, and locally near-vertical, slopes.

3.2 Generalized Geologic Section

During our field investigative work in August 1995, we conducted geologic mapping along two profiles down to the then-current water elevation (273.7 feet) within the limits of our original study area. Generalized geologic cross sections along both profiles were prepared and included in our September 1, 1995, concept design report. A more detailed description of the variations in stratigraphy of the Quaternary alluvial fan exposed along the Bond Avenue slope is presented in the 1985 WCC report, and elsewhere in the other referenced documents. In all of our analyses, we have utilized the generalized geologic cross section, B-B, reported in our September 1, 1995, report, and have superimposed on this cross section the seismic stability buttress reported by WCC in their June 1995 report.

3.3 Seismic Hazards and Site Seismicity

Potential seismic hazards for any site include ground rupture, slope instability, subsidence, liquefaction, seismic compaction/settlement, and ground shaking. Our scope of work did not include an assessment of seismic hazards other than ground shaking.

The site is located in a seismically-active area and thus ground shaking due to nearby and distant earthquakes should be anticipated during the life of the recharge basin. The closest active fault is the Whittier Elsinore fault located about 8 miles to the northeast of the site. This fault is capable of generating a magnitude 7.5 earthquake at distances of 8 miles from the site.

We performed a deterministic seismic hazard analysis using the computer program EQFAULT (Blake, 1989). The program computes the peak horizontal ground acceleration from the "maximum credible" and "maximum probable" (100 year return period) earthquakes on each of the faults found within a user specified radius. The computation of the peak acceleration is based on the closest distance between the site and each digitized fault and a user specified attenuation relationship. For our analysis, we used a 75-mile radius and the average of three attenuation relations (Joyner and Boore 1982; Sadigh 1987; Campbell 1993). Based on our deterministic analysis, the peak horizontal ground accelerations associated with the maximum probable and maximum credible earthquakes are 0.17g and 0.37g, respectively.

We also performed a probabilistic seismic hazard analysis using the computer program FRISK89 (Blake, 1989). We again considered faults within a 75 mile radius from the site and used the three attenuation relationships by Joyner and Boore, Sadigh, and Campbell. We also included the effects of magnitude weighting factor (Idriss, 1985). Probabilistic seismic hazard evaluation involves obtaining, through a formal mathematical process, the level of ground motion parameter that has a selected probability of being exceeded during a specified time interval. A probabilistic seismic hazard evaluation at a site due to a particular source involves combining three probability functions:

- The recurrence rate which defines the probability that an earthquake of a particular magnitude will occur on this source during a specified time interval. This probability is usually expressed as mean number of earthquakes with a given magnitude per year on the given source.

- The probability that the rupture surface is a specified distance from the site. This probability is obtained from fault geometry and the magnitude-rupture length relationships.

- The probability that the ground motions from an earthquake of a given magnitude occurring at a certain distance from the site will exceed a specified level. This probability is based on the selected attenuation relationship.

The probabilistic approach incorporates the contributions from all faults and considers the likelihood of the occurrence of earthquakes at any point on the fault. It also incorporates the contributions from various magnitude earthquakes up to and including the maximum credible. This approach is described by Idriss (1985).

The results of these analyses are summarized in Figure 1 and are presented below. These peak site accelerations are approximately 10 percent higher than those reported by WCC in 1985.

Peak Horizontal Ground Acceleration	Average Return Period (years)	Probability of Exceedance (P_e) in 50 Years (%)
0.16	72	50
0.18	100	39
0.23	200	22
0.31	475	10
0.38	1000	5

For pseudo-static stability analyses, the selected seismic coefficient generally ranges between 1/3 to 1/2 of the maximum peak acceleration (U.S.A.C.E, 1984). For our pseudo-static stability analyses, we used a horizontal acceleration of 0.15g, corresponding to one-half of the value of peak ground acceleration associated with a 10% probability of exceedance in 50 years. A value of 0.15g as a pseudo-static seismic coefficient is also generally accepted by local building departments for southern California sites.

3.4 Rapid Drawdown

Considerable discussion exists in the previous documents regarding the impact of rapid drawdown and the significant reduction in surficial slope stability, which has likely contributed to recent slope failures noted throughout Santiago Basin. During our September 28, 1995, meeting with OCWD staff, considerable discussion was devoted to the impact of rapid drawdown and its effect on the stability of the proposed Terramesh® wall on top of a 2:1 slope extending up from the existing seismic stability buttress placed in 1989. We used Section B-B reported in our September 1, 1995, concept design report and utilized the soil strength parameters originally developed by WCC for deep-seated stability. We evaluated this slope geometry for a variety of phreatic surfaces, illustrating the impact of the rapid drawdown phreatic surface on surficial stability. In summary, gently inclined phreatic surfaces (very high isotropic aquifer transmissivity) resulted in static factors of safety in excess of 1.5, while a clogged aquifer precludes pore pressure dissipation and results in static factors of safety considerably below 1.0.

3.5 Wave-Induced Scour

An unobstructed 4,500±-foot-long fetch exists for the prevailing winds out of Santa Ana Canyon to develop wind-generated waves and the associated wave-induced erosion along the Bond Avenue slope. A sustained wind duration of less than one hour will develop 1-foot waves with 20-mile-per-hour winds, and 2-foot waves with 40-mile-per-hour winds (USCOE 1984). Similarly, 1-foot waves will displace 3-pound stones, and 2-foot waves will displace 25-pound stones (USCOE 1984). Wave-induced erosion is common to most shoreline environments, and a variety of stabilization measures are available to mitigate wave-induced erosion.

4 RECOMMENDED EMBANKMENT STABILIZATION

Recognizing the importance of maintaining the hydraulic conductivity of the recharge basin, we have developed a slope stabilization program that we believe fulfills the various desires of OCWD staff. In summary, we are recommending the construction of a 15-foot-high tied-back wall constructed along the OCWD's existing fence alignment adjacent to Bond Avenue. Additionally, we are recommending that the elevation of the perimeter roadway in this area be lowered to the base of the wall to both facilitate its construction and improve embankment stability. We recommend that the wall be designed with a 42-inch-tall structural parapet to accommodate a glancing impact from a 15,000-pound vehicle leaving the Bond Avenue travelway. We recommend that foundation pedestals be incorporated into the tied-back wall for support of new power poles for the Southern California Edison power line adjacent to the OCWD property line. A 6½-foot-tall masonry block privacy wall could also be constructed on top of the structural parapet, if desired, along with any new landscaping between the wall and the Bond Avenue travelway. A typical section depicting this design is shown on Figure 2.

In reviewing the 1-inch equals 100-foot scale topographic base map prepared by Robert J. Long & Associates (photo flight date 11/4/83), it appears that the area of most concern adjacent to Bond Avenue extends for a length of 1,500 to 1,700 feet. We have used a 1,500-foot project length in all of our cost estimates, and these values may be adjusted accordingly for different project lengths.

5 TECHNICAL BASIS OF DESIGN

5.1 Tied-Back Wall Design Criteria

We are recommending that the tied-back wall be designed to accommodate both earth pressures and vehicle loads. Due to the limited amount of anticipated wall movements, the design earth pressure should assume "at-rest" conditions, which, for the gravelly alluvial soils, is equal to an equivalent fluid pressure of approximately 41 pounds per cubic foot. An additional uniform pressure of 60 pounds per square foot should be added to simulate transient vehicular traffic adjacent the wall. We recommend that the parapet reinforcing, along with internal wall stability, be designed for a 15,000-pound vehicle striking the wall at 10 mph at a 30-degree angle to the wall. This condition further assumes that two-thirds of the impact energy is absorbed by the vehicle, and that a 20-foot-long section of wall resists the remaining energy. Impact loads in excess of this design will break the parapet, thereby preserving the integrity of the tied-back wall. We recommend that the wall be designed to resist a seismic design acceleration of 0.15g. Anchor capacity should be sized to accommodate the larger of the following two criteria:

- An overload factor of 1.5 x (earth pressure + vehicular load + vehicular impact load)
- 1.25 x (earth pressure + seismic increment)

Seismic loading criteria was based on a Mononabe-Okabe analysis (Seed and Whitman 1978) method. For a design acceleration of 0.15g, and a total unit weight of 140 pcf, we calculated an equivalent inverted seismic-induced fluid pressure of 10 pcf. This equivalent seismic-induced fluid pressure should be added as an additional inverted triangular load to the 41 pcf earth pressure.

5.2 Structural Shotcrete Wall

We recommend that the proposed wall utilize a free-form structural shotcrete skin that can be carved and colored to increase its natural appearance. We anticipate that the structural face would likely be 15- to 18-inches thick, and be constructed on a 1/4:1 inclination extending down from the existing OCWD fence line. The proposed site grading would develop a 1/4:1 temporary construction cut slope 15 feet in height at the OCWD fence line, into which two rows of tieback anchors would be installed, steel reinforcing placed, and the shotcrete surface applied directly to the construction cut slope. If construction-period instability dictates, the wall could be constructed in two vertical lifts.

5.3 Tieback Design Criteria

We recommend that the tiebacks used for restraining the proposed structural concrete skin utilize friction from straight-shaft cylindrical bored holes. Allowable anchor capacity is based on the surface area of the bonded anchor. We recommend using an allowable design shaft friction of 500 psf. All tiebacks should be proof-tested in the field in general accordance with the City of Los Angeles Code Requirements for Anchor Testing. Twenty-four hour creep tests should also be conducted on the first two anchors, and on an additional six anchors at approximately 200-foot horizontal intervals.

We recommend that the tiebacks be installed at a minimum batter of 5 on 1 (11.3 degrees below the horizontal), and possibly as steep as 4:1, with the design batter predicated on structural considerations and increased anchor capacity with increased overburden. We are recommending that tieback anchors utilize a non-shrink, low-slump (viscous) grout, with no subsequent post-grouting to minimize degradation of the hydraulic conductivity of the aquifer. The use of a non-shrink, low slump grout will not permeate the aquifer, and will have essentially no impact on hydraulic conductivity of the aquifer. The presence of gravels and cobbles within the alluvium will likely dictate the use of cased, bored holes. For cost estimating purposes, we have assumed that all tiebacks will be installed in cased holes.

5.4 Corrosion Considerations

The reinforced structural shotcrete skin can be made relatively durable by utilizing a rich concrete mix that produces a rather impermeable concrete skin. With a minimum of 4 inches of cover over the reinforcing steel, corrosion during the design life of the structure is quite low. The corrosion protection systems currently available for tiebacks are both excellent and dependable, and can essentially provide a guaranteed service life in excess of 100 years. We have previously transmitted under separate cover technical brochures from DYWIDAG Systems International (DSI), a tieback manufacturer whose product we typically specify. We recommend the use of their double-corrosion protection anchorage system, which we believe minimizes the potential for corrosion of the tendons during the design life of the structure.

5.5 Slope Stability Considerations

Slope stability considerations include the potential for both deep-seated and surficial instability, seismic-induced instability, and slope instability caused by rapid drawdown. As previously indicated, the entire 1,500±-foot slope adjacent to Bond Avenue is marginally stable with the original abandoned quarry slopes at an inclination of approximately 50 degrees. The introduction of water into the Santiago Basin has aggravated slope instability due to differential pore pressures as water levels recede, due to reduction in soil strengths of the fine-grained fraction with increase in soil moisture, and due to erosion from wind-driven waves. The potential for slope instability represents a very real concern; however, in our opinion, implementation of the proposed slope stabilization program produces a manageable level of risk that we believe should be acceptable to the OCWD. Risks associated with these slope stability considerations are described in greater detail in the following paragraphs.

5.5.1 **Surficial Stability**

Without the presence of water, relatively steep cohesionless slopes are most typically analyzed using an infinite-slope method, which, in its simplest terms, results in a safety factor of 1.0 when the slope inclination equals the angle of internal friction of the soil. As previously indicated, WCC originally recommended a soil friction angle of 50 degrees for surficial stability considerations, suggesting that a 50 degree slope would have a factor of safety of 1.0. The actual computed factor of safety is as follows:

$$\text{F.S.} = \tan^{-1} (\tan\phi/\tan\beta)$$

where: ϕ is the angle of internal friction of the soil
β is the inclination of the slope

GROUP
DELTA
CONSULTANTS

Considering surficial stability calculations, small reductions in slope angle, although producing only small increases in factor of safety, render the slope essentially stable unless subjected to an external force such as seismic shaking, rapid drawdown, or seepage. The proposed slope stabilization project provides a sacrificial bench ranging from 40 to 60+ feet in width, thereby allowing a long-term eroded profile to flatten out to approximately 25 degrees prior to undermining the tied-back wall. This results in a computed surficial factor of safety (without seepage) of 2.1 when utilizing ϕ equal to 45 degrees.

5.5.2 **Deep-Seated Slope Stability**

As suggested in the previous section, stability calculations performed on hypothetical failure geometries approaching the inclination of the slope yield the same safety factors obtained from surficial stability analyses. In our evaluation of deep-seated or global slope stability, we forced failure geometries back to Bond Avenue, and examined hypothetical failure geometries exiting the ground surface from approximately 20 feet southerly of the existing slope top to the southerly edge of the east-bound Bond Avenue travelway. The most significant finding is that by lowering the slope top 15 feet, as proposed, there is a corresponding increase in global stability of approximately 15 to 30 percent, depending upon the hypothetical failure geometry analyzed. In general, for deep-seated slope failures propagating back to Bond Avenue, the existing static factor of safety is on the order of 1.5. By lowering grades northerly of the OCWD fence line by 15 feet, as currently proposed, the static factor of safety increases to approximately 1.7. The 15-foot tied-back wall further increases the static factor of safety for deep-seated slope failures to approximately 1.8.

5.5.3 **Seismic Stability**

Seismic shaking produces oscillating forces, which can have a significant impact on both structures and slopes. The conventional method for analyzing the effects of seismic loading on slope stability is by applying an additional horizontal load equal to the design acceleration, multiplied by the mass of the entire hypothetical failure geometry. This pseudo-static approach does not acknowledge the oscillatory loading condition, nor the relatively short duration of dynamic loading. The U.S. Army Corps of Engineers (USCOE 1985) has recommended that pseudo-static slope stability analyses utilize one-third to one-half of the peak ground surface acceleration to more accurately characterize the damage potential from seismically-induced slope failures. Moreover, with computed factors of safety on the order of 1.1, seismically-induced slope movements should be limited to permanent displacements on the order of a few inches, in addition to surface distortion, ravelling, and shallow sloughing. We have used a design peak ground acceleration of 0.31, reduced to 0.15 for pseudo-static analyses, and have computed safety factors on the order of 1.1. The presence of the tied-back wall increases seismic stability by 3 to 4 percent and, due to its design, should tend to reduce surface manifestations of seismically-induced slope deformation.

5.5.4 **Rapid Drawdown Considerations**

Slope instability associated with rapid drawdown represents the greatest potential for damage to Bond Avenue. As we have discussed in previous correspondence, the geometry of the phreatic surface has a significant impact upon slope stability, and the potential for clogging would suggest the potential for a steeply-sloping phreatic surface exiting the slope face. As with surficial stability considerations, as the slope becomes flatter, slope instability associated with rapid drawdown becomes less severe, and a series of progressive failures will eventually reach an equilibrium profile.

Working in association with Dr. David Huntley, we have performed limited parametric stability analyses of varying slopes to simulate surficial failures associated with rapid drawdown events, ultimately achieving a factor of safety of 1.05, with progressive failures encroaching to within 10 feet of the proposed tied-back wall. This analysis was based on the Corps of Engineers' proposal for using Santiago Basin as a flood detention facility as described in their memorandum of understanding between the Orange County Water District and Orange County Flood Control District (Corps of Engineers Flood Control Facility No. E08B01). This assumes a maximum static water level within the basin at elevation 293 feet, and a requirement to reduce the water level to elevation 270 feet in 12 hours to accommodate an expected storm event. This scenario likely represents the greatest risk to the stability of the Bond Pit slope and its associated impact to Bond Avenue. Under the guidelines of the above-referenced memorandum, this rapid drawdown scenario could conceivably occur several times a year, subjecting the Bond Avenue slope to continued surficial sloughing, which would likely eventually encroach upon the base of the proposed tied-back wall.

As indicated in our previous reports and other correspondence, we have previously considered the use of either Gabion Walls or Terramesh® walls as an economical retaining wall system. However, if subjected to undermining from slope failures from repeated rapid drawdown events, these gravity-type structures could become undermined and rapidly fail, compromising Bond Avenue. The design of the currently-proposed tied-back wall is less sensitive to undermining and provides better protection for Bond Avenue.

We have previously discussed the need for an accurate characterization of a design phreatic surface, and remain of the opinion that if OCWD agrees to the Corps of Engineers' flood control proposal, OCWD should attempt to better define the likely variations in the phreatic surface. Due to the significance of the Corps' proposal, the OCWD may wish to encourage the Corps to conduct the necessary hydrogeologic investigative work, or at least fund the necessary work that could be completed by the OCWD's Hydrogeology Department.

We have also reanalyzed rapid drawdown conditions for a maximum static pool elevation of 288 feet in lieu of 293 feet. Limiting the maximum pool elevation to 288 feet results in an increase of 5 percent in the factor of safety for surficial stability under rapid drawdown conditions, as compared to a pool elevation of 293 feet. Other alternatives for increasing surficial stability include the construction of a higher tied-back wall, encroaching closer onto Bond Avenue, or, possibly best case, eliminating Santiago Basin from the Corps' proposed flood control project.

5.6 Wave Protection

Protection from erosion due to wind-driven waves can most easily be provided by installing an erosion control blanket such as a Maccaferri Reno Mattress, or a cellular concrete mat such as Armorflex. Product literature for both of these products has been previously submitted by WCC (1995). For an erosion control blanket to be effective, it must extend a short distance below the pool elevation during the generation of wind-driven waves. If, after completion of this project, the design pool elevation were maintained at say elevation 288 feet, an erosion control blanket could economically be placed down to elevation 285 feet, thereby essentially eliminating erosion from wind-driven waves. If it is practical to maintain the pool elevation near 288 feet, it may be desirable to grade a 3:1 slope 25 to 30 feet out from the base of the tied-back wall, and place the erosion control blanket on the slope extending down to elevation 285± feet. This is also advantageous in that it locally improves the stability of the top-of-slope, and acknowledges that with any rapid drawdown, the slope will quickly recede to this inclination anyway. This also precludes the need to provide any special anchorage provisions for the erosion control blanket, which would otherwise be necessary for steeper slopes. Moreover, with a relatively simple anchorage system, considerable erosion can be accommodated under the toe of the erosion control blanket without quickly compromising the blanket.

Recognizing that OCWD staff does not wish to propose any improvements below elevation 290 feet, an erosion control blanket could also be anchored from the lowered perimeter roadway. However, this solution would likely be somewhat more expensive and more susceptible to subsequent failure. If desired, simply allowing some additional erosion will tend to flatten out the slope, eventually achieving a flatter slope, at which time the erosion control blanket could be installed. As an alternate, a detached floating breakwater could also be constructed for a similar cost

An important item to remember is that the current erosion rate is on the order of 2½ to 5 feet per year, which could increase with repeated rapid drawdown events contemplated by the Corps of Engineers. Some consideration should be given to preserving the minimum 40-foot-wide sacrificial bench, since erosion will continue to encroach upon the proposed tied-back wall.

6 ESTIMATED TOTAL PROJECT COST

Estimated costs for construction of the proposed improvements described herein were developed in dollars per cubic yard for total in-place cost for the reinforced structural shotcrete skin; in dollars per facial square foot for the privacy wall and erosion control blanket; as a unit price for each tied-back anchor, including installation and prestressing; and in dollars per cubic yard for grading. It should be noted that these costs are based on manufacturer's suggested prices and present contractor's average installation cost. These costs could vary somewhat depending on availability, suppliers, and bidding costs. Cost estimates include installation of all elements of the proposed plan, plus the required grading for a 1,500-foot section along Bond Avenue. Total estimated costs can be proportioned based on actual proposed project length. A breakdown of total estimated project costs is provided in Table 2. Unit costs used in our estimate of construction costs are listed below:

ESTIMATED UNIT COSTS

Item	Unit Cost
Site Grading with On-Site Disposal	$3/cu.yd.
Site Grading with Off-Site Disposal	$10/cu.yd.[(1)]
Reinforced Shotcrete Wall, Carved & Colored	$500/cu.yd.
Cased Tieback Anchors, 80± feet long, 50-kip capacity	$2500/ea.
6½-foot-tall Reinforced Concrete Masonry Privacy Wall	$5/sq.ft.
Mounting Pedestal to Accommodate Power Poles	$1000/ea.
Erosion Control Blanket	$2/sq.ft.

(1) Cost for off-site disposal may vary greatly depending upon available disposal areas. A total of approximately 50,000 cubic yards of material will be required for removal and, if off-site disposal is required, a staging area would likely be set up at the corner of Bond Avenue and Prospect Street, where trucks could be loaded for subsequent off-site disposal. As these materials comprise relatively clean alluvial deposits, we would, at a minimum suggest that these materials be placed in the Santa Ana River, or wherever an economical disposal location would be available.

7 LIMITATIONS

The sketches and recommendations provided in this report are preliminary in nature. Detailed plans and specifications must be prepared, and must be approved by the permitting agency before construction.

Geotechnical Engineering and the Earth Sciences are characterized by uncertainty. Professional judgements presented herein are based partly on our evaluation of the technical information gathered, partly on our understanding of the proposed construction, and partly on our general experience.

Considerable geotechnical work has previously been conducted to evaluate the stability of the perimeter slopes surrounding the gravel pits that comprise Santiago Basin. These previous geotechnical studies have included detailed geologic mapping, test borings, and laboratory testing to characterize the subsurface conditions and to develop soil strengths for use in evaluating the stability of the perimeter slopes. A geotechnical investigation addressing the overall stability of the Bond Pit slope is beyond the scope of our work. Such an investigation would cost significantly more than the cost of the scope of work performed for this study. It is understood that GDC is relying upon geotechnical studies conducted by other consultants on behalf of the OCWD. Although we performed limited geologic mapping of the existing natural exposures, we are relying upon the geotechnical information provided by the OCWD's previous geotechnical consultants for our geotechnical characterization of the project slope.

REFERENCES

1. Blake, T. F., July 1989, EQFAULT -- A computer program for the deterministic prediction of peak horizontal acceleration from digitized California faults.

2. Blake, T. F., July 1989, FRISK89 -- A computer program for the probabilistic estimation of seismic hazard using faults as earthquake sources.

3. Bolton, Malcolm, 1979, A guide to soil mechanics: John Wiley & Sons, New York.

4. Environmental Management Agency, April 1993, Budgetary report for utilization of Strawberry Field east of Hewes Avenue in Santiago Basin (Blue Diamond and Bond Pits) for slope stabilization.

5. Geosoils, Inc., February 1992, Geotechnical evaluation of the potential impact of drilled caissons (soldier piles) on the permeability of slopes within Bond Pit, Santa Ana Mainstem project, County of Orange, California.

6. Geosoils, Inc., May 1992, Material evaluation, Strawberry Field, Santa Ana Mainstem project, City of Orange, California.

7. Geosoils, Inc., January 1991, Slope evaluation, Blue Diamond and Bond Pits, Santiago Creek (Flood Control Facility No. E08) County of Orange, California.

8. Group Delta Consultants, Inc., December 11, 1995, Tied-back wall alternate, Bond Pit embankment stabilization, Santiago Basin slope remediation project, Orange County, California.

9. Group Delta Consultants, Inc., October 25, 1995, Proposal for engineering services, scoping program optimizing slope stabilization, Bond Pit in Santiago Basin, Orange County, California.

10. Group Delta Consultants, Inc., September 1, 1995, concept design slope stabilization test program, Bond Pit in Santiago Basin, Orange, California.

REFERENCES
- continued -

11. Herndon, Roy L., 1992, Hydrogeology of the Orange County groundwater basin - an overview, in The Regressive Pleistocene Shoreline, Coastal Southern California, Annual Field Trip Guide Book No. 20: South Coast Geological Society, Inc., p. 237-259.

12. Idriss, I.M., 1985, Evaluating seismic risk in engineering practice, in Proceedings of the 11th International Conference on Soil Mechanics and Foundation Engineering, San Francisco, Vol. 1, 255-320.

13. Joyner, W.B. and Boore, D.M., 1982, Prediction of earthquake response spectra, in Proceedings, 51st Annual Convention of the Structural Engineers of California, also USGS Open-File Report 82-977, 16 pp.

14. Lung, Robert J., & Associates, 11/4/83 (date of photography), 1-inch equals 100-foot scale photogrammetrically-prepared topographic base map.

15. Orange County Water District, October 1991, Santiago Creek recharge project, Phase I, hydrogeologic investigation.

16. Orange, County of, Environmental Management Agency, Public Works, May 22, 1992, Memorandum of understanding between the Orange County Water District and Orange County Flood Control District.

17. U.S. Army Corps of Engineers, 1984, Shore protection manual, U.S. Army Coastal Engineering Research Center, Fort Belvoir, Virginia, v. I and II.

18. U.S. Army Corps of Engineers, July 1984, Rationalizing the seismic coefficient method, Waterways Experiment Station.

19. Woodward-Clyde Consultants, June 1995, Reevaluation of slope distress and mitigation measures at Santiago Basin.

20. Woodward-Clyde Consultants, January 1993, Final report of evaluation of slope mitigation measures for the Santiago Basin, Orange County, California.

REFERENCES
- continued -

21. Woodward-Clyde Consultants, November 1989, Geotechnical investigation, proposed Prospect Avenue, buttress fill, Orange County Water District, Santiago Basin improvements.

22. Woodward-Clyde Consultants, August 1985, Geotechnical investigation Santiago Creek replenishment project site improvements, Orange County, California.

23. Miscellaneous water level data from Bond Pit and a variety of nearby wells maintained by the Orange County Water District.

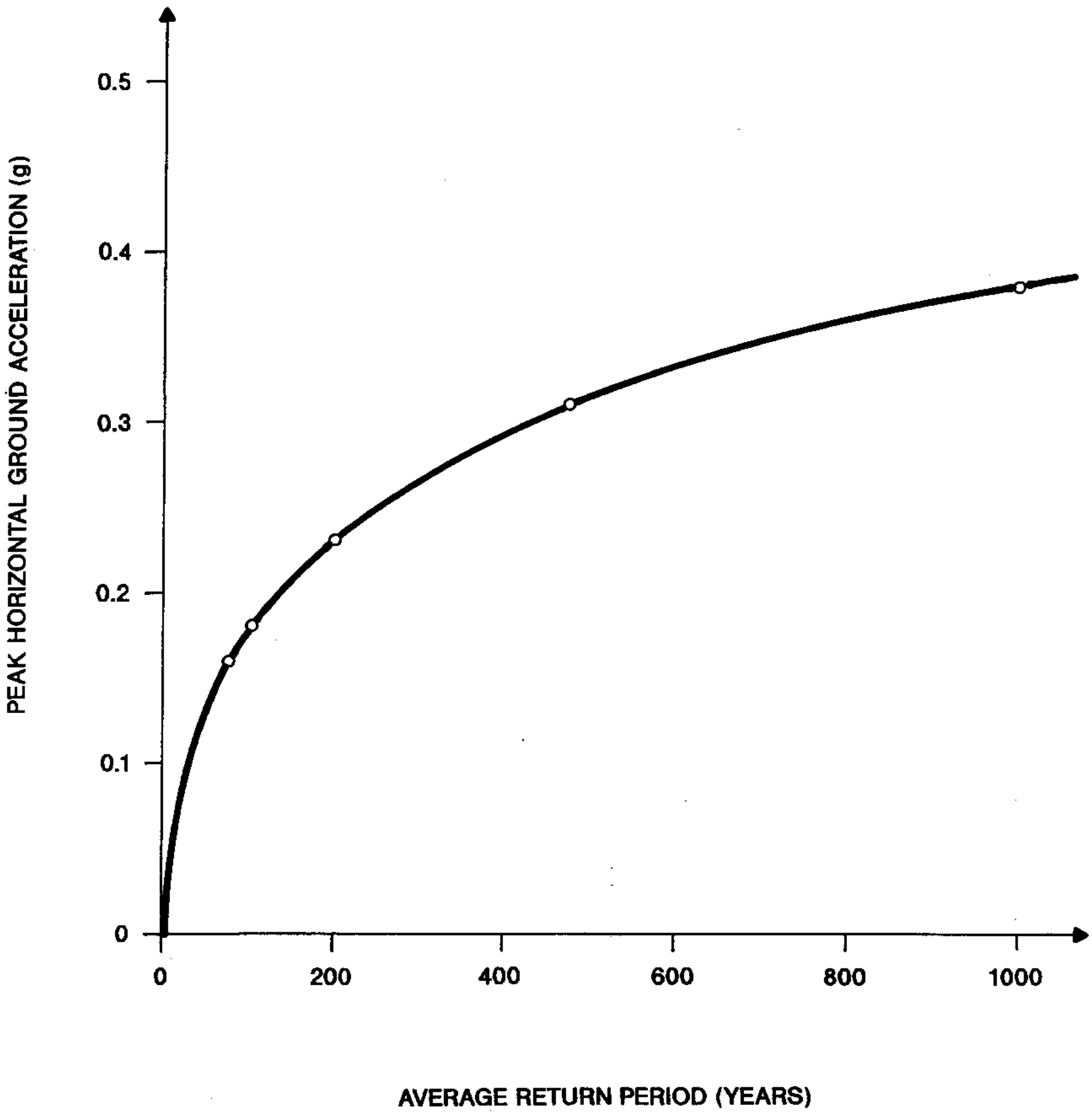
TABLE 1

SUMMARY OF SOIL STRENGTHS USED IN PREVIOUS STABILITY STUDIES

Source	Surficial	Deep Seated	Total Unit Wt
Woodward-Clyde Consultants (1985)	$\phi = 50°$ c = 0 psf	$\phi = 45°$ c = 0 psf	140 pcf
GeoSoils (1991)	N/A	$\phi = 40°$ c = 120 psf	140 pcf

TABLE 2
ESTIMATED PROJECT COSTS

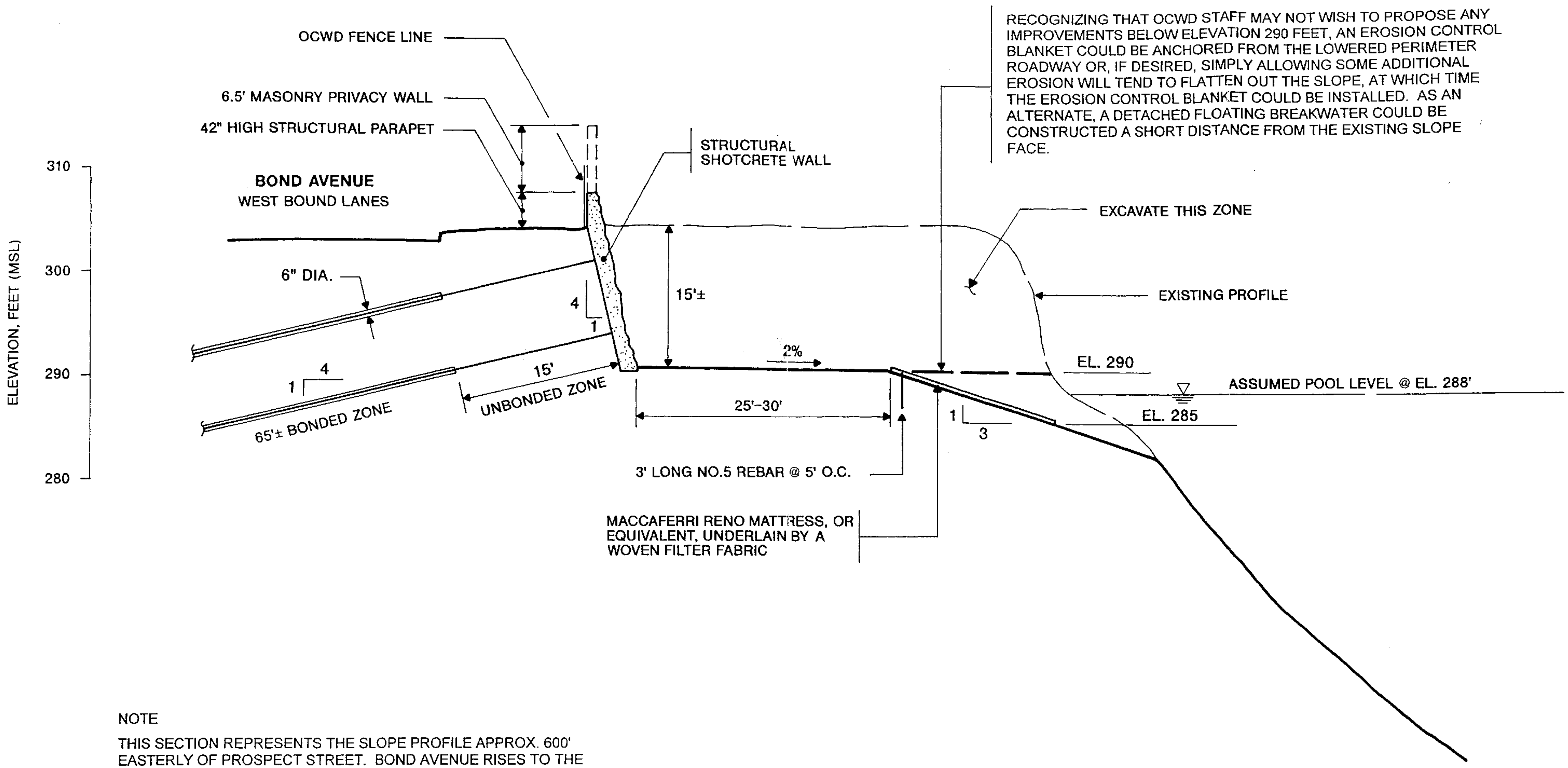
DEVELOP CONSTRUCTION BENCH			
40 to 60+ feet wide w/15-foot-high, 1/4:1 construction backcut			
50,000 cu.yd. @ $10/cu.yd.		$	500,000
If On-Site Disposal is Possible			
50,000 cu.yd. @ $3/cu.yd.	$150,000		
STRUCTURAL SHOTCRETE WALL			
Including Reinforcing Steel, Coloring & Carving			
1250 cu.yd. @ $500/cu.yd.		$	625,000
TIEBACK ANCHORS			
Install 80±-ft-long, 50-kip capacity cased tieback anchors, including post-tensioning and any necessary testing (2 rows of anchors with horizontal anchor spacing of 10 feet)			
300 anchors @ $2500/ea.		$	750,000
42-IN.-HIGH STRUCTURAL PARAPET			
292 cu.yd. @ $500/cu.yd.			146,000
6½-FT-TALL MASONRY PRIVACY WALL			
9750 sq.ft. @ $5/sq.ft.		$	48,750
INSTALLATION OF POWER POLE MOUNTING PLATFORMS BUILT INTO TIED-BACK WALL			
10 power poles @ $1000/ea.		$	10,000
EROSION CONTROL BLANKET			
Maccaferri 6" Reno Mattress, or equivalent, backed with a woven filter fabric and anchored with 3-foot-long No. 5 rebar at 5 feet on O.C. extending from elevation 285 to 291 feet, laid on a 3:1 slope			
9487 sq.ft. @ $2/sq.ft.		$	18,974
TOTAL CONSTRUCTION COST ESTIMATE		$2,098,724	



DESIGN PEAK SITE ACCELERATIONS

Project No. 1664	BOND PIT SLOPE STABILIZATION PROJECT	Figure 1

GROUP DELTA CONSULTANTS, INC.



NOTE

THIS SECTION REPRESENTS THE SLOPE PROFILE APPROX. 600' EASTERLY OF PROSPECT STREET. BOND AVENUE RISES TO THE EAST BY UPWARDS OF 12 FEET, AND THUS THE BASE OF THE WALL NEAR ITS EASTERN END WOULD BE AT EL. 302'.

PROFILE SCALE: 1"= 10' HORIZ. & VERT.

Project		Project No.	Figure
BOND PIT SLOPE STABILIZATION PROJECT	**TIED-BACK WALL SECTION**	1664	2

GROUP DELTA CONSULTANTS, INC.

203 North Golden Circle Drive
Santa Ana, CA 92705
(714) 835-6886
(213) 581-7164
Telex 68-3420

Woodward-Clyde Consultants

17 November 1988
Project No. 8840269A

Orange County Water District
10500 Ellis Avenue
P.O. Box 8300
Fountain Valley, California 92728-8300

Attention: Mr. John M. Chaufty, P.E.
Assistant to General Manager

SUBJECT: GEOTECHNICAL DESIGN AND SPECIFICATIONS FOR THE BOND PIT BUTTRESS WITH OPTIONS FOR ENHANCED PERCOLATION-OCWD PROPOSED SANTIAGO BASIN IMPROVEMENTS

Gentlemen:

In accordance with Woodward-Clyde Consultants (WCC) proposal dated 16 September 1988 and your subsequent authorization of 26 September 1988, we have completed the subject work and this report summarizes the results. Previously, WCC completed a preliminary geotechnical investigation for the Santiago Creek Replenishment project site which recommended construction of a buttress to enhance slope stability in Bond Pit. This report presents four optional design schemes for the Bond Pit buttress and drain system. These design schemes considered the effect of the buttress on the percolation of water into the pit. Three of the four design schemes considered resulted in no reduction in percolation rate due to the presence of the buttress.

The Santiago Creek Replenishment project site is located in the Cities of Orange and Villa Park and in unincorporated areas of Orange County. The site is bounded as follows: Santiago Canyon Road (north), Hewes Avenue (east), Bond Avenue (south), and a residential development (west). The location of the site is indicated in Figure 1. Basically, the site consists of two abandoned sand and gravel pits referred to as Bond Basin (south basin) and Blue Diamond Basin (north basin). The configuration of these basins is indicated in Figure 2.

The scope of work for the current investigation consisted of:

1. Review of existing geotechnical investigation reports for the site, hydrologic investigation reports of pit percolation analyses performed by Camp, Dresser, McKee (CDM) and Orange County Water District (OCWD), and related Villa Park Dam hydrologic data.

2. Field investigations consisting of additional geologic mapping and performance of four test pits (work completed under separate contract, but reported herein).

3. Laboratory testing of selected samples obtained from the test pits, including soil classification (work completed under separate contract, but reported herein).

4. Engineering analyses to develop conclusions and recommendations regarding:

   - site geologic constraints that may affect pit percolation rates;
   - natural recharge capacity of Bond and Blue Diamond Pits;
   - soil characteristics of native fill materials that may be used in the construction of the buttress;
   - specific dimensions and material specifications of the buttress drain elements;
   - earthwork specifications for the berm materials (work deleted from current scope but replaced by consideration of four alternative drainage schemes in lieu of one drainage scheme).

In the following sections of this report, we present a description of the project, the results of field investigations and laboratory testing, the results of percolation rate analyses together with other parametric analyses, a comparison of four buttress/drainage schemes, conclusions derived from our investigation, and general conditions.

PROJECT DESCRIPTION

The site is proposed as a ground water recharge and flood control basin for OCWD. The existing Santiago Creek, as well as a proposed 66-inch diameter pipeline leading from Burris Pit, will be used as sources of the water. The water elevation will vary from approximate elevation 160 feet (a dry condition) to elevation 280 feet in Bond and Blue Diamond pits depending upon precipitation, available water, time of year, etc. The general geometry has been developed for the buttress that will be placed on the slope of the Bond Pit, extending to elevation 230 feet adjacent to Bond Avenue and Hewes Avenue as illustrated by the shaded region in Figure 3.

The final buttress design should include specifications to maintain a percolation rate (recharge rate) through the buttress that is equal to or greater than the recharge capacity of the native formation. The natural recharge capacity of Bond and Blue Diamond pits was estimated from OCWD hydrologic data in order to develop critical design parameters. Four different construction and drainage schemes are presented in this report and the recharge rate of each is compared to recharge in Bond and Blue Diamond pits under natural conditions. The designs include: (1) a homogeneous buttress constructed of miscellaneous native material from the pit bottom; (2) a homogeneous buttress constructed of washed and selectively graded native material; (3) a buttress with drainage blankets constructed of 3/4-inch gravel to enhance recharge; and (4) a buttress with drainage blankets constructed of Caltrans Class II Permeable material. Potential fouling mechanisms that could affect flow through the buttress drainage system, as well as system maintenance are discussed.

FIELD INVESTIGATION AND LAB TESTING

The stratigraphy within the Bond Pit was mapped extensively from the top of the pit, (at approximately 300 feet in elevation), to the top of the talus, (at approximately 200 feet in elevation) as a requisite for the 1985 Geotechnical Investigation (WCC). To locate any low permeability silt interbeds behind the talus at the base of the side slope, a backhoe test pit was excavated. This test pit and existing outcrops along the bottom of the south wall of the Bond Pit were mapped; the stratigraphy observed is generally illustrated in the schematic stratigraphic section shown on Figure 3. Buttress drains are designed to align with, and recharge into, the gravel layers which are estimated to be approximately three orders of magnitude more permeable than the silt lenses as described below.

Four test pits were dug in the floor of the Bond Pit to determine the character of the underlying sediments. The lithology exposed in each pit was logged and selected samples from the test pits were taken to the WCC soil laboratory for mechanical sieve analysis. The results of the mechanical sieve analyses and the logs of the test pits are included in Appendix A.

ANALYSIS OF PERCOLATION

The natural recharge capacity of Bond and Blue Diamond pits was estimated from data collected by the OCWD in order to provide criteria to design the buttress so as to minimize its affect on the recharge capacity of the Bond Pit. During a 1980 storm event, Villa Park Dam floodwater releases inundated the Santiago basins. Typical watershed management data were recorded by OCWD, including Villa Park Dam outflow and corresponding fluctuations in water level elevation in Bond and Blue Diamond Basins. When outflow from Villa Park Dam ceased, water elevation in the Bond and Blue Diamond Basins immediately began to recede. The receding water level was monitored and recorded by OCWD during the ensuing 10 weeks. From these data, the rate of change in water level (ft/day) was calculated. These data are summarized in Table 1. Because both Bond and Blue Diamond Pits were completely flooded during the 1980 storm, the water levels and rates recorded reflect recharge through both Bond and Blue Diamond Pits.

Because recharge into the Santiago Pits occurs under bank storage conditions, similar to water recharge into stream banks during flood events, the recharge capacity of the basin should be greater during rising stage than during falling stage. As discussed above, the data available to make calculations of the recharge occurring from the Bond Pit were collected while the water level in the pit was dropping (a falling head condition), therefore, analyses of the data may result in a low estimate of the hydraulic conductivity of the native formation. For purposes of the design analysis of a pervious buttress, so as not to adversely affect the recharge capacity of the pit, it is necessary to use a high recharge rate for the pit. To develop a high estimate of the recharge capacity and hydraulic conductivity of native material around the Bond Pit, the total recharge estimated from the falling water level data was considered to flow only through two sides of the Bond Pit as discussed below.

To calculate a recharge rate for Bond Pit, the volume of water that flowed out through the pit during a specified time interval following the 1980 storm was determined by multiplying the rate of change in water level by the surface area of the basin determined by measuring the pit dimensions at specified elevations. The pit dimensions were measured from a topographic map prepared on 4 November 1983 by Robert J. Lung and Associates. It was assumed that minimal changes in the pit configuration occurred between 1980 and 1983.

Similar calculations were performed to determine flow through Blue Diamond Pit, and these values were added to those for the Bond Pit to determine the total recharge rate for both pits at specific elevations as summarized in Table 2. The maximum recharge rate for Bond Pit at full stage was calculated to be approximately 79.5 cfs. Figure 4 graphically illustrates the recharge rate for each pit at various water elevations as calculated from the data for the 1980 flood event.

Because both Bond and Blue Diamond Pits will probably be simultaneously operated, significant outflow from either pit will not occur through the berm between them (north wall of Bond Pit). Little outflow is expected through the west wall of Bond Pit because the permeability is probably low due to concrete and fine debris which has been poured over the side of the pit at that location. Therefore, only the south and east walls of Bond Pit were considered to transmit water from the pit into the surrounding native formation.

The effective wetted side slope area of the south and east walls of Bond Pit was measured, using a planimeter, from the 1983 topographic map of Santiago Basins. That area, measured from the base of the pit at approximately 180 feet elevation to the overflow elevation at approximately 288 feet, was roughly 14.35 acres. The area of the base of the pit was measured to be approximately 72.1 acres.

Natural recharge in Bond Pit is believed to be predominantly horizontal through the gravel sand units of the formation. This occurs because the sedimentary sequence is interstratified gravel, sand and silt. Interbedded silt lenses which are relatively impermeable, are expected to greatly reduce downward vertical migration of recharged water.

Figure 5 graphically illustrates the correlation between recharge rate and wetted side slope area available for recharge at different water elevations. The straight line relationship (solid line) indicates that recharge, does not taper off at lower water elevations. If the base of the pit was impermeable, the flow rate through the sides of the pit would gradually be reduced due to saturated conditions and the curve would exhibit an exponential shape (dashed line). Therefore, it is likely that some recharge is occurring through the bottom of the pit.

If we assume that the permeability of the base material is 1/10 that of the side slopes, then the effective permeable area of the base would be reduced from 72.1 acres to approximately 7.2 acres. Based on this assumed relative permeability, the effective permeable base area is roughly one-third of the total recharge area available in Bond Pit ([7.2 acres] ÷ [14.35 acres + 7.2 acres] = 0.33). Following this assumption, our analysis indicates that the recharge flowing through the side slopes of Bond Pit effects a horizontal area equivalent to 14.4 acres, computed to extend a distance of 1,235 feet around the perimeter of the pit. Based on this result, water flowing through the side slopes of the pit into the natural formation can be assumed to flow under a hydraulic gradient of 0.089 ([maximum vertical head] ÷ [effective flow distance] = [110 ft/1235 ft]).

The hydraulic conductivity of the natural formation material can be estimated by Darcy's law:

$$Q = KiA$$

where,

- Q = recharge flow rate, cfs;
- i = hydraulic gradient, ft/ft;
- A = area, ft^2; and
- K = hydraulic conductivity, ft/sec.

As a conservative estimate, K can be determined assuming that the total recharge (79.5 cfs) flows only through the side slope area (14.35 acres) with a gradient of 0.089, disregarding any vertical recharge through the base of the pit. For design purposes, the hydraulic conductivity of the native material can therefore be estimated at 1.4×10^{-3} ft/sec or approximately 4.3×10^{-2} cm/sec. Because of the assumption that the flow only occurs through two sides of the pit, this value is considered higher than the actual permeability for the pit walls behind the buttress. Therefore, using this value for hydraulic conductivity provides a conservative basis for comparison in the evaluation of the impact of buttress design on the recharge capacity of Bond Pit.

PARAMETRIC ANALYSIS OF DRAINAGE SCHEMES

Results of the previous WCC geotechnical investigation (23 October 1985) recommend that a buttress be constructed to reinforce the south and east walls of Bond Pit. The proposed buttress should extend from the base of the pit (approximately 160 feet in elevation after completion of current excavation operations) to 230 feet in elevation and should be constructed with a 2:1 slope with a 20-foot wide maintenance road above the buttress as shown in Figure 6. At each end of the buttress, the maintenance road was assumed to slope to the bottom of the pit with a 15 percent grade as shown in Figure 3.

Four different buttress designs are described in the paragraphs that follow. The drainage characteristics of each design and the associated material requirements are assessed. The potential effects of the designed buttresses on the recharge capacity of Bond Pit are estimated.

HOMOGENEOUS BUTTRESS DESIGN

Miscellaneous Fill. The simple buttress design illustrated in cross-section on Figure 6 may be constructed with material excavated from the bottom of the pit. The volume of material required to construct the buttress with the 15 percent grade access road at each end of the buttress is estimated to be 345,000 cubic yards based on the shaded area shown in Figure 3. Due to the occurrence of sandy silt lenses in the predominantly gravel layers of the natural formation and the silty sand matrix of these gravel layers, the percentage of fine material that would occur when the native material is mixed for use in the construction of the buttress fill would significantly lower the permeability of the buttress fill relative to the surrounding native formation.

If we assume zero permeability for the buttress, the effective side slope area available for recharge would be reduced by 4.5 acres (the area covered by the buttress), resulting in a total effective recharge area of 17 acres at full stage, including 7.2 acres of effective base area. This is equivalent to a 21 percent reduction in area and can be directly correlated to a 21 percent loss of recharge capacity in Bond Pit or an 11.3 percent reduction in flow recharged by Bond and Blue Diamond pits combined. The results of calculations tabulated in Tables 3 and 4 indicate that as water level drops, the impact of the impermeable buttress increases, especially below El. 230 feet where the loss of recharge capacity becomes approximately 33 percent in Bond Pit.

If the bottoms of the pits are considered impermeable, the effective area available for recharge in Bond Pit with the impermeable buttress would be 9.8 acres. The resultant loss in recharge capacity would be 32 percent at full stage in Bond Pit and 17 percent at full stage in Bond and Blue Diamond pits, combined.

The area available for recharge may be enhanced by exposing native material below the buttress in a trench, excavated to obtain material to construct the buttress. An example of approximate dimensions of a trench which would yield the estimated 345,000 cubic yards required to construct the buttress, is schematically illustrated in Figure 6a.

<u>Washed Fill</u>. The permeability of the simple buttress design illustrated in Figure 6 may be enhanced by washing silt and other fines from the gravel and sand materials excavated from the bottom of the pit, and then constructing the buttress with the coarse fraction. Mechanical analyses of selected test pit samples indicate that fine materials passing the 200 mesh screen ranged from 2 to 34 percent of the total matrix, as summarized in Table 5.

The estimated volume of material that should be excavated for this scheme, approximately 415,000 to 450,000 cubic yards, is 20 to 30 percent more than the volume actually required for construction of the buttress to account for the fines that will be washed out plus some provisions for shrinkage. Separated fines can be relocated to a small area in the central region of the bottom of the pit to fill deep excavations, thereby minimizing any reduction of the effective recharge area of the base of Bond Pit.

To enhance horizontal recharge into the sides of the pit, construction fill material should be primarily excavated from trenches near the base of the proposed buttress location, thereby exposing approximately 67 feet of the underlying native formation slope, as schematically illustrated in Figure 6a.

Once fines have been removed by gravel washing, the well graded sand and gravel may be used to construct the buttress. In order to reduce the potential for clogging of the buttress gravel by infiltration of fine debris from poor quality water or storm flows in the basin, a finer grained filter mat should be placed on the surface of the buttress. The filter mat can be composed of a relatively fine grained sand that is compatible with the well graded gravel in the buttress (for example, Class II permeable material). The thickness of the filter mat should be sufficient to limit clogging to the surface of the buttress and should be consistent with grading maintenance operations. A similar filter zone or filter fabric should be placed adjacent to the native slope between the buttress and the native soil.

The frequency of maintenance operations should depend upon the degree of percolation reduction through the buttress due to clogging in the filter mat on the buttress surface. The loss in head through the buttress can be monitored by installing water level monitoring standpipes adjacent to the natural slope behind the buttress, and perforated between 230 feet and 160 feet elevation, as illustrated in Figure 6. The recommended placement of the standpipes is indicated in Figure 3.

ZONED DRAINAGE BLANKET BUTTRESS DESIGN

An alternative solution to enhancing the permeability of the buttress is to construct it predominantly with miscellaneous fill and include blankets of permeable material in discrete layers throughout the buttress. These permeable "blankets" control recharge through the buttress and the drain system is designed to attain a recharge capacity equivalent to the full-stage natural recharge capacity of Bond Pit. Therefore, the required thickness of the drainage blankets is determined by the hydraulic conductivity of the material used to construct them. Two different buttress/drain designs are described in the paragraphs that follow; the drainage blankets in each design are constructed with a different type of permeable material.

Imported 3/4-Inch Gravel. The buttress and drainage blanket scheme designed with imported 3/4-inch gravel drainage blankets is depicted in Figure 7. The hydraulic conductivity of the imported gravel fill is approximately 5 cm/sec or 0.164 ft/sec. The full-stage natural recharge capacity of Bond Pit can be estimated by multiplying the hydraulic conductivity (1.4×10^{-3} ft/sec) by the vertical height of water at full stage (288-180 = 108 ft). In order to meet or exceed the full-stage natural recharge capacity of Bond Pit, the drainage layer thickness should be approximately one foot ([thickness of gravel]) = ([K of formation] x [full stage height]) ÷ [K of gravel] = ([1.4×10^{-3} ft/sec] x [108 ft]) ÷ ([0.164 ft/sec]), as a minimum.

As previously mentioned, recharge in Bond Basin occurs under bank storage conditions and because our initial data was collected during falling head in the basin, the estimated natural hydraulic conductivity may be a low value. Therefore, a 50 percent safety factor, at least, should be added to the calculated thickness of the gravel drain. Figure 7 illustrates a buttress constructed of miscellaneous fill excavated from the pit bottom, having three 1-foot thick horizontal gravel blankets inlaid through the buttress to enhance its permeability and to provide a percolation rate equal to or greater than that of the natural formation.

To prevent infiltration of fine material into the gravel blankets, which would significantly curtail their permeability, filter fabric should be placed on both sides of each layer and along the back and the front of the buttress. However, calculations indicated that siltation on the filter fabric surface of the buttress can have a large limiting effect on the recharge flow rate into the permeable layers of the buttress. As an example, consider an 0.1-foot thick layer of silt deposited on the buttress surface. Assume the hydraulic conductivity of silt is approximately 3.3 x 10^{-7} ft/sec, and there is a 5-foot head loss due to water flowing through the silt layer (i.e., 5 percent reduction at a stage height of 110 feet). If two-thirds of the recharge flows through the side slopes of Bond Basin, the length of the silt covered slope required to overcome the head loss due to the reduced permeability can be estimated by the relationship,

$$L_N K_N i_N = L\ K_{silt}\ (H_L/L_{silt})$$

where,

L_N = length of the natural slope (70 ft);

K_N = hydraulic conductivity of the natural formation for 53 cfs recharge (9.6 x 10^{-4} ft/sec);

i_N = hydraulic gradient of the natural formation (0.089);

K_{silt} = hydraulic conductivity of the silt layer (3.3 x 10^{-7} ft/sec);

(H_L/L_{silt}) = hydraulic gradient through the silt layer (5 ft/0.1 ft); and

L = length of the effective wetted buttress area covered by silt (to be solved).

The length of the side slope of the buttress, uniformly covered by 0.1 foot of silt, necessary to allow 53 cfs of recharge would be 365 feet. Therefore, in order to reduce the effect of siltation on recharge reduction, the permeability of the face of the buttress should be maximized by placing a 1-foot thick layer of gravel over the surface. The layer should be enclosed by filter fabric except at intersections with horizontal gravel blankets. To maximize recharge at the interface between the buttress and the natural slope, another 1-foot thick gravel layer should be placed there and enclosed with filter fabric as previously detailed for the surface layer. To monitor any head difference between the point of recharge behind the buttress and in the basin, which would indicate need for maintenance, monitoring standpipes can be installed as illustrated in Figures 3 and 7.

The estimated volume of materials required to construct this buttress design include 36,000 yd^3 of gravel, 1.9 MSF of filter fabric, and 309,000 yd^3 of miscellaneous fill material. Maintenance of the buttress surface may be complicated when water level exceeds the buttress height.

IMPORTED CLASS II PERMEABLE MATERIAL

The buttress and drainage blanket scheme designed with imported Class II permeable material drainage blankets is depicted in Figure 8. The hydraulic conductivity of the Class II permeable material is approximately 8.2×10^{-3} ft/sec. By following the method and the rationale described previously for 3/4-inch gravel drainage blankets, it can be shown that a minimum drainage layer thickness of approximately 13 feet is required to meet or exceed the full-stage natural recharge capacity of Bond Pit. A minimum 50 percent safety factor should be applied. The permeability of available recharge areas along the surface of the buttress and at the interface with the natural slope can be increased by including 5-foot thick permeable layers. The aforementioned monitoring standpipes are included to monitor head loss due to clogging at the surface of the buttress, and filter fabric is not necessarily required because the permeable material is filter graded to prevent silt infiltration.

The volume of materials required to construct this buttress design include approximately 190,000 yd^3 of Class II permeable material, and approximately 155,000 yd^3 of miscellaneous fill material. Maintenance will be complicated, as previously mentioned.

CONCLUSIONS

Four different buttress designs have been discussed and the estimated recharge capacity of the basins with each design has been compared to the natural recharge capacity estimated in Bond Basin and Bond and Blue Diamond Basins combined. The materials required for each of the four buttress designs herein described, are listed in Table 6 along with costs approximated for imported materials and filter fabric. Costs for gravel washing operations, buttress construction, and imported material delivery costs are not included.

Construction of a homogenous buttress with randomly mixed native materials excavated from the bottom of the pit would likely effectively reduce to zero the relative permeability of the 4.5 acre side slope area covered by the buttress. Figure 9 graphically summarizes the effect of the relatively impermeable buttress on the total recharge capacity of Bond Pit and of Bond and Blue Diamond Pits combined, as water level in the pits declines.

By washing fine material from the native gravel excavated from the bottom of the pit, the permeability of the homogeneous fill can be greatly increased, probably approximating the permeability of the native formation. A filter mat should be placed at the surface of the buttress to reduce infiltration of fine materials into gravel buttress fill, which could significantly decrease permeability. Siltation at the surface and resultant head loss of recharging flow through the buttress can be monitored with standpipes, perforated below the buttress, as illustrated in Figures 3 and 6. The filter mat can be maintained by grading. A filter mat or fabric should also be placed between the buttress and the native slope.

Purpose ?

Excavation of the estimated 414,000 to 450,000 cubic yards required for washing and construction operations can be performed in trenches located at the base of the buttress slopes. As schematically represented in Figure 6a, excavation of these trenches will expose approximately 67 feet of natural side slope area which should further enhance recharge.

Drainage blankets may be included in a buttress constructed predominantly of miscellaneous fill, in order to increase the overall permeability of the buttress and reduce the volume of permeable material to be imported. The required thickness of the blankets is controlled by the permeability of the drain material.

A buttress constructed with 3/4-inch gravel as a drain material can accommodate a sufficient recharge flow with three 1-foot thick gravel blankets, as illustrated in Figure 7. To reduce the effects of siltation and enhance recharge into the native formation, 1-foot thick layers should be placed along the face of the buttress and at the slope interface of the buttress and natural formation. The volume of gravel material required to construct this design is approximately 36,000 cubic yards. The cost for 3/4-inch gravel is approximately $12 per yard, and the estimated cost for the volume of gravel required for this design is approximately $430,000. Filter fabric lining around each gravel layer is required to prevent infiltration of fine debris. The total cost for the estimated 1.9×10^6 square feet of fabric needed (at $0.08 per sq. ft.) would be $150,000.

Monitoring standpipes, similar to those previously described, should be installed as illustrated in Figures 3 and 7. Because a small amount of siltation on the filter fabric can dramatically reduce recharge into the buttress, regular maintenance is a necessity. When the water level in the basin exceeds the height of the buttress, maintenance may be difficult.

Class II permeable material may be used instead of gravel in the drainage blankets. Because this type of material is filter graded, filter fabric should not be required. However, continued deposition of silt on the surface of the buttress with time may eventually contaminate the material in the drains by infiltration, and would reduce permeability. Class II material has a much lower permeability than 3/4-inch gravel, and two 13-foot thick layers are required to fulfill the recharge criteria for the buttress, as illustrated in Figure 8. The volume of Class II permeable material required for this drainage scheme is 190,000 cubic yards (300,000 tons). The cost is estimated to be $11.75 per ton or $3.6 million for the volume of Class II permeable material required.

Monitoring standpipes would be installed as illustrated in Figures 3 and 8. Maintenance required can be accomplished by normal grading.

GENERAL CONDITIONS

Professional judgments represented in this report are based on evaluations of the technical information gathered, on our understanding of the project, and on our general experience in the geotechnical field. We do no guarantee the performance of the project in any respect, only that our engineering work and judgments rendered meet the standard of care of our profession at this time.

Woodward-Clyde Consultants hope this report meets the current needs of this project. The staff hydrogeologist assigned to this project is Ms. Catherine Quinn. If you have any questions or need any additional information, please call.

Very truly yours,

WOODWARD-CLYDE CONSULTANTS

John A Barneich

John Barneich
RGE 116

REGISTERED PROFESSIONAL ENGINEER
JOHN A. BARNEICH
No. 116
Exp. 6/30/89
GEOTECHNICAL
STATE OF CALIFORNIA

S. Thomas Freeman

S. Thomas Freeman
RGE

CERTIFIED ENGINEERING GEOLOGIST
S. THOMAS FREEMAN
No. EG1015
Exp. 6/1990
STATE OF CALIFORNIA

JAB:STF:lk
(L-JAB/Chafty-L)

TABLE 1

**1980 STORM EVENT
WATER ELEVATION MEASUREMENTS**

Date of Measurement	Water Elevation (ft msl)	Rate of Water Elevation Decline (ft/day)
4/9/80	288.25	-1.74
4/11/80	285	-1.5
5/2/80	260	-1.26
5/15/80	247	-1.0
6/3/80	232.5	-0.8
6/24/80	220	-0.6

(2L-JAB/SP-Tab1)

TABLE 2

SANTIAGO PITS
RECHARGE CAPACITY, Q (cfs)

Pit Name	Water Elevation (ft msl)	Rate of Water Elevation Decline (ft/day)	Basin Area (acres)	Q (cfs)
Bond	288.25	-1.74	90.5	79.5
	285	-1.5	90.1	68
	260	-1.26	86.2	54
	247	-1.0	84.1	42
	232.5	-0.8	81.8	33
	220	-0.6	79.6	24
Blue Diamond	288.25	-1.74	77.4	68
	285	-1.5	75.5	57
	260	-1.26	60	38
	247	-1.0	51.7	26
	232.5	-0.8	42.2	17
	220	-0.6	36.5	11
Bond & Blue Diamond	288.25	-1.74	--	147.5
	285	-1.5	--	125
	260	-1.26	--	92
	247	-1.0	--	68
	232.5	-0.8	--	50
	220	-0.6	--	35

TABLE 3

RELATIVELY IMPERMEABLE BUTTRESS CONSTRUCTION
RECHARGE AREA REDUCTION, BOND BASIN

Water Elevation (feet/msl)	Wetted Side Slope Area of Bond (initial) (acres)	Area of Impermeable Buttress (acres)	Base Area (acres)	Effective* Base Area (acres)	Total Effective Recharge Area (initial) (acres)	Total Effective Recharge Area (with impermeable buttress) (acres)	Area Ratio
288.25	14.3	4.5	72.1	7.2	21.5	17	0.79
285	13.9	4.5	72.1	7.2	21.1	16.6	0.79
260	10.4	4.5	72.1	7.2	17.6	13.1	0.74
247	8.6	4.5	72.1	7.2	15.8	11.3	0.74
232.5	6.7	4.5	72.1	7.2	13.8	9.3	0.67
220	4.9	3.9	72.1	7.2	12.1	8.2	0.68

* Assume permeability of base is 1/10 that of the side slopes

TABLE 4

RELATIVELY IMPERMEABLE BUTTRESS CONSTRUCTION-REDUCTION IN RECHARGE CAPACITY, BOND AND BLUE DIAMOND BASINS

Water Elevation (ft/msl)	Natural Recharge Bond Basin (cfs)	Natural Recharge Bond and Blue Diamond Basins (cfs)	Reduced Recharge in Bond Basin (cfs)	% Loss in Bond Basin	Reduced Recharge in Bond and Blue Diamond Basins (cfs)	% Loss in Bond and Blue Diamond Basins
288.25	79.5	147.5	62.8	21	130.8	11.3
285	68	125	53.8	21	110.8	11.4
260	54	92	40.2	26	78.2	15.3
247	42	68	30.3	28	56.3	17.3
232.5	33	50	21.9	33	38.9	21.7
220	24	35	16.3	32	27.3	21.8

TABLE 5

TEST PIT SAMPLES
PERCENTAGE OF FINE MATERIAL

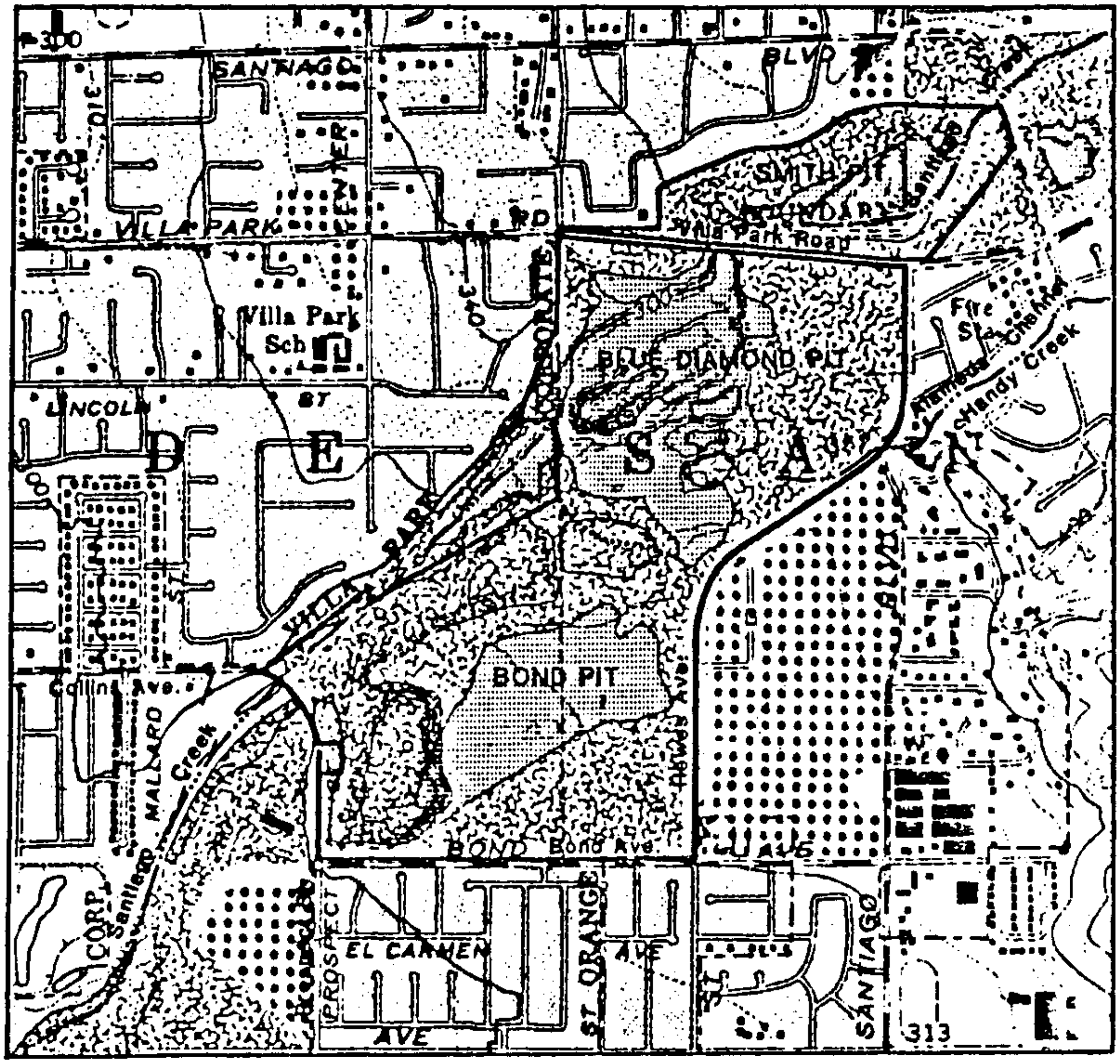
Sample Number	-200 Sieve
TP-1-SK-2	2%
TP-2-SK-1	6%
TP-3-SK-1	3%
TP-3-SK-2	34%
TP-4-SK-2	2%

TABLE 6

CONSTRUCTION MATERIALS

Type of Buttress	Buttress Fill Material	Estimated Buttress Fill Volume	Buttress Drain Material	Drainage Blanket					Filter Fabric	
				K	Thickness	Volume	Estimated Cost	Estimated Total Cost	Area of Filter Fabric	Estimated Total Cost
Homogeneous	Native, randomly mixed	345,000 cubic yds	N/A*	N/A	N/A	N/A	N/A	N/A	N/A	N/A
Homogeneous	Native, washed and selectively graded	414,00 cubic yds (to be excavated for washing)	Class II permeable material	8x10^{-3} ft/sec	12 inches	7,700 yd^3 (125,000 tons)	$11.75/ton	$1.4 M	2.1 x 10^{-5} sq ft	$16,800 ($0.08/ft^2)
Drainage Blanket	Native, randomly mixed	309,000 cubic yds	3/4" gravel	0.164 ft/sec	1" x 3	36,000 cubic yds	$12/yd	$430,000	1.9 x 10^6 sq ft	$150,000 ($0.08/ft^2)
Drainage Blanket	Native, randomly mixed	155,000 cubic yds	Class II permeable material	8 x 10^{-3} ft/sec	13" x 2	19,000 cubic yds (300,000 tons)	$11.75/ton	$3.6 M	N/A	N/A

*NA = Not Applicable



LEGEND

—— Boundary of Existing and Proposed OCWD Properties

N

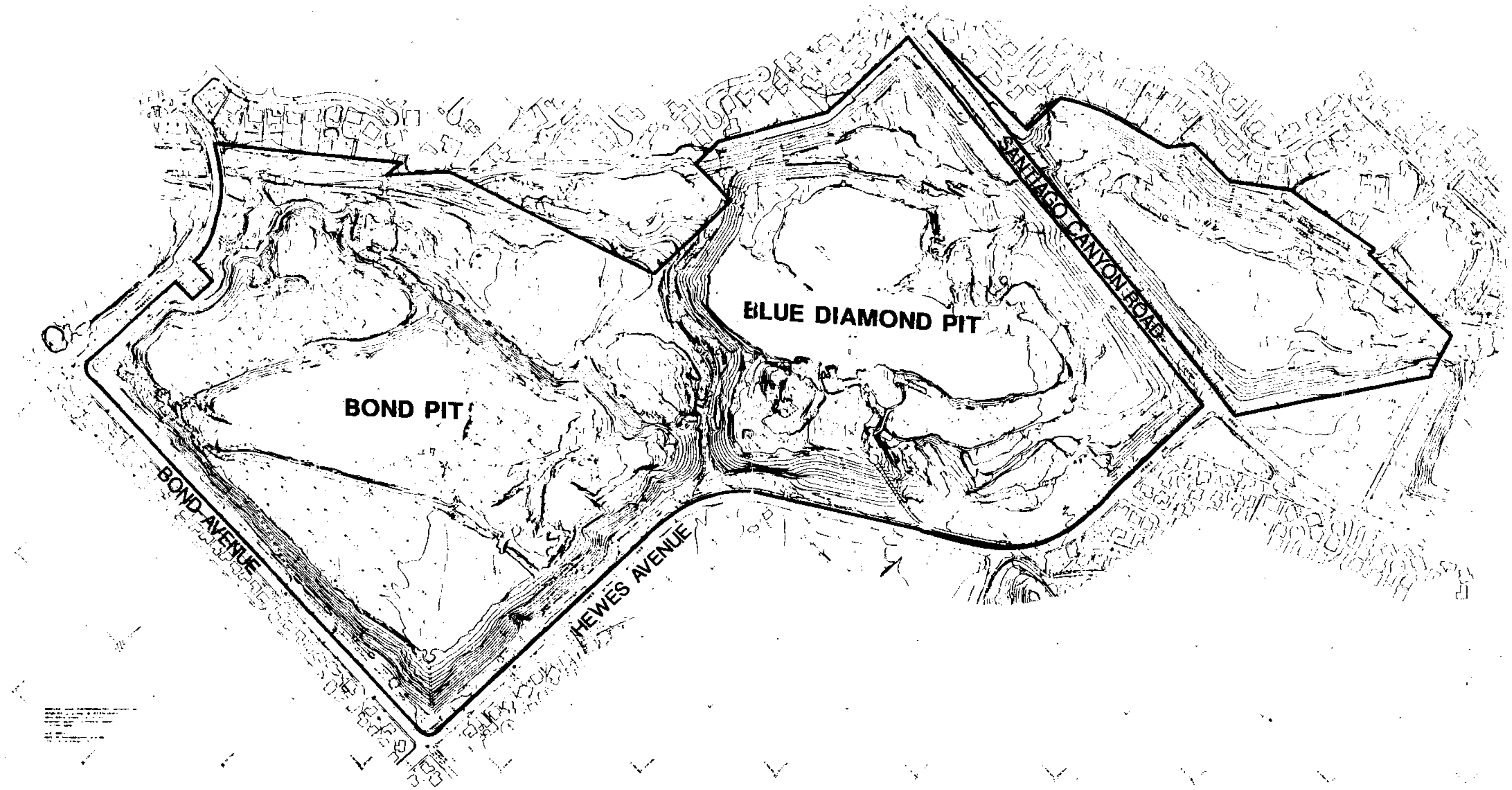
0 1000 2000

SCALE, feet

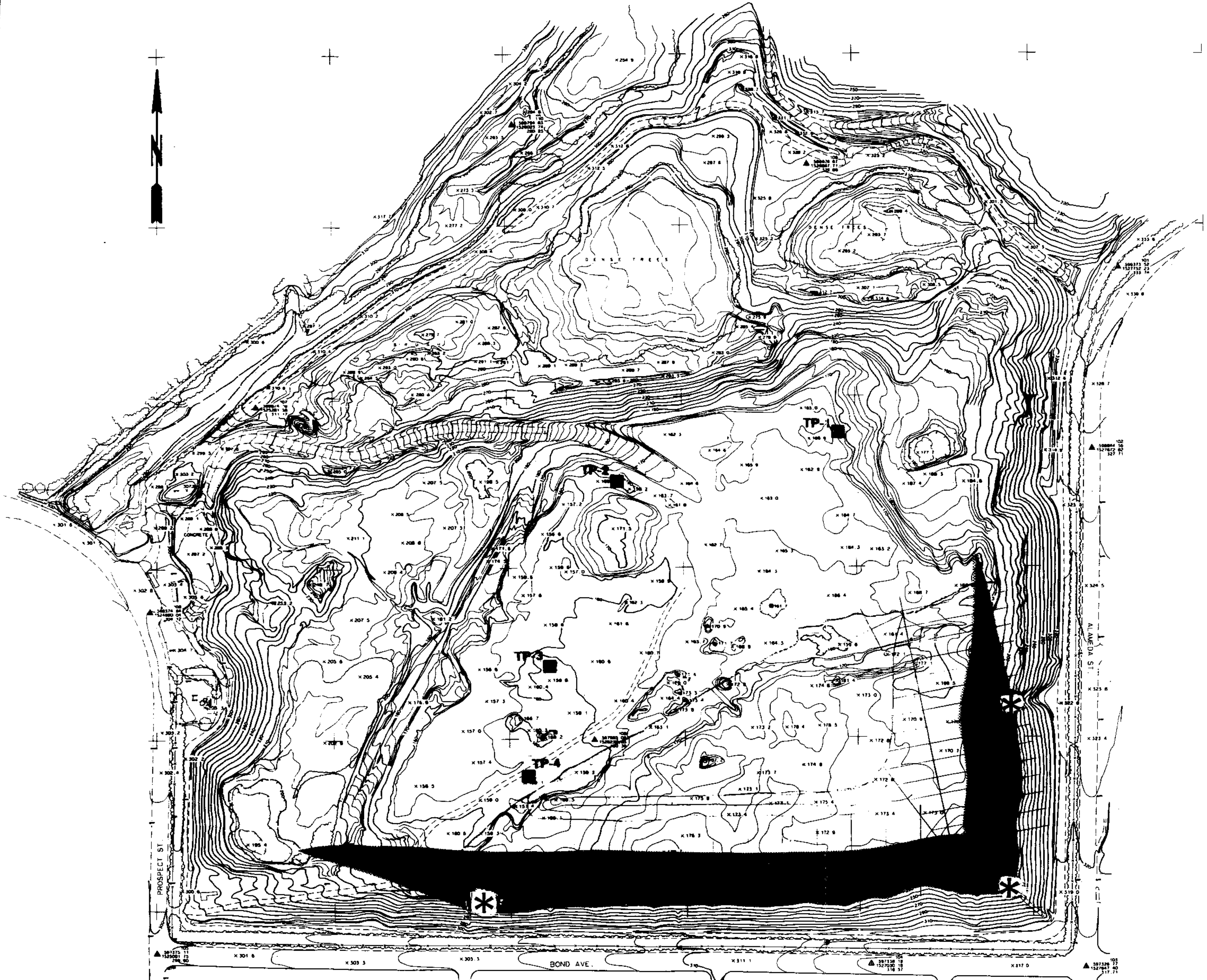
(After PRC[June 1984])

LOCATION OF SANTIAGO CREEK BASINS			
Project No.: 8840269A	Date: OCT. 1988	Project: SANTIAGO BASINS	Fig. 1

Woodward-Clyde Consultants



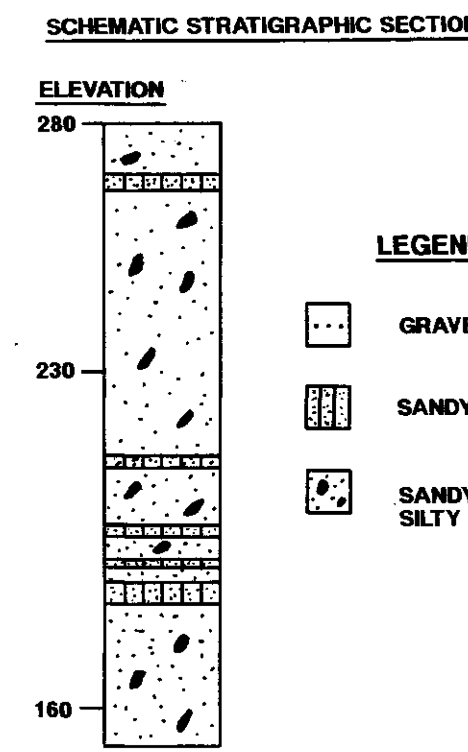
Woodward-Clyde Consultants			
BOND AND BLUE DIAMOND PIT SITE PLAN			
Project No.:	8840269A	Date:	OCT. 1988
Project:	SANTIAGO BASINS	Figure:	2



■ APPROXIMATE LOCATION OF TEST PIT PERFORMED ON 13 SEPTEMBER 1988

APPROXIMATE LIMITS OF BUTTRESS

* APPROXIMATE LOCATION OF MONITORING STANDPIPE



LEGEND

GRAVEL ZONE

SANDY SILT (ML)

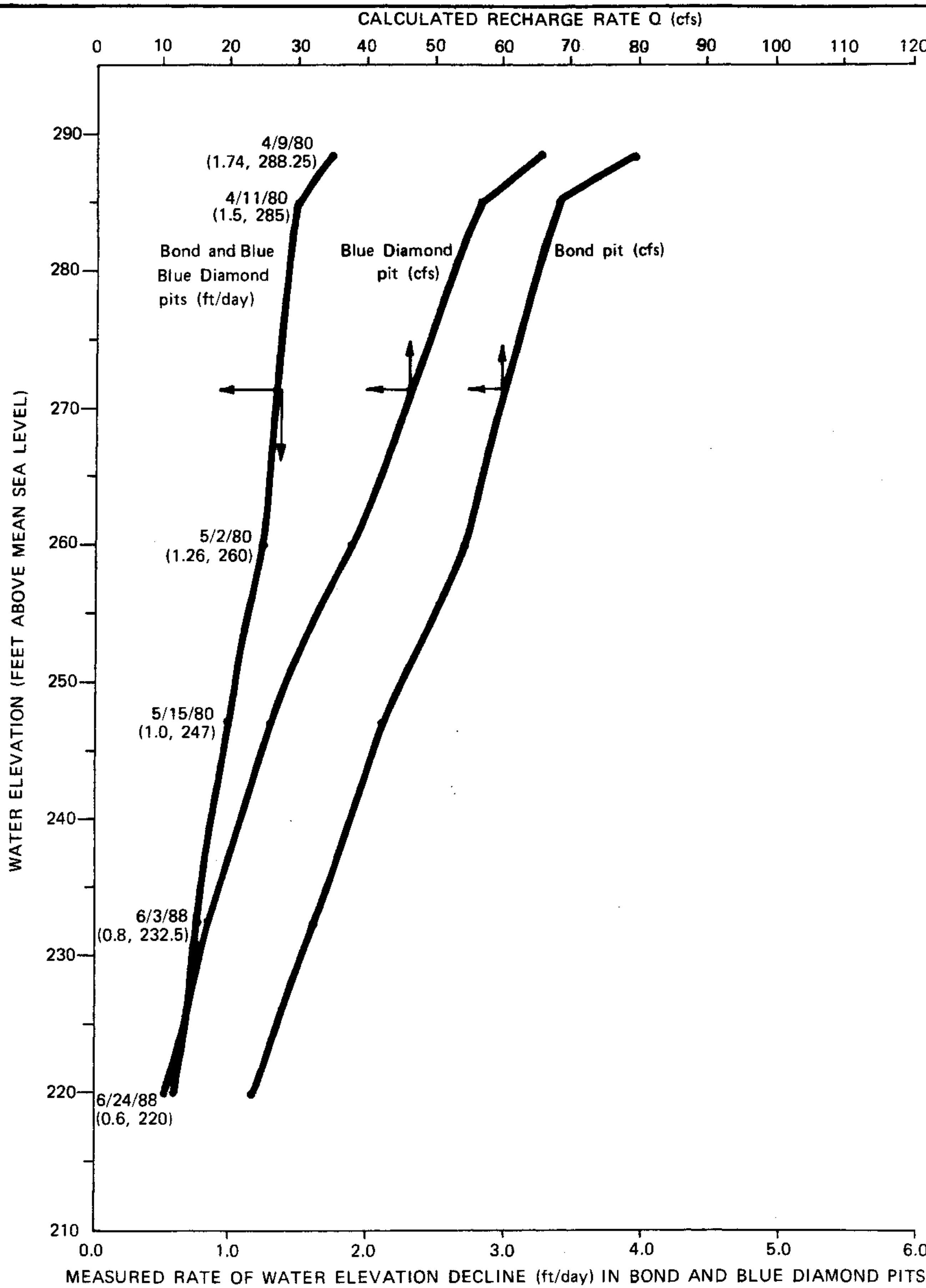
SANDY GRAVEL (GW) SILTY SAND MATRIX

0 500 FEET

Woodward-Clyde Consultants

BOND PIT SITE PLAN AND PROPOSED BUTTRESS LOCATION

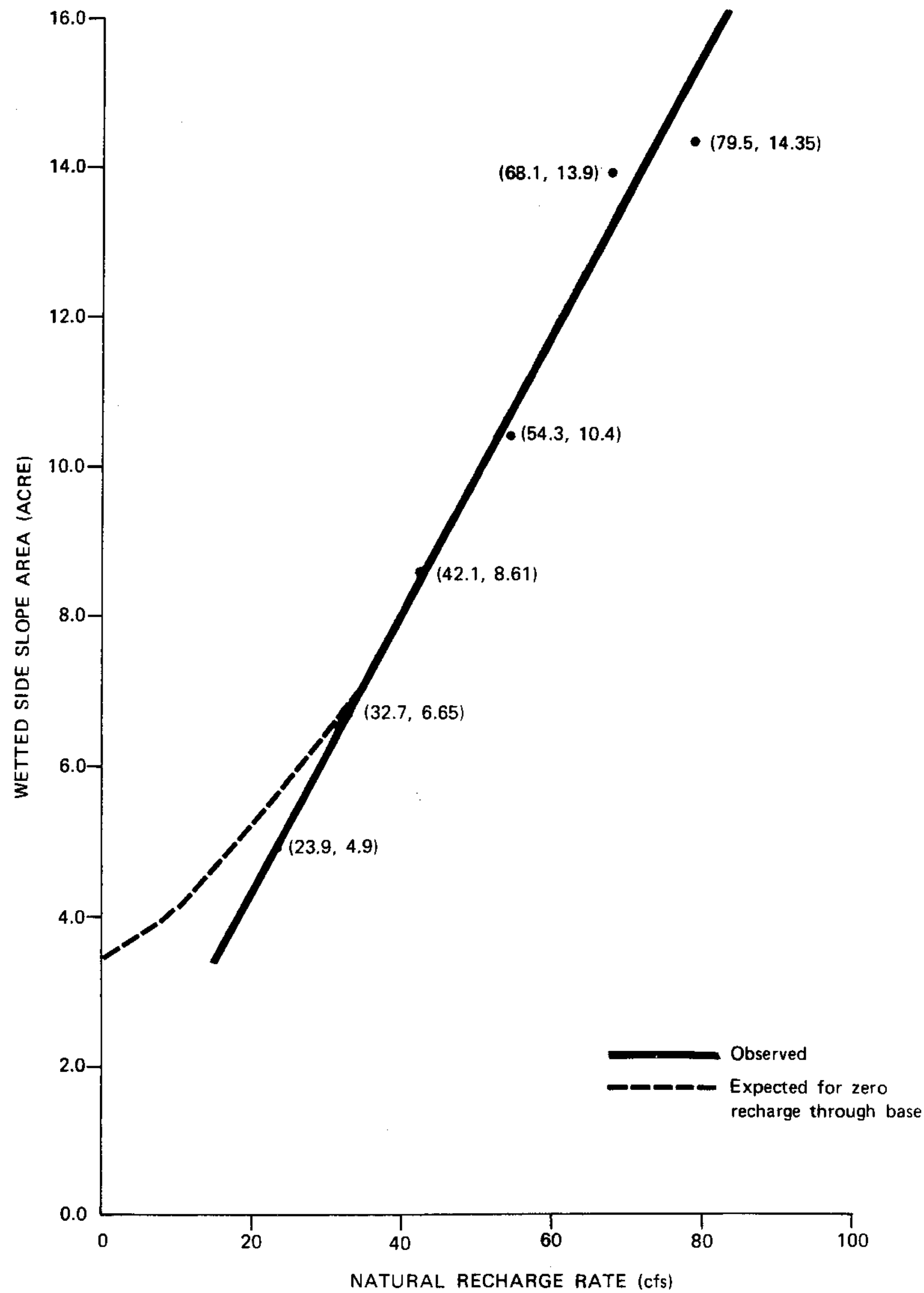
Project No.:	8840269A	Date:	OCT. 1988
Project:	SANTIAGO BASINS	Figure:	3



WATER ELEVATION VS. RECHARGE RATE IN SANTIAGO CREEK BASINS

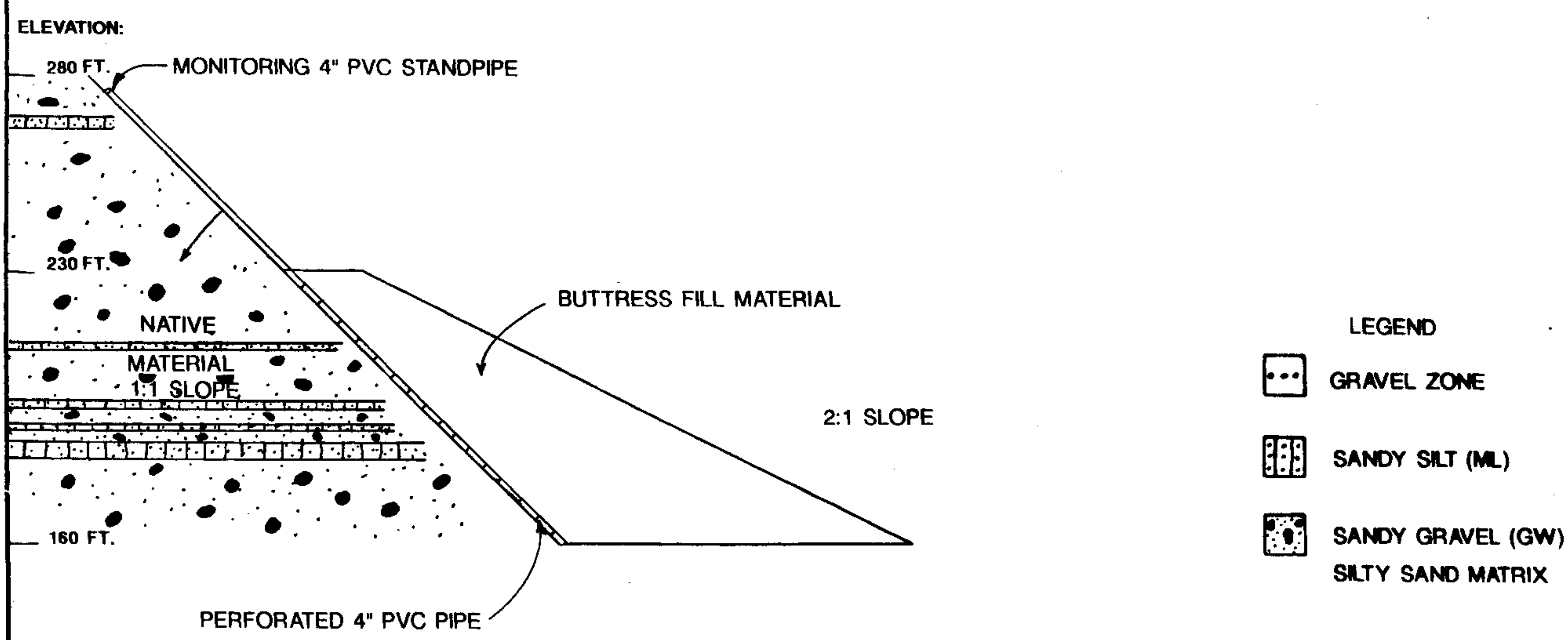
Project No.: 8840269A	Date: October 1988	Project: SANTIAGO BASINS	Fig. 4

Woodward-Clyde Consultants



EFFECT OF IMPERMEABLE BUTTRESS ON NATURAL RECHARGE CAPACITY			
Project No.: 8840269A	Date: October 1988	Project: SANTIAGO BASINS	Fig. 5

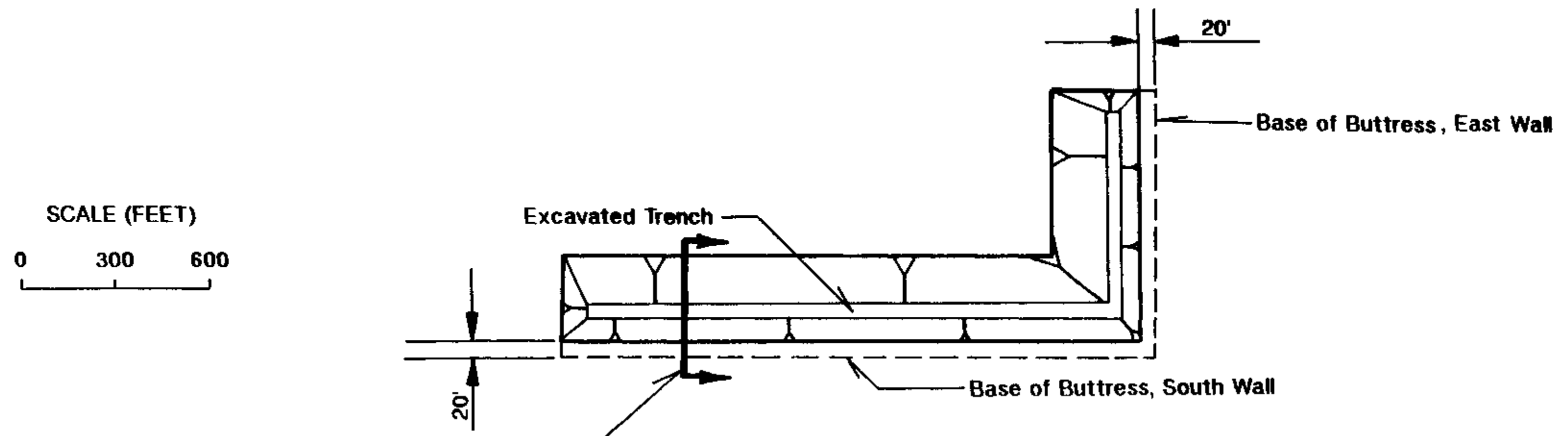
Woodward-Clyde Consultants



BUTTRESS WITHOUT DRAINAGE BLANKETS-SCHEMATIC DIAGRAM

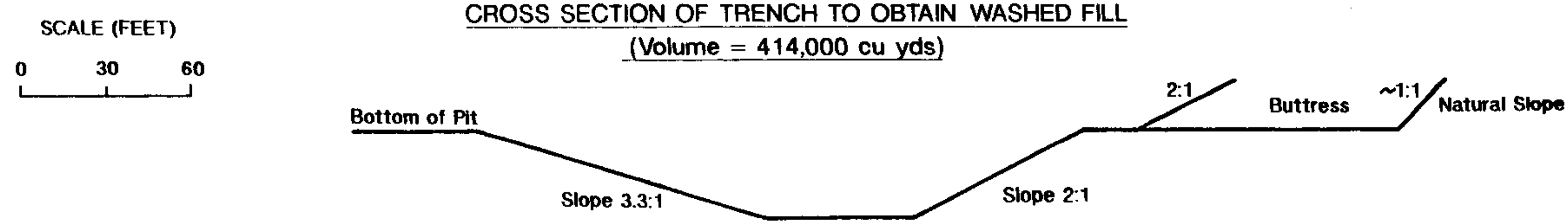
Project No.: 8840269A	Date: 10/88	Project: SANTIAGO BASINS	Fig. 6

Woodward-Clyde Consultants



PLAN VIEW
(Volume = 414,000 cu yds)

CROSS SECTION OF TRENCH TO OBTAIN WASHED FILL
(Volume = 414,000 cu yds)

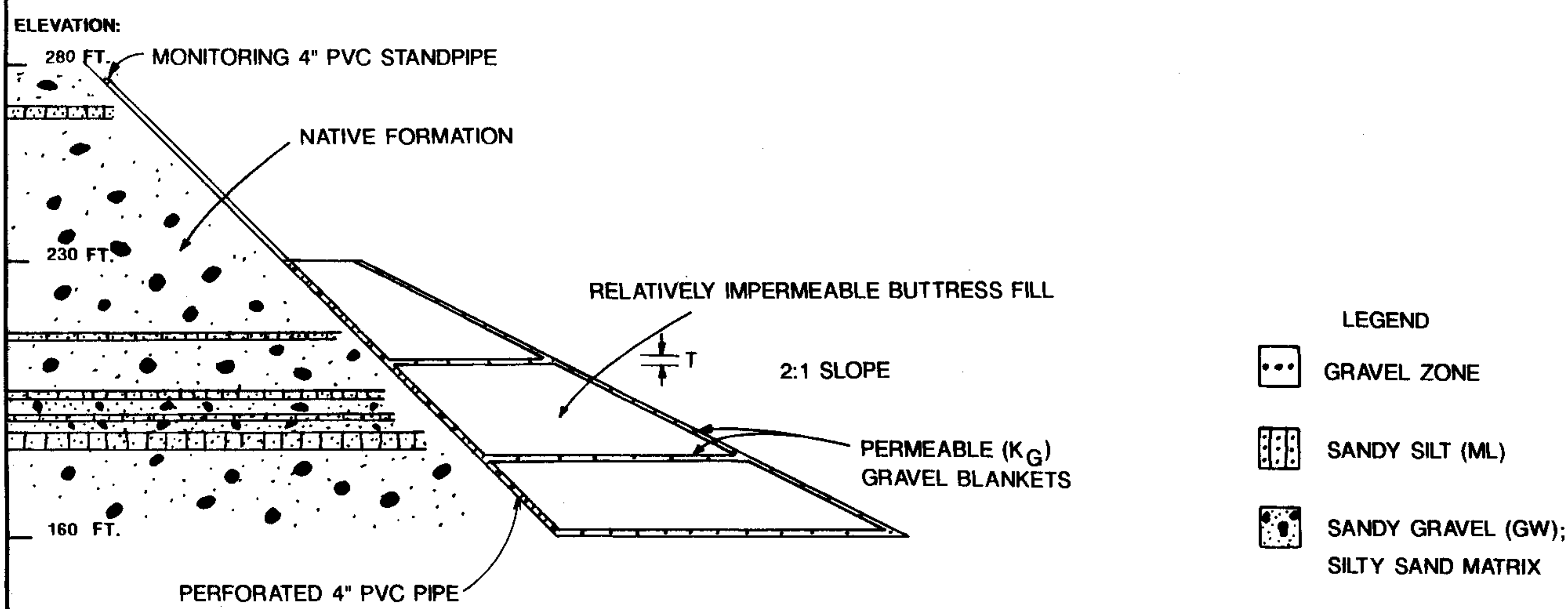


CROSS SECTION OF TRENCH TO OBTAIN MISCELLANEOUS FILL
(Volume = 345,000 cu yds)

BOND BASIN FILL EXCAVATION-HOMOGENEOUS BUTTRESS CONSTRUCTION

Project No.: 8840269A	Date: OCT. 1988	Project: SANTIAGO BASINS	Fig. 6A

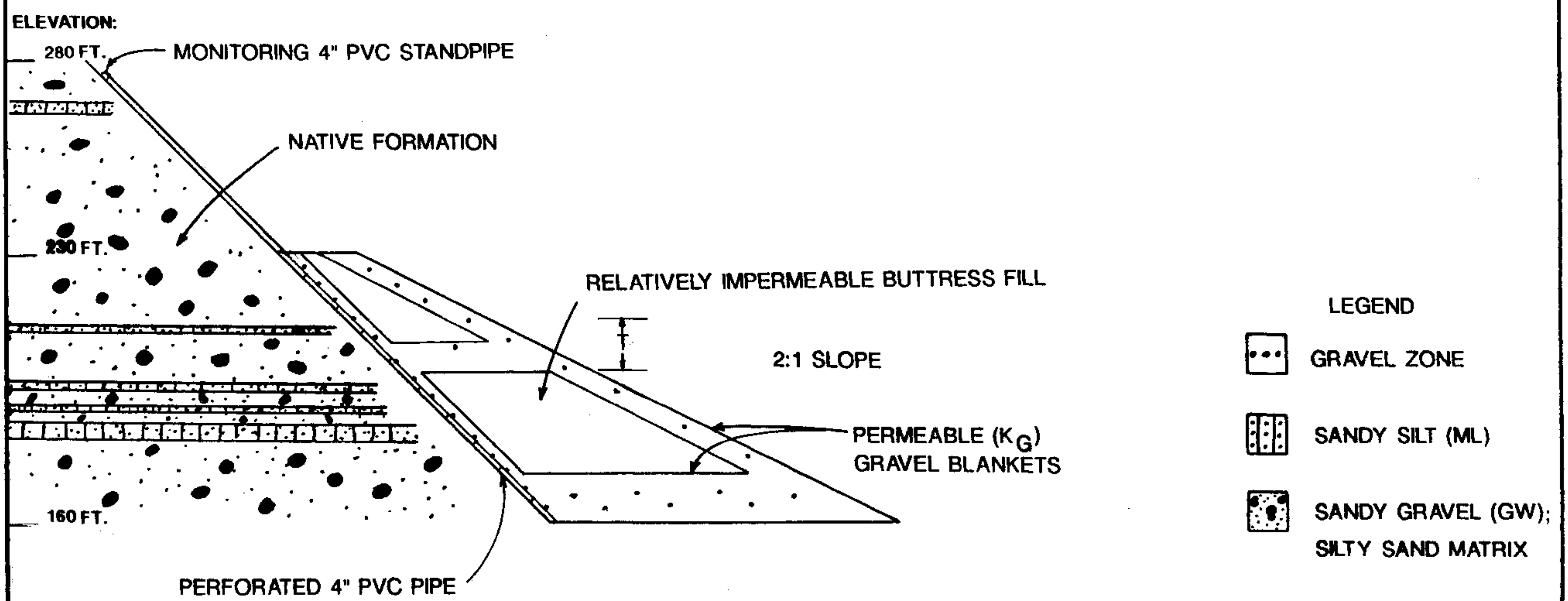
Woodward-Clyde Consultants



BUTTRESS WITH DRAINAGE BLANKETS (K_G = 5 cm/sec; T = 1FT) - SCHEMATIC DIAGRAM

Project No.: 8840269A	Date: 10/88	Project: SANTIAGO BASINS	Fig. 7

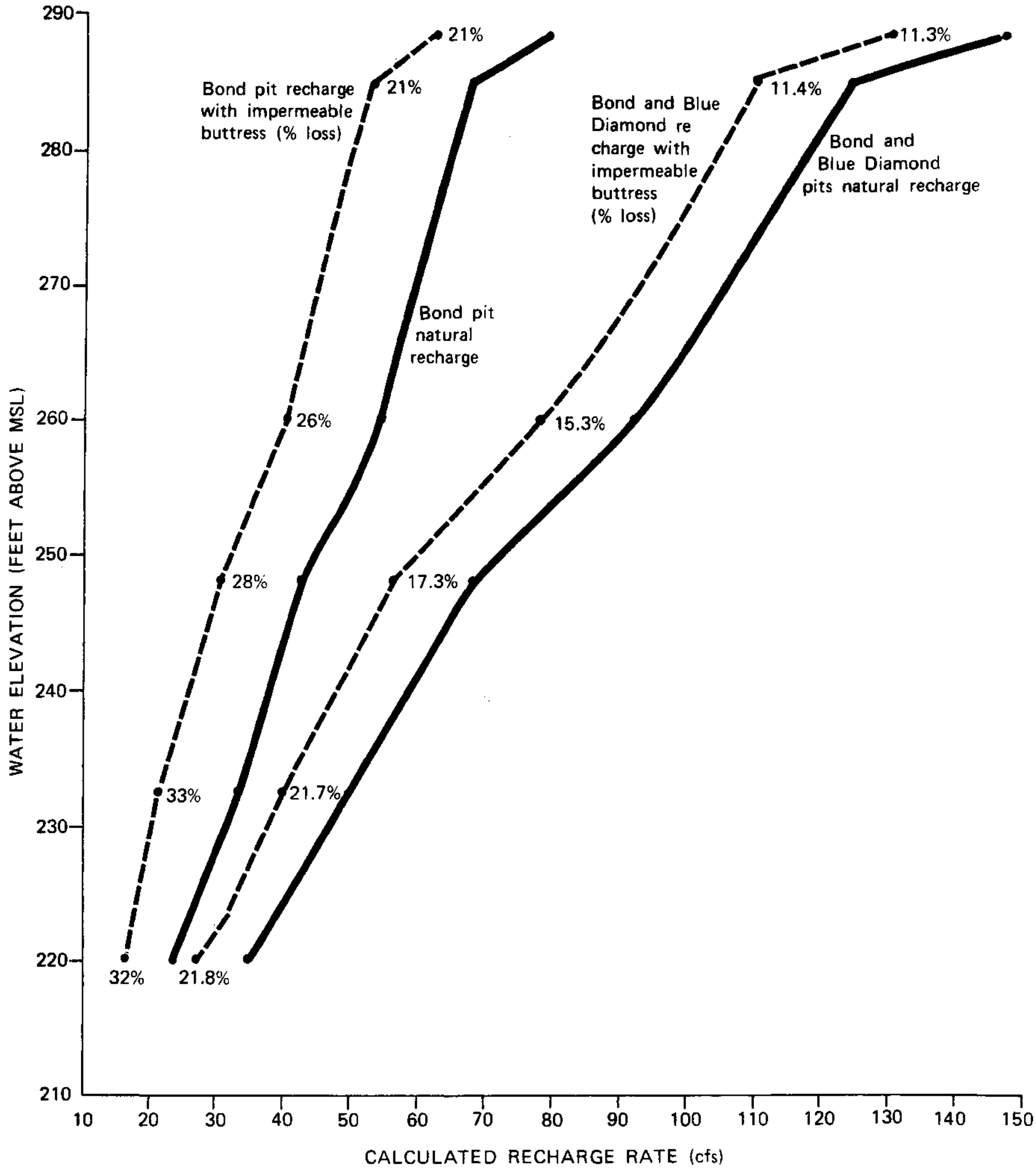
Woodward-Clyde Consultants



BUTTRESS WITH DRAINAGE BLANKETS (K_G = 0.25 cm/sec; T=13 FT) – SCHEMATIC DIAGRAM

Project No.: 8840269A	Date: 10/88	Project: SANTIAGO BASINS	Fig. 8

Woodward-Clyde Consultants



NATURAL RECHARGE RATE VS. RECHARGING SIDE SLOPE AREA

Project No.: 8840269A	Date: October 1988	Project: SANTIAGO BASINS	Fig. 9

Woodward-Clyde Consultants

APPENDIX A

TEST PITS AND LABORATORY TESTS

APPENDIX A

TEST PITS AND LABORATORY TESTS

Woodward-Clyde Consultants logged and sampled four test pits on the northwestern side of Bond Pit on 13 September 1988. The location of the test pits are shown on Figure 3, site plan.

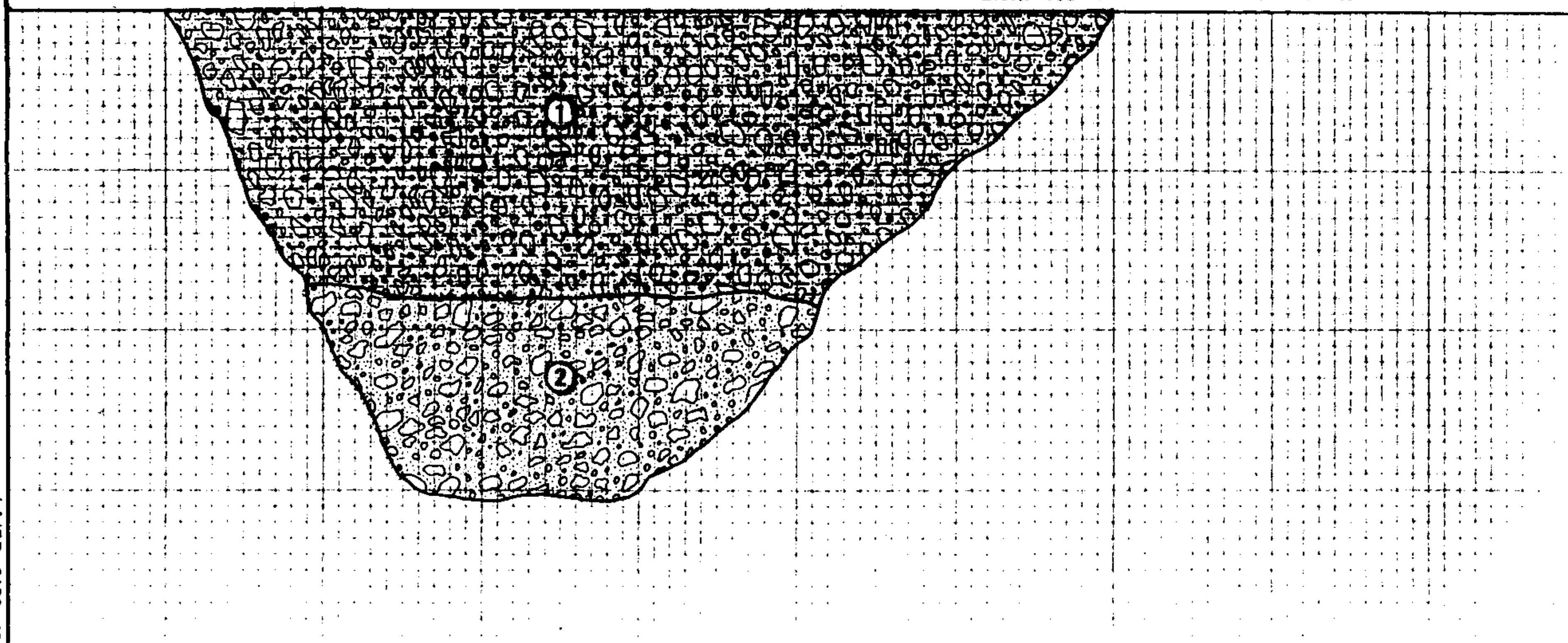
Test pits numbered 1 through 4 were excavated by OCWD with a backhoe to depths of about 6 feet. The test pits were logged by a senior field technician under the guidance of a senior project geologist. The logs of the test pits are presented in Figures A-1 through A-4. The elevations shown on the logs were estimated from the topographic map provided to us and dated August 1988.

Bucket samples of soil were collected and returned to our laboratory. Sieve analyses on the minus number four sieve were performed on selected samples. The results are presented in Figures A-5 through A-9.

Excavated By: OWNER	Geologist / Engineer: H. REYES	Woodward-Clyde Consultants	Project: SANTIAGO BASINS	
Equipment Used: BACKHOE	Shoring: NONE	LOG OF TEST PIT NO. 1	Project No.: 8840269A	Figure No.: A-1

Description	Physical Condition	Attitudes	Samples
1. Moist, dark yellowish brown medium to coarse grained SILTY SAND (SM) with abundant gravel 1/4 inch to 3 inch subrounded and cobbles to 10 inch diameter, subrounded.			SK-1
2. Moist, grayish brown, GRAVEL with SAND (GW), gravel to 3 inch diameter and some cobbles to 10 inch diameter subrounded.			SK-2

Scale: 1"=2' Date: 9-13-88 GRAPHIC REPRESENTATION Elev.: 165 Feet Pit Trend: N55°W

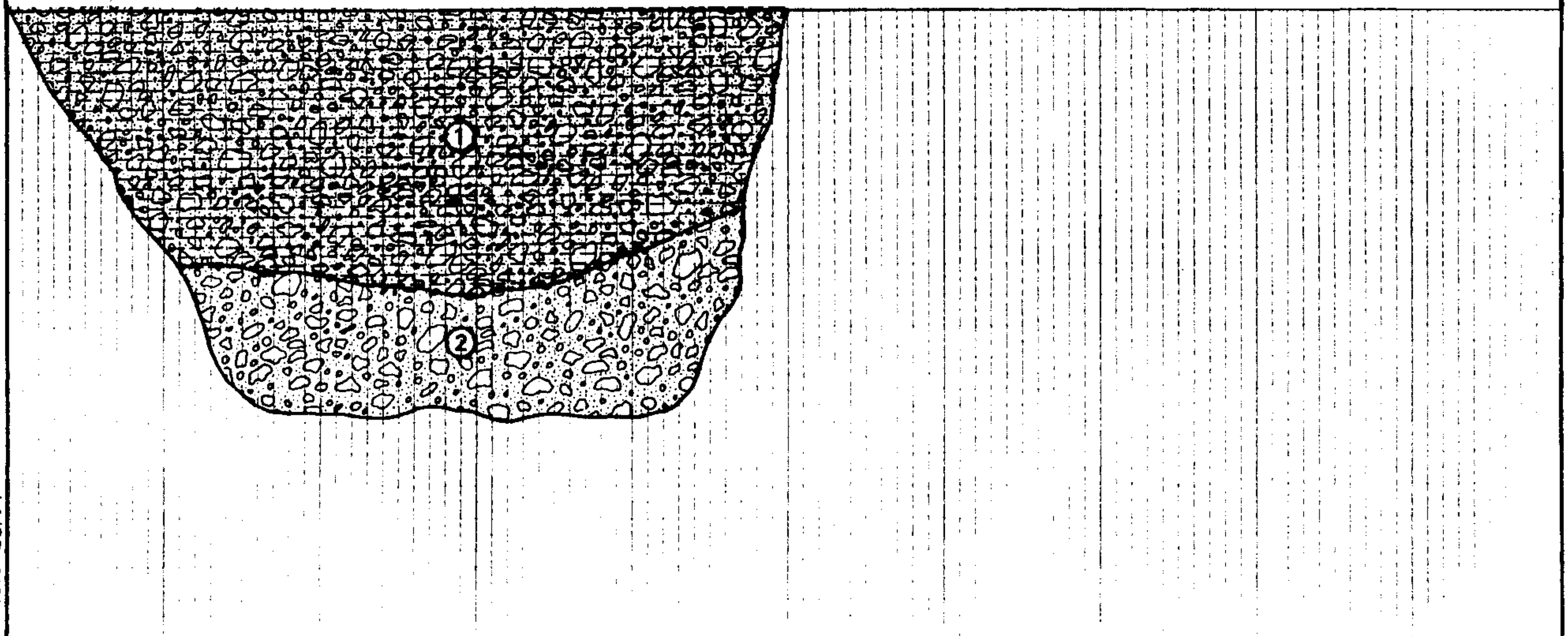


LA/OR–0183–420

Excavated By: OWNER	Geologist / Engineer: H. REYES	Woodward-Clyde Consultants	Project: SANTIAGO BASINS	
Equipment Used: BACKHOE	Shoring: NONE	LOG OF TEST PIT NO. 2	Project No.: 8840269A	Figure No.: A-2

Description	Physical Condition	Attitudes	Samples
1. Moist, dark yellowish brown, GRAVEL with SAND (GW), 3 inch diameter subrounded, and cobbles to 8 inch diameter subrounded.			SK-1
2. Moist, grayish brown, medium to coarse grained, GRAVELLY SAND (SW), gravel to 1 1/2 inch diameter with occasional cobbles 5 inch to 6 inch.			SK-2

Scale: 1"=2'   Date: 9-13-88   GRAPHIC REPRESENTATION   Elev.: 161 Feet   Pit Trend: N45° E

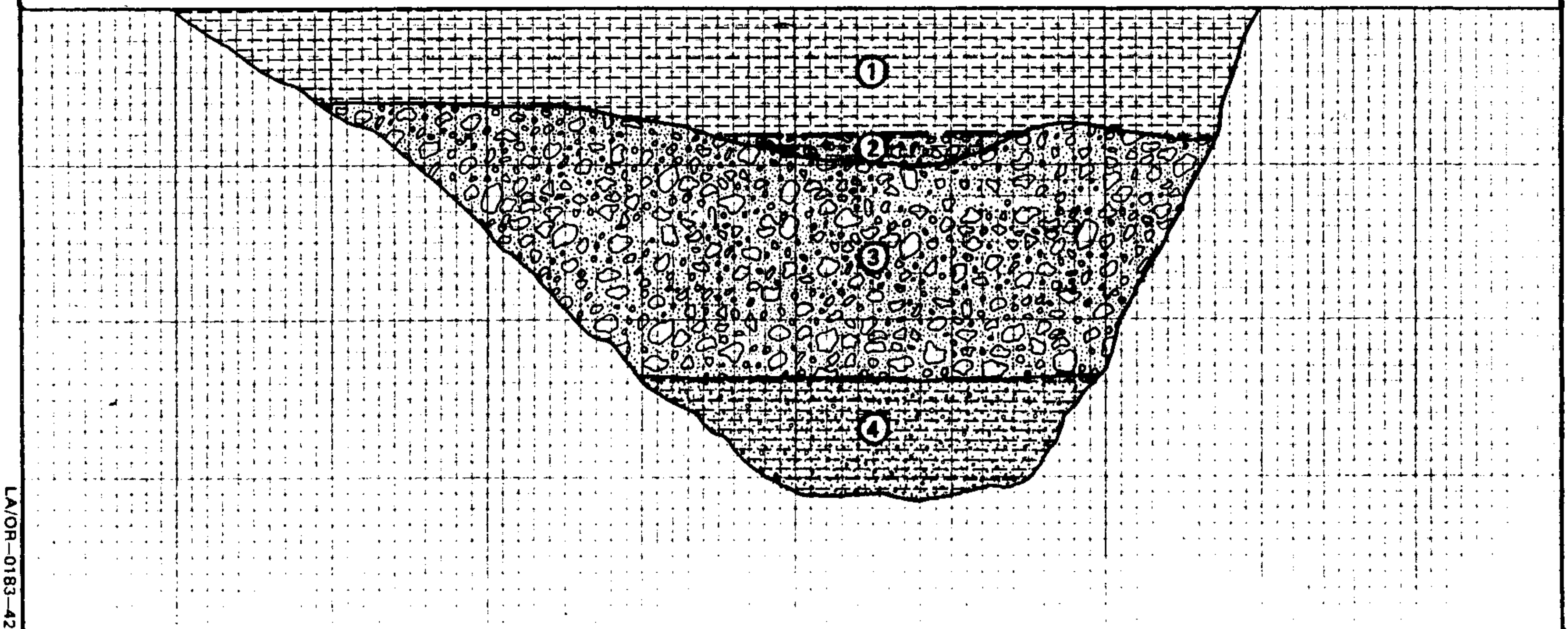


LA/OR–0183–420

Excavated By: OWNER	Geologist / Engineer: H. REYES	**Woodward-Clyde Consultants**	Project: SANTIAGO BASINS	
Equipment Used: BACKHOE	Shoring: NONE	LOG OF TEST PIT NO. 3	Project No.: 8840269A	Figure No.: A-3

Description	Physical Condition	Attitudes	Samples
1. Dry to damp, light brown, CLAYEY SILT (MH).			
2. Dry, light grayish brown, fine grained SILTY SAND (SM).			
3. Moist, grayish brown GRAVEL with SAND (GW), gravel 1/4 inch to 3 inch diameter, some cobbles to 10 inch diameter subrounded.			SK-1
4. Moist, dark yellowish brown, medium grained SILTY SAND (SM).			SK-2

Scale: 1"=2' Date: 9-13-88 GRAPHIC REPRESENTATION W. Wall Elev.: 160 Feet Pit Trend: N75°W

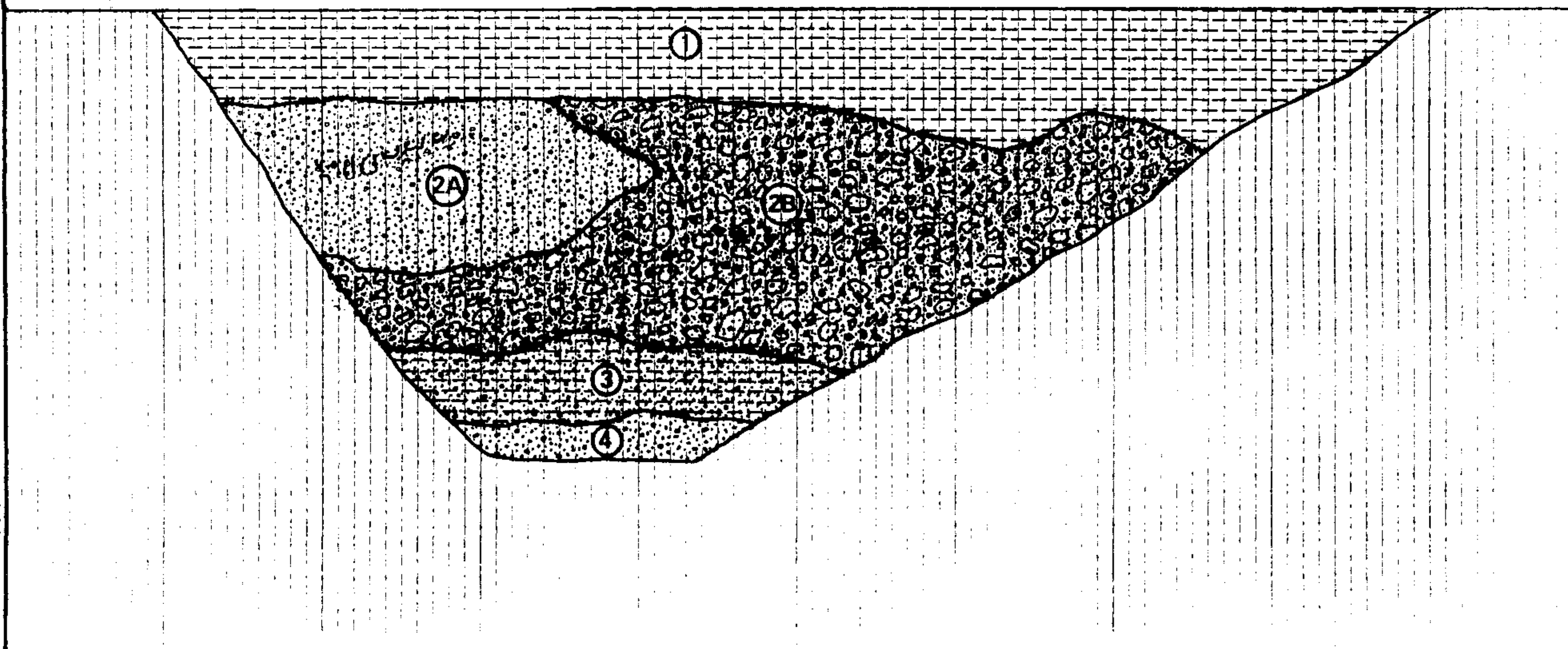


LA/OR—0183—420

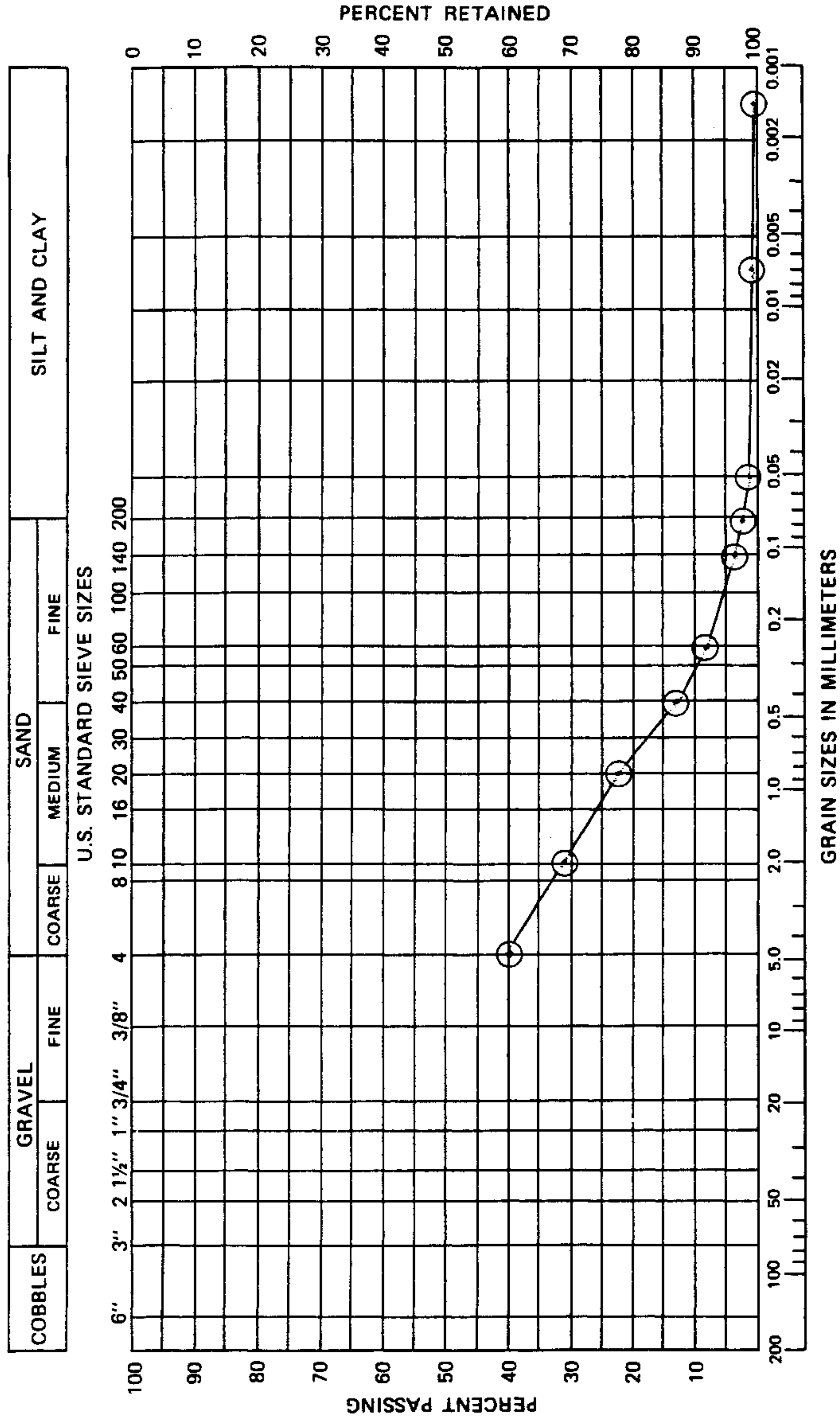
Excavated By: OWNER	Geologist / Engineer: H. REYES	**Woodward-Clyde Consultants**	Project: SANTIAGO BASINS	
Equipment Used: CAT. HOE	Shoring:	LOG OF TEST PIT NO. 4	Project No.: 8840269A	Figure No.: A-4

Description	Physical Condition	Attitudes	Samples
1. Dry, light brown, CLAYEY SILT (MH) with some gravel.			
2a. Moist, dark yellowish brown, medium to coarse grained GRAVELLY SAND (SW).			SK-1
2b. SANDY GRAVEL (GW) some cobbles.			
3. Very moist, dark yellowish brown, medium grained SILTY SAND (SM) with some small diameter gravel.			
4. Moist, grayish brown, GRAVEL with SAND (GW), gravel to 3 1/2" and some cobbles.			SK-2

Scale: 1"=3' Date: 9-13-88 GRAPHIC REPRESENTATION W. Wall Elev.: 159 Feet Pit Trend: N60°W



LA/OR–0183–420



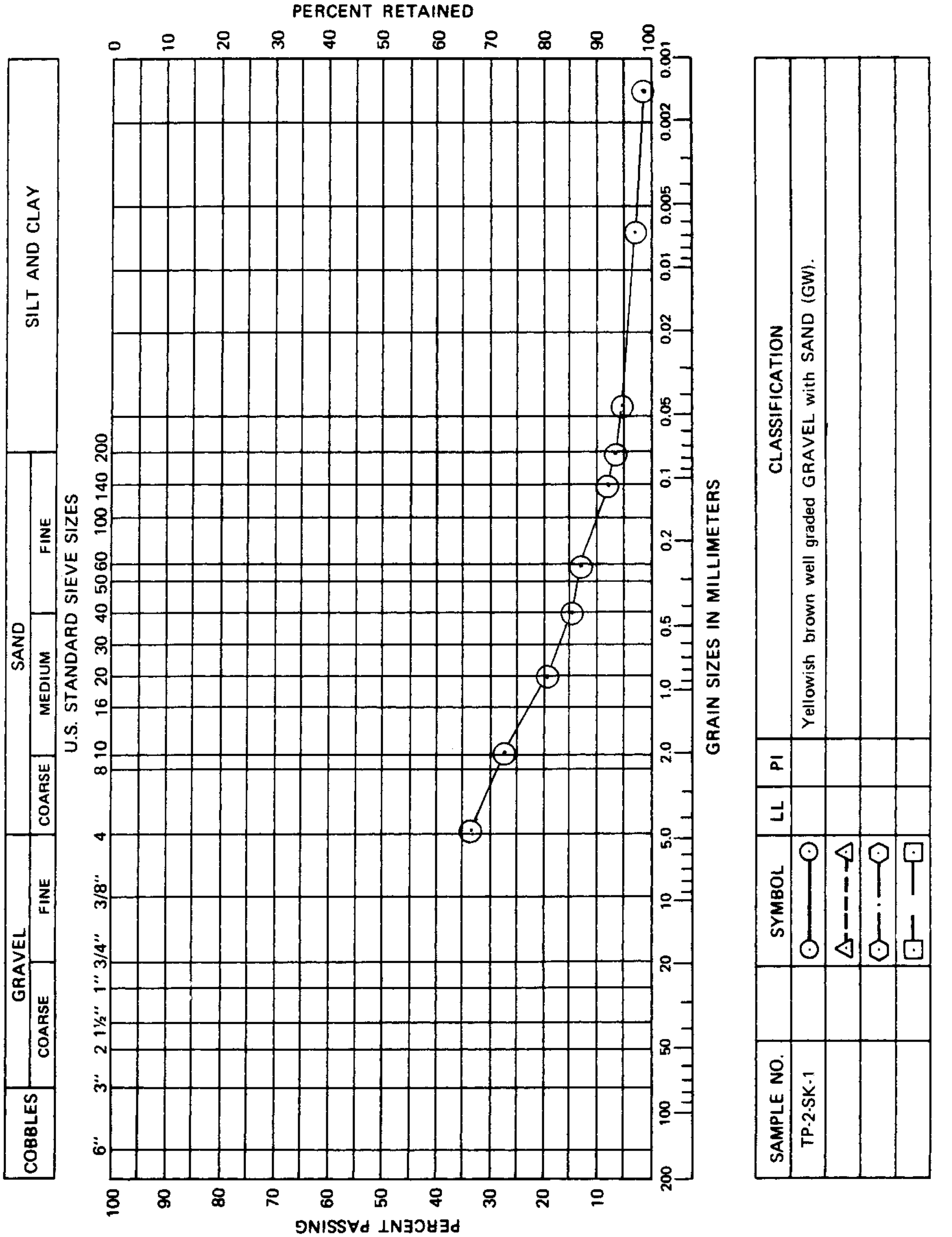
SAMPLE NO.		SYMBOL	LL	PI	CLASSIFICATION
TP-1-SK-2		○—○			Grayish brown well graded GRAVEL with SAND (GW).
		△- - -△			
		⬡—·—⬡			
		□— —□			

Project: SANTIAGO BASINS
Project No. 8840269A

GRAIN SIZE DISTRIBUTION CURVES

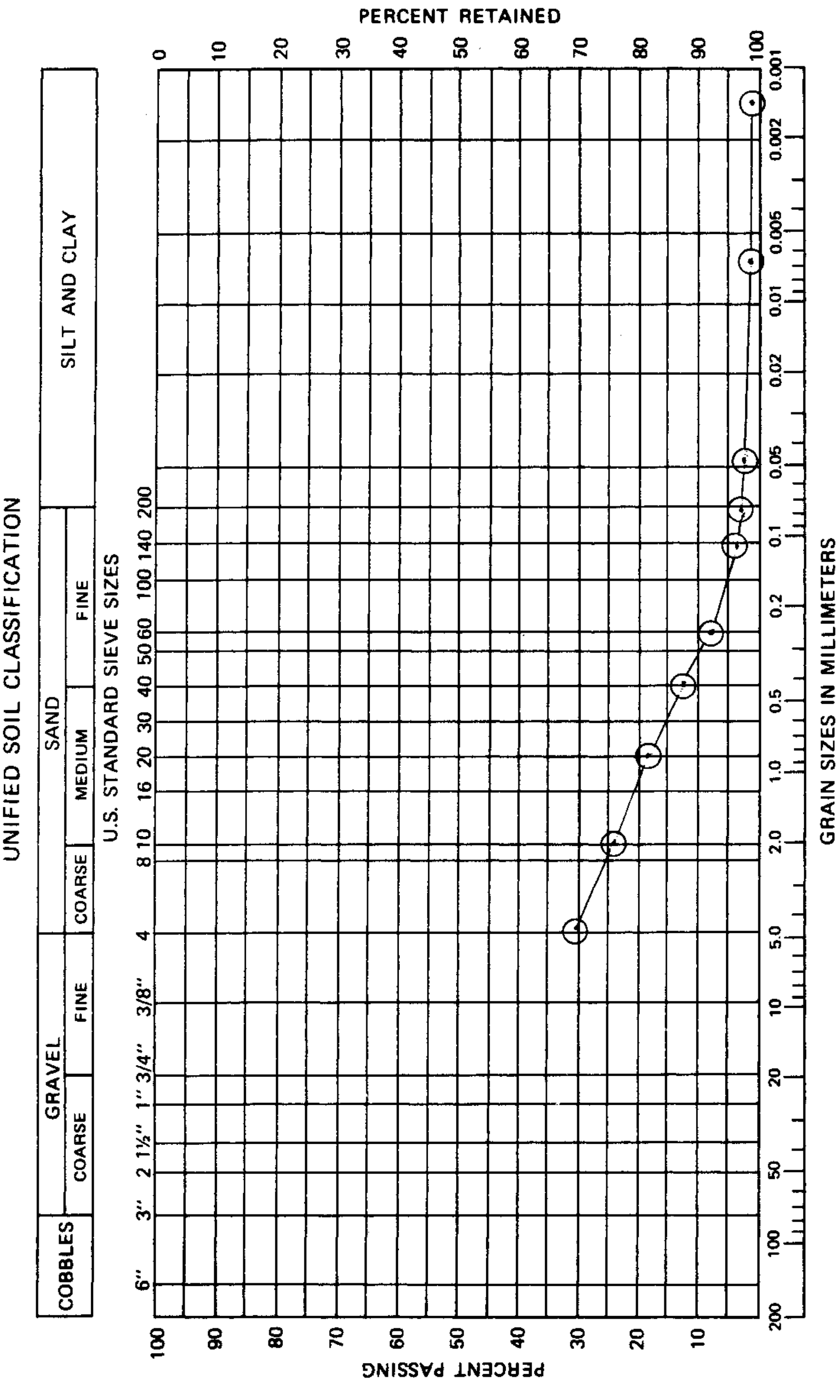
Fig. A-5

WOODWARD–CLYDE CONSULTANTS



Project: SANTIAGO BASINS Project No. 8840269A	**GRAIN SIZE DISTRIBUTION CURVES**	Fig. A-6

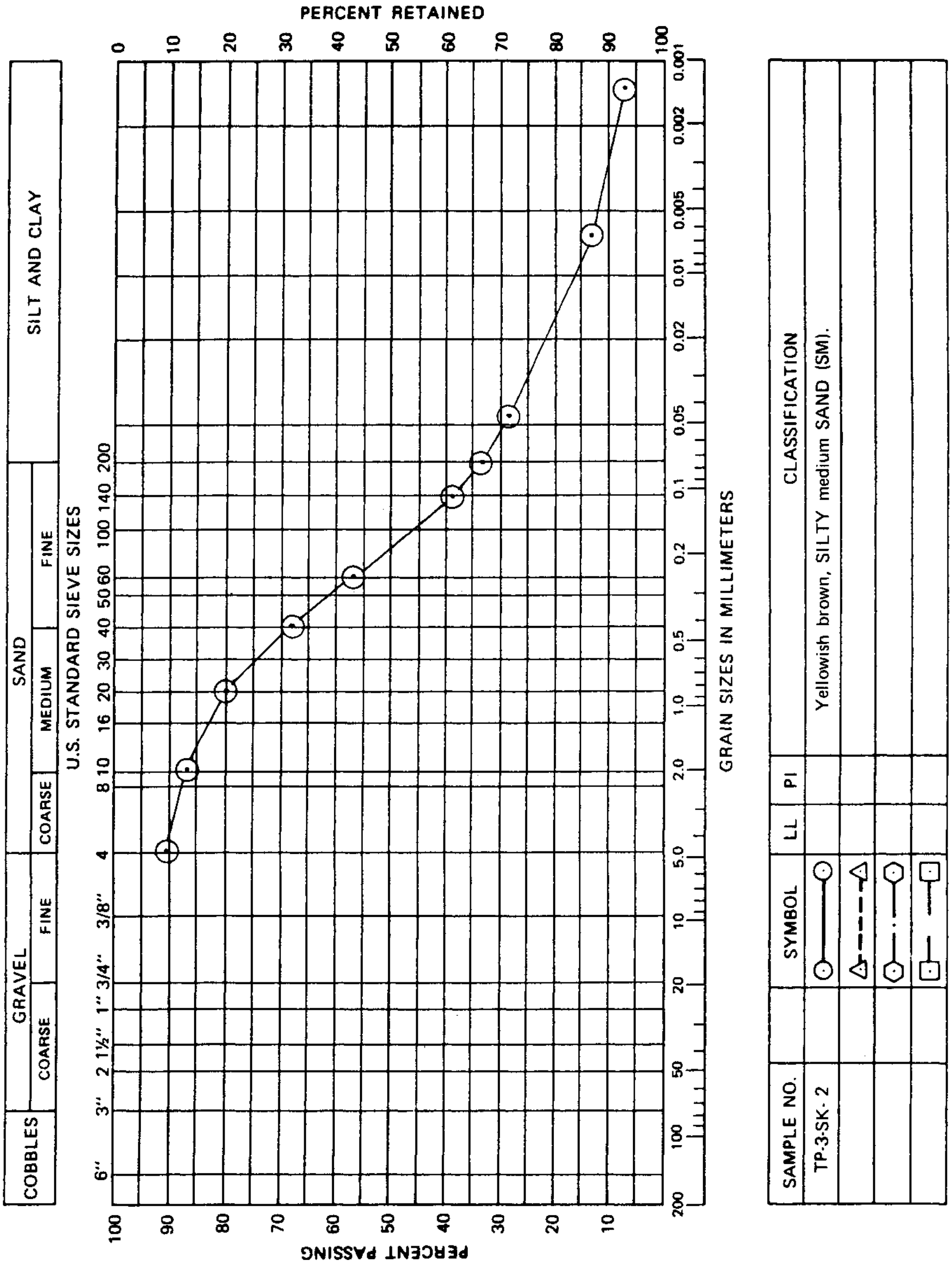
WOODWARD–CLYDE CONSULTANTS



SAMPLE NO.	SYMBOL	LL	PI	CLASSIFICATION
TP-3-SK-1				Grayish brown well graded GRAVEL with SAND (GW).

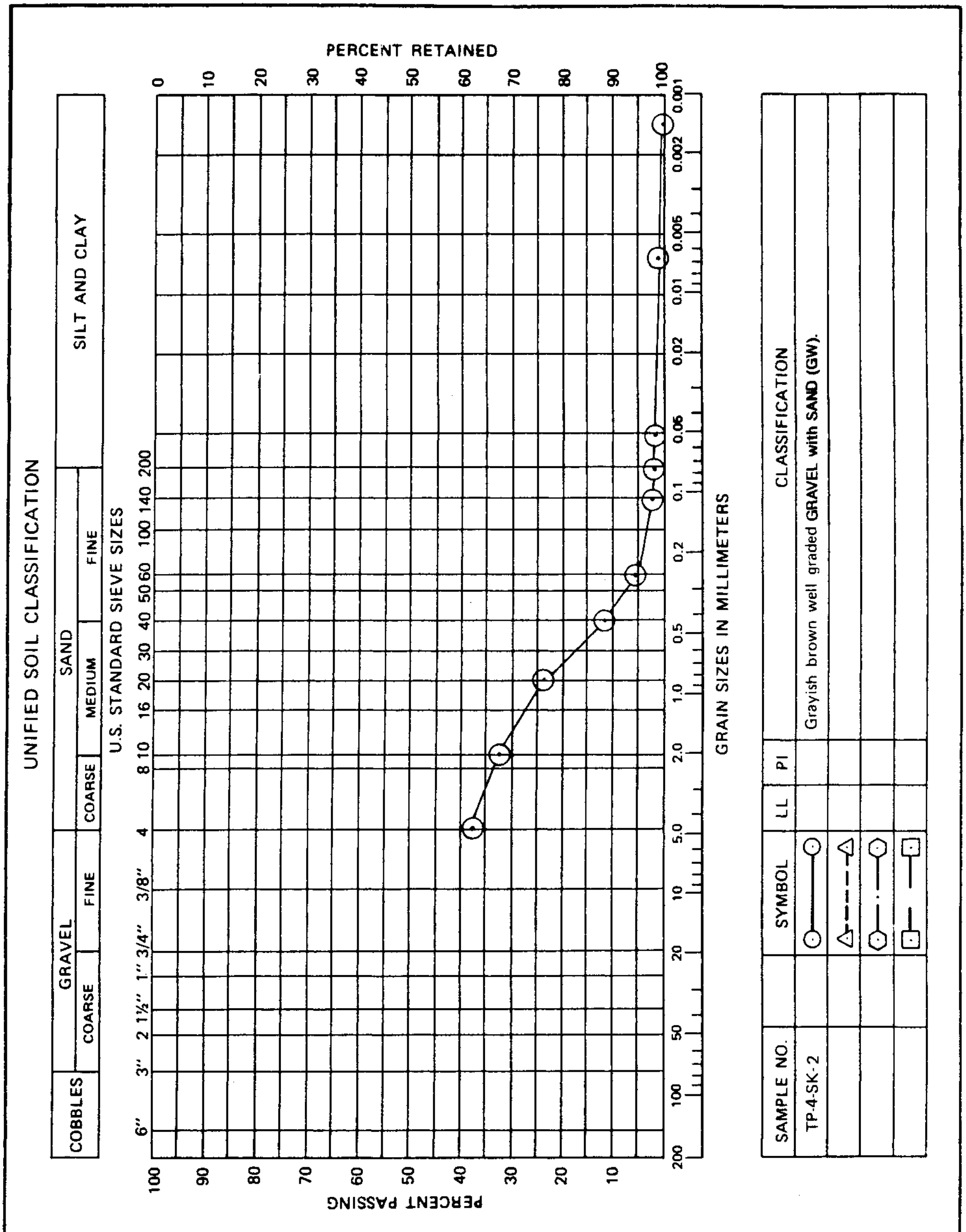
Project: SANTIAGO BASINS Project No. 8840269A	GRAIN SIZE DISTRIBUTION CURVES	Fig. A-7

WOODWARD–CLYDE CONSULTANTS



Project: SANTIAGO BASINS Project No. 8840269A	**GRAIN SIZE DISTRIBUTION CURVES**	Fig. A-8

WOODWARD–CLYDE CONSULTANTS



Project: SANTIAGO BASINS Project No. 8840269A	GRAIN SIZE DISTRIBUTION CURVES	Fig. A-9

WOODWARD-CLYDE CONSULTANTS

docusign

Certificate Of Completion

Envelope Id: 02CE183B-8EDA-40EF-86F6-66E3D5587511 — Status: Completed

Subject: Complete with Docusign: Addendum No.1 - Bond Basin Repair

Source Envelope:

Document Pages: 3	Signatures: 1	Envelope Originator:
Certificate Pages: 1	Initials: 0	Melody Wu
AutoNav: Enabled		PO Box 8300
EnvelopeId Stamping: Enabled		nil
Time Zone: (UTC-08:00) Pacific Time (US & Canada)		Fountain Valley, CA 92728
		mwu@ocwd.com
		IP Address: 67.52.122.245

Record Tracking

Status: Original 3/6/2025 3:51:44 PM	Holder: Melody Wu mwu@ocwd.com	Location: DocuSign

Signer Events	Signature	Timestamp
john kennedy jkennedy@ocwd.com General Manager Security Level: Email, Account Authentication (None)	 Signature Adoption: Pre-selected Style Using IP Address: 67.52.122.245	Sent: 3/6/2025 3:57:32 PM Viewed: 3/7/2025 8:30:33 AM Signed: 3/7/2025 8:30:42 AM

Electronic Record and Signature Disclosure:
Not Offered via Docusign

In Person Signer Events	Signature	Timestamp

Editor Delivery Events	Status	Timestamp

Agent Delivery Events	Status	Timestamp

Intermediary Delivery Events	Status	Timestamp

Certified Delivery Events	Status	Timestamp

Carbon Copy Events	Status	Timestamp

Witness Events	Signature	Timestamp

Notary Events	Signature	Timestamp

Envelope Summary Events	Status	Timestamps
Envelope Sent	Hashed/Encrypted	3/6/2025 3:57:32 PM
Certified Delivered	Security Checked	3/7/2025 8:30:33 AM
Signing Complete	Security Checked	3/7/2025 8:30:42 AM
Completed	Security Checked	3/7/2025 8:30:42 AM

Payment Events	Status	Timestamps