INFORMATION HANDOUT

For Contract No. 08-0Q3004
At 08-SBd-138-R17.1/R19.2

Identified by
Project ID 08-00020191

STRUCTURE REPORTS

1. Final Hydraulics Report, dated December 20, 2013
2. Foundation Recommendation for Hog Ranch Creek Bridge, dated December 23, 2013
3. Revised Foundation Report for Miner's Shack Creek Bridge, dated February 4, 2014
4. Foundation Report for Double Drain Creek Bridge, dated December 20, 2013
5. Seismic Design Recommendations for Hog Ranch Creek Bridge, dated November 27, 2013
6. Seismic Design Recommendations for Miner's Shack Creek Bridge, dated November 27, 2013
7. Seismic Design Recommendations for Double Drain Creek Bridge, dated November 27, 2013
8. P-Y Curves at Pier 2, Miner's Shack Creek Bridge, dated November 22, 2013
9. Materials Report by JASA, dated May 29, 1996 [Preliminary Geotechnical Design Report]
10. Additional Geotechnical Data by JASA, dated September 17, 1996
11. Supplemental Geotechnical Design Report by AGRA, dated December 20, 1996
12. Addendum to Geotechnical Design Report, by AGRA, dated 9-16-1997
13. Geotechnical Design Report by Caltrans, dated May 2003
14. Metrification of Geotechnical Design Report by Caltrans, dated August 12, 2003
15. Infiltration/Detention Basin Percolation Tests by Caltrans, dated January 22, 2004
16. Materials Report by Caltrans, dated December 30, 2013
17. Materials Report Correction E-mail by Caltrans, dated August 26, 2014
18. Revised Hydraulic Report Hog Ranch 11-5-13
19. Foundation Report Miners Shack 12-19-13

State of California
DEPARTMENT OF TRANSPORTATION

Business, Transportation and Housing Agency

Memorandum

Flex your power!
Be energy efficient!

To: GORDON DANKE
Office of Structure Design–West
Design Branch 9

Date: December 20, 2013

File: 08-SBd-138-R18.1
08-0Q3001
Proj. ID: 0800020191
Double Drain Creek Br.
Br. #54-1220

Attention: Pete Norboe

From: **DEPARTMENT OF TRANSPORTATION**
DIVISION OF ENGINEERING SERVICES
GEOTECHNICAL SERVICES
OFFICE OF GEOTECHNICAL DESIGN – SOUTH 2
DESIGN BRANCH B, MS #5

Subject: Foundation Report for Double Drain Creek Bridge

This report presents the foundation recommendations for the proposed Double Drain Creek Bridges (Br. No. 54-1220). The proposed Double Darin Creek Bridge was originally designed in 2003 using Working Stress Design (WSD) for abutments and Load Factor Design (LFD) for piers. Due to lack of funding, the project was put on hold until recently. The design of the bridge is being updated to the current AASHTO Load and Resistance Factored Design (LRFD) bridge specifications with California Amendments and the contract plans being converted from metric to english units.

This report supersedes previously generated Foundation Report, dated October 28, 2003. The following foundation recommendations are based on subsurface information gathered during the 2002 subsurface investigation performed by Caltrans. All elevations referenced within this report are based on the NGVD 1929 vertical datum, unless otherwise indicated.

Project Description

The proposed bridge site is located on newly aligned Route 138, located approximately 2.4 miles east of Interstate 15 in Cajon Pass in San Bernardino County. The proposed Double Drain Creek Bridge is to be approximately 670.0 ft long and consists of a 4-span, cast-in-place, prestressed, concrete box girder structure with seat abutments.

Site Geology

The "Geologic Map of the Cajon Quadrangle" (Dibblee, 2003) indicates the site is underlain by Quaternary Alluvium deposits which overlie Crowder Formation, which Dibblee describes as a fine to coarse, arkosic, pebbly sandstone.

State of California – Department of Transportation
Division of Engineering Services
Structure Design Services

FINAL HYDRAULIC REPORT

Hog Ranch Creek Bridge
08-SBD-138-R16.8, R18.7
EA 08-437001

Prepared by:

Jimmie Pallares, PE
Structure Hydraulics and Hydrology
December 20, 2013

REGISTERED PROFESSIONAL ENGINEER
JIMMIE PALLARES
C 65225
EXP. 9/30/15
CIVIL
STATE OF CALIFORNIA

This report has been prepared under my direction as the professional engineer in responsible charge of the work, in accordance with the provisions of the Professional Engineers Act of the State of California.

This final hydraulic report revision supersedes the original final hydraulic report dated March 18, 2003 for the Hog Ranch Creek Bridge project, which includes three new bridge structures. The three structures are Hog Ranch Creek Bridge (bridge number 54-1218), Miners Shack Creek Bridge (bridge number 54-1219), and Double Drain Creek Bridge (bridge number 54-1220).

Hydrology & Hydraulics Report

General

It is proposed to construct three new bridges on the new State Route138 alignment. The new alignment is located north of the existing SR 138 and is approximately 1.7 miles east of the State Route 138/15 Separation. The new SR 138 will reduce traffic delays, congestion, and improve safety. The new Hog Ranch Creek, Miners Shack Creek, and Double Drain Creek bridges will be 2-lanes prestressed box girders and will allow wildlife under crossing.

Hog Ranch Creek Bridge (Br. No. 54-1218, 08-SBd-138-PM R16.8)

The single span bridge dimensions are 147'-6" long by 42'-10" wide by 6'-6" deep will replace the existing 3-foot diameter corrugated metal pipe culvert on the existing SR 138. The fill at the culvert site will be removed to allow unimpeded flow of Hog Ranch Creek.

Miners Shack Creek Bridge (Br. No. 54-1219, 08-SBd-138-PM R17.2)

The dimensions of the double span bridge are 249'- 4" long by 42'-10" wide by 5'-0" deep. The bent will consist of two 5.5 feet diameter columns that are placed on 7 feet diameter CIDH piles. The columns will be placed within the Miners Shack Creek channel bed. There is a 38.58 degree hydraulic skew between the columns and the stream centerline.

Double Drain Creek Bridge (Br. No. 54-1220,, 08-SBd-138-PM R17.8)

The dimensions of the four span bridge are 669'-4" long by 42'-11.5" wide by 6'-6" deep. The bents consist of two 5.5 feet diameter columns that are placed on 7 feet diameter CIDH piles.

This final hydraulic report makes references to data found in the Federal Emergency Management Agency Flood Insurance Study titled "SAN BERNARDINO, CALIFORNIA" volume 1 through 4, dated January 17, 1997; Environmental Protection Agency website at www.epa.gov

Note: Elevations for this report are based on CAiCE file 1110 which was provided by Paul Phan, of District 8, and the original ground elevations used for scour elevations were provided by Pete Norboe, of DES. Please verify datum references on the new bridge layouts and make elevation adjustments as required.

Basin

The bridge sites are located within the Mojave Watershed (USGS Hydrologic Unit Code 18090208) which is part of the South Lahontan Hydrologic Region. The project site is located in the San Bernardino Mountains adjacent to the Cajon Summit.

The before mentioned FEMA Flood Insurance Study states that the San Bernardino Mountain resort and residential areas located in the canyons experience major problems of high velocity flood flows in the steep channels, which results in the occurrence of extreme amounts of erosion and deposition.

The area upstream of the project site is undeveloped with poor ground cover typical of the area. During the spring months, the ground cover consist of grasses and buck brush. During the rest of the year the ground is typically dry with little ground cover. The U.S. Forest Service controls most of the land with no consideration given for development. The table below list basin areas for the streams.

Stream	Basin Area (acres)
Hog Ranch Creek	358
Miners Shack Creek	289
Double Drain Creek	198

The annual rainfall for the project site ranges from 14 to 20 inches.

Streambed

The riverbeds are composed of silty sand with gravel and cobbles, fine to coarse-grained sand, and fine-grained sub angular to angular gravel round cobbles. Also, the bed has organics (alluvium).

Discharge

The discharges for the bridge sites were computed using Regional Regression Method in the U.S.G.S. Water-Resources Investigations Manual 77-21 and the standard statistical procedures as specified by the Water Resource Council Bulletin 17B for flood frequency analysis.

The table below lists the 50-year and 100-year flowrate events for the streams.

Stream	Q_{50} (cfs)	Q_{100} (cfs)
Hog Ranch Creek	473	717
Miners Shack Creek	406	611
Double Drain Creek	318	473

Velocity and Stage

HEC-RAS Version 4.1, a one-dimensional, step backwater, computer program developed by the U.S. Army Corps of Engineers, was used to model the bridge sites and their contributing waterways. This program estimates the flow velocities and water surface elevation (WSEL) for any given discharge. The model was based on the before mentioned CAiCE file. A Manning's roughness coefficient of 0.025 was based on site photographs and was applied to the channels.

The table below lists velocities and WSEL elevations for the streams.

Stream	Average V_{100} (ft/s)	Q_{50} WSEL Elevation (ft)	Q_{100} WSEL Elevation (ft)
Hog Ranch Creek	20.6	3355	3356
Miners Shack Creek	11.4	3428	3429
Double Drain Creek	13.6	3526	3527

Scour and Channel Degradation

The stream channels are very unstable. High local pier scour and channel degradation potential are probable due to high velocities and channel bed materials. The potential for thalweg migration is high. The table below lists the calculated scour depths for each structure. The scour elevations were based on the channel ground elevations (GE) provided by Pete Norboe, of DES, and the calculated scour depth.

Structure	Degradation (ft)	Contraction (ft)	Local (ft)	Total (ft)	GE (ft)	Scour Elev. (ft)
Miners Shack (54-1219)	3	.66	19.1	22.8	3426	3403
Hog Ranch (54-1218)	3	0	0	3	3355	3352
Double Drain (54-1220)	3	.5	4.5	8	3525	3517

Drift

Photographs and type of vegetation indicate that there is a small potential for debris at the project site.

Minimum Soffit Elevation

The freeboard distance is measured from the bridge soffit elevation to the 100-year water surface elevation (WSEL). Soffit elevations were provided by Pete Norboe, of DES.

Structure	Soffit Elev. (ft)	Q_{100} WSEL (ft)	Freeboard (ft)
Miner's Shack (54-1219)	3469.25	3429	40.25
Hog Ranch (54-1218)	3377.73	3356	21.73
Double Drain (54-1220)	3596.68	3527	69.68

There are no freeboard concerns.

Bank Protection

Due to the degradable characteristics of the site soils Structure Hydraulics recommends Rock Slope Protection (RSP) be placed at both abutments for Miner's Shack Creek and Double Drain Creek. The new rock slope protection (RSP) will be designed by District. The velocities have been provided to assist the District hydraulics engineers in the design of bank protection.

Summary Information for the Bridge Designer

Below is a summary of key design parameters based on the hydrology and hydraulic analysis performed for each structure:

Hog Ranch Creek Bridge

Minimum Soffit Elevation	3358 feet
Potential Scour Elevation at Piers/Abutment	3352 feet
Required Waterway	34.8 feet^2
Average Velocity	20.6 feet/s

HYDROLOGIC SUMMARY			
Drainage Area: 358 acres			
	Design Flood	Base Flood	Overtopping
Frequency (yrs)	50	100	>>500
Discharge (cfs)	473	717	
Water Surface Elevation at Bridge (ft)	3355	3356	3381.27
Flood plain data are based upon information available when the plans were prepared and are shown to meet federal requirements. The accuracy of said information is not warranted by the State and interested of affected parties should make their investigation.			

LongTerm Scour Depths,		
Support	Degradation Scour Depth	Contraction Scour Depth
Abut 1 & 2	3	0
Scour Data (Elevation and Depth),		
Support	Long Term Scour Elevation	Short Term (Local) Scour Depth
Abut 1 & 2	3352	0

Suggest District provide approach bank fill protection.

Miner's Shack Creek Bridge

Minimum Soffit Elevation	3431 feet
Potential Scour Elevation at Piers/Abutment	3403 feet
Required Waterway	53.5 $feet^2$
Average Velocity	11.4 feet/s

HYDROLOGIC SUMMARY			
Drainage Area: 358 acres			
	Design Flood	Base Flood	Overtopping
Frequency (yrs)	50	100	>>500
Discharge (cfs)	406	611	
Water Surface Elevation at Bridge (ft)	3428	3429	3469.23
Flood plain data are based upon information available when the plans were prepared and are shown to meet federal requirements. The accuracy of said information is not warranted by the State and interested of affected parties should make their investigation.			

LongTerm Scour Depths,		
Support	Degradation Scour Depth	Contraction Scour Depth
Bent 2	3	.66

Scour Data (Elevation and Depth),		
Support	Long Term Scour Elevation	Short Term (Local) Scour Depth
Bent 2	3422.3	19.1

Double Drain Creek Bridge

Minimum Soffit Elevation	3529 feet
Potential Scour Elevation at Piers/Abutment	3517 feet
Required Waterway	34.8 feet^2
Average Velocity	13.6 feet/s

HYDROLOGIC SUMMARY			
Drainage Area: 358 acres			
	Design Flood	Base Flood	Overtopping
Frequency (yrs)	50	100	>>500
Discharge (cfs)	318	473	
Water Surface Elevation at Bridge (ft)	3526	3527	3584.09
Flood plain data are based upon information available when the plans were prepared and are shown to meet federal requirements. The accuracy of said information is not warranted by the State and interested of affected parties should make their investigation.			

LongTerm Scour Depths,		
Support	Degradation Scour Depth	Contraction Scour Depth
Bent 3	3	.5

Scour Data (Elevation and Depth),		
Support	Long Term Scour Elevation	Short Term (Local) Scour Depth
Bent 3	3521.5	4.5

Lazaro, Ramon E@DOT

From: Ahmed, Kazi F@DOT
Sent: Tuesday, August 26, 2014 10:42 AM
To: Lazaro, Ramon E@DOT
Cc: Raouf, Mustapha Y@DOT; Phan, Paul@DOT; Kean, Bruce W@DOT
Subject: RE: EA 08-0Q3001 - Project Report Info Update/Verification

There is no need to have an updated Materials Report. As the SSP 39-1.23 for HMA-Type C is not available, instead use Type A HMA and there would be no change in the HMA thickness. We appreciate if you can send us the revised 95% PS&E package for our 2nd review and comments. Thanks.

Kazi Ahmed, PE
Caltrans District 8
Construction Division, Materials Engineering Branch
MS 860 (909) 888-2090

State of California

Business, Transportation and Housing Agency

Memorandum

To : **Renee Sasse, Senior Engineer**
Attention: Paul Phan

Date : **Dec 30, 2013**

File No: **08-SBd-138 PM R17.1/R19.2**
Construct Two Lanes New Alignment
EA: 0Q300 PN: 0800020191

From: **DEPARTMENT OF TRANSPORTATION**
Bruce W. Kean, District 8 Materials Engineer

Materials Report

1.0 GENERAL

1.1 Proposed Improvements

This project proposes to realign SR-138 from 1.7 miles east of Interstate 15 (I-15) to 0.2 mile west of Summit Post Office Road near the City of Hesperia. This new alignment will be constructed with two 12-foot lanes and 8-foot shoulders in each direction. The project scope also includes construction of temporary detour in three different locations within the project limit, installation of new culverts for drainage systems, and AC dikes.

1.2 Existing Facility

Within the project limit, SR-138 is a mountainous conventional two-lane highway with no paved shoulders and little or no earth shoulders. Existing drainage facilities at major floodways within the project limits consist of culverts to pass minor runoff, with major flows overtopping the roadway resulting in lengthy road closures. The existing pavement shows signs of distress and cracks. The existing facility within the project limit consists of asphalt concrete (AC) mainline which would be removed after the construction of the proposed new alignment.

1.3 Climate and Geology

The average annual rainfall in the project area ranges from ten to forty inches. The rainfall occurs during the months of November through May. The annual temperature range is from 22 to 116 degrees Fahrenheit. Elevations range from 3,270 to 3,687 feet.

John A Sayers & Associates (JASA) indicates in a Materials Report for Proposed Highway Construction Rte 138 in 1996 that the project area is underlain by sandstone, fanglomerate and conglomerate of the

Crowder Formation, older alluvium, and terrace deposits. These materials are light grey to slightly pink, grey to white and light reddish brown respectively. Each formation has similar characteristics. In effect, they consist of sands and gravels and are very dense. A thin soil cover mantles most of the natural slopes and recent alluvium and wash deposits exist in gullies and canyons. A relatively thick alluvial deposit exists at the east end of the project. The west end of the project is approximately 2.0 miles northeast of the seismically active San Andreas Fault.

1.4 Ground Water

According to the Geotechnical Consultants JASA's Materials Report (1996), generally, there is little or no ground water within the project area except at great depth. Locally, in two boring locations (Borings 1 and 2), perched groundwater was encountered at shallow depths. This would affect excavations and embankment placement. JASA recommended subdrains wherever groundwater is encountered or expected. Surface water would be expected during or after rainstorms.

2.0. PAVEMENT DESIGN PARAMETERS

2.1 Traffic Index (T.I)

The following Traffic Index (TI) values were provided in the Memorandum dated August 12, 2013 from the Office of Forecasting.

Forecasted Period	**SBd-138 PM R17.1/R19.2**	
	Mainline + first 2' of Shoulder	**Shoulder**
20-Year T.I.	8.5	5.5
40-Year T.I.	9.5	6.0

In addition to the TI values, forecasted ADT was also provided:

	Year 2013	**Year 2017**	**Year 2034**	**Year 2054**
Annual Average Daily Traffic (AADT)	4,500	4,800	5,900	7,100
Design Hour Volume (DHV)	450	450	490	580
Truck Daily Traffic (ADT%)	13%	13%	13%	13%
Truck Design Volume (DHV%)	7%	6%	6%	6%

Using the above 20-Year TI (8.5) and the HDM Table 613.3C "Conversion of ESAL to Traffic Index", the TI for the Detour to be used during the construction phase is estimated as 6.0.

2.2 R-value for Basement Soil

Materials Report for Proposed Highway Construction between I-15 and Summit Valley on State Rte 138 PM R15.2/R19.0 in San Bernardino County, EA 08204-066611, and dated October, 1971 indicates that the soils consist of generally gravelly sands and sandy gravel with pebbles and cobbles. It also indicates that near the east end of the project, a few lenses of silt will be encountered in excavation which can either be placed in the lower elevations of the embankment or mixed thoroughly with good soil material and used. That Materials Report recommends to use R-value of 55 for any imported borrow.

Materials Report prepared by John A. Sayers & Associates (JASA) for the EA:08-268311 Route SBd-138 PM 16.5/19.7 dated May 29, 1996 indicates that most on site soils have R-values greater than 50, although soils collected from a Boring (apx. PM 18.8) had an R-value of less than 20 which represents the more clayey portion of Crowder Formation. JASA recommends that these Clayey soils should be removed from the cut sections and not be placed in fill sections within 4 feet of finish grade. They suggest an R-value of 50 for structural sections.

In the District 08 DME Memo "Responding to the Request date 8/15/02" dated August 20, 2002, EA: 437001, SBd-138 PM R16.8/R18.7, it was recommended to use R-value of 40 for the entire length of the project based on the recorded documents and the preliminary project layout plans.

Based on this information, we recommend that if cohesive (silt and clay) soils are exposed in cut road bed sections, overexcavation may be necessary to improve the roadway section. If necessary, overexcavations in roadbeds should be four feet below subgrade and replaced with a sandy soil. A conservative R-value of 40 will be selected for the project design.

2.3 Pavement Design Life

For the pavement design of the Detour, we will provide temporary pavement design using TI as 6.0. In the Memorandum from the Office of Forecasting dated August 12, 2013, the Average Daily Traffic (ADT) for the Year 2034 is 5,900. Based on this information and according to Caltrans Highway Design Manual, Section 612.2, the minimum pavement design life for New Construction and Reconstruction of the Mainline Traveled way within the project limit should be 20 or 40-year and the one with the lowest life-cycle cost should be selected.

3.0 PAVEMENT STRUCTURAL SECTIONS

Rigid (JPCP) structural sections for the new alignment for SR-138 are shown below. JPCP is recommended for travel lane and shoulder to provide longer term resistance to continuous truck acceleration and deceleration as well as stress from braking and oil dripping. As the project area is located in a mountainous region and at higher elevation (more than 3000'), JPCP is also very effective against tire chain wearing during icy winter. However should the cost of JPCP proves prohibitive, then all other alternatives are listed below. We have provided pavement designs for the 20 and 40-Year design life. Rubberized Hot Mix Asphalt Type G (RHMA-G) is also offered to comply with Caltrans

guidelines to use RHMA-G as much as possible, if feasible. Designer may select the one which is the most cost effective based on the LCCA analysis results.

As the Highway Design Manual (HDM), Section 613.5 (page 610-10, dated Nov 2, 2012) says "The total depth of the shoulder pavement structure (depth from the surface to the subgrade) shall match the pavement structure grading plane of the adjacent traffic lane", so in preparing the shoulder structural sections, the depth of the shoulder base and/or subbase has been increased in order to match the grading plane of the shoulder pavement structure to that of the adjacent traffic lane. This will provide a path for water in the pavement structure to drain away from the lane and into the shoulder. It can also provide a more cost effective means to upgrade the shoulder to a traffic lane in the future. It is important to mention that Project Engineer should refer to Caltrans HDM Topic 613.5 (2) (b) to determine whether the shoulder shall have the same structural section as the adjacent mainline.

3.1 Alternative 1: JPCP and LCB

Utilizing the current TI values (Section 2.1) and the R-value (from historical research, Section 2.2), we recommend the following rigid pavement structural section for the mainline and shoulder for the proposed project. In determining the pavement structural sections, we have used the procedure described in Section 623.1 of the July 1, 2008 Edition of the HDM. This procedure utilized "Type II" soil, "South Mountain" climate region, and Table 623.1 (K).

Structural Section	**20-Yr Design**		**40-Yr Design**	
	Mainline + first 2' of Shoulder (TI=8.50)	**Shoulder (TI=5.50)**	**Mainline + first 2' of Shoulder (TI=9.50)**	**Shoulder (TI=6.0)**
JPCP	0.75'	0.75'	0.80'	0.75'
HMA Type A	0.10'	0.10'	0.10'	0.10'
Lean Concrete Base (LCB)	0.50'	0.50'	0.50'	0.50'
Class 2 Aggregate Subbase (AS)	0.50'	0.50'	0.50'	0.55'

3.2 Alternative 2: JPCP and AB

Structural Section	**20-Yr Design**		**40-Yr Design**	
	Mainline + first 2' of Shoulder (TI=8.50)	**Shoulder (TI=5.50)**	**Mainline + first 2' of Shoulder (TI=9.50)**	**Shoulder (TI=6.0)**
JPCP	0.75'	0.75'	0.85'	0.75'
Class 2 Aggregate Base (AB)	1.00'	1.00'	1.00'	1.10'

3.3 Alternative 3: Flexible Pavement

Pavement sections shown below were obtained employing CalFP version 1.1, a computer program based on design methodology as documented in Chapter 630 of the Caltrans Highway Design Manual (HDM).

Option 1: HMA-Type C

Structural Section	**20-Year**		**40-Year**	
	Mainlines + first 2' of Shoulder (TI=8.50)	**Shoulder** (TI=5.50)	**Mainlines + first 2' of Shoulder** (TI=9.50)	**Shoulder** (TI=6.0)
RHMA-G	-	-	0.10'	0.10'
Hot Mix Asphalt Type C (HMA-C)	0.50'	0.40'	0.90'	0.55'
Class 2 Aggregate Base	0.60'	0.70'	0.50'	0.85'

Option 2: Rubberized HMA-Type G (RHMA-G)

Structural Section	**20-Year**		**40-Year**	
	Mainlines + first 2' of Shoulder (TI=8.50)	**Shoulder** (TI=5.50)	**Mainlines + first 2' of Shoulder** (TI=9.50)	**Shoulder** (TI=6.0)
RHMA-G	0.20'	0.20'	0.20'	0.20'
HMA-Type C	0.30'	0.20'	0.80'	0.45'
Class 2 Aggregate Base	0.60'	0.70'	0.50'	0.85'

3.4 Detour Pavement Structural Sections

A flexible pavement design is recommended. Pavement sections shown below were obtained employing CalFP version 1.1, a computer program based on design methodology as documented in Chapter 630 of the Caltrans Highway Design Manual (HDM).

R-value 40

Traffic Index (TI)	**Pavement Structural Section**
6.0	0.40' HMA-Type A over 0.35' Class 2 Aggregate Base

4.0 CULVERTS & CORROSION POTENTIAL

The project scope includes the installation of culverts of various types and sizes (12", 18", 24", 36", 42", 48", 60", 72", 84" and 90" diameter) in order to ensure proper drainage systems within the project limit.

Section 4.1 of Caltrans "Corrosion Guidelines" dated September 2003, considers a site to be corrosive if one or more of the following conditions exist for the representative soil and/or water samples taken at the site: Chloride concentration is 500 ppm or greater, Sulfate concentration is 2000 ppm or greater, or the pH is 5.5 or less. In general, a minimum resistivity value for soil and/or water less than 1000 ohm-cm indicates the presence of high quantities of soluble salts and a higher propensity for corrosion.

Materials Report prepared by John A. Sayers & Associates for the EA:08-268311 Route SBd-138 PM 16.5/19.7 dated May 29, 1996 indicated the corrosivity characteristics of the existing soil. According to that Materials Report, resistivity of soil ranged between 3,960 to 14,250 ohm and pH ranged between 7.43 to 8.90. Based on the soils' pH and resistivity values and using the computer program AltPipe, the following is recommended to achieve a 50-year service life culvert:

4.1 Recommendations for Steel Pipe

Thickness for Corrugated Steel Pipe (CSP) due to Structural Overfill Requirement

Minimum Thickness Gauge (inches)	CSP with Helical Corrugations (22/3" x ½")	CSP with Helical Corrugations (3" x 1")	CSP with Helical Corrugations (5" x 1")	CSP with Annular Corrugations (22/3" x ½")
18 (0.052)	12" to 42" size allowable	-	-	-
16 (0.064)	48" size allowable	48" to 90" size allowable except 54"	48" to 90" size allowable	18" to 48" size allowable
12 (0.109)	60" size allowable	-	-	60" size allowable
10 (0.138)	66" & 72" size allowable	-	-	66" & 72" size allowable
8 (0.168)	78" & 84" size allowable	-	-	78" & 84" size allowable

Thickness for other Steel Pipe due to Structural Overfill Requirement

Minimum Thickness Gauge (inches)	Steel Spiral Rib Pipe with 3/4" x 1" Rib at 11 ½" Pitch	Steel Spiral Rib Pipe with 3/4" x 1" Rib at 8 ½" Pitch	Steel Spiral Rib Pipe with 3/4" x 1" Rib at 7 ½" Pitch	Structural Steel Plate Pipe with 6" x 2" Corrugations
16 (0.064)	24" to 60" size allowable except 27" & 45"	24" to 72" size allowable except 27" & 45"	24" to 54" size allowable except 27" & 45"	-
14 (0.079)	66" & 72" size allowable	78" & 84" size allowable	60" & 66" size allowable	-
12 (0.109)	78" to 90" size allowable	90" size allowable	72" to 84" size allowable	60" to 90" size allowable

In order to provide adequate abrasion resistance to meet or exceed a 50-year design service life, steel pipe selected from the Tables in the above Section 4.1 should also meet the following requirement:

a) 48" to 90" size steel pipe should be 16 gauge (0.064") or thicker and should be or have one of the following:

- plain galvanized
-aluminized steel pipe (Type 2)
-composite SSRP
- bituminous coating (hot dipped)
- bituminous coating & paved invert
-polymerized asphalt invert
-polymeric sheet coating

b)12" to 42" size steel pipe should be 18 gauge (0.052") or thicker and should be or have one of the following:

- aluminized steel pipe (Type 2)
- bituminous coating (hot dipped)
- bituminous coating & paved invert
- polymerized asphalt invert
- polymeric sheet coating.

c)12" to 42" size plain galvanized steel pipe should be 16 gauge (0.064") or thicker.

d)36" & 42" size composite SSRP should be 16 gauge (0.064") or thicker.

4.2 Recommendations for Aluminium Pipe

Thickness due to Structural Overfill Requirement

Min. Thickness Gauge (inches)	Corrugated Aluminium Pipe with Annular Corrugations (22/3" x ½")	Corrugated Aluminium Pipe with Annular Corrugations (3" x 1")	Corrugated Aluminium Pipe with Helical Corrugations (22/3" x ½")	Corrugated Aluminium Pipe with Helical Corrugations (3" x 1")	Structural Aluminium Plate Pipe (9" x2 ½")	Aluminium Spiral Rib
16 (0.064)	12" to 24" size allowable	30" to 72" size allowable except 45"	12" to 24" size allowable	30" to 72" size allowable except 45"		24", 30" & 36" size allowable
14 (0.079)	30" & 36" size allowable	78" size allowable	30" & 36" size allowable	78" size allowable	-	42" size allowable
12 (0.109)	42" & 48" size allowable	84" & 90" size allowable	42" & 48" size allowable	84" & 90" size allowable	60" to 90" size allowable	48" to 60" size allowable
10 (0.138)	54" & 60" size allowable	-	54" & 60" size allowable	-	-	-
8 (0.168)	66" & 72" size allowable	-	66" & 72" size allowable			

4.3 Recommendations for Plastic Pipe

Corrugated HDPE (Type S)	Corrugated HDPE (Type C)	Ribbed HDPE	Ribbed PVC	Corrugated PVC
18" to 60" size allowable except 21" & 27"	18" & 24" size allowable	18 to 24" size allowable	18" to 48" size allowable	12" to 36" size allowable except 27"

4.4 Recommendations for Reinforced Concrete Pipe (RCP): Steel cover should be 1".

The above recommendations in Section 4.1, 4.2, 4.3 and 4.4 are to prevent corrosion only, and these recommendations are based on available laboratory test results and reasonable assumptions. For the computer calculations, the water flow velocity was assumed to be 5 fps with no-abrasive conditions and maximum 10' backfill over pipe. A reevaluation would be required if conditions differ from these assumptions.

5.0 LIFE CYCLE COST ANALYSIS (LCCA)

It is required that an LCCA be performed for this project. The LCCA is an effective and useful tool for comparing the value of alternative pavement structures and strategies. It can be used to compare life-cycle cost for:

- Different pavement types (rigid, flexible, composite)
- Different rehabilitation strategies
- Different pavement design lives

The LCCA must be conducted by the Project Engineer during the early stages of the project. Recommended structural sections in this Materials Report should be used in the various alternatives in the LCCA analysis. For information and guidance, please refer to Topic 619.1 of the Caltrans HDM, and to the "Life-Cycle Cost Analysis Procedures Manual", available at the Caltrans Pavement Engineering website shown below:

http://www.dot.ca.gov/hq/maint/Pavement/Offices/Pavement_Engineering/LCCA_index.html

6.0 EARTHWORK

Cut and fill slopes in the project area may subject to slope erosion. Any proposed cut or fill slopes should be grossly and surficially stable at the proposed heights and proposed gradients. Proper erosion control measures as per Highway Design Manual should be required. Installation of benches at various intervals, and downdrains at various locations may be necessary to control and minimize severe erosion. Any imported or local borrow required should conform to Section 19-7 of the Standard Specifications 2010 and the minimum R-value of 40 for all segments within the project limit.

7.0 MATERIALS SPECIFICATIONS

7.1 Earthwork

- Clearing and grubbing is recommended as per Section 16 of the Standard Specifications, to remove vegetation, topsoil, and any artificial fills or debris, and to prepare the site for the proposed facilities.
- Earthwork should conform to Section 19 of the Standard Specifications.
- The subgrades for paved areas should be compacted to a minimum relative compaction of 95%, as per Section 19-5.03B "Relative Compaction (95 Percent)" of the Standard Specifications.
- The subgrades to receive pavements should have a minimum R-value of 40 for all Segments within the project limit including any imported or local borrow.

7.2 Base Materials

- Aggregate Base (AB) should be Class 2 Aggregate Base, and conform to Section 26 of the Standard Specifications.
- Aggregate Subbase (AS) should be Class 2 Aggregate Subbase, and conform to Section 25 of the Standard Specifications.
- We prefer Lean Concrete Base (LCB) for the base layer in the JPCP structural sections. LCB should conform to Section 28-2 of the Standard Specifications.

7.3 Rigid Pavement

- Jointed Plain Concrete Pavement (JPCP), previously referred to as Portland Cement Concrete (PCC) pavement, shall conform to Section 40 of the Standard Specifications.
- Lean Concrete Base shall conform to Section 28-2 of the Standard Specifications.
- HMA-A for rigid pavement shall comply with 3/8-inch maximum aggregate grading, the additional one percent oil above optimal binder content. The grade of asphalt binder shall be PG 64-16.

7.4 Flexible Pavement

- **Hot Mix Asphalt (HMA):**

a) Type A (HMA-A): For ¾ inch aggregate grading, the minimum lift thickness is 0.20 foot and the maximum lift thickness is 0.35 foot. For ½ inch aggregate grading, minimum lift thickness is 0.15 foot and the maximum lift thickness is 0.20 foot.
b) Type C (HMA-C): For 1-inch aggregate grading, the minimum lift thickness is 0.25 foot and the maximum lift thickness is 0.45 foot. For ½ inch aggregate grading, minimum lift thickness is 0.15 foot and the maximum lift thickness is 0.25 foot.
c) Type G (RHMA-G): For ¾ inch aggregate grading, the minimum lift thickness is 0.20 foot and the maximum lift thickness is 0.20 foot. For ½ inch aggregate grading, the minimum lift thickness is 0.10 foot and the maximum lift thickness is 0.15 foot.

- **Asphalt Binder:**

  Caltrans is now using the performance graded (PG) system (replacing the "aged residue" or "AR" classification) for asphalt binder specification.

  a) HMA-Type A: PG 64-28 PM for ¾ inch aggregate grading
  b) HMA-Type A: PG 64-16 for ½ inch aggregate grading
  c) HMA-Type A (in rigid pavement): PG 64-16
  d) HMA-Type C: PG 64-28 PM
  e) RHMA-Type G: PG 64-16

- Prime Coat shall be applied to base material prior to placing hot mix asphalt concrete.
- Tack Coat shall be applied to in between the successive layers of HMA.

8.0 REFERECES

1. – Preliminary Materials Report for Proposed Construction of Two Lanes New Alignment, dated April 16, 2012, EA: 0Q300, SBd-138 PM R17.1/R19.2.

2. – District 08 DME Memo "Responding to the Request date 8/15/02" dated August 20, 2002. EA: 437001, SBd-138 PM R16.8/R18.7.

3. - Materials Report prepared by John A. Sayers & Associates for the EA:08-268311, Route SBd-138 PM 16.5/19.7, dated May 29, 1996.

4. - Materials Report for Proposed Highway Construction between I-15 and Summit Valley on State Rte138 PM R15.2/R19.0 in San Bernardino County. EA 08204-066611, dated October, 1971.

9.0 CLOSURE

Our findings and recommendations were obtained in accordance with generally accepted professional principles in Materials Engineering, and are based on the results of the historical research, and Caltrans Highway Design Manual guidelines. If any change (i.e., structure type, location, scope of the project etc.) is implemented which materially alters the project, our recommendations may need to be revised.

If you have any questions, you may call Kazi Ahmed at (909) 888-2090, or me at (909) 888-2029.

BWK: KA

file copy

State of California

Business, Transportation and Housing Agency

Memorandum

Save the trees!
Be energy efficient!

To: MS. CHRISTY CONNORS – 08
Design "N"

Attention: Mr. Chris Nguyen

Date: January 22, 2004

File: 08-SBd-138-KP R27.5/R30.9
Fr. 3.0 km E of Rte 15 to 0.2 km
W of Summit Post Office Rd

E.A.: 08-437001

From: **DEPARTMENT OF TRANSPORTATION**
DIVISION OF ENGINEERING SERVICES
Geotechnical Services, MS#5
Office of Geotechnical Design – South 2

Subject: Infiltration/Detention Basin Percolation Tests

1. Introduction

Per your request, we are providing percolation test results for an infiltration/detention basin proposed on a segment of State Highway 138 at KP 27.7 to 27.8 in San Bernardino County, California. District 08 Project Development is proposing to convert Route 138 from the existing two-lane rural route to a new 12 m realigned paved section from 3.0 km east of I 15 to 0.2 km west of Summit Post Office Road. The purpose of this report is to document subsurface geotechnical conditions and to provide engineering assessments to recommend criteria for the proposed basins.

2. Pertinent Reports and Investigations

The following is a list of documents, reports and maps that were utilized in preparing this report.

- Memorandum dated April 22, 2003 "Request for Infiltration Testing" from Ms. Christy Connors, Senior, Design "N", District 08 and its attachments.
- Preliminary Geotechnical Reports dated May 29, 1996 by Sayer & Assoc., Dec. 20, 1996 by AGRA, and Sept. 16, 1997 by AGRA.
- California Seismic Hazard Map prepared by Lalliana Mualchin, Caltrans Engineering Seismologist, dated 1996.
- Storm Water Quality Handbooks, Project Planning and Design Guide, Appendix B dated September 2002 prepared by Caltrans.
- Method for determining the percolation rate of soil using 6-inch-diameter-test hole, California Test 750-1986.

3. Existing Facilities and Proposed Improvements

Within this segment, Highway 138 is currently a two-lane rural route paved with asphalt concrete, built on fill, and aligned in a general east/west direction. It is hilly terrain with sparse vegetation, sandy washes and public utilities. There are several minor cuts on the alignment and one major cut at the eastern end of this segment.

A new alignment is proposed for a two-lane safety route adjacent to the old alignment. The storm water infiltration or detention basin is planned to filter and/or detain runoff on the western end of the alignment.

This report addresses the geotechnical issues associated with the construction of the basins and assessment of the local material. Geotechnical considerations for the structure foundations are not addressed in this report.

4. Physical Setting

4.1 Climate

This project is located in the inland "mountains and high desert" region. The climatic conditions at the project site are considered temperate with moderate winters and hot summers. Based on the climatic data available the range of temperature is from below freezing to during the winter months and to often over 100 degrees F during the summer months. Rainfall, which occurs predominately in the midwinter to midspring, averages 15 to 20 inches per year. This may come in the form of relatively large quantities of moisture in a short period of time and can be accompanied by very high winds. Flash floods may develop in normally dry to intermittent drainages and washes.

4.2 Topography and Drainage

The topography and drainage in the "basin area" is relatively flat. However, the topography above the basin site is hilly to steep on the east end. Drainages around the basin are wide and deep compared to the incised drainages on the east end. Large bedloads and high volume of water are possible in the basin area. Refer to Boring Logs, attached, for location of site.

4.3 Geology

The site is situated in the Cajon Basin within the Transverse Range geomorphic province of California. This province is a large east-west trending mountain range bounded by the Mojave Desert province to the north, the San Bernardino Valley to the south. The Geologic Map of San Bernardino quadrangle sheet, California, 1998 published by CDMG, indicates that the site is underlain by Quaternary sedimentary formations, which were deposited as fan deposits (gravel, sand, silt and clay) eroded from the mountains.

Uplift has occurred on the south side of the fans from the San Andreas Fault system uplift. Erosion has scraped out a basin of which Crowder Canyon Creek is the major drainage along Route 138 east of Interstate 15.

4.4 Seismicity

Based upon the Department's California Seismic Hazard Map, dated 1996, the controlling fault is the San Andreas Fault with a maximum credible earthquake moment magnitude of M_w=7.0 and the Cleghorn Fault of a Mw=7.75 event. The basin is located about 2 kilometers northeast of the faults. The Peak Horizontal Bedrock Acceleration, based on the above-mentioned map, is estimated to be 0.6g at the site. The potential for surface rupture at the site due to fault

movement is considered low since there are no known faults projecting towards or passing directly through the project site.

5. Groundwater

Groundwater in the project area originates from infiltration of creek water and rainwater through the alluvial fans. Borings were drilled to a maximum depth elevation of 1024.3 m (3360.6 ft.). Groundwater was not encountered above that elevation. The basin bottom elevation is 1033 m.

Additional information for the historical well data can be found on the DWR web site at: http://well.water.ca.gov/.

6. Subsurface Investigation

We performed our subsurface exploration program from December 8, 2003 to December 10, 2003. Five borings were drilled to depths up to 44 ft. below the existing ground surface. Table 3 presents the summary of the five borings performed.

The borings were drilled using a trailer-mounted "Mobile Drill" rig and advanced using a 152 mm (6 in.) diameter hollow stem auger. SPT (Standard Penetration Tests) were conducted with a 63.5 kg (140 lb.) safety hammer dropped 762 mm (30 in.). The number of hammer blows required to drive the sampler the last 300 mm (12 in.) into the soil formation was recorded and is presented on the boring logs. Samples recovered from the sampler were used to classify the soil types. Bulk samples of loose material were randomly collected using plastic bags at varying depths below the basin. The logs of the 5 borings and the associated legends are attached. There were three percolation tests performed and two water level wells drilled as shown in Table 1.

Table 1: Summary of Borings

Boring No.	Stationing (Proposed Rte.)	Offset from Proposed Rte. (m)	Surface Elevation (m)	Depth drilled (m)	Remarks
B 1-03	28+02	30 m Right	± 1035.3	4.1	Percolation Test
B 2-03	28+32	27 m Right	± 1037.0	5.5	Percolation Test
B 3-03	28+33	37 m Right	± 1036.5	12.2	Groundwater test
B 4-03	28+57	31 m Right	± 1038.0	13.4	Groundwater test
B 5-03	28+58	42 m Right	± 1037.0	6.1	Percolation Test

7. Subsurface Soil Conditions

Based on our field investigations, the near-surface materials encountered consist of medium dense to dense combinations of micaceous silty sand and clayey sand. In general, the materials are more non-cohesive than cohesive. A trace to minor gravel, cobbles and boulders are also present. Below these materials, at elevation 1025.2 m (3363.5 ft.) in Boring 3-03, is a stiff silty clay, and in Boring 4-03 at elevation 1027.7 m (3371.7 ft.) is a stiff silty sandy clay. All materials were in a dry to damp condition.

8. Laboratory Testing

The following laboratory tests were performed on selected samples obtained from the borings.

- CTM 202, 203 – Sieve Analysis and Mechanical Analysis
- CTM 204 – Atterberg Limits
- CTM 643 – pH
- EPA 9081 – Organic Content and Cation Exchange Capacity (NA)

The tests above were used in classifying the soils encountered. The test results are also attached. All samples just below basin bottom, elevation 1033 m (3389 ft.), were non-plastic.

Table 2: Test Summary

Boring No.	-#200 (% Passing)	Atterberg Limits	pH	Organic Content (%)	EPA 9081 Sodium, mg/kg
B1-03/8.5'-12'	17	NP	6.94	0.68	67.90
B2-03/13'-16.5'	22	NP	6.90	0.61	58.92
B3-03/20'-21.5'	25	NP	6.90	0.58	77.46
B4-03/25'-26.5'	17	NP	6.90	0.54	59.85
B5-03/14'-15.5'	13	NP	6.90	0.44	78.30

NP-Non plastic

9. Percolation Testing

Three borings were tested for percolation during our field investigation. Each percolation boring was drilled down to 1 m (5 ft.) below the basin bottom elevation of 1033 m (3389.1 ft.). A 1.875 in. (OD 47.6 mm, ID 38.1 mm) plastic pipe , with the bottom 1 m (5 ft.) perforated, was inserted into the hole. Clean pea gravel was filled up 1 m (5 ft.), then 0.3 m (1 ft.) of bentonite chips was added. Then the hole was backfilled with native and water added to stand overnight. The tests were performed the following day.

The test was performed by filling the holes with relatively clean water, then monitoring the rate in minutes at which the water surface dropped. Specifically, the beginning water level depth was measured against the drop in water until the drop leveled off and the stabilized percolation rates were obtained. Infiltration range according to the Storm Water Guide is 2.5 inches per hour (6.4 cm/hour) to 0.5 in. per hour (1.3 cm/hour).

Four percolation tests were performed for B1-03 and B2-03, and three tests were performed for B5-03. The tests were averaged for each boring. Boring B1-03, closest to the egress, averaged 0.54 inches/hour (1.37 cm/hr). Boring B2-03 averaged 0.40 in/hr (1.02 cm/hr). Boring B5-03, closest to ingress, averaged 0.37 in/hr (0.94 cm/hr). The overall average was 0.44 in/hr (1.12 cm/hr).

Testing was performed just after a rainy day and may have caused the numbers to be lower than normal. We note that relatively clean water was used to perform the tests above. However, highway

runoff water will likely contain silt, sand, oils, and/or other materials that would decrease the percolation rates. This is bordering the minimum acceptable infiltration rate of 0.5 in/hr (1.3 cm/hr).

10. Construction Considerations

We recommend that an overall infiltration rate of 0.44 in/hr (1.12 cm/hr) and a factor of safety of 2.0 be used when designing the infiltration or detention basin.

11. Notes

During the winter months, periods of ground freezing occur. During the time the ground is frozen, the percolation rates will be near zero and basins should be designed accordingly.

If excavation of the basin produces a basin bottom different from the 1033 m (3389 ft.) elevation, please contact this office for further investigation. Otherwise, no further fieldwork is planned.

If you have any questions or comments, please call Chris Hoadley at (916) 227-4515.

Prepared by: Date: 2/5/04

REGISTERED GEOLOGIST
CHRIS HOADLEY
No. 2309
Exp. 11/05
CERTIFIED ENGINEERING GEOLOGIST
STATE OF CALIFORNIA

Chris Hoadley, CEG No. 2309
Associate Engineering Geologist
Geotechnical Branch C

Reviewed by: Date: 2-5-04

Shawn Wei, P.E., Chief
Branch C
Office of Geotechnical Design – South 2

c: AAbghari
BKean-08
Project File

ROTARY FIELD NOTES

TL-1271a (REV. 01/31/00)

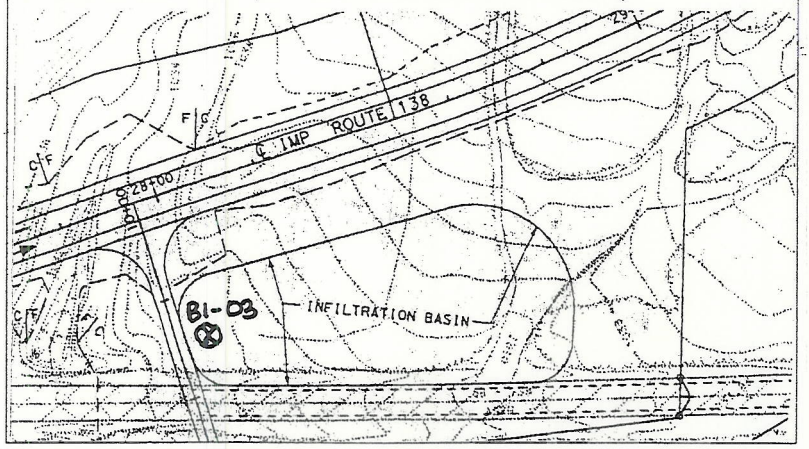
BORING NUMBER	DATE	DIST.	CO.	RTE.	~~P.M.~~ (K.P.)	BRIDGE #
B1-03	12/09/03	08-437001	SBd	138	27.6-27.7	

LOCATION (STA/OFFSET or NORTHING/EASTING)	BRIDGE NAME	EA NUMBER
Station 28+02, 30m Rt. CL		08-437001

TOP HOLE ELEVATION	CREW	EQUIPMENT	CHC NUMBER
1035.3m	Bob Eneix	B47	

HAMMER ID#
Standard

SITE LOCATION MAP (Inc. North Arrow & Benchmark Datum)



LOGGER	
Chris Hoadley, AEG	
GWS	DATE
GWS	DATE
CASING SIZE	CASING DEPTH
CASING SIZE	CASING DEPTH
SLURRY TYPE	

SURFACE CONDITIONS (Slope, Water, Vegetation, etc)
Littered, cobbles and boulders

REMARKS (Tool Sizes/Type - Rods & Bits, etc) (Hole Condition – Caving, Squeezing, Loss of Circulation, etc. Drill Rig reactions – slowing, chattering, skipping, blocking off)	SAMPLE #	BLOWS PER 6"	SPT (N)		DEPTH Ft.	GRAPHIC LOG	DESCRIPTION Soil Classification (group name, group symbol, consistency/relative density, color, moisture, particle size, gradation, plasticity, structure, cementation, organics, fill, q_u, s_u, Other characteristics) Rock Classification (rock name, color, degree of weathering, relative hardness, bedding, discontinuity characteristics, voids, slaking, odor, other characteristics)
Bit 6.5"					1		Surface-Silty SAND (SM) medium dense,
					2		fine to medium grained, light brown, moist,
					3		trace of fine to medium gravel
					4		
					5		
					6		
					7		
	1	5			8		Clayey SAND (SC) medium dense, medium
		8			9		brown, moist, trace of gravel
		8	16		10		
					11		
					12		
					13		
					14		BOH 13.5', install slotted pipe, 5' clean sand
							bentonite chips, backfill with native, add
							water

ROTARY FIELD NOTES

TL-1271a (REV. 01/31/00)

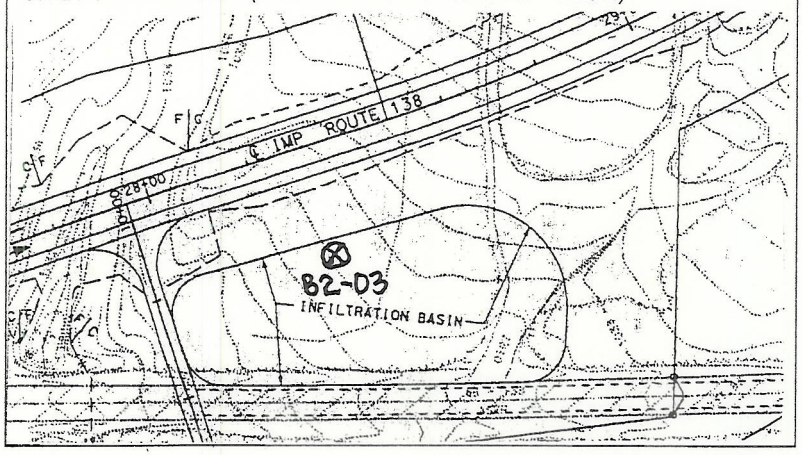
BORING NUMBER	DATE	DIST.	CO.	RTE.	~~P.M.~~ (K.P.)	BRIDGE #
B2-03	12/08/03	08-437001	SBd	138	27.6-27.7 KP	

LOCATION (STA/OFFSET or NORTHING/EASTING)	BRIDGE NAME	EA NUMBER
Station 28+32, 27m Rt. CL		08-437001

TOP HOLE ELEVATION	CREW	EQUIPMENT	CHC NUMBER
1037m	Bob Eneix	B47	

HAMMER ID#
Standard

SITE LOCATION MAP (Inc. North Arrow & Benchmark Datum)



LOGGER	
Chris Hoadley, AEG	
GWS	DATE
GWS	DATE
CASING SIZE	CASING DEPTH
CASING SIZE	CASING DEPTH
SLURRY TYPE	

SURFACE CONDITIONS (Slope, Water, Vegetation, etc)
Littered, cobbles and boulders

REMARKS (Tool Sizes/Type - Rods & Bits, etc) (Hole Condition – Caving, Squeezing, Loss of Circulation, etc. Drill Rig reactions – slowing, chattering, skipping, blocking off)	FIELD TESTING: SAMPLE #	BLOWS PER 6"	SPT (N)		DEPTH -Ft.	GRAPHIC LOG	DESCRIPTION Soil Classification (group name, group symbol, consistency/relative density, color, moisture, particle size, gradation, plasticity, structure, cementation, organics, fill, q_u, s_u, Other characteristics) Rock Classification (rock name, color, degree of weathering, relative hardness, bedding, discontinuity characteristics, voids, slaking, odor, other characteristics)
Bit 6.5"					1		Surface-Silty SAND (SM) medium dense,
					2		medium brown, moist, fine to coarse
					3		grained with trace fine gravel
					4		
					5		
					6		
					7		
					8		
					9		
					10		
					11		
					12		
	1	2			13		As above
		6			14		
		8	14		15		
	2	7			16		A/a
		9			17		
		12	21		18		BOH 18', install slotted pipe, 5' sand,
					19		bentonite chips, backfill with native, water
					20		
					21		

ROTARY FIELD NOTES

TL-1271a (REV. 01/31/00)

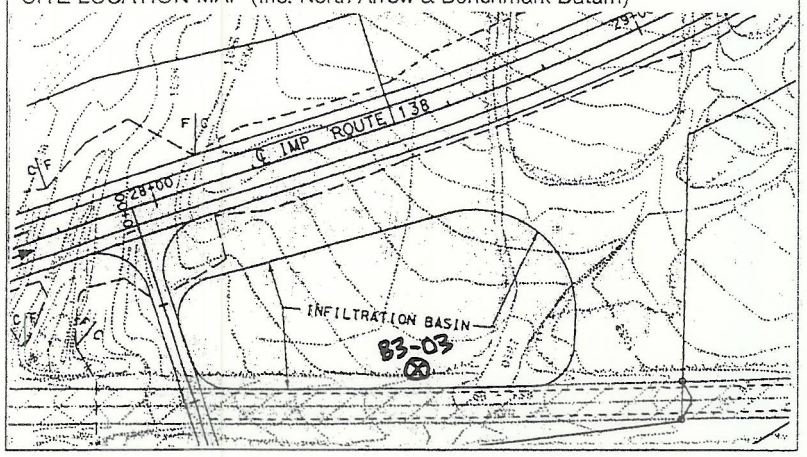
BORING NUMBER	DATE	DIST.	CO.	RTE.	~~P.M.~~ (K.P.)	BRIDGE #
B3-03	12/09/03	08-437001	SBd	138	27.6-27.7 KP	

LOCATION (STA/OFFSET or NORTHING/EASTING)	BRIDGE NAME	EA NUMBER
Station 28+33, 37m Rt. CL		08-437001

TOP HOLE ELEVATION	CREW	EQUIPMENT	CHC NUMBER
1036.5m	Bob Eneix	B47	

HAMMER ID#
Standard

SITE LOCATION MAP (Inc. North Arrow & Benchmark Datum)



LOGGER	
Chris Hoadley, AEG	
GWS	DATE
GWS	DATE
CASING SIZE	CASING DEPTH
CASING SIZE	CASING DEPTH
SLURRY TYPE	

SURFACE CONDITIONS (Slope, Water, Vegetation, etc)
Littered, cobbles and boulders

REMARKS (Tool Sizes/Type - Rods & Bits, etc) (Hole Condition – Caving, Squeezing, Loss of Circulation, etc. Drill Rig reactions – slowing, chattering, skipping, blocking off)	FIELD TESTING: SAMPLE #	BLOWS PER 6"	SPT (N)		DEPTH - FT.	GRAPHIC LOG	DESCRIPTION Soil Classification *(group name, group symbol, consistency/relative density, color, moisture, particle size, gradation, plasticity, structure, cementation, organics, fill, q_u, s_u, Other characteristics)* Rock Classification *(rock name, color, degree of weathering, relative hardness, bedding, discontinuity characteristics, voids, slaking, odor, other characteristics)*
Bit 6.5"					1		Surface-Silty SAND (SM) medium dense,
					2		light brown, moist, medium to coarse
					3		grained with minor med. to c. gravel to 3"
					4		
					5		
					6		
					7		
					8		
					9		
					10		
					11		
					12		
					13		
					14		
					15	—	finer grained, v. moist
					16		
					17		
					18	—	med. brown
	1	11			19		Silty SAND (SM) med. dense, lt. brown,
		10			20		sl. to v. moist, fn. To c. sand with minor
		8	18		21		gravel to 1"

ROTARY FIELD NOTES

TL-1271b (REV. 01/31/00)

BORING NUMBER	DATE	DIST.	CO.	RTE.	~~P.M.~~ (K.P.)
B3-03	12/09/03	08-437001	SBd	138	27.6-27.7 KP

LOCATION (STA/OFFSET or NORTHING/EASTING)	TOP HOLE ELEVATION	BRIDGE #	EA NUMBER
Station 28+33, 37m Rt. CL	1036.5m		08-437001

REMARKS (Tool Sizes/Type - Rods & Bits, etc) (Hole Condition – Caving, Squeezing, Loss of Circulation, etc. Drill Rig reactions – slowing, chattering, skipping, blocking off)	FIELD TESTING: SAMPLE #	BLOWS PER 6"	SPT (N)		DEPTH-Ft.	GRAPHIC LOG	DESCRIPTION Soil Classification *(group name, group symbol, consistency/relative density, color, moisture, particle size, gradation, plasticity, structure, cementation, organics, fill, q_u, s_u, Other characteristics)* Rock Classification *(rock name, color, degree of weathering, relative hardness, bedding, discontinuity characteristics, voids, slaking, odor, other characteristics)*
					22		
					23		
					24		
	2	10			25		Silty SAND (SM) med. dense, lt. to med.
		12			26		brown, moist, fine sand and tr fn. gravel
		11	23		27		
					28		gravelly to 2"
					29		
					30		
					31		
					32		
					33		
					34		
					35		
					36		
					37		
					38		Silty CLAY (CL) stiff, med. brown,
					39		moist, slightly plastic
					40		
							BOH 40', backfill with native
							No groundwater

ROTARY FIELD NOTES

TL-1271a (REV. 01/31/00)

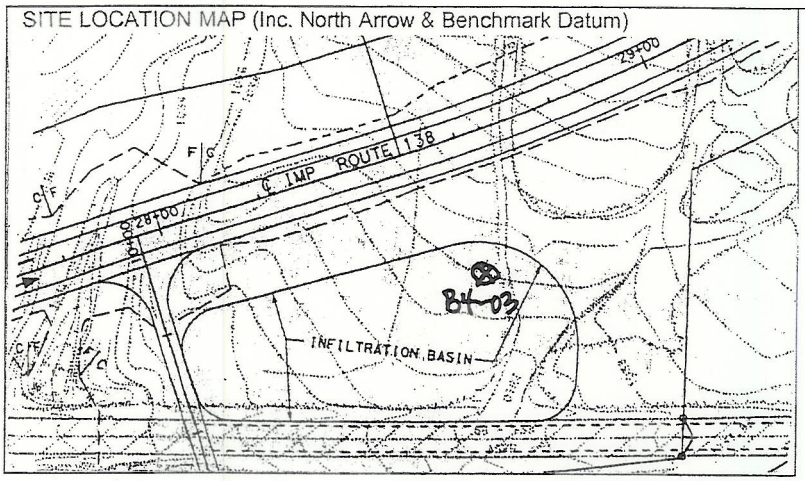
BORING NUMBER	DATE	DIST.	CO.	RTE.	~~P.M.~~ (K.P.)	BRIDGE #
B4-03	12/10/03	08-437001	SBd	138	27.6-27.7 KP	

LOCATION (STA/OFFSET or NORTHING/EASTING)	BRIDGE NAME	EA NUMBER
Station 28+57, 31m Rt. CL		08-437001

TOP HOLE ELEVATION	CREW	EQUIPMENT	CHC NUMBER
1038m	Bob Eneix	B47	

HAMMER ID#
Standard

SITE LOCATION MAP (Inc. North Arrow & Benchmark Datum)



LOGGER	
Chris Hoadley, AEG	
GWS	DATE
GWS	DATE
CASING SIZE	CASING DEPTH
CASING SIZE	CASING DEPTH
SLURRY TYPE	

SURFACE CONDITIONS (Slope, Water, Vegetation, etc)
Littered, cobbles and boulders

REMARKS (Tool Sizes/Type - Rods & Bits, etc) (Hole Condition – Caving, Squeezing, Loss of Circulation, etc. Drill Rig reactions – slowing, chattering, skipping, blocking off)	FIELD TESTING: SAMPLE #	BLOWS PER 6"	SPT (N)		DEPTH ~Ft.	GRAPHIC LOG	DESCRIPTION Soil Classification *(group name, group symbol, consistency/relative density, color, moisture, particle size, gradation, plasticity, structure, cementation, organics, fill, q_u, s_u, Other characteristics)* Rock Classification *(rock name, color, degree of weathering, relative hardness, bedding, discontinuity characteristics, voids, slaking, odor, other characteristics)*
Bit 6.5"					1		Surface-Silty SAND (SM) medium dense,
					2		light brown, dry, fine to coarse grained with
					3		trace gravel to boulder size
					4		below gravel to 2.5", moist
					5		
					6		
					7		
					8		
					9		
					10		
					11		
					12		
					13		
					14		med. den., moist, fn. to med. grained, med.
					15		br., moist, tr. fn. to med. gravel
					16		
					17		sand c. grain, med. green br., some grav. to
					18		1 ½", den., sl. moist, then grav. to 2 ½ "
					19		
					20		As 14-17' med. den. , moist, fn. to med. gr.,
					21		med. br., tr. fn. to med. gravel

ROTARY FIELD NOTES

TL-1271b (REV. 01/31/00)

BORING NUMBER	DATE	DIST.	CO.	RTE.	~~P.M.~~ (K.P.)
B4-03	12/10/03	08-437001	SBd	138	27.6-27.7 KP

LOCATION (STA/OFFSET or NORTHING/EASTING)	TOP HOLE ELEVATION	BRIDGE #	EA NUMBER
Station 28+57, 31m Rt. CL	1038m		08-437001

REMARKS (Tool Sizes/Type - Rods & Bits, etc) (Hole Condition – Caving, Squeezing, Loss of Circulation, etc. Drill Rig reactions – slowing, chattering, skipping, blocking off)	FIELD TESTING: SAMPLE #	BLOWS PER 6"	SPT (N)		DEPTH - ft	GRAPHIC LOG	DESCRIPTION: Soil Classification (group name, group symbol, consistency/relative density, color, moisture, particle size, gradation, plasticity, structure, cementation, organics, fill, q_u, s_u, Other characteristics) Rock Classification (rock name, color, degree of weathering, relative hardness, bedding, discontinuity characteristics, voids, slaking, odor, other characteristics)
					22		
					23		
	1	10			24		Clayey SAND (SC) med. den., med. brown,
		6			25		moist
		8	14		26		
					27		Silty SAND (SM) den., greenish brown,
					28		sl. moist, trace gravel
					29		fn. to med. grain, damp, c. grav. and trace
					30		cobble to 3 ½"
					31		below grav. to 2 ¼"
					32		
					33		
					34		
					35		Silty sandy CLAY (CL) stiff, med. br.,
					36		damp, slightly plastic
					37		
					38		gravel lense
					39		
					40		
					41		
					42		
					43		
					44		
							BOH 44', backfill with native
							No groundwater

ROTARY FIELD NOTES

TL-1271a (REV. 01/31/00)

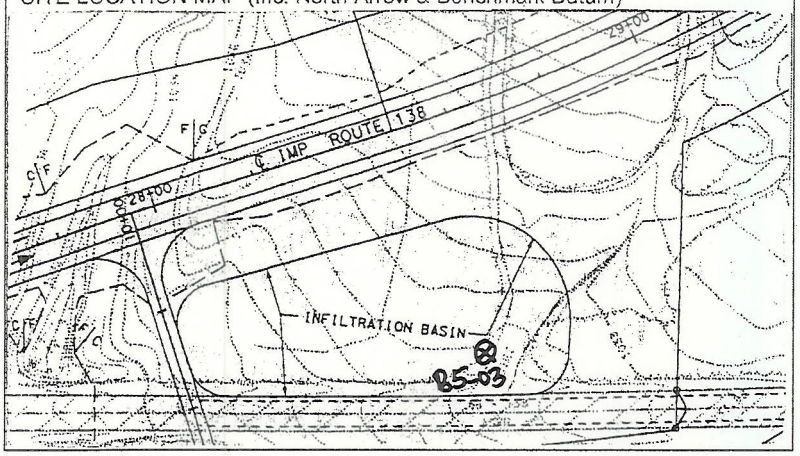
BORING NUMBER	DATE	DIST.	CO.	RTE.	~~P.M.~~ (K.P.)	BRIDGE #
B5-03	12/10/03	08-437001	SBd	138	27.6-27.7 KP	

LOCATION (STA/OFFSET or NORTHING/EASTING)	BRIDGE NAME	EA NUMBER
Station 28+58, 42m Rt. CL		08-437001

TOP HOLE ELEVATION	CREW	EQUIPMENT	CHC NUMBER
1037m	Bob Eneix	B47	

HAMMER ID#
Standard

SITE LOCATION MAP (Inc. North Arrow & Benchmark Datum)



LOGGER	
Chris Hoadley, AEG	
GWS	DATE
GWS	DATE
CASING SIZE	CASING DEPTH
CASING SIZE	CASING DEPTH
SLURRY TYPE	

SURFACE CONDITIONS (Slope, Water, Vegetation, etc)
Littered, cobbles and boulders

REMARKS (Tool Sizes/Type - Rods & Bits, etc) (Hole Condition – Caving, Squeezing, Loss of Circulation, etc. Drill Rig reactions – slowing, chattering, skipping, blocking off)	FIELD TESTING: SAMPLE #	BLOWS PER 6"	SPT (N)		DEPTH ft.	GRAPHIC LOG	DESCRIPTION Soil Classification *(group name, group symbol, consistency/relative density, color, moisture, particle size, gradation, plasticity, structure, cementation, organics, fill, q_u, s_u, Other characteristics)* Rock Classification *(rock name, color, degree of weathering, relative hardness, bedding, discontinuity characteristics, voids, slaking, odor, other characteristics)*
Bit 6.5"					1		Surface-Silty SAND (SM) medium dense,
					2		light brown, sl. moist, fine to coarse grained
					3		with gravel to cobble size
					4		
					5		
					6		
					7		
					8		
					9	—	smaller gravel
					10		
					11		
					12	—	gravelly
	1	23			13		
		22			14	—	dense, gravel to 1 3/8"
		14	36		15		
	2	15			16		Clayey SAND (SC) med. dense, lt. brown,
		14			17		sl. moist, trace of gravel
		11	25		18		
					19		
					20		BOH 20', install slotted pipe, 5' sand,
					21		bentonite chips, backfill with native, water

STATE OF CALIFORNIA
DEPT OF TRANSPORTATION
GEOTECHNICAL LAB

E.A. 08-437001

Br. #

CLASSIFICATION TEST SUMMARY

LOCATION	% FINER THAN															ATT. LIMITS		Gs	AS RECEIVED		LAB. COMP	
Sample I.D.	1 1/2	1	3/4	1/2	3/8	4	8	16	30	50	100	200	5μ	2μ	1μ	LL	PI		γ_d	%m	γ_d	%m
B-1-8.5-12'	CS		100	99	99	98	94	83	65	42	26	17	7	-	4		NP					
B-2-13-16.5'	SS				100	99	97	93	81	59	37	22	8	-	4		NP					
B-3-20-21.5'	SS		100	98	98	95	92	86	77	61	40	25	8	-	5		NP					
* B-4-42-44'	BEYOND BASIN				SiSaCl	100	99	97	92	82	69	56	30	-	20	30	13					
B4-25-26.5'	CS		100	99	99	96	91	81	68	48	27	17	7	-	4		NP					
B-5-14-15.5'	100 CS	97	95	92	89	82	72	58	42	28	19	13	6	-	4		NP					

NP - Non-plastic
* Sample received fractured -- Not suitable for testing
** Sample disintegrated while preparing for testing -- Not suitable for testing
*** Sample received desiccated -- Test results may not represent the true soil conditions

FINAL

State of California

Business, Transportation and Housing Agency

Memorandum

Flex your power!
Be energy efficient!

To: MS. CHRISTY CONNORS – 08
Design "N"

Attention: Mr. Chris Nguyen

Date: August 12, 2003
File: 08-SBd-138-KP R27.5/R30.9
Fr 3.1 km E of Rte 15 to 0.5 km
W of Summit Post Office Rd
E.A.: 08-437001

From: **DEPARTMENT OF TRANSPORTATION**
DIVISION OF ENGINEERING SERVICES
Geotechnical Services, MS#5
Office of Geotechnical Design – South 2

Subject: Metrication of the Geotechnical Design Report for Route 138, Phase 2

This report is prepared as requested by District 8 to convert the unit system to metric for the Geotechnical Design Report (GDR) prepared by AGRA Earth and Environmental, Inc. (AGRA), dated Dec. 20, 1996.

The GDR report was prepared for a proposed realignment of the existing two-lane State Highway 138. Realignment will be offset mostly to the north relative to the present centerline. The previously proposed project was a four-lane realignment. Due to problems with the approved environmental document, the locally funded four-lane project was suspended. However, within the limits of the four-lane project was a two-lane HB1 safety project identified by the District on the same alignment from 3.1 km east of Route 15 to 0.2 km west of Summit Post Office Road. The difference is approximately 610 m less on the west end and 506 m less on the east end of the project.

The current project is proposing a different centerline and stationing compared to the previously proposed project, and three (3) bridges (not covered in this report), an infiltration basin (in a separate report), and some newly proposed 1(V):2(H) cut slopes and 1(V):3(H) fill slopes.

Our Office prepared this report based on a site reconnaissance, available literature review, including reports on highway projects and bridges adjacent to and along this project site, and on local geology. This report is intended to be used as a supplemental report to the AGRA prepared GDR reports.

The following table converts all units in US System to Metric (SI) system within the text portion of the GDR reports. For any conversions needed in the Plates or Appendices, please request from District 8, Design "N".

Proposed project limits extend from approximately Station 24+87 to 58+78 (meters). This is 24.4 meters upstation from Station 80+80 in US units. Conversion to new stationing is approximate based on visual measurements of plan sheets.

Page No.	Section No.	Paragraph No.	Line No. within the Paragraph	U.S. Units	Metric Units
1	1.0	1	2	1.2 miles	1.9 km
				0.2 miles	0.3 km
	2.0	1	1	Sta. 62+00	Sta. 24+87
				Sta. 209+20	Sta. 58+78
			4	3,270 ft.	996.7 m
				3,850 ft.	1,173.5 m
3	2.0	1	1	0.75:1.0 (H:V)	1.0:0.75 (V:H)
				Sta. 175+00	Sta. 53+20
				Sta. 182+00	Sta. 55+50
			3	105 ft.	32.0 m
			4	0.5:1.0	1.0:0.5
			5	20 ft.	6.1 m
		2	1	30 ft.	9.1 m
				2:1 (H:V)	1:2 (V:H)
				1.25:1 (H:V)	1:1.25 (V:H)
		3	1	3,270 ft.	996.7 m
				3,800 ft.	1,158.2 m
			2	70 ft.	21.3 m
				160 ft.	48.8 m
4	4.1	1	7	100 deg. F	37.8 deg. C
			8	15 in.	381 mm
				20 in.	508 mm
			10	¼ in.	6 mm
		2	1	4000 ft.	1,219.2 m
	4.2	1	2	3220 ft.	981.5 m
				3,953 ft.	1,204.9 m
				733 ft.	223.4 m
			3	Sta. 135+00	Sta. 41+10
5	4.2	1	1	20-70 ft.	6.1-21.3 m
			3	1.5:1	1:1.5
				2:1	1:2
				0.75:1 (H:V)	1:0.75 (V:H)
			5	2-feet	0.61 m
		2	3	Sta. 175 to 185	Sta. 53+20 to 56+20
			4	100-feet	30.5 m
			2	(0.5:1)	(1:0.5)
				2:1	1:2
				5:1	1:5
			3	Sta. 140 to 150, ?? ft Lt. CL	Sta. 42+60 to 45+60, 30 m Lt. CL
			4	10 to 20:1	1:10 to 20
			5	2:1	1:2
	4.3	2	1	Sta. 175 to 190	Sta. 53+20 to 57+70
7	4.4.2	1	4	3-miles	4.8 km

Page No.	Section No.	Paragraph No.	Line No. within the Paragraph	U.S. Units	Metric Units
			6	2 mi.	3.2 km
				2.5 mi.	4.0 km
				8 mi.	12.9 km
				10 mi.(x3)	16.1 km
		2	2	17-miles	27.4 km
8	Fault map			30 miles, 60 miles	49.3 km, 96.6 km
9	4.4.2	1	1	14-feet	4.3 m
				230-miles	370.1 km
	5.1	1	2	81-feet	24.7 m
		2	4	2.5-inch ring	63.5 mm ring
				140 lb.	63.5 kg (623 N)
				30-inches	762 mm
10	5.1	2	2	24-inch	609.6 mm
			5	140 lb.	63.5 kg (623 N)
				30-inches	762 mm
11	5.3	1	3	330 to 230 feet	100.6 m to 70.1 m
13	7.1.1	2	2	Sta. 175+00 to185+00	Sta. 53+20 to 56+20
14	7.1.2	2	1	Sta. 180 to 190	Sta. 54+70 to 57+70
		3	4	Sta. 135+00	Sta. 41+10
				Sta. 185+00 to 195+00	Sta. 56+20 to 59+20
15	7.1.3	3	1	Sta. 175+00 to 185+00	Sta. 53+20 to 56+20
		4	1	2:1	1:2
			2	2:1	1:2
		5	1	2:1	1:2
17	7.3.1	1	1	3 miles	4.8 km
				2 miles	3.2 km
19			Unit Wt.	135 pcf	21.20 kN/m^3
				125 pcf	19.63 kN/m^3
				128 pcf	20.10 kN/m^3
			Sat. Un. Wt.	141 pcf	22.14 kN/m^3
				131 pcf	20.57 kN/m^3
				138 pcf	21.67 kN/m^3
			Peak	300 psf	14.36 kPa
				175 psf	8.38 kPa
			Ultimate	180 psf	8.62 kPa
				75 psf	3.59 kPa
				150 psf	7.18 kPa
	8.1.1	1	1	300 psf	14.36 kPa
			2	750 psf	35.91 kPa
			3	175 psf	8.38 kPa
				Sta. 180+00	Sta. 54+70
	8.1.2.1	1	1	Sta. 176+00 to 185+00	Sta. 51+65 to 56+20
			4	160 feet	48.8 m
				Sta.176+00	Sta. 53+45

Page No.	Section No.	Paragraph No.	Line No. within the Paragraph	U.S. Units	Metric Units
				1.35:1.0	1.0:1.35
19	8.1.2.1	1	6	20 feet	6.1 m
			7	125 feet	38.1 m
				Sta. 183+00	Sta. 55+60
				1:35:1.0	1.0:1.35
				20 feet	6.1 m
		2	1	Sta. 157+00	Sta. 47+72, 65 m Lt. CL
				Sta. 174+00	Sta. 52+85
			2	57feet	17.4 m
				Sta. 165+50	Sta. 50+30
				1.35:1.0	1.0:1.35
20	8.1.2.1	1	1	Sta. 60+00	Changed to Sta. 24+87
				Sta. 157+00	Sta. 47+72, 65 m Lt. CL
			3	30 feet	9.1 m
				Sta. 120+00	Sta. 36+55
				1.5:1.0	1.0:1.5
		2	1	Sta. 157+00	Sta. 47+72, 65 m Lt. CL
				Sta. 190+00	Sta. 57+70
				1.35:1.0	1.0:1.35
			2	70 feet	21.3 m
				20 feet	6.1 m
			3	Sta. 62+00	Changed to 24+87
				Sta. 157+00	Sta. 47+72, 65 m Lt. CL
				1.5:1.0	1.0:1.5
	8.1.2.2	1	3	1.2 feet	0.37 m
				2.6 feet	0.79 m
				2.8 feet	0.85 m
		3	1	2 feet	0.61 m
21	Chart	Fig. 3		1.35:1.0	1.0:1.35
				1.5:1.0	1.0:1.5
				1,2,3,4,5 feet	0.3,.61,.91,1.22,1.52 m
22	8.1.2.3	1	End	1.01 feet	0.31 m
				1.04 feet	0.32 m
				1.22 feet	0.37 m
				0.63 feet	0.19 m
	8.1.3	1	4	2,000 to 4,000 fps	609.6 to 1219.2 mps
				2,000 to 5,000 fps	609.6 to 1524.0 mps
23	8.1.4	2	1	Sta. 164+00	Sta. 49+70, 45 m Lt. CL
			2	Sta. 157+00	Sta. 47+70, 65 m Lt. CL
	8.2.1	1	4	3 to 4 feet	0.9 to 1.2 m
			5	Sta. 189+00	Sta. 57+40
			6	3 feet	0.9 m
			7	Sta. 158+00	Sta. 48+05, 65 m Lt. CL
				Sta. 163+00	Sta. 49+50, 50 m Lt. CL

Page No.	Section No.	Paragraph No.	Line No. within the Paragraph	U.S. Units	Metric Units
			9	Sta. 110+00 to 112+00	Sta. 33+50 to 34+10
			10	15 feet	4.6 m
		2	1	1 to 4 inches	25.4 to 101.6 mm
				20 to 70 feet	6.1 to 21.3 m
24	8.2.2	1	4	100 psf	4.79 kPa
	8.2.3.1	1	3	70 feet	21.3 m
				Sta. 137+00	Sta. 41+70
				2.0:1.0	1.0:2.0
			6	2.0:1.0	1.0:2.0
	8.2.3.2	1	3	2.5 feet	0.76 m
			4	2.5 feet	0.76 m
25	Chart			2.0:1.0	1.0:2.0
				1,2,3,4,5 feet	0.31, 0.61, 0.91, 1.22, 1.52 m
26	8.2.3.3	1	End	0.52 feet	0.16 m
				0.58 feet	0.18 m
				0.55 feet	0.17 m
				0.30 feet	0.09 m
	9.0	Bullets	1	Sta. 157+00	Sta. 47+72, 65 m Lt. CL
				1.5:1.0	1.0:1.5
				Sta. 190+00	Sta. 57+70
				1.35:1.0	1.0:1.35
			2	70 feet	21.3 m
				20 foot	6.1 m
27	9.0	Bullets	6	8 feet	2.4 m
				70 feet	21.3 m
			7	3 – 4 feet	0.9 – 1.2 m
				Sta. 189+00	Sta. 57+40
				3 feet	0.9 m
				Sta. 110+00 to 112+00	Sta. 33+50 to 34+10
				5 – 15 feet	1.5 – 4.6 m
			8	2.0:1.0	1.0:2.0
			10	30 feet	9.1 m
				7.5 feet	2.3 m
			13	4 feet	1.2 m
			14	Sta. 144 and 189	Sta. 43+85 and 57+40
				4 feet	1.2 m

If you have any questions or comments, please call Chris Hoadley at (916) 227-4515.

Prepared by: Date: 8/12/03

Chris Hoadley, CEG No. 2309
Associate Engineering Geologist
Geotechnical Branch C

REGISTERED GEOLOGIST
CHRIS HOADLEY
No. 2309
Exp. 11/23/05
CERTIFIED ENGINEERING GEOLOGIST
STATE OF CALIFORNIA

Supervised by: Date: 8-12-03

Shawn Wei, P.E., Chief
Branch C
Office of Geotechnical Design – South 2

cc: AAbghari
Project File

Reference:

- Materials Report, prepared by John A. Sayers and Associates (JASA), dated May 29, 1996.
- Supplemental Geotechnical Design Report, prepared by AGRA, dated Dec. 20, 1996.
- Addendum to the Supplemental Geotechnical Design Report, prepared by AGRA, dated Sept. 16, 1997.
- Review of New Proposed Alignment from Station 44+80 to 51+20, prepared by Office of Geotechnical Design South 2, Caltrans, dated March 8, 2001.
- 60% Review Plans and Cross-Sections, prepared by District 08, Caltrans, dated March 11, 2003.

GEOTECHNICAL DESIGN REPORT

For
Realigning the Existing Two-Lane Highway
On
State Route138 in San Bernardino County

From 3.0 Km East of Route 15
To 0.5 Km West of Summit Post Office Road

08-SBd-138-KP R27.5/R30.9
EA: 08-437001

Prepared By

Chris Hoadley
Associate Engineering Geologist
Geotechnical Branch C

REGISTERED GEOLOGIST
CHRIS HOADLEY
No. 2309
Exp. 11/05
CERTIFIED ENGINEERING GEOLOGIST
STATE OF CALIFORNIA

DIVISION OF ENGINEERING SERVICES
Geotechnical Services
Office of Geotechnical Design – South II
May 2003

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Drainage Location (One Sheet)

Groundwater Level Data, Department of Water Resources (Seven Sheets)

Seismic Refraction Survey for Rippability (Seven Sheets)

Corrosion Test Results (Seven Sheets)

Ritchie Criteria (One Sheet)

Log of Test Borings (LOTB) (Five Drawings)

1. Introduction

This Geotechnical Design Report (GDR) is for the proposed realignment of the existing two-lane State Highway 138. Realignment will be offset to the north or south relative to the present centerline. The original proposed project was a four-lane realignment. Due to problems with the approved environmental document, the locally-funded four-lane project was suspended indefinitely. However, within the limits of the four-lane project was a two-lane HB1 safety project identified by the District on the same alignment from 3.0 km east of Route 15 to 0.2 km west of Summit Post Office Road.

District 8 requested a New/Revised GDR that reflects a slightly different alignment than was studied for the four-lane locally funded project in the previous GDR's. This difference is approximately 610 m less on the west end and 506 m less on the east end of the project. In addition, the current project is in metric, has different centerline stationing, has three (3) bridges (not covered in this report), proposes an infiltration basin, and a new typical cross-section with 1(Vertical):2(Horizontal) cut slopes and 1:3 fill slopes.

Our Office prepared this GDR based on a surface reconnaissance of the site, literature study, including reports on highway projects and bridges adjacent to and along this project site, and on local geology. This report is intended for use by the project roadway design engineer, construction personnel, bidders and contractors.

2. Pertinent Report and Investigations

District 8 has provided us with 60% Review Plans and Cross-Sections. From our previous review of the four-lane project we have the reports listed below. Our literature search yielded several reports and maps, which were utilized in preparing this report and are cited in the attached List of References.

- Materials Report, John A. Sayers and Associates (JASA), May 29, 1996.
- Supplemental Geotechnical Design Report, AGRA Earth and Environmental, Inc., Dec. 20, 1996.
- Addendum to the Supplemental Geotechnical Design Report, AGRA, Sept. 16, 1997.
- Review of New Proposed Alignment from Station 44+80 to 51+20, Chris Hoadley, March 8, 2001.

2. Description of Existing Facilities

The current project limits extend from approximately Station 24+97 to 58+77. The existing and proposed alignment are shown on the Geologic and Field Exploration Plans ( ). Existing Route 138 within the project limits is a two-lane paved roadway traversing moderately hilly terrain. Ground elevation along the current alignment varies from approximately 1,024 m at the west end of the project to approximately 1,146 m near the east end of the project. The new alignment will match the the existing roadway at those elevations and the highest point on the new alignment is approximately 1,159 m.

Existing cut slope gradients along the existing roadway generally vary from about 1(V):¾(H) to 1:1 within the eastern portion of the alignment (Station 52+00 to 55+00). These cuts are made in both Crowder Formation and Terrace deposits. Slope heights vary up to approximately 32 m. Steeper cut slopes, with gradients of 1:½ or steeper are encountered locally along the remainder of the roadway within the Crowder Formation, Terrace deposits and alluvial fan deposits. These slope heights are generally less than 7 m in height. Based on visual observations, and discussion with District 8 personnel, the existing cut slopes are performing relatively well. Minor to moderate ravelling of coarser material within the steeper cuts does occur particularly during periods of rainfall. Where upslope runoff occurs over the face of cut slopes there is evidence of gullying, erosion and surficial slumping.

Fill slopes along the existing roadway vary in height up to about 9 m with gradients varying from flatter than 1:2 to locally as steep as 1:1.25. Most fill slope are covered with vegetation and appear to be performing adequately. The vegetation within the project limits includes species associated with well-drained areas of the Mojave Desert, such as creosote bush, cheese bush, burro bush, and Mormon tea. Cacti species are more common in the drier valley areas but other areas exhibit varying densities of low lying shrubs and brush.

The proposed grade will necessitate fill embankments up to approximately 20 m in height and cut slopes up to approximately 47 m in height. The maximum elevation will be approximately 1,158 m at Station 55+45. Culverts are proposed where fill embankments will transect existing drainages.

4. Physical Setting

4.1 Climate

The climatic conditions at the project area are typical of a desert region. This arid region in southeastern California is characterized by extreme variation in daily temperature and has an average annual precipitation of less than 12.7 cm (5 in.), mostly rain with some snow. Almost all the precipitation arrives in winter. Freezing temperatures occur in winter, while summers are hot, dry and windy.

The following tabulated data was obtained from the Webster: http://www.accuweather.com/. This climatology data was recorded for the year 2000 at George AFB, California (Elevation 893 m/2874 ft) which is located Approx. 25 km (15.5 miles) NE of the intersection of the San Bernardino/Los Angeles County line with Rte. 138.

16.3 Climatology

Climatology 2000	Jan	Feb	Mar	Apr	May	Jun	Jul	Aug	Sep	Oct	Nov	Dec
Highest Temp C	21	25	28	31	29	38	41	39	40	33	23	22
(F)	70	77	83	88	103	100	105	103	104	91	73	72
Lowest Temp. C	-3	-3	1	1	6	12	14	13	11	6	-4	-3
(F)	27	27	33	33	42	53	57	56	51	42	25	27
Average Temp. C	7	8	13	14	22	24	27	27	23	17	10	10
(F)	44	46.7	56	56.4	72.1	75.0	80.0	81.2	73.9	62.5	49.4	49.1
Total Precipt. cm	0.56	2.8	0.33	1.32	0.00	0.00	0.00	2.11	0.03	trace	0.00	trace
(in)	0.22	1.12	0.13	0.52	0.00	0.00	0.00	0.83	0.01	trace	0.00	trace

Total Snowfall cm	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
(in)	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
Normal Precipt. cm	2.1	2.2	2.2	0.9	0.5	0.1	0.4	0.8	1.0	0.7	1.5	1.8
(in)	0.82	0.88	0.85	0.36	0.18	0.05	0.15	0.31	0.38	0.26	0.58	0.69

4.2 Topography and Drainage

In general the topography for the project area descends gradually from the south/southwest to the north/northeast, from the San Gabriel Mountains (approximate elevation 2134 m/7000 ft.) to the Victor Valley, (approximate elevation 914 m/3000 ft.). Table Mountain and Lone Pine Ridge impede drainage of north-flowing San Gabriel Mountain streams, directing the drainage along Lone Pine Canyon.

At several isolated locations, such as Sheep Creek (see Plate No. 17.2A) and Mescal Creek (see Plate 17.2C), the composite ridge is breached and the streams flow north, onto the Mojave Desert. Along Segment 1 the profile grade generally rises from (approximate elevation 1176 m/3858 ft.) at the San Bernardino /Los Angeles County line (KP 0.0/PM 0.0) to elevation 1324 m/4343 ft. at Sheep Creek (KP 5.8 /PM 3.6). The Segment 2 profile grade generally inclines downward to elevation 949 m/3112 ft. at Cajon Creek.

16.4 Ground Elevations

Bridge Name	Bridge Number	KP(PM)	Approx. ground Elevation
Sheep Creek Bridge	54-0810	5.8(3.6)	1324m/4343ft
Pine Lodge West Overhead	54-1056	23.0(14.3)	984m/3229ft
Pine Lodge East Overhead	54-1057	23.8(14.8)	958m/3142ft
Cajon Creek Bridge	54-0832	24.0(14.9)	949m/3112ft
California Aqueduct	53-2174	112.8(70.1)	1062m/3483ft

As mentioned in Segment 3 "Existing Facilities," Segment 3 passes through rolling terrain. Where natural drainage courses descend from the south toward the highway, they flow over the roadway at dips in the profile grade (see Exhibit G1). At most of these dips, minor debris basins from 0.5 m to about 2 m deep have been excavated on the south side of the highway to collect sediments from the storm flow and allow cleaner water to pass over the traveled way. Most of these basins had been cleaned of deposits from last winter's storms at the time of our field review. However, one at KP 118.2 (PM 73.45) contained an approximately 150 mm thick deposit of silt or clayey silt exhibiting desiccation cracks up to 50 mm wide. Such soils will not be acceptable foundation material for new embankment, but might help to retain moisture for establishing vegetation if well dispersed within new embankment fill.

On-site drainage within Segment 3 is controlled by soil berms 0.3 m high at the outer edges of the dirt shoulders. These berms both direct on-site flow to the dips, where the flow then continues northerly in existing natural drainage courses, and also help to direct sheet flow from desert floor areas south of the alignment to the debris basins.

In the San Bernardino County segments, earthen trapezoidal channels locally collect runoff from offsite areas to the south and southwest and direct the flow to cross culverts. An example can be observed between PM 0.0 to 0.5 where a 1 m deep, 2 m (bottom width) channel with 1(V):2(H) or flatter side slopes diverts runoff to a corrugated metal culvert at PM 0.49. These channels are irregular and hard to distinguish due to desert vegetation and disturbance due to either deposition or dumping of debris in the channel and/or maintenance efforts to clear such deposits. There are 104 culverts for unnamed drainage courses that cross the alignment in Segments 1 & 2 (see attached drainage data sheet in appendix).

About 2 m deep, easterly draining drainage course lies approximately 15 m north of the edge of pavement for the existing westbound lane from the beginning of Segment 2 at KP 13.8 (PM 8.6) to a dry wash at KP 14.6 (PM 9.0). This channel is about 4.5 m below the elevation of the existing road surface at the beginning of Segment 2, but it rises up, gets shallower and spreads out wider as it approaches the Cajon wash. The drainage course contains heavy growths of brush and likely contains loose alluvial deposits (see Exhibit H). New embankment placed to widen the highway northerly of its current location will likely encroach into, or bury this drainage course.

The existing drainage courses throughout the project alignment are poorly to well eroded, usually displaying considerable gravel, cobbles and occasional boulders.

4.3 Prior Land Use

To the best of our knowledge, no agricultural land that would contain plowed, loosened surface soils exists close to or along the sides of the current alignment, except for along Segment 2 where the road passes an orchard at approximately KP 18.0 (PM 11.3) and a vineyard at KP 17.7 (PM 11.0). Most of the alignment, and all of Segment 3, passes through vacant high desert. Segment 1 passes through the communities of Pinon Hills and Phelan. Segment 1 - 2 is the Highway 2 interchange and the Summit Restaurant set back from the roadway. Segment 2 passes through lands of the San Bernardino National Forest. Segment 3 passes through the community limits of Llano. However, where the current alignment passes through the communities, residential and commercial facilities are on adjoining land that might be impacted by this project. Right of Way lines shown on plans available at the time of our review show that existing commercial buildings in the area of Segment 1 will not be within the future Right of Way. If existing buildings do exist within the future Right of Way, there could be potential impacts to future highway earthwork due to disturbed soils from building demolition, possible on-site sewage disposal systems, etc.

5. Geology

5.1 Regional Geology

The subject site lies at the boundary between two geomorphic provinces, the Great Basin/Mojave Block Province and the Transverse Range Province. The border between these provinces is in close proximity to the San Andreas Fault, the boundary between the Pacific and North American continental plates. This boundary is strike-slip and has resulted in significant seismic activity in the past, and should be expected to continue to do so in the future (Plate 17.3A).

The upper part of the project, north of the San Gabriel Mountains, lies within the Great Basin/Mojave Block Province, which is characterized by isolated mountain ranges separated by expanses of desert plains (Plate 17.3B). There are two topographic trends within the province, a northwest-southeast trend controlled by the San Andreas Fault along the southwestern border of the province, and a secondary east-west trend controlled by the Garlock Fault, which is the northern boundary of the province. The southern section of the subject site lies within the Transverse Range geomorphic province, a complex series of east-west trending mountain ranges and valleys.

5.2 Site Geology

Segments 1 and 3 of Antelope Highway lie within the Upper Mojave River Valley Basin. The geologic formation through which these segments will pass has been mapped as Quarternary Alluvium as shown on Plate 17.3 A. Deposits at the southeasterly end of Segment 1 are mapped as well dissected alluvial fan deposits, which have much the same character as the previously mentioned alluvial deposits.

Segment 1-2 and Segment 2 passes across the southern edge of the Cajon Basin west of Cajon Pass, the natural pass separating the San Gabriel Mountains from the San Bernardino Mountains. This portion also lies southwesterly and roughly parallel to the flow line of Cajon Canyon, which is also located just beyond the base of Lone Pine Ridge to the southwest. Lone Pine Ridge, underlain by Mesozoic plutonic rock and pre-Mesozoic gneiss and marble, separates Cajon Canyon from Lone Pine Canyon on the southwest. The San Andreas Fault, which trends west-northwest through the Transverse Ranges, lies within Lone Pine Canyon.

As it traverses the floor of Cajon Canyon, Segment 2 is also founded on Quarternary Alluvium. However, two geologic formations were encountered in two locations. As seen on Plate 17.2B, between KP 13.8 to 14.1 (PM 8.63 to 8.76) the northwest end of Segment 2 has been cut into the flank of a northwest trending ridge consisting of Crowder Formation sandstone. This formation is a light tan to gray, fine to coarse, highly weathered, friable sandstone with frequent cobbles (see Exhibit D). Distinct geologic bedding was not discerned in this rock where exposed on the surface of the road cut. However, the geologic map consulted depicts bedding as dipping down to the northeast at 20 degrees from the horizontal. At that angle, the bedding is dipping slightly out of slope in the present cut, and would be unsupported should the cut be widened at a slope steeper than 20 degrees.

The second location of formational rock along Segment 2 occurs at KP 21.4 to 21.66 (PM 13.3 to 13.46) where the highway threads between outcrops known as the Mormon Rocks (see Exhibit C). The Mormon Rocks belong to the Punchbowl Formation of sandstone and conglomerate, very hard, unfractured, gravelly, coarse sandstone with laterally discontinuous cobble lenses that dips to the northeast at 40 to 60 degrees (see Plate 17.2B). A bedding measurement taken during our rippability study found the bedding to be dipping at 45 degrees to the northeast.

For confirmation of the types of soils to be encountered along the project alignment, we compared the soil descriptions listed in the Log of Test Boring (LOTB) sheets (attached) for the five bridges on the existing highway to the material types depicted by the geologic mapping. The borings encountered alluvial deposits of interbedded layers of slightly compact to very dense silty sand, sand, gravelly sand, and sandy gravel with scattered cobbles.

6. Geologic Considerations

6.1 Mineral Resources

Other than soil (alluvial) deposits that might be processed to generate sand, no known mineral resources were identified within the project limits.

6.2 Aggregate/Construction Material Source

The alluvial soils within the project limits could potentially be used as a source of sand and aggregate for construction.

At this time no specific borrow sites have been identified nor tested. However, during our reconnaissance we noted trapezoidal shaped stockpiles of silty sand with gravel adjacent to the California Aqueduct near KP 112.8 (PM 70.1) in Segment 3 (see Exhibit I). The stockpiled soils were likely removed during excavation of the aqueduct and placed as mounds on either side of the aqueduct (see Exhibit I). We do not know if these mounds serve other purposes than simply a means of disposing of the excavated soil during aqueduct construction, but the District could inquire with the Department of Water Resources about the availability of that material for use as common borrow.

6.3 Excavation Characteristics

For most of the project length, we expect that excavation of alluvial soil required for constructing the two new lanes and the median left-turn lanes can be carried out by conventional earth moving equipment. Based on our surficial observations and on the geologic mapping, as substantiated by the Log of Test Borings for the five bridges, sandy gravel with scattered cobbles and boulders will be encountered to depths of at least 2 m to 3 m below ground surface.

Other than at two locations, we do not anticipate cuts, if any, to be deeper than 0.5 to 1.0 m for the new lanes. However, if occasional boulders are encountered during excavations, they will need to be separated from the rest of the excavated material, treated as "surplus material" and handled as per Standard Specification 19-2.06, assuming the proposed fills will be too thin to accommodate the boulder/cobble dimensions.

At the time of this report, the depth of excavation to widen the existing northeasterly facing cut slope at PM 8.64 was not known to us. Nevertheless, it appears that the Crowder Formation sandstone is highly weathered. We expect that excavation can be conducted with conventional earthmoving equipment, perhaps with heavy ripping at times.

The Mormon Rocks (KP 22/PM 13.4) outcrop is very hard, unfractured, gravelly, coarse sandstone with laterally discontinuous cobble lenses that dip to the north at 45 degrees (see Exhibit J). These outcrops contain a weathered horizon that appears rippable to a depth of approximately 2 meters. Depths greater than 2 meters are unrippable mechanically to subgrade elevation. Blasting to a subgrade elevation may be considered as an option, or alternatively evaluate realignment. The hardness of this formation, the favorable bedding dip relative to the orientation of the proposed cuts and the lack of other discontinuities enable us to tentatively recommend cut slope ratios in the Mormon Rock outcrops steeper than the other cuts on the project.

6.4 Erosion

The performance at the existing embankments for the Pine Lodge Overheads, the Oasis Road intersection and the Mescal Creek culvert shows that the erosion performance of embankments constructed at 1(V):2(H) or flatter slope ratios can be dependent on effective landscaping treatments (vegetation), width of paved area providing tributary runoff,

prevention of both concentrated flows and sheet flow over the face of the slope, and the amount of gravel and cobble on the slope surface. As the proposed widened highway will create a paved width similar to that existing at the Oasis Road intersection, runoff from the pavement could erode unprotected fill slopes. Asphalt concrete dikes that prevent sheet flow of water from the pavement and over the slope appear to be highly effective in preventing erosion, even where they are cracked as has occurred at the Sheep Creek Bridge. However, AC dikes would require provisions to collect the flow from the pavement and discharge it in a non-erosive manner. For fill slopes of minor height that can be easily accessed for maintenance, flatter slope ratios and landscaping measures could serve satisfactorily.

We anticipate that the proposed cut in the Mormon Rocks will not be subject to erosion due to the hardness of the rock. These outcrops stand naturally at steep slope ratios with little vegetation, their continued existence showing the erosion resistant nature of the rock.

While the existing 1(V):1(H) cut in the Crowder Formation sandstone at KP 13.9 (PM 8.64) has apparently stood well since the construction of the highway without significant gross failure, the bedrock is friable and subject to surficial erosion as the rock weathers. Erosion of the present cut is evident (see Exhibit D). Increasing the height and exposed surface of the cut face will subject the cut to added runoff, therefore, increasing the potential for erosion. If the top of the future widened cut does not extend to the top of the southwesterly adjacent ridge where the proposed R/W line is shown, then we recommend provisions be provided to prevent runoff from the superjacent natural slope from flowing over the cut face.

Minor cuts in the alluvium have been made at about 1:2 slope ratios such as on the south side of the intersection at Oasis Road. Where the ground surface above the top of such cuts is relatively flat and no concentrated flows are directed toward the top of the cut, they appear to perform well with minor erosion. Where such cuts are planned across even minor drainage courses on the desert floor, provisions should be made to prevent the intermittent flows in the gullies from flowing over the cut face.

Erosion mitigation measures are proposed under Section 12.4 "Slope Stability." Nevertheless, District Landscaping should be made aware of this project and request provisions to prevent and/or control erosion.

Erosion of offsite areas to the south may also have an effect on the design of offsite drainage systems. Observation of sharply incised existing drainage courses and the sand/gravel/cobbles exposed in those courses infers that culverts will be subjected to heavy bedload and potential scour. We defer to the Hydrology Section for the design measures to accommodate stream bedload.

6.5 Scour

All creeks and streams, which lie within the project, were dry during our site reviews (April and July 2001). The bridges listed in chart 16.5 are not scour critical as per Mr. Mark Palmer, Senior Engineering Geologist, Scour Critical Program. (Ref. 15)

7. Groundwater Conditions

Our inquiry (Ref. 5) with Department of Water Resources revealed six wells located near the project limits:

16.5 Groundwater Elevation from Department of Water Resources

Well	Approx. groundwater Elevation	Approx. ground Elevation	Date
04N06W23M01S	895m/2937ft	1195m/3920ft	02/11/ 98
04N08W07C01S	1212m/3976ft	1281m/4202ft	03/17/99
05N09W25A01S	897m/2944ft	1042m/3417ft	11/26/56
05N07W30D02S	890m/2919ft	1033m/3390ft	10/2098

Note: Wells 04N07W35G01S and 04N07W27A01S reported by DWR are not included because no ground water was encountered. For well locations, see "Groundwater Level Data Retrieval Map Interface," attached in the appendices.

Groundwater was also encountered at different elevations during soil investigation, drilling, or construction of the existing bridges along the alignment as follows:

16.6 Groundwater Elevation at Bridges Along the Alignment

Bridge Name	Approx. groundwater Elevation	Approx. ground Elevation	Date	KP(PM)
Sheep Creek Bridge	Not encountered	1324m/4343ft	March 1966	5.8(3.6)
Pine Lodge West Overhead	974m/3194.5ft	984m/3229ft	1979/1980	23.0(14.3)
Pine Lodge East Overhead	952m/3125ft	958m/3142ft	1979/1980	23.8(14.8)
Cajon Creek Bridge	941m/3088ft	949m/3112ft	March 1966	24.0(14.9)
California Aqueduct	Not encountered	1062m/3483ft	Sept 1967	112.8(70.1)

We do not anticipate that groundwater will be encountered during earthwork operations. However, groundwater levels fluctuate by season and might be expected to be higher near the mouth of Cajon Canyon in Segment 2.

Availability of groundwater for use in construction operations has not been investigated. In Segments 1 and 3, water for earthwork and other construction purposes might be available from public hydrants in the communities adjoining the alignment.

8. Seismicity

Historical records indicate that the area has been subjected to a number of seismic events over the course of the last 182 years. Strong ground motion (defined as a horizontal ground acceleration in excess of about 0.5 g) was likely experienced in the project area in 1812, 1827, 1852, 1855, 1857, 1893, 1936, 1952, 1956, 1965, 1971, 1974, 1977, 1991, 1992, 1994 and 1999. The 1812 and 1857 events (estimated to have had moment magnitudes of approximately 7.0, and 8.0, respectively) are

thought to have occurred along the Mojave segment of the San Andreas fault and caused significant damage to developed areas of southern and central California. Similarly, the 1992 Landers earthquake and the 1999 Hector Mine earthquake had magnitudes of 7.5 and 7.1 respectively, however these latter two events did not damage existing Route 138 within the project limits (Ref. 16).

The geologic processes that have caused earthquakes in the past should be expected to continue. Seismic Map (Plate 17.4) is an excerpt from the Department's "California Seismic Hazard Map" of 1996. It depicts the location of the project and nearby State routes (route numbers shown in boxes) to nearby faults that might affect the site, and gives contours of peak bedrock accelerations. From the map we anticipate the project area could experience peak bedrock accelerations of 0.55g to 0.7g.

16.7 Seismicity

Fault Name	Type	Magnitude	Peak acceleration	Dist. to Fault	KP/PM
San Andreas/C (SAC)	Strike-Slip	8.00	0.7	2.6km/1.6mi	22.8/14.2
Cleghorn-North (CNF)	Reverse	7.75	0.7	1.9km/1.2mi	23.6/14.7
Llano Fault (LLO)	Unknown	6.00	0.6	3.8km/2.4mi	112.8/70.1

As shown on the California Seismic Hazard Map of 1996, the closest fault to the project site is the Cleghorn-North Frontal fault, a reverse-thrust fault thought capable of generating a Magnitude 7.75 event. That fault is 1.9 km (1.2 miles) from the alignment of Route 138. As such, it is unlikely that surface rupture will cause significant horizontal and vertical surface displacement at the site.

Engineering implications of the site seismicity are discussed in Section 12.2.

9. Landslides and Rockfall

Neither landslides nor rockfall are anticipated within the three project segments in San Bernardino and Los Angeles Counties, because of the generally flat topography of the area. Proposed cuts in bedrock in Segment 1-2 are expected to perform well at 1(V):2(H) slope ratios, as judged by either the performance of the existing cut slopes and natural slopes above at PM 8.64±, or due to the hardness of the rock exposed at the Mormon Rocks. Slope failures are not likely on proposed fill slopes graded at 1(V):2(H) or flatter assuming they are constructed in accordance with the Standard Specifications of earth materials generated from the vicinity of the project alignment, and are of heights less than or similar to the existing embankments at the Pine Lodge Overheads (10± m).

10. Snow Avalanches

Although snow does fall within the project limits, a snow avalanche is highly unlikely due to the generally flat topography of this project and the distance from the neighboring mountains. This project lies approximately 8 km(5 mi.) northeast of the base of the San Gabriel Mountains, so even the potential run-off zone from a very large snow avalanche is considered very low.

11. Geothermal Activity

No known geothermal activity was identified within the project limits.

12. Geotechnical Considerations

12.1 Embankment

The geotechnical aspects discussed in this section are based on review of the existing facilities plus the previous Log of Test Borings for five bridges on the project site. In general, alluvial soils encountered at the site and excavation from the proposed cut slopes at Mormon Rocks and PM 8.64 can be used as fill for embankments. Depending on how the contractor rips or blasts the rock at Mormon Rocks, oversized fragments may be generated that may be unsuitable either for incorporating into the embankment, or requiring additional effort to to break up for placement in the fill.

For Segments 1 and 2, we anticipate the height of embankment will generally be less than 1 to 1.5 m except near PM 8.75 (Segment 2), where we anticipate the embankment will be about 5 m above the drainage course. Fill depths within the existing drainage course (see Exhibit H) may be deeper such as Segment 1-2 where the fill is a maximum of 20 m deep. Within Segment 2, existing approach fills to the Pine Lodge Overheads will be widened and will have a height of about 10± m. For Segment 3, we anticipate new embankment heights up to 5± m.

The Earthwork Factor (EWF) for the alluvial soils may be assumed to be 0.8 and for materials derived from the Mormon Rocks EWF may be assumed to be 1.15. Occasional boulders can be expected in the alluvial soils of Segment 1, 1-2 and 3.

12.2 Ground Shaking and Liquefaction

Ground shaking is expected to occur at the site considering the predicted magnitude and peak ground accelerations of earthquakes along nearby faults. Ground shaking could cause densification of loose soil layers and consequently some distress to the roadway structure. However, the potential for liquefaction is not anticipated based on ground water depth and generally dense nature of the subsurface granular soils. Furthermore, a nearly at-grade highway with flexible pavement is less susceptible to the effects of earthquake induced ground shaking than perhaps any other types of highway.

Surface fault rupture is not anticipated to affect the roadway.

12.3 Settlement

Embankment settlement from foundation compression will not be a major issue for the project due to deep groundwater level, granular foundation soils and low to moderate embankment heights. In the absence of high groundwater and fine grained foundation soils, we anticipate that settlement will occur as the embankments are being constructed. Waiting periods are not expected. However, loose alluvial deposits in drainage courses to be filled or, in the bottoms of existing debris basins in Segment 1-2 and 3 should be at least partially subexcavated. Depths of subexcavation will be explored in the design phase, but are not anticipated to exceed 0.5 to 1 m.

12.4 Slope Stability

In general, with future cut and fill slopes being graded at 1:2 or flatter and being constructed of, or in, granular materials to moderate slope heights of 20± m for fills and 67 m for cuts, slopes should be stable against deep seated failure. However, for the cut slope at KP 13.9 to KP 14.0 (PM 8.64 to PM 8.71) the geologic map depicts the bedding as dipping at 20 degrees to the north. Our field investigation revealed the bedding dipping to the northeast away from the roadway. As this slope is currently stable, we feel with the flattening, this slope will be stable at 1(V):2(H).

With the bedding plane orientation in the Mormon Rocks dipping favorably to the north (the roadway turns to the east), and in the absence of other, adversely oriented rock discontinuities, the rock hardness and geologic structure appear to be favorable for making a steep cut. Accordingly, it is our opinion that this cut may be made at a slope ratio of 1:0.5 or flatter, but that a catchment area be provided along the toe of the cut in conformity with the Ritchie criteria (see Appendix).

13. Corrosion

Seven soil samples were collected at selected sites for a few proposed culverts and structures and tested for corrosion potential as per Caltrans Corrosion Guidelines 1996. The test results show soils to be noncorrosive.

16.8 Corrosion Test Results Summary

Sample Identification	KP/PM	Depth (ft)	pH	Minimum Resistivity (Ohm-cm)	Sulphate Content (ppm)	Chloride Content (ppm)
LA3-1	112.65/70	.5 to 2	7	12000	N/A	N/A
LA3-2	117.48/73	.5 to 2	7.1	12000	N/A	N/A
SBd1-3	1.61/1	.5 to 2	7.5	22000	N/A	N/A
SBd1-4	3.22/2	.5 to 2	7.9	9000	N/A	N/A
SBd1-5	4.83/3	.5 to 2	7.7	11000	N/A	N/A
SBd2-6	16.09/10	.5 to 2	7.5	18000	N/A	N/A
SBd2-7	19.31/12	.5 to 2	8.6	12000	N/A	N/A

Note: Samples were obtain from Soil Profiler/hammer and shovel in zip bags

District Materials Report has additional corrosion information.

14. Hazardous Waste Impact

As of the time of writing this report, we were not aware of any locations along the proposed alignment suspected of containing hazardous materials. We will need information from the District Hazardous Waste Unit about the location of any hazardous materials prior to conducting our subsurface exploration.

15. Recommendations

15.1 Exploration and Investigations

The granular nature of the material for the project will provide an acceptable base for a roadway. As LOTB sheets already exists for the existing bridges, this exploration is already done. At the location of the proposed cut at PM 8.6±, if during grading adverse bedding is discovered, please contact this Office for additional exploration.

15.2 Cut Slopes

The cut slope at Segment 2 KP 13.9 (PM 8.6) may be designed at a 1(V):2(H) slope ratio. If the top of the cut will daylight with the natural rising slope above, a brow ditch is to be installed. Slopes refered to in the "Geotechnical Investigation", June 28, 2001, Segment 1-2, Cuts 1L, 6L, 7L, 8L, 9L, 11L, 29L, 35L, 36L, 38L, 6bR, 14R and 15R shall have a brow ditch installed. Cuts 18L,19L, 20L, 6aR, 6bR, 6cR and PM 8.6 shall have an erosion mat installed on the entire slope face not just intermittent straw wattle.

The cut slope at the Mormon Rocks may be graded at a slope ratio of 1:0.5 or flatter to a height of 12 m. A 4.5 m wide catchment area should be graded between the toe of the cut and the edge of the traveled way. The catchment area should be graded uniformly from the edge of shoulder to the toe of cut, and the toe of the cut slope should be 0.5 m below the edge of traveled way elevation.

15.3 Embankment

Embankment slopes should be graded at 1(V):2(H) or flatter. Measures such as AC dikes at the top of embankments are recommended to prevent flow from the roadway over the slope face; however, dikes may be omitted where erosion control measures, as selected by the District Landscape Architects, are used.

Embankment foundations shall be prepared in accordance with Section 19 of the Standard Specifications. For estimating purposes, assume 0.5 m of alluvium must be subexcavated from embankment foundation areas in drainage course channels and replaced as compacted fill.

15.4 Excavation Techniques

Excavations can be accomplished by conventional techniques for the project, except for cuts in the Mormon Rocks in Segment 2. Seismic Refraction Surveying has been performed to provide an assessment of the rippability of these resistant sandstone features. These outcrops contain a weathered horizon that appears rippable to a depth of approximately 2 meters. At depths greater than 2 meters, the outcrop is unrippable to subgrade elevation and blasting will likely be required. If blasting is not viable then realignment of the proposed road should be considered.

97/8051
EF
due 10/23/97

Rec'd 9/30/97 B.D.

AGRA
Earth & Environmental

AGRA Earth &
Environmental, Inc.
1290 North Hancock Street
P.O. Box 19079
Anaheim, CA 92817
Tel (714) 779-2591
Fax (714) 779-8377

September 16, 1997
Job No. 6-212-105200

RECEIVED
SEP 22 REC'D
DEPT OF TRANSPORTATION
CONSULTANT SERVICES

Rancho Las Flores Limited Partnership
33971 Selva Road
Suite 230
Dana Point, California 92629

Attention: Mr. Clifford C. Hood

Re: Addendum To Geotechnical Design Report
Route 138, Phase II
1.2 Miles East Of Interstate 15 To Summit Post Office Road
08-SBd-138-16.4/19.7

1.0 INTRODUCTION

This addendum report presents revisions to the Geotechnical Design Report (dated December 10, 1996) prepared for this project by AGRA Earth & Environmental, Inc. (AEE). The revisions are a result of comments presented in the Review Memorandum prepared by Caltrans (dated January 22, 1997) and a subsequent change in the project alignment. Unless specifically superseded by this addendum, the recommendations presented in the original Geotechnical Design Report are still considered applicable to this project.

2.0 GENERAL DISCUSSION

The Caltrans review comments necessitate changes to both Appendix E - Pavement Design Report and Appendix F - Corrosion Study. Revised appendices are attached.

Subsequent to the issuance of the Geotechnical Design Report portions of the proposed Route 138 alignment were modified. Between about Station 85 to Station 165, the alignment was shifted to the left a distance varying up to approximately 260 feet. In order to evaluate the geotechnical impact of the alignment shift additional geologic field mapping was performed. The geologic mapping in relation to the revised alignment is presented on Plates I through V. Revised cross-sections are presented on Plates VI and VII.

Based on the geologic conditions encountered during the supplemental field work no change in the recommendations for cut or fill slope gradients are recommended. The major cut slopes in the Crowder Formation and Terrace Deposits within the eastern portion of the project were

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unaffected by the alignment shift. The roadway embankment of the new alignment will traverse several drainages and areas of landslides and/or surficial slumps. These areas will require remedial grading (clean-outs) prior to embankment construction as outlined in the following section.

3.0 EMBANKMENT FOUNDATIONS

Complete removal of loose alluvium (Qal), slope wash (Qsw), landslide material (Qls), and existing fill (Af, Afe) below proposed embankment was recommended in the original Geotechnical Design Report. This recommendation is also applicable to the revised alignment. (Refer to Plates I through V for location of these materials). Several landslide areas will be traversed by the new alignment. Although these features are considered to be fairly surficial in nature, it is recommended that they be completely removed. Where removals are required at the toe of proposed fill slopes a slot key (minimum depth of 3 feet and width of 15 feet) should be excavated into competent material below the toe of the slope. Typical removals and slot key excavation are shown on the cross-sections. Removal depths in areas of alluvium slope wash, and existing fill are expected to generally range from approximately 3 to 10 feet. Landslide removal depths are expected to generally be less than 15 feet, however, within the larger landslide mass encountered between approximately Station 141 and 144, removal depths on the order of 20 to 25 feet should be anticipated.

4.0 CLOSURE

This addendum report is based on the project as described in the original Geotechnical Design Report and as modified herein, information obtained from referenced reports prepared by others, and from the field exploration and laboratory testing performed by AEE. Our findings are based on the results of the field, laboratory, and office studies, combined with an interpolation and extrapolation of soil conditions between and beyond the borings. The results reflect our interpretation of the limited direct evidence obtained. Our firm should be notified of any pertinent change in the project plans. If subsurface conditions are found to differ from those described herein, it may require a re-evaluation of the recommendations.

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

Respectfully submitted,

AGRA Earth & Environmental, Inc.

Larry E. Fanning
Senior Engineering Geologist
Registered Assessor REA 04677
RG 6118, CEG 1907
(Expires January 31, 1999)

Douglas R. Bell
Supervising Engineer
GE 2140
(Expires March 31, 1999)

LEF/DRB/ljo

Encl.: Revised Appendix E
Revised Appendix F
Plates I through V, Geologic and Field Exploration Plans
Plates VI and VII, Cross Sections

c: Mr. Clifford C. Hood, Addressee (2)
Mr. John Starner, C. M. Engineering (8)

01/6-1052.AD/09-16-97

ENCLOSURES

APPENDIX E

PAVEMENT DESIGN REPORT
ROUTE 138 PHASE 2
08-SBd-138 - 16.4/19.7

The recommendations presented in this report are based on field exploration and testing performed by AGRA Earth & Environmental, Inc. (AEE) as presented in the Supplemental Geotechnical Design Report. Reference should be made to the geologic and geotechnical conditions described in that report as well as the presentation of laboratory testing on the Soil Summary Sheets, Plate VI through X. Previous testing performed by John A. Sayers & Associates (JASA) was also used as a data base. The testing by JASA is also compiled in the Supplemental Geotechnical Design Report.

A summary of Resistance Value testing performed is presented below:

Boring No.	@ Depth/ Sample No.	Material Type	R-Value (Stabilometer)
B1	@ 7.5'	Older Alluvium: Silty Sand	78
B6	@ 3.5'	Older Alluvium: Silty Sand	68
B10	@ 0 - 3'	Slope Wash: Silty Sand	61
B15	@ 0 - 3'	Alluvium: Silty Clayey Sand	67
B16	@ 0 - 3'	Alluvium: Silty Sand	61
101	2	Fanglomerate: Fine to Medium Sand	70
Test Pit 2	5	Fanglomerate: Silty Sand with Gravel	73
103	2	Crowder Formation: Silty Sandstone	42
107	3	Crowder Formation: Clayey Sandstone	48
108	9	Crowder Formation: Clayey Sandstone	15
109	7	Terrace Deposits: Fine to Medium Silty Sand	74

Based on the testing performed, it is our opinion that a project design R-Value of 50 is appropriate. An R-Value of 50 should be readily achievable within the Terrace Deposits and younger alluvial soils which is expected to provide most of the fill embankment material for this project. It is expected that the Crowder Formation, particularly the more clayey units will not yield an R-Value of 50. Fill soils derived from the Crowder Formation should be omitted from the upper 4 feet of embankment fill or be mixed with other on-site soils to attain an R-Value of greater than 50. Where pavement subgrade will be founded in Crowder Formation (predominantly cut areas between approximately Station 144 and 189) lower R-Value material may be exposed at subgrade elevation. In this case, overexcavation of the unsuitable material to a depth of 4 feet and replacement with suitable R-Value material will be required.

With the above conditions the recommended pavement sections, as designed in accordance with the Caltrans Highway Design Manual, are as follows:

Design Condition	Drained Section		Undrained Section		Full Depth Section	
Main Line - Highway 138, 10-year T.I. = 8.0	0.40′	Asphaltic Concrete	0.40′	Asphaltic Concrete	0.65′	Asphaltic Concrete
	over 0.25′	Asphalt Treated Permeable Base	over 0.45′	Class 2 Aggregate Base		
	over 0.35′	Class 2 Aggregate Base				
Main Line - Highway 138, 20-year T.I. = 10.0	0.50′	Asphaltic Concrete	0.50′	Asphaltic Concrete	0.85′	Asphaltic Concrete
	over 0.25′	Asphalt Treated Permeable Base	over 0.65′	Class 2 Aggregate Base		
	over 0.35′	Class 2 Aggregate Base				
Shoulder - Highway 138, 10-year T.I. = 5.0			0.20′	Asphaltic Concrete	0.35′	Asphaltic Concrete
			over 0.35′	Class 2 Aggregate Base		
Shoulder - Highway 138, 20-year T.I. = 6.0			0.25′	Asphaltic Concrete	0.45′	Asphaltic Concrete
			over 0.35′	Class 2 Aggregate Base		

Asphaltic concrete, asphalt treated permeable base and Class 2 Aggregate Base should conform to the current Caltrans Specifications. Class 2 Aggregate Base should have a maximum size of 3/4-inch. Asphaltic concrete should be Type A with an aggregate grading of 3/4-inch maximum, medium.

If a drained structural section is utilized for the main line, the design should incorporate edge drains discharging to a positive outlet (refer to Caltrans Highway Design Manual Section 606.2).

A number of aggregate quarry sites exist in the general area of the Route 138, Phase 2 project. A list of active quarries which were contacted and stated that they can supply Class 2 Aggregate Base is presented below. Testing of quarry material was not performed as part of this study, therefore the quality of available material has not been confirmed. The list is for informational purposes and is not intended to be exclusive. Other suitable quarry sites may exist in the proximity of this project.

Operator	SMARA I.D. No.
Sun West Materials Lytle Creek Facility 3221 North Riverside Avenue Rialto, California 92376 (909) 350-1846 or (800) 801-7625	91-36-0040
Calmat 2400 West Highland Avenue San Bernardino, California 92405 (909) 875-1150 or (800) 225-6280	91-36-0012
Hi-Grade Materials Co. 17671 Bear Valley Road Hesperia, California 92345 (619) 244-9325	91-36-0042 (Lucerne Valley Quarry) 91-19-0026 (Little Rock Quarry) 91-36-0046 (Oro Grande Quarry)

APPENDIX F

CORROSION STUDY
ROUTE 138, PHASE 2
08-SBd-138 - 16.4/19/7

This study presents design recommendations for culvert construction for the Route 138 Phase 2 project. The recommendations are based on field exploration and laboratory testing performed by AGRA Earth & Environmental, Inc. (AEE) as presented in the Supplemental Geotechnical Design Report. Reference should be made to the geologic and geotechnical conditions described in that report as well as the presentation of laboratory testing on the Soil Summary Sheets.

During the field exploration a cursory inspection of existing culverts within the current roadway alignment was performed. The culverts were constructed of corrugated steel pipe (CSP). The ages of the culverts are not known, however, it is likely that they date back to the original construction of existing Route 138. The condition of the culverts was rated as fair to good. No perforations of the culvert shells were noted within sight of either the inlet or outlet.

Corrosion samples were obtained at the proposed major culvert crossings using hand auger techniques (Samples CS-1 through CS-24). In addition, a number of samples were selected from deep borings drilled within proposed cut areas. Resistivity, pH and sulphate tests were performed on all the samples. Chloride tests were performed on selected samples with the lowest minimum resistivity values. The sample locations and test results are summarized on the Soil Survey Sheets, Plate VI through X.

Based on the testing performed it is evident that the geologic units along the alignment can be grouped into two classes. The Terrace Deposits and younger alluvial soils, which will be encountered at the invert elevation of proposed culverts have low corrosion potential. The Crowder Formation, which could potentially be included in embankment fill over proposed culverts, has moderate corrosion potential. A summary of recommended design parameters for both groups is presented as follows.

Material	Type A Terrace Deposits and Younger Alluvial Soils	Type B Crowder Formation
Minimum Resistivity (ohm-cm)	6,980	1,190
pH	6.8	7.2
Sulphate Content (ppm)	227	138
Chloride Content (ppm)	43	96
Design Life of Galvanized 18 Gage CSP (years)	26	23
Required Gage Thickness For CSP 25 Year Design Life		
- Galvanized	18	16
- Galvanized, Bitumen Coating	18	18
- Galvanized, Bitumen Coating, Paved Invert	18	18
Required Gage Thickness For CSP 50 Year Design Life		
- Galvanized	12	12
- Galvanized, Bitumen Coating	14	12
- Galvanized, Bitumen Coating, Paved Invert	16	14
Culvert Material Alternatives	Aluminum or aluminized steel is acceptable.	Neither aluminum nor aluminized steel is acceptable.

Where culverts will be founded within alluvial soils and embankment fill to be placed around the culvert will be composed of similar material, the Type A design parameters may be used for metal conduit. If there is uncertainty as to the type of embankment fill to be used at a specific culvert location we recommend that the Type B design parameters be applied to metal conduit.

The corrosion testing indicates that none of the geologic units within the roadway alignment present a corrosive environment for unreinforced or reinforced concrete pipe.

PLATES

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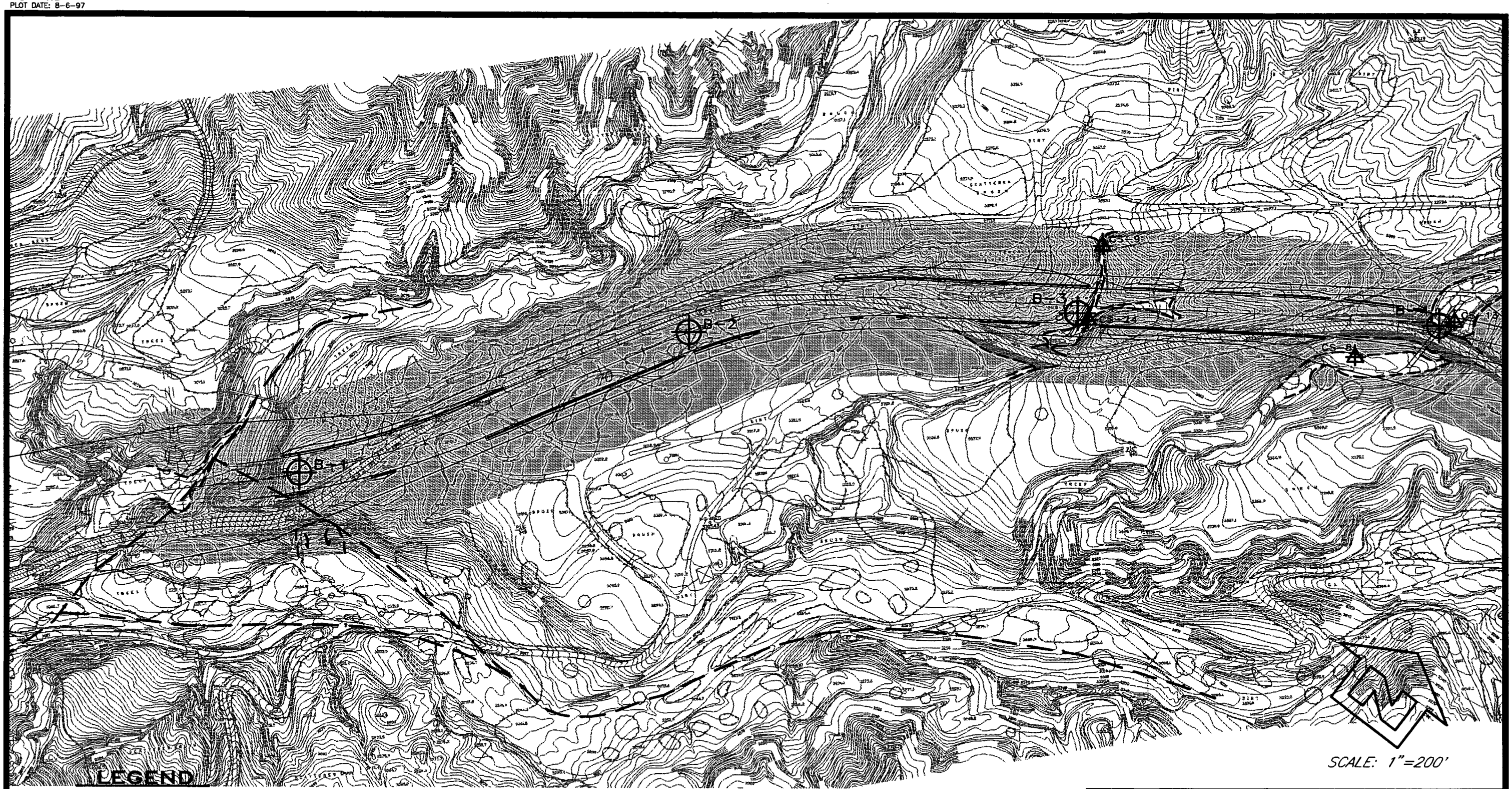

LEGEND

- Af – NONENGINEERED FILL
- Afe – ENGINEERED FILL
- Qal – ALLUVIUM
- Qoal – OLDER ALLUVIUM
- Qls – LANDSLIDE
- Qf – FANGLOMERATE
- Qsw – SLOPE WASH
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- Qh – HAROLD FORMATION
- QTc – CROWDER FORMATION
- MARSH AREA
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- AGRA EARTH & ENVIRONMENTAL, Inc. TEST PIT LOCATION
- AGRA EARTH & ENVIRONMENTAL, Inc. CORROSION SAMPLE LOCATION
- BORING LOCATION BY JOHN A. SAYERS & ASSOCIATES
- EXISTING ROADWAY
- GEOPHYSICAL SEISMIC LINE
- GEOLOGIC CONTACT–DASHED WHERE APPROXIMATE, DOTTED WHERE CONCEALED
- FAULT–DASHED WHERE APPROXIMATE, DOTTED WHERE CONCEALED

ROUTE 138, PHASE 2
POST MILE 16.4/19.7
SAN BERNARDINO COUNTY, CALIFORNIA

GEOLOGIC AND FIELD EXPLORATION PLAN

AGRA
Earth & Environmental, Inc.

FIELD	DRAFT	APPROV	DATE	JOB No.
GB/LEF	JBD		9/16/97	6-212-1052

PLATE 1

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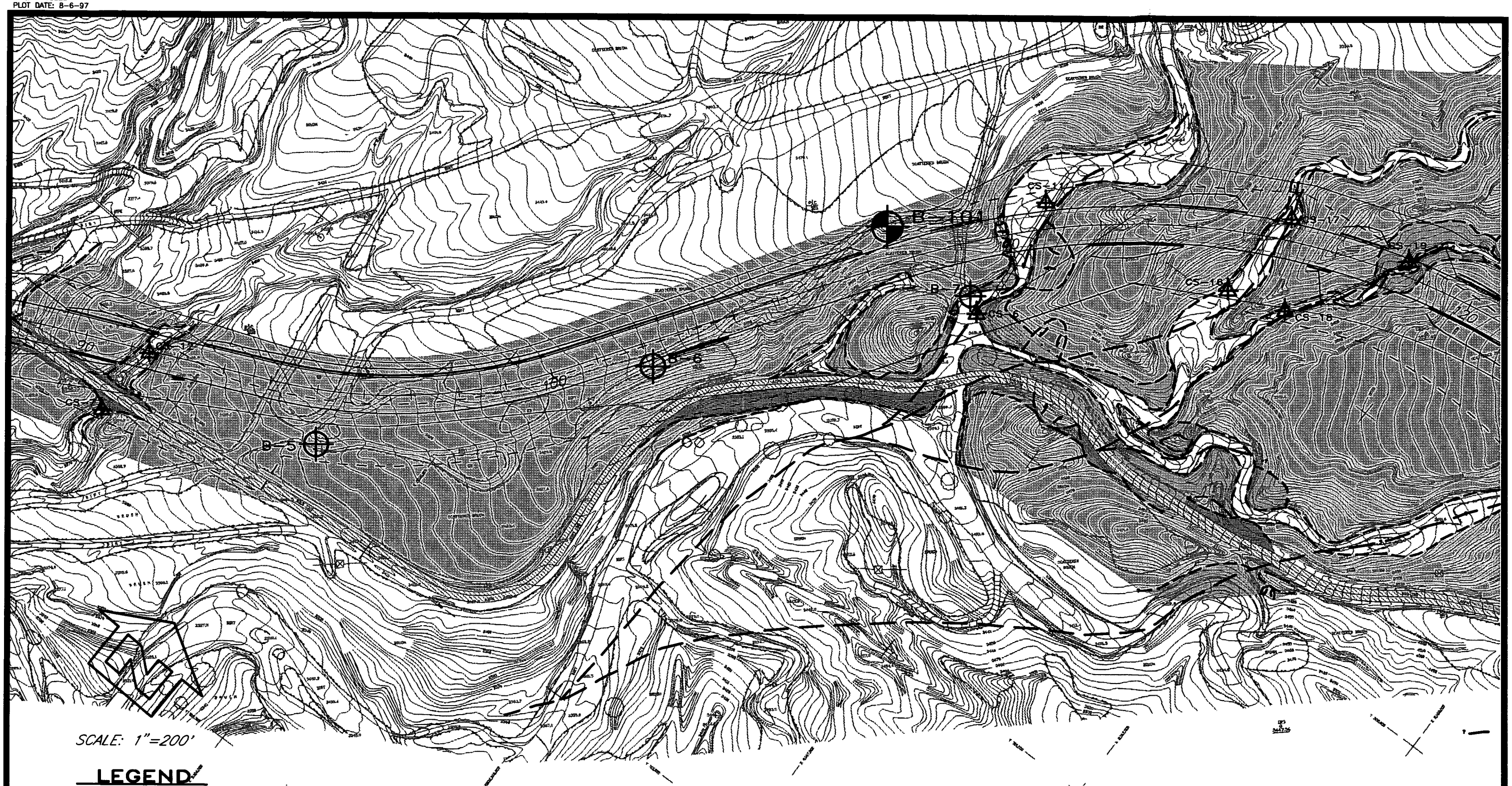
PLOT DATE: 8-6-97



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ROUTE 138, PHASE 2
POST MILE 16.4/19.7
SAN BERNARDINO COUNTY, CALIFORNIA

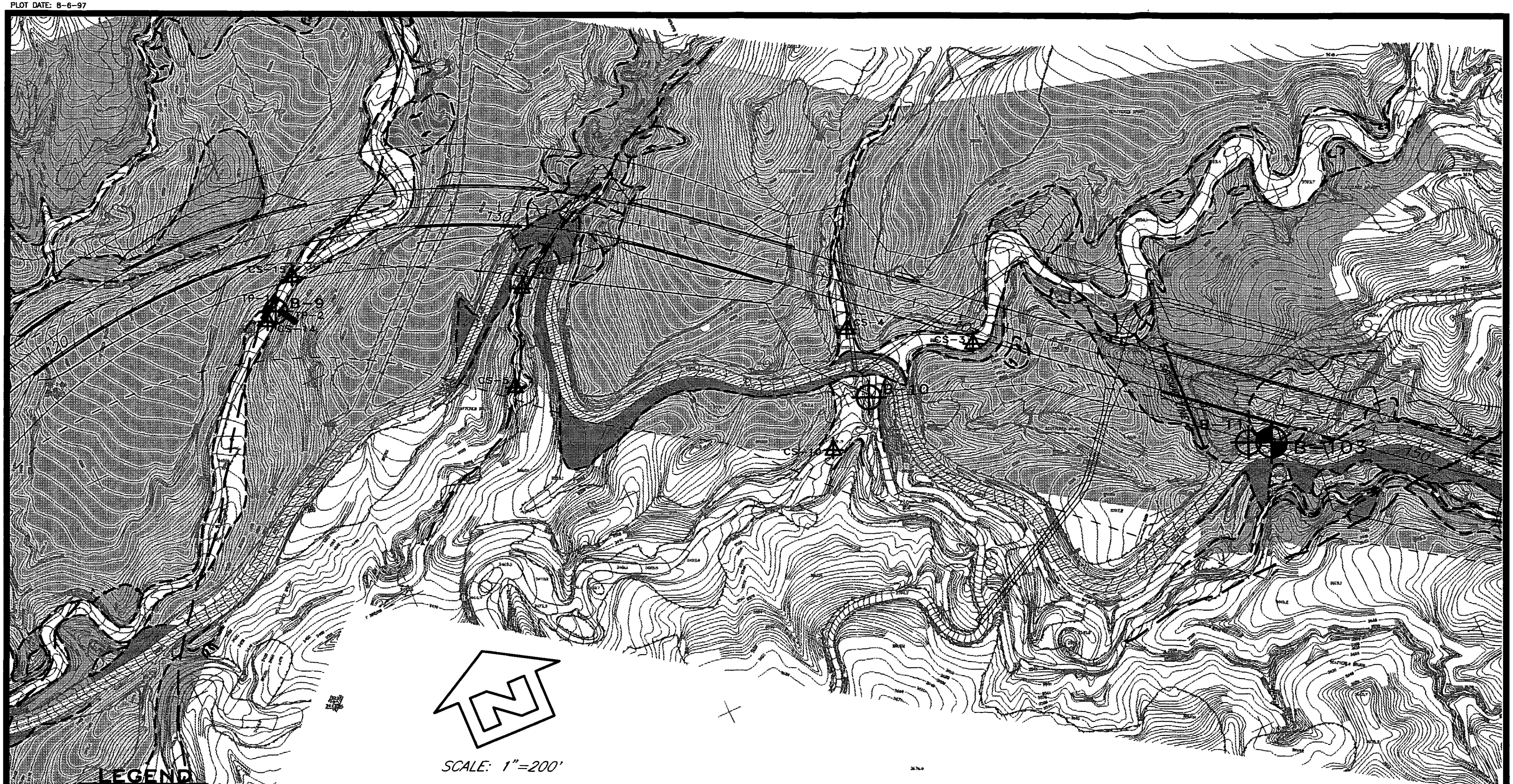
GEOLOGIC AND FIELD EXPLORATION PLAN

AGRA
Earth & Environmental, Inc.

FIELD	DRAFT	APPROV	DATE	JOB No.
GB/LEF	JBD		9/16/97	6-212-1052

PLATE II

FILE: 6-1052M
PLOT DATE: 6-6-97



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ROUTE 138, PHASE 2
POST MILE 16.4/19.7
SAN BERNARDINO, CALIFORNIA

GEOLOGIC AN FIELD EXPLORATION PLAN

AGRA
Earth & Environmental, Inc.

FIELD	DRAFT	APPROV.	DATE	JOB No.
GB/LEF	JBD		9/16/97	6-212-1052

PLATE III

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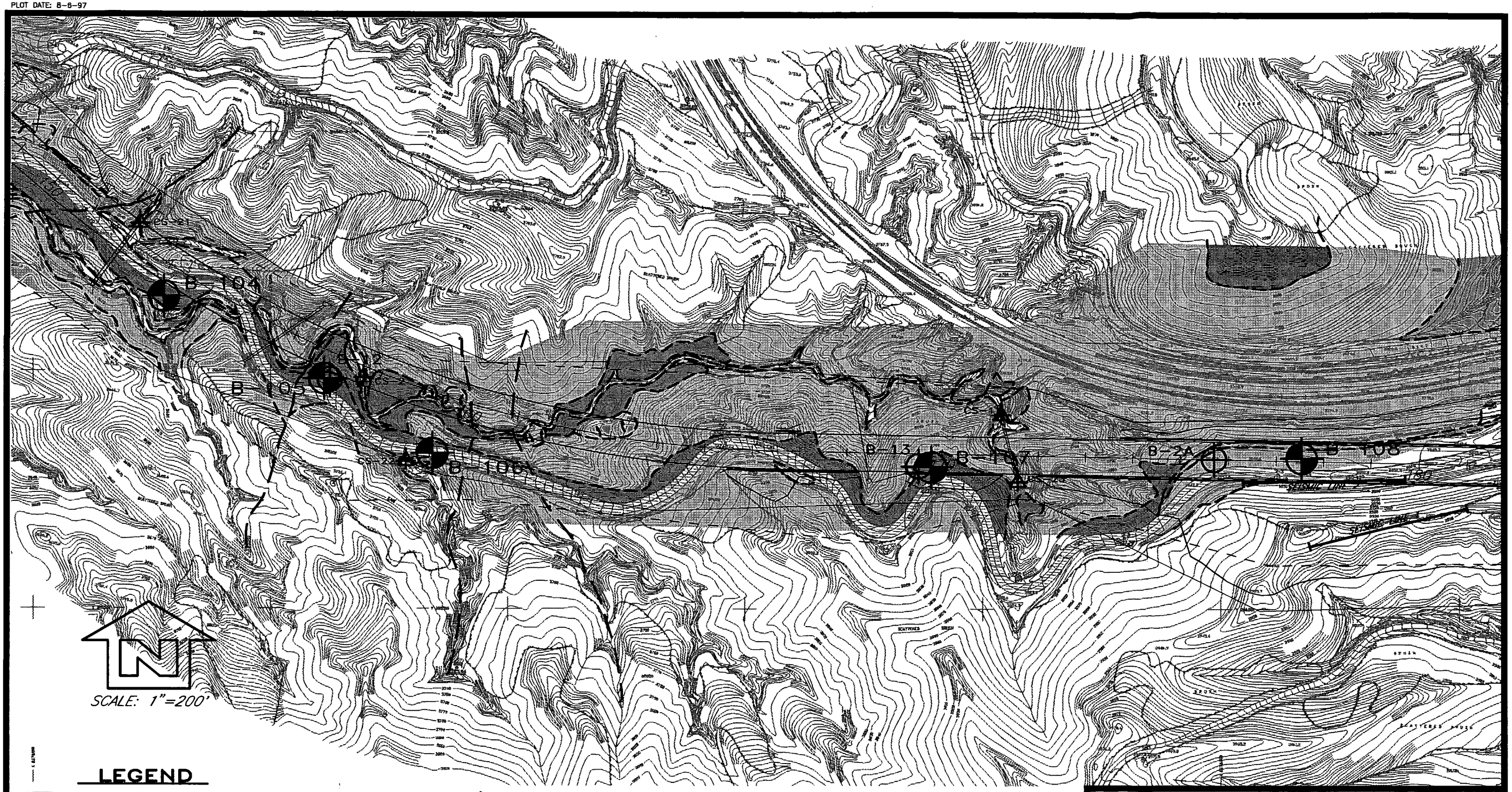
PLOT DATE: 8-6-97



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ROUTE 138, PHASE 2
POST MILE 16.4/19.7
SAN BERNARDINO COUNTY, CALIFORNIA

GEOLOGIC AND FIELD EXPLORATION PLAN

AGRA
Earth & Environmental, Inc.

FIELD	DRAFT	APPROV.	DATE	JOB No.
GB/LEF	JBD		9/16/97	6-212-1052

PLATE IV

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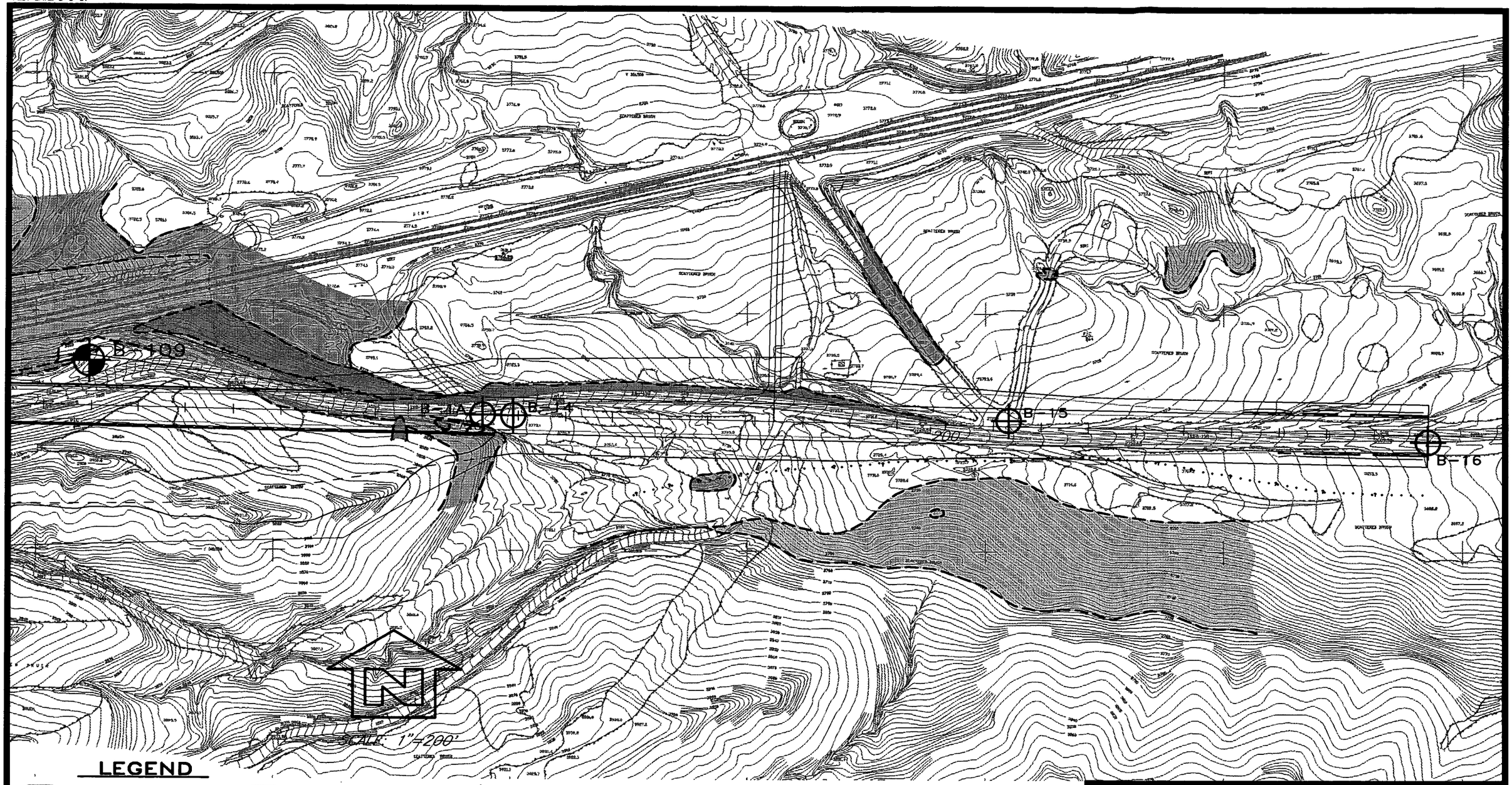

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ROUTE 138, PHASE 2
POST MILE 16.4/19.7
SAN BERNARDINO COUNTY, CALIFORNIA

GEOLOGIC AND FIELD EXPLORATION PLAN

AGRA
Earth & Environmental, Inc.

FIELD	DRAFT	APPROV.	DATE	JOB No.
GB/LEF	JBD		9/16/97	6-212-1052

PLATE V

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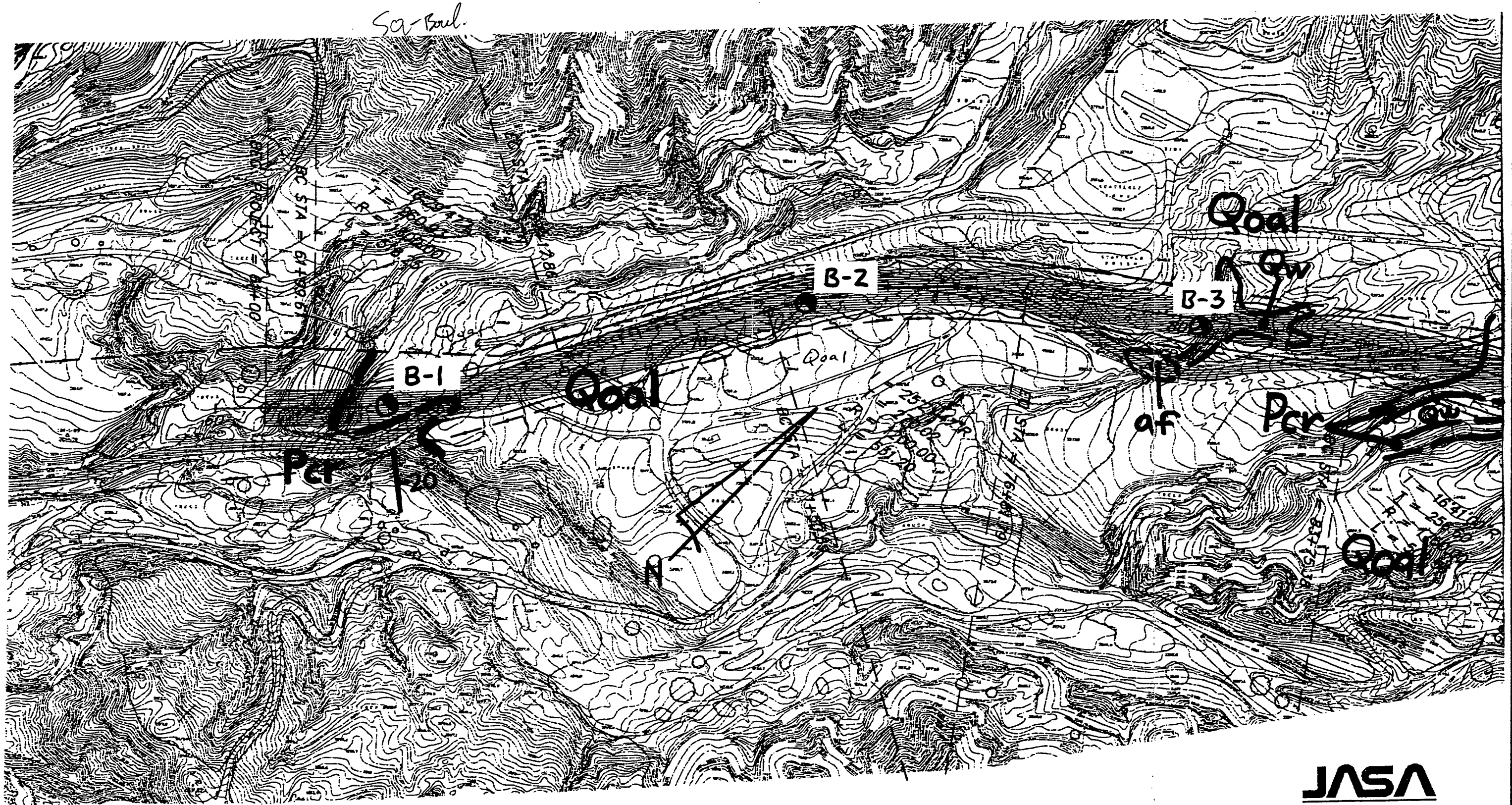

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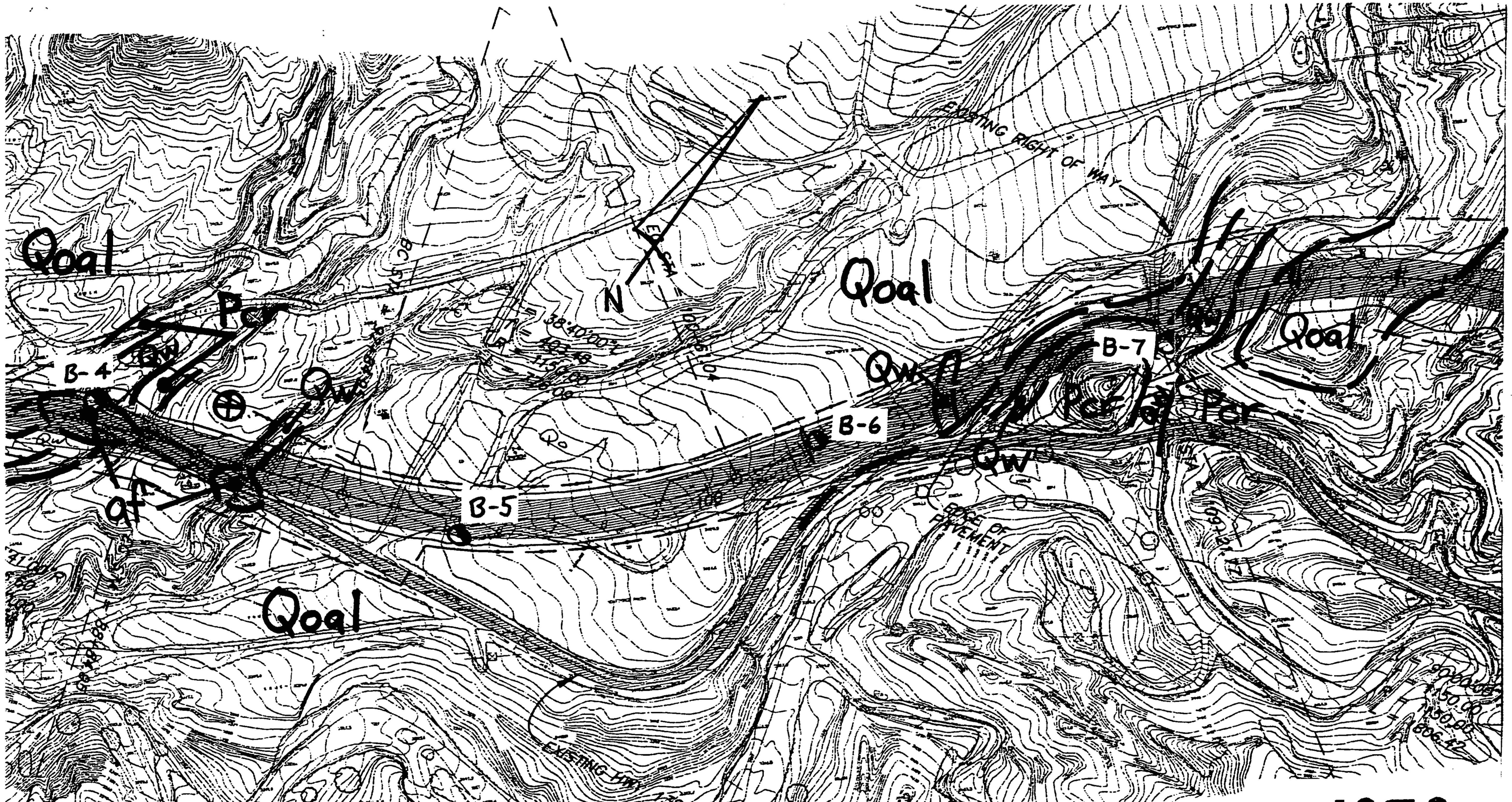
1.3 mi.

JASA

John A Sayers & Associates

W.O. 215-1-03 DATE 3/29/96

SCALE 1" = 200' PLATE D-2



JASA
John A Sayers & Associates

W.O. 215-1-03 DATE 5/29/96

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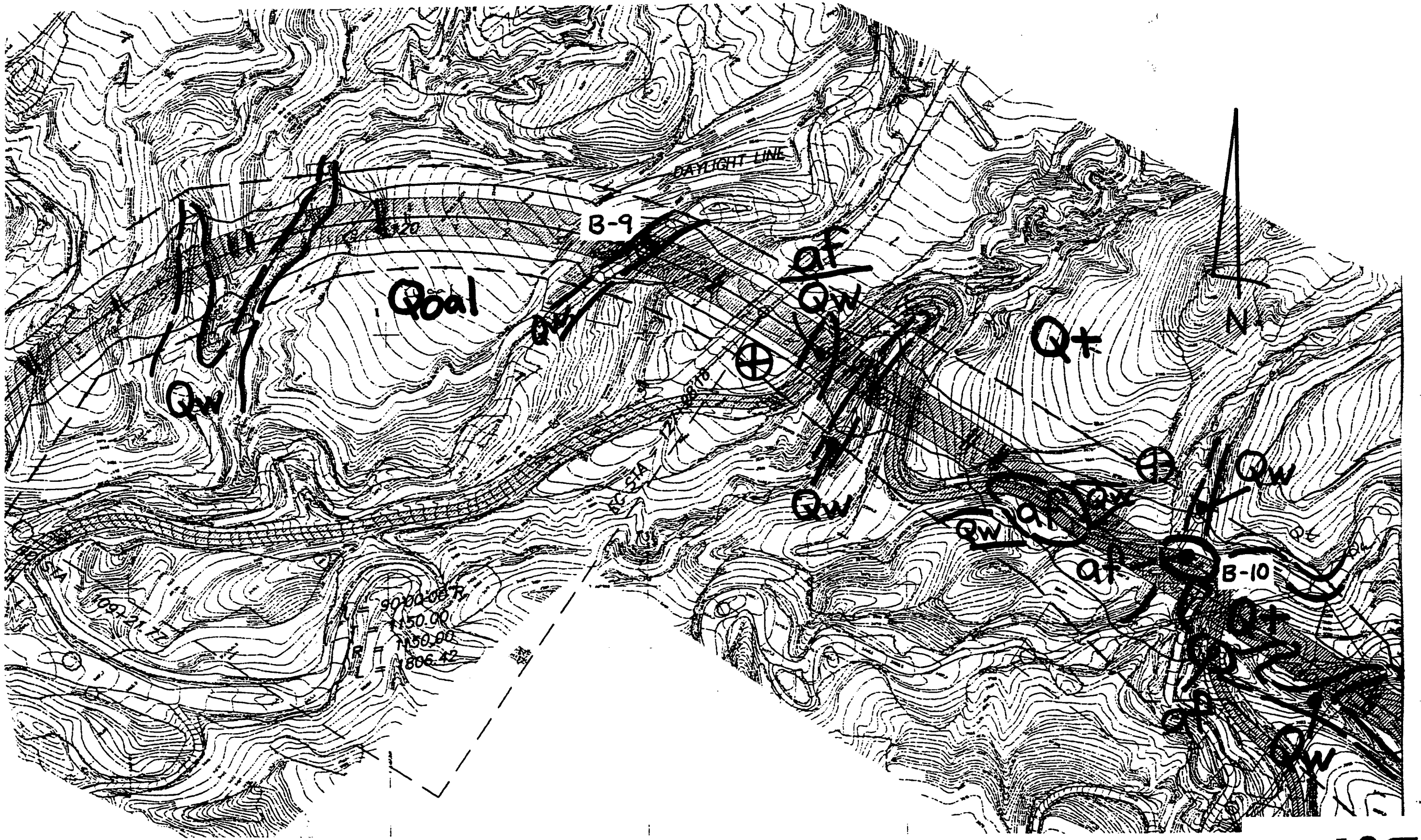

JASA

John A Sayers & Associates

W.O. 215-1-03 DATE 5/29/96

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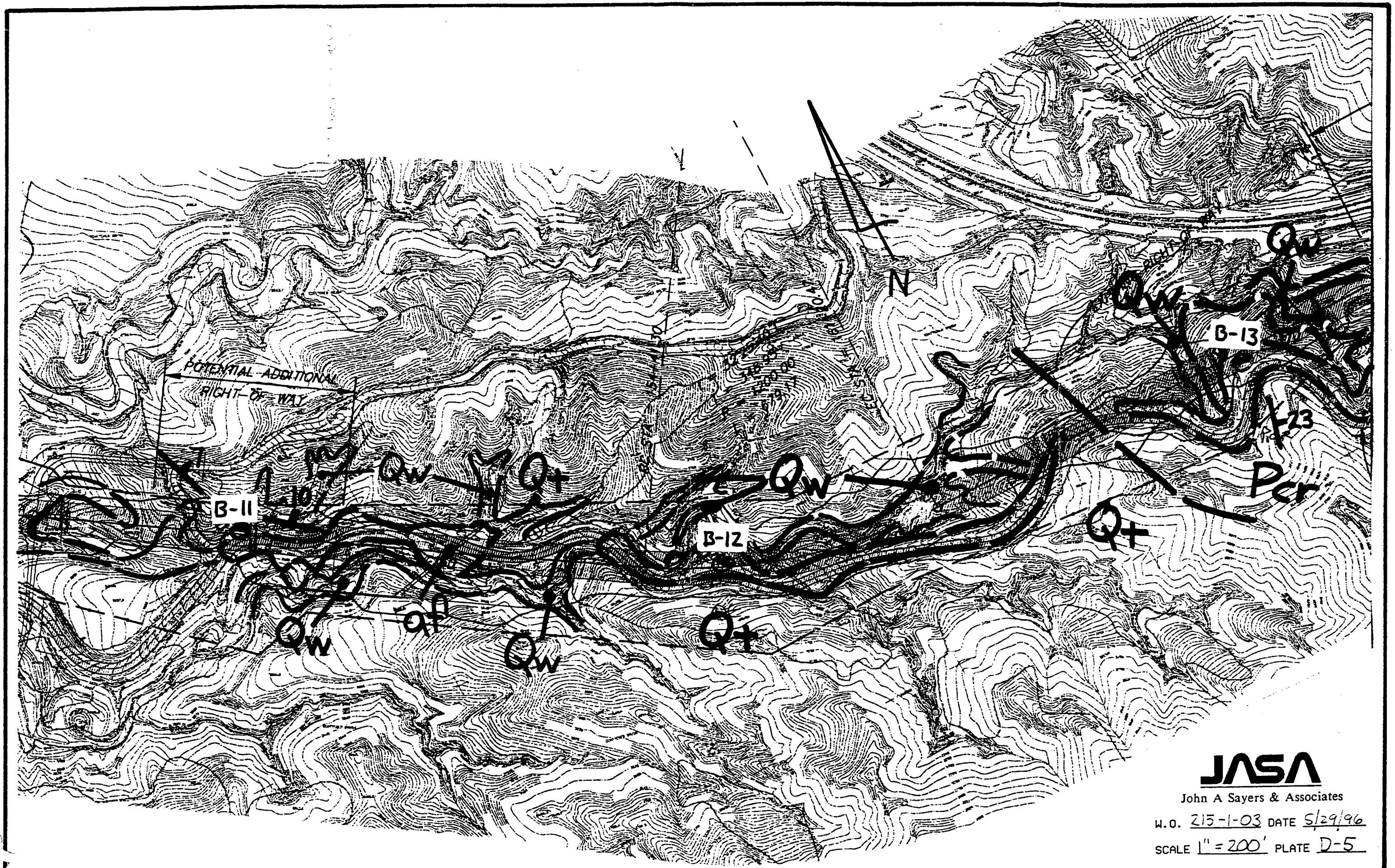
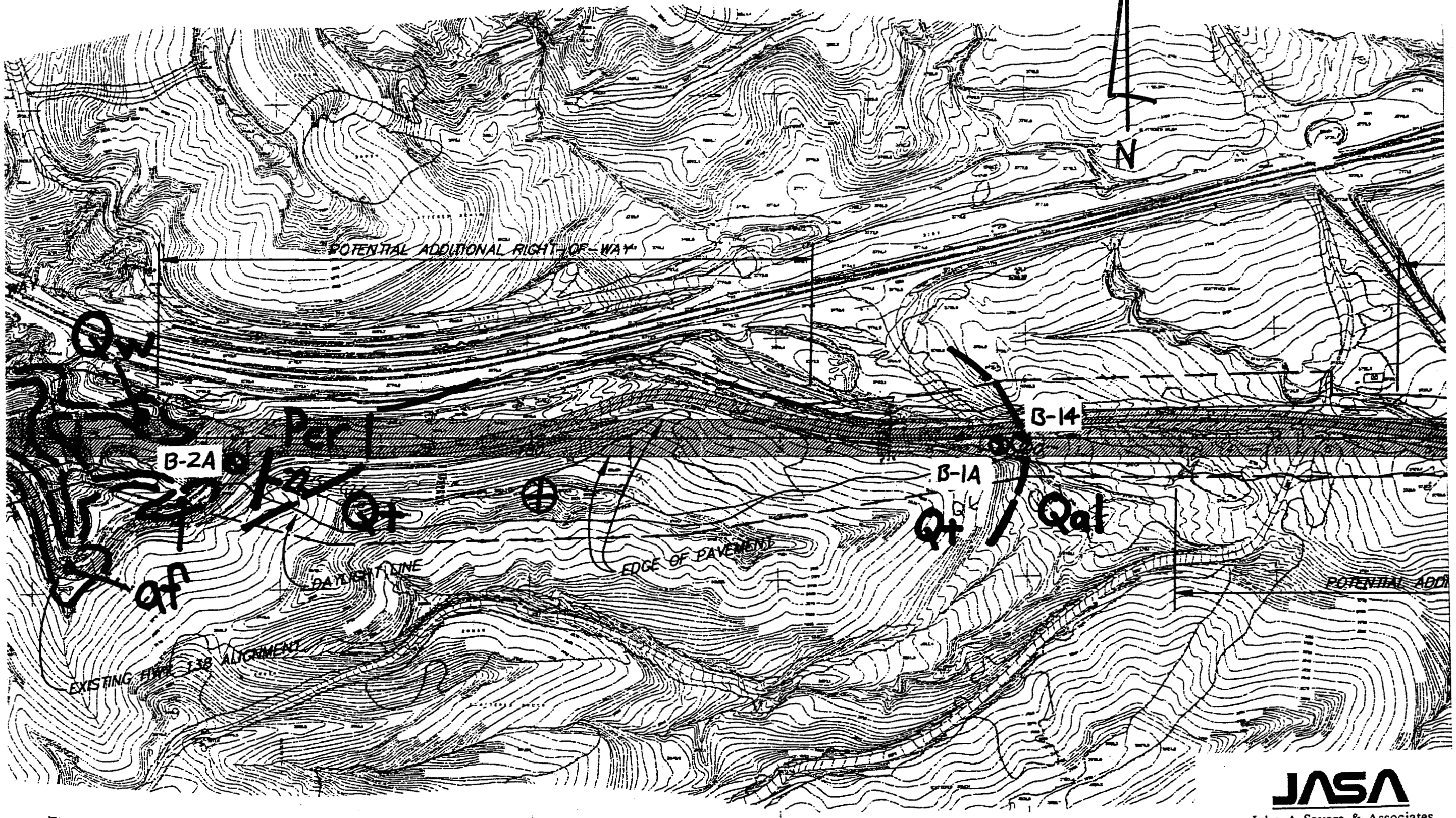


JASA

John A Sayers & Associates

W.O. 215-1-03 DATE 5/29/96

SCALE 1" = 200' PLATE D-5



JASA
John A Sayers & Associates

W.O. 215-1-03 DATE 5/29/96

SCALE 1"=200' PLATE D-6

AGRA
Earth & Environmental

AGRA Earth &
Environmental, Inc.
1290 North Hancock Street
P.O. Box 19079
Anaheim, CA 92817
Tel (714) 779-2591
Fax (714) 779-8377

December 20, 1996
Job No. 6-212-105200

Rancho Las Flores Limited Partnership
33971 Selva Road, Suite 230
Laguna Niguel, California 92629

Attention: Mr. Clifford C. Hood

Re: Letter Of Transmittal
Supplemental Geotechnical Design Report
Route 138, Phase 2, 1.2 Miles East of
Interstate 15 to Summit Post Office Road
08-SBd-138 - 16.4/19.7

Attached is the Supplemental Geotechnical Design Report for the above project. This report defines the geologic and geotechnical conditions as evaluated from review of previous reports performed by others and from field exploration and laboratory testing performed by AGRA Earth & Environmental, Inc. (AEE). This data was used in the development of the geotechnical design. This report provides recommendations and specifications for project design and construction.

Thank you for this opportunity to be of service. If you should have any questions please contact this office.

AGRA Earth & Environmental, Inc.

Larry E. Fanning
Senior Engineering Geologist
Registered Assessor REA 04677
RG 6118, CEG 1907
(Expires January 14, 1997)
LEF/DRB/ljo

REGISTERED GEOLOGIST
LARRY E. FANNING
No. 1907
CERTIFIED ENGINEERING GEOLOGIST
STATE OF CALIFORNIA

Douglas R. Bell
Supervising Engineer
GE 2140
(Expires March 31, 1999)

REGISTERED PROFESSIONAL ENGINEER
DOUGLAS R. BELL
No. GE 2140
Exp. 3-31-99
GEOTECHNICAL
STATE OF CALIFORNIA

Encl.: Supplemental Geotechnical Design Report

c: Mr. Clifford C. Hood, Addressee (2)
Mr. Mohamad Younes, C. M. Engineering (8)

01/6-1052C.LTR/12-20-96

SUPPLEMENTAL GEOTECHNICAL DESIGN REPORT

Route 138, Phase 2, 1.2 Miles East Of Interstate 15 To .2 Miles East Of Summit Post Office Road 08-SBd-138 - 16.4/19.7

Submitted To:

Rancho Las Flores Limited Partnership
33971 Selva Road
Suite 230
Laguna Niguel, California 92629

Submitted By:

AGRA Earth & Environmental, Inc.
1290 North Hancock
P.O. Box 19079
Anaheim, California 92817

December 20, 1996

Job No. 6-212-105200

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AGRA
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1.0 INTRODUCTION

This report presents geotechnical design recommendations for the Route 138 Phase 2 project. The proposed improvements involve replacement of the existing two lane Route 138 with a four-lane roadway from 1.2 miles east of Interstate 15 to 0.2 miles east of Summit Post Office Road in San Bernardino County, California. (Refer to Project Location Map - Figure 1). The new roadway will incorporate longer curve radii and provide significant safety improvements. Roadway grading will include both widening of portions of the existing alignment as well as construction of realigned roadway.

Previous geotechnical evaluation of the site has been performed by John A. Sayers & Associates (JASA) as summarized in the document entitled "Materials Report for Proposed Highway Construction Route 138, Phase II" dated May 29, 1996. After review of the Materials Report by Caltrans - Office of Structural Foundations, Roadway Geotechnical Engineering, several significant grading design issues remained unresolved. Additional testing and analysis was deemed necessary in order to resolve these issues. This Supplemental Geotechnical Design Report compiles existing available data, presents the results of additional study and provides supplemental recommendations for roadway construction.

Existing data from the JASA Materials Report was reviewed and used in the analysis. This data is presented in Appendix H of this report. Supplemental data was obtained by AGRA Earth & Environmental, Inc. (AEE) by review of published data, site reconnaissance and mapping, drilling of exploratory borings, excavation of test pits, and performance of geophysical surveys and laboratory testing.

The purpose of this report is to document subsurface geotechnical conditions, provide analyses of anticipated site conditions as they pertain to the project described herein, and to recommend design and construction criteria for the roadway portions of the project. This report also establishes a geotechnical baseline to be used in assessing the existence and scope of changed site conditions.

This report is intended for use by the project roadway design engineer, construction personnel, bidders and contractors.

2.0 EXISTING FACILITIES AND PROPOSED IMPROVEMENTS

The current project limits extend from approximately Station 62 + 00 to Station 209 + 20. The existing and proposed alignment are shown on the Geologic and Field Exploration Plans (Plates I through V). Existing Route 138 within the project limits is a two-lane paved roadway traversing moderately hilly terrain. Ground elevation along the current alignment varies from approximately 3,270 at the west end of the project to approximately 3,850 near the east end of the project.

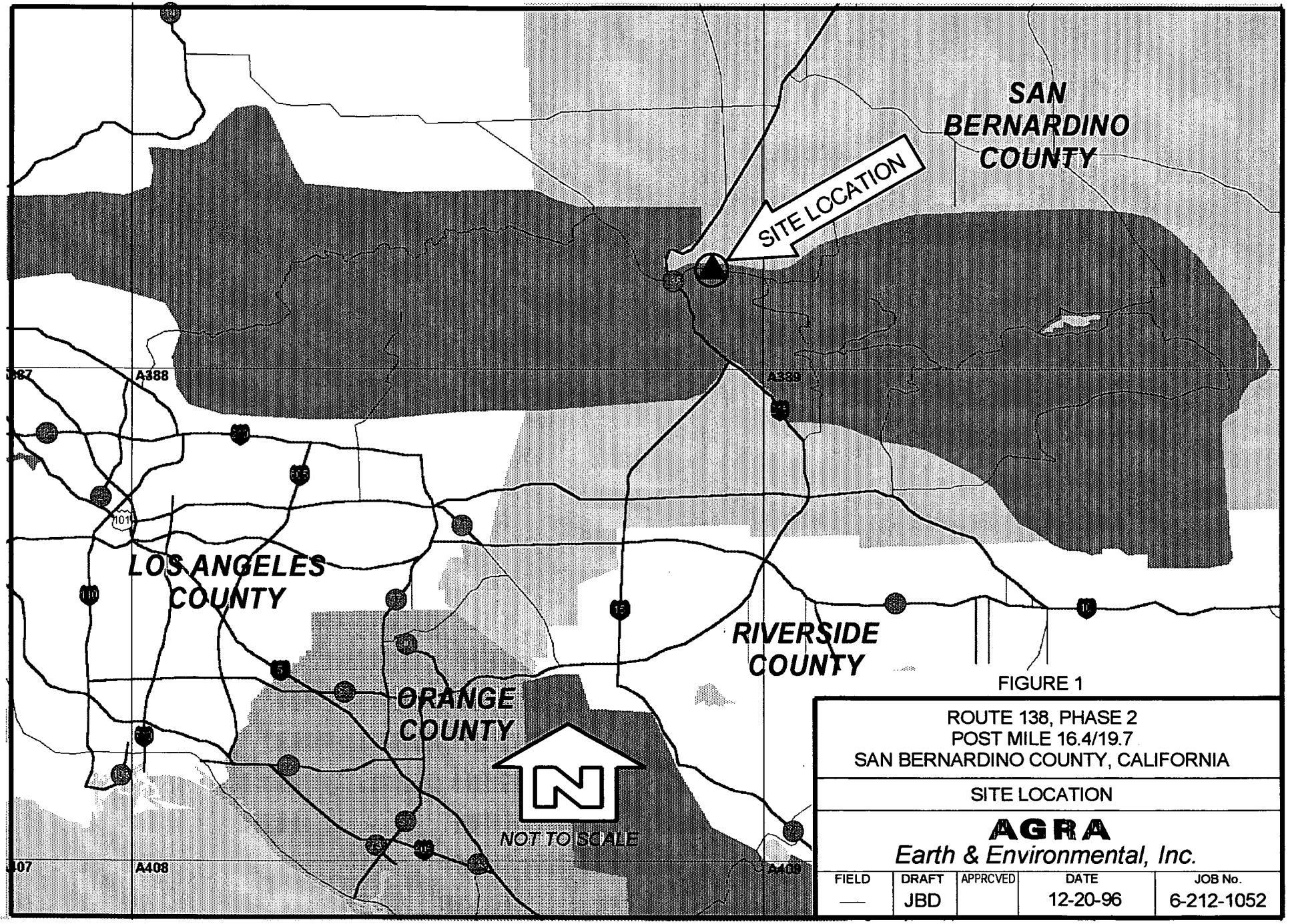

FIGURE 1

ROUTE 138, PHASE 2
POST MILE 16.4/19.7
SAN BERNARDINO COUNTY, CALIFORNIA

SITE LOCATION

AGRA
Earth & Environmental, Inc.

FIELD	DRAFT	APPRCVED	DATE	JOB No.
—	JBD		12-20-96	6-212-1052

FILE: 6-212-1082
PLOT DATE: 12-17-96

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Existing cut slopes gradients along the existing roadway generally vary from about 0.75:1.0 (horizontal:vertical) to 1.0:1.0 within the eastern portion of the alignment (Station 175+00 to 182+00). These cuts are made in both Crowder Formation and Terrace Deposits. Slope heights vary up to approximately 105 feet. Steeper cut slopes, with gradients of 0.5:1.0 or steeper are encountered locally along the remainder of the roadway within the Crowder Formation, Terrace deposits and alluvial fan deposits. These slope heights are generally less than 20 feet in height. Based on visual observations, and discussion with Caltrans District 8 maintenance personnel, the existing cut slopes are performing relatively well. Minor to moderate ravelling of coarser material within the steeper cuts does occur particularly during periods of rainfall. Where upslope runoff occurs over the face of cut slopes there is evidence of gullying, erosion, and surficial slumping.

Fill slopes along the existing roadway vary in height up to about 30 feet with gradients varying from flatter than 2.0:1.0 to locally as steep as 1.25:1.0. Most fill slopes are covered with vegetation and appear to be performing adequately. District 8 personnel reported that the only significant fill slope problem along Route 138 was within the newly constructed Phase 1 portion near Interstate 15. At this location the fill slopes apparently experienced both large scale erosion and surficial instability due to sheetflow and infiltration of surface water. The problem was remediated by regrading the slopes and installing sheetflow interruption devices on the slope faces.

Proposed grade elevation along the new Route 138 alignment varies from about 3,270 at the west end of the project to about 3,800 within the large cut areas near the east end of the project. The proposed grade will necessitate fill embankments up to approximately 70 feet in height and cut slopes up to approximately 160 feet in height. Culverts are proposed where fill embankments will transect existing drainages.

3.0 PERTINENT REPORTS AND INVESTIGATIONS

The following documents and reports were reviewed as part of this study:

- Caltrans, Report of Geophysical Investigation, Route 08-SBd-138, P.M. 15.2/19.1, dated August 30, 1971.
- Caltrans, Materials Report, 08-SBd-138, P.M. 15.2/19.0, dated October 14, 1971.
- Caltrans, Structural Section Design, 08-SBd-138-R15.2/16.4, dated January 4, 1989.
- Caltrans, Updated Materials Report, 08-SBd-138-R15.2/R16.5, dated January 12, 1989.
- Caltrans, Memorandum on Embankment Slope Stabilization, 08-SBd-R15.2/16.4, dated March 27, 1984.
- Caltrans, Draft Project Report, 08-SBd-138-16.5/19.7, dated August, 1989.

- John A. Sayers & Associates, Materials Report, Route 138, Phase II, dated June 1, 1990.
- John A. Sayers & Associates, Materials Report for Proposed Highway Construction, Route 138, Phase II, dated May 29, 1996.
- John A. Sayers & Associates, Additional Data, Proposed Highway Construction Route 138, Phase II, dated September 17, 1996.

4.0 PHYSICAL SETTING

4.1 CLIMATE

As a portion of this study, the aspects of the climate of the site which could affect design parameters and maintenance were evaluated. The project is located in the Southern California northeastern inland "mountains and high desert" region. Based on direct experience, as well as information and telephone discussions with NOAA-NWS (National Oceanic and Atmospheric Administration - National Weather Service), the climate is relatively variable seasonally. It is characterized by a mild, windy and dry fall with mild days and cool nights, followed by a cold, breezy and moist to wet winter. Spring is typically mild, with cool nights and a tapering of rain showers. The summer months are typically dry, warm to hot, and often breezy. The range of temperatures is from below freezing during winter nights to often over 100 degrees F during the day in summer. Rainfall, which occurs predominately in the mid winter to mid spring, averages 15 to 20 inches per year. Rainfall in the area, owing to the effects of mountainous terrain and high desert altitude, is often in the form of thunderstorms and other fast moving, relatively intense storms. These weather phenomena may precipitate relatively large quantities of moisture in a short period of time, on the order of 1/4 to a few inches, in less than an hour. These storms are also often associated with localized but very high winds. Because the storms may impart a large quantity of water in a short period of time, flash flooding may develop in normally dry to intermittent drainages and washes.

The elevation of the site is less than 4000 feet. As such, snowfall is infrequent and not typically heavy. Rainfall, however, may freeze during cold winter nights, and create road ice.

4.2 TOPOGRAPHY AND DRAINAGE

The site is located in mountainous pass terrain. Ground elevation in the vicinity of the project ranges from approximately 3220 feet to 3953 feet above sea level; a topographic relief of 733 feet. As can be noted on the topographic expressions shown on Plates I through V, the morphology of the site may be divided into two distinct regions: a relative "lowland" down station of approximately Station 135; and the "highlands" upstation of Station 135.

The "lowland" portion of the site is geomorphically dominated by terrain generated by the dissection of broad alluvial fans and terraces. The landform, as a result, is a series of low terraces or mesas which slope gently to the south and southwest, separated by relatively

steep, narrow channels of low to moderate depth below the mesa tops (on average 20 - 70 feet). These channels also are generally aligned northeast - southwest. The walls of these drainages are typically 1.5:1 (H:V) and may range from 2:1 or flatter to locally 0.75:1 or steeper. These drainage channels, with the exception of small, localized seeps and springs, are intermittent, and only carry significant, concentrated flow during exceptional rainfall periods. As may be discerned by the prevalence of coarse particles ranging to boulders commonly greater than 2-feet in diameter however, these flows may be very strong and move very large quantities of bedload materials (rocks and debris) in their short period of occurrence.

The "highland" portion of the site is somewhat different in that it has been uplifted and more strongly deformed. This has resulted in the development of hilly and ridgeline terrain which has been modified by narrow, deeply incised, steep walled drainages. The ridges tend to be aligned northwest-southeast to, in the case of the most pronounced ridge in the vicinity of Stations 175 to 185, east-west. Relief between the ridgetops and adjacent drainage bottoms is much greater than that of the "lowland portion," and in many areas exceeds 100-feet.

In the "highland", drainage habits are similar to those described for the "lowland." The slopes in the "highland" area range from very steep (0.5:1) locally to approximately 2:1 to less than 5:1 on the flanks of the hills. A few remnants of terraces (mesas) occur just south of Station 140 to 150. These are very gently sloping, on the order of 10 to 20:1. The sides of these mesas, however, are steep sided, on the order of 2:1 to 1:1.

4.3 MAN-MADE AND NATURAL FEATURES OF ENGINEERING AND CONSTRUCTION SIGNIFICANCE

The dominant man-made feature is that of the existing Highway 138 alignment. This feature is a two-lane undivided highway. The road is surfaced in asphalt-concrete and was created by minor to moderate cuts and fills which closely follow the existing topography. The significance of this with respect to engineering and construction of the new alignment is two fold: 1) It is the main existing access to the proposed alignment; and 2) It will be within the zone of grading influence and therefore affect the grading of the new alignment.

Another significant man-made feature is the existing AT&SF railway, which tangentially approaches the proposed alignment in the area between Stations 175 to 190. The primary concern with the railway is its effect on the construction of the proposed high cuts in that area.

Other man-made features of significance include overhead high-tension electric transmission lines, which cross the road at several locations, as well as buried to semi-exposed utilities, namely a large diameter high pressure natural gas line generally located northerly of the proposed alignment. These utilities will require protection where they may be compromised by construction. There are several unimproved dirt access roads and abandoned structural features (small concrete slabs and foundation remnants). The influence of these on proposed construction is minimal. The exposed portions of these features in the area of the proposed alignment are shown on Plates I through V.

4.4 REGIONAL GEOLOGY AND SEISMICITY

4.4.1 *Regional Geologic Setting*

The site is located in the central portion of the Transverse Ranges Geologic Province. This province is characterized by mountain ranges, deformation belts and basins which trend roughly east-west. This is at odds with other mountain ranges in California which generally trend roughly north-south. The Transverse Ranges include the oldest known rocks exposed in California, as well as very young formations.

The structure of the Transverse Ranges is strongly controlled by the activity of the San Andreas Fault Zone, which is a highly active right-lateral transform fault forming the boundary between the Pacific Plate to the west, which includes the San Gabriel and Santa Monica Mountains; and the North American Plate to the east, which includes the San Bernardino Mountains and the Mojave structural block. Due to influence of the Garlock and other faults, the San Andreas fault system has developed a "kink" or "dog-leg" within the area of the Transverse Ranges. This "kink" causes the development of a large compression component in the motions of the plates, where they are forced into each other due to strike slip motion on the plate boundary. This "kinking," which is active today, has two geotechnical ramifications which are sympathetic to this action: 1) The development of folding and deformation belts, which have induced a "structural fabric" into the rocks of the area; and 2) The development of secondary faultings, many of which are considered active. These secondary faults may be capable of very large ground motions, and may be right lateral (San Gabriel), left lateral (Cleghorn), thrust (Cucamonga), or oblique (Cajon Valley) in nature.

The site is located in the northwesterly portion of the San Bernardino Mountains. These mountains are cored by older crystalline basement rocks which include both igneous (granite, syenite, diorite, volcanics, etc.) and metamorphic (schist, greenstone, and gneiss) rocks. These basement rocks have a very complex deformation history, and have experienced several periods of uplift and deformation since the development of the San Andreas (mid-Miocene time, about 15-million years ago), several of which have occurred since Quaternary time (beginning roughly 2-million years ago).

During times of uplift, the mountainous terrain is subjected to the effects of weathering and erosion, which results in the deposition of the eroded clastics into adjacent basins. It is this process that has created the geologic conditions which underlay the subject site.

The development of an ancient mountain range in the vicinity of the site in early Quaternary time resulted in the development of a tectonic basin in which the site now sits. The conditions were such that extremely rapid deposition of material eroded from the mountains occurred. This deposition was in the form of large coalescing fans of coarse sandy alluvium which formed the thick arkosic (sandstone with abundant feldspar) of the Crowder Formation. This was deposited unconformably on pre-existing eroded ground underlain by older sequences of terrestrial sediments and crystalline basement. The form of this deposit was broad sheets and interfingering lenses of sands and sandy gravels with cobble and boulder lenses. The

sediments of the Crowder underwent consolidation by loading with additional sediments. Chemical weathering, especially of the feldspathic and mafic minerals produced clays and other cementaceous materials (silica salts such as carbonates, etc.). These processes produced induration, resulting in an arkosic sandstone.

Changes in the tectonic scene, likely controlled predominately by continued activity on the San Andreas, resulted in uplift and deformation of the Crowder and older units, inducing a regional dip to the north and east. This uplift was followed by erosion of the sediments previously over the Crowder, as well as some of the Crowder itself. The regional mountain building processes which are now currently building the San Gabriel and San Bernardino Mountains induced additional uplift and deformation, in the form of additional dip to the north and east, and the development of gentle to open fold belts and faulting. Several episodes of uplift/deformation, deposition of clastic sediments, erosion, and renewed uplift/downcutting in the area formed additional alluvial units, including Harold Formation, Alluvial Terrace, Fanglomerate, and Older and Younger Alluvium. The last two units mentioned are generally limited to existing drainages.

4.4.2 Regional Seismicity

The site, as is all of the Southern California area, is located within an active seismic region. As described above, the source of the seismic activity is related to the tectonic activity of the right lateral movement of the Pacific Plate relative to the North American Plate. Based on review of published information by Dibblee (1987, 1975, Dibblee and Hill, 1953; and others), Woodburne (1975), Foster, and others, the planned improvements of Highway 138 are located in close proximity to several large fault systems. The site is located approximately 3-miles northeasterly of the San Andreas Fault Zone. The site is also located in the relative proximity of several other faults which have exhibited movement during Quaternary to recent time. These faults include (listed in increasing distance) the Cleghorn Fault (2 mi. S), the Cajon Valley Fault (2.5 mi. E), the San Gabriel Fault Zone (8 mi. SW), the Cucamonga Fault Zone (10 mi. S), the westerly portion of the North Frontal Fault (10 mi. NE), and the north end of the San Jacinto Fault Zone (10 mi. S). The relation of these faults to the site are shown on the attached Earthquake Epicenter and Fault Map (Figure 2).

The dominating seismic exposure of the proposed Highway 138 improvements is that of the San Andreas Fault Zone. This fault has shown, and/or demonstrated by dated recent sediments, very strong ground motions and produced permanent ground deformation (demonstrated at nearby Pallet Creek, less than 17-miles northwest) in the general vicinity of the site in 1858 (includes action on the San Jacinto, M6.5, per Ziony, 1985), 1857 (M8.3), two in 1812 (M6.9 & 7.1), ~1720, ~1550, ~1350, ~1080, ~1015, ~1010, ~640, ~630, ~305, and ~135 AD (Sieh, 1984, et al., and others). This translates into a major earthquake (ie. M7.0 and greater) on the San Andreas in the area of Highway 138 every 50 to 300 years, with a mean recurrence of roughly 160 years.

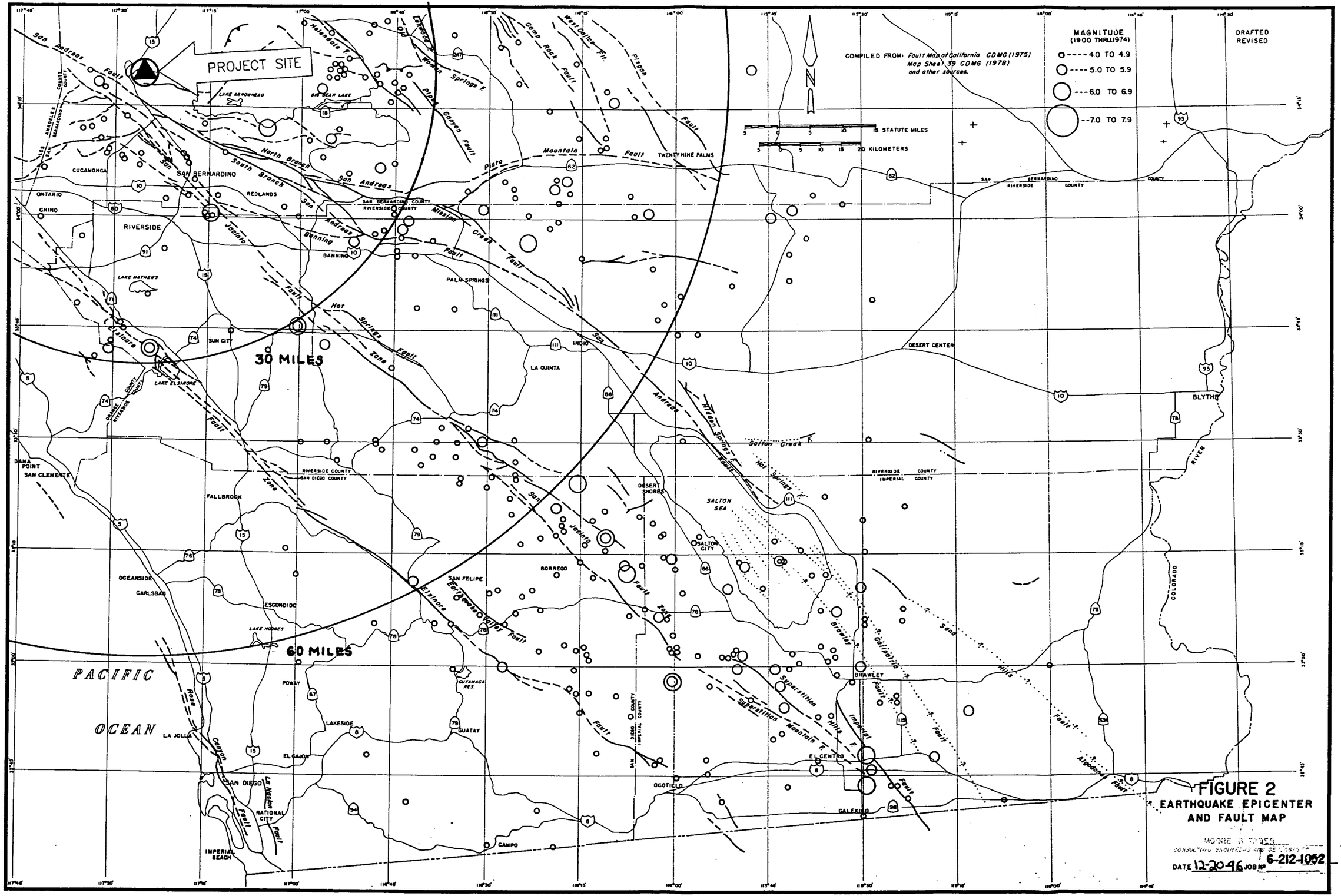


FIGURE 2
EARTHQUAKE EPICENTER AND FAULT MAP

MOORE & TAYLOR
CONSULTING ENGINEERS AND GEOLOGISTS
DATE 12-20-96 JOB No. 6-212-1052

The San Andreas Fault Zone in 1857 produced up to 14-feet of movement and involved over 230-miles of rupture. The strong ground motion was felt as far as Las Vegas, Nevada; Yuma Arizona; Mexico; and the Marysville area north of Sacramento (per Sieh, 1978; Ziony, 1985, and others).

The project site ground accelerations potentially generated by the events described above are expected to exceed that of any other known fault, except that possibly of the Cleghorn Fault. The Cleghorn Fault is identified by the State of California and the USGS as "potentially active" (ie. shown activity in the past ~ 600,000 years).

The Cleghorn Fault is essentially a complex "flower structure" (sympathetic side splay) associated with compression and deformation induced by the San Andreas. Based on review by Foster (1982), Dibblee, and others, the Cleghorn system is a oblique left-lateral fault with both strike-slip and thrust (reverse) components. No information has been reviewed by this firm that indicates offset of Holocene deposits by this fault, however, it's proximity and relation to the highly active San Andreas, it is considered a significant seismic exposure hazard to the site.

Cal Trans identifies the Cleghorn as capable of a M7.75 event. Ziony, and others (1985) have assigned it a M6.5 postulated (probable) rating.

5.0 EXPLORATION

The geotechnical and geologic aspects of the area of planned improvements for Highway 138 were investigated by this firm, as well as by JASA (per their report dated 1996), and Cal Trans in 1971. This section presents the details of exploration performed by AGRA Earth & Environmental (AEE) in the fall/winter of 1996.

The AEE field exploration included detailed surface mapping of geology at both 50 and 200-scale; subsurface exploration by auger boring, bucket rig, backhoe, and hand excavation; and geophysical assessment.

5.1 DRILLING AND SAMPLING

A total of eight exploratory borings, two backhoe excavated testpits, and 24 hand excavated sample pits were advanced for this study. The depths of exploration ranged from a few feet in the manual sample pits to 81-feet by bucket rig. The locations of these exploration points are shown on the attached Geologic and Field Exploration Plan (Plates I through V). The logs of the exploration points are presented in Appendix A.

Five borings were performed using hollow stem auger equipment, using either a Failing F-10, or CME 75 (B-101 and B-103 through B-106). Boring B-102 could not be drilled due to inaccessible terrain. Bulk samples were obtained by a field geologist. Relatively undisturbed sampling (2.5-inch ring) and SPT sampling were performed using either an SPT Autohammer or blows delivered by a 140-pound weight falling 30-inches in accordance with ASTM D1586.

The driving effort (blowcount) for each was recorded and the samples were field logged by the geologist.

Three borings were advanced by use of a bucket rig (B-107 through B-109). This type of drill rig produces a 24-inch diameter hole, which facilitates sampling in bouldery or cemented ground, and allows the field geologist to downhole log the geologic conditions in detail. The ring samples were driven using the "Kelly Bar" (drill drive stem) as a weight. The weight of the bar is depth dependent, and is shown on the attached boring logs. The SPT samples were obtained by use of a 140-pound weight falling 30-inches in accordance with ASTM D1586. Blowcounts for each were recorded, and the samples field logged by the geologist.

Two testpits were excavated by use of a Case 640K Extendahoe (TP-1 and TP-2). The testpits allowed for the exposure and logging of relatively long sections of the subsurface. Materials representative of the subsurface encountered were also sampled. Sampling involved collection of bulk samples, as well as undisturbed ring samples. The bulk samples were obtained selectively by the geologist. The ring samples were advanced by application of the backhoe bucket on the sampler, pressing it into the ground. The samples were then excavated and recovered, resulting in very little disturbance.

Twenty-four hand excavated sample pits were also dug (CS-1 through CS-24). The purpose of the pits was to evaluate the shallow subsurface in the areas of proposed culverts. A bulk sample was obtained for laboratory testing at each location, and the geologic conditions noted by the geologist.

The exploration points were all backfilled, and where possible, tamped. The backfill consisted of excavation spoils and cuttings. The exploration points were sampled and logged under the supervision of an experienced Certified Engineering Geologist.

5.2 GEOLOGIC MAPPING

Detailed geologic mapping was performed as a portion of the investigation. The mapping was performed or supervised by a Certified Engineering Geologist using 50 and 200-scale topographic base plans. The geologic features mapped included the lithologic units, contact relations between them, (ancient) faulting, bedding structure, and the extent of alluvium, landslides, slopewash, and man-made features (ie. fill).

Mapping was performed by foot traverses of the site in a detailed format. The results of the geologic mapping of the site is shown on Plates I through V. Geologic cross sections are also attached as Plates XI through XIV.

5.3 GEOPHYSICAL STUDIES

The geophysical aspects of the site were investigated by refractive seismic techniques. The primary intent of the geophysics was to provide rippability information of the Terrace deposits and the Crowder Formation. A total of three lines, ranging in length from 330 to 230 feet, were executed. The work was directed and supervised by a Certified Engineering Geologist from AEE. The actual seismic refraction survey work was performed by Goffman, McCormick & Urban, Inc. A copy of this report is included in Appendix G.

6.0 LABORATORY TESTING

In order to provide a basis for design recommendations the following laboratory tests were performed.

- Dry Density and Moisture Content
- Maximum Dry Density/Optimum Moisture Content (CA Test Method 216)
- Grain Size Analysis (ASTM D422)
- Consolidation (ASTM D2435)
- Corrosivity (CA Test Method 643)
- Sulphate Content (CA Test Method 417)
- Chloride Content (CA Test Method 422)

Test results are presented in Appendix B.

In addition to the above tests, the shear strengths of the on-site geologic units were evaluated by direct shear and triaxial strength testing. The direct shear tests were performed in general conformance with ASTM D3080. Tests were performed on samples at in-situ moisture content, and on samples under submerged conditions. Shear strength values and Mohr-Coulomb strength parameters were evaluated for both "peak" and "ultimate" conditions. The "peak" strength is defined as the maximum shear strength exhibited by the material. The "ultimate" or post-peak strength is defined as the shear strength at 10 percent strain. A summary of normal stress/shear stress values, stress/strain plots, and interpreted strength envelopes are presented in Appendix B.

A consolidated, undrained triaxial strength test was performed on a sample of Crowder Formation. Stress/strain and Mohr circle plots are presented in Appendix B.

7.0 GEOTECHNICAL CONDITIONS

7.1 SITE GEOLOGY

The following subsections describe the lithology, structure, and natural slope stability characteristics of the site as encountered in exploration.

7.1.1 Lithology

The geotechnical aspects of the site subsurface may be broken down into mappable units. The engineering character of each is described below, in order of decreasing age.

<u>Crowder Formation (TQc)</u>

The Crowder Formation consists of overconsolidated, mildly to moderately indurated arkosic sandstones which are of late Pleistocene to early Quaternary in age. These deposits are terrestrial and alluvial in nature, and are the depositionary result of erosion of ancient highlands. Because of this, they are generally crudely bedded, and often thickly interbedded or form series of interfingering lenses. The unit includes lenses and crude layerings of gravel, cobbles, and boulders. This unit also includes what appear to be paleosols (ancient soil horizons) which may form thin intermittent clayey zones that are generally concordant with bedding. Bedding features may be traceable for a few to over several hundred feet. The sands include both quartz and feldspathic grains, and are well sorted locally to well graded overall, and tend to be in the fine to medium coarse range.

The color of the Crowder formation is generally a light grayish to yellowish brown to brownish gray. It may also be locally light pinkish or medium brownish depending on weathering. The unit also has characteristic light colored salt-like infillings in exposed fractures and joints.

Induration and lithification of the Crowder has resulted in a mild to moderate cementation of the unit. This cementation appears to be somewhat more pronounced in the westerly portion of the site. The degree of cementation varies from being very difficult to excavate more than a few inches by handpick to excavation of a few feet with moderate to significant difficulty where relatively fresh exposures occur. This cementation was found to be moderately to greatly reduced by weathering exposure and inundation by water. Slaking was noted to be moderate to heavy when samples were exposed to repeated soakings and drying periods. The effects of water indicate that the cementation consists likely of carbonates, silica and chemical salts.

The results of geologic mapping, subsurface exploration, and geophysics suggest that the Crowder may be expected to behave more as a "massive" unit rather than a bedding-plane dependent unit. The reason may be the effect of interbedding, crossbedding, and the interfingering of lensed deposits disrupting clear bedding planes. A lack of consistent fine grained beds also adds to the overall strength of this unit.

Terrace Deposits (Qt)

The Terrace deposits are coarse clastic, terrestrial based materials of Quaternary age. These materials were laid down as thick sheets and crude interlayerings created by rapid erosion of the surrounding highlands. These deposits were laid down unconformably over the older units (ie. Crowder Formation). Like the Crowder Formation, the Terrace deposits are overconsolidated.

Terrace deposits at the site were found to be generally limited to the "highland" portion of the site as remnant cappings on hill and ridgetops. Terrace deposits will form a major portion of proposed roadway cut in the vicinity of Station 175 to 185.

The Terrace deposits were found to be thinly to thickly bedded to interlensed sands, sandy gravels, and included lenses and discontinuous layers of coarse sand, cobbles, and boulders. These bedding features were found to be traceable over the distance of a few feet to several hundred feet. The fraction of coarse to oversized materials was noted to be much higher overall than that in the Crowder.

Terrace deposits were found to be medium grayish brown to ruddy brown. The deposits lack any significant cementation however, due to their overconsolidated nature, the Terrace deposits do exhibit a minor amount of cohesion.

Fanglomerate (Qf)

Fanglomerate consist of interfingering alluvial fans of mid to late Quaternary age. These deposits, although generally younger than the Terrace deposits, are very similar depositionally and materially. Fanglomerate was generally limited to the "lowland" portion of the site, and forms an unconformable contact with the Crowder Formation.

The Fanglomerate forms the incised alluvial fans in the lowlands of the site. It consists of thickly to thinly, crudely bedded to interlensing fine to coarse sands, gravelly sands, with commonly discontinuous lenses of cobbles and boulders. Locally fine silts to silty sands were also encountered.

This unit is essentially a normally consolidated to overconsolidated material. It lacks in general significant cementation.

Older and Younger Alluvium (Qal)

These formations are associated in the mapping area with active drainages and washes. They are of late Quaternary to recent age. Younger Alluvium is restricted to existing drainages of current age, and consists of poorly sorted and poorly consolidated sands with abundant coarse clastics. Older alluvium is slightly better consolidated, and may be characterized as youthful stranded floodplain deposits abandoned by actively downcutting streams.

Landslide (Qls) and Slopewash (Qw)

Landslide deposits consist of late Quaternary to recent aged slope failures. They are generally comprised of the failed remains of the surrounding geologic units. Landslides were noted to vary from moderate sized block and rotational failures to relatively shallow slumps. Their occurrence appears more strongly dependent on weathering, slope inclination and undercutting by streams than by bedding fabric. The landslide debris are considered to be poorly consolidated and disarticulated.

Slopewash is essentially a thin colluvial unit restricted to slopes. It consists of poorly consolidated wash deposits and creep materials.

Fill (Af & Ef)

Man-made fill was encountered in the project area. These materials were generally "engineered" fills associated with the existing roadway. Other man made fills were encountered which were of a much more limited extent, and were associated with dirt roads, utilities, and other minor features. The fill was generally a sand to gravelly sand comprised of materials obtained from the surrounding geologic units.

7.1.2 Structure

The bedding within the Crowder formation generally dips to the north and east 15 to 25 degrees. The terrace deposits have only shallow dips less than 10 degrees, also to the north and east. The younger Fanglomerate and alluvial units are essentially flat-lying. As stated previously, bedding in the units onsite is typically crude and discontinuous.

Gentle to open folding of small scale was observed associated with ancient faulting locally, namely in the area of Station 180 to 190. In general, however, dips are consistent.

Faulting on-site appears to involve faults which do not offset youthful to recent sediments or topsoil. They are typically high angle (50-90 degrees) and strike north to northeast. These faults were found to produce narrow shear zones, the gouge zones of which were found to be typically a few inches wide at most. The most notable faulting is associated with the fault system dividing the site highlands from the lowlands, which cross the roadway in the vicinity of Station 135; and the fault system believed to have locally deformed the Crowder Formation materials in the easterly portion of the existing railway cut near Station 185 to 195.

Other than the bedding deformation induced by onsite ancient faults and regional faulting, the only structural fabric of significance is the development of fractures within the Crowder Formation of apparent random orientation and short length. These fractures range from tightly closed to slightly open, and are typically high angle. They are typically filled with salt-like precipitates.

7.1.3 Natural Slope Stability

Natural slope stability at the site was noted to be good where slopes were protected from attack of erosive waters and had developed a cover of chaparral. Man made cutslopes associated with the roadway and railway appear to be performing well where protected from erosive attack of upslope drainage waters. With only direct saturation by rain, degradation was generally limited to minor raveling and erosion.

Instabilities in natural slopes at the site appear to be strongly related to oversteepening in drainages, which are then attacked by uphill runoff, which results in gullying, as well as by undercutting of slope toes by intermittent stream/flash flood activity, resulting in landslide.

Unprotected cutslopes, such as the one associated with the railway cut north of the railway and roadway around Stations 175 to 185, which are not protected from direct attack by rain and uphill run off were noted to ravel and gully deeply.

Vegetation appeared to establish itself well on natural slopes 2:1 or flatter in non-Crowder terrain, and moderately well on such slopes in Crowder terrain. Vegetation was noted to be only moderate on natural slopes steeper than 2:1. Steep and man-made slopes of the Crowder Formation were noted to have only light to very sparse growth.

no veg it ation

The Terrace and Fanglomerate slopes of moderate steepness (2:1 and steeper) were found to develop further protection over time by what appears to be "self-armoring." Self-armoring is a process where coarse grained sands, gravels, cobbles, and boulders accumulate on a slope face as the fines are washed out by sheet flows from heavy rainfall. This accumulation of coarse material acts as a form of rip-rap, and protects the slope from direct attack, as well as by reducing the velocity of water flowing on it. Such armoring likely takes some time to develop.

This can not be justified

The nature of the climate plays a significant role in the surficial stability of slopes in the project area. As discussed previously, the rainfall at the site occurs generally in fast moving thunderstorms, consisting of high intensity, short duration rainfall in any particular area. This, combined with the development of "self armoring" protection afforded by coarse materials exposed in the slope create a condition where water has a limited ability to infiltrate. The result is a situation where only limited saturation of the slopes is likely to occur. Based on evidence observed in slopes (both natural and older cuts) directly after heavy rainfall, saturation did not extend more than a few inches in the Crowder, and less than a foot in the younger formations.

7.2 WATER

Water exists at the site in two forms, surface and subsurface. The character and potential for impact to proposed construction differs significantly between the two forms.

7.2.1 Surface Waters

In the subject area of Highway 138, surface waters occur in three forms: localized intermittent ponding, sheet and other more disorganized flows during rainfall, and concentrated flows within existing channels.

Significant localized ponding at the site is limited to a small area just north of the extreme west end of the project. This appears to be the result of intermittent high local groundwater which has been interrupted and forced to the surface in the form of seeps as a result of ancient faulting. This has produced the marshy area shown on the attached Geologic Field Exploration Plan (Plate I). The existence of trees and other vegetation within the marsh which do not tolerate saturated ground for long periods, and the exposure to shallow occurrence of bedrock (Crowder Formation) indicate this marsh likely occurs only seasonally. Based on the location of the marsh with respect to proposed construction which is limited to minor cuts and fills in the area, the impact of this feature is considered minor.

Surface waters occur in localized channels and drainages only intermittently as a result of heavy rainfall events. Some of this water is translated into the ground by permeation, the rest flows in sheets or more organized formats into the drainages, where it becomes concentrated and flows as a stream when rainfall is sufficient.

As described elsewhere in this report, rainfall occurs as infrequent, but often heavy to exceptional events. This results in a large amount of water concentrating into relatively narrow, confined drainages which have moderate to steep slopes. The effect is to produce flash flood events where normally dry washes become torrents. The effects of water flowing over the ground as such has two primary effects which are detrimental to proposed construction: scouring, and erosion. These effects are described below.

7.2.1.1 Scour

Scouring is an effect caused by moving, concentrated flows of water containing significant particulate matter. The action abrades and loosens materials which the flows come in contact with. This occurs by a combination of the force of the fluid pushing on the materials which it contacts, as well as the battering and scrubbing afforded by the suspended coarse sands, gravels, cobbles, and during exceptional periods, boulders. It is by this process that the bedload portion of the channel sediments are mobilized and transported.

Based on the large percentage of cobble to boulder sized particles within the younger Alluvium observed at the site, moderate to heavy scouring may be expected for improvements located in drainages during exceptional rainfall events. Resistance to scour may be improved by good drainage control, velocity reduction, and armoring structures by large rip-rap within the larger drainages.

7.2.1.2 Erosion

Erosion is the effect of flowing water disarticulating and removing soil from the surface on which it is flowing. For the purposes of this study, erosion is limited to effects on slopes and non channelized surfaces.

The greatest erosion hazard at the site is the uncontrolled flow of surface water onto and along slopes. The capacity for flowing water to cause erosion is critically dependent on the velocity of the flow, and is roughly geometrically proportional to velocity. As may be observed on existing slopes in the area, water flowing uncontrolled onto a slope from uphill sources will eventually become concentrated and water begins to erode away point defects in the slope face, eventually changing from sheet flow to the forming of rills and gullies. Once gullies form, the water is then concentrated into them creating a "run-away" condition that may eat up large portions of a slope face in a short period of time.

7.2.2 Groundwater

Groundwater, as source of regional free water within formational materials, was not encountered in the course of our work at the site to the depths explored. Two exceptions to this exist: 1) localized springs and seeps in the westerly project area as described herein, and 2) localized and intermittent perched subsurface waters confined in drainages within younger Alluvium at the contact between it and formation ground. As such, groundwater is not anticipated to be a problem during the proposed grading.

7.3 PROJECT SITE SEISMICITY

7.3.1 Ground Motions

The nearest active or potentially active faults to the roadway alignment are the San Andreas Fault (3 miles) and Cleghorn Fault (2 miles). These faults have been assigned a maximum credible magnitude (MCE) of 8.0 and 7.75, respectively (Mualchin, 1996). Based on these magnitudes and distances a MCE peak ground acceleration of 0.66g is considered appropriate for this project.

7.3.2 Ground Rupture

No active faulting was identified within the immediate area of the proposed roadway alignment. The potential for ground rupture affecting the proposed improvements is considered very low.

8.0 GEOTECHNICAL ANALYSIS AND DESIGN

8.1 CUTS AND EXCAVATIONS

Analyses of cut slopes for this project are dependent on the specific geologic unit as well as the proposed slope height, consequently analyses of a number of different critical cross-sections were performed.

8.1.1 Strength Parameters

Depending on their location, proposed cut slopes will encounter either Crowder Formation, Terrace Deposits, Fanglomerate, or Older Alluvium. A summary of Mohr-Coulomb strength parameters interpreted from the direct shear testing performed by AEE and JASA is presented in Appendix C. The summary includes values for tests performed on samples at both in-situ moisture content and under submerged conditions. Values are presented for both peak and ultimate shear strength conditions. Average strength parameters are presented for each case. Graphical presentation of the data is also provided in Appendix C.

Visual observation and laboratory testing indicate that the Crowder Formation does exhibit weak cementation under undisturbed, confined conditions. Consequently, it is our opinion that the use of peak direct shear strength is appropriate for evaluating gross stability of cuts in this material. The Terrace Deposits are heavily overconsolidated soils which do exhibit some nominal cohesive properties under confined conditions. The use of peak strength is also considered appropriate for evaluating gross stability of cuts in this material. To provide for a measure of conservatism for design, the appropriate average peak shear strength parameters for each unit were based on tests performed under submerged conditions.

Within the relatively younger, normally consolidated, alluvial sediments (Fanglomerate, Older Alluvium), where larger strain deformation is more probable, the use of ultimate shear strength values is considered more appropriate. Again, the average shear strength parameters were based on the direct shear tests performed under submerged conditions.

In the evaluation of surficial stability the potential for surface deformation due to changes in moisture conditions, weathering, and other processes must be considered. Consequently, submerged, ultimate shear strength parameters were used for all geologic units.

A summary of design parameters is as follows:

Geologic Unit	Unit Weight (Moist) (pcf)	Unit Weight (Saturated) (pcf)	Peak Strength		Ultimate Strength	
			Cohesion (psf)	Phi (degrees)	Cohesion (psf)	Phi (degrees)
Crowder Formation	135	141	300	37	180	35
Terrace Deposits	125	131	175	41	75	36
Fanglomerate/Older Alluvium	128	138	--	--	150	35

average?

The average peak strength value of c = 300 psf, ϕ = 37° for the Crowder Formation compares favorably to the results from a consolidated, undrained triaxial strength test performed on the material. A value of c = 750 psf and ϕ = 35° was determined by this test. (Refer to Appendix B, Page B-45). The average peak strength value of c = 175 psf, ϕ = 41° for the Terrace Deposits compares very closely to the back calculated value for the steep existing slope found in this material at Station 180+00 (Refer to Appendix D, Page D-1).

8.1.2 Slope Stability

8.1.2.1 Gross Stability

The highest proposed cuts on this project will be within both Terrace Deposits and Crowder Formation within the eastern portion of the alignment (between approximate Station 174 and 185). Slope stability analysis of critical sections were calculated using the computer program SLOPEW. Factors of safety were calculated using the Morgenstern-Price Method. The overall highest proposed cut slope (approximately 160 feet at Station 176+00) was found to have a factor of safety in excess of 1.5 at a slope gradient of 1.35:1.0 (horizontal:vertical). (Refer to Appendix D, Page D-2). The section included a 20 foot wide horizontal bench near mid-slope. The highest proposed cut within Terrace Deposits (approximately 125 feet at Station 183+00) was also found to have a factor of safety of greater than 1.5 at a gradient of 1.35:1.0 with a mid-slope 20 foot wide bench. (Refer to Appendix D, Page D-3).

Smaller cut slopes are proposed within the Crowder Formation between approximate Station 157 and 174. An analysis of the highest cut slope in this area (approximately 57 feet at Station 165+50 yielded a factor of safety greater than 1.5 at a gradient of 1.35:1.0. (Refer to Appendix D, Page D-4). No benches were included in this analysis.

Within the western portion of the alignment (Station 60 to 157) there are numerous proposed cuts within the Fanglomerate and Older Alluvium. Some of these cuts will also expose underlying Crowder Formation. An analysis of highest proposed cut in this area (approximately 30 feet at Station 120+00) yielded a factor of safety greater than 1.5 at a gradient of 1.5:1.0 (Refer to Appendix D, Page D-5). No benches were included in this analysis.

Based on the stability analyses performed, it is recommended that cut slopes between Station 157 and 190 be constructed at a gradient no steeper than 1.35:1.0. Cut slopes with heights greater than 70 feet should also include a 20 foot horizontal bench at or close to mid-slope height. Cut slopes between Station 62 and 157 should be constructed at a gradient of 1.5:1.0.

8.1.2.2 Surficial Stability and Erosion Control

Analysis of surficial stability of cut slopes under saturated conditions was performed using submerged ultimate strengths and saturated unit weights. The calculated factors of safety as a function of saturation depth are presented in Figure 3. Based on the analysis a factor of safety of 1.5 would correspond to saturation depths of about 1.2 feet, 2.6 feet, and 2.8 feet within the Terrace Deposits, Fanglomerate and Crowder Formation, respectively. Deeper saturation depths would lower the factor of safety. It is evident that cut slope faces could be prone to surficial failures if excessive saturation of the slope face occurred. In addition, these materials would be susceptible to erosion from concentrated surface water flow. These conditions would exist even with significantly flatter slopes.

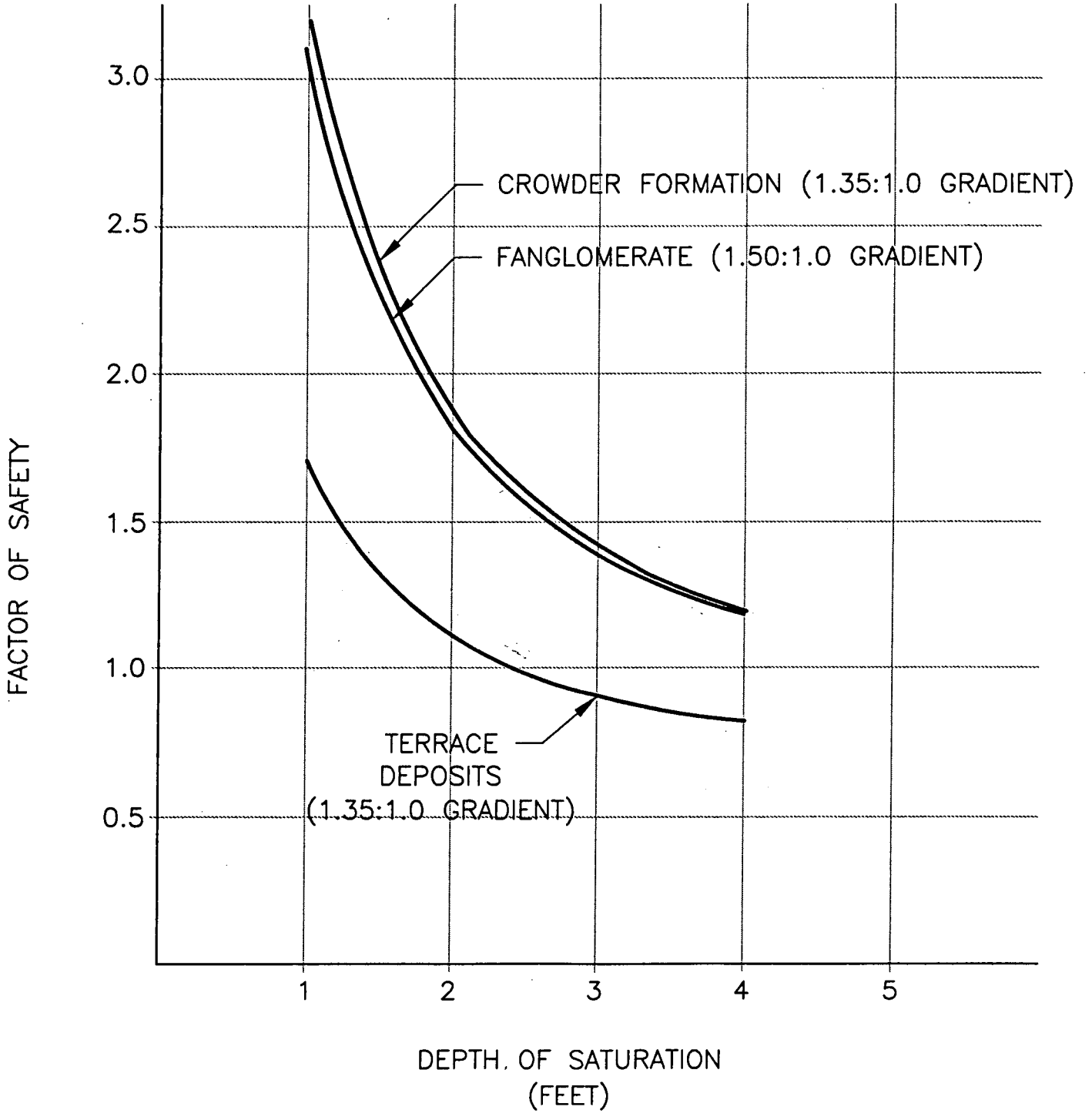
In order to minimize the potential of slope damage due to water flow and/or infiltration it is considered critical that adequate surface drainage be provided. In this regard it is recommended that upslope tributary drainage areas be intercepted by a paved brow ditch at the top of all cut slopes. The brow ditch should flow into paved slope drains or other suitable points of discharge. All slope benches should incorporate a paved drainage swale at the toe of the ascending slope. The bench should be sloped at a gradient of at least one percent towards the drainage swale.

To reduce the potential for erosion of slope faces and to promote vegetative growth it is recommended that cut slopes be stepped. This will reduce water flow velocity on the slope face and provide a horizontal surface for vegetative growth. A minimum horizontal tread width of 2 feet is recommended. After step cutting of the slope is completed consideration should be given to applying hydroseed or other landscaping techniques to establish vegetation on the slope face.

Even with good surface drainage provisions some periodic maintenance due to slope ravelling should be anticipated. It is recommended that a truck width maintenance strip be provided between the roadway shoulder and the toe of slope to facilitate cleanup of slope debris.

FIGURE 3

SURFICIAL STABILITY
FACTOR OF SAFETY
CUT SLOPES



8.1.2.3 Dynamic Stability

Pseudostatic analysis of the maximum proposed cut slope height utilizing a seismic coefficient of 0.15 yielded a factor of safety greater than 1.1 (Page D-6). With the local seismic conditions as described previously, peak ground accelerations on the order of 0.66g could be expected during a maximum credible event. In order to evaluate the potential movement of proposed slopes during maximum credible shaking a deformation analysis was performed using the computer program TNMN - Version 1.2. The program computes the displacement of a potential sliding mass by double integration of the difference between an applied acceleration history and the slope yield acceleration. The yield acceleration for the highest proposed cut was calculated as Ky = .22g (Page D-7). Acceleration time histories for a number of recorded events, as well as a synthetic one developed for the San Andreas Fault, were scaled to the maximum site acceleration. The analysis yielded a range of expected deformation as summarized below.

Event	Calculated Deformation (feet)
San Andreas (Synthetic)	1.01
El Centro	1.04
Hayward	1.22
Taft	0.63

Based on the analysis, some deformation of cut slopes should be expected during a maximum credible event.

8.1.3 Rippability

Drilling refusal was not encountered in any of the exploratory borings. In order to evaluate the rippability of the on-site materials three geophysical seismic line surveys were performed within major proposed cut areas. The findings of the survey are detailed in Appendix G. Based on data obtained from the study, seismic velocities within the Terrace Deposits are interpreted to range from about 2,000 to 4,000 feet per second. Seismic velocities within the Crowder Formation are interpreted to range from 2,000 to 5,000 feet per second within the depth of proposed cut. These materials are expected to present easy to moderately difficult ripping conditions for a Caterpillar Model D-9L or equivalent rip-dozer equipment.

8.1.4 Grading Factors

Based on the in-situ dry density and maximum density tests performed the following average grading factors are considered appropriate.

Crowder Formation	-	Bulking Factor 1.05
Terrace Deposits	-	Bulking Factor 1.03
Fanglomerate/Older Alluvium	-	Shrinkage Factor 0.97
Recent Alluvium/Slope Wash	-	Shrinkage Factor 0.90
Landslide Material/Existing Fill		

Material from the Crowder Formation and Terrace Deposits will be predominately encountered within cuts along the eastern portion of the alignment (east of approximately Station 164). Fanglomerate and Older Alluvium will predominately be encountered within cuts along the western portion (west of Station 157). Recent alluvium, slope wash, and landslide material will generally be encountered in cleanouts below proposed embankment. Existing fills will be encountered as fairly shallow sliver fills along the existing Route 138 roadway.

8.2 EMBANKMENTS

8.2.1 Foundation Preparation and Settlement

Based on the density and consolidation testing performed it is our opinion that on-site soils will generally provide on adequate foundation for proposed fill embankments. Removal and recompaction of loose alluvium, slope wash, landslide material, and existing fill will be required prior to placing embankment fill. Recent alluvium and slope wash exists within existing drainages (refer to Geologic and Field Exploration Plans Plates I through V). The estimated depth of removal in these areas is 3 to 4 feet. Deep alluvial soils exist at the extreme east end of the alignment (east of approximately Station 189). Removal and recompaction of the upper 3 feet of soil is recommended in this area. Landslide material in the area between approximately Station 158 and Station 163 are generally shallow slumps on the sides of existing drainages. It is expected that benching (as stipulated in Caltrans Standard Specification 19-6.01) will effectively remove these materials. Deeper rotational slide masses may be encountered between Station 110 and Station 112. Localized removals on the order of 15 feet is expected in this area. Existing roadway fills exist predominately as shallow sliver masses or shallow embankments crossing existing drainages. Required slope benching and excavation for proposed culvert construction is expected to effectively remove most of this material.

Settlement analysis generally indicates potential ground subsidence ranging from approximately 1 to 4 inches for embankment heights varying from 20 to 70 feet. Due to the granular nature of the subsurface soils most of this subsidence is expected to occur during roadway grading.

8.2.2 Strength Parameters

Direct shear tests were performed on samples of Crowder Formation and Terrace Deposits remolded to 90 percent of maximum dry density. A summary of Mohr-Coulomb strength parameters interpreted from tests performed by both AEE and JASA are presented in Appendix C. Graphical presentation of the data is also provided. Based on the test results average ultimate strength parameters of c = 100 psf and ϕ = 34° have been selected to evaluate both gross and surficial stability of fill slopes.

8.2.3 Slope Stability

8.2.3.1 Gross Stability

In order to evaluate gross stability of proposed fill slopes a critical section was analyzed using the computer program SLOPEW. Factors of safety were calculated using the Morgenstern - Price method. The highest proposed fill slope (approximately 70 feet at Station 137+00) was found to have a factor of safety in excess of 1.5 at a slope gradient of 2.0:1.0 (Page D-8). The critical surface was a toe failure. Basal failure through underlying alluvial soils is not considered a design factor. Based on the analysis it is recommended that fill slopes be constructed at a gradient of 2.0:1.0.

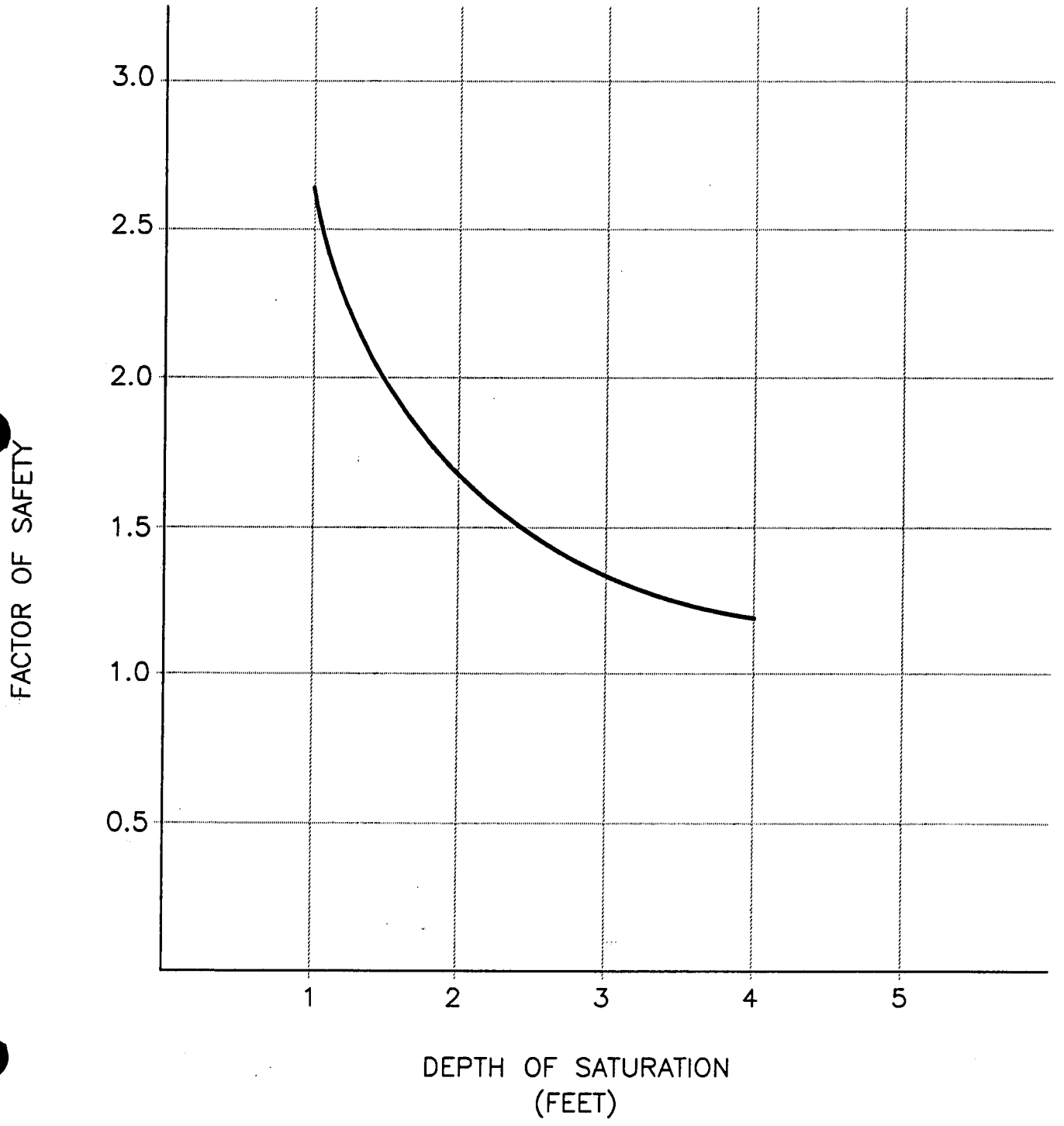
8.2.3.2 Surficial Stability

Analysis of surficial stability of fill slopes under saturated conditions was performed using submerged ultimate strengths and the saturated unit weight of the fill. The calculated factors of safety as a function of saturation depth are presented in Figure 4. Based on the analysis, a saturation depth less than 2.5 feet would result in a factor of safety greater than 1.5. Saturation depths of greater that 2.5 feet would result in commensurately lower factors of safety.

In order to minimize the potential for slope damage due to water infiltration, it is recommended that surface drainage be designed such that all roadway runoff is prevented from flowing onto the face of fill slopes. After construction of the fill embankments it is recommended that drainage terraces, straw wattles, or other approved method of slope interruption, be installed on the slope face. This will reduce the potential for sheetflow erosion and enhance revegetation of the slope face. Seeding of the slope faces should also be initiated as soon after grading as possible.

FIGURE 4

SURFICIAL STABILITY
FACTOR OF SAFETY
2.0 : 1.0 FILL SLOPES



8.2.3.3 Dynamic Stability

Pseudostatic analysis of the maximum fill slope height utilizing a seismic coefficient of 0.15 yielded a factor of safety greater than 1.1 (Page D-9). Potential dynamic movement for the maximum credible seismic event was evaluated using the computer program TNMN - Version 1.2. The program computes the displacement of a potential sliding mass by double integration of the difference between an applied acceleration history and the slope yield acceleration. Time histories of several earthquakes were scaled to the maximum credible acceleration of 0.66g. A yield acceleration of Ky = 0.28 was calculated for the critical fill slope (Page D-10). The analysis yielded a range of expected deformation as summarized below.

Event	Calculated Deformation (feet)
San Andreas (synthetic)	0.52
El Centro	0.58
Hayward	0.55
Taft	0.30

Based on the analysis some deformation of fill slopes should be expected during a maximum credible event.

8.3 CULVERT FOUNDATIONS

Based on the field exploration performed it is our opinion that the on-site soils will generally provide an adequate foundation for proposed culverts. Removal requirements, as outlined in the Section 8.2.1 Embankments - Foundation Preparation and Settlement should be completed prior to culvert construction. Corrosion effects on proposed culverts are discussed in Appendix F.

9.0 RECOMMENDATIONS FOR PLANS AND SPECIFICATIONS

A summary of recommendations that should be included in the plans and specifications are presented as follows:

- Recommended maximum cut slope gradients.

  - Beginning of Project to Station 157 — 1.5:1.0 (H:V)
  - Station 157 to Station 190 — 1.35:1.0 (H:V)

- All cut slopes higher than 70 feet should have a 20 foot wide mid-slope horizontal bench.

- Cut slopes should be step cut with a tread width of 2 feet.

- Brow ditches should be constructed at the top of all cut slopes where upslope drainage areas exist.

- Slope benches should have a paved drainage swale at the toe of the ascending slope. The benches should slope toward the swale at a gradient of at least one percent.

- A maintenance strip (minimum 8 feet wide) should be provided between the roadway shoulder and the toe of cut slopes greater than 70 feet in height.

- Removal of loose surficial soils below embankment fills will be necessary. The estimated depths of removals are as follows:

  - Existing drainages 3 - 4 feet.
  - Alluvial area (Station 189 to End of Project) 3 feet
  - Landslide material (Station 110 to 112) 5 - 15 feet

- Fill slopes should be constructed at a gradient no steeper than 2.0:1.0.

- Roadway runoff should be prevented from spilling over fill slopes.

- Fill slopes should incorporate paved drainage terraces spaced at vertical intervals of 30 feet or incorporate straw wattles or other flow interruption devices at vertical intervals of 7.5 feet.

- Hydroseeding or other acceptable planting techniques should be initiated on fill slopes immediately after grading is completed.

- Pavement subgrade soils must have a minimum Resistance Value (R-Value) of 50. (Refer to Pavement Design Report - Appendix E).

- The upper 4 feet of embankment material must have an R-Value greater than 50.

- Within cut areas between approximate Stations 144 and 189 some low R-Value material within the Crowder Formation should be expected at subgrade elevation. Removal of this material to a depth of 4 feet and replacement with material with an R-Value greater than 50 will be required.

10.0 CLOSURE

This report is based on the project as described, information obtained from referenced reports prepared by others, and from the field exploration, laboratory testing performed by AEE. Our findings are based on the results of the field, laboratory, and office studies, combined with an interpolation and extrapolation of soil conditions between and beyond the borings. The results reflect our interpretation of the limited direct evidence obtained. Our firm should be notified of any pertinent change in the project plans. If subsurface conditions are found to differ from those described herein, it may require a re-evaluation of the recommendations.

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

Respectfully submitted,

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01/6-1052D.J12/12-20-96

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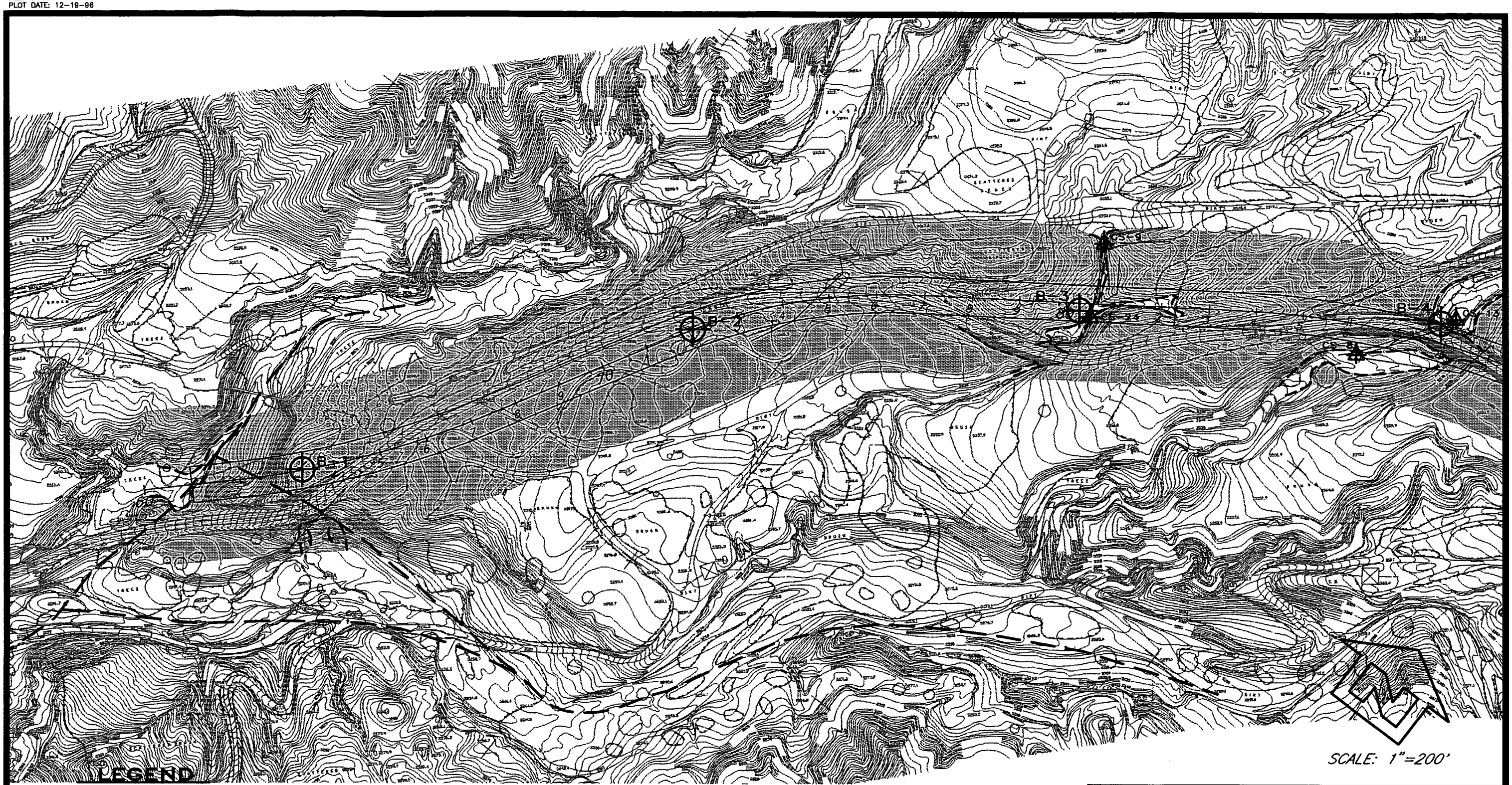
PLATES

FILE: 6-1052K
PLOT DATE: 12-19-96



LEGEND

- Af – NONENGINEERED FILL
- Afe – ENGINEERED FILL
- Qal – ALLUVIUM
- Qoal – OLDER ALLUVIUM
- Qls – LANDSLIDE
- Qf – FANGLOMERATE
- Qsw – SLOPE WASH
- Qt – TERRACE DEPOSIT
- Qh – HAROLD FORMATION
- QTc – CROWDER FORMATION
- MARSH AREA
- AGRA EARTH & ENVIRONMENTAL, Inc. BORING LOCATION
- AGRA EARTH & ENVIRONMENTAL, Inc. TEST PIT LOCATION
- AGRA EARTH & ENVIRONMENTAL, Inc. CORROSION SAMPLE LOCATION
- BORING LOCATION BY JOHN A. SAYERS & ASSOCIATES
- EXISTING ROADWAY
- GEOPHYSICAL SEISMIC LINE
- GEOLOGIC CONTACT–DASHED WHERE APPROXIMATE, DOTTED WHERE CONCEALED
- FAULT–DASHED WHERE APPROXIMATE, DOTTED WHERE CONCEALED

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POST MILE 16.4/19.7
SAN BERNARDINO COUNTY, CALIFORNIA

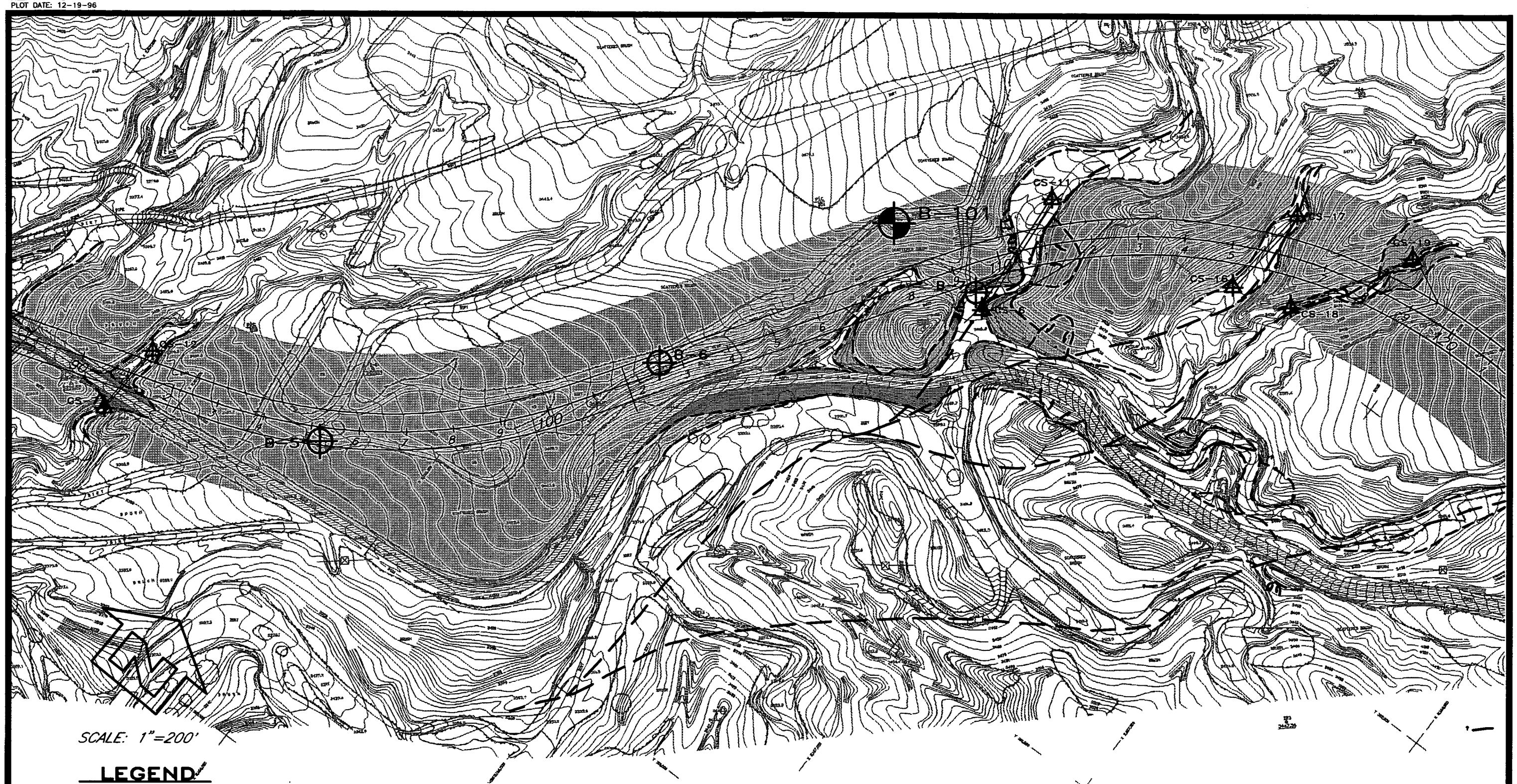
GEOLOGIC AND FIELD EXPLORATION PLAN

AGRA
Earth & Environmental, Inc.

FIELD	DRAFT	APPROV	DATE	JOB No.
GB/LEF	JBD		12-20-96	6-212-1052

PLATE 1

FILE: 6-1052L
PLOT DATE: 12-19-96



LEGEND

- Af – NONENGINEERED FILL
- Afe – ENGINEERED FILL
- Qal – ALLUVIUM
- Qoal – OLDER ALLUVIUM
- Qls – LANDSLIDE
- Qf – FANGLOMERATE
- Qsw – SLOPE WASH
- Qt – TERRACE DEPOSIT
- Qh – HAROLD FORMATION
- QTc – CROWDER FORMATION
- MARSH AREA
- AGRA EARTH & ENVIRONMENTAL, Inc. BORING LOCATION
- AGRA EARTH & ENVIRONMENTAL, Inc. TEST PIT LOCATION
- AGRA EARTH & ENVIRONMENTAL, Inc. CORROSION SAMPLE LOCATION
- BORING LOCATION BY JOHN A. SAYERS & ASSOCIATES
- EXISTING ROADWAY
- GEOPHYSICAL SEISMIC LINE
- GEOLOGIC CONTACT–DASHED WHERE APPROXIMATE, DOTTED WHERE CONCEALED
- FAULT–DASHED WHERE APPROXIMATE, DOTTED WHERE CONCEALED

ROUTE 138, PHASE 2
POST MILE 16.4/19.7
SAN BERNARDINO COUNTY, CALIFORNIA

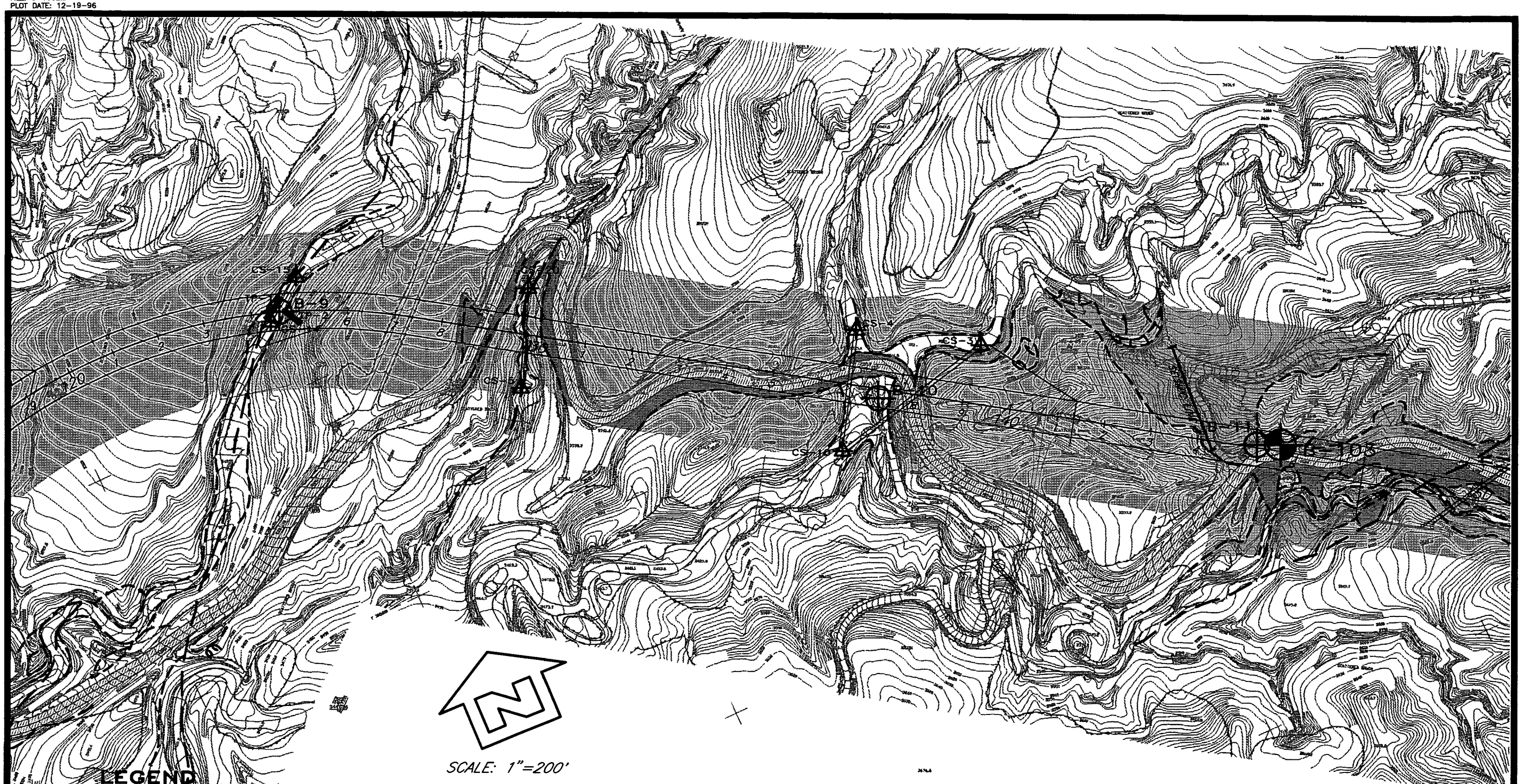
GEOLOGIC AND FIELD EXPLORATION PLAN

AGRA
Earth & Environmental, Inc.

FIELD	DRAFT	APPROV.	DATE	JOB No.
GB/LEF	JBD		12-20-96	6-212-1052

PLATE II

FILE: 6-1052M
PLOT DATE: 12-19-96



LEGEND

- Af – NONENGINEERED FILL
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- GEOPHYSICAL SEISMIC LINE
- GEOLOGIC CONTACT–DASHED WHERE APPROXIMATE, DOTTED WHERE CONCEALED
- FAULT–DASHED WHERE APPROXIMATE, DOTTED WHERE CONCEALED

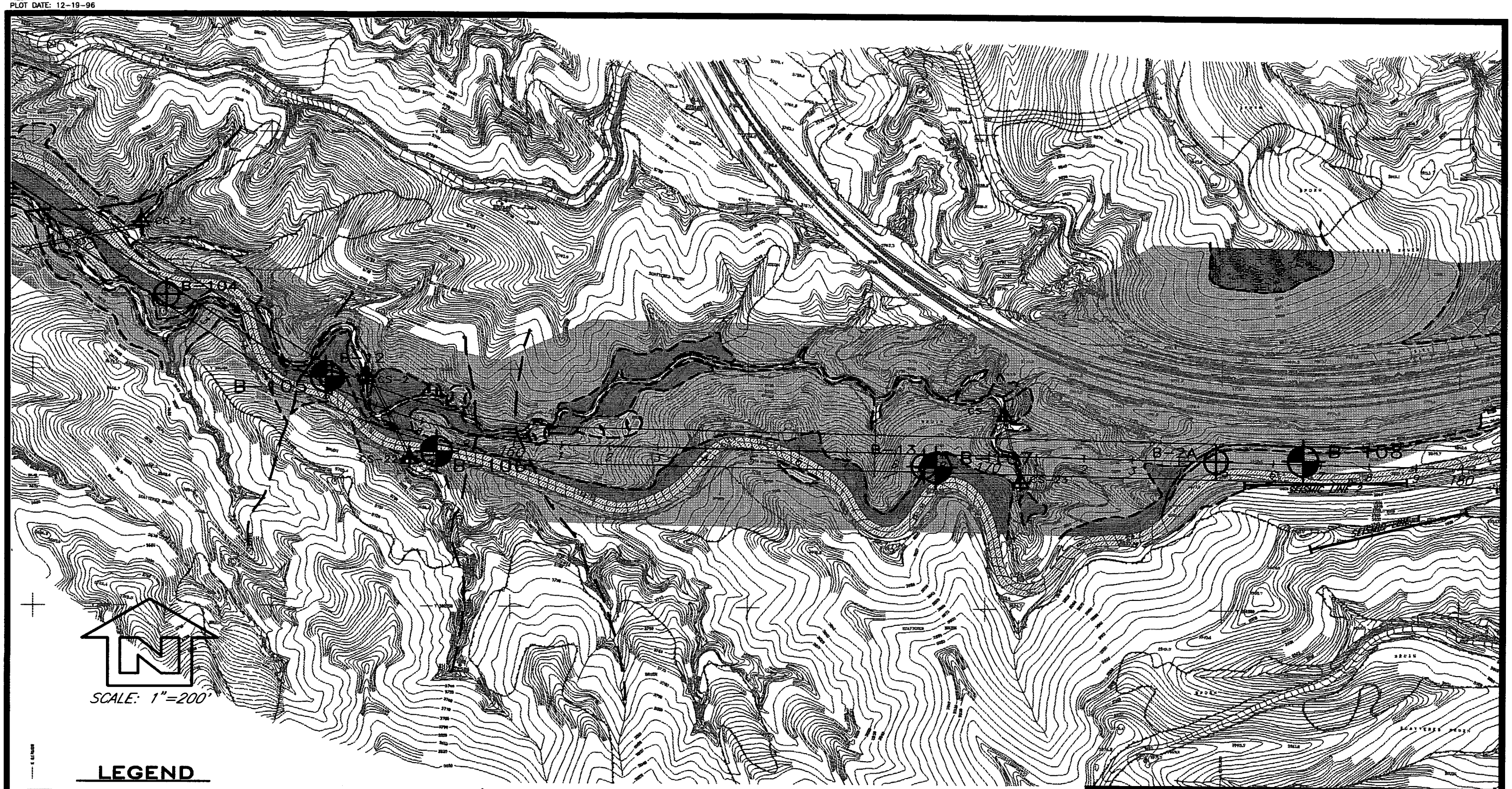
ROUTE 138, PHASE 2
POST MILE 16.4/19.7
SAN BERNARDINO, CALIFORNIA

GEOLOGIC AN FIELD EXPLORATION PLAN

AGRA
Earth & Environmental, Inc.

FIELD	DRAFT	APPROV.	DATE	JOB No.
GB/LEF	JBD		12-20-96	6-212-1052

FILE: 6-1052N
PLOT DATE: 12-19-96



LEGEND

- Af – NONENGINEERED FILL
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- EXISTING ROADWAY
- GEOPHYSICAL SEISMIC LINE
- GEOLOGIC CONTACT–DASHED WHERE APPROXIMATE, DOTTED WHERE CONCEALED
- FAULT–DASHED WHERE APPROXIMATE, DOTTED WHERE CONCEALED

ROUTE 138, PHASE 2
POST MILE 16.4/19.7
SAN BERNARDINO COUNTY, CALIFORNIA

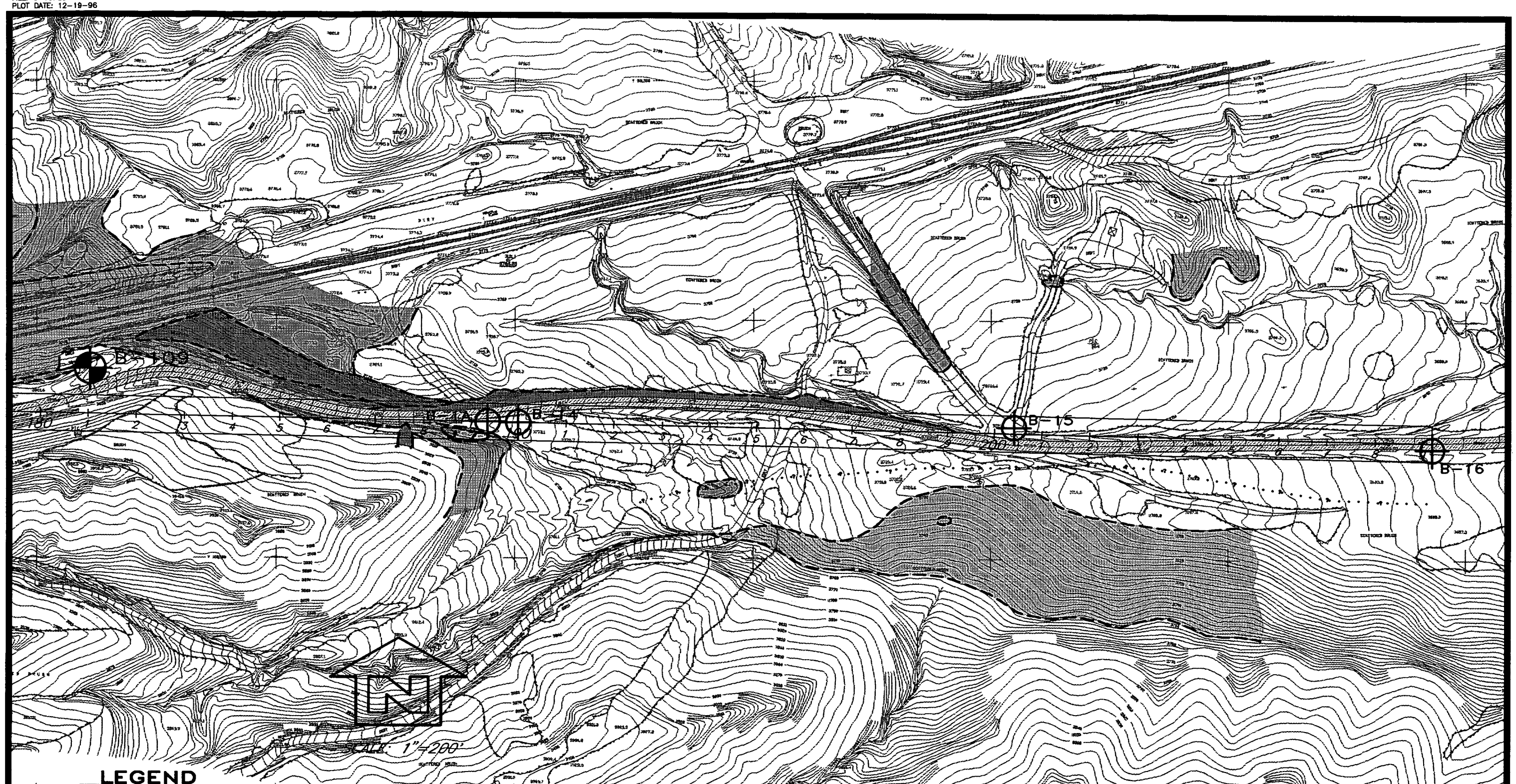
GEOLOGIC AND FIELD EXPLORATION PLAN

AGRA
Earth & Environmental, Inc.

FIELD	DRAFT	APPROV.	DATE	JOB No.
GB/LEF	JBD		12-20-96	6-212-1052

PLATE IV

FILE: 6-10520
PLOT DATE: 12-19-96



LEGEND

- Af – NONENGINEERED FILL
- Afe – ENGINEERED FILL
- Qal – ALLUVIUM
- Qoal – OLDER ALLUVIUM
- Qls – LANDSLIDE
- Qf – FANGLOMERATE
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- EXISTING ROADWAY
- GEOPHYSICAL SEISMIC LINE
- GEOLOGIC CONTACT–DASHED WHERE APPROXIMATE, DOTTED WHERE CONCEALED
- FAULT–DASHED WHERE APPROXIMATE, DOTTED WHERE CONCEALED

ROUTE 138, PHASE 2
POST MILE 16.4/19.7
SAN BERNARDINO COUNTY, CALIFORNIA

GEOLOGIC AND FIELD EXPLORATION PLAN

AGRA
Earth & Environmental, Inc.

FIELD	DRAFT	APPROV	DATE	JOB No.
GB/LEF	JBD		12–20–96	6-212-1052

PLATE V

SOIL SURVEY SHEET

GEN. INFO.	BORING NO./SAMPLE NO.												CS-24		CS-9		CS-8		CS-13	
	SAMPLE DEPTH (feet)												1.0		2.0		2.0		1.0	
	UNIFIED SOIL CLASSIFICATION												SP		SP		SM		SM	
	DATE SAMPLED												12-4-96		11-6-96		11-6-96		11-6-96	
SIEVE ANALYSIS (Percent Passing)	1 ½" SQUARE OPENING																			
	¾" SQUARE OPENING																			
	⅜" SQUARE OPENING																			
	No. 4 (4760 MICRONS)																			
	No. 8 (2380 MICRONS)																			
	No. 16 (1190 MICRONS)																			
	No. 30 (590 MICRONS)																			
	No. 50 (297 MICRONS)																			
	No. 100 (148 MICRONS)																			
	No. 200 (74 MICRONS)																			
	5 MICRONS																			
	2 MICRONS																			
PHYSICAL TESTS	MAXIMUM DRY DENSITY (pcf)																			
	OPTIMUM MOISTURE (%)																			
	LIQUID LIMIT																			
	PLASTICITY INDEX																			
	SAND EQUIVALENT																			
CHEM. TESTS	SOLUBLE CHLORIDE (%)																			
	SOLUBLE SULPHATE (%)												0.0111		0.0082		0.0065		0.0101	
	RESISTIVITY (ohm-cm)												>16750		16750		11170		16750	
	pH												6.9		6.9		7.4		7.1	
R-VALUE	TEST SPECIMEN																			
	DRY DENSITY (pcf)																			
	MOISTURE CONTENT (%)																			
	EXUDATION PRESSURE (psi)																			
	CORRECTED R-VALUE																			
	R-VALUE STABILOMETER																			
	R-VALUE BY EXPANSION																			
DIRECT SHEAR	TEST SPECIMEN																			
	NORMAL STRESS (psf)																			
	PEAK SHEAR STRESS (psf)																			
	ULTIMATE SHEAR STRESS (psf)																			
	ANGLE OF INTERNAL FRICTION (Ø°)																			
	COHESION (psf)																			
EXPAN. TEST	INITIAL DRY DENSITY (psf)																			
	INITIAL MOISTURE (%)																			
	FINAL MOISTURE (%)																			
	SWELL (%)																			
	EXPANSION INDEX																			

DIST	COUNTY	ROUTE	POST MILES	SHEET NO.	TOTAL SHEETS
08	SBD	138	16.4–19.7		

REGISTERED PROFESSIONAL ENGINEER

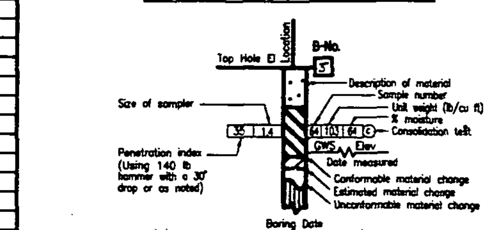
PLANS APPROVAL DATE

REGISTERED PROFESSIONAL ENGINEER DOUGLAS R. BELL No. GE 2140 Exp. 3-31-99 GEOTECHNICAL STATE OF CALIFORNIA

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

ROUTE 138 IMPROVEMENTS

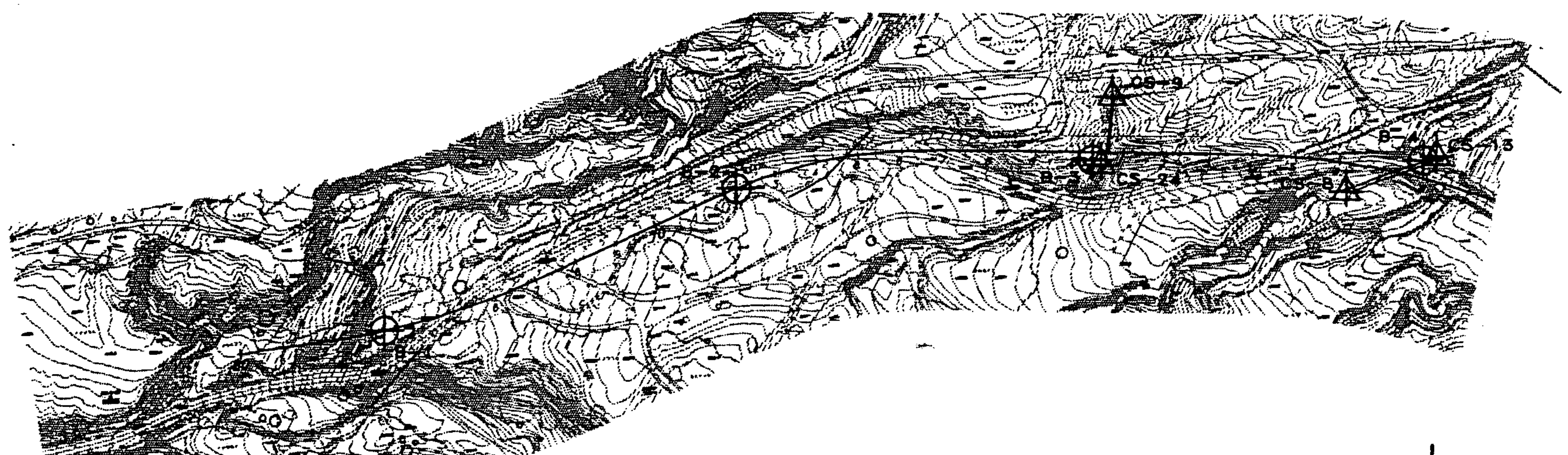
LEGEND OF BORING OPERATIONS



LEGEND OF EARTH MATERIALS

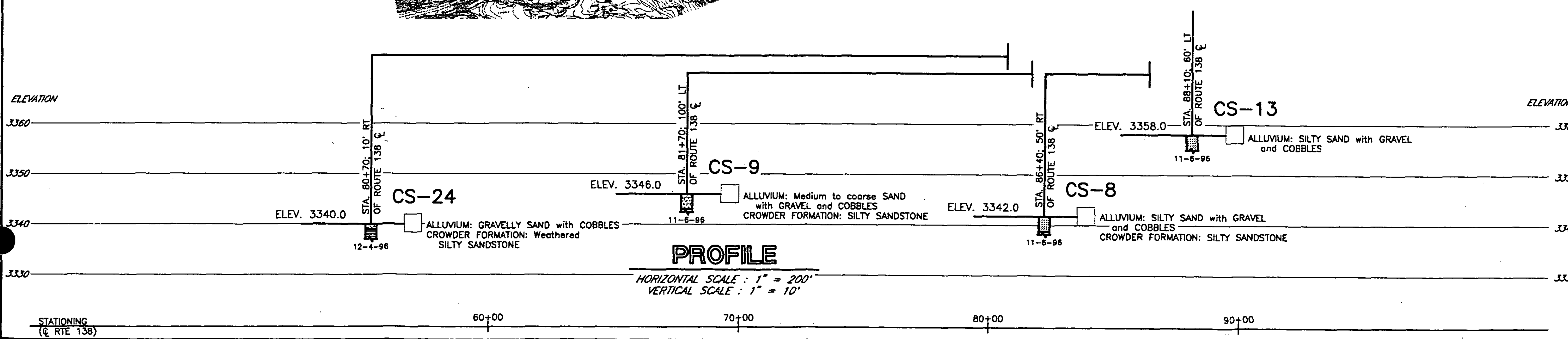
GRAVEL; SAND; SILT; CLAY; SANDY CLAY or CLAYEY SAND; SANDY SILT or SILTY SAND; SILTY CLAY; CLAYEY SILT; PEAT and/or ORGANIC MATTER; FILL MATERIAL; IGNEOUS ROCK; SEDIMENTARY ROCK; METAMORPHIC ROCK

ORIGINAL SCALE IN INCHES FOR REDUCED PLANS 0 1 2 3



PLAN
SCALE : 1" = 200'

LEGEND
- AGRA EARTH & ENVIRONMENTAL, Inc. HAND SAMPLE LOCATION
- AGRA EARTH & ENVIRONMENTAL, Inc. BORING LOCATION
- AGRA EARTH & ENVIRONMENTAL, Inc. TEST PIT LOCATION
- BORING LOCATION BY JOHN A. SAYERS & ASSOCIATES



PROFILE
HORIZONTAL SCALE : 1" = 200'
VERTICAL SCALE : 1" = 10'

PREPARED FOR THE **STATE OF CALIFORNIA** DEPARTMENT OF TRANSPORTATION

G. BUTLER/L.E. FANNING — FIELD INVESTIGATOR
DECEMBER 4, 1996 — DATE
J.B. DePOORTER — DRAWN BY
CHECK BY
Job No. 6-212-1052
PLATE VI

SOIL SURVEY SHEET

Group	Test	CS-7	CS-12	B-101/2	B-101/8	B-101/10	CS-6	CS-11	CS-16	CS-17	CS-18	CS-19
GEN. INFO.	BORING NO./SAMPLE NO.	CS-7	CS-12	B-101/2	B-101/8	B-101/10	CS-6	CS-11	CS-16	CS-17	CS-18	CS-19
	SAMPLE DEPTH (feet)	1.0	2.0	3.5	22.5	27.5	1.0	1.0	1.0	2.0	2.0	1.0
	UNIFIED SOIL CLASSIFICATION	SM	SM	SM	SM	SM	SP	SP	SM	SP	SP	SP
	DATE SAMPLED	11-6-96	11-6-96	11-26-96	11-26-96	11-26-96	11-6-96	11-6-96	12-4-96	12-4-96	12-4-96	12-4-96
SIEVE ANALYSIS (Percent Passing)	1 ½" SQUARE OPENING											
	¾" SQUARE OPENING			98								
	⅜" SQUARE OPENING			93								
	No. 4 (4760 MICRONS)			86								
	No. 8 (2380 MICRONS)			75								
	No. 16 (1190 MICRONS)			61								
	No. 30 (590 MICRONS)			46								
	No. 50 (297 MICRONS)			32								
	No. 100 (148 MICRONS)			22								
	No. 200 (74 MICRONS)			15								
	5 MICRONS											
	2 MICRONS											
PHYSICAL TESTS	MAXIMUM DRY DENSITY (pcf)											
	OPTIMUM MOISTURE (%)											
	LIQUID LIMIT											
	PLASTICITY INDEX											
	SAND EQUIVALENT											
CHEM. TESTS	SOLUBLE CHLORIDE (%)			0.0031								
	SOLUBLE SULPHATE (%)	0.0126	0.0080	0.0084			0.0083	0.0184	0.0149	0.0138	0.0141	0.0139
	RESISTIVITY (ohm-cm)	16750	16750	>16750			6980	16750	16750	16750	>16750	>16750
	pH	7.4	7.0	7.7			7.9	7.1	7.2	7.0	7.1	7.0
R-VALUE	TEST SPECIMEN											
	DRY DENSITY (pcf)			127 126 127								
	MOISTURE CONTENT (%)			1.8 1.8 1.8								
	EXUDATION PRESSURE (psi)			351 226 547								
	CORRECTED R-VALUE			71 68 74								
	R-VALUE STABILOMETER			70								
	R-VALUE BY EXPANSION											
DIRECT SHEAR	TEST SPECIMEN				SAT SAT SAT	SAT SAT SAT						
	NORMAL STRESS (psf)				1000 2000 3000	1000 2000 3000						
	PEAK SHEAR STRESS (psf)				1442 2577 3506	1953 3027 4152						
	ULTIMATE SHEAR STRESS (psf)				1196 1779 2689	1421 2531 3640						
	ANGLE OF INTERNAL FRICTION (Ø°)				PEAK 45.5 / ULT 37.5	PEAK 47 / ULT 47						
	COHESION (psf)				500 / 320	900 / 370						
EXPAN. TEST	INITIAL DRY DENSITY (psf)											
	INITIAL MOISTURE (%)											
	FINAL MOISTURE (%)											
	SWELL (%)											
	EXPANSION INDEX											

DIST	COUNTY	ROUTE	POST MILES	SHEET NO.	TOTAL SHEETS
08	SBD	138	16.4–19.7		

REGISTERED PROFESSIONAL ENGINEER

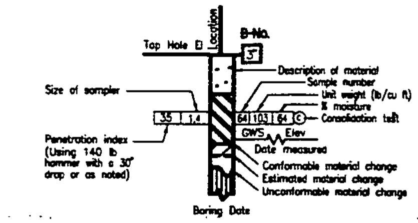
PLANS APPROVAL DATE

REGISTERED PROFESSIONAL ENGINEER DOUGLAS R. BELL No. GE 2140 Exp. 3-21-99 GEOTECHNICAL STATE OF CALIFORNIA

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

ROUTE 138 IMPROVEMENTS

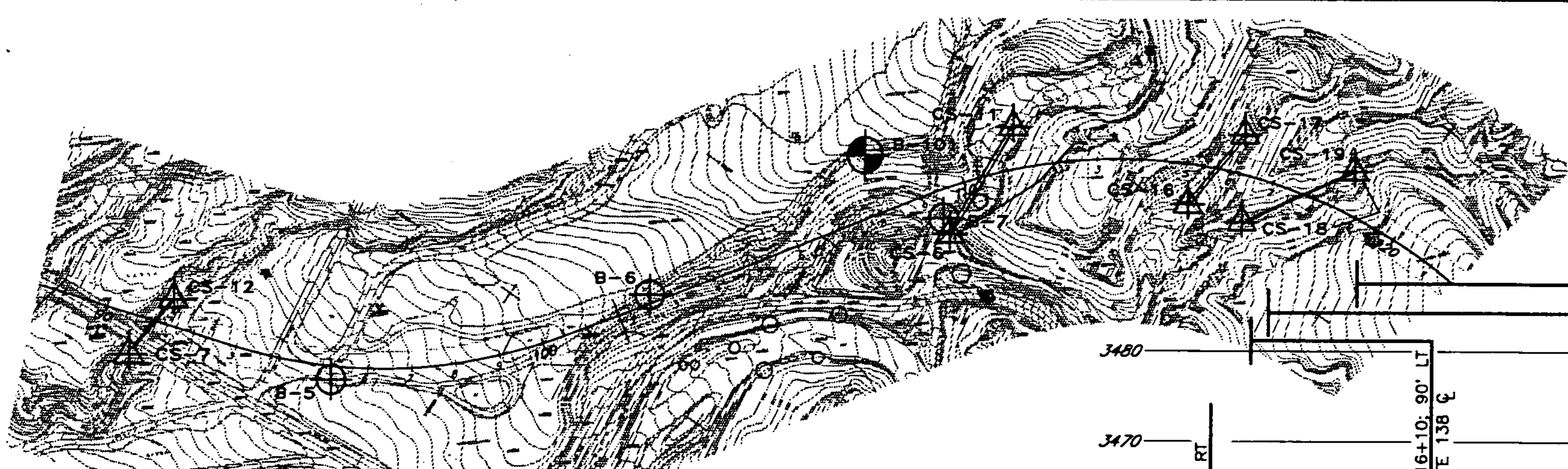
LEGEND OF BORING OPERATIONS



LEGEND OF EARTH MATERIALS

GRAVEL — SANDY CLAY or CLAYEY SAND — PEAT and/or ORGANIC MATTER — METAMORPHIC ROCK
SAND — SANDY SILT or SILTY SAND — FILL MATERIAL
SILT — SILTY CLAY — IGNEOUS ROCK
CLAY — CLAYEY SILT — SEDIMENTARY ROCK

ORIGINAL SCALE IN INCHES FOR REDUCED PLANS 0 1 2 3

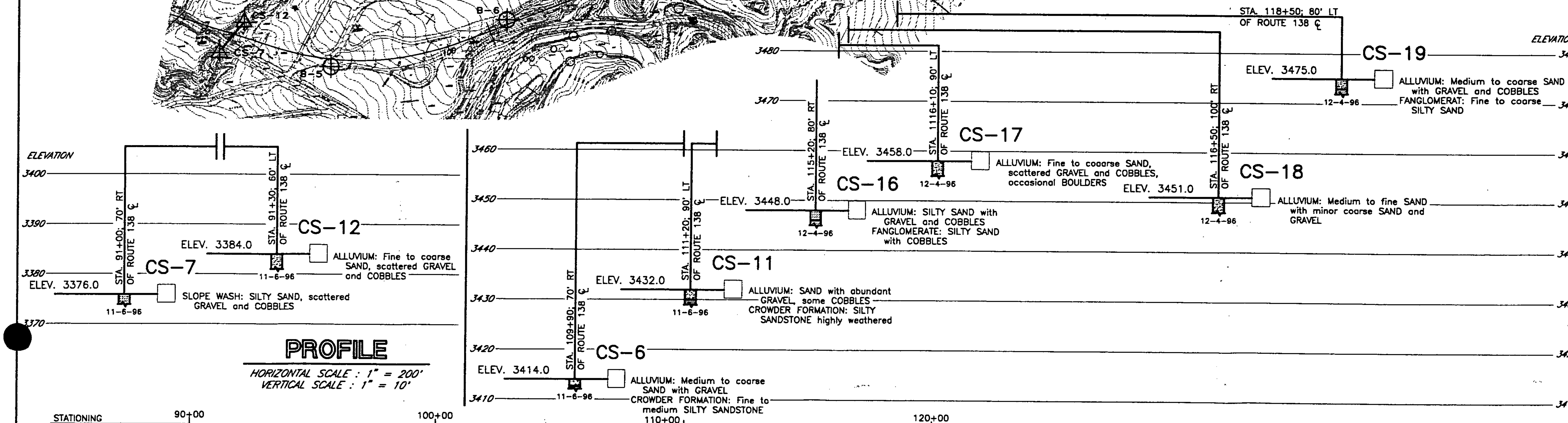


PLAN

SCALE : 1" = 200'

LEGEND

- *AGRA EARTH & ENVIRONMENTAL, Inc. HAND SAMPLE LOCATION*
- *AGRA EARTH & ENVIRONMENTAL, Inc. BORING LOCATION*
- *AGRA EARTH & ENVIRONMENTAL, Inc. TEST PIT LOCATION*
- *BORING LOCATION BY JOHN A. SAYERS & ASSOCIATES*



PROFILE

HORIZONTAL SCALE : 1" = 200'
VERTICAL SCALE : 1" = 10'

PREPARED FOR THE **STATE OF CALIFORNIA** DEPARTMENT OF TRANSPORTATION

G. BUTLER/L.E. FANNING — FIELD INVESTIGATOR
DATE: DECEMBER 4, 1996

DRAWN BY: J.B. DePOORTER
CHECK BY:

Job No. 6-212-1052
PLATE VII

SOIL SURVEY SHEET

DIST	COUNTY	ROUTE	POST MILES	SHEET NO.	TOTAL SHEETS
08	SBD	138	16.4–19.7		

REGISTERED PROFESSIONAL ENGINEER

PLANS APPROVAL DATE

REGISTERED PROFESSIONAL ENGINEER DOUGLAS R. BELL No. GE 2140 Exp. 3-31-99 GEOTECHNICAL STATE OF CALIFORNIA

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

ROUTE 138
IMPROVEMENTS

| | | |

SOIL SURVEY SHEET

DIST	COUNTY	ROUTE	POST MILES	SHEET NO.	TOTAL SHEETS
08	SBD	138	16.4–19.7		

REGISTERED PROFESSIONAL ENGINEER

PLANS APPROVAL DATE

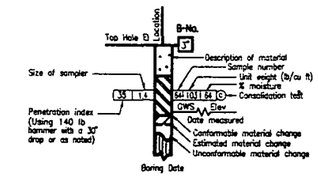
REGISTERED PROFESSIONAL ENGINEER — NICHOLAS R. BELL — No. GE 2140 — Exp. 3-31-99 — GEOTECHNICAL — STATE OF CALIFORNIA

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

ROUTE 138
IMPROVEMENTS

	BORING NO./SAMPLE NO.	CS-21	CS-2	CS-22	B-107/3	B107/6	B-107/10	B-107/10	CS-1	CS-23	B-108/2	B-108/4	B-108/6	B-108/9	B-108/15
GEN. INFO.	SAMPLE DEPTH (feet)	1.0	1.0	2.0	6.0	20.0	40.0	40.0	2.0	2.0	2.5	10.0	20.0	33.0	60.0
	UNIFIED SOIL CLASSIFICATION	SP	SP	SP					SP	SP					
	DATE SAMPLED	12-4-96	11-6-96	12-4-96	11-8-96	11-8-96	11-8-96	11-8-96	11-6-96	12-4-96	11-7-96	11-7-96	11-7-96	11-7-96	11-7-96
SIEVE ANALYSIS (Percent Passing)	1 ½" SQUARE OPENING														
	¾" SQUARE OPENING				98										
	⅜" SQUARE OPENING				95										
	No. 4 (4760 MICRONS)				89									99	
	No. 8 (2380 MICRONS)				82									98	
	No. 16 (1190 MICRONS)				70									90	
	No. 30 (590 MICRONS)				57									81	
	No. 50 (297 MICRONS)				45									71	
	No. 100 (148 MICRONS)				33									57	
	No. 200 (74 MICRONS)				23									47	
	5 MICRONS														
	2 MICRONS														
PHYSICAL TESTS	MAXIMUM DRY DENSITY (pcf)										129.5				
	OPTIMUM MOISTURE (%)										9.8				
	LIQUID LIMIT														
	PLASTICITY INDEX														
	SAND EQUIVALENT														
CHEM. TESTS	SOLUBLE CHLORIDE (%)													0.0022	
	SOLUBLE SULPHATE (%)	0.0147	0.0089	0.0099					0.0071	0.0227				0.01380	
	RESISTIVITY (ohm-cm)	11170	11170	6980					11960	16750				1270	
	pH	7.2	7.0	6.8					7.0	7.0				7.6	
R-VALUE	TEST SPECIMEN														
	DRY DENSITY (pcf)				126.3 125.0 128.8									119.8 114.3 110.4	
	MOISTURE CONTENT (%)				6.3 6.3 6.3									11.0 11.0 11.0	
	EXUDATION PRESSURE (psi)				315 213 555									511 295 156	
	CORRECTED R-VALUE				50 29 68									34 14 7	
	R-VALUE STABILOMETER				48									15	
	R-VALUE BY EXPANSION														
DIRECT SHEAR	TEST SPECIMEN					SAT SAT SAT	NAT NAT NAT	SAT SAT SAT			SAT SAT SAT	SAT SAT SAT	SAT SAT SAT		NAT NAT NAT
	NORMAL STRESS (psf)					500 1000 1500	1000 2000 3000	1000 2000 3000			500 1000 1500	500 1000 1500	1000 2000 3000		1000 2000 3000
	PEAK SHEAR STRESS (psf)					685 1156 1618	2372 3303 4929	1667 1974 3201			675 1084 1268	859 1605 2434	3191 3477 3722		1452 2648 3344
	ULTIMATE SHEAR STRESS (psf)					450 839 1196	1064 2372 2853	982 1861 2925			409 767 1135	501 818 1268	1166 1738 2465		1299 2260 3098
	ANGLE OF INTERNAL FRICTION (Ø°)					PEAK 44 ULT 36	PEAK 48 ULT 41	PEAK 44 ULT 44			PEAK 38 ULT 36	PEAK 57 ULT 39	PEAK 15 ULT 33		PEAK 47 ULT 42
	COHESION (psf)					200 100	1200 200	300 0			300 50	75 75	2900 500		400 400
EXPAN. TEST	INITIAL DRY DENSITY (psf)														
	INITIAL MOISTURE (%)														
	FINAL MOISTURE (%)														
	SWELL (%)														
	EXPANSION INDEX														

LEGEND OF BORING OPERATIONS



LEGEND OF EARTH MATERIALS

GRAVEL; SAND; SILT; CLAY; SANDY CLAY or CLAYEY SAND; SANDY SILT or SILTY SAND; SILTY CLAY; CLAYEY SILT; PEAT and/or ORGANIC MATTER; FILL MATERIAL; IGNEOUS ROCK; SEDIMENTARY ROCK; METAMORPHIC ROCK

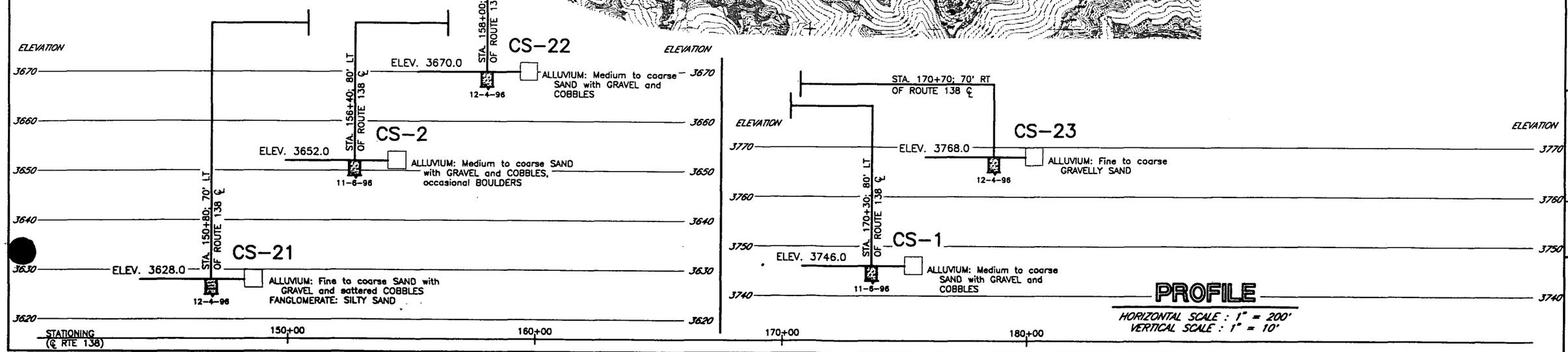
ORIGINAL SCALE IN INCHES FOR REDUCED PLANS 0 1 2 3

PLAN

SCALE : 1" = 200'

LEGEND

- AGRA EARTH & ENVIRONMENTAL, Inc. HAND SAMPLE LOCATION
- AGRA EARTH & ENVIRONMENTAL, Inc. BORING LOCATION
- AGRA EARTH & ENVIRONMENTAL, Inc. TEST PIT LOCATION
- BORING LOCATION BY JOHN A. SAYERS & ASSOCIATES



PROFILE

HORIZONTAL SCALE : 1" = 200'
VERTICAL SCALE : 1" = 10'

PREPARED FOR THE **STATE OF CALIFORNIA** DEPARTMENT OF TRANSPORTATION

G. BUTLER/L.E. FANNING — FIELD INVESTIGATOR
DATE DECEMBER 4, 1996
DRAWN BY JOE MAGLIATO
CHECK BY

Job No. 6-212-1052

PLATE IX

SOIL SURVEY SHEET

DIST	COUNTY	ROUTE	POST MILES	SHEET NO.	TOTAL SHEETS
08	SBD	138	16.4–19.7		

REGISTERED PROFESSIONAL ENGINEER

PLANS APPROVAL DATE

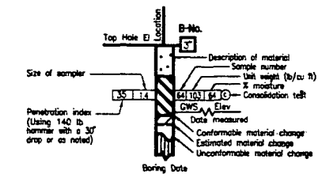
REGISTERED PROFESSIONAL ENGINEER DOUGLAS R. BELL No. GE 2140 Exp. 3-31-99 GEOTECHNICAL STATE OF CALIFORNIA

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

ROUTE 138
IMPROVEMENTS

	BORING NO./SAMPLE NO.	B-109/1	B-109/2	B-109/3	B-109/5	B-109/7	B-109/8	B-109/11	B-109/11	B-109/12	B-109/15	B-109/23
GEN. INFO.	SAMPLE DEPTH (feet)	2.5	2.5	5.0	10.0	10.0	12.5	17.5	17.5	17.5	23.5	60.0
	UNIFIED SOIL CLASSIFICATION	SM	SM	SM	SM	SM	SM					
	DATE SAMPLED	11-6-96	11-6-96	11-6-96	11-6-96	11-6-96	11-6-96	11-6-96	11-6-96	11-6-96	11-6-96	11-6-96
SIEVE ANALYSIS (Percent Passing)	1½" SQUARE OPENING					98						
	¾" SQUARE OPENING					93						
	⅜" SQUARE OPENING					85						
	No. 4 (4760 MICRONS)					74						
	No. 8 (2380 MICRONS)					62						
	No. 16 (1190 MICRONS)					42						
	No. 30 (590 MICRONS)					25						
	No. 50 (297 MICRONS)					15						
	No. 100 (148 MICRONS)					11						
	No. 200 (74 MICRONS)					8						
	5 MICRONS											
	2 MICRONS											
PHYSICAL TESTS	MAXIMUM DRY DENSITY (pcf)		124.5									
	OPTIMUM MOISTURE (%)		9.0									
	LIQUID LIMIT											
	PLASTICITY INDEX											
	SAND EQUIVALENT											
CHEM. TESTS	SOLUBLE CHLORIDE (%)											
	SOLUBLE SULPHATE (%)									0.0087	0.00772	
	RESISTIVITY (ohm-cm)									16750	12890	
	pH									7.4	7.4	
R-VALUE	TEST SPECIMEN											
	DRY DENSITY (pcf)					126.2	125.3	118.7				
	MOISTURE CONTENT (%)					3.8	3.8	3.8				
	EXUDATION PRESSURE (psi)					686	405	244				
	CORRECTED R-VALUE					76	75	73				
	R-VALUE STABILOMETER					74						
	R-VALUE BY EXPANSION											
DIRECT SHEAR	TEST SPECIMEN	SAT SAT SAT	SAT SAT SAT	SAT SAT SAT	NAT NAT NAT		NAT NAT NAT	NAT NAT NAT	SAT SAT SAT			SAT SAT SAT
	NORMAL STRESS (psf)	500 1000 1500	1000 2000 3000	1000 2000 3000	1000 2000 3000		1000 2000 3000	1000 2000 3000	1000 2000 3000			1000 2000 3000
	PEAK SHEAR STRESS (psf)	536 928 1808	1158 2035 2853	1194 1803 2801	1554 2925 3743		1790 2904 3815	1616 2567 3428	2158 3252 4407			1350 2301 3610
	ULTIMATE SHEAR STRESS (psf)	341 656 1047	757 1513 2352	877 1473 2807	1217 2808 3078		1370 2158 3610	1432 2413 3405	1871 3037 4162			971 1810 2838
	ANGLE OF INTERNAL FRICTION (Ø°)	PEAK 47 ULT 35	PEAK 41 ULT 38	PEAK 40 ULT 38	PEAK 50 ULT 42		PEAK 45 ULT 45	PEAK 42.5 ULT 42.5	PEAK 48 ULT 48			PEAK 47 ULT 40
	COHESION (psf)	0 0	300 0	350 150	300 300		850 330	700 500	1090 800			300 100
EXPAN. TEST	INITIAL DRY DENSITY (psf)											
	INITIAL MOISTURE (%)											
	FINAL MOISTURE (%)											
	SWELL (%)											
	EXPANSION INDEX											

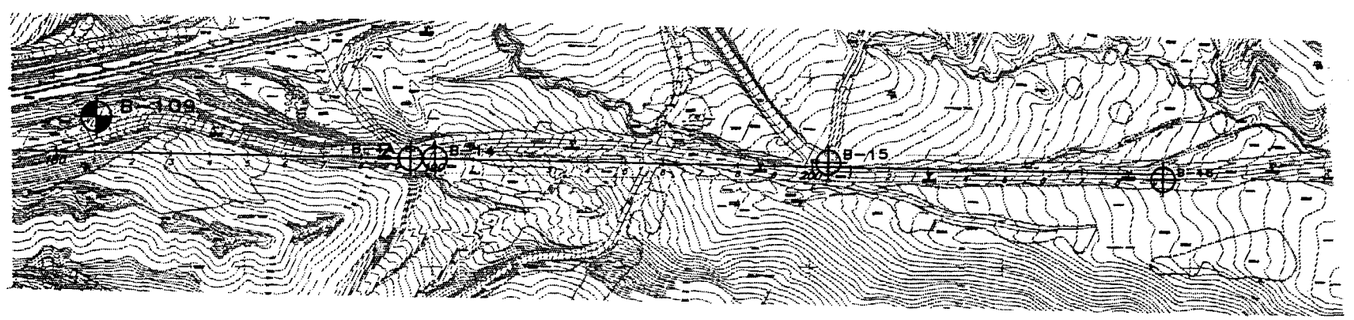
LEGEND OF BORING OPERATIONS



LEGEND OF EARTH MATERIALS

GRAVEL, SAND, SILT, CLAY, SANDY CLAY or CLAYEY SAND, SANDY SILT or SILTY SAND, SILTY CLAY, CLAYEY SILT, PEAT and/or ORGANIC MATTER, FILL MATERIAL, IGNEOUS ROCK, SEDIMENTARY ROCK, METAMORPHIC ROCK

ORIGINAL SCALE IN INCHES FOR REDUCED PLANS 0 1 2 3



N

PLAN

SCALE : 1" = 200'

LEGEND

- AGRA EARTH & ENVIRONMENTAL, Inc. HAND SAMPLE LOCATION
- AGRA EARTH & ENVIRONMENTAL, Inc. BORING LOCATION
- AGRA EARTH & ENVIRONMENTAL, Inc. TEST PIT LOCATION
- BORING LOCATION BY JOHN A. SAYERS & ASSOCIATES

STATIONING (℄ RTE 138) 180+00 190+00 200+00 210+00

PREPARED FOR THE **STATE OF CALIFORNIA** DEPARTMENT OF TRANSPORTATION

G. BUTLER/L.E. FANNING — FIELD INVESTIGATOR
DATE NOVEMBER 6, 1996

JOE MAGLIATO

DRAWN BY

CHECK BY

Job No. 6-212-1052

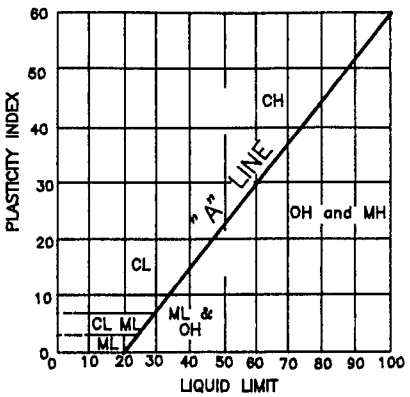
PLATE X

APPENDIX A

TEST BORING LOGS

UNIFIED SOIL CLASSIFICATION

Pt	OH	CH	MH	OL	CL	ML	SC	SM	SP	SW	GC	GM	GP	GW
Highly organic soils	Silts and clays Liquid limit >50%			Silts and clays Liquid limit <50%			Sands with fines >12% fines		Clean sands <5% fines		Gravels with fines >12% fines		Clean gravels <5% fines	
							Sands—more than 50% of coarse fraction is smaller than No. 4 sieve.				Gravels—more than 50% of coarse fraction is larger than No 4 sieve.			
Fine grained soils (More than 50% is smaller than No. 200 sieve)							Coarse grained soils (More than 50% is larger than No. 200 sieve)							



LABORATORY CLASSIFICATION CRITERIA

GW and SW—$C_u = \frac{D_{60}}{D_{10}}$ greater than 4 for SW & 6 for GW ; $C_c = \frac{(D_{30})^2}{D_{10} \times D_{60}}$ between 1 and 3.

GP and SP — Clean gravel or sand not meeting requirments for GW and SW.
GM and SM — Atterberg limits below "A" line or P.I. less than 4.
GC and SC — Atterberg limits above "A" line with P.I. greater than 7.

FINES (silt or clay)	Fine sand	Medium sand	Coarse sand	Fine gravel	Coarse gravel	Cobbles	Boulders
Sieve sizes 200	40		10	4	3/4"	3"	10"

Classification of earth materials shown on this sheet is based on field inspection and should not be construed to imply laboratory anaylsis unless so stated.

MATERIAL SYMBOLS

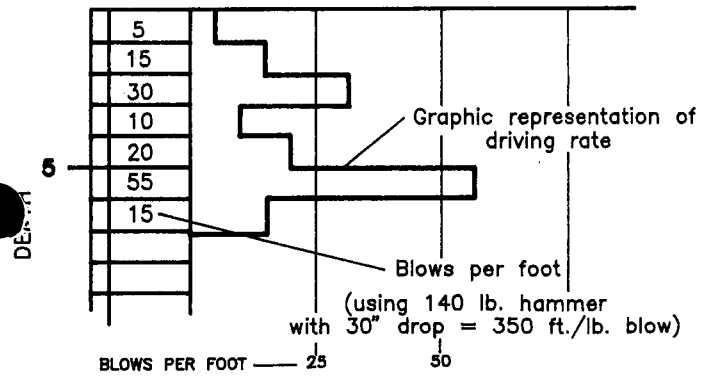
- EXTRUSIVE IGNEOUS ROCK
- ASPHALT
- CLAYSTONE
- CLAYEY SANDSTONE
- CLAYEY SILTSTONE
- METAMORPHIC ROCK
- CONCRETE
- CONGLOMERATE
- INTRUSIVE IGNEOUS ROCK
- INTERBEDDED LIMESTONE AND SHALE
- LIMESTONE
- SANDY CLAYSTONE
- SANDSTONE
- SANDY SILTSTONE
- SILTSTONE
- SILTY CLAYSTONE
- SILTY SANDSTONE

CONSISTENCY CLASSIFICATION FOR SOILS

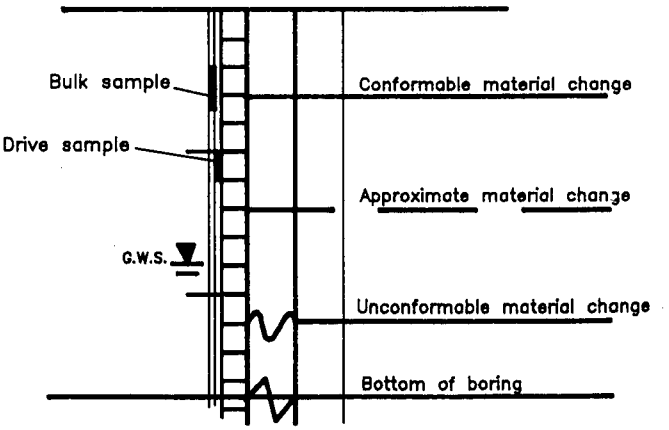
According to the Standard Penetration Test

No. of blows	Granular	Cohesive
0–5	Very loose	Very soft
6–10	Loose	Soft
11–20	Semicompact	Stiff
21–35	compact	Very stiff
36–70	dense	Hard
>70	Very dense	Very hard

LEGEND OF PENETRATION TEST



LEGEND OF BORING



TEST BORING LOG

TYPE 8" Hollow Stem | ELEV./ LOC.: 3474' / Sta. 108+15, 134' Lt | BORING **B-101**

SAMPLE TYPE	DRY DENSITY (lbs/cu.ft.)	MOISTURE (%)	BLOWS/FOOT	SAMPLE SIZE (INCHES)	SAMPLE NO.	DEPTH IN FEET	MATERIAL SYMBOL	UNIFIED SOIL CLASS.	
								SM	**FANGLOMERATE:** Moderate brown fine to medium SILTY SAND wtih minor coarse SAND to fine subangular GRAVEL, dry
BAG					1				
DRV	109	3.2	92	2.5	2				
						5			
DRV			30	NSR					... (5 feet) color change to light olive-gray
DRV			27	NSR					
DRV			20	1.4	3				... (7.5 feet) semi-compact
						10			
DRV			22	NSR					
DRV			36	NSR					
DRV	124	4.2	38	2.5	4				
						15			
DRV	128	4.1	60	2.5	5				
DRV	129	5.7	53	2.5	6				
						20			
DRV			37	1.4	7				... (20 feet) dense
DRV	125	8.0	42	2.5	8				
						25			
DRV	122	5.3	55	2.5	9				
DRV			42	1.4	10				... (27.5 feet) dense
						30			
DRV			50	NSR					... (30 feet) rock push
DRV	118	5.4	80	2.5	11				
						35			
DRV			50	1.4	12				... (35 feet) dense
DRV			50	NSR					... (37.5 feet) rock push
						40			
DRV			76	2.5	13				... (40 feet) rock push
DRV	119	11.5	63	2.5	14				**BEDROCK (CROWDER FORMATION):** Yellow-gray fine to medium SILTY SANDSTONE with minor fine subangular GRAVEL
						45			
DRV			80	1.4	15				

Continued

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

LOGGED BY LEF/GB | DATE 11-26-96

TEST BORING LOG

TYPE 8" Hollow Stem	ELEV./ LOC.: 3474' / Sta. 108+15, 134' Lt	BORING B-101

NOTES:

1) Total depth of boring: 49 feet.
2) No caving.
3) No groundwater was encountered.
4) Elevation and stationing from 50 scale plans.
5) Boring backfilled on November 26, 1996.
6) NSR = No Sample Recovered.
   DRV = Drive Cylinder Sample.
   SPT = Standard Penetration Test.

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

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

LOGGED BY LEF/GB | DATE 11-26-96

TEST BORING LOG

TYPE 8" Hollow Stem | ELEV./ LOC.: 3616' / Sta. 145+47, 10' Lt | BORING **B-103**

SAMPLE TYPE	DRY DENSITY (lbs/cu.ft.)	MOISTURE (%)	BLOWS/FOOT	SAMPLE SIZE (INCHES)	SAMPLE NO.	DEPTH IN FEET	MATERIAL SYMBOL	UNIFIED SOIL CLASS.	
									BEDROCK (CROWDER FORMATION): Gray, pink, orange fine SILTY SANDSTONE with and fine angular GRAVEL to medium COBBLES, damp
DRV BAG	108	5.1	94	2.5	1 2	5			
DRV			200	1.4	3	10			
DRV			100	NSR		15			
DRV			100	1.4	4	20			

NOTES:

1) Total depth of boring: 21.5 feet.
2) No caving.
3) No groundwater was encountered.
4) Elevation and stationing from 50 scale plans.
5) Boring backfilled on November 12, 1996.
6) NSR = No Sample Recovered.
   DRV = Drive Cylinder Sample.
   SPT = Standard Penetration Test.

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

LOGGED BY LEF/GB | DATE 11-12-96

TEST BORING LOG

TYPE 8" Hollow Stem | ELEV./ LOC.: 3631' / Sta. 152+13, 15' Rt | BORING **B-104**

SAMPLE TYPE	DRY DENSITY (lbs/cu.ft.)	MOISTURE (%)	BLOWS/FOOT	SAMPLE SIZE (INCHES)	SAMPLE NO.	DEPTH IN FEET	MATERIAL SYMBOL	UNIFIED SOIL CLASS.	
BAG					1			SP	**FILL:** Moderate brown fine to medium SAND, minor coarse SAND to fine angular GRAVEL, damp
DRV	104	3.7	40	2.5	2				
						5			
DRV			40	NSR					
DRV			40	2.5	3				
								SP	**FANGLOMERATE:** Moderate red-brown fine to medium SAND with minor SILT, coarse SAND and fine angular GRAVEL, damp
DRV	113	1.8	55	1.4	4				
						10			
DRV	117	2.0	53	NSR					... (7.5 feet) dense
DRV			84	2.5	5				... (11 feet) disturbed sample
DRV	122	9.5	94	2.5	6				... (12.5 feet) rock refusal, moved boring approximately 10 feet west to sample at 12.5 feet
DRV			69	1.4	7				
						15			**BEDROCK (CROWDER FORMATION):** Gray-orange to moderate brown fine SANDSTONE with moderate CLAY, minor medium to coarse SAND and fine angular GRAVEL to medium angular COBBLES, damp
DRV			100	1.4	8				
						20			
DRV		11.1	99	2.5	9				

NOTES:

1) Total depth of boring: 21 feet.
2) No caving.
3) No groundwater was encountered.
4) Rock refusal at 12.5 feet, boring moved approximately 10 feet west to collect sample at 12.5 feet.
5) Elevation and stationing from 50 scale plans.
6) Boring backfilled on November 12, 1996.
7) NSR = No Sample Recovered.
   DRV = Drive Cylinder Sample.
   SPT = Standard Penetration Test.

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

LOGGED BY LEF/GB | DATE 11-12-96

TEST BORING LOG

TYPE 8" Hollow Stem | ELEV./ LOC.: 3657' / Sta. 155+90, 17'Lt | BORING **B-105**

SAMPLE TYPE	DRY DENSITY (lbs/cu.ft.)	MOISTURE (%)	BLOWS/FOOT	SAMPLE SIZE (INCHES)	SAMPLE NO.	DEPTH IN FEET	MATERIAL SYMBOL	UNIFIED SOIL CLASS.	
DRV BAG			68	NSR				SP	**FILL:** Moderate brown fine to medium SAND, moderate SILT and fine angular GRAVEL, dry
									... (2.5 feet) damp
DRV			70	NSR		5			
DRV			68	1.4	1				... (7.5 feet) dense, minor CLAY
DRV	102	9.2	80	2.5	2	10		SP	**OLDER ALLUVIUM (?):** Moderate brown to moderate red-brown medium SAND, minor SILT, minor fine and coarse SAND and fine angular GRAVEL, damp
DRV	117	10.4	83	2.5	3				
DRV			100	1.4	4	15			**BEDROCK (CROWDER FORMATION)** Gray, orange-pink fine SANDSTONE with minor CLAY and SILT, minor medium to coarse SAND and fine angular GRAVEL, damp
DRV			100	NSR		20			
DRV			100	1.4	5	25			

NOTES:

1) Total depth of boring: 26.5 feet.
2) No caving.
3) No groundwater was encountered.
4) Elevation and stationing from 50 scale plans.
5) Boring backfilled on November 12, 1996.
6) NSR = No Sample Recovered.
   DRV = Drive Cylinder Sample.
   SPT = Standard Penetration Test.

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

LOGGED BY LEF/GB | DATE 11-12-96

TEST BORING LOG

TYPE 8" Hollow Stem | ELEV./ LOC.: 3679' / Sta. 158+58, 33' Rt | BORING **B-106**

SAMPLE TYPE	DRY DENSITY (lbs/cu.ft.)	MOISTURE (%)	BLOWS/FOOT	SAMPLE SIZE (INCHES)	SAMPLE NO.	DEPTH IN FEET	MATERIAL SYMBOL	UNIFIED SOIL CLASS.	
								SM	**FILL:** Pale to moderate brown fine to medium SILTY SAND with minor fine subangular GRAVEL, damp
BAG DRV	116	3.3	36	2.5	1 2				
DRV	99	3.7	70	2.5	3	5			**BEDROCK (CROWDER FORMATION):** Yellow-gray fine to medium SANDSTONE with minor SILTY, coarse SAND to medium subangular GRAVEL, moderate brown fine to medium CLAYEY SANDSTONE intervals, moist
DRV			11	1.4	4				
DRV	95	6.2	30	2.5	5	10			
DRV	107	4.5	88	2.5	6	15			
			85	1.4	7	20			

NOTES:

1) Total depth of boring: 21.5 feet.
2) No caving.
3) No groundwater was encountered.
4) Elevation and stationing from 50 scale plans.
5) Boring backfilled on November 26, 1996.
6) NSR = No Sample Recovered.
   DRV = Drive Cylinder Sample.
   SPT = Standard Penetration Test.

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

LOGGED BY LEF/GB | DATE 11-26-96

AGRA Earth & Environmental, Inc.

TEST BORING LOG

TYPE 24" Bucket Rig | ELEV./ LOC.: 3796' / Sta. 169+00, 28' Rt | BORING **B-107**

SAMPLE TYPE	DRY DENSITY (lbs/cu.ft.)	MOISTURE (%)	BLOWS/FOOT	SAMPLE SIZE (INCHES)	SAMPLE NO.	DEPTH IN FEET	MATERIAL SYMBOL	UNIFIED SOIL CLASS.	
BAG					1			SP	**FILL:** Moderate brown fine to medium SAND with minor SILT and fine subangular GRAVEL, damp
DRV			90	1.4	2	5			**BEDROCK (CROWDER FORMATION):** Yellow-gray medium to coarse SILTY SANDSTONE with subangular fine crystalline GRAVEL to small COBBLES, fine SILTY SANDSTONE laminations
BAG					3				... (4 to 5 feet) moderate brown fine SANDSTONE
									... (6.3 to 9.8 feet) moderate brown fine CLAYEY SANDSTONE
DRV	112	43.0	9	2.5	4	10			... (10 feet) rock push
									... (10.5 feet) pushing rock, bucket bounce
									... (12.2 feet) lamination attitude - N83E/19N
DRV			100	1.4	5	15			... (14.8 to 17.6 feet) abundant fine to coarse subangular crystalline GRAVEL
									... (17.7 feet) lamination attitude - N67W/24N
DRV	128	6.0	18	2.5	6	20			... (20 feet) gray-orange medium SAND
									... (22.3 feet) lamination attitude - N87E/27N
DRV			100	1.4	7	25			... (26.2 - 27 feet) abundant GRAVEL
									... (27.9 feet) 6-inch thick fine SANDSTONE attitude - N66E/27N
DRV		3.0	15	2.5	8	30			... (30.6 to 31 feet) abundant GRAVEL
									... (32 to 32.5 feet) abundant GRAVEL
									... (33.3 feet) lamination attitude - N45W/20N
DRV			100	1.4	9	35			
DRV	139	4.1	18	2.5	10	40			
DRV			100	1.4	11	45			

NOTES:

Continued

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

LOGGED BY LEF/GB | DATE 11-8-96

TEST BORING LOG

TYPE 24" Bucket Rig	ELEV./ LOC.: 3796' / Sta. 169+00, 28' Rt	BORING B-107

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

1) Total depth of boring: 46.5 feet.
2) No caving.
3) Casing set to 2.5 feet.
4) No groundwater was encountered.
5) Elevation and stationing from 50 scale plans.
6) 2.5 inch samples driven with following Kelly bar weights (30-inch drop):
   0 - 30 feet: 3,450 lbs.
   31 - 60 feet: 2,050 lbs.
   61 - 90 feet: 1,140 lbs.
7) 1.4 inch samples driven with 140 lb. hammer falling 30 inches.
8) Boring backfilled and tamped on November 8, 1996.
9) NSR = No Sample Recovered.
   DRV = Drive Cylinder Sample.
   SPT = Standard Penetration Test.

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

LOGGED BY LEF/GB | DATE 11-8-96

TEST BORING LOG

TYPE 24" Bucket Rig | ELEV./ LOC.: 3850' / Sta. 176+75, 08' Lt | BORING **B-108**

SAMPLE TYPE	DRY DENSITY (lbs/cu.ft.)	MOISTURE (%)	BLOWS/FOOT	SAMPLE SIZE (INCHES)	SAMPLE NO.	DEPTH IN FEET	MATERIAL SYMBOL	UNIFIED SOIL CLASS.	
									BEDROCK (CROWDER FORMATION): Gray, orange-pink fine to medium SILTY SANDSTONE and pale yellow-gray medium to coarse SANDSTONE, with subangular crystalline, fine GRAVEL to small COBBLES and minor moderate brown medium SANDY CLAYSTONE to fine, SILTY SANDSTONE intervals or lenses, slightly damp
DRV	135	5.4	19	2.5	1				
BAG					2				
DRV			105	1.4	3	5			... (5.2 feet) erosion surface attitude - N78E/18N
DRV	128	6.5	18	2.5	4	10			... (11.3 feet) fine SAND lamination attitude - N60E/22N
									... (13.8 feet) fine SAND lamination attitude - N68W/22N
DRV			100	1.4	5	15			... (15.4 feet) erosional surface attitude - N87W/34N
DRV	133	9.4	10	2.5	6	20			
DRV			111	1.4	7	25			
DRV	126	11.1	10	2.5	8	30			
BAG					9				... (33.2 feet) 1-inch thick GRAVEL attitude - N85E/24N
DRV			155	1.4	10	35			... (34.5 feet) very moist, a top fine CLAYEY SANDSTONE interval
DRV	133	7.5	15	2.5	11	40			
DRV			155	1.4	12	45			

Continued

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

LOGGED BY LEF/GB | DATE 11-7-96

TEST BORING LOG

TYPE 24" Bucket Rig | ELEV./ LOC.: 3850' / Sta. 176+75, 08' Lt | BORING **B-108**

SAMPLE TYPE	DRY DENSITY (lbs/cu.ft.)	MOISTURE (%)	BLOWS/FOOT	SAMPLE SIZE (INCHES)	SAMPLE NO.	DEPTH IN FEET	MATERIAL SYMBOL	UNIFIED SOIL CLASS.	
DRV	120	8.1	31	2.5	13				**BEDROCK (CROWDER FORMATION):** Gray, orange-pink fine to medium SILTY SANDSTONE and pale yellow-gray medium to coarse SANDSTONE, with subangular crystalline, fine GRAVEL to small COBBLES and minor moderate brown medium SANDY CLAYSTONE to fine, SILTY SANDSTONE intervals or lenses, slightly damp
DRV			100	1.4	14	55			
DRV	114	8.9	44	2.5	15	60			... (59 feet) very moist above fine SILTY SANDSTONE interval
DRV			100	1.4	16	65			
DRV	113	8.2	36	2.5	17	70			... (72.8 feet) 2-inch thick fine SAND attitude - N77E/18N
DRV			100	1.4	18	75			
DRV	118	6.0	51	2.5	19	80			

NOTES:

1) Total depth of boring: 81 feet.
2) No caving.
3) Casing set to 2.0 feet.
4) No groundwater was encountered.
5) Elevation and stationing from 50 scale plans.
6) 2.5 inch samples driven with following Kelly bar weights:
   0 - 30 feet: 3,450 lbs.
   31 - 60 feet: 2,050 lbs.
   61 - 90 feet: 1,140 lbs.
7) 1.4 inch samples driven with 140 lb. hammer falling 30 inches.
8) Boring backfilled and tamped on November 8, 1996.
9) NSR = No Sample Recovered.
   DRV = Drive Cylinder Sample.
   SPT = Standard Penetration Test.

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

LOGGED BY LEF/GB | DATE 11-7-96

TEST BORING LOG

TYPE 24" Bucket Rig | ELEV./ LOC.: 3844' / Sta. 181+05, 95' Lt | BORING **B-109**

SAMPLE TYPE	DRY DENSITY (lbs/cu.ft.)	MOISTURE (%)	BLOWS/FOOT	SAMPLE SIZE (INCHES)	SAMPLE NO.	DEPTH IN FEET	MATERIAL SYMBOL	UNIFIED SOIL CLASS.	
								SM	**TERRACE DEPOSIT:** Moderate yellow-brown fine to medium SILTY SAND with moderate coarse SAND and fine angular crystalline GRAVEL to small COBBLES, moderately friable, moist
DRV	113	8.1	5	2.5	1				
BAG					2				
						5			... (4.5 feet) fine SAND lamination attitude - N50E/45E
DRV	118	6.3	3	2.5	3				
BAG					4				
DRV			31	1.4	5				... 7.5 feet) compact
						10			
DRV	119	8.2	3	2.5	6				... (10 feet) increasing moisture
BAG					7				... (11.2 feet) fine SAND lamination attitude - N13W/06NE
DRV	117	5.0	4	2.5	8				... (13 feet) very moist
BAG					9				
						15			
DRV			56	1.4	10				... (15 to 17.5 feet) abundant GRAVEL to COBBLE at basal contact
DRV	121	5.0	3	2.5	11				**BEDROCK (CROWDER FORMATION):** Gray, orange-pink fine to medium SILTY SANDSTONE with pale yellow-brown CLAYEY SILTSTONE with medium SAND intervals, minor subangular crystalline fine GRAVEL to small COBBLES, moderately cemented damp, erosional contact with terrace
BAG					12				
						20			
DRV	127	10.1	6	2.5	13				
BAG					14				
BAG					15				... (22 feet) fine SAND lamination attitude - N25W/21NE
						25			
DRV			153	1.4	16				
									... (28.6 feet) fine SAND lamination attitude - N86E/20N
						30			
DRV	125	7.8	14	2.5	17				... 29.8 feet) 2-inch thick SILTSTONE attitude - N82W/16N
						35			
DRV			184	1.4	18				
						40			
DRV	127	6.5	15	2.5	19				
						45			
DRV			110	1.4	20				... (45 feet) erosional surface attitude - N30W/23NE
									... (46 feet) biotite? lamination attitude - N42W/21NE

Continued

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

LOGGED BY LEF/GB | DATE 11-6-96

TEST BORING LOG

TYPE 24" Bucket Rig | ELEV./ LOC.: 3844' / Sta. 181+05, 95' Lt | BORING **B-109**

SAMPLE TYPE	DRY DENSITY (lbs/cu.ft.)	MOISTURE (%)	BLOWS/FOOT	SAMPLE SIZE (INCHES)	SAMPLE NO.	DEPTH IN FEET	MATERIAL SYMBOL	UNIFIED SOIL CLASS.	
DRV	127	6.5	27	2.5	21				... (50.7 feet) fine SAND lamination attitude - N27W/15NE
DRV			161	1.4	22	55			... (56.5 feet) GRAVEL interval attitude - N85W/13N
DRV	118	5.9	43	2.5	23	60			
DRV			109	1.4	24	65			
DRV	130	7.8	36	2.5	25	70			

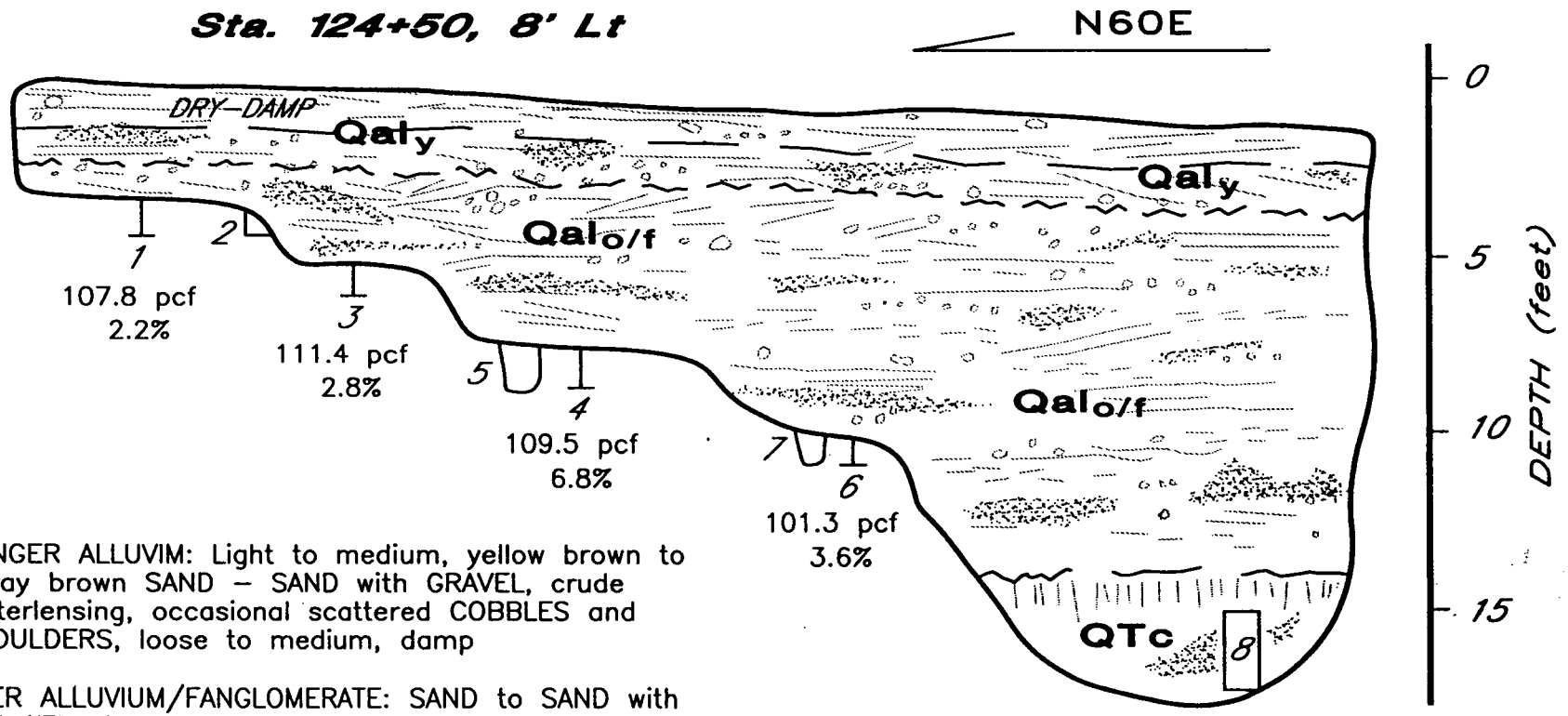
NOTES:

1) Total depth of boring: 71 feet.
2) No caving.
3) Casing set to 3.0 feet.
4) No groundwater was encountered.
5) Elevation and stationing from 50 scale plans.
6) 2.5 inch samples driven with following Kelly bar weights:
   0 - 30 feet: 3,450 lbs.
   31 - 60 feet: 2,050 lbs.
   61 - 90 feet: 1,140 lbs.
7) 1.4 inch samples driven with 140 lb. hammer falling 30 inches.
8) Boring backfilled and tamped on November 6, 1996.
9) NSR = No Sample Recovered.
   DRV = Drive Cylinder Sample.
   SPT = Standard Penetration Test.

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

LOGGED BY LEF/GB | DATE 11-6-96

Sta. 124+50, 8' Lt



0–3' YOUNGER ALLUVIM: Light to medium, yellow brown to gray brown SAND – SAND with GRAVEL, crude interlensing, occasional scattered COBBLES and BOULDERS, loose to medium, damp

3'–14' OLDER ALLUVIUM/FANGLOMERATE: SAND to SAND with GRAVEL, light to medium, ruddy gray brown to light medium ruddy yellow brown, interlensed fines SANDS/ SILTY SANDS and coarser GRAVELS and SANDS, bedded crudely <1/2"–12", scattered BOULDERS and COBBLES to >12", medium dense, appears dense below 8', damp, becomes moist below 3'

14'+ CROWDER FORMATION: SANDSTONE, ruddy yellow brown, weathered, arkosic, SILTY, scattered GRAVEL, much cleaner/coherent than Qal, soft to medium; weak cementation, moist to very moist at contact, moist to damp below

NOTES:

1. Total Depth = 17 feet
2. No Groundwater Encountered
3. Moderate Slough/Cave of Qal_y
4. Minor Cave of $Qal_{o/f}$

SCALE: 1"=5'
HORIZONTAL=VERTICAL

ROUTE 138. PHASE 2
POST MILE 16.4/19.7
SAN BERNARDINO COUNTY, CALIFORNIA

TEST PIT LOG – TP–1

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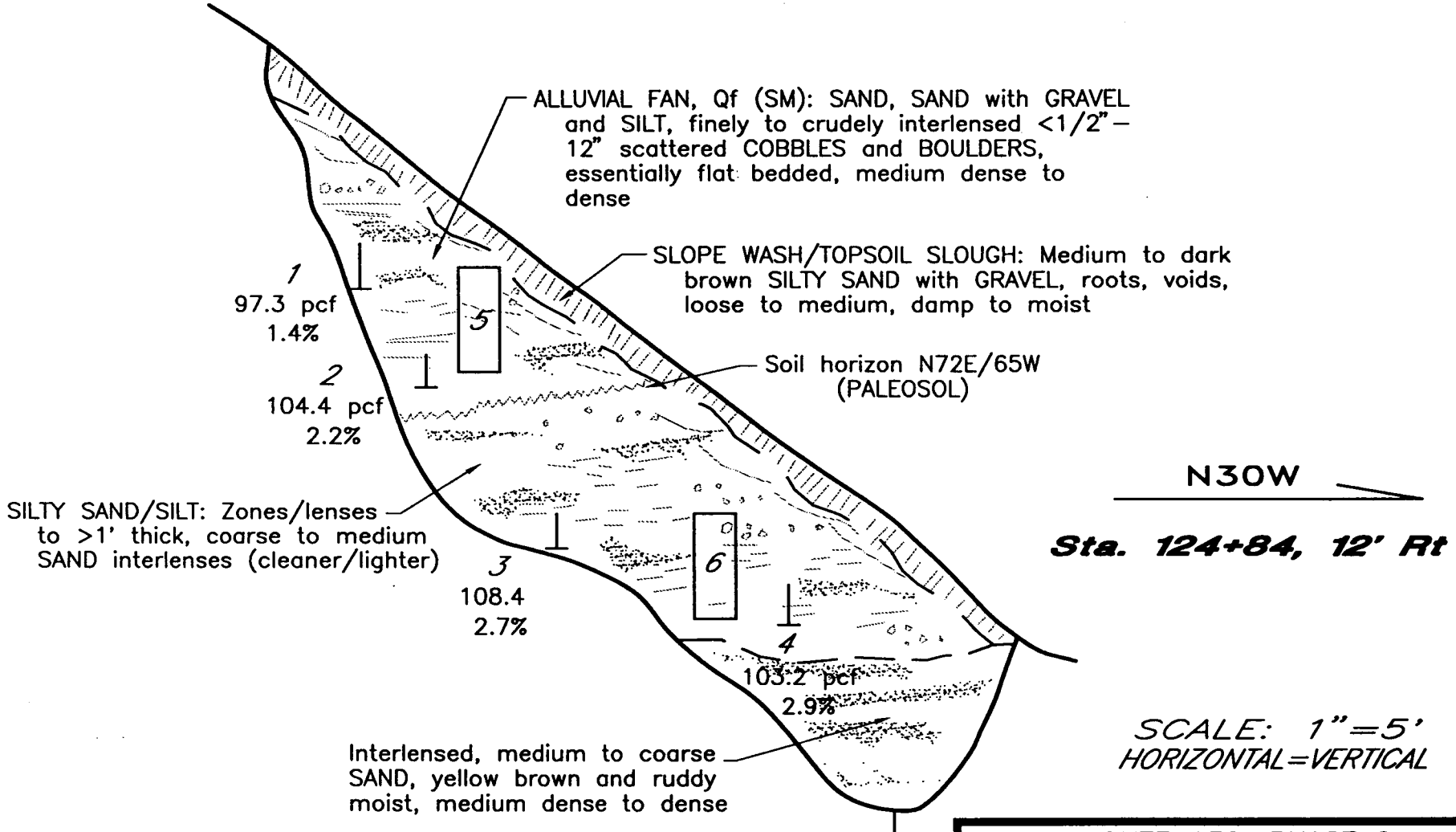
FIELD	DRAFT	APPROV	DATE	JOB No.
LEF	JBD		12–20–96	6–212–1052



ROUTE 138. PHASE 2
POST MILE 16.4/19.7
SAN BERNARDINO COUNTY, CALIFORNIA

TEST PIT LOG – TP-2

AGRA
Earth & Environmental, Inc.

FIELD	DRAFT	APPROV.	DATE	JOB No.
LEF	JBD		12-20-96	6-212-1052

FILE: 6-1052Z
PLOT DATE: 12-20-96

APPENDIX B

LABORATORY TEST RESULTS

LABORATORY TESTING

The laboratory testing was designed to fit the specific needs of this project and was limited to testing on-site materials. A brief description of each type of test is presented below. Specific results are given on pages B-2 through B-54.

In addition to the in-situ field tests, strength characteristics of the subsurface soils were determined in the laboratory by direct shear tests performed on 17 undisturbed samples and 2 remolded samples. Samples were tested under natural moisture or submerged conditions. Samples were tested under three different normal loads. All samples were tested in a 2.5-inch I.D. circular shear box, using a controlled displacement rate of 0.04-inch per minute in general accordance with ASTM D3080. Both peak and ultimate shear strength values were recorded. Peak strength is the maximum strength recorded. Ultimate strength is defined as the shear strength at .25 inch displacement.

A consolidated-undrained triaxial compression test was conducted on one sample in general accordance with ASTM D4767.

Settlement characteristics of 7 soil samples were evaluated by means of laboratory consolidation tests. Samples were tested in a floating ring consolidometer using a dead weight lever system for load application in general accordance with ASTM D2435.

Maximum density/optimum moisture relationship tests were performed on 2 soil samples in general accordance with California Test Method 216.

Particle size analyses were performed on 5 soil samples in general accordance with ASTM D422.

Resistance (R-value) tests were performed on 6 samples. Testing was conducted in general accordance with California Test Method No. 301.

The concentration of soluble sulphate was determined for soil samples in general accordance with California Test Method No. 417. The chloride content of four samples were determined in accordance with California Test Method 422.

Corrosivity tests were performed on 29 soil samples to determine the pH and minimum electrical resistance of the soils. These tests were conducted in general accordance with California Test Method No. 643.

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

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DIRECT SHEAR TESTS

Boring No./ Sample No.	Normal Stress (psf)	Peak Shear Strength		Shear Stress at 0.25-Inch Displacement (psf)
		Stress (psf)	Displacement (inch)	
101/8	1000	1442	0.150	1196
101/8	2000	2577	0.090	1776
101/8	3000	3508	0.120	2689
101/10	1000	1953	0.140	1421
101/10	2000	3027	0.150	2531
101/10	3000	4152	0.160	3640
TP2/2	1000	920	0.200	890
TP2/2	2000	1513	0.240	1503
TP2/2	3000	2659	0.200	2587
103/1	1000	757	0.250	757
103/1	2000	1340	0.250	1340
103/1	3000	2158	0.250	2150
107/6	500	685	0.100	450
107/6	1000	1156	0.110	839
107/6	1500	1616	0.100	1196
107/10	1000	1687	0.085	982
107/10	2000	1974	0.120	1861
107/10	3000	3201	0.130	2925
*107/10	1000	2372	0.050	1084
*107/10	2000	3303	0.060	2372
*107/10	3000	4929	0.080	2853
**108/2	500	675	0.045	409
**108/2	1000	1084	0.050	767
**108/2	1500	1268	0.065	1135

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DIRECT SHEAR TESTS

(Continued)

Boring No./ Sample No.	Normal Stress (psf)	Peak Shear Strength		Shear Stress at 0.25-Inch Displacement (psf)
		Stress (psf)	Displacement (inch)	
108/4	500	859	0.095	501
108/4	1000	1605	0.070	818
108/4	1500	2434	0.070	1268
108/6	1000	3191	0.095	1166
108/6	2000	3477	0.120	1738
108/6	3000	3722	0.110	2485
*108/15	1000	1452	0.095	1299
*108/15	2000	2649	0.100	2260
*108/15	3000	3344	0.140	3098
109/1	500	536	0.087	341
109/1	1000	926	0.125	658
109/1	1500	1608	0.125	1047
**109/2	1000	1156	0.075	757
**109/2	2000	2035	0.090	1513
**109/2	3000	2853	0.110	2352
109/3	1000	1194	0.087	877
109/3	2000	1803	0.125	1413
109/3	3000	2801	0.175	2607
*109/6	1000	1554	0.110	1217
*109/6	2000	2925	0.170	2608
*109/6	3000	3743	0.160	3078
*109/8	1000	1790	0.085	1370
*109/8	2000	2904	0.130	2158
*109/8	3000	3865	0.190	3610

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DIRECT SHEAR TESTS

(Continued)

Boring No./ Sample No.	Normal Stress (psf)	Peak Shear Strength		Shear Stress at 0.25-Inch Displacement (psf)
		Stress (psf)	Displacement (inch)	
*109/11	1000	1616	0.110	1432
*109/11	2000	2567	0.170	2413
*109/11	3000	3426	0.220	3405
109/11	1000	2168	0.150	1871
109/11	2000	3252	0.170	3037
109/11	3000	4407	0.180	4162
109/23	1000	1350	0.090	971
109/23	2000	2301	0.095	1810
109/23	3000	3610	0.150	2638

* Tested at natural moisture.
** Remolded to 90 percent of maximum dry density.

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MAXIMUM DENSITY TESTS

Boring No./Sample No.	Maximum Dry Density (pcf)	Optimum Moisture Content (%)
108/2	129.5	9.8
109/2	124.5	9.0

RESISTANCE VALUE

Boring No./ Sample No.	Initial Moisture (%)	Dry Density (pcf)	Exudation Pressure (psi)	Expansion Dial Reading (inch)	Corrected R-Value
101/2A	1.8	127.0	351	0	71
101/2B	1.8	126.3	226	0	68
101/2C	1.8	127.3	547	0	74
T2/5A	2.1	118.2	648	0	75
T2/5B	2.1	116.5	215	0	72
T2/5C	2.1	117.9	356	0	74
103/2A	7.0	127.1	467	0.0023	54
103/2B	7.0	124.9	350	0.0020	48
103/2C	7.0	123.0	240	0.0010	24
107/3A	6.3	126.3	315	0.0013	50
107/3B	6.3	125.0	213	0.0001	29
107/3C	6.3	128.8	555	0.0022	68
108/9A	11.0	119.8	511	0.0085	34
108/9B	11.0	114.3	295	0.0036	14
108/9C	11.0	110.4	156	0.0010	7
109/7A	3.8	126.2	686	0	76
109/7B	3.8	125.3	405	0	75
109/7C	3.8	118.7	244	0	73

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CORROSIVITY TESTS

Boring No./ Sample No.	Soluble Chloride Content (ppm)	Soluble Sulphate Content (ppm)	Resistivity (ohm-cm)	pH
CS1/1		70.9	11,960	7.0
CS2/1		88.5	11,170	7.0
CS3/1		106.1	>16,750	6.9
CS4/1		95	16,750	6.8
CS5/1		123	16,750	7.5
CS6/1		82.7	6,980	7.9
CS7/1		126	16,750	7.4
CS8/1		65	11,170	7.4
CS9/1		82	16,750	6.9
CS10/1		96	10,470	7.2
CS11/1		184	16,750	7.1
CS12/1		80	16,750	7.0
CS13/1		101	16,750	7.1
CS14/1		80	16,750	7.0
CS15/1		117	16,750	7.0
CS16/1		149	16750	7.2
CS17/1		138	16,750	7.0
CS18/1		141	>16,750	7.1
CS19/1		139	>16,750	7.0
CS20/1		94	16,750	6.9
CS21/1		147	11,170	7.2
CS22/1	43	99	6,980	6.8
CS23/1		227	>16,750	7.0
CS24/1		111	>16,750	6.9
101/2	31	84	>16,750	7.7
103/2	96	67	1,190	7.2
108/9	22	138	1,270	7.6
109/12		87	16,750	7.4
109/15		72	12,890	7.4

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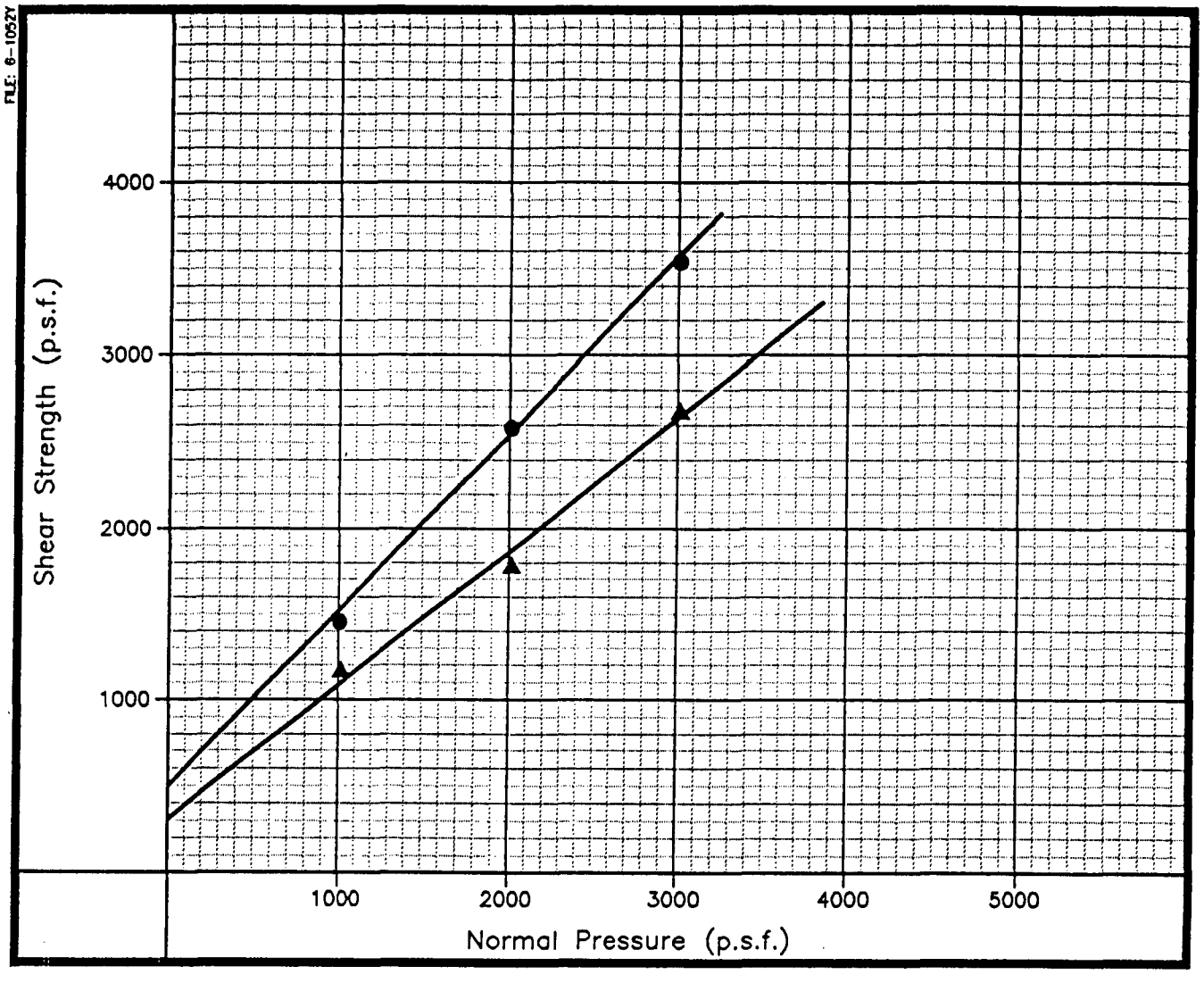
RESULTS OF DIRECT SHEAR TESTS



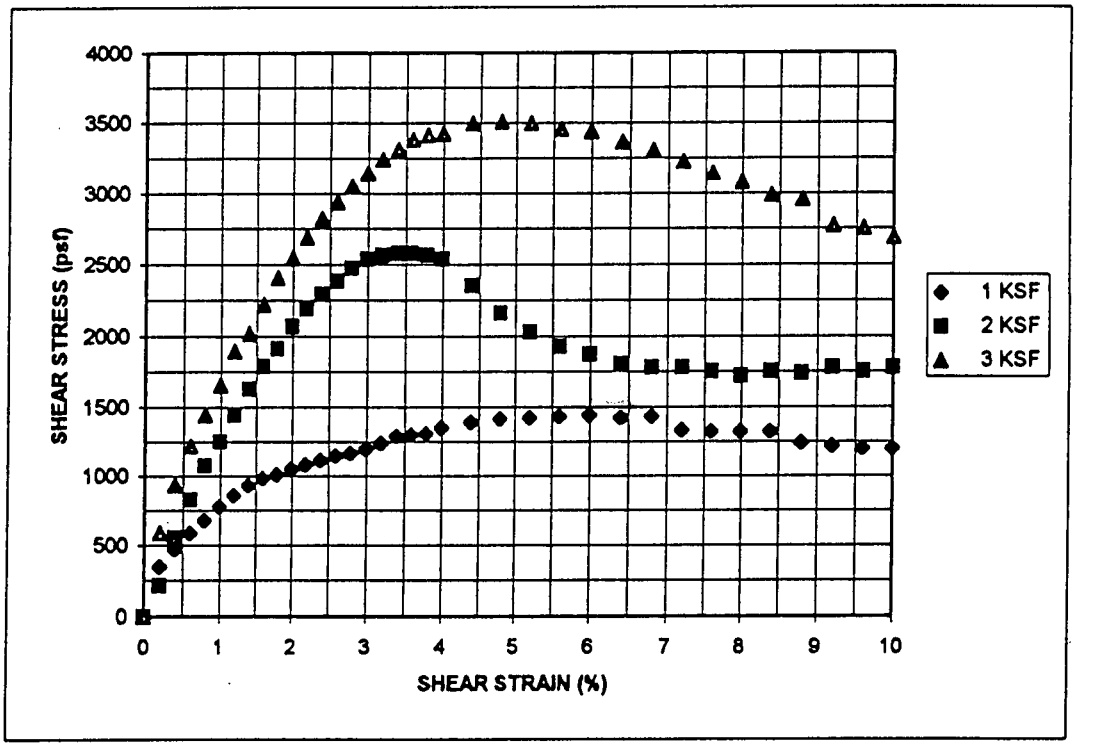
○ peak shear strength △ strength at 1/4 inch displacement

Open symbols (△) signify tests at natural moisture;
Solid symbols (▲) signify tests on submerged samples.

BORING/SAMPLE	MATERIAL	PEAK	ULTIMATE
101/8	FANGLOMERATE: fine to coarse SAND (SUBMERGED)	c = 500 psf ø = 45.5°	c = 320 psf ø = 37.5°

ROUTE 138 - PHASE 2

BORING : 101
SAMPLE : 8 (SUBMERGED)



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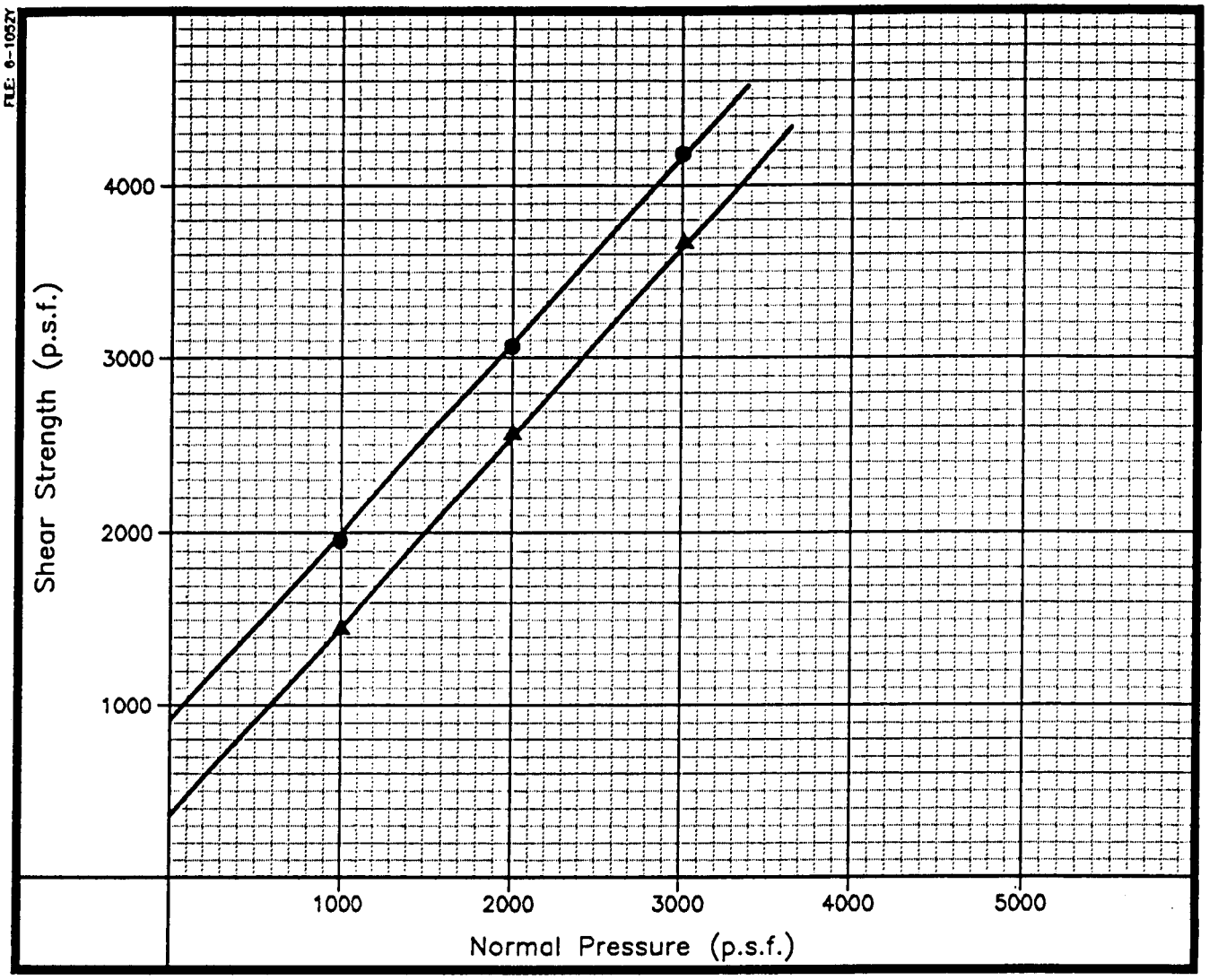
RESULTS OF DIRECT SHEAR TESTS



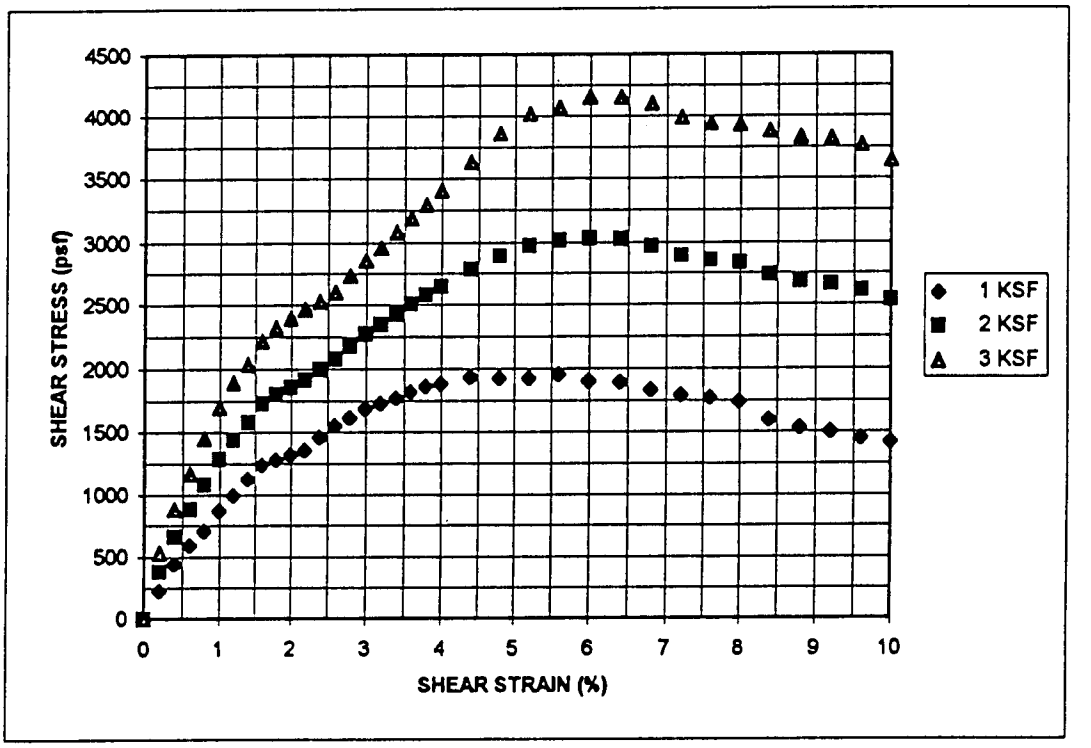
○ peak shear strength △ strength at 1/4 inch displacement

Open symbols (△) signify tests at natural moisture;
Solid symbols (▲) signify tests on submerged samples.

BORING/SAMPLE	MATERIAL	PEAK	ULTIMATE
101/10	FANGLOMERATE: fine to coarse SAND (SUBMERGED)	C = 900 psf ø = 47°	c = 370 psf ø = 47°

ROUTE 138 - PHASE 2

BORING : 101
SAMPLE : 10 (SUBMERGED)



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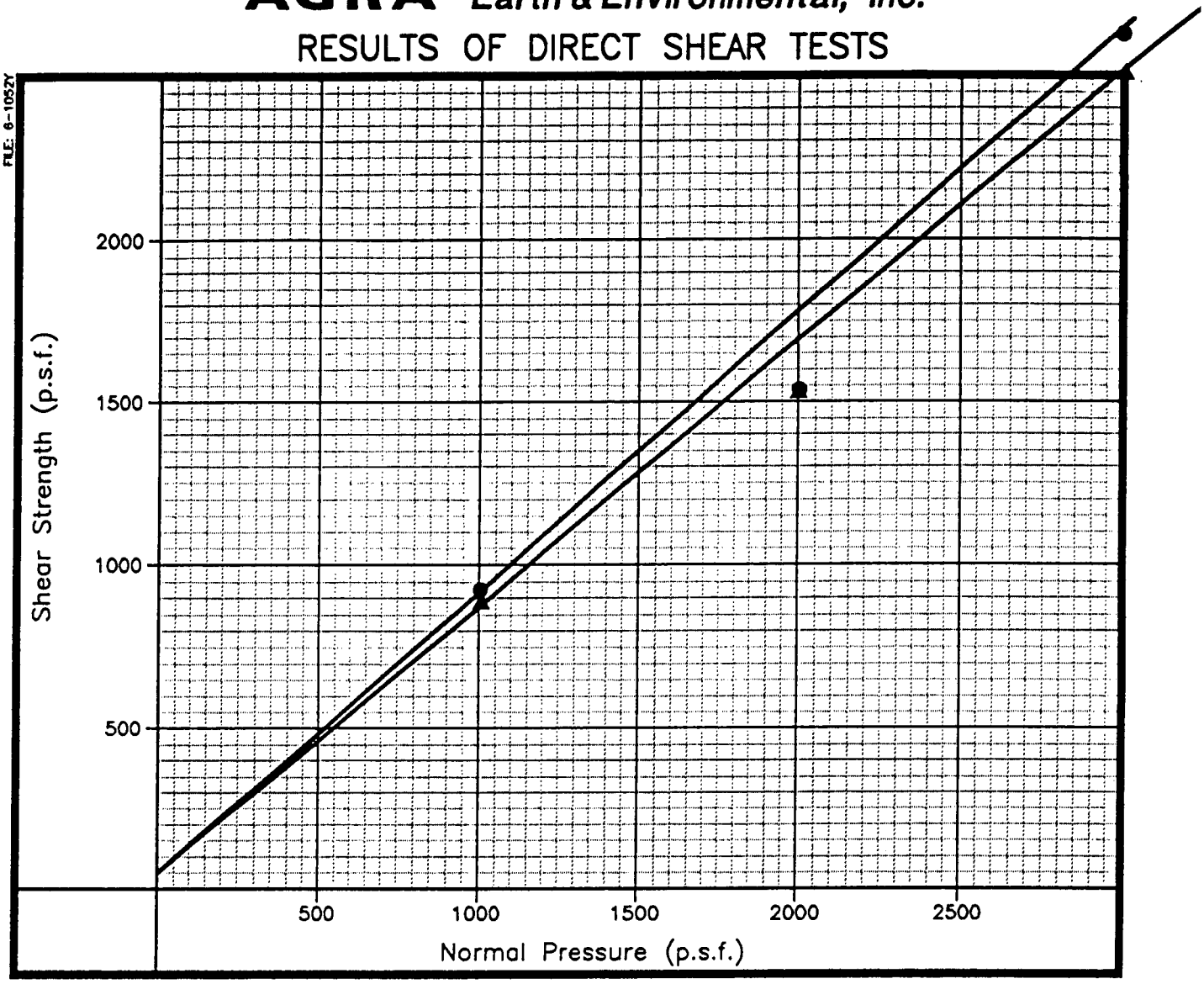
RESULTS OF DIRECT SHEAR TESTS



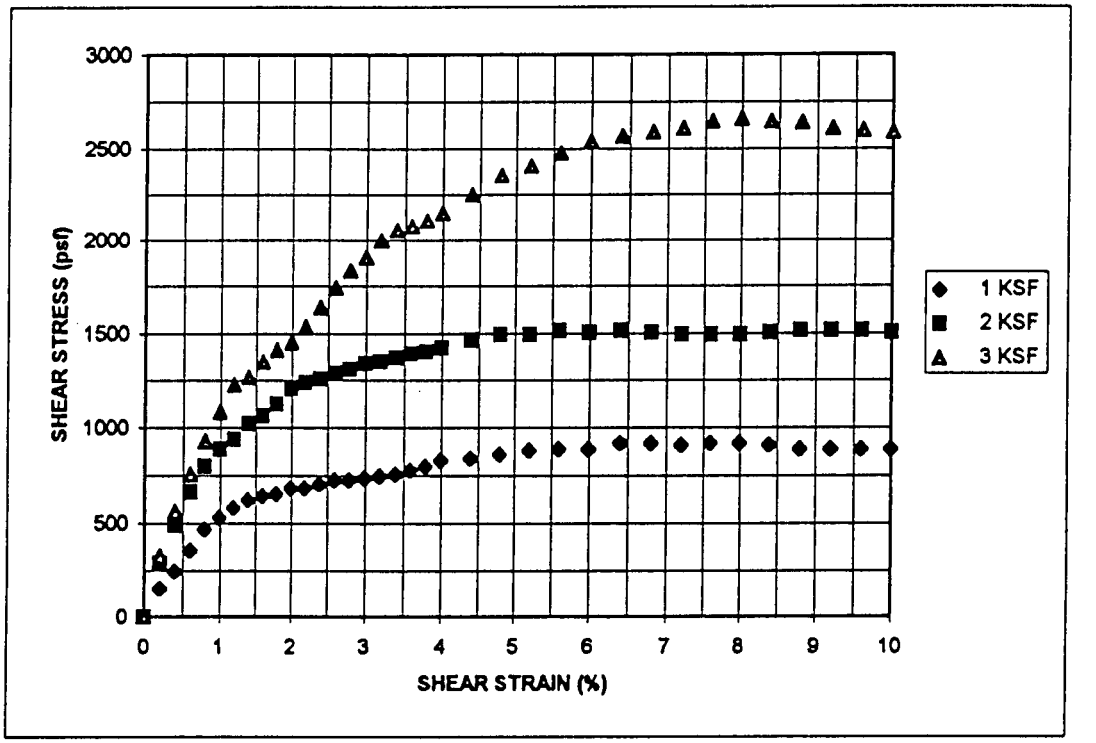
O peak shear strength Δ strength at 1/4 inch displacement

Open symbols (Δ) signify tests at natural moisture;
Solid symbols (▲) signify tests on submerged samples.

BORING/SAMPLE	*MATERIAL*	*PEAK*	*ULTIMATE*
TP 2/2	FANGLOMERATE: fine to medium SILTY SAND (SUBMERGED)	c = 50 psf ø = 41°	c = 50 psf ø = 40°

ROUTE 138 - PHASE 2

BORING : TP2
SAMPLE : 2 (SUBMERGED)



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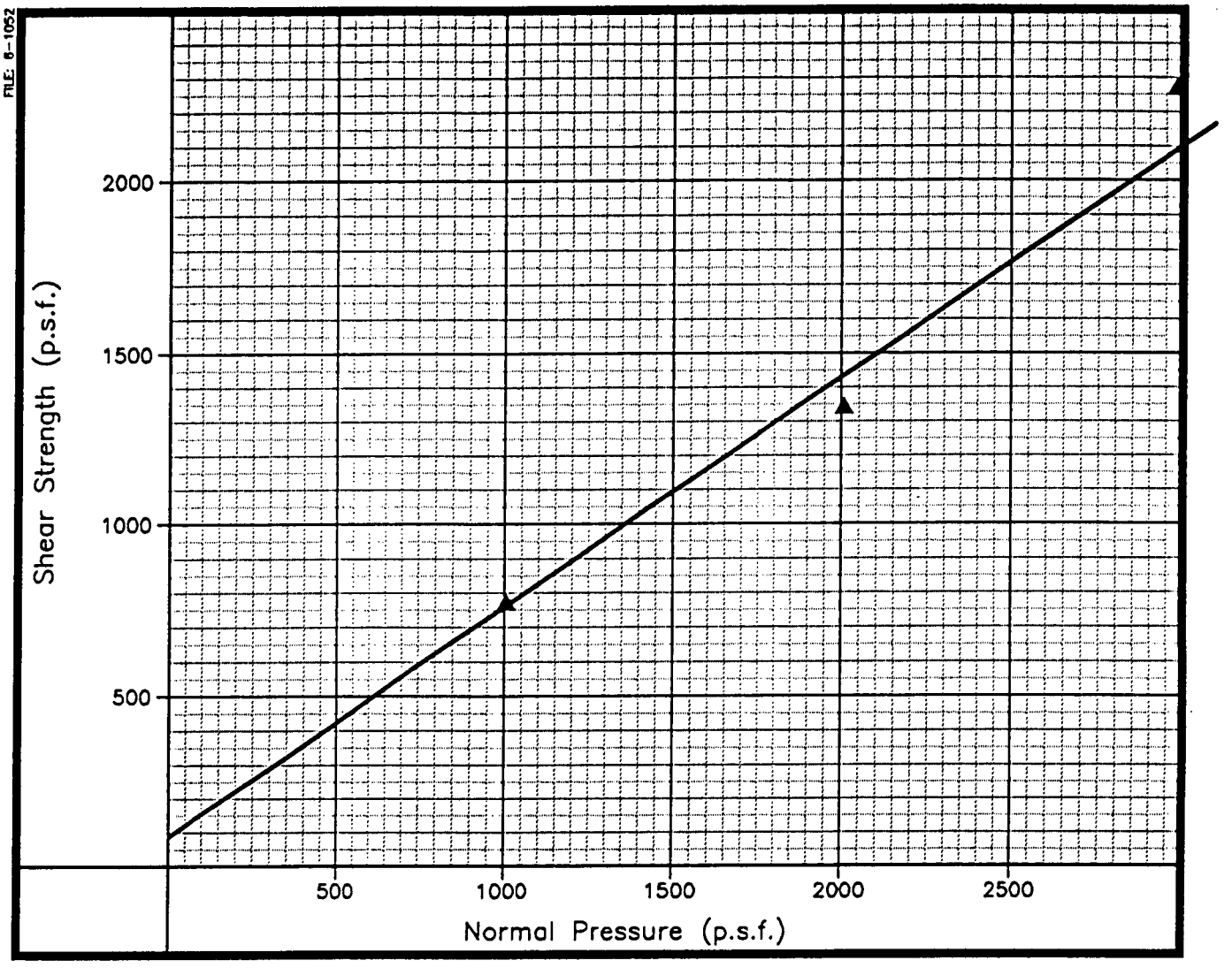
RESULTS OF DIRECT SHEAR TESTS



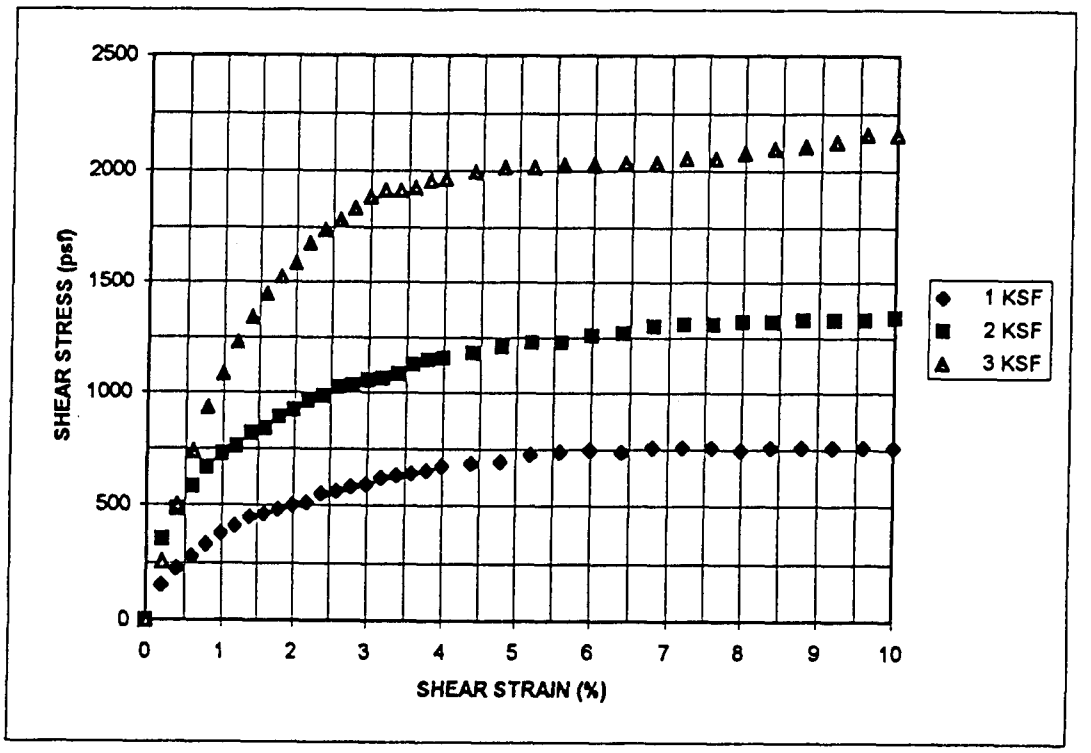
O peak shear strength Δ strength at 1/4 inch displacement

Open symbols (Δ) signify tests at natural moisture;
Solid symbols (▲) signify tests on submerged samples.

BORING/SAMPLE	MATERIAL	PEAK+ULTIMATE
103/1	CROWDER FORMATON: fine to medium CLAYEY SANDSTONE (SUBMERGED)	c = 100 psf ø = 34°

ROUTE 138 - PHASE 2

BORING : 103
SAMPLE : 1 (SUBMERGED)



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JOB # : 6-212-1052

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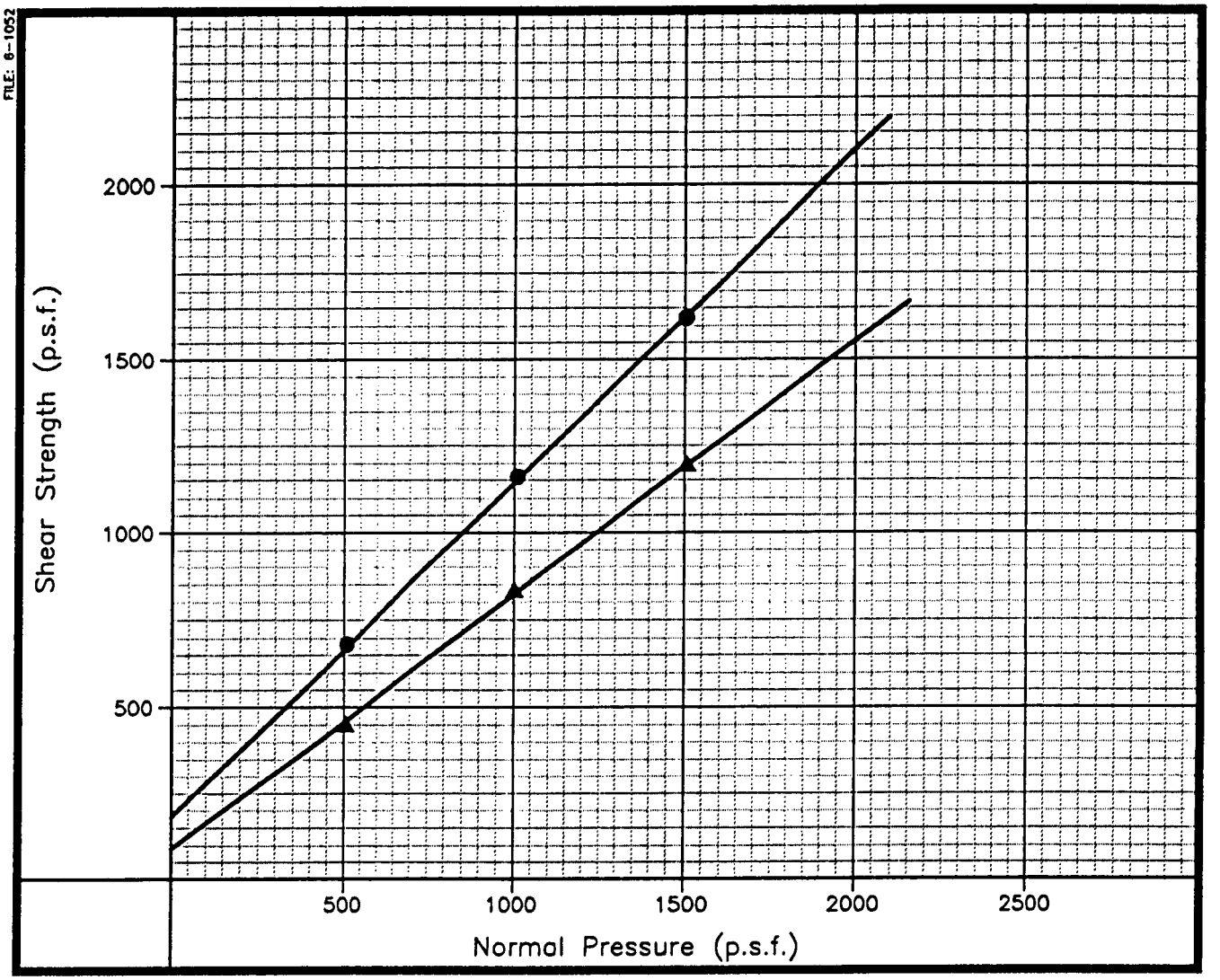
RESULTS OF DIRECT SHEAR TESTS



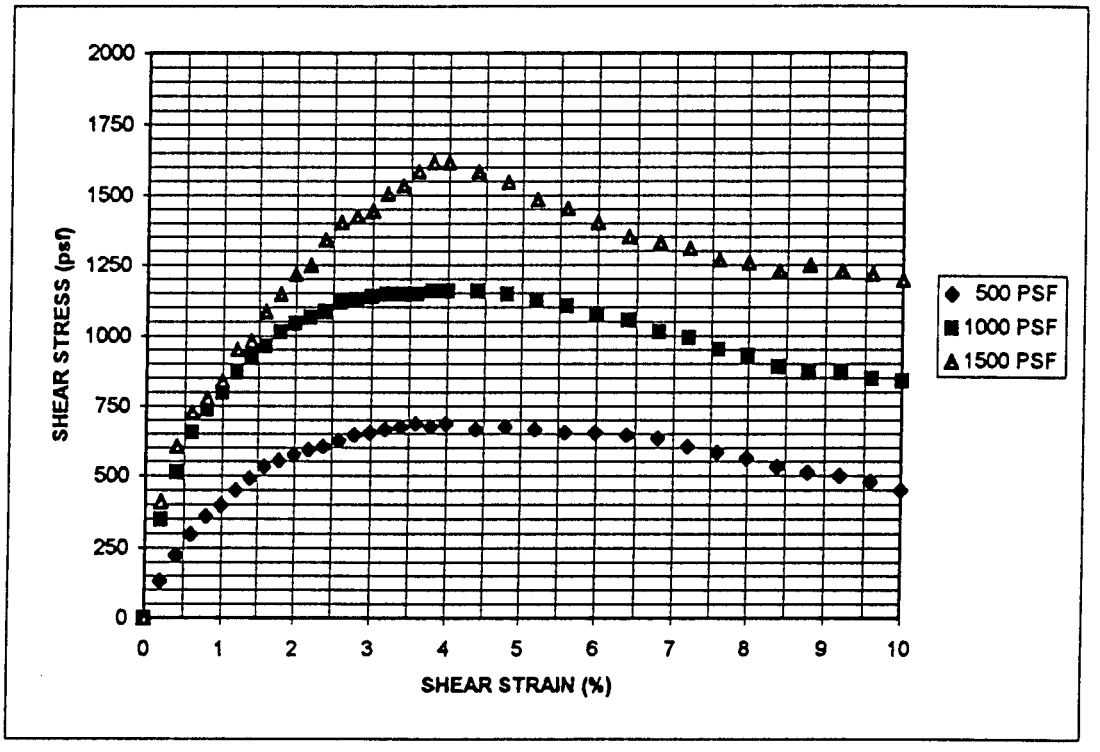
○ peak shear strength △ strength at 1/4 inch displacement

Open symbols (△) signify tests at natural moisture;
Solid symbols (▲) signify tests on submerged samples.

BORING/SAMPLE	MATERIAL	PEAK	ULTIMATE
107/6	CROWDER FORMATION: fine to coarse SANDSTONE (SUBMERGED)	c = 200 psf ø = 44°	c = 100 psf ø = 36°

ROUTE 138 - PHASE 2

BORING : 107
SAMPLE : 6 (SUBMERGED)



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JOB # : 6-212-1052

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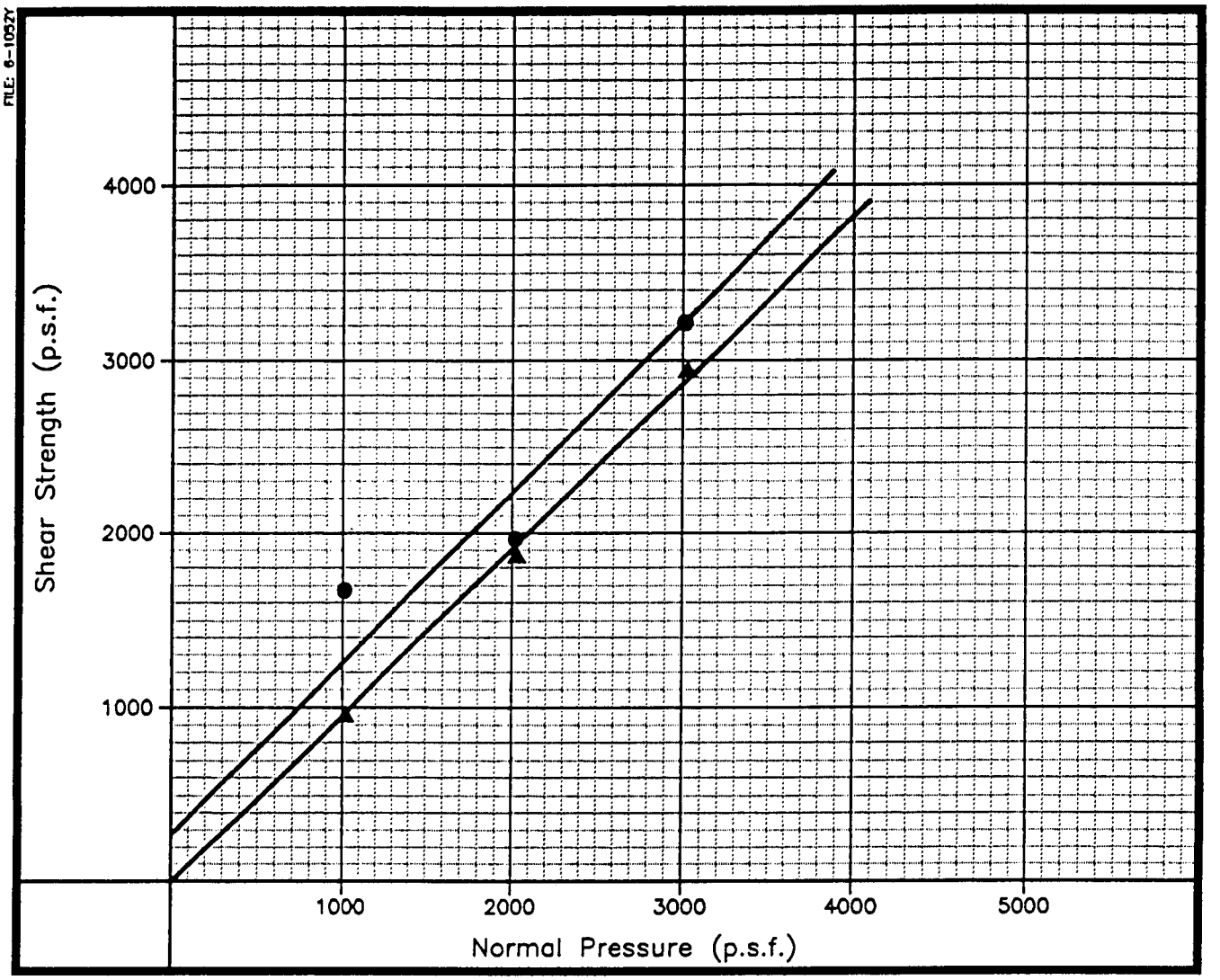
RESULTS OF DIRECT SHEAR TESTS



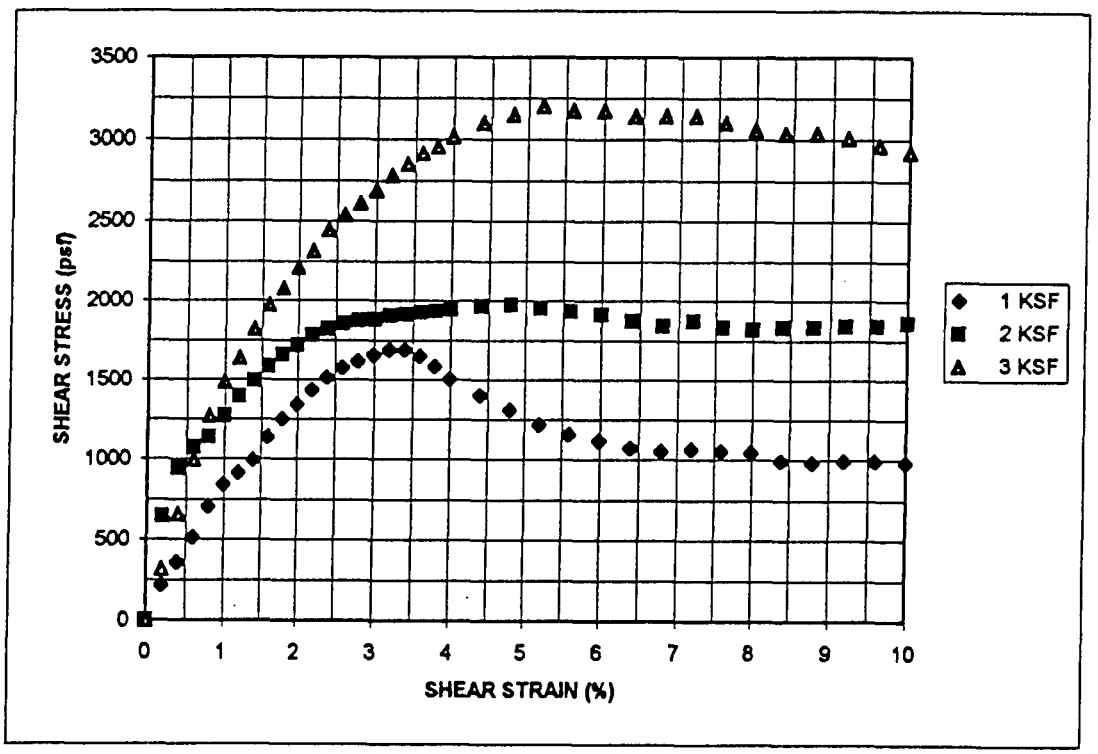
○ peak shear strength △ strength at 1/4 inch displacement

Open symbols (△) signify tests at natural moisture;
Solid symbols (▲) signify tests on submerged samples.

BORING/SAMPLE	MATERIAL	PEAK	ULTIMATE
107/10	CROWDER FORMATION: fine to coarse SILTY SANDSTONE (SUBMERGED)	c = 300 psf ø = 44°	c = 0 psf ø = 44°

ROUTE 138 - PHASE 2

BORING : 107
SAMPLE : 10 (SUBMERGED)



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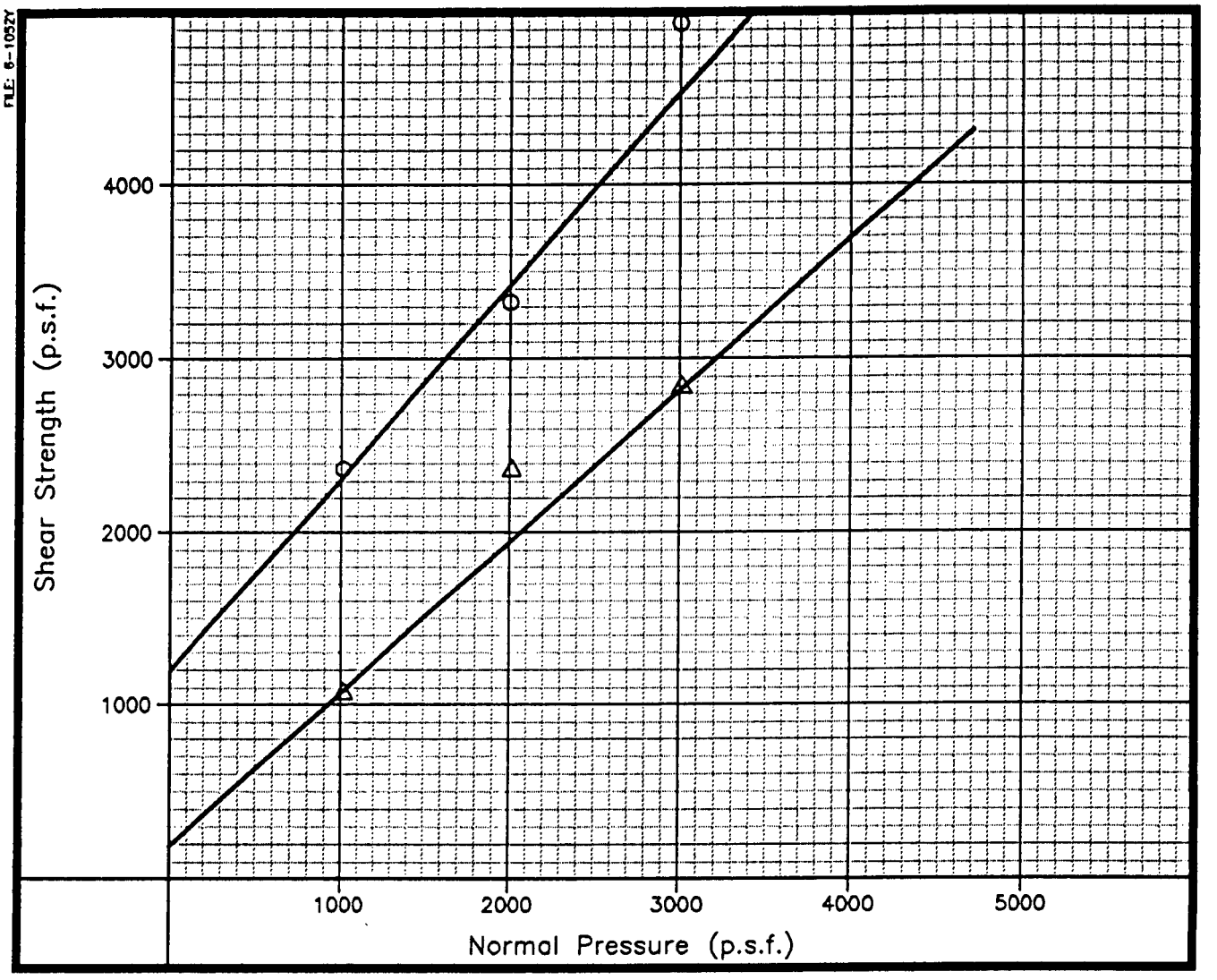
JOB # : 6-212-1052

AGRA Earth & Environmental, Inc.

RESULTS OF DIRECT SHEAR TESTS



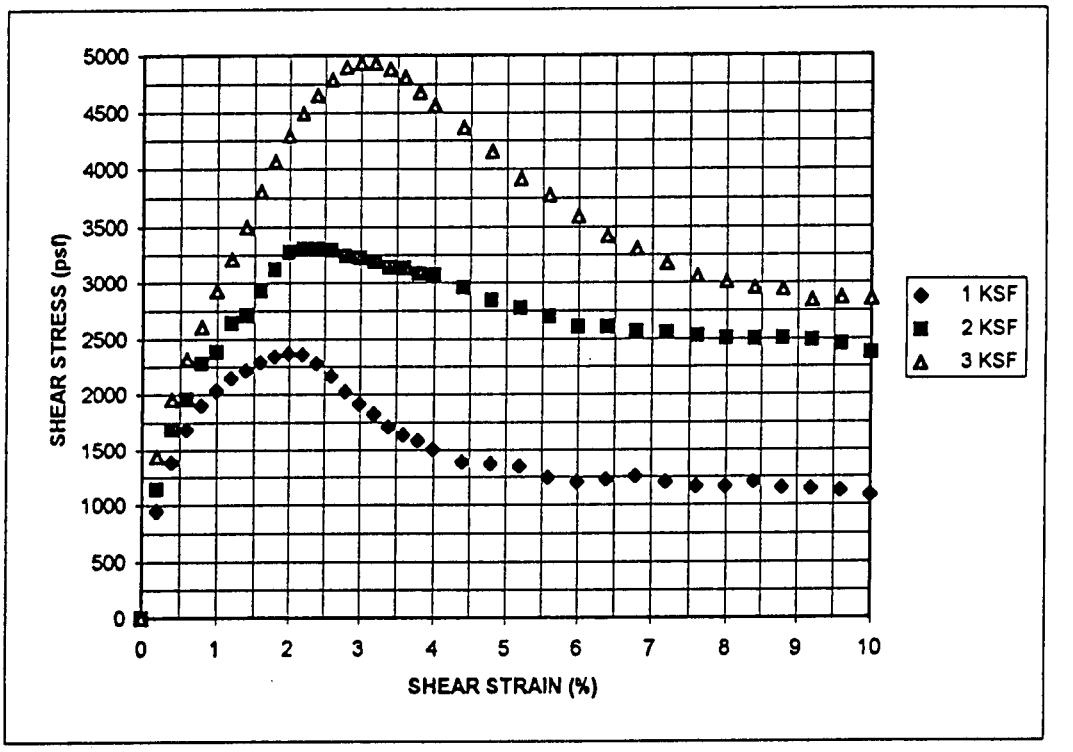
○ peak shear strength △ strength at 1/4 inch displacement

Open symbols (△) signify tests at natural moisture;
Solid symbols (▲) signify tests on submerged samples.

BORING/SAMPLE	MATERIAL	PEAK	ULTIMATE
107/10	CROWDER FORMATION: fine to coarse SILTY SANDSTONE (NATURAL MOISTURE)	c = 1200 psf ø = 48°	c = 200 psf ø = 41°

ROUTE 138 - PHASE 2

BORING : 107
SAMPLE : 10 (NATURAL)

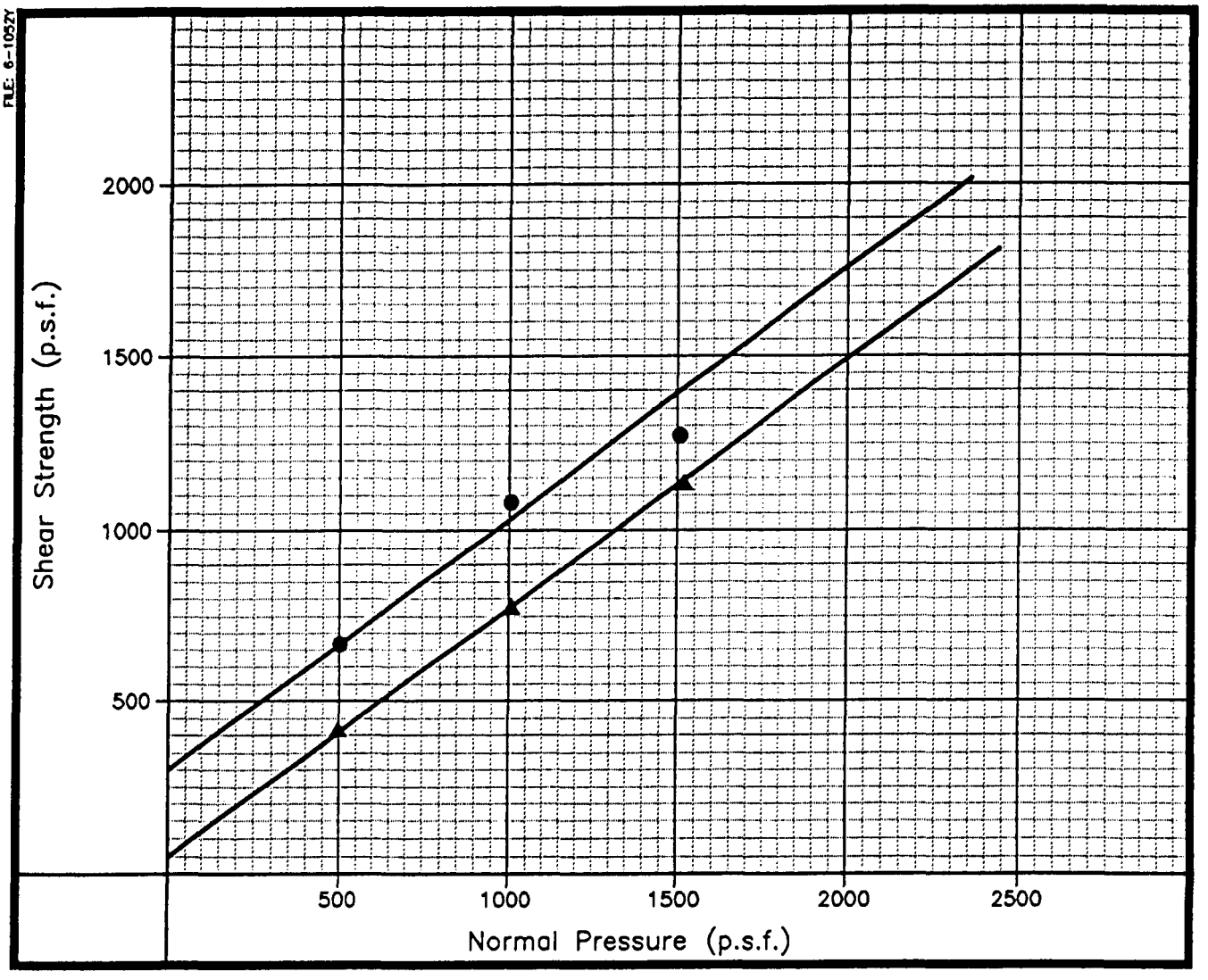


AGRA EARTH & ENVIRONMENTAL, INC.

JOB # : 6-212-1052

AGRA Earth & Environmental, Inc.

RESULTS OF DIRECT SHEAR TESTS



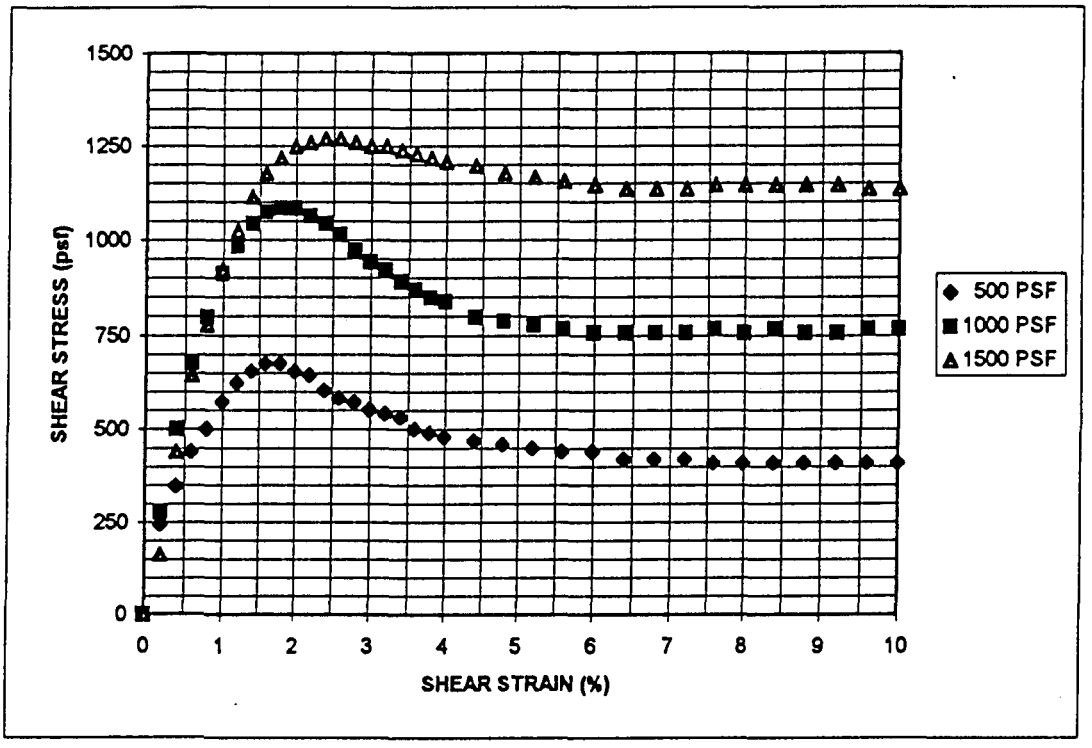
○ peak shear strength △ strength at 1/4 inch displacement

Open symbols (△) signify tests at natural moisture;
Solid symbols (▲) signify tests on submerged samples.

BORING/SAMPLE	MATERIAL	PEAK	ULTIMATE
108/2	fine to medium SILTY SANDSTONE (REMOLDED SUBMERGED)	c = 300 psf ø = 36°	c = 50 psf ø = 36°

ROUTE 138 - PHASE 2

BORING : 108
SAMPLE : 2 (REMOLDED & SUBMERGED)

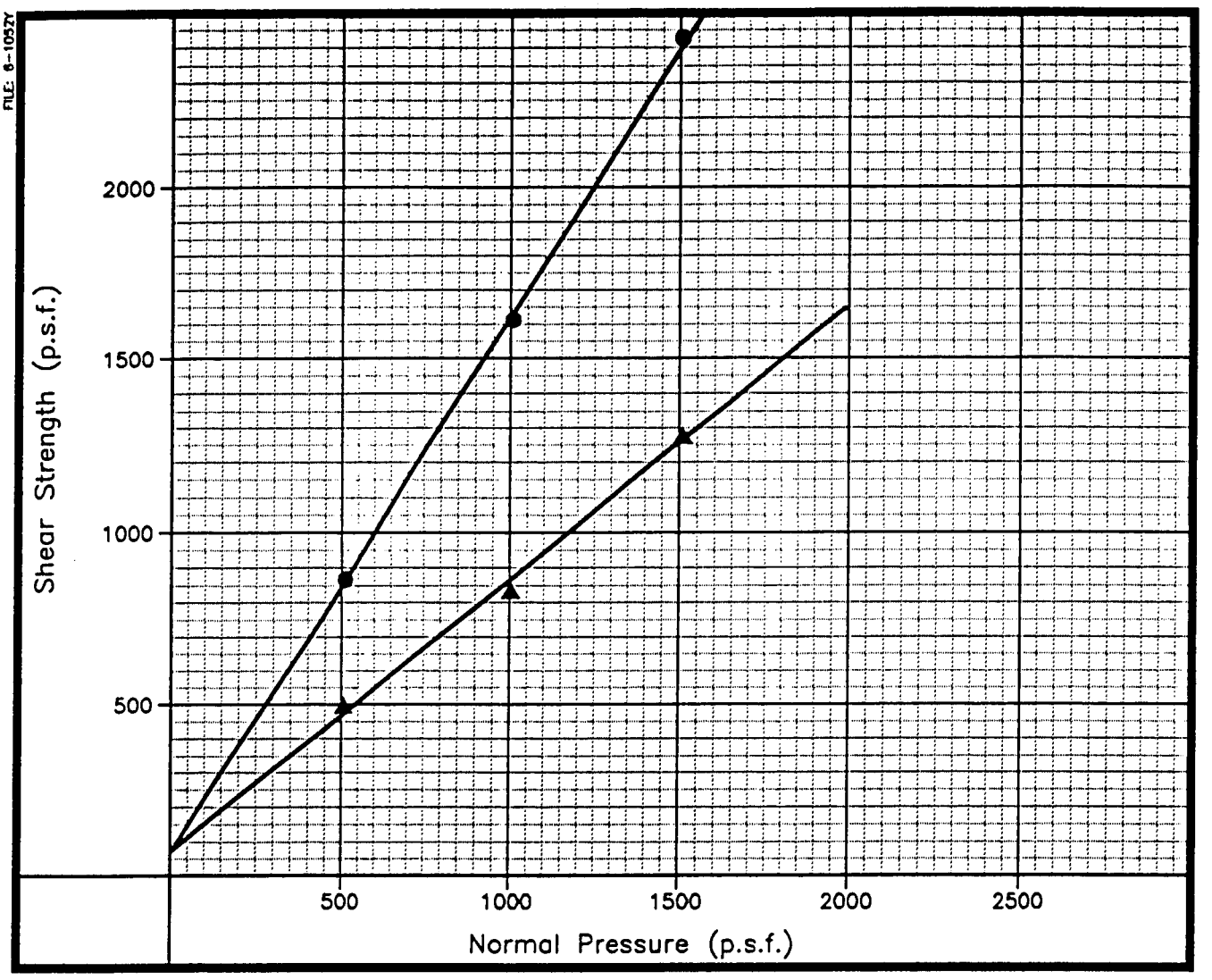


AGRA EARTH & ENVIRONMENTAL, INC.

JOB # : 6-212-1052

AGRA Earth & Environmental, Inc.

RESULTS OF DIRECT SHEAR TESTS



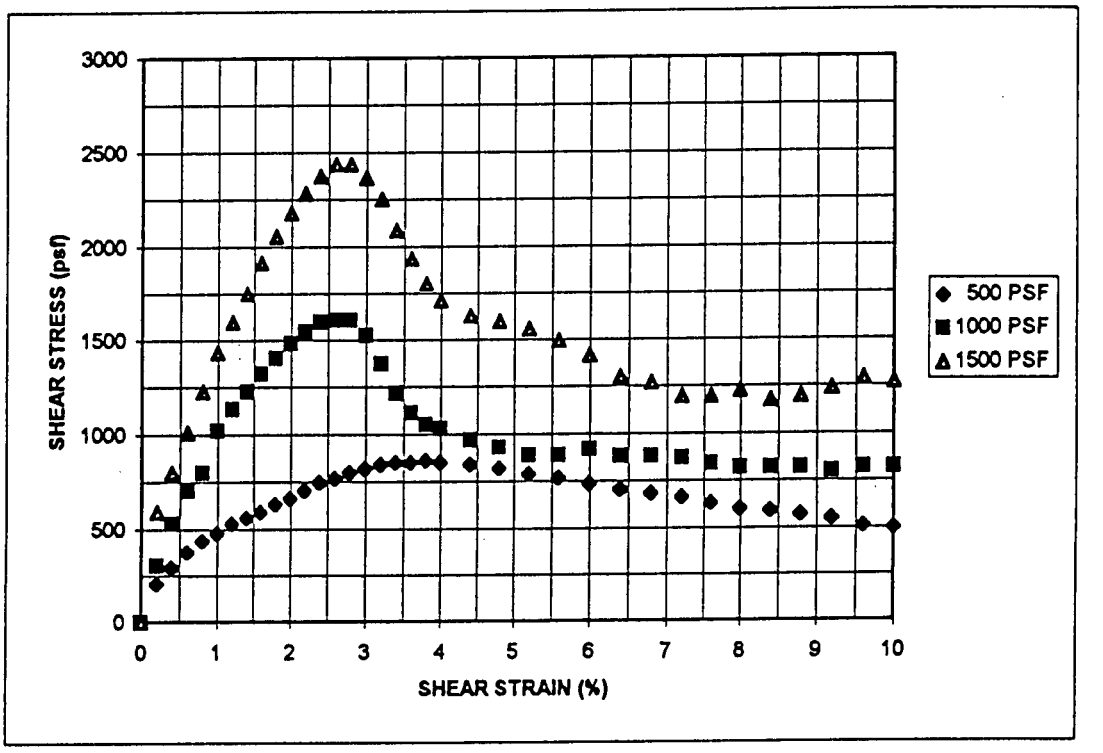
○ peak shear strength △ strength at 1/4 inch displacement

Open symbols (△) signify tests at natural moisture;
Solid symbols (▲) signify tests on submerged samples.

BORING/SAMPLE	MATERIAL	PEAK	ULTIMATE
108/4	CROWDER FORMATION: fine to coarse SANDSTONE (SUBMERGED)	c = 75 psf ø = 57°	c = 75 psf ø = 39°

ROUTE 138 - PHASE 2

BORING : 108
SAMPLE : 4 (SUBMERGED)

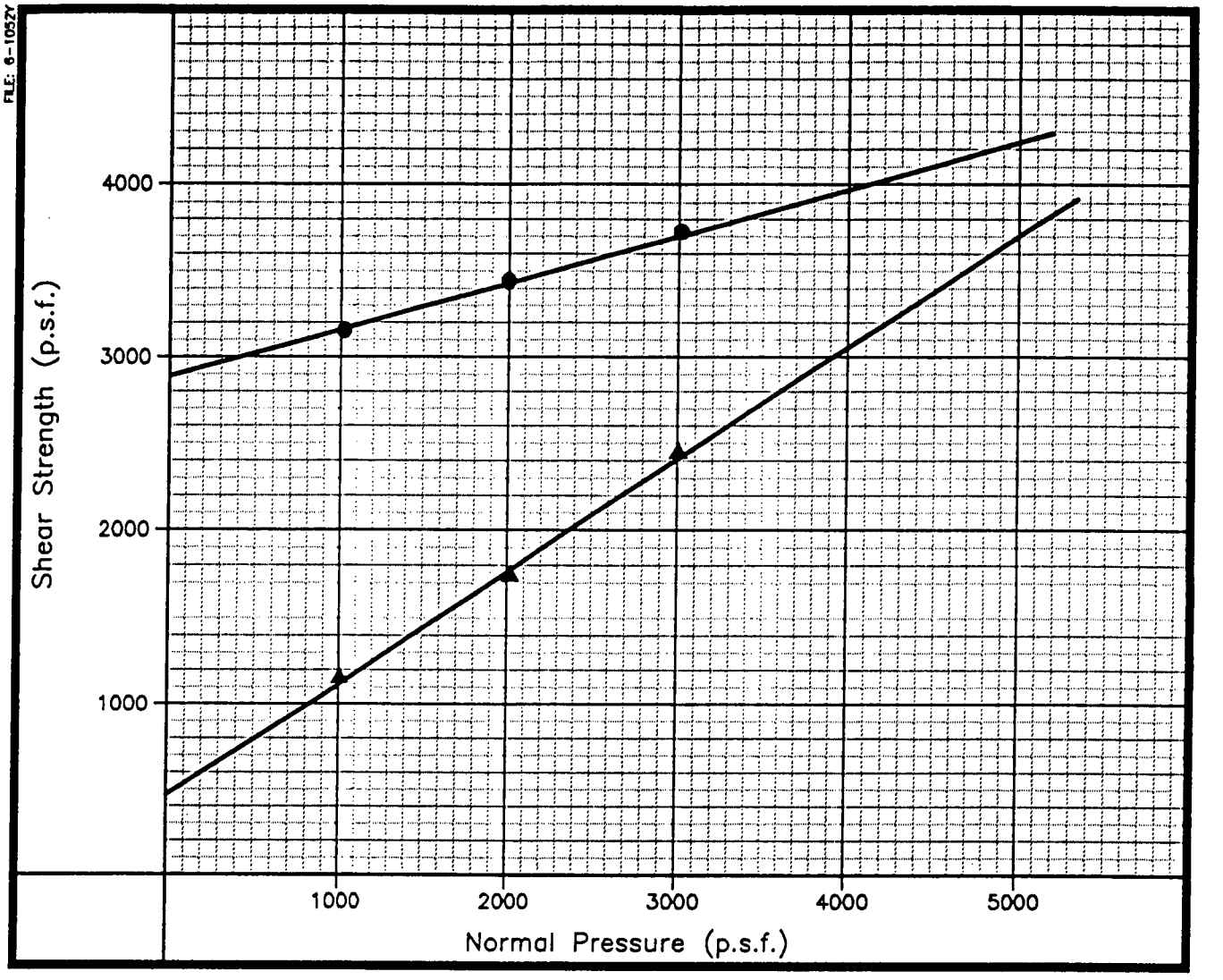


AGRA EARTH & ENVIRONMENTAL, INC.

JOB # : 6-212-1052

AGRA Earth & Environmental, Inc.

RESULTS OF DIRECT SHEAR TESTS



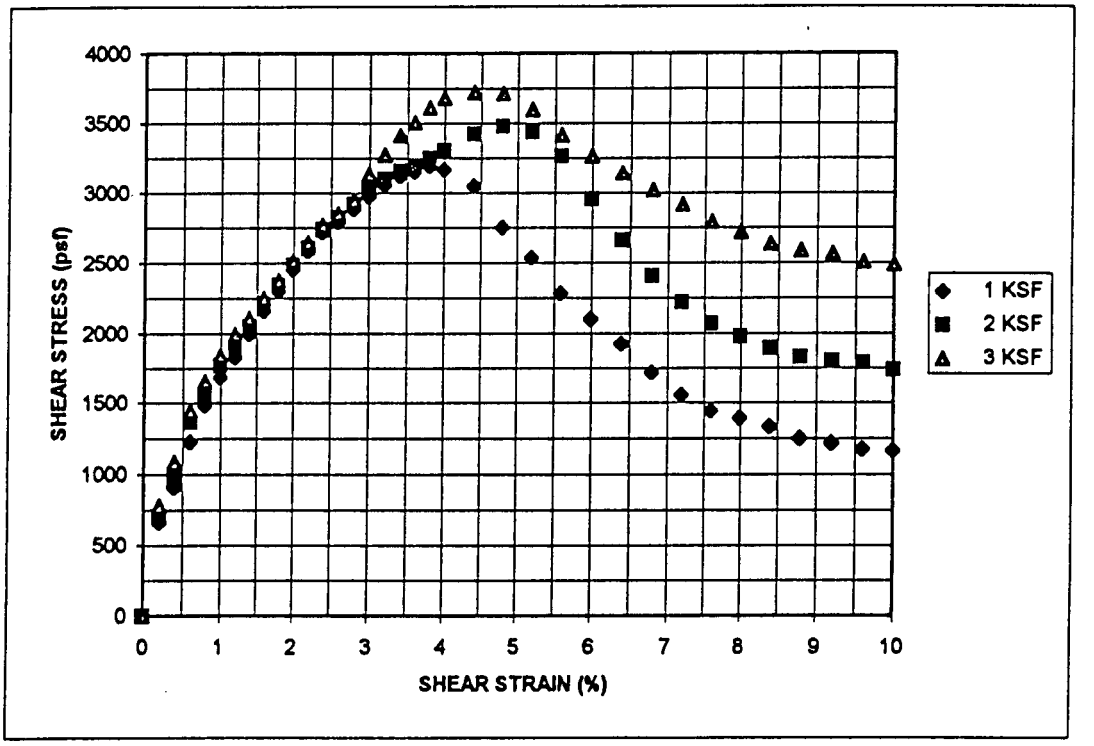
○ peak shear strength △ strength at 1/4 inch displacement

Open symbols (△) signify tests at natural moisture;
Solid symbols (▲) signify tests on submerged samples.

BORING/SAMPLE	MATERIAL	PEAK	ULTIMATE
108/6	CROWDER FORMATION: SANDSTONE (SUBMERGED)	c = 2900 psf ø = 15°	c = 500 psf ø = 33°

ROUTE 138 - PHASE 2

BORING : 108
SAMPLE : 6 (SUBMERGED)



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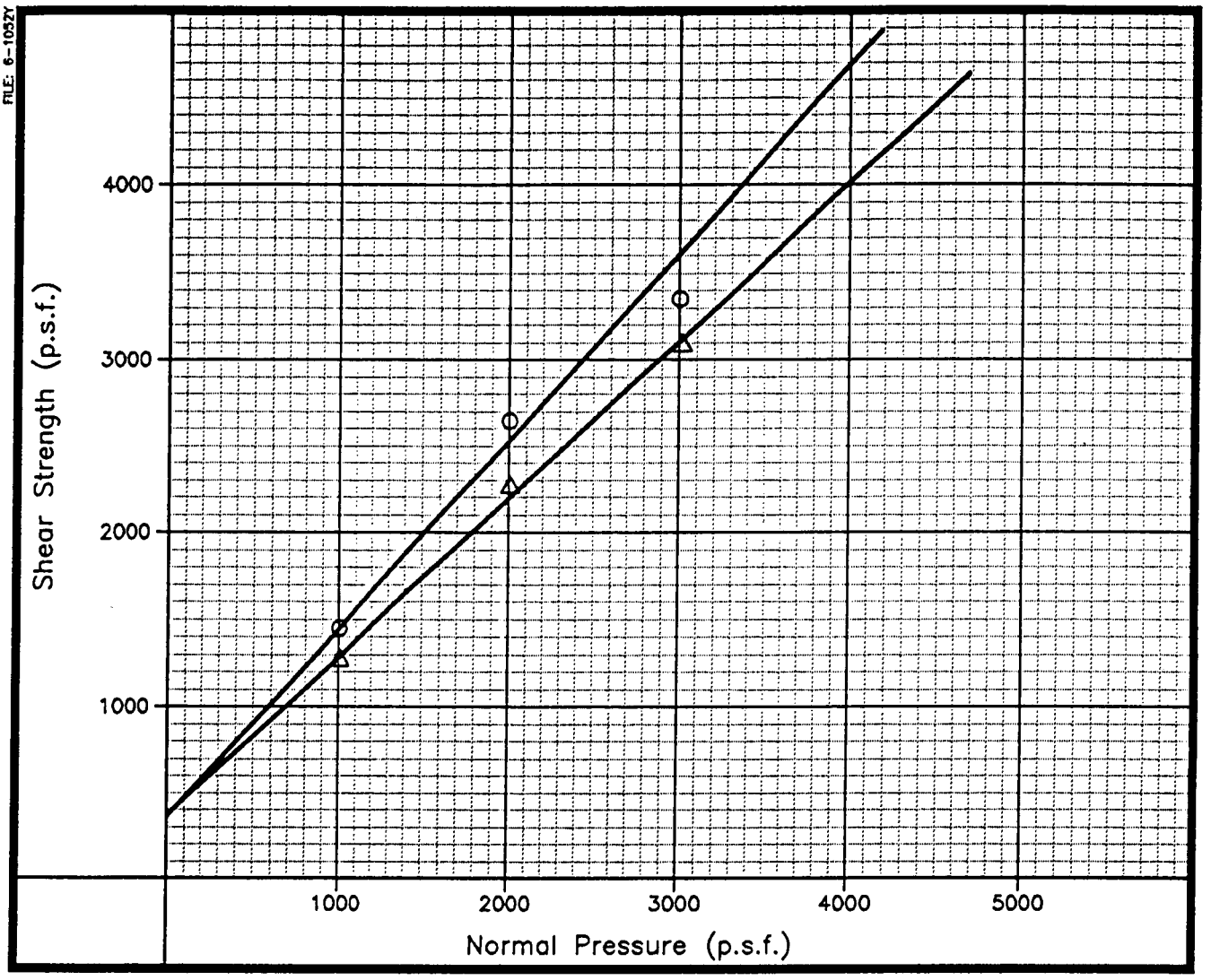
JOB # : 6-212-1052

AGRA Earth & Environmental, Inc.

RESULTS OF DIRECT SHEAR TESTS



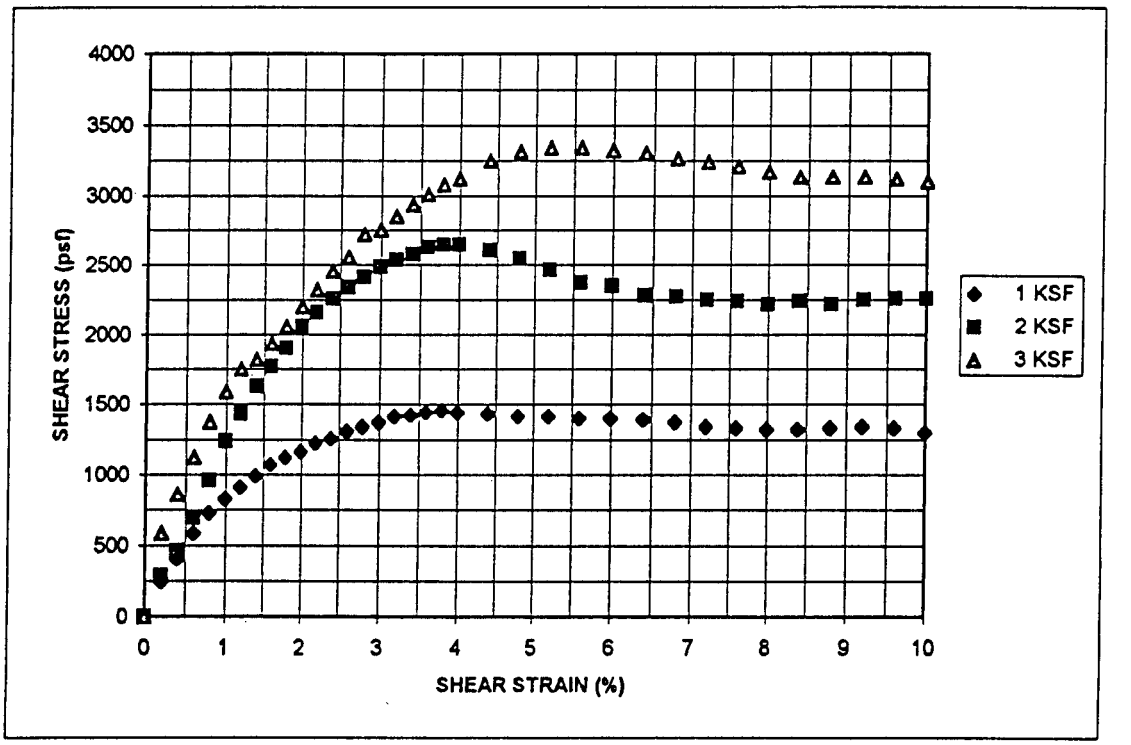
○ peak shear strength △ strength at 1/4 inch displacement

Open symbols (△) signify tests at natural moisture;
Solid symbols (▲) signify tests on submerged samples.

BORING/SAMPLE	MATERIAL	PEAK	ULTIMATE
108/15	CROWDER FORMATION: SILTY SANDSTONE (NATURAL MOISTURE)	c = 400 psf ø = 47°	c = 400 psf ø = 42°

ROUTE 138 - PHASE 2

BORING : 108
SAMPLE : 15 (NATURAL)

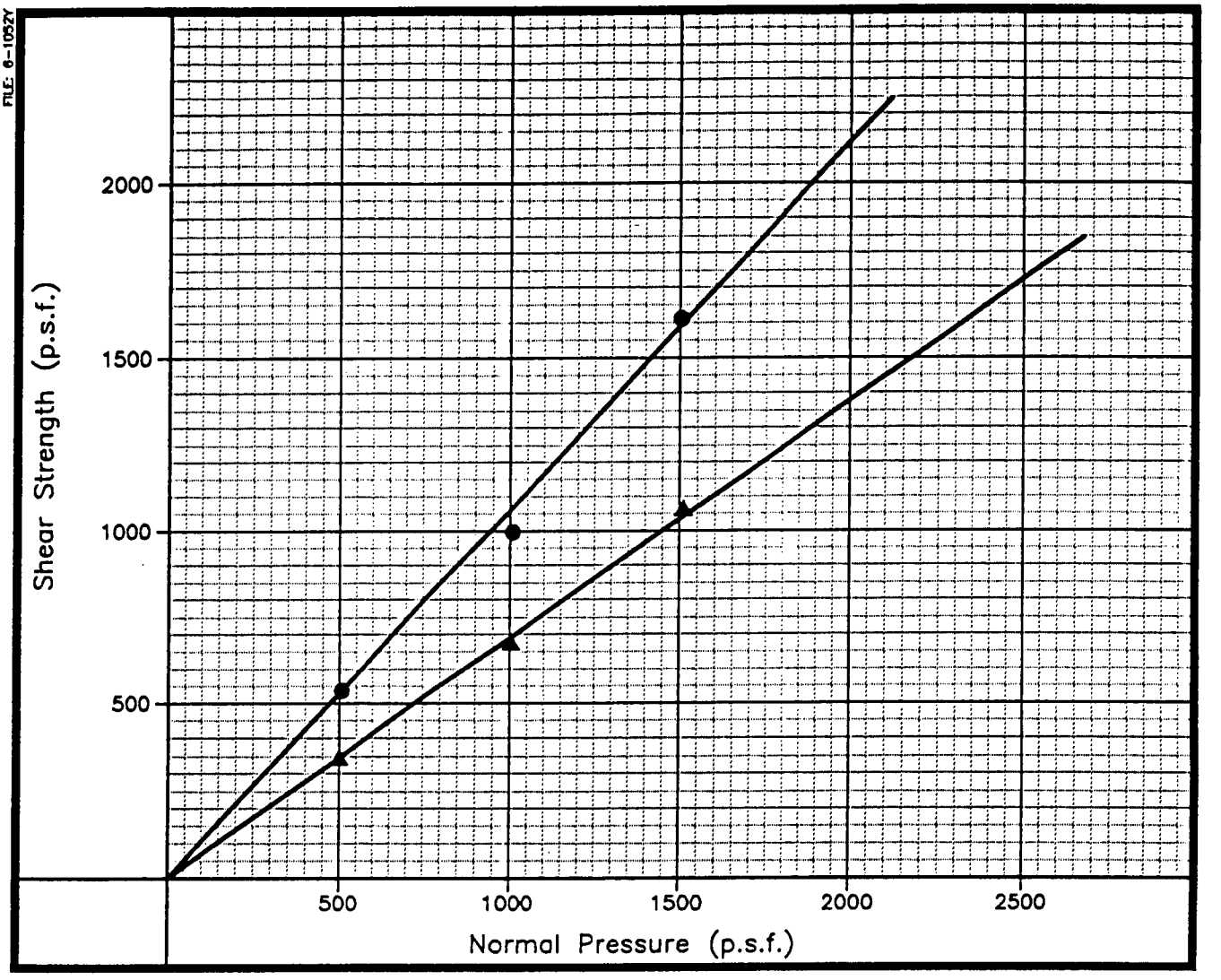


AGRA EARTH & ENVIRONMENTAL, INC.

JOB # : 6-212-1052

AGRA Earth & Environmental, Inc.

RESULTS OF DIRECT SHEAR TESTS



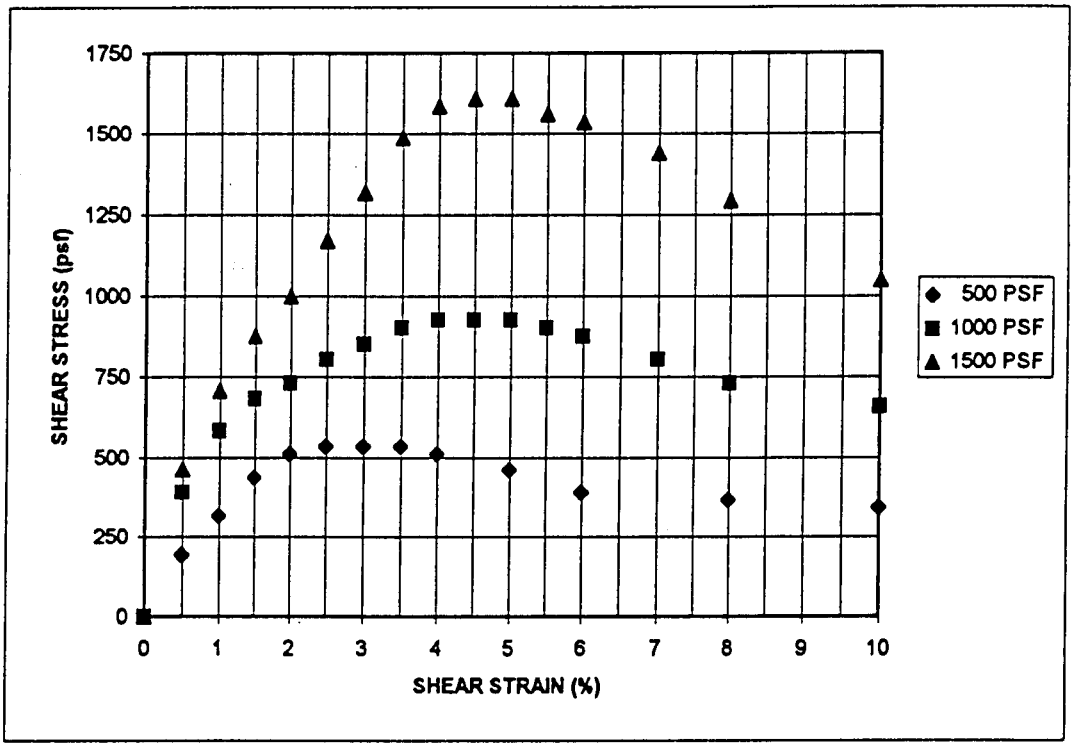
O peak shear strength △ strength at 1/4 inch displacement

Open symbols (△) signify tests at natural moisture;
Solid symbols (▲) signify tests on submerged samples.

BORING/SAMPLE	MATERIAL	PEAK	ULTIMATE
109/1	TERRACE DEPOSITS: fine to coarse SILTY SAND (SUBMERGED)	c = 0 psf ø = 47°	c = 0 psf ø = 35°

ROUTE 138 - PHASE 2

BORING : 109
SAMPLE : 1 (SUBMERGED)



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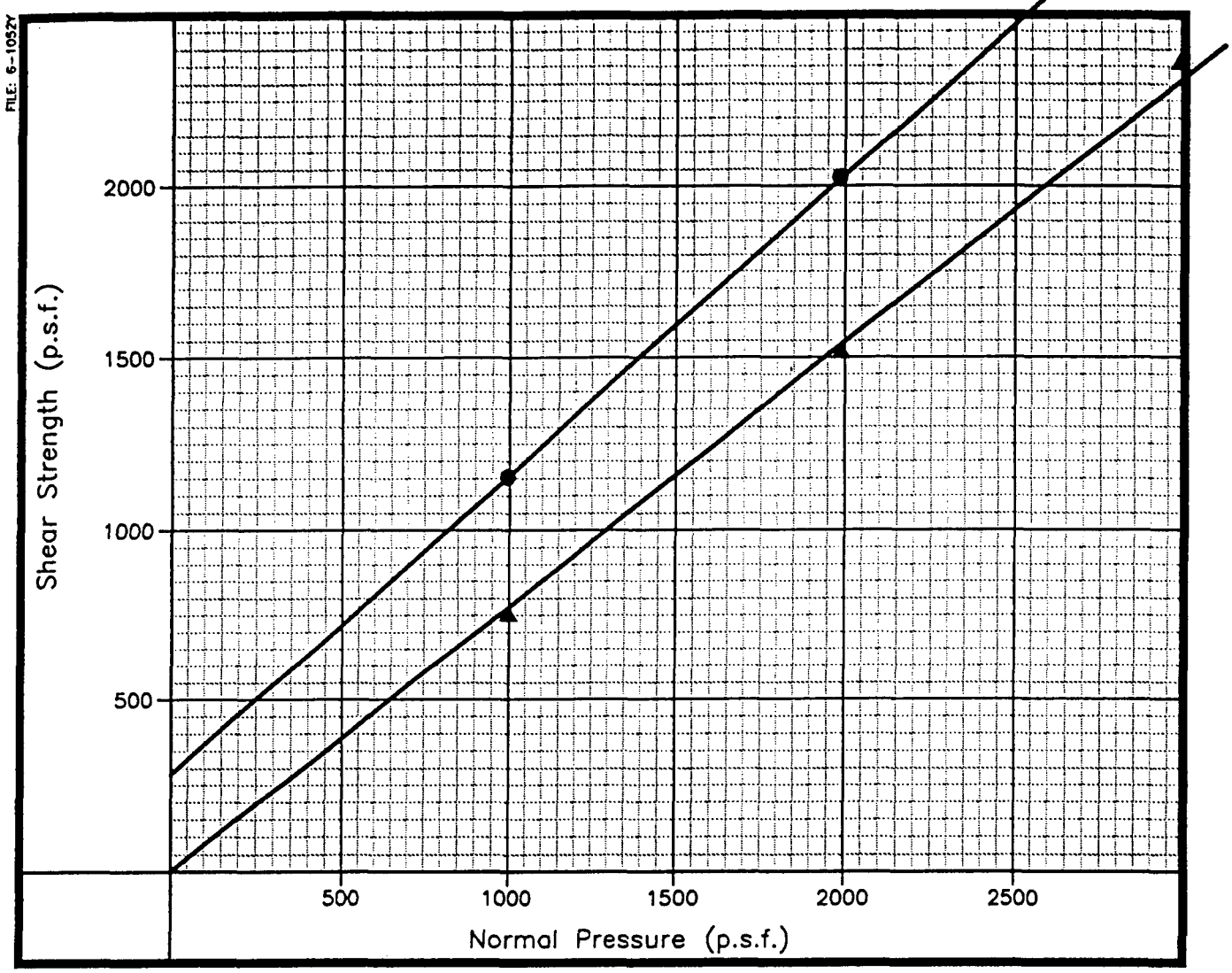
JOB # : 6-212-1052

AGRA *Earth & Environmental, Inc.*

RESULTS OF DIRECT SHEAR TESTS



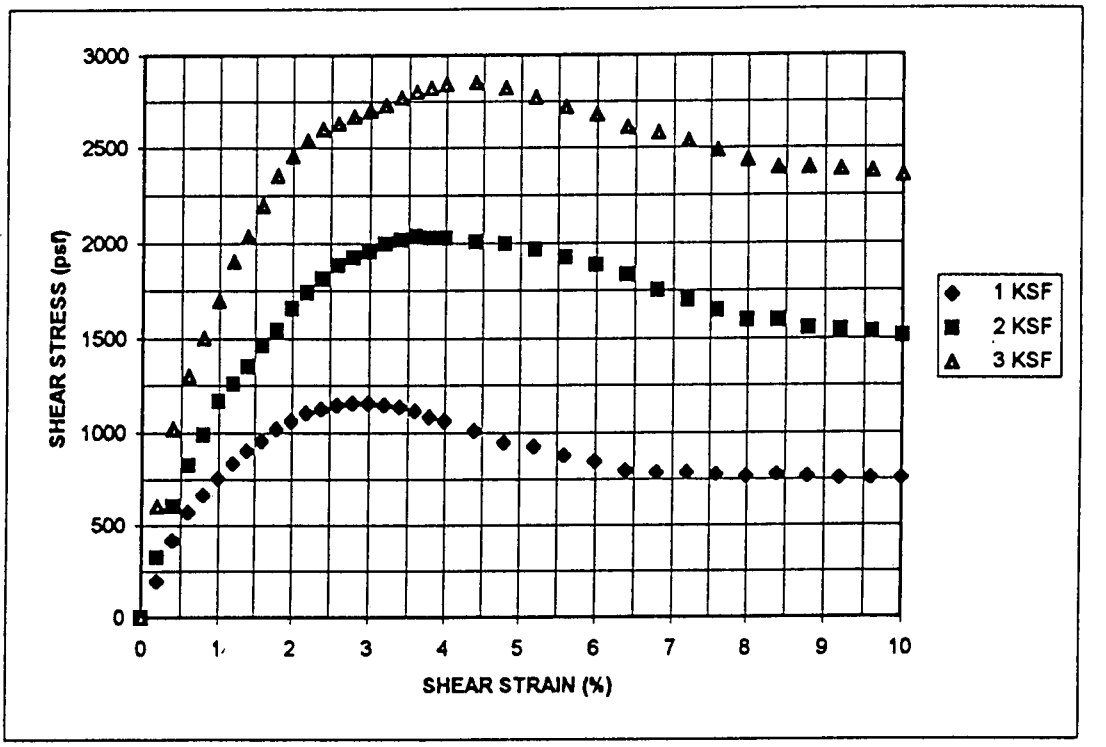
○ peak shear strength △ strength at 1/4 inch displacement

Open symbols (△) signify tests at natural moisture;
Solid symbols (▲) signify tests on submerged samples.

BORING/SAMPLE	*MATERIAL*	*PEAK*	*ULTIMATE*
109/2	fine to coarse SILTY SAND (REMOLDED SUBMERGED)	c = 300 psf ø = 41°	c = 0 psf ø = 38°

ROUTE 138 - PHASE 2

BORING : 109
SAMPLE : 2 (REMOLDED & SUBMERGED)

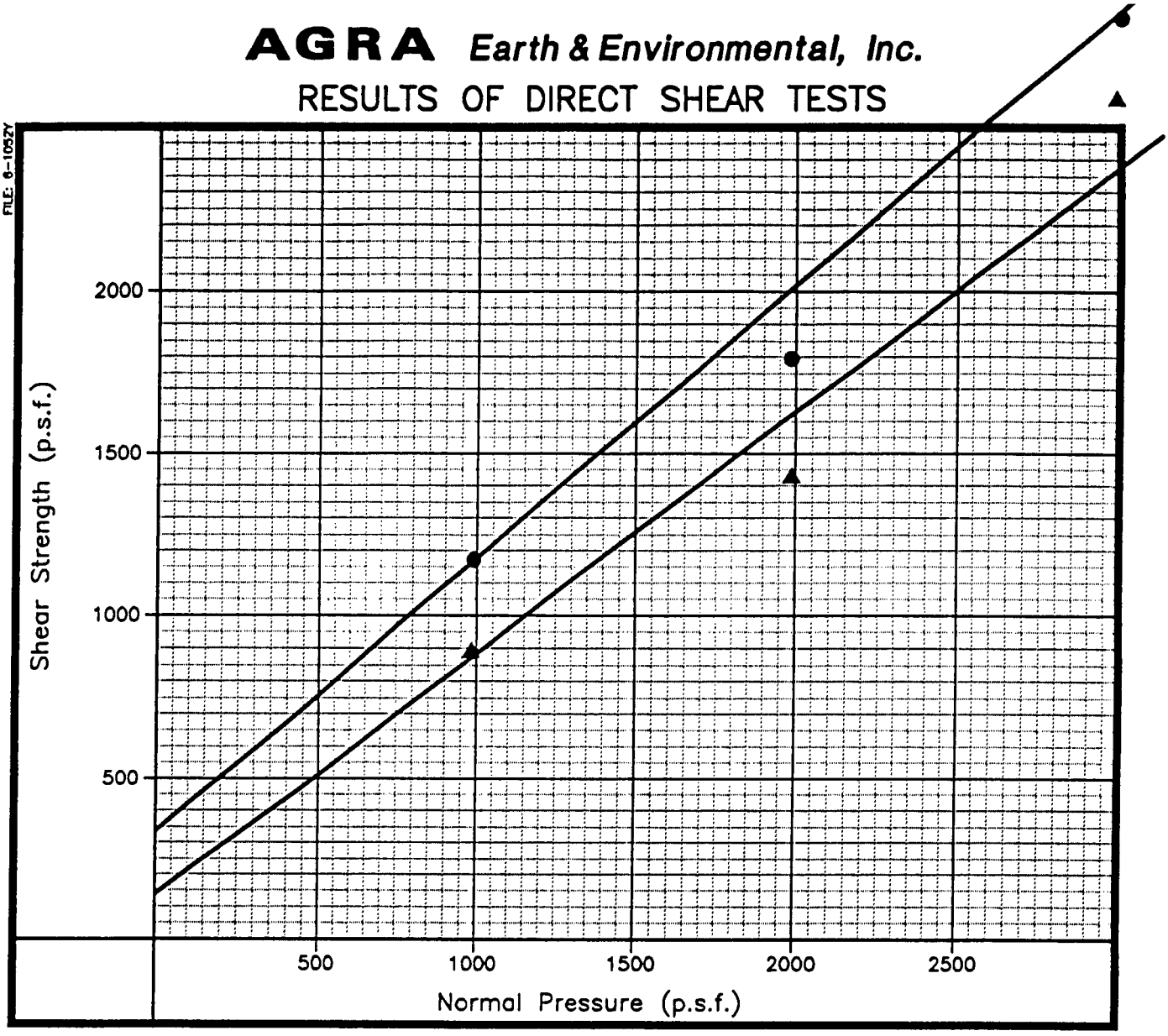


AGRA EARTH & ENVIRONMENTAL, INC.

JOB # : 6-212-1052

AGRA Earth & Environmental, Inc.

RESULTS OF DIRECT SHEAR TESTS



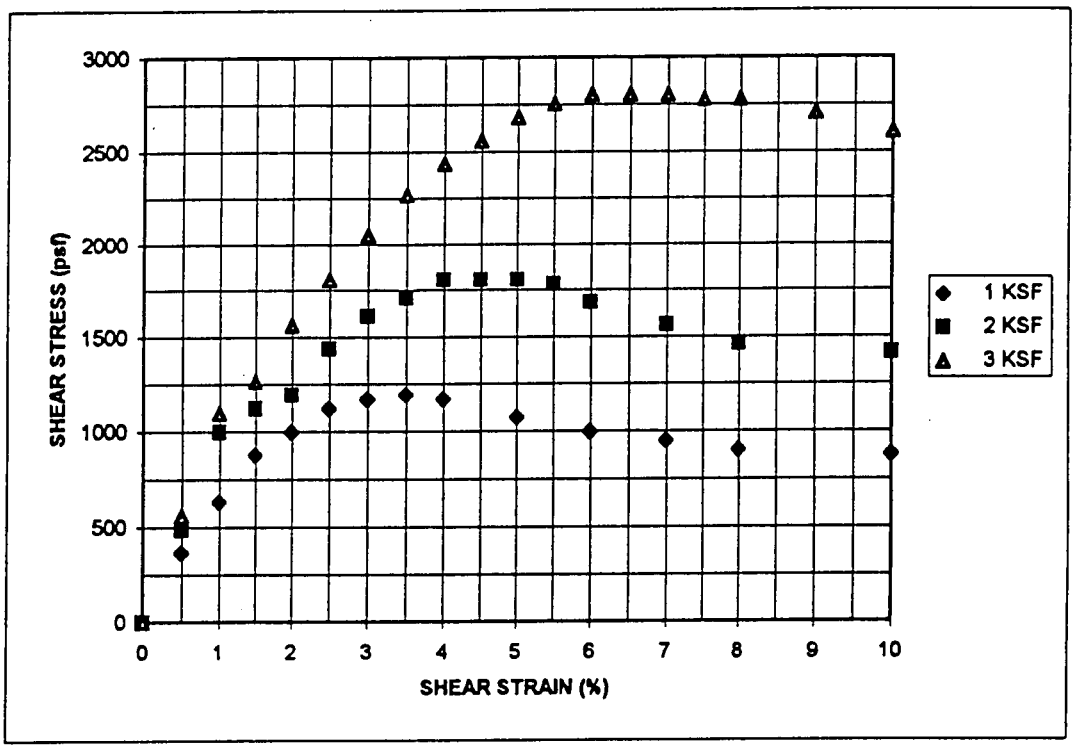
O peak shear strength Δ strength at 1/4 inch displacement

Open symbols (Δ) signify tests at natural moisture;
Solid symbols (▲) signify tests on submerged samples.

BORING/SAMPLE	MATERIAL	PEAK	ULTIMATE
109/3	TERRACE DEPOSITS: fine to coarse SILTY SAND (SUBMERGED)	c = 350 psf ø = 40°	c = 150 psf ø = 38°

ROUTE 138 - PHASE 2

BORING : 109
SAMPLE : 3 (SUBMERGED)

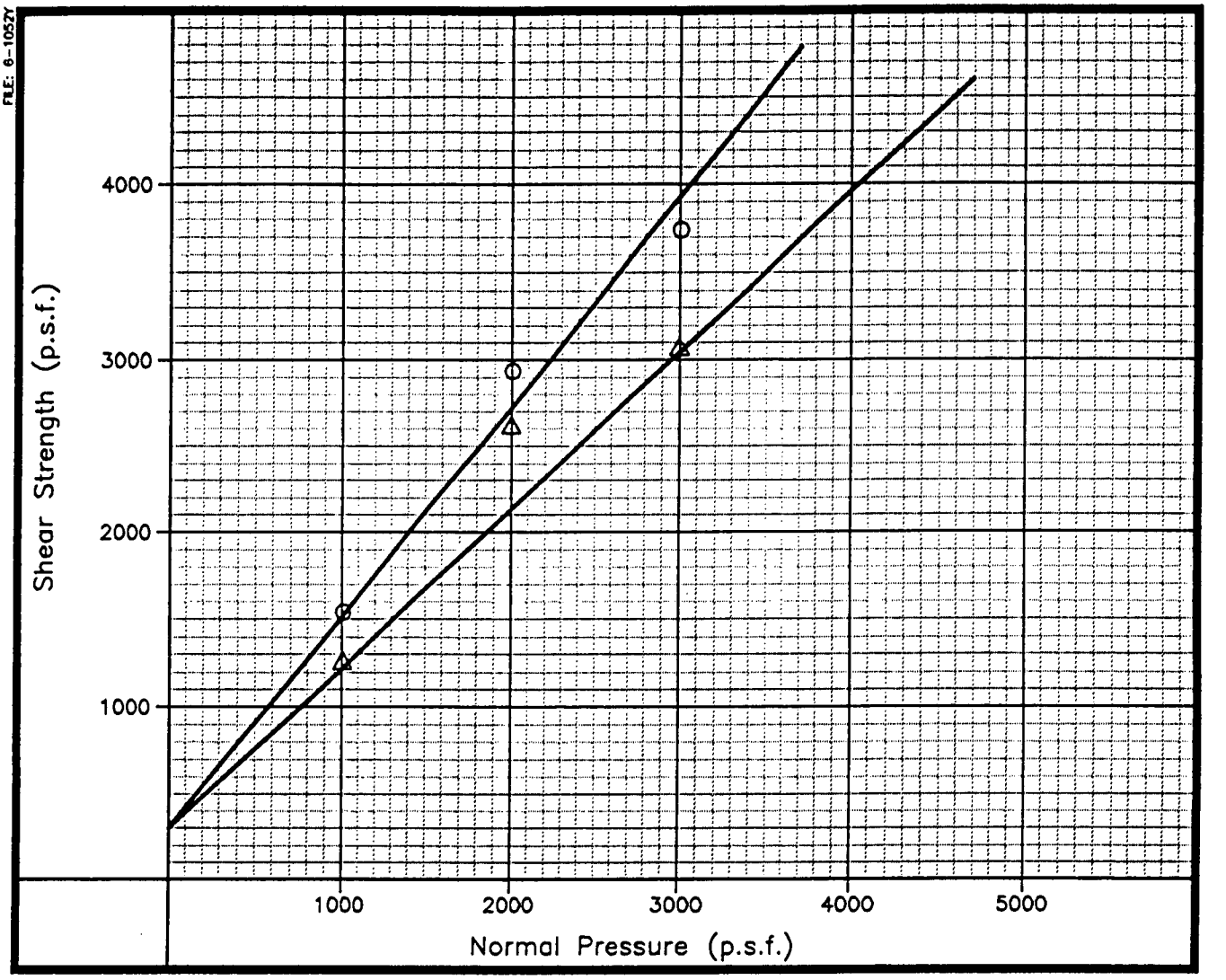


AGRA EARTH & ENVIRONMENTAL, INC.

JOB # : 6-212-1052

AGRA Earth & Environmental, Inc.

RESULTS OF DIRECT SHEAR TESTS



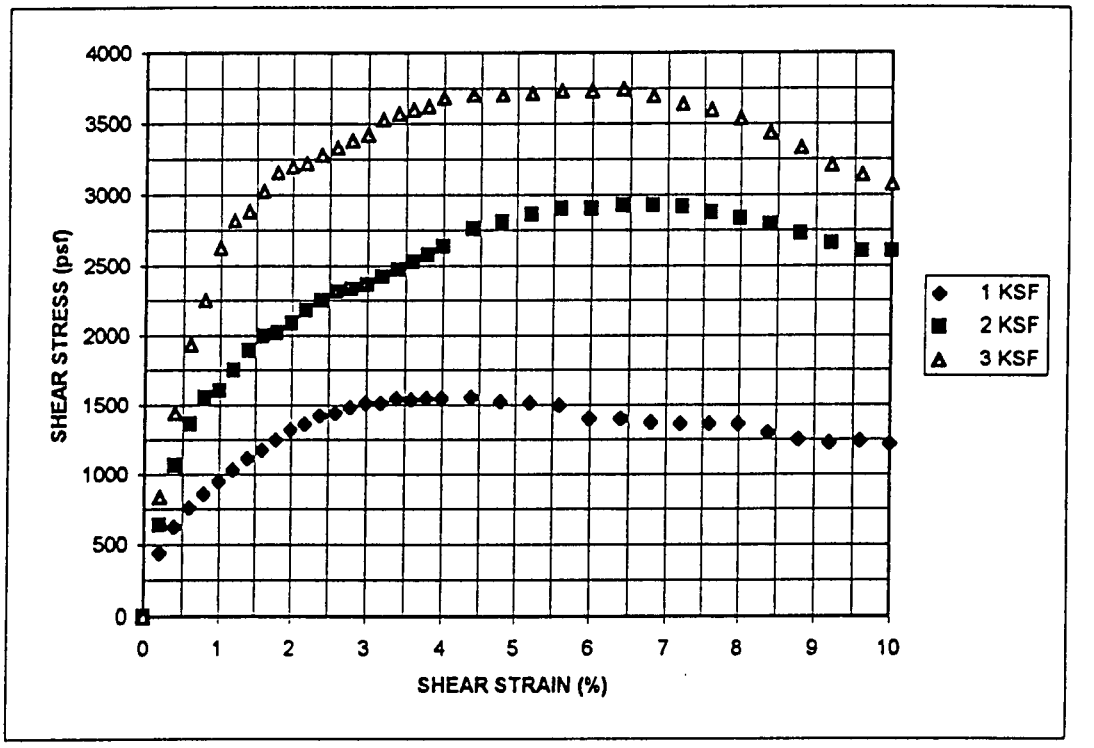
○ peak shear strength △ strength at 1/4 inch displacement

Open symbols (△) signify tests at natural moisture;
Solid symbols (▲) signify tests on submerged samples.

BORING/SAMPLE	MATERIAL	PEAK	ULTIMATE
109/6	TERRACE DEPOSITS: medium to coarse SILTY SAND (NATURAL MOISTURE)	c = 300 psf ø = 50°	c = 300 psf ø = 42°

ROUTE 138 - PHASE 2

BORING : 109
SAMPLE : 6 (NATURAL)

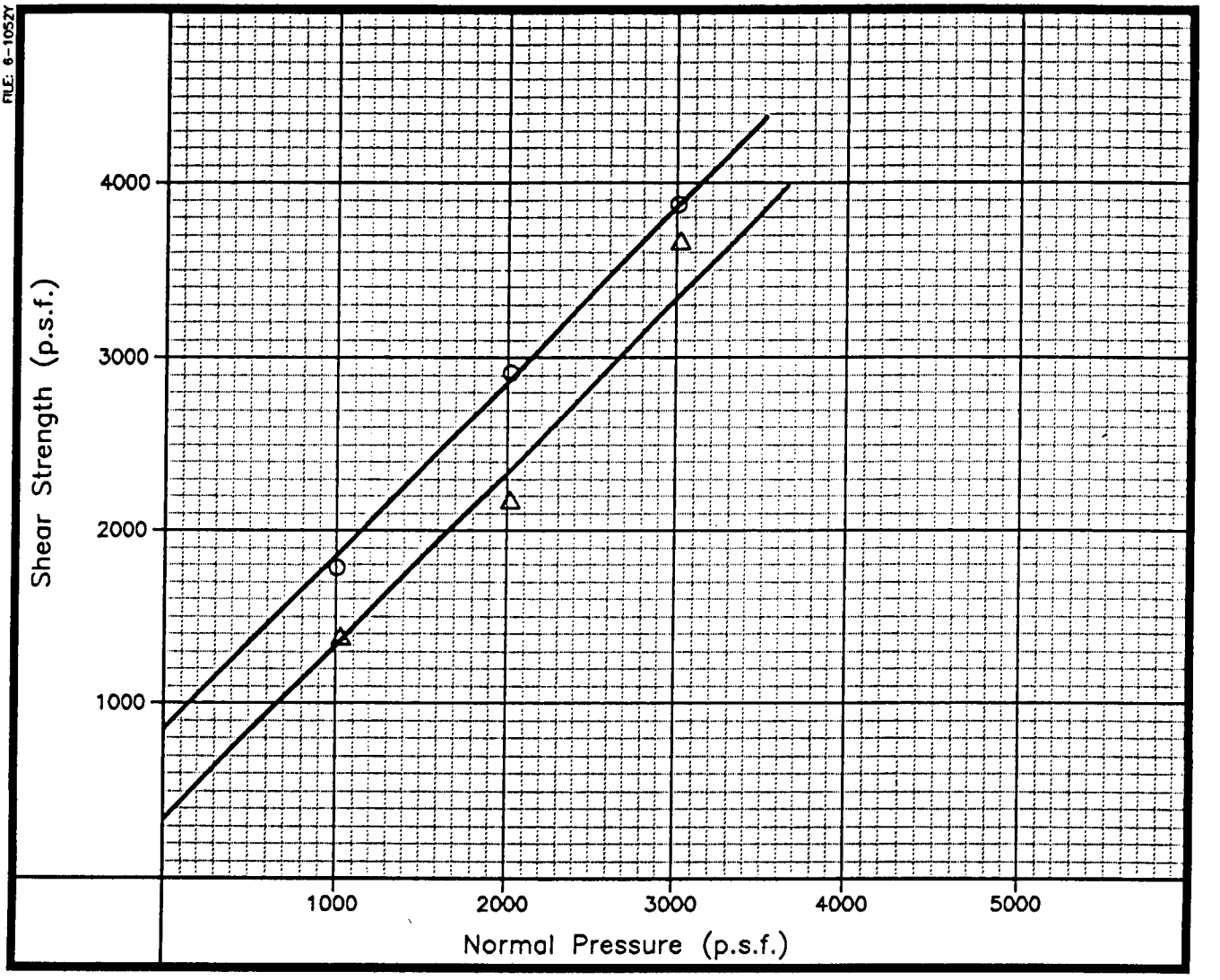


AGRA EARTH & ENVIRONMENTAL, INC.

JOB # : 6-212-1052

AGRA Earth & Environmental, Inc.

RESULTS OF DIRECT SHEAR TESTS



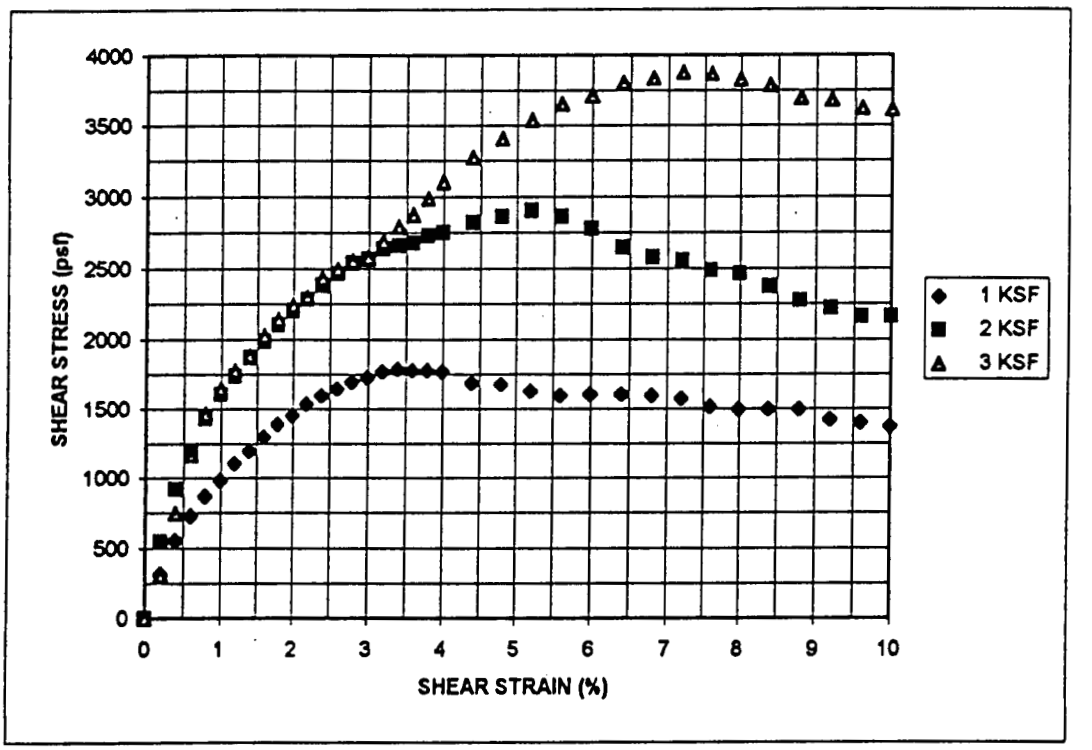
O peak shear strength △ strength at 1/4 inch displacement

Open symbols (△) signify tests at natural moisture;
Solid symbols (▲) signify tests on submerged samples.

BORING/SAMPLE	MATERIAL	PEAK	ULTIMATE
109/8	TERRACE DEPOSITS: medium to coarse SAND WITH GRAVEL (NATURAL MOISURE)	c = 850 psf ø = 45°	c = 330 psf ø = 45°

ROUTE 138 - PHASE 2

BORING : 109
SAMPLE : 8 (NATURAL)



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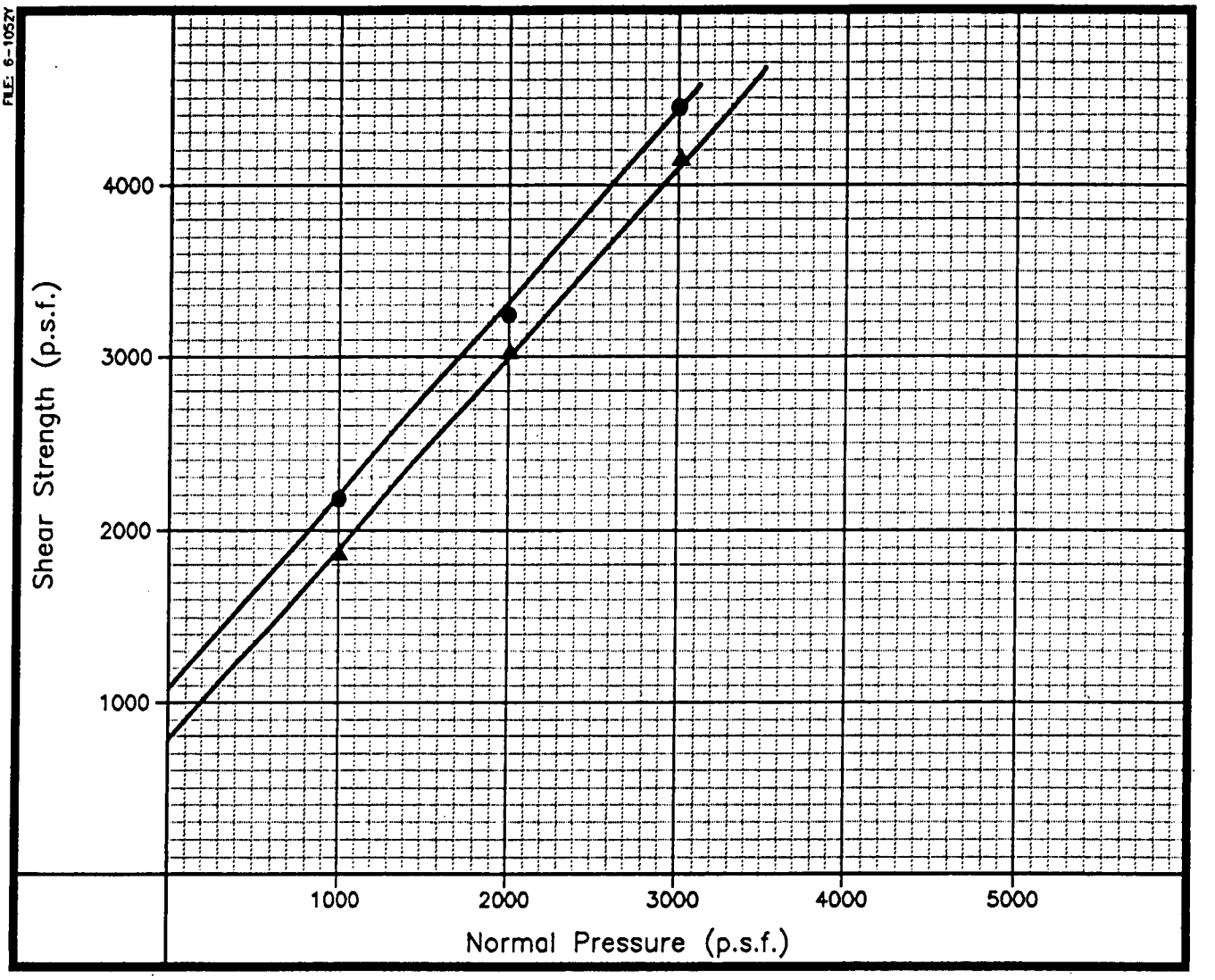
JOB # : 6-212-1052

AGRA Earth & Environmental, Inc.

RESULTS OF DIRECT SHEAR TESTS



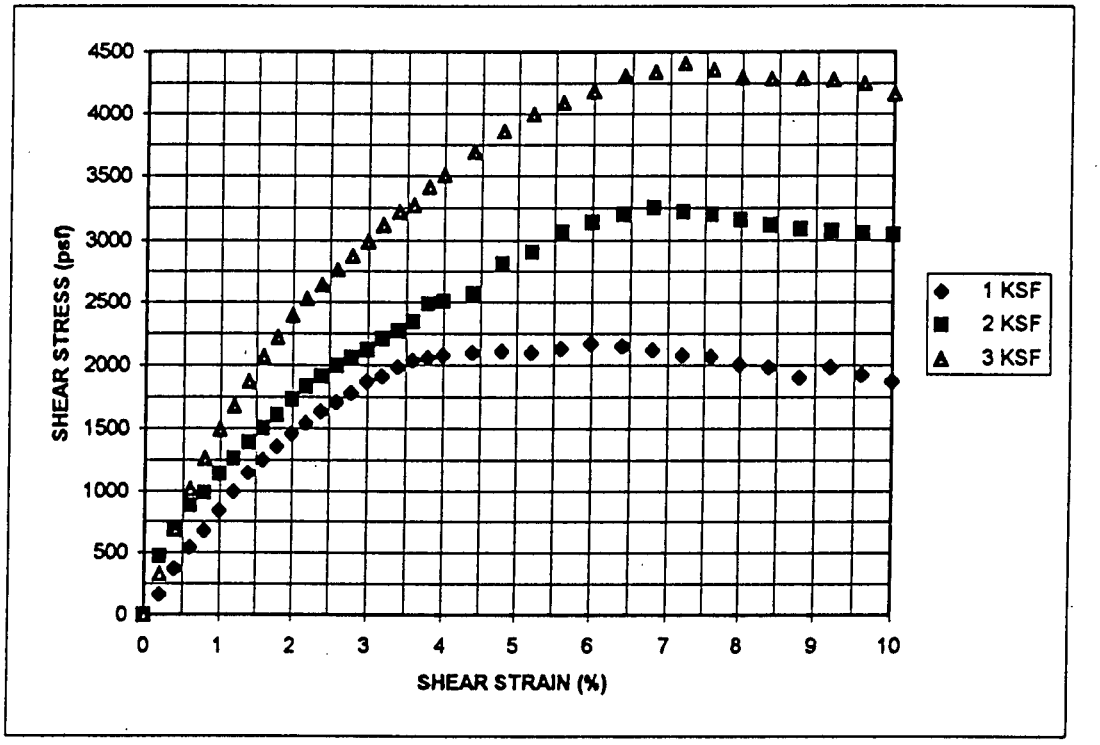
○ peak shear strength △ strength at 1/4 inch displacement

Open symbols (△) signify tests at natural moisture;
Solid symbols (▲) signify tests on submerged samples.

BORING/SAMPLE	MATERIAL	PEAK	ULTIMATE
109/11	TERRACE DEPOSITS: fine to coarse SAND WITH GRAVEL (SUBMERGED)	c = 1090 psf ø = 48°	c = 800 psf ø = 48°

ROUTE 138 - PHASE 2

BORING : 109
SAMPLE : 11 (SUBMERGED)

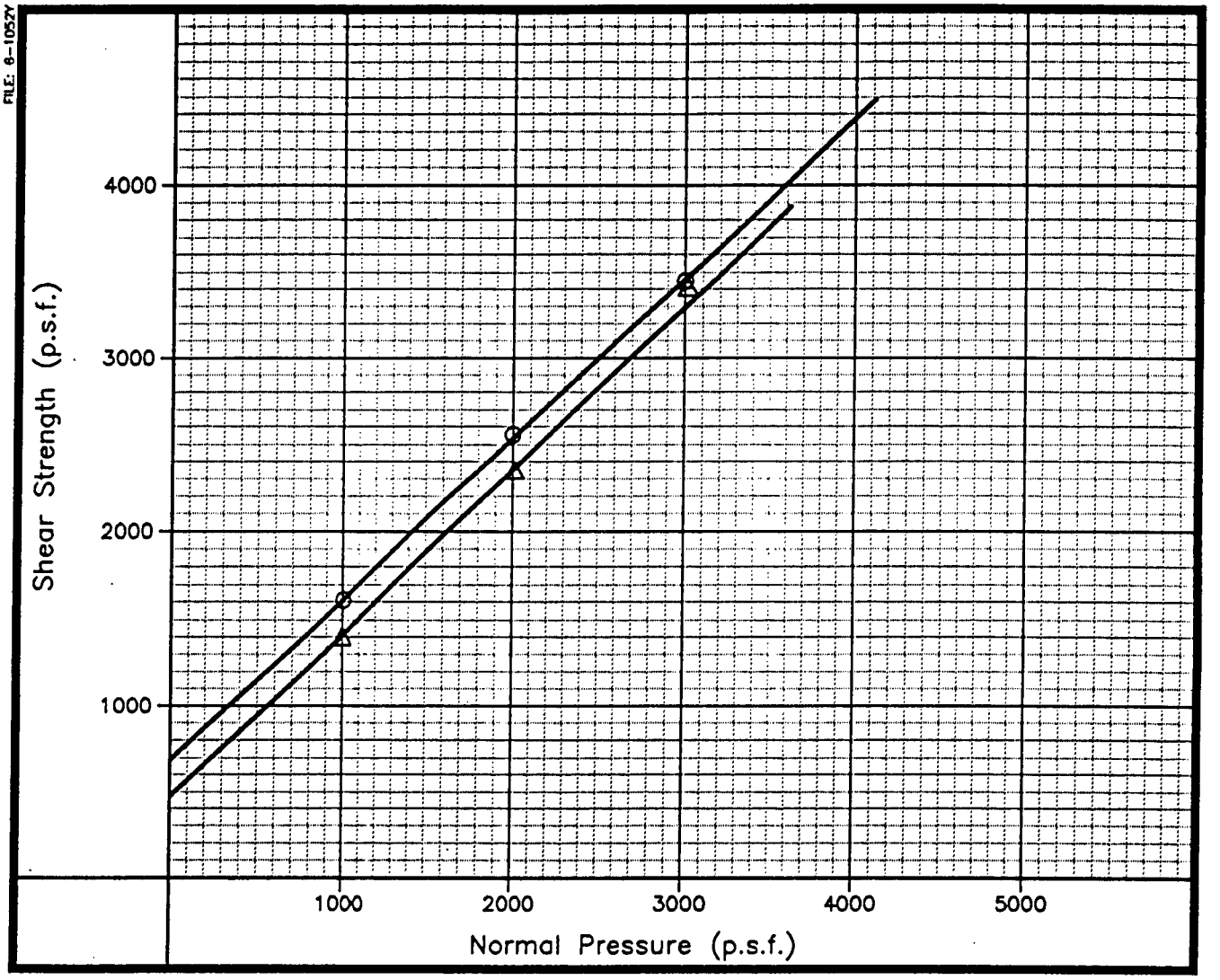


AGRA EARTH & ENVIRONMENTAL, INC.

JOB # : 6-212-1052

AGRA Earth & Environmental, Inc.

RESULTS OF DIRECT SHEAR TESTS



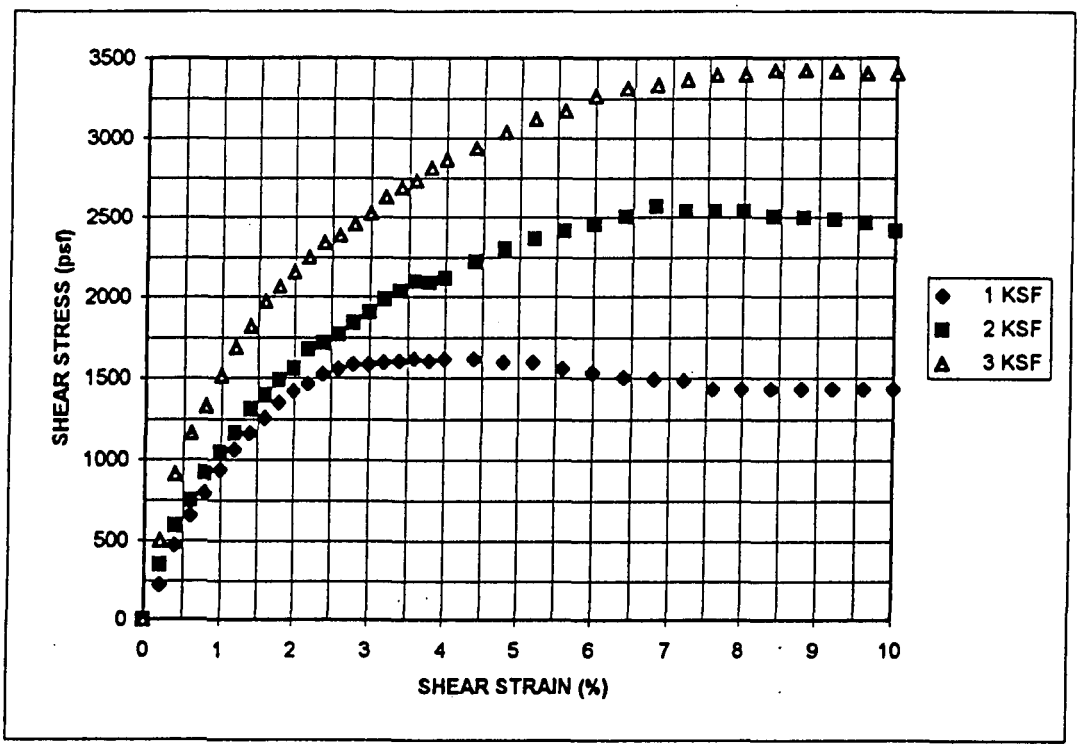
○ peak shear strength △ strength at 1/4 inch displacement

Open symbols (△) signify tests at natural moisture;
Solid symbols (▲) signify tests on submerged samples.

BORING/SAMPLE	MATERIAL	PEAK	ULTIMATE
109/11	TERRACE DEPOSITS: fine to coarse SAND WITH GRAVEL (NATURAL MOISTURE)	c = 700 psf ø = 42.5°	c = 500 psf ø = 42.5°

ROUTE 138 - PHASE 2

BORING : 109
SAMPLE : 11 (NATURAL)

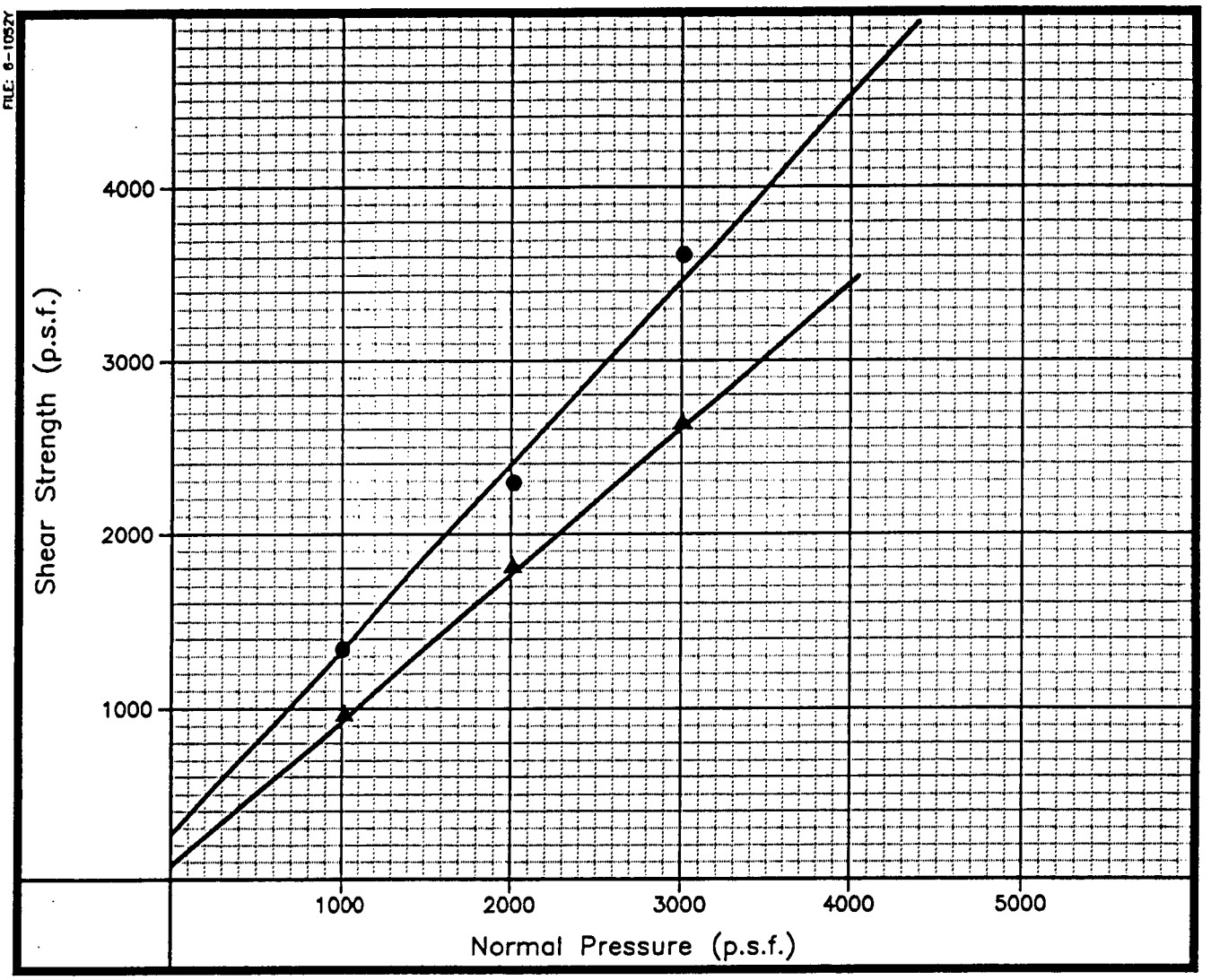


AGRA EARTH & ENVIRONMENTAL, INC.

JOB # : 6-212-1052

AGRA Earth & Environmental, Inc.

RESULTS OF DIRECT SHEAR TESTS



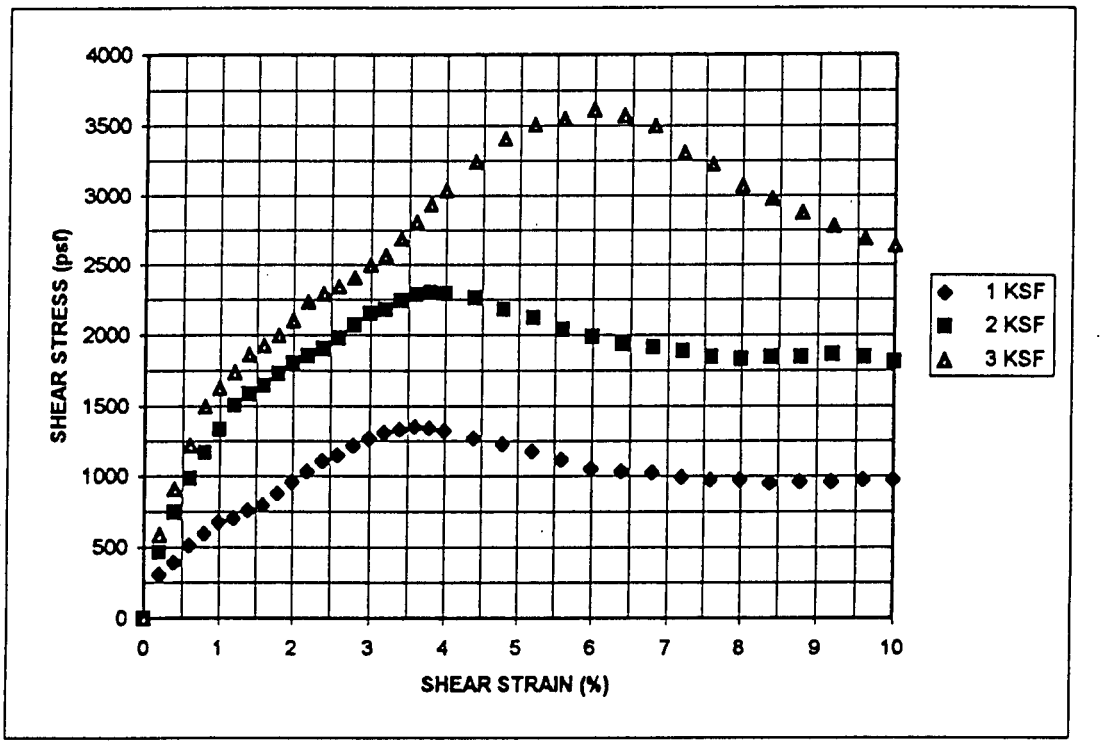
○ peak shear strength △ strength at 1/4 inch displacement

Open symbols (△) signify tests at natural moisture;
Solid symbols (▲) signify tests on submerged samples.

BORING/SAMPLE	MATERIAL	PEAK	ULTIMATE
109/23	CROWDER FORMATION: fine to coarse SANDSTONE (SUBMERGED)	c = 300 psf ø = 47°	c = 100 psf ø = 40°

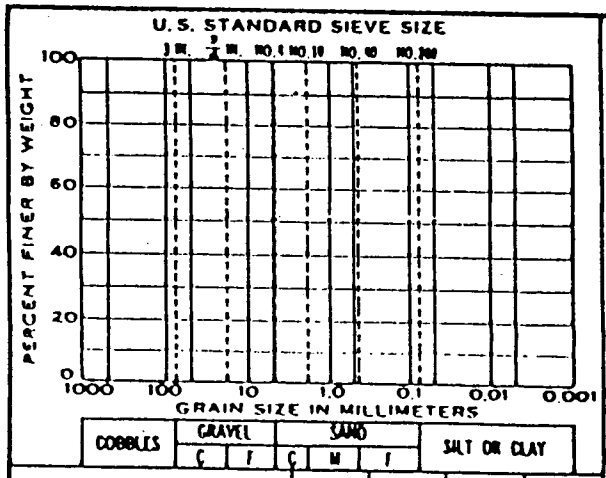
ROUTE 138 - PHASE 2

BORING : 109
SAMPLE : 23 (SUBMERGED)

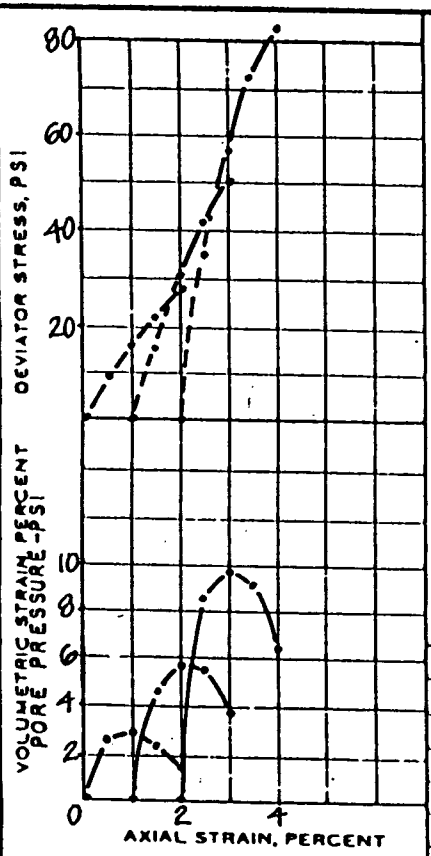


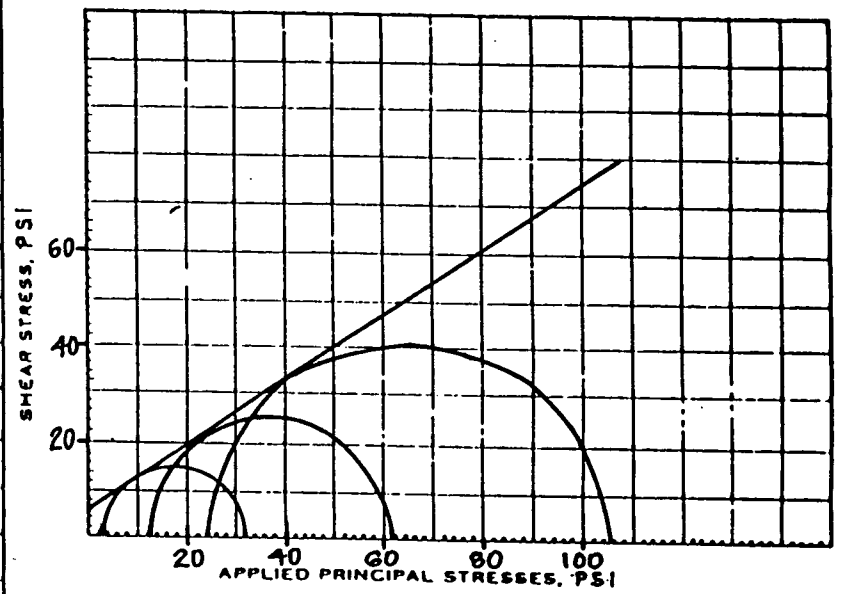
AGRA EARTH & ENVIRONMENTAL, INC.

JOB # : 6-212-1052



	Test No.				
INITIAL	Water Content, W_o %	10.1			
	Dry Density Lbs/Cu Ft	127.4			
	Void Ratio, e_o				
	Saturation, S_o %				
BEFORE TEST	W.C. after Saturation, W_s %	13.9	13.7	13.0	
	Saturation, S %	100	100	100	
	Consol Pressure (PSF)	5	15	30	
	W.C. after Consol, W_c %	13.7	13.0	12.5	
	Void Ratio after Consol e_c				
AT FAILURE	Maj Prin. Stress σ_I (PSI)	31.1	61.4	105.5	
	Min Prin Stress σ_{III} (PSI)	3.6	11.3	23.7	
	Pore Pressure, u (PSI)	1.4	3.7	6.3	
	Water Content, W_F %	13.7	13.0	12.5	
	Void Ratio, e_F				
Specimen Dia. INCHES		2.5	2.53	2.55	
Initial Height INCHES		5.0	4.95	4.90	
Test Time to Failure Min.		50	50	50	





Remarks:

Type of Test

CONSOLIDATED UNDRAINED

Type of Specimen

ø - 35 °	Tan ø - .700	c - 5.2 PSI

Classification

LL	G
PL	D_s

AGRA Earth & Environmental, Inc.

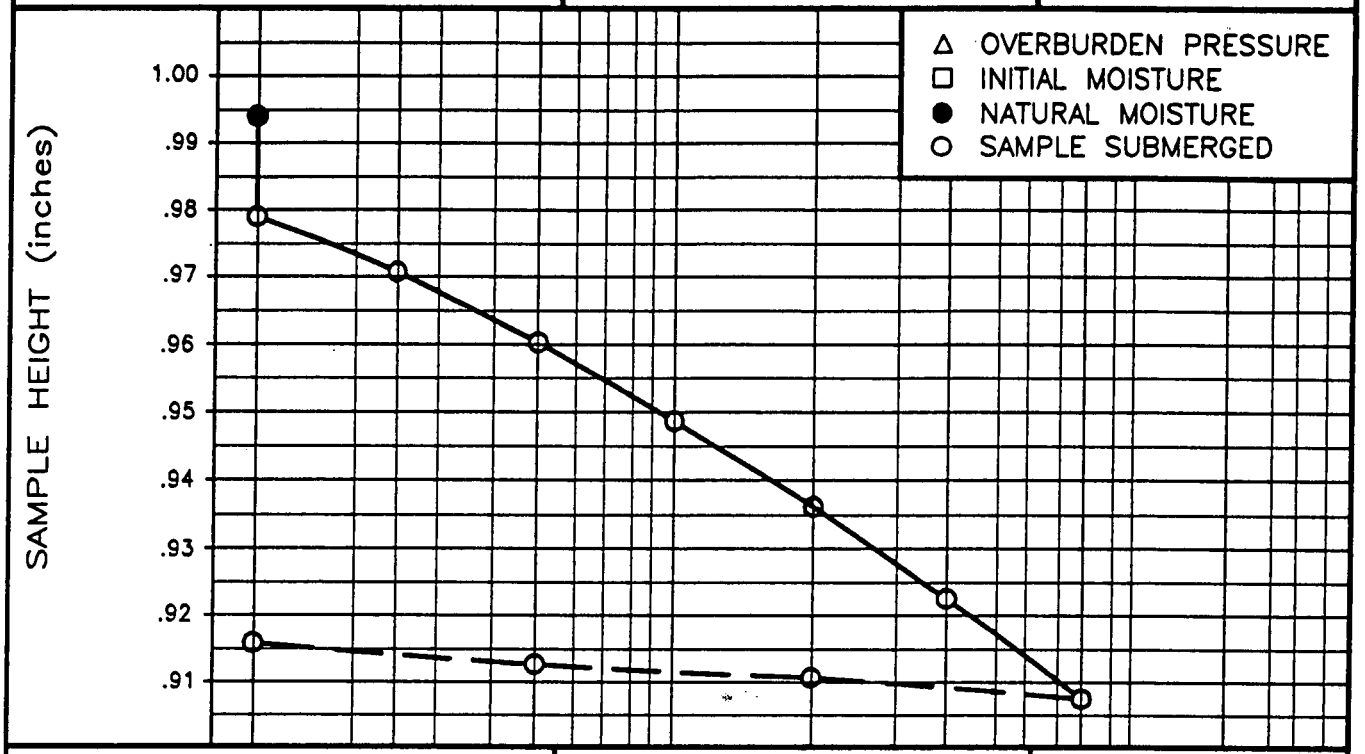
Project	ROUTE 138, PHASE 2	№ 6-212-1052	
Client	RANCHO LAS FLORES		
Boring No.	108	Sample No.	8
Elev or Depth	30'	Date	12-12-96

TRIAXIAL COMPRESSION TEST REPORT

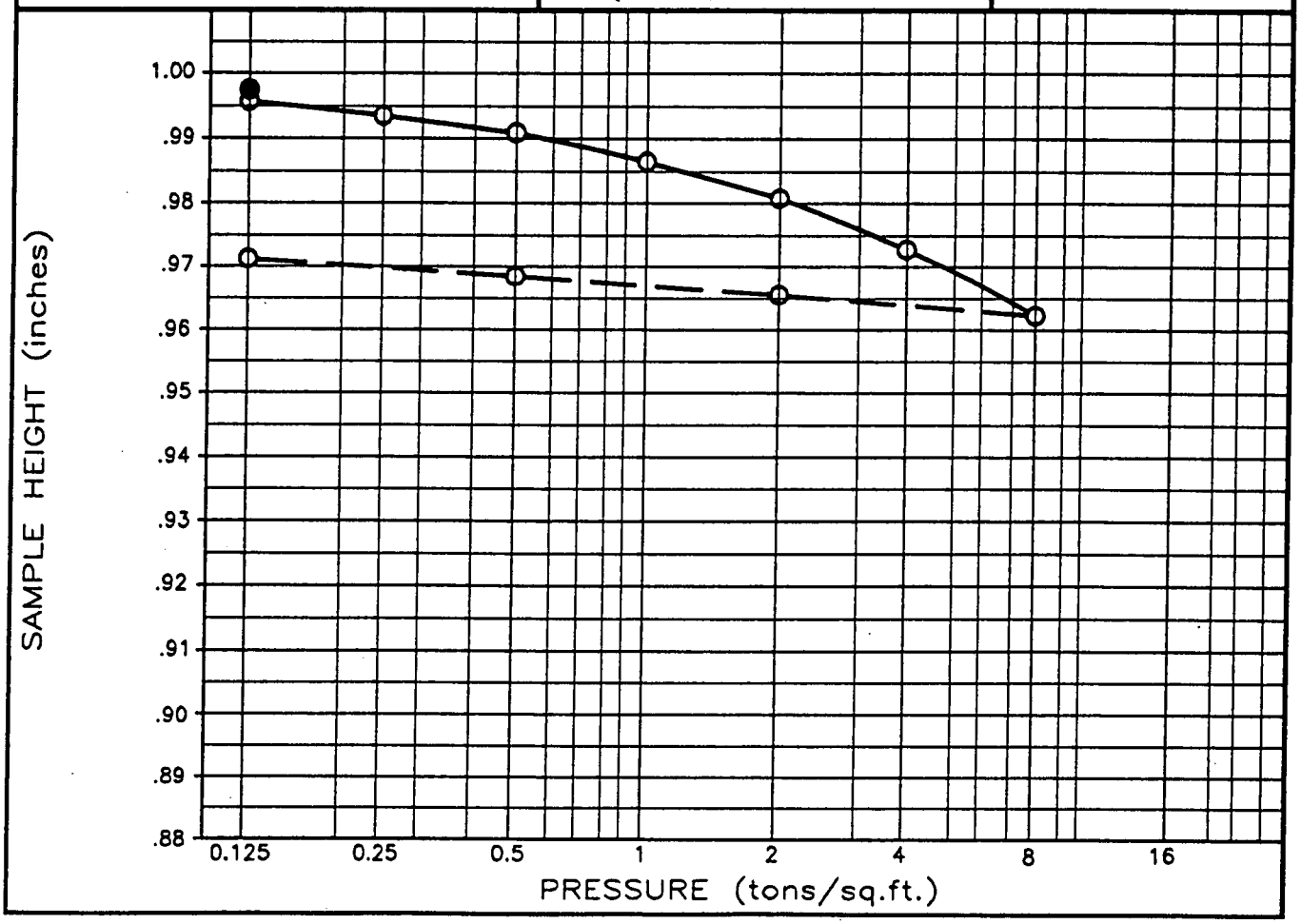
AGRA Earth & Environmental, Inc.

CONSOLIDATION TEST – PRESSURE CURVES

Bor./Sample № TP 1/1	~~Elev.~~ – Depth 3'	Date 12-12-96



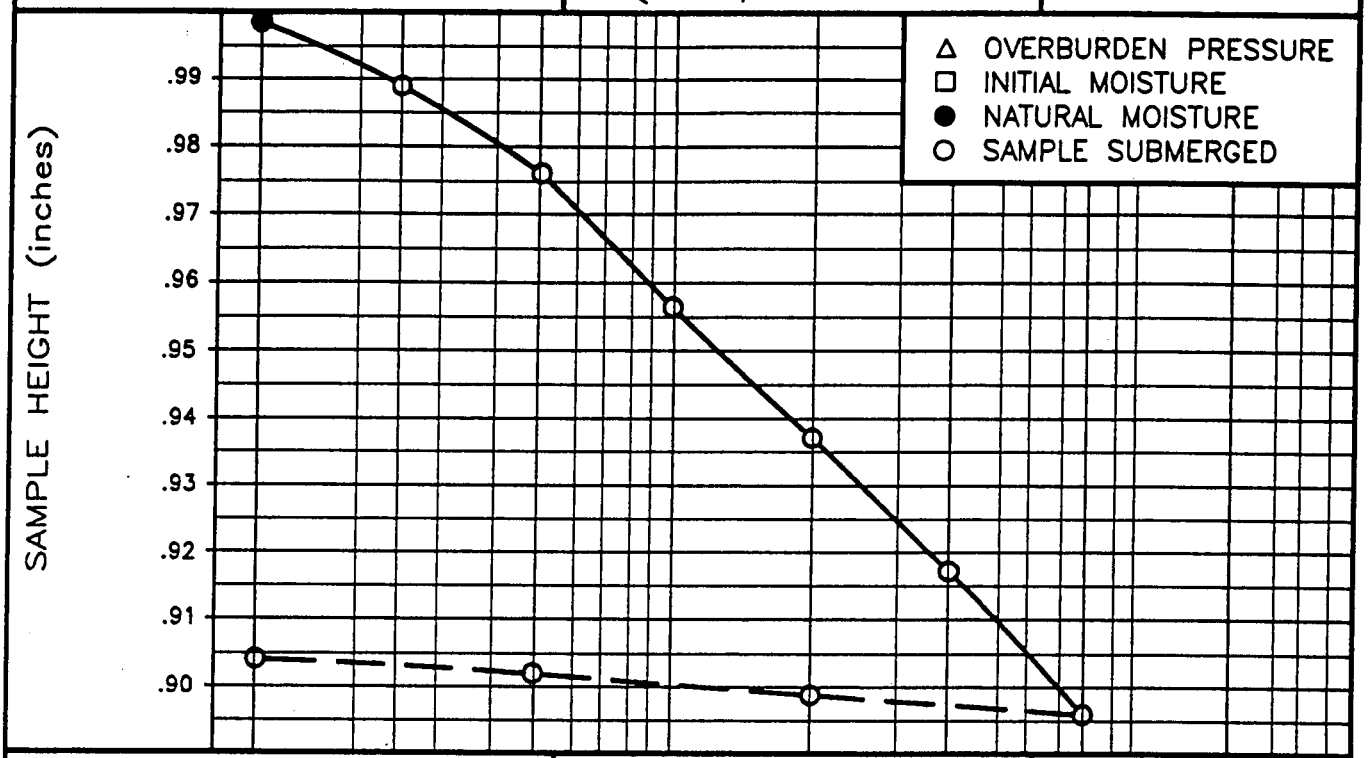
Bor./Sample № TP 1/4	~~Elev.~~ – Depth 7'	Date



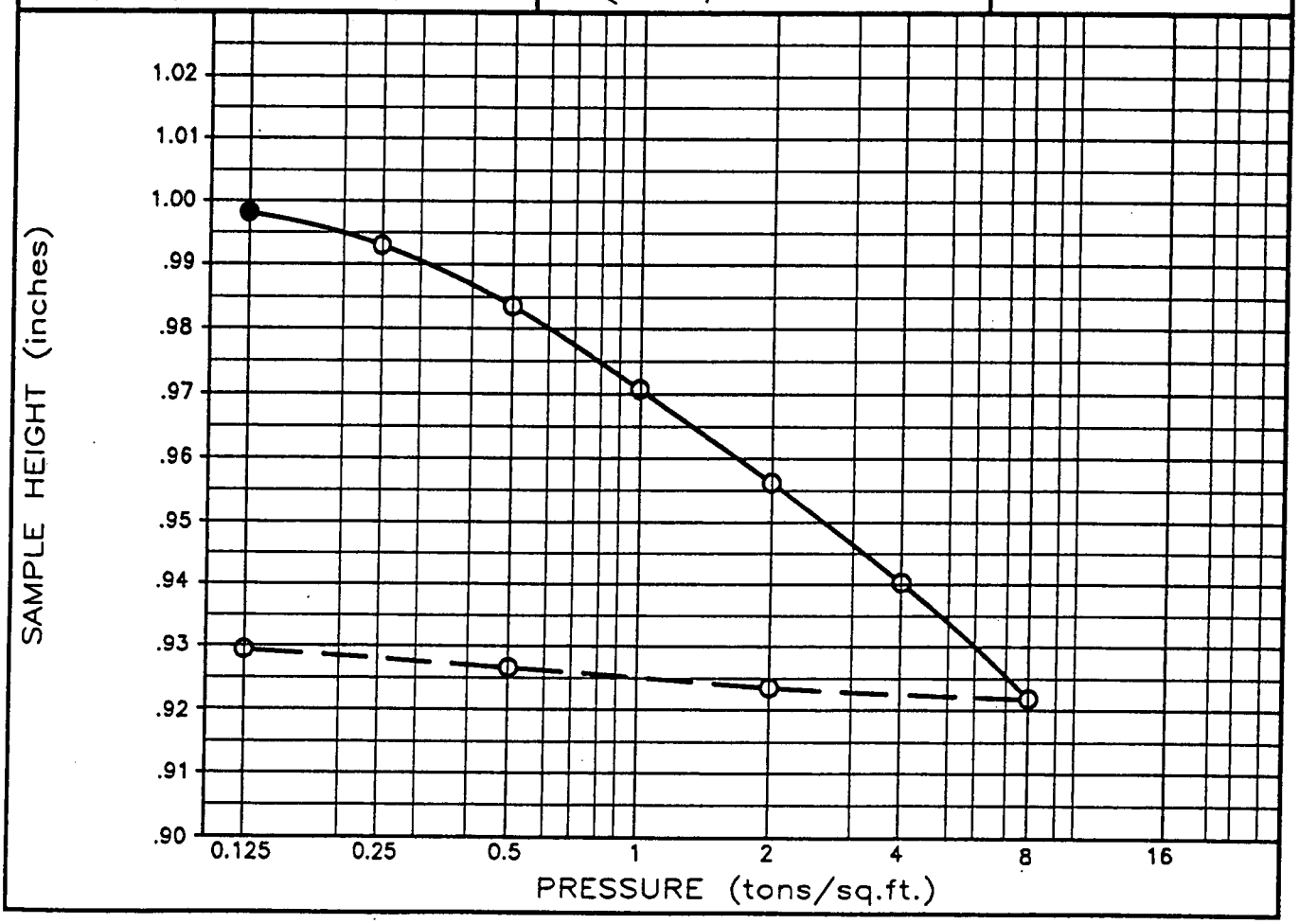
AGRA Earth & Environmental, Inc.

CONSOLIDATION TEST – PRESSURE CURVES

Bor./Sample № 104/1	~~Elev.~~ – Depth 3'	Date 12-2-96



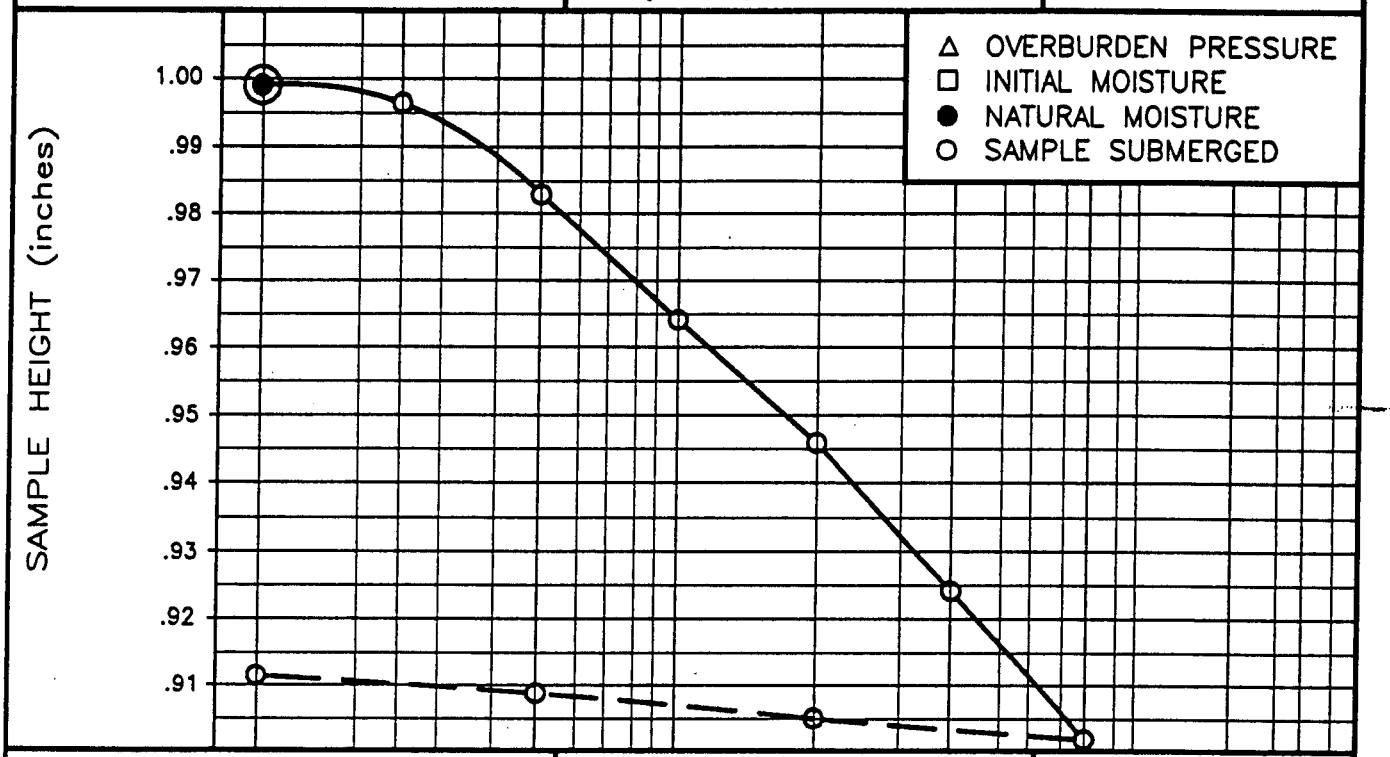
Bor./Sample № 104/4	~~Elev.~~ – Depth 6'	Date



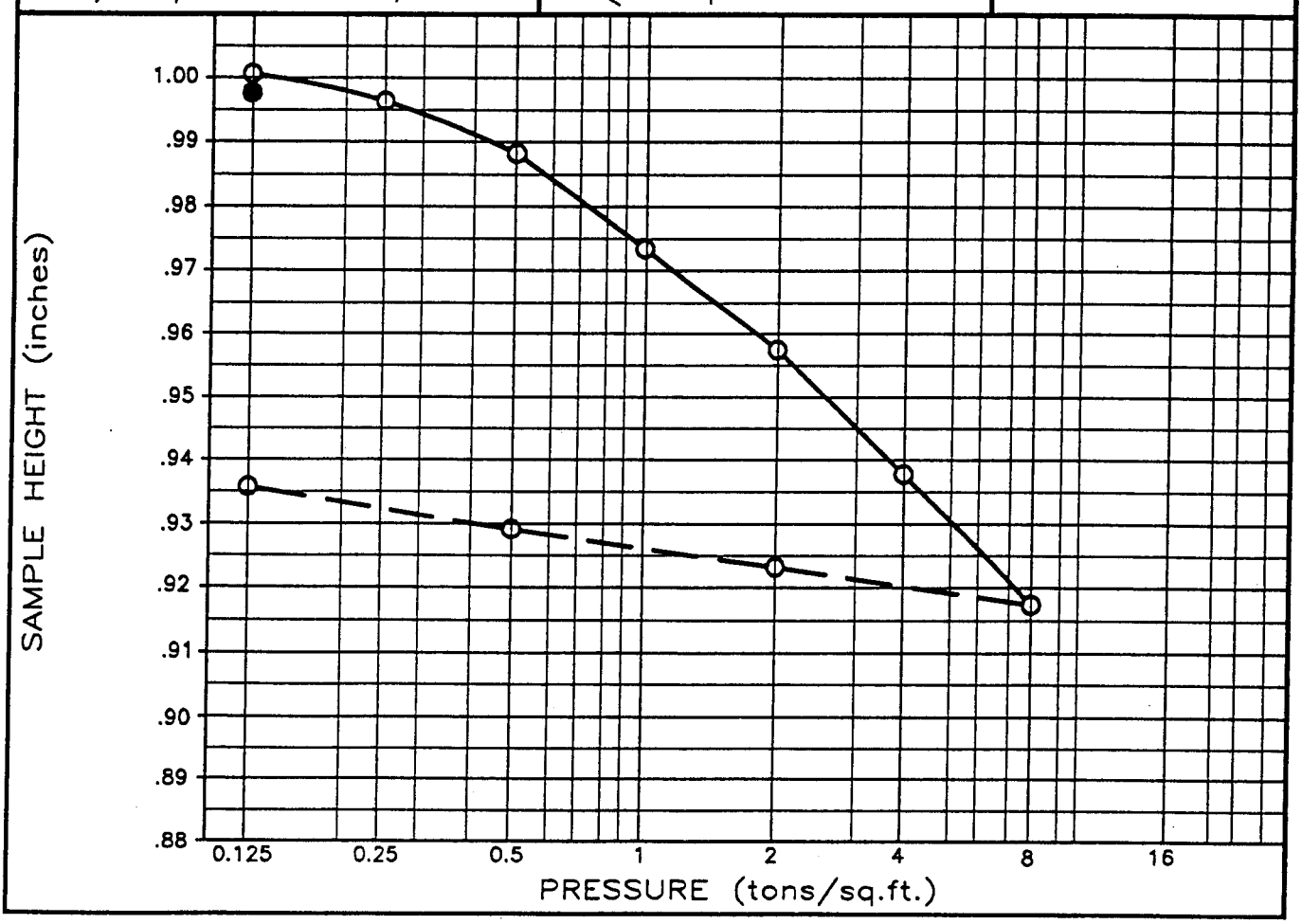
AGRA Earth & Environmental, Inc.

CONSOLIDATION TEST – PRESSURE CURVES

Bor./Sample № 105/4	~~Elev.~~ – Depth 10'	Date 12-2-96



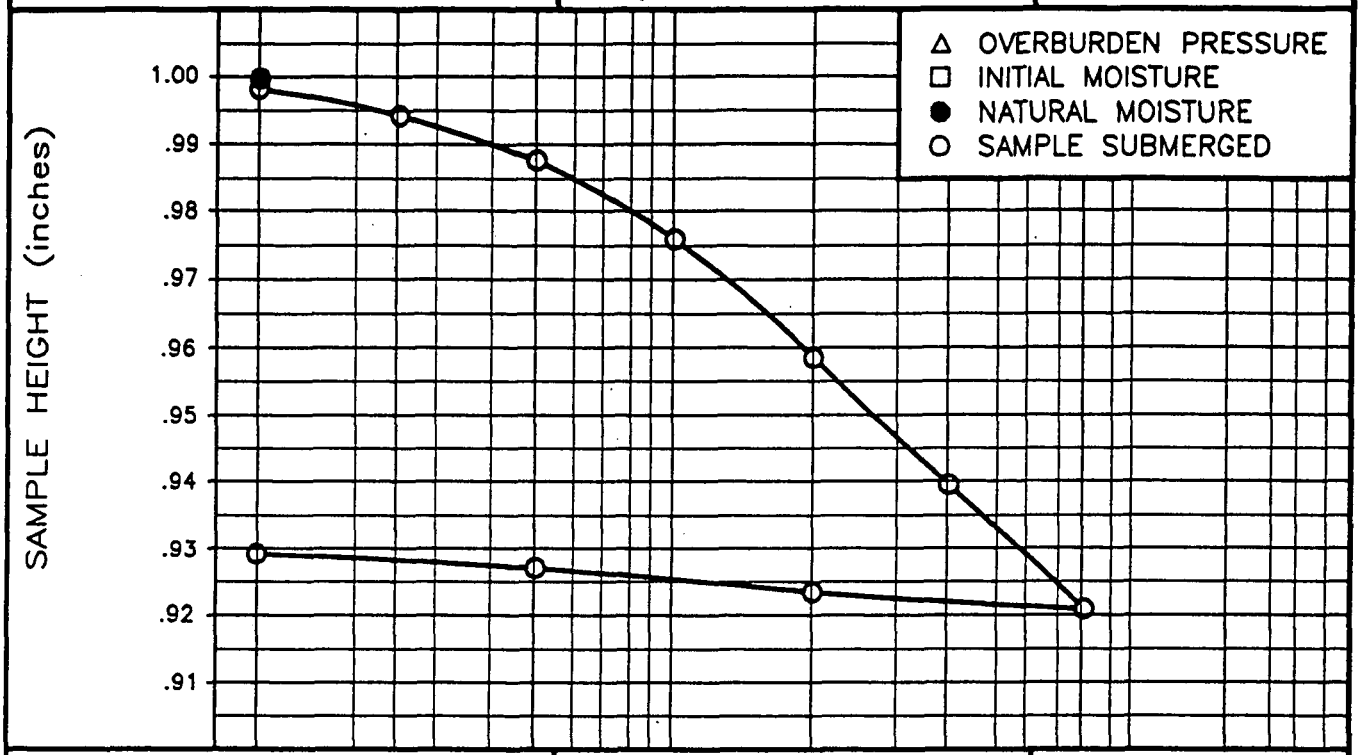
Bor./Sample № 105/5	~~Elev.~~ – Depth 13'	Date



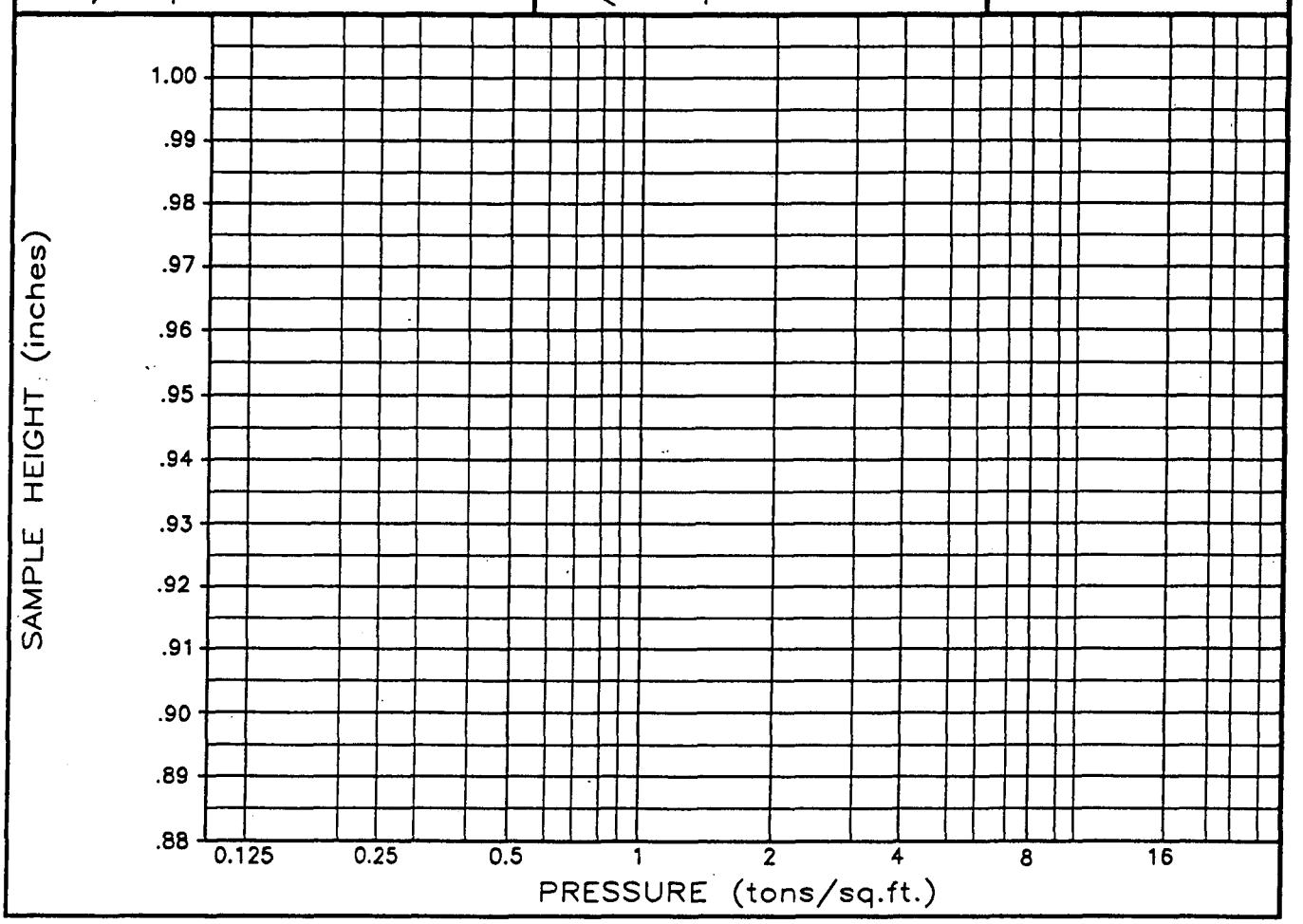
AGRA Earth & Environmental, Inc.

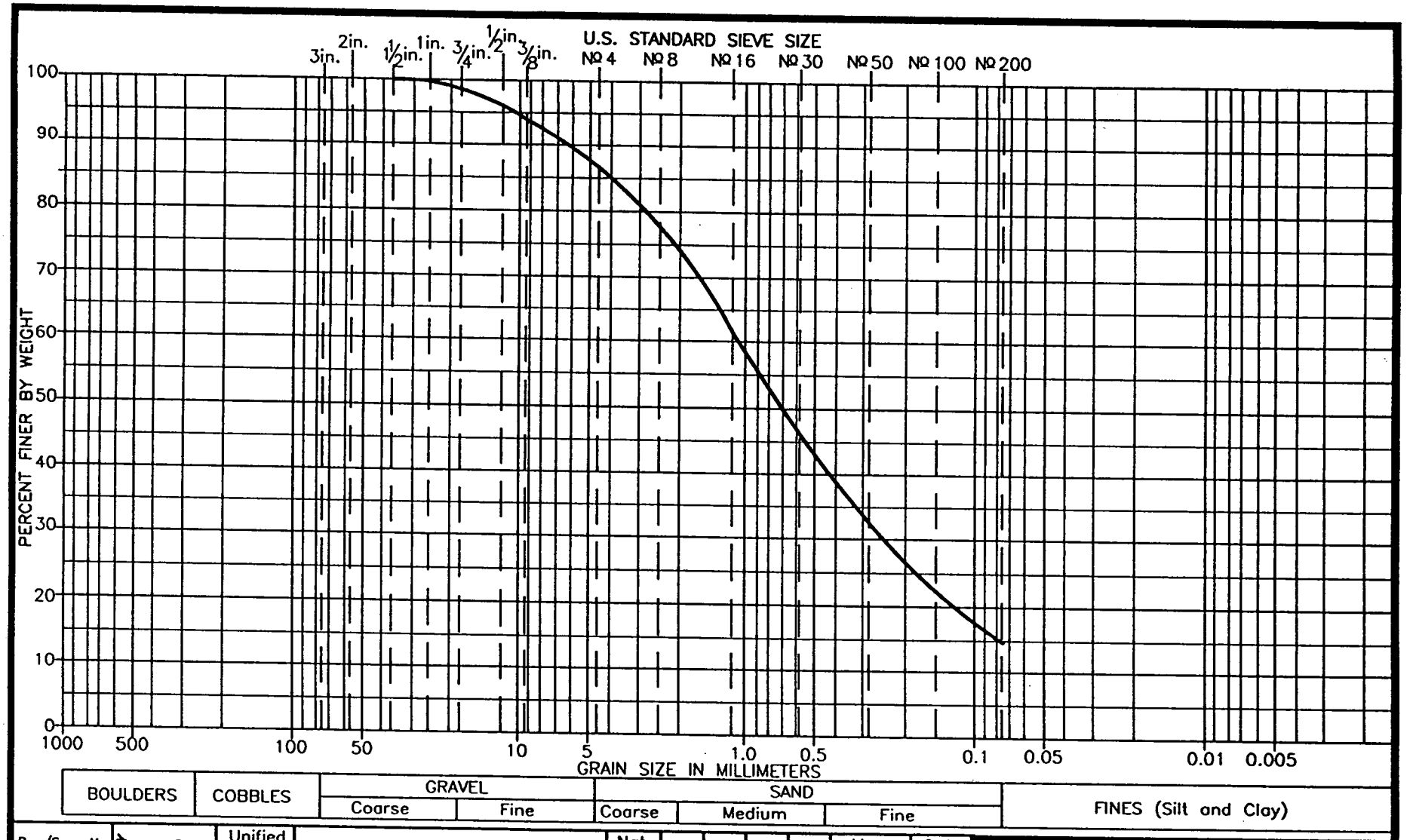
CONSOLIDATION TEST – PRESSURE CURVES

Bor./Sample № 106/1	~~Elev.~~ – Depth 3'	Date 12-12-96



Bor./Sample №	~~Elev.~~ – Depth	Date





Bor./Samp.No.	Elev. or Depth	Unified Soil Class	Description	Nat. Water	LL	PL	PI	SE	Max Dry Dens.	Opt. Moist.
101/2	3'	SM	fine to medium SILTY SAND							
— —										
— — — —										
. . . . .										

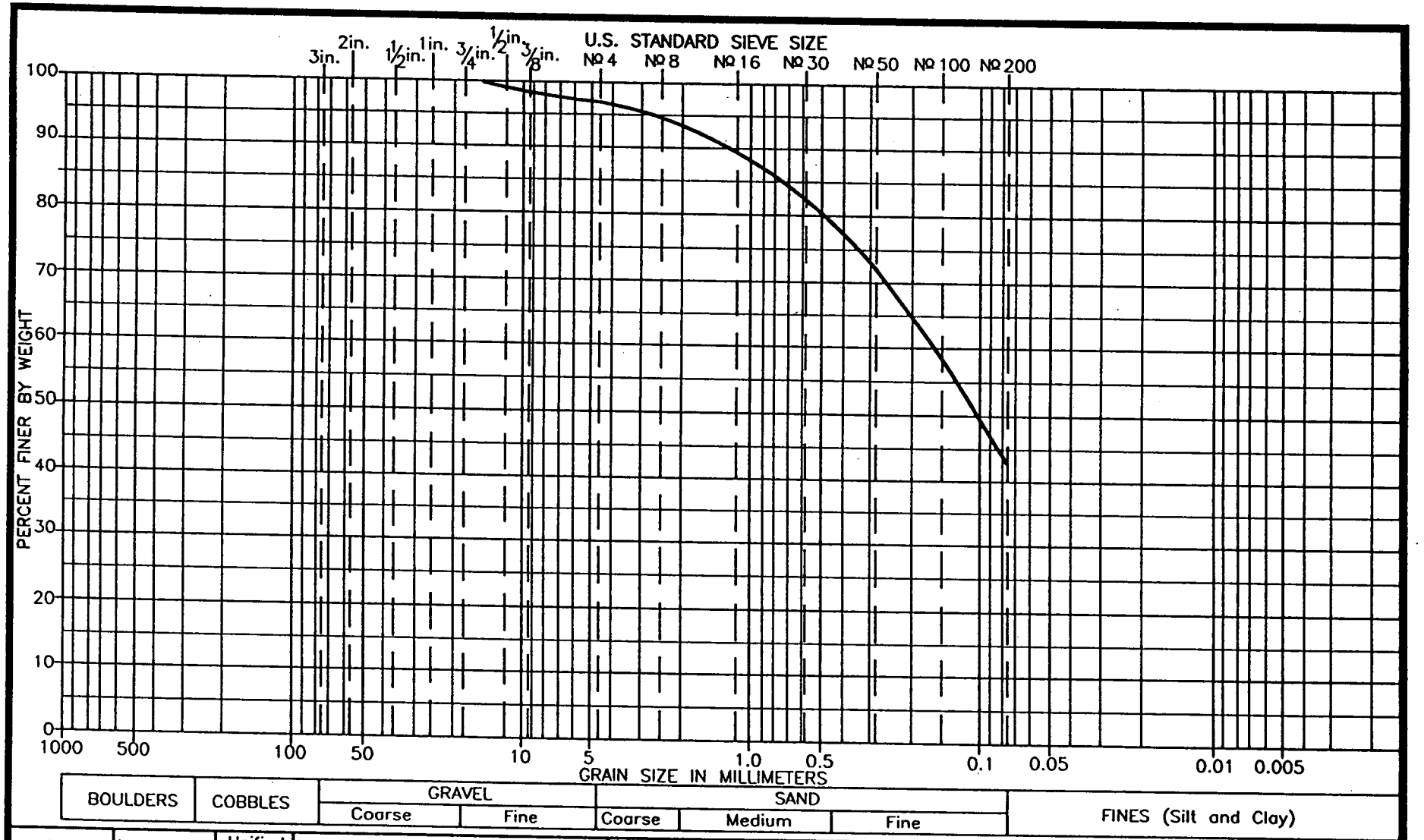
GRAIN SIZE DISTRIBUTION

AGRA
Earth & Environmental, Inc.

Project ROUTE 138, PHASE 2

By RN | Date 12-12-96

6-212-1052



Bor./Samp.No.	~~Elev.~~ or Depth	Unified Soil Class	Description	Nat. Water	LL	PL	PI	SE	Max Dry Dens.	Opt. Moist.
103/2	5'–7'	SM	fine to medium SILTY SAND							

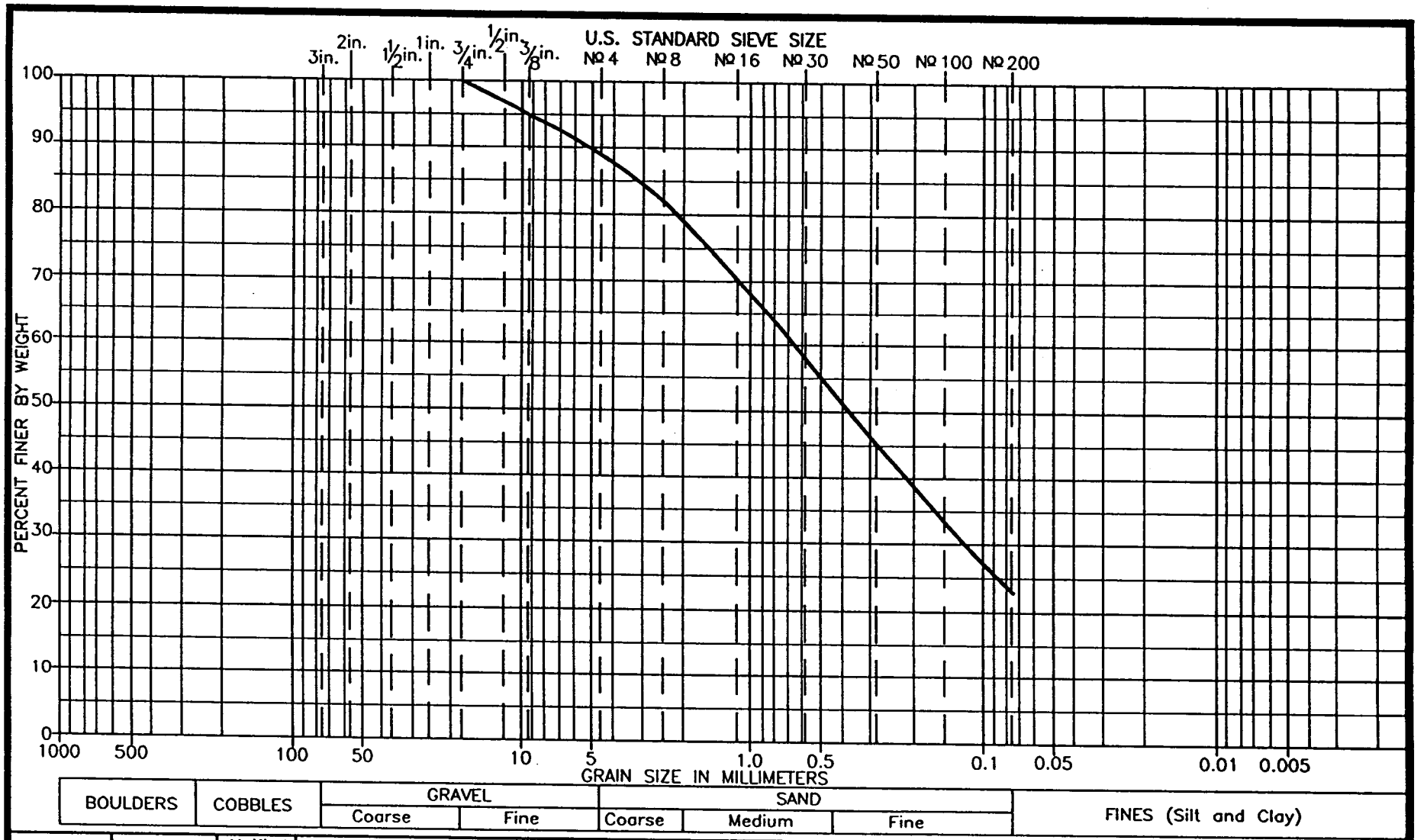
GRAIN SIZE DISTRIBUTION

AGRA
Earth & Environmental, Inc.

Project ROUTE 138, PHASE 2

By RN | Date 12–2–96

6–212–1052



Bor./Samp.No.	Elev. or Depth	Unified Soil Class	Description	Nat. Water	LL	PL	PI	SE	Max Dry Dens.	Opt. Moist.
107/3	6'–8'	SM	fine to medium SILTY SAND							

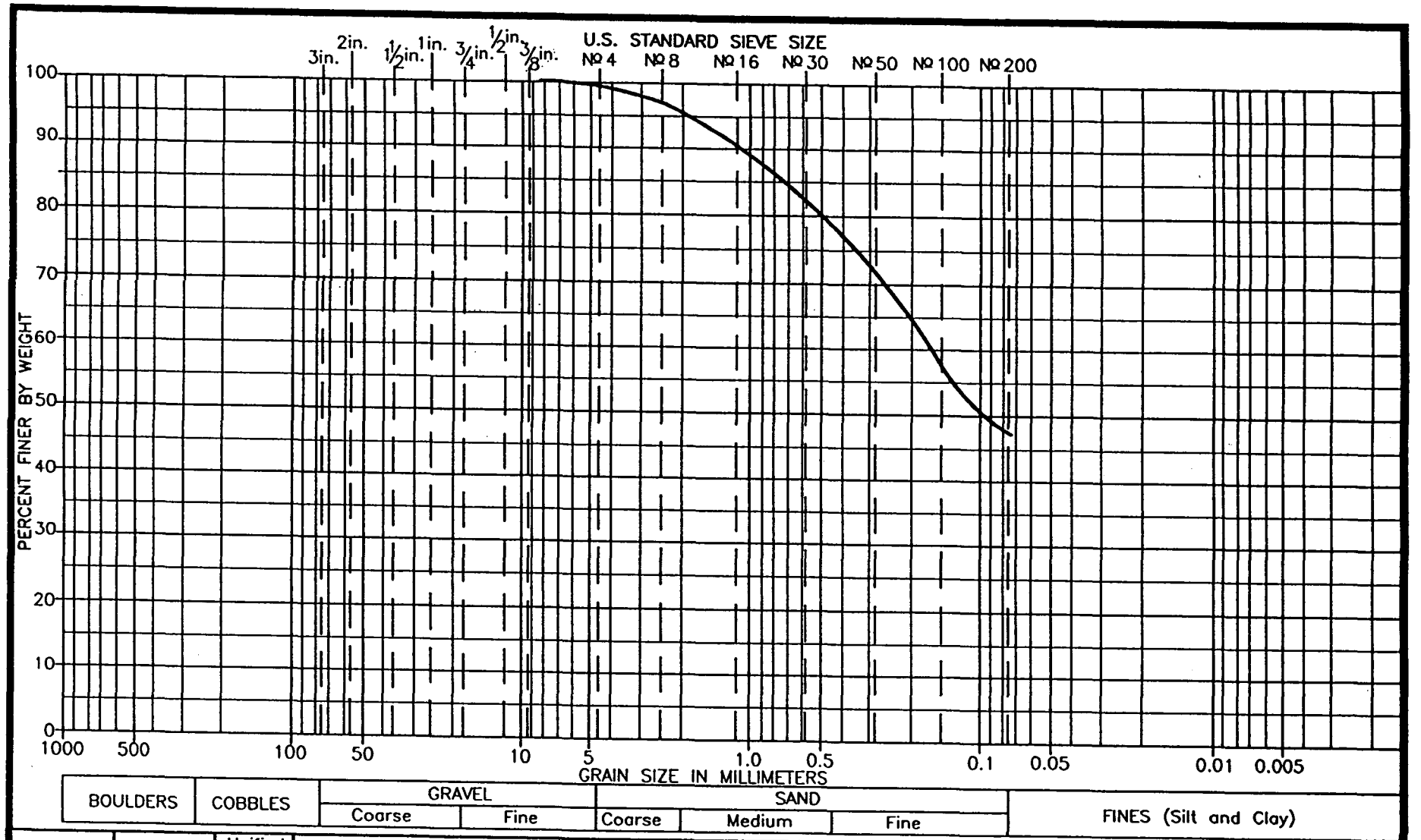
GRAIN SIZE DISTRIBUTION

AGRA
Earth & Environmental, Inc.

Project ROUTE 138, PHASE 2

By RN | Date 12–2–96

6–212–1052



Bor./Samp.No.	Elev. or Depth	Unified Soil Class	Description	Nat. Water	LL	PL	PI	SE	Max Dry Dens.	Opt. Moist.
108/9	33'–35'	SM	fine to medium SILTY SAND							

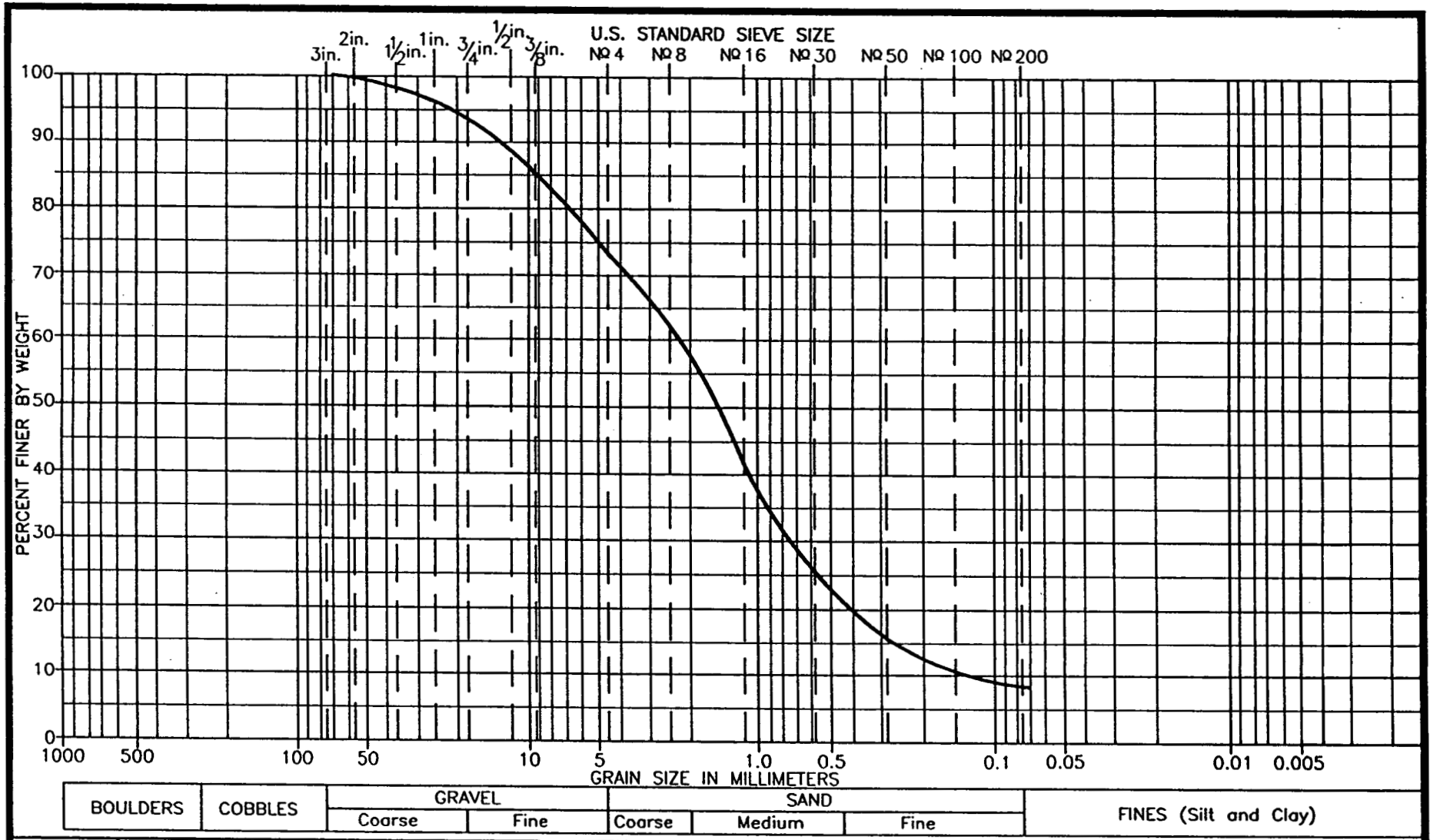
GRAIN SIZE DISTRIBUTION

AGRA
Earth & Environmental, Inc.

Project ROUTE 138, PHASE 2

By RN | Date 12–2–96

6–212–1052



Bor./Samp.No.	~~Elev.~~ or Depth	Unified Soil Class	Description	Nat. Water	LL	PL	PI	SE	Max Dry Dens.	Opt. Moist.
109/7	11'–12'	SW/SM	fine to medium SILTY SAND W/ GRAVEL							

GRAIN SIZE DISTRIBUTION

AGRA
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Project ROUTE 138, PHASE 2

By RN | Date 12–2–96

6–212–1052

APPENDIX C

SUMMARY OF SHEAR STRENGTH VALUES

A summary of Mohr-Coulomb shear strength parameters are presented in this Appendix. Samples from Borings B-1 through B-16, B1A and B2A were tested by John A. Sayers & Associates. Samples from Borings B-101 through B-109 and TP-2 were tested by AGRA Earth & Environmental, Inc. The shear strength values are differentiated by geologic unit, testing moisture conditions (natural moisture or submerged conditions), and stress/strain conditions (peak and ultimate strengths). An "average" set of strength parameters are also presented for each case. The average values are conservative estimates. Unusually high strength points have been omitted from the averaging process.

SUMMARY OF DIRECT SHEAR STRENGTH VALUES

Crowder Formation (Submerged)

Boring No. @ Depth or/ Sample No.	Material Description	Peak Strength		Ultimate Strength	
		c (psf)	ϕ (degrees)	c (psf)	ϕ (degrees)
B11 @ 5'	Silty fine SANDSTONE	608	31	360	29
B12 @ 20'	Silty SANDSTONE	1032	24	560	25
B13 @ 5'	Silty SANDSTONE with CLAY	230	39	0	40
B13 @ 15'	Silty SANDSTONE with CLAY	120	37	0	35
B103 / 1	Fine to medium CLAYEY SANDSTONE	100	34	100	34
B107 / 6	Fine to coarse SANDSTONE	200	44	100	36
B107 / 10	Fine to coarse SILTY SANDSTONE	300	44	0	44
B108 / 4	Fine to coarse SANDSTONE	75	57	75	39
B108 / 6	SANDSTONE	2900	15	500	33
B109 / 23	Fine to coarse SANDSTONE	300	47	100	40
	AVERAGE VALUE	**300**	**37**	**180**	**35**

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Earth & Environmental

SUMMARY OF DIRECT SHEAR STRENGTH VALUES

Crowder Formation (Natural Moisture)

Boring No. @ Depth or/ Sample No.	Material Description	Peak Strength		Ultimate Strength	
		c (psf)	ϕ (degrees)	c (psf)	ϕ (degrees)
B1A @ 10′	SILTY SANDSTONE	0	50	0	38
B1A @ 20′	SILTY SANDSTONE	1636	44	0	39
B1A @ 40′	SANDSTONE with GRAVEL	2420	28	250	34
B2A @ 10′	SILTY fine to medium SANDSTONE	1956	46	0	38
B2A @ 15′	SILTY SANDSTONE with CLAY	1604	47	530	30
B2A @ 20′	Fine to medium SANDSTONE with GRAVEL	0	50	0	36
B2A @ 35′	Fine to medium SANDSTONE with GRAVEL	0	51	0	42
B2A @ 45′	Fine to coarse SANDSTONE	124	49	400	36
B107 / 10	Fine to coarse SILTY SANDSTONE	1200	48	200	41
B108 / 15	SILTY SANDSTONE	400	47	400	42
	AVERAGE VALUE	**900**	**46**	**180**	**37**

Recycled Paper

SUMMARY OF DIRECT SHEAR STRENGTH VALUES

Terrace Deposits (Submerged)

Boring No./ Sample No.	Material Description	Peak Strength		Ultimate Strength	
		c (psf)	ϕ (degrees)	c (psf)	ϕ (degrees)
B109 / 1'	Fine to coarse SILTY SAND	0	47	0	35
B109 / 3'	Fine to coarse SILTY SAND	350	40	150	38
B109 / 11'	Fine to coarse SAND with GRAVEL	1090	48	800	48
	AVERAGE VALUE	**175**	**41**	**75**	**36**

Terrace Deposits (Natural Moisture)

Boring No./ Sample No.	Material Description	Peak Strength		Ultimate Strength	
		c (psf)	ϕ (degrees)	c (psf)	ϕ (degrees)
B109 / 6'	Medium to coarse SILTY SAND	300	50	300	42
B109 / 8'	Medium to coarse SAND with GRAVEL	850	45	330	45
B109 / 11'	Fine to coarse SAND with GRAVEL	700	42	500	42
	AVERAGE VALUE	**600**	**45**	**350**	**44**

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AGRA
Earth & Environmental

SUMMARY OF DIRECT SHEAR STRENGTH VALUES

Older Alluvium/Fanglomerate (Submerged)

Boring or Test Pit No. @ Depth or/ Sample No.	Material Description	Peak Strength		Ultimate Strength	
		c (psf)	ϕ (degrees)	c (psf)	ϕ (degrees)
B5 @ 10'	SILTY SAND	152	35	200	33
B10 @ 10'	SILTY to CLAYEY SAND	384	36	100	30
B101 / 8	Fine to coarse SAND	500	45	320	37.5
B101 / 10	Fine to coarse SAND	900	47	370	47
TP 2/2	Fine to medium SILTY SAND	50	41	50	40
	AVERAGE VALUE	**250**	**39**	**150**	**35**

Remolded Samples (Submerged)

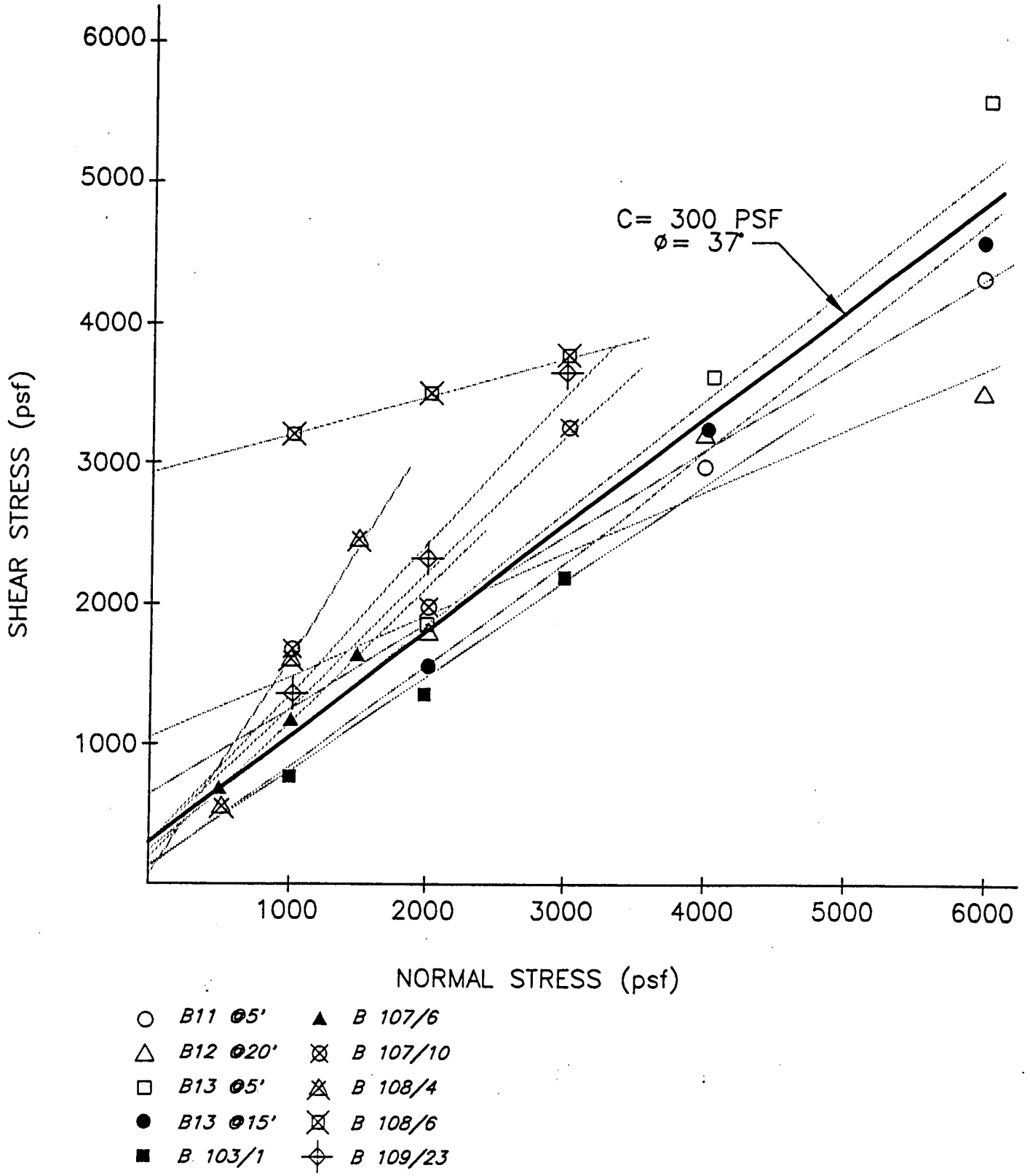
Boring No. @ Depth or/ Sample No.	Material Description	Peak Strength		Ultimate Strength	
		c (psf)	ϕ (degrees)	c (psf)	ϕ (degrees)
B1A @ 10'	Crowder SANDSTONE	980	25	100	32
B1A @ 20'	Crowder fine SANDSTONE	1220	30	240	32
B108 / 2	Crowder CLAYEY SANDSTONE	300	36	50	36
B109 / 2	Terrace medium to coarse SAND	300	41	0	38
	AVERAGE VALUES	**700**	**33**	**100**	**34**

Recycled Paper

AGRA
Earth & Environmental

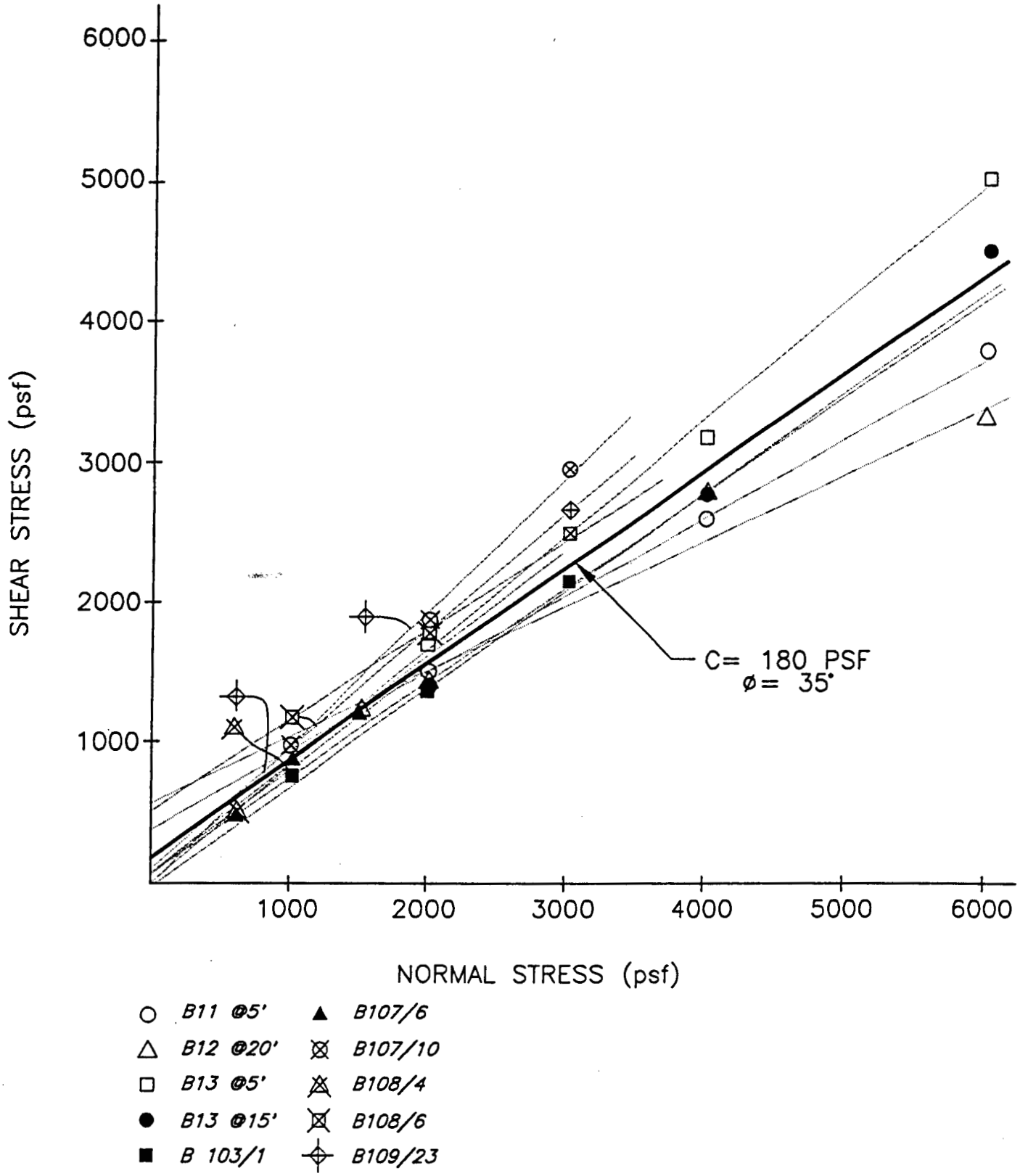
SUMMARY OF DIRECT SHEAR TEST DATA

CROWDER FORMATION
SUBMERGED – PEAK STRENGTH



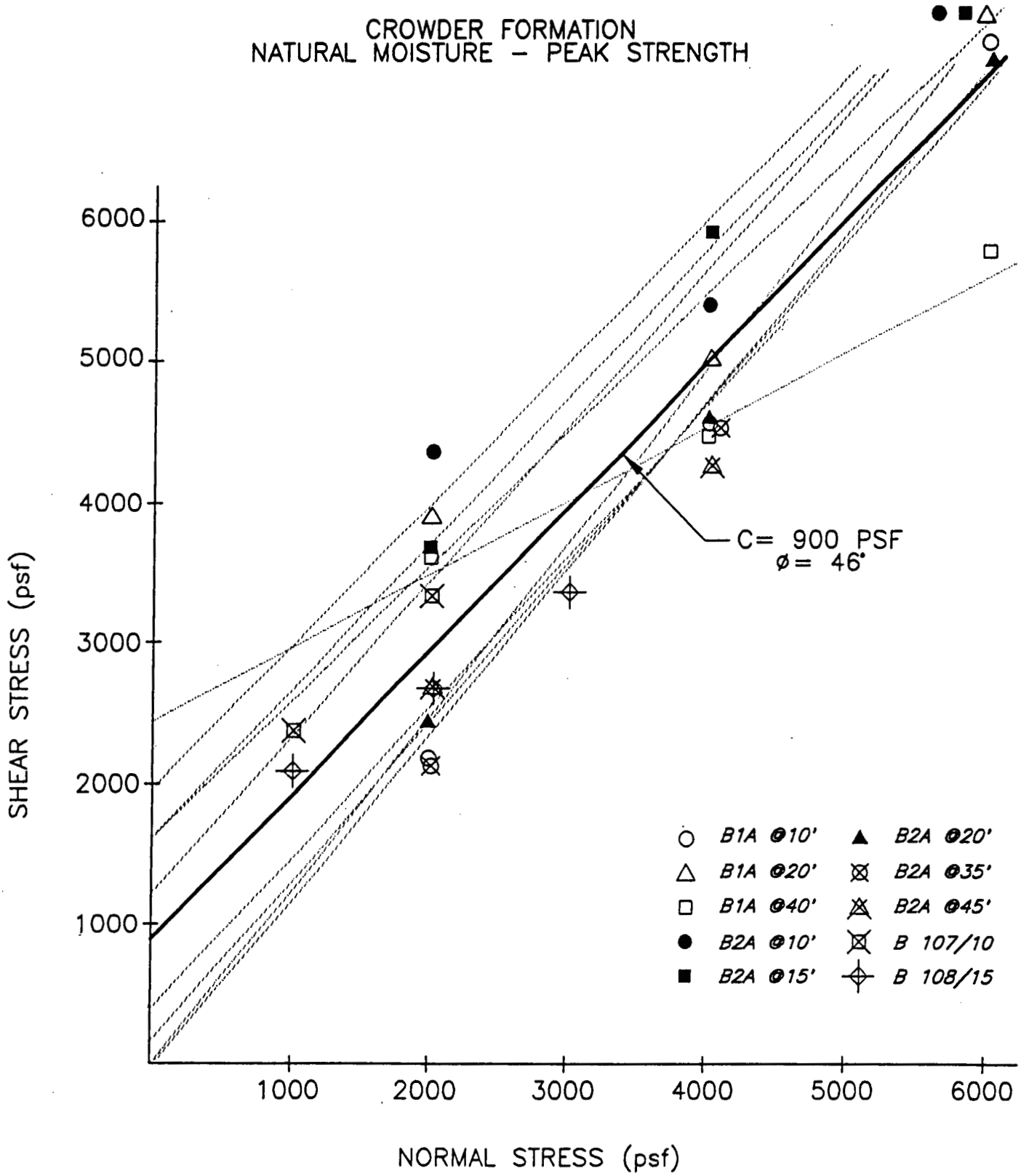
SUMMARY OF DIRECT SHEAR TEST DATA

CROWDER FORMATION
SUBMERGED – ULTIMATE STRENGTH



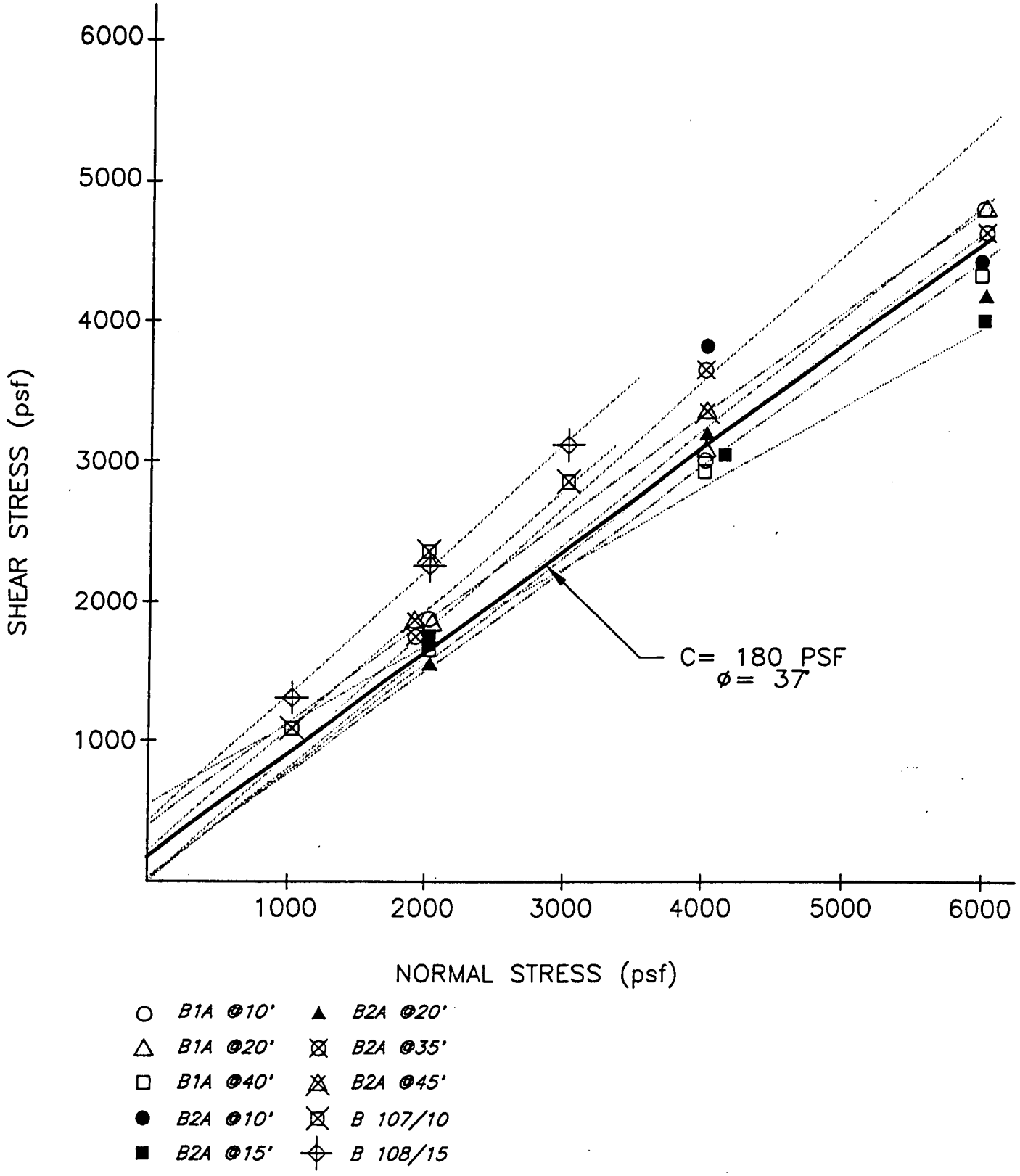
SUMMARY OF DIRECT SHEAR TEST DATA

CROWDER FORMATION
NATURAL MOISTURE – PEAK STRENGTH



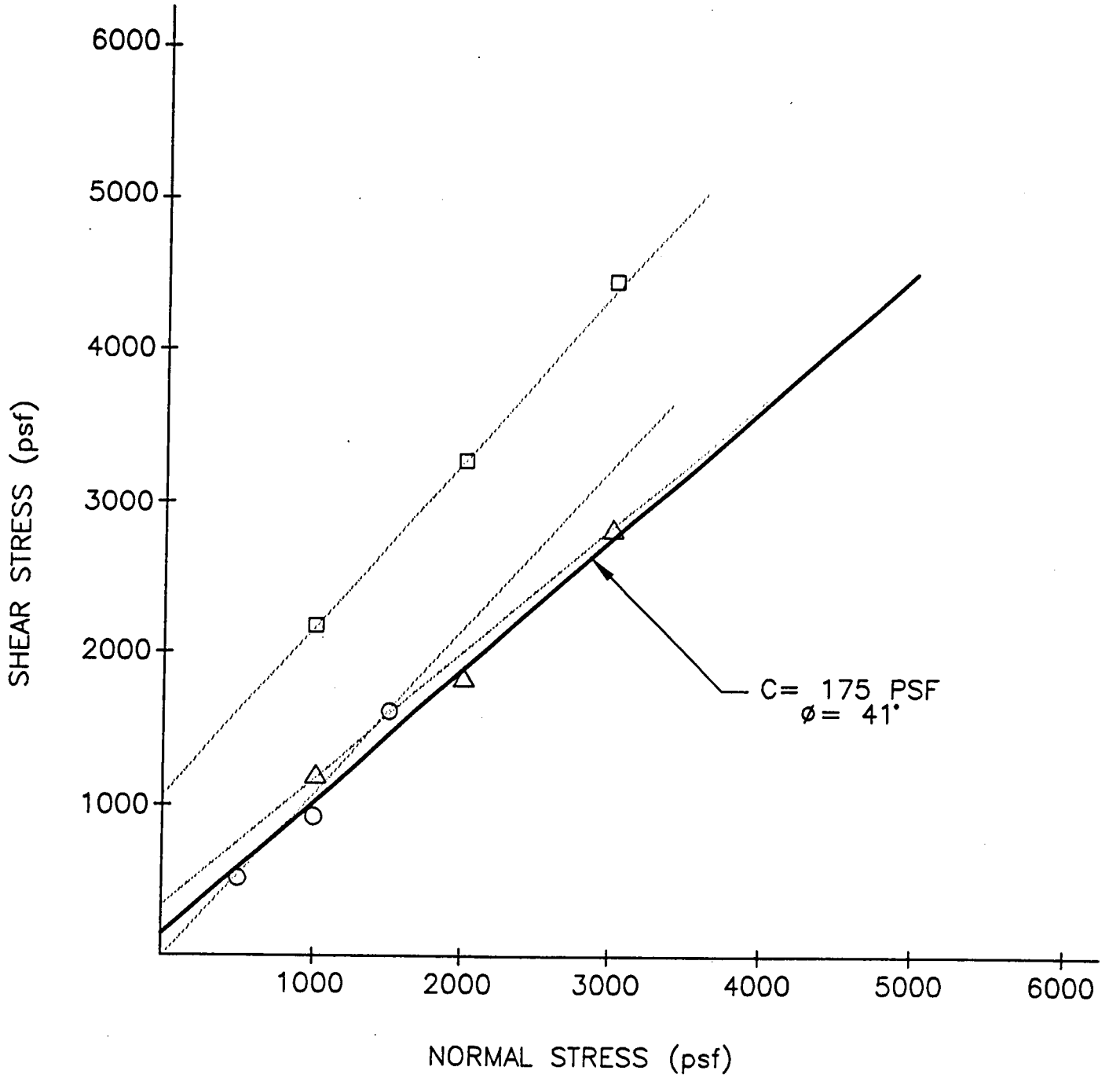
SUMMARY OF DIRECT SHEAR TEST DATA

CROWDER FORMATION
NATURAL MOISTURE – ULTIMATE STRENGTH



SUMMARY OF DIRECT SHEAR TEST DATA

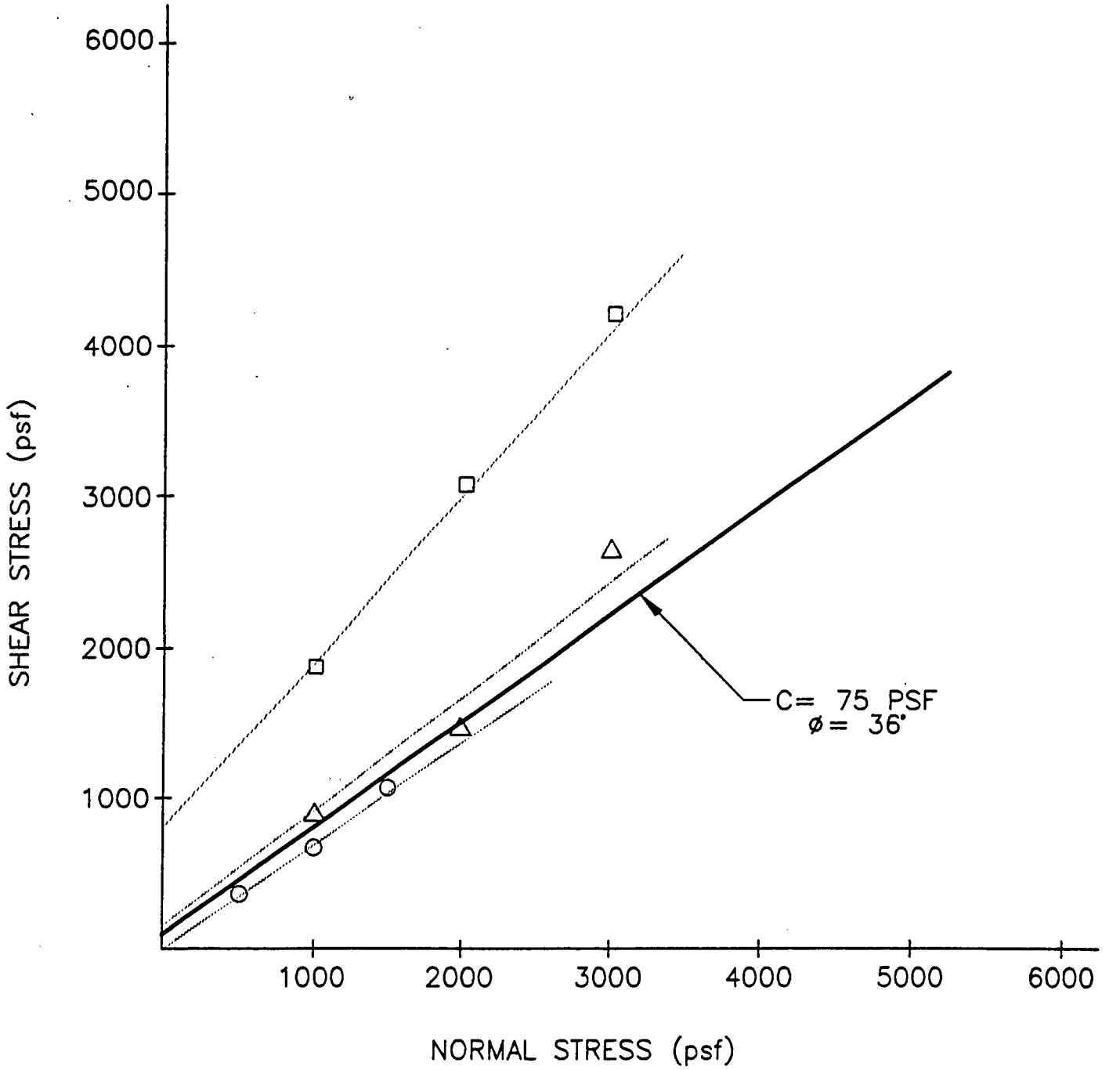
TERRACE DEPOSITS
SUBMERGED – PEAK STRENGTH



○ *B 109/1*
△ *B 109/3*
□ *B 109/11*

SUMMARY OF DIRECT SHEAR TEST DATA

TERRACE DEPOSITS
SUBMERGED – ULTIMATE STRENGTH

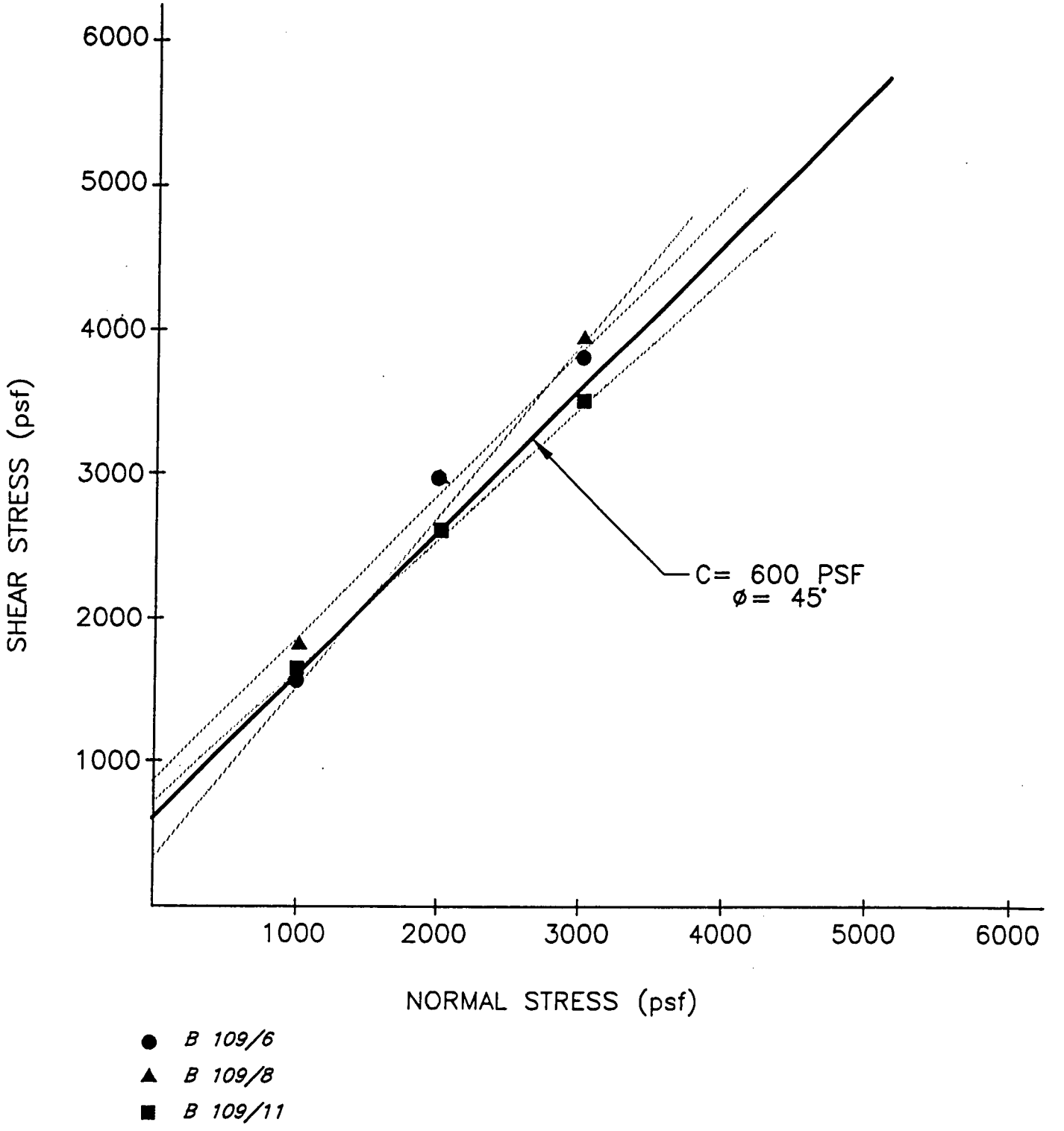


○ *B 109/1*
△ *B 109/3*
□ *B 109/11*

LE: 6-1052Y

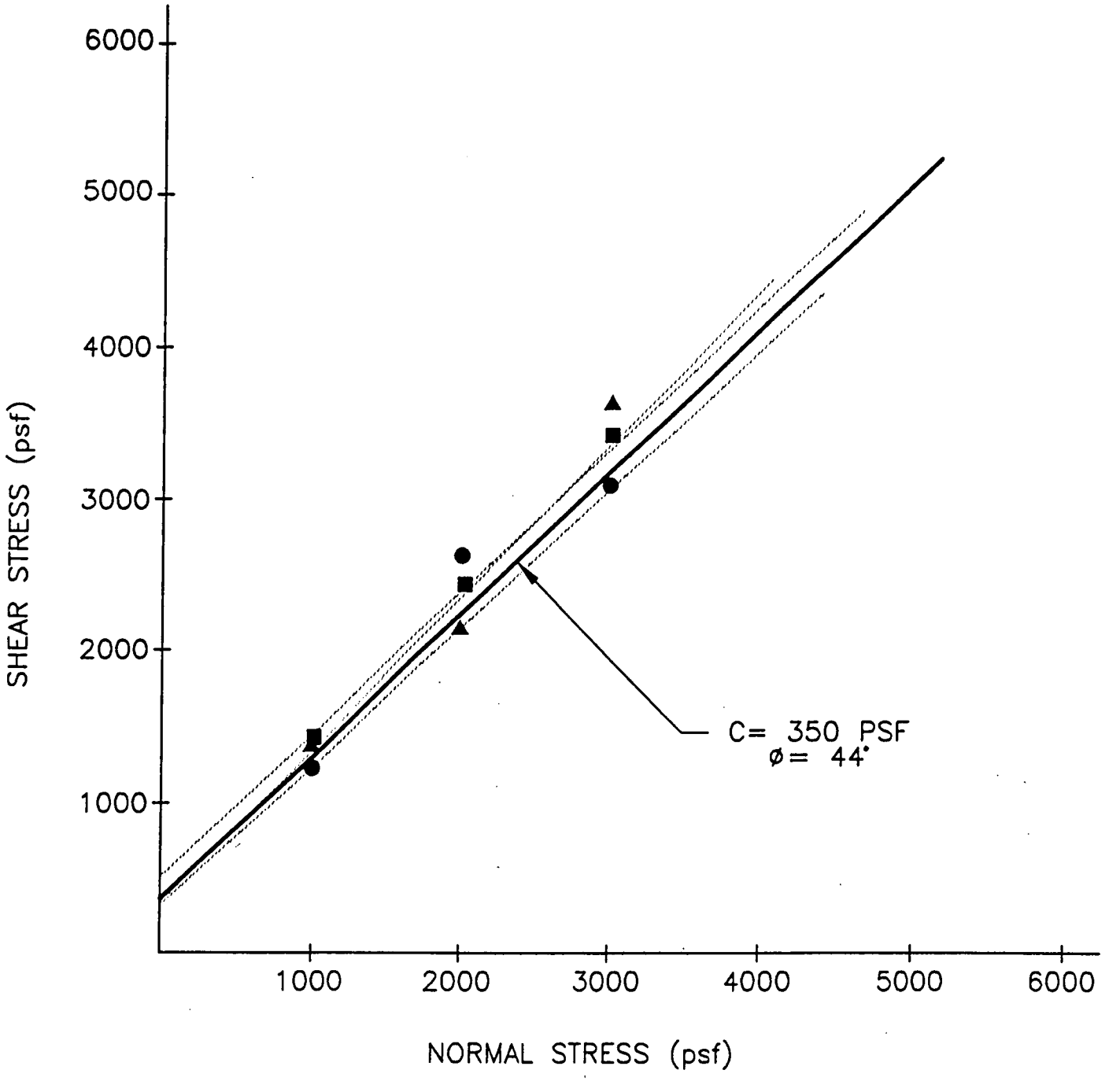
SUMMARY OF DIRECT SHEAR TEST DATA

TERRACE DEPOSITS
NATURAL MOISTURE – PEAK STRENGTH



SUMMARY OF DIRECT SHEAR TEST DATA

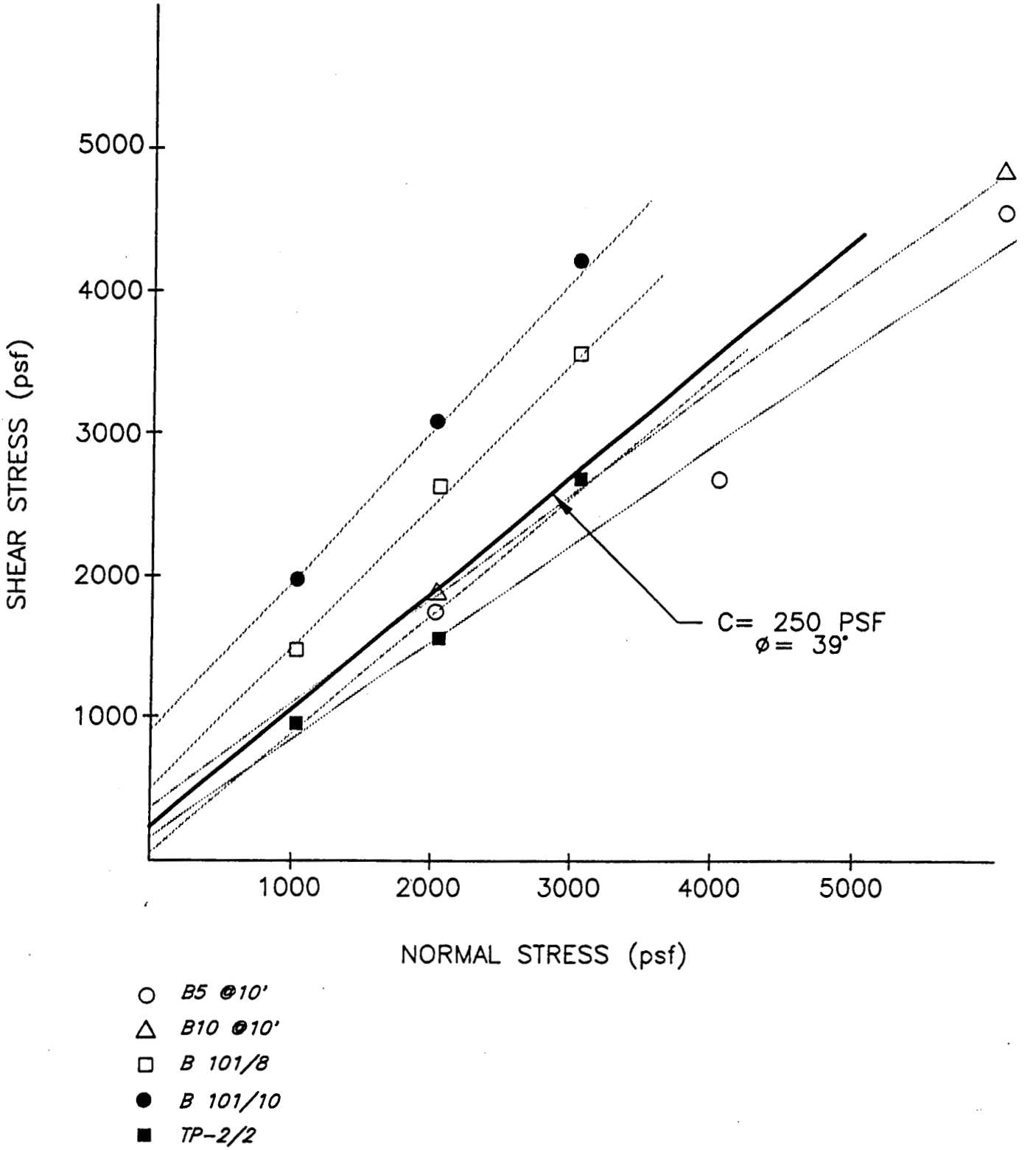
TERRACE DEPOSITS
NATURAL MOISTURE – ULTIMATE STRENGTH



● *B 109/6*
▲ *B 109/8*
■ *B 109/11*

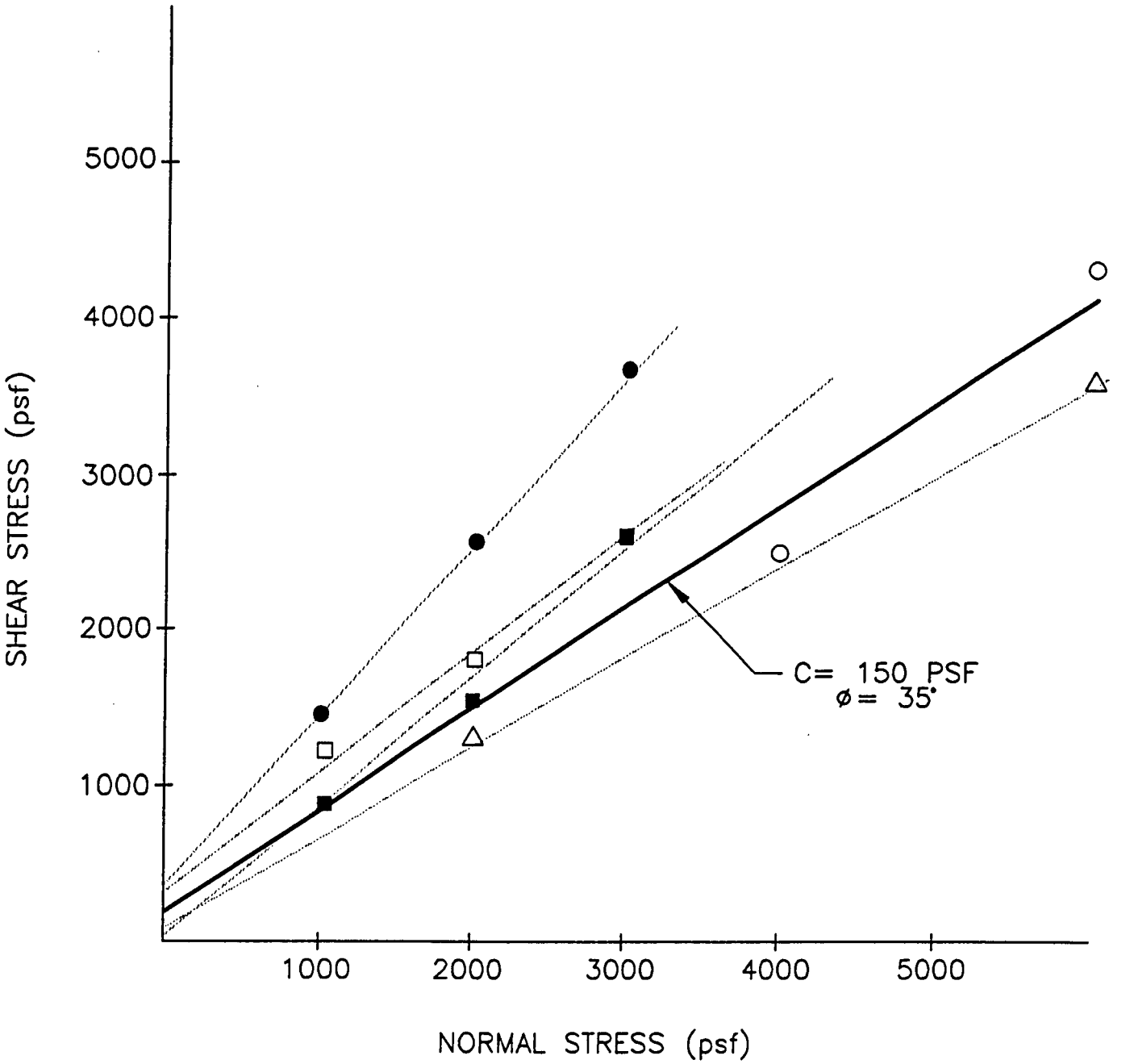
SUMMARY OF DIRECT SHEAR TEST DATA

FANGLOMERATE – OLDER ALLUVIUM
SUBMERGED – PEAK STRENGTH



SUMMARY OF DIRECT SHEAR TEST DATA

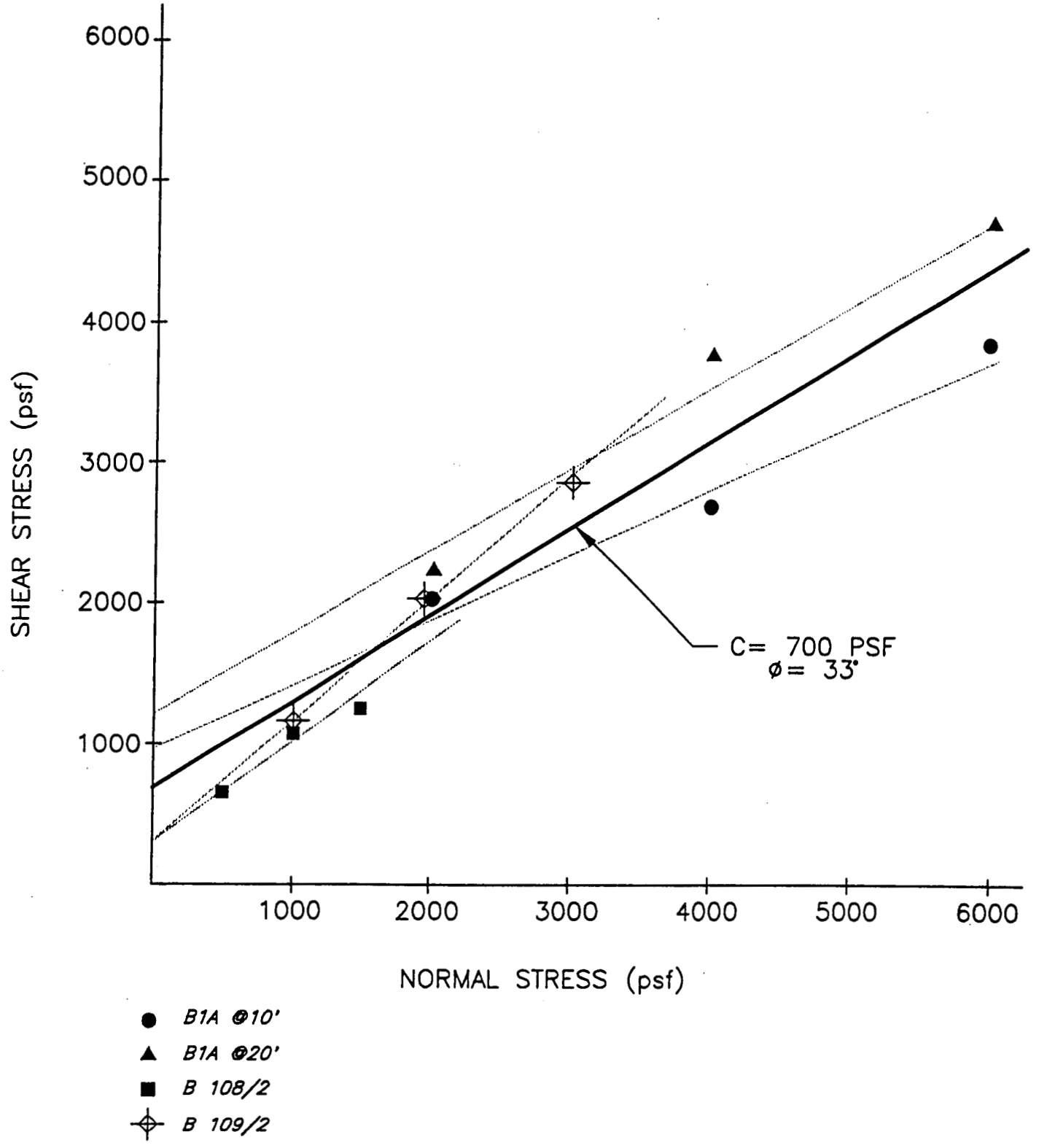
FANGLOMERATE – OLDER ALLUVIUM
SUBMERGED – ULTIMATE STRENGTH



○ *B5 @10'*
△ *B10 @10'*
□ *B 101/8*
● *B 101/10*
■ *TP-2/2*

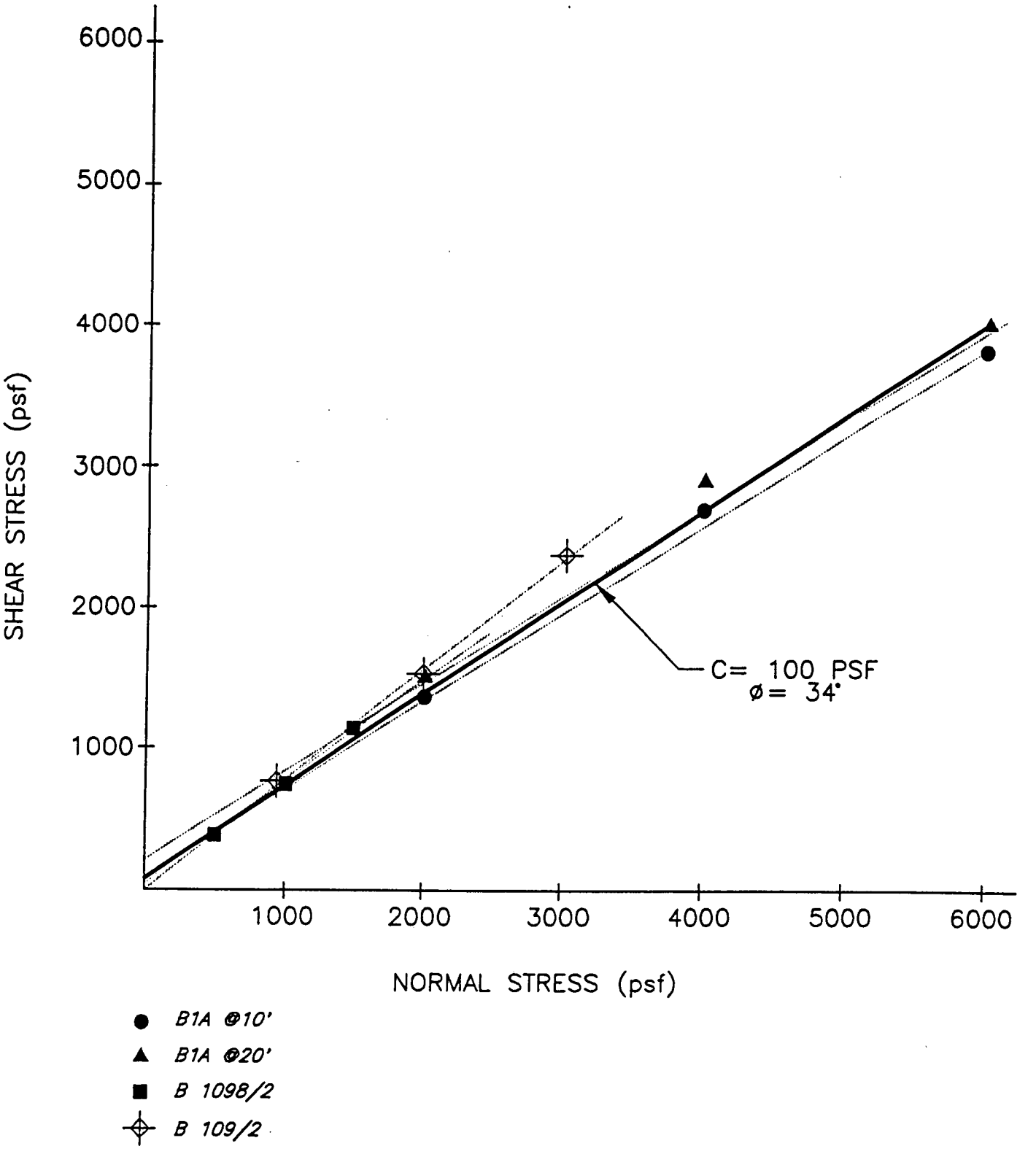
SUMMARY OF DIRECT SHEAR TEST DATA

REMOLDED MATERIAL
SUBMERGED – PEAK STRENGTH



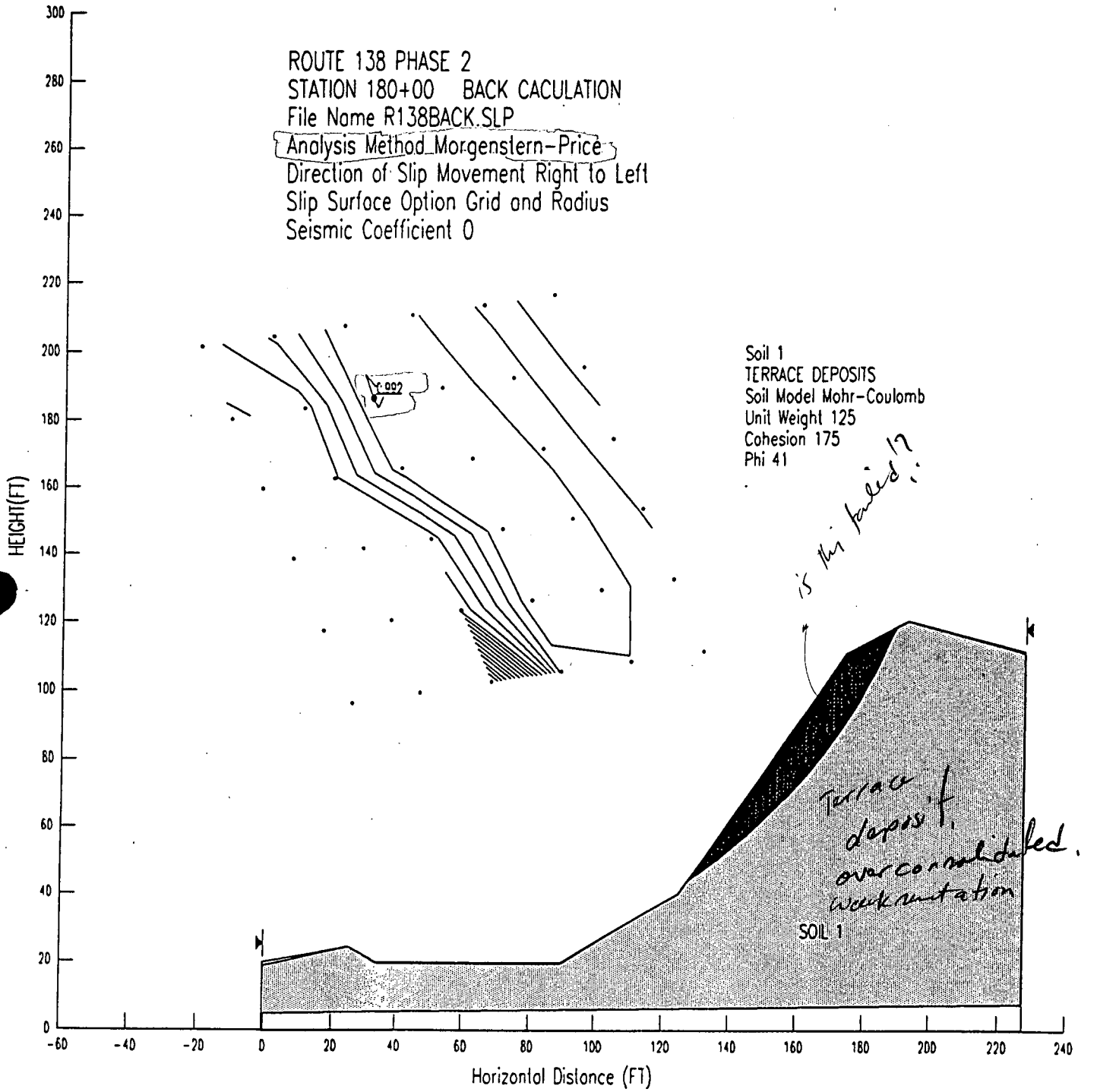
SUMMARY OF DIRECT SHEAR TEST DATA

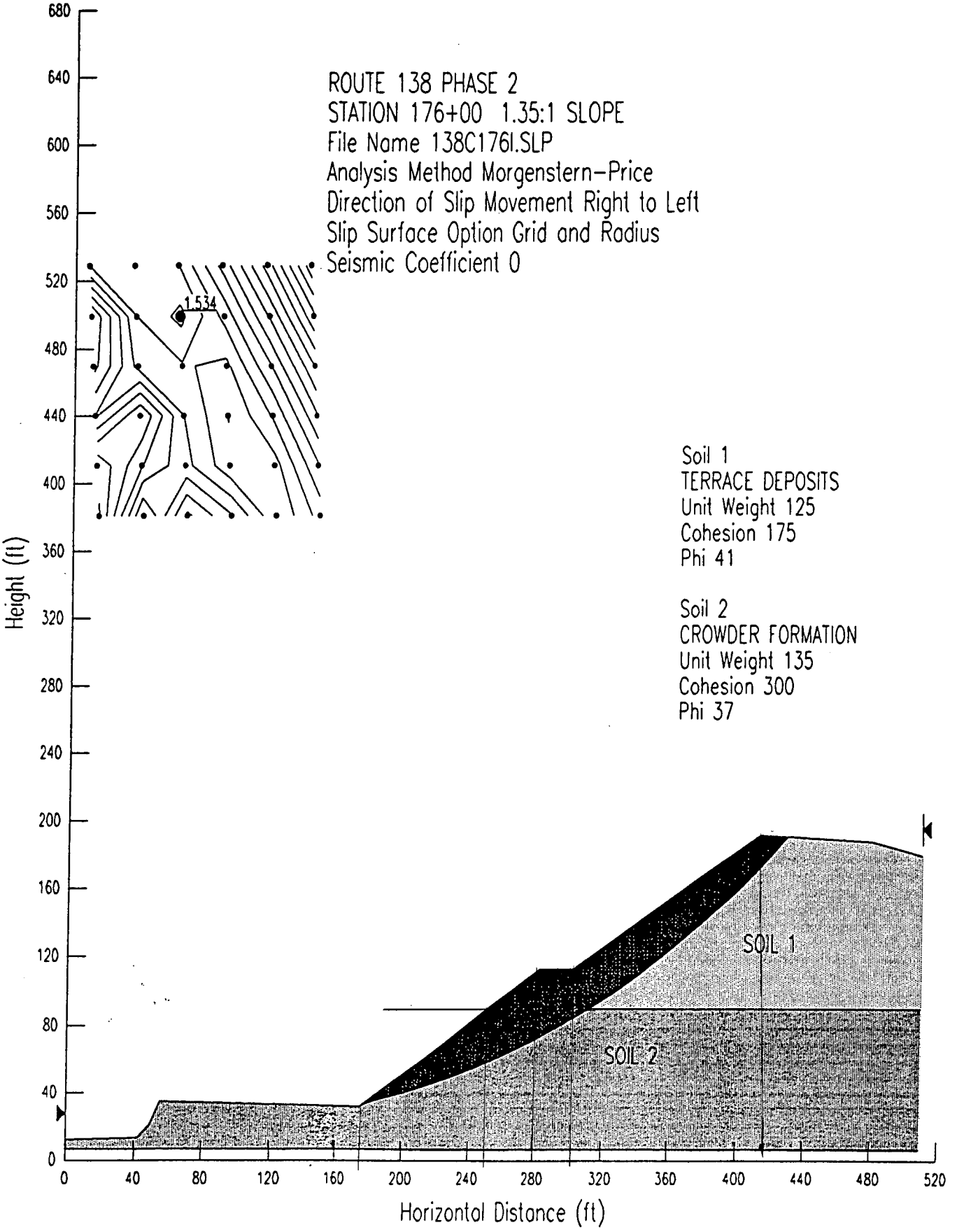
REMOLDED MATERIAL
SUBMERGED – ULTIMATE STRENGTH

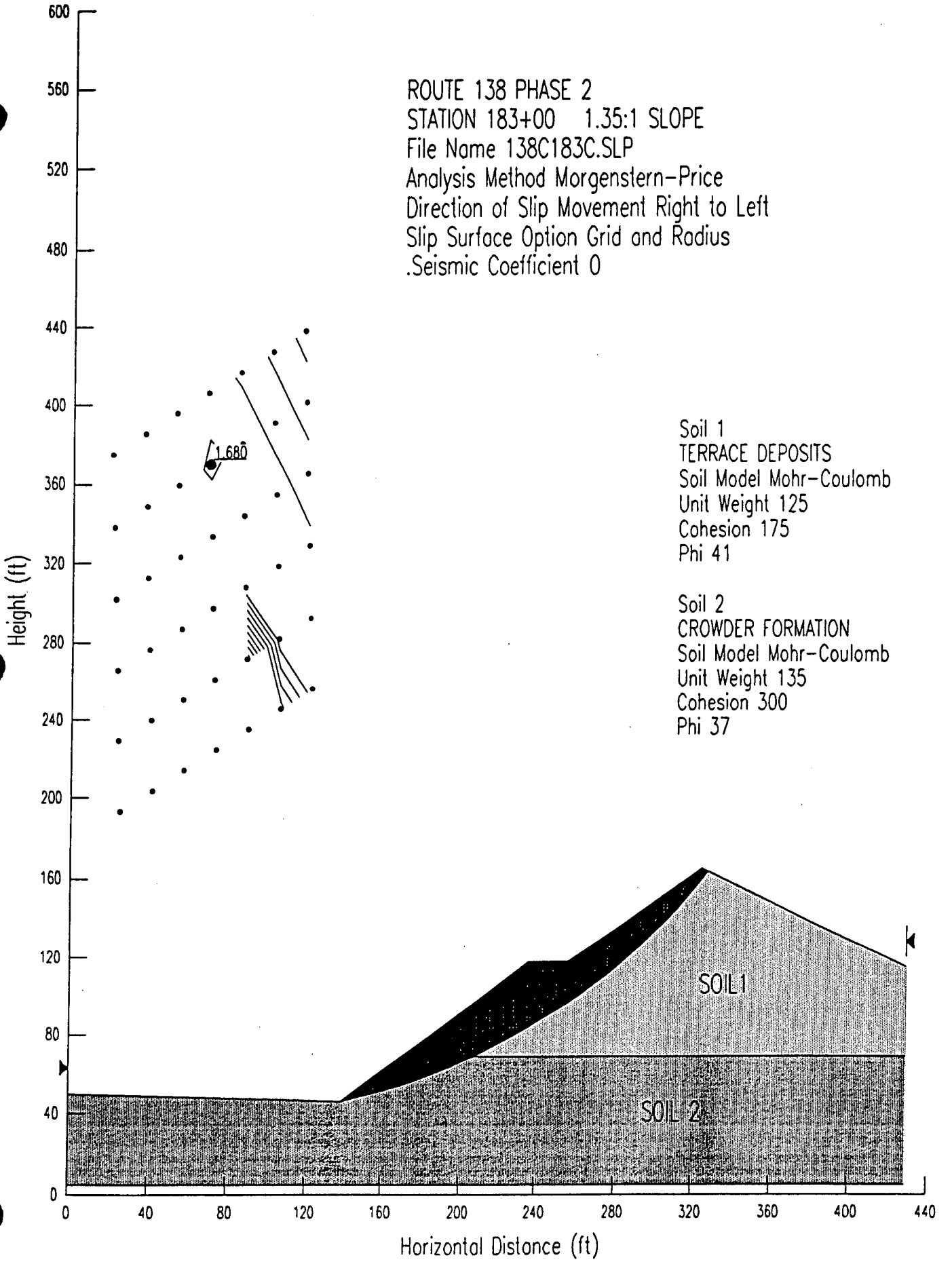


APPENDIX D

SLOPE STABILITY CALCULATIONS


ROUTE 138 PHASE 2
STATION 180+00 BACK CACULATION
File Name R138BACK.SLP
Analysis Method Morgenstern-Price
Direction of Slip Movement Right to Left
Slip Surface Option Grid and Radius
Seismic Coefficient 0
Soil 1
TERRACE DEPOSITS
Soil Model Mohr-Coulomb
Unit Weight 125
Cohesion 175
Phi 41
is this failed?
Terrace deposit, overconsolidated, weak cementation
SOIL 1
HEIGHT(FT)
Horizontal Distance (FT)


ROUTE 138 PHASE 2
STATION 176+00 1.35:1 SLOPE
File Name 138C176I.SLP
Analysis Method Morgenstern-Price
Direction of Slip Movement Right to Left
Slip Surface Option Grid and Radius
Seismic Coefficient 0
1.534
Soil 1
TERRACE DEPOSITS
Unit Weight 125
Cohesion 175
Phi 41
Soil 2
CROWDER FORMATION
Unit Weight 135
Cohesion 300
Phi 37
SOIL 1
SOIL 2
Height (ft)
Horizontal Distance (ft)
0
40
80
120
160
200
240
280
320
360
400
440
480
520
560
600
640
680


ROUTE 138 PHASE 2
STATION 183+00 1.35:1 SLOPE
File Name 138C183C.SLP
Analysis Method Morgenstern-Price
Direction of Slip Movement Right to Left
Slip Surface Option Grid and Radius
.Seismic Coefficient 0
Soil 1
TERRACE DEPOSITS
Soil Model Mohr-Coulomb
Unit Weight 125
Cohesion 175
Phi 41
Soil 2
CROWDER FORMATION
Soil Model Mohr-Coulomb
Unit Weight 135
Cohesion 300
Phi 37
1.680
SOIL1
SOIL 2
Height (ft)
Horizontal Distance (ft)
0
40
80
120
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240
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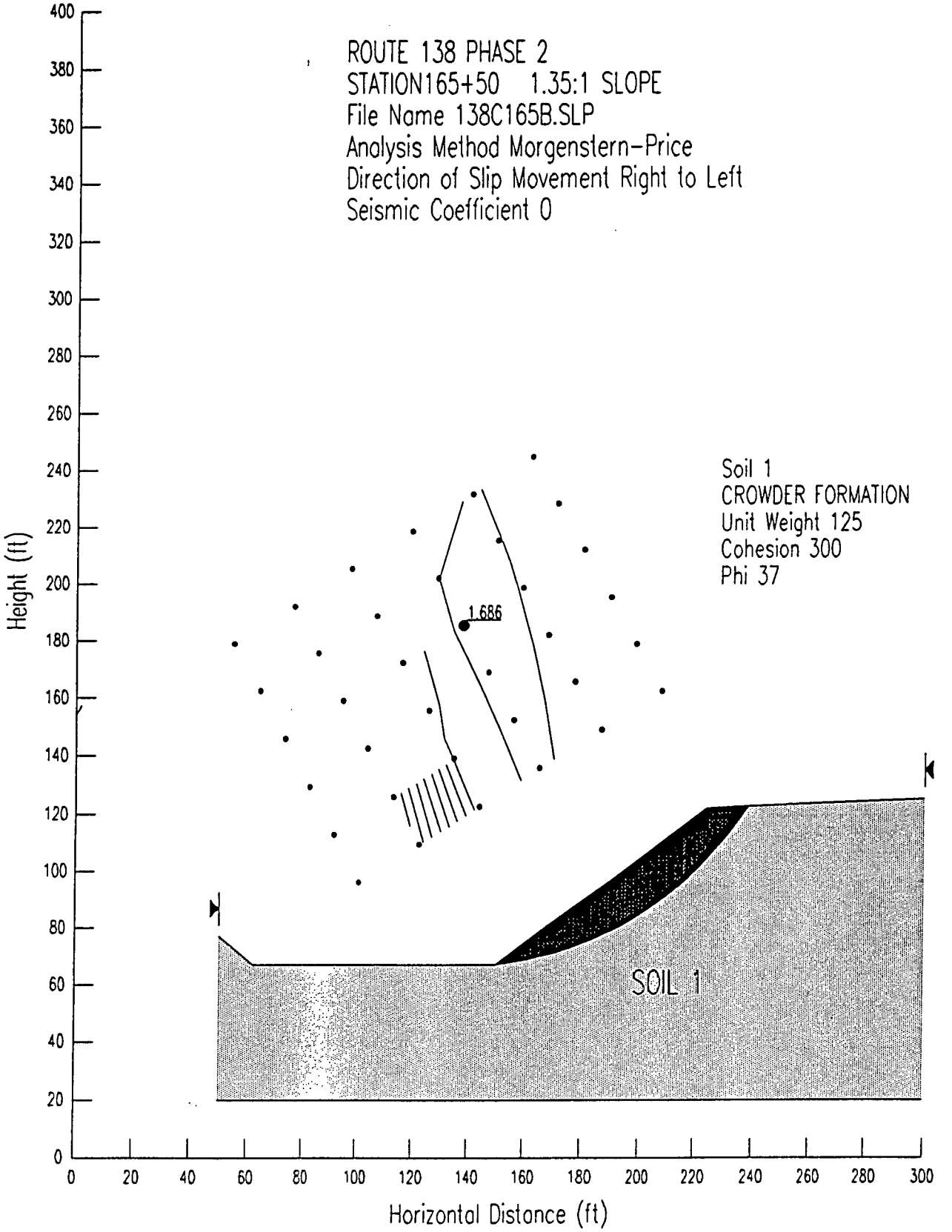
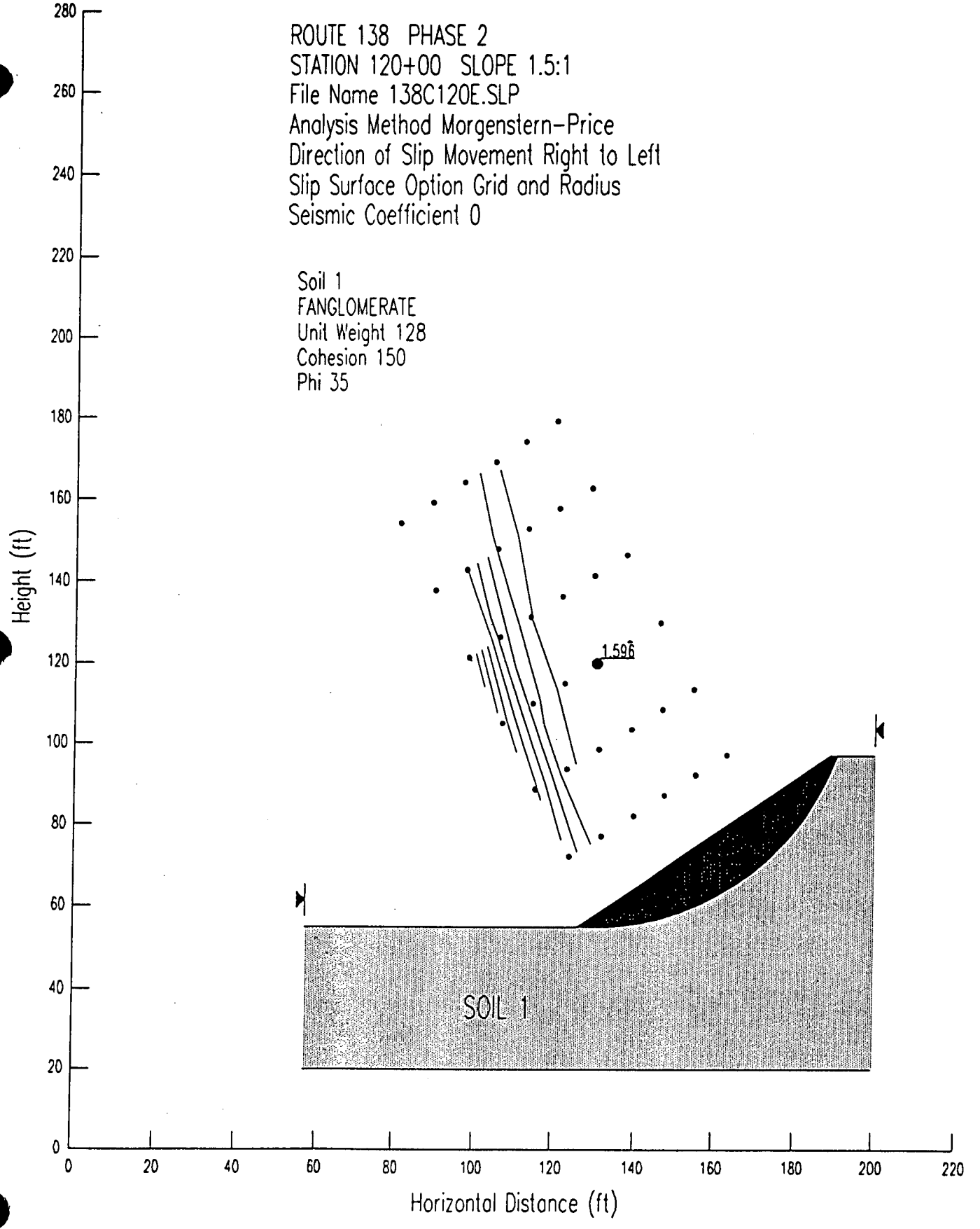
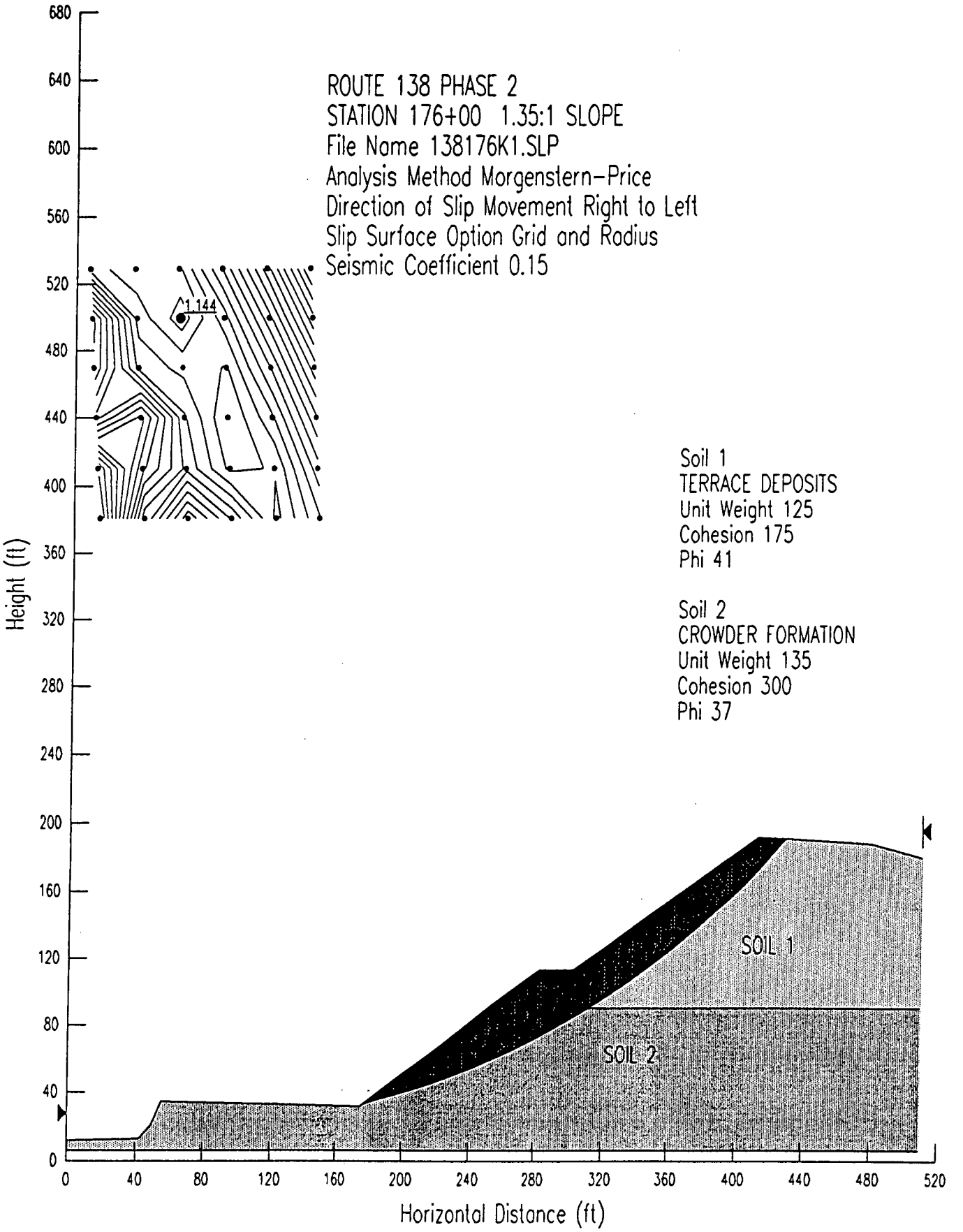
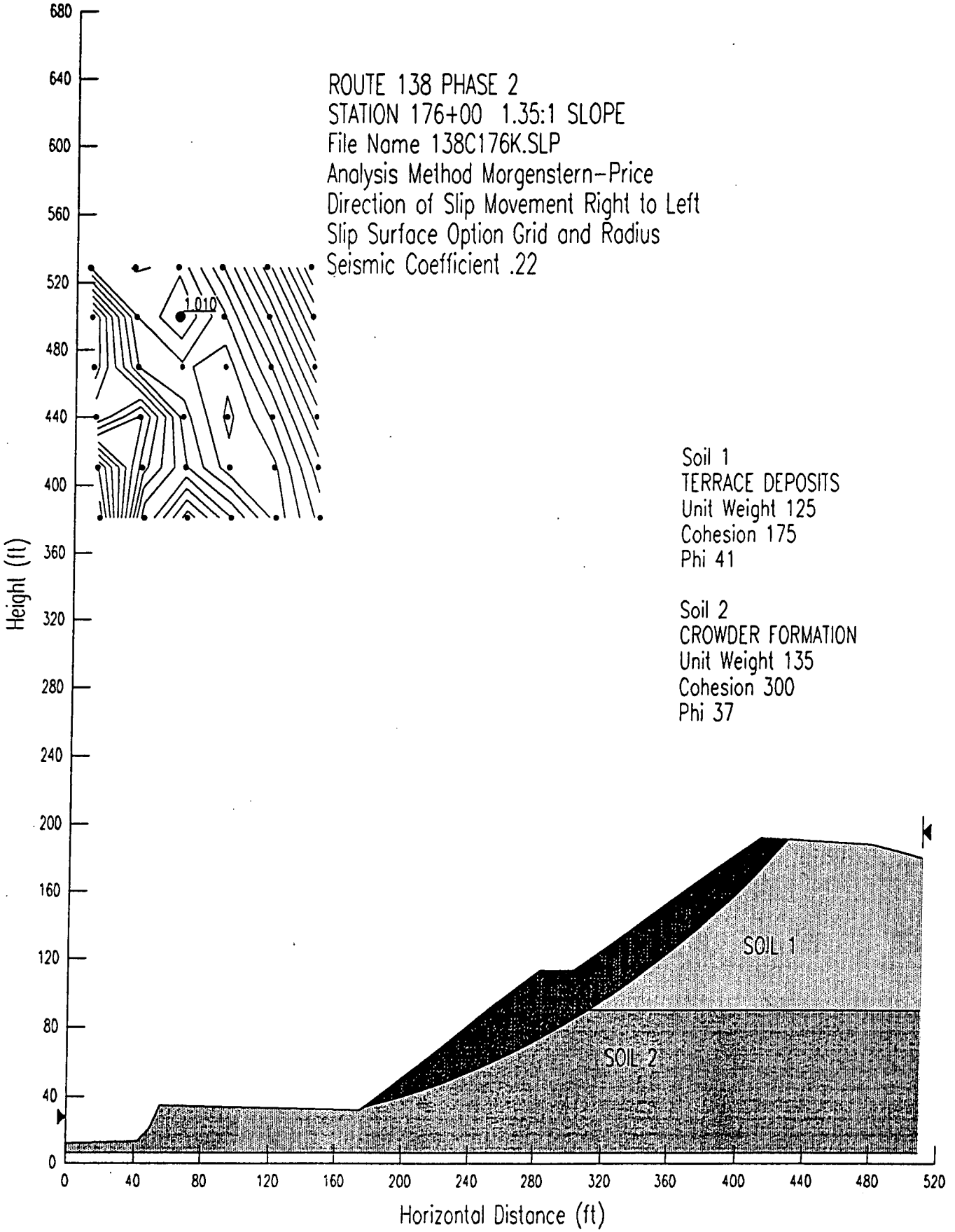
320
360
400
440
480
520
560
600


ROUTE 138 PHASE 2
STATION165+50 1.35:1 SLOPE
File Name 138C165B.SLP
Analysis Method Morgenstern-Price
Direction of Slip Movement Right to Left
Seismic Coefficient 0
Soil 1
CROWDER FORMATION
Unit Weight 125
Cohesion 300
Phi 37
1.686
SOIL 1
Height (ft)
Horizontal Distance (ft)
0
20
40
60
80
100
120
140
160
180
200
220
240
260
280
300
320
340
360
380
400


ROUTE 138 PHASE 2
STATION 120+00 SLOPE 1.5:1
File Name 138C120E.SLP
Analysis Method Morgenstern-Price
Direction of Slip Movement Right to Left
Slip Surface Option Grid and Radius
Seismic Coefficient 0
Soil 1
FANGLOMERATE
Unit Weight 128
Cohesion 150
Phi 35
1.596
SOIL 1
Height (ft)
Horizontal Distance (ft)
0
20
40
60
80
100
120
140
160
180
200
220
260
280


ROUTE 138 PHASE 2
STATION 176+00 1.35:1 SLOPE
File Name 138176K1.SLP
Analysis Method Morgenstern-Price
Direction of Slip Movement Right to Left
Slip Surface Option Grid and Radius
Seismic Coefficient 0.15
1.144
Soil 1
TERRACE DEPOSITS
Unit Weight 125
Cohesion 175
Phi 41
Soil 2
CROWDER FORMATION
Unit Weight 135
Cohesion 300
Phi 37
SOIL 1
SOIL 2
Height (ft)
Horizontal Distance (ft)
0
40
80
120
160
200
240
280
320
360
400
440
480
520
560
600
640
680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STATION 176+00 1.35:1 SLOPE
File Name 138C176K.SLP
Analysis Method Morgenstern-Price
Direction of Slip Movement Right to Left
Slip Surface Option Grid and Radius
Seismic Coefficient .22
1.010
Soil 1
TERRACE DEPOSITS
Unit Weight 125
Cohesion 175
Phi 41
Soil 2
CROWDER FORMATION
Unit Weight 135
Cohesion 300
Phi 37
SOIL 1
SOIL 2
Height (ft)
Horizontal Distance (ft)
0
40
80
120
160
200
240
280
320
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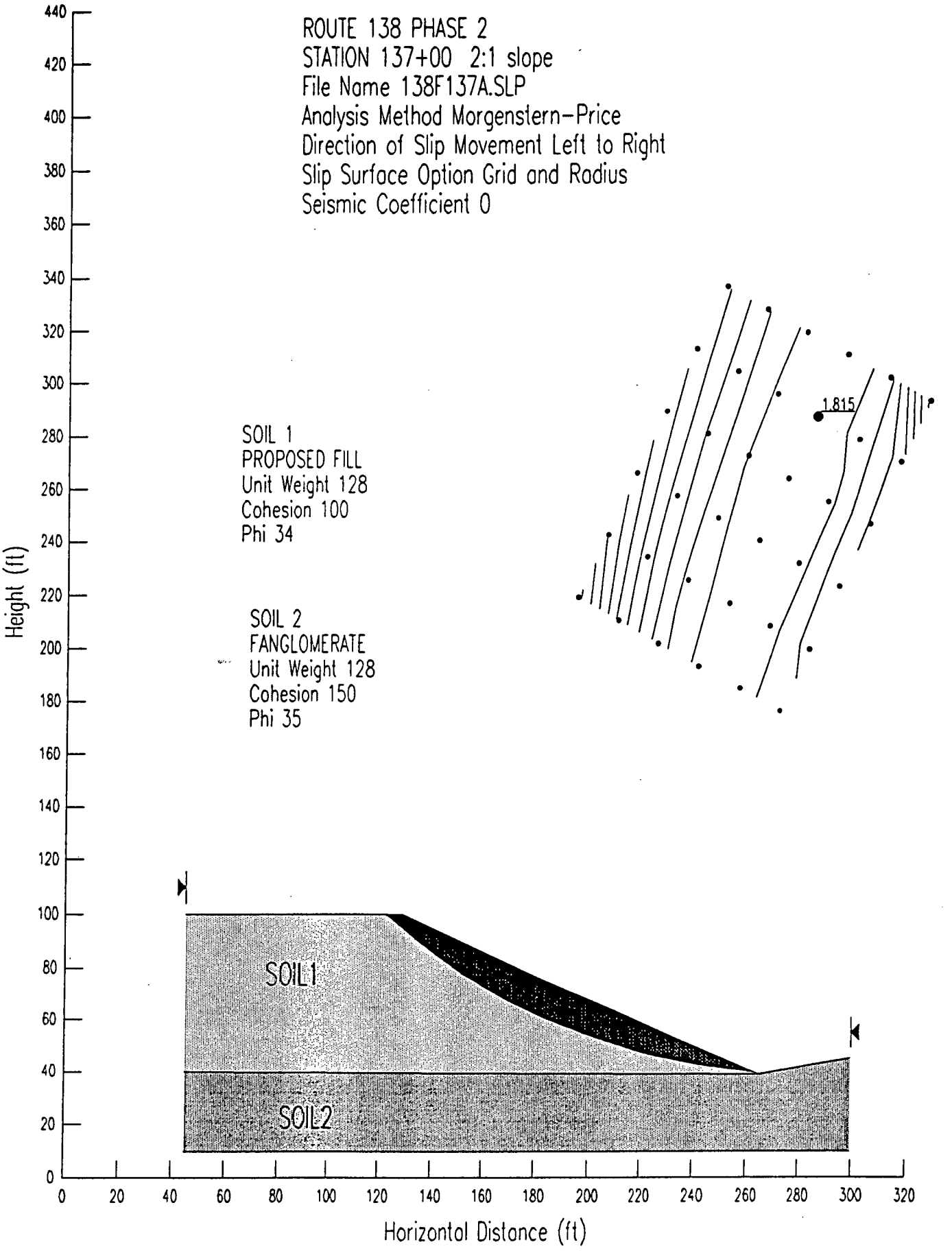
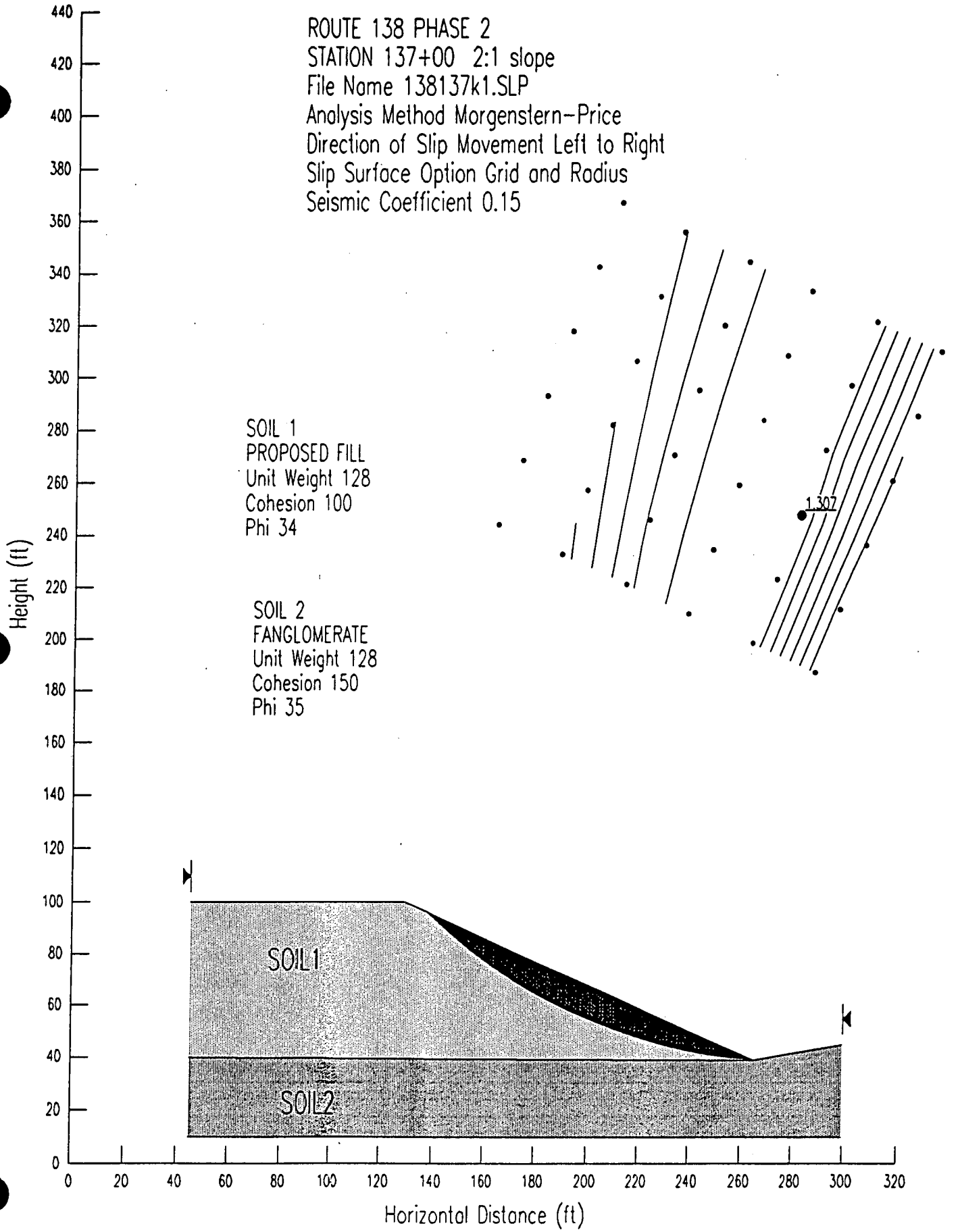
400
440
480
520
560
600
640
680


ROUTE 138 PHASE 2
STATION 137+00 2:1 slope
File Name 138F137A.SLP
Analysis Method Morgenstern-Price
Direction of Slip Movement Left to Right
Slip Surface Option Grid and Radius
Seismic Coefficient 0
SOIL 1
PROPOSED FILL
Unit Weight 128
Cohesion 100
Phi 34
SOIL 2
FANGLOMERATE
Unit Weight 128
Cohesion 150
Phi 35
1.815
SOIL1
SOIL2
Height (ft)
Horizontal Distance (ft)
0
20
40
60
80
100
120
140
160
180
200
220
240
260
280
300
320
340
360
380
400
420
440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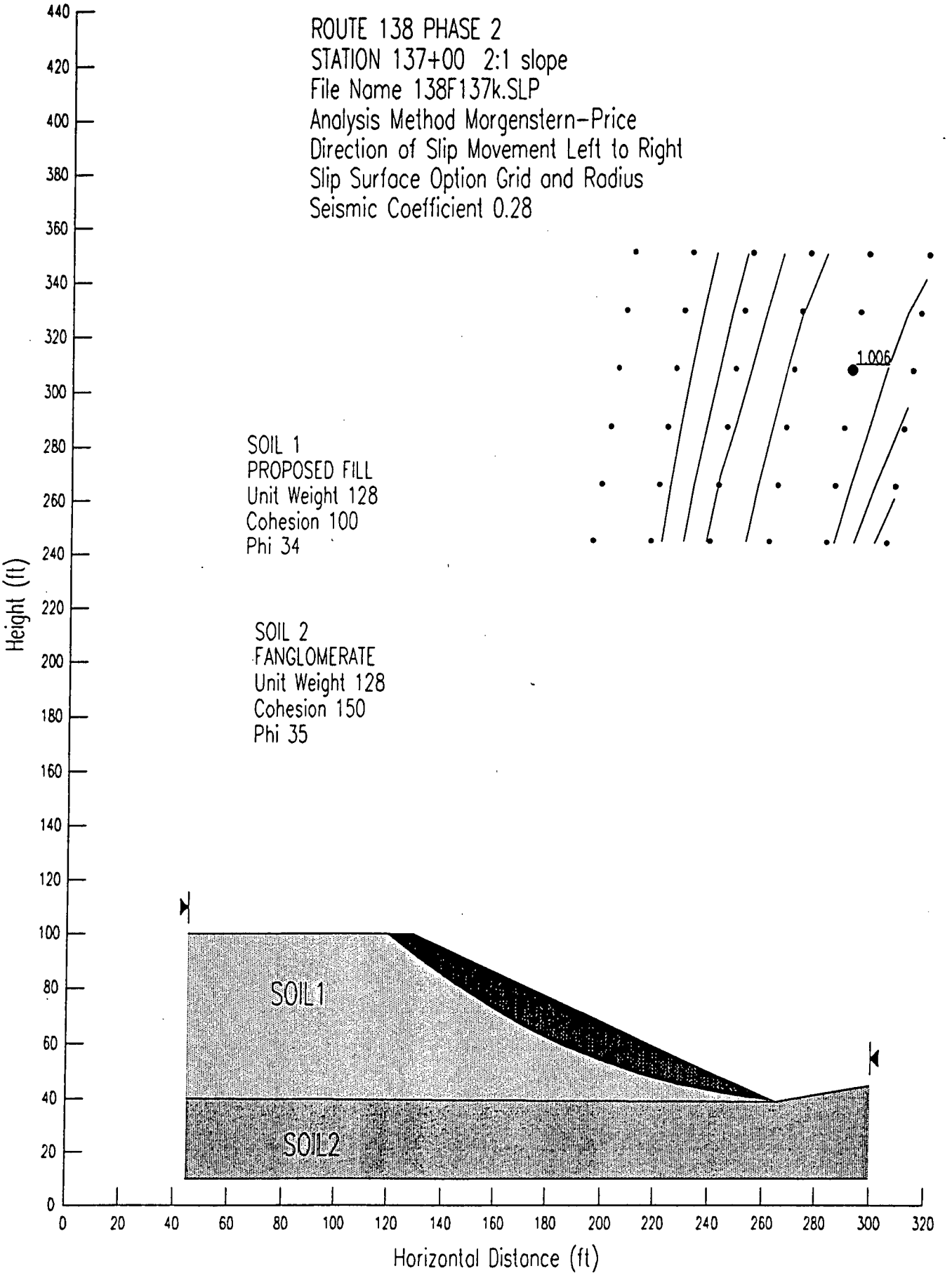
STATION 137+00 2:1 slope
File Name 138137k1.SLP
Analysis Method Morgenstern-Price
Direction of Slip Movement Left to Right
Slip Surface Option Grid and Radius
Seismic Coefficient 0.15
SOIL 1
PROPOSED FILL
Unit Weight 128
Cohesion 100
Phi 34
SOIL 2
FANGLOMERATE
Unit Weight 128
Cohesion 150
Phi 35
1.307
SOIL1
SOIL2
Height (ft)
Horizontal Distance (ft)
0
20
40
60
80
100
120
140
160
180
200
220
240
260
280
300
320
340
360
380
400
420
440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STATION 137+00 2:1 slope
File Name 138F137k.SLP
Analysis Method Morgenstern-Price
Direction of Slip Movement Left to Right
Slip Surface Option Grid and Radius
Seismic Coefficient 0.28
1.006
SOIL 1
PROPOSED FILL
Unit Weight 128
Cohesion 100
Phi 34
SOIL 2
FANGLOMERATE
Unit Weight 128
Cohesion 150
Phi 35
SOIL1
SOIL2
Height (ft)
Horizontal Distance (ft)
0
20
40
60
80
100
120
140
160
180
200
220
240
260
280
300
320
340
360
380
400
420
440

SURFICIAL SLOPE STABILITY

PROJECT **ROUTE 138**

CROWDER @ TERRACE DEPOSITS 1.35:1

JOB NO **6-212-105200**

COHESION (psf)	PHI ANGLE (deg)	TOTAL UNIT WT(pcf)	DEPTH TO FAILURE (ft)	HT OF WATER (ft)*	SLOPE ANGLE (deg)	FACTOR OF SAFETY
180	35	141	1	1	36.5	3.20
180	35	141	2	1	36.5	1.86
180	35	141	3	1	36.5	1.42
180	35	141	4	1	36.5	1.19
75	36	131	1	1	36.5	1.71
75	36	131	2	1	36.5	1.11
75	36	131	3	1	36.5	0.91
75	36	131	4	1	36.5	0.81

*Normalized to failure depth

123r3/surfail

SURFICIAL SLOPE STABILITY

PROJECT **ROUTE 138**

FANGLOMERATE 1.5:1 @ FILL 2:1

JOB NO **6-212-105200**

COHESION (psf)	PHI ANGLE (deg)	TOTAL UNIT WT(pcf)	DEPTH TO FAILURE (ft)	HT OF WATER (ft)*	SLOPE ANGLE (deg)	FACTOR OF SAFETY
150	35	138	1	1	33.7	2.93
150	35	138	2	1	33.7	1.75
150	35	138	3	1	33.7	1.36
150	35	138	4	1	33.7	1.16
100	34	128	1	1	26.6	2.64
100	34	128	2	1	26.6	1.67
100	34	128	3	1	26.6	1.34
100	34	128	4	1	26.6	1.18

*Normalized to failure depth

123r3/surfail

```
Again  Buffer  Copy  Delete  Find  -find  Get  Insert  Jump
----- reading file: HWY138F.DAT ... 31538 characters
HIGHWAY 138. DISPLACEMENT
DISPLACEMENT FOR MAXIMUM CREDIBLE SITE ACCELARATION
C SEED.IDRISS
C UNITS OF DISP ARE IN FEET

C NACC     DT     AMAX     G
  1536    0.02    0.66    32.2

C KODE     XAXLEN     XSCALE
   1         0          0

C NUMBER OF YIELD ACCEL
           6

C YIELD ACCELERATION
  0.16 0.19 0.22 0.25 0.28 0.31

C ACCEL. HISTORY


C SEED-IDRISS (SAN.ANDREAS)  (BASE LINE CORRECTED)  NPT=3072 DT=0.02
 0.111076 0.121594 0.127002 0.126753 0.103202 0.082824 0.082863 0.065653
 0.037451 0.028367 0.024310 0.030191 0.033200 0.033113 0.017113-0.001049
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  DATE: 12:10:1996
  TIME:  9: 3:41
  INPUT/OUTPUT FILE NAME: HWY138F


*********************************************
  HIGHWAY 138. DISPLACEMENT
*********************************************

*********************************************
 DISPLACEMENT FOR MAXIMUM CREDIBLE SITE A
*********************************************


FOR NORMAL DIRECTION OF MOTION
AND A YIELD ACCELERATION OF    .16
MAXIMUM DISPLACEMENT =   1.859 UNITS OF LENGTH


FOR REVERSED MOTION
AND A YIELD ACCELERATION OF    .16
MAXIMUM DISPLACEMENT =   2.015 UNITS OF LENGTH


FOR NORMAL DIRECTION OF MOTION
AND A YIELD ACCELERATION OF    .19
MAXIMUM DISPLACEMENT =   1.281 UNITS OF LENGTH


FOR REVERSED MOTION
AND A YIELD ACCELERATION OF    .19
MAXIMUM DISPLACEMENT =   1.385 UNITS OF LENGTH


FOR NORMAL DIRECTION OF MOTION
AND A YIELD ACCELERATION OF    .22
MAXIMUM DISPLACEMENT =    .913 UNITS OF LENGTH


FOR REVERSED MOTION
AND A YIELD ACCELERATION OF    .22
MAXIMUM DISPLACEMENT =   1.014 UNITS OF LENGTH
```

```
FOR NORMAL DIRECTION OF MOTION
AND A YIELD ACCELERATION OF    .25
MAXIMUM DISPLACEMENT =    .583 UNITS OF LENGTH

FOR REVERSED MOTION
AND A YIELD ACCELERATION OF    .25
MAXIMUM DISPLACEMENT =    .722 UNITS OF LENGTH

FOR NORMAL DIRECTION OF MOTION
AND A YIELD ACCELERATION OF    .28
MAXIMUM DISPLACEMENT =    .406 UNITS OF LENGTH

FOR REVERSED MOTION
AND A YIELD ACCELERATION OF    .28
MAXIMUM DISPLACEMENT =    .522 UNITS OF LENGTH

FOR NORMAL DIRECTION OF MOTION
AND A YIELD ACCELERATION OF    .31
MAXIMUM DISPLACEMENT =    .280 UNITS OF LENGTH

FOR REVERSED MOTION
AND A YIELD ACCELERATION OF    .31
MAXIMUM DISPLACEMENT =    .374 UNITS OF LENGTH

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ELAPSED TIME =  0 MINS  3 SECS
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Insert: <cursor keys>, Esc to exit, Ins for Exchange
----- reading file: HWY138.DAT ... 15963 characters
HIGHWAY 138. DISPLACEMENT
2 DISPLACEMENT FOR MAXIMUM CREDIBLE SITE ACCELERATION
C EL CENTRO ACCELERGRAM
C UNITS OF DISP ARE IN FEET

C NACC     DT     AMAX     G
  1536    0.02    0.66    32.2

C KODE     XAXLEN     XSCALE
   1         0          0

C NUMBER OF YIELD ACCEL
          6

C YIELD ACCELERATION
  0.16 0.19 0.22 0.25 0.28 0.31

C ACCEL. HISTORY


C ELCENTRO -S90W- (BASE LINE CORRECTED)  NPT=1536  DT=0.02
 0.013375 0.026959 0.048846 0.031057 0.030420 0.042742 0.056136 0.042116
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  DATE: 12:10:1996
  TIME:  7:34:51
  INPUT/OUTPUT FILE NAME: HWY138


*******************************************
  HIGHWAY 138. DISPLACEMENT
*******************************************

*******************************************
 2 DISPLACEMENT FOR MAXIMUM CREDIBLE SITE A
*******************************************


FOR NORMAL DIRECTION OF MOTION
AND A YIELD ACCELERATION OF    .16
MAXIMUM DISPLACEMENT =   2.048 UNITS OF LENGTH


FOR REVERSED MOTION
AND A YIELD ACCELERATION OF    .16
MAXIMUM DISPLACEMENT =   1.722 UNITS OF LENGTH


FOR NORMAL DIRECTION OF MOTION
AND A YIELD ACCELERATION OF    .19
MAXIMUM DISPLACEMENT =   1.413 UNITS OF LENGTH


FOR REVERSED MOTION
AND A YIELD ACCELERATION OF    .19
MAXIMUM DISPLACEMENT =   1.266 UNITS OF LENGTH


FOR NORMAL DIRECTION OF MOTION
AND A YIELD ACCELERATION OF    .22
MAXIMUM DISPLACEMENT =   1.038 UNITS OF LENGTH


FOR REVERSED MOTION
AND A YIELD ACCELERATION OF    .22
MAXIMUM DISPLACEMENT =    .868 UNITS OF LENGTH
```

```
FOR NORMAL DIRECTION OF MOTION
AND A YIELD ACCELERATION OF    .25
MAXIMUM DISPLACEMENT =    .805 UNITS OF LENGTH

FOR REVERSED MOTION
AND A YIELD ACCELERATION OF    .25
MAXIMUM DISPLACEMENT =    .633 UNITS OF LENGTH

FOR NORMAL DIRECTION OF MOTION
AND A YIELD ACCELERATION OF    .28
MAXIMUM DISPLACEMENT =    .580 UNITS OF LENGTH

FOR REVERSED MOTION
AND A YIELD ACCELERATION OF    .28
MAXIMUM DISPLACEMENT =    .423 UNITS OF LENGTH

FOR NORMAL DIRECTION OF MOTION
AND A YIELD ACCELERATION OF    .31
MAXIMUM DISPLACEMENT =    .421 UNITS OF LENGTH

FOR REVERSED MOTION
AND A YIELD ACCELERATION OF    .31
MAXIMUM DISPLACEMENT =    .262 UNITS OF LENGTH

*******************************
ELAPSED TIME =  0 MINS  2 SECS
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----- Bye! (HWY138B.DAT)
HIGHWAY 138. DISPLACEMENT
3 DISP FOR MAXIMUM CREDIBLE SITE ACCELERATION
C HAYWARD 40 ACCELERGRAM
C UNITS OF DISP ARE IN FEET

C NACC     DT     AMAX     G
  2048    0.02    0.66    32.2

C KODE     XAXLEN     XSCALE
   1         0          0

C NUMBER OF YIELD ACCEL
          6

C YIELD ACCELERATION
  0.16 0.19 0.22 0.25 0.28 0.31

C ACCEL. HISTORY



C HAYWARD  (BASE LINE CORRECTED)  NPT=2048  DT=.02
Press any key to continue
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  DATE: 12:10:1996
  TIME:  7:35:29
  INPUT/OUTPUT FILE NAME: HWY138B


*********************************************
  HIGHWAY 138. DISPLACEMENT
*********************************************

*********************************************
 3 DISP FOR MAXIMUM CREDIBLE SITE ACCELERAT
*********************************************


FOR NORMAL DIRECTION OF MOTION
AND A YIELD ACCELERATION OF    .16
MAXIMUM DISPLACEMENT =   2.793 UNITS OF LENGTH


FOR REVERSED MOTION
AND A YIELD ACCELERATION OF    .16
MAXIMUM DISPLACEMENT =   2.470 UNITS OF LENGTH


FOR NORMAL DIRECTION OF MOTION
AND A YIELD ACCELERATION OF    .19
MAXIMUM DISPLACEMENT =   1.778 UNITS OF LENGTH


FOR REVERSED MOTION
AND A YIELD ACCELERATION OF    .19
MAXIMUM DISPLACEMENT =   1.955 UNITS OF LENGTH


FOR NORMAL DIRECTION OF MOTION
AND A YIELD ACCELERATION OF    .22
MAXIMUM DISPLACEMENT =    .994 UNITS OF LENGTH


FOR REVERSED MOTION
AND A YIELD ACCELERATION OF    .22
MAXIMUM DISPLACEMENT =   1.222 UNITS OF LENGTH
```

```
FOR NORMAL DIRECTION OF MOTION
AND A YIELD ACCELERATION OF    .25
MAXIMUM DISPLACEMENT =    .635 UNITS OF LENGTH

FOR REVERSED MOTION
AND A YIELD ACCELERATION OF    .25
MAXIMUM DISPLACEMENT =    .793 UNITS OF LENGTH

FOR NORMAL DIRECTION OF MOTION
AND A YIELD ACCELERATION OF    .28
MAXIMUM DISPLACEMENT =    .419 UNITS OF LENGTH

FOR REVERSED MOTION
AND A YIELD ACCELERATION OF    .28
MAXIMUM DISPLACEMENT =    .552 UNITS OF LENGTH

FOR NORMAL DIRECTION OF MOTION
AND A YIELD ACCELERATION OF    .31
MAXIMUM DISPLACEMENT =    .246 UNITS OF LENGTH

FOR REVERSED MOTION
AND A YIELD ACCELERATION OF    .31
MAXIMUM DISPLACEMENT =    .373 UNITS OF LENGTH

*******************************
3 ELAPSED TIME =  0 MINS  2 SECS
*******************************
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Again  Buffer  Copy  Delete  Find  -find  Get  Insert  Jump
----- reading file: HWY138D.DAT ... 15988 characters
HIGHWAY 138. DISPLACEMENT
DISPLACEMENT FOR MAXIMUM CREDIBLE SITE ACCELERATION
C TAFTN21E ACCELERGRAM
C UNITS ARE IN FEET

C NACC     DT     AMAX     G
  1536    0.02    0.66    32.2

C KODE     XAXLEN     XSCALE
   1         0          0

C NUMBER OF YIELD ACCEL
          6

C YIELD ACCELERATION
  0.16 0.19 0.22 0.25 0.28 0.31

C ACCEL HISTORY


C TAFT -N21E- (BASE LINE CORRECTED)  NPT=1536  DT=0.02
-0.076134 0.005743 0.002287 0.024638 0.031752 0.018700 0.017331 0.028483
-0.005611-0.035063-0.009571 0.021214 0.026416 0.002848 0.006094 0.014604
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  DATE: 12:10:1996
  TIME:  7:19:27
  INPUT/OUTPUT FILE NAME: HWY138D


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  HIGHWAY 138. DISPLACEMENT
*********************************************

*********************************************
+ DISPLACEMENT FOR MAXIMUM CREDIBLE SITE A
*********************************************


FOR NORMAL DIRECTION OF MOTION
AND A YIELD ACCELERATION OF    .16
MAXIMUM DISPLACEMENT =   1.407 UNITS OF LENGTH


FOR REVERSED MOTION
AND A YIELD ACCELERATION OF    .16
MAXIMUM DISPLACEMENT =   1.206 UNITS OF LENGTH


FOR NORMAL DIRECTION OF MOTION
AND A YIELD ACCELERATION OF    .19
MAXIMUM DISPLACEMENT =    .968 UNITS OF LENGTH


FOR REVERSED MOTION
AND A YIELD ACCELERATION OF    .19
MAXIMUM DISPLACEMENT =    .862 UNITS OF LENGTH


FOR NORMAL DIRECTION OF MOTION
AND A YIELD ACCELERATION OF    .22
MAXIMUM DISPLACEMENT =    .582 UNITS OF LENGTH


FOR REVERSED MOTION
AND A YIELD ACCELERATION OF    .22
MAXIMUM DISPLACEMENT =    .626 UNITS OF LENGTH
```

```
FOR NORMAL DIRECTION OF MOTION
AND A YIELD ACCELERATION OF    .25
MAXIMUM DISPLACEMENT =    .403 UNITS OF LENGTH


FOR REVERSED MOTION
AND A YIELD ACCELERATION OF    .25
MAXIMUM DISPLACEMENT =    .428 UNITS OF LENGTH


FOR NORMAL DIRECTION OF MOTION
AND A YIELD ACCELERATION OF    .28
MAXIMUM DISPLACEMENT =    .272 UNITS OF LENGTH


FOR REVERSED MOTION
AND A YIELD ACCELERATION OF    .28
MAXIMUM DISPLACEMENT =    .302 UNITS OF LENGTH


FOR NORMAL DIRECTION OF MOTION
AND A YIELD ACCELERATION OF    .31
MAXIMUM DISPLACEMENT =    .179 UNITS OF LENGTH


FOR REVERSED MOTION
AND A YIELD ACCELERATION OF    .31
MAXIMUM DISPLACEMENT =    .204 UNITS OF LENGTH


*******************************
ELAPSED TIME =  0 MINS  3 SECS
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APPENDIX E

PAVEMENT DESIGN REPORT

PAVEMENT DESIGN REPORT
ROUTE 138 PHASE 2
08-SBd-138 - 16.4/19.7

The recommendations presented in this report are based on field exploration and testing performed by AGRA Earth & Environmental, Inc. (AEE) as presented in the Supplemental Geotechnical Design Report. Reference should be made to the geologic and geotechnical conditions described in that report as well as the presentation of laboratory testing on the Soil Summary Sheets, Plate VI through X. Previous testing performed by John A. Sayers & Associates (JASA) was also used as a data base. The testing by JASA is also compiled in the Supplemental Geotechnical Design Report.

A summary of Resistance Value testing performed is presented below:

Boring No.	@ Depth/ Sample No.	Material Type	R-Value (Stabilometer)
B1	@ 7.5'	Older Alluvium: Silty Sand	78
B6	@ 3.5'	Older Alluvium: Silty Sand	68
B10	@ 0 - 3'	Slope Wash: Silty Sand	61
B15	@ 0 - 3'	Alluvium: Silty Clayey Sand	67
B16	@ 0 - 3'	Alluvium: Silty Sand	61
101	2	Fanglomerate: Fine to Medium Sand	70
Test Pit 2	5	Fanglomerate: Silty Sand with Gravel	73
103	2	Crowder Formation: Silty Sandstone	42
107	3	Crowder Formation: Clayey Sandstone	48
108	9	Crowder Formation: Clayey Sandstone	15
109	7	Terrace Deposits: Fine to Medium Silty Sand	74

Based on the testing performed, it is our opinion that a project design R-Value of 50 is appropriate. An R-Value of 50 should be readily achievable within the Terrace Deposits and younger alluvial soils which is expected to provide most of the fill embankment material for this project. It is expected that the Crowder Formation, particularly the more clayey units will not yield an R-Value of 50. Fill soils derived from the Crowder Formation should be omitted from the upper 4 feet of embankment fill or be mixed with other on-site soils to attain an R-Value of greater than 50. Where pavement subgrade will be founded in Crowder Formation (predominantly cut areas between approximately Station 144 and 189) lower R-Value material

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Earth & Environmental

may be exposed at subgrade elevation. In this case, overexcavation of the unsuitable material to a depth of 4 feet and replacement with suitable R-Value material will be required.

With the above conditions the recommended pavement sections, as designed in accordance with the Caltrans Highway Design Manual, are as follows:

Main Line - Highway 138, 10-year T.I. = 8.0

	0.40 feet	Asphaltic Concrete
over	0.25 feet	Asphalt Treated Permeable Base
over	0.35 feet	Class 2 Aggregate Base

Main Line - Highway 138, 20-year T.I. = 10.0

	0.50 feet	Asphaltic Concrete
over	0.25 feet	Asphalt Treated Permeable Base
over	0.35 feet	Class 2 Aggregate Base

Shoulder - Highway 138, 10-year T.I. = 5.0

	0.20 feet	Asphaltic Concrete
over	0.35 feet	Class 2 Aggregate Base

Shoulder - Highway 138, 20-year T.I. = 6.0

	0.25 feet	Asphaltic Concrete
over	0.35 feet	Class 2 Aggregate Base

Asphaltic concrete, asphalt treated permeable base and Class 2 Aggregate Base should conform to the current Caltrans Specifications.

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APPENDIX F

CORROSION STUDY

CORROSION STUDY
ROUTE 138, PHASE 2
08-SBd-138 - 16.4/19/7

This study presents design recommendations for culvert construction for the Route 138 Phase 2 project. The recommendations are based on field exploration and laboratory testing performed by AGRA Earth & Environmental, Inc. (AEE) as presented in the Supplemental Geotechnical Design Report. Reference should be made to the geologic and geotechnical conditions described in that report as well as the presentation of laboratory testing on the Soil Summary Sheets.

During the field exploration a cursory inspection of existing culverts within the current roadway alignment was performed. The culverts were constructed of corrugated steel pipe (CSP). The ages of the culverts are not known, however, it is likely that they date back to the original construction of existing Route 138. The condition of the culverts was rated as fair to good. No perforations of the culvert shells were noted within sight of either the inlet or outlet.

Corrosion samples were obtained at the proposed major culvert crossings using hand auger techniques (Samples CS-1 through CS-24). In addition, a number of samples were selected from deep borings drilled within proposed cut areas. Resistivity, pH and sulphate tests were performed on all the samples. Chloride tests were performed on selected samples with the lowest minimum resistivity values. The sample locations and test results are summarized on the Soil Survey Sheets, Plate VI through X.

Based on the testing performed it is evident that the geologic units along the alignment can be grouped into two classes. The Terrace Deposits and younger alluvial soils, which will be encountered at the invert elevation of proposed culverts have low corrosion potential. The Crowder Formation, which could potentially be included in embankment fill over proposed culverts, has moderate corrosion potential. A summary of recommended design parameters for both groups is presented as follows.

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Material	Type A Terrace Deposits and Younger Alluvial Soils	Type B Crowder Formation
Minimum Resistivity (ohm-cm)	6,980	1,190
pH	6.8	7.2
Sulphate Content (ppm)	227	138
Chloride Content (ppm)	43	96
Design Life of Galvanized 18 Gage CSP (years)	26	23
Required Gage Thickness For CSP 25 Year Design Life		
- Galvanized	18	16
- Galvanized, Bitumen Coating	18	18
- Galvanized, Bitumen Coating, Paved Invert	18	18
Required Gage Thickness For CSP 50 Year Design Life		
- Galvanized	12	12
- Galvanized, Bitumen Coating	14	12
- Galvanized, Bitumen Coating, Paved Invert	16	14
Culvert Material Alternatives	Alluvium or aluminized steel is acceptable.	Neither aluminum nor aluminized steel is acceptable.

Where culverts will be founded within alluvial soils and embankment fill to be placed around the culvert will be composed of similar material, the Type A design parameters may be used for metal conduit. If there is uncertainty as to the type of embankment fill to be used at a specific culvert location we recommend that the Type B design parameters be applied to metal conduit.

The corrosion testing indicates that none of the geologic units within the roadway alignment present a corrosive environment for unreinforced or reinforced concrete pipe.

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APPENDIX G

GEOPHYSICAL SURVEY REPORT (PREPARED BY GOFFMANN, MCCORMICK & URBAN INC.)

Goffman, McCormick & Urban, Inc.
ENGINEERING AND GEOLOGICAL
CONSULTANTS IN APPLIED
EARTH SCIENCES

22822 Granite Way, Suite A
Laguna Hills, CA 92653-1299
Telephone (714) 951-1808
FAX (714) 951-8093

November 26, 1996

Mr. Doug Bell
AGRA EARTH & ENVIRONMENTAL, INC.
1290 North Hancock Street
Anaheim, CA 92817

Project 96-76

Subject: Seismic Refractive Classification, Highway 138 Project in San Bernardino County, California

Dear Mr. Bell:

As requested, a seismic refractive study was performed at the Highway 138 Project in San Bernardino County.

PURPOSE

The study was conducted to obtain seismic refractive velocity and depth data representative of the earth materials to be excavated. This data is intended to serve as an aid in classifying earth materials to be excavated. Rippability of rock material is a function of seismic velocity, geologic structure, ripping equipment capacity and operating technique.

SCOPE

This study consisted of shooting three seismic refractive profiles. The length of each traverse ranged from 223 feet to 333 feet. A site reconnaissance was performed on November 12, 1996. The field work was performed on November 14, 1996.

LOCATION

The location of the seismic lines are indicated on Plate 1.1 (Lines 1 and 2) and Plate 1.2 (Line 3). The estimated locations of the seismic lines are indicated on portions of the project layout plans. The location of the seismic lines were determined by your firm.

FIELD PROCEDURE

Each seismic profile consisted of 12 geophones spaced in line at uniform intervals of either 20 or 30 feet on the ground surface. The distance scale of the individual Time versus Distance graphs indicates the geophone spacing.

Shotpoints were located three feet from each end of each geophone spread. In essence, the shotpoints were located at the first geophone, "zero-time" being essentially the first energy arrival at the first geophone.

The depth of seismic investigation is a function of the length of the seismic line on the ground surface and the seismic velocity of the subsurface materials. Good practice provides for locating the line of geophones on level ground or on a uniform slope and making the line as straight as possible.

In practice, the total seismic traveltime from shotpoint to end geophone should be the same for the opposite ends of each geophone spread. The data obtained in this study essentially meets this basic criterion. The exceptions are attributed to disturbance of the ground in the vicinity of the shotpoints and geologic conditions within the shotpoint offset interval.

INSTRUMENTATION

A Geo Space Corporation Model GT-2B 12-channel seismic system and Hall-Sears geophones were used for recording the traveltime of first arrival seismic energy between shotpoint and each geophone. Energy traveltime is photographically recorded by this equipment. Traveltime data read from the records are graphically plotted with respect to distance traveled to determine seismic velocity and velocity-depth relationships of the earth materials; reference the Appendix.

EXCAVATION MATERIALS

The soil and rock to be excavated consist primarily of terrace deposits and sedimentary rock (Crowder formation) materials in various stages of alteration and weathering. Existing road cuts and outcrops were observed to be both terrace deposits and Crowder formation rock materials.

Terrace deposits were observed at the surface in the vicinity of Seismic Line 1. Terrace deposits and/or Crowder formation conglomerate materials were observed at the surface in the areas where Seismic Lines 2 and 3 were located. The conglomerate materials tend to be moderately cemented. Some boulders in excess of 12 inches in diameter were observed within exposures of both the terrace deposits and Crowder formation materials.

For purposes of classifying these materials in terms of excavation effort, six classifications have been defined as functions of seismic velocity:

Excavation Method	Seismic Velocity (feet per second)
Easy processing	up to 3000
Easy ripping	3000 to 4000
Moderately difficult ripping	4000 to 5000
Difficult ripping	5000 to 6000
Very difficult ripping	greater than 6000
Probable blasting	greater than 7000

This table assumes double ripper shank Cat. Model D-9L, or equivalent, rip-dozer equipment. Difficult ripping represents the start of difficulty in achieving and maintaining ripper shank penetration, the start of "riding out".

These classifications are a generalized means of indicating relative excavation effort. The rippability of the rock will depend greatly on the spacing and development of the rock fractures present at depth.

Trenches excavated in materials represented by seismic velocities greater than about 3500 to 4000 feet per second will require extraordinary excavation effort.

SUMMARY OF SEISMIC FINDINGS

The results of the seismic refractive investigation are contained in the attached Seismic Traveltime versus Distance graphs. The seismic velocity is "flagged" to the straight line segments connecting the data points, and calculated depths to velocity interfaces below the existing ground surface are circled.

The slope of the lines through the time-distance data points (e.g. Plates 2 through 4) indicates seismic velocity (distance divided by traveltime). The steeper the slope, the lower the velocity. The lines were drawn through the data points on the basis of our interpretation of "best fit". The reader may elect to draw other lines through the data points in order to evaluate the possible extremes of velocity variation.

The quality of the seismic recordings is graded as fair to good on a scale of poor, fair, good, excellent.

If subsurface conditions are uniform over the entire length of the seismic lines, both limbs of the time-distance graphs would be symmetrical with respect to velocities and depths. The graphs are fairly symmetrical with variations which are reasonable for the geological conditions present.

The velocity-depth data for both limbs of each seismic profile have been combined to yield true* velocity and average depths to velocity contrasts in order to provide a convenient visual summary of the subsurface data.

It is recommended that the individual time-distance graphs be reviewed for specific details of velocity and depth variation in the process of classifying the materials for excavation.

LINE 1 - Plate 2

Depth (feet)	**Velocity (feet per second)**	**Excavation (D-9L)**
0-70	2,000	easy processing
70-138	3,850	easy to moderately difficult ripping

If 7,000 feet per second material is present, the average depth is more than 138 feet and, based on the data from Seismic Line 2, it is probably more than 162 feet.

* reference the Appendix

LINE 2 - Plate 3

Depth (feet)	Velocity (feet per second)	Excavation (D-9L)
0-4	3,850	easy to moderately difficult ripping
4-61	4,950	moderately difficult to difficult ripping

If pervasive 7,000 feet per second material is present, the average depth is more than 61 feet.

LINE 3 - Plate 4

Depth (feet)	Velocity (feet per second)	Excavation (D-9L)
0-26	2,100	easy processing
26-77	3,850	easy to moderately difficult ripping

If pervasive 7,000 feet per second material is present, the average depth is more than 77 feet.

Plates 1.1 through 4 and the Appendix are attached and complete this report.

Respectfully submitted,

GOFFMAN, McCORMICK & URBAN, INC.

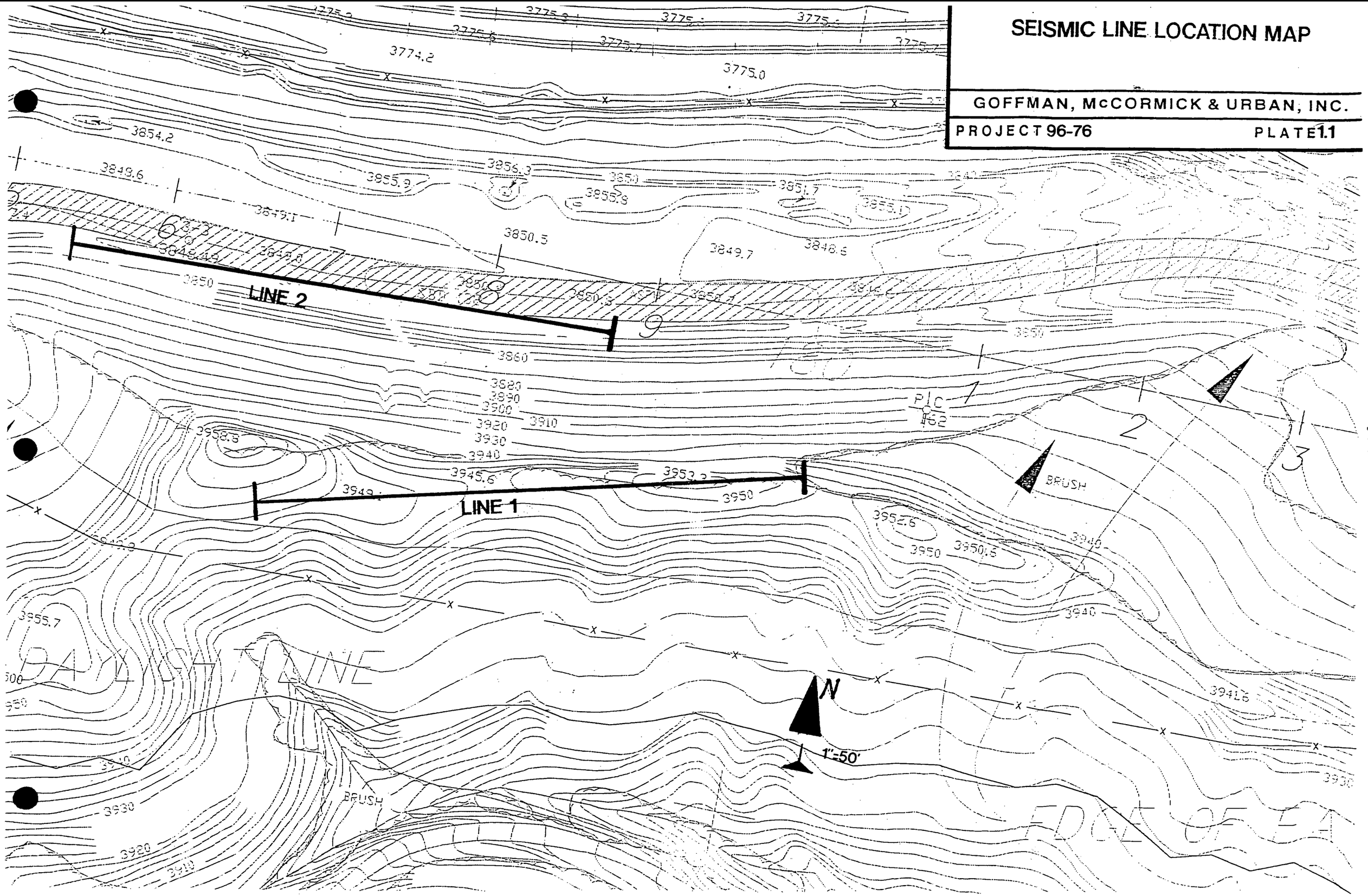
Alan B. Mutchnick
Engineering Geologist 1789

bm/96-76r1

SEISMIC LINE LOCATION MAP

GOFFMAN, McCORMICK & URBAN, INC.

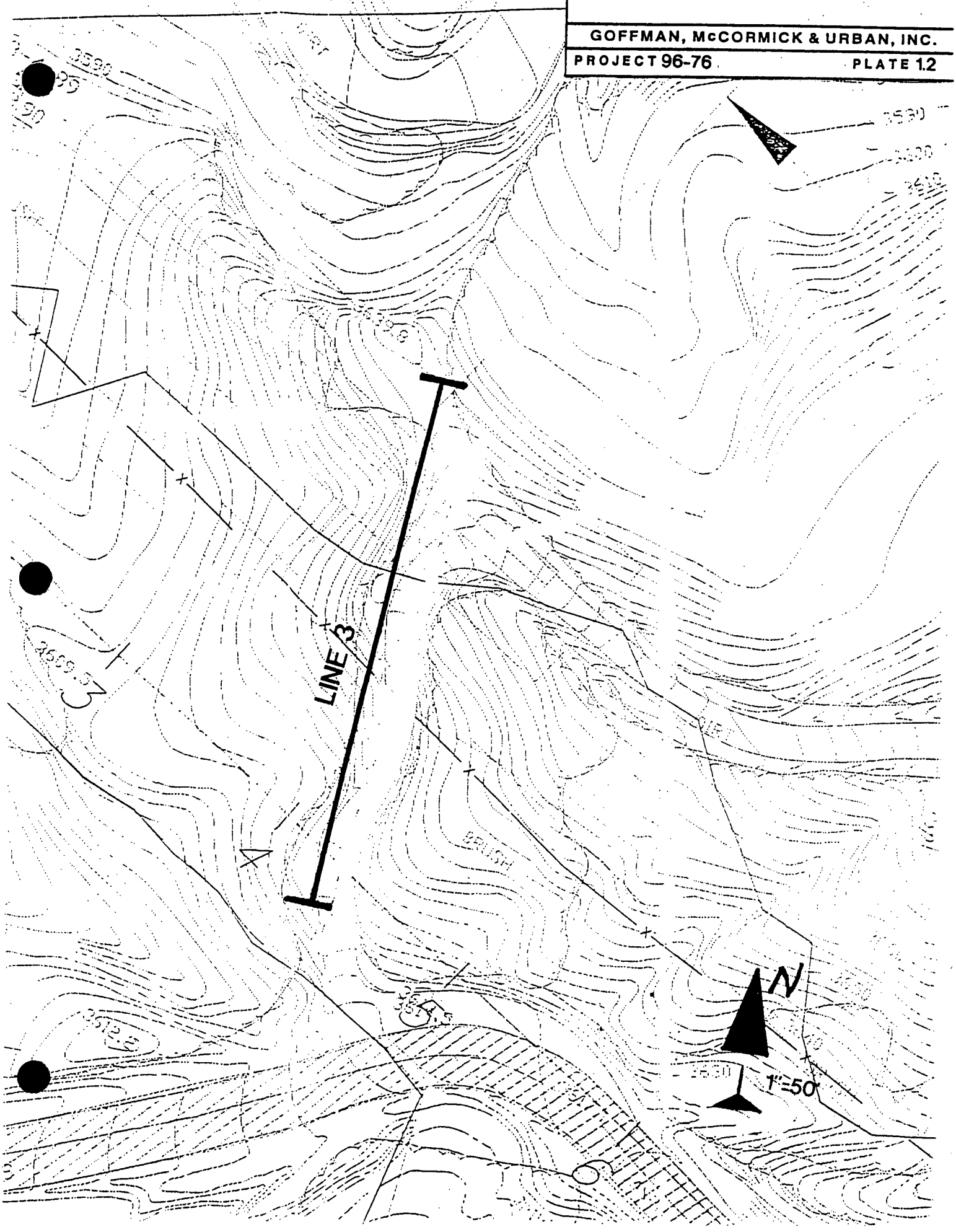
PROJECT 96-76 PLATE 1.1

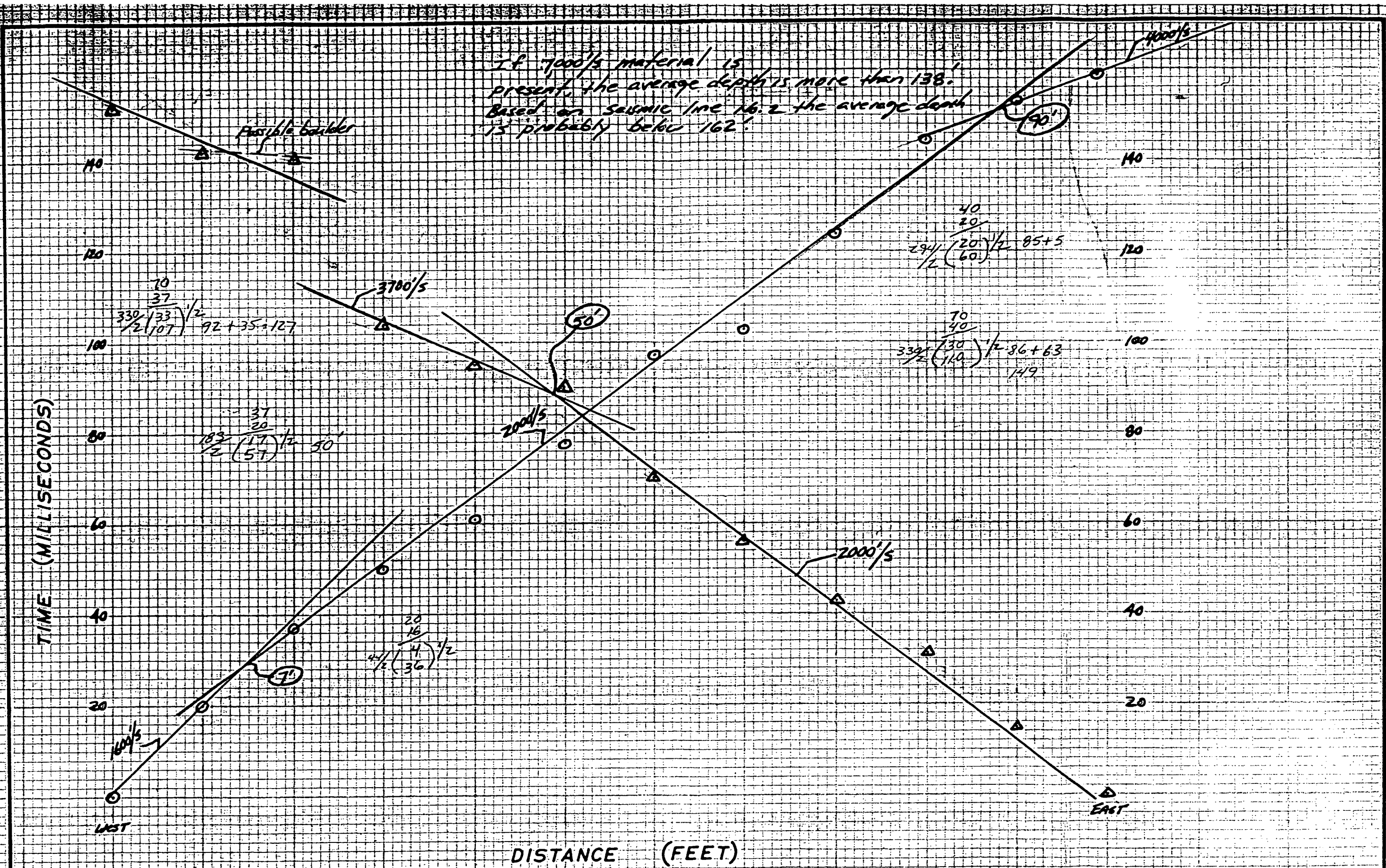


SEISMIC LINE LOCATION MAP

GOFFMAN, McCORMICK & URBAN, INC.

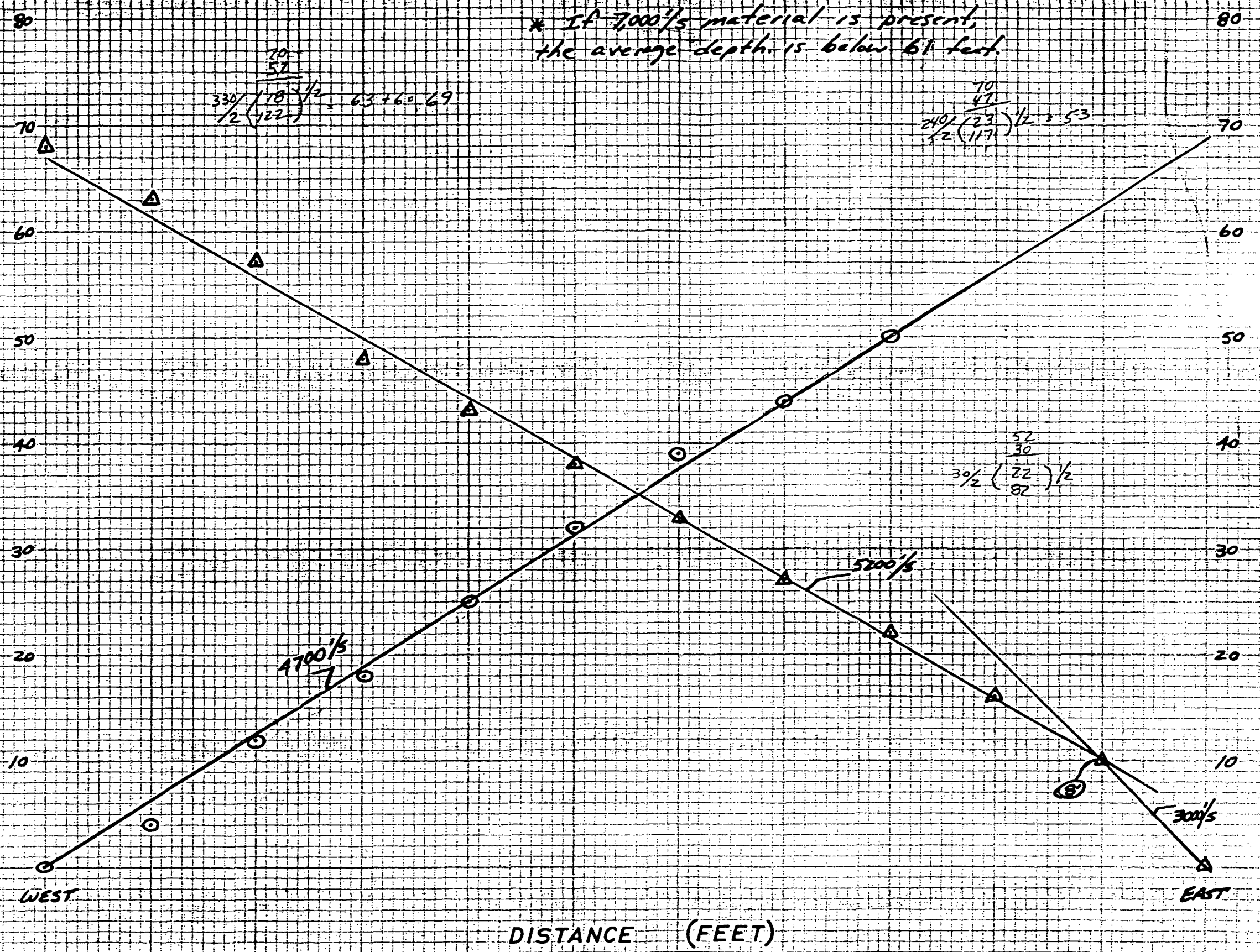
PROJECT 96-76 PLATE 1.2





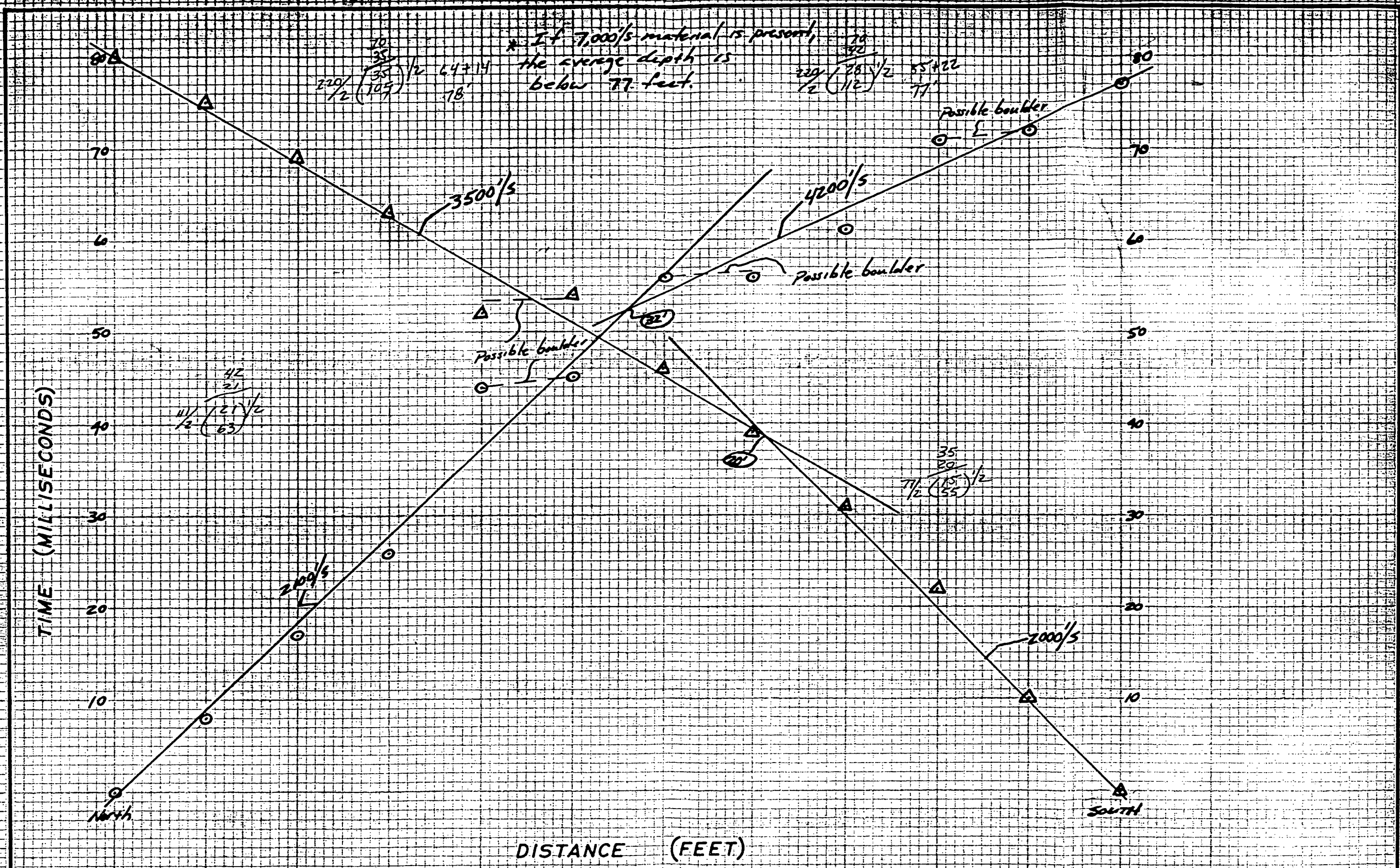
SCALE 1" = 30 FEET
1" = 20 MS

LINE NO. 1	PROJECT Hwy 138	
DATE 11/23/96	BY [illegible]	
JOB 96-76	Goffman, McCormick & Urban, Inc.	PLATE - 2



SCALE 1" = 30 FEET
1" = 10 MS

LINE NO. 2	PROJECT Hwy 138
DATE 11/23/96	BY ABM
JOB 96-76 Goffman, McCormick & Urban, Inc.	PLATE - 3



SCALE 1" = 20 FEET
1" = 10 MS

LINE NO. 3	PROJECT Hwy 138	
DATE 11/23/96	BY RBM	
JOB 96-76	Goffman, McCormick & Urban, Inc.	PLATE 4

APPENDIX

THEORY AND DISCUSSION

Seismic energy introduced into the earth by means of a shallow subsurface detonation propagates in the form of elastic waves. The behavior of seismic waves is controlled by the elastic properties of the earth materials through which they propagate and the laws of optics.

The basic quantity recorded in a seismic refractive investigation is the compressional (fastest traveling) elastic wave traveltime between the sources (detonation) and surface detectors (geophones) spaced at known distances from the detonation point.

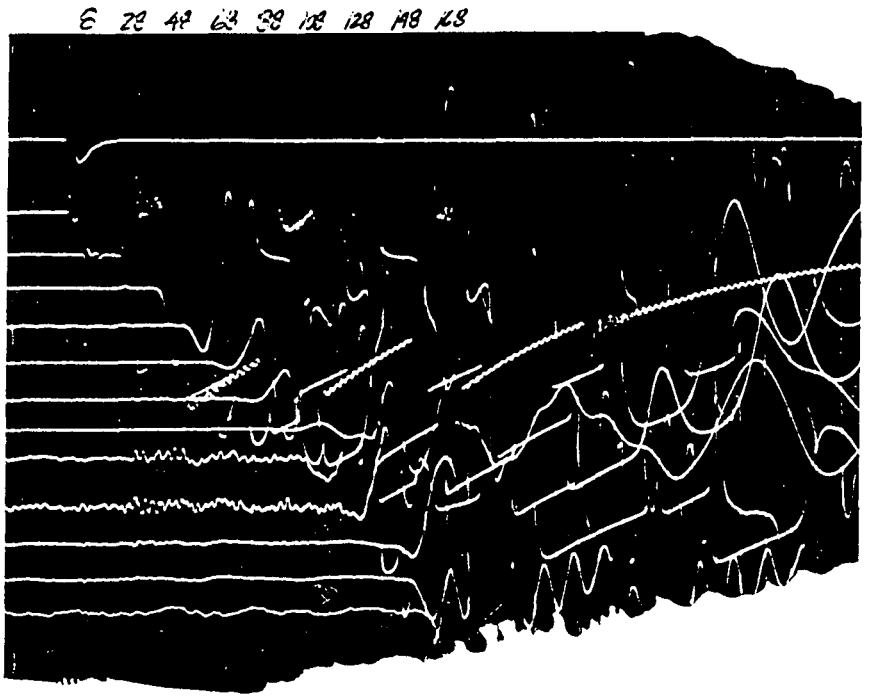
The constitution of the earth is generally such that materials at depth have properties related to density and modulus of elasticity which transmit seismic energy at greater velocity than shallower materials transmit the energy.

A basic seismic principle (Fermat's) states that the compressional wave follows the minimum time path between detonation point and geophone. With the existence of high velocity material beneath low velocity material and sufficient distance between detonation point and geophones, the minimum time path (traveltime) is not over the shortest distance path, but over a refracted (bent) travel path, a portion of which lies within the deeper high velocity material.

Graphic plots of elastic wave traveltime versus ground surface distance exhibit the details of subsurface seismic velocity variations. Utilizing the principles of seismic wave refraction (Snell's Law), seismic velocity variations are interpreted in terms of subsurface geologic conditions. Our time-distance worksheets for this project are included in this report.

Seismic refractive data does not preclude the possibility that softer, lower velocity materials may exist beneath high velocity materials. The depth and thickness of low velocity materials underlying high velocity materials are not directly definable by surface seismic refractive methods.

Seismic refractive velocity data in intercalated high and low velocity materials generally represent an "average" of the velocities of the materials. Whether the "average" is a true average or not is a function of the individual volumes of the various material types, the subsurface attitude of the materials, the length of the seismic energy travel path in each material, and the relative velocity contrast between the material types.



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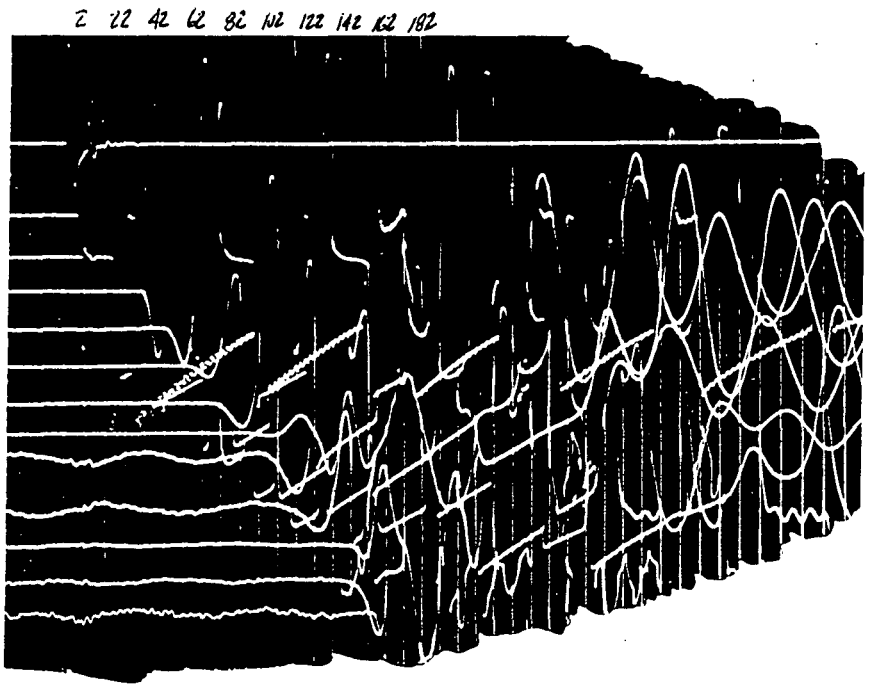
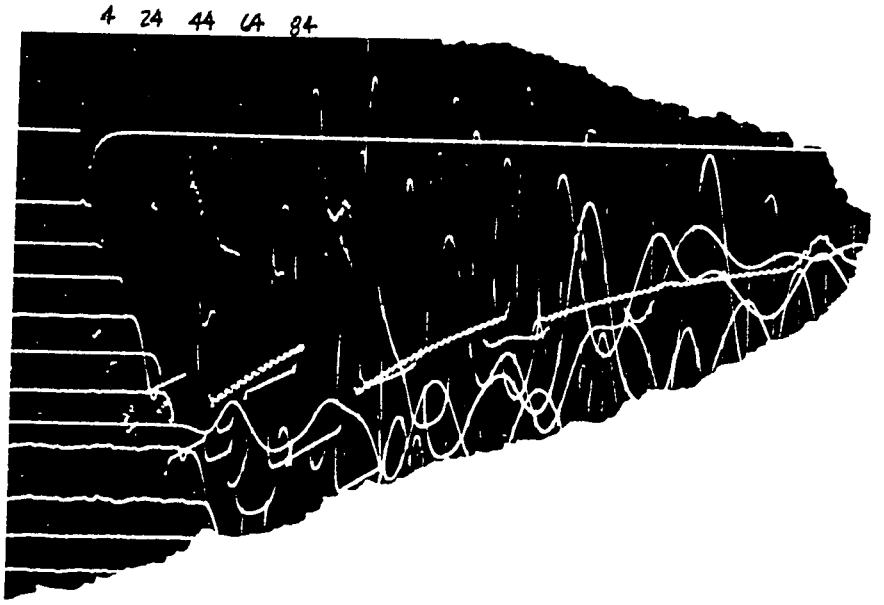


Line 1 E

TRAVELTIME SEISMOGRAMS



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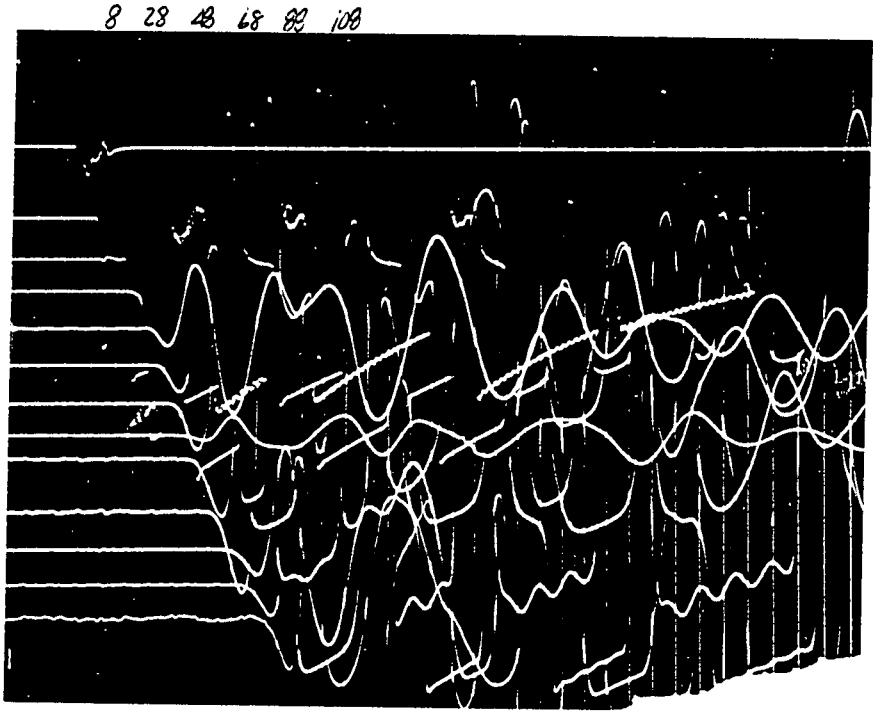

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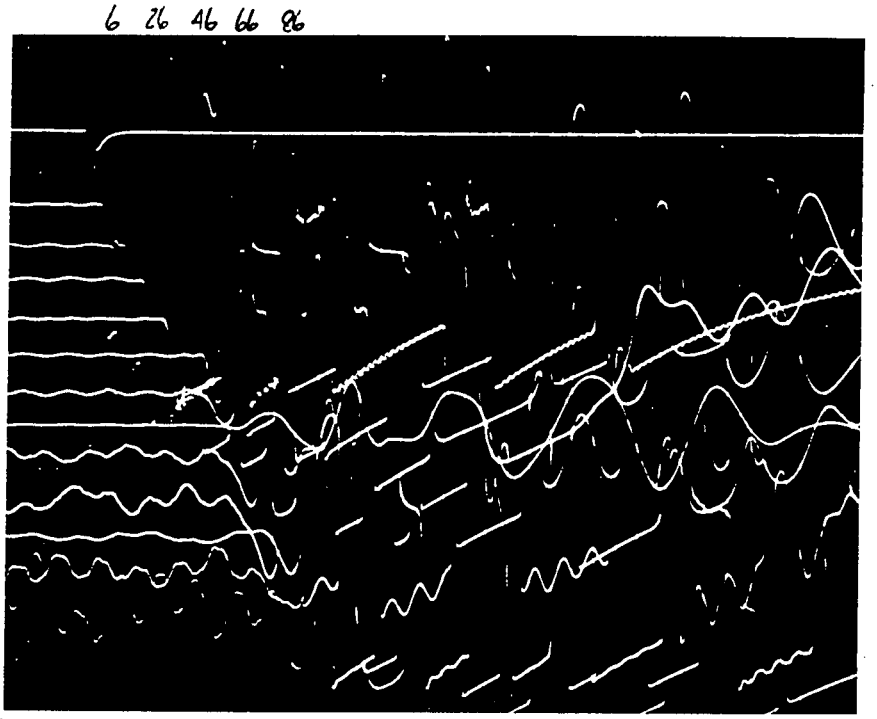
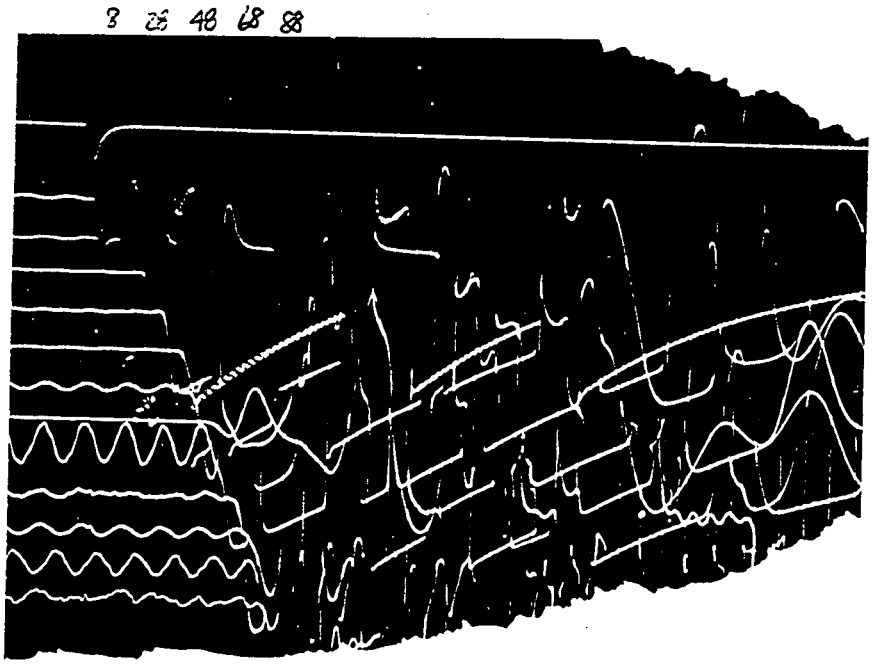
TRAVELTIME SEISMOGRAMS



Line 3 N



Line 3 S

TRAVELTIME SEISMOGRAMS

APPENDIX H

DATA FROM PREVIOUS TESTING (PERFORMED BY JOHN A. SAYERS & ASSOCIATES)

SAMPLING METHODS AND LABORATORY TESTING

Drilling Techniques

Borings B-1 through B-16 were drilled with a Mobile B-52 Hollow Stem Auger (2-R Drilling Company, Ontario, California) using an eight inch diameter auger. Samples are taken from within the hollow stem by driving a sampler with a 140 pound weight according to the standard penetration test procedure ASTM D 1586-76. Borings B-1A and B-2A were drilled with a EZ Bore Drill (Al-Roy Drilling, Yorba Linda, California) using an 18 inch diameter bucket. With this rig the samples are taken by driving the sampler with the weight of a telescoping Kelly bar. As the Kelly bar extends the weight is reduced by sections depending upon the depth. These blow counts may be converted to standard penetration blows (N-values) with the following formula:

$$N = N' \times \frac{WH}{4200} \times \frac{2.00^2 - 1.38^2}{D_o^2 - D_i^2}$$

Where

N	=	equivalent SPT blowout (uncorrected)
N'	=	measured blowout
W	=	weight of hammer (Kelly), pounds
H	=	distance of hammer drop (inches) = 12
D_o	=	outside diameter of sample (inches) = 3.0
D_i	=	inside diameter of sample (inches) = 2.416

Example: B-1A @ 20' $N' = 5 + 9 = 14$

$$N = 14 \times \frac{3350\ (12")}{4200} \times \frac{2.0^2 - 1.38^2}{3.0^2 - 2.416^2} = 119.7$$

Sampling Methods

Soil samples have been collected as bulk or bag samples and/or as in-situ or relatively "undisturbed" core samples. The latter were obtained with a split-barrel sampler containing a

series of one-inch high brass rings, 2.416 inches in inside diameter (ASTM D 3550-84). The sampler is driven into the ground with successive free fall blows of the drill rig's Kelly bar dropping one foot. The weight of the Kelly bar and number of blows per foot are typically recorded on the boring logs. Alternatively, the sampler may be driven with a 140-pound weight and the sampler may be a Standard Penetration Sampler which obtains a disturbed sample. If the standard penetration test is used, it is designated on the boring log as explained in the Boring Log Legend.

The Standard Penetration Test is performed in borings during field exploration and may be used to correlate materials and/or to determine a variety of engineering properties. The test uses a split-barrel sampler, 2 inches in diameter, which is driven into the ground with successive blows of a 140-pound weight. The blow counts are recorded in six inch intervals for further analysis and the sample contained in the sampler is retained for identification and testing. The test procedure used is the American Society for Testing and Materials (ASTM) D 1586-76.

Bulk samples were collected in plastic bags to retain their original moisture content. The core samples were placed in molded plastic containers, sealed, stored in protective boxes and transported to our laboratory.

Classification

Soils extracted from trenches/borings in the field were identified using the Unified Soil Classification System (USCS). Field identifications may be supplemented by additional visual classifications and/or by test procedures. The soil classifications are shown on the Logs of Test Pits and/or Borings presented within this report. Where blow counts were very high or refusal was encountered, the soils were described as very hard.

Laboratory Testing

The physical and engineering properties of the soils were determined by laboratory testing of samples. The types of tests typically conducted are briefly discussed below. All test results using these procedures are specific to the soils collected and variations could be expected at other depths and locations.

Natural Density and Moisture Content

The natural density of the soil has been determined for all "undisturbed" samples obtained during subsurface explorations. These densities are recorded as dry unit weights in pounds per cubic foot. The moisture contents for these and standard penetration samples are determined as a percentage of the dry weight. These test results are shown on the Boring Logs and in the Soil Survey Sheets.

Maximum Dry Density and Optimum Moisture Content

Typical bulk samples have been tested for their maximum dry density and optimum moisture content. The results are obtained using the ASTM standard procedure D 1557-78. These tests are generally used to compare natural densities and may be used for compaction testing. Additional testing for other soil types or soil mixes may be necessary at a later date.

The maximum dry densities and optimum moisture contents are presented on the Soil Survey Sheets accompanying this report.

Direct Shear

The shear strength of typical on-site soils has been determined using a direct shear machine designed to receive either "undisturbed" or remolded samples of the soil in one-inch high brass rings. The samples are extruded from the rings and into the machine apparatus, loaded with varying normal loads, allowed to consolidate and then sheared at a strain rate of 0.02 inches per minute. The samples may be sheared at their natural moisture content or may be presaturated and inundated with water prior to testing, as indicated on the test results. The test procedure is according to ASTM D 3080-79.

The test data has been plotted on stress-strain diagrams and the peak strength has been plotted on Shear Test diagrams. Using the stress-strain diagram the percent shear should be multiplied by 120 to obtain the shear strength in pounds per square foot. Both have used computer interfaces as the plotting mechanism with the friction angle and cohesion calculated by the computer, except for cohesionless soils where the cohesion was set to zero and the friction angle determined as a best fit.

Our shear testing was performed for four situations, one being undisturbed samples at natural moisture contents. The second is undisturbed samples being saturated prior and during testing. The third and fourth consist of samples remolded to 90 percent relative compaction at optimum moisture content and then either sheared at optimum moisture or at saturation. The four conditions are considered representative of specific formations and locations along the road alignment as follows:

1) Undisturbed samples at natural moisture content should be used for steep topographic areas along eastern portion of road alignment exposing Crowder Formation and Terrace deposits.

2) Undisturbed samples at saturated conditions should be used for the more gentle topographic areas along western portion of alignment exposing older alluvium and recent alluvium (cohesionless and uncemented sands).

3) Remolded samples at saturation should be used for proposed fill slopes where cementation would be lost and cohesion is minimized by recompaction.

4) Remolded samples at optimum moisture content would typically be representative of unsaturated fill, but were not utilized in our analyses.

Shear test results are presented in Appendices B through E to follow. They include a shear test summary, a summary diagram showing the values used followed by individual shear test diagrams and stress-strain diagrams.

One-Dimensional Consolidation

A soil sample from alluvium within Boring B-15 has been tested for its consolidation potential. The consolidation testing apparatus is designed to receive either natural or remolded soils in a one-inch high ring. Each sample is then loaded incrementally in a geometric progression and the resulting consolidation is recorded at selected time intervals. Porous stones are in contact with both the top and bottom of the soil to permit the addition or release of pore water. The samples are typically inundated with water at intermediate loads. The test procedure used was California Test 219 which is similar to ASTM D 2435-80.

The consolidation test result is plotted on the Consolidation Pressure Curves accompanying this appendix as Plate A-1.

Grain Size Distribution

The grain size distribution of selected soils has been determined using a range of wire mesh screens and a mechanical sieve shaker (ASTM D 422-72). The weight of the residue contained on each screen is determined and the percent finer or coarser of the total weight is calculated. The percent passing the 200-sieve wet by either the dry sieve method (above) or by the wet sieve method (ASTM D 1140-71) are shown on the Soil Survey sheets.

Grain Size Distributions are graphically plotted and are presented at the end of this appendix.

Sand Equivalent

The Sand Equivalent (S.E.) value for selected soils was determined using CALTRANS Test Method No. 217. This test determines the relative cleanliness of sandy soils and relates to their suitability for bedding, backfill and other uses. A higher S.E. reading indicates a relatively clean sand. An S.E. of 30 or greater is usually required for trench backfill which may need only flooding to obtain compaction.

Test results of the Sand Equivalent test are presented in the attached Soil Survey Sheets.

Plasticity Index

The Plasticity Index (PI) for selected soils were determined according to California Test 204, which is similar to ASTM D 4318-83. This test measures the liquid limit (LL) and Plastic Limit (PL) of the soils passing the No. 40 sieve. In each case tested, the samples had less than 50 percent and more than 12 percent passing the No. 200 sieve and liquid limits less than 50. This indicates the fines (< No. 40) are low plastic silts or clays and when combined with the sands are silty to clayey sands (SM/SC).

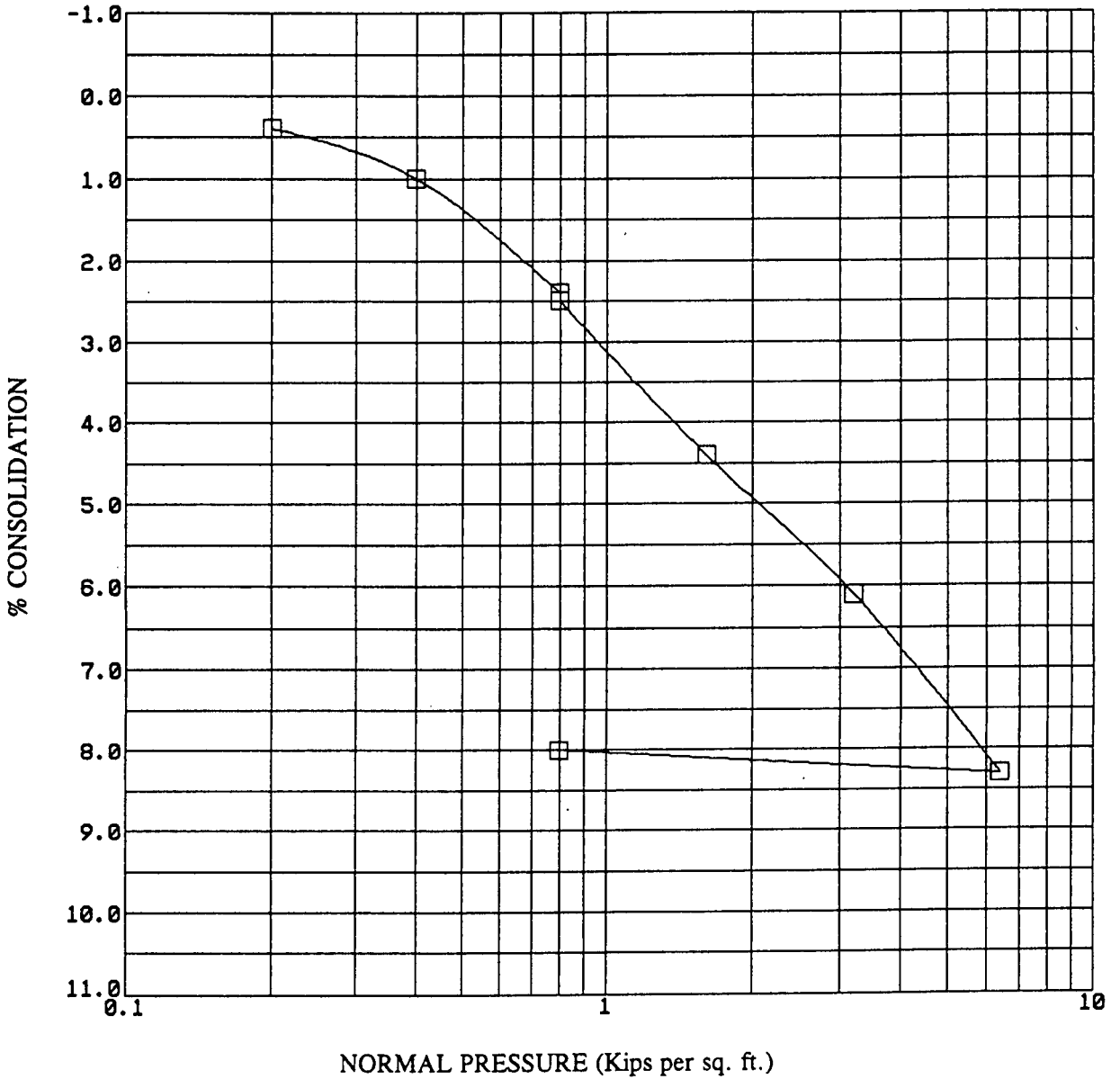
The results of these tests are shown on Plates A-2 and A-3.

Corrosion Testing

Soils resistivity and pH testing was performed for us by Associated Laboratories of Orange, California. The test method used was California 643B and the results are indicated on the Soil Survey Sheets. Associated Laboratories report is attached as Plate A-4.

CONSOLIDATION TEST RESULTS

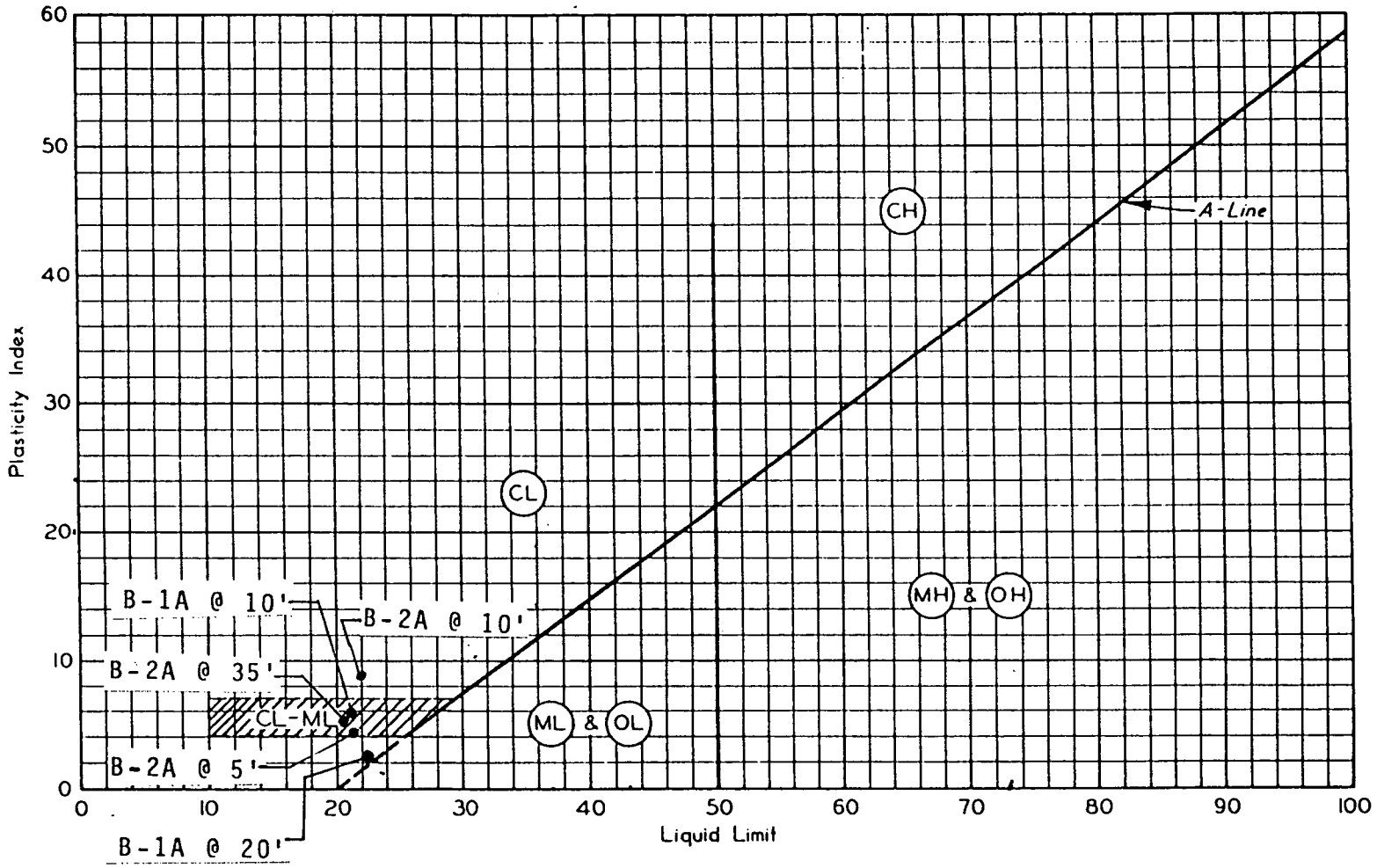
Client: **Rancho Las Flores Limited Partnership**	W.O. **215-1**
Project: **Highway 138**	Location: **San Bernardino County**



Sample: **B-15 @ 5.0 ft.**
Undisturbed
Water Added 0.8 Kips

PLASTICITY INDEX TEST RESULTS

Sample Location	Material Type	% Passing No. 200	Liquid Limit (%)	Plastic Limit (%)	Plasticity Index (%)	USCS
B-1A @ 5'	Sandstone (silty sand)	14.2	(Non-Plastic)			SM
B-1A @ 10'	Sandstone (silty to clayey sand)	40.7	27.2	21.3	5.9	SM/SC
B-1A @ 20'	Sandstone (silty sand)	27.3	24.6	22.3	2.3	SM
B-2A @ 5'	Sandstone (silty to clayey sand)	28.3	25.8	21.7	4.1	SM/SC
B-2A @10'	Sandstone (clayey sand)	40.3	31.0	22.1	8.9	SC
B-2A @ 35'	Sandstone (silty to clayey sand)	24.7	26.0	20.8	5.2	SM/SC



PLASTICITY CHART

215

ASSOCIATED LABORATORIES

806 North Batavia - Orange, California 92668 - 714/771-6900

FAX 714/538-1209

CLIENT

John A. Sayers & Associates (2562)
27071 Cabot Rd.
Ste. 132
Laguna Hills, CA 92653
Attn: Carisa Davis

LAB NO F81294

REPORTED 02/15/90

SAMPLE Soil

RECEIVED 02/08/90

IDENTIFICATION W.O.#215

BASED ON SAMPLE As Submitted

	Sulfate	pH	Minimum Resistivity (Calif. 643B) (ohm/cm)
B-6 @ 3-6'	ND<0.001%	6.80	14,250
B-13 @ 12-15'	ND<0.001%	8.31	3,960
B-1 @ 6-8'	ND<0.001%	7.43	14,250
B-15 @ 0-3'	ND<0.001%	7.59	5,280
B-10 @ 0-3'	ND<0.001%	8.09	11,880

ASSOCIATED LABORATORIES

Tito L. Parola

TLP/ql

RECEIVED 2-27-90

NOTE: Unless notified in writing, all samples will be discarded by appropriate disposal protocol 30 days from date reported.

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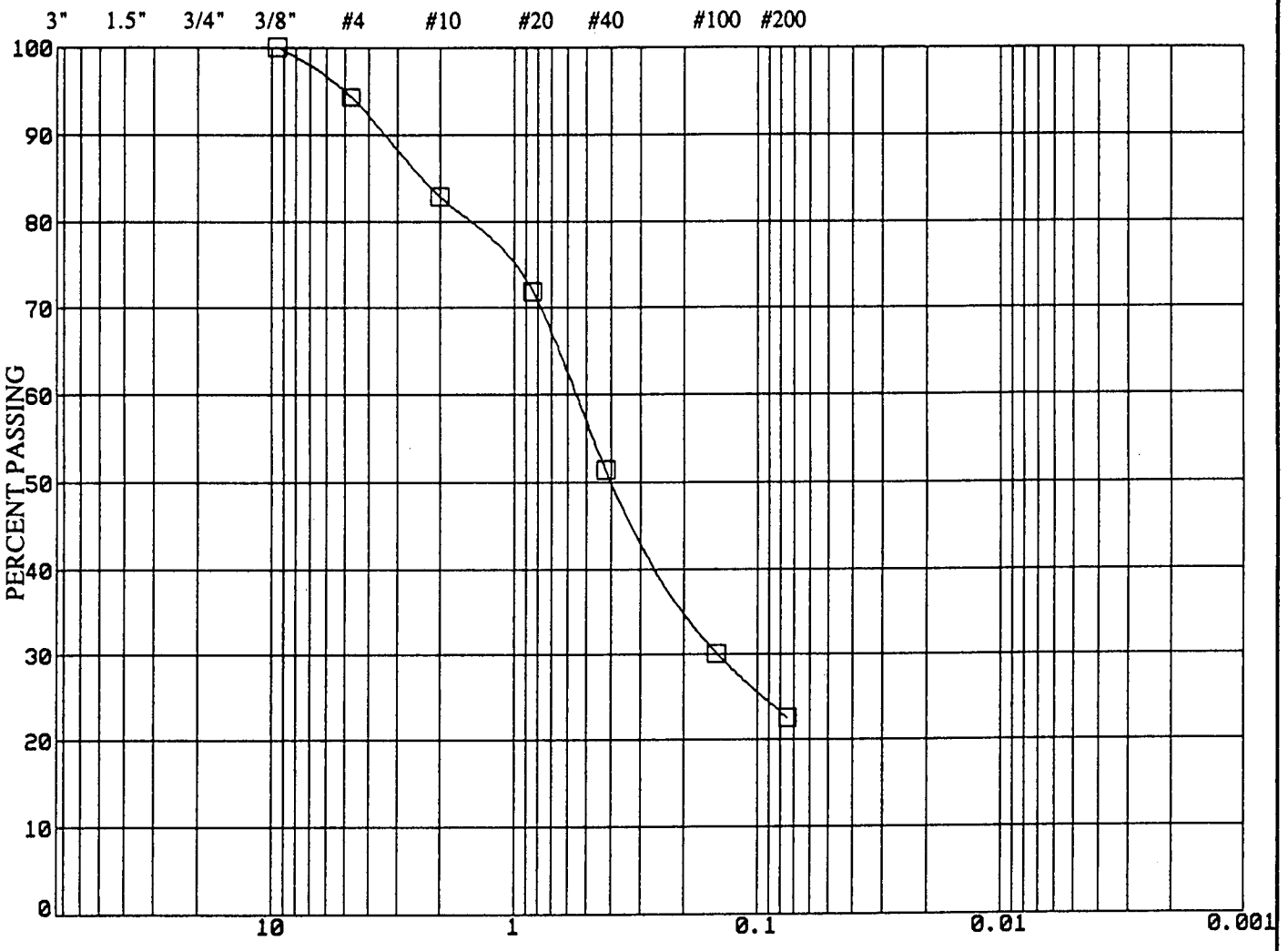
TESTING & CONSULTING
Chemical •
Microbiological •
Environmental •

GRAIN SIZE DISTRIBUTION

Client: **Rancho Las Flores Limited Partnership**	W.O. **215-1**
Project: **Highway 138**	Location: **San Bernardino County**

SIEVE ANALYSIS | HYDROMETER ANALYSIS



PARTICLE SIZE IN MILLIMETERS

GRAVEL		SAND			SILT	CLAY
coarse	fine	coarse	medium	fine		

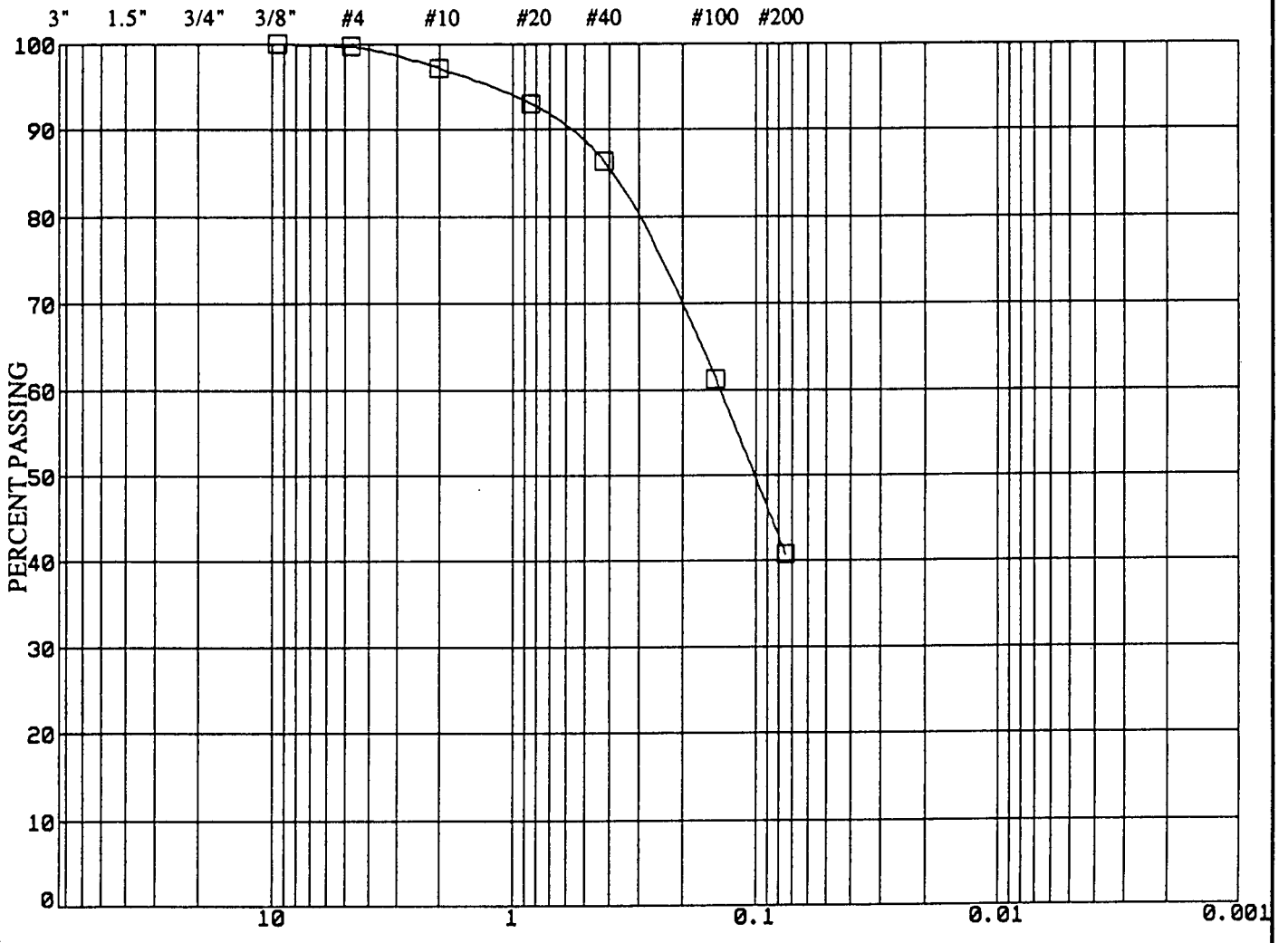
Sample: **B-1A @ 5.0 ft.**

John A. Sayers & Associates GEOTECHNICAL CONSULTANTS	Plate

GRAIN SIZE DISTRIBUTION

Client: **Rancho Las Flores Limited Partnership**	W.O. **215-1**
Project: **Highway 138**	Location: **San Bernardino County**

SIEVE ANALYSIS | HYDROMETER ANALYSIS



PARTICLE SIZE IN MILLIMETERS

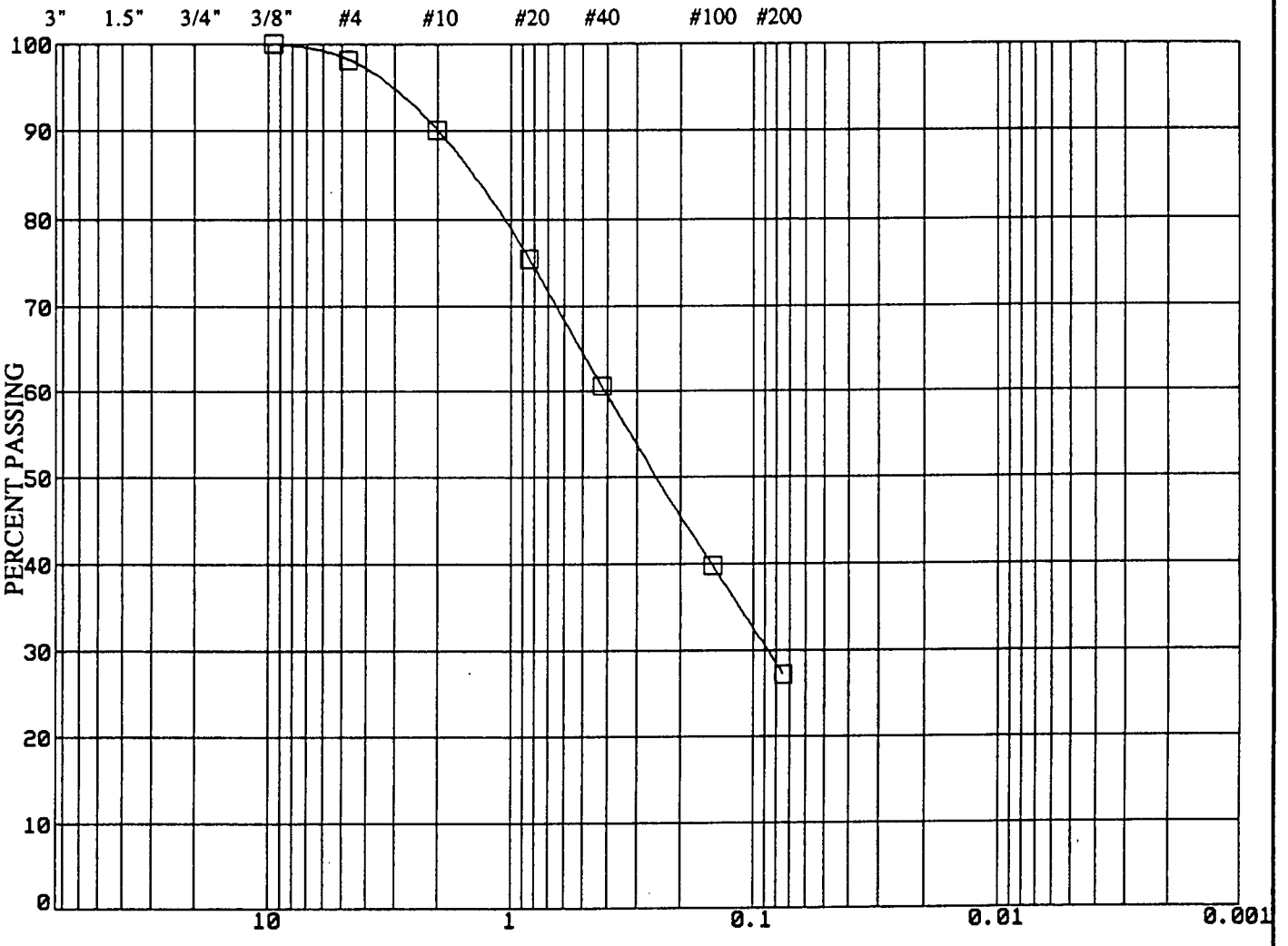
GRAVEL		SAND			SILT	CLAY
coarse	fine	coarse	medium	fine		

Sample: **B-1A @ 10.0 ft.**

GRAIN SIZE DISTRIBUTION

Client: **Rancho Las Flores Limited Partnership**	W.O. **215-1**
Project: **Highway 138**	Location: **San Bernardino County**

SIEVE ANALYSIS | HYDROMETER ANALYSIS



PARTICLE SIZE IN MILLIMETERS

GRAVEL		SAND			SILT	CLAY
coarse	fine	coarse	medium	fine		

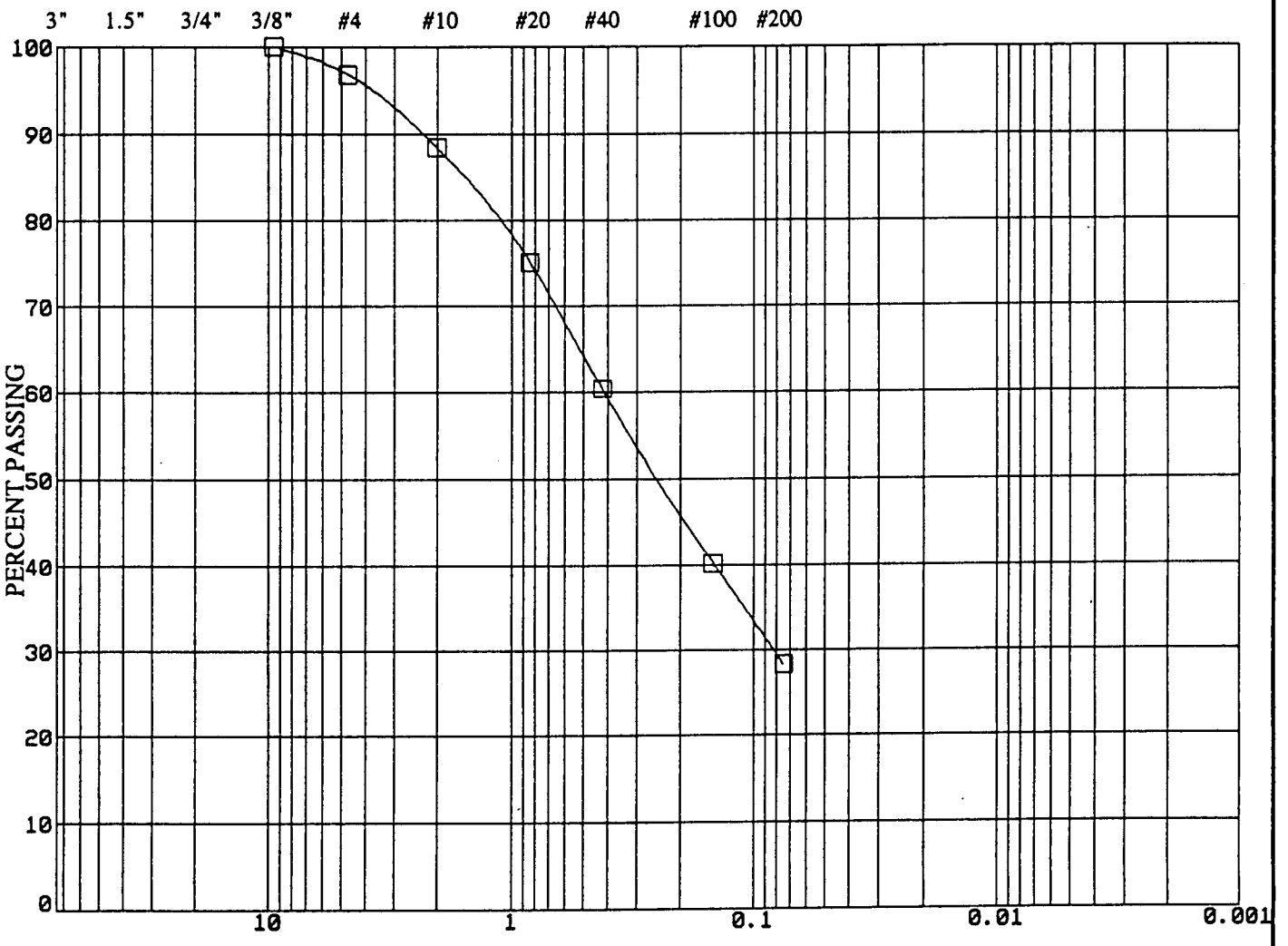
Sample: **B-1A @ 20.0 ft.**

John A. Sayers & Associates GEOTECHNICAL CONSULTANTS	Plate

GRAIN SIZE DISTRIBUTION

Client: **Rancho Las Flores Limited Partnership**	W.O. **215-1**
Project: **Highway 138**	Location: **San Bernardino County**

SIEVE ANALYSIS | HYDROMETER ANALYSIS



PARTICLE SIZE IN MILLIMETERS

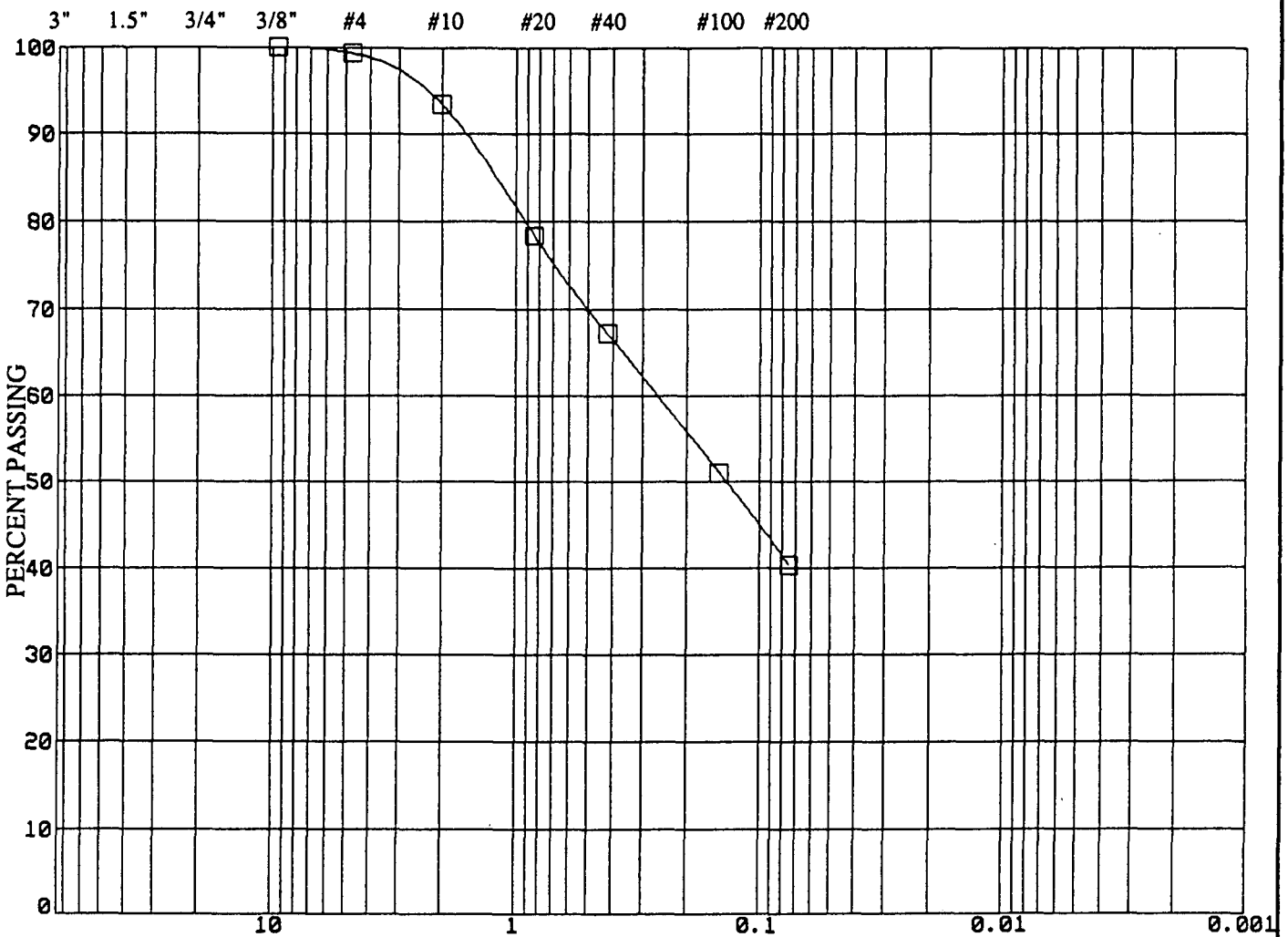
GRAVEL		SAND			SILT	CLAY
coarse	fine	coarse	medium	fine		

Sample: **B-2A @ 5.0 ft.**

GRAIN SIZE DISTRIBUTION

Client: **Rancho Las Flores Limited Partnership**	W.O. **215-1**
Project: **Highway 138**	Location: **San Bernardino County**

SIEVE ANALYSIS | HYDROMETER ANALYSIS



PARTICLE SIZE IN MILLIMETERS

GRAVEL		SAND			SILT	CLAY
coarse	fine	coarse	medium	fine		

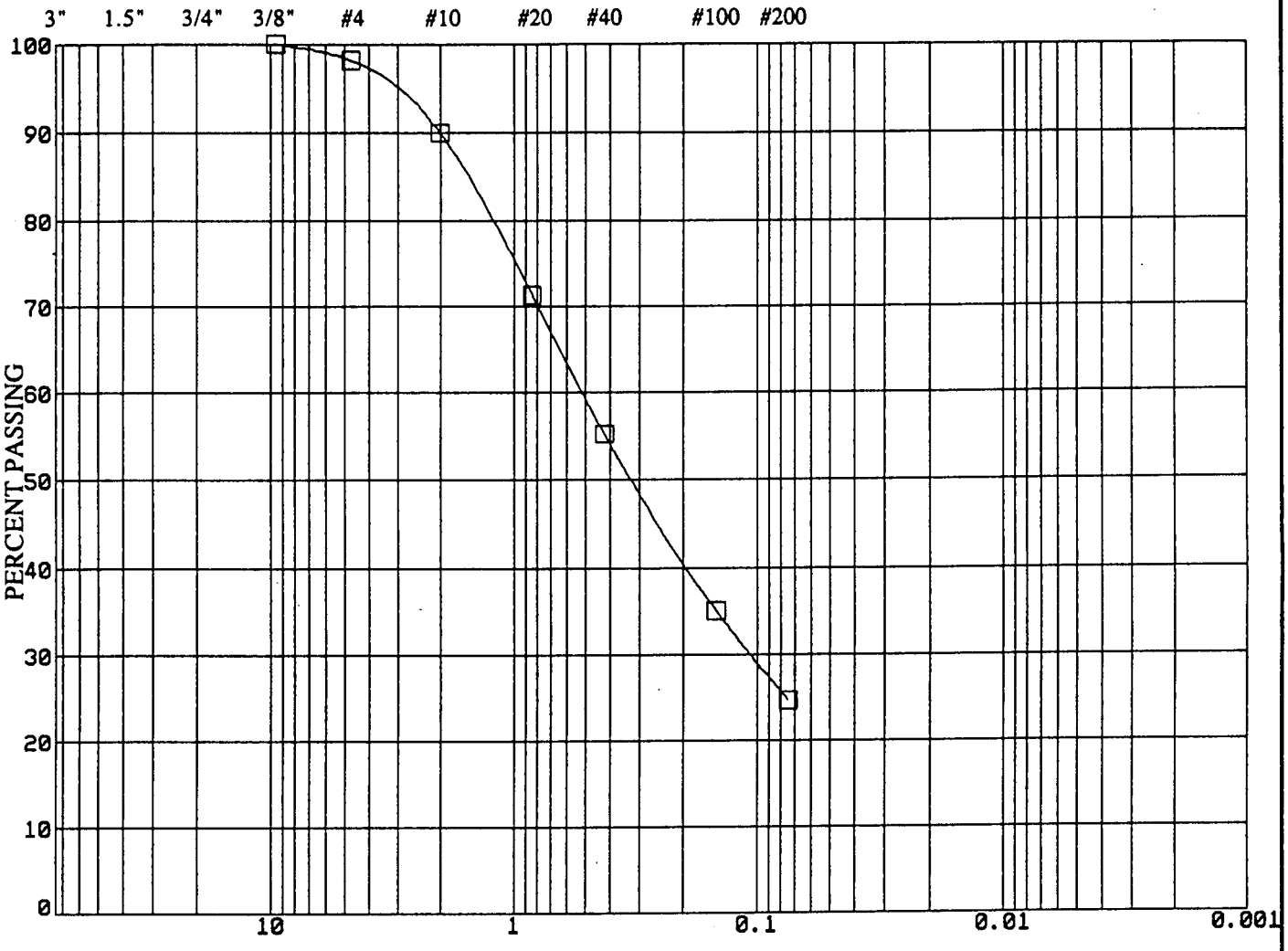
Sample: **B-2A @ 10.0 ft.**

John A. Sayers & Associates GEOTECHNICAL CONSULTANTS	Plate

GRAIN SIZE DISTRIBUTION

Client: **Rancho Las Flores Limited Partnership**	W.O. **215-1**
Project: **Highway 138**	Location: **San Bernardino County**

SIEVE ANALYSIS | HYDROMETER ANALYSIS



PARTICLE SIZE IN MILLIMETERS

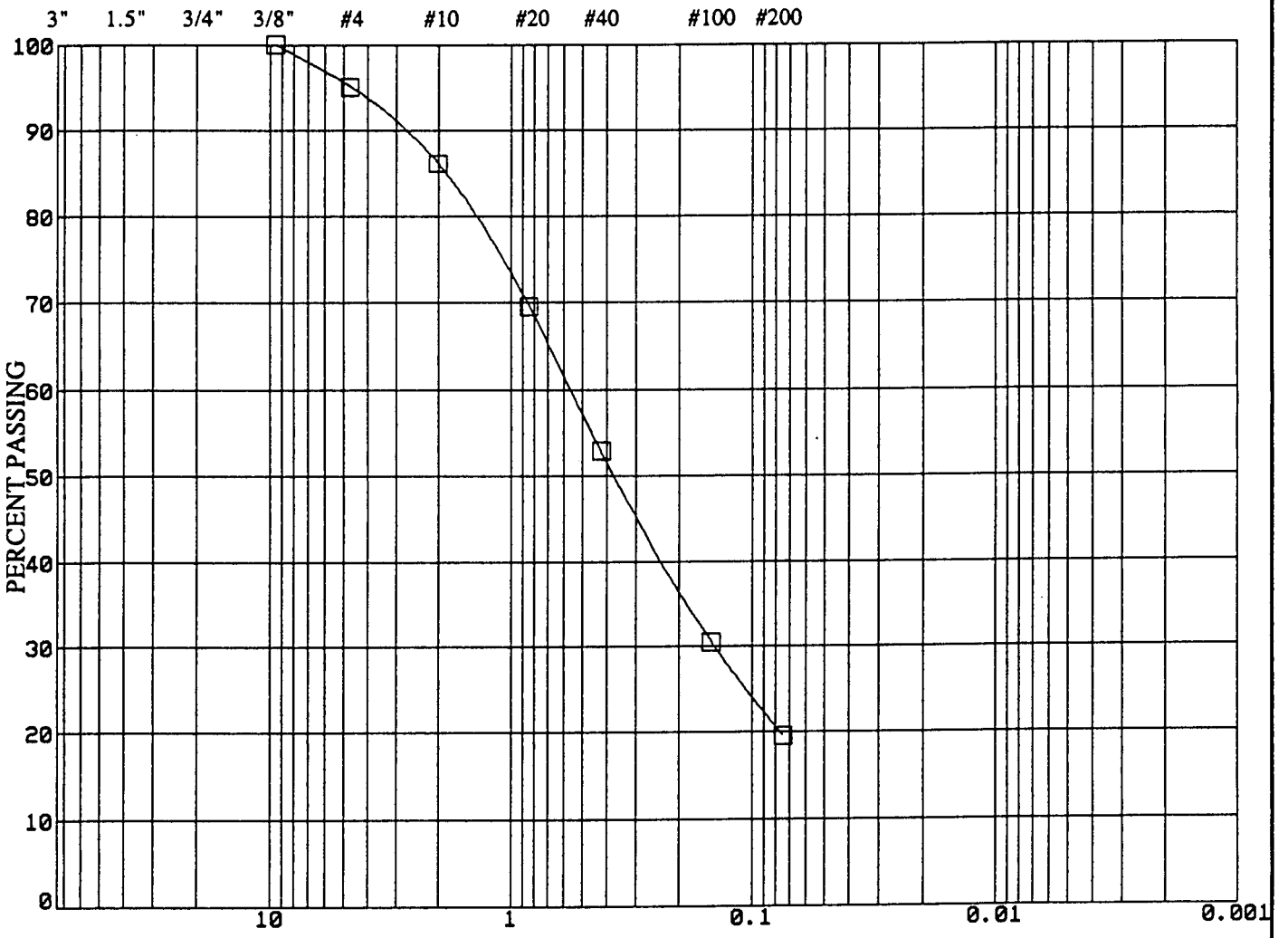
GRAVEL		SAND			SILT	CLAY
coarse	fine	coarse	medium	fine		

Sample: **B-2A @ 35.0 ft.**

GRAIN SIZE DISTRIBUTION

Client: **Rancho Las Flores Limited Partnership**	W.O. **215-1**
Project: **Highway 138**	Location: **San Bernardino County**

SIEVE ANALYSIS | HYDROMETER ANALYSIS



PARTICLE SIZE IN MILLIMETERS

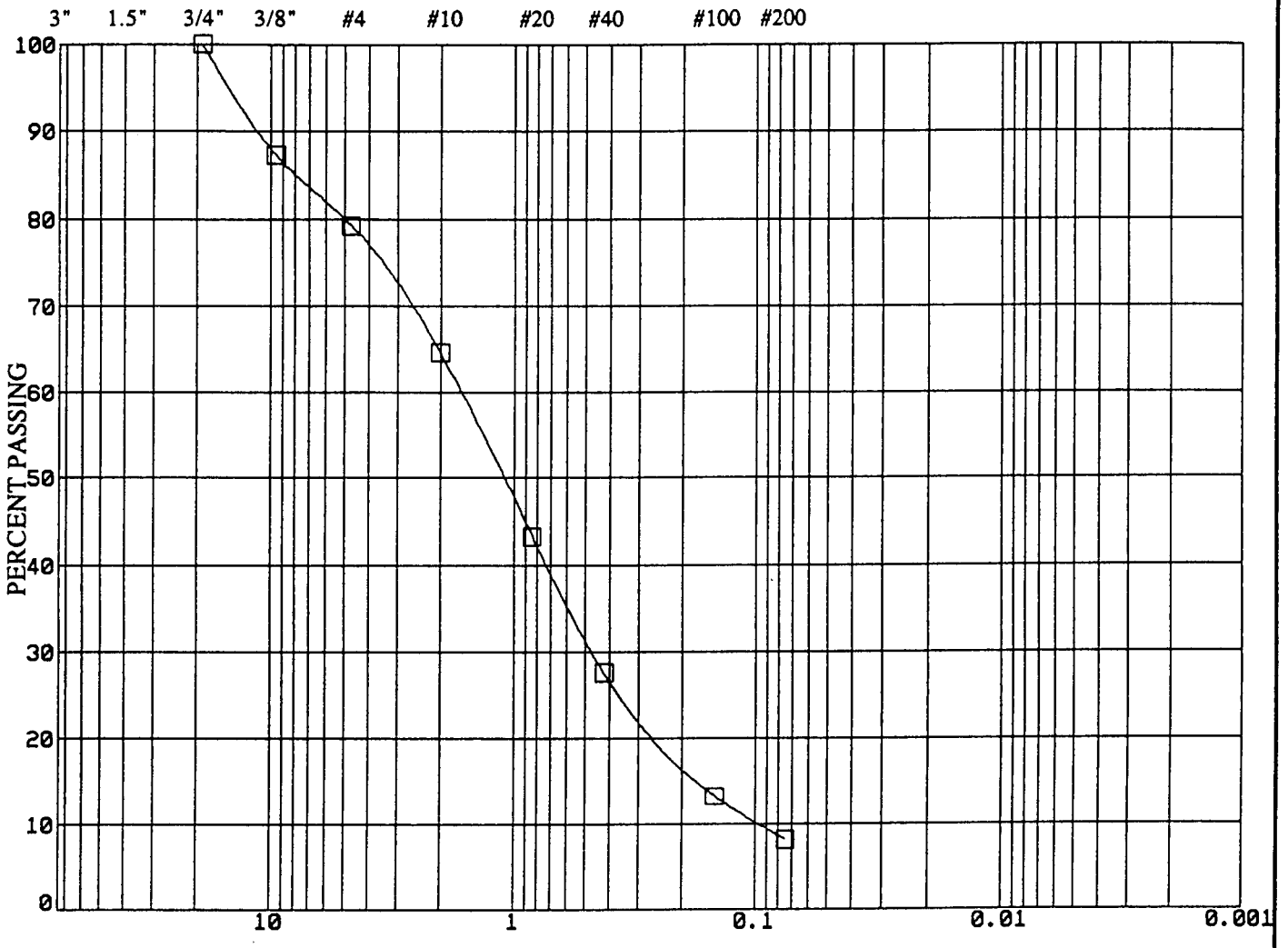
GRAVEL		SAND			SILT	CLAY
coarse	fine	coarse	medium	fine		

Sample: **B-2A @ 45.0 ft.**

GRAIN SIZE DISTRIBUTION

Client: **Rancho Las Flores Limited Partnership**	W.O. **215-1**
Project: **Highway 138**	Location: **San Bernardino County**

SIEVE ANALYSIS | HYDROMETER ANALYSIS



PARTICLE SIZE IN MILLIMETERS

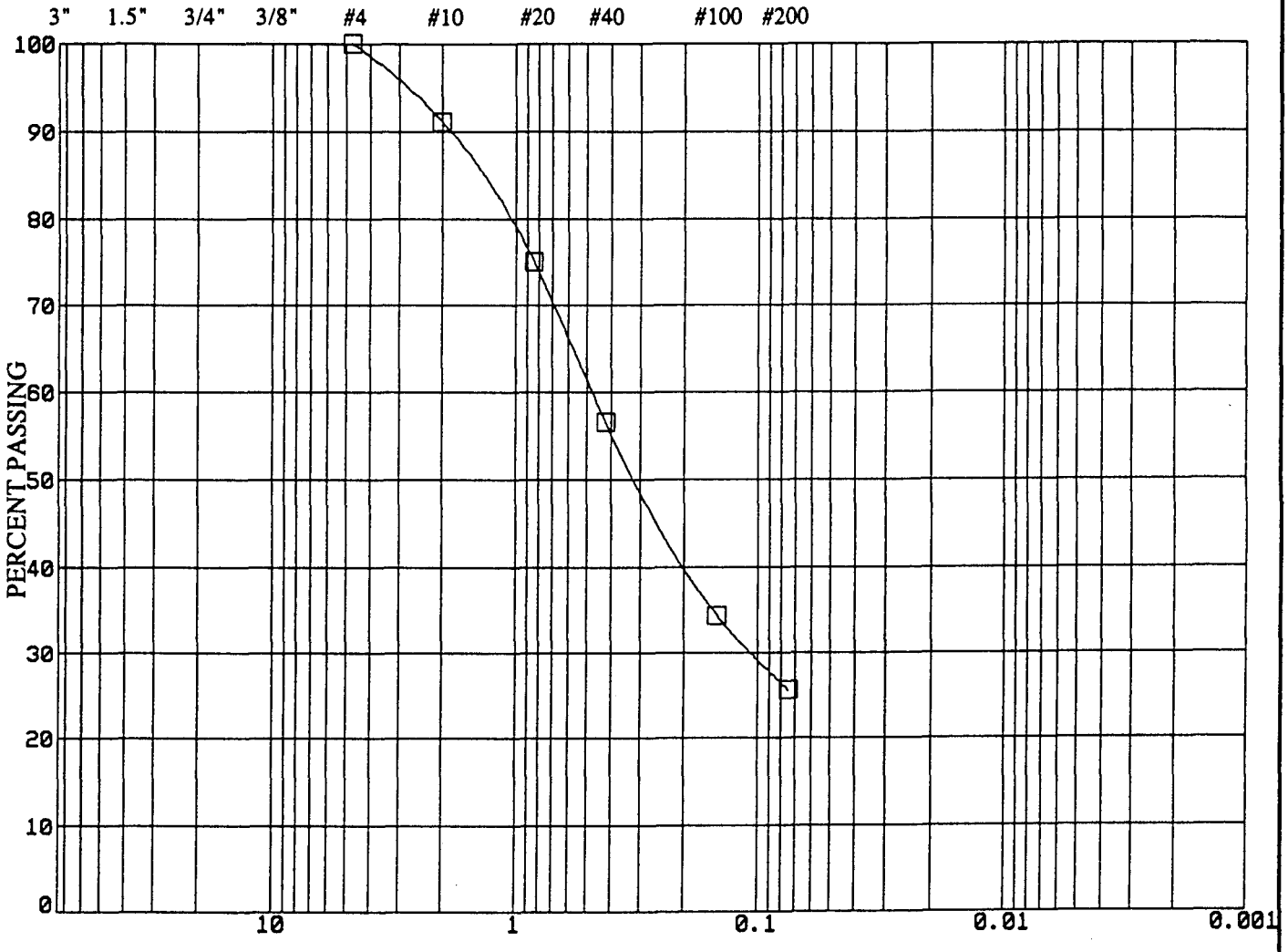
GRAVEL		SAND			SILT	CLAY
coarse	fine	coarse	medium	fine		

Sample: **B-1 @ 8.0 ft.**

GRAIN SIZE DISTRIBUTION

Client: **Rancho Las Flores Limited Partnership**	W.O. **215-1**
Project: **Highway 138**	Location: **San Bernardino County**

SIEVE ANALYSIS | HYDROMETER ANALYSIS



PARTICLE SIZE IN MILLIMETERS

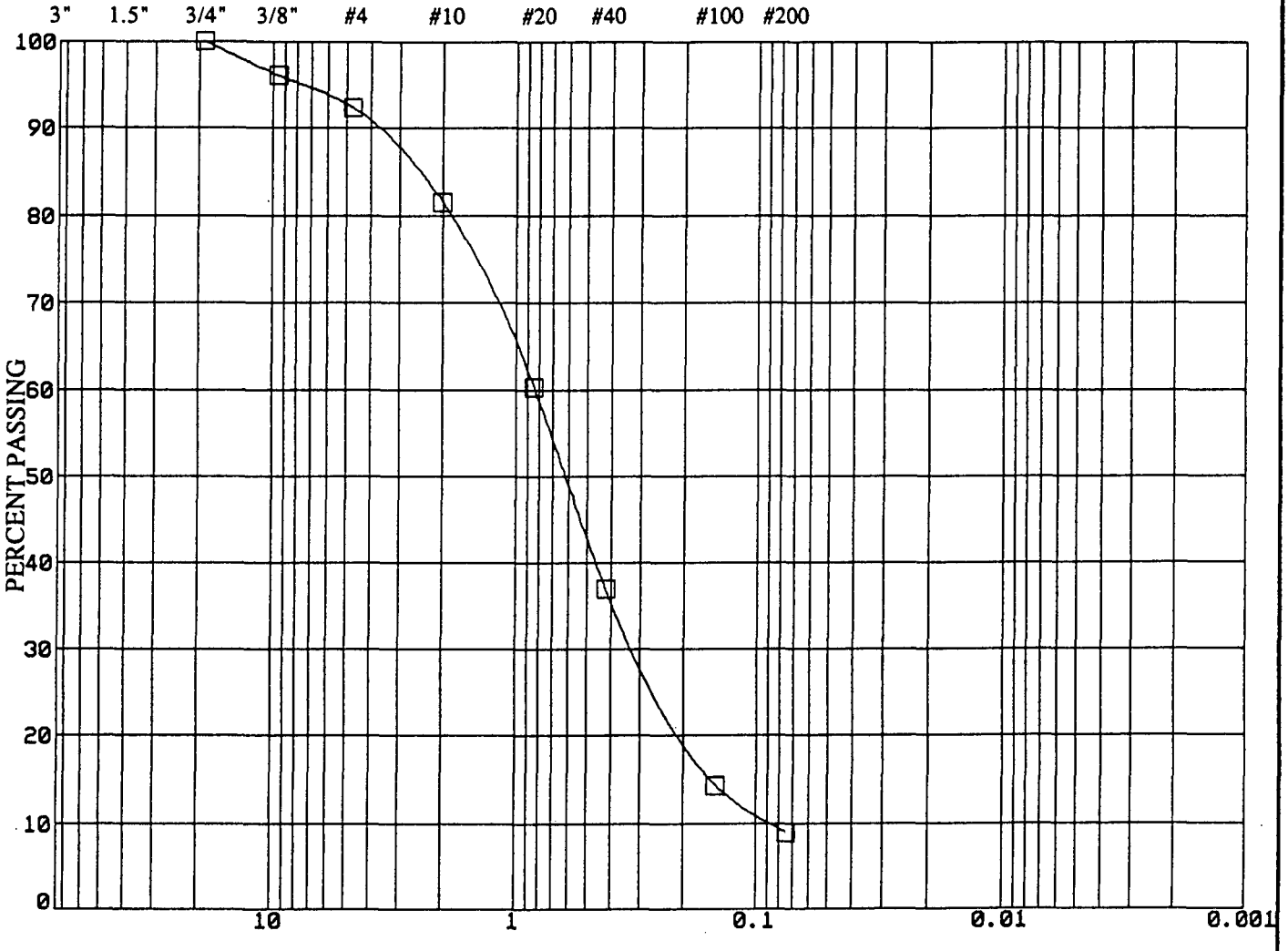
GRAVEL		SAND			SILT	CLAY
coarse	fine	coarse	medium	fine		

Sample: **B-2 @ 2.0 ft.**

GRAIN SIZE DISTRIBUTION

Client: **Rancho Las Flores Limited Partnership**	W.O. **215-1**
Project: **Highway 138**	Location: **San Bernardino County**

SIEVE ANALYSIS | HYDROMETER ANALYSIS



PARTICLE SIZE IN MILLIMETERS

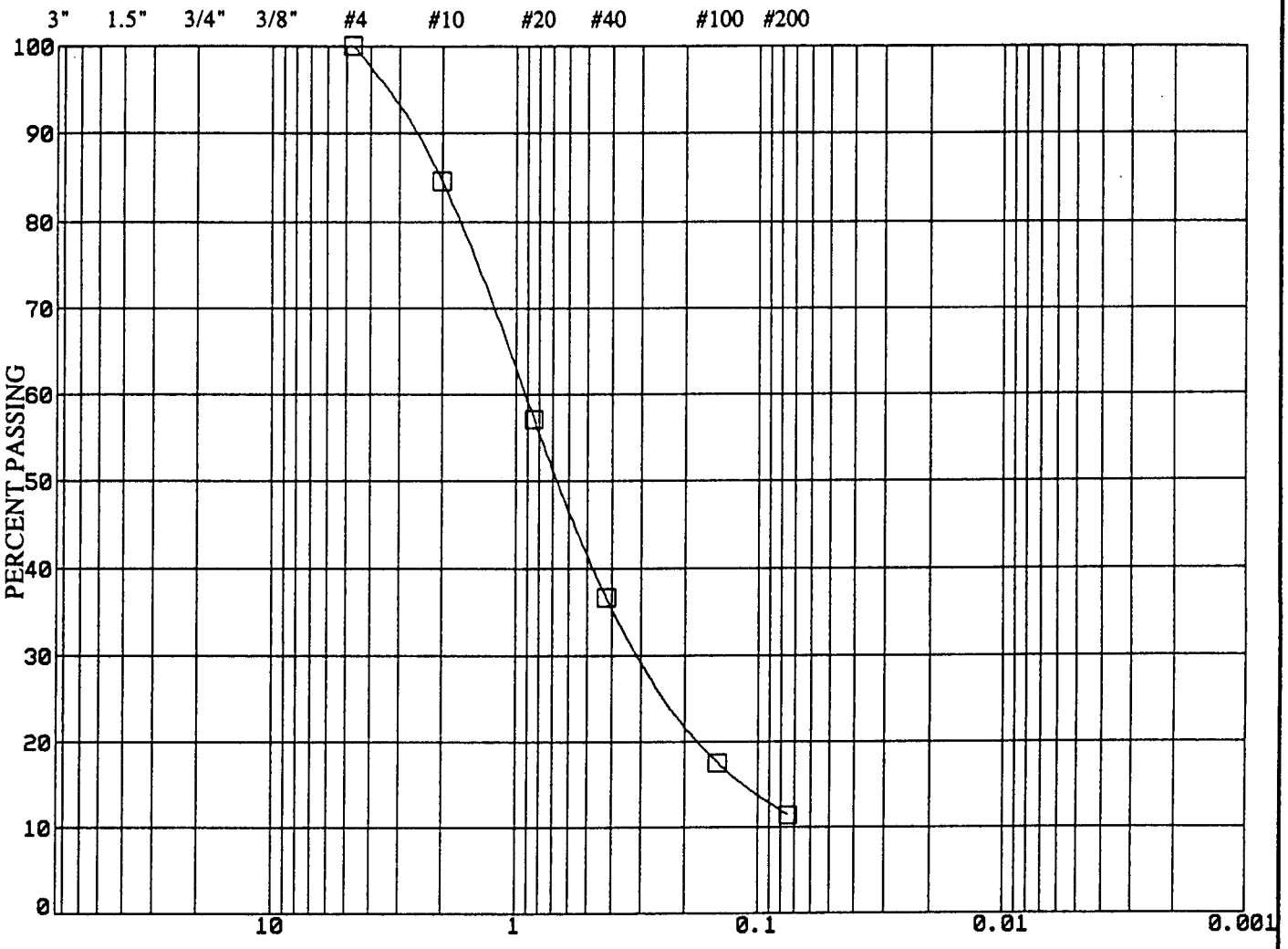
GRAVEL		SAND			SILT	CLAY
coarse	fine	coarse	medium	fine		

Sample: **B-4 @ 5.0 ft.**

GRAIN SIZE DISTRIBUTION

Client: **Rancho Las Flores Limited Partnership**	W.O. **215-1**
Project: **Highway 138**	Location: **San Bernardino County**

SIEVE ANALYSIS | HYDROMETER ANALYSIS



PARTICLE SIZE IN MILLIMETERS

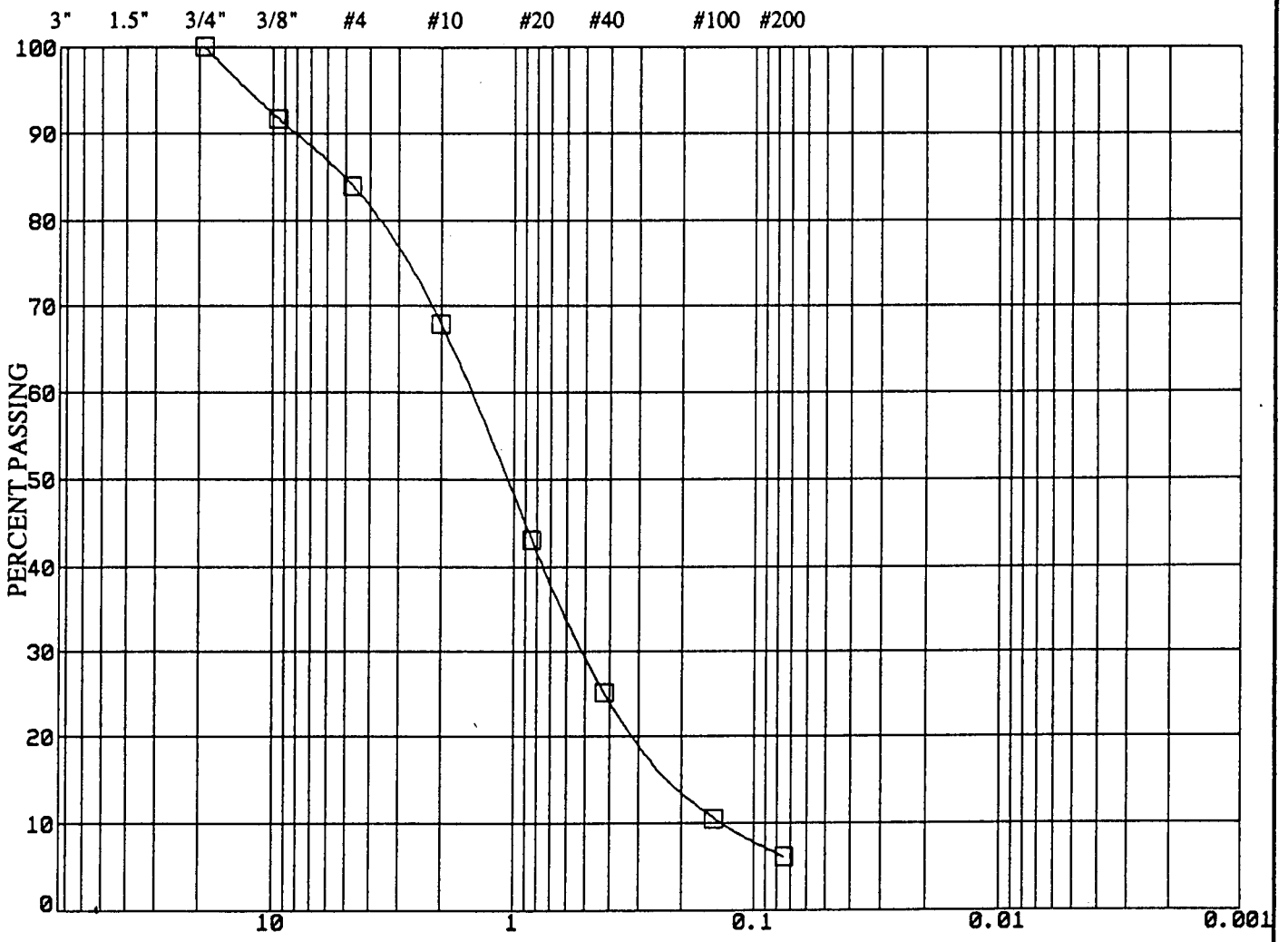
GRAVEL		SAND			SILT	CLAY
coarse	fine	coarse	medium	fine		

Sample: **B-6 @ 6.0 ft.**

GRAIN SIZE DISTRIBUTION

Client: **Rancho Las Flores Limited Partnership**	W.O. **215-1**
Project: **Highway 138**	Location: **San Bernardino County**

SIEVE ANALYSIS | HYDROMETER ANALYSIS



PARTICLE SIZE IN MILLIMETERS

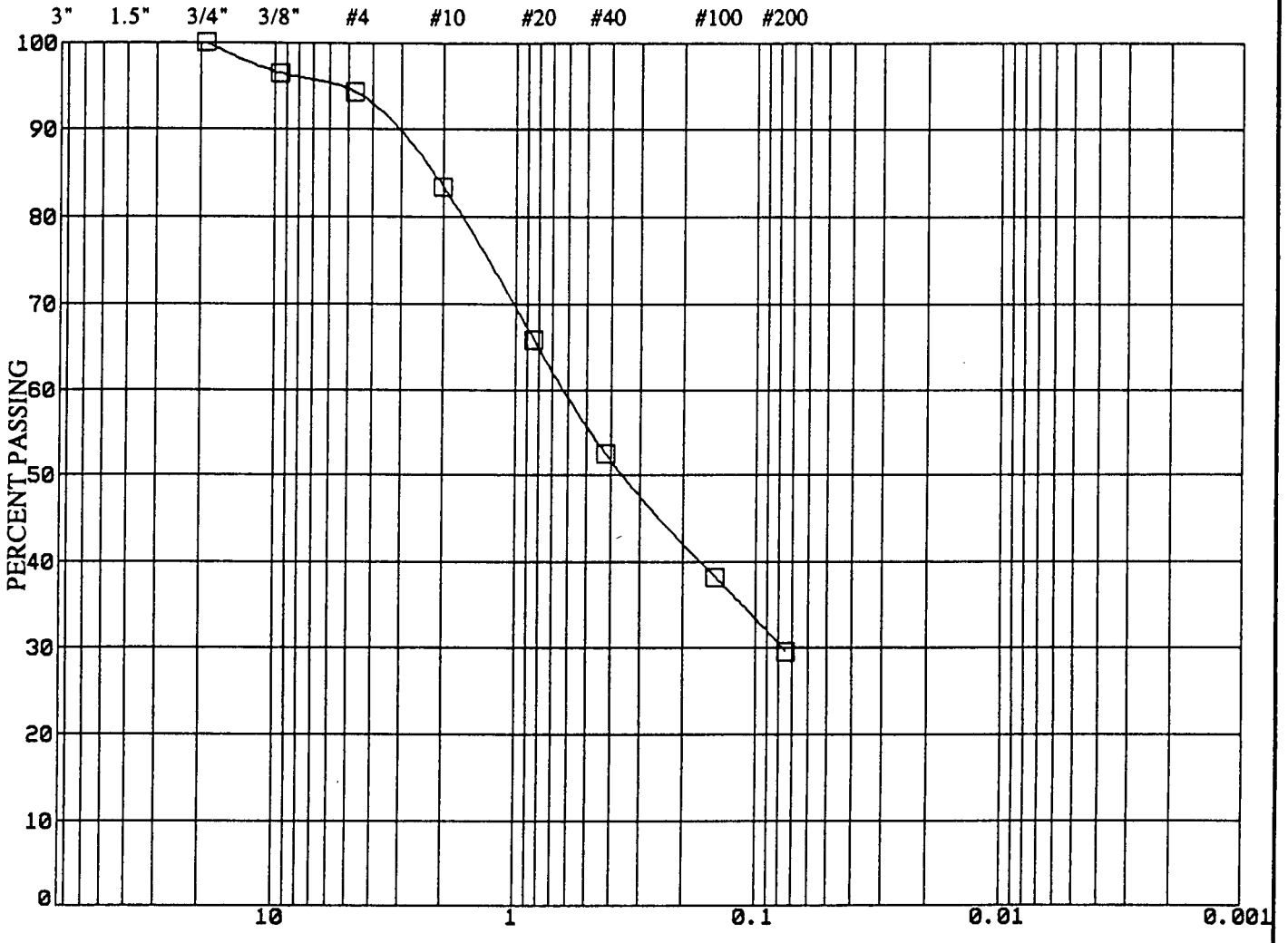
GRAVEL		SAND			SILT	CLAY
coarse	fine	coarse	medium	fine		

Sample: **B-9 @ 2.5 ft.**

GRAIN SIZE DISTRIBUTION

Client: **Rancho Las Flores Limited Partnership**	W.O. **215-1**
Project: **Highway 138**	Location: **San Bernardino County**

SIEVE ANALYSIS | HYDROMETER ANALYSIS



PARTICLE SIZE IN MILLIMETERS

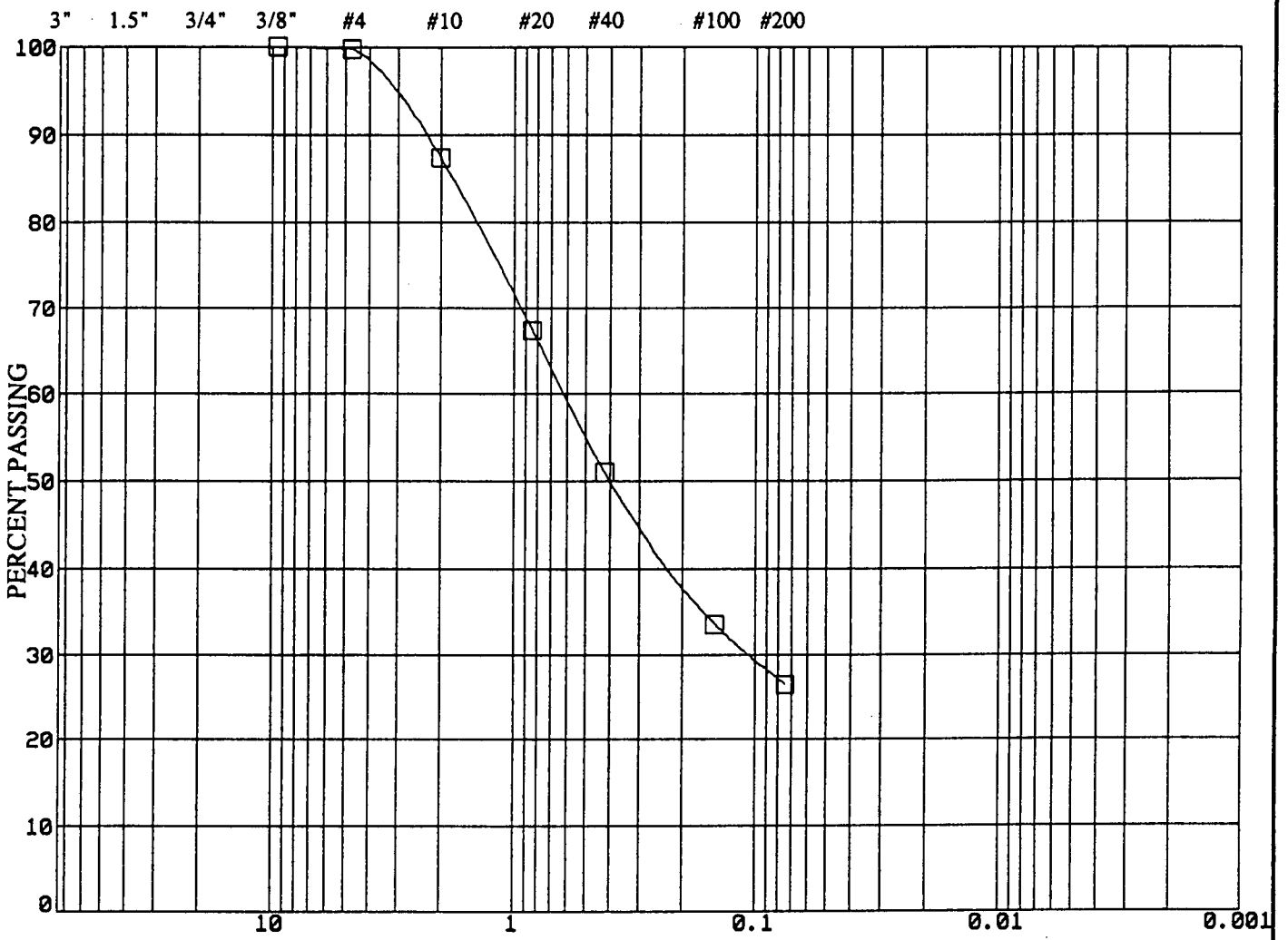
GRAVEL		SAND			SILT	CLAY
coarse	fine	coarse	medium	fine		

Sample: **B-10 @ 3.0 ft.**

GRAIN SIZE DISTRIBUTION

Client: **Rancho Las Flores Limited Partnership**	W.O. **215-1**
Project: **Highway 138**	Location: **San Bernardino County**

SIEVE ANALYSIS | HYDROMETER ANALYSIS



PARTICLE SIZE IN MILLIMETERS

GRAVEL		SAND			SILT	CLAY
coarse	fine	coarse	medium	fine		

Sample: **B-13 @ 15.0 ft.**

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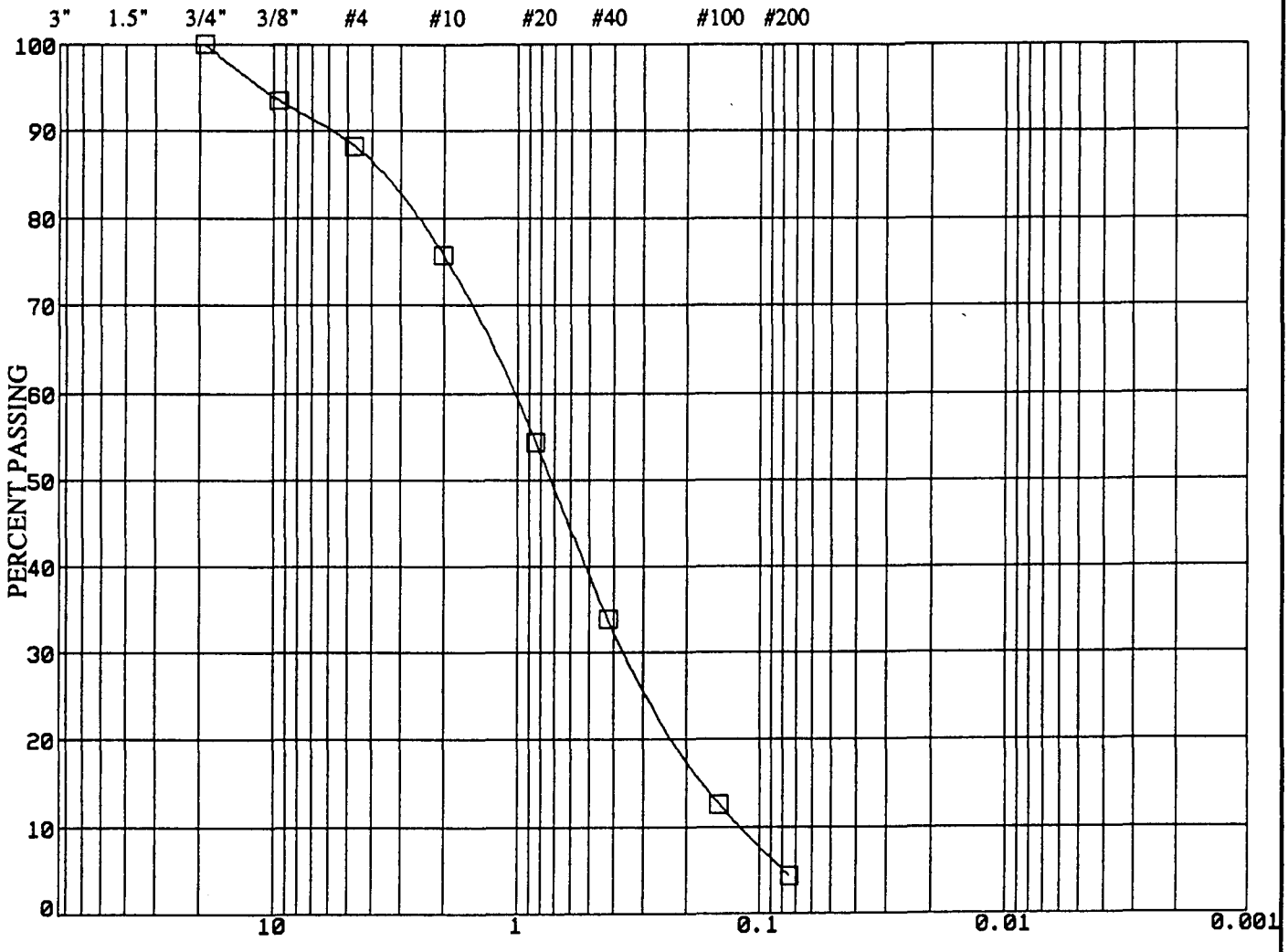
GRAIN SIZE DISTRIBUTION

Client: **Rancho Las Flores Limited Partnership**	W.O. **215-1**
Project: **Highway 138**	Location: **San Bernardino County**

SIEVE ANALYSIS | HYDROMETER ANALYSIS



PARTICLE SIZE IN MILLIMETERS

GRAVEL		SAND			SILT	CLAY
coarse	fine	coarse	medium	fine		

Sample: **B-14 @ 2.5 ft.**

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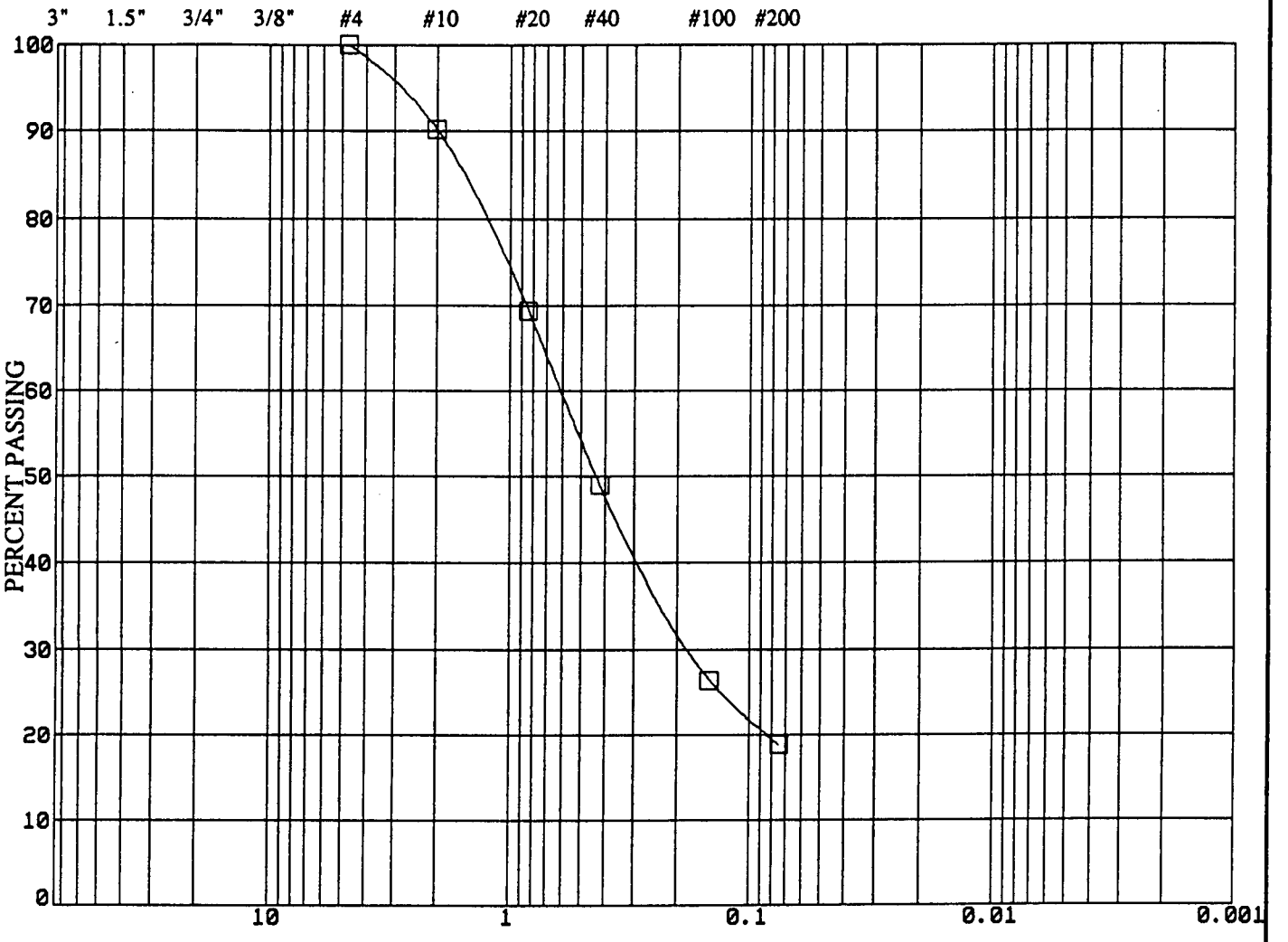
GRAIN SIZE DISTRIBUTION

Client: **Rancho Las Flores Limited Partnership**	W.O. **215-1**
Project: **Highway 138**	Location: **San Bernardino County**

SIEVE ANALYSIS | HYDROMETER ANALYSIS



PARTICLE SIZE IN MILLIMETERS

GRAVEL		SAND			SILT	CLAY
coarse	fine	coarse	medium	fine		

Sample: **B-16 @ 3.0 ft.**

SHEAR TEST SUMMARY

Undisturbed, Tested at Natural Moisture Content

Sample Location		Normal Pressure (psf) 2000	4000	6000
B-1A @ 10'	Shear Strength (psf)	2172	4560	7404
B-1A @ 20'		3864	4956	7752
B-1A @ 40'		3576	4428	5736 (Low)
B-2A @ 10'		4332 (High)	5364	8412 (High)
B-2A @ 15'		3684	5892 (High)	7908
B-2A @ 20'		2436	4548	7164
B-2A @ 35'		2136 (Low)	4512	7752
B-2A @ 45'		2676	4212 (Low)	7272
Median		3234	5052	7074

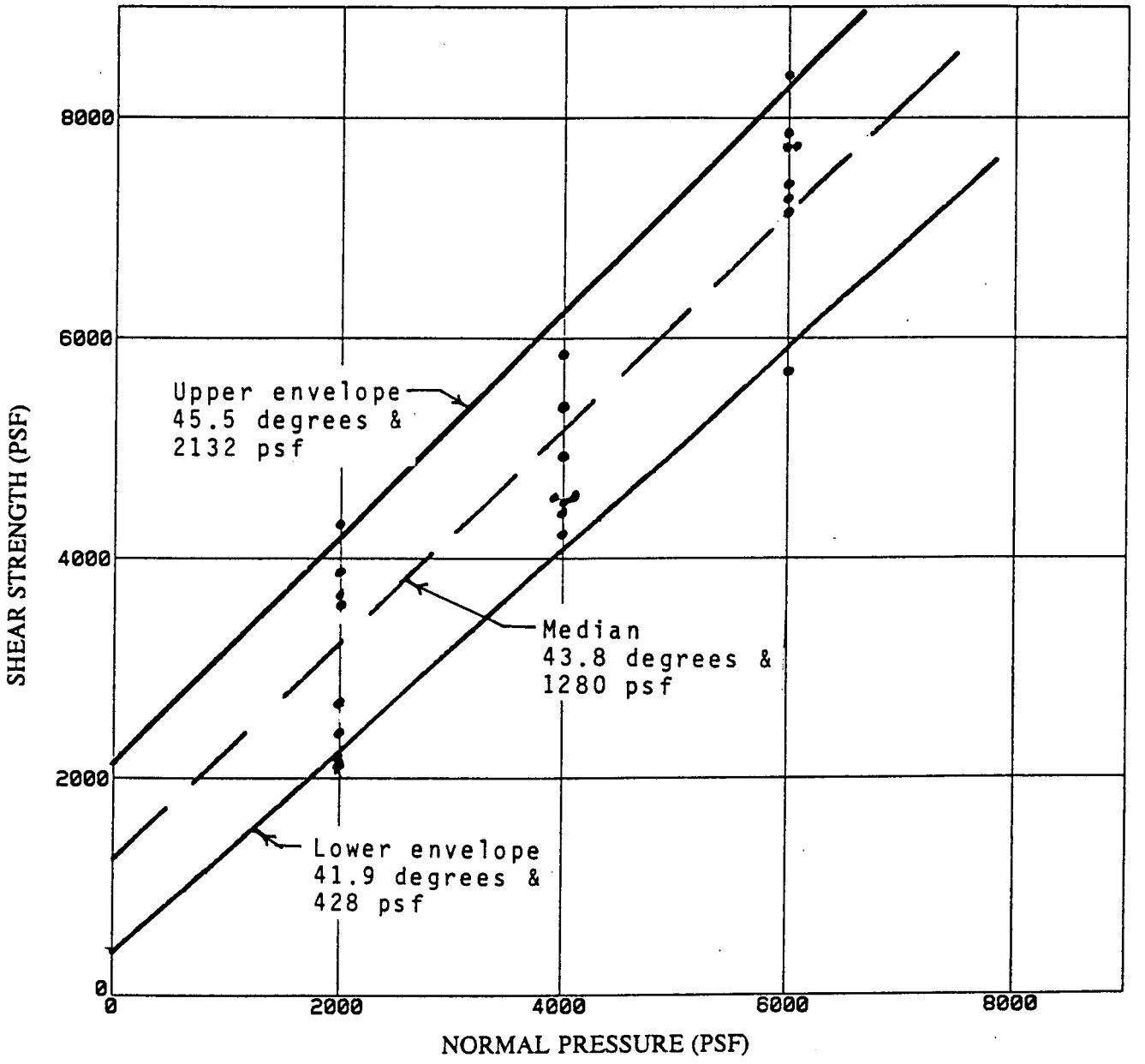
Upper envelope (1)	45.5 degrees and 2132 psf
Lower envelope (1)	41.9 degrees and 428 psf
Median of envelopes (2)	43.8 degrees and 1280 psf

(1) Calculated by computer using low and high shear strengths

(2) Calculated by computer as mathematical average of high and low shear strengths

SHEAR TEST DIAGRAM

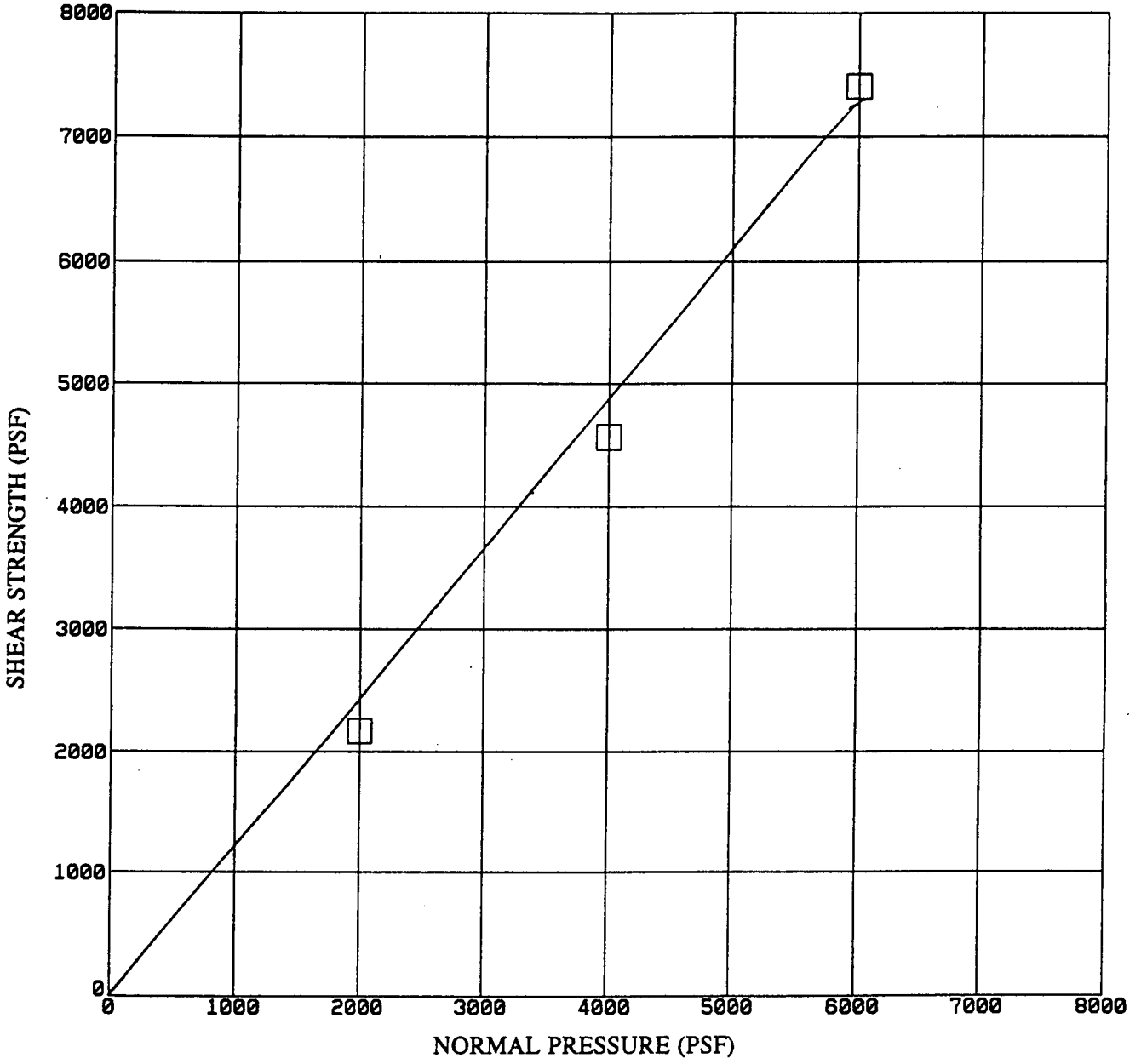
Client: Rancho Las Flores Limited Partnership	W.O. 215-1
Project: Highway 138	Location: San Bernardino County



SHEAR TEST Summary
Undisturbed Samples
Sheared at Natural Moisture

SHEAR TEST DIAGRAM

Client: **Rancho Las Flores Limited Partnership**	W.O. **215-1**
Project: **Highway 138**	Location: **San Bernardino County**



Initial Sample

Dry Density = 126.1 pcf
Moist. Cont. = 8.1%
Silty Sand
L.L. = 27.2
P.I. = 5.9

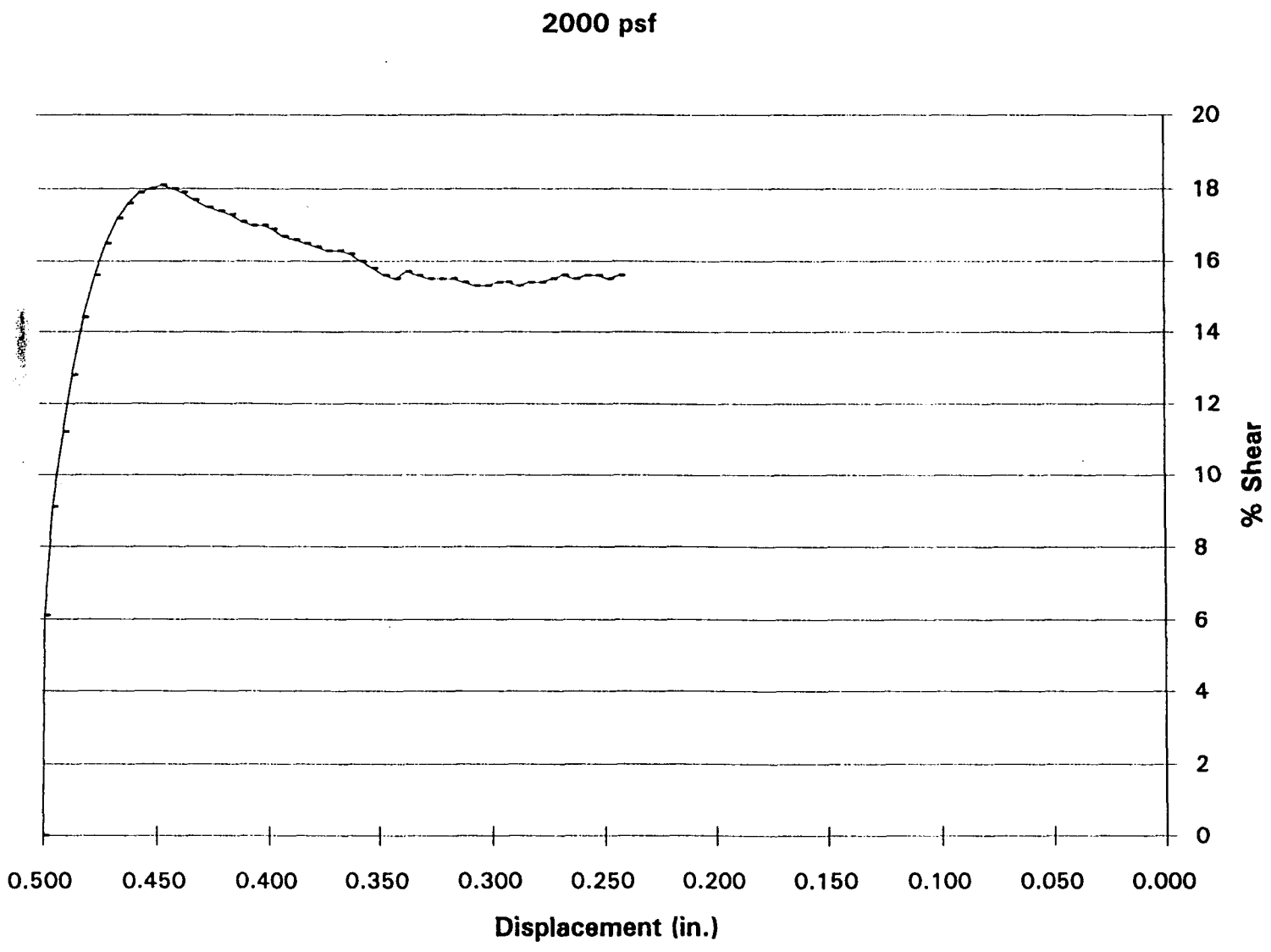
Sample: **B-1A @ 10.0 ft.**

Phi Angle: **50 deg.**

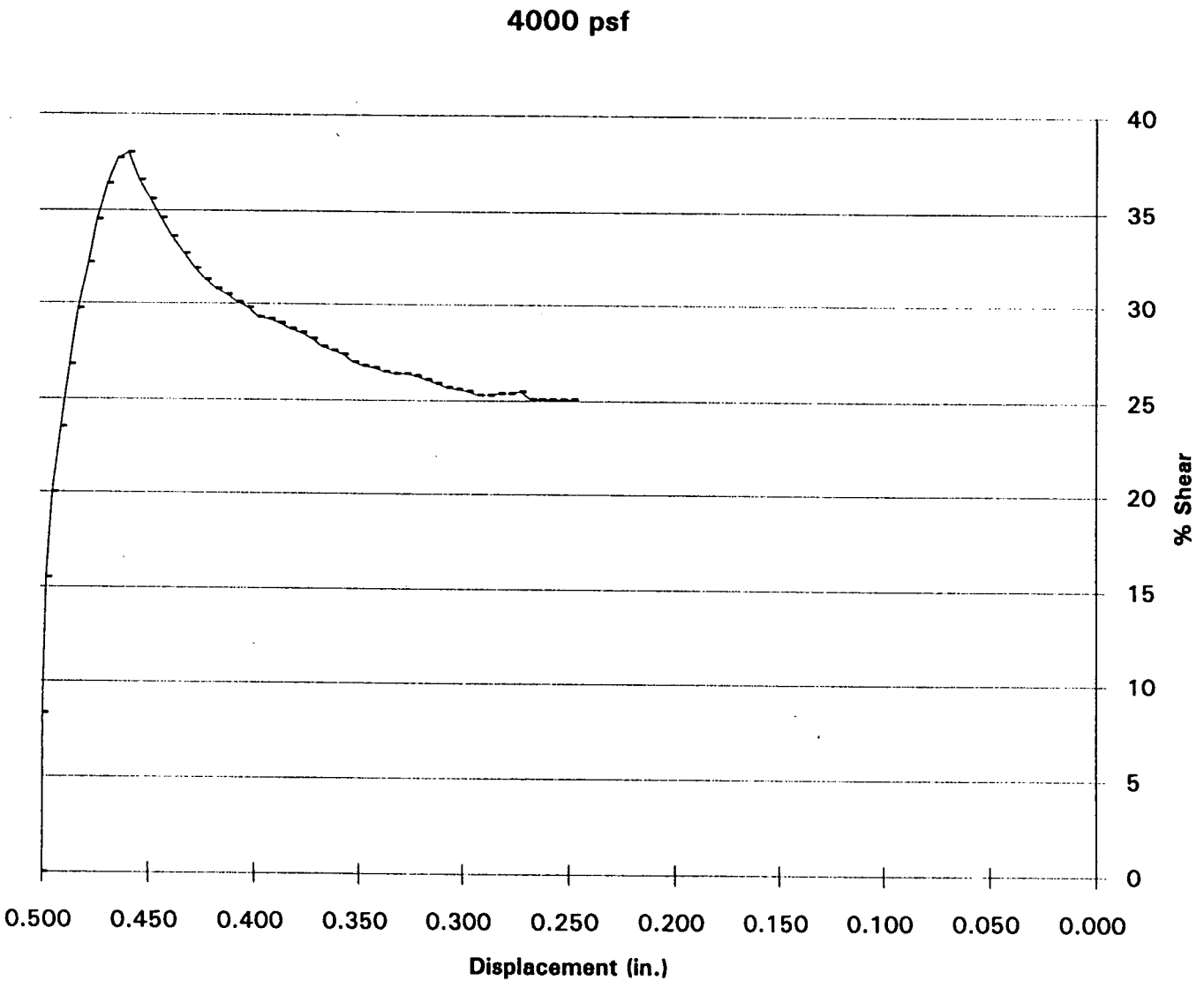
Cohesion: **0 psf**

TEST METHOD: Undisturbed, shear at natural moisture

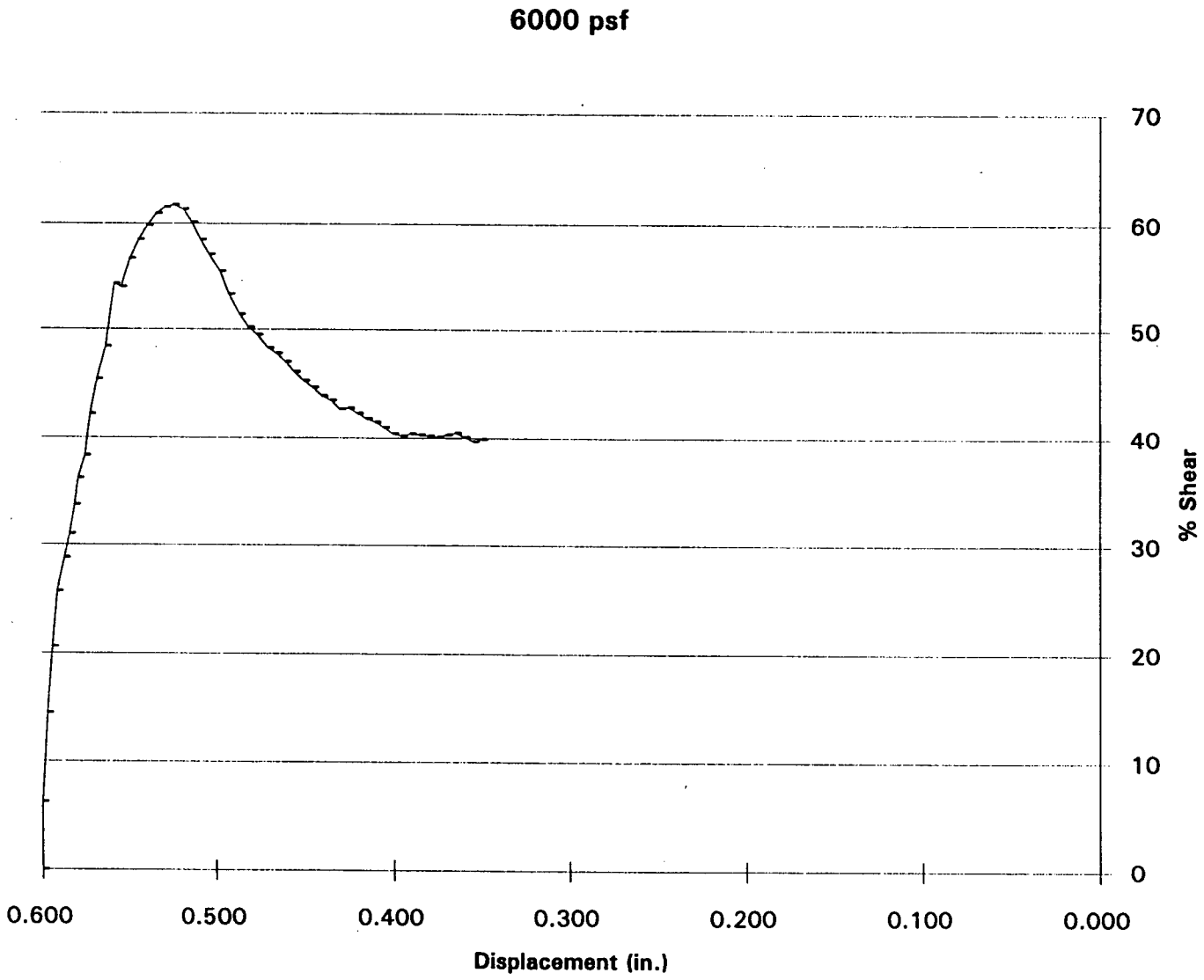
DIRECT SHEAR TEST PLOT OF B1A @ 10 FT.



DIRECT SHEAR TEST PLOT OF B1A @ 10 FT.

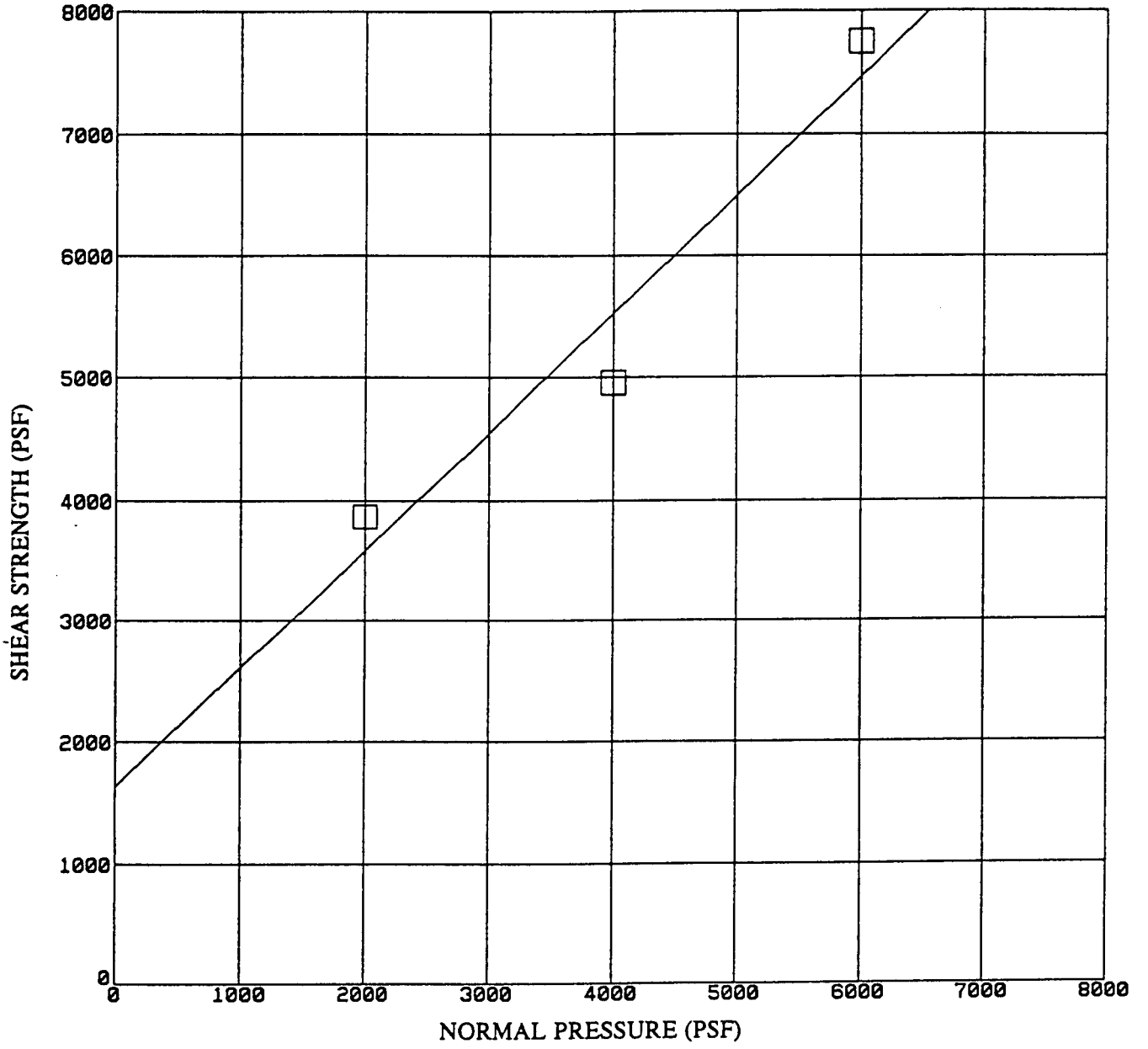


DIRECT SHEAR TEST PLOT OF B1A @ 10 FT.



SHEAR TEST DIAGRAM

Client: Rancho Las Flores Limited Partnership	W.O. 215-1
Project: Highway 138	Location: San Bernardino County



Initial Sample

Dry Density = 127.4 pcf
Moist. Cont. = 10.4%
Silty Sand
L.L. = 24.6
P.I. = 2.3

Sample: B-1A @ 20.0 ft.

Phi Angle: 44 deg.
Cohesion: 1636 psf

TEST CONDITION: Undisturbed, sheared at natural moisture

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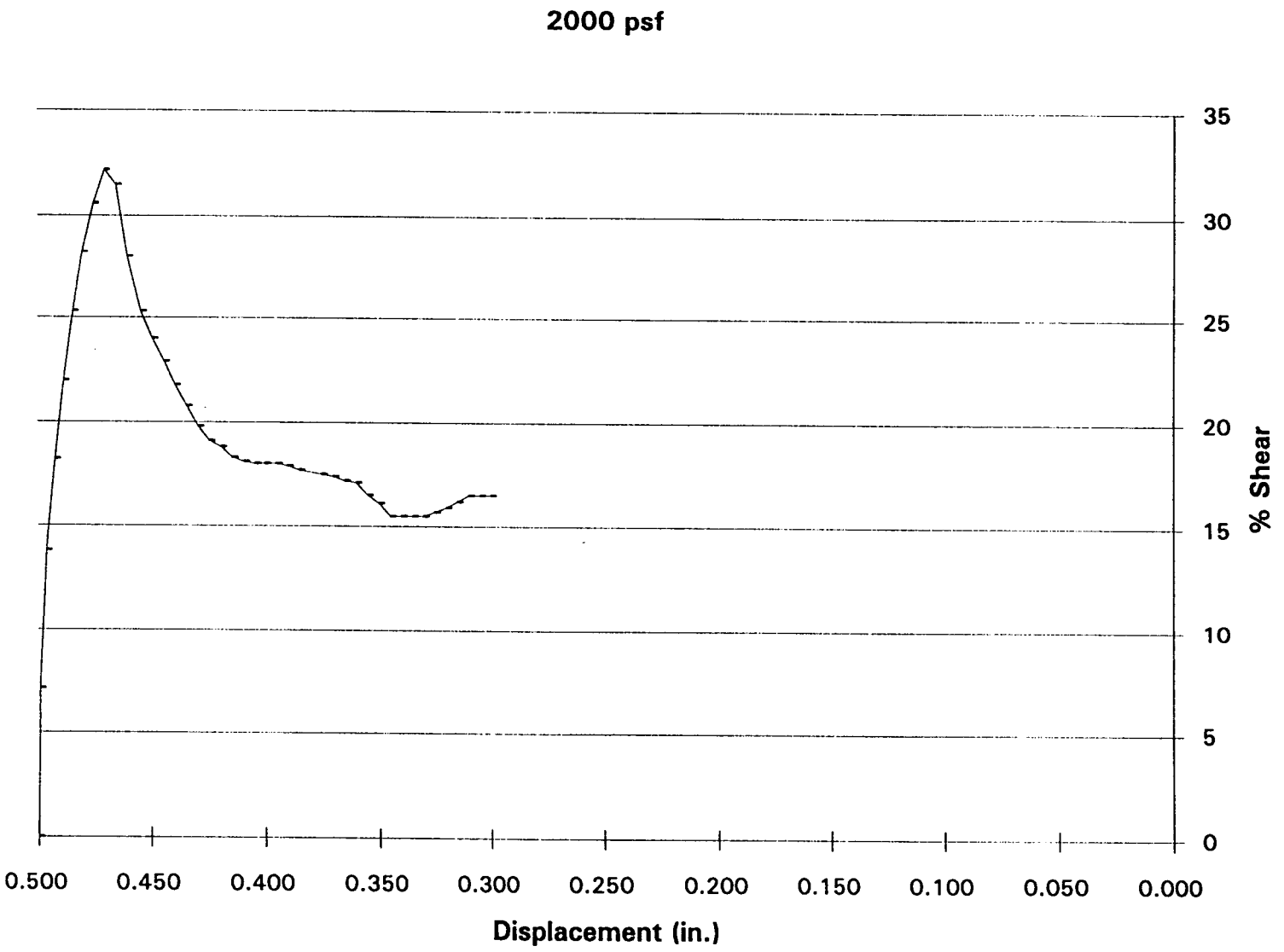
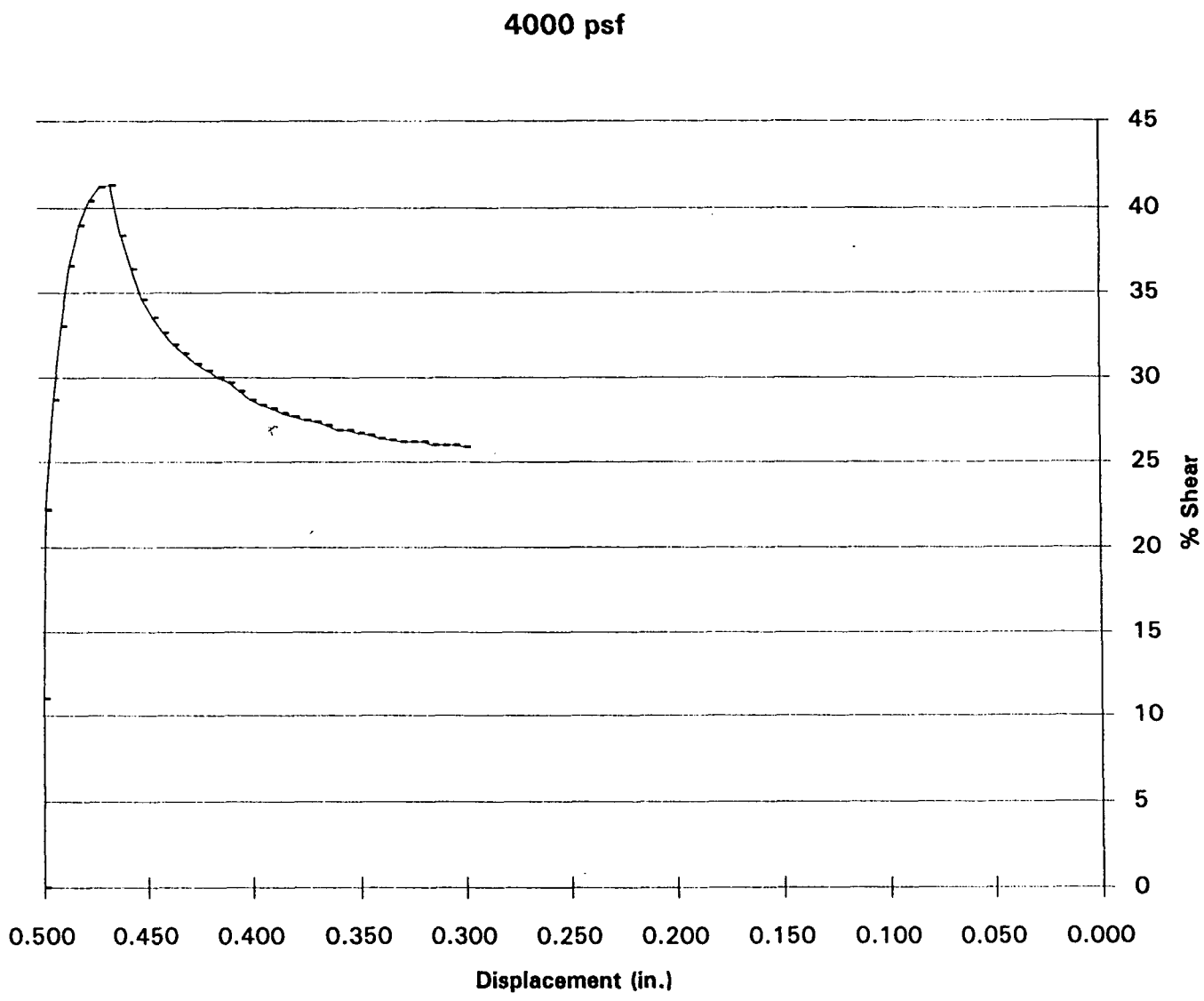
GEOTECHNICAL CONSULTANTS

Plate

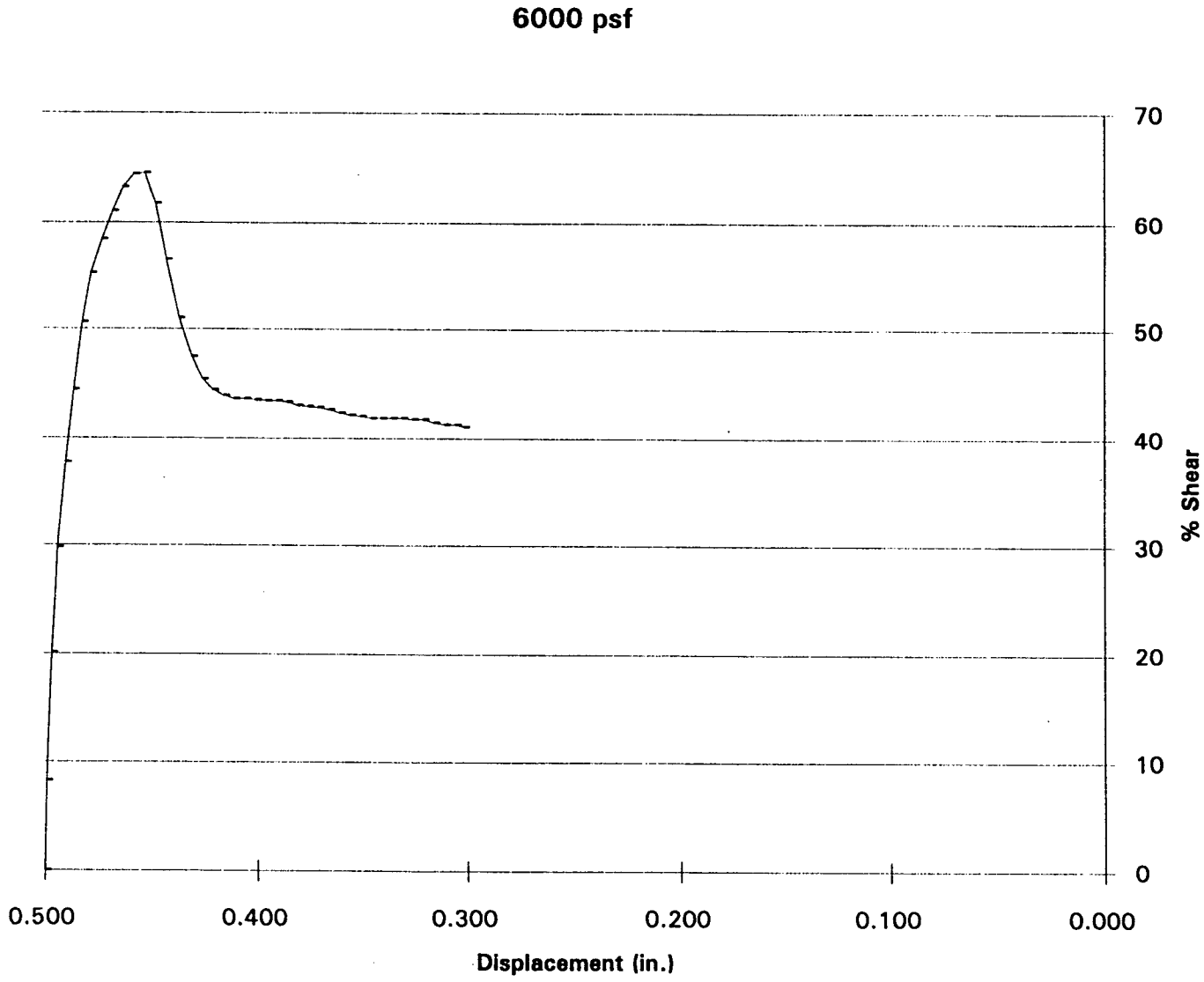
DIRECT SHEAR TEST PLOT OF B1A @ 20 FT.



DIRECT SHEAR TEST PLOT OF B1A @ 20 FT.

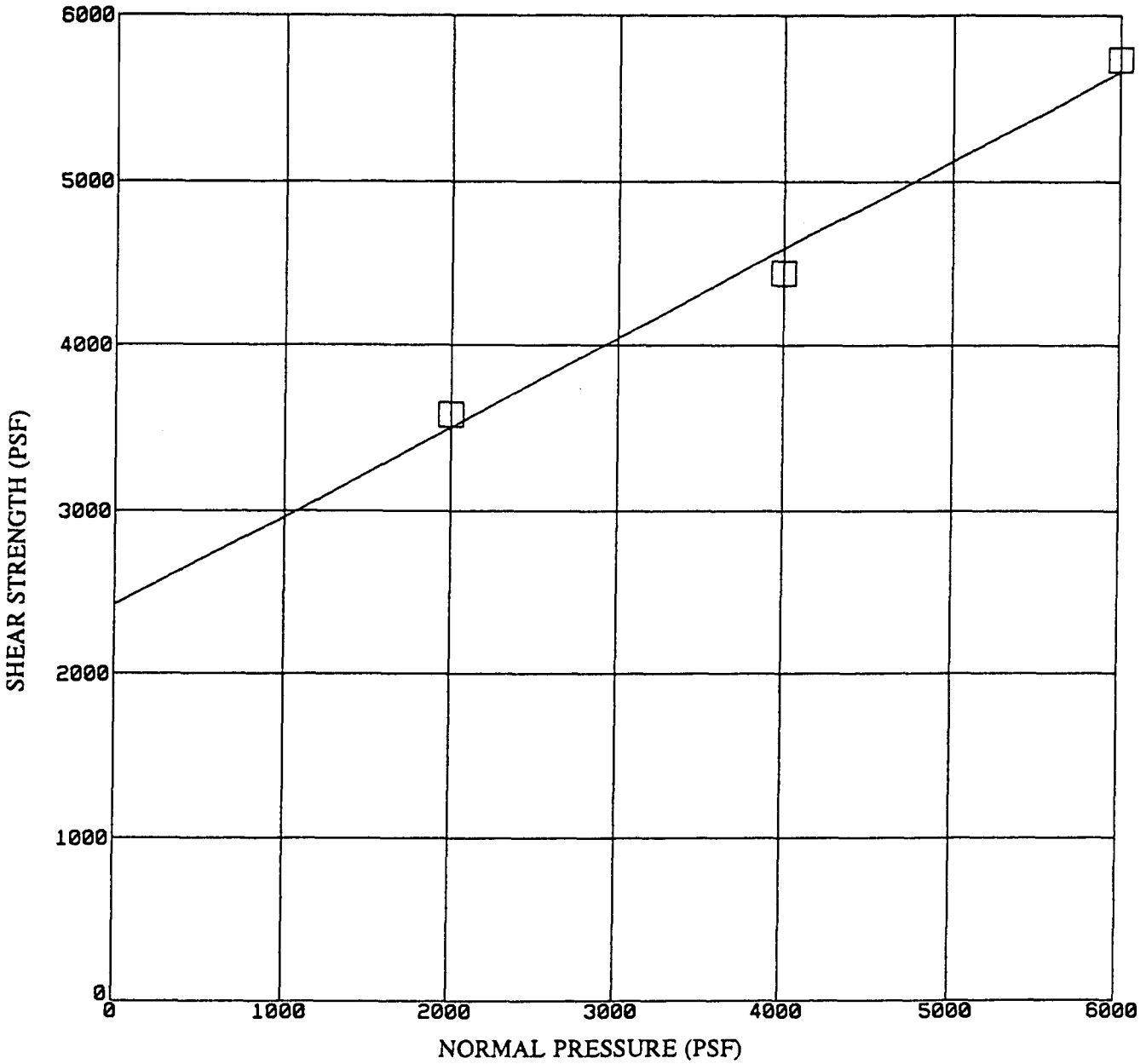


DIRECT SHEAR TEST PLOT OF B1A @ 20 FT.



SHEAR TEST DIAGRAM

Client: **Rancho Las Flores Limited Partnership**	W.O. **215-1**
Project: **Highway 138**	Location: **San Bernardino County**



Initial Sample

Dry Density = 124.5 pcf
Moist. Cont. = 4.7%

Sample: **B-1A @ 40.0 ft.**

Phi Angle: **28 deg.**

Cohesion: **2420 psf**

TEST CONDITION: Undisturbed, sheared at natural moisture

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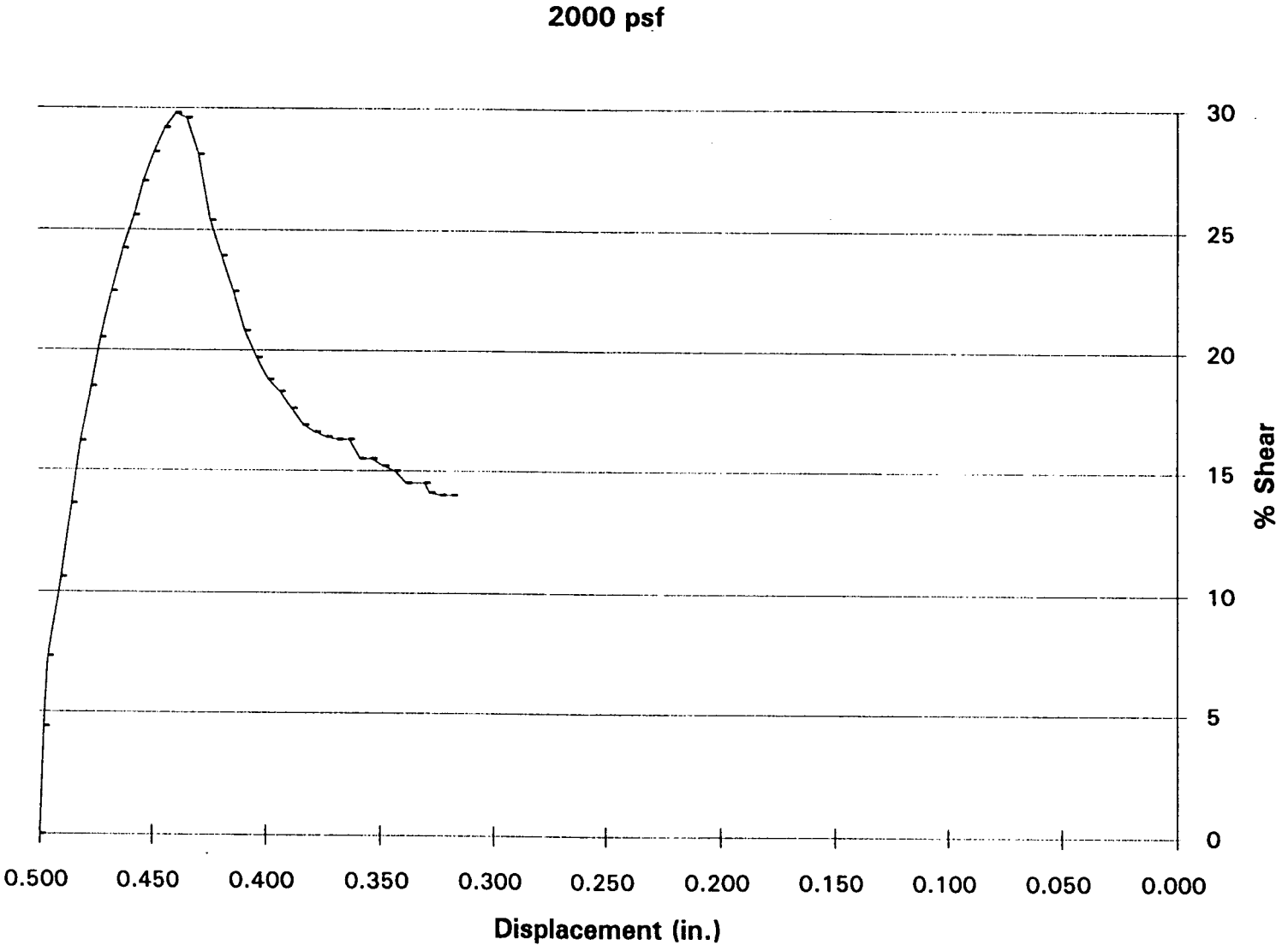
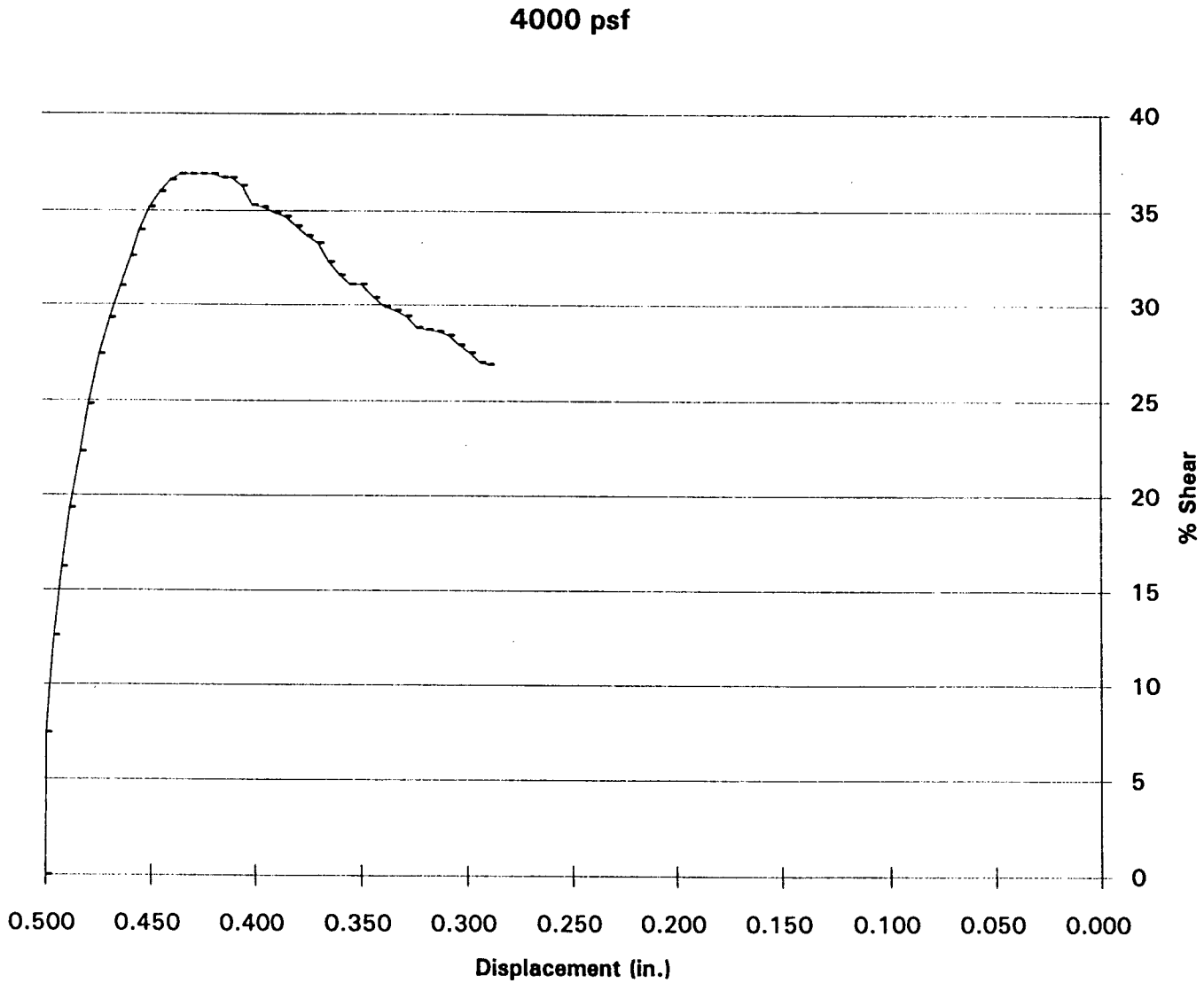
Plate

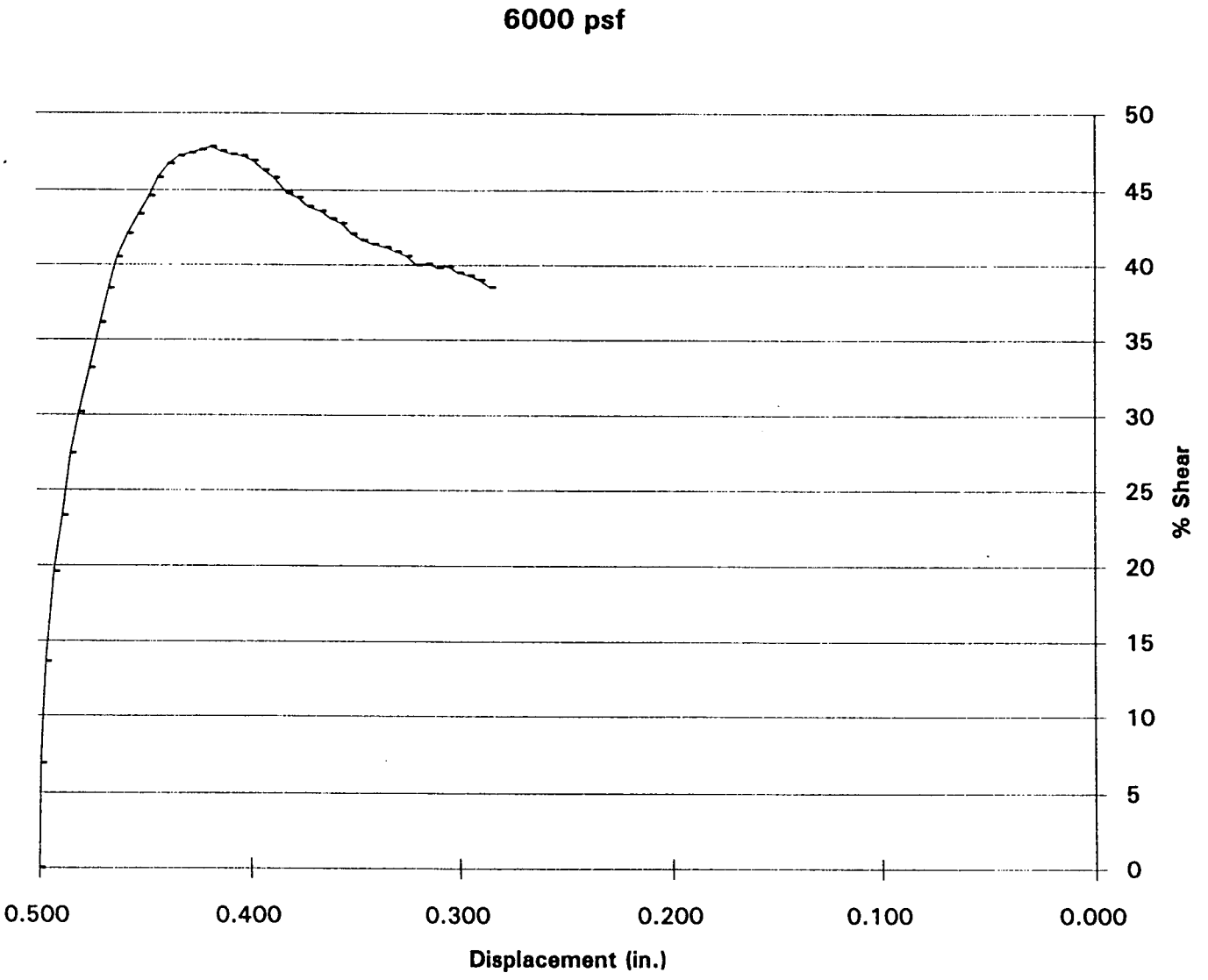
DIRECT SHEAR TEST PLOT OF B1A @ 40 FT.



DIRECT SHEAR TEST PLOT OF B1A @ 40 FT.

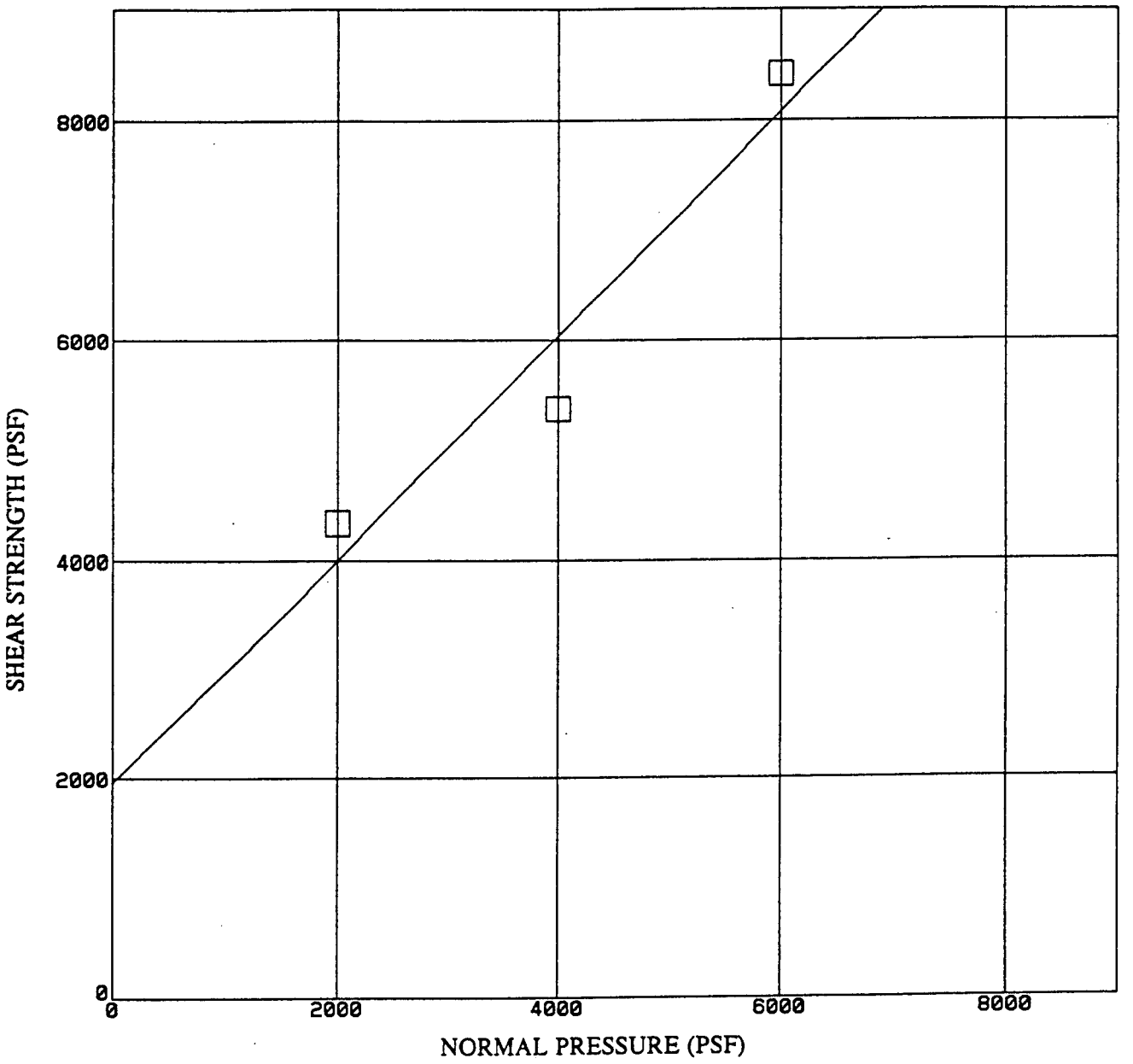


DIRECT SHEAR TEST PLOT OF B1A @ 40 FT.



SHEAR TEST DIAGRAM

Client: Rancho Las Flores Limited Partnership	W.O. 215-1
Project: Highway 138	Location: San Bernardino County



Initial Sample

Dry Density = 127.5 pcf
Moist. Cont. = 12.0%
Clayey Sand
L.L. = 31.0
P.I. = 8.9

Sample: B-2A @ 10.0 ft.

Phi Angle: 46 deg.
Cohesion: 1956 psf

TEST CONDITION: Undisturbed, sheared at natural moisture

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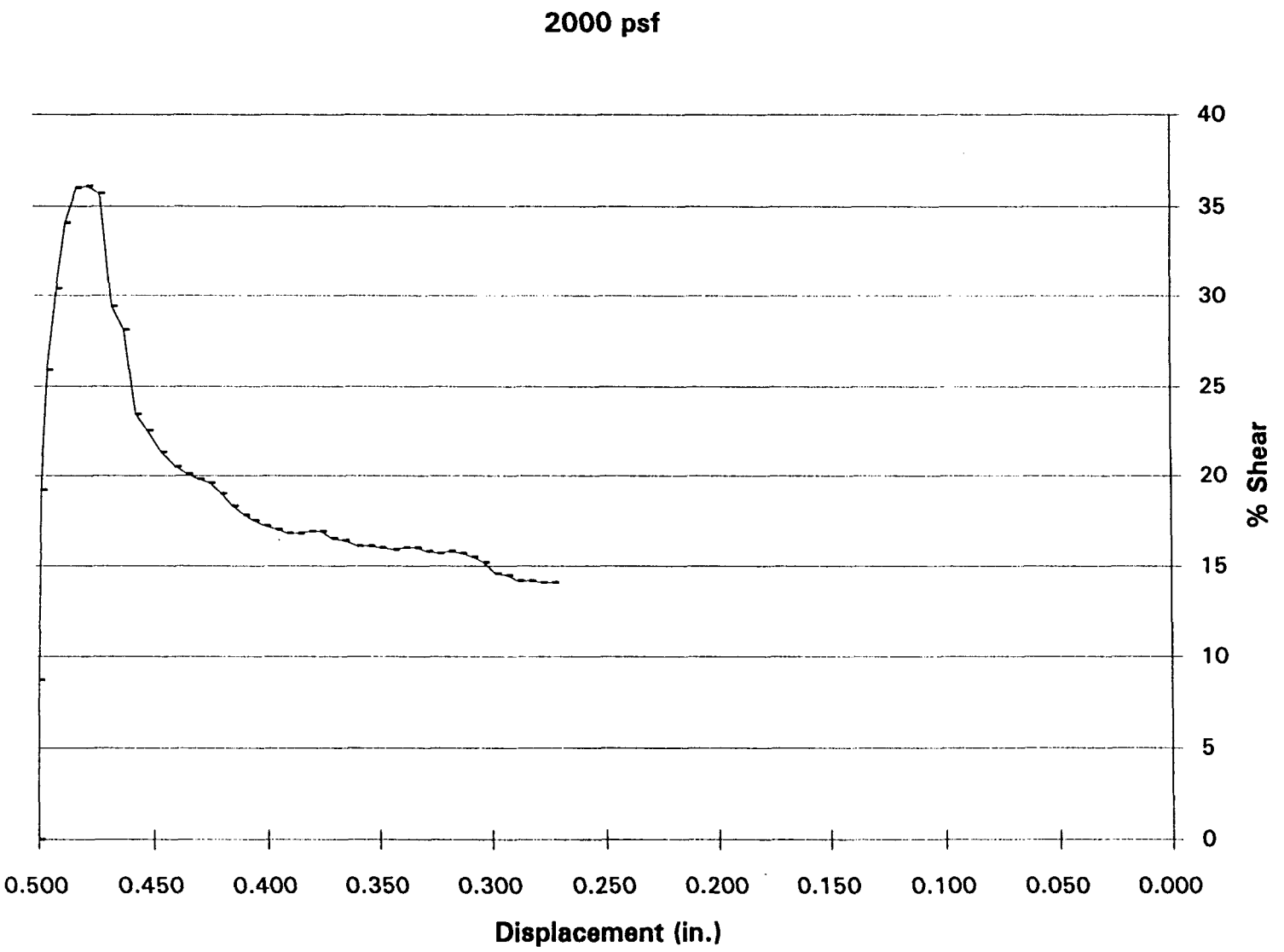
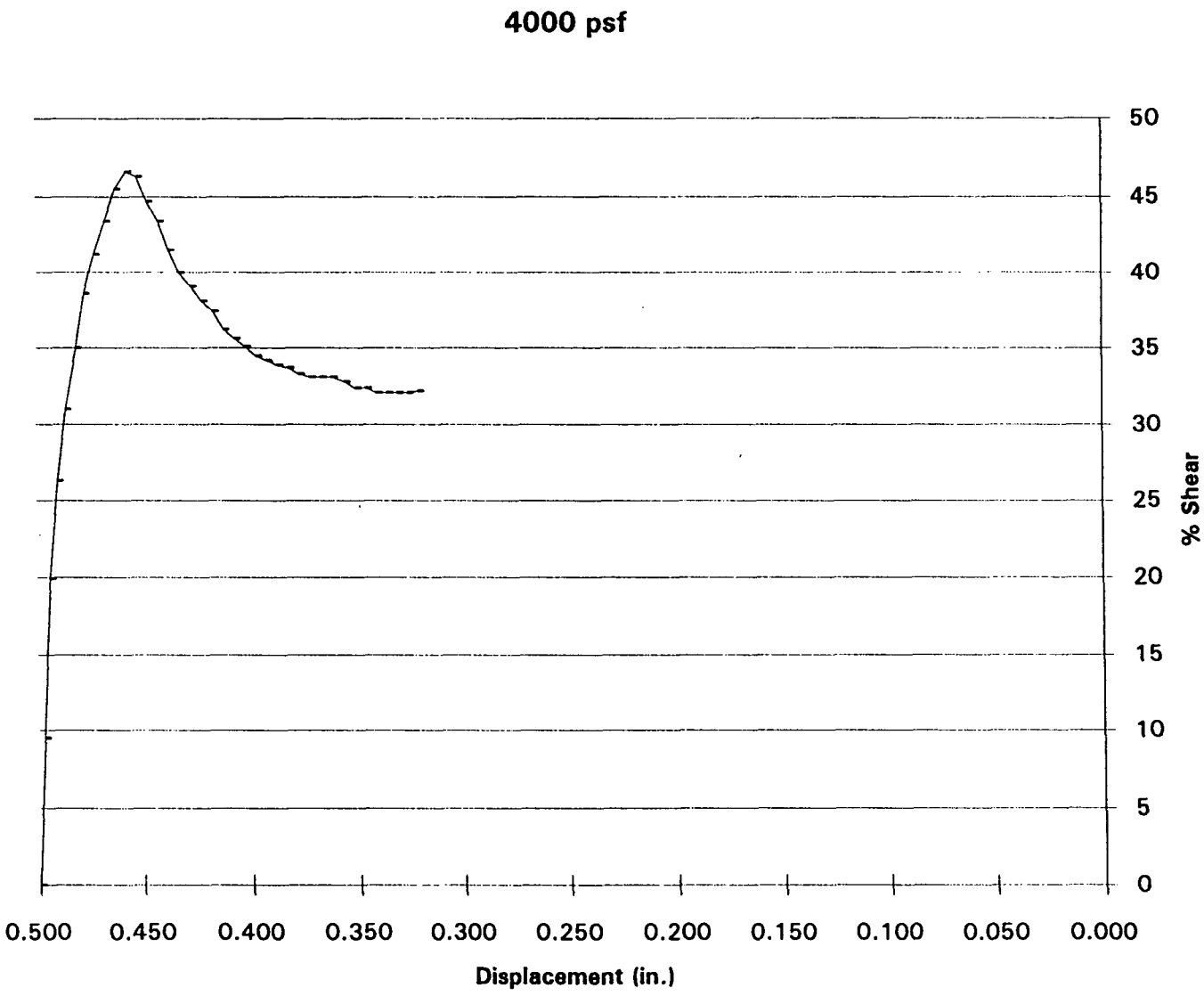
Plate

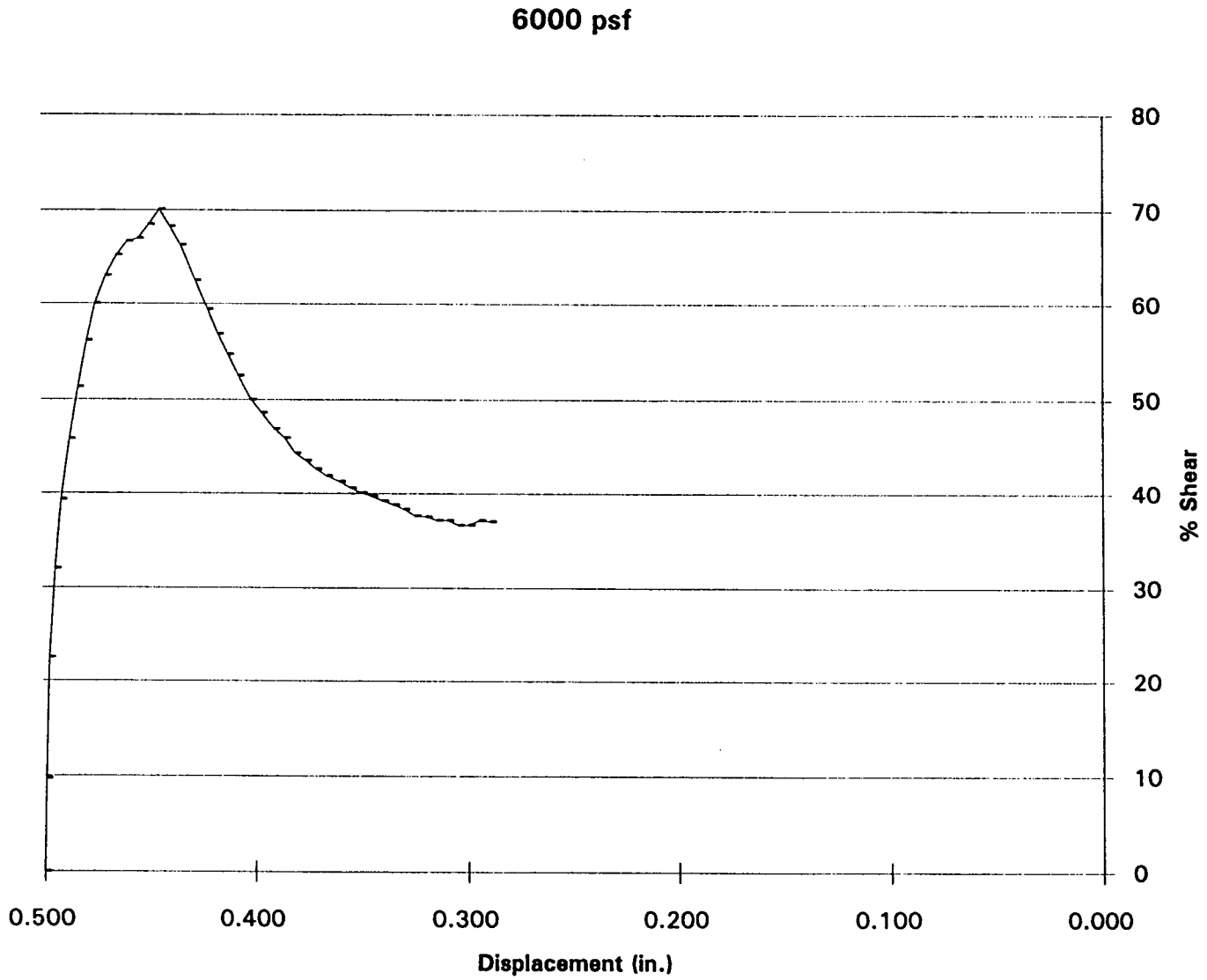
DIRECT SHEAR TEST PLOT OF B2A @ 10 FT.



DIRECT SHEAR TEST PLOT OF B2A @ 10 FT.

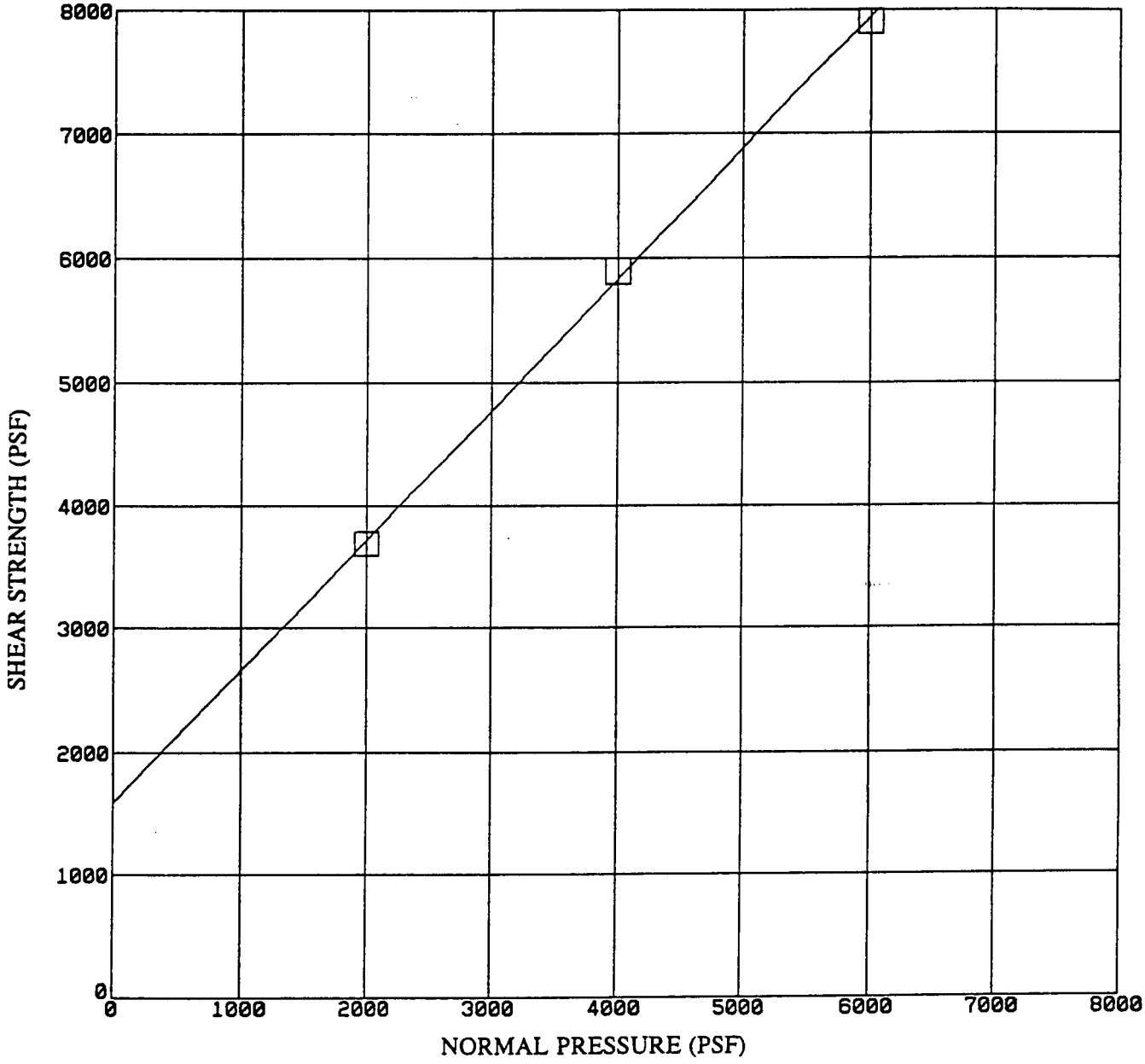


DIRECT SHEAR TEST PLOT OF B2A @ 10 FT.



SHEAR TEST DIAGRAM

Client: Rancho Las Flores Limited Partnership	W.O. 215-1
Project: Highway 138	Location: San Bernardino County



Initial Sample

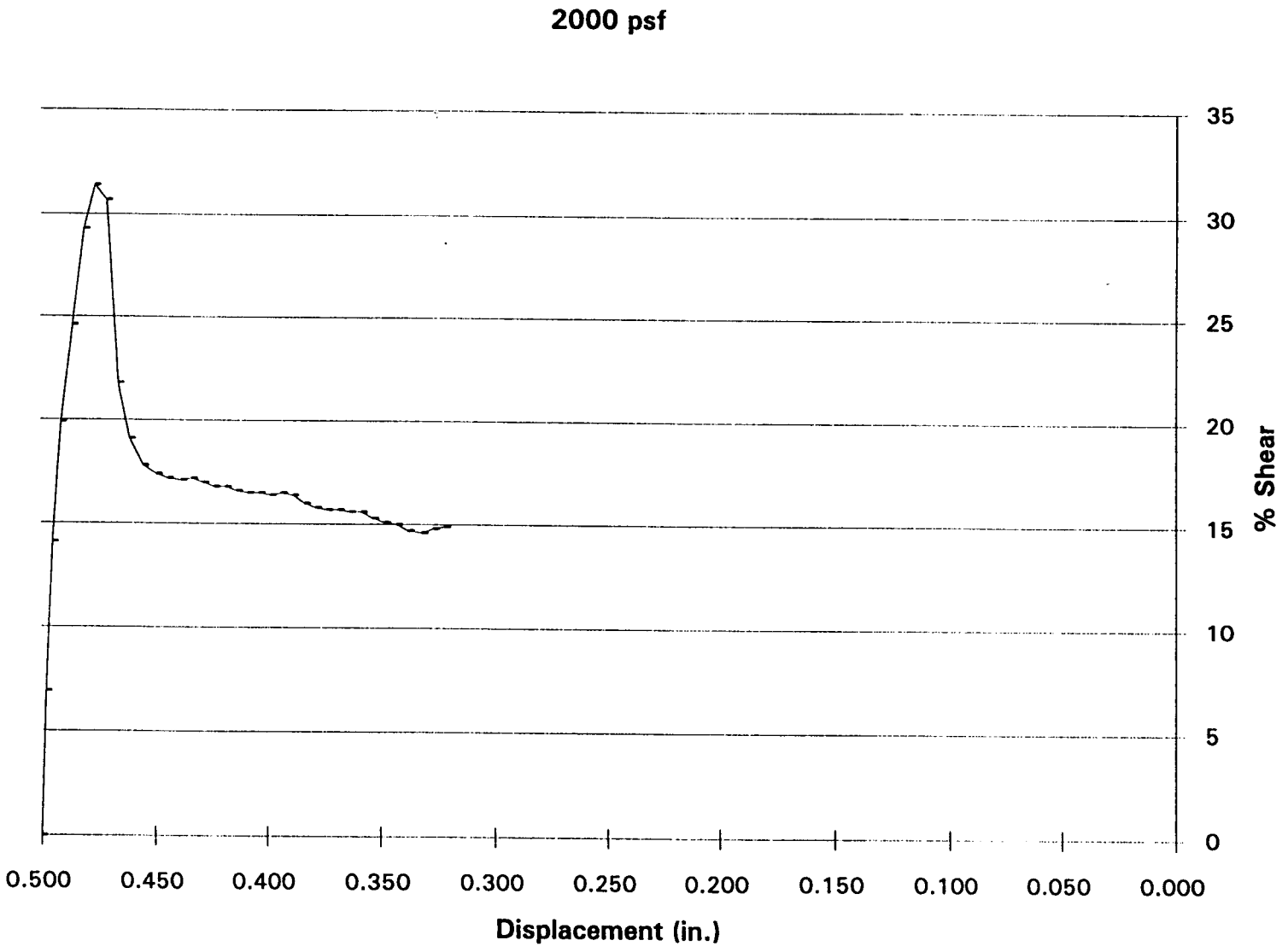
Dry Density = 128.0 pcf
Moist. Cont. = 8.5%

Sample: B-2A @ 15.0 ft.

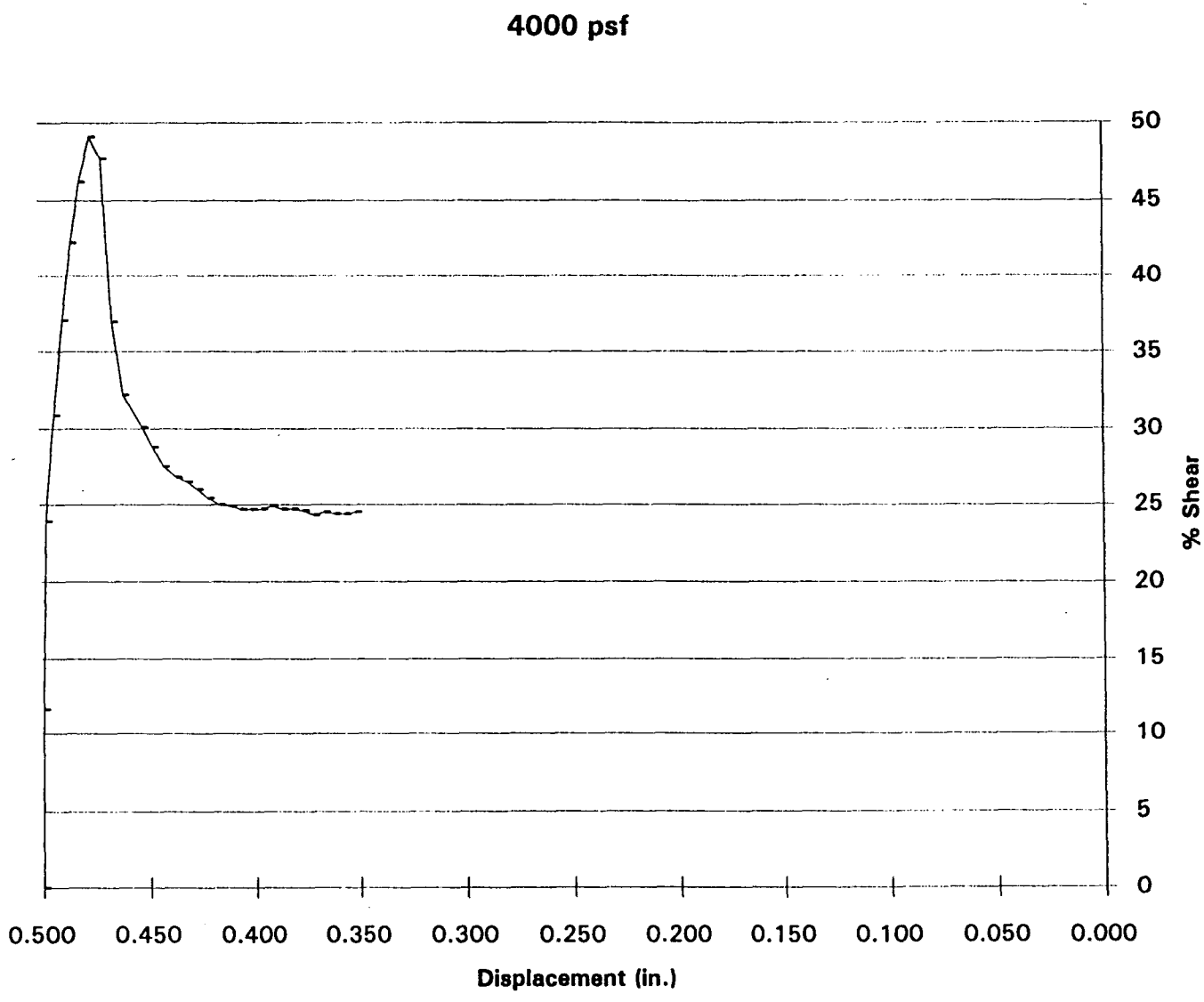
Phi Angle: 47 deg.
Cohesion: 1604 psf

TEST CONDITION: Undisturbed, sheared at natural moisture

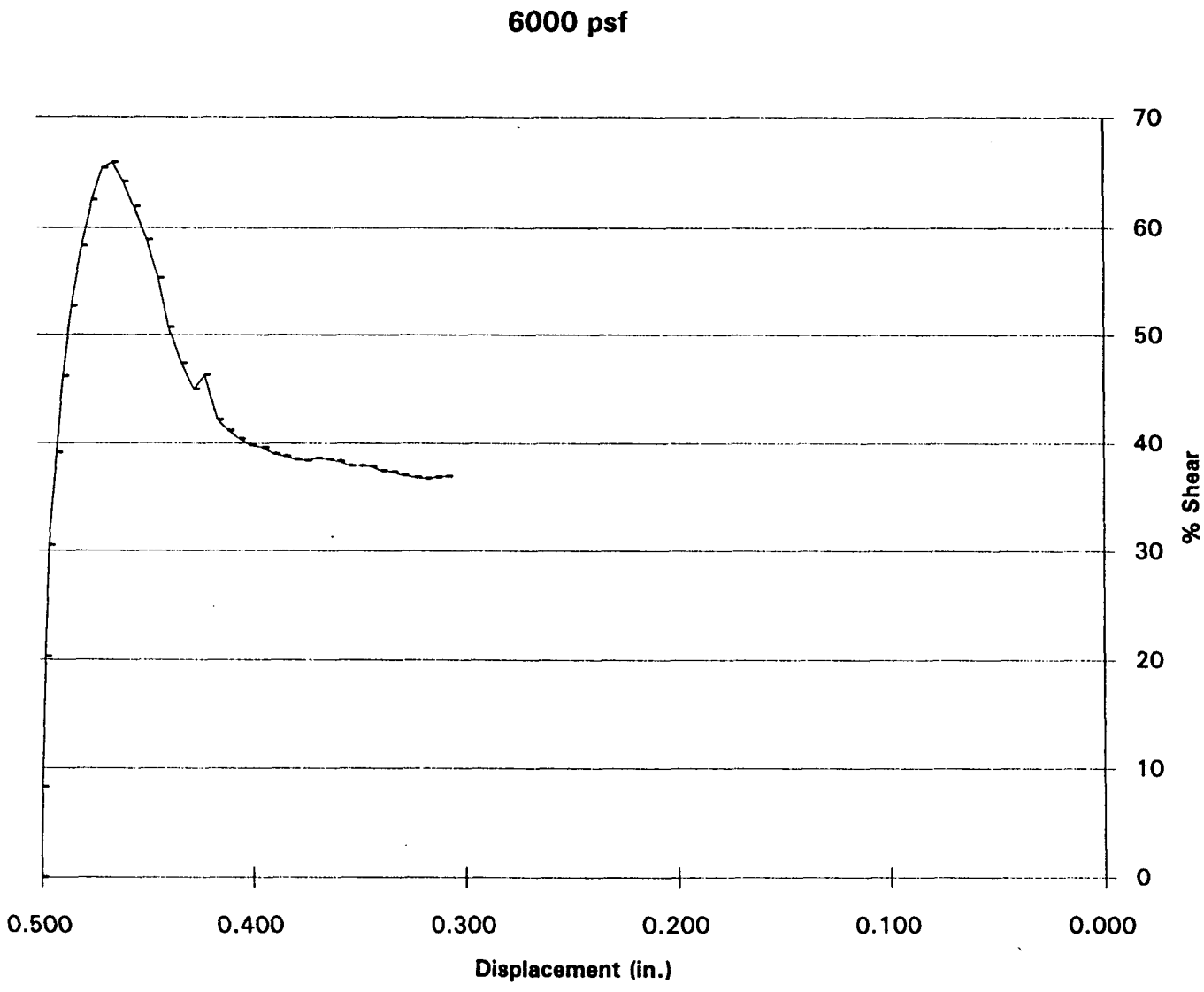
DIRECT SHEAR TEST PLOT OF B2A @ 15 FT.



DIRECT SHEAR TEST PLOT OF B2A @ 15 FT.

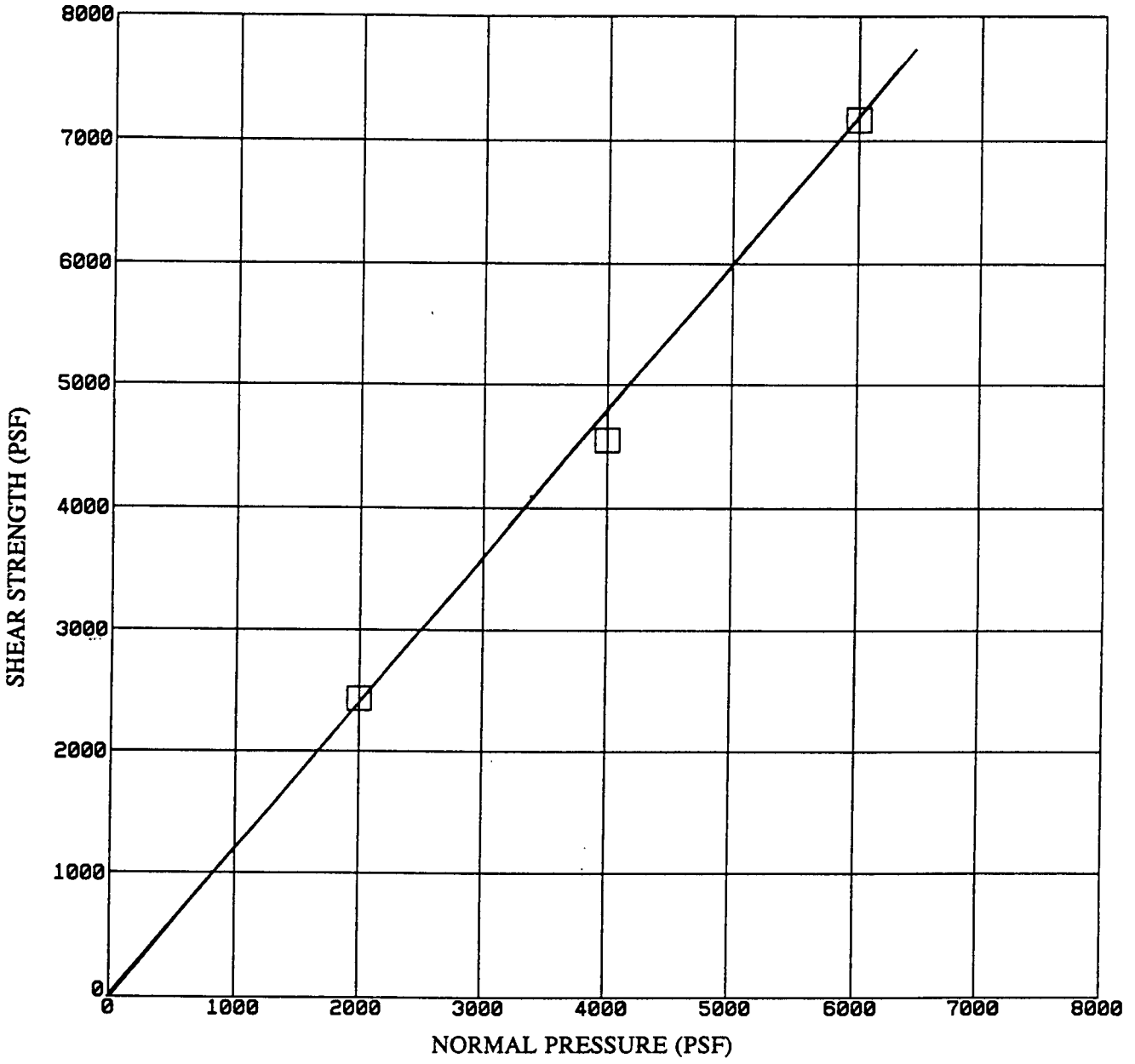


DIRECT SHEAR TEST PLOT OF B2A @ 15 FT.



SHEAR TEST DIAGRAM

Client: **Rancho Las Flores Limited Partnership**	W.O. **215-1**
Project: **Highway 138**	Location: **San Bernardino County**



Initial Sample

Dry Density = 118.5 pcf
Moist. Cont. = 5.3%

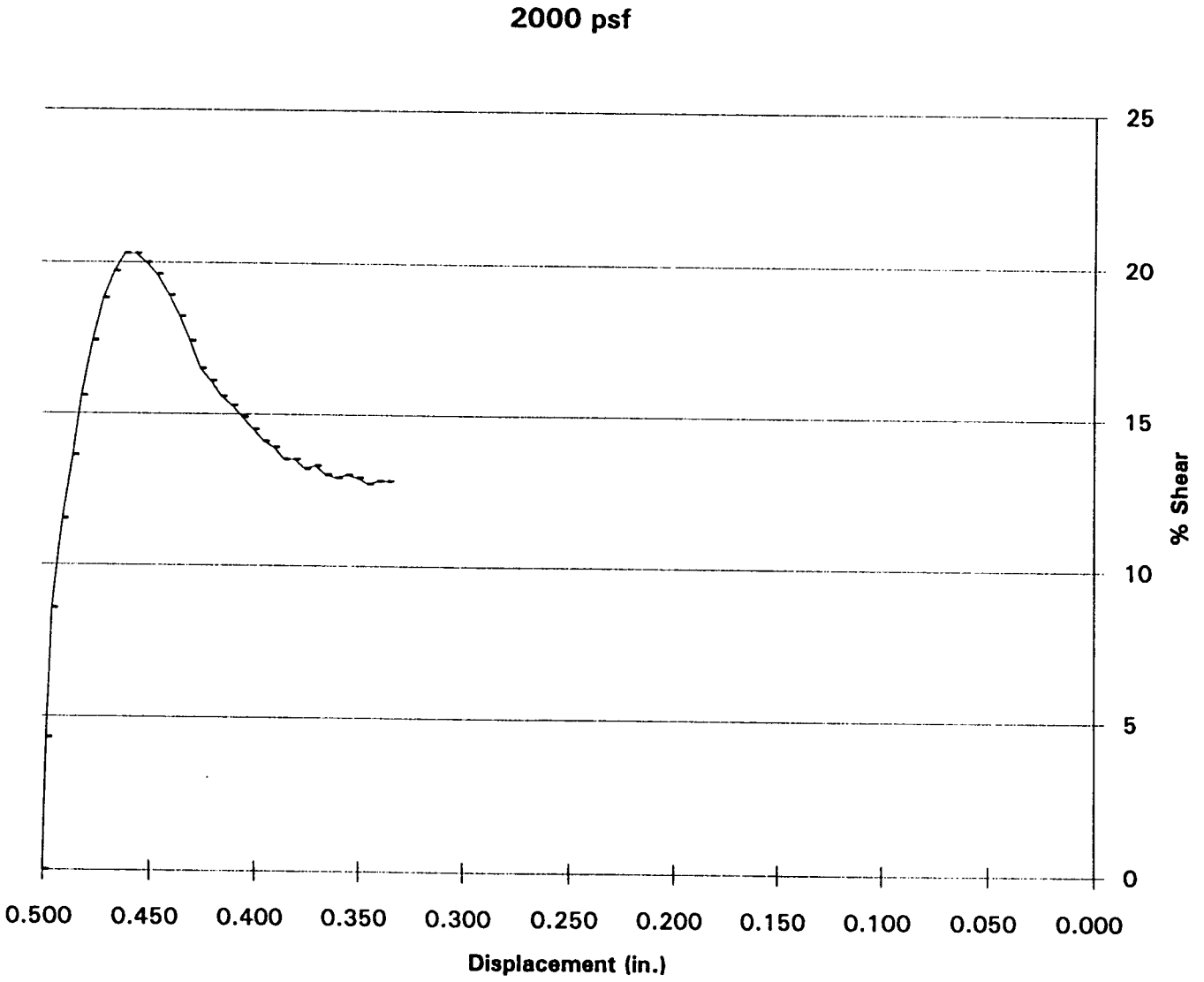
Sample: **B-2A @ 20.0 ft.**

Phi Angle: **50 deg.**

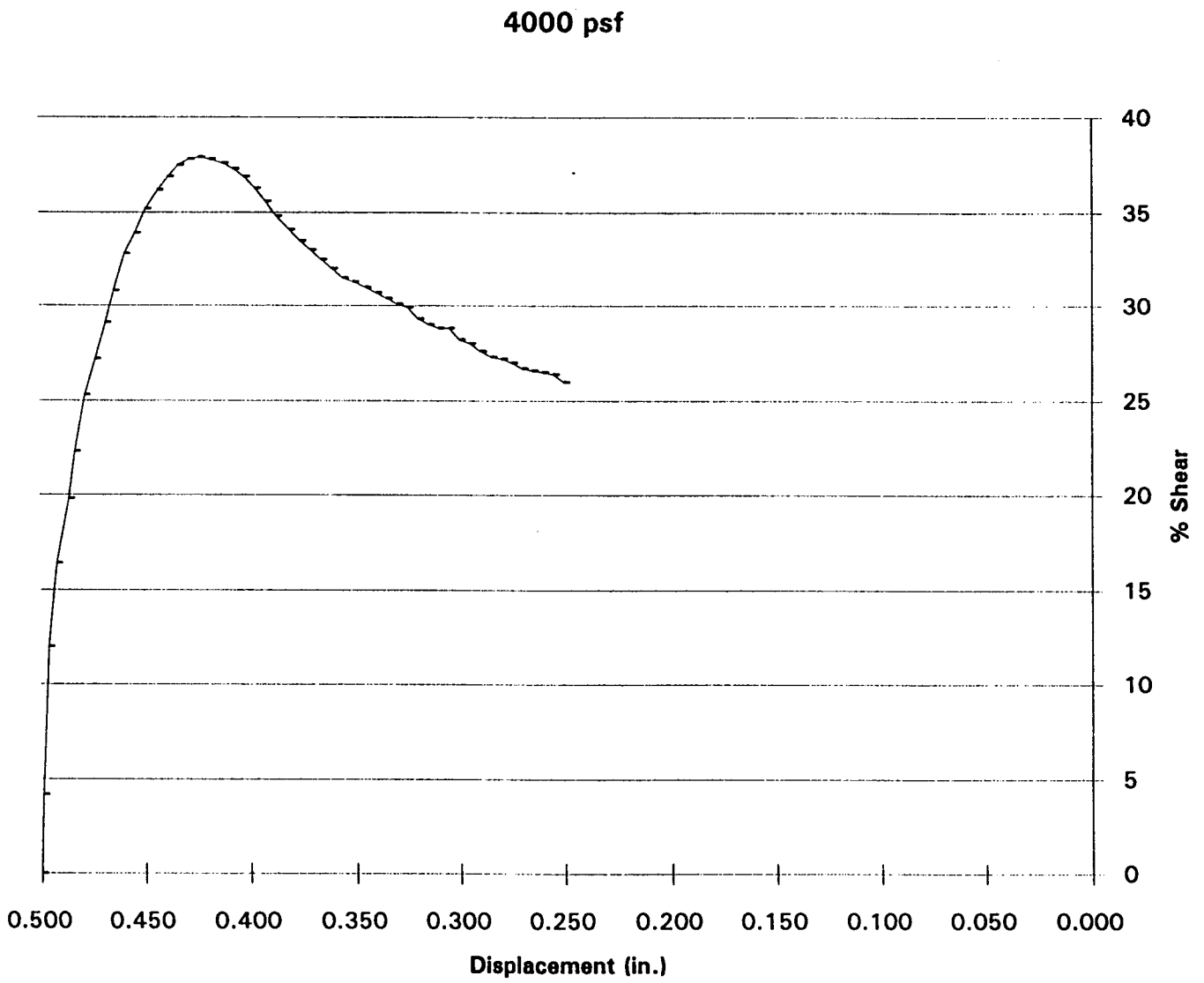
Cohesion: **0 psf**

TEST METHOD: Undisturbed, sheared at natural moisture

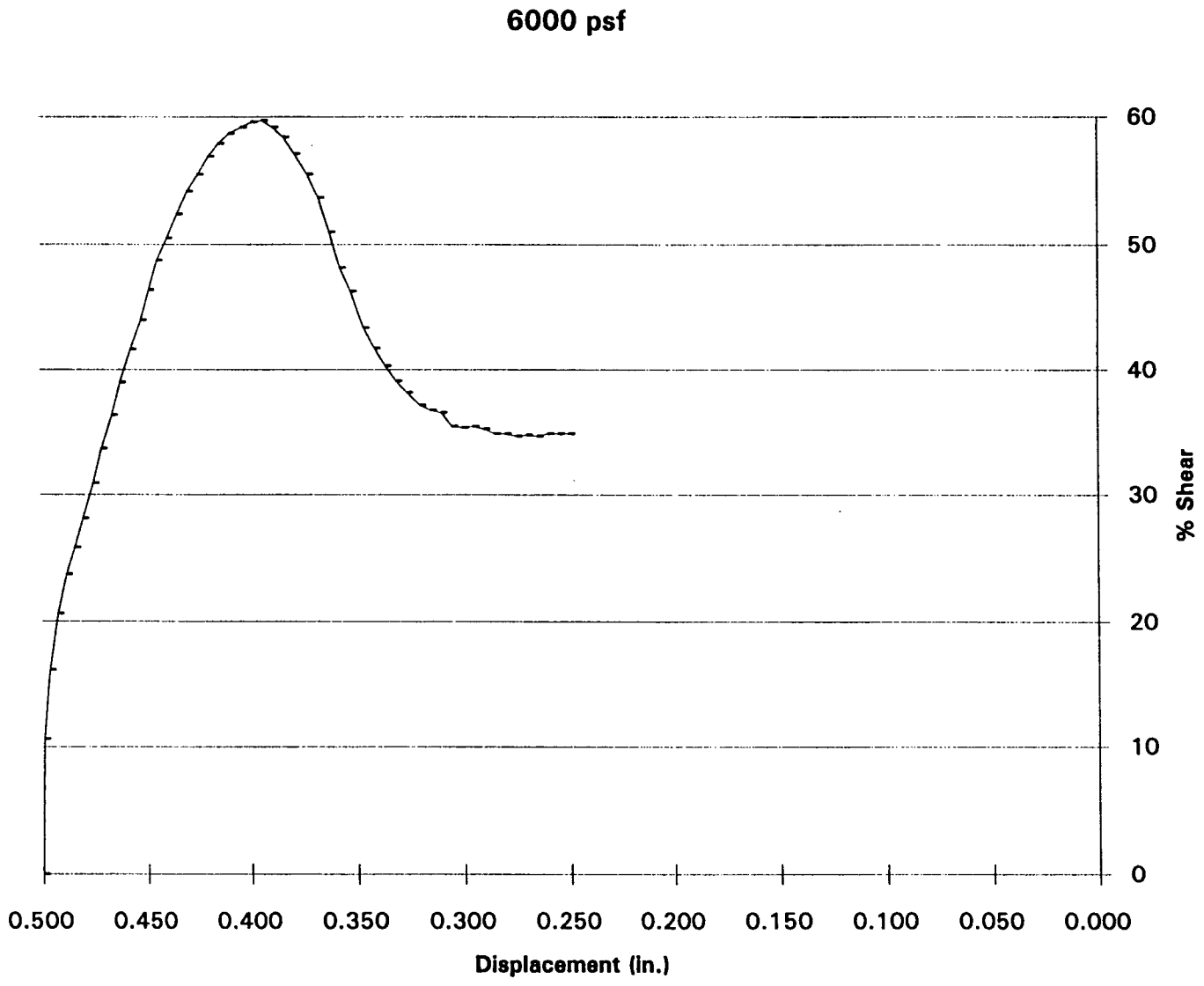
DIRECT SHEAR TEST PLOT OF B2A @ 20 FT.



DIRECT SHEAR TEST PLOT OF B2A @ 20 FT.

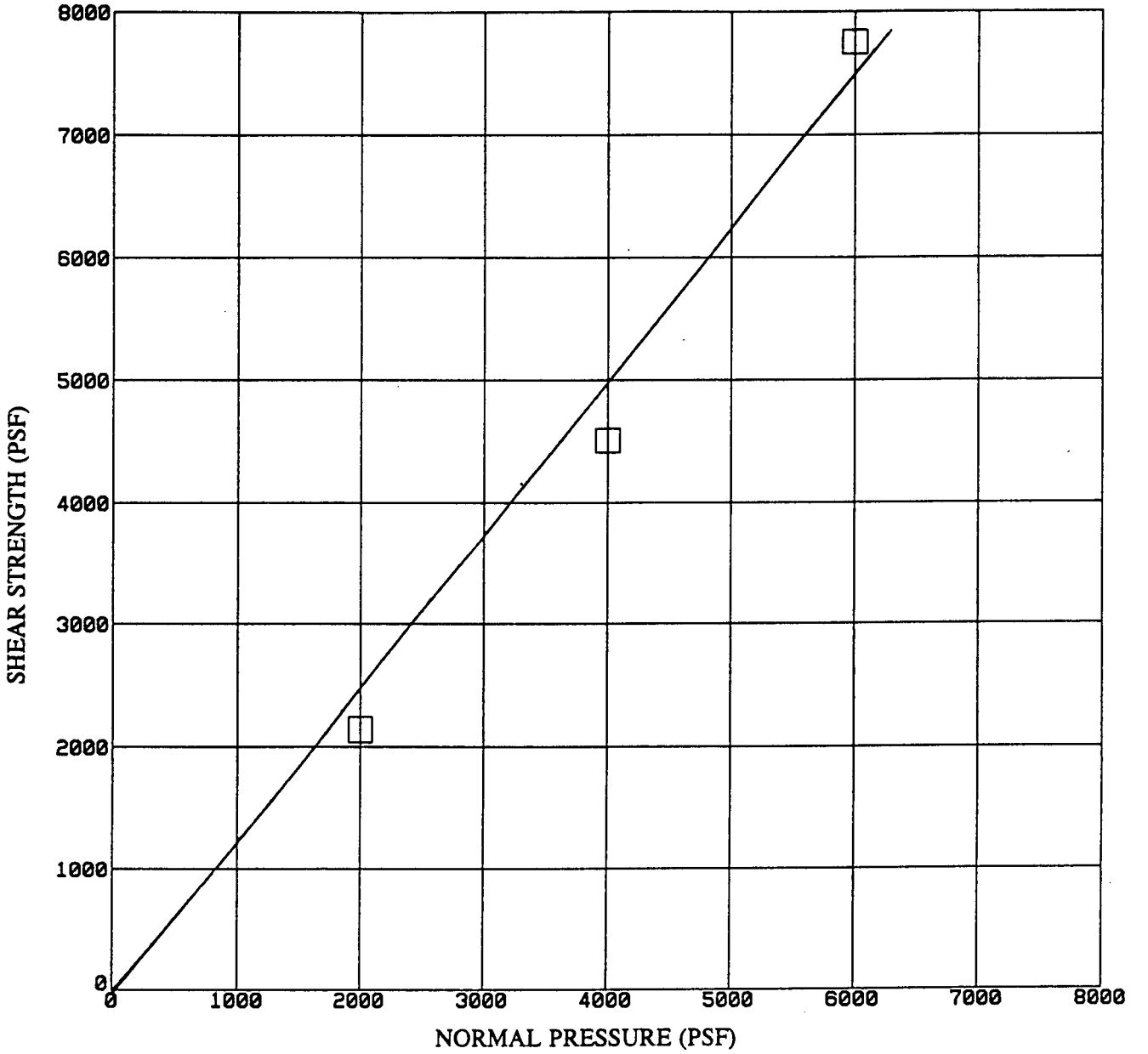


DIRECT SHEAR TEST PLOT OF B2A @ 20 FT.



SHEAR TEST DIAGRAM

Client: **Rancho Las Flores Limited Partnership**	W.O. **215-1**
Project: **Highway 138**	Location: **San Bernardino County**



Initial Sample

Dry Density = 123.3 pcf
Moist. Cont. = 6.0%
Silty to Clayey Sand
L.L. = 26.0
P.I. = 5.2

Sample: **B-2A @ 35.0 ft.**

Phi Angle: **51 deg.**

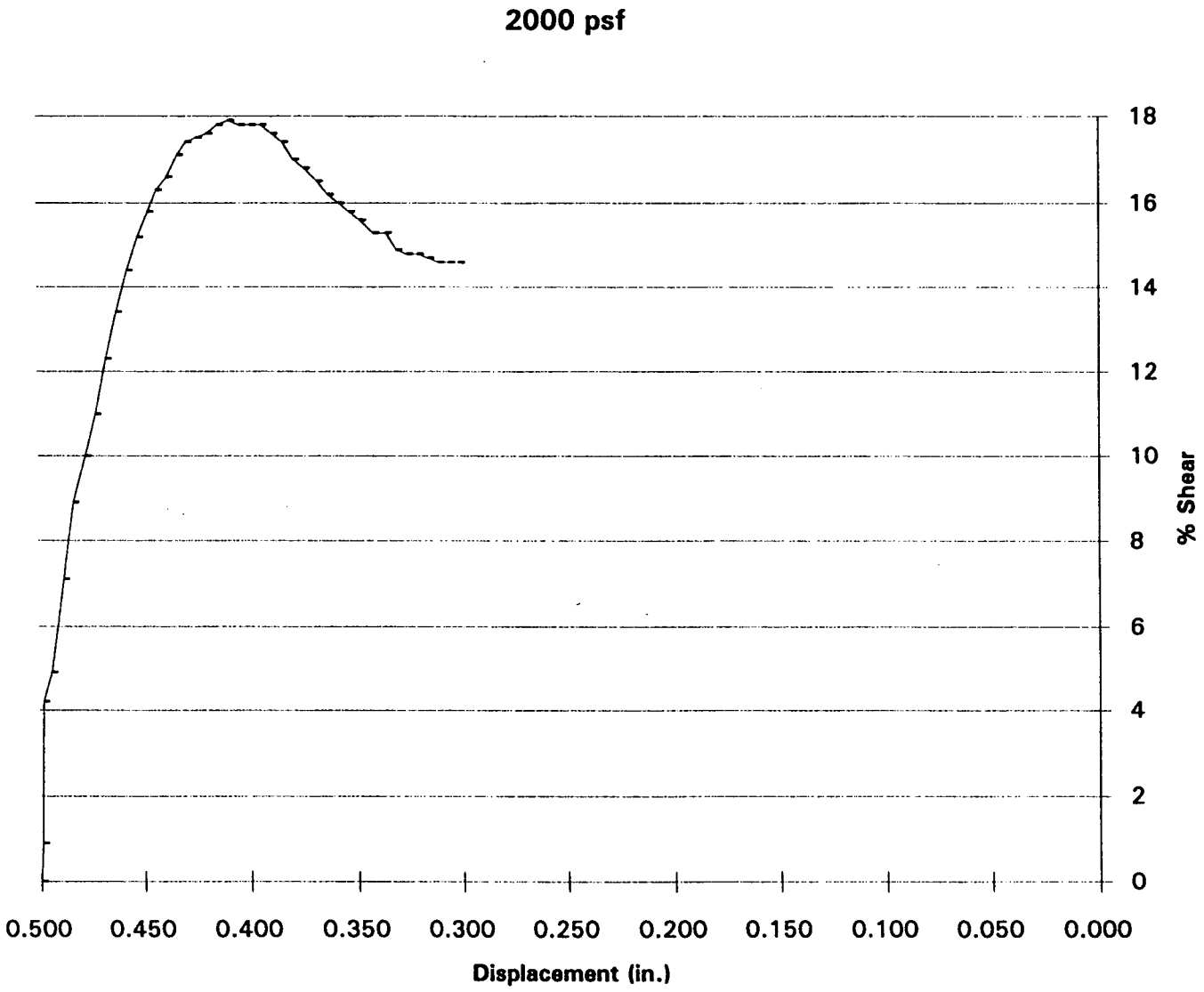
Cohesion: **0 psf**

TEST METHOD: Undisturbed, sheared at natural moisture

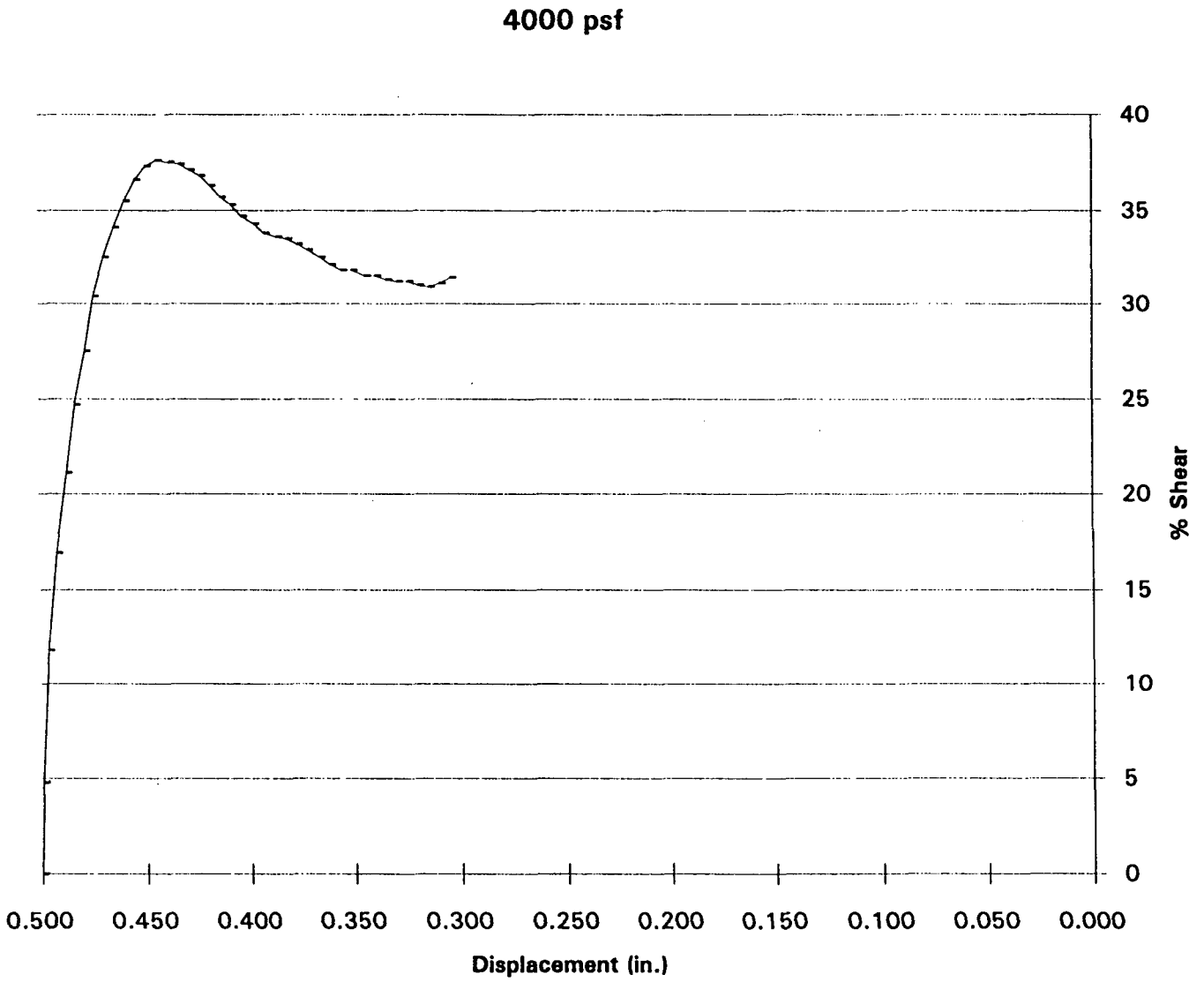
John A. Sayers & Associates
GEOTECHNICAL CONSULTANTS

Plate

DIRECT SHEAR TEST PLOT OF B2A @ 35 FT.

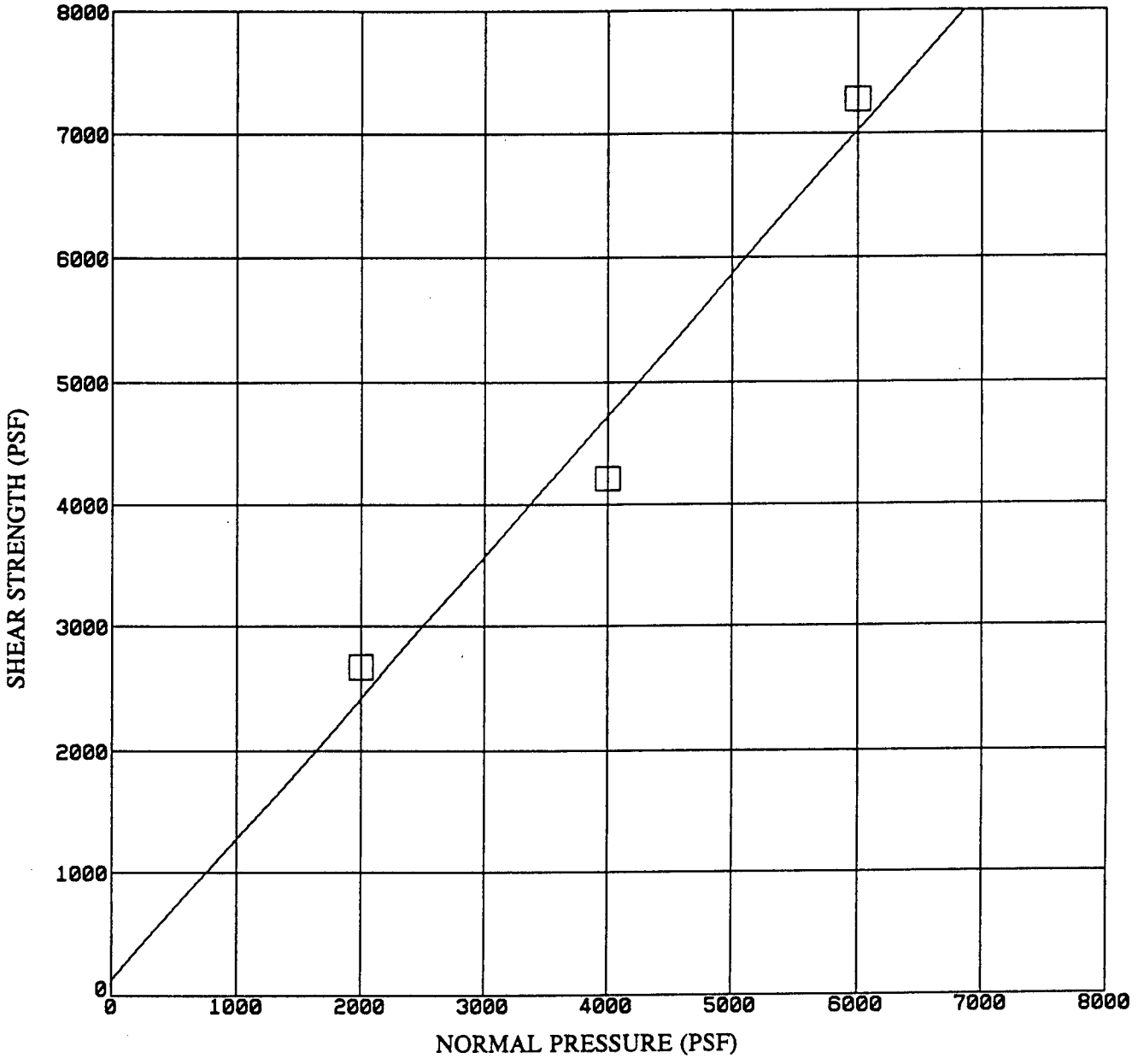


DIRECT SHEAR TEST PLOT OF B2A @ 35 FT.



SHEAR TEST DIAGRAM

Client: Rancho Las Flores Limited Partnership	W.O. 215-1
Project: Highway 138	Location: San Bernardino County



Initial Sample

Dry Density - 120.1 pcf
Moist. Cont. = 6.0%

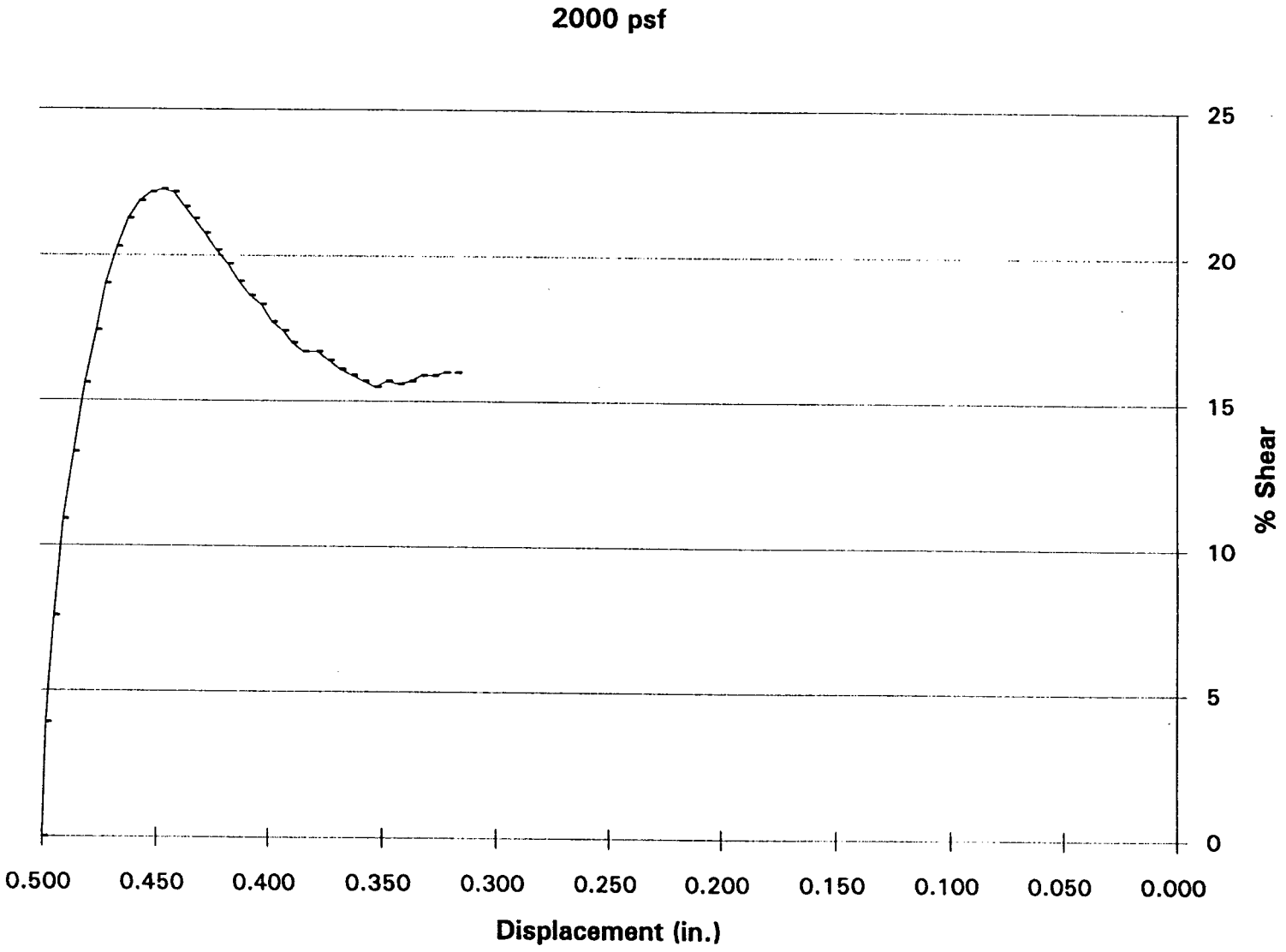
Sample: B-2A @ 45.0 ft.

Phi Angle: 49 deg.
Cohesion: 124 psf

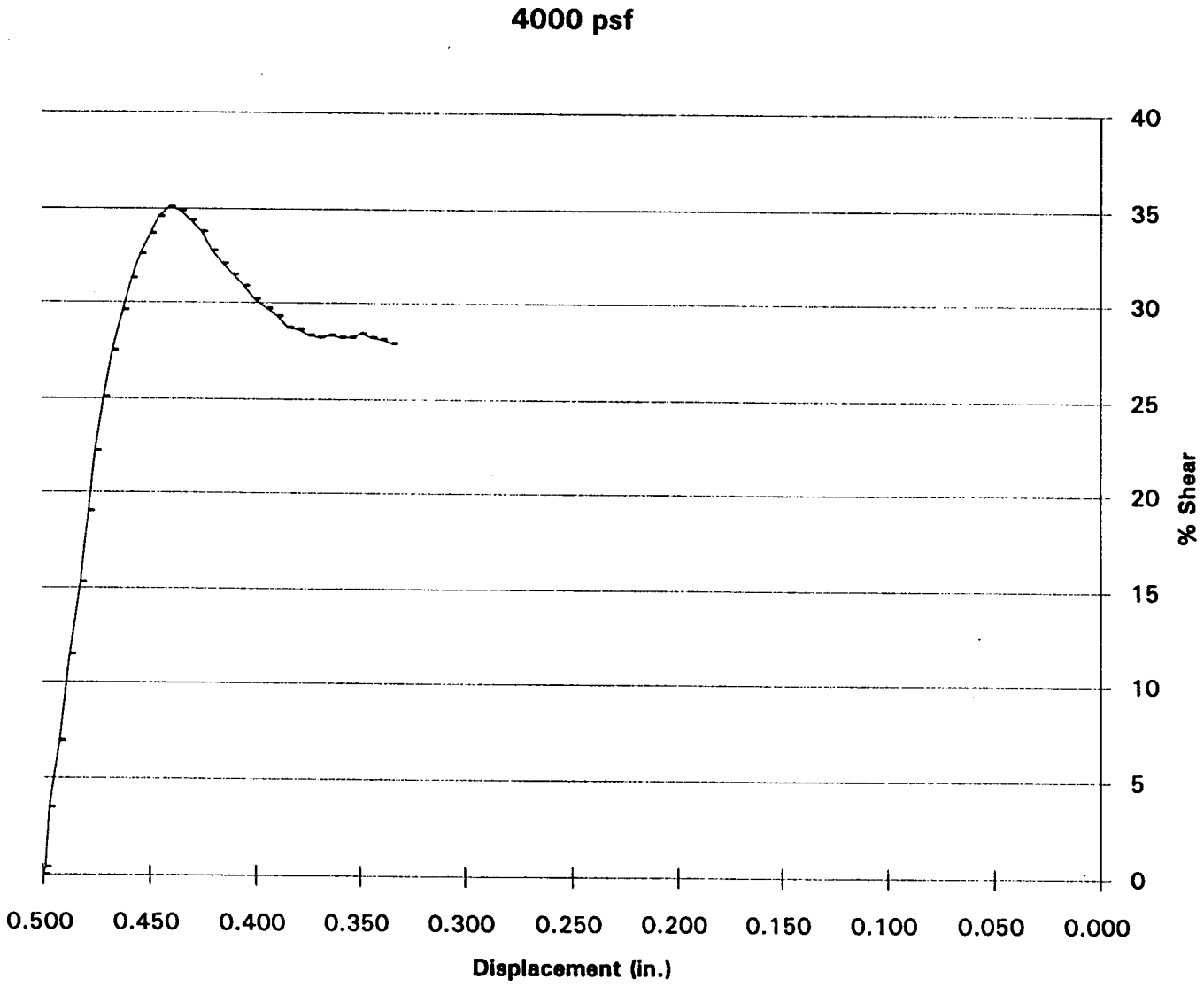
TEST CONDITION: Undisturbed, sheared at natural moisture

John A. Sayers & Associates GEOTECHNICAL CONSULTANTS	Plate

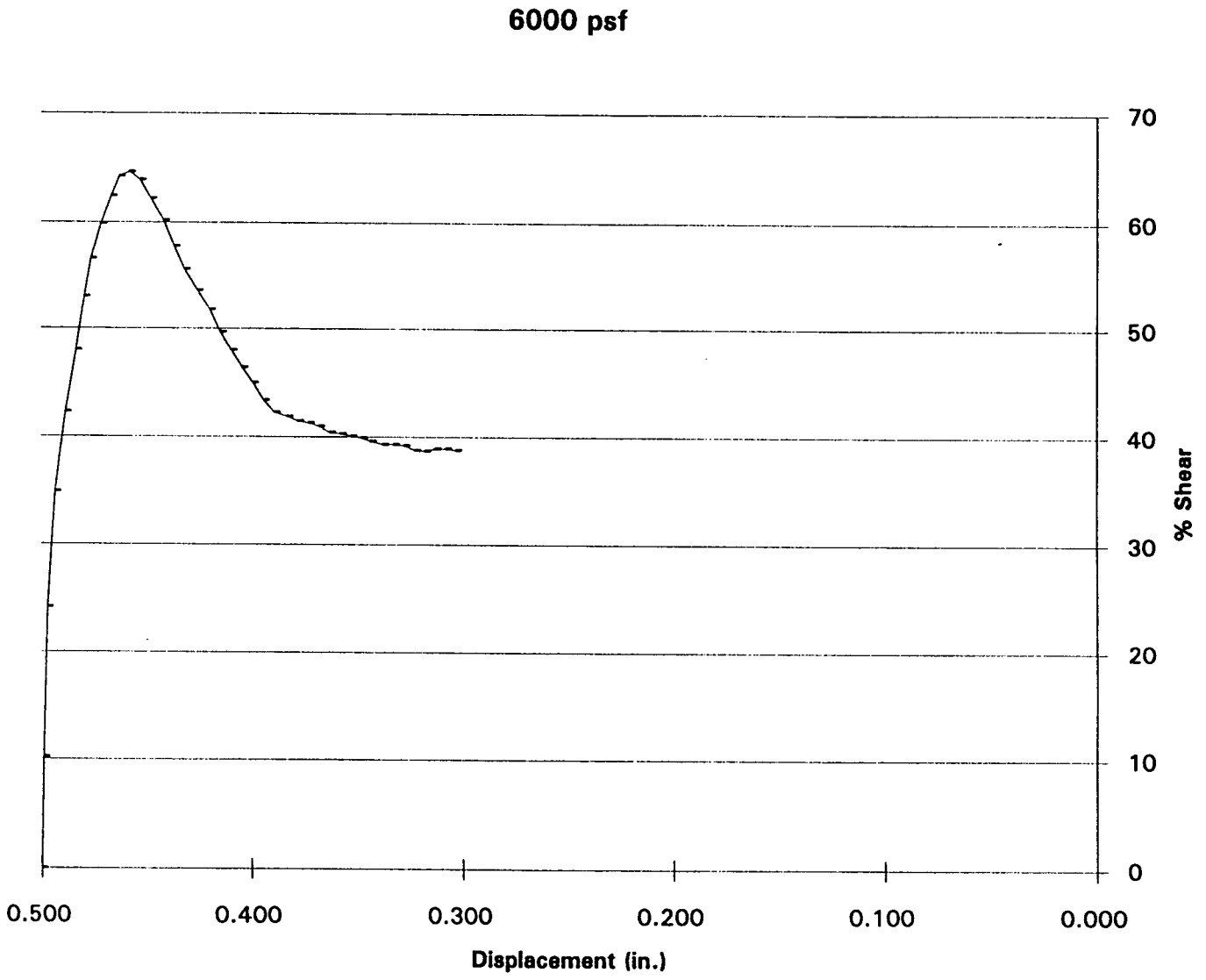
DIRECT SHEAR TEST PLOT OF B2A @ 45 FT.



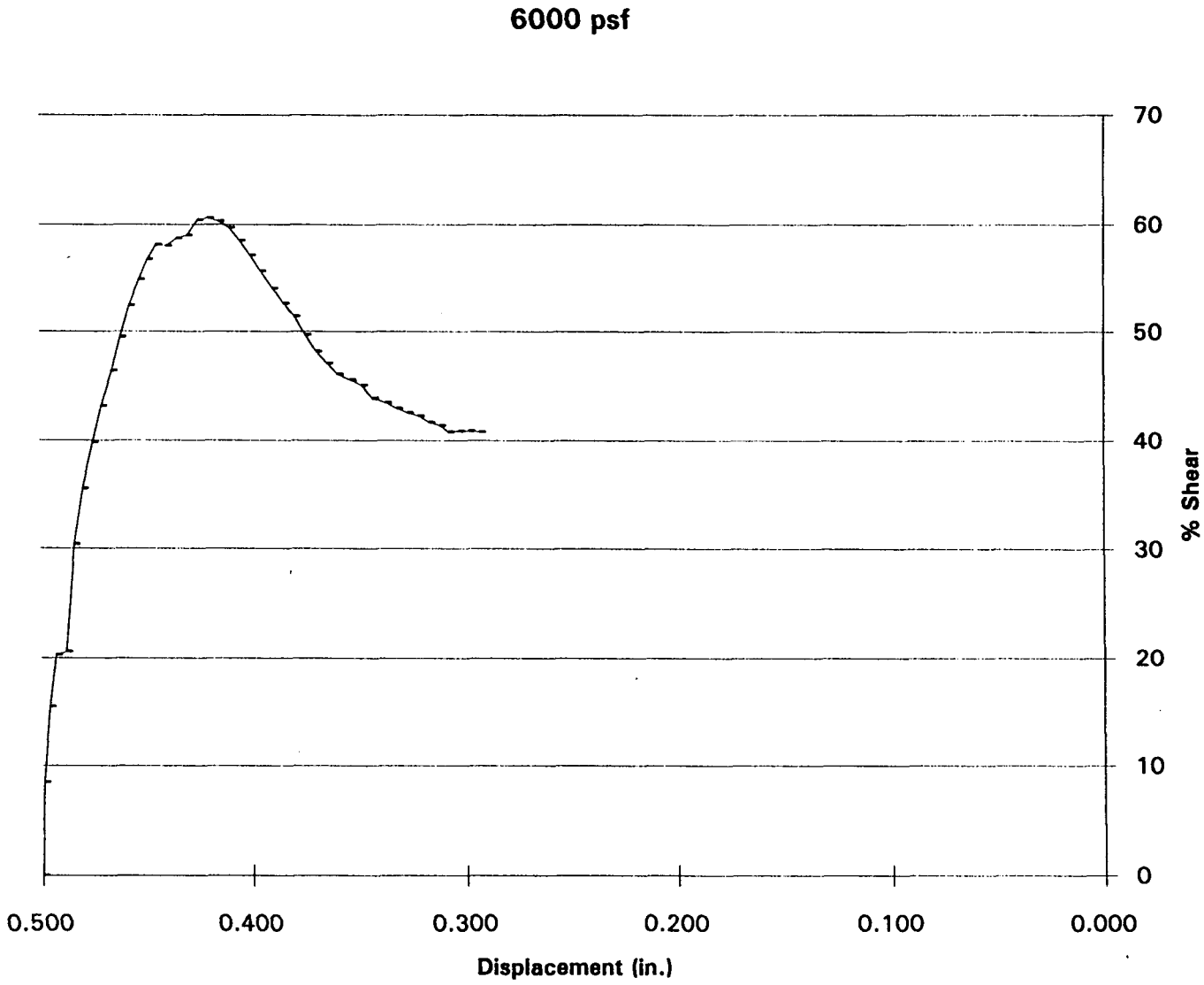
DIRECT SHEAR TEST PLOT OF B2A @ 45 FT.



DIRECT SHEAR TEST PLOT OF B2A @ 35 FT.



DIRECT SHEAR TEST PLOT OF B2A @ 45 FT.



SHEAR TEST SUMMARY

Undisturbed, Tested after Saturation

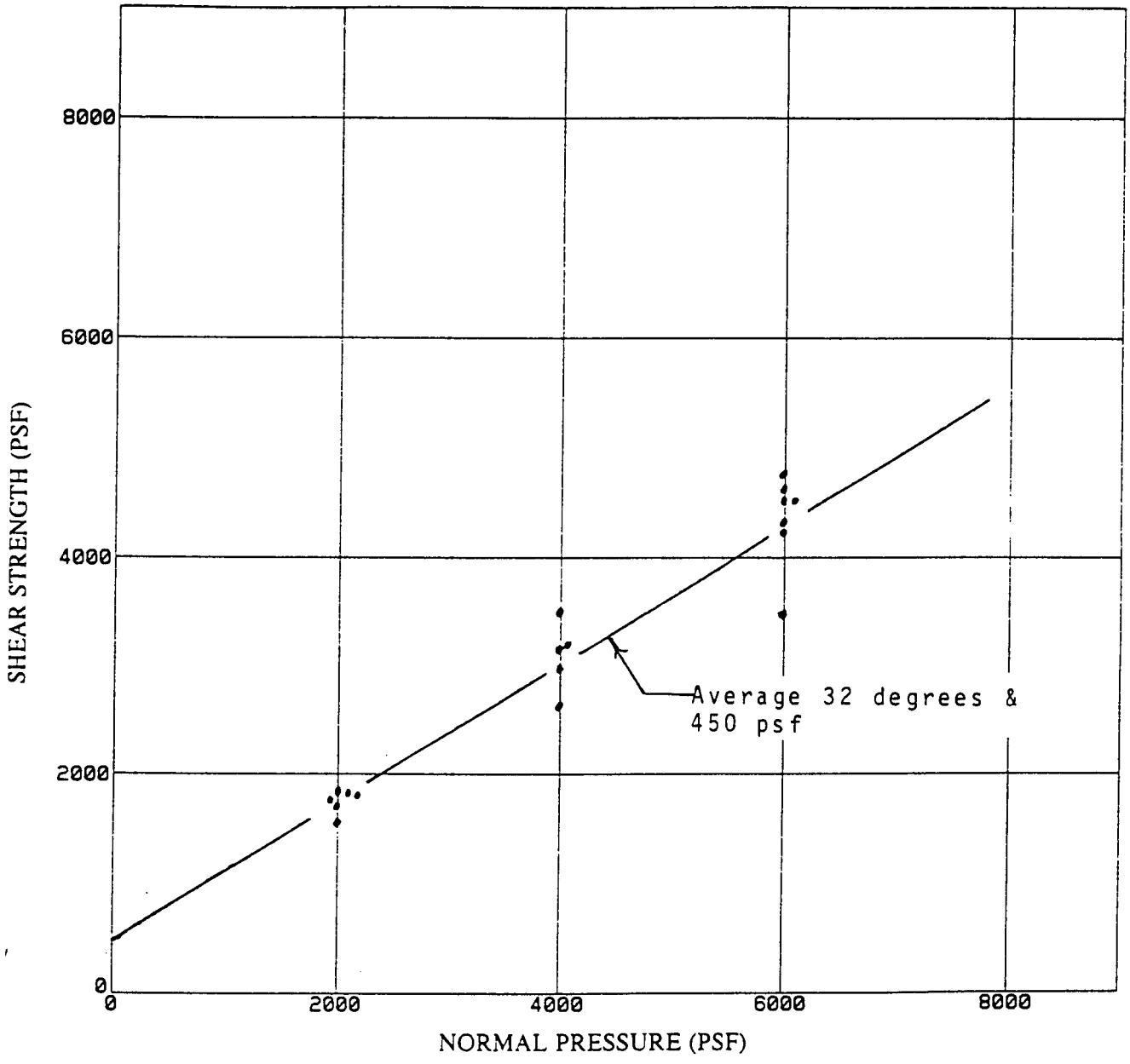
Sample Location	Shear Strength (psf)	Normal Pressure (psf) 2000	4000	6000
B-5 @ 10'		1704	2640	4500
B-10 @ 10'		1848	(1)	4776
B-11 @ 5'		1860	2976	4296
B-12 @ 20'		1776	3168	3468
B-13 @ 5'		1848	3516	5472 (1) (3)
B-13 @ 15'		1548	3192	4512
B-1A @ 20' (2)			5760 (3)	
B-1A @ 40' (2)				4596
B-2A @ 15'				4212
Average		1764	3098	4337

Average strength (4) 32.0 degrees and 450 psf

(1) Rock in sample
(2) Insufficient sample for more points
(3) Too high, not used for average
(4) Calculated as a mathematical average

SHEAR TEST DIAGRAM

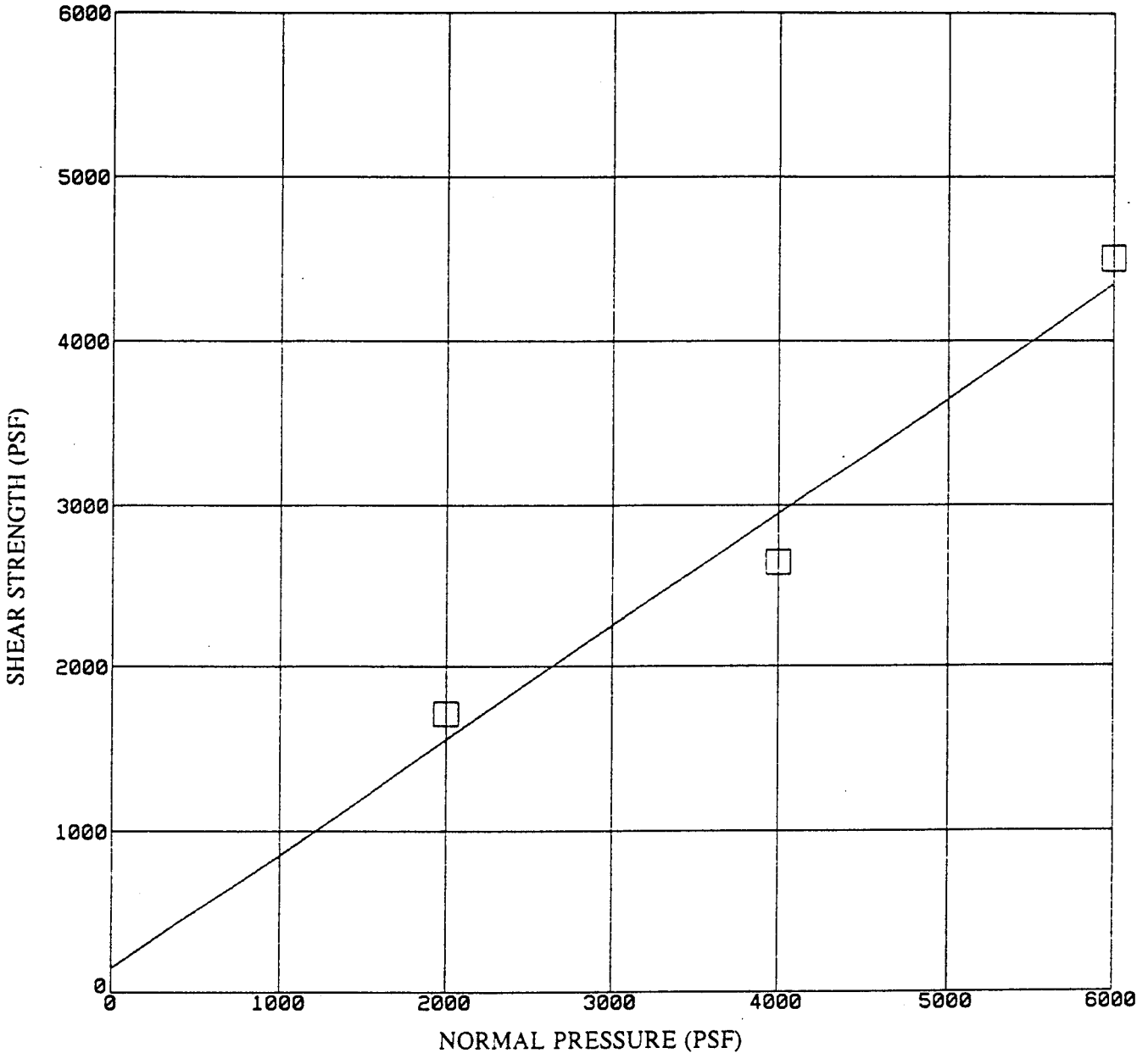
Client: **Rancho Las Flores Limited Partnership**	W.O. **215-1**
Project: **Highway 138**	Location: **San Bernardino County**



Shear Test Summary
Undisturbed Samples
Sheared after Saturation

SHEAR TEST DIAGRAM

Client: Rancho Las Flores Limited Partnership	W.O. 215-1
Project: Highway 138	Location: San Bernardino County



Initial Sample

Dry Density = 113.7 pcf
Moist. Cont. = 2.9%

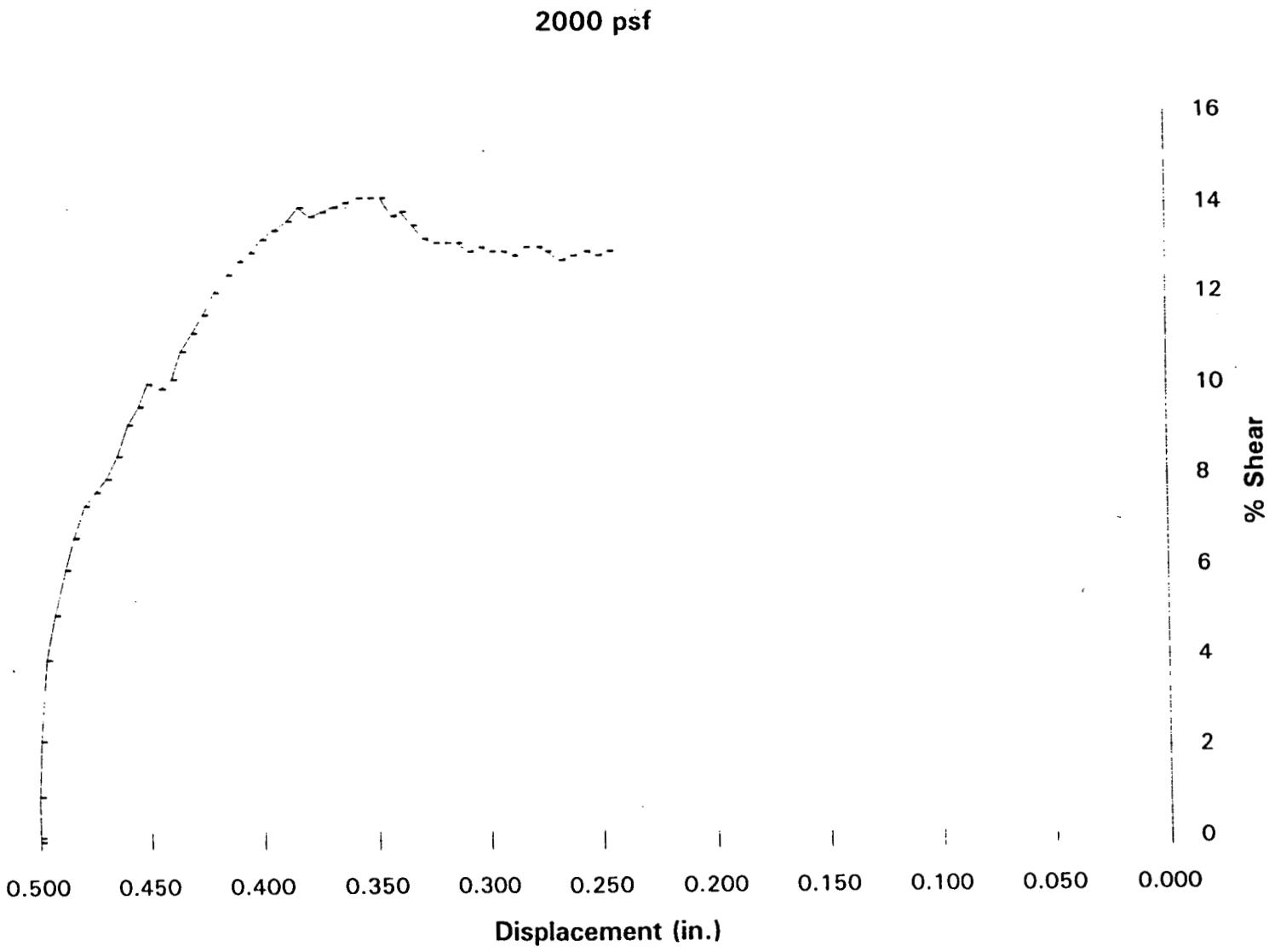
Sample: **B-5 @ 10.0 ft.**

Phi Angle: **35 deg.**

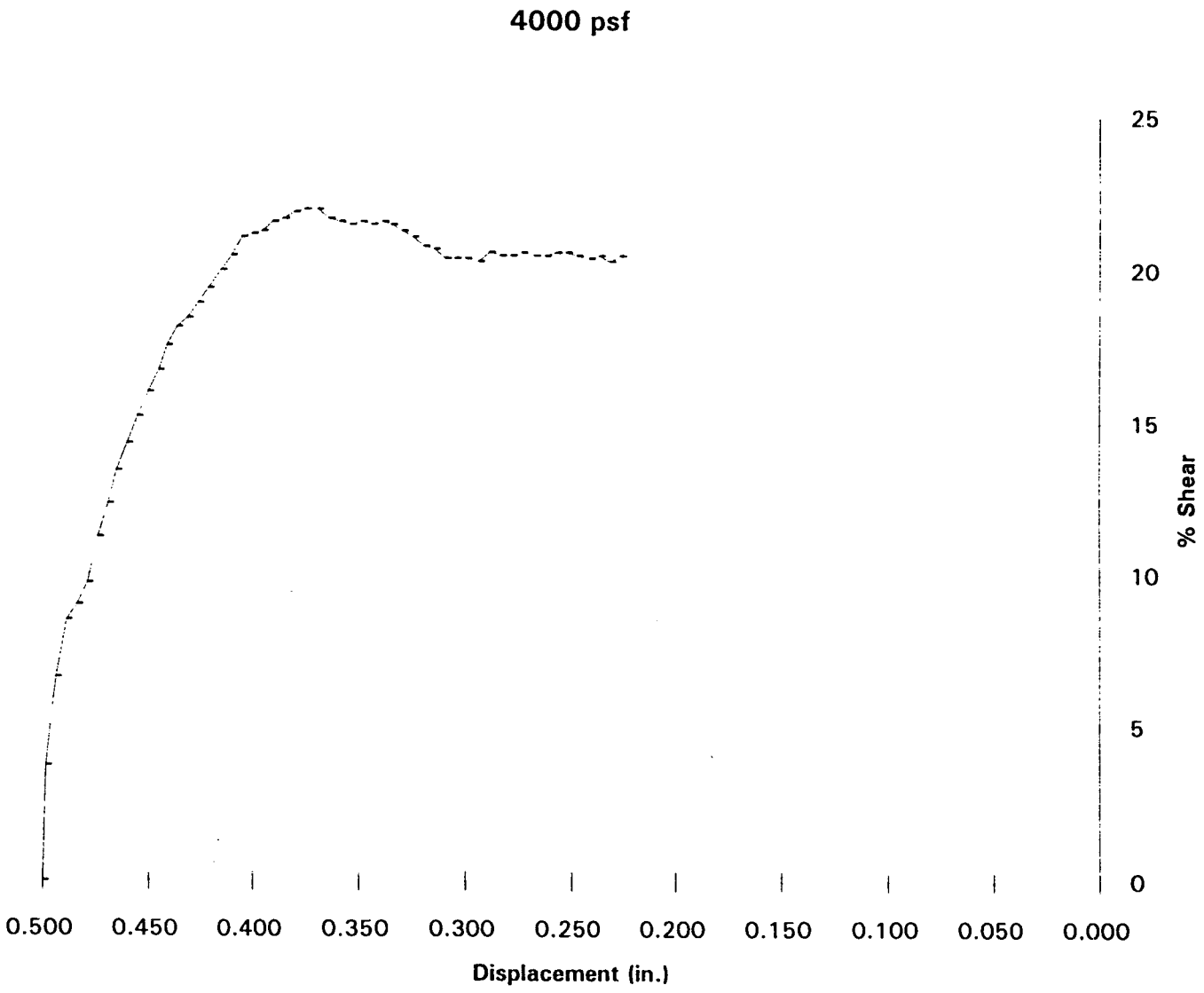
Cohesion: **152 psf**

TEST CONDITION: Undisturbed. tested at saturation

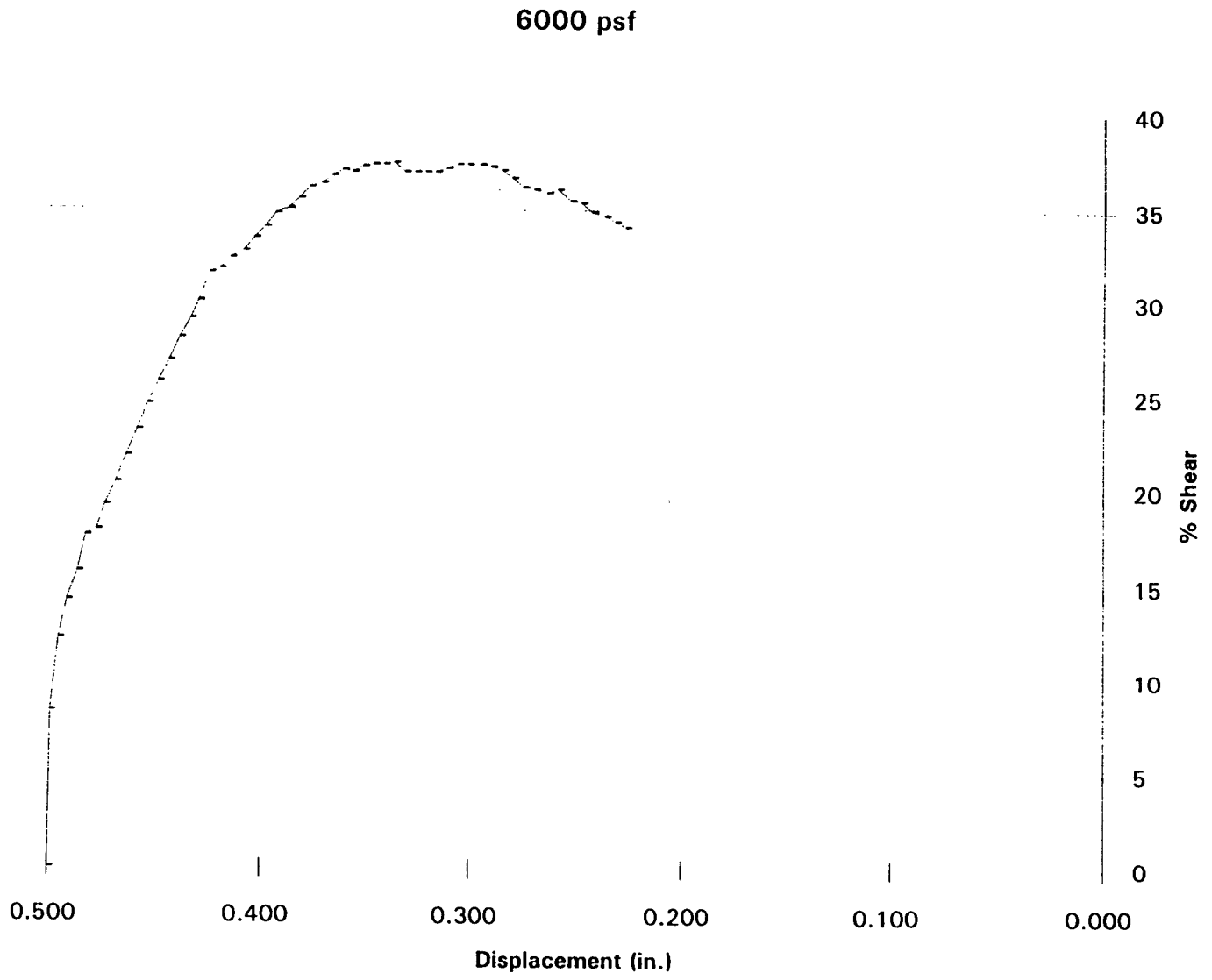
DIRECT SHEAR TEST PLOT OF B5 @ 10 FT.



DIRECT SHEAR TEST PLOT OF B5 @ 10 FT.

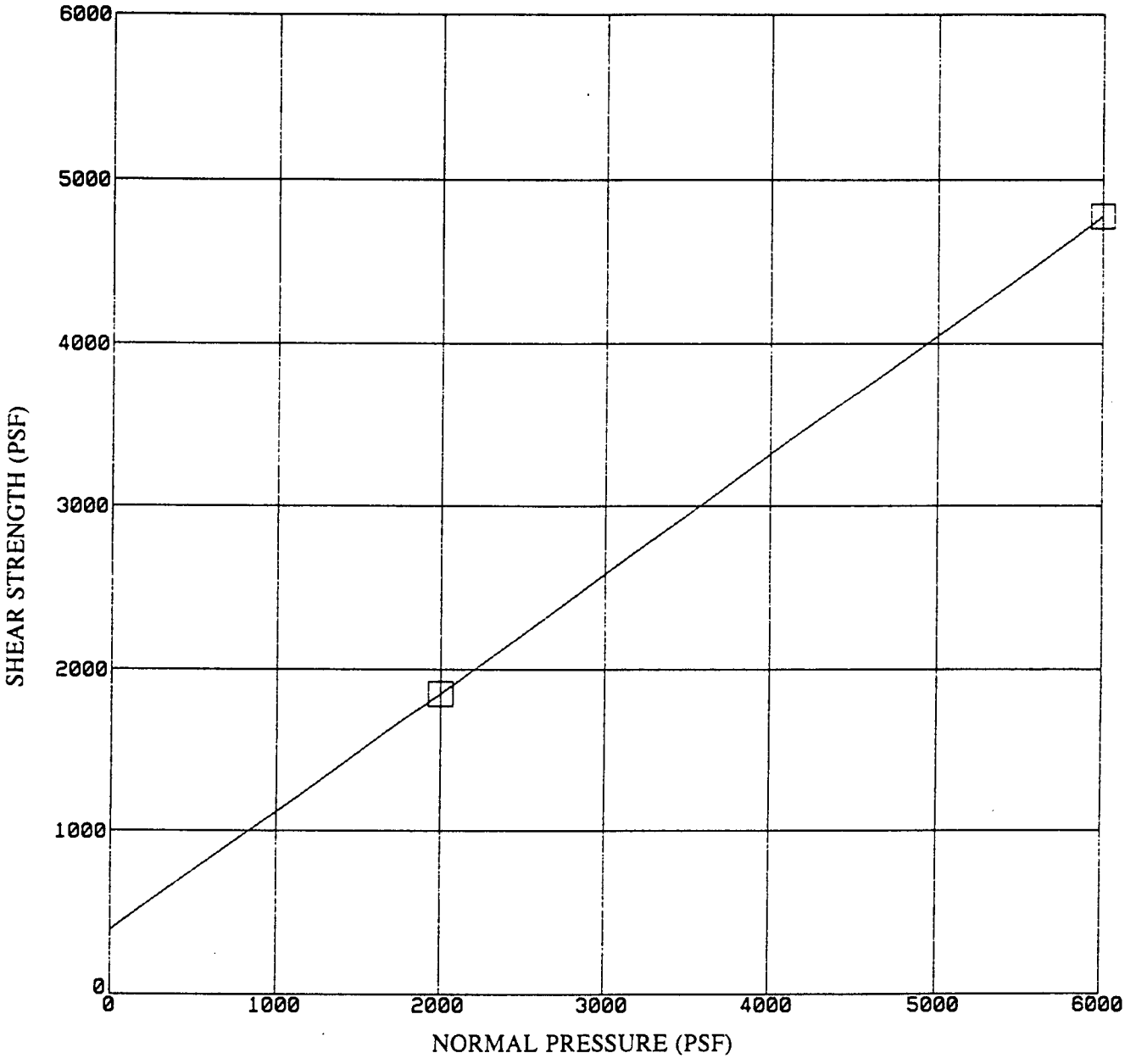


DIRECT SHEAR TEST PLOT OF B5 @ 10 FT.



SHEAR TEST DIAGRAM

Client: **Rancho Las Flores Limited Partnership**	W.O. **215-1**
Project: **Highway 138**	Location: **San Bernardino County**



Sample: **B-10 @ 10.0 ft.**

Phi Angle: **36 deg.**
Cohesion: **384 psf**

TEST CONDITION: Undisturbed, tested at saturation

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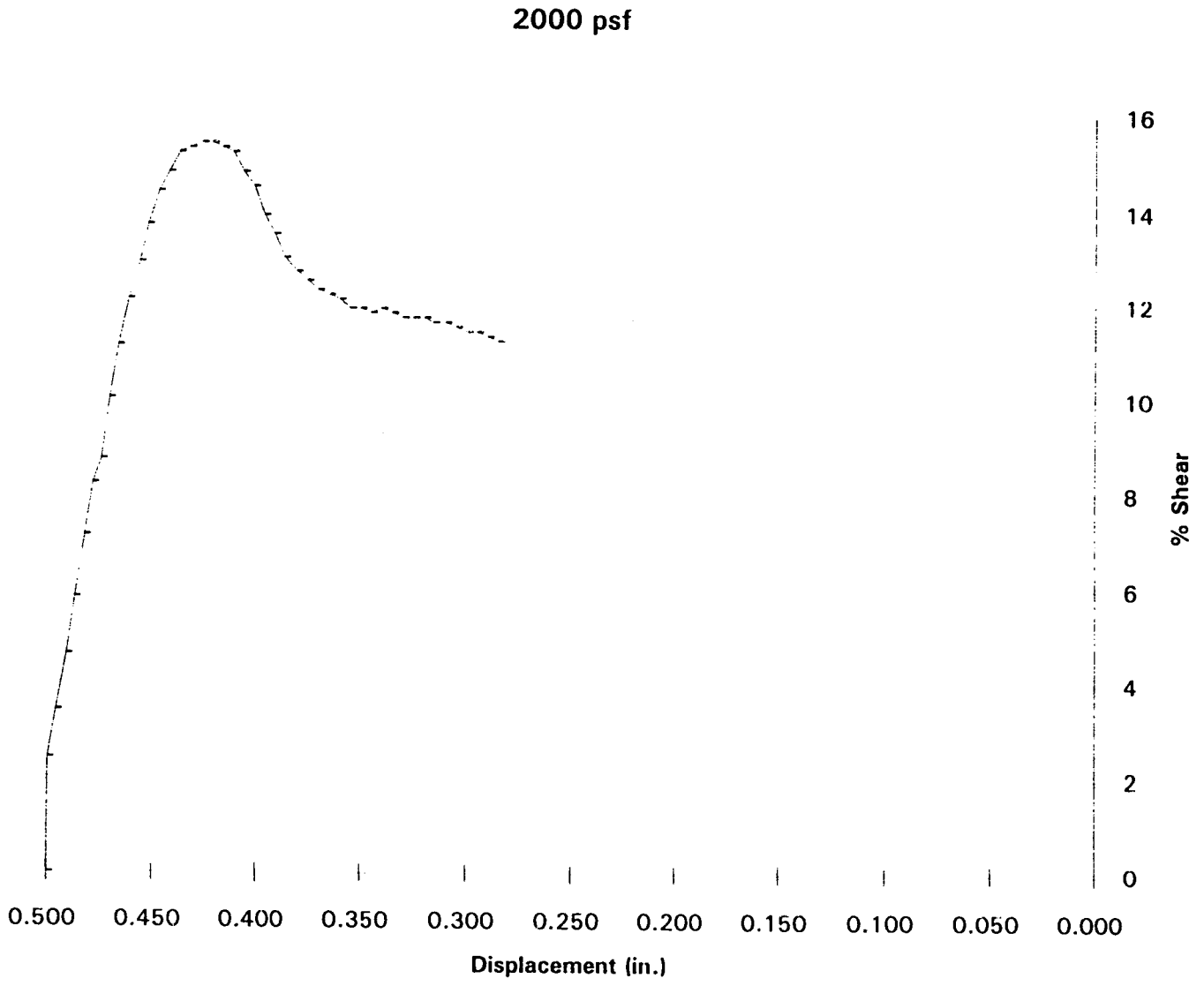
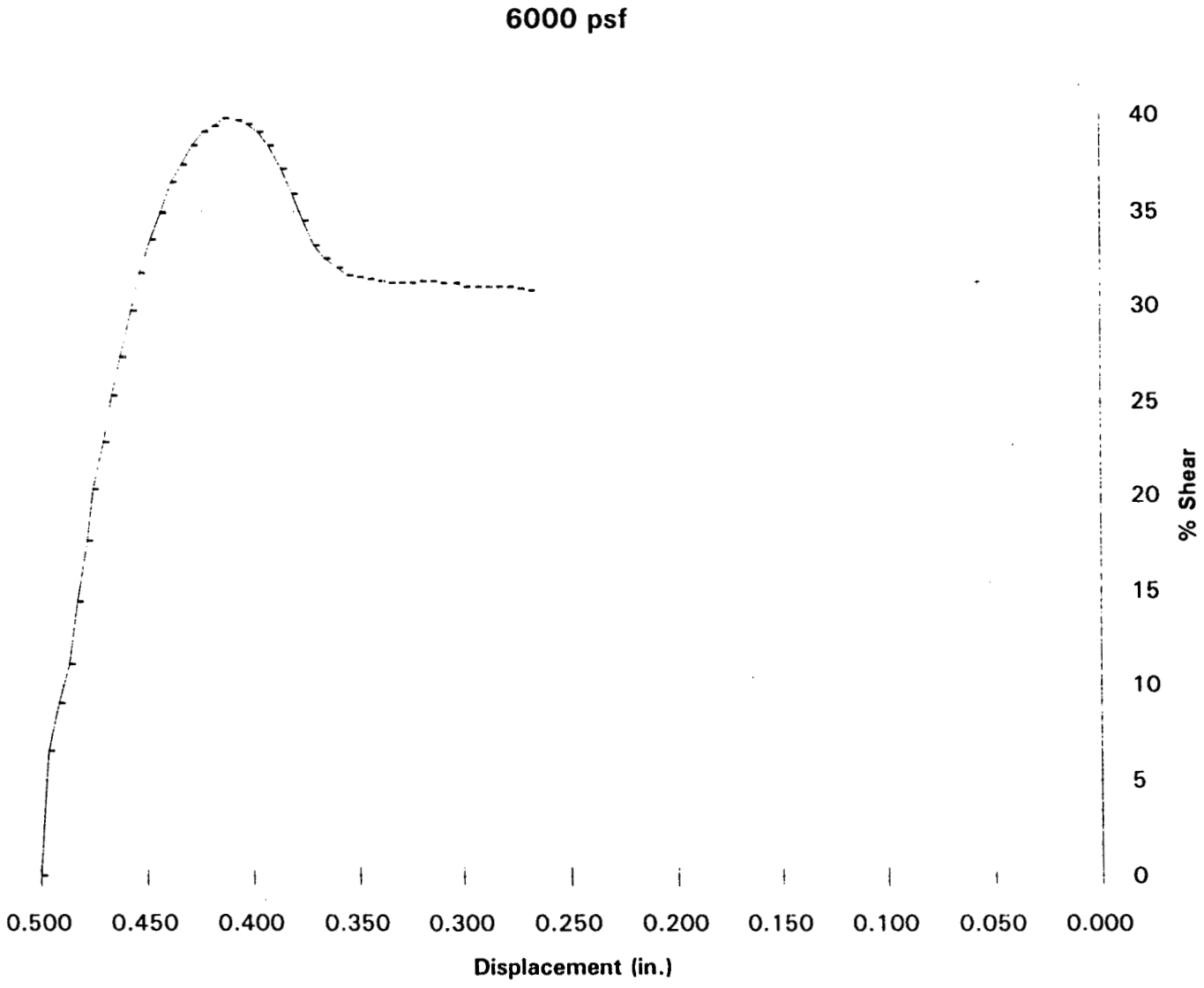
Plate

DIRECT SHEAR TEST PLOT OF B10 @ 10 FT.

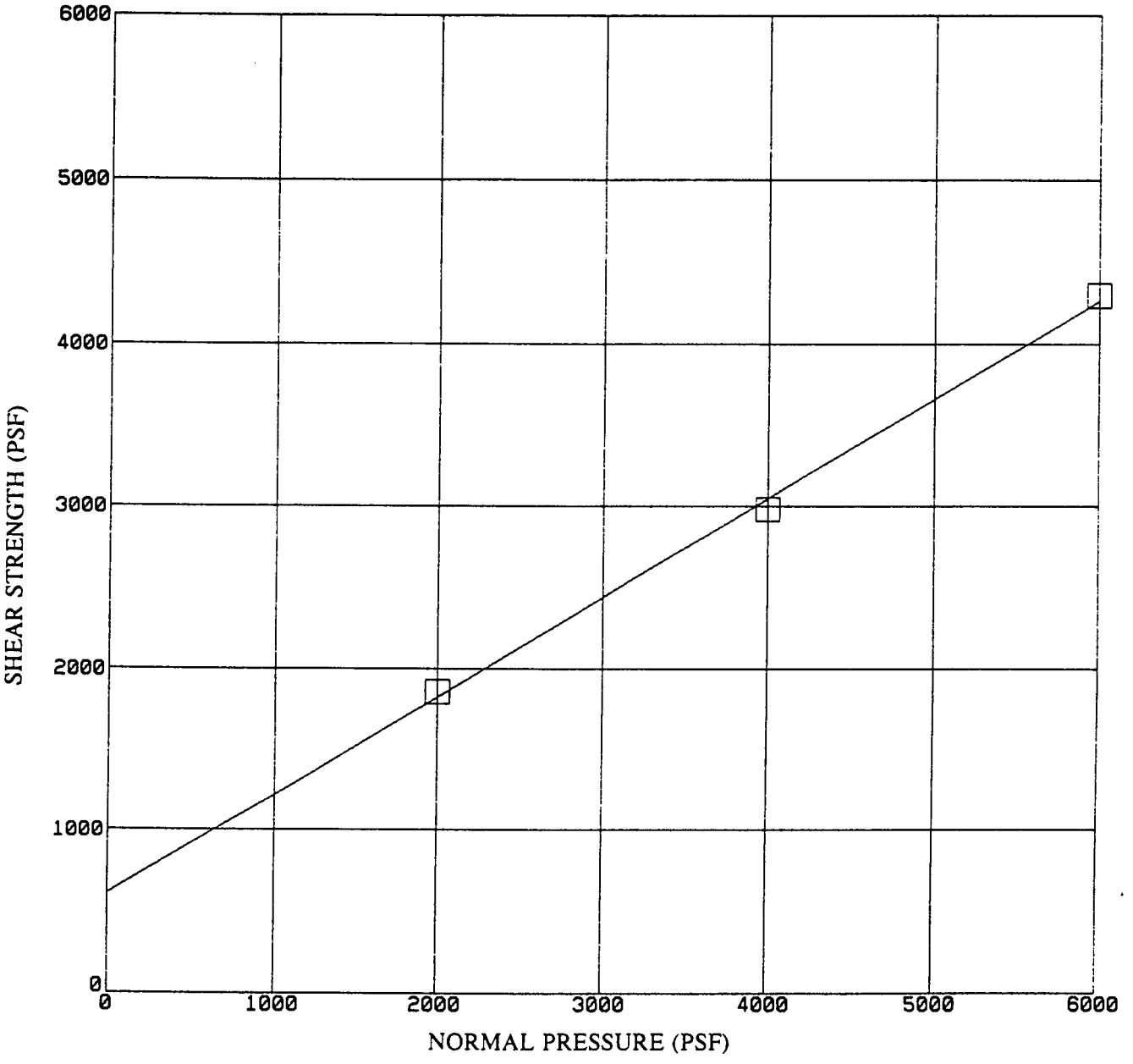


DIRECT SHEAR TEST PLOT OF B10 @ 10 FT.



SHEAR TEST DIAGRAM

Client: **Rancho Las Flores Limited Partnership**	W.O. **215-1**
Project: **Highway 138**	Location: **San Bernardino County**



Initial Sample

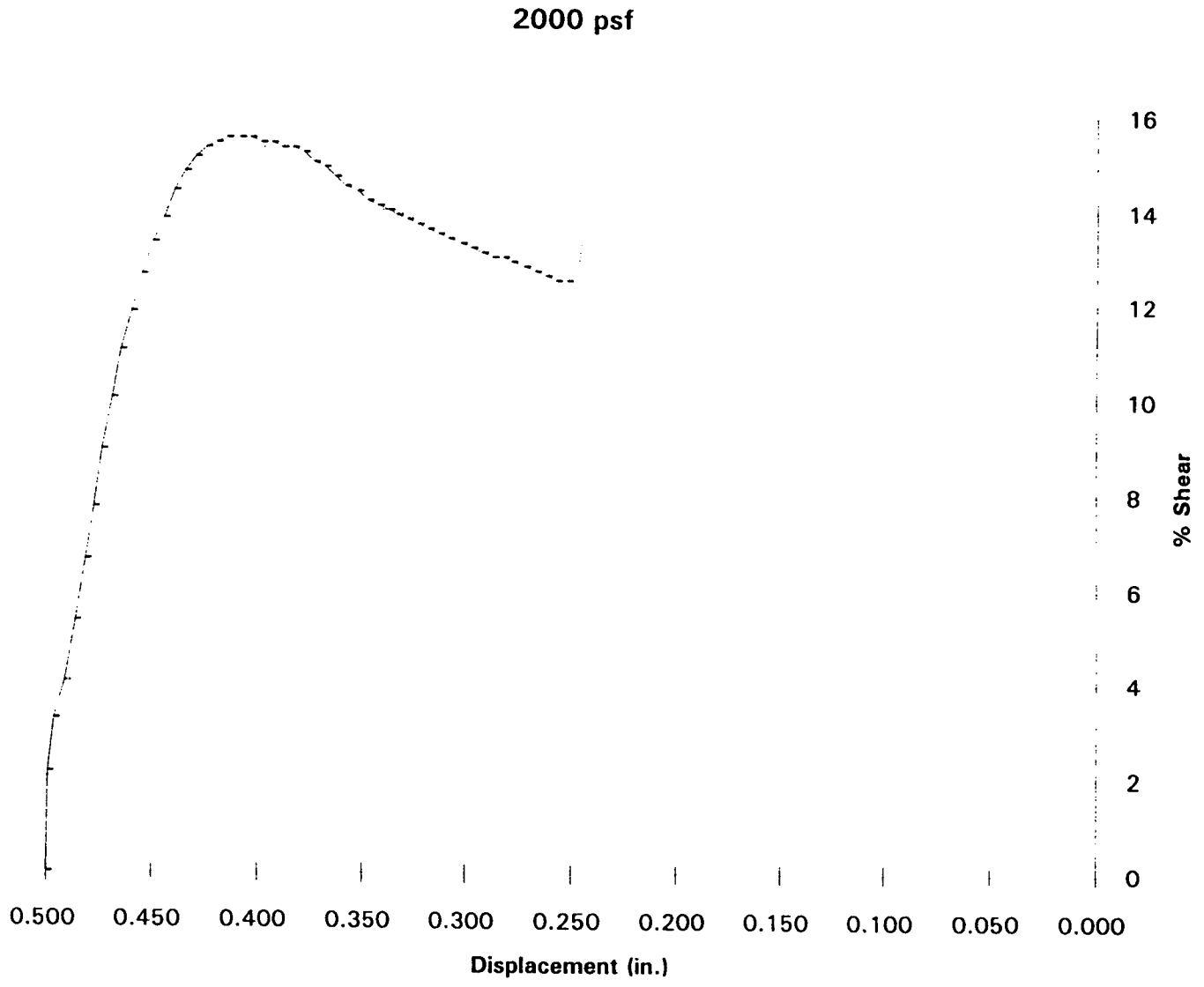
Dry Density = 117.9 pcf
Moist. Cont. = 6.4%

Sample: **B-11 @ 5.0 ft.**

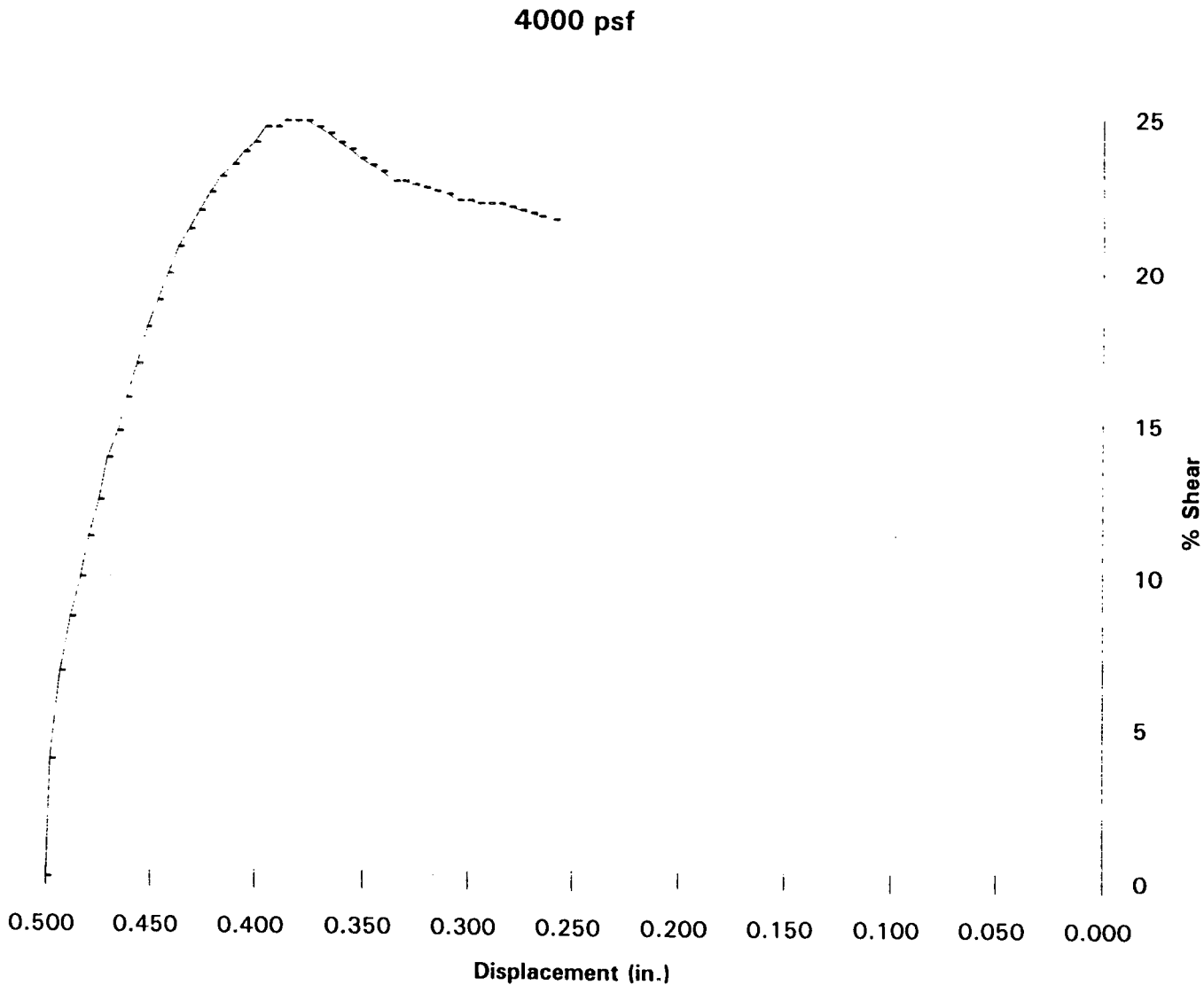
Phi Angle: **31 deg.**
Cohesion: **608 psf**

TEST CONDITION: Undisturbed, tested at saturation

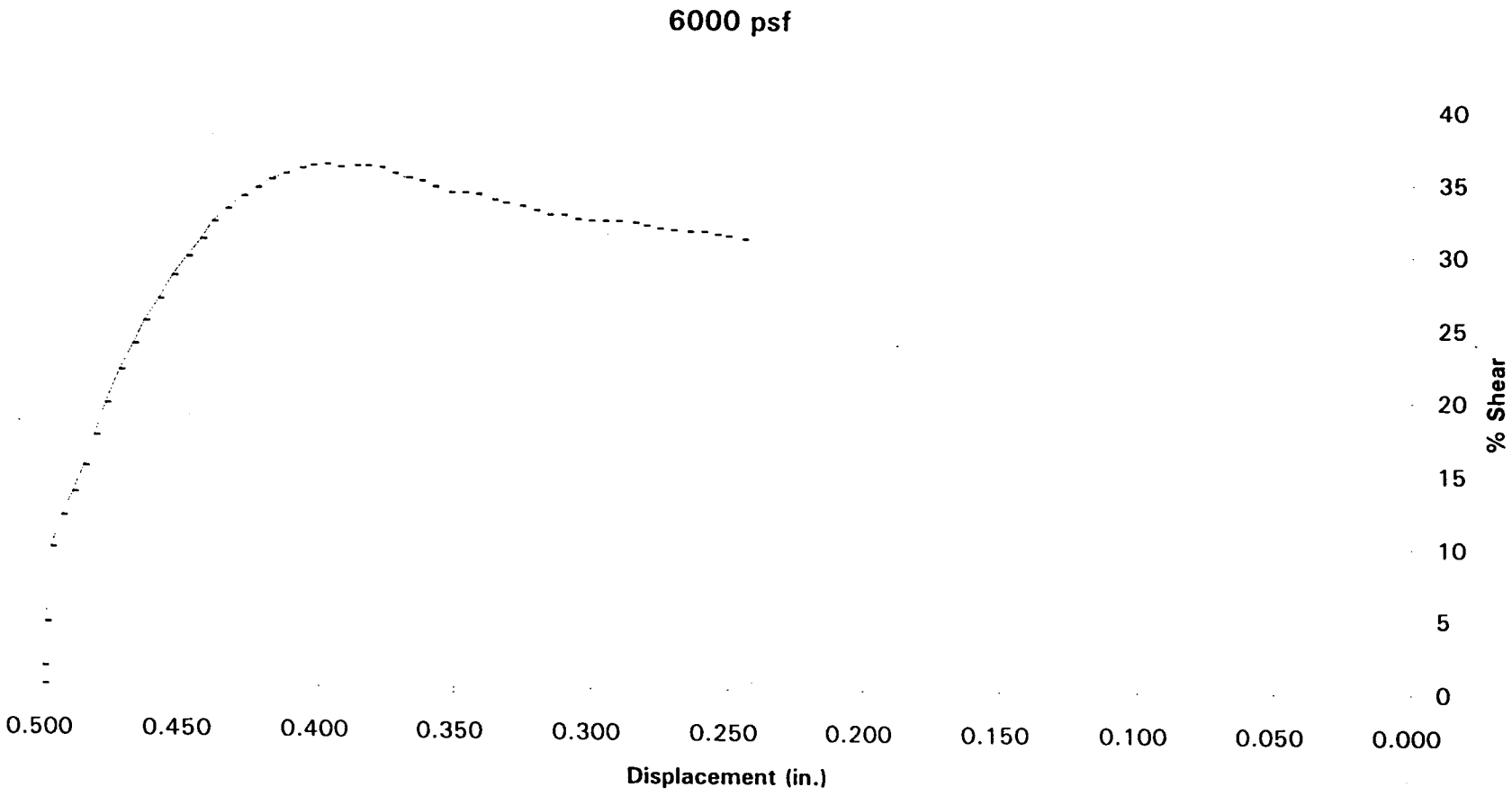
DIRECT SHEAR TEST PLOT OF B11 @ 5 FT.



DIRECT SHEAR TEST PLOT OF B11 @ 5 FT.

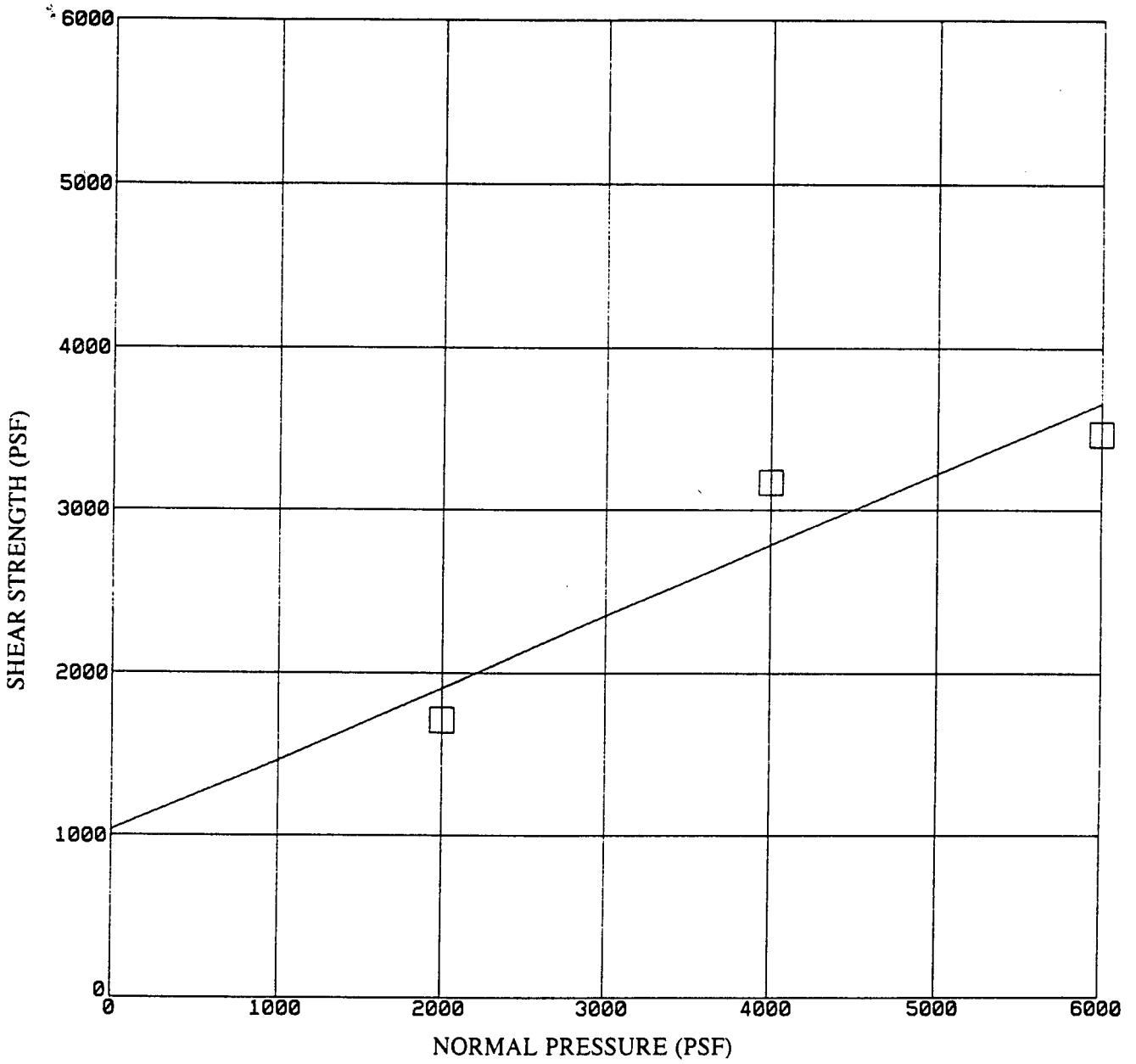


DIRECT SHEAR TEST PLOT OF B-11 @ 5 FT.



SHEAR TEST DIAGRAM

Client: **Rancho Las Flores Limited Partnership**	W.O. **215-1**
Project: **Highway 138**	Location: **San Bernardino County**



Initial Sample

Dry Density = 120.3 pcf
Moist. Cont. = 5.4%

Sample: **B-12 @ 20.0 ft.**

Phi Angle: **24 deg.**

Cohesion: **1032 psf**

TEST CONDITION: Undisturbed, tested at saturation

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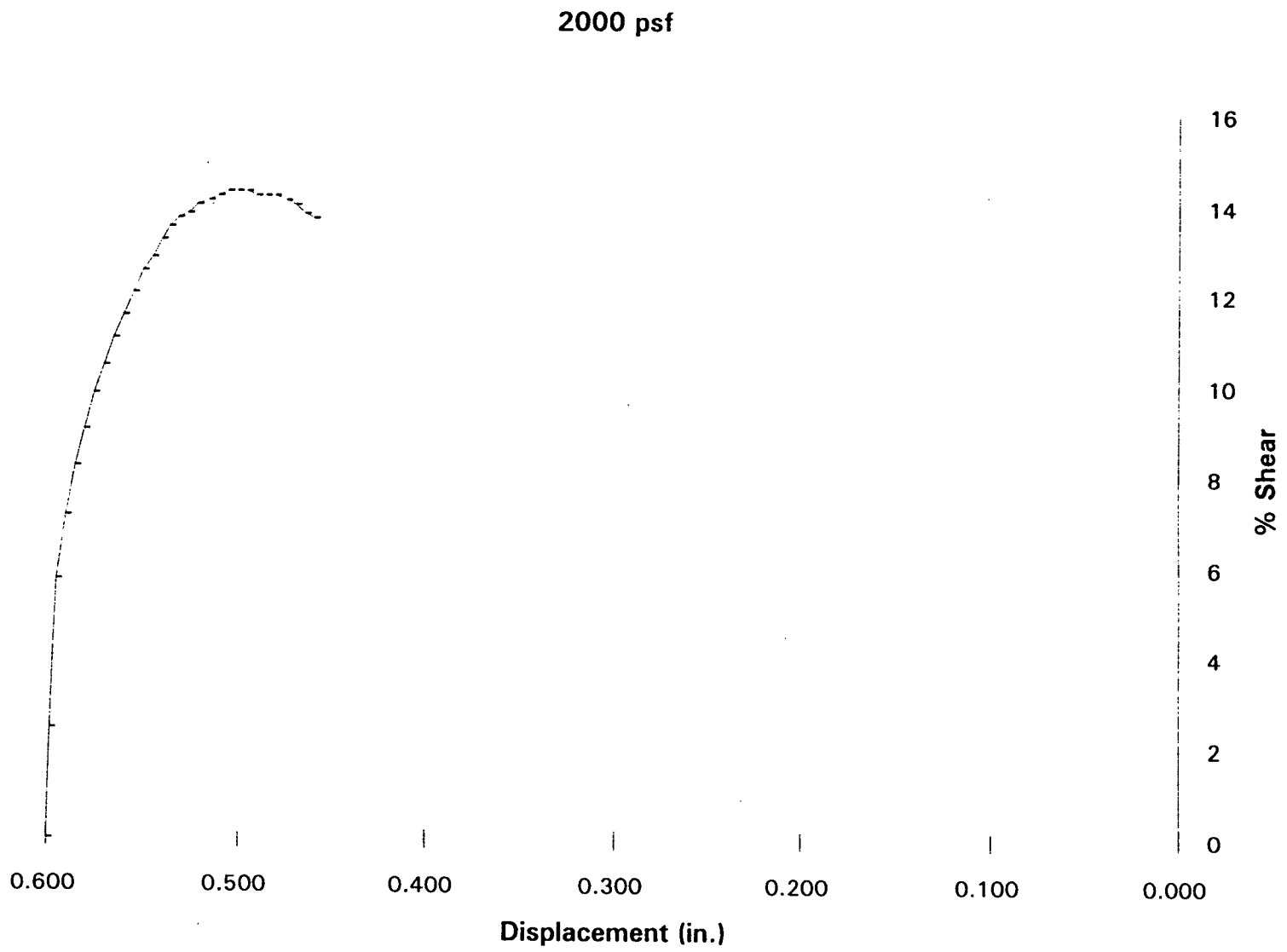
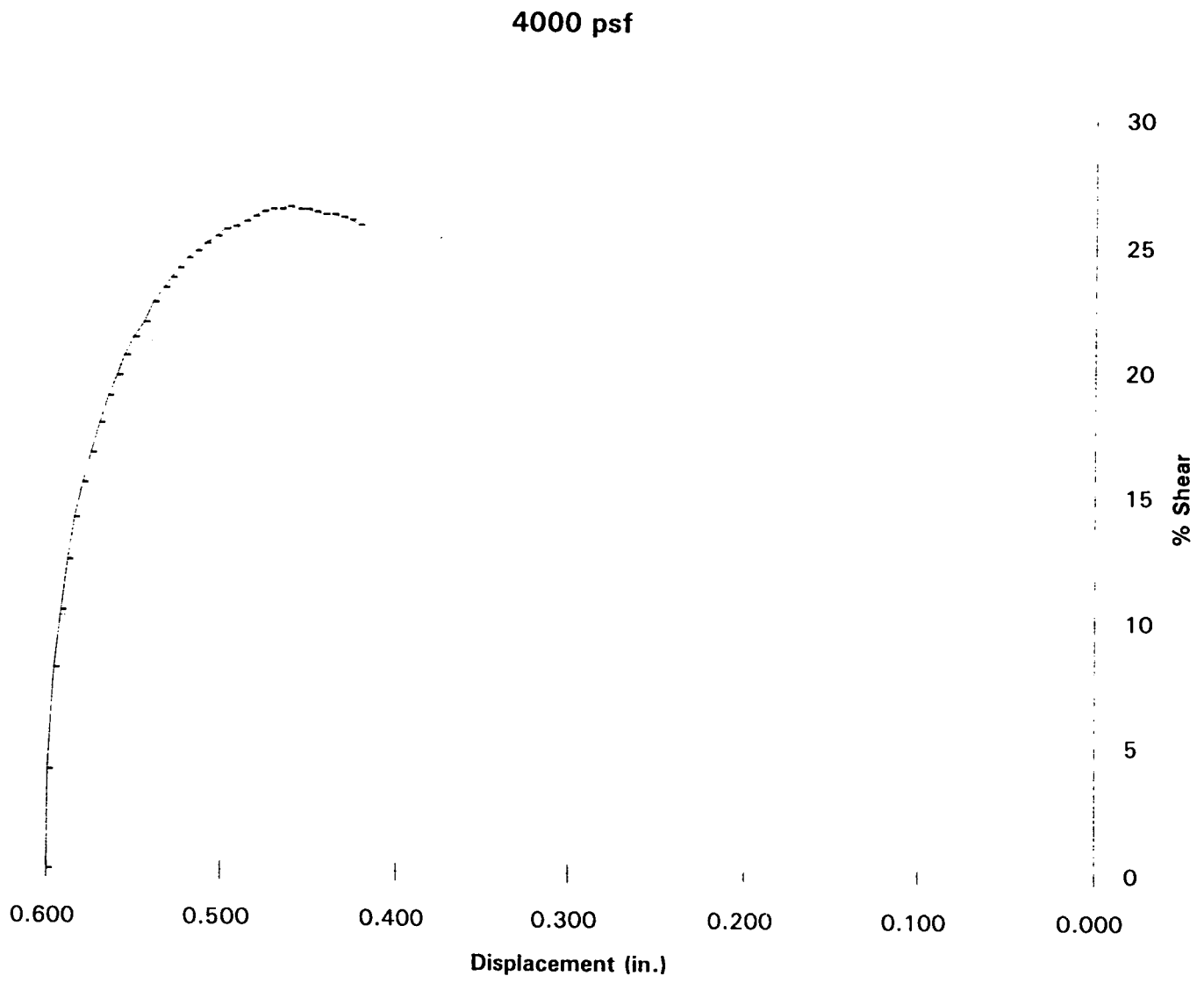
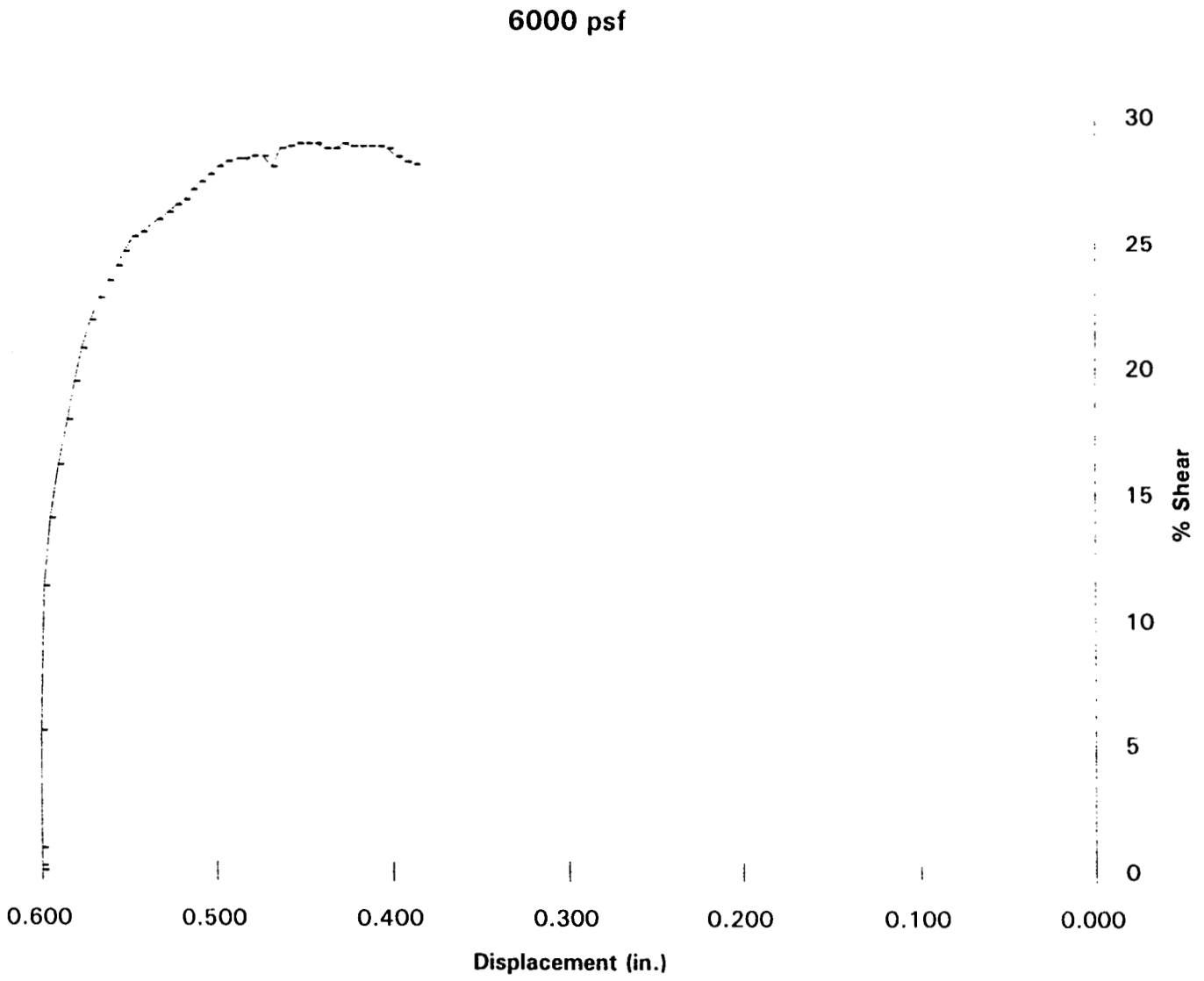
DIRECT SHEAR TEST PLOT OF B12 @ 20 FT.



DIRECT SHEAR TEST PLOT OF B12 @ 20 FT.

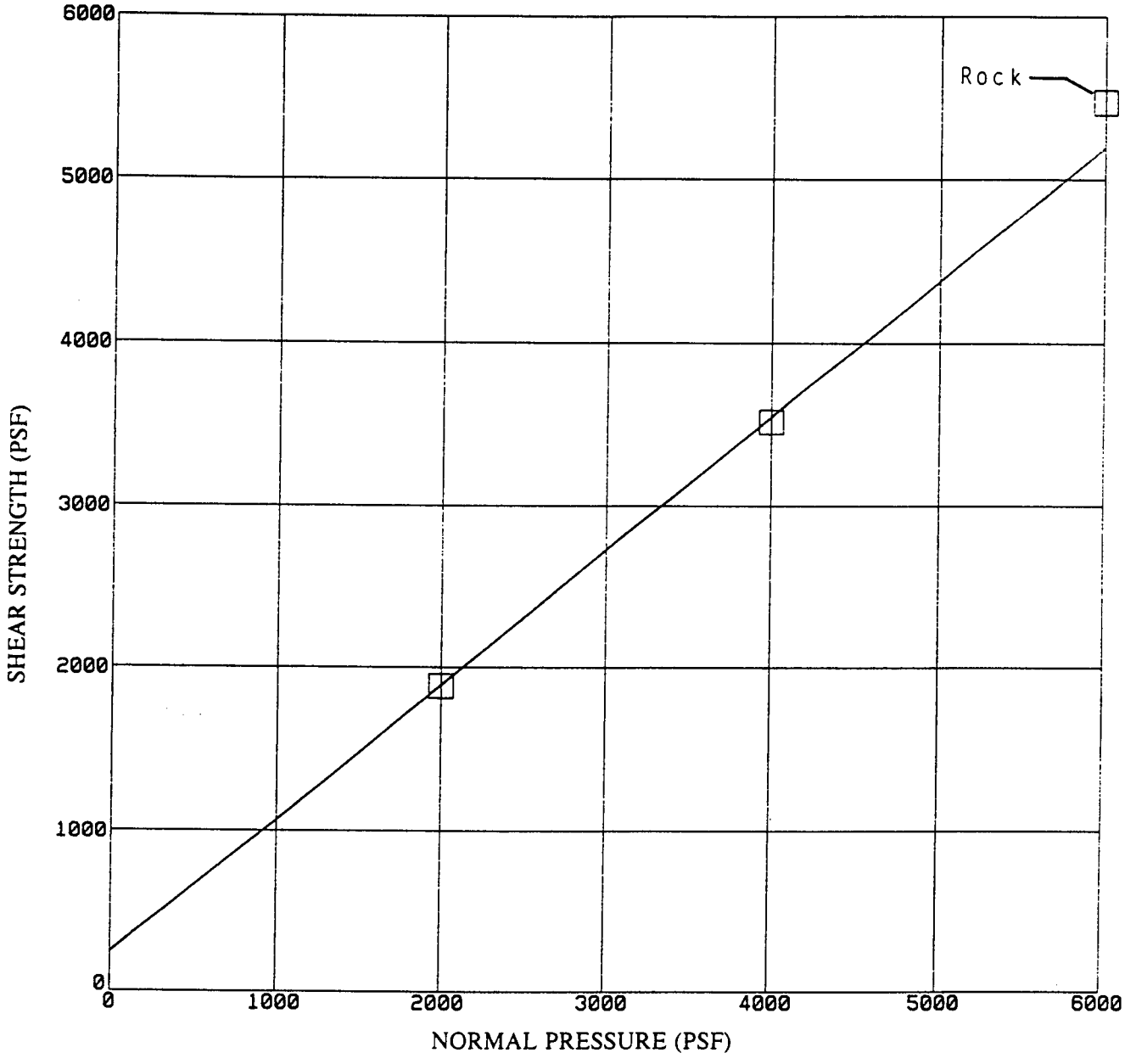


DIRECT SHEAR TEST PLOT OF B12 @ 20 FT.



SHEAR TEST DIAGRAM

Client: **Rancho Las Flores Limited Partnership**		W.O. **215-1**
Project: **Highway 138**	Location: **San Bernardino County**	



Initial Sample

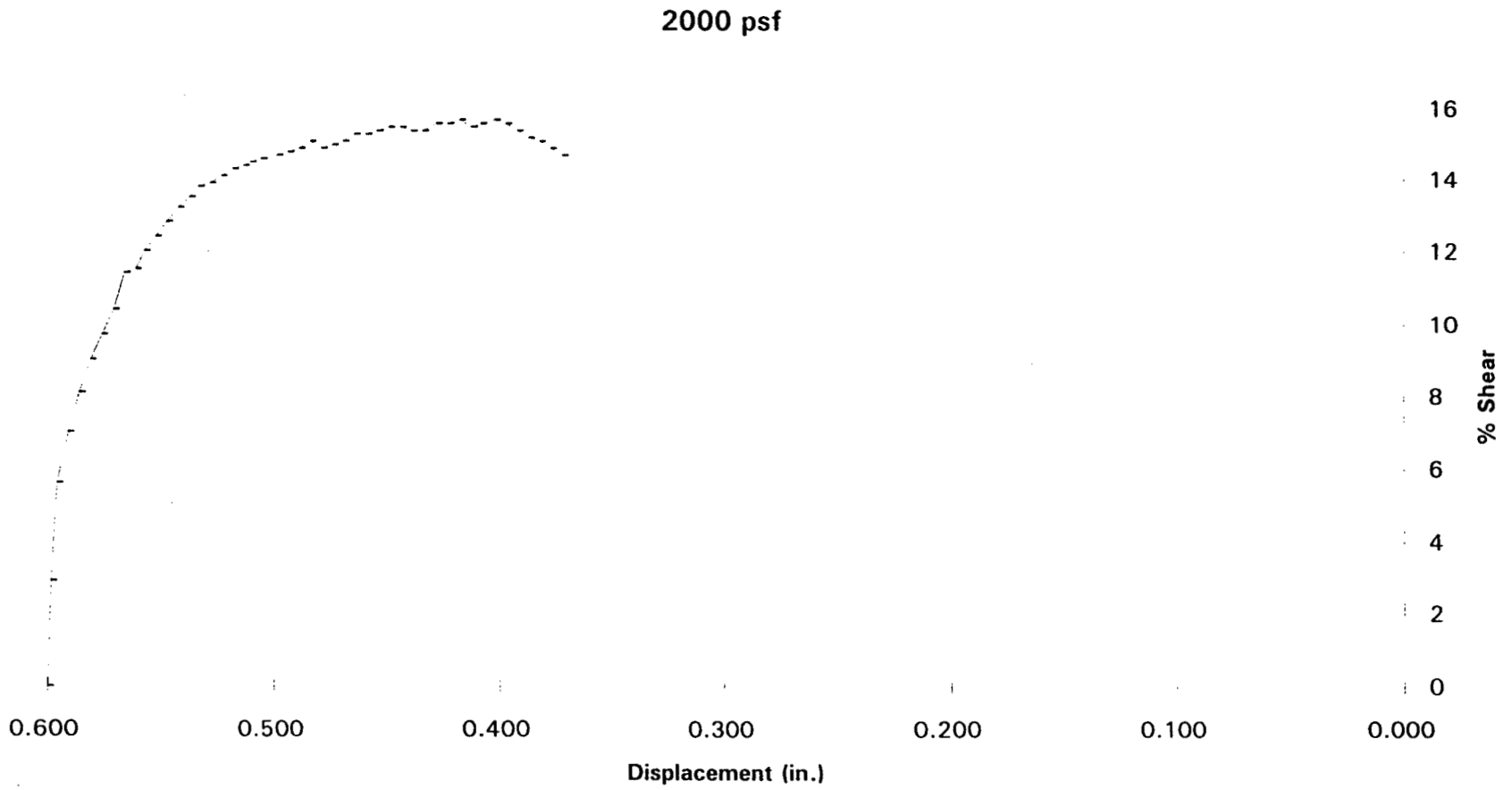
Dry Density = 111.5 pcf
Moist. Cont. = 5.3%

Sample: **B-13 @ 5.0 ft.**

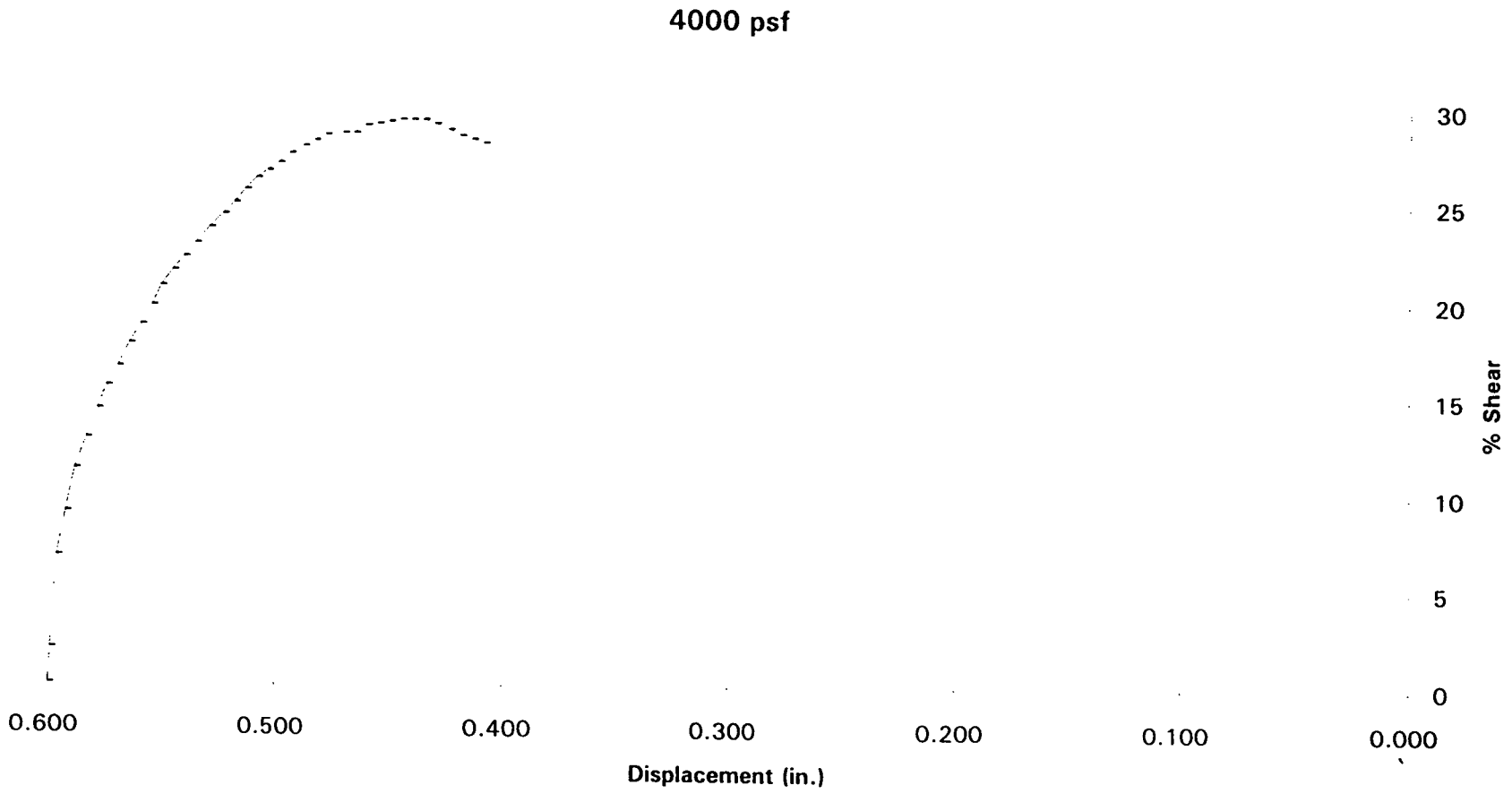
Phi Angle: **39 deg.**
Cohesion: **230 psf**

TEST CONDITION: Undisturbed, tested at saturation

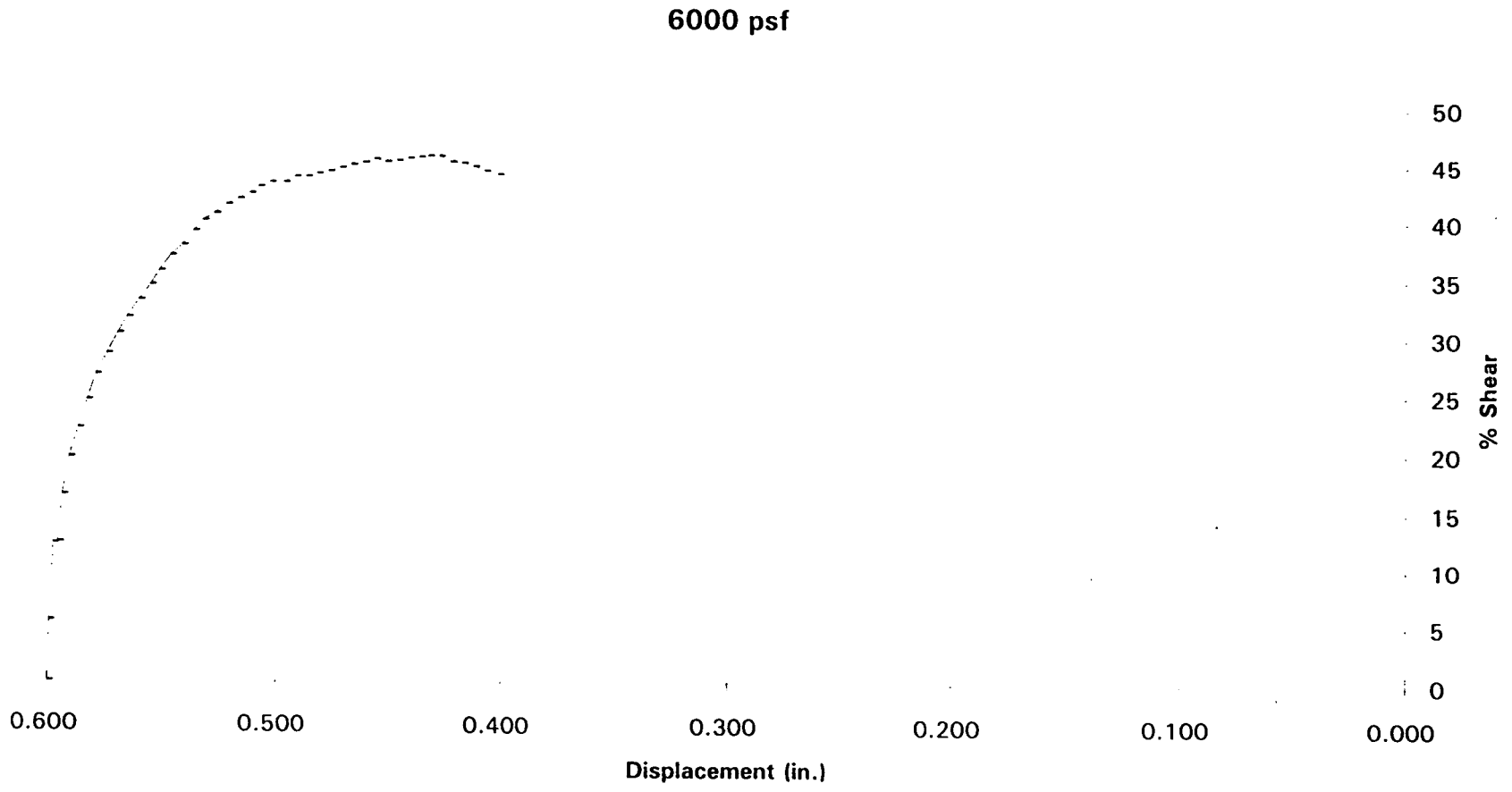
DIRECT SHEAR TEST PLOT OF B-13 @ 5 FT.



DIRECT SHEAR TEST PLOT OF B-13 @ 5 FT.

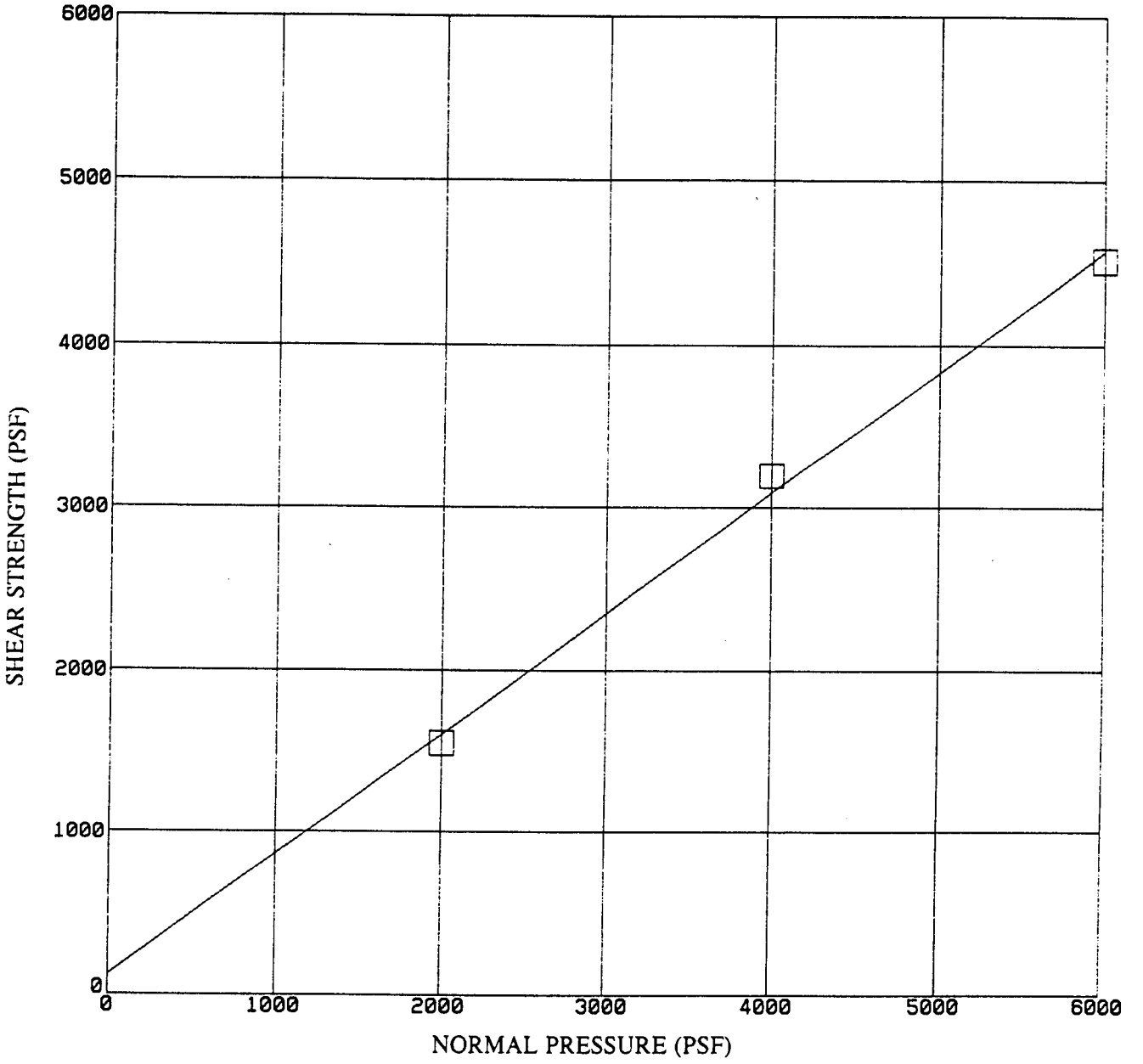


DIRECT SHEAR TEST PLOT OF B-13 @ 5 FT.



SHEAR TEST DIAGRAM

Client: **Rancho Las Flores Limited Partnership**		W.O. **215-1**
Project: **Highway 138**	Location: **San Bernardino County**	



Initial Sample

Dry Density = 121.9 pcf
Moist. Cont. = 3.1%

Sample: **B-13 @ 15.0 ft.**

Phi Angle: **37 deg.**
Cohesion: **120 psf**

TEST CONDITION: Undisturbed, tested at saturation

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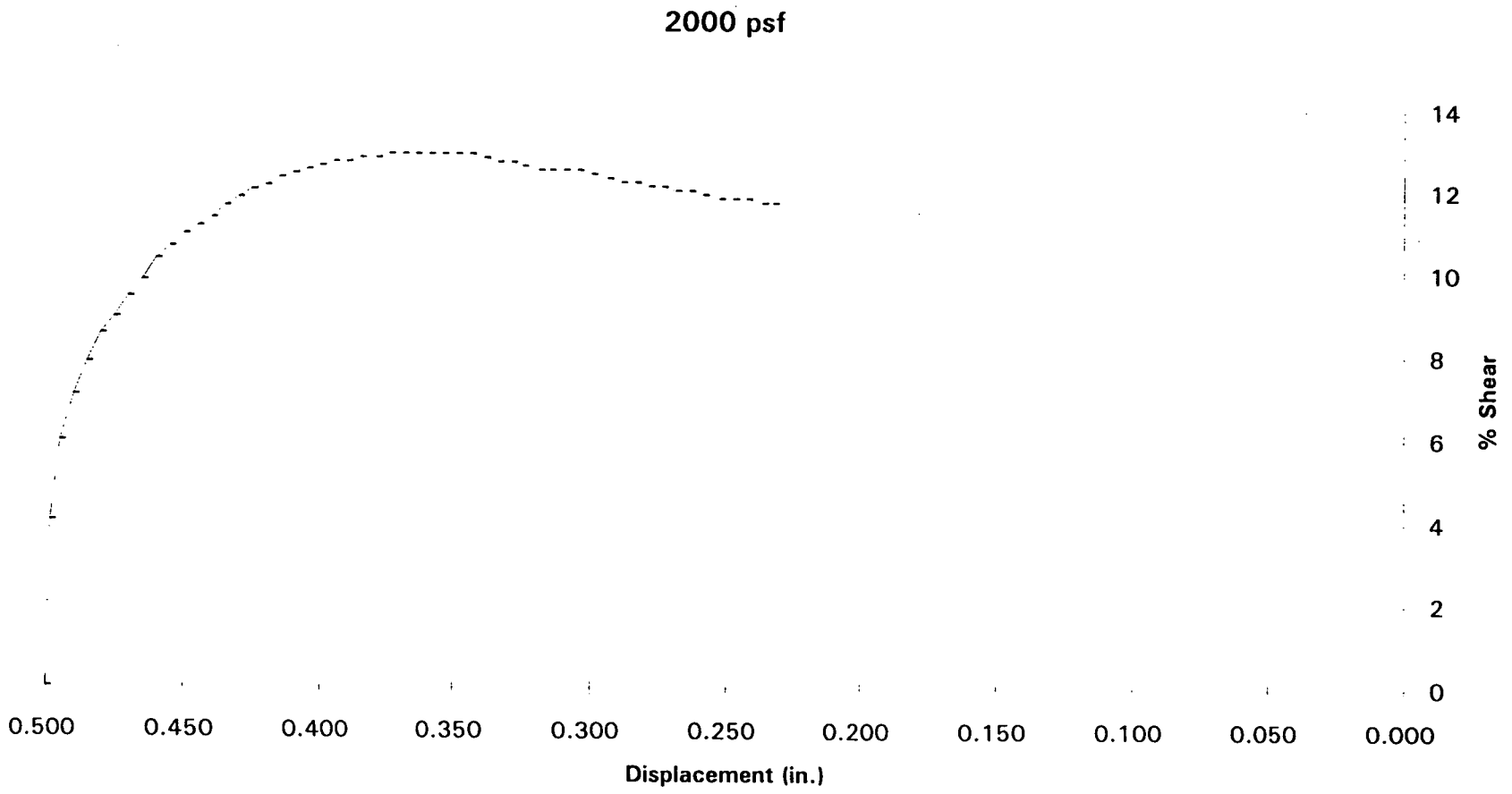
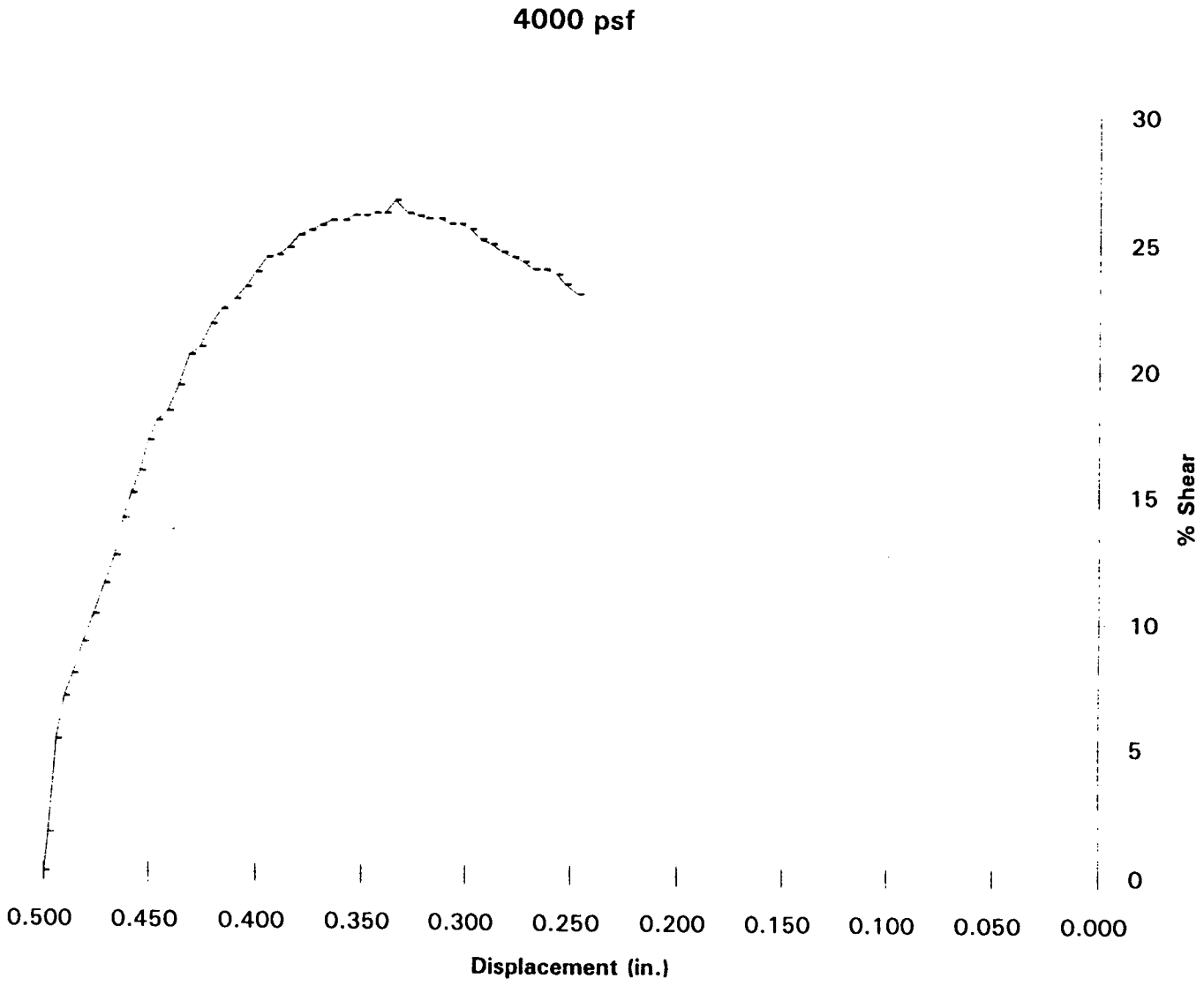
Plate

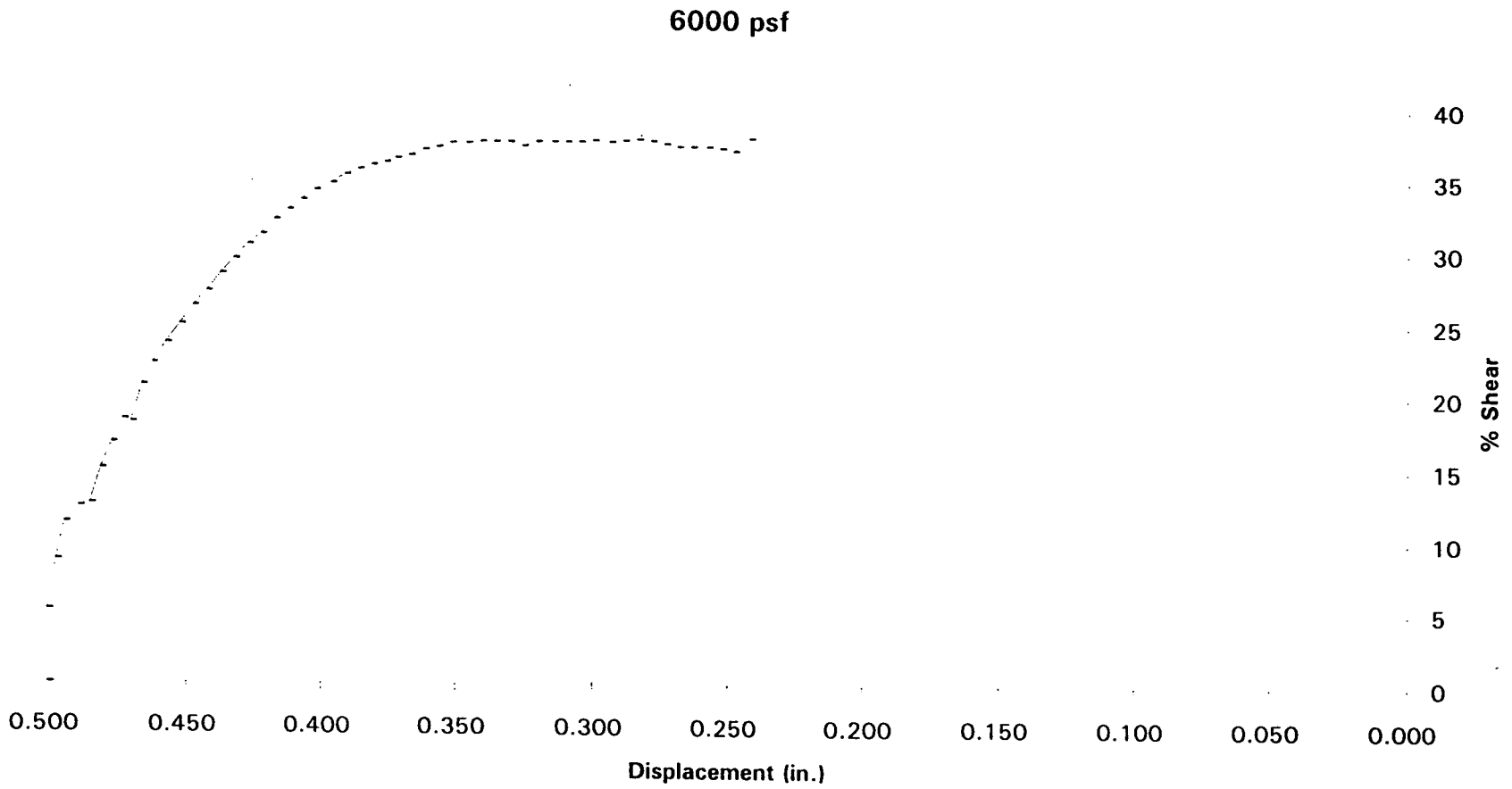
DIRECT SHEAR TEST PLOT OF B-13 @ 15 FT.



DIRECT SHEAR TEST PLOT OF B13 @ 15 FT.

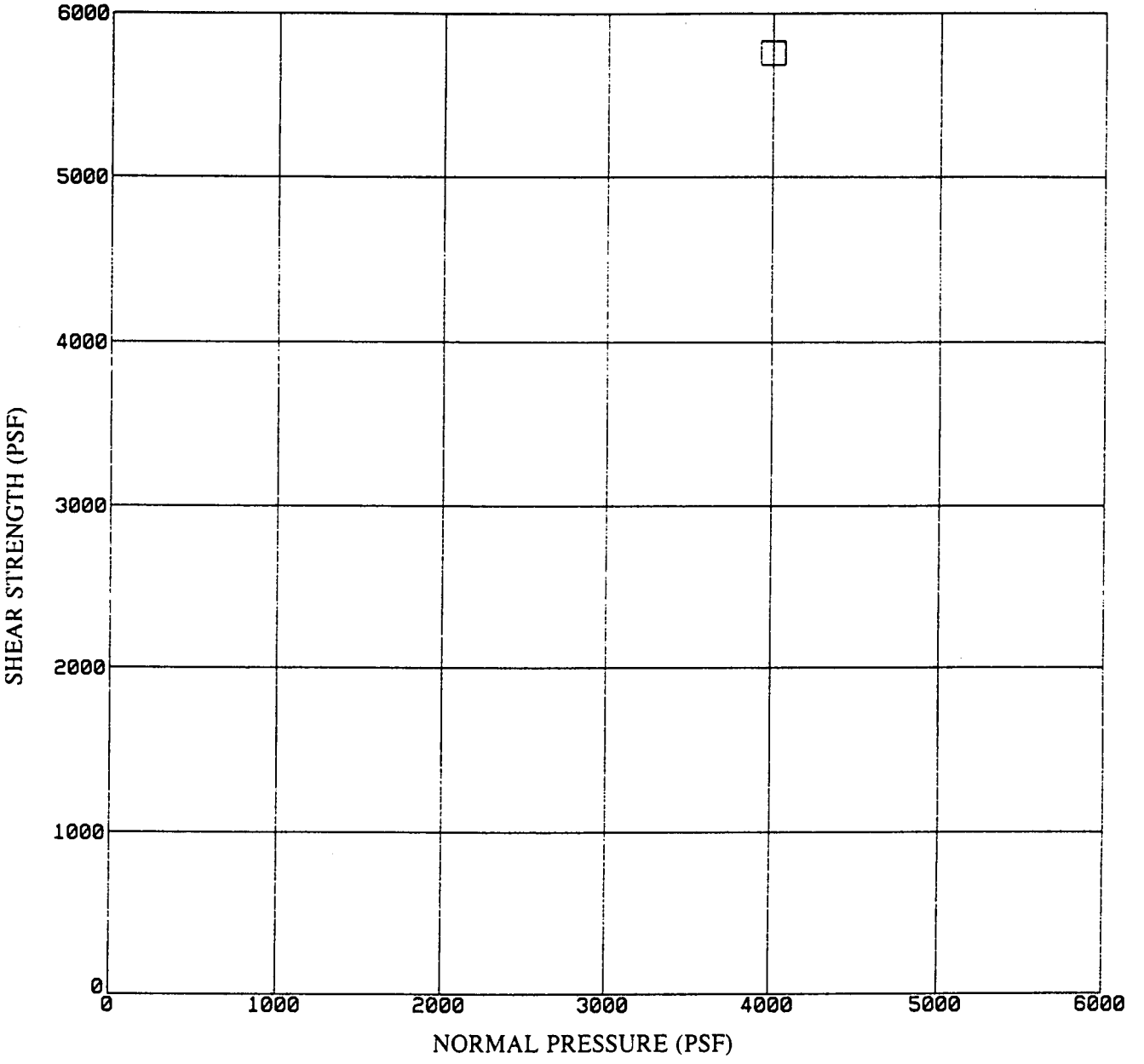


DIRECT SHEAR TEST PLOT OF B-13 @ 15 FT.



SHEAR TEST DIAGRAM

Client: **Rancho Las Flores Limited Partnership**	W.O. **215-1**
Project: **Highway 138**	Location: **San Bernardino County**



Initial Sample

Dry Density = 127.4 pcf
Moist. Cont. = 10.4%

Sample: **B-1A @ 20.0 ft.**

Phi Angle: **deg.**

Cohesion: **psf**

TEST CONDITION: Undisturbed, tested at saturation

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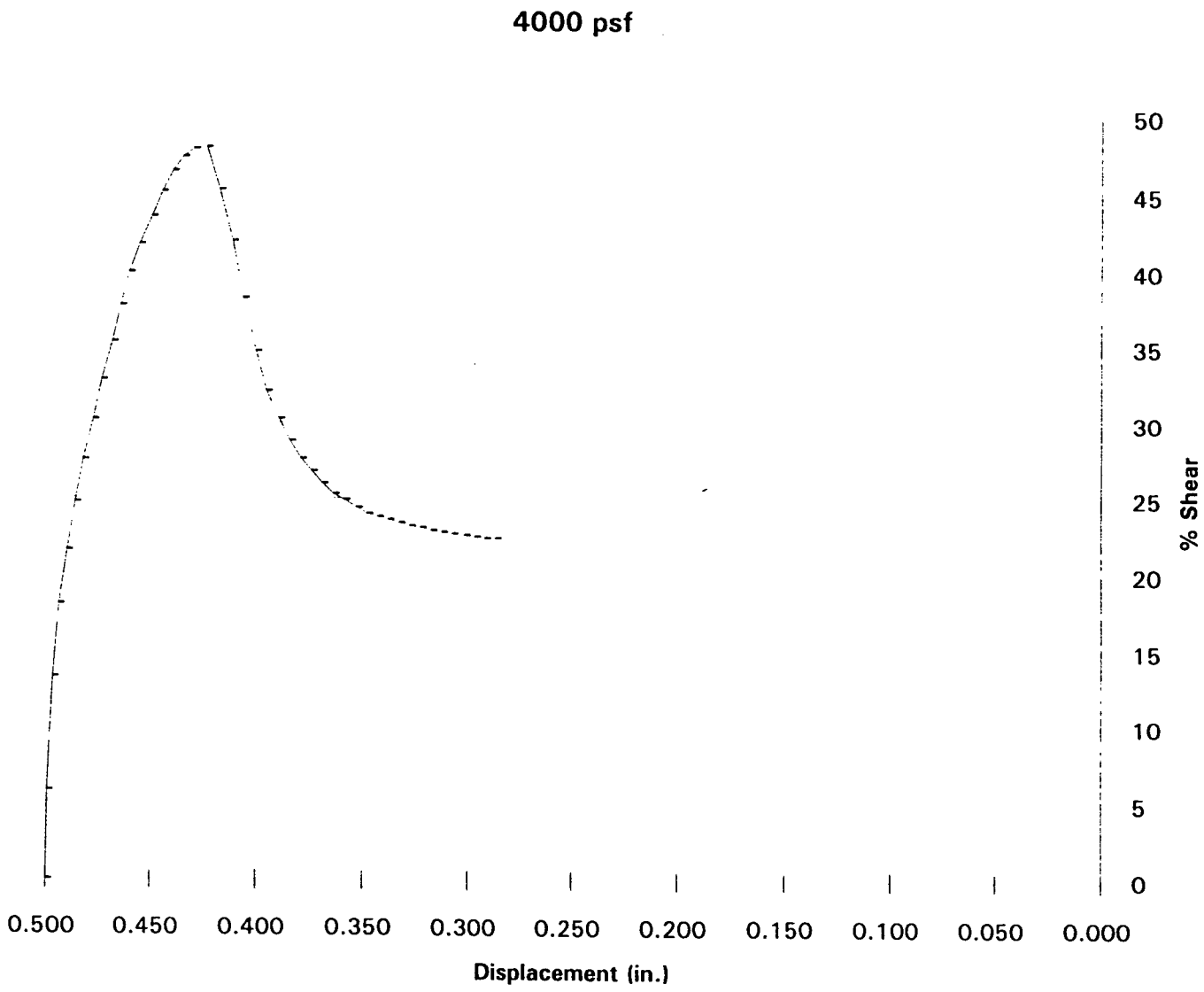
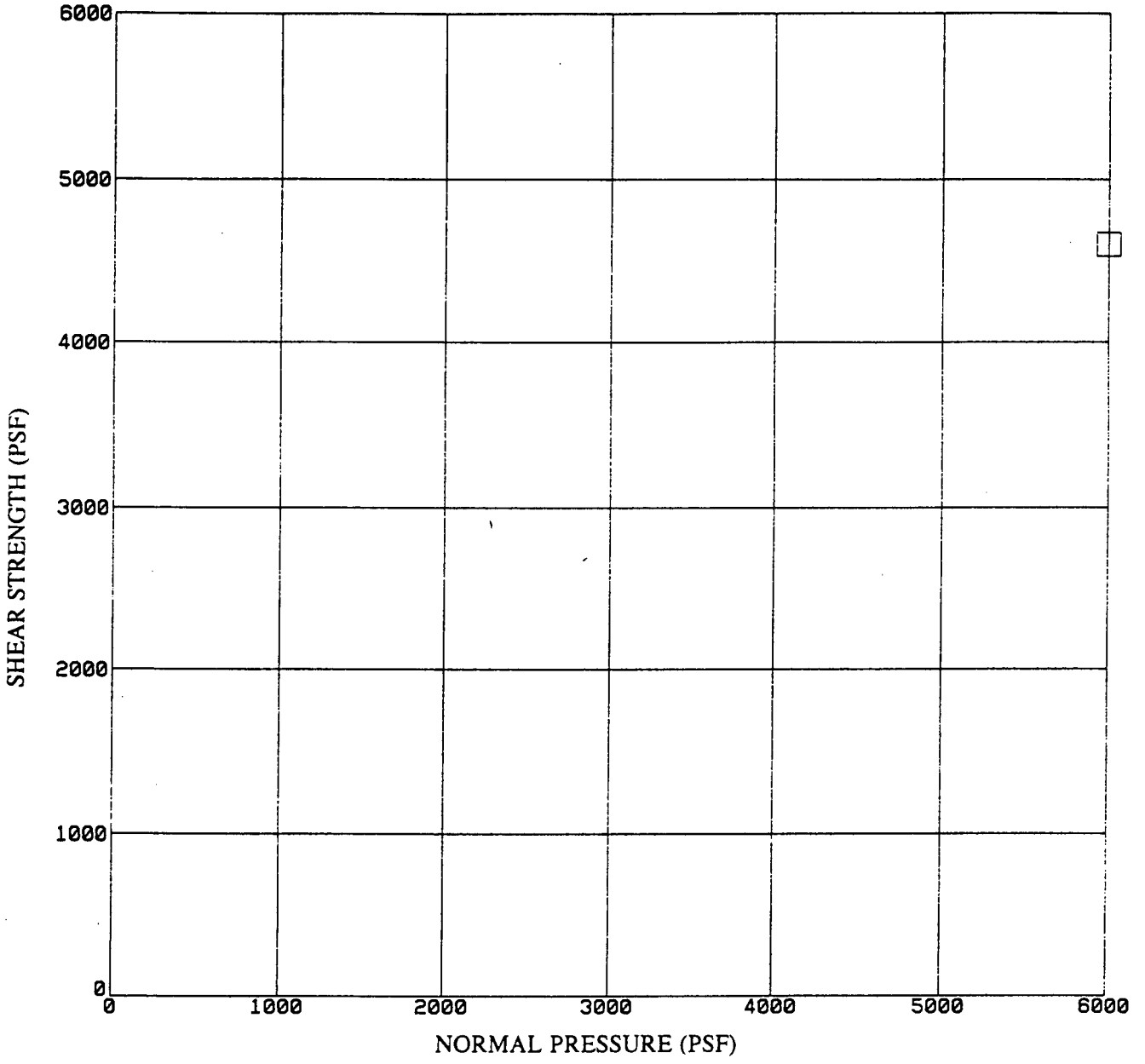
Plate

DIRECT SHEAR TEST PLOT OF B1A @ 20 FT. (SATURARTED)



SHEAR TEST DIAGRAM

Client: **Rancho Las Flores Limited Partnership**		W.O. **215-1**
Project: **Highway 138**	Location: **San Bernardino County**	



Initial Sample

Dry Density = 124.5 pcf
Moist. Cont. = 4.7%

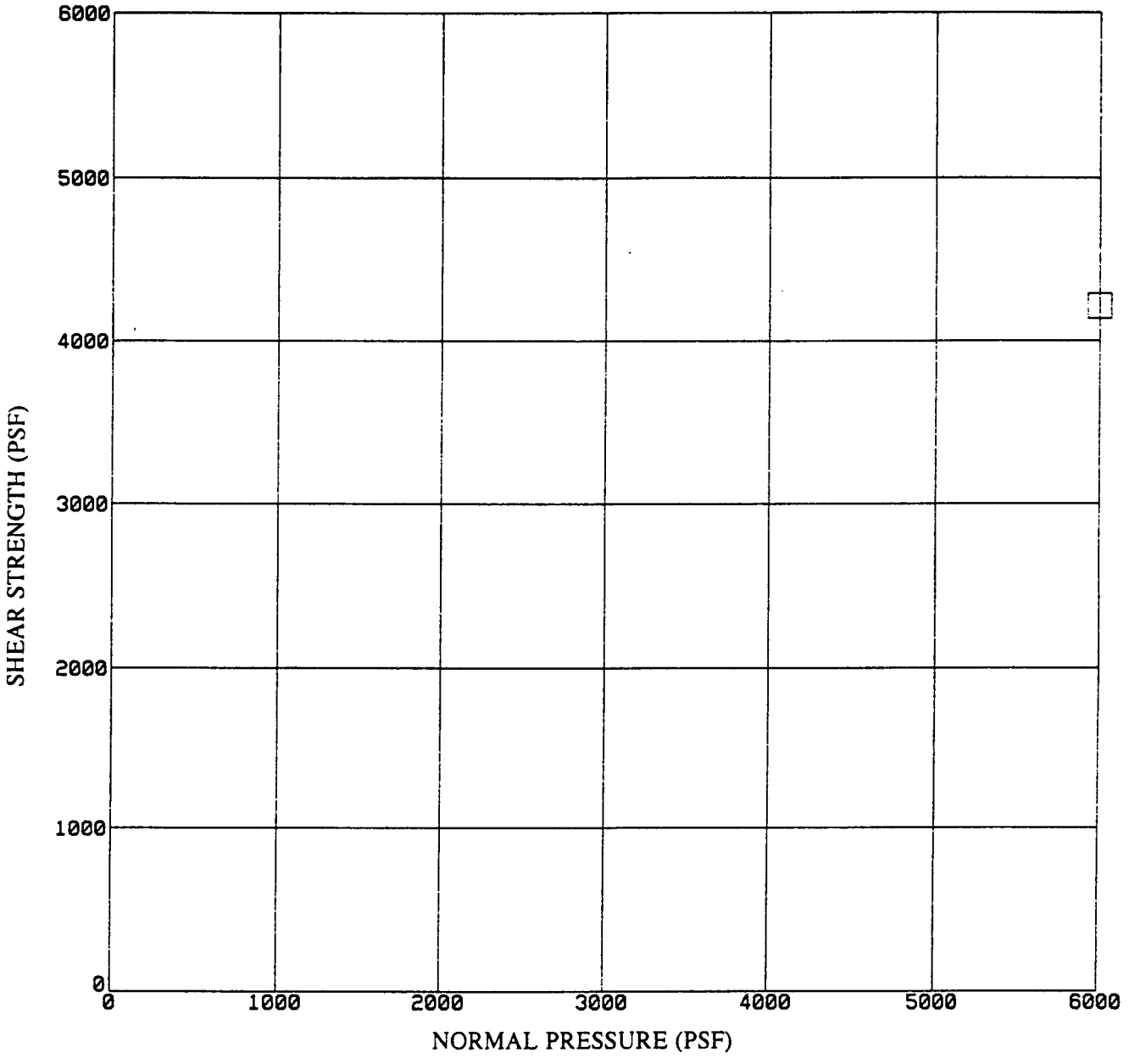
Sample: **B-1A @ 40.0 ft.**

Phi Angle: **deg.**
Cohesion: **psf**

TEST CONDITION: Undisturbed, tested at saturation

SHEAR TEST DIAGRAM

Client: **Rancho Las Flores Limited Partnership**	W.O. **215-1**
Project: **Highway 138**	Location: **San Bernardino County**



Initial Sample

Dry Density = 129.0 pcf
Moist. Cont. = 8.5%

Sample: **B-2A @ 15.0 ft.**

Phi Angle: **deg.**
Cohesion: **psf**

TEST CONDITION: Undisturbed, tested at saturation

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Plate

SHEAR TEST SUMMARY

Remolded to 90% Relative Compaction
at Optimum Moisture
Tested at Optimum Moisture

Sample Location	Shear Strength (psf)	Normal Pressure (psf) 2000	4000	6000
B-1A @ 10'		3000	4392	5520
B-1A @ 20'		3060	3528	5352
Median (Average) (1)		3030	3960	5436

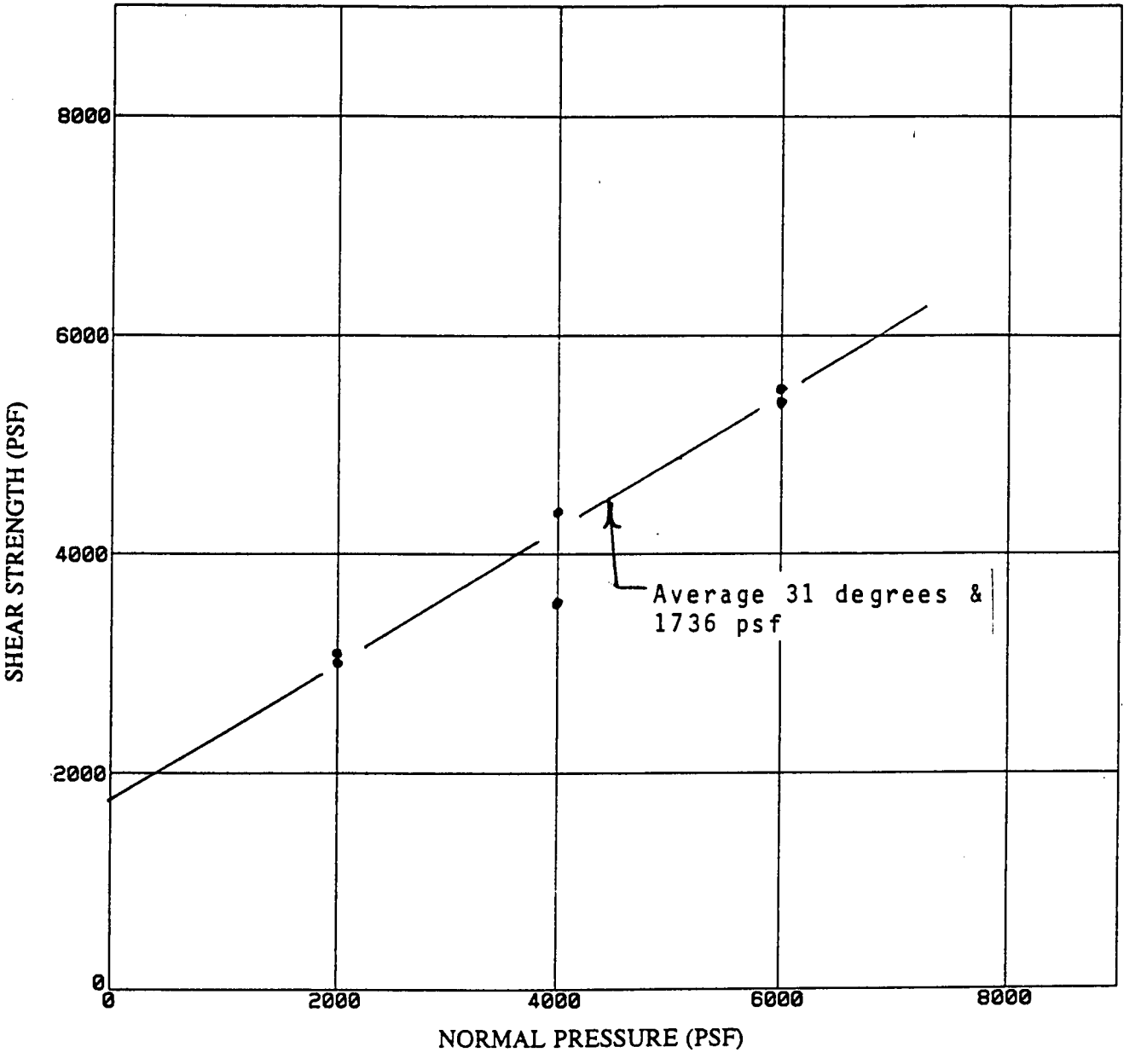
Median (Average) (2) = 31 degrees and 1736 psf

(1) Calculated as a mathematical average
(2) Calculated by computer

NOTE: These values were not used in calculations
See remolded and saturated Shear Test Data (Appendix E)

SHEAR TEST DIAGRAM

Client: Rancho Las Flores Limited Partnership	W.O. 215-1
Project: Highway 138	Location: San Bernardino County

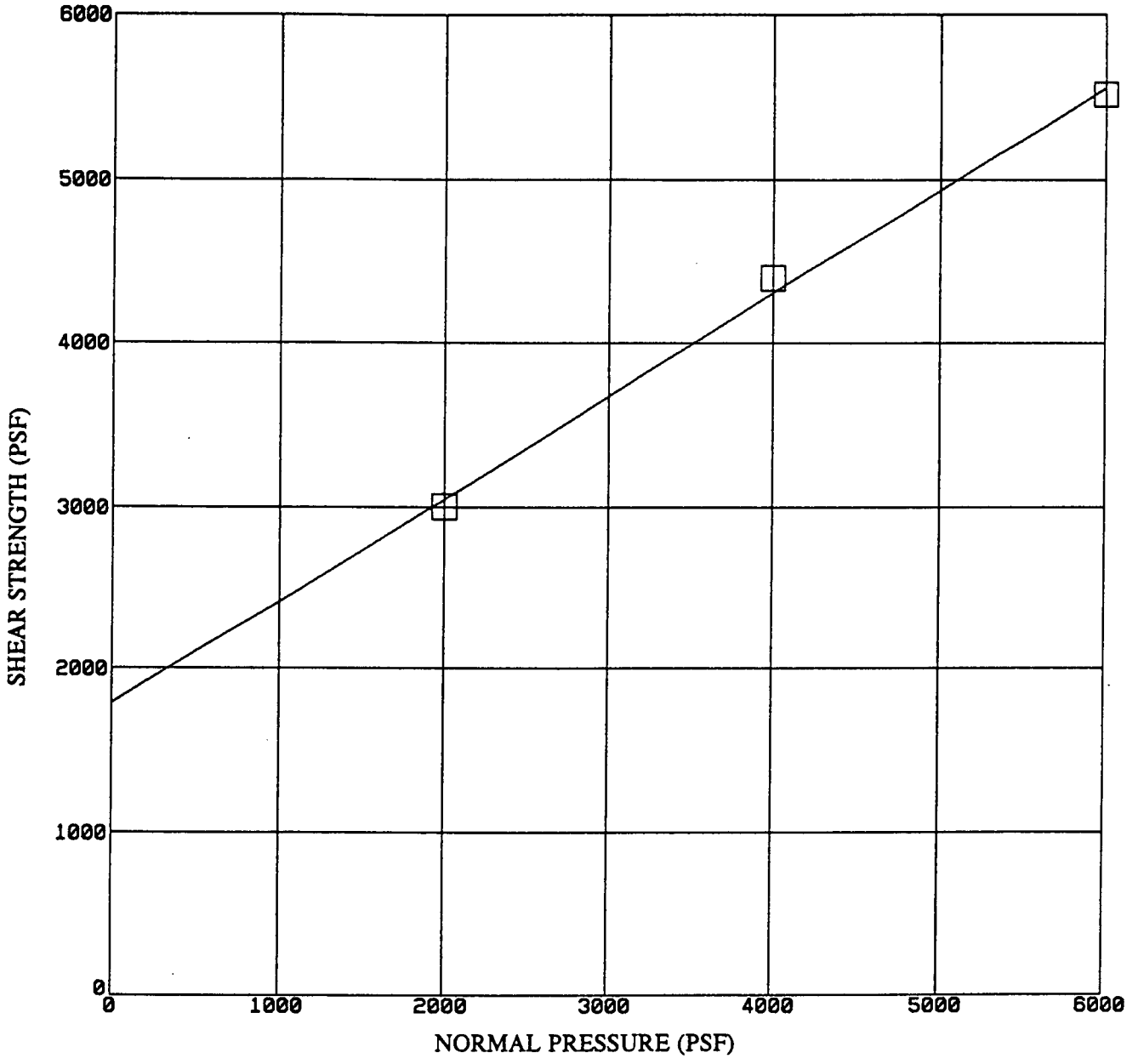


Shear Test Summary
Remolded to 90% R.C.
at Optimum Moisture
Sheared at Optimum

NOTE: These values were not used in calculations
See Remolded and saturated Shear Test Data
(Appendix E)

SHEAR TEST DIAGRAM

Client: Rancho Las Flores Limited Partnership	W.O. 215-1
Project: Highway 138	Location: San Bernardino County



Initial Sample

Dry Density = 115.7 pcf
Moist. Cont. = 9.5%

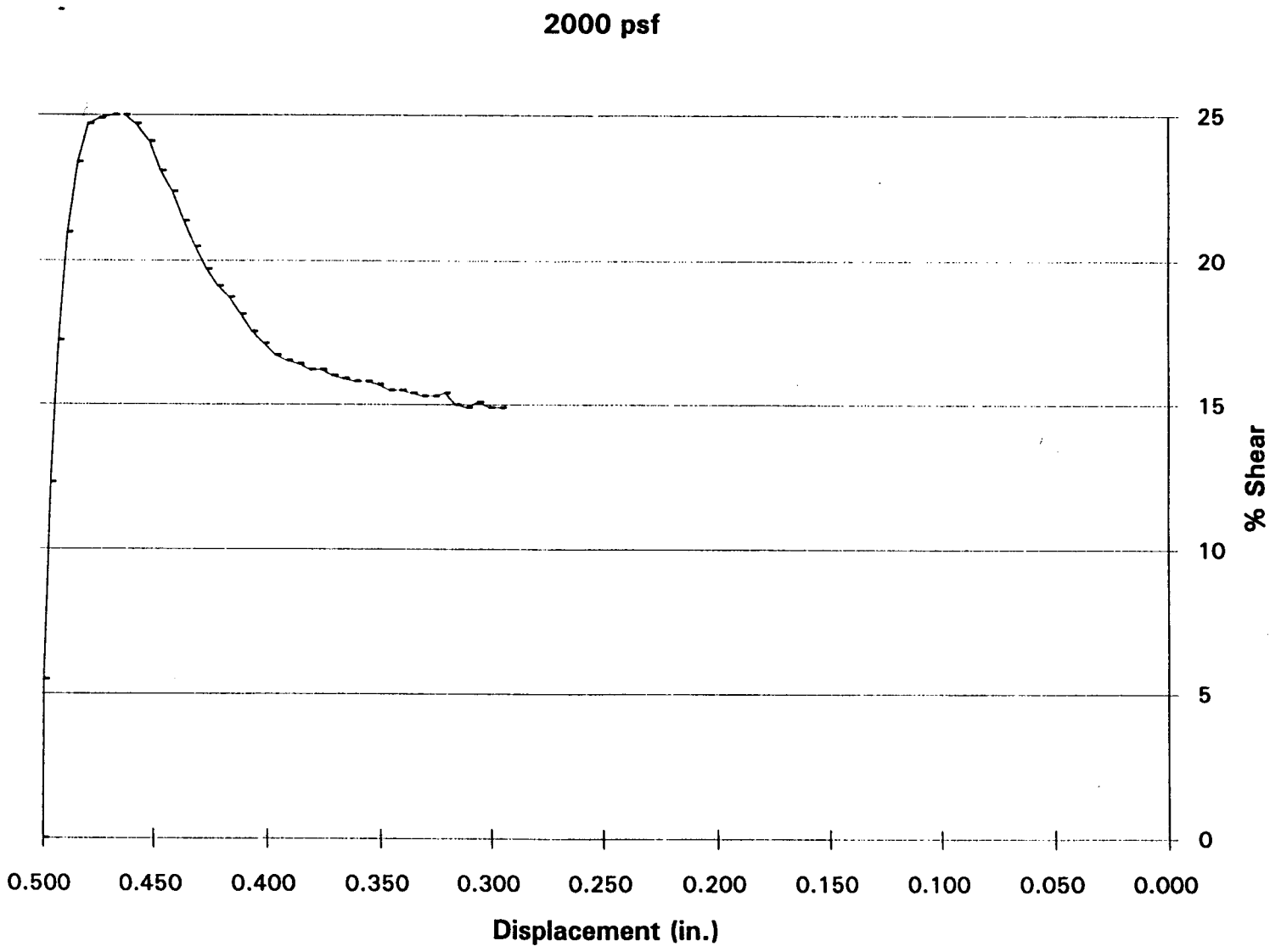
Sample: B-1A @ 10.0 ft.

Phi Angle: 32 deg.

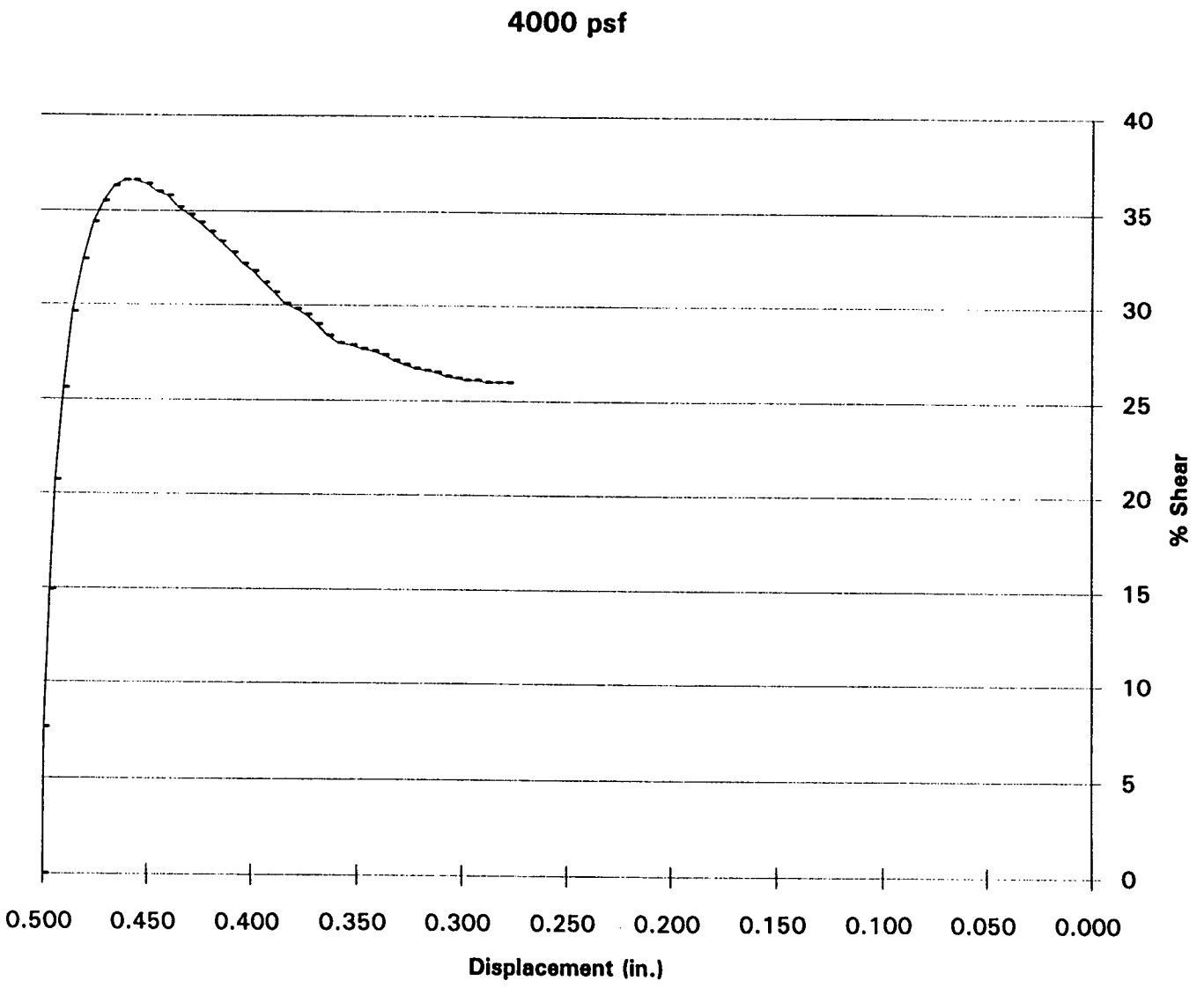
Cohesion: 1720 psf

TEST CONDITION: Remolded to 90% R.C. @ optimum moisture and tested at optimum

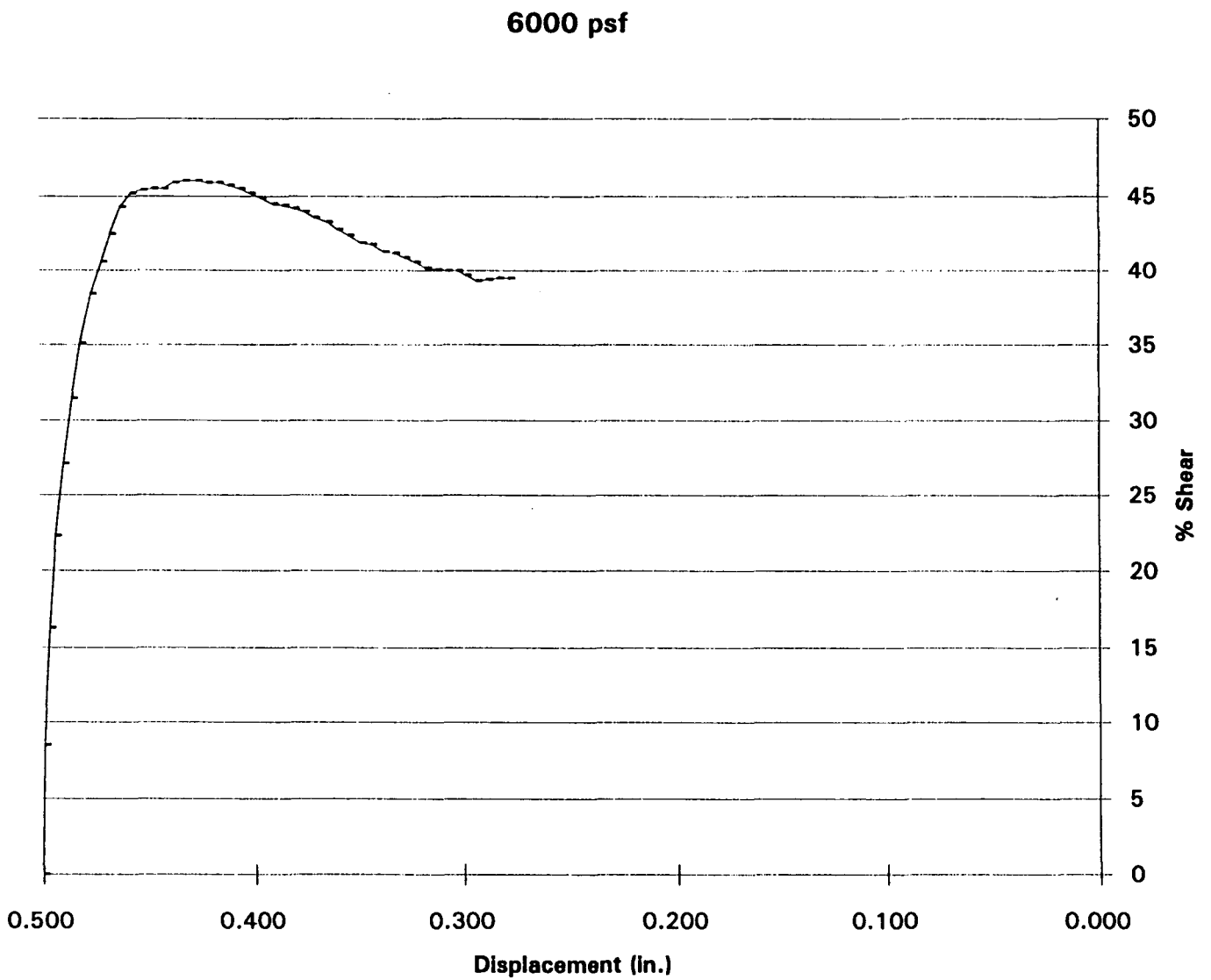
DIRECT SHEAR TEST PLOT OF B1A @ 10 FT. (REMOLDED)



DIRECT SHEAR TEST PLOT OF B1A @ 20 FT. (REMOLDED)

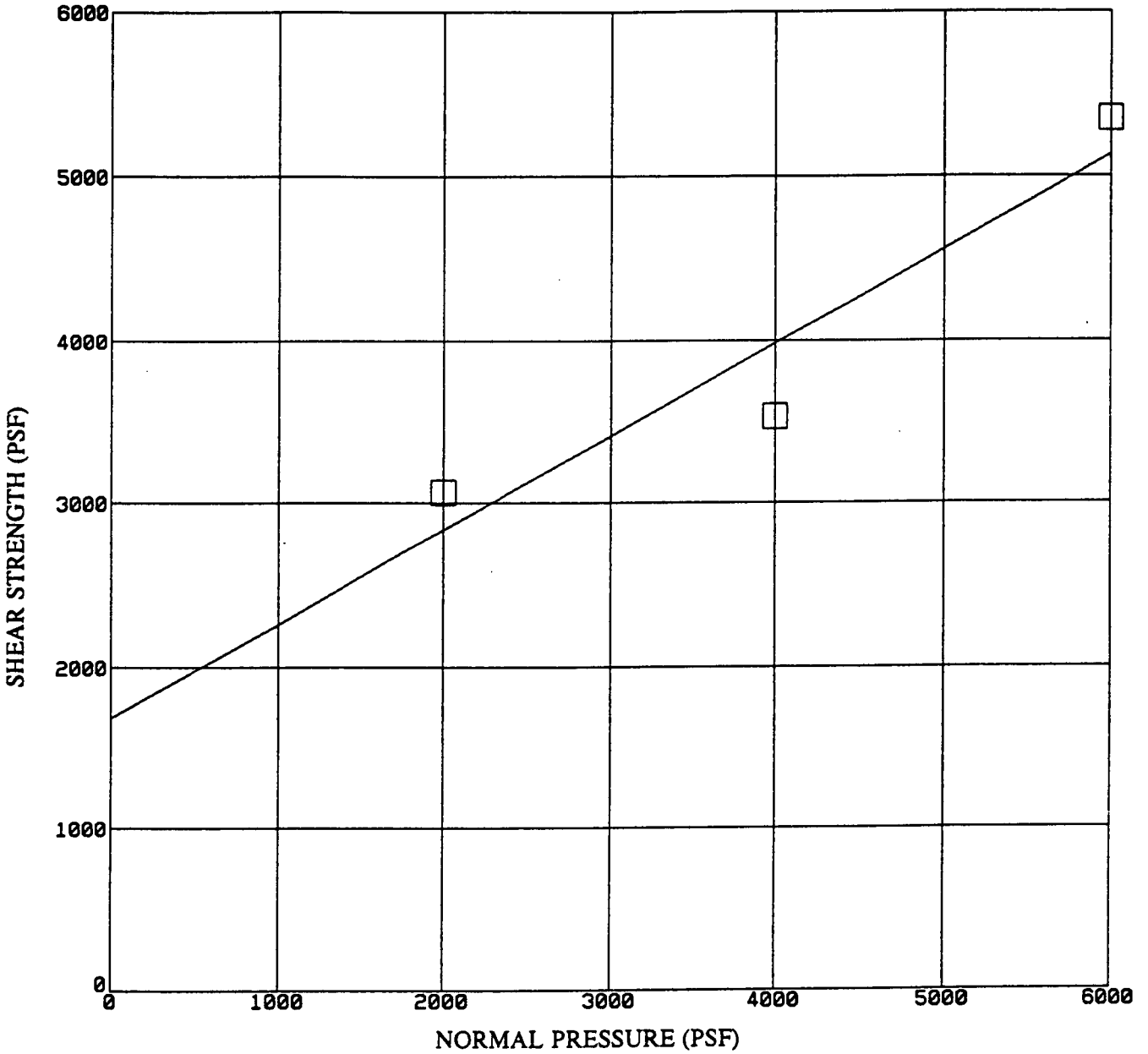


DIRECT SHEAR TEST PLOT OF B1A @ 10 FT. (REMOLDED)



SHEAR TEST DIAGRAM

Client: Rancho Las Flores Limited Partnership	W.O. 215-1
Project: Highway 138	Location: San Bernardino County



Initial Sample

Dry Density = 112.1 pcf
Moist. Cont. = 11.0%

Sample: B-1A @ 20.0 ft.

Phi Angle: 30 deg.

Cohesion: 1600 psf

TEST CONDITION: Remolded to 90% R.C. @ optimum moisture and tested at optimum

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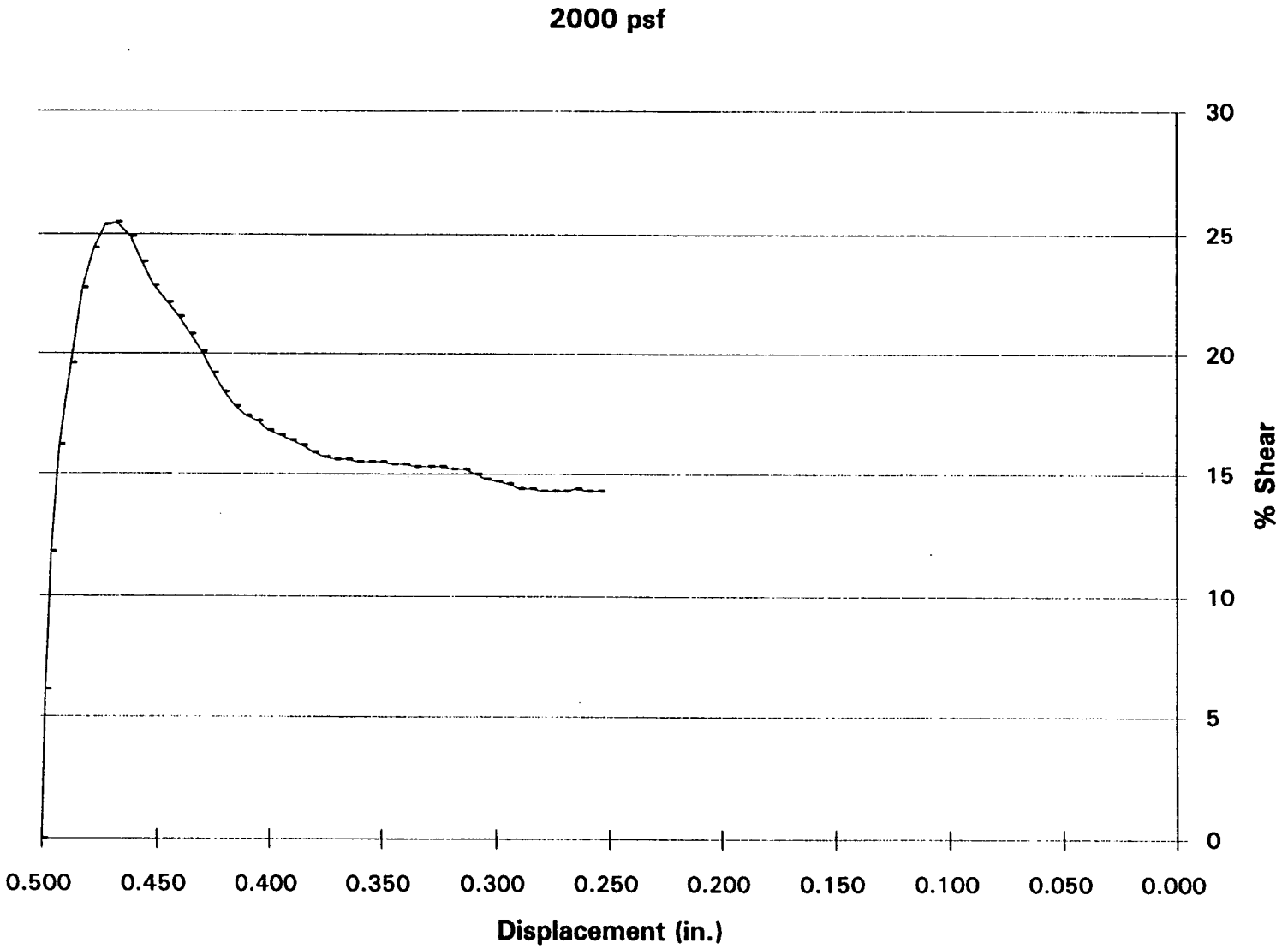
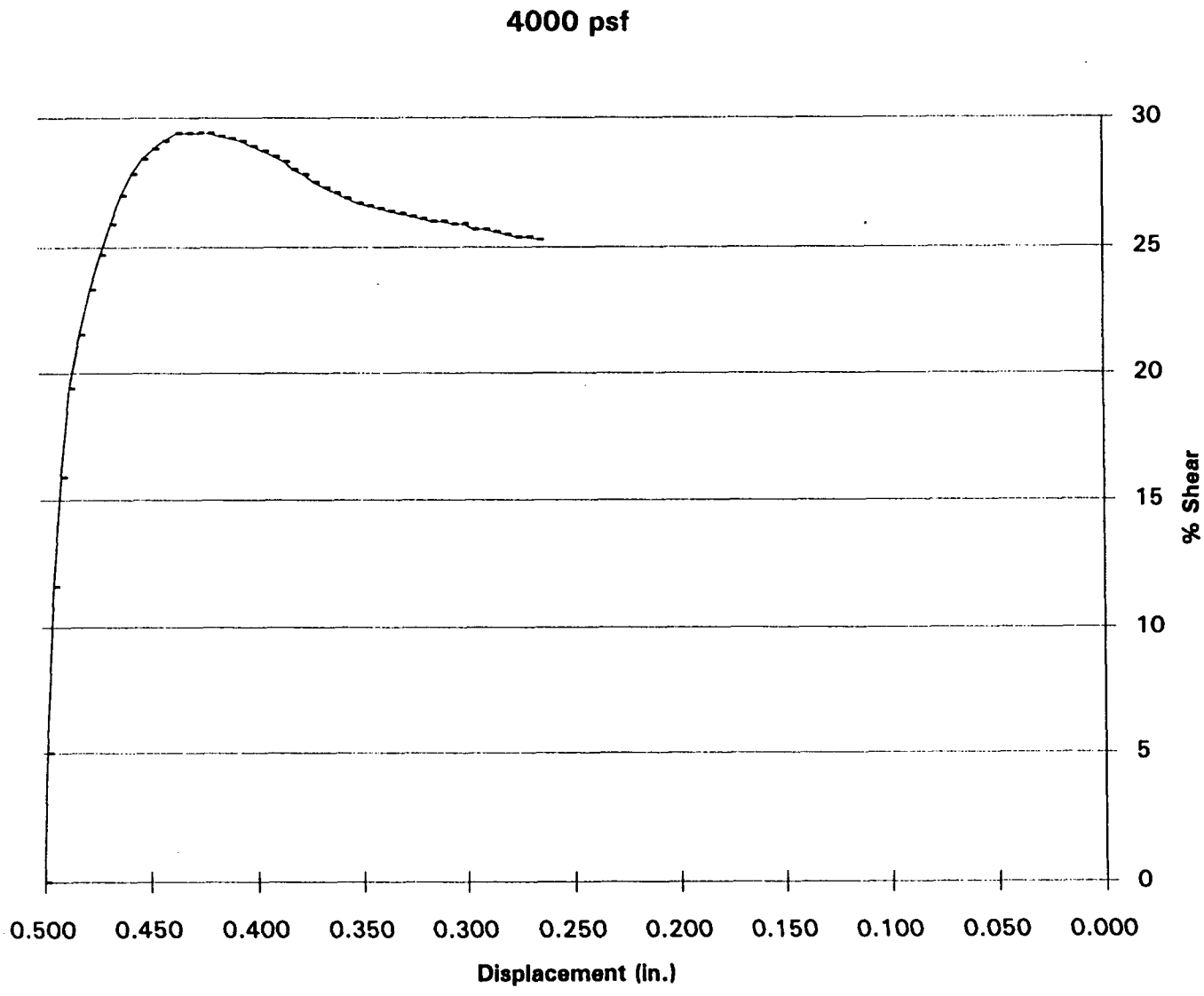
Plate

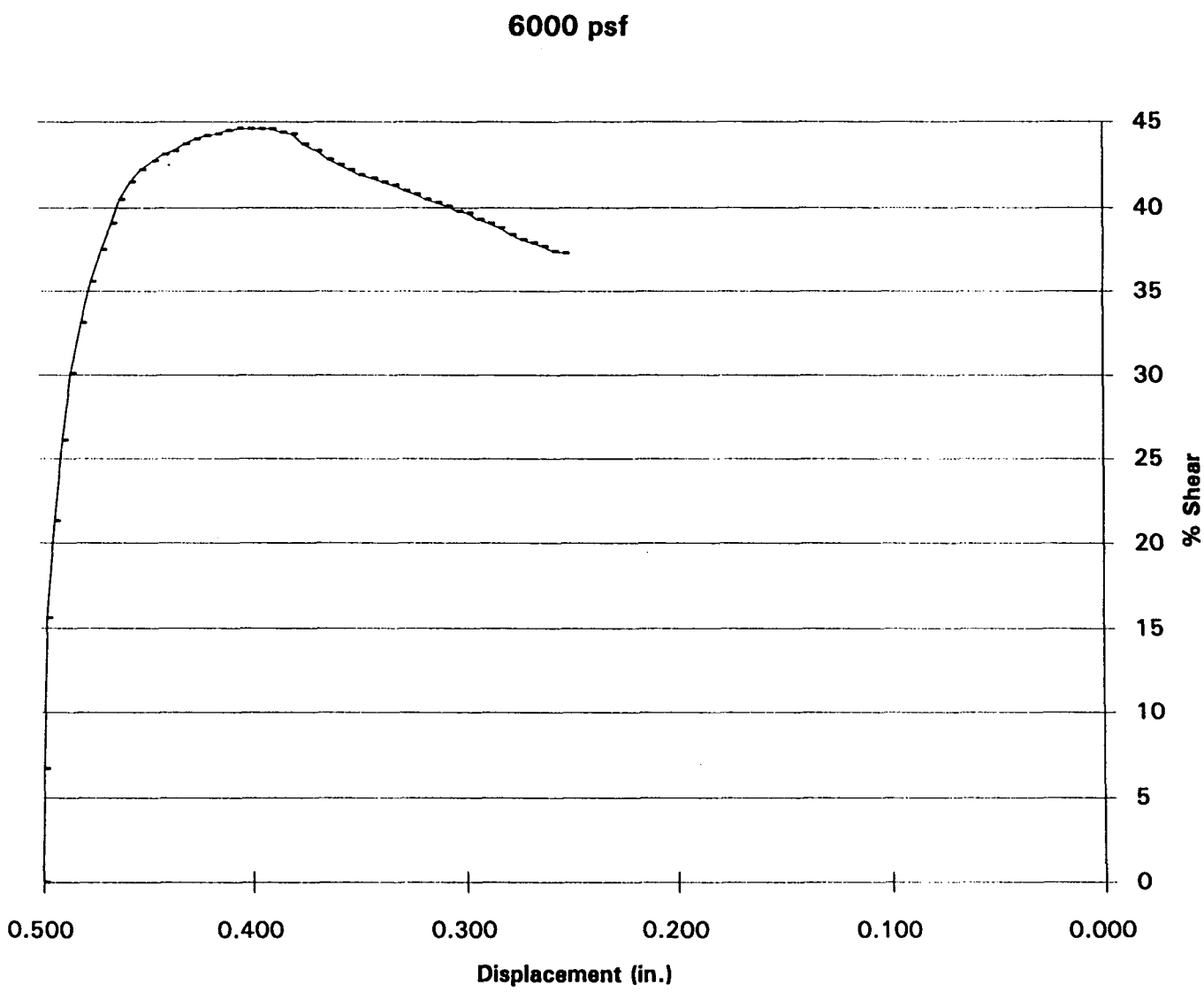
DIRECT SHEAR TEST PLOT OF B1A @ 20 FT. (REMOLDED)



DIRECT SHEAR TEST PLOT OF B1A @ 20 FT. (REMOLDED)



DIRECT SHEAR TEST PLOT OF B1A @ 20 FT. (REMOLDED)



SHEAR TEST SUMMARY

Remolded to 90% Relative Compaction
at Optimum Moisture
Test after Saturation

Sample Location	Shear Strength (psf)	Normal Pressure (psf) 2000	4000	6000
B-1A @ 10'		2004 (Low)	2676 (Low)	3828 (Low)
B-1A @ 20'		2232	3756	4668
Average		2118	3216	4248

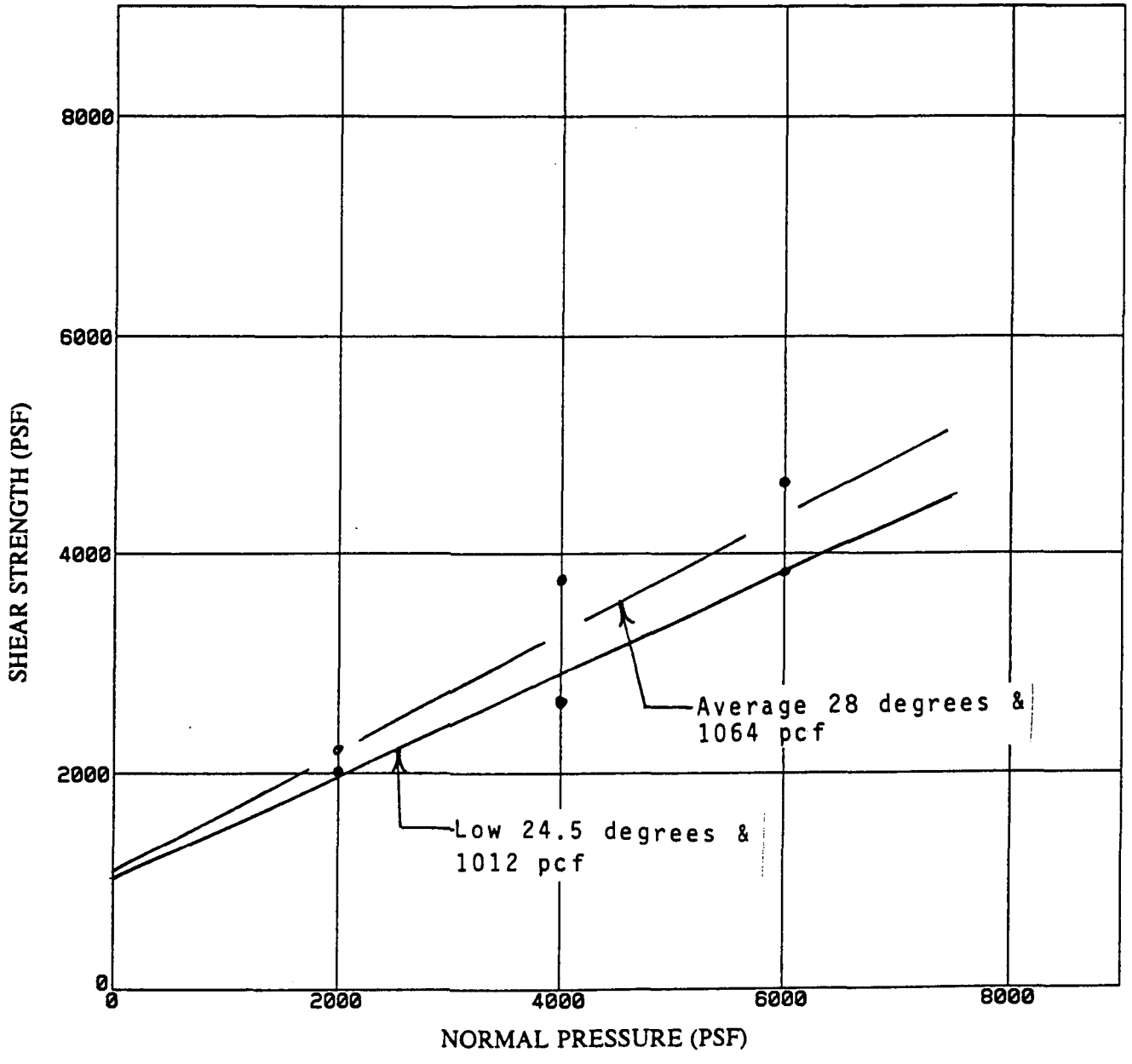
Low (1) 24.5 degrees and 1012 psf
Average (1) (2) 28 degrees and 1064 psf

(1) Calculated by computer
(2) Mathematical average

NOTE: Calculations used low value with half the cohesion.

SHEAR TEST DIAGRAM

Client: Rancho Las Flores Limited Partnership	W.O. 215-1
Project: Highway 138	Location: San Bernardino County

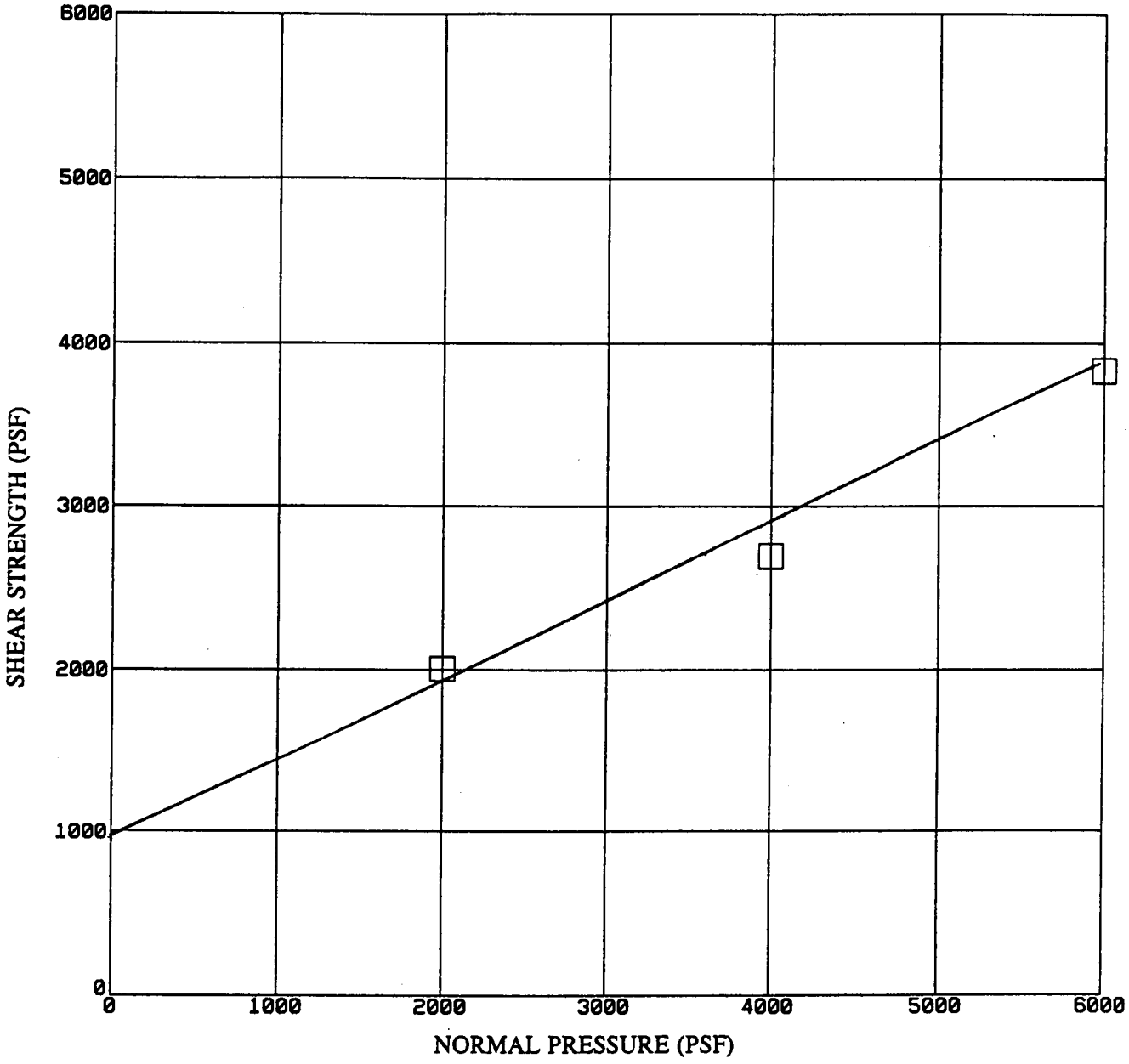


Shear Test Summary
Remolded to 90% R.C.
at Optimum Moisture
Sheared After Saturation

NOTE: Low value used in calculations with half cohesion

SHEAR TEST DIAGRAM

Client: **Rancho Las Flores Limited Partnership**	W.O. **215-1**
Project: **Highway 138**	Location: **San Bernardino County**



Initial Sample

Dry Density = 126.1 pcf
Moist. Cont.= 8.1%

Sample: **B-1A @ 10.0 ft.**

Phi Angle: **25 deg.**
Cohesion: **980 psf**

TEST CONDITION: Remolded to 90% R.C. @ optimum, saturated

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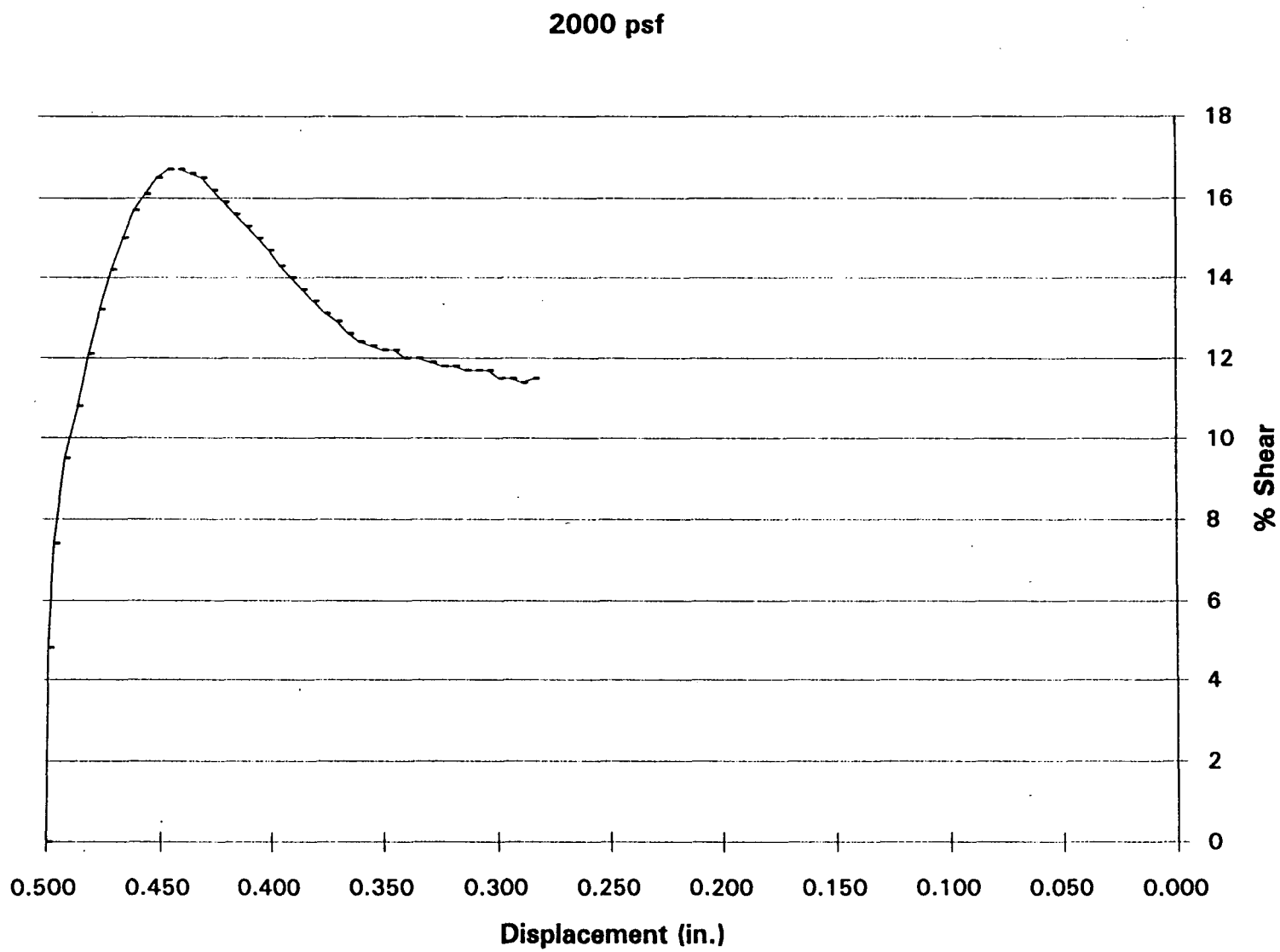
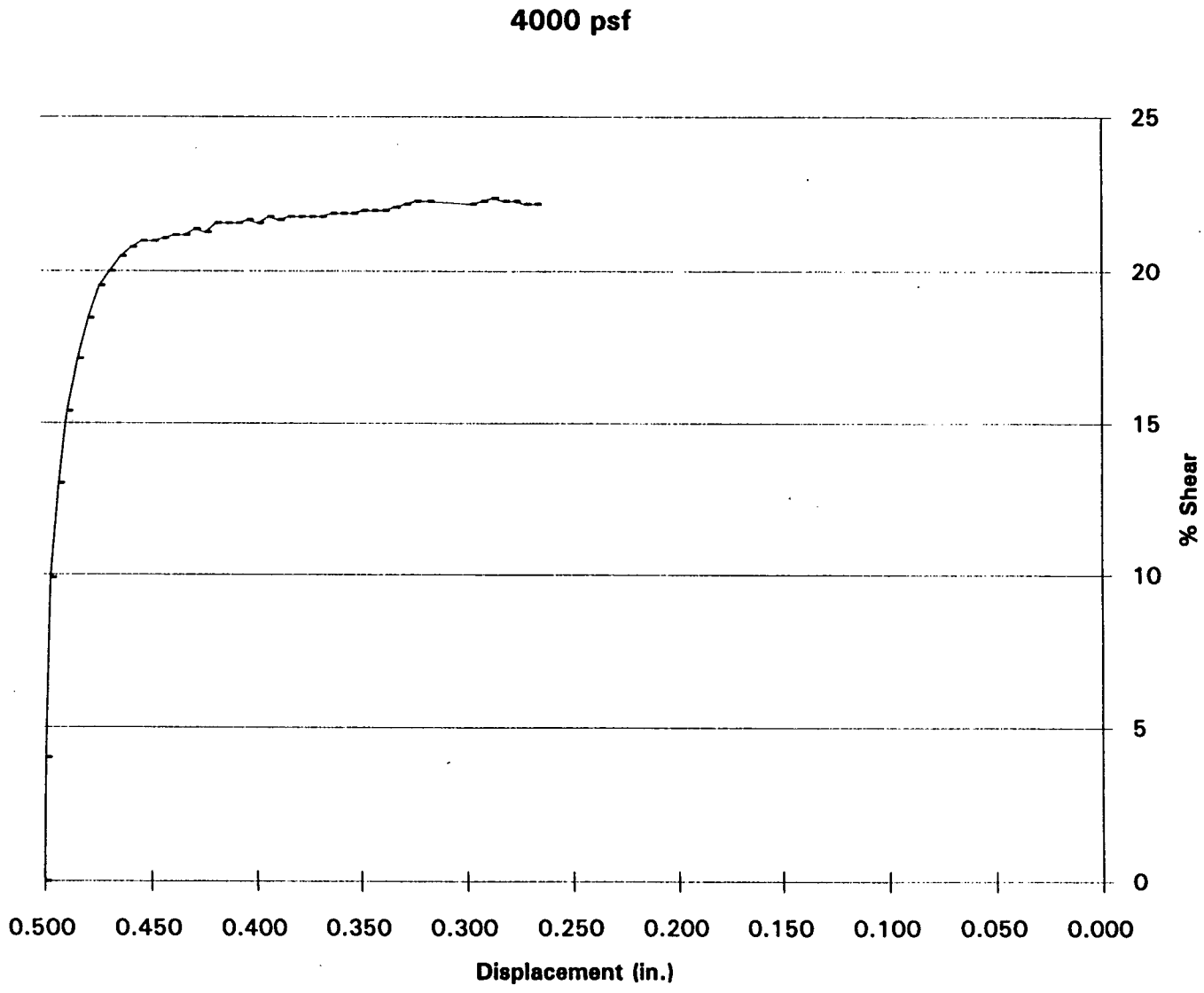
Plate

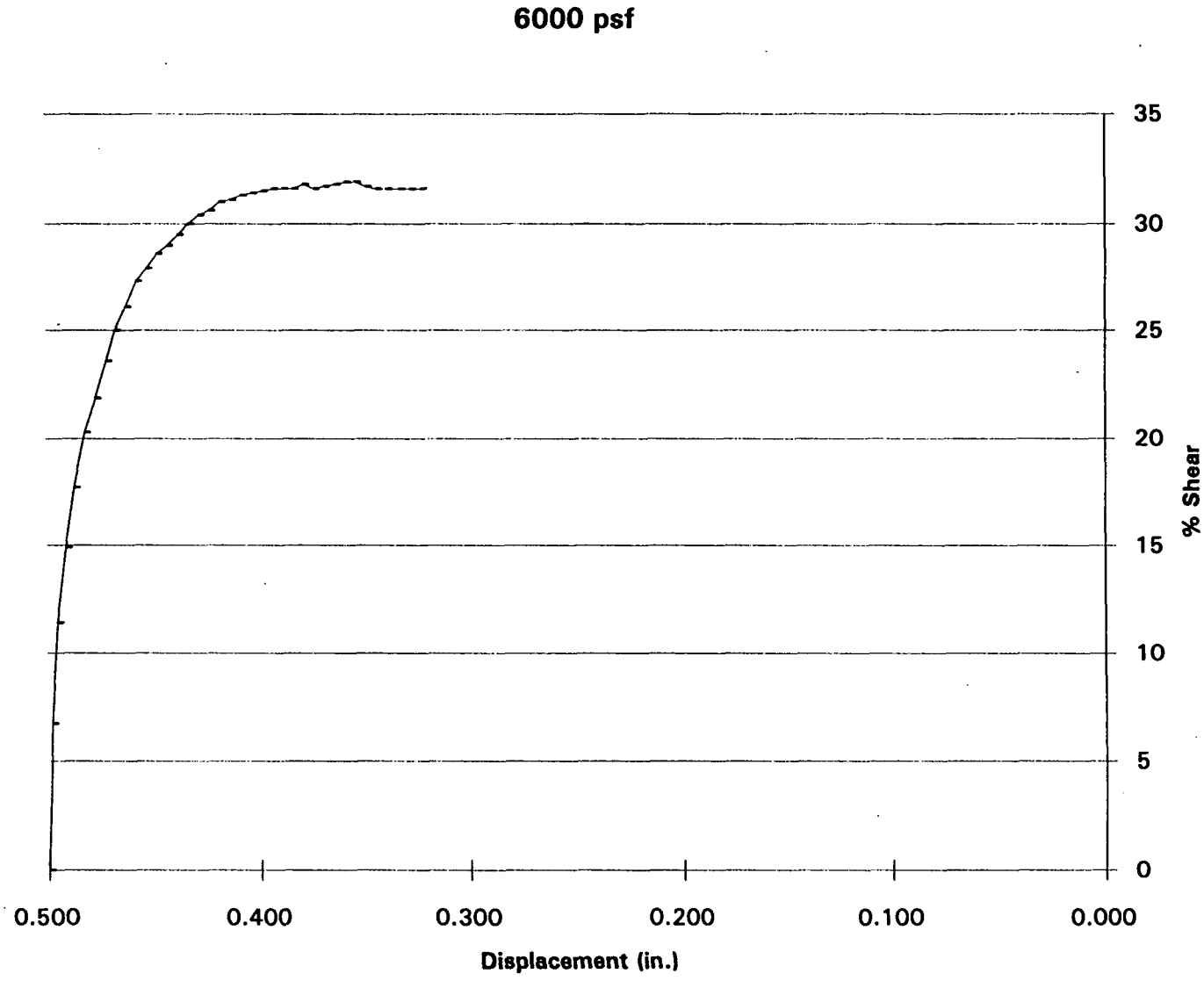
DIRECT SHEAR TEST PLOT OF B1A @ 10 FT. (REMOLDED AND SATURATED)



DIRECT SHEAR TEST PLOT OF B1A @ 10 FT. (REMOLDED AND SATURATED)

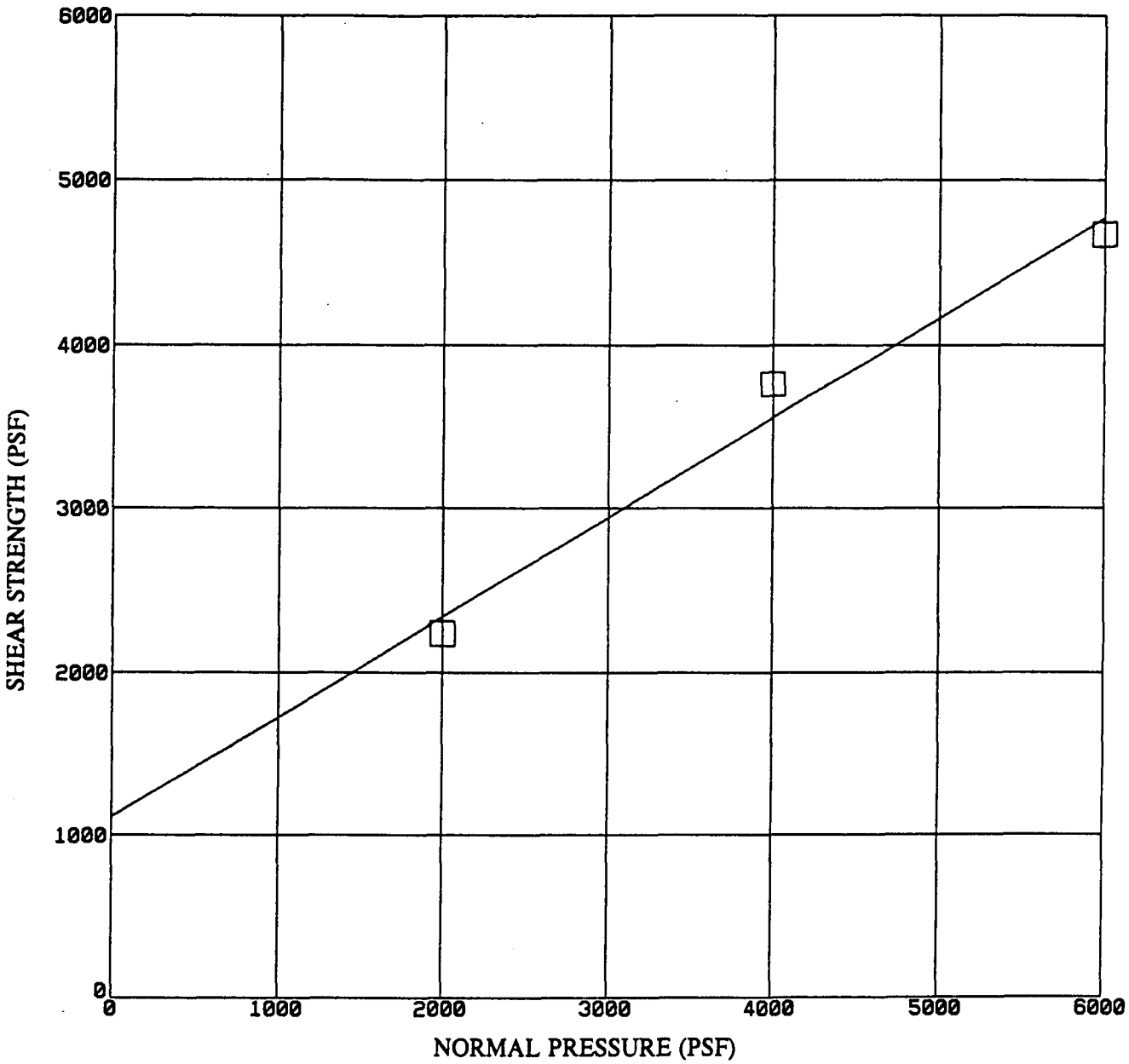


DIRECT SHEAR TEST PLOT OF B1A @ 10 FT. (REMOLDED AND SATURATED)



SHEAR TEST DIAGRAM

Client: Rancho Las Flores Limited Partnership	W.O. 215-1
Project: Highway 138	Location: San Bernardino County



Initial Sample

Dry Density = 127.4 pcf
Moist.Cont. = 10.4%

Sample: B-1A @ 20.0 ft.

Phi Angle: 30 deg.
Cohesion: 1220 psf

TEST CONDITION: Remolded to 90% R.C. @ optimum, saturated

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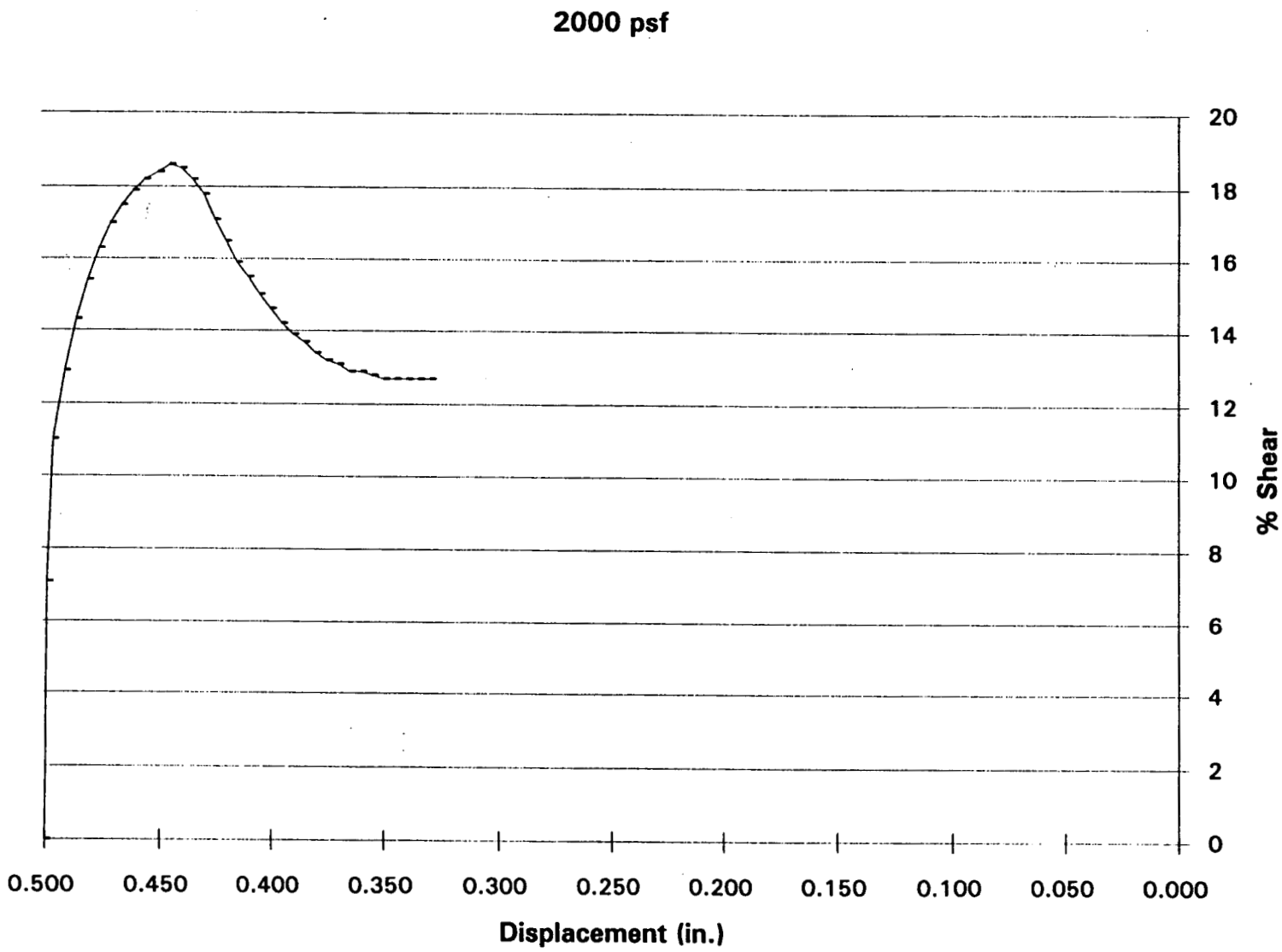
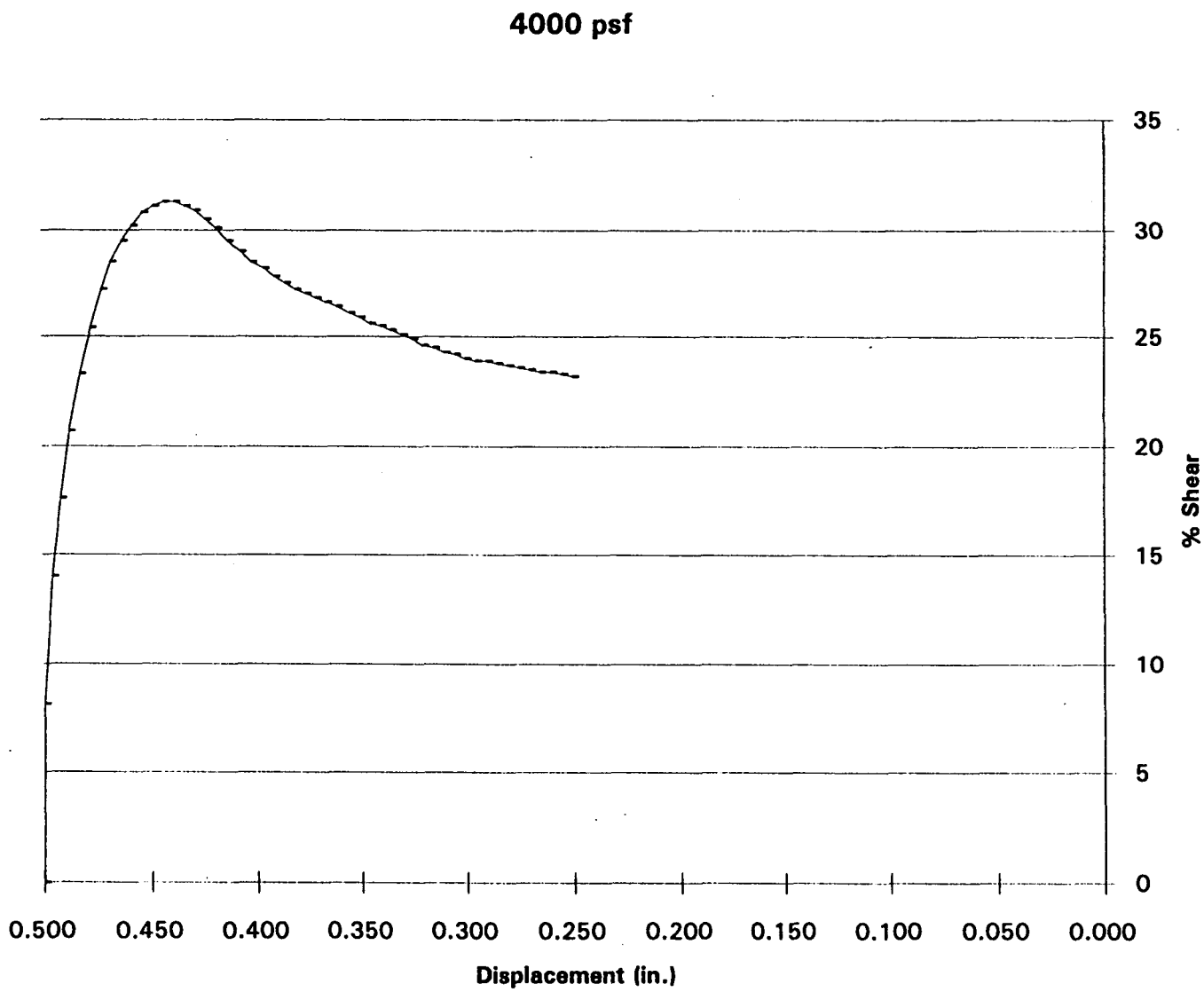
Plate

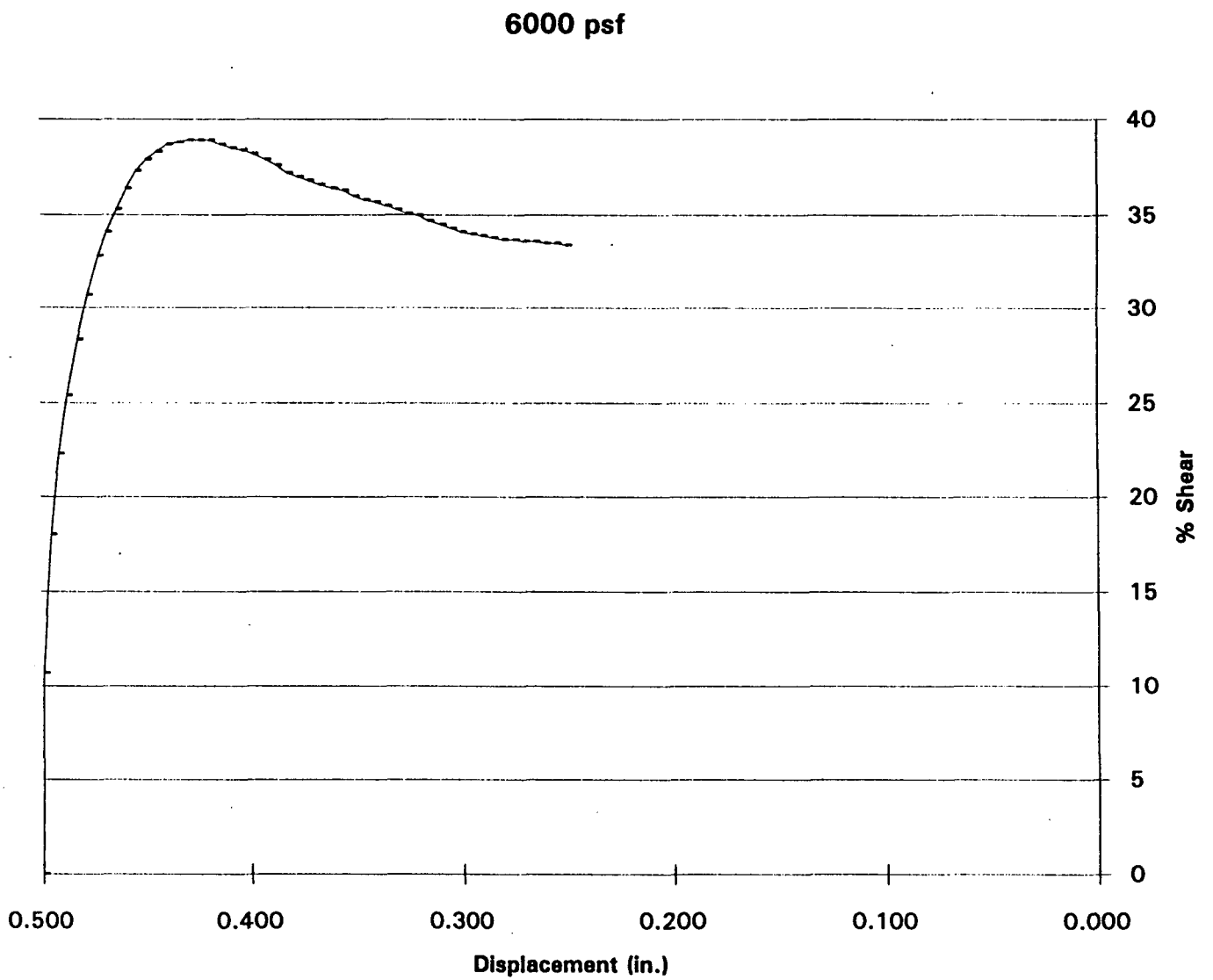
DIRECT SHEAR TEST PLOT OF B1A @ 20 FT. (REMOLDED AND SATURATED)



DIRECT SHEAR TEST PLOT OF B1A @ 20 FT. (REMOLDED AND SATURATED)



DIRECT SHEAR TEST PLOT OF B1A @ 20 FT. (REMOLDED AND SATURATED)



Legend for Test Pit Logs

Sample Type:

CORE SAMPLE

BULK SAMPLE

CORE AND BULK SAMPLES

Density terminology used for soil descriptions (e.g., medium dense, dense) reported in test pit logs has been based on observation and engineering experience.

UNIFIED SOIL CLASSIFICATION SYSTEM

Major Divisions			Symbols	Typical Names
COARSE GRAINED SOILS (More than 50% of material is LARGER than 200 sieve size.)	GRAVELS (More than 50% of coarse fraction is LARGER than No.4 sieve.)	CLEAN GRAVELS (Little or no fines.)	GW	Well-graded gravels, gravel-sand mixtures, little or no fines.
			GP	Poorly graded gravels or gravel-sand mixtures, little or no fines.
		GRAVELS W/FINES (Appreciable amount of fines.)	GM	Silty gravels, gravel-sand-silt mixtures.
			GC	Clayey gravels, gravel-sand-clay mixtures.
	SANDS (More than 50% of the coarse fraction SMALLER than the No.4 seive.)	CLEAN SANDS	SW	Well-graded sands, gravelly sands, little or no fines.
			SP	Poorly graded sands or gravelly sands, little or no fines.
		SANDS W/FINES (Appreciable amount of fines.)	SM	Silty sands, sand-silt mixtures.
			SC	Clayey sands, sand-clay mixtures.
FINE GRAINED SOILS (More than 50% of material is SMALLER than 200 sieve size.)	SILTS AND CLAYS (Liquid limit less than 50.)		ML	Inorganic silts & fine sands, rock flour, silty or clayey fine sands or clayey silts with slight plasticity
			CL	Inorganic clays of low to medium plasticity, gravelly clays, sandy clays, silty clays, lean clays
			OL	Organic silts and organic silty clays.
	SILTS AND CLAYS (Liquid limit more 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.

BOUNDARY CLASSIFICATIONS: Soils possessing characteristics of two groups are designated by combinations of group symbols.

JOHN A. SAYERS & ASSOCIATES

GEOTECHNICAL CONSULTANTS

Log of Exploratory Boring B-1

Client: A.R.C. Las Flores Ltd.	Date: 12-11-89	W.O. 215-01
Project: Highway 138	Location: San Bernardino	
	Surface Elevation: ±	
	Total Depth: 15.5	

FIELD					LAB DATA						
Water	Core	Bulk	Blows*	Depth (ft)	Moisture Content (%)	Dry Density (Pcf)	Other	USCS	Graphics	Depth (ft)	DESCRIPTION AND REMARKS
								SM			TOPSOIL: Silty Sand, brown, loose to medium dense, dry, with roots and fine gravel.
								SM			OLDER ALLUVIUM: Silty Sand, reddish brown, medium dense, moist, fine to medium grained, with a few fine to coarse gravel, with occasional coarse grains.
	■		3	5	1.4	113.3				5	
		■	8 9				SE=46 <200 8%				
▽	■		8 9 17		12.7	114.6	<200 8%	SM SP			Silty Sand/Sand, reddish brown, medium dense, wet to saturated, with gravel. @8.4' - Groundwater observed, brown, medium dense.
	■		3 8 10	10	10.9	125.3		SM		10	CROWDER FORMATION: Silty Sand, reddish brown, very dense, moist to wet, with some clay and gravel.
	▩		50	15			<200 20%			15	

DESCRIPTION AND REMARKS

This log is a representation of conditions at the time & place of drilling. With the passage of time or at any other location there may be consequential changes in conditions.

Total Depth 15.5 feet
Groundwater at 8' 4"
Caving at 3 feet
Hole Backfilled

See Legend for Important Notes | *Driving Weight: CME-75, Automatic Hammer System

JOHN A. SAYERS & ASSOCIATES
GEOTECHNICAL CONSULTANTS

Boring B-1
Sheet 1 of 1

Log of Exploratory Boring B-2

Client: A.R.C. Las Flores Ltd.	Date: 12-11-89	W.O. 215-01
Project: Highway 138	Location: San Bernardino	
	Surface Elevation: ±	
	Total Depth: 15.5	

FIELD					LAB DATA						
Water	Core	Bulk	Blows*	Depth (ft)	Moisture Content (%)	Dry Density (Pcf)	Other	USCS	Graphics	Depth (ft)	DESCRIPTION AND REMARKS
								SM			TOPSOIL: Silty Sand, brown, loose, dry.
								SM			OLDER ALLUVIUM: Silty Sand, brown, medium dense, dry to damp, with trace of fine gravel.
	■		8 10 23		4.8	106.9	SE=37 <200 25%				
	■		5 10 11	5	1.3	121.6		SM SP		5	Silty Sand/Sand, light to medium brown, medium dense to dense, damp, with gravel.
	■		5 7 14		1.7	107.5	<200 5%				
	▩		28 50	10						10	
▽											@14' - Groundwater encountered.
	▩		7 28 29	15				SM SP		15	CROWDER FORMATION: Silty Sand with Some Clay, reddish brown, very dense, moist to wet.

Total Depth 15.5 feet
Groundwater at 14 feet
Caving at 6 feet
Hole Backfilled

DESCRIPTION AND REMARKS

This log is a representation of conditions at the time & place of drilling. With the passage of time or at any other location there may be consequential changes in conditions.

e Legend for Important Notes | *Driving Weight: CME-75 Automatic Hammer System

JOHN A. SAYERS & ASSOCIATES
GEOTECHNICAL CONSULTANTS

Boring B-2
Sheet 1 of 1

Log of Exploratory Boring B-3

Client: A.R.C. Las Flores Ltd.	Date: 12-11-89	W.O. 215-01
Project: Highway 138	Location: San Bernardino	
	Surface Elevation: ±	
	Total Depth: 16.5	

FIELD					LAB DATA					
Water	Core	Bulk	Blows*	Depth (ft)	Moisture Content (%)	Dry Density (Pcf)	Other	USCS	Graphics	Depth (ft)
			20 22 23					GW		
			10 22 23	5	3.2	106.2		SM		5
			5 10 15							
			44 40 43	10	1.4	106.2				10
			11 20 25	15						15

DESCRIPTION AND REMARKS

This log is a representation of conditions at the time & place of drilling. With the passage of time or at any other location there may be consequential changes in conditions.

ALLUVIUM: Sandy Gravel, light to medium brown, dry to damp, loose to dense, cobbles and boulders at surface.

OLDER ALLUVIUM: Silty Sand, reddish to light brown and gray, medium dense to dense, dry to slightly damp.

Total Depth 16.5 feet
No Groundwater
No Caving
Hole Backfilled

See Legend for Important Notes | *Driving Weight:

JOHN A. SAYERS & ASSOCIATES
GEOTECHNICAL CONSULTANTS

Boring B-3
Sheet 1 of 1

Log of Exploratory Boring B-4

Client: A.R.C. Las Flores Ltd.	Date: 12-11-89	W.O. 215-01
Project: Highway 138	Location: San Bernardino	
	Surface Elevation: ±	
	Total Depth: 25.5	

FIELD					LAB DATA						
Water	Core	Bulk	Blows*	Depth (ft)	Moisture Content (%)	Dry Density (Pcf)	Other	USCS	Graphics	Depth (ft)	DESCRIPTION AND REMARKS
								SM			**ROADFILL:** Silty Sand with Gravel, brown, medium dense to dense, dry to damp, minor roots, gravel and cobbles encountered at 3.5 feet.
	■		7 17 44		2.6	126.9					
				5	2.0					5	
	■		10 11 11								
	▩		5 8 18					SP SM			**WASH DEPOSITS:** Sand/Silty Sand, light brown, dense, dry to damp, some gravel.
	■		22 29 40	10	1.0	114.9				10	
								SM			**OLDER ALLUVIUM:** Silty Sand with Gravel, light brown, very dense, dry to damp.
	▩		15 27 28	15						15	
	■		21 33 36	20	3.0	121.8				20	
	■		50	25	2.8					25	

Total Depth 25.5 feet
Caving at 8 to 11 feet
No Groundwater
Hole Backfilled

See Legend for Important Notes | *Driving Weight:

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Boring B-4
Sheet 1 of 1

Log of Exploratory Boring B-5

Client: A.R.C. Las Flores Ltd.	Date: 12-11-89	W.O. 215-01
Project: Highway 138	Location: San Bernardino	
	Surface Elevation: ±	
	Total Depth: 16.5	

FIELD					LAB DATA						
Water	Core	Bulk	Blows*	Depth (ft)	Moisture Content (%)	Dry Density (Pcf)	Other	USCS	Graphics	Depth (ft)	DESCRIPTION AND REMARKS
								SM SP			TOPSOIL: 2" to 3" thick OLDER ALLUVIUM: Silty Sand/Sand, brown, loose to medium dense, dry.
	■		7 8 8		0.8	104.3					
	■		5 7 8	5	2.0	107.8				5	@5' - dry to damp, dense.
	■		10 19 19		2.9	110.6		SM			Silty Sand, brown, medium dense, damp, with gravel, dry to damp.
	■		9 16 18	10	2.9	113.7				10	
	▒		7 11 12	15						15	

Total Depth 16.5 feet
No Groundwater
No Caving
Hole Backfilled

This log is a representation of conditions at the time & place of drilling. With the passage of time or at any other location there may be consequential changes in conditions.

See Legend for Important Notes | *Driving Weight:

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Boring B-5
Sheet 1 of 1

Log of Exploratory Boring B-6

Client: A.R.C. Las Flores Ltd.	Date: 12-11-89	W.O. 215-01
Project: Highway 138	Location: San Bernardino	
	Surface Elevation: ±	
	Total Depth: 15.0	

FIELD					LAB DATA						
Water	Core	Bulk	Blows*	Depth (ft)	Moisture Content (%)	Dry Density (Pcf)	Other	USCS	Graphics	Depth (ft)	DESCRIPTION AND REMARKS
								SM			TOPSOIL: Silty Sand, medium brown, loose, dry.
								SP SM			OLDER ALLUVIUM: Sand/Silty Sand, brown, medium dense, damp, with gravel.
	■		10 12 14		1.3	106.1					
	■		10 18 25	5	1.3	117.9				5	@5' - dense
	■		17 19 19		1.1	115.1					
	▩		14 22 24	10				SM		10	Silty Sand, brown, dense, damp to moist, with gravel.
	▩		10 18 26	15						15	Total Depth 15 feet No Groundwater Hole Backfilled

This log is a representation of conditions at the time & place of drilling. With the passage of time or at any other location there may be consequential changes in conditions.

See Legend for Important Notes | *Driving Weight:

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Boring B-6
Sheet 1 of 1

Log of Exploratory Boring B-7

Client: A.R.C. Las Flores Ltd.	Date: 12-11-89	W.O. 215-01
Project: Highway 138	Location: San Bernardino	
	Surface Elevation: ±	
	Total Depth: 15.0	

FIELD					LAB DATA						
Water	Core	Bulk	Blows*	Depth (ft)	Moisture Content (%)	Dry Density (Pcf)	Other	USCS	Graphics	Depth (ft)	DESCRIPTION AND REMARKS
								SM			WASH DEPOSIT: Silty Sand with Gravel, brown, loose, dry.
	■		64 87		5.0	129.2		SC			CROWDER FORMATION (WEATHERED): Clayey Sand, brown, damp, dense.
	■		15 39 67	5	11.3	123.5				5	@5' - less weathered, although still clayey.
	■		41 50/3"	10	8.0	124.3		SC		10	Silty Sand with Clay, some gravel, brown, very dense, moist.
	▩		18 33 51	15	13.4					15	Total Depth 15 feet No Groundwater No Caving Hole Backfilled

This log is a representation of conditions at the time & place of drilling. With the passage of time or at any other location there may be consequential changes in conditions.

See Legend for Important Notes | *Driving Weight:

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Boring B-7
Sheet 1 of 1

Log of Exploratory Boring B-8

Client: A.R.C. Las Flores Ltd.	Date:	W.O. 215-01
Project: Highway 138	Location: San Bernardino	
	Surface Elevation: ±	
	Total Depth: 1.0	

FIELD					LAB DATA						
Water	Core	Bulk	Blows*	Depth (ft)	Moisture Content (%)	Dry Density (Pcf)	Other	USCS	Graphics	Depth (ft)	DESCRIPTION AND REMARKS
											This log is a representation of conditions at the time & place of drilling. With the passage of time or at any other location there may be consequential changes in conditions.
											This boring was deleted due to lack of access.

See Legend for Important Notes | *Driving Weight:

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Boring B-8
Sheet 1 of 1

Log of Exploratory Boring B-9

Client: A.R.C. Las Flores Ltd.	Date: 12-11-89	W.O. 215-01
Project: Highway 138	Location: San Bernardino	
	Surface Elevation: ±	
	Total Depth: 21.0	

FIELD					LAB DATA						
Water	Core	Bulk	Blows*	Depth (ft)	Moisture Content (%)	Dry Density (Pcf)	Other	USCS	Graphics	Depth (ft)	DESCRIPTION AND REMARKS
								SM SP			WASH DEPOSITS: Sand/Silty Sand, light brown, loose, dry.
	■		12 21 27		1.0	116.4	<200 6%				@2.5' - becomes medium brown, medium dense, fine to medium grained.
	■		20 20 23	5	1.5	110.8				5	@5' - some gravel, dry to damp.
				10						10	
	■		72/6"	15	4.3	120.1	<200 40%	SM SC		15	OLDER ALLUVIUM (?): Silty to Clayey Sand, some gravel and cobbles, brown, very dense, damp.
	■		50/4"	20						20	
	▨		50/3"								Total Depth 21 feet No Groundwater Hole Backfilled

The Legend for Important Notes | *Driving Weight: 140 lbs/automatic

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Boring B-9
Sheet 1 of 1

This log is a representation of conditions at the time & place of drilling. With the passage of time or at any other location there may be consequential changes in conditions.

Log of Exploratory Boring B-10

Client: A.R.C. Las Flores Ltd.	Date: 12-12-89	W.O. 215-01
Project: Highway 138	Location: San Bernardino	
	Surface Elevation: ±	
	Total Depth: 15.0	

FIELD					LAB DATA						
Water	Core	Bulk	Blows*	Depth (ft)	Moisture Content (%)	Dry Density (Pcf)	Other	USCS	Graphics	Depth (ft)	DESCRIPTION AND REMARKS
								SP SM			WASH DEPOSITS: Sand/Silty Sand, light brown, loose, dry to slightly damp.
	■		2 5 32		2.2	120.5	<200 30%				@2.5' - some gravel and clay, loose to medium dense.
	■		36 58 59	5	1.6	121.3				5	
								SM SC			TERRACE DEPOSITS (?): Silty to Clayey Sand, medium to light brown, dense to very dense, damp.
	■		50 104	10	4.6	126.0	<200 49%			10	
				15						15	
	▨		15 30 42								Total Depth 15 feet No Groundwater Hole Backfilled

This log is a representation of conditions at the time & place of drilling. With the passage of time or at any other location there may be consequential changes in conditions.

See Legend for Important Notes | *Driving Weight: 140 lbs/automatic

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Boring B-10
Sheet 1 of 1

Log of Exploratory Boring B-11

Client: A.R.C. Las Flores Ltd.	Date: 12-12-89	W.O. 215-01
Project: Highway 138	Location: San Bernardino	
	Surface Elevation: ±	
	Total Depth: 10.0	

FIELD					LAB DATA						
Water	Core	Bulk	Blows*	Depth (ft)	Moisture Content (%)	Dry Density (Pcf)	Other	USCS	Graphics	Depth (ft)	DESCRIPTION AND REMARKS
								SC			FILL: Clayey Sand, brown, medium dense, dry.
								SM			CROWDER FORMATION (?): Silty Sand with Some Gravel, light brown, very dense, damp.
	■		35 60		2.7	120.6					
	■		33 30/4"	5	6.4	117.9	<200 53%	SM ML		5	@5' - fine grained, very silty
	▩		33 42 50/5"	10						10	@10' - Silty Fine Sand/Fine Sandy Silt, brown, very dense, damp, some clay.

This log is a representation of conditions at the time & place of drilling. With the passage of time or at any other location there may be consequential changes in conditions.

Total Depth 10 feet
No Groundwater
Hole Backfilled

See Legend for Important Notes | *Driving Weight: 140 lbs/automatic

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Boring B-11
Sheet 1 of 1

Log of Exploratory Boring B-12

Client: A.R.C. Las Flores Ltd.	Date: 12-12-89	W.O. 215-01
Project: Highway 138	Location: San Bernardino	
	Surface Elevation: ±	
	Total Depth: 20.0	

FIELD					LAB DATA							DESCRIPTION AND REMARKS
Water	Core	Bulk	Blows*	Depth (ft)	Moisture Content (%)	Dry Density (Pcf)	Other	USCS	Graphics	Depth (ft)		This log is a representation of conditions at the time & place of drilling. With the passage of time or at any other location there may be consequential changes in conditions.
								SM				FILL/WASH DEPOSITS: Silty Sand with Clay and Gravel, brown, loose to medium dense, dry to damp.
	■		12 11 14		3.7	104.7	<200 12%					
				5						5		
	■		7 12 18		1.9	109.0						
	■		8 11 12	10	7.9	113.1				10		@10' - becoming clayey
								SM ML				CROWDER FORMATIOM (?): Silty Sand/Sandy Silt with clay, light brown, very dense, damp, some gravel.
	▩		30 51	15			<200 51%			15		
	■		55/6"	20	5.4	120.3				20		Total Depth 20 feet No Groundwater Hole Backfilled

See Legend for Important Notes | *Driving Weight: 140 lbs./automatic

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Boring B-12
Sheet 1 of 1

Log of Exploratory Boring B-13

Client: A.R.C. Las Flores Ltd.	Date: 12-12-89	W.O. 215-01
Project: Highway 138	Location: San Bernardino	
	Surface Elevation: ±	
	Total Depth: 15.0	

FIELD					LAB DATA						
Water	Core	Bulk	Blows*	Depth (ft)	Moisture Content (%)	Dry Density (Pcf)	Other	USCS	Graphics	Depth (ft)	DESCRIPTION AND REMARKS
								SM SC			FILL: Silty to Clayey Sand with Gravel, brown, loose to medium dense, dry to damp.
	■		57 132	5	3.1	121.9	<200 32%	SM		5	CROWDER FORMATION: Silty Sand with Clay, light brown, very dense, damp, some small gravel.
	▩		40 38 50/5"	10			<200 27%			10	
	■		50/6"	15						15	Total Depth 15 feet No Groundwater Hole Backfilled

This log is a representation of conditions at the time & place of drilling. With the passage of time or at any other location there may be consequential changes in conditions.

See Legend for Important Notes | *Driving Weight: 140 lbs/automatic

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Boring B-13
Sheet 1 of 1

Log of Exploratory Boring B-14

Client: A.R.C. Las Flores Ltd.	Date: 12-12-89	W.O. 215-01
Project: Highway 138	Location: San Bernardino	
	Surface Elevation: ±	
	Total Depth: 10.0	

FIELD					LAB DATA						
Water	Core	Bulk	Blows*	Depth (ft)	Moisture Content (%)	Dry Density (Pcf)	Other	USCS	Graphics	Depth (ft)	DESCRIPTION AND REMARKS
								SM			FILL: Silty Sand with Gravel, reddish brown, medium dense, dry to damp.
								SP SM			TERRACE DEPOSIT: Sand/Silty Sand with Gravel, light reddish brown, very dense, damp.
	■		34 62		3.9	122.2	<200 4%				
	■		70/6	5	4.1	114.2				5	
	▩		28 58	10						10	Total Depth 10 feet No Groundwater Hole Backfilled

This log is a representation of conditions at the time & place of drilling. With the passage of time or at any other location there may be consequential changes in conditions.

See Legend for Important Notes | *Driving Weight: 140 lbs/automatic

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Boring B-14
Sheet 1 of 1

Log of Exploratory Boring B-15

Client: A.R.C. Las Flores Ltd.	Date: 12-12-89	W.O. 215-01
Project: Highway 138	Location: San Bernardino	
	Surface Elevation: ±	
	Total Depth: 35.0	

FIELD					LAB DATA						
Water	Core	Bulk	Blows*	Depth (ft)	Moisture Content (%)	Dry Density (Pcf)	Other	USCS	Graphics	Depth (ft)	DESCRIPTION AND REMARKS
								SM SC			ALLUVIUM: Silty/Clayey Sand, dark to medium brown, loose to medium dense, damp, scattered gravel.
	■		23 27 27		6.0	120.3	<200 30%				
	■		6 8 6	5	7.2	109.5				5	
	■		3 4 4	10	8.2	106.3	<200 43%			10	
	■		4 5 7	15	5.5	110.3				15	
	▩		2 3 4	20						20	

This log is a representation of conditions at the time & place of drilling. With the passage of time or at any other location there may be consequential changes in conditions.

See Legend for Important Notes | *Driving Weight: 140 lbs/automatic

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Boring B-15
Sheet 1 of 2

Log of Exploratory Boring B-15

Client: A.R.C. Las Flores Ltd.	Date: 12-12-89	W.O. 215-01
Project: Highway 138	Location: San Bernardino	
	Surface Elevation: ±	
	Total Depth: 35.0	

FIELD					LAB DATA							DESCRIPTION AND REMARKS
Water	Core	Bulk	Blows*	Depth (ft)	Moisture Content (%)	Dry Density (Pcf)	Other	USCS	Graphics	Depth (ft)		This log is a representation of conditions at the time & place of drilling. With the passage of time or at any other location there may be consequential changes in conditions.
			6 7 10		16.0	101.1						Alluvium continues, silty/clayey sand, medium dense, damp.
			3 2 5	30						30		
			8 28 30	35	3.7	115.2				35		Total Depth 35 feet No Groundwater Hole Backfilled

See Legend for Important Notes | *Driving Weight: 140 lbs/automatic

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Boring B-15
Sheet 2 of 2

Log of Exploratory Boring B-16

Client: A.R.C. Las Flores Ltd.	Date: 12-12-89	W.O. 215-01
Project: Highway 138	Location: San Bernardino	
	Surface Elevation: ±	
	Total Depth: 14.5	

FIELD					LAB DATA						
Water	Core	Bulk	Blows*	Depth (ft)	Moisture Content (%)	Dry Density (Pcf)	Other	USCS	Graphics	Depth (ft)	DESCRIPTION AND REMARKS
								ML			ALLUVIUM: Silty Sand, medium brown, damp, loose to medium dense, some gravel, portions slightly clayey.
							<200 19%				
	■		3 4 4		7.3	105.2					
				5						5	
	■		3 3 5		5.7	99.3					
				10						10	
	■		3 3 5		9.6	109.9					

This log is a representation of conditions at the time & place of drilling. With the passage of time or at any other location there may be consequential changes in conditions.

Total Depth 14.5 feet
No Groundwater
Hole Backfilled

See Legend for Important Notes | *Driving Weight: 140 lbs/automatic

JOHN A. SAYERS & ASSOCIATES
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Boring B-16
Sheet 1 of 1

Log of Exploratory Boring B-1A

JASA

Client: **Rancho Las Flores Limited Partnership**	Date: **5-15-96**	W.O. **215-1**
Project: **Highway 138**	Location: **San Bernardino County**	
	Surface Elevation: ~	
	Total Depth: ~47	

FIELD					LAB DATA						
Water	Core	Bulk	Blows*	Depth (ft)	Moisture Content (%)	Dry Density(pcf)	Relative Compaction	USCS	Graphics	Depth (ft)	DESCRIPTION AND REMARKS
											This log is a representation of conditions at the time & place of drilling. With the passage of time or at any other location there may be consequential changes in conditions.
											CROWDER FORMATION: Sandstone fine to medium grained, scattered pebbles, very light brown, hard, slightly weathered, damp
	■		6/6"	5	8.0	122.5				5	Slightly cemented, very hard
	■		8/6" 3/1"	10	8.1	126.1				10	
				15						15	
	■		5/6" 9/6"	20	10.4	127.4				20	Fine-grained with silt, very hard

See Legend for Important Notes

*Driving Weight: 3350 to 29' - 2040 to 58'

John A. Sayers & Associates
GEOTECHNICAL CONSULTANTS

Boring B-1A
Sheet 1 of 2

Log of Exploratory Boring B-1A

JASA

Client: Rancho Las Flores Limited Partnership	Date: 5-15-96	W.O. 215-1
Project: Highway 138	Location: San Bernardino County	
	Surface Elevation: ~	
	Total Depth: ~47	

FIELD					LAB DATA						
Water	Core	Bulk	Blows*	Depth (ft)	Moisture Content (%)	Dry Density(pcf)	Relative Compaction	USCS	Graphics	Depth (ft)	DESCRIPTION AND REMARKS
											This log is a representation of conditions at the time & place of drilling. With the passage of time or at any other location there may be consequential changes in conditions.
											Crowder Formation continued
	■		8/6" 12/1"	30	6.2	128.7				30	Scattered pebbles, very hard
				35						35	
	■		18/6" 7/1"	40	4.7	124.5				40	Scattered pebbles and gravel, very hard
				45						45	
											@ 47 ' - encountered large cobble or small boulder, could not get past with bucket, discontinued drilling
											Total depth 47 feet. No water. No caving.

See Legend for Important Notes

*Driving Weight: 3350 to 29' - 2040 to 58'

John A. Sayers & Associates
GEOTECHNICAL CONSULTANTS

Boring B-1A
Sheet 2 of 2

Log of Exploratory Boring B-1A

JASA

Client: Rancho Las Flores Limited Partnership	Date: 5-15-96	W.O. 215-1
Project: Highway 138	Location: San Bernardino County	
	Surface Elevation: ~	
	Total Depth: ~47	

FIELD					LAB DATA						DESCRIPTION AND REMARKS
Water	Core	Bulk	Blows*	Depth (ft)	Moisture Content (%)	Dry Density(pcf)	Relative Compaction	USCS	Graphics	Depth (ft)	This log is a representation of conditions at the time & place of drilling. With the passage of time or at any other location there may be consequential changes in conditions.
											Crowder Formation continued
	■		8/6" 12/1"	30	6.2	128.7				30	Scattered pebbles, very hard
				35						35	
	■		18/6" 7/1"	40	4.7	124.5				40	Scattered pebbles and gravel, very hard
				45						45	
											@ 47 ' - encountered large cobble or small boulder, could not get past with bucket, discontinued drilling

Total depth 47 feet.
No water. No caving.

See Legend for Important Notes | *Driving Weight: 3350 to 29' - 2040 to 58'

John A. Sayers & Associates
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Boring B-1A
Sheet 2 of 2

Log of Exploratory Boring B-2A

JASA

Client: Rancho Las Flores Limited Partnership	Date: 5-15-96	W.O. 215-1
Project: Highway 138	Location: San Bernardino County	
	Surface Elevation: ~	
	Total Depth: ~45	

FIELD					LAB DATA						
Water	Core	Bulk	Blows*	Depth (ft)	Moisture Content (%)	Dry Density(pcf)	Relative Compaction	USCS	Graphics	Depth (ft)	DESCRIPTION AND REMARKS
											This log is a representation of conditions at the time & place of drilling. With the passage of time or at any other location there may be consequential changes in conditions.
											CROWDER FORMATION: Sandstone fine to medium grained, scattered pebbles, very light brown, hard, slightly weathered, damp
	■	■	5/6" 9/1"	5	8.4	131.5				5	Very hard
	■	■	M	10	12.0	127.5				10	Silty Fine to Medium Sand, very hard
	■		8/6" 7/2"	15	8.5	128.0				15	Silty with Trace Clay, very hard
	■		15/6	20	5.3	118.5				20	Gravelly Fine tóo Medium Sand, scattered gravels continue below, very hard

ee Legend for Important Notes

*Driving Weight: 3350 to 29' - 2040 to 58'

John A. Sayers & Associates
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Boring B-2A
Sheet 1 of 2

Log of Exploratory Boring B-2A

JASA

Client: Rancho Las Flores Limited Partnership	Date: 5-15-96	W.O. 215-1
Project: Highway 138	Location: San Bernardino County	
	Surface Elevation: ~	
	Total Depth: ~45	

FIELD					LAB DATA						
Water	Core	Bulk	Blows*	Depth (ft)	Moisture Content (%)	Dry Density(pcf)	Relative Compaction	USCS	Graphics	Depth (ft)	DESCRIPTION AND REMARKS
			15/6"		4.6	122.0					Crowder Formation continued, very hard
				30						30	
			17/6 8/1"	35	6.0	123.3				35	Scattered gravel, very hard
				40						40	
			16/6 9/1"	45	6.0	120.1				45	Very hard, contains coarse sand

This log is a representation of conditions at the time & place of drilling. With the passage of time or at any other location there may be consequential changes in conditions.

Total depth 45 feet.
No water. No caving.

See Legend for Important Notes

*Driving Weight: 3350 to 29' - 2040 to 58'

John A. Sayers & Associates
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Boring B-2A
Sheet 2 of 2

TE JUNE 1, 1990

SOIL SURVEY SHEET

LIMITS PROPOSED ROUTE 138
STA. 61+99.61 TO STA. 209+00

ESAL 300,000
EWL

Combined Cohesion of Layers Above

Traffic Index ____ Base ____ Subbase ____ Basement Soil ____

BORING OR SAMPLE No.	B-1	B-1	B-1	B-2	B-2	B-2	B-3	B-3
TEST NUMBER (D or H)								
DATE SAMPLED								
STATION	63+34	63+34	63+34	72+04	72+04	72+04	80+30	80+30
LINE								
DISTANCE FROM LINE (RT. or LT.)	—	—	—	—	—	—	19' RT.	19' RT
DEPTH OR ELEVATION	5'	7.5'	10'	2'	5'	7.5'	2.5'	5'
TYPE OF MATERIAL	SILTY SAND	SILTY SAND	SILTY SAND	SILTY SAND	SILTY SAND	SILTY SAND	SILTY SAND	SILTY SAN
1½" SQUARE OPENING								
½" " "								
⅜" " "				100				
No. 4 (4760 MICRONS)				100				
No. 10 ~~8 (2380 ")~~				91.3				
No. 20 ~~16 (1190 ")~~				74.2				
No. 40 ~~30 (590 ")~~				56.6				
No. 100 ~~50 (297 ")~~				34.5				
No. 200 (74 ")		8.3	20.2	25.7		5.1		22.1
5 MICRON								
1 MICRON								
IN-PLACE DENSITY (DRY WT.- lb/ft³)	113.3	114.6	125.3	106.9	121.6	107.5	—	106.2
IN-PLACE MOISTURE (PERCENT)	1.4	12.7	10.9	4.8	1.3	1.7	0.8	3.2
MAXIMUM DENSITY (DRY WT.- lb/ft³)		131.5			133.0			
OPT. MOISTURE (PERCENT)		8.5			7.8			
RELATIVE COMPACTION (PERCENT)								
OIL RATIO - TYPE OF BITUMEN								
STABILOMETER VALUE								
COHESION								
SWELL								
STABILOMETER VALUE AFTER MVS								
% CEMENT								
OPT. MOISTURE								
DENSITY								
COMP. STRENGTH AT 7 DAYS								
LIQUID LIMIT								
PLASTICITY INDEX								
SAND EQUIVALENT		46.0						
PRODUCT P.I. X-200								
MOISTURE % AT COMPACTION		8.8						
DENSITY		130.4						
R-VALUE - STABILOMETER		78						
R-VALUE BY EXPANSION								
COHESION								
THICKNESS OF COVER REQUIRED		G.E. = 0.56 FT.						

SEE SHT 7 FOR CORROSION DATA

JOHN A. SAYERS & ASSOCIATES

GEOTECHNICAL CONSULTANTS

DATE JUNE 1, 1990

SOIL SURVEY SHEET

LIMITS PROPOSED ROUTE 138
STA. 61+99.61 TO STA. 209+00

ESAL 300,000
EWL

Combined Cohesion of Layers Above

Traffic Index ____ Base ____ in. Subbase ____ Basement Soil ____

DESCRIPTION	BORING OR SAMPLE No.	B-3	B-4	B-4	B-4	B-4	B-4	B-5	B-5
	TEST NUMBER (D or H)								
	DATE SAMPLED								
	STATION	80+30	87+85	87+85	87+85	87+85	87+85	95+24	95+24
	LINE								
	DISTANCE FROM LINE (RT. or LT.)	19' RT.	40' LT.	40' LT.	40' LT.	40' LT.	40' LT.	30' RT.	30' RT.
	DEPTH OR ELEVATION	10'	2.5'	5'	10'	20'	25'	2.5'	5'
	TYPE OF MATERIAL	SILTY SAND	SILTY SAND	SILTY SAND	SILTY SAND	SILTY SAND	SILTY SAND	SILTY SAND	SILTY SAND
ANALYSIS	1½" SQUARE OPENING								
	½" " "								
	⅜" " "			96.2					
	No. 4 (4760 MICRONS)			92.3					
	No. 10 ~~8 (2380 ")~~			81.6					
	No. 20 ~~16 (1190 ")~~			60.3					
	No. 40 ~~30 (590 ")~~			36.9					
	No. 100 ~~50 (297 ")~~			14.4					
	No. 200 (74 ")		15.7	9.1				3.5	
	5 MICRON								
	1 MICRON								
COMPACTION	IN-PLACE DENSITY (DRY WT.- lb/ft³)	106.2	126.9	—	114.9	121.8	—	104.3	107.8
	IN-PLACE MOISTURE (PERCENT)	1.4	2.6	2.0	1.0	3.0	2.8	0.8	2.0
	MAXIMUM DENSITY (DRY WT.- lb/ft³)								
	OPT. MOISTURE (PERCENT)								
	RELATIVE COMPACTION (PERCENT)								
MIX.	OIL RATIO - TYPE OF BITUMEN								
	STABILOMETER VALUE								
	COHESION								
	SWELL								
	STABILOMETER VALUE AFTER MVS								
CTB	% CEMENT								
	OPT. MOISTURE								
	DENSITY								
	COMP. STRENGTH AT 7 DAYS								
	LIQUID LIMIT								
	PLASTICITY INDEX								
	SAND EQUIVALENT								
	PRODUCT P.I. X-200								
R-VALUE	MOISTURE % AT COMPACTION								
	DENSITY								
	R-VALUE - STABILOMETER								
	R-VALUE BY EXPANSION								
	COHESION								
	THICKNESS OF COVER REQUIRED								

SEE SHT 7 FOR CORROSION DATA

JOHN A. SAYERS & ASSOCIATES

Geotechnical Consultants

DATE JUNE 1, 1990

SOIL SURVEY SHEET

LIMITS PROPOSED ROUTE 138

STA. 61+99.61 TO STA. 209+00

ESAL 300,000

EWL

Combined Cohesion of Layers Above

Traffic Index ____ Base ____ Subbase ____ Basement Soil ____

BORING OR SAMPLE No.	B-5	B-5	B-6	B-6	B-6	B-7	B-7	B-7
TEST NUMBER (D or H)								
DATE SAMPLED								
STATION	95+24	95+24	102+55	102+55	102+55	109+30	109+30	109+30
LINE								
DISTANCE FROM LINE (RT. or LT.)	30' RT.	30' RT.	18' LT.	18' LT.	18' LT.	60' RT.	60' RT.	60' RT.
DEPTH OR ELEVATION	7.5'	10'	2.5'	3'-6'	7.5'	0'-3'	5'	10'
TYPE OF MATERIAL	SILTY SAND	SILTY SAND	SILTY SAND	SILTY SAND	SILTY SAND	CLAYEY SAND	CLAYEY SAND	CLAYEY SAND
1½" SQUARE OPENING								
¾" " "								
⅜" " "				100				
No. 4 (4760 MICRONS)				100				
No. 10 ~~8 (2380 ")~~				84.7				
No. 20 ~~16 (1190 ")~~				57.2				
No. 40 ~~30 (590 ")~~				36.8				
No. 100 ~~50 (297 ")~~				17.7				
No. 200 (74 ")	12.1			11.5		46.1	62.1	56.3
5 MICRON								
1 MICRON								
IN-PLACE DENSITY (DRY WT.- lb/ft³)	110.6	113.7	106.1	117.9	115.1	129.2	123.5	124.3
IN-PLACE MOISTURE (PERCENT)	2.9	2.9	1.3	1.3	1.1	5.0	11.3	8.0
MAXIMUM DENSITY (DRY WT.- lb/ft³)				130.0		131.0		
OPT. MOISTURE (PERCENT)				10.1		8.5		
RELATIVE COMPACTION (PERCENT)								
OIL RATIO - TYPE OF BITUMEN								
STABILOMETER VALUE								
COHESION								
SWELL								
STABILOMETER VALUE AFTER MVS								
% CEMENT								
OPT. MOISTURE								
DENSITY								
COMP. STRENGTH AT 7 DAYS								
LIQUID LIMIT								
PLASTICITY INDEX								
SAND EQUIVALENT				37				
PRODUCT P.I. X-200								
MOISTURE % AT COMPACTION				9.4				
DENSITY				129.1				
R-VALUE - STABILOMETER				68				
R-VALUE BY EXPANSION								
COHESION								
THICKNESS OF COVER REQUIRED				G.E. = 0.82 FT.				

SEE SHT 7 FOR CORROSION DATA

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GEOTECHNICAL CONSULTANTS

TE JUNE 1, 1990

SOIL SURVEY SHEET

LIMITS PROPOSED ROUTE 138
STA. 61+99.61 TO STA. 209+00

ESAL 300,000
EWL

Combined Cohesion of Layers Above

Traffic Index ____ Base ____ Subbase ____ Basement Soil ____

BORING OR SAMPLE No.		B-9	B-9	B-9	B-10	B-10	B-10	
TEST NUMBER (D or H)								
DATE SAMPLED								
STATION		124+58	124+58	124+58	137+25	137+25	137+25	
LINE								
DISTANCE FROM LINE (RT. or LT.)		—	—	—	18' RT.	18' RT.	18' RT.	
DEPTH OR ELEVATION		2.5'	5'	15'	0'-3'	5'	10'	
TYPE OF MATERIAL		SILTY SAND	SILTY SAND	CLAYEY SAND	SILTY SAND	SILTY SAND	SILTY SAND	
1½" SQUARE OPENING								
¾" " "								
⅜" " "		91.7			96.5			
No. 4 (4760 MICRONS)		83.9			94.2			
No. 10 ~~8 (2380 ")~~		68.0			83.5			
No. 20 ~~16 (1190 ")~~		43.2			66.0			
No. 40 ~~30 (590 ")~~		25.2			52.7			
No. 100 ~~50 (297 ")~~		10.6			38.2			
No. 200 (74 ")		6.1		39.7	29.6		49.2	
5 MICRON								
1 MICRON								
IN-PLACE DENSITY (DRY WT.- lb/ft³)		115.4	110.3	120.1	120.5	121.3	126.0	
IN-PLACE MOISTURE (PERCENT)		1.0	1.5	4.3	2.2	1.6	4.6	
MAXIMUM DENSITY (DRY WT.- lb/ft³)					134.5			
OPT. MOISTURE (PERCENT)					8.5			
RELATIVE COMPACTION (PERCENT)								
OIL RATIO - TYPE OF BITUMEN								
STABILOMETER VALUE								
COHESION								
SWELL								
STABILOMETER VALUE AFTER MVS								
% CEMENT								
OPT. MOISTURE								
DENSITY								
COMP. STRENGTH AT 7 DAYS								
LIQUID LIMIT								
PLASTICITY INDEX								
SAND EQUIVALENT								
PRODUCT P.I. X-200								
MOISTURE % AT COMPACTION					10.1			
DENSITY					127.5			
R-VALUE - STABILOMETER					61			
R-VALUE BY EXPANSION								
COHESION								
THICKNESS OF COVER REQUIRED					G.E. = 0.99 FT.			

SEE SHT 7 FOR CORROSION DATA

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GEOTECHNICAL CONSULTANTS

DATE JUNE 1 1990

SOIL SURVEY SHEET

LIMITS PROPOSED ROUTE 138
STA. 61+99.61 TO STA. 209+00

ESAL 300,000
EWL

Combined Cohesion of Layers Above

Traffic Index ____ Base ____ in. Subbase ____ Basement Soil ____

DESCRIPTION	BORING OR SAMPLE No.	B-11	B-11	B-12	B-12	B-12	B-12	B-13	B-13
	TEST NUMBER (D or H)								
	DATE SAMPLED								
	STATION	145+34	145+34	156+00	156+00	156+00	156+00	168+70	168+70
	LINE								
	DISTANCE FROM LINE (RT. or LT.)	8' LT.	8' LT.	20' LT.	20' LT.	20' LT.	20' LT.	26' RT.	26' RT.
	DEPTH OR ELEVATION	2.5'	5'	2.5'	5'	10'	20'	5'	15'
	TYPE OF MATERIAL	SILTY SAND	SILTY SAND	SILTY SAND	SILTY SAND	CLAYEY SAND	SILTY SAND	SILTY SAND	SILTY SAND
ANALYSIS	1½" SQUARE OPENING								
	¾" " "								
	⅜" " "								100
	No. 4 (4760 MICRONS)								99.8
	No. 10 ~~8 (2380 ")~~								87.4
	No. 20 ~~16 (1190 ")~~								67.4
	No. 40 ~~30 (590 ")~~								51.1
	No. 100 ~~50 (297 ")~~								33.7
	No. 200 (74 ")		52.7	12.2		51.0		32.2	26.5
	5 MICRON								
	1 MICRON								
COMPACTION	IN-PLACE DENSITY (DRY WT.- lb/ft³)	120.6	117.9	104.7	109.0	113.1	120.3	121.9	–
	IN-PLACE MOISTURE (PERCENT)	2.7	6.4	3.7	1.9	7.9	5.4	3.1	–
	MAXIMUM DENSITY (DRY WT.- lb/ft³)								
	OPT. MOISTURE (PERCENT)								
	RELATIVE COMPACTION (PERCENT)								
MIX.	OIL RATIO - TYPE OF BITUMEN								
	STABILOMETER VALUE								
	COHESION								
	SWELL								
	STABILOMETER VALUE AFTER MVS								
CTB	% CEMENT								
	OPT. MOISTURE								
	DENSITY								
	COMP. STRENGTH AT 7 DAYS								
	LIQUID LIMIT								
	PLASTICITY INDEX								
	SAND EQUIVALENT								30
	PRODUCT P.I. X-200								
MATERIAL	MOISTURE % AT COMPACTION								
	DENSITY								
	R-VALUE - STABILOMETER								
	R-VALUE BY EXPANSION								
	COHESION								
	THICKNESS OF COVER REQUIRED								

SEE SHT 7 FOR CORROSION DATA

JOHN A. SAYERS & ASSOCIATES

GEOTECHNICAL CONSULTANTS

ATE JUNE 1, 1990

SOIL SURVEY SHEET

LIMITS PROPOSED ROUTE 138
STA. 61+99.61 TO STA. 209+00

ESAL 300,000
EWL

6 OF
Combined Cohesion of Layers Above

Traffic Index ____ Base ____ Subbase ____ Basement Soil ____

BORING OR SAMPLE No.	B-14	B-14	B-15	B-15	B-15	B-15	B-15	B-15
TEST NUMBER (D or H)								
DATE SAMPLED								
STATION	190+00	190+00	200+42	200+42	200+42	200+42	200+42	200+42
LINE								
DISTANCE FROM LINE (RT. or LT.)	—	—	12' LT.	12' LT.	12' LT.	12' LT.	12' LT.	12' LT.
DEPTH OR ELEVATION	2.5'	5.0'	0'-3'	2.5'	5.0'	10.0'	15.0	25.0'
TYPE OF MATERIAL	SILTY SAND	SILTY SAND	SILTY SAND	SILTY SAND	SILTY SAND	SILTY SAND	SILTY SAND	SILTY SAND
1½" SQUARE OPENING								
½" " "								
⅜" " "	93.5							
No. 4 (4760 MICRONS)	88.3							
No. 8 (2380 ")	75.8							
No. 16 (1190 ")	54.4							
No. 30 (590 ")	33.9							
No. 50 (297 ")	12.6							
No. 200 (74 ")	4.4		30			43		
5 MICRON								
1 MICRON								
IN-PLACE DENSITY (DRY WT.- lb/ft³)	122.2	114.3	101.1	120.3	109.5	106.3	110.3	101.1
IN-PLACE MOISTURE (PERCENT)	3.9	4.1	16.0	6.0	7.2	8.2	5.5	16.0
MAXIMUM DENSITY (DRY WT.- lb/ft³)			133.0					
OPT. MOISTURE (PERCENT)			8.0					
RELATIVE COMPACTION (PERCENT)								
OIL RATIO - TYPE OF BITUMEN								
STABILOMETER VALUE								
COHESION								
SWELL								
STABILOMETER VALUE AFTER MVS								
% CEMENT								
OPT. MOISTURE								
DENSITY								
COMP. STRENGTH AT 7 DAYS								
LIQUID LIMIT								
PLASTICITY INDEX								
SAND EQUIVALENT								
PRODUCT P.I. X-200								
MOISTURE % AT COMPACTION			10.0					
DENSITY			130.2					
R-VALUE - STABILOMETER			67.0					
R-VALUE BY EXPANSION								
COHESION								
THICKNESS OF COVER REQUIRED			G.E. = 0.84 FT.					

SEE SHT 7 FOR CORROSION DATA

JOHN A. SAYERS & ASSOCIATES

GEOTECHNICAL CONSULTANTS

TE JUNE 1, 1990

SOIL SURVEY SHEET

LIMITS PROPOSED ROUTE 138
STA. 61+99.61 TO STA. 209+00

ESAL 300,000
EWL

Combined Cohesion of Layers Above

Traffic Index ____ Base ____ Subbase ____ Basement Soil ____

BORING OR SAMPLE No.	B-16	B-16		B-1	B-6	B-10	B-13	B-15
TEST NUMBER (D or H)								
DATE SAMPLED								
STATION	209+20	209+20		63+34	102+50	137+00	169+00	200+25
LINE								
DISTANCE FROM LINE (RT. or LT.)	15' RT.	15' RT.		—	18' LT.	18' RT.	26' RT.	12' LT.
DEPTH OR ELEVATION	0'-3'	10.0'		6'-8'	3'-6'	0'-3'	12'-15'	0'-3'
TYPE OF MATERIAL	SILTY SAND	SILTY SAND		SILTY SAND	SILTY SAND	SILTY SAND	SILTY SAND	SILTY SAND
1½" SQUARE OPENING								
¾" " "				CORROSION DATA	CORROSION DATA	CORROSION DATA	CORROSION DATA	CORROSION DA
⅜" " "	100		Ohm/cm³	14250	14250	11880	3960	5280
No. 4 (4760 MICRONS)	100		pH	7.43	6.80	8.90	8.31	7.59
No. 8 (2380 ")	90.2		SO_4	<.001%	<.001%	<.001%	<.001%	<.001%
No. 16 (1190 ")	69.5							
No. 30 (590 ")	49.2							
No. 50 (297 ")	26.2							
No. 200 (74 ")	19.0							
5 MICRON								
1 MICRON								
IN-PLACE DENSITY (DRY WT.- lb/ft³)	105.2	109.9						
IN-PLACE MOISTURE (PERCENT)	7.3	9.5						
MAXIMUM DENSITY (DRY WT.- lb/ft³)	131.7							
OPT. MOISTURE (PERCENT)	8.1							
RELATIVE COMPACTION (PERCENT)								
OIL RATIO - TYPE OF BITUMEN								
STABILOMETER VALUE								
COHESION								
SWELL								
STABILOMETER VALUE AFTER MVS								
% CEMENT								
OPT. MOISTURE								
DENSITY								
COMP. STRENGTH AT 7 DAYS								
LIQUID LIMIT								
PLASTICITY INDEX								
SAND EQUIVALENT	29							
PRODUCT P.I. X-200								
MOISTURE % AT COMPACTION	12.0							
DENSITY	123.6							
R-VALUE - STABILOMETER	61							
R-VALUE BY EXPANSION								
COHESION								
THICKNESS OF COVER REQUIRED	G.E. = 0.99							

SEE SHT 7 FOR CORROSION DATA

JOHN A. SAYERS & ASSOCIATES

GEOTECHNICAL CONSULTANTS

DATE JUNE 1, 1990

SOIL SURVEY SHEET

LIMITS PROPOSED ROUTE 138
STA. 61+99.61 TO STA. 209+00

ESAL 300,000
EWL

Combined Cohesion of Layers Above

Traffic Index ____ Base ____ Subbase ____ Basement Soil ____

	BORING OR SAMPLE No.	B-1A	B-1A	B-1A	B-1A	B-1A	B-2A	B-2A	B-2A
	TEST NUMBER (D or H)								
	DATE SAMPLED	5-15-96	5-15-96	5-15-96	5-15-96	5-15-96	5-15-96	5-15-96	5-15-96
	STATION	189 + 35	189 + 35	189 + 35	189 + 35	189 + 35	174 + 90	174 + 90	174 + 90
	LINE								
	DISTANCE FROM LINE (RT. or LT.)	—	—	—	—	—	—		
	DEPTH OR ELEVATION	5'	10'	20'	30'	40'	5'	10'	15'
	TYPE OF MATERIAL	SILTY SAND	SILTY TO CLAYEY SAND	SILTY SAND	SILTY SAND	SILTY SAND	SILTY TO CLAYEY SAND	CLAYEY SAND	SILTY SAND
ANALYSIS	1½" SQUARE OPENING								
	¾" " "								
	⅜" " "								
	No. 4 (4760 MICRONS)	94.3	99.7	100			96.8	99.3	
	No. 10 ~~8 (2380 ")~~	82.8	97.2	90.1			88.4	93.4	
	No. 20 ~~16 (1190 ")~~	63.5	93.0	75.4			75.0	78.3	
	No. 40 ~~30 (590 ")~~	43.2	86.3	60.6			60.4	67.2	
	No. 100 ~~50 (297 ")~~	21.8	61.3	39.7			40.2	51.1	
	No. 200 (74 ")	14.2	40.7	27.3			28.3	40.3	
	5 MICRON								
	1 MICRON								
	IN-PLACE DENSITY (DRY WT.- lb/ft³)	122.5	126.1	127.4	128.7	124.5	131.5	127.5	128.0
	IN-PLACE MOISTURE (PERCENT)	8.0	8.1	10.4	6.2	4.7	8.4	12.0	8.5
	MAXIMUM DENSITY (DRY WT.- lb/ft³)		128.5	124.5					
	OPT. MOISTURE (PERCENT)		9.5	11.0					
	RELATIVE COMPACTION (PERCENT)		98.1						
MIX.	OIL RATIO - TYPE OF BITUMEN								
	STABILOMETER VALUE								
	COHESION								
	SWELL								
	STABILOMETER VALUE AFTER MVS								
	% CEMENT								
	OPT. MOISTURE								
	DENSITY								
	COMP. STRENGTH AT 7 DAYS								
	LIQUID LIMIT		27.2	24.6			25.8	31.0	
	PLASTICITY INDEX	NON - PLASTIC	5.9	2.3			4.1	8.9	
	SAND EQUIVALENT								
	PRODUCT P.I. X-200								
	MOISTURE % AT COMPACTION								
	DENSITY								
	R-VALUE - STABILOMETER								
	R-VALUE BY EXPANSION								
	COHESION								
	THICKNESS OF COVER REQUIRED								

JOHN A. SAYERS & ASSOCIATES

GEOTECHNICAL CONSULTANTS

ATE JUNE 1, 1990

SOIL SURVEY SHEET

LIMITS PROPOSED ROUTE 138
STA. 61+99.61 TO STA. 209+00

ESAL 300,000
EWL
Combined Cohesion of Layers Above
Traffic Index ____ Base ____ Subbase ____ Basement Soil ____

BORING OR SAMPLE No.	B-2A	B-2A	B-2A	B-2A	B-	B-	B-	B-
TEST NUMBER (D or H)								
DATE SAMPLED	5-15-96	5-15-96	5-15-96	5-15-96				
STATION	174+90	174+90	174+90	174+90				
LINE								
DISTANCE FROM LINE (RT. or LT.)	—	—	—	—	—	—	—	
DEPTH OR ELEVATION	20'	25'	35'	45'				
TYPE OF MATERIAL	SILTY SAND	SILTY SAND	SILTY TO CLAYEY SAND	SILTY SAND				
1½" SQUARE OPENING								
¾" " "								
⅜" " "								
No. 4 (4760 MICRONS)			98.1	95.1				
No. 10 ~~8 (2380 ")~~			90.0	86.1				
No. 20 ~~16 (1190 ")~~			71.2	69.5				
No. 40 ~~30 (590 ")~~			55.3	52.9				
No. 100 ~~50 (297 ")~~			35.0	30.5				
No. 200 (74 ")			24.7	19.6				
5 MICRON								
1 MICRON								
IN-PLACE DENSITY (DRY WT.- lb/ft³)	118.5	122.0	123.3	120.1				
IN-PLACE MOISTURE (PERCENT)	5.3	4.6	6.0	6.0				
MAXIMUM DENSITY (DRY WT.- lb/ft³)								
OPT. MOISTURE (PERCENT)								
RELATIVE COMPACTION (PERCENT)								
OIL RATIO - TYPE OF BITUMEN								
STABILOMETER VALUE								
COHESION								
SWELL								
STABILOMETER VALUE AFTER MVS								
% CEMENT								
OPT. MOISTURE								
DENSITY								
COMP. STRENGTH AT 7 DAYS								
LIQUID LIMIT			26.0					
PLASTICITY INDEX			5.2	NON-PLASTIC				
SAND EQUIVALENT								
PRODUCT P.I. X-200								
MOISTURE % AT COMPACTION								
DENSITY								
R-VALUE - STABILOMETER								
R-VALUE BY EXPANSION								
COHESION								
THICKNESS OF COVER REQUIRED								

ANALYSIS

MIX.

R-VALUE

JOHN A. SAYERS & ASSOCIATES

GEOTECHNICAL CONSULTANTS

JASA

John A Sayers and Associates, Inc.

Geotechnical Consultants

TRANSMITTAL

Mr. Ebi M. Fini
Office of Structural Foundation, South
5900 Folsom Boulevard
Sacramento, California 95819

September 17, 1996
W.O. 215

Subject: Additional Data Pertaining to Proposed Highway Construction, Route 138, 1.2 miles East of Route 15 to Summit Post Office Road, San Bernardino County, 08-SBd-138-16.5/19.7

Dear Mr. Fini:

Attached to this transmittal are several appendices providing more details on drilling and sampling methods and laboratory testing. Specifically, at your request, we are providing direct shear test diagrams, individual stress-strain plots and plasticity index and grain size data for your review.

Through this transmittal, we are hoping we can come to some agreement on the applicable shear strengths to use for slope designs so we can progress with the profiles and calculations.

Please call if you have any questions or wish to discuss the information.

Respectfully submitted,

JOHN A. SAYERS & ASSOCIATES

By: _____
John A. Sayers
Principal Geologist, CEG 907

By: _____
John A. Seminara
Principal Engineer, GE 2294

REGISTERED PROFESSIONAL ENGINEER
JOHN A. SEMINARA
No. 2294
Exp. 12/31/98
GEOTECHNICAL
STATE OF CALIFORNIA

JAS/JS/mw

27071 Cabot Road, Suite 101 • Laguna Hills, California 92653-7009 • (714) 582-2144

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Appendix A	Sampling Methods and Laboratory Testing
Appendix B	Shear Test Data - Undisturbed Samples Sheared at Natural Moisture Content
Appendix C	Shear Tests Data - Undisturbed Samples Sheared after Saturation
Appendix D	Shear Test Data - Remolded Samples Sheared at Optimum
Appendix E	Shear Test Data - Remolded Samples Sheared after Saturation
Appendix F	Boring Logs and Soil Survey Sheets

APPENDIX A

SAMPLING METHODS AND LABORATORY TESTING

SAMPLING METHODS AND LABORATORY TESTING

Drilling Techniques

Borings B-1 through B-16 were drilled with a Mobile B-52 Hollow Stem Auger (2-R Drilling Company, Ontario, California) using an eight inch diameter auger. Samples are taken from within the hollow stem by driving a sampler with a 140 pound weight according to the standard penetration test procedure ASTM D 1586-76. Borings B-1A and B-2A were drilled with a EZ Bore Drill (Al-Roy Drilling, Yorba Linda, California) using an 18 inch diameter bucket. With this rig the samples are taken by driving the sampler with the weight of a telescoping Kelly bar. As the Kelly bar extends the weight is reduced by sections depending upon the depth. These blow counts may be converted to standard penetration blows (N-values) with the following formula:

$$N = N' \times \frac{WH}{4200} \times \frac{2.00^2 - 1.38^2}{D_o^2 - D_i^2}$$

Where

N	=	equivalent SPT blowout (uncorrected)
N'	=	measured blowout
W	=	weight of hammer (Kelly), pounds
H	=	distance of hammer drop (inches) = 12
D_o	=	outside diameter of sample (inches) = 3.0
D_i	=	inside diameter of sample (inches) = 2.416

Example: B-1A @ 20' $N' = 5 + 9 = 14$

$$N = 14 \times \frac{3350\ (12")}{4200} \times \frac{2.0^2 - 1.38^2}{3.0^2 - 2.416^2} = 119.7$$

Sampling Methods

Soil samples have been collected as bulk or bag samples and/or as in-situ or relatively "undisturbed" core samples. . The latter were obtained with a split-barrel sampler containing a

series of one-inch high brass rings, 2.416 inches in inside diameter (ASTM D 3550-84). The sampler is driven into the ground with successive free fall blows of the drill rig's Kelly bar dropping one foot. The weight of the Kelly bar and number of blows per foot are typically recorded on the boring logs. Alternatively, the sampler may be driven with a 140-pound weight and the sampler may be a Standard Penetration Sampler which obtains a disturbed sample. If the standard penetration test is used, it is designated on the boring log as explained in the Boring Log Legend.

The Standard Penetration Test is performed in borings during field exploration and may be used to correlate materials and/or to determine a variety of engineering properties. The test uses a split-barrel sampler, 2 inches in diameter, which is driven into the ground with successive blows of a 140-pound weight. The blow counts are recorded in six inch intervals for further analysis and the sample contained in the sampler is retained for identification and testing. The test procedure used is the American Society for Testing and Materials (ASTM) D 1586-76.

Bulk samples were collected in plastic bags to retain their original moisture content. The core samples were placed in molded plastic containers, sealed, stored in protective boxes and transported to our laboratory.

Classification

Soils extracted from trenches/borings in the field were identified using the Unified Soil Classification System (USCS). Field identifications may be supplemented by additional visual classifications and/or by test procedures. The soil classifications are shown on the Logs of Test Pits and/or Borings presented within this report. Where blow counts were very high or refusal was encountered, the soils were described as very hard.

Laboratory Testing

The physical and engineering properties of the soils were determined by laboratory testing of samples. The types of tests typically conducted are briefly discussed below. All test results using these procedures are specific to the soils collected and variations could be expected at other depths and locations.

Natural Density and Moisture Content

The natural density of the soil has been determined for all "undisturbed" samples obtained during subsurface explorations. These densities are recorded as dry unit weights in pounds per cubic foot. The moisture contents for these and standard penetration samples are determined as a percentage of the dry weight. These test results are shown on the Boring Logs and in the Soil Survey Sheets.

Maximum Dry Density and Optimum Moisture Content

Typical bulk samples have been tested for their maximum dry density and optimum moisture content. The results are obtained using the ASTM standard procedure D 1557-78. These tests are generally used to compare natural densities and may be used for compaction testing. Additional testing for other soil types or soil mixes may be necessary at a later date.

The maximum dry densities and optimum moisture contents are presented on the Soil Survey Sheets accompanying this report.

Direct Shear

The shear strength of typical on-site soils has been determined using a direct shear machine designed to receive either "undisturbed" or remolded samples of the soil in one-inch high brass rings. The samples are extruded from the rings and into the machine apparatus, loaded with varying normal loads, allowed to consolidate and then sheared at a strain rate of 0.02 inches per minute. The samples may be sheared at their natural moisture content or may be presaturated and inundated with water prior to testing, as indicated on the test results. The test procedure is according to ASTM D 3080-79.

The test data has been plotted on stress-strain diagrams and the peak strength has been plotted on Shear Test diagrams. Using the stress-strain diagram the percent shear should be multiplied by 120 to obtain the shear strength in pounds per square foot. Both have used computer interfaces as the plotting mechanism with the friction angle and cohesion calculated by the computer, except for cohesionless soils where the cohesion was set to zero and the friction angle determined as a best fit.

Our shear testing was performed for four situations, one being undisturbed samples at natural moisture contents. The second is undisturbed samples being saturated prior and during testing. The third and fourth consist of samples remolded to 90 percent relative compaction at optimum moisture content and then either sheared at optimum moisture or at saturation. The four conditions are considered representative of specific formations and locations along the road alignment as follows:

1) Undisturbed samples at natural moisture content should be used for steep topographic areas along eastern portion of road alignment exposing Crowder Formation and Terrace deposits.

2) Undisturbed samples at saturated conditions should be used for the more gentle topographic areas along western portion of alignment exposing older alluvium and recent alluvium (cohesionless and uncemented sands).

3) Remolded samples at saturation should be used for proposed fill slopes where cementation would be lost and cohesion is minimized by recompaction.

4) Remolded samples at optimum moisture content would typically be representative of unsaturated fill, but were not utilized in our analyses.

Shear test results are presented in Appendices B through E to follow. They include a shear test summary, a summary diagram showing the values used followed by individual shear test diagrams and stress-strain diagrams.

One-Dimensional Consolidation

A soil sample from alluvium within Boring B-15 has been tested for its consolidation potential. The consolidation testing apparatus is designed to receive either natural or remolded soils in a one-inch high ring. Each sample is then loaded incrementally in a geometric progression and the resulting consolidation is recorded at selected time intervals. Porous stones are in contact with both the top and bottom of the soil to permit the addition or release of pore water. The samples are typically inundated with water at intermediate loads. The test procedure used was California Test 219 which is similar to ASTM D 2435-80.

The consolidation test result is plotted on the Consolidation Pressure Curves accompanying this appendix as Plate A-1.

Grain Size Distribution

The grain size distribution of selected soils has been determined using a range of wire mesh screens and a mechanical sieve shaker (ASTM D 422-72). The weight of the residue contained on each screen is determined and the percent finer or coarser of the total weight is calculated. The percent passing the 200-sieve wet by either the dry sieve method (above) or by the wet sieve method (ASTM D 1140-71) are shown on the Soil Survey sheets.

Grain Size Distributions are graphically plotted and are presented at the end of this appendix.

Sand Equivalent

The Sand Equivalent (S.E.) value for selected soils was determined using CALTRANS Test Method No. 217. This test determines the relative cleanliness of sandy soils and relates to their suitability for bedding, backfill and other uses. A higher S.E. reading indicates a relatively clean sand. An S.E. of 30 or greater is usually required for trench backfill which may need only flooding to obtain compaction.

Test results of the Sand Equivalent test are presented in the attached Soil Survey Sheets.

Plasticity Index

The Plasticity Index (PI) for selected soils were determined according to California Test 204, which is similar to ASTM D 4318-83. This test measures the liquid limit (LL) and Plastic Limit (PL) of the soils passing the No. 40 sieve. In each case tested, the samples had less than 50 percent and more than 12 percent passing the No. 200 sieve and liquid limits less than 50. This indicates the fines (< No. 40) are low plastic silts or clays and when combined with the sands are silty to clayey sands (SM/SC).

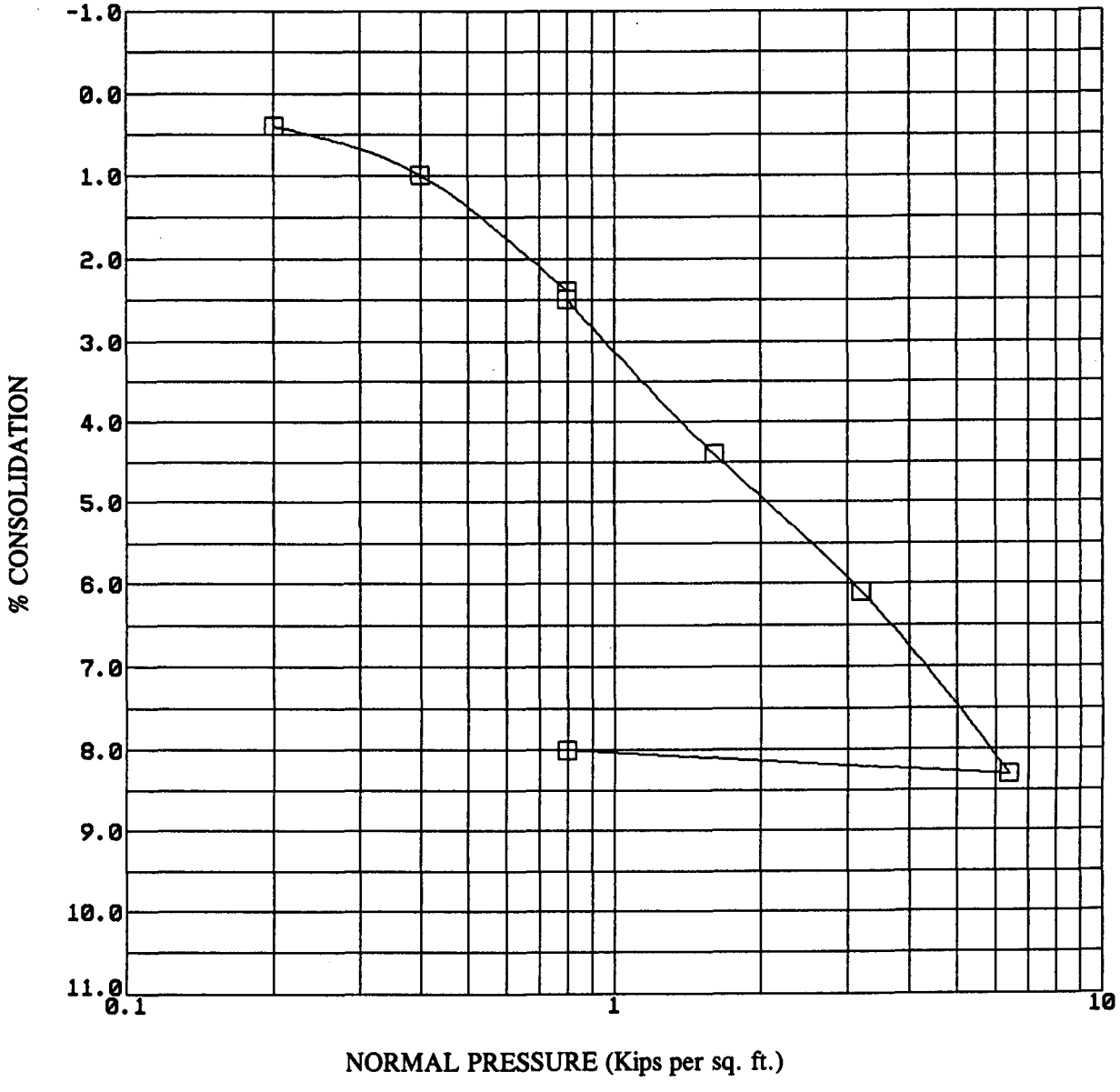
The results of these tests are shown on Plates A-2 and A-3.

Corrosion Testing

Soils resistivity and pH testing was performed for us by Associated Laboratories of Orange, California. The test method used was California 643B and the results are indicated on the Soil Survey Sheets. Associated Laboratories report is attached as Plate A-4.

CONSOLIDATION TEST RESULTS

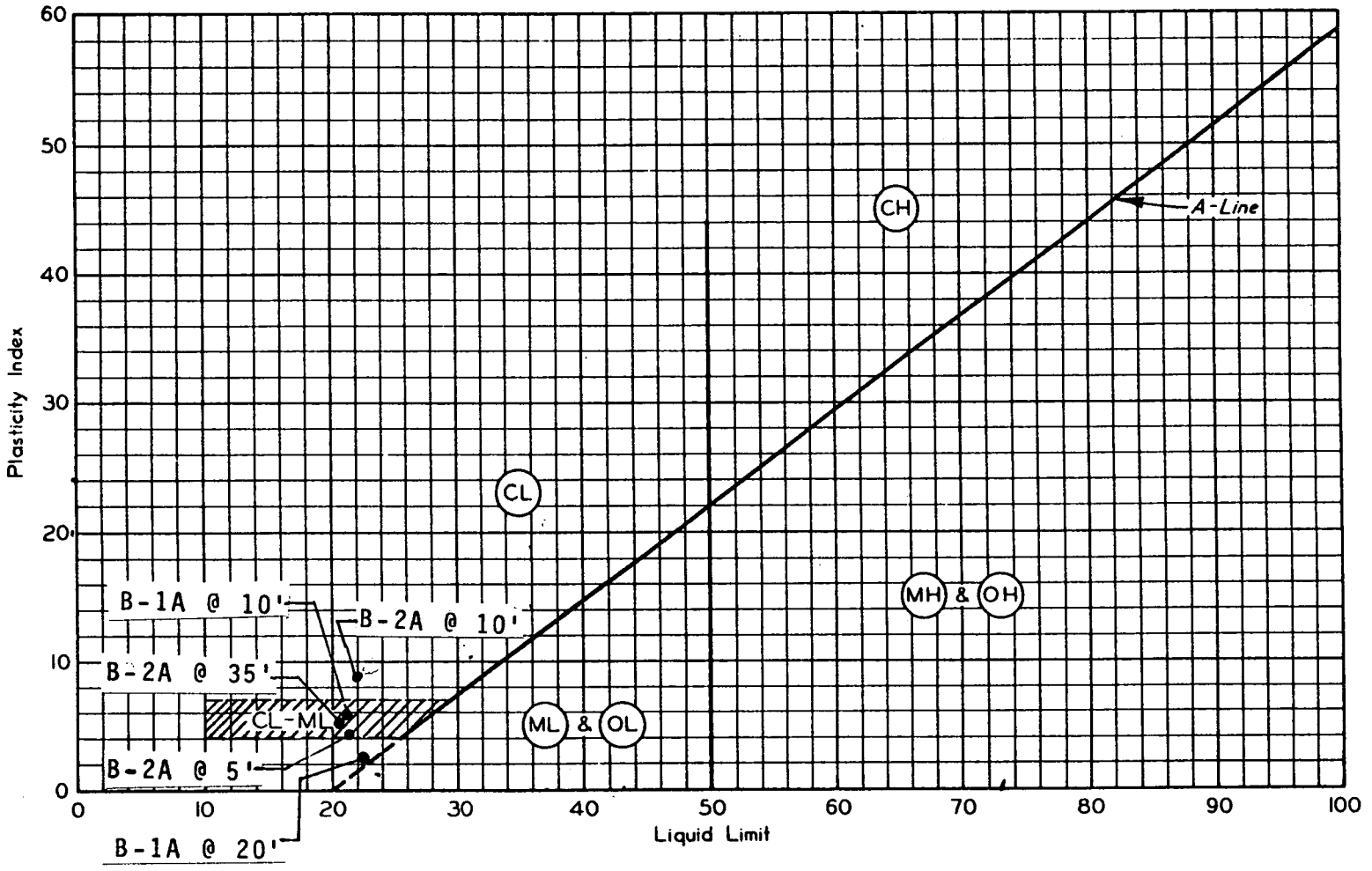
Client: **Rancho Las Flores Limited Partnership**	W.O. **215-1**
Project: **Highway 138**	Location: **San Bernardino County**



Sample: **B-15 @ 5.0 ft.**
Undisturbed
Water Added 0.8 Kips

PLASTICITY INDEX TEST RESULTS

Sample Location	Material Type	% Passing No. 200	Liquid Limit (%)	Plastic Limit (%)	Plasticity Index (%)	USCS
B-1A @ 5'	Sandstone (silty sand)	14.2	(Non-Plastic)			SM
B-1A @ 10'	Sandstone (silty to clayey sand)	40.7	27.2	21.3	5.9	SM/SC
B-1A @ 20'	Sandstone (silty sand)	27.3	24.6	22.3	2.3	SM
B-2A @ 5'	Sandstone (silty to clayey sand)	28.3	25.8	21.7	4.1	SM/SC
B-2A @10'	Sandstone (clayey sand)	40.3	31.0	22.1	8.9	SC
B-2A @ 35'	Sandstone (silty to clayey sand)	24.7	26.0	20.8	5.2	SM/SC



PLASTICITY CHART

215

ASSOCIATED LABORATORIES
806 North Batavia - Orange, California 92668 - 714/771-6900
FAX 714/538-1209

CLIENT
John A. Sayers & Associates (2562)
27071 Cabot Rd.
Ste. 132
Laguna Hills, CA 92653
Attn: Carisa Davis

LAB NO. F81294
REPORTED 02/15/90

SAMPLE Soil
RECEIVED 02/08/90
IDENTIFICATION W.O.#215
BASED ON SAMPLE As Submitted

	Sulfate	pH	Minimum Resistivity (Calif. 643B) (ohm/cm)
B-6 @ 3-6'	ND<0.001%	6.80	14,250
B-13 @ 12-15'	ND<0.001%	8.31	3,960
B-1 @ 6-8'	ND<0.001%	7.43	14,250
B-15 @ 0-3'	ND<0.001%	7.59	5,280
B-10 @ 0-3'	ND<0.001%	8.09	11,880

ASSOCIATED LABORATORIES

Tito L. Parola

TLP/ql

RECEIVED 2-27-90

NOTE: Unless notified in writing, all samples will be discarded by appropriate disposal protocol 30 days from date reported.

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TESTING & CONSULTING
Chemical •
Microbiological •
Environmental •

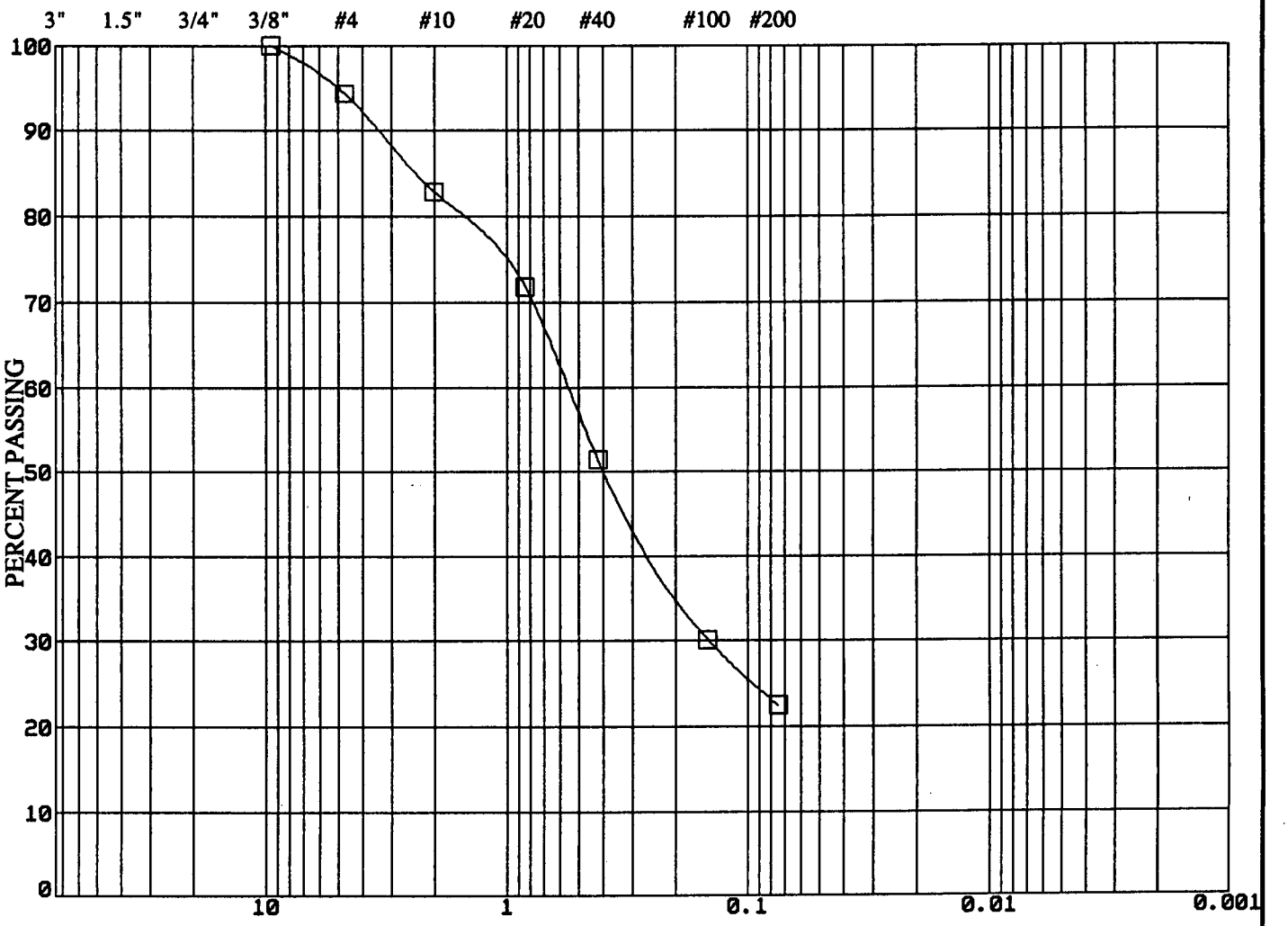
PLATE A-4

C-1 10M

GRAIN SIZE DISTRIBUTION

Client: **Rancho Las Flores Limited Partnership**	W.O. **215-1**
Project: **Highway 138**	Location: **San Bernardino County**

SIEVE ANALYSIS | HYDROMETER ANALYSIS



PARTICLE SIZE IN MILLIMETERS

GRAVEL		SAND			SILT	CLAY
coarse	fine	coarse	medium	fine		

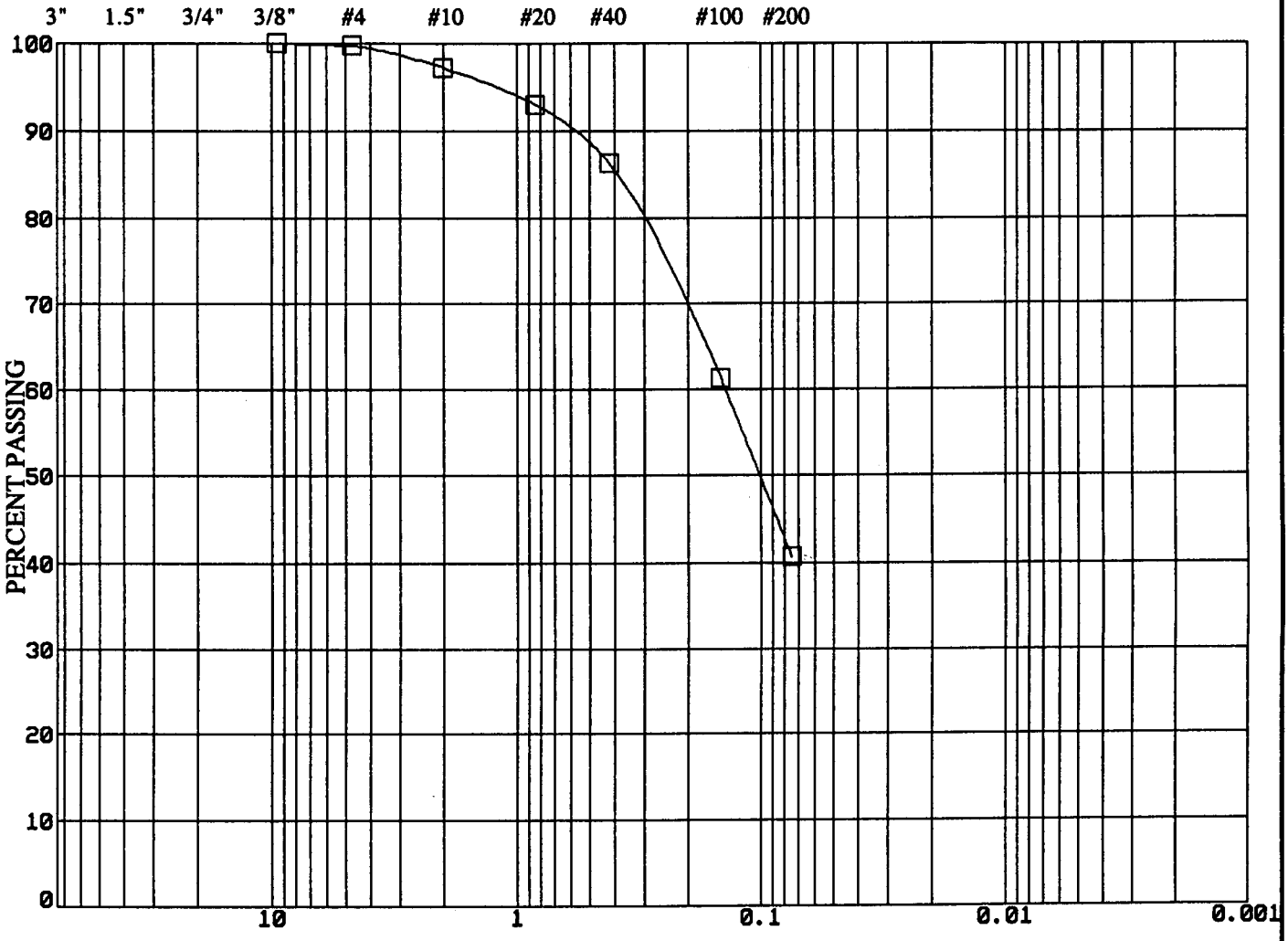
Sample: **B-1A @ 5.0 ft.**

John A. Sayers & Associates GEOTECHNICAL CONSULTANTS	Plate

GRAIN SIZE DISTRIBUTION

Client: **Rancho Las Flores Limited Partnership**	W.O. **215-1**
Project: **Highway 138**	Location: **San Bernardino County**

SIEVE ANALYSIS | HYDROMETER ANALYSIS



PARTICLE SIZE IN MILLIMETERS

GRAVEL		SAND			SILT	CLAY
coarse	fine	coarse	medium	fine		

Sample: **B-1A @ 10.0 ft.**

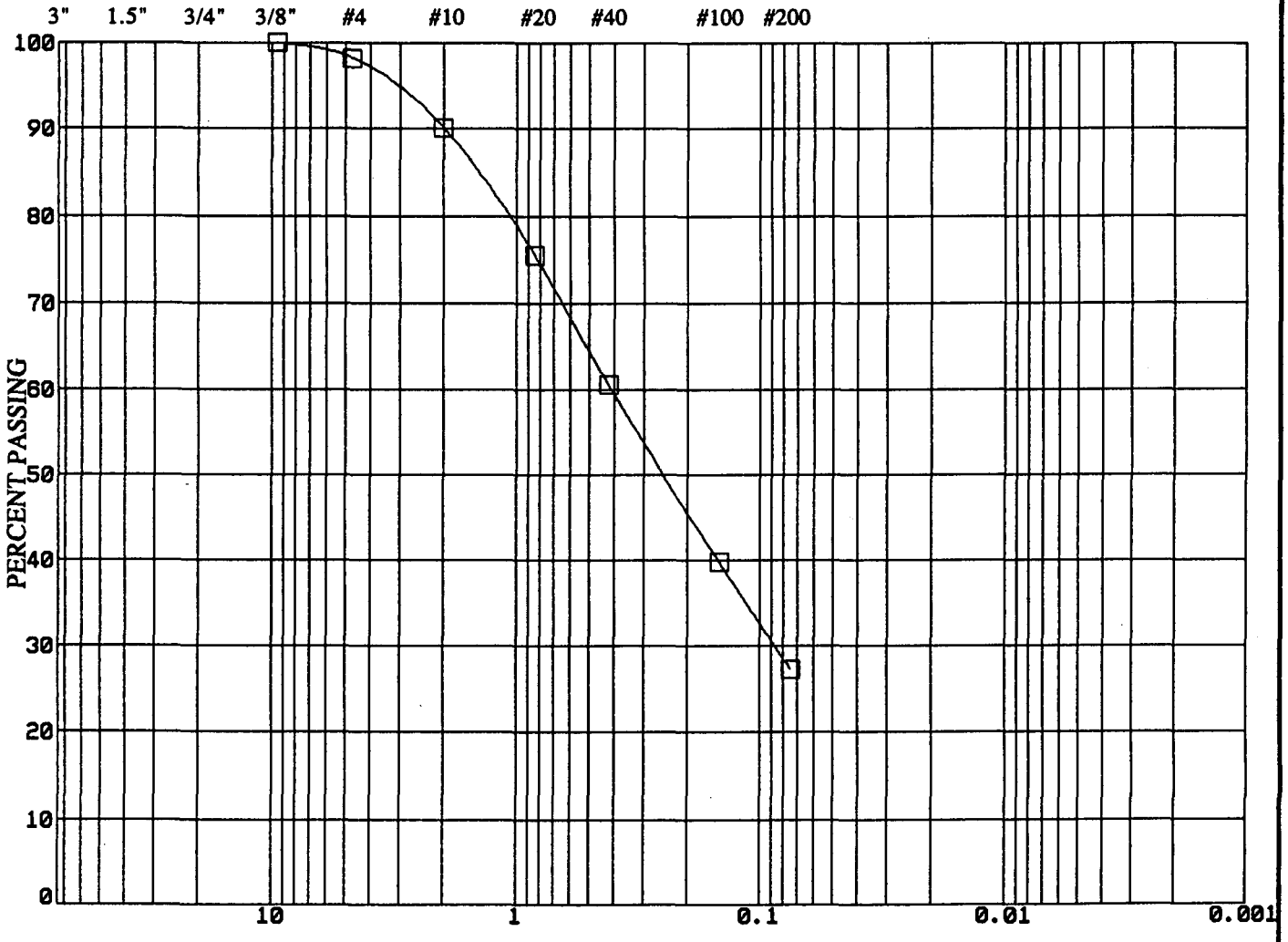
SM/SC

John A. Sayers & Associates GEOTECHNICAL CONSULTANTS	Plate

GRAIN SIZE DISTRIBUTION

Client: **Rancho Las Flores Limited Partnership**	W.O. **215-1**
Project: **Highway 138**	Location: **San Bernardino County**

SIEVE ANALYSIS | HYDROMETER ANALYSIS



PARTICLE SIZE IN MILLIMETERS

GRAVEL		SAND			SILT	CLAY
coarse	fine	coarse	medium	fine		

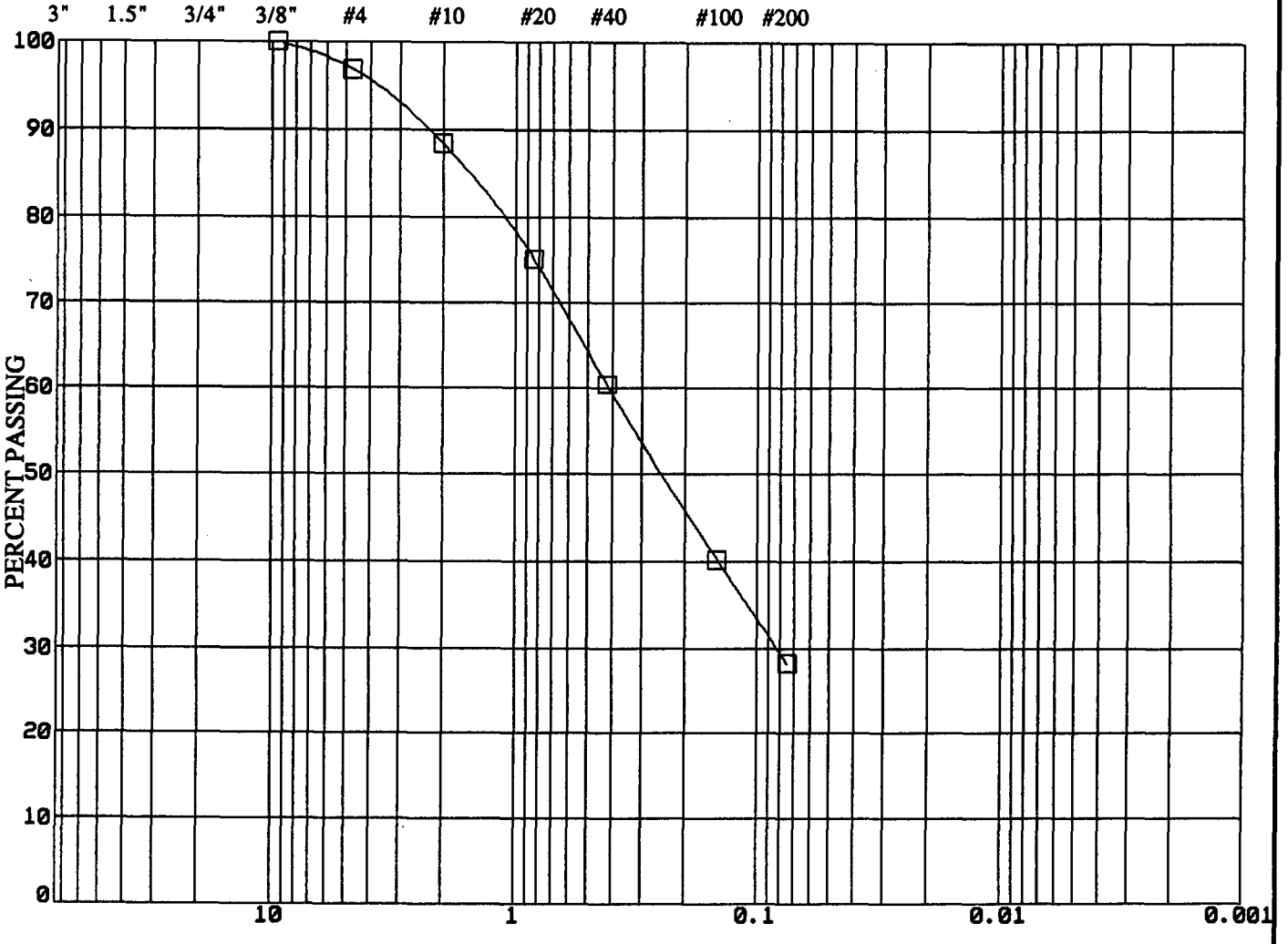
Sample: **B-1A @ 20.0 ft.**

John A. Sayers & Associates GEOTECHNICAL CONSULTANTS	Plate

GRAIN SIZE DISTRIBUTION

Client: **Rancho Las Flores Limited Partnership**	W.O. **215-1**
Project: **Highway 138**	Location: **San Bernardino County**

SIEVE ANALYSIS | HYDROMETER ANALYSIS



PARTICLE SIZE IN MILLIMETERS

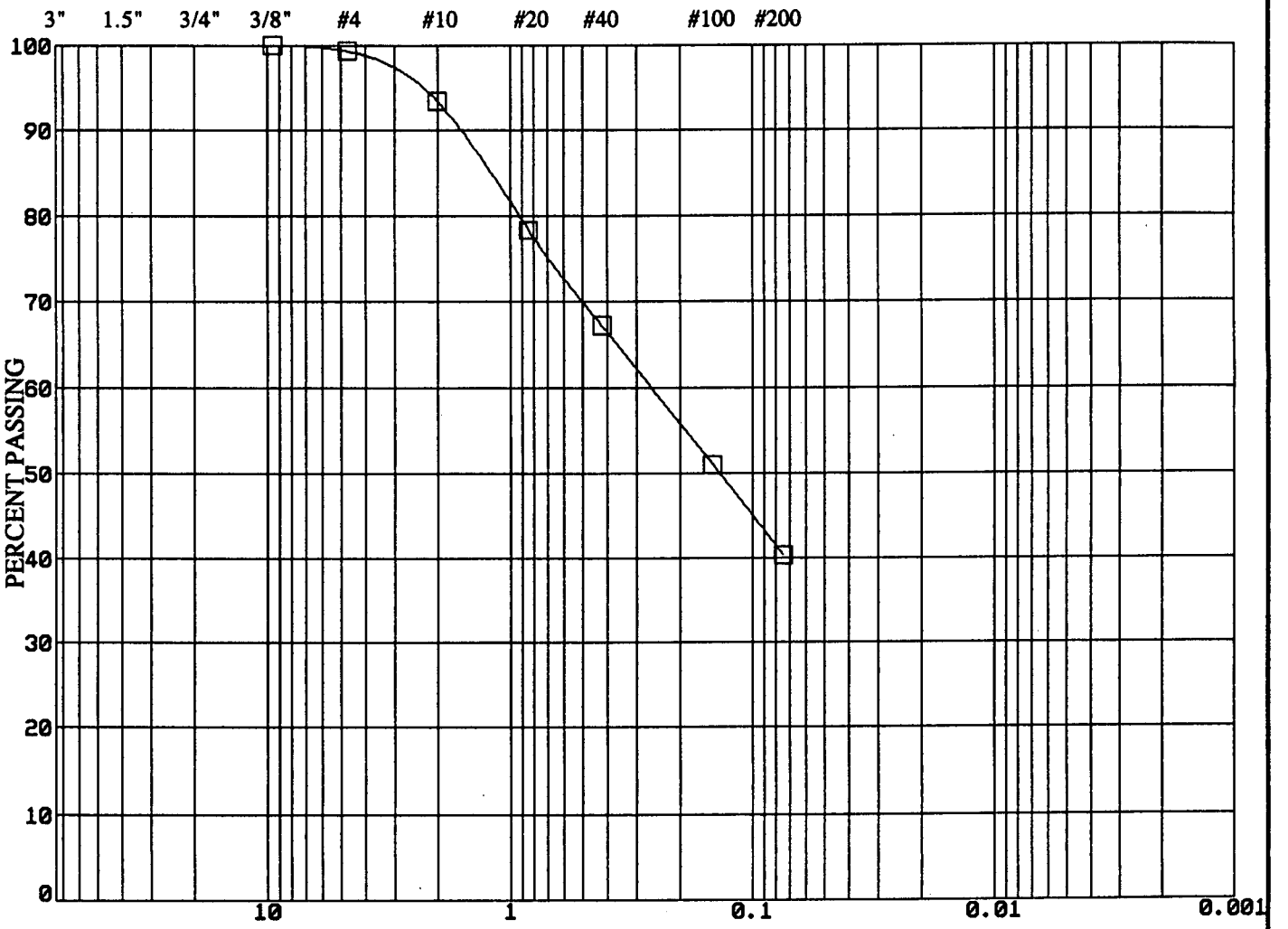
GRAVEL		SAND			SILT	CLAY
coarse	fine	coarse	medium	fine		

Sample: **B-2A @ 5.0 ft.**

GRAIN SIZE DISTRIBUTION

Client: **Rancho Las Flores Limited Partnership**	W.O. **215-1**
Project: **Highway 138**	Location: **San Bernardino County**

SIEVE ANALYSIS | HYDROMETER ANALYSIS



PARTICLE SIZE IN MILLIMETERS

GRAVEL		SAND			SILT	CLAY
coarse	fine	coarse	medium	fine		

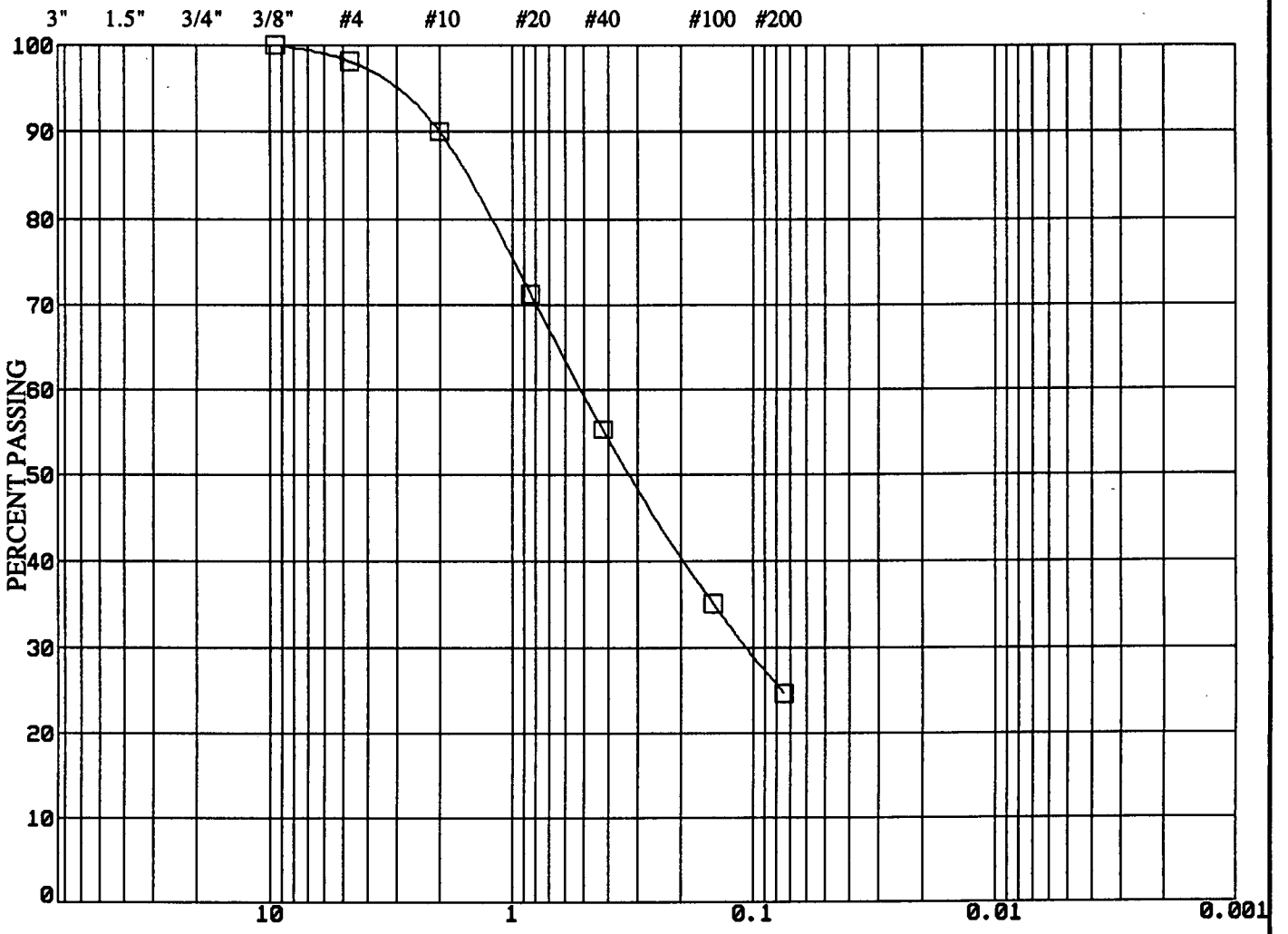
Sample: **B-2A @ 10.0 ft.**

John A. Sayers & Associates GEOTECHNICAL CONSULTANTS	Plate

GRAIN SIZE DISTRIBUTION

Client: **Rancho Las Flores Limited Partnership**	W.O. **215-1**
Project: **Highway 138**	Location: **San Bernardino County**

SIEVE ANALYSIS | HYDROMETER ANALYSIS



PARTICLE SIZE IN MILLIMETERS

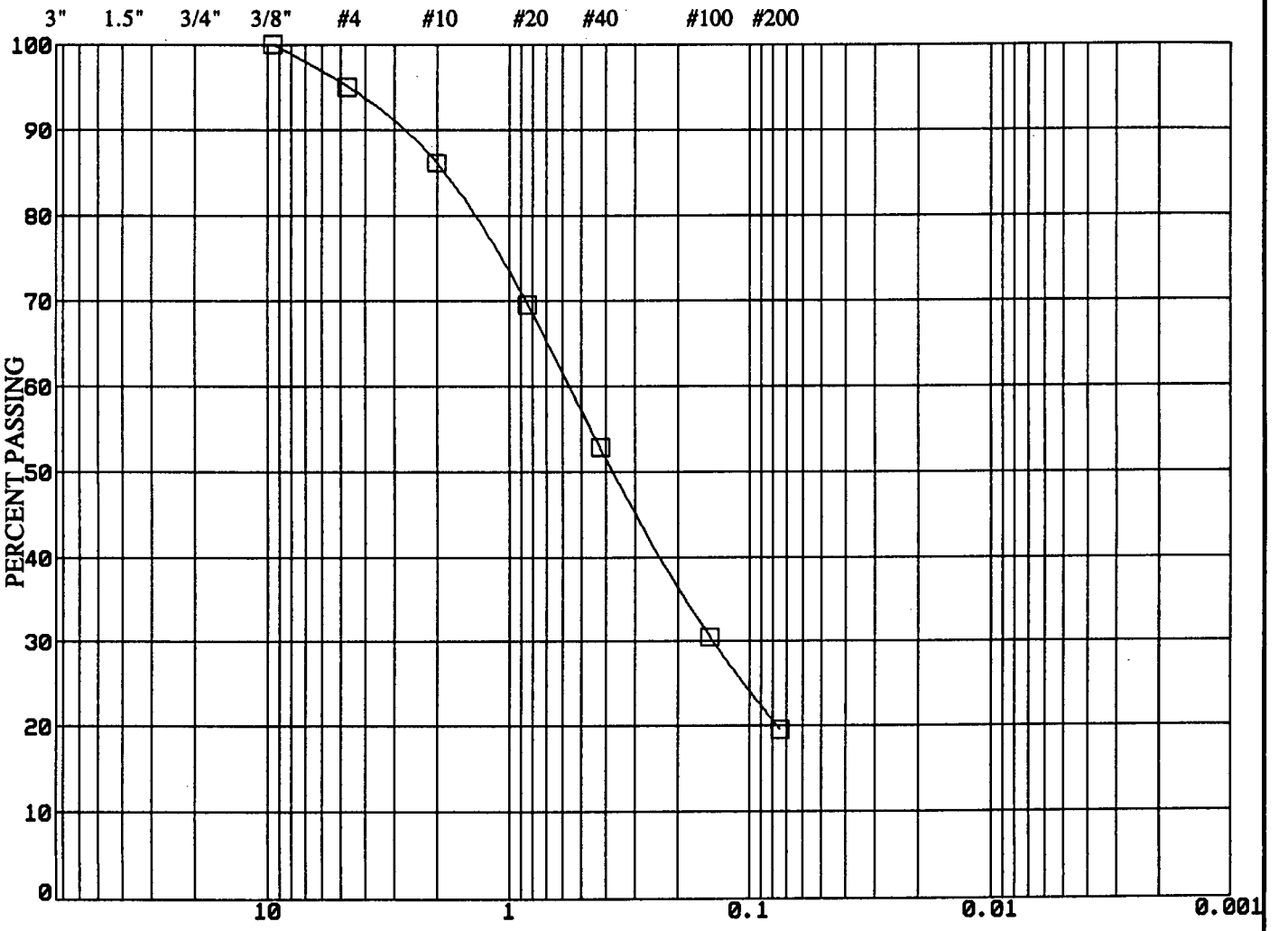
GRAVEL		SAND			SILT	CLAY
coarse	fine	coarse	medium	fine		

Sample: **B-2A @ 35.0 ft.**

GRAIN SIZE DISTRIBUTION

Client: **Rancho Las Flores Limited Partnership**	W.O. **215-1**
Project: **Highway 138**	Location: **San Bernardino County**

SIEVE ANALYSIS | HYDROMETER ANALYSIS



PARTICLE SIZE IN MILLIMETERS

GRAVEL		SAND			SILT	CLAY
coarse	fine	coarse	medium	fine		

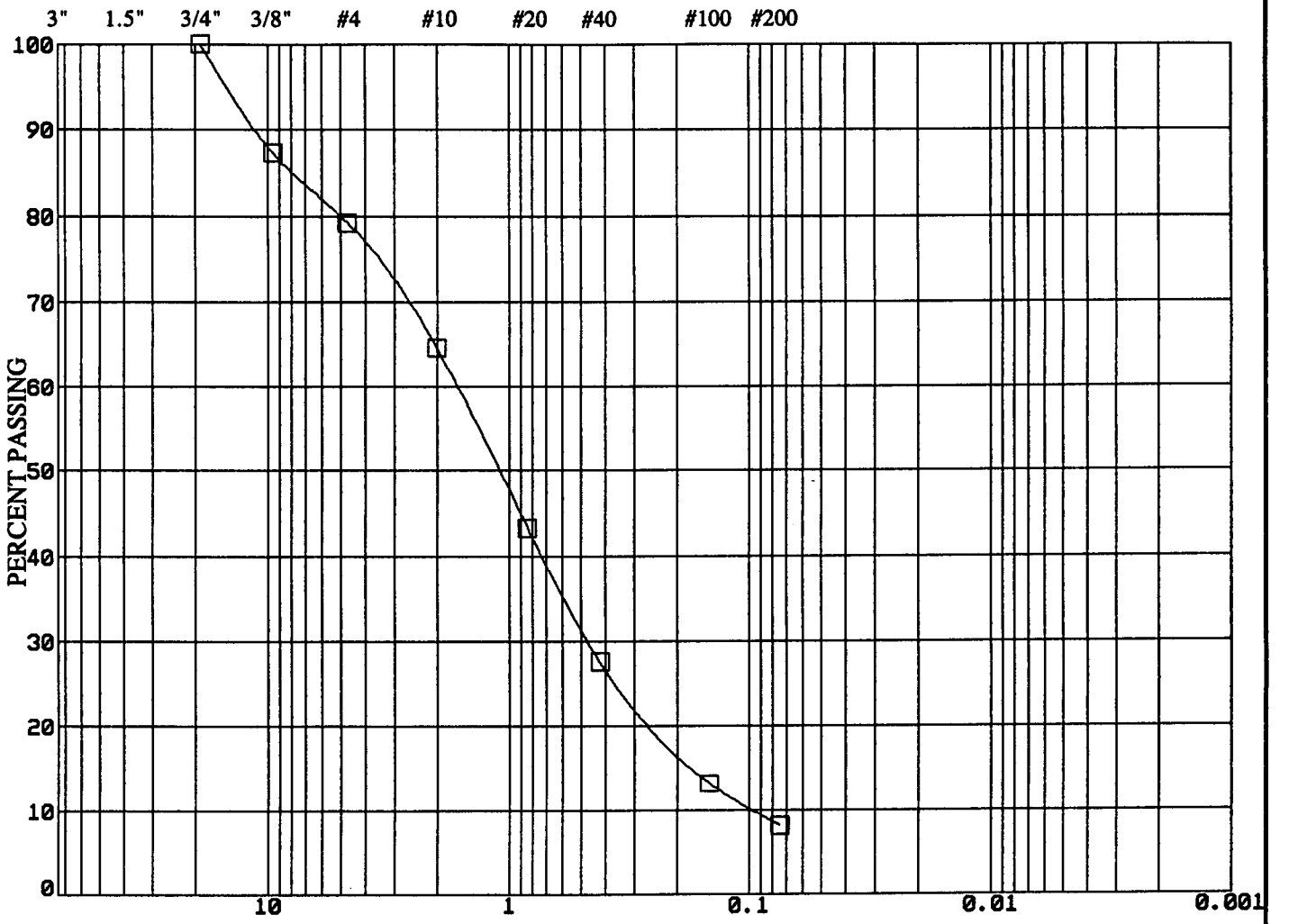
Sample: **B-2A @ 45.0 ft.**

John A. Sayers & Associates GEOTECHNICAL CONSULTANTS	Plate

GRAIN SIZE DISTRIBUTION

Client: **Rancho Las Flores Limited Partnership**	W.O. **215-1**
Project: **Highway 138**	Location: **San Bernardino County**

SIEVE ANALYSIS | HYDROMETER ANALYSIS



PARTICLE SIZE IN MILLIMETERS

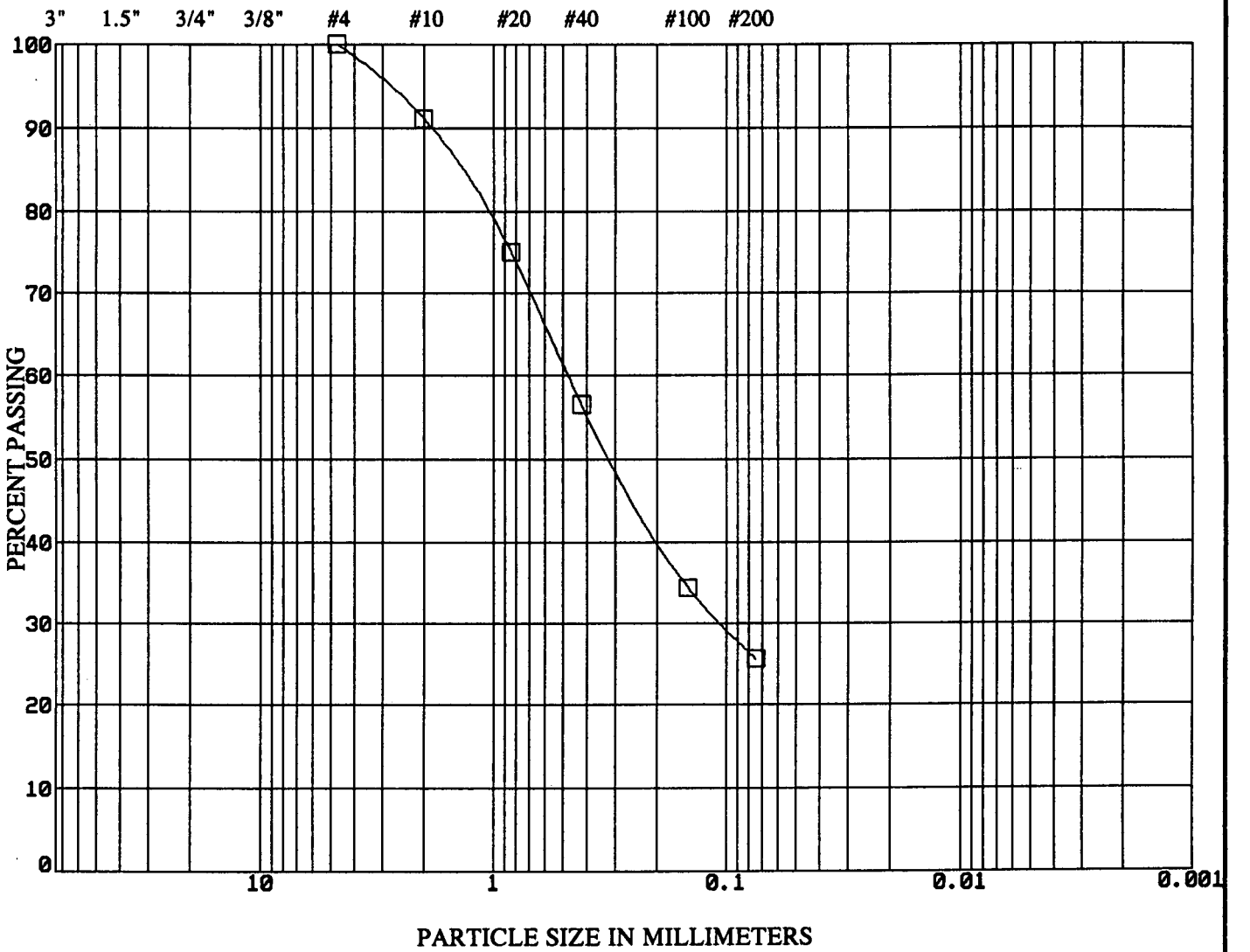
GRAVEL		SAND			SILT	CLAY
coarse	fine	coarse	medium	fine		

Sample: **B-1 @ 8.0 ft.**

GRAIN SIZE DISTRIBUTION

Client: **Rancho Las Flores Limited Partnership**	W.O. **215-1**
Project: **Highway 138**	Location: **San Bernardino County**

SIEVE ANALYSIS | HYDROMETER ANALYSIS



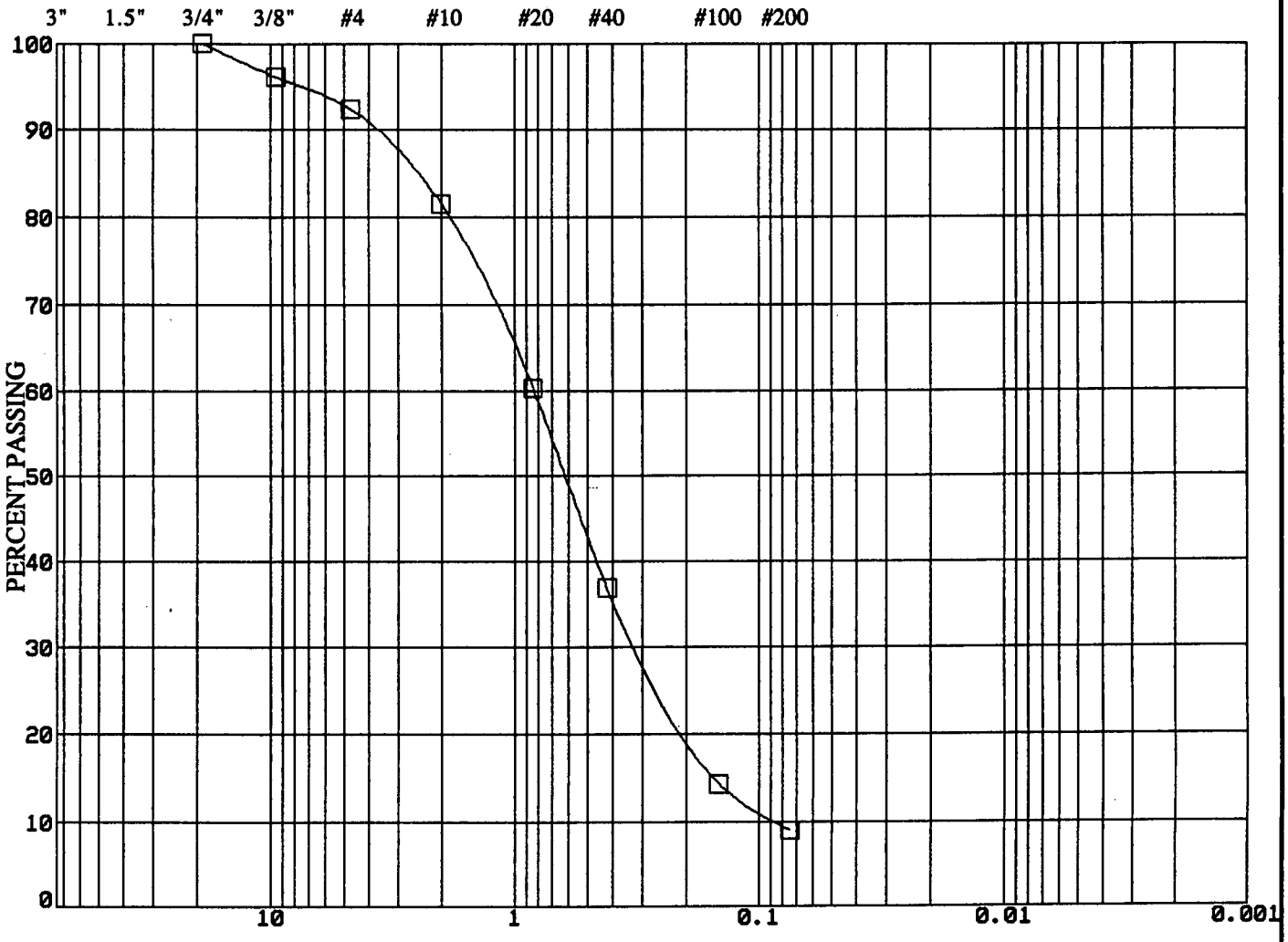
GRAVEL		SAND			SILT	CLAY
coarse	fine	coarse	medium	fine		

Sample: **B-2 @ 2.0 ft.**

GRAIN SIZE DISTRIBUTION

Client: **Rancho Las Flores Limited Partnership**	W.O. **215-1**
Project: **Highway 138**	Location: **San Bernardino County**

SIEVE ANALYSIS | HYDROMETER ANALYSIS



PARTICLE SIZE IN MILLIMETERS

GRAVEL		SAND			SILT	CLAY
coarse	fine	coarse	medium	fine		

Sample: **B-4 @ 5.0 ft.**

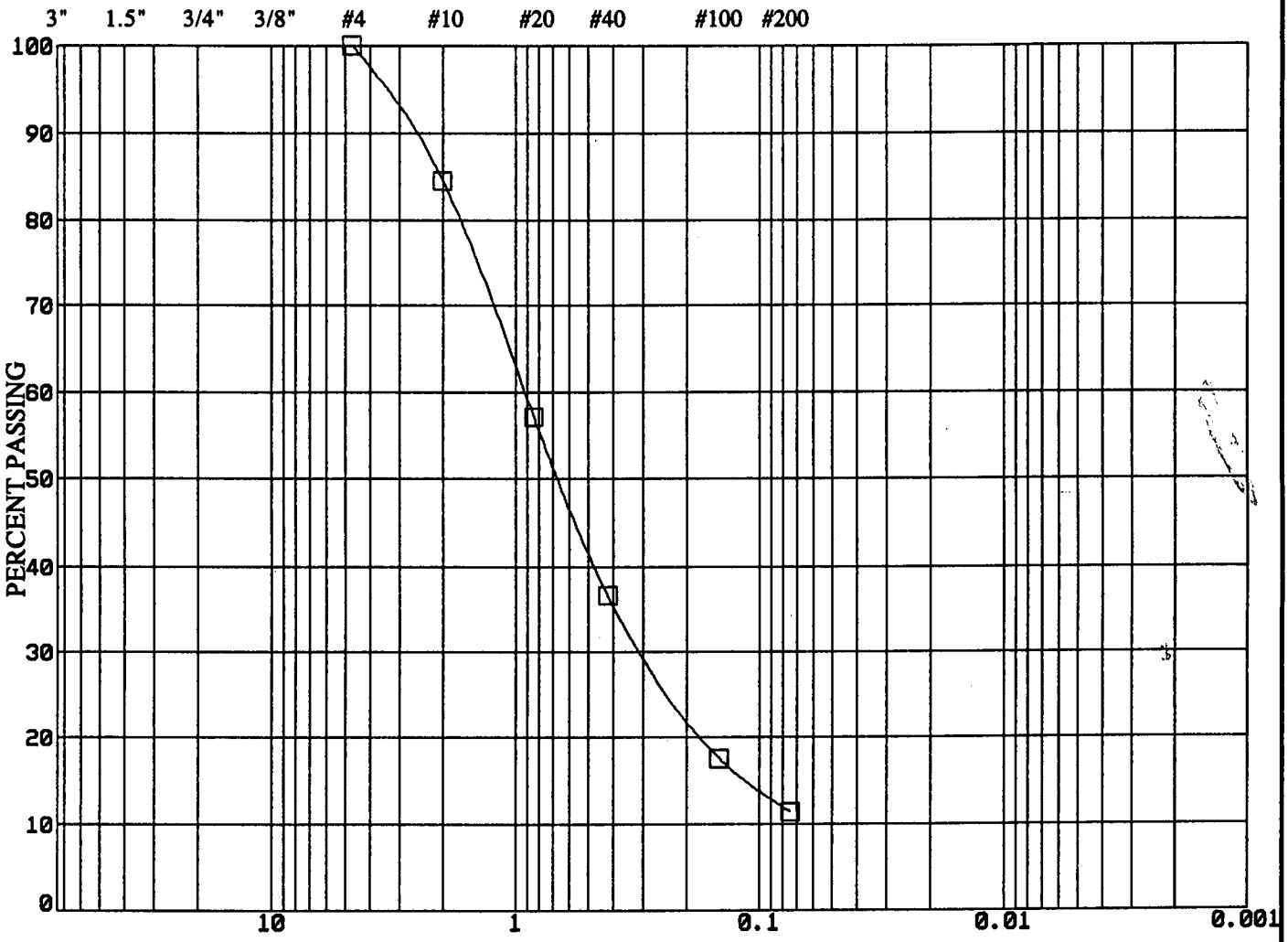
John A. Sayers & Associates
GEOTECHNICAL CONSULTANTS

Plate

GRAIN SIZE DISTRIBUTION

Client: **Rancho Las Flores Limited Partnership**	W.O. **215-1**
Project: **Highway 138**	Location: **San Bernardino County**

SIEVE ANALYSIS | HYDROMETER ANALYSIS



PARTICLE SIZE IN MILLIMETERS

GRAVEL		SAND			SILT	CLAY
coarse	fine	coarse	medium	fine		

Sample: **B-6 @ 6.0 ft.**

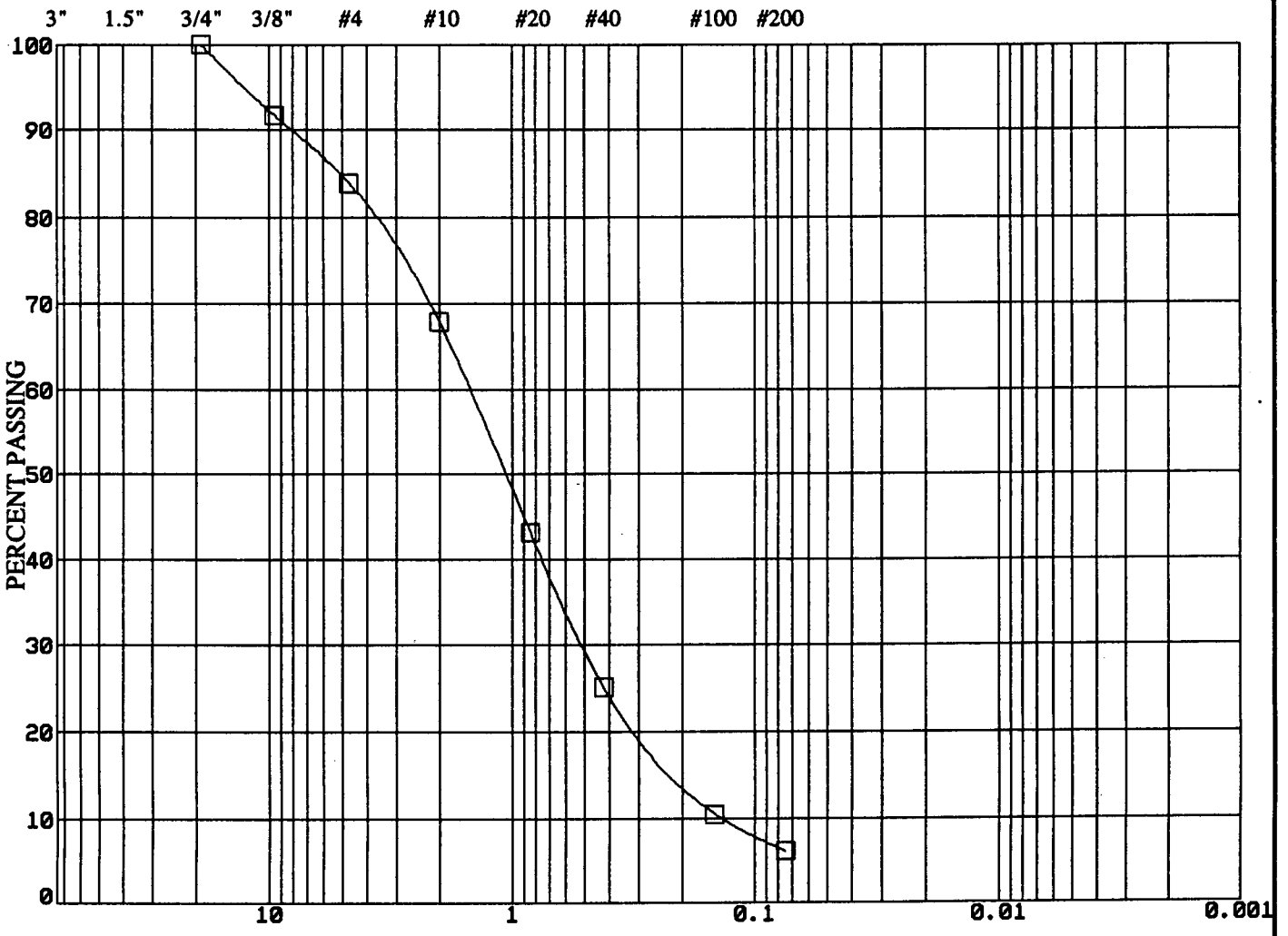
John A. Sayers & Associates
GEOTECHNICAL CONSULTANTS

Plate

GRAIN SIZE DISTRIBUTION

Client: **Rancho Las Flores Limited Partnership**	W.O. **215-1**
Project: **Highway 138**	Location: **San Bernardino County**

SIEVE ANALYSIS | HYDROMETER ANALYSIS



PARTICLE SIZE IN MILLIMETERS

GRAVEL		SAND			SILT	CLAY
coarse	fine	coarse	medium	fine		

Sample: **B-9 @ 2.5 ft.**

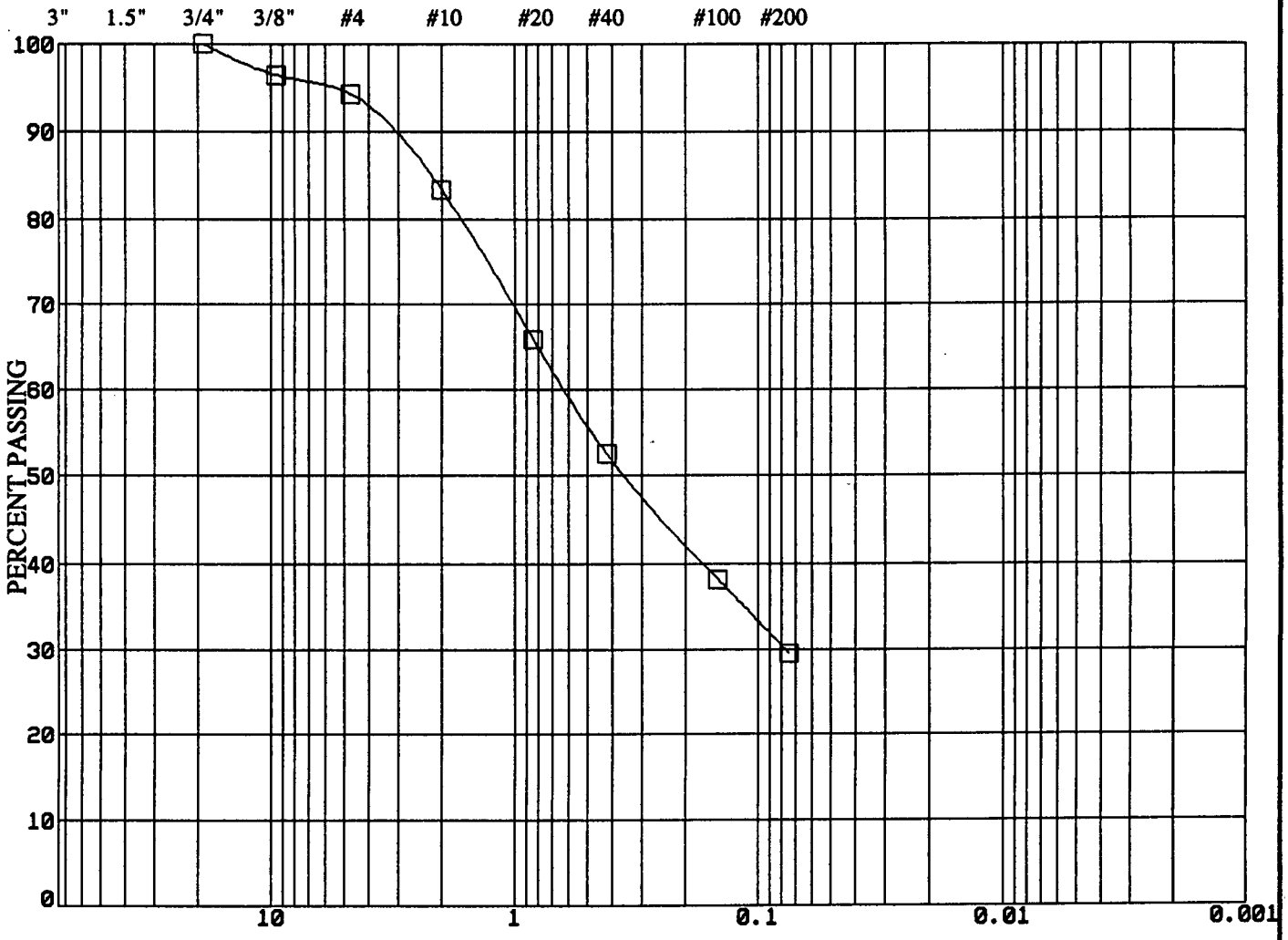
John A. Sayers & Associates
GEOTECHNICAL CONSULTANTS

Plate

GRAIN SIZE DISTRIBUTION

Client: **Rancho Las Flores Limited Partnership**	W.O. **215-1**
Project: **Highway 138**	Location: **San Bernardino County**

SIEVE ANALYSIS | HYDROMETER ANALYSIS



PARTICLE SIZE IN MILLIMETERS

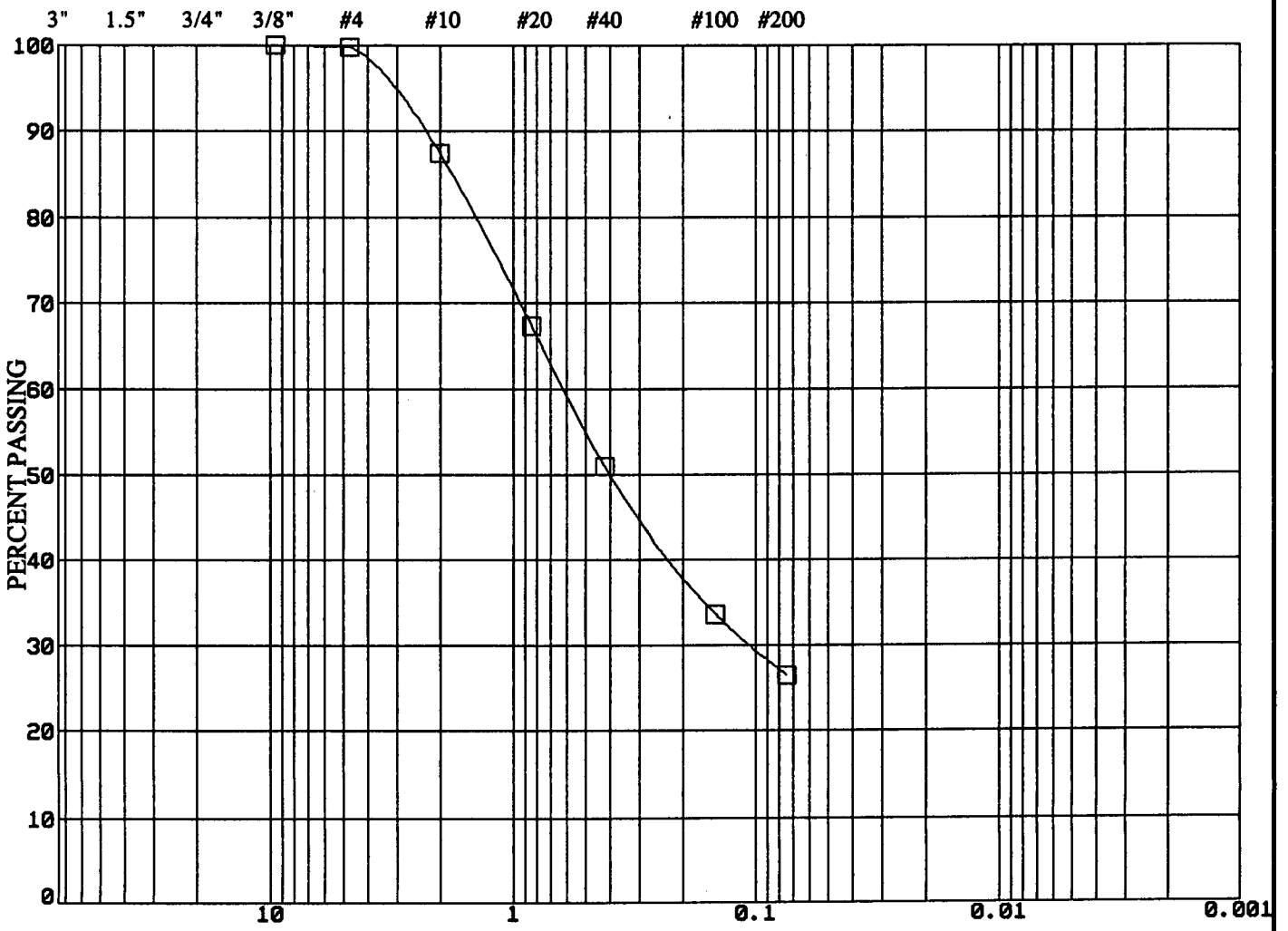
GRAVEL		SAND			SILT	CLAY
coarse	fine	coarse	medium	fine		

Sample: **B-10 @ 3.0 ft.**

GRAIN SIZE DISTRIBUTION

Client: **Rancho Las Flores Limited Partnership**	W.O. **215-1**
Project: **Highway 138**	Location: **San Bernardino County**

SIEVE ANALYSIS | HYDROMETER ANALYSIS



PARTICLE SIZE IN MILLIMETERS

GRAVEL		SAND			SILT	CLAY
coarse	fine	coarse	medium	fine		

Sample: **B-13 @ 15.0 ft.**

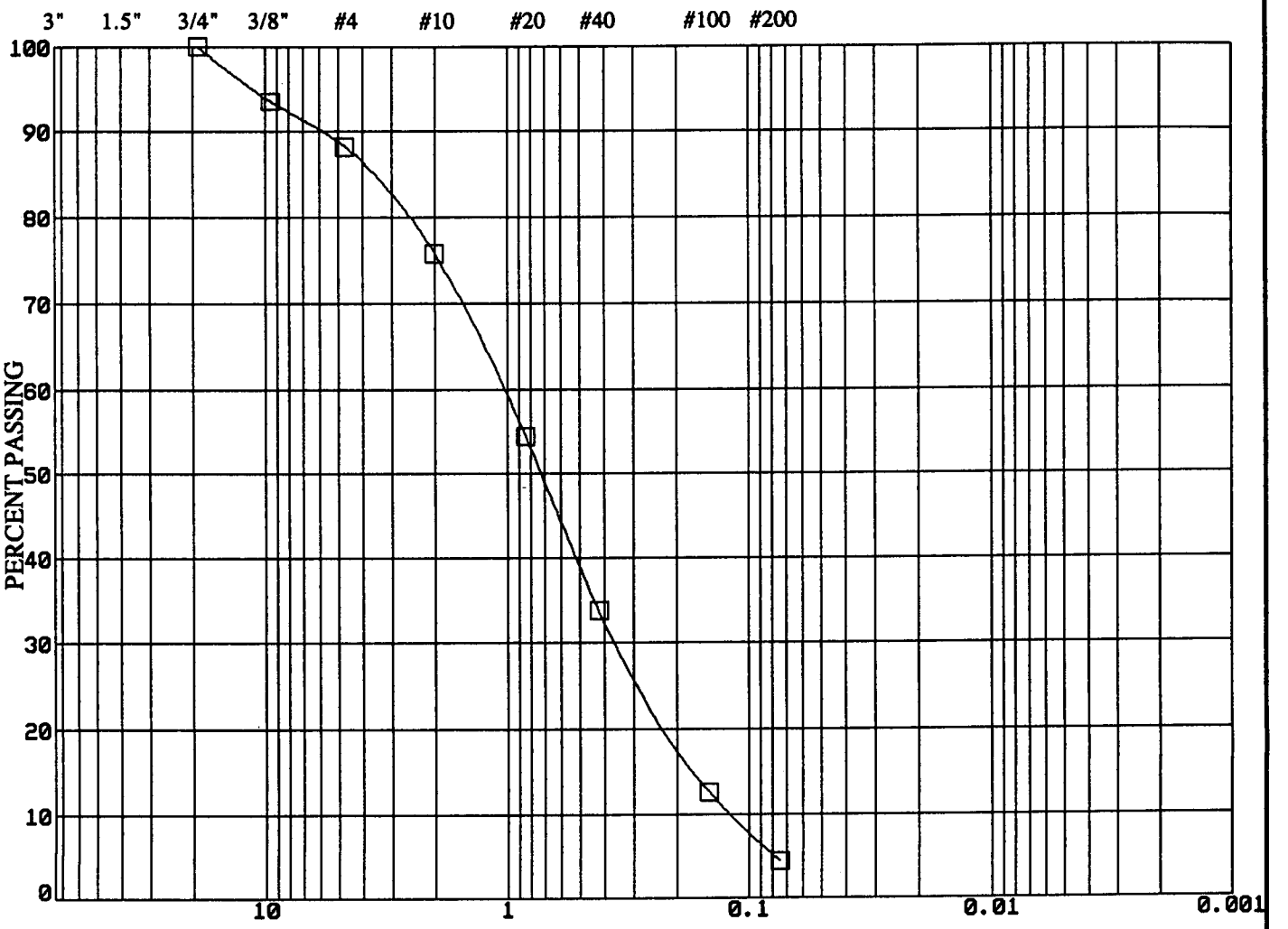
John A. Sayers & Associates
GEOTECHNICAL CONSULTANTS

Plate

GRAIN SIZE DISTRIBUTION

Client: **Rancho Las Flores Limited Partnership**	W.O. **215-1**
Project: **Highway 138**	Location: **San Bernardino County**

SIEVE ANALYSIS | HYDROMETER ANALYSIS



PARTICLE SIZE IN MILLIMETERS

GRAVEL		SAND			SILT	CLAY
coarse	fine	coarse	medium	fine		

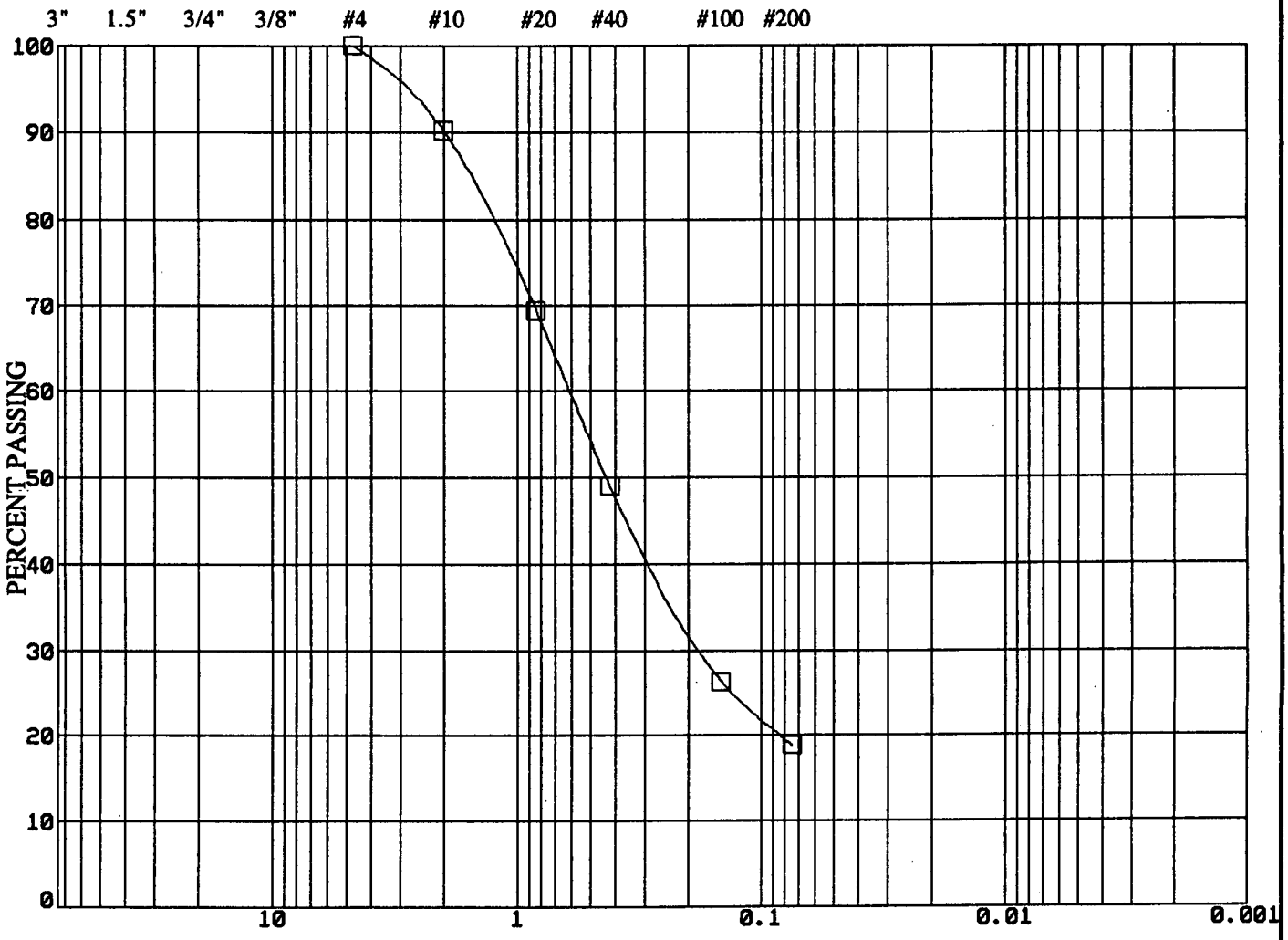
Sample: **B-14 @ 2.5 ft.**

John A. Sayers & Associates GEOTECHNICAL CONSULTANTS	Plate

GRAIN SIZE DISTRIBUTION

Client: **Rancho Las Flores Limited Partnership**	W.O. **215-1**
Project: **Highway 138**	Location: **San Bernardino County**

SIEVE ANALYSIS | HYDROMETER ANALYSIS



PARTICLE SIZE IN MILLIMETERS

GRAVEL		SAND			SILT	CLAY
coarse	fine	coarse	medium	fine		

Sample: **B-16 @ 3.0 ft.**

John A. Sayers & Associates GEOTECHNICAL CONSULTANTS	Plate

APPENDIX B

SHEAR TEST DATA UNDISTURBED SAMPLES SHEARED AT NATURAL MOISTURE CONTENT

CONTENTS:

Shear Test Summary
Shear Test Diagram Summary
Individual Shear Test Diagrams with Stress-Strain Diagrams

JOHN A. SAYERS & ASSOCIATES

SHEAR TEST SUMMARY

Undisturbed, Tested at Natural Moisture Content

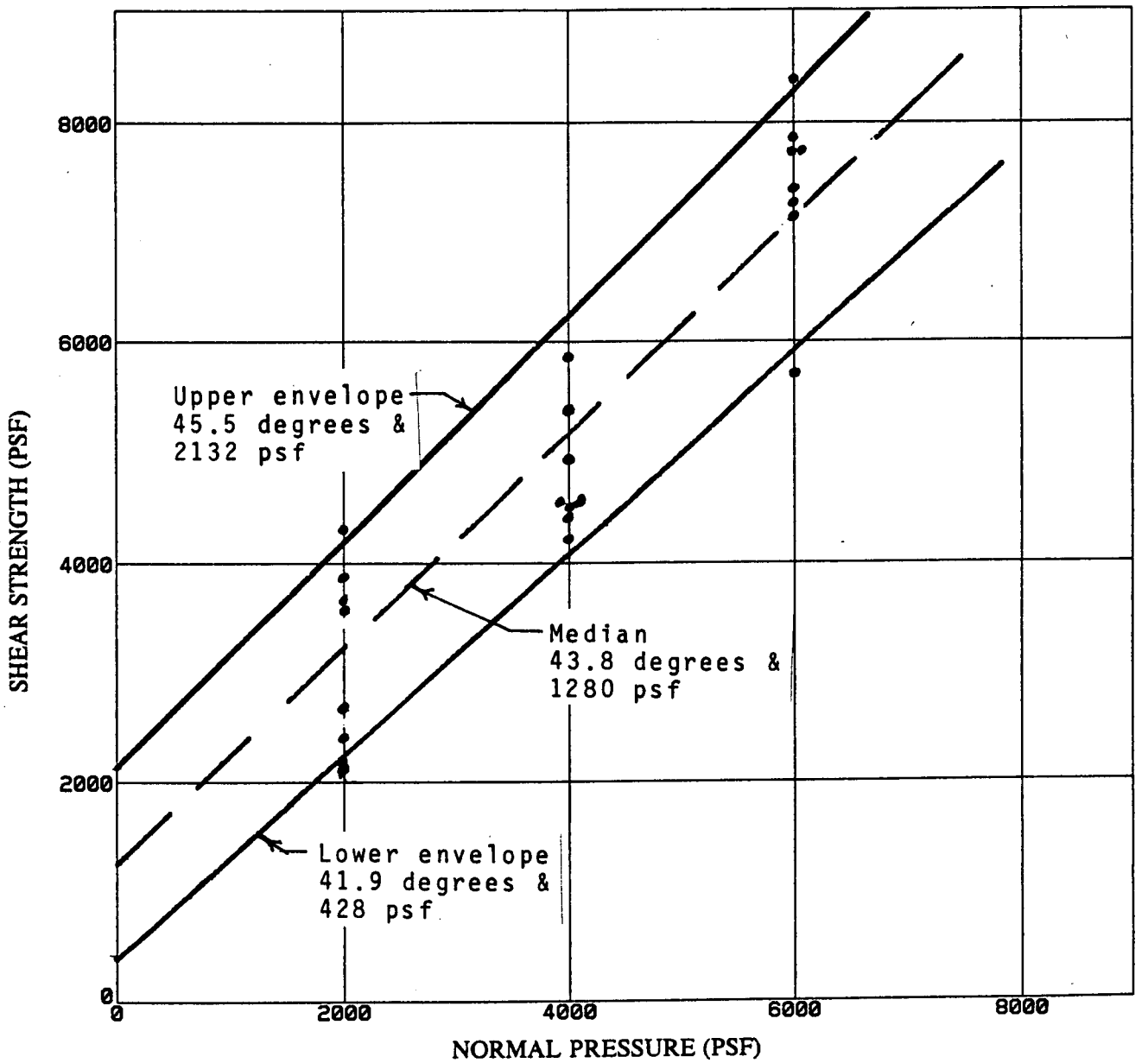
Sample Location		Normal Pressure (psf) 2000	4000	6000
B-1A @ 10'	Shear Strength (psf)	2172	4560	7404
B-1A @ 20'		3864	4956	7752
B-1A @ 40'		3576	4428	5736 (Low)
B-2A @ 10'		4332 (High)	5364	8412 (High)
B-2A @ 15'		3684	5892 (High)	7908
B-2A @ 20'		2436	4548	7164
B-2A @ 35'		2136 (Low)	4512	7752
B-2A @ 45'		2676	4212 (Low)	7272
Median		3234	5052	7074

Upper envelope (1)	45.5 degrees and 2132 psf
Lower envelope (1)	41.9 degrees and 428 psf
Median of envelopes (2)	43.8 degrees and 1280 psf

(1) Calculated by computer using low and high shear strengths
(2) Calculated by computer as mathematical average of high and low shear strengths

SHEAR TEST DIAGRAM

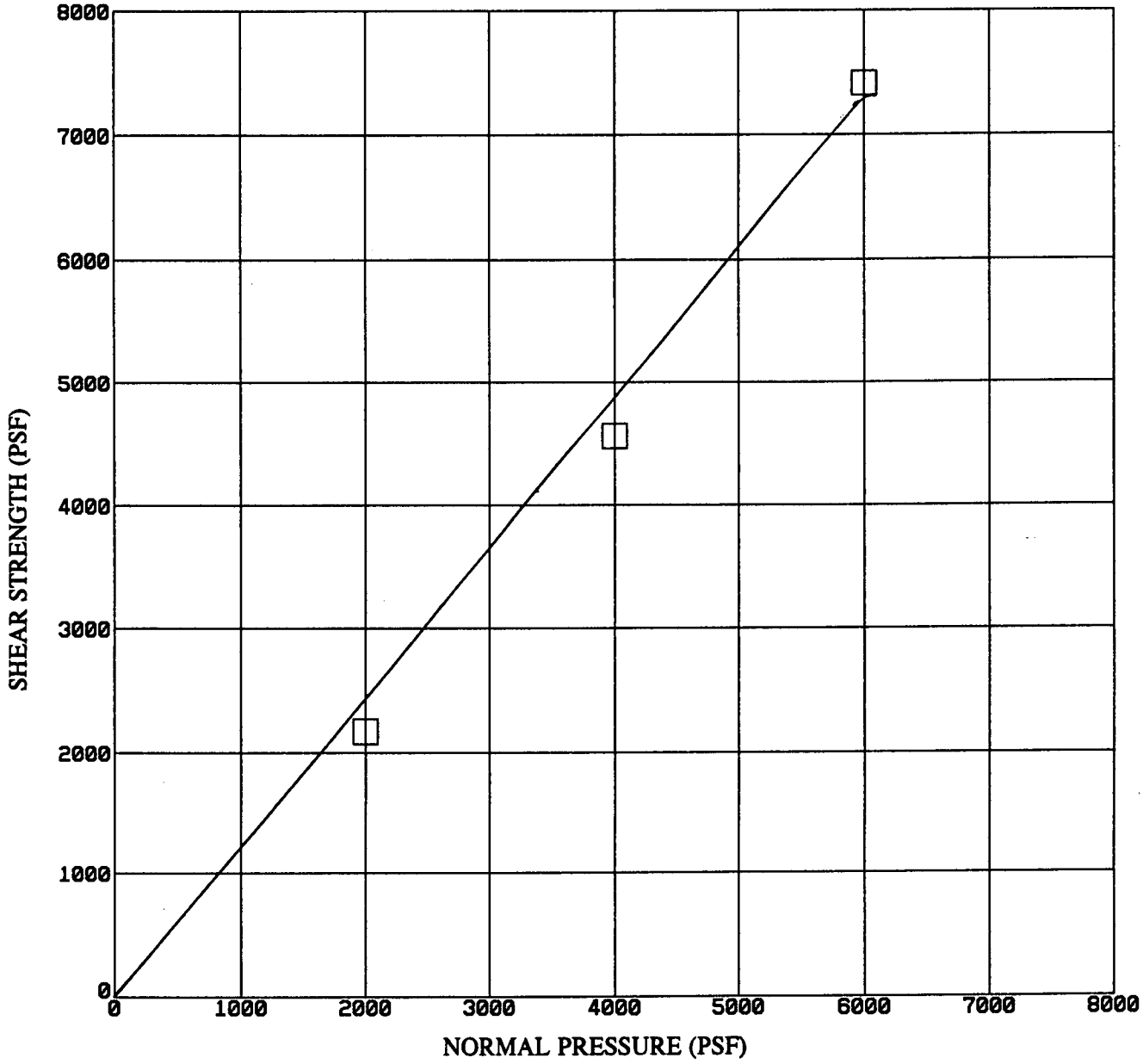
Client: Rancho Las Flores Limited Partnership	W.O. 215-1
Project: Highway 138	Location: San Bernardino County



SHEAR TEST Summary
Undisturbed Samples
Sheared at Natural Moisture

SHEAR TEST DIAGRAM

Client: **Rancho Las Flores Limited Partnership**	W.O. **215-1**
Project: **Highway 138**	Location: **San Bernardino County**



Initial Sample

Dry Density = 126.1 pcf
Moist. Cont. = 8.1%
Silty Sand
L.L. = 27.2
P.I. = 5.9

Sample: **B-1A @ 10.0 ft.**

Phi Angle: **50 deg.**
Cohesion: **0 psf**

TEST METHOD: Undisturbed, shear at natural moisture

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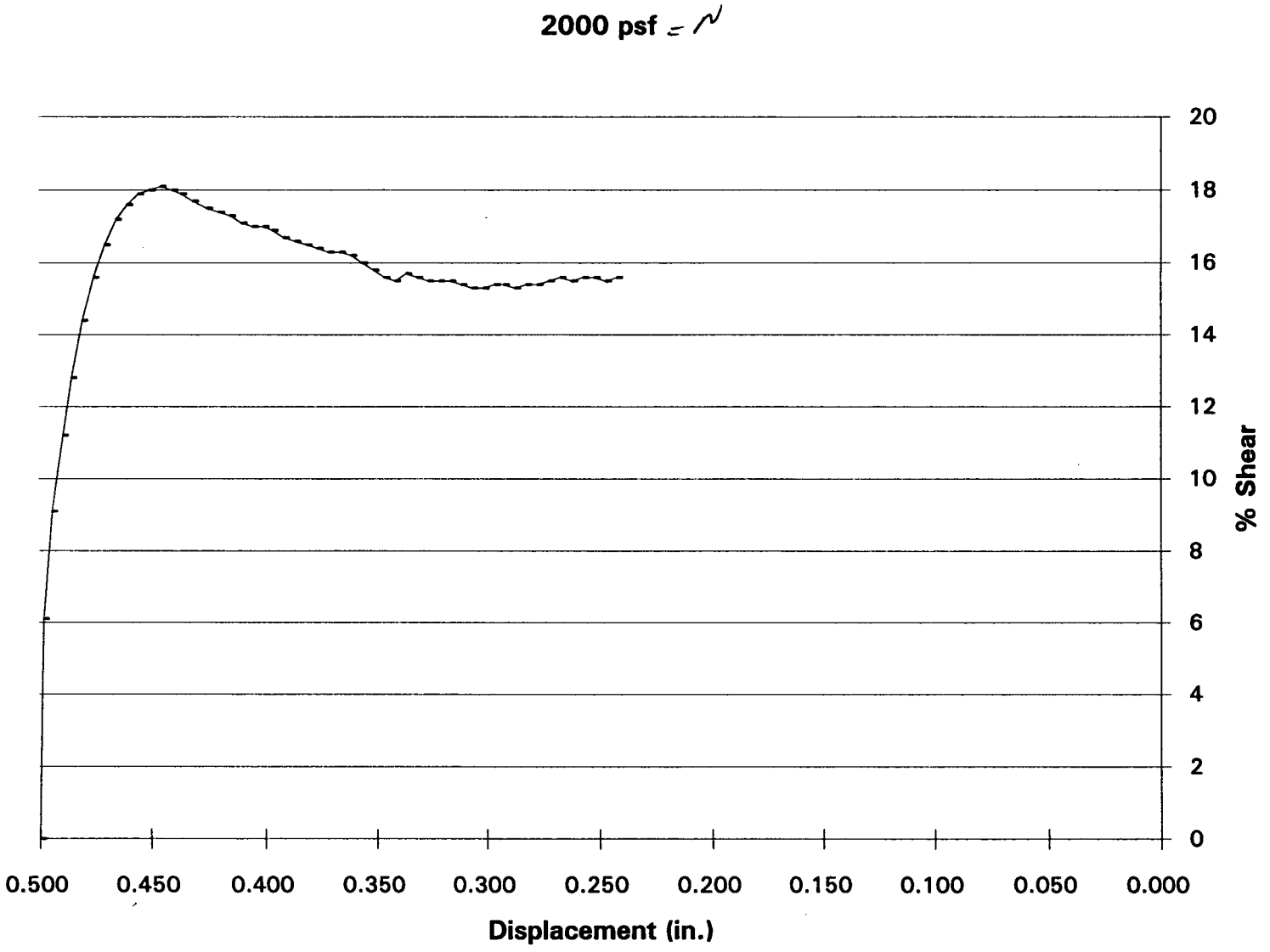
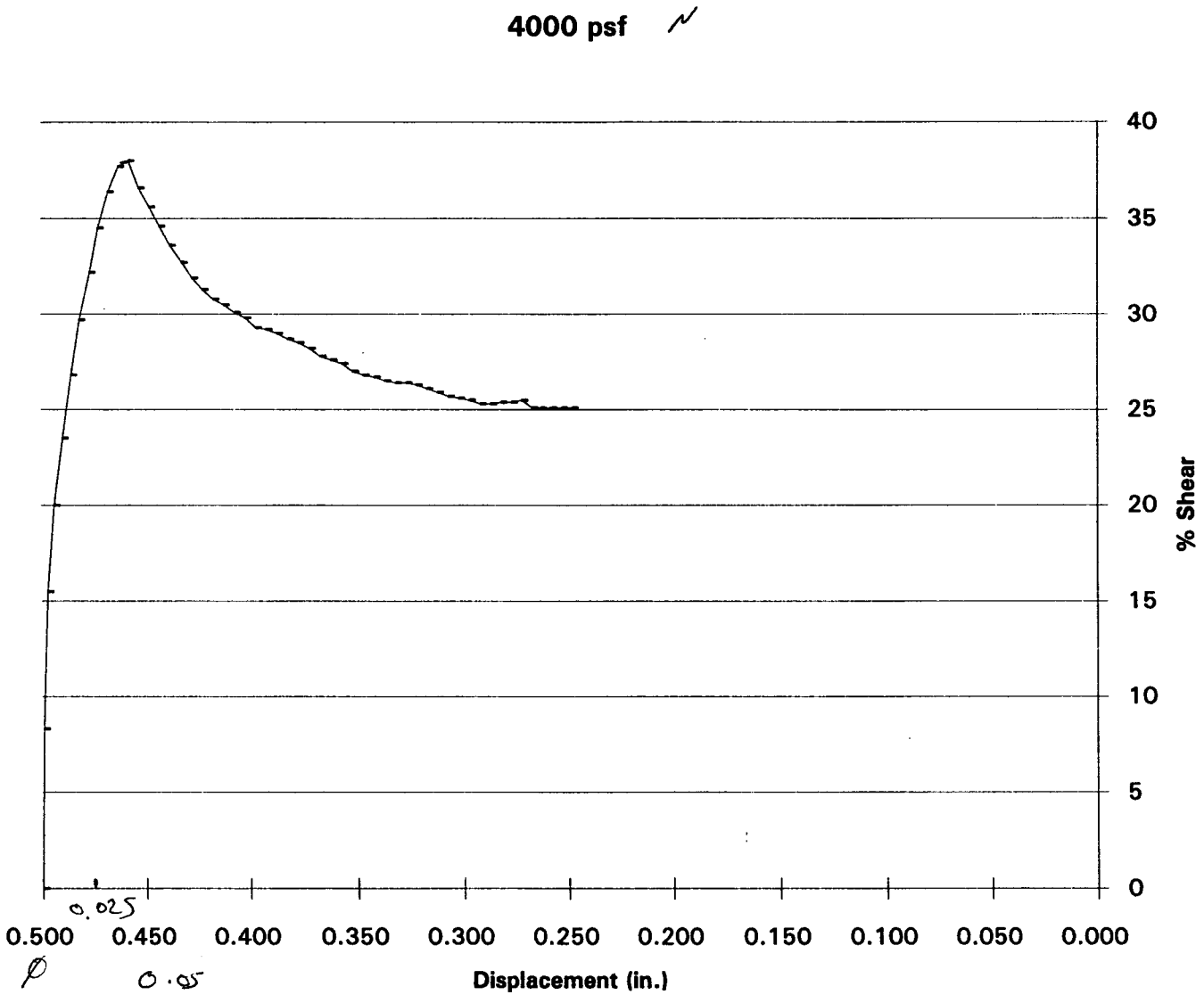
Plate

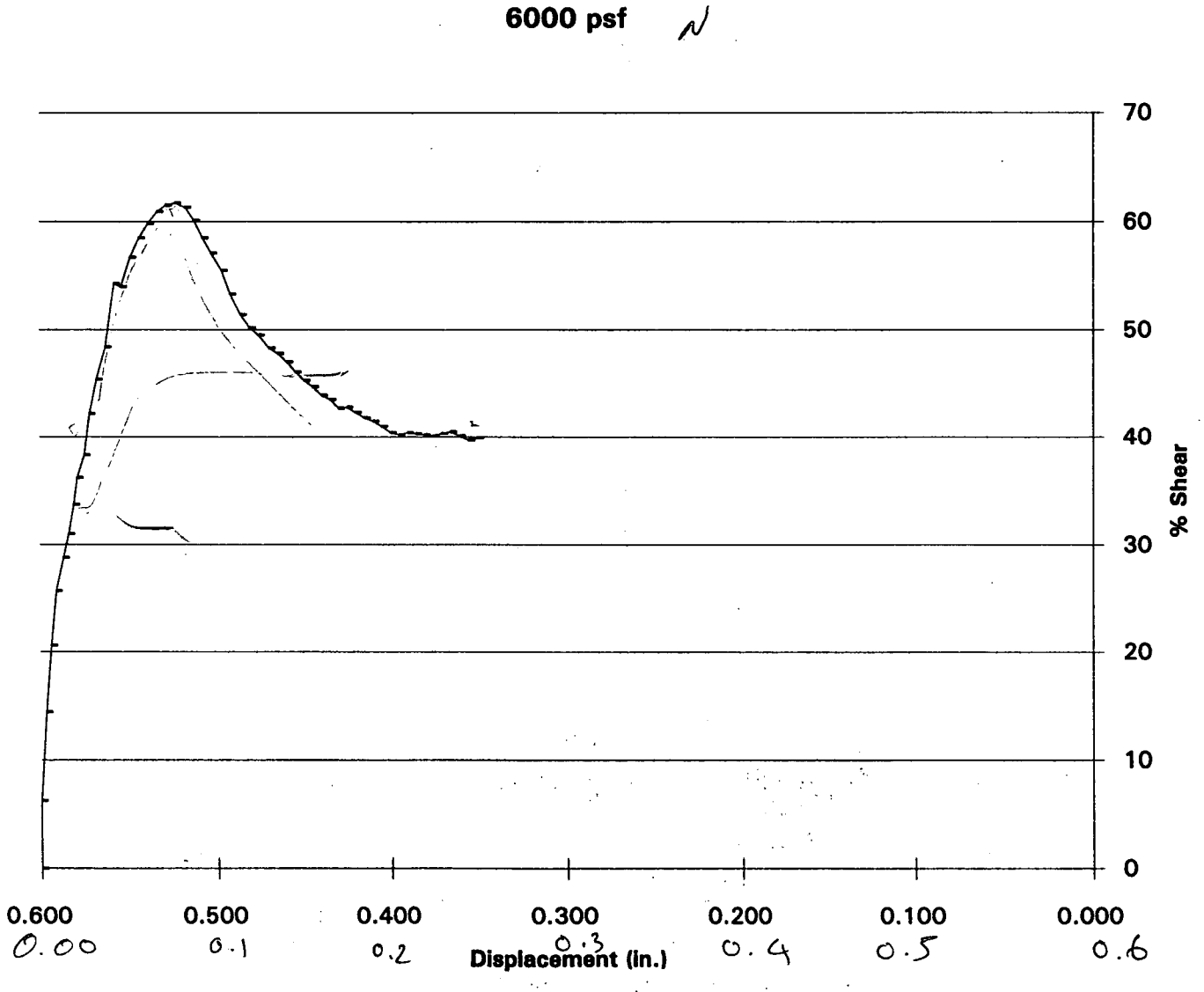
DIRECT SHEAR TEST PLOT OF B1A @ 10 FT.



DIRECT SHEAR TEST PLOT OF B1A @ 10 FT.

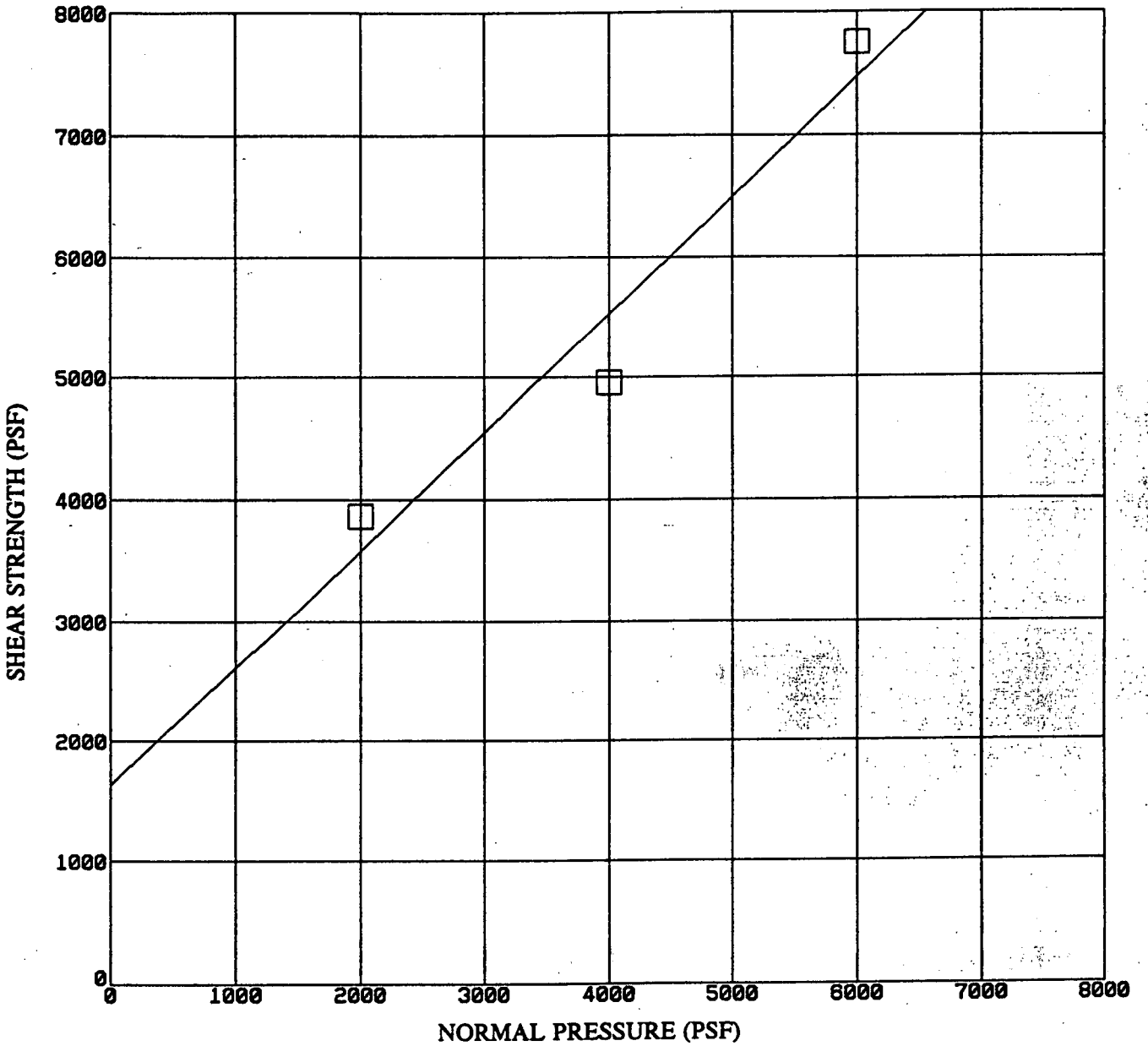


DIRECT SHEAR TEST PLOT OF B1A @ 10 FT.



SHEAR TEST DIAGRAM

Client: Rancho Las Flores Limited Partnership	W.O. 215-1
Project: Highway 138	Location: San Bernardino County



Initial Sample

Dry Density = 127.4 pcf
Moist. Cont. = 10.4%
Silty Sand
L.L. = 24.6
P.I. = 2.3

Sample: B-1A @ 20.0 ft.

Phi Angle: 44 deg.
Cohesion: 1636 psf

TEST CONDITION: Undisturbed, sheared at natural moisture

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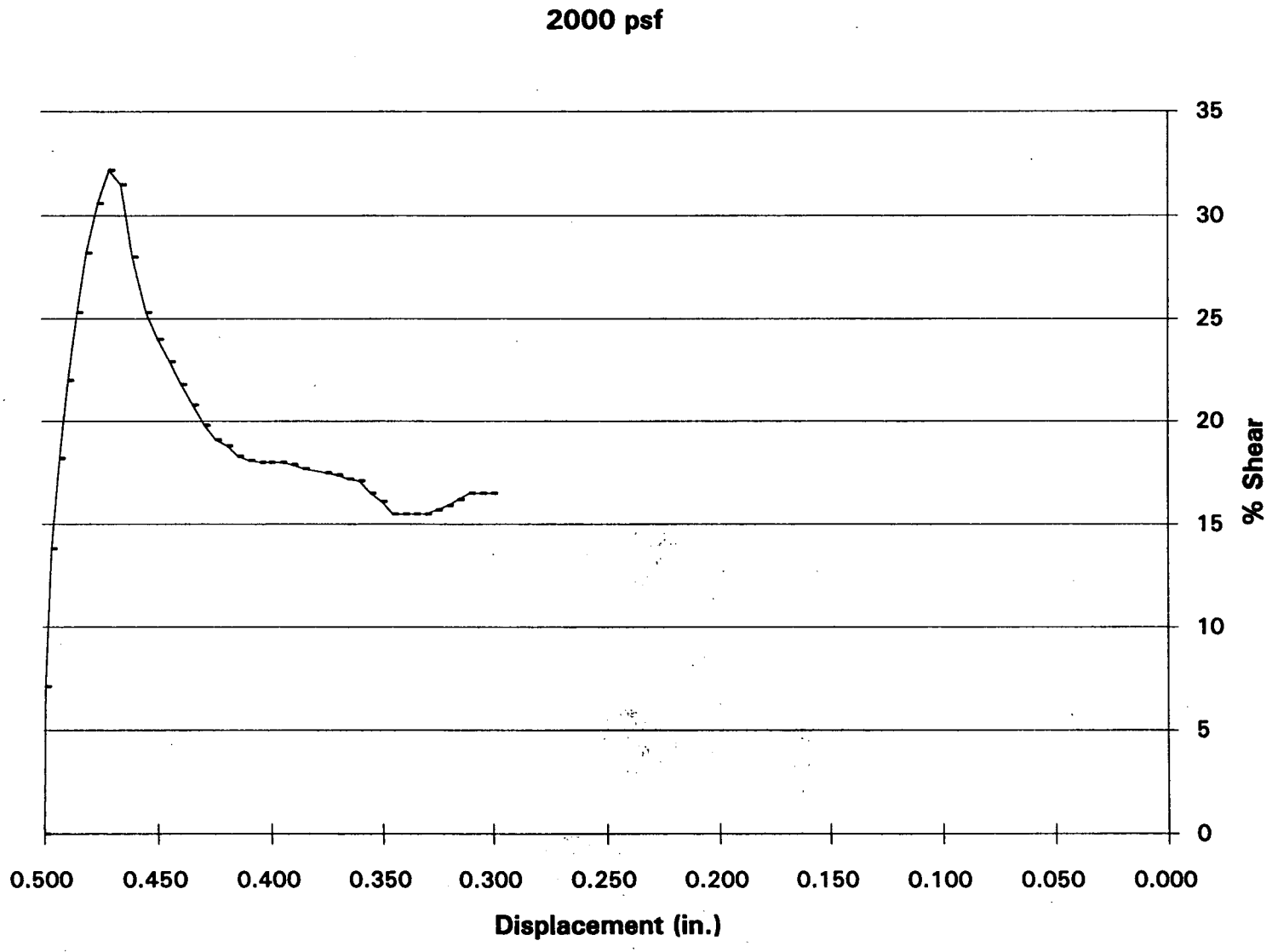
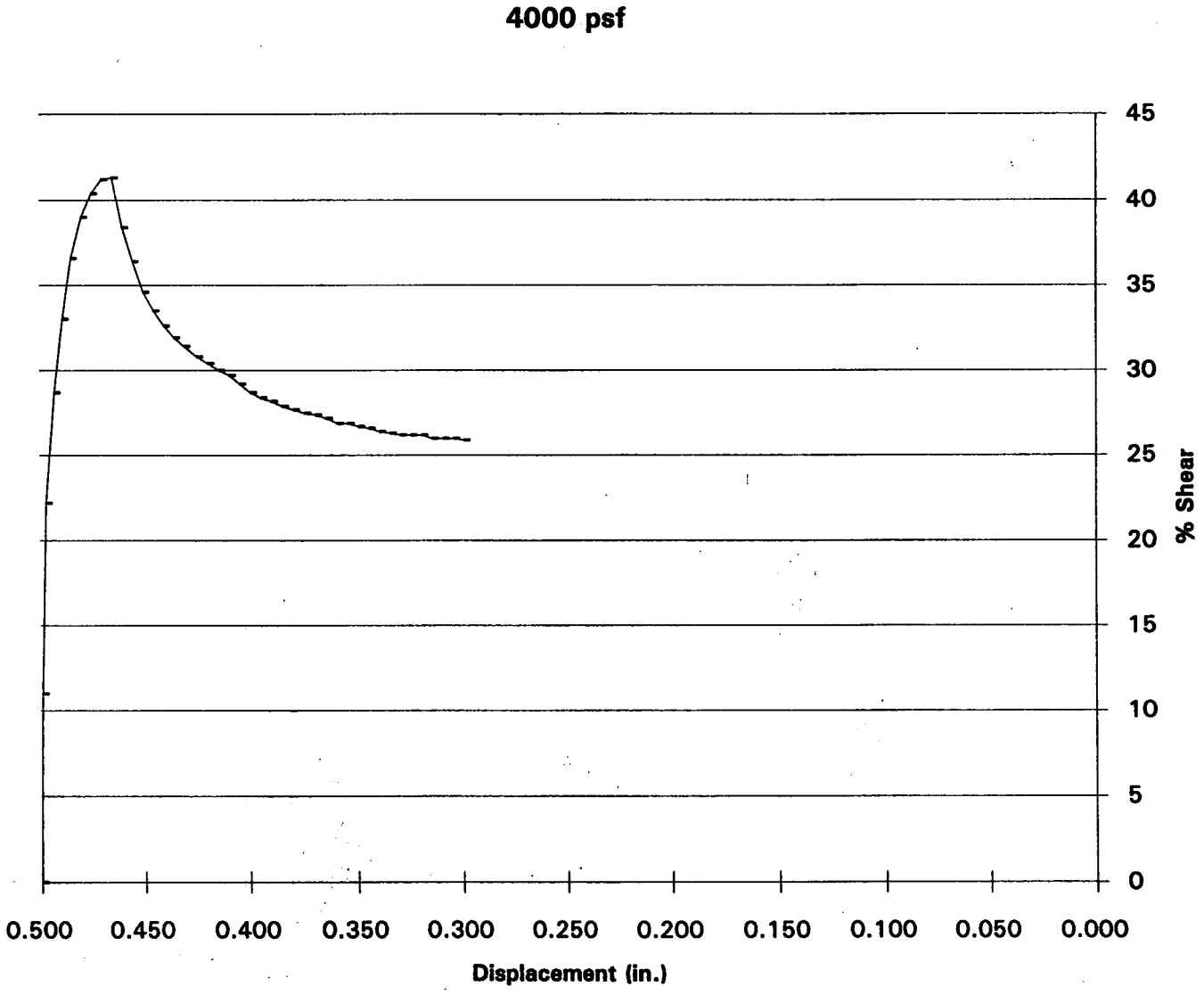
Plate

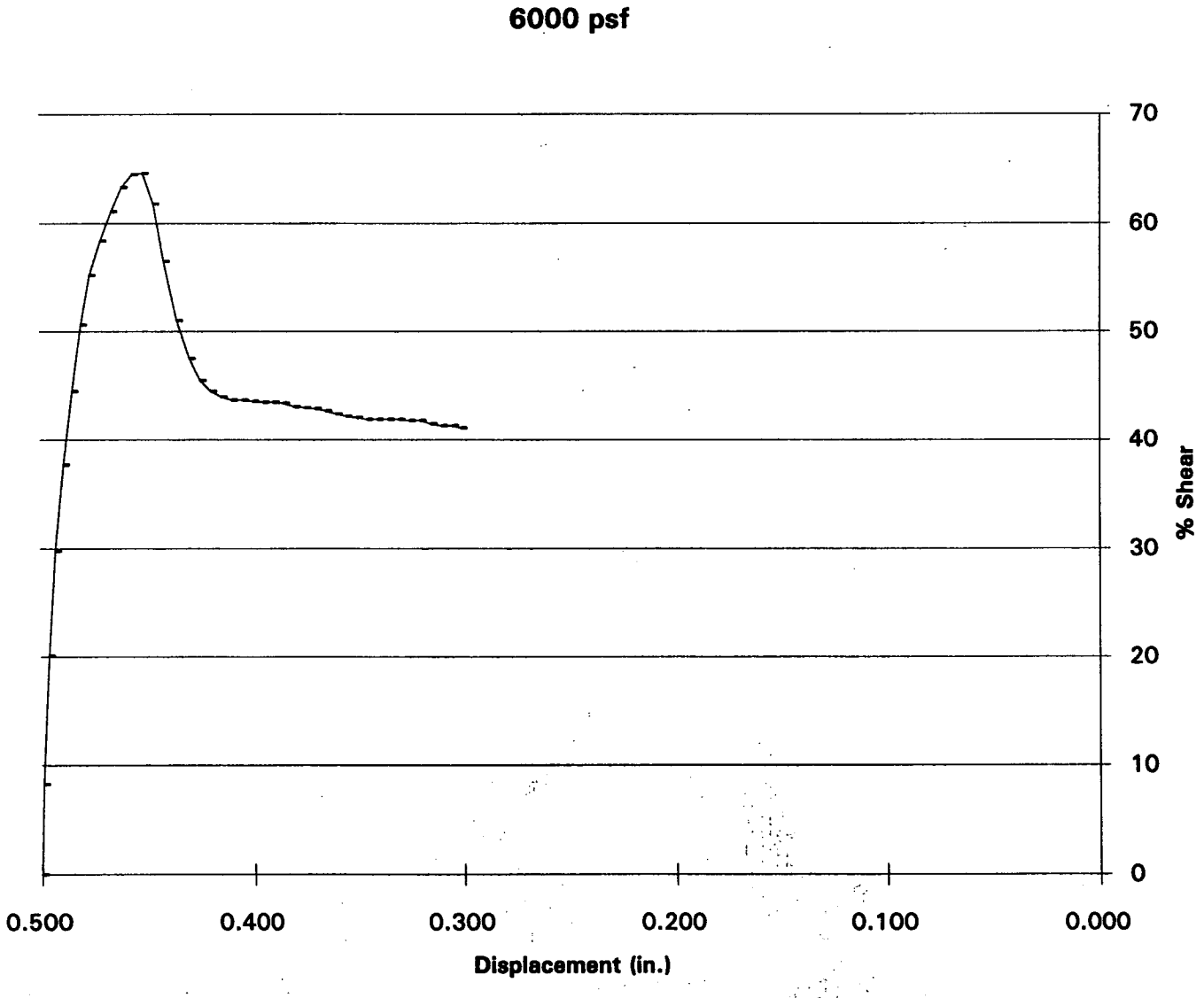
DIRECT SHEAR TEST PLOT OF B1A @ 20 FT.



DIRECT SHEAR TEST PLOT OF B1A @ 20 FT.

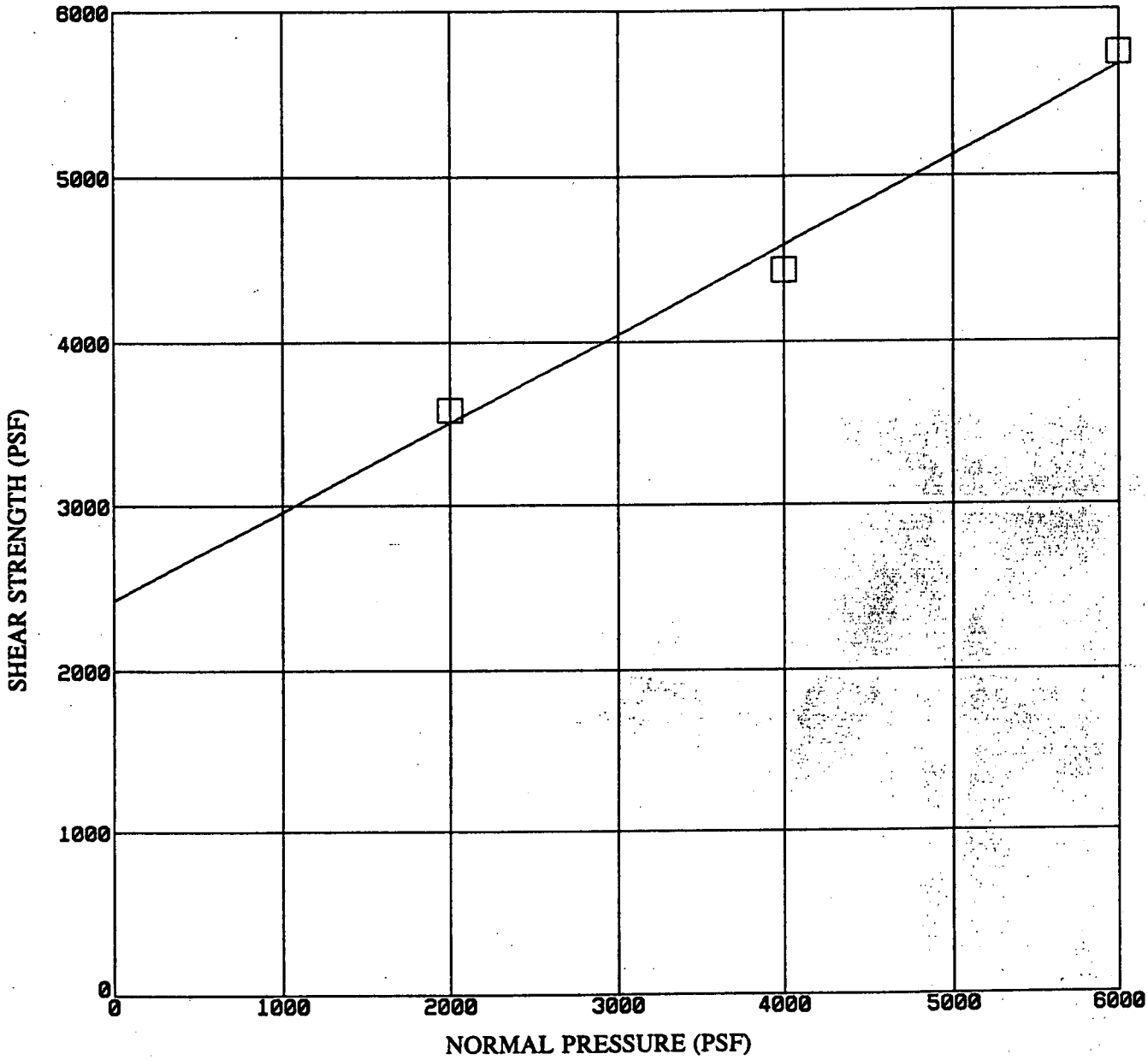


DIRECT SHEAR TEST PLOT OF B1A @ 20 FT.



SHEAR TEST DIAGRAM

Client: Rancho Las Flores Limited Partnership	W.O. 215-1
Project: Highway 138	Location: San Bernardino County



Initial Sample

Dry Density = 124.5 pcf
Moist. Cont. = 4.7%

Sample: **B-1A @ 40.0 ft.**

Phi Angle: **28 deg.**

Cohesion: **2420 psf**

TEST CONDITION: Undisturbed, sheared at natural moisture

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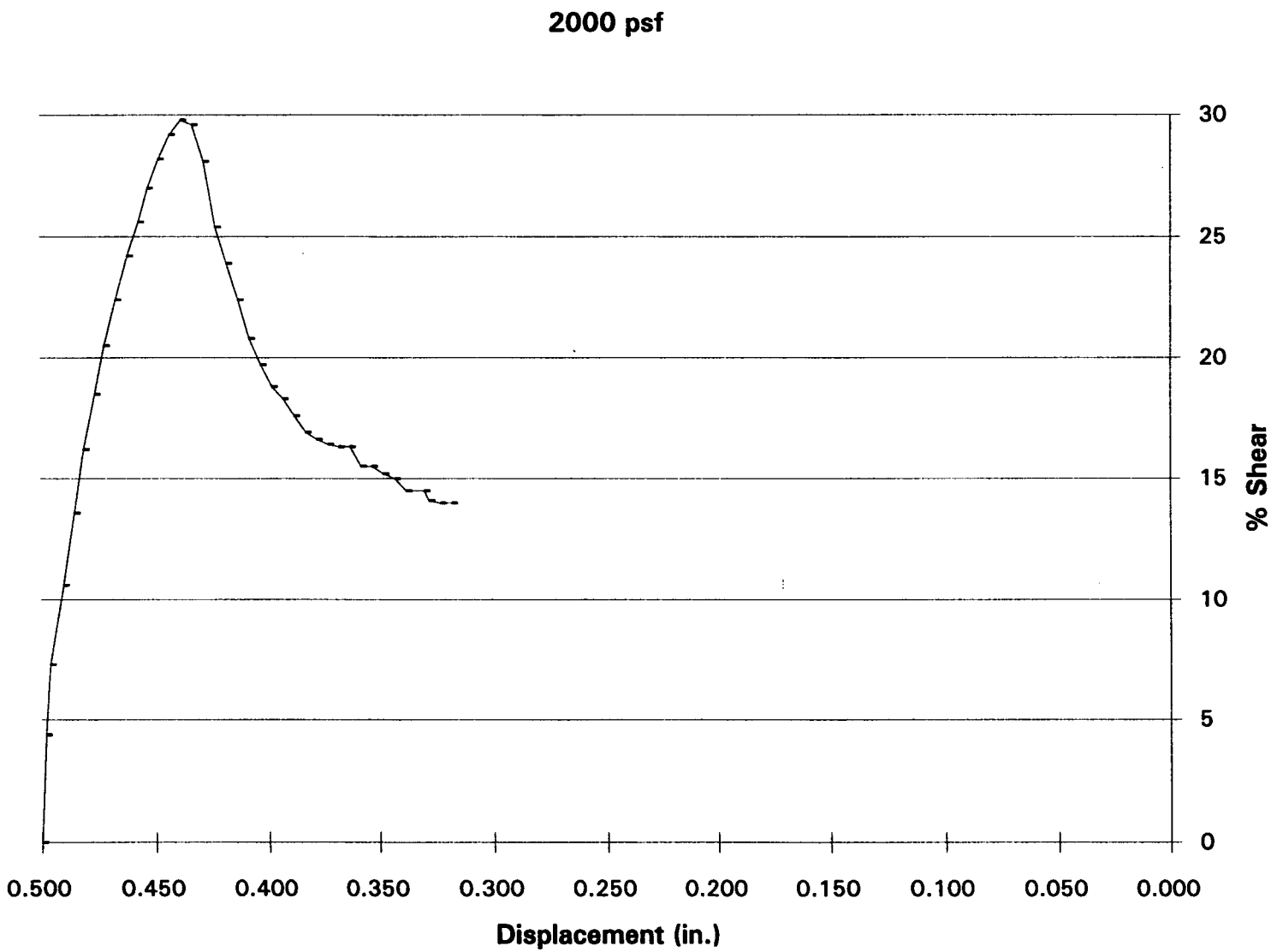
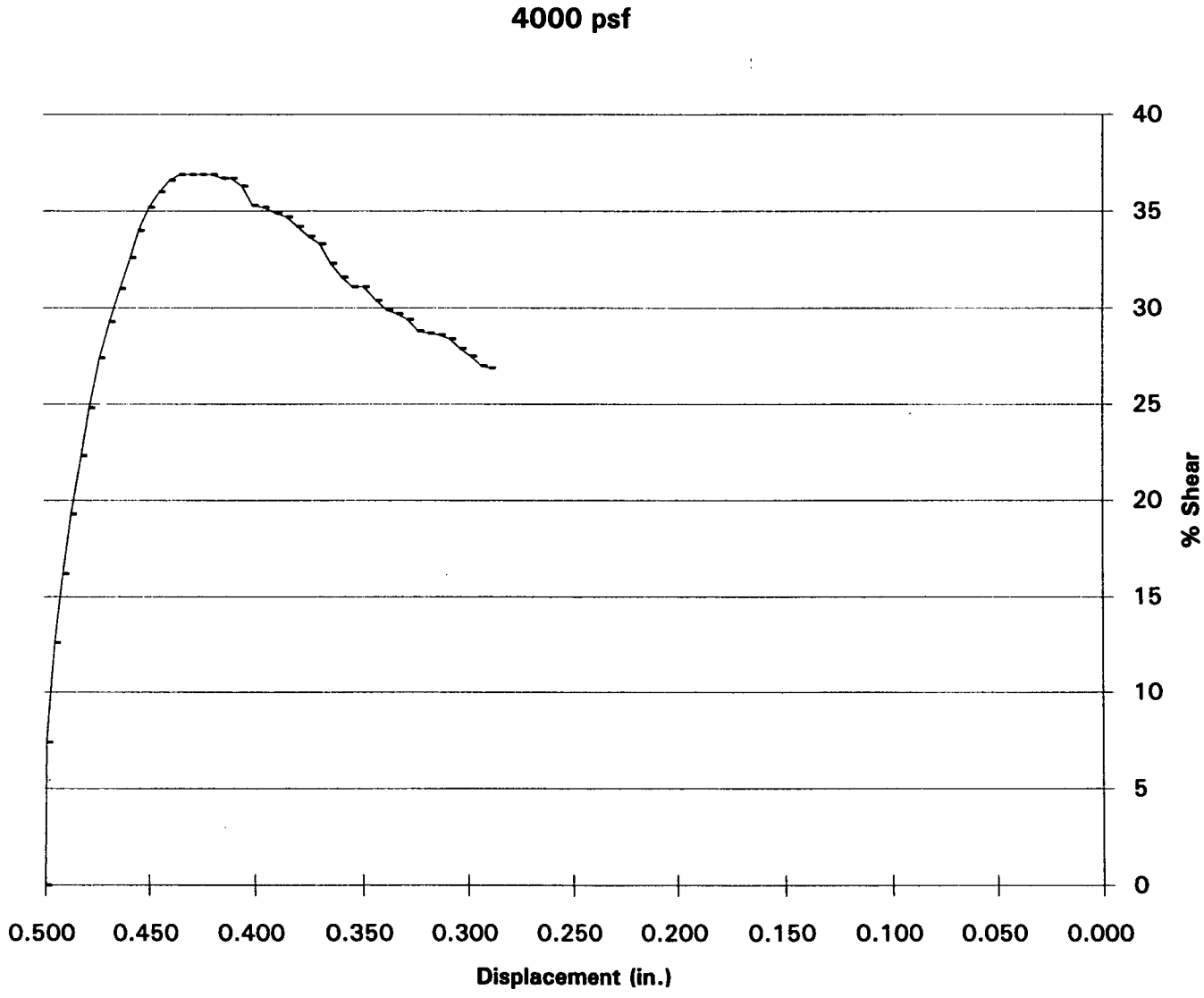
Plate

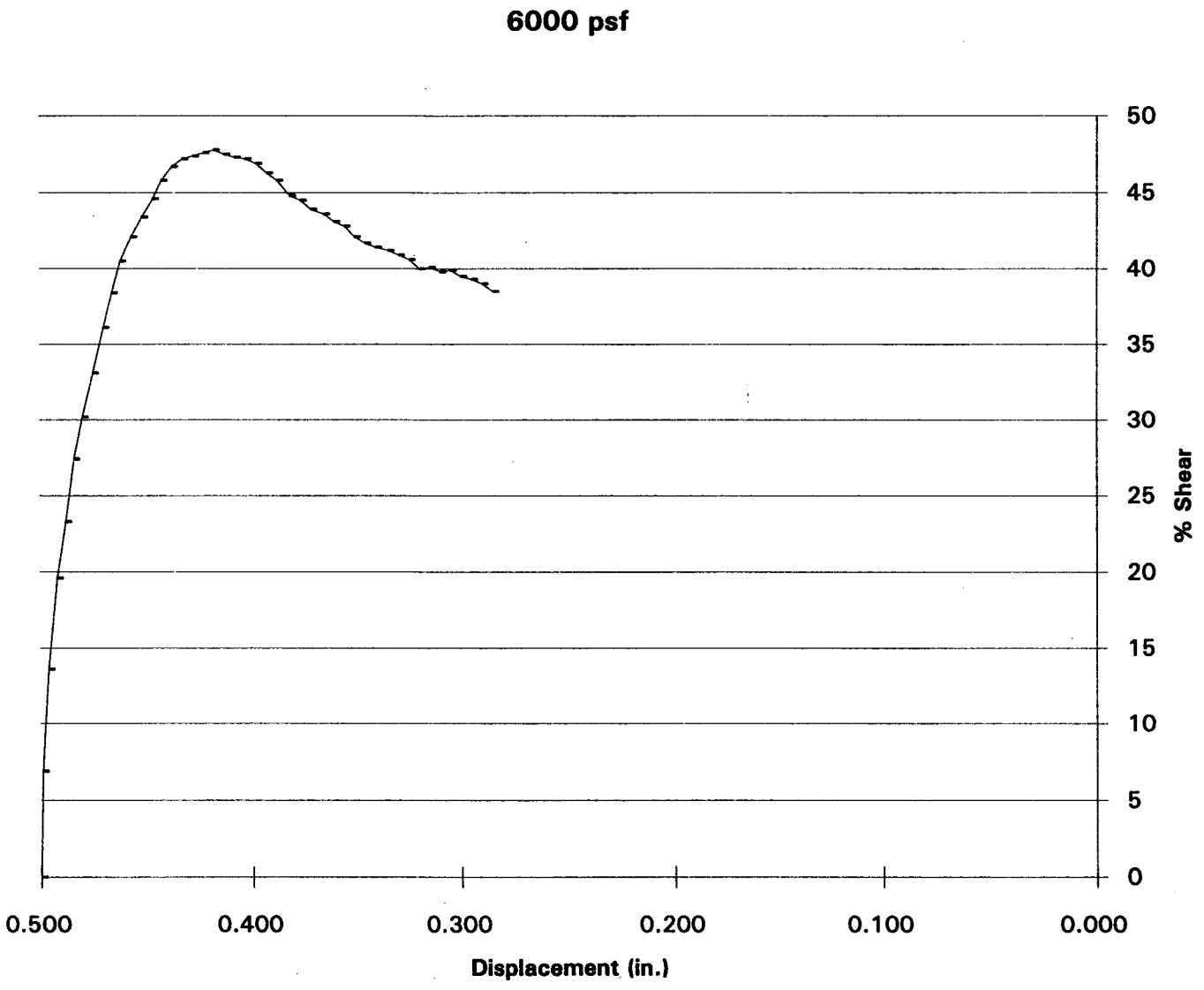
DIRECT SHEAR TEST PLOT OF B1A @ 40 FT.



DIRECT SHEAR TEST PLOT OF B1A @ 40 FT.

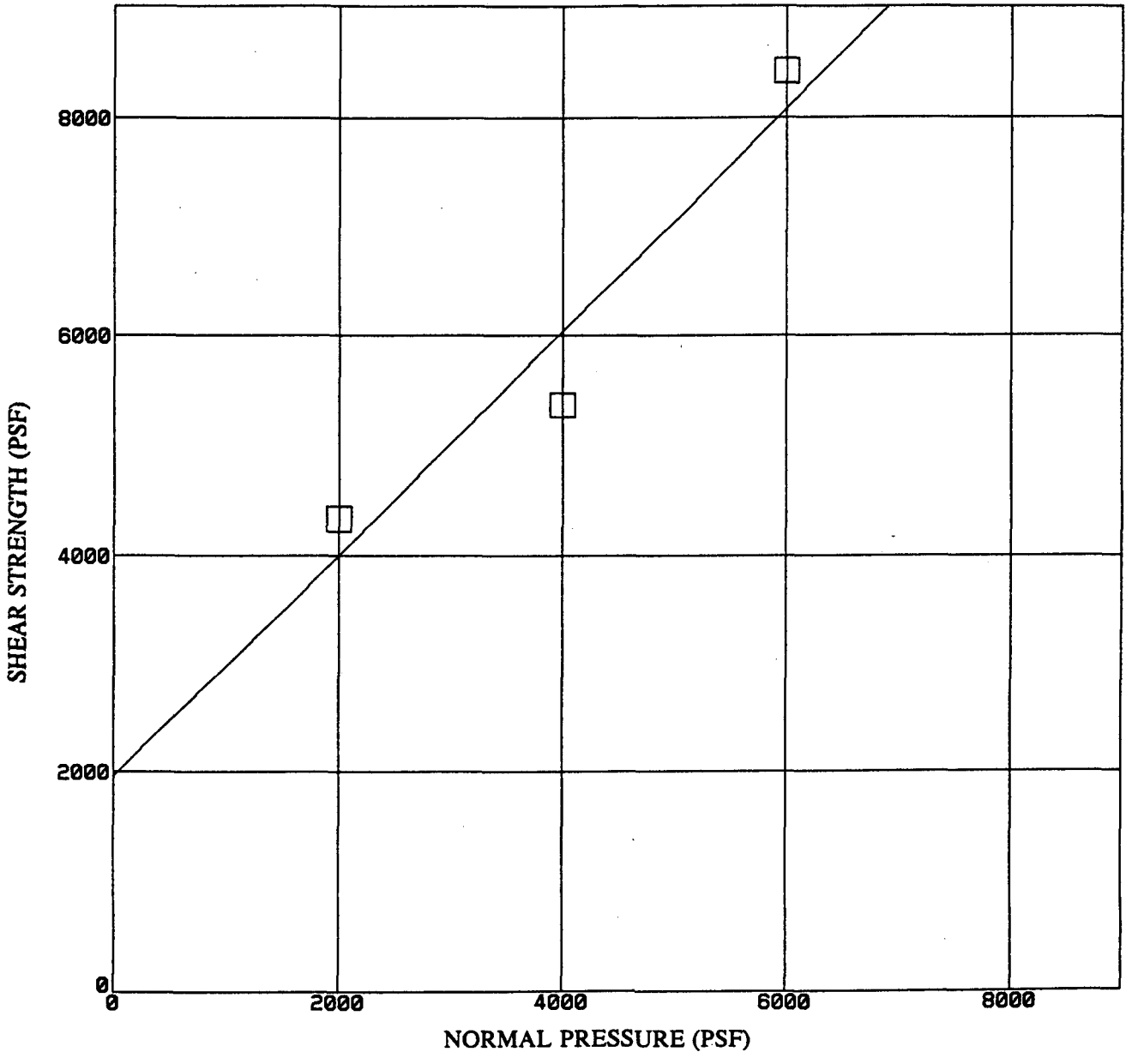


DIRECT SHEAR TEST PLOT OF B1A @ 40 FT.



SHEAR TEST DIAGRAM

Client: **Rancho Las Flores Limited Partnership**	W.O. **215-1**
Project: **Highway 138**	Location: **San Bernardino County**



Initial Sample

Dry Density = 127.5 pcf
Moist. Cont. = 12.0%
Clayey Sand
L.L. = 31.0
P.I. = 8.9

Sample: **B-2A @ 10.0 ft.**

Phi Angle: **46 deg.**
Cohesion: **1956 psf**

TEST CONDITION: Undisturbed, sheared at natural moisture

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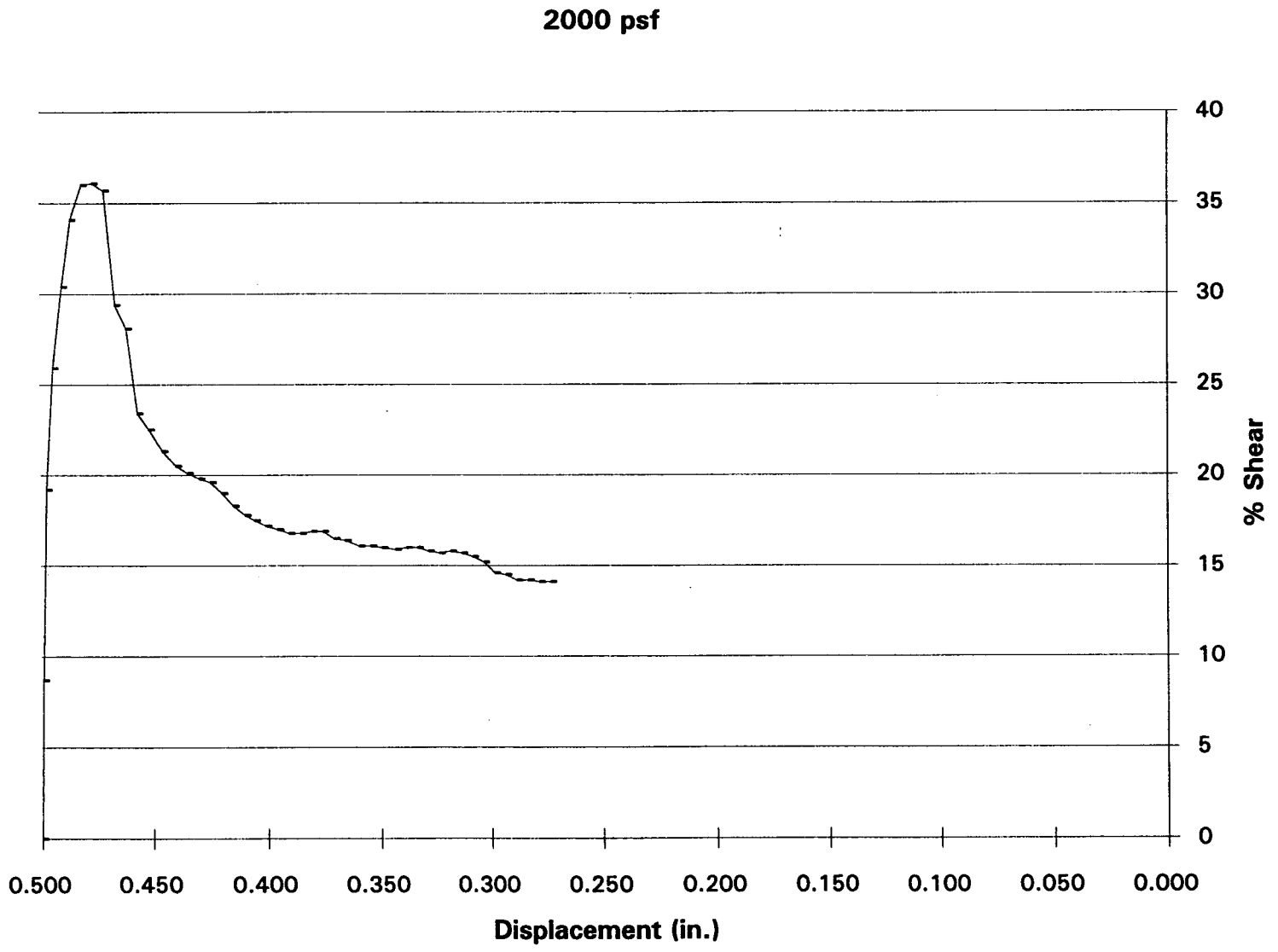
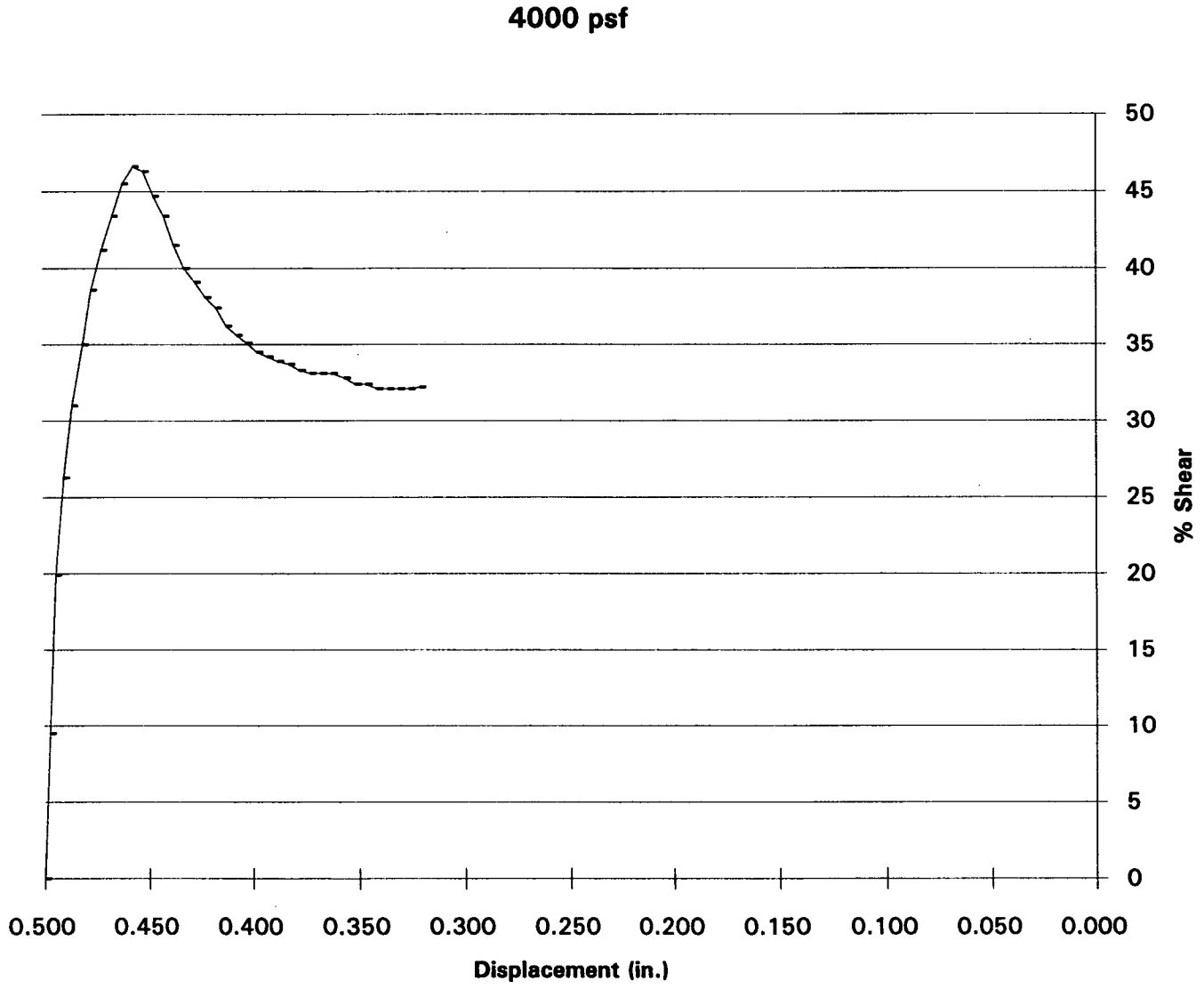
Plate

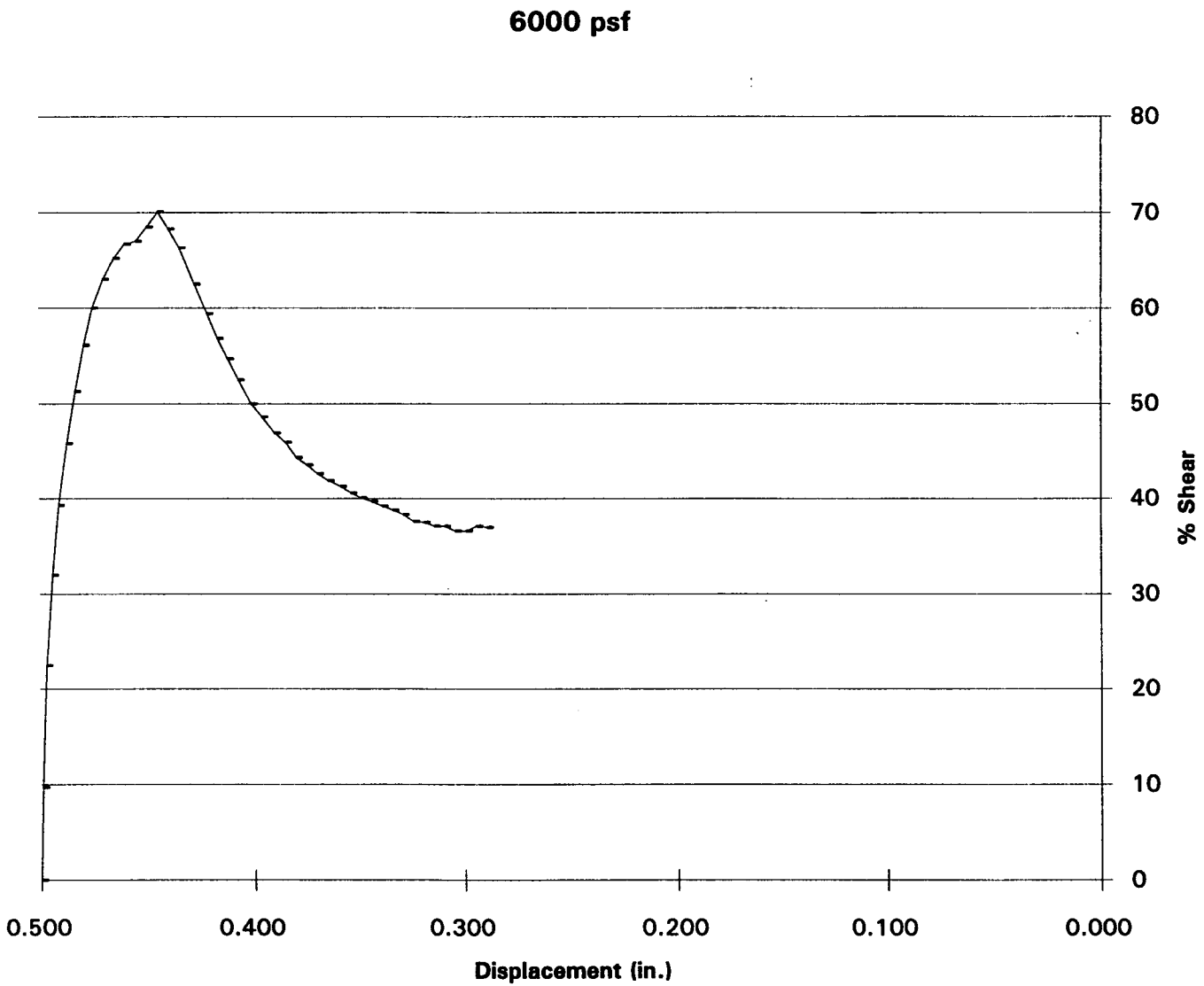
DIRECT SHEAR TEST PLOT OF B2A @ 10 FT.



DIRECT SHEAR TEST PLOT OF B2A @ 10 FT.

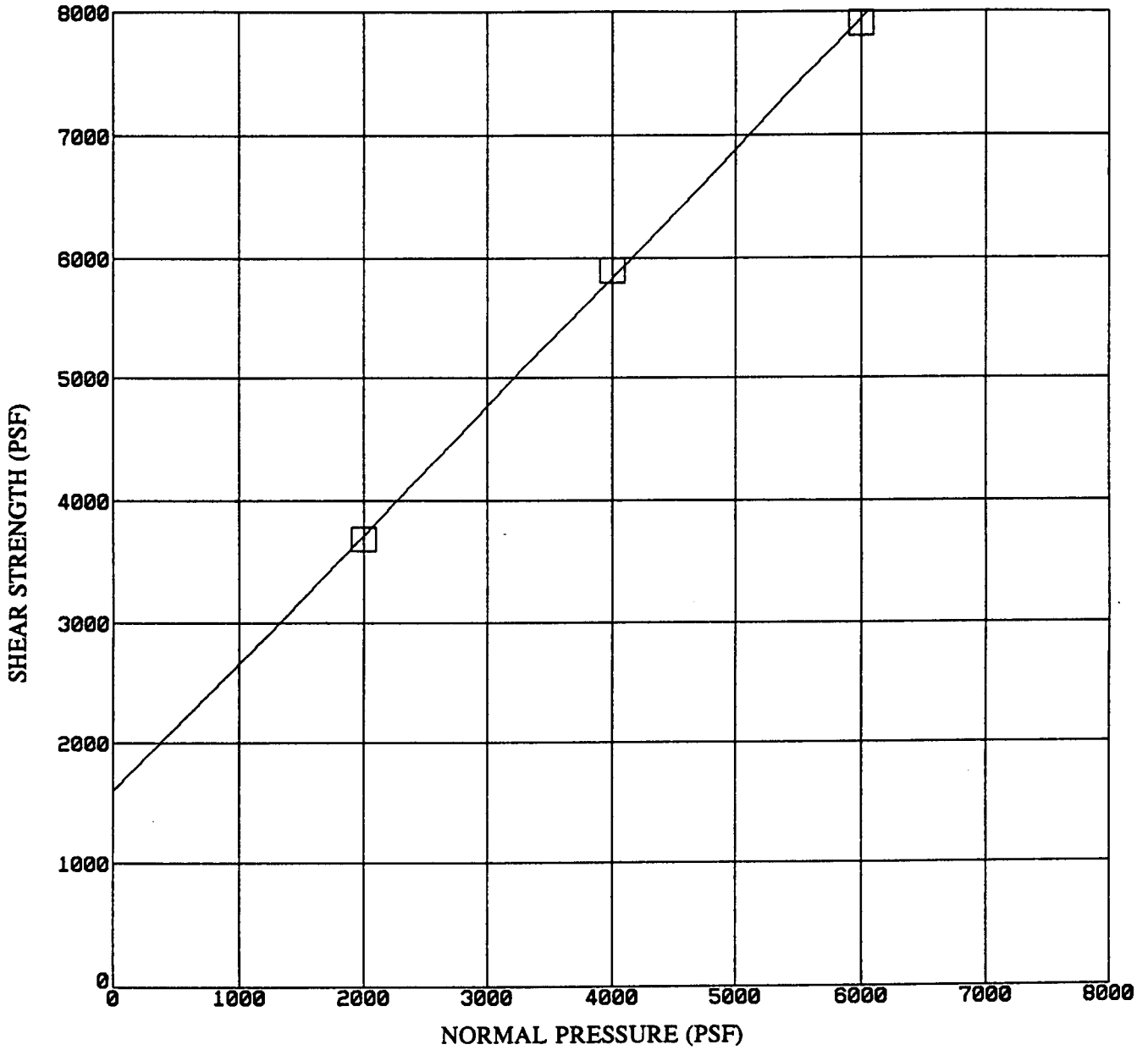


DIRECT SHEAR TEST PLOT OF B2A @ 10 FT.



SHEAR TEST DIAGRAM

Client: **Rancho Las Flores Limited Partnership**	W.O. **215-1**
Project: **Highway 138**	Location: **San Bernardino County**



Initial Sample

Dry Density = 128.0 pcf
Moist. Cont. = 8.5%

Sample: **B-2A @ 15.0 ft.**

Phi Angle: **47 deg.**
Cohesion: **1604 psf**

TEST CONDITION: Undisturbed, sheared at natural moisture

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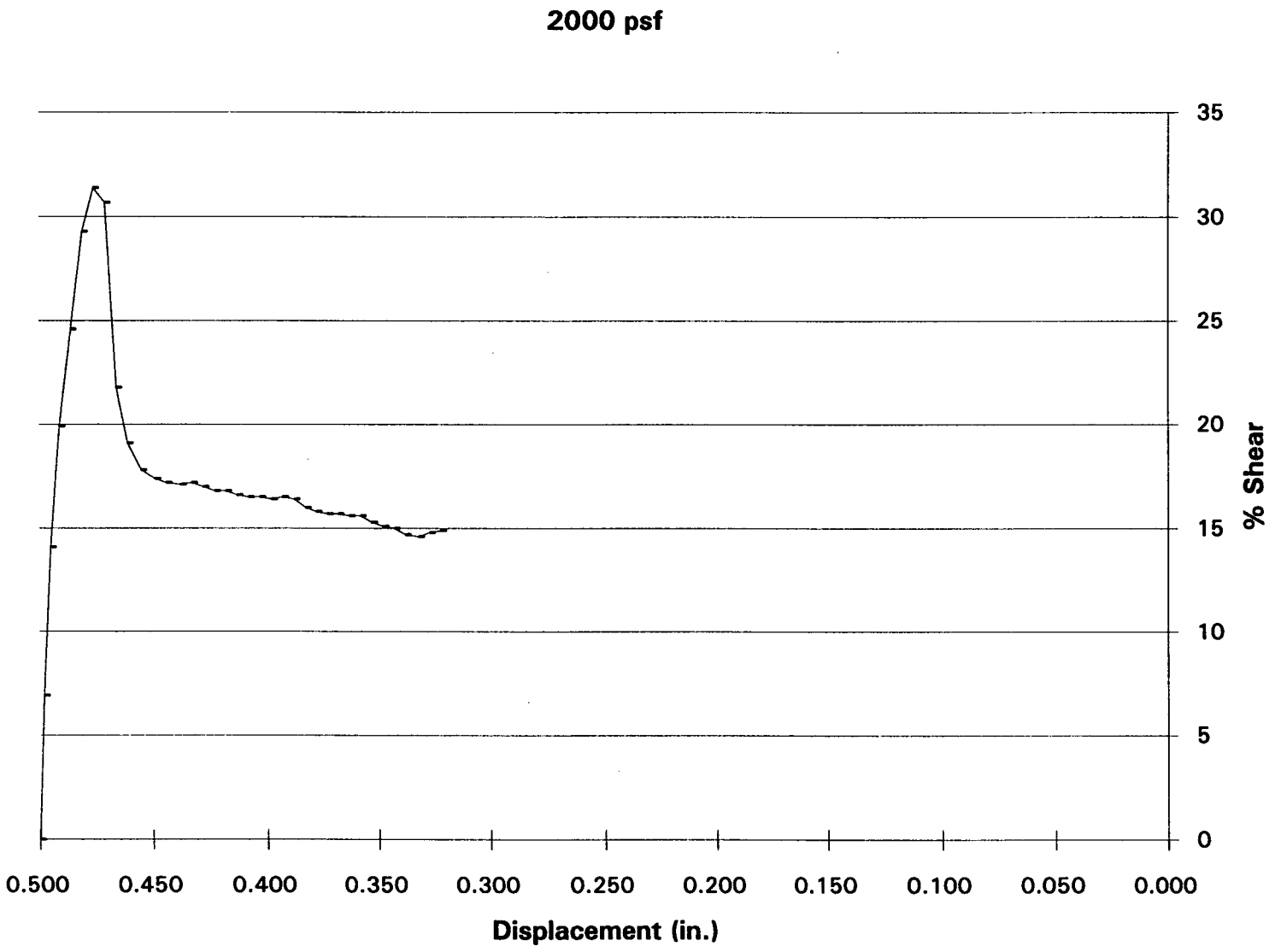
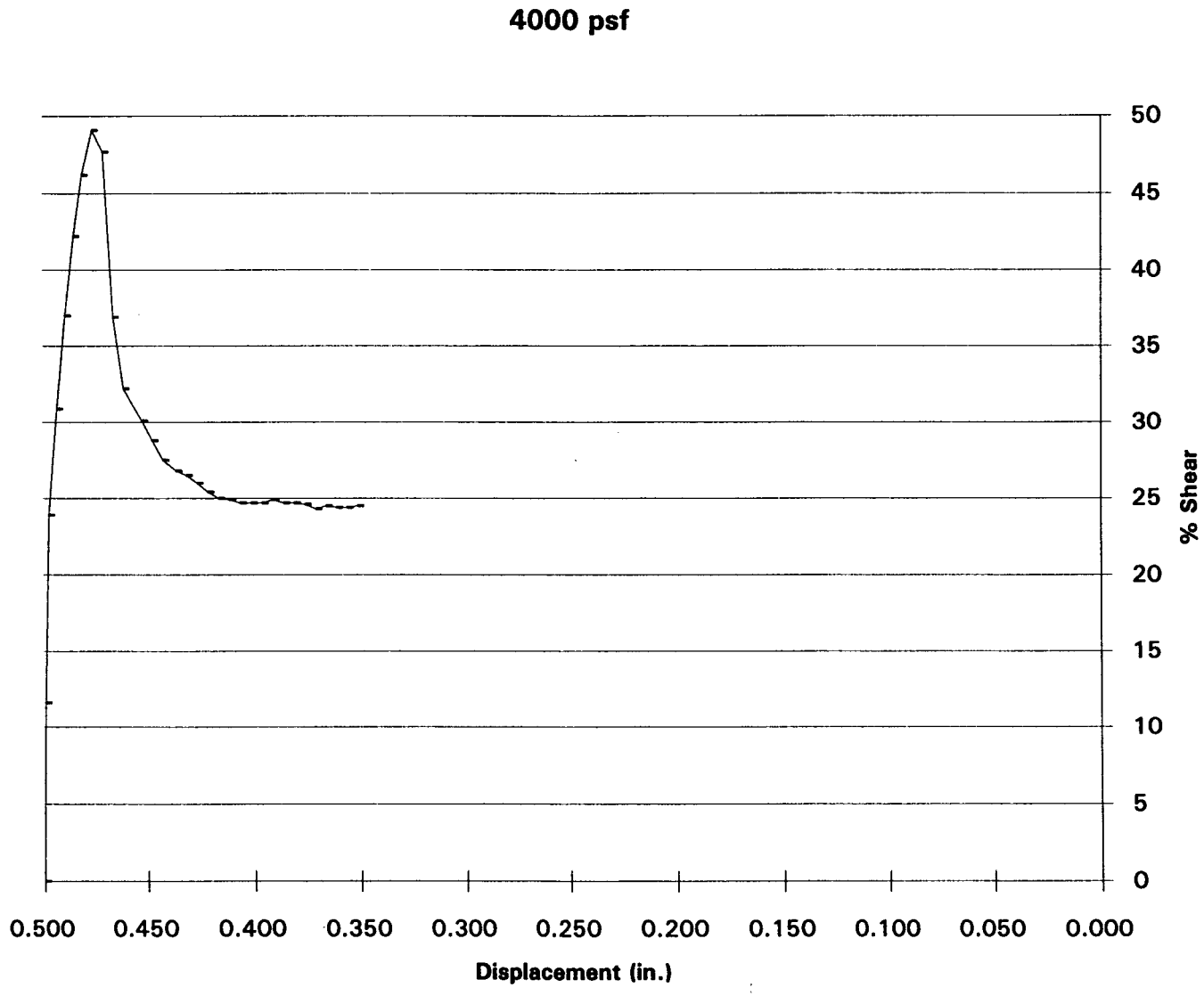
Plate

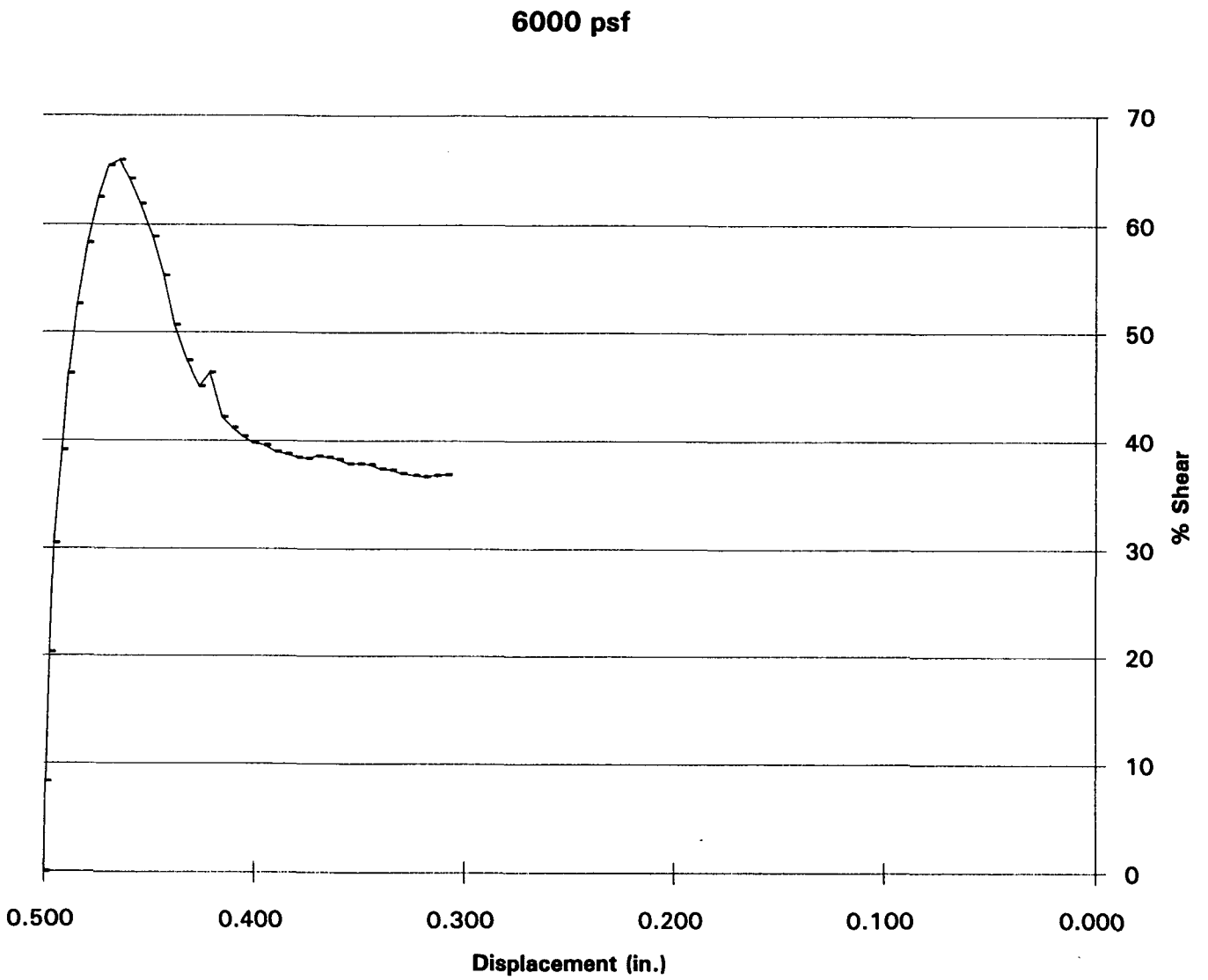
DIRECT SHEAR TEST PLOT OF B2A @ 15 FT.



DIRECT SHEAR TEST PLOT OF B2A @ 15 FT.

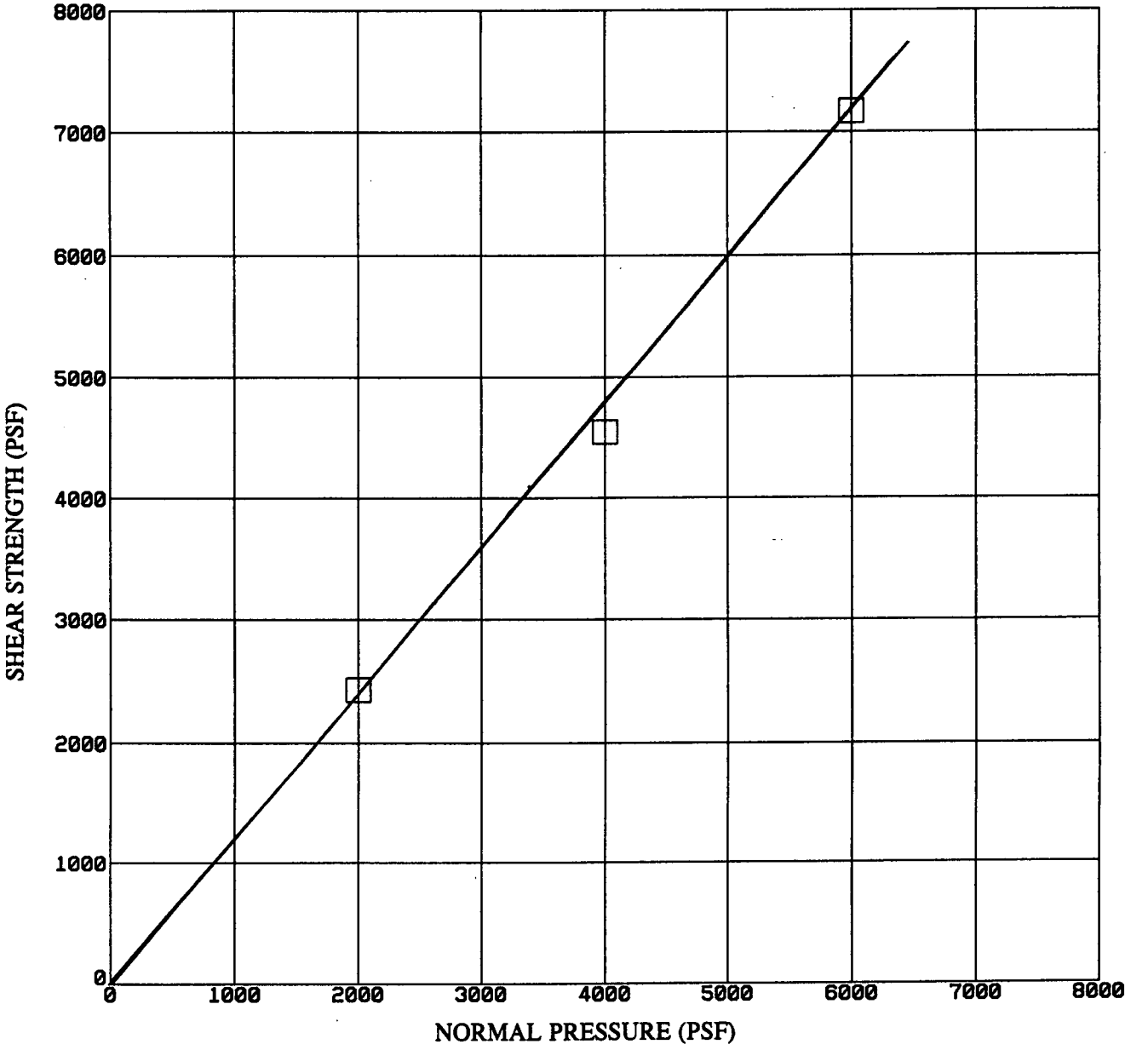


DIRECT SHEAR TEST PLOT OF B2A @ 15 FT.



SHEAR TEST DIAGRAM

Client: **Rancho Las Flores Limited Partnership**	W.O. **215-1**
Project: **Highway 138**	Location: **San Bernardino County**



Initial Sample

Dry Density = 118.5 pcf
Moist. Cont. = 5.3%

Sample: **B-2A @ 20.0 ft.**

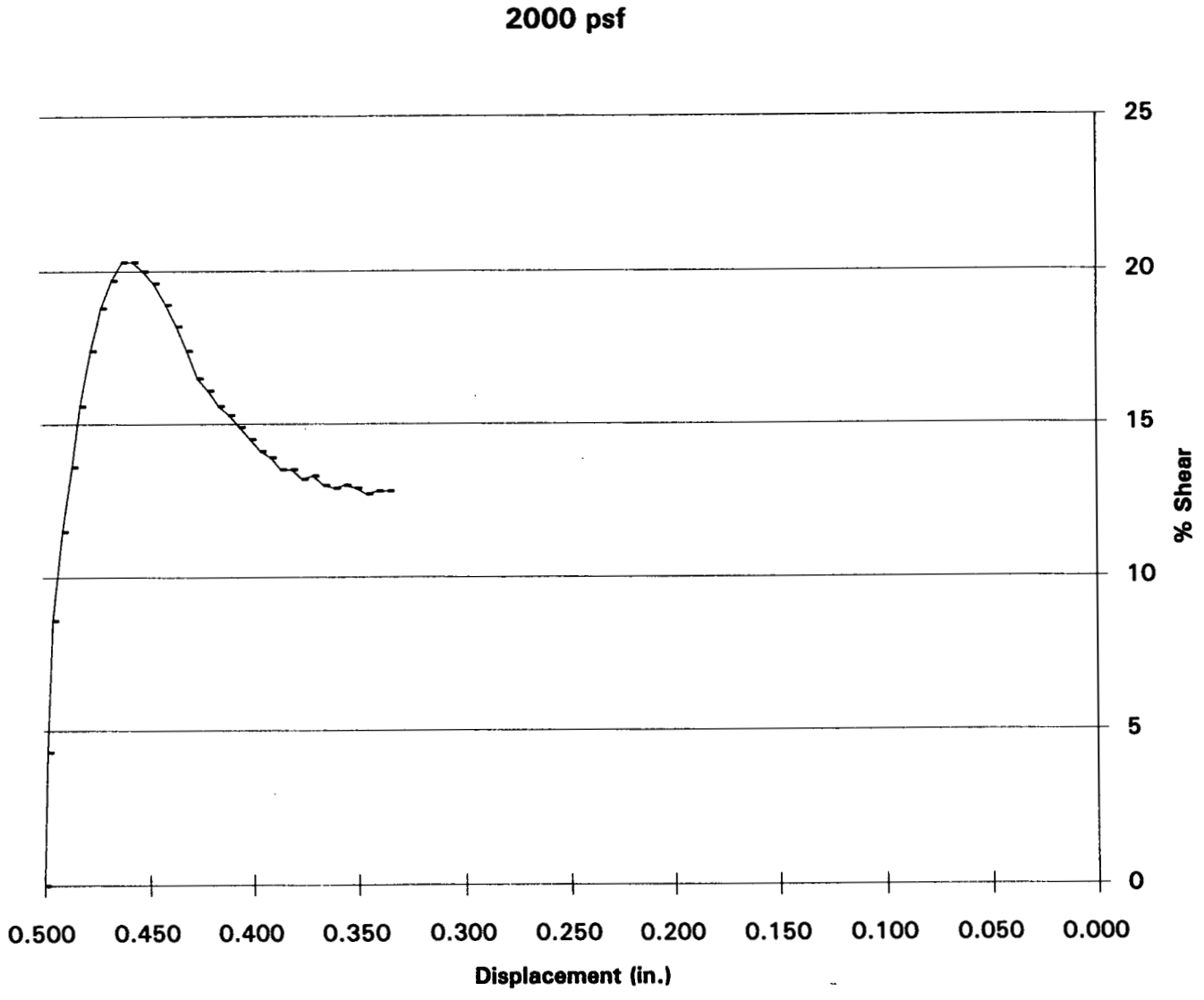
Phi Angle: **50 deg.**

Cohesion: **0 psf**

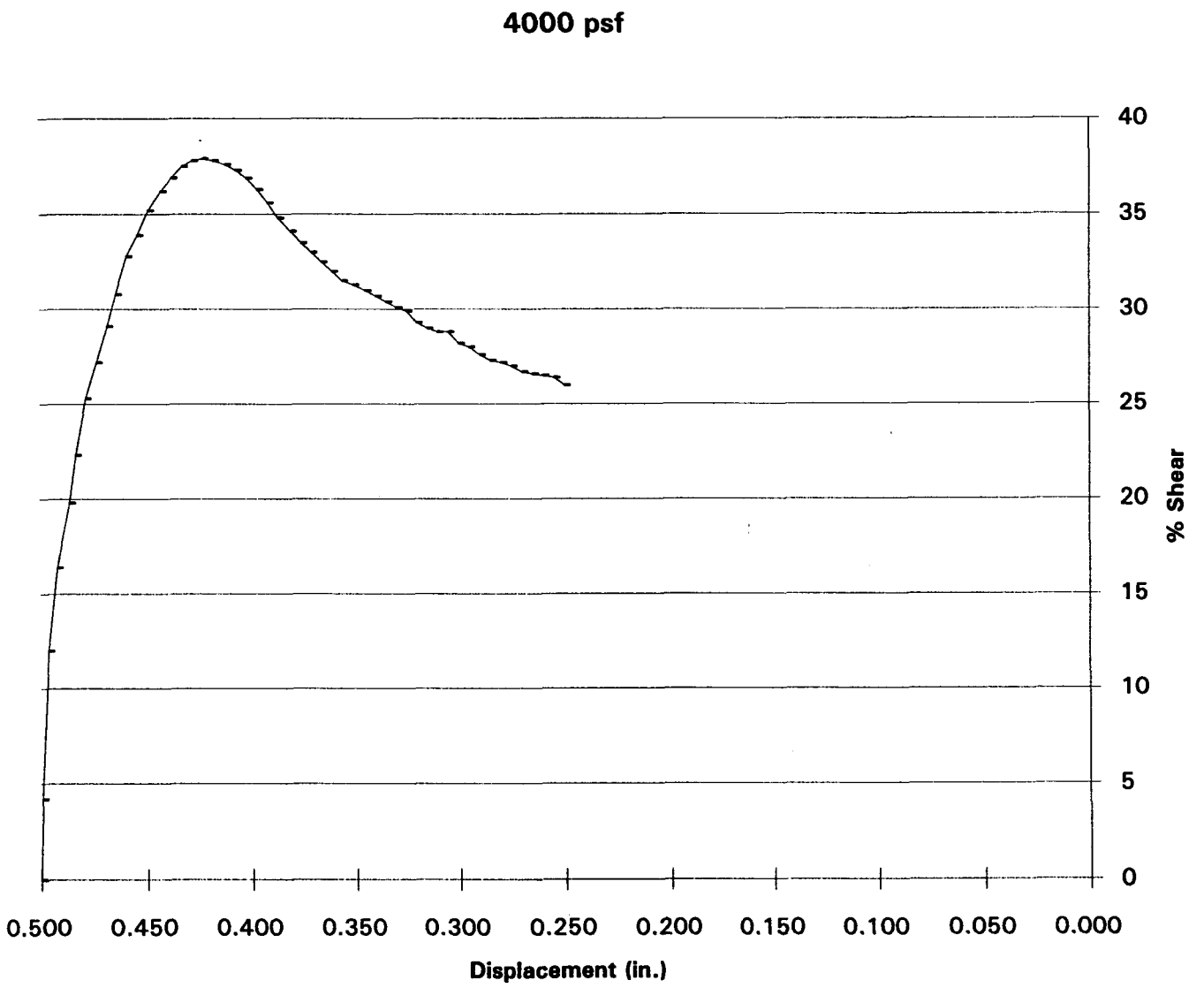
TEST METHOD: Undisturbed, sheared at natural moisture

John A. Sayers & Associates GEOTECHNICAL CONSULTANTS	Plate

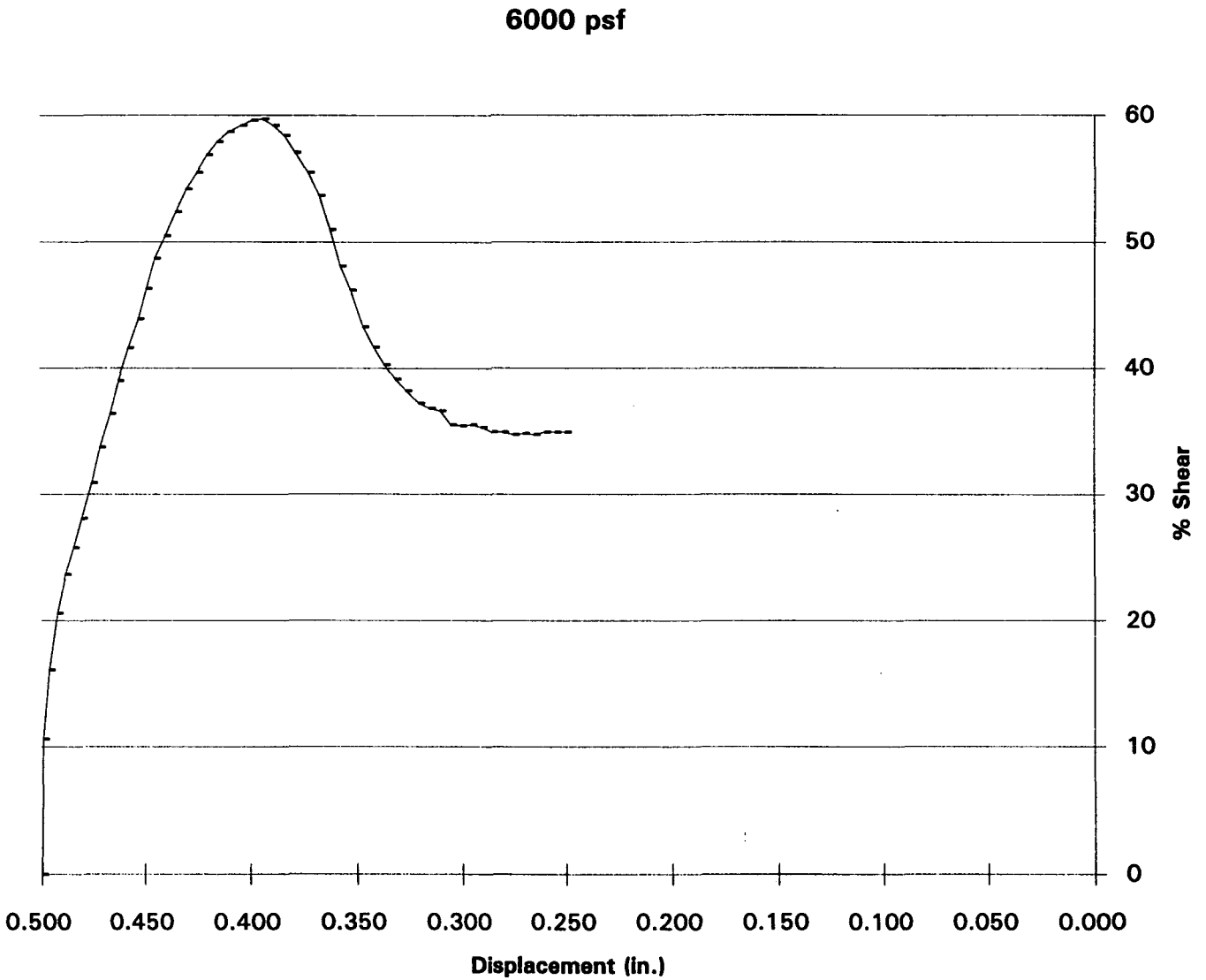
DIRECT SHEAR TEST PLOT OF B2A @ 20 FT.



DIRECT SHEAR TEST PLOT OF B2A @ 20 FT.

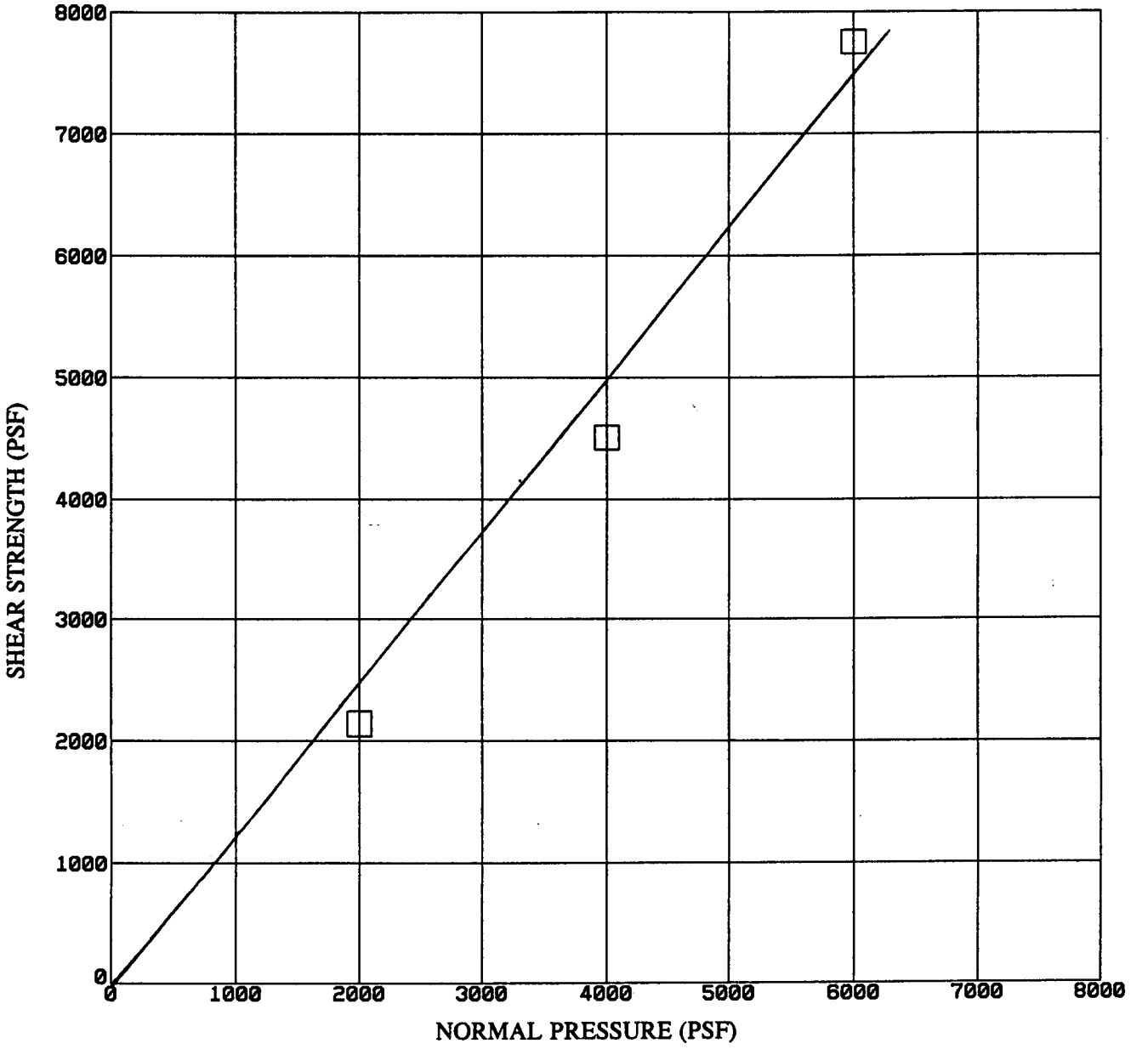


DIRECT SHEAR TEST PLOT OF B2A @ 20 FT.



SHEAR TEST DIAGRAM

Client: **Rancho Las Flores Limited Partnership**	W.O. **215-1**
Project: **Highway 138**	Location: **San Bernardino County**



Initial Sample

Dry Density = 123.3 pcf
Moist. Cont. = 6.0%
Silty to Clayey Sand
L.L. = 26.0
P.I. = 5.2

Sample: **B-2A @ 35.0 ft.**

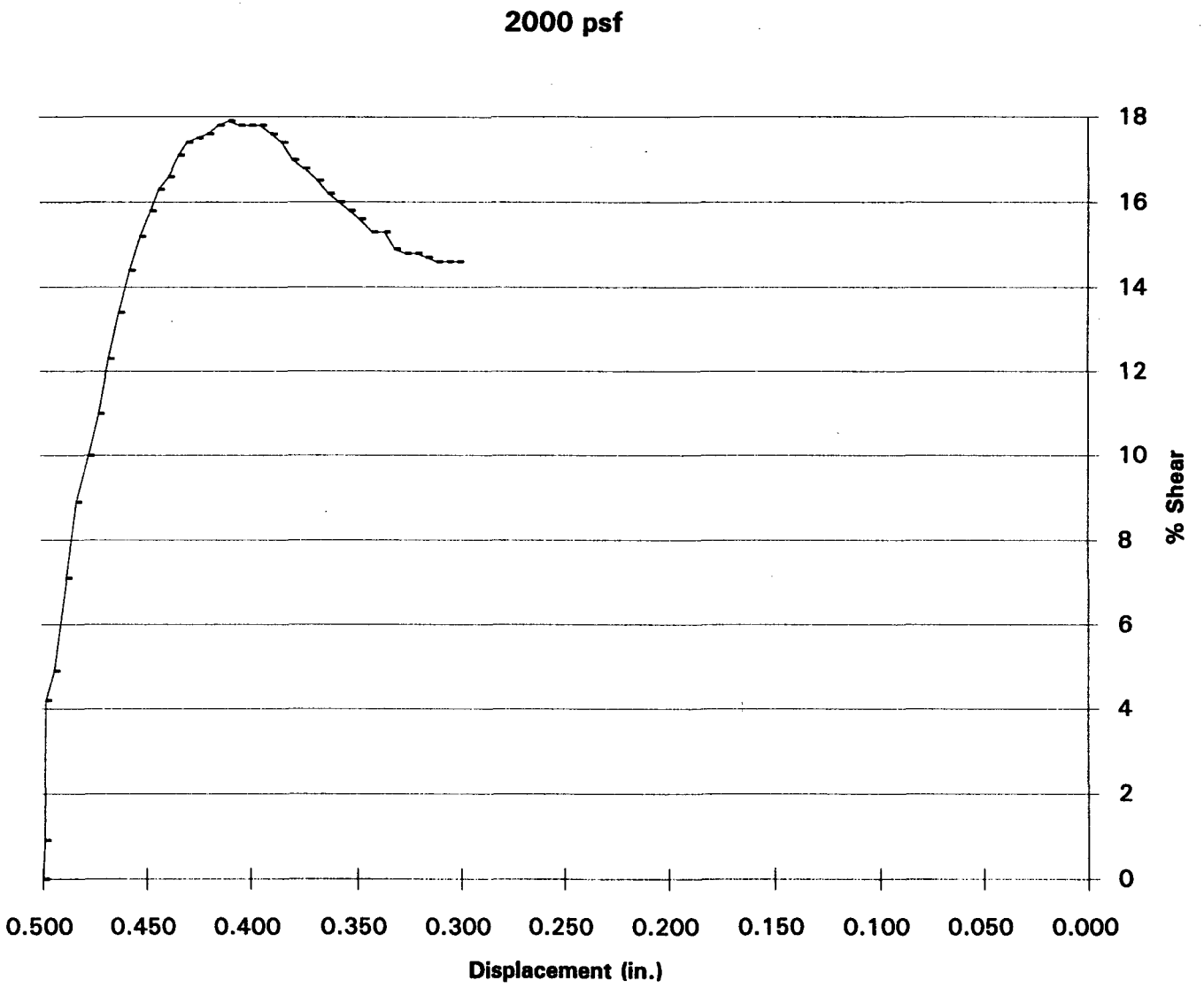
Phi Angle: **51 deg.**
Cohesion: **0 psf**

TEST METHOD: Undisturbed, sheared at natural moisture

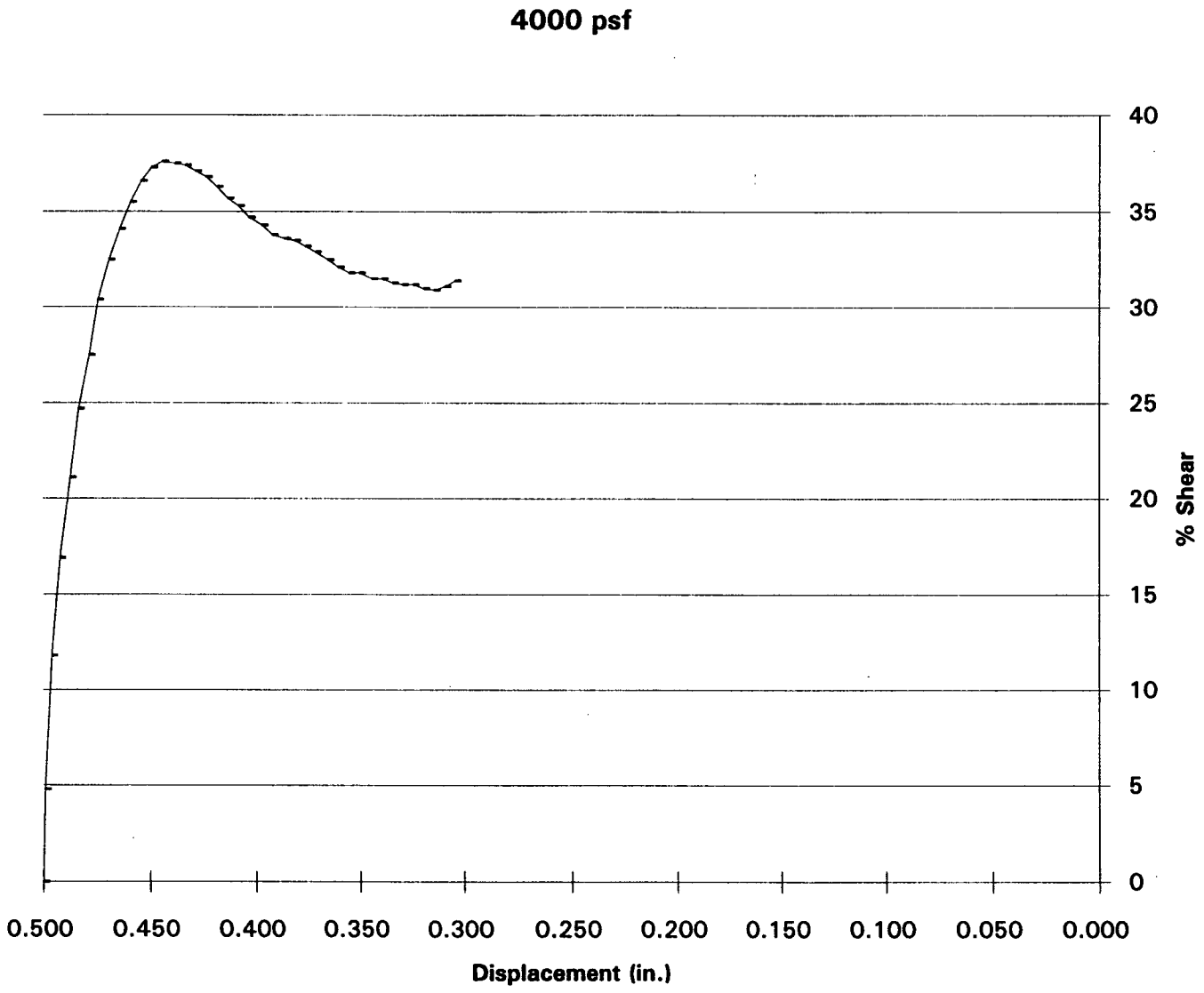
John A. Sayers & Associates
GEOTECHNICAL CONSULTANTS

Plate

DIRECT SHEAR TEST PLOT OF B2A @ 35 FT.

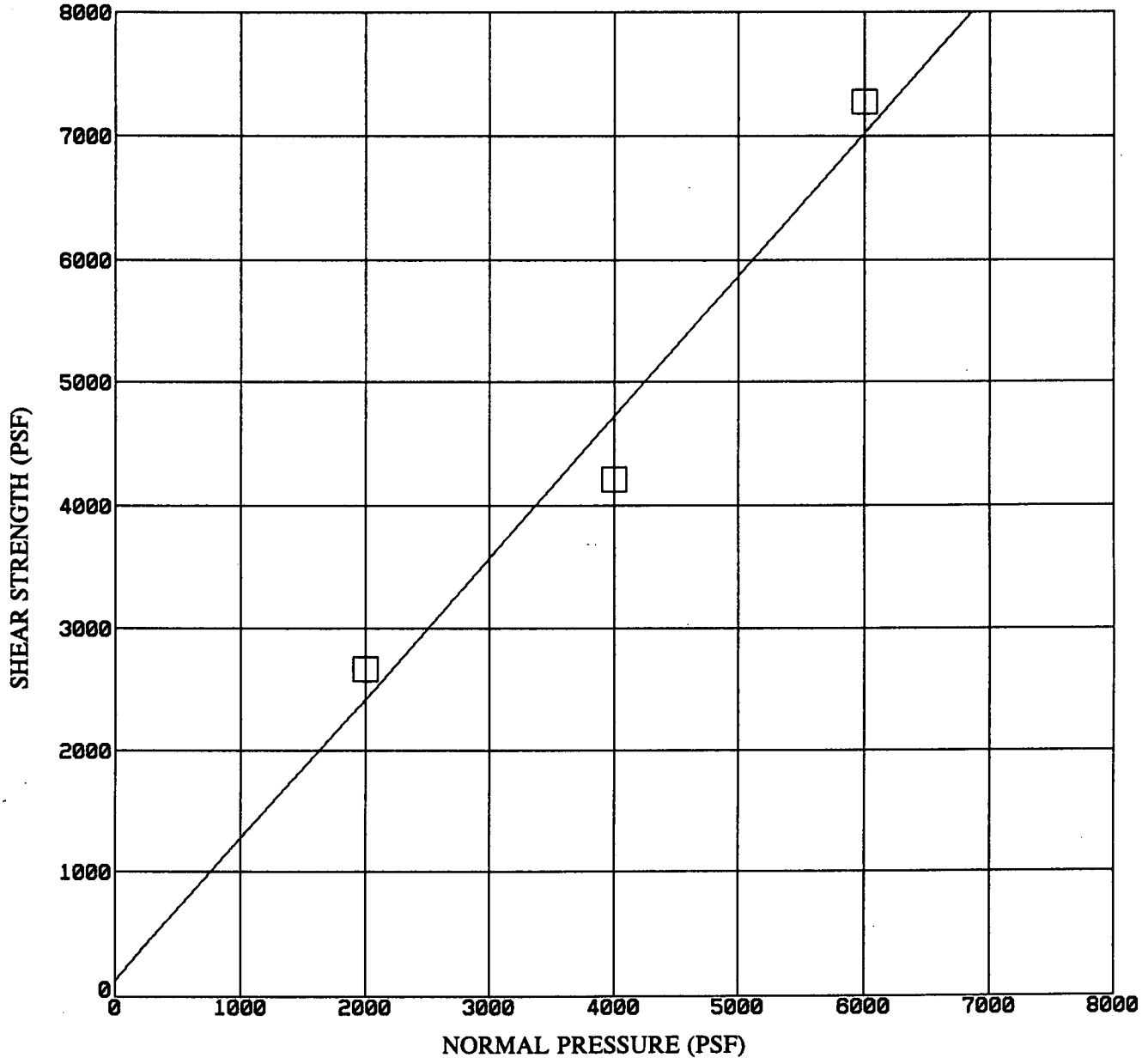


DIRECT SHEAR TEST PLOT OF B2A @ 35 FT.



SHEAR TEST DIAGRAM

Client: **Rancho Las Flores Limited Partnership**	W.O. **215-1**
Project: **Highway 138**	Location: **San Bernardino County**



Initial Sample

Dry Density - 120.1 pcf
Moist. Cont. = 6.0%

Sample: **B-2A @ 45.0 ft.**

Phi Angle: **49 deg.**

Cohesion: **124 psf**

TEST CONDITION: Undisturbed, sheared at natural moisture

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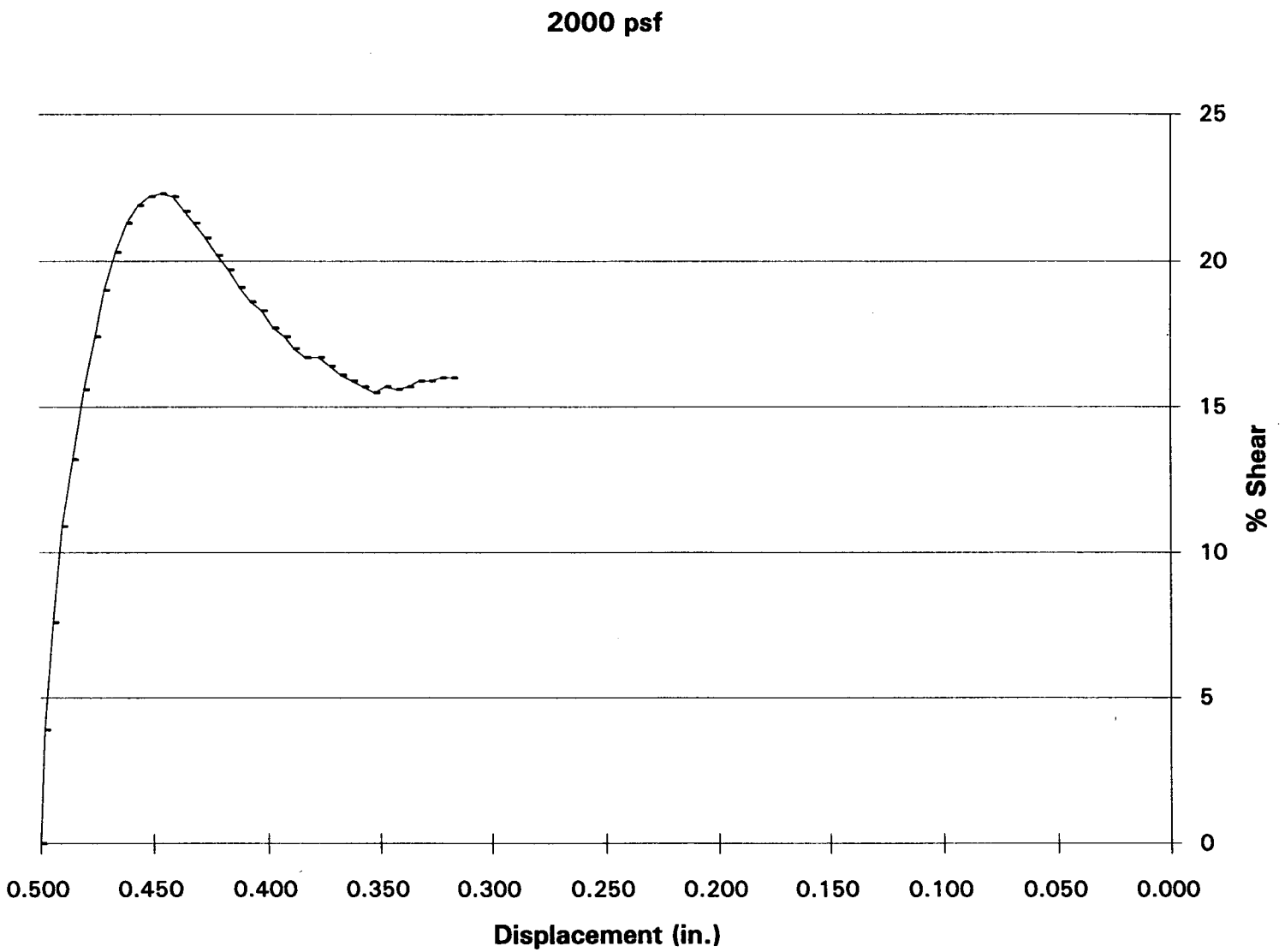
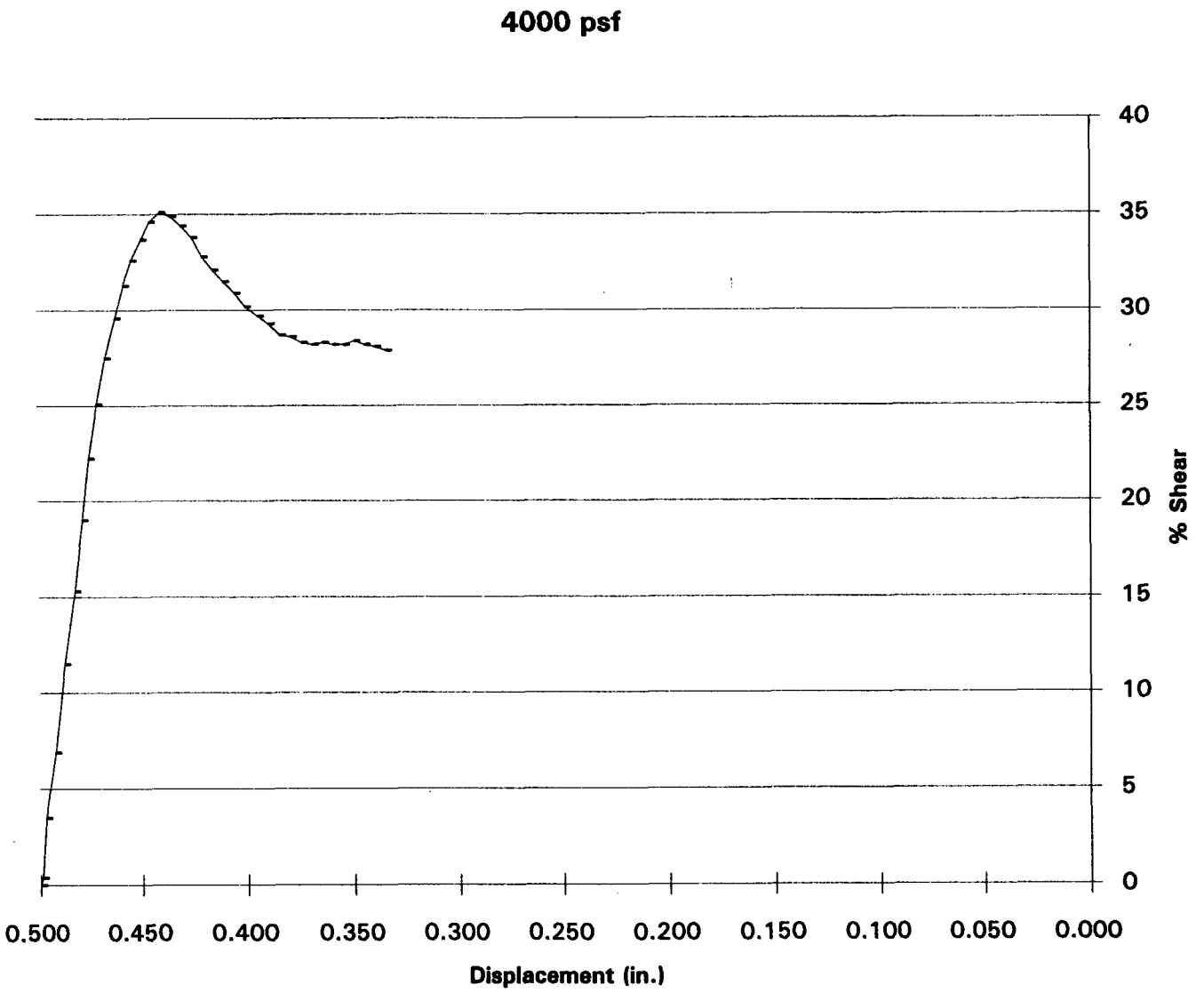
Plate

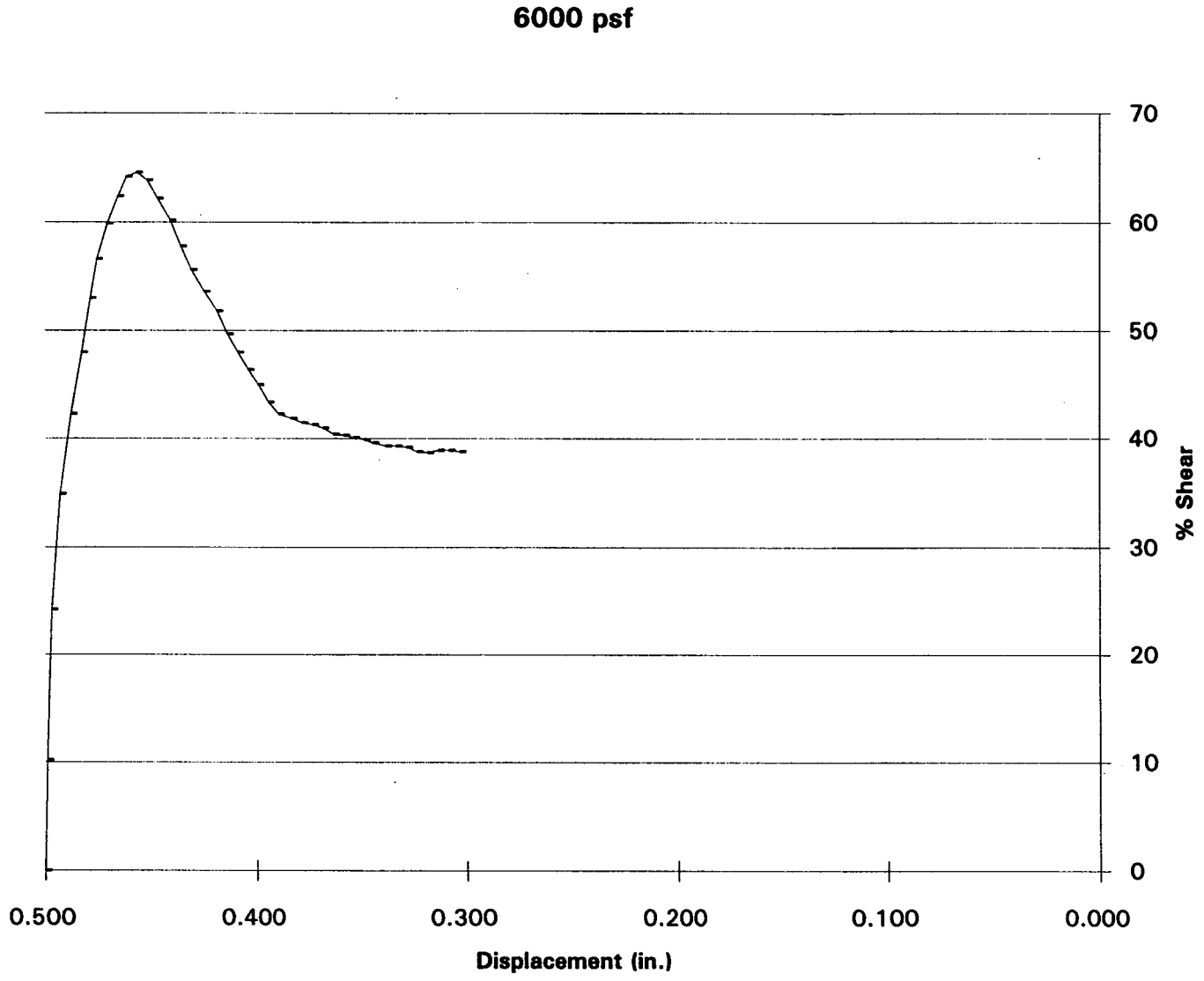
DIRECT SHEAR TEST PLOT OF B2A @ 45 FT.



DIRECT SHEAR TEST PLOT OF B2A @ 45 FT.



DIRECT SHEAR TEST PLOT OF B2A @ 35 FT.



APPENDIX C

SHEAR TEST DATA UNDISTURBED SAMPLES SHEARED AFTER SATURATION

CONTENTS:

Shear Test Summary
Shear Test Diagram Summary
Individual Shear Test Diagrams with Stress-Strain Diagrams

JOHN A. SAYERS & ASSOCIATES

SHEAR TEST SUMMARY

Undisturbed, Tested after Saturation

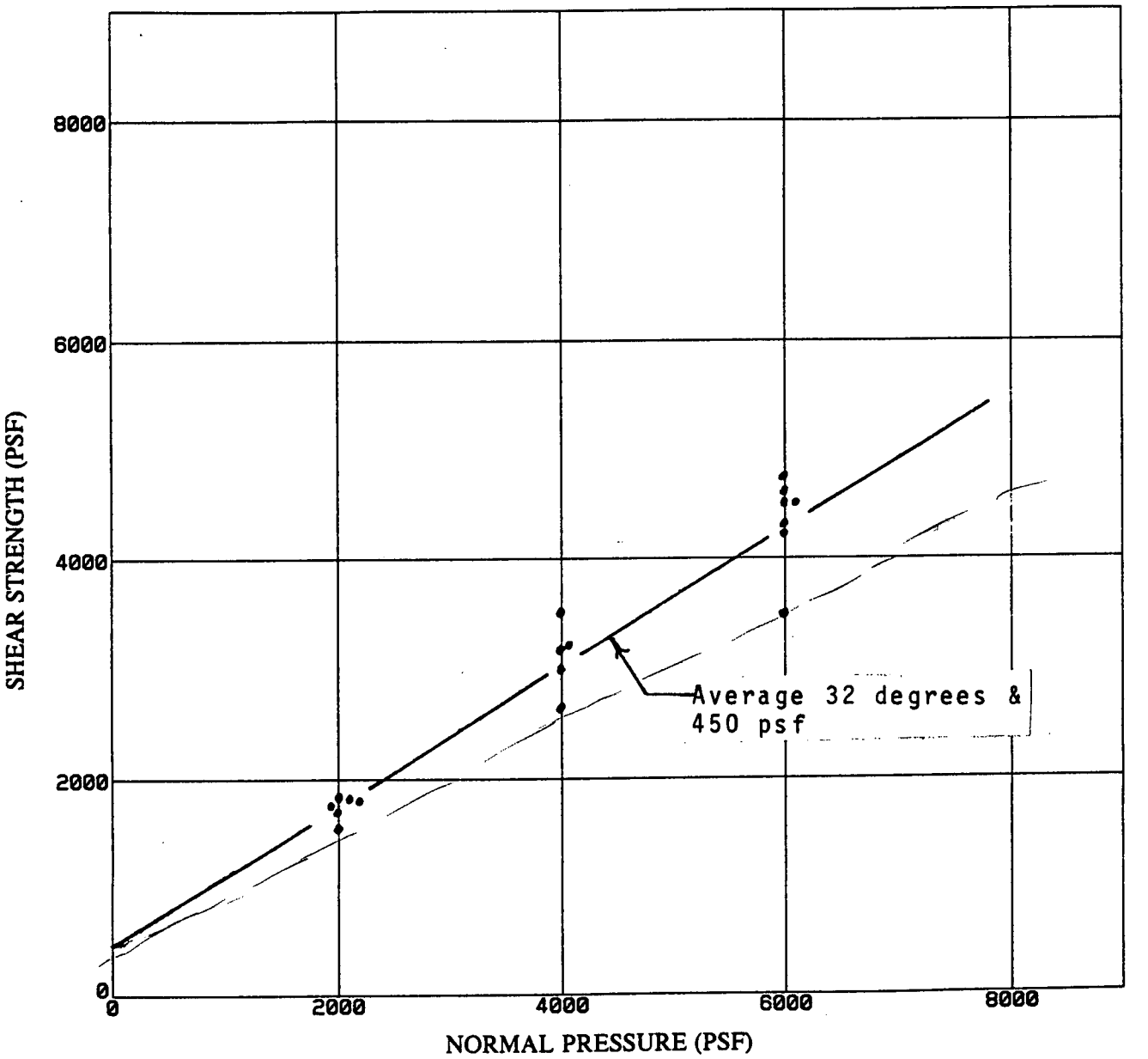
Sample Location	Shear Strength (psf)	Normal Pressure (psf) 2000	4000	6000
B-5 @ 10'		1704	2640	4500
B-10 @ 10'		1848	(1)	4776
B-11 @ 5'		1860	2976	4296
B-12 @ 20'		1776	3168	3468
B-13 @ 5'		1848	3516	5472 (1) (3)
B-13 @ 15'		1548	3192	4512
B-1A @ 20' (2)			5760 (3)	
B-1A @ 40' (2)				4596
B-2A @ 15'				4212
Average		1764	3098	4337

Average strength (4) 32.0 degrees and 450 psf

(1) Rock in sample
(2) Insufficient sample for more points
(3) Too high, not used for average
(4) Calculated as a mathematical average

SHEAR TEST DIAGRAM

Client: Rancho Las Flores Limited Partnership	W.O. 215-1
Project: Highway 138	Location: San Bernardino County



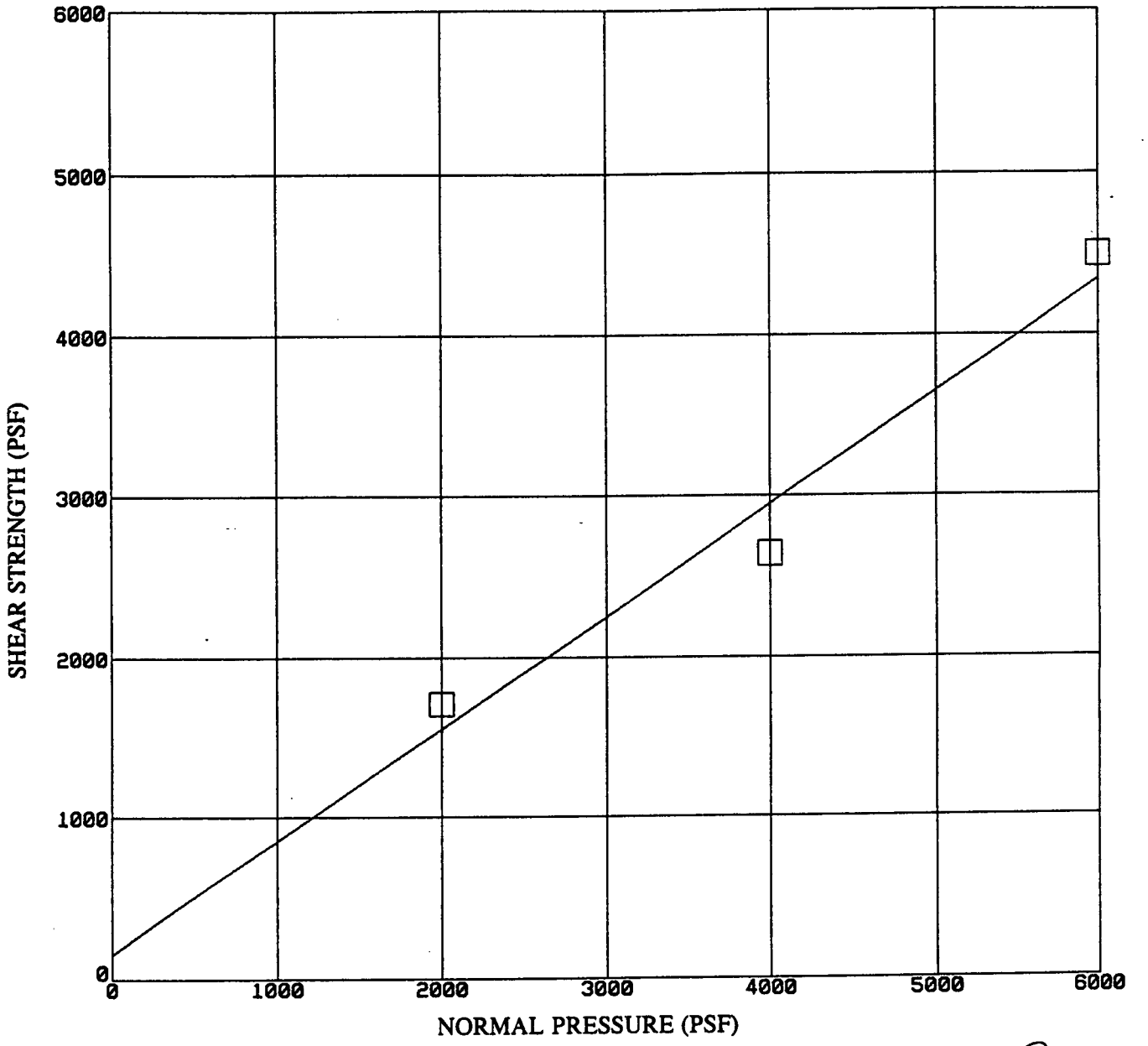
Shear Test Summary
Undisturbed Samples
Sheared after Saturation

John A. Sayers & Associates
GEOTECHNICAL CONSULTANTS

Plate

SHEAR TEST DIAGRAM

Client: Rancho Las Flores Limited Partnership	W.O. 215-1
Project: Highway 138	Location: San Bernardino County



Initial Sample

Dry Density = 113.7 pcf
Moist. Cont. = 2.9%

Sample: B-5 @ 10.0 ft.

Phi Angle: 35 deg.

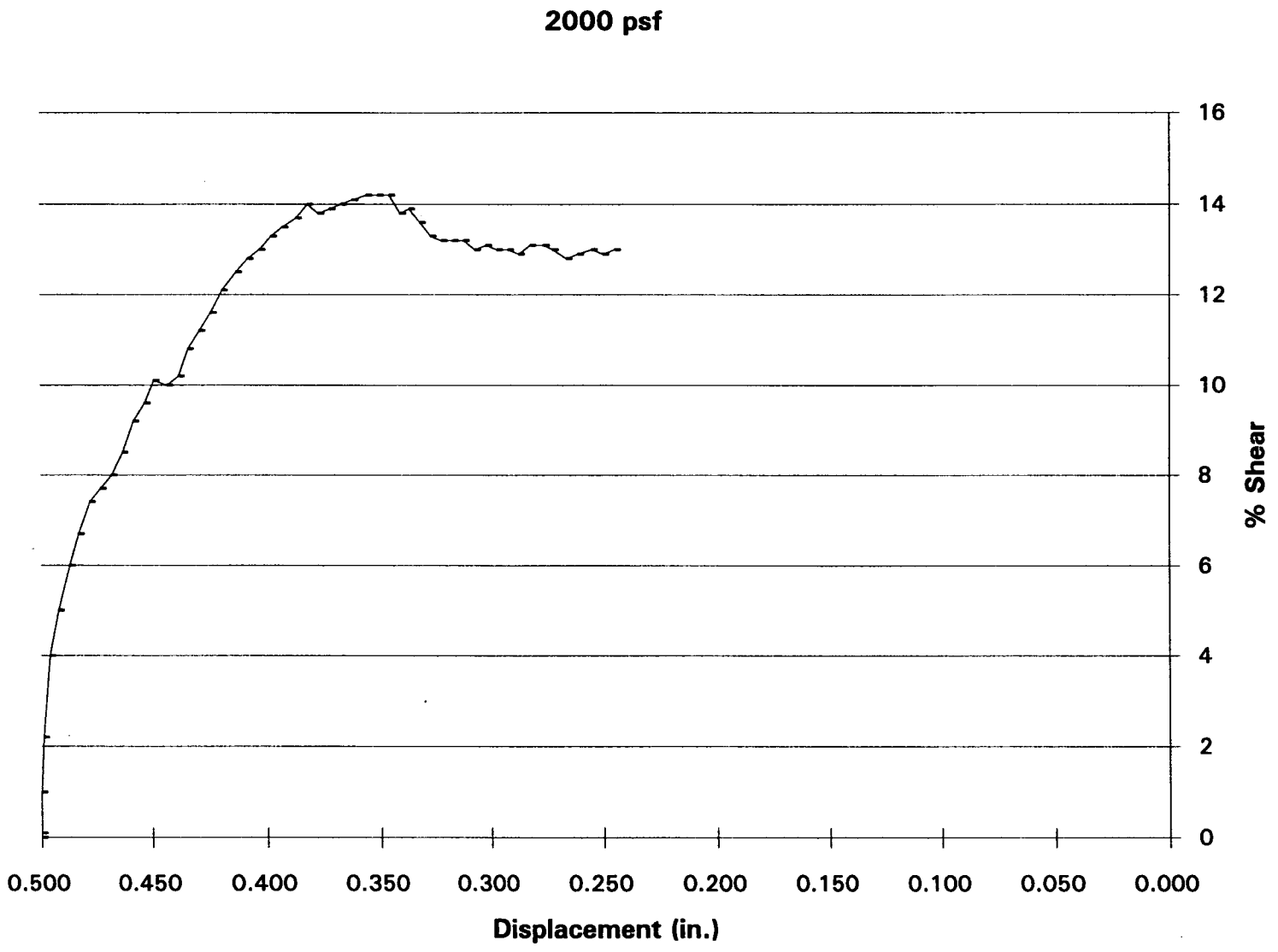
Cohesion: 152 psf

PI ?
Sr.

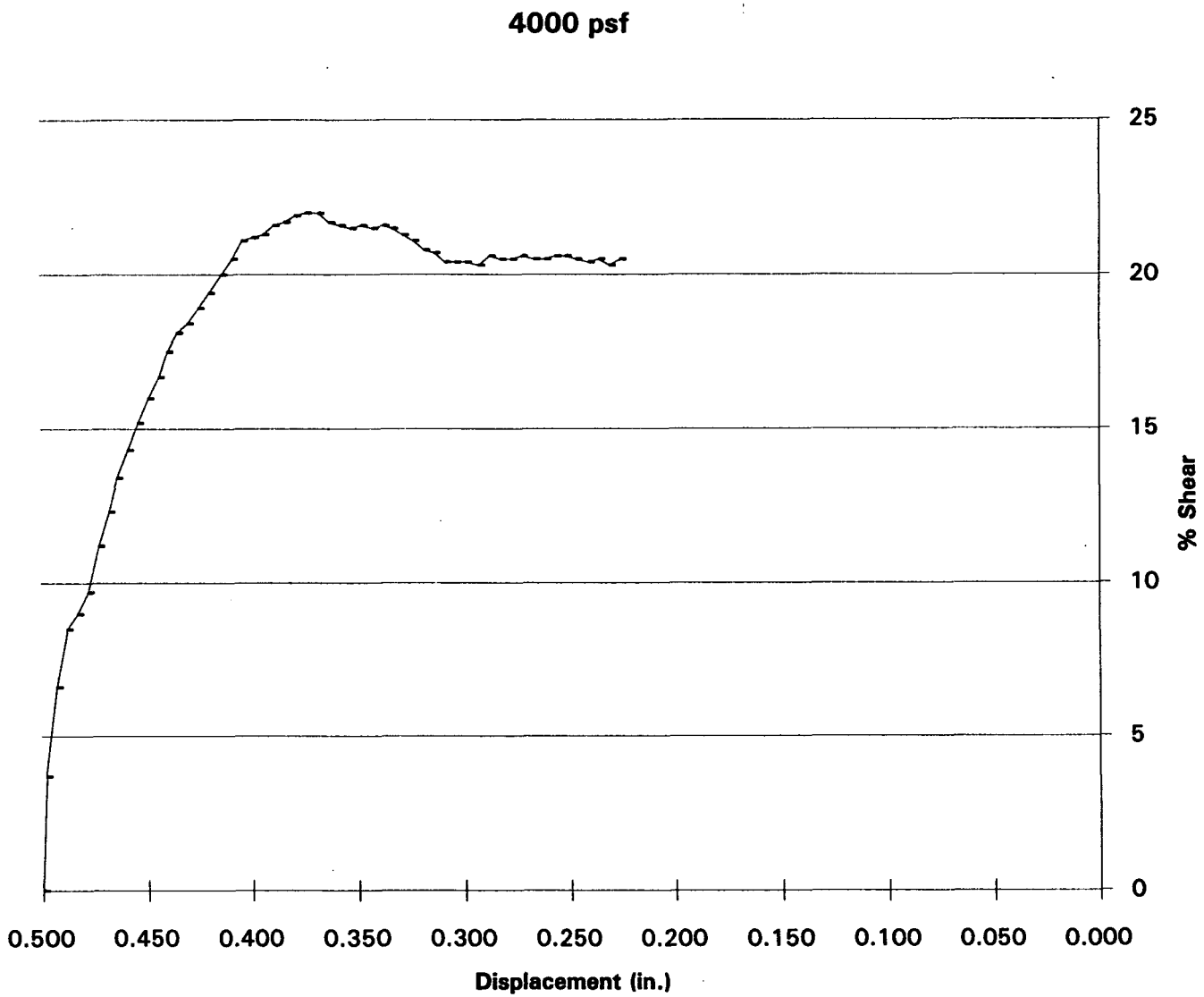
TEST CONDITION: Undisturbed. tested at saturation

John A. Sayers & Associates GEOTECHNICAL CONSULTANTS	Plate

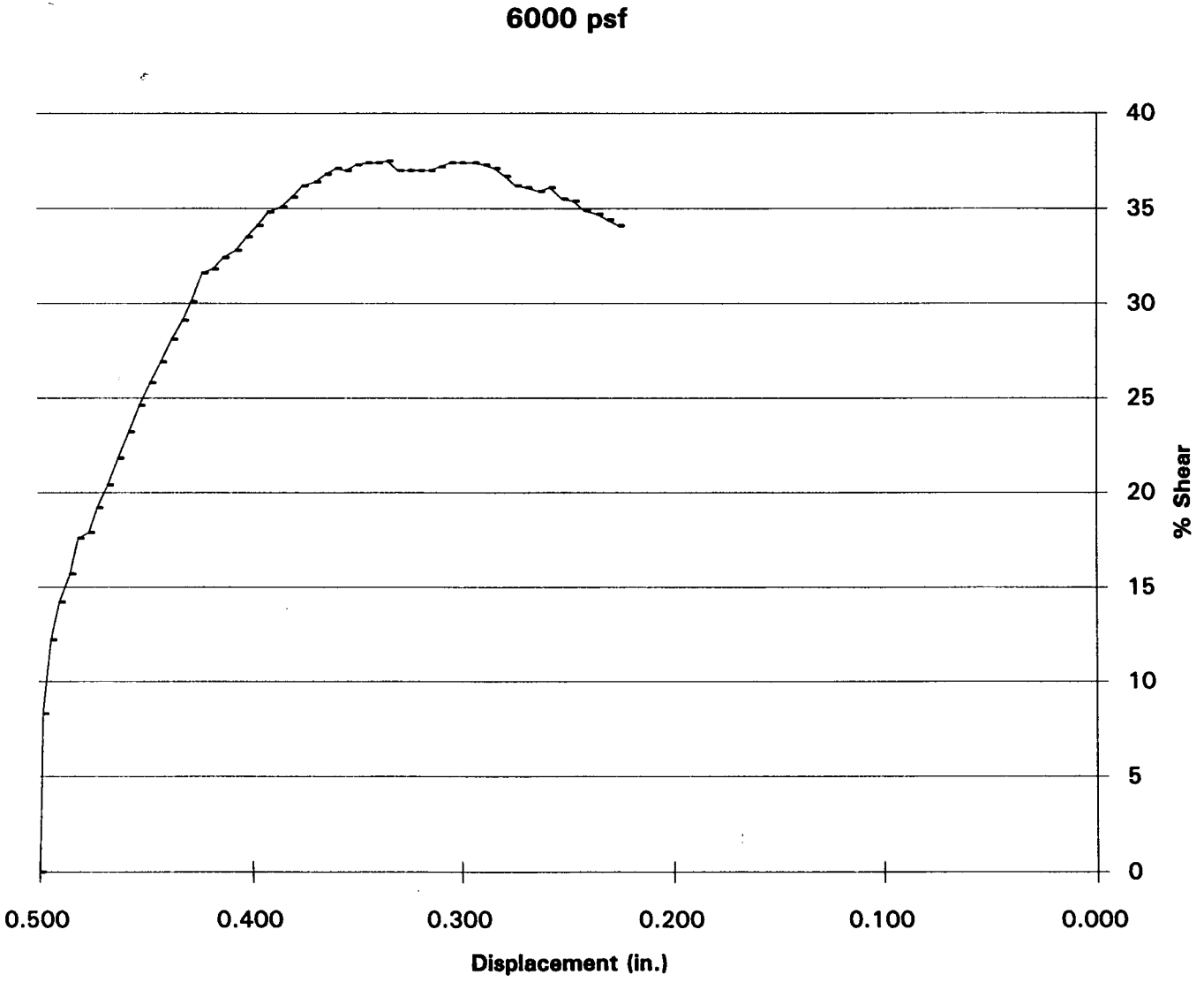
DIRECT SHEAR TEST PLOT OF B5 @ 10 FT.



DIRECT SHEAR TEST PLOT OF B5 @ 10 FT.

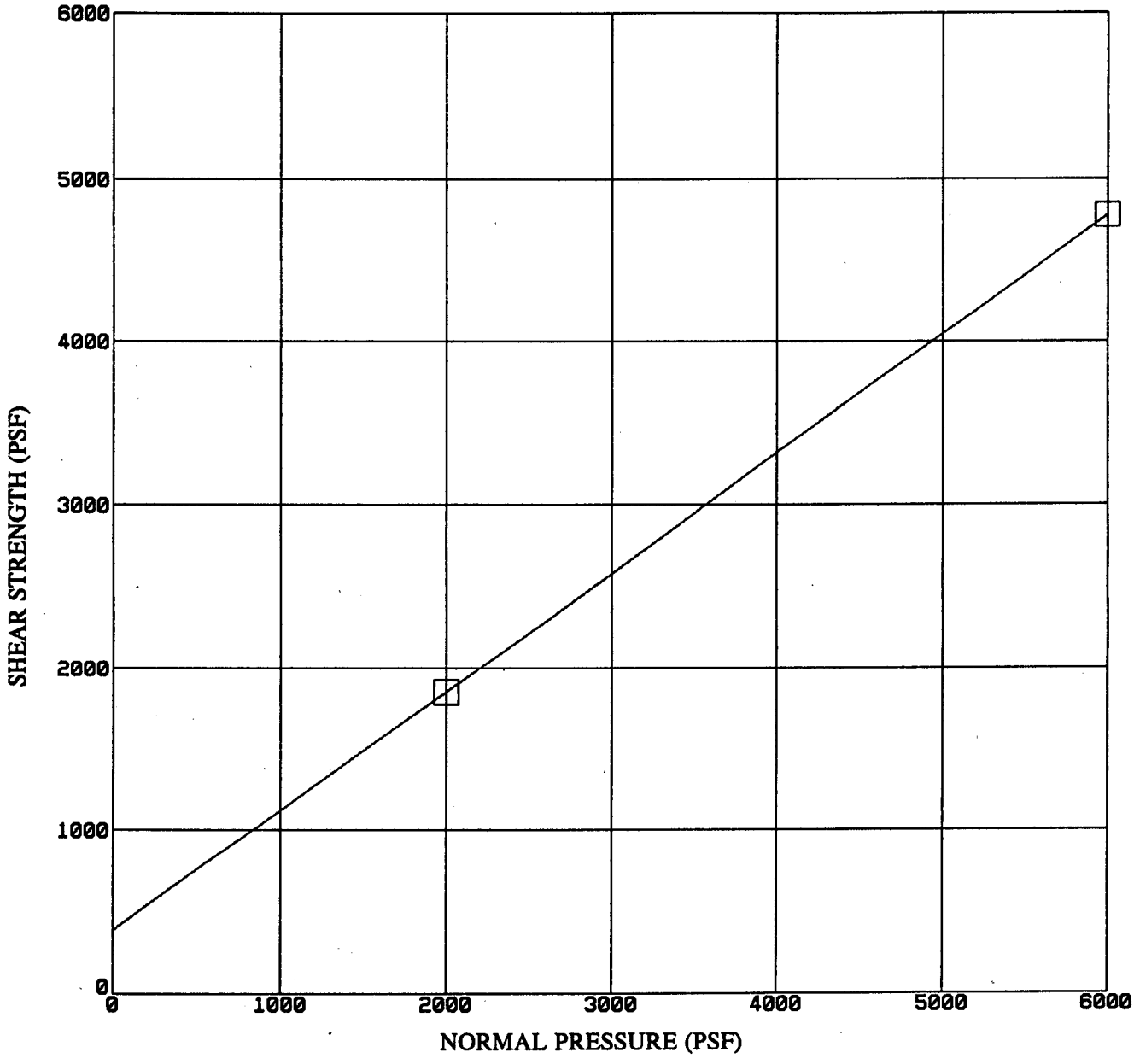


DIRECT SHEAR TEST PLOT OF B5 @ 10 FT.



SHEAR TEST DIAGRAM

Client: Rancho Las Flores Limited Partnership	W.O. 215-1
Project: Highway 138	Location: San Bernardino County

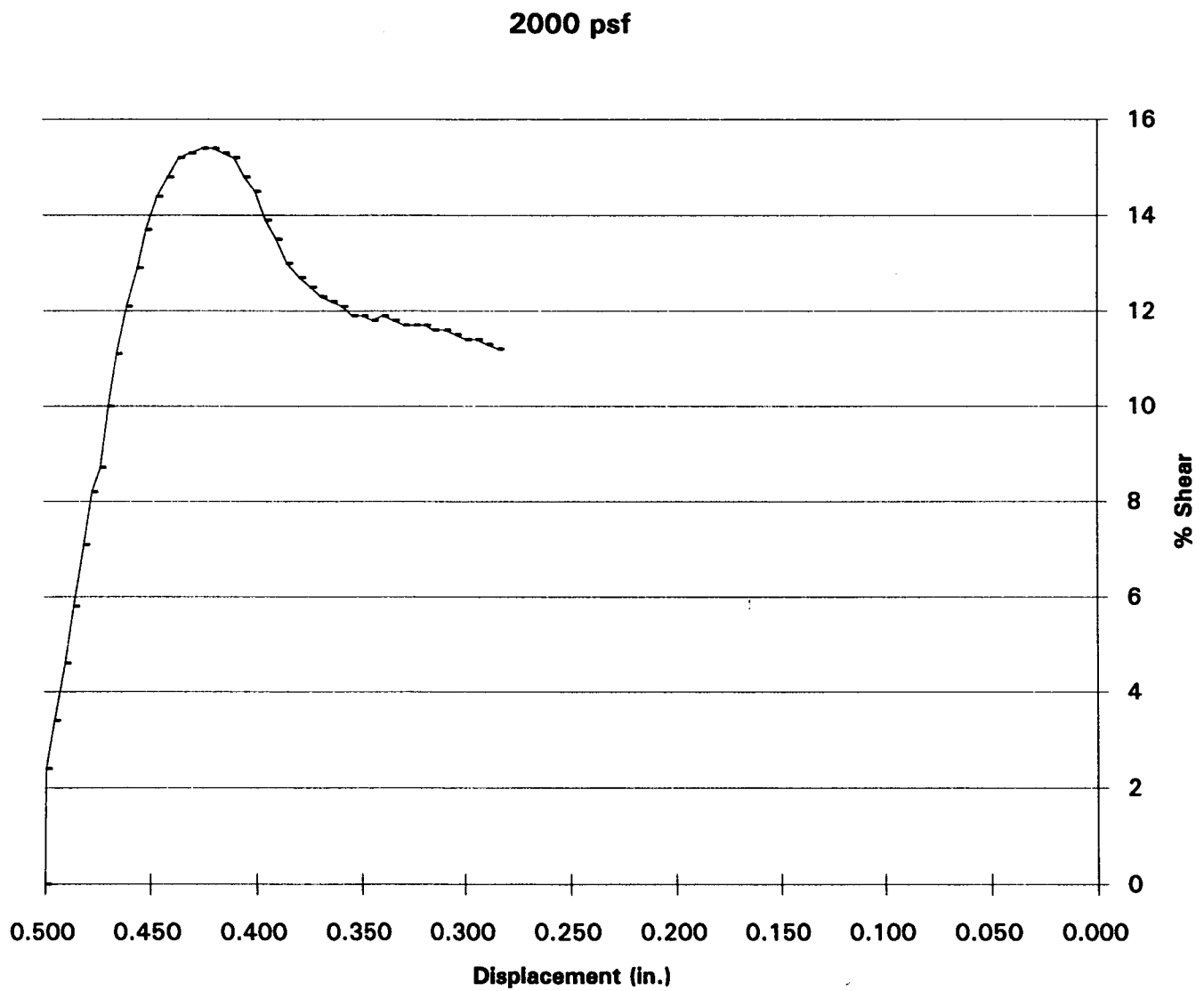


Sample: B-10 @ 10.0 ft.

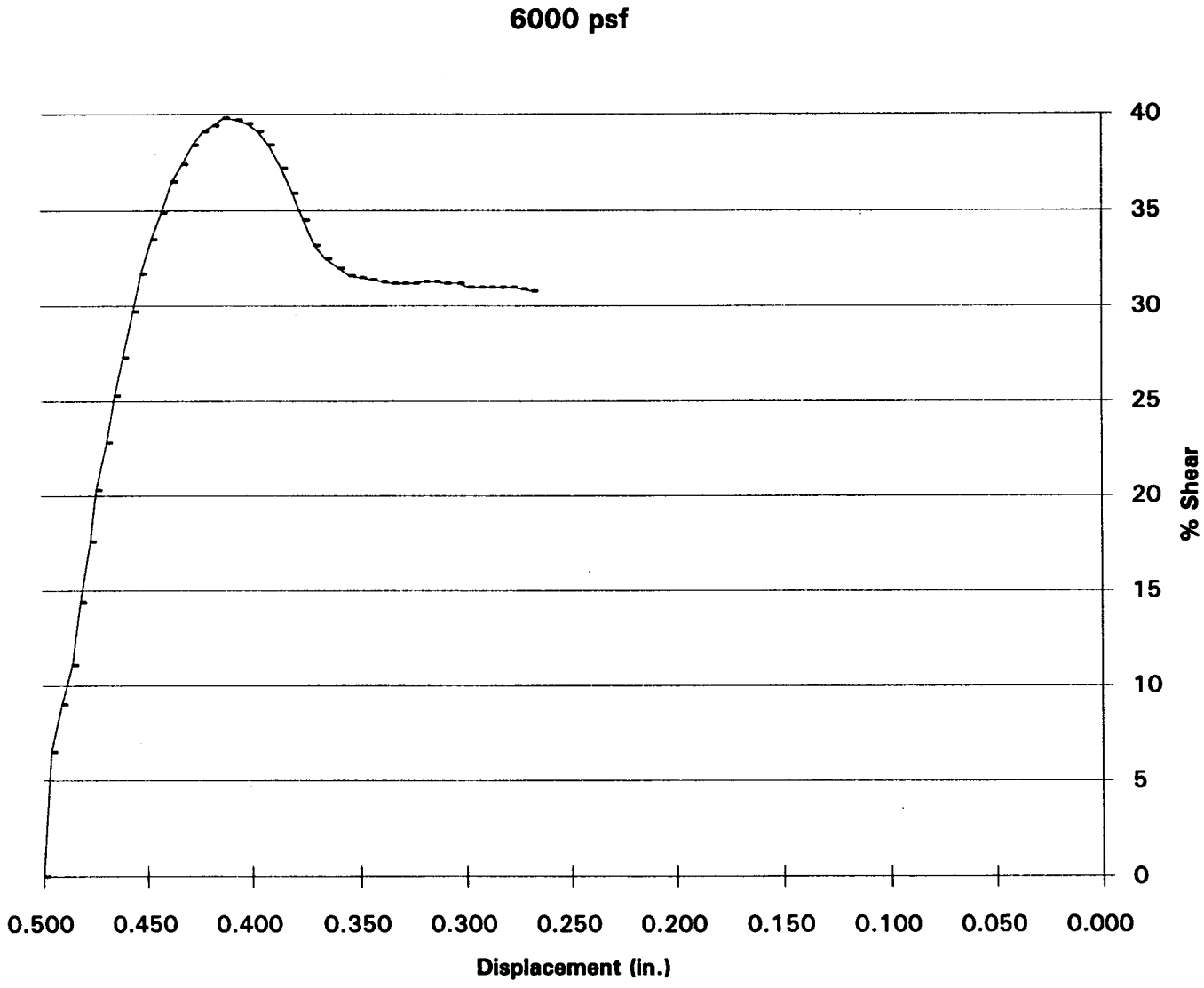
Phi Angle: 36 deg.
Cohesion: 384 psf

TEST CONDITION: Undisturbed, tested at saturation

DIRECT SHEAR TEST PLOT OF B10 @ 10 FT.

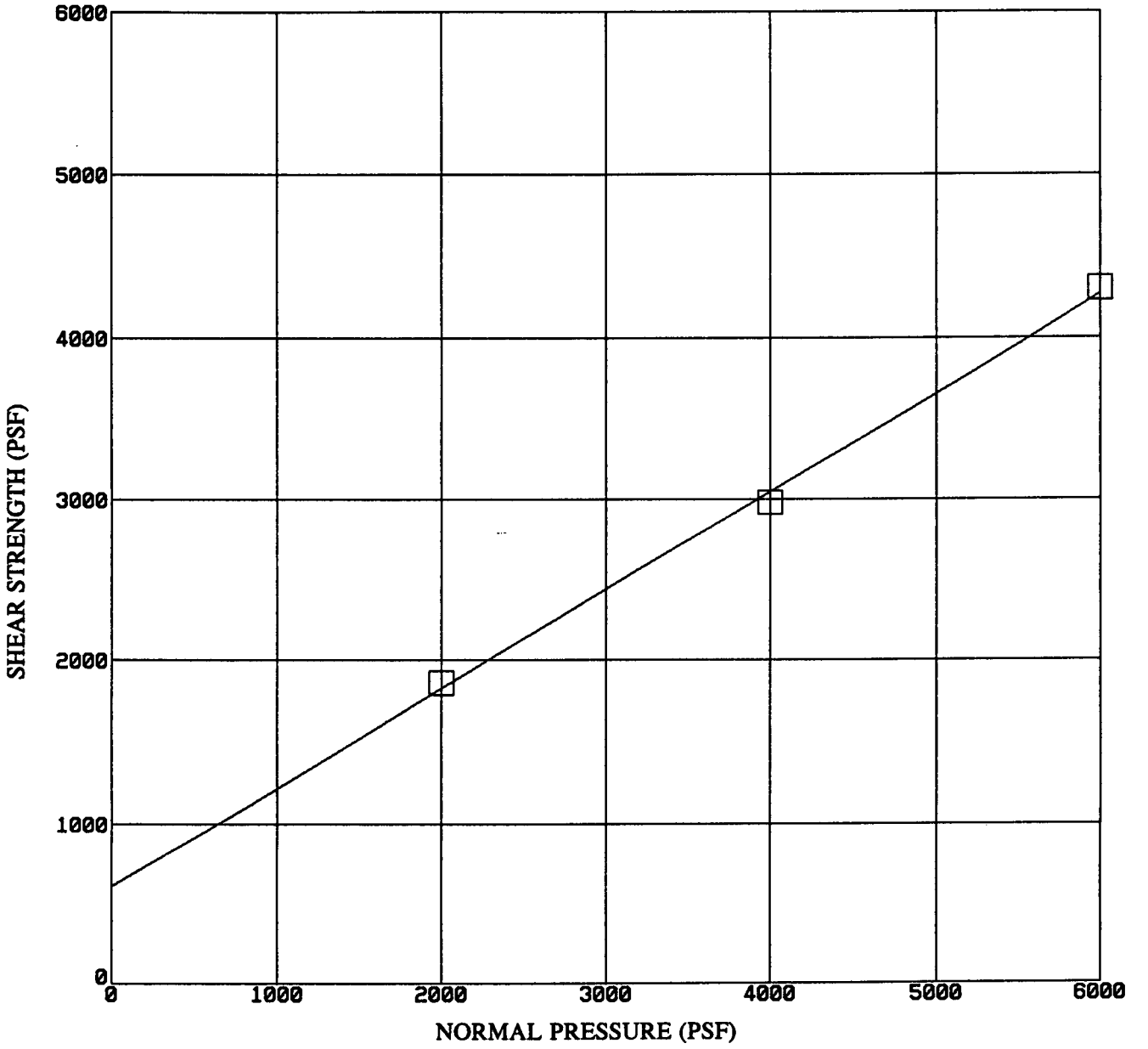


DIRECT SHEAR TEST PLOT OF B10 @ 10 FT.



SHEAR TEST DIAGRAM

Client: Rancho Las Flores Limited Partnership	W.O. 215-1
Project: Highway 138	Location: San Bernardino County



Initial Sample

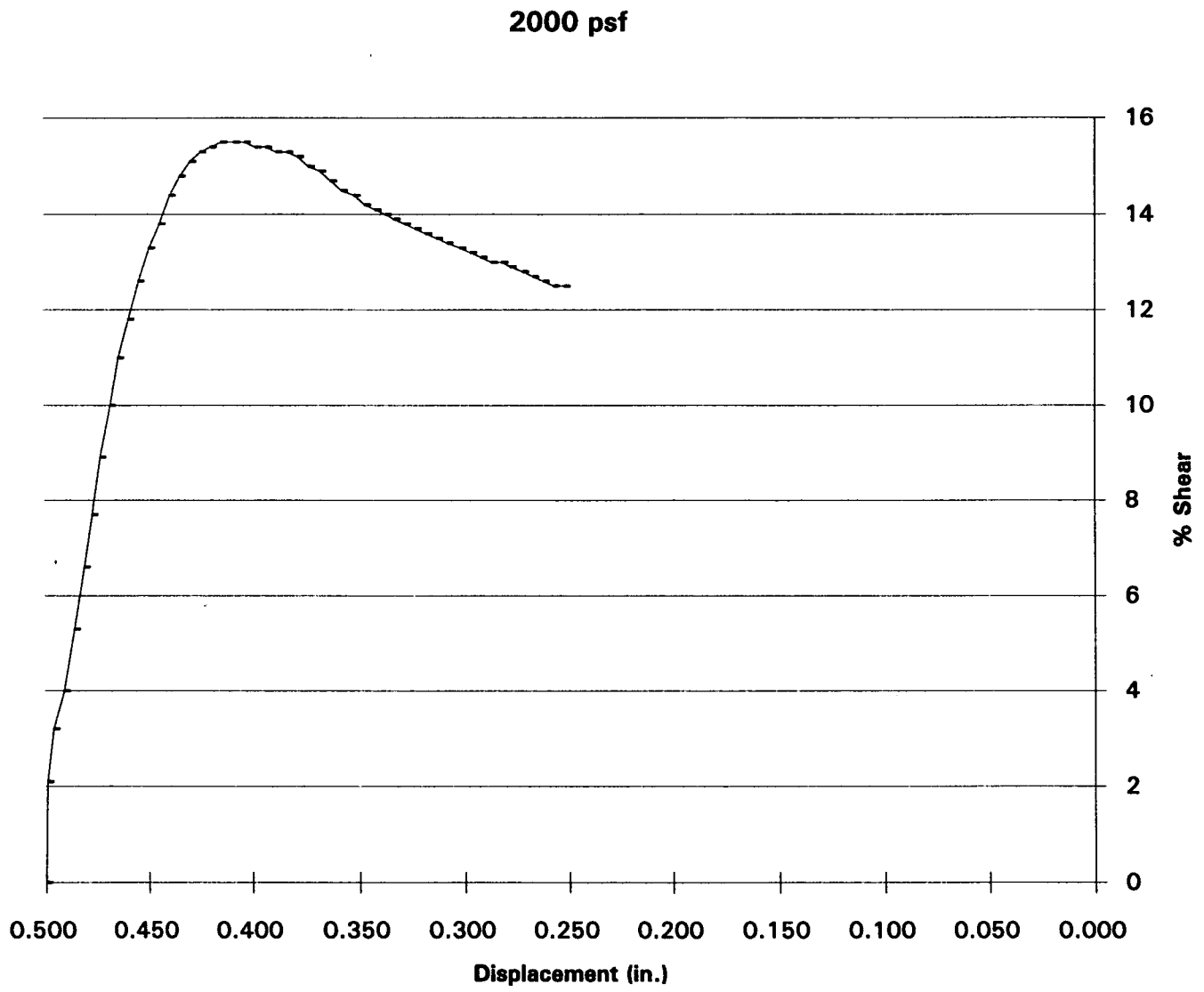
Dry Density = 117.9 pcf
Moist. Cont. = 6.4%

Sample: **B-11 @ 5.0 ft.**

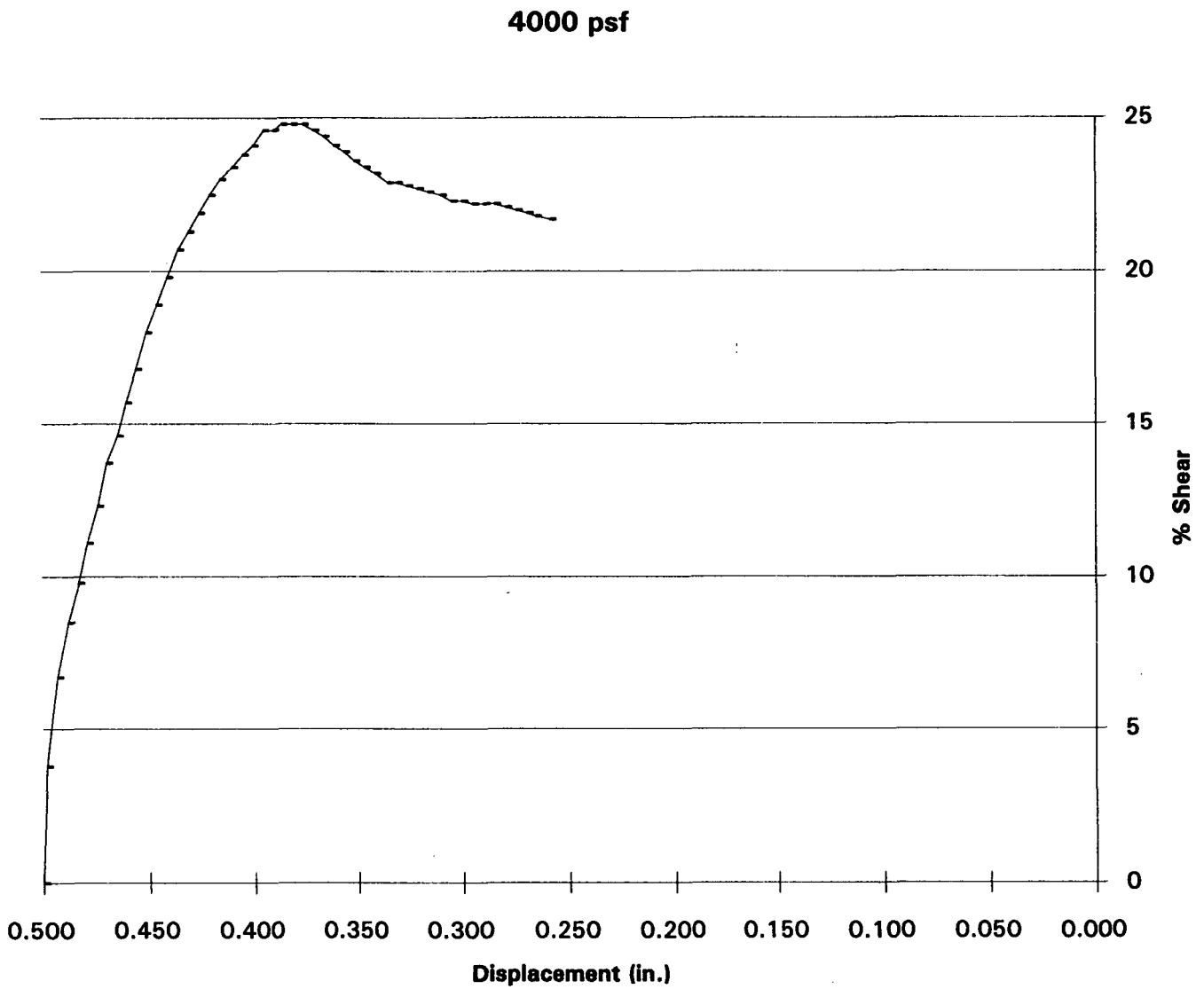
Phi Angle: **31 deg.**
Cohesion: **608 psf**

TEST CONDITION: Undisturbed, tested at saturation

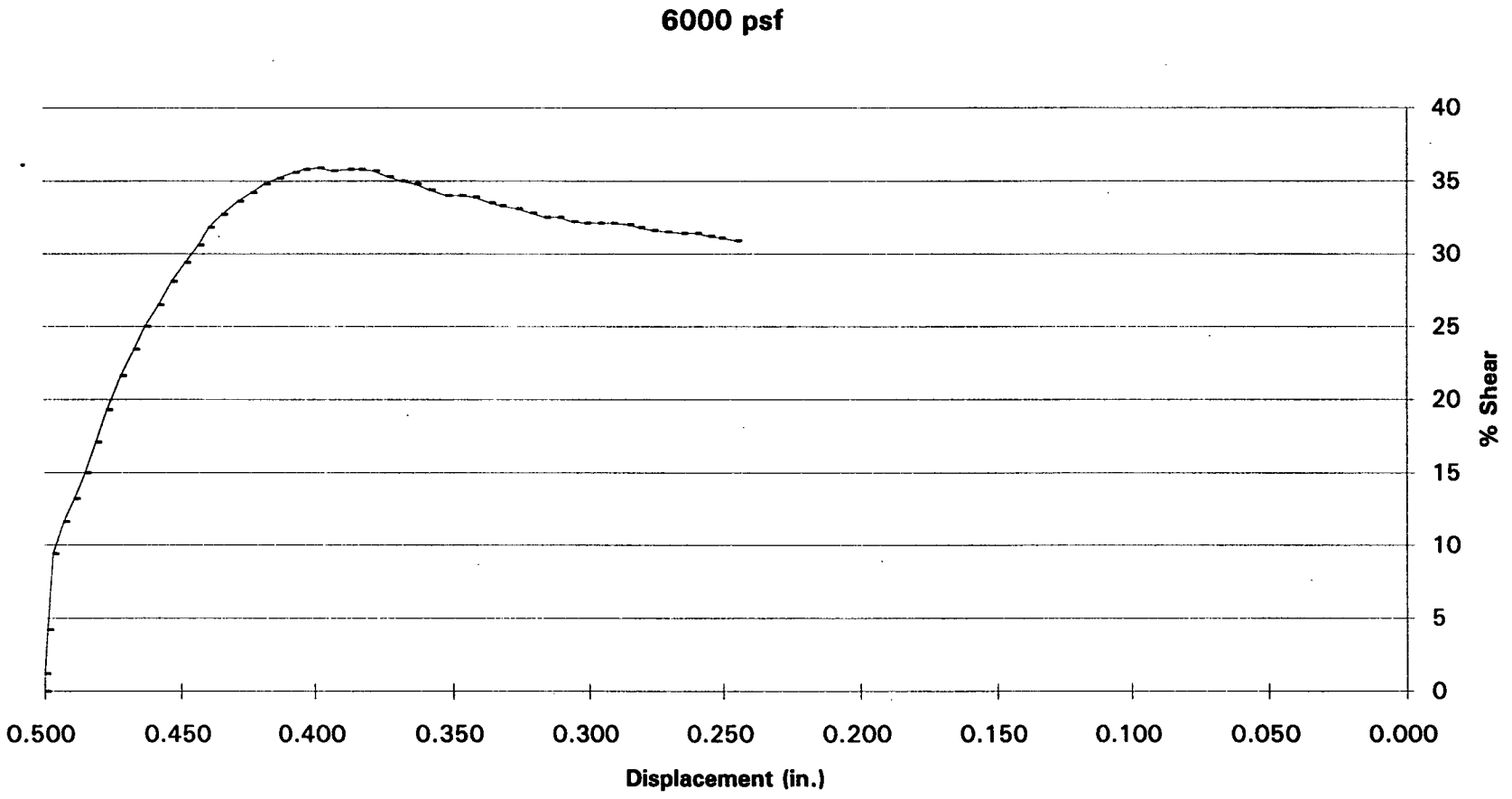
DIRECT SHEAR TEST PLOT OF B11 @ 5 FT.



DIRECT SHEAR TEST PLOT OF B11 @ 5 FT.

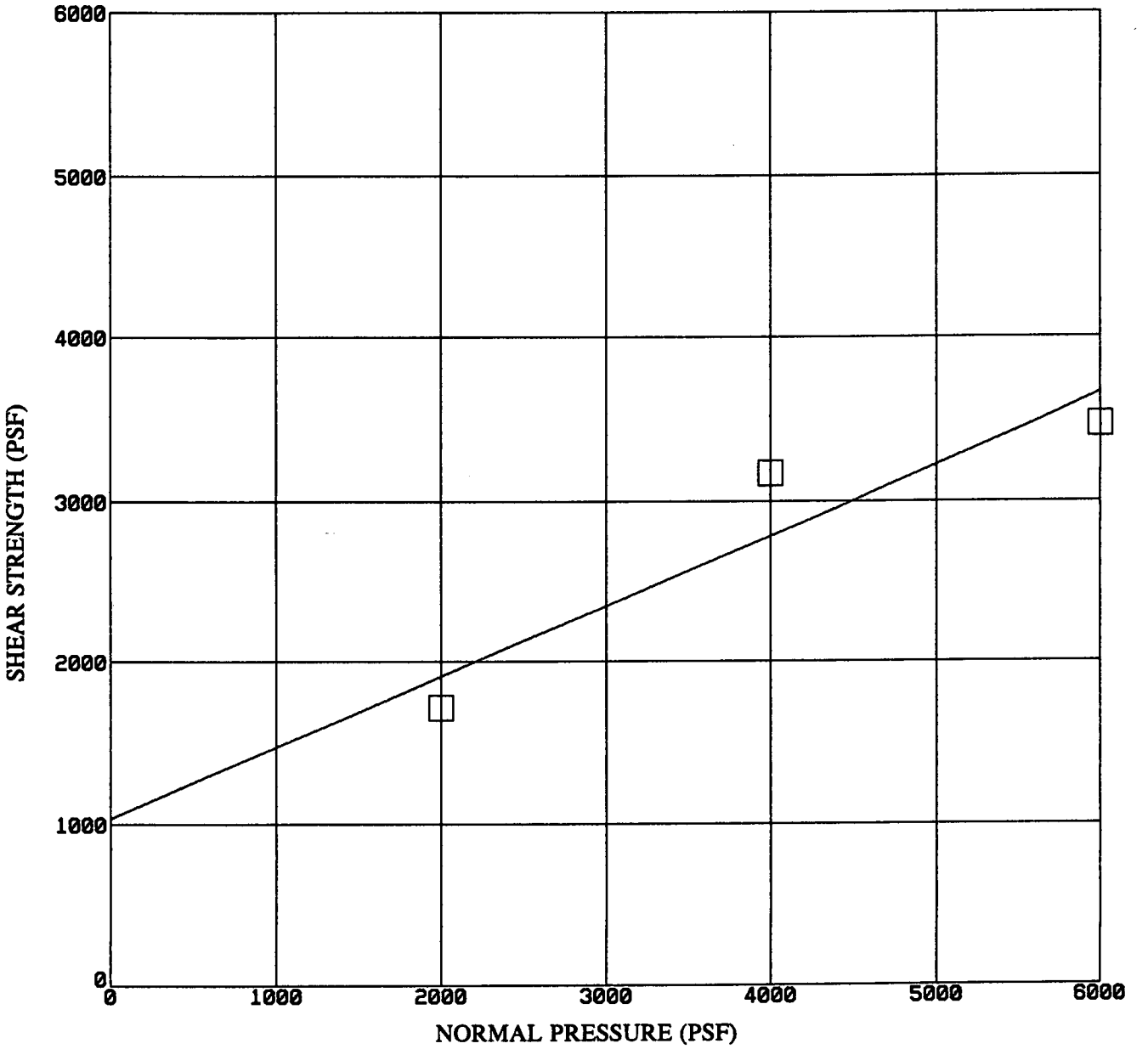


DIRECT SHEAR TEST PLOT OF B-11 @ 5 FT.



SHEAR TEST DIAGRAM

Client: **Rancho Las Flores Limited Partnership**	W.O. **215-1**
Project: **Highway 138**	Location: **San Bernardino County**



Initial Sample

Dry Density = 120.3 pcf
Moist. Cont. = 5.4%

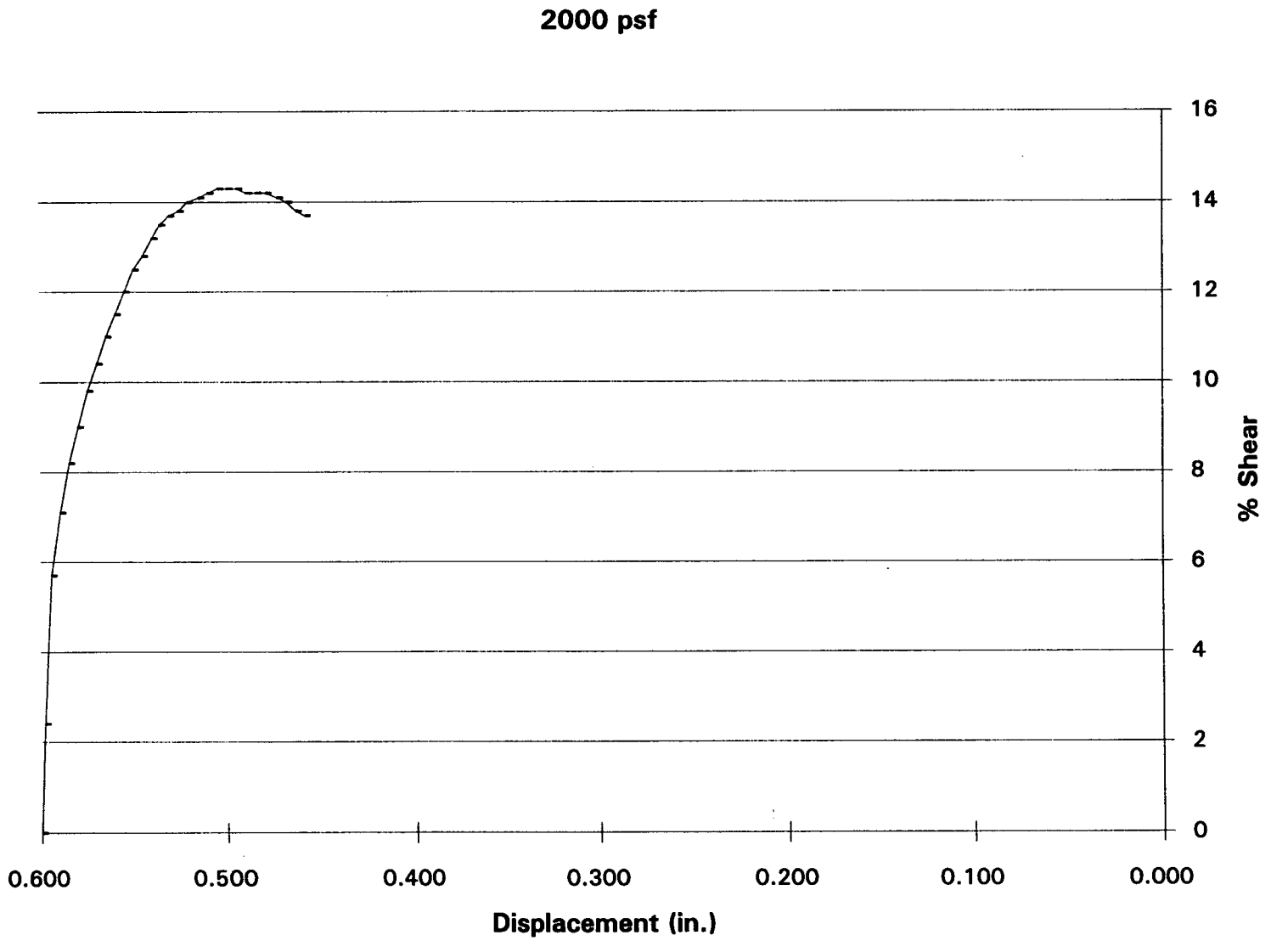
Sample: **B-12 @ 20.0 ft.**

Phi Angle: **24 deg.**
Cohesion: **1032 psf**

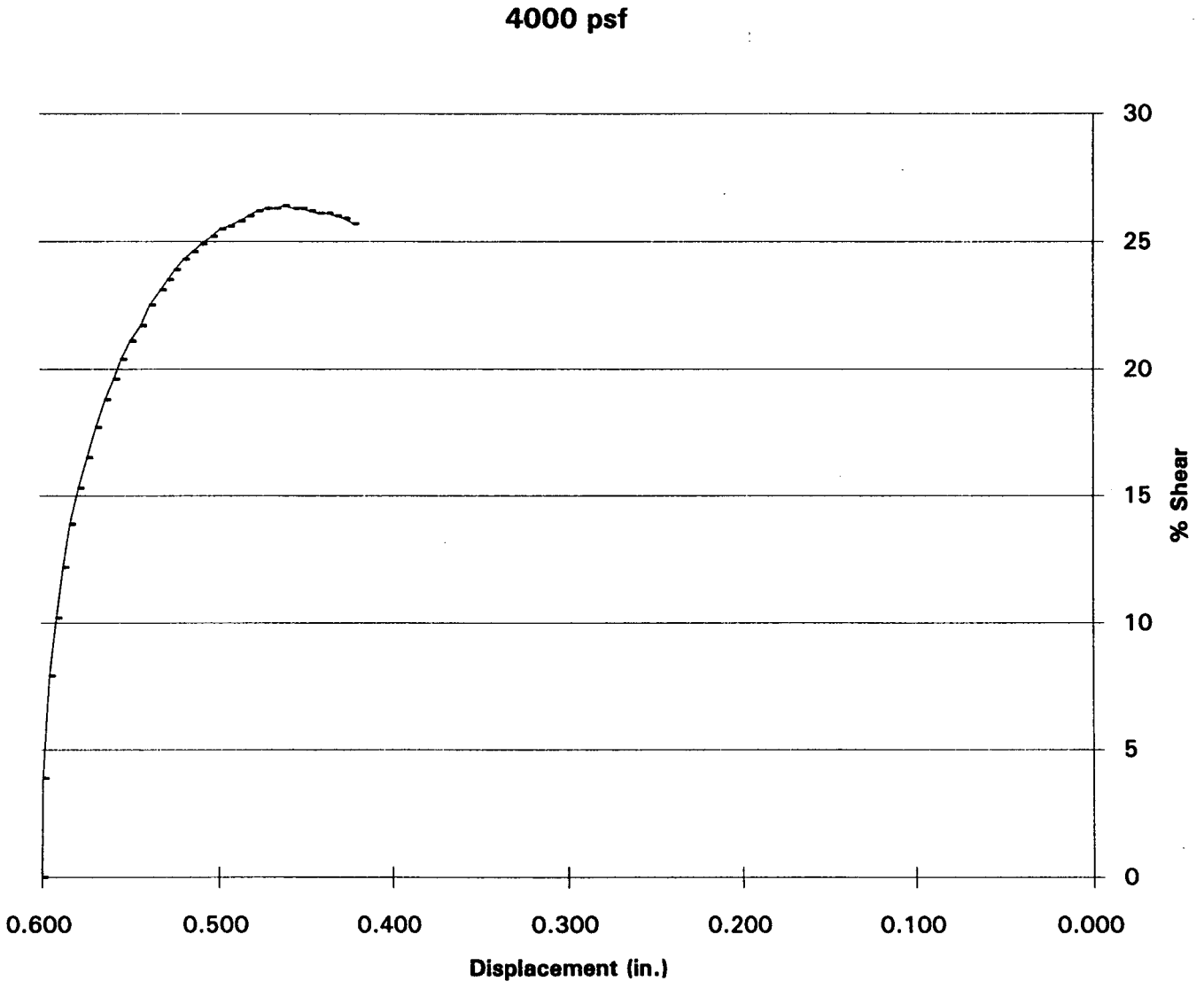
TEST CONDITION: Undisturbed, tested at saturation

John A. Sayers & Associates GEOTECHNICAL CONSULTANTS	Plate

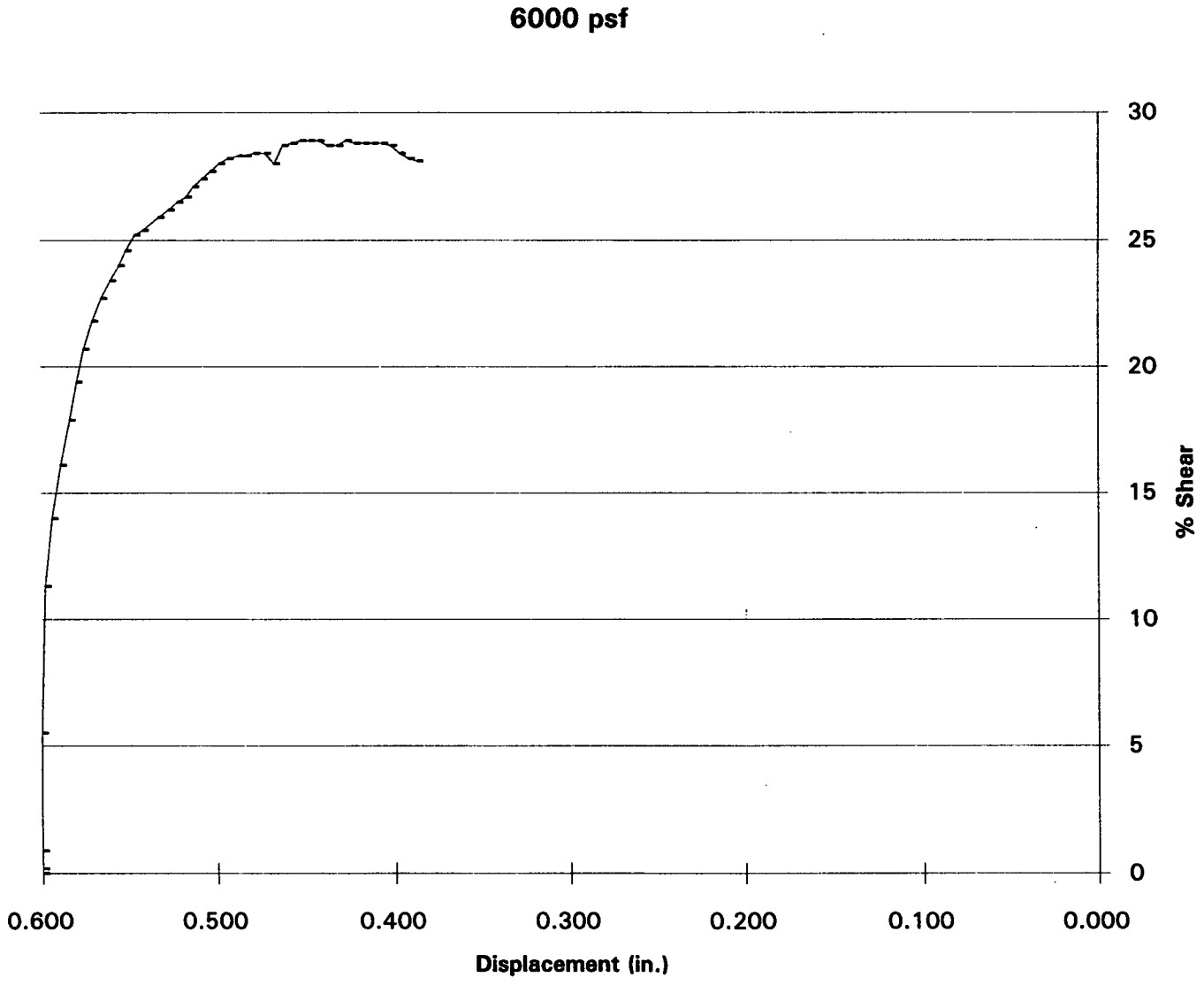
DIRECT SHEAR TEST PLOT OF B12 @ 20 FT.



DIRECT SHEAR TEST PLOT OF B12 @ 20 FT.

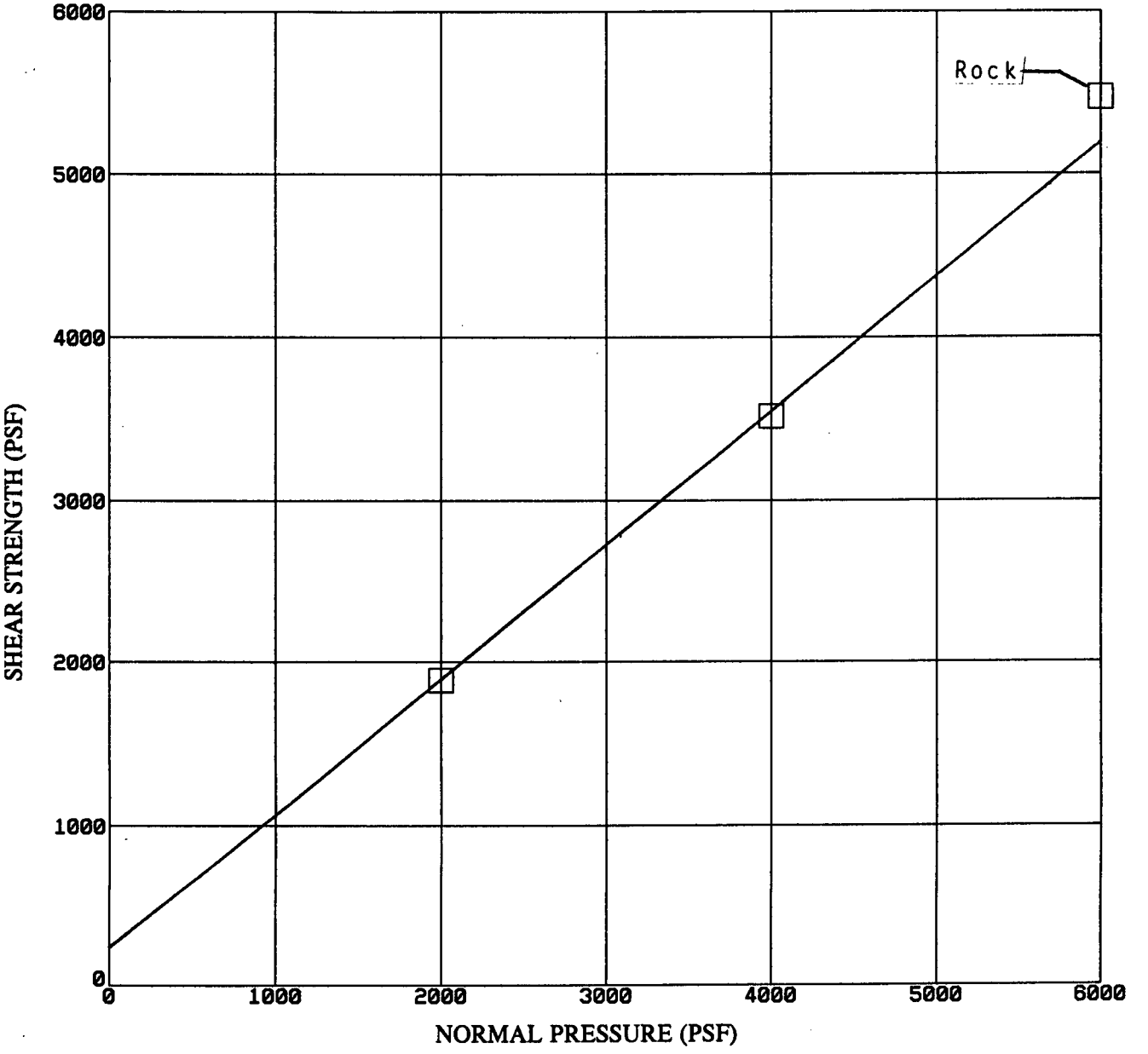


DIRECT SHEAR TEST PLOT OF B12 @ 20 FT.



SHEAR TEST DIAGRAM

Client: **Rancho Las Flores Limited Partnership**	W.O. **215-1**
Project: **Highway 138**	Location: **San Bernardino County**



Initial Sample

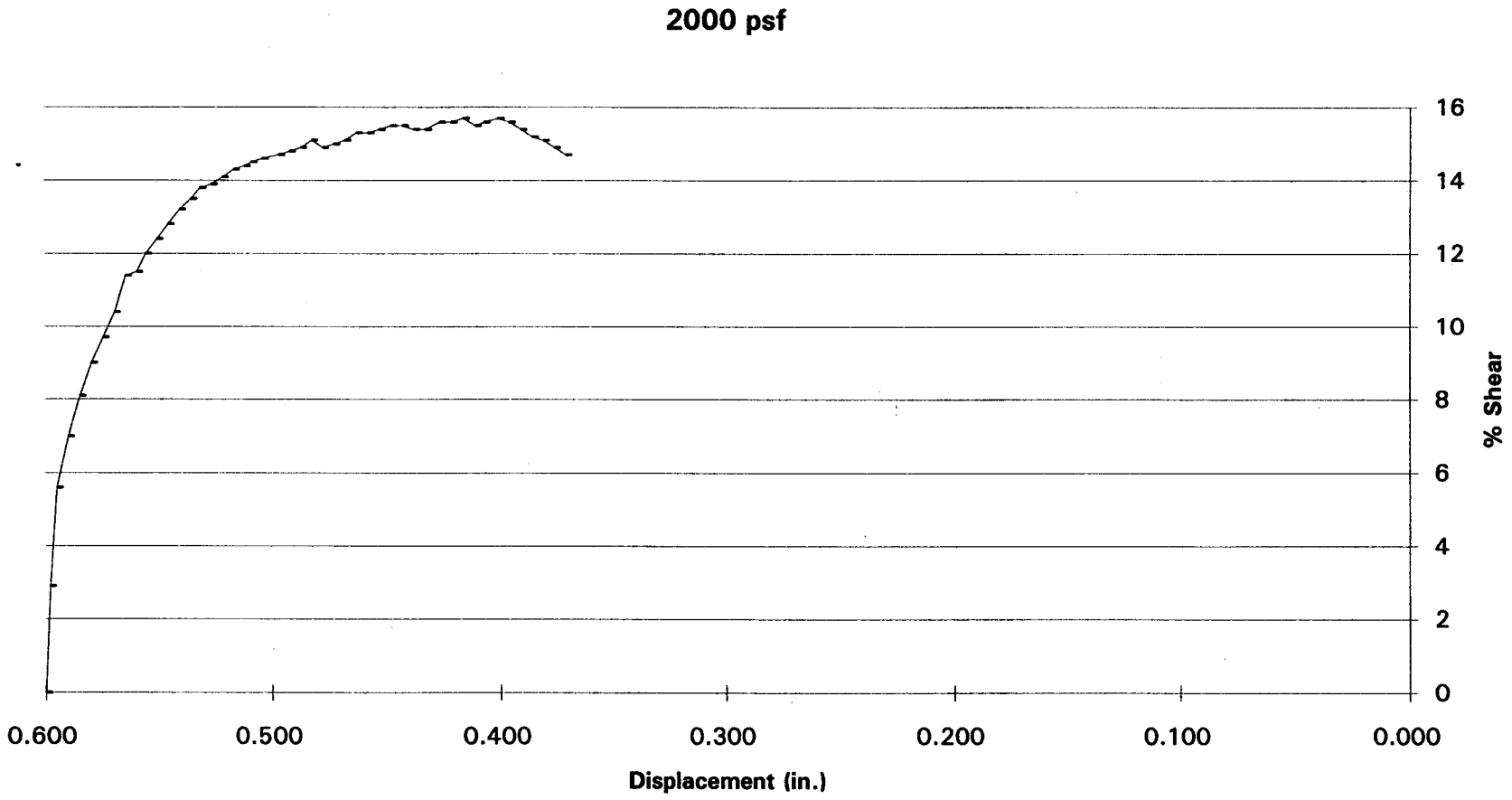
Dry Density = 111.5 pcf
Moist. Cont. = 5.3%

Sample: **B-13 @ 5.0 ft.**

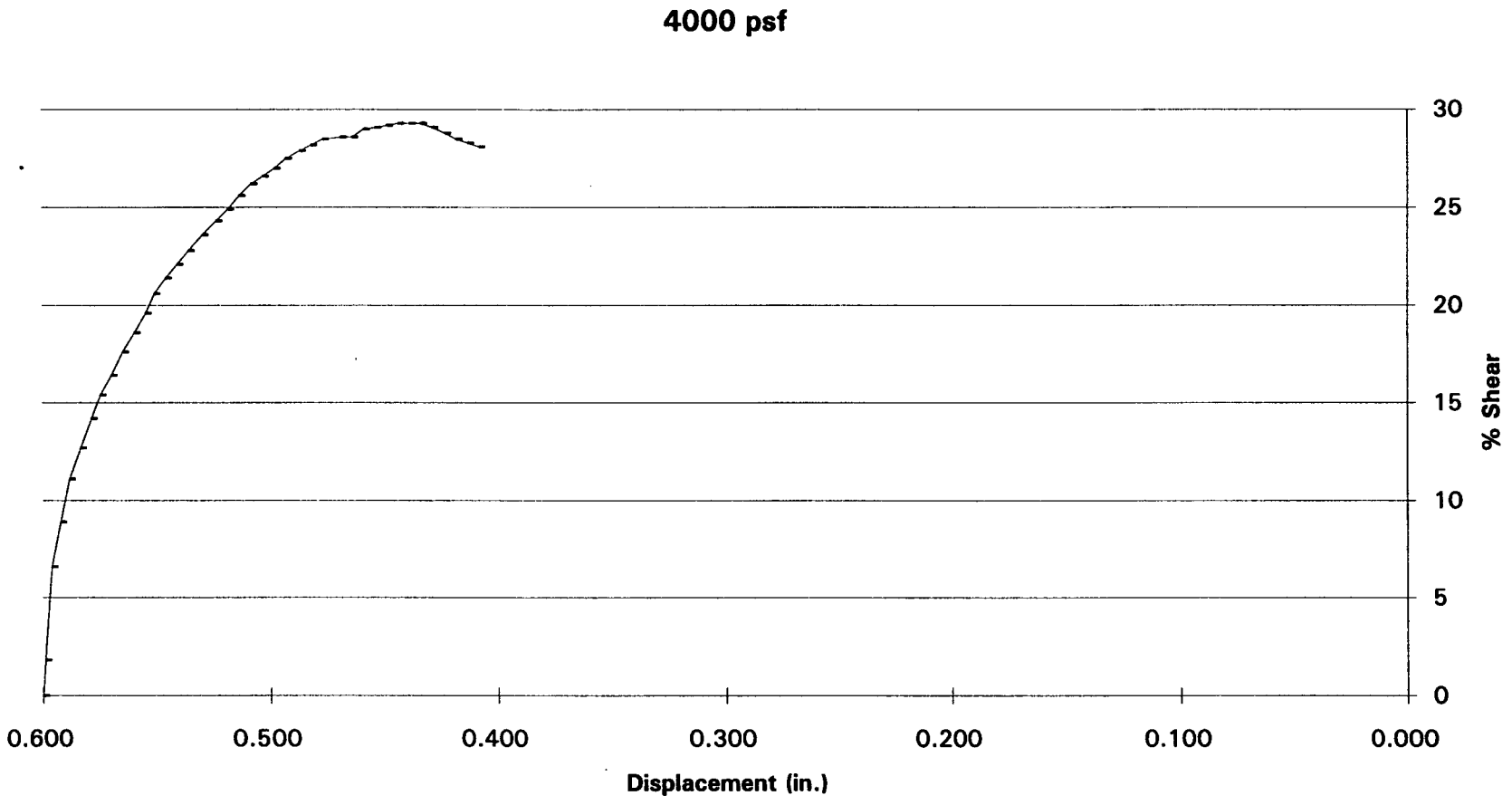
Phi Angle: **39 deg.**
Cohesion: **230 psf**

TEST CONDITION: Undisturbed, tested at saturation

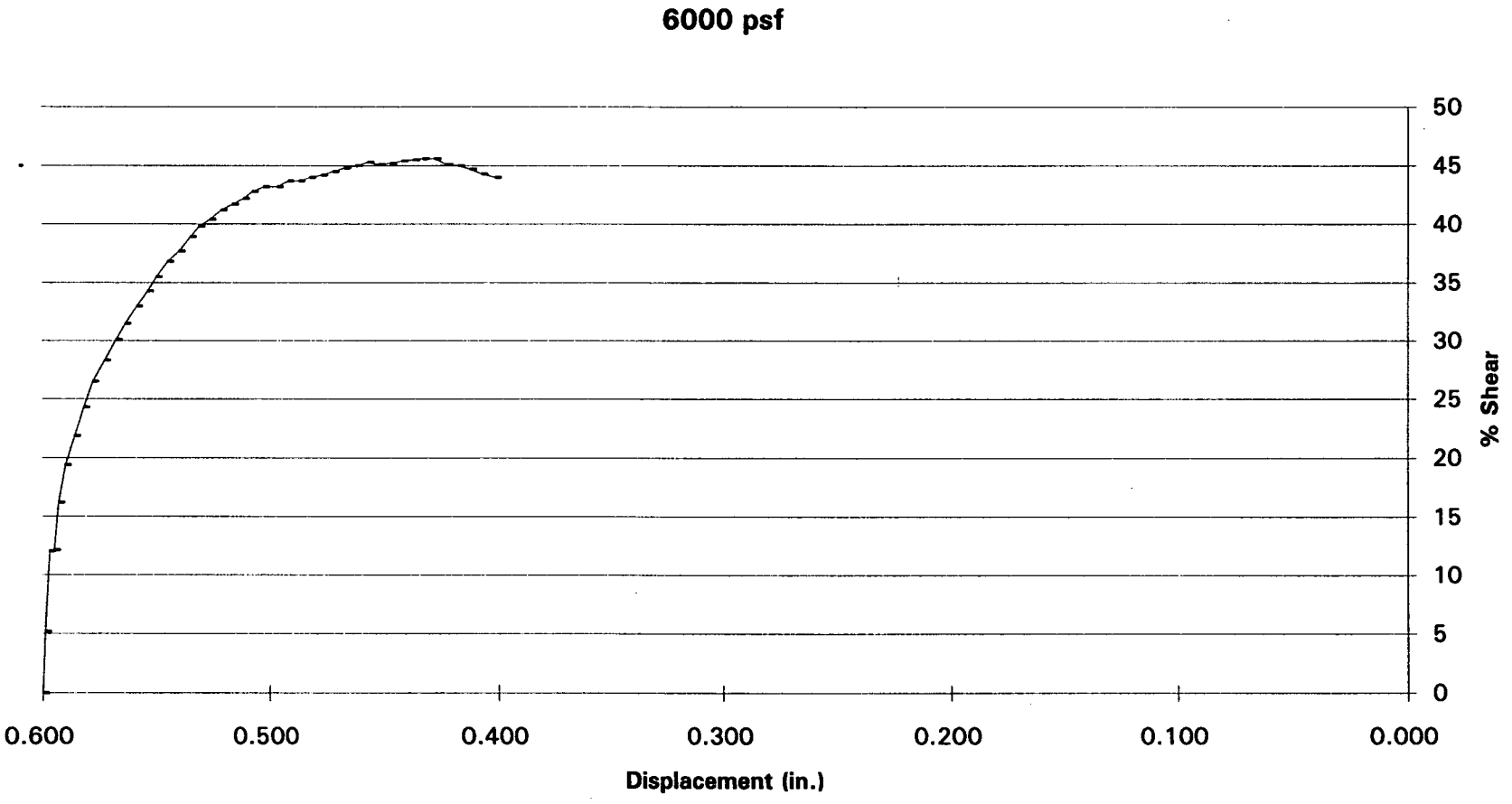
DIRECT SHEAR TEST PLOT OF B-13 @ 5 FT.



DIRECT SHEAR TEST PLOT OF B-13 @ 5 FT.

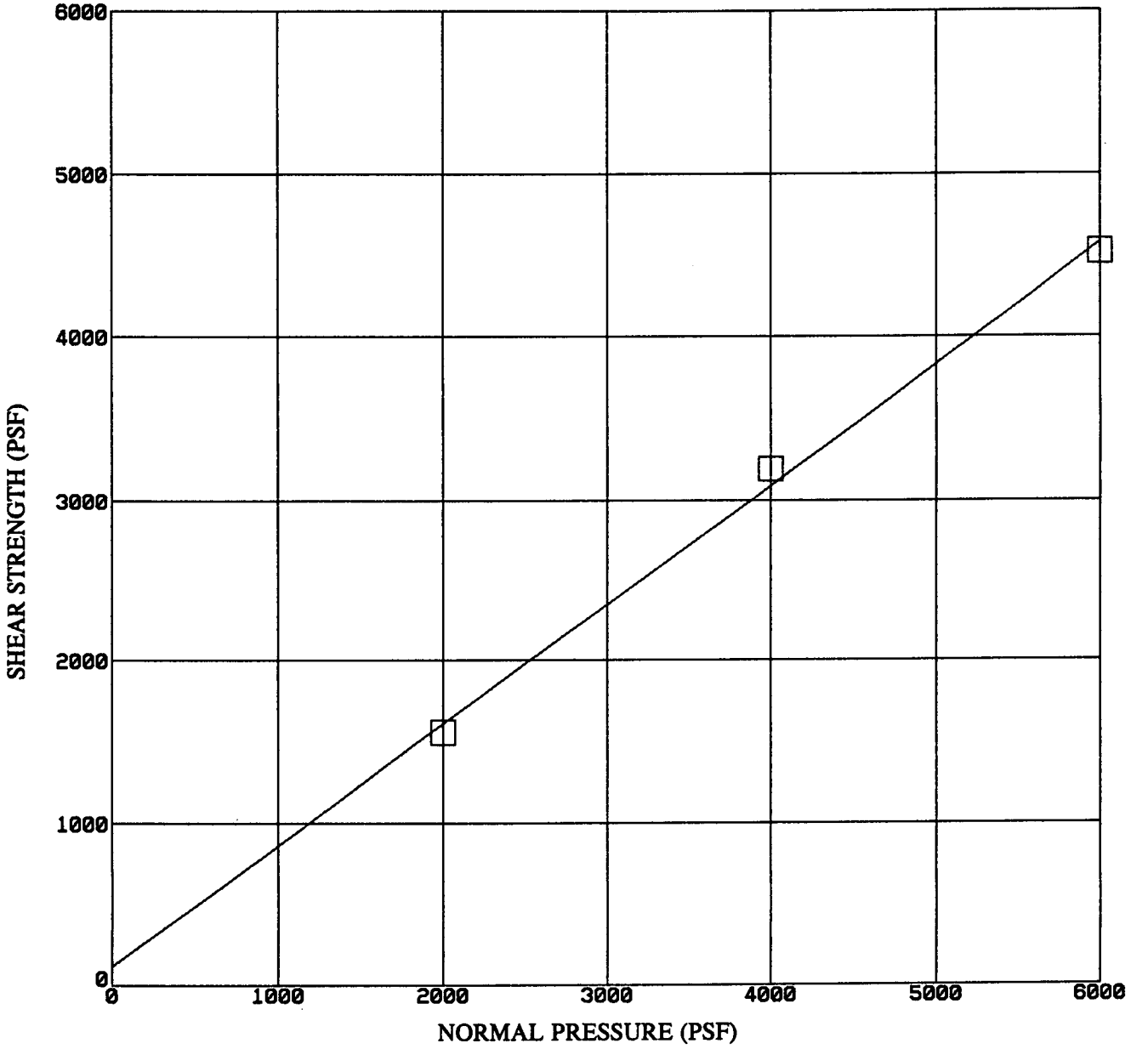


DIRECT SHEAR TEST PLOT OF B-13 @ 5 FT.



SHEAR TEST DIAGRAM

Client: **Rancho Las Flores Limited Partnership**	W.O. **215-1**
Project: **Highway 138**	Location: **San Bernardino County**



Initial Sample

Dry Density = 121.9 pcf
Moist. Cont. = 3.1%

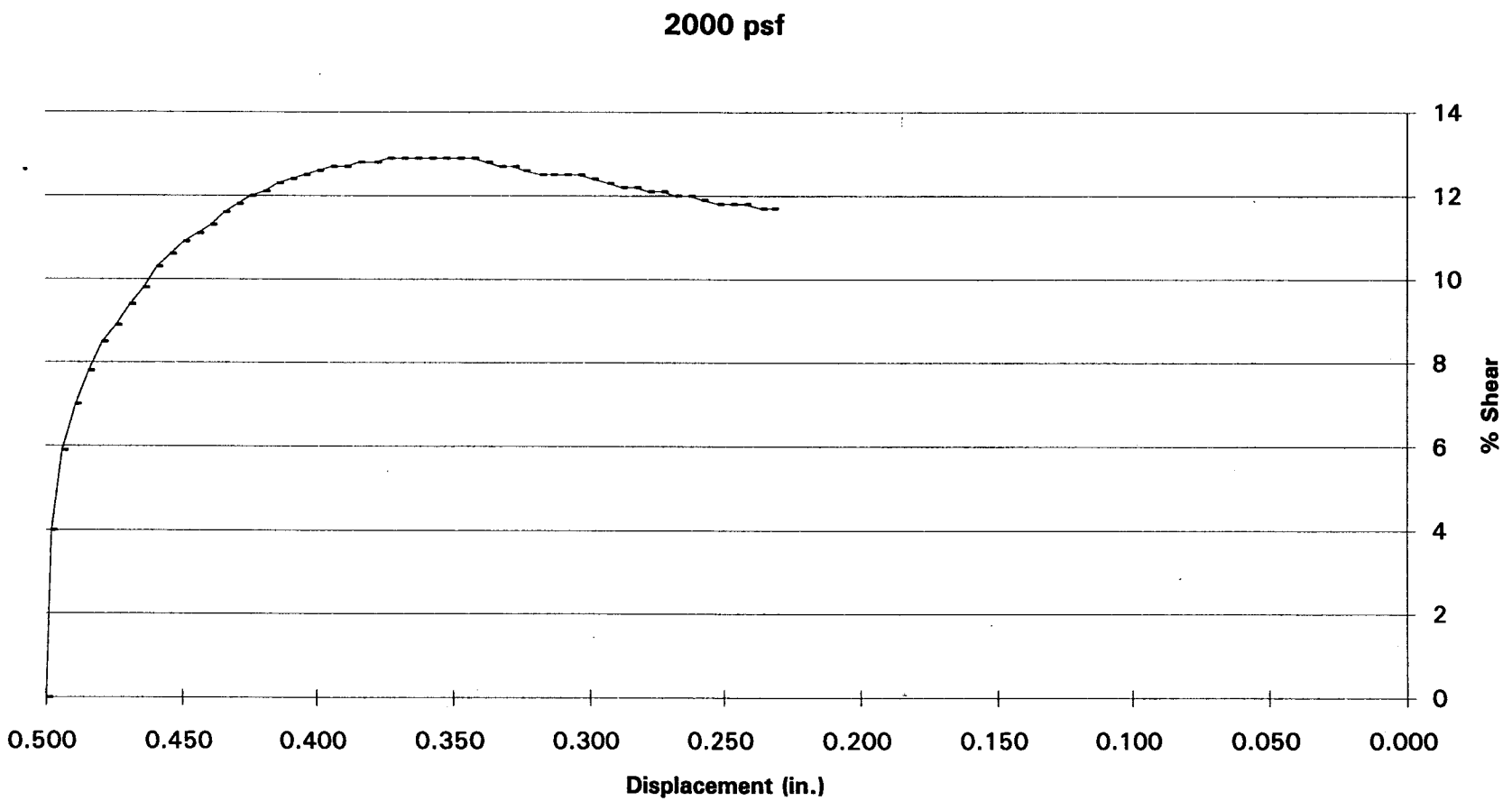
Sample: **B-13 @ 15.0 ft.**

Phi Angle: **37 deg.**
Cohesion: **120 psf**

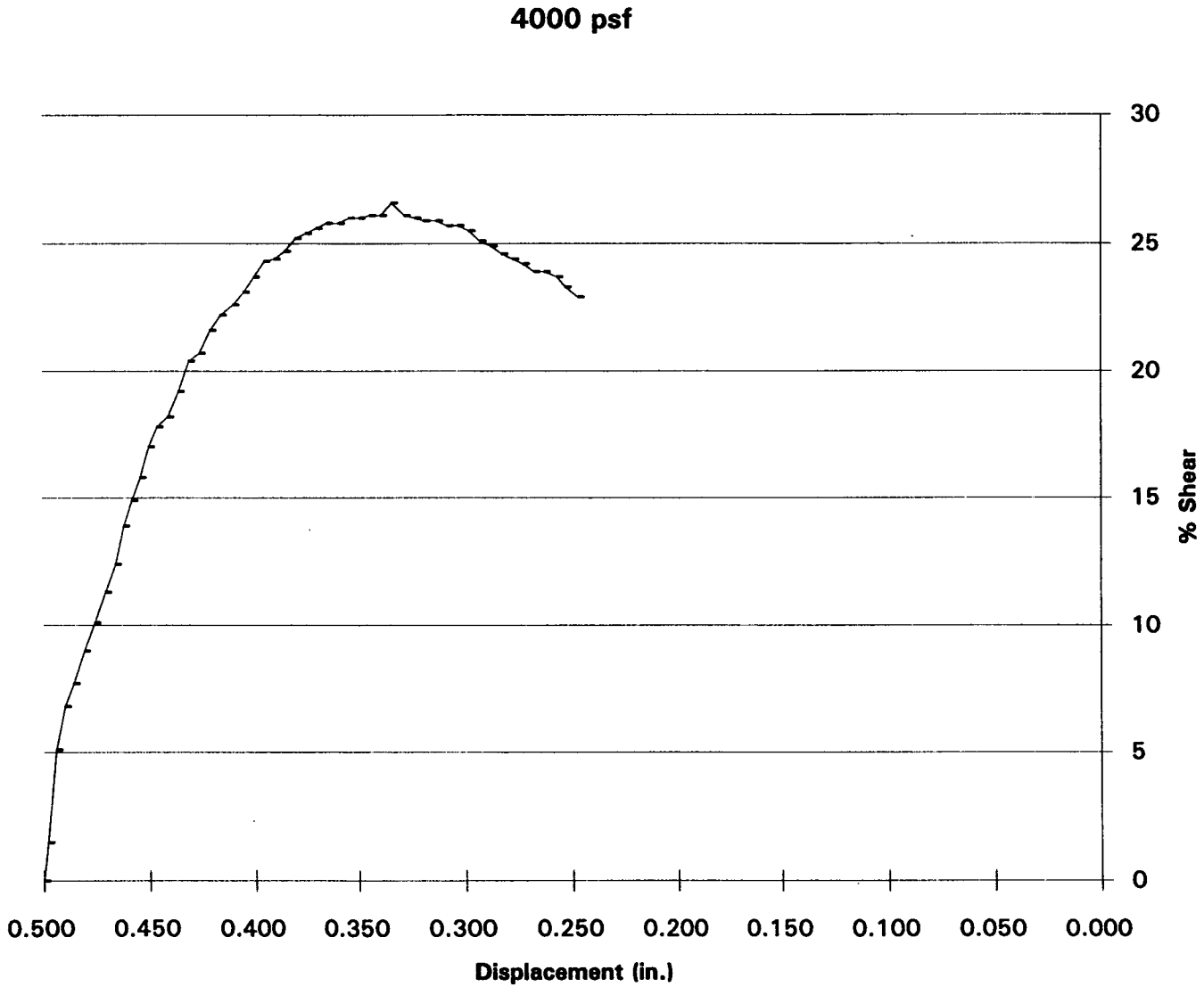
TEST CONDITION: Undisturbed, tested at saturation

John A. Sayers & Associates GEOTECHNICAL CONSULTANTS	Plate

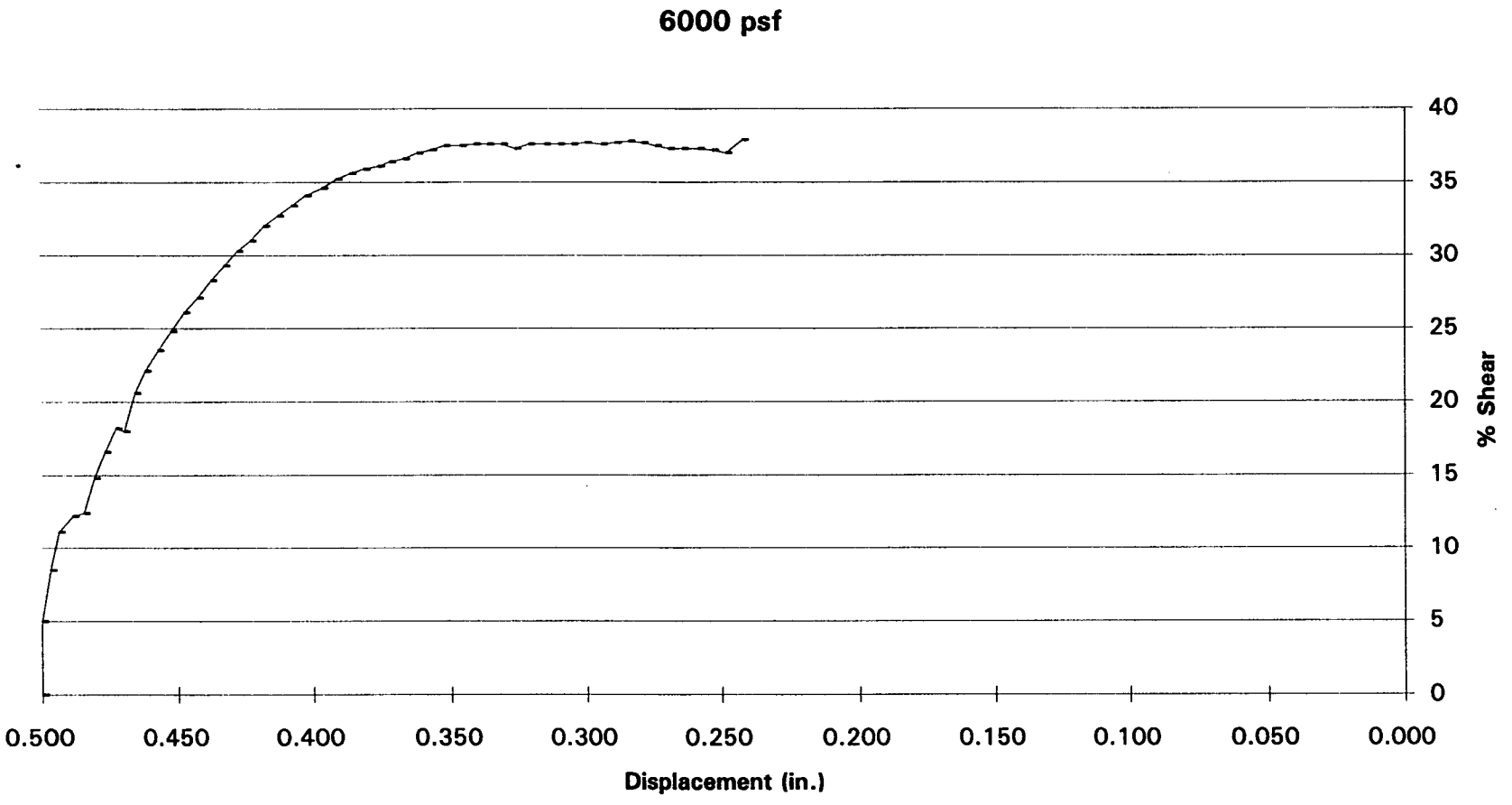
DIRECT SHEAR TEST PLOT OF B-13 @ 15 FT.



DIRECT SHEAR TEST PLOT OF B13 @ 15 FT.

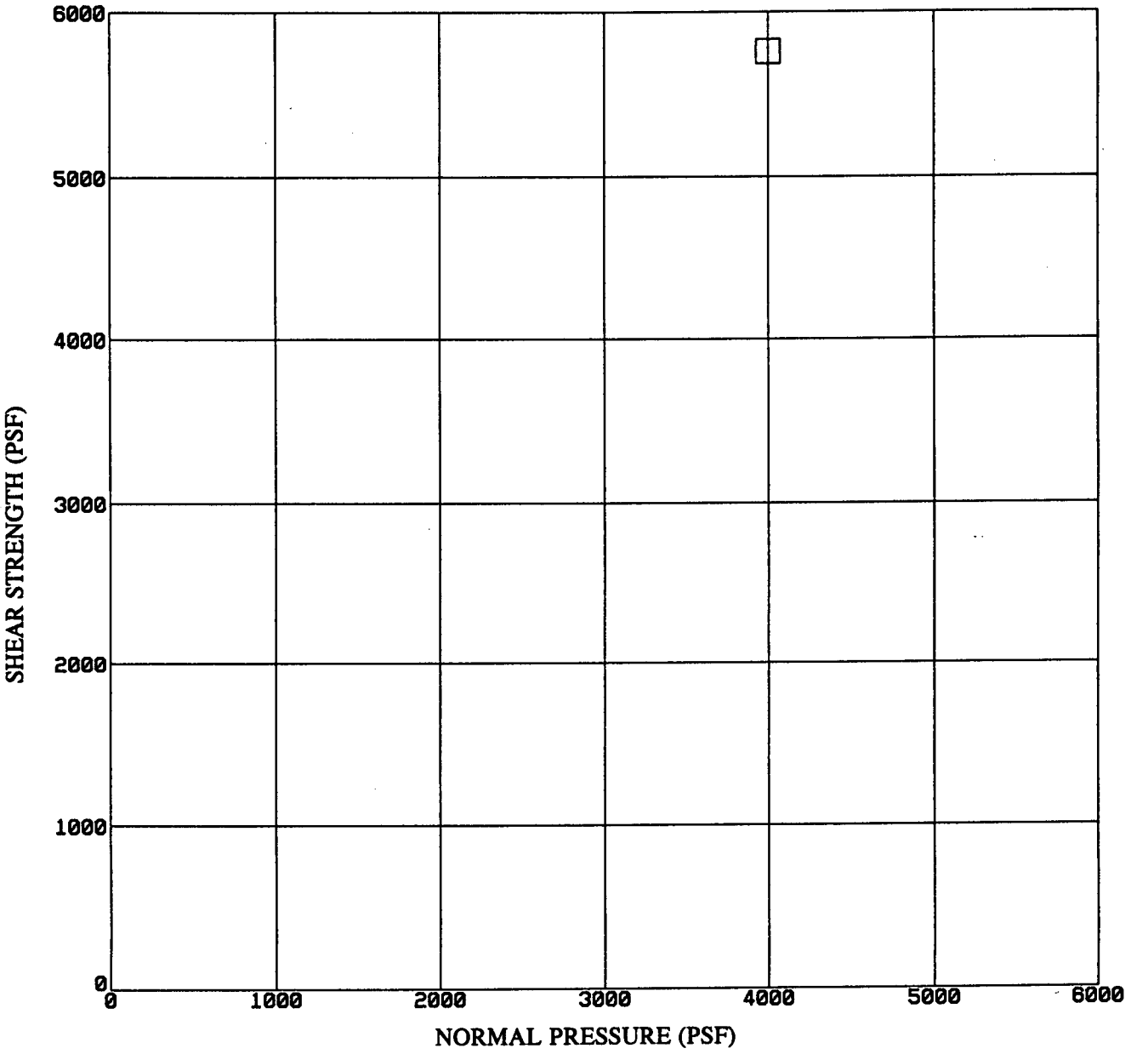


DIRECT SHEAR TEST PLOT OF B-13 @ 15 FT.



SHEAR TEST DIAGRAM

Client: **Rancho Las Flores Limited Partnership**	W.O. **215-1**
Project: **Highway 138**	Location: **San Bernardino County**



Initial Sample

Dry Density = 127.4 pcf
Moist. Cont. = 10.4%

Sample: **B-1A @ 20.0 ft.**

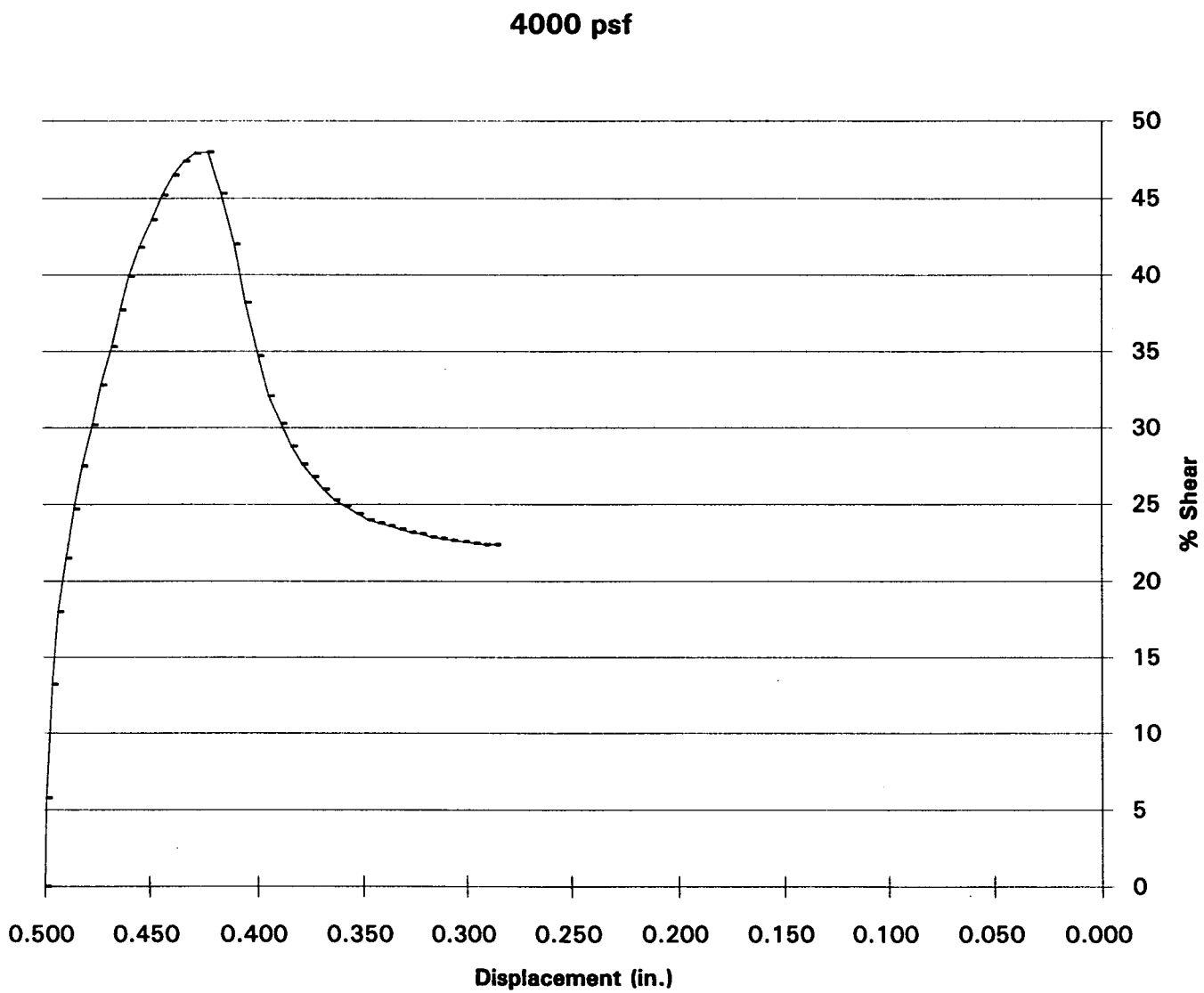
Phi Angle: **deg.**

Cohesion: **psf**

TEST CONDITION: Undisturbed, tested at saturation

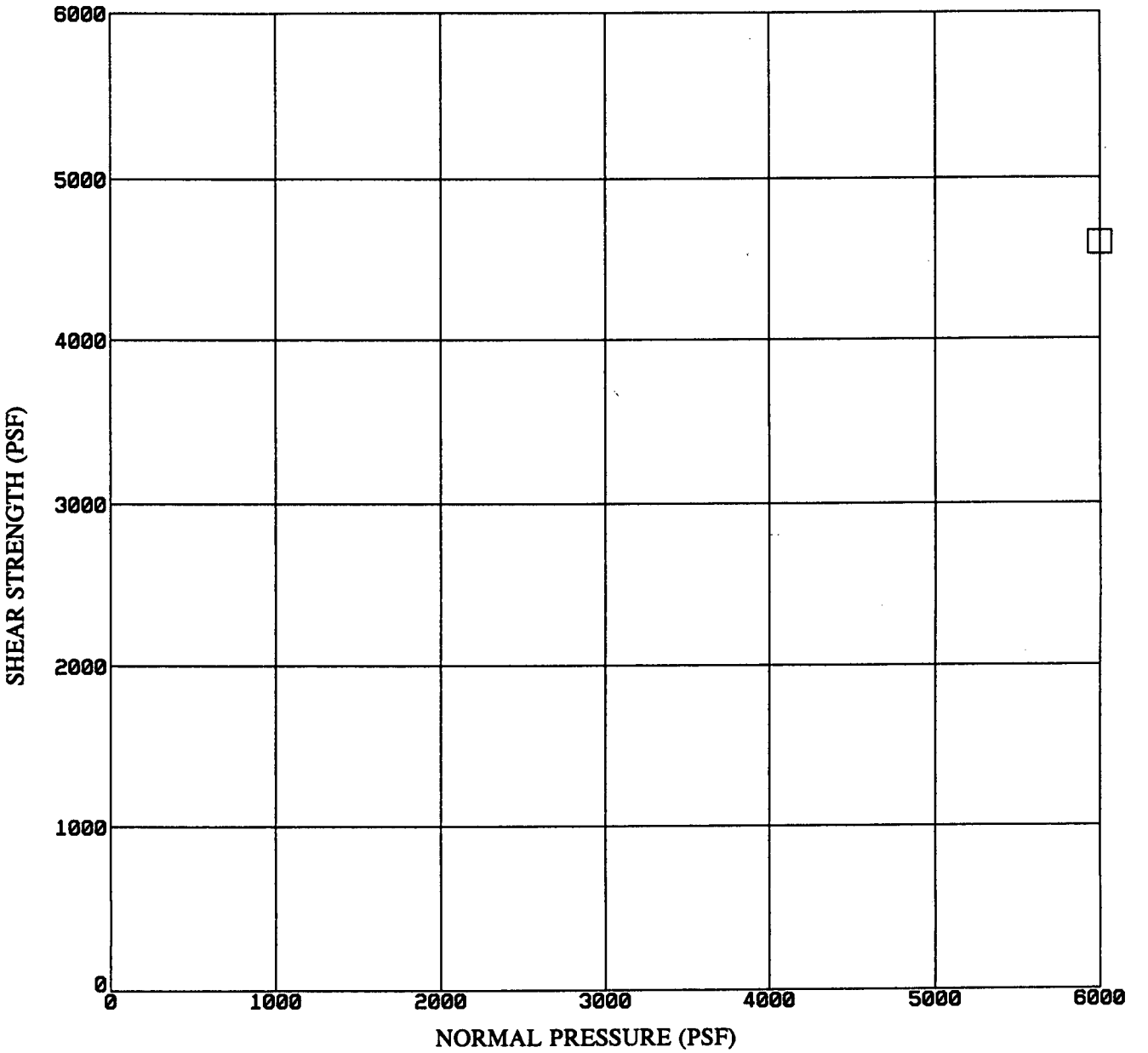
John A. Sayers & Associates GEOTECHNICAL CONSULTANTS	Plate

DIRECT SHEAR TEST PLOT OF B1A @ 20 FT. (SATURARTED)



SHEAR TEST DIAGRAM

Client: **Rancho Las Flores Limited Partnership**	W.O. **215-1**
Project: **Highway 138**	Location: **San Bernardino County**



Initial Sample

Dry Density = 124.5 pcf
Moist. Cont. = 4.7%

Sample: **B-1A @ 40.0 ft.**

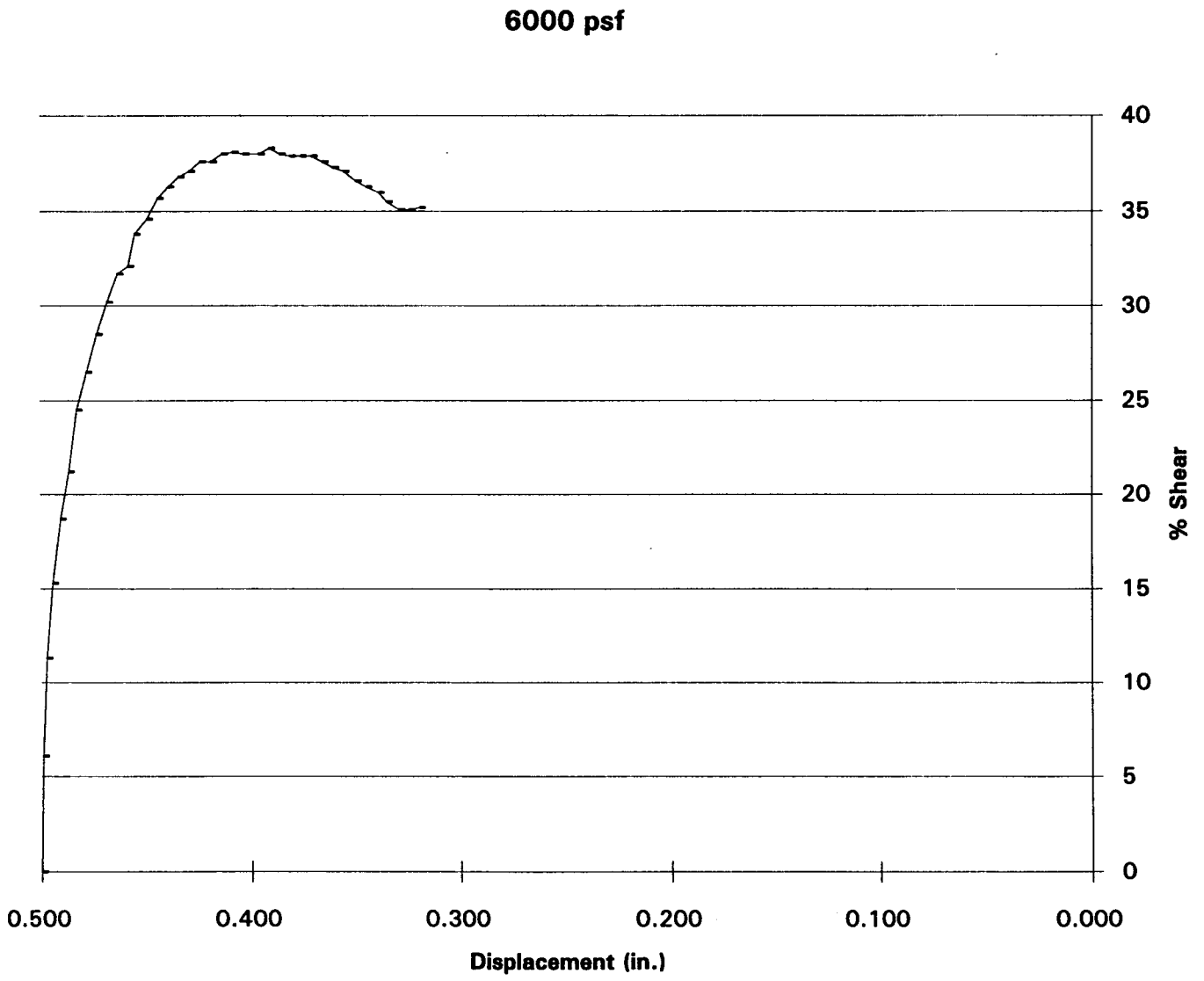
Phi Angle: **deg.**
Cohesion: **psf**

TEST CONDITION: Undisturbed, tested at saturation

John A. Sayers & Associates
GEOTECHNICAL CONSULTANTS

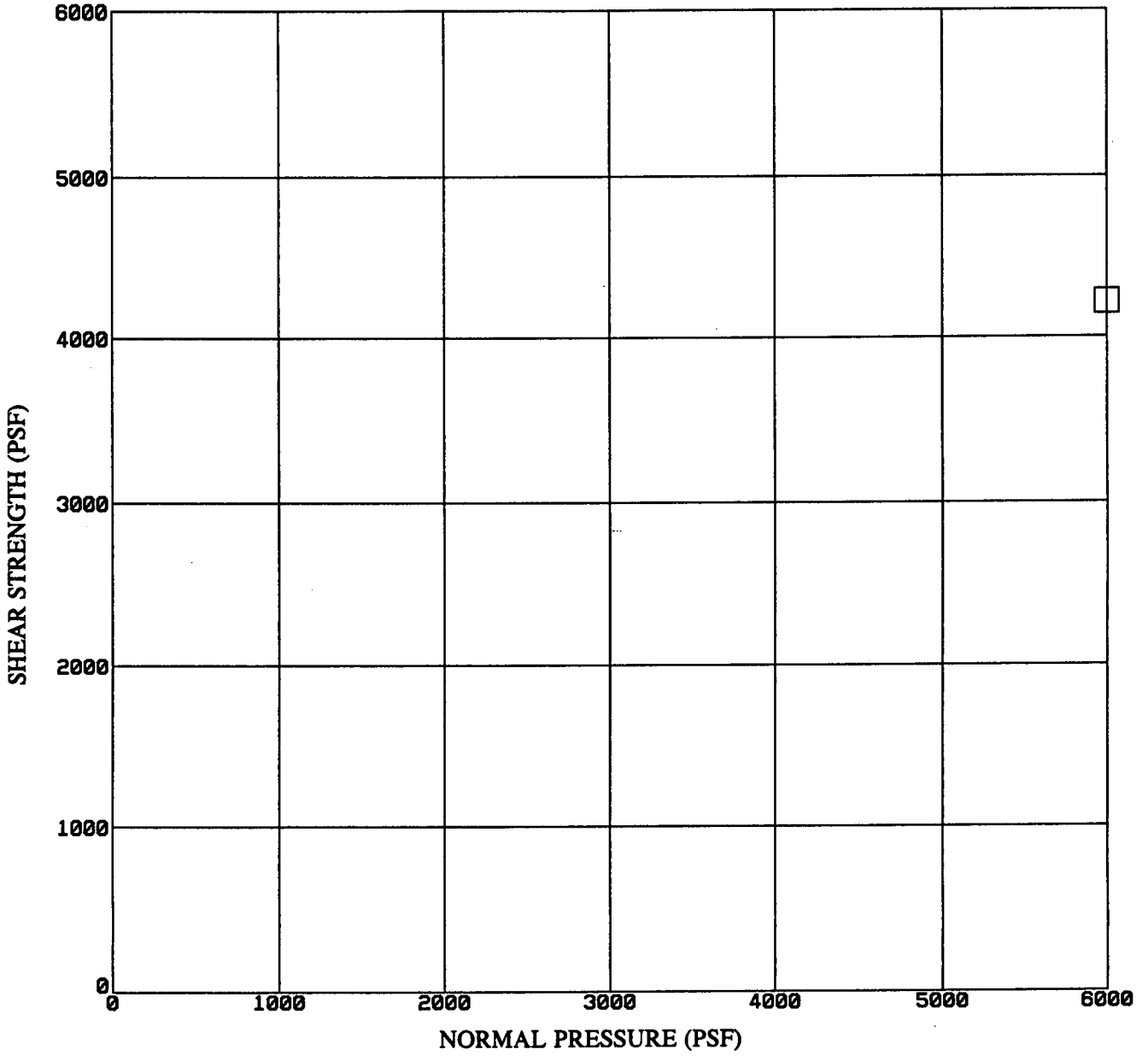
Plate

DIRECT SHEAR TEST PLOT OF B1A @ 40 FT. (SATURATED)



SHEAR TEST DIAGRAM

Client: Rancho Las Flores Limited Partnership	W.O. 215-1
Project: Highway 138	Location: San Bernardino County



Initial Sample

Dry Density = 129.0 pcf
Moist. Cont. = 8.5%

Sample: **B-2A @ 15.0 ft.**

Phi Angle: **deg.**
Cohesion: **psf**

TEST CONDITION: Undisturbed, tested at saturation

John A. Sayers & Associates GEOTECHNICAL CONSULTANTS	Plate

APPENDIX D

SHEAR TEST DATA REMOLDED SAMPLES SHEARED AT OPTIMUM

SHEAR TEST SUMMARY

Remolded to 90% Relative Compaction
at Optimum Moisture
Tested at Optimum Moisture

Sample Location	Shear Strength (psf)	Normal Pressure (psf) 2000	4000	6000
B-1A @ 10'		3000	4392	5520
B-1A @ 20'		3060	3528	5352
Median (Average) (1)		3030	3960	5436

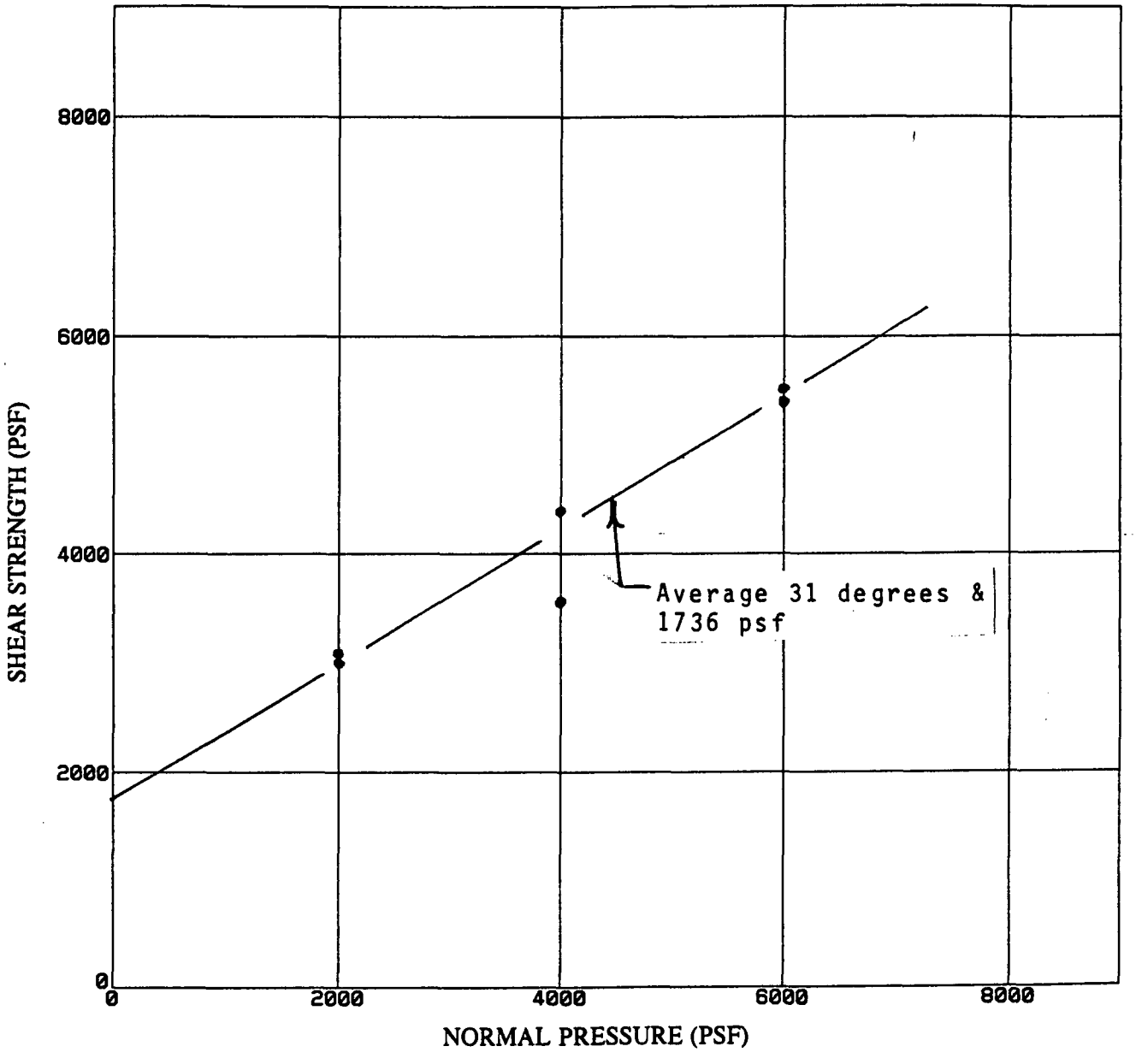
Median (Average) (2) = 31 degrees and 1736 psf

(1) Calculated as a mathematical average
(2) Calculated by computer

NOTE: These values were not used in calculations
See remolded and saturated Shear Test Data (Appendix E)

SHEAR TEST DIAGRAM

Client: Rancho Las Flores Limited Partnership	W.O. 215-1
Project: Highway 138	Location: San Bernardino County

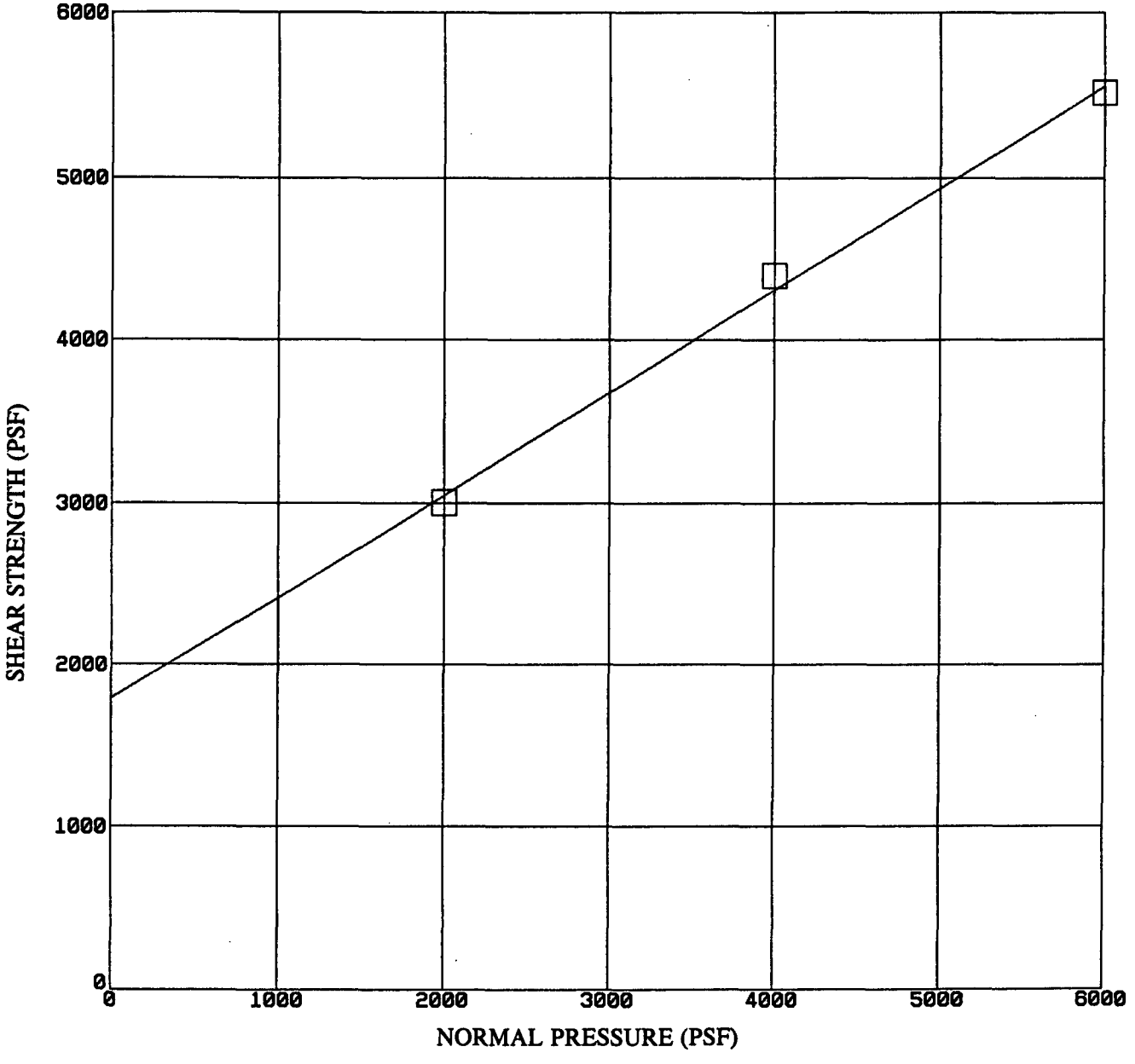


Shear Test Summary
Remolded to 90% R.C.
at Optimum Moisture
Sheared at Optimum

NOTE: These values were not used in calculations
See Remolded and saturated Shear Test Data
(Appendix E)

SHEAR TEST DIAGRAM

Client: **Rancho Las Flores Limited Partnership**	W.O. **215-1**
Project: **Highway 138**	Location: **San Bernardino County**



Initial Sample

Dry Density = 115.7 pcf
Moist. Cont. = 9.5%

Sample: **B-1A @ 10.0 ft.**

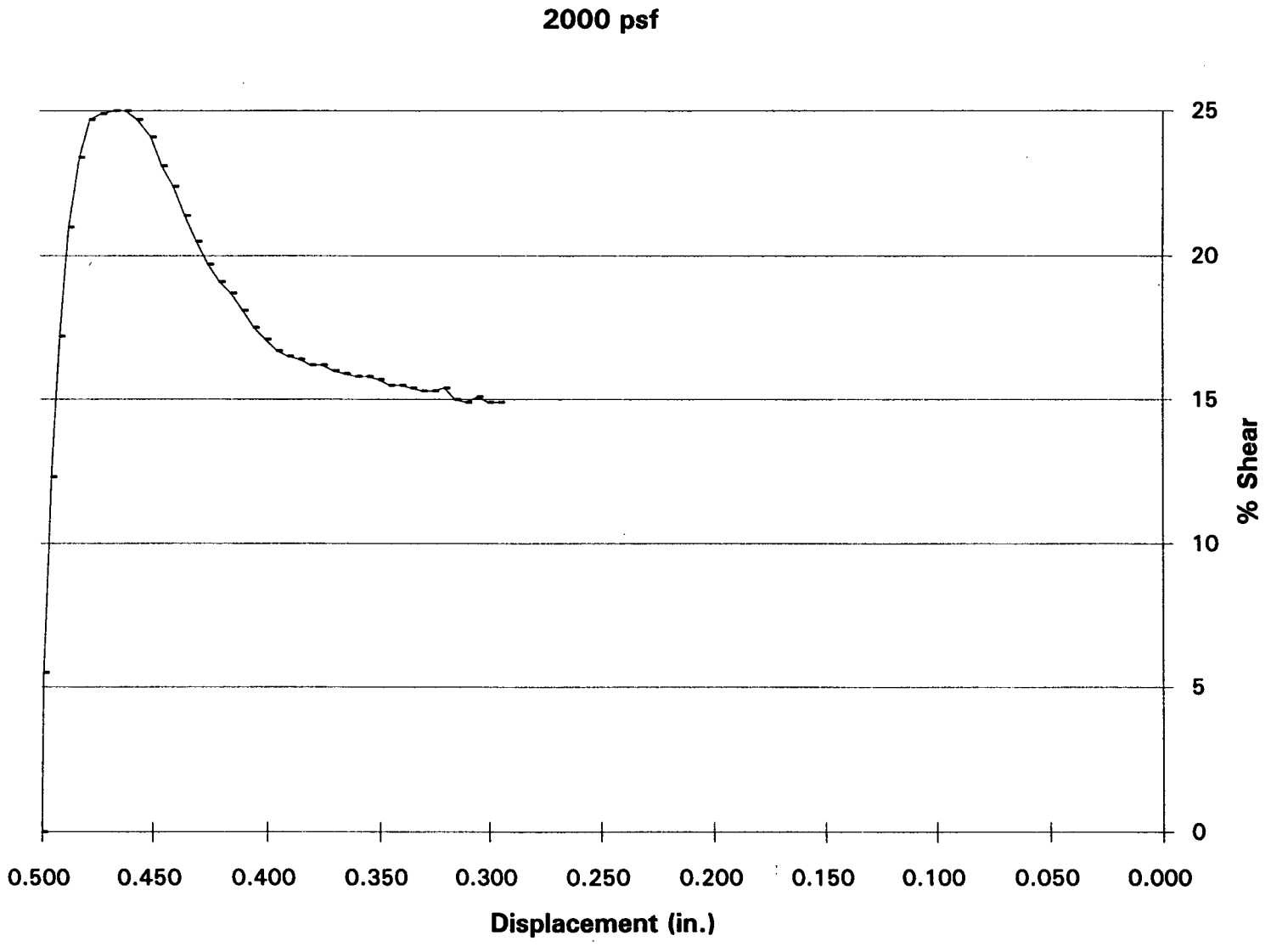
Phi Angle: **32 deg.**
Cohesion: **1720 psf**

TEST CONDITION: Remolded to 90% R.C. @ optimum moisture and tested at optimum

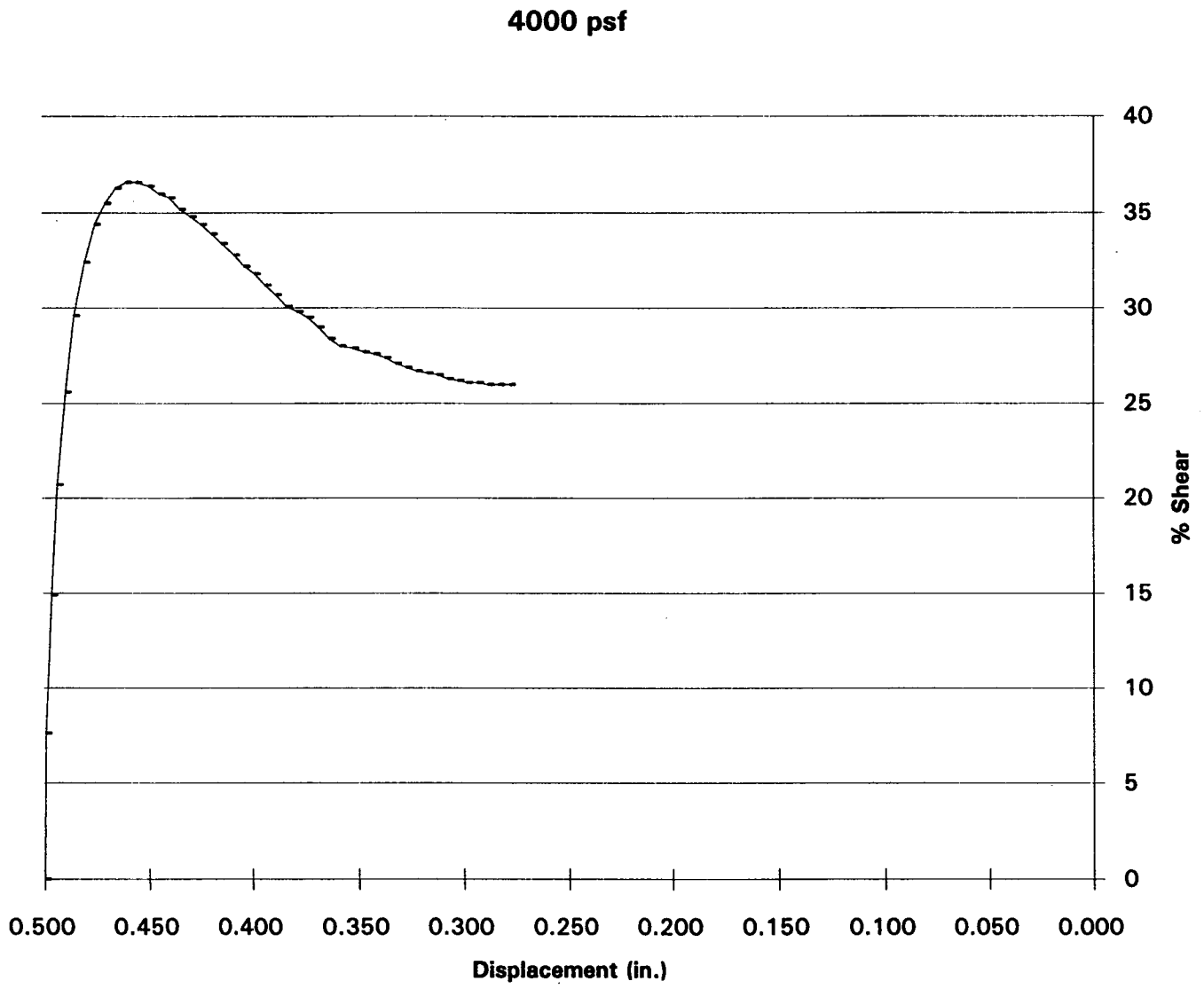
John A. Sayers & Associates
GEOTECHNICAL CONSULTANTS

Plate

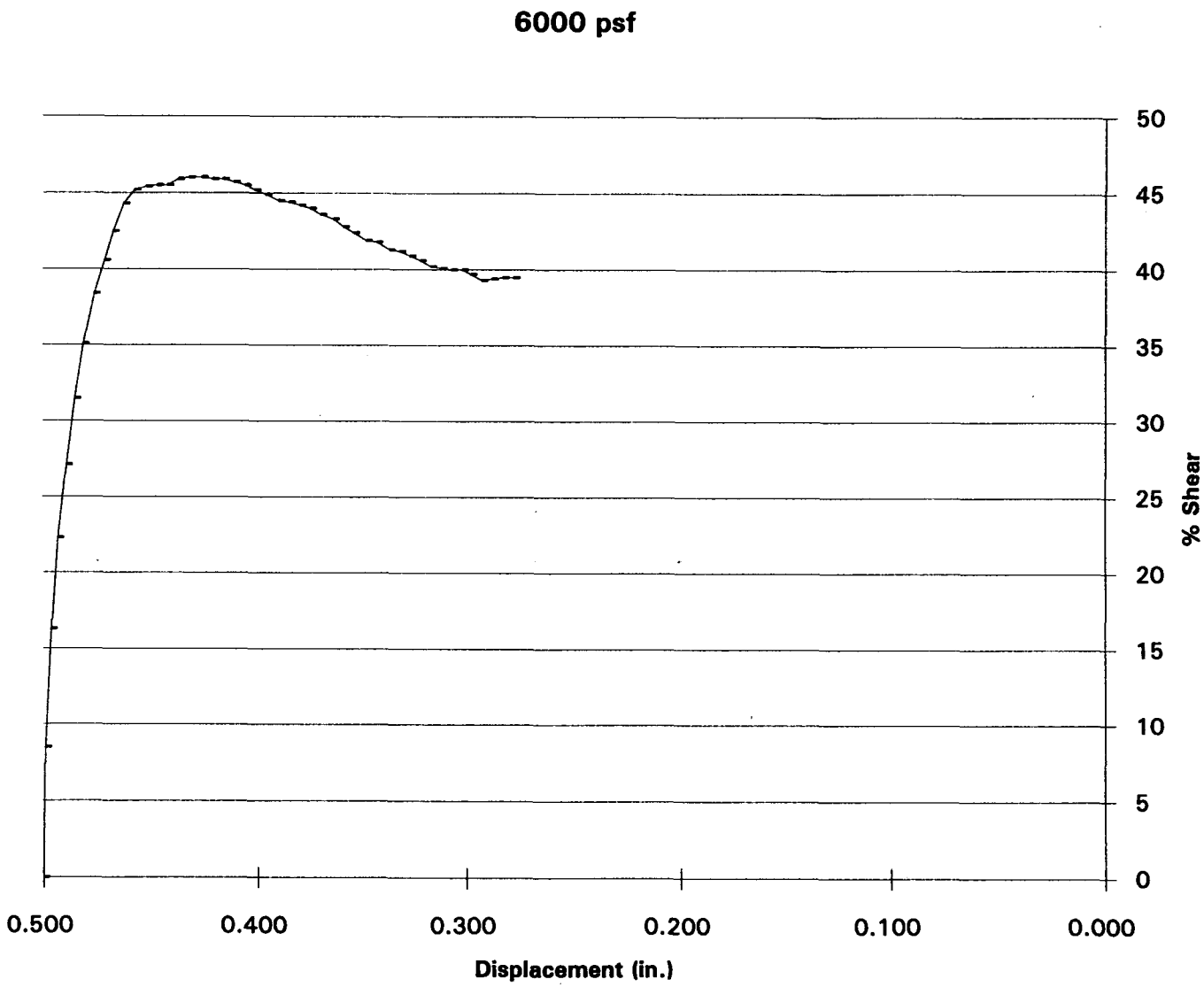
DIRECT SHEAR TEST PLOT OF B1A @ 10 FT. (REMOLDED)



DIRECT SHEAR TEST PLOT OF B1A @ 20 FT. (REMOLDED)

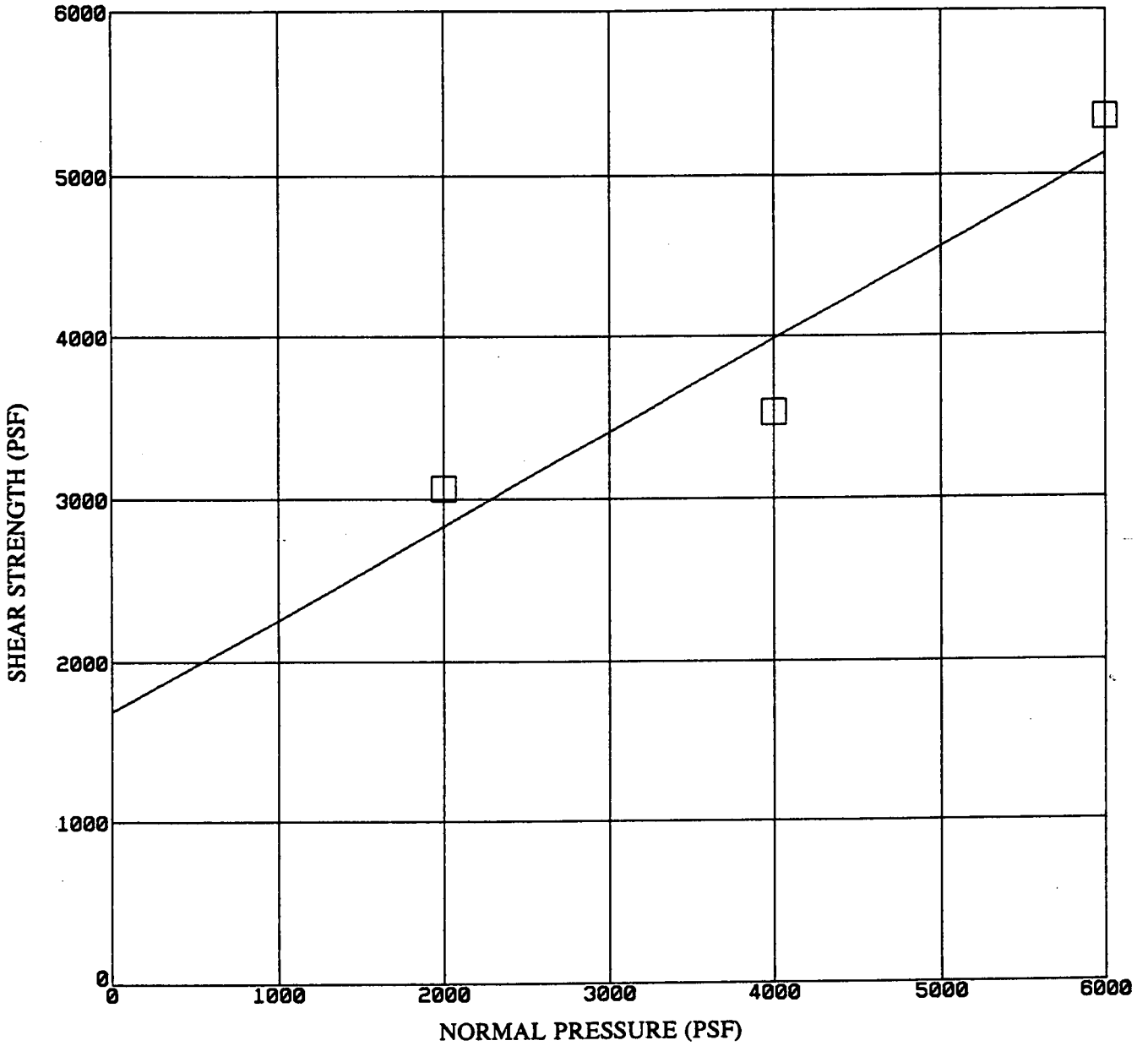


DIRECT SHEAR TEST PLOT OF B1A @ 10 FT. (REMOLDED)



SHEAR TEST DIAGRAM

Client: **Rancho Las Flores Limited Partnership**	W.O. **215-1**
Project: **Highway 138**	Location: **San Bernardino County**



Initial Sample

Dry Density = 112.1 pcf
Moist. Cont. = 11.0%

Sample: **B-1A @ 20.0 ft.**

Phi Angle: **30 deg.**

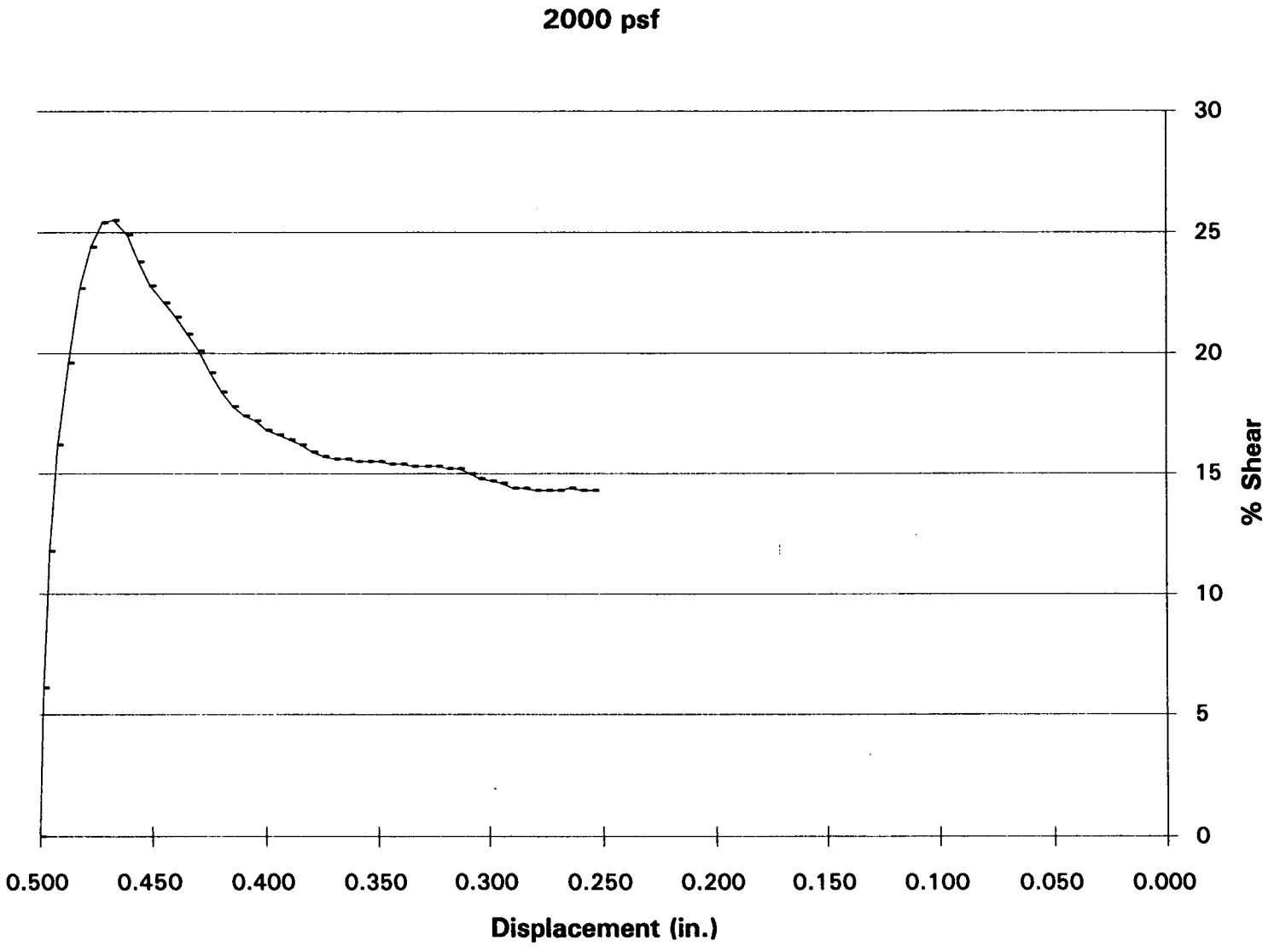
Cohesion: **1600 psf**

TEST CONDITION: Remolded to 90% R.C. @ optimum moisture and tested at optimum

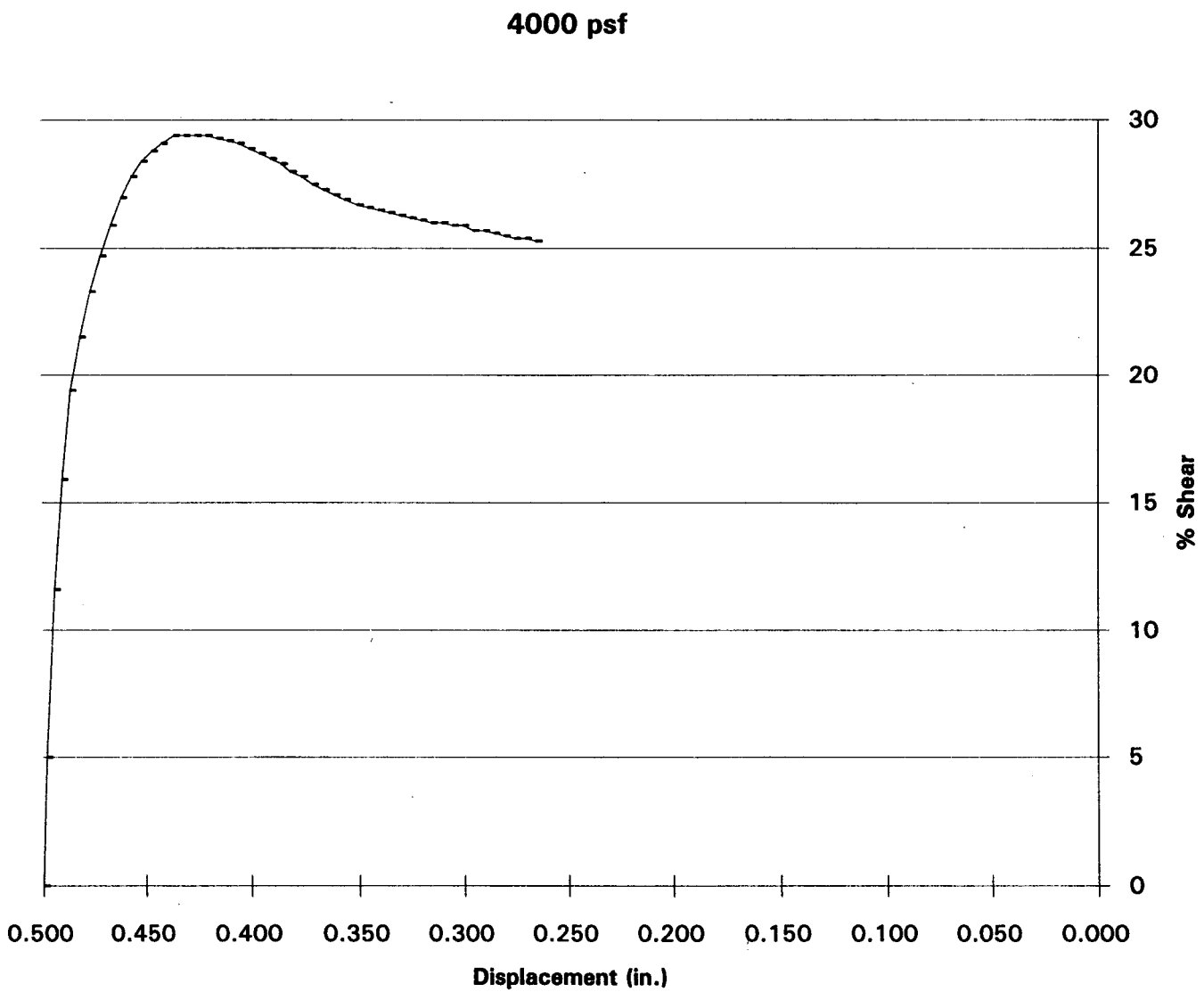
John A. Sayers & Associates
GEOTECHNICAL CONSULTANTS

Plate

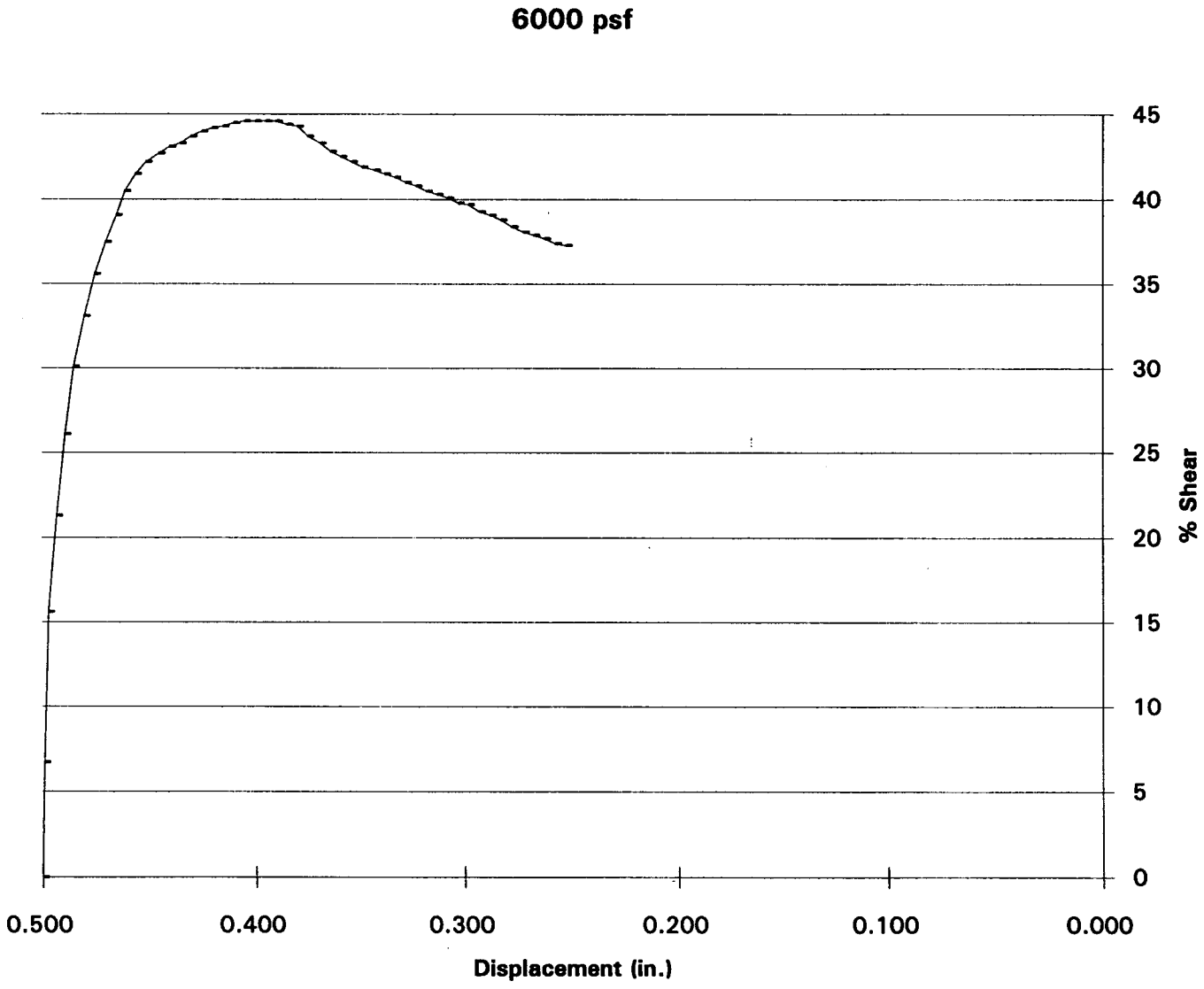
DIRECT SHEAR TEST PLOT OF B1A @ 20 FT. (REMOLDED)



DIRECT SHEAR TEST PLOT OF B1A @ 20 FT. (REMOLDED)



DIRECT SHEAR TEST PLOT OF B1A @ 20 FT. (REMOLDED)



APPENDIX E

SHEAR TEST DATA REMOLDED SAMPLES AT OPTIMUM AND SHEARED AFTER SATURATION

SHEAR TEST SUMMARY

Remolded to 90% Relative Compaction
at Optimum Moisture
Test after Saturation

Sample Location	Shear Strength (psf)	Normal Pressure (psf) 2000	4000	6000
B-1A @ 10'		2004 (Low)	2676 (Low)	3828 (Low)
B-1A @ 20'		2232	3756	4668
Average		2118	3216	4248

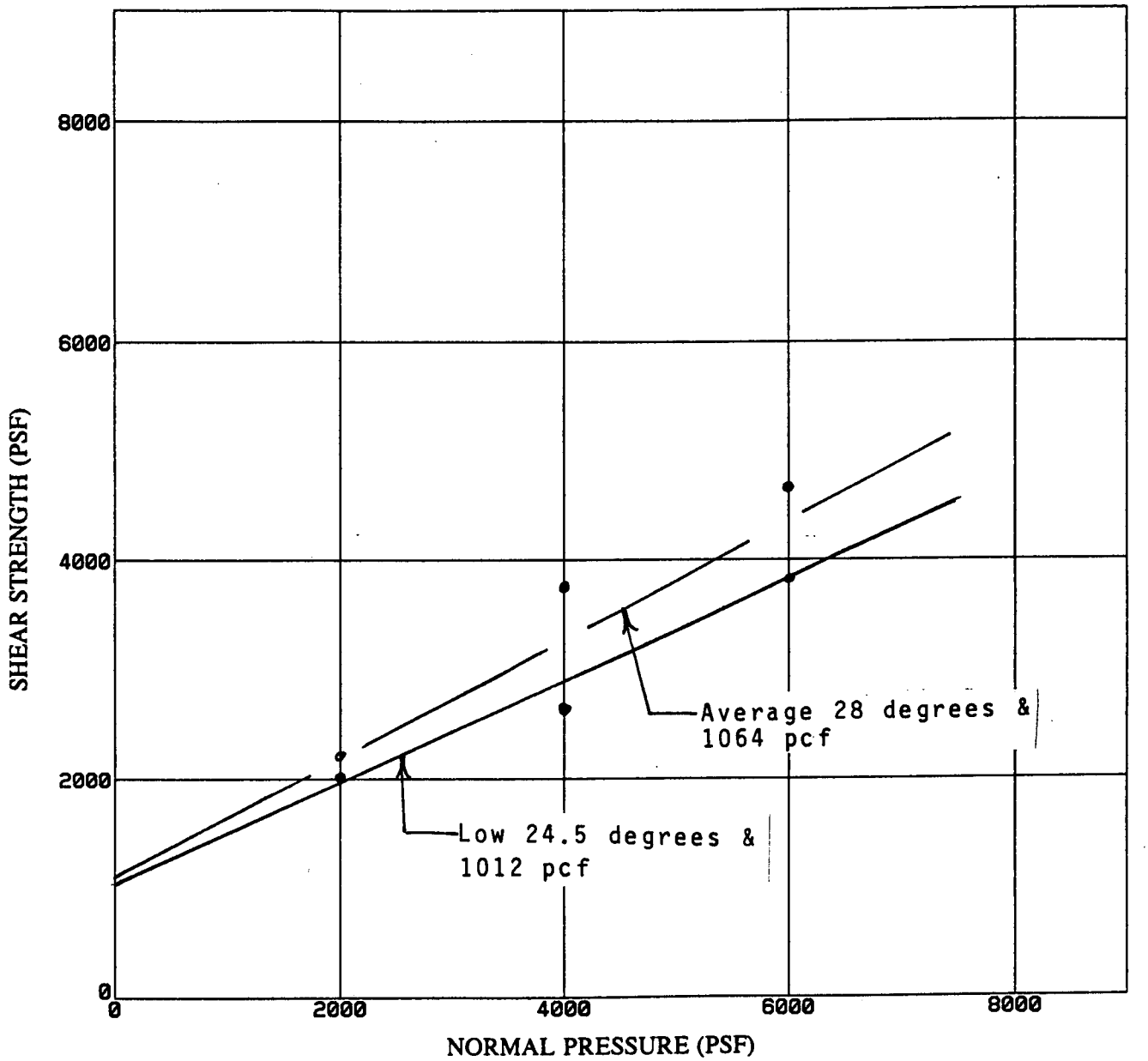
Low (1) 24.5 degrees and 1012 psf
Average (1) (2) 28 degrees and 1064 psf

(1) Calculated by computer
(2) Mathematical average

NOTE: Calculations used low value with half the cohesion.

SHEAR TEST DIAGRAM

Client: Rancho Las Flores Limited Partnership	W.O. 215-1
Project: Highway 138	Location: San Bernardino County



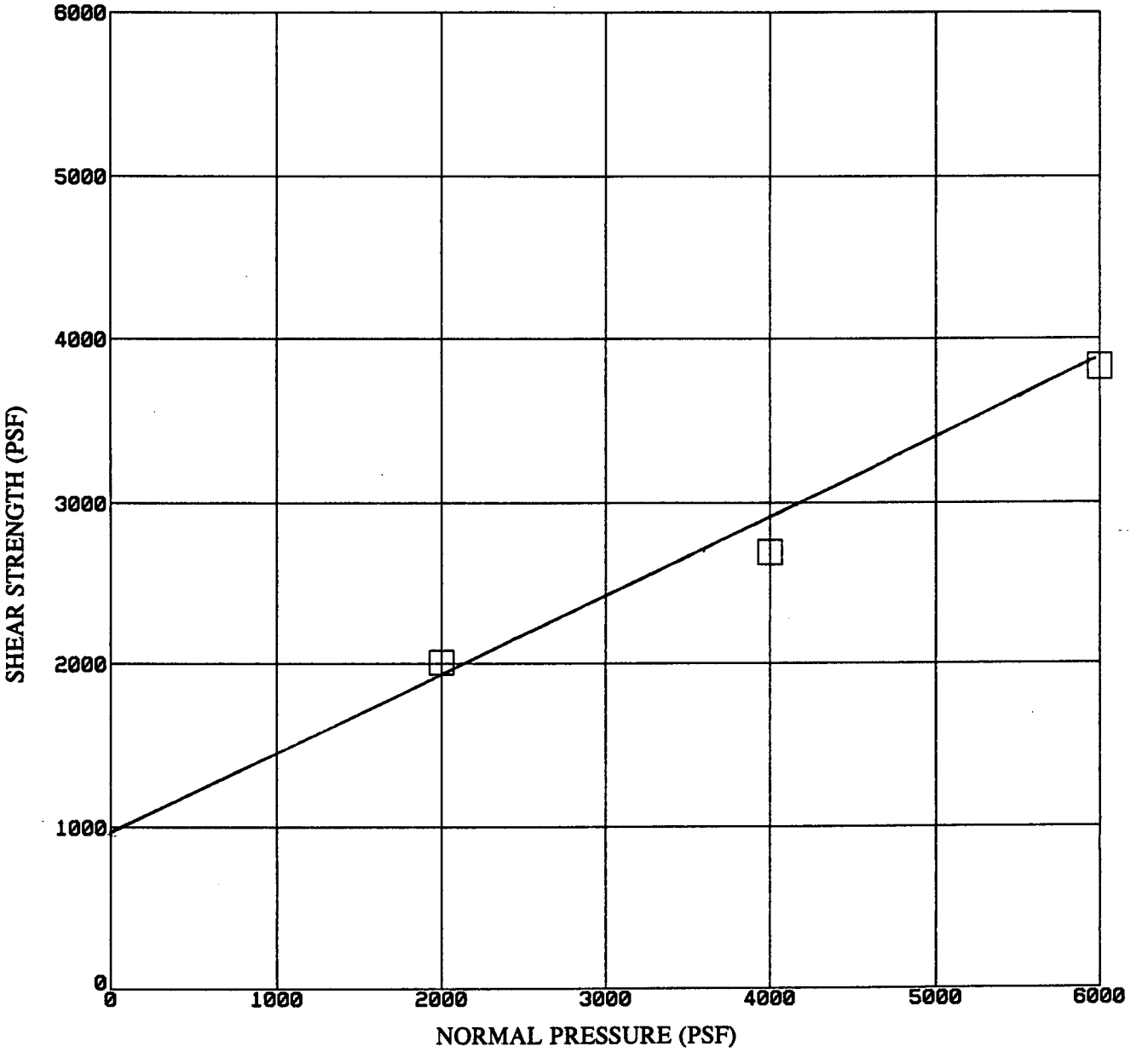
Shear Test Summary
Remolded to 90% R.C.
at Optimum Moisture
Sheared After Saturation

NOTE: Low value used in calculations with half cohesion

John A. Sayers & Associates GEOTECHNICAL CONSULTANTS	Plate

SHEAR TEST DIAGRAM

Client: **Rancho Las Flores Limited Partnership**	W.O. **215-1**
Project: **Highway 138**	Location: **San Bernardino County**



Initial Sample

Dry Density = 126.1 pcf
Moist. Cont.= 8.1%

Sample: **B-1A @ 10.0 ft.**

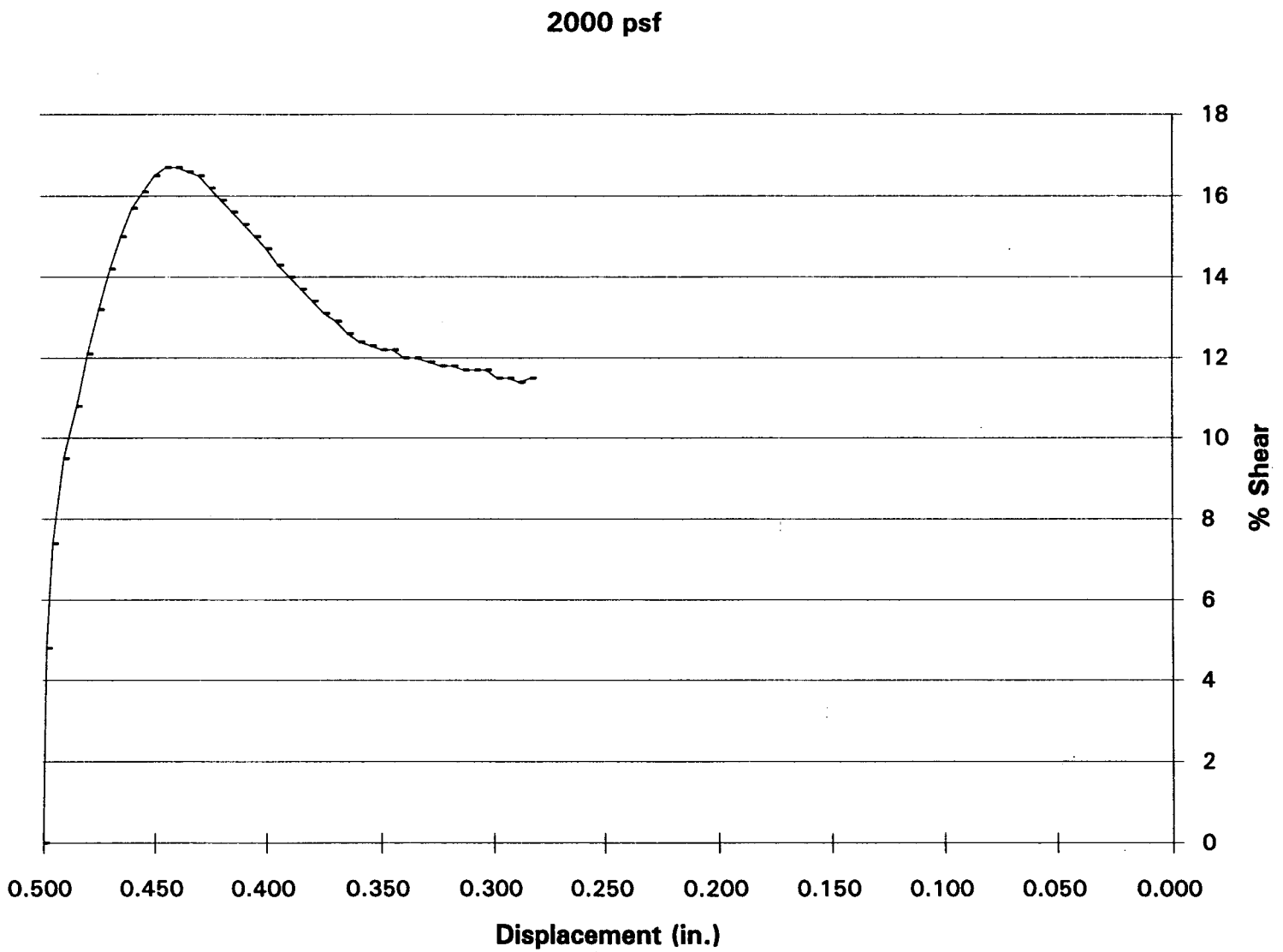
Phi Angle: **25 deg.**
Cohesion: **980 psf**

TEST CONDITION: Remolded to 90% R.C. @ optimum, saturated

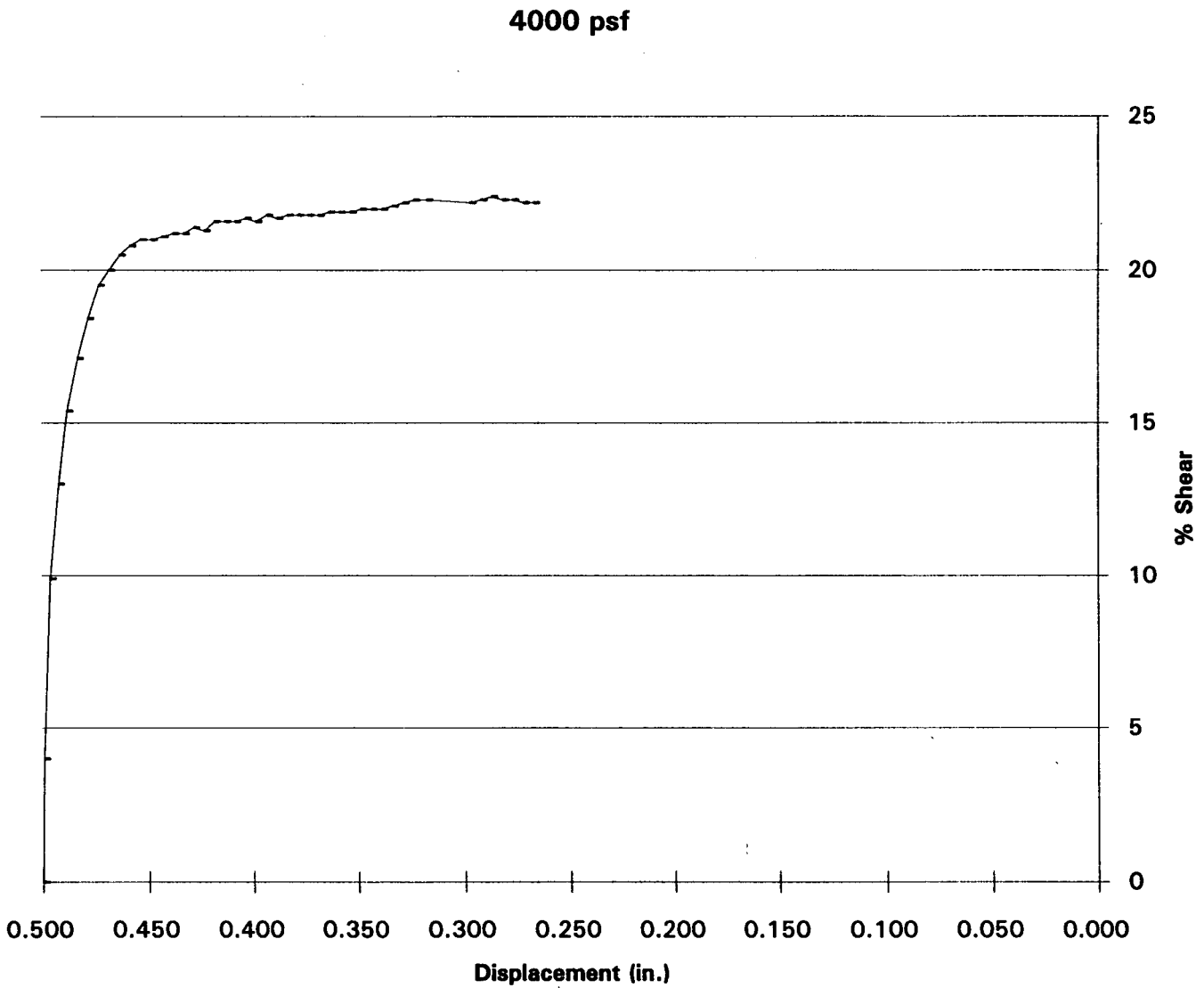
John A. Sayers & Associates
GEOTECHNICAL CONSULTANTS

Plate

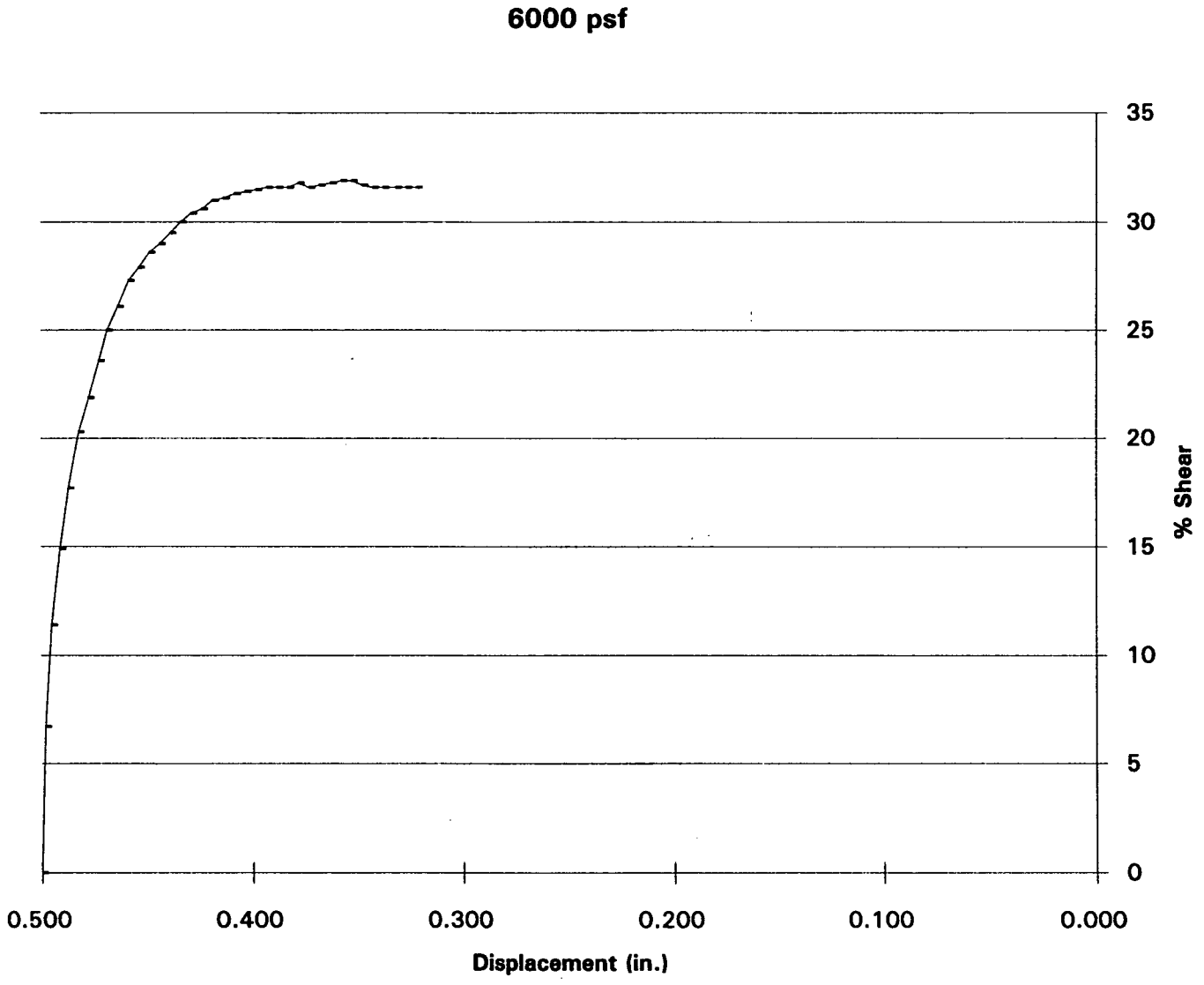
DIRECT SHEAR TEST PLOT OF B1A @ 10 FT. (REMOLDED AND SATURATED)



DIRECT SHEAR TEST PLOT OF B1A @ 10 FT. (REMOLDED AND SATURATED)

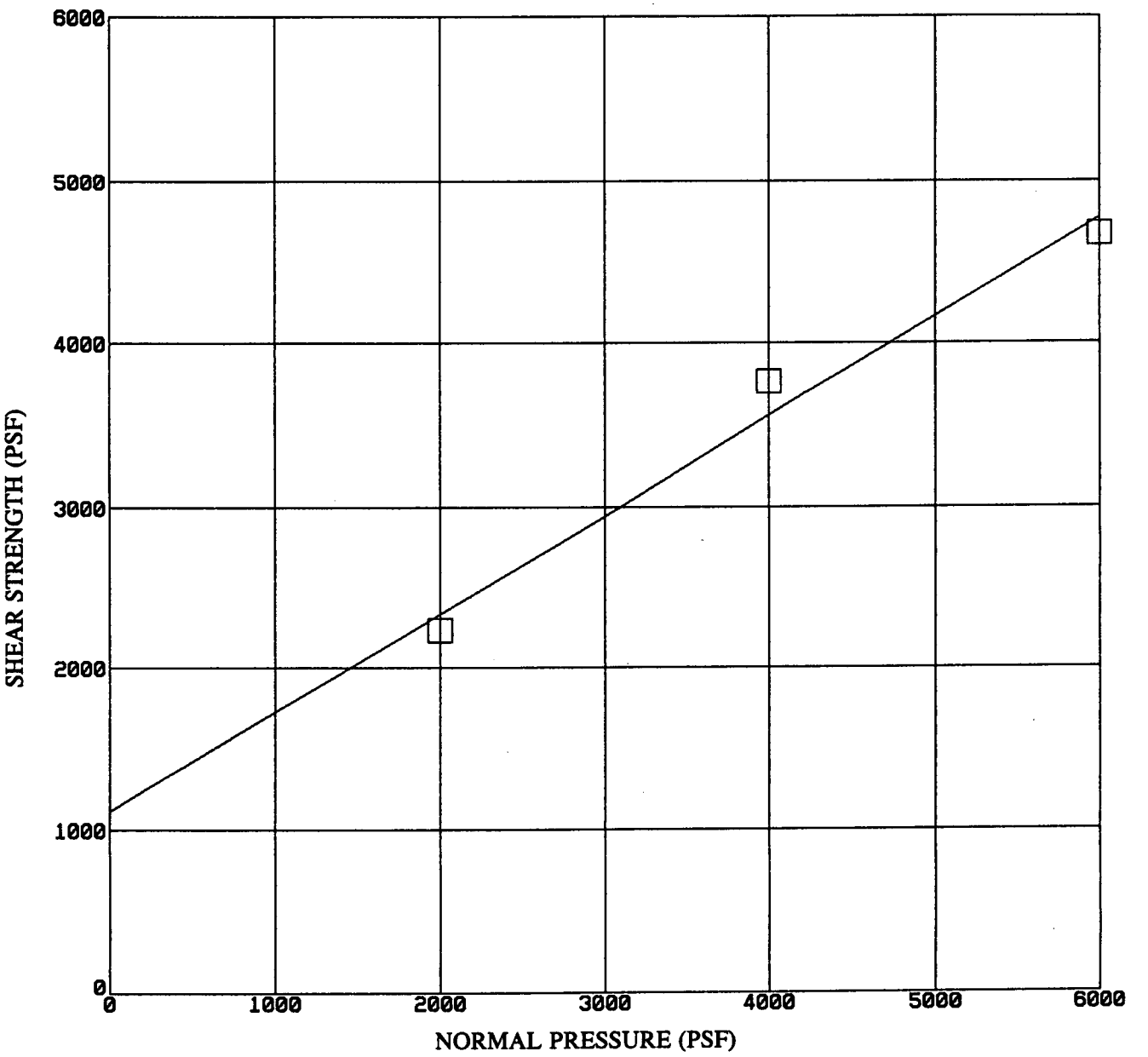


DIRECT SHEAR TEST PLOT OF B1A @ 10 FT. (REMOLDED AND SATURATED)



SHEAR TEST DIAGRAM

Client: **Rancho Las Flores Limited Partnership**	W.O. **215-1**
Project: **Highway 138**	Location: **San Bernardino County**



Initial Sample

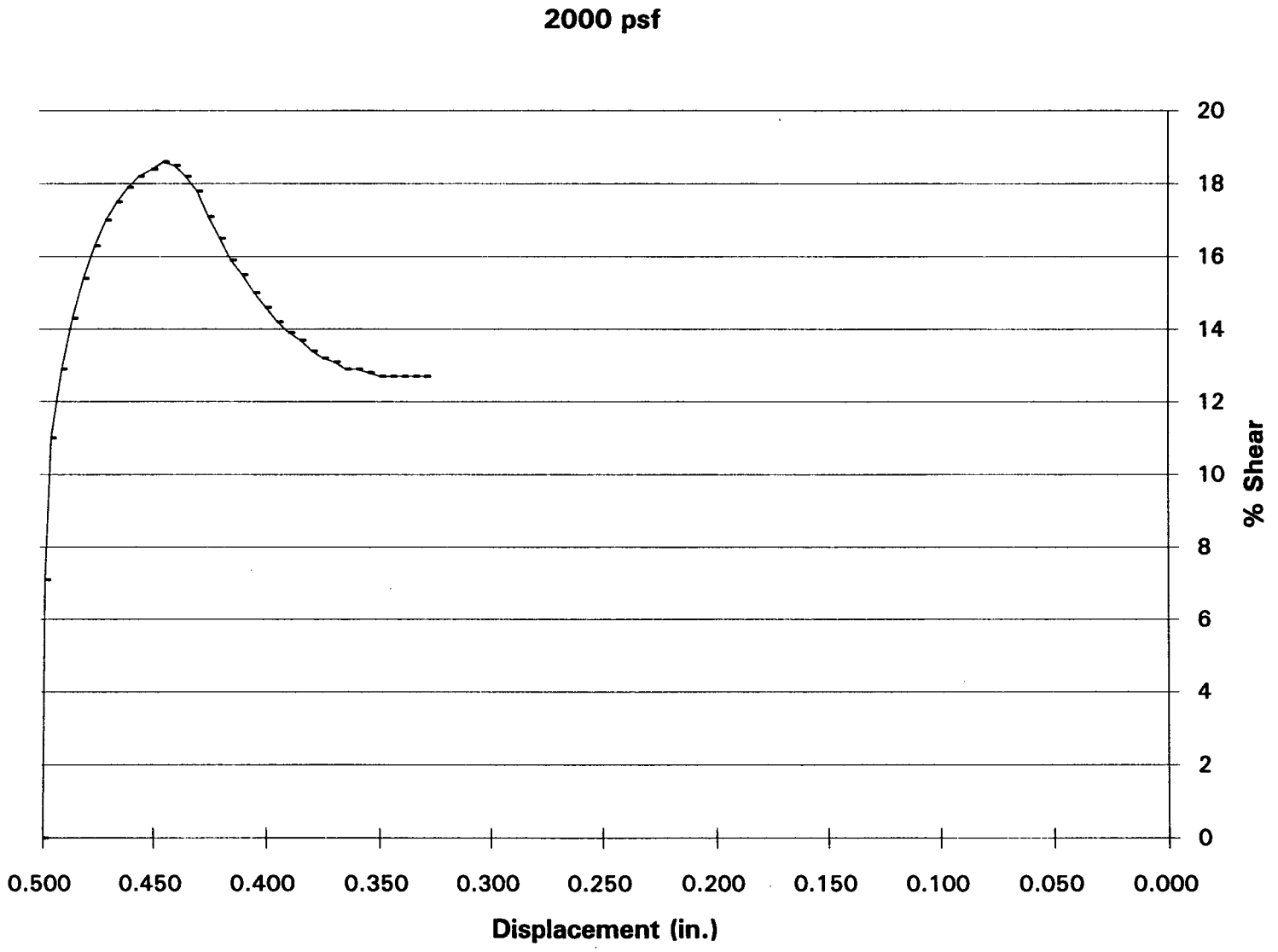
Dry Density = 127.4 pcf
Moist.Cont. = 10.4%

Sample: **B-1A @ 20.0 ft.**

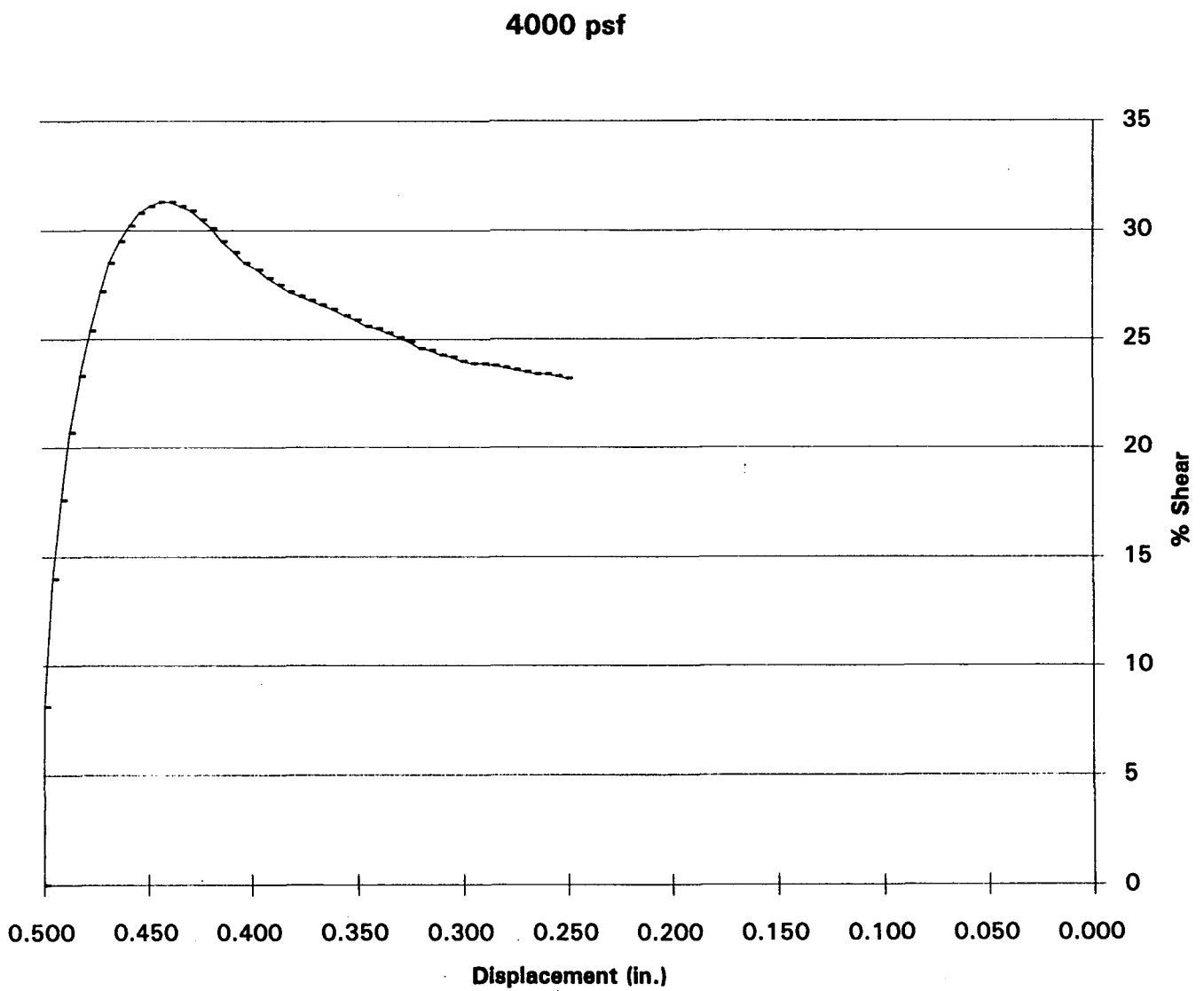
Phi Angle: **30 deg.**
Cohesion: **1220 psf**

TEST CONDITION: Remolded to 90% R.C. @ optimum, saturated

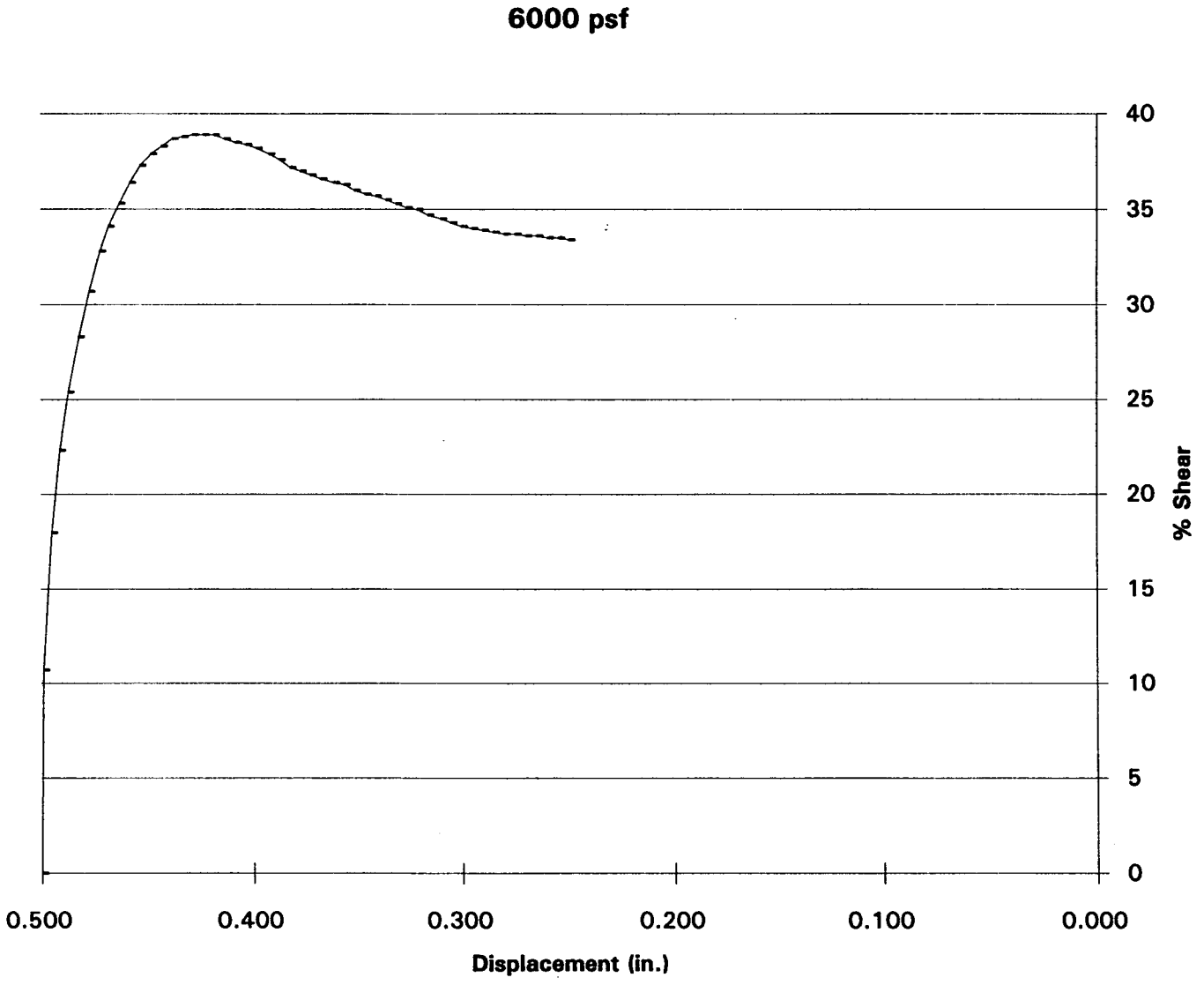
DIRECT SHEAR TEST PLOT OF B1A @ 20 FT. (REMOLDED AND SATURATED)



DIRECT SHEAR TEST PLOT OF B1A @ 20 FT. (REMOLDED AND SATURATED)



DIRECT SHEAR TEST PLOT OF B1A @ 20 FT. (REMOLDED AND SATURATED)



APPENDIX F

BORING LOGS
B-1 THROUGH B-16
AND B-1A AND B-2A
SOIL SURVEY SHEETS
1 THROUGH 9

JOHN A. SAYERS & ASSOCIATES

Legend for Test Pit Logs

Sample Type:

CORE SAMPLE

BULK SAMPLE

CORE AND BULK SAMPLES

Density terminology used for soil descriptions (e.g., medium dense, dense) reported in test pit logs has been based on observation and engineering experience.

UNIFIED SOIL CLASSIFICATION SYSTEM

Major Divisions			Symbols	Typical Names
COARSE GRAINED SOILS (More than 50% of material is LARGER than 200 sieve size.)	GRAVELS (More than 50% of coarse fraction is LARGER than No.4 sieve.)	CLEAN GRAVELS (Little or no fines.)	GW	Well-graded gravels, gravel-sand mixtures, little or no fines.
			GP	Poorly graded gravels or gravel-sand mixtures, little or no fines.
		GRAVELS W/FINES (Appreciable amount of fines.)	GM	Silty gravels, gravel-sand-silt mixtures.
			GC	Clayey gravels, gravel-sand-clay mixtures.
	SANDS (More than 50% of the coarse fraction SMALLER than the No.4 seive.)	CLEAN SANDS	SW	Well-graded sands, gravelly sands, little or no fines.
			SP	Poorly graded sands or gravelly sands, little or no fines.
		SANDS W/FINES (Appreciable amount of fines.)	SM	Silty sands, sand-silt mixtures.
			SC	Clayey sands, sand-clay mixtures.
FINE GRAINED SOILS (More than 50% of material is SMALLER than 200 sieve size.)	SILTS AND CLAYS (Liquid limit less than 50.)		ML	Inorganic silts & fine sands, rock flour, silty or clayey fine sands or clayey silts with slight plasticity.
			CL	Inorganic clays of low to medium plasticity, gravelly clays, sandy clays, silty clays, lean clays.
			OL	Organic silts and organic silty clays.
	SILTS AND CLAYS (Liquid limit more 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.

BOUNDARY CLASSIFICATIONS: Soils possessing characteristics of two groups are designated by combinations of group symbols.

Log of Exploratory Boring B-1

Client: A.R.C. Las Flores Ltd.	Date: 12-11-89	W.O. 215-01
Project: Highway 138	Location: San Bernardino	
	Surface Elevation: ±	
	Total Depth: 15.5	

FIELD					LAB DATA						
Water	Core	Bulk	Blows*	Depth (ft)	Moisture Content (%)	Dry Density (Pcf)	Other	USCS	Graphics	Depth (ft)	DESCRIPTION AND REMARKS
								SM			**TOPSOIL:** Silty Sand, brown, loose to medium dense, dry, with roots and fine gravel.
								SM			**OLDER ALLUVIUM:** Silty Sand, reddish brown, medium dense, moist, fine to medium grained, with a few fine to coarse gravel, with occasional coarse grains.
	■		3	5	1.4	113.3				5	
		■	8 9				SE=46 <200 8%				
	■		8 9 17		12.7	114.6	<200 8%	SM SP			Silty Sand/Sand, reddish brown, medium dense, wet to saturated, with gravel. @8.4' - Groundwater observed, brown, medium dense.
▽											
	■		3 8 10	10	10.9	125.3		SM		10	**CROWDER FORMATION:** Silty Sand, reddish brown, very dense, moist to wet, with some clay and gravel.
	▨		50	15			<200 20%			15	

This log is a representation of conditions at the time & place of drilling. With the passage of time or at any other location there may be consequential changes in conditions.

Total Depth 15.5 feet
Groundwater at 8' 4"
Caving at 3 feet
Hole Backfilled

See Legend for Important Notes | *Driving Weight: CME-75, Automatic Hammer System

JOHN A. SAYERS & ASSOCIATES
GEOTECHNICAL CONSULTANTS

Boring B-1
Sheet 1 of 1

Log of Exploratory Boring B-2

Client: A.R.C. Las Flores Ltd.	Date: 12-11-89	W.O. 215-01
Project: Highway 138	Location: San Bernardino	
	Surface Elevation: ±	
	Total Depth: 15.5	

FIELD					LAB DATA						
Water	Core	Bulk	Blows*	Depth (ft)	Moisture Content (%)	Dry Density (Pcf)	Other	USCS	Graphics	Depth (ft)	DESCRIPTION AND REMARKS
								SM			TOPSOIL: Silty Sand, brown, loose, dry.
								SM			OLDER ALLUVIUM: Silty Sand, brown, medium dense, dry to damp, with trace of fine gravel.
	■		8 10 23		4.8	106.9	SE=37 <200 25%				
		■									
	■		5 10 11	5	1.3	121.6		SM SP		5	Silty Sand/Sand, light to medium brown, medium dense to dense, damp, with gravel.
	■		5 7 14		1.7	107.5	<200 5%				
	▨		28 50	10						10	
▽											@14' - Groundwater encountered.
	▨		7 28 29	15				SM SP		15	CROWDER FORMATION: Silty Sand with Some Clay, reddish brown, very dense, moist to wet.

Total Depth 15.5 feet
Groundwater at 14 feet
Caving at 6 feet
Hole Backfilled

This log is a representation of conditions at the time & place of drilling. With the passage of time or at any other location there may be consequential changes in conditions.

See Legend for Important Notes | *Driving Weight: CME-75 Automatic Hammer System

JOHN A. SAYERS & ASSOCIATES
GEOTECHNICAL CONSULTANTS

Boring B-2
Sheet 1 of 1

Log of Exploratory Boring B-3

Client: A.R.C. Las Flores Ltd.	Date: 12-11-89	W.O. 215-01
Project: Highway 138	Location: San Bernardino	
	Surface Elevation: ±	
	Total Depth: 16.5	

FIELD					LAB DATA						
Water	Core	Bulk	Blows*	Depth (ft)	Moisture Content (%)	Dry Density (Pcf)	Other	USCS	Graphics	Depth (ft)	DESCRIPTION AND REMARKS
								GW			**ALLUVIUM:** Sandy Gravel, light to medium brown, dry to damp, loose to dense, cobbles and boulders at surface.
			20 22 23								
			10 22 23	5	3.2	106.2		SM		5	**OLDER ALLUVIUM:** Silty Sand, reddish to light brown and gray, medium dense to dense, dry to slightly damp.
			5 10 15								
			44 40 43	10	1.4	106.2				10	
			11 20 25	15						15	

This log is a representation of conditions at the time & place of drilling. With the passage of time or at any other location there may be consequential changes in conditions.

Total Depth 16.5 feet
No Groundwater
No Caving
Hole Backfilled

See Legend for Important Notes | *Driving Weight:

JOHN A. SAYERS & ASSOCIATES
GEOTECHNICAL CONSULTANTS

Boring B-3
Sheet 1 of 1

Log of Exploratory Boring B-4

Client: A.R.C. Las Flores Ltd.	Date: 12-11-89	W.O. 215-01
Project: Highway 138	Location: San Bernardino	
	Surface Elevation: ±	
	Total Depth: 25.5	

FIELD					LAB DATA						
Water	Core	Bulk	Blows*	Depth (ft)	Moisture Content (%)	Dry Density (Pcf)	Other	USCS	Graphics	Depth (ft)	DESCRIPTION AND REMARKS
								SM			**ROADFILL:** Silty Sand with Gravel, brown, medium dense to dense, dry to damp, minor roots, gravel and cobbles encountered at 3.5 feet.
	■		7 17 44		2.6	126.9					
				5						5	
	■		10 11 11		2.0						
	▩		5 8 18					SP SM			**WASH DEPOSITS:** Sand/Silty Sand, light brown, dense, dry to damp, some gravel.
	■		22 29 40	10	1.0	114.9				10	
								SM			**OLDER ALLUVIUM:** Silty Sand with Gravel, light brown, very dense, dry to damp.
	▩		15 27 28	15						15	
	■		21 33 36	20	3.0	121.8				20	
	■		50	25	2.8					25	

This log is a representation of conditions at the time & place of drilling. With the passage of time or at any other location there may be consequential changes in conditions.

Total Depth 25.5 feet
Caving at 8 to 11 feet
No Groundwater
Hole Backfilled

See Legend for Important Notes | *Driving Weight:

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GEOTECHNICAL CONSULTANTS

Boring B-4
Sheet 1 of 1

Log of Exploratory Boring B-5

Client: A.R.C. Las Flores Ltd.	Date: 12-11-89	W.O. 215-01
Project: Highway 138	Location: San Bernardino	
	Surface Elevation: ±	
	Total Depth: 16.5	

FIELD					LAB DATA						
Water	Core	Bulk	Blows*	Depth (ft)	Moisture Content (%)	Dry Density (Pcf)	Other	USCS	Graphics	Depth (ft)	DESCRIPTION AND REMARKS
								SM SP			TOPSOIL: 2" to 3" thick OLDER ALLUVIUM: Silty Sand/Sand, brown, loose to medium dense, dry.
	■		7 8 8		0.8	104.3					
				5						5	
	■		5 7 8		2.0	107.8					@5' - dry to damp, dense.
	■		10 19 19		2.9	110.6		SM			Silty Sand, brown, medium dense, damp, with gravel, dry to damp.
				10						10	
	■		9 16 18		2.9	113.7					
				15						15	
	▨		7 11 12								

Total Depth 16.5 feet
No Groundwater
No Caving
Hole Backfilled

See Legend for Important Notes | *Driving Weight:

JOHN A. SAYERS & ASSOCIATES
GEOTECHNICAL CONSULTANTS

Boring B-5
Sheet 1 of 1

Log of Exploratory Boring B-6

Client: A.R.C. Las Flores Ltd.	Date: 12-11-89	W.O. 215-01
Project: Highway 138	Location: San Bernardino	
	Surface Elevation: ±	
	Total Depth: 15.0	

FIELD					LAB DATA						
Water	Core	Bulk	Blows*	Depth (ft)	Moisture Content (%)	Dry Density (Pcf)	Other	USCS	Graphics	Depth (ft)	DESCRIPTION AND REMARKS
								SM			TOPSOIL: Silty Sand, medium brown, loose, dry.
								SP SM			OLDER ALLUVIUM: Sand/Silty Sand, brown, medium dense, damp, with gravel.
	■		10 12 14		1.3	106.1					
	■		10 18 25	5	1.3	117.9				5	@5' - dense
	■		17 19 19		1.1	115.1					
	▩		14 22 24	10				SM		10	Silty Sand, brown, dense, damp to moist, with gravel.
	▩		10 18 26	15						15	Total Depth 15 feet No Groundwater Hole Backfilled

This log is a representation of conditions at the time & place of drilling. With the passage of time or at any other location there may be consequential changes in conditions.

See Legend for Important Notes | *Driving Weight:

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Boring B-6
Sheet 1 of 1

Log of Exploratory Boring B-7

Client: A.R.C. Las Flores Ltd.	Date: 12-11-89	W.O. 215-01
Project: Highway 138	Location: San Bernardino	
	Surface Elevation: ±	
	Total Depth: 15.0	

FIELD					LAB DATA						DESCRIPTION AND REMARKS
Water	Core	Bulk	Blows*	Depth (ft)	Moisture Content (%)	Dry Density (Pcf)	Other	USCS	Graphics	Depth (ft)	This log is a representation of conditions at the time & place of drilling. With the passage of time or at any other location there may be consequential changes in conditions.
								SM			WASH DEPOSIT: Silty Sand with Gravel, brown, loose, dry.
	■		64 87		5.0	129.2		SC			CROWDER FORMATION (WEATHERED): Clayey Sand, brown, damp, dense.
	■		15 39 67	5	11.3	123.5				5	@5' - less weathered, although still clayey.
	■		41 50/3"	10	8.0	124.3		SC		10	Silty Sand with Clay, some gravel, brown, very dense, moist.
	▨		18 33 51	15	13.4					15	Total Depth 15 feet No Groundwater No Caving Hole Backfilled

See Legend for Important Notes | *Driving Weight:

JOHN A. SAYERS & ASSOCIATES
GEOTECHNICAL CONSULTANTS

Boring B-7
Sheet 1 of 1

Log of Exploratory Boring B-8

Client: A.R.C. Las Flores Ltd.	Date:	W.O. 215-01
Project: Highway 138	Location: San Bernardino	
	Surface Elevation: ±	
	Total Depth: 1.0	

FIELD					LAB DATA						
Water	Core	Bulk	Blows*	Depth (ft)	Moisture Content (%)	Dry Density (Pcf)	Other	USCS	Graphics	Depth (ft)	DESCRIPTION AND REMARKS
											This log is a representation of conditions at the time & place of drilling. With the passage of time or at any other location there may be consequential changes in conditions.
											This boring was deleted due to lack of access.

See Legend for Important Notes | *Driving Weight:

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GEOTECHNICAL CONSULTANTS

Boring B-8
Sheet 1 of 1

Log of Exploratory Boring B-9

Client: A.R.C. Las Flores Ltd.	Date: 12-11-89	W.O. 215-01
Project: Highway 138	Location: San Bernardino	
	Surface Elevation: ±	
	Total Depth: 21.0	

FIELD					LAB DATA						DESCRIPTION AND REMARKS
Water	Core	Bulk	Blows*	Depth (ft)	Moisture Content (%)	Dry Density (Pcf)	Other	USCS	Graphics	Depth (ft)	This log is a representation of conditions at the time & place of drilling. With the passage of time or at any other location there may be consequential changes in conditions.
								SM SP			**WASH DEPOSITS:** Sand/Silty Sand, light brown, loose, dry.
	■		12 21 27		1.0	116.4	<200 6%				@2.5' - becomes medium brown, medium dense, fine to medium grained.
	■		20 20 23	5	1.5	110.8				5	@5' - some gravel, dry to damp.
				10						10	
	■		72/6"	15	4.3	120.1	<200 40%	SM SC		15	**OLDER ALLUVIUM (?):** Silty to Clayey Sand, some gravel and cobbles, brown, very dense, damp.
	■		50/4"	20						20	
	▩		50/3"								Total Depth 21 feet No Groundwater Hole Backfilled

See Legend for Important Notes | *Driving Weight: 140 lbs/automatic

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GEOTECHNICAL CONSULTANTS

Boring B-9
Sheet 1 of 1

Log of Exploratory Boring B-10

Client: A.R.C. Las Flores Ltd.	Date: 12-12-89	W.O. 215-01
Project: Highway 138	Location: San Bernardino	
	Surface Elevation: ±	
	Total Depth: 15.0	

FIELD					LAB DATA						
Water	Core	Bulk	Blows*	Depth (ft)	Moisture Content (%)	Dry Density (Pcf)	Other	USCS	Graphics	Depth (ft)	DESCRIPTION AND REMARKS
								SP SM			**WASH DEPOSITS:** Sand/Silty Sand, light brown, loose, dry to slightly damp.
	■		2 5 32		2.2	120.5	<200 30%				@2.5' - some gravel and clay, loose to medium dense.
	■		36 58 59	5	1.6	121.3				5	
								SM SC			**TERRACE DEPOSITS (?):** Silty to Clayey Sand, medium to light brown, dense to very dense, damp.
	■		50 104	10	4.6	126.0	<200 49%			10	
	▨		15 30 42	15						15	Total Depth 15 feet No Groundwater Hole Backfilled

This log is a representation of conditions at the time & place of drilling. With the passage of time or at any other location there may be consequential changes in conditions.

See Legend for Important Notes | *Driving Weight: 140 lbs/automatic

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GEOTECHNICAL CONSULTANTS

Boring B-10
Sheet 1 of 1

Log of Exploratory Boring B-11

Client: A.R.C. Las Flores Ltd.	Date: 12-12-89	W.O. 215-01
Project: Highway 138	Location: San Bernardino	
	Surface Elevation: ±	
	Total Depth: 10.0	

FIELD					LAB DATA						
Water	Core	Bulk	Blows*	Depth (ft)	Moisture Content (%)	Dry Density (Pcf)	Other	USCS	Graphics	Depth (ft)	DESCRIPTION AND REMARKS
								SC			FILL: Clayey Sand, brown, medium dense, dry.
								SM			CROWDER FORMATION (?): Silty Sand with Some Gravel, light brown, very dense, damp.
	■		35 60		2.7	120.6					
	■		33 30/4"	5	6.4	117.9	<200 53%	SM ML		5	@5' - fine grained, very silty
	▩		33 42 50/5"	10						10	@10' - Silty Fine Sand/Fine Sandy Silt, brown, very dense, damp, some clay.

This log is a representation of conditions at the time & place of drilling. With the passage of time or at any other location there may be consequential changes in conditions.

Total Depth 10 feet
No Groundwater
Hole Backfilled

See Legend for Important Notes | *Driving Weight: 140 lbs/automatic

JOHN A. SAYERS & ASSOCIATES
GEOTECHNICAL CONSULTANTS

Boring B-11
Sheet 1 of 1

Log of Exploratory Boring B-12

Client: A.R.C. Las Flores Ltd.	Date: 12-12-89	W.O. 215-01
Project: Highway 138	Location: San Bernardino	
	Surface Elevation: ±	
	Total Depth: 20.0	

FIELD					LAB DATA						DESCRIPTION AND REMARKS
Water	Core	Bulk	Blows*	Depth (ft)	Moisture Content (%)	Dry Density (Pcf)	Other	USCS	Graphics	Depth (ft)	This log is a representation of conditions at the time & place of drilling. With the passage of time or at any other location there may be consequential changes in conditions.
								SM			**FILL/WASH DEPOSITS:** Silty Sand with Clay and Gravel, brown, loose to medium dense, dry to damp.
	■		12 11 14		3.7	104.7	<200 12%				
	■		7 12 18	5	1.9	109.0				5	
	■		8 11 12	10	7.9	113.1				10	@10' - becoming clayey
	▩		30 51	15			<200 51%	SM ML		15	**CROWDER FORMATIOM (?):** Silty Sand/Sandy Silt with clay, light brown, very dense, damp, some gravel.
	■		65/6"	20	5.4	120.3				20	Total Depth 20 feet No Groundwater Hole Backfilled

See Legend for Important Notes | *Driving Weight: 140 lbs./automatic

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Boring B-12
Sheet 1 of 1

Log of Exploratory Boring B-15

Client: A.R.C. Las Flores Ltd.	Date: 12-12-89	W.O. 215-01
Project: Highway 138	Location: San Bernardino	
	Surface Elevation: ±	
	Total Depth: 35.0	

FIELD					LAB DATA						
Water	Core	Bulk	Blows*	Depth (ft)	Moisture Content (%)	Dry Density (Pcf)	Other	USCS	Graphics	Depth (ft)	DESCRIPTION AND REMARKS
								SM SC			ALLUVIUM: Silty/Clayey Sand, dark to medium brown, loose to medium dense, damp, scattered gravel.
	■		23 27 27		6.0	120.3	<200 30%				
	■		6 8 6	5	7.2	109.5				5	
	■		3 4 4	10	8.2	106.3	<200 43%			10	
	■		4 5 7	15	5.5	110.3				15	
	▩		2 3 4	20						20	

This log is a representation of conditions at the time & place of drilling. With the passage of time or at any other location there may be consequential changes in conditions.

See Legend for Important Notes | *Driving Weight: 140 lbs/automatic

JOHN A. SAYERS & ASSOCIATES
GEOTECHNICAL CONSULTANTS

Boring B-15
Sheet 1 of 2

Log of Exploratory Boring B-13

Client: A.R.C. Las Flores Ltd.	Date: 12-12-89	W.O. 215-01
Project: Highway 138	Location: San Bernardino	
	Surface Elevation: ±	
	Total Depth: 15.0	

FIELD					LAB DATA						
Water	Core	Bulk	Blows*	Depth (ft)	Moisture Content (%)	Dry Density (Pcf)	Other	USCS	Graphics	Depth (ft)	DESCRIPTION AND REMARKS
								SM SC			**FILL:** Silty to Clayey Sand with Gravel, brown, loose to medium dense, dry to damp.
			57 132	5	3.1	121.9	<200 32%	SM		5	**CROWDER FORMATION:** Silty Sand with Clay, light brown, very dense, damp, some small gravel.
			40 38 50/5"	10			<200 27%			10	
			50/6"	15						15	Total Depth 15 feet No Groundwater Hole Backfilled

This log is a representation of conditions at the time & place of drilling. With the passage of time or at any other location there may be consequential changes in conditions.

See Legend for Important Notes | *Driving Weight: 140 lbs/automatic

JOHN A. SAYERS & ASSOCIATES
GEOTECHNICAL CONSULTANTS

Boring B-13
Sheet 1 of 1

Log of Exploratory Boring B-14

Client: A.R.C. Las Flores Ltd.	Date: 12-12-89	W.O. 215-01
Project: Highway 138	Location: San Bernardino	
	Surface Elevation: ±	
	Total Depth: 10.0	

FIELD					LAB DATA						
Water	Core	Bulk	Blows*	Depth (ft)	Moisture Content (%)	Dry Density (Pcf)	Other	USCS	Graphics	Depth (ft)	DESCRIPTION AND REMARKS
								SM			FILL: Silty Sand with Gravel, reddish brown, medium dense, dry to damp.
								SP SM			TERRACE DEPOSIT: Sand/Silty Sand with Gravel, light reddish brown, very dense, damp.
	■		34 62		3.9	122.2	<200 4%				
	■		70/6"	5	4.1	114.2				5	
	▩		28 58	10						10	Total Depth 10 feet No Groundwater Hole Backfilled

This log is a representation of conditions at the time & place of drilling. With the passage of time or at any other location there may be consequential changes in conditions.

See Legend for Important Notes | *Driving Weight: 140 lbs/automatic

JOHN A. SAYERS & ASSOCIATES
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Boring B-14
Sheet 1 of 1

Log of Exploratory Boring B-15

Client: A.R.C. Las Flores Ltd.	Date: 12-12-89	W.O. 215-01
Project: Highway 138	Location: San Bernardino	
	Surface Elevation: ±	
	Total Depth: 35.0	

FIELD					LAB DATA						DESCRIPTION AND REMARKS
Water	Core	Bulk	Blows*	Depth (ft)	Moisture Content (%)	Dry Density (Pcf)	Other	USCS	Graphics	Depth (ft)	This log is a representation of conditions at the time & place of drilling. With the passage of time or at any other location there may be consequential changes in conditions.
			6 7 10		16.0	101.1					Alluvium continues, silty/clayey sand, medium dense, damp.
			3 2 5	30						30	
			8 28 30	35	3.7	115.2				35	Total Depth 35 feet No Groundwater Hole Backfilled

See Legend for Important Notes | *Driving Weight: 140 lbs/automatic

JOHN A. SAYERS & ASSOCIATES
GEOTECHNICAL CONSULTANTS

Boring B-15
Sheet 2 of 2

Log of Exploratory Boring B-16

Client: A.R.C. Las Flores Ltd.	Date: 12-12-89	W.O. 215-01
Project: Highway 138	Location: San Bernardino	
	Surface Elevation: ±	
	Total Depth: 14.5	

FIELD					LAB DATA						
Water	Core	Bulk	Blows*	Depth (ft)	Moisture Content (%)	Dry Density (Pcf)	Other	USCS	Graphics	Depth (ft)	DESCRIPTION AND REMARKS
								ML			ALLUVIUM: Silty Sand, medium brown, damp, loose to medium dense, some gravel, portions slightly clayey.
							<200 19%				
	■		3 4 4		7.3	105.2					
				5						5	
	■		3 3 5		5.7	99.3					
				10						10	
	■		3 3 5		9.6	109.9					

This log is a representation of conditions at the time & place of drilling. With the passage of time or at any other location there may be consequential changes in conditions.

Total Depth 14.5 feet
No Groundwater
Hole Backfilled

See Legend for Important Notes | *Driving Weight: 140 lbs/automatic

JOHN A. SAYERS & ASSOCIATES
GEOTECHNICAL CONSULTANTS

Boring B-16
Sheet 1 of 1

Log of Exploratory Boring B-1A

JASA

Client: **Rancho Las Flores Limited Partnership**	Date: **5-15-96**	W.O. **215-1**
Project: **Highway 138**	Location: **San Bernardino County**	

Surface Elevation: ~

Total Depth: ~47

FIELD					LAB DATA						
Water	Core	Bulk	Blows*	Depth (ft)	Moisture Content (%)	Dry Density(pcf)	Relative Compaction	USCS	Graphics	Depth (ft)	DESCRIPTION AND REMARKS
											This log is a representation of conditions at the time & place of drilling. With the passage of time or at any other location there may be consequential changes in conditions.
											CROWDER FORMATION: Sandstone fine to medium grained, scattered pebbles, very light brown, hard, slightly weathered, damp
	■	■	6/6"	5	8.0	122.5				5	Slightly cemented, very hard
	■	■	8/6" 3/1"	10	8.1	126.1				10	
				15						15	
	■	■	5/6" 9/6"	20	10.4	127.4				20	Fine-grained with silt, very hard

See Legend for Important Notes

*Driving Weight: **3350 to 29' - 2040 to 58'**

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Boring B-1A

Sheet 1 of 2

Log of Exploratory Boring B-1A

JASA

Client: **Rancho Las Flores Limited Partnership**	Date: **5-15-96**	W.O. **215-1**
Project: **Highway 138**	Location: **San Bernardino County**	
	Surface Elevation: ~	
	Total Depth: ~ **47**	

FIELD					LAB DATA						DESCRIPTION AND REMARKS
Water	Core	Bulk	Blows*	Depth (ft)	Moisture Content (%)	Dry Density(pcf)	Relative Compaction	USCS	Graphics	Depth (ft)	This log is a representation of conditions at the time & place of drilling. With the passage of time or at any other location there may be consequential changes in conditions.
											Crowder Formation continued
	■		8/6" 12/1"	30	6.2	128.7				30	Scattered pebbles, very hard
				35						35	
	■		18/6" 7/1"	40	4.7	124.5				40	Scattered pebbles and gravel, very hard
				45						45	
											@ 47 ' - encountered large cobble or small boulder, could not get past with bucket, discontinued drilling
											Total depth 47 feet. No water. No caving.

See Legend for Important Notes

*Driving Weight: **3350 to 29' - 2040 to 58'**

John A. Sayers & Associates
GEOTECHNICAL CONSULTANTS

Boring B-1A
Sheet 2 of 2

Log of Exploratory Boring B-1A

JASA

Client: **Rancho Las Flores Limited Partnership**	Date: **5-15-96**	W.O. **215-1**
Project: **Highway 138**	Location: **San Bernardino County**	
	Surface Elevation: ~	
	Total Depth: ~ **47**	

FIELD					LAB DATA						
Water	Core	Bulk	Blows*	Depth (ft)	Moisture Content (%)	Dry Density(pcf)	Relative Compaction	USCS	Graphics	Depth (ft)	DESCRIPTION AND REMARKS
											This log is a representation of conditions at the time & place of drilling. With the passage of time or at any other location there may be consequential changes in conditions.
											Crowder Formation continued
	■		8/6" 12/1"	30	6.2	128.7				30	Scattered pebbles, very hard
				35						35	
	■		18/6" 7/1"	40	4.7	124.5				40	Scattered pebbles and gravel, very hard
				45						45	
											@ 47 ' - encountered large cobble or small boulder, could not get past with bucket, discontinued drilling
											Total depth 47 feet. No water. No caving.

See Legend for Important Notes

*Driving Weight: **3350 to 29' - 2040 to 58'**

John A. Sayers & Associates
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Boring B-1A
Sheet 2 of 2

Log of Exploratory Boring B-2A

JASA

Client: **Rancho Las Flores Limited Partnership**	Date: **5-15-96**	W.O. **215-1**
Project: **Highway 138**	Location: **San Bernardino County**	
	Surface Elevation: ~	
	Total Depth: ~ **45**	

FIELD					LAB DATA						
Water	Core	Bulk	Blows*	Depth (ft)	Moisture Content (%)	Dry Density(pcf)	Relative Compaction	USCS	Graphics	Depth (ft)	DESCRIPTION AND REMARKS
											This log is a representation of conditions at the time & place of drilling. With the passage of time or at any other location there may be consequential changes in conditions.
											CROWDER FORMATION: Sandstone fine to medium grained, scattered pebbles, very light brown, hard, slightly weathered, damp
	■	■	5/6" 9/1"	5	8.4	131.5				5	Very hard
	■	■	M	10	12.0	127.5				10	Silty Fine to Medium Sand, very hard
	■		8/6" 7/2"	15	8.5	128.0				15	Silty with Trace Clay, very hard
	■		15/6"	20	5.3	118.5				20	Gravelly Fine tóo Medium Sand, scattered gravels continue below, very hard

See Legend for Important Notes

*Driving Weight: **3350 to 29' - 2040 to 58'**

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Boring B-2A

Sheet 1 of 2

Log of Exploratory Boring B-2A

JASA

Client: **Rancho Las Flores Limited Partnership**	Date: **5-15-96**	W.O. **215-1**
Project: **Highway 138**	Location: **San Bernardino County**	
	Surface Elevation: ~	
	Total Depth: ~ **45**	

FIELD					LAB DATA						
Water	Core	Bulk	Blows*	Depth (ft)	Moisture Content (%)	Dry Density(pcf)	Relative Compaction	USCS	Graphics	Depth (ft)	DESCRIPTION AND REMARKS
			15/6"		4.6	122.0					Crowder Formation continued, very hard
				30						30	
			17/6" 8/1"	35	6.0	123.3				35	Scattered gravel, very hard
				40						40	
			16/6" 9/1"	45	6.0	120.1				45	Very hard, contains coarse sand

This log is a representation of conditions at the time & place of drilling. With the passage of time or at any other location there may be consequential changes in conditions.

Total depth 45 feet.
No water. No caving.

See Legend for Important Notes

*Driving Weight: **3350 to 29' - 2040 to 58'**

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Boring B-2A
Sheet 2 of 2

DATE JUNE 1, 1990

SOIL SURVEY SHEET

LIMITS PROPOSED ROUTE 138
STA. 61+99.61 TO STA. 209+00

ESAL 300,000
EWL
Traffic Index ____ Base ____ Subbase ____ Basement Soil ____
Combined Cohesion of Layers Above

BORING OR SAMPLE No.	B-1	B-1	B-1	B-2	B-2	B-2	B-3	B-3
TEST NUMBER (D or H)								
DATE SAMPLED								
STATION	63+34	63+34	63+34	72+04	72+04	72+04	80+30	80+30
LINE								
DISTANCE FROM LINE (RT. or LT.)	—	—	—	—	—	—	19' RT.	19' RT
DEPTH OR ELEVATION	5'	7.5'	10'	2'	5'	7.5'	2.5'	5'
TYPE OF MATERIAL	SILTY SAND	SILTY SAND	SILTY SAND	SILTY SAND	SILTY SAND	SILTY SAND	SILTY SAND	SILTY SAND
1½" SQUARE OPENING								
¾" " "								
⅜" " "				100				
No. 4 (4760 MICRONS)				100				
No. 10 ~~8 (2380 ")~~				91.3				
No. 20 ~~16 (1190 ")~~				74.2				
No. 40 ~~30 (590 ")~~				56.6				
No. 100 ~~50 (297 ")~~				34.5				
No. 200 (74 ")		8.3	20.2	25.7		5.1		22.1
5 MICRON								
1 MICRON								
IN-PLACE DENSITY (DRY WT.- lb/ft³)	113.3	114.6	125.3	106.9	121.6	107.5	—	106.2
IN-PLACE MOISTURE (PERCENT)	1.4	12.7	10.9	4.8	1.3	1.7	0.8	3.2
MAXIMUM DENSITY (DRY WT.- lb/ft³)		131.5			133.0			
OPT. MOISTURE (PERCENT)		8.5			7.8			
RELATIVE COMPACTION (PERCENT)								
OIL RATIO - TYPE OF BITUMEN								
STABILOMETER VALUE								
COHESION								
SWELL								
STABILOMETER VALUE AFTER MVS								
% CEMENT								
OPT. MOISTURE								
DENSITY								
COMP. STRENGTH AT 7 DAYS								
LIQUID LIMIT								
PLASTICITY INDEX								
SAND EQUIVALENT		46.0						
PRODUCT P.I. X-200								
MOISTURE % AT COMPACTION		8.8						
DENSITY		130.4						
R-VALUE - STABILOMETER		78						
R-VALUE BY EXPANSION								
COHESION								
THICKNESS OF COVER REQUIRED		G.E. = 0.56 FT.						

SEE SHT 7 FOR CORROSION DATA

JOHN A. SAYERS & ASSOCIATES

GEOTECHNICAL CONSULTANTS

DATE JUNE 1, 1990

SOIL SURVEY SHEET

LIMITS PROPOSED ROUTE 138
STA. 61+99.61 TO STA. 209+00

ESAL 300,000
EWL

Combined Cohesion of Layers Above

Traffic Index ____ Base ____ Subbase ____ Basement Soil ____

DESCRIPTION	BORING OR SAMPLE No.	B-3	B-4	B-4	B-4	B-4	B-4	B-5	B-5
	TEST NUMBER (D or H)								
	DATE SAMPLED								
	STATION	80+30	87+85	87+85	87+85	87+85	87+85	95+24	95+24
	LINE								
	DISTANCE FROM LINE (RT. or LT.)	19' RT.	40' LT.	40' LT.	40' LT.	40' LT.	40' LT.	30' RT.	30' RT.
	DEPTH OR ELEVATION	10'	2.5'	5'	10'	20'	25'	2.5'	5'
	TYPE OF MATERIAL	SILTY SAND	SILTY SAND	SILTY SAND	SILTY SAND	SILTY SAND	SILTY SAND	SILTY SAND	SILTY SAND
SIEVE ANALYSIS	1½" SQUARE OPENING								
	¾" " "								
	⅜" " "			96.2					
	No. 4 (4760 MICRONS)			92.3					
	No. 10 ~~8 (2380 ")~~			81.6					
	No. 20 ~~16 (1190 ")~~			60.3					
	No. 40 ~~30 (590 ")~~			36.9					
	No. 100 ~~50 (297 ")~~			14.4					
	No. 200 (74 ")		15.7	9.1				3.5	
	5 MICRON								
	1 MICRON								
COMPACTION	IN-PLACE DENSITY (DRY WT.- lb/ft³)	106.2	126.9	—	114.9	121.8	—	104.3	107.8
	IN-PLACE MOISTURE (PERCENT)	1.4	2.6	2.0	1.0	3.0	2.8	0.8	2.0
	MAXIMUM DENSITY (DRY WT.- lb/ft³)								
	OPT. MOISTURE (PERCENT)								
	RELATIVE COMPACTION (PERCENT)								
OIL MIX.	OIL RATIO - TYPE OF BITUMEN								
	STABILOMETER VALUE								
	COHESION								
	SWELL								
	STABILOMETER VALUE AFTER MVS								
CTB	% CEMENT								
	OPT. MOISTURE								
	DENSITY								
	COMP. STRENGTH AT 7 DAYS								
	LIQUID LIMIT								
	PLASTICITY INDEX								
	SAND EQUIVALENT								
	PRODUCT P.I. X-200								
R-VALUE	MOISTURE % AT COMPACTION								
	DENSITY								
	R-VALUE - STABILOMETER								
	R-VALUE BY EXPANSION								
	COHESION								
	THICKNESS OF COVER REQUIRED								

SEE SHT 7 FOR CORROSION DATA

JOHN A. SAYERS & ASSOCIATES

GEOTECHNICAL CONSULTANTS

A E JUNE 1, 1990

SOIL SURVEY SHEET

LIMITS PROPOSED ROUTE 138

STA. 61+99.61 TO STA. 209+00

ESAL 300,000

EWL

Traffic Index ____ Base ____ Subbase ____ Basement Soil ____

Combined Cohesion of Layers Above

BORING OR SAMPLE No.	B-5	B-5	B-6	B-6	B-6	B-7	B-7	B-7
TEST NUMBER (D or H)								
DATE SAMPLED								
STATION	95+24	95+24	102+55	102+55	102+55	109+30	109+30	109+30
LINE								
DISTANCE FROM LINE (RT. or LT.)	30' RT.	30' RT.	18' LT.	18' LT.	18' LT.	60' RT.	60' RT.	60' RT.
DEPTH OR ELEVATION	7.5'	10'	2.5'	3'-6'	7.5'	0'-3'	5'	10'
TYPE OF MATERIAL	SILTY SAND	SILTY SAND	SILTY SAND	SILTY SAND	SILTY SAND	CLAYEY SAND	CLAYEY SAND	CLAYEY SAND
1½" SQUARE OPENING								
¾" " "								
⅜" " "				100				
No. 4 (4760 MICRONS)				100				
No. 10 ~~8 (2380 ")~~				84.7				
No. 20 ~~16 (1190 ")~~				57.2				
No. 40 ~~30 (590 ")~~				36.8				
No. 100 ~~50 (297 ")~~				17.7				
No. 200 (74 ")	12.1			11.5		46.1	62.1	56.3
5 MICRON								
1 MICRON								
IN-PLACE DENSITY (DRY WT.- lb/ft³)	110.6	113.7	106.1	117.9	115.1	129.2	123.5	124.3
IN-PLACE MOISTURE (PERCENT)	2.9	2.9	1.3	1.3	1.1	5.0	11.3	8.0
MAXIMUM DENSITY (DRY WT.- lb/ft³)				130.0		131.0		
OPT. MOISTURE (PERCENT)				10.1		8.5		
RELATIVE COMPACTION (PERCENT)								
OIL RATIO - TYPE OF BITUMEN								
STABILOMETER VALUE								
COHESION								
SWELL								
STABILOMETER VALUE AFTER MVS								
% CEMENT								
OPT. MOISTURE								
DENSITY								
COMP. STRENGTH AT 7 DAYS								
LIQUID LIMIT								
PLASTICITY INDEX								
SAND EQUIVALENT				37				
PRODUCT P.I. X-200								
MOISTURE % AT COMPACTION				9.4				
DENSITY				129.1				
R-VALUE - STABILOMETER				68				
R-VALUE BY EXPANSION								
COHESION								
THICKNESS OF COVER REQUIRED				G.E. = 0.82 FT.				

SEE SHT 7 FOR CORROSION DATA

JOHN A. SAYERS & ASSOCIATES

GEOTECHNICAL CONSULTANTS

DATE JUNE 1, 1990

SOIL SURVEY SHEET

LIMITS PROPOSED ROUTE 138
STA. 61+99.61 TO STA. 209+00

ESAL 300,000
EWL

Combined Cohesion of Layers Above

Traffic Index ____ Base ____ Subbase ____ Basement Soil ____

BORING OR SAMPLE No.		B-9	B-9	B-9	B-10	B-10	B-10	
TEST NUMBER (D or H)								
DATE SAMPLED								
STATION		124+58	124+58	124+58	137+25	137+25	137+25	
LINE								
DISTANCE FROM LINE (RT. or LT.)		—	—	—	18' RT.	18' RT.	18' RT.	
DEPTH OR ELEVATION		2.5'	5'	15'	0'-3'	5'	10'	
TYPE OF MATERIAL		SILTY SAND	SILTY SAND	CLAYEY SAND	SILTY SAND	SILTY SAND	SILTY SAND	
ANALYSIS 1½" SQUARE OPENING								
¾" " "								
⅜" " "		91.7			96.5			
No. 4 (4760 MICRONS)		83.9			94.2			
No. 10 ~~8 (2380 ")~~		68.0			83.5			
No. 20 ~~16 (1190 ")~~		43.2			66.0			
No. 40 ~~30 (590 ")~~		25.2			52.7			
No. 100 ~~50 (297 ")~~		10.6			38.2			
No. 200 (74 ")		6.1		39.7	29.6		49.2	
5 MICRON								
1 MICRON								
IN-PLACE DENSITY (DRY WT.- lb/ft³)		116.4	110.8	120.1	120.5	121.3	126.0	
IN-PLACE MOISTURE (PERCENT)		1.0	1.5	4.3	2.2	1.6	4.6	
MAXIMUM DENSITY (DRY WT.- lb/ft³)					134.5			
OPT. MOISTURE (PERCENT)					8.5			
RELATIVE COMPACTION (PERCENT)								
OIL RATIO - TYPE OF BITUMEN								
STABILOMETER VALUE								
MIX. COHESION								
SWELL								
STABILOMETER VALUE AFTER MVS								
% CEMENT								
OPT. MOISTURE								
DENSITY								
COMP. STRENGTH AT 7 DAYS								
LIQUID LIMIT								
PLASTICITY INDEX								
SAND EQUIVALENT								
PRODUCT P.I. X-200								
MOISTURE % AT COMPACTION					10.1			
DENSITY					127.5			
R-VALUE - STABILOMETER					61			
R-VALUE BY EXPANSION								
COHESION								
THICKNESS OF COVER REQUIRED					G.E. = 0.99 FT.			

SEE SHT 7 FOR CORROSION DATA

JOHN A. SAYERS & ASSOCIATES

GEOTECHNICAL CONSULTANTS

DATE JUNE 1 1990

SOIL SURVEY SHEET

LIMITS PROPOSED ROUTE 138
STA. 61+99.61 TO STA. 209+00

ESAL 300,000
EWL

Combined Cohesion of Layers Above

Traffic Index ____ Base ____ Subbase ____ Basement Soil ____

DESCRIPTION	BORING OR SAMPLE No.	B-11	B-11	B-12	B-12	B-12	B-12	B-13	B-13
	TEST NUMBER (D or H)								
	DATE SAMPLED								
	STATION	145+34	145+34	156+00	156+00	156+00	156+00	168+70	168+70
	LINE								
	DISTANCE FROM LINE (RT. or LT.)	8' LT.	8' LT.	20' LT.	20' LT.	20' LT.	20' LT.	26' RT.	26' RT.
	DEPTH OR ELEVATION	2.5'	5'	2.5'	5'	10'	20'	5'	15'
	TYPE OF MATERIAL	SILTY SAND	SILTY SAND	SILTY SAND	SILTY SAND	CLAYEY SAND	SILTY SAND	SILTY SAND	SILTY SAND
SIEVE ANALYSIS	1½" SQUARE OPENING								
	¾" " "								
	⅜" " "								100
	No. 4 (4760 MICRONS)								99.8
	No. 10 ~~8 (2380 ")~~								87.4
	No. 20 ~~16 (1190 ")~~								67.4
	No. 40 ~~30 (590 ")~~								51.1
	No. 100 ~~50 (297 ")~~								33.7
	No. 200 (74 ")		52.7	12.2		51.0		32.2	26.5
	5 MICRON								
	1 MICRON								
COMPACTION	IN-PLACE DENSITY (DRY WT.- lb/ft³)	120.6	117.9	104.7	109.0	113.1	120.3	121.9	–
	IN-PLACE MOISTURE (PERCENT)	2.7	6.4	3.7	1.9	7.9	5.4	3.1	–
	MAXIMUM DENSITY (DRY WT.- lb/ft³)								
	OPT. MOISTURE (PERCENT)								
	RELATIVE COMPACTION (PERCENT)								
BIT. MIX.	OIL RATIO - TYPE OF BITUMEN								
	STABILOMETER VALUE								
	COHESION								
	SWELL								
	STABILOMETER VALUE AFTER MVS								
CTB	% CEMENT								
	OPT. MOISTURE								
	DENSITY								
	COMP. STRENGTH AT 7 DAYS								
	LIQUID LIMIT								
	PLASTICITY INDEX								
	SAND EQUIVALENT								30
	PRODUCT P.I. X-200								
	MOISTURE % AT COMPACTION								
R-VALUE	DENSITY								
	R-VALUE - STABILOMETER								
	R-VALUE BY EXPANSION								
	COHESION								
	THICKNESS OF COVER REQUIRED								

SEE SHT 7 FOR CORROSION DATA

JOHN A. SAYERS & ASSOCIATES

GEOTECHNICAL CONSULTANTS

DATE JUNE 1, 1990

SOIL SURVEY SHEET

LIMITS PROPOSED ROUTE 138
STA. 61+99.61 TO STA. 209+00

ESAL 300,000
EWL

Combined Cohesion of Layers Above

Traffic Index ____ Base ____ Subbase ____ Basement Soil ____

BORING OR SAMPLE No.	B-14	B-14	B-15	B-15	B-15	B-15	B-15	B-15
TEST NUMBER (D or H)								
DATE SAMPLED								
STATION	190+00	190+00	200+42	200+42	200+42	200+42	200+42	200+42
LINE								
DISTANCE FROM LINE (RT. or LT.)	—	—	12' LT.	12' LT.	12' LT.	12' LT.	12' LT.	12' LT.
DEPTH OR ELEVATION	2.5'	5.0'	0'-3'	2.5'	5.0'	10.0'	15.0	25.0'
TYPE OF MATERIAL	SILTY SAND	SILTY SAND	SILTY SAND	SILTY SAND	SILTY SAND	SILTY SAND	SILTY SAND	SILTY SAND
AVAILOID 1½" SQUARE OPENING								
¾" " "								
⅜" " "	93.5							
No. 4 (4760 MICRONS)	88.3							
No. 8 (2380 ")	75.8							
No. 16 (1190 ")	54.4							
No. 30 (590 ")	33.9							
No. 50 (297 ")	12.6							
No. 200 (74 ")	4.4		30			43		
5 MICRON								
1 MICRON								
IN-PLACE DENSITY (DRY WT.- lb/ft³)	122.2	114.3	101.1	120.3	109.5	106.3	110.3	101.1
IN-PLACE MOISTURE (PERCENT)	3.9	4.1	16.0	6.0	7.2	8.2	5.5	16.0
MAXIMUM DENSITY (DRY WT.- lb/ft³)			133.0					
OPT. MOISTURE (PERCENT)			8.0					
RELATIVE COMPACTION (PERCENT)								
MIX. OIL RATIO - TYPE OF BITUMEN								
STABILOMETER VALUE								
COHESION								
SWELL								
STABILOMETER VALUE AFTER MVS								
% CEMENT								
OPT. MOISTURE								
DENSITY								
COMP. STRENGTH AT 7 DAYS								
LIQUID LIMIT								
PLASTICITY INDEX								
SAND EQUIVALENT								
PRODUCT P.I. X-200								
MOISTURE % AT COMPACTION			10.0					
DENSITY			130.2					
R-VALUE - STABILOMETER			67.0					
R-VALUE BY EXPANSION								
COHESION								
THICKNESS OF COVER REQUIRED			G.E. = 0.84 FT.					

SEE SHT 7 FOR CORROSION DATA

JOHN A. SAYERS & ASSOCIATES

GEOTECHNICAL CONSULTANTS

A E JUNE 1, 1990

SOIL SURVEY SHEET

LIMITS PROPOSED ROUTE 138
STA. 61+99.61 TO STA. 209+00

ESAL 300,000
EWL

Combined Cohesion of Layers Above

Traffic Index ____ Base ____ Subbase ____ Basement Soil ____

BORING OR SAMPLE No.	B-16	B-16		B-1	B-6	B-10	B-13	B-15
TEST NUMBER (D or H)								
DATE SAMPLED								
STATION	209+20	209+20		63+34	102+50	137+00	169+00	200+25
LINE								
DISTANCE FROM LINE (RT. or LT.)	15' RT.	15' RT.		–	18' LT.	18' RT.	26' RT.	12' LT.
DEPTH OR ELEVATION	0'-3'	10.0'		6'-8'	3'-6'	0'-3'	12'-15'	0'-3'
TYPE OF MATERIAL	SILTY SAND	SILTY SAND		SILTY SAND	SILTY SAND	SILTY SAND	SILTY SAND	SILTY SAND
1½" SQUARE OPENING								
¾" " "				CORROSION DATA	CORROSION DATA	CORROSION DATA	CORROSION DATA	CORROSION DA
⅜" " "	100		Ohm/cm³	14250	14250	11880	3960	5280
No. 4 (4760 MICRONS)	100		pH	7.43	6.80	8.90	8.31	7.59
No. 8 (2380 ")	90.2		SO_4	<.001%	<.001%	<.001%	<.001%	<.001%
No. 16 (1190 ")	69.5							
No. 30 (590 ")	49.2							
No. 50 (297 ")	26.2							
No. 200 (74 ")	19.0							
5 MICRON								
1 MICRON								
IN-PLACE DENSITY (DRY WT.- lb/ft³)	105.2	109.9						
IN-PLACE MOISTURE (PERCENT)	7.3	9.6						
MAXIMUM DENSITY (DRY WT.- lb/ft³)	131.7							
OPT. MOISTURE (PERCENT)	8.1							
RELATIVE COMPACTION (PERCENT)								
OIL RATIO - TYPE OF BITUMEN								
STABILOMETER VALUE								
COHESION								
SWELL								
STABILOMETER VALUE AFTER MVS								
% CEMENT								
OPT. MOISTURE								
DENSITY								
COMP. STRENGTH AT 7 DAYS								
LIQUID LIMIT								
PLASTICITY INDEX								
SAND EQUIVALENT	29							
PRODUCT P.I. X-200								
MOISTURE % AT COMPACTION	12.0							
DENSITY	123.6							
R-VALUE - STABILOMETER	61							
R-VALUE BY EXPANSION								
COHESION								
THICKNESS OF COVER REQUIRED	G.E. = 0.99							

SEE SHT 7 FOR CORROSION DATA

JOHN A. SAYERS & ASSOCIATES

GEOTECHNICAL CONSULTANTS

DATE JUNE 1, 1990

SOIL SURVEY SHEET

LIMITS PROPOSED ROUTE 138
STA. 61+99.61 TO STA. 209+00

ESAL 300,000
EWL

Combined Cohesion of Layers Above

Traffic Index ____ Base ____ Subbase ____ Basement Soil ____

DESCRIPTION	BORING OR SAMPLE No.	B-1A	B-1A	B-1A	B-1A	B-1A	B-2A	B-2A	B-2A
	TEST NUMBER (D or H)								
	DATE SAMPLED	5-15-96	5-15-96	5-15-96	5-15-96	5-15-96	5-15-96	5-15-96	5-15-96
	STATION	189 + 35	189 + 35	189 + 35	189 + 35	189 + 35	174 + 90	174 + 90	174 + 90
	LINE								
	DISTANCE FROM LINE (RT. or LT.)	—	—	—	—	—	—		
	DEPTH OR ELEVATION	5'	10'	20'	30'	40'	5'	10'	15'
	TYPE OF MATERIAL	SILTY SAND	SILTY TO CLAYEY SAND	SILTY SAND	SILTY SAND	SILTY SAND	SILTY TO CLAYEY SAND	CLAYEY SAND	SILTY SAND
SIEVE ANALYSIS	1½" SQUARE OPENING								
	¾" " "								
	⅜" " "								
	No. 4 (4760 MICRONS)	94.3	99.7	100			96.8	99.3	
	No. 10 ~~8 (2380 ")~~	82.8	97.2	90.1			88.4	93.4	
	No. 20 ~~16 (1190 ")~~	63.5	93.0	75.4			75.0	78.3	
	No. 40 ~~30 (590 ")~~	43.2	86.3	60.6			60.4	67.2	
	No. 100 ~~50 (297 ")~~	21.8	61.3	39.7			40.2	51.1	
	No. 200 (74 ")	14.2	40.7	27.3			28.3	40.3	
	5 MICRON								
	1 MICRON								
COMPACTION	IN-PLACE DENSITY (DRY WT.- lb/ft³)	122.5	126.1	127.4	128.7	124.5	131.5	127.5	128.0
	IN-PLACE MOISTURE (PERCENT)	8.0	8.1	10.4	6.2	4.7	8.4	12.0	8.5
	MAXIMUM DENSITY (DRY WT.- lb/ft³)		128.5	124.5					
	OPT. MOISTURE (PERCENT)		9.5	11.0					
	RELATIVE COMPACTION (PERCENT)		98.1						
BIT. MIX.	OIL RATIO - TYPE OF BITUMEN								
	STABILOMETER VALUE								
	COHESION								
	SWELL								
	STABILOMETER VALUE AFTER MVS								
CTB	% CEMENT								
	OPT. MOISTURE								
	DENSITY								
	COMP. STRENGTH AT 7 DAYS								
R-VALUE	LIQUID LIMIT		27.2	24.6			25.8	31.0	
	PLASTICITY INDEX	NON - PLASTIC	5.9	2.3			4.1	8.9	
	SAND EQUIVALENT								
	PRODUCT P.I. X-200								
	MOISTURE % AT COMPACTION								
	DENSITY								
	R-VALUE - STABILOMETER								
	R-VALUE BY EXPANSION								
	COHESION								
	THICKNESS OF COVER REQUIRED								

JOHN A. SAYERS & ASSOCIATES

GEOTECHNICAL CONSULTANTS

DATE JUNE 1, 1990

SOIL SURVEY SHEET

LIMITS PROPOSED ROUTE 138
STA. 61+99.61 TO STA. 209+00

ESAL 300,000
EWL
Combined Cohesion of Layers Above
Traffic Index ____ Base ____ Subbase ____ Basement Soil ____

DESCRIPTION	BORING OR SAMPLE No.	B-2A	B-2A	B-2A	B-2A	B-	B-	B-	B-
	TEST NUMBER (D or H)								
	DATE SAMPLED	5-15-96	5-15-96	5-15-96	5-15-96				
	STATION	174+90	174+90	174+90	174+90				
	LINE								
	DISTANCE FROM LINE (RT. or LT.)	—	—	—	—	—	—		
	DEPTH OR ELEVATION	20'	25'	35'	45'				
	TYPE OF MATERIAL	SILTY SAND	SILTY SAND	SILTY TO CLAYEY SAND	SILTY SAND				
SIEVE ANALYSIS	1½" SQUARE OPENING								
	¾" " "								
	⅜" " "								
	No. 4 (4760 MICRONS)			98.1	95.1				
	No. 10 ~~8 (2380 ")~~			90.0	86.1				
	No. 20 ~~16 (1190 ")~~			71.2	69.5				
	No. 40 ~~30 (590 ")~~			55.3	52.9				
	No. 100 ~~50 (297 ")~~			35.0	30.5				
	No. 200 (74 ")			24.7	19.6				
	5 MICRON								
	1 MICRON								
COMPACTION	IN-PLACE DENSITY (DRY WT.- lb/ft³)	118.5	122.0	123.3	120.1				
	IN-PLACE MOISTURE (PERCENT)	5.3	4.6	6.0	6.0				
	MAXIMUM DENSITY (DRY WT.- lb/ft³)								
	OPT. MOISTURE (PERCENT)								
	RELATIVE COMPACTION (PERCENT)								
BIT MIX	OIL RATIO - TYPE OF BITUMEN								
	STABILOMETER VALUE								
	COHESION								
	SWELL								
	STABILOMETER VALUE AFTER MVS								
CTB	% CEMENT								
	OPT. MOISTURE								
	DENSITY								
	COMP. STRENGTH AT 7 DAYS								
R-VALUE	LIQUID LIMIT			26.0					
	PLASTICITY INDEX			5.2	NON-PLASTIC				
	SAND EQUIVALENT								
	PRODUCT P.I. X-200								
	MOISTURE % AT COMPACTION								
	DENSITY								
	R-VALUE - STABILOMETER								
	R-VALUE BY EXPANSION								
	COHESION								
	THICKNESS OF COVER REQUIRED								

JOHN A. SAYERS & ASSOCIATES

GEOTECHNICAL CONSULTANTS

441.02 Material Report

RECEIVED
PROJECT DEVELOPMENT
96 JUN 11 PM 2:57

MATERIALS REPORT

FOR

PROPOSED HIGHWAY CONSTRUCTION

ROUTE 138, PHASE II

W.O. 215-1-03

MAY 29, 1996

JOHN A SAYERS & ASSOCIATES

JASA

John A Sayers and Associates, Inc.

Geotechnical Consultants

TRANSMITTAL

Rancho Las Flores Limited Partnership
33971 Selva Road, Suite 230
Laguna Niguel, CA 92677

May 29, 1996
W.O. 215-1-03

Attention: Mr. Cliff Hood

Subject: Materials Report, Route 138, Phase II, 1.2 Miles East of Route 15 to Summit Post Office Road, 08-SBd-138-16.5/19.7

Gentlemen:

Attached is a revised version of our Materials Report for Phase II of the proposed construction of Highway 138. This report presents the results of our recent and previous explorations, reconnaissance, and laboratory testing. The route and conditions have been described, as well as the geology and soil materials, which would be involved in construction.

Thank you for this opportunity to be of continued service. If you have any questions regarding the project or the conditions, please contact us.

Respectfully submitted,

JOHN A. SAYERS & ASSOCIATES

By: John A. Sayers
Engineering Geologist
CEG 907

By: John A. Seminara
Principal Engineer
GE 2294

REGISTERED PROFESSIONAL ENGINEER
JOHN A. SEMINARA
No. 2294
Exp. 12/31/98
GEOTECHNICAL
STATE OF CALIFORNIA

JAS/JS/mw

Dist: (1) Addressee
(4) C.M. Engineering, Attention: Mr. Mohamad Younes

Encl: (1) Materials Report for Proposed Highway Construction, Route 138, Phase II

27071 Cabot Road, Suite 101 • Laguna Hills, California 92653-7009 • (714) 582-2144

MATERIALS REPORT

FOR

PROPOSED HIGHWAY CONSTRUCTION

ROUTE 138, PHASE II

1.2 MILES EAST OF ROUTE 15
TO SUMMIT POST OFFICE ROAD

08-SBd-138-16.5/19.7

08201-26810

MAY 29, 1996
W.O. 215-1-03

JOHN A SAYERS & ASSOCIATES

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C. Soil Survey Sheets, Plates C-1 through C-7

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I. General

A. Description of Project

1. Proposed Improvements

The proposed project is to replace the existing two-lane roadway with a new four-lane roadway along Crowder Creek from 1.2 miles east of Interstate Route 15 (Mojave Freeway) to 0.2 miles east of Summit Post Office Road. The proposed work would include major grading, paving, and the installation of drainage facilities.

2. Climatic Conditions

The average annual rainfall in this area ranges between 14 and 20 inches, occurring during the winter months. Annual temperatures range from about 22 degrees to 116 degrees fahrenheit.

3. Terrain

The proposed alignment uses the same general corridor as the existing route. This area consists of rugged foothill terrain supporting chaparral-type brush, grasses, and occasional trees. Elevations vary from 3,270 feet at the west end of the improvement to 3,687 feet on the east end.

4. Geology

The project area is underlain by sandstone, fanglomerate and conglomerate of the Crowder Formation, older alluvium, and terrace deposits. These materials are light grey to slightly pink, grey to white and light reddish brown, respectively. Each formation has similar characteristics. In effect, they consist of sands and gravels and are very dense. These materials are exposed in cut slopes along the existing highway and nearby railway.

The older alluvium and terrace deposits overlie the Crowder Formation in portions of the proposed alignment. The Crowder Formation, in turn, overlies granitic bedrock at depth.

Bedding within the Crowder Formation is poorly developed and defined by localized and discontinuous pebble alignments and grain-size variations. The formation is best described as massive and lacks bedding surfaces considered as potential failure surfaces. Slope instabilities and/or landslides are lacking in the area. Where observed, bedding is dipping gently to the east at about 20 degrees. Regionally it is shown as dipping north to northeast.

The older alluvium and terrace deposits have similar bedding characteristics as the Crowder Formation, except that, where observed, the beds are essentially flat. Again, the lack of bedding surfaces is favorable to slopes.

A thin soil cover mantles most of the natural slopes and recent alluvium and wash deposits exist in gullies and canyons. A relatively thick alluvial deposit exists at the east end of the proposed alignment.

The west end of the project is approximately 1.5 miles northeast of the seismically active San Andreas Fault.

5. Groundwater

Generally, there is little or no groundwater within the project area except at great depth. Locally, such as at Borings 1 and 2, perched groundwater was encountered at shallow depths. This would affect excavations and embankment placement. Subdrains are recommended wherever groundwater is encountered or expected. Surface water would be expected during or after rainstorms.

B. Pertinent Reports (all prepared by Caltrans)

1. Project Report - for the proposed improvement of Route 138 in San Bernardino County, 1.2 miles east (08-SBd-138-R15.2/16.2), dated January 4, 1989.

2. Materials Report - for proposed highway construction between Interstate Route 15 and Crowder Creek on Route 138, dated January 12, 1989.

3. Typical Structural Section Approval - for Route 138 from Route 15 to 1.2 miles east (08-SBd-138-R15.2/16.4), dated January 4, 1989.

4. Report of Geophysical Investigation - along portions of Route 08-SBd-138, post miles 15.20/19.10, dated August 30, 1971.

II. Existing Facility

The existing road alignment consists of a typical pavement width of about 20 feet with narrow shoulders. The horizontal alignment contains short curve radii (50 feet) and steep grades (15 percent). The first 1.2 miles of Highway 138 east of Highway 15 (Phase 1) was completed in 1993.

III. Roadway, Culvert and Embankment Foundations

A. Description

The proposed roadway would be constructed on cut and fill sections with maximum vertical depths of about 100 feet at about station 182+00. Fill slopes are planned at 1-1/2:1 (horizontal to vertical) to maximum heights of about 70 feet. Cut slopes are planned at 1:1 to maximum heights of approximately 150 feet.

B. Foundation Treatment

The soil and bedrock materials underlying the proposed road alignment are generally sandy in nature and contain gravel.

Sand with about 50 percent cohesive (silt and clay) materials were encountered locally, although they are expected to be a small percentage and would likely be mixed with the sandy soils. If cohesive soils are exposed in cut road bed sections, overexcavation may be necessarv to improve the roadway section. Cohesive soils should also be avoided in the upper fill section affecting the roadway. We encountered higher cohesive percentages in Borings 9 through 13, although Borings 9 and 10 are in areas to receive fill. If necessary, overexcavations in roadbeds should be four feet below subgrade and replaced with a sandy soil.

Surface water or springs were not observed, although groundwater was encountered in Borings 1 and 2 at shallow depths. This water is locally perched in older alluvium above the Crowder Formation bedrock at the west end of the project (Borings 1 and 2, Stations ±63+00 to ±72+00) where shallow cuts and embankments are planned. Extra removals of wet soils, installation of subdrains, and overexcavations may be necessary to create a stable road section. Dewatering is not considered necessary as the water is typically perched one to two feet above the Crowder Formation. Subdrain recommendations are presented below in section III D.

Removal of loose fill, topsoil, colluvium and wash deposits is recommended in narrow canyon areas to minimize differential settlement. These removals are localized and expected to be on the order of a few feet to 15 feet deep (see Table below). Topsoil and shallow fill removals on slopes steeper than 5:1 could be removed by benching.

Table I
Approximate Locations of Canyons or Areas
With Expected Removal *

Approximate Station No.	Boring No.	Expected Depth (feet)
80+50 to 85+00	3	5
87+00 to 89+00	4	13
91+00	-	10-15
108+50 to 112+00	7	5-10
115+00 to 116+00	-	3-6
117+00 to 119+00	-	3-6
123+50 to 125+50	9	5
129+00 to 130+00	-	5-8
133+00 to 165+00	10	5-20
167+50	-	3-5
170+50 to 173+50	-	3-5
190+00 to end	16	3

* fill keys, benching and removals of existing fills, topsoil, colluvium occur locally along the entire alignment. There may be other removal areas not cited above.

C. Settlement

Subsidence of natural bedrock, older alluvium and terrace deposits is expected to be nil or less than 0.1 feet for most of the alignment. The eastern end of the alignment would be over a thick section of natural alluvial soils which would be expected to subside on the order of 0.25 to 0.5 feet. This is based upon an expected maximum embankment thickness of 20 feet. Most of this subsidence is expected to occur during construction.

D. Subdrains

Subdrains should consist of six-inch (6") diameter, perforated pipe, embedded in nine (9) cubic feet of permeable material enclosed in filter fabric. Discharge pipes which may extend through embankment fill should be non-perforated.

IV. Cut and Excavations

Previous geophysical investigations (Pertinent Report No. 4) were conducted along this alignment in 1971. This report should be consulted for conditions. We have plotted the approximate locations of these seismic lines on our geotechnical maps (enclosed). In general, they indicated rippable materials to the depths explored. Seismic velocities varied from about 2,500 to 3,400 feet per second material.

Cut slopes along the existing roadway and railway appear to have been easily excavated and are performing reasonably well. Minor surficial erosion and small, localized slumping is apparent on some of the slopes. The existing slopes are typically steeper than 1:1 and locally near vertical.

Shear strength parameters have been determined based on direct shear tests on in-situ samples taken during our 1990 study and more recently in May of 1996. The recent samples were taken near the eastern portion of the proposed road (Stations 169+00 and 195+00) where the highest cut slopes are programmed. Currently existing slopes in this area are nearly 100 feet high and at a 3/4:1 grade.

Shear strength parameters for samples at natural moisture content are shown on Plate A-1. These may be used in the eastern portion of the roadway where the natural topography is steeper and where the topography adjacent to the proposed cut falls away from the cut making a narrow ridgeline. The latter creates shedding of rainfall in both directions and therefore little opportunity for penetration. Because of this, it is unlikely

that the slope materials would become saturated. The performance of the existing, steep slopes along this route or along Route 15 where Crowder and similar materials are exposed has been good. We know of no landslide or gross instabilities in this area.

Shear strength parameters for saturated conditions are shown on Plate A-2. These are appropriate for the more gentle topographic conditions at the western portions of the highway improvement where water could penetrate and cause saturation.

Remolded shear strength parameters are shown on Plate A-3 which are appropriate for fill embankments.

As a conservative approach, we used half the cohesion for each case and these are shown in Table 2 and plotted on Plates A-1, A-2 and A-3.

A slope stability analysis was performed for various height slopes at gradients of 1:1 for cut slopes and 1½:1 for fill slopes. The maximum height of the proposed slopes are 160 and 75 feet, respectively. The analyses were conducted using PCSTABL Version 6H with the STED Version 6.54 interface, and using the modified Bishop Method. Factors of safety for the ten most critical circles for each case are shown on Plates A-4 through A-7. These are then summarized on Table 2 below.

Table 2
Summary of Slope Stability Calculations

Condition	Height (ft)	Gradient (H:V)	Min. Static F.S.	Min. Seismic F.S.	$\phi°$	C(psf)
Cut	100	1:1	1.77	1.51	43.5	650
Cut	160	1:1	1.51	1.20	43.5	650
Cut	40	1:1	1.99	1.47	31.5	450
Fill	75	1½:1	1.52	1.13	25	490

We have also performed surficial stability calculations and they are presented on Plates A-8 through A-12.

The results of our analyses indicate that the proposed cut slopes should be grossly and surficially stable at the proposed heights and proposed gradients. The addition of benches at 75 foot intervals for higher slopes tends to improve the factors of safety. Surficial erosion, however, is more difficult to analyze. Observation of the existing (old) cut slopes indicate minor erosion for slopes influenced only by direct rainfall and moderate erosion where perimeter drainage is allowed to drain onto a slope. The latter was especially true for the first 1.2 mile section of Highway 138 (Phase I) that was completed a couple of years ago. Brow drains and downdrains at specific locations to control offsite drainage should minimize the more severe erosion; and re-vegetation would mitigate less severe erosion. Flattening the slopes is not recommended as this tends to capture more direct rainfall thereby introducing more erosion potential. Likewise, a flatter slope tends to be higher which also captures more rainfall. Flatter slopes also increases the cost of excavation, would remove more native vegetation, increases maintenance due to more slope being exposed, and is less appealing due to the larger exposure. The latter are more visible at great distances. Flatter slopes would also require more right-of-way.

Our analyses also indicated that the proposed fill slopes should be grossly stable at the proposed heights and gradients. Fill slopes, however, are more subject to erosion and surficial damage due to the decrease in strength caused by remolding the soil. However, fill slopes can be reasonably stable at 1½:1 given adequate compaction and slope finishing, and by preventing any runoff from flowing onto the slope face. Two methods of slope compaction have proven successful. One method is to overfill the slope face and later trim off the excess to expose a compacted core. The second method calls for back rolling the slope every four vertical feet with a double drum sheepsfoot during the slope building process and to run heavy equipment close to the top of the slope. This creates a compacted core with only a minor amount of sluff on the slope. The slope should be built as close to grade as possible during this process. Then at completion of the entire slope, the slope should be shaped with a small tractor, sheepsfoot rolled using a side boom cat and after successful compaction testing, the slope should be grid rolled. We also recommend six (6) foot wide terrace drain benches at no more than 30 vertical feet and/or mid-height for slopes higher than 30 feet. In addition, drainage should not be allowed to spill onto the slope face.

V. Embankments

It is our understanding that the project is expected to balance the cut with the fill, with a provision for earthwork factors.

VI. Earthwork Factors

On the average, grading factors for cuts in the Crowder Formation and Terrace deposits would be expected to be 1.0, and for cuts in the older alluvium on the order of 0.90. Fill, topsoil, and wash deposit should have a grading factor of 0.9. The locations of the various deposits are shown on the attached maps. The bulk of the grading, i.e., higher cuts would be in the Crowder Formation and Terrace deposits at the eastern half of the proposed roadway.

VII. Corrosion Investigation

A. Corrosion testing was performed, however, all the planned culverts will be reinforced concrete pipe (R.C.P.).

B. There are no planned bridges.

VIII. Pavement Structural Design

Testing indicates that most on-site soils have R-values greater than 50, although soils collected from Borings 13 had an R-value of less than 20. The latter represents the more clayey portion of Crowder Formation. These clayey soils should be removed from cut sections and not be placed in fill sections within four feet of finished subgrade.

A proposed structural design is based on an R-value of 50, and an Average Daily Traffic Count of 30,000 for year 2020 and using a 13 percent Truck Traffic. The current Traffic data was provided by Caltrans in a District 8 Transportation Planning memorandum dated April 29, 1996. Therefore, a preliminary section is shown below based upon the Caltrans Highway Design Manual, Chapter 600 (July 1, 1990).

T.I.	=	10.5			
R-value	=	50			
A.C.	=	0.55 feet	=	1.68 cm	16.8 cm
AB	=	0.73 feet	=	2.23 cm	22.3 cm

The actual section should be based on testing at completion of roadway grading and based on soils exposed.

IX. Materials Available

A. Location Sources

The soil material needed for the fills is proposed to be generated from the cut sections.

The materials report for Phase I of Highway 138, dated January 12, 1989 (Pertinent Report No. 3), cited a local source as the DLH gravel deposit, located about three miles west of Interstate Route 15. Other sources may also be available.

B. Commercial Sources

The contractor for Phase I of the Highway 138 improvements reportedly used Owl Rock, the Lytle Creek Facility (1-800-232-3695), or C.L. Pharris as a secondary source (714-798-1144). Additionally, Rancho Las Flores expects to have materials available and other sources may also be available.

X. Water Availability

A. Construction Needs

The soils are expected to need considerable water to achieve near optimum moisture content for compaction. This need may vary with the time of year. Specific water sources are unknown at this time. Possible sources would be from the Rancho Las Flores area (well water and pump to temporary reservoir), an existing well at the intersection of Highway 138 and Route 15 (used for Phase 1) and a third well drilled in 1987 near the proposed alignment.

XI. Recommended Materials Specifications

A. Imported Borrow

Imported borrow is presently not proposed for this project. Should this become a contract item, it is suggested that an R-value of 50 be specified for materials within four feet of finished subgrade.

B. Other

Quality requirements for elements of the structural section and rock slope protection, as outlined in the Standard Specifications, are considered satisfactory.

XII. Estimates for Quantity Calculations

The following estimates can be used in quantity calculations for the preliminary report. Densities are based on 100 percent relative compaction.

Imported Borrow	125 pcf
Aggregate Subbase	130 pcf
Aggregate Base	132 pcf
Aggregate Types A and B, Asphalt Concrete	138 pcf
Aggregate OG Asphalt Concrete	120 pcf
Maximum Paving Asphalt for ATPB	1-½ percent of dry aggregate

XIII ATTACHMENTS

A. Shear Strength and Slope Stability Calculations, Plates A-1 through A-12

B. Boring Logs, Plates B-1 through B-16, B-1A and B-2A

C. Soil Survey Sheets, Plates C-1 through C-7

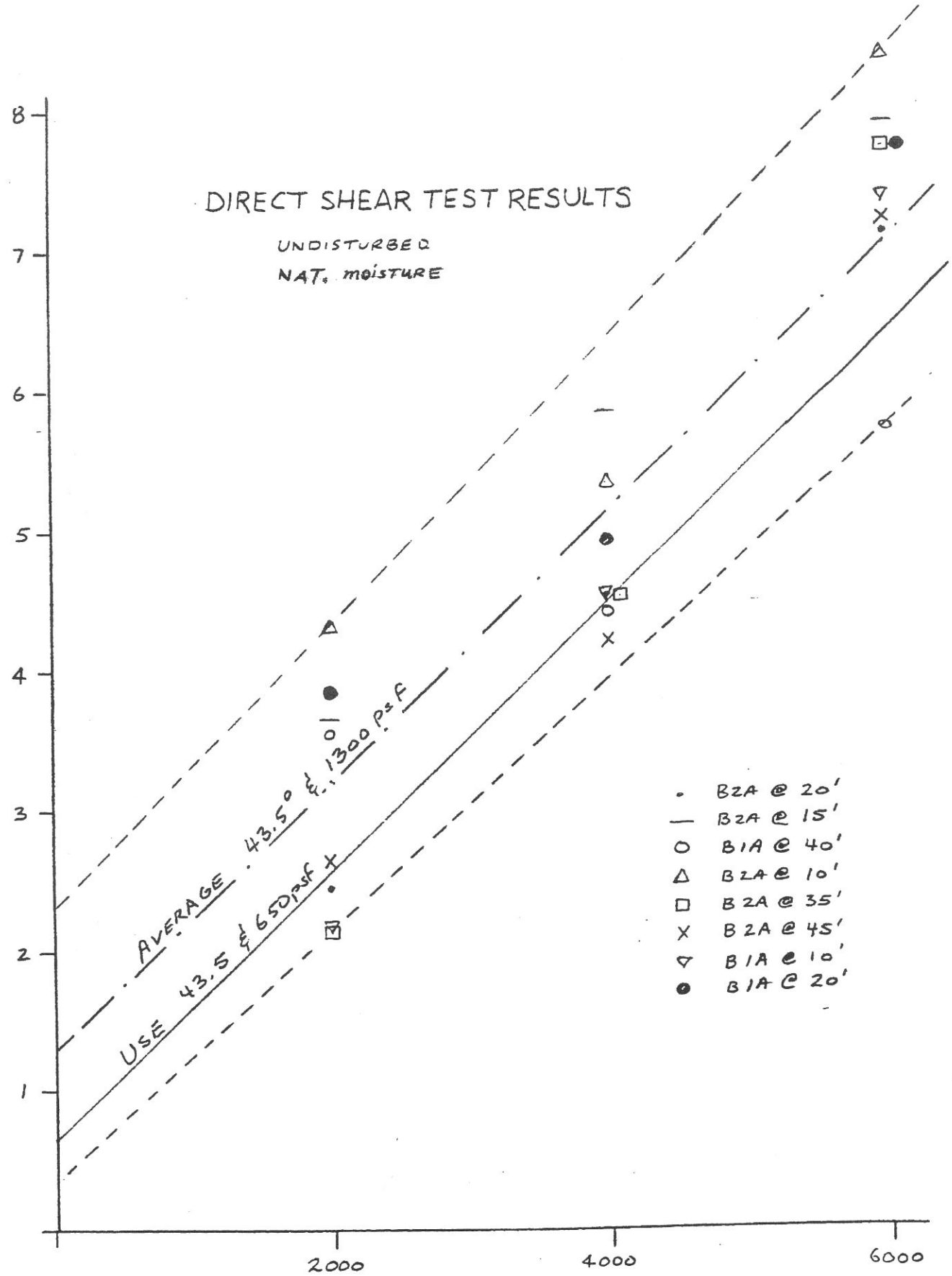
D. Map Legend, Plate D-1 and Boring Location Maps, Plates D-2 through D-21

ATTACHMENT A

Shear Strength and Slope Stability Calculation Results

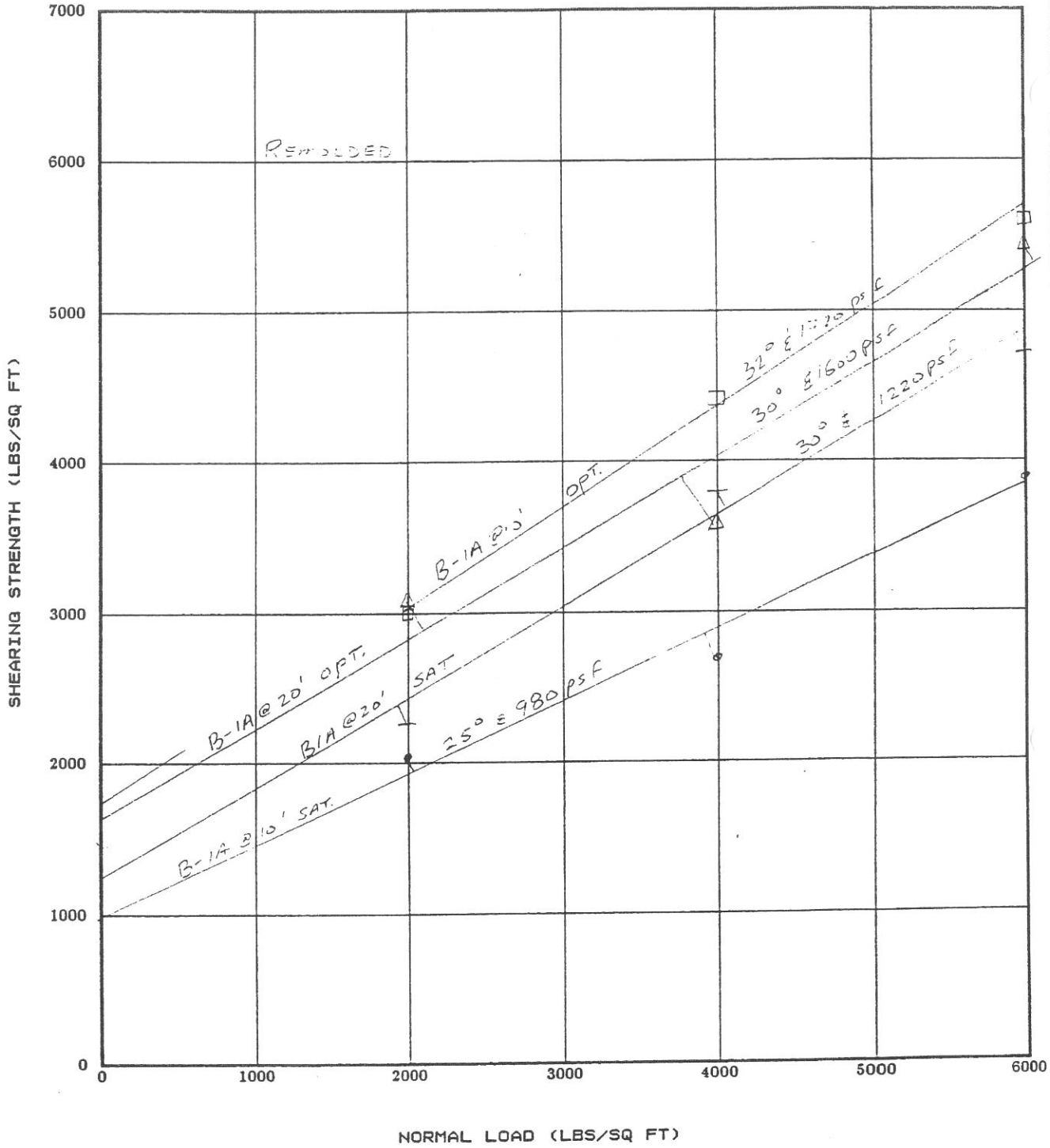
Plates A-1 through A-12

JOHN A SAYERS & ASSOCIATES



W.O. 215-1-03
May 29, 1996

PLATE A-1



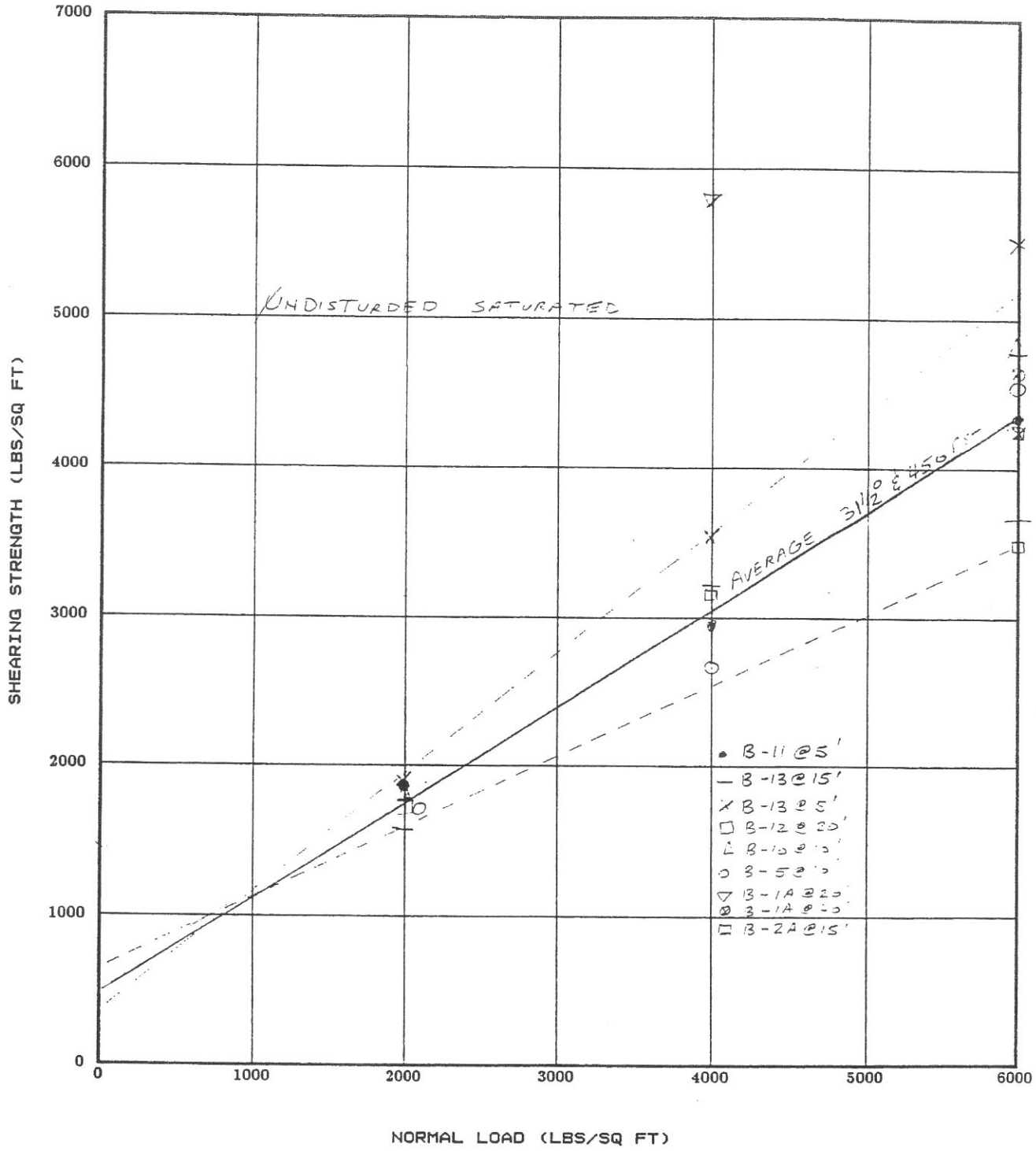
DIRECT SHEAR TEST RESULTS

W.O. 215-1-03	Project: Route 138, Phase II	Date: 5/29/96

JOHN A. SAYERS & ASSOCIATES

PLATE A-2

form: SHEARFRM (6103)



DIRECT SHEAR TEST RESULTS

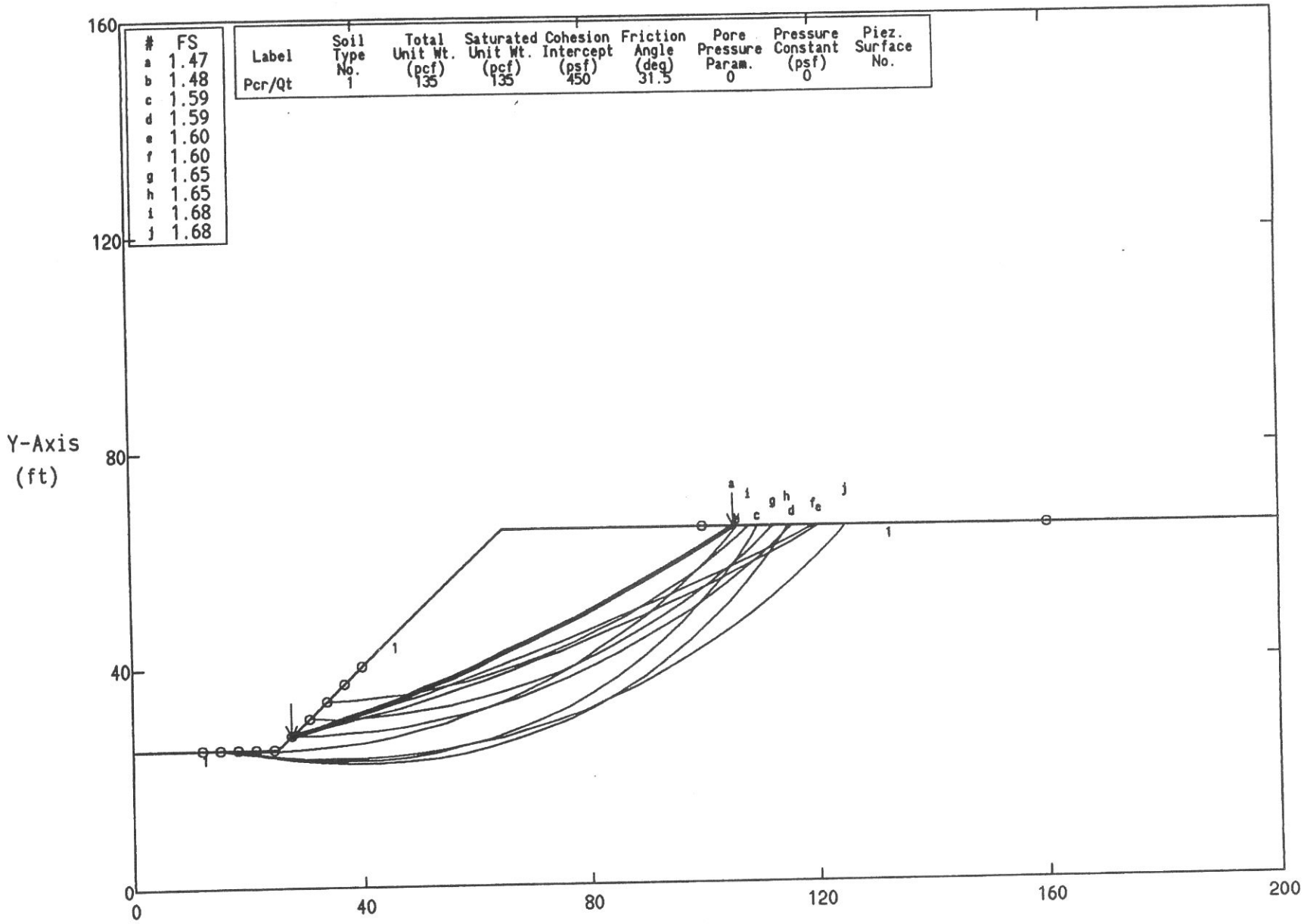
W.O. 215-1-03	Project: Route 138, Phase II	Date: 5/29/96

JOHN A. SAYERS & ASSOCIATES

form: SHE RM (6103)

WO 215 - HWY 138 - 1:1 - 40 FT CUT/SAT SEISMIC = 0.15G

Ten Most Critical. C:CUT3E.PLT By: JRM 05-29-96 5:26pm



#	FS
a	1.47
b	1.48
c	1.59
d	1.59
e	1.60
f	1.60
g	1.65
h	1.65
i	1.68
j	1.68

Label	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.
Pcr/Qt	1	135	135	450	31.5	0	0	

STABL6H FSmin=1.47 X-Axis (ft)

Factors Of Safety Calculated By The Modified Bishop Method

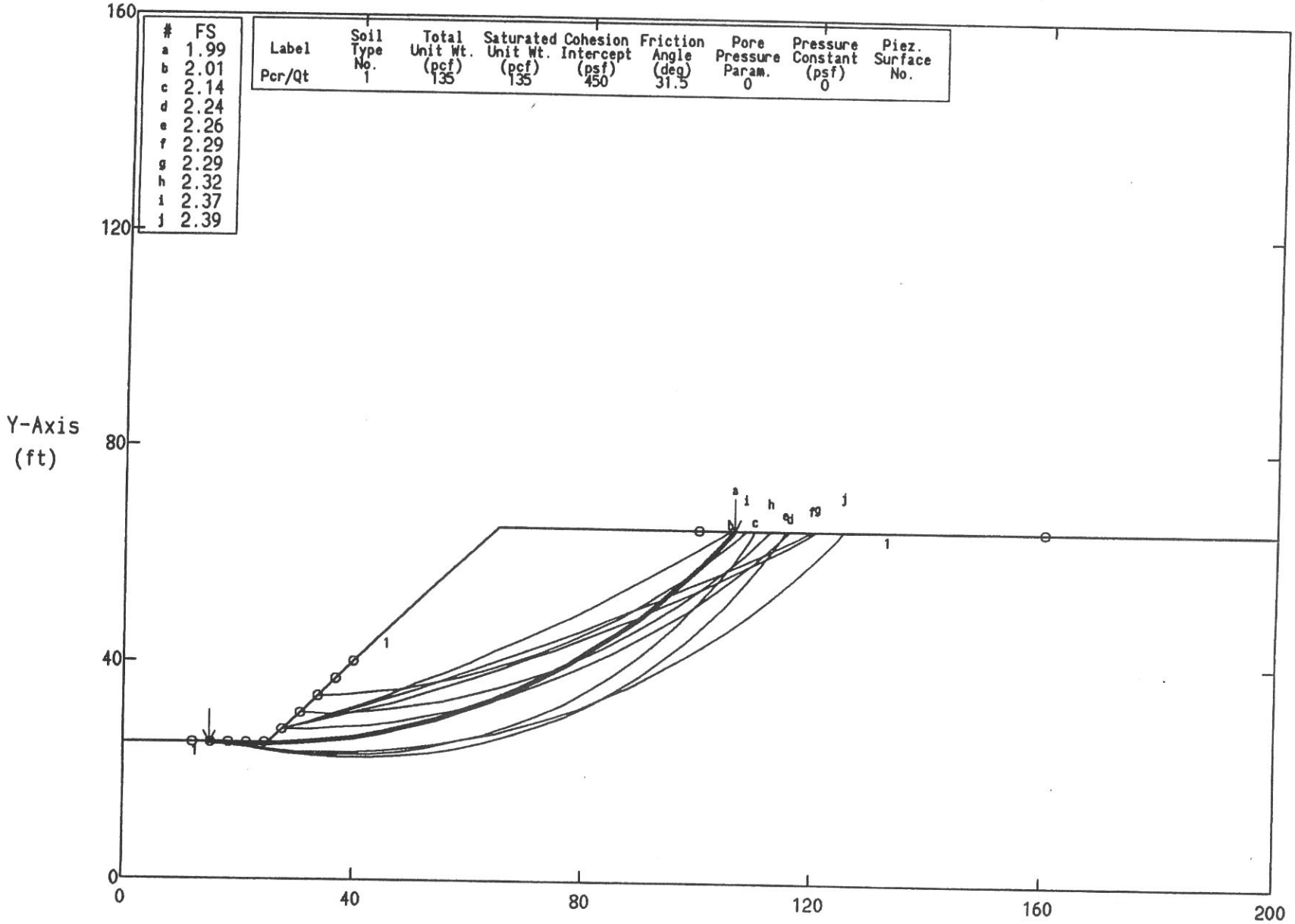
W.O. 215-1-03
May 29, 1996

PLATE A-4

W.O. 215-1-03
May 29, 1996

WO 215 - HWY 138 - 40 FT. CUT/SAT

Ten Most Critical. C:CUT3.PLT By: JRM 05-24-96 4:35pm



STABL6H FSmin=1.99 X-Axis (ft)

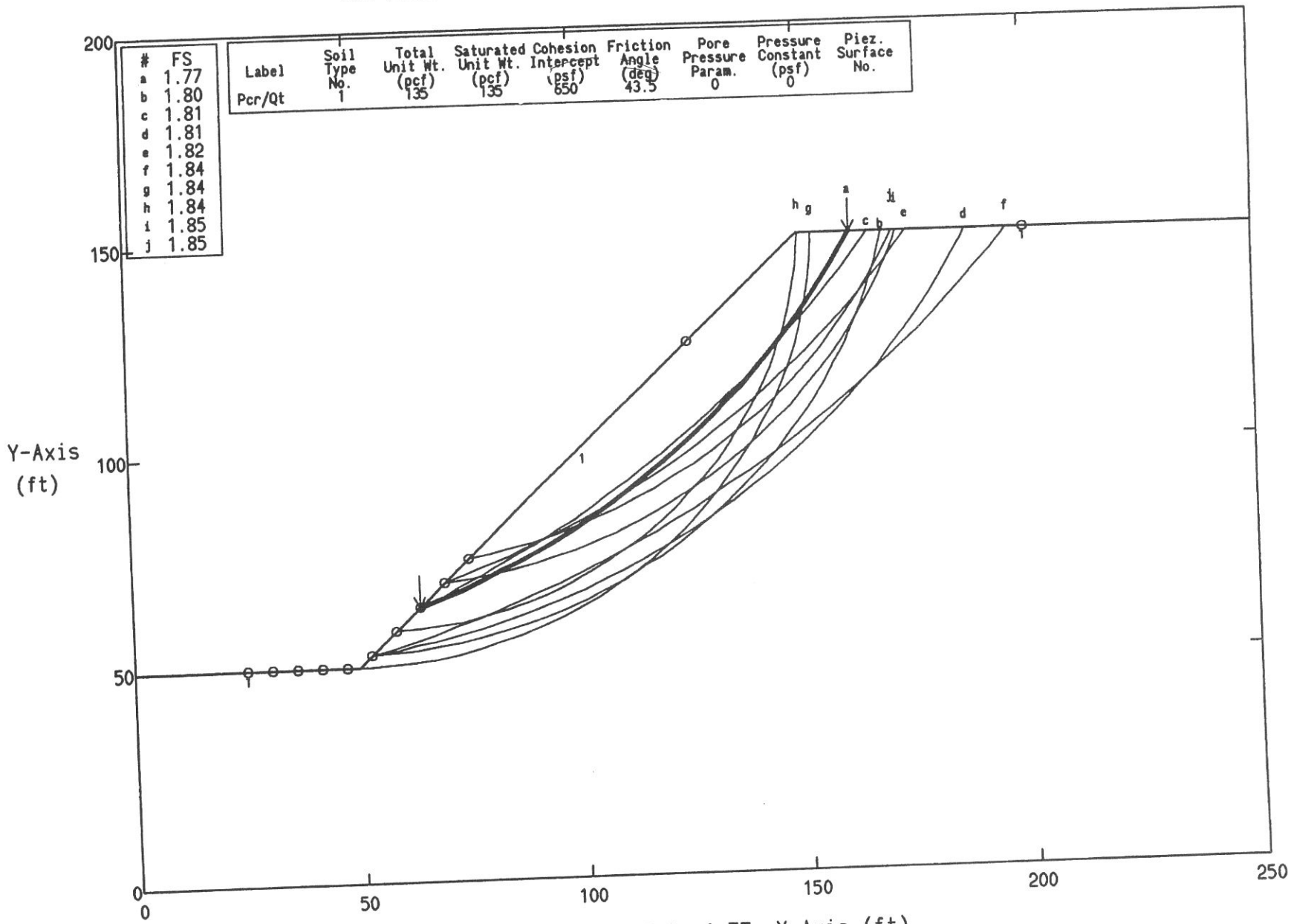
Factors Of Safety Calculated By The Modified Bishop Method

PLATE A-5

W.O. 215-1-03
May 29, 1996

WO 215 - HWY 138 - 1:1 - 100 FT, cut/nat

Ten Most Critical. C:CUT1.PLT By: JRM 05-29-96 5:09pm



STABL6H FSmin=1.77 X-Axis (ft)

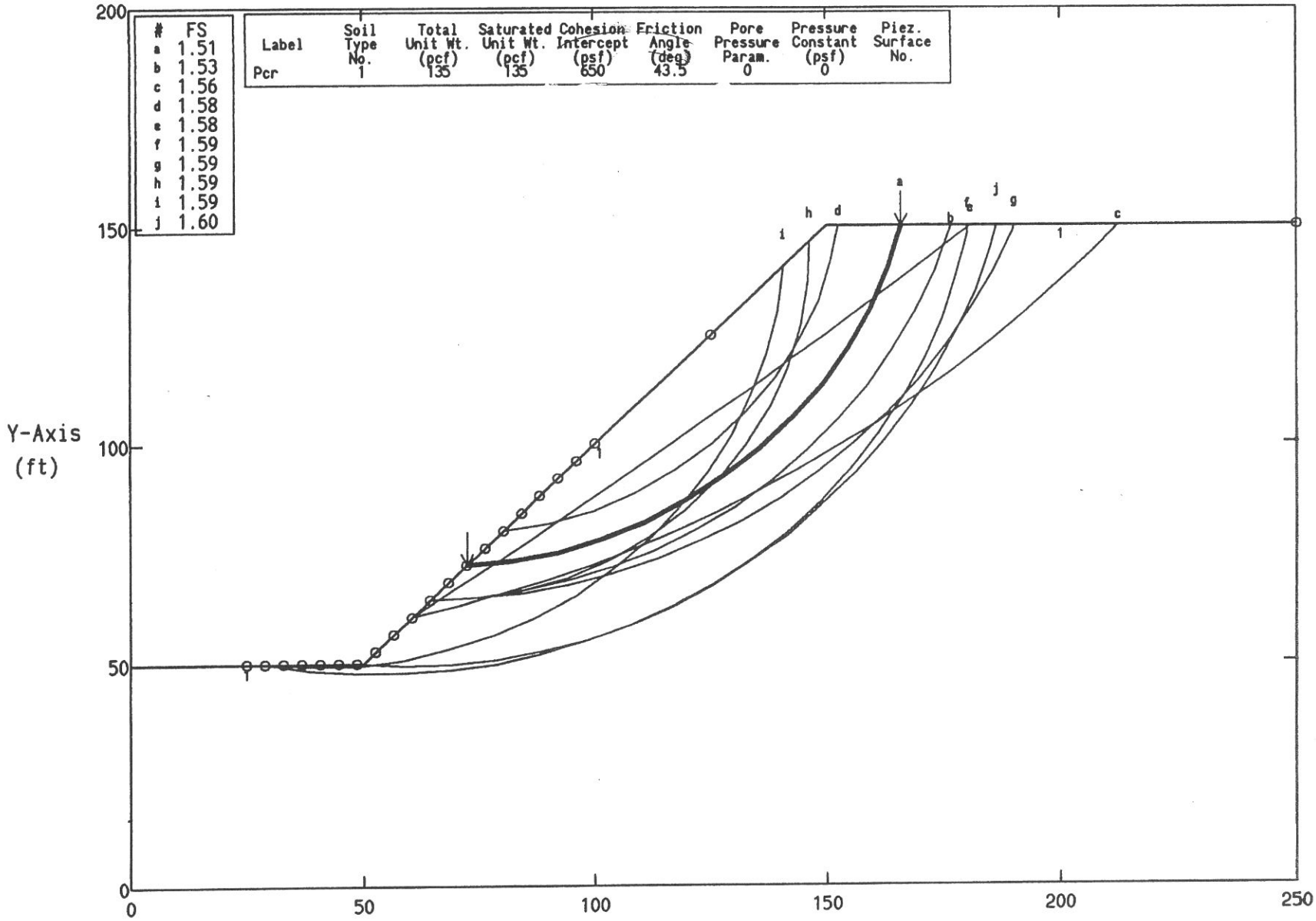
Factors Of Safety Calculated By The Modified Bishop Method

PLATE A-6

W.O. 215-1-03
May 29, 1996

WO 215 - HWY 138 - 1:1 - 100 ft SEISMIC = 0.15g

Ten Most Critical. C:CUT1E.PLT By: JRM 05-29-96 5:41pm



STABL6H FSmin=1.51 X-Axis (ft)

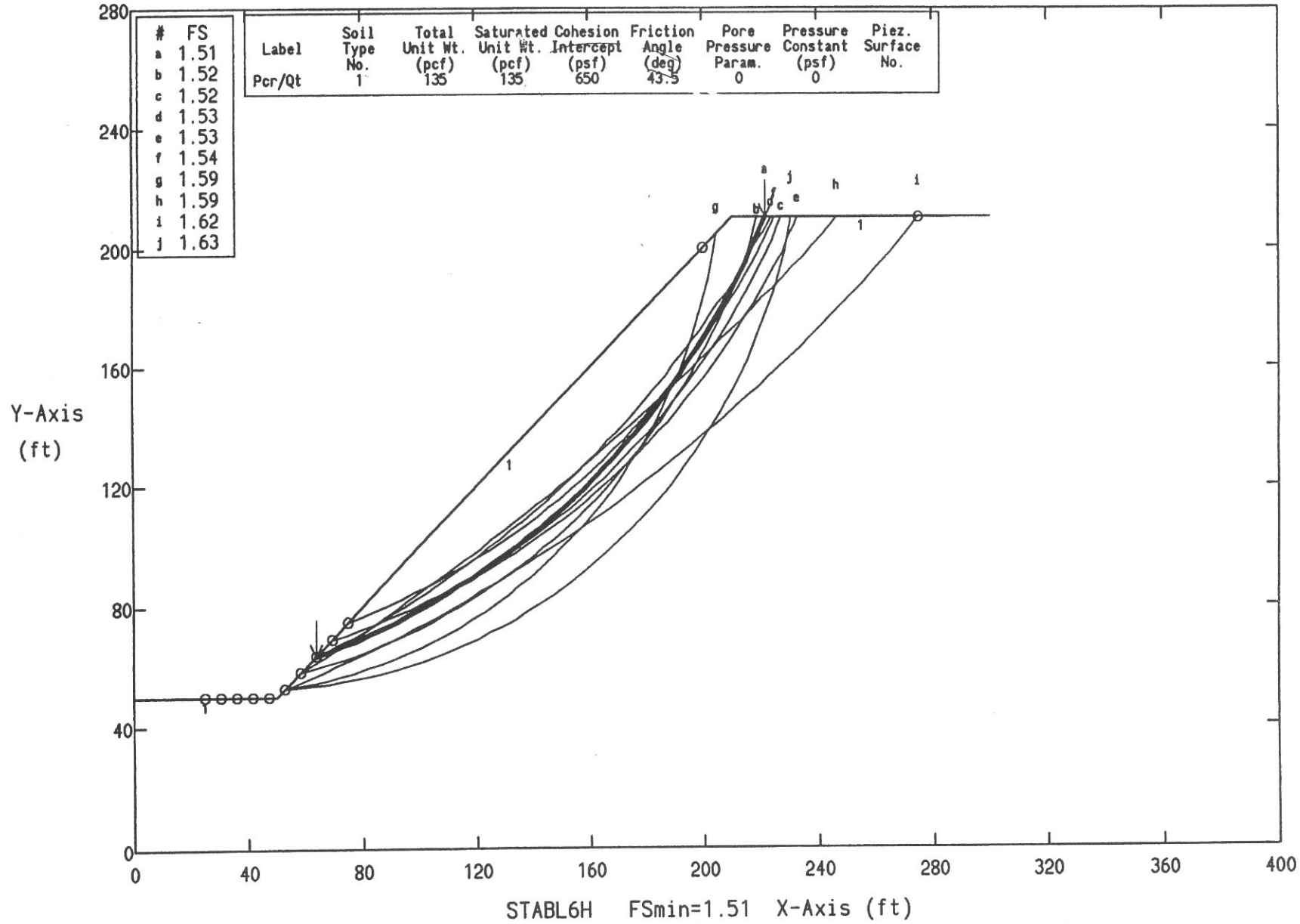
Factors Of Safety Calculated By The Modified Bishop Method

PLATE A-7

W.O. 215-1-03
May 29, 1996

WO 215 - HWY 138 - 1:1 - 160 FT, cut/nat

Ten Most Critical. C:CUT2.PLT By: JRM 05-29-96 5:14pm



STABL6H FSmin=1.51 X-Axis (ft)

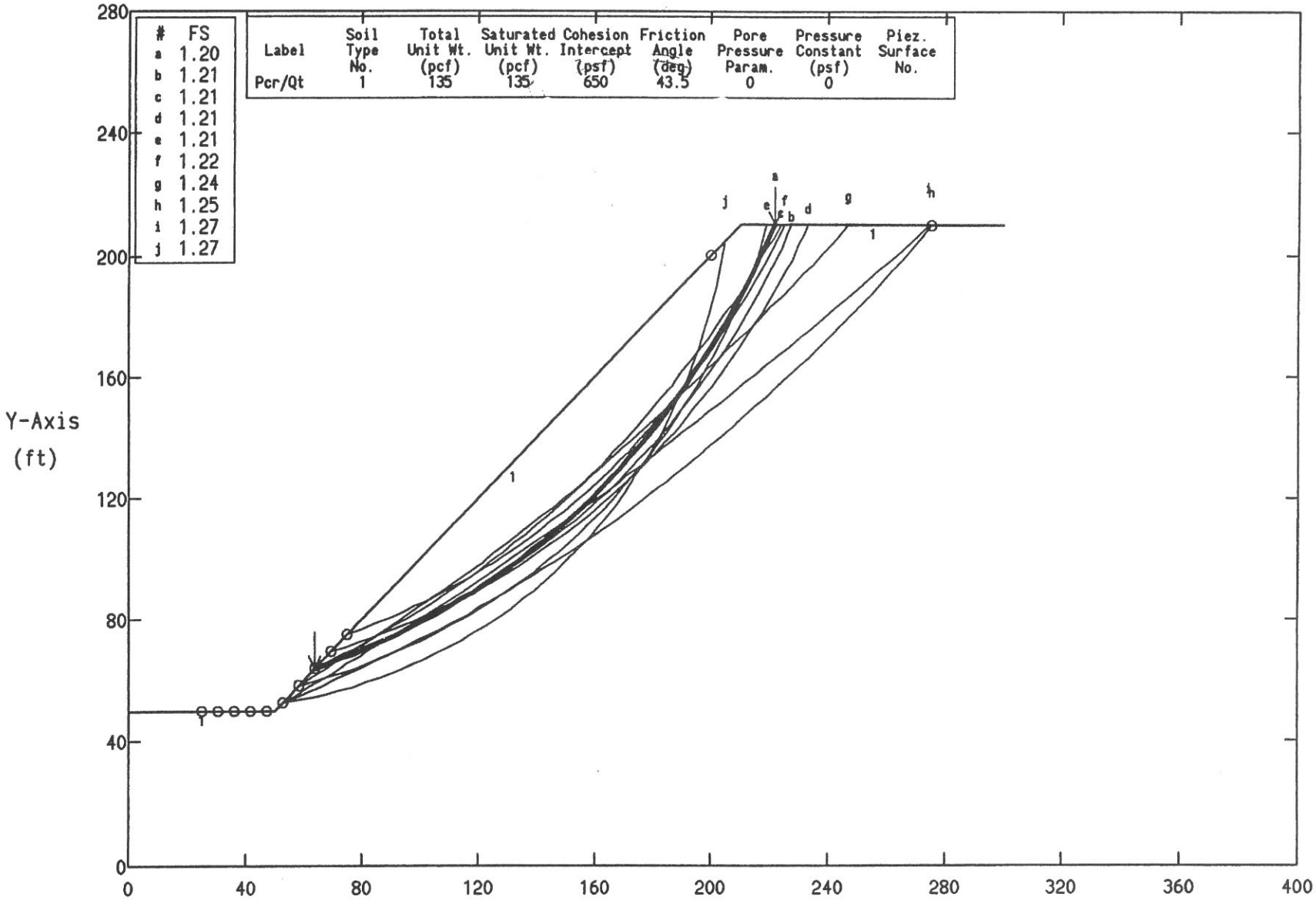
Factors Of Safety Calculated By The Modified Bishop Method

PLATE A-8

W.O. 215-1-03
May 29, 1996

WO 215 - HWY 138 - 1:1 - 160 FT CUT/NAT SEISMIC = 0.15G

Ten Most Critical. C:CUT2E.PLT By: JRM 05-29-96 5:18pm



STABL6H FSmin=1.20 X-Axis (ft)

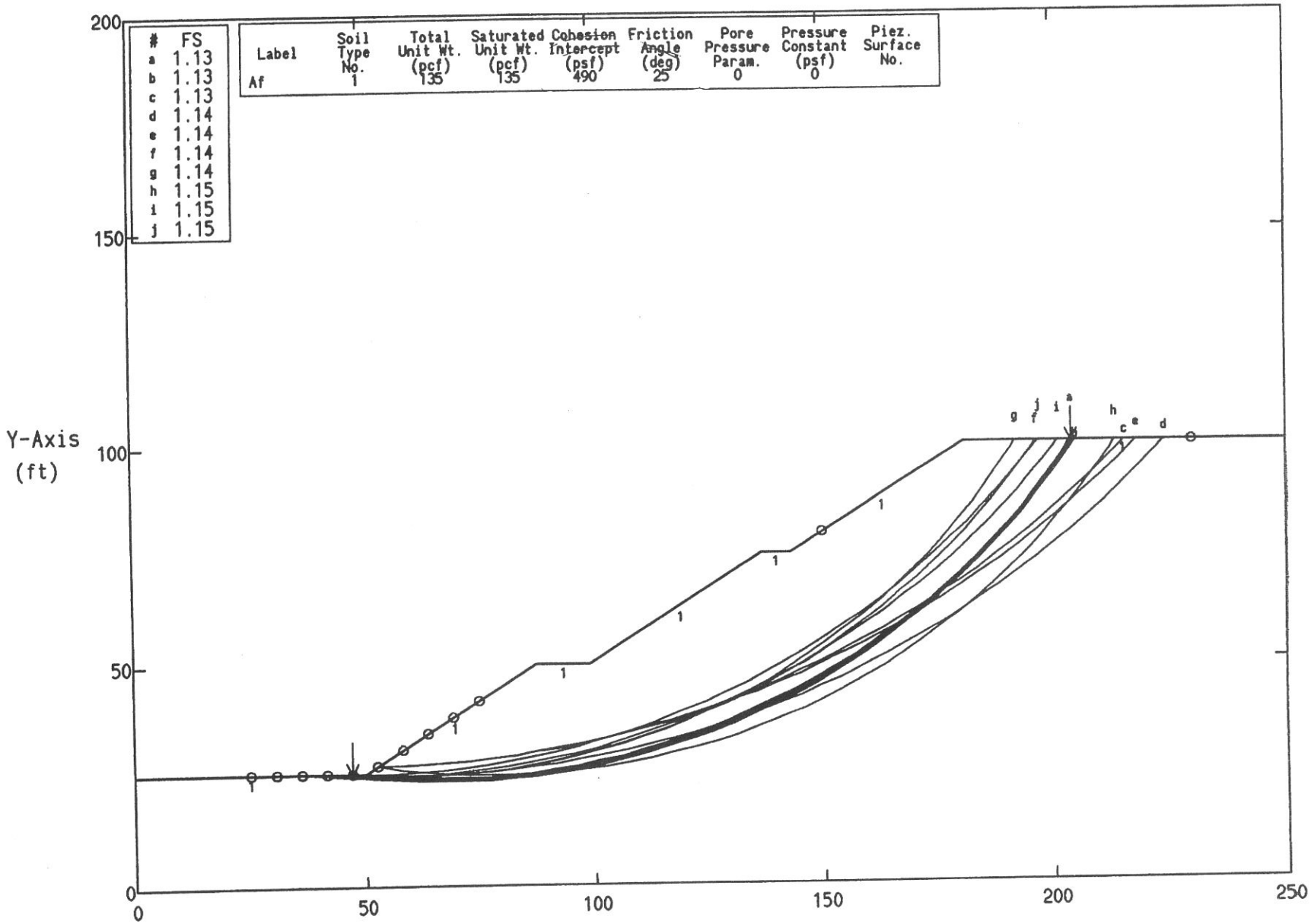
Factors Of Safety Calculated By The Modified Bishop Method

PLATE A-9

W.O. 215-1-03
May 29, 1996

HWY 138 LAS FLORES 75 FT. FILL SLOPE WITH BENCHES seismic

Ten Most Critical. C:FILL2.PLT By: JRM 05-29-96 5:06pm



STABL6H FSmin=1.13 X-Axis (ft)

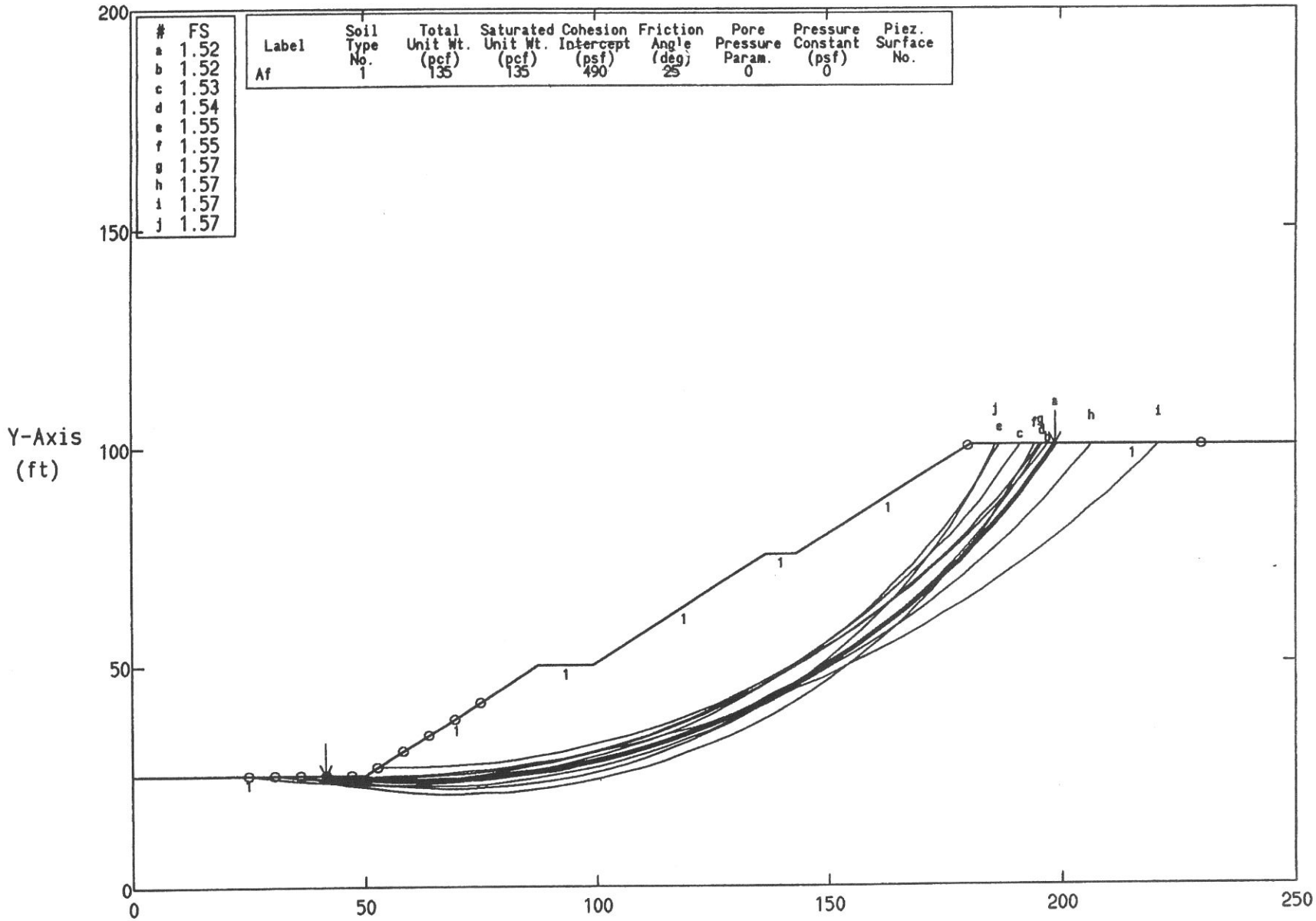
Factors Of Safety Calculated By The Modified Bishop Method

PLATE A-10

W.O. 215-1-03
May 29, 1996

HWY 138 LAS FLORES 75 FT. FILL SLOPE WITH BENCHES

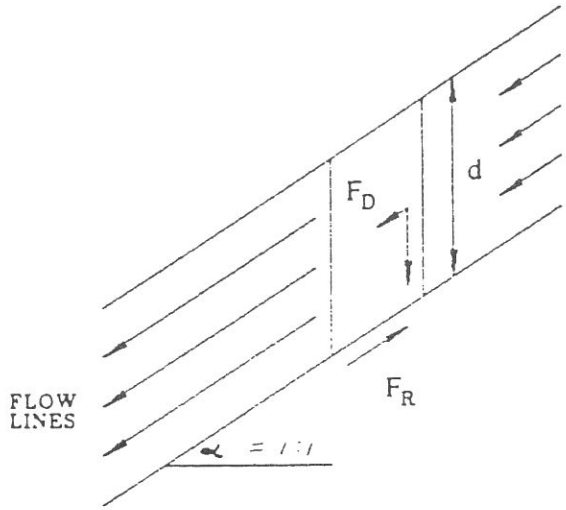
Ten Most Critical. C:FILL.PLT By: JRM 05-29-96 5:03pm



STABL6H FSmin=1.52 X-Axis (ft)

Factors Of Safety Calculated By The Modified Bishop Method

PLATE A-11



d = ASSUMED DEPTH OF SATURATION = 4 feet
γ_w = UNIT WEIGHT OF WATER = 62.4 pcf
γ_t = UNIT WEIGHT OF SOIL = 135 pcf
α = SLOPE ANGLE = 45
ϕ = ANGLE OF INTERNAL FRICTION = 43
c = COHESION = 350 psf

FORCE TENDING TO CAUSE MOVEMENT:

$F_D = d \cdot \gamma_t \cdot \cos\alpha \cdot \sin\alpha =$ 270 psf

FORCE TENDING TO RESIST MOVEMENT:

$F_R = d(\gamma_t - \gamma_w)\cos^2\alpha \cdot \tan\phi + c =$ 485 psf

FACTOR OF SAFETY:

F.S. $= F_R \div F_D =$ 1.79

SURFICIAL SLOPE STABILITY CALCULATION

W.O. 215-1-03	Project: Route 138, Phase II	Date: 5/29/96

JOHN A. SAYERS & ASSOCIATES

ATTACHMENT B

Boring Logs, Plates B-1 through B-16

and

Plates B-1A and B-2A

JOHN A SAYERS & ASSOCIATES

Legend for Boring Logs

Sample Type:

- CORE SAMPLE
- BULK SAMPLE
- CORE AND BULK SAMPLES
- STANDARD PENETRATION TEST
- SEEPAGE
- PERCHED WATER
- GROUNDWATER

Density terminology used for soil descriptions (e.g., medium dense, dense) reported in boring logs has been based on observation and engineering experience.

UNIFIED SOIL CLASSIFICATION SYSTEM

Major Divisions			Symbols	Typical Names
COARSE GRAINED SOILS (More than 50% of material is LARGER than 200 sieve size.)	GRAVELS (More than 50% of coarse fraction is LARGER than No.4 sieve.)	CLEAN GRAVELS (Little or no fines.)	GW	Well-graded gravels, gravel-sand mixtures, little or no fines.
			GP	Poorly graded gravels or gravel-sand mixtures, little or no fines.
		GRAVELS W/FINES (Appreciable amount of fines.)	GM	Silty gravels, gravel-sand-silt mixtures.
			GC	Clayey gravels, gravel-sand-clay mixtures.
	SANDS (More than 50% of the coarse fraction is SMALLER than the No.4 seive.)	CLEAN SANDS	SW	Well-graded sands, gravelly sands, little or no fines.
			SP	Poorly graded sands or gravelly sands, little or no fines.
		SANDS W/FINES (Appreciable amount of fines.)	SM	Silty sands, sand-silt mixtures.
			SC	Clayey sands, sand-clay mixtures.
FINE GRAINED SOILS (More than 50% of material is SMALLER than 200 sieve size.)	SILTS AND CLAYS (Liquid limit less than 50.)		ML	Inorganic silts & fine sands, rock flour, silty or clayey fine sands or clayey silts with slight plasticity.
			CL	Inorganic clays of low to medium plasticity, gravelly clays, sandy clays, silty clays, lean clays.
			OL	Organic silts and organic silty clays.
	SILTS AND CLAYS (Liquid limit more 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.

BOUNDARY CLASSIFICATIONS: Soils possessing characteristics of two groups are designated by combinations of group symbols.

JASA

John A. Sayers & Associates

form: B-LEGEND

Log of Exploratory Boring B-1

Client: A.R.C. Las Flores Ltd.	Date: 12-11-89	W.O. 215-01
Project: Highway 138	Location: San Bernardino	
	Surface Elevation: ±	
	Total Depth: 15.5	

FIELD					LAB DATA						
Water	Core	Bulk	Blows*	Depth (ft)	Moisture Content (%)	Dry Density (Pcf)	Other	USCS	Graphics	Depth (ft)	DESCRIPTION AND REMARKS
								SM			TOPSOIL: Silty Sand, brown, loose to medium dense, dry, with roots and fine gravel.
								SM			OLDER ALLUVIUM: Silty Sand, reddish brown, medium dense, moist, fine to medium grained, with a few fine to coarse gravel, with occasional coarse grains.
	■		3 8 9	5	1.4	113.3				5	
		■					SE=46 <200 8%				
▽	■		8 9 17		12.7	114.6	<200 8%	SM SP			Silty Sand/Sand, reddish brown, medium dense, wet to saturated, with gravel. @8.4' - Groundwater observed, brown, medium dense.
	■		3 8 10	10	10.9	125.3		SM		10	CROWDER FORMATION: Silty Sand, reddish brown, very dense, moist to wet, with some clay and gravel.
	▨		50	15			<200 20%			15	

Total Depth 15.5 feet
Groundwater at 8' 4"
Caving at 3 feet
Hole Backfilled

This log is a representation of conditions at the time & place of drilling. With the passage of time or at any other location there may be consequential changes in conditions.

See Legend for Important Notes | *Driving Weight: CME-75, Automatic Hammer System

JOHN A. SAYERS & ASSOCIATES
GEOTECHNICAL CONSULTANTS

Boring B-1
Sheet 1 of 1

Log of Exploratory Boring B-2

Client: A.R.C. Las Flores Ltd.	Date: 12-11-89	W.O. 215-01
Project: Highway 138	Location: San Bernardino	
	Surface Elevation: ±	
	Total Depth: 15.5	

FIELD					LAB DATA						
Water	Core	Bulk	Blows*	Depth (ft)	Moisture Content (%)	Dry Density (Pcf)	Other	USCS	Graphics	Depth (ft)	DESCRIPTION AND REMARKS
								SM			**TOPSOIL:** Silty Sand, brown, loose, dry.
								SM			**OLDER ALLUVIUM:** Silty Sand, brown, medium dense, dry to damp, with trace fine gravel.
	■		8 10 23		4.8	106.9	SE=37 <200 25%				
		■									
	■		5 10 11	5	1.3	121.6		SM SP		5	Silty Sand/Sand, light to medium brown, med. dense to dense, damp, with gravel.
	■		5 7 14		1.7	107.5	<200 5%				
	▨		28 50	10						10	
▽											@14' - Groundwater encountered.
	▨		7 28 29	15				SM SP		15	**CROWDER FORMATION:** Silty Sand with S Clay, reddish brown, very dense, moist wet.

This log is a representation of conditions at the time & plac drilling. With the passage of time or at any other location may be consequential changes in conditions.

Total Depth 15.5 feet
Groundwater at 14 feet
Caving at 6 feet
Hole Backfilled

See Legend for Important Notes | *Driving Weight: CME-75 Automatic Hammer System

JOHN A. SAYERS & ASSOCIATES
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Boring B-
Sheet 1 of 1

Log of Exploratory Boring B-3

Client: A.R.C. Las Flores Ltd.	Date: 12-11-89	W.O. 215-01
Project: Highway 138	Location: San Bernardino	
	Surface Elevation: ±	
	Total Depth: 16.5	

FIELD					LAB DATA						
Water	Core	Bulk	Blows*	Depth (ft)	Moisture Content (%)	Dry Density (Pcf)	Other	USCS	Graphics	Depth (ft)	DESCRIPTION AND REMARKS
								GW			**ALLUVIUM:** Sandy Gravel, light to medium brown, dry to damp, loose to dense, cobbles and boulders at surface.
	■		20 22 23								
								SM			**OLDER ALLUVIUM:** Silty Sand, reddish to light brown and gray, medium dense to dense, dry to slightly damp.
	■		10 22 23	5	3.2	106.2				5	
	▨		5 10 15								
	■		44 40 43	10	1.4	106.2				10	
	▨		11 20 25	15						15	

This log is a representation of conditions at the time & place of drilling. With the passage of time or at any other location there may be consequential changes in conditions.

Total Depth 16.5 feet
No Groundwater
No Caving
Hole Backfilled

See Legend for Important Notes | *Driving Weight:

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Boring B-3
Sheet 1 of 1

Log of Exploratory Boring B-4

Client: A.R.C. Las Flores Ltd.	Date: 12-11-89	W.O. 215-01
Project: Highway 138	Location: San Bernardino	
	Surface Elevation: ±	
	Total Depth: 25.5	

FIELD					LAB DATA					
Water	Core	Bulk	Blows*	Depth (ft)	Moisture Content (%)	Dry Density (Pcf)	Other	USCS	Graphics	Depth (ft)
			7 17 44		2.6	126.9		SM		
			10 11 11	5	2.0					5
			5 8 18					SP SM		
			22 29 40	10	1.0	114.9				10
								SM		
			15 27 28	15						15
			21 33 36	20	3.0	121.8				20
			50	25	2.8					25

DESCRIPTION AND REMARKS

This log is a representation of conditions at the time & pla drilling. With the passage of time or at any other location may be consequential changes in conditions.

ROADFILL: Silty Sand with Gravel, brown, medium dense to dense, dry to damp, n roots, gravel and cobbles encountered a feet.

WASH DEPOSITS: Sand/Silty Sand, light brown, dense, dry to damp, some grave

OLDER ALLUVIUM: Silty Sand with Grave light brown, very dense, dry to damp.

Total Depth 25.5 feet
Caving at 8 to 11 feet
No Groundwater
Hole Backfilled

See Legend for Important Notes | *Driving Weight:

JOHN A. SAYERS & ASSOCIATES
GEOTECHNICAL CONSULTANTS

Boring B-
Sheet 1 of 1

Log of Exploratory Boring B-5

Client: A.R.C. Las Flores Ltd.	Date: 12-11-89	W.O. 215-01
Project: Highway 138	Location: San Bernardino	
	Surface Elevation: ±	
	Total Depth: 16.5	

FIELD					LAB DATA						
Water	Core	Bulk	Blows*	Depth (ft)	Moisture Content (%)	Dry Density (Pcf)	Other	USCS	Graphics	Depth (ft)	DESCRIPTION AND REMARKS
								SM SP			TOPSOIL: 2" to 3" thick **OLDER ALLUVIUM:** Silty Sand/Sand, brown, loose to medium dense, dry.
	■		7 8 8		0.8	104.3					
	■		5 7 8	5	2.0	107.8				5	@5' - dry to damp, dense.
	■		10 19 19		2.9	110.6		SM			Silty Sand, brown, medium dense, damp, with gravel, dry to damp.
	■		9 16 18	10	2.9	113.7				10	
	▨		7 11 12	15						15	

This log is a representation of conditions at the time & place of drilling. With the passage of time or at any other location there may be consequential changes in conditions.

Total Depth 16.5 feet
No Groundwater
No Caving
Hole Backfilled

See Legend for Important Notes | *Driving Weight:

JOHN A. SAYERS & ASSOCIATES
GEOTECHNICAL CONSULTANTS

Boring B-5
Sheet 1 of 1

Log of Exploratory Boring B-6

Client: A.R.C. Las Flores Ltd. | Date: 12-11-89 | W.O. 215-01

Project: Highway 138 | Location: San Bernardino

Surface Elevation: ±

Total Depth: 15.0

FIELD					LAB DATA						
Water	Core	Bulk	Blows*	Depth (ft)	Moisture Content (%)	Dry Density (Pcf)	Other	USCS	Graphics	Depth (ft)	DESCRIPTION AND REMARKS
								SM			TOPSOIL: Silty Sand, medium brown, loose, dry.
								SP SM			OLDER ALLUVIUM: Sand/Silty Sand, brown, medium dense, damp, with gravel.
	■		10 12 14		1.3	106.1					
	■		10 18 25	5	1.3	117.9				5	@5' - dense
	■		17 19 19		1.1	115.1					
	▨		14 22 24	10				SM		10	Silty Sand, brown, dense, damp to moist, with gravel.
	▨		10 18 26	15						15	Total Depth 15 feet No Groundwater Hole Backfilled

This log is a representation of conditions at the time & place of drilling. With the passage of time or at any other location there may be consequential changes in conditions.

See Legend for Important Notes | *Driving Weight:

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Boring B-6

Sheet 1 of 1

Log of Exploratory Boring B-7

Client: A.R.C. Las Flores Ltd.	Date: 12-11-89	W.O. 215-01
Project: Highway 138	Location: San Bernardino	
	Surface Elevation: ±	
	Total Depth: 15.0	

FIELD					LAB DATA						
Water	Core	Bulk	Blows*	Depth (ft)	Moisture Content (%)	Dry Density (Pcf)	Other	USCS	Graphics	Depth (ft)	DESCRIPTION AND REMARKS
								SM			WASH DEPOSIT: Silty Sand with Gravel, brown, loose, dry.
	■		64 87		5.0	129.2		SC			CROWDER FORMATION (WEATHERED): Clayey Sand, brown, damp, dense.
	■		15 39 67	5	11.3	123.5				5	@5' - less weathered, although still clayey.
	■		41 50/3"	10	8.0	124.3		SC		10	Silty Sand with Clay, some gravel, brown, very dense, moist.
	▨		18 33 51	15	13.4					15	Total Depth 15 feet No Groundwater No Caving Hole Backfilled

This log is a representation of conditions at the time & place of drilling. With the passage of time or at any other location there may be consequential changes in conditions.

See Legend for Important Notes | *Driving Weight:

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Boring B-7
Sheet 1 of 1

Log of Exploratory Boring B-8

Client: A.R.C. Las Flores Ltd.	Date:	W.O. 215-01
Project: Highway 138	Location: San Bernardino	
	Surface Elevation: ±	
	Total Depth: 1.0	

FIELD					LAB DATA						
Water	Core	Bulk	Blows*	Depth (ft)	Moisture Content (%)	Dry Density (Pcf)	Other	USCS	Graphics	Depth (ft)	DESCRIPTION AND REMARKS
											This log is a representation of conditions at the time & place of drilling. With the passage of time or at any other location there may be consequential changes in conditions.
											This boring was deleted due to lack of access.

See Legend for Important Notes | *Driving Weight:

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Boring B-8
Sheet 1 of 1

Log of Exploratory Boring B-9

Client: A.R.C. Las Flores Ltd.	Date: 12-11-89	W.O. 215-01
Project: Highway 138	Location: San Bernardino	
	Surface Elevation: ±	
	Total Depth: 21.0	

FIELD					LAB DATA							
Water	Core	Bulk	Blows*	Depth (ft)	Moisture Content (%)	Dry Density (Pcf)	Other	USCS	Graphics	Depth (ft)	DESCRIPTION AND REMARKS	
								SM SP			**WASH DEPOSITS:** Sand/Silty Sand, light brown, loose, dry.	
	■		12 21 27		1.0	116.4	<200 6%				@2.5' - becomes medium brown, medium dense, fine to medium grained.	
	■		20 20 23	5	1.5	110.8				5	@5' - some gravel, dry to damp.	
				10						10		
	■		72/6"	15	4.3	120.1	<200 40%	SM SC		15	**OLDER ALLUVIUM (?):** Silty to Clayey Sand, some gravel and cobbles, brown, very dense, damp.	
	■		50/4"	20						20		
	▨		50/3"								Total Depth 21 feet No Groundwater Hole Backfilled	

This log is a representation of conditions at the time & place of drilling. With the passage of time or at any other location there may be consequential changes in conditions.

'ee Legend for Important Notes | *Driving Weight: 140 lbs/automatic

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Boring B-9
Sheet 1 of 1

Log of Exploratory Boring B-10

Client: A.R.C. Las Flores Ltd.	Date: 12-12-89	W.O. 215-01
Project: Highway 138	Location: San Bernardino	
	Surface Elevation: ±	
	Total Depth: 15.0	

FIELD					LAB DATA						
Water	Core	Bulk	Blows*	Depth (ft)	Moisture Content (%)	Dry Density (Pcf)	Other	USCS	Graphics	Depth (ft)	DESCRIPTION AND REMARKS
								SP SM			WASH DEPOSITS: Sand/Silty Sand, light brown, loose, dry to slightly damp.
	■		2 5 32		2.2	120.5	<200 30%				@2.5' - some gravel and clay, loose to medium dense.
	■		36 58 59	5	1.6	121.3				5	
								SM SC			TERRACE DEPOSITS (?): Silty to Clayey Sand, medium to light brown, dense to ve dense, damp.
	■		50 104	10	4.6	126.0	<200 49%			10	
	▨		15 30 42	15						15	Total Depth 15 feet No Groundwater Hole Backfilled

This log is a representation of conditions at the time & place drilling. With the passage of time or at any other location th may be consequential changes in conditions.

See Legend for Important Notes | *Driving Weight: 140 lbs/automatic

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Boring B-1(
Sheet 1 of 1

Log of Exploratory Boring B-11

Client: A.R.C. Las Flores Ltd.	Date: 12-12-89	W.O. 215-01
Project: Highway 138	Location: San Bernardino	
	Surface Elevation: ±	
	Total Depth: 10.0	

FIELD					LAB DATA						
Water	Core	Bulk	Blows*	Depth (ft)	Moisture Content (%)	Dry Density (Pcf)	Other	USCS	Graphics	Depth (ft)	DESCRIPTION AND REMARKS
								SC			FILL: Clayey Sand, brown, medium dense, dry.
								SM			CROWDER FORMATION (?): Silty Sand with Some Gravel, light brown, very dense, damp.
	■		35 60		2.7	120.6					
	■		33 30/4"	5	6.4	117.9	<200 53%	SM ML		5	@5' - fine grained, very silty
	▨		33 42 50/5"	10						10	@10' - Silty Fine Sand/Fine Sandy Silt, brown, very dense, damp, some clay.

This log is a representation of conditions at the time & place of drilling. With the passage of time or at any other location there may be consequential changes in conditions.

Total Depth 10 feet
No Groundwater
Hole Backfilled

See Legend for Important Notes | *Driving Weight: 140 lbs/automatic

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Boring B-11
Sheet 1 of 1

Log of Exploratory Boring B-12

Client: A.R.C. Las Flores Ltd.	Date: 12-12-89	W.O. 215-01
Project: Highway 138	Location: San Bernardino	
	Surface Elevation: ±	
	Total Depth: 20.0	

FIELD					LAB DATA						
Water	Core	Bulk	Blows*	Depth (ft)	Moisture Content (%)	Dry Density (Pcf)	Other	USCS	Graphics	Depth (ft)	DESCRIPTION AND REMARKS
								SM			This log is a representation of conditions at the time & place c drilling. With the passage of time or at any other location the may be consequential changes in conditions.
											FILL/WASH DEPOSITS: Silty Sand with Clay and Gravel, brown, loose to medium dense dry to damp.
	■		12 11 14		3.7	104.7	<200 12%				
				5						5	
	■		7 12 18		1.9	109.0					
				10						10	@10' - becoming clayey
	■		8 11 12		7.9	113.1					
								SM ML			CROWDER FORMATIOM (?): Silty Sand/Sandy Silt with clay, light brown, very dense, damp, some gravel.
	▩		30 51	15			<200 51%			15	
	■		65/6"	20	5.4	120.3				20	Total Depth 20 feet No Groundwater Hole Backfilled

See Legend for Important Notes | *Driving Weight: 140 lbs./automatic

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Boring B-12
Sheet 1 of 1

Log of Exploratory Boring B-13

Client: A.R.C. Las Flores Ltd.	Date: 12-12-89	W.O. 215-01
Project: Highway 138	Location: San Bernardino	
	Surface Elevation: ±	
	Total Depth: 15.0	

FIELD					LAB DATA						
Water	Core	Bulk	Blows*	Depth (ft)	Moisture Content (%)	Dry Density (Pcf)	Other	USCS	Graphics	Depth (ft)	DESCRIPTION AND REMARKS
								SM SC			FILL: Silty to Clayey Sand with Gravel, brown, loose to medium dense, dry to damp.
	■		57 132	5	3.1	121.9	<200 32%	SM		5	CROWDER FORMATION: Silty Sand with Clay, light brown, very dense, damp, some small gravel.
	▨		40 38 50/5"	10						10	
							<200 27%				
	■		60/6"	15						15	Total Depth 15 feet No Groundwater Hole Backfilled

This log is a representation of conditions at the time & place of drilling. With the passage of time or at any other location there may be consequential changes in conditions.

See Legend for Important Notes | *Driving Weight: 140 lbs/automatic

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Boring B-13
Sheet 1 of 1

Log of Exploratory Boring B-14

Client: A.R.C. Las Flores Ltd.	Date: 12-12-89	W.O. 215-01
Project: Highway 138	Location: San Bernardino	
	Surface Elevation: ±	
	Total Depth: 10.0	

FIELD					LAB DATA						
Water	Core	Bulk	Blows*	Depth (ft)	Moisture Content (%)	Dry Density (Pcf)	Other	USCS	Graphics	Depth (ft)	DESCRIPTION AND REMARKS
								SM			FILL: Silty Sand with Gravel, reddish brown, medium dense, dry to damp.
								SP SM			TERRACE DEPOSIT: Sand/Silty Sand with Gravel, light reddish brown, very dense, damp.
	■		34 62		3.9	122.2	<200 4%				
	■		70/6	5	4.1	114.2				5	
	▨		28 58	10						10	Total Depth 10 feet No Groundwater Hole Backfilled

This log is a representation of conditions at the time & place of drilling. With the passage of time or at any other location there may be consequential changes in conditions.

See Legend for Important Notes | *Driving Weight: 140 lbs/automatic

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Boring B-14
Sheet 1 of 1

Log of Exploratory Boring B-15

Client: A.R.C. Las Flores Ltd.	Date: 12-12-89	W.O. 215-01
Project: Highway 138	Location: San Bernardino	
	Surface Elevation: ±	
	Total Depth: 35.0	

FIELD					LAB DATA					
Water	Core	Bulk	Blows*	Depth (ft)	Moisture Content (%)	Dry Density (Pcf)	Other	USCS	Graphics	Depth (ft)
								SM SC		
	■		23 27 27		6.0	120.3	<200 30%			
	■		6 8 6	5	7.2	109.5				5
	■		3 4 4	10	8.2	106.3	<200 43%			10
	■		4 5 7	15	5.5	110.3				15
	▨		2 3 4	20						20

DESCRIPTION AND REMARKS

This log is a representation of conditions at the time & place of drilling. With the passage of time or at any other location there may be consequential changes in conditions.

ALLUVIUM: Silty/Clayey Sand, dark to medium brown, loose to medium dense, damp, scattered gravel.

See Legend for Important Notes | *Driving Weight: 140 lbs/automatic

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Boring B-15
Sheet 1 of 2

Log of Exploratory Boring B-15

Client: A.R.C. Las Flores Ltd.	Date: 12-12-89	W.O. 215-01
Project: Highway 138	Location: San Bernardino	
	Surface Elevation: ±	
	Total Depth: 35.0	

FIELD					LAB DATA							DESCRIPTION AND REMARKS
Water	Core	Bulk	Blows*	Depth (ft)	Moisture Content (%)	Dry Density (Pcf)	Other	USCS	Graphics	Depth (ft)		This log is a representation of conditions at the time & place drilling. With the passage of time or at any other location th may be consequential changes in conditions.
			6 7 10		16.0	101.1						Alluvium continues, silty/clayey sand, medium dense, damp.
			3 2 5	30						30		
			8 28 30	35	3.7	115.2				35		Total Depth 35 feet No Groundwater Hole Backfilled

See Legend for Important Notes | *Driving Weight: 140 lbs/automatic

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Boring B-15
Sheet 2 of 2

Log of Exploratory Boring B-16

Client: A.R.C. Las Flores Ltd.	Date: 12-12-89	W.O. 215-01
Project: Highway 138	Location: San Bernardino	
	Surface Elevation: ±	
	Total Depth: 14.5	

FIELD					LAB DATA						
Water	Core	Bulk	Blows*	Depth (ft)	Moisture Content (%)	Dry Density (Pcf)	Other	USCS	Graphics	Depth (ft)	DESCRIPTION AND REMARKS
								ML			ALLUVIUM: Silty Sand, medium brown, damp, loose to medium dense, some gravel. portions slightly clayey.
							<200 19%				
	■		3 4 4		7.3	105.2					
				5						5	
	■		3 3 5		5.7	99.3					
				10						10	
	■		3 3 5		9.6	109.9					

This log is a representation of conditions at the time & place of drilling. With the passage of time or at any other location there may be consequential changes in conditions.

Total Depth 14.5 feet
No Groundwater
Hole Backfilled

See Legend for Important Notes | *Driving Weight: 140 lbs/automatic

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Boring B-16
Sheet 1 of 1

Log of Exploratory Boring B-1A

JASA

Client: **Rancho Las Flores Limited Partnership**	Date: **5-15-96**	W.O. **215-1**
Project: **Highway 138**	Location: **San Bernardino County**	
	Surface Elevation: ~	
	Total Depth: ~ **47**	

FIELD					LAB DATA						
Water	Core	Bulk	Blows*	Depth (ft)	Moisture Content (%)	Dry Density(pcf)	Relative Compaction	USCS	Graphics	Depth (ft)	DESCRIPTION AND REMARKS
											This log is a representation of conditions at the time & plac drilling. With the passage of time or at any other location may be consequential changes in conditions.
											CROWDER FORMATION: Sandstone fine to medi grained, scattered pebbles, very light brown, hard, slightly weathered, damp
	■	■	6/6"	5	8.0	122.5				5	Slightly cemented, very hard
	■	■	8/6" 3/1"	10	8.1	126.1				10	
				15						15	
	■	■	5/6" 9/6"	20	10.4	127.4				20	Fine-grained with silt, very hard

See Legend for Important Notes

*Driving Weight: **3350 to 29' - 2040 to 58'**

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GEOTECHNICAL CONSULTANTS

Boring B-1A

Sheet 1 of 2

Log of Exploratory Boring B-1A

JASA

Client: **Rancho Las Flores Limited Partnership**	Date: **5-15-96**	W.O. **215-1**
Project: **Highway 138**	Location: **San Bernardino County**	
	Surface Elevation: ~	
	Total Depth: ~ **47**	

FIELD					LAB DATA						
Water	Core	Bulk	Blows*	Depth (ft)	Moisture Content (%)	Dry Density(pcf)	Relative Compaction	USCS	Graphics	Depth (ft)	*DESCRIPTION AND REMARKS*
											This log is a representation of conditions at the time & place of drilling. With the passage of time or at any other location there may be consequential changes in conditions.
											Crowder Formation continued
	■		8/6" 12/1"	30	6.2	128.7				30	Scattered pebbles, very hard
				35						35	
	■		18/6" 7/1"	40	4.7	124.5				40	Scattered pebbles and gravel, very hard
				45						45	
											@ 47 ' - encountered large cobble or small boulder, could not get past with bucket, discontinued drilling
											Total depth 47 feet. No water. No caving.

See Legend for Important Notes

*Driving Weight: **3350 to 29' - 2040 to 58'**

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Boring B-1A

Sheet 2 of 2

Log of Exploratory Boring B-2A

JASA

Client: **Rancho Las Flores Limited Partnership**	Date: **5-15-96**	W.O. **215-1**
Project: **Highway 138**	Location: **San Bernardino County**	
		Surface Elevation: ~
		Total Depth: ~4

FIELD					LAB DATA						
Water	Core	Bulk	Blows*	Depth (ft)	Moisture Content (%)	Dry Density(pcf)	Relative Compaction	USCS	Graphics	Depth (ft)	DESCRIPTION AND REMARKS
											This log is a representation of conditions at the time drilling. With the passage of time or at any other loc may be consequential changes in conditions.
											CROWDER FORMATION: Sandstone fine to grained, scattered pebbles, very light bro hard, slightly weathered, damp
	■	■	5/6" 9/1"	5	8.4	131.5				5	Very hard
	■	■	M	10	12.0	127.5				10	Silty Fine to Medium Sand, very hard
	■		8/6" 7/2"	15	8.5	128.0				15	Silty with Trace Clay, very hard
	■		15/6"	20	5.3	118.5				20	Gravelly Fine t6o Medium Sand, scattered gra continue below, very hard

See Legend for Important Notes

*Driving Weight: **3350 to 29' - 2040 to 58'**

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Boring B

Sheet 1 of 2

Log of Exploratory Boring B-2A

JASA

Client: **Rancho Las Flores Limited Partnership**	Date: **5-15-96**	W.O. **215-1**
Project: **Highway 138**	Location: **San Bernardino County**	
	Surface Elevation: ~	
	Total Depth: ~ **45**	

FIELD					LAB DATA						
Water	Core	Bulk	Blows*	Depth (ft)	Moisture Content (%)	Dry Density(pcf)	Relative Compaction	USCS	Graphics	Depth (ft)	DESCRIPTION AND REMARKS
			15/6"		4.6	122.0					Crowder Formation continued, very hard
				30						30	
			17/6" 8/1"	35	6.0	123.3				35	Scattered gravel, very hard
				40						40	
			16/6" 9/1"	45	6.0	120.1				45	Very hard, contains coarse sand

This log is a representation of conditions at the time & place of drilling. With the passage of time or at any other location there may be consequential changes in conditions.

Total depth 45 feet.
No water. No caving.

See Legend for Important Notes

*Driving Weight: **3350 to 29' - 2040 to 58'**

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Boring B-2A
Sheet 2 of 2

ATTACHMENT C

Soil Survey Sheets

Plates C-1 through C-7

JOHN A SAYERS & ASSOCIATES

MAP LEGEND

Qw	Wash deposits
Qal	Alluvium
Qt	Terrace Deposits
Qoal	Older alluvium
Pcr	Crowder Formation
	Bedding attitude
	Horizontal bedding
B-16	Boring location

PLATE D-1

ATTACHMENT D

Map Legend, Plate D-1

and

Boring Location Maps, Plates D-2 through D-7

JOHN A SAYERS & ASSOCIATES

State of California
DEPARTMENT OF TRANSPORTATION

Business, Transportation and Housing Agency

M e m o r a n d u m

Flex your power!
Be energy efficient!

To: GORDON DANKE
Office of Structure Design–West
Design Branch 9

Date: December 19, 2013

File: 08-SBd-138-R17.6
08-0Q3001
Proj. ID: 0800020191
Miner's Shack Creek Br.
Br. #54-1219

Attention: Pete Norboe

From: **DEPARTMENT OF TRANSPORTATION**
DIVISION OF ENGINEERING SERVICES
GEOTECHNICAL SERVICES
OFFICE OF GEOTECHNICAL DESIGN – SOUTH 2
DESIGN BRANCH B, MS #5

Subject: Foundation Report for Miner's Shack Creek Bridge

This report presents the foundation recommendations for the proposed Miner's Shack Creek Bridges (Br. No. 54-1219). In 2003, the proposed Miner's Shack Creek Bridge was originally designed in metric using Working Stress Design (WSD) for abutments and Load Factor Design (LFD) for the pier. The project was put on hold for many years, and now it is being reactivated. The bridge is being updated to the current AASHTO Load and Resistance Factor Design (LRFD) bridge specifications with California Amendments and the structure plans being coverted from metric to english units.

This foundation report supersedes the previously generated Foundation Report, dated 11-28-03 and any other foundation reports generated for this structure. The following foundation recommendations are based on the 2002 subsurface investigation performed by geotechnical services and design information provided by Structures Design Branch 9. All elevations referenced within this report are based on the NGVD 1929 vertical datum.

Project Description/History

The proposed bridge site is located on the proposed newly re-aligned Route 138, located approximately 500 feet north of the existing State Route 138 (PM 17.1) in the Cajon Pass area of San Bernardino County. Based on the General Plan (dated 11-21-13), the proposed Miner's Shack Creek Bridge is to be approximately 250 ft long and consists of a two-span, cast-in-place, prestressed, concrete box girder structure with seat abutments.

Site Geology

The "Geologic Map of the Cajon Quadrangle" (Dibblee & Minch, 2003) indicates the site is underlain by Quaternary Alluvium deposits which overlie the Crowder Formation, which Dibblee describes as a fine to coarse, arkosic, pebbly sandstone.

The subsurface investigation performed in 2002 consisted of 2 mud rotary borings which generally revealed alluvial materials overlying sandstone. The alluvial material consists of medium dense to very dense layers of silty sand with gravel and cobbles, poorly graded sand with gravel and cobbles, and silt with sand. Underlying the alluvium, the sandstone was described as very intensely weathered, weakly to moderately cemented with soil like characteristics which consist of a very dense silty sand with gravel and cobbles, sandy/silty gravel, poorly graded sand with gravel and cobbles, with thin intermittent lean clay lenses. For site-specific soil and rock descriptions, refer to the Log of Test Borings sheets.

Groundwater

Groundwater was encountered during the 2002 subsurface investigation. In Boring 02-1, near the Abutment 1 location, the measured groundwater level was at elevation 3417.0 ft. In Boring 02-2, near the Pier 2 location, the measured groundwater level was at elevation 3414.2 ft. The structure supports are located within an intermittent creek, therefore, the groundwater surface elevations are subject to large fluctuations due to seasonal precipitation (heavy rainfall/flash floods), and will be encountered at higher or lower elevations at time of construction.

Scour Potential

The bridge site is located within an intermittent creek that during times of heavy precipitation has large amounts of surface water flowing in the channel and has the potential for scour. The scour data shown in Table 1 is based on the hydraulic data provided to our office. At the time of the report, a final hydraulic report was not completed. For details and information used in developing the estimated scour information, please contact Jimmie Pallares with the Structure Hydraulics and Hydrology Branch at 916-227-8705.

Table 1 - Scour Data

Support Location	Long Term Scour (Degradation and Contraction) Elevation (ft)	Short Term Scour (Local) Depth (ft)
Abutment 1	N/A	N/A
Pier 2	3.7	19.1
Abutment 3	N/A	N/A

Corrosion

Corrosion test results for soils samples collected from Boring 02-2 are shown in Table 2 below. All of the soil samples tested are considered non-corrosive by current Caltrans standards.

Table 2 - Corrosion Test Summary

Sample Location	pH	Minimum Resistivity (Ohm-Cm)	Chloride Content (ppm)	Sulfate Content (ppm)
Elev. 3430-3409 ft	7.31	2950	-	-
Elev. 3409-3390 ft	8.16	2200	-	-
Elev. 3390-3370 ft	9.39	2500	-	-
Elev. 3370-3340 ft	8.93	1900	-	-
Elev. 3340-3290 ft	8.86	2400	-	-

Note: Caltrans currently defines a corrosive environment as an area where the soil has either a chloride concentration of 500 ppm or greater, a sulfate concentration of 2000 ppm or greater, or has a pH of 5.5 or less. With the exception of MSE walls, soil and water are not tested for chlorides and sulfates if the minimum resistivity is greater than 1,000 ohm-cm.

Fault and Seismic Data

The structure site is potentially subject to ground motions from nearby earthquake sources during the design life of the new structure. Based on the 2012 Caltrans Fault Database (V2.2.06) for the deterministic procedure, the controlling fault for the site is the San Andreas (San Bernardino South) fault zone (Fault ID: 325, strike-slip, dip=90 deg.) with a maximum moment magnitude (M_{Max}) of 7.9, located approximately 3.9 miles (6.2 km) southwest of the site. The probabilistic response spectrum is obtained for the 5% probability of exceedance in 50 years from the 2008 USGS National Seismic Hazard Map.

Based on the 2002 Log of Test Borings and general geology at the site, the average shear wave velocity for the upper 100 ft of the subsurface materials is estimated as V_{S30} = 1150 ft/s (350 m/s).

For this site, the design response spectrum is controlled solely by the probabilistic response spectrum, with a Peak Ground Acceleration (PGA) of 0.97g.

Surface Rupture Potential

The site is not located within an Alquist Priolo Earthquake Fault Zone or an unzoned fault with an age of Holocene or younger, therefore, the potential for surface fault rupture hazard is low.

Liquefaction Potential

Due to the dense nature of the alluvial material underlying the site and the groundwater elevation, the liquefaction potential at the site is anticipated to be low. For site specific seismic data and design recommendations mentioned above, refer to the memorandum concerning seismic design recommendations (dated November 27, 2013), by Asef Wardak (916-227-1219) of the Office of Geotechnical Design South 2 (OGDS2).

Foundation Recommendations

The following Foundation Recommendations are for the Miner's Shack Creek Bridge (Replacement), Br. No. 54-1219, as shown on General Plan, dated November 21, 2013.

At the abutment support locations, driven "Modified" Class 140, Alternative "W" steel pipe piles are recommended to be used for support. The "Modified" Class 140, Alternative "W" steel pipe pile shall be specified with a circular steel plate or conical steel tip welded to the bottom of the pile, similar to the tip detail of Alternative "V" pipe pile as shown in the 2010 Standard Plans. Spread footings are not feasible at this location due to the presence of potential loose to medium dense granular soil and high PGA (MTD 5-1). CIDH piles are also not recommended at the abutments due to the need for battered piles.

At the Pier 2 support location, large diameter (84-inch) Type-II CIDH piles are recommended for support. At the Pier 2 support location, spread footings and small diameter piles are not feasible due to the large local scour potential within the channel and the anticipated structural loads.

The foundation design at the abutment locations was based on working stress design (WSD) and Load Resistance Factored Design (LRFD) was used at the pier location. The design information shown in Table 3 & 4 was provided to our office by Structure Design, Branch 9 on May 8, 2013 and supplemented on December 13, 2013.

Table 3 - Foundation Data Provided by Structure Design

Support Location	Design Method	Pile Type	Finished Grade Elevation (ft)	Pile Cut-Off Elevation (ft)	Pile Cap Size (ft)		Permissible Settlement under Service Load (in)	Number of Piles per Support
					B	L		
Abut 1	WSD	Modified Class 140 Alt W (PP 14 x 0.438)	3454.5	3452.92	7	43	1	14
Pier 2	LRFD	84 in CIDH	3428.5	3422.0	N/A	N/A	1	2
Abut 3	WSD	Modified Class 140 Alt W (PP 14 x 0.438)	3464.0	3462.42	7	43	1	14

Table 4 - Foundation Design Loads Provided by Structure Design

Support Location	Service-I Limit State (kips)			Strength Limit State (Controlling Group, kips)				Extreme Limit State (Controlling Group, kips)			
	Total Load		Perm. Loads	Compression		Tension		Compression		Tension	
	Per Support	Max. Per Pile	Per Support	Per Support	Max. Per Pile	Per Support	Max. Per Pile	Per Support	Max. Per Pile	Per Support	Max. Per Pile
Abut 1	1550	140	1230	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A
Pier 2	4200	N/A	2320	6060	3030	N/A	0	2320	4050	N/A	-1430
Abut 3	1550	140	1230	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A

The specific pile tip elevations for Abutments 1 and 3, and Pier 2 are shown below in Table 5 and Table 6.

Table 5 - Foundation Design Recommendations for Abutments 1 and 3

Support Location	Pile Type	Cut-off Elevation (ft)	LRFD Service-I Limit State Load per Support (kips)		LRFD Service-I Limit State Total Load per Pile (kips)	Nominal Resistance (kips)	Design Tip Elevation (ft)	Specified Tip Elevation (ft)	Nominal Driving Resistance Required (kips)
			Total	Perm.					
Abut 1	Modified Class 140 Alt W (PP 14 x 0.438)	3452.92	1550	1230	140	280	3432.8 (a)	3432.8	280
Abut 3	Modified Class 140 Alt W (PP 14 x 0.438)	3462.42	1550	1230	140	280	3442.5 (a)	3442.5	280

Note: 1) Design tip elevations are controlled by: (a) Compression
2) "Modified" Class 140, Alternative "W" pipe pile is to be driven with either a flat or conical steel tip welded to the pile tip.

Table 6 - Foundation Design Recommendations for Pier 2

Support Location	Pile Type	Cut-Off Elev. (ft)	Service-I Limit State Load per Support (kips)	Total Permissible Support Settlement (inches)	Required Nominal Resistance (R_n) (kips)				Design Tip Elev. (ft)	Specified Tip Elev. (ft)
					Strength Limit		Extreme Limit			
					Comp. (ϕ=0.7)	Tension (ϕ=0.7)	Comp. (ϕ=1.0)	Tension (ϕ=1.0)		
Pier 2	84 in CIDH	3422.0	4200	1	4330	0	4050	1430	3353.0 (a-I) 3360.0 (a-II) 3382.0 (b-II)	3353.0

Note: Design tip elevations are controlled by: (a-I) Compression (Strength), (a-II) Compression (Extreme), (b-II) Tension (Extreme)

The Pile Data Table is presented in Table 7, below. The ultimate geotechnical pile capacity of the CIDH piles will meet or exceed the required nominal resistance in compression.

Table 7 - Pile Data Table

Support Location	Pile Type	Nominal Resistance (R_n) (kips)		Design Tip Elev. (ft)	Specified Tip Elev. (ft)	Nominal Driving Resistance (kips)
		Compression	Tension			
Abut 1	Modified Class 140 Alt W (PP 14 x 0.438)	280	0	3432.8 (a)	3432.8	280
Pier 2	84 in CIDH	4330	1430	3353.0 (a) 3382.0 (b)	3353.0	N/A
Abut 3	Modified Class 140 Alt W (PP 14 x 0.438)	280	0	3442.5 (a)	3442.5	280

Note: 1) Design tip elevations are controlled by: (a) Compression, (b) Tension
2) "Modified" Class 140, Alternative "W" pipe pile is to be driven with either a flat or conical steel tip welded to the pile tip.

At the Pier 2 pile location, a Type II shaft with an optional construction joint is shown on the structure plans. If the optional construction joint is used, then the permanent steel casing used must be a corrugated metal pipe CMP because it will be located within the CIDH skin friction zone. For the CIDH piles, the skin friction was utilized from the construction joint elevation down to a distance within 4ft above the specified pile tip elevation listed in Table 7. Due to the anticipated use of slurry displacement methods to construct the pile, no end bearing was considered. The skin friction zone is listed in Table 9 in the construction considerations section of this report. All foundation design loads and corresponding limit states (listed in Table 4) were considered in developing the foundation design recommendations provided in Tables 5 & 6.

At the abutment locations, "drilling" to assist driving (Standard Specifications 49-2.01C(3)) will be required to minimize erratic driving and to advance the pile down to the specified pile tip elevation listed in Table 7. The design of the piles was based predominantly on end bearing with some skin friction. The nominal axial structural resistance for the driven steel pipe piles will meet or exceed the required nominal resistance in compression.

General Notes:

1) The structure engineer shall show on the plans, in the pile data table, the minimum pile design tip elevation required to meet the lateral load demands. If the specified pile tip elevation required to meet lateral load demands exceed the specified pile tip elevation given within this report, the Office of Geotechnical Design South 2, Branch B should be contacted for further recommendations.

2) "Modified" Class 140, Alternative "W" steel pipe pile details are to be shown on the project plans. The "Modified" Class 140, Alternative "W" steel pipe piles shall be shown with a circular steel plate or conical steel tip with a minimum thickness of ¾ inch welded to the pile tip, similar to the Alternative "V" pile tip detail shown in the 2010 Standard Plans.

3) The special provisions shall specify the requirements of Tunnel Safety Orders for the CIDH shaft work that meets the definition of a tunnel or shaft as described in the Highway Design Manual, Section 110.12 "Tunnel Safety Orders."

Construction Considerations:

- **Rock Core Samples and Laboratory Testing**

1) Rock core samples from the 2002 Caltrans foundation investigation are available for viewing by bidders at the Geotechnical Services (Transportation Laboratory), Department of Transportation, 5900 Folsom Blvd., Sacramento, CA. Specifically, only Boring B-02-2 was retained for viewing. Potential bidders are to allow the State seven (7) days to prepare and display the rock cores.

- **Driven Piles**

1) At Abutments 1 & 2, "Drilling" to assist driving, per Standard Specification Section 49-2.01C(3), will be required to minimize erratic pile driving and to advance the piles to specified pile tip elevation. The bottom of the "drilled" hole elevation is shown below in Table 8.

Table 8: "Drilling" to Assist Driving for Abutment 1 & 3

Location	Bottom of Drilled Hole Elevation (ft)
Abutment 1	3442.8
Abutment 3	3452.5

2) At the abutment locations, the contractor should anticipate hard driving due to the physical characteristic of the very dense alluvium overly the sedimentary rock as shown in the Log of Test Boring sheets.

3) Pile acceptance is to be based on Standard Specifications 49-2.01A(4)(b) "Pile Driving Acceptance Criteria". At Abutments 1 and 3 support locations, any pile that achieves 1½ times the required nominal resistance in compression, as shown on the contract plans, within 6 feet of the specified pile tip elevation, may be considered satisfactory and cut off with written approval from the engineer. (e.g. 1½ times the required nominal resistance in compression will be 420 kips at Abutment 1 & 3).

- **CIDH Piles**

1) During the 2002 foundation investigation, groundwater was encountered at the bridge site. Due to the location of Pier 2 being within the channel, the contractor should anticipate that surface and/or groundwater will be encountered during the construction of the CIDH piles. Groundwater elevations at this bridge site are subject to seasonal fluctuations and will be encountered at higher or lower elevations depending on seasonal precipitation rates leading up to and at the time of construction. For more details refer to the site geology and groundwater section of this report, and the Log of Test Boring Sheets.

2) Groundwater is expected to easily permeate through the alluvial soils overlying the rock. It is also anticipated that groundwater will also permeate through the upper portions of the rock due to the weakly cemented and granular nature of the rock. Temporary casing extended into the rock may not eliminate groundwater entering the CIDH pile excavation.

3) At the Pier 2, it is anticipated that groundwater and/or surface water will be encountered and will require concrete placement of CIDH piles using slurry displacement methods. If the CIDH piles are to be constructed using slurry displacement method, the slurry shall consist of mineral or synthetic slurry. Use of water shall not be allowed as slurry.

4) The 2002 subsurface investigation identified medium dense to very dense alluvium, which in general consisted of sand with gravel and localized hard cobbles overlying moderately to weakly cemented, sedimentary rock with soil-like characteristics. At the pier pile locations, the contractor will need to excavate through these earth materials to construct the shaft. Caving conditions are anticipated in both the alluvium and within the weakly cemented zones of the rock. The amount of caving the contractor will experience will be dependent upon the methods and means the contractor chooses to use to construct the CIDH piles. Temporary casing may be necessary to control caving during construction. All temporary casing is to be removed during concrete placement.

5) The calculated geotechnical capacity of all CIDH piles is based on skin friction only and no end-bearing was considered. For the CIDH piles, the skin friction zone used to develop the geotechnical capacity is from the construction joint elevation down to a location 4 ft above the specified pile tip elevation. The skin friction zone of the CIDH piles is summarized below in Table 9.

Table 9: CIDH Pile Skin Friction Zone Elevations

Location *	**Skin Friction Zone Start Elevation (ft)**	**Skin Friction Zone End Elevation (ft)**
Pier 2, Column 1 & 2	3410	3457

* Pier columns are numbered from 1 to 2 (from left to right) looking up-station.

6) At the Pier 2, Type II CIDH piles are specified with an optional construction joint that is shown on the contract plans. If the contractor chooses to construct the optional construction joint, then it shall be in conformance to Standard Specifications 49-3.02C(6) and possibly 49-3.02C(5). Because of the soil-like characteristics of the rock, any permanent casing used must extend 5 feet below the optional construction joint shown on the plans. If the contractor chooses to construct the optional construction joint, the permanent steel casing must consist of a corrugated metal pipe (CMP) since it is located within the skin friction zone listed in Table 9 above.

This Foundation Report is based on specific project information that has been provided by the Office of Structure Design - West, Design Branch 9. If any conceptual changes are made during final project design, the Office of Geotechnical Design-South 2, Design Branch B should review those changes to determine if this report is still applicable. Any questions regarding the above report should be directed to the attention of Hector Valencia, (916) 227-4555, or Mark DeSalvatore, (916) 227-5391 at the Office of Geotechnical Design-South 2, Branch B.

Prepared by: Date: 12-19-13

Hector Valencia, P.E., #C65257
Engineering Geologist
Office of Geotechnical Design-South 2
Design Branch B

cc: Mohammad Mollazadeh- Dist. 8 Project Manager
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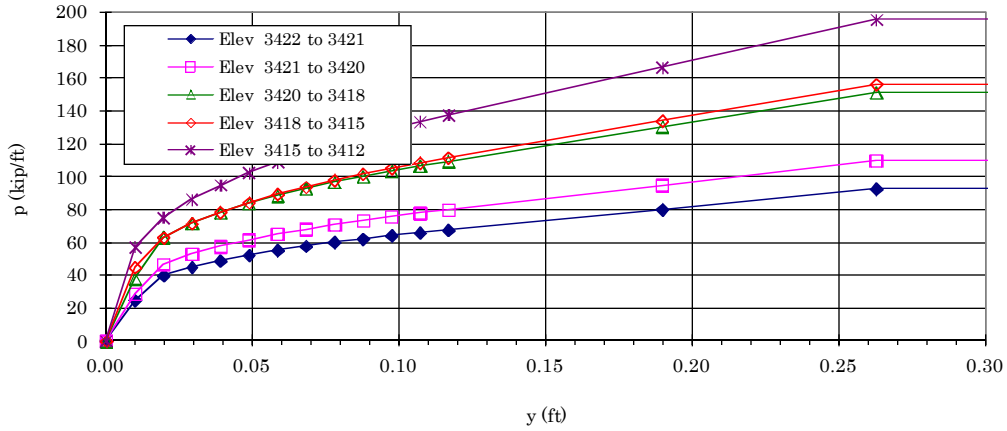
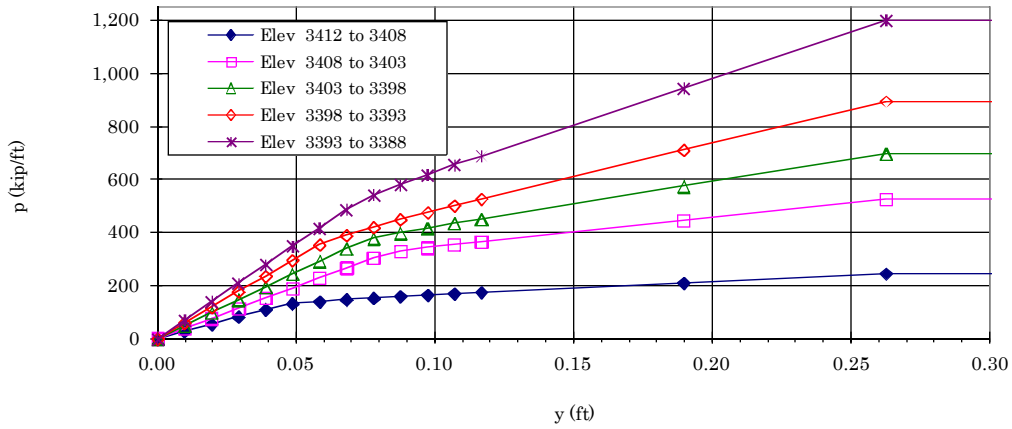
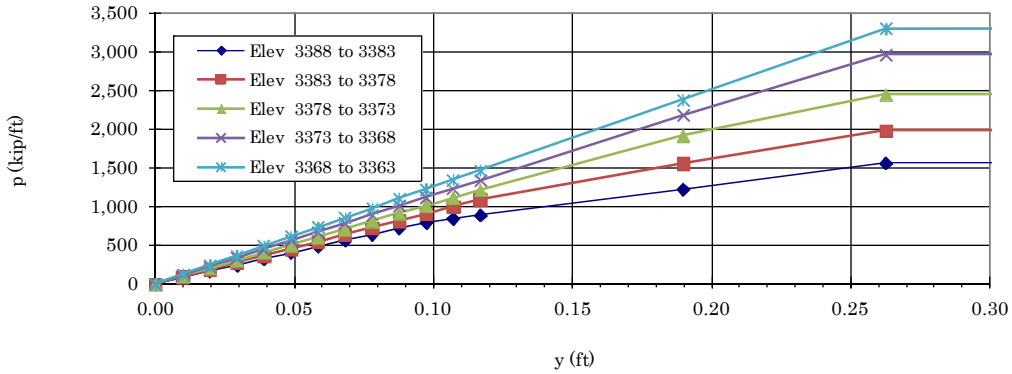
Mark DeSalvatore – OGDS2
Geotechnical Archive

Miner's Shack Creeek Bridge
Bridge NO. 54-1219
EA# 08-0Q300K
Project No. 08000020191

Pier 02		**O.G.**	3428 ft	**Pile**	**7 ft CIDH**
Sta.	110+40.0	**Cut-off**	3422 ft	**GWS**	3414.1-ft





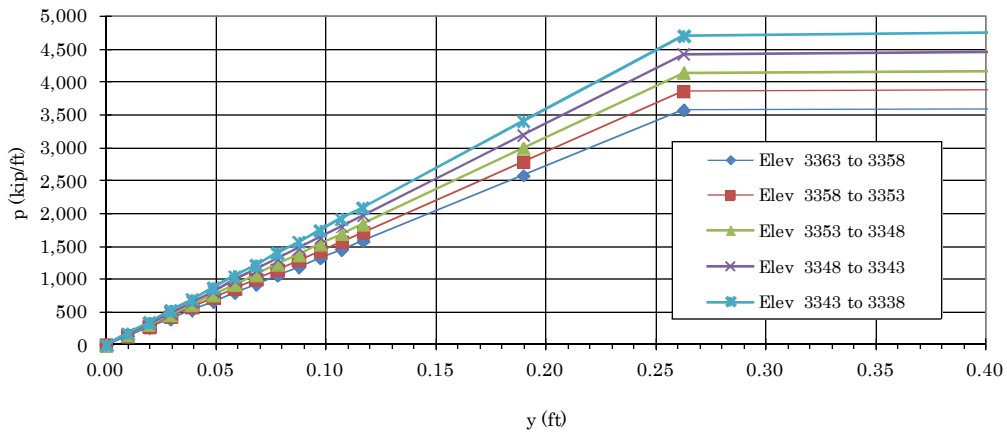


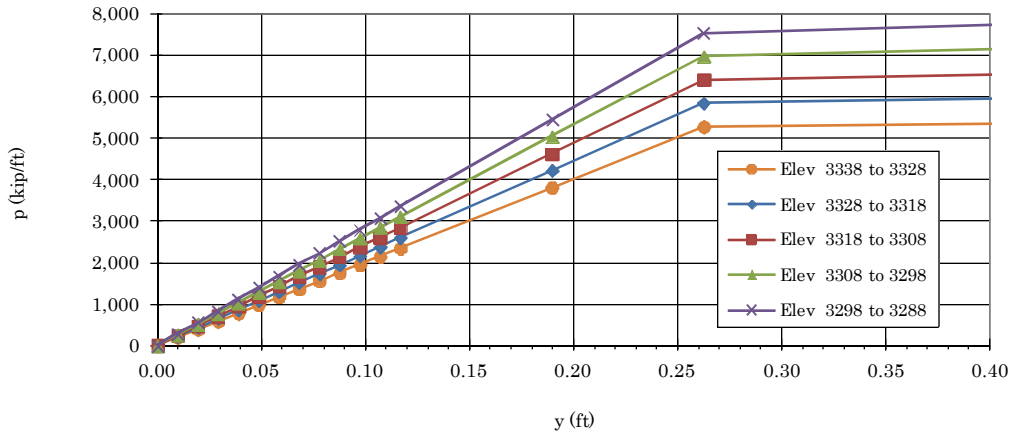
Post-Liq. **B-02-2** 5.25' Lt @ Sta. 110+60.2 "C/L Rte. 138" 5/8/02

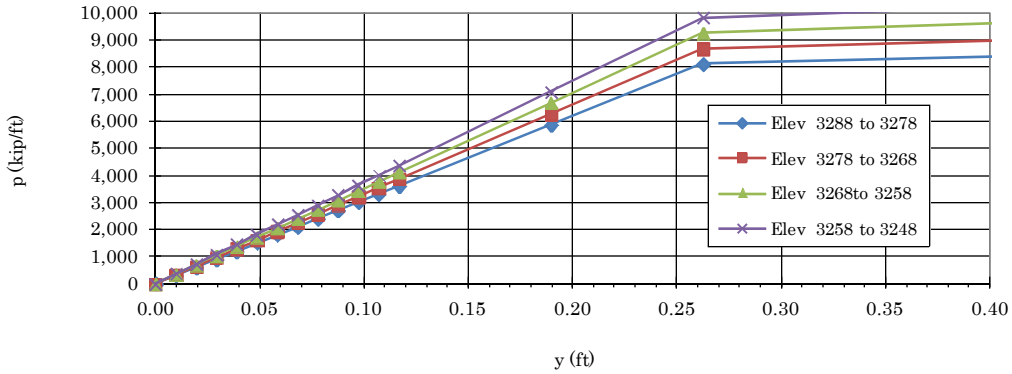
p-y Curves

Miner's Shack Creeek Bridge
Bridge NO. 54-1219
EA# 08-0Q300K
Project No. 08000020191

Pier 02		**O.G.**	3428 ft	**Pile**	**7 ft CIDH**
Sta.	110+40.0	**Cut-off**	3422 ft	**GWS**	3414.1-ft







Post-Liq. **B-13-01** 88.58' RT Sta. 190+21.26 "C/L Rte. 15" 10/13/01

p-y Curves

Miner's Shack Creeek Bridge
Bridge NO. 54-1219
EA# 08-0Q300K
Project No. 08000020191

Pier 02						O.G.	3428 ft	Pile	7 ft CIDH
Sta.	110+40.0					Cut-off	3422 ft	GWS	3414.1-ft
y (ft)	Elev 3422 to 3421	y (ft)	Elev 3421 to 3420	y (ft)	Elev 3420 to 3418	y (ft)	Elev 3418 to 3415	y (ft)	Elev 3415 to 3412
0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000
0.010	24.997	0.010	28.777	0.010	38.227	0.010	44.981	0.010	57.329
0.019	39.967	0.019	46.927	0.019	63.230	0.019	62.950	0.019	75.187
0.029	45.037	0.029	52.954	0.029	71.612	0.029	71.671	0.029	86.213
0.039	49.020	0.039	57.693	0.039	78.225	0.039	78.583	0.039	95.003
0.049	52.351	0.049	61.660	0.049	83.772	0.049	84.399	0.049	102.434
0.058	55.239	0.058	65.102	0.058	88.595	0.058	89.469	0.058	108.935
0.068	57.805	0.068	68.162	0.068	92.889	0.068	93.993	0.068	114.752
0.078	60.124	0.078	70.929	0.078	96.776	0.078	98.097	0.078	120.041
0.088	62.247	0.088	73.463	0.088	100.340	0.088	101.865	0.088	124.909
0.097	64.209	0.097	75.806	0.097	103.638	0.097	105.358	0.097	129.431
0.107	66.037	0.107	77.989	0.107	106.716	0.107	108.620	0.107	133.662
0.117	67.751	0.117	80.038	0.117	109.605	0.117	111.687	0.117	137.645
0.190	80.225	0.190	94.944	0.190	130.637	0.190	134.025	0.190	166.678
0.263	92.698	0.263	109.849	0.263	151.668	0.263	156.362	0.263	195.712
7.263	92.698	7.263	109.849	7.263	151.668	7.263	156.362	7.263	195.712
14.263	92.698	14.263	109.849	14.263	151.668	14.263	156.362	14.263	195.712
y (ft)	Elev 3412 to 3408	y (ft)	Elev 3408 to 3403	y (ft)	Elev 3403 to 3398	y (ft)	Elev 3398 to 3393	y (ft)	Elev 3393 to 3388
0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000
0.010	27.609	0.010	38.109	0.010	48.609	0.010	59.109	0.010	69.609
0.019	55.217	0.019	76.217	0.019	97.217	0.019	118.217	0.019	139.217
0.029	82.826	0.029	114.326	0.029	145.826	0.029	177.326	0.029	208.826
0.039	110.434	0.039	152.434	0.039	194.434	0.039	236.434	0.039	278.434
0.049	132.038	0.049	190.543	0.049	243.043	0.049	295.543	0.049	348.043
0.058	140.030	0.058	228.652	0.058	291.652	0.058	354.652	0.058	417.652
0.068	147.162	0.068	266.760	0.068	340.260	0.068	389.893	0.068	487.260
0.078	153.634	0.078	304.869	0.078	378.546	0.078	420.136	0.078	541.988
0.088	159.578	0.088	330.553	0.088	398.452	0.088	448.754	0.088	581.288
0.097	165.090	0.097	343.046	0.097	417.143	0.097	476.002	0.097	618.851
0.107	170.241	0.107	354.754	0.107	434.806	0.107	502.073	0.107	654.918
0.117	175.082	0.117	365.790	0.117	451.583	0.117	527.118	0.117	689.676
0.190	210.351	0.190	446.287	0.190	574.389	0.190	711.436	0.190	945.859
0.263	245.619	0.263	526.783	0.263	697.194	0.263	895.754	0.263	1202.041
7.263	245.619	7.263	526.783	7.263	697.194	7.263	895.754	7.263	1202.041
14.263	245.619	14.263	526.783	14.263	697.194	14.263	895.754	14.263	1202.041
y (ft)	Elev 3388 to 3383	y (ft)	Elev 3383 to 3378	y (ft)	Elev 3378 to 3373	y (ft)	Elev 3373 to 3368	y (ft)	Elev 3368 to 3363
0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000
0.010	80.109	0.010	90.609	0.010	101.109	0.010	111.609	0.010	122.109
0.019	160.217	0.019	181.217	0.019	202.217	0.019	223.217	0.019	244.217
0.029	240.326	0.029	271.826	0.029	303.326	0.029	334.826	0.029	366.326
0.039	320.434	0.039	362.434	0.039	404.434	0.039	446.434	0.039	488.434
0.049	400.543	0.049	453.043	0.049	505.543	0.049	558.043	0.049	610.543
0.058	480.652	0.058	543.652	0.058	606.652	0.058	669.652	0.058	732.652
0.068	560.760	0.068	634.260	0.068	707.760	0.068	781.260	0.068	854.760
0.078	640.869	0.078	724.869	0.078	808.869	0.078	892.869	0.078	976.869
0.088	720.977	0.088	815.477	0.088	909.977	0.088	1004.477	0.088	1098.977
0.097	794.730	0.097	906.086	0.097	1011.086	0.097	1116.086	0.097	1221.086
0.107	842.144	0.107	996.695	0.107	1112.195	0.107	1227.695	0.107	1343.195
0.117	887.895	0.117	1087.303	0.117	1213.303	0.117	1339.303	0.117	1465.303
0.190	1225.295	0.190	1552.335	0.190	1917.995	0.190	2176.368	0.190	2381.118
0.263	1562.695	0.263	1979.789	0.263	2446.138	0.263	2961.744	0.263	3296.932
7.263	1562.695	7.263	1979.789	7.263	2446.138	7.263	2961.744	7.263	3526.608
14.263	1562.695	14.263	1979.789	14.263	2446.138	14.263	2961.744	14.263	3526.608
Note:	p (kip/ft)	Post-Liq.			B-02-2	5.25' Lt @ Sta. 110+60.2 "C/L Rte. 138"			5/8/2002

Miner's Shack Creeek Bridge
Bridge NO. 54-1219
EA# 08-0Q300K
Project No. 08000020191

Pier 02						O.G.	3428 ft	Pile	7 ft CIDH
Sta.	110+40.0					Cut-off	3422 ft	GWS	3414.1-ft
y (ft)	Elev 3363 to 3358	y (ft)	Elev 3358 to 3353	y (ft)	Elev 3353 to 3348	y (ft)	Elev 3348 to 3343	y (ft)	Elev 3343 to 3338
0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000
0.010	132.609	0.010	143.109	0.010	153.609	0.010	164.109	0.010	174.609
0.019	265.217	0.019	286.217	0.019	307.217	0.019	328.217	0.019	349.217
0.029	397.826	0.029	429.326	0.029	460.826	0.029	492.326	0.029	523.826
0.039	530.434	0.039	572.434	0.039	614.434	0.039	656.434	0.039	698.434
0.049	663.043	0.049	715.543	0.049	768.043	0.049	820.543	0.049	873.043
0.058	795.652	0.058	858.652	0.058	921.652	0.058	984.652	0.058	1047.652
0.068	928.260	0.068	1001.760	0.068	1075.260	0.068	1148.760	0.068	1222.260
0.078	1060.869	0.078	1144.869	0.078	1228.869	0.078	1312.869	0.078	1396.869
0.088	1193.477	0.088	1287.977	0.088	1382.477	0.088	1476.977	0.088	1571.477
0.097	1326.086	0.097	1431.086	0.097	1536.086	0.097	1641.086	0.097	1746.086
0.107	1458.695	0.107	1574.195	0.107	1689.695	0.107	1805.195	0.107	1920.695
0.117	1591.303	0.117	1717.303	0.117	1843.303	0.117	1969.303	0.117	2095.303
0.190	2585.868	0.190	2790.618	0.190	2995.368	0.190	3200.118	0.190	3404.868
0.263	3580.432	0.263	3863.932	0.263	4147.432	0.263	4430.932	0.263	4714.432
7.263	4140.731	7.263	4804.114	7.263	5516.758	7.263	6278.663	7.263	7089.829
14.263	4140.731	14.263	4804.114	14.263	5516.758	14.263	6278.663	14.263	7089.829
y (ft)	Elev 3338 to 3328	y (ft)	Elev 3328 to 3318	y (ft)	Elev 3318 to 3308	y (ft)	Elev 3308 to 3298	y (ft)	Elev 3298 to 3288
0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000
0.010	195.609	0.010	216.609	0.010	237.609	0.010	258.609	0.010	279.609
0.019	391.217	0.019	433.217	0.019	475.217	0.019	517.217	0.019	559.217
0.029	586.826	0.029	649.826	0.029	712.826	0.029	775.826	0.029	838.826
0.039	782.434	0.039	866.434	0.039	950.434	0.039	1034.434	0.039	1118.434
0.049	978.043	0.049	1083.043	0.049	1188.043	0.049	1293.043	0.049	1398.043
0.058	1173.652	0.058	1299.652	0.058	1425.652	0.058	1551.652	0.058	1677.652
0.068	1369.260	0.068	1516.260	0.068	1663.260	0.068	1810.260	0.068	1957.260
0.078	1564.869	0.078	1732.869	0.078	1900.869	0.078	2068.869	0.078	2236.869
0.088	1760.477	0.088	1949.477	0.088	2138.477	0.088	2327.477	0.088	2516.477
0.097	1956.086	0.097	2166.086	0.097	2376.086	0.097	2586.086	0.097	2796.086
0.107	2151.695	0.107	2382.695	0.107	2613.695	0.107	2844.695	0.107	3075.695
0.117	2347.303	0.117	2599.303	0.117	2851.303	0.117	3103.303	0.117	3355.303
0.190	3814.368	0.190	4223.868	0.190	4633.368	0.190	5042.868	0.190	5452.368
0.263	5281.432	0.263	5848.432	0.263	6415.432	0.263	6982.432	0.263	7549.432
7.263	8859.946	7.263	10827.111	7.263	12991.326	7.263	15352.589	7.263	17910.903
14.263	8859.946	14.263	10827.111	14.263	12991.326	14.263	15352.589	14.263	17910.903
y (ft)	Elev 3288 to 3278	y (ft)	Elev 3278 to 3268	y (ft)	Elev 3268to 3258	y (ft)	Elev 3258 to 3248		
0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000		
0.010	300.609	0.010	321.609	0.010	342.609	0.010	363.609		
0.019	601.217	0.019	643.217	0.019	685.217	0.019	727.217		
0.029	901.826	0.029	964.826	0.029	1027.826	0.029	1090.826		
0.039	1202.434	0.039	1286.434	0.039	1370.434	0.039	1454.434		
0.049	1503.043	0.049	1608.043	0.049	1713.043	0.049	1818.043		
0.058	1803.652	0.058	1929.652	0.058	2055.652	0.058	2181.652		
0.068	2104.260	0.068	2251.260	0.068	2398.260	0.068	2545.260		
0.078	2404.869	0.078	2572.869	0.078	2740.869	0.078	2908.869		
0.088	2705.477	0.088	2894.477	0.088	3083.477	0.088	3272.477		
0.097	3006.086	0.097	3216.086	0.097	3426.086	0.097	3636.086		
0.107	3306.695	0.107	3537.695	0.107	3768.695	0.107	3999.695		
0.117	3607.303	0.117	3859.303	0.117	4111.303	0.117	4363.303		
0.190	5861.868	0.190	6271.368	0.190	6680.868	0.190	7090.368		
0.263	8116.432	0.263	8683.432	0.263	9250.432	0.263	9817.432		
7.263	20666.268	7.263	23618.684	7.263	26768.150	7.263	30114.668		
14.263	20666.268	14.263	23618.684	14.263	26768.150	14.263	30114.668		
Note:	p (kip/ft)	Post-Liq.			B-02-2		5.25' Lt @ Sta. 110+60.2 "C/L Rte. 138"		5/8/2002

State of California Business, Transportation and Housing Agency

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Be energy efficient!

M e m o r a n d u m

To: MR. GORDON DANKE, BRANCH CHIEF
OFFICE OF BRIDGE DESIGN - West
DESIGN BRANCH 9

Date: November 27, 2013

File: 08-SBd-138-PM-R17.6
EA No. 08-0Q300G
Project No. 0800020191
Miner's Shack Crk Bridge
Bridge No. 54-1219

Attn.: Mr. Pete Norboe

From: **DEPARTMENT OF TRANSPORTATION**
DIVISION OF ENGINEERING SERVICES
GEOTECHNICAL SERVICES - MS #5
OFFICE OF GEOTECHNICAL DESIGN SOUTH-2

Subject: **Seismic Design Recommendations for Miner's Shack Creek Bridge**

Introduction

This memorandum presents seismic design recommendations for the Miner's Shack Creek Bridge 54-1219. Ground motion recommendations are based on the Caltrans 2009 Seismic Design Procedure (SDP) as described in the Seismic Design Criteria Version 1.7 (SDC) Appendix B, the Acceleration Response Spectrum (ARS) Online Tool v2.2.06, USGS 2008 Interactive Deaggregations (Beta), and 2002 Log of Test Boring (LOTB) for geotechnical subsurface investigations.

Seismicity

Based on the 2012 Caltrans faults database and Caltrans ARS Online (v2.2.06) Tool, the site is located about 3.9 miles (6.2 km) from the San Andreas (San Bernardino South) fault. This fault (Fault ID 325, MMax = 7.9, strike-slip, dip = 90 degrees, vertical, Bottom and Top of Rupture Plane approximately 8 and 0 miles, respectively) is the controlling fault for the deterministic seismic design procedure. A map showing the location of the bridge and the nearby faults is attached in Figure 1.

Soil Profile

The Office of Geotechnical Design South-2 had performed subsurface geotechnical investigations dated May 8, 2002 and August 7, 2002. This subsurface geotechnical exploration included two (2) 4-inch diameter rotary wash borings.

The maximum depth of the investigation was approximate 181 feet below the existing ground surface.

The geologic map "Geologic Map of the Cajon Quadrangle" (Dibblee, 2003) indicates the site is underlain by Quaternary Alluvium deposits which overlie Crowder Formation. Crowder Formation consists of very dense, weakly to moderately cemented arkosic sandstones.

The 2002 subsurface investigation revealed alluvial materials overlying sandstone. The alluvial material consists of medium dense to very dense layers of silty sand with gravel and cobbles, poorly graded sand with gravel and cobbles, and silt with sand. Underlying the alluvium is sandstone, which is very intensely weathered, weakly to moderately cemented. The sandstone consists of very dense layers of silty sand with gravel and cobbles, sandy gravel, poorly graded sand with gravel and cobbles, with thin intermittent lean clay lenses. Groundwater was measured on August 13 and 14, 2002 at elevation 3417 feet and 3414 feet, respectively along the bridge alignment. For site-specific soil and rock descriptions, refer to the Log of Test Borings sheets and Foundation Report (FR) prepared by the Office of Geotechnical Design South 2, Branch B (OGDS-2B) for additional information.

Per 2002's LOTBs and general geology at this site, the average shear wave velocity for the upper 100 feet of subsurface materials is estimated as V_{S30} = 350 m/s.

Design Response Spectrum

Based on the 2009 SDP, the design response spectrum is the upper envelope of the deterministic and probabilistic response, but is not less than a minimum deterministic response spectrum resulting from a Mmax = 6.5 earthquake on a vertical strike-slip fault at a distance of 7.5 miles (12 km).

The deterministic response spectrum is obtained by taking the arithmetic average of the median response spectrum calculated using the 2008 Campbell-Bozorgnia and 2008 Chiou-Youngs ground motion prediction equations. The probabilistic response spectrum is obtained for 5 percent probability of exceedance in 50 years (corresponding to 975 year return period).

For this site, the design response spectrum is controlled by probabilistic response spectrum. The corresponding peak horizontal ground acceleration at proposed site is 0.97g. The recommended acceleration response spectrum is attached in Figure 2.

Liquefaction Potential Evaluation

Based on the 2002 LOTBs, the soil below groundwater at Miner's Shack Creek Bridge may be prone to liquefaction due to strong ground shaking. It must be noted that this liquefaction analysis is preliminary based on the available limited soil information. Field and laboratory geotechnical investigations are necessary to determine liquefaction potential and seismic settlement at supports locations.

Surface Fault Rupture Hazard

The proposed site is not within an Alquist-Priolo Earthquake Fault Zone or an unzoned faults (Holocene or younger in age). The site is more than 2.0 miles (3.3 km) from the nearest Caltrans-active fault, Cleghorn fault zone (Southern Cleghorn section), which extends to the ground surface. Potential for surface rupture is low, and no further work or design for surface rupture is required.

The seismic design recommendations contained in this report are based on specific project information that was provided by the Office of Bridge Design West. If any conceptual changes are made during final project design, the Office of Geotechnical Design South-2 should review those changes to determine whether the seismic design recommendations provided in this report are still applicable. Any questions regarding the above recommendations should be directed to the attention of Asef Wardak (916) 227-1219 or Angel Perez-Cobo (916) 227-7167, Office of Geotechnical Design South-2.

Prepared by: Date: 11-27-2013

Asef Wardak
Transportation Engineer
Office of Geotechnical Design South-2

REGISTERED PROFESSIONAL ENGINEER
ASEF WARDAK
No. C 61860
Exp. 9/30/2015
CIVIL
STATE OF CALIFORNIA

Attachments:
Figure 1. Nearby Major Faults with Reference to the Project Site
Figure 2. Recommended Acceleration Response Spectrum (ARS) Curve

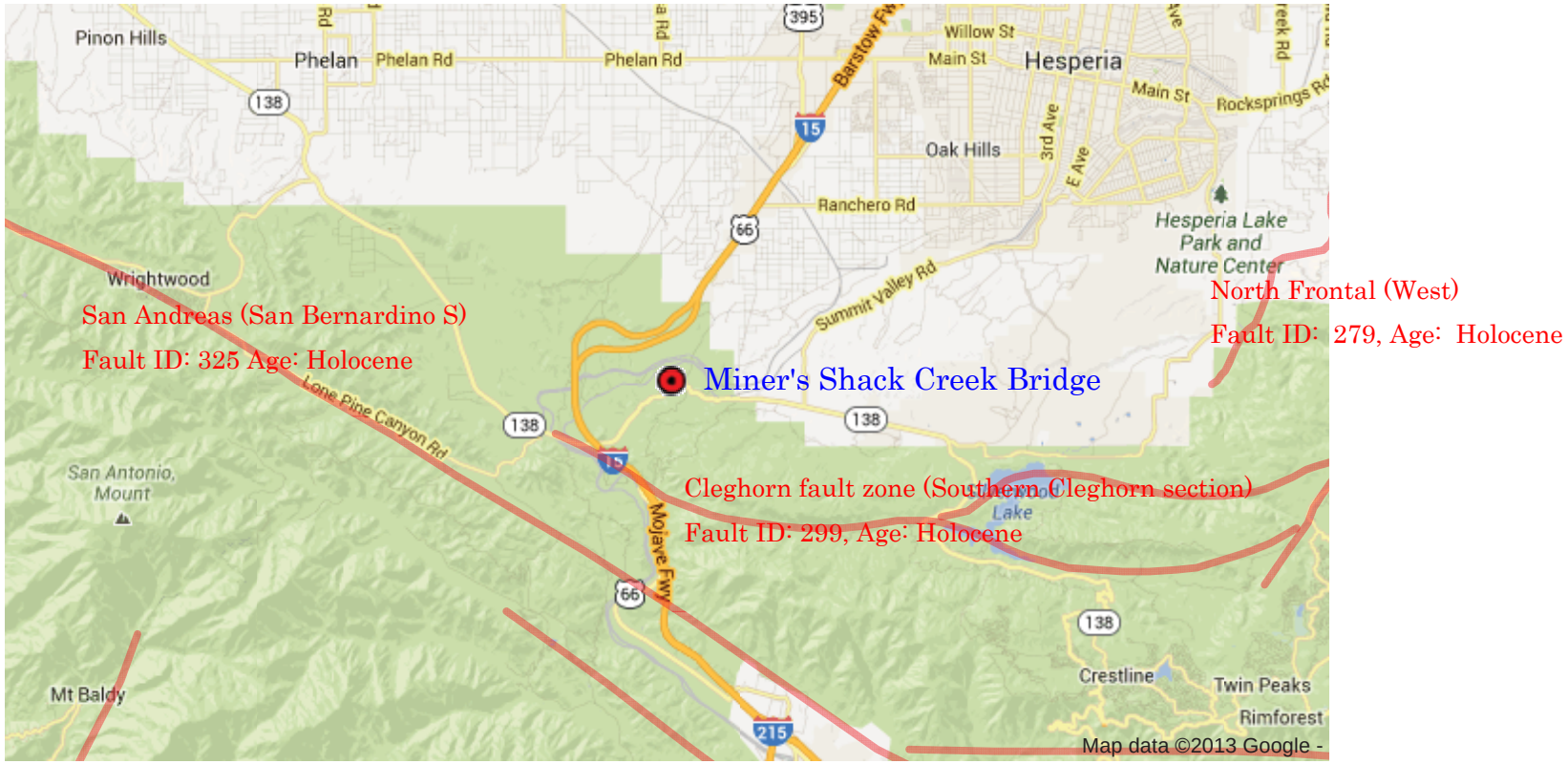
cc: District 08 Structure Construction R.E. – Pending File
District 8 – Project Manager – Jason Bennecke (Electronic File)
Specs & Estimates – Ofelia Alcantara (Electronic File)
District 8, District Materials Engineer – Bruce Kean (Electronic File)
HQ Geotechnical Design South-2 – Abbas Abghari – OGDS-2 (Electronic File)
HQ Geotechnical Design South-2 – Mark Desalvatore – GDS-2 (Electronic File)
HQ Geotechnical Design South-2 – Angel Perez-Cobo (Electronic File) APC
HQ Geotechnical Design South-2 – Erich Neupert – GDS-2 (Electronic File)
HQ Geotechnical Design South-2 – David Liao – GDS-2 (Electronic File)
HQ Geotechnical Design South-2 – Cipriano Manansala – GDS-2 (Electronic File)
HQ Geotechnical Archive

CALIFORNIA DEPARTMENT OF
TRANSPORTATION

Caltrans ARS Online (v2.2.06)

This web-based tool calculates both deterministic and probabilistic acceleration response spectra for any location in California based on criteria provided in *Appendix B of Caltrans Seismic Design Criteria*. More...

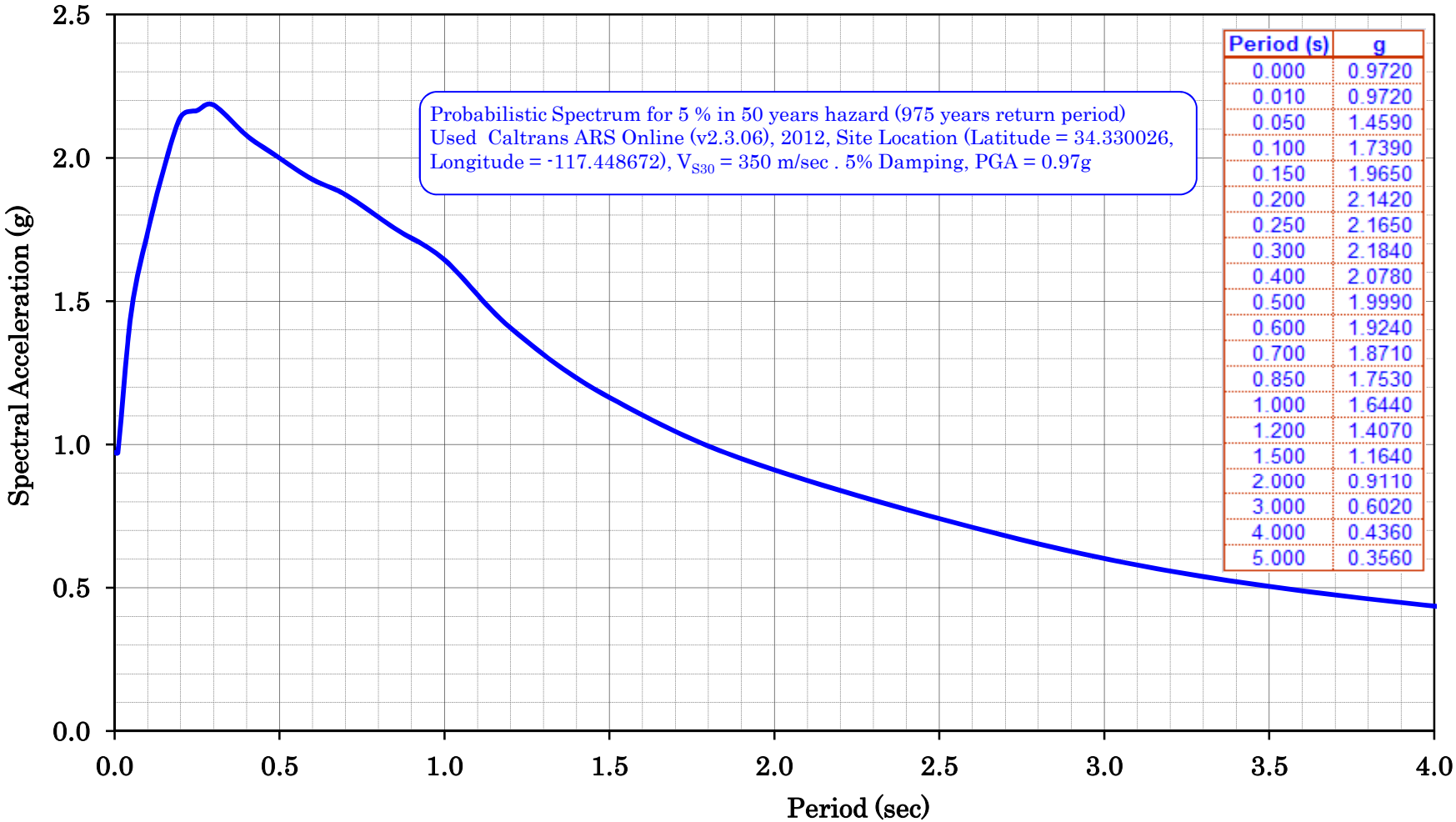
SELECT SITE LOCATION



Latitude: 34.330026 **Longitude:** -117.448672 ***Vs30*:** 290 m/s Calculate

Figure 1. Project site and the nearby faults is attached

Recommended Acceleration Response Spectrum for the Miner's Shack Creek Bridge (New) (Bridge # 54-1219)



Period (s)	g
0.000	0.9720
0.010	0.9720
0.050	1.4590
0.100	1.7390
0.150	1.9650
0.200	2.1420
0.250	2.1650
0.300	2.1840
0.400	2.0780
0.500	1.9990
0.600	1.9240
0.700	1.8710
0.850	1.7530
1.000	1.6440
1.200	1.4070
1.500	1.1640
2.000	0.9110
3.000	0.6020
4.000	0.4360
5.000	0.3560

Figure 2. Recommended Acceleration Response Spectrum (ARS) Curve

State of California
DEPARTMENT OF TRANSPORTATION

Business, Transportation and Housing Agency

M e m o r a n d u m

Flex your power!
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To: GORDON DANKE
Office of Structure Design–West
Design Branch 9

Date: February 4, 2014

File: 08-SBd-138-R17.6
08-0Q3001
Proj. ID: 0800020191
Miner's Shack Creek Br.
Br. #54-1219

Attention: Pete Norboe

From: **DEPARTMENT OF TRANSPORTATION**
DIVISION OF ENGINEERING SERVICES
GEOTECHNICAL SERVICES
OFFICE OF GEOTECHNICAL DESIGN – SOUTH 2
DESIGN BRANCH B, MS #5

Subject: Revised Foundation Report for Miner's Shack Creek Bridge

This report presents the revised foundation recommendations for the proposed Miner's Shack Creek Bridge (Br. No. 54-1219). The revised foundation report is in response to an in-house review of the original foundation report (dated 12-19-13) that identified a typographical error in Table 1 (Scour section) and another in Table 9 (Construction Consideration section) of the report. To indentify the changes, the corrections are bolded and italicized in the corresponding sections of the report. All other portions of the original report (dated 12-19-13) remain unchanged.

In 2003, the proposed Miner's Shack Creek Bridge was originally designed in metric using Working Stress Design (WSD) for abutments and Load Factor Design (LFD) for the pier. The project was put on hold for many years, and now it is being reactivated. The bridge is being updated to the current AASHTO Load and Resistance Factor Design (LRFD) bridge specifications with California Amendments and the structure plans being converted from metric to English units.

This foundation report supersedes the previously generated Foundation Report, dated 11-28-03 and any other foundation reports generated for this structure. The following foundation recommendations are based on the 2002 subsurface investigation performed by geotechnical services and design information provided by Structures Design Branch 9. All elevations referenced within this report are based on the NGVD 1929 vertical datum.

Project Description/History

The proposed bridge site is located on the proposed newly re-aligned Route 138, located approximately 500 feet north of the existing State Route 138 (PM 17.1) in the Cajon Pass area of San Bernardino County. Based on the General Plan (dated 11-21-13), the proposed Miner's Shack Creek Bridge is to be approximately 250 ft long and consists of a two-span, cast-in-place, prestressed, concrete box girder structure with seat abutments.

Site Geology

The "Geologic Map of the Cajon Quadrangle" (Dibblee & Minch, 2003) indicates the site is underlain by Quaternary Alluvium deposits which overlie the Crowder Formation, which Dibblee describes as a fine to coarse, arkosic, pebbly sandstone.

The subsurface investigation performed in 2002 consisted of 2 mud rotary borings which generally revealed alluvial materials overlying sandstone. The alluvial material consists of medium dense to very dense layers of silty sand with gravel and cobbles, poorly graded sand with gravel and cobbles, and silt with sand. Underlying the alluvium, the sandstone was described as very intensely weathered, weakly to moderately cemented with soil like characteristics which consist of a very dense silty sand with gravel and cobbles, sandy/silty gravel, poorly graded sand with gravel and cobbles, with thin intermittent lean clay lenses. For site-specific soil and rock descriptions, refer to the Log of Test Borings sheets.

Groundwater

Groundwater was encountered during the 2002 subsurface investigation. In Boring 02-1, near the Abutment 1 location, the measured groundwater level was at elevation 3417.0 ft. In Boring 02-2, near the Pier 2 location, the measured groundwater level was at elevation 3414.2 ft. The structure supports are located within an intermittent creek, therefore, the groundwater surface elevations are subject to large fluctuations due to seasonal precipitation (heavy rainfall/flash floods), and will be encountered at higher or lower elevations at time of construction.

Scour Potential

The bridge site is located within an intermittent creek that during times of heavy precipitation has large amounts of surface water flowing in the channel and has the potential for scour. The scour data shown in Table 1 is based on the hydraulic data provided to our office. At the time of the report, a final hydraulic report was not completed. For details and information used in developing the estimated scour information, please contact Jimmie Pallares with the Structure Hydraulics and Hydrology Branch at 916-227-8705.

Table 1 - Scour Data

Support Location	Long Term Scour (Degradation and Contraction) *Depth* (ft)	Short Term Scour (Local) Depth (ft)
Abutment 1	N/A	N/A
Pier 2	3.7	19.1
Abutment 3	N/A	N/A

Corrosion

Corrosion test results for soils samples collected from Boring 02-2 are shown in Table 2 below. All of the soil samples tested are considered non-corrosive by current Caltrans standards.

Table 2 - Corrosion Test Summary

Sample Location	pH	Minimum Resistivity (Ohm-Cm)	Chloride Content (ppm)	Sulfate Content (ppm)
Elev. 3430-3409 ft	7.31	2950	-	-
Elev. 3409-3390 ft	8.16	2200	-	-
Elev. 3390-3370 ft	9.39	2500	-	-
Elev. 3370-3340 ft	8.93	1900	-	-
Elev. 3340-3290 ft	8.86	2400	-	-

Note: Caltrans currently defines a corrosive environment as an area where the soil has either a chloride concentration of 500 ppm or greater, a sulfate concentration of 2000 ppm or greater, or has a pH of 5.5 or less. With the exception of MSE walls, soil and water are not tested for chlorides and sulfates if the minimum resistivity is greater than 1,000 ohm-cm.

Fault and Seismic Data

The structure site is potentially subject to ground motions from nearby earthquake sources during the design life of the new structure. Based on the 2012 Caltrans Fault Database (V2.2.06) for the deterministic procedure, the controlling fault for the site is the San Andreas (San Bernardino South) fault zone (Fault ID: 325, strike-slip, dip=90 deg.) with a maximum moment magnitude (M_{Max}) of 7.9, located approximately 3.9 miles (6.2 km) southwest of the site. The probabilistic response spectrum is obtained for the 5% probability of exceedance in 50 years from the 2008 USGS National Seismic Hazard Map.

Based on the 2002 Log of Test Borings and general geology at the site, the average shear wave velocity for the upper 100 ft of the subsurface materials is estimated as V_{S30} = 1150 ft/s (350 m/s).

For this site, the design response spectrum is controlled solely by the probabilistic response spectrum, with a Peak Ground Acceleration (PGA) of 0.97g.

Surface Rupture Potential

The site is not located within an Alquist Priolo Earthquake Fault Zone or an unzoned fault with an age of Holocene or younger, therefore, the potential for surface fault rupture hazard is low.

Liquefaction Potential

Due to the dense nature of the alluvial material underlying the site and the groundwater elevation, the liquefaction potential at the site is anticipated to be low. For site specific seismic data and design recommendations mentioned above, refer to the memorandum concerning seismic design recommendations (dated November 27, 2013), by Asef Wardak (916-227-1219) of the Office of Geotechnical Design South 2 (OGDS2).

Foundation Recommendations

The following Foundation Recommendations are for the Miner's Shack Creek Bridge (Replacement), Br. No. 54-1219, as shown on General Plan, dated November 21, 2013.

At the abutment support locations, driven "Modified" Class 140, Alternative "W" steel pipe piles are recommended to be used for support. The "Modified" Class 140, Alternative "W" steel pipe pile shall be specified with a circular steel plate or conical steel tip welded to the bottom of the pile, similar to the tip detail of Alternative "V" pipe pile as shown in the 2010 Standard Plans. Spread footings are not feasible at this location due to the presence of potential loose to medium dense granular soil and high PGA (MTD 5-1). CIDH piles are also not recommended at the abutments due to the need for battered piles.

At the Pier 2 support location, large diameter (84-inch) Type-II CIDH piles are recommended for support. At the Pier 2 support location, spread footings and small diameter piles are not feasible due to the large local scour potential within the channel and the anticipated structural loads.

The foundation design at the abutment locations was based on working stress design (WSD) and Load Resistance Factored Design (LRFD) was used at the pier location. The design information shown in Table 3 & 4 was provided to our office by Structure Design, Branch 9 on May 8, 2013 and supplemented on December 13, 2013.

Table 3 - Foundation Data Provided by Structure Design

Support Location	Design Method	Pile Type	Finished Grade Elevation (ft)	Pile Cut-Off Elevation (ft)	Pile Cap Size (ft)		Permissible Settlement under Service Load (in)	Number of Piles per Support
					B	L		
Abut 1	WSD	Modified Class 140 Alt W (PP 14 x 0.438)	3454.5	3452.92	7	43	1	14
Pier 2	LRFD	84 in CIDH	3428.5	3422.0	N/A	N/A	1	2
Abut 3	WSD	Modified Class 140 Alt W (PP 14 x 0.438)	3464.0	3462.42	7	43	1	14

Table 4 - Foundation Design Loads Provided by Structure Design

Support Location	Service-I Limit State (kips)			Strength Limit State (Controlling Group, kips)				Extreme Limit State (Controlling Group, kips)			
	Total Load		Perm. Loads	Compression		Tension		Compression		Tension	
	Per Support	Max. Per Pile	Per Support	Per Support	Max. Per Pile	Per Support	Max. Per Pile	Per Support	Max. Per Pile	Per Support	Max. Per Pile
Abut 1	1550	140	1230	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A
Pier 2	4200	N/A	2320	6060	3030	N/A	0	2320	4050	N/A	-1430
Abut 3	1550	140	1230	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A

The specific pile tip elevations for Abutments 1 and 3, and Pier 2 are shown below in Table 5 and Table 6.

Table 5 - Foundation Design Recommendations for Abutments 1 and 3

Support Location	Pile Type	Cut-off Elevation (ft)	LRFD Service-I Limit State Load per Support (kips)		LRFD Service-I Limit State Total Load per Pile (kips)	Nominal Resistance (kips)	Design Tip Elevation (ft)	Specified Tip Elevation (ft)	Nominal Driving Resistance Required (kips)
			Total	Perm.					
Abut 1	Modified Class 140 Alt W (PP 14 x 0.438)	3452.92	1550	1230	140	280	3432.8 (a)	3432.8	280
Abut 3	Modified Class 140 Alt W (PP 14 x 0.438)	3462.42	1550	1230	140	280	3442.5 (a)	3442.5	280

Note: 1) Design tip elevations are controlled by: (a) Compression
2) "Modified" Class 140, Alternative "W" pipe pile is to be driven with either a flat or conical steel tip welded to the pile tip.

Table 6 - Foundation Design Recommendations for Pier 2

Support Location	Pile Type	Cut-Off Elev. (ft)	Service-I Limit State Load per Support (kips)	Total Permissible Support Settlement (inches)	Required Nominal Resistance (R_n) (kips)				Design Tip Elev. (ft)	Specified Tip Elev. (ft)
					Strength Limit		Extreme Limit			
					Comp. (ϕ=0.7)	Tension (ϕ=0.7)	Comp. (ϕ=1.0)	Tension (ϕ=1.0)		
Pier 2	84 in CIDH	3422.0	4200	1	4330	0	4050	1430	3353.0 (a-I) 3360.0 (a-II) 3382.0 (b-II)	3353.0

Note: Design tip elevations are controlled by: (a-I) Compression (Strength), (a-II) Compression (Extreme), (b-II) Tension (Extreme)

The Pile Data Table is presented in Table 7, below. The ultimate geotechnical pile capacity of the CIDH piles will meet or exceed the required nominal resistance in compression.

Table 7 - Pile Data Table

Support Location	Pile Type	Nominal Resistance (R_n) (kips)		Design Tip Elev. (ft)	Specified Tip Elev. (ft)	Nominal Driving Resistance (kips)
		Compression	Tension			
Abut 1	Modified Class 140 Alt W (PP 14 x 0.438)	280	0	3432.8 (a)	3432.8	280
Pier 2	84 in CIDH	4330	1430	3353.0 (a) 3382.0 (b)	3353.0	N/A
Abut 3	Modified Class 140 Alt W (PP 14 x 0.438)	280	0	3442.5 (a)	3442.5	280

Note: 1) Design tip elevations are controlled by: (a) Compression, (b) Tension
2) "Modified" Class 140, Alternative "W" pipe pile is to be driven with either a flat or conical steel tip welded to the pile tip.

At the Pier 2 pile location, a Type II shaft with an optional construction joint is shown on the structure plans. If the optional construction joint is used, then the permanent steel casing used must be a corrugated metal pipe CMP because it will be located within the CIDH skin friction zone. For the CIDH piles, the skin friction was utilized from the construction joint elevation down to a distance within 4ft above the specified pile tip elevation listed in Table 7. Due to the anticipated use of slurry displacement methods to construct the pile, no end bearing was considered. The skin friction zone is listed in Table 9 in the construction considerations section of this report. All foundation design loads and corresponding limit states (listed in Table 4) were considered in developing the foundation design recommendations provided in Tables 5 & 6.

At the abutment locations, "drilling" to assist driving (Standard Specifications 49-2.01C(3)) will be required to minimize erratic driving and to advance the pile down to the specified pile tip elevation listed in Table 7. The design of the piles was based predominantly on end bearing with some skin friction. The nominal axial structural resistance for the driven steel pipe piles will meet or exceed the required nominal resistance in compression.

General Notes:

1) The structure engineer shall show on the plans, in the pile data table, the minimum pile design tip elevation required to meet the lateral load demands. If the specified pile tip elevation required to meet lateral load demands exceed the specified pile tip elevation given within this report, the Office of Geotechnical Design South 2, Branch B should be contacted for further recommendations.

2) "Modified" Class 140, Alternative "W" steel pipe pile details are to be shown on the project plans. The "Modified" Class 140, Alternative "W" steel pipe piles shall be shown with a circular steel plate or conical steel tip with a minimum thickness of ¾ inch welded to the pile tip, similar to the Alternative "V" pile tip detail shown in the 2010 Standard Plans.

3) The special provisions shall specify the requirements of Tunnel Safety Orders for the CIDH shaft work that meets the definition of a tunnel or shaft as described in the Highway Design Manual, Section 110.12 "Tunnel Safety Orders."

Construction Considerations:

- **Rock Core Samples and Laboratory Testing**

1) Rock core samples from the 2002 Caltrans foundation investigation are available for viewing by bidders at the Geotechnical Services (Transportation Laboratory), Department of Transportation, 5900 Folsom Blvd., Sacramento, CA. Specifically, only Boring B-02-2 was retained for viewing. Potential bidders are to allow the State seven (7) days to prepare and display the rock cores.

- **Driven Piles**

1) At Abutments 1 & 2, "Drilling" to assist driving, per Standard Specification Section 49-2.01C(3), will be required to minimize erratic pile driving and to advance the piles to specified pile tip elevation. The bottom of the "drilled" hole elevation is shown below in Table 8.

Table 8: "Drilling" to Assist Driving for Abutment 1 & 3

Location	Bottom of Drilled Hole Elevation (ft)
Abutment 1	3442.8
Abutment 3	3452.5

2) At the abutment locations, the contractor should anticipate hard driving due to the physical characteristic of the very dense alluvium overly the sedimentary rock as shown in the Log of Test Boring sheets.

3) Pile acceptance is to be based on Standard Specifications 49-2.01A(4)(b) "Pile Driving Acceptance Criteria". At Abutments 1 and 3 support locations, any pile that achieves 1½ times the required nominal resistance in compression, as shown on the contract plans, within 6 feet of the specified pile tip elevation, may be considered satisfactory and cut off with written approval from the engineer. (e.g. 1½ times the required nominal resistance in compression will be 420 kips at Abutment 1 & 3).

- **CIDH Piles**

1) During the 2002 foundation investigation, groundwater was encountered at the bridge site. Due to the location of Pier 2 being within the channel, the contractor should anticipate that surface and/or groundwater will be encountered during the construction of the CIDH piles. Groundwater elevations at this bridge site are subject to seasonal fluctuations and will be encountered at higher or lower elevations depending on seasonal precipitation rates leading up to and at the time of construction. For more details refer to the site geology and groundwater section of this report, and the Log of Test Boring Sheets.

2) Groundwater is expected to easily permeate through the alluvial soils overlying the rock. It is also anticipated that groundwater will also permeate through the upper portions of the rock due to the weakly cemented and granular nature of the rock. Temporary casing extended into the rock may not eliminate groundwater entering the CIDH pile excavation.

3) At the Pier 2, it is anticipated that groundwater and/or surface water will be encountered and will require concrete placement of CIDH piles using slurry displacement methods. If the CIDH piles are to be constructed using slurry displacement method, the slurry shall consist of mineral or synthetic slurry. Use of water shall not be allowed as slurry.

4) The 2002 subsurface investigation identified medium dense to very dense alluvium, which in general consisted of sand with gravel and localized hard cobbles overlying moderately to weakly cemented, sedimentary rock with soil-like characteristics. At the pier pile locations, the contractor will need to excavate through these earth materials to construct the shaft. Caving conditions are anticipated in both the alluvium and within the weakly cemented zones of the rock. The amount of caving the contractor will experience will be dependent upon the methods and means the contractor chooses to use to construct the CIDH piles. Temporary casing may be necessary to control caving during construction. All temporary casing is to be removed during concrete placement.

5) The calculated geotechnical capacity of all CIDH piles is based on skin friction only and no end-bearing was considered. For the CIDH piles, the skin friction zone used to develop the geotechnical capacity is from the construction joint elevation down to a location 4 ft above the specified pile tip elevation. The skin friction zone of the CIDH piles is summarized below in Table 9.

Table 9: CIDH Pile Skin Friction Zone Elevations

Location *	**Skin Friction Zone Start Elevation (ft)**	**Skin Friction Zone End Elevation (ft)**
Pier 2, Column 1 & 2	3410	***3357***

* Pier columns are numbered from 1 to 2 (from left to right) looking up-station.

6) At the Pier 2, Type II CIDH piles are specified with an optional construction joint that is shown on the contract plans. If the contractor chooses to construct the optional construction joint, then it shall be in conformance to Standard Specifications 49-3.02C(6) and possibly 49-3.02C(5). Because of the soil-like characteristics of the rock, any permanent casing used must extend 5 feet below the optional construction joint shown on the plans. If the contractor chooses to construct the optional construction joint, the permanent steel casing must consist of a corrugated metal pipe (CMP) since it is located within the skin friction zone listed in Table 9 above.

This Foundation Report is based on specific project information that has been provided by the Office of Structure Design - West, Design Branch 9. If any conceptual changes are made during final project design, the Office of Geotechnical Design-South 2, Design Branch B should review those changes to determine if this report is still applicable. Any questions regarding the above report should be directed to the attention of Hector Valencia, (916) 227-4555, or Mark DeSalvatore, (916) 227-5391 at the Office of Geotechnical Design-South 2, Branch B.

Prepared by: Date: 02-04-14

Hector Valencia, P.E., #C65257
Engineering Geologist
Office of Geotechnical Design-South 2
Design Branch B

cc: Mohammad Mollazadeh- Dist. 8 Project Manager
Bruce Kean – Dist 8 Materials Engineer
Structures Construction - R.E. Pending File
Ofelia Alcantara - Structures Office Engineer (Specs)
Abbas Abghari – OGDS2
Mark DeSalvatore – OGDS2
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To: Gorden Dankee
Branch Chief
Bridge Design Branch 9
Office of Structure Design

Date: November 05, 2013
File: 08-SBD-138-R16.8, R18.7
Hog Ranch Creek Bridge
EA 08-437001

Attn: Pete Norboe

From: **Jimmie Pallares**
Structures Hydraulic Engineer
Structure Design Services
Structure Hydraulics and Hydrology
MS #9-HYD-1/2i

Subject: Revised Final Hydraulic Report for the Hog Ranch Creek Bridge Project.

This memorandum report is a revision to the original final hydraulic report dated March 18, 2003 for the Hog Ranch Creek Bridge project, which includes three new bridge structures. The three structures are Hog Ranch Creek Bridge (bridge number 54-1218), Miners Shack Creek Bridge (bridge number 54-1219), and Double Drain Creek Bridge (bridge number 54-1220).

Hydrology & Hydraulics Report

General

It is proposed to construct three new bridges on the new State Route138 alignment. The new alignment is located north of the existing SR 138 and is approximately 1.7 miles east of the State Route 138/15 Seperation. The new SR 138 will reduce traffic delays, congestion, and improve safety. The new Hog Ranch Creek, Miner's Shack Creek, and Double Drain Creek will be a 2-lanes prestressed box girders and will allow wildlife under crossings.

Hog Ranch Creek Bridge (Br. No. 54-1218, 08-SBd-138-PM R16.8)

The 125-foot long by 42-foot wide by 5-foot deep single span bridge will replace the existing 3-foot diameter corrugated metal pipe culver on the existing SR 138. The fill at the culvert site will be removed to allow unimpeded flow of Hog Ranch Creek.

Miner's Shack Creek Bridge (Br. No. 54-1219, 08-SBd-138-PM R17.2)

The dimensions of the double span bridge are 225-feet long by 42-feet wide by 4.4-feet deep. The bent cap will consist of two 5.5-feet diameter columns that are placed on 6.5-feet diameter CIDH piles. The columns will be placed within the Miner's Shack Creek channel bed. There is a 12 degree hydraulic skew between the columns and the stream.

Double Drain Creek Bridge (Br. No. 54-1220,, 08-SBd-138-PM R17.8)

The dimensions of the four span bridge are 385.5-feet long by 42.2-feet wide by 6.1-feet deep. The bent cap will consist of two 7-feet diameter columns that are placed on 8-feet diameter CIDH piles.

This final hydraulic report makes references to data found in the Federal Emergency Management Agency Flood Insurance Study titled "SAN BERNARDINO, CALIFORNIA" volume 1 through 4, dated January 17, 1997; Environmental Protection Agency website at www.epa.gov

Note: All calculated elevations in this report are based on the Microstation files MSC Layout(1-27).dgn, HRCLayout.dgn, and DDC Layout(1-22).dgn Please verify datum references on the new bridge layouts and make elevations adjustments as required.

Basin

The bridge sites are located within the Mojave Watershed (USGS Hydrologic Unit Code 18090208) which is part of the South Lahontan Hydrologic Region. The project site is located in the San Bernardino Mountains adjacent to the Cajon Summit.

The before mentioned FEMA Flood Insurance Study states that the San Bernardino Mountain resort and residential areas located in the canyons experience major problems of high velocity flood flows in the steep channels, which results in the occurrence of extreme amounts of erosion and deposition.

The area upstream of the project site is undeveloped with poor ground cover typical of the area. During the spring months, the ground cover is good with grasses and buck brush. During the rest of the year the ground is typically dry with little ground cover. The U.S. Forest Service controls most of the land with no consideration given for development. The table below list basin areas for the streams.

Stream	Basin Area (acres)
Hog Ranch Creek	358
Miner's Shack Creek	289
Double Drain Creek	198

The annual rainfall for the project site ranges from 14 to 20 inches.

Streambed

The riverbeds are composed of silty sand with gravel and cobbles, fine to coarse-grained sand, and fine-grained sub angular to angular gravel round cobbles. Also, the bed has organics (alluvium).

Discharge

The discharges for the bridges site were computed using Regional Regression Method in the U.S.G.S. Water-Resources Investigations Manual 77-21 and the standard statistical procedures as specified by the Water Resource Council Bulletin 17B for flood frequency analysis.

The table below lists the 100-year and 50-year flowrate events for the streams.

Stream	Q_{100} (cfs)	Q_{50} (cfs)
Hog Ranch Creek	717	473
Miner's Shack Creek	611	406
Double Drain Creek	473	318

Velocity and Stage

HEC-RAS Version 4.1, a one-dimensional, step backwater, computer program developed by the U.S. Army Corps of Engineers, was used to model the bridge sites and its contributing waterways. This program estimates the flood velocities and stages for any given return period flood events. The model was based on the mentioned Micro Station files. Based on photographs the Manning's roughness coefficient of 0.025 was applied on the main channel and over banks. The table below list velocities and stage values for the streams.

Stream	Average V_{100} (ft/s)	Q_{100} Stage Elevation (ft)	Q_{50} Stage Elevation (ft)
Hog Ranch Creek	17	3361	3361
Miner's Shack Creek	12.6	3432	3431
Double Drain Creek	11.7	3529	3529

Scour and Channel Degradation

The stream channels are very unstable. High local pier scour and channel degradation potential are susceptible due to high velocities and channel bed materials. The potential for thalweg migration is high. The table below list the potential scours depths for the streams.

Stream	Potential Scour Depth (ft)
Hog Ranch Creek	6.9
Miner's Shack Creek	16.1
Double Drain Creek	25.6

Drift

Photographs and type of vegetation indicate that there is a small potential for debris at the project site.

Minimum Soffit Elevation

The freeboard distance is measured from the lowest bridge soffit elevation to the l00-year water surface elevation. The table below lists freeboard distances for the streams.

Stream	Freeboard (ft)
Hog Ranch Creek	18
Miner's Shack Creek	35.1
Double Drain Creek	52.8

There are no freeboard distance concerns.

Bank Protection

Due to the degradable characteristics of the site soils Structure Hydraulics recommends Rock Slope Protection (RSP) be placed at both abutments for Miner's Shack Creek and Double Drain Creek. The new rock slope protection (RSP) will be designed by District. The velocities have been provided to assist the District hydraulics engineers in the design of bank protection.

Summary Information for the Bridge Designer

Below is a summary of key design parameters based on the hydrology and hydraulic analysis performed for each structure:

Hog Ranch Creek Bridge

Minimum Soffit Elevation	3364.2 feet
Potential Scour Elevation at Piers/Abutment	3351.7 feet
Required Waterway	42 feet^2
Average Velocity	17.4 feet/s

HYDROLOGIC SUMMARY			
Drainage Area: 358 acres			
	Design Flood	Base Flood	Overtopping
Frequency (yrs)	50	100	>>500
Discharge (cfs)	473	717	
Water Surface Elevation at Bridge (ft)	3360.6	3361	3382
Flood plain data are based upon information available when the plans were prepared and are shown to meet federal requirements. The accuracy of said information is not warranted by the State and interested of affected parties should make their investigation.			

Miner's Shack Creek Bridge

Minimum Soffit Elevation	3437 feet
Potential Scour Elevation at Piers/Abutment	3414.4 feet
Required Waterway	48.4 feet^2
Average Velocity	12.5 feet/s

HYDROLOGIC SUMMARY			
Drainage Area: 358 acres			
	Design Flood	Base Flood	Overtopping
Frequency (yrs)	50	100	>>500
Discharge (cfs)	406	611	
Water Surface Elevation at Bridge (ft)	3431	3433.7	3469.5
Flood plain data are based upon information available when the plans were prepared and are shown to meet federal requirements. The accuracy of said information is not warranted by the State and interested of affected parties should make their investigation.			

Double Drain Creek Bridge

Minimum Soffit Elevation	3532.5 feet
Potential Scour Elevation at Piers/Abutment	3501.6 feet
Required Waterway	42 $feet^2$
Average Velocity	11.8 feet/s

HYDROLOGIC SUMMARY			
Drainage Area: 358 acres			
	Design Flood	Base Flood	Overtopping
Frequency (yrs)	50	100	>>500
Discharge (cfs)	318	473	
Water Surface Elevation at Bridge (ft)	3528.9	3529.2	3586.5
Flood plain data are based upon information available when the plans were prepared and are shown to meet federal requirements. The accuracy of said information is not warranted by the State and interested of affected parties should make their investigation.			

State of California Business, Transportation and Housing Agency

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M e m o r a n d u m

To: MR. GORDON DANKE, BRANCH CHIEF
OFFICE OF BRIDGE DESIGN - West
DESIGN BRANCH 9

Date: November 27, 2013

File: 08-SBd-138-PM-R17.2
EA No. 08-0Q300G
Project No. 0800020191
Hog Ranch Creek Bridge
Bridge No. 54-1218

Attn.: Mr. Pete Norboe

From: **DEPARTMENT OF TRANSPORTATION**
DIVISION OF ENGINEERING SERVICES
GEOTECHNICAL SERVICES - MS #5
OFFICE OF GEOTECHNICAL DESIGN SOUTH-2

Subject: **Seismic Design Recommendations for Hog Ranch Creek Bridge**

Introduction

This memorandum presents seismic design recommendations for the Hog Ranch Creek Bridge 54-1218. Ground motion recommendations are based on the Caltrans 2009 Seismic Design Procedure (SDP) as described in the Seismic Design Criteria Version 1.7 (SDC) Appendix B, the Acceleration Response Spectrum (ARS) Online Tool v2.2.06, USGS 2008 Interactive Deaggregations (Beta), and 2002 Log of Test Boring (LOTB) for geotechnical subsurface investigations.

Seismicity

Based on the 2012 Caltrans faults database and Caltrans ARS Online (v2.2.06) Tool, the site is located about 3.5 miles (5.6 km) from the San Andreas (San Bernardino South) fault. This fault (Fault ID 325, MMax = 7.9, strike-slip, dip = 90 degrees, vertical, Bottom and Top of Rupture Plane approximately 7.9 and 0 miles, respectively) is the controlling fault for the deterministic seismic design procedure. A map showing the location of the bridge and the nearby faults is attached in Figure 1.

Soil Profile

The Office of Geotechnical Design South-2 had performed subsurface geotechnical investigations from August 13 through August 14 in 2002.

This subsurface geotechnical exploration included two (2) 4-inch diameter rotary wash borings. The maximum depth of the investigation was approximate 120 feet below the existing ground surface.

The subsurface condition at the Hog Ranch Creek bridge site is composed young Quaternary Alluvium consists of medium dense to dense, sitly sand with gravel interbedded with layers of poorly graded sand with gravel and cobbles. The young Quaternary alluvium extends from the ground surface at an approximate elevation of 3390.6 feet to an approximate elevation of 3383.5 feet. Underlying the young Quaternary Alluvium is the old Quaternary Alluvium. It is composed of interbedded layers with various thicknesses of very dense fine silts to silty sands, fine to coarse sands, gravelly sands, with discontinuous lenses of cobbles. The old Quaternary Alluvium extended from an approximate elevation of 3390.6 feet to a maximum elevation of 3270 feet. Groundwater was measured on November 13, 2002 at elevation 3334 feet. Please refer to Foundation Report (FR) prepared by the Office of Geotechnical Design South 2, Branch B (OGDS-2B) for additional information.

Per 2002's LOTBs and general geology at this site, the average shear wave velocity for the upper 100 feet of subsurface materials is estimated as V_{S30} = 320 m/s, , based on the correlation of Shear wave as function of standard penetration test resistance and vertical effective stress by Brandenberg, S.J., Bellana, N. and Shantz, T., 2010 UCLA.

Design Response Spectrum

Based on the 2009 SDP, the design response spectrum is the upper envelope of the deterministic and probabilistic response, but is not less than a minimum deterministic response spectrum resulting from a Mmax = 6.5 earthquake on a vertical strike-slip fault at a distance of 7.5 miles (12 km).

The deterministic response spectrum is obtained by taking the arithmetic average of the median response spectrum calculated using the 2008 Campbell-Bozorgnia and 2008 Chiou-Youngs ground motion prediction equations. The probabilistic response spectrum is obtained for 5 percent probability of exceedance in 50 years (corresponding to 975 year return period).

For this site, the design response spectrum is controlled by probabilistic response spectrum. The corresponding peak horizontal ground acceleration at proposed site is 1.0g. The recommended acceleration response spectrum is attached in Figure 2.

Liquefaction Potential Evaluation

Based on the 2002 LOTBs, the soil below groundwater at Hog Ranch Creek Bridge may be prone to liquefaction due to strong ground shaking. It must be noted that this liquefaction analysis is preliminary based on the available limited soil information. Field and laboratory geotechnical investigations are necessary to determine liquefaction potential and seismic settlement at supports locations.

Surface Fault Rupture Hazard

The proposed site is not within an Alquist-Priolo Earthquake Fault Zone or an unzoned faults (Holocene or younger in age). The site is more than 1.6 miles (2.6 km) from the nearest active fault (Caltrans faults database 2012), Cleghorn fault zone (Southern Cleghorn section), which extends to the ground surface. Potential for surface rupture is low, and no further work or design for surface rupture is required.

The seismic design recommendations contained in this report are based on specific project information that was provided by the Office of Bridge Design West. If any conceptual changes are made during final project design, the Office of Geotechnical Design South-2 should review those changes to determine whether the seismic design recommendations provided in this report are still applicable. Any questions regarding the above recommendations should be directed to the attention of Asef Wardak (916) 227-1219 or Angel Perez-Cobo (916) 227-7167, Office of Geotechnical Design South-2.

Prepared by: Date: 11-27-2013

Awardak.

REGISTERED PROFESSIONAL ENGINEER
ASEF WARDAK
No. C 61960
Exp. 9/30/2015
CIVIL
STATE OF CALIFORNIA

Asef Wardak
Transportation Engineer
Office of Geotechnical Design South-2

Attachments:

Figure 1. Nearby Major Faults with Reference to the Project Site
Figure 2. Recommended Acceleration Response Spectrum (ARS) Curve

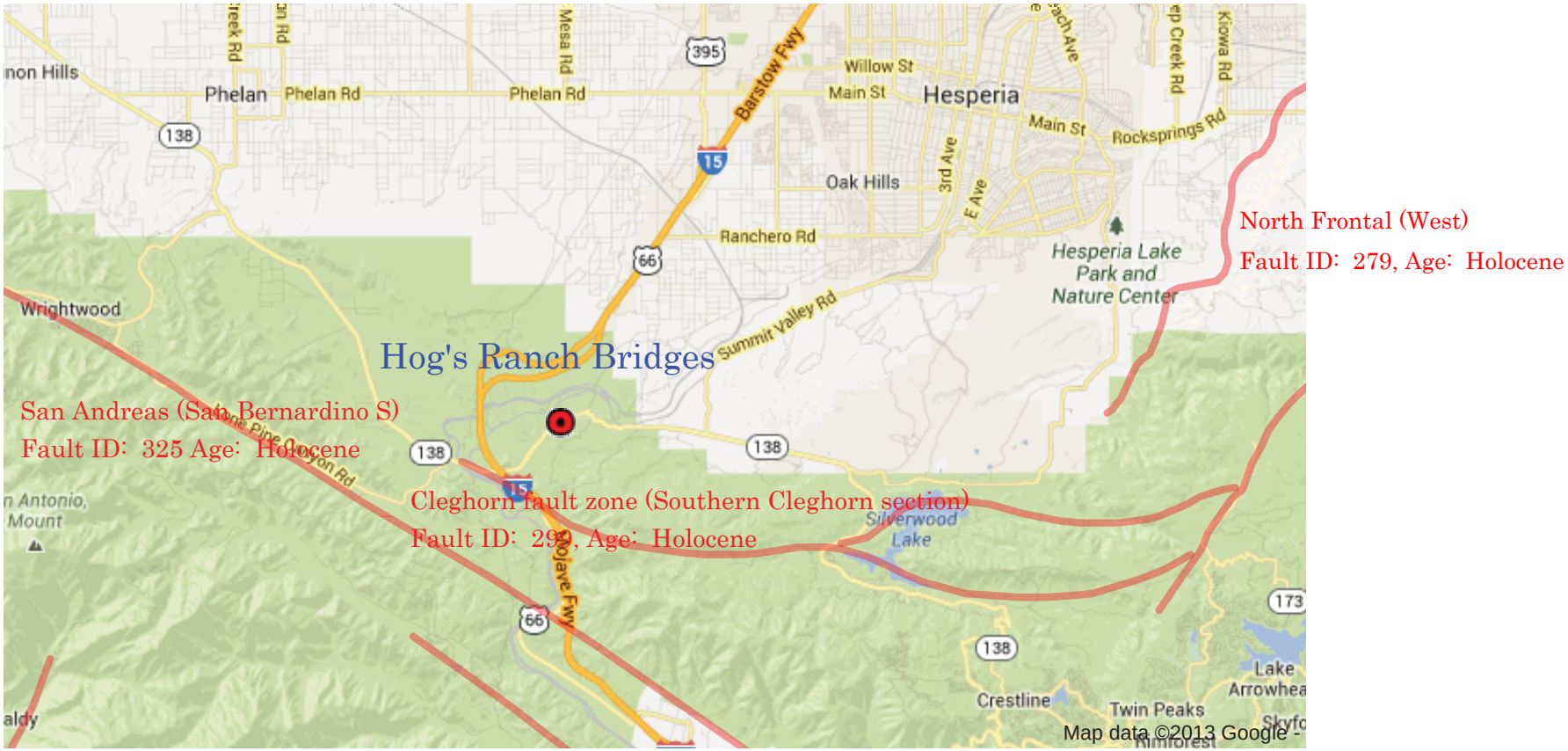
cc: District 08 Structure Construction R.E. – Pending File
District 8 – Project Manager – Jason Bennecke (Electronic File)
Specs & Estimates – Ofelia Alcantara (Electronic File)
District 8, District Materials Engineer – Bruce Kean (Electronic File)
HQ Geotechnical Design South-2 – Abbas Abghari – OGDS-2 (Electronic File)
HQ Geotechnical Design South-2 – Mark Desalvatore – GDS-2 (Electronic File)
HQ Geotechnical Design South-2 – Angel Perez-Cobo (Electronic File) APC
HQ Geotechnical Design South-2 – Erich Neupert – GDS-2 (Electronic File)
HQ Geotechnical Design South-2 – David Liao – GDS-2 (Electronic File)
HQ Geotechnical Design South-2 – Cipriano Manansala – GDS-2 (Electronic File)
HQ Geotechnical Archive

CALIFORNIA DEPARTMENT OF
TRANSPORTATION

Caltrans ARS Online (v2.2.06)

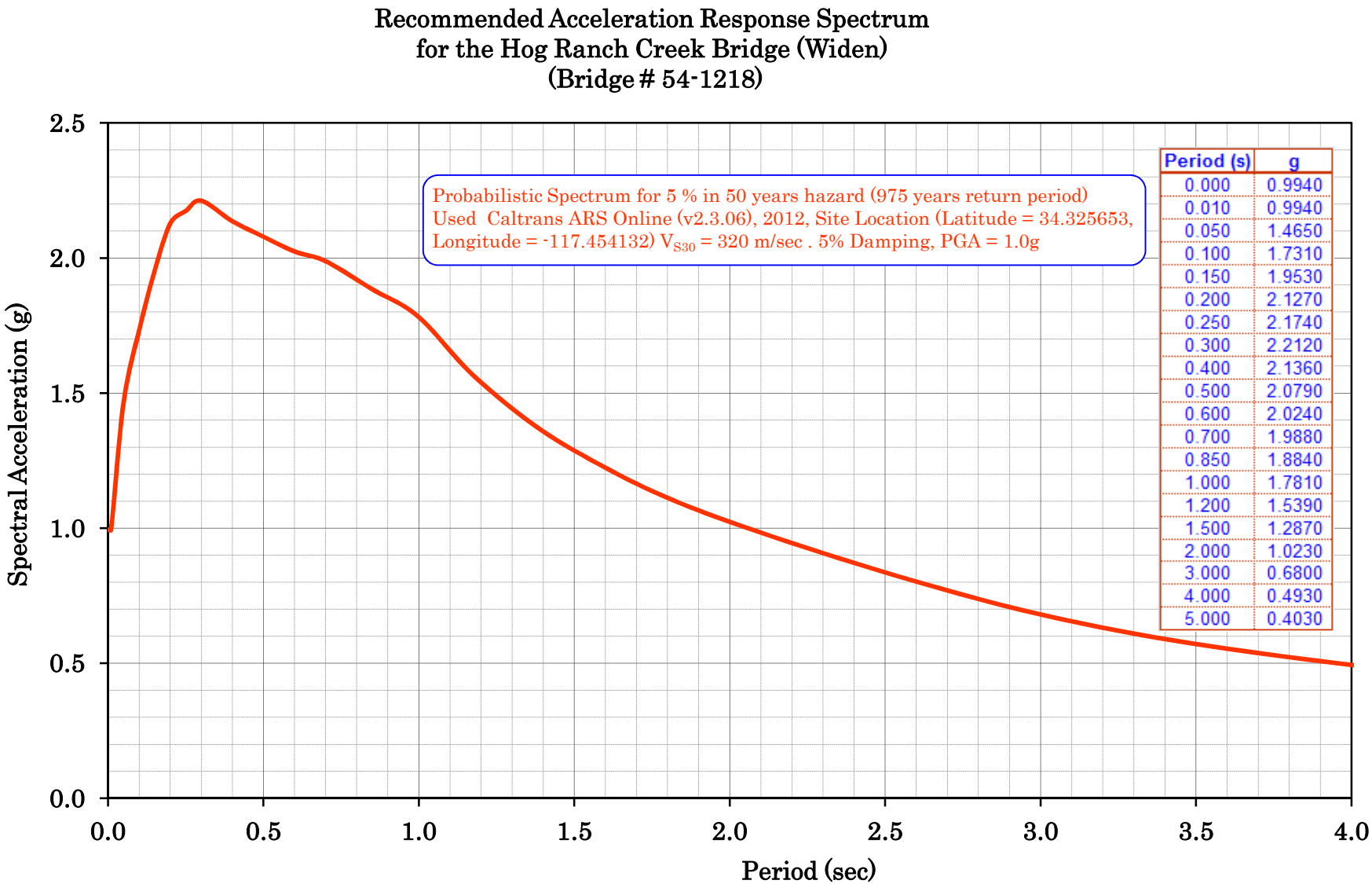
This web-based tool calculates both deterministic and probabilistic acceleration response spectra for any location in California based on criteria provided in *Appendix B of Caltrans Seismic Design Criteria*. More...

SELECT SITE LOCATION



Latitude: 34.325653 **Longitude:** -117.454132 ***Vs30*:** 320 m/s Calculate

Figure 1. Project site and the nearby faults is attached



Period (s)	g
0.000	0.9940
0.010	0.9940
0.050	1.4650
0.100	1.7310
0.150	1.9530
0.200	2.1270
0.250	2.1740
0.300	2.2120
0.400	2.1360
0.500	2.0790
0.600	2.0240
0.700	1.9880
0.850	1.8840
1.000	1.7810
1.200	1.5390
1.500	1.2870
2.000	1.0230
3.000	0.6800
4.000	0.4930
5.000	0.4030

Figure 2. Recommended Acceleration Response Spectrum (ARS) Curve

State of California

Business, Transportation and Housing Agency

Memorandum

Flex your power!
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MR. GORDON DANKE, Chief
Senior Bridge Engineer
Office of Structure Design-West
Bridge Design Branch 9

Date: December 23, 2013

File: 08-SBD-138- PM R17.2
0800020191
08-0Q3001
Hog Ranch Creek Bridge
Br. #54-1218

Attention: Pete Norboe

From: DEPARTMENT OF TRANSPORTATION
Division of Engineering Services
Geotechnical Services
Office of Geotechnical Design – South 2
Design Branch B

Subject: Foundation Recommendation for Hog Ranch Creek Bridge

Introduction

This report presents the foundation recommendations for the proposed Hog Ranch Creek Bridge (Br. #54-1218) which is associated with the newly realign Route 138. The proposed bridge was originally designed in 2003 in metric using Working Stress Design (WSD) for the abutments. The project was put on hold for many years, and now it is being reactivated. The bridge is being updated to the current AASHTO LRFD bridge specifications with California Amendments and the plans being updated from metric to english units. This foundation report supersedes the previously generated Revised Foundation Recommendations, dated October 28, 2003 and any other foundation reports generated for this structure.

The recommendations provided below are based on a review of the Revised Foundation Recommendations in October 2003, Log of Test Borings (LOTB) in 2002, General Plan dated December 4, 2013, the Foundation Plan dated December 17, 2013, hydraulics data emailed on November 18 & 20, 2013, foundation data emailed on May 8 and Dec 18, 2013. After reviewing the 2002 LOTB's, our office determined that the subsurface data was sufficient to develop foundation recommendations

The elevations shown on the Log of Test Borings are based on the NGVD 1929 vertical datum. For subsurface data and boring locations, please refer to the LOTB sheets.

Project/Site Description

The project site is located in the San Bernardino Mountains near Cajon Pass on Route 138 at Post mile 17.2. The bridge is approximately 1.3 miles northeast from the intersection of Interstate 15 and State Route 138. This proposed bridge will replace the existing 20 and 40 inch corrugated culvert pipe and embankment, and will provide a much larger opening for water flows and possible animal passage. The proposed new bridge is to be a single-span, cast in

place, prestressed concrete box girder structure with seat type abutments which is approximately 147.5 ft long and 42.8 ft wide.

Geology

The "Geologic Map of the Cajon Quadrangle" (Dibblee and Minch, 2003), shows that the bridge site is underlain by Quaternary alluvial deposits over Late Cenozoic (Pliocene) Crowder Formation which is described as a pebbly sandstone. The subsurface conditions were explored in August 2002 by drilling 2 mud rotary sample borings 02-1 and 02-2 located near the proposed support locations. The borings were advanced using a fully cased wire line drilling system. Standard Penetration Tests were performed at regular intervals in the soil during the subsurface investigation. Soil samples were collected for corrosion testing.

The subsurface investigation revealed that the soils underlying the site consist of dense to very dense silty sand with gravel, poorly graded sand with gravel and silty gravel with sand and all 3 units contained scattered cobbles. The alluvium deposits extend to approximate depths of 77 ft and 91 ft (elev 3314 ft and 3300 ft respectively) below the ground surface. Below the alluvium deposits in boring 02-1, the bedrock was very intensely weathered, weakly to moderately cemented sandstone consisting of a very dense, silty sand to poorly graded sand with gravel and cobbles. In boring 02-1, the rock extended down to the maximum explored depth of 154 ft (elev 3237ft). In boring 02-2, rock was not encountered down to the maximum explored depth. For more details, refer to the LOTB sheets.

According to the Revised Foundation Recommendation dated October 10, 2003, boulders were not encountered during the subsurface investigation but were visible and scattered within the slopes of the wash from the terraces to the bottom of the wash. The boulders are hard, moderately weathered, subrounded to round, ranging in size from 1 to 3.5 ft.

Groundwater

Groundwater was measured in November 2002 at elevation of 3334.0 ft. in boring 02-1 as shown on the LOTB. Groundwater elevations are subject to seasonal fluctuations and may be encountered at higher or lower elevations depending on current conditions.

Scour Evaluation

The bridge site spans over an intermittent creek that has the potential for scour. Based on the structure hydraulics data emailed November 18, 2013, the long term degradation scour is estimated to be 3 feet and contraction and short term local scour was estimated to be 0 ft at the proposed bridge site. The total potential scour during the 75-year design life of the proposed structure is estimated at 3 feet below the existing channel grade. For details regarding the estimated scour, please contact Jimmie Pallares with the Structure Hydraulics and Hydrology at 916-227-8705.

Corrosion Evaluation

Corrosion test results for the soil samples collected from boring 02-2 are shown below in Table 2. The soil samples tested are considered non-corrosive by current Caltrans standards.

Table 2: Corrosion Test Summary

Location	Corrosion Number	Sample Depth (ft)	Minimum Resistivity (ohm-cm)	pH	Chloride Content (ppm)	Sulfate Content (ppm)
Boring 02-2	#02-0679	0-65	8000	7.95	N/A	N/A
Boring 02-2	#02-0680	65-100	4800	8.94	N/A	N/A

Note: Caltrans currently defines a corrosive environment as an area where the soil has a minimum resistivity of less than 1000 ohm-cm, and either contains greater than or equal to 500 ppm of chlorides, greater than or equal to 2000 ppm of sulfates, or has a pH of 5.5 or less.

Fault and Seismic Data

The structure site is potentially subject to ground motions from nearby earthquake sources during the design life of the new structure. For the deterministic procedure, the controlling fault for the site is the San Andreas Fault (San Bernardino S, Fault ID 325, strike-slip, dip=90°) with a maximum moment magnitude M_{max}=7.9, located approximately 3.5 miles southwest of the bridge site.

For the probabilistic procedure, a response spectrum was obtained for the 5 % probability of exceedance in 50 years from the 2008 USGS Seismic Hazard Map. Based on the 2013 foundation investigation, the average shear wave velocity of the upper 100 ft is estimated as V_{S30}=320 m/s. For this site, the probabilistic response spectrum controls, with the corresponding peak ground acceleration (PGA) estimated to be 1.0g.

Seismic Design Recommendation report has indicated that due to the dense nature of the local soils, the site is not susceptible to liquefaction. The bridge is located approximately 3.5 miles from the San Andreas (San Bernardino South) fault therefore; there is no potential for surface rupture.

For site specific seismic data and design recommendations, refer to the memorandum concerning seismic design recommendations (dated November 27, 2013), by Asef Wardak (916-227-1219) of the Office of Geotechnical Design South 2 (OGDS2)."

Foundation Recommendations

The following Foundation Recommendations for the proposed Hog Ranch Creek Bridge (Br. No. 54-1218) as shown on the General Plan dated December 4, 2013, are based on the structure design information provided to our office by Structure Design on May 8, 2013 and updated on December 18, 2013. This design information is summarized in Tables 3 and 4.

At the Abutment 1 and Abutment 2 support locations, driven "Modified" Class 140, Alternative "W" steel pipe piles are recommended to be used for support. The "Modified" Class 140, Alternative "W" steel pipe pile shall be specified with a circular steel plate or conical steel tip welded to the bottom of the pile, similar to the tip detail of Alternative "V" pipe pile as shown in the 2010 Standard Plans.

The Specified Pile Tip Elevations (SPTE) for driven closed end steel pipe piles are shown below in Table 5 and 6. The Specified Pile Tip Elevations will provide piles with an ultimate geotechnical capacity that will meet or exceed the required nominal resistance in compression.

Table 3: Foundation and Structure Information Provided by Structure Design

Support Location	Design Method	Pile Type	Finished Grade Elevation (ft)	Cut-off Elevation (ft)	Pile Cap Size (ft)		Permissible Settlement under Service Load (in)	Number of Piles per Support
					B	L		
Abut. 1	WSD	Modified Class 140 Alt W (PP 14 x 0.438)	3363.5	3361.92	7	43	1	22
Abut. 2	WSD	Modified Class 140 Alt W (PP 14 x 0.438)	3369.5	3367.92	7	43	1	22

Table 4: Abutment Foundation Design Loads Provided by Structure Design

Support Location	Service-I Limit State Load (kips)		
	Total Load		Permanent Load
	Per Support	Max. Per Pile	Per Support
Abut. 1	2090	140	1750
Abut. 2	2090	140	1750

Table 5: Abutment Foundations Design Recommendations

Support Location	Pile Type (b)	Cut-off Elevation (ft)	LRFD Service-I Limit State Load (kips) per Support		LRFD Service-I Limit State Total Load (kips) per Pile (Compression)	Nominal Resistance (kips)	Design Tip Elevations (ft)	Specified Tip Elevation (ft)	Nominal Driving Resistance Required (kips)
			Total	Permanent					
Abut 1	Modified Class 140 Alt W (PP 14 x 0.438)	3361.92	2090	1750	140	280	3337 (a)	3337	280
Abut 2	Modified Class 140 Alt W (PP 14 x 0.438)	3367.92	2090	1750	140	280	3337 (a)	3337	280

Note:
a) Design tip elevations are controlled by: Compression
b) "Modified" Class 140, Alternative "W" pipe pile is to be driven with either a flat or conical steel tip welded to the pile tip.

Table 6: Pile Data Table

Support Location	Pile Type (b)	Nominal Resistance (kips)		Design Tip Elevations (ft)	Specified Tip Elevation (ft)	Nominal Driving Resistance (kips)
		Compression	Tension			
Abut 1	Modified Class 140 Alt W (PP 14 x 0.438)	280	0	3337 (a)	3337	280
Abut 2	Modified Class 140 Alt W (PP 14 x 0.438)	280	0	3337 (a)	3337	280

Note:
a) Design tip elevations are controlled by: Compression
b) "Modified" Class 140, Alternative "W" pipe pile is to be driven with either a flat or conical steel tip welded to the pile tip.

General Notes

1) The"Modified" Class 140, Alternative "W" steel pipe piles shall be shown with a circular steel plate or conical steel tip with a minimum thickness of ¾ inch welded to the pile tip, similar to the Alternative "V" pile tip detail shown in the 2010 Standard Plans.

Construction Considerations

1. At Abutments 1 & 2, drilling to assist driving will be required to minimize erratic driving and to advance piles to the specified pile tip elevation (SPTE). The drilled holes shall be done in accordance with "Drilling", Standard Specification Section 49-2.01C(3). The bottom of the drilled hole elevation is shown on Table 7.

Table 7: "Drilling" to Assist Driving for Abutment 1 & 2

Location	Bottom of Drilled Hole Elevation (ft)
Abutment 1 & 2	3342

2. Difficult drilling conditions and hard pile driving is anticipated due to the dense to very dense alluvial soils encountered during the 2002 foundation investigation. Boulders were not encountered during the 2002 foundation investigation though were visible in the slopes and may be encountered during the driving of the piles. The cobbles and boulders up to approximately 3.5 ft in size ranged from slightly weathered to moderately weathered and hard.

3. The calculated geotechnical capacity of the "Modified" Class 140, Alternative "W" steel pipe piles is based predominantly on end-bearing with some skin friction.

4. Pile acceptance is to be based on Standard Specifications 49-2.01A(4)(b) "Pile Driving Acceptance Criteria". At the Abutment 1 and Abutment 2 locations, any pile that achieves 1½ times the required nominal resistance in compression, as shown on the contract plans, within 5 feet of the specified pile tip elevation, may be considered satisfactory and cut off with written approval from the Engineer.

5. Groundwater was encountered during the 2002 subsurface investigation. Refer to groundwater section of this report and the LOTB's for details regarding the groundwater. Groundwater levels indicated on the LOTB reflect the measured groundwater levels at the time of the subsurface investigation.

The recommendations contained in this report are based on specific project information regarding structure type, location, and design loads that have been provided by the Office of Structure Design-West, Design Branch 9. If any conceptual changes are made during final project design, the Office of Geotechnical Design-South 2, Design Branch B should review those changes to determine if these foundation recommendations are still applicable. Any questions regarding the above recommendations should be directed to the attention of Cipriano Manansala, (916) 227-5399, or Hector Valencia, (916) 227-4555, at the Office of Geotechnical Design-South 2, Branch B.

Prepared by: Date: 12-23-13

Cipriano Manansala
Transportation (Civil) Engineer
Office of Geotechnical Design-South 2
Design Branch B

Reviewed by: Date: 12-23-13

Hector Valencia, P.E., #C65257
Engineering Geologist
Office of Geotechnical Design-South 2
Design Branch B

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cc: Jason Bennecke - District 8 Project Manager
Structures Construction R.E. Pending File: RE_Pending_File@dot.ca.gov
Ofelia Alcantara - Structures Office Engineer
Theresa Nedwick - Structure Specifications
Bruce Kean - District 8 Materials Engineer
Abbas Abghari - OGDS-2
Mark Desalvatore - OGDS-2 FOH for Mark Desalvatore
Geotechnical Archive

M e m o r a n d u m

To: MR. GORDON DANKE, BRANCH CHIEF
OFFICE OF BRIDGE DESIGN - West
DESIGN BRANCH 9

Date: November 27, 2013

File: 08-SBd-138-PM-R18.1
EA No. 08-0Q300G
Project No. 0800020191
Double Drain Crk Bridge
Bridge No. 54-1220

Attn.: Mr. Pete Norboe

From: **DEPARTMENT OF TRANSPORTATION**
DIVISION OF ENGINEERING SERVICES
GEOTECHNICAL SERVICES - MS #5
OFFICE OF GEOTECHNICAL DESIGN SOUTH-2

Subject: **Seismic Design Recommendations for Double Drain Creek Bridge**

Introduction

This memorandum presents seismic design recommendations for the Double Drain Creek Bridge 54-1220. Ground motion recommendations are based on the Caltrans 2009 Seismic Design Procedure (SDP) as described in the Seismic Design Criteria Version 1.7 (SDC) Appendix B, the Acceleration Response Spectrum (ARS) Online Tool v2.2.06, USGS 2008 Interactive Deaggregations (Beta), and 2002 Log of Test Boring (LOTB) for geotechnical subsurface investigations.

Seismicity

Based on the 2012 Caltrans faults database and Caltrans ARS Online (v2.2.06) Tool, the site is located about 4.1 miles (6.5 km) from the San Andreas (San Bernardino South) fault. This fault (Fault ID 325, MMax = 7.9, strike-slip, dip = 90 degrees, vertical, Bottom and Top of Rupture Plane approximately 8.0 and 0 miles, respectively) is the controlling fault for the deterministic seismic design procedure. A map showing the location of the bridge and the nearby faults is attached in Figure 1.

Soil Profile

The Office of Geotechnical Design South-2 had performed subsurface geotechnical investigations from May through the end of July, 2002. This subsurface geotechnical exploration included five (5) 4-inch diameter rotary wash borings.

The maximum depth of the investigation was approximate 230 feet below the existing ground surface.

The geologic map "Geologic Map of the Cajon Quadrangle" (Dibblee, 2003) indicates the site is underlain by Quaternary Alluvium deposits which overlie Crowder Formation. Crowder Formation consists of very dense, weakly to moderately cemented arkosic sandstones.

The 2002 subsurface investigation reveled alluvial materials overlying sandstone. The alluvial material consists of medium to very dense layers of fine silty sands, fine to coarse sands, gravelly sands, with discontinuous lenses of bobbles. Underlying the alluvium is sandstone, which is very intensely weathered, weakly to moderately cemented. The sandstone consists of very dense layers of silty sand with gravel and cobbles, sandy gravel, poorly graded sand with gravel and cobbles, with thin intermittent lean clay lenses. Groundwater was measured on July 24 and November 13, 2002 at elevation 3470 feet and 3504 feet, respectively along the bridge alignment. For site-specific soil and rock descriptions, refer to the Log of Test Borings sheets and Foundation Report (FR) prepared by the Office of Geotechnical Design South 2, Branch B (OGDS-2B) for additional information.

Per 2002's LOTBs and general geology at this site, the average shear wave velocity for the upper 100 feet of subsurface materials is estimated as V_{S30} = 400 m/s.

Design Response Spectrum

Based on the 2009 SDP, the design response spectrum is the upper envelope of the deterministic and probabilistic response, but is not less than a minimum deterministic response spectrum resulting from a Mmax = 6.5 earthquake on a vertical strike-slip fault at a distance of 7.5 miles (12 km).

The deterministic response spectrum is obtained by taking the arithmetic average of the median response spectrum calculated using the 2008 Campbell-Bozorgnia and 2008 Chiou-Youngs ground motion prediction equations. The probabilistic response spectrum is obtained for 5 percent probability of exceedance in 50 years (corresponding to 975 year return period).

For this site, the design response spectrum is controlled by probabilistic response spectrum. The corresponding peak horizontal ground acceleration at proposed site is 0.96g. The recommended acceleration response spectrum is attached in Figure 2.

Liquefaction Potential Evaluation

Based on available subsurface information from 2002 LOTBs, soil liquefaction is not likely due to the dense nature of the local soils.

Surface Fault Rupture Hazard

The proposed site is not within an Alquist-Priolo Earthquake Fault Zone or an unzoned faults (Holocene or younger in age). The site is more than 2.0 miles (3.3 km) from the nearest Caltrans-active fault, Cleghorn fault zone (Southern Cleghorn section), which extends to the ground surface. Potential for surface rupture is low, and no further work or design for surface rupture is required.

The seismic design recommendations contained in this report are based on specific project information that was provided by the Office of Bridge Design West. If any conceptual changes are made during final project design, the Office of Geotechnical Design South-2 should review those changes to determine whether the seismic design recommendations provided in this report are still applicable. Any questions regarding the above recommendations should be directed to the attention of Asef Wardak (916) 227-1219 or Angel Perez-Cobo (916) 227-7167, Office of Geotechnical Design South-2.

Prepared by: Date: 11-27-2013

REGISTERED PROFESSIONAL ENGINEER
ASEF WARDAK
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Exp. 9/30/2015
CIVIL
STATE OF CALIFORNIA

Asef Wardak
Transportation Engineer
Office of Geotechnical Design South-2

Attachments:

Figure 1. Nearby Major Faults with Reference to the Project Site
Figure 2. Recommended Acceleration Response Spectrum (ARS) Curve

cc: District 08 Structure Construction R.E. – Pending File
District 8 – Project Manager – Jason Bennecke (Electronic File)
Specs & Estimates – Ofelia Alcantara (Electronic File)
District 8, District Materials Engineer – Bruce Kean (Electronic File)
HQ Geotechnical Design South-2 – Abbas Abghari – OGDS-2 (Electronic File)
HQ Geotechnical Design South-2 – Mark Desalvatore – GDS-2 (Electronic File)
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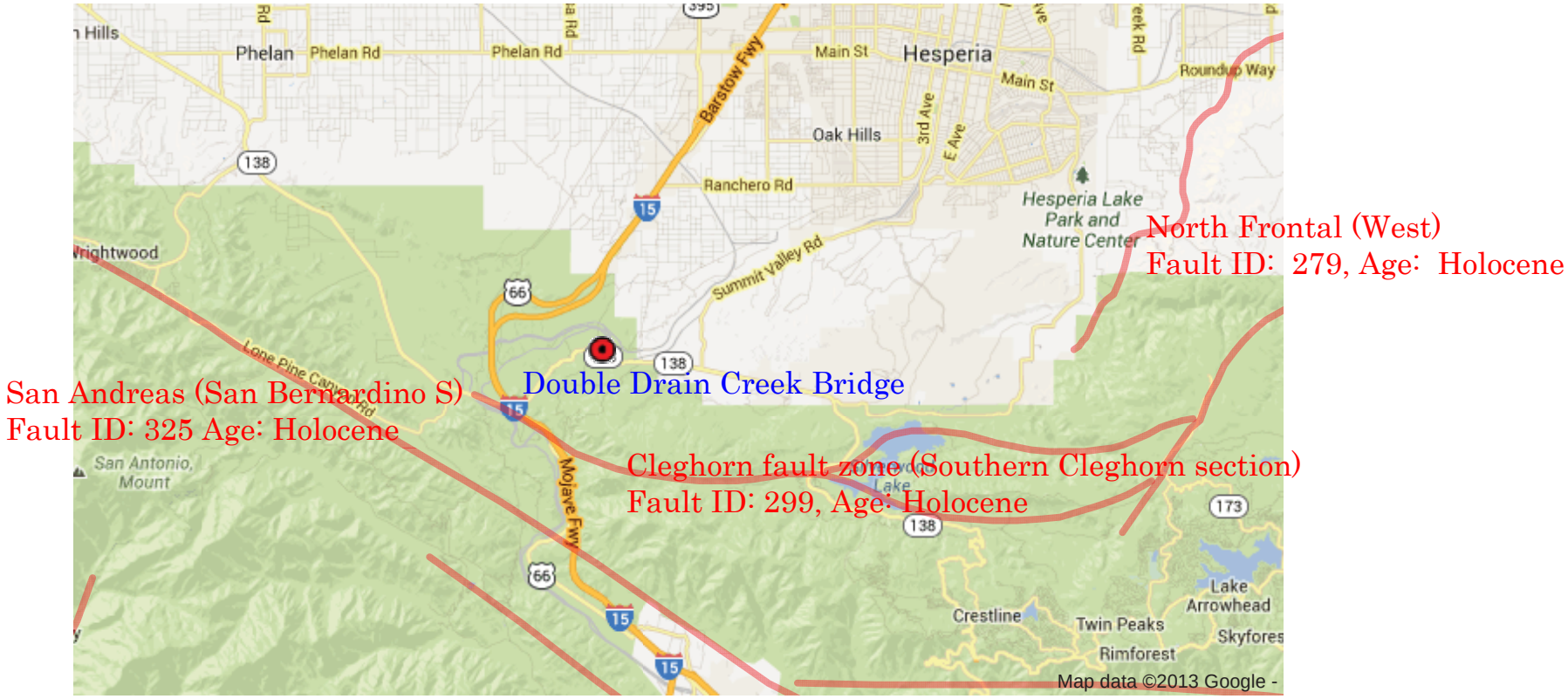
HQ Geotechnical Design South-2 – Erich Neupert – GDS-2 (Electronic File)
HQ Geotechnical Design South-2 – David Liao – GDS-2 (Electronic File)
HQ Geotechnical Design South-2 – Cipriano Manansala – GDS-2 (Electronic File)
HQ Geotechnical Archive

CALIFORNIA DEPARTMENT OF
TRANSPORTATION

Caltrans ARS Online (v2.2.06)

This web-based tool calculates both deterministic and probabilistic acceleration response spectra for any location in California based on criteria provided in *Appendix B of Caltrans Seismic Design Criteria*. More...

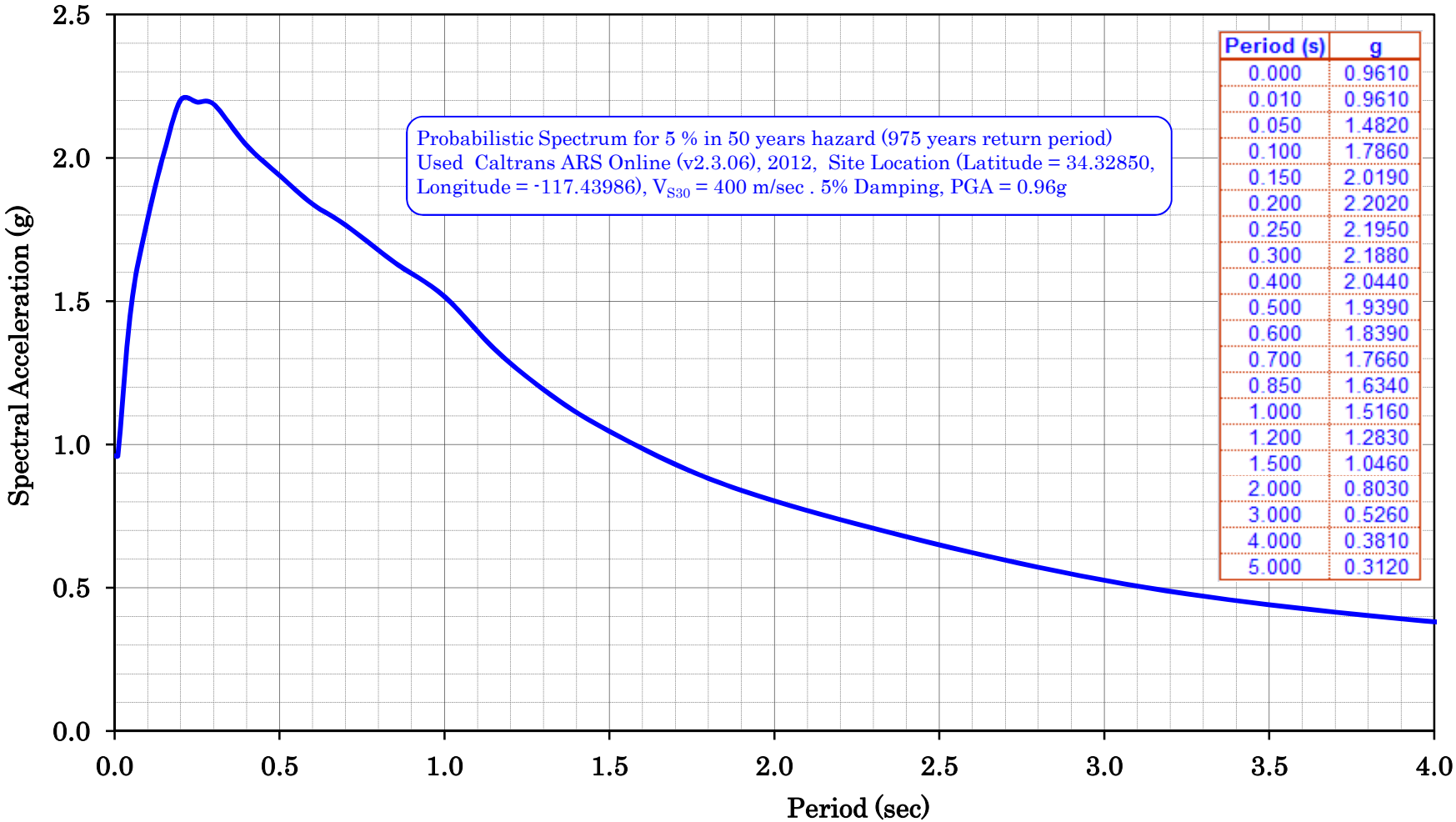
SELECT SITE LOCATION



Latitude: 34.3285 **Longitude:** -117.43986 ***Vs30:*** 400 m/s Calculate

Figure 1. Project site and the nearby faults is attached

Recommended Acceleration Response Spectrum for the Double Drain Creek Bridge (New) (Bridge # 54-1220)



Period (s)	g
0.000	0.9610
0.010	0.9610
0.050	1.4820
0.100	1.7860
0.150	2.0190
0.200	2.2020
0.250	2.1950
0.300	2.1880
0.400	2.0440
0.500	1.9390
0.600	1.8390
0.700	1.7660
0.850	1.6340
1.000	1.5160
1.200	1.2830
1.500	1.0460
2.000	0.8030
3.000	0.5260
4.000	0.3810
5.000	0.3120

Figure 2. Recommended Acceleration Response Spectrum (ARS) Curve

The subsurface investigation performed in 2002 consisted of 5 mud rotary borings which generally reveled alluvial materials overlying sandstone. The alluvial material consists of medium dense to very dense layers of silty sand and poorly graded sand with gravel and cobbles. Underlying the alluvium is sandstone, which is very intensely weathered, weakly to strongly cemented. The sandstone with soil-like characteristics which consists of very dense layers of silty sand with gravel and cobbles, poorly graded sand with gravel and cobbles, with thin intermittent lean clay lenses. For site-specific soil and rock descriptions, refer to the Log of Test Borings sheets.

Groundwater

Groundwater was encountered during the 2002 subsurface investigation. In Boring 02-2, near the pier 2 location, the measured static groundwater level was at elevation 3470 ft. In Boring 02-5, located between pier 4 and abutment 5, the measured static groundwater level was at elevation 3504 ft. The structure supports are located within an intermittent creek, therefore, the groundwater surface elevations are subject to large fluctuations in elevation due to seasonal precipitation (heavy rainfall/flash floods), and will be encountered at higher or lower elevations at time of construction.

Scour Potential

The bridge site is located within an intermittent creek that during times of heavy precipitation has large amounts of surface water flowing in the channel and has the potential for scour. The scour data shown in Table 1 is based on the hydraulic data provided to our office. At this time, a final hydraulic report has not been completed. For details and information used in developing the estimated scour information, please contact Jimmie Pallares with the Structure Hydraulics and Hydrology Branch at 916-227-8705.

Table 1. Scour Data

Support Location	Long Term Scour (Degradation and Contraction) Elevation (ft)	Short Term Scour (Local) Depth (ft)
Abutment 1	N/A	N/A
Pier 2	3.5	4.5
Pier 3	3.5	4.5
Pier 4	N/A	N/A
Abutment 5	N/A	N/A

Corrosion

Corrosion test results for soils samples collected from Boring 02-1, 02-3, and 02-5 are shown in Table 2 below. All of the soil samples tested are considered non-corrosive by current Caltrans standards.

Table 2. Corrosion Test Summary

Sample Location	pH	Minimum Resistivity (Ohm-Cm)	Chloride Content (ppm)	Sulfate Content (ppm)
Boring 02-1 Elev. 3517.3-3476.6	8.09	3300	N/A	N/A
Boring 02-1 Elev. 3476.6-3452.6	8.05	1900	N/A	N/A
Boring 02-1 Elev. 3452.6-3422.4	8.13	2200	N/A	N/A
Boring 02-1 Elev. 3422.4-3387.3	8.24	3000	N/A	N/A
Boring 02-3 Elev. 3532.9-3482.4	8.14	2000	N/A	N/A
Boring 02-3 Elev. 3482.4-3432.5	8.15	2700	N/A	N/A
Boring 02-3 Elev. 3432.5-3392.5	8.45	2600	N/A	N/A
Boring 02-3 Elev. 3392.5-3352.4	8.67	2000	N/A	N/A
Boring 02-5 Elev. 3606.3-3575.8	8.18	4300	N/A	N/A
Boring 02-5 Elev. 3575.8-3525.9	8.12	1900	N/A	N/A
Boring 02-5 Elev. 3525.9-3505.9	8.27	3200	N/A	N/A
Boring 02-5 Elev. 3505.9-3456.3	8.30	2600	N/A	N/A

Note: Caltrans currently defines a corrosive environment as an area where the soil has either a chloride concentration of 500 ppm or greater, a sulfate concentration of 2000 ppm or greater, or has a pH of 5.5 or less. With the exception of MSE walls, soil and water are not tested for chlorides and sulfates if the minimum resistivity is greater than 1,000 ohm-cm.

Fault and Seismic Data

The structure site is potentially subject to ground motions from nearby earthquake sources during the design life of the new structure. Based on the 2012 Caltrans Fault Database (V2.2.06) for the deterministic procedure, the controlling fault for the site is the San Andrea (San Bernardino South) fault zone (Fault ID: 325, strike-slip, dip=90 deg.) with a maximum moment magnitude, M_{Max}=7.9, located approximately 4.1 miles (6.5 km) southwest of the site. The probabilistic response spectrum is obtained for the 5% probability of exceedance in 50 years from the 2008 USGS Seismic Hazard Map.

Based on the 2002 Log of Test Borings and general geology at the site, the average shear wave velocity for the upper 100 ft of the subsurface materials is estimated as V_{S30} = 1312 ft/s (400 m/s).

For this site, the design response spectrum is controlled solely by the probabilistic response spectrum, with a Peak Ground Acceleration (PGA) of 0.96g.

Surface Rupture Potential

The site is not located within the Earthquake Fault Hazard Zone in California. Therefore, the potential for surface fault rupture hazard is considered low.

Liquefaction Potential

Due to the dense nature of the alluvial material underlying the site, as well as the expected absence of high ground water table, the liquefaction potential at the site is anticipated to be low.

For sites specific seismic data and design recommendations mentioned above, refer to the Seismic Design Recommendations Report, dated November 27, 2013, by Asef Wardak (916-227-1219) of the Office of Geotechnical Design South 2 (OGDS2).

Foundation Recommendations

The following foundation recommendations are for the Double Drain Creek Bridge as shown on General Plan, dated November 7, 2013.

At Abutments 1 and 5 support locations, 42-inch Cast-In-Drilled-Hole (CIDH) piles may be used for support of the proposed structure. At Piers 2 through 4 support locations, 84-inch CIDH piles may be used for support of the proposed structure.

The foundation design at the abutment locations was based on working stress design (WSD) and Load and Resistance Factored Design (LRFD) was used at the pier locations. The information shown in Table 3 & 4 is based on foundation design information provided to our office by Structure Design, Branch 9 on May 8, 2013 and supplemented on December 13, 2013.

Table 3. Foundation Data Provided by Structure Design

Support Location	Design Method	Pile Type	Finished Grade Elevation (ft)	Pile Cut-Off Elevation (ft)	Pile Cap Size (ft)		Permissible Settlement under Service Load (in)	Number of Piles per Support
					B	L		
Abut 1	WSD	42 in CIDH	3573.8	3572.0	4	42	1	4
Pier 2	LRFD	84 in CIDH	varies	3528.5	N/A	N/A	1	2
Pier 3	LRFD	84 in CIDH	varies	3536.8	N/A	N/A	1	2
Pier 4	LRFD	84 in CIDH	varies	3548.3	N/A	N/A	1	2
Abut 5	WSD	42 in CIDH	3610.8	3609.0	4	42	1	4

Table 4. Foundation Design Loads Provided by Structure Design

Foundation Design Loads											
Support Location	Service-I Limit State (kips)			Strength Limit State (Controlling Group, kips)				Extreme Limit State (Controlling Group, kips)			
	Total Load		Perm. Loads	Compression		Tension		Compression		Tension	
	Per Support	Max. Per Pile	Per Support	Per Support	Max. Per Pile	Per Support	Max. Per Pile	Per Support	Max. Per Pile	Per Support	Max. Per Pile
Abut 1	1750	440	1360	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A
Pier 2	4880	N/A	2900	6820	3410	0	0	4970	4970	N/A	-2070
Pier 3	4240	N/A	2680	6080	3040	0	0	4860	4860	N/A	-2180
Pier 4	5060	N/A	3140	7300	3650	0	0	5090	5090	N/A	-1950
Abut 5	1710	430	1350	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A

The specific CIDH pile tip elevations for Abutments 1 and 5, and Pier 2 through 4 are shown below in Tables 5 and 6.

Table 5. Foundation Design Recommendations for Abutments 1 and 5

Abutment Foundations Design Recommendations								
Support Location	Pile Type	Cut-off Elevation (ft)	LRFD Service-I Limit State Load per Support (kips)		LRFD Service-I Limit State Total Load per Pile (kips)	Nominal Resistance (kips)	Design Tip Elevations (ft)	Specified Tip Elevation (ft)
			Total	Perm.				
Abut 1	42 in CIDH	3572.0	1750	1360	440	880	3513.0 (a)	3513.0
Abut 5	42 in CIDH	3609.0	1710	1350	430	860	3564.0 (a)	3564.0

Note: 1) Design tips are controlled by: (a) Compression.

Table 6. Foundation Design Recommendations for Pier 2 to4

Pier Foundation Design Recommendations										
Support Location	Pile Type	Cut-Off Elev. (ft)	Service-I Limit State Load per Support (kips)	Total Permissible Support Settlement (inches)	Required Nominal Resistance (R_n) (kips)				Design Tip Elev. (ft)	Specified Tip Elev. (ft)
					Strength Limit		Extreme Limit			
					Comp. (ϕ=0.7)	Tension (ϕ=0.7)	Comp. (ϕ=1.0)	Tension (ϕ=1.0)		
Pier 2	84 in CIDH	3528.5	4880	1	4880	0	4970	2070	3447.0 (a-I) 3442.0 (a-II) 3465.0 (b-II)	3442.0
Pier 3	84 in CIDH	3536.8	4240	1	4350	0	4860	2180	3461.0 (a-I) 3452.0 (a-II) 3473.0 (b-II)	3452.0
Pier 4	84 in CIDH	3548.3	5060	1	5220	0	5090	1950	3486.0 (a-I) 3491.0 (a-II) 3511.0 (b-II)	3486.0

Note: 1) Design tip elevations are controlled by: (a-I) Compression (Strength Limit), (a-II) Compression (Extreme Event), (b-II) Tension (Extreme Event).
2) The specified tip elevation shall not be raised above the design tip elevations for tension.

The Pile Data Table is presented in Table 7, below. The ultimate geotechnical pile capacity of the CIDH piles will meet or exceed the required nominal resistance in compression.

Table 7. Pile Data Table

Pile Data Table					
Support Location	Pile Type	Nominal Resistance (R_n) (kips)		Design Tip Elev. (ft)	Specified Tip Elev. (ft)
		Compression	Tension		
Abut 1	42 in CIDH	880	0	3513.0 (a)	3513.0
Pier 2	84 in CIDH	4970	2070	3442.0 (a) 3465.0 (b)	3442.0
Pier 3	84 in CIDH	4860	2180	3452.0 (a) 3473.0 (b)	3452.0
Pier 4	84 in CIDH	5220	1950	3486.0 (a) 3511.0 (b)	3486.0
Abut 5	42 in CIDH	860	0	3564.0 (a)	3564.0

Note: 1) Design tip elevations are controlled by: (a) Compression, (b) Tension.

General Notes:

1) The structure engineer shall show on the plans, in the pile data table, the minimum pile design tip elevation required to meet the lateral load demands. If the specified pile tip elevation required to meet lateral load demands exceed the specified pile tip elevation given within this report, OGDS2 should be contacted for further recommendations.

2) The special provisions shall specify the requirements of Tunnel Safety Orders, for the CIDH shaft work that meets the definition of a tunnel or shaft as described in the Highway Design Manual, Section 110.12 "Tunnel Safety Orders."

Construction Considerations:

Rock Core Samples

1) Rock core samples from Boring No. 02-2 and 02-4 of the 2002 Caltrans foundation investigation were retained and are available for viewing by bidders at the Geotechnical Services (Transportation Laboratory), Department of Transportation, 5900 Folsom Blvd., Sacramento, CA. Potential bidders are to allow the State seven (7) days to prepare and display the rock cores.

CIDH Piles

1) During the 2002 foundation investigation, groundwater was encountered at the bridge site. Contractor should anticipate the possibility that surface and/or groundwater may be encountered during the construction. Groundwater elevations at this bridge site are subject to seasonal fluctuations and will be encountered at higher or lower elevations depending on conditions at the time of construction. For more details refer to the site geology and groundwater section of this report and the Log of Test Boring Sheets.

2) Due to the anticipation that concrete placement for the CIDH piles will require slurry displacement methods, the calculated geotechnical capacity of all CIDH piles is based on skin friction only and no end-bearing was considered. The skin friction zones of the CIDH piles are summarized below in Table 8.

Table 8: CIDH Pile Skin Friction Zone Elevations

Support Location	Skin Friction Zone Start Elevation (ft)	Skin Friction Zone End Elevation (ft)
Abutment 1	3557.0	3515.0
Pier 2	3497.5	3446.0
Pier 3	3507.0	3456.0
Pier 4	3536.3	3490.0
Abutment 5	3604.0	3566.0

3) If the CIDH piles are to be constructed using slurry displacement method, the slurry shall consist of mineral or synthetic slurry only. Use of water shall not be allowed as slurry.

4) The contractor will need to excavate alluvium soil over sedimentary formational material which consists of sandstone with soil-like characteristics in order to construct the CIDH piles. The amount of difficulty the contractor will experience will be dependent upon the methods and means the contractor chooses to use to construct the CIDH piles. Temporary casing may be necessary to control caving during construction. All temporary casing is to be removed during concrete placement.

5) At the pier locations, Type II CIDH piles are specified with a construction joint that is shown on the contract plans. The construction joint shall be in conformance to Standard Specifications 49-3.02C(5) and 49-3.02C(6). Because of the soil-like characteristics of the

rock, permanent casing must extend 5 feet below the construction joint shown on the contract plans. The permanent casing must consist of a corrugated metal pipe (CMP).

This Foundation Report is based on specific project information that has been provided by the Office of Bridge Design, Design Branch 9. If any conceptual changes are made during final project design, the Office of Geotechnical Design-South 2, Design Branch B should review those changes to determine if this report is still applicable. Any questions regarding the above report should be directed to the attention of David TM Liao, (916) 227-5756, or Mark DeSalvatore, (916) 227-5391 at the Office of Geotechnical Design-South 2, Branch B.

Prepared by: Date: 12/20/13

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