Earth Systems
Southwest

UPDATE TO
GEOTECHNICAL ENGINEERING REPORT
REVISED COMMERCIAL WECS 20
WEST 1/2 OF SECTION 31, T2S, R4E, S.B.B.M.
RIVERSIDE COUNTY, CALIFORNIA

Consulting Engineers and Geologists

ENERGY UNLIMITED, INC.
638 LINDERO CANYON ROAD, #273
OAK PARK, CALIFORNIA 91301

**UPDATE TO
GEOTECHNICAL ENGINEERING REPORT
REVISED COMMERCIAL WECS 20
WEST 1/2 OF SECTION 31, T2S, R4E, S.B.B.M.
RIVERSIDE COUNTY, CALIFORNIA**

File No.: 07164-03
00-11-728

Earth Systems
Southwest

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November 10, 2000

File No.: 07164-03
00-11-728

Energy Unlimited, Inc.
638 Lindero Canyon Road, #273
Oak Park, California 91301

Attention: Mr. David Lamm

Project: **Revised Commercial WECS 20**
West 1/2 of Section 31, T2S, R4E, S.B.B.M.
Riverside County, California

Subject: **Update to Geotechnical Engineering Reports**

References: Pioneer Consultants, Preliminary Soils and Geologic Investigation, WECS 20, W 1/2 of Section 31, T2S, R4E, S.B.B.M., Riverside County, California, Project No. J.N. 3923-001, dated February 4, 1985.

As requested, we have reviewed the referenced documents for purposes of providing an updated report for the WECS 20 revised permit. This letter provides a description of the proposed revisions to the WECS permit and provides foundation and seismic design criteria.

Project Description

The WECS 20 permit area is located in the west 1/2 of Section 31, T2S, R4E, S.B.B.M. in the Whitewater area of Riverside County, California. Figure 1 shows the site location and vicinity. Seven wind turbines are currently proposed to add to an existing north-south trending row of smaller turbines. Based on information presented on the Plot Plan prepared by Krieger and Stewart, Inc., dated October 31, 2000, we understand that the proposed wind turbines will be Nordex N62 1,300kW turbines mounted on approximately 197-foot high (to the hub) monopole towers. These turbine locations are shown on Figure 2.

Site Description

The topography of the site is highly irregular consisting of eastward and southward sloping ridges and stream channels. The overall effect is that of a highly dissected alluvial fan surface. Regional slope on the ridgetops is 5 degrees to the southwest. Approximately half of the site consists of slopes that are 25 percent grade or greater.

The ground surface is characterized by desert pavement consisting of very coarse sands, gravels, and cobbles. Boulders to 3 feet in diameter are scattered on the surface but are generally confined to the ridgetop areas. Vegetation on the site consists of sparse scrub brush and cactus.

Summary of Geologic Conditions

The WECS 20 site is located at the extreme eastern end of the San Gorgonio Pass. This pass forms the boundary between the Transverse Ranges geomorphic province to the north, and the Peninsular Ranges province to the south.

The San Andreas Fault zone is the most significant potential seismic source in the site vicinity. In the eastern San Gorgonio Pass and the upper portion of the Coachella Valley, the San Andreas Fault zone is comprised of the Garnet Hill, the Banning, and the Mission Creek faults.

Previous geologic mapping by Proctor in 1968 and field mapping conducted by Pioneer Consultants in 1985 indicate that the site is underlain by three distinctive geologic units. The oldest exposed geologic unit on the site is the Painted Hill formation (Unit Tph) of early Pleistocene Age. This alluvial unit consists of gray and light brown, poorly sorted beds of conglomerate and/or arkosic formation is the Quaternary age Cabazon Fanglomerate (Unit Qoc). Capping the Cabazon Fanglomerate on ridgetops is a thin mantle of terrace deposits consisting of orange-tinted sands and gravels and is further characterized by large boulders protruding conspicuously from the ground surface. The boulders are present in trace amounts. Overlying the Painted Hill formation and the Cabazon fanglomerate in the valley areas is recent stream alluvium (Unit Qal) consisting of coarse sands and pebble gravels.

The proposed turbines will be located on a ridge that is underlain by the Pleistocene-aged Cabazon Fanglomerate. This formation consists of semi-consolidated, poorly-bedded, poorly-sorted, pebbly to bouldery conglomerate. These deposits are alluvial in origin (deposited by flowing water) and have been uplifted by tectonic forces related to movements along the San Andreas Fault. The rugged topography at the site is primarily the result of the dissection of these deposits by erosion along currently active stream channels.

The Cabazon Fanglomerate was investigated by Pioneer Consultants with soil borings BH-3 and BH-4. These materials were found to consist primarily of a surface, loose silty sand, approximately 1 foot in depth, underlain by dense to very dense, coarser to gravelly sands and silty gravels to the total depths encountered. The borings logs from the Pioneer Consultants Report is attached as Appendix A.

Seismic Hazards

Seismic Sources: Our research of regional faulting indicates that several active faults or seismic zones lie within 62 miles (100 kilometers) of the project site as shown on Table 1. The Maximum Magnitude Earthquake (M_{max}) listed is from published geologic information available for each fault (CDMG, 1996). The M_{max} corresponds to the maximum earthquake believed to be tectonically possible.

Surface Fault Rupture: The project site does not lie within a currently delineated State of California, *Alquist-Priolo* Earthquake Fault Zone (Hart, 1994). Well-delineated fault lines cross through this region as shown on California Division of Mines and Geology (CDMG) maps (Jennings, 1994). Therefore, active fault rupture is unlikely to occur at the project site. While fault rupture would most likely occur along previously established fault traces, future fault rupture could occur at other locations.

Historic Seismicity: Six historic seismic events (5.9 M or greater) have significantly affected the region in the last 100 years. They are as follows:

- *Desert Hot Springs Earthquake* - On December 4, 1948, a magnitude 6.5 M_L (6.0M_W) earthquake occurred east of Desert Hot Springs. This event was strongly felt in the Palm Springs area.
- *Palm Springs Earthquake* - A magnitude 5.9 M_L (6.2M_W) earthquake occurred on July 8, 1986 in the Painted Hills beneath the WECS 20 site causing minor surface creep of the Banning segment of the San Andreas Fault. This event was strongly felt in the Palm Springs area and caused structural damage, as well as injuries.
- *Joshua Tree Earthquake* - On April 22, 1992, a magnitude 6.1 M_L (6.1M_W) earthquake occurred in the mountains 9 miles east of Desert Hot Springs. Structural damage and minor injuries occurred in the Palm Springs area because of this earthquake.
- *Landers & Big Bear Earthquakes* - Early on June 28, 1992, a magnitude 7.5 M_S (7.3M_W) earthquake occurred near Landers, the largest seismic event in Southern California for 40 years. Surface rupture occurred just south of the town of Yucca Valley and extended some 43 miles toward Barstow. About three hours later, a magnitude 6.6 M_S (6.4M_W) earthquake occurred near Big Bear Lake. No significant structural damage from these earthquakes was reported in the Palm Springs area.
- *Hector Mine Earthquake* - On October 16, 1999, a magnitude 7.1M_W earthquake occurred on the Lavic Lake and Bullion Mountain Faults north of 29 Palms. This event while widely felt, no significant structural damage has been reported in the Coachella Valley.

Seismic Risk: While accurate earthquake predictions are not possible, various agencies have conducted statistical risk analyses. In 1996, the California Division of Mines and Geology (CDMG) and the United States Geological Survey (USGS) completed the latest generation of probabilistic seismic hazard maps for use in the 1997 UBC. We have used these maps in our evaluation of the seismic risk at the site. The Working Group of California Earthquake Probabilities (WGCEP, 1995) estimated a 22% conditional probability that a magnitude 7 or greater earthquake may occur between 1994 to 2024 along the Coachella segment of the San Andreas Fault.

The primary seismic risk at the site is a potential earthquake along the San Andreas Fault. Geologists believe that the San Andreas Fault has characteristic earthquakes that result from rupture of each fault segment. The estimated characteristic earthquake is magnitude 7.4 for the Southern Segment of the fault. This segment has the longest elapsed time since rupture than any other portion of the San Andreas Fault. The last rupture occurred about 1690 AD, based on dating by the USGS near Indio (WGCEP, 1995). This segment has also ruptured on about 1020, 1300, and 1450 AD, with an average recurrence interval of about 220 years. The San Andreas Fault may rupture in multiple segments producing a higher magnitude earthquake. Recent paleoseismic studies suggest that the San Bernardino Mountain Segment to the north and the Coachella Segment may have both ruptured together in 1450 and 1690 AD (WGCEP, 1995).

Site Acceleration: The potential intensity of ground motion may be estimated the horizontal peak ground acceleration (PGA), measured in "g" forces. Included in Table 1 are deterministic estimates of site acceleration from possible earthquakes at nearby faults. Ground motions are dependent primarily on the earthquake magnitude and distance to the seismogenic (rupture) zone. Accelerations also are dependent upon attenuation by rock and soil deposits, direction of rupture, and type of fault. For these reasons, ground motions may vary considerably in the same general area. This variability can be expressed statistically by a standard deviation about a mean relationship.

The PGA is an inconsistent scaling factor to compare to the UBC Z factor and is generally a poor indicator of potential structural damage during an earthquake. Important factors influencing the structural performance are the duration and frequency of strong ground motion, local subsurface conditions, soil-structure interaction, and structural details. Because of these factors, an effective peak acceleration (EPA) is used in structural design.

The following table provides the probabilistic estimate of the PGA and EPA taken from the 1996 CDMG/USGS seismic hazard maps.

Estimate of PGA and EPA from 1996 CDMG/USGS Probabilistic Seismic Hazard Maps

Risk	Equivalent Return Period (years)	PGA (g) [1]	Approximate EPA (g) [2]
10% exceedance in 50 years	475	0.96	0.75

Notes:
1. Based on a soft rock site, $S_{B/C}$ and soil amplification factor of 1.0 for Soil Profile Type S_C.
2. Spectral acceleration (S_A) at period of 0.3 seconds divided by 2.5 for 5% damping, as defined by the Structural Engineers Association of California (SEAOC, 1996).

Recommended Future Geotechnical Studies

We recommend that additional geologic and geotechnical studies be conducted prior to project construction of the proposed Nordex N62/1300kW wind turbine foundations. These studies should include additional soil borings to a depth of 40 feet or refusal along the alignment of proposed turbines in WECS 20. At least three soil borings should be conducted to evaluate the soil conditions to support these large wind turbine structures..

Foundations

We understand that the tower foundations will consist of the proprietary Patrick and Henderson tensionless pier (PHTP) using a large diameter, cast-in-place pier. This type of pier would be constructed by excavating to the desired depth and size with an excavator. Within the excavation a smaller diameter, corrugated-steel casing is set concentrically within the larger diameter corrugated-steel casing. Steel tie rods within PVC sleeves are placed vertically and concrete placed in the annular space between the casings. The tie rods are post-tensioned to keep the concrete in compression (hence tensionless) during loading. Soil backfill is placed within the central casing. The annular space between the outer casing and the excavation walls are to be backfilled with sand/cement slurry.

All details of the foundation system are to be designed by the design engineer. The diameter and depth of the pier as well as spacing and connection of steel tie rods are to be determined by the design engineer, proportioned to support the design loads. The outside annular space should be grouted to near the surface to maintain intimate contact between the composite caisson and the undisturbed native soil. Caving conditions may occur in the soil consisting of relatively cohesionless sand, gravel, and cobbles. Sidewall sloughing will result in larger excavation and greater grout quantities for backfill.

The following table present allowable axial and lateral capacities that may be used in the PHTP design contingent on any change of condition that future soil borings may indicate. The capacities for axial loads may be based on skin friction with some end bearing. These values have an estimated factor of two to ultimate values. We anticipate the size of the PHTP may range from 14 to 15 feet in outer diameter and about 25 to 35 feet deep.

Allowable Axial End Bearing Capacity, Settlement Criteria Governs From depth of 20 to 35 feet below grade	10,000 psf
Allowable Positive Skin Friction per foot of depth Maximum	55 psf/ft 2000 psf
Allowable Uplift Skin Friction per foot of depth Maximum	35 psf/ft 2000 psf
Passive Earth Pressure Unit Soil Weight Friction Angle of Soil Secant Modulus of Lateral Subgrade Reaction at e_{50} Initial Modulus of Lateral Subgrade Reaction	480 pcf 110 pcf 38 degrees 65 lbs./cu.-in. 225 lbs./cu.-in.

Lateral pile capacity: Lateral pile capacity may be calculated for deflections at the pile head for a pile free to rotate. Deflection can be assumed proportional to the applied load. Deep foundations placed in granular soils and subjected to cyclic lateral loading will eventually experience deflection of approximately two times their initial lateral top deflection. We assume plumb tolerance of the turbines should be within 0.002 radians for operating conditions. For extreme design loads, a tilt tolerance of 100:1 (vertical: horizontal) is acceptable.

Seismic Design Criteria

This site is subject to strong ground shaking due to potential fault movements along the San Andreas Fault. The *minimum* seismic design should comply with the 1997 edition of the Uniform Building Code (UBC) for non-building structures. The UBC provisions are generally intended to protect human life safety and prevent structural collapse. It is not necessarily intended to prevent structural damage or preserve functionality after a large earthquake. The following are 1997 UBC seismic design values:

1997 UBC Seismic Coefficients for Chapter 16 Seismic Provisions

			Reference
Seismic Zone:	4		Figure 16-2
Seismic Zone Factor, Z:	0.4		Table 16-I
Soil Profile Type:	S_C		Table 16-J
Seismic Source Type:	A		Table 16-U
Closest Distance to Known Seismic Source:	<2 km		(San Andreas Fault)
Near Source Factor, Na:	1.5		Table 16-S
Near Source Factor, Nv:	2.0		Table 16-T
Seismic Coefficient, Ca:	0.60	= 0.40Na	Table 16-Q
Seismic Coefficient, Cv:	1.12	= 0.56Nv	Table 16-R

Closing

Except as modified by this update report, it is our opinion that the referenced documents are applicable to the proposed revision to the WECS permit. The recommendations contained within our geotechnical engineering report and the supplement regarding special grading provisions for access roads on steep slopes remain applicable.

This report is issued with the understanding that the owner, or the owner's representative, has the responsibility to bring the information and recommendations contained herein to the attention of the engineers for the project so that they are incorporated into the plans and specifications for the project. The owner, or the owner's representative, also has the responsibility to take the necessary steps to see that the general contractor and all subcontractors follow such recommendations. It is further understood that the owner or the owner's representative is responsible for submittal of this report to the appropriate governing agencies.

As the Geotechnical Engineer of Record for this project, Earth Systems Southwest (ESSW) has striven to provide our services in accordance with generally accepted geotechnical engineering practices in this locality at this time. No warranty or guarantee is express or implied. This report was prepared for the exclusive use of the Client and the Client's authorized agents.

ESSW should be provided the opportunity for a general review of final design and specifications in order that earthwork and foundation recommendations may be properly interpreted and implemented in the design and specifications. If ESSW is not accorded the privilege of making this recommended review, we can assume no responsibility for misinterpretation of our recommendations.

This report is based on the assumption that an adequate program of client consultation, construction monitoring, and testing will be performed during the final design and construction phases to check compliance with these recommendations. Maintaining ESSW as the geotechnical consultant from beginning to end of the project will provide continuity of services. The geotechnical engineering firm providing tests and observations shall assume the responsibility of Geotechnical Engineer of Record.

Should you have any questions concerning this update report please give us a call and we will be pleased to assist you.

Sincerely,
EARTH SYSTEMS SOUTHWEST

Shelton L. Stringer

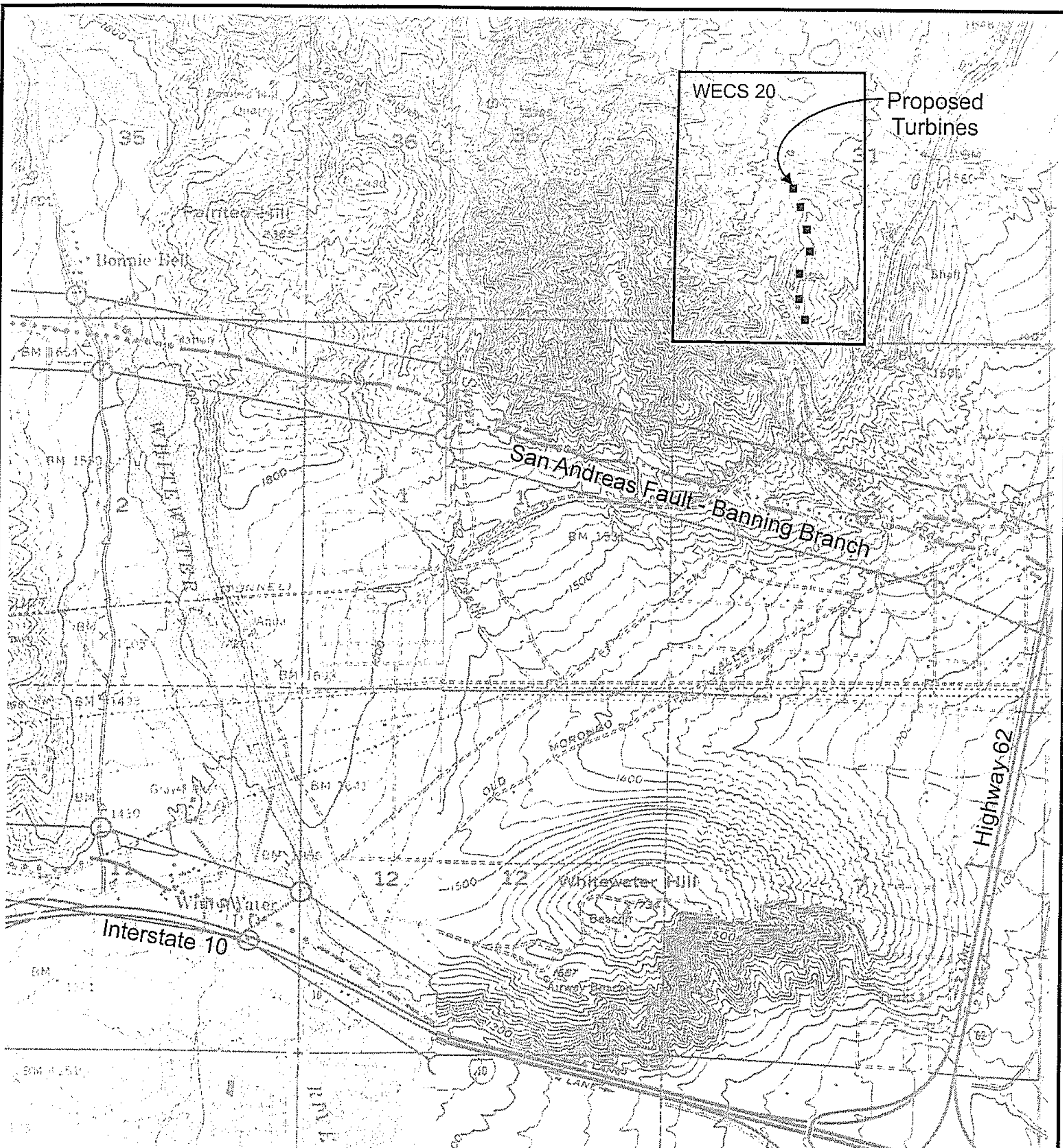
Shelton L. Stringer
GE 2266

REGISTERED PROFESSIONAL ENGINEER
SHELTON L. STRINGER
No. 2266
Exp. 6-30-04
GEOTECHNICAL
STATE OF CALIFORNIA

Distribution: 2/Energy Unlimited, Inc.
5/Krieger and Stewart, Inc.
1/VTA File
2/BD File

The following are attached and complete this report:

Figure 1	-	Site Location
Figure 2	-	Geologic Map & Exploration Locations, WECS 20
Table 1	-	Fault Parameters
Appendix A	-	Boring Logs from Preliminary Soils and Geologic Investigations, WECS 20



Base Map: California Division of Mines and Geology, Alquist-Priolo Earthquake Fault Zone Maps, Desert Hot Springs and Whitewater Quadrangles, effective 1980 and 1995, respectively.

N

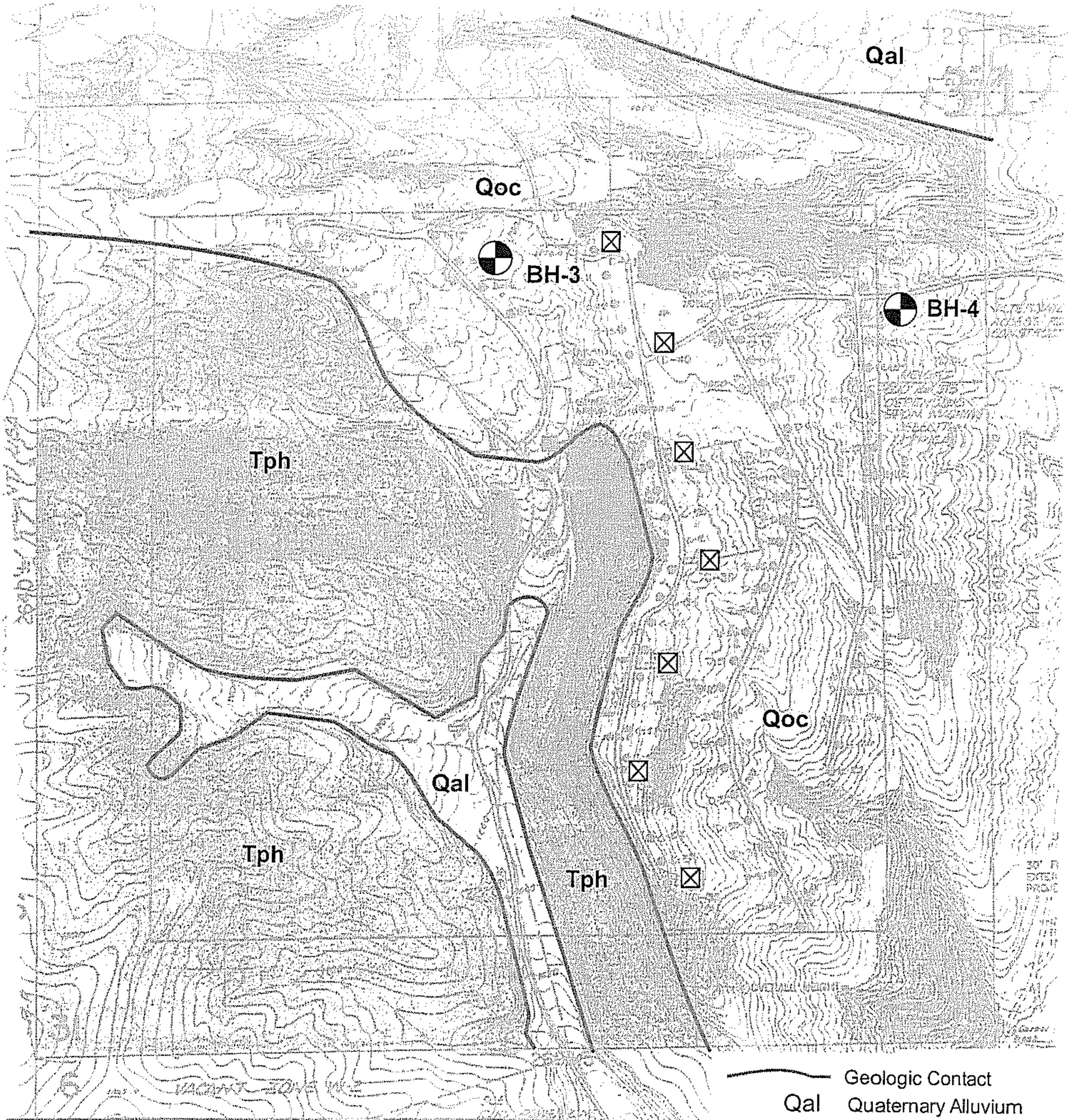
Scale: 1" = 2,000'

0 2,000 4,000

Figure 1 - Site Location

Energy Unlimited, Inc. - Revised WECS 20
Project No.: 07164-03

Earth Systems
Southwest



Geologic Contact
Qal Quaternary Alluvium
Qoc Cabazon Fanglomerate
Tph Painted Hills Formation

Note: Geology after Pioneer Consultants, 1985

Base Map: Revised WECS 20 Permit, Plot Plan, prepared by Krieger and Stewart, Inc., dated October 31, 2000

LEGEND

☒ Proposed Turbine Location

Approximate Boring Location

N

Scale: 1" = 400'

0 400 800

Figure 2 - Geologic Map and Boring Locations

Energy Unlimited, Inc. - Revised WECS 20
Project No.: 07164-03

Earth Systems
Southwest

Table 1
Fault Parameters &
Deterministic Estimates of Mean Peak Ground Acceleration (PGA)

Fault Name or Seismic Zone	Distance from Site (mi)	Distance from Site (km)	Fault Type	UBC	Maximum Magnitude Mmax (Mw)	Avg Slip Rate (mm/yr)	Avg Return Period (yrs)	Fault Length (km)	Date of Last Rupture (year)	Largest Historic Event >5.5M	Largest Historic Event (year)	Mean Site PGA (g)
Reference Notes: (1)			(2)	(3)	(4)	(2)	(2)	(2)		(5)		(6)
San Andreas - Southern (C V +S B M)	0.2	0.2	SS	A	7.4	24	220	203	c. 1690			0.61
San Andreas - San Bernardino Mtn.	0.2	0.2	SS	A	7.3	24	433	107	1812	7.0	1812	0.60
San Andreas - Banning Branch	0.7	1.2	SS	A	7.1	10	220	98		6.2	1986	0.56
San Andreas - Mission Crk. Branch	4.1	6.7	SS	A	7.1	25	220	95		6.5	1948	0.42
Morongo	5.1	8.2	SS	C	6.5	0.6	1170	23		5.5	1947	0.29
San Andreas - Coachella Valley	8.2	13.2	SS	A	7.1	25	220	95	c. 1690			0.28
Pinto Mountain	9.2	14.8	SS	B	7.0	2.5	500	73				0.24
Burnt Mountain	13	21	SS	B	6.4	0.6	5000	20	1992	7.3	1992	0.13
Eureka Peak	16	25	SS	B	6.4	0.6	5000	19	1992	6.1	1992	0.11
San Jacinto (Hot Spgs - Buck Ridge)	18	30	SS	C	6.5	2	354	70		6.3	1937	0.10
Landers	19	30	SS	B	7.3	0.6	5000	83	1992	7.3	1992	0.16
Blue Cut	21	33	SS	C	6.8	1	760	30		--		0.11
San Jacinto -Anza	23	36	SS	A	7.2	12	250	90	1918	6.8	1918	0.12
San Jacinto -San Jacinto Valley	23	37	SS	B	6.9	12	83	42		6.8	1899	0.10
North Frontal Fault Zone (East)	26	41	DS	B	6.7	0.5	1730	27				0.09
Johnson Valley (Northern)	27	44	SS	B	6.7	0.6	5000	36		--		0.07
Emerson So. - Copper Mtn.	27	44	SS	B	6.9	0.6	5000	54		--		0.08
Lenwood-Lockhart-Old Woman Spgs	28	45	SS	B	7.3	0.6	5000	149				0.11
North Frontal Fault Zone (West)	32	52	DS	B	7.0	1	1310	50				0.09
Helendale - S. Lockhardt	33	53	SS	B	7.1	0.6	5000	97				0.08
San Jacinto - Coyote Creek	34	55	SS	B	6.8	4	175	40	1968	6.5	1968	0.06
Calico -Hidalgo	35	56	SS	B	7.1	0.6	5000	95				0.07
Pisgah-Bullion Mtn.-Mesquite Lk	36	58	SS	B	7.0	0.6	5000	88	1999	7.1	1999	0.07
San Jacinto -San Bernardino	36	58	SS	B	6.7	12	100	35		6.0	1923	0.05
Cleghorn	41	66	SS	B	6.5	3	216	25				0.04
Elsinore - Temecula	45	73	SS	B	6.8	5	240	42				0.05
Elsinore - Julian	46	74	SS	A	7.1	5	340	75				0.05
Elsinore- Glen Ivy	47	76	SS	B	6.8	5	340	38		6.0	1910	0.04
Cucamonga	50	80	DS	A	7.0	5	650	28				0.05
Earthquake Valley	53	85	SS	B	6.5	2	351	20				0.03
Chino-Central Avenue	56	89	DS	B	6.7	1	882	28				0.04
San Jacinto - Borrego Mountain	57	92	SS	B	6.6	4	175	29		6.5	1942	0.03
Whittier	59	95	SS	B	6.8	2.5	641	37				0.03

Notes:
1. Jennings (1994) and CDMG (1996)
2. CDMG & USGS (1996), SS = Strike-Slip, DS = Dip Slip
3. ICBO (1997), where Type A faults: Mmax > 7 and slip rate >5 mm/yr &Type C faults: Mmax <6.5 and slip rate < 2 mm/yr
4. CDMG (1996) based on Wells & Coppersmith (1994), Mw = moment magnitude
5. Modified from Ellsworth Catalog (1990) in USGS Professional Paper 1515
6. The estimates of the mean Site PGA are based on the following attenuation relationships:
   Average of: (1) 1997 Boore, Joyner & Fumal; (2) 1997 Sadigh et al; (3) 1997 Campbell
   (mean plus sigma values are about 1.6 times higher)
   Based on Site Coordinates: 33.948 N Latitude, 116.609 W Longtude and Site Soil Type C

APPENDIX A

Boring Logs from Preliminary Soils and Geologic Investigations, WECS 20

BORING SUMMARY NO. BH-3

ELEVATION: N/I

DATE DRILLED: January 7, 1985

DEPTH IN FEET	SAMPLES	BLOW COUNT PER FOOT	FIELD MOISTURE % DRY WEIGHT	DRY DENSITY LB./CU. FT.	RELATIVE COMPACTION %	UNIFIED SOIL CLASSIFICATION	MATERIAL DESCRIPTION			
1							Silty sand, fine to coarse, with trace of roots	loose	slightly moist	red/ brown
2	1*	27/5"	13.5	102.0			Silty sand, fine, to ½-inch gravel	dense		
3							Sand, fine to coarse, to ½-inch gravel	very dense	dry	light brown
4	SPT	35/4"	5.3	77.9						
5										
6										
7										
8	SPT	50/ 5½"	8.5	116.2		SW-SM	Sand, fine to coarse, with ½ to 1-inch gravel			
9										
10										
11							2 to 3-inch gravel			
12										
13	SPT	50/ 5½"	3.1	125.6			Gravelly			
14										
15										
16										
17										
18	SPT	50/6"	5.1	128.4						
19										
20										
21										
22										
23	SPT	50/ 5½"	5.2	126.3			Sand to ½-inch gravels			
24										
25							(continued)			

* 2.5" I.D. Ring Sampler
SPT - Standard Penetration Test

WECS 20
Desert Hot Springs Area, California
for Energy Unlimited, Inc.

EXHIBIT NUMBER 5

Pioneer Consultants

JOB NUMBER: 3823-001

Approved For Report On ______ by ______

BORING SUMMARY NO. BH-3

ELEVATION: N/I

DATE DRILLED: January 7, 1985

DEPTH IN FEET	SAMPLES	BLOW COUNT PER FOOT	FIELD MOISTURE % DRY WEIGHT	DRY DENSITY LB./CU. FT.	RELATIVE COMPACTION %	UNIFIED SOIL CLASSIFICATION	MATERIAL DESCRIPTION			
26							Sand	very dense	dry	light brown
27										
28	SPT	50/5"	4.5	132.7						
29							Less coarse sand			
30										
31										
32										
33	SPT	50/4"	6.5	129.3						
34										
35										
36							TOTAL BORING DEPTH 35.0 FEET NO GROUNDWATER ENCOUNTERED			
37										
38										
39										
40										
41										
42										
43										
44										
45										
46										
47										
48										
49										
50										

SPT - Standard Penetration Test

Approved For Report On ______ By ______

WECS 20
Desert Hot Springs Area, California
for Energy Unlimited, Inc.

EXHIBIT NUMBER 5A

Pioneer Consultants

BORING SUMMARY NO. BH-4

ELEVATION: N/I

DATE DRILLED: January 8 and 9, 1985

DEPTH IN FEET	SAMPLES	BLOW COUNT PER FOOT	FIELD MOISTURE % DRY WEIGHT	DRY DENSITY LB./CU. FT.	RELATIVE COMPACTION %	UNIFIED SOIL CLASSIFICATION	MATERIAL DESCRIPTION			
1							Silty sand	loose	slightly moist	red/ brown
2	1*	50/5"	3.5	106.7			Silty sand to ½-inch gravel	very dense	dry	light reddish brown
3										
4	2*	31/2"	4.3	83.5		SP-SC				
5							Increasing gravel ½ to 1 inch			
6										
7										
8	SPT	50/ 4½"	3.8	138.8						light brown
9										
10										
11										
12										
13	SPT	50/5"	2.3	130.5			Gravelly sand			
14										
15										
16							Sand, fine to very coarse, trace of ½-inch gravel			
17										
18	SPT	50/4"	2.3	136.1						
19										
20							Increasing gravel			
21										
22										
23	SPT	50/ 1½"	2.2	135.8						
24							TOTAL BORING DEPTH 23.0 FEET (REFUSAL ON COBBLES) NO GROUNDWATER ENCOUNTERED			
25										

* 2.5" I.D. Ring Sampler
SPT - Standard Penetration Test

WECS 20
Desert Hot Springs Area, California
for Energy Unlimited, Inc.

EXHIBIT NUMBER 6

Pioneer Consultants

JOB NUMBER:

PRELIMINARY SOILS AND GEOLOGIC INVESTIGATION
WECS 20, W½ OF SECTION 31, TOWNSHIP 2 SOUTH, RANGE 4 EAST, S.B.B.M.
RIVERSIDE COUNTY, CALIFORNIA
PIONEER CONSULTANTS
1985

554-1.2

PRELIMINARY SOILS AND GEOLOGIC INVESTIGATION

WECS 20

W½ of Section 31, T2S, R4E, S.B.B.M.

Riverside Couinty, California

J.N. 3923-001

February 4, 1985

for

Energy Unlimited, Inc.
19345 Indian Avenue
Desert Hot Springs, California 92240

prepared by

Pioneer Consultants
251 Tennessee Street
Redlands, California 92373

pioneer consultants
251 TENNESSEE STREET • REDLANDS, CALIFORNIA 92373 • (714) 793-2691

J.N. 3923-001
August 1, 1985

Energy Unlimited, Inc.
11360 Palm Drive, Suite C
Desert Hot Springs, California 92240

Attention: Mr. B. C. Lees

Re: WECS 20 - W½ of Section 31, T2S, R4E, S.B.B.M.
Riverside County, California

Subject: Passive Pressure Calculations and Potential for Water Scour

Gentlemen:

Subsequent to a review of our report dated February 4, 1985 by the County, Mr. Jon Reynolds of Krieger & Stewart requested that we respond to the subject areas of concern. Included with this letter as Appendix A are the calculations used to develop the passive pressures and lateral loads presented in the above-referenced report.

The "Potential for Water Erosion, Sedimentation and Flooding" section in the above report addresses in general terms the potential for scour. As pointed out under the above section, considerable quantities of water will move down the normally dry canyons during periods of intense rainfall. However, this water will not exceed ½ to 1 foot in depth across the channels during peak flows. Based on field observations, whatever scour is initially produced will be quickly replaced by sedimentation during the reduced water flows at the end of flooding. The short duration of desert storms (on the order of minutes) and lack of incised channels observed in the field indicate a lack of scour in all the existing channels. The braided stream pattern in the bottom of the channels does not promote erosion but rather deposition. It is our opinion that the minor scour which may occur should not affect foundation integrity.

We hope this information is sufficient for your needs. If you should have any questions, please feel free to contact this office.

Very truly yours,

PIONEER CONSULTANTS

Warren L. Sherling, C.E.G. #1182
Project Geologist

WLS:ljs
Attachment

cc: (4) Krieger & Stewart Incorporated, Attention: Mr. Jon Reynolds

pioneer consultants
251 TENNESSEE STREET • REDLANDS, CALIFORNIA 92373 (714) 889-8506
(714) 793-2691

Qal unit (wash)

Lateral loads

$\phi = 41-43^\circ$ $\gamma_{dry} = 113$ (min) $\gamma_{wet} = 129$ max.

Level backfill set back = 3.2-3.5H

log spiral (NAVFAC) $\phi = 41^\circ$

$K_a = .215 \ (129) = 27.7$ use 28 PSF/ft

$K_o = (1-\sin\phi)\gamma = .34 \ (129) = 44.3$ use 44 PSF/ft

$K_p = 19 \ (\overset{.735}{.770} - .665)(113) = 1578$ use 1500 PSF/ft

Rankine (smooth element)

$K_a = \tan^2(45 - \phi/2)\ \gamma = .207 \ (129) = 26.8 \approx 27$ PSF/ft

$K_p = \tan^2(45 + \phi/2)\ \gamma = 4.81 \ (113) = 544$ PSF/ft

$\delta = 2/3\ 40 = 26^\circ$

Coulomb (rough element) $\alpha = 90^\circ$, $\beta = 0$, $\phi = 40^\circ$ Table 11-1/2 Bowles

$K_a = .199\,(\gamma) = .199\,(129) = 25.6$ use 26 PSF/ft

$K_p = 13.364\,(113) = 1510$ PSF/ft use 1500 PSF

∴ for level ground 3.5H away from element

$K_a = 28$ PSF/ft

$K_o = 44$ PSF/ft

$K_p = 1500$ PSF/ft

Level

natural ground slopes 20' in 200' or 1:10 $\beta = 6^\circ$ $\beta/\phi = .14$

log spiral (NAVFAC) $\phi = 41$ $\beta/\phi = .14$

slope towards $K_a = .225 \ (129) = 29$ PSF/ft

slope away $K_p = 14.5 \ (.735)(113) = 1204$ PSF/ft

Coulomb (rough) $\alpha = 90^\circ$ $\beta = 6^\circ$ $\delta = 26^\circ$ $\phi = 40^\circ$ Table 11-1 Bowles

toward $K_a = .211 \ (129) = 27$ PSF/ft

away $$K_p = \frac{\sin^2(\alpha - \phi)}{\sin^2\alpha \,\sin(\alpha-\delta)\left[1 - \sqrt{\dfrac{\sin(\phi+\delta)\sin(\phi+\beta)}{\sin(\alpha+\delta)\sin(\alpha+\beta)}}\right]^2} \ (\gamma) =$$

(EQ 11-4 Bowles)

$\delta = 26^\circ$ $\phi = 41^\circ$

$\beta = -6^\circ$

$$\frac{.56958}{.89879\left[\ .2314\ \right]^2} = 11.829$$

$K_p = 11.829\,(113) = 1336$ PSF/ft

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Qal unit (wash)

Rankine (smooth element) $\phi = 41^\circ$ $\beta = 6^\circ$

K_a (Table 11-3 Bowles) =
5 .2099
6 .2104 $\times \gamma =$ 25 PSF/ft
10 .2145

$\beta = -6^\circ$ $\phi = 41^\circ$

$$K_p \text{ (EQ 11-9)} = \cos -6^\circ \frac{\cos -6 + \sqrt{\cos^2 -6 - \cos^2 41}}{\cos -6 - \sqrt{\cos^2 -6 - \cos^2 41}} = \frac{.6406}{.3468} = 1.8469$$

$K_p = 208$ PSF/ft

∴ Use sloping N.G.

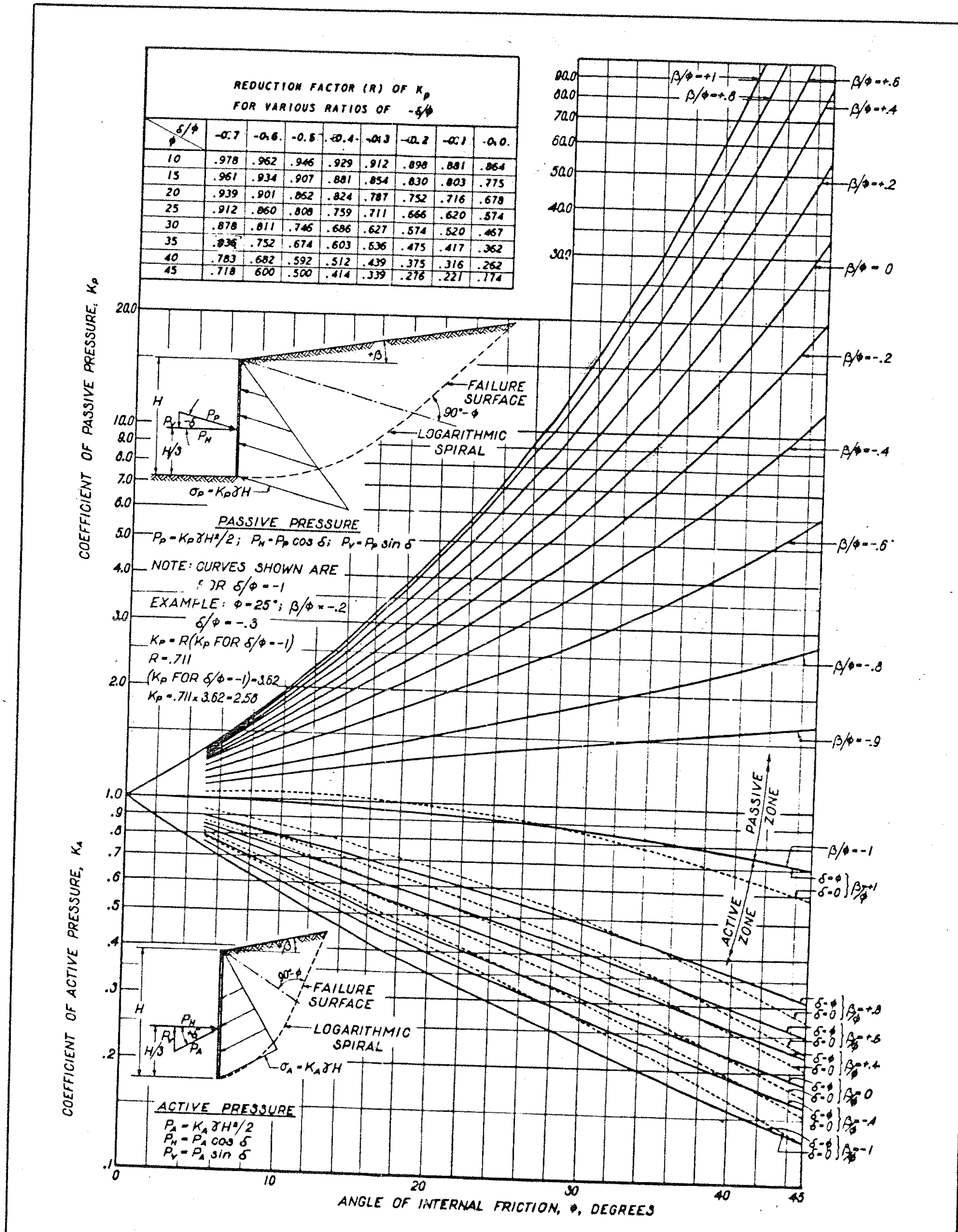
$K_a = 29$ PSF/ft

$K_p = 1200$ PSF/ft

JOB NO.: 3923-001 PROJECT: EN-UNLIM BY: RBR DATE: 24 Jan 85

FORM 204 - 9/84 - 3M

APPENDIX A SHEET 3 OF

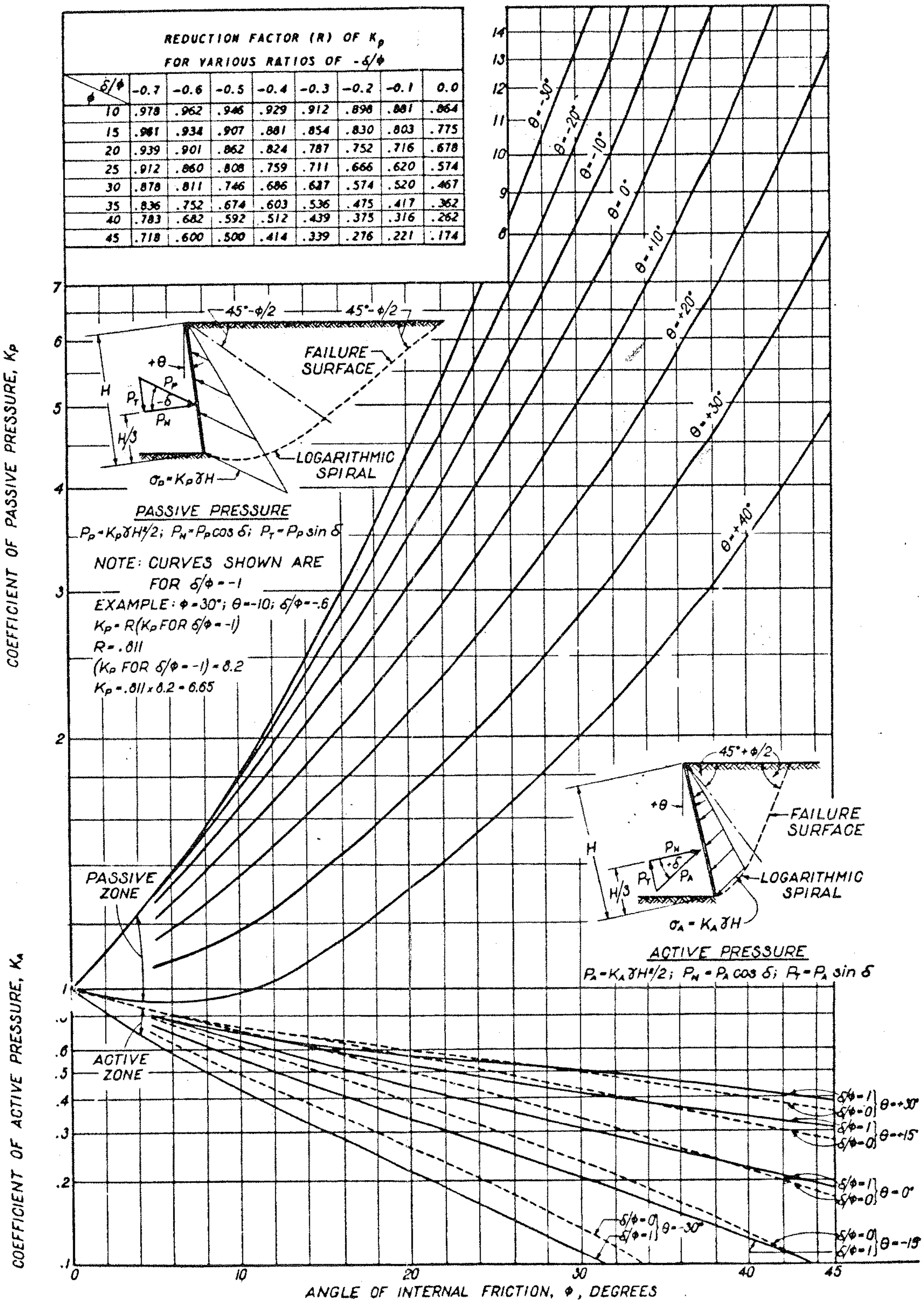


REDUCTION FACTOR (R) OF K_p FOR VARIOUS RATIOS OF $-\delta/\phi$

ϕ \ δ/ϕ	-0.7	-0.6	-0.5	-0.4	-0.3	-0.2	-0.1	-0.0
10	.978	.962	.946	.929	.912	.898	.881	.864
15	.961	.934	.907	.881	.854	.830	.803	.775
20	.939	.901	.862	.824	.787	.752	.716	.678
25	.912	.860	.808	.759	.711	.666	.620	.574
30	.878	.811	.746	.686	.627	.574	.520	.467
35	.836	.752	.674	.603	.536	.475	.417	.362
40	.783	.682	.592	.512	.439	.375	.316	.262
45	.718	600	.500	.414	.339	.276	.221	.174

FIGURE 10-3
Active and Passive Coefficients with Wall Friction (Sloping Backfill)

NAVFAC



REDUCTION FACTOR (R) OF K_p FOR VARIOUS RATIOS OF $-\delta/\phi$

ϕ \ δ/ϕ	-0.7	-0.6	-0.5	-0.4	-0.3	-0.2	-0.1	0.0
10	.978	.962	.946	.929	.912	.898	.881	.864
15	.961	.934	.907	.881	.854	.830	.803	.775
20	.939	.901	.862	.824	.787	.752	.716	.678
25	.912	.860	.808	.759	.711	.666	.620	.574
30	.878	.811	.746	.686	.627	.574	.520	.467
35	.836	.752	.674	.603	.536	.475	.417	.362
40	.783	.682	.592	.512	.439	.375	.316	.262
45	.718	.600	.500	.414	.339	.276	.221	.174

FIGURE 10-4
Active and Passive Coefficients with Wall Friction (Sloping Wall)

Table 11-1 Active-earth-pressure coefficients K_a based on the Coulomb equation (11-3)

ALPHA =90. BETA = 0.

δ	ϕ = 26	28	30	32	34	36	38	40
0	0.390	0.361	0.333	0.307	0.283	0.260	0.238	0.217
16	0.349	0.324	0.300	0.278	0.257	0.237	0.218	0.201
17	0.348	0.323	0.299	0.277	0.256	0.237	0.218	0.200
20	0.345	0.320	0.297	0.276	0.255	0.235	0.217	0.199
22	0.343	0.319	0.296	0.275	0.254	0.235	0.217	0.199

ALPHA =90. BETA = 5.

δ	ϕ = 26	28	30	32	34	36	38	40
0	0.414	0.382	0.352	0.323	0.297	0.272	0.249	0.227
16	0.373	0.345	0.319	0.295	0.272	0.250	0.229	0.210
17	0.372	0.344	0.318	0.294	0.271	0.249	0.229	0.210
20	0.370	0.342	0.316	0.292	0.270	0.248	0.228	0.209
22	0.369	0.341	0.316	0.292	0.269	0.248	0.228	0.209

ALPHA =90. BETA =10.

δ	ϕ = 26	28	30	32	34	36	38	40
0	0.443	0.407	0.374	0.343	0.314	0.286	0.261	0.238
16	0.404	0.372	0.342	0.315	0.289	0.265	0.242	0.221
17	0.404	0.371	0.342	0.314	0.288	0.264	0.242	0.221
20	0.402	0.370	0.340	0.313	0.287	0.263	0.241	0.220
22	0.401	0.369	0.340	0.312	0.287	0.263	0.241	0.220

Table 11-2 Passive-earth-pressure coefficients K_p based on the Coulomb equation (11-6)

ALPHA =90. BETA = 0.

δ	ϕ = 26	28	30	32	34	36	38	40
0	2.561	2.770	3.000	3.255	3.537	3.852	4.204	4.599
16	4.195	4.652	5.174	5.775	6.469	7.279	8.230	9.356
17	4.346	4.830	5.385	6.025	6.767	7.636	8.662	9.882
20	4.857	5.436	6.105	6.886	7.804	8.892	10.194	11.771
22	5.253	5.910	6.675	7.574	8.641	9.919	11.466	13.364

ALPHA =90. BETA = 5.

δ	ϕ = 26	28	30	32	34	36	38	40
0	2.943	3.203	3.492	3.815	4.177	4.585	5.046	5.572
16	5.250	5.878	6.609	7.464	8.474	9.678	11.128	12.894
17	5.475	6.146	6.929	7.850	8.942	10.251	11.836	13.781
20	6.249	7.074	8.049	9.212	10.613	12.321	14.433	17.083
22	6.864	7.820	8.960	10.334	12.011	14.083	16.685	20.011

ALPHA =90. BETA =10.

δ	ϕ = 26	28	30	32	34	36	38	40
0	3.385	3.713	4.080	4.496	4.968	5.507	6.125	6.841
16	6.652	7.545	8.605	9.876	11.417	13.309	15.665	18.647
17	6.992	7.956	9.105	10.492	12.183	14.274	16.899	20.254
20	8.186	9.414	10.903	12.733	15.014	17.903	21.636	26.569
22	9.164	10.625	12.421	14.659	17.497	21.164	26.013	32.602

APPENDIX A

(from Bowles)

pioneer consultants
251 TENNESSEE STREET • REDLANDS, CALIFORNIA 92373 • (714) 793-2691

J.N. 3923-001
February 4, 1985

Energy Unlimited, Inc.
19345 Indian Avenue
Desert Hot Springs, California 92240

Attention: Mr. Clare Lees

Re: WECS 20 - W½ of Section 31, T2S, R4E, S.B.B.M.
Riverside County, California

Subject: Preliminary Soils and Geologic Investigation

Gentlemen:

At your request, we have made a preliminary soils and geologic investigation on the above-referenced site. The purpose of our investigation was to evaluate the subsurface condition of the site and make geotechnical recommendations for wind generator foundations and access roadway grading. Our investigation included five rotary auger borings on the site, office and field geologic investigation of the site, laboratory testing of the acquired soil samples, geotechnical engineering analysis and the preparation of this report.

We have found the site to be suitable for both shallow footings and deep caisson foundations. Our specific recommendations are included in the attached report.

It has been our pleasure to be of service and, if there are any further questions, please call at your convenience.

Very truly yours,

PIONEER CONSULTANTS

Robert B. Rogers

Robert B. Rogers, P.E.
Geotechnical Engineer

Michael C. Shea

Michael C. Shea, R.G. #3262
Project Geologist

REGISTERED PROFESSIONAL ENGINEER
ROBERT B. ROGERS
RBR
NO. 26370
4 Feb 1985
CIVIL
STATE OF CALIFORNIA

RBR:ljs
Addressee (2)
Attachment

cc: Krieger & Stewart Incorporated

SITE AND PROJECT DESCRIPTION

The proposed WECS 20 site is located in the W½ of Section 31, T2S, R4E and consists of the entire area with the exception of the north half of the northwest quarter. Its location is shown on Exhibit 1 attached.

The main access to the proposed wind farm site was via a dirt service road associated with the Colorado River Aqueduct. The road trends south and west from West Pierson Boulevard. An additional access road is proposed in the NW¼ of Section 6 to the south.

The site is at present unoccupied and undeveloped except for: (a) evidence of past mine activity; (b) dirt access roads and (c) scattered anemometer locations. Drainage across the site is to the southeast in the northern portion and to the south in the southern portion.

The topography of the site is highly irregular consisting of eastward and southward sloping ridges and stream channels. The overall effect is that of a highly desiccated alluvial fan surface. Regional slope on the ridgetops is 5 degrees to the southwest. Approximately half of the site consists of slopes which are equal to or exceed 25 percent grade. These areas are indicated with cross hatching on the enclosed site plan (Exhibit 2, 2 sheets) prepared by Krieger & Stewart of Riverside. Those slopes in excess of 25 percent are not included in this study. The Colorado River Aqueduct of the Metropolitan Water District lies just east of the eastern boundary of the site and is shown on the enclosed plan (Exhibit 2).

The ground surface is characterized by desert pavement consisting of very coarse sands, gravels, and cobbles. Boulders to 3 feet in diameter are scattered on the surface but are generally confined to the ridgetop areas. Vegetation on the site consists of scrub brush and cactus and covers approximately 10 to 15 percent of the ground surface. As is shown on Exhibit 2 (Sheets 1 and 2), the proposed WECS 20 development will consist of pylon-mounted wind generators adjacent to compacted soil access roads. These will be located primarily in the alluvial valleys and on the native sloping ground that does not exceed 25 percent in grade. Each wind generator will be spaced approximately 70 feet apart.

Access road grading is proposed in the area immediately south of the NW¼ of Section 6 as shown on Exhibit 2 (Sheet 2 of 2).

SCOPE OF SERVICES

A preliminary soil and geologic investigation of the wind farm site was made that included:

1. Examination of pertinent available published and unpublished information relative to the site and the surrounding general region.

2. A field investigation consisting of five 8-inch hollow stem auger borings to depths as great as 35 feet and a reconnaissance geologic mapping of the site by our staff geologist. Logs of the borings are included as Exhibits 3 through 7. Representative bulk and undisturbed samples were obtained during the subsurface investigation and returned to our laboratory for testing and analysis.

3. Laboratory testing of representative samples retained and the results are included as Exhibits 8 through 14.

4. Preparation of a geotechnical report presenting our findings, conclusions and recommendations concerning the development of the proposed wind farm site. Our evaluation addressed all of the points outlined in the Riverside County Planning Department's requirements for "WECS Geotechnical Reports".

FIELD INVESTIGATION

The field investigation consisted of five borings to depths of 15.5 to 35.0 feet from the existing ground surface. The borings were advanced using an 8-inch hollow stem rotary flight auger. Drive samples were obtained using both the 2.5-inch internal diameter ring sampler and the Standard Penetration Test sampler. The samplers were advanced using a 140-pound hammer free-falling 30 inches, and the number of blows required to advance the sampler the last 12 inches of an 18-inch drive are recorded on the log sheets. These values are used to evaluate the relative consistency of the subsoils.

All of the samples retained in the investigation were returned to our laboratory for testing and analysis. The locations of the borings are shown on the attached plot plan, Exhibit 2 (Sheets 1 and 2 of 2); and the boring logs accompany this report as Exhibits 3 through 7.

LABORATORY TESTING

The following tests were performed for this project in our laboratory in accordance with the American Society for Testing and Materials or contemporary practices of the soil engineering profession.

In-Situ Moisture and Density: This test consisted of weighing and measuring the drive samples obtained from the borings to determine their in-place moisture and density. These results are used to analyze the consistency of the subsoils.

Standard Penetration Test: The Standard Penetration Test, although a field test, is used to determine the relative density of the subsoil and also the allowable bearing value and total settlement. This is basically for cohesionless materials.

Maximum Density - Optimum Moisture Determination: This test determines the density that a soil can be compacted at various moisture contents. For each soil mixture, there is a maximum dry density obtained and the associated optimum moisture content. The results are used to evaluate the natural compaction, the control of the grading process and as an aid in developing the soil bearing capacity. This test is based on the ASTM Standard D1557.

Sieve Analysis: This test determines the size of the soil grains which constitutes a soil and is used in generating an engineering classification of the soil.

Plasticity Index: This is determined from the liquid limit and the plastic limit of the soils. The liquid limit is the moisture content at which the soil changes from a plastic to a liquid state, and the plastic limit is the moisture content at which the soil changes from a semi-solid to a plastic state. The difference in these values is the Plasticity Index and is the range of moisture content at which the soil is in a plastic condition. It is used to aid in the classification of the soil.

Direct Shear: This test is used to confirm the ultimate soil bearing value, as well as, lateral load resistance and slope stability analysis.

GEOLOGY

Previous mapping (Proctor, 1968) and our field mapping show that the site is underlain by three distinctive geologic units. The oldest exposed geologic unit on the site is the Painted Hill formation (Unit Tph) of early Pleistocene Age. This alluvial unit consists of gray and light brown, poorly-sorted beds of conglomerate and/or arkosic conglomerate sandstone. Unconformably overlying the Painted Hill formation is the Quaternary age Cabezon fanglomerate (Unit Qc). The Cabezon fanglomerate consists of poorly-sorted and bedded, pebbly and bouldery tan arkosic sandstone with clasts of gneiss, granitic rocks and minor amounts of basalt. Capping the Cabezon fanglomerate on ridgetops is a thin mantle of terrace deposits consisting of orange-tinted sands and gravels and is further characterized by large boulders protruding conspicuously from the ground surface. The boulders are present in trace amounts. Overlying the Painted Hill formation and the Cabezon fanglomerate in the valley areas is recent stream alluvium (Unit Qal) consisting of coarse sands and pebble gravels.

Faulting

No active or potentially active faulting is known or has previously been mapped crossing the proposed wind farm site, and no portion of the wind farm site is included within an Alquist-Priolo Special Studies Zone. The nearest active or potentially active faults to the site include the Banning Fault (considered to be the southern branch of the San Andreas Fault), located approximately ½ mile south of the main wind farm site, and the Mission Creek Fault (considered to be the northern branch of the San Andreas Fault), located approximately 4 miles northeast of the main wind farm site. Both the Mission Creek and the Banning Fault are considered active by the State of California (Hart, 1980). The Banning Fault is shown on Exhibit 2 (Sheet 2 of 2) passing northwesterly through the central portion of Section 6 and, thus, through the alignment of the proposed access road to the site. The limits of the Special Studies Zone to the north and south of the fault are also shown.

Seismic Setting

The regional seismic setting is shown on Exhibit 15. As discussed under "Faulting", the nearest active or potentially active faults to the site include the Banning and Mission Creek Faults and, slightly more distant, the San Jacinto Fault (24 miles to the southwest). Although the Banning Fault passes nearest the wind farm site, no historic seismic activity has been assigned to this branch of the San Andreas. By comparison, the Mission Creek Fault is considered the more active northern branch of the San Andreas Fault (Proctor, 1968; Riverside County, 1976). For the evaluation of the site, we have chosen the more active Mission Creek Fault as the "design fault" when evaluating seismic parameters which could affect the site.

Historic Seismicity

Only one earthquake of greater than 6.0 magnitude has been assigned to the San Andreas Fault within 100 miles of the site during the last 100 years (Hileman, Allen and Nordquist, 1974). This was the magnitude 6.5 event at Desert Hot Springs in 1948, approximately 3.8 miles to the east. An event of estimated magnitude 6.5 occurred along the San Andreas in 1868 near the Salton Sea, approximately 30 miles to the southeast. However, these records are incomplete. Some surface rupture also occurred along the trace of the San Andreas Fault near the Salton Sea as a result of sympathetic movement during the 1968 magnitude 6.5 earthquake centered on the Coyote Creek Fault. No surface rupture is anticipated at any point on the site as the result of a major earthquake along either the Banning or Mission Creek Faults. However, surface rupture is anticipated to occur through the proposed access roadway during a major earthquake along the Banning Fault.

Seismic Exposure

Although no precise method has been developed to evaluate the seismic potential of a specific fault, available information on historic seismic activity may be projected to estimate the future activity of the fault. This is usually done by plotting the historic seismic activity in terms of number of events in a given time interval versus magnitude of the events. The other method of determining potential seismicity of a fault is by evaluating accumulated stress and determining the size of the earthquake necessary to release this accumulated stress. Based on such data and plots, recurrence intervals for the earthquakes in given magnitudes may be estimated. Such data has been presented for the San Andreas Fault by Lamar, Merifield and Proctor (1973) and by the County of Riverside (1976). For this project, the maximum probable earthquake is defined as that with a recurrence period of 100 years or the greatest historic event, whichever is greater, as recommended by CDMG Note 43. Using the information referenced above, we estimate the maximum probable or "design earthquake" for the Mission Creek Fault to be 6.5 magnitude. Based on data presented by Greensfelder (1974), we estimate that the maximum credible event for the San Andreas Fault in this area [which includes both the Mission Creek Fault (north branch) and the Banning Fault (south branch)] to be an event of magnitude 7.5. The maximum credible event is the greatest event that the fault appears theoretically capable of producing without a consideration of time interval based upon the present tectonic framework.

Ground Motion Characteristics

The ground motion characteristics for the postulated maximum probable earthquake of magnitude 6.5 on the Mission Creek Fault were estimated. Available information in literature about the maximum peak bedrock acceleration and attenuation with distance (Schnabel and Seed, 1973), the effects of site-soil conditions on surface ground motion parameters (Idriss, 1978) and site-response criteria (Hays, 1980) were utilized. This information indicates that a maximum peak rock acceleration on the order of 0.52 g may be anticipated at the site for a magnitude 6.5 event epicentered along the Mission Creek Fault at a point nearest the site (4 miles). The extreme soils engineering differences between the recent alluvium and the older bedrock (either terraced gravels in Cabezon fanglomerate or only Cabezon fanglomerate) would produce considerably different maximum ground surface acceleration values. The maximum ground acceleration in those areas underlain by bedrock is expected to be only slightly less than peak rock acceleration. This is due to the response of very thick, but essentially rocklike, older alluvial deposits above the hard, high-velocity crystalline bedrock. Therefore, in these areas underlain by either Tph, or Qc with thin cover of terrace deposits, we estimate the maximum ground surface acceleration to be 0.39 g based on the work by Idriss (1978). However, within those areas underlain by the younger alluvium (Qal Unit), the relatively thin deposit of alluvium (estimated at less than 100 feet) would amplify, rather than dampen, the peak ground surface acceleration according to the data presented by Riverside County (1976). Therefore, in the alluvial areas, we estimate the maximum surface acceleration to be 0.46 g based on a slight damping by the underlying Cabezon fanglomerate and a slight amplification by the overlying thin alluvial deposits. Repeatable ground acceleration can be estimated at 65 percent of peak ground acceleration for design purposes (Ploessel and Slosson, 1974) with a value of 0.25 g for the areas underlain by Cabezon fanglomerate and about 0.30 g for the areas underlain by 100 feet or less of recent alluvium. The predominant period of bedrock acceleration is expected to be 0.35 second or less with more than 18 seconds of strong ground shaking.

Secondary Seismic Hazards

Of the other secondary seismic hazards, such as, surface rupture, liquefaction, settlement, or seismically-triggered rock slides/landslides, only seismically-triggered rock slides might affect the site. These potential rock slides would be very local in nature and would only occur along the very steep [1½:1 (horizontal to vertical) or steeper] slopes which presently exist at the site. It should be noted that these steep slopes have very limited heights. Actual run out of the boulders down the slope faces would probably be on the order of less than 5 feet.

SUBSURFACE CONDITIONS

Three basic geologic units were encountered on this site consisting of the Tertiary Painted Hill Formation (Tph), the Quaternary Cabezon fanglomerate (Qc) and the Quaternary recent alluvium (Qal) which are outlined on our plot plan, Exhibit 2 (Sheets 1 and 2 of 2). The majority of the construction is proposed to be in the Quaternary Cabezon fanglomerate (Qc) and the Quaternary recent alluvium (Qal). All of the subsurface explorations were in one of these two units.

The recent alluvium was investigated by boring BH-2. The material consisted of a clean, medium to coarse sand in the upper 4 to 5 feet with coarse sand and interbedded 1 to 4-inch thick gravel beds underneath that. Moisture contents ranged from a low of 1.1 percent at a depth of 8.0 feet to a high of 4.7 percent at a depth of 12.5 feet. Dry densities ranged from a low of 112.7 pounds per cubic foot (pcf) at a depth of 4.5 feet to a high of 137.9 pcf at a depth of 8.0 feet. The higher densities reflect greater gravel contents; the lesser density reported at a depth of 12 to 13 feet is felt to be a disturbed sample due to the very dense nature and high blow counts present.

The Cabezon fanglomerate was investigated by borings BH-1, BH-3 and BH-5. These materials were found to consist primarily of a surface, loose silty sand, approximately 1 foot in depth, underlain by coarser to gravelly sands to silty gravels to the total depths encountered. Moisture contents ranged from a low of 0.8 percent at a depth of 17.0 feet in boring BH-1 to a high of 13.5 percent at a depth of 1.5 feet in BH-3. The higher moisture contents reflect silt content and recent rainfall. Dry densities ranged from a low of 94.0 pcf at a depth of 2.5 to 3.0 feet in BH-5 to 151.3 pcf at a depth of 23 feet in BH-5. The higher densities reflect an increase in gravel content of the soils; the lesser densities probably indicate sampling disturbance due to the high blow count.

Generally, we would characterize both soil types as being only slightly dissimilar in undisturbed characteristics and to be the same material when recompacted as compacted fill. The in-situ properties of the Qal unit at the site had a dry unit weight of approximately 118 pcf, a cohesion intercept of 0 psf and friction angles ranging from 41 to 43 degrees. The native ground slopes approximately 10:1 (horizontal to vertical).

In the Qc unit the soil has a unit weight of approximately 110 pcf in the upper 5 to 6 feet, with a minimum friction angle of 40 degrees and a cohesion intercept of 0 to 100 psf. The ground slopes as steep as 4.5:1 (horizontal to vertical).

Thus, both units where construction is proposed consist of coarse sands and gravels that are in a dense to very dense native condition.

Potential for Wind Erosion

According to the work prepared by Halseth (1967), the site would be included in the Tujunga-Soboba Soil Association with between 10 and 20-percent slopes. This soil association consists of excessively drained, coarse-textured soils developed on a granitic alluvial base. Based upon the findings in that report and our field mapping which indicated very little evidence of blowing sand, the site has little or no wind-erosion potential. This is based on the current conditions where there is no source of finer sand to blow. Disturbed soils have from 16 to 30 percent passing the No. 40 sieve and therefore would be a very small source for wind-blown sand.

Potential for Water Erosion, Sedimentation and Flooding

Based upon the flood insurance maps prepared by the Federal Emergency Management Agency, the entire site lies above the Mission

Creek Drainage Flood Plain. However, based upon the topographic expression at the site and our field mapping (which indicated that the deeply-incised streams have been backfilled by rather coarse alluvial debris), these channels carried considerable quantities of moving water at some time in the past. Therefore, it is our opinion that isolated portions of the site presently within the existing southeast-draining stream channels would be subjected to minor flooding. Sedimentation would occur if the velocity of the water was retarded in anyway by the proposed project.

Because of the steep natural slopes involved, flooding will more than likely precipitate erosion rather than sedimentation for a given rainfall event. Periodic maintenance after storm events can be expected especially for proposed access roads that cross drainage channels.

Stability of Existing Cut and Fill Slopes

Presently, all of the slopes at the site are natural slopes. Within some of the more deeply incised canyons, the canyon walls founded in Cabezon fanglomerate stand at inclinations of near vertical with essentially no evidence of instability or raveling. On the other hand, the recent alluvium (despite its very dense nature) is noncohesive and easily eroded. Therefore, it is anticipated that cut or fill slopes within recent alluvial material should not be steeper than 2:1 (horizontal to vertical) and should be protected from surface erosion.

Stability of Proposed Cut and Fill Slopes

Because both geologic units have similar recompacted fill soil characteristics, we recommend that roadway fills be constructed less than 15 feet in height. Where these fills are constructed at a maximum slope of 2:1 (horizontal to vertical), the factor of safety is between 1.3 and 1.4. Where the slopes are constructed not steeper than 2.5:1 (horizontal to vertical), the factor of safety increases to the 1.6 to 1.7 range. These factors of safety are based upon static condition with internal pore pressure equal to half the slope height (worst condition). It is our professional opinion that fill slopes should not be constructed steeper than 2:1 (horizontal to vertical); and to achieve a minimum factor of safety of 1.5 for these conditions, they should not be constructed steeper than 2.5:1 (horizontal to vertical).

With the exception of certain portions of the Qal unit where the soils are loose in the upper 2 to 5 feet, the soils present on this site are predominantly more dense in a natural condition than they would be in a compacted fill condition. Therefore, we recommend that cuts not greater than 15 feet in height be constructed not steeper than 2:1 (horizontal to vertical). Proposed cut slopes greater than 15 feet in height might require flatter slopes based on individual cut slope field conditions at the time the grading is conducted.

Foundation Material Type

Two predominant soil types were encountered on this site, and separate recommendations are made for each. Both consist of sands to increasingly coarser gravels at depth which were encountered in a dense to very dense condition between 1 to 5 feet below the existing ground surface, depending upon the unit. The site is not level and each unit has a different natural slope to it.

Collapsible or Expansive Soils

No clays or other expansive soils were encountered on this site, nor was there evidence, in any of the subsurface explorations, of collapsible soils. Extremely low dry densities obtained in some of the samples are primarily a result of sampler disturbance due to the extremely dense nature of the subsurface soils encountered.

Settlements

The following recommendations are predicated on a 1-inch nominal settlement with a 3/4-inch differential settlement between adjacent footings of similar sizes and loads, normally to be anticipated in sandy and noncohesive foundation materials. These settlements should occur as the footings are constructed and loaded and should not be of a long-term nature.

Shallow Footing Recommendations

Spread footing foundations are a foundation option on this site. The recommended safe bearing capacity for each geologic unit and site condition is given below:

For the Qal Unit (Wash Area):

Footing Width (feet)	Level Ground (psf)	10:1 Ground Sloping Away (psf)	
2	4,200	3,300	4,800
4	10,200	8,100	
6	13,400	10,600	2,500

For the Qc Unit (Highland Area):

Footing Width (feet)	Level Ground (psf)	4:1 Ground Sloping Away (psf)
2	6,000	4,100
4	11,000	7,600
6	14,000	9,700

Isolated spread footings are recommended to have a minimum depth of embedment of 18 inches from lowest adjacent finish grade. Footings may be founded on dense, native, undisturbed soil or properly compacted fill soils.

Lateral Soil Loads

In the Qal Unit, the following pressures are recommended:

For level ground:

Active - 28 pounds per square foot per foot of soil depth (psf/ft)

At Rest - 44 psf/ft

Passive - 1500 psf/ft

When ground slopes 10:1 (horizontal to vertical):

Active - 29 psf/ft (when sloped toward the element)

Passive - 1200 psf/ft (when the ground slopes away from the element)

In the Qc Unit, the following pressures are recommended:

For level ground:

Active - 27 psf/ft

At Rest - 45 psf/ft

Passive - 1400 psf/ft

When the ground slopes at 4:1 (horizontal to vertical):

Active - 31 psf/ft (when sloped toward the element)
Passive - 730 psf/ft (when the ground slopes away from the element)

Active means the resisting element moves away from the soil; at rest means the resisting element does not move relative to the soil; and passive means the resisting element moves into the soil. The coefficient of friction between concrete footings and/or caissons and the soil may be taken as 0.5 in both units. This may be increased by one-third to resist wind and seismic loads.

Caisson Foundations

Caisson foundations are also a foundation option for the site. There is an insignificant applied vertical load and the previous recommendations for spread footings may be used at the bottom of any caisson to resist vertical loads and/or rotation. The bottom of the caisson in that case should be set back a minimum of seven times the caisson diameter from an exposed slope. This means that the horizontal distance from the open face of a slope to the bottom of the caisson would be 7 times the caisson diameter when the elevation at the bottom of the caisson and the elevation at the face of the slope are equal. Both soil units encountered on this site may be excavated vertically for caissons. The top 4.5 feet may require casing in the Qal unit, however.

The primary design loads on a caisson foundation would be a lateral load of 25.9 kips and a 1600 foot-kip moment applied to the top of the caisson. Caisson sizes were analyzed for level ground conditions for the soil conditions encountered at this site. The results are summarized below:

Caisson Diameter (feet)	Ground Surface Deflection (inches)	Maximum Moment in a Caisson (foot-kip)
5	0.25	1645
4	0.45	1640
3	1.65	1642

The above parameters are predicated on a level setback of 3.2 times the caisson length. Because the existing site slopes significantly in a direction against the passive resistance of overturning for the caissons, we recommend that either: (1) the access road run on the uphill side of each foundation; or, alternatively (2) the depth of the caisson be measured from the level of the road if it is placed on the downhill side of the caisson.

In consultation with Krieger & Stewart, a 5-foot diameter caisson was analyzed for ground sloping at 4:1 (horizontal to vertical) and the resultant moment diagram for a 20-foot deep caisson is included as

Exhibit 16. However, this is a short caisson, and it is more likely to rotate rather than to deflect due to moment loading. There are significant resistances to rotation due to the end bearing of the caisson. However, we recommend the full passive pressure of 730 psf/ft be used in the design of a 5-foot diameter caisson over the full 5-foot width of the caisson. If the 5-foot diameter caisson is sufficiently fixed that it does not rotate out of position, it is not likely to deflect more than ¼ inch under the above design loads of lateral load and overturning moment. The lateral stability against rotation should be addressed by the designer of the caisson.

Access Roadway Construction

It is not anticipated at this time that the access roadways would be paved but merely compacted in-place soils. Cuts and fills would be constructed as necessary to provide drivable road surface to access all of the wind machines. Basically, this technique of unimproved road construction has worked well in the past in the immediate area. Periodic maintenance will be required at stream crossings after rainfall and runoff events. Surfacing these roads with gravel in excess of ¼ inch would reduce the possibility of blowing sand and fines erosion.

General Site Grading

1. Clearing and Grubbing: The site as it exists had little to no development in the past, and currently 40 percent of the site is covered with native desert flora. We recommend that those areas of improvements and access roadways be excavated to ensure flora and root removal and that the resulting excavation be backfilled following procedures for compacted fill as recommended below.

2. Preparation of Footing Areas: Shallow footings may rest on dense, undisturbed, native soils. If loose native materials are present, a minimum of 12 inches of additional material should be removed and recompacted to meet the minimum density recommended below under "Placement of Compacted Fill".

3. Placement of Compacted Fill: Compacted fill is defined as that material which will be replaced in the areas of removal due to root removal, the placement of footings and wherever the grade is to be raised. All fill shall be compacted to a minimum 90 percent based upon the maximum density obtained in accordance with ASTM Standard D1557. The area to be filled will be prepared by benching completely into cut, scarifying the cut material remaining for a depth of 6 inches, adjusting the moisture to near optimum and recompacting to 90 percent of ASTM D1557 procedure maximum dry density.

4. Review of Grading Plan and Specifications: We recommend that the soil engineer have the opportunity to review the final grading plan and construction specifications to assure that they include the items of this soil and geology report for the benefit of the owner and the contractor.

5. Pre-Job Conference: Prior to the commencement of grading, a pre-job conference should be held with representatives of the owner, contractor, engineer and soil engineer in attendance. The purpose of this meeting shall be to clarify any questions relating to the intent of the grading recommendations and to verify that the project specifications comply with the recommendations of this report.

6. Testing and Inspection: During grading, density testing should be performed by a representative of the soil engineer in order to determine the degree of compaction being obtained. Where testing indicates insufficient density, additional compactive effort shall be applied, with the adjustment of moisture content where necessary, until 90 percent relative compaction is obtained. The maximum dry density shall be determined in accordance with ASTM Standard D1557 in all cases.

   Caisson foundations should be inspected as they are drilled and the concrete placed the same day as the caisson excavation is drilled. The concrete should be placed by either pump or tremie so that no concrete is allowed to drop or free-fall in the caisson excavation and so that the bottom of the pump or tremie pipe remains submerged the entire time the caisson is being filled. The bottom of the caisson shall be clean and free of loose material. The bottom of the caisson need not be flat.

GENERAL

The recommendations of this report are based on the assumption that all footings will be founded on either dense, undisturbed, native soils or recompacted fill soils. All footing/caisson excavations should be inspected prior to the placement of concrete in order to verify that footings/caissons are founded on satisfactory soils and are free of loose and disturbed materials. All grading and fill placement should be performed under the testing and inspection of a representative of the soil engineer.

The findings and recommendations of this report were prepared in accordance with contemporary geotechnical engineering principles and practice. We make no other warranty, either express or implied. Our recommendations are based on an interpolation of soil conditions between boring locations. Should conditions be encountered during excavation that appear to be different from those indicated by this report, this office should be notified.

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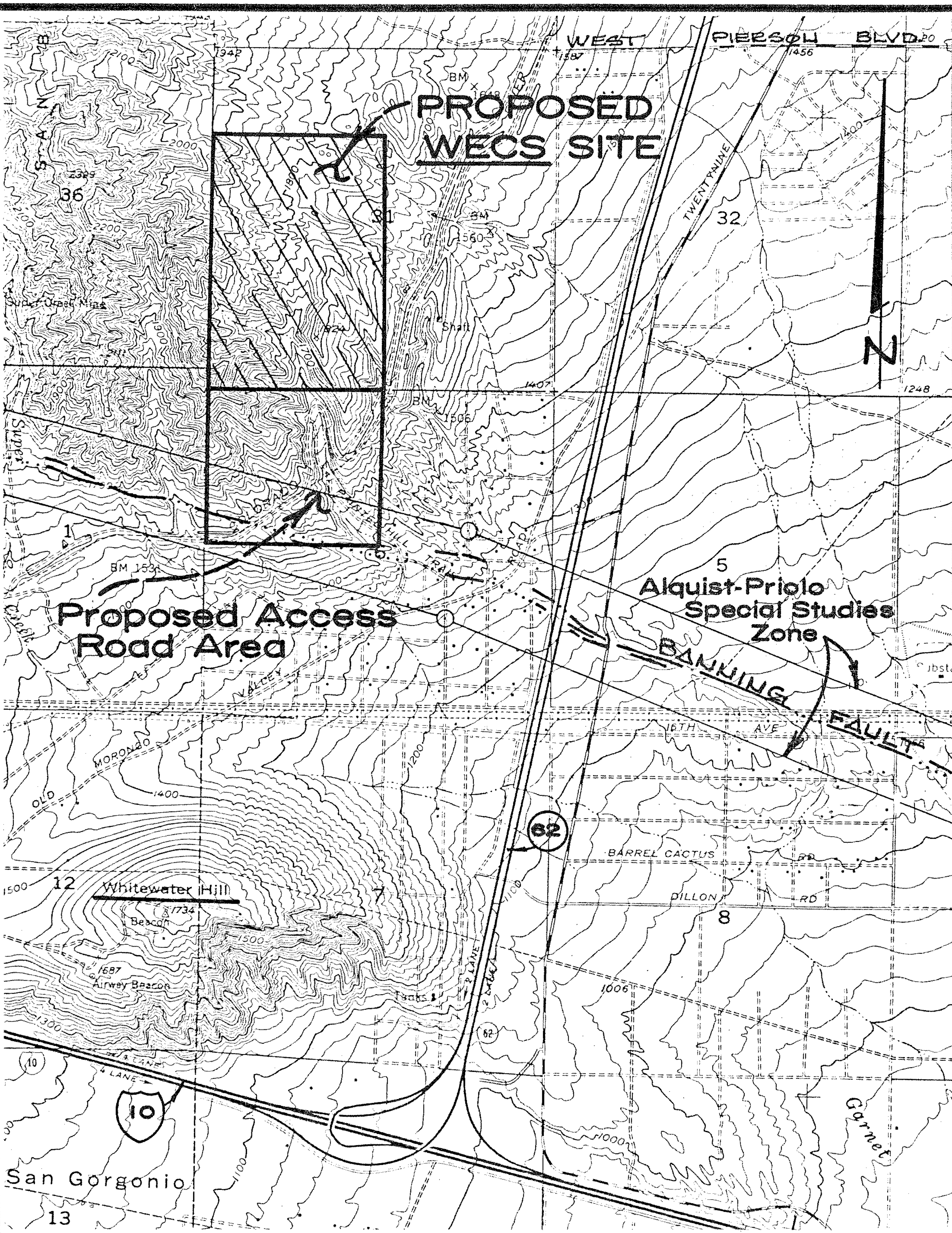
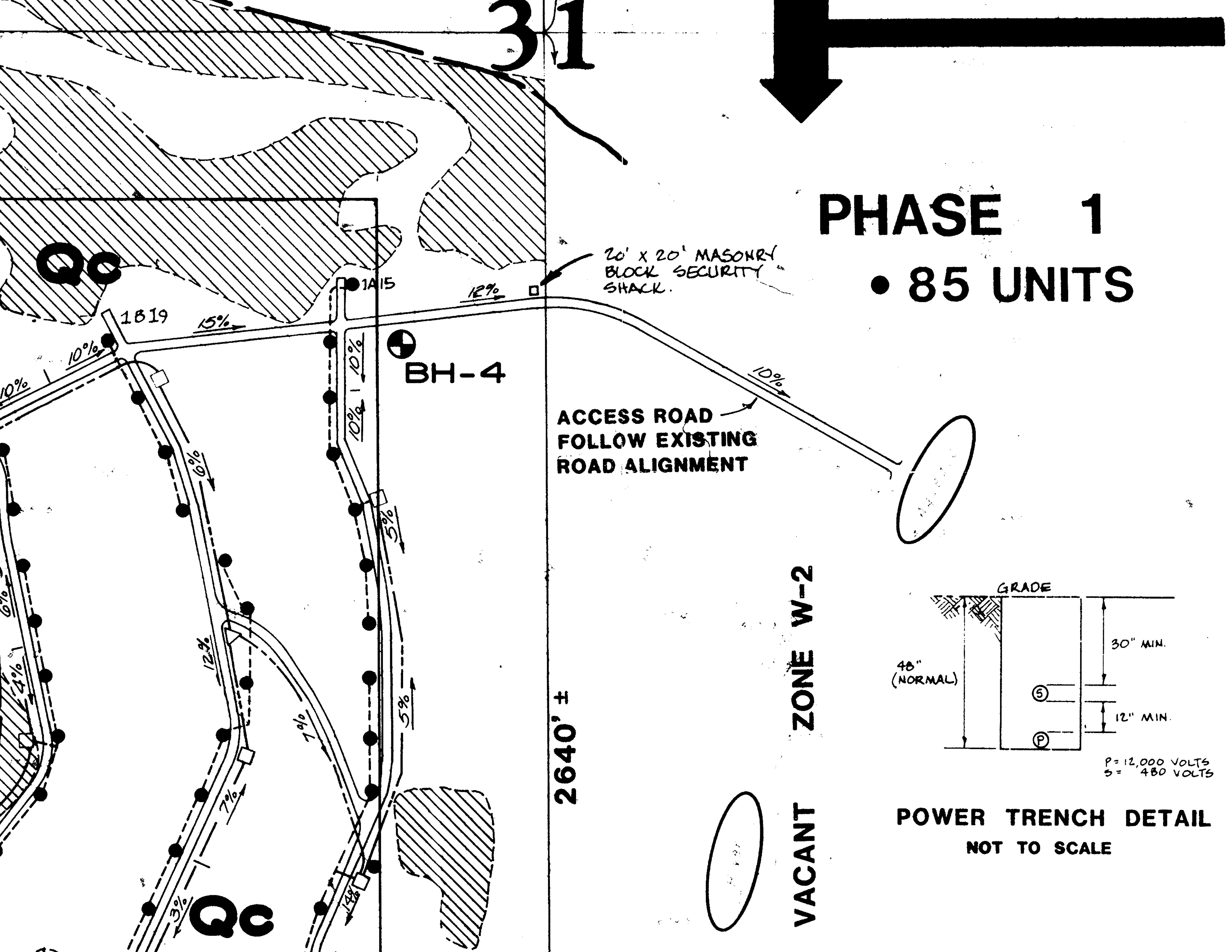


pioneer consultants GEOTECHNICAL ENGINEERS Redlands, California	DATE JAN. '85	JOB NO. 3923-001
	SCALE 1" = 200'	EXHIBIT 1.



31
PHASE 1
• 85 UNITS
20' X 20' MASONRY BLOCK SECURITY SHACK.
Qc
1A15
1819
12%
15%
10%
BH-4
ACCESS ROAD
FOLLOW EXISTING
ROAD ALIGNMENT
6%
5%
12%
7%
14%
3%
Qc
2640' ±
ZONE W-2
VACANT
GRADE
30" MIN.
48" (NORMAL)
12" MIN.
P = 12,000 VOLTS
S = 480 VOLTS
POWER TRENCH DETAIL
NOT TO SCALE

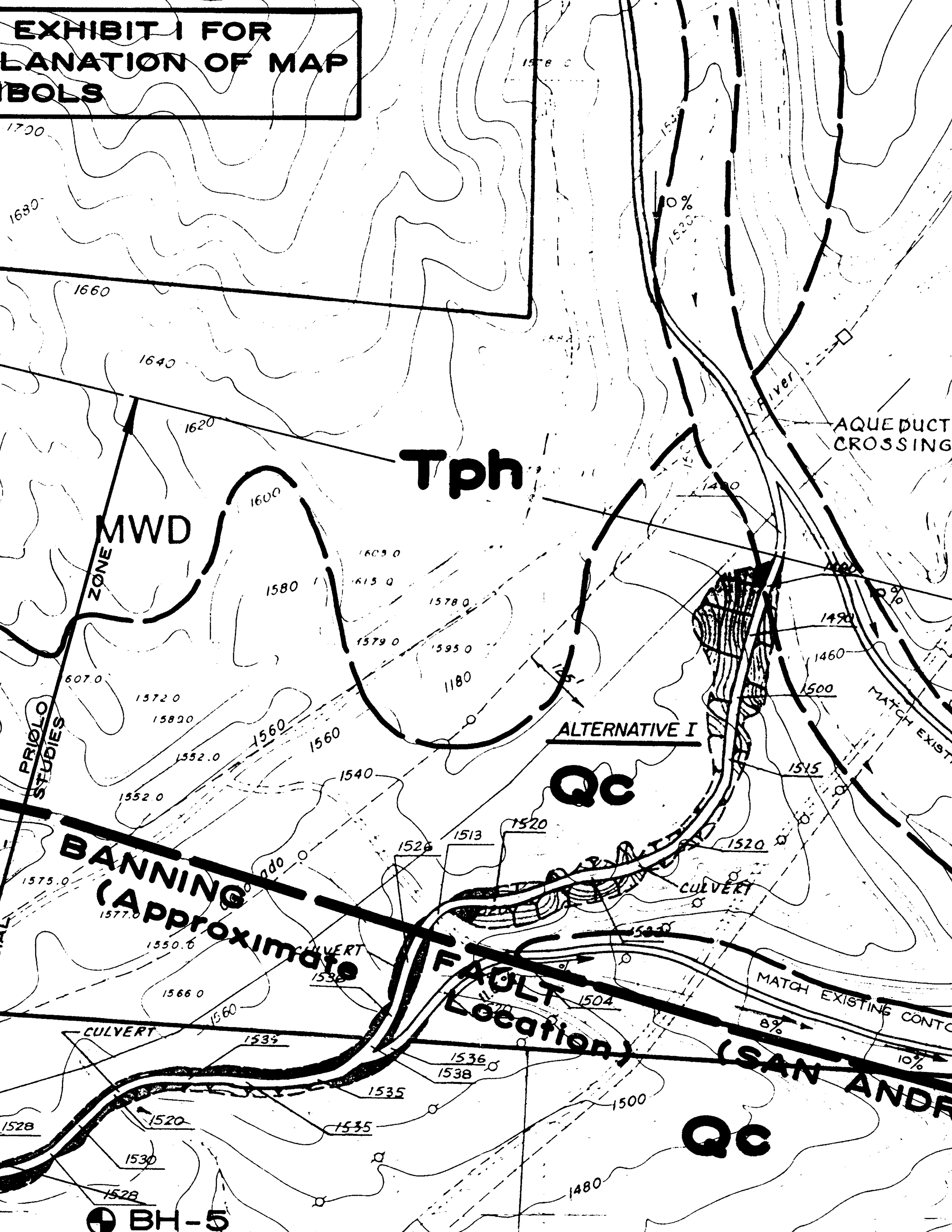


EXHIBIT I FOR
LANATION OF MAP
IBOLS
1700
1680
1660
1640
1620
1600
1580
1560
1540
Tph
MWD
ZONE
PRIOLO
STUDIES
10%
River
AQUEDUCT
CROSSING
1480
1490
1460
1500
1515
MATCH EXIST
ALTERNATIVE I
Qc
1520
1513
1526
CULVERT
BANNING
(Approximate
FAULT
Location)
(SAN ANDR
MATCH EXISTING CONTO
8%
10%
1504
1535
1536
1538
1528
1530
CULVERT
Qc
1480
BH-5

BORING SUMMARY NO. BH-1

ELEVATION: N/I
DATE DRILLED: January 7, 1985

DEPTH IN FEET	SAMPLES	BLOW COUNT PER FOOT	FIELD MOISTURE % DRY WEIGHT	DRY DENSITY LB./CU. FT.	RELATIVE COMPACTION %	UNIFIED SOIL CLASSIFICATION	MATERIAL DESCRIPTION			
0–1							Silty sand, with trace of ½-inch gravel	loose	slightly moist	red/brown
1–2	1*	50/4"					Sand, fine to very coarse to 1-inch gravel (no recovery)	very dense		
4	SPT	50/3"	3.1	117.5		SW-SC	Gravels		dry	lighter
7						SP-SM	Sand, fine to coarse, trace of ½ to 1-inch gravel and 3-inch cobbles (no recovery)			light brown/gray
8	SPT	50/4½"								
13	SPT	50/4½"	1.9	92.0		SP-SM	Sand, fine to very coarse, gravels to 3 inches			
17	SPT	50/½"	0.8	116.5			Gravel			
17.5							TOTAL BORING DEPTH 17.5 FEET (REFUSAL) NO GROUNDWATER ENCOUNTERED			

* 2.5" I.D. Ring Sampler
SPT - Standard Penetration Test

WECS 20
Desert Hot Springs Area, California
for Energy Unlimited, Inc.

EXHIBIT NUMBER 3

Pioneer Consultants

JOB NUMBER: 3923-001

BORING SUMMARY NO. BH-2

ELEVATION: N/I

DATE DRILLED: January 7, 1985

DEPTH IN FEET	SAMPLES	BLOW COUNT PER FOOT	FIELD MOISTURE % DRY WEIGHT	DRY DENSITY LB./CU. FT.	RELATIVE COMPACTION %	UNIFIED SOIL CLASSIFICATION	MATERIAL DESCRIPTION			
	1*	71	3.7	118.8			Sand, clean, medium to coarse	loose	moist	brown/gray
			3.5	112.7						
	2*	50/2"	4.1	130.9		SP	Sand, fine to coarse, to 1 to 4-inch gravels	very dense		
	SPT	50/ 4½"	1.1	137.9			Sand to a trace of ½" gravel		slightly moist	
	SPT	50/5"	4.7	103.5		SP-SM	Sand, fine to medium, trace of coarse to 1-inch gravel			

TOTAL BORING DEPTH 15.5 FEET (REFUSAL)
NO GROUNDWATER ENCOUNTERED

* 2.5" I.D. Ring Sampler
SPT - Standard Penetration Test

WECS 20
Desert Hot Springs Area, California
for Energy Unlimited, Inc.

EXHIBIT NUMBER 4

Pioneer Consultants

JOB NUMBER: 3923-001

BORING SUMMARY NO. BH-3

ELEVATION: N/I

DATE DRILLED: January 7, 1985

DEPTH IN FEET	SAMPLES	BLOW COUNT PER FOOT	FIELD MOISTURE % DRY WEIGHT	DRY DENSITY LB./CU. FT.	RELATIVE COMPACTION %	UNIFIED SOIL CLASSIFICATION	MATERIAL DESCRIPTION			
1							Silty sand, fine to coarse, with trace of roots	loose	slightly moist	red/ brown
2	1*	27/5"	13.5	102.0			Silty sand, fine, to ½-inch gravel	dense		
3							Sand, fine to coarse, to ½-inch gravel	very dense	dry	light brown
4	SPT	35/4"	5.3	77.9						
5										
6										
7										
8	SPT	50/ 5½"	8.5	116.2		SW-SM	Sand, fine to coarse, with ½ to 1-inch gravel			
9										
10										
11							2 to 3-inch gravel			
12										
13	SPT	50/ 5½"	3.1	125.6			Gravelly			
14										
15										
16										
17										
18	SPT	50/6"	5.1	128.4						
19										
20										
21										
22							Sand to ½-inch gravels			
23	SPT	50/ 5½"	5.2	126.3						
24										
25							(continued)			

* 2.5" I.D. Ring Sampler
SPT - Standard Penetration Test

WECS 20
Desert Hot Springs Area, California
for Energy Unlimited, Inc.

EXHIBIT NUMBER 5

Pioneer Consultants

JOB NUMBER: 3923-001

Approved For Report On ______ By ______

BORING SUMMARY NO. BH-3

ELEVATION: N/I

DATE DRILLED: January 7, 1985

DEPTH IN FEET	SAMPLES	BLOW COUNT PER FOOT	FIELD MOISTURE % DRY WEIGHT	DRY DENSITY LB./CU. FT.	RELATIVE COMPACTION %	UNIFIED SOIL CLASSIFICATION	MATERIAL DESCRIPTION			
26							Sand	very dense	dry	light brown
27										
28	SPT	50/5"	4.5	132.7			Less coarse sand			
29										
30										
31										
32										
33	SPT	50/4"	6.5	129.3						
34										
35										
36							TOTAL BORING DEPTH 35.0 FEET NO GROUNDWATER ENCOUNTERED			
37										
38										
39										
40										
41										
42										
43										
44										
45										
46										
47										
48										
49										
50										

SPT - Standard Penetration Test

Approved For Report On ____ By ____

WECS 20
Desert Hot Springs Area, California
for Energy Unlimited, Inc.

EXHIBIT NUMBER 5A

Pioneer Consultants

JOB NUMBER: 3923-001

BORING SUMMARY NO. BH-4

ELEVATION: N/I

DATE DRILLED: January 8 and 9, 1985

DEPTH IN FEET	SAMPLES	BLOW COUNT PER FOOT	FIELD MOISTURE % DRY WEIGHT	DRY DENSITY LB./CU. FT.	RELATIVE COMPACTION %	UNIFIED SOIL CLASSIFICATION	MATERIAL DESCRIPTION			
1							Silty sand	loose	slightly moist	red/ brown
2	1*	50/5"	3.5	106.7			Silty sand to ½-inch gravel	very dense	dry	light reddish brown
3										
4	2*	31/2"	4.3	83.5		SP-SC				
5							Increasing gravel ½ to 1 inch			
6										
7										
8	SPT	50/ 4½"	3.8	138.8						light brown
9										
10										
11										
12										
13	SPT	50/5"	2.3	130.5			Gravelly sand			
14										
15										
16							Sand, fine to very coarse, trace of ½-inch gravel			
17										
18	SPT	50/4"	2.3	136.1						
19										
20							Increasing gravel			
21										
22										
23	SPT	50/ 1½"	2.2	135.8						
24							TOTAL BORING DEPTH 23.0 FEET (REFUSAL ON COBBLES) NO GROUNDWATER ENCOUNTERED			
25										

* 2.5" I.D. Ring Sampler
SPT - Standard Penetration Test

WECS 20
Desert Hot Springs Area, California
for Energy Unlimited, Inc.

EXHIBIT NUMBER 6

Pioneer Consultants

JOB NUMBER: 3923-001

Approved For Report On ____ By ____

BORING SUMMARY NO. BH-5

ELEVATION: N/I

DATE DRILLED: January 9, 1985

DEPTH IN FEET	SAMPLES	BLOW COUNT PER FOOT	FIELD MOISTURE % DRY WEIGHT	DRY DENSITY LB./CU. FT.	RELATIVE COMPACTION %	UNIFIED SOIL CLASSIFICATION	MATERIAL DESCRIPTION			
1							Sand, fine to medium, trace of very coarse	loose	slightly moist	brown
2	1*	100	5.4	93.9			Sand to ½-inch gravel	dense		
3							Trace of 3-inch gravel			
5	2*	62	2.4	118.7						
7						SP-SM	Sand, fine to coarse, to 3-inch gravel	very dense	dry	light brown
8	SPT	92	1.4	149.2						
9							Increasing gravel			
13	SPT	50/5"	1.0	147.9						
18	SPT	84/10½"	1.5	148.5						
21							Sand to 3-inch gravel			
23	SPT	50/4"	1.4	151.3						
25							(continued)			

* 2.5" I.D. Ring Sampler
SPT - Standard Penetration Test

WECS 20
Desert Hot Springs Area, California
for Energy Unlimited, Inc.

EXHIBIT NUMBER 7

Pioneer Consultants

JOB NUMBER: 3923-001

Approved For Report On ______ By ______

BORING SUMMARY NO. BH-5

ELEVATION: N/I

DATE DRILLED: January 9, 1985

DEPTH IN FEET	SAMPLES	BLOW COUNT PER FOOT	FIELD MOISTURE % DRY WEIGHT	DRY DENSITY LB./CU. FT.	RELATIVE COMPACTION %	UNIFIED SOIL CLASSIFICATION	MATERIAL DESCRIPTION			
26							Sand with ½-inch gravel	very dense	dry	light brown
27										
28	SPT	50/5"	1.6	150.8						
29										
30										
31							TOTAL BORING DEPTH 30.0 FEET NO GROUNDWATER ENCOUNTERED			
32										
33										
34										
35										
36										
37										
38										
39										
40										
41										
42										
43										
44										
45										
46										
47										
48										
49										
50										

SPT - Standard Penetration Test

WECS 20
Desert Hot Springs Area, California
for Energy Unlimited, Inc.

EXHIBIT NUMBER 7A

Pioneer Consultants

JOB NUMBER: 3923-001

Approved For Report On ______ By ______

The maximum density was determined in accordance with ASTM Standard D1557. The results by laboratory checkpoint are:

Boring Number	Depth (feet)	Soil Description	Maximum Dry Density (P.C.F.)	Optimum Moisture (Percent)
BH-1	1.0- 6.0	Clayey silt, very fine, trace of fine to coarse sand and fine to medium gravel, brown	131.0	9.0
BH-2	6.0-12.0	Sand, fine to coarse, trace of very fine silty sand, brown	134.5	7.5
BH-3	7.0-35.0	Silt, very fine, trace 50% fine to coarse sand, brown	130.7	9.0
BH-4	2.0- 8.0	Silty, very fine, trace 50% fine to coarse sand, brown	133.5	8.3
BH-5	5.0-30.0	Sand, medium, trace of fine to coarse sand, fine to medium gravel, gray	130.5	7.0

MAXIMUM DENSITY - OPTIMUM MOISTURE DETERMINATION

PREPARED BY: ljs	WECS 20 Desert Hot Springs Area, California for Energy Unlimited, Inc.		EXHIBIT NUMBER 8
CHECKED BY: RBR			
APPROVED BY: DWT	DATE: 1/85	SCALE:	JOB NUMBER: 3923-001

Pioneer Consultants — *Consulting Engineers and Geologists*

Boring Number	Depth (feet)	Percent Passing Individual Sieve											
		2"	1½"	1"	3/4"	1/2"	3/8"	#4	#10	#20	#40	#100	#200
BH-1	1.0- 6.0					100	96	88	69	47	33	18	12
BH-1	6.0-12.0	100	96	94	92	90	87	75	56	38	25	12	7
BH-1	12.0-17.5						100	89	60	33	20	10	6
BH-2	6.0-12.0	100	97	92	87	79	73	54	42	26	16	7	4
BH-2	12.0-15.0	100	99	97	93	87	83	74	58	40	29	11	7
BH-3	7.0-35.0						100	88	72	49	32	16	10
BH-4	2.0- 8.0				100	92	88	73	52	33	22	11	8
BH-5	5.0-30.0			100	93	84	77	63	47	29	18	9	5

Boring Number	Depth (feet)	Liquid Limit	Plasticity Index	Classification	
				A.A.S.H.T.O.	Unified
BH-1	1.0- 6.0	27	8	A-2-4	SW-SC
BH-1	6.0-12.0	-	Nonplastic	A-1-b	SP-SM
BH-1	12.0-17.5	-	Nonplastic	A-1-b	SP-SM
BH-2	6.0-12.0	-	Nonplastic	A-1-a	SP
BH-2	12.0-15.0	-	Nonplastic	A-1-b	SP-SM
BH-3	7.0-35.0	-	Nonplastic	A-1-b	SW-SM
BH-4	2.0- 8.0	-	-	-	SP-SC
BH-5	5.0-30.0	-	-	-	SP-SM

SUMMARY OF LABORATORY TESTING

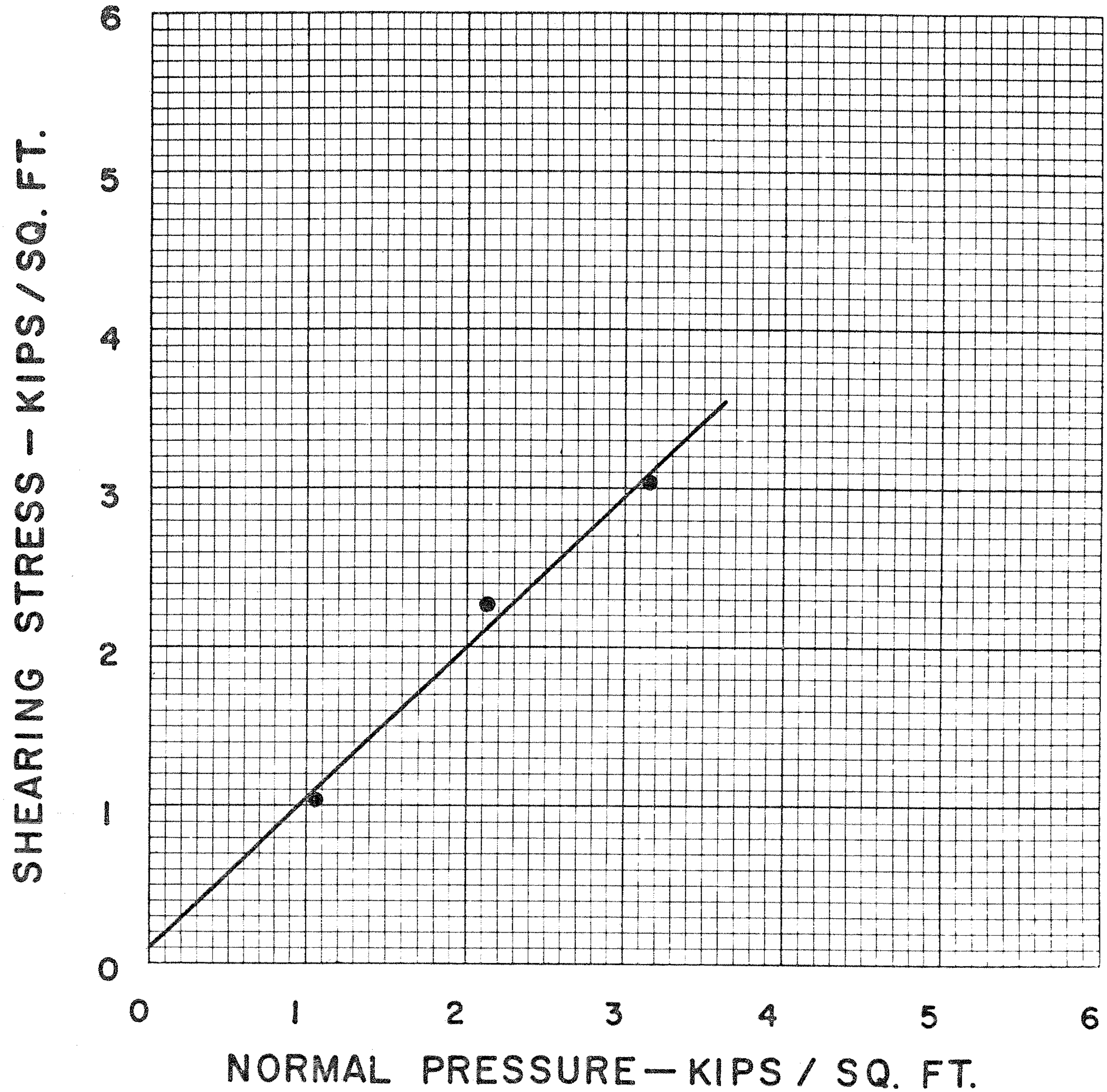
PREPARED BY: ljs
CHECKED BY: RBR
APPROVED BY: DWT

WECS 20
Desert Hot Springs Area, California
for Energy Unlimited, Inc.

EXHIBIT NUMBER 9

DATE: 1/85 SCALE: JOB NUMBER: 3923-001

Pioneer Consultants — Consulting Engineers and Geologists



EXCAVATION NO. BH-1 DEPTH: 1.0 - 6.0 FEET

SATURATED TEST

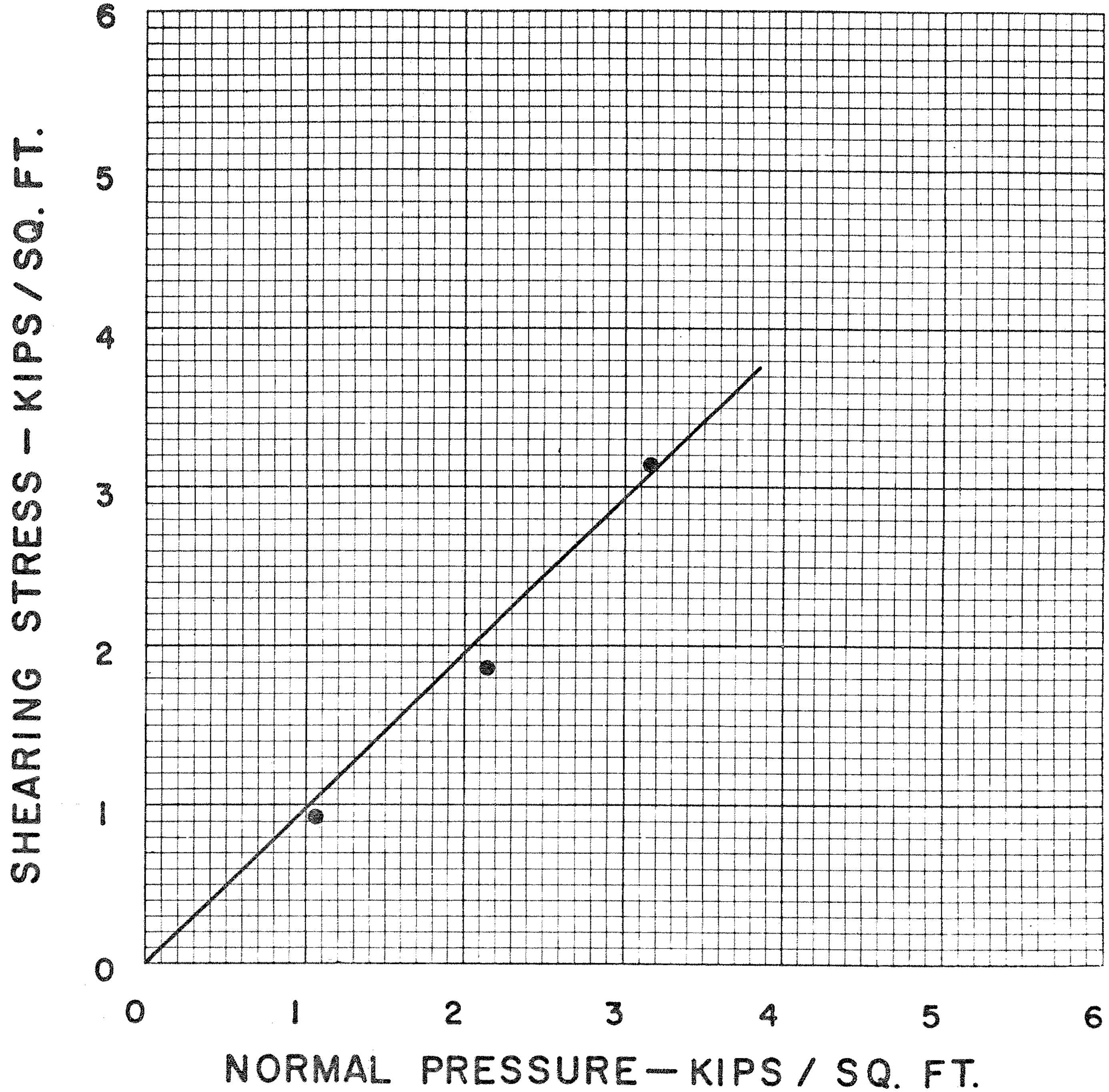
Ø = °
C = P.S.F

IN SITU MOISTURE TEST

Ø = 40 °
C = 110 P.S.F

DIRECT SHEAR TEST DATA

PROJECT: WECS 20, Desert Hot Springs Area, California	PIONEER CONSULTANTS REDLANDS, CALIFORNIA	EXHIBIT 10
JOB NO.: 3923-001 DATE: 1/85		

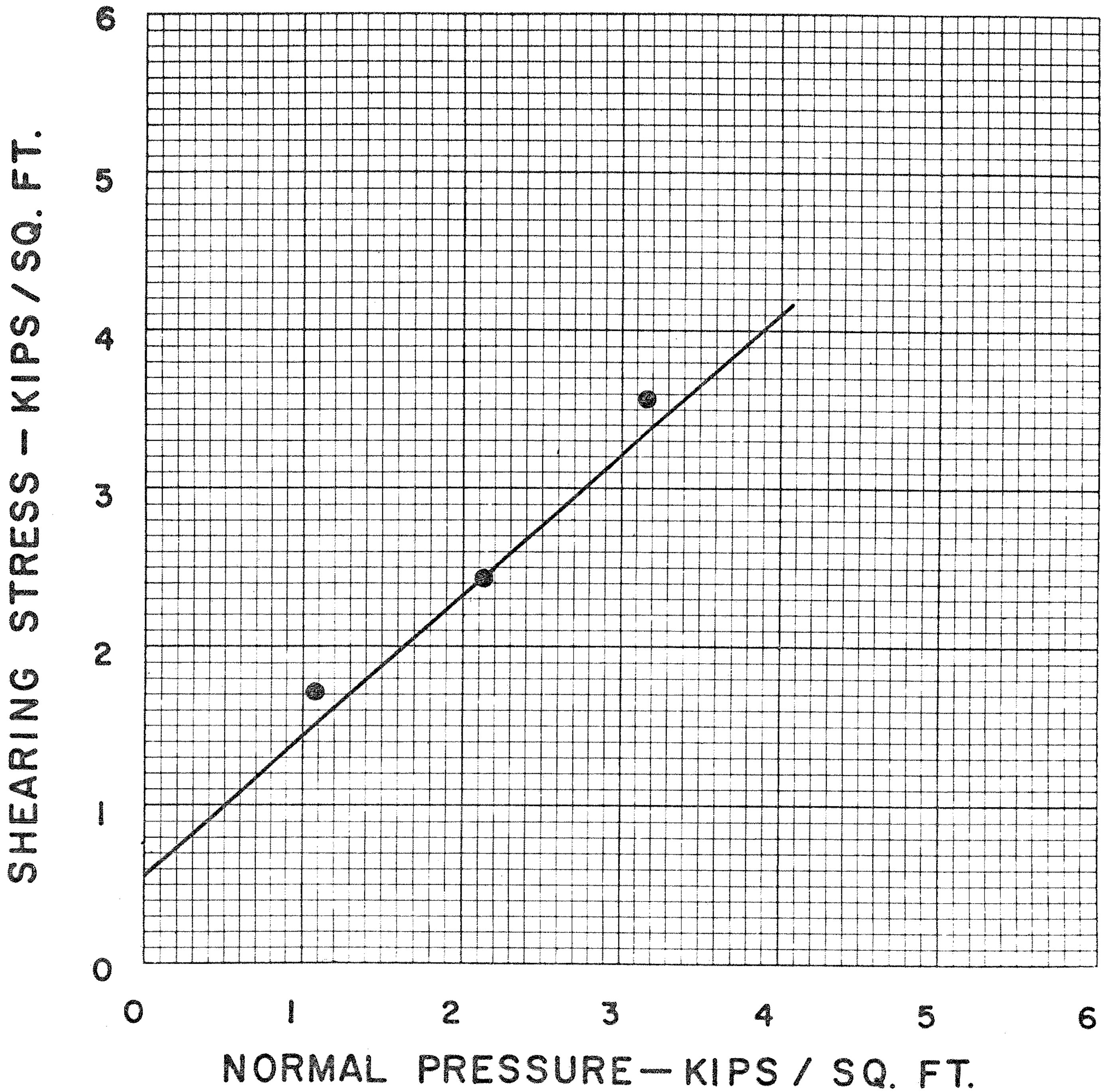


EXCAVATION NO. BH-2 DEPTH: 2.5 - 3.0 FEET

SATURATED TEST		IN SITU MOISTURE TEST	
Ø =	°	Ø =	40 °
C =	P.S.F	C =	0 P.S.F

DIRECT SHEAR TEST DATA

PROJECT:	WECS 20, Desert Hot Springs Area, California	PIONEER CONSULTANTS REDLANDS, CALIFORNIA	EXHIBIT 11
JOB NO.:	3923-001 DATE: 1/85		



EXCAVATION NO. BH-2

DEPTH: 6.0 - 12.0 FEET

$\gamma = 120.5$ $m_c = 7.3\%$

SATURATED TEST

ϕ = °

C = P.S.F

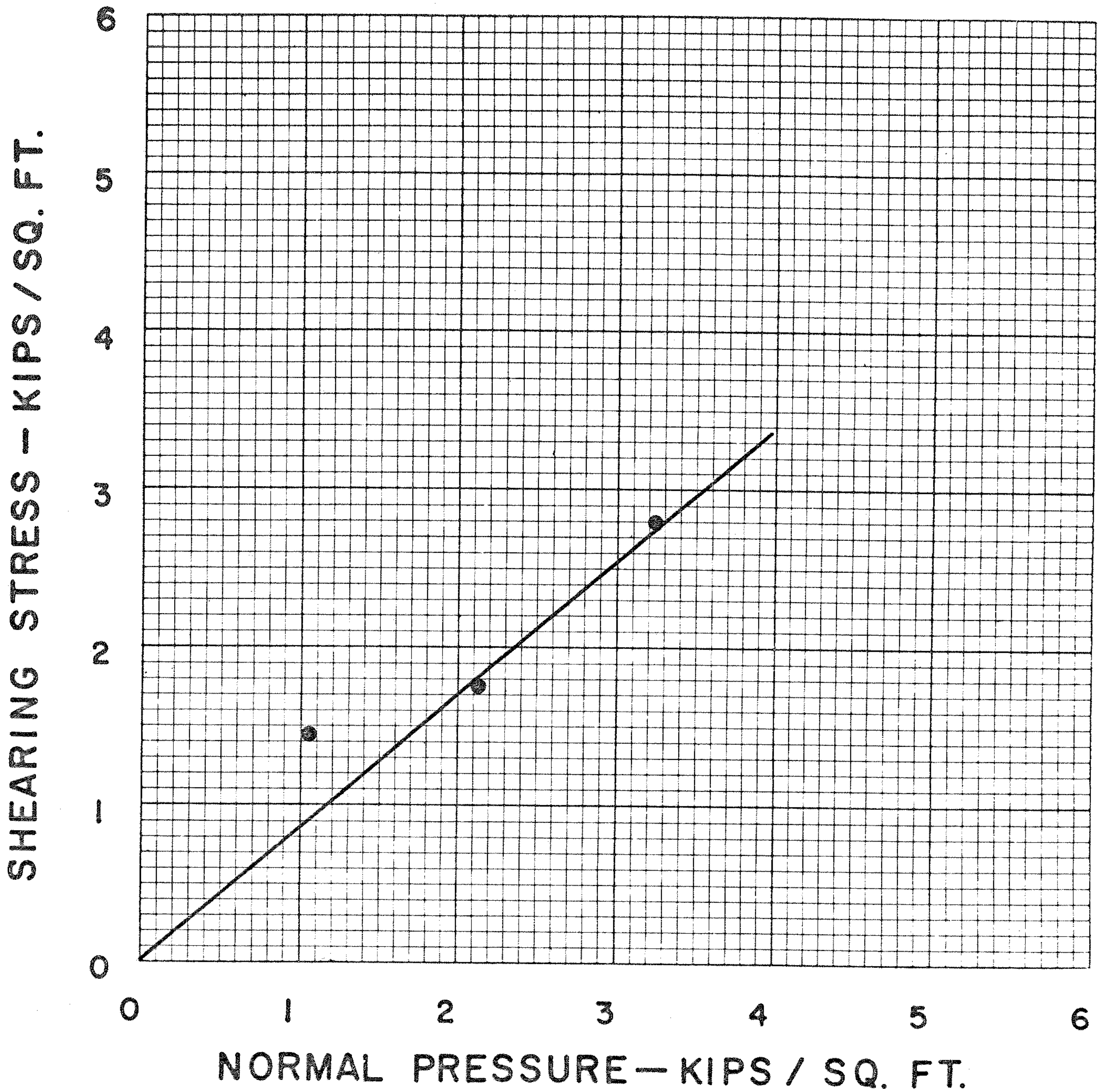
REMOLDED TEST

ϕ = 41 °

C = 560 P.S.F

DIRECT SHEAR TEST DATA

PROJECT:	WECS 20, Desert Hot Springs Area, California	PIONEER CONSULTANTS REDLANDS, CALIFORNIA	EXHIBIT 12
JOB NO.:	3923-001 DATE: 1/85		



EXCAVATION NO. BH-5 DEPTH: 4.5 - 5.0 FEET

SATURATED TEST

Ø = °

C = P.S.F

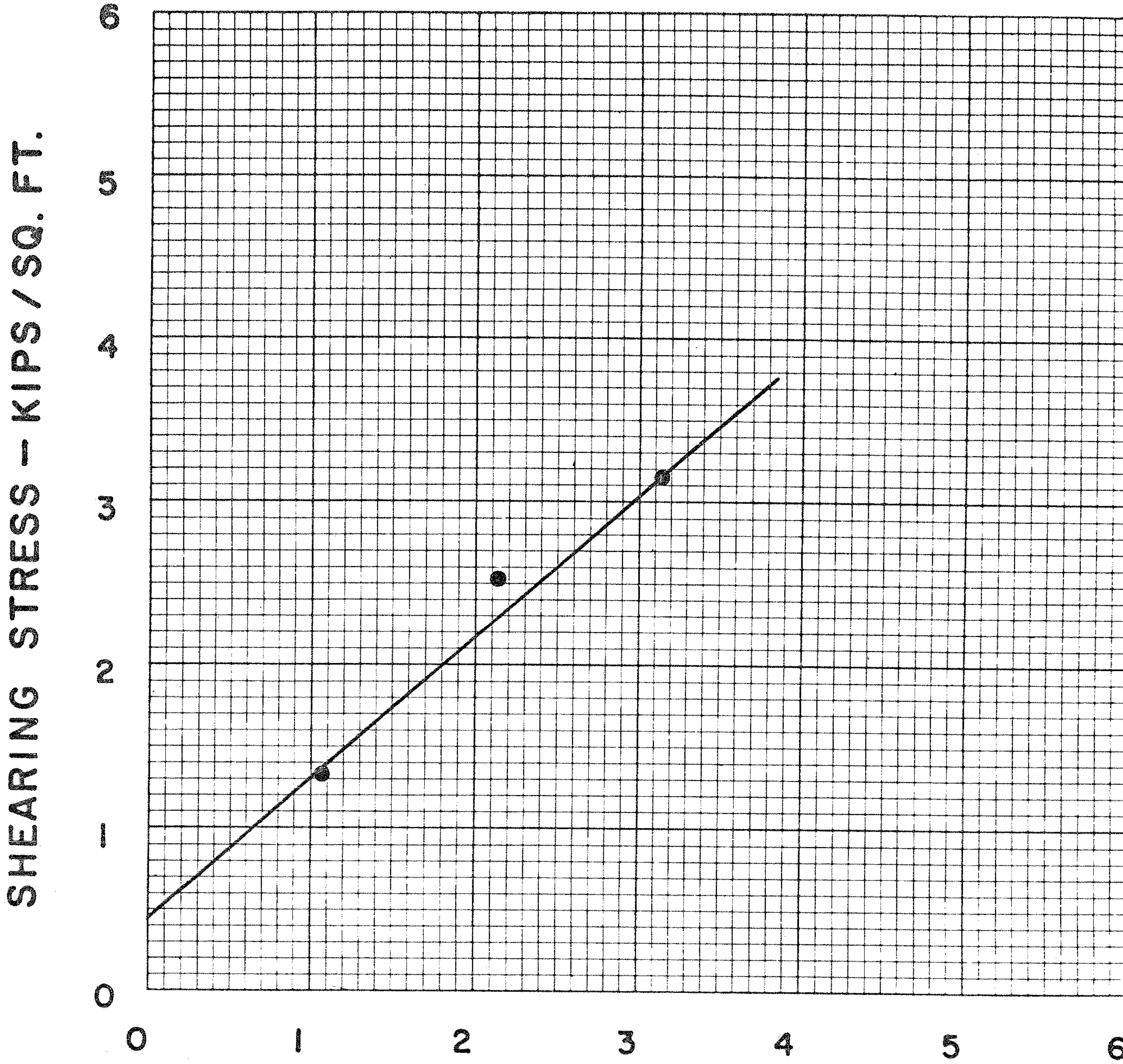
IN SITU MOISTURE TEST

Ø = 40 °

C = 0 P.S.F

DIRECT SHEAR TEST DATA

PROJECT: WECS 20, Desert Hot Springs Area, California		PIONEER CONSULTANTS REDLANDS, CALIFORNIA	EXHIBIT 13
JOB NO.: 3923-001	DATE: 1/85		



NORMAL PRESSURE—KIPS / SQ. FT.

EXCAVATION NO. BH-5 DEPTH: 5.0 - 30.0 FEET

SATURATED TEST

Ø = °

C = P.S.F

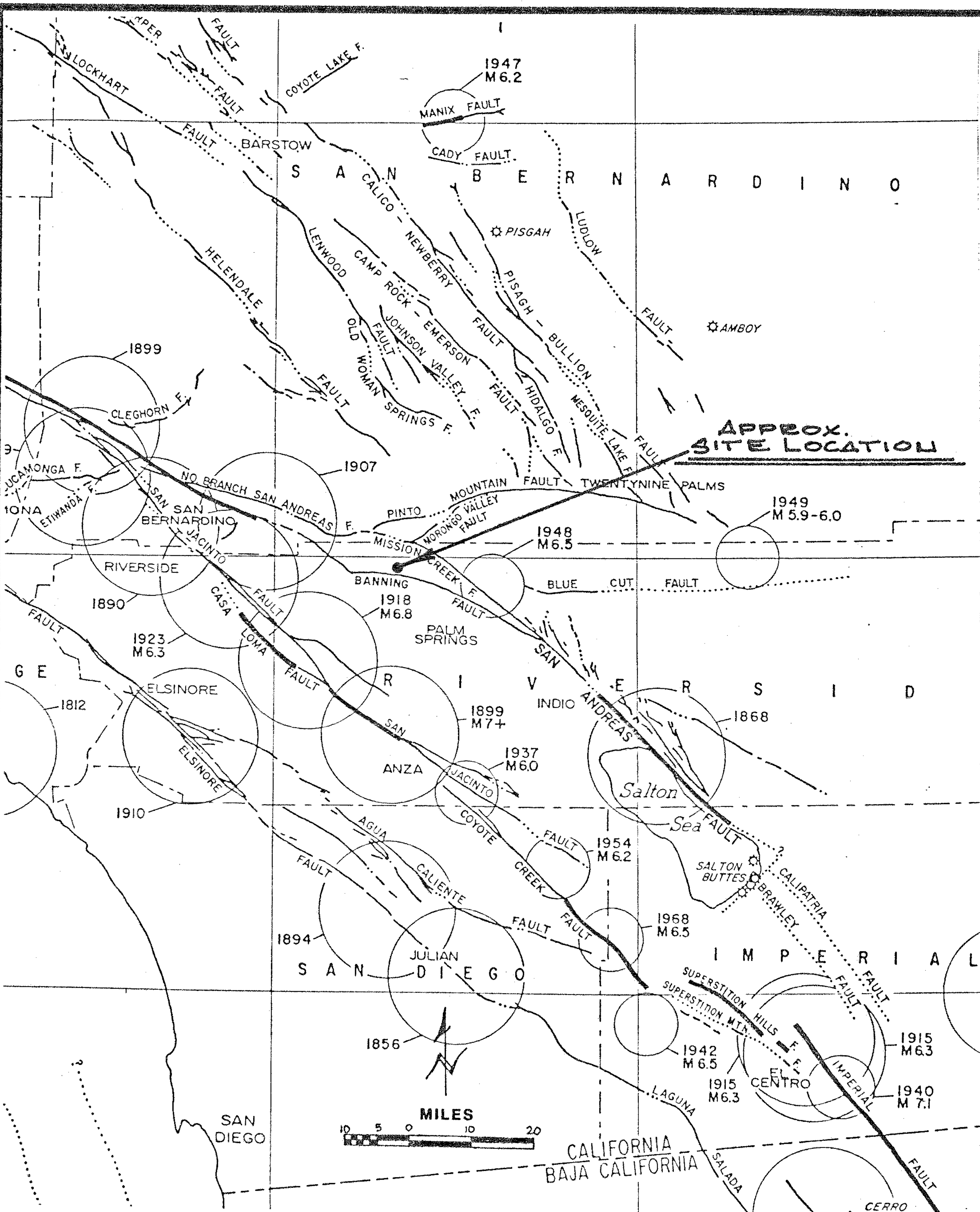
IN SITU MOISTURE TEST

Ø = 40 °

C = 460 P.S.F

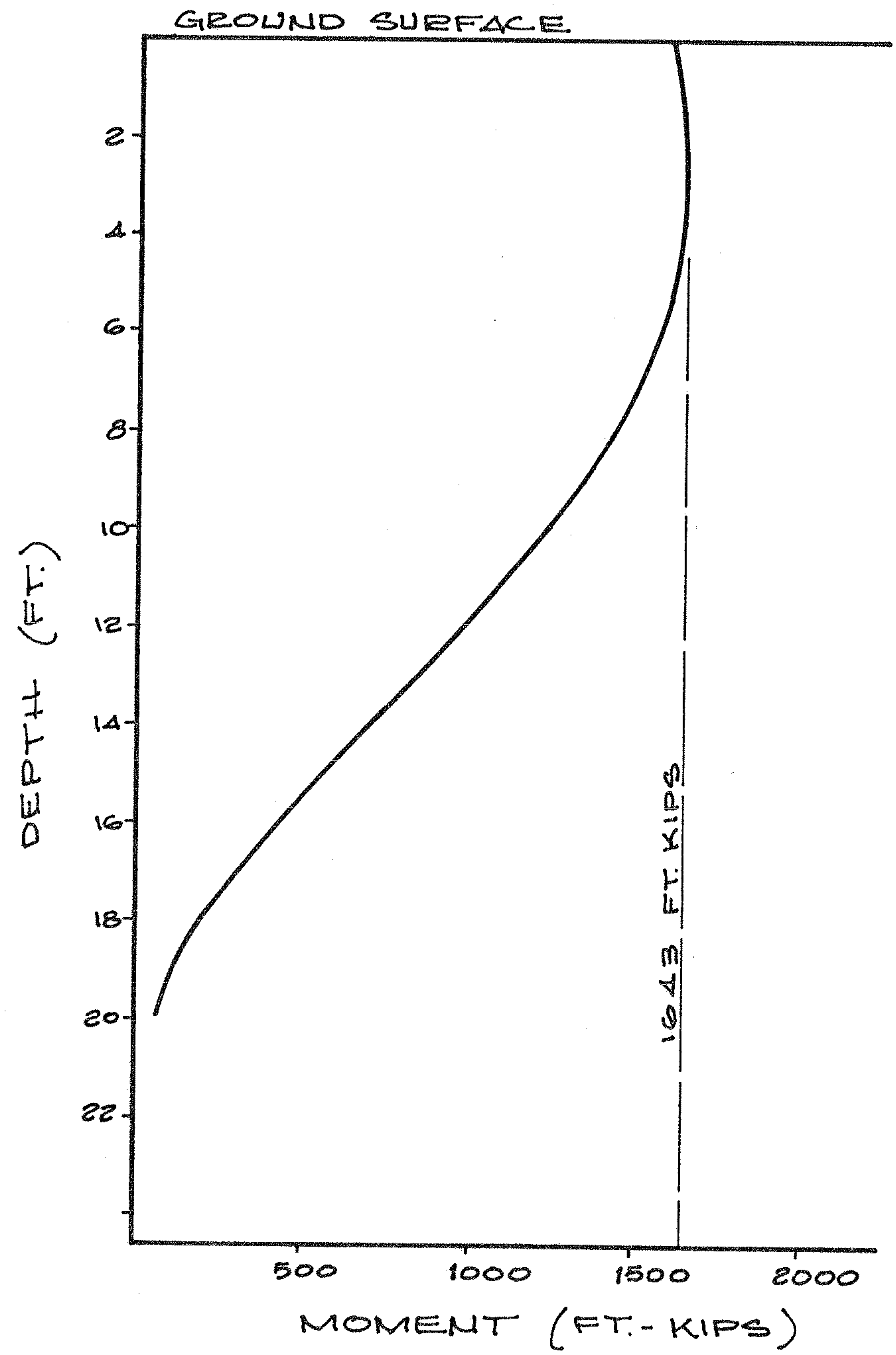
DIRECT SHEAR TEST DATA

PROJECT: WECS 20, Desert Hot Springs Area, California	PIONEER CONSULTANTS REDLANDS, CALIFORNIA	EXHIBIT 14
JOB NO.: 3923-001 DATE: 1/85		



MAJOR EARTHQUAKES and RECENTLY ACTIVE FAULTS
SOUTHERN CALIFORNIA REGION

pioneer consultants GEOTECHNICAL ENGINEERS Redlands, California	DATE FEB. '85	JOB NO. 3923-001
	SCALE AS SHOWN	EXHIBIT 15



pioneer consultants GEOTECHNICAL ENGINEERS Redlands, California	DATE FEB. '85	JOB NO. 3923-001
	SCALE AS SHOWN	EXHIBIT 16