APPENDIX B.2

Geotechnical Report from Spec 639S
(For Reference Only)

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DESIGN DEPARTMENT
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GEOTECHNICAL INVESTIGATION REPORT
Reach 4, Reclaimed Water and Brine Water Transmission Pipelines, Contract II (Station 356+80.24 to Station 480+00 and Station 562+00 to Station 698+77.58) Riverside County, California

Prepared for:

Mr. Arlen Nielson
Eastern Municipal Water District
P. O. Box 8300
San Jacinto, CA 92581-8300

CCIE Project No. 92-81-504-01

October 6, 1994

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San Jacinto, CA 92581-8300

Subject: **GEOTECHNICAL INVESTIGATION REPORT**
Reach 4, Reclaimed Water and Brine Water
Transmission Pipelines, Contract II
(Station 356+80.24 to Station 480+00
and Station 562+00 to Station 698+77.58)
Riverside County, California
CCIE Project No. 92-81-504-01

Dear Mr. Nielson:

Enclosed is the report of our geotechnical investigation performed along the alignment of the proposed Reach 4 Reclaimed Water and Brine Water Transmission Pipelines, Contract II (Station 356+80.24 to Station 480+00 and Station 562+00 to Station 4696+77.58), Riverside County, California. This investigation was performed in accordance with our agreement with Black and Veatch, dated July 9, 1993.

A final geotechnical report, dated January 14, 1994, was submitted to Black and Veatch. The enclosed report is a modified version of that final geotechnical report. The modifications are introduced as per your request to reflect construction-related concerns only as opposed to both design and construction concerns. For design purposes, the final report to Black and Veatch is still applicable.

Results of our investigation indicate that, from a geotechnical standpoint, the proposed alignment is suitable for the construction of the pipelines, provided the recommendations contained in this report are incorporated into the design and construction of the project.

Earth materials encountered in the exploratory borings drilled along the proposed alignment varied widely and consisted of fill and native alluvial soils underlain by severely weathered to relatively unweathered granitic and metamorphic bedrock. Fill and alluvial soils are composed of mainly silty sand and clayey sands with traces of clay.

At the time of our field exploration, groundwater was not encountered within the borings drilled to a maximum depth of 20 feet along the proposed alignment from Station 356+80.24 to Station 480+00. The depth of the water table along the proposed alignment from Station 562+00 to Station 696+77.58 varied from 12 to 25 feet below existing ground surface. Seasonal variation in the groundwater table should be expected.

Liquefaction potential of the site is considered low, as the depth from existing ground surface to the present water table is greater than the depth to weathered bedrock. Also, the overlying medium dense native silty and/or clayey sands contain appreciable amounts of fines and are not like to liquefy.

Based on the study of soil corrosivity performed by M. J. Schiff and Associates, the portion of the alignment from Station 562+00 to Station 696+77.58 is classified as severely corrosive to ferrous metals and deleterious to concrete. The remainder of the alignment is classified as moderately corrosive to ferrous metals.

Temporary excavations of up to five feet deep can be constructed vertically. The recommended side slopes of temporary excavations of five to 12 feet and 12 to 25 feet deep are 0.5:1 (H:V) and 1:1, respectively. If steeper temporary excavations are constructed, they should be shored to provide necessary support.

If you have any questions, please feel free to contact the undersigned, or Richard Escandon, Senior Geologist. This opportunity to be of service to Eastern Municipal Water District is greatly appreciated.

Very truly yours,

CONVERSE CONSULTANTS INLAND EMPIRE

Quazi S. E. Hashmi, Ph.D., P. E.
Principal Engineer/Branch Manager

Dist: 3/Addressee

MSI/QSH/dmd

PROFESSIONAL CERTIFICATION

This report has been prepared by the staff of Converse Consultants Inland Empire (CCIE) under the professional supervision of the individuals whose seals and signatures appear hereon.

The findings, recommendations, specifications or professional opinions contained in this report were prepared in accordance with generally accepted professional engineering and engineering geologic principles and practice in this area of California. There is no warranty, either express or implied.

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1.0 INTRODUCTION

This report contains the findings of CCIE's geotechnical investigation performed for the proposed Reach 4, Reclaimed Water and Brine Water Transmission Pipelines, Contract II (Stations 356+80.24 to Station 480+00 and Station 562+00 to Station 696+77.56) located in Riverside County, California. The purpose of the investigation was to evaluate the nature and engineering properties of the subsurface soils and to provide geotechnical recommendations for design and construction of the proposed pipelines.

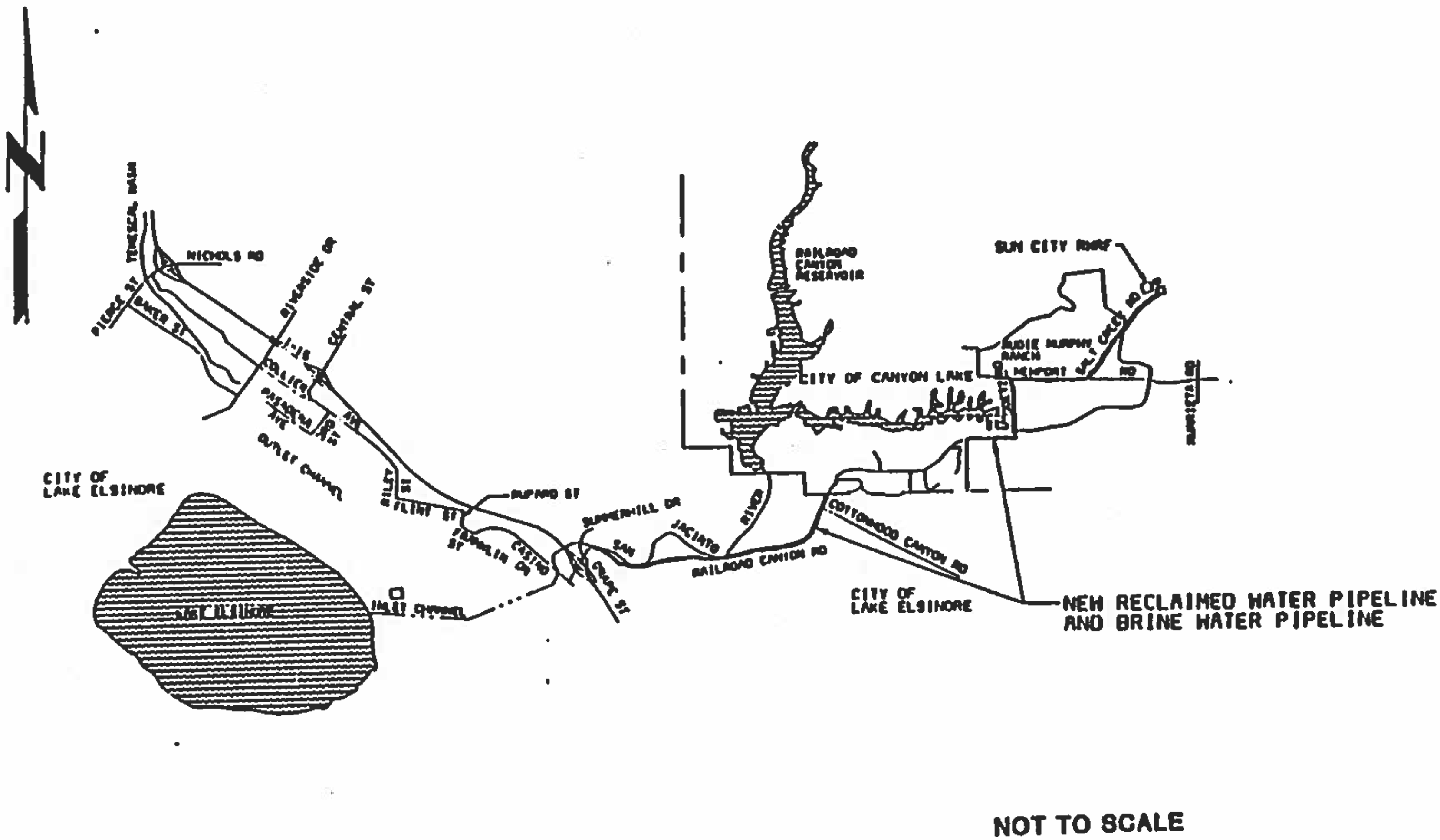
The location of the proposed pipeline alignment is shown in Figure No. 1, *Location Map*, and Drawing No. 1, *Geologic/Boring Location Map*. The proposed alignment comprises two separate segments referred to in this report as Segment I and Segment II.

Segment I (Station 356+80.24 to Station 480+00): This portion of the Reach 4 Pipelines, Contract II, alignment runs easterly along Railroad Canyon Road and is located within the city of Lake Elsinore. Segment I originates near the intersection of Railroad Canyon Road and Interstate I-15 (Station 356+80.24) and terminates at the city limit between the city of Lake Elsinore and the city of Canyon Lake (Station 480+00).

Segment II (Station 562+00 to Station 696+77.58): This segment of the Contract II alignment originates near the eastern end of Railroad Canyon Road (Station 562+00) and traverses northerly along Goetz Road to the intersection with Newport Road. The proposed alignment then continues easterly along Newport Road and northeasterly along Salt Creek Road to the Sun City Regional Water Reclamation Facility (Station 696+77.58).

The proposed pipeline alignment, which is about 25,798 feet long, forms part of the Reach 4, Reclaimed Water and Brine Water Transmission Pipelines (RWBWTP). The Reach 4 Pipelines will carry reclaimed water and brine water from the Sun City Regional Water Reclamation Facility (SCRWRF) to the proposed dechlorination facilities near the intersection of Collier Street and Third Street in the city of Lake Elsinore.

Based on information provided by Black and Veatch, it is our understanding that the proposed project will comprise a 54-inch reclaimed water pipeline and a 24-inch brine water pipeline. These pipes will be made of either cement-mortar-lined steel pipe (AWWA Standard C-205) or tape-wrapped steel pipe (AWWA Standard C-214). For most of the alignment, the depth to invert of the 54-inch reclaimed water pipeline will be approximately eight to 10 feet below the existing ground surface. Due to the presence of utilities and other structures, the pipes will be placed at depths greater than 10 feet at a number of locations as presented in Table No. 1, *Locations with Depth of Invert Greater Than 10 Feet Below Existing Grade*. The invert of the 24-inch brine water pipeline will be above the invert of the 54-inch reclaimed water pipeline.



LOCATION MAP

REACH 4, RWBWTP, CONTRACT II, EMWD
Riverside County, California
for: Black & Veatch

Project No. 92-81-604-01

Figure No. 1

Converse Consultants Inland Empire

Table No. 1. Locations with Depth of Invert Greater Than 10 Feet Below Existing Grade

Location	Approximate Length of Section (ft)	Approximate Maximum Depth of Invert Below Existing Grade (ft)
Sta. 356+80.24 to Sta. 357+90	110	13.0
Sta. 363+00 to Sta. 363+22	22	15.0
Sta. 367+04 to Sta. 367+28	24	16.0
Sta. 374+30 to Sta. 374+53	23	15.0
Sta. 379+29 to Sta. 379+50	21	13.0
Sta. 381+98 to Sta. 382+25	27	15.5
Sta. 387+58 to Sta. 387+85	27	15.5
Sta. 396+49 to Sta. 396+69	20	13.0
Sta. 407+69 to Sta. 408+09	40	19.0
Sta. 423+01 to Sta. 423+28	27	16.0
Sta. 428+88 to Sta. 429+12	24	15.0
Sta. 430+65 to Sta. 433+07	242	16.0
Sta. 441+00 to Sta. 442+80	180	12.0
Sta. 446+40 to Sta. 447+92	152	11.0
Sta. 460+60 to Sta. 462+00	140	12.0
Sta. 470+27 to Sta. 472+00	173	16.0
Sta. 618+39 to Sta. 618+62	23	13.0

2.0 SCOPE OF WORK

Our investigation consisted of the following scope of work:

2.1 Geologic Reconnaissance and Mapping

Our geologists performed a field reconnaissance along the proposed alignment to identify geologic and/or geotechnical features that can have an impact on the project. Detailed geologic mapping of the proposed alignment was performed. The mylar of the alignment was not available at the time of preparation of this report. The results of the geologic mapping will be presented in the final report.

2.2 Subsurface Exploration

Nine borings were drilled to obtain subsurface information along the proposed pipeline alignment. The location of the Boring BC-1, drilled as part of the "*Reach 4 Pipelines, Contract I*," is located within the subject alignment by about 400 feet. The information derived from this boring was also utilized in the preparation of this report. The approximate location and depth of borings, existing grade elevation and the depth of water table, if encountered, are included in Table No. 2, *Summary of Information on Exploration Borings.*

Table No. 2. Summary of Information on Exploration Borings Drilled in September, 1993.

Boring Number	Approximate Location	Existing Grade Elevation (feet)	Depth of Boring (feet)	Depth to Water Table (feet)
BF-1	Sta. 670+00	1405	25	19
BF-2	Sta. 656+00	1400	25	12
BT-1	Sta. 640+70	1403	25	16
BT-2	Sta. 620+00	1418	24	22
BT-3	Sta. 605+00	1420	30	26
BC-1	Sta. 566+00	1398	15	None Encountered
BE-1	Sta. 478+30	1422	20	None Encountered
BE-2	Sta. 443+00	1445	15[1]	None Encountered
BE-3	Sta. 406+00	1313	10[1]	None Encountered
BE-4	Sta. 373+30	1303	6[1]	None Encountered

[1] Borings ended due to refusal

The existing grade elevations at the locations of the borings included in Table No. 2, *Summary of Information on Exploration Borings,* are based on the contours shown in the Reach 4 Pipelines Contract II drawing prepared by Black and Veatch. The locations of the borings are plotted in Drawing No. 1, *Geologic/Boring Location Map.*

Subsurface conditions encountered during drilling were continuously logged by our field engineer. Relatively undisturbed and bulk samples of the subsurface materials were obtained from the borings at frequent intervals for laboratory testing and visual classification. A detailed description of the field exploration procedures and boring log summary sheets is presented in Appendix A, *Field Exploration.*

2.3 Laboratory Testing

Representative soil samples obtained during the subsurface exploration were tested in our laboratory to determine their engineering properties. The tests included moisture-density determination, gradation analysis, laboratory maximum density determination, direct shear, and consolidation. Descriptions and results of the laboratory tests are presented in Appendix B, *Laboratory Testing Program*.

2.4 Seismic Refraction Survey

A seismic refraction survey along approximately 10,500 linear feet of the proposed alignment was performed by Subsurface Surveys.

The purpose of the seismic refraction survey was twofold:

- To estimate the depth to bedrock along the alignment

- To determine the seismic velocities of the subsurface materials along the alignment for evaluation of rock excavatability

A detailed description of the seismic refraction survey procedures and interpretation of results obtained are included in Appendix C, *Seismic Refraction Survey*.

2.5 Corrosivity Testing

A soil corrosivity study was performed by M. J. Schiff and Associates. Representative soil samples retrieved during drilling were tested in their laboratory for pH, chloride, sulfate, chloride and electrical resistivity. The *in situ* electrical resistivity of the soils was measured using the Wenner Four Pin Method. Descriptions and results of field and laboratory testing are presented in Appendix D, *Soil Corrosivity Study*.

2.6 Analyses and Report

Data obtained from the exploratory field work and laboratory testing program were evaluated. Geotechnical analyses were performed, and we prepared this report to present our findings and recommendations developed during the investigation.

3.0 SITE CONDITIONS

3.1 General

The portion of the proposed pipeline alignment addressed in this report consists of two separate segments, as defined in Section 1, *Introduction*, and is about 25,798 linear feet.

Segment I: Existing topography consists of a moderate to deeply incised canyon. Existing improvements along the alignment include areas of extensive cuts as well as relatively deep fill slopes associated with the construction of Railroad Canyon Road.

Segment II: The topography along most of this segment of the alignment can be considered to be mildly sloped to relatively flat.

3.2 Subsurface Conditions

Our evaluation of the subsurface conditions along the proposed pipeline alignment was based on observations made during geologic mapping, field exploration, soil samples, and laboratory test results.

Segment I: The depth to bedrock along this segment of the alignment varies from about a foot to greater than 12 feet below existing grade. The degree of weathering of the granitic bedrock encountered within the depths of exploratory borings ranges from very little to moderate. The subsurface materials overlying bedrock are composed of either fill or native alluvium deposits. The depth of fill materials is about one foot. The fills soils are very dense and composed of silty sands. These fills form the subgrade of the pavement along Railroad Canyon Road.

The native soils occur as relatively loose alluvium deposits within the natural drainage channels that cross Railroad Canyon Road. The alluvial materials were encountered in Boring BE-1 to a depth of 12 feet near the eastern end of the segment and comprised mainly silty sand with trace clay and gravel.

Segment II: The depth to bedrock along this segment of the project alignment varies from about three to 18 feet below existing grade. The degree of weathering of the granitic bedrock encountered within the depths of exploratory borings ranges from severe to moderate. The subsurface materials overlying bedrock are composed mainly of native alluvial deposits with one to two feet of fills. The native soils occur as relatively loose to medium dense alluvium deposits and are composed mainly of silty and clayey sands.

The compressibility of the subsurface materials at the elevation of the proposed pipe invert is in general low and is slightly to moderately susceptible to moisture variation. The direct shear tests indicate that the friction angle and cohesion of the native alluvial soils varied from about 38 to 41 degrees and from 675 to 1,176 psf, respectively. The corresponding shear strength parameters for the weathered granitic bedrock from the elevation of the proposed pipe invert varies from 37 to 46 degrees and from 580 to 1,000 psf, respectively.

The Sand Equivalent (SE) of the representative medium-grained soils resulting from disintegration of weathered granitic bedrock along Segment I is found to be 27. The SE value for native alluvium soils from Segment I and II varies from eight to 15.

The resistance (R) value of the soils at the road subgrade elevation varies from 27 to 67 along the alignment.

Detailed descriptions of the subsurface conditions and laboratory tests are included in the logs of borings, Appendix A, *Field Observation*, and Appendix B, *Laboratory Testing*, respectively.

4.0 GEOLOGY

<u>4.1 Geologic Setting</u>

The project area is located within the Peninsular Ranges Physiographic Province. The Peninsular Ranges Physiographic Province consists of a series of northwest-trending mountain ranges and valleys bounded on the north by the San Bernardino and San Gabriel mountains, on the west by the Los Angeles Basin, and on the south by the Pacific Ocean.

The province is a seismically active region characterized by a series of northwest-trending, strike-slip faults. The most prominent of these faults include the San Jacinto and Elsinore faults, both of which have been shown to be active within Quaternary time.

Topography within the province is generally characterized by broad alluvial valleys separated by linear mountain ranges. This northwest-trending linear fabric is created by regional faulting within the granitic basement rock of the Southern California Batholith.

Broad linear alluvial valleys have been formed by erosion of these principally granitic mountain ranges.

4.2 Site Geology

Earth materials underlying the study area consist of, from oldest to youngest, Triassic-age metamorphic rocks, including siliceous metasandstones and shales, Cretaceous- age granitic rocks, including granite, granodiorite, and diorite, Pleistocene-age alluvium, recent and older alluvial deposits and artificial fill. The aerial distribution of these earth materials is shown in Drawing No. 1, *Geologic/Boring Location Map*, and is described below.

4.2.1 Metasandstone and Shale (Map unit Trsa) Quartz-rich metasandstone and shale of the Triassic-age Santa Ana Formation underlie a portion of the alignment from the entrance to the Railroad Canyon at Salt Creek to the vicinity of Sorrel Lane. The formation is visible in roadcuts and outcrops within the canyon and was encountered in Boring BC-1 at a depth of 10 feet. The formation generally consists of resistant, light grey to brown, quartz-rich shale and metasandstone.

4.2.2 Granitic Rock (Map units Ku and Kg) Cretaceous-age granite, granodiorite, and diorite underlie much of the alignment as evidenced from numerous roadcuts. These units generally consist of moderately to severely weathered granitic, granodiorite and diorite near the surface to locally, relatively resistant unweathered granitic rock at depth.

4.2.3 Older Alluvium (Map unit Qoal) Older (Pleistocene-age) alluvium blankets the majority of the alignment between approximate Station 574+00 and 696+77. The older alluvium typically consists of moderately to well consolidated mixtures of sands, gravels, silts and clays. The older alluvium commonly exhibits moderate to strong soil development. The older alluvium between Station 574+00 and 696+77 generally ranges from three to 18 feet deep based on the alluvial deposits encountered in borings BF-1, BF-2, and BT-1 through BT-3. The older alluvium may be deeper or shallower in areas between borings.

4.2.4 Recent Alluvium (Map unit Qal) Younger (Holocene-age) alluvium occurs in the active drainage courses within the alignment. These deposits vary in thickness from less than one foot thick to over 10 feet thick (estimated) in the drainage for Salt Creek. These deposits typically consist of unconsolidated mixtures of silts, sands and gravel.

4.2.5 Artificial Fill (Map unit af) Artificial fill occurs locally within the alignment and is associated with road construction and residential development adjacent to the alignment.

Artificial fill typically consists of moderately dense to dense mixtures of silty sands and clayey sands. The existing fill underlying Railroad Canyon Road between approximate Station 370+00 and 480+00 was measured in borings BE-1 through BE-4 at about one foot thick. Fill underlying the northeastern portion of the alignment between Salt Creek and Station 696+77 also appears to be shallow based on the borings.

5.0 FAULTING AND SEISMICITY

5.1 Faulting

There are no known active faults projecting toward or extending across the alignment. The project site is not situated within a currently designated State of California Alquist-Priolo Special Studies Zone. For pipeline Segment I (from Station 356+80.24 to Station 480+00), the nearest known active fault zones are the Elsinore, Whittier-North Elsinore, Chino and Casa Loma-Clark (San Jacinto) fault zones, located approximately three to 19 miles southwest, northwest, north and northeast, respectively, from the site. For pipeline Segment II (from Station 562+00 to Station 696+77.58), the nearest known active fault zones are the Elsinore, Casa Loma-Clark (San Jacinto), Glen Helen-Lytle Creek-Claremont and Whittier-North Elsinore fault zones, located approximately six to 18 miles southwest, northeast, north and northwest, respectively, from the site. Table nos. 3 and 4, both entitled, *Seismic Characteristics of Regional Faults,* present distances of the various nearby faults from the two ends of the proposed pipeline Segments I and II, respectively.

Of those faults listed in Table nos. 3 and 4, both entitled, *Seismic Characteristics of Regional Faults*, the Elsinore, Whittier-North Elsinore, Casa Loma-Clark (San Jacinto), Chino and Glen Helen-Lytle Creek-Claremont fault zones are considered most capable of generating significant ground motions along the proposed alignment.

5.2 Seismicity

The proposed pipeline segments are situated in a seismically active region. As is the case for most areas of Southern California, ground shaking resulting from earthquakes associated with nearby and distant faults will occur. During the life of the project, seismic activity associated with active faults in the area may generate moderate to strong ground shaking at the project site.

According to the Uniform Building Code (1991 edition), the proposed pipeline segments are situated in Seismic Zone 4. Major damage corresponding to intensities VIII or higher on the Modified Mercalli Intensity Scale should be expected within this zone. Seismic Zone 4 also includes those areas that lie within a zone of major (Richter magnitude greater than seven) historic earthquakes and recent high levels of seismicity.

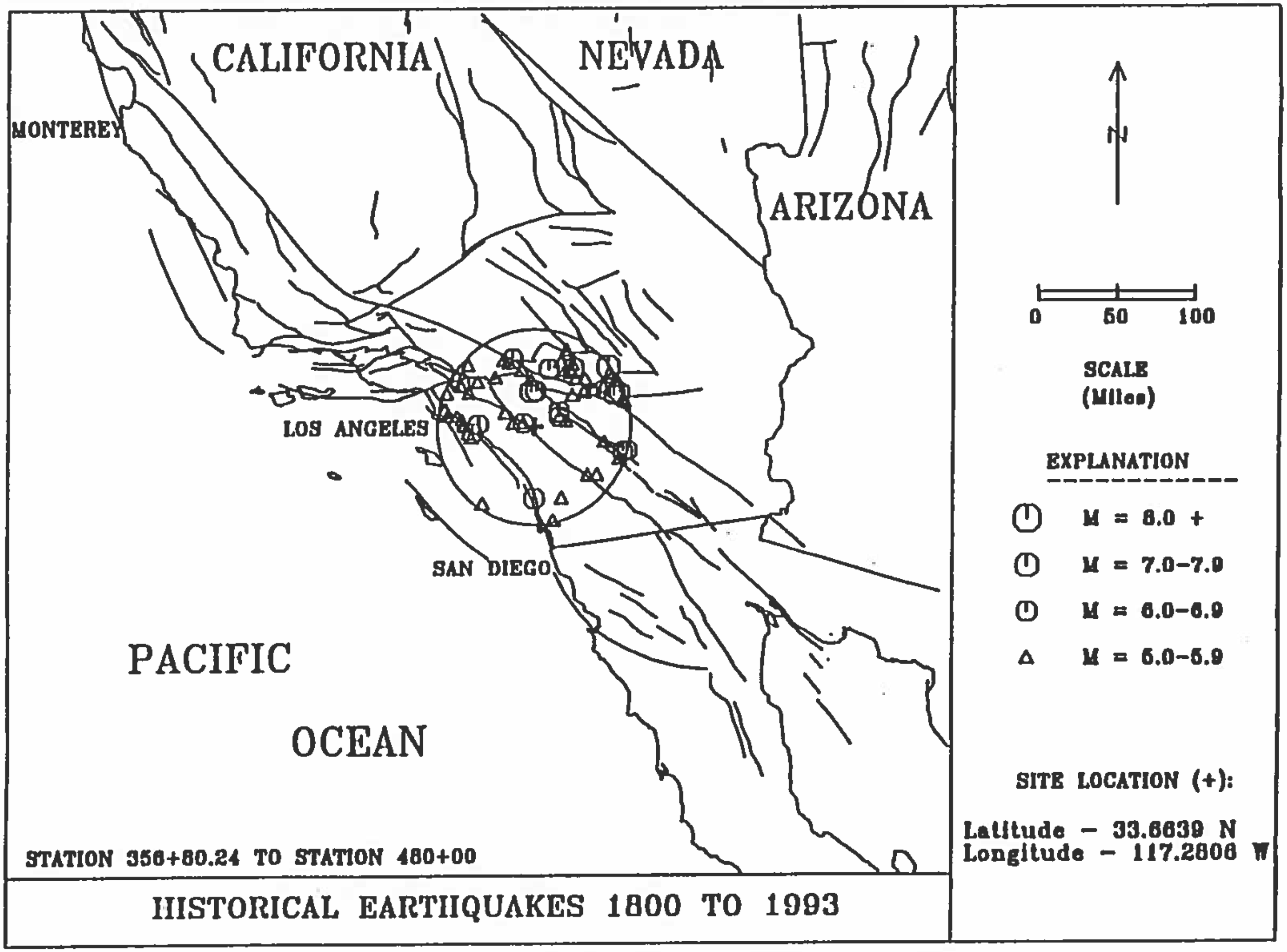
A maximum credible earthquake is defined as the maximum seismic event that a particular fault is theoretically capable of producing and is evaluated based upon existing geologic and seismologic evidence.

Various active faults within a 100 km (62-mile) radius of the site are shown in Figure No. 2, *Fault Map*. Historical seismic events within a 100 km (62-mile) radius of the mid-points of pipeline Segments I and II are shown in Figure Nos. 3 and 4, both entitled, *Earthquake Epicenter Map*, respectively.

Maximum credible earthquakes and associated seismic parameters for various active faults within a 100-km (62-mile) radius of the site shown in Figure No. 2, *Fault Map*, are included in Table Nos. 3 and 4, both entitled, *Seismic Characteristics of Regional Faults*, respectively.

Based on the location of the faults, for pipeline Segment I a maximum credible horizontal bedrock acceleration of 0.60g and 0.71g, where g is the acceleration due to gravity, should be expected at the western end (Station 356+80.24) and the eastern end (Station 480+00), respectively. The corresponding maximum credible horizontal bedrock accelerations at the western end (Station 562+00) and the eastern end (Station 696+77.58) of pipeline Segment II are 0.53g and 0.45g, respectively.

Based on a deterministic analysis, a peak horizontal ground surface acceleration of 0.40g and 0.48g should be expected at Stations 356+80.24 and 480+00, respectively, of pipeline Segment I within a 100-year design period. The corresponding values of the accelerations at stations 562+00 and 696+77.58 of pipeline Segment II are 0.29g and 0.36g, respectively.



EARTHQUAKE EPICENTER MAP (SEGMENT I)

REACH 4, RWBWTP, CONTRACT II, EMWD
Riverside County, California
for: Black & Veatch

Project No. 92-81-504-01

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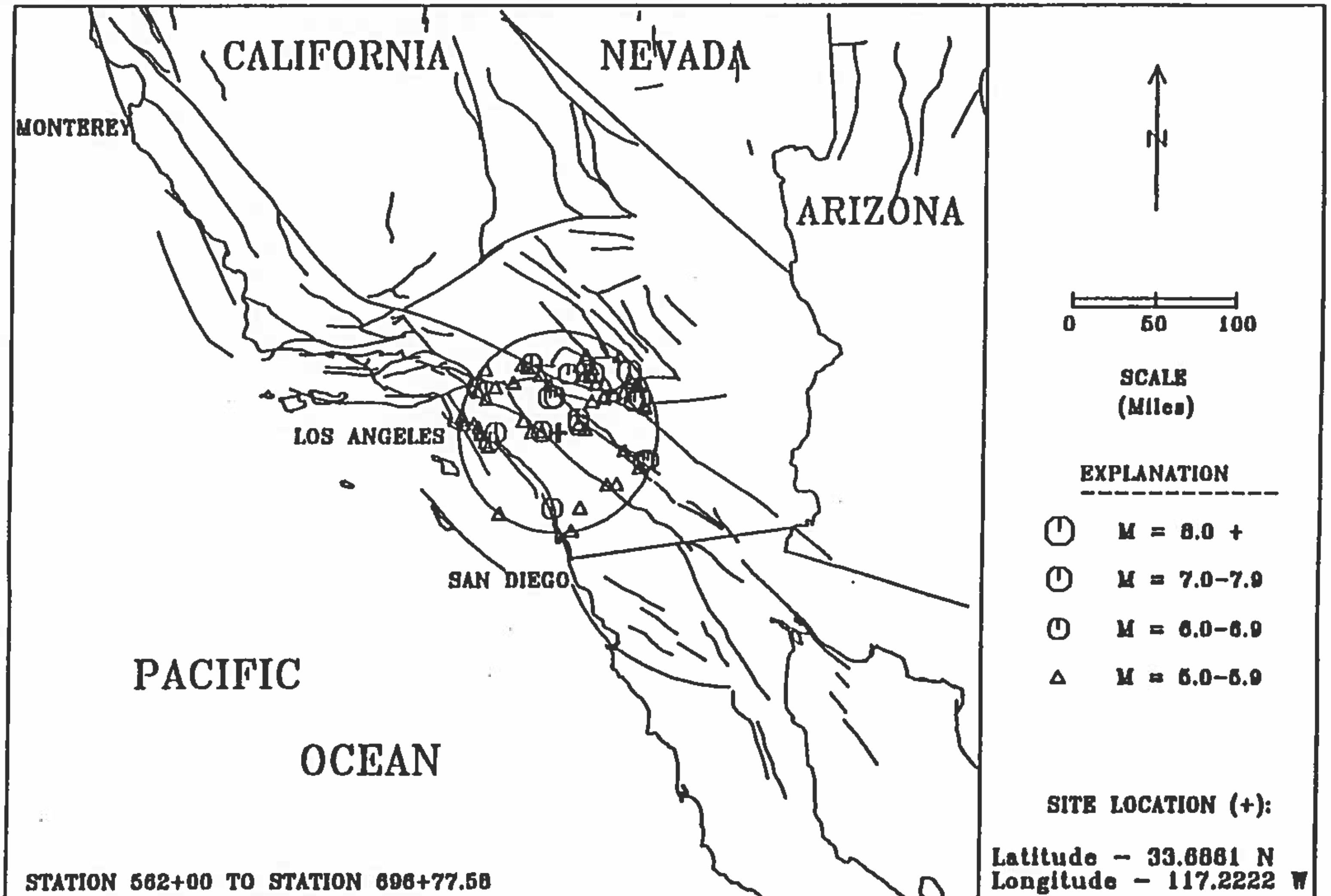
Figure No. 3



HISTORICAL EARTHQUAKES 1800 TO 1993

EARTHQUAKE EPICENTER MAP (SEGMENT II)

REACH 4, RWBWTP, CONTRACT II, EMWD
Riverside County, California
for: Black & Veatch

Project No. 92-81-604-01

Converse Consultants Inland Empire

Figure No. 4

Table No. 3. Seismic Characteristics of Regional Faults (Segment I: Sta. 356+80.24 to Sta. 480+00)

Fault Name	Minimum Distance To Sta. 356+80.24/ STA. 480+00 (miles)	Approx. Total Fault Length (miles)	Site Maximum Credible Earthquake Moment Magnitude (Mw)[1]	Maximum Peak Horizontal Bedrock Acceleration at Sta. 356+80.24/ Sta. 480+00 (g)[2]	Duration of Strong Shaking[3] (sec)
Elsinore	2.6 / 4.4	125.0	7.5	0.71 / 0.60	30 - 35
Whittier-North Elsinore	15 / 17	43.8	7.5	0.23 / 0.21	25 - 30
Chino	17 / 18	21.9	7.0	0.16 / 0.07	22 - 27
Casa Loma-Clark (San Jacinto)	19 / 17	62.5	7.0	0.14 / 0.16	22 - 27
Glen Helen-Lytle Creek-Claremont (San Jacinto)	20 / 19	50.0	7.0	0.13 / 0.14	21 - 26
Hot Spring-Buck Ridge (San Jacinto)	23 / 21	93.8	7.0	0.11 / 0.12	21 - 26
San Gorgonio-Banning	25 / 25	56.3	7.5	0.13 / 0.13	24 - 29
Newport-Inglewood-Offshore Zone	29 / 31	102.0	7.0	0.08 / 0.08	20 - 25
San Andreas (San Bernardino Mountains)	34 / 33	139.2	8.0+	0.12 / 0.12	24 - 29
Cucamonga	37 / 37	25.0	7.0	0.06 / 0.06	15 - 20
Rose Canyon	37 / 38	46.9	7.0	0.06 / 0.06	15 - 20

Notes:

(1) Moment Magnitude Mw of earthquake expected for rupture of entire fault length, estimated with slip rate dependent empirical relations between seismic moment Mo and fault length and assuming the empirical relationship Log Mo=1.5 Mw + 16.1 (Hanks and Kanamori, 1979).

(2) Based on computer program EQFAULT By Thomas F. Blake using Joyner & Boore (1982)-Larger and Mean Method.

(3) Bolt, 1977, bracketed duration.

Table No. 4. Seismic Characteristics of Regional Faults (Segment II: Sta. 562+00 to Sta. 696+77.58)

Fault Name	Minimum Distance to Sta. 562+00/ Sta.696+77.58 (miles)	Approx. Total Fault Length (miles)	Maximum Credible Earthquake Moment Magnitude (Mw)[1]	Maximum Peak Horizontal Bedrock Acceleration at Sta. 562+00/Sta. 696+77.58[2] (g)	DURATION OF STRONG SHAKING[3] (sec)
Elsinore	5.6 / 7.4	125.0	7.5	0.53 / 0.45	30 - 35
Casa Loma-Clark (San Jacinto)	15 / 14	62.5	7.0	0.17 / 0.19	22 - 27
Glen Helen-Lytle Creek-Claremont (San Jacinto)	18 / 16	50.0	7.0	0.15 / 0.17	21 - 26
Whittier-North Elsinore	18 / 18	43.8	7.5	0.19 / 0.19	25 - 30
Chino	19 / 20	21.9	7.0	0.14 / 0.13	24 - 29
Hot Spring-Buck Ridge (San Jacinto)	20 / 18	93.8	7.0	0.13 / 0.14	21 - 26
San Gorgonio-Banning	24 / 22	56.3	7.5	0.14 / 0.15	25 - 30
San Andreas (San Bernardino Mountains)	32 / 30	139.2	8.0+	0.13 / 0.13	26 - 31
Newport-Inglewood-Offshore Zone	33 / 35	102.0	7.0	0.07 / 0.07	19 - 24
Cucamonga	37 / 36	25.0	7.0	0.06 / 0.06	16 - 21
Rose Canyon	39 / 40	46.9	7.0	0.06 / 0.05	13 - 18

Notes:

(1) Moment Magnitude Mw of earthquake expected for rupture of entire fault length, estimated with slip-rate dependent empirical relations between seismic moment Mo and fault length and assuming the empirical relationship Log Mo=1.5 Mw + 16.1 (Hanks and Kanamori, 1979).

(2) Based on computer program EQFAULT by Thomas F. Blake using Joyner & Boore (1982)-Larger and Mean Method.

(3) Bolt, 1977, bracketed duration.

6.0 GROUNDWATER

The depth of ground water within proposed pipeline Segment II is governed by the elevation of the water in the Canyon Lake Reservoir. The elevation of water in the reservoir at the spillway location is about 1,382 feet above Mean sea level (msl). The regional ground-water elevation is expected to be near that elevation. Based on the information provided by Mr. Dick Morton with the Eastern Municipal Water District (EMWD), as of January 11, 1993, the depth of ground water in a monitoring well approximately 200 feet north of Newport Road and near the Sun City RWRF was about 19 feet below surface. The ground elevation at the location of the well is about 1,408 feet above msl.

Groundwater was encountered in all of the borings drilled along pipeline Segment II except Boring No. BC-1, which was drilled to a depth of 15 feet only. The depths of the water table below existing grade at each of the boring locations are shown in Table No. 2, *Summary of Information on Exploratory Borings*. At the time of our field exploration, the depth of the ground-water table along Pipeline Segment II varied from 12 to 25 below existing ground surface. The subsurface materials at these depths were composed of decomposed granite and acted as the water-bearing strata.

No ground water was encountered in the exploratory boring drilled to a maximum depth of 20 feet below existing grade along the pipeline Segment I. The hydrogeologic setting along this segment of the proposed alignment is characterized by steep hills on the south side. On the north side of the alignment the ground slopes downward to the bottom level of the San Jacinto River. Seasonal variations in the depth of groundwater should be expected.

7.0 LIQUEFACTION EVALUATION

Soil liquefaction can occur during or immediately following a strong ground shaking due to earthquakes caused by the movement of nearby faults. It has been well documented in the literature, based on research and experience over the last three decades, that liquefaction due to seismic shaking occurs only in saturated contractive granular soils. The contractiveness of a given soil element subjected to cyclic shear stress depends both on the *in situ* effective confining stress and the density (void ratio). At low effective stress, even a loose granular soil element may exhibit dilation when subjected to shear stresses. A very dense granular soil element under high confining pressure may contract due to applied shear stresses.

Furthermore, the applied shear stress caused by the ground motion must be high enough to exceed the steady-state shear strength of the granular soil element. The duration of the shaking must also be long enough to develop excess pore pressure required to bring the state of element to a steady state of deformation. The magnitude of the applied shear stress and the duration of its application required to cause liquefaction also depends on the static shear stresses acting on the element at the onset of shaking.

Liquefaction potential along the proposed pipeline alignment is estimated to be low for the following reasons:

- Present ground-water levels that are generally below proposed pipe invert elevations

- Shallow weathered bedrock that extends above the elevations of the present water table and/or pipe invert

- The native silty and/or clayey sand materials, as observed during drilling operations and confirmed by laboratory testing, are mainly medium dense and contain significant amount of fines and are not likely to liquefy.

Earthquake damage to buried pipelines can occur due to permanent ground deformation or seismic wave propagation. Permanent ground deformation refers to landslides, surface faulting, settlement, or liquefaction-induced lateral spreading. Damage potential due to permanent deformation is considered to be low for the proposed pipeline.

Seismic wave propagation refers to strains and curvatures that result from seismic waves traversing the ground. Wave propagation damage to buried pipelines can occur over the entire alignment. However, the potential of wave propagation damage to buried pipelines is higher in areas of relatively abrupt changes in subsurface conditions, such as rock to soils and vice versa, due to differential ground strain.

There are two types of action that can be taken against seismic damage to buried pipelines. These are physical damage mitigation and impact mitigation. Physical damage mitigation involves strengthening and retrofitting various system components with the objective of reducing or eliminating potential seismic damage.

Since damage to pipelines due to wave propagation can occur through the entire alignment, physical damage mitigation may not be economically justifiable. For the subject project, physical damage mitigation measures in areas of abrupt changes in subsurface conditions should greatly reduce the potential of seismic damage. The extent of seismic damage also depends on the pipe material. Welded steel gas-welded joint (WSGWJ) pipes suffered less damage in the past compared to pipes made of other common materials such as cast iron, PVC, ductile iron, etc.

Impact mitigation for the subject project should include plans for rapid repair of the system following seismic damage.

8.0 SEISMIC REFRACTION SURVEY

8.1 General

A seismic refraction survey was performed along approximately 10,500 linear feet of the proposed alignment. The seismic lines (S-1, S-2, and S-10) are shown in Drawing No. 1, *Geologic/Boring Location Map*. The purpose of the seismic refraction survey was twofold:

- To determine the seismic velocities of the soil and bedrock along the alignment for evaluation of rock excavatability.
- To estimate the depth to bedrock.

The description of the methodology that was followed is included in Appendix C, *Seismic Refraction Survey*.

8.2 Bedrock Excavatability

Bedrock excavatability characteristics along the proposed alignment were evaluated based on:

- Seismic refraction survey data
- Subsurface exploration
- Observation of existing bedrock exposures along road cuts
- Observation of recent trench and retaining wall excavations along the proposed alignment

The excavatability of the bedrock (degree of excavation difficulty) along the proposed alignment may be correlated with seismic velocities. In general, the higher the seismic velocity, the harder the bedrock and the more difficult the excavation. Excavatability is also dependant upon pervasiveness and character of joints and fractures in the rock, type of equipment used, and experience and ability of the operator.

The information provided herein is intended to aid in selecting the proper equipment for the subject project based on the excavation characteristics of the subsurface materials. We recommend that the contractor review our data and conclusions contained in this section and in Appendix C, *Seismic Refraction Survey*, and evaluate the excavation characteristics based on his experience, equipment available and knowledge of local conditions. It is the responsibility of the contractor to choose the appropriate equipment and methods of excavation based on the excavation characteristics and on his/her own experience and knowledge of local conditions.

9.0 SOIL CORROSIVITY EVALUATION

M. J. Schiff & Associates was retained by CCIE to evaluate soil corrosivity along the proposed pipeline alignment. Detailed discussion on the soil corrosivity study procedure, results and recommended mitigative measures is included in Appendix D, *Soil Corrosivity Study*.

10.0 CONCLUSIONS

- Based on the results of our geotechnical investigation, we conclude that the proposed alignment is geotechnically suitable for the pipeline project, provided the recommendations contained in this report are incorporated into the design and construction.

- The subsurface materials along the alignment vary significantly. The proposed pipeline alignment is underlain by weathered granitic bedrock to depths varying from about one foot to 18 feet below existing grade. The overlying loose to very dense materials are composed of fills and native alluvium soils consisting of silty sand and clayey sand.

- No groundwater was encountered at the location of the borings drilled along pipeline Segment I. Groundwater depth along pipeline Segment II varied from about 12 feet to 25 feet below existing grade. Seasonal fluctuations in groundwater level should be anticipated.

- Due to the shallow depth of bedrock, the relatively deep ground water table and medium dense native soils that contain appreciable amounts of fines, liquefaction potential of the subsurface soils along the alignment is considered low.

- Based on the study of soil corrosivity, the portion of the alignment from station 562+00 to Station 696+77.58 is classified as severely corrosive to ferrous metals and deleterious to concrete. The remainder of the alignment is classified as moderately corrosive to ferrous metals.

11.0 DESIGN RECOMMENDATIONS

11.1 Soil Parameters for Pipeline Design

Structural design of pipes requires proper evaluation of all possible loads acting on the pipeline, including dead and live or transient loads. The stresses induced in a buried pipe by the imposed dead and live or transient loading depend on the type of pipe, i.e., either rigid or flexible.

The maximum dead load imposed on the pipeline by the backfilled soils is a function of the depth and width of the trench, soil unit weight, angle of internal friction, coefficient of active earth pressure, and coefficient of friction at the interface between the backfill and native soils.

The recommended values of the various soil parameters that the pipe design engineer may use for the design of the subject pipelines are provided in the following sections.

11.1.1 Segment I (Station 356+80.24 to Station 480+00) The following soil parameters are recommended for this segment of the proposed pipeline.

- Soil bulk unit weight — γ = 137 pcf
- Angle of internal friction of soils — ϕ = 37 degrees
- Soil cohesion — c = 500 psf
- Coefficient of friction between backfill and native soils — f = 0.45
- Coefficient of active earth pressure — K_a = 0.40
- Coefficient of friction at cement-mortar-coated steel pipe-soil interface — = 0.35-0.40

•	Coefficient of friction at tape-wrapped steel pipe-soil interface	= 0.20-0.25

The modulus of soils reaction, E', for *in situ* soils at pipe invert elevation may be assumed to be in the range of 1,500 to 1,800 pounds/in^2 for flexible pipe deflection calculation.

11.1.2 Segment II (Station 562+00 to Station 696+77.58) The following soil parameters are recommended for this segment of the proposed pipeline.

•	Soil bulk unit weight	γ = 128 pcf
•	Angle of internal friction of soils	ϕ = 40 degrees
•	Soil cohesion	c = 300 psf
•	Coefficient of friction between backfill and native soils	f = 0.50
•	Coefficient of active earth pressure	K_a = 0.37
•	Coefficient of friction at cement mortar coated steel pipe-soil interface	= 0.35-0.40
•	Coefficient of friction at tape-wrapped steel pipe-soil interface	= 0.20-0.25

The modulus of soils reaction, E', for *in situ* soils at pipe invert elevation may be assumed to be in the range of 1,200 to 1,500 pounds/in^2 for flexible pipe deflection calculation.

11.2 Allowable Bearing Pressures for Footings

The air and vacuum relief manholes and the blowoff structures can be supported on strip and rectangular footing foundations, respectively. In addition to soil characteristics, the allowable bearing pressures of footing foundations depend on the width and the embedment depth. Assuming that the width of footing would be at least 18 inches, and that the minimum embedment depth would be 4.0 feet, (as is shown in the Reach 4 Contract I drawings prepared by Black & Veatch), the following allowable bearing capacities are recommended for the entire alignment:

- Footings founded on decomposed granite = 4,500 psf

- Footings founded on native and/or compacted soils = 2,500 psf

For short-term seismic and wind loadings the above-recommended allowable bearing capacities can be increased by 33 percent.

11.3 Lateral Earth Pressure on Permanent Retaining Walls

11.3.1 Seismic Lateral Earth Pressures For design of buried walls retaining granular backfills, seismic lateral earth pressures should be applied in the form of an inverted triangle as shown in Figure No. 5, *Recommended Seismic Lateral Earth Pressures*. The maximum pressure at the top of the walls within the pipeline Segments I and II should be 28H pounds/ft^2 and 40H pounds/ft^2, respectively, where H is the height of the wall in feet.

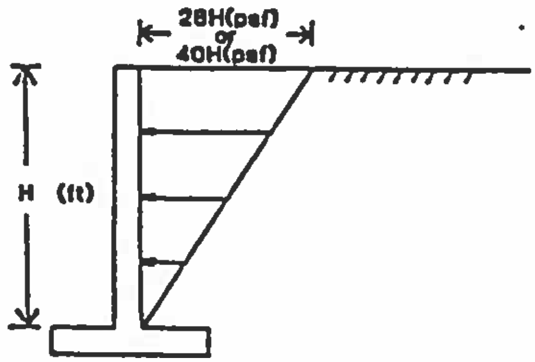
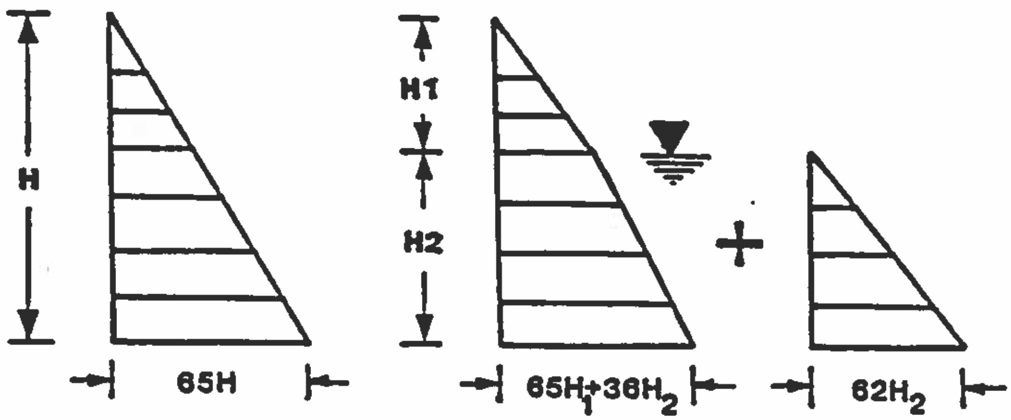


Figure No. 5, *Recommended Seismic Lateral Earth Pressures*

11.3.2 Static Lateral Earth Pressures

Cantilevered Walls An active earth pressure equal to that developed by a fluid with a density of 50 pounds per cubic foot (pcf) for drained soils may be used for design of cantilevered walls. For walls below water table, an earth pressure developed by a fluid with a density of 88 pcf may be used for the design of cantilever walls. Surcharge pressure should be added to the earth pressures for surcharge within a distance from the top of the wall less than or equal to the wall height. One-third of any uniform surcharge load should be added to the above pressure.

Nonyielding Walls For design of nonyielding retaining walls, we recommend that "at rest" earth pressure with a triangular distribution as shown in Figure No. 6, *Lateral Pressures on Nonyielding Walls,* be used. Walls should be designed to resist hydrostatic pressure below water table. The recommended earth pressure and hydrostatic pressure distribution are shown in Figure No. 6, *Lateral Pressures on Nonyielding Walls,* Half of any uniform surcharge load should be added to the above pressure.



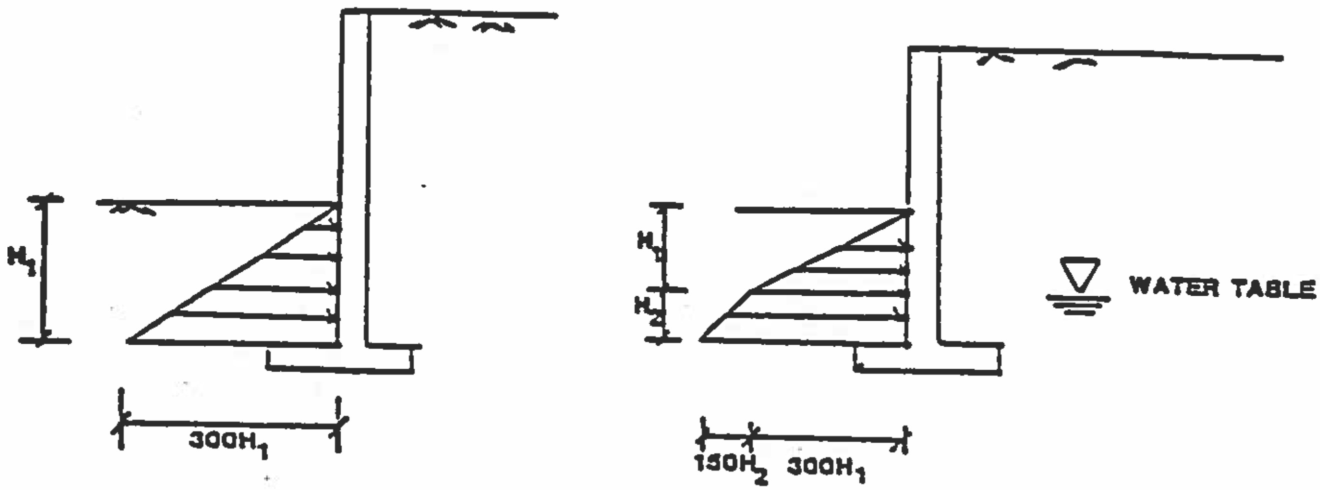
(a) Drained (b) Undrained

Figure No. 6, *Lateral Pressures on Nonyielding Walls*

In addition to the above recommended static and seismic lateral earth pressures, the upper 10 feet of retaining walls adjacent to roadways should be designed to resist a uniform lateral pressure of 100 psf, acting as a result of an assumed 300 psf surcharge behind the wall due to normal traffic. If the traffic is kept 10 feet from the wall, the traffic surcharge may be neglected.

11.4 Resistance to Lateral Loads

Resistance to lateral loads can be assumed to be provided by friction acting at the base of foundations of underground structures and by passive earth pressures. An allowable coefficient of friction of 0.40 may be applied to the dead load forces. An allowable passive earth pressure of 300 pounds per square foot per foot of depth may be used for the sides of footings poured against undisturbed or compacted on-site soils and for retaining walls. The corresponding value for passive resistances from the soils below water table may be assumed to be ($300H_1+150H_2$), as shown in Figure No. 7, *Lateral Resistance Due to Passive Earth Pressures.*



(a) Drained (b) Undrained

Figure No. 7, *Lateral Resistance Due to Passive Earth Pressures*

The value of the passive lateral earth pressure should be limited to 3,000 psf. The allowable passive pressure can be increased by 33 percent for seismic or wind forces. The frictional resistance and the passive resistance of the soils may be combined without reduction in determining the total allowable lateral resistance.

11.5 Bearing Pressure for Anchor and Thrust Blocks

Allowable bearing pressure of 2,500 pounds per square foot may be used for anchor and thrust block design buried below a depth of at least four feet. Resistance to lateral forces can be assumed to be provided by friction at the base of thrust blocks and by passive earth pressure. A friction coefficient of 0.4 may be used between the thrust block and the supporting natural soil or compacted fill. An allowable passive earth pressure of 300 psf per foot of depth may be used for the sides of thrust blocks or anchors poured against undisturbed or recompacted soils. The value of the passive lateral earth pressure should be limited to 3,000 psf.

The allowable passive pressure may be increased by 33 percent for seismic or wind forces. Frictional and passive resistance can be combined for the design of anchors and thrust blocks.

11.6 Settlements

During excavation of the pipeline trench, the soils below invert elevations will undergo elastic rebound due to the removal of overburden pressures. During backfilling of the trench, the soils beneath the pipe can settle. Based upon our test results, pipe settlement due to recompression is estimated to be on the order of 0.50 to 1.00 inch.

11.7 Special Considerations

Based on the information provided to us by Black and Veatch, various utility lines exist along the proposed pipeline alignment. The depths and the locations of the existing utility lines are such that to protect these lifelines during construction, special construction considerations would be required during excavation.

Excavations for the proposed pipelines should not extend below a 1:1 (horizontal:vertical) plane extending beyond and down from the bottom of the existing utility lines. The proposed excavation should not cause loss of bearing and/or lateral supports of the existing utilities.

Open cuts adjacent to existing roadways and/or adjacent structures are not recommended within a 1:1 (horizontal:vertical) plane extending beyond and down from the roadway or structure perimeter.

Spoils from the trench excavation should not be stockpiled more than six feet high adjacent to an open trench. Soils should not be stockpiled behind the shoring to a horizontal distance equal to the depth of the trench unless the shoring has been designed for such loads.

12.0 EARTHWORK RECOMMENDATIONS

12.1 General

Earthwork for this project will consist primarily of trench excavation for construction of the pipeline. The approximate depth of excavation will vary from about nine feet to 11 feet for most of the alignment. Excavation depths greater than 11 feet and up to a maximum of about 20 feet would be required at certain sections of the alignment as shown in Table No. 1, *Locations with Depth of Invert Greater Than 10 Feet Below Existing Grade.*

Suggested materials for pipe bedding and trench zone backfill and backfill compaction procedures are included in the following sections.

12.2 Pipe Bedding

To provide uniform and firm support for the pipeline, free-draining granular soil should be used as pipe bedding material. For flexible pipes, sand can be used as bedding material. Crushed rock or gravel can be used under rigid pipes. The thickness of the bedding material should be selected by the pipeline design engineer.

Prior to placing the bedding material, the trench excavation should be cleared of soft or disturbed materials. Bedding material should be placed on firm, undisturbed native soils or rocks. All cobbles, boulders, or rocks projecting into the pipe zone should be removed prior to placement of bedding material. Bedding material should be placed a minimum of 12 inches above the top of pipe or as required by the pipe design engineer.

Bedding material should be vibrated in-place, and care should be taken to densify the bedding material below the spring line of the pipe. Flooding or jetting of the bedding material should not be attempted because the water from the trench is not expected to drain freely. Long-term accumulation of water in the pipe trench from any sources should be avoided, and trenches should be pumped dry if water collects inside.

12.3 Trench Backfill

Backfill for the remainder of the trench above pipe bedding (Trench Zone) should be placed in lifts as recommended in Section 12.4, *Recommended Specifications for Placement of Trench Backfill*. Excavated on-site soils and/or imported soils used as trench zone backfill should meet the following criteria:

- No particles larger than three inches in largest dimension
- Less than 30 percent by weight retained in 3/4" sieve.
- Free of all perishable materials
- Plasticity index of 10 or less

After removal of oversize materials, excavated on-site soils are expected to meet these criteria.

Imported backfill soils, in addition to satisfying the above criteria, should have a Sand Equivalent of at least 30.

Trench backfill should be compacted to at least 90 percent of the maximum laboratory density at a moisture content within 2.5 percent of the optimum moisture content. Detailed trench backfill compaction recommendations are included in Section 12.4, *Recommended Specifications for Placement of Trench Backfill*. Compaction of the soils containing appreciable amount of fines (SM and SC) to 90 percent relative compaction may require more compactive effort than the more granular soils. Trench backfills underlying pavements should be compacted to at least 95 percent of the maximum laboratory density at a moisture content within two percent of the optimum moisture content, to a depth of at least 12 inches below the pavement base.

12.4 Recommended Specifications for Placement of Trench Backfill

- Trench excavations to receive backfill shall be free of trash, debris or other unsatisfactory materials at the time of backfill placement.

- Trench backfills shall be compacted to a minimum relative compaction of 90 percent. Relative compaction is defined as the ratio of the in-place soil density to the laboratory maximum dry density as determined by the ASTM Standard D1557-91 test method.

- Trench backfill underlying pavements, shoulders and sidewalks shall be compacted to not less than 95 percent of the maximum dry density, to a depth of at least 12 inches below the pavement section.

- Rocks generated from the trench excavation not exceeding three inches in largest dimension may be used as backfill material. However, such materials may not be placed within 12 inches of the top of pipe and the upper 12 inches of the trench underneath pavements. No more than 30 percent of the backfill volume shall be larger 19.5 mm in diameter, and rocks shall be well mixed with finer soil.

- Trench backfill shall be compacted manually or mechanically to achieve the density specified herein. The backfill materials shall be brought to near optimum moisture content, placed in eight-inch thick uncompacted horizontal layers. Each layer shall be evenly spread, moistened or dried as necessary and compacted until the specified density has been achieved.

- The contractor shall select the equipment and processes to be used to achieve the specified density without damage to adjacent ground, existing improvements or completed work.

- Observation and field tests should be performed by a qualified geotechnical firm to confirm that the required degree of compaction has been obtained. Where compaction is less than that specified, additional compactive effort shall be made with adjustment of the moisture content as necessary until the specified compaction is obtained.

- Wherever, in the opinion of the soils consultants or the Owner's representatives, an unstable condition is being created, the work shall not proceed in that area until an investigation has been made and the excavation plan revised, if found necessary.

- Fill material shall not be placed, spread or completed during unfavorable weather conditions. When the work is interrupted by heavy rain, fill operations shall not be resumed until field tests by a qualified geotechnical firm indicate that the moisture content and density of the fill are as previously specified.

- Whenever the words "supervision," "inspection," or "control" appear they shall mean <u>observation</u> of the work and <u>testing</u> of the fill placement as necessary by a qualified geotechnical firm for substantial compliance with plans, specifications and design concepts. The equipment and methods necessary to properly compact the soils and the safety at the work site shall be the sole responsibility of the contractor.

13.0 CONSTRUCTION RECOMMENDATIONS

<u>13.1 General</u>

Sloped trench excavations may not be feasible along the majority of the alignment due to proximity of existing utility lines, roadways or structures. Recommendations pertaining to temporary excavations and temporary shoring design are included in Section 13.2, *Temporary Excavations* and Section 13.3, *Temporary Shoring*, respectively.

13.2 Temporary Excavations

Based on the materials encountered in the borings, sloped temporary excavations may be constructed according to the slope ratios presented in Table No. 5, *Slope Ratios for Temporary Excavations*. Any loose utility trench backfill or other fill encountered in excavations will be less stable than the native soils. Temporary cuts encountering loose fill or loose dry sand may have to be constructed at a flatter gradient than presented in Table No. 5, *Slope Ratios for Temporary Excavations*.

Table No. 5. Slope Ratios for Temporary Excavation

Maximum Depth of Cut (feet)	Maximum Slope Ratio* (horizontal:vertical)
0 - 5	Vertical
5 - 12	0.5:1
12 - 25	1:1

*Slope ratio assumed to be uniform from top to toe of slope.

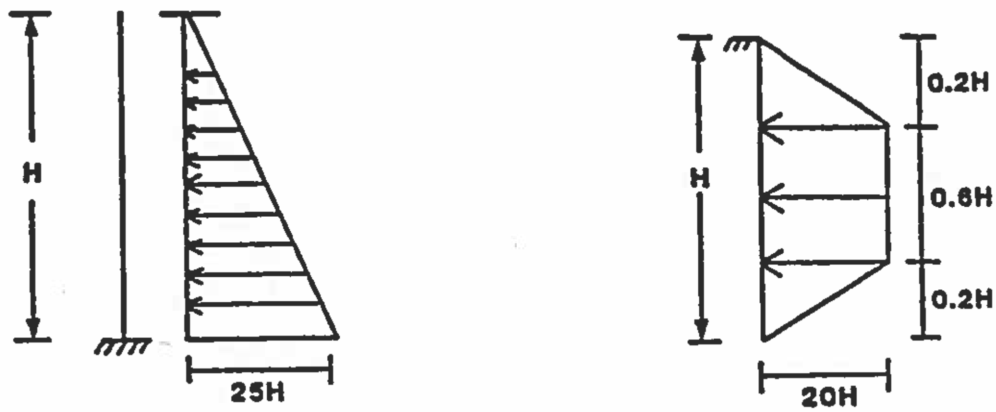
Construction of open cuts adjacent to existing roadways and/or adjacent structures is not recommended within a 1:1 (horizontal:vertical) plane extending beyond and down from the perimeter of the roadway or structure.

Surfaces exposed in slope excavations should be kept moist but not saturated to retard raveling and sloughing during construction. Adequate provisions should be made to protect the slopes from erosion during periods of rainfall. Surcharge loads, including construction, should not be placed within five feet of the trench edge. The above maximum slopes are based on a maximum height of stockpiled soils of six feet adjacent to the trench. Cuts that are proposed within five feet of light standards, other utilities or pavement should be provided with temporary shoring.

All applicable requirements of the California Construction and General Industry Safety Orders, the Occupational Safety and Health Act of 1987 and current amendments, and the Construction Safety Act should be met. The soils exposed in cuts should be observed during excavation by the project geotechnical consultant. If potentially unstable soil conditions are encountered, modifications of slope ratios for temporary cuts may be required.

13.3 Temporary Shoring

Soldier pile, sheet pile or shield shoring systems may be used to maintain temporary support of excavations. Temporary shoring should be designed using the earth pressure shown in Figure No. 8, *Recommended Lateral Earth Pressures for Temporary Retaining Walls*. The lateral earth pressure shown is based on the assumption that ground water is at least five feet below the base of the proposed excavations.



(a) Cantilevered

(b) Braced

Figure No. 8, *Recommended Lateral Earth Pressures for Temporary Retaining Walls*

Surcharge pressures should be added to the earth pressures for surcharges within a distance from the top of the shoring less than or equal to the shoring height. Surcharge coefficients of 33 percent and 50 percent of any uniform vertical surcharge should be added as a horizontal shoring pressure for cantilevered and braced shoring, respectively. In addition to the recommended lateral earth pressure, the upper 10 feet of shoring adjacent to traffic areas should be designed to resist a uniform lateral pressure of 100 psf per foot of depth, as a result of an assumed 300 psf surcharge behind the shoring due to normal traffic. If the traffic is kept back at least 10 feet from the shoring, the traffic surcharge may be neglected. Adjacent to existing structures, the shoring should also be designed for any surcharge loading imposed by the foundations of the adjacent structures. Where unusual surcharge loads due to construction equipment are anticipated, the shoring should be designed for such loads. Recommendations for design of shoring for unusual surcharge conditions can be provided when load information is available.

It is recommended that a qualified geotechnical firm review plans and specifications for proposed shoring and that a representative from the firm observe the installation of shoring.

Lateral resistance for piles may be assumed to be provided by passive pressure acting below the bottom of the excavation. The allowable passive resistance of 300 psf per foot of depth may be used under drained conditions and may be doubled for piles that are placed at center-to-center spacings equal to at least three times the pile diameter. For portion of the pile below water table, the passive resistance should be reduced 50 percent. The allowable passive resistance should not exceed 3,000 psf per foot of depth. To develop the full lateral resistance, provisions should be made to assure firm contact between the soldier piles and the undisturbed soils.

14.0 GEOTECHNICAL SERVICES DURING CONSTRUCTION

This report has been prepared to aid in the evaluation of the proposed pipeline alignment and to aid the engineer in the design of the project. It is recommended that this office be provided the opportunity to review the final design drawings, shoring design, and specifications to evaluate if the recommendations of this report have been properly implemented.

Design recommendations given in this report are based on the assumption that the pipeline will be placed on firm undisturbed native soils and/or rocks. All trench excavations and shoring installations should be observed by the project geotechnical consultant prior to placement of the pipe to verify that the pipeline will be founded on satisfactory materials and that excavations are free of loose and disturbed soils. All trench backfill should be placed and compacted during observation and testing by the project geotechnical consultant.

15.0 CLOSURE

The findings and recommendations of this report were prepared in accordance with generally accepted professional engineering and engineering geologic principles and practice at this time in this area. We make no other warranty, either express or implied. Our conclusions and recommendations are based on the results of the field investigations and laboratory tests, combined with interpolation of soil conditions beyond the boring location. If the conditions encountered during construction appear to be different from those shown by the borings, this office should be notified.

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APPENDIX A

FIELD EXPLORATION

APPENDIX A

FIELD EXPLORATION

Field exploration included a reconnaissance along the proposed pipeline alignment and a subsurface drilling program. During the site reconnaissance, the surface conditions were noted, and the locations of the test borings were determined. The borings were located using existing topography and boundary features as a guide. The borings were located in the field by representatives from CCIE and Alex Wesner from Black & Veatch. It is our understanding that the boring locations were approved by Betsy Henderson with Black & Veatch.

Elevations of the boreholes have been obtained by extrapolating from contours shown on "Reach 4 Pipelines Contract II" drawings prepared by Black & Veatch.

Nine borings were advanced using an eight-inch diameter hollow-stem auger drill rig. Soils were continuously logged by our field engineer and classified in the field by visual examination in accordance with the Unified Soil Classification System. The field descriptions have been modified, where appropriate, to reflect laboratory test results.

Relatively undisturbed and bulk samples of the subsurface soils were obtained from the borings at frequent intervals. The undisturbed samples were obtained using a thin-walled, steel sampler (2.4-inch-inside-diameter, three-inch-outside-diameter) lined with brass sample rings.

The sampler was driven into the bottom of the borehole with successive drops of a 140-pound hammer falling 30 inches. The number of successive drops of the driving weight ("blows") required for one foot of penetration of the sampler are shown on the boring summary sheets in the "blow/foot" column. The energy delivered to the rod during driving the sampler is the same as in the standard penetration test (SPT). However, the effective end area of the SPT sampler, to which this energy is delivered, is about 0.67 times that of the thin-walled steel sampler. As a result, the blow counts from the thin-walled steel sampler is multiplied by 0.67 to obtain equivalent SPT blow counts.

The soil was retained in brass rings (2.4 inches in diameter, one inch in height). The central portion of the sample was retained and carefully sealed in waterproof plastic containers for shipment to the laboratory. Bulk soil samples were collected in plastic bags.

Logs of the borings are presented on summary sheets A-1 through A-10. Boring BC-1 was drilled as part of the investigation for the "*Reach 4 Pipelines, Contract I*" but located at approximate Station 566+00, which is about 400 feet east of the western end of the Segment II of the "*Reach 4 Pipelines, Contract II.*" We have decided to include the information derived from this boring in this report. The boring log is included as Drawing No. A-6.

The boring summary sheets include descriptions of the soils, pertinent field data, and supplementary laboratory results. A key to soil symbols and terminology is presented in Drawing No. A-11.

An estimate of the relative density of sandy soils can be estimated based on the equivalent SPT blow counts, obtained as explained above, in conjunction with Table No. A-1, *Standard Penetration Test.*

Table No. A-1. Standard Penetration Test

Penetration Resistance N (Blows/305 mm)	Relative Density
0-4	Very Loose
4-10	Loose
10-30	Medium
30-50	Dense
>50	Very Dense

information derived from this boring in this report. The boring log is included as Drawing No. A-6.

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Penetration Resistance N (Blows/305 mm)	Relative Density
0-4	Very Loose
4-10	Loose
10-30	Medium
30-50	Dense
>50	Very Dense

Log of Boring No. BF- 1

Dates Drilled: 9/16/93 - | Logged by: WNP | Checked by: MSI

Equipment: 8" HSA | Driving Weight and Drop: 140 lb / 30 in

Ground Surface Elevation(ft): 1405 | Depth to Water(ft): 19 Feet

SUMMARY OF SUBSURFACE CONDITIONS

This log is part of the report prepared by Converse for this project and should be read together with the report. This summary applies only at the location of the boring and at the time of drilling. Subsurface conditions may differ at other locations and may change at this location with the passage of time. The data presented is a simplification of actual conditions encountered.

DEPTH (ft)	SUMMARY OF SUBSURFACE CONDITIONS	BLOWS/FOOT	MOISTURE (%)	DRY UNIT WT. (pcf)	LABORATORY TESTS
	FILL: SILTY SAND (SM): fine grained, yellowish-brown, loose.	11	10	106	
	ALLUVIUM (Qal): SILTY SAND (SM): fine to medium grained, with clay, trace of caliche, brown, loose.				
5		29	18	90	ds
	CLAYEY SAND (SC): fine to medium grained, some gravel, brown, medium dense.				ma,se, max
	SANDY SILT (ML): some clay, trace gravel, reddish, firm.				
10		25	17	112	c
	CLAYEY SAND (SC): fine to coarse grained, yellowish-gray, medium dense.				
15		22	16	114	ct
	∇ BEDROCK: GRANITE: moderately to highly weathered, light-to dark-gray, medium dense.				
20		39	18	107	
	- highly weathered, silty clay zone at 24 feet	24	30	93	
25	End of boring at 25 feet. Boring backfilled.				

Converse Consultants Inland Empire

Project Name. REACH 4, RWBWTP, CONTRACT II, EMWD

Project No. 92-81-504-01

Drawing No. A-1

Log of Boring No. BF- 2

Dates Drilled: 9/16/93 - Logged by: WNP Checked by: MSI

Equipment: 8" HSA Driving Weight and Drop: 140 lb / 30 in

Ground Surface Elevation(ft): 1400 Depth to Water(ft): 12 Feet

DEPTH (ft)	GRAPHIC LOG	SUMMARY OF SUBSURFACE CONDITIONS This log is part of the report prepared by Converse for this project and should be read together with the report. This summary applies only at the location of the boring and at the time of drilling. Subsurface conditions may differ at other locations and may change at this location with the passage of time. The data presented is a simplification of actual conditions encountered.	SAMPLES: DRIVE	SAMPLES: BULK	BLOWS/FOOT	MOISTURE (%)	DRY UNIT WT. (pcf)	LABORATORY TESTS
		FILL: SILTY SAND (SM): fine grained, brown, loose.						
		ALLUVIUM (Qal): SILTY SAND (SM): fine grained, trace caliche, light brown, trace clay, caliche and gravel, medium dense.						
5					31	11		
		- fine to coarse grained with clay, below 7', yellowish-brown						
10		BEDROCK: GRANITE: highly weathered, yellowish-brown, medium dense.			26	19	106	
15					29	16	110	
20					34	17	112	
25					33	16	108	
		End of boring at 25 feet. Boring backfilled.						

Log of Boring No. BT- 1

Dates Drilled: 9/16/93 - | Logged by: WNP | Checked by: MSI

Equipment: 8" HSA | Driving Weight and Drop: 140 lb / 30 in

Ground Surface Elevation(ft): 1403 | Depth to Water(ft): 16 Feet

SUMMARY OF SUBSURFACE CONDITIONS

This log is part of the report prepared by Converse for this project and should be read together with the report. This summary applies only at the location of the boring and at the time of drilling. Subsurface conditions may differ at other locations and may change at this location with the passage of time. The data presented is a simplification of actual conditions encountered.

DEPTH (ft)	SUMMARY OF SUBSURFACE CONDITIONS	SAMPLES: DRIVE	SAMPLES: BULK	BLOWS/FOOT	MOISTURE (%)	DRY UNIT WT. (pcf)	LABORATORY TESTS
0	ALLUVIUM (Qal): SILTY SAND (SM): fine to coarse grained, trace gravel, dark brown, loose to medium dense.						
1.5		X		12	10	88	
3–8			X				
5		X		13	22	104	ma
8	BEDROCK: GRANITE: weathered, white-brown, medium dense.						
10		X		29	18	106	c,ds
10–15			X				
15		X		27	16	105	
16	▽ (water)						
18	- highly weathered, below 18', silty clay, olive-gray						
20		X		23	31	90	
25		X		19	31	88	
25	End of boring at 25 feet. Boring backfilled.						

Converse Consultants Inland Empire

Project Name. REACH 4, RWBWTP, CONTRACT II, EMWD

Project No. 92-81-504-01

Drawing No. A-3

Log of Boring No. BT- 2

Dates Drilled: 9/16/93 - Logged by: WNP Checked by: MSI

Equipment: 8" HSA Driving Weight and Drop: 140 lb / 30 in

Ground Surface Elevation(ft): 1418 Depth to Water(ft): 22 Feet

DEPTH (ft)	GRAPHIC LOG	SUMMARY OF SUBSURFACE CONDITIONS This log is part of the report prepared by Converse for this project and should be read together with the report. This summary applies only at the location of the boring and at the time of drilling. Subsurface conditions may differ at other locations and may change at this location with the passage of time. The data presented is a simplification of actual conditions encountered.	SAMPLES DRIVE	SAMPLES BULK	BLOWS/FOOT	MOISTURE (%)	DRY UNIT WT. (pcf)	LABORATORY TESTS
		ALLUVIUM (Qal): SILTY SAND (SM): fine to coarse grained, reddish-brown, loose to medium dense.						
		BEDROCK: GRANITE: moderately to highly weathered, yellowish-brown, medium dense.						
5			■		25	15	109	
10			■		38	13	111	
15			■	■	27	18	106	
20			■	■	35	15	112	
		∇						
		End of boring at 24 feet. Boring backfilled.						

Converse Consultants Inland Empire

Project Name. REACH 4, RWBWTP, CONTRACT II, EMWD

Project No. 92-81-504-01

Drawing No. A-4

Log of Boring No. BT- 3

Dates Drilled: 9/16/93 -
Logged by: WNP
Checked by: MSI
Equipment: 8" HSA
Driving Weight and Drop: 140 lb / 30 in
Ground Surface Elevation(ft): 1420
Depth to Water(ft): 26 Feet

DEPTH (ft)	SUMMARY OF SUBSURFACE CONDITIONS	DRIVE	BULK	BLOWS/FOOT	MOISTURE (%)	DRY UNIT WT. (pcf)	LABORATORY TESTS
	FILL: SILTY SAND (SM): fine to medium grained, trace clay, brown, medium dense.	■	▒	17	11	112	
5	ALLUVIUM (Qal): SAND (SP): fine to medium grained, trace silt, yellowish-brown, medium dense.	■	▒	30	15	107	
10	BEDROCK: GRANITE: moderately to highly weathered, white-gray, medium dense.	■		36	13	107	
15		■	▒	23	20	106	
20		■	▒	32	15	104	
25	▽	■	▒	25	20	106	
30		■		46	15	112	
	End of boring at 30 feet. Boring backfilled.						

This log is part of the report prepared by Converse for this project and should be read together with the report. This summary applies only at the location of the boring and at the time of drilling. Subsurface conditions may differ at other locations and may change at this location with the passage of time. The data presented is a simplification of actual conditions encountered.

Converse Consultants Inland Empire

Project Name. REACH 4, RWBWTP, CONTRACT II, EMWD

Project No. 92-81-504-01

Drawing No. A-5

Log of Boring No. BC- 1

Dates Drilled: 9/1/93 -　　Logged by: WNP　　Checked by: WNP

Equipment: 8" HSA　　Driving Weight and Drop: 140 lb / 30 in

Ground Surface Elevation(ft): 1398　　Depth to Water(ft): None Encountered

DEPTH (ft)	GRAPHIC LOG	SUMMARY OF SUBSURFACE CONDITIONS This log is part of the report prepared by Converse for this project and should be read together with the report. This summary applies only at the location of the boring and at the time of drilling. Subsurface conditions may differ at other locations and may change at this location with the passage of time. The data presented is a simplification of actual conditions encountered.	SAMPLES DRIVE	SAMPLES BULK	BLOWS/FOOT	MOISTURE (%)	DRY UNIT WT. (pcf)	LABORATORY TESTS
		ALLUVIUM (Qal): SILTY SAND (SM): fine-to coarse-grained, some gravel, trace clay, brown, loose to medium dense.			7	5	101	ma,r
5					7	10	105	
								max se
10					24	12	109	c,ds
		BEDROCK: weathered metamorphic rock, possible metasandstone, white, medium dense.						
15					28	18	105	
		End of boring at 15 feet. Boring backfilled.						

Converse Consultants Inland Empire

Project Name. REACH 4, RWBWTP, CONTRACT II, EMWD

Project No. 92-81-504-01

Drawing No. A-6

Log of Boring No. BE- 1

Dates Drilled: 9/2/93 - | Logged by: WNP | Checked by: MSI

Equipment: 8" HSA | Driving Weight and Drop: 140 lb / 30 in

Ground Surface Elevation(ft): 1422 | Depth to Water(ft): None Encountered

DEPTH (ft)	GRAPHIC LOG	SUMMARY OF SUBSURFACE CONDITIONS This log is part of the report prepared by Converse for this project and should be read together with the report. This summary applies only at the location of the boring and at the time of drilling. Subsurface conditions may differ at other locations and may change at this location with the passage of time. The data presented is a simplification of actual conditions encountered.	SAMPLES DRIVE	SAMPLES BULK	BLOWS/FOOT	MOISTURE (%)	DRY UNIT WT. (pcf)	LABORATORY TESTS
		FILL: SILTY SAND (SM): fine to coarse grained, brown, loose.			11	14	112	
		ALLUVIUM (Qal): SILTY SAND (SM): fine to coarse grained, some gravel, trace clay, dark brown, medium dense to dense.						
5					12	10	117	
10					51	10	106	ct se
		BEDROCK: GRANITE: weathered, grayish-brown, dense to very dense.						
15					41	10	114	
		- hole caved in at 17 feet grayish-black below 18 feet						
20					170	9	116	
		End of boring at 30 feet due to refusal. Boring backfilled.						

Converse Consultants
Inland Empire

Project Name.
REACH 4, RWBWTP, CONTRACT II, EMWD

Project No.
92-81-504-01

Drawing No.
A-7

Log of Boring No. BE- 2

Dates Drilled: 9/2/93 - Logged by: WNP Checked by: MSI

Equipment: 8" HSA Driving Weight and Drop: 140 lb / 30 in

Ground Surface Elevation(ft): 1445 Depth to Water(ft): None Encountered

DEPTH (ft)	GRAPHIC LOG	SUMMARY OF SUBSURFACE CONDITIONS This log is part of the report prepared by Converse for this project and should be read together with the report. This summary applies only at the location of the boring and at the time of drilling. Subsurface conditions may differ at other locations and may change at this location with the passage of time. The data presented is a simplification of actual conditions encountered.	SAMPLES DRIVE	SAMPLES BULK	BLOWS/FOOT	MOISTURE (%)	DRY UNIT WT. (pcf)	LABORATORY TESTS
		FILL: SILTY SAND (SM): fine to coarse grained, brown, loose.			49	6	119	
		BEDROCK: GRANITE: weathered, grayish-brown, dense to very dense.						
5					32			
10					85/3"			N/R
15					90/4"			N/R
		End of boring at 15 feet, due to refusal. Boring backfilled.						

Converse Consultants
Inland Empire

Project Name.
REACH 4, RWBWTP, CONTRACT II, EMWD

Project No.
92-81-504-01

Drawing No.
A-8

Log of Boring No. BE- 3

Dates Drilled: 9/2/93 - Logged by: WNP Checked by: MSI

Equipment: 8" HSA Driving Weight and Drop: 140 lb / 30 in

Ground Surface Elevation(ft): 1313 Depth to Water(ft): None Encountered

DEPTH (ft)	GRAPHIC LOG	SUMMARY OF SUBSURFACE CONDITIONS This log is part of the report prepared by Converse for this project and should be read together with the report. This summary applies only at the location of the boring and at the time of drilling. Subsurface conditions may differ at other locations and may change at this location with the passage of time. The data presented is a simplification of actual conditions encountered.	SAMPLES DRIVE	SAMPLES BULK	BLOWS/FOOT	MOISTURE (%)	DRY UNIT WT. (pcf)	LABORATORY TESTS
		FILL: SILTY SAND (SM): fine to coarse grained, brown, medium dense.			21	7	117	
		BEDROCK: GRANITE: weathered, medium grained, grayish-brown, dense to very dense.						
5					48	6	118	
		- black below 7'						ma,max se
10					141	9	122	ds
		End of boring at 10 feet, due to refusal. Boring backfilled.						

Converse Consultants Inland Empire

Project Name. REACH 4, RWBWTP, CONTRACT II, EMWD

Project No. 92-81-504-01

Drawing No. A-9

Log of Boring No. BE- 4

Dates Drilled: 9/2/93 -
Logged by: WNP
Checked by: MSI

Equipment: 8" HSA
Driving Weight and Drop: 140 lb / 30 in

Ground Surface Elevation(ft): 1303
Depth to Water(ft): None Encountered

DEPTH (ft)	GRAPHIC LOG	SUMMARY OF SUBSURFACE CONDITIONS This log is part of the report prepared by Converse for this project and should be read together with the report. This summary applies only at the location of the boring and at the time of drilling. Subsurface conditions may differ at other locations and may change at this location with the passage of time. The data presented is a simplification of actual conditions encountered.	SAMPLES DRIVE	SAMPLES BULK	BLOWS/FOOT	MOISTURE (%)	DRY UNIT WT. (pcf)	LABORATORY TESTS
		<u>FILL:</u> SILTY SAND (SM): fine to coarse grained, light brown, medium dense.			27	6	111	
5		<u>BEDROCK:</u> GRANITE: weathered, brown, very dense.			50/0.5"			N/R
		End of boring at 6 feet, due to refusal. Boring backfilled.						

UNIFIED SOIL CLASSIFICATION

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

BORING LOG SYMBOLS

SAMPLE TYPE

STANDARD PENETRATION TEST, Split barrel sampler in accordance with ASTM D1587-84 Standard Test Method

DRIVE SAMPLE, 2.42" ID sampler, driven with 140 lb. weight, 30 in. drop

DRIVE SAMPLE, No Recovery

DISTURBED BULK SAMPLE

TEST TYPE

(Results shown in Appendix B)

TEST TYPE	OTHER
SAMPLE DISTURBED	—
CLASSIFICATION	
Plasticity	pl
Grain Size Analysis	ma
Sand Equivalent	se
Specific Gravity	sg
Expansion Index	ei
Compaction Curve	max
STRENGTH	
Pocket Penetrometer	p
Direct Shear	ds
Unconfined Compression	uc
Triaxial Compression	tx
Vane Shear	vs
CONSOLIDATION	c
COLLAPSE TEST	ct
RESISTANCE (R) VALUE	r
CHEMICAL ANALYSIS	ca
ELECTRICAL RESISTIVITY	er

UNIFIED SOIL CLASSIFICATION AND KEY TO BORING LOG SYMBOLS

REACH 4 PUMP STATION & FACILITIES
Sun City, California
for: Black & Veatch

Project No. 92-81-504-01

Converse Consultants Inland Empire

Drawing No. A-11

APPENDIX B

LABORATORY TESTING PROGRAM

APPENDIX B

LABORATORY TESTING PROGRAM

Tests were conducted in our laboratory on representative soil samples for the purpose of classification and evaluation of their physical properties and engineering characteristics. The amount and selection of tests were based on the geotechnical parameters of the project. Test results are presented herein and on the boring logs (Appendix A, *Field Exploration*). A summary of the various laboratory tests conducted for this project is provided below.

Moisture Content and Dry Density

Results of these tests, performed on relatively undisturbed samples, were used to aid in the classification and correlation of the soils and to provide qualitative information regarding soil strength and compressibility. Test results are included on the boring logs (Appendix A, *Field Exploration*).

Sieve Analysis

To aid in the classification of the soils, sieve analyses were performed on selected soil samples. The analyses were performed in accordance with the ASTM Standard D422-63 Standard Test Method. The grain-size distributions are provided in Figure No. B-1, *Grain-Size Distribution*.

Laboratory Maximum Density Tests

Laboratory dry density and moisture content relationship tests were performed on representative bulk samples. The test was conducted in accordance with ASTM Standard Method D1557-91. The test results are included in Figure No. B-2, *Compaction Test*.

Direct Shear Tests

Direct shear tests were performed on representative undisturbed samples under drained conditions. Three samples contained in brass sampler rings were placed, one at a time, directly into the test apparatus and subjected to a range of normal loads appropriate for the anticipated conditions. Each sample was then sheared at a constant strain rate. Shear deformation was recorded until a maximum of about one-half inch shear displacement was achieved. Either peak or residual strength can be selected from the shear-stress deformation data and plotted to determine the shear strength parameters.

Test data, including sample density and moisture content, are provided in Figure No. B-3 through B-6, *Direct Shear Test*. One of the tests, included as Figure No. B-3, *Direct*

Shear Test, was saturated before shearing. The remaining tests were performed under field moisture conditions.

Consolidation Tests

Data obtained from this test, performed on relatively undisturbed soil samples, was used to evaluate the settlement characteristics of the on-site soils under applied loads. Preparation for this test involved trimming the sample and placing it in a one-inch high brass ring and loading it into the test apparatus, which contained porous stones to accommodate drainage during testing. Normal axial loads were applied to one end of the sample through the porous stones, and the resulting deflections were recorded at various time periods. The load was increased after the sample reached a reasonable state of equilibrium. Normal loads were applied at a constant load-increment ratio, successive loads being generally twice the preceding load.

Samples were tested at field moisture and submerged conditions. Data corresponding to field moisture conditions are indicated by an open circle; data corresponding to submerged conditions are designated by solid circles. Test results, including sample density and moisture content, are provided in Figure No. B-7 though B-9, *Consolidation Test*.

Collapse Test

To evaluate the moisture sensitivity (collapse) potential of the encountered soils, representative ring samples were loaded up to 1.0 ksf, allowed to stabilize under load, and then submerged. For test results, see Table No. B-1, *Collapse Potential Test Results*.

Table No. B-1. Collapse Potential Test Results

Boring No./Sample No.	Soil Classification	Percent Collapse
BF-1/S4 @14-15	Clayey Sand (SC)	-0.41 (Swell)
BE-1/S3 @ 9-10	Silty Sands (SM)	4.2

Sand Equivalent (SE) Tests

Representative soil samples were tested for Sand Equivalent (SE) value to evaluate the suitability of the on-site soil for placement as bedding and/or backfill material. Also, sand equivalent values can be used to evaluate the suitability of the on-site soils for trench backfill by flooding or jetting methods:

Boring No.	Depth (feet)	Soil Description	Sand Equivalent
BF-1	6-10	Sandy Silts (ML) with Clay	8
BC-1	7-12	Clayey Sand (SC) with Silts	15
BE-1	8-13	Silty Sand (SM), Trace Clay	15
BE-3	7-10	Granite: weathered, disintegrated, medium grained	27

R-Value Tests

Representative bulk samples from the upper five feet of the two borings were tested for resistance (R-value) in accordance with the State of California Standard Method 301. The test is designed to provide a relative measure of soil strength for use in the pavement design.

Boring No.	Depth (feet)	Soil Description	R-Value
BC-1	0-5	Silty Sand (SM), Trace Clay, Brown	27
BE-4	0-5	Silty Sand (SM), disintegrated weathered granite	67

Soil Corrosivity

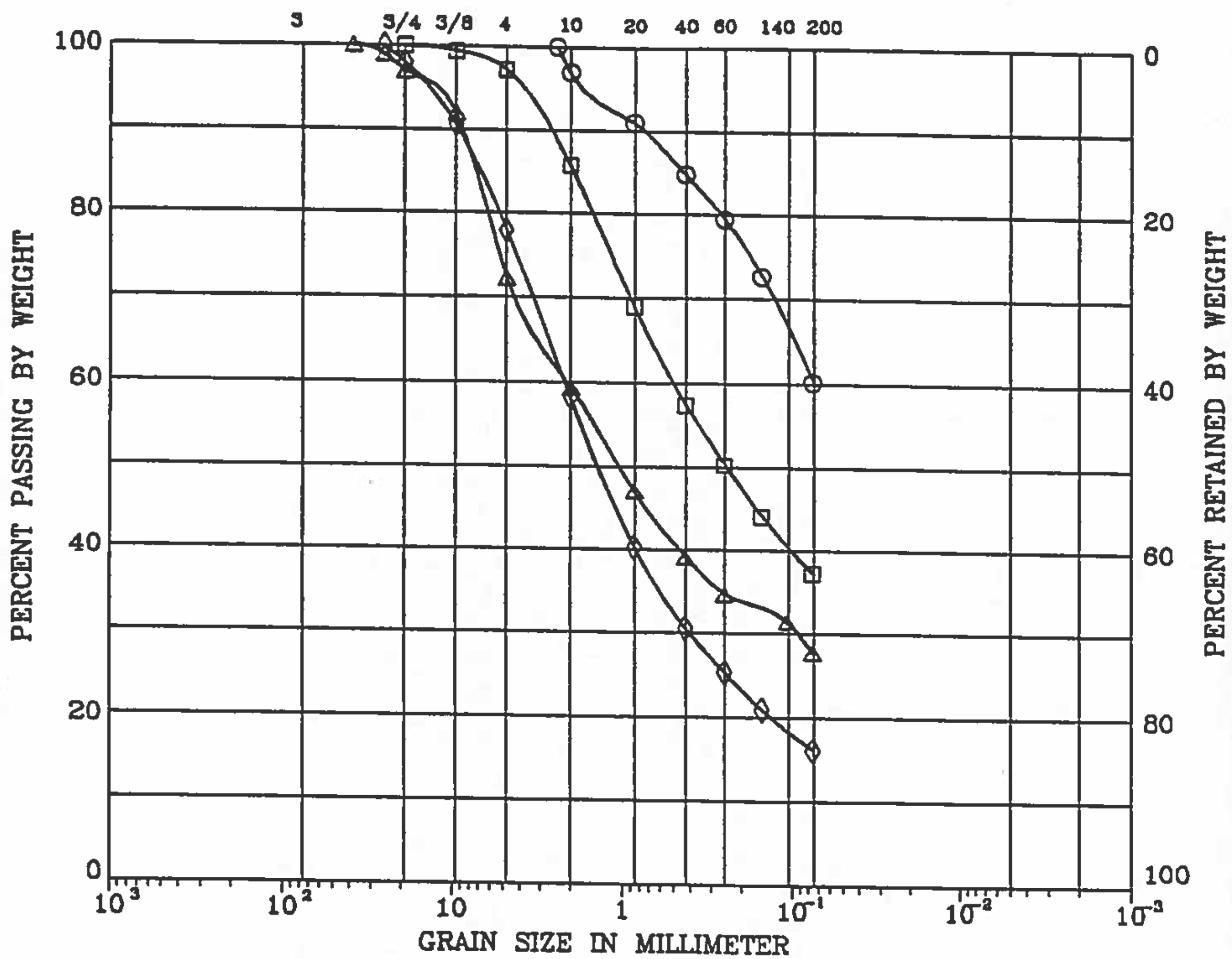
Field and laboratory tests were conducted to evaluate the corrosion potential of the pipe material in contact with soil. These tests were performed by M.J. Schiff & Associates, and the test results are enclosed in Appendix D, *Soil Corrosivity Study*.

Sample Storage

Soil samples presently stored in our laboratory will be discarded 30 days after the date of this report unless this office receives a specific request to retain the samples for a longer period.

UNIFIED SOIL CLASSIFICATION

COBBLES	GRAVEL		SAND			SILT OR CLAY
	COARSE	FINE	COARSE	MEDIUM	FINE	
U.S. SIEVE SIZE IN INCHES			U.S. STANDARD SIEVE No.			HYDROMETER



SYMBOL	BORING	DEPTH (ft)	LL (%)	PI (%)	DESCRIPTION
○	BF1/Blk2	6–10			Sandy Silt(ML)
□	BT1/Blk1	3–8			Silty Sand (SM)
△	BC1/Blk1	0–5			Silty Sand (SM)
◇	BE3/Blk1	7–10			Granite, Disintegrated

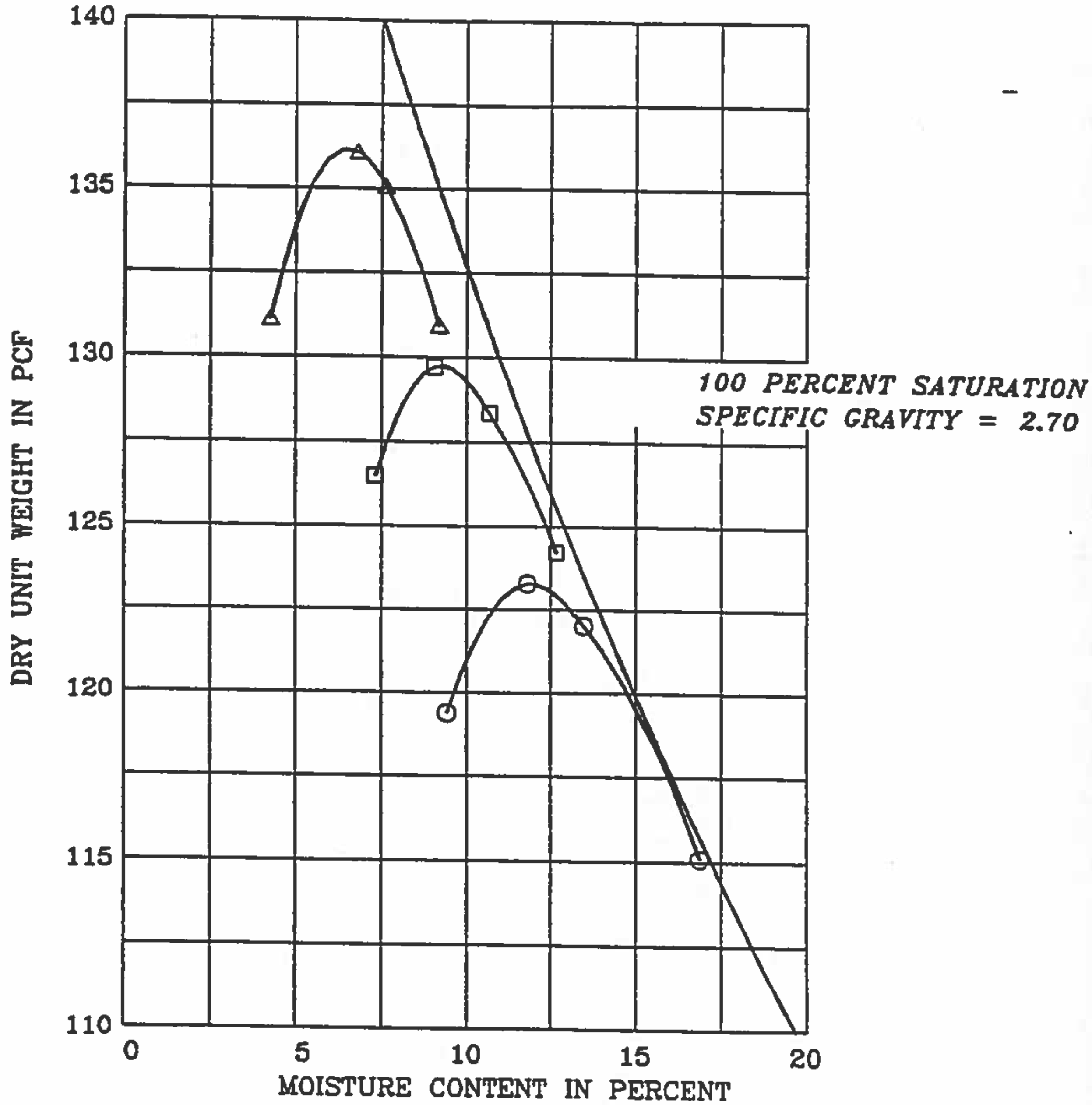
GRAIN SIZE DISTRIBUTION

Reach 4, RWBWTP, Contract II, EMWD
For: Black and Veatch

Project No.
92–81–504–01

Converse Consultants Inland Empire

Figure No. B–1



SYMBOL	SAMPLE LOCATION	DEPTH (ft)	DESCRIPTION	TEST METHOD	OPTIMUM MOISTURE (%)	MAXIMUM DRY DENSITY (pcf)
○	BF-1/Blk2	6-10	Sandy Silt(ML)	D1557-91	11.8	123.3
□	BC-1/Blk2	7-12	Silty Sand, w/some gravel	D1557-91	9.3	129.7
△	BE-3/Blk1	7-10	Silty Sands (SM)	D1557-91	6.4	136.1

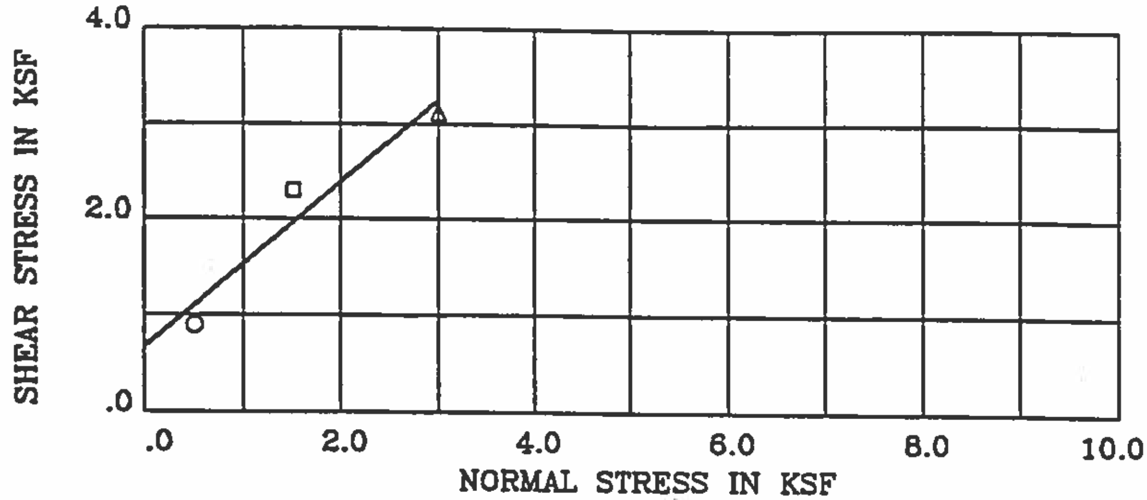
COMPACTION TEST

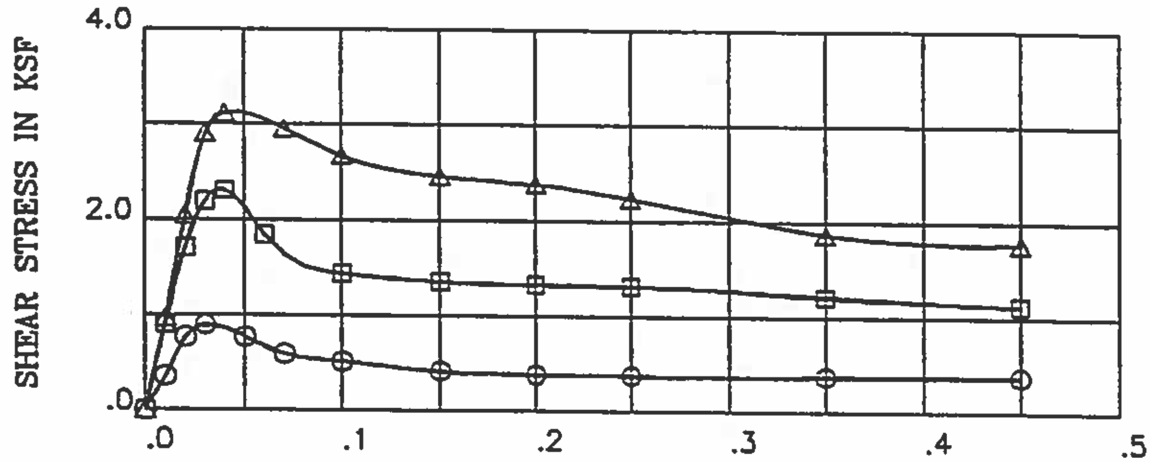
Reach 4, RWBWTP, Contract II, EMWD
For: Black and Veatch

Project No.
92-81-504-01

Converse Consultants Inland Empire

Figure No. B-2





BORING/SAMPLE : BF-1/S2 DEPTH (ft) : 4-5

DESCRIPTION : Silty Sand(SM),with clay saturated

STRENGTH INTERCEPT (ksf) : .675 (PEAK STRENGTH)

FRICTION ANGLE (degree) : 40.7 (PEAK STRENGTH)

SYMBOL	MOISTURE CONTENT (%)	DRY DENSITY (pcf)	VOID RATIO	NORMAL STRESS (ksf)	PEAK SHEAR (ksf)	RESIDUAL SHEAR (ksf)
○	19.9	110.5	.525	.50	.90	.38
□	20.3	107.7	.565	1.50	2.30	1.13
△	20.3	107.7	.565	3.00	3.12	1.77

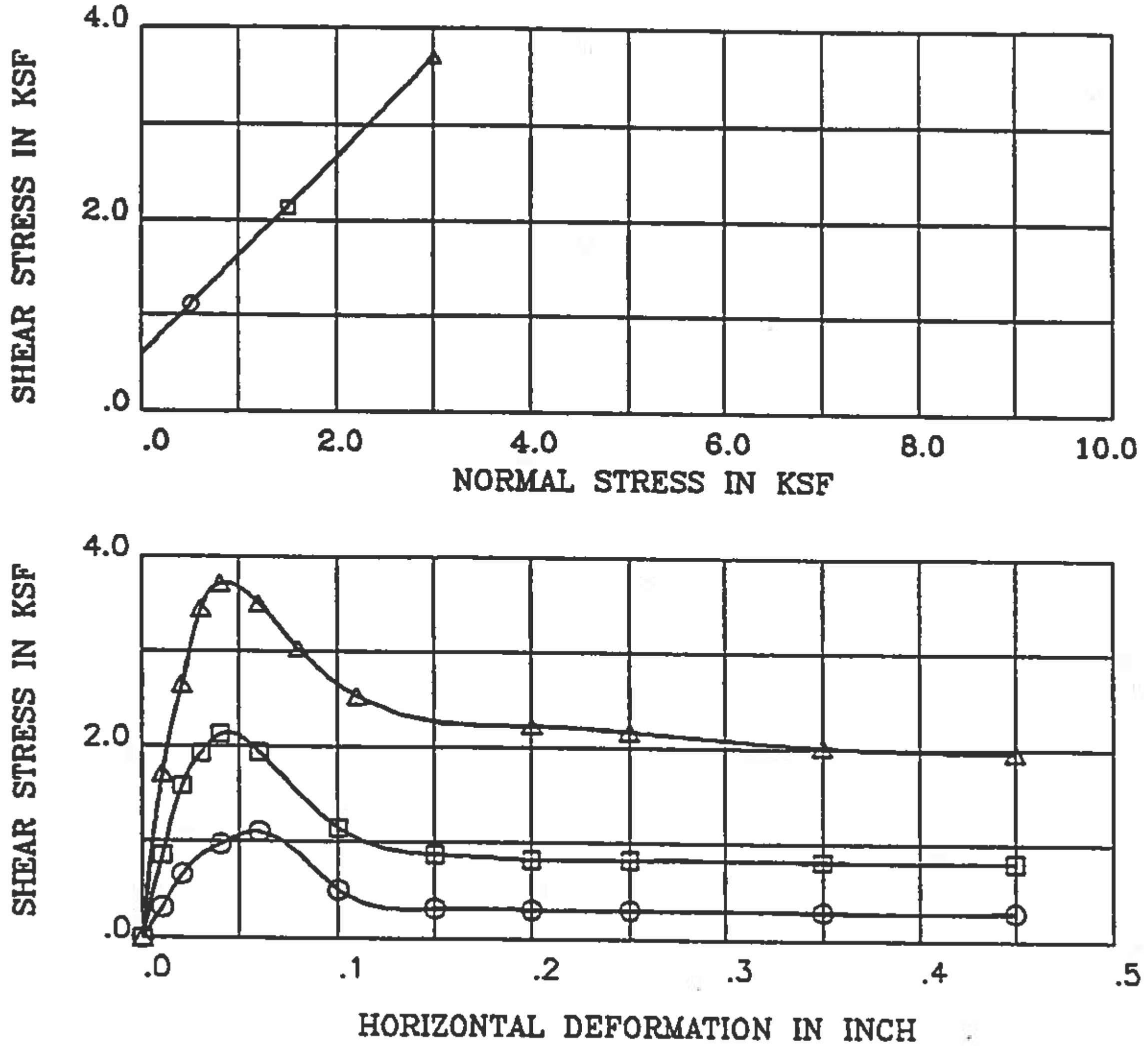
DIRECT SHEAR TEST

Reach 4, RWBWTP, Contract II, EMWD
For: Black and Veatch

Project No.
92-81-504-01

Converse Consultants Inland Empire

Figure No. B-3



BORING/SAMPLE : BT-1/S3 DEPTH (ft) : 9-10
DESCRIPTION : Weathered Granite, field moisture
STRENGTH INTERCEPT (ksf) : .583 (PEAK STRENGTH)
FRICTION ANGLE (degree) : 46.2 (PEAK STRENGTH)

SYMBOL	MOISTURE CONTENT (%)	DRY DENSITY (pcf)	VOID RATIO	NORMAL STRESS (ksf)	PEAK SHEAR (ksf)	RESIDUAL SHEAR (ksf)
O	17.9	105.5	.597	.50	1.11	.30
□	17.6	107.4	.568	1.50	2.14	.81
△	17.8	108.5	.553	3.00	3.71	1.96

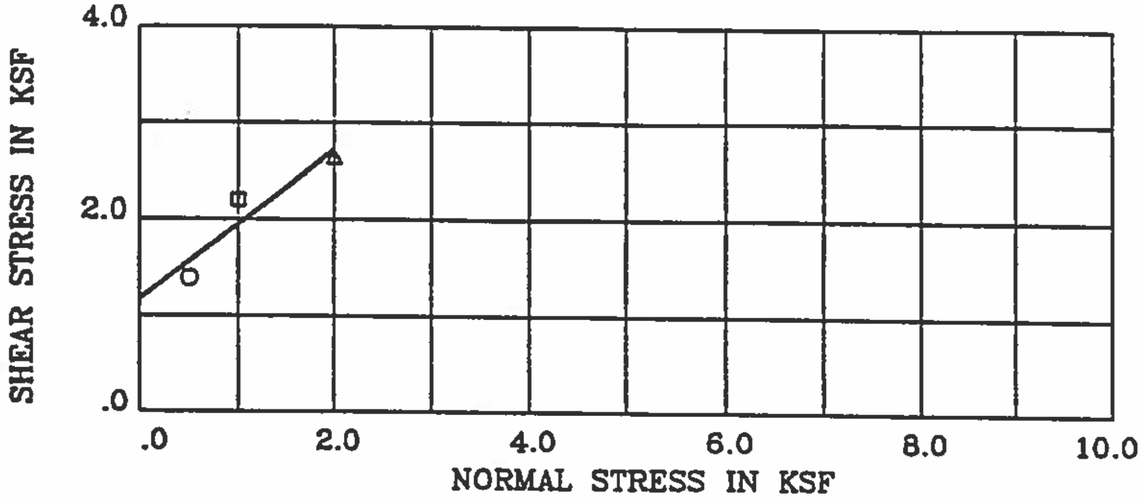
DIRECT SHEAR TEST

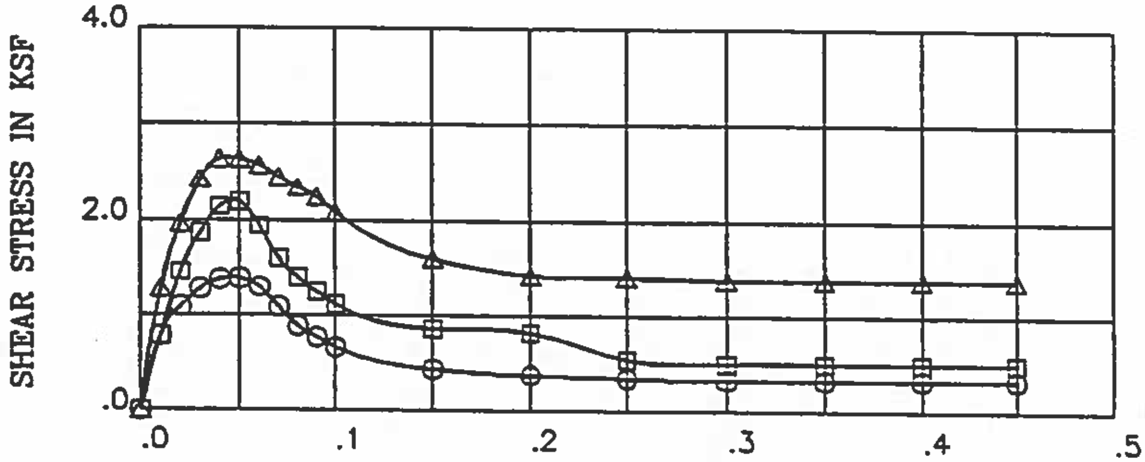
Reach 4, RWBWTP, Contract II, EMWD
For: EMWD

Project No.
92-81-504-01

Converse Consultants Inland Empire

Figure No. B-4





BORING/SAMPLE : BC-1/S3 DEPTH (ft) : 9-10

DESCRIPTION : Silty Sands (SM), tr clay field moisture

STRENGTH INTERCEPT (ksf) : 1.176 (PEAK STRENGTH)

FRICTION ANGLE (degree) : 37.9 (PEAK STRENGTH)

SYMBOL	MOISTURE CONTENT (%)	DRY DENSITY (pcf)	VOID RATIO	NORMAL STRESS (ksf)	PEAK SHEAR (ksf)	RESIDUAL SHEAR (ksf)
○	17.4	106.8	.577	.50	1.40	.33
□	14.8	112.8	.494	1.00	2.21	.50
△	16.5	110.2	.529	2.00	2.65	1.39

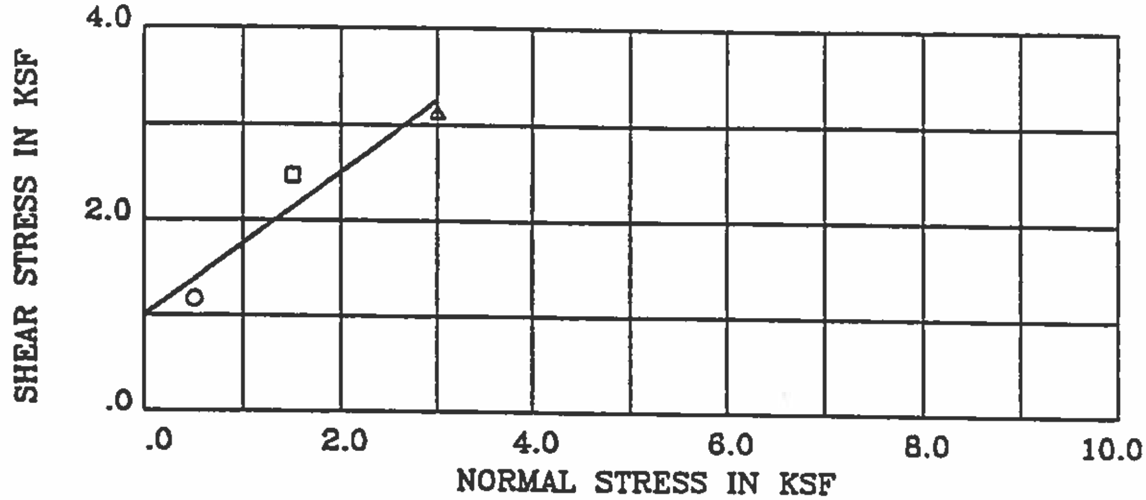
DIRECT SHEAR TEST

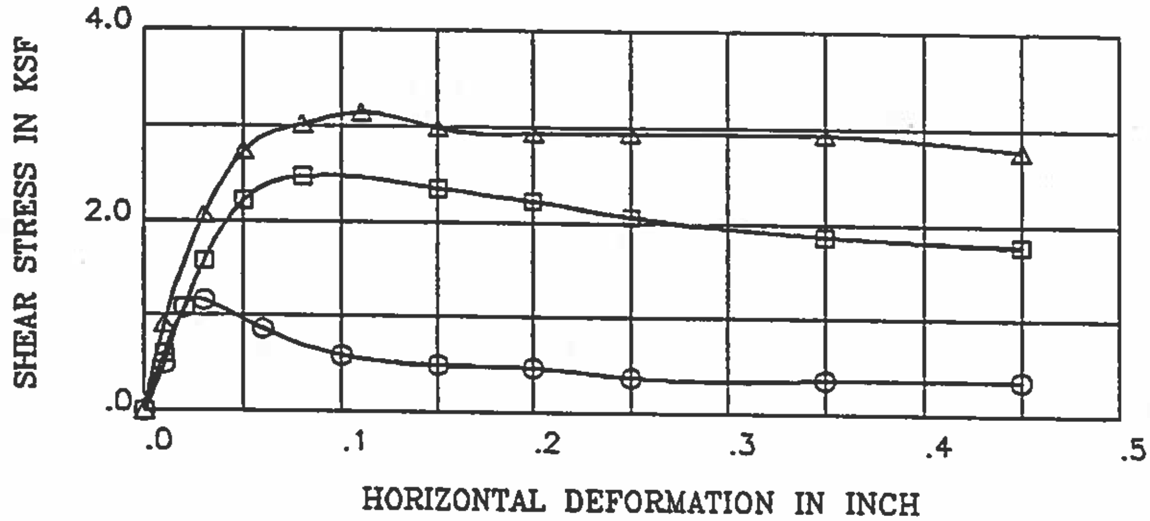
Reach 4, RWBWTP, Contract II, EMWD
For: Black and Veatch

Project No.
92-81-504-01

Converse Consultants Inland Empire

Figure No. B-5





BORING/SAMPLE : BE-3/S3 DEPTH (ft) : 9-10
DESCRIPTION : Weathered Granite, field moisture
STRENGTH INTERCEPT (ksf) : 1.000 (PEAK STRENGTH)
FRICTION ANGLE (degree) : 37.2 (PEAK STRENGTH)

SYMBOL	MOISTURE CONTENT (%)	DRY DENSITY (pcf)	VOID RATIO	NORMAL STRESS (ksf)	PEAK SHEAR (ksf)	RESIDUAL SHEAR (ksf)
○	8.6	119.8	.407	.50	1.18	.34
□	8.2	112.5	.498	1.50	2.48	1.78
△	8.5	122.4	.376	3.00	3.14	2.77

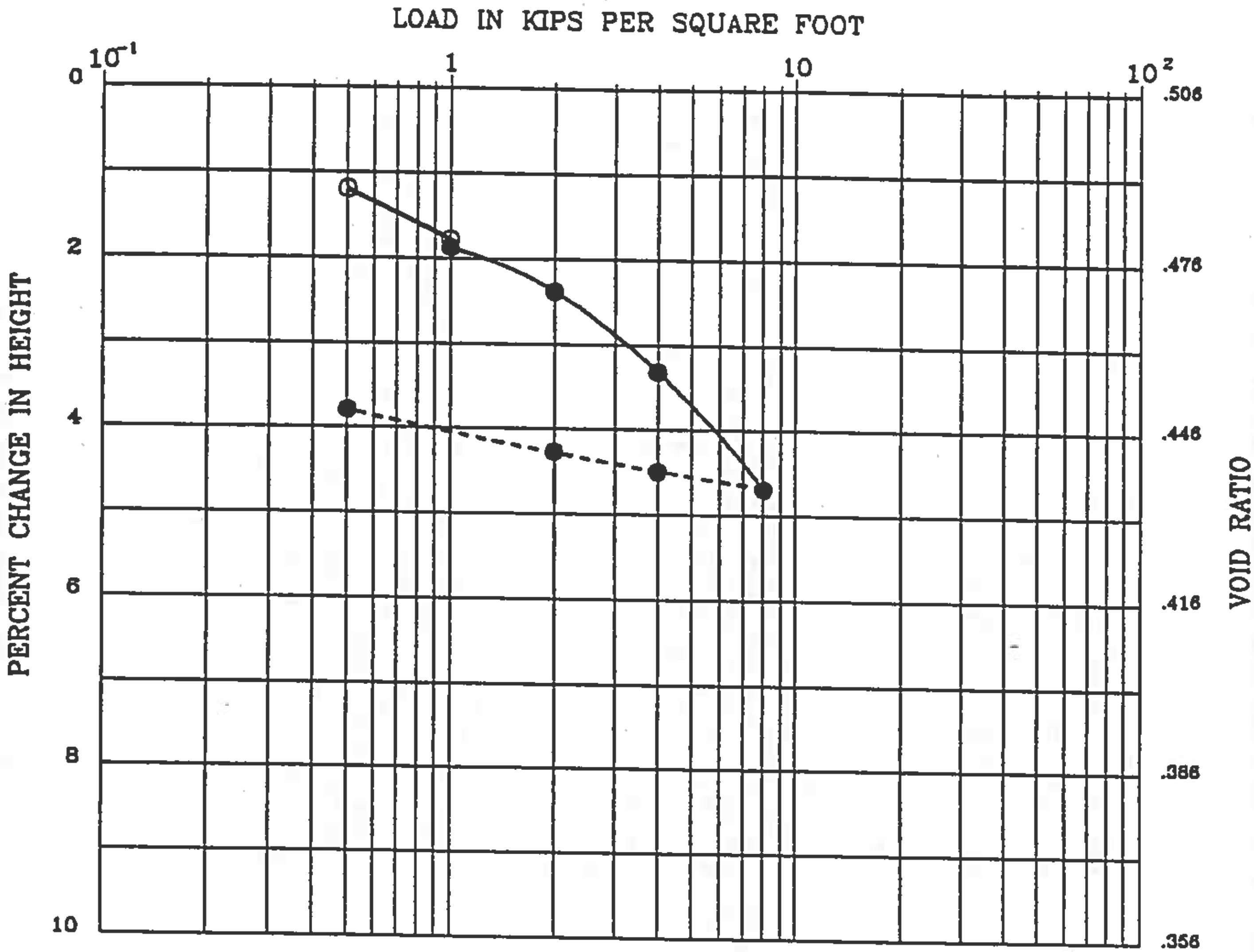
DIRECT SHEAR TEST

Reach 4, RWBWTP, Contract II, EMWD
For: Black and Veatch

Project No.
92-81-504-01

Converse Consultants Inland Empire

Figure No. B-6



BORING : BF-1/S3
DEPTH (ft) : 9-10

DESCRIPTION : Sandy Silt (ML), tr clay

	MOISTURE CONTENT (%)	DRY DENSITY (pcf)	PERCENT SATURATION	VOID RATIO
INITIAL	15.6	112.6	84	.506
FINAL	16.5	117.1	100	.449

Note: Solid circles indicate readings after addition of water

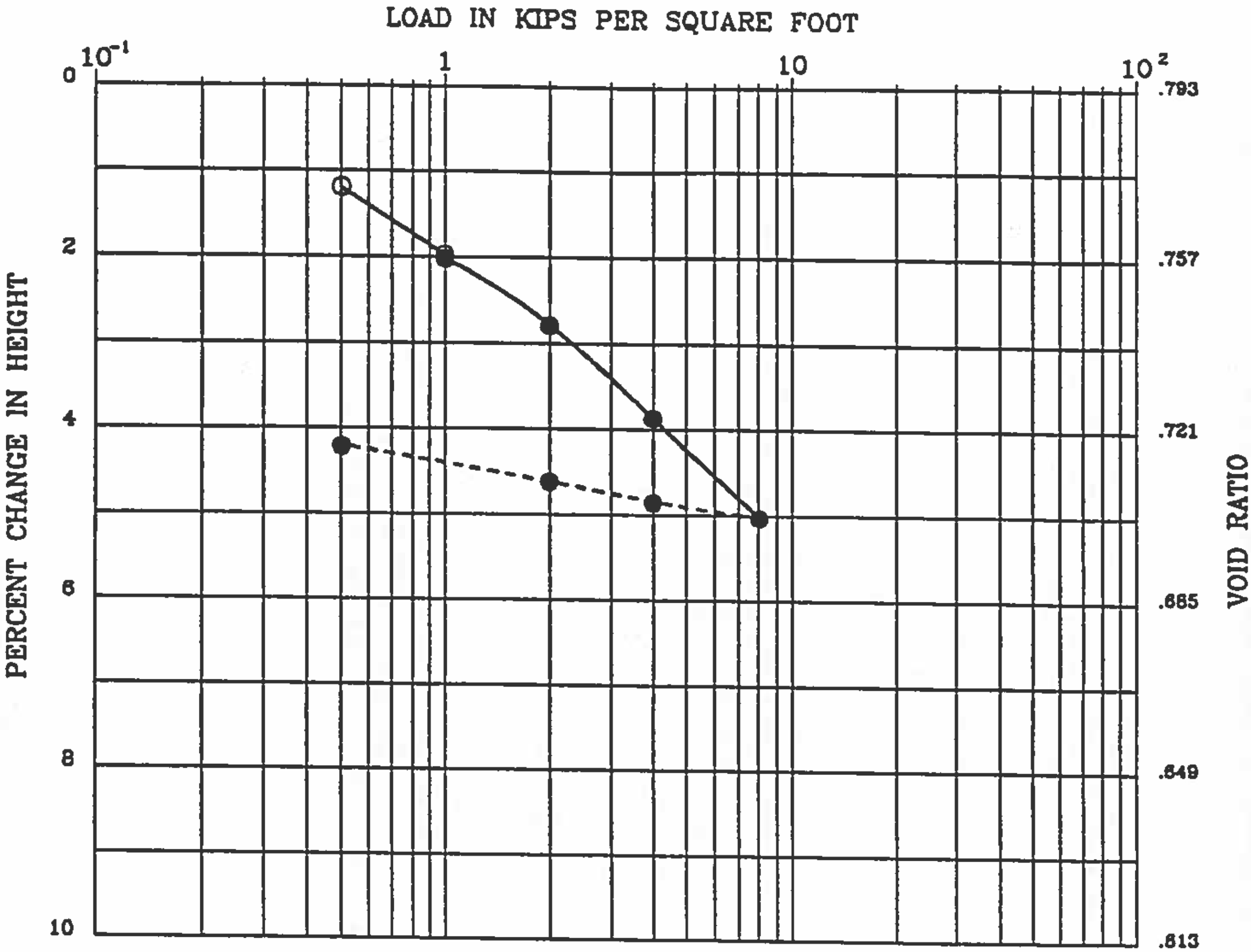
CONSOLIDATION TEST

Reach 4, RWBWTP, Contract II
For: Black and Veatch

Project No.
92-81-504-01

Converse Consultants Inland Empire

Figure No. B-7



BORING : BT-1/S3
DEPTH (ft) : 9-10

DESCRIPTION : Weathered Granite

	MOISTURE CONTENT (%)	DRY DENSITY (pcf)	PERCENT SATURATION	VOID RATIO
INITIAL	16.7	109.2	66	.793
FINAL	22.8	113.9	100	.717

Note: Solid circles indicate readings after addition of water

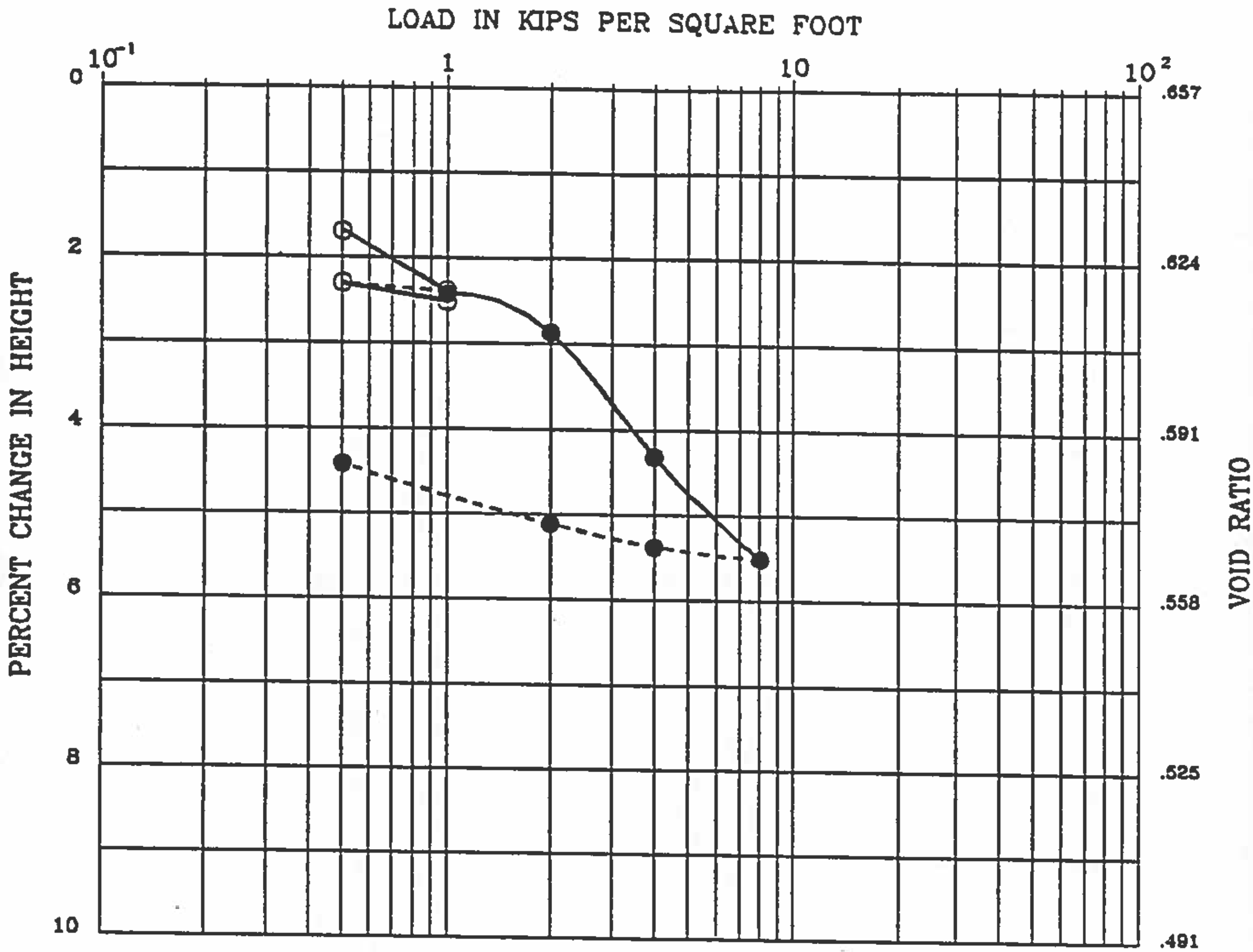
CONSOLIDATION TEST

Reach 4, RWBWTP, Contract II
For: Black and Veatch

Project No.
92-81-504-01

Converse Consultants Inland Empire

Figure No. B-8



BORING : BC-1/S3
DEPTH (ft) : 9-10

DESCRIPTION : Silty Sand (SM), trace clay'

	MOISTURE CONTENT (%)	DRY DENSITY (pcf)	PERCENT SATURATION	VOID RATIO
INITIAL	18.4	101.8	76	.657
FINAL	20.2	106.7	94	.581

Note: Solid circles indicate readings after addition of water

CONSOLIDATION TEST

Reach 4, RWBWTP, Contract II
For: Black and Veatch

Project No.
92-81-504-01

Converse Consultants Inland Empire

Figure No. B-9

APPENDIX C

SEISMIC REFRACTION SURVEY

Sub Surface Surveys

215 So. Highway 101, Suite 203 P.O. Box 1152 Solana Beach, CA 92075 (619) 481-8949

September 22, 1992

Converse Consultants Inland Empire
10391 Corporate Drive
Redlands, CA 92374

Attn: Mohammed Islam re: Lake Elsinore-RR Canyon Seismic Survey

This brief letter report is to convey the results of our seismic refraction survey carried out along Railroad Canyon Road northeast of Lake Elsinore, California (Fig. 1) on September 14-18, 1993. Purpose of the survey was to determine rippability of the granitic clan bedrock and minor metamorphics, generally exposed along both sides of the road, to aid in planning excavations for a water line to be installed within the road right-of-way.

Instrumentation - A Bison 9024 system was used to obtain the basic time-distance data. This 24-channel system is equipped with DIFP, differential instantaneous floating point capability; that is, the wiggle traces from each geophone output is recorded digitally in real time, and in virtual time the on-board computer analyses the digital signals, and "goes back" and applies optimum instrumental settings to the traces, and outputs the optimum traces. Source was a EWG-I accelerated weight drop. Four or five impacts on a striker plate, or more, were stacked (summed) for each record.

Survey Design - The seismic lines were laid out along the southeast side of the roadway, averaging 9 feet from the edge of the asphalt. The controlling factor that determined the selection of the southeast side of the road was the position of installed utility pipes. Along this portion of Railroad Canyon Road the utility pipes are installed beneath the northwest side of the road. Close proximity to the steel pipes was avoided so that the seismic wave would not produce first arrivals through a high velocity pipe, and thereby eliminate first arrivals through the ground.

There was a break in the otherwise continuous line that was designated by the client on the basis of geology, wherein alluvial materials appeared to be anomalously thick. In the terminology used in this report a "spread" is one layout of the seismic line (cables and geophones). These are 480 feet long, according to the layout aforementioned. A "section" is a continuous line terminated at breaks on each end. In this survey each of the two sections are more than one spread long. The survey starts at the first outcrop on Railroad Canyon Road in Lake Elsinore, near I-15, and ends at the mutual boundary of Lake Elsinore and Canyon Lake villages.

The overall seismic line was shot from west to east. The west end of each spread is designated the forward end, and the east end is designated the reverse end. All spreads were shot forward and reverse, and split, that is, a shot in the middle of the spread. Each shot point

RAILROAD CANYON SEISMIC LINE

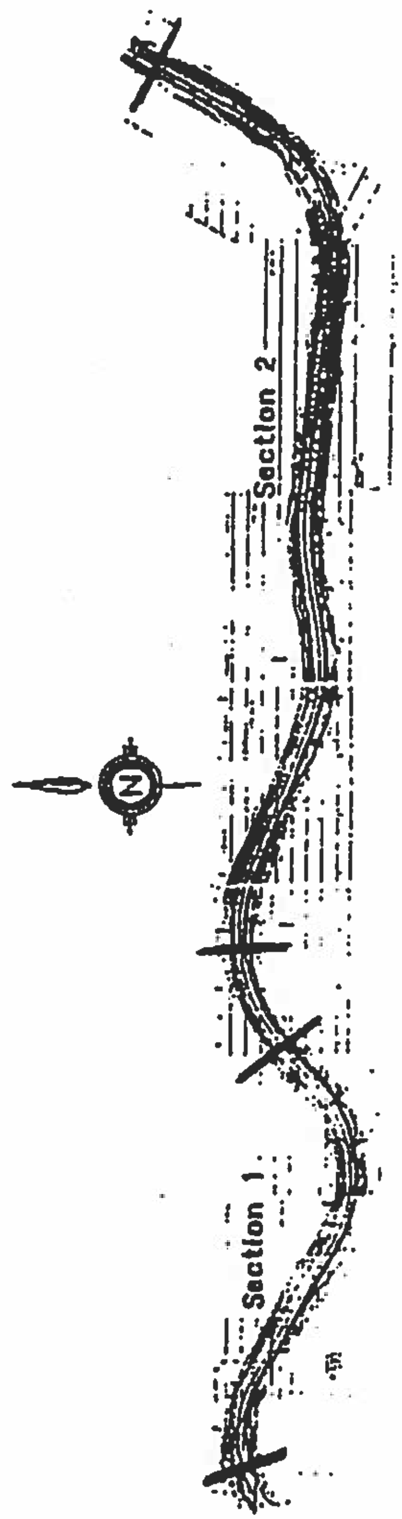


FIGURE 1

was marked with paint on the edge of the asphalt, at the shoulder of the road, and was numbered consecutively from west to east through each section. Shot point numbering started anew at the beginning of each new section, from west to east.

Engineering survey station numbers were not available, as they were in the Canyon Lake sections; hence, other fixed objects along the route are referenced for the purpose of accurate location of the sections, and these appear with the annotations at the top of the seismic sections. Numerous metal culvert pipes extend beneath Railroad Canyon Road, and their positions on the seismic line are annotated at the top of the structure sections (Plate I). In addition, four survey markers were located along the roadway, HV123, HV121, HV118 and HV117, and these are annotated on the structure sections. It should be possible, therefore, to accurately locate any point on the seismic structure sections.

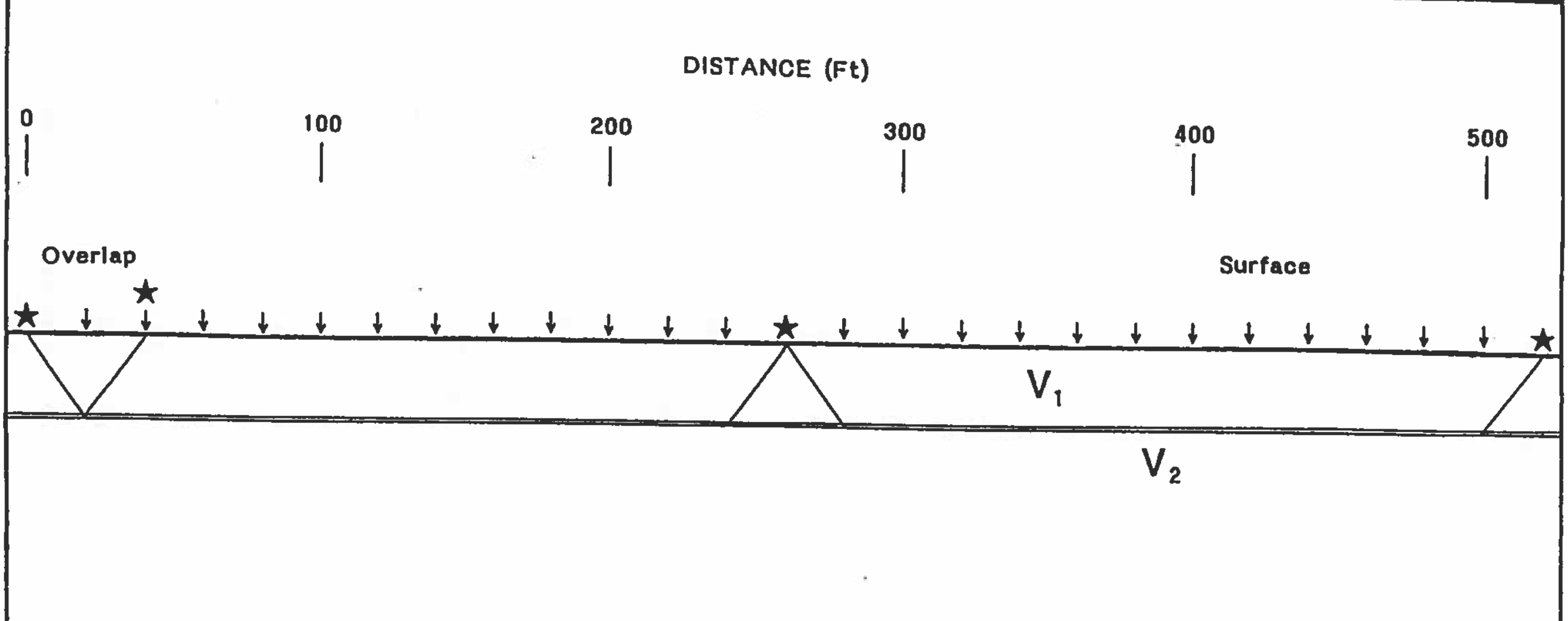
Geophone spacing was 20 feet with 20 foot shot offsets (Fig. 2). Both ends and the center of the spreads were shot. In leap-frogging to the next spread, the forwardmost geophone became the trailing geophone. Thus, with geophone overlap, plus shot offsets, there was 100 percent coverage at the depths of interest, nominally 30 feet. With this set up, total length of each spread was 480 feet, and maximum depth investigated was approximately 100 feet.

The shooting parameters are summarized in Table I.

The crew waited for windows in traffic and construction activity, to make each shot, in order to obtain the most definitive records. All shots were stacked shots.

<u>Brief Description of the Geophysical Method Applied</u> - Seismic refraction investigates the subsurface by generating arrival time and offset distance information to determine the path and velocity of an elastic disturbance. The disturbance is created by shot, hammer, weight drop or some comparable method of putting impulsive energy into the ground. Detectors are laid out at regular intervals in a line to measure the first arrival energy and the time of its arrival. The data are plotted in time-distance graphs, from which velocity of, and depth to, layers can be calculated. This is possible because rays (a continuum point on an expanding wave front) of the disturbance wave are refracted across layer boundaries where there is a difference in elastic and density properties. The critically refracted rays travel along the layer interfaces and continuously "feed" energy back to the surface.

Shots are normally reversed from one end of the line to the other to determine whether or not the layering is horizontal or dipping. The acquired data are computationally intense. A ray-tracing computer program, SIPT2, is used to iteratively honor all detector information to determine dip and irregularities in the refracting surfaces, and to be able to consider a large number of layers, where they are developed.



SEISMIC REFRACTION LAYOUT

FIGURE 2

TABLE I

The Exploration Problem

Seismic Parameters To Be Defined	Depth of Interest	Reflection Quality	Required Resolution	Steepest Dip	Type of Feature	Noise Problems	Access Problems	Special Processing	SHOOTING PARAMETERS
Far-Trace Offset (Distance to Farthest Geophones)	●								500 Ft
Near-Trace Offset	●						●		20 Ft
Group Interval			●	●					20 Ft
Charge Size (Or Other Source Effort Required)	●		●			●			Accel Wt Drop
Charge Depth		●	●			●			Surface
Multiplicity (CDP Fold)		●	●			●	●	●	Single
Sample Rate	●		●					●	.05 ms
Low-cut Filter	●		●			●		●	8 Hz
Geophone Frequency	●		●			●		●	30 Hz
Record Length	●							●	500 ms
Geophone Array Size (And Design)			●			●		●	Single
Spread Type (Split, End-On, Offset, Etc)						●	●		End on & Split
Line Length	●				●		●	●	480 Ft
Line Direction					●		●	●	Pipeline Route
Line Spacing			●		●		●	●	Pipeline Route

Note: Testing could suggest minor modifications.

Geologic Setting - Granite clan rocks, dominantly tonalites of Cretaceous age, form the bedrock along the route. Roof pendants of metamorphosed Triassic/Jurassic marine and non-marine sediments, and minor volcanic rocks, are mapped locally in a few places. A near vertical boundary between granites and metamorphics occurs in the road cut at the beginning point of the survey.

The bedrock, dominantly tonalites of the Peninsular Range batholith, weathers differentially as dictated by exposure, surface and subsurface drainages, water and air access in fracture zones, differing chemical compositions, dike development, structural development in the composite batholith, and other lesser factors. Generally, the local exposures in the road cuts are quite friable, and are thus deeply weathered. "Core" rocks were not conspicuously developed along the route of the seismic line. Further, some minor aplite dike development is seen in the road cuts. Thus, there is obviously differential weathering.

Interpretation - It may be stated that the construction crews installing the gas pipelines in the town of Canyon Lake reported that all rock within their ditching excavations through the community was ripped with large caterpillar equipment with ripper. They did report also that in a few places rippability was marginal and that a pneumatic "hammer" had to be installed on the ripper in order to complete the work.

Because of the 20 foot geophone spacing, surface layers less than approximately 6 feet thick are not resolved in the data. This was, of course, known and anticipated in the planning stages of the survey. Inasmuch as it is a virtual certainty that any thin surface layer is rippable, this is of no particular consequence in terms of the objective of the survey, and the savings of longer geophones intervals were opted for. There are places, nevertheless, where the surface, low velocity layer reaches thicknesses considerably greater than 6 feet, and is readily resolved and shown in the seismic sections.

Where no such layer is shown, it can be assumed that there is no low velocity layer, or it is thinner than 6 feet. The prepared road bed, even where laid down on bedrock, may be in the order of 3 feet. Except for the prepared road bed, any natural surface materials were, in places, removed during road construction. In fact, in some of the deeper road cuts, much of the weathered bedrock layer was undoubtedly removed. It would not be surprising, therefore, to identify places, in the deeper road cuts, where unweathered bedrock is fairly near the surface.

The seismic velocity of the compacted road bed materials is similar to natural alluvial and colluvial deposits. Thus, inasmuch as the route of the seismic line is on the roadway, it appears that there is a low velocity surface layer everywhere, but in some places it is less than 6 feet thick, and is not depicted on the seismic sections.

Generally, then, the shallow earth along the seismic line is characterized by a three layer case. The top layer is alluvium, colluvium, and artificial fill that makes up the prepared road bed. Average velocity of this layer is in the order of 1600 ft/s. The range of velocities is about 1200 to 1900 ft/s. Thickness of the surface layer ranges from less than 6 feet to 20 feet.

The next deeper layer is weathered bedrock, mostly weathered granite clan rocks and locally metamorphosed sediments. Average velocity is 3765 ft/s. Its range, however, is large, which is not surprising owing to the variability in local conditions that bring about differential weathering. The range is from near 2800 to 5500 ft/s. Its thickness is similarly variable, ranging from about 11 to 58 feet.

The third layer, which extents to depth (based on geologic considerations), probably well below the 100 plus foot investigation depth, has an average velocity in the order of 11,200 ft/s. This layer (of probable great thickness) has a trend to slightly lower velocities near the east end of the survey. This high velocity layer comes to within 10 feet of the surface about 1000 feet west of the survey marker HV123, and locally the depth is less than 20 feet in 3 or 4 other places.

<u>Conclusions</u> - The seismic refraction data indicate that the shallow subsurface is a three layer case, with laterally variable depth caused by differential weathering.

All data acquired in this survey are in confidential file in this office, and are available for review by your staff, or by us at your request, at any time. We appreciate the opportunity to participate in this project. Please call, if there are questions.

Gary W. Crosby

Gary W. Crosby, PhD, GP 960

GWC:arr

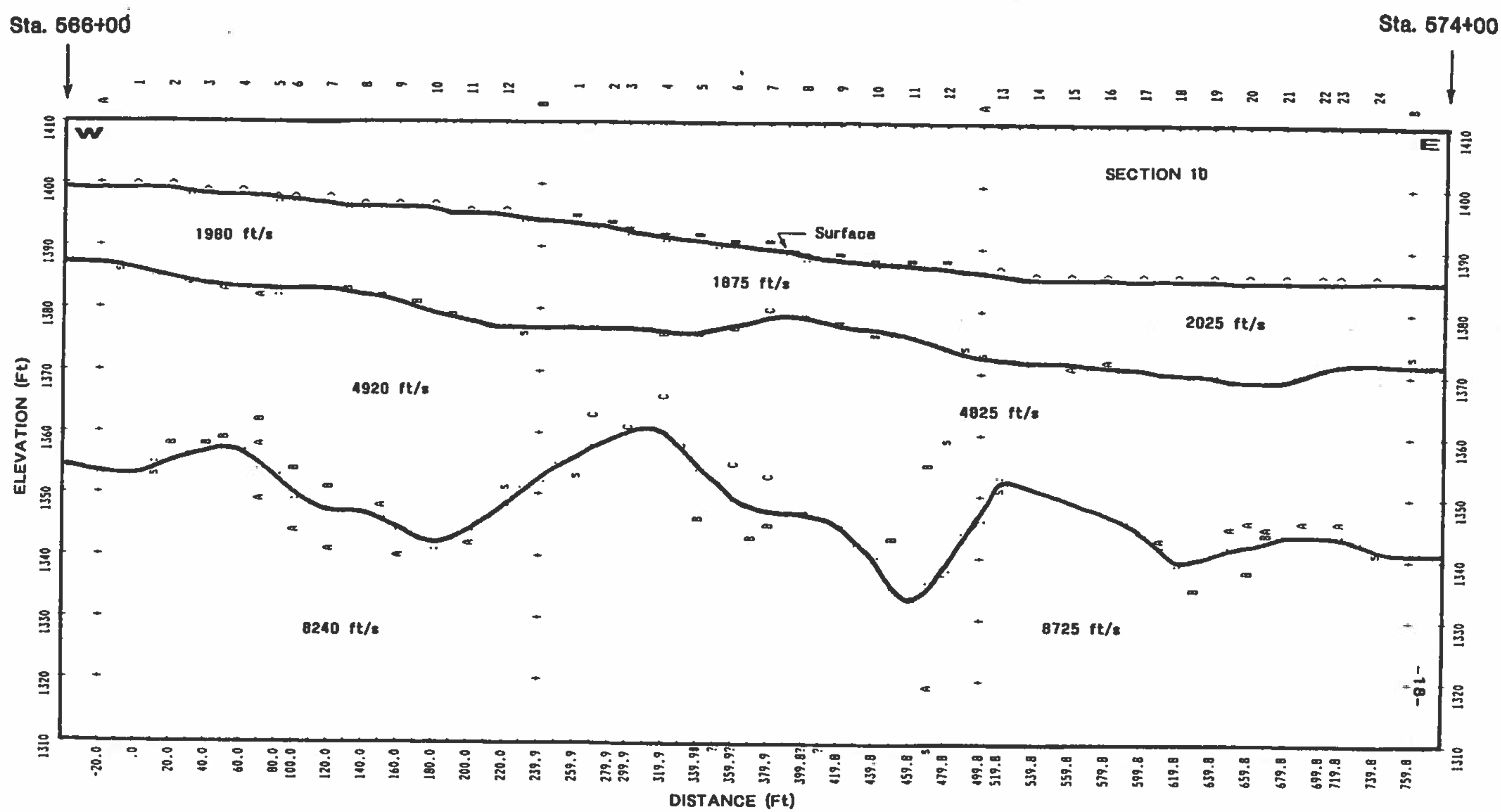

Sta. 566+00
Sta. 574+00
W
E
SECTION 10
Surface
1980 ft/s
1875 ft/s
2025 ft/s
4920 ft/s
4825 ft/s
8240 ft/s
8725 ft/s
ELEVATION (Ft)
DISTANCE (Ft)
-18-

PLATE II

APPENDIX D

SOIL CORROSIVITY STUDY

M. J. SCHIFF & ASSOCIATES

Consulting Corrosion Engineers

1291 NORTH INDIAN HILL BOULEVARD
CLAREMONT, CALIFORNIA 91711-3897
909/626-0967
FAX 909/621-1419

December 23, 1993

CONVERSE CONSULTANTS INLAND EMPIRE
10391 Corporate Drive
Redlands, California 92374

Attention: Mr Mohammed Islam

Re: Soil Corrosivity Study
EMWD - Reach 4, Contract 2
Riverside County, California
Your #92-81-504-01, MJS&A #92186

INTRODUCTION

Field and laboratory tests have been completed on soils along and from the route of the subject pipeline. The purpose of these tests was to determine soil corrosivity regarding the proposed waterline and concrete structures.

Reach 4, Contract 2 will run from the Sun City Regional Water Reclamation Facility (SCRWRF) to the east end of Contract 1, and from the west end of Contract 1 to Contract 3. The total length of Contract 2 is about 4.5 miles. Contract 1 runs through the city of Canyon Lake; Contract 3 begins at the east side of Interstate 15. Materials being considered for the proposed 54-inch diameter waterline are cement-mortar coated steel pipe (CMCSP) and tape wrapped steel pipe (TWSP).

The scope of this study is limited to a determination of soil corrosivity and its general effects on materials likely to be used for construction. If the engineers desire more specific information, designs, specifications, or review of design, we will be happy to work with them as a separate phase of this project.

TEST PROCEDURES

The electrical resistivity of the soil was measured in situ using the Wenner Four Pin Method. This procedure measures the average electrical resistivity of the soil from the surface to a depth equal to the pin spacings. Pin spacings of 2.5, 5, 7.5, 10, and 15 feet were used so that variations with depth could be evaluated. Stratum resistivities were calculated by the Barnes Procedure. Test results are shown on Table 1.

The electrical resistivity of each soil sample was measured in its as-received condition and again with distilled water added to create the standardized condition of saturation. Soil resistivities are at about their lowest value when the soil is saturated. The samples were chemically analyzed for the major anions and cations and pH was measured. Results are shown on Table 2.

DISCUSSION

A useful factor in determining soil corrosivity is electrical resistivity. The electrical resistivity of a soil is a measure of it resistance to the flow of electrical current. Corrosion of buried metal is an electrochemical process in which the amount of metal loss due to corrosion is directly proportional to the flow of electrical current (DC) from the metal into the soil. Corrosion currents, following Ohm's Law, are inversely proportional to soil resistivity. Lower electrical resistivities result from higher moisture and chemical contents and indicate corrosive soil.

A correlation between electrical resistivity and corrosivity toward ferrous metals is:

Soil Resistivity in ohm-centimeters	Corrosivity Category
0 to 1,000	severely corrosive
1,000 to 2,000	corrosive
2,000 to 10,000	moderately corrosive
over 10,000	mildly corrosive

Average resistivities at all seven test locations and all but two stratum resistivities were in the moderately corrosive category. The 10 to 15 foot stratum resistivities at FR 1 and 2 were in the corrosive category.

The four electrical resistivities measured in the laboratory with as-received moisture content were in mildly and moderately corrosive categories. When saturated, they dropped into moderately through severely corrosive categories. The resistivities dropped considerably with added moisture indicating that the samples were dry as-received. The wide variations in resistivity can create concentration cells that increase corrosion rates above what would be expected from the soil characteristics alone.

Correlation between field and laboratory resistivities was good. The lowest resistivities measured in the field and laboratory were near the SCRWRF.

Soil pH values varied from 7.7 to 8.1. This range is slightly to moderately alkaline and does not have a significant effect on corrosivity.

The chemical content of the sample from BF-1 was high and the others were low. Sodium sulfate was the predominant compound at BF-1, and the concentration was in a range where sulfate resistant cement should be used.

The reach from the SCRWRF to Newport Road (BF to BT 1) is classified as severely corrosive to ferrous metals and deleterious to concrete; the remainder is classified as moderately corrosive to ferrous metals.

A check of cathodic protection system locations listed by members of the Southern California Cathodic Protection Committee was made to determine if there are any cathodic protection systems that might cause corrosion due to stray DC current interference. There were no rectifiers listed in the area and none were seen along the route.

RECOMMENDATIONS FOR STEEL PIPELINE WITH CEMENT-MORTAR COATING

1. Steel pipe with a cement-mortar coating per AWWA Standard C-205 from the SCRWRF to Newport Road should be made sulfate resistant by using type 2 cement with 25 percent replaced by class F bituminous fly ash with as sulfate resistance factor less than 1.0 or type 5 cement. The remainder may be made with type 1 or 2 cement.

2. Buried steel and iron pipe and fittings in appurtenances such as air valves and blowoffs should be cement-mortar coated or encased in concrete.

3. Buried cast and ductile iron valves should be cement-mortar coated or encased in concrete or cement slurry where possible. Where not possible, they should be coated with a fusion-bonded epoxy, coal-tar epoxy, pipe wrapping tape, or equivalent and wrapped with 8 mil polyethylene per AWWA Standard C-105/ANSI 21.5. Backfill with an alkalized sand (50 pounds of hydrated lime mixed with each cubic yard of sand) at least 6 inches thick surrounding the valve.

4. Bond pipe joints for electrical continuity by means of three steel bonds welded between the bell and spigot and coated with cement-mortar per EMWD Standard Drawing B-638, Detail C. Bond any flexible couplings or other nonconductive type joints with 3 #4 copper wires.

5. Install insulated joints at the connection to SCRWRF and to Contracts 1 and 3 except where they use CMCSP. Insulated joints should be placed above grade or in vaults where possible. Install an insulated test connection per EMWD Standard Drawing B-379 at all buried or otherwise inaccessible insulated joints.

6. Install corrosion monitoring test stations at each end, one end of any casings, where any cathodically protected lines cross, and other locations as necessary so the interval between test stations does not exceed 1,500 feet. Pipeline test stations should use a #12 and #6 or larger wire with type THWN insulation per EMWD Standard Drawing B-582 with every fourth test station a four-wire type for measuring line current. Independently braze each wire to the pipe. Install 2 wire test stations at casings, and braze two additional wires of a different color to the casing.

7. Cathodic protection is not recommended at this time. The joint bonds and test stations will permit cathodic protection to be easily and economically installed in the future if necessary.

8. Preliminary construction drawings should be reviewed by a qualified corrosion engineer to insure that corrosion control recommendations are properly incorporated.

9. After the pipeline is backfilled, but before the construction contract is completed, the pipeline should be tested to insure that the joint bonds are intact and test stations properly installed. These data will be useful in determining if pipeline conditions change in the future.

10. Pipe-to-soil potentials should be measured biennially to determine if conditions on the pipeline are changing.

RECOMMENDATIONS FOR TAPE WRAPPED STEEL PIPE

1. Wrap steel pipe with pipe-wrapping tape per AWWA Standard C-214, 50 mils thick if bedded and backfilled with sand. If bedding or backfill contains rock that could be harmful to the tape, use 80 mil tape or 50 mil tape protected by a 3/4 inch cement-mortar coating.

2. Coat field joints with a tape wrap per AWWA Standard C-209 or with heat shrink sleeves per AWWA Standard C-216.

3. Coat buried iron and steel pipe, fittings, and valves in appurtenances with a dielectric coating such as fusion-bonded epoxy, coal-tar epoxy, tape wrap, or equivalent.

4. Bond pipe joints for electrical continuity by means of three steel bonds welded between the bell and the spigot per EMWD Standard Drawing B-638, Detail A or B.

5. Install insulated joints at connections to the SCRWRF and Contracts 1 and 3. Insulated joints should be placed above grade or in vaults where possible. Install an insulated test

connection per EMWD Standard Drawing B-379 at all buried or otherwise inaccessible insulated joints.

6. Install corrosion monitoring test stations at each end, one end of any casings, where any cathodically protected lines cross, and other locations as necessary so the interval between test stations does not exceed 1,500 feet. Pipeline test stations should use a #12 and #6 or larger wire with type THWN insulation per EMWD Standard Drawing B-582 with every fourth test station a four-wire type for measuring line current. Independently braze each wire to the pipe. Install 2 wire test stations at casings, and braze two additional wires of a different color to the casing.

7. Apply cathodic protection to the reach from the SCRWRF to Contract 1.

8. Preliminary construction drawings should be reviewed by a qualified corrosion engineer, to insure that corrosion control recommendations are properly incorporated.

9. After the pipeline is backfilled, but before the construction contract is completed, the pipeline should be tested to insure that the joint bonds are intact and test stations properly installed. Also, baseline pipe-to-soil potentials should be measured and recorded. These data will be useful in determining if pipeline conditions change in the future.

10. Pipe-to-soil potentials should be measured annually to confirm cathodic protection and determine if conditions on the pipeline are changing.

RECOMMENDATIONS FOR CEMENT-MORTAR COATINGS AND CONCRETE STRUCTURES

Concrete, cement-mortar, or cement slurry contacting soils in the reach from the SCRWRF to Newport Road should be made sulfate resistant by using type 2 cement with 25 percent replaced by class F bituminous fly ash with a sulfate resistance factor less than 1.0 or type 5 cement. In other locations type 1 or 2 cement may be used.

Respectfully Submitted,
M.J. SCHIFF & ASSOCIATES

Robert A. Pannell

Robert A. Pannell
jjj

Enc: Table 1 - Soil Resistivity - Field Tests
Table 2 - Laboratory Tests on Soil Samples

WPL\92186

Test Date
08 September 93

TABLE 1
SOIL RESISTIVITY - FIELD TESTS
EMWD - REACH 4, CONTRACT 2
RIVERSIDE COUNTY, CALIFORNIA
MJS&A #92186

LOCATION	DEPTH feet	MEASURED RESISTANCE ohms	AVERAGE RESISTIVITY TO DEPTH ohm-cm	STRATUM RESISTIVITY ohm-cm
FR 1 near Sun City WWRF bare, dry ground	2.50	4.60	2300	2300
	5.00	2.60	2600	2990
	7.50	2.00	3000	4333
	10.00	1.40	2800	2333
	15.00	0.78	2340	1761
FR 2 Newport Road bare, dry ground	2.50	8.80	4400	4400
	5.00	2.70	2700	1948
	7.50	1.90	2850	3206
	10.00	1.70	3400	8075
	15.00	0.84	2520	1660
FR 3 Newport & Goetz Roads bare, dry ground	2.50	14.00	7000	7000
	5.00	7.00	7000	7000
	7.50	5.80	8700	16917
	10.00	4.40	8800	9114
	15.00	3.30	9900	13200
FR 4 RR Canyon & Cottonwood bare, dry ground	2.50	16.00	8000	8000
	5.00	6.60	6600	5617
	7.50	4.10	6150	5412
	10.00	2.90	5800	4954
	15.00	1.70	5100	4108

Test Date
08 September 93

TABLE 1
SOIL RESISTIVITY - FIELD TESTS
EMWD - REACH 4, CONTRACT 2
RIVERSIDE COUNTY, CALIFORNIA
MJS&A #92186

LOCATION	DEPTH feet	MEASURED RESISTANCE ohms	AVERAGE RESISTIVITY TO DEPTH ohm-cm	STRATUM RESISTIVITY ohm-cm
FR 5	2.50	7.60	3800	3800
RR Canyon Rd				
0.5 mi. west of FR 4	5.00	3.50	3500	3244
bare, dry ground				
	7.50	2.00	3000	2333
	10.00	1.40	2800	2333
	15.00	0.93	2790	2770
FR 6	2.50	11.00	5500	5500
RR Canyon Rd				
0.5 mi. west of FR 5	5.00	5.80	5800	6135
bare, dry ground				
	7.50	3.40	5100	4108
	10.00	2.10	4200	2746
	15.00	1.20	3600	2800
FR 7	2.50	9.00	4500	4500
RR Canyon Rd				
0.5 mi. west of FR 6	5.00	4.10	4100	3765
bare, dry ground				
	7.50	2.90	4350	4954
	10.00	2.00	4000	3222
	15.00	1.20	3600	3000

TABLE 2

RESULTS OF LABORATORY ANALYSIS ON SOIL SAMPLES

Sample ID	Soil Type	Soil Resistivity		Saturated Soil pH	EC* (mS/cm)	Chemical Analysis in mg/kg (ppm) of dry soil										
		As-received (ohm-cm)	Saturated (ohm-cm)			Ca	Mg	Na	NH_4	CO_3	HCO_3	Cl	SO_4	NO_3	S^{--}	Redox (mv)
BF1 6-10'	clayey sand	5,500	420	7.7	1.42	240	60	1,242	NA	nd	183	35	7,296	NA	NA	NA
BT1 4-8'	silty sand and clay	6,600	1,600	8.0	0.19	nd	nd	218	NA	nd	244	35	408	NA	NA	NA
BE1 9-14'	silty sand and gravel	17,000	2,300	8.1	0.10	nd	nd	115	NA	nd	183	70	168	NA	NA	NA
BE3 7-10'	silty sand and cobbles	44,000	7,600	8.0	0.06	nd	nd	69	NA	nd	183	35	120	NA	NA	NA

nd = not detected
NA = Not Analyzed
*Electrical Conductivity measured on a 1:5 soil to distilled water extract.

Soil Corrosivity Study
EMWD - Reach 4, Contract 2
Riverside County, California
Your #92-81-504-01, MJS&A #92186
F1