APPENDIX B.1

Geotechnical Report from Spec 604S
(For Reference Only)

[PAGE LEFT INTENTIONALLY BLANK]

Converse Consultants
Inland Empire

Consulting Engineering
and Applied Sciences

10391 Corporate Drive
Redlands, CA 92374

Telephone 909 / 796-0544
FAX 909 796-7675

FINAL GEOTECHNICAL INVESTIGATION REPORT

Reach 4, Reclaimed Water and Brine Water
Transmission Pipelines
Contract I (Station 480+00 to 562+00)
City of Canyon Lake
Riverside County, California

Prepared For:

Black & Veatch
6 Venture, Suite 315
Irvine, CA 92718-3317

CCIE Project No. 92-81-504-01

November 11, 1993

A Wholly Owned Subsidiary of
The Converse Professional Group

November 11, 1993

Mr. Steven N. Foellmi
Project Manager
Black & Veatch
6 Venture, Suite 315
Irvine, CA 92718-3317

Subject: **FINAL GEOTECHNICAL INVESTIGATION REPORT**
Reach 4, Reclaimed Water and Brine Water Transmission Pipelines
Contract I (Station 480+00 to 562+00)
City of Canyon Lake
Riverside County, California
CCIE Project No. 92-81-504-01

Dear Mr. Foellmi:

Enclosed is the final report of our geotechnical investigation performed along the alignment of the proposed Reach 4 Reclaimed Water and Brine Water Transmission Pipelines, Contract I (Station 480+00 to 562+00), City of Canyon Lake, Riverside County, California. This investigation was performed in accordance with our agreement dated July 9, 1993. We have addressed review comments on the draft report received from Black & Veatch, dated October 29, 1993 and incorporated them into this final report.

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 weathered granitic and metamorphic bedrock. Fill and alluvial soils are composed of sands, silts, and clays.

Laboratory tests and field investigations indicate that disintegrated weathered bedrock at the elevation of the pipe invert, free of oversized particles, may be used as pipe bedding. Fill or native alluvial soils at pipe invert elevations may not be suitable as bedding material (Sand Equivalent < 30).

Ground water was encountered in exploratory borings BC-3 and BC-4 at a depth of about 17 to 18 feet below existing ground surface. For most of the alignment, the pipe invert will be approximately eight to 10 feet below existing grade, and the effects of water need not be considered during design and construction.

Liquefaction potential of the site is considered low, as the depth of weathered bedrock extends above the pipe invert for most of the alignment. Also, the overlying soil deposits for the remainder of the alignment occur above ground-water table and consist of soils that contain significant amount of fines and therefore are not likely to liquefy.

Where the pipe invert will be required to be placed below about 17 feet, effects of water seeping into the excavation should be considered. Dewatering may be required during construction in these areas. The contractor should perform the necessary field testing to evaluate the pumping rates for dewatering.

Based on the nature of subsurface materials encountered within the depth of the pipe invert elevations, the majority of the trenches for the pipeline construction should be excavatable with conventional heavy-duty trenching equipment such as Caterpillar D9N Rippers. Some very hard ripping requiring extra effort, such as use of a pneumatic hammer or blasting, may be encountered locally.

Based on the study of soil corrosivity, this route is classified as moderately to mildly corrosive to ferrous metals and low corrosive to concrete.

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 Black & Veatch is appreciated.

Very truly yours,

CONVERSE CONSULTANTS INLAND EMPIRE

Quazi S.E. Hashmi, Ph.D., P.E.
Branch Manager

Dist: 3/Addressee

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.

Mohammed S. Islam, Ph.D.
Senior Staff Engineer

Jason Holcomb
Senior Staff Geologist

Quazi S. E. Hashmi, Ph.D., P.E.
Engineering Manager

Richard F. Escandon, C.E.G. 1213
Senior Geologist

CONTENTS

TABLES

ILLUSTRATIONS

Figures

Drawings

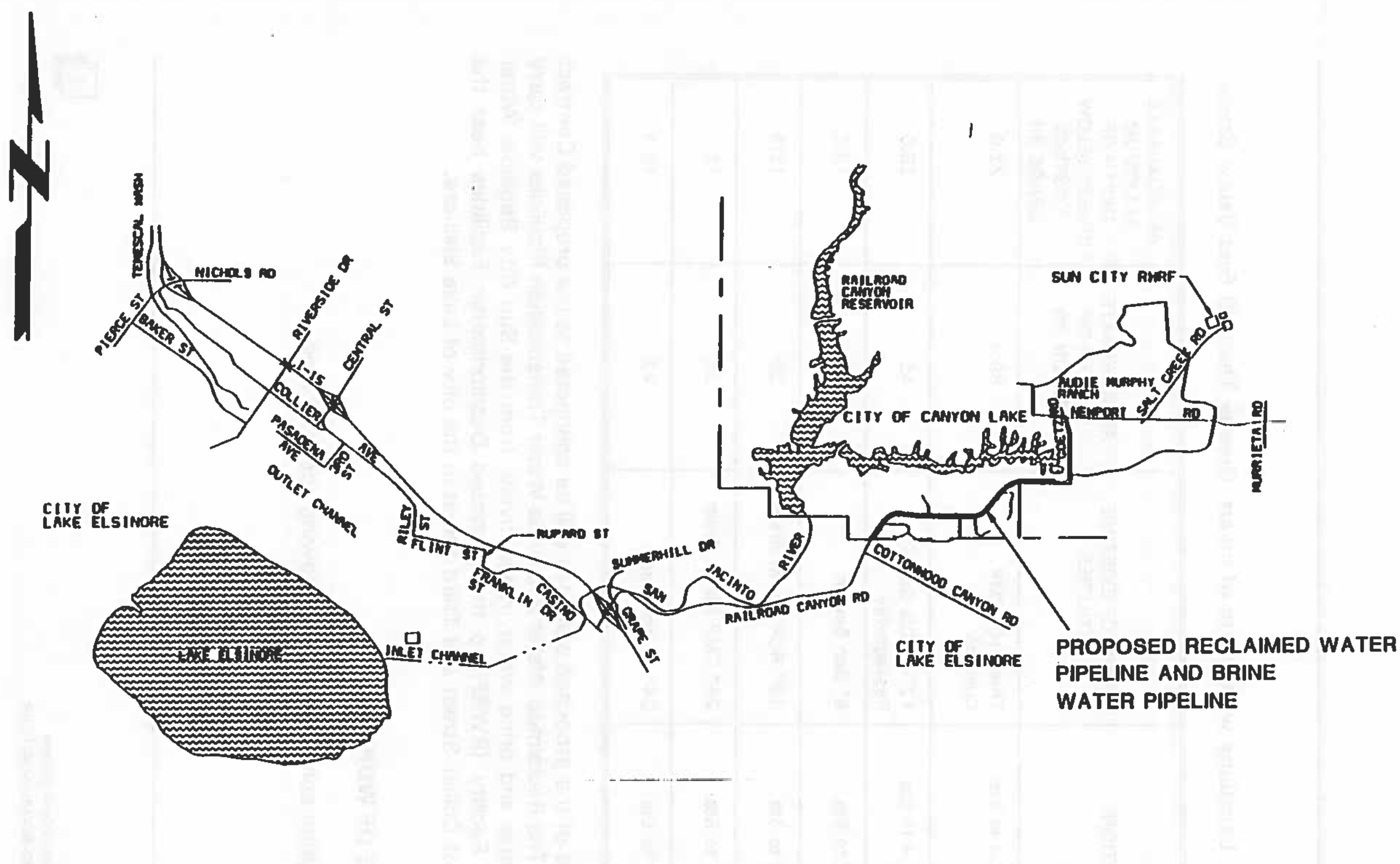
1.0 INTRODUCTION

This report contains the findings of CCIE's geotechnical investigation performed for the proposed Reach 4 Reclaimed Water and Brine Water Transmission Pipelines, Contract I (Stations 480+00 to 562+00) located in the city of Canyon Lake, 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 pipelines.

The pipeline alignment is shown in Figure No. 1, *Location Map*, and Drawing No. 1, *Geologic/Boring Location Map*. The proposed alignment under Contract 1 runs along Railroad Canyon Road from near the intersection with Cottonwood Canyon Road (Station 480+00) to near the eastern end of Railroad Canyon Road (Station 562+00) at Goetz Road. The total length of the alignment is about 8,200 feet.

The Reclaimed Water and Brine Water Transmission Pipelines will carry reclaimed water and brine water, respectively, from the Sun City Regional Water Reclamation Facility (RWRF) 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 pipelines 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, invert elevation of the 54-inch reclaimed water pipeline will be approximately eight feet to 10 feet below the existing ground surface. Due to the presence of existing 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 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 I, EMWD
City of Canyon Lake, California
for: Black and 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 Grade.

LOCATION	TYPE OF EXISTING UTILITIES	APPROXIMATE LENGTH OF SECTION (ft)	APPROXIMATE MAXIMUM DEPTH OF INVEST BELOW EXISTING GRADE (ft)
Sta. 483+93.9 to Sta. 484+63.3	Three 10' CMP Culverts	69.4	22.0
Sta. 496+77.4 to Sta. 497+21.5	12'-6" Dia Golf Cart Passageway	45	25.0
Sta. 507+60 to Sta. 509+60	8" San Sewer	200	13.2
Sta. 520+15 to Sta. 520+43.8	33" RCP Storm Drain	29	15.5
Sta. 539+52 to Sta. 539+76.7	24" CMP Storm Drain	25	15
Sta. 555+28 to Sta. 555+70	24" Storm Drain	42	12.5

The two ends of the proposed pipelines will be connected to the proposed Contract II pipelines. The Reclaimed Water and Brine Water Transmission Pipelines will carry reclaimed water and brine water, respectively, from the Sun City Regional Water Reclamation Facility (RWRF) to the proposed Dechlorination Facilities near the intersection of Collier Street and Third Street in the city of Lake Elsinore.

2.0 SCOPE OF WORK

Our investigation consisted of the following scope of work:

Geologic Reconnaissance and Mapping

Our geologists performed a field reconnaissance along the proposed alignment to identify possible geologic and/or geotechnical impacts. Detailed geologic mapping of the proposed alignment was performed as presented in Drawing No. 1, *Geologic Map/Boring Location Map*.

Subsurface Exploration

Four borings were drilled to obtain subsurface information along the proposed pipeline alignment. Borings BC-1 and BC-2 were drilled to a depth 15 feet below existing grade; and Borings BC-3 and BC-4 were drilled to a depth of 25 feet below existing surface grade. The location of the borings are shown 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 are presented in Appendix A, *Field Exploration*.

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*.

Seismic Refraction Survey

A seismic refraction survey along about 5,500 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*.

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*.

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 section of the proposed pipeline alignment addressed in this report is about 8,200 feet long and runs along the existing Railroad Canyon Road within the city of Canyon Lake, California. 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. The areas to the north and south of the alignment consist of residential as well as commercial developments.

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. Based on the data, the subsurface conditions underlying the site vary widely. The depth to bedrock varies from more than 15 feet at the east end to about one foot at the west end of the alignment. The degree of weathering of the bedrock encountered within the depths of exploratory borings ranges from severe to moderate.

The subsurface materials overlying bedrock are composed of either fill or native alluvium deposits. The depth of fill material varies from about one foot to greater than 15 feet. The fill materials are generally dense and composed of silty sands with appreciable amounts of gravel and trace clay. These fills were likely associated with the construction of the Railroad Canyon Road and other developments.

The native soils occur as relatively loose alluvium deposits. The alluvial materials were encountered in Boring BC-1 to a depth of 10 feet in the eastern portion of the alignment and comprised of mainly silty and/or clayey sands.

The compressibility of the subsurface materials at the elevation of the pipe invert is in general low and is not susceptible to moisture variation. The direct shear tests indicate that the friction angle of the native alluvial soils is about 37 degrees and cohesion of about 1,150 psf. The friction angle and cohesion of the decomposed bedrock was found to be about 47 degrees and 800 psf, respectively.

The Sand Equivalent (SE) of the representative native soils resulted from disintegration of weathered bedrock at pipe invert elevation is found to be about 45. A native soil sample from Boring BC-1 indicated a SE of 13. This boring is located at approximate Station 566+00. This station is located outside the east end of the proposed alignment by about 400 feet. An examination of the seismic velocity profile shows that alluvial soils with low seismic velocity that occur at Station 566+00 do not extend toward west beyond Station 562+00, the east end of the proposed alignment.

The Resistance (R) value of the soils at the road subgrade elevation is 27.

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

4.1 Geologic Setting

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 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 the eastern 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. The formation generally consists of resistant, light grey to brown, quartz-rich shale and metasandstone.

4.2.2 Granodiorite and diorite (Map unit Ku) Cretaceous-age granodiorite and diorite underlies much of the alignment as evidenced from numerous roadcuts. The unit generally consists of moderately to severely weathered granodiorite and diorite near the surface. Locally, relatively resistant "core rock" can be observed in roadcuts where the less resistant weathering rind has been removed.

4.2.3 Older Alluvium (Map unit Qoal) Older (Pleistocene-age) alluvium was encountered in the western portion of the alignment near Canyon Lake Golf Course. The older alluvium typically consists of moderately to well consolidated mixtures of silty sands and gravelly sands, commonly exhibiting moderate to strong soil development.

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 up to several feet thick in the drainage for Salt Creek. These deposits typically consist of unconsolidated mixtures of silts, sands and gravel.

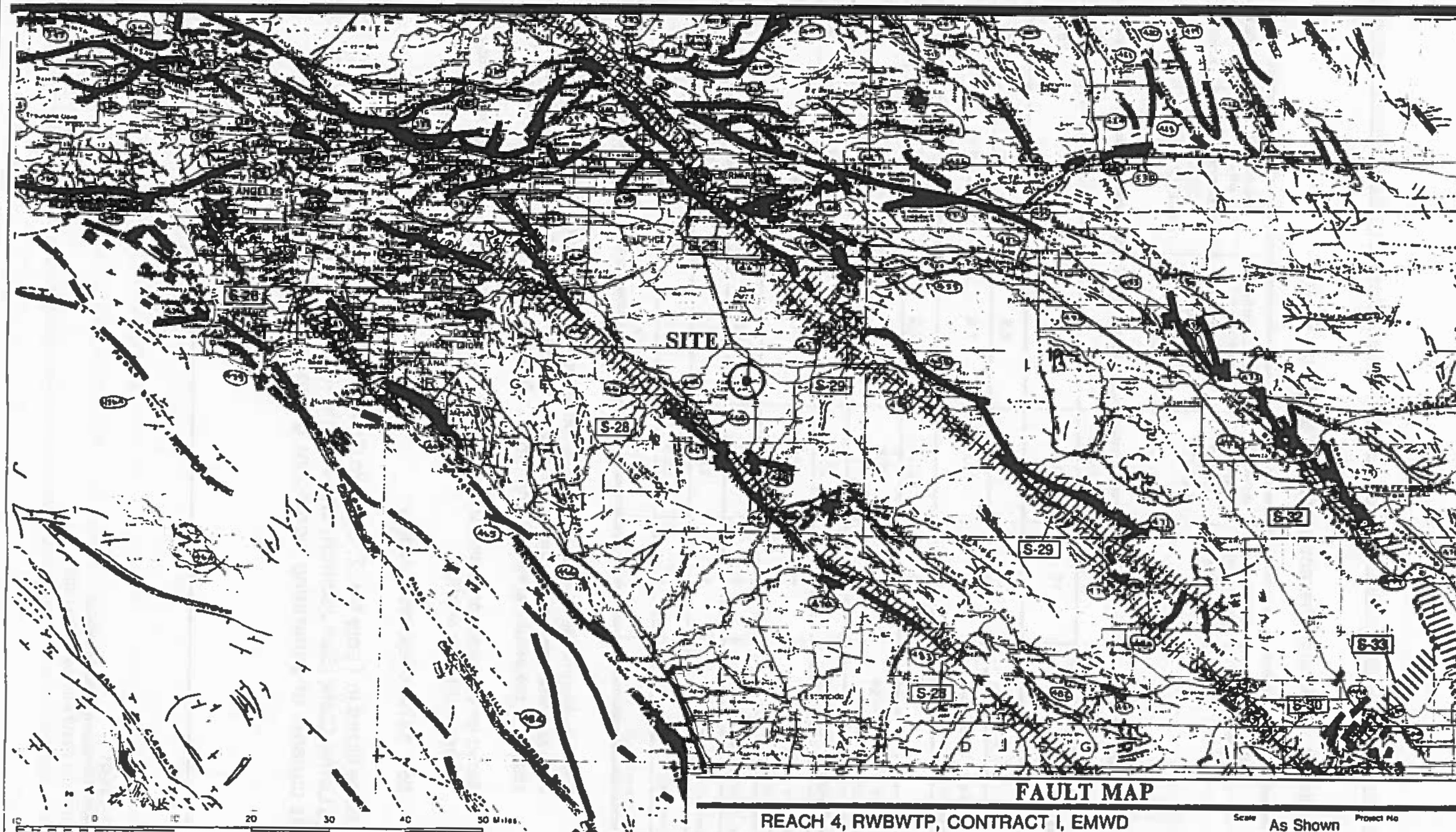
4.2.5 Artificial Fill (Map unit af) Artificial fill occurs locally within the alignment associated with road construction and residential development adjacent to the alignment. Artificial fill typically consists of moderately dense to dense mixtures of silty sands and clayey sands. An embankment fill for the Canyon Lake Reservoir is located along the eastern portion of this section of the alignment. This embankment fill consists of an approximately 10-foot high fill slope with a rip-rap cover.

5.0 FAULTING & SEISMICITY

5.1 Faulting

There are no known active faults projecting toward or extending across the alignment. The alignment is not situated within a currently designated State of California Alquist-Priolo Special Studies Zone. The nearest known active fault zones are the Elsinore, Casa-Loma (San Jacinto) and Whittier-North Elsinore fault zones, located approximately five, six and 17 miles west, north and northwest of the site, respectively. These distances are measured from the center of the alignment.

The maximum credible earthquake is the maximum seismic event that a particular fault is theoretically capable of producing, based upon existing geologic and seismologic evidence. The maximum credible event does not imply that an earthquake of that magnitude has occurred or will occur along the particular fault but simply implies that the potential for such an earthquake does exist. Maximum credible earthquakes and associated seismic parameters for active faults within a 100-km (62-mile) radius of the site are shown in Figure No. 2, *Fault Map*, and Table No. 2, *Seismic Characteristics of Regional Faults.*



OTHER SYMBOLS

(204) Numbers refer to annotations listed in the Appendices of the accompanying report. Annotations include fault name, age of fault movement, and pertinent references including Special Studies Zone maps where a fault has been zoned by the Alquist-Priolo Special Studies Zone Act of 1972 (amended 1974 and 1975). This Act requires the State Geologist to delineate zones to encompass all potentially and recently active faults.

FAULT MAP

REACH 4, RWBWTP, CONTRACT I, EMWD
City of Canyon Lake, California
for: Black & Veatch

Converse Consultants Inland Empire

Scale	As Shown
Date	10/12/93
Prepared By	LDH
Checked By	WHC
Approved By	QSH
Project No	92-81-504-01
Drawing No	2

MODERN PASADENA BLUEPRINT / 99934

Table No. 2. Seismic Characteristics of Regional Faults

FAULT NAME	MINIMUM DISTANCE TO SITE (miles)	APPROX. TOTAL FAULT LENGTH (miles)	MAXIMUM CREDIBLE EARTHQUAKE MOMENT MAGNITUDE (Mw)[1]	MAXIMUM PEAK HORIZONTAL GROUND ACCELERATION[2] (g)	DURATION OF STRONG SHAKING[3] (sec)
Elsinore	5	125.0	7.5	0.57	29 - 34
Casa Loma - Clark (San Jacinto)	16	62.5	7.5	0.21	26 -31
Whittier - North Elsinore	17	43.8	7.5	0.20	25 - 30
Glen Helen - Lytle Creek-Claremont	18	50.0	7.5	0.19	25 - 30
Chino	19	21.9	7.0	0.14	20 - 25
Hot Spring - Buck Ridge (San Jacinto)	21	43.8	7.5	0.16	24 - 29
San Gorgonio - Banning	24	56.3	8.0	0.17	26 - 31
Offshore Zone of Deformation	32	37.5	7.5	0.10	24 - 29
San Andreas (Southern)	32	118.8	8.0+	0.12	27 - 32
Newport - Inglewood	35	37.5	7.5	0.08	22 - 27
Cucamonga	37	25.0	7.0	0.06	16 - 21
Rose Canyon	38	46.9	7.5	0.08	19 - 24

(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 relation 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.

Of those listed in Table No. 2, *Seismic Characteristics of Regional Faults*, the Elsinore, Casa Loma-Clark (San Jacinto), and Whittier-North Elsinore fault zones are considered most capable of generating significant ground motions at the site.

5.2 Seismicity

The subject site is situated in a seismically active region. As is the case in 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 site.

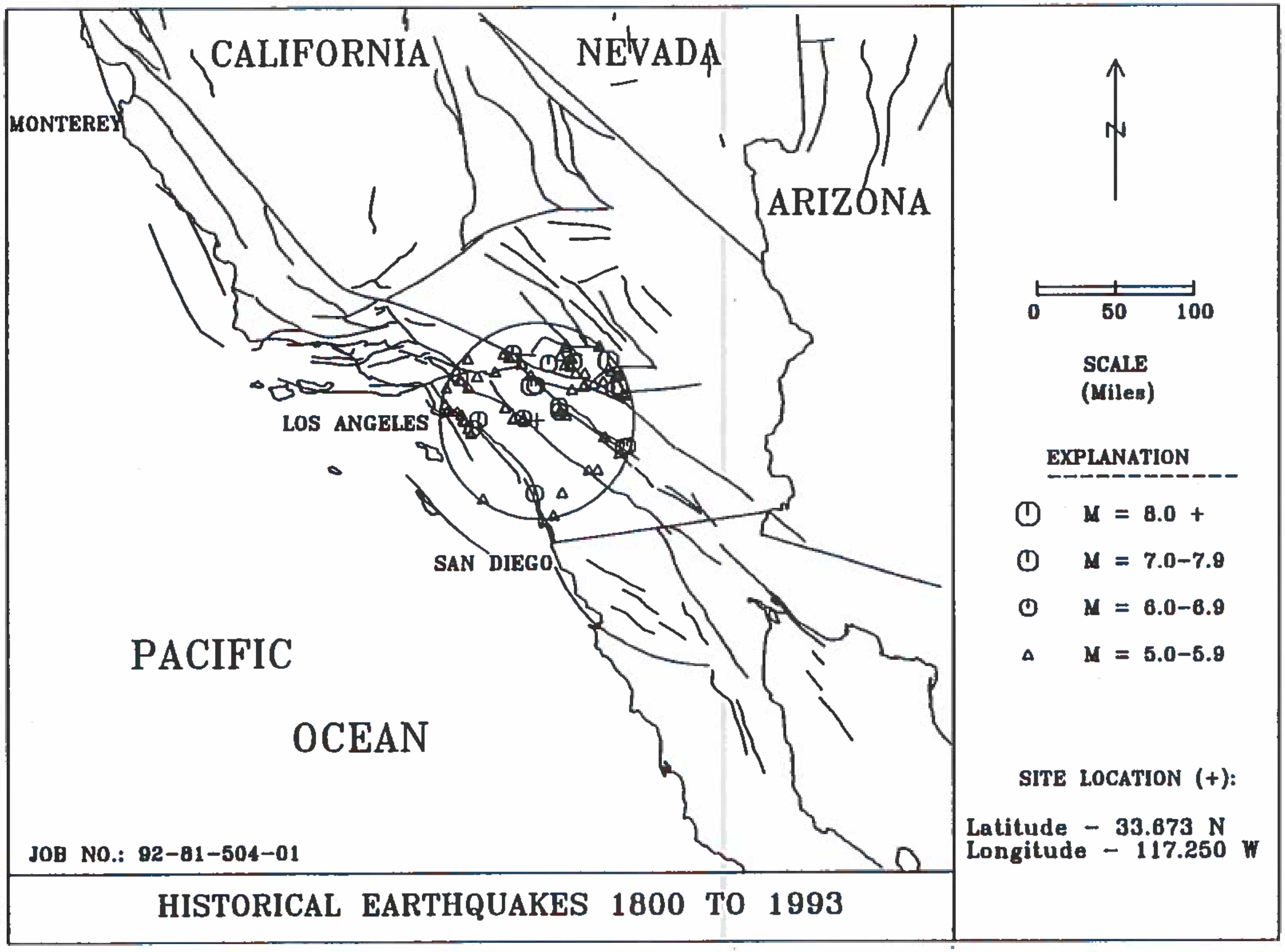
According to the Uniform Building Code (1991 edition), the project site is situated in Seismic Zone 4. Major damage corresponding to intensities VIII or higher on the Modified Mercalli Intensity Scale can be expected within this zone. Seismic Zone 4 also includes those areas that lie within a zone of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

Based on the location of the faults, maximum credible ground acceleration of 0.57g is expected in the vicinity of the alignment. Historical seismic events research within a 100-km (62-mile) radius of the site is illustrated in Figure No. 3, *Earthquake Epicenter Map*. Based on a 100-year return period, a ground acceleration of 0.39g is expected.

6.0 GROUND WATER

The depth of ground water in the area of the proposed pipeline alignment 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 at 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 about 200 feet north of Newport Road and near the Sun City RWRF is about 19 feet below surface. The ground elevation at the location of the well is about 1,408 feet above msl.

Ground water was encountered in the Borings BC-3 and BC-4 at depths of about 17 to 18 feet below existing ground surface. The ground elevation at the two boring locations is 1,477 feet and 1,418 feet, respectively, above msl. The subsurface materials at these depths were composed of decomposed granite and acted as the water bearing strata. The hydrogeologic setting along the proposed alignment is characterized by steep hills on the east side. On the west side of the alignment the ground slopes downward to the reservoir level. During the last rainy season the area received heavy precipitation. It is our opinion that heavy precipitation has resulted in



EARTHQUAKE EPICENTER MAP

REACH 4, RWBWTP, CONTRACT I, EMWD
City of Canyon Lake, California
for: Black and Veatch

Converse Consultants Inland Empire

Project No. 92-81-504-01

Figure No. 3

temporary storage of large quantities of water within the decomposed granite layer. The rate of downward movement of this water is very slow due to low transmissivity of the decomposed granite.

Excavations below about 17 feet from existing surface grade along the alignment may intercept this stored water. Dewatering may be required during the excavation of these portions of the trench. Seasonal variations in the depth of ground water should be expected.

7.0 SEISMIC REFRACTION SURVEY

7.1 General

A seismic refraction survey was performed along approximately 5,500 linear feet of the proposed alignment. The seismic lines (S-3 to S-9) 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 followed and interpretation of the results obtained are included in Appendix C, *Seismic Refraction Survey*.

7.2 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

In general, it is anticipated that trenches for the proposed pipeline(s) will be excavatable using conventional heavy-duty trenching equipment. At locations where relatively unweathered hard bedrock occurs, blasting or use of heavy-duty breakers may be required to fragment the rock prior to excavation. It is the responsibility of the contractor to chose the appropriate equipment and methods of excavation based on the excavation characteristics described herein and on his/her own experience and knowledge of local conditions.

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 more difficult the excavation. Guidelines for evaluating the excavatability of earth materials from seismic P-wave velocity measurements have been published in terms of rippability by Caterpillar Tractor Company (1989), Peurifoy (1979), and Church (1981, 1972). These guidelines have been developed from case histories in which the seismic velocities of various soil and rock types have been compared with equipment performance (tractors and ripper).

In general, rippability of tractor-rippers versus bedrock types can be correlated between tractor-ripper weight/horsepower and P-wave velocities. Church (1981) provides suggested guidelines for rippability and blasting based on tractor-ripper size and seismic velocities, see Table No. 3, *Seismic Velocity Versus Rippability*.

Table No. 3. Seismic Velocity Versus Rippability

Medium-Weight Tractor-Rippers 200- to 300-Engine-HP and 60,000- to 90,000-lb Working-Weight Ranges	
SEISMIC VELOCITY (feet/second)	RIPPABILITY
0 to 1,500 ft/s	No Ripping
1,500 to 3,000 ft/s	Soft Ripping
3,000 to 4,000 ft/s	Medium Ripping
4,000 to 5,000 ft/s	Hard Ripping
5,000 to 6,000 ft/s	Extremely Hard Ripping or Blasting
6,000 ft/s and higher	Blasting

Heavyweight Tractor-Rippers 300- to 525-Engine-HP and 100,000- to 160,000-lb Working-Weight Ranges	
SEISMIC VELOCITY (feet/second)	RIPPABILITY
0 to 1,500 ft/s	No Ripping
1,500 to 4,000 ft/s	Soft Ripping
4,000 to 5,000 ft/s	Medium Ripping
5,000 to 6,000 ft/s	Hard Ripping
6,000 to 7,000 ft/s	Extremely Hard Ripping or Blasting
7,000 ft/s and higher	Blasting

In this report, estimates on the excavatability of the subsurface materials within the areas planned for excavation are based, in part, on CCIE's own field experience and results of the seismic refraction survey presented in Appendix C, *Seismic Refraction Survey*.

Our conclusions regarding excavatability of the proposed pipeline(s) trench are:

- The surficial materials and deeply to moderately weathered granitic bedrock should be excavatable using standard heavy-duty trenching equipment. The seismic velocity range of the surficial materials varies between 1,700 feet per second (fps) to 2,900 fps and 2,900 fps to 4,000 fps for weathered granitic bedrocks.

- Bedrock with seismic velocities greater than 4000 fps may be difficult to excavate with conventional heavy-duty trenching equipment and may require blasting or the use of heavy-duty breakers to fragment the rock prior to excavation.

- Within the sections of the trench requiring excavation below about 13 feet from the existing grade, relatively unweathered nonrippable granitic and metamorphic rocks may be encountered. Blasting may be required at these locations.

Anticipated bedrock excavatability characteristics along the alignment are summarized in Table No. 4, *Anticipated Excavatability Characteristics of Bedrocks*.

Table No. 4. Anticipated Excavatability Characteristics of Bedrocks

APPROXIMATE STATION	BEDROCK TYPE	SEISMIC VELOCITY (feet per second)	ANTICIPATED EXCAVATABILITY
Sta. 480+00 to Sta. 493+00	Weathered Granitic Rock	2820 to 3080	Likely excavatable with conventional heavy-duty trenching equipment
Sta. 493+00 to Sta. 503+00	Weathered Granitic Rock	3300 to 5000	Moderate to difficult excavation with conventional heavy-duty trenching equipment. May require local blasting or breaking where bedrock velocities are in excess of 4000 fps.
Sta. 503+00 to Sta. 531+00	Weathered Granitic Rock	up to 3680	Likely excavatable with conventional heavy-duty trenching equipment.
Sta. 531+00 to Sta. 545+00	Weathered Granitic Rock	2650 to 6900	Generally excavatable with conventional heavy-duty trenching equipment with local isolated hard rock areas. Blasting or breaking may be required locally.
Sta. 545+00 to Sta. 562+00	Metamorphic Bedrock	3260 to 5850	Likely very difficult excavation throughout most of this section. Will likely require blasting and/or breakers to fragment frequently prior to excavation.

It should be understood that the foregoing discussion and the tables do not imply that excavation with tractor-rippers is the recommended method of trench excavation for the project. Rather, 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.

8.0 LIQUEFACTION EVALUATION

Soil liquefaction can occur during or immediately following a strong ground shaking event because of 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 loose granular soils may exhibit dilation when subjected to shear stress whereas even a very dense granular soil element within a deposit may contract due to applied shear stress if the effective confining stress is high.

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 depend on the static shear 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 bedrock that extends above the elevations of the pipe invert
- The native silty and/or clayey sand materials above the elevation of the pipe invert, as observed during drilling operations and confirmed by laboratory testing, contain significant amount of fines and are not likely to liquefy.

Earthquake damage to buried pipelines can be due to permanent ground deformation or seismic wave propagation. Permanent ground deformation refers to landslide, 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 damages to buried pipelines. These are physical damage mitigation and impact mitigation. Physical damage mitigation includes 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 damages.

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*.

Average and stratum soil resistivity based on field tests ranged from 3,854 ohm-cm to 16,917 ohm-cm. The majority of the measured resistivity values, however, fall within 2,000 to 10,000 ohm-cm, a range representing moderately corrosive soils. Electrical resistivity based on laboratory tests on samples at *in situ* moisture contents as in the mildly corrosive category. When saturated, the measured resistivity increased to mildly corrosive range.

Soil pH values of the two soil samples tested in the laboratory were 6.6 and 8.0. These values indicate that in general, soil corrosivity is not significant. Only trace soluble sulphate as detected in the two soil samples. The measured soluble chloride concentrations were 35 parts per million (ppm) each.

These results indicate that at present the proposed alignment is moderately to mildly corrosive to steel and is not significantly deleterious to concrete structures. However, conditions may change in the future, and to enhance the durability and, hence, the life-time, of the proposed cement-mortar coated steel pipes (CMCSP) and related structures, the corrosion measures recommended in Appendix D, *Soil Corrosivity Study*, should be considered in the design.

10.0 CONCLUSIONS

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

- The subsurface materials along the alignment vary significantly. The site is underlain by weathered granitic bedrock to depths varying from about one foot to 15 feet below existing grade. The overlying dense to very dense materials are composed of fills and native alluvium soils consisting of silty sand and clayey sand with some gravel.

- Disintegrated weathered bedrock materials, free from oversize particles, at the elevation of the pipe invert may be used as the pipe bedding material for most of the proposed alignment. Localized small sections with fill, or native alluvial soils at pipe invert elevations containing materials unsuitable for use as bedding material (SE < 30) may occur. Majority of excavated fill and alluvial soils may be used as backfill for the trench zone.

- Ground water to a depth of about 17 to 18 feet below existing surface grade should be expected along the proposed alignment. Seasonal fluctuations in ground-water levels should be anticipated.

- A dewatering system should be anticipated where ground water is above the proposed pipeline invert.

- Due to the shallow depth to bedrock and the relatively deep ground-water table, liquefaction potential of the subsurface soils along the alignment is considered low.

- Based on our field observations, seismic refraction survey, and laboratory testing, the surficial native soils, fills and deeply weathered bedrock to depths of proposed pipe invert should be excavatable with conventional heavy-duty trenching equipment. Difficult to marginally excavatable metamorphic bedrock occurs from approximate Station 545+00 to Station 562+00. Small local sections of hard ripping requiring additional effort such as a pneumatic hammer or blasting may be encountered. Trench excavation below about 13 feet from existing grade may require blasting.

- The pipeline alignment is classified as moderately to mildly corrosive to ferrous metals and is not significantly deleterious to concrete.

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 soil unit weight, angle of internal friction, coefficient of active earth pressure, and coefficient of friction at the interface between the backfill soils and native soils.

The following list provides recommended values of the various soil parameters that the pipe design engineer may use for the design of the subject pipelines:

- Soil bulk unit weight — $\gamma = 125$ pcf
- Angle of internal friction of soils — $\phi = 37$ 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 = 0.20-25

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.

11.2 Uplift Pressures

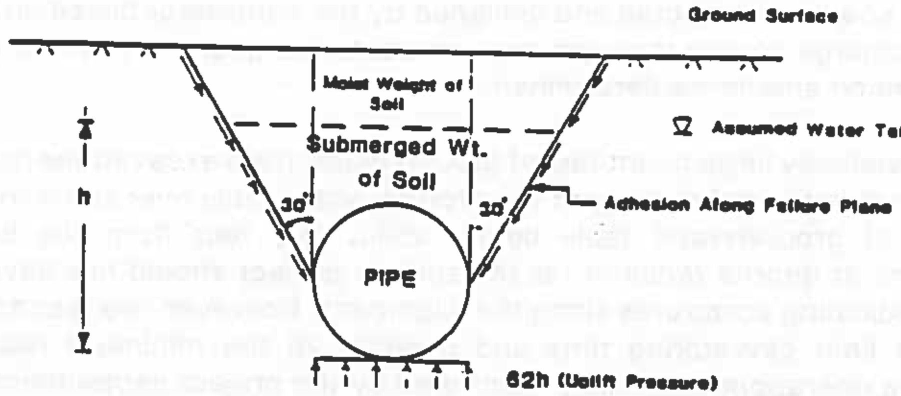
We understand that the majority of the proposed pipeline invert elevations will range from approximately eight to 10 feet below existing ground surface. For the few locations where the invert depths are more than about 17 feet, listed in Table No. 1, *Locations with Depth of Invert Greater Than 10 Feet Below Grade*, the pipe may be subjected to uplift pressures. We recommend that the performance of the pipes at these locations be checked against hydrostatic uplift force.

The hydrostatic uplift pressure acting on the pipe is shown in Figure No. 4, *Schematic Uplift Pressure Diagram*. The hydrostatic uplift force in pounds per foot length of the pipe, P_{uplift}, can be obtained from the following expression,

$$P_{uplift} = 62hD$$

where D is the diameter of the pipe in feet.

The shearing resistance between soils along the assumed failure plane, shown in Figure No. 4, *Schematic Uplift Pressure Diagram*, can be considered to resist the uplift force. Either an average soils adhesion of 250 psf or the total weight of the soil block within the failure planes should be used to estimate design uplift resisting force. The total weight for the block should be calculated based on a moist unit weight of 110 pcf for the soils above ground-water table and submerged unit of 125 psf for soils below ground-water table.



Either the soil adhesion or the Total Weight
should be used to resist uplift pressures.

Figure No. 4, *Schematic Uplift Pressure Diagram*

Possible measures to provide additional pipeline restraint against uplift pressures include the following:

- Use of concrete encasement to increase the dead weight of the pipe.

- Restraining the pipeline by use of helical anchors.

11.3 Excavation Dewatering

Excavations below about 17 feet from the existing surface, at the locations listed in Table No. 1, *Locations with Depth of Invert More Than Ten Feet Below Grade*, may encounter ground water. The ground-water bearing strata is composed of decomposed granite. Rock masses owe their permeability mainly to the joints and fissures. As a result, it is difficult to estimate *in situ* permeability from laboratory test specimens. Field pumping tests must be performed to estimate *in situ* permeability. It should be noted that the presence of joints and fissures also makes the *in situ* determination of permeability very erratic. Therefore, large variations in the value of the permeability may occur.

Based on the above considerations, we recommend that field pumping tests be performed prior to the design of a dewatering system. The method of dewatering to be utilized should be selected and designed by the contractor based on field pumping test. A discharge source that can accommodate the quantity of water pumped from the excavation should be determined.

Pumping relatively large quantities of ground water from excavations has been known to result in significant lowering of the ground-water table over substantial distances. Lowering of groundwater table below about five feet from the bottom of the excavations at depths required for the subject project should not have any adverse effect to adjoining structures along the alignment. However, we recommend that the contractor limit dewatering time and quantity to the minimum required and the dewatering operations be closely monitored by the project geotechnical consultant.

11.4 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, the following allowable bearing capacities are recommended:

- Footings founded on decomposed granite = 4,500 psf
- Footings founded on native and/or compacted soils = 2,500 psf

11.5 Lateral Earth Pressure on Subterranean Walls

Seismic Pressure

For seismic design of buried walls resisting horizontal forces from granular backfill, lateral pressure distribution in the form of an inverted triangle, with the maximum pressure of 30H pounds/ft^2 should be applied against the wall, see Figure No. 5, *Recommended Seismic Earth Pressures,* where H is the height of the wall in feet.

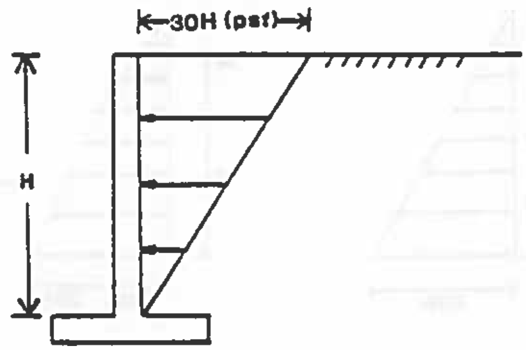


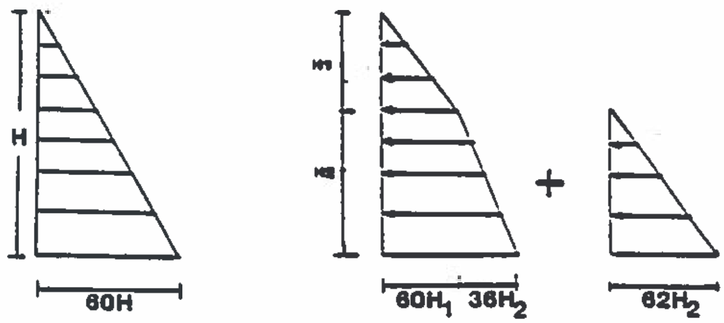
Figure No. 5, *Recommended Seismic Earth Pressures*

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 level, an earth pressure developed by a fluid with a density of 88 pcf may be used for design of cantilevered 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.

Unyielding Walls

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



a) Drained b) Undrained

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

In addition to the recommended lateral earth pressures, the portions of the walls within 10 feet below grade adjacent to any roadway 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.

11.6 Resistances 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. A 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 above water table and poured against undisturbed or compacted on-site drained soils. The corresponding value for passive resistance from the soils below water table may be assumed to be $(300H_1 + 150H_2)$, as shown in Figure No. 7, *Lateral Resistances Due to Passive Earth Pressures*.

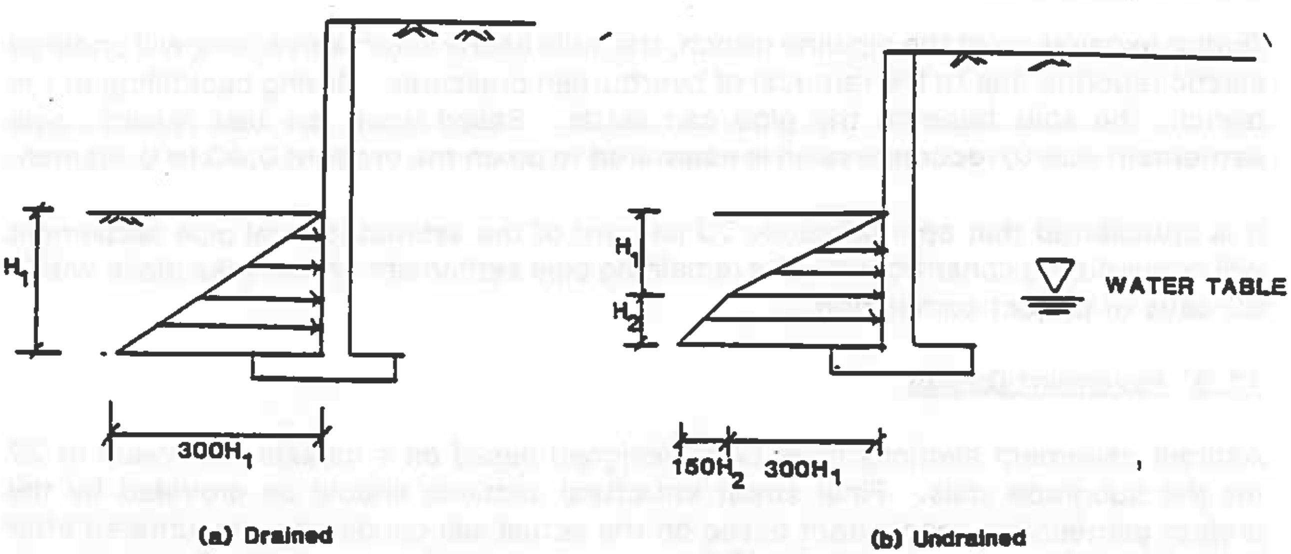


Figure No. 7, *Lateral Resistances Due to Passive Earth Pressures*

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 resistances and the passive resistances of the soils may be combined without reduction in determining the total lateral resistance.

11.7 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 resistances can be combined for the design of anchors and thrust blocks.

11.8 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 0.40 to 0.60 inch.

It is anticipated that approximately 70 percent of the estimated total pipe settlement will occur during construction. The remaining pipe settlement should take place within 60 days of project completion.

11.9 Pavement Design

Asphalt pavement sections have been designed based on a measured R-value of 27 for the subgrade soils. Final street structural sections should be provided by the project geotechnical consultant based on the actual soil conditions encountered after pipeline construction. For preliminary design purposes, recommended pavement sections corresponding to different Traffic Indexes (TIs) are illustrated in Table No. 5, *Recommended Flexible Pavement Sections (A.C. Over Base)*. It is our understanding that the city of Canyon Lake will provide the design of the final pavement section.

Table No. 5. Recommended Flexible Pavement Sections (A.C. Over Base)

(R-value = 27) Railroad Canyon Road		
Traffic Index	Base (inch)	A.C. (inch)
8	10	6
9	12	7
10	14	8
11	16	9

The final thickness of new sections should match with the existing pavement thickness. At least the upper 12 inches of the pavement subgrade areas should be excavated, moisture conditioned and recompacted to 95 percent of the modified proctor maximum density.

Base materials should conform with Section 200-2.2, "Crushed Aggregate Base," of the Standard Specifications for Public Works Construction (SSPWC) and should be placed in accordance with Section 301.2 of the SSPWC. Asphalt concrete materials should conform with Section 203 of the SSPWC and should be placed in accordance with Section 302.5 of the SSPWC (1978).

11.10 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 26 feet would be required at certain section of the alignment as shown in Table No. 1, *Locations with Depth of Invert Greater Than 10 Feet Below Existing Grade.*

Based on our field observations, seismic refraction survey, and laboratory testing, majority of trench excavation should be possible with conventional heavy-duty trenching equipment, except for the areas indicated in Table No. 4, *Anticipated Excavatability Characteristics of Bedrocks*.

Suggested guidelines for pipe bedding and trench zone backfill are included below.

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.

We recommend that a granular material with a sand equivalent (SE) greater than 30 be used for bedding material. The sand equivalent of the weathered bedrock material at the elevation of the pipe invert disintegrated due to drilling was about 45. The native fill and alluvial soils have sand equivalent of about 13 and should not be used as pipe bedding. The majority of bedrock materials at pipe invert elevations are expected to disintegrate during excavation. These disintegrated bedrock materials, separated from oversize particles (greater than 3 inches), appear to be suitable for use as pipe bedding.

The excavated *in situ* materials along the alignment should be inspected and sand equivalent determined, if deemed necessary, by the project geotechnical consultant during construction to ascertain suitability as bedding.

If imported materials are used as bedding material, migration of fines from the surrounding soils must be considered. We recommended that the following gradation criteria be satisfied by the selected imported bedding materials:

$$D_{15} < 1.5 \text{ mm and } D_{50} < 19.5 \text{ mm},$$

where, D_{15} and D_{50} represent particle sizes of the bedding material corresponding to 15 percent and 50 percent passing by weight, respectively. The maximum particle size of bedding materials should not exceed three inches.

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 due to low permeability of the subsurface materials. Long-term accumulation of water in the pipe trench from any sources should be avoided, and trenches should be pumped dry if water collects in the trench.

12.3 Trench Backfill

Backfill for the remainder of the trench above pipe bedding (Trench Zone) should be placed in lifts as recommended in Appendix E, *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 passing No. 4
- 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, as specified in Appendix E, *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, to a depth of at least 12 inches below the pavement section.

13.0 CONSTRUCTION RECOMMENDATIONS

13.1 General

Site soils should be excavatable using conventional heavy-duty excavating equipment. Areas of bedrock will be readily excavatable to marginally rippable with some areas requiring localized blasting. Sloped trench excavations may not be feasible along the majority of the alignment due to proximity of existing utility lines, roadway or structures. Recommendations pertaining to temporary excavations and temporary shoring design are included in sections 13.2, *Temporary Excavations* and 13.3, *Temporary Shoring*, respectively.

13.2 Temporary Excavations

Based upon 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. 6, *Slope Ratios for Temporary Excavations*.

Table No. 6. 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 due to equipment 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 on Figure No. 8, *Recommended Lateral Earth Pressures for: (a) Cantilevered, (b) Braced Shoring*. 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.

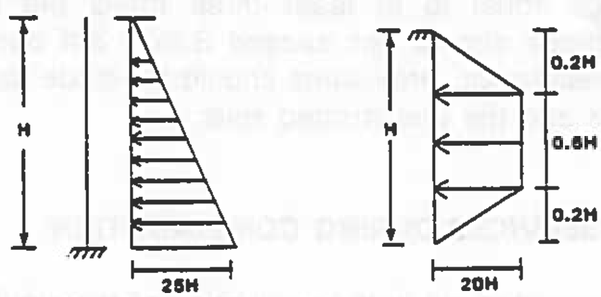


Figure No. 8, *Recommended Lateral Earth Pressures for:*
(a) Cantilevered, (b) Braced Shoring

The design earth pressures indicated in Figure No. 8, *Recommended Lateral Earth Pressures for: (a) Cantilevered, (b) Braced Shoring* are only for temporary shoring.

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 for piles and may be doubled for piles that are placed at center-to-center spacings equal to at least three times the pile diameter. 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 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 determine 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.

REFERENCES

Advisory Notes on Lifeline Earthquake Engineering, 1983, Technical Council on Lifeline Earthquake Engineering, ASCE, New York, N.Y., 232.

California Division of Mines and Geology, Geologic Map of California, Santa Ana Sheet, 1986, Scale 1:250,000.

California Division of Mines and Geology, Preliminary Fault Activity Map of California, 1992, DMG Open-File Report 92-03.

California Division of Mines and Geology, 1988, State of California, Special Studies Zones, Elsinore Quadrangle, Effective March 1, Scale 1:24,000.

Caterpillar Tractor Co., 1979, Caterpillar Performance Handbook, Tenth Edition, Peoria, Ill.

Church, C. E., 1972, 433 Seismic Excavation Studies: What They Tell About Rippability, Roads and Streets.

Church, H. K., 1981, Excavation Handbook, McGraw-Hill, New York.

Engle, René, "Geology of the Lake Elsinore Quadrangle California" California Division of Mines and Geology Bulletin 146, 1959.

Greensfelder, R., 1974, Maximum credible rock accelerations from earthquakes in California: California Division of Mines and Geology, Map Sheet 23.

International Conference of Building Officials, 1991, Uniform Building Code (UBC).

Marachi, N. D., and Dixon, S. J., 1972; A method for evaluation of seismicity: Proceedings of the International Conference on Microzonation, October, 1972, Seattle, Washington.

Mauchin, L. and Jones, A. L., 1987; Peak acceleration from maximum credible earthquakes in California: CDMG Open File Report (in program), p. 1-79.

Morton, D., 1993, Verbal Communication, Eastern Municipal Water District.

Morton, D. M., 1991; "Geologic Map of the Romoland 7.5-minute Quadrangle, Riverside County, California" USGS Open-file Report 90-701.

O'Rourke, M. and Ayala, G., 1993; Pipeline Damage Due to Wave Propagation, Journal of Geotechnical Engineering, ASCE, Vol. 119, No. 9, pp. 1490-1498.

Peurifoy, R. L., 1979, Construction Planning, Equipment and Methods, Third Edition, McGraw-Hill.

NAVFAC, May 1982, Soil Mechanics Design Manual 7.1.

Sharp, R. P. 1976, "Geology Field Guide to Southern California" (Revised Edition) Published by Kendall/Hunt, Dubuque, Iowa.

Sherard, J. L., Dunnigan, L. P. and Talbot, J. R., 1984, Basic Properties of Sand and Gravel Filters, Journal of Geotechnical Engineering, ASCE, Vol. 110, No. 6, June, pp. 684-700.

Slemmons, D. B., 1977; State-of-the-art for assessing earthquake hazards in the United States," U.S. Army Engineer Corps, Report 6, contract DACW39-76-C-0009, 129 p., appendix 37 p.

United States Geological Survey, 1982, Lake Elsinore Quadrangle, California, Riverside County, 7.5-Minute Series (Topographic) Scale 1:24,000.

United States Geological Survey, 1979, Romoland Quadrangle, California, Riverside County, 7.5-Minute Series (Topographic), Scale 1:24,000.

Waring, Gerald A., 1919, Groundwater in the San Jacinto and Temecula Basins, California: U.S. Geological Survey Water-Supply Paper 429, 113 pp.

Winterkorn, Hans F., and Fang, Hsai-Yang, Foundation Engineering Handbook, edited, 1975, Van Nostrand Reinhold Company Inc.

Ziony, J. I. and Jones, L. M., "Map Showing Late Quaternary Faults And 1978-84 Seismicity of the Los Angeles Region, California, 1989, MAP MF-1964.

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 I" drawings prepared by Black & Veatch.

Four 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 (2.42-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 required to be 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-4. The boring summary sheets also 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-5.

Log of Boring No. BC- 1

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

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

Ground Surface Elevation: 1319 ft. Depth to Water: 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.			7	5	101	ma,r
5					7	10	105	
								max se
10					24	12	109	c,ds
		BEDROCK: weathered metamorphic rock, possible metasandstone, white.						
15					28	18	105	
		End of boring at 15 feet. Boring backfilled.						

Converse Consultants Inland Empire

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

Project No. 92-81-504-01

Drawing No. A-1

Log of Boring No. BC- 2

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

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

Ground Surface Elevation: 1497 ft. Depth to Water: 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): with gravel, fine to coarse grained, brown.			33	8	—	
5		- trace clay below 4 feet			22	10	—	
10		- up to 3" gravel below 10'			41	7	—	
15		- moist below 14 feet			31	9	—	
		End of boring at 14 feet. Boring backfilled.						

Converse Consultants
Inland Empire

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

Project No.
92-81-504-01

Drawing No.
A-2

Log of Boring No. BC- 3

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

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

Ground Surface Elevation: 1477 ft. Depth to Water: 18 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 to medium grained, brown.			21	9	117	
5		- fine to coarse grained, below 4 feet			15	6	115	
		BEDROCK: weathered granite, greenish-brown.						
10					79	3	120	ma
15					139	4	120	
20					124	10	124	
25					104	9	133	
		End of boring at 25 feet. Boring backfilled.						

Converse Consultants Inland Empire

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

Project No. 92-81-504-01

Drawing No. A-3

Log of Boring No. BC- 4

Dates Drilled: 9/1/93 | Logged by: WNP | Checked by: OSH

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

Ground Surface Elevation: 1418 ft. | Depth to Water: 17 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)	DESCRIPTION	SAMPLES: DRIVE	SAMPLES: BULK	BLOWS/FOOT	MOISTURE (%)	DRY UNIT WT. (pcf)	LABORATORY TESTS
0	3 INCH ASPHALT CONCRETE						
	FILL: SAND (SP): fine to medium grained, with some silt, yellowish.			31	7	120	
	BEDROCK: weathered granitic rock, grayish-brown.						
				73	5	134	
5							max
				47	5	121	se
10							
				75	7	142	ds
15							
17	▽ (water)						
				143	13	120	
20							
				100/4"			
25	End of boring at 25 feet. Boring backfilled.						

Converse Consultants Inland Empire

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

Project No. 92-81-504-01

Drawing No. A-4

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 of 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, RWBWTP, CONTRACT I, EMWD
City of Canyon Lake, California
for: Black & Veatch

Project No. 92-81-504-01

Drawing No. A-5

Converse Consultants Inland Empire

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

Direct Shear Tests

Two direct shear tests were performed on undisturbed samples under field moisture conditions. Three samples contained in brass sampler rings were placed, one at a time, directly in 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-3A and 3B, *Direct Shear Test*.

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 load. 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-4, *Consolidation Test*.

Sand Equivalent Tests

Two representative soil samples were tested for sand equivalent value to evaluate the suitability of the on-site soil for placement as bedding material. Also, sand equivalent values can be used to determine the suitability of the on-site soils for trench backfill by flooding or jetting methods:

BORING NO.	DEPTH (feet)	SOIL DESCRIPTION	SAND EQUIVALENT
BC-1	7-12	Clayey Sand	13
BC-4	10-12	Gravelly Sand with silts	45

R-Value Tests

A bulk sample of the representative soils at the elevation of pavement subgrade was 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.

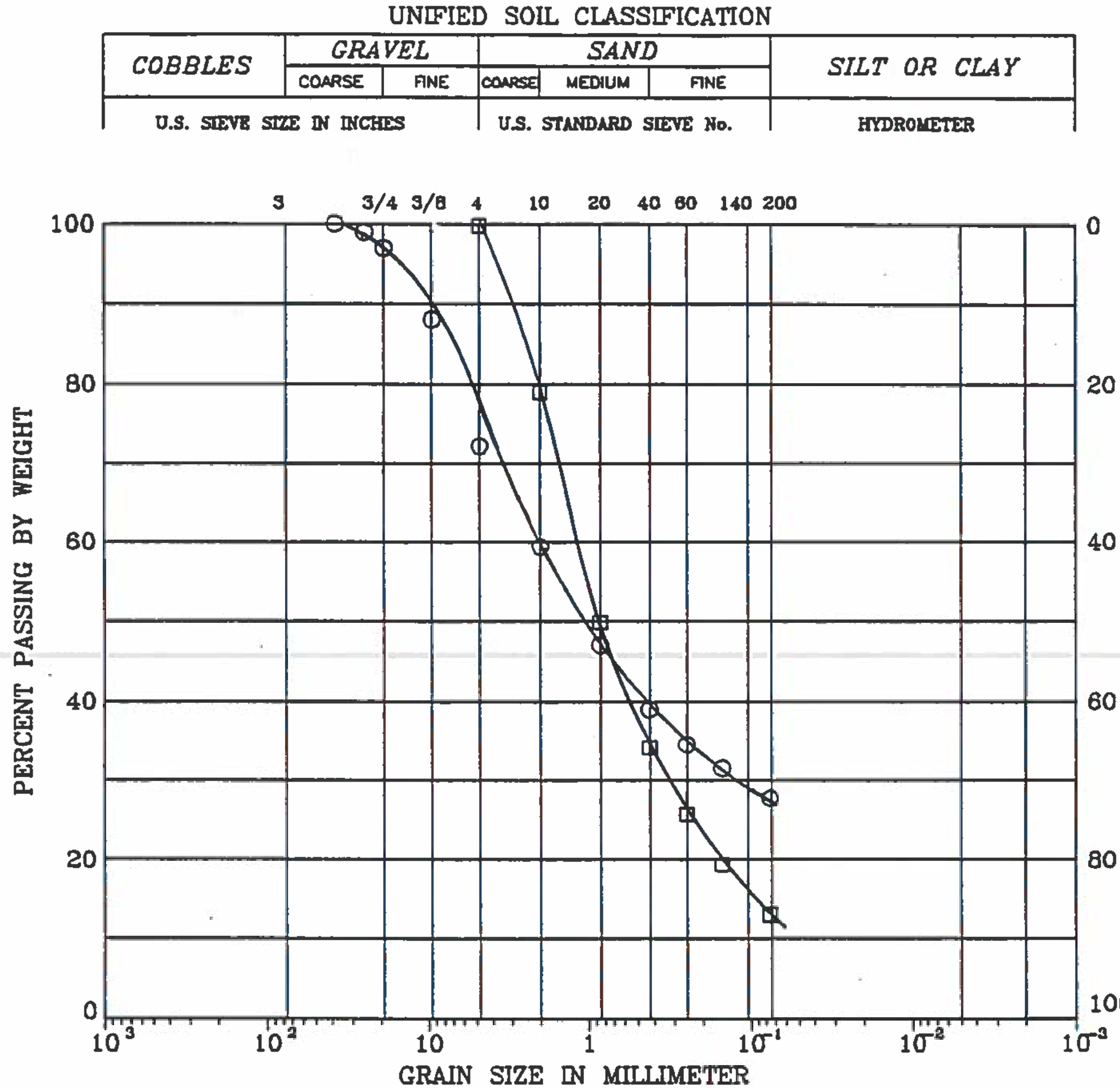
Boring No.	Depth (feet)	SOIL DESCRIPTION	R-VALUE
BC-1	0-5	Silty Sands (SM), Trace Clay, brown	27

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.



SYMBOL	BORING	DEPTH (ft)	LL (%)	PI (%)	DESCRIPTION
○	BC1/Blk1	0–5			Silty Sand (SM)
□	BC3/Blk2	9–14			Silty Sand (SM)

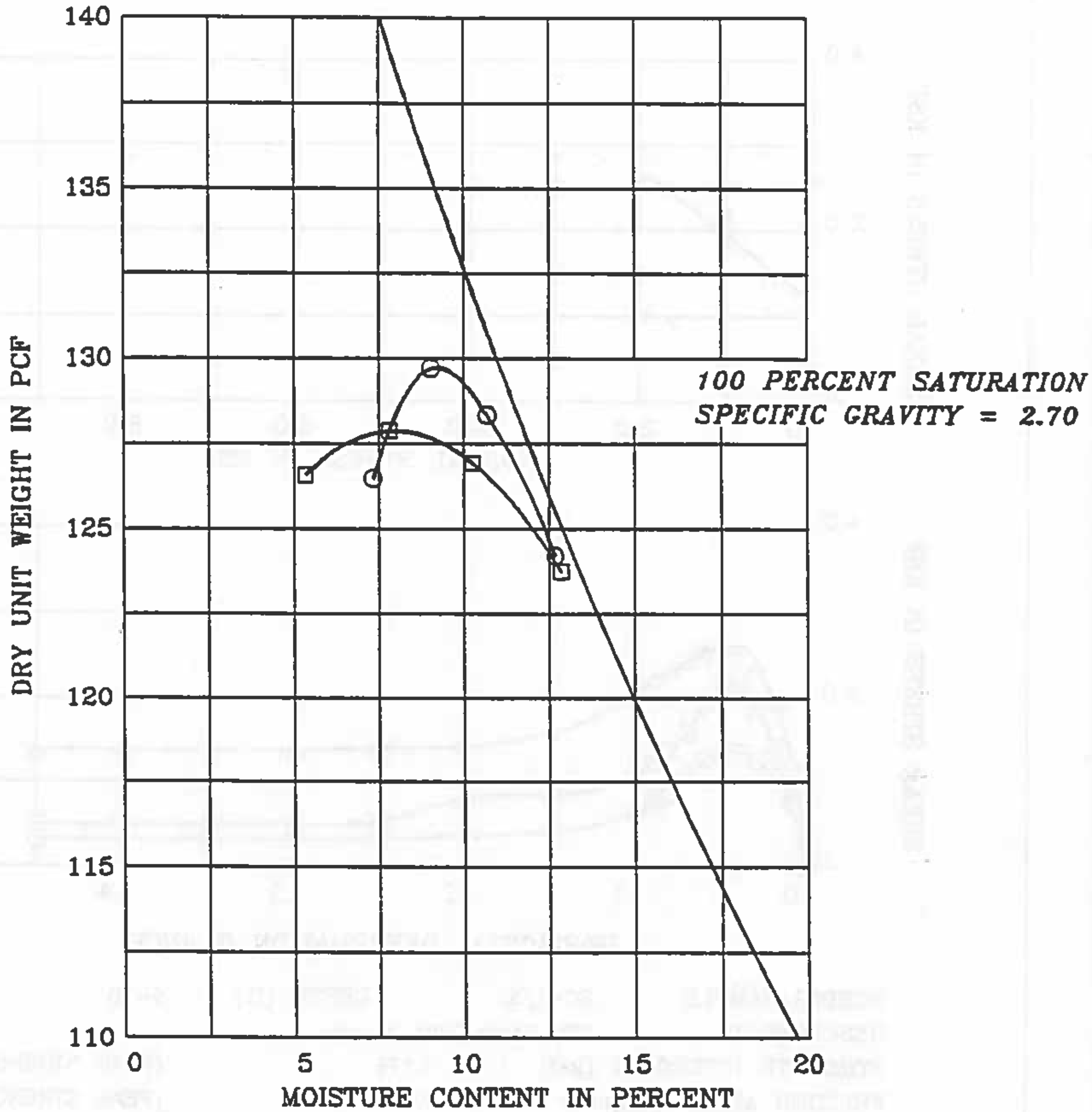
GRAIN SIZE DISTRIBUTION

Reach 4, RWBWTP, Contract I, 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)
○	BC1/Bulk 2	7-12	Silty Sand, w/some Gravel	D1557-91	9.3	129.7
□	BC4/Bulk 2	5-10	Poorly Graded Sand (SP)	D1557-91	7.8	127.9

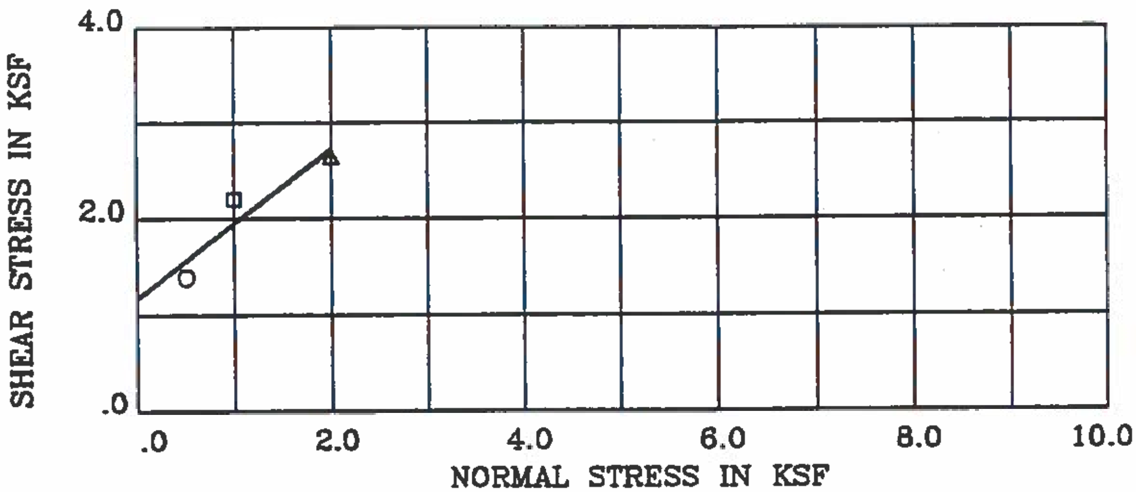
COMPACTION TEST

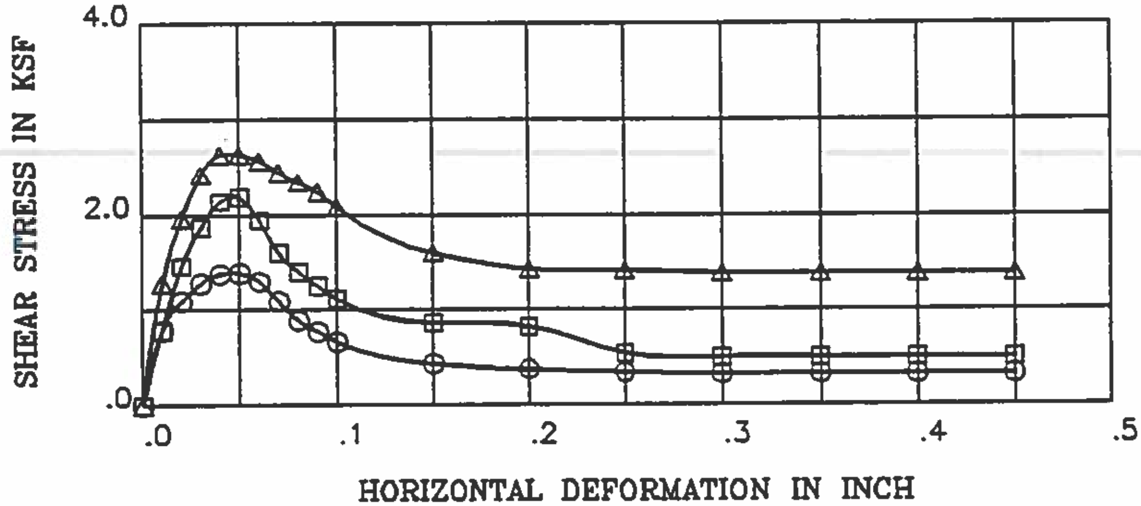
Reach 4, RWBWTP, Contract I, EMWD
For: Black and Veatch

Project No.
92-81-504-01

Converse Consultants Inland Empire

Figure No. B-2





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

DESCRIPTION : Silty Sands (SM), tr clay

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)
O	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	2.6[illegible]	1.39

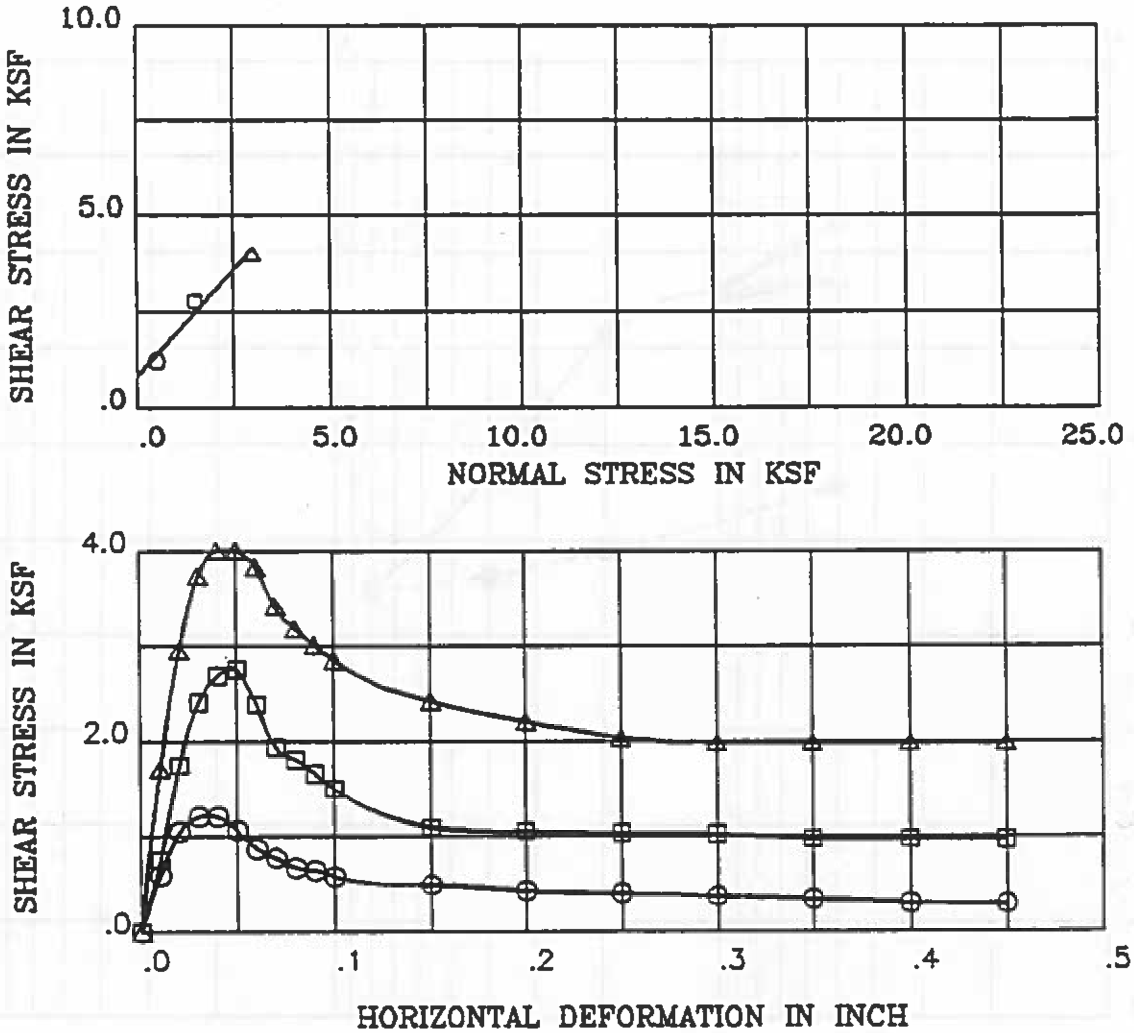
DIRECT SHEAR TEST

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

Project No.
92-81-504-01

Converse Consultants Inland Empire

Figure No. B-3A



BORING/SAMPLE : BC-4/S4 DEPTH (ft) : 14-15

DESCRIPTION : Weathered Granite

STRENGTH INTERCEPT (ksf) : .847 (PEAK STRENGTH)

FRICTION ANGLE (degree) : 47.6 (PEAK STRENGTH)

SYMBOL	MOISTURE CONTENT (%)	DRY DENSITY (pcf)	VOID RATIO	NORMAL STRESS (ksf)	PEAK SHEAR (ksf)	RESIDUAL SHEAR (ksf)
○	13.6	118.2	.426	.50	1.23	.30
□	6.3	135.7	.242	1.50	2.77	.97
△	8.4	130.9	.287	3.00	4.02	1.98

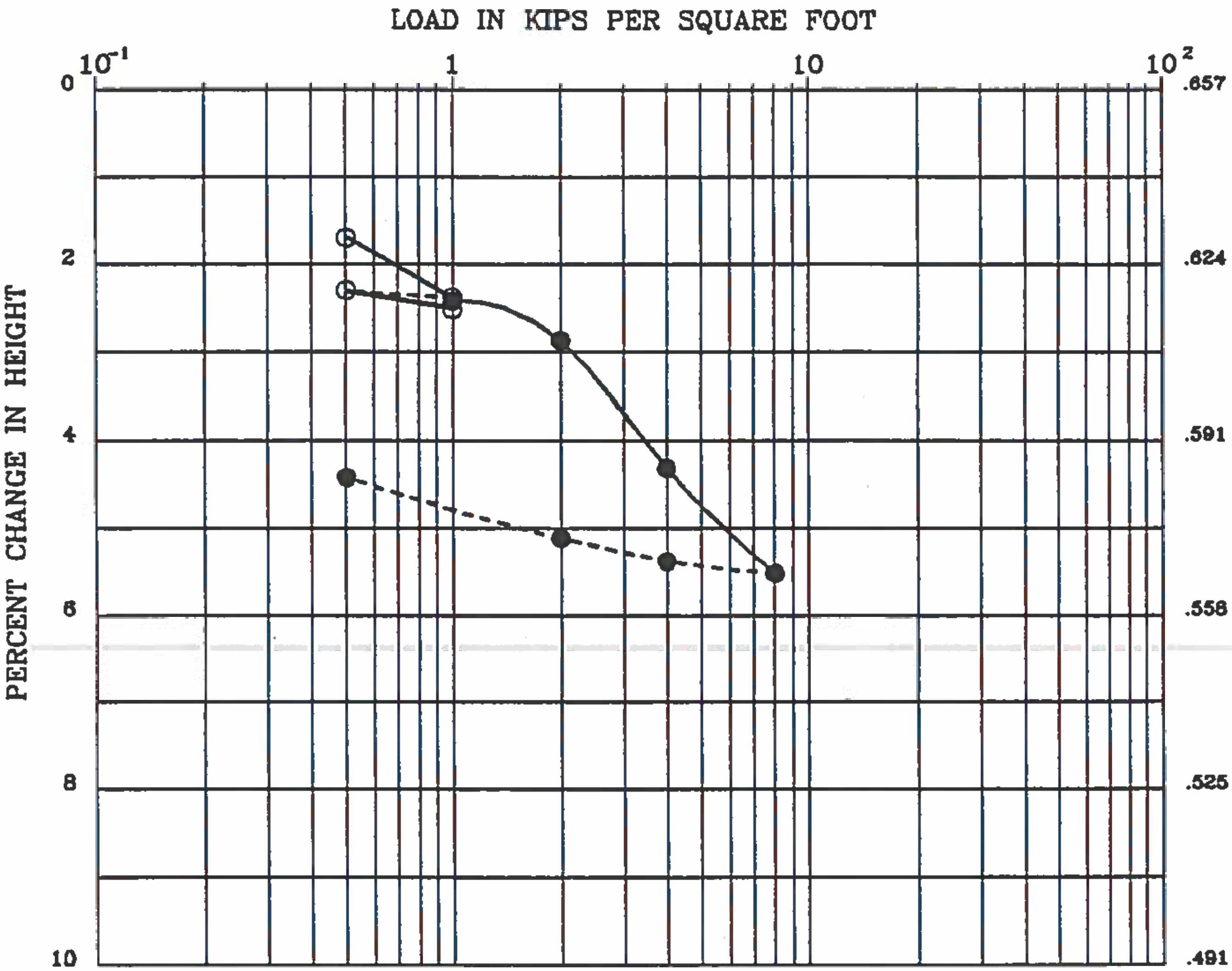
DIRECT SHEAR TEST

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

Project No.
92-81-504-01

Converse Consultants Inland Empire

Figure No. B-3B



BORING	:	BC-1/S3	DESCRIPTION	:	Silty Sand (SM), trace clay
DEPTH (ft)	:	9-10	LIQUID LIMIT	:	
SPEC. GRAVITY	:	2.70	PLASTIC LIMIT	:	

	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 I
For: Black and Veatch

Project No.
92-81-504-01

Converse Consultants Inland Empire

Figure No. B-4

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: Canyon Lake Seismic Survey

This brief letter report is to convey the results of our seismic refraction survey carried out along Railroad Canyon Road in Canyon Lake, 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. Geophone spacing was 20 feet with 20 foot shot offsets. 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.

Survey Design - The seismic lines were laid out mostly along the center island of the roadway. Two small sections were laid out on the north and south sides of the road at the west side of Canyon Lake. The controlling factor that determined the position of the layout, relative to the roadway, was the position of installed steel pipes. A number of utility pipes were previously installed beneath Railroad Canyon Road, and two gas lines were installed just days before our survey was carried out. 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.

Because installed steel pipes, in the final analysis, dictated the position of the layout, other factors determined minor breaks in the line. These, for the most part, are cross streets where traffic could not be shut off. Geophones, along with their cables, could not be operated with constant traffic going over the seismic lines. In one instance, a large excavation and dirt pile dictated a short break. Of

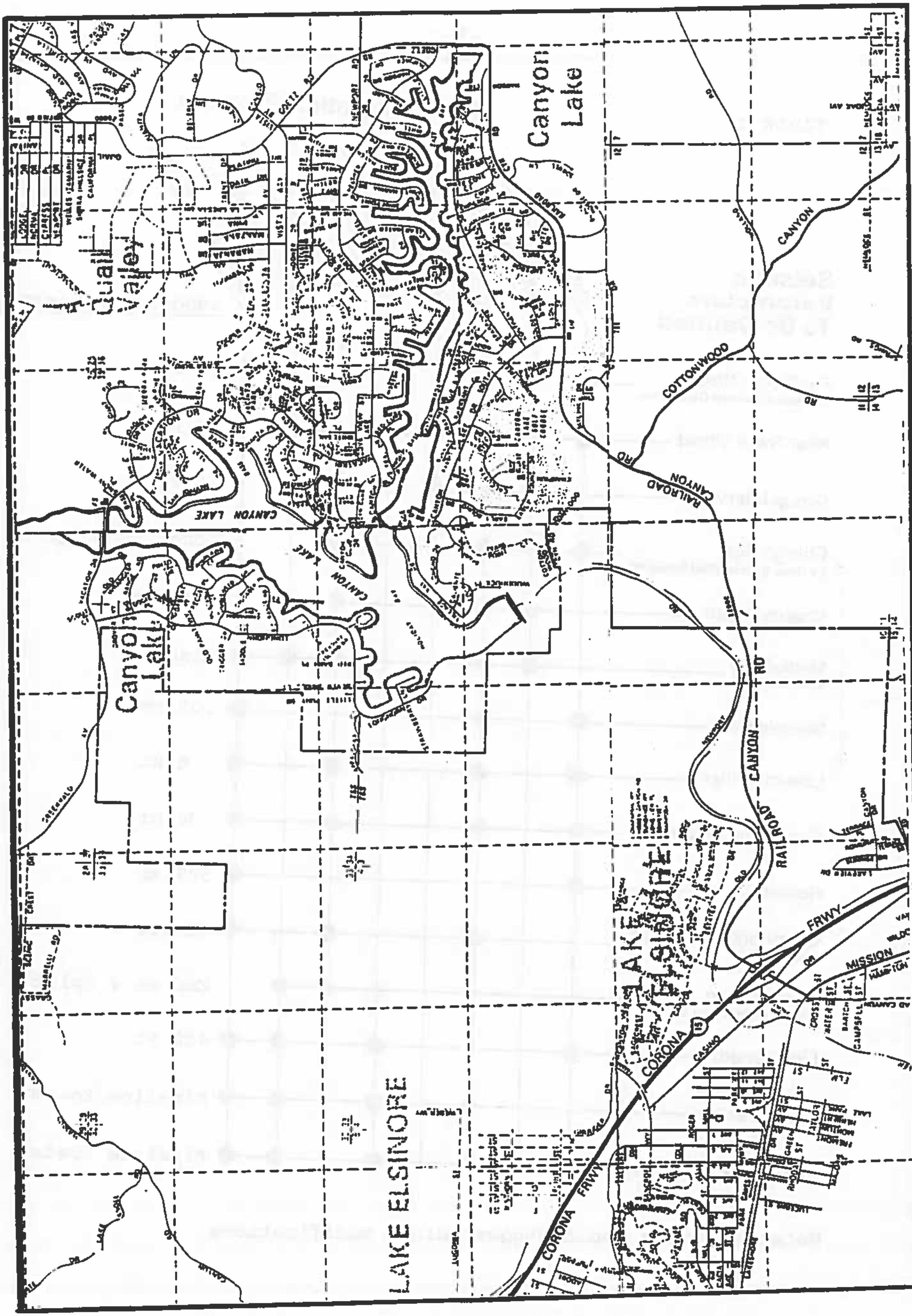

Canyon Lake
Quail Valley
Canyon Lake
LAKE ELSINORE
COTTONWOOD
CANYON
RAILROAD CANYON RD
CORONA FRWY
MISSION

FIGURE 1

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.

course, there were a couple of short breaks 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). Nominally, these are 480 feet long, but a few are shorter, to accommodate the minor breaks aforementioned. A "section" is a continuous line terminated at breaks on each end. The sections may be more than one spread, and can be less than a spread.

The overall seismic line, from Lake Elsinore near I-15 to the east side of Canyon Lake along Railroad Canyon Road, 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 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.

A number of engineering survey station numbers are posted on the seismic section illustrations, for purpose of accurate location of the sections. The location of borehole BC-1, near the east side of the Lake Canyon, is also posted with the annotations at the top of the illustration.

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.

<u>Geologic Setting</u> - 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.

The bedrock, 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 - The presence of steel utility piping was of considerable concern to us, in that the information from the records would not represent ground conditions, if the first arrival energy traveled along the pipe. From review of the informational content of the seismic records, we believe that we were able to keep the line sufficiently far from these pipes, such that the data are not denigrated. Nevertheless, it is manifest that, if the seismic wave traveled along utility piping, it could only increase the measured velocity, and could never decrease it. Thus, if the measured velocities indicate a rippable rock, the real situation could only be better, and never worse, than that reported. Moreover, if the first arrival energy traveled along pipes at a higher velocity than in the native ground, the data would not show deeper layers in the first energy arrival data vs distance. Deeper layers are shown to be present everywhere beneath the line.

The portion of the overall seismic line that falls in the municipality of Lake Elsinore was shot in two sections, and these will be reported separately. Accordingly, all remaining sections are within the limits of Canyon Lake, and they start with section 3 at the west side of town.

It may be stated that the construction crews installing the gas pipelines reported that all rock within their ditching excavations through Canyon Lake was ripped with large catapillar 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.

Most of the route of the seismic line through Canyon Lake is on weathered bedrock. The road bed, however, is highly compacted fill beneath the asphalt, and it, in most places, was laid down directly on weathered bedrock. Its seismic velocity 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 roadbed. Average velocity of this layer is in the order of 2500 ft/s. The range of velocities is about 1700 to 2900 ft/s. These materials, of course, are easily rippable.

The next deeper layer is weathered bedrock, mostly weathered granite clan rocks and locally metamorphosed sediments. Average velocity is 4200 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 the velocity of the surface layer to approximately 6900 ft/s. Its thickness is similarly variable, ranging from about 13 to 55 feet. Generally, this layer is rippable, but locally would respond only to heavy equipment.

The third layer, which extends 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 material is not rippable. This high velocity layer comes to within 25 feet of the surface in several places, and locally to within 12 feet in section 9.

The rippability chart for a D9 Cat (Fig. 2), taken from the Caterpillar Performance Handbook, indicates that granites become non-rippable somewhere in the range 7000 to 8000 ft/s.

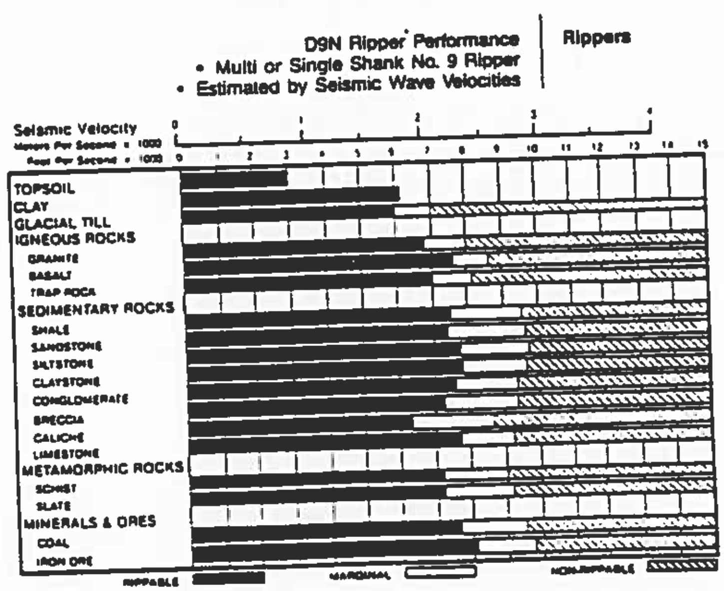


FIGURE 2

Layer 1 is seen to thin to below the thickness of resolution (about 6 feet) in a few places (e.g. in section 5, Fig. 6). Also, layer 1 is recognized just briefly along the line in section 4 (Fig. 5).

The location map (Fig. 3) and the seismic structure sections for line sections 3 through 10 are presented in the Appendix (Figs. 4-13). The engineering survey station numbers locate the section accurately.

Conclusions - The seismic refraction data indicate that granitic bedrock, in the depth and zones investigated within the municipality of Canyon Lake, is rippable to 25 feet and deeper below the surface over most of the route through Canyon Lake. The exception is in section 9 at the east side of town; here non-rippable rocks come to within 12 feet of the surface.

All data acquired in these surveys 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

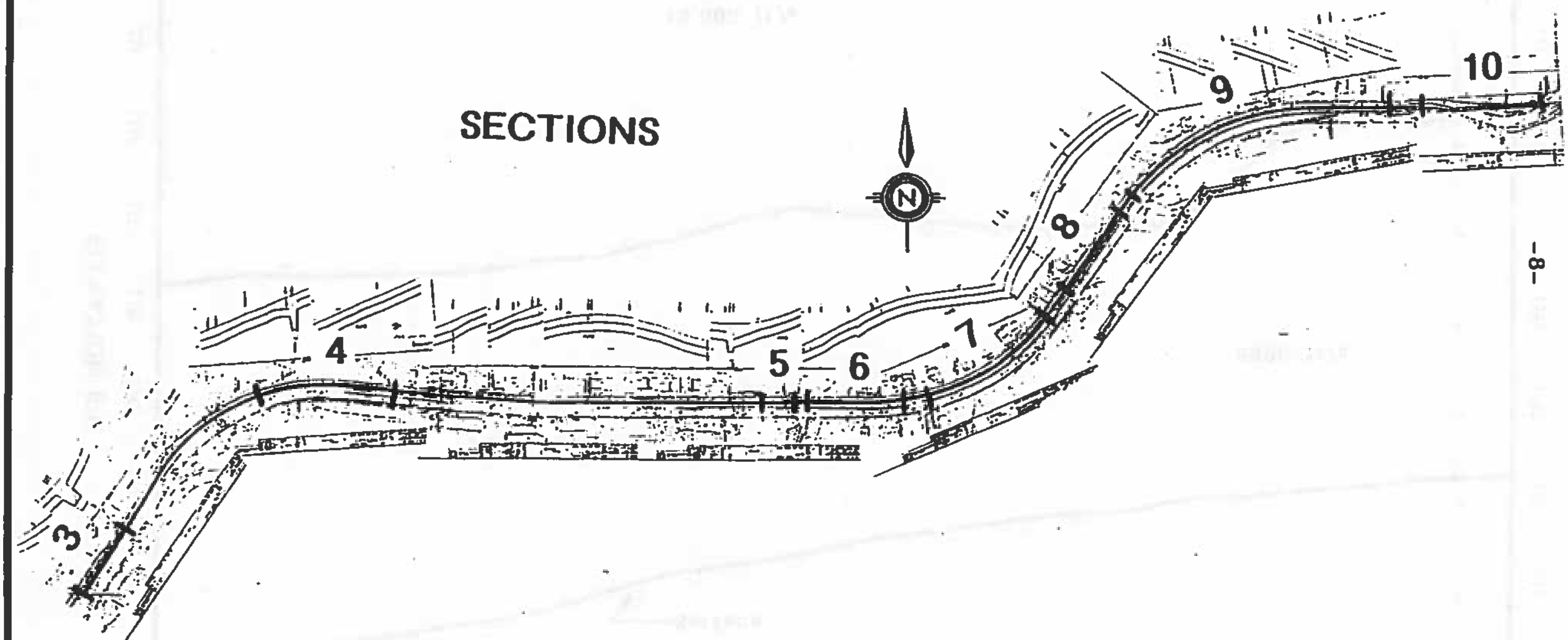


FIGURE 3

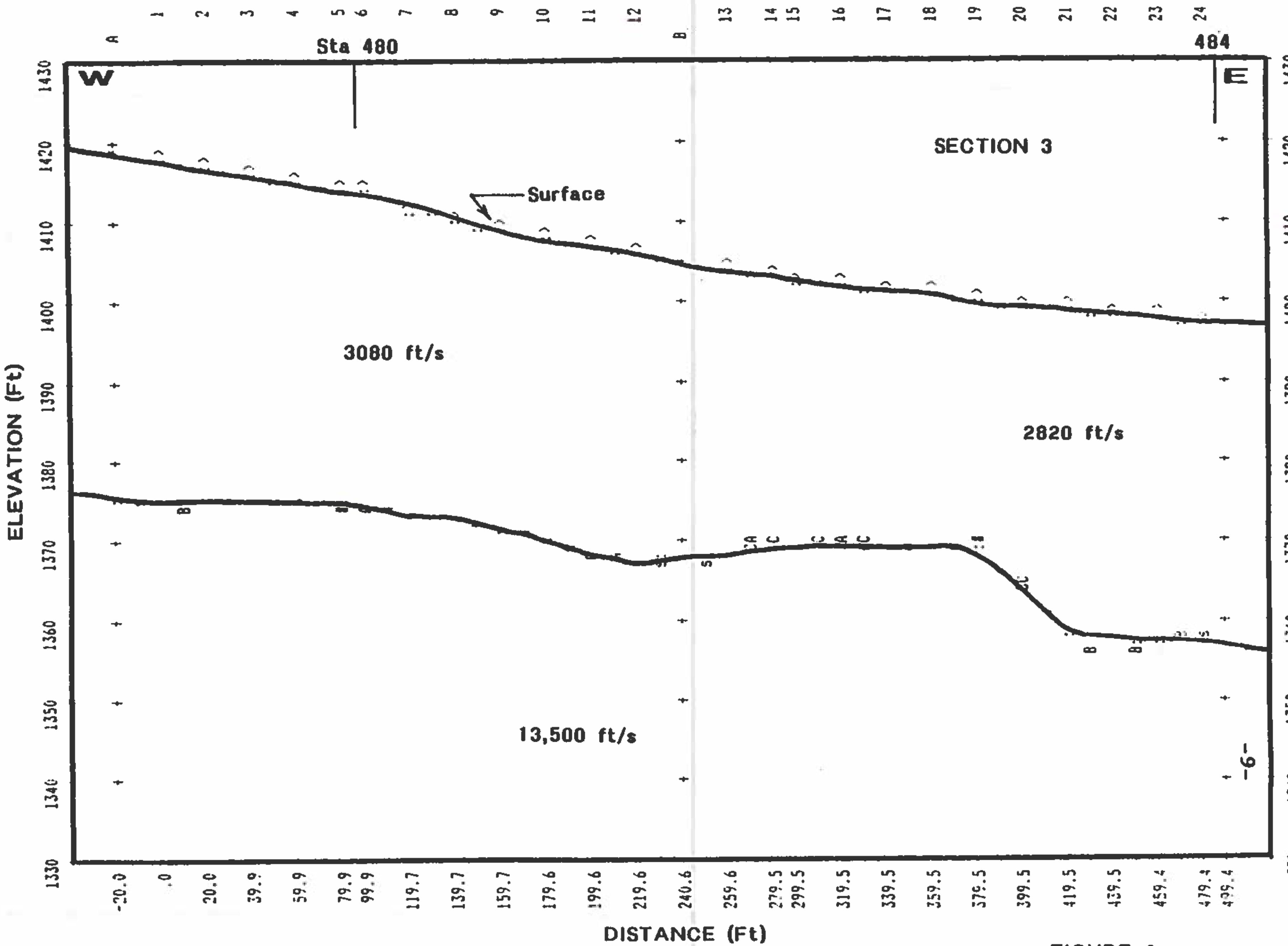

SECTION 3
W
E
Sta 480
484
Surface
3080 ft/s
2820 ft/s
13,500 ft/s
ELEVATION (Ft)
DISTANCE (Ft)
1430
1420
1410
1400
1390
1380
1370
1360
1350
1340
1330
-20.0
0
20.0
39.9
59.9
79.9
99.9
119.7
139.7
159.7
179.6
199.6
219.6
240.6
259.6
279.5
299.5
319.5
339.5
359.5
379.5
399.5
419.5
439.5
459.4
479.4
499.4
A
B
1
2
3
4
5
6
7
8
9
10
11
12
13
14
15
16
17
18
19
20
21
22
23
24

FIGURE 4

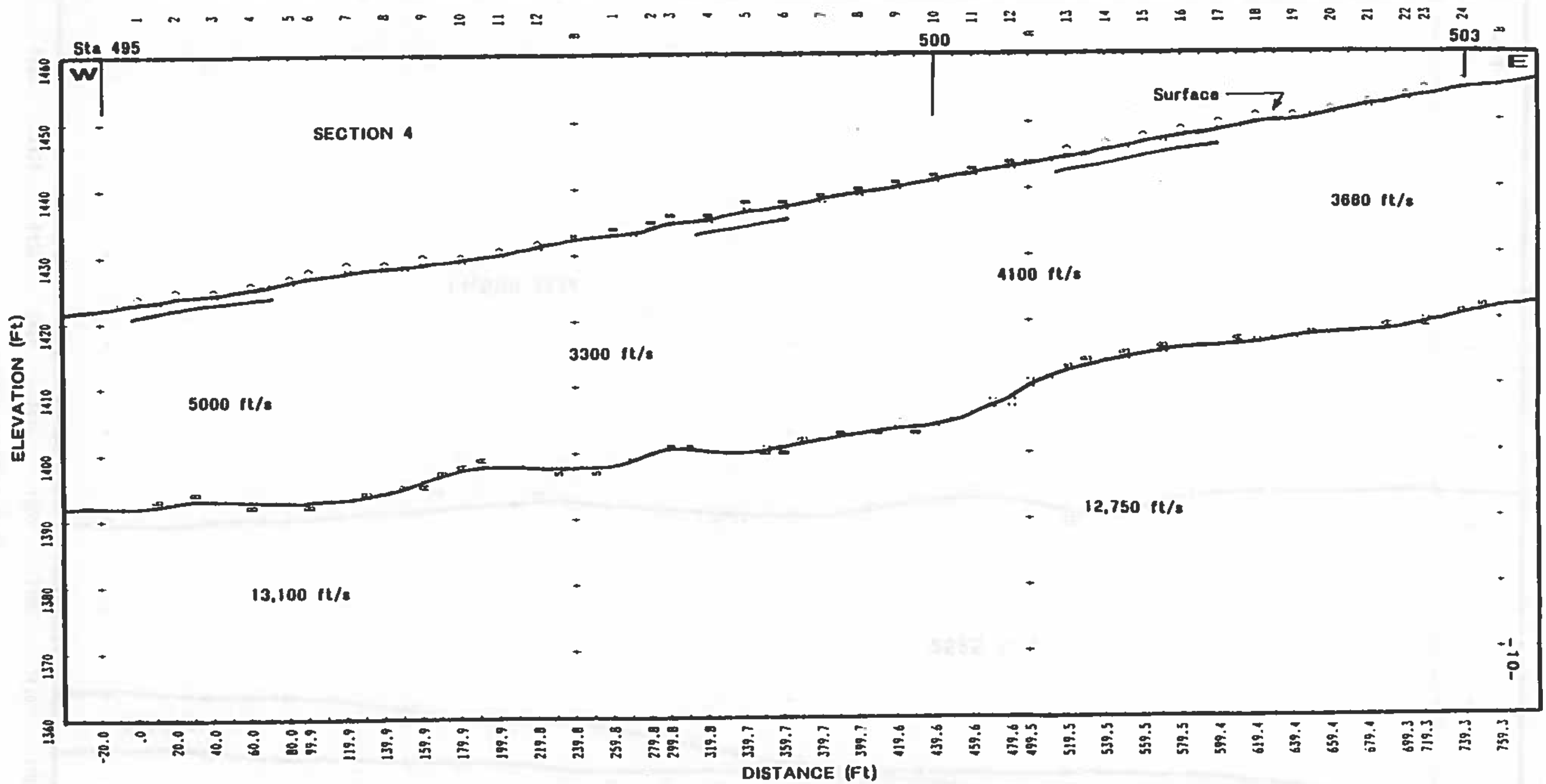
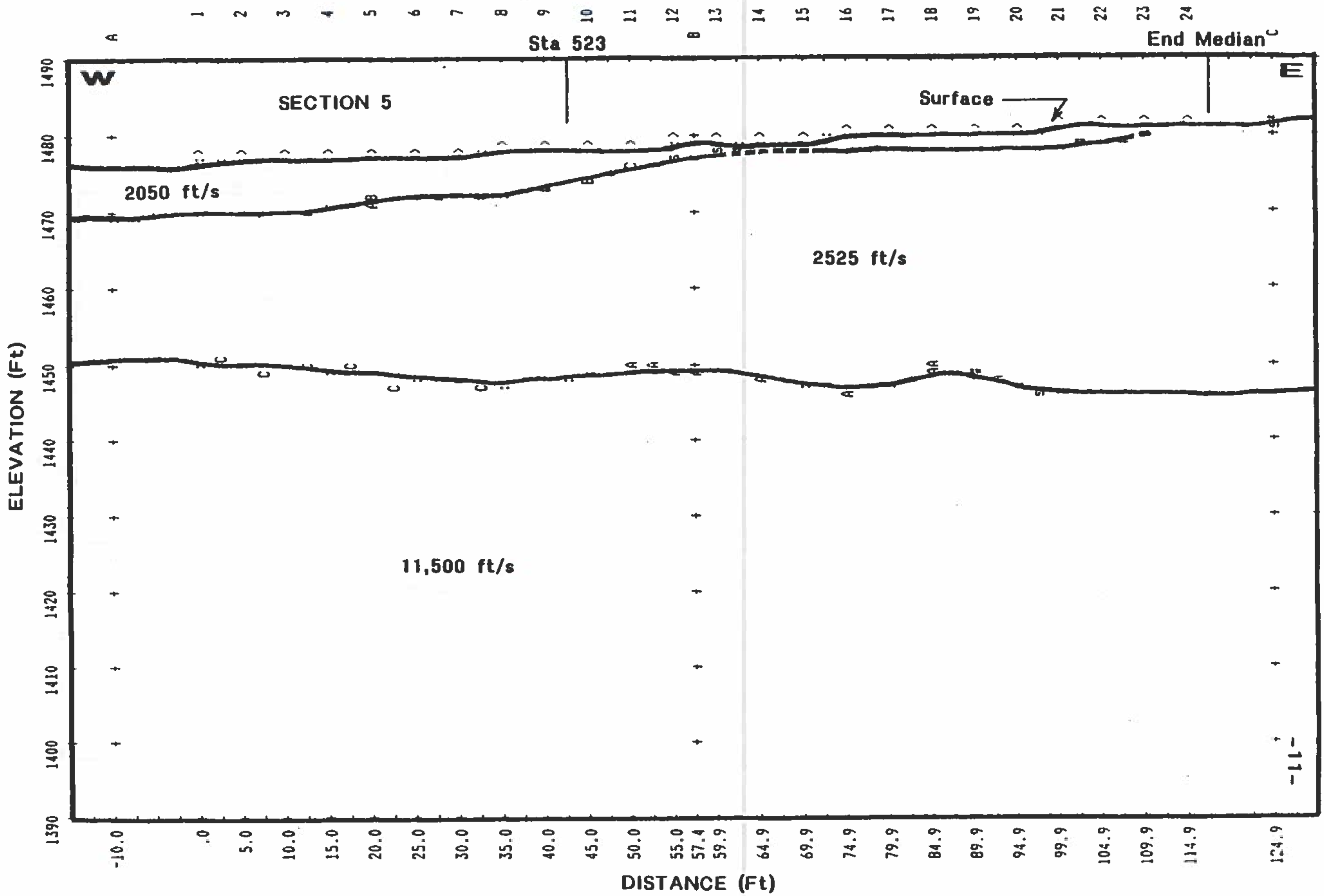
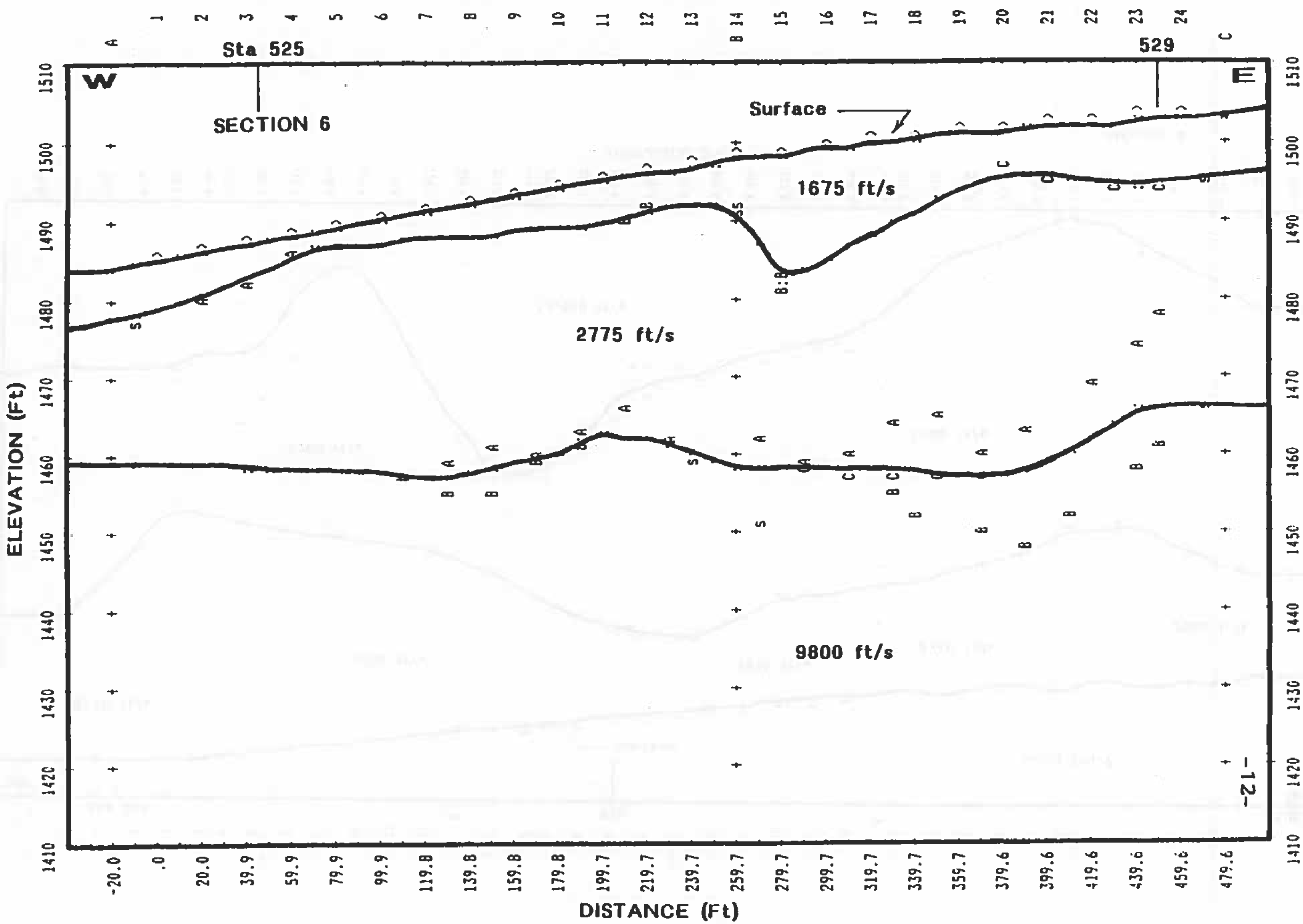

Sta 495
500
503
W
E
SECTION 4
Surface
3680 ft/s
4100 ft/s
3300 ft/s
5000 ft/s
12,750 ft/s
13,100 ft/s
ELEVATION (Ft)
DISTANCE (Ft)

FIGURE 5



Sta 523
End Median
W
E
SECTION 5
Surface
2050 ft/s
2525 ft/s
11,500 ft/s
ELEVATION (Ft)
DISTANCE (Ft)

FIGURE 6



W
E
Sta 525
SECTION 6
529
Surface
1675 ft/s
2775 ft/s
9800 ft/s
ELEVATION (Ft)
1510
1500
1490
1480
1470
1460
1450
1440
1430
1420
1410
DISTANCE (Ft)
-20.0
.0
20.0
39.9
59.9
79.9
99.9
119.8
139.8
159.8
179.8
199.7
219.7
239.7
259.7
279.7
299.7
319.7
339.7
359.7
379.6
399.6
419.6
439.6
459.6
479.6
1 2 3 4 5 6 7 8 9 10 11 12 13 14 15 16 17 18 19 20 21 22 23 24

FIGURE 7

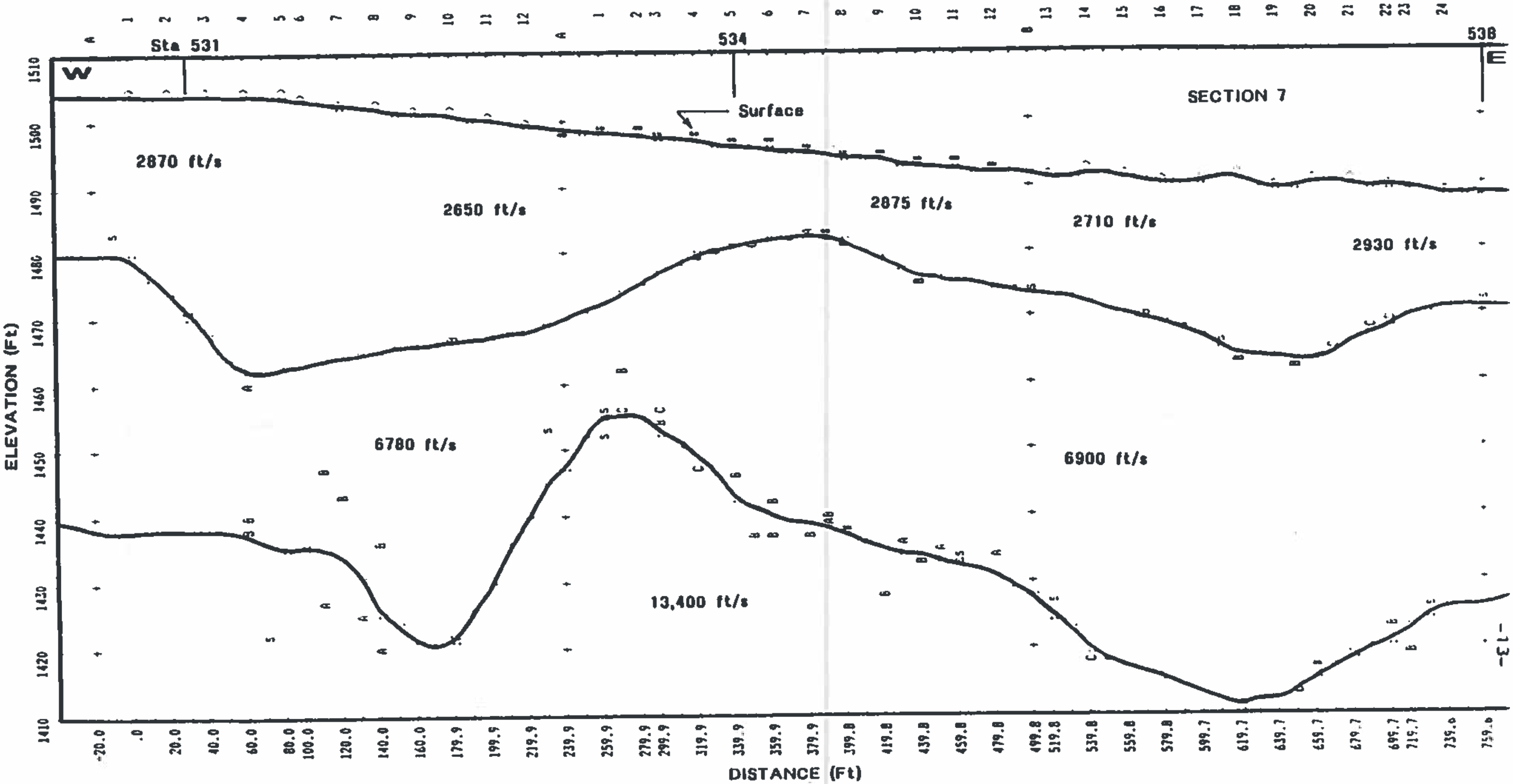

SECTION 7
W
E
Sta 531
534
538
Surface
2870 ft/s
2650 ft/s
2875 ft/s
2710 ft/s
2930 ft/s
6780 ft/s
6900 ft/s
13,400 ft/s
ELEVATION (Ft)
DISTANCE (Ft)

FIGURE 8

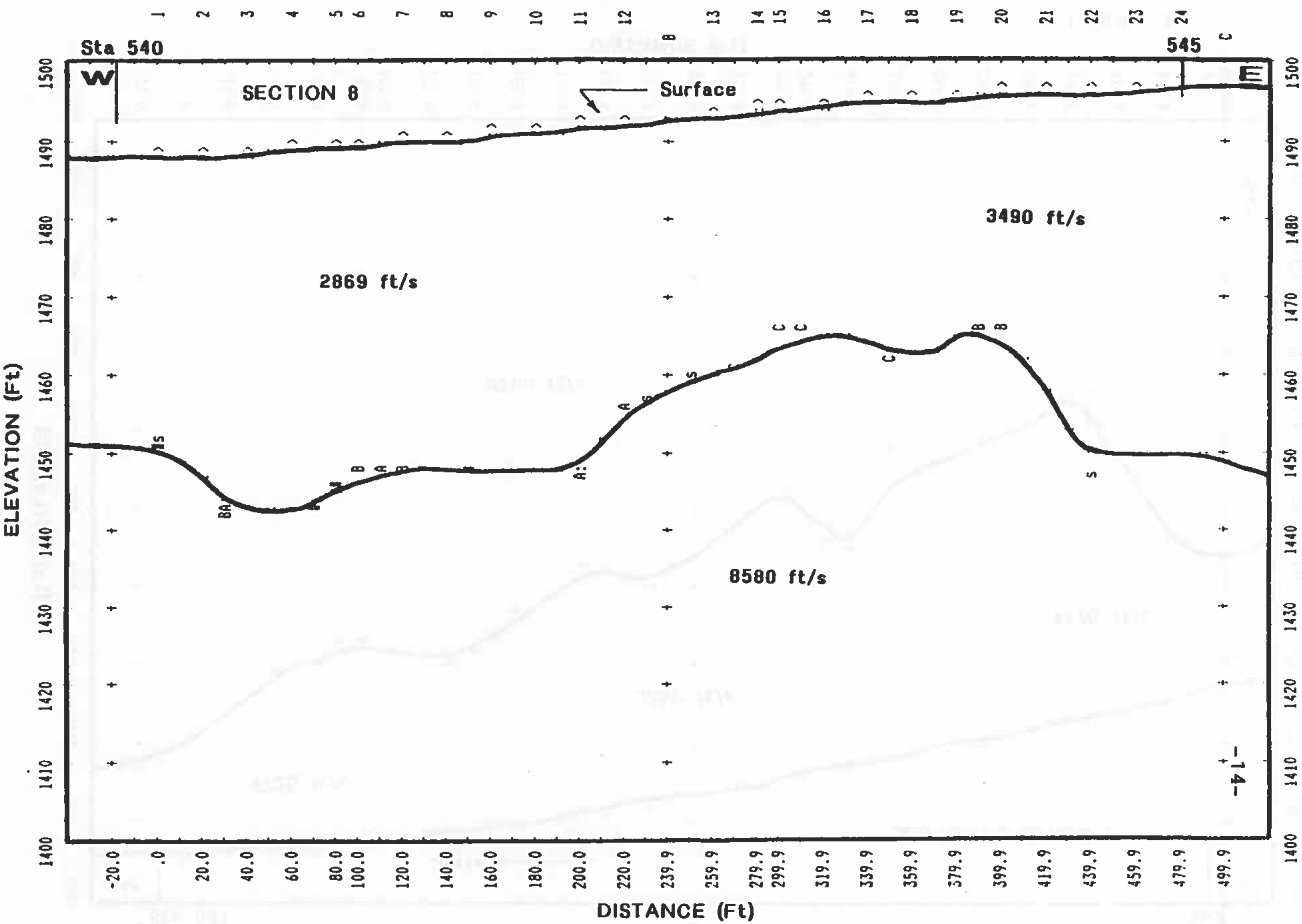
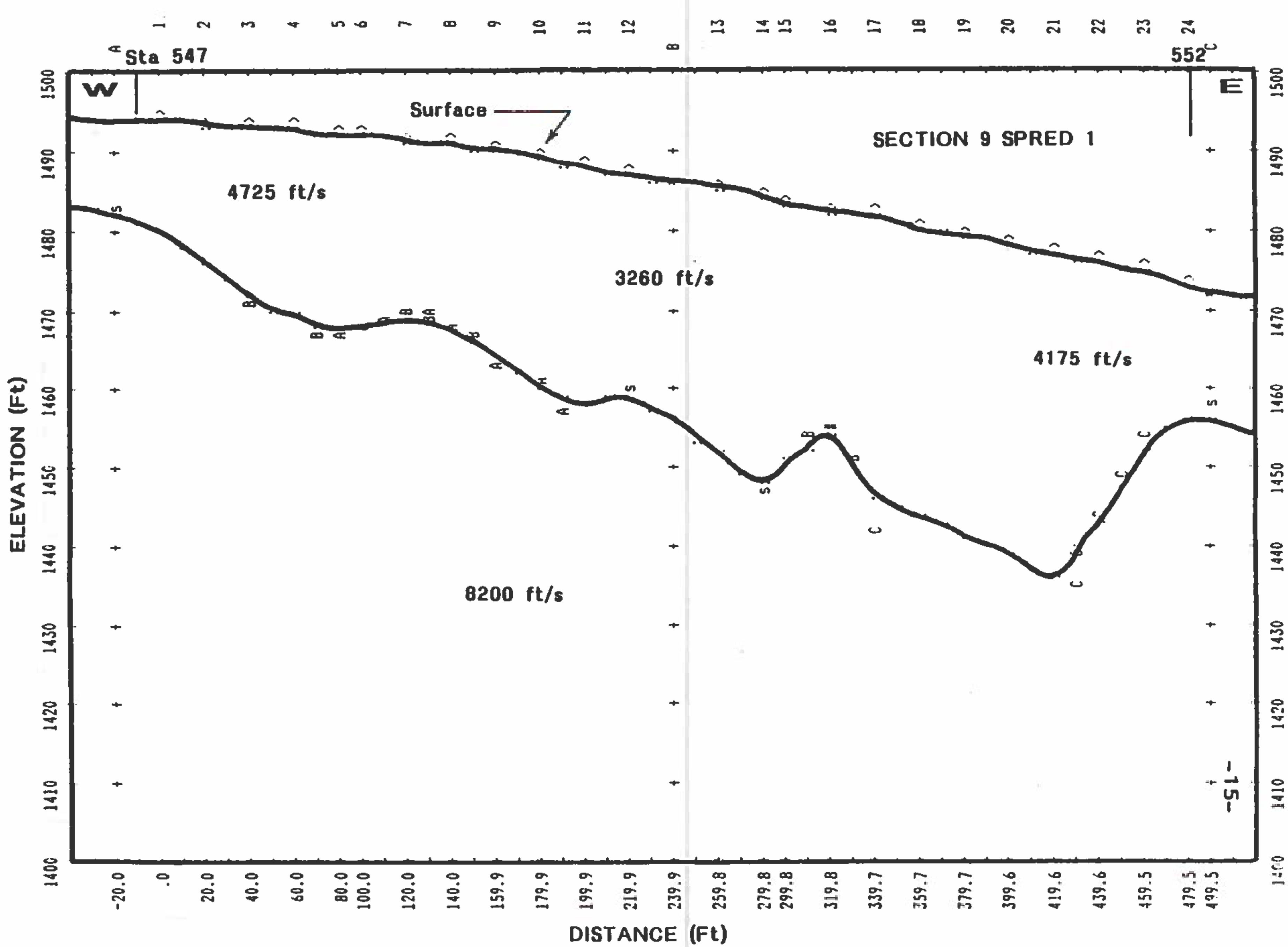

Sta 540
545
W
E
SECTION 8
Surface
2869 ft/s
3490 ft/s
8580 ft/s
ELEVATION (Ft)
DISTANCE (Ft)
1400
1410
1420
1430
1440
1450
1460
1470
1480
1490
1500
-20.0
.0
20.0
40.0
60.0
80.0
100.0
120.0
140.0
160.0
180.0
200.0
220.0
239.9
259.9
279.9
299.9
319.9
339.9
359.9
379.9
399.9
419.9
439.9
459.9
479.9
499.9

FIGURE 9



Sta 547
552
W
E
Surface
SECTION 9 SPRED 1
4725 ft/s
3260 ft/s
4175 ft/s
8200 ft/s
ELEVATION (Ft)
DISTANCE (Ft)
1500
1490
1480
1470
1460
1450
1440
1430
1420
1410
1400
-20.0 .0 20.0 40.0 60.0 80.0 100.0 120.0 140.0 159.9 179.9 199.9 219.9 239.9 259.8 279.8 299.8 319.8 339.7 359.7 379.7 399.6 419.6 439.6 459.5 479.5 499.5
1 2 3 4 5 6 7 8 9 10 11 12 13 14 15 16 17 18 19 20 21 22 23 24

FIGURE 10

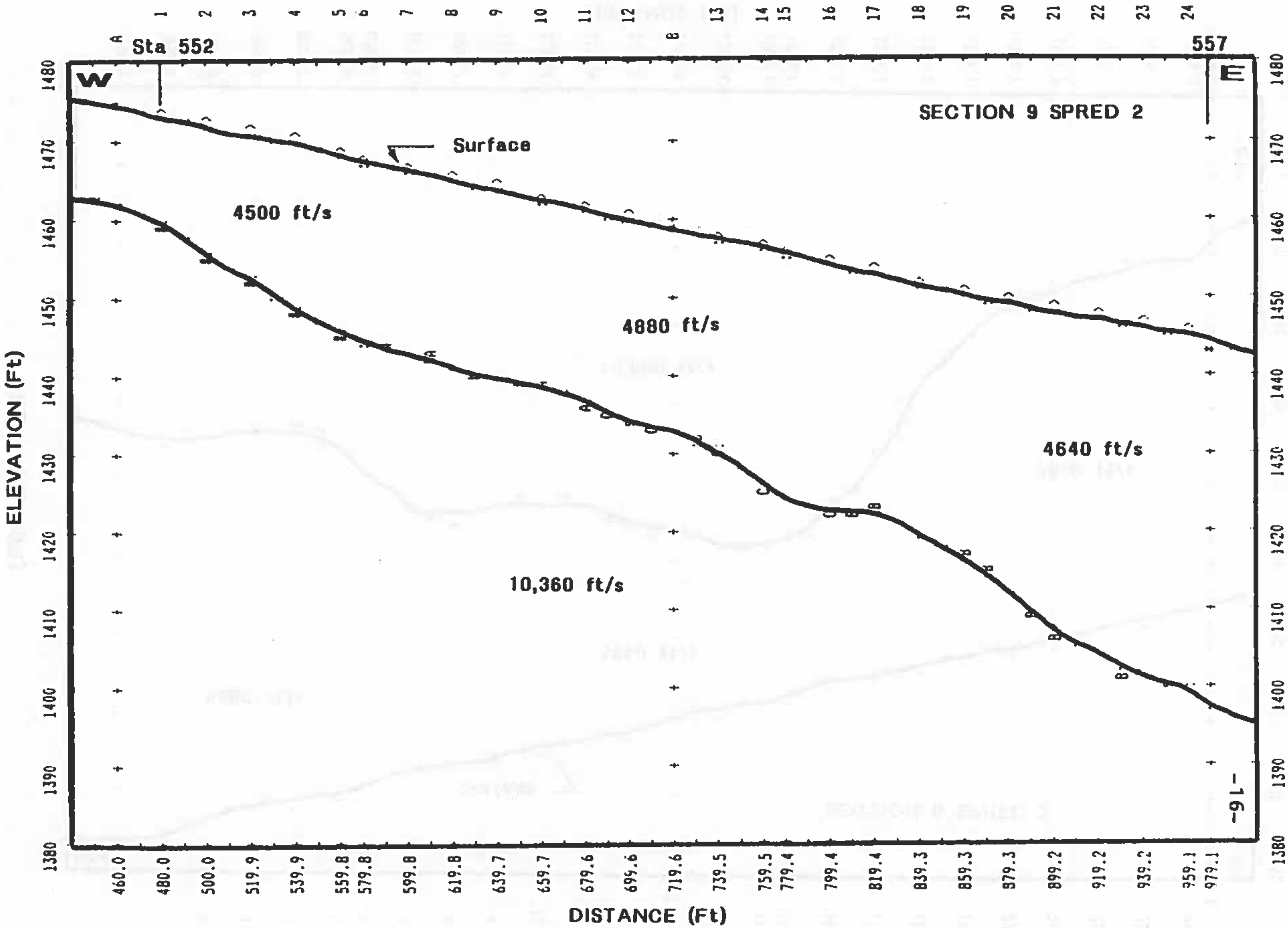

Sta 552
557
W
E
SECTION 9 SPRED 2
Surface
4500 ft/s
4880 ft/s
4640 ft/s
10,360 ft/s
ELEVATION (Ft)
DISTANCE (Ft)
-16-

FIGURE 11

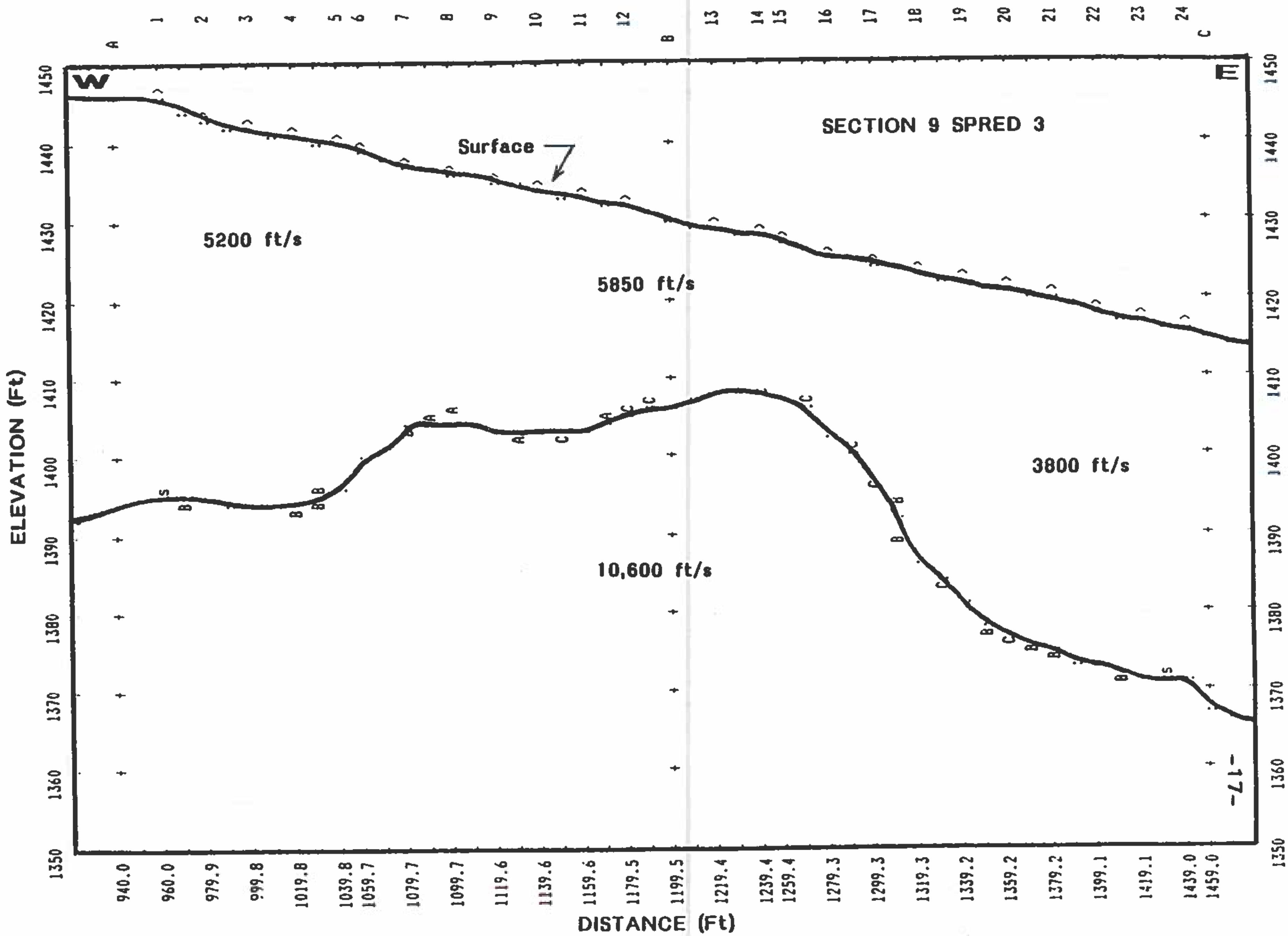

W
E
SECTION 9 SPRED 3
Surface
5200 ft/s
5850 ft/s
3800 ft/s
10,600 ft/s
ELEVATION (Ft)
DISTANCE (Ft)
1350
1360
1370
1380
1390
1400
1410
1420
1430
1440
1450
940.0
960.0
979.9
999.8
1019.8
1039.8
1059.7
1079.7
1099.7
1119.6
1139.6
1159.6
1179.5
1199.5
1219.4
1239.4
1259.4
1279.3
1299.3
1319.3
1339.2
1359.2
1379.2
1399.1
1419.1
1439.0
1459.0

FIGURE 12

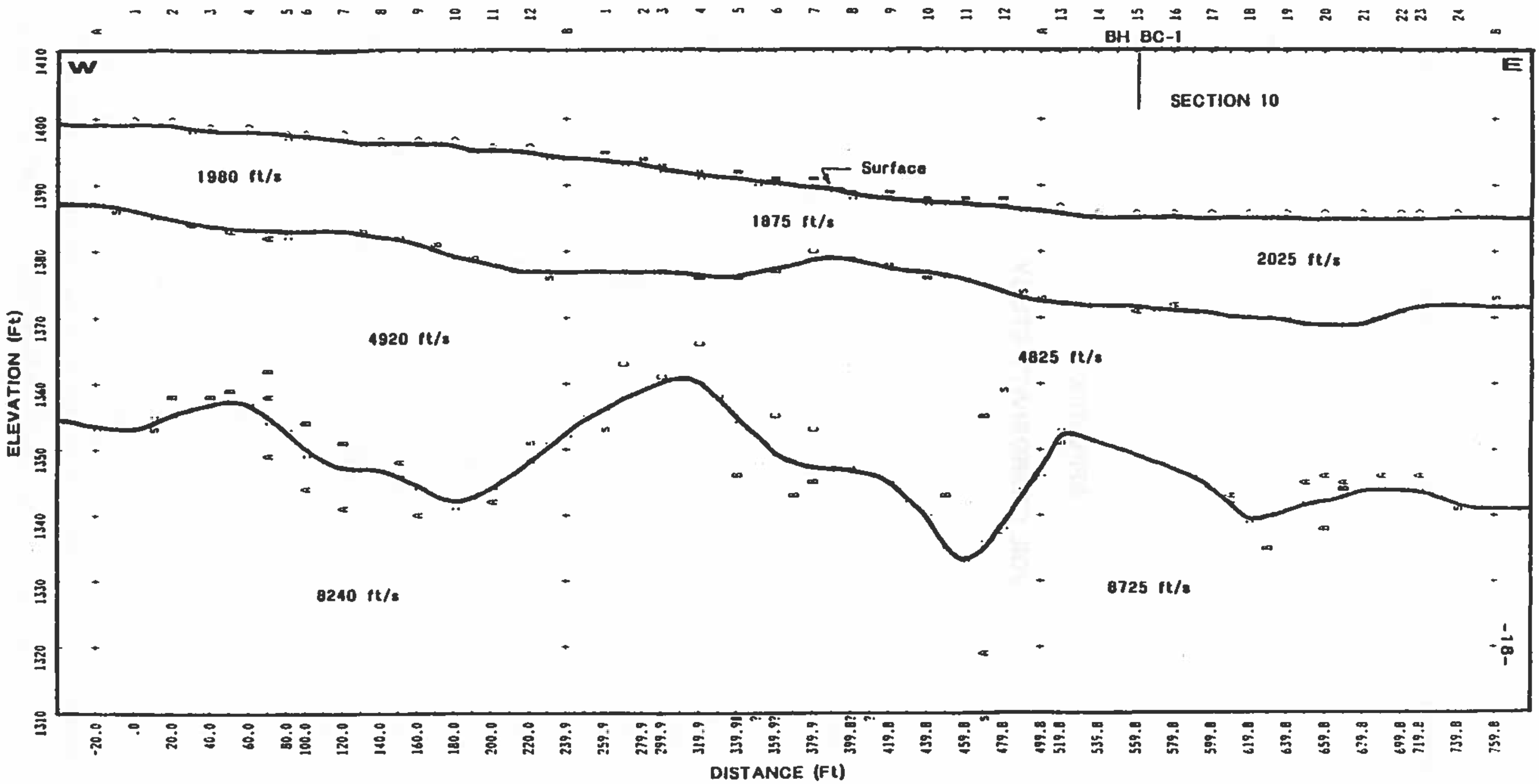

BH BC-1
SECTION 10
W
E
Surface
1980 ft/s
1875 ft/s
2025 ft/s
4920 ft/s
4825 ft/s
8240 ft/s
8725 ft/s
ELEVATION (Ft)
DISTANCE (Ft)

FIGURE 13

APPENDIX D

SOIL CORROSIVITY STUDY

Consulting Corrosion Engineers

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

October 1, 1993

CONVERSE CONSULTANTS INLAND EMPIRE
10391 Corporate Drive
Redlands, California 92374

Attention: Mr. Quazi Hashmi

Re: Soil Corrosivity Study
EMWD - Reach 4, Contract 1
Canyon Lake, California
Your # 92-81-504-01
MJS&A #92186

INTRODUCTION

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

The Canyon Lake reach of the proposed 54-inch diameter waterline will run about 8,200 feet in Railroad Canyon Road. Materials being considered 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 gives the average resistivity to a depth equal to the spacing between the pins. Pin spacings of 2.5, 5, 7.5, 10, and 15 feet were used so that variations with depth could be evaluated. Strata resistivities were calculated from resistance data using 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 its 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 field resistivities were in the moderately corrosive category. Stratum resistivities were mostly moderately corrosive and the others were mildly corrosive. The lowest resistivity was 3,854 ohm-cm at Black Horse Drive.

Electrical resistivities measured in the laboratory with as-received moisture content were in the mildly corrosive category. When saturated, they dropped into the moderately corrosive category. The resistivities dropped considerably with added moisture indicating that the samples were dry as-received.

Soil pH values were 6.6 and 8.0. These values are neutral and moderately alkaline and do not have a significant on corrosivity.

The chemical content of the samples was low.

Even though this route is classified as only moderately corrosive to ferrous metals, some corrosion control measures should be taken.

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. Further, there is a new 6 or 8 inch diameter, steel, natural gas line being installed in Railroad Canyon Road. It has been reported that this line will be protected by a galvanic system.

RECOMMENDATIONS FOR STEEL PIPELINE WITH CEMENT-MORTAR COATING

1. Steel pipe with a cement-mortar coating per AWWA Standard C-205 requires no additional protection.

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 or cement slurry.

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, wax 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 the spigot and coated with cement-mortar per EMWD Standard Drawing B-638, Detail C.

5. Install insulated joints at any existing metallic piping connecting to the subject waterline. 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 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. Independently braze each wire to the pipe. For test stations at casings, 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. Also, native 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 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 50 mil tape, use 80 mil tape or 50 mil tape protected by a 3/4 inch cement-mortar coating.

2. 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.

3. Coat field joints with a tape wrap per AWWA Standard C-109 or with heat shrink sleeves per AWWA Standard C-216.

4. 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.

5. Install insulated joints at any existing metallic piping connecting to the subject waterline. 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 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. Independently braze each wire to the pipe. For test stations at casings, braze two additional wires of a different color to the casing.

7. Cathodic protection is not needed 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. Also, native 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 determine if conditions on the pipeline are changing.

RECOMMENDATIONS FOR CONCRETE STRUCTURES

Standard construction practices and concrete mixes may be used for concrete in contact with soils using type 1 or 2 cement.

Please call if you have any questions.

Respectfully Submitted,
M. J. SCHIFF & ASSOCIATES

Paul R Smith

Paul R. Smith, P.E.

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

L76/92186-2

TABLE 2
LABORATORY TESTS ON SOIL SAMPLES

Location and Depth	Soil Type	Soil Resistivity ohm-centimeters As Rec'd	Sat'd	pH	----------Chemical Analysis in mg/kg (ppm) of dry soil--------- Calcium Ca	Magnesium Mg	Sodium Na	Bicarbonate HCO3	Chloride Cl	Sulfate SO4
BC-1 7-12'	sandy silt	50,000	4,600	6.6	trace	trace	34	122	35	trace
BC-4 5-10'	clayey sand	50,000	7,100	8.0	trace	trace	46	122	35	trace

Carbonate = 0 for all samples

EMWD - Reach 4, Phase I
Canyon Lake, California
Your #92-81-504-01, MJS&A #92186
F19

Test date:
September 8, 1993

TABLE 1
SOIL RESISTIVITY - FIELD TESTS

EMWD - REACH 4, SUN CITY TO LAKE ELSINORE
MJS&A #92186

LOCATION	DEPTH feet	MEASURED RESISTANCE ohms	AVERAGE RESISTIVITY TO DEPTH ohm-cm	STRATUM RESISTIVITY ohm-cm
RR Canyon Rd. / Goetz Rd.	2.50	14.00	7000	7000
	5.00	7.00	7000	7000
bare dry ground	7.50	5.80	8700	16917
	10.00	4.40	8800	9114
	15.00	3.30	9900	13200
RR Canyon Rd. & Black Horse Dr.	2.50	9.50	4750	4750
	5.00	6.00	6000	8143
bare dry ground	7.50	3.70	5550	4826
	10.00	2.50	5000	3854
	15.00	1.60	4800	4444
RR Canyon RD. & Skylink Dr.	2.50	16.00	8000	8000
	5.00	6.60	6600	5617
bare dry ground	7.50	4.10	6150	5412
	10.00	2.90	5800	4954
	15.00	1.70	5100	4108

APPENDIX E

RECOMMENDED SPECIFICATIONS FOR PLACEMENT OF TRENCH BACKFILL

APPENDIX E

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 backfills 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.

- Except as stipulated herein, soils obtained from the excavation, free of organic and deleterious material, may be used as backfill.

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

- Granular bedding material should have a sand equivalent (SE) equal to or greater than 30, as determined by the ASTM Standard D2419-74 test method.

- Trench backfill shall be compacted by mechanical methods such as tamping sheepsfoot, vibrating or pneumatic rollers or mechanical tampers to achieve the density specified herein. The backfill materials shall be brought to near optimum moisture content, placed in eight to 12 inches thick uncompacted horizontal layers. Each layer shall be evenly spread, moistened or dried as necessary and then tamped or rolled 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.

REACH IV- BORING LOCATIONS