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Converse Consultants
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GEOTECHNICAL REPORT

METRO RAIL PROJECT

DESIGN UNIT A100

BY

CONVERSE CONSULTANTS, INC.
EARTH SCIENCES ASSOCIATES
GEO/RESOURCE CONSULTANTS

FEBRUARY 1984

Funding for this Project is provided by grants to the Southern California Rapid Transit District from the United States Department of Transportation, the State of California and the Los Angeles County Transportation Commission.

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February 17, 1984

Metro Rail Transit Consultants
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Attention: Mr. B.I. Maduke, Senior Geotechnical Engineer

Gentlemen:

This letter transmits our final geotechnical investigation report for Design Unit A100, Main Yard and Shops, prepared in accordance with our Contract No. 503 agreement dated September 30, 1984 between Converse Consultants, Inc. and Metro Rail Transit Consultants (MRTC). This report provides geotechnical information and recommendations to be used by design firms in preparing designs for Design Unit A100.

Our study team appreciate the assistance provided by the MRTC staff, especially Bud Maduke. We also want to acknowledge the efforts of each member of the Converse team, in particular Fred Chen and Jim Doolittle.

Respectfully submitted,

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RMP:d

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PROFESSIONAL CERTIFICATION

This report has been prepared by CCI/ESA/GRC under the professional supervision of the principal soils engineer and engineering geologist whose seals and signatures appear hereon.

The findings, recommendations, specifications or professional opinions are presented, within the limits prescribed by the client, after being prepared in accordance with generally accepted professional engineering and geologic principles and practice. There is no other warranty, either express or implied.

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Section 1.0

Executive Summary

1.0 EXECUTIVE SUMMARY

This report presents the results of a geotechnical investigation for a segment of subsurface lead tracks east of the Union Station and the proposed Main Yard and Shops, which will be located southeast of the Union Station. The proposed at-grade yard and shops area will be located along approximately 6000 lineal feet of the existing Santa Fe railyard. Access to the yard and shops area will be via cut-and-cover construction of a box structure approximately 1750 feet long passing under the existing elevated Hollywood Freeway southeast of the Union Station, and daylighting just north of First Street in the Main Yard and Shops area. The purpose of this investigation is to provide geotechnical information to be used for design and construction.

Our subsurface investigation encountered approximately 90 feet of Young Alluvium in the vicinity of Union Station. These soils consisted primarily of gravelly sands with an upper sandy zone and a lower bouldery zone. Generally, the fill thickness in the Union Station area and the Hollywood Freeway crossing area is expected to be around 14 feet; in the Yard and Shops area further to the southeast fill thicknesses are generally expected to range between 1 and 12 feet. Underlying the Young Alluvium, a weak claystone bedrock of the Puente Formation was encountered in the deep borings drilled during our 1981 and 1983 explorations. Ground water depth was measured between 23 and 35 feet below the existing ground surface. These levels correspond to approximately 38 feet of water above the bottom of the proposed cut-and-cover box excavation east of the Union Station area, and 10 feet below the depth of the box excavation at the portal.

Gas consisting of a mixture of hydrogen sulfide and hydrocarbons was released from the water during a field permeability pumping test near the west end of the Union Station site. A mixture of the gases exploded within the central well when it was probed during a gas detection test. The potential for production of similar gas mixtures is believed to exist along the cut-and-cover construction within Design Unit A100 where lowering of the ground water may be necessary to complete the construction.

Construction of the cut-and-cover box structure passing under the existing Hollywood Freeway will involve extensive underpinning operations for the existing piled and non-piled foundations supporting the Freeway. At the deep portion of the proposed box section near Center Street, 70-foot deep excavations will be required through predominately dense granular soil to about 38 feet below the static water level. This will involve a major dewatering effort and may require special construction provisions to mitigate production of gases during dewatering.

Construction of the Yard and Shops area will generally require cutting the area on the average of 2 to 3 feet to achieve the design subgrade of the yard. Typically, top-of-track elevations at the northern end of the Metro Rail (MR #3) Yard Track will require cuts on the order of 6 to 23 feet below existing grade. The southern end of the yard, as proposed, will require approximately 2.5 feet of fill above the existing ground surface elevation.

The proposed structures will include the Main Shop Building, Maintenance-of-way Building, Transportation Building and several minor structures. Essentially all of the proposed structures will be supported at grade and may require some excavation and recompaction of existing fill soils for support of spread footings and floor slabs.

Based on our understanding of the proposed structures and considering the subsurface data developed during this investigation, geotechnical evaluations and design criteria presented in this report include:

- EXCAVATION DEWATERING: Since the approximate depth of excavation for the A-track boxes within this study area will extend some 38 feet below the static water table in permeable sand and gravel, a major lowering of the water level and careful evaluation of possible construction problems will be required. The report presents ground water data and general criteria to be satisfied by the contractor-designed, -installed, and -operated dewatering system.

- UNDERPINNING: The report presents general guidelines for assessing the need to underpin existing buildings and the foundations for the existing elevated Hollywood Freeway structure. Careful evaluation of possible construction problems relating to underpinning methods and procedures will be required.

- TEMPORARY EXCAVATION SUPPORT: A number of methods may be employed to support vertical sided excavations along the cut-and-cover segment of this design unit. Flexible wall and rigid wall support systems will be feasible with some systems being more difficult to install than others. Internal lateral bracing or tiebacks may be utilized with the latter requiring special construction procedures to avoid the collapse and loss of the hole for the installation of the tieback rods and grout. Design criteria for the two classes of support systems are presented and discussed in the report. The appropriate support system will be decided upon by the construction contractor and will be reviewed prior to approval by the Metro Rail Construction Manager.

- MAIN YARD AND SHOPS BUILDING FOUNDATION PREPARATION: Foundation preparation criteria are provided for all at-grade building footings to be founded within the natural alluvial sand and upper fills at the site. The recommended foundation preparation will provide a uniformly compacted building pad for the proposed structures. Allowable bearing capacities for various embedment depths and footing widths are provided with estimated settlements to be anticipated.

- RAILYARD SITE PREPARATION: The proposed new rail tracks within the Main Yard and Shops will be supported at or near existing grade. The report presents recommended subgrade and site preparation criteria for the new railyard, the roadway, and the parking areas.

- EXCAVATION INSTRUMENTATION PROGRAM: In our opinion the proposed cut and cover excavation and the elevated Hollywood Freeway structure should be instrumented. The recommended instrumentation program includes a preconstruction survey, subgrade heave monitoring, surface survey control, bracing load measurements, inclinometer measurements, and gas and oil monitoring.

- LOADS ON PERMANENT SLABS AND WALLS: The report presents recommended design loads on the permanent structures. These include vertical and lateral soil and hydrostatic pressures on slabs, walls and roof.

- LIQUEFACTION POTENTIAL: Since the site is underlain by 50 to 60 feet of saturated granular soils, a liquefaction analysis was performed. Based on the results of the analysis and engineering judgement, it is our opinion that the site would not be subject to liquefaction during the postulated design ground motion.

To emphasize the critical items in this report relating to possible construction problems, the subject items of discussion are underscored for the reader in the report.

Section 2.0

Introduction

2.0 INTRODUCTION

This report presents the results of a geotechnical investigation for Design Unit A100. The unit consists of a 1750-foot subsurface line proceeding in a southeasterly direction from the Union Station, passing under the Hollywood and Santa Ana Freeways, and an at-grade Main Yard and Shops areas along the west bank of the Los Angeles River. This area forms the eastern terminus and maintenance facility for the 18.6-mile starter line of the Southern California Rapid Transit District. The work performed for this report includes borings, laboratory tests, engineering analyses and the development of recommendations and specifications for design and construction purposes.

Additional geotechnical information on the Metro Rail project is included in the following reports, some of which may pertain to Design Unit A100:

- "Geotechnical Investigation Report, Metro Rail Project", Volume I - Report, and Volume II - Appendices, prepared by Converse Ward Davis Dixon, Earth Sciences Associates and Geo/Resource Consultants, submitted to SCRTD in November 1981: This report presents preliminary geologic and geotechnical data for the entire project. The report also comments on tunneling and shoring experience and practices in the Los Angeles area.

- "Geotechnical Report, Metro Rail Project, Design Unit A135", prepared by Converse Consultants, Inc., Earth Sciences Associates, and Geo/Resource Consultants, submitted to SCRTD in September 1983. This report presents our results of the findings for the Union Station design unit.

- "Memorandum - Bridge Foundation Investigation and Recommendations, Los Angeles River Busway Bridge and Overhead Bridge No. 53-2673", issued by California State Department of Transportation, Office of Transportation Laboratory to Mr. R.C. Cassano, Chief, Office of Structures, dated July 14, 1981, File No. 07-LA-010-17.6.

- "Seismological Investigation & Design Criteria Metro Rail Project", prepared by Converse Consultants, Lindvall, Richter & Associates, Earth Sciences Associates and Geo/Resource Consultants, submitted to SCRTD in May 1983: This report presents the results of a seismological investigation.

- "Geologic Aspects of Tunneling in the Los Angeles Area" (USGS Map No. MF866, 1977), prepared for the U.S. Geological Survey in cooperation with the U.S. Department of Transportation. This publication includes a compilation of geotechnical data in the general vicinity of the proposed Metro Rail Project and this design unit.

Pertinent data from these previous reports have been incorporated in this report.

The design concepts evaluated in this geotechnical report are based on the "Draft Report for the Development of Milestone 10: Main Yard and Shops: dated June 1983 drawing nos. AP-16AAB-C-001(0) through -010, and C-101, -103, and -104. These plans were prepared for SCRTD by DMJM/PBOD. The Milestone 10 data in the draft report may be subject to change.

Section 3.0

Project and Site Description

3.0 PROJECT AND SITE DESCRIPTION

3.1 SITE DESCRIPTION

The Main Yard and Shops area occupies an area south of the Union Station and adjacent to the west side of the Los Angeles River. The subsurface access line, south of the station, passes beneath numerous public streets, private and public properties with existing commercial and industrial buildings and major public transportation facilities. The latter consist of the Hollywood and Santa Ana Freeways and the future El Monte Busway on the north side of the freeways. The freeway pavement elevation varies between approximately 285 and 298 feet above the metro alignment. The existing ground surface elevation along the alignment varies between approximately 270 and 280 feet.

South of the freeway, the subsurface line emerges into an existing rail yard owned by the Atcheson Topeka and the Santa Fe Railway Company. This area is bounded by the Los Angeles River on the east and Center, Santa Fe and Mesquit Streets on the west. Ramirez Street forms the north limit of the yard area whereas Seventh Street forms the south limit. The yard area consists of numerous existing rail tracks, commercial rail and truck transhipping and warehousing facilities, storage silos and industrial plant structures and offices.

The surface of the yard area is comparatively flat and level with some improved or paved areas on the west side of the site. Existing ground surface elevations vary between approximately 280 feet near Commercial Street and 244 feet in the area between Sixth and Seventh Streets. Major permanent transportation facilities crossing the site include the First, Fourth and Sixth Street vehicular bridges which carry traffic across the Los Angeles River.

3.2 PROJECT DESCRIPTION

Design Unit A100 consists of a number of significantly different design and construction sections; subsurface lead tracks and at-grade yard and shops facilities.

1. Subsurface Tracks - The subsurface tracks originate at the south end of the Union Station structure and extend southward some 3100 feet into the yard area. For purposes of this report, the subsurface lead tracks are defined to consist of approximately 1800 feet of cut-and-cover construction; 650 feet of U-section construction and 650 feet of cantilever retaining wall construction; the latter ending just north of the First Street vehicular bridge crossing the site.

   The box structure south of the station will have a width of approximately 70 feet and a height of 25 feet, containing both the Y-tracks to the yard and the future A-tracks to the east. The Y-tracks cross over and above the AL tracks at approximately station AL93+00. The AR and AL tracks will be temporarily truncated at approximately stations AR93+50 and AL91+50 in a closed end box structure. Invert elevations along the Y-line vary between approximately elevation 232, near the station, and elevation 250, near the portal. Invert elevations along the A-line vary between approximately elevation 232 and 210 near the point of temporary truncation. Cut-and-cover excavation at this point will approach a depth of 70 feet.

South of station approximately Y92+50, the width of Y-line box structure reduces to approximately 40 feet and passes beneath the future El Monte busway, and the existing Hollywood and Santa Ana Freeways. South of Station Y86+00, the box structure containing the Y tracks increases in width to approximately 75 feet to accommodate future X line tracks to and from the east. The box structure ends at approximately station MR#3 66+70. South beyond this station the yard lead tracks are contained within a 650-foot long U-section and a 650-foot long retaining wall section.

We understand that the existing elevated Hollywood Freeway structure is founded on both spread footings and pile foundations. At the Y-track box crossing area, the proposed top of box structure is at approximately elevation 264 feet. The pile tip at those locations supporting the Hollywood Freeway where the Y-track box passes is at approximately Elevation 255.

We also understand that the proposed El Monte Busway will also be founded on piles. However, the foundations of the Busway structure will span over the proposed box structures and the piles for the Busway will not affect the box construction.

We understand that the permanent box structure will be designed as a rigid reinforced concrete box with up to three rows of interior columns or panels to support the roof slab. We understand that the box roof will be designed to support the bridge pier or building foundation loads and the earth cover above it. We also understand that designated above-grade building structures will be removed as necessary along the Y-track alignment to facilitate construction.

2. At-grade Yard and Shops - Approximately 5900 lineal feet of surface construction is proposed between the Portal and Seventh Street. The proposed construction will include the main shop, transportation, maintenance-of-way, car wash, cleaning and substation buildings, storage and operating tracks and minor ancillary structures.

Section 4.0

Field Exploration and Laboratory Testing

4.0 FIELD EXPLORATION AND LABORATORY TESTING

4.1 GENERAL

The conclusions and recommendations presented in this report are based primarily on the field and laboratory investigations performed in 1983 for this study and those performed in 1981 for the initial Geotechnical Investigation Report. In addition, the subsurface data compiled in 1981 by the California Department of Transportation, in 1977 by the USGS, and by Kaiser Engineers in 1962 were reviewed. We also reviewed a soils report for the Soule Metal Building by Foundation Engineering Co. Inc. dated August 1, 1979.

In general the geotechnical studies included borings, man-sized auger borings, ground water observation wells, soil/rock laboratory tests, and geotechnical engineering analysis.

4.2 BORINGS

Test borings drilled in 1983 for this Design Unit include 33 borings (3-1 to 3-33) ranging in depth from 10 to 51 feet. In addition 2 borings (B-10 and B-11) were drilled at locations of proposed El Monte Busway piers. The 1981 exploration program included 150-foot deep borings CEG-3 to CEG-5, and the Union Station borings 5-4 and 5-5 in the general site area. Kaiser borings 114 to 116 are also in the site vicinity (not shown in this report). Locations of the pertinent borings are shown on Drawings 2 through 5. Ground water observation wells were installed in borings CEG-3, CEG-4, CEG-5, 5-5, and 3-1, 3-7, 3-9, 3-15 and 3-24. Observation wells included a perforated section within the lower 20 feet of the boring with gravel backfill placed to the surface seal. Section 5.2 presents a summary of ground water level measurements obtained from the observations wells. The logs of the 1981 and 1983 borings along with descriptions of the field procedures are included in Appendix A.

4.3 GEOTECHNICAL LABORATORY TESTING

Laboratory testing programs were performed on representative soil and rock samples for both the 1981 investigation and this investigation. These consisted of classification tests, consolidation tests, triaxial compression tests, direct shear tests, and permeability tests. Since the bedrock formation encountered at the site is "soil-like", the bedrock samples were subjected to the same type of testing as the soil samples.

Appendix F summarizes the testing procedures and presents the results from both the 1983 and 1981 laboratory testing programs.

4.4 OTHER TESTING

Several related pertinent field and laboratory tests were conducted during the investigation for Design Unit 135 at Union Station. The data and information obtained from those tests are included here for general reference and use.

Most of the tests were performed in adjacent locations where the subsurface conditions are essentially similar in nature to that found in the cut and cover box section in this design unit.

4.4.1 Pump Test

A pump test was performed, during the Union Station investigation, adjacent to boring 5-1 as shown on Drawing 5. The well was 12 inches in diameter (I.D.), 82.5 feet deep, and perforated below the ground water table. Two 4-inch diameter observation wells were installed to evaluate water level drawdown at distances of 60 and 120 feet west of the test well. Boring 5-1, 25 feet away from the pump well, was also used as an observation well during the test.

The test well was pumped initially at a discharge rate of 175 gpm for about 8 hours. The test was terminated early due to problems with gas in the discharge affecting the flow meter measurements. Following a 15-hour recovery period a second test was performed at a reduced discharge rate of 150 gpm for 23 hours. The maximum drawdown at the 60-foot piezometer was 2.8 feet (after 23 hours of pumping at 150 gpm).

Appendix B provides a report of the pump test procedures and results. The well casing is capped but open at the time of this writing. It may be possible to re-use the casing in the future provided that effects of ground water corrosion have not made it inoperable

4.4.2 Geophysical Measurements

Downhole and crosshole shear wave velocity surveys were performed in boring CEG-5 (see Drawing 5). Downhole shear wave velocity measurements were made at 5-foot vertical intervals from ground surface to a depth of 130 feet. Crosshole shear wave velocity measurements were performed in Boring CEG-5 at 5-foot vertical intervals. An array of three borings, spaced about 15 feet apart, was used for the crosshole survey. The results of the downhole and crosshole surveys are presented in Appendix C.

4.4.3 Oil and Gas Analyses

In the 1981 and 1983 borings, noxious sulfur odors (H_2S) were detected. Gas was ignited, causing an explosion, in the pump test well (see Appendix B). During the 1981 investigation gas chromatograph analyses (Appendix D) were performed in Boring CEG-2 located about 2000 feet to the east; however, no chemical analyses of the oil, nor gas chromatographic analyses, were performed at the of Union Station vicinity area in the 1981 or 1983 borings.

4.4.4 Water Quality Analyses

Water quality analyses were performed on samples taken from Borings CEG-4, CEG-6, and man-size Boring 6A (not shown). The results are presented in Appendix E. The water was tested for basic cations, anions, conductivity, total dissolved solids, pH, turbidity and boron.

Section 5.0

Subsurface Conditions

5.0 SUBSURFACE CONDITIONS

5.1 SUBSOILS

The geologic sequence in the site area consists of man-made Fill, Young Alluvium (A_1), and bedrock of the Puente Formation (C).

The Los Angeles anticline (upfold), a major geologic structure trending about N70W, influences the dip of the bedrock strata. There are no known active or inactive faults identified in the A100 site.

Ground water was measured at elevations ranging from about 236 to 255 in the Main Yard and Shops area. Assuming the deepest bottom excavation elevation of about 214, the proposed excavation would extend through fill, sands and gravelly sands. Drawing Nos. 2 through 5 show the geologic sections through the site.

5.1.1 Fill

The fill underlying the existing Santa Fe Track Yard consisted of silt, silty sand and gravel to depths varying between 1 and 12 feet (see Drawings 2 through 5). Based on the borings drilled during this investigation, we believe the fill thickness appears to vary between 1 and 3 feet over the majority of the existing warehouse and pavement areas. Local areas were encountered where the fill thickness was found to be on the order of 8 to 12 feet.

Borings CEG-4, 5-4 and 5-5 located outside the Track Yard, east of the Union Station encountered about 5 to 14 feet of fill consisting of dense and loose silts and sands. Borings B-10 and B-11 drilled for the Cal Trans Bridges encountered 9 and 29 feet of fill respectively. The fill consisted of sandy silt and silty sand with brick debris.

Boring 3-1 encountered 12 feet of fill. Borings 3-14, 3-18, 3-31 and 3-33 were drilled to depths of 30 and 45 feet, encountered fill between 5 and 8 feet. The remainder of the Track Yard showed a fill thickness of approximately 1 to 3 feet. Based on the probable fill history of the area, bricks, timber, concrete blocks, and other building debris may be encountered, although not much debris was seen in the holes drilled. Some 14 abandoned reinforced concrete blocks are known to exist in the area of the intersection of Ramirez and Keller Streets, as shown on Drawing Nos. 4 and 5. These old concrete generator blocks measured approximately 28'x28'x12' in dimensions and probably were buried underneath the existing ground surface and the pavements.

5.1.2 Young Alluvium (A_1)

The fill is underlain by Young Alluvium deposited during Holocene age. Based on the borings, the Young Alluvium consists of a thin upper sandy unit, a main gravelly and cobbly sand unit, a lower sand unit and a bouldery zone overlying the Puente Formation (see Drawing 5). Specific descriptions of these materials are as follows:

- Upper Sand Unit: Below the fill, the borings encountered approximately 27 feet of silty fine to medium sand between Borings 3-2 and 3-12 inclusive. In other locations, the fine silty sand layer was confined to the upper 10 feet or so. Typical grain size curves for these materials are shown in Appendix F.

- Main Gravelly and Cobbly Sand Unit: The Young Alluvium consisted of well graded gravelly sands with about 10% fines as encountered in Borings 3, 4, 5, 5-4 and 5-5. The gravel content decreases with depth. Due to the gravel content, sample recovery was generally poor and limited to soil particles smaller than the inside diameter of the samplers (1.4 to 2.4 inches). It is believed that the material probably grades through the coarse gravel sizes (2 to 3 inches) with significant cobbly zones. Occasional boulders 2 to 4 feet in diameter are likely to be encountered within the depths of excavation. These interpretations are based on the drilling action (such as rig chatter and drilling difficulty), observations in localized excavations, and man-size auger Boring 6A drilled for the Union Station (not shown in this report), and the nature of the depositional environment.

  Thin layers/lenses of sand, silts and clays were occasionally encountered within the main gravelly sand unit in Borings 5-4 and 3-9. Boring 5-4, drilled for the Union Station Study, encountered a zone of sand interbedded with thin beds of stiff silty clay between a depth of about 28 to 41 feet. Boring 3-9 drilled in the yard and shops area encountered a silt layer between 15 and 25 feet. The occurrences of finer units within the main gravelly sand unit are consistent with the depositional environment of the Young Alluvium. Thus, layers of sands, silts, and/or clays ranging in thickness from a few inches to more than a foot may be expected during excavation in the main gravelly sand unit. These finer grained interbeds are expected to be of limited lateral extent.

  Due to the gravel content, the high standard penetration resistances recorded through this unit are of doubtful value for design purposes. However, based on the standard penetration resistance values in the intermediate sand layers, it is believed that the main gravelly sand unit has a relative density varying between compact and very dense.

- Lower Sand Unit: Borings 5-4 and 5-5, drilled for the Union Station, encountered 7 to 16 feet of dense, poorly graded fine and fine to medium sand with occasional silty lenses. This layer was encountered at an elevation of about 215. None of the borings drilled for the Yard and Shops area penetrated to deeper than elevation 220 feet. Borings B-10 and B-11 drilled for the Cal Trans busway did not encounter a distinguishable lower sand unit.

- Bouldery Zone: The deep borings encountered a boulder zone some 10 to 15 feet thick between elevations 200 and 210 feet overlying the Puente Formation. Due to the composition of this unit, the only samples obtained were cuttings from the drilling mud. These cuttings indicated

> that the boulders were predominately of granitic composition. It is likely this zone is composed of boulders 2 to 3 feet in diameter in a matrix of sands and gravels.

The following description of bouldery ground, applying to Young Alluvial deposits in the vicinity of the Los Angeles River, is excerpted from the 1981 Geotechnical Investigation, Volume I, page 29:

> "Bouldery Ground - Contains occasional boulders in the ancestral Los Angeles River channels (Reaches 1, 2 and 8) up to 2 ft diameter; boulders observed at the surface, prior to lining the Los Angeles River at the Macy Street crossing, were reported to be 4 ft diameter. The presence of boulders and cobbles is noted in the boring logs. However, boulders were noted only where encountered; their absence, therefore, cannot be assumed where not noted, especially near the Los Angeles River. The possibility of undetected irregular-shaped lenses of large and small boulders and cobbles should be assumed."

5.1.3 Bedrock (C)

Borings B-10 and B-11, drilled in 1983 for the Cal Trans Busway and Boring 5-5 of the Union Station investigation penetrated 5 to 10 feet, respectively, into the Puente Formation underlying the boulder zone (Drawing No. 5). The 10 feet penetrated in Borings 5-5 and B-10 consisted of weak claystone laminated with thin silty claystone and sandstone beds. The Puente Formation is believed to contain local hard, cemented sandstone beds ranging from less than 1 inch to 3 feet thick. The top of the bedrock appears to be at about Elevation 190 at the east end of the Union Station.

Borings CEG-3, CEG-4 and CEG-5 drilled during the 1981 geotechnical investigation were advanced some 50 to 70 feet into the Puente Formation (Drawing No. 5).

5.2 GROUND WATER

The proposed Yard and Shops area lies within the Los Angeles Forebay portion of the Central Ground Water Basin. The term "forebay" refers to a recharge area where substantial infiltration of surface water can occur. Due to heavy urban development, the area for direct infiltration has been significantly reduced. The Los Angeles River flows in a concrete-lined channel some 250 feet east of the yard shops area. Ground water elevations and fluctuations measured in Borings CEG-3, CEG-4, CEG-5, 5-5 of the Union Station Study and Borings 3-1, 3-7, 3-9, 3-15 and 3-24 drilled for this study are shown below:

GROUND WATER ELEVATIONS (ft*)

BORING	3/10/81	6/17/81	4/28/82	2/08/83	4/27/83	6/08/83	9/02/83	10/12/83	12/15/83
CEG-3	240								243
CEG-4	249	249	249						250
CEG-5	255**								
5-5				252	253	254	254		
3-1								245	245
3-7								241	241
3-9								245	244
3-15								239	241
3-24								236	235

*Rounded to the nearest foot.

**Piezometer tubing not installed.

It appears that the ground water varies across the site ranging from about Elevation 254 in boring 5-5 in the north to about Elevation 235 in boring 3-24 in the south. This represents a ground water gradient of about .003 foot per foot (vertical/horizontal) in a southward direction. The regional water table aquifer reportedly slopes to the south at a gradient of about .009 (Los Angeles County Flood Control, 1976). Thus, the measured local gradient appears to be in reasonable agreement with the reported regional gradients. Measured fluctuations were about one foot in Boring CEG-4, and 3 feet in Boring CEG-3 during the observational period.

The oil field brine encountered in the Puente Formation in Boring CEG-4 is not representative of ground water quality in the overlying Young Alluvium. Selected water quality parameters at CEG-4 from the Union Station Study are shown below for exemplification purpose only.

SELECTED WATER QUALITY PARAMETERS
(Water Sample in Alluvium)

Boring No.	Depth Water Sampled (ft)	Date Sampled	pH @ 25° C	Total Dissolved Solids (ppm)	Sulfate SO_4 (ppm)	Chloride (ppm)
CEG-4	30	02-19-81	7.6	5,085	79	2,800

5.3 GAS

Hydrogen sulfide odors were detected during drilling of the deep holes for the Union Station Study in borings CEG-4 (trace), CEG-6, and 5-1 through 5-5. During the pump test, considerable gas was released causing discharge flow measurement problems. The gas appears to have been hydrogen sulfide mixed with hydrocarbons including methane. In man-sized auger Boring 6A, located some 500 feet west of the pump test well, no gas was detected, even though the bottom of the hole was extended in free air to a depth of some 35 feet below the static ground water table.

The Union Station Oil Field is located underneath the proposed Yard and Shops area (portion of the boundary of the oil field is shown on Drawing Nos. 2, 3, and 4). Little is known about the oil field, but it does produce from the Puente Formation at very shallow depths.

Boring CEG-2 located some 2000 feet east of the proposed cut-and-cover section (east of the Los Angeles River) penetrated the Young Alluvium/Puente Formation contact at a depth of 12 feet. Oil was encountered at this contact as well as in sandstone layers within the Puente Formation between 12 and 100 feet (bottom of hole). Gas was first detected at a depth of 37 feet in Boring CEG-2. Gas chromatograms run in Boring CEG-2 indicate 100 ppm methane and 500 ppm ethane, thereby resulting in a classification in the lower explosive limit (5% LEL). Borings 10 and 11 drilled for the Cal Trans Bridge also encountered limited amount of tar and sulferous odor in the sand above the bedrock.

It is believed that free gas mixtures may be contained in the Young Alluvium or underlying Puente Formation and/or a portion of the ground water underlying the site may contain gas in solution which originated from the underlying Puente Formation. Stress relief due to ground water lowering during the pump test is believed to have caused the release of gases into the central well. Additional data would be required to confirm these concepts and delineate the problem. The engineering implications of these observations are discussed in Section 6.3, Excavation Dewatering. Details of the field test results and data of CEG-2 are contained in Appendix D.

5.4 ENGINEERING PROPERTIES OF SUBSURFACE MATERIALS

5.4.1 General

For purposes of our engineering evaluations and development of design recommendations, the geologic units at the site were grouped as fill, Young Alluvium, (upper sands, main gravelly and cobbly sand unit, lower sand and boulder zone) and Puente Formation bedrock. This section includes engineering descriptions of each geologic unit and, in Table 5-1, presents the engineering parameters to be used for analyses. These parameters are based on the laboratory test results (Appendix F), data from other investigations, and published data of observed and recorded field behavior on recent construction projects.

5.4.2 Fill

The fill soils were variable and included soil types such as gravel, clayey silts, sandy silt and silty sands with a variety of building debris. The relative density of these materials is considered to be generally dense in the Union Station area, and loose to medium dense in the yard and shops area, based on sampling resistance and density measurement. However, the possible occurrence of loose near surface horizons should not be discounted. The lack of adequate samples did not permit the carrying out of any significant engineering tests on the fill materials. Therefore, engineering judgement was required in selecting design parameters. It was our opinion that drained (effective strength) properties should be utilized in design since these materials are generally above the water table and only partially saturated.

TABLE 5-1

MATERIAL PROPERTIES SELECTED FOR STATIC DESIGN

MATERIAL PROPERTY	FILL	YOUNG ALLUVIUM (A_1)	PUENTE BEDROCK (C)
Moist Density Above Ground Water (pcf)	125	125	120
Saturated Density (pcf)	130	130	120
Effective Stress Strength			
ø' (degrees)	33	35-37**	35
c' (psf)	0	0	0
Total Stress Strength			
ø (degrees)	-	-	0
c (psf)	-	-	5000
Unconfined Compressive Strength (psf)	-	-	10,000
Permeability (cm/sec)	-	3×10^{-2}	10^{-6} to 10^{-7}
Initial Tangent Modulus (psf)	$200 \cdot \sigma_{v'}$*	$350 \cdot \sigma_{v'}$*	2×10^{6}
Poissons Ratio	0.3	0.35	0.35

* $\sigma_{v'}$ is the effective overburden pressure (psf) equal to effective density times over-burden depth. Moist density should be used to determine $\sigma_{v'}$ above the water table and submerged density (saturated density minus water density) was be used for the effective density of soils below the water table.

**ø' = 35° for Yard and Shops area; ø = 37° for cut-and-cover box structure design.

5.4.3 Young Alluvium

The Young Alluvium encountered consisted primarily of silt-sand-gravel mixtures, and these materials were generally classified as silty sand, sand and gravelly sand with cobbles. Standard Penetration Test (SPT) results and laboratory densities indicate the Alluvium is dense to very dense but the SPT values in the gravels are qualitative only. Much of these materials lie near and below the water table. Selected moist and saturated densities are presented in Table 5-1.

The hydraulic properties of the Young Alluvium were investigated by a pump test during the Union Station Study. The testing procedures and test results are described in Appendix B. The general hydraulic characteristics of the Young Alluvium determined on the basis of the test results are as follows:

- Transmissivity: Computed to vary between 21,500 to 43,000 gpd/ft
- Storage Coefficient: Estimated to be about 0.1 to 0.15
- Boundaries: Boundaries were not observed during the pump test but may have been observed had the test duration been longer
- Saturated Thickness: Estimated to be about 65 feet
- Average Formation Permeability: Computed to be about $2x10^{-2}$ to $4x10^{-2}$ cm/sec. However, individual layers may have widely varying permeabilities. Laboratory tests on a silty sand sample indicate permeabilities less than $4x10^{-4}$ cm/sec, and clayey interbeds probably have values less than $1x10^{-6}$ cm/sec.

The above hydraulic characteristics were developed during and for the Union Station investigation. These results are intended to be used only as possible indicators of the hydraulic properties of materials along the cut-and-cover construction in Design Unit A100, and should not be construed as data obtained for this investigation.

Strength tests performed on the sand soils included both direct shear and triaxial compression tests. Considering the relatively high permeability of the sands, drained (effective) strength parameters are considered appropriate for static design. Effective stress strength data for granular alluvium is summarized in Figure F-1 of Appendix F. Figure F-1 also includes test data obtained on similar soils from other Sections of the Rail Project.

Elastic properties for the sands were based on the laboratory triaxial and consolidation tests combined with published data and engineering judgement. The modulus data for soil samples tested from this investigation and similar soil samples from Design Unit A135 and other Rail Sections are summarized on Figure F-2 of Appendix F. The data indicate the modulus increases linearly with depth of confinement. This characteristic is consistent with published data for cohesionless type materials. The modulus value is presented in terms of the effective overburden pressure (effects of submergence must be considered in determining the effective overburden pressure).

5.4.4 Puente Formation Bedrock

The weak Puente Formation claystone and siltstone materials were considered to act as very stiff to hard highly overconsolidated fine-grained soils for the purpose of our engineering evaluations.

Bedrock elastic properties of Table 5-1 were based on field performance data, published data, and engineering judgement. A constant modulus value is considered appropriate for the highly overconsolidated bedrock materials for depths under consideration. The selected constant modulus value presented in Table 5-1 is consistent with observed bedrock heave in the Los Angeles area (Evans, 1968).

The undrained strength parameters (0° friction and 5000 psf cohesion) were selected to represent the expected cohesive characteristics of the in situ bedrock. Measured permeability of intact samples of the bedrock was very low (10^{-7} to 10^{-8} cm/sec). However, the mass permeability of this unit including effects of fractures, joints and sandstone beds would be higher. The permeability value of Table 5-1 is intended to represent the mass permeability of the bedrock.

Section 6.0

Geotechnical Evaluations and Design Criteria – Union Station to First Street (Cut and Cover Portion)

6.0 GEOTECHNICAL EVALUATIONS AND DESIGN CRITERIA - Union Station to First Street (Cut-and-Cover Portion)

6.1 GENERAL

Construction of the proposed cut-and-cover section leading to the Yard and Shops will involve making a 70-foot deep excavation with major underpinning of the existing Hollywood Freeway, portions of which are supported on pile foundations penetrating the dense granular soils to depths below the roof elevations of the Y-line box structures. Such excavations will extend some 38 feet below the static water table. This will involve a major extensive dewatering effort and may require special construction provisions due to the possible occurrence of gases in the ground water and the high quantity of water to be handled. The permanent reinforced concrete box structures will bear on and retain dense granular soil. It is proposed that some of the freeway foundations will be supported on the completed box structures. Typical sections of the underpinning requirements are depicted at three locations along the alignment and are shown on Drawings 9, 10 and 11.

The primary geotechnical considerations along the cut-and-cover section of this design unit consist of:

- Construction dewatering and subsidence considerations;
- Design and construction of the underpinning of adjacent structures, and the Hollywood Freeway structures;
- Design of the temporary shoring systems;
- Identifying construction problems and solutions relating to the possible production of gas and oil during dewatering;
- Recommendations for soil and water loads on the permanent structure.

6.2 EARTHWORK

Earthwork required for the cut-and-cover box structure, the U-section structure and retaining walls will consist primarily of excavation for the subsurface construction. The earthwork will include removal of existing 28'x28'x12' concrete generator blocks, excavation and slab subgrade preparation, backfill for the subterranean box walls, underpinning of footings, and temporary dewatering installation. Recommended specifications for compaction of fill are presented in Appendix H. Suggested guidelines for subgrade preparation, structural backfill, trench backfill are presented below. Construction specifications should clearly define the responsibilities of the contractor for construction safety in accordance with CALOSHA standards.

- Structural Backfill: Excavated onsite clean natural sandy soils are suitable for use as backfill. Fine grained clay soils and bedrock are not considered suitable for fill due to their potential for expansion or consolidation. Loose soil, formwork and debris should be removed prior to backfilling the walls. Approved onsite materials or imported sand backfill should be placed and compacted in accordance with the recommended specifications of Appendix H. Where space limitations do not allow for conventional backfill compaction operations, special backfill materials and procedures may be required. Pea gravel, lean concrete, or other select backfill may be used in limited space areas. Recommendations for placement and densification of pea gravel or other special backfill can be provided during construction.

- Subgrade Preparation: Concrete box slabs-on-grade at the subterranean levels may be supported directly on undisturbed natural approved materials. The subgrade should be dense and undisturbed, and where disturbed, such areas should be recompacted in accordance with the recommended specifications set forth in Appendix H. For loose dry fills within the 24-inch zone below the slab should be moisture-conditioned to obtain near optimum water content and then compacted to at least 90% of the maximum dry density as determined by the ASTM D1557-70 compaction test method.

- Site Drainage: Adequate positive surface drainage should be provided away from the excavation to prevent seepage water from entering the excavation.

- Curing of Concrete: At the intersections of the different yard tracks east of the proposed Union Station, the A-Left track box will cross underneath the upper Yard and Shops Y-tracks box. The backfilling of the box structures should be sequenced by completion of the lower box, before the construction of the upper box. A delay period is required for backfilling after each concrete pour to allow the concrete to develop sufficient strength. Duration of the delay period should be determined by the design engineer.

6.3 EXCAVATION DEWATERING

6.3.1 General

Assuming an excavation depth to Elevation 214 at around Station AL93+50, the proposed excavation will extend approximately to 38 feet below the static ground water table (as measured in 1981 and 1983). Since the soils consist primarily of permeable sand and gravel, an extensive dewatering system will be required. Section 5.4.3 summarized the hydraulic characteristics of the soils to be dewatered based on the pumping test performed at the Union Station (see Appendix B). This section provides dewatering criteria, presents a possible dewatering scheme to aid in evaluating project feasibility, and identifies potential operating problems.

A deep slurry wall (diaphragm wall) could be installed through the permeable soils into the underlying Puente Formation. A deep slurry wall would provide an effective cut-off and essentially stop ground water flow into the excavation. This would eliminate the need for an extensive dewatering system although excavation sumps would still be required initially to dispose of the ground water in the excavated soils. However, the slurry wall would need to extend some 30 to 40 feet below the minimum wall depth required for excavation support. If properly designed and installed, the slurry wall would provide a technically acceptable alternative to extensive dewatering. However, such construction would be expected to experience delays when penetrating through horizons of cobbles and boulders.

As discussed in Section 5.3, gases, including methane and hydrogen sulfide, were released from the ground water during pumping. If comparable volumes of gas were to be released during construction dewatering then such releases may well give rise to several unsolved problems such as the effects of gas releases on pumping installations, the seriousness of offensive odors and the nature and magnitude of the hazards on construction safety within the construction site and downstream from it. Possible ways of mitigating these problems may be gas separators at the pump, well venting, gas removal systems at the surface, and use of construction provisions applied to gassy formations. No oil was detected in the borings or during the pump test performed during the Union Station investigation. However, the Union Station Oil Field, which reportedly produces oil in the Puente from very shallow depths, is located southeast of and in the vicinity of this site. In addition, Boring CEG-2 (location shown on Drawing No. 5), located some 2000 feet to the east, encountered oil and gas at a depth 38 feet. Sustained dewatering conceivably could draw oil into the area and result in pumping oil.

The impacts of water quality, as discussed in Section 5.2, also need to be incorporated into the design, installation, and operating procedures of a dewatering system.

6.3.2 Criteria for Dewatering Systems

The contractor is responsible for designing, installing, and operating a suitable construction dewatering system and should satisfy the following criteria:

- The dewatering system should be installed and in operation for a sufficient period prior to the excavation reaching the elevation of the static ground water level. The lead time depends on the maximum installed pumping capacity of the system.

- The system should maintain the ground water levels at least 2 feet below the lowest excavation level in the center and 4 feet along the excavation perimeter.

- The system should be operated continuously. Emergency power and backup pumps should be required to insure continuous excavation dewatering. This is essential at the Union Station and Hollywood Freeway crossing area since even a temporary disruption in dewatering could result in rapid flooding of the excavation and the possible development of localized "quick" conditions and possible localized failure of temporary support systems.

- The contractor should be made to recognize the potential environmental and operational problems caused by noise, poor quality ground water, strong gas odors from dissolved gases, and the possibility of pumping oil when designing his methods of disposing of the discharge from the wells. He should satisfy the applicable codes and ordinances for such discharge or disposal systems. The contractor will be required to submit water samples to the Water Quality Control Board for establishing guidelines as required by the Board.

- The wells must be designed and developed to eliminate loss of ground due to piping of fines. The well operations should be constantly monitored for evidence of piping.

6.3.3 Preliminary Analysis of a Dewatering Installation

It is believed that an appropriate dewatering system would utilize a number of 24-inch diameter wells penetrating the Puente Formation to maximize the capacity of the wells. This will require drilling the wells through the boulder zone. Alternatively, more wells could be installed terminating above the boulder zone. The wells would be located outside the proposed excavation and spaced at equal intervals along the sides of the excavation. The wells should consist of properly designed screens throughout the saturated zone with a bottom blank section to minimize piping of the fine-grained materials. It is estimated that the maximum drawdown at the well would have to be on the order of 45 feet or more to maintain the water levels noted in Section 6.3.2.

Dewatering is expected to release a mixture of gases and possibly oil if the wells penetrate into the lower sand and the Puente. Methods exist for separating gases from liquids at pump inlets. For example, gas shrouds on pumps can separate gases from the liquids being pumped. Special procedures may also be required to mitigate or prevent the accumulation of explosive and other hazardous gases within the work spaces. It is beyond the scope of our services to provide definitive data on these problems and to provide viable solutions. Working solutions to these problems can be obtained only by further investigations well in advance of or immediately prior to construction.

6.3.4 Subsidence

A major dewatering program operated for an extended period of time will lower the ground water table over a relatively large area. Depending on the dewatering system used, it is estimated that the drawdown will be on the order of 20 feet at a distance of 500 feet, 15 feet at a distance of 1000 feet, and 5 feet at a distance of 2000 feet from the Yard and Shops. This drawdown will cause an increase in the effective stress in the subsurface formations and, theoretically, result in some surface settlement due primarily to elastic compression of the alluvium affected. Assuming that the subsurface conditions within 1000 feet of the site are similar to those encountered in the borings, the theoretical settlement which may be caused by dewatering was estimated by means of computation. These calculations indicate that the surface settlement would be less than 1/2 inch for 20 feet of drawdown and less than 1/5 inch for 5 feet of drawdown. The ground would rebound to essentially its original elevations after dewatering is terminated as the effective stresses reduce to their original levels. In addition, the subsidence slope would be very gentle with differential settlement along any single structure being considerably less than the total.

Due to the proximity to existing facilities (buildings, the Hollywood Freeway crossing, rails, streets, utilities, etc.) the dewatering system must be designed and maintained to minimize loss of ground due to piping. Loss of ground due to piping could lead to severe ground subsidence, particularly near the wells.

6.4 UNDERPINNING

The need to underpin existing elevated Hollywood Freeway structures adjacent to deep excavations depends on many factors including the existing pile foundation design and construction records, soil conditions, depth of excavation, type of structures and proximity to the excavation, type of shoring, and consequence of potential ground movements. Figure 6-1 presents general guidelines for assessing when underpinning should be considered.

Based on the proximity of existing structures to the proposed cut-and-cover excavations as shown on Drawings 4 and 5, we believe underpinning will be required. Both the Hollywood Freeway structures and office buildings fall within the 1H:1-1/2V guideline shown on Figure 6-1. Anticipated ground movements adjacent to the excavation are discussed in Sections 6.3.4 and 6.5.6.

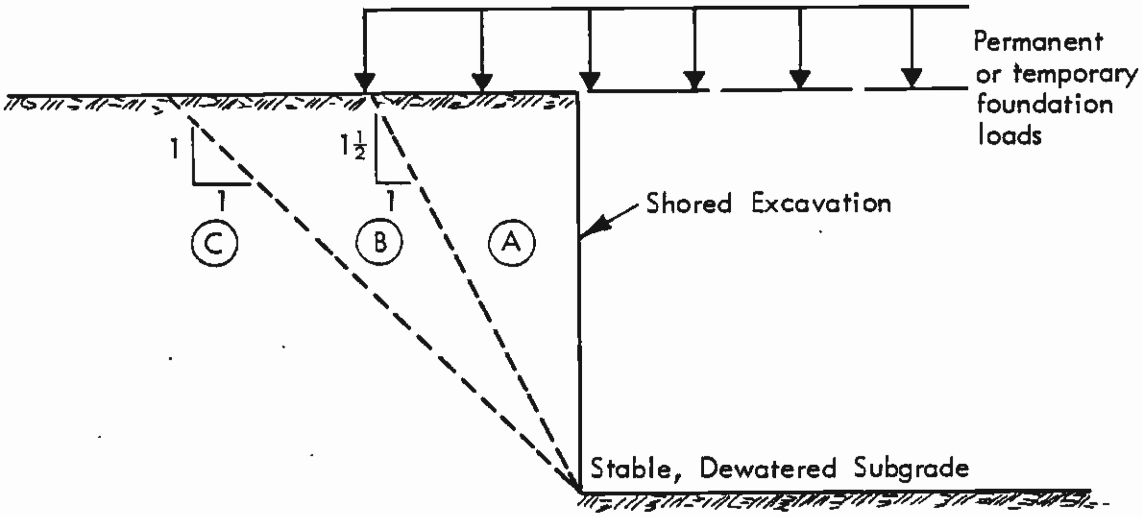
Careful consideration should be given to the underpinning method used due to both high caving potential and difficult driving conditions of the gravelly subsoils at the site.

If space limitation becomes a problem, a diaphragm wall system may be considered as one alternative method for functioning as an underpinning and a structural load bearing wall.

6.4.1 Underpinning of Existing Buildings, Foundation and Tracks

It is understood that all existing at-grade structures within the right of way of the proposed yard and shops track alignment will be removed. It is not known at the time of this writing whether the warehouse building to the northeast of Center Street and Commercial Street will be removed in its entirety or whether only a portion of it will be removed. However, some underpinning may be required even in the latter case. Drawings 8 and 9 show typical sections of the U-section and box in relation to the existing warehouse buildings.

We understand that one of the existing tracks located between the west leg of the retaining wall sections and the warehouse on the west will remain and be operational during construction. The track occurs approximately 20 feet west of the retaining walls while the warehouse is some 40 to 45 feet west of the walls. Though the underpinning criteria, Figure 6-1, suggest that underpinning of the warehouse will not be required, this preliminary conclusion presumes that a temporary vertical support system is utilized and that the quality of construction through this area will be superior. When such an assumption is not valid, the warehouse may require some underpinning and the track may require more than the usual maintenance to keep it continuously operational. We do not believe that a sloped excavation along the west side of this portal access area would be a feasible method of construction because of the proximity of the track and warehouse and the cohesionless nature of the soils involved. Section C-C', Drawing 8, is typical of the geometry in this area.



NOTE: These guidelines are applicable only for stiff or dense stable ground conditions. Other soil and/or foundation conditions will require other analyses.

Ⓐ <u>Special Provisions Required for Important Structures:</u>

Underpinning or construction of conservative shoring system (designed to support lateral loads from building foundations with acceptably small ground movements) must be considered.

Ⓑ <u>Generally No Special Provisions Required:</u>

Properly designed shoring system generally adequate without underpinning unless underlain by poor soils or adjacent to especially sensitive structures.

Ⓒ <u>No Special Provisions</u>

UNDERPINNING GUIDELINES

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Project No. 83-1140

Figure No. 6-1

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6.4.2 Underpinning of Hollywood Freeway

It is believed that underpinning of the existing foundations supporting the elevated Hollywood freeway will be necessary to construct the box structures passing beneath the freeway. It is assumed that the underpinning may be achieved by predrilled or driven piles which will serve as temporary support for the bridge piers and piles. Existing pile foundation tip elevations may vary between 255 and 265 feet. Typical sections of the proposed underpinning portions of the freeway structure are shown on Drawing Nos. 10 and 11, Geotechnical Sections E-E' and F-F'. The respective location of these two sections are indicated on Drawing Nos. 4 and 5.

The proposed bottom of the Y-Tracks box excavation will be approximately at elevation 240 feet, and the approximate top of box elevation is 265 feet based on the drawings provided by DMJM/PBQD. No underpinning support should be assumed above a line drawn at 45° upward from the toe of the excavation. Temporary underpinning piles shall be pre-drilled or driven to a point below the 45° "no-support" line prior to start of any excavations for the box structures. The vertical end bearing capacities for the temporary underpinning piles versus the penetration depth required beneath the "no-support" line are provided on Figure 6-2 for various pre-drilled and driven pile diameters. The sandy and gravelly nature of the Young Alluvium at the proposed box structure passing underneath the Freeway may give rise to some construction problems for both cast-in-place and driven type piles. However, we believe that the seamless steel pipe piles, driven open-ended may be the most desirable type of underpinning piles to be used. The pipe after having been driven to a firm bearing should be cleaned out with air or water jet and filled with concrete. The load-bearing capacity of the open pipe pile is figured on its supporting capacity by friction below the bottom of excavation and bearing in the dense to very dense sands.

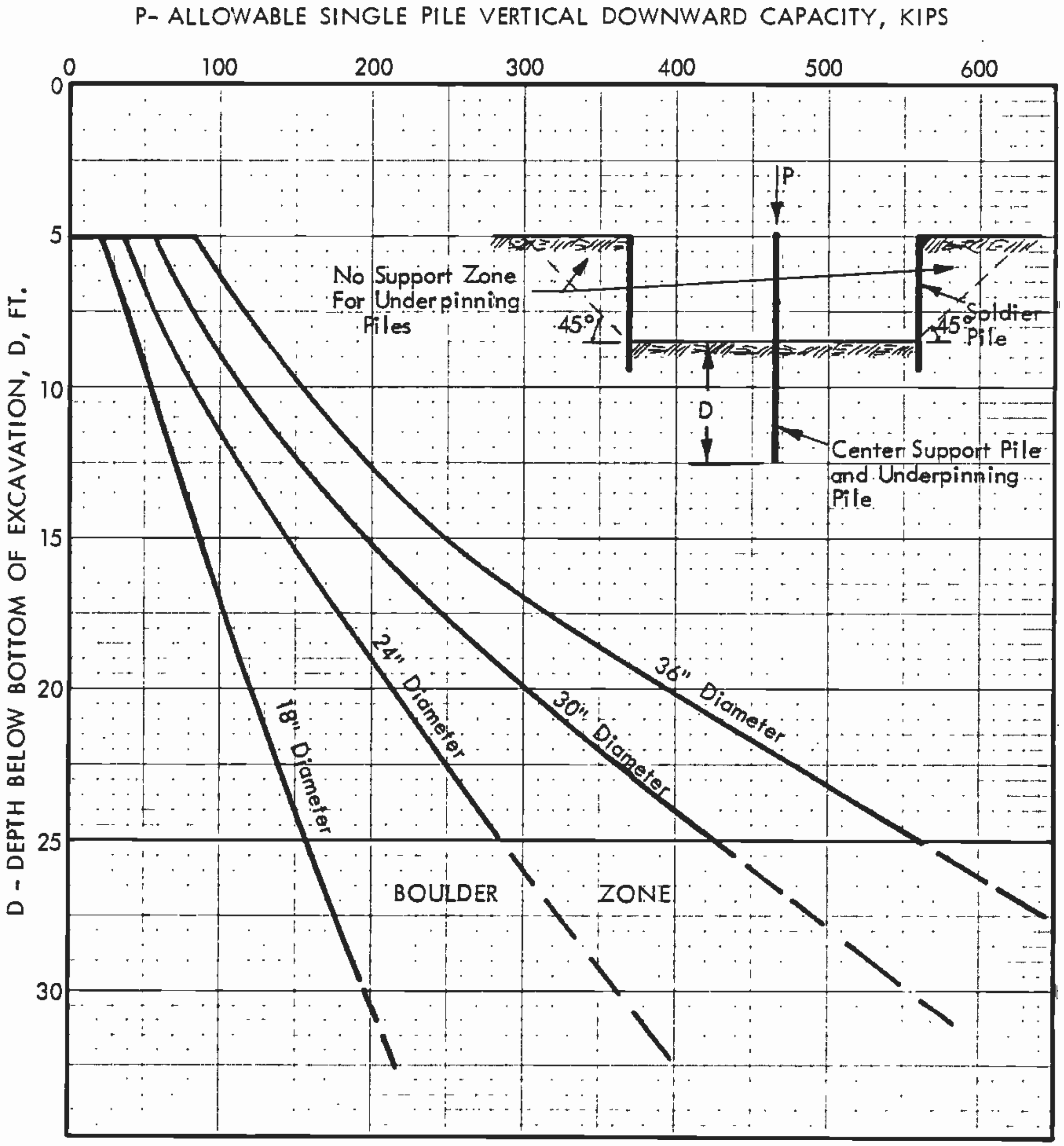
We recommend that after installation of the underpinning piles, a special pile load testing program should be performed prior to construction of the box excavation. Discussions on pile load tests for the underpinning piles are provided in Section 6.7 of this report.

6.5 TEMPORARY EXCAVATIONS

6.5.1 General

We understand that construction between the station and the freeway will consist in large measure of vertical faced excavations because of the existing physical environment such as streets and structures. In the yard proper and near Union Station, partially sloped excavations may be viable in certain areas.

Based on present design features, the depths of the excavations will vary between 70 feet in the vicinity of the station and freeway and 30 feet at the portal to the south.



NOTES:
1.) Bottom of excavation assumed at elev. 235 ft. for Y-Track Box and lower than 235 ft. for the individual A-Track Boxes.
2.) For seismic design, capacities may be increased 33%.
3.) Proper excavation clean-out assumed for End Bearing Support.

VERTICAL CAPACITY OF CENTER SUPPORT AND UNDERPINNING PILES

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Figure No.
6-2

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A number of excavation support systems may be utilized to support vertical sided excavations. Slurry or diaphragm wall systems, though feasible in the subsurface conditions described, may require more time to complete because of the presence of cobbles and boulders. Drilled soldier piles with lagging and tiebacks or internal cross lot bracing will also be a feasible support system. Drilling for soldier piles through the cobbles and boulders will have certain problems. Drilling and maintaining open shafts for tiebacks within the coarse granular alluvium will, however, test the ingenuity of contractors to maintain a cost effective rate of installation and a structurally sound tieback system. Excavations supported with steel sheet piles and internal bracing would be another possible excavation support system. However, we do not believe that steel sheet piles can be driven to great depths through the coarse granular materials without an inordinate amount of assistance, such as jetting, and at the same time maintaining the integrity of the interlocks.

The recommendations and discussions presented in this section pertain primarily to soldier pile and lagging type excavation support systems and sloped excavations.

6.5.2 Sloped Excavations

Portions of the required excavation nearest Union Station and the track yard area south of the Hollywood Freeway could possibly be made with a sloped excavation. To minimize slope instability and potential loss of ground due to occurrences of perched water which may persist after dewatering, sloped excavations should be limited to the soil zone above the static ground water table. Sloped excavations extending to the static ground water table would reduce the height of the temporary shoring by approximately 50%. Construction of a wide bench at the toe of the cut slope would provide access to the shored excavation but would increase the volume of excavated soil and the subsequent backfill. Sloped excavations would be feasible in the rail yard area where easements are available due to track removal.

Factors which will influence determination of a safe, stable slope include soil conditions, ground water conditions, the weather (i.e., dry or heavy rain), construction procedures and scheduling. Applicable governmental safety codes must also be complied with (refer to CALOSHA Article 6, 1540 d). In the final analysis, safe, stable construction slopes are normally the responsibility of the contractor and must be established in the field based on actual construction conditions.

Based on previous experience in similar soils, temporary construction slopes of 1.5H:1V through the fill and upper silty sands and 1H:1V through the dense gravelly sands would probably be suitable above the ground water level. This assumes suitable site dewatering, no heavy loads at the top of the slope, protection of the slope surface and some slope maintenance. These observations should not be construed by the contractor to be a guaranteed permissible slope. The actual slope used by the contractor may be different.

6.5.3 Soldier Pile Shoring System

A soldier pile shoring system consisting of soldier piles installed in predrilled holes and braced with tiebacks or internal struts is a common method of shoring deep excavations in the Los Angeles area. Appendix G summarizes several case studies in the Los Angeles area involving soldier pile and tieback excavations to depths exceeding 100 feet.

Soldier piles have been installed in the Los Angeles area in granular soils similar to those encountered at the proposed Yard and Shops area. In these types of soils, particularly below the ground water table, caving will be a problem. This may be more pronounced at the yard site due to the gravelly composition of the subsoils and the occurrences of cobbles and boulders. Potential caving conditions in drilled holes have sometimes been resolved by maintaining a head of water or slurry in the hole.

Granular soils exposed in vertical excavations at the Yard and Shops site will require support between soldier piles to eliminate loss of ground. Typically, wooden lagging is used although precast concrete or steel panels could also be used. Gunite has also been used in areas where soil arching between soldier piles allows time for guniting before soil sloughing occurs. At the Yard and Shops site the granular soils may be unsuitable for guniting due to the potential for sloughing, particularly below the static ground water table.

6.5.4 Tiebacks and Internal Bracing

Tiebacks and/or internal bracing will be required to support the temporary shoring walls for the proposed excavations. Tiebacks have the advantage of producing a clean, open excavation which can significantly simplify the excavation procedure and construction of the permanent structure. Previous experience indicates that there is not a significant difference in the maximum movement of properly designed and constructed internally braced walls and tieback walls. There may, however, be a difference in the distribution of the ground and wall movements. Generally, the maximum lateral movement of a tieback wall occurs near the top, while movement of an internally braced wall occurs near the base of the excavation. These differences are not the rule as soil type, prestress loads and construction details can alter the distribution. With tiebacks, there is less incentive for the contractor to excavate a significant distance below the designated support level prior to installing the supports. Thus the tieback wall is less prone to the contractor excavating too deep prior to installing supports. This potential problem can be minimized with appropriate specifications and construction inspection.

Tiebacks may be subject to creep which could result in additional movements particularly if the excavation is to be open for an extended period. When anchors are conservatively designed, creep will not be a problem in the competent soils for the box structures. On the other hand, struts are subject to strain and stress variations due to temperature changes. Such effects would be significant only if large temperature variations were permitted to occur.

There are numerous types of tieback anchors available including large diameter straight shaft friction anchors, belled anchors, driven anchors, high pressure grouted anchors, high pressure regroutable anchors, and others. Generally in the Los Angeles area, high capacity straight shaft anchors have been used. Due to the gravelly composition of the subsoils and the occurrences of cobbles and boulders, tieback installations will be a problem requiring use of casing, slurry, hollow stem auger and/or other procedures to maintain hole stability and minimize caving. Belled anchors are considered impractical at the Yard and Shops area site due to the potential caving problems. Driven, high pressure grouted anchors may be a feasible alternative but there is little experience with such anchors in the Los Angeles area, and anchor capacities would need to be verified in the field prior to full scale production of such a system.

6.5.5 Design Criteria

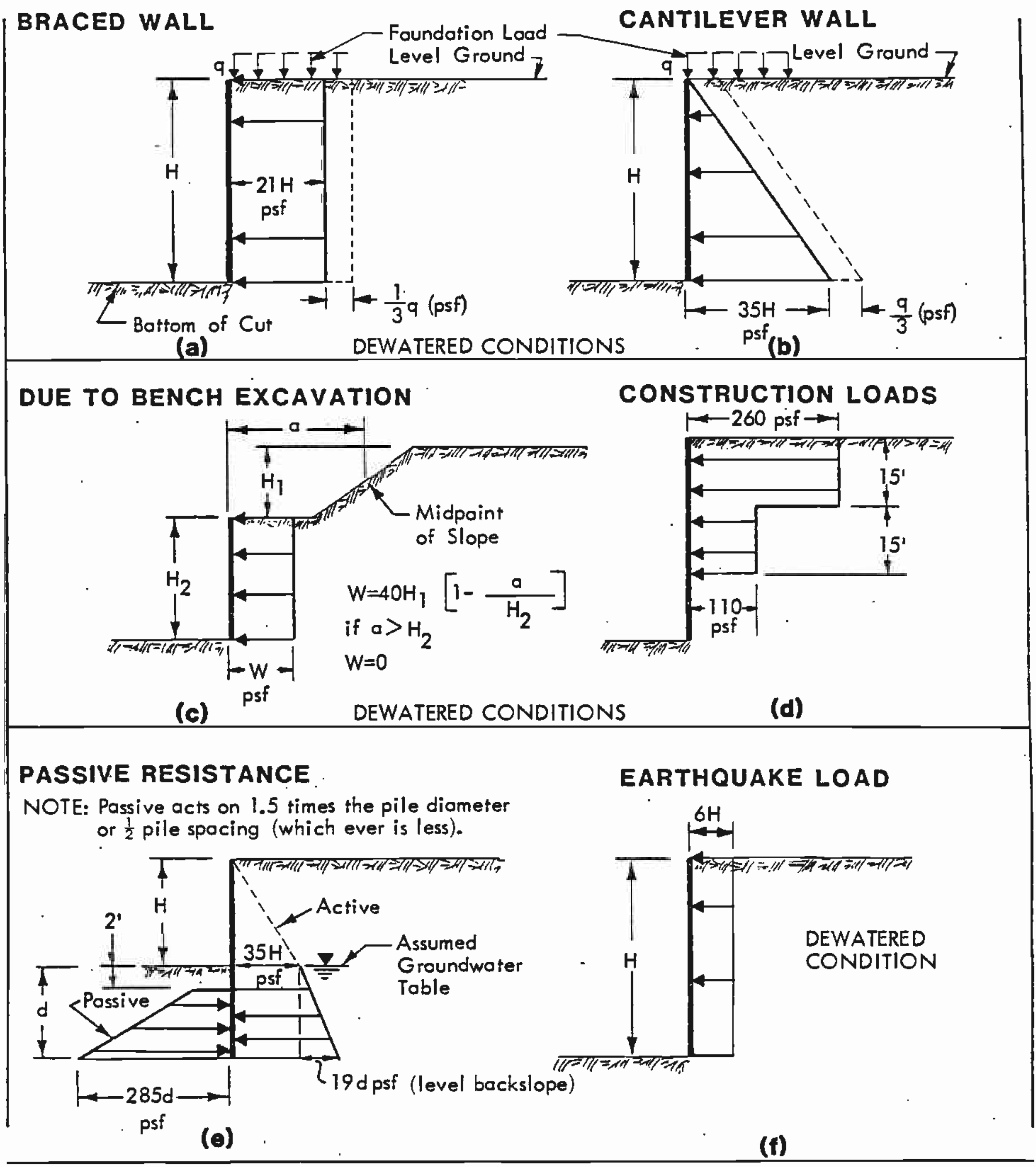
This section provides design criteria for a shoring system consisting of soldier piles, wooden lagging, and bracing - both cased tiebacks and struts. The soldier piles are assumed to consist of steel W- or H-sections installed in predrilled circular holes and that the drilled hole will be filled with structural concrete below the invert of the excavation and lean mix concrete above. Thus for computing the allowable vertical and horizontal pile loads on the soil, a circular concrete section was assumed.

Specific shoring design criteria include:

- Design Wall Pressure: Figure 6-3 presents the recommended design lateral earth pressure on the temporary walls. The figure includes the case of full depth shoring and the case of partial sloped cuts. The full loading diagram should be used to determine the design loads on tieback anchors or struts and the required depth of embedment of the soldier piles. For computing design stresses in the soldier piles, the computed values can be multiplied by 0.8. For sizing lagging, the earth pressures can be reduced by a factor of 0.5.

  The recommended design static lateral earth pressures are based on standard design practices and experience in the Los Angeles area. Appendix G summarizes the design shoring pressures used on nine typical projects in the Los Angeles area. Several of these involved excavation into Young Alluvium - similar to but not as coarse-grained as the Alluvium underlying Main Yard and Shops area. The range of shoring loads shown on Table G-1 were actual design values used. However, no field measurements were made to provide a comparison between actual pressures experienced and design values assumed.

- Depth of Soldier Piles: The depth of the soldier pile below the lowest anticipated excavation depth must be sufficient to safely carry both the lateral and vertical loads under static and dynamic loading conditions. The vertical loads on soldier piles supported with tiebacks can be substantial. In general the vertical load is equal to the imposed vertical load on top of the soldier pile plus the vertical component of tieback anchor load.



LOADS ON TEMPORARY FLEXIBLE WALLS (WITH DEWATERING)

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83-1140

Figure No.
6-3

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The required depth of soldier pile embedment to safely carry a given total vertical load in kips is shown on Figure 6-4.

The imposed lateral load on the embedded portion of the soldier pile should be computed based on the contributing area of the earth pressure envelope diagrams (Figures 6-3a to 6-3d, and 6-3f) below the lowest tieback or strut. The allowable passive resistance developed by the pile should be based on the passive pressure minus the active pressure below the bottom of the excavation, as shown on Figure 6-3e. Due to arching effects, it is recommended that the effective pile diameter be assumed equal 1.5 times the actual diameter or half of the pile spacing, whichever is less. Figure 6-3e indicates the recommended method to compute passive resistance.

The required depth of pile penetration should be based on both vertical loads and lateral loads, with the greatest value controlling the design.

- Pile Spacing and Lagging: The optimum pile spacing depends on many factors including soil loads, member sizes and costs, anchor load, and the ability for soils to arch. At the Yard and Shops area, the soils are granular and may contain pockets of water not fully drained by the construction dewatering system. Thus, it is recommended that the pile spacing be limited to about 6 feet and that continuous lagging be placed to minimize ravelling of soils and loss of ground between soldier piles. Use of geotextiles and limiting the temporarily exposed soil height should be implemented by the contractor to control ravelling and loss of ground problems.

- Tieback Anchor Design: The final tieback anchor capacity can be determined only in the field based on anchor load tests. For estimating purposes we recommend that the allowable design capacity of straight shaft friction anchors installed in drilled shafts be computed based on the following equation:

$$P = \pi DLq$$

where:

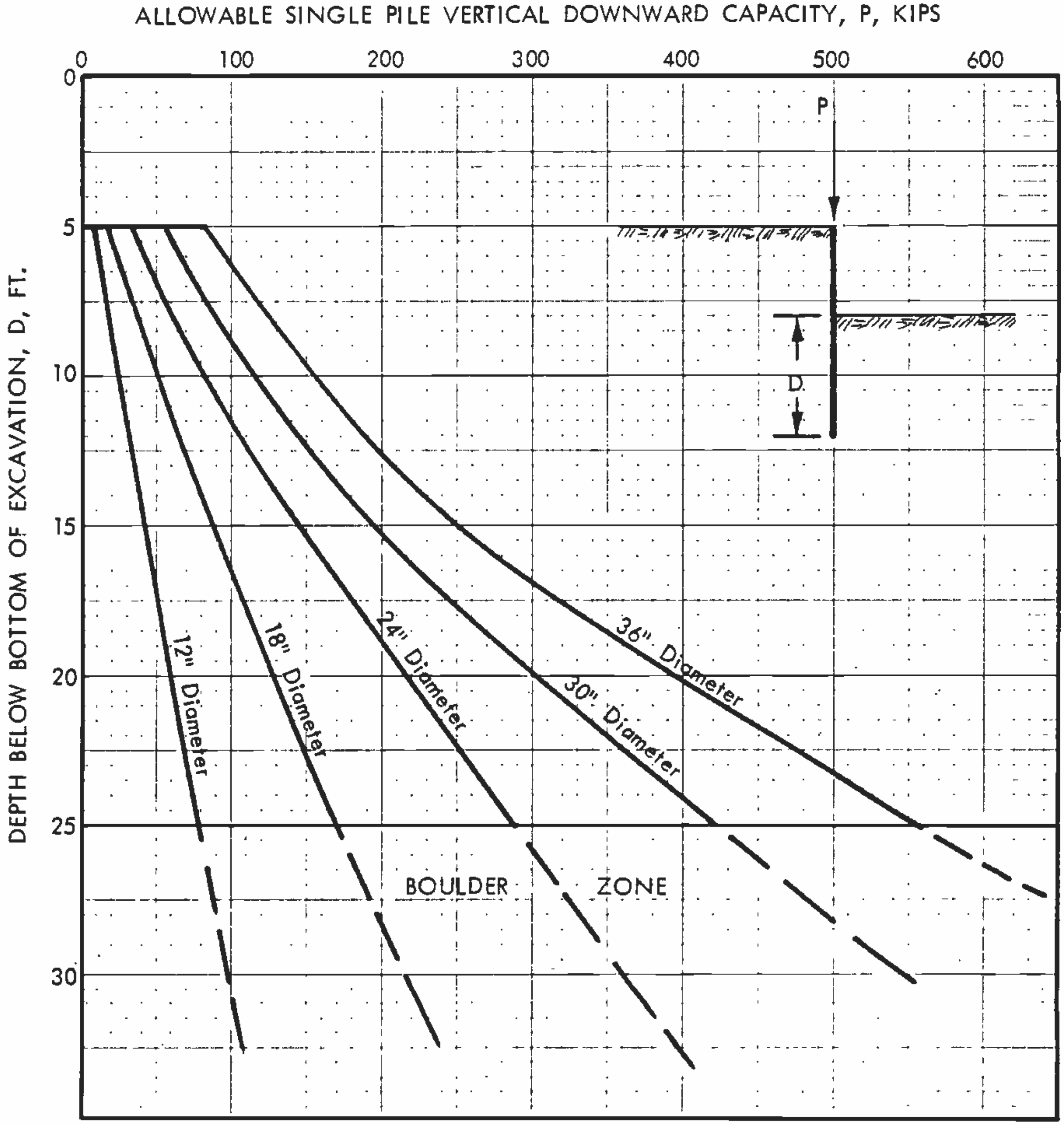
P = allowable anchor design load in pounds
D = drilled hole diameter in feet
L = anchor length beyond no load zone in feet
q = average soil resistance in psf.

The average soil resistance (q) can be assumed equal to:

$$q = 20d \leq 1000 \text{ psf}$$

where:

d = average depth of the anchor (in feet) beyond the potential failure plane; measured vertically from the ground surface.



NOTES: 1.) Bottom of excavation assumed at elev. 235 ft. for Y-Track Box and lower for the A-Track Boxes.
2.) For Seismic design, capacities may be increased 33%.
3.) Proper excavation clean-out assumed for End Bearing Support.

VERTICAL CAPACITY OF SOLDIER PILES

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Figure No.
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For design purposes, the potential failure surface behind the shored excavation may be a plane drawn at 35° with the vertical through the bottom of the excavation. Only the frictional resistance developed beyond the assumed failure plane should be assumed effective in resisting lateral loads. In addition no resistance should be assumed within the fill.

The anchors may be installed at angles between 20° and 50° below the horizontal. The contractor should be required to use a tieback anchor installation method which will minimize loss of ground due to caving. Uncontrolled caving not only causes installation problems but could result in surface subsidence and distress to adjacent structures and areas. To minimize caving, casing should be installed as the hole is advanced but must be pulled as the concrete is poured. Alternatively, a hollow stem auger could be used. Structural concrete should be placed in the lower portion of the drilled shaft up to the assumed failure plane. Pouring of the anchors should be done by pumping the concrete through a tremie or pipe extending to the bottom of the shaft. The anchor shaft between the failure plane and the face of the shoring must be backfilled with a sand slurry or equivalent after concrete placement.

Allowable anchor capacity for tieback types other than straight shaft friction anchors cannot be generalized. Capacity of anchors such as high pressure grouted anchors and high pressure regroutable anchors can be determined only in the field based on the results of field pull out tests.

- Tieback Prestressing and Testing: It is recommended that each tieback anchor be load tested to 150% of the design load and then locked off at the design load. At 150% of the design load, the anchor creep should not exceed 0.1 inch over a 15-minute period. In addition, 5% to 10% of the anchors should be test-loaded to 200% of the design load and then locked off at the design load. At 200% of the design load, the anchor creep should not exceed 0.15 inch over a 15-minute period. During each test procedure, the unit rates of creep during the observational period should also be ascertained to establish whether creep is in a decreasing or increasing mode. An increasing creep rate will in most cases lead to eventual failure under test load conditions.

- Internal Bracing: The contractor should not be allowed to extend the excavation an excessive distance below a strut level prior to installing the next level of struts. The maximum allowable distance depends somewhat on the tolerances for ground movements. A maximum vertical distance between an installed strut level and the base of the excavation of about 15 feet may be appropriate.

  To remove slack and limit ground movement, the struts should be preloaded. A preload equal to 50% of the design load is normally desirable for struts greater than 5 feet below the ground surface. The shoring design and preload procedures must provide for the effects of temperature changes.

  Joints between struts, piles and/or walls should be designed and constructed to carry a tension load equal to 10% of the design compression load of the strut.

6.5.6 Anticipated Ground Movements

Based on review of published data on shoring performance, we believe that the maximum horizontal deflection for a properly designed and installed wall will be equal to about 0.1% to 0.2% of the excavation depth. Thus for the proposed 30- to 70-foot depth excavation, the maximum wall deflection may vary between ½ to 1½ inches, depending on the depth of the excavation.

The maximum horizontal movement should occur near the top of the wall and decrease with depth if tiebacks are used. The maximum horizontal movement should occur near the base of a cross-lot braced shoring wall. The maximum settlement immediately behind and adjacent to the wall should be equal to about 50% to 100% of the maximum horizontal deflection.

6.6 SUPPORT OF TEMPORARY DECKING

Depending on the Yard and Shops operation requirements, it may be necessary to construct temporary decking over portions of the shored excavations. The deck span may be minimized by installation of piles along the centerline of the excavation. The natural dense granular soils encountered below the proposed excavation level are suitable for supporting pile loads. To minimize installation problems, piles should be designed to develop the required capacity at depths above the bouldery zone encountered in the Borings CEG 3, 4 and 5.

Recommended allowable design loads for piles installed in predrilled holes are shown on Figures 6-2 or 6-4, and include both end bearing and shaft friction. The end bearing component includes a higher factor of safety due to the greater movement required to develop end bearing.

Due to the occurrence of cobbles and boulders within the alluvium, we believe that driven piles may be difficult to install and will probably need to be predrilled to at least the bottom of the proposed excavation if installed in advance of any excavation. In addition, driven piles may induce settlements in the soil due to driving vibrations. For these reasons, we believe that driven piles would be a less favorable type of pile for the support of temporary decking.

6.7 PILE LOAD TEST(S)

Piles installed for underpinning support of the existing Hollywood Freeway foundations should be designed based on Figure 6-2 and the depth of embedment below the 45° "no-support" line. It is understood that each pile may be loaded on the order of 125 tons. We recommend that a pile load testing program be performed on at least one pile after the pile is installed. A static load test carrying up to at least 200 percent of the design load should be applied for a total of 24 hours during the test period. The testing procedures should be approved by the Construction Manager prior to installation. As a guide for the method of the pile load test, the ASTM Designation D1143-81 should be studied and followed.

6.8 INSTRUMENTATION OF THE EXCAVATION

In our opinion the proposed excavation at the Yard and Shops Cut-and-Cover Section should be instrumented to minimize liability (by having documentation of performance), to validate design and construction specifications, to identify problems before they become critical, and to obtain data valuable for future designs.

We recommend the following instrumentation program:

- Preconstruction Survey: A qualified civil engineer should complete a visual and photographic log of all streets, structures, major utilities, and pedestrian tunnel adjacent to the site prior to construction. This will minimize the risk associated with claims against the owner/contractor. If substantial cracks are noted in the existing structures, they should be measured and periodically remeasured during the construction period.

- Surface Survey Control: It is recommended that several locations around the excavation and on any nearby structures and the Hollywood Freeway structure be surveyed prior to any construction activity and then periodically monitored to detect vertical and horizontal movement to the nearest 0.01 feet. These should include points on the adjacent buildings and on top of the shoring wall (every fourth pile or a maximum distance of about 25 feet). The monitoring program should continue until after all construction and backfill is complete at the site.

- Heave Monitoring: The magnitude of the total ground heave should be measured. This information will be valuable in determining the ground response to load change and as an indirect check on the magnitude of the predicted settlement of the station structure.

  We recommend that heave monitoring devices be installed along the longitudinal centerline of the excavation on about 200-foot centers. The devices could consist of conical steel points, installed with the apex up in a borehole, and monitored with a probing rod that mates with the top of the conical point. The borehole may be filled with a thick colored slurry to maintain an open hole and allow for easy hole location as the excavation progresses. The top of the points should be at least 2 feet below the bottom of the final excavation to protect it from equipment yet allow for easy access should the hole collapse.

  The points should be installed and surveyed prior to starting excavation. Once the excavation begins, readings should be taken at regular intervals of excavation progress until the excavation is completed and then at about two-week intervals until all heave has stopped.

- Inclinometers: It is recommended that several inclinometers be installed and monitored around the excavation. At least one inclinometer should be located next to the building at Commercial Street close to the excavation. The casing could be installed within the soldier pile holes or in separate holes immediately adjacent to the shoring wall. If a slurry wall is used, the inclinometer casing should be installed in separate boreholes outside the proposed excavation prior to digging the slurry trench. This would permit the performance of the wall to be monitored throughout the installation phase. The casing should extend at least 30 feet below the final excavation level to ensure base fixity. Baseline readings of the inclinometers should be made immediately upon installation. Subsequent readings should be made at regular intervals of excavation progress.

- Gas and Oil Monitoring: The occurrence and concentration of gas and oil in the excavations and the discharge from dewatering should be monitored. It may also be prudent to install several shallow gas monitoring wells near adjacent structures to detect any potential increase in gas levels caused by site dewatering.

- Strut Load Measurements: If struts are used, it is recommended that strain gauges be used to monitor the stress conditions within the strut members. The strain gauges should be installed a short distance away from the ends to prevent end effects and to protect them from damage during construction. The location and number of gauges to be installed should be determined when a proper shoring system is being designed. If steel pipe struts are used, the strain gauges may be installed in sets of three around the pipe surface at 120 degree angles apart from each other. When H- or W-sections are to be used, it would be desirable to have both the flanges and the web members instrumented. The strain gauges should be securely installed prior to mounting and jacking of the members. The reading of the strain gauges should be performed early in the morning or late in the evening to avoid the thermal effects on the readings.

- Frequency of Readings: An appropriate frequency of instrumentation readings depends on many factors including the construction progress, the results of the instrumentation readings (i.e., if any unusual readings are obtained), costs, and other factors which cannot be generalized. All initial readings should be taken well in advance of any major construction activity. Subsequent readings should be taken as frequently as necessary to determine the behavior being monitored. For ground movements this should be no greater than one to two-week intervals during the major excavation phases of the work. Strut load measurements should be more frequent, possibly even daily, when significant construction activity is occurring near the strut (such as excavation, placement of another level of struts, etc.).

  The frequency of the readings should be increased if unusual behavior is observed.

- Supplementary Instrumentation: In addition to the above preplanned program, additional instrumentation may be appropriate during construction as a tool to aid in resolving specific construction concerns.

In our opinion, it is important that the installation and measurement of the instrumentation devices be under the direction and control of the Construction Manager. Experience has shown when the entire instrumentation program has been included in the bid package as a furnish and install item, the quality of the work has often been inadequate such that the data become questionable.

6.9 EXCAVATION HEAVE AND SETTLEMENT OF STRUCTURES

The major excavations for this design unit occur in coarse granular alluvium above and below the water table. Maximum excavation depths reach 70 feet with 50% of the excavation depth above and 50% below the water table. The coarse granular nature of the subsurface soils indicates that heave and settlements will occur rapidly and will not be time-dependent. Stress reductions due to excavations and removal of some freeway loads may attain magnitudes of 9000 psf. Individual box structure loads will probably not exceed 3000 psf. Hence time-dependent settlements of structures are not expected to be a problem. This conclusion does not apply to clayey pockets which may occur within the plan area of the structures (see Section 6.10.1).

On the other hand, stress relief due to excavations is expected to result in elastic shear deformations leading to elastic rebound at the bottom of the excavation. We estimate that, for excavations of maximum depth, the elastic rebound at the center of the excavation will be on the order of 1.5 to 3 inches; the majority of which is expected to have occurred upon completion of the excavations. For lesser depths of excavation, a linear relationship may be assumed. We estimate that imposed loads from the structures and backfill will result in immediate elastic settlements on the order of 1 to 1.5 inches, depending upon the loading conditions. Such settlements are expected to take place during the course of construction of the box structures and backfilling operations. The long and narrow nature and rigidity of the box structures will limit differential settlements to very low values; less than theoretically predicted.

6.10 FOUNDATION SYSTEMS FOR PERMANENT STRUCTURES

6.10.1 Union Station to Yard Portal Box Structures

At the proposed foundation level, the box structures are expected to bear on dense sand and gravels. Local interbeds of dense silty sands and/or stiff clay may be encountered at the foundation level. When encountered, they should be removed and replaced with approved materials prior to any placement of concrete. We estimate that the permanent average bearing pressure on the soil will vary between 500 and 3000 psf. In our opinion the natural soils at foundation levels can adequately support the box structures with minimal settlements. Section 6.9 presents estimated settlements for the proposed structures.

6.10.2 Support of the U-Section and the Retaining Wall Section

The transfer zone from the underground box structure to the at-grade rail yard consists of an approximately 648-foot U-section, and a 675-foot retaining wall section. The U-section will consist of a reinforced monolithic concrete slab structurally connecting cantilevered side walls. The section of final egress from the ground consists of reinforced concrete cantilevered retaining walls with no structural slab connecting the wall footings.

Based on the boring information, the U-section will be supported by the natural soil beneath the fill in this area. The natural alluvial silty sands are dense to very dense. Average foundation bearing pressures of the

U-section slab and side walls will probably be about 1000 psf. The ground water table is expected to be at around elevation 245 feet which is at approximately the bottom of the concrete slab if not a few feet below the bottom of slab.

Footing design for the retaining walls may be based on the allowable bearing values presented in Figure 6-5. The values shown are for full dead plus live loads. For transient loads, such as seismic and wind loads, the bearing values may be increased by 1/3.

For computing lateral resistance of retaining wall footings, a passive pressure of 285 pcf per foot of depth may be used together with a base friction value of 0.4.

6.10.3 Ducommun Street Storm Drain Crossing

An existing 11½-foot-diameter underground storm drain pipe passes beneath the yard area between Ducommun Street and the Los Angeles River. We understand that the existing storm drain will be replaced by a box structure where it passes beneath the north end of the proposed U-section. Based on the subsoil conditions found in Boring 3-2 and the adjacent Borings 3-1 and 3-3, the box will be founded within the gravelly sand layer beneath the upper alluvial sand. Due to its closeness to the bottom of the U-section, it may adversely affect the differential settlement characteristics between the U-section and the box section. We recommended that the alignment of the storm drain be moved south of the construction joint in this area or that the location of the construction joint be made to avoid the storm drain alignment.

6.11 LOADS ON PERMANENT SLAB AND WALLS

6.11.1 Hydrostatic Pressures

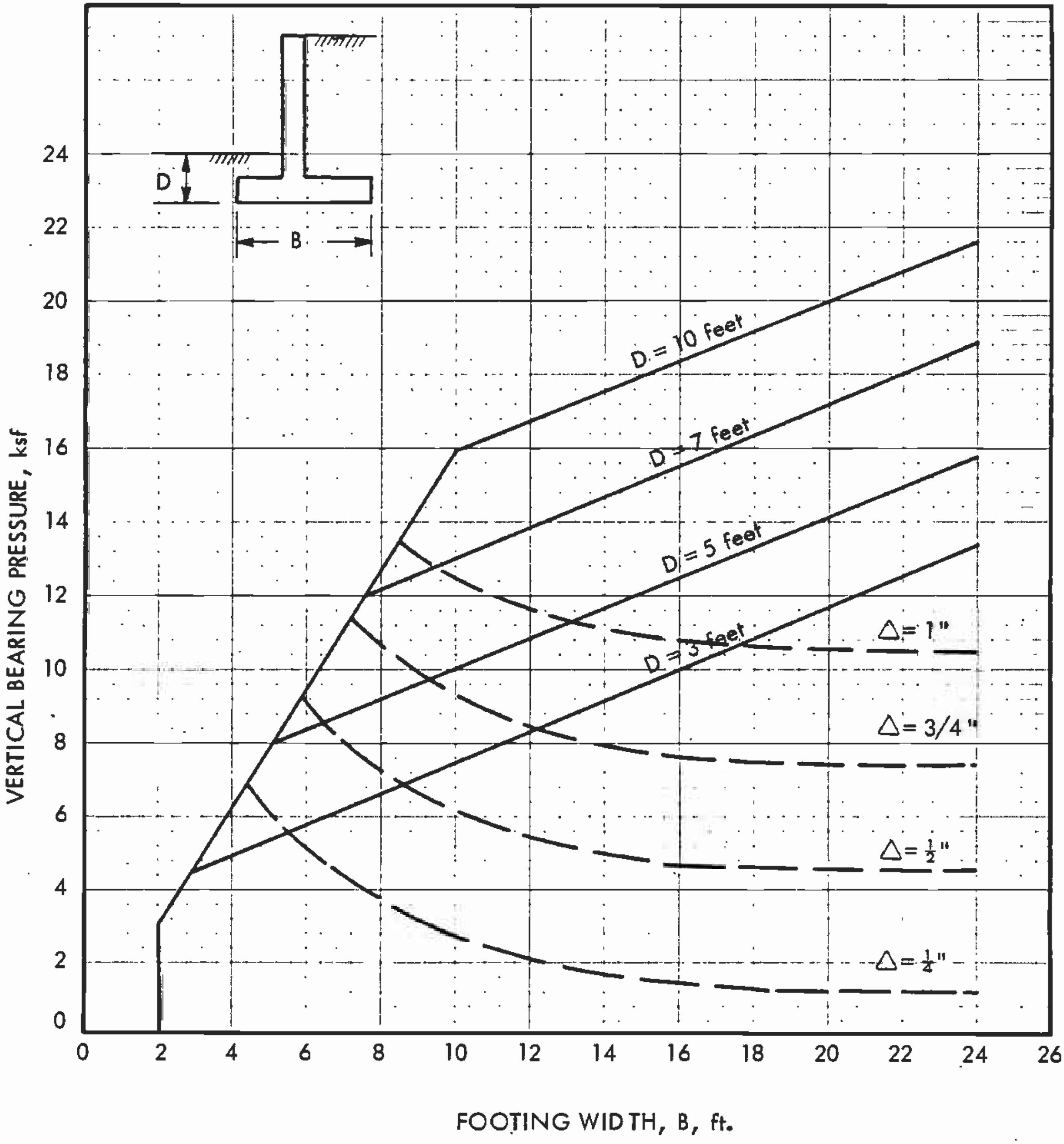
The ground water table as measured during 1983 varies across the site and appears to range between approximately Elevation 260 feet at the Union Station end and Elevation 245 at the north end of the Yard and Shops area and Elevation 236 at the south end of the Yard and Shops. Considering that the winter of 1982 was one of the five wettest years in the past 100 years, we recommend that the following maximum ground water levels be assumed for design purposes.

Location	Elevation	Hydrostatic Pressure Uplift On the Slab* (psf)
Union Station End of Box	260	2450
Yard Portal Box Structure	245	500

*Assuming bottom of slab, Elevation 215 at Union Station End of Box (for Track AR) and Elevation 240' at Portal for Box (Track AL).

Maximum water level elevations for areas between the ends of the structures may be linearly interpolated.

NOTE: 1) Applicable only to footings on dense granular alluvium or properly compacted granular fill above ground water table.
2) D = depth below the bottom of final grade.
3) △ = total footing settlement



GEOTECHNICAL DESIGN CRITERIA FOR RETAINING WALL FOOTINGS

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Figure No.
6-5

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6.11.2 Permanent Static Earth Pressures

We recommend that the permanent static lateral earth pressures be based on the anticipated at-rest conditions. For this condition, we recommend that the pressure be computed on the basis of an equivalent fluid with a density of 50 pcf above the ground water table and 30 pcf below the ground water table.

The pressures on the roof should be assumed equal to the full weight of the overburden soil (which may be assumed to be 130 pcf above the water level and 70 pcf below the water level) plus hydrostatic pressures and any surcharge for static loading conditions. In addition, surcharge loads due to surface rail or vehicular traffic should be considered as well as loads due to Hollywood Freeway structure and earthmoving equipment during construction operations.

6.11.3 Recommended Earth Pressure Diagrams

Two sets of recommended permanent earth pressure diagrams are provided. Figure 6-6 presents the recommended design earth pressure diagrams for the box structure walls, and Figure 6-7 presents the recommended design earth pressure diagrams for the U-section and retaining walls section.

6.12 COEFFICIENT OF VERTICAL SUBGRADE REACTION

6.12.1 General

The SCRTD System Design Criteria and Standards, Underground Structures, Section 2.4.1.B states that foundation stress analyses for stations and similar underground structures shall consider variations in the elastic support of the subgrade for different conditions. This is interpreted by the section designers to mean a method of analysis which employs a coefficient of subgrade reaction Ks. Though no comparable criterion is specified for analyses of at-grade foundation systems, the designer, nevertheless, has requested that values of K_s be provided also for the design of shallow systems in the Yard and Shops area. To meet these requirements, this report presents such a value with the following qualifications.

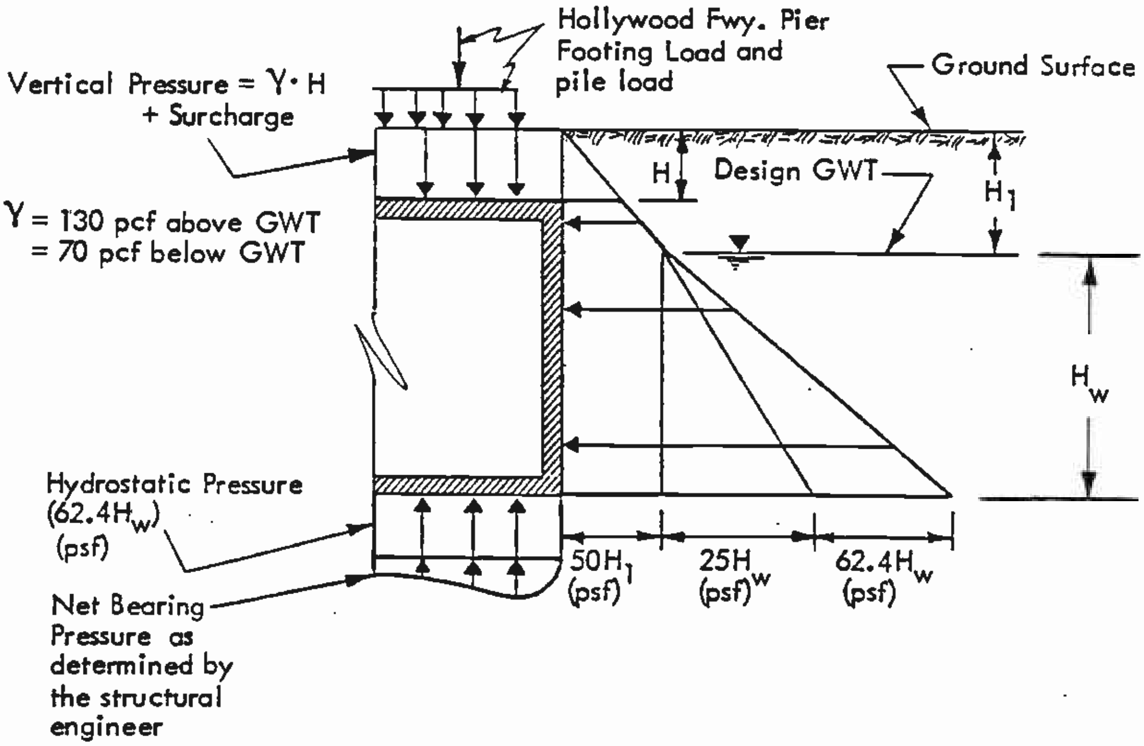
The true value of Ks depends on a variety of factors such as: stress deformation characteristics of the soil (i.e., is the soil compressible or not for all practical purposes); shape of the loaded area; size of the loaded area; magnitude of the load on the area; the depth of the loaded area, and the position of nearby loads. It is apparent that the evaluation of Ks for design purposes requires much consideration to overcome the many uncertainties inherent in this method of analysis. The value and relationships suggested in the following section are based upon a review of the literature, experience and judgement. Actual values may vary from that given.

6.12.2 At-Grade Foundations

We suggest the basic value of $\bar{K}_{s1}$ for a 1'x 1' square rigid plate at the ground surface of a compact to dense cohesionless granular material common to this site be:

$$\bar{K}_{s1} = 150 \text{ tons/ft}^3$$

STATIC PRESSURE



Design GWT:

1) During Construction: Below Floor Slab

2) Permanent Construction:

South End at Portal: El 245'
East End of Union Station: El 260'
Locations Between: Interpolate

LOADS ON PERMANENT WALLS (BOX)

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Figure No. 6-6

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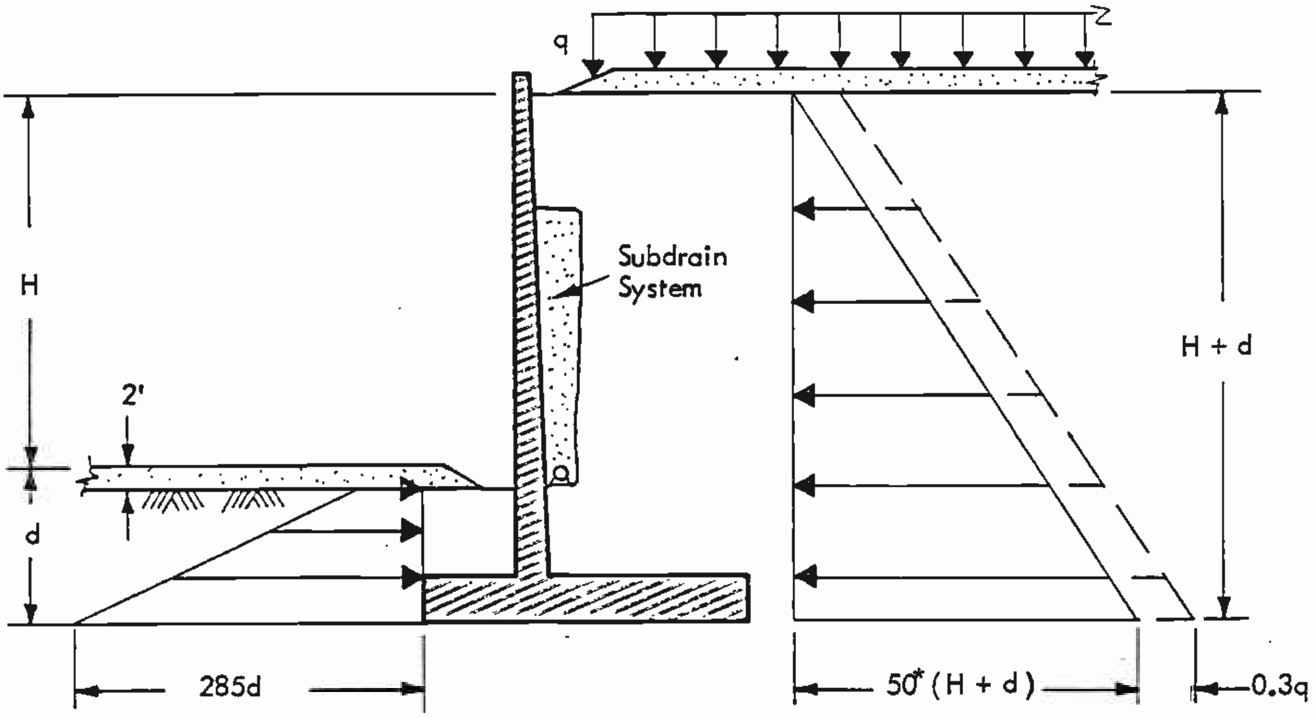

NOTE:
1) Ground water table not within a depth equal to footing width.
2) For earth surcharge, q = rh
   Where: r = 130 pcf
   h = depth of earth surcharge in feet
3)* The Lateral pressure 50 pcf allows for backfill compaction effects.

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LOADS ON PERMANENT WALLS (U-SECTION AND RETAINING WALLS)

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Figure No.
6-7

Values for larger square footings, rectangular footings and concrete floor slabs with column loads may be derived from the following relationships assuming that $\bar{K}_{s1}$ remains essentially constant for depths not exceeding 5 feet.

- Square footings of width B, the vertical coefficient of subgrade reaction,

$$K_s = \bar{K}_{s1}\left(\frac{B+1}{2b}\right)^2$$

- Finite rectangular footing of width B and Length L,

$$K_s = K_{s1}\left(\frac{1}{B}\right) \text{ and } K_{s1} = \bar{K}_{s1}\left(\frac{L+0.5}{1.5L}\right)$$

- Long rectangular or continuous footing of width B.

$$K_s = \bar{K}_{s1}\left(\frac{1}{1.5B}\right)$$

- Concrete floor slab with column loads.

$$K_s = \bar{K}_{s1}\left(\frac{1}{2R_1}\right)$$

As a general rule R_1 may be assumed equal to 7 times the thickness of the slab.

The range of influence of the concentrated load, and that portion of the mat which is located within a distance R from the point of load application is;

$$R = \sqrt[4]{\frac{10Eh^3}{3(1-\nu^2)K_s}} = 2.5r_o$$

where r_o = radius of stiffness of the slab

E = Modulus of Elasticity of concrete

ν = Poisson's ratio of concrete

h = thickness of slab

If the difference between the assumed R_1 and the estimated R is greater than 50%, the computed value of R should be used to recalculate the K_s and the R until the difference between R_1 and R is less than 50%.

Use $\bar{K}_{s1}$ for computing settlement for a footing 1.5B above the water table. With the ground water at the base of the footing, use $(\bar{K}_{s1})/2$. For intermediate positions of the water level, interpolate between the above two values.

6.12.3 Buried Box Structure and U-Section

The proposed cut-and-cover structures will vary in their bottom elevations for the different line tracks. Generally, the invert elevations will be near or below the ground water level. The coefficient of subgrade reaction, $\bar{K}_{s1}$, for the cut-and-cover box structure may be assumed to increase linearly with depth. We suggest that, for foundations embedded more than 5 feet below the lowest adjacent final grade, the rate of increase of the coefficient of subgrade reaction be 6 tons per cubic foot per foot of depth, or

$$\bar{K}_{s1} = 150 + 6D \text{ (tons/ft}^3\text{)}$$

where

D = depth to the invert of the box or U-section structure in feet.

The design coefficient of subgrade reaction, K_s, can be evaluated from the long beam or continuous footing relationship provided in Section 6.12.2.

We understand that MRTC is contemplating to modify the Design Criteria and Standards for underground structures to permit use of a simplifying and slightly more conservative assumption resulting in a uniform net foundation bearing pressure for the design of the invert slabs of box structures. The use of the elastic soil-structure analysis or the above simplifying approach is left to the discretion of MRTC and the Section Designer.

Section 7.0

Geotechnical Evaluations and Design Criteria – First Street to Main Yard and Shops (At-Grade Portion)

7.0 GEOTECHNICAL EVALUATION AND DESIGN CRITERIA - First Street to Main Yard and Shops (At-Grade Portion)

7.1 GENERAL

The plans for the Yard track's area call for cutting the area on the average of 2 to 10 feet to achieve the design subgrade of the yard south of the First Street crossing, and maintaining or raising existing grades within the shop building areas. North of First Street, the yard area will be excavated on the order of 6 and 23 feet, whereas the yard area in the south will be filled approximately 2.5 feet above the existing ground surface elevation.

The at-grade shop buildings and parking lots located to the west of the track yard will include

- a Main Shop Building approximately 300 by 340 feet in plan dimensions
- a Maintenance-of-Way Building of approximately 128 by 264 feet
- a Transportation Building of approximately 120 by 120 feet
- a Car Washing and a Car Cleaner Building, each approximately 30 by 130 feet in plan dimension
- a substation area of approximately 50 by 120 feet
- roughly some 2.8 acres of paved parking areas.

7.2 EARTHWORK

The grading work noted above will include:

- Removal of old tracks and structures, and site preparation
- Rail track yard drainage systems and placement of sub ballast and ballast courses
- Fill and backfill in the proposed Shops and pavement areas.

Suggested guidelines for specifications for compaction of fill are presented in Appendix H.

7.2.1 Site Preparation - General

Most of the existing Santa Fe tracks and structures within the Yard and Shops are to be removed. This will involve removal of all existing rails, railroad ties, switch-boxes, conduits, light stands, utilities, drainage ditches, ballast, A.C. pavement, and retaining walls of comparatively low height. The most easterly lead and main Santa Fe tracks adjacent to the Los Angeles River will remain in service during construction.

Within the existing warehouse and truck trailer parking areas, existing structures, facilities, buildings, corn storage silos, and roadway pavements will be demolished and removed from the site to permit preparation of the subgrade for the yard and storage tracks, and the new Transportation, Maintenance-of-Way, Main Shop facilities, substation, car cleaners and car washing buildings.

Cuts on the order of 10 feet are required in the northern portion of the yard between First Street and east of the proposed Transportation Building. Generally between 2 and 5 feet of cut will be required in the track yard areas. We suggest that all concrete foundations, pavements, conduits, rail ties, and debris in the upper zone within the planned area of the yard and shops be removed off the site. The existing subsoils, which are comprised of existing fill and natural alluvial sand, can be either removed from the site or stockpiled for further use as fill. If the contractor chooses to incorporate concrete into certain fill areas, the concrete should be crushed to a maximum size not to exceed 4 inches before mixing with finer granular fill materials. There appears to be more cut than fill according to the grading cross-sections.

Upon completion of demolition, stripping and rough grading, the subgrade should be proof-rolled to identify loose and/or compressible zones which may occur locally across the site.

Abandoned or buried structures, if encountered during rough grading operations, should be excavated, broken up where possible, and removed from the site. Depressions resulting from such removal should be graded and backfilled in layers with approved materials, and compacted to the compaction requirements noted in Appendix H.

Upon completion of the rough grading operations, the subgrade should be prepared for accepting structures by scarifying the surface to a depth of 6 inches, removing oversized materials, and recompacting to a minimum relative compaction of 90%. The finished subgrade should be sloped to provide positive drainage at all times prior to the placement of the sub ballast or structural fill. Sub ballast fill should be of granular materials meeting the gradation and compaction requirements of the American Railway Engineering Association specifications as provided by the 1982 edition.

7.2.2 Site Preparation - Shops and Pavement Areas

The proposed shop buildings are to be founded on prepared bases either at existing grade or on a few feet of engineered fill above existing grades. The majority of these sites are now occupied by warehouses, office buildings and/or paved roadways and parking lots. The new building areas are underlain by granular fill having a relative density varying between medium dense and compact, the thicknesses of which varied between one and 12 feet. The nature of the random occurrence of the granular fill suggests a relative compaction which may vary from that recorded during the exploration. To avoid potential future problems with completed buildings, it is recommended that, upon completion of stripping and rough grading and before preparation of the subgrade, the new building sites be proof-rolled with a 50-ton rubber-tired roller. Excessive and abnormal depressions should be identified, the unsuitable materials removed and replaced with approved materials compacted in layers. A similar procedure should be followed in the proposed new roadway and parking areas. All compaction specifications should be in accordance with the recommendations in Appendix H.

Proposed finish floor slab elevation of the main shop building, and the maintenance pits are schematically shown on Drawing No. 6 with the existing ground surface in the typical section A-A'. The proposed grading and finish floor elevations of the Maintenance-of-Way Building and the Transportation Building are depicted on Drawing No. 7, Section B-B'.

7.2.3 Relocation of Storm Drains, Oil Pipelines

Preliminary plans indicate that the existing underground 11½-foot-diameter and 42-inch-diameter storm drain system underneath Ducommun Street will be relocated. Details are not available at this time, but we understand that the drains may be lowered in elevation to pass under the U-section and box structures along their present horizontal alignment. To achieve these revisions to the drains, it may prove necessary to utilize shoring and dewatering installations similar to those described in Section 6.5, Temporary Excavations, of this report.

The details for relocating the Chevron oil pipeline are not known at this time. When details of this and other relocations of existing facilities are available, this office should review the proposed designs and comment on their viability.

7.3 DESIGN COLUMN LOADS AND SPREAD FOOTING FOUNDATIONS

Table 7-1 provides the general design loads for the buildings in the yard areas as provided by the Section Designer.

TABLE 7-1
PROPOSED SHOP BUILDING DESIGN LOADS

Item	Estimated Total Column Load (D.L. + L.L.) kips	Crane Load (kips)	Column Spacing (ft)
Main Shop Building	50	200	24 - 80
Maintenance-of-Way Building	60	110	24 - 80
Transportation Building	40	none	24 - 24
Car Washing Building	10	none	24 - 24
Car Cleaning Building	20	none	12 - 36

Conventional spread footings founded within dense granular alluvium or properly compacted granular fill are proposed for the support of the structures in the Main Yard and Shops area.

Borings drilled within the proposed Yard and Shops area indicated that localized zones of loose to medium dense granular fill up to 8 and 12 feet thick may be encountered. It is unlikely that deeper fills would exist in great extent at the site; however, some localized deeper fill may exist. We have developed geotechnical footing design criteria for square and rectangular spread footings considering the possibility that fill zones may exist in the foundation areas. We recommend that, where existing fill soils at column locations are found to be unsatisfactory, the fill be replaced with a zone of compacted structural fill with a minimum depth beneath the bottom of footings equal to 1.5 times the minimum footing width, B, or to dense natural soils, whichever is less. Excavated granular fill soils may be used for construction of the structural fill zone if approved in the field by the soils engineer. Schematic illustration of the recommended structural fill zone for footing support is shown on Figure 7-1. Similarly, if a floor slab is underlain by loose to medium dense fill, a minimum of 18 inches of recompaction is required. Recommended floor slab design and subgrade recompaction is shown on Figure 7-2.

Allowable bearing capacities for footings founded on natural soils and/or structural fill as discussed above, are presented on Figures 7-3 and 7-4 for spread footings for all proposed shop buildings and structures. These bearing values are for non submerged conditions, since ground water is not anticipated within 20 feet of the existing ground surface. The figures provide settlement estimates for the different footing embedment depths, and dimensions shown. In the event it is necessary to extrapolate bearing capacities and settlements beyond the limits of Figure 7-3 and 7-4, this office should be notified.

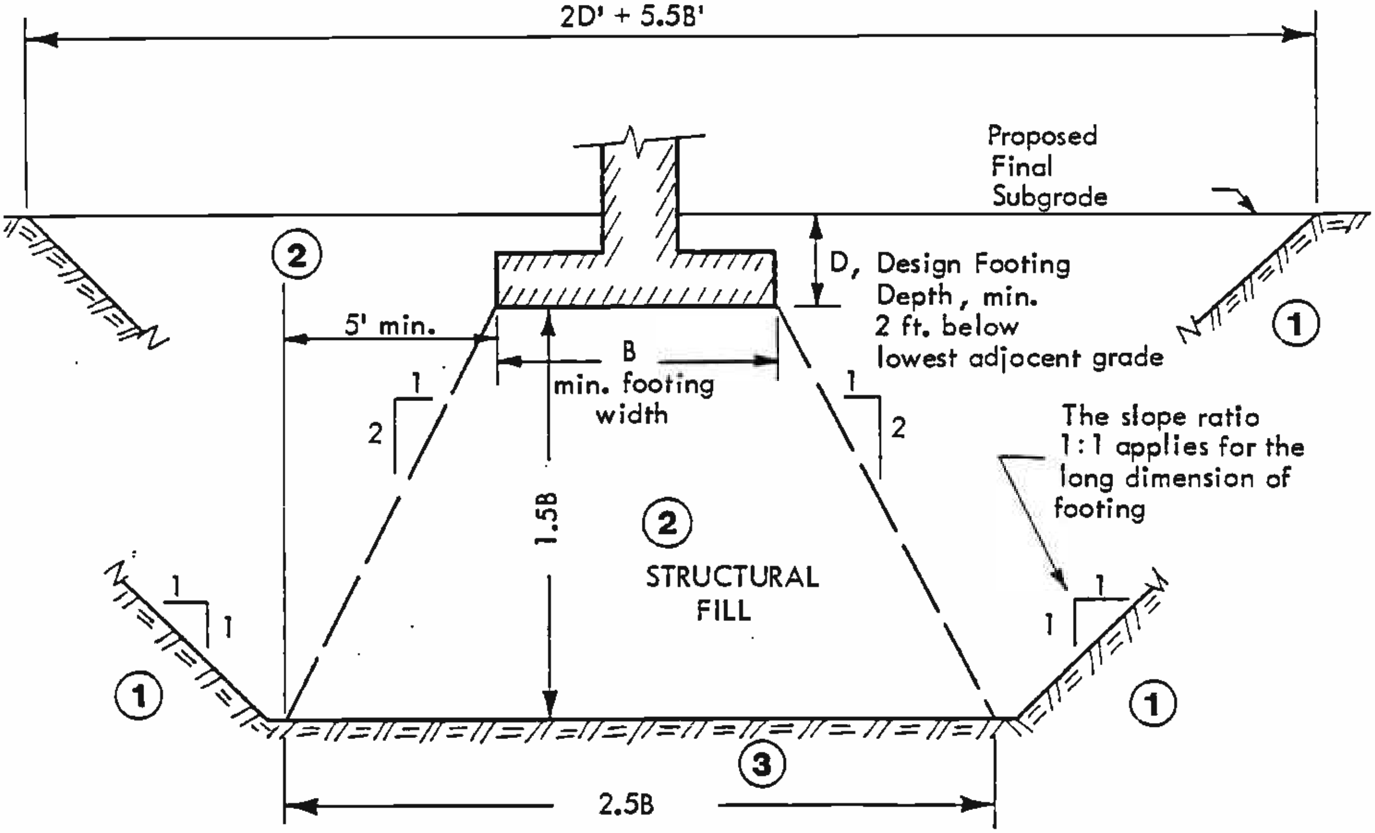
Differential settlements between isolated square footings may be taken to be the difference between the estimated total settlements presented on Figure 7-3 and 7-4 or a minimum of 1/4 of the average estimated total settlement, whichever is greater. Based on the expected range of structural loads and the settlement data of Figures 7-3 and 7-4, it appears that differential settlements between square footings can be kept to tolerable levels by utilizing bearing pressures in the range of 4 ksf to 6 ksf.

Foundation resistance to lateral loads can be assumed to be provided by friction acting at the base of the footings and by passive earth pressure. A coefficient of friction of 0.4 may be assumed with dead load forces. An allowable passive earth pressure of 285 pound per square foot per foot depth may be used for the sides of footings to resist transient loadings due to wind and seismic forces.

The bearing values indicated above are for the total working loads consisting of dead loads and frequently applied live loads and the crane loads. The suggested vertical bearing values may be exceeded by 33% for live loads of short duration caused by wind or seismic forces. The allowable passive pressure may also be increased by 33% for the same loading conditions.

7.4 SERVICE PIT, LOADING DOCK AND BASEMENT WALLS

Proposed service and inspection pits in the Main Shop Building vary between 9 to 14 feet deep, and are 14 feet wide. The pits are utilized for the car hoist, body stand and other servicing and maintenance functions. The side walls of these pits should be designed as retaining walls, supporting the full lateral earth pressures for the native soils, the surcharge and train track loading as necessary. The lateral pressure diagram shown on Figure 6-7 should be used for a cantilever wall design, and that shown on Figure 6-6 should be used if the wall is rigidly tied in with the floor slab. Since the rail



① Existing fill to be excavated and recompacted (if suitable to be used as structural fill). Minimum depth of removal to be 1.5B below footing bottom, or to dense natural soils, whichever is less.

② Structural fill should be compacted by mechanical means in accordance with Earthwork recommendations, Appendix H.

③ Bottom excavation width may be increased at contractor's convenience to permit use of earthmoving equipment.

NOTE: 1) Ground water level below a depth of 1.5B.

RECOMPACTION ZONE FOR SUPPORT OF SPREAD FOOTINGS

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Figure No.
7-1

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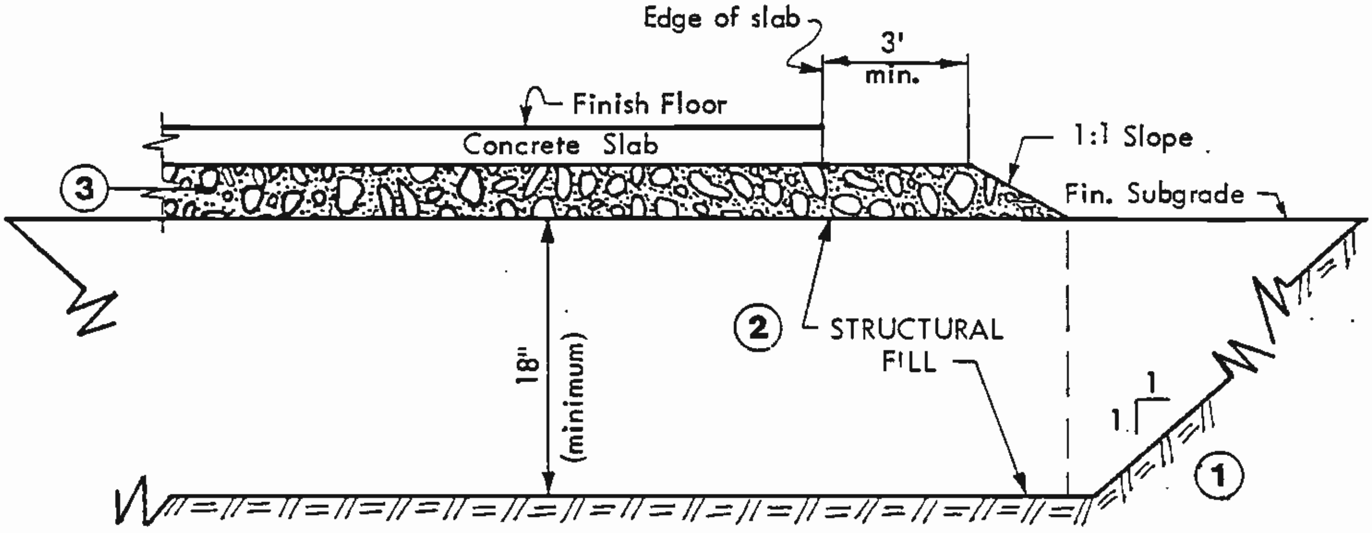

① Existing fill to be excavated and recompacted (if suitable to be used as structural fill). Minimum depth 18 inches below the finished subgrade of floor slab system.

② Structural fill should be compacted by mechanical means in accordance with Earthwork recommendations, Appendix H.

③ 4-inch layer of crushed rock conforming to 3/4" maximum size, compacted to 95% relative compaction as specified in the Standard Specification for Public Works Construction. Section 200-1.

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RECOMPACTION ZONE FOR SUPPORT OF CONCRETE FLOOR SLAB

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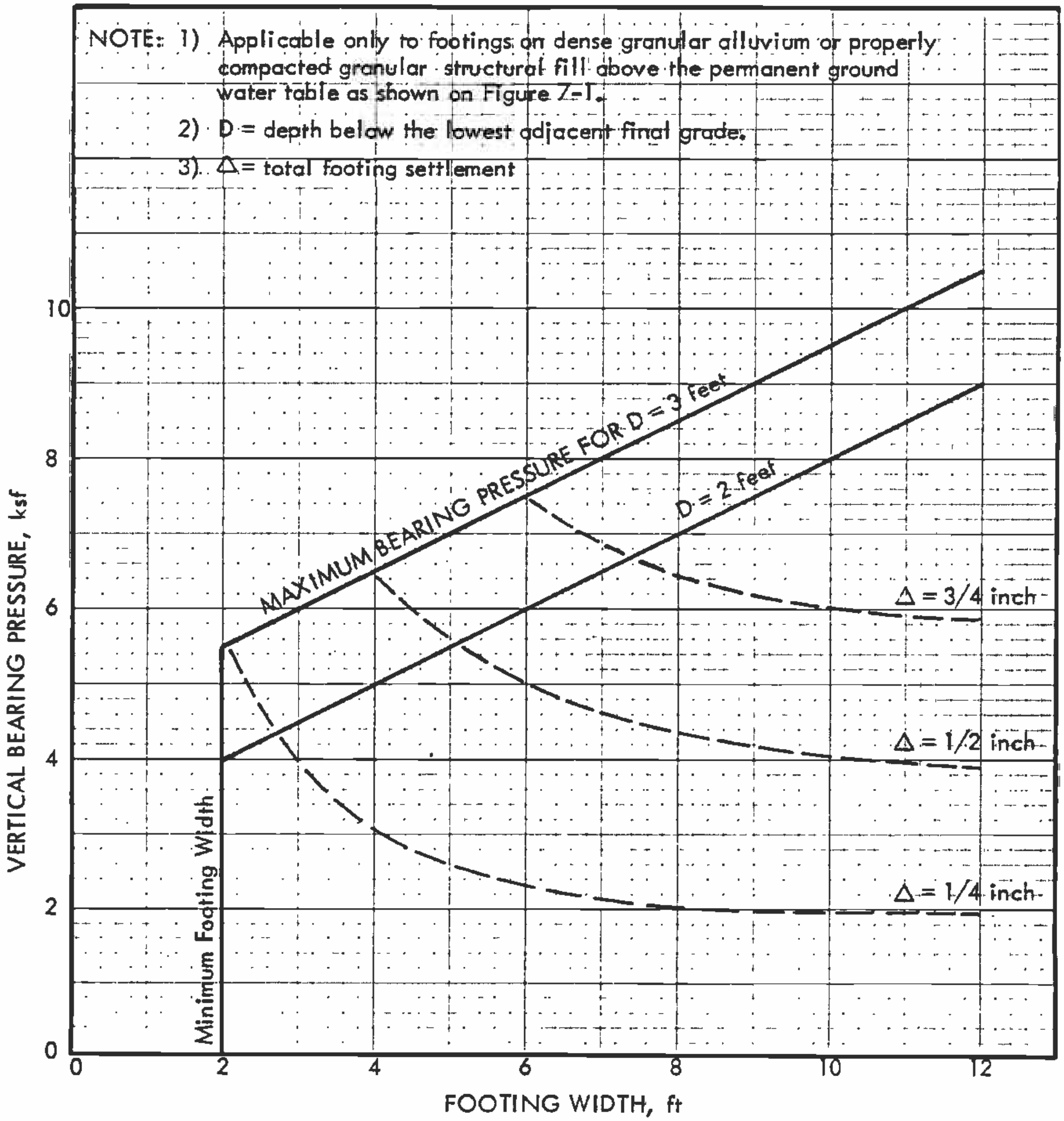
Figure No.
7-2



GEOTECHNICAL DESIGN CRITERIA FOR SPREAD FOOTINGS; DEPTH 2 & 3 FEET

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Figure No.
7-3

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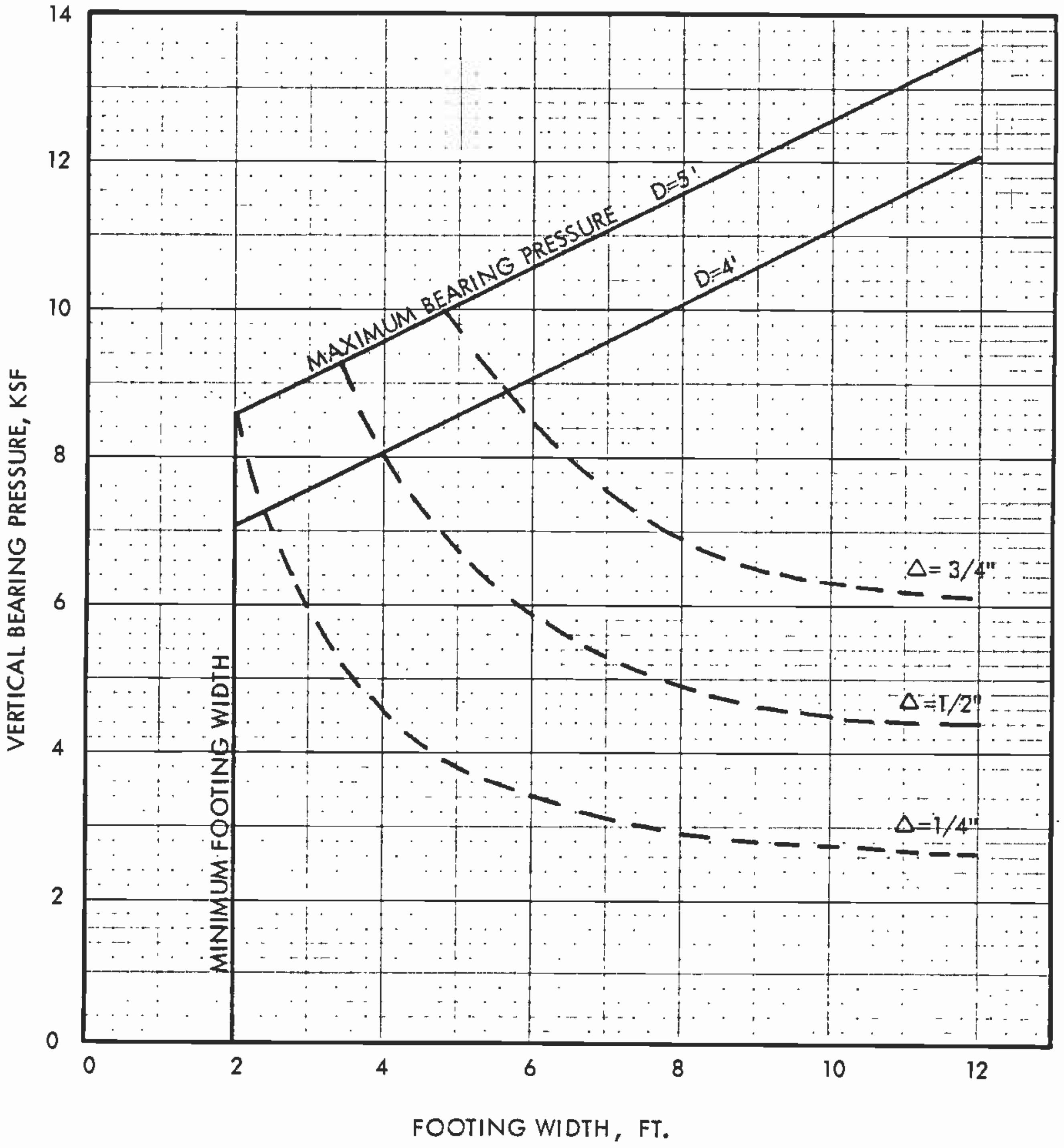
NOTE: 1.) Applicable only to footings on dense granular alluvium or properly compacted granular structural fill above the permanent water table, as shown on Fig. 7-1.

2.) D = depth below the lowest adjacent final grade.

3.) △= total footing settlement



GEOTECHNICAL DESIGN CRITERIA FOR SPREAD FOOTINGS; DEPTH 4 & 5 FEET

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Figure No.
7-4

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tracks are to be located on separate rail support systems, no lateral pressures will be induced on the retaining walls. If the rail tracks are to be located behind the wall, then an additional wheel load surcharge should be applied on the wall.

A depressed loading dock is planned for the Maintenance-of-Way Building. Exact dimensions of the loading dock have not been defined. Loading dock retaining walls should also be designed for the lateral soil pressure distributions provided on Figures 6-6 and 6-7, whichever is the applicable case.

7.5 PAVEMENT SECTION

The proposed parking lot and roadway pavement will probably be paved with flexible asphaltic concrete. It is assumed that both light passenger and maintenance vehicles and heavy trucks will be utilizing the parking areas and the roadways. The minimum flexible pavement sections for assumed Traffic Index (TI) values of 4.0, 5.0 and 7.0, and a subgrade R-value of 40 were developed for the Yard and Shops. The following pavement sections provide the recommended thickness of compacted subgrade, the base course and the asphaltic concrete for the different Traffic Indices.

		THICKNESS (in inches)				
		A.C. with Base Course			Compacted Subgrade (R ≥ 40)	
SERVICE CONDITIONS	ASSUMED TRAFFIC INDEX (TI)	A.C.	Base Course	Full Depth Asphaltic Concrete	Native Sandy Soil	Existing Fill
Passenger Car Parking	4.0	2.0	5.0	4.0	12.0	24.0
Light Truck Parking & Driveway	5.0	2.0	6.5	4.5	12.0	24.0
Heavy Truck & Loading Areas	7.0	3.0	8.5	7.0	18.0	36.0

Subgrade soil preparation should include processing of any disturbed subgrade areas, and excavation and replacement as required to provide a properly compacted subgrade of select granular material ("R" Value ≥40) to the depths indicated above. Subgrade fill compaction should be performed in accordance with recommended specifications presented in Appendix H.

Base course material should be Type II aggregate base conforming with Section 26-1.023 of CALTRANS' Standard Specifications (1978).

Section 8.0

Parameters for Seismic Design

8.0 PARAMETERS FOR SEISMIC DESIGN

8.1 GENERAL

Seismic criteria for design of the Southern California Rapid Transit Metro Rail Project have been previously developed and are presented in the "Seismological Investigation and Design Criteria" report dated May, 1983. The Part I investigation of this report contains an evaluation of the seismological conditions which may affect the project, and selection of 100-year probable and maximum credible earthquake ground motions and response spectra for the project. The Part II investigation provides geotechnical and structural seismic design criteria to be used for design of both underground and above ground structures.

8.2 DYNAMIC MATERIAL PROPERTIES

Values of apparent wave propagation velocities for use in travelling wave analyses have been presented in Table B-2 of Part II, Appendix B, of the seismic design criteria report. Other dynamic soil parameters will also be required for input into the various types of analyses recommended in the seismic design criteria report. These include values of dynamic Young's modulus, dynamic constrained modulus, and dynamic shear modulus at low strain levels. In addition, certain types of equivalent linear analyses required that the variation of dynamic shear modulus and soil hysteretic damping with the level of shear strain be known.

Average values of compression and shear wave velocities based on interpretation of limited downhole and crosshole geophysical surveys performed in Boring CEG-5 and other borings in similar materials during the 1981 investigation (see Appendix C) are presented in Table 8-1. These velocities have been used together with the corresponding values of density and Poisson's ratio to establish appropriate modulus values at low strain levels for the Young Alluvium and Puente bedrock.

TABLE 8-1
MATERIAL PROPERTIES SELECTED FOR SEISMIC DESIGN

		YOUNG ALLUVIUM	PUENTE BEDROCK
Average Compression Wave Velocity, V_c (ft/sec)	- moist	4500	5700
	- saturated	5300	
Average Shear Wave Velocity, V_s (ft/sec)		1100	1300
*Poisson's Ratio		0.35	0.35
**Young's Modulus, E, (psi)	- moist	474,000	530,000
	- saturated	211,000	
**Constrained Modulus, E_c, (psi)	- moist	760,000	850,000
	- saturated	800,000	
**Shear Modulus, G_{max}, (psi)	- moist	32,600	45,000
	- saturated	34,500	

* For saturated Young Alluvium, use value of 0.45.

** Saturated values of modulus should be used for undrained loading conditions in saturated Young Alluvium.

The variation of dynamic shear modulus, expressed as the ratio of G/G_{max}, with the level of shear strain is presented in Figure 8-1 for the various geologic units. Similar relationships for soil hysteretic damping are presented in Figure 8-2.

8.3 LIQUEFACTION POTENTIAL

The site is underlain by 50 to 60 feet of saturated granular alluvium. Since saturated granular soils can be susceptible to liquefaction, a liquefaction analysis of the site was performed using data obtained from the 1981 and 1983 investigations. The details of the analysis are presented in Appendix G.3. The analysis was based on:

- Recent procedures developed by Seed et al., 1983, which relate observed field behavior with common soil properties. The main soil properties examined at this site included Standard Penetration Test (SPT) resistances, field shear wave velocity measurements and gradation.
- Observations made in man-sized Boring 6A (not shown on drawings).

The analysis indicated that the alluvium has a low risk of liquefaction due to its relatively high density and coarse gradation. Based on the results of the analysis presented in Appendix G.3 and engineering judgement, it is our opinion that the alluvium underlying the site would not be subject to liquefaction during strong ground shaking produced at the site by the postulated earthquake motions.

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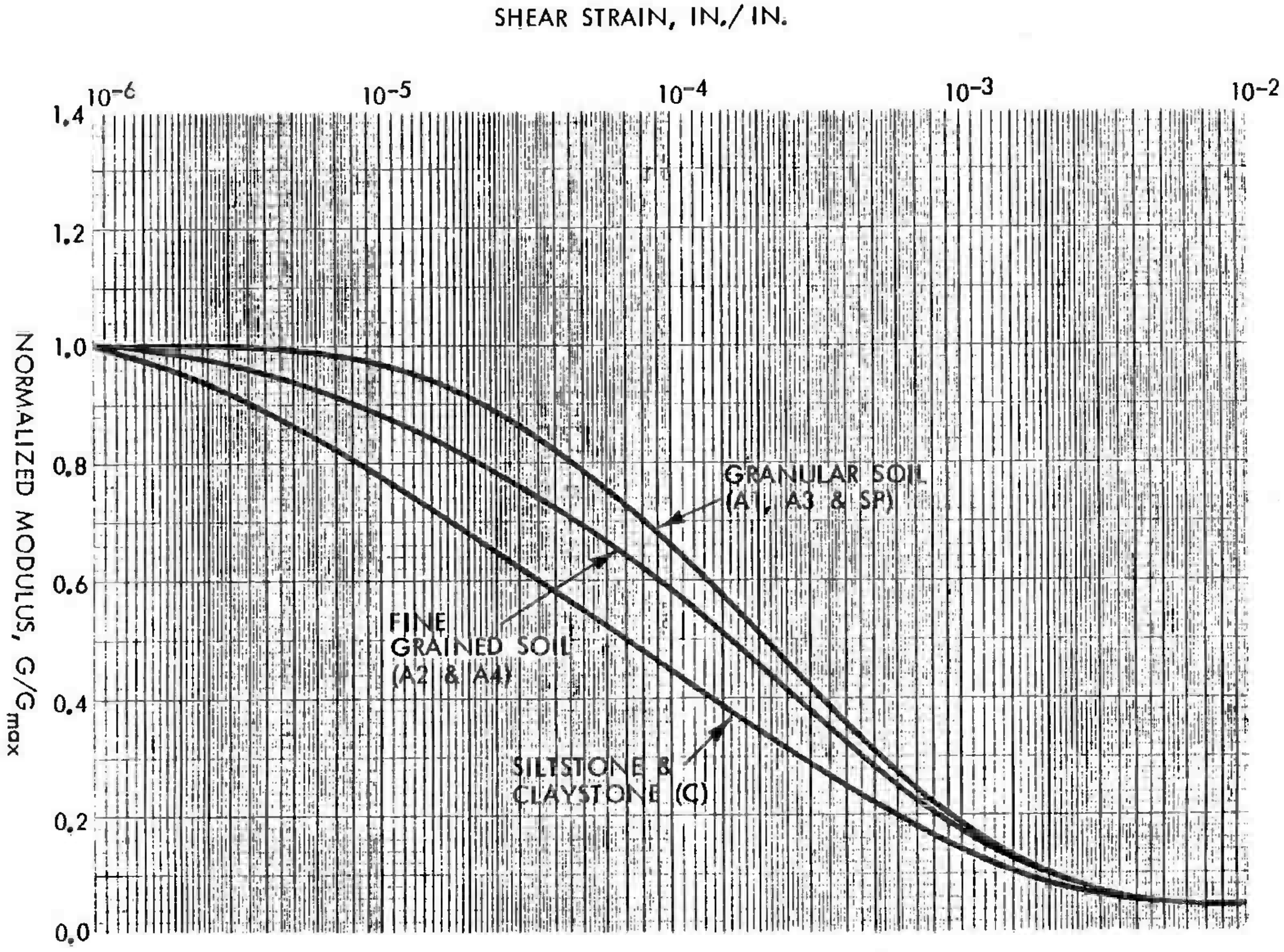

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RECOMMENDED DYNAMIC SHEAR MODULUS RELATIONSHIPS

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Figure No.
8-1

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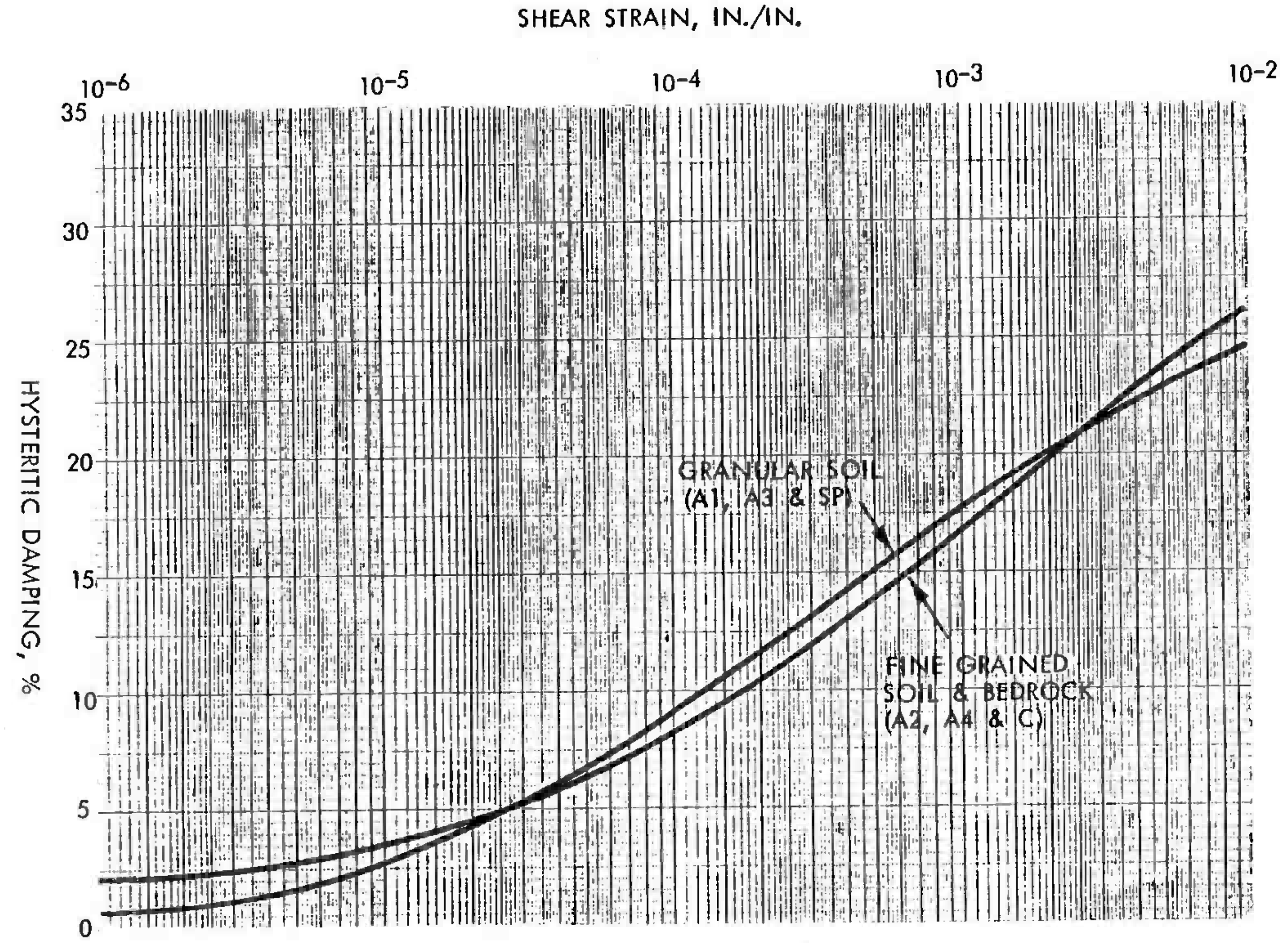

RECOMMENDED DYNAMIC DAMPING RELATIONSHIPS

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Figure No.
8-2

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Section 9.0

Supplementary Exploration and Geotechnical Services

9.0 SUPPLEMENTARY EXPLORATION AND GEOTECHNICAL SERVICES

Based on the available data and current design concepts, the following supplementary exploration and geotechnical services are strongly recommended:

- Additional Pumping Test and Observation Wells: We recommend that an additional pumping test be conducted in the area of the cut-and-cover portion leading into the Yard and Shops area to supplement the data for construction dewatering in the Young Alluvium immediately west of the los Angeles River. We recommend that at least four additional observation wells be installed for time-drawdown observations. These observation wells should also be utilized as test borings with sampling and testing to supplement and verify subsurface soil information in this area.

- Observation Well Monitoring: The ground water observation wells, already installed in this study, should be read several times a year prior to construction and more frequently during construction if the wells can be maintained. These data will aid in confirming the maximum design ground water levels. It will also provide valuable data to the contractor in determining his construction schedule and procedures prior to construction and evaluating dewatering during construction.

- Evaluation of the Hollywood Freeway Underpinning Construction Problems: Proposed Hollywood Freeway underpinning methodology and related construction problems should be evaluated in full detail. The existing elevated Freeway structure design and construction records should be reviewed in conjunction with more detailed actual subsurface soil and ground water conditions.

- Additional Oil and Gas Explorations: The available data may be insufficient to evaluate adequately the anticipated problems associated with gas release during dewatering, occurrence of gas in the permanent structure, and the possibility of oil migrating to the pumps during dewatering.

- Review Final Design Plans and Specifications: We recommend that we have the opportunity to participate in the development of the final design concepts and to review the contract plans and specifications for geotechnical aspects of the construction.

- Shoring Plan Review: Assuming that the shoring and dewatering systems are designed by the contractor, we request the opportunity to review the proposed systems in detail including review of engineering computations. This review is not a certification of the contractor's plan but rather an independent review made with respect to the owner's interests.

- Construction Observations: We recommend that a geotechnical engineer be on site full time during installation of the dewatering system, installation of the shoring system, preparation of foundation bearing surfaces, and placement of structural backfills. The geotechnical engineer should also be available for consultation to review recommended instrumentation data and respond to any specific geotechnical problems that may occur.

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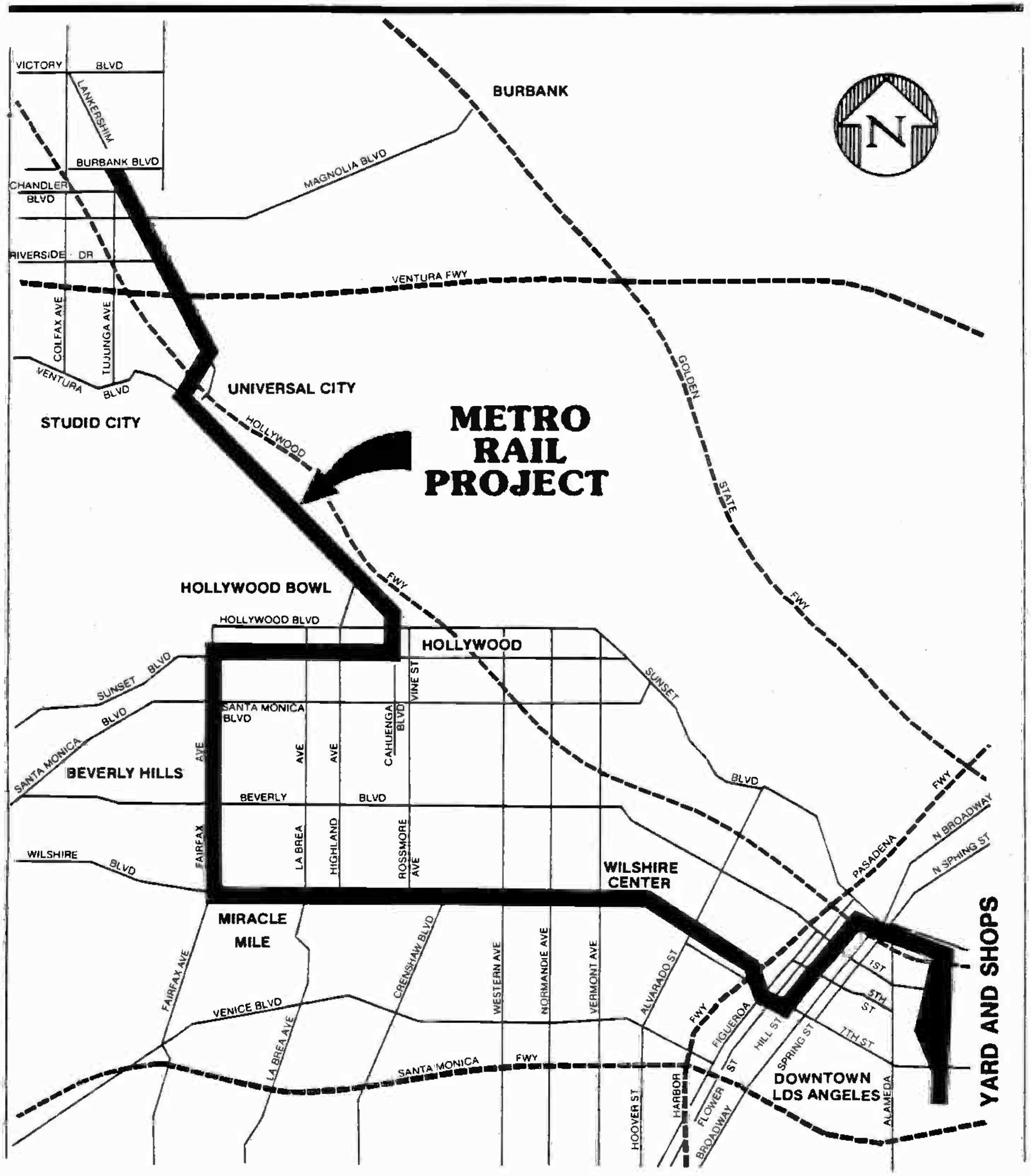
Drawings



VICINITY MAP

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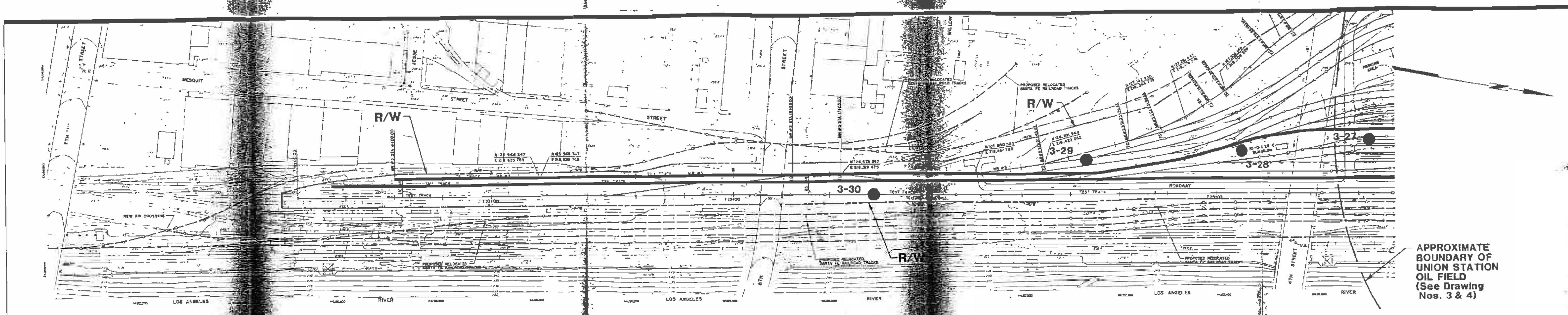
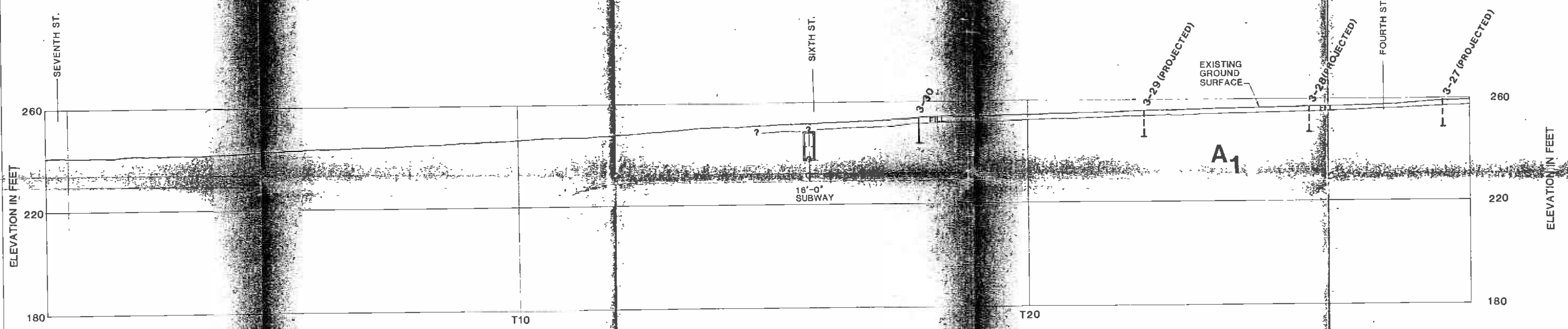


LOCATION OF BORINGS



GEOLOGIC SECTION

SCALE (IN FEET)

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VER. 20 0 20 40

NOTES: 1.) TEST TRACK STATIONING WAS USED AS THE REFERENCE BASE LINE FOR THE GEOLOGIC SECTION.

2.) FOR EXPLANATION OF SYMBOLS SEE DRAWING NO. 12

REFERENCE: "MAIN YARD AND SHOPS, YARD LAYOUT, SHEETS 8, 9, 10, 11, 12, 13, 14, 15 AND 16 OF 173; AND CBD TO NORTH HOLLYWOOD LINE PLAN, SHEETS 150, 152 AND 153 OF 173, DATED JULY 1983, PREPARED BY JOINT VENTURE OF DMJM/PBOD, GENERAL ENGINEERING CONSULTANTS - WAYS AND STRUCTURES."

LOCATION OF BORINGS AND GEOLOGIC SECTION

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Checked By	FYC
Approved By	RMP
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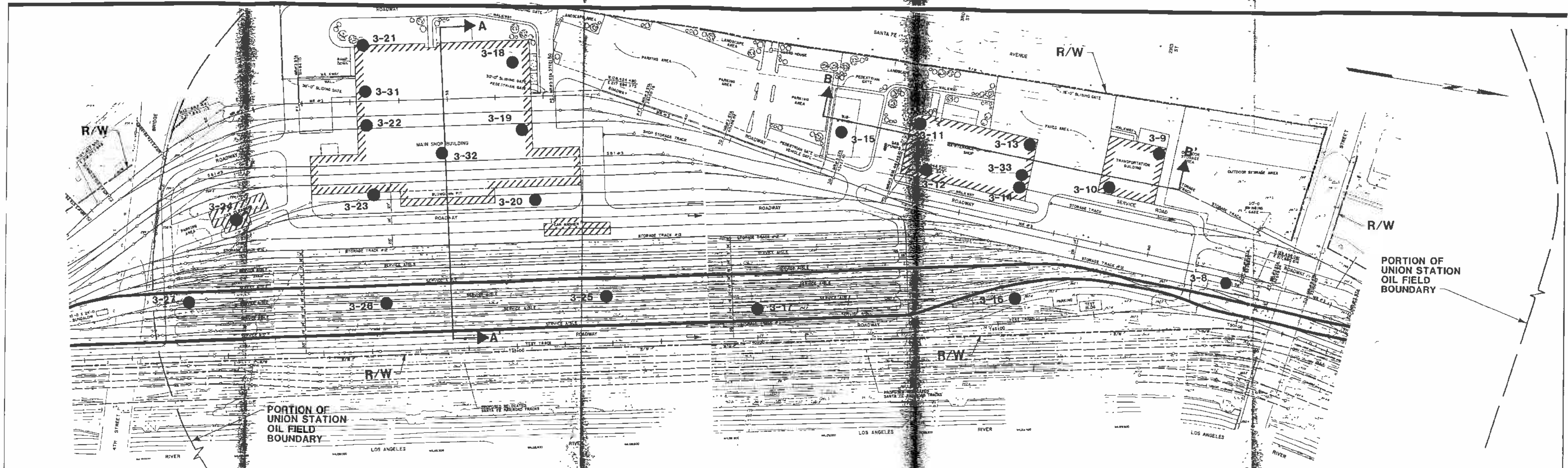
Drawing No.	2



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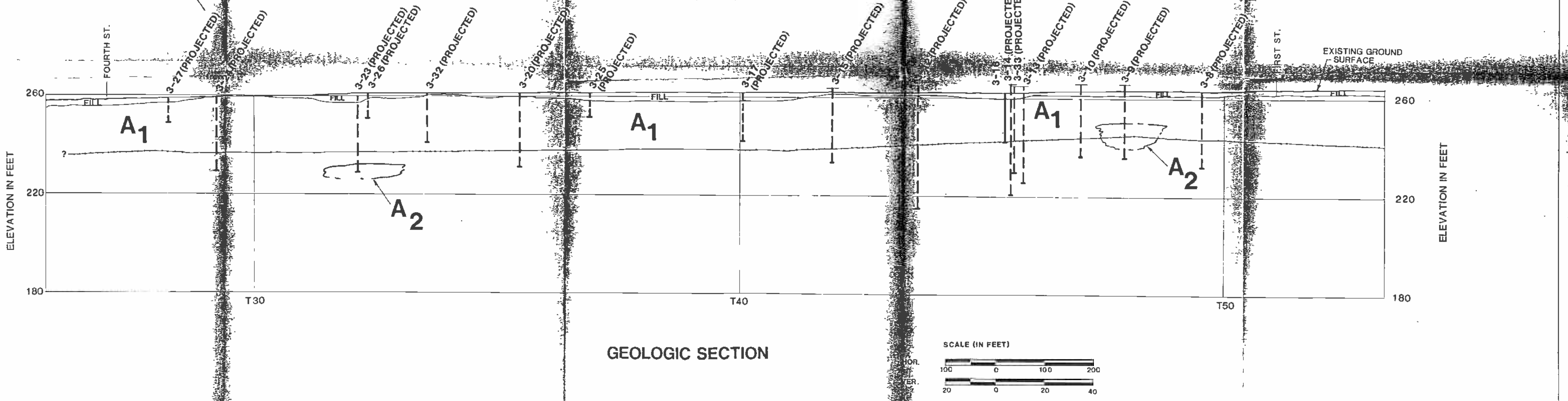

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3.) FOR GEOLOGIC SECTION A-A' SEE DRAWING 6.
4.) FOR GEOLOGIC SECTION B-B' SEE DRAWING NO. 7.
5.) FOR REFERENCE SEE DRAWING NO. 2

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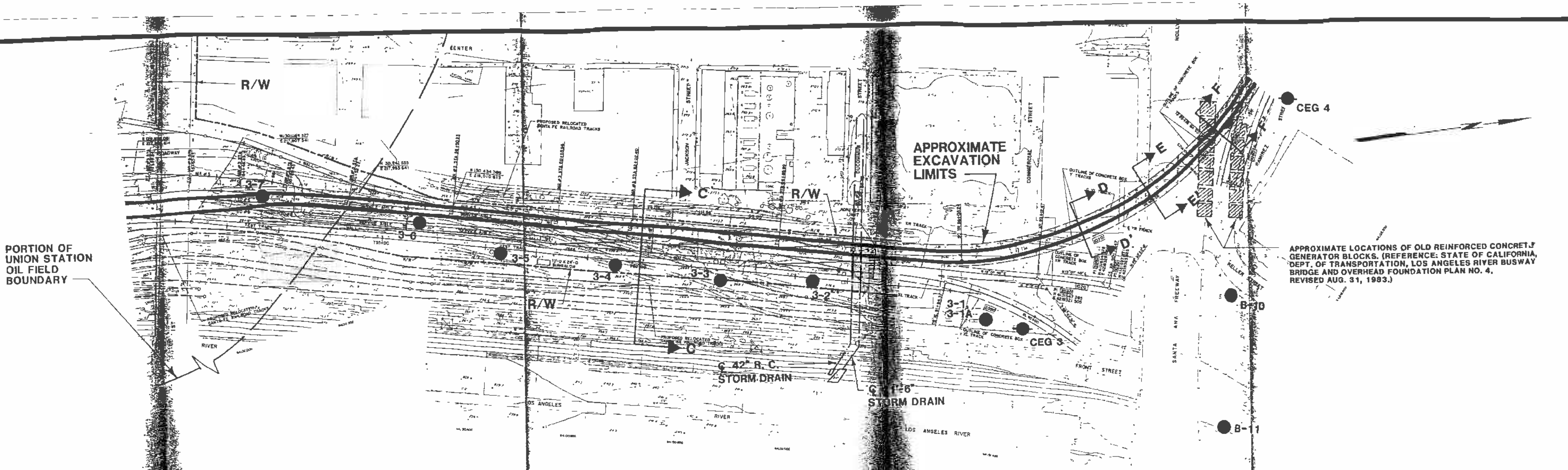

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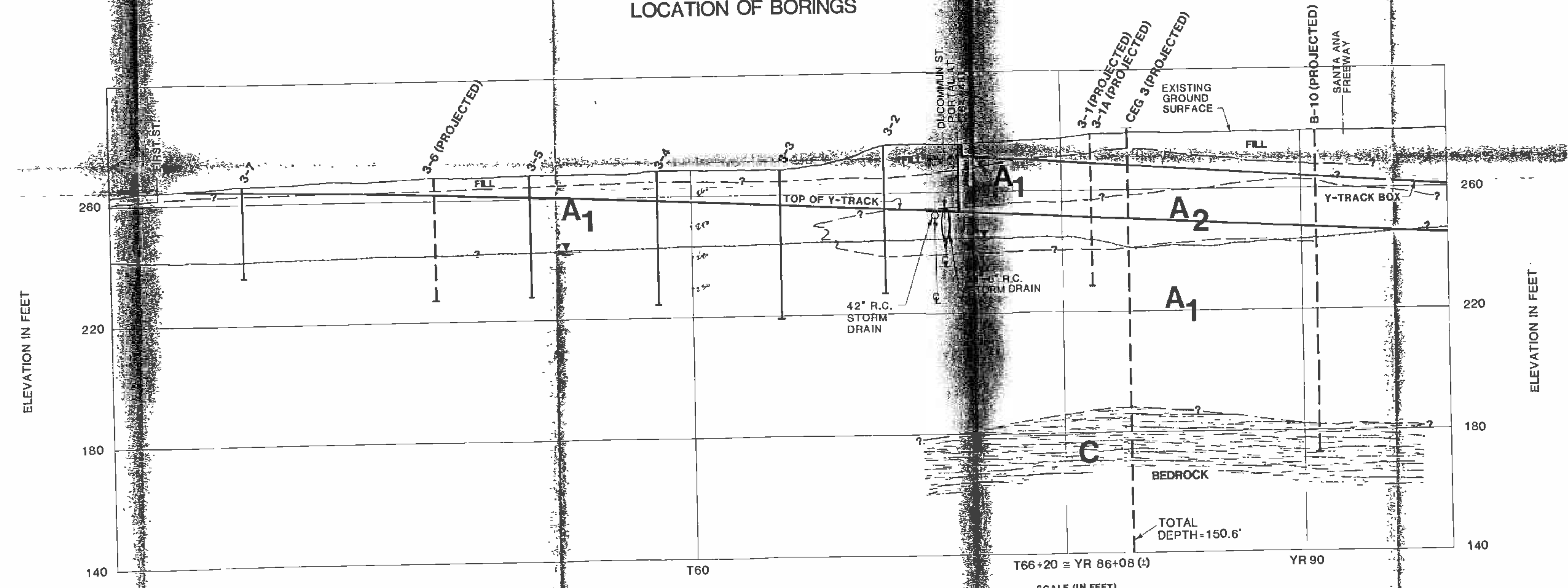

GEOLOGIC SECTION

SCALE (IN FEET)

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2.) FOR EXPLANATION OF SYMBOLS SEE DRAWING NO. 12.
3.) FOR GEOLOGIC SECTION C-C' SEE DRAWING NO. 8.
4.) FOR GEOLOGIC SECTION D-D' SEE DRAWING NO. 9.
5.) FOR GEOLOGIC SECTION E-E' SEE DRAWING NO. 10.
6.) FOR GEOLOGIC SECTION F-F' SEE DRAWING NO. 11.
7.) FOR REFERENCE SEE DRAWING NO. 2

LOCATION OF BORINGS AND GEOLOGIC SECTION

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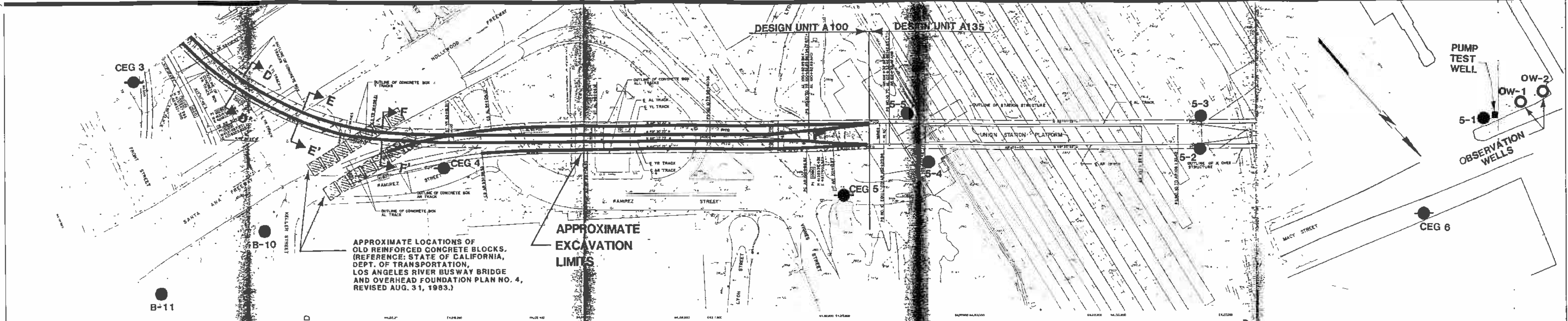
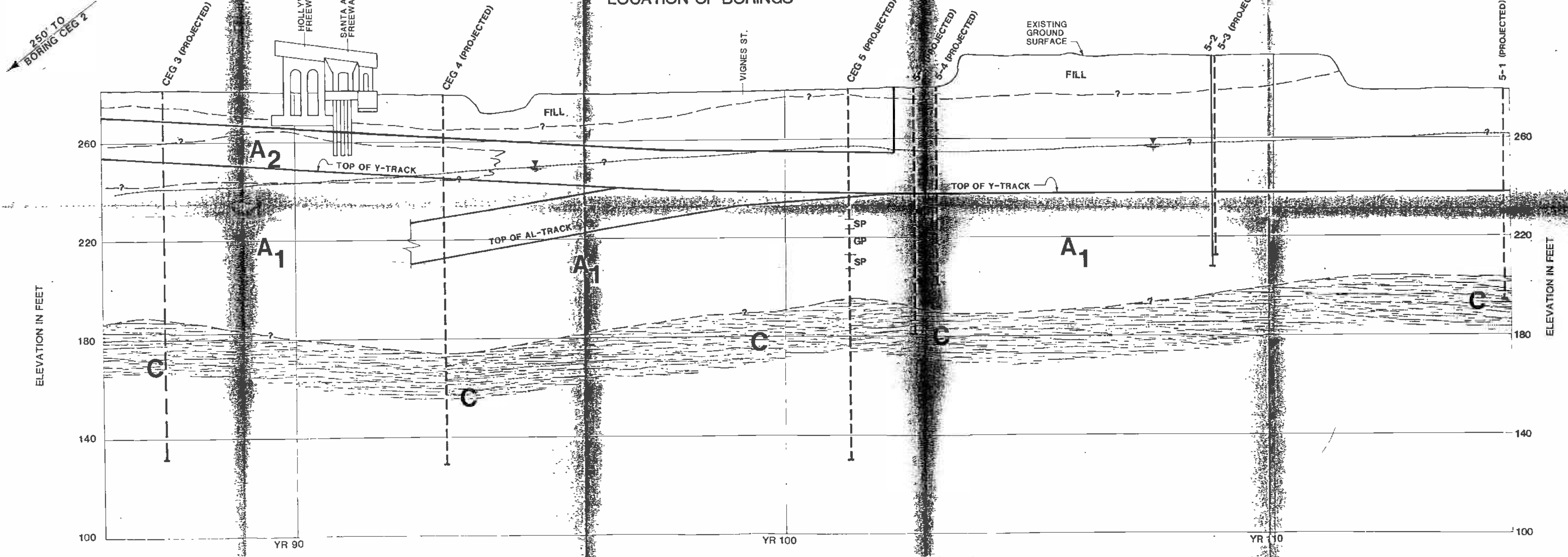


APPROXIMATE LOCATIONS OF OLD REINFORCED CONCRETE BLOCKS. (REFERENCE: STATE OF CALIFORNIA, DEPT. OF TRANSPORTATION, LOS ANGELES RIVER BUSWAY BRIDGE AND OVERHEAD FOUNDATION PLAN NO. 4, REVISED AUG. 31, 1983.)

APPROXIMATE EXCAVATION LIMITS

LOCATION OF BORINGS



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4.) FOR GEOLOGIC SECTION E-E' SEE DRAWING NO. 10.
5.) FOR GEOLOGIC SECTION F-F' SEE DRAWING NO. 11.
6.) FOR REFERENCE SEE DRAWING NO. 2

SCALE (IN FEET)
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LOCATION OF BORINGS AND GEOLOGIC SECTION

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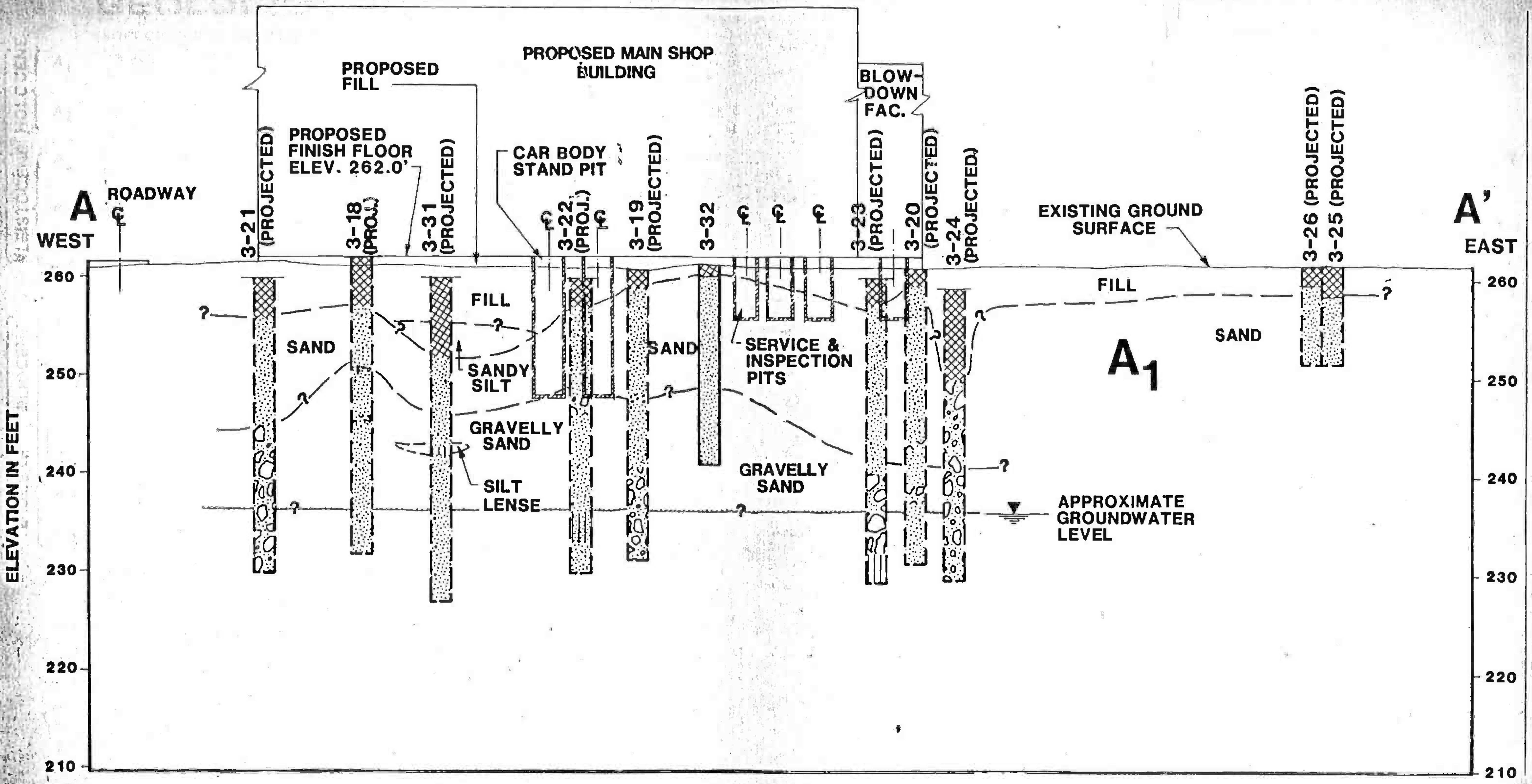
Drawing No	5



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2.) FOR EXPLANATION OF SYMBOLS SEE DRAWING NO.12

GEOLOGIC SECTION A–A'

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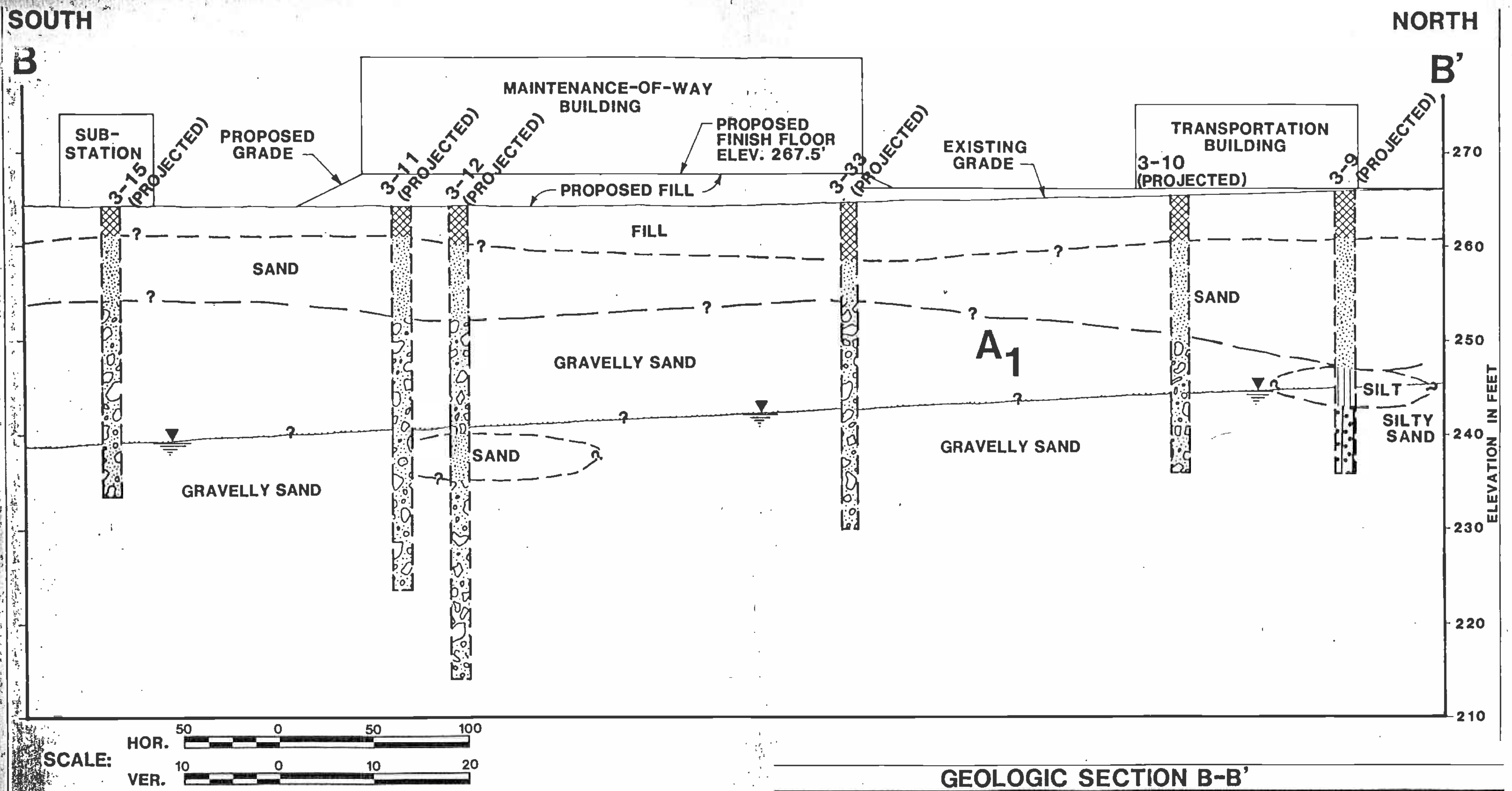

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2.) FOR EXPLANATION OF SYMBOLS SEE DRAWING NO. 12

GEOLOGIC SECTION B-B'

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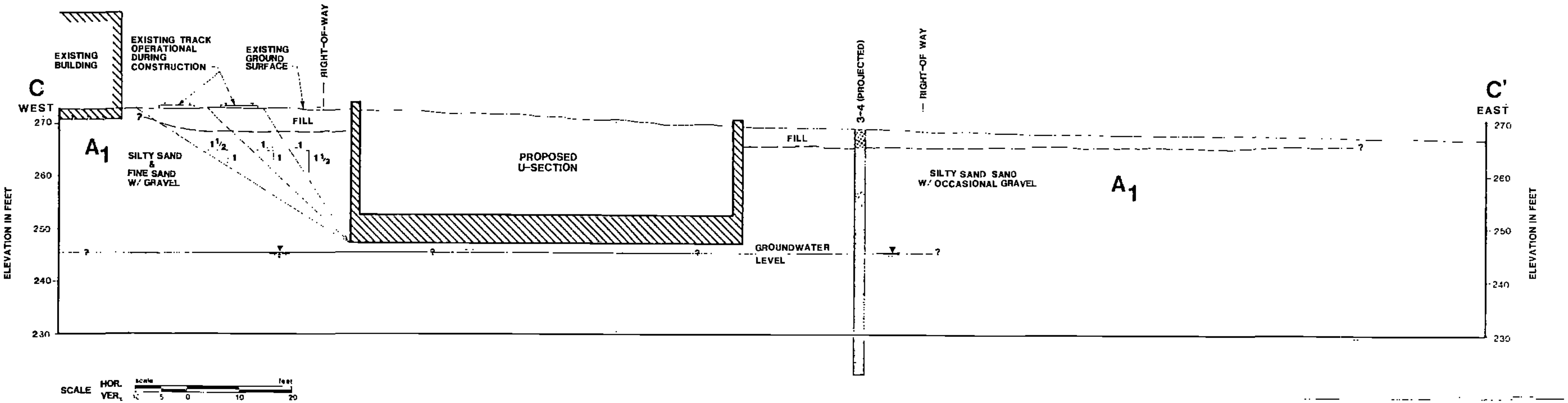

NOTES 1) FOR LOCATION OF SECTION SEE DRAWING NO 4
2) FOR EXPLANATION OF SYMBOLS SEE DRAWING NO. 12

GEOLOGIC SECTION C-C

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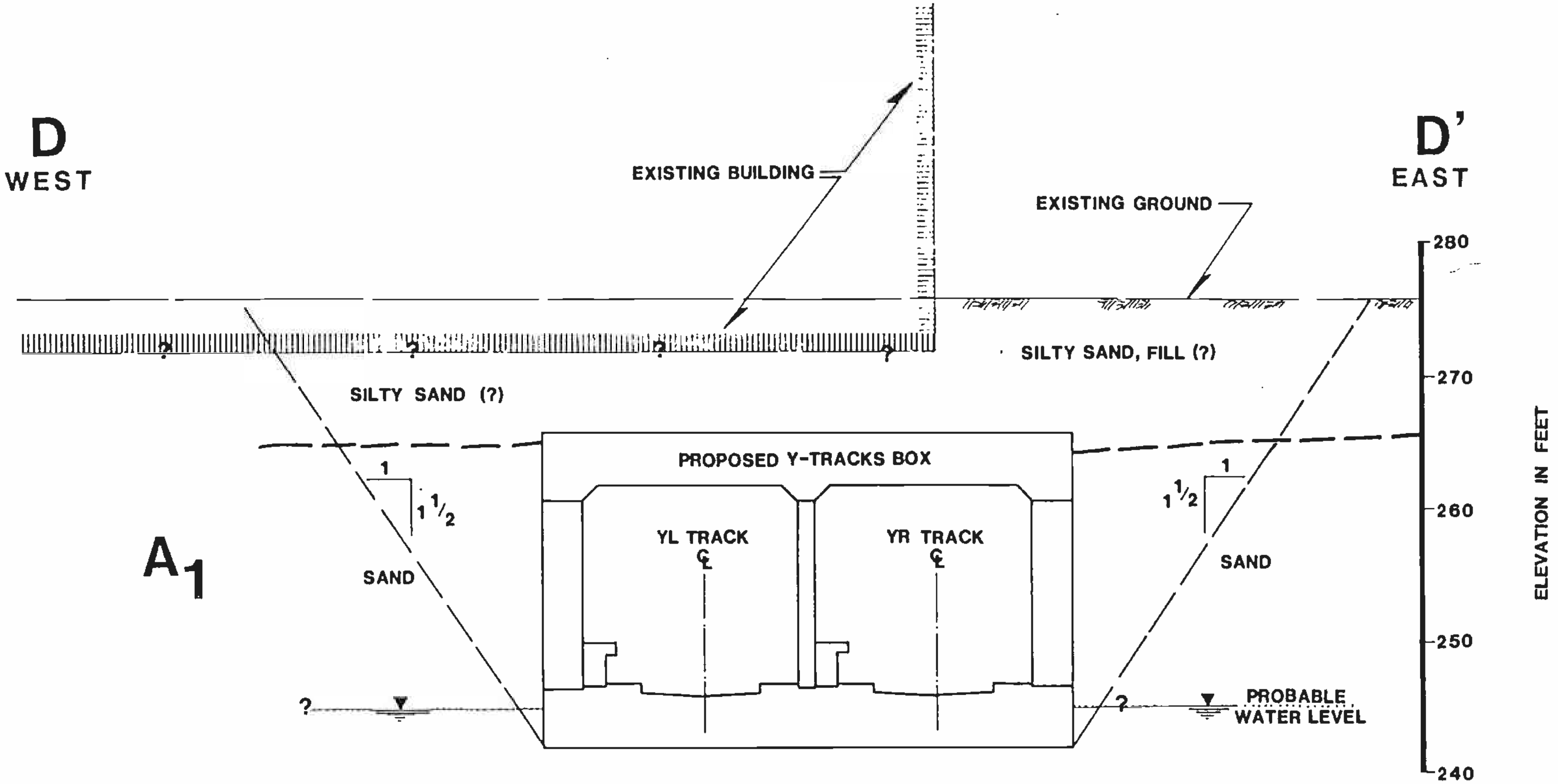
RG	8
FYC	



SCALE: HOR. VER.

NOTE: 1.) FOR LOCATION OF SECTION SEE DRAWING NO. 4.
2.) FOR EXPLANATION OF SYMBOLS SEE DRAWING NO. 12.

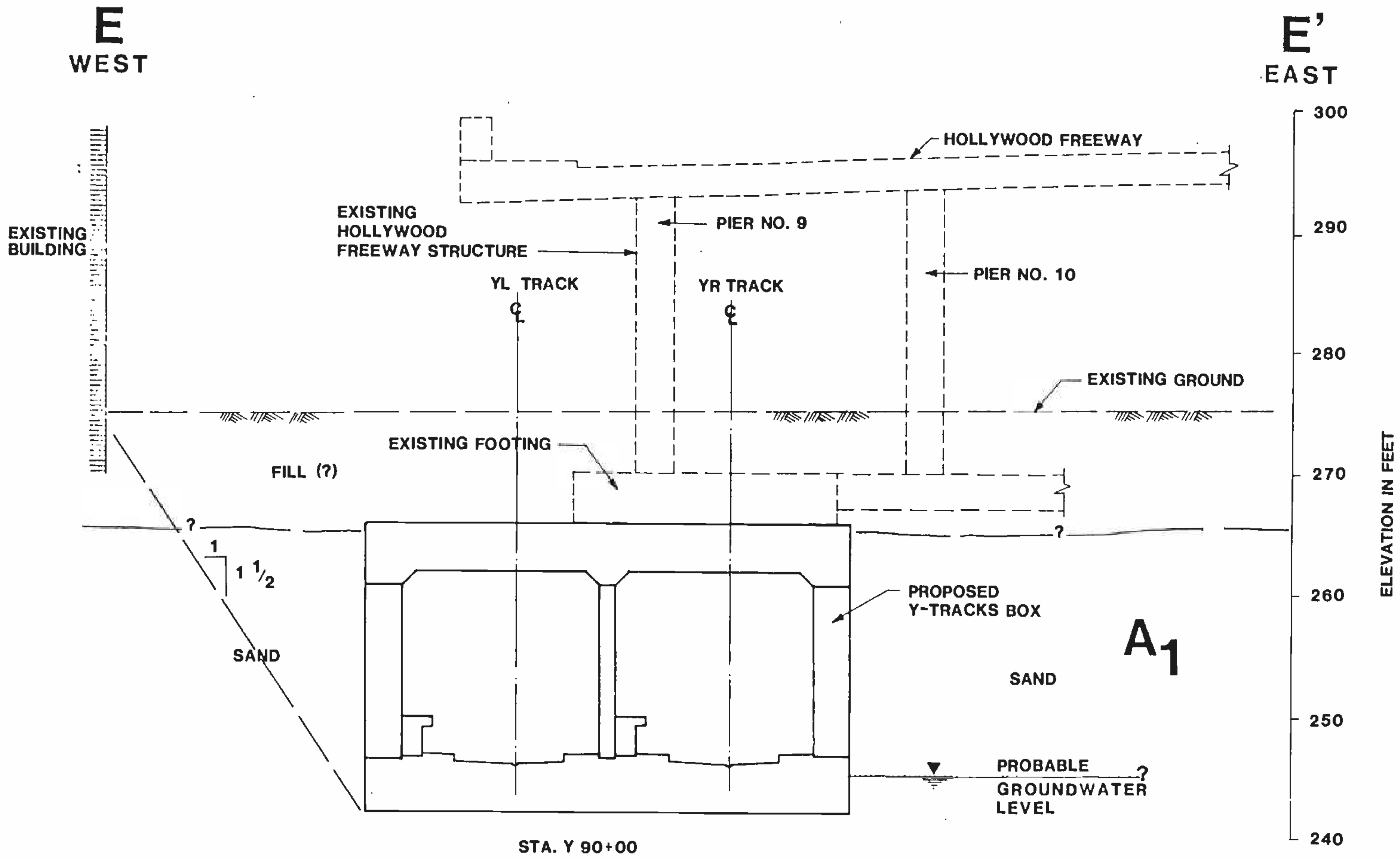
DRAWING REFERENCE: SCRTD METRO RAIL PROJECT,
DMJM/PBQD WORK DRAWING,
UNTITLED, UNDATED.

GEOLOGIC SECTION D-D'

DESIGN UNIT A100
Southern California Rapid Transit District
METRO RAIL PROJECT

Converse Consultants Geotechnical Engineering and Applied Sciences

Scale As Shown
Date FEB., 1984
Prepared by CSJ
Checked by RG
Approved By FYC
Project No 83-1140
Drawing No 9



SCALE: HOR. 10 5 0 10 20

VER. scale feet

NOTE: 1.) FOR LOCATION OF SECTION SEE DRAWING NO. 4.
2.) FOR EXPLANATION OF SYMBOLS SEE DRAWING NO. 12.

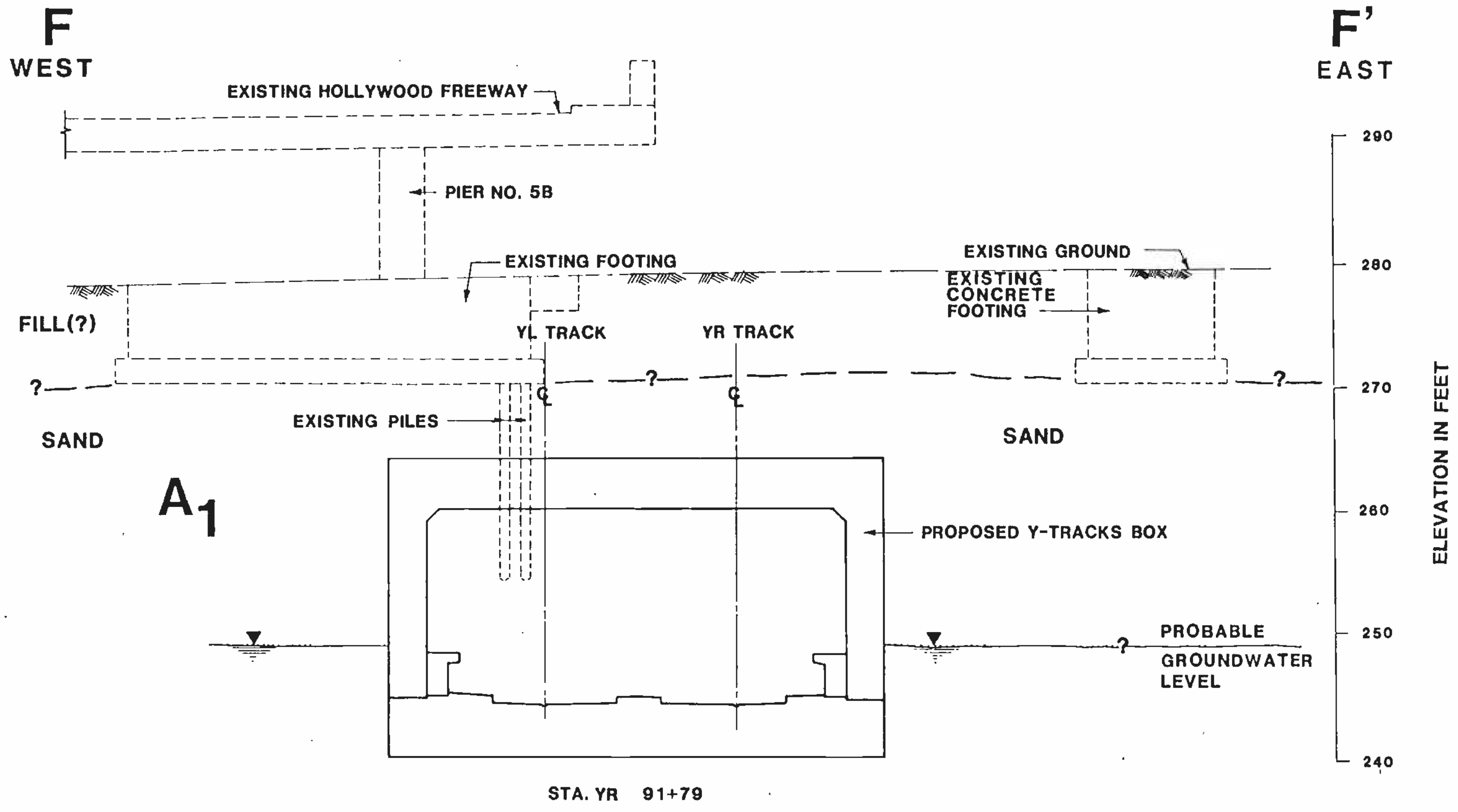
DRAWING REFERENCE: SCRTD METRO RAIL PROJECT,
DMJM/PBQD WORK DRAWING,
UNTITLED, UNDATED.

GEOLOGIC SECTION E-E'

DESIGN UNIT A100
Southern California Rapid Transit District
METRO RAIL PROJECT

Converse Consultants
Geotechnical Engineering
and Applied Sciences

Scale As Shown
Date FEB., 1984
Prepared by CSJ
Checked by RG
Approved By FYC
Project No 83-1140
Drawing No 10



SCALE: HOR. VER.

10 5 0 10 20

scale feet

NOTE: 1.) FOR LOCATION OF SECTION SEE DRAWING NO. 4.
2.) FOR EXPLANATION OF SYMBOLS SEE DRAWING NO. 12.

DRAWING REFERENCE: SCRTD METRO RAIL PROJECT,
DMJM/PBQD WORK DRAWING,
UNTITLED, UNDATED.

GEOLOGIC SECTION F-F'

DESIGN UNIT A100
Southern California Rapid Transit District
METRO RAIL PROJECT

Converse Consultants
Geotechnical Engineering and Applied Sciences

Scale	As Shown
Date	FEB., 1984
Prepared by	CSJ
Checked by	RG
Approved By	FYC
Project No	83-1140
Drawing No	11

GEOLOGIC UNITS

SOFT GROUND TUNNELLING

QUATERNARY	HOLOCENE	A_1	YOUNG ALLUVIUM (Granular): Includes clean sands, silty sands, gravelly sands, sandy gravels, and locally contains cobbles and boulders. Primarily dense, but ranges from loose to very dense.
		A_2	YOUNG ALLUVIUM (Fine-grained): Includes clays, clayey silts, sandy silts, sandy clays, clayey sands. Primarily stiff, but ranges from firm to hard.
	PLEISTOCENE	A_3	OLD ALLUVIUM (Granular): Includes clean sands, silty sands, gravelly sands, and sandy gravels. Primarily dense, but ranges from medium dense to very dense.
		A_4	OLD ALLUVIUM (Fine-grained): Includes clays, clayey silts, sandy silts, sandy clays, and clayey sands. Primarily stiff, but ranges from firm to hard.
		SP	SAN PEDRO FORMATION: Predominantly clean, cohesionless, fine to medium-grained sands, but includes layers of silts, silty sands, and fine gravels. Primarily dense, but ranges from medium dense to very dense. Locally impregnated with oil or tar.
TERTIARY	PLIOCENE	C	FERNANDO AND PUENTE FORMATIONS: Claystone, siltstone, and sandstone; thinly to thickly bedded. Primarily low hardness, weak to moderately strong. Locally contains very hard, thin cemented beds and cemented nodules.

ROCK TUNNELLING

(Terzaghi Rock Condition Numbers apply)*

3 ← Terzaghi Rock Condition Number

→ Approximate boundary between Terzaghi numbers

TERTIARY	MIOCENE	2-5	TOPANGA FORMATION: Conglomerate, sandstone, and siltstone; thickly bedded; primarily hard and strong (Geologic symbol Tt).
		1-5	TOPANGA FORMATION: Basalt; intrusive, primarily hard and strong (Geologic symbol Tb).

TERZACHI ROCK CONDITION NUMBERS:*

1 Hard and intact

2 Hard and stratified or schistose

3 Massive, moderately jointed

4 Moderately blocky and seamy

5 Very blocky and seamy (closely jointed)

6 Crushed but chemically intact rock or unconsolidated sand; may be running or flowing ground

7 Squeezing rock, moderate depth

8 Squeezing rock, great depth

9 Swelling rock

*In practice, there are not sharp boundaries between these categories, and a range of several Terzaghi Numbers may best describe some rock.

SYMBOLS

- - - - ?	Geologic contact: approximately located; queried where inferred
⇄ ?	Fault: approximately located; queried where inferred; arrows indicate probable movement; attitude in profile is an apparent dip and is not corrected for scale distortion
← 40	Dip of bedding: from unoriented core samples; bedding attitudes may not be correctly oriented to the plane of the profile, but represent dips to illustrate regional geologic trends; number gives true dip in degrees, as encountered in boring
........▽........?	Perched water level: approximately located; queried where inferred
———▽———?	Permanent water level: approximately located; queried where inferred
●	Boring — CEG (1981)
●	Boring — CCI/ESA/GRC (1983)
○	Boring — Nuclear Regulatory Commission (1980)
⊕	Boring — Woodward-Clyde (1977)
⦶	Boring — Kaiser Engineers (1962)
⊖	Boring — Other (USGS 1977 and various foundation studies)

NOTES:

1) The geologic sections are based on interpolation between borings and were prepared as an aid in developing design recommendations. Actual conditions encountered during construction may be different.

2) The locations of the tunnel line and stations are based on the Metro Rail Project, Milestone 10 alignment as of February, 1983.

3) Borings projected more than 200' to the profile line were considered in some of the interpretation of subsurface conditions. However, final interpretation is based on numerous factors and may not reflect the boring logs as presented in Appendix A

4) Displacements shown along faults are graphic representations. Actual vertical offsets are unknown.

▥	SILT
▨	CLAY
▥	SANDY SILT
▨	SANDY CLAY
▥	CLAYEY SILT
▨	SILTY CLAY
▦	SILTY SAND
▧	CLAYEY SAND
░	SAND
▒	GRAVELLY SAND
▒	SANDY GRAVEL
▒	GRAVEL
▧	GRAVELLY CLAY
▤	TAR SILT & CLAY
▤	TAR SAND
▩	FILL
▭	SILTSTONE
▤	CLAYSTONE
▭	INTERBEDDED SANDSTONE WITH SILTSTONE OR CLAYSTONE
░	SANDSTONE
▒	SANDSTONE, CONGLOMERATE
▭	CEMENTED ZONE
▩	META—SANDSTONE
▭	BASALT
░	BRECCIA
▭	SHEAR ZONE

GEOLOGIC EXPLANATION

DESIGN UNIT A100
Southern California Rapid Transit District
METRO RAIL PROJECT

Converse Consultants
Geotechnical Engineering and Applied Sciences

Scale N/A
Date FEB., 1984
Prepared by RG
Checked by JAD
Approved By HAS
Project No 83-1140
Drawing No 12

BR G 40 107

Appendix A

Field Exploration

APPENDIX A FIELD EXPLORATION

A.1 GENERAL

Field exploration data presented in this report for Design Unit A-100 includes information from borings drilled for the 1981 Geotechnical Investigation Report and additional borings drilled for the 1983 investigation. Four borings (CEG-2, 3, 4 and 5) were drilled near or within Design Unit A100 during the 1981 investigation and the logs are reproduced in this appendix. Two additional borings (5-4 and 5-5) were drilled in 1983 for the Union Station investigation. Borings 3-1 through 3-33 were drilled in 1983. Locations of the borings are shown on Drawings 2 through 5. Ground water observation wells (piezometers) were installed in borings CEG-4, and 5-5. A ground water pump test was performed adjacent to boring 5-1 (see Appendix E). Geophysical downhole and crosshole surveys were made at boring CEG-5.

The borings drilled in 1983 for this design unit include 33 borings (3-1 through 3-33) ranging in depth from 10 to 51 feet. All borings were sampled at regular intervals using the Converse ring sampler, pitcher barrel sampler and the standard split spoon sampler. Sample recovery was good in the upper silty sands but generally poor in the coarse granular alluvium (Unit A_1). Overall sample recovery average about 70%.

The following subsections describe the field exploration procedures and provide explanations of symbols and notation used in preparing the field boring logs. Copies of the field boring logs are presented following the text of this appendix.

A.2 FIELD STAFF AND EQUIPMENT

A.2.1 Technical Staff

Members of Converse Consultants and Geo/Resource Consultants participated in the drilling exploration program. The field geologist continuously supervised each boring during the drilling and sampling operation. The geologist was also responsible for preparing detailed lithologic log of the rotary wash cuttings and for sample/core identification, labeling and storage of all samples, and installation of piezometer pipe, gravel pack and bentonite seals.

A.2.2 Drilling Contractor and Equipment

Drilling was performed by Pitcher Drilling Company of East Palo Alto, California, with Failing 1500 rotary wash rigs, each operated by a two man crew.

A.3 SAMPLING AND LOGGING PROCEDURES

Logging and sampling were performed in the field by the geologist. The following describes sampling equipment and procedures and notations used on the lithologic logs to indicate drilling and sampling modes.

A.3.1 Sampling

In the overburden at about 10-foot intervals, the Converse ring sampler was driven using a down-hole 450-pound slip-jar hammer. The Converse sampler was followed with the standard split spoon sample (SPT) driven with a 140-pound hammer with a 30 inch stroke.

The most common cause for loss of samples or altering the sample interval was when gravels were encountered at the desired sampling depth. Standard penetration blow count information can often be misleading in this type of formation, and it is difficult to recover an undisturbed sample. Therefore at some locations borings were advanced until drill response and cutting suggested a change in formation.

The following symbols were used on the logs to indicate the type of sample and the drilling mode:

Log Symbol	Sample Type	Type of Sampler
B	Bag	-
J	Jar	Split Spoon
C	Can	Converse Ring
S	Shelby Tube	Pitcher Barrel
Box	Box	Pitcher Barrel, Core Barrel

Log Symbol	Drilling Mode
AD	Auger Drill
RD	Rotary Drill
PB	Pitcher Barrel Sampling
SS	Split Spoon
DR	Converse Drive Sample
C	Coring

A.3.2 Field Classification of Soils

All soil types were classified in the field by the site geologist using the "Unified Soil Classification System". Based on the characteristics of the soil, this system indicates the behavior of the soil as an engineering construction material.* Although particle size distribution estimates were based on volume rather than weight, the field estimates should fall within an acceptable range of accuracy.

Table A-1 shows the correlation of standard penetration information and the physical description of the consistency of clays (hand-specimen) and the compactness of sands used by the field geologists for describing the materials encountered.

TABLE A-1 Correlation of N-Values and Consistency/Compactness of Soil Obtained in the Field

N-Values (blows/foot)	Hand-Specimen (clay only)	Consistency (clay or silt)	Compactness (sand only)	N-Values (blows/foot)
0 - 2	Will squeeze between fingers when hand is closed	Very soft	Very loose	0 - 4
2 - 4	Easily molded by fingers	Soft	Loose	4 - 10
4 - 8	Molded by strong pressure of fingers	Firm	---	---
8 - 16	Dented by strong pressure of fingers	Stiff	Medium dense	10 - 30
16 - 32	Dented only slightly by finger pressure	Very stiff	Dense	30 - 50
32+	Dented only slightly by pencil point	Hard	Very dense	50+

A.3.3 Field Description of the Formations

The description of the formations is subdivided in two parts: lithology and physical condition. The lithologic description consists of:

- rock name;
- color of wet core (from GSA rock color chart);
- mineralogy, textural and structural features; and
- any other distinctive features which aid in correlating or interpreting the geology.

The physical condition describes the physical characteristics of the rock believed important for engineering design consideration. The form for the description is as follows:

Physical condition: ____________ fractured, minimum ______,
maximum ________, mostly ____________; ____________ hardness;
________ strength; ____________ weathered.

Bedrock description terms used on the boring logs are given on Table A-2.

---

* For a more complete discussion of the Unified Soil Classification System, refer to Corps of Engineers, Technical Memorandum No. 3-357, March 1953, or Department of the Interior, Bureau of Reclamation, Earth Manual, 1963.

TABLE A-2 Bedrock Description Terms

PHYSICAL CONDITION*	SIZE RANGE	REMARKS
Crushed	-5 microns to 0.1 ft	Contains clay
Intensely Fractured	0.05 ft to 0.1 ft	Contains no clay
Closely Fractured	0.1 ft to 0.5 ft	
Moderately Fractured	0.5 ft to 1.0 ft	
Little Fractured	1.0 ft to 3.0 ft	
Massive	4.0 ft and larger	

HARDNESS**	
Soft	- Reserved for plastic material
Friable	- Easily crumbled or reduced to powder by fingers
Low Hardness	- Can be gouged deeply or carved with pocket knife
Moderately Hard	- Can be readily scratched by a knife blade; scratch leaves heavy trace of dust
Hard	- Can be scratched with difficulty; scratch produces little powder & is often faintly visible
Very Hard	- Cannot be scratched with knife blade

STRENGTH	
Plastic	- Easily deformed by finger pressure
Friable	- Crumbles when rubbed with fingers
Weak	- Unfractured outcrop would crumble under light hammer blows
Moderately Strong	- Outcrop would withstand a few firm hammer blows before breaking
Strong	- Outcrop would withstand a few heavy ringing hammer blows but would yield, with difficulty, only dust & small fragments
Very Strong	- Outcrops would resist heavy ringing hammer blows & will yield with difficulty, only dust & small fragments

WEATHERING	DECOMPOSITION	DISCOLORATION	FRACTURE CONDITION
Deep	- Moderate to complete alteration of minerals, feldspars altered to clay, etc.	Deep & thorough	All fractures extensively coated with oxides, carbonates, or clay
Moderate	- Slight alteration of minerals, cleavage surfaces lusterless & stained	Moderate or localized & intense	Thin coatings or stains
Little	- No megascopic alteration in minerals	Slight & intermittent & localized	Few stains on fracture surfaces
Fresh	- Unaltered, cleavage surface glistening	None	

*Joints and fractures are considered the same for physical description, and both are referred to as "fractures"; however, mechanical breaks caused by drilling operation were not included.

**Scale for rock hardness differs from scale for soil hardness.

A.4 PIEZOMETER INSTALLATION

Piezometers were installed in borings CEG-4 and 5 in 1981 and two additional piezometers were installed in borings 5-1 and 5-5. For this study open wells were installed in borings 3-1, 3-7, 3-9, 3-15, and 3-24 in 1983. Procedures for piezometer installation were the following:

A two-inch diameter plastic ABS pipe was installed in the boring and the annulus of the boring around the pipe was backfilled with a coarse sand/pea gravel aggregate. A 5-foot thick surface bentonite seal was placed around the holes to prevent surface water from artificially recharging the gravel-packed hole or contaminating local ground water. After the piezometer was installed, the boring was flushed using air lift provided by a trailer-mounted air compressor. The piezometer was covered with a standard 7-inch diameter steel water meter cap held at surface grade by a grouted in-place 3- to 4-foot long, 5-inch diameter plastic sleeve. Ground water data obtained from the piezometer is presented in Section 5.4 of the text.

Converse Consultants, Inc.
Earth Sciences Associates
Geo/Resource Consultants

BORING LOG 3-1

Proj: DESIGN UNIT A-100
Date Drilled 7/26/83
Ground Elev. 279'
Drill Rig FAILING 1500
Logged By L. Schoeberlein
Total Depth 21.2
Hole Diameter 4 7/8"
Hammer Weight & Fall 140 lb @ 30"

DEPTH	USCS	MATERIAL CLASSIFICATION	SAMPLE	SPT (6")	DRILL MODE	REMARKS
0	SM	FILL SILTY SAND: mod. yellowish brown; dry			GB	FILL to 12'
2	SP	SAND: mod. yellowish brown; medium angular sand; moist				
	CL	SANDY CLAY: yellowish brown; moist				
4	SM			9	DR	1.0/1.0
		SILTY SAND: mod. yellowish brown; fine subangular sand; low to mod. plastic fines; gravels rounded; loose; moist; occasional brick fragments	C-1	12		
					RD	set 5' casing, set up mud tub mixed 1 sack mud
6			J-1	3	SS	
				3		0.6/1.5
				3		
8		increased gravel content to GRAVELLY SILTY SAND			RD	
				5	DR	0.0/1.0
10				9		drove rock ahead
					RD	
			J-2	3	SS	0.4/1.5
12	SP			3		
				4		
		GRAVELLY SAND, loose			RD	
14				25	DR	0.4/1.0
			C-2	25		drove rock ahead
					RD	
16		grades moderately dense		4	SS	0.0/1.5
				6		drove rock ahead
				9		
18					RD	
						lost circulation;mixed 2 1/2 sacks mud
				7	DR	
20				19		0.0/1.0

Project DESIGN UNIT A-100 Date Drilled 7-26-83 Hole No. 3-1

DEPTH	USCS	MATERIAL CLASSIFICATION	SAMPLE	SPT (6")	DRILL MODE	REMARKS
20	SP	GRAVELLY SAND				drove casing 20.5' lost circulation at bottom of casing, got load of H20
		B.H. 21.2' Terminated hole, backfilled				21.2' hard probably steel can't advance abandoned hole 12:30 attempted drive bent shoe Moved rig and drilled Boring 3-1A
22						
24						
26						
28						
30						
32						
34						
36						
38						
40						
42						
44						

Sheet 2 of 5

Converse Consultants, Inc.
Earth Sciences Associates
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BORING LOG 3-1A

Proj: DESIGN UNIT A-100
Date Drilled 7-26-83
Ground Elev. 279
Drill Rig FAILING 1500
Logged By L. Schoeberlein
Total Depth 49.5
Hole Diameter 4 7/8
Hammer Weight & Fall 140 lb. @ 30"

DEPTH	USCS	MATERIAL CLASSIFICATION	SAMPLE	SPT. (6")	DRILL MODE	REMARKS
0	SM	FILL SILTY SAND: mod. yellowish brown; loose; dry; contains brick		GB		
4	CL	SANDY CLAY: mottled white & grey; med stiff, paint slop white				
6	SP	GRAVELLY SAND: mod. yellowish brown; medium dense, moist; mostly fine subangular sand, and rounded gravel and cobbles to 5"; granitic and metamorphic				
12				RD		
20		encountered metamorphic boulder, fresh ~2' thick				

Sheet 3 of 5

Project DESIGN UNIT A-100 **Date Drilled** 7-26-83 **Hole No.** 3-1A

DEPTH	USCS	MATERIAL CLASSIFICATION	SAMPLE	SPT (6")	DRILL MODE	REMARKS
20	SP	metamorphosed granite		RD		
	SW	YOUNG ALLUVIUM: GRAVELLY SAND:salt & pepper, v. dense; moist; fine to med. sand; with some granitic and metamorphic subangular to subround gravels; fresh				broke through @ 21' lost circulation mixed mud
22						
			J-3	6 6 8	SS	0.7/1.5
24					RD	
26				45	DR	0.7/1.0
			C-3	50		
					RD	set casing to 26' mixed mud
28						
		decreasing gravel content	J-4	45 50-4"	SS	0.5/1.0
30	SP	FINE TO MEDIUM SAND: med. grey, white and pink; v. dense; moist to wet; fine sand with fines; some gravel & granitic sand			RD	
				77	DR	0.8/1.0
32			C-4	95		
					RD	
			J-5	55 70	SS	0.7/1.0
34					RD	
36				77 100-5"	CR	0.0/1.0
38	SP	GRAVELLY SAND: med. greenish grey; med. sand, silt and gravel; v. dense wet	J-6	80	SS	0.6/0.5
					RD	dry in a.m.
40						
			C-5	150-4"	DR	0.3/0.3
42		increased gravel content				heavy chatter
			J-7	57 74	SS	0.8/1.0
44						

Project DESIGN UNIT A-100 Date Drilled 7-26-83 Hole No. 3-1A

DEPTH	USCS	MATERIAL CLASSIFICATION	SAMPLE	SPT (6")	DRILL MODE	REMARKS
44	SP	(cont.) <u>GRAVELLY SAND</u>			RD	heavy chatter
46			C-6	100-3"	DR	0.2/0.2
					RD	
48						
			J-8	137	SS	0.4/0.5
50		B.H. 49.5' terminated hole; installed 49' of 2" ABS; bottom 20' slotted backfilled around perforations w/ pea gravel, then pulled casing				8:30 a.m. used 26 sacks bentonite mixed mud continuously after setting 26' of casing
52						
54						
56						
58						
60						
62						
64						
66						
68						

Sheet 5 of 5

Converse Consultants, Inc.
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Geo/Resource Consultants

BORING LOG 3-2

Proj: DESIGN UNIT A-100
Date Drilled 7-27-83
Ground Elev. 276
Drill Rig FAILING 1500
Logged By L. Schoeberlein
Total Depth 49.5
Hole Diameter 4 7/8"
Hammer Weight & Fall 140 lb. @ 30"

DEPTH	USCS	MATERIAL CLASSIFICATION	SAMPLE	SPT (6")	DRILL MODE	REMARKS
0	SM	FILL. <u>SILTY SAND</u>:dark yellowish brown fine sand; and coarse gravel & fine cobbles; v. dense; dry to moist; contains brick and concrete chunks			GB	FILL to 11' (?)
2						
4				19	DR	0.5/1.0
		becomes siltier	C-1	25		
					GB	
6						
8						
		clayey zone~6"				
10						
	SM	<u>SILTY SAND</u>: yellowish brown, loose to med. dense	J-1	5 4 3	SS	1.5/1.5 CASING to 14.5'
12						
14						
		some gravel	C-2	10 6	DR	1.0/1.0
16						
		sand becoming v. fine	J-2	5 5 10	SS	0.7/1.5
18						
				12 20	DR	0/1.0
20						

Sheet 1 of 3

Project DESIGN UNIT A-100 **Date Drilled** 7-27-84 **Hole No.** 3-2

DEPTH	USCS	MATERIAL CLASSIFICATION	SAMPLE	SPT (6")	DRILL MODE	REMARKS
20	SM	(Cont.) SILTY SAND				Lost circulation
		YOUNG ALLUVIUM -	J-3	2 3 21	SS	1.0/1.5
22	SP	GRAVELLY SAND: salt & pepper; fine to medium sand; some fine to medium gravel, subangular to subround granite origin; v. dense; dry to moist Metagranitics to > 2"				drove casing to 21.5'
24			C-3	76 65-	DR 1½"	0.5/1.0 Rock in shoe
26			J-4	25 76	SS	0.5/1.0
28		some brown mottling	C-4	100 65-	DR 3"	0.5/0.7
30						
32		occasional thin clayey lenses	J-5	25 32 42	SS	0.6/1.5
34		grain size of sand increasing with depth	C-5	93 60-	DR 2"	0.5/0.7
36			J-6	30 81	SS	0.5/1.0
38	SP	GRAVELLY SAND: Medium greenish gray; fine sand, fine to coarse gravel; v. dense; wet				
40				130.3	"DR	intense chatter lost circulation metagranitic rock in shoe 0/0.3
				80	SS	intense chatter 0.0.5
42						intense rig chatter
44						

Sheet 2 of 3

Project DESIGN UNIT A-100 Date Drilled 7-27-84 Hole No. 3-2

DEPTH	USCS	MATERIAL CLASSIFICATION	SAMPLE	SPT (6")	DRILL MODE	REMARKS
44	SD	GRAVELLY SAND:	B-1	150.3	DR	0.2/0.2
46			J-7	120.4"	SS	0.3/0.3
48						
				120	DR	0/0.5
50		B.H. 49.5' Terminated, backfilled w/cuttings and grouted surface				used 9 sacks bentonite
52						
54						
56						
58						
60						
62						
64						
66						
68						

Sheet 3 of 3

Converse Consultants, Inc.
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BORING LOG 3-3

Proj: DESIGN UNIT A-100 **Date Drilled** 6/28/83 **Ground Elev.** ±269'

Drill Rig ROTARY **Logged By** J.R. Stellar **Total Depth** 50'

Hole Diameter 4 3/4" **Hammer Weight & Fall** 320 lb @ 36"

DEPTH	USCS	MATERIAL CLASSIFICATION	SAMPLE	SPT (6")	DRILL MODE	REMARKS
0	SP/SM	FILL MIXED SAND AND SILTY SAND with broken brick; loose to med. dense; dry to slightly moist			RD	
2						
4	SP	ALLUVIUM SAND alternating with GRAVELLY SAND: medium brown; medium dense				
				4	S	
				6	P	
6			J-1	21	T	
					RD	
8						
		becomes dense mostly medium grained sand and gravel				
10		grades to SANDY GRAVEL	30K 12" C-1		DR	
				15	S	
12				30	P	
			J-2	60	T	
					RD	
14						
				6	S	
				10	P	
16				17	T	
					RD	
18		grades to GRAVELLY SAND				
20						

Sheet 1 of 3

Project DESIGN UNIT A-100 **Date Drilled** 6/28/83 **Hole No.** 3-3

DEPTH	USCS	MATERIAL CLASSIFICATION	SAMPLE	SPT (6")	DRILL MODE	REMARKS
20	SP		40K 12"		DR	no recovery @ 20'
				16 30		
22			J-3	25		
					RD	
24						
				20 30	S P	
26		grades dense to very dense	J-4	50	T	
					RD	
28						
30			50K 12"		DR	no recovery @ 30'
				15 20	S P	
32			J-5	100	T	
					RD	
34		color changes to olive gray to black; wet; very dense; poorly graded				
				20 50 100	S P	
36			J-6	2"	T	
					RD	very hard, slow drilling
38						
40				90	S P	too hard for Converse sampler
			J-7	50-2"	T	
42						
44						

Sheet 2 **of** 3

Project DESIGN UNIT A-100 **Date Drilled** 6/28/83 **Hole No.** 3-3

DEPTH	USCS	MATERIAL CLASSIFICATION	SAMPLE	SPT (6")	DRILL MODE	REMARKS
44	SP	ALLUVIUM (continued)			RD	
			60K 8" C-2		DR	
					RD	
46				30	S	
				50	P	
			J-8	90	T	
					RD	
48						
50				30	SPT	stop drilling @ 1210 PM
				60		
		END BORING @ 51.0				
52						
54						
56						
58						
60						
62						
64						
66						
68						

Sheet 3 of 3

Converse Consultants, Inc.
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BORING LOG 3-4

Proj: DESIGN UNIT A-100
Date Drilled 6/28/83
Ground Elev. ±269
Drill Rig ROTARY
Logged By J.R. Stellar
Total Depth 45
Hole Diameter 4 3/4"
Hammer Weight & Fall 320 lb @ 30"

DEPTH	USCS	MATERIAL CLASSIFICATION	SAMPLE	SPT (6")	DRILL MODE	REMARKS
0	SP/SM	FILL MIXED SAND AND SILTY SAND with broken brick; loose; dry to slightly moist			RD	
2						
	SP	ALLUVIUM SAND alternating with GRAVELLY SAND: brown; medium dense				
4						
6			J-1	4 12 22	SPT	
8						
10			25K 12" C-1			continuous caving from 10' - 46.5'
12				8 12 22	SPT RD	no recovery at 11'
14						
16			J-2	12 17 25	SPT RD	
18		increasing gravel content				some rig chatter
20						

Sheet 1 of 3

Project DESIGN UNIT A-100 **Date Drilled** 6/28/83 **Hole No.** 3-4

DEPTH	USCS	MATERIAL CLASSIFICATION	SAMPLE	SPT (6")	DRILL MODE	REMARKS
20		ALLUVIUM (continued)	40K 12" C-2		DR	
				14 50 56	S P T	
22					RD	
24						
		increasing gravel content	J-3	25 35 100	S P T	
26					RD	
28						
30			75K 6" C-3		DR	
					RD	
			J-4	25 30 100	S P T	
32					RD	
34						
				25 30 100	S P T	no recovery @ 35'
36					RD	
38		color change to olive gray to black; silt; fine to coarse-grained sand and gravel to 3"; very moist to wet; dense to very dense; poorly graded				
40			70K 12"		DR	no recovery @ 40'
			J-5	35 40 60	S P T	
42					RD	
44						

Sheet 2 of 3

Project DESIGN UNIT A-100 Date Drilled 6/28/83 Hole No. 3-4

DEPTH	USCS	MATERIAL CLASSIFICATION	SAMPLE	SPT (6")	DRILL MODE	REMARKS
44		ALLUVIUM (continued)			RD	
				35	S	almost continuous
				40	P	caving problems from
46			J-6	50	T	10'-46.5'
		end boring @ 46.5				stop drilling @ 6:15 PM 6-28-83
48						
50						
52						
54						
56						
58						
60						
62						
64						
66						
68						

Sheet 3 of 3

THIS BORING LOG IS BASED ON FIELD CLASSIFICATION AND VISUAL SOIL DESCRIPTION, BUT IS MODIFIED TO INCLUDE RESULTS OF LABORATORY CLASSIFICATION TESTS WHERE AVAILABLE. THIS LOG IS APPLICABLE ONLY AT THIS LOCATION AND TIME. CONDITIONS MAY DIFFER AT OTHER LOCATIONS OR TIME.

Converse Consultants, Inc.
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BORING LOG 3-5

Proj: DESIGN UNIT A-100
Date Drilled 6-29-83
Ground Elev. 268'
Drill Rig ROTARY
Logged By J. R. Stellar
Total Depth 40.5'
Hole Diameter 4 3/4"
Hammer Weight & Fall 320 lb @ 30"

DEPTH	USCS	MATERIAL CLASSIFICATION	SAMPLE	SPT (6")	DRILL MODE	REMARKS
0	SP/SM	FILL <u>MIXED SAND AND SILTY SAND</u>: with with broken brick, loose, dry			RD	
2						
	SP	ALLUVIUM <u>SAND</u>: alternating with <u>GRAVELLY SAND</u>: medium brown, medium dense, slightly moist				
4						
				12 16 20	S P T	No recovery @ 5'
6					RD	
8						
		Grades to GRAVELLY SAND				
10			40K 12" C-1		DR	
12			J-1	12 16 25	S P T	
					RD	
14						
16			J-2	6 20 30	S P T	
					RD	
18		Grades to very dense				
20						

Sheet 1 of 2

Project DESIGN UNIT A-100 **Date Drilled** 6-29-83 **Hole No.** 3-5

DEPTH	USCS	MATERIAL CLASSIFICATION	SAMPLE	SPT (6")	DRILL MODE	REMARKS
20	SP	ALLUVIUM	60K 12" C-2		DR	
				40	S	
				70	P	
22			J-3	85	T	
					RD	
24						
			50K 12" C-3		DR	
26				30	S	
				60	P	
			J-4	83	T	
					RD	
28						
30				50	SPT	
			J-5	160 4"		
					RD	
32						
34		Grades to olive gray and black, very moist to wet, dense to very dense, poorly graded				
			80K 3"		DR	No recovery @ 35'
					RD	
36				100 5"	SPT	No recovery @ 36'
					RD	
38						
40				100 5"	SPT	No recovery @ 40'
		END OF BORING @ 40.5'				
42						
44						

Sheet 2 of 2

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BORING LOG 3-6

Proj: DESIGN UNIT A-100 Date Drilled 6-29-83 Ground Elev. ±268'
Drill Rig Rotary Logged By J. Stellar Total Depth 40.8'
Hole Diameter 4 3/4" Hammer Weight & Fall 320 lb. @ 30"

DEPTH	USCS	MATERIAL CLASSIFICATION	SAMPLE	SPT (6")	DRILL MODE	REMARKS
0		FILL MIXED SAND, GRAVELLY SAND, AND SILTY SAND: dry Loose			RD	
2						
4	SP	ALLUVIUM SAND alternating with GRAVELLY SAND, brown medium dense slightly moist				
				18	S	
				20	P	
6			J-1	22	T	
					RD	
8						
		becomes dense				
10			30k 12"			no recovery @ 10'
					DR	
				25	S	
				40	P	
12			J-2	50	T	
					RD	
14		becomes v. dense				
				30	S	
				60	P	
16			J-3	60	T	
		grades more gravelly				
18						
20						

Sheet 1 of 2

Project DESIGN UNIT A-100 **Date Drilled** 6-29-83 **Hole No.** 3-6

DEPTH	USCS	MATERIAL CLASSIFICATION	SAMPLE	SPT (6")	DRILL MODE	REMARKS
20	SP	ALLUVIUM SAND	50k 12 C-1		DR	
				40 10 2"	S,P T	poss. recovery @ 20' no recovery @ 21'
22					RD	
24						
			J-4	45 100	S P T	
26					RD	
28						
30			60k2" J-5	100	SPT	no recovery @ 30'
					RD	
32						
		grading to olive gray & black v. moist to wet, dense to v. dense, poorly graded				
34						
			J-6		SPT RD	
36						
38						
40			60k1" J-7	100	SPT	no recovery @ 40' finished boring @ 1:30 p.m.
		end boring at 40.8'				
42						
44						

Sheet 2 of 2

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BORING LOG 3-7

Proj: DESIGN UNIT A-100
Date Drilled 6-29-83
Ground Elev. ±266'
Drill Rig ROTARY
Logged By J. Stellar
Total Depth 30.5'
Hole Diameter 4 3/4"
Hammer Weight & Fall 320 lb. @ 30"

DEPTH	USCS	MATERIAL CLASSIFICATION	SAMPLE	SPT (6")	DRILL MODE	REMARKS
0	SP	FILL MIXED SAND AND GRAVELLY SAND: dry, loose			RD	
2						
	SP	ALLUVIUM SAND: alternating with GRAVELLY SAND, brown, medium dense, slightly moist				
4						
			J-1	12 18 24	S P T	
6					RD	
8						
10			50k 12"		DR	no recovery @ 10'
			J-2	4 16 24	S P T	
12					RD	
14		grades to dense				
			J-3	12 24 36	S P T	
16						
18						
20		grades to v. dense				

Sheet 1 of 2

Project DESIGN UNIT A -100 Date Drilled 6-29-83 Hole No. 3-7

DEPTH	USCS	MATERIAL CLASSIFICATION	SAMPLE	SPT (6")	DRILL MODE	REMARKS
20		ALLUVIUM SAND	60k		DR	
				50	SPT	
22			J-4	100		
24						
				25	SPT	
				50		
26			J-5	50		
28						
30			60k1"			no recovery @ 30' tip of sample barrel broken stop drilling @ 4:30 p.m.
			J-6	100	SPT	
		End Boring @ 30.5; set 2" ABS Piezometer to 30.5				
32						
34						
36						
38						
40						
42						
44						

Sheet 2 of 2

THIS BORING LOG IS BASED ON FIELD CLASSIFICATION AND VISUAL SOIL DESCRIPTION, BUT IS MODIFIED TO INCLUDE RESULTS OF LABORATORY CLASSIFICATION TESTS WHERE AVAILABLE. THIS LOG IS APPLICABLE ONLY AT THIS LOCATION AND TIME. CONDITIONS MAY DIFFER AT OTHER LOCATIONS OR TIME.

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BORING LOG 3-8

Proj: DESIGN UNIT A-100 **Date Drilled** 6-30-83 **Ground Elev.** 263'

Drill Rig ROTARY **Logged By** J. S. Stellar **Total Depth** 30.7'

Hole Diameter 4 3/4" **Hammer Weight & Fall** 320 lb @ 30"

DEPTH	USCS	MATERIAL CLASSIFICATION	SAMPLE	SPT (6")	DRILL MODE	REMARKS
0	SM/SP	FILL MIXED SAND AND GRAVELLY SAND: dry, loose				Start drilling @ 7:00 am
2	SP	ALLUVIUM-SAND: alternating with GRAVELLY SAND, brown, dense, slightly moist				
4						
				50	S	
				50	P	
6			J-1	40	T	
					RD	
8						
		becomes very dense				
10			30K 12" C-1			
				12	S	
				30	P	
12			J-2	60	T	
					RD	
14						
				6	S	
				34	P	
16			J-3	40	T	
					RD	
18		increasing gravel content				
20						

Sheet 1 of 2

Project DESIGN UNIT A-100 **Date Drilled** 6-30-83 **Hole No.** 3-8

DEPTH	USCS	MATERIAL CLASSIFICATION	SAMPLE	SPT (6")	DRILL MODE	REMARKS
20	SP	ALLUVIUM-SAND:	70K 12"		DR	No recovery @ 20', too gravelly to stay in sampler
				50	S	
				50	P	
22			J-4	60	T	
					RD	
24						
				50	S	
26				50	P	
			J-5	75	T	
					RD	
28						
30			10K 4"			No recovery @ 30'
			J-6	100 5"	SPT	
		END OF BORING AT 30.7'				stop drilling @ 7:40 am
32						
34						
36						
38						
40						
42						
44						

Sheet 2 of 2

Converse Consultants, Inc.
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BORING LOG 3-9

Proj: DESIGN UNIT A-100 **Date Drilled** 7-28-83 **Ground Elev.** 266'
Drill Rig FAILING 1500 **Logged By** L. Schoeberlein **Total Depth** 30.0'
Hole Diameter 4 7/8" **Hammer Weight & Fall** 140 lb @ 30"

DEPTH	USCS	MATERIAL CLASSIFICATION	SAMPLE	SPT (6")	DRILL MODE	REMARKS
0	AC	2" ASPHALT			GB	
	SP	FILL SAND: yellowish brown, fine sand, loose, dry				
2	SP	SAND: salt and pepper, fine to medium sand, fine gravel, medium dense, dry to moist gravels rounded granitic and meta-granitic, bedded with very fine sand lenses to 6" thick	C-1	7 12	DR	1.0/1.0
					GB	
4			J-1	3 8 11	SS	1.2/1.5
					GB	
6			C-2	9 12	DR	1.0/1.0
					GB	
8		increased gravel content, medium to coarse well-rounded to subangular	J-2	14 9 7	SS	0.5/1.0
					GB	
10			C-3	14 25	DR	1.0/1.0
					GB	
12			J-3	14 24 43	SS	1.2/1.5
14					RD	set up tub and placed 13.5' casing mixed mud
			B-1	18 13	DR	0.2/1.0 sample fell out
16	ML	SANDY SILT: yellowish brown, nonplastic fines, very fine sand, medium dense, v. stiff, moist, contains occasional medium subangular to sub-rounded gravels			RD	
18			J-4	51 60	SS	0.3/1.0 rock in shoe
					RD	
20						

Sheet 1 of 2

Project DESIGN UNIT A-100 Date Drilled 7-28-83 Hole No. 3-9

DEPTH	USCS	MATERIAL CLASSIFICATION	SAMPLE	SPT (6")	DRILL MODE	REMARKS
20	SM	grades to SILTY SAND, SM, very fine sand, nonplastic.		16	DR	0.9/1.0
			C-4	12		
					RD	
22		increased gravel				
				55- 1"	SS	0 recovery bouncing, intense rig chatter
24						
				25	DR	0.8/1.0
26			C-5	40		
				22	SS	0.3/1.0
			J-5	68		
					RD	
28						
	SP	SAND: medium to coarse subangular sand		67	DR	0.5/1.0
			C-6	99		
30		B.H. 30.0' Terminated, installed piezometer to 29', 20' slotted section at bottom				complete drilling 10:15, used 3 sacks mud
32						
34						
36						
38						
40						
42						
44						

Sheet 2 of 2

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BORING LOG 3-10

Proj: DESIGN UNIT A-100
Date Drilled 7-7-83
Ground Elev. 266'
Drill Rig ROTARY
Logged By J. R. Stellar
Total Depth 29.5'
Hole Diameter 4 3/4"
Hammer Weight & Fall 320 lb @ 30"

DEPTH	USCS	MATERIAL CLASSIFICATION	SAMPLE	SPT (6")	DRILL MODE	REMARKS
0		1.0' A.C. PAVEMENT				
	SM SP	FILL-MIXED SAND, GRAVELLY SAND, SILTY SAND AND SILT				
2	SP	ALLUVIUM SAND: alternating with GRAVELLY SAND, brown, dense to very dense, slightly moist				
4						
				14	S	
				20	P	
6			J-1	30	T	
8						
10			20K 12"			No recovery @ 10'
12		grades to GRAVELLY SAND				
14						
				60	S P	
16			J-2	75/4"	T	
18						
20						

Sheet 1 of 2

Project DESIGN UNIT A-100 Date Drilled 7-7-83 Hole No. 3-10

DEPTH	USCS	MATERIAL CLASSIFICATION	SAMPLE	SPT (6")	DRILL MODE	REMARKS
20	SP	ALLUVIUM GRAVELLY SAND	70K 12" C-1		DR	
					RD	
22						
24						
				26	S	
				35	P	
26			J-3	50	T	
					RD	
28						
			75K 6"		DR	No recovery @ 29'
30		END BORING @ 29.5'				
32						
34						
36						
38						
40						
42						
44						

Sheet 2 of 2

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BORING LOG 3-11

Proj: DESIGN UNIT A-100
Date Drilled 7-8-83
Ground Elev. 264'
Drill Rig ROTARY
Logged By J.D. Stellar
Total Depth 40.5'
Hole Diameter 4 3/4"
Hammer Weight & Fall 320 lb @ 30"

DEPTH	USCS	MATERIAL CLASSIFICATION	SAMPLE	SPT (6")	DRILL MODE	REMARKS
0	SM SP	FILL MIXED SAND, GRAVELLY SAND AND SILTY SAND: dry, loose				
2	SP	ALLUVIUM-SAND: alternating with GRAVELLY SAND, brown, dense, slightly moist				
4						
				9	S	
				18	P	
6			J-1	25	T	
					RD	
8						
10			40K 12"		DR	No recovery @ 10'
					RD	
12						
14						
				18	S	No recovery @ 15'
				24	P	
16				40	T	
					RD	
18						
20						

Sheet 1 of 2

Project DESIGN UNIT A-100 **Date Drilled** 7-8-83 **Hole No.** 3-11

DEPTH	USCS	MATERIAL CLASSIFICATION	SAMPLE	SPT (6")	DRILL MODE	REMARKS
20	SP	ALLUVIUM-SAND	60K 12" C-1		DR	Poor recovery @ 20' 2 rings only
					RD	
22						
24						
				26	S	
				40	P	
26			J-2	45	T	
					RD	
28						
30			50K-1"		DR	No recovery @ 30'
32						
34						
			25K 12" C-2		DR	
36						
38						
		at 39' alluvium becomes olive black, very dense, poorly graded, wet				
40			J-3	100	SPT	
		END BORING @ 40.5'				
42						
44						

Sheet 2 of 2

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BORING LOG 3-12

Proj: DESIGN UNIT A-100 **Date Drilled** 7-9-83 **Ground Elev.** ±265'

Drill Rig Rotary **Logged By** J. Stellar **Total Depth** 50.0'

Hole Diameter 4 3/4" **Hammer Weight & Fall** 320 lb. @ 30"

DEPTH	USCS	MATERIAL CLASSIFICATION	SAMPLE	SPT (6")	DRILL MODE	REMARKS
0		1.0 A. C. Pavement FILL-MIXED SAND, GRAVELLY SAND, AND SILTY SAND: loose, dry			RD	A. C. is 1.0' thick
2	SM/ SP					
	SP	ALLUVIUM-SAND alternating with GRAVELLY SAND, medium brown, medium dense, slightly moist				
4						
				4 9	S P	
6			J-1	12	T	
					RD	
8						
10			20k 12" C-1		DR	
12						
		grades to GRAVELLY SAND				
14						
				16 40	S P	
16			J-2	45	T	
					RD	
18						
20						

Sheet 1 of 3

Project DESIGN UNIT A-100 Date Drilled 7-9-83 Hole No. 3-12

DEPTH	USCS	MATERIAL CLASSIFICATION	SAMPLE	SPT (6")	DRILL MODE	REMARKS
20	SP	ALLUVIUM-GRAVELLY SAND	35k 12"		DR	No recovery @ 20'
22						
24		grades to SAND				
				16 30	S P	
26			J-3	30	T	
					RD	
28		grades to GRAVELLY SAND				
30			50k½"		DR	no recovery @ 30'
					RD	
32						
34						
		color change to olive black, v. dense, poorly graded, wet		100/5"	SPT RD	no recovery @ 35'
36						
38						
40			50k½"		DR RD	
42						
44						

Sheet 2 of 3

Project DESIGN UNIT A-100 **Date Drilled** 7-9-83 **Hole No.** 3-12

DEPTH	USCS	MATERIAL CLASSIFICATION	SAMPLE	SPT (6")	DRILL MODE	REMARKS
44	SW	ALLUVIUM -SAND			RD	no recovery @ 45'
				100-4"	SPT	
46						
48						
				30	SPT	
				100-5"		
50		end boring @ 50.0'				
52						
54						
56						
58						
60						
62						
64						
66						
68						

Sheet 3 of 3

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BORING LOG 3-13

Proj: DESIGN UNIT A-100
Date Drilled 7-8-83
Ground Elev. 265'
Drill Rig ROTARY
Logged By J. D. Stellar
Total Depth 39.2'
Hole Diameter 4 3/4"
Hammer Weight & Fall 320 lb @ 30"

DEPTH	USCS	MATERIAL CLASSIFICATION	SAMPLE	SPT (6")	DRILL MODE	REMARKS
0		FILL-A.C. PAVEMENT			RD	A.C. is 1' thick
	SP/SM	MIXED SAND AND GRAVELLY SAND AND SILTY SAND				
2						
	SP	ALLUVIUM-SAND: alternating with GRAVELLY SAND, brown, medium dense, slightly moist				
4						
				18	S	
6				26	P	
			J-1	40	T	
					RD	
8		gravel layer at 7.5'				
10			25K 12" C-1		DR	
					RD	
12						
14						
				16	S	
				25	P	
16			J-2	35	T	
					RD	
18						
20						

Sheet 1 of 2

Project DESIGN UNIT A-100 Date Drilled 7-8-83 Hole No. 3-13

DEPTH	USCS	MATERIAL CLASSIFICATION	SAMPLE	SPT (6")	DRILL MODE	REMARKS
20	SP	ALLUVIUM-SAND	36K 12"		DR	No recovery @ 20'
					RD	
22						
24		grades to GRAVELLY SAND				
				50	S	
				50	P	
26			J-3	40	T	
28						
30			35K 12" C-2		DR	
					RD	
32						
34		color change to olive black, very dense, poorly graded, wet				
				100	SPT	No recovery @35'
36						
38						
		END BORING @ 39.2'				
40						
42						
44						

Sheet 2 of 2

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BORING LOG 3-14

Proj: DESIGN UNIT A-100 **Date Drilled** 7-8-83 **Ground Elev.** 266'

Drill Rig ROTARY **Logged By** J. R. Stellar **Total Depth** 45.1'

Hole Diameter 4 3/4" **Hammer Weight & Fall** 320 lb @ 30"

DEPTH	USCS	MATERIAL CLASSIFICATION	SAMPLE	SPT (6")	DRILL MODE	REMARKS
0	AC	FILL-1.0' A.C. PAVEMENT				A.C. is 1.0' thick
	SP/SM	MIXED SAND, GRAVELLY SAND AND SILTY SAND: dry, loose				
2						
	SP	ALLUVIUM-SAND: alternating with GRAVELLY SAND, brown, medium dense, slightly moist				
4						
				5	S	
				12	P	
6			J-1	15	T	
					RD	
8						
10			40K 12" C-1		DR	
12						
14						
				15	S	
				30	P	
16			J 2	30	T	
					RD	
18						
20						

Sheet 1 of 3

Project DESIGN UNIT A-100 Date Drilled 7-8-83 Hole No. 3-14

DEPTH	USCS	MATERIAL CLASSIFICATION	SAMPLE	SPT (6")	DRILL MODE	REMARKS
20	SP	ALLUVIUM-SAND	50K 12"		DR	No recovery @ 20'
22						
24				17	S	
				30	P	
26			J-3	40	T	
					RD	
28						
30			75K 6"		DR RD	
32						
34		color change to olive black, very dense, poorly graded, wet				
				75 5"	SPT	No recovery @ 35'
36						
38						
40			75K 2"		DR RD	No recovery @ 40'
42						
44						

Sheet 2 of 3

Project DESIGN UNIT A-100 **Date Drilled** 7-8-83 **Hole No.** 3-14

DEPTH	USCS	MATERIAL CLASSIFICATION	SAMPLE	SPT (6")	DRILL MODE	REMARKS
44	SP	ALLUVIUM-SAND		100-1"	SPT	
		END BORING @ 45.1' due to extremely hard, slow drilling				
46						
48						
50						
52						
54						
56						
58						
60						
62						
64						
66						
68						

Sheet 3 of 3

THIS BORING LOG IS BASED ON FIELD CLASSIFICATION AND VISUAL SOIL DESCRIPTION, BUT IS MODIFIED TO INCLUDE RESULTS OF LABORATORY CLASSIFICATION TESTS WHERE AVAILABLE. THIS LOG IS APPLICABLE ONLY AT THIS LOCATION AND TIME. CONDITIONS MAY DIFFER AT OTHER LOCATIONS OR TIME.

Converse Consultants, Inc.
Earth Sciences Associates
Geo/Resource Consultants

BORING LOG 3-15

Proj: DESIGN UNIT A-100 Date Drilled 7-28-83 Ground Elev. 264'

Drill Rig FAILING 1500 Logged By L. Schoeberlein Total Depth 30.5'

Hole Diameter 4 7/8" Hammer Weight & Fall 140 lb @ 30"

DEPTH	USCS	MATERIAL CLASSIFICATION	SAMPLE	SPT (6")	DRILL MODE	REMARKS
0	AC GP SP	FILL-2" A.C. PAVEMENT 6" BASE ROCK SAND: salt and pepper, very fine to fine sand, silt, medium dense to dense			GB	
				21 32	DR	Fell out
2					GB	
	SP	ALLUVIUM-SAND AND GRAVELLY SAND	J-1	12 13 16	SS	1.5/1.5
4					GB	
			C-1	14 20	DR	1.0/1.0
6					GB	
			J-2	9 12 22	SS	1.5/1.5
8					GB	
			C-2	21 40	DR	1.0/1.0
10		becoming slighty coarser grained with occasional rounded gravels			RD	Set tub, pushed 10' casing
			J-3	5 12 33	SS	0.6/1.5
12					RD	
14		becoming coarser, fine gravel		60 60-1"	DR	No recovery, bent bit
					RD	Heavy chatter
16			J-4	52 66	SS	0.5/1.0
					RD	
18		cobbles		100 100-2.5"	DR	3" rock in shoe drive
20			J-5	90-6"	DR	

Sheet 1 of 2

Project DESIGN UNIT A-100 **Date Drilled** 7-28-83 **Hole No.** 3-15

DEPTH	USCS	MATERIAL CLASSIFICATION	SAMPLE	SPT (6")	DRILL MODE	REMARKS
20	SP	ALLUVIUM - SAND AND GRAVELLY SAND	C-3		DR	Disturbed
					RD	
			J-5	66 67	SS	0.5/1.0
22					RD	
24				72 3"	DR	No recovery
					RD	
26						
			J-6	38 54 80	SS	1.0/1.5
28					RD	
30			C-4	190	DR	0.2/0.5
		B.H. 30.5' Terminated, installed 30' of 2" ABS bottom 20' slotted, flushed hole through PVC in case of cave-in				Completed drilling 4:00 pm 4 sacks bentonite used
32						
34						
36						
38						
40						
42						
44						

Sheet 2 of 2

Converse Consultants, Inc.
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BORING LOG 3-16

Proj: DESIGN UNIT A-100
Date Drilled 6-30-83
Ground Elev. 262'
Drill Rig ROTARY
Logged By J. R. Stellar
Total Depth 20'
Hole Diameter 4 3/4"
Hammer Weight & Fall 320 lb @ 30"

DEPTH	USCS	MATERIAL CLASSIFICATION	SAMPLE	SPT (6")	DRILL MODE	REMARKS
0	SM SP	FILL-MIXED SAND AND GRAVELLY SAND loose, dry			RD	Start drilling with air
2						
	SP	ALLUVIUM-SAND: alternating with GRAVELLY SAND, brown, loose, slightly moist				
4						
				12	S	
				20	P	
6			J-1	20	T	
					RD	
8						
10		grades to medium dense	40K 12" C-1		DR	
				12	S	
12				20	P	
			J-2	40	T	
					RD	
14						Switch to mud @ 14'
				20	S	
16				30	P	
			J-3	45	T	
18		grades to dense				
20		END OF BORING @ 20'	80K 12" C-2		DR	Sheet 1 of 1

Converse Consultants, Inc.
Earth Sciences Associates
Geo/Resource Consultants

BORING LOG 3-17

Proj: DESIGN UNIT A-100
Date Drilled 6-30-83
Ground Elev. 262'
Drill Rig ROTARY
Logged By J. Stellar
Total Depth 20'
Hole Diameter 4 3/4"
Hammer Weight & Fall 320 lb @ 30"

DEPTH	USCS	MATERIAL CLASSIFICATION	SAMPLE	SPT (6")	DRILL MODE	REMARKS
0	SM/SP	FILL- MIXED SAND AND GRAVELLY SAND loose, dry			RD	Start drilling @ 10:55 AM
2						
4	SP	ALLUVIUM-SAND: alternating with GRAVELLY SAND, brown, medium dense, slightly moist				
				25	S	
				45	P	
6			J-1	50	T	
					RD	
8						
		becomes dense to very dense				
10			40K 12" C-1		DR	
				30	SpT	
			J-2	65-3"		
12					RD	
14						
				20	S	
				50	P	
16			J-3	40	T	
					RD	
18						
						No recovery @ 19' Stopped drilling @ 12
20		END OF BORING @ 20.0'	40K 12"		DR	

Sheet 1 of 1

THIS BORING LOG IS BASED ON FIELD CLASSIFICATION AND VISUAL SOIL DESCRIPTION, BUT IS MODIFIED TO INCLUDE RESULTS OF LABORATORY CLASSIFICATION TESTS WHERE AVAILABLE. THIS LOG IS APPLICABLE ONLY AT THIS LOCATION AND TIME. CONDITIONS MAY DIFFER AT OTHER LOCATIONS OR TIME.

Converse Consultants, Inc.
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BORING LOG 3-18

Proj: DESIGN UNIT A-100
Date Drilled 7-7-83
Ground Elev. 262'
Drill Rig ROTARY
Logged By J. R. Stellar
Total Depth 30.0'
Hole Diameter 4 3/4"
Hammer Weight & Fall 320 lb @ 30"

DEPTH	USCS	MATERIAL CLASSIFICATION	SAMPLE	SPT (6")	DRILL MODE	REMARKS
0	AC	FILL-MIXED GRAVELLY SAND, SAND, SILTY SAND: loose dry			RD	A.C. is 0.5' thick
	SM/SP					
2						
4						
	SP	ALLUVIUM-SAND alternating with GRAVELLY SAND, brown, medium dense, slightly moist		24	S	
				30	P	
6			J-1	30	T	
					RD	
8						
10			40K 12" C-1		DR	
					RD	
12						
14						
				28	S	
				36	P	
16			J-2	46	T	
					RD	
18						
20						

Sheet 1 of 2

Project DESIGN UNIT A-100 Date Drilled 7-7-83 Hole No. 3-18

DEPTH	USCS	MATERIAL CLASSIFICATION	SAMPLE	SPT (6")	DRILL MODE	REMARKS
20	SP	ALLUVIUM-SAND	75K 12"		DR	no recovery @ 20'
					RD	
22						
24						
				36	S	
26				50	P	
			J-3	45	T	
					RD	
28						
			75K 12" C-2		DR	
30		end of boring @ 30'				
32						
34						
36						
38						
40						
42						
44						

Sheet 2 of 2

Converse Consultants, Inc.
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BORING LOG 3-19

Proj: DESIGN UNIT A-100
Date Drilled 7-7-83
Ground Elev. 261'
Drill Rig ROTARY
Logged By J. R. Stellar
Total Depth 29.5'
Hole Diameter 4 3/4"
Hammer Weight & Fall 320 lb @ 30"

DEPTH	USCS	MATERIAL CLASSIFICATION	SAMPLE	SPT (6")	DRILL MODE	REMARKS
0	SM/SP	FILL-MIXED SANDY SILT, SAND, GRAVELLY SAND, AND SILT: soft to loose, dry to slightly moist			RD	
2	SP	ALLUVIUM-SAND alternating with GRAVELLY SAND, brown, medium dense, slightly moist				
4						
				2	S	
				3	P	
6			J-1	4	T	
					RD	
8		grades to coarse SAND				
10			20K 12" C-1		DR	
					RD	
12						
		gravel layer @ 13'				
14						
				26	SPT	
			J-2	75		
16						
18						
20						

Sheet 1 of 2

Project DESIGN UNIT A-100 Date Drilled 7-7-83 Hole No. 3-19

DEPTH	USCS	MATERIAL CLASSIFICATION	SAMPLE	SPT (6")	DRILL MODE	REMARKS
20	SP	ALLUVIUM-GRAVELLY SAND	40K 12"		DR	no recovery @ 20'
					RD	
22						
24						
				25	SPT	
			J-3	100 5"		
26					RD	
28						
			25K-6 C-2			
30		end boring @ 29.5'				
32						
34						
36						
38						
40						
42						
44						

Sheet 2 of 2

THIS BORING LOG IS BASED ON FIELD CLASSIFICATION AND VISUAL SOIL DESCRIPTION, BUT IS MODIFIED TO INCLUDE RESULTS OF LABORATORY CLASSIFICATION TESTS WHERE AVAILABLE. THIS LOG IS APPLICABLE ONLY AT THIS LOCATION AND TIME. CONDITIONS MAY DIFFER AT OTHER LOCATIONS OR TIME.

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BORING LOG 3-20

Proj: DESIGN UNIT A-100 **Date Drilled** 7-7-83 **Ground Elev.** 261'

Drill Rig Rotary **Logged By** J. R. Stellar **Total Depth** 30'

Hole Diameter 4 3/4 **Hammer Weight & Fall** 320 lb @ 30"

DEPTH	USCS	MATERIAL CLASSIFICATION	SAMPLE	SPT (6")	DRILL MODE	REMARKS
0	SM/SP	FILL-MIXED SAND, GRAVELLY SAND, SANDY SILT, AND SILT: soft to moderately dense, dry			RD	drill with mud
2	SP	ALLUVIUM-SAND alternating with GRAVELLY SAND, brown, medium dense, slightly moist				
4						
				12	S	
6				20	P	
			J-1	30	T	
					RD	
8						
10			15K 12"		DR	no recovery @ 10'
					RD	
12						
14						
				20	S	
				36	P	
16			J-2	40	T	
					RD	
18		gravel layer @ 18-19'				
20						

Sheet 1 of 2

Project DESIGN UNIT A-100 Date Drilled 7-7-83 Hole No. 3-20

DEPTH	USCS	MATERIAL CLASSIFICATION	SAMPLE	SPT (6")	DRILL MODE	REMARKS
20	SP	ALLUVIUM-SAND	60K 12"		DR	no recovery @ 20'
					RD	
22		gravel layer at 22-24'				
24						
				24	S	
				36	P	
26			J-3	46	T	
					RD	
28						
			70K 12"		DR	no recovery @ 30'
30		end boring @ 30'				
32						
34						
36						
38						
40						
42						
44						

Sheet 2 of 2

THIS BORING LOG IS BASED ON FIELD CLASSIFICATION AND VISUAL SOIL DESCRIPTION, BUT IS MODIFIED TO INCLUDE RESULTS OF LABORATORY CLASSIFICATION TESTS WHERE AVAILABLE. THIS LOG IS APPLICABLE ONLY AT THIS LOCATION AND TIME. CONDITIONS MAY DIFFER AT OTHER LOCATIONS OR TIME.

Converse Consultants, Inc.
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BORING LOG 3-21

Proj: DESIGN UNIT A-100 **Date Drilled** 7-7-83 **Ground Elev.** 260'

Drill Rig ROTARY **Logged By** J. R. Stellar **Total Depth** 30.0'

Hole Diameter 4 3/4" **Hammer Weight & Fall** 320 lb @ 30"

DEPTH	USCS	MATERIAL CLASSIFICATION	SAMPLE	SPT (6")	DRILL MODE	REMARKS
0	AC	FILL-0.5' A.C. PAVEMENT			RD	
	SM/SP	MIXED GRAVELLY SAND, SAND, AND SILTY SAND: loose to soft, dry to slightly moist				
2						
4	SP	ALLUVIUM-SAND alternating with GRAVELLY SAND, brown, medium dense, slightly moist				
				4	S	
				11	P	
6			J-1	11	T	
					RD	
8						
10			35K 12"		DR	no recovery @ 10'
					RD	
12						
14						
		grades to GRAVELLY SAND		12	S	
				45	P	
16			J-1	45	T	
					RD	
18						
20						

Sheet 1 of 2

Project DESIGN UNIT A-100 Date Drilled 7-7-83 Hole No. 3-21

DEPTH	USCS	MATERIAL CLASSIFICATION	SAMPLE	SPT (6")	DRILL MODE	REMARKS
20	SP	ALLUVIUM-GRAVELLY SAND AND SAND	50K 12"		DR	no recovery @ 20'
					RD	
22						
24						
				26	S	
26				40	P	
			J-3	40	T	
					RD	
28		interlayers of SILTY SAND				
			50K 12" C-1		DR	
30		end boring @ 30.0'				
32						
34						
36						
38						
40						
42						
44						

Sheet 2 of 2

Converse Consultants, Inc.
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BORING LOG 3-22

Proj: DESIGN UNIT A-100
Date Drilled 7-6-83
Ground Elev. 260'
Drill Rig ROTARY
Logged By J. R. Stellar
Total Depth 30.0'
Hole Diameter 4 3/4"
Hammer Weight & Fall 320 lb @ 330"

DEPTH	USCS	MATERIAL CLASSIFICATION	SAMPLE	SPT (6")	DRILL MODE	REMARKS
0	SM/SP	FILL-MIXED SAND, GRAVELLY SAND, SILTY SAND: dry, loose			RD	
2						
	SP	ALLUVIUM-SAND alternating with GRAVELLY SAND, brown, medium dense, slightly moist				
4						
				6	S	
6				6	P	
			J-1	8	T	
					RD	
8						
10			26K 12"		DR	no recovery @ 10'
					RD	
12		gravel layer 12-14'				rig chatter 12-14' hole caving almost continuously at 12'
14		grades to coarse SAND AND GRAVELLY SAND				
				12	S	
16				46	P	
			J-2	60	T	
					RD	
18						
20						

Sheet 1 of 2

Project DESIGN UNIT A-100 Date Drilled 7-6-83 Hole No. 3-22

DEPTH	USCS	MATERIAL CLASSIFICATION	SAMPLE	SPT 6"	DRILL MODE	REMARKS
20	SP	ALLUVIUM-COARSE SAND AND GRAVELLY SAND	24K 12"		DR	no recovery @ 20'
22						
24		layer of SANDY SILT @ 24-27'				
				12	S	
26				18	P	
			J-3	26	T	
		grades to coarse SAND				
28						
			50K 12" C-1			
30		end boring @ 30.0'				
32						
34						
36						
38						
40						
42						
44						

Sheet 2 of 2

THIS BORING LOG IS BASED ON FIELD CLASSIFICATION AND VISUAL SOIL DESCRIPTION, BUT IS MODIFIED TO INCLUDE RESULTS OF LABORATORY CLASSIFICATION TESTS WHERE AVAILABLE. THIS LOG IS APPLICABLE ONLY AT THIS LOCATION AND TIME. CONDITIONS MAY DIFFER AT OTHER LOCATIONS OR TIME.

Converse Consultants, Inc.
Earth Sciences Associates
Geo/Resource Consultants

BORING LOG 3-23

Proj: DESIGN UNIT A-100
Date Drilled 7-6-83
Ground Elev. ±260'
Drill Rig Rotary
Logged By J. Stellar
Total Depth 31.0'
Hole Diameter 4 3/4"
Hammer Weight & Fall 320 lb. @ 30'

DEPTH	USCS	MATERIAL CLASSIFICATION	SAMPLE	SPT (6")	DRILL MODE	REMARKS
0	SM SP	FILL-MIXED SAND, GRAVELLY SAND AND SILTY SAND: loose to medium dense, dry			RD	Drill with mud
2						
	SP	ALLUVIUM-SAND alternating with GRAVELLY SAND, brown, medium dense, slightly moist				
4						
				8	S	
				12	P	
6				50	T	
					RD	
8						
10			25k 12h C-1		DR	
					RD	
12						
14						
				12	S	
				24	P	
16			J-1	36	T	
					RD	
18						
20						

Sheet 1 of 2

Project DESIGN UNIT A-100 **Date Drilled** 7-6-83 **Hole No.** 3-23

DEPTH	USCS	MATERIAL CLASSIFICATION	SAMPLE	SPT (6")	DRILL MODE	REMARKS
20	SP	ALLUVIUM-SAND AND GRAVELLY SAND	60k 12		DR	No recovery @ 20'
					RD	
22						
24						
				24	S	
				36	P	
26			J-2	75	T	
					RD	
28		grades to silty				
30			40k 12" C-2		DR	
		end boring @ 31.0'				
32						
34						
36						
38						
40						
42						
44						

Sheet 2 **of** 2

THIS BORING LOG IS BASED ON FIELD CLASSIFICATION AND VISUAL SOIL DESCRIPTION, BUT IS MODIFIED TO INCLUDE RESULTS OF LABORATORY CLASSIFICATION TESTS WHERE AVAILABLE. THIS LOG IS APPLICABLE ONLY AT THIS LOCATION AND TIME. CONDITIONS MAY DIFFER AT OTHER LOCATIONS OR TIME.

Converse Consultants, Inc.
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BORING LOG 3-24

Proj: DESIGN UNIT A-100 **Date Drilled** 7-29-83 **Ground Elev.** 259'

Drill Rig FAILING 1500 **Logged By** L. Schoeberlein **Total Depth** 29.7'

Hole Diameter 4 7/8" **Hammer Weight & Fall** 140 lb @ 30"

DEPTH	USCS	MATERIAL CLASSIFICATION	SAMPLE	SPT (6")	DRILL MODE	REMARKS
0	SM/ML	FILL-SANDY SILT-SILTY SAND: dark yellowish brown to greenish black, interbedded of varying composition, dense, dry to moist			GB	
2			C-1	14 18	DR	1.0/1.0
					GB	
4			J-1	3 9 15	SS	1.5/1.5
					GB	
6			C-2	5 5	DR	0.8/1.0
					GB	
8			J-2	10 21 31	SS	1.0/1.5
					GB	intense chatter
	SP	YOUNG ALLUVIUM - GRAVELLY SAND: dark yellowish brown to salt and pepper; lenses of fine sand and medium to coarse sand; angular, granitic origin; contains fine to coarse gravel and cobbles throughout, angular, granitic, metamorphic and volcanic; very dense; moist to wet	C-3	32 37	DR	1.0/1.0 set 10' casing
10					RD	set up tub, mixed mud
12			J-3	28 20 34	SS	1.0/1.5
					RD	
14			C-4	21 40	DR	1.0/1.0
					RD	
16			J-4	24 68 86	SS	0.7/1.5
18					RD	
20			C-5	35 67	DR	0.9/1.0

Sheet 1 of 2

Project DESIGN UNIT A-100 Date Drilled 7-29-83 Hole No. 3-24

DEPTH	USCS	MATERIAL CLASSIFICATION	SAMPLE	SPT (6")	DRILL MODE	REMARKS
20	SP	YOUNG ALLUVIUM - GRAVELLY SAND			RD	
			J-5	36 70	SS	0.5/1.0
22					RD	
24		clean fine sand lens		62	DR	1.0/1.0
			C-6	74		
					RD	
26		grain size increasing to coarse sand	J-6	27 35 50	SS	0.6/1.5
28					RD	
				90	DR	0.8/0.9
			C-7	100 3"		
30		B. H. @ 29.7 Terminated Installed 29' of 2" ABS, bottom 20' slotted				completed drilling 10:45 4 sacks mud used moved hole 12' 9" to west of hub
32						
34						
36						
38						
40						
42						
44						

Sheet 2 of 2

Converse Consultants, Inc.
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BORING LOG 3-25

Proj: DESIGN UNIT A-100
Date Drilled 6-30-83
Ground Elev. ±261'
Drill Rig Rotary
Logged By J. Stellar
Total Depth 10
Hole Diameter 4 3/4"
Hammer Weight & Fall 320 lb. @ 30"

DEPTH	USCS	MATERIAL CLASSIFICATION	SAMPLE	SPT (6")	DRILL MODE	REMARKS
0	SM	FILL MIXED SAND AND GRAVELLY SAND: loose, dry				
2						
	SP	ALLUVIUM-SAND alternating with GRAVELLY SAND, brown, medium dense, slightly moist				
4						
				12	S	
				20	P	
6			J-1	26	T	
8			20k 12" C-1		DR	
				18	S	
				20	P	
			J-2	20	T	
10		end boring @ 10.0'				
12						
14						
16						
18						
20						

Sheet 1 of 1

Converse Consultants, Inc.
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BORING LOG 3-26

Proj: DESIGN UNIT A-100
Date Drilled 6-30-83
Ground Elev. ±261'
Drill Rig Rotary
Logged By J. Stellar
Total Depth 10'
Hole Diameter 4 3/4 "
Hammer Weight & Fall 320 lb. @ 30"

DEPTH	USCS	MATERIAL CLASSIFICATION	SAMPLE	SPT (6")	DRILL MODE	REMARKS
0	SM SP	FILL-MIXED SAND AND GRAVELLY SAND: loose, dry		RD		drilling with air
2	SP	ALLUVIUM-SAND alternating with GRAVELLY SAND, brown, medium dense, slightly moist				
4						
				14	S	
				16	P	
6			J-1	20	T	
					RD	
8			40k 12" C-1		DR	
				18	S	
		grades to dense		26	P	
10			J-2	45	T	
		end boring @ 10'				
12						
14						
16						
18						
20						

Sheet 1 of 1

Converse Consultants, Inc.
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Geo/Resource Consultants

BORING LOG 3-27

Proj: DESIGN UNIT A-100
Date Drilled 6-30-83
Ground Elev. ±259'
Drill Rig Rotary
Logged By J. Stellar
Total Depth 10.0'
Hole Diameter 4 3/4"
Hammer Weight & Fall 320 lb. @ 30"

DEPTH	USCS	MATERIAL CLASSIFICATION	SAMPLE	SPT (6")	DRILL MODE	REMARKS
0	SM SP	FILL-MIXED SAND AND GRAVELLY SAND: sandy silt, loose, dry			RD	drilling with air
2	SP ML	ALLUVIUM-SAND alternating with GRAVELLY SAND, and SANDY SILT, medium dense, slightly moist to moist				
4		sandy silt layer 4 to 6'				
			10k 12" C-1		DR	poor recovery @ 5' 2 rings only
6				2 4 12	SPT	
8		grades to sandy silt			PD	
			20k 12" C-2		DR	poor recovery @ 9' 3 rings only
10		end boring @ 10.0'				
12						
14						
16						
18						
20						

Sheet 1 of 1

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Converse Consultants, Inc.
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BORING LOG 3-28

Proj: DESIGN UNIT A-100
Date Drilled 6-30-83
Ground Elev. 257'
Drill Rig ROTARY
Logged By J. R. Stellar
Total Depth 10.0'
Hole Diameter 4 3/4"
Hammer Weight & Fall 320 lb @ 30"

DEPTH	USCS	MATERIAL CLASSIFICATION	SAMPLE	SPT (6")	DRILL MODE	REMARKS
0	SM/SP	FILL-MIXED GRAVELLY SAND AND SAND: dry, loose			RD	drilling with air
2	SP/SM	ALLUVIUM-SAND alternating with GRAVELLY SAND and SILTY SAND, brown, medium dense, slightly moist				
4			17K 12" C-1		DR	
				2 5 11	S P T	
6					RD	
8						
			40K 12" C-2		DR	
10		end boring @ 10.0'				
12						
14						
16						
18						
20						

Sheet 1 of 1

Converse Consultants, Inc.
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BORING LOG 3-29

Proj: DESIGN UNIT A-100
Date Drilled 6-30-83
Ground Elev. 256'
Drill Rig ROTARY
Logged By J. R. Stellar
Total Depth 10.0'
Hole Diameter 4 3/4"
Hammer Weight & Fall 320 lb @ 30"

DEPTH	USCS	MATERIAL CLASSIFICATION	SAMPLE	SPT (6")	DRILL MODE	REMARKS
0	SM/SP	FILL- MIXED SAND, GRAVELLY SAND AND SILTY SAND: loose, dry			RD	drilling with air
2	SP/SM	ALLUVIUM-SAND alternating with GRAVELLY SAND and SILT, brown, medium dense to firm, slightly moist				
4			23K 12" C-1		DR	
				2 4 6	S P T	
6		layer of SILT 6' to 8'			RD	
8						
			25K 12" C-2		DR	
10		end boring @ 10.0'				
12						
14						
16						
18						
20						

Sheet 1 of 1

Converse Consultants, Inc.
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BORING LOG 3-30

Proj: DESIGN UNIT A-100
Date Drilled 6-3-83
Ground Elev. 254'
Drill Rig ROTARY
Logged By J. R. Stellar
Total Depth 10.0'
Hole Diameter 4 3/4"
Hammer Weight & Fall 320 lb @ 30"

DEPTH	USCS	MATERIAL CLASSIFICATION	SAMPLE	SPT (6")	DRILL MODE	REMARKS
0	SM / SP	FILL-MIXED SAND, GRAVELLY SAND AND SILTY SAND: dry to slightly moist, loose			RD	drilling with air
2	SP / SM	ALLUVIUM-SAND alternating with SILTY SAND brown, medium dense, slightly moist				petroleum odor
4			40K 12" C-1		DR	
				4 4 6	S P T	
6		intermittent SILTY SAND layers				
8			38K 12" C-2		DR	
10		end boring @ 10.0'				rig mast broken @ 3:45 pm 6-30-83
12						
14						
16						
18						
20						

Sheet 1 of 1

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BORING LOG 3-31

Proj: DESIGN UNIT A-100
Date Drilled 9-23-83
Ground Elev. 260.4'
Drill Rig BUCKET
Logged By J. Doolittle
Total Depth 33'
Hole Diameter 36"
Hammer Weight & Fall

DEPTH	USCS	MATERIAL CLASSIFICATION	SAMPLE	SPT (6")	DRILL MODE	REMARKS
0	SM	FILL-SILTY SAND: dark brown with gravel to 1/2"				
	SP	SAND: fine, light brown, moist				
2						
4						
	ML/SM	SANDY SILT AND VERY FINE SILTY SAND: moist to very moist				
6						
8	SP	SAND: fine to medium, light brown, moist with gravel to 2"				
10		trace gravel to 1"				
12		SAND: medium to coarse, light brown, with gravel to 2"				
14		gravel to 1"				
						cobbles and boulders @ 15', difficult drilling and caving
16						
18		layer of SILT				switched to 24" rod bucket, took out 12" boulder
		SAND: medium to coarse, light brown, with gravel to 1"				
20						

Sheet 1 of 2

Project DESIGN UNIT A-100 Date Drilled 9-23-83 Hole No. 3-31

DEPTH	USCS	MATERIAL CLASSIFICATION	SAMPLE	SPT (6")	DRILL MODE	REMARKS
20	SP	SAND: medium to coarse, light brown, gravel and cobbles to 6"				switched back to 30" bucket
22						
24		SAND: fine to medium, light brown				
26		SAND: medium to coarse, trace gravel				difficult drilling hit cobble/boulder
		SAND: fine to medium, trace gravel and silt lens				
28						
30						
32						
		end of boring @ 33'				
34						
36						
38						
40						
42						
44						

Sheet 2 of 2

Converse Consultants, Inc.
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BORING LOG 3-32

Proj: DESIGN UNIT A-100
Date Drilled 9-23-83
Ground Elev. 261.5'
Drill Rig BUCKET
Logged By J. Doolittle
Total Depth 20'
Hole Diameter 36"
Hammer Weight & Fall

DEPTH	USCS	MATERIAL CLASSIFICATION	SAMPLE	SPT (6")	DRILL MODE	REMARKS
0	SM	FILL-SILTY SAND: fine to medium, very dry, with gravel to 3/4"				
	SP	SAND: fine, light brown, trace gravel, very dry				stratification in deposits with 12" drop across hole
2		slightly moist				
4		moist				
6		SAND: fine to medium, light brown, moist				
8						caving from 7'
10		SAND: fine to medium, brown, moist				caving 16" boulder at 11' had to put on rock bucket
12		SAND: fine to medium, brown, moist, with gravel to 2"				caving
		SAND: fine to medium, light brown, moist, gravel with occasional cobble/boulder				
14						boulder at 14-15'
16						caving
18						
20		end of boring @ 20'				

Sheet 1 of 1

Converse Consultants, Inc.
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BORING LOG 3-33

Proj: DESIGN UNIT A-100
Date Drilled 9-23-83
Ground Elev. 365.7'
Drill Rig BUCKET
Logged By J. Doolittle
Total Depth 35.0'
Hole Diameter 36"
Hammer Weight & Fall NA

DEPTH	USCS	MATERIAL CLASSIFICATION	SAMPLE	SPT 6"	DRILL MODE	REMARKS
0	SM	FILL 3" A.C. SILTY SAND: lt. to red brn. w/gravel SILTY SAND: drk. brn. w/gravel SAND: light brown SILTY SAND: brown, fine SILT: brown to drk. brown w/ SAND SAND: fine to medium to coarse, lt. brown to gray 10% gravel to 1" SAND: v. fine, micaceous				
						Bulk Sample #1 5'-7"
6	SP	SAND: fine to medium. gravel to 1"				
						caving and belling 9' to 17'
10	SP	SAND: medium to coarse, lt. brown with gravels and cobbles				
						Bulk Sample #2 13' to 14'
18		grades to fine to medium sand with gravel to 2" SAND: fine to medium to coarse, lt. brn. w/gravels & cobbles				
20						

Sheet 1 of 2

Project DESIGN UNIT A-100 Date Drilled 9-23-83 Hole No. 3-33

DEPTH	USCS	MATERIAL CLASSIFICATION	SAMPLE	SPT (6")	DRILL MODE	REMARKS
20	SP	SAND: medium to coarse, gravels and cobbles				
22						
24						
26	SP	SAND: medium to coarse with gravels and cobbles, light brown with scattered olive black (gravel)				
28						
30	SP	SAND: medium to coarse, light brown with scattered olive black gravel to 3"				
32						
34						
		End boring at 35'				
36						
38						
40						
42						
44						

Sheet 2 of 2

THIS BORING LOG IS BASED ON FIELD CLASSIFICATION AND VISUAL SOIL DESCRIPTION, BUT IS MODIFIED TO INCLUDE RESULTS OF LABORATORY CLASSIFICATION TESTS WHERE AVAILABLE. THIS LOG IS APPLICABLE ONLY AT THIS LOCATION AND TIME. CONDITIONS MAY DIFFER AT OTHER LOCATIONS OR TIME.

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BORING LOG 3

Proj: DESIGN UNIT A-100
Date Drilled 3-9/2-81
Ground Elev. 281'
Drill Rig FAILING 1500
Logged By S. Slaff
Total Depth 150.6'
Hole Diameter 4 7/8"
Hammer Weight & Fall 140 lb @ 30"

DEPTH	USCS	MATERIAL CLASSIFICATION	SAMPLE	SPT (6")	DRILL MODE	REMARKS
0	SM	BRICK AND MORTAR PAVEMENT COBBLESTONE PAVEMENT FILL SILTY SAND: dark yellowish brown (10YR4/2), 60% sand, 35% low plasticity fines, 5% gravel, moist, medium dense, sand is subangular, gravel is well rounded brick, metamorphic rocks, and quartz		AD		10:30, 2-3-81 began drilling 0.0-0.8' drilled with "garbage barrel" (cylindrical bit). 0.8-3.5' drilled with 6" flight auger set 6" steel surface casing to 3.5' with 1.5' stick-up 3.5-10.0' drilled with new 4 7/8" tri-cone tooth bit drilling with 230 psi hydraulic draw-down
2						
4				RD		
6	SP	GRAVELLY SAND: mottled, yellowish gray, saturated, medium dense				
8		?				rig chattering
10				17 17 18	SS	0.0 recovery
12					RD	rig chattering
14						
16						rig chattering
18						11:45
20						

Sheet 1 of 7

Project DESIGN UNIT A-100 Date Drilled 2-3-81 Hole No. 3

DEPTH	USCS	MATERIAL CLASSIFICATION	SAMPLE	SPT (6")	DRILL MODE	REMARKS
20	SP	GRAVELLY SAND: continued	C-1 T		RD	DR blowcount: 58,100 .75/1.0 recovery
			J-1	29 36 51		350 lb slip jars, 24" drop, used for drive samples 1.5/1.5 recovery
22	SP SM	SILTY SAND: light olive gray, fine sand, low plasticity, moist, dense, sand is subangular			RD	rig chattering
24						
26						
28						12:30
30			J-2	23 33 40	SS	1.0/1.5 recovery
32					RD	12:45
34						
36						drill rate = 1.25/min.
38						13:00
	SM	SILTY SAND: medium dark gray, fine to medium grained sand, moderate plasticity fines, moist, very dense, subrounded med. grained sand, some rounded granitic rock-derived metamorphic cobbles	C-2		DR	DR blowcount: 100 .5/.5 recovery
				50+	SS	then refusal, intense rig chattering, refusal-no sample 0.0/0.5 recovery
40					RD	
42						intense rig chattering (metamorphic cobbles and boulders)
		gravel lens				
44						

Project DESIGN UNIT A-100 Date Drilled 2-3-81 Hole No. 3

DEPTH	USCS	MATERIAL CLASSIFICATION	SAMPLE	SPT (6")	DRILL MODE	REMARKS
44	SM	<u>SILTY SAND</u>: continued			RD	drilling with 100 psi hydraulic draw-down
46						
48						14:25
50			J-3	38 37 22	SS	1.3/1.5 recovery metamorphic boulder at 44.2' caused problems running back into hole after taking J-3 sample
					RD	
52		gravel lens				
54		gravel lens				rig chattering
56						
		gravel content increases to 10% of formation				
58						DR blow count: 8,33 0.3/1.0 recovery
			C-3 T		DR	
60			J-4	32 50	SS	0.5/1.0' recovery 16:10
					RD	
62						
64						
66						
68						

Sheet 3 of 7

Project DESIGN UNIT A-100 **Date Drilled** 2/3-4/81 **Hole No.** 3

DEPTH	USCS	MATERIAL CLASSIFICATION	SAMPLE	SPT (6")	DRILL MODE	REMARKS
68	SM	SILTY SAND: continued			RD	
70		gravel content increases to 20% of formation	J-5	42 50	SS	14:40 .75/.75 recovery refusal at 70.75'
					RD	
72						
74						
76						
78		grading to fine and medium grained sand				17:00 2-3-81
			C-4		DR	07:45 2-4-81
			J-6	50	SS	.25/.25 recovery DR blowcount: 100 cleaned out hole before sampling. obstruction at 41.2" gone now.
80					RD	
82		gravel lens				refusal on both samples rig chattering violently drilling with 150 psi hydraulic drawdown mixed 1 sack bentonite
84						
86		gravel lens				rig chattering
88						08:30
	CL/SC	PUENTE FORMATION-CLAYSTONE: grayish olive green, high plasticity fines, fine sand, very stiff, moist, sticky, sand is sub-angular, sulfurous-organic odor, micaceous, poorly developed, fissility				
90			J-7	26 30 48	SS	1.5/1.5 recovery unconfined strength >4.5 tons/ft²
92					RD	

Sheet 4 of 7

Project DESIGN UNIT A-100 **Date Drilled** 2-4-81 **Hole No.** 3

DEPTH	USCS	MATERIAL CLASSIFICATION	SAMPLE	SPT (6")	DRILL MODE	REMARKS
92		PUENTE FORMATION CLAYSTONE: Physical Condition: massive, soft to friable hardness, soft to friable strength, fresh			RD	
94						
96						
98						
100			box #1	PB		0.4/2.8 recovery
102						
			S-1			2.5.2.7 recovery
104						unconfined strength >4.5 tons/ft^2 10:00
106		no longer any odor banded with grayish olive green alternating with greenish black				2.8/2.8 recovery adding water drill rate = 1/4'/minute
108						1.9/2.7 recovery
110		massive structure				
						2.8/2.8 recovery
112						11:20
114						2.7/2.7 recovery
116			box #2			

Sheet 5 of 7

Project DESIGN UNIT A-100 Date Drilled 2/4-9/81 Hole No. 3

DEPTH	USCS	MATERIAL CLASSIFICATION	SAMPLE	SPT (6")	DRILL MODE	REMARKS
116		PUENTE FORMATION CLAYSTONE: continues Physical Condition: continues as previously described	box #2		PB	2.7/2.7 recovery 2.7/2.8 recovery 11:45
118						2.7/2.7 recovery
120						
122					RD	2-9-81 groundwater deeper than 40'. Hole caved in from 40'-88.8'. Drilled out with 50-200 psi hydraulic draw-down 09:45-10:20 mixed 2 sacks bentonite, small amount synthetic polymer. Cave-in ended at bedrock. Small bridge at 100.3' mixed 1 more sack bentonite. Drilled to 123.0'. 11:30
			S-2		PB	
124						
126		moderately well-developed fissility, cleavage planes are dark greenish gray to greenish gray	box #2			
128		yellowish gray silt blebs .1-.3" long comprise 1-2% of sample	box #3			2.8/2.8 recovery run #9
130						2.2/2.7 recovery run #10 drilling with 250 psi hydraulic draw-down
132						2.7/2.7 recovery disturbed by bent Pitcher barrel
		poorly-developed bedding				
134						2.8/2.8 recovery disturbed by bent Pitcher barrel drill rate = .33'/min.
136						2.3/2.8 recovery
138			box #4			13:30 2.3/2.8 recovery
140			S-3		15	

Sheet 6 of 7

Project DESIGN UNIT A-100 **Date Drilled** 2-9-81 **Hole No.** 3

DEPTH	USCS	MATERIAL CLASSIFICATION	SAMPLE	SPT (6")	DRILL MODE	REMARKS
140		PUENTE FORMATION <u>CLAYSTONE</u> continues as previously described	S-3		PB	2.3/2.8 recovery
142			box #4			2.25/2.7 recovery drill rate= 1 ft./ 5 minutes
144						0.3/2.8 recovery
146						
148			S-4			2.8/2.8 recovery
150						14:45 2-9-81
		end boring @ 150.6'				ran electric logs 15:00-17:00 Set 150' of 2" dia. PVC casing, perforated from 105-150'
152						
154						07:15 2-10-81 0% combustible gas groundwater at 31.8' below surface. water sampled 2-19-81
156						
158						
160						
162						
164						

Sheet 7 of 7

THIS BORING LOG IS BASED ON FIELD CLASSIFICATION AND VISUAL SOIL DESCRIPTION, BUT IS MODIFIED TO INCLUDE RESULTS OF LABORATORY CLASSIFICATION TESTS WHERE AVAILABLE. THIS LOG IS APPLICABLE ONLY AT THIS LOCATION AND TIME. CONDITIONS MAY DIFFER AT OTHER LOCATIONS OR TIME.

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BORING LOG 4

Proj: DESIGN UNIT A-100
Date Drilled 2-9-81
Ground Elev. 279'
Drill Rig FAILING 1500
Logged By Stephen M. Testa
Total Depth 150.0'
Hole Diameter
Hammer Weight & Fall 140 lb @ 30"

DEPTH	USCS	MATERIAL CLASSIFICATION	SAMPLE	SPT (6")	DRILL MODE	REMARKS
0		CONCRETE FILL CRUSHED ASPHALT AND BRICK DEBRIS			AD	started drilling at 8:45, cloudy day, light drizzle, down to 10.0'
2						
4						
6						
8						
10					RD	
12						
14	SP	GRAVELLY SAND: olive black, non-plastic, fine to medium sand, gravel up to 2.0 mm in. max. diameter, wet, very dense, poorly graded, odorous (oil)				
				19	SS	
16			J-1	30		1.0/1.5 recovery
				50		
					RD	
18						
20						

Sheet 1 of 7

Project DESIGN UNIT A-100 **Date Drilled** 2-9-81 **Hole No.** 4

DEPTH	USCS	MATERIAL CLASSIFICATION	SAMPLE	SPT (6")	DRILL MODE	REMARKS
20	SP	GRAVELLY SAND: continued		50	DR	(continued) Converse sample at 20.0', no recovery, SPT at 20.5', no recovery
				50/5"	SS	
	GP	SANDY GRAVEL: numerous cobbles and boulders				moderate to heavy rod chatter (episodic) from 21.0-30.0'
22						
24						
26						
28						
30				50/4"	SS	SPT @ 30.0', no recovery
					RD	caving is preventing advancement of hole, added two sacks of bentonite, cuttings sampled at 32.0'
32		gravels range up to 6.0 mm in maximum diameter, consist primarily of granites and metamorphics, subangular to subrounded, poorly graded	J-2			
34						
36						
38						
40				50/3"	SS	SPT at 40.0', no recovery, much difficulty getting back into hole
					RD	
42						
44						

Sheet 2 of 7

Project DESIGN UNIT A-100 **Date Drilled** 2/9/81-2-10-81 **Hole No.** 4

DEPTH	USCS	MATERIAL CLASSIFICATION	SAMPLE	SPT (6")	DRILL MODE	REMARKS
44	GP	SANDY GRAVEL: continued numerous cobbles			RD	(Continued) moderate to heavy rig shaking continues
46						
48						
50						due to heavy rig shaking, no SPT was taken at 50.0'
52						
54						
56						
58						
60		continued, numerous cobbles		50/3"	SS RD	SPT at 60.0', no recovery stopped drilling at a depth of 70.0'
62						2-10-81 resumed drilling at 7:00 am, clear day
64						
66						
68						

Project DESIGN UNIT A-100 **Date Drilled** 2-10-81 **Hole No.** 4

DEPTH	USCS	MATERIAL CLASSIFICATION	SAMPLE	SPT (6")	DRILL MODE	REMARKS
68	GP	SANDY GRAVEL: continued cobbles and possibly boulders			RD	(continued) moderate to heavy rig shaking continues
70				50/2"	SS	SPT at 70.0', no recovery, much difficulty getting back into hole
72						
74						
76						
78						
		cobbles and boulders				
80				50/4"	SS RD	SPT at 80.0', no recovery
82						
84						
86						
88						
90						
92						

Project DESIGN UNIT A-100 Date Drilled 2-10-81 Hole No. 4

DEPTH	USCS	MATERIAL CLASSIFICATION	SAMPLE	SPT (6")	DRILL MODE	REMARKS
92	GP	SANDY GRAVEL: continued fine to coarse sand with numerous cobbles and boulders			RD	(Continued) moderate to heavy rig to 101.5'
94						
96						
98						
100						
102		PUENTE FORMATION CLAYSTONE: olive gray, micaceous claystone, very fine sand, compositional banding apparent. Physical Conditions: massive, soft to friable hardness, plastic to weak strength, fresh, tends to fracture along bedding planes				gas check 0.0% LEL no gas encountered,
					PB	continuous Pitcher barrel sampling from 102.5', had to ream hole out to 7.0" with tri-cone bit from 12:45 to 1:20 pm in order to pitcher barrel, samples are at 2.8' intervals
104						
106			box #1			
108		from 107.8, very thin to medium compositional banding, contacts sharp and parallel, hairline fractures at 108.4', dipping 80° to core axis with an oriented strike 70° to bedding				200 psi pocket penetrometer >4.5 kg/cm²
110						
112		primarily claystone exhibiting compositional banding, at 112.1' clay filled fracture with undeterminable offset				200 psi
114		continued, very thin to medium compositional banding evident in claystone (depositional features)	box #2			200 psi
116						

Project DESIGN UNIT A-100 **Date Drilled** 2-10-92/2-11-81 **Hole No.** 4

DEPTH	USCS	MATERIAL CLASSIFICATION	SAMPLE	SPT (6")	DRILL MODE	REMARKS
116		PUENTE FORMATION CLAYSTONE: continued Physical Conditions: cont. massive, soft to friable hardness, plastic to weak strength, fresh, tends to fracture along bedding planes, notably sandstone laminae from 119.7' alternating claystone and very thin grayish brown fine sandstone laminae sandstone (5%) laminae, 119.3-119.9' intensely fractured (orientation undeterminable)	box #2		PB	(Continued) 200 psi
			S-1			
118						200 psi
120						
122						200 psi
124		continued, primarily claystone (80-85%) fine sandstone (10-15%)	box #3			stopped drilling at a depth of 124.9' @ 5:00 2-11-81 resumed drilling @ 7:00 am, clear day
126						
128						200 psi
130		continued, compositional banding apparent in claystone (bluish green clay)	box #4			200 psi pocket penetrometer >4.5 kg/cm²
132						200 psi
134			S-2			
136						200 psi
138			box #4			
140						200 psi

Project DESIGN UNIT A-100 **Date Drilled** 2-11-81 **Hole No.** 4

DEPTH	USCS	MATERIAL CLASSIFICATION	SAMPLE	SPT (6")	DRILL MODE	REMARKS
140		PUENTE FORMATION CLAYSTONE: continued well cemented, very fine greenish gray sandstone Physical Condition: continues as previously described	box #4		PB	(Continued) 200 psi
142						pocket penetrometer >4.5 kg/cm^2
144		color change to grayish brown, fine to medium sand, partially saturated with oil from 145.3', primarily mottled claystone (70%), oil saturated, fine to medium sand (30%)	box #5			petroleum sample 145-146'
146						
148			S-3			
150		end boring at 150'				terminated hole at a depth of 150.0' at 11:20 am E-log conducted from 11:30-2:00 pm Installed 150.0' of 2" PVC piezometer (PVC slotted from 110.0' to 145') and backfilled hole with pea gravel. 5' bentonite plug at surface water sampled 2/19/81
152						
154						
156						
158						
160						
162						
164						

Sheet 7 of 7

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BORING LOG 5

Proj: DESIGN UNIT A-100
Date Drilled 1-2-4-81
Ground Elev. 280'
Drill Rig Failing 1500
Logged By S. M. Testa
Total Depth 150.0'
Hole Diameter 6 7/8", 4 7/8"
Hammer Weight & Fall 140 lb @ 30"

DEPTH	USCS	MATERIAL CLASSIFICATION	SAMPLE	SPT (6")	DRILL MODE	REMARKS
0		ASPHALTIC CONCRETE PAVEMENT			RD	started drilling @ 9AM
	GP	SANDY GRAVEL with COBBLES: medium to coarse sand, gravel and cobbles ranging to 6"				moderate to heavy rig chatter from 3.0'; did not begin sampling schedule until 52.0' due to the cobbly and gravelly nature of the material
2						
4						
6						
8						
10						
12						
14						
16						
18						
20						

Sheet 1 of 7

Project DESIGN UNIT A-100 Date Drilled 1-2-81 Hole No. 5

DEPTH	USCS	MATERIAL CLASSIFICATION	SAMPLE	SPT (6")	DRILL MODE	REMARKS
20	GP	SANDY GRAVEL with COBBLES: cont.			RD	moderate to heavy rig chatter
22						
24						
26						
28						
30						
32						
34						
36						
38						
40						
42						
44						

Sheet 2 of 7

Project DESIGN UNIT A-100 **Date Drilled** 1-2-31 **Hole No.** 5

DEPTH	USCS	MATERIAL CLASSIFICATION	SAMPLE	SPT (6")	DRILL MODE	REMARKS
44	GP	SANDY GRAVEL with COBBLES: cont.			RD	heavy shaking to 52.0'
46						
48						
50						
52	SP	GRAVELLY SAND: dense; medium dark grey; medium to coarse sand; gravel up to 1/2"; primarily granitics	J-1	47 50/4"	SS	.5'/1.5' recovery
					RD	easy drilling from 53.0'-55.0'; heavy rig shaking from 55.0 to 60.0'
54						
	GP	SANDY GRAVEL with COBBLES:				
56						
58						
60				40 50/2"	SS	no recovery
					RD	heavy rig shaking from 60.6' to 66.0'
62						
64						
66			J-2	8 20 50/4"	SS	.3'/1.5' recovery
					RD	
68						

Project DESIGN UNIT A-100 Date Drilled 1-2-3-81 Hole No. 5

DEPTH	USCS	MATERIAL CLASSIFICATION	SAMPLE	SPT (6")	DRILL MODE	REMARKS
68	GP	SANDY GRAVEL with COBBLES: cont.			RD	
70				50 50/1"	DR	no recovery
					RD	moderate to heavy rig shaking from 70.6' to 84.2'
72						
74						heavy rig shaking at 75.0'
76						
78						
80						heavy rig shaking at 80.0'
82						
84	CL/ SC 43° ←40°	PUENTE FORMATION - CLAYSTONE: wavy, parallel bedding; primarily olive grey claystone w/alt. very thin to thin lamina of dusky yellowish brown siltier claystone and light grey friable sandstone; micaceous; claystone fossiliferous; opp. dip orientation apparent in PB-1 Physical Conditions: soft to friable hardness; plastic to friable strength; fresh; tends to fracture along bedding planes, notably friable sandstone beds and lamina. sandstone blebs apparent	Box #1		PB 1	easy drilling from 84.2'
86	47°				PB 2	
88	50°				PB 3	
90						
92					PB 4	

Sheet 4 of 7

Project DESIGN UNIT A-100 Date Drilled 1-3-81 Hole No. 5

DEPTH	USCS	MATERIAL CLASSIFICATION	SAMPLE	SPT (6")	DRILL MODE	REMARKS
92	52° CL/SC	PUENTE FORMATION - CLAYSTONE: cont. Physical Conditions: cont. folding of lamina apparent at 92.0'; primarily claystone from 93.5'-100.4', some faintly discernable very thin discontinuous sandstone lamina	Box #1		PB 4	pocket penetrometer >4.5 tsf
94					PB 5	
96	40°	thin sandstone lamina at 96.6, 97.1	Box #2		PB 6	
98	20-30°				PB 7	pocket penetrometer >4.5 tsf
100	51°	alt. very thin to med. lamina of claystone, sandstone and silty claystone; some lamina truncated, x-bedding(upright)			PB 8	
102	40°	repeating sequences of v. thin to thin sandstone, claystone and silty claystone; organics (trace)			PB 9	
104						
106	43°	primarily claystone; wavy discontinuous sandstone, dark green wavy stringers(up to 2cm) from 108.0; preferred orientation dipping about 43°			PB 10	pocket penetrometer >4.5 tsf
108	43°	primarily claystone, siltier claystone rounded concretions apparent, randomly distributed from 108.5-112.5	Box #3		PB 11	
110		primarily claystone w/occasional discontinuous wavy sandstone up to 4cm in max. thickness from 111.0 -116.0; variable bedding 5-40°			PB 12	
112						gas checked; no gas encountered
114		sandstone lamina occasionally discontinuous			PB 13	
116						

Sheet 5 of 7

Project DESIGN UNIT A-100 Date Drilled 1-3-81 Hole No. 5

DEPTH	USCS	MATERIAL CLASSIFICATION	SAMPLE	SPT (6")	DRILL MODE	REMARKS
116	44° CL/SC	PUENTE FORMATION - CLAYSTONE: cont. primarily claystone with alt. wavy, parallel v.thin to med. lamina of sandstone, siltier claystone w/minor siltstone; micaceous; fossiliferous claystone Physical Conditions: cont. occasional discontinuous , nonparallel lamina; folding apparent, depth 119.0'	Box #3		PB 14	
118	48°				PB 15	
120		bedding planes sharp, dip of bedding planes variable(10-46°) folding apparent	Box #4		PB 16	
122	46°				PB 17	
124		folding apparent from 125-126.0'				
126	46°	sharp contorts between alt. lamina; cross-bedding evident(seq. is upright)			PB 18	
128	46°	wavy parallel alt. lamina of sandstone, claystone & silty claystone; dark yellowish brown siltstone at 129.6'; graded bedding apparent in v.fine to fine grained sandstone lamina @ 129.7 to 130.0'			PB 19	pocket penetrometer >4.5 tsf
130		alt. lamina of sandstone, claystone, silty claystone and siltstone; clay filled fracture hairline fractures w/offsets apparent at 132.0'; prominent fracture plane dips 47°.			PB 20	
132					PB 21	
134	46°	some folding evident in sandstone lamina at 134.5'				
136	42°	cross-bedding apparent (seq. upright) Physical Conditions: soft to friable hardness; plastic to friable strength; fresh; tends to fracture along bedding planes, notably friable sandstone lamina	Box #5		PB 22	
138					PB 23	
140						

Sheet 6 of 7

Project DESIGN UNIT A-100 **Date Drilled** 1-3-81 **Hole No.** 5

DEPTH	USCS	MATERIAL CLASSIFICATION	SAMPLE	SPT (6")	DRILL MODE	REMARKS
140	CL/SC 44°	PUENTE FORMATION - CLAYSTONE: cont. wavy, parallel v.thin to med. alt. lamina of claystone, subordinate sandstone and siltier claystone, and minor siltstone; variable thickness of lamina; cross-bedding apparent (seq. is upright)	Box #5		PB 23	
142					PB 24	
		primarily claystone to 142.3'; very fine grained sandstone from 142.3 to 142.8, 143.1 to 145.3				
144		alt. wavy parallel lamina; sandstone from 144.5-144.7, 145.1-145.6, 145.9-146.2			PB 25	
146					PB 26	
148	44°	primarily claystone from 147.3 with fine to coarse grained sandstone at 147.3-147.6, 148.0-148.4, 148.6-148.9 and 149.1-149.3				5:30 1-3-81
150		end boring @ 150.0'				Installed 100.0' of 4" PVC and grouted
152						
154						
156						
158						
160						
162						
164						

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BORING LOG 5-4

Proj: DESIGN UNIT A-100 **Date Drilled** 1-3-81 **Ground Elev.** 280.6'

Drill Rig Mayhew 1000 **Logged By** B. Ingram **Total Depth** 80.0'

Hole Diameter 4 3/4" **Hammer Weight & Fall** 320 lbs @ 36"

DEPTH	USCS	MATERIAL CLASSIFICATION	SAMPLE	SPT (6")	DRILL MODE	REMARKS
0		ASPHALT PAVEMENT				
	ML	SANDY SILT: dark brown, moist, stiff & brick chunks			RD	begin drilling 8:15 am weather: clear, warm
			C-1 16K		DR	
2					RD	
		concrete block at 3.5'				
		FILL				
4	SM	YOUNG ALLUVIUM-SILTY SAND: gray-brown, moist, dense, fine to very fine				
			J-1	30 18 24	S P T	
6					RD	
	SP	SAND: gray, moist, dense, clean, uniform fine sand				
8						
10			C-2 17K		DR	
					RD	
12		becomes coarser, to medium grained				
14						
			J-2	5 4 11	S P T	
16					RD	
18	SW	GRAVELLY SAND: brown/gray/white, moist, dense, medium to coarse clean sand, gravel to 2", subrounded to subangular, granitic comp.				
20						

Sheet 1 of 4

Project DESIGN UNIT A-100 Date Drilled 1-31-83 Hole No. 5-4

DEPTH	USCS	MATERIAL CLASSIFICATION	SAMPLE	SPT (6")	DRILL MODE	REMARKS
20	SW	GRAVELLY SAND: Continued	C -3 75K		DR	highly disturbed sample due to large gravel pieces in sample barrel
					RD	
22						
24						
			5"	100	SPT	refusal
					RD	
26						
28	SP/CL	SAND/SILTY CLAY: gray, moist, dense, clean sand, interbeds of dark gray, stiff, moist, silty clay to 31.5'				
30			C-4 55K		DR	
					RD	
	SP	end clay interbeds				
32						
34						
			J-3	67 55 80	S P T	
					RD	
36						
38						
40			30K 0"			no sample recovery refusal
	SW	GRAVELLY SAND: dark gray, very dense subrounded grains, well graded, very fine to coarse sand & gravel, highly micaceous, dioritic composition				
42						
44						

Sheet 2 of 4

Project DESIGN UNIT A-100 Date Drilled 1-31-83 Hole No. 5-4

DEPTH	USCS	MATERIAL CLASSIFICATION	SAMPLE	SPT (6")	DRILL MODE	REMARKS
44	SW	GRAVELLY SAND: Continued			RD	
				75	S	
				78	P	
46			5"-	100	T	
48						
50		clay layer, decreasing gravel content	C-5 70K		DR	4 rings only sulphurous odor in sample
					RD	
52						
54						
			3"	100	SPT	sulphurous odor in refusal
					RD	
56						
58						
60			C-6 76K/9"		DR	sulphur/hydrothermal odor
					RD	
62						
64						
				105	SpT	sulphur odor
66			1"-	50		
					RD	
68						

Sheet 3 of 4

Project DESIGN UNIT A-100 Date Drilled 1-31-83 Hole No. 5-4

DEPTH	USCS	MATERIAL CLASSIFICATION	SAMPLE	SPT (6")	DRILL MODE	REMARKS
68	SP	**SAND**: gray, dense, uniform, micaceous, fine to very fine grained			RD	
70			C -7 25K		DR	sulphur/hydrothermal odor in sample
					RD	
72						
74						
	SW	**GRAVELLY SAND**: gray, dense to v. dense, up to 1½"				
76						
78						
			C-8 75K		DR	stop drilling 2:30 pm
80		end boring 80.0'				
82						
84						
86						
88						
90						
92						

Sheet 4 of 4

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BORING LOG 5-5

Proj: DESIGN UNIT A-100
Date Drilled 2-1-83
Ground Elev. 280.8'
Drill Rig Mayhew 1000
Logged By B. Ingram
Total Depth 100.0'
Hole Diameter 4 3/4"
Hammer Weight & Fall 320 lb @ 36"

DEPTH	USCS	MATERIAL CLASSIFICATION	SAMPLE	SPT (6")	DRILL MODE	REMARKS
0		FILL-ASPHALT PAVEMENT			RD	set up 2:45 pm, 1-31 begin drilling 7:15 am 2-1-83 weather: clear, warm
	ML & SM	SANDY SILT & SILTY SAND: mottled & intermixed, moist, stiff to medium dense, with brick debris		16 25 57	S P T	
2						
					RD	drilled to 5' with 7" bit for piezo installation, 4 3/4" bit below
4						
			C -1 3K		DR	contact contained within sample
6	SM	YOUNG ALLUVIUM-SILTY SAND: gray brown. moist, medium dense, fine to very fine sand-70%, 30% silt				
8						
10				10 12 25	S P T	
	SP	SAND: gray brown, moist, medium dense to dense, poorly graded fine sand, with trace silt			RD	
12						
14	SW	GRAVELLY SAND: brown, dense, well graded, medium to coarse clean sand, gravel sub-angular to sub-rounded grains, granitic composition				
			C-2 50K		DR	gravelly-disturbed sample
16					RD	
18						
20						

Sheet 1 of 5

Project DESIGN UNIT A-100 Date Drilled 2-1-83 Hole No. 5-5

DEPTH	USCS	MATERIAL CLASSIFICATION	SAMPLE	SPT (6")	DRILL MODE	REMARKS
20	SW	<u>GRAVELLY SAND</u>: Continued	5"	100	SPT	refusal
					RD	
22						
24						
		occasional lens of fine sand	C -3 36K		DR	
26					RD	
28						
30		color change to dark gray	3"	75 100	SpT	slight odor of gas refusal
					RD	
32						
34						
			C-4 40K		DR	slight gas odor gravelly sample-- only 5 good rings
36					RD	
38						
40			J-1 2"	25 75 100	S P T	refusal
					RD	
42						
44						

Sheet 2 of 5

Project DESIGN UNIT A- 100 Date Drilled 2-1-83 Hole No. 5-5

DEPTH	USCS	MATERIAL CLASSIFICATION	SAMPLE	SPT (6")	DRILL MODE	REMARKS
44	SW	GRAVELLY SAND: Continued			RD	
46						
48						
50			75K		DR RD	sulphur odor poor sample recovery remaining sample destroyed in handling
52						
54		gradually decreasing gravel content				
56						
58						
60			C-5 50K		DR RD	slight sulphur odor
62	SP	SAND: dark gray, dense, poorly graded, uniform, fine to very fine grained, micaceous				
64						
66						
68						

Sheet 3 of 5

Project DESIGN UNIT A- 100 **Date Drilled** 2-1-83 **Hole No.** 5-5

DEPTH	USCS	MATERIAL CLASSIFICATION	SAMPLE	SPT (6")	DRILL MODE	REMARKS
68	SP	SAND: Continued			RD	
70			C -6		DR	stronger sulphur odor poor sample recovery
					RD	
72						
74						
76						
78	GP	BOULDERS AND GRAVEL: medium to coarse, subangular grains, granitic composition from cuttings				78.0'--change in drilling conditions, very hard drilling full weight of rig (10 tons) on bit, too hard to sample to 90.0' refusal
80			60K-1"		DR RD	no sample recovery
82						
84			100K-1"			refusal--attempted to sample--unsuccessful
					RD	
86						
			J-2			jar sample of cuttings taken
88						
90	CL	PUENTE FORMATION-CLAYSTONE (see next page)				easy drilling from 90.0', claystone cuttings obtained
92						

Sheet 4 **of** 5

Project DESIGN UNIT A-100 Date Drilled 2-1-83 Hole No. 5-5

DEPTH	USCS	MATERIAL CLASSIFICATION	SAMPLE	SPT (6")	DRILL MODE	REMARKS
92		PUENTE FORMATION-CLAYSTONE: Continued olive gray color, moist, plastic to friable strength, soft friable hardness, thinly laminated with SILTY CLAYSTONE, SANDSTONE blebs tends to fracture along laminations			RD	
94						
96						
98			C-7 50K		DR	
100		end boring 100.0' piezometer set to 100' perforated in lowest 40'				
102						
104						
106						
108						
110						
112						
114						
116						

Sheet 5 of 5

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BORING LOG B-10

Proj: DESIGN UNIT A-100
Date Drilled 6-22-84
Ground Elev. ±279'
Drill Rig Rotary
Logged By J. Stellar
Total Depth 107'
Hole Diameter 4 3/4"
Hammer Weight & Fall 320 lb. @ 36"

DEPTH	USCS	MATERIAL CLASSIFICATION	SAMPLE	SPT (6")	DRILL MODE	REMARKS
0	ML SM	FILL-MIXED SANDY SILT AND SILTY SAND: moist, firm, with brick debris			RD	
2						
4						
			18k 12" B10-1		DR	
6					RD	
8						
	SW	ALLUVIUM-GRAVELLY SAND WITH FINE SAND: interlayers, gray brown, fine to medium grained sand and gravel				
10				12	S	
				18	P	
				21	T	
					RD	
12						
14						
		petroleum odor gravel to 3/4"	50k 12" B10-2		DR	poor recovery 2 rings only
16					RD	
		increase in % gravel @ 17'				rig chatter @ 17'
18						
20						

Sheet 1 of 5

Project DESIGN UNIT A-100 **Date Drilled** 6-22-83 **Hole No.** B10

DEPTH	USCS	MATERIAL CLASSIFICATION	SAMPLE	SPT (6")	DRILL MODE	REMARKS
20	SW	<u>GRAVELLY SAND</u>: change in color to blue grey		6 30 50	S P T	cuttings in first 6" of SPT sample has strong petroleum odor change to 1½" bit @21'
22						
		gravel to 2"				
24						hole caving-problems getting sampler to bottom of hole
			100K 4" B10-3		DR	upper rings of sample B10-3 are probable cuttings
26						
28						
30		interlayered lenses of dense medium grained sand		16 50 70	S P T	
					RD	
32						
34						
			100K 10" B10-4		DR RD	poor recovery in sample B10-4, 1 ring only
36						
	SP	<u>SAND</u>: dark gray to blue gray, dense to v. dense, med. grained contains interbeds of <u>GRAVELLY SAND</u>				
38						
40				30 35 60	S P T	
					RD	
42						
44						

Sheet 2 of 5

Project DESIGN UNIT A-100 Date Drilled 6-22-83 Hole No. B-10

DEPTH	USCS	MATERIAL CLASSIFICATION	SAMPLE	SPT (6")	DRILL MODE	REMARKS
44	SP	SAND: with interbeds of GRAVELLY SAND				
			100K 11" B10-5		DR	
46						
48						
50		medium grained sand 5% gravel to ½"		30 40 50	S P T RD	sample has strong petroleum odor
52						
54						
			70 K 12" B10-6		DR	
56						
58		medium to coarse grained sand 15% gravel to 1"				
60			B10 J1	30 50 70	S P T RD	sulfurous odor
62						
64						
			100K 10"		DR	no recovery @ 65'
66						
68						

Sheet 3 of 5

Project DESIGN UNIT A-100 **Date Drilled** 6-22-83 **Hole No.** B-10

DEPTH	USCS	MATERIAL CLASSIFICATION	SAMPLE	SPT (6")	DRILL MODE	REMARKS
68	SP	SAND: with interlayers of GRAVELLY SAND			RD	
70			B10	40 50 55	S P T	sulfurous odor
72					RD	
74						
		SAND, 5% gravel	100K 12" B10-7		DR	petroleum odor
76					RD	
78						
80			B10 J-3	35 50 50/5"	S P T	
82					RD	
84						heavy rig chatter @ 83.0'
			B-10 J-4	80 80/9"	SPT	
86		streaks & blebs of tar in GRAVELLY SAND, fragments of blue gray claystone			RD	
88						
90			100K 4"		DR	no recovery @ 90' stop @ 90', 6-22-83
					RD	6-23-83 start drilling 7:30 am
92						

Sheet 4 **of** 5

Project DESIGN UNIT A-100 **Date Drilled** 6-22-83 **Hole No.** B-10

DEPTH	USCS	MATERIAL CLASSIFICATION	SAMPLE	SPT (6")	DRILL MODE	REMARKS
	SP	SAND: with interlayers of GRAVELLY SAND			RD	
			B-10 J-5	20 50 75/1"	S P T	
		PUENTE FORMATION interbedded SILTSTONE AND CLAYSTONE, bedding is near vertical, olive gray, fine SANDSTONE interbeds Physical conditions: soft to friable, weak, fresh, fractures along bedding planes			RD	
			70K 12" B10-8		DR	
					RD	start to ream hole with 7" bit for PB sampler
			35K 12" B10-9		DR	Driller having trouble getting PB sample to
			PB10-1 0.6'		PB	stay in Shelby tube
			PB 10-2 1.1		PB	Try to wash gravel from bottom of hole
					RD	PB 10-2--tip of Shelby tube collapsed. Stop at 104.7', 6-23-83.
			50K 12" B10-10		DR	took CCI Sample due to unsuccessful PB attempt
		end boring @ 107.0' no excessive caving				Gravel at bottom of hole grinding up Shelby tubes, & sandy interbeds of Puente formation too hard to sample with pitcher barrel

Sheet 5 of 5

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BORING LOG B-11

Proj: DESIGN UNIT A-100
Date Drilled 6-24-83
Ground Elev. 271'
Drill Rig ROTARY
Logged By J. R. Stellar
Total Depth 107'
Hole Diameter 7"
Hammer Weight & Fall 320 lb @ 30"

DEPTH	USCS	MATERIAL CLASSIFICATION	SAMPLE	SPT (6")	DRILL MODE	REMARKS
0	GP/SP	FILL-MIXED GRAVEL AND SAND: with broken brick			RD	set-up @ 10:45 am start drilling @ 11:00 waited from 7:30 am to get across tracks 3¼ hrs. standby
2						
4						
			15K 12"		DR	no recovery at 5.0'
6					RD	
8						
10				4	S	
				5	P	
				9	T	
					RD	
12		alternating with GRAVELLY SILT layers				
14						
			11K 12" B11-1		DR	
16						
18						
20						

Sheet 1 of 5

Project DESIGN UNIT A-100 Date Drilled 6-24-83 Hole No. B-11

DEPTH	USCS	MATERIAL CLASSIFICATION	SAMPLE	SPT (6")	DRILL MODE	REMARKS
20		FILL MIXED GRAVELLY SAND AND GRAVELLY SILT	B-11 J-1	45 50 50	S P T	
					RD	
22						
24						
			30K 12"		DR	no recovery at 25'
26					RD	
28						
	SP	ALLUVIUM SAND alternating with GRAVELLY SAND: blue gray to greenish gray, fines < 5%, medium to coarse grained sand, 85%, gravel to 1", dense to very dense				heavy rig chatter @ 29'
30			B-11 J-2	25 40 100	S P T	
					RD	
32						
34						
			70K 12" B11-2		DR	poor recovery 3 rings only at 35'
36					RD	
38						
40			B-11 J-3	20 30 110	S P T	
42						
44						

Sheet 2 of 5

Project DESIGN UNIT A-100 **Date Drilled** ________ **Hole No.** B-11

DEPTH	USCS	MATERIAL CLASSIFICATION	SAMPLE	SPT (6")	DRILL MODE	REMARKS
44	SP	SAND alternating with GRAVELLY SAND			RD	
			70K 12"		DR	no recovery at 45'
46					RD	
48						
50				30	S	no recovery at 50'
				50	P	
				75	T	
52					RD	
54						
			100K 12" B11-3		DR	Petroleum odor
56						
58						
60				40	S	
				50	P	
			B11U-4	100	T	
62					RD	
64		SAND becomes very fine & silty calcareous streaks				
			60K 12" B11-4		RD	6-24-83 stop @ 66'
66						
68						

Project DESIGN UNIT A-100 Date Drilled 6-24-83 Hole No. B-11

DEPTH	USCS	MATERIAL CLASSIFICATION	SAMPLE	SPT (6")	DRILL MODE	REMARKS
68	SP	SAND alternating with layers of Gravelly Sand, layers of Tar Sand			RD	
70				30	S	sulfurous odor
			B11	40	P	
				100	T	
72					RD	
		increase in percentage GRAVEL at 73'				heavy rig chatter at 73'
74						
			75K 1"		DR	no recovery @ 75'
76					RD	
78						
80				75	S	
				100	P	
		high percent of tar in sands	B11J-6	100	T	
82					RD	
84						
			70K 2"		DR	no recovery @ 85'
86					RD	
88						
90				40	S	
				75	P	
			B11J-7	100	T	
92					RD	

Sheet 4 of 5

Project DESIGN UNIT A-100 **Date Drilled** 6-24-83 **Hole No.** B-11

DEPTH	USCS	MATERIAL CLASSIFICATION	SAMPLE	SPT (6")	DRILL MODE	REMARKS
92	SP	SAND alternating with GRAVELLY SAND ±20% tar			RD	
94						
		gravel to 2"	75K 12"		DR	no recovery @ 95'
96					RD	
98		BEDROCK-PUENTE FORMATION: interbedded SILTSTONE & CLAYSTONE olive gray, fine, SANDSTONE interbeds Physical Conditions: massive, soft to friable, weak, fresh				
100				20	S	
				40	P	
			B11J-8	40	T	
					RD	
102			45K 12" B11-5		DR	
			50K 12" B11-6		RD	pitcher barrel samples from 104.0-104.5'
104			B11PB-1		PB	Shelby tube collapsed rock too hard to sample with Shelby tube
					RD	
			45K 12" B11-7		DR	
106			45K 12" B11-8		DR	
		end boring @ 107.0'				
108						
110						
112						
114						
116						

Sheet 5 of 5

Appendix B

Pump Test Results

APPENDIX B PUMP TEST RESULTS

B.1 INTRODUCTION AND SUMMARY

A pump test was performed near Union Station (approximately 1400 feet west of the northwest end of Design Unit A100) to provide data for construction dewatering. Two pump tests were run at the same well to determine aquifer properties and boundary conditions. Two pump tests were performed because gas, entrained in the water, caused the first test to be terminated prematurely and additional testing was needed to confirm test results. The location of the pumping well and observation wells, OW-1, OW-2, and 5-1 are shown on Drawing 5.

The methodology used for the test consisted of constant discharge tests with time-drawdown measurements in the observation wells. These measurements were plotted on log-log paper as drawdown versus t/r^2 where t = time in days and r = the radial distance of the observation well from the pumped well in feet. The data plots for the test were matched to a family of type curves by Newman (1975) for wells fully penetrating an unconfined aquifer. Under these conditions the typical log of drawdown versus the log of time response is an S-shaped curve with delayed drainage causing a flattening of the curve between early and late responses. Field data plots are shown for each test along with matching curves, formulas used and computations. Aquifer test data sheets for each test and observation well are attached to Appendix B.

An effective hydraulic conductivity of 500 gpd/ft^2 is believed appropriate due to methane and hydrogen sulfide gas in the water. Gas bubbles in the pores reduce the ability of the aquifer to transmit water. Also, gas flow through the orifice plate used to measure well discharges result in readings higher than actual water discharge. Both factors combine to reduce computed transmissivities. However, if gas production dissipates during dewatering, hydraulic conductivity may increase to an estimated value on the order of 1,000 gpd/ft^2.

B.2 SITE CONDITIONS

The pump test was located at the north end of the public parking lot near Macy Street (see Drawing 5). Bedrock of the Puente Formation underlies the test area at a depth of 80 feet. This formation consists of siltstone/claystone and clayey sandstone. The bedrock acts as an aquiclude and for practical dewatering purposes is impermeable.

Overlying the Puente Formation is 78 feet of Young (coarse-grained) Alluvium and about 2 feet of artificial fill and asphalt pavement. The alluvium thickness is very uniform over the test area. The alluvium consists of (from top to bottom) silty sand/sandy silt to a depth of about 8 feet, gravelly sand with cobbles from 8 to 70 feet, and a boulder-gravel zone from a depth of 70 to 80 feet (see Drawing 4). Based on boring samples, the aquifer generally contains high percentages of fine to medium sand.

Static water level was 20 feet below ground surface. The saturated thickness of the unconfined aquifer was 60 feet.

B.3 WELL CONSTRUCTION AND DEVELOPMENT

The test well is located about 11 feet southwest of test Boring 5-1. The well was drilled by the cable tool method, driving 12-inch double walled casing. The well was drilled to a depth of 82.5 feet and the casing was perforated inplace from 21 feet to 77 feet. Perforations consist of 12 horizontal punched slots per foot that are 1-1/4 by 5/32 inches, in staggered rows.

The existing test boring, 5-1, has 2-inch PVC casing that is slotted from 45 feet to 85 feet. The boring was 4-3/4 inches in diameter drilled by the mud rotary method. The annulus between the PVC casing and the well bore was filled with pea gravel and sealed with bentonite above the perforations. This well was used as a supplemental observation well during the pump tests.

Two new observations wells were drilled by mud rotary methods, in line to the west of the test well. OW-1 is located 51 feet west of the well and was drilled to a depth of 84 feet. PVC casing, 4 inches in diameter, was installed with perforations from 5 to 70 feet. The annulus of the 6-inch bore was backfilled with pea gravel and sealed with 4 feet of cement and gravel.

Observations well OW-2 is located 110.6 feet west of the well and was drilled to a depth of 85 feet. PVC casing, 4 inches in diameter, was installed to a depth of 83 feet with perforations from 5 to 75 feet. The annulus of the 6-inch bore was backfilled with pea gravel and sealed with 4 feet of cement and gravel. The layout of the observation wells and the test well are shown on Drawing 5.

The pumping well was developed to flush mud and cuttings to provide hydraulic communication with the aquifer. The 12-inch test well was surged by bailing and then developed with the test pump for about 20 hours. Gas was noticed in the pump discharge during development pumping at maximum drawdowns. Near the end of the pump tests, distant drawdown measurements indicate that the pumped well was operating at a hydraulic efficiency of about 30%. That is, 70% of well drawdown was due to well losses in the pumped well.

B.4 PUMP TESTING PROCEDURE

A constant discharge test was planned with a test duration of 24 to 48 hours. However, because of gas problems that developed with time, two relatively short duration tests were performed.

The gas problems are discussed at the end of the Section under comments on test results.

The first test was run on March 24, 1983 for approximately 500 minutes at a well discharge rate of 175 gpm. The test was terminated because gas was causing the discharge rate to fluctuate and observation wells began to recover indicating a reduced well discharge. The second test was run at a reduced discharge rate of 150 gpm for 1380 minutes during March 25 and 26, 1983. Again, the test was terminated because of gas interference resulting in recovery of observation wells.

The test well was pumped with an Aurora lineshaft turbine pump (capacity 500 gpm) powered by a Cummings diesel engine. Discharge was measured with an inline orifice plate and a mercury manometer. The base of the 10-inch diameter bowls were set at a depth of 78 feet below the ground surface. Water from the well was discharged through a pipe to a storm drain near the southeast corner of the intersection of Alameda and Macy Streets.

Drawdowns in the pumped well (maximum 30 feet) were measured occasionally by air line during the tests, but were not used for test interpretations because of the high well losses. Drawdowns were measured in each observation well at times selected to provide suitable logarithmic distributions of time. Drawdown measurements in observation wells OW-1 and OW-2 were made with Stevens Recorders. Times were recorded manually on the chart paper because the recorder clocks are not sensitive enough particularly during the early more frequent measurements. Drawdowns in observation well 5-1 were measured with a hand-held electric sounder. Generally, measurements of drawdown were accurate within ±0.01 foot.

Recovery tests were planned after the pumping ceased. However, during both tests, recoveries had already started prior to stopping the pump due to gas problems. Regardless, recovery measurements were made in OW-1 and OW-2 after the first test. These measurements, however, produced unreliable results because of the premature recovery combined with delayed drainage effects. As a result, it was decided to test the gas responses at various pumping rates after the second test (in place of recovery measurements) using a gas detector.

Rubber tubing with a metal tip, attached to the methane reading gas detector (made by Gastech Inc.), was inserted to a depth of 5 feet in the pumping well's water level measuring hole, immediately after the pump was turned off. Instantly the methane gas detector needle surged to a reading of 100% lower explosive limit (LEL) and for some unexplainable reason the gas ignited in the instrument causing a small explosion that blew the rubber tubing out of the hole. The gas detector indicated around 30% LEL methane gas each time gas was measured up to the explosion.

B.5 TEST INTERPRETATIONS

Time-drawdown data were plotted on log-log graphs as shown on interpretation charts. Figures B-1 and B-2 show data for the first and second tests, respectively, for observation wells OW-1 and OW-2 with the log of drawdown(s) in feet plotted versus the log of time (days) divided by the distance (feet) from the pump well squared (t/r^2). These data plots were matched to the type curves indicated and appropriate match points were selected to determine values of s and t/r^2 for corresponding values of $W(\upsilon)$ and $1/\upsilon$. The calculations for transmissivity (T), Storativity (S) or Specific yield (Sy) are shown.

Figure B-3 shows data plots, match points and calculations for observation well 5-1 for both the first and second tests.

The water level responses of OW-1 (well location not shown) indicate the typical S-shaped curve of an unconfined aquifer with delayed drainage for both tests. The plots of the first test are somewhat distorted possibly due to gas and a recharge event (intense rainfall) just prior to the test. The second test at the reduced pumping rate provided better matches for both the A region and B region of the type curves.

Data from well 5-1 (well location not shown) indicate on both tests that most of the drawdown occurred in the first minute of pumping and the plot is in the region of delayed drainage. However, using OW-1 as a guide, reasonable matches were obtained.

Data from OW-2 indicate the possibility of delayed well response. OW-2 may have been damaged by siltation when runoff water flowed into the well immediately prior to the first test. However, good matches were obtained in the B region of the curves for both tests. Results from OW-2 for both tests must remain somewhat suspect, however.

Distance-drawdown plots shown on Figure B-4, were used as a check where log of the distance is plotted against drawdown on a semi-log chart. Wells OW-1 and OW-2 were used in this plot primarily. Well 5-1 is very close to the pumped well in a region where potential lines are relatively distorted. Since the bottom 40 feet of this observation well are perforated, drawdowns at a given time would tend to be greater than should be the case in the distance-drawdown relationship. The results of the analyses from the best fit type of curves along with distance drawdown analyses are summarized in Table B-1 below.

TABLE B-1
SUMMARY OF PUMP TEST RESULTS

Test	Observation Well	Curve Match	Transmissivity (gpd/ft)	Average Hydraulic Conductivity (gpd/ft^2)	Storativity (S) or Specific Yield (Sy)
1st	OW-1	B Type, β=0.06	20,055	334	0.064 (Sy)
1st	OW-2	B Type, β=6.0	30,386	506	0.021 (Sy)
1st	5-1	B Type, β=0.01	20,055	334	0.23 (Sy)
1st	OW-1, OW-2, 5-1	Dist. d.d.	24,973	416	0.028 (Sy)
2nd	OW-1	A Type, β=0.06	20,464	341	0.0044 (S)
2nd	OW-1	B Type, β=0.06	21.488	358	0.069 (Sy)
2nd	OW-2	B Type. β=6.0	41.927	699	0.022 (Sy)
2nd	5-1	B Type β=0.01	15,627	260	0.27 (Sy)
2nd	OW-1, OW-2, 5-1	Dist. d.d.	21.405	357	0.061 (Sy)

The transmissivities range from 16,000 to 42,000 gpd/ft with a mean of about 24,000 gpd/ft. The average hydraulic conductivity is the transmissivity divided by the saturated thickness of 60 feet. Average hydraulic conductivities range from 250 to 700 gpd/ft² (~1.3 x 10^{-2} to 3.2 x 10^{-2} cm/sec). These values are in a low range for clean stream channel deposits, but these low values may be explained by free gas, which would tend to reduce hydraulic conductivities, influencing results significantly. A hydraulic conductivity of 1,000 gpd/ft² or more is judged reasonable for these deposits near Union Station, if gas were not present.

Most of the computed specific yields appear to be very low for this type of aquifer. The values computed range from 0.021 to 0.27. A specific yield in the 0.15 to 0.25 range would be reasonable, and a value of 0.2 to 0.25 is more probable. For dewatering purposes the use of a specific yield of 0.25 is recommended because this would be more conservative.

B.6 COMMENTS ON TEST RESULTS

Boundary conditions were not detected from the pump tests at Union Station. This was primarily due to the relatively short duration of both tests. Longer term dewatering operations should encounter barrier boundary effects as the pumping cone intersects the boundary of the aquifer about 1000 feet west of the pumping well as shown on Milestone 10 Drawings 2 and 3. This will enhance dewatering to some degree. The effect can be estimated by determining distance to a barrier(s) from geologic maps and factoring the barrier(s) into dewatering computations.

The gases encountered are assumed to be methane and hydrogen sulfide, based upon the methane gas detector readings and the characteristic odor of hydrogen sulfide. Other gases may have been present also in lesser quantities as described in Appendix D. Gases such as these can be hazardous at comparatively low concentrations, and their release can affect hydraulic efficiencies of pumps and wells as was observed during these tests.

The hypothesis for the origin of the gases is as follows: It is assumed that the gases originate from the underlying Puente Formation and are confined within the aquifer. When the water level is drawn down, the reduction in hydrostatic pressure causes a release of the gases from solution or from depth, or both. This hypothesis implies that the volumes of gas may increase as the drawdown increases - which the pump tests suggested may be the case. There appeared to be a threshold of time drawdown of about 9 or 10 feet where relatively large amounts of gases were observed as being released. The tests were not conducted for a long enough period to determine if, with time, the volumes of gas released would or would not decrease.

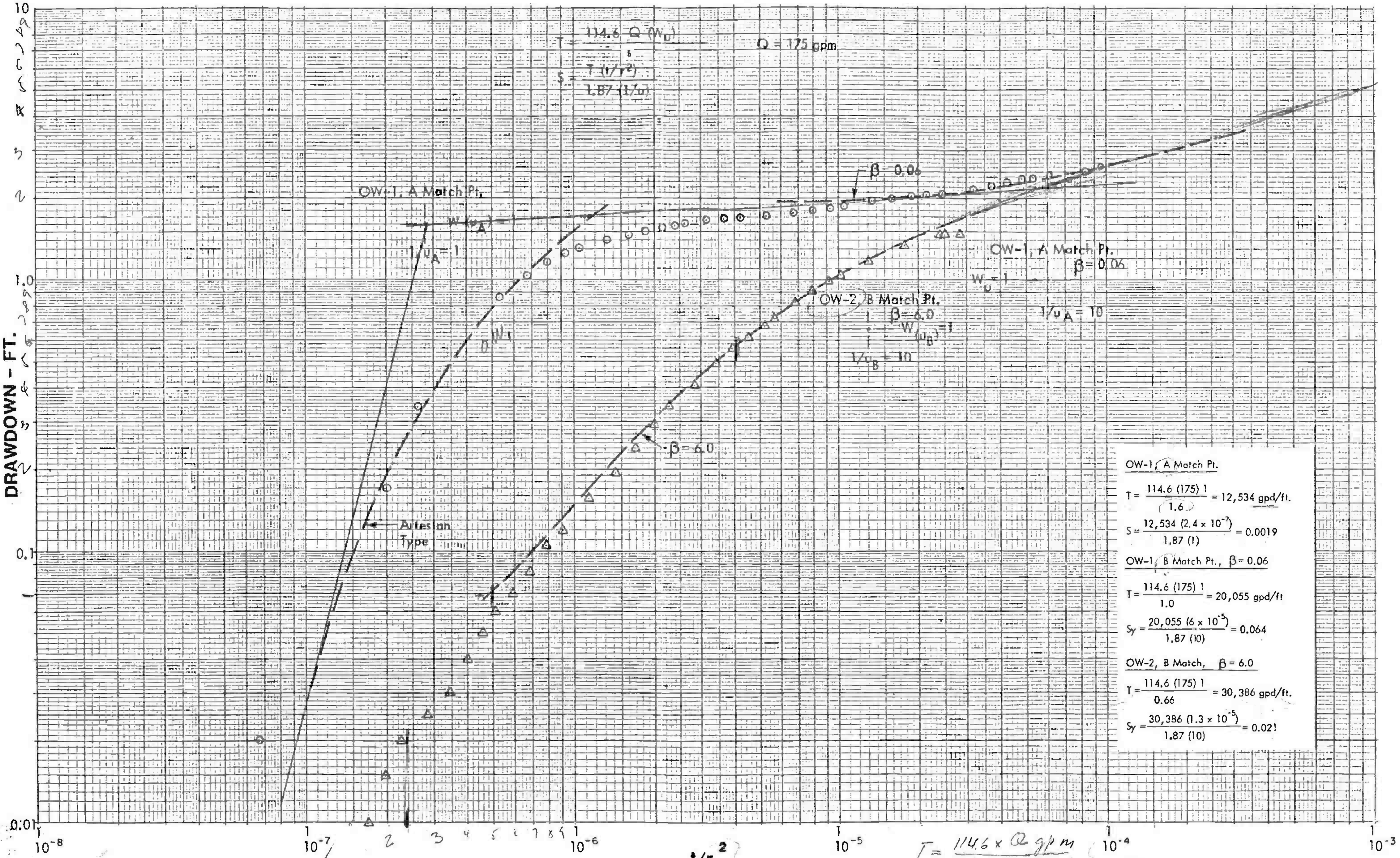
The following information regarding gas and oil in this area helps explain the presence, if not the pressure or quantity, of gas in the pump test well:

- The Union Station Oil Field is located beneath the proposed Yard and Shops area. Little is known about the oil field, but it does produce from the Puente Formation at very shallow depths.
- Boring CEG-2 (see Drawing 5) located some 2000 feet east of the Union Station site penetrated the Young Alluvium/Puente Formation contact at a depth of 38 feet. Oil was encountered at this contact as well as in sandstone layers within the Puente Formation from 38 to 100 feet (bottom of hole). Gas (hydrogen sulfide) was first detected by odor at a depth of 37 feet in Boring CEG-2. Gas chromatograms run in Boring CEG-1 indicate 100 ppm methane and 500 ppm ethane. This would thus be classified in the lower explosive limit (5% LEL).

It is believed that at least a portion of the ground water underlying the site may be saturated with gas which originated from the underlying Puente Formation and/or may contain free gas in the aquifer or underlying Puente Formation that is released as hydrostatic pressures are reduced during pumping. During the pump test, there was a considerable drop in pressure head near the well as water flowed into the pump. This pressure drop would have resulted in release of the gas and into the well head. Additional data would be required to confirm these concepts and delineate the problem. The engineering implications of these observations are discussed in Chapter 6.

The pump tests resulted in reasonably consistent data in terms of transmissivity and hydraulic conductivity. A mean transmissivity of 24,000 gpd/ft is considered a good effective value for gassy conditions, however permeability along the length of the Union Station and Yard and Shops box structure excavation may vary significantly because of the variable geologic conditions.

It is beyond the scope of this report to recommend specific dewatering systems. However, the aquifer is amenable to well dewatering, providing the gases are controlled safely and effectively. Also, more efficient wells would be needed to be cost effective. Mainly well screens with larger open areas are needed such as wire wound screens or Rosco Moss' "Full Flo" louvered screen, along with careful well development. The presence of hydrogen sulfide gas suggests some corrosion potential which should be considered with the time that dewatering is required for construction.



OW-1, A Match Pt.

$T = \frac{114.6\ (175)\ 1}{1.6} = 12,534$ gpd/ft.

$S = \frac{12,534\ (2.4 \times 10^{-7})}{1.87\ (1)} = 0.0019$

OW-1, B Match Pt., $\beta = 0.06$

$T = \frac{114.6\ (175)\ 1}{1.0} = 20,055$ gpd/ft

$S_y = \frac{20,055\ (6 \times 10^{-5})}{1.87\ (10)} = 0.064$

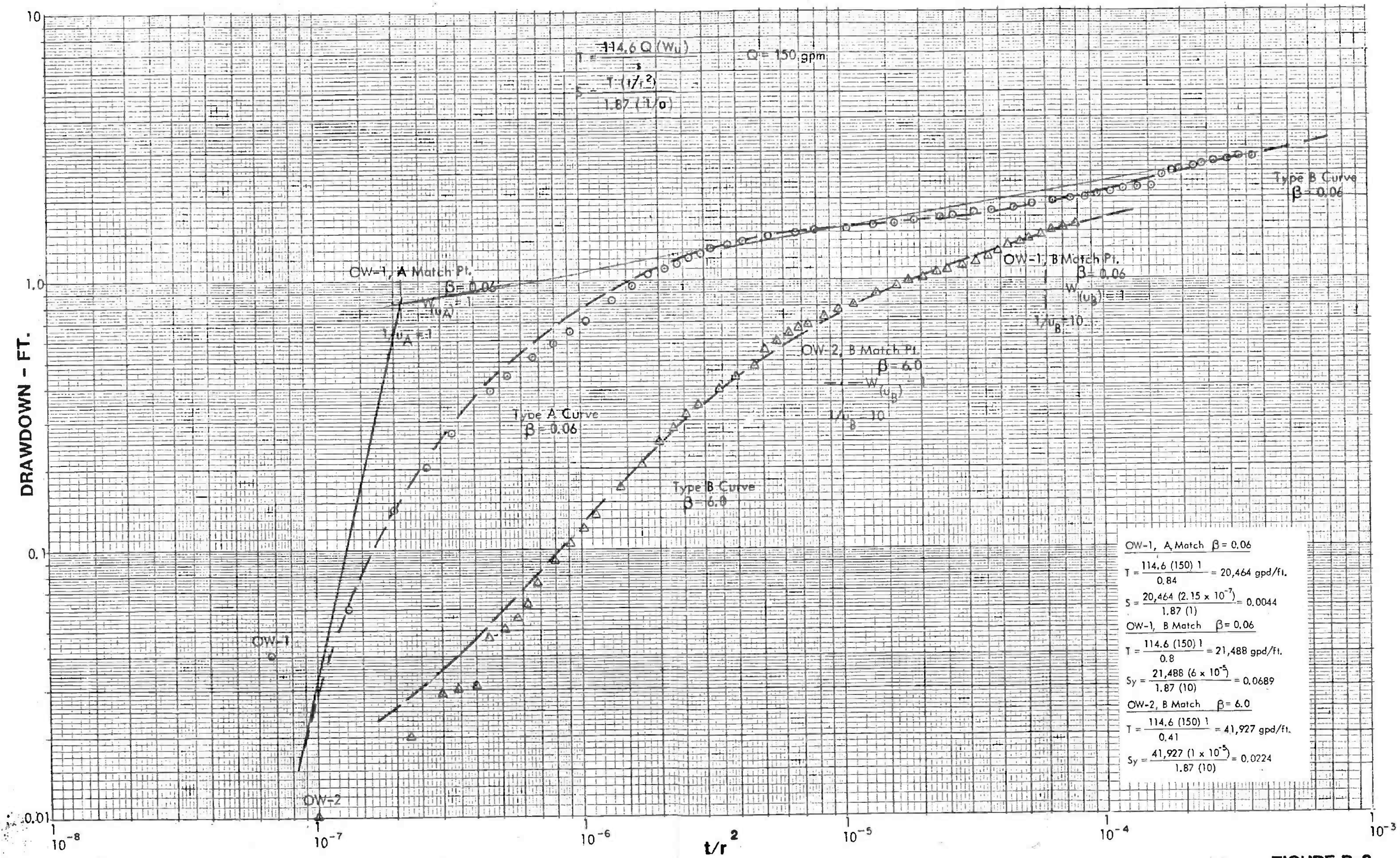
OW-2, B Match, $\beta = 6.0$

$T = \frac{114.6\ (175)\ 1}{0.66} = 30,386$ gpd/ft.

$S_y = \frac{30,386\ (1.3 \times 10^{-5})}{1.87\ (10)} = 0.021$

$T = \frac{114.6 \times Q\ gpm}{lg\left(R_2^2/R_1^2\right)} \frac{60}{6.5}$ $\frac{360}{.25}$

FIGURE B-1



$$T = \frac{114.6\ Q\ (W_u)}{s}$$

$$S = \frac{T\ (t/r^2)}{1.87\ (1/u)}$$

OW-1, A Match β = 0.06

$$T = \frac{114.6\ (150)\ 1}{0.84} = 20{,}464 \text{ gpd/ft.}$$

$$S = \frac{20{,}464\ (2.15 \times 10^{-7})}{1.87\ (1)} = 0.0044$$

OW-1, B Match β = 0.06

$$T = \frac{114.6\ (150)\ 1}{0.8} = 21{,}488 \text{ gpd/ft.}$$

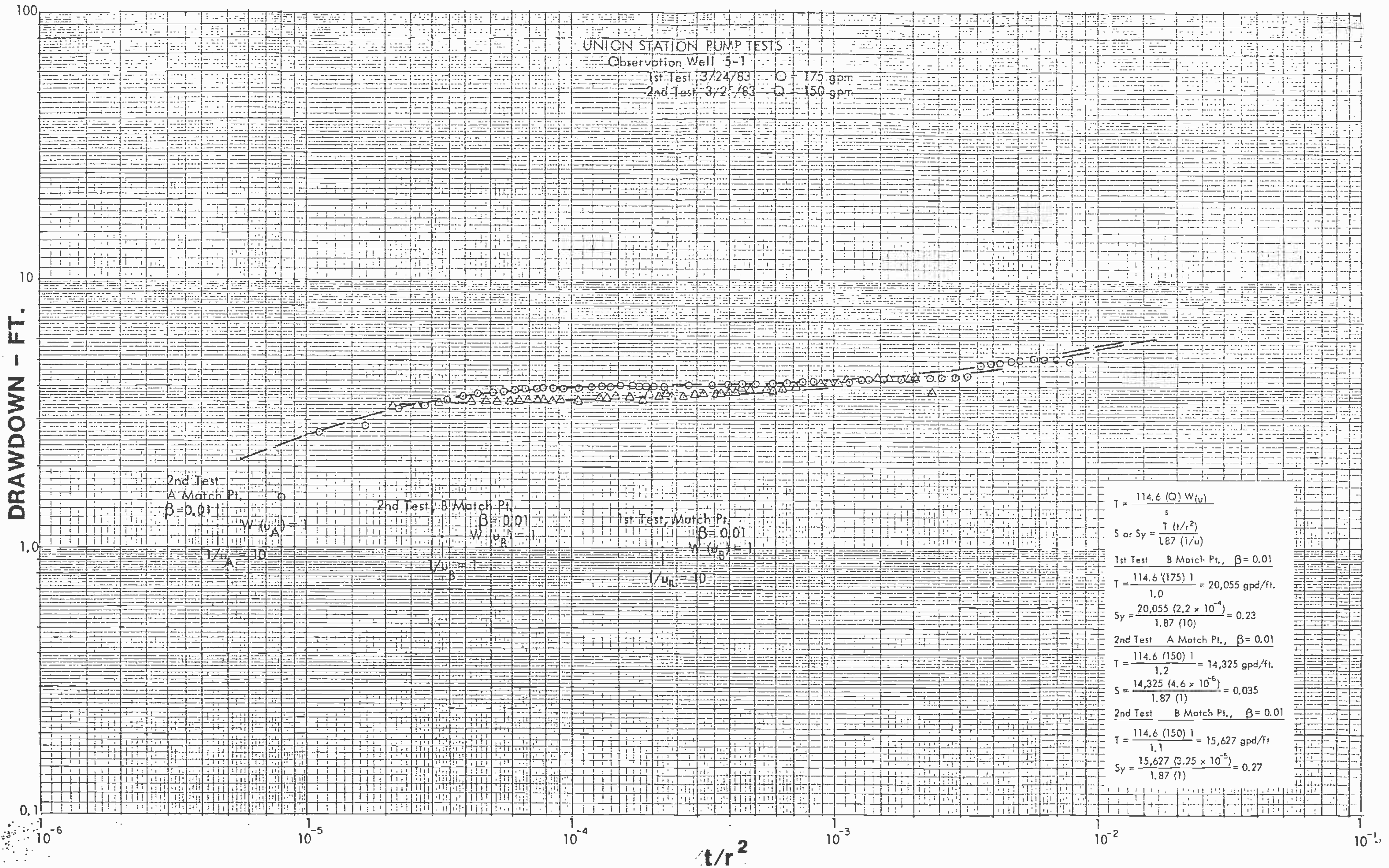
$$Sy = \frac{21{,}488\ (6 \times 10^{-5})}{1.87\ (10)} = 0.0689$$

OW-2, B Match β = 6.0

$$T = \frac{114.6\ (150)\ 1}{0.41} = 41{,}927 \text{ gpd/ft.}$$

$$Sy = \frac{41{,}927\ (1 \times 10^{-5})}{1.87\ (10)} = 0.0224$$

FIGURE B-2



$$T = \frac{114.6\ (Q)\ W_{(u)}}{s}$$

$$S \text{ or } Sy = \frac{T\ (t/r^2)}{1.87\ (1/u)}$$

1st Test B Match Pt., β= 0.01

$$T = \frac{114.6\ (175)\ 1}{1.0} = 20{,}055 \text{ gpd/ft.}$$

$$Sy = \frac{20{,}055\ (2.2 \times 10^{-4})}{1.87\ (10)} = 0.23$$

2nd Test A Match Pt., β= 0.01

$$T = \frac{114.6\ (150)\ 1}{1.2} = 14{,}325 \text{ gpd/ft.}$$

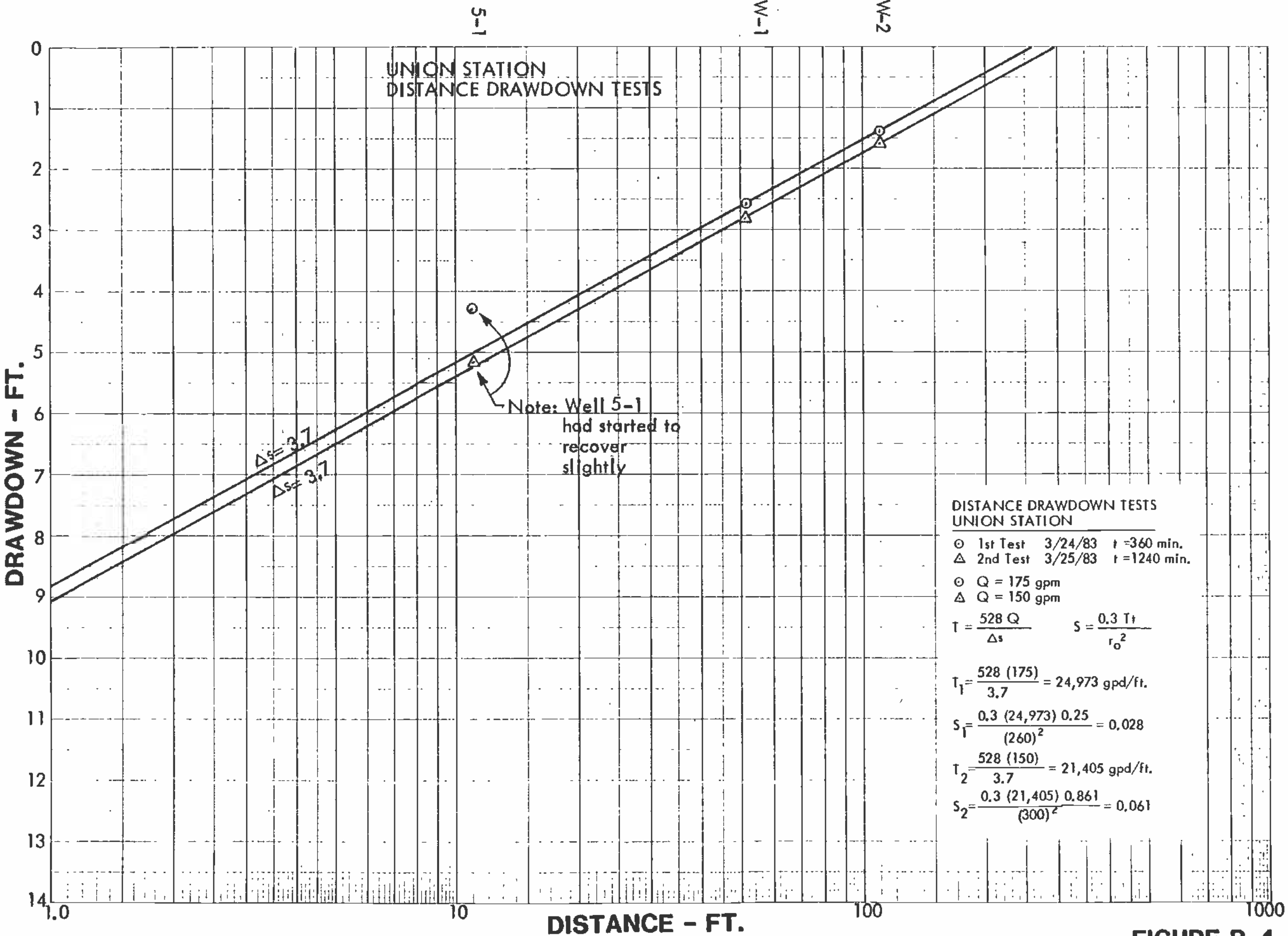
$$S = \frac{14{,}325\ (4.6 \times 10^{-6})}{1.87\ (1)} = 0.035$$

2nd Test B Match Pt., β= 0.01

$$T = \frac{114.6\ (150)\ 1}{1.1} = 15{,}627 \text{ gpd/ft}$$

$$Sy = \frac{15{,}627\ (3.25 \times 10^{-5})}{1.87\ (1)} = 0.27$$

FIGURE B-3



UNION STATION
DISTANCE DRAWDOWN TESTS

Note: Well 5-1 had started to recover slightly

DISTANCE DRAWDOWN TESTS
UNION STATION

⊙ 1st Test 3/24/83 t =360 min.
△ 2nd Test 3/25/83 t =1240 min.

⊙ Q = 175 gpm
△ Q = 150 gpm

$T = \frac{528\ Q}{\Delta s}$ $S = \frac{0.3\ Tt}{r_o^2}$

$T_1 = \frac{528\ (175)}{3.7} = 24,973$ gpd/ft.

$S_1 = \frac{0.3\ (24,973)\ 0.25}{(260)^2} = 0.028$

$T_2 = \frac{528\ (150)}{3.7} = 21,405$ gpd/ft.

$S_2 = \frac{0.3\ (21,405)\ 0.861}{(300)^2} = 0.061$

FIGURE B-4

AQUIFER TEST DATA SHEET

Observation Well No. OW-1

Project No. E167

Test Well No. Union Station

Date of Test 03/24/83

Static Water Level 20.49

Observed By WRH, TDH

Radius from Pumped Well 51.1

Average Discharge 175 gpm

Time	t min.	t days	t/r^2	Water Level feet	Drawdown, s feet	Remarks
11:45	0	---	---	20.49	0	
	.25	$1.74x10^{-4}$	$6.66x10^{-8}$	20.51	0.02	
	.50	$3.47x10^{-4}$	$1.33x10^{-7}$	20.56	0.07	
	.75	$5.21x10^{-4}$	$2.00x10^{-7}$	20.66	0.17	
	1	$6.94x10^{-4}$	$2.66x10^{-7}$	20.83	0.34	
	1.5	$1.04x10^{-3}$	$3.98x10^{-7}$	21.09	0.60	
	2	$1.39x10^{-3}$	$5.32x10^{-7}$	21.34	0.85	
	2.5	$1.74x10^{-3}$	$6.66x10^{-7}$	21.52	1.03	
	3	$2.08x10^{-3}$	$7.97x10^{-7}$	21.65	1.16	
	3.5	$2.43x10^{-3}$	$9.31x10^{-7}$	21.74	1.25	
	4	$2.77x10^{-3}$	$1.06x10^{-6}$	21.79	1.30	
	5	$3.47x10^{-3}$	$1.33x10^{-6}$	21.89	1.40	
	6	$4.17x10^{-3}$	$1.60x10^{-6}$	21.94	1.45	
	7	$4.86x10^{-3}$	$1.86x10^{-6}$	21.99	1.50	
	8	$5.56x10^{-3}$	$2.13x10^{-6}$	22.03	1.54	
	9	$6.25x10^{-3}$	$2.39x10^{-6}$	22.07	1.58	
	10	$6.94x10^{-3}$	$2.66x10^{-6}$	22.09	1.60	
	12	$8.33x10^{-3}$	$3.19x10^{-6}$	22.13	1.64	
	14	$9.72x10^{-3}$	$3.72x10^{-6}$	22.16	1.67	
	16	$1.11x10^{-2}$	$4.25x10^{-6}$	22.18	1.69	
	20	$1.39x10^{-2}$	$5.32x10^{-6}$	22.21	1.72	
	25	$1.74x10^{-2}$	$6.66x10^{-6}$	22.26	1.77	

Well No. [illegible]-1 Page 2 of 2

Time	t min.	t days	t/r^2	Water Level feet	Drawdown, s feet	Remarks
	30	$2.08\text{x}10^{-2}$	$7.97\text{x}10^{-6}$	22.30	1.81	
	35	$2.43\text{x}10^{-2}$	$9.31\text{x}10^{-6}$	22.32	1.83	
	40	$2.77\text{x}10^{-2}$	$1.06\text{x}10^{-5}$	22.37	1.88	
	50	$3.47\text{x}10^{-2}$	$1.33\text{x}10^{-5}$	22.44	1.95	
	60	$4.17\text{x}10^{-2}$	$1.60\text{x}10^{-5}$	22.47	1.98	
	70	$4.86\text{x}10^{-2}$	$1.86\text{x}10^{-5}$	22.51	2.02	
	80	$5.56\text{x}10^{-2}$	$2.13\text{x}10^{-5}$	22.55	2.06	
	92	$6.39\text{x}10^{-2}$	$2.45\text{x}10^{-5}$	22.57	2.08	
	100	$6.94\text{x}10^{-2}$	$2.66\text{x}10^{-5}$	22.57	2.08	
1:45p	120	$8.33\text{x}10^{-2}$	$3.19\text{x}10^{-5}$	22.67	2.18	
	140	$9.72\text{x}10^{-2}$	$3.72\text{x}10^{-5}$	22.71	2.22	
2:25p	160	$1.11\text{x}10^{-1}$	$4.25\text{x}10^{-5}$	22.77	2.28	
2:45p	180	$1.25\text{x}10^{-1}$	$4.79\text{x}10^{-5}$	22.82	2.33	
	200	$1.39\text{x}10^{-1}$	$5.32\text{x}10^{-5}$	22.86	2.37	
3:35	230	$1.60\text{x}10^{-1}$	$6.13\text{x}10^{-5}$	22.91	2.42	
5:01	316	$2.19\text{x}10^{-1}$	$8.39\text{x}10^{-5}$	23.01	2.52	
5:45	360	$2.50\text{x}10^{-1}$	$9.57\text{x}10^{-5}$	23.08	2.59	
6:45	420	$2.92\text{x}10^{-1}$	$1.12\text{x}10^{-4}$	23.01	2.52	
7:06	441	$3.06\text{x}10^{-1}$	$1.17\text{x}10^{-4}$	22.96	2.47	
8:02	497	$3.45\text{x}10^{-1}$	$1.32\text{x}10^{-4}$	22.91	2.42	

AQUIFER TEST DATA SHEET

Observation Well No. OW-1

Project No. E167

Test Well No. Union Station

Date of Test 03/25/83

Static Water Level 20.84

Observed By WRH, TDH

Radius from Pumped Well 51.1

Average Discharge 150 gpm

Time	t min.	t days	t/r^2	Water Level feet	Drawdown, s feet	Remarks
11:30a	0	--	--	20.84	0	
	.25	$1.74x10^{-4}$	$6.66x10^{-8}$	20.88	0.04	
	.50	$3.47x10^{-4}$	$1.33x10^{-7}$	20.90	0.06	
	.75	$5.21x10^{-4}$	$2.00x10^{-7}$	20.98	0.14	
	1	$6.94x10^{-4}$	$2.66x10^{-7}$	21.04	0.20	
	1.25	$8.68x10^{-4}$	$3.32x10^{-7}$	21.11	0.27	
	1.75	$1.21x10^{-3}$	$4.63x10^{-7}$	21.23	0.39	
	2	$1.39x10^{-3}$	$5.32x10^{-7}$	21.28	0.44	
	2.5	$1.74x10^{-3}$	$6.66x10^{-7}$	21.36	0.52	
	3	$2.08x10^{-3}$	$7.97x10^{-7}$	21.42	0.58	
	3.5	$2.43x10^{-3}$	$9.31x10^{-7}$	21.49	0.65	
	4	$2.77x10^{-3}$	$1.06x10^{-6}$	21.55	0.71	
	5	$3.47x10^{-3}$	$1.33x10^{-6}$	21.68	0.84	
	6	$4.17x10^{-3}$	$1.60x10^{-6}$	21.79	0.95	
	7	$4.86x10^{-3}$	$1.86x10^{-6}$	21.88	1.04	
	8	$5.56x10^{-3}$	$2.13x10^{-6}$	21.95	1.11	
	9	$6.25x10^{-3}$	$2.39x10^{-6}$	21.98	1.14	
	10	$6.94x10^{-3}$	$2.66x10^{-6}$	22.05	1.21	
	11	$7.64x10^{-3}$	$2.93x10^{-6}$	22.10	1.26	
	12	$8.33x10^{-3}$	$3.19x10^{-6}$	22.14	1.30	
	14	$9.72x10^{-3}$	$3.72x10^{-6}$	22.18	1.34	
	16	$1.11x10^{-3}$	$4.25x10^{-6}$	22.22	1.38	

Well No. [illegible] Page 2 of 5

Time	t min.	t days	t/r^2	Water Level feet	Drawdown, s feet	Remarks
	18	$1.25x10^{-2}$	$4.79x10^{-6}$	22.28	1.44	
	20	$1.39x10^{-2}$	$5.32x10^{-6}$	22.30	1.46	
	25	$1.74x10^{-2}$	$6.66x10^{-6}$	22.33	1.49	
	30	$2.08x10^{-2}$	$7.97x10^{-6}$	22.36	1.52	
	40	$2.77x10^{-2}$	$1.06x10^{-5}$	22.40	1.56	
	50	$3.47x10^{-2}$	$1.33x10^{-5}$	22.43	1.59	
	60	$4.17x10^{-2}$	$1.60x10^{-5}$	22.46	1.62	
	70	$4.86x10^{-2}$	$1.86x10^{-5}$	22.48	1.64	
	90	$6.25x10^{-2}$	$2.39x10^{-5}$	22.53	1.69	
	100	$6.94x10^{-2}$	$2.66x10^{-5}$	22.55	1.71	
1:30p	120	$8.33x10^{-2}$	$3.19x10^{-5}$	22.60	1.76	
	140	$9.72x10^{-2}$	$3.72x10^{-5}$	22.63	1.79	
	170	$1.18x10^{-1}$	$4.52x10^{-5}$	22.67	1.83	
	200	$1.39x10^{-1}$	$5.32x10^{-5}$	22.73	1.89	
3:30	240	$1.67x10^{-1}$	$6.40x10^{-5}$	22.78	1.94	
4:15	285	$1.98x10^{-1}$	$7.58x10^{-5}$	22.82	1.98	
4:49	319	$2.22x10^{-1}$	$8.50x10^{-5}$	22.85	2.01	
5:30	360	$2.50x10^{-1}$	$9.57x10^{-5}$	22.89	2.05	
6:15	405	$2.81x10^{-1}$	$1.08x10^{-4}$	22.93	2.09	
7:00	450	$3.13x10^{-1}$	$1.20x10^{-4}$	22.97	2.13	
8:00	510	$3.54x10^{-1}$	$1.36x10^{-4}$	23.01	2.17	
9:04	574	$3.99x10^{-1}$	$1.53x10^{-4}$	23.04	2.20	
10:02	632	$4.39x10^{-1}$	$1.68x10^{-4}$	23.26	2.42	
10:58	688	$4.78x10^{-1}$	$1.83x10^{-4}$	23.34	2.50	
11:58	748	$5.19x10^{-1}$	$1.99x10^{-4}$	23.38	2.54	
1:30a	840	$5.83x10^{-1}$	$2.23x10^{-4}$	23.46	2.62	
	900	$6.25x10^{-1}$	$2.39x10^{-4}$	23.51	2.67	
4:00a	1000	$6.94x10^{-1}$	$2.66x10^{-4}$	23.56	2.72	

Well No. GW-1

Time	t min.	t days	t/r^2	Water Level feet	Drawdown, s feet	Remarks
6:00a	1120	$7.77x10^{-1}$	$2.98x10^{-4}$	23.61	2.77	
8:00a	1240	$8.61x10^{-1}$	$3.30x10^{-4}$	23.66	2.82	
10:00a	1380	$9.58x10^{-1}$	$3.67x10^{-4}$	23.65	2.81	

AQUIFER TEST DATA SHEET

Observation Well No. OW-2

Project No. E167

Test Well No. Union Station

Date of Test 03/24/83

Static Water Level 19.51

Observed By WRH, TDH

Radius from Pumped Well 110.6

Average Discharge 175 gpm

Time	t min.	t days	t/r^2	Water Level feet	Drawdown, s feet	Remarks
11:45	0	--	--	19.510	0	
	.25	$1.74x10^{-4}$	$1.42x10^{-8}$	19.510	0	
	.50	$3.47x10^{-4}$	$2.84x10^{-8}$	19.510	0	
	.75	$5.21x10^{-4}$	$4.26x10^{-8}$	19.510	0	
	1	$6.94x10^{-4}$	$5.67x10^{-8}$	19.510	0	
	1.5	$1.04x10^{-3}$	$8.50x10^{-8}$	19.515	0.005	
	2	$1.39x10^{-3}$	$1.14x10^{-7}$	19.515	0.005	
	2.5	$1.74x10^{-3}$	$1.42x10^{-7}$	19.515	0.005	
	3	$2.08x10^{-3}$	$1.70x10^{-7}$	19.520	0.010	
	3.5	$2.43x10^{-3}$	$1.99x10^{-7}$	19.525	0.015	
	4	$2.78x10^{-3}$	$2.27x10^{-7}$	19.530	0.020	
	5	$3.47x10^{-3}$	$2.83x10^{-7}$	19.535	0.025	
	6	$4.17x10^{-3}$	$3.41x10^{-7}$	19.540	0.030	
	7	$4.86x10^{-3}$	$3.97x10^{-7}$	19.550	0.040	
	8	$5.56x10^{-3}$	$4.55x10^{-7}$	19.560	0.050	
	9	$6.25x10^{-3}$	$5.11x10^{-7}$	19.570	0.060	
	10.5	$7.29x10^{-3}$	$5.96x10^{-7}$	19.580	0.070	
	12	$8.33x10^{-3}$	$6.81x10^{-7}$	19.595	0.085	
	14	$9.72x10^{-3}$	$7.95x10^{-7}$	19.615	0.105	
	16	$1.11x10^{-2}$	$9.07x10^{-7}$	19.630	0.120	
	20	$1.39x10^{-2}$	$1.14x10^{-6}$	19.665	0.155	
	25	$1.74x10^{-2}$	$1.42x10^{-6}$	19.705	0.195	

Well No. [illegible] Page [illegible] of 2

Time	t min.	t days	t/r^2	Water Level feet	Drawdown, s feet	Remarks
	30	2.08×10^{-2}	1.70×10^{-6}	19.750	0.240	
	35	2.43×10^{-2}	1.99×10^{-6}	19.805	0.295	
	40	2.78×10^{-2}	2.27×10^{-6}	19.850	0.340	
	50	3.47×10^{-2}	2.84×10^{-6}	19.920	0.410	
	60	4.17×10^{-2}	3.41×10^{-6}	20.000	0.490	
	70	4.86×10^{-2}	3.97×10^{-6}	20.070	0.560	
	80	5.56×10^{-2}	4.55×10^{-6}	20.125	0.615	
	92	6.39×10^{-2}	5.22×10^{-6}	20.190	0.680	
	100	6.94×10^{-2}	5.67×10^{-6}	20.230	0.720	
	120	8.33×10^{-2}	6.81×10^{-6}	20.340	0.830	
	140	9.72×10^{-2}	7.95×10^{-6}	20.420	0.910	
	160	1.11×10^{-1}	9.07×10^{-6}	20.490	0.980	
2:45	180	1.25×10^{-1}	1.02×10^{-5}	20.560	1.050	
3:05	200	1.39×10^{-1}	1.14×10^{-5}	20.620	1.110	
3:35	230	1.60×10^{-1}	1.31×10^{-5}	20.690	1.180	
4:05	260	1.81×10^{-1}	1.48×10^{-5}	--	--	
4:56	311	2.16×10^{-1}	1.77×10^{-5}	20.850	1.340	
5:47	362	2.51×10^{-1}	2.05×10^{-5}	20.920	1.410	
6:47	422	2.93×10^{-1}	2.40×10^{-5}	20.980	1.470	
7:07	442	3.07×10^{-1}	2.51×10^{-5}	20.985	1.475	
8:04	499	3.47×10^{-1}	2.84×10^{-5}	20.990	1.480	

AQUIFER TEST DATA SHEET

Observation Well No. OW-2

Project No. E167

Test Well No. Union Station

Date of Test 03/25/83

Static Water Level 19.50

Observed By WRH, TDH

Radius from Pumped Well 110.6

Average Discharge 150 gpm

Time	t min.	t days	t/r^2	Water Level feet	Drawdown, s feet	Remarks
11:30	0	--	--	19.500	0	
	.25	$1.74x10^{-4}$	$1.42x10^{-8}$	19.500	0	
	.50	$3.47x10^{-4}$	$2.84x10^{-8}$	19.500	0	
	.75	$5.21x10^{-4}$	$4.26x10^{-8}$	19.504	0.004	
	1	$6.94x10^{-4}$	$5.67x10^{-8}$	19.504	0.004	
	1.25	$8.68x10^{-4}$	$7.10x10^{-8}$	19.504	0.004	
	1.50	$1.04x10^{-3}$	$8.50x10^{-8}$	19.504	0.004	
	1.75	$1.22x10^{-3}$	$9.97x10^{-8}$	19.510	0.010	
	3	$2.08x10^{-3}$	$1.70x10^{-7}$	19.515	0.015	
	3.50	$2.43x10^{-3}$	$1.99x10^{-7}$	19.515	0.015	
	4	$2.78x10^{-3}$	$2.27x10^{-7}$	19.520	0.020	
	5	$3.47x10^{-3}$	$2.83x10^{-7}$	19.520	0.020	
	5.25	$3.65x10^{-3}$	$2.98x10^{-7}$	19.529	0.029	
	6	$4.17x10^{-3}$	$3.41x10^{-7}$	19.530	0.030	
	7	$4.86x10^{-3}$	$3.97x10^{-7}$	19.532	0.032	
	8	$5.56x10^{-3}$	$4.55x10^{-7}$	19.547	0.047	
	9	$6.25x10^{-3}$	$5.11x10^{-7}$	19.550	0.050	
	10	$6.94x10^{-3}$	$5.67x10^{-7}$	19.555	0.055	
	11	$7.64x10^{-3}$	$6.25x10^{-7}$	19.562	0.062	
	12	$8.33x10^{-3}$	$6.81x10^{-7}$	19.575	0.075	
	14	$9.72x10^{-3}$	$7.95x10^{-7}$	19.590	0.090	
	16	$1.11x10^{-3}$	$9.07x10^{-7}$	19.605	0.105	

Well No. OW-2 Page 2 of 3

Time	t min.	t days	t/r^2	Water Level feet	Drawdown, s feet	Remarks
	18	$1.25x10^{-2}$	$1.02x10^{-6}$	19.620	0.120	
	20	$1.39x10^{-2}$	$1.14x10^{-6}$	19.635	0.135	
	25	$1.74x10^{-2}$	$1.42x10^{-6}$	19.670	0.170	
	30	$2.08x10^{-2}$	$1.70x10^{-6}$	19.705	0.205	
12:05	35	$2.43x10^{-2}$	$1.99x10^{-6}$	19.750	0.250	
12:10	40	$2.77x10^{-2}$	$2.27x10^{-6}$	19.780	0.280	
	45	$3.13x10^{-2}$	$2.56x10^{-6}$	19.815	0.315	
12:20	50	$3.47x10^{-2}$	$2.84x10^{-6}$	19.840	0.340	
12:30	60	$4.17x10^{-2}$	$3.41x10^{-6}$	19.890	0.390	
12:40	70	$4.86x10^{-2}$	$3.97x10^{-6}$	19.930	0.430	
12:52	82	$5.69x10^{-2}$	$4.65x10^{-6}$	19.980	0.480	
1:00	90	$6.25x10^{-2}$	$5.11x10^{-6}$	20.050	0.550	
1:10	100	$6.94x10^{-2}$	$5.67x10^{-6}$	20.090	0.590	
1:20	110	$7.64x10^{-2}$	$6.25x10^{-6}$	20.130	0.630	
1:30	120	$8.33x10^{-2}$	$6.81x10^{-6}$	20.160	0.660	
1:40	130	$9.03x10^{-2}$	$7.38x10^{-6}$	20.180	0.680	
2:00	150	$1.04x10^{-1}$	$8.50x10^{-6}$	20.230	0.730	
2:20	170	$1.18x10^{-1}$	$9.65x10^{-6}$	20.270	0.770	
2:46	196	$1.36x10^{-1}$	$1.11x10^{-5}$	20.310	0.810	
3:30	240	$1.67x10^{-1}$	$1.37x10^{-5}$	20.390	0.890	
4:15	285	$1.98x10^{-1}$	$1.62x10^{-5}$	20.440	0.940	
4:47	317	$2.20x10^{-1}$	$1.80x10^{-5}$	20.480	0.980	
5:30	360	$2.50x10^{-1}$	$2.04x10^{-5}$	20.510	1.010	
6:14	404	$2.81x10^{-1}$	$2.30x10^{-5}$	20.550	1.050	
7:00	450	$3.13x10^{-1}$	$2.56x10^{-5}$	20.580	1.080	
7:56	506	$3.51x10^{-1}$	$2.87x10^{-5}$	20.620	1.120	
9:00	570	$3.96x10^{-1}$	$3.24x10^{-5}$	20.660	1.160	
10:00	630	$4.38x10^{-1}$	$3.58x10^{-5}$	20.700	1.200	

Well No. OW-2

Time	t min.	t days	t/r^2	Water Level feet	Drawdown, s feet	Remarks
11:00	690	4.79×10^{-1}	3.92×10^{-5}	20.770	1.270	
12:00	750	5.21×10^{-1}	4.26×10^{-5}	20.820	1.320	
	840	5.83×10^{-1}	4.77×10^{-5}	20.880	1.380	
	900	6.25×10^{-1}	5.11×10^{-5}	20.910	1.410	
	1000	6.94×10^{-1}	5.67×10^{-5}	20.960	1.460	
	1120	7.77×10^{-1}	6.35×10^{-5}	21.010	1.510	
	1240	8.61×10^{-1}	7.04×10^{-5}	21.060	1.560	
	1380	9.58×10^{-1}	7.83×10^{-5}	21.100	1.600	

AQUIFER TEST DATA SHEET

Observation Well No. 5-1

Project No. E167

Test Well No. Union Station

Date of Test 03/24/83

Static Water Level 19.92

Observed By DG

Radius from Pumped Well 11.1 ft.

Average Discharge 175 gpm

Time	t min.	t days	t/r^2	Water Level feet	Drawdown, s feet	Remarks
11:45	0	0	---	19.92	0.00	large initial drawdown-missed 1 min. reading
	1	6.94x1$\bar{0}^{4}$	5.33x1$\bar{0}^{6}$	--	--	
	2	1.39x1$\bar{0}^{3}$	1.07x1$\bar{0}^{5}$	23.58	3.66	
	3	2.08x1$\bar{0}^{3}$	1.60x1$\bar{0}^{5}$	23.60	3.68	
	4	2.78x1$\bar{0}^{3}$	2.14x1$\bar{0}^{5}$	23.31	3.39	
	5	3.47x1$\bar{0}^{3}$	2.67x1$\bar{0}^{5}$	23.32	3.40	
	6	4.17x1$\bar{0}^{3}$	3.20x1$\bar{0}^{5}$	23.40	3.48	
	7	4.86x1$\bar{0}^{3}$	3.73x1$\bar{0}^{5}$	--	--	
	8	5.56x1$\bar{0}^{3}$	4.27x1$\bar{0}^{5}$	23.42	3.50	
	9	6.25x1$\bar{0}^{3}$	4.80x1$\bar{0}^{5}$	23.42	3.50	
	10	6.94x1$\bar{0}^{3}$	5.33x1$\bar{0}^{5}$	23.43	3.51	
	11	7.64x1$\bar{0}^{3}$	5.87x1$\bar{0}^{5}$	23.43	3.51	
	12	8.33x1$\bar{0}^{3}$	6.40x1$\bar{0}^{5}$	23.46	3.54	
	13	9.03x1$\bar{0}^{3}$	6.94x1$\bar{0}^{5}$	23.47	3.55	
	14	9.72x1$\bar{0}^{3}$	7.47x1$\bar{0}^{5}$	23.47	3.55	
	15	1.04x1$\bar{0}^{2}$	7.99x1$\bar{0}^{5}$	23.50	3.58	
	16	1.11x1$\bar{0}^{2}$	8.53x1$\bar{0}^{5}$	23.48	3.56	
	17	1.18x1$\bar{0}^{2}$	9.06x1$\bar{0}^{5}$	23.47	3.55	
	18	1.25x1$\bar{0}^{2}$	9.60x1$\bar{0}^{5}$	--	--	
	19	1.32x1$\bar{0}^{2}$	1.01x1$\bar{0}^{4}$	--	--	
	20	1.39x1$\bar{0}^{2}$	1.07x1$\bar{0}^{4}$	23.47	3.55	
	22	1.53x1$\bar{0}^{2}$	1.18x1$\bar{0}^{4}$	--	--	

Well No. [illegible]

Page 2 of 3

Time	t min.	t days	t/r^2	Water Level feet	Drawdown, s feet	Remarks
	24	1.67×10^{-2}	1.28×10^{-4}	23.53	3.61	
	25	1.74×10^{-2}	1.34×10^{-4}	23.55	3.63	
	28	1.94×10^{-2}	1.49×10^{-4}	23.57	3.65	
	31	2.15×10^{-2}	1.65×10^{-4}	23.55	3.63	
	35	2.43×10^{-2}	1.87×10^{-4}	23.50	3.58	
	37	2.57×10^{-2}	1.97×10^{-4}	23.71	3.79	
	40	2.78×10^{-2}	2.14×10^{-4}	23.62	3.70	
	43	2.99×10^{-2}	2.30×10^{-4}	23.67	3.75	
	45	3.13×10^{-2}	2.40×10^{-4}	23.64	3.72	
	50	3.47×10^{-2}	2.67×10^{-4}	23.65	3.73	
	56	3.89×10^{-2}	2.99×10^{-4}	23.66	3.74	
	60 67	4.17×10^{-2} 4.65×10^{-2}	3.20×10^{-4} 3.57×10^{-4}	23.66 23.70	3.74 3.78	
	70	4.86×10^{-2}	3.73×10^{-4}	23.69	3.77	
	75	5.21×10^{-2}	4.00×10^{-4}	23.71	3.79	
	80	5.56×10^{-2}	4.27×10^{-4}	23.71	3.79	
	85	5.90×10^{-2}	4.53×10^{-4}	23.83	3.91	
	90	6.25×10^{-2}	4.80×10^{-4}	23.75	3.83	
	95	6.60×10^{-2}	5.07×10^{-4}	23.80	3.88	
	100	6.94×10^{-2}	5.33×10^{-4}	--	--	
1:30	105	7.29×10^{-2}	5.60×10^{-4}	23.77	3.85	
	110	7.64×10^{-2}	5.87×10^{-4}	23.78	3.86	
1:45	120	8.33×10^{-2}	6.40×10^{-4}	23.90	3.98	
1:50	125	8.68×10^{-2}	6.67×10^{-4}	23.92	4.00	
2:00	135	9.38×10^{-2}	7.20×10^{-4}	23.91	3.99	
2:10	145	1.01×10^{-1}	7.76×10^{-4}	23.98	4.06	
2:26	155	1.08×10^{-1}	8.29×10^{-4}	24.03	4.11	
2:30	165	1.15×10^{-1}	8.83×10^{-4}	24.03	4.11	
2:45	180	1.25×10^{-1}	9.60×10^{-4}	24.01	4.09	

Well No. S-1

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Time	t min.	t days	t/r^2	Water Level feet	Drawdown, s feet	Remarks
3:00	195	$1.35\text{x}10^{-1}$	$1.04\text{x}10^{-3}$	24.07	4.15	
3:15	210	$1.46\text{x}10^{-1}$	$1.12\text{x}10^{-3}$	24.16	4.24	
3:45	240	$1.67\text{x}10^{-1}$	$1.28\text{x}10^{-3}$	24.20	4.28	
4:15	270	$1.88\text{x}10^{-1}$	$1.44\text{x}10^{-3}$	24.23	4.31	
4:45	300	$2.08\text{x}10^{-1}$	$1.60\text{x}10^{-3}$	24.19	4.27	
5:50	365	$2.53\text{x}10^{-1}$	$1.94\text{x}10^{-3}$	24.22	4.30	
7:04	439	$3.05\text{x}10^{-1}$	$2.34\text{x}10^{-3}$	23.68	3.76	

AQUIFER TEST DATA SHEET

Observation Well No. 5-1

Project No. E167

Test Well No. Union Station

Date of Test 03/25/83

Static Water Level 19.01

Observed By DG

Radius from Pumped Well 11.1 ft.

Average Discharge 150 gpm

Time	t min.	t days	t/r^2	Water Level feet	Drawdown, s feet	Remarks
11:30	0	0	--	19.01	0.0	started pump 11:15, stopped
	1	$6.94x10^{-4}$	$5.63x10^{-6}$	--	--	11:20-start 11:30 static before 1st
	2	$1.39x10^{-3}$	$1.13x10^{-5}$	21.72	2.71	start-19.01
	3	$2.08x10^{-3}$	$1.69x10^{-5}$	21.85	2.84	
	4	$2.78x10^{-3}$	$2.26x10^{-5}$	22.30	3.29	
	5	$3.47x10^{-3}$	$2.82x10^{-5}$	22.35	3.34	
	6	$4.17x10^{-3}$	$3.38x10^{-5}$	22.55	3.54	
	7	$4.86x10^{-3}$	$3.94x10^{-5}$	22.69	3.68	
	8	$5.56x10^{-3}$	$4.51x10^{-5}$	22.76	3.75	
	9	$6.25x10^{-3}$	$5.07x10^{-5}$	22.77	3.76	
	10	$6.94x10^{-3}$	$5.63x10^{-5}$	22.82	3.81	
	11	$7.64x10^{-3}$	$6.20x10^{-5}$	22.86	3.85	
	12	$8.33x10^{-3}$	$6.76x10^{-5}$	22.90	3.89	
	13	$9.03x10^{-3}$	$7.55x10^{-5}$	22.86	3.85	
	14	$9.72x10^{-3}$	$7.89x10^{-5}$	22.90	3.89	
	15	$1.04x10^{-2}$	$8.44x10^{-5}$	22.90	3.89	
	17	$1.18x10^{-2}$	$9.58x10^{-5}$	22.94	3.93	
	19	$1.32x10^{-2}$	$1.07x10^{-4}$	22.95	3.94	
	21	$1.46x10^{-2}$	$1.19x10^{-4}$	22.97	3.96	
	23	$1.60x10^{-2}$	$1.30x10^{-4}$	23.01	4.00	
	25	$1.74x10^{-2}$	$1.41x10^{-4}$	23.00	3.99	
	27	$1.88x10^{-2}$	$1.53x10^{-4}$	23.02	4.01	

Well No. 5-1 Page 2 of 5

Time	t min.	t days	t/r^{2}	Water Level feet	Drawdown, s feet	Remarks
12:00	30	2.08x10^{-2}	1.69x10^{-4}	23.01	4.00	
	32	2.22x10^{-2}	1.80x10^{-4}	23.02	4.01	
	34	2.36x10^{-2}	1.92x10^{-4}	22.99	3.98	
	36	2.50x10^{-2}	2.03x10^{-4}	23.00	3.99	
	40	2.78x10^{-2}	2.26x10^{-4}	23.00	3.99	
12:20	50	3.47x10^{-2}	2.82x10^{-4}	23.03	4.02	
12:30	60	4.17x10^{-2}	3.38x10^{-4}	23.05	4.04	
12:40	70	4.86x10^{-2}	3.94x10^{-4}	23.06	4.05	
12:50	80	5.56x10^{-2}	4.51x10^{-4}	23.09	4.08	
1:00	90	6.25x10^{-2}	5.07x10^{-4}	23.08	4.07	
1:15	105	7.29x10^{-2}	5.92x10^{-4}	23.10	4.09	
1:30	120	8.33x20^{-2}	6.76x10^{-4}	23.13	4.12	
1:45	135	9.38x10^{-2}	7.61x10^{-4}	23.14	4.13	
2:00	150	1.04x10^{-1}	8.44x10^{-4}	23.17	4.16	
2:50	200	1.39x10^{-1}	1.13x10^{-3}	23.20	4.19	
3:15	225	1.56x10^{-1}	1.27x10^{-3}	23.26	4.25	
3:30	240	1.67x10^{-1}	1.36x10^{-3}	23.30	4.29	
4:00	270	1.88x10^{-1}	1.53x10^{-3}	23.31	4.30	
4:50	320	2.22x10^{-1}	1.80x10^{-3}	23.31	4.30	
5:34	364	2.53x10^{-1}	2.05x10^{-3}	23.33	4.32	
6:20	410	2.85x10^{-1}	2.31x10^{-3}	23.36	4.35	
7:03	453	3.15x10^{-1}	2.56x10^{-3}	23.35	4.34	
8:04	514	3.57x10^{-1}	2.90x10^{-3}	23.37	4.36	
9:08	578	4.01x10^{-1}	3.25x10^{-3}	23.38	4.37	
10:06	636	4.42x10^{-1}	3.59x10^{-3}	23.82	4.81	
11:04	694	4.82x10^{-1}	3.91x10^{-3}	23.91	4.90	
5/83 12:03	753	5.23x10^{-1}	4.25x10^{-3}	23.92	4.91	
	843	5.85x10^{-1}	4.75x10^{-3}	23.95	4.94	

Well No. 5-1 Page 5 of [illegible]

Time	t min.	t days	t/r^2	Water Level feet	Drawdown, s feet	Remarks
	905	6.28×10^{-1}	5.10×10^{-3}	24.03	5.02	
	1000	6.94×10^{-1}	5.63×10^{-3}	24.09	5.08	
	1120	7.78×10^{-1}	6.31×10^{-3}	24.05	5.04	
	1240	8.61×10^{-1}	6.99×10^{-3}	24.08	5.07	
	1380	9.58×10^{-1}	7.78×10^{-3}	23.98	4.97	

Appendix C

Geophysical Exploration

CCI/ESA/GRC

APPENDIX C GEOPHYSICAL EXPLORATION

C.1 DOWNHOLE SURVEY

C.1.1 Summary

A downhole shear wave velocity survey was performed in Boring CEG-5 Design Unit A135 (shown on Drawing No. 5). Measurements were made at 5-foot intervals from the ground surface to depths of 130 feet. A description of the technique and a summary of the results are attached.

C.1.2 Field Procedure

Shearing energy was generated by using a sledge hammer source on the ends of a 4-by-6-inch timber positioned under the tires of a station wagon, tangential to the borehole. A 12-channel signal enhancement seismograph (Geometrics Model ES1210) allowed the summing of several blows in one direction when necessary to increase the signal-to-noise ratio. Shear waves were identified by recording wave arrivals with opposite first motions on adjacent channels of the seismograph.

C.1.3 Data Analysis

For the purpose of illustration, typical wave arrival records from a downhole geophysical survey are reproduced in Figure C-1. The timing line shows a 20 millisecond (MS) break at the end of the record, indicating that each vertical line is 10 MS. The time of the first arrivals of compressional shear energy is indicated by P and S, respectively. Wave arrival records similar to Figure C-1 were analyzed to estimate wave travel times and velocities for CEG-5.

C.1.4 Discussion of Results

Estimated velocity structures are summarized in Table C-1. Velocity estimates are based on selection of linear portions of the downhole arrival time curves (see Figure C-2).

The error analysis performed for these surveys involved a least squares fit of these data by estimating the mean of the slope ($\bar{V}$) in Table C-1) and the standard deviation of this estimate of the slope. This estimate of the standard deviation was combined with an estimate of the overall accuracy to produce the best estimated velocity (V*). Vp* are the values to be used for studies of the response of these sites. N is the number of data points used for the straight line fit for each velocity estimate.

In general, near-surface shear wave velocity was found to be in the range of 1000 feet per second. To depths of 130 feet, shear velocity estimates increased to 1400±100 feet per second.

C.2 CROSSHOLE SURVEY

C.2.1 Summary

Crosshole measurements for the determination of seismic wave velocities were performed in Boring CEG-5. The crosshole technique for determining shear wave velocities of in-situ materials was utilized in a three-borehole array. The array consisted of boring CEG-5 and two additional holes drilled approximately 15 feet away. All boreholes were drilled to a depth of 100 feet. Compressional wave and shear wave velocities are presented in Table C-2.

C.2.2 Field Procedure

The shear wave hammer is placed in an end hole of the array, and vertical geophones are placed in the remaining two boreholes. The shear wave generating hammer and the two geophones are lowered to the same depth in all boreholes. The hammer is coupled to the wall of the hole by means of hydraulic jacks, and the geophones are coupled by means of expanding heavy rubber balloons which protrude from one side of the geophone housings. The hammer is then used to create vertically polarized shear waves with either an up or down first motion. A 12-channel signal enhancement seismograph with oscilloscope and electrostatic paper camera is used as a signal storage device.

C.2.3 Data Analysis

For the data analysis actual crosshole distances were determined to within ±0.01 feet. These distances were computed between each of the three boreholes at the elevations of shear measurements. From the crosshole records (seismograms), the travel times for both compressional and shear wave arrivals at each borehole and at each depth were measured . Shear wave arrivals were identified by the reversed first motion on the seismograms. Compression and shear wave estimates were based on the wave arrival records.

C.2.4 Discussion of Data Analysis

Seismic wave velocity determinations were made at 5-foot intervals from 10 feet below ground surface to a depth of 100 feet (see Figures C-3 and C-4). The shear wave velocity (V_s) is equal to the difference in travel path distance from the shear source to each geophone divided by the difference in shear wave arrival times. The results of the compressional and shear wave velocity analyses are shown in Table C-2. It should be noted that compression wave velocities below the ground water table may be masked by the compression wave response of the water (V_c = 5000 fps) particularly in highly porous materials.

TABLE C-1 Downhole Velocities

Boring No.	Depth (ft)	COMPRESSIONAL WAVE					SHEAR WAVE				
		$\overline{V}p$	σ p	Ep	Np	Vp*	$\overline{V}s$	σs	Es	Ns	Vs*
5	20-75	4831	736	242	10	4830±98	994	79	50	12	990±130
	75-130	5458	500	273	12	5460±770	1391	39	70	12	1390±110

$\overline{V}p$ = mean estimate of compressional wave velocity

$\overline{V}s$ = mean estimate of shear wave velocity

σp = standard deviation of estimated compressional wave velocity

σs = standard deviation of estimated shear wave velocity

Ep = estimated accuracy of compressional survey

Es = estimated accuracy of shear survey

Np = number of points used for straight line fit of compressional wave

Vp* = overall accuracy of compressional wave velocity estimate

Vs* = overall accuracy of shear wave velocity estimate

Ns = number of points used for straight line fit of shear wave velocity data

TABLE C-2 Crosshole Velocities

		COMPRESSIONAL WAVE					SHEAR WAVE				
Boring No.	Depth (ft)	$\bar{V}_p$	σp	Ep	Np	Vp*	$\bar{V}_s$	σs	Es	Ns	Vs*
5	10	2555	---	510	1	2560±510	948	3	47	6	950±50
	15	4948	319	247	4	4950±570	1081	17	54	10	1080±70
	20	5847	524	292	4	5850±820	1107	4	55	4	1110±60
	25	4886	---	490	2	4890±490	1122	41	56	9	1120±100
	30	4512	1056	226	10	4510±1280	1103	4	55	8	1100±60
	40	5307	300	265	8	5310±570	1193	63	60	8	1190±120
	45	5981	305	300	4	5980±600	1158	7	58	12	1160±60
	50	6256	590	313	4	6260±900	1025	6	51	13	1025±60
	55	5810	369	290	4	5180±660	1168	28	58	12	1170±90
	60	4651	607	233	4	4650±840	1167	4	58	13	1170±60
	65	5147	1193	257	9	5150±1450	1173	37	59	12	1170±100
	70	5041	422	252	6	5040±670	1080	12	54	6	1080±70
	75	5803	947	290	4	5800±1240	1224	42	61	12	1220±100
	80	7852	---	1570	2	7850±1570	1400	22	70	16	1400±90
	85	5200	426	260	8	5200±690	1186	42	59	10	1190±100
	90	5740	409	287	4	5740±700	1364	10	68	12	1360±80
	95	5873	1180	294	4	5870±1470	1304	8	65	10	1300±70
	99	5592	291	280	8	5590±570	1417	14	71	16	1410±90

$\bar{V}_p$ = mean estimate of compressional wave velocity

$\bar{V}_s$ = mean estimate of shear wave velocity

σp = standard deviation of estimated compressional wave velocity

σs = standard deviation of estimated shear wave velocity

Ep = estimated accuracy of compressional survey

Es = estimated accuracy of shear survey

Np = number of points used for straight line fit of compressional wave

Vp* = overall accuracy of compressional wave velocity estimate

Vs* = overall accuracy of shear wave velocity estimate

Ns = number of points used for straight line fit of shear wave velocity data

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DOWNHOLE SAMPLE RECORD

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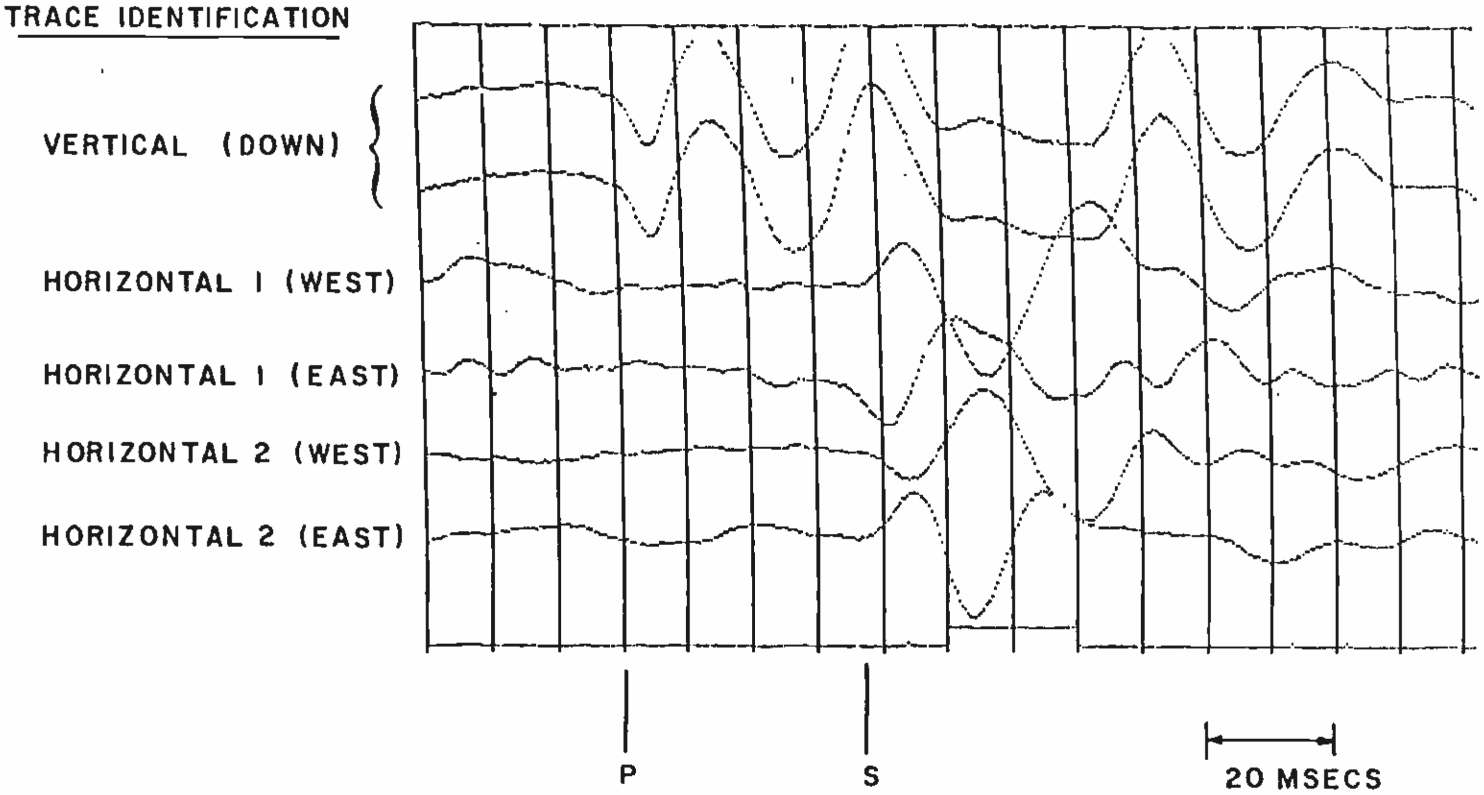
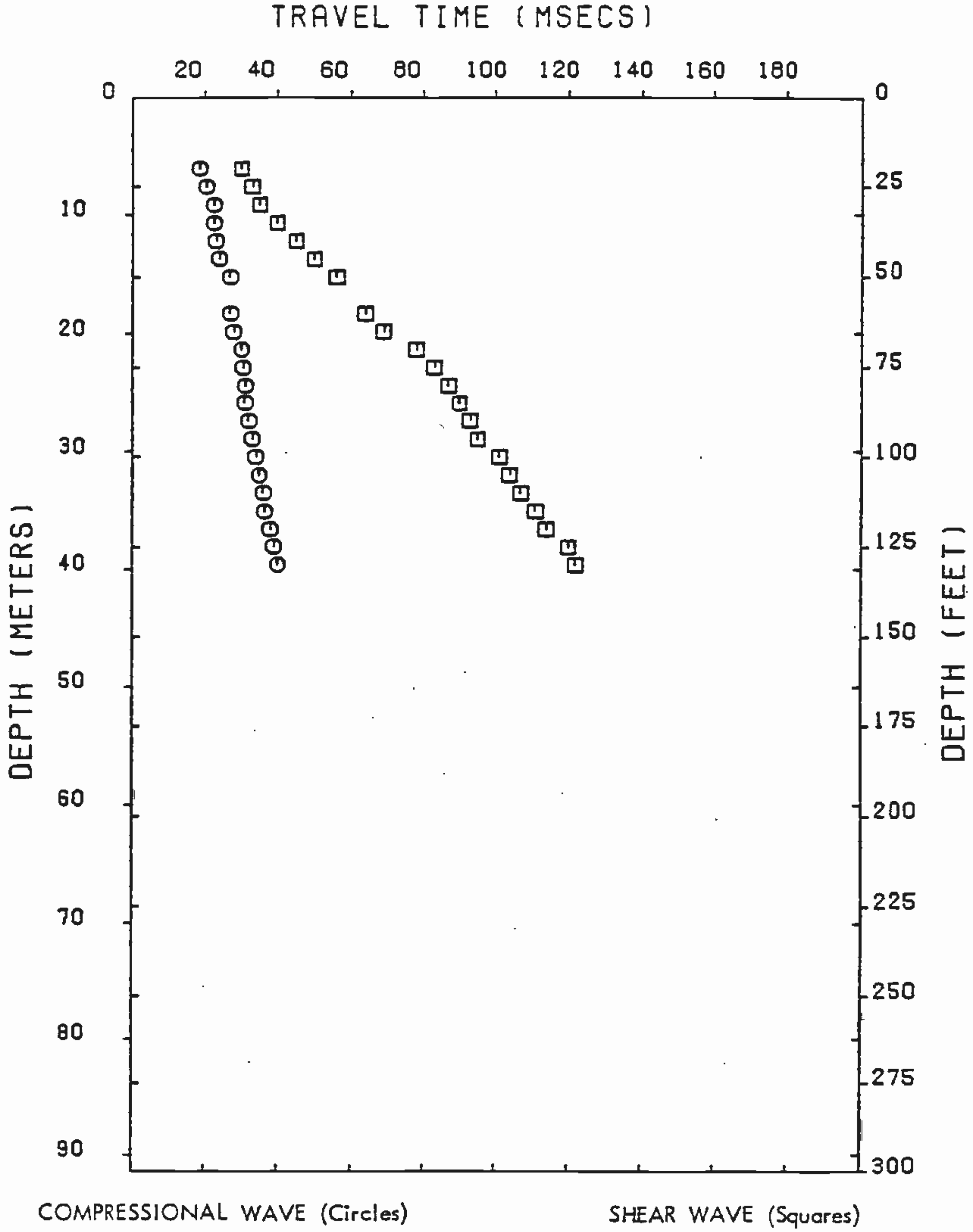
Figure No.
C-1



BOREHOLE: 13

DEPTH: 70 FT



DOWNHOLE TRAVEL TIME PROFILE - BORING 5

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Figure No.
C-2

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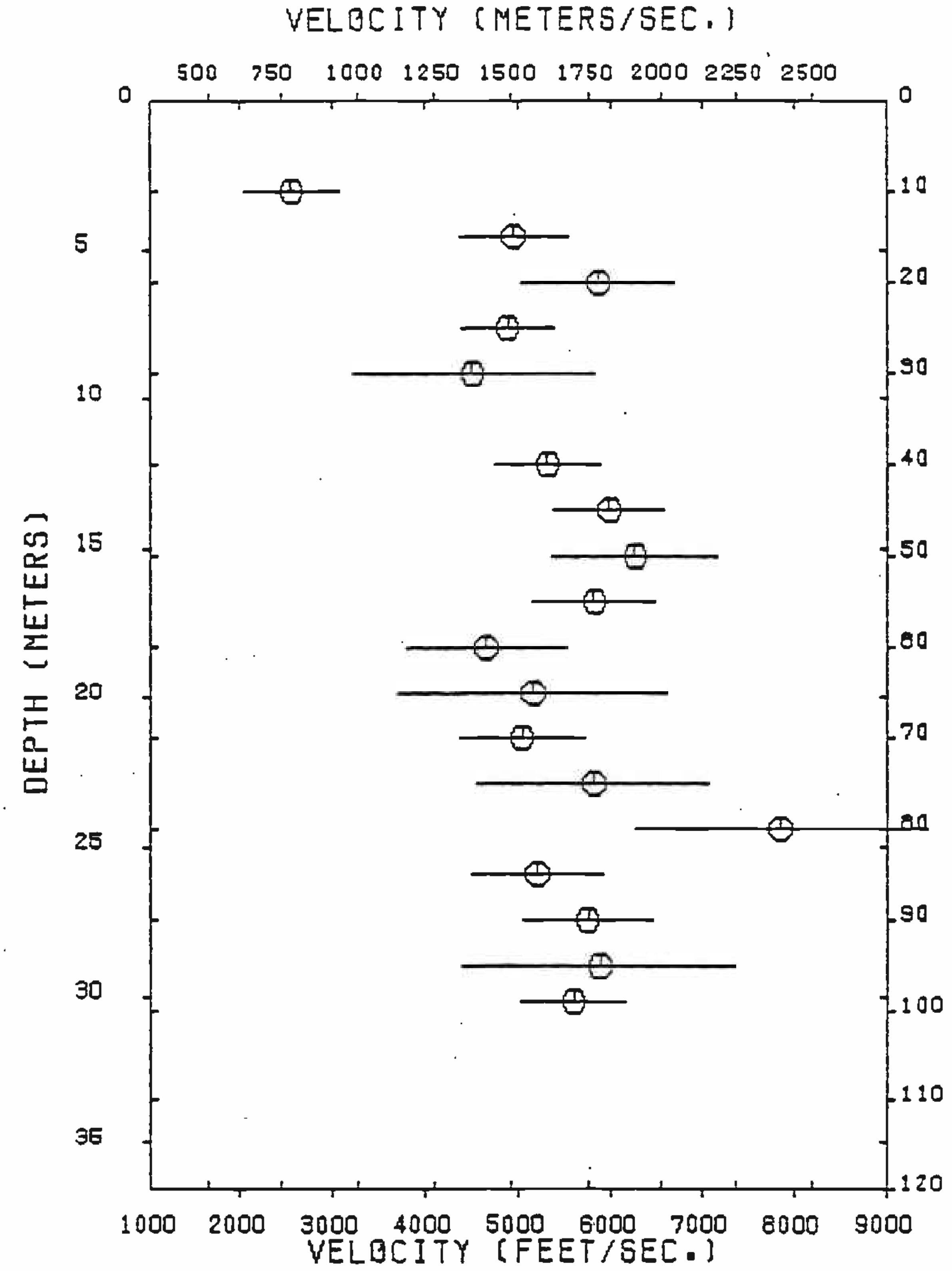

COMPRESSIONAL WAVE VELOCITY/DEPTH PROFILE-BORING SITE 5

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Figure No.
C-3

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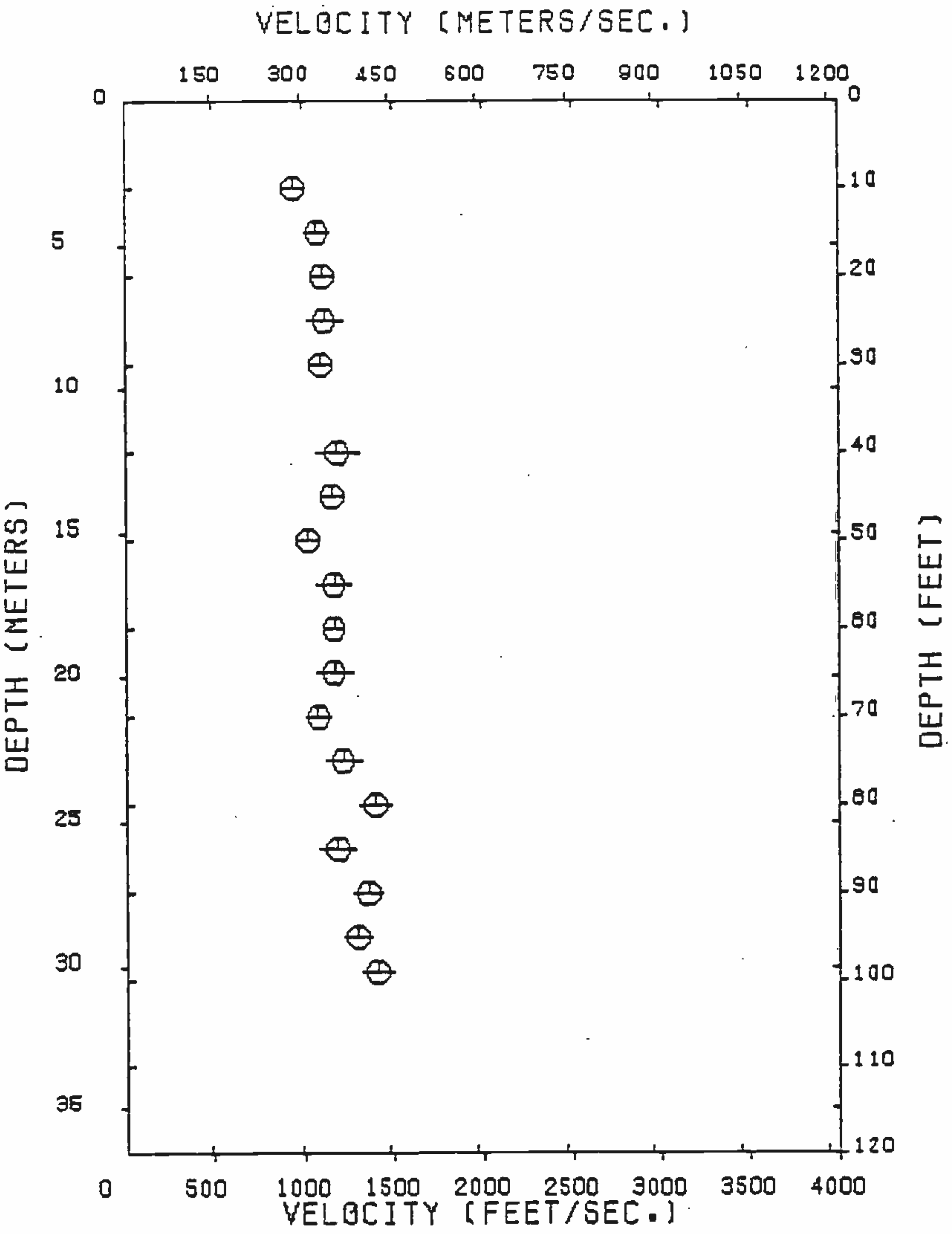

SHEAR WAVE VELOCITY/DEPTH PROFILE - BORING SITE 5

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Figure No.
C-4

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Appendix D

Gas Chromatographic Analysis

APPENDIX D GAS CHROMATOGRAPHIC ANALYSIS

D.1 INTRODUCTION

The Gas Chromatographic analysis was performed at CEG-2, which is located approximately 1400 feet northeast of the Yard and Shops site. Due to the proximity of the test hole and the Yard and Shops area, methane and other natural hydrocarbon gases may occur along the proposed Yard and Shops excavation, especially where the alignment encroaches oil fields. Certain non-hydrocarbon gases can be corrosive or result in health hazards to the miners, and these gases are also expected. These gases include hydrogen sulfide, carbon monoxide, and carbon dioxide. To provide a measure of the distribution and extent of the hazardous hydrocarbon and non-hydrocarbon gases, a program of in-situ quantitative analyses was conducted by Converse's special consultant, RYLAND-CUMMINGS, INC.

The hydrocarbon gases identified were: methane, ethane; propane; n-butane; isobutane; n-pentane, isopentane; and C_6+, undifferentiated. The non-hydrocarbon gases identified were: nitrogen; oxygen; carbon monoxide; carbon dioxide; and hydrogen sulfide.

D.2 FIELD PROGRAM

Specific hydrocarbon and non-hydrocarbon gases were collected during the 1981 investigation at shallow depths in Boring CEG-2 (see Drawing No. 5 for approximate location), located 2,000 feet east of Union Station. Samples of air were analyzed to provide an ambient base. Approximately 10 ml of gas were analyzed for each sample. All samples were analyzed in the field using an analytical gas chromatograph.

Gas Collection - Air Samples

Samples of air were collected, using a syringe specifically designed for gas chromatographic analysis. The air sample was injected into the gas chromatograph and analyzed in the field.

Gas Collection - Borehole Samples

Most of the natural hydrocarbon gases are heavier than air and must be drawn to the surface to be sampled. One gas, methane, is lighter than air; and another gas, ethane, has approximately the same density as air.

The gas in the borehole was collected through a perforated tube that was inserted into the borehole, and the gas was drawn to the surface by a vacuum pump. The vacuum pump was operated by a portable 120-volt, 1500-watt generator; the generator also supplied power to the gas chromatograph and strip chart recorder. The borehole was temporarily sealed above the level of sampling. The seal prevented contamination of air or gases from the surface.

The hole was pumped for several minutes; the air and gases wasted before a representative sample was collected for analysis. The purpose for wasting these gases was to purge the borehole of any anomalous accumulations of gas or air due to the drilling operation. After this purge, a sample of gas was collected using the special syringe, and the gas was inserted into the gas chromatograph for analysis in the field.

D.3 DESCRIPTION OF ANALYTICAL GAS CHROMATOGRAPH

The instrument used for quantitative analysis was a Carle thermal conductivity analytical gas chromatograph, Series-S, with a minimum detectability limit of 5×10^{-10} g/ml of propane at 150°C. The unit uses a built-in valve programmer that automatically actuates the correct sequence of internal switching events that are required to perform the complete analysis. Because the instrument is fully automated, errors that might be introduced during the analysis by the operator are eliminated. The gases that were detected were recorded on a strip chart; the written record is called a chromatogram. Chromatograms of the samples and a legend are attached to Appendix D.

Chromatographic System and Operation

A sample of gas is injected into the chromatograph. The injected sample is carried through the instrument by an inert gas (helium) at a constant temperature (70°C), at a constant pressure (60 psi), and at a constant flow rate (30 ml/min). The gas flows through a series of columns, or tubes, that are packed with materials that have specific adsorptive properties; these properties help to separate individual gases from the sample as it flows through the instrument. Each column is designed to separate and identify specific gases. A pressure regulator is used to assure uniform pressure to the column inlet, thereby resulting in a constant rate of flow throughout the analysis.

Depending on the complexity of the gas to be detected, the gas stream may be shunted through a series of valves that direct the gas sample into different columns containing the appropriate adsorptive materials for proper separation.

The column selectively retards the gas components according to their molecular weight and polar characteristics until the components form separate concentrations, or bands, in the carrier (helium) gas. These bands are recorded on a strip chart as a function of time.

The Chromatograph; Methods of Interpretation

The record of the gases is printed on a strip chart; the abscissa is time, and the ordinate is millivolts. The chromatogram can be used immediately to qualitatively identify the gases in the sample. Quantitative analyses require additional steps and auxiliary operations. Several different methods can be used to quantify the data; each method has advantages and disadvantages, and not every method is applicable to a particular problem.

A series of gas standards that have different, known percents of the components are allowed to flow through the instrument; the components are recorded on a strip chart. The areas and heights of the peaks are calculated for each different component and for each percent; these data are used to draw a set of graphs of percent of gas vs. peak area or peak height. These graphs provide a basis for comparison to the unknown volumes of gas sampled in the field. The procedure would be as follows: the area corresponding to a gas depicted on the field chromatogram is measured (using, for example, a compensating polar planimeter); that area can be compared to the standard to determine the volume percent of gas in the unknown sample.

To determine weight percent, the data on the field chromatogram must be normalized with respect to the total area of all components. To convert the field data to weight percent, a correction factor corresponding to the gas must be used. The correction factor is necessary because the areas on the graph corresponding to each component are not directly proportional to the percent composition. This is so because different compounds have different responses to the detector depending on the molecular weight of the gas. To determine the correction factor, the relative thermal response per mole of the gas is divided into the molecular weight.

Both the volume method and weight method were used in our analyses of the data for this project. The results of one method provide a check of the other.

D.4 RESULTS

The chromatogram for Boring CEG-2 is attached. The results of the analyses, reported as parts per million, are given in Table D-1. The reason for selecting "parts per million" to report the results is because this measure provides the most direct conversion to percent by volume; percent by volume is the basis for classifying tunnels in terms of safety (California Administrative Code, Title 8, Article 8, Section 8422). Table D-1 also identifies (1) the lower limit of flammability, (2) tunnel classification at the 5 percent and 20 percent lower explosive limit (LEL), and (3) the threshold limit values of selected non-hydrocarbon gases. These columns, abstracted from the more complete Tables D-2 and D-3 are included in Table D-1 for convenience. Table D-2 indicates the limits of flammability for the gases. Table D-3 indicates the threshold limit value (TLV) of selected non-hydrocarbon gases.

Samples Collected in Air

None of the gases detected reached a value that would be considered hazardous (Table D-1).

Hydrocarbon gases in air are not necessarily from natural sources, such as emanations from oil fields. Automobile exhaust is a major source. Exhaust from automobiles includes ethane, propane, isobutane, n-butane, isopentane, n-pentane, C_6+ (California Air Resources Board, Nov. 1980, Hydrocarbon profile of motor vehicle exhaust, 1980, Project HS-11-SHC, 4p). Hydrogen sulfide can come from either natural or industrial sources. There is no need for differentiating the sources for this project. However, they can be differentiated by studying the isotopic composition of the gases.

Methane is likely to have a natural source. Because the gas is lighter than air, it can work its way up through the rocks and soils, eventually reaching the surface. Some of the hydrogen sulfide undoubtedly has a natural source. The gas, could be smelled near some of the open boreholes and from the water pumped from the subsurface; the gas is highly soluble in water (Table D-4). During our testing, we noticed that the gas did not flow continuously out of the boreholes; rather, it came out in pulses. Detection of hydrogen sulfide by smell does not necessarily indicate a hazardous condition; the lower limit of detection can be less than 10 ppm (Table D-3), depending on the sensitivity of the individual.

Samples Collected in Boreholes

Gas samples were collected in the boreholes from levels above the uppermost perched water table or within the saturated zone of the uppermost perched water table. A sample from Boring CEG-2 was collected in a cased piezometer; perforations in the casing were within the saturated zone and the gas sampling point was above the line of the water in the cased piezometer. Field conditions did not allow for sampling of gas below the perched water table or at tunnel level or at the point of origin of the gas. Details of the sampling depth and the depth of the water at the time of sampling are given in Table D-1.

Sources of Gas

Geologic exploration for natural gas fields clearly indicates that perched ground water acts to seal the gases below the water (Masters, 1979). The water inhibits the upward migration of the gases. In some field examples discussed in Masters (1979), the gases and water are in the same permeable sandstone, and no impermeable barrier or lithology exists between the water and the gases. Although small amounts of hydrocarbon gases can be absorbed in the water, the limit of saturation for these gases is extremely low, not exceeding 65 ppm (Table D-4). Among the non-hydrocarbon gases, only carbon dioxide and hydrogen sulfide are significantly soluble (1449 ppm and 3375 ppm, respectively; Table D-4). Because only small amounts of gas can be present in the water, only small amounts can come out of the water. Thus, only a very small amount of hydrocarbon gases detected in the boreholes came from within the water. The gases can enter the water and bubble up through it if the gases are subjected to a high differential pressure. Gases can also enter the water-saturated zone and bubble up through it if the source of the gases is within the saturated zone.

A review of the lithologic logs of the boreholes along the proposed alignment indicates geologic conditions analogous to those described in Masters (1979). Direct evidence of such conditions along the alignment comes from reports of the drilling operations. The gas "sniffers" detected gas concentrations during the drilling and after the holes had been capped temporarily. The lower level of detection of the "sniffers" was above the lowest limit of sensitivity of the gas chromatograph; the chromatograph recorded levels of gas concentrations lower than that which would trigger the "sniffers." Apparently, the "sniffers" detected the pulse of the gas that was trapped below the water table when the water table was pierced by the drilling. These geologic conditions have significance along the proposed alignment because the natural gases that formed at depth and related to the oil fields are likely to be trapped below the perched water tables. The gases that accumulate along the base of the perched water would likely migrate laterally. Because the gases can migrate laterally below the perched water table, the gases may be present outside the immediate vicinity of known oil fields. The concentrations of gas would depend on the permeability of the rock and soils as well as the concentration and production of gases at the source. Consequently, gases may also be present along the alignment in areas away from the known oil fields. The gases can accumulate in pockets or zones in the soils or bedrock against faults, or against other impermeable barriers such as igneous dikes. These accumulations can be miles away from known or suspected sources.

The lateral migration of gases from their source in one oil field can cause them to mix with other gases from another oil field. A gas sample from a borehole may not provide a characteristic signature of the gases produced by the nearby oil field due to contamination related to the lateral migration of these gases.

D.5 CONCLUSIONS

The known Union Station Oil Field is located to the south of the cut and cover box structure area and the chromatogram Boring CEG-2. It's proximity, as mapped, is directly underneath the Yard and Shops area. The shallow borings drilled for the Yard and Shops investigation did not encounter any of the subsurface gas. However, Borings B-10 and B-11 drilled for the proposed Cal Trans Bridge did encounter oil tar within the samples obtained between depths of 82 and 97 feet below existing ground surface. We may expect to find subsurface gas either within the Puente Formation or trapped within the alluvium below the ground water table in the deep box excavation of the A-track and X-track alignments.

Because of the lateral migration of gases below the zones of perched water, it is likely that gases have accumulated under pressure in the stratigraphic and structural traps (e.g., faults or igneous dikes along the southern part of the Santa Monica Mountains) at distances away from the immediate areas of known oil fields. Such areas should be approached cautiously with appropriate testing of gases during the excavation of the box structures. In addition, extreme caution should be exercised whenever the excavation of the box structures approaches the area below a perched water zone, and appropriate gas testing should be done.

TABLE D-1 Summary of Data from Gas Chromatograms

GAS	Lower Limit of Flammability* (ppm)	Total Allowable Limits** [illegible] (ppm)	Total Allowable Limits** [illegible] (ppm)	1 Air (ppm)	1 [illegible] (ppm)	2 Air (ppm)	2 [illegible] (ppm)	10 Air (ppm)	10 [illegible] water level (ppm)	11 Air (ppm)	11 [illegible] Sampled @ [illegible] water level @ [illegible] (ppm)	1[illegible] Air (ppm)	1[illegible] [illegible] water level (ppm)	21 Air (ppm)	21 [illegible] (ppm)	22 Air (ppm)	22 Alluvium Open Hole Sampled [illegible] water level [illegible] (ppm)	23 Air (ppm)	23 [illegible] water level (ppm)	[illegible] Air (ppm)	[illegible] [illegible] water level (ppm)
Methane	5[illegible],000	[illegible]	1,[illegible]	-	[illegible]	-	[illegible]	-	-			-	trace	-	a trace b - c -	trace	trace	-	-	-	trace
Ethane	[illegible]	1,500	6,[illegible]	trace	[illegible]	-	[illegible]	-	-			-	[illegible]	[illegible]	a trace b trace c trace	[illegible]	[illegible]	-	-	[illegible]	[illegible]
Propane	21,200	1,[illegible]	4,[illegible]	-	trace	-	trace	-	-			-	-	-	a trace b - c -	-	-	-	-	trace	-
n-Butane	18,[illegible]	950	3,[illegible]	-	trace	-	trace	-	-			-	trace	-	a trace b - c -	-	trace		trace	[illegible]	
Isobutane	18,000	900	3,[illegible]	-	trace	-	trace	-	-			-	trace	-	a trace b - c -	-	trace	-	[illegible]	[illegible]	-
n-Pentane	14,000	700	2,800	-	trace	-	trace	-	-			-	trace	-	a trace b - c -	-	-		trace	trace	-
Isopentane	13,200	660	2,640	-	trace	-	trace	-	-			-	trace	-	a trace b - c -	-	-	-	trace	[illegible]	-
C_6+	‡	-	-	600	1,000	1,600	1,6[illegible]	450	1,200			[illegible]	4,5[illegible]	5[illegible]	a [illegible] b [illegible] c trace	[illegible]	[illegible]	5,5[illegible]	[illegible]	[illegible]	[illegible]
Nitrogen	-	-	-	772,000	771,000	770,000	773,000	773,000	773,000			775,[illegible]	[illegible]	77[illegible]	a 77[illegible],000 b 770,000 c 770,000	773,000	773,000	[illegible]	[illegible]	[illegible]	[illegible]
Oxygen	-	-	-	200,000	200,000	2[illegible]0,000	[illegible]	201,000	200,000			[illegible]	[illegible]	201,[illegible]	a 201,000 b 201,000 c 201,000	201,000	200,000	200,000	[illegible]	[illegible]	200,000
Carbon monoxide	125,000	6,250	25,000	-	trace	-	trace	-	-			trace	trace	trace	a trace b - c -	-	trace	-	trace	trace	trace
Carbon dioxide	-	-	-	27,000	28,000	28,000	[illegible]	25,[illegible]	28,000			25,000	29,000	28,000	a 28,[illegible] b 27,000 c 28,000	29,000	27,000	27,000	[illegible]	[illegible]	27,[illegible]
Hydrogen sulfide	43,000	2,175	8,700	trace	trace	-	trace	-	trace			trace	trace	trace	a trace b trace c [illegible]	trace	trace	trace	trace	trace	trace

GAS	Threshold Limit Values of Selected Non-Hydrocarbon Gases ppm §	Threshold Limit Values of Selected Non-Hydrocarbon Gases ppm §§	1 Air (ppm)	1 Cased Piezometer @ 10' (ppm)	2 Air (ppm)	2 Cased Piezometer @ 10' (ppm)	10 Air (ppm)	10 [illegible] Piezometer @ [illegible] water level @ [illegible] (ppm)	11 Air (ppm)	11 Hole (ppm)	1[illegible] Air (ppm)	1[illegible] Hole (ppm)	21 Air (ppm)	21 Hole (ppm)	22 Air (ppm)	22 Hole (ppm)	23 Air (ppm)	23 Hole (ppm)	[illegible] Air (ppm)	[illegible] Hole (ppm)
Carbon monoxide	50	100, 200, 400, 1,200, 2,000	-	trace	-	trace	-	-			trace	trace	trace	a trace b - c -	-	trace	-	trace	trace	trace
Carbon dioxide	5,000	5,000, 50,000, 90,000	27,000	28,000	25,000	25,000	25,000	25,000			2[illegible],000	[illegible]	28,000	a 28,000 b 27,000 c 28,000	26,000	27,000	27,000	[illegible]	[illegible]	[illegible]
Hydrogen sulfide	10	10, 100, 200	trace	trace	-	trace	-	trace			trace	trace	trace	a trace b trace c [illegible]	trace	trace	trace	trace	trace	trace
Oxygen	#	-	200,000	200,000	200,000	[illegible]	201,000	200,000			200,[illegible]	[illegible]	201,000	a 201,000 b 201,000 c 201,000	201,000	200,000	[illegible]	[illegible]	[illegible]	202,000

* See Table F1-2 for levels of selected gases.

** Based on information in Table F1-2, see California Administrative Code, Title 8, Article 8, Appendix B, Part 9, section 7955a, 55b, 7955c, 7955d. NOTE: samples normalized to indicate ppm. Small errors result from rounding of values.

† Less than 100 ppm.

‡ Not differentiated.

§ Title 8, California Administrative Code, General Industry Safety Order. NOTE: Nitrogen Dioxide not tested. TLV requirements, not more than 5 ppm.

§§ See Table F1-3 for details of different levels.

\# Not less than 180,000 ppm.

Notes for Boring 21 Description
a = Alluvium Open Hole Sampled @ 13'
b = Cased 3/4" Piezometer Sampled @ 15', water @ 16'
c = Cased 2" Piezometer Sampled @ 53', water @ 55'

TABLE D-2 Limits of Flammability

Gas	Formula	Limits of Flammability in Air			
		Percent by Volume*		Parts per Million	
		Lower	Upper	Lower	Upper
Methane	CH_1	5.00	15.00	50,000	150,000
Ethane	C_2H_6	3.00	12.50	30,000	125,000
Propane	C_3H_8	2.12	9.35	21,200	93,500
n-Butane	C_4H_{10}	1.86	8.41	18,600	84,100
Isobutane	C_4H_{10}	1.80	8.44	18,000	84,400
n-Pentane	C_5H_{12}	1.40	7.80	14,000	78,000
Isopentane	C_5H_{12}	1.32	-	13,200	-
Hexane**	C_6H_{14}	1.18	7.40	11,800	74,000
Heptane (C_7)	-	1.10	6.70	11,000	67,000
Octane (C_8)	-	0.95	-	9,500	-
Nonane (C_9)	-	0.83	-	8,300	-
Decane (C_{10})	-	0.77	5.35	7,700	53,000
Carbon monoxide	CO	12.50	74.20	125,000	742,000
Hydrogen sulfide	H_2S	4.30	28.50	43,000	285,000

*Handbook of Chemistry and Physics, 41st ed., p. 1927-1929.

**Instrument used in analyses combined all hydrocarbon gases, C_6 and greater, including those greater than C_{10}.

TABLE D-3 Threshold Limit Value of Selected Non-Hydrocarbon Gases

Gas	Concentration by Volume in Air* Parts per Million	Comments*
Carbon monoxide	100	Threshold limit value (TLV); no adverse effects.
	200	Headache after about 7 hours if resting; about 2 hours of work.
	400	Headache and discomfort, possibility of collapse after 2 hours at rest or 45 minutes of exertion.
	1,200	Palpitation after 30 minutes rest or 10 minutes of exertion.
	2,000	Unconsciousness after 30 minutes rest or 10 minutes of exertion.
Carbon dioxide	5,000	TLV; lung ventilation slightly increased.
	50,000	Breathing is labored.
	90,000	Depression of breathing begins.
Hydrogen sulfide	10	TLV.
	100	Irritation to eyes and throat; headache.
	200	Maximum concentration tolerable for one hour.
	1,000	Immediate unconsciousness.
Sulfur dioxide (not tested)	1 to 5	Can be detected by taste at lower level, by smell at upper level.
	5	TLV; onset or irritation to nose and throat.
	20	Irritation to eyes.
	400	Immediately dangerous to life.

*National Coal Board, 1978, Spoil Heaps and Lagoons, Technical Handbook, N.C.B., London.

TABLE D-4 Solubility of Gases in Water

Gas	Solubility in Water Parts per Million
Hydrocarbon*	
Methane	24.4 ± 1.0
Ethane	60.4 ± 1.3
Propane	6.24 ± 2.1
n-Butane	61.4 ± 2.6
Isobutane	48.9 ± 2.1
n-Pentane	38.5 ± 2.0
Isopentane	48.9 ± 1.6
(C_6)	9.5 ± 1.3
(C_7)	2.93 ± 0.20
(C_8)	0.66 ± 0.06
Non-Hydrocarbon**	
Nitrogen	17.5
Oxygen	39.3
Carbon monoxide	26.0
Carbon dioxide	1,449
Hydrogen sulfide	3,375

*McAuliffe, C., 1963, Solubility in Water of C_1 - C_9 hydrocarbons: Nature, v. 200, no. 4911, p. 1092-1093.

**Handbook of Chemistry and Physics, 41st ed., p. 1706-1707.

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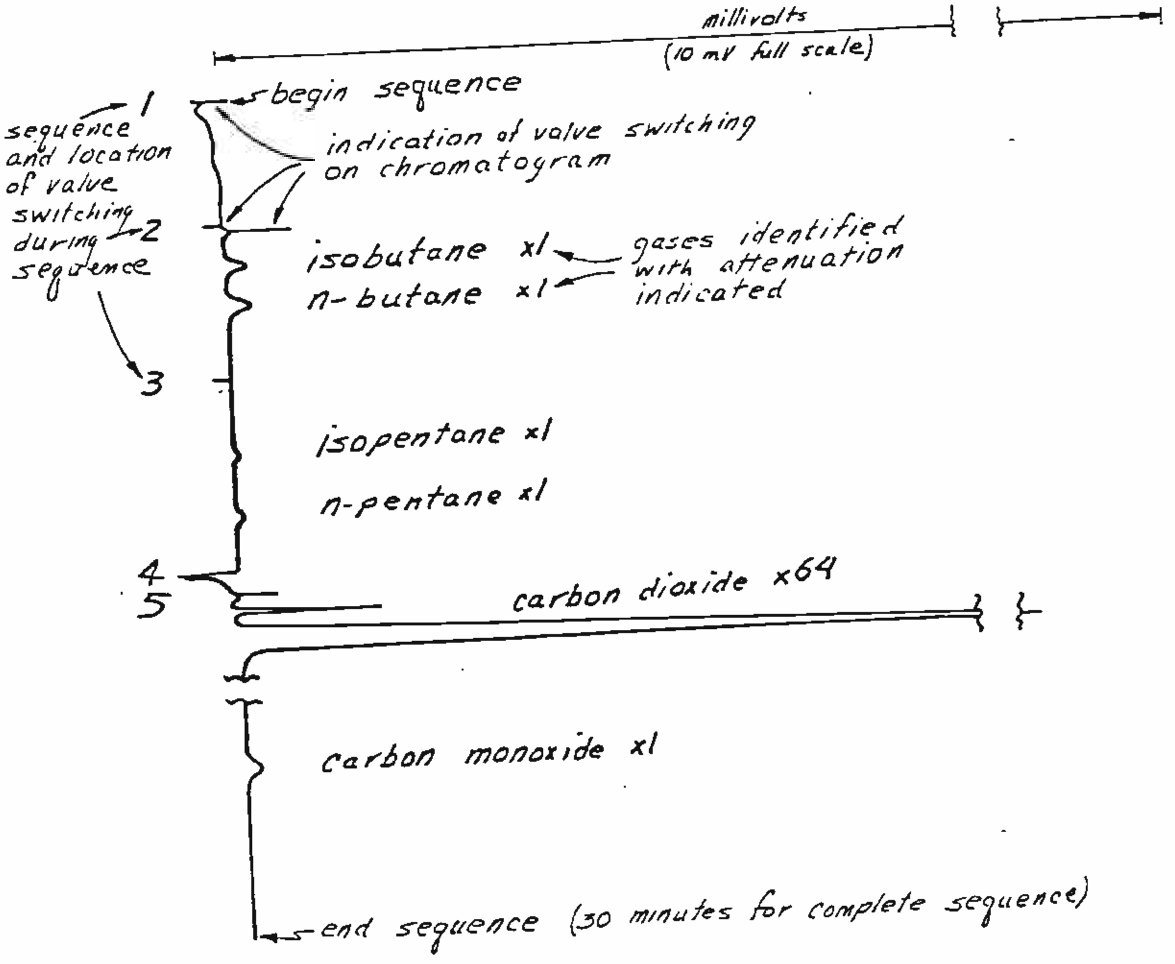
Ryland-Cummings, Inc.

Gas Chromatography
Boring No. ____

Date Sampled ____
Depth of Sample ____
Formation ____
Sample Size ____
Tested by ____
Column Temp. ____ Chart Speed ____
Helium ____
Attenuation Range ____

LEGEND



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Ryland-Cummings, Inc.

Gas Chromatography
Boring No. 1

Date Sampled 2/12/81

Depth of Sample Air

Formation

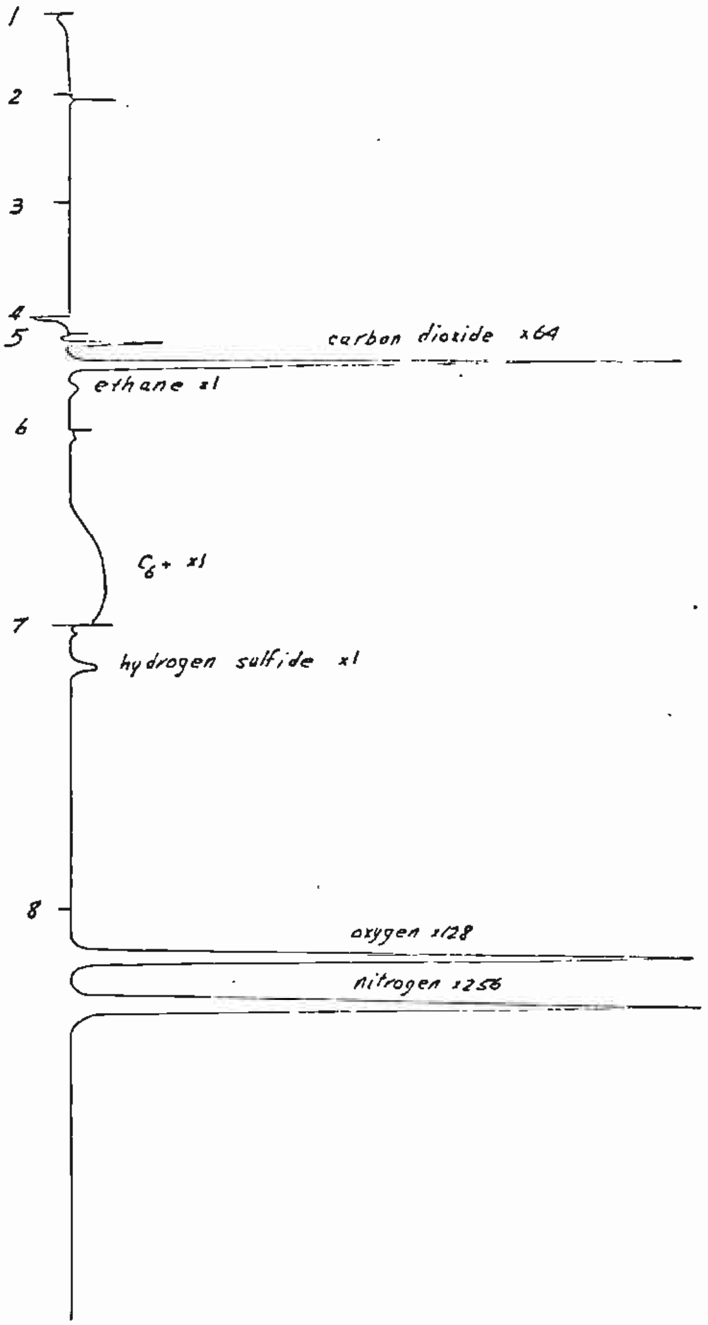
Sample Size 10 ml

Tested by DC SLR

Column Temp. 70°C Chart Speed 0.5 in/min

Helium 30 ml/min flow rate @ 60 psig

Attenuation Range as shown



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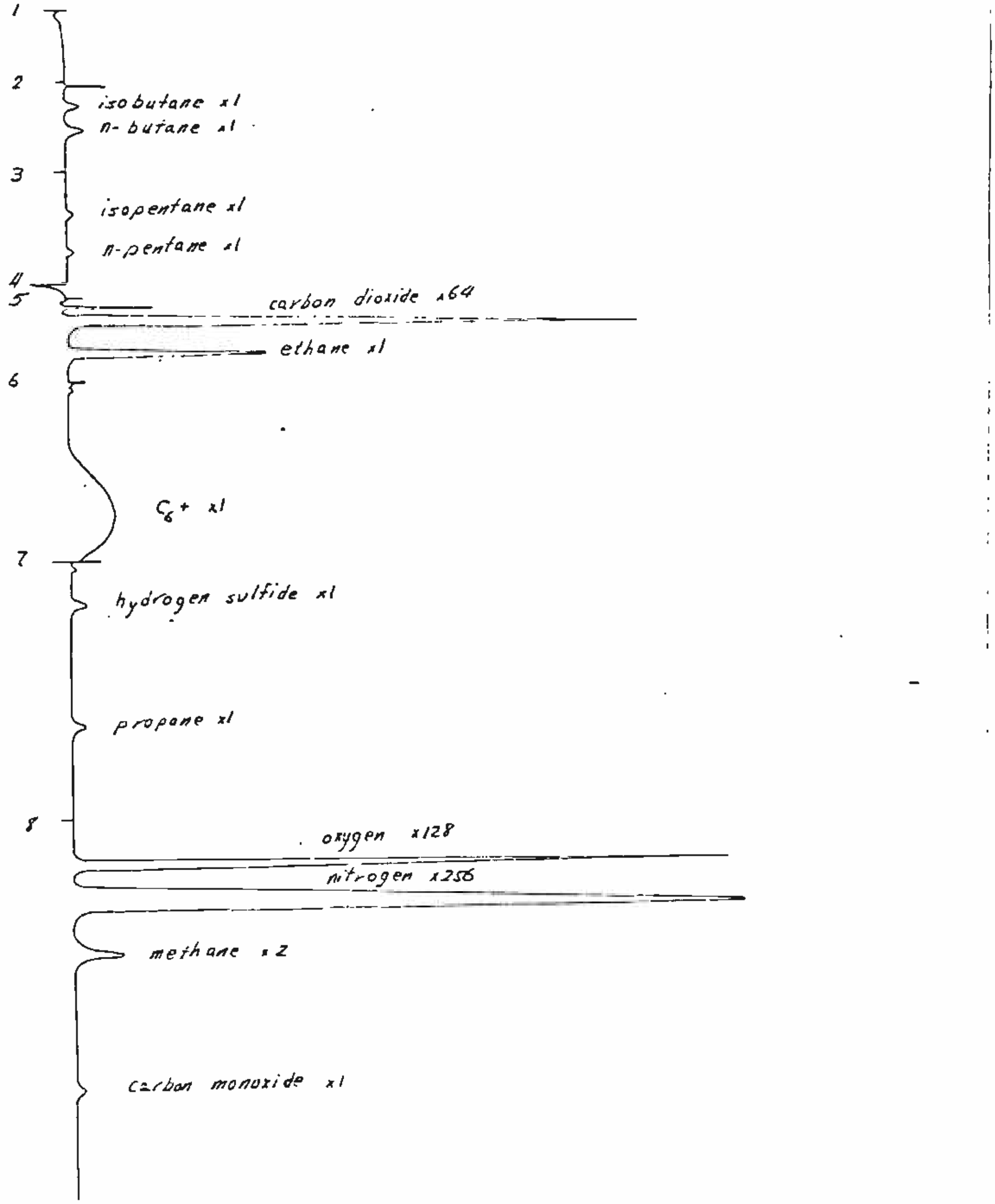
Ryland-Cummings, Inc.

Gas Chromatography
Boring No. 1

Date Sampled 2/12/81 Tested by DC SLR
Depth of Sample 10 ft water @ 10 ft Column Temp. 70°C Chart Speed 0.5 in/min
Formation cased piezometer Helium 30 ml/min flow rate @ 60 psig
Sample Size 10 ml Attenuation Range as shown



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Gas Chromatography
Boring No. 2

Date Sampled 2/14/81

Tested by DC

Depth of Sample Air

Column Temp. 70°C

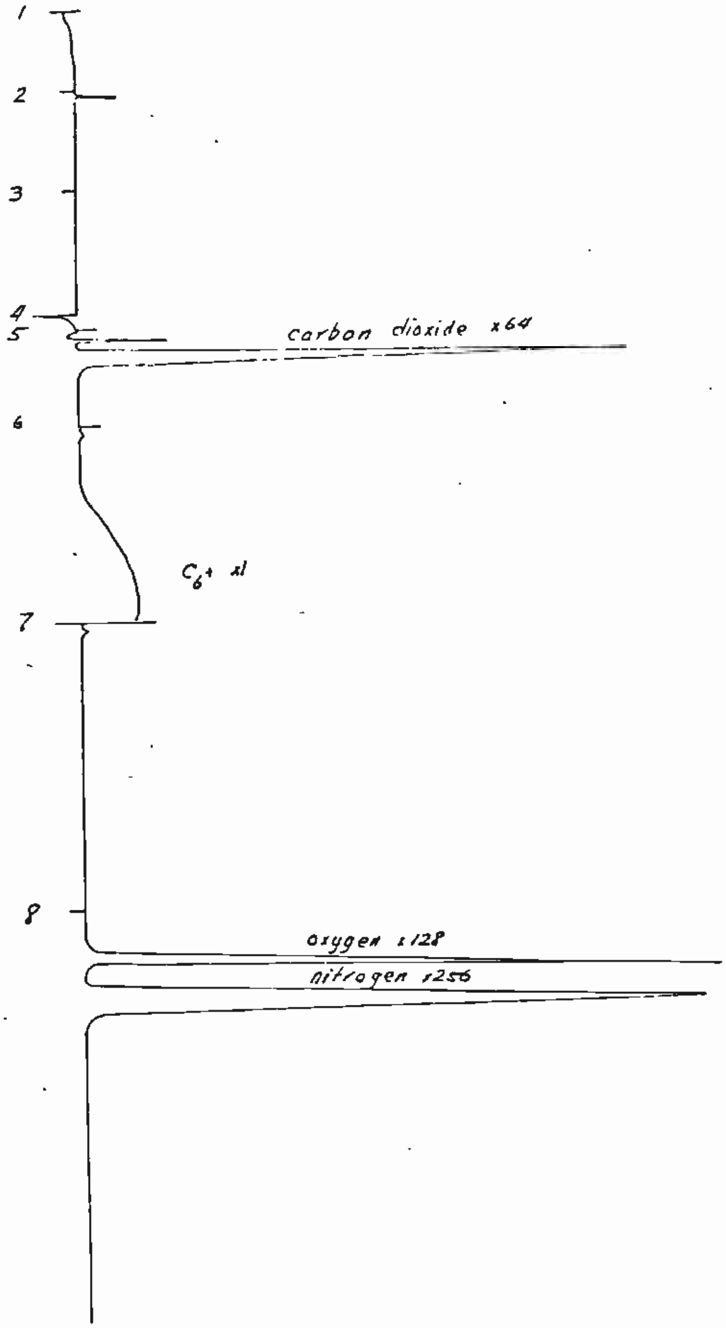
Chart Speed 0.5 in/min

Formation

Helium 30 ml/min flow rate @ 60 psig

Sample Size 10 ml

Attenuation Range as shown



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Gas Chromatography
Boring No. 2

Date Sampled 2/14/81

Depth of Sample 18 ft water @ 20 ft

Formation cased piezometer

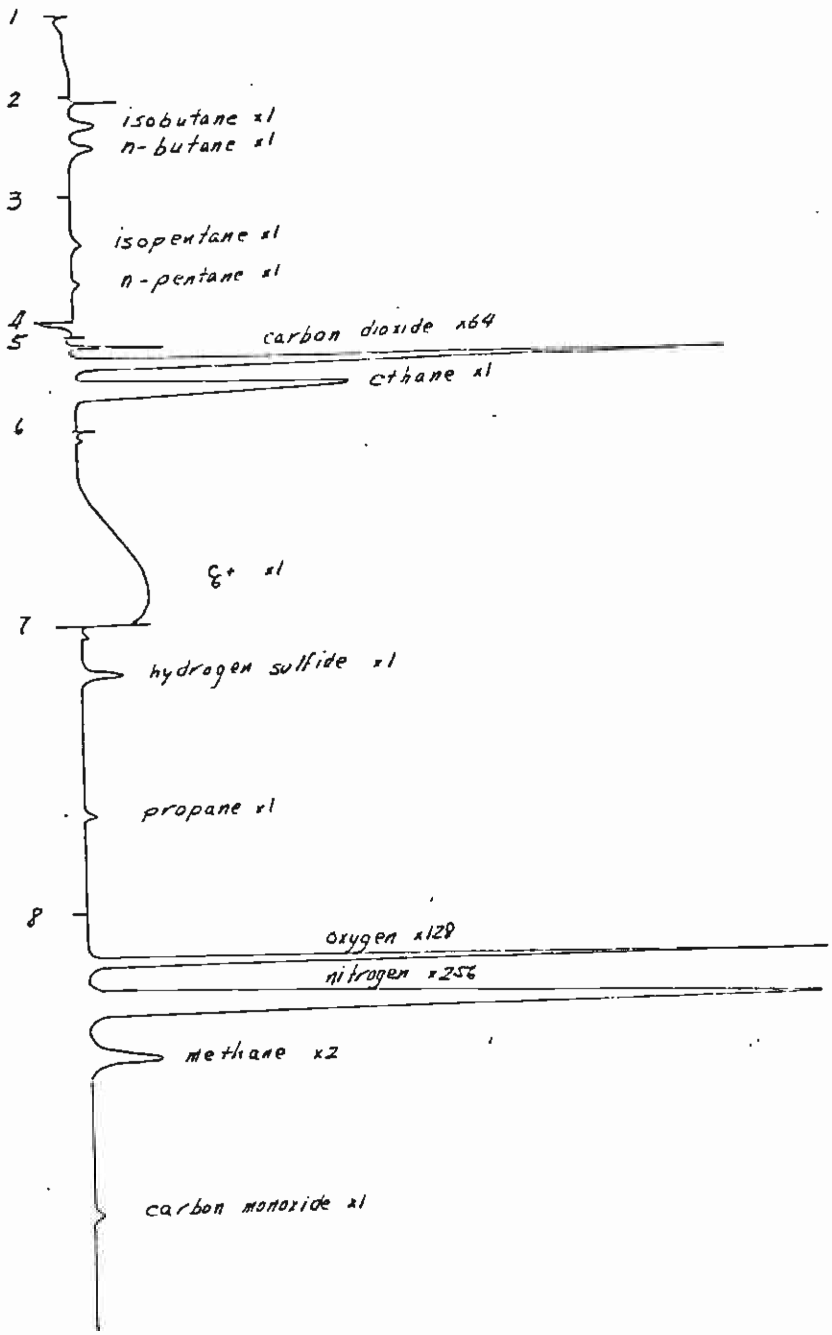
Sample Size 10 ml

Tested by DC

Column Temp. 70°C Chart Speed 0.5 in/min

Helium 30 ml/min flow rate @ 60 psig

Attenuation Range as shown



Appendix E

Water Quality Analysis

APPENDIX E WATER QUALITY ANALYSIS

E.1 RESULTS

Water samples were taken from Borings CEG-4 and 6. The purpose was to evaluate water chemicals that could have significant influence on design requirements and to identify chemical constituents for compliance with EPA requirements for cut and cover excavation dewatering activities. Chemical constituents tested are attached.

E.2 FIELD PROGRAM

The borehole was flushed and established as a piezometer. At a later date (often several weeks) the established piezometer hole was again flushed and cleaned out. Upon achieving a clean hole, water samples were collected with an air-lifting procedure from various depths within the borehole. The water samples were collected in sterilized one-quart glass containers which were properly identified and marked in the field. The water samples taken were delivered to Jacobs Laboratories for testing.

Converse Ward Davis Dixon

Lab No. P81-02-186-2

No. Samples : 7
Sampled By : Client
Brought By : Client
Date Received: 2-20-81

Sample labeled: HOLE #6-2"

Conductivity: 30,000 µ mhos/cm

Turbidity: NTU

pH 7.5 @ 25°C
pHs @ 60°F (15.6°C)
pHs @ 140°F (60°C)

	Milligrams per liter (ppm)		Milli-equivalents per liter
Cations determined:			
Calcium, Ca	1,055		52.75
Magnesium, Mg	210		17.28
Sodium, Na	6,450		280.58
Potassium, K	38		0.97
		Total	351.58
Anions determined:			
Bicarbonate, as HCO_3	230		3.77
Chloride, Cl	12,255		345.60
Sulfate, SO_4	27		0.56
Fluoride, F	0.4		0.02
Nitrate, as N	0.5		0.04
		Total	349.99
Carbon dioxide, CO_2, Calc.	10		
Hardness, as $CaCO_3$	3,500		
Silica, SiO_2	39		
Iron, Fe	0.08		
Manganese, Mn	0.64		
Boron, B	38		
Total Dissolved Minerals, (by addition: HCO_3 -> CO_3)	20,230		

Converse Ward Davis Dixon

Lab No. F81-02-186-7

No. Samples : 7
Sampled By : Client
Brought By : Client
Date Received: 2-20-81

Sample labeled: HOLE #4-2"

Conductivity: 8,450 μ mhos/cm

Turbidity: NTU

pH 7.6 @ 25°C
pHs @ 60°F (15.6°C)
pHs @ 140°F (60°C)

	Milligrams per liter (ppm)	Milli-equivalents per liter
Cations determined:		
Calcium, Ca	76	3.80
Magnesium, Mg	64	5.27
Sodium, Na	1,800	78.30
Potassium, K	18	0.46
Total		87.83
Anions determined:		
Bicarbonate, as HCO_3	404	6.62
Chloride, Cl	2,800	79.18
Sulfate, SO_4	79	1.65
Fluoride, F	0.6	0.03
Nitrate, as N	0.3	0.02
Total		87.50
Carbon dioxide, CO_2, Calc.	15	
Hardness, as $CaCO_3$	453	
Silica, SiO_2	39	
Iron, Fe	0.08	
Manganese, Mn	< 0.01	
Boron, B	7.0	
Total Dissolved Minerals, (by addition: HCO_3 -> CO_3)	5,085	

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Geo/Resource Consultants

Jacobs Laboratories

Water Quality

April 6, 1981

Converse Ward Davis Dixon
126 W. Del Mar Blvd.
P.O. Box 2268D
Pasadena, CA 91105

Lab No. P81-02-123
P81-02-142
P81-02-159
P81-02-186
P81-03-017

Attention: Buzz Spellman

Report of Chemical Analysis

The enclosed analytical results are for thirty (30) samples of ground water received by this laboratory on February 12, 17, 18, 20 and March 3, 1981. The samples were collected and delivered by Converse, Ward, Davis, Dixon personnel.

Cation/Anion balance was not acheived on many of the samples due to the presence of an unmeasured cation, probably aluminum or barium. This fact is reflected in the large difference between the milliequivalents of total hardness, (Milligrams $CaCO_3/1 \div 50$ = milliequivalents) and the summed milliequivalents of calcium and magnesium. These samples balance electrically using the total hardness in place of the calcium and magnesium. This indicates a cation (or cations) was not measured. The most common ions are aluminum and barium. If you so desired, we may analyze these samples for the missing element(s).

Respectfully submitted,

William, R. Ray
Manager, Water Laboratory

asl

Appendix F

Geotechnical Laboratory Testing

APPENDIX F GEOTECHNICAL LABORATORY TESTING

F.1 INTRODUCTION

Laboratory geotechnical tests were performed on selected soil samples obtained from the borings.

The soil tests performed may be classified into two broad categories:

- Index or identification tests which included visual classification, grain-size distribution, Atterberg Limits, moisture content, and unit weight testing;
- Engineering properties testing which included unconfined compression, triaxial compression, direct shear, consolidation, permeability, porosity, resonant column, cyclic triaxial, and dynamic triaxial tests.

The laboratory test data from the 1983 investigation are presented in Table F-2, while data from the 1981 geotechnical investigation are presented in Table F-3. The geologic units listed in these tables are described in Section 5.0 of the report.

F.1.1 Data Analysis

The summary of laboratory test results is presented in Table F-1. Figures F-1 and F-2 summarize strength and modulus data for granular alluvium existing at the site.

Data from the various tests were organized by test type and geologic unit. Where the numbers were sufficient to warrant, a statistical evaluation including averaging and computation of standard deviation was performed. The arithmetic average, or mean, was computed for each test type except for the permeability tests. The geometric mean was used for the permeability tests. The geometric mean, m_s, of a population of n samples is defined as:

$$m_s = (a_1 \times a_2 \times \ldots \times a_n)^1/n$$

Data obtained for each geological unit were summarized, averaged and evaluated for use in developing recommendations for the design unit. Test results which were considered non-representative due to sample disturbance or other factors were not reported or summarized.

F.2 INDEX AND IDENTIFICATION

F.2.1 Visual Classification

Field classification was verified in the laboratory by visual examination in accordance with the unified Soil Classification System and ASTM D-2488-69 test method. When necessary to substantiate visual classifications, tests were conducted in accordance with the ASTM D-2478-69 test method.

F.2.2 Grain-Size Distribution

Grain-size distribution tests were performed on representative samples of the geologic units to assist in the soils classification and to correlate test data between various samples. Sieve analyses were performed on that portion of the sample retained on the No. 200 sieve in accordance with ASTM D-422-63 test method. Combined sieve and hydrometer analyses were performed on selected samples which had a significant percentage of soil particles passing the No. 200 sieve. Results of these analyses are presented in the form of grain-size distribution or gradation curves on Figures F-3 through F-21.

It should be noted that the grain-size distribution tests were performed on samples secured with 2.42- and 2.87-inch ID samplers. Thus, material larger than those dimensions may be present in the natural deposits although not indicated on the gradation curves.

F.2.3 Atterberg Limits

Atterberg Limit Tests were performed on selected soil samples to evaluate their plasticity and to aid in their classification. The testing procedure was in accordance with ASTM D-423-66 and D-424-59 test methods. Test results are presented on Figures F-22 and F-23 and on Tables F-2 and F-3.

F.2.4 Moisture Content

Moisture content determinations were performed on selected soil samples to assist in their classification and to evaluate ground water location. The testing procedure was a modified version of the ASTM D-2261 test method. Test results are presented on Tables F-2 and F-3.

F.2.5 Unit Weight

Unit weight determinations were performed on selected undisturbed soil samples to assist in their classification and in the selection of samples for engineering properties testing. Samples were generally the same as those selected for moisture content determinations.

The test procedure entailed measuring specimen dimensions with a precision ruler or micrometer. Weights of the sample were than determined at natural moisture content. Total unit weight was computed directly from data obtained from the two previous steps. Dry density was calculated from the moisture content found in Section F.2.4 and the total unit weight. Results of the unit weight tests are presented as dry densities on Tables F-2 and F-3.

F.3 ENGINEERING PROPERTIES: STATIC

F.3.1 Unconfined Compression

Unconfined compression tests were performed on selected samples of cohesive soils and bedrock from the test borings for the purpose of evaluating the undrained, unconfined shear strength of the various fine-grained geologic units. The tests were performed in accordance with the ASTM D-2166 test method. Results of the unconfined compression tests are presented on Tables F-2 and F-3.

F.3.2 Triaxial Compression

Consolidated undrained and unconsolidated undrained (quick) triaxial compression tests were performed on selected undisturbed soil samples. The tests were conducted in the following manner:

F.3.2.1 Consolidated Undrained (CU) Tests

- The undisturbed test specimen was trimmed to a length to diameter ratio of approximately 2.0
- The specimen was then covered with a rubber membrane and placed in the triaxial cell.
- The triaxial cell was filled with water and pressurized, and the specimen was saturated using back-pressure.
- When saturation was complete, the specimen was consolidated at the desired effective confining pressure.
- After consolidation, an axial load was applied at a controlled rate of strain. In the case of the undrained test, flow of water from the specimen was not permitted, and the resulting pore water pressure change was measured.
- The specimen was then sheared to failure or until a maximum strain of 15% to 20% was reached.

Some of the tests were performed as progressive tests to more fully utilize the available samples. The procedure was the same as above except that, when the soil specimen approached but did not reach failure (usually to peak effective stress ratio), the axial load was removed and the specimen was consolidated at a higher confining pressure. The axial load was again applied at a constant rate of strain, and the load was removed before the specimen failed. This process was sometimes repeated a third time at a still higher confining pressure, and the sample was loaded until failure occurred.

F.3.2.2 Unconsolidated Undrained (UU), Quick (Q) Triaxial Tests

- The undisturbed test specimen was trimmed to a length to diameter ratio of approximately 2.0.
- The specimen was then covered with a rubber membrane and placed in the triaxial cell.
- The triaxial cell was filled with water and pressurized.
- The specimen was loaded at a controlled rate of strain to failure, or to 15% to 20% axial strain. The specimen was neither saturated nor consolidated prior to loading.

Results of the triaxial compression tests are presented in Figures F-24 through F-27.

F.3.3 Direct Shear

Direct shear tests were performed on selected undisturbed soil samples using a constant strain rate direct shear machine.

Each test specimen was trimmed, soaked and placed in the shear machine, a specified normal load was applied, and the specimen was sheared until a maximum shear strength was developed. Fine-grained samples were allowed to consolidate prior to shearing. The maximum developed shear strengths are summarized on Tables F-2 and F-3.

Progressive direct shear tests were performed on selected undisturbed samples of coarse-grained material. After the soil specimen had developed maximum shear resistance under the first normal load, the normal load was removed and the specimen was pushed back to its original undeformed configuration. A new normal load was then applied, and the specimen was sheared a second time. This process was repeated for several different normal loads. Results of the progressive direct shear tests are summarized on Tables F-2 and F-3.

F.3.4 Swell

No swell tests were performed for this design unit.

F.3.5 Consolidation

Consolidation tests were performed on selected undisturbed soil samples. Test specimens were 1-inch thick by 2.42-inch diameter and were obtained either directly from a Converse ring sample or by trimming a 3-inch diameter Shelby tube sample.

Apparatus used for the consolidation test is designed to receive the 1 inch high brass rings directly. Porous stones were placed in contact with both sides of the specimens to permit ready addition or release of water. Loads were applied to the test specimens in several increments, and the resulting settlements recorded.

Results of consolidation tests on the undisturbed samples are presented on Figures F-28 and F-33.

F.3.6 Permeability

Permeability tests were performed on undisturbed specimens selected for testing, or in conjunction with the static and cyclic triaxial tests, using the same selected undisturbed samples of soil. Permeability was measured during back-pressure saturation by applying a differential pressure to the ends of the sample and measuring the resulting flow. Results of the tests are tabulated on Tables F-2 and F-3.

F.3.7 Porosity

Porosity, or void ratio, of selected undisturbed samples was determined by measuring the dry unit weight and specific gravity, then calculating the void ratio, e, and porosity, n, using the following formula:

$$e = \frac{1-Vs}{Vs} \text{ where } Vs = \frac{\gamma_d}{G \times \gamma_w} \text{ and } n = \frac{e}{1+e}$$

Where:

γ_w = unit weight of water
γ_d = unit dry weight of soil
G = specific gravity of soil solids.

In some cases, an assumed average value for the specific gravity, based on the measured values for other specimens, was used for the calculation.

F.4 ENGINEERING PROPERTIES: DYNAMIC

F.4.1 Dynamic Triaxial Compression

This test evolved from the static triaxial procedure and is designed to evaluate the stress-strain properties of the soils under dynamic loading conditions. This test differs from the cyclic triaxial test in that it is designed to obtain dynamic stress-strain data at various strain levels, while the cyclic test measures deformation and liquefaction susceptibility at a given level of cyclic stress. Shear strain data is obtained generally in the range of 10^{-4} to 10^{-2} inch/inch.

F.4.1.1 Sample Preparation and Handling

These tests were performed on undisturbed cylindrical samples obtained from rotary borings using a sampler lined with either brass rings or Shelby tubes. Samples from the brass rings were 2.42 inches in diameter by 5 inches in length; those from the Shelby tubes were 2.87 inches in diameter by 6 inches in length. The samples were extruded, weighed and placed in the test cell.

F.4.1.2 Test Conditions and Parameters

Test conditions and parameters may vary in the dynamic triaxial test. The procedures followed for this project were:

- Stress controlled: After specimen preparation, the specimens were loaded cyclically at several levels of cyclic stress. Generally, one or two cycles of a relatively low stress were applied, the specimen was reconsolidated and loaded again for one or two additional cycles at a slightly higher stress level. This procedure was repeated until the resulting strain levels became large enough to cause significant permanent strain, precluding further satisfactory data (strain of about 10^{-2} inch/inch or until the maximum cycle stress level possible with the procedure was reached, corresponding to $\sigma_{cyclic}/2\sigma_{3c}$ = 0.5.

- Saturation: The specimens were artificially saturated using flushing and back pressure techniques. Typical back pressures of 60 to 100 psi were required to saturate the specimens. The degree of saturation was measured using Skempton's B parameter, $\Delta u/\Delta\sigma_{3c}$. A minimum value of B = 0.95 was obtained for all test specimens which were saturated.

- A few of the test specimens were tested in their in situ moisture condition, without artificial saturation, in order to evaluate the stress-strain properties of unsaturated samples. The tests which were not saturated are identified on the figures.

- Consolidation: Specimens were allowed to consolidate under the specified static ambient stress levels. Consolidation was monitored either by measuring specimen volume changes or by closing the drainage lines and verifying that buildup of pore pressures did not occur. A consolidation ratio ($K_c = \sigma_{1c}/\sigma_{3c}$) of 1.0 was used for this program.

- Waveform and Frequency: A sinusoidal waveform at a freauency of 0.5Hz was used for this test program.

F.4.1.3 Apparatus

The apparatus described in Section F.4.2.3 was used for this test. In addition, for the dynamic triaxial tests, an x-y flatbed recorder was utilized to record the hysteretic stress stain curve for each load cycle.

F.4.1.4 Data Reduction

The following methods and definitions were employed in the reduction of test data from the dynamic triaxial tests.

- Axial stress: Given in terms of axial load and the unconsolidated specimen crosssectional area.

- Axial strain: Given in terms of the consolidated specimen length.

- Dynamic axial strain: The peak-to-peak axial strain for any given loading cycle.

- Shear modulus and shear strain conversion: Axial stress, axial strain and Young's modulus, E, were converted to equivalent shear stress, shear strain and shear modulus, G, using a Poisson's ratio of 0.5 (undrained, zero volume change condition) for tests on saturated samples, and an assumed Poisson's ratio of 0.40 for tests on saturated specimens tested at their in situ moisture contents. Shear strain values are the strains on a plane located at 45° to the principal stress plane, which has been shown to be the plane of maximum shear strain during triaxial loading.

- Modulus: Shear modulus values are defined as the equivalent linear modulus corresponding to the straight line connecting the end points of the hysteresis loop of each loading cycle.
- Shear strain: Shear strain values given are the maximum shear strains between the end points of the hysteresis loop for a given cycle. The maximum shear strain is calculated according to the equations of solid body mechanics as 1.5 x the maximum axial strain.

Results of the dynamic triaxial tests are presented in Figures F-34 and F-37.

TABLE F-1 SUMMARY OF LABORATORY TEST RESULTS

	TOTAL DENSITY (pcf)	DRY DENSITY (pcf)	MOISTURE CONTENT (%)	UNCONFINED COMPRESSIVE STRENGTH (ksf)	DIRECT SHEAR TEST		TRIAXIAL TEST				YOUNG MODULUS E/σ_{3c} (ksf/ksf)	PERMEABILITY (cm/sec)
					PEAK SHEAR STRENGTH		PEAK UNDRAINED STRENGTH		PEAK EFFECTIVE STRENGTH			
					ϕ (deg)	c (ksf)	ϕ (deg)	c (ksf)	ϕ (deg)	c (ksf)		
COARSE-GRAINED ALLUVIUM												
NUMBER OF TESTS:	65	65	65	0	17 DS		0		4 TX		32 C + 9TX	3
HIGH	146	136	28	-	44	0.45	-	-	44	13	750	8.7×10^{-4}
LOW	96	84	2	-	22	0.02	-	-	36.5	0	210	2.1×10^{-4}
MEAN	122	111	10	-	36	0.21	-	-	38.6	5	385	6.4×10^{-4}
STANDARD DEVIATION	±12.8	±11.8	±5.7	-	-	-	-	-	-	-	-	-
CLAYSTONE/SILTSTONE												
NUMBER OF TESTS:	21	21	21	7	0 DS		0 TX		0 TX		0 C + 0 TX	0
HIGH	126	107	48	139								
LOW	101	73	18	13.8								
MEAN	117	91	29	46.5								
STANDARD DEVIATION	±5.4	±8.2	±6.5	±43.3								

TABLE F-2 LABORATORY TEST DATA

BORING No.	SAMPLE No.	DEPTH (ft)	VISUAL CLASSIFICATION	GEOLOGIC UNIT	DRY DENSITY (pcf)	MOISTURE CONTENT (%)	ATTERBERG LIMITS LL	ATTERBERG LIMITS PI	Kv, COEFFICIENT OF PERMEABILITY (cm/sec) (Confining Pressure, psi)	UNCONFINED COMPRESSIVE STRENGTH (ksf)	DIRECT SHEAR STRENGTH ENVELOPE φ, deg	DIRECT SHEAR STRENGTH ENVELOPE c, ksf	ONE-DIMENSIONAL SWELL (%) (Normal Load, ksf)	SWELL PRESSURE (ksf)	SIEVE ANALYSIS	HYDROMETER ANALYSIS	OEDOMETER	TRIAXIAL COMPRESSION (Stages)
3-1	C-1	5	Silty Sand	A_1	136	7					44	0.18						
	C-2	15	Silty Sand	A_1	96	26												
	C-3	25	Silty Sand	A_1	109	12												
	C-4	32	Gravelly Sand	A_1	110	13			$8.4x10^{-4}$(25)						X			
	C-5	41	Gravelly Sand	A_1	109	10												
	C-6	46	-		---	--												
	J-3	24	Gravelly Sand	A_1	---	--									X			
3-2	C-1	5	Silty Sand	A_1	91	7												
	C-2	15	Silty Sand, Trace of Gravel	A_1	119	13									X			X(2)
	C-3	25	Gravelly Sand	A_1	118	10												
	C-4	30	Gravelly Sand	A_1	121	12												
	C-5	35	Sandy Gravel	A_1	124	9												
3-3	C-1	11	Sandy Gravel	A_1	132	5					59*	0.38*			X			
	C-2	46	Sandy Gravel	A_1	128	8												
	J-1	7	Gravelly Sand	A_1	---	--									X			
	J-3	23	Gravelly Sand	A_1	---	--									X			

*Gravel in shear plane.

TABLE F-2 LABORATORY TEST DATA

BORING No.	SAMPLE No.	DEPTH (ft)	VISUAL CLASSIFICATION	GEOLOGIC UNIT	DRY DENSITY (pcf)	MOISTURE CONTENT (%)	ATTERBERG LIMITS LL	ATTERBERG LIMITS PI	Kv, COEFFICIENT OF PERMEABILITY (cm/sec) (Confining Pressure, psi)	UNCONFINED COMPRESSIVE STRENGTH (ksf)	DIRECT SHEAR STRENGTH ENVELOPE ø, deg	DIRECT SHEAR STRENGTH ENVELOPE c, ksf	ONE-DIMENSIONAL SWELL (%) (Normal Load, ksf)	SWELL PRESSURE (ksf)	SIEVE ANALYSIS	HYDROMETER ANALYSIS	OEDOMETER	TRIAXIAL COMPRESSION (Stages)
3-4	C-1	11	Sandy Gravel	A_1	124	5					†	†						
	C-2	21	Sandy Gravel	A_1	113	9												
	C-3	31	Sandy Gravel	A_1	127	7												
	J-1	7	Gravelly Sand	A_1	---	--									X			
	J-2	17	Gravelly Sand	A_1	---	--									X			
	J-3	27	Gravelly Sand	A_1	---	--									X			
3-5	C-1	11	Sandy Gravel	A_1	126	10					44*	0.23*						
	C-2	21	Gravelly Sand	A_1	---	7												
	C-3	26	Silty Gravelly Sand	A_1	122	11												
3-6	C-1	21	-		---	--												
	J-1	7	Gravelly Sand	A_1	---	--									X			
	J-2	12	Gravelly Sand	A_1	---	--									X			
	J-4	26	Gravelly Sand	A_1	---	--									X			
3-7	J-1	7	Gravelly Sand	A_1	---	--									X			
	J-3	16	Gravelly Sand	A_1	---	--									X			

†Too gravelly to perform test.

*Gravel in shear plane.

TABLE F-2 LABORATORY TEST DATA

BORING No.	SAMPLE No.	DEPTH (ft)	VISUAL CLASSIFICATION	GEOLOGIC UNIT	DRY DENSITY (pcf)	MOISTURE CONTENT (%)	ATTERBERG LIMITS LL	ATTERBERG LIMITS PI	Kv, COEFFICIENT OF PERMEABILITY (cm/sec) (Confining Pressure, psi)	UNCONFINED COMPRESSIVE STRENGTH (ksf)	DIRECT SHEAR STRENGTH ENVELOPE ϕ, deg	DIRECT SHEAR STRENGTH ENVELOPE c, ksf	ONE-DIMENSIONAL SWELL (%) (Normal Load, ksf)	SWELL PRESSURE (ksf)	SIEVE ANALYSIS	HYDROMETER ANALYSIS	OEDOMETER	TRIAXIAL COMPRESSION (Stages)
3-8	C-1	11	Silty Sand	A_1	104	14					31	0.15						
	J-1	7	Sand	A_1	---	--									X			
	J-3	17	Gravelly Sand	A_1	---	--									X			
3-9	C-1	2	Sand	A_1	95	4									X			
	C-2	6	Sand	A_1	96	7					33	0.08					X	
	C-3	10	Gravelly Sand	A_1	112	3					44	0.12					X	
	C-4	20	Silty Gravelly Sand	A_1	106	18												
	C-5	25	Sandy Silt	A_1	98	25					32	0.28			X			
	C-6	29	Gravelly Sand	A_1	112	10												
3-10	C-1	21	Gravelly Sand	A_1	123	10											X	
	J-1	6	Sand	A_1	---	--									X			
3-11	C-1	21	Silty Gravelly Sand	A_1	115	14												
	C-2	36	Silty Sand	A_1	105	18												
	J-1	6	Gravelly Sand	A_1	---	--									X			
3-12	C-1	11	Sand	A_1	94	6					34	0.20			X		X	
	J-1	7	Silty Sand	A_1	---	--									X			

TABLE F-2 LABORATORY TEST DATA

BORING No.	SAMPLE No.	DEPTH (ft)	VISUAL CLASSIFICATION	GEOLOGIC UNIT	DRY DENSITY (pcf)	MOISTURE CONTENT (%)	ATTERBERG LIMITS		Kv, COEFFICIENT OF PERMEABILITY (cm/sec) (Confining Pressure, psi)	UNCONFINED COMPRESSIVE STRENGTH (ksf)	DIRECT SHEAR STRENGTH ENVELOPE		ONE-DIMENSIONAL SWELL (%) (Normal Load, ksf)	SWELL PRESSURE (ksf)	SIEVE ANALYSIS	HYDROMETER ANALYSIS	OEDOMETER	TRIAXIAL COMPRESSION (Stages)
							LL	PI			ϕ, deg	c, ksf						
3-13	C-1	11	Sand	A_1	113	5									X			X(2)
	C-2	31	Sandy Gravel	A_1	130	8												
	J-1	7	Gravelly Sand	A_1	---	--									X			
3-14	C-1	11	Gravelly Sand	A_1	120	5					38	0.45			X		X	
	C-2	31	Gravelly Sand	A_1	122	10					48	0.40			X		X	
	J-1	7	Silty Gravelly Sand	A_1	---	--									X			
	J-2	17	Gravelly Sand	A_1	---	--									X			
3-15	C-1	6	Sand	A_1	100	4					31	0.25					X	
	C-2	10	Silty Sand	A_1	113	3												
	C-3	20	Gravelly Sand	A_1	115	12												
	C-4	30	Sandy Gravel	A_1	130	9									X			
	C-5	25	Sandy Silt	A_1	98	25												
3-16	C-1	11	Gravelly Sand	A_1	120	5					39	0.26						
	C-2	20	-	--	---	--												
3-17	C-1	11	Silty Gravelly Sand	A_1	118	11												
	J-1	6	Gravelly Sand	A_1	---	--									X			

TABLE F-2 LABORATORY TEST DATA

BORING No.	SAMPLE No.	DEPTH (ft)	VISUAL CLASSIFICATION	GEOLOGIC UNIT	DRY DENSITY (pcf)	MOISTURE CONTENT (%)	ATTERBERG LIMITS		Kv, COEFFICIENT OF PERMEABILITY (cm/sec) (Confining Pressure, psi)	UNCONFINED COMPRESSIVE STRENGTH (ksf)	DIRECT SHEAR STRENGTH ENVELOPE		ONE-DIMENSIONAL SWELL (%) (Normal Load, ksf)	SWELL PRESSURE (ksf)	SIEVE ANALYSIS	HYDROMETER ANALYSIS	OEDOMETER	TRIAXIAL COMPRESSION (Stages)
							LL	PI			ø, deg	c, ksf						
3-18	C-1	11	Sand	A_1	98	8									X			X(2)
	C-2	30	Silty Sand	A_1	113	11												
	J-1	7	Silty Sand	A_1	---	--									X			
3-19	C-1	11	Sand	A_1	98	11					33	0.18					X	
	C-2	30	Gravelly Sand	A_1	125	12												
	J-1	6	Sandy Silt	A_1	---	--									X			
3-20	J-1	7	Sand	A_1	---	--									X			
	J-2	17	Gravelly Sand	A_1	---	--									X			
3-21	C-1	30	Silty Sand	A_1	109	10												
	J-1	6	Sandy Silt	A_1	---	--									X			
	J-2	17	Gravelly Sand	A_1	---	--									X			
3-22	C-1	30	Silty Gravelly Sand	A_1	113	9												
	J-1	7	Sandy Silt	A_1	---	--									X			
	J-2	17	Gravelly Sand	A_1	---	--									X			
3-23	C-1	11	Gravelly Sand	A_1	108	12					39	0.02			X		X	
	J-1	17	Gravelly Sand	A_1	---	--									X			

TABLE F-2 LABORATORY TEST DATA

BORING No.	SAMPLE No.	DEPTH (ft)	VISUAL CLASSIFICATION	GEOLOGIC UNIT	DRY DENSITY (pcf)	MOISTURE CONTENT (%)	ATTERBERG LIMITS LL	ATTERBERG LIMITS PI	Kv, COEFFICIENT OF PERMEABILITY (cm/sec) (Confining Pressure, psi)	UNCONFINED COMPRESSIVE STRENGTH (ksf)	DIRECT SHEAR STRENGTH ENVELOPE ϕ, deg	DIRECT SHEAR STRENGTH ENVELOPE c, ksf	ONE-DIMENSIONAL SWELL (%) (Normal Load, ksf)	SWELL PRESSURE (ksf)	SIEVE ANALYSIS	HYDROMETER ANALYSIS	OEDOMETER	TRIAXIAL COMPRESSION (Stages)
3-24	C-1	2	Sandy Silt	A_1	96	11												
	C-2	6	Silty Sand	A_1	84	28												
	C-3	10	Sandy Gravel	A_1	112	3					35*	0.42*					X	
	C-4	14	Gravelly Sand	A_1	107	16									X			
	C-5	20	Silty Gravelly Sand	A_1	128	8												
	C-6	25	Sand	A_1	106	13									X			
	C-7	30	Gravelly Sand	A_1	118	13												
3-25	C-1	9	Sand	A_1	106	4												
	J-1	7	Sand	A_1	---	--									X			
3-26	C-1	9	Gravelly Sand	A_1	110	6					33	0.15			X			
3-27	C-1	6	Silty Gravelly Sand	A_1	96	9									X			
	C-2	10	Silty Sand	A_1	90	7												
3-28	C-1	5	Sand	A_1	102	3									X		X	
	C-2	10	Sand	A_1	100	5												
3-29	C-1	5	Sand	A_1	108	4									X			
	C-2	10	Sand	A_1	106	3												

*Gravel in shear plane.

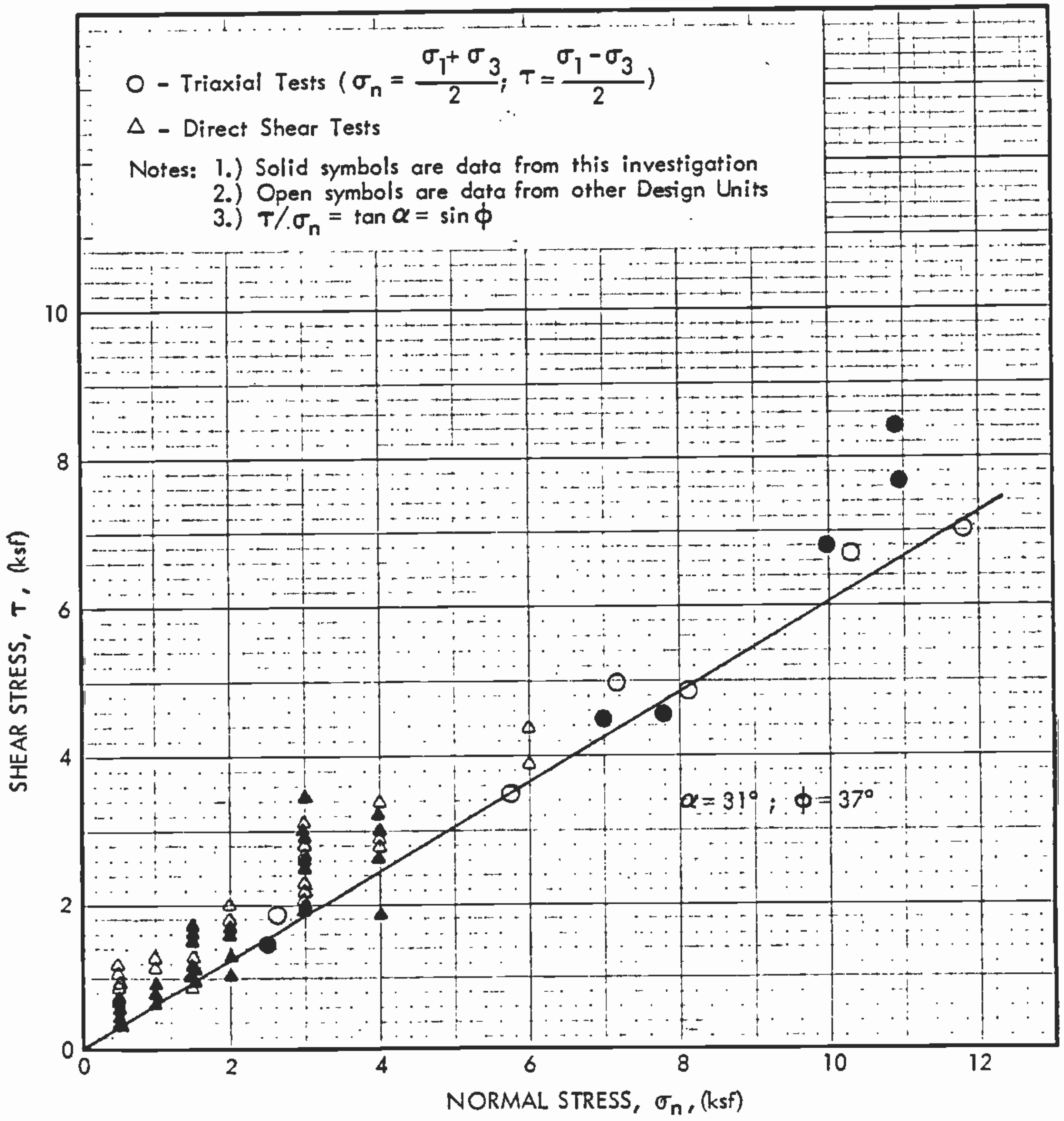
TABLE F-2 LABORATORY TEST DATA

BORING No.	SAMPLE No.	DEPTH (ft)	VISUAL CLASSIFICATION	GEOLOGIC UNIT	DRY DENSITY (pcf)	MOISTURE CONTENT (%)	ATTERBERG LIMITS		Kv, COEFFICIENT OF PERMEABILITY (cm/sec) (Confining Pressure, psi)	UNCONFINED COMPRESSIVE STRENGTH (ksf)	DIRECT SHEAR STRENGTH ENVELOPE		ONE-DIMENSIONAL SWELL (%) (Normal Load, ksf)	SWELL PRESSURE (ksf)	SIEVE ANALYSIS	HYDROMETER ANALYSIS	OEDOMETER	TRIAXIAL COMPRESSION (Stages)
							LL	PI			φ, deg	c, ksf						
3-30	C-1	5	Sandy Gravel	A_1	110	2					34	0.04			X			
	C-2	10	Silty Sand	A_1	106	21												
B-10	C-6	56	Silty Sand	A_1	91	11			$5.2x10^{-11}$(15)‡		22	0.43						
	C-7	76	Silty Sand	A_1	118	13			$8.7x10^{-4}$ (15)						X			X(3)
	C-8	101	Clayey Siltstone	C	96	25	52	18		21.6					X	X		
	C-9	103	Clayey Siltstone	C	93	26												
	C-10	107	Clayey Siltstone	C	97	25	52	18		27.4					X	X		
	J-1	62	Sand & Gravel	A_1		10									X			
	J-3	82	Silty Sand & Gravel	A_1		16									X			
B-11	C-5	103	Clayey Siltstone	C	93	28	52	18		13.8					X	X		
	C-6	104	Clayey Siltstone	C	93	27	51	17		29.5					X	X		
	J-5	72	Sand & Gravel	A_1											X			

‡Sample was impermeable hard silt.

TABLE F-3 COMPREHENSIVE LIST OF SOILS ENGINEERING PROPERTIES FROM 1981 LABORATORY TESTS

CEG Boring No.	Sample No.	Depth (ft)	Visual Classification	Geologic Unit	Dry Density (pcf)	Moisture Content (%)	Atterberg Limits		Particle Size Cumulative % Passing Sieve No.			Unconfined Compression Strength (psi)	Kv, Coefficient of Permeability (cm/sec)	Coefficient of Consolidation Cc (in/in Per Log Cycle)	Specific Gravity	Porosity (n)	Undrained Quick Direct Shear		One-Dimensional Swell % Swell (ksf Normal Load)	Cyclic Triaxial (Liquefaction)	Dynamic Triaxial (Stress/Strain)	Resonant Column Test	Triaxial Compression Test
							LL	PI	4	40	200						φ, deg	c, ksf					
2	S1	18	Silty claystone	C	92	25																	
	S2	28	Clayey siltstone	C	73	38															X		
	S2	29	Silty claystone, folded	C	73	48																	
	S3	34	Clayey siltstone	C	91	27																	
	S4	51	Bedded siltstone & claystone	C	90	29																	
	S5	67	Bedded siltstone & claystone	C	89	30	49	13															
	S6	85	Bedded siltstone & claystone	C	92	28																	
	S7	98	Bedded siltstone & claystone	C	100	21	46	11						.034									
3	C4	79	Granitic fine to coarse sand	A_1	116	13							2.1E-4			31.2	41	0.14					
	S1	104	Clayey siltstone	C	89	32																	
	S2	124	Silty claystone, bedded	C	84	35																	
	S3	140	Silty claystone, massive	C	89	30															X		
	S3	141	Silty claystone, massive	C	90	31	69	38															
	S4	149	Silty claystone, massive	C	88	32																	
4	S1	118	Clayey siltstone, massive	C	89	31						60.5											
	S2	135	Silty claystone, massive	C	107	18						139											
	S3	148	Silty claystone, massive	C	105	20	49	15				34.1											
5	PB3	90	Claystone	C			49	18															
	PB15	119	Micaceous claystone	C			40	13															
	PB26	148	Claystone, bedded	C			40	13															



SUMMARY OF EFFECTIVE STRENGTH DATA - GRANULAR ALLUVIUM

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Figure No.
F-1

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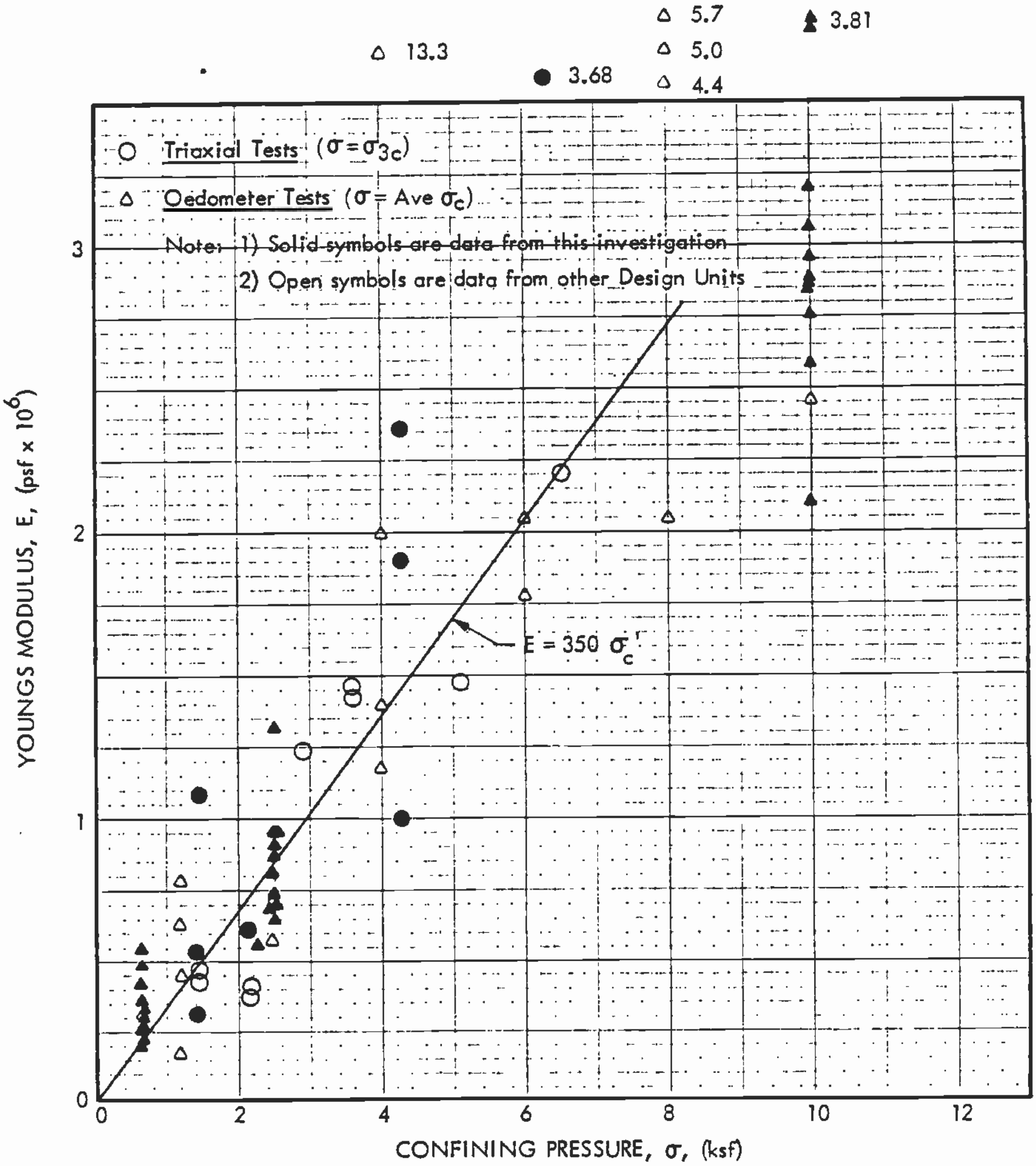

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SUMMARY OF MODULUS DATA - GRANULAR ALLUVIUM

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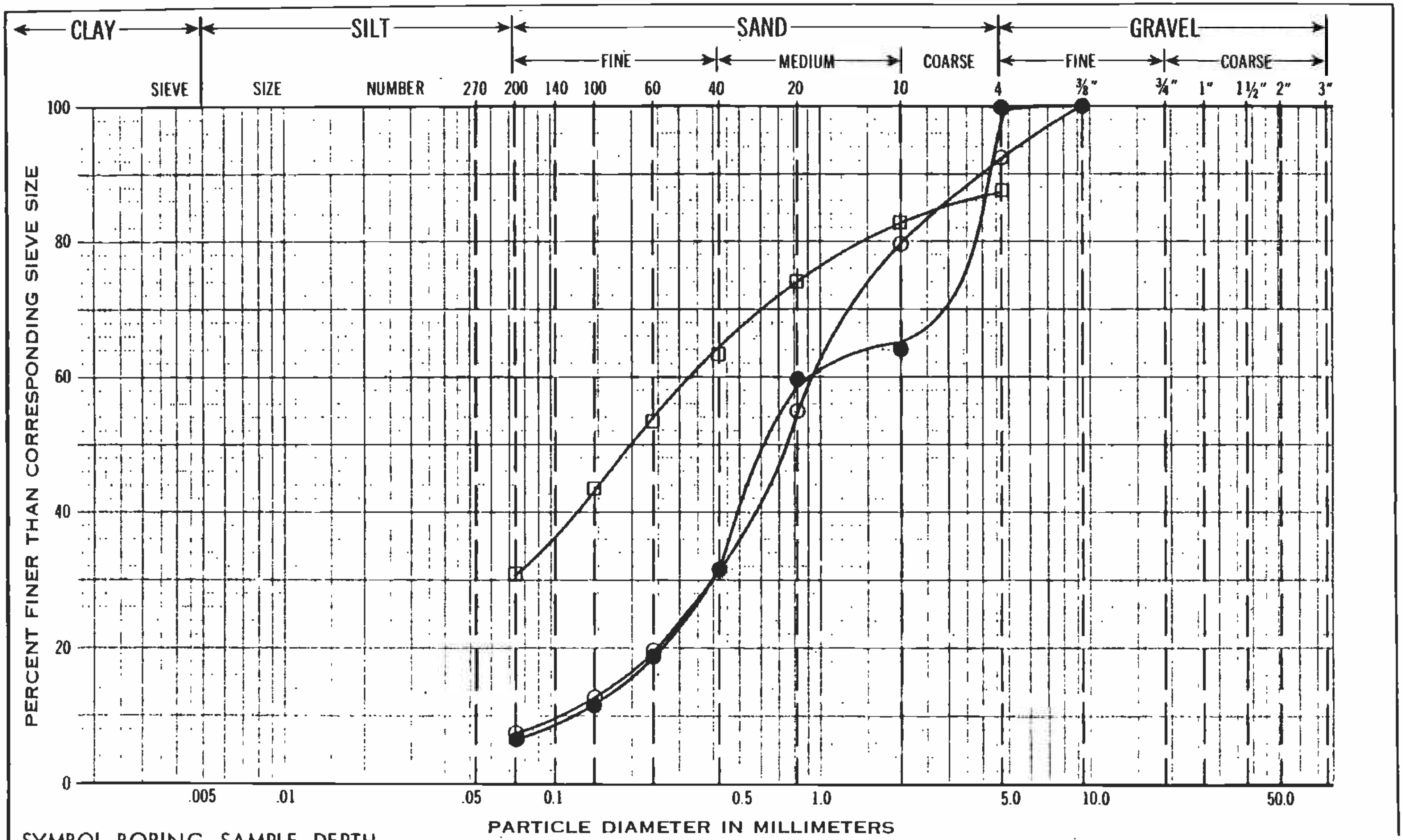
F-2



SYMBOL	BORING	SAMPLE	DEPTH
○	3-1	J-3	23'-24.5'
●	3-1	C-4	31'8"-31'11"
□	3-2	C-2	15'1"-15'6"

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Figure No.
F-3

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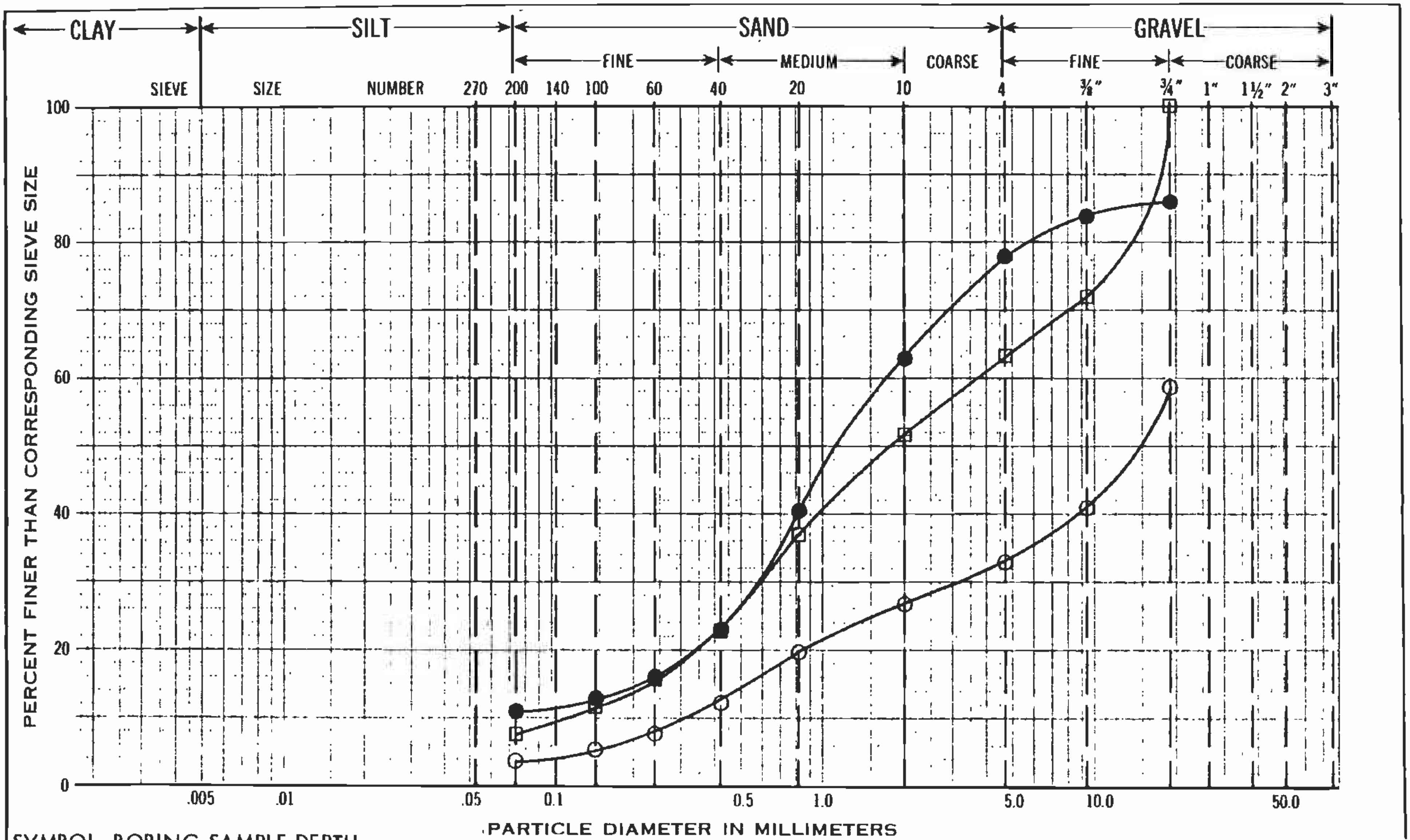

SYMBOL	BORING	SAMPLE	DEPTH
○	3-3	C-1	10'6"-10'9"
●	3-3	J-1	6'6"
☐	3-3	J-3	22'6"

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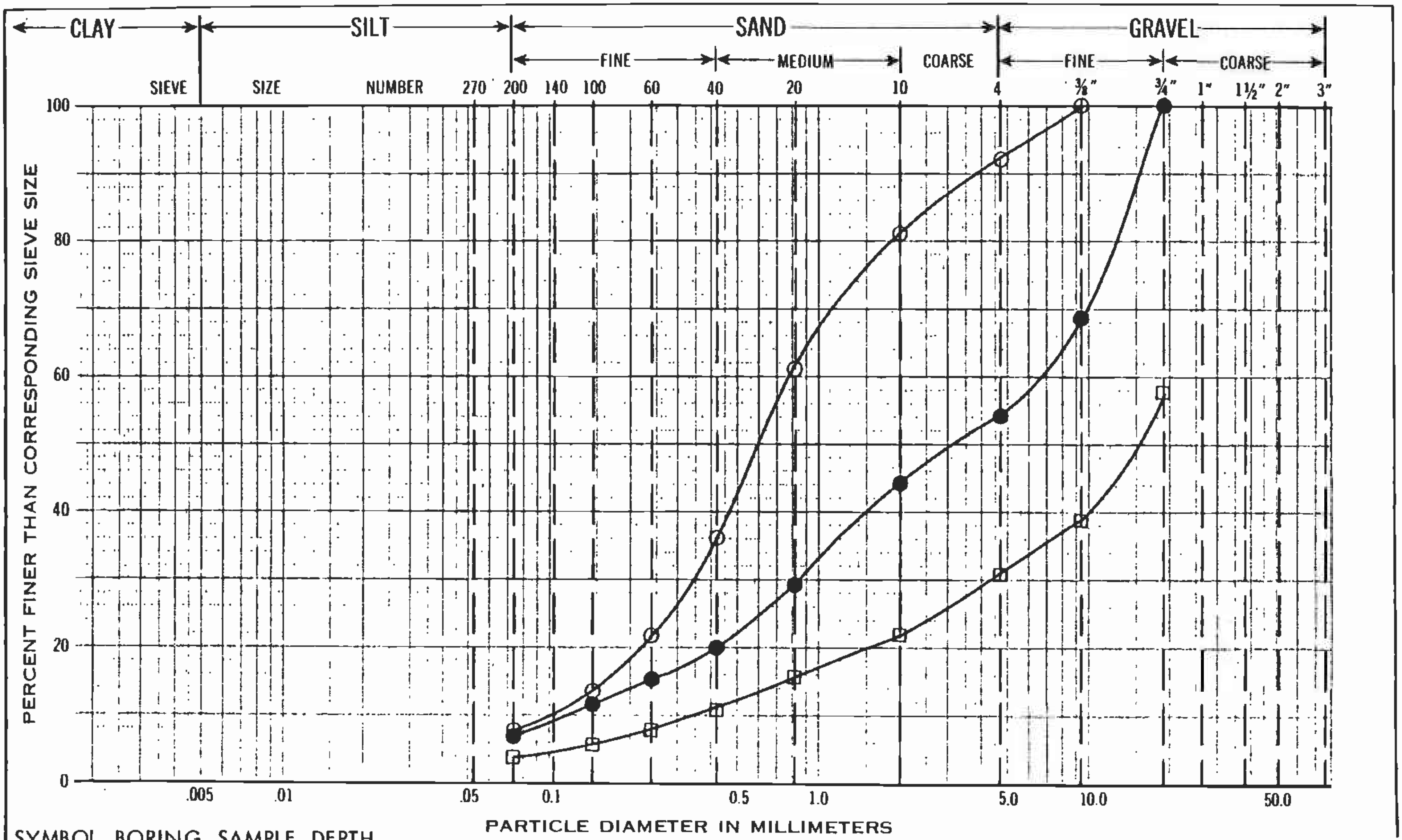

SYMBOL	BORING	SAMPLE	DEPTH
○	3-4	J-1	6'6"
●	3-4	J-2	16'5"
□	3-4	J-3	26'6"

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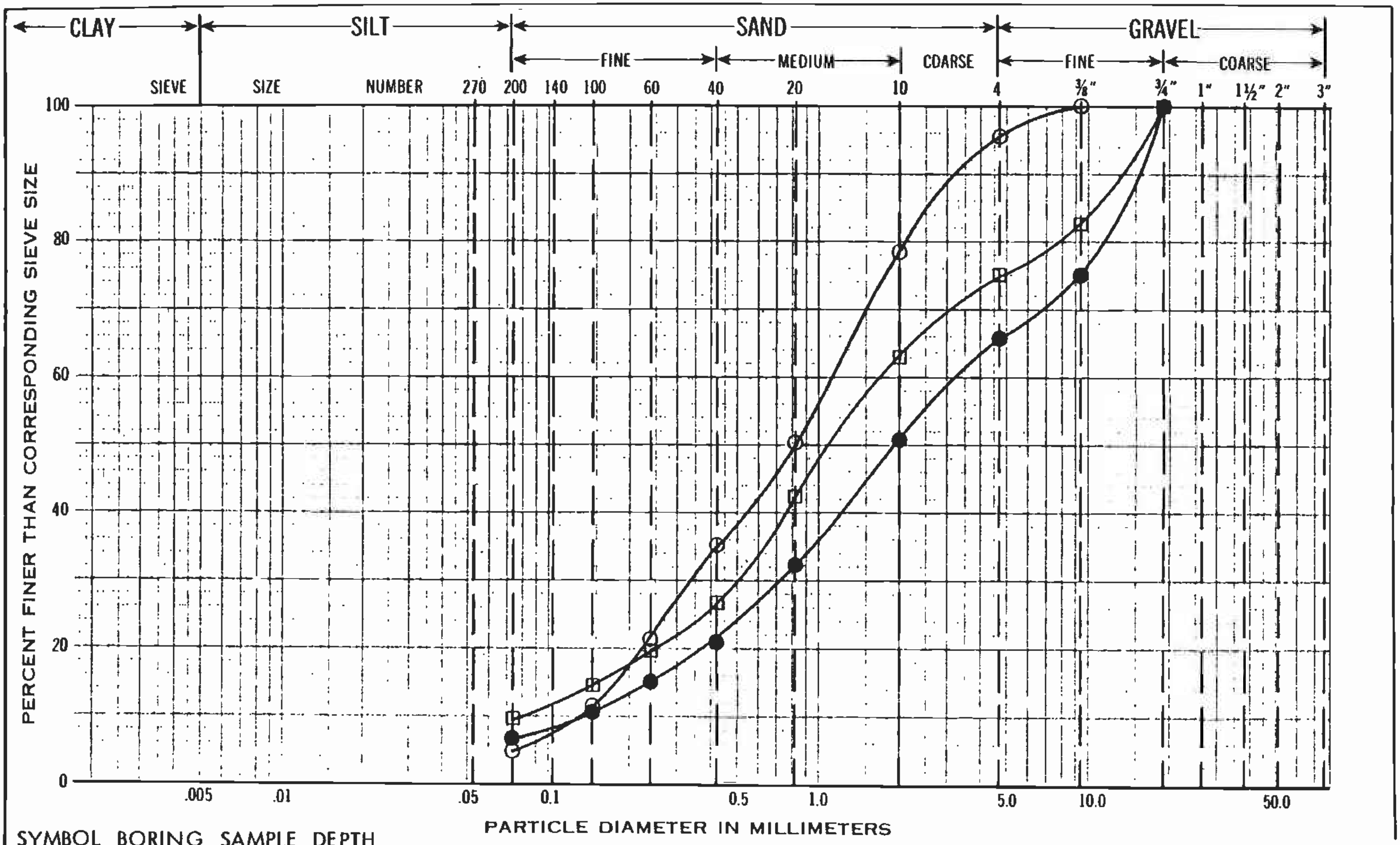

SYMBOL	BORING	SAMPLE	DEPTH
○	3-6	J-1	6'6"
●	3-6	J-2	12'-12.5'
□	3-6	J-4	26'0"

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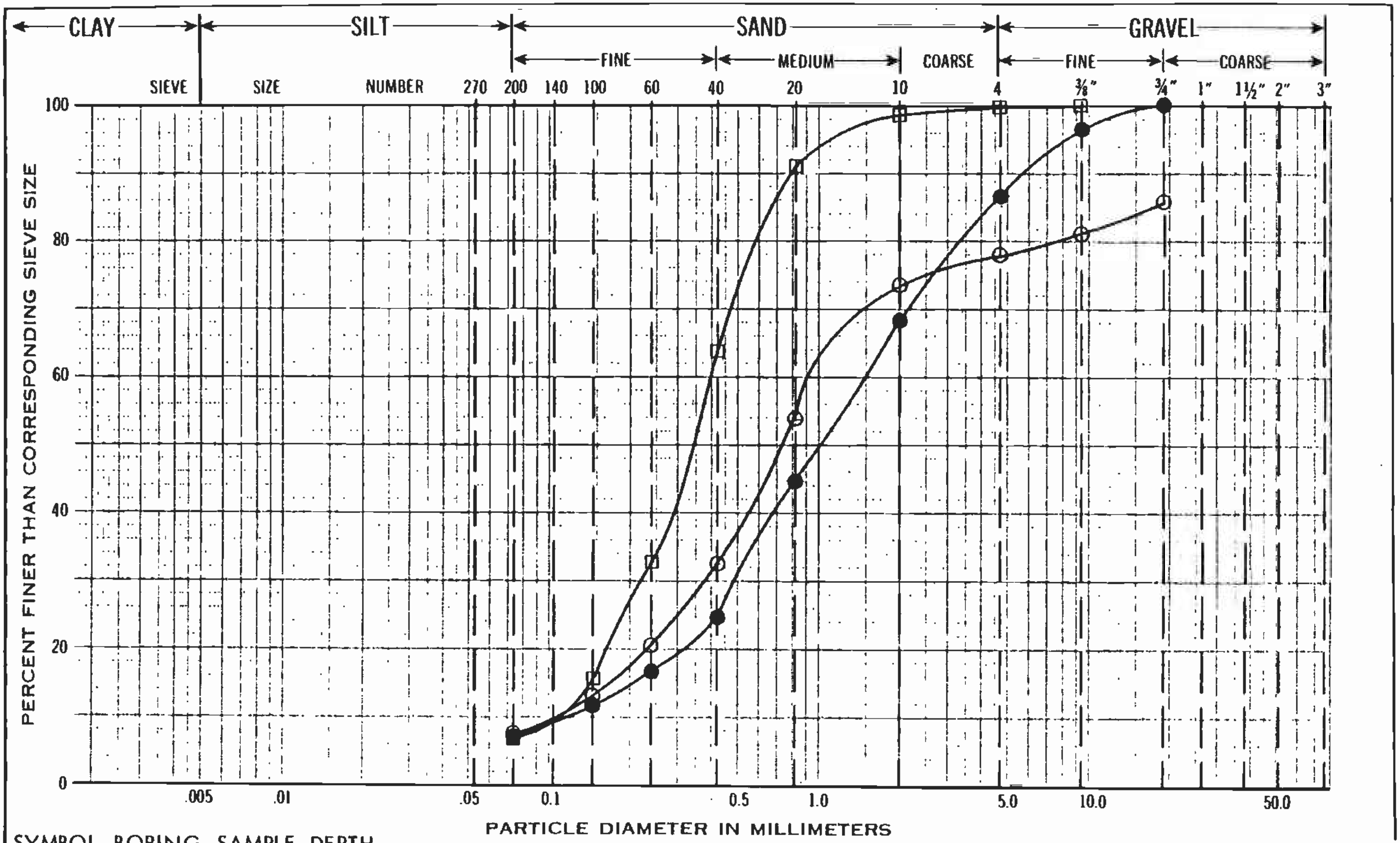

SYMBOL	BORING	SAMPLE	DEPTH
○	3-7	J-1	6'6"
●	3-7	J-3	16'-16.5'
☐	3-8	J-1	6'6"

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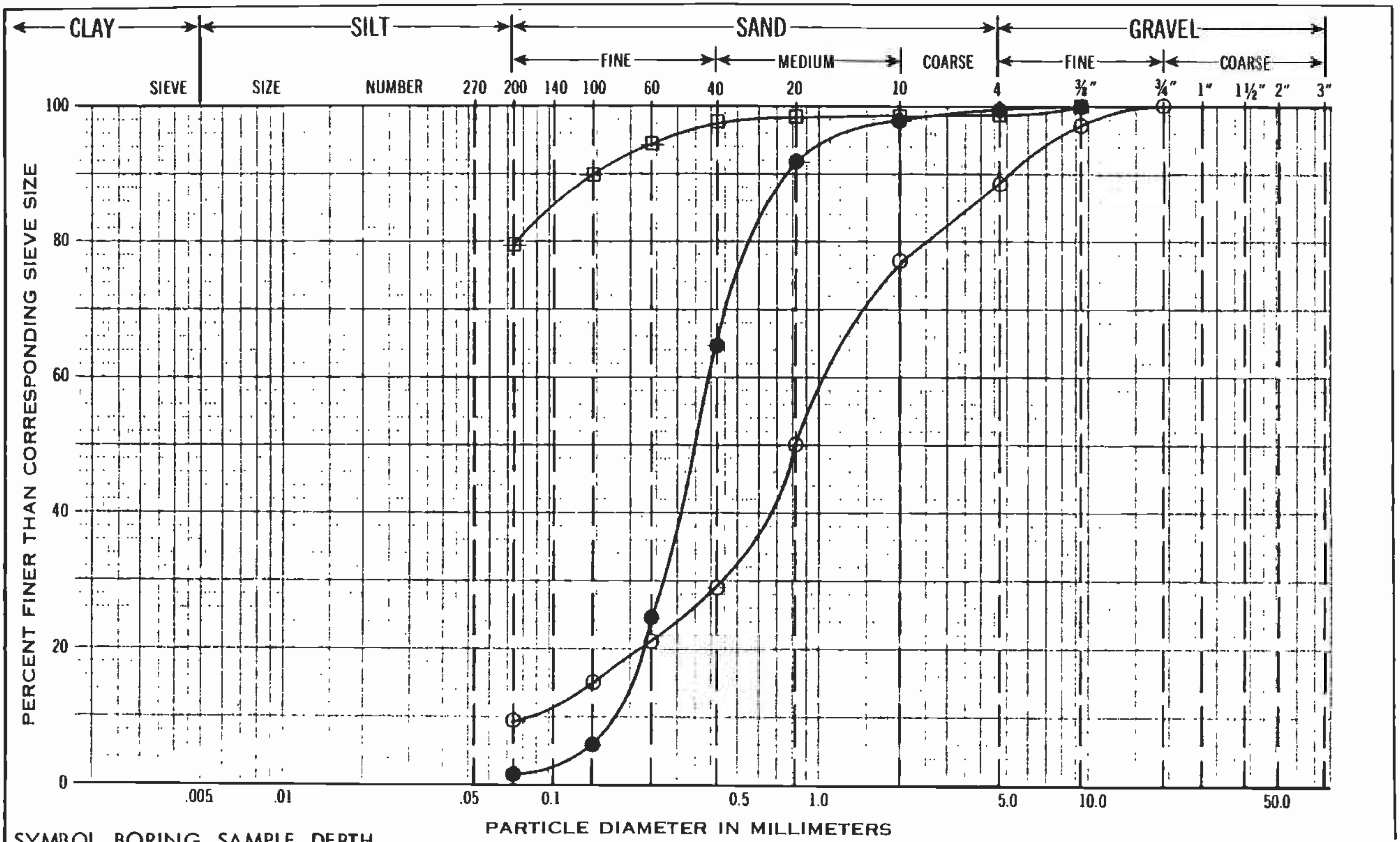

SYMBOL	BORING	SAMPLE	DEPTH
○	3-8	J-3	16'7"
●	3-9	C-1	3'
□	3-9	C-5	25'8"

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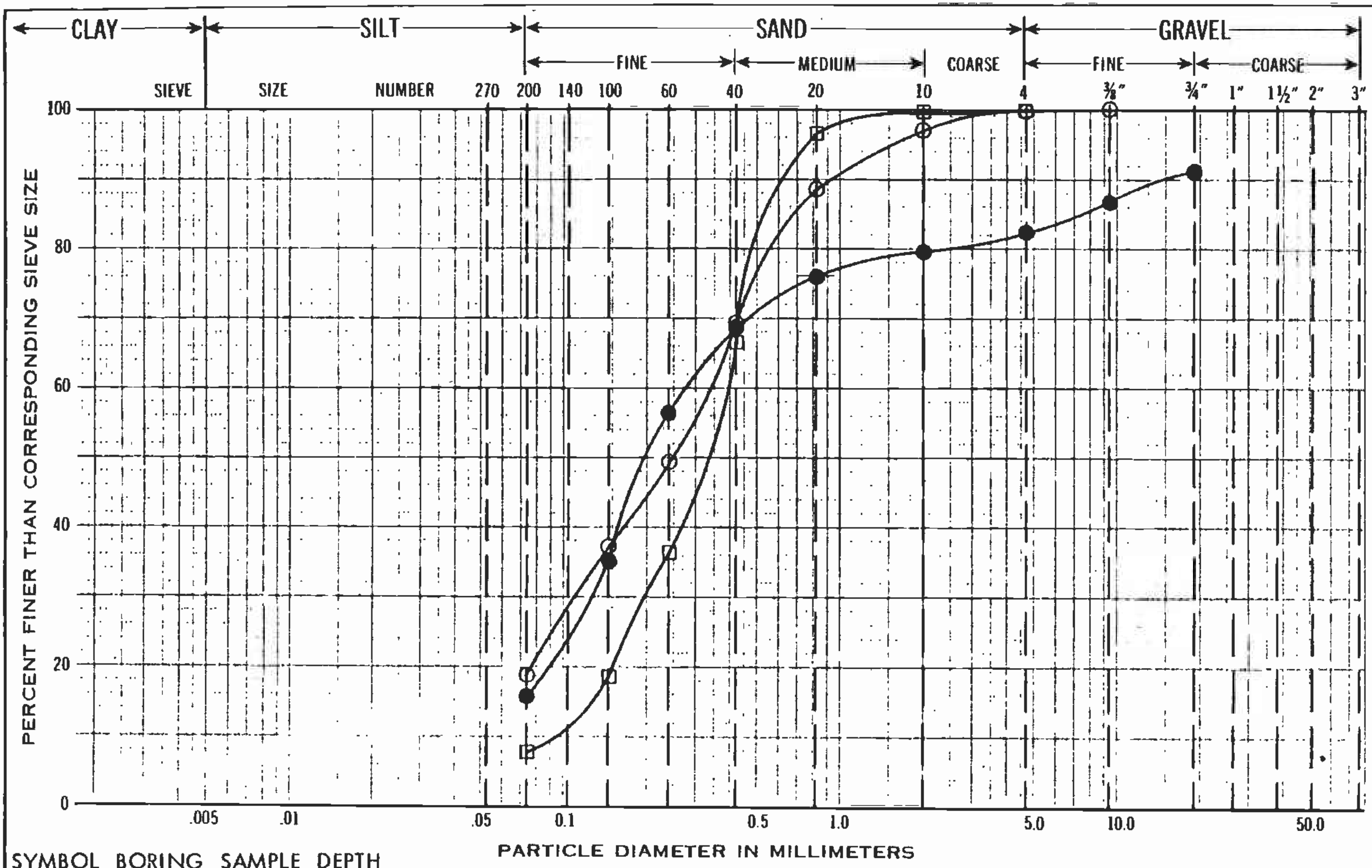

SYMBOL	BORING	SAMPLE	DEPTH
○	3-10	J-1	6'-6.5'
●	3-11	J-1	6'-6.5'
□	3-12	C-1	10'7"-11'0"

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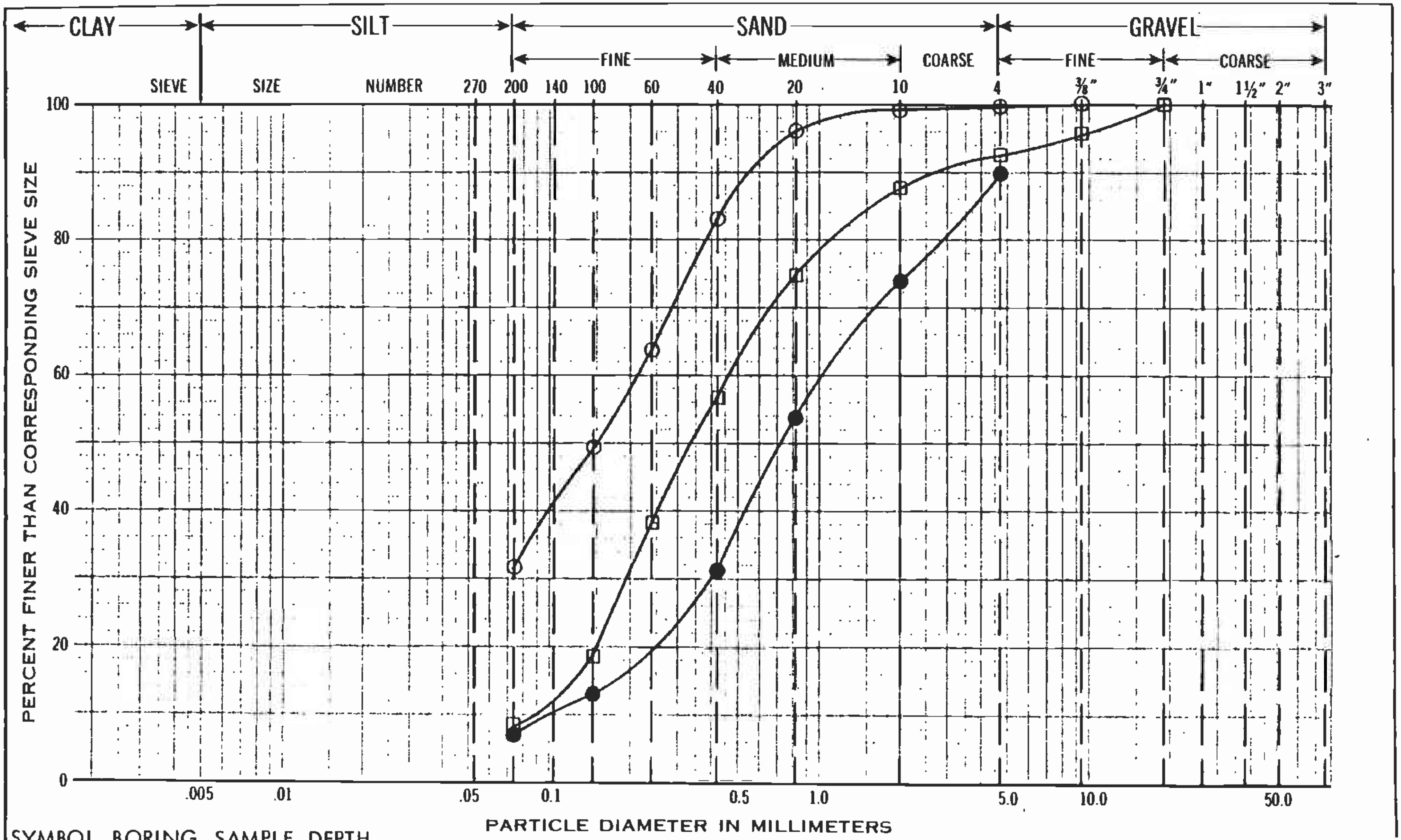

SYMBOL	BORING	SAMPLE	DEPTH
○	3-12	J-1	6'6"
●	3-13	C-1	10'11"
□	3-13	J-1	6'6"

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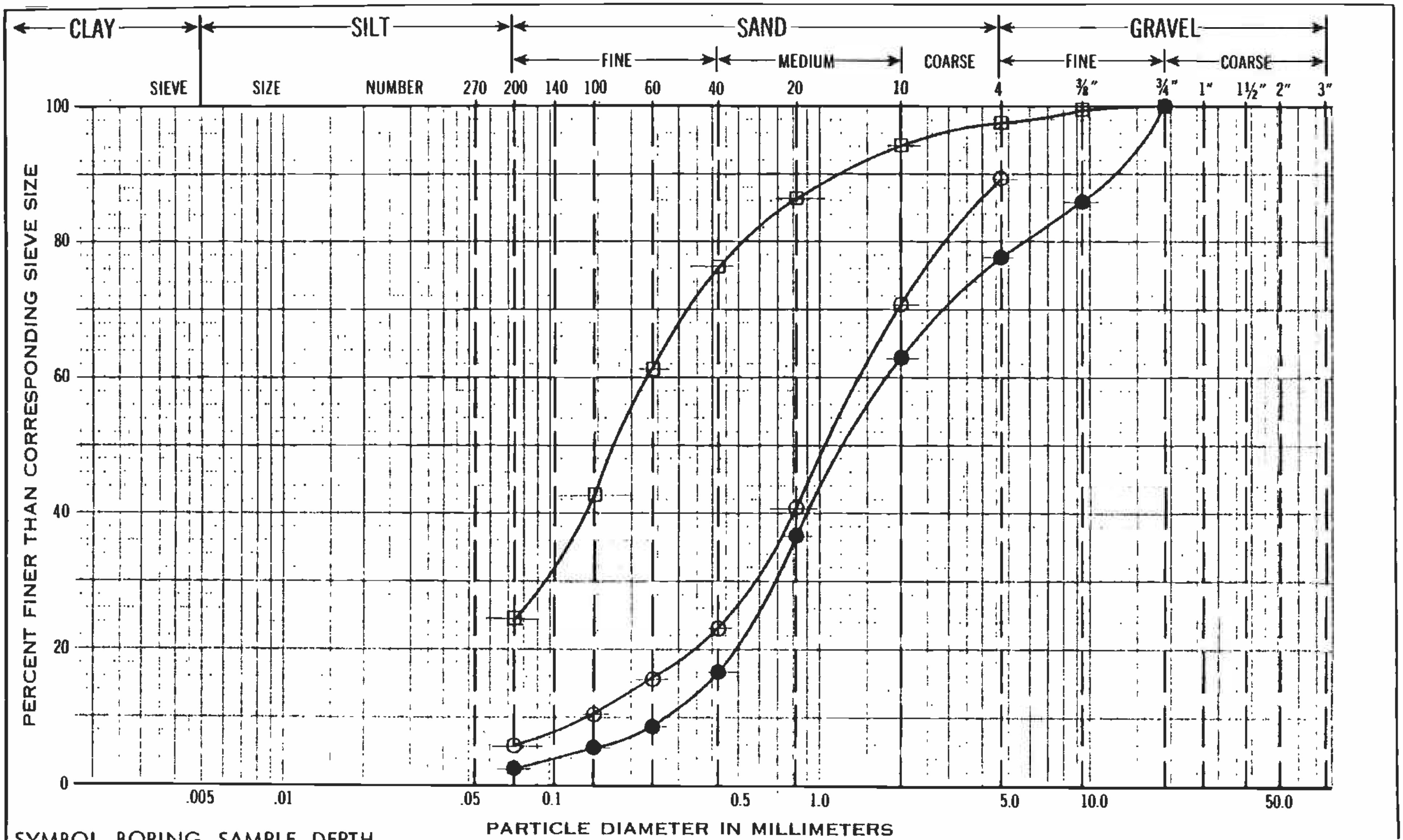

SYMBOL	BORING	SAMPLE	DEPTH
○	3-14	C-1	10'7"-11'0"
●	3-14	C-2	30'1"-30'3"
☐	3-14	J-1	6'6"

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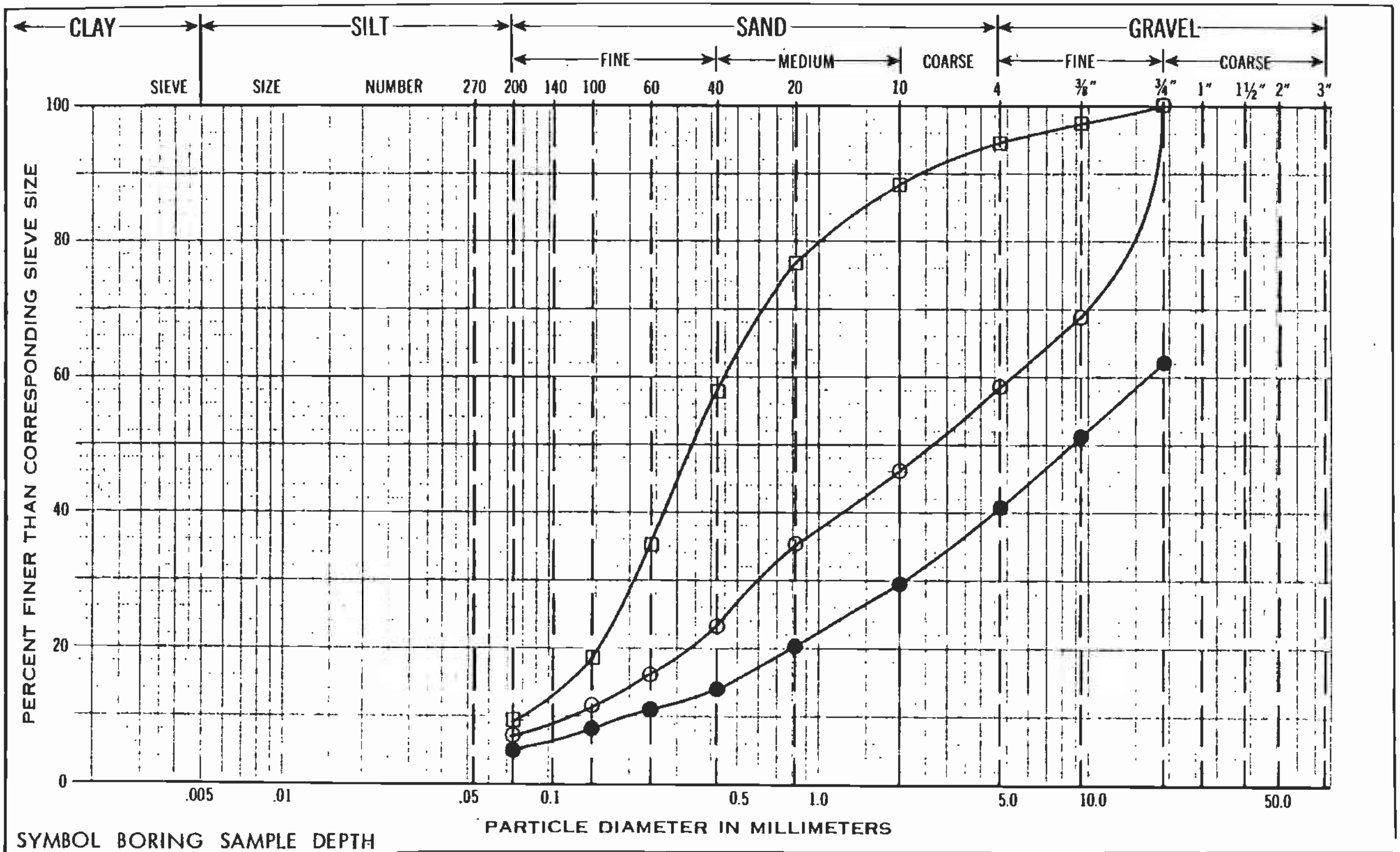

SYMBOL	BORING	SAMPLE	DEPTH
○	3-14	J-2	16'5"
●	3-15	C-4	30'2"-30'6"
□	3-17	J-1	6'-6.5'

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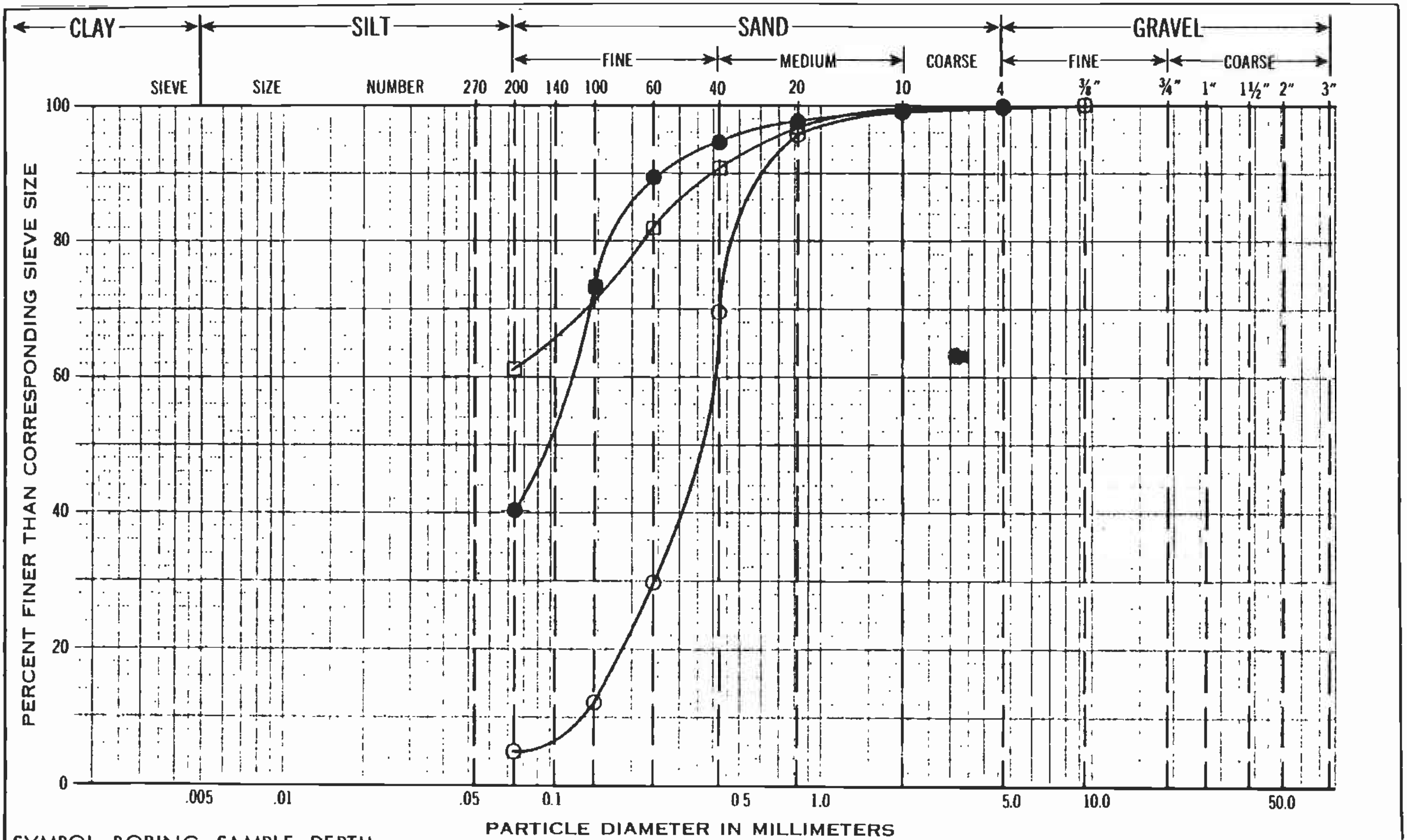

SYMBOL	BORING	SAMPLE	DEPTH
○	3-18	C-1	11'0"
●	3-18	J-1	6'6"
☐	3-19	J-1	6'-6.5'

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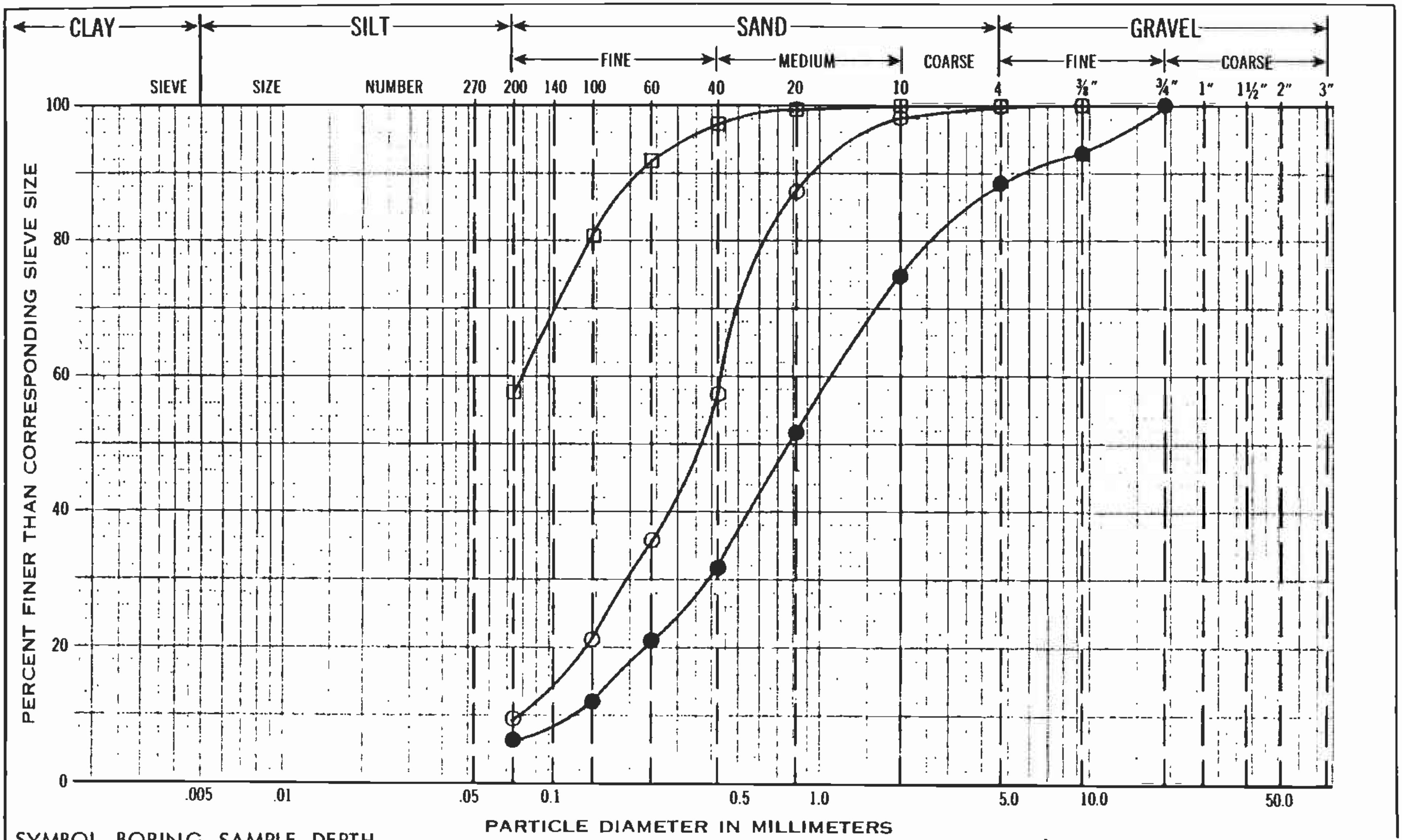

SYMBOL	BORING	SAMPLE	DEPTH
○	3-20	J-1	6'5"
●	3-20	J-2	16'6"
□	3-21	J-1	6'-6.5'

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F-14

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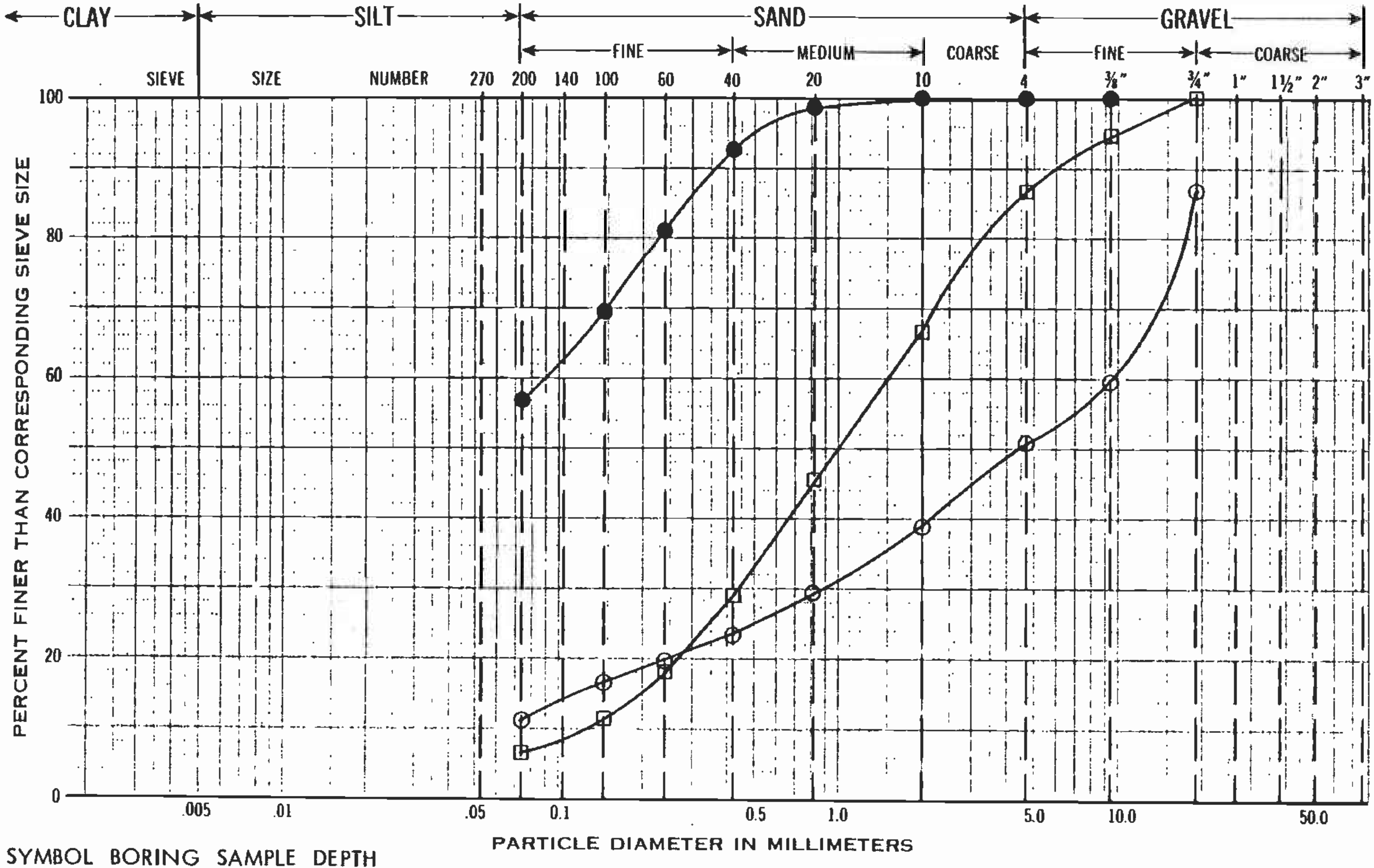

SYMBOL	BORING	SAMPLE	DEPTH
○	3-21	J-2	16'5"
●	3-22	J-1	6'-6.5'
□	3-22	J-2	16'6"

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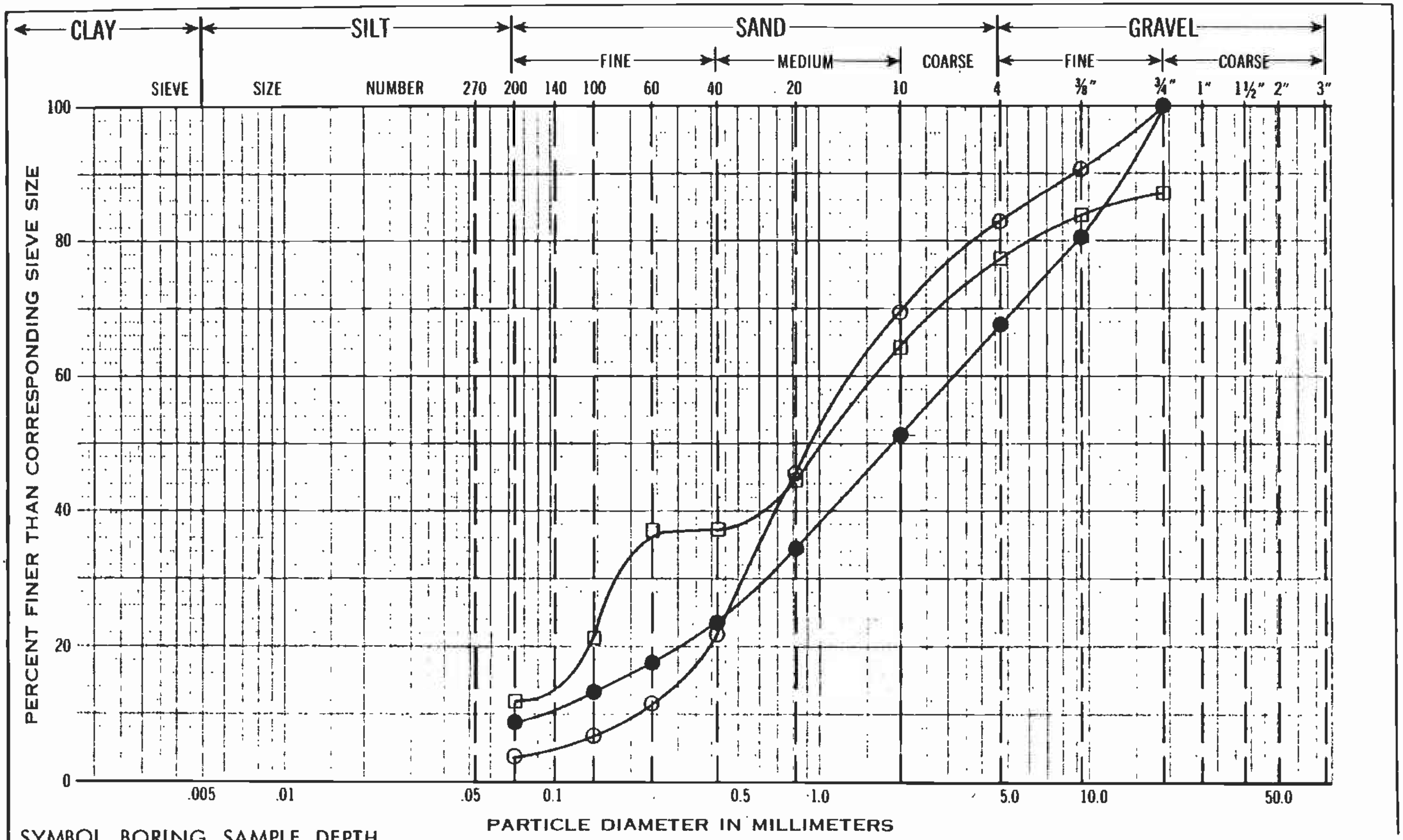

SYMBOL	BORING	SAMPLE	DEPTH
○	3-23	C-1	10'8"-10'10"
●	3-23	J-1	16'-16.5'
□	3-24	C-4	14'6"

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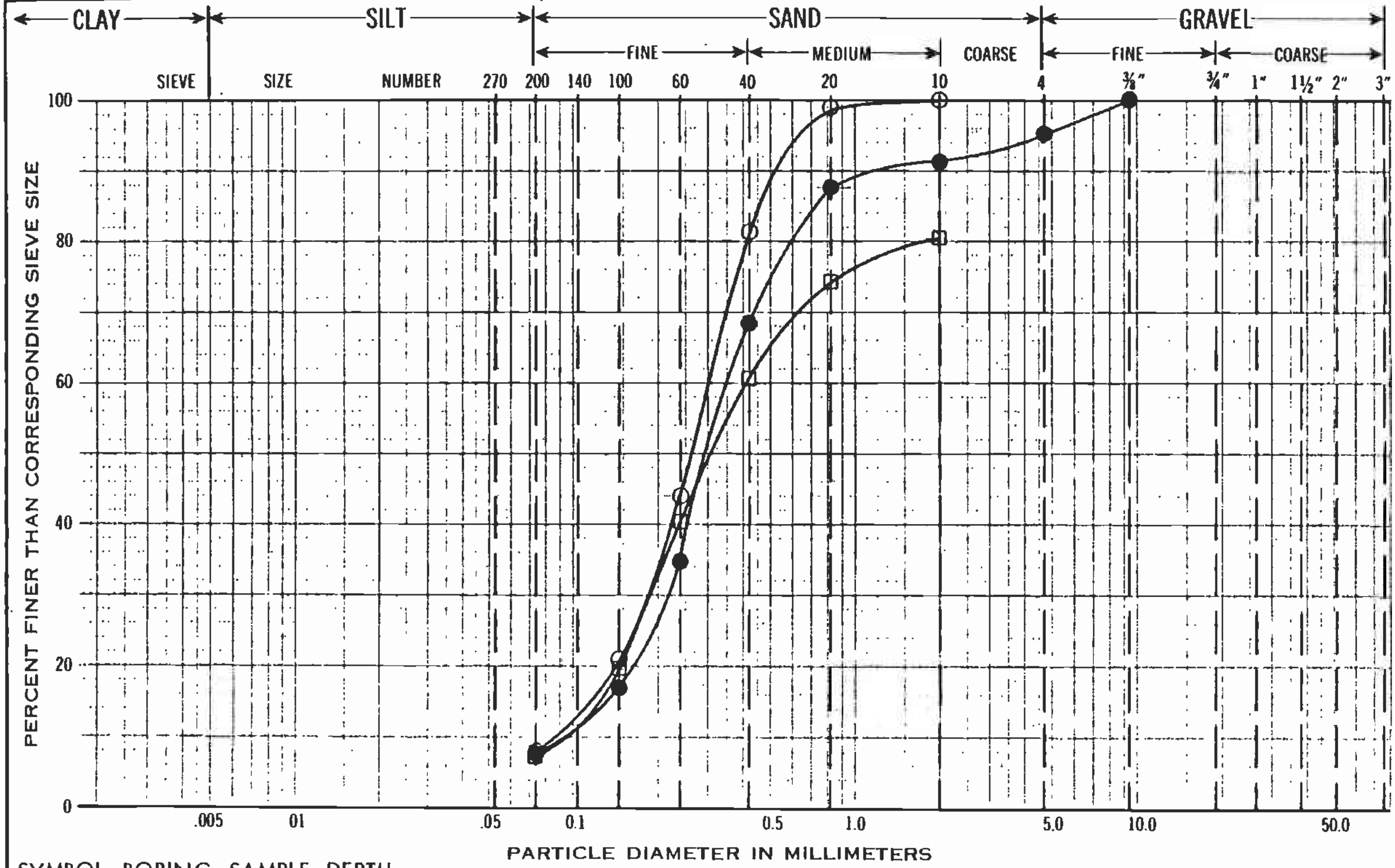

SYMBOL	BORING	SAMPLE	DEPTH
○	3-24	C-6	24'11"
●	3-25	J-1	6'-6.5'
□	3-26	C-1	8'11'-8'5"

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F-17

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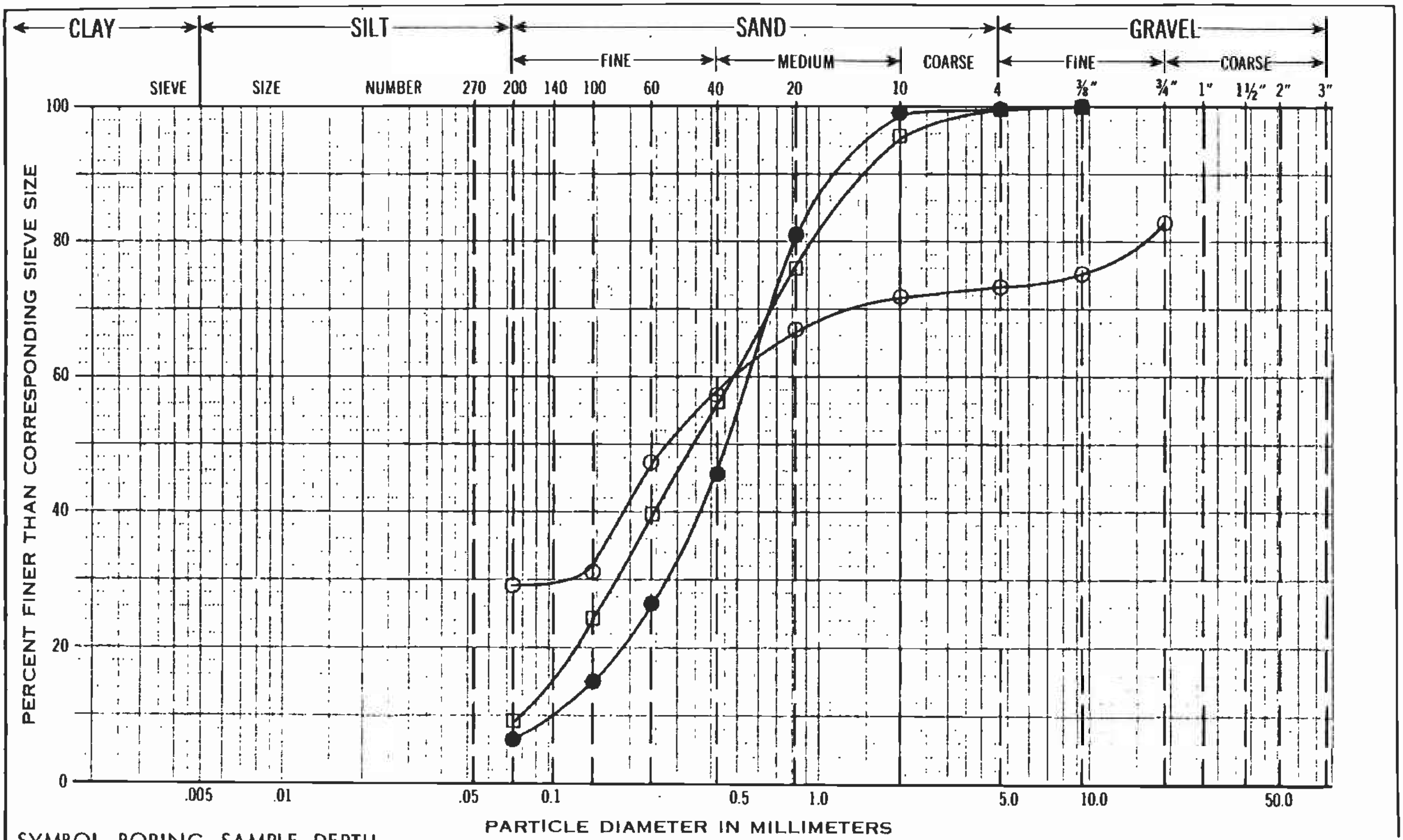

SYMBOL	BORING	SAMPLE	DEPTH
○	3-27	C-1	5'9"-6'0"
●	3-28	C-1	4'11"-5'0"
□	3-29	C-1	4'7"

GRAIN-SIZE DISTRIBUTION CHART

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83-1140

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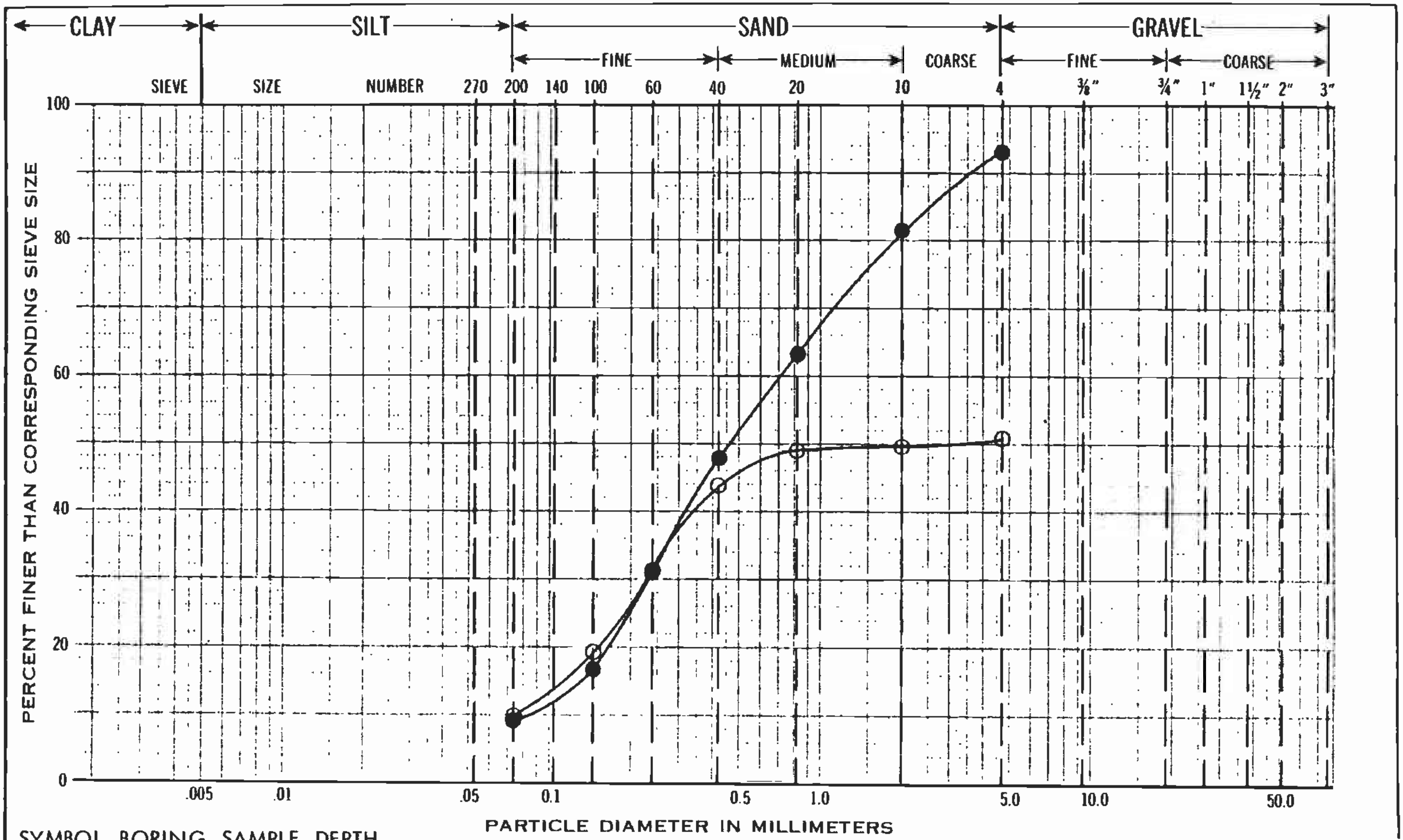
Figure No.
F-18

FORM NO. D-3 Approved for publication ____________ by ____________



SYMBOL	BORING	SAMPLE	DEPTH
○	3-30	C-1	4'10"-5'0"
●	B-10	C-7	75'7"-76'0"

GRAIN-SIZE DISTRIBUTION CHART

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Figure No
F-19

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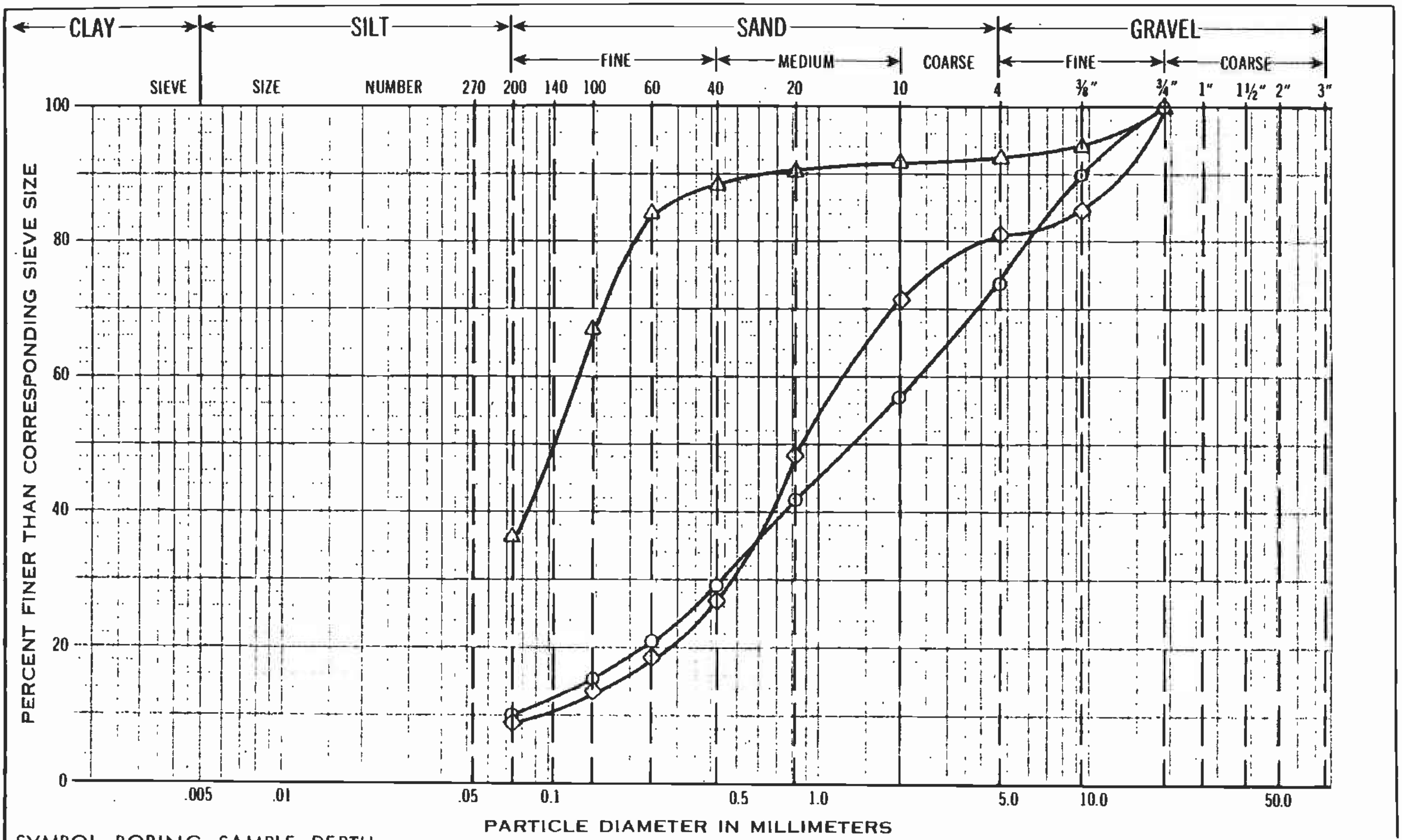

SYMBOL	BORING	SAMPLE	DEPTH
○	B-10	J-1	61.5'
◇	B-10	J-3	81.5'
△	B-11	J-5	71.5'

GRAIN-SIZE DISTRIBUTION CHART

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Figure No
F-20

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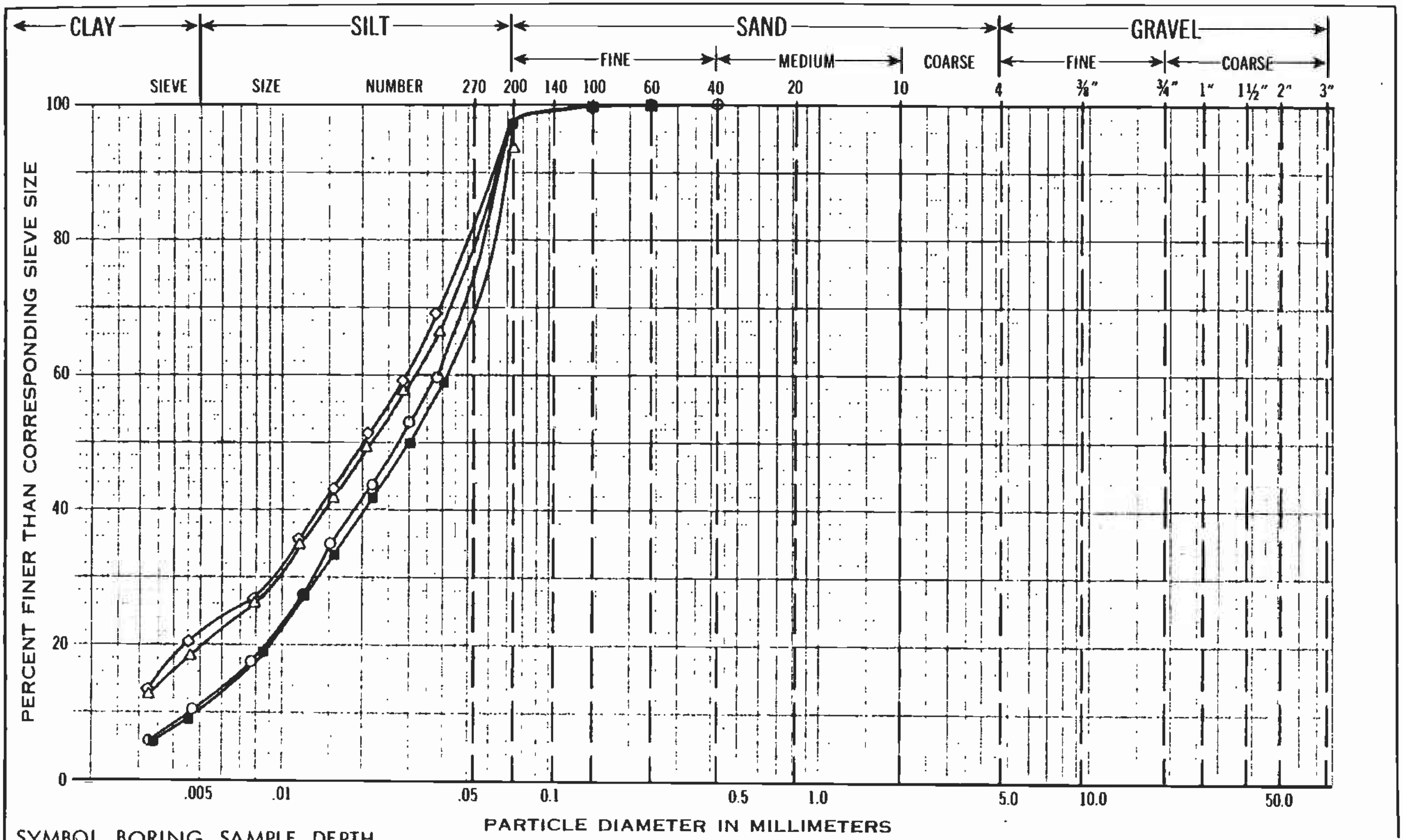

SYMBOL	BORING	SAMPLE	DEPTH
○	B-10	C-8	101'
■	B-10	C-10	107'
◇	B-11	C-5	102.5'
△	B-11	C-6	104'

GRAIN-SIZE DISTRIBUTION CHART

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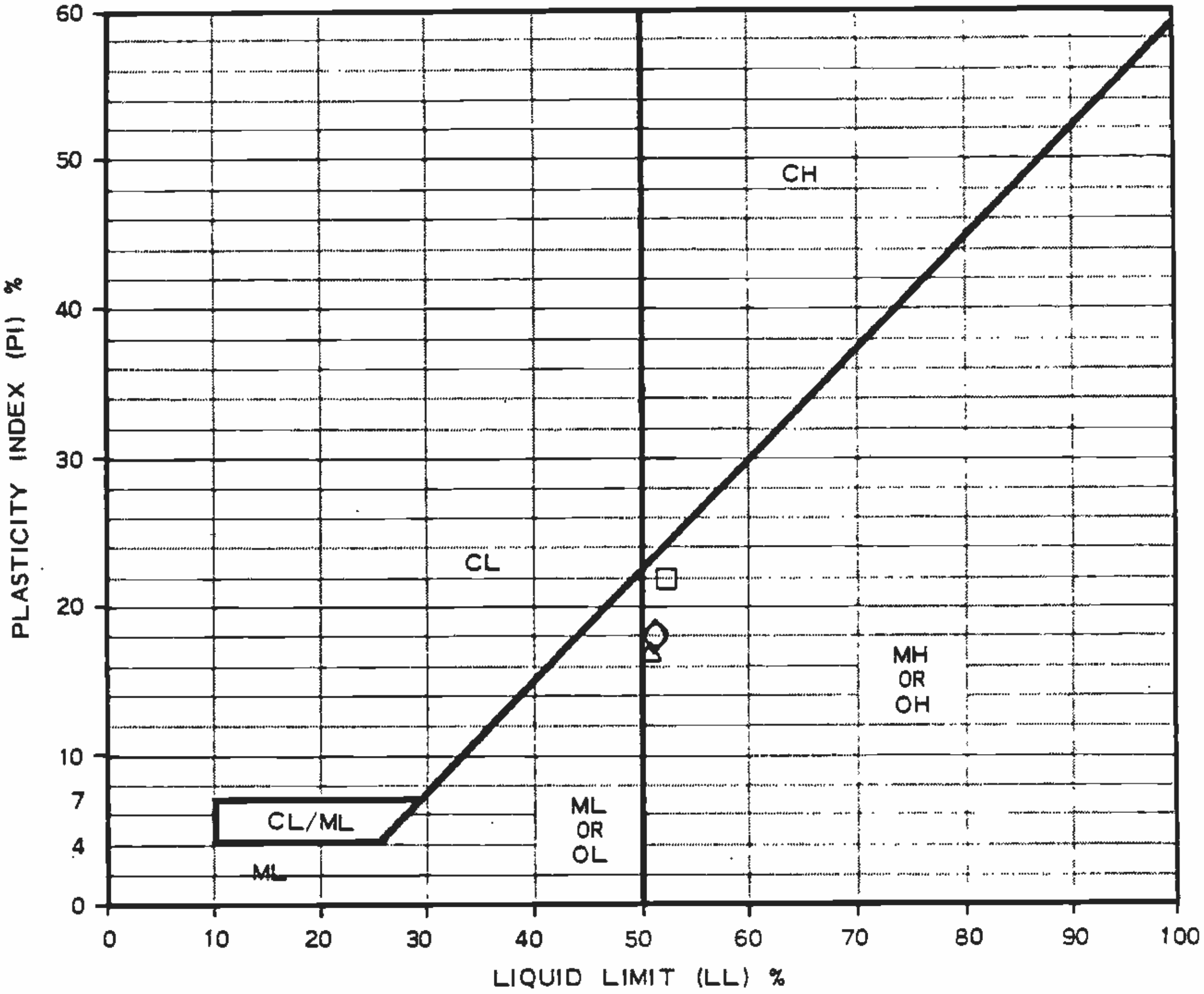
Figure No.
F-21



Symbol	Classification and Source	Liquid Limit (%)	Plastic Limit (%)	Plasticity Index (%)	% Passing No. 200 Seive
○	B10, C8, 101', CH	52	34	18	97
□	B10, C10, 107', CH	53	31	22	97
◇	B11, C5, 102.5', CH	52	34	18	94
△	B11, C6, 104', CH	51	34	17	97

PLASTICITY CHART

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Project No.
83-1140

Figure No.
F-22

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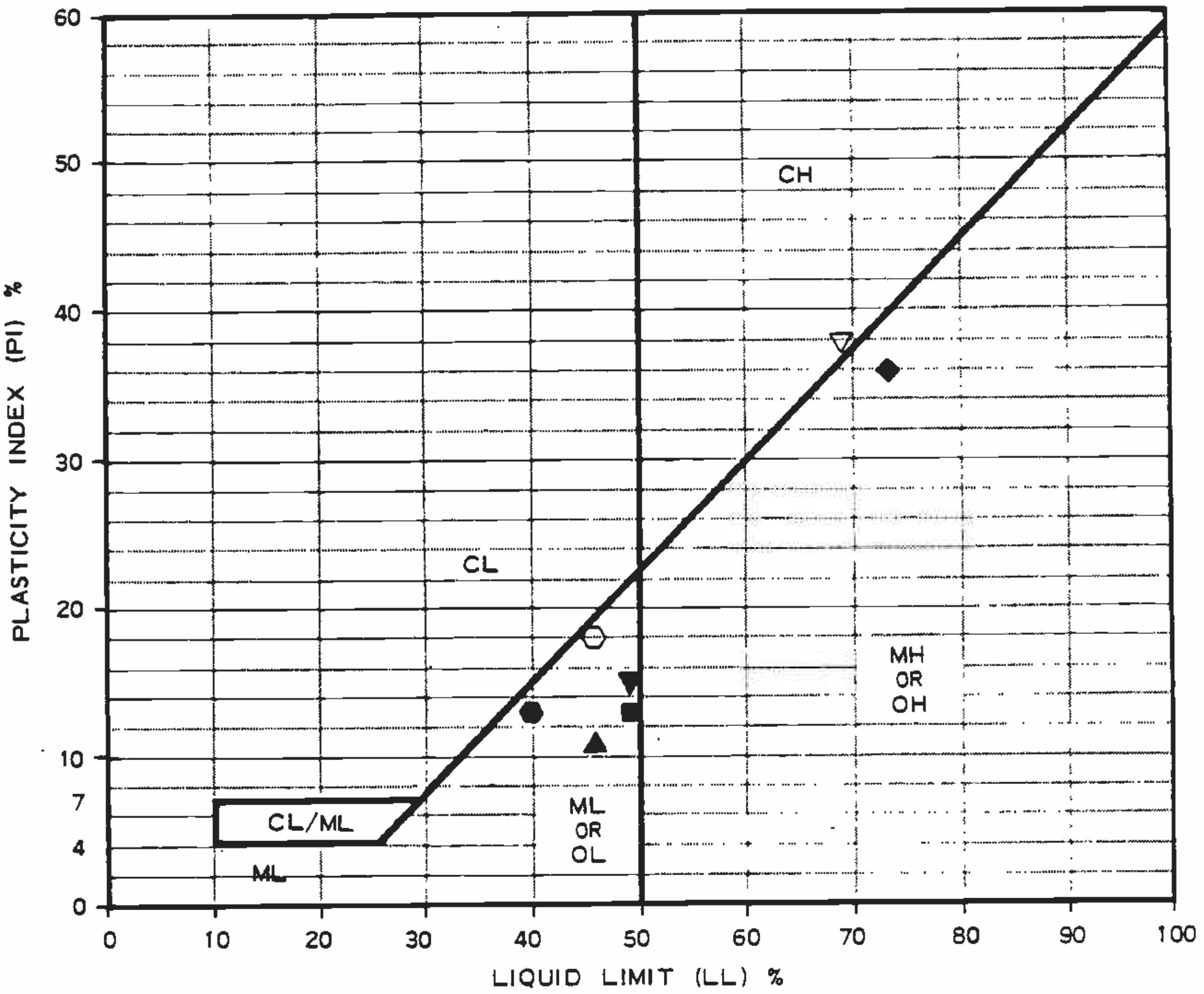

Symbol	Classification and Source	Liquid Limit (%)	Plastic Limit (%)	Plasticity Index (%)	% Passing No. 200 Seive
■	BH 2, 29' (ML)	49	36	13	-
▲	BH 2, 98' (ML)	46	35	11	-
◆	BH 3, 104' (MH)	73	37	36	-
▽	BH 3, 139' (CH)	69	31	38	-
▼	BH 5, 90' (ML)	49	34	15	-
○	BH 5, 119' (ML)	46	28	18	-
●	BH 5, 148' (ML)	40	27	13	-

PLASTICITY CHART

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Project No.
83-1140

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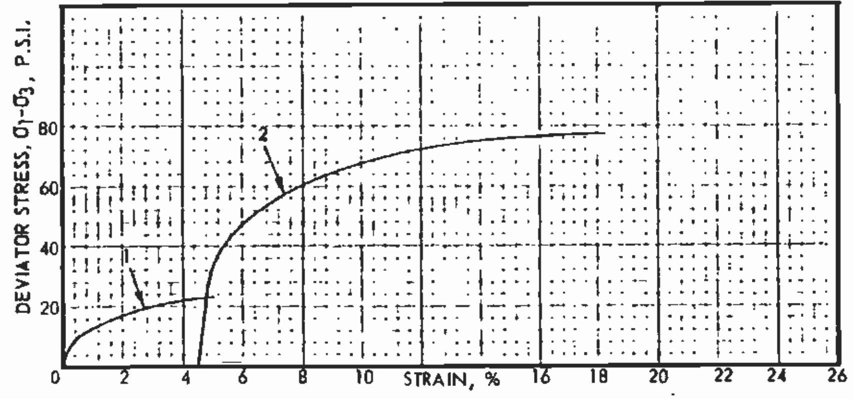
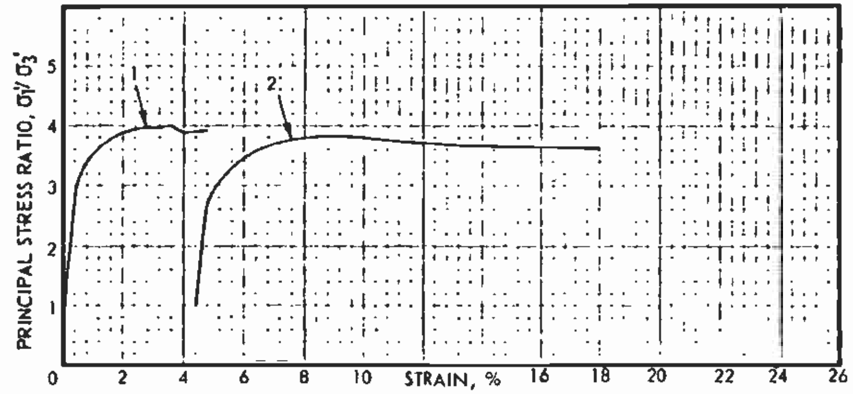
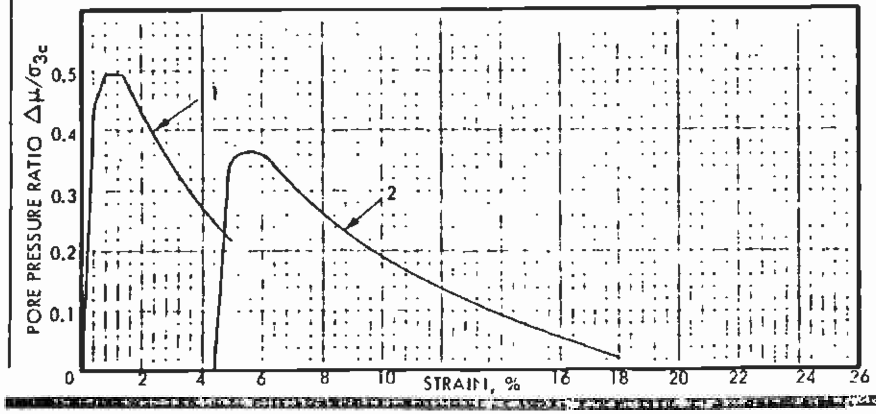
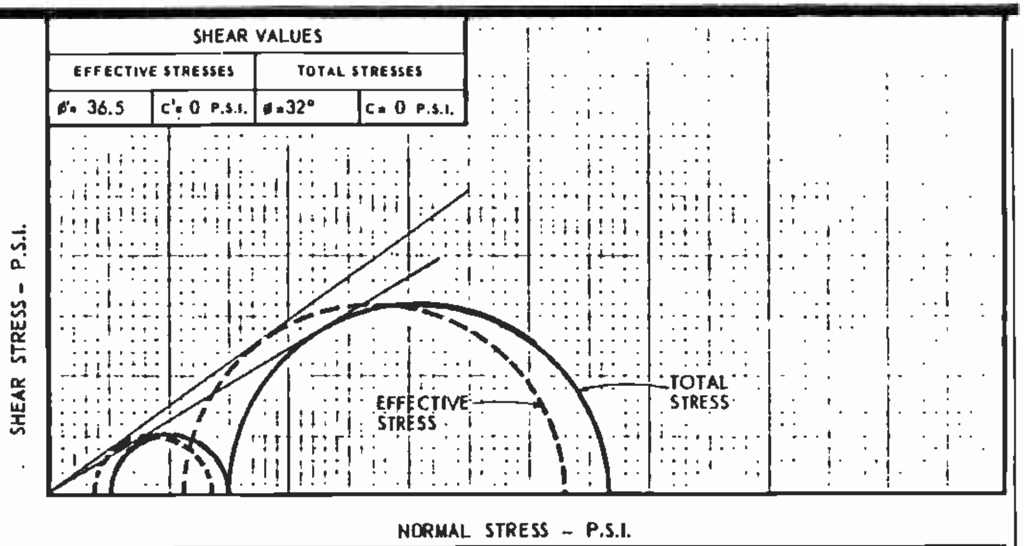
Figure No.
F-23

Approved for publication ______ by ______









SHEAR VALUES			
EFFECTIVE STRESSES		TOTAL STRESSES	
ϕ'= 36.5	c'= 0 P.S.I.	ϕ=32°	c= 0 P.S.I.

SPECIMEN NUMBER	SPECIMEN LOCATION		INITIAL SPECIMEN DATA					TYPE OF SAMPLE
	BORING NUMBER	DEPTH IN FEET	SOIL CLASSIFICATION	LENGTH IN INCHES	DIAMETER IN INCHES	DRY DENSITY (P.C.F.)	MOISTURE CONTENT IN PERCENT	
C-2	3-2	15' 1"	SM	4.96	2.42	119	13.2	5 RING (CONVERSE)
C-2	3-2	15' 1"	SM	4.96	2.42	119	13.2	

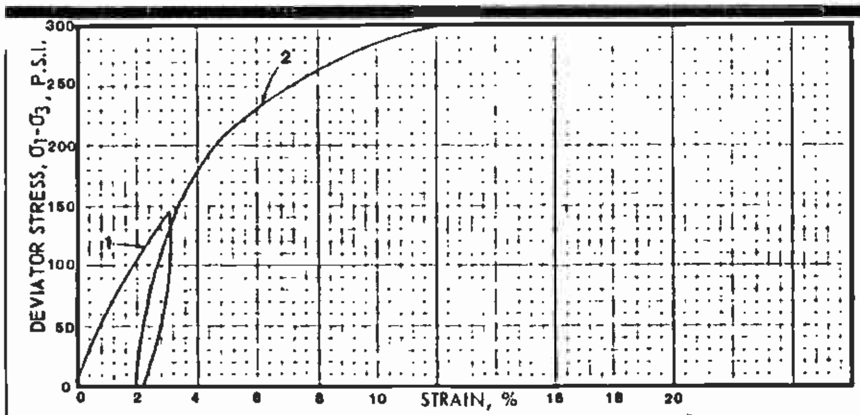
SYMBOLS	SPECIMEN NUMBER	APPLIED LATERAL PRESSURE σ_3 (P.S.I.)	MAXIMUM DEVIATOR STRESS $\sigma_1 - \sigma_3$ (P.S.I.)	TEST VALUES AT FAILURE - MAXIMUM σ_1'/σ_3'		BACK PRESSURE (P.S.I.)	TYPE OF TEST
				EFFECTIVE LATERAL PRESSURE σ_3' (P.S.I.)	MAJOR EFFECTIVE STRESS σ_1' (P.S.I.)		
1	C-2	10	20.8	6.9	27.7	70	PROGRESSIVE CUE
2	C-2	30	63.5	22.6	86.1	70	

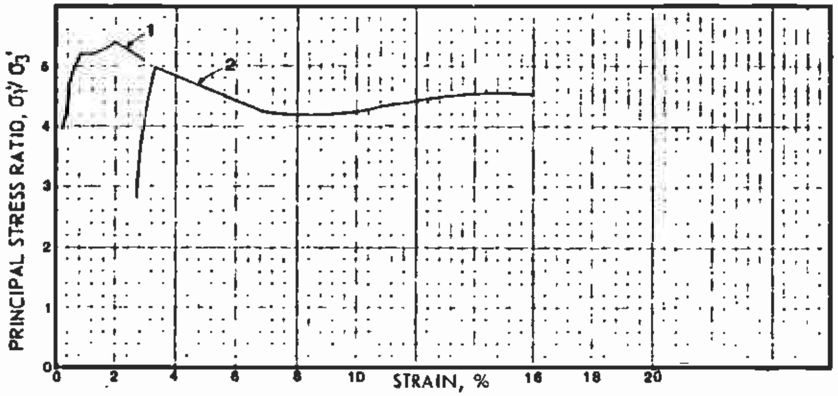
TRIAXIAL COMPRESSION TEST

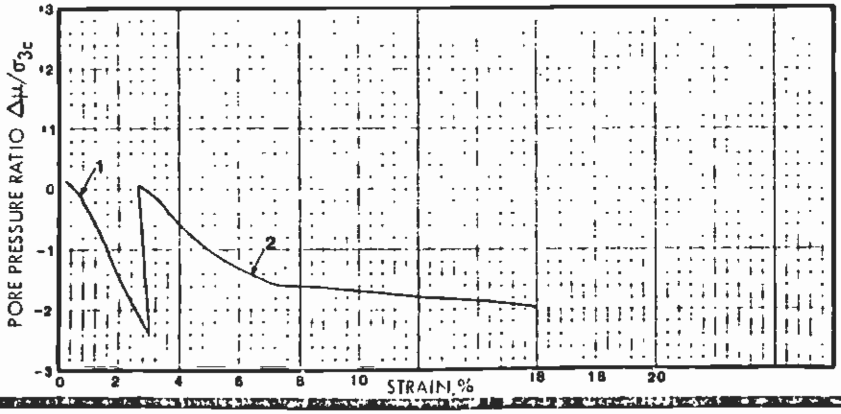
DESIGN UNIT A100
Southern California Rapid Teansit District
METRO RAIL PROJECT

Converse Consultants
Geotechnical Engineering and Applied Sciences

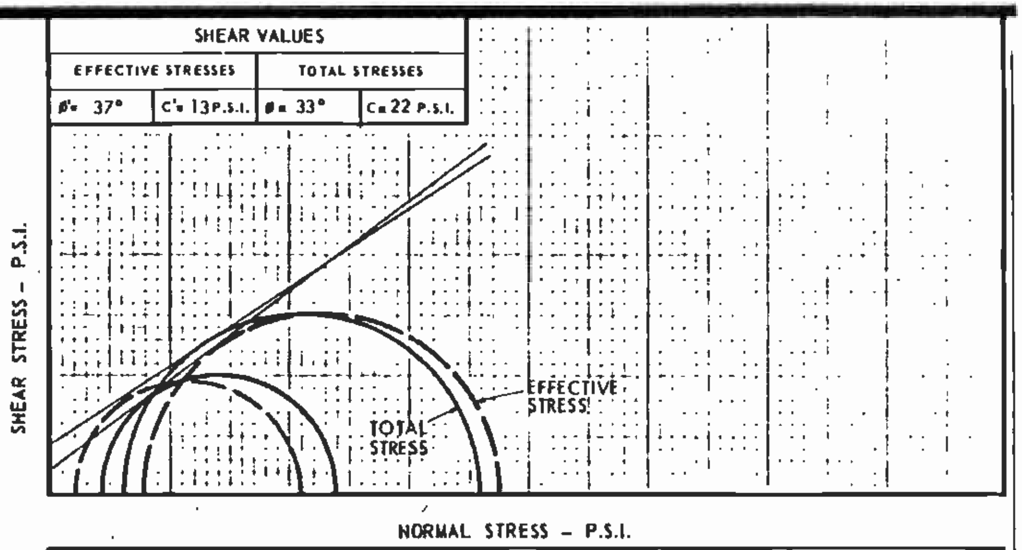
Scale As Shown
Date Feb., 1984
Prepared by KLF
Checked by JAD
Approved by JAD
Project No 83-1140
Figure No F-24







SHEAR VALUES			
EFFECTIVE STRESSES		TOTAL STRESSES	
ϕ'= 37°	C'= 13 P.S.I.	ϕ= 33°	C= 22 P.S.I.



SPECIMEN NUMBER	SPECIMEN LOCATION		INITIAL SPECIMEN DATA					TYPE OF SAMPLE
	BORING NUMBER	DEPTH IN FEET	SOIL CLASSI-FICATION	LENGTH IN INCHES	DIAMETER IN INCHES	DRY DENSITY (P.C.F.)	MOISTURE CONTENT IN PERCENT	
C-1	3/13	10	SP	5.05	2.42	114	5	5 RING (CONVERSE)
C-1	3/13	10	SP	5.05	2.42	114	5	

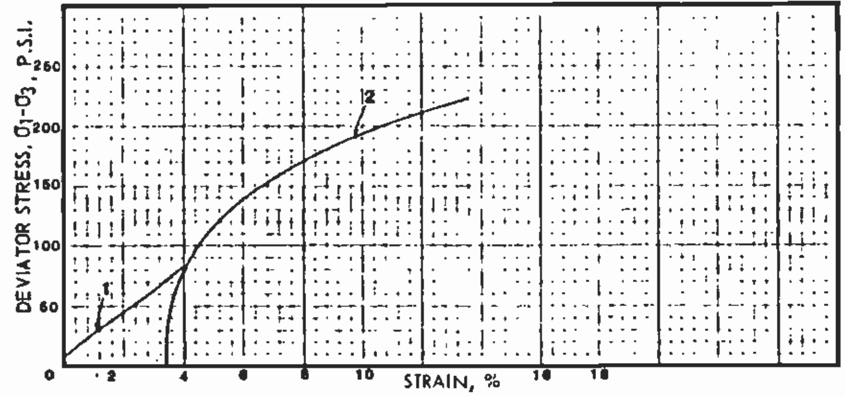
SYMBOLS	SPECIMEN NUMBER	APPLIED LATERAL PRESSURE σ_3 (P.S.I.)	MAXIMUM DEVIATOR STRESS $\sigma_1 - \sigma_3$ (P.S.I.)	TEST VALUES AT FAILURE – MAXIMUM σ_1'/σ_3'		BACK PRESSURE (P.S.I.)	TYPE OF TEST
				EFFECTIVE LATERAL PRESSURE σ_3' (P.S.I.)	MAJOR EFFECTIVE STRESS σ_1' (P.S.I.)		
1	C-1	10	140.6	22.0	117.0	80	PROGRESSIVE CUE
2	C-1	30	320.0	38.0	188.0	80	

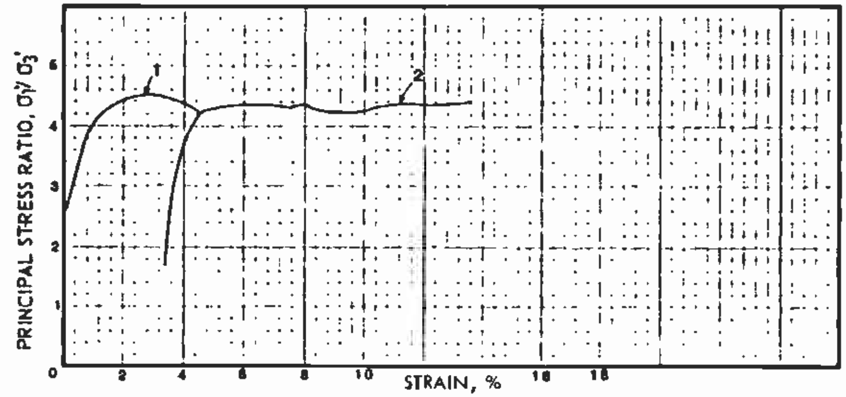
TRIAXIAL COMPRESSION TEST

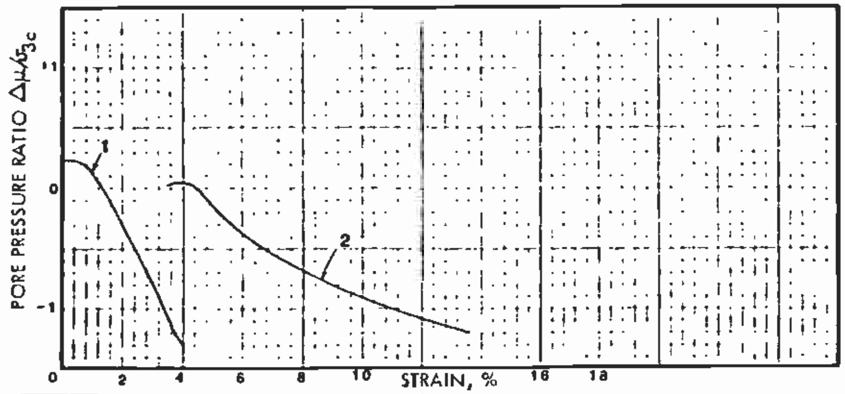
DESIGN UNIT A100
Southern California Rapid Transit District
METRO RAIL PROJECT

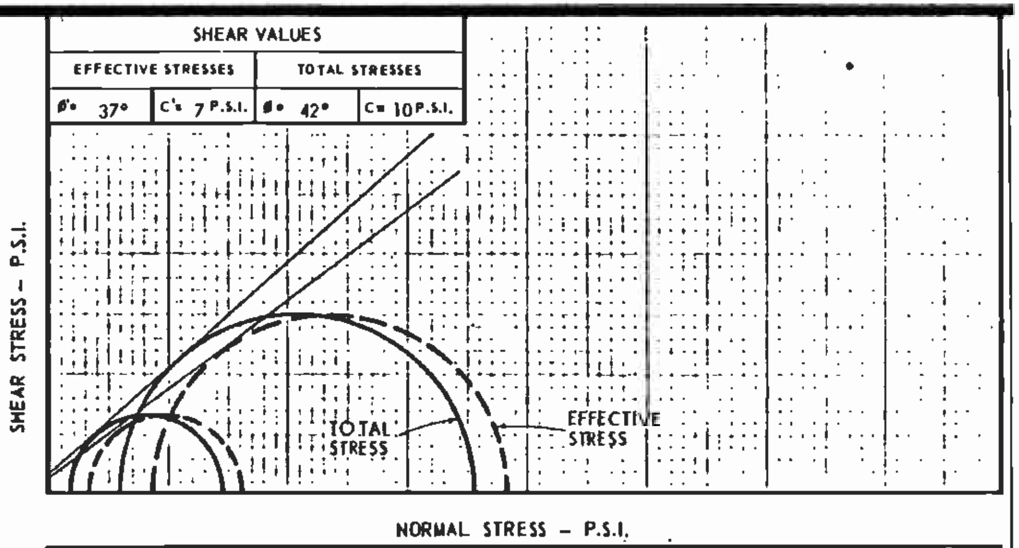
Converse Consultants Geotechnical Engineering and Applied Sciences

Scale As Shown
Date Feb., 1984
Prepared by KLH
Checked by JAD
Approved by JAD
Project No 83-1140
Figure No F-25









SHEAR VALUES			
EFFECTIVE STRESSES		TOTAL STRESSES	
ø= 37°	C'= 7 P.S.I.	ø= 42°	C= 10 P.S.I.

SPECIMEN NUMBER	SPECIMEN LOCATION		INITIAL SPECIMEN DATA					TYPE OF SAMPLE
	BORING NUMBER	DEPTH IN FEET	SOIL CLASSI-FICATION	LENGTH IN INCHES	DIAMETER IN INCHES	DRY DENSITY (P.C.F.)	MOISTURE CONTENT IN PERCENT	
C-1	3-18	10' 7"	SP	5.02	2.42	97	15	5 RING (CONVERSE)
C-1	3-18	10' 7"	SP	5.02	2.42	97	15	

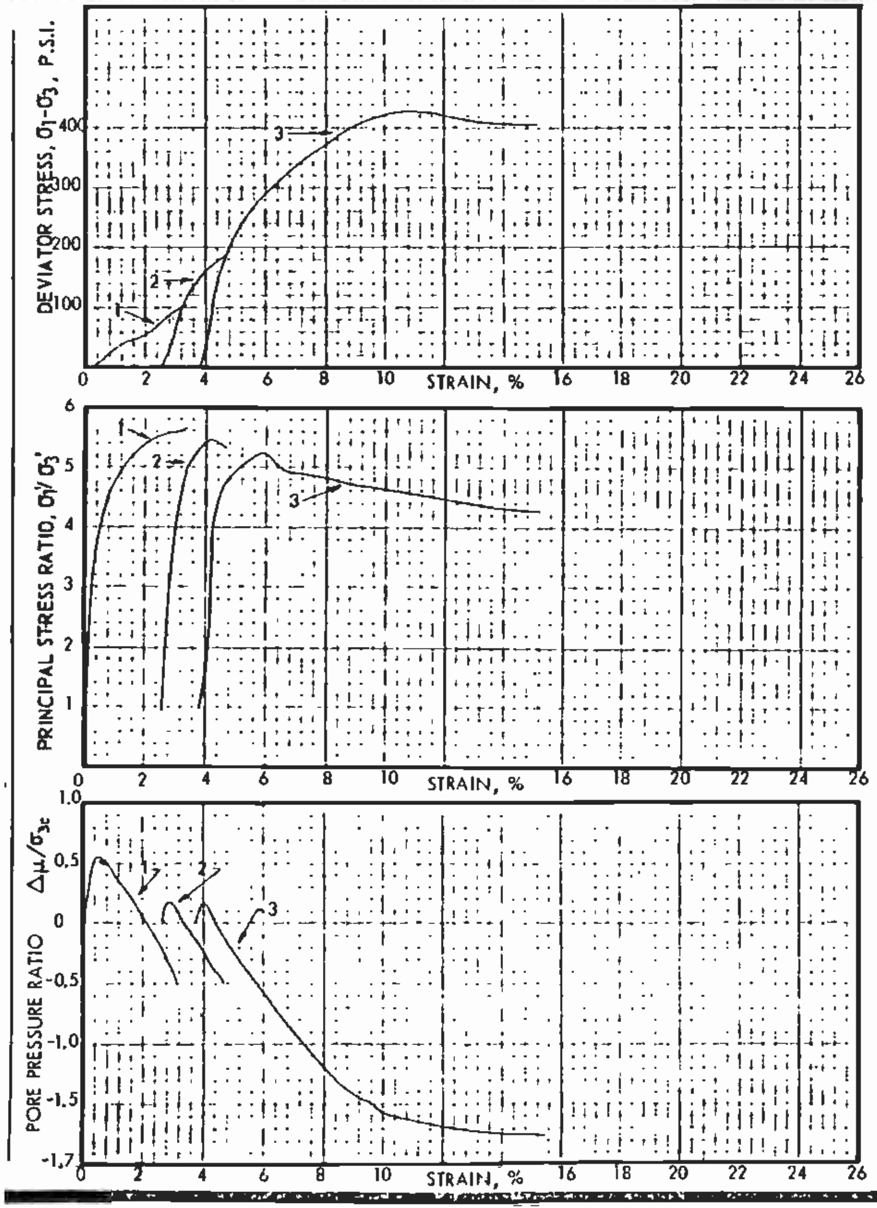
SYMBOLS	SPECIMEN NUMBER	APPLIED LATERAL PRESSURE σ_3 (P.S.I.)	MAXIMUM DEVIATOR STRESS $\sigma_1 - \sigma_3$ (P.S.I.)	TEST VALUES AT FAILURE – MAXIMUM σ_1'/σ_3'		BACK PRESSURE (P.S.I.)	TYPE OF TEST
				EFFECTIVE LATERAL PRESSURE σ_3' (P.S.I.)	MAJOR EFFECTIVE STRESS σ_1' (P.S.I.)		
1	C-1	10	81.2	17.7	80.2	50	PROGRESSIVE CUE
2	C-1	30	225.0	44.0	192.2	50	

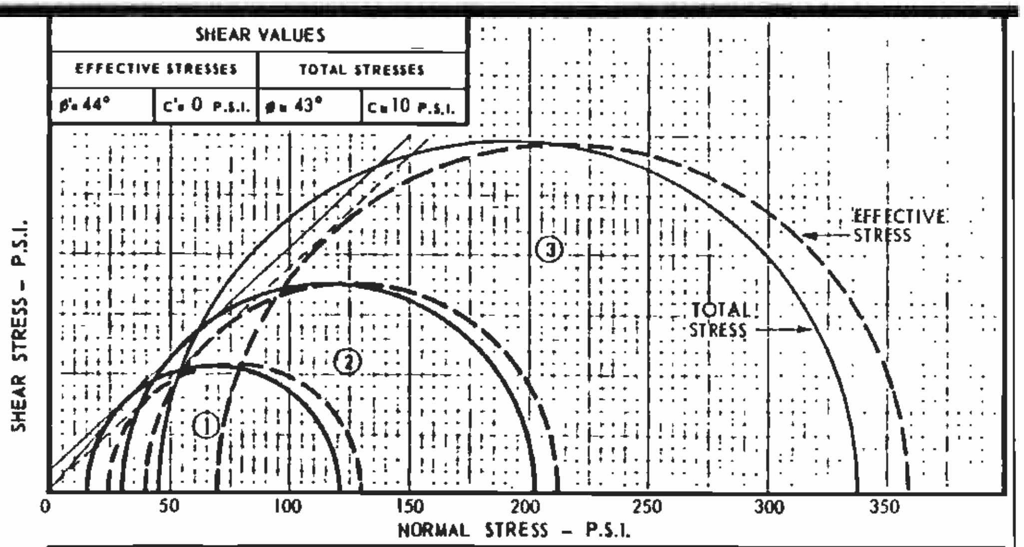
TRIAXIAL COMPRESSION TEST

DESIGN UNIT A100
Southern California Rapid Transit District
METRO RAIL PROJECT

Converse Consultants Geotechnical Engineering and Applied Sciences

Scale As Shown
Date Feb., 1984
Prepared by KLF
Checked by JAD
Approved by JAD
Project No 83-1140
Figure No F-26





SHEAR VALUES			
EFFECTIVE STRESSES		TOTAL STRESSES	
ø'= 44°	c'= 0 P.S.I.	ø= 43°	c=10 P.S.I.

SPECIMEN NUMBER	SPECIMEN LOCATION		INITIAL SPECIMEN DATA					TYPE OF SAMPLE
	BORING NUMBER	DEPTH IN FEET	SOIL CLASSIFICATION	LENGTH IN INCHES	DIAMETER IN INCHES	DRY DENSITY (P.C.F.)	MOISTURE CONTENT IN PERCENT	
C - 7	B - 10	75'7"-76'0"	SW / SM	4.933"	2.42"	118.2	13.35	5 RING CONVERSE SAMPLE
C - 7	B - 10	75'7"-76'0"	SW / SM	4.933"	2.42"	118.2	13.35	
C - 7	B - 10	75'7"-76'0"	SW / SM	4.933"	2.42"	118.2	13.35	

SYMBOLS	SPECIMEN NUMBER	APPLIED LATERAL PRESSURE σ_{3c} (P.S.I.)	MAXIMUM DEVIATOR STRESS $\sigma_1 - \sigma_3$ (P.S.I.)	TEST VALUES AT FAILURE - MAXIMUM σ_1'/σ_3'		BACK PRESSURE (P.S.I.)	TYPE OF TEST
				EFFECTIVE LATERAL PRESSURE σ_3' (P.S.I.)	MAJOR EFFECTIVE STRESS σ_1' (P.S.I.)		
1	C - 7	15	106.3	22.9	129.2	70	PROGRESSIVE CUE TX NO. 4
2	C - 7	30	173.3	39.0	212.3	70	
3	C - 7	45	292.6	68.6	361.2	70	

TRIAXIAL COMPRESSION TEST

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Southern California Rapid Transit District
METRO RAIL PROJECT

Converse Consultants
Geotechnical Engineering and Applied Sciences

Scale As Shown
Date
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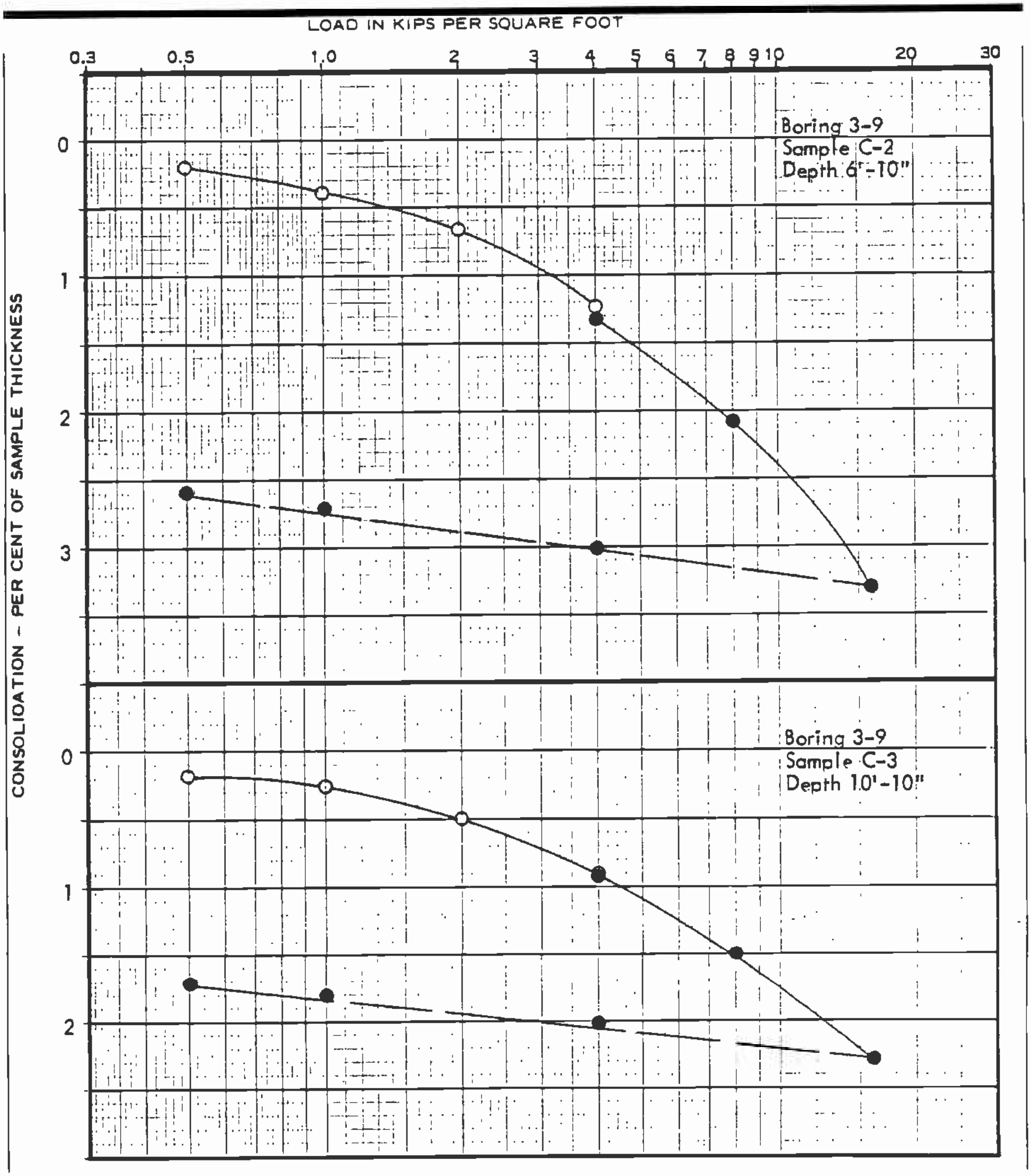
Checked by KDM
Approved by JAD
Project No 83-1140
Figure No F-27



● READINGS AFTER SATURATION WITH WATER

CONSOLIDATION TESTS

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Southern California Rapid Transit District
METRO RAIL PROJECT

Project No
83-1140-11

Figure No.
F-28

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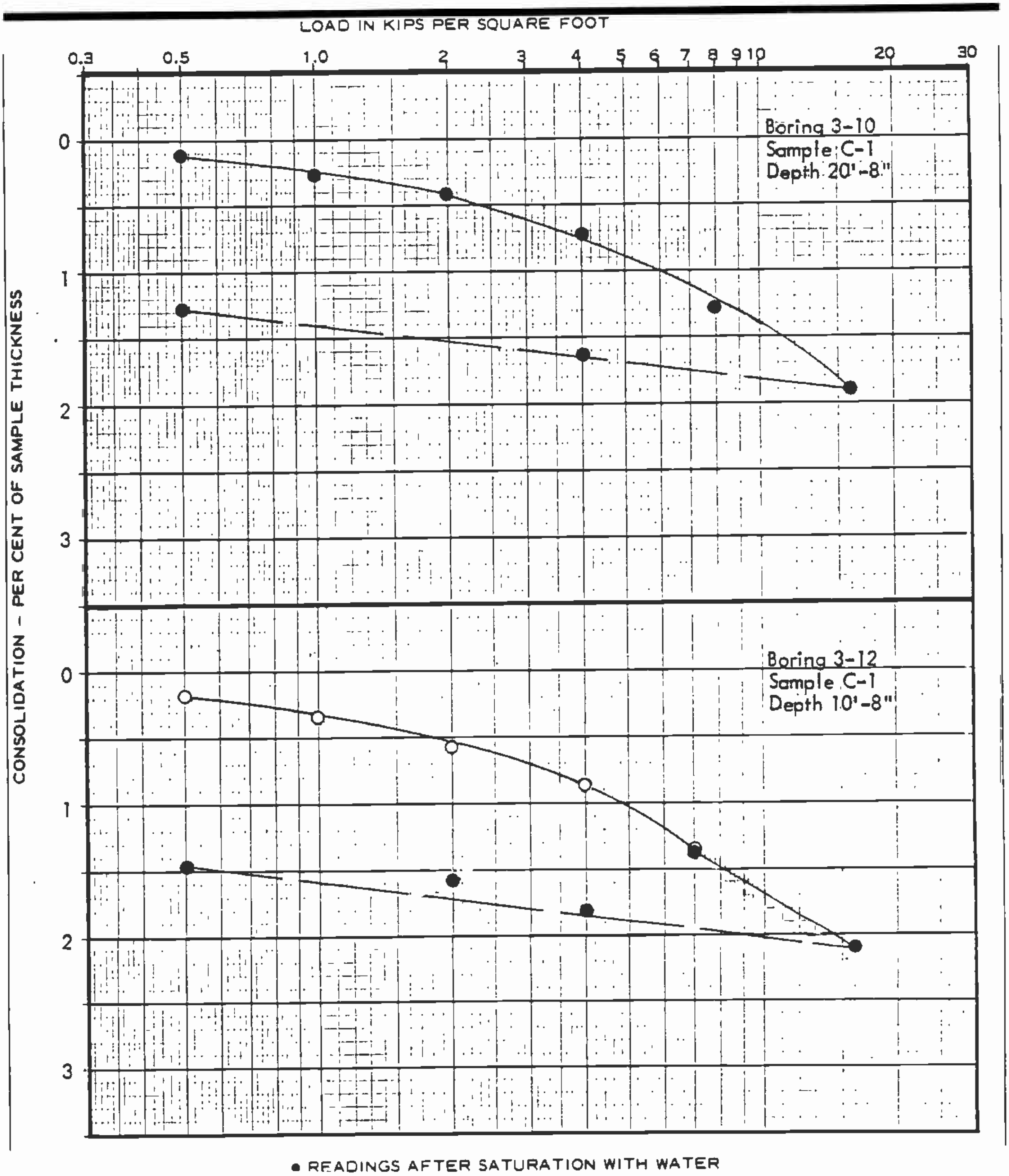

CONSOLIDATION TESTS

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Southern California Rapid Transit District
METRO RAIL PROJECT

Project No
83-1140-01

Figure No
F-29

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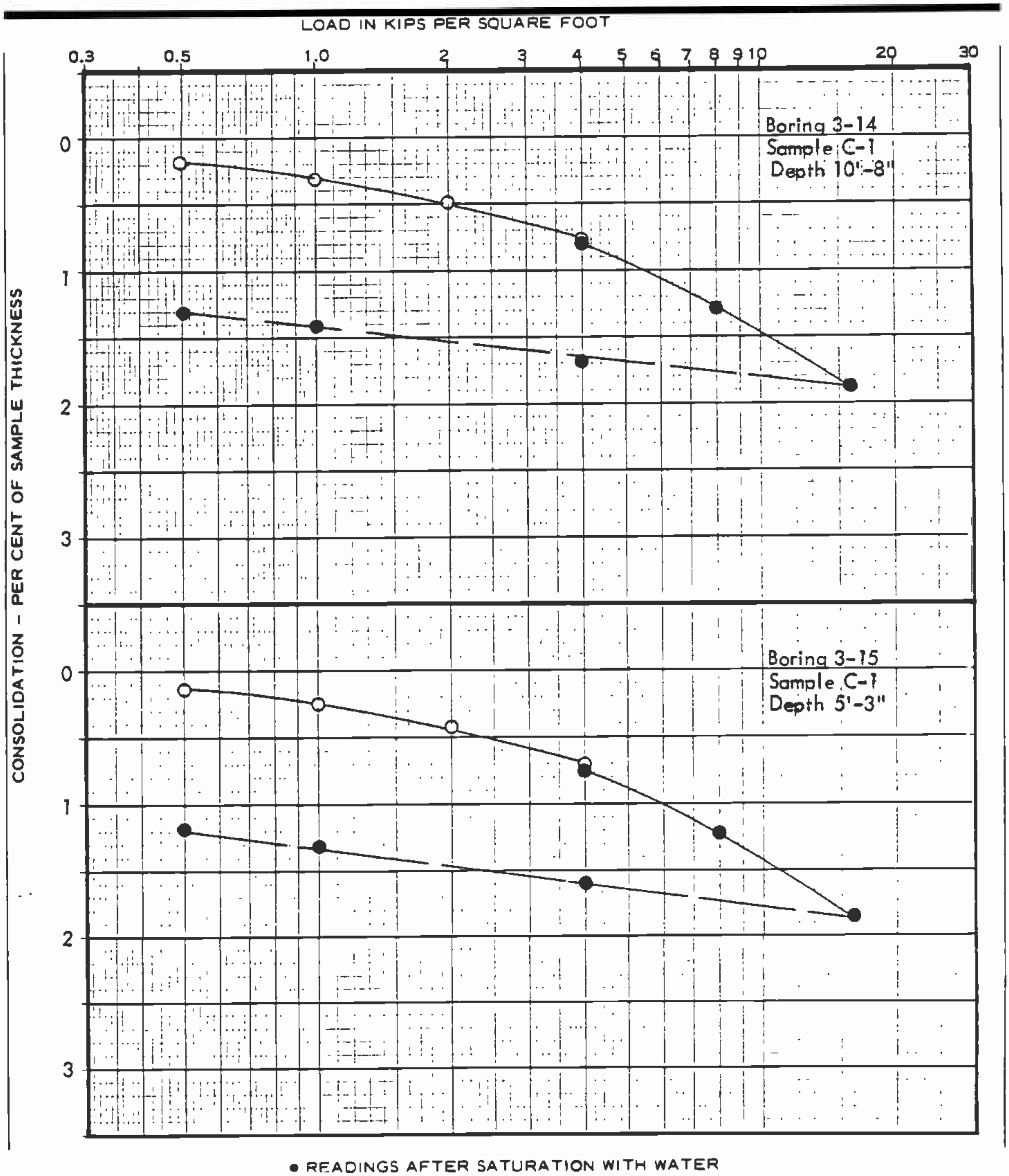

CONSOLIDATION TESTS

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Southern California Rapid Transit District
METRO RAIL PROJECT

Project No
83-1140-11

Figure No.
F-30

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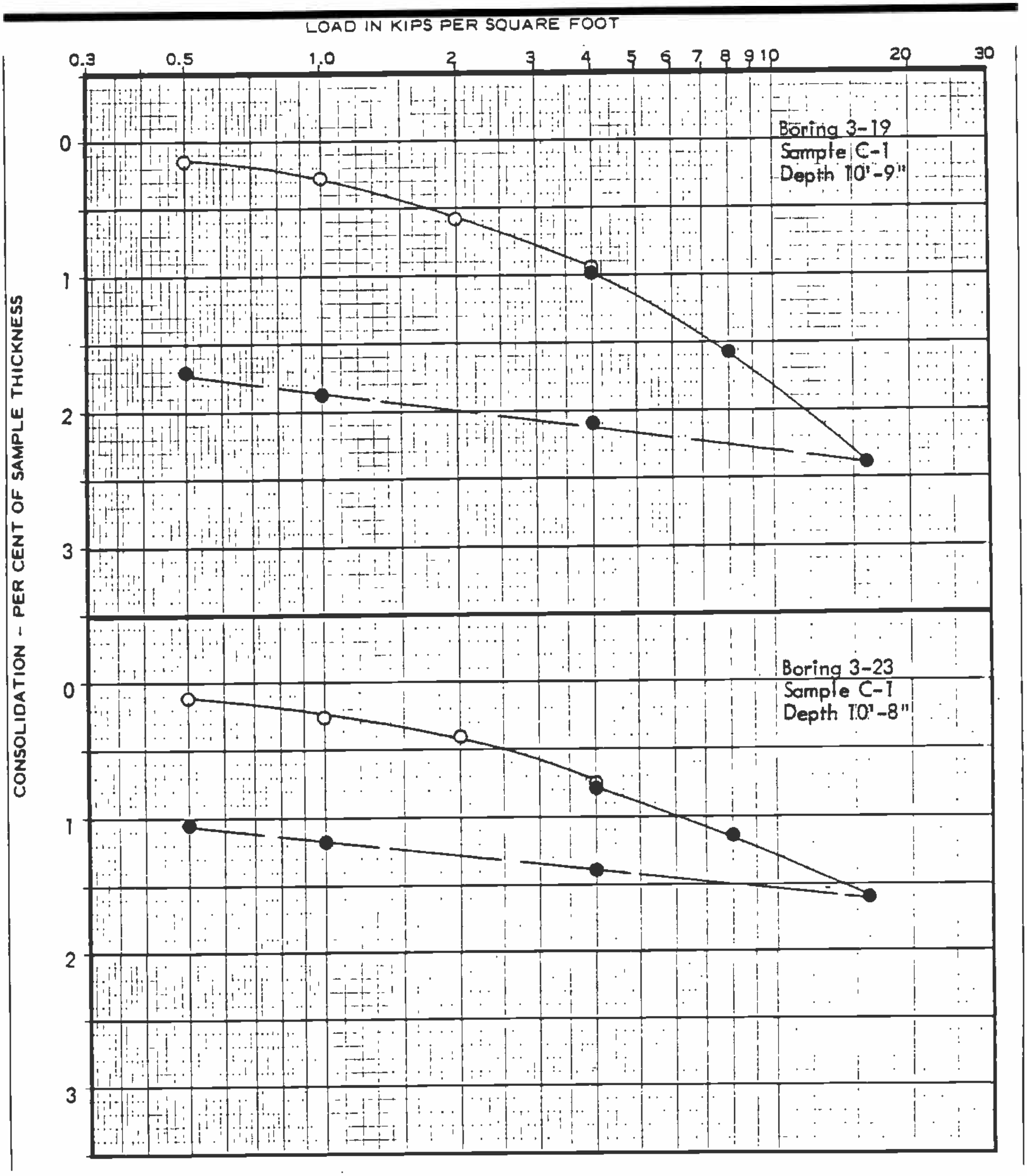

● READINGS AFTER SATURATION WITH WATER

CONSOLIDATION TESTS

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Southern California Rapid Transit District
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Project No
83-1140-11

Figure No.
F-31

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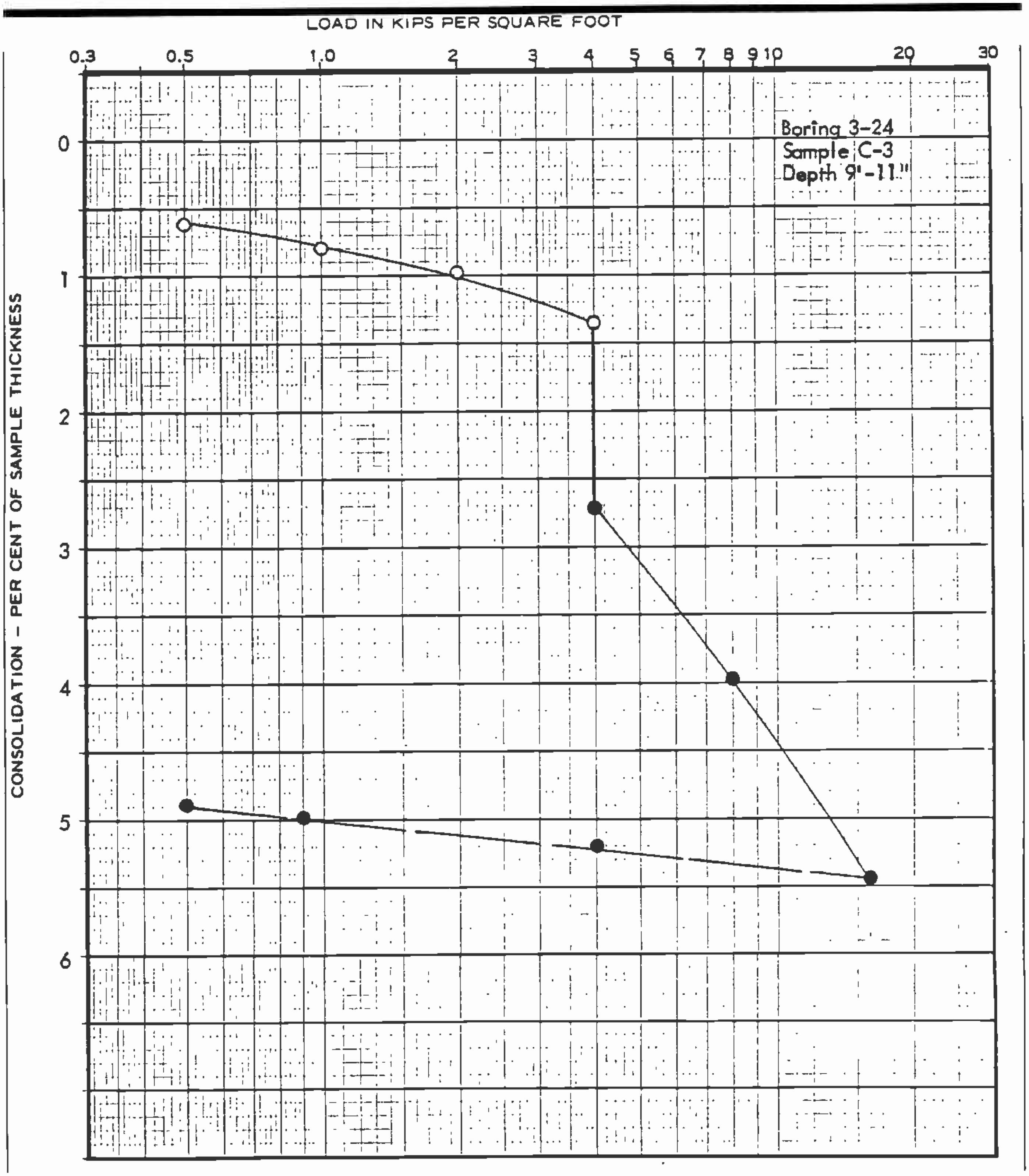

● READINGS AFTER SATURATION WITH WATER

CONSOLIDATION TESTS

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Southern California Rapid Transit District
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Project No
83-1140-11

Figure No.
F-32

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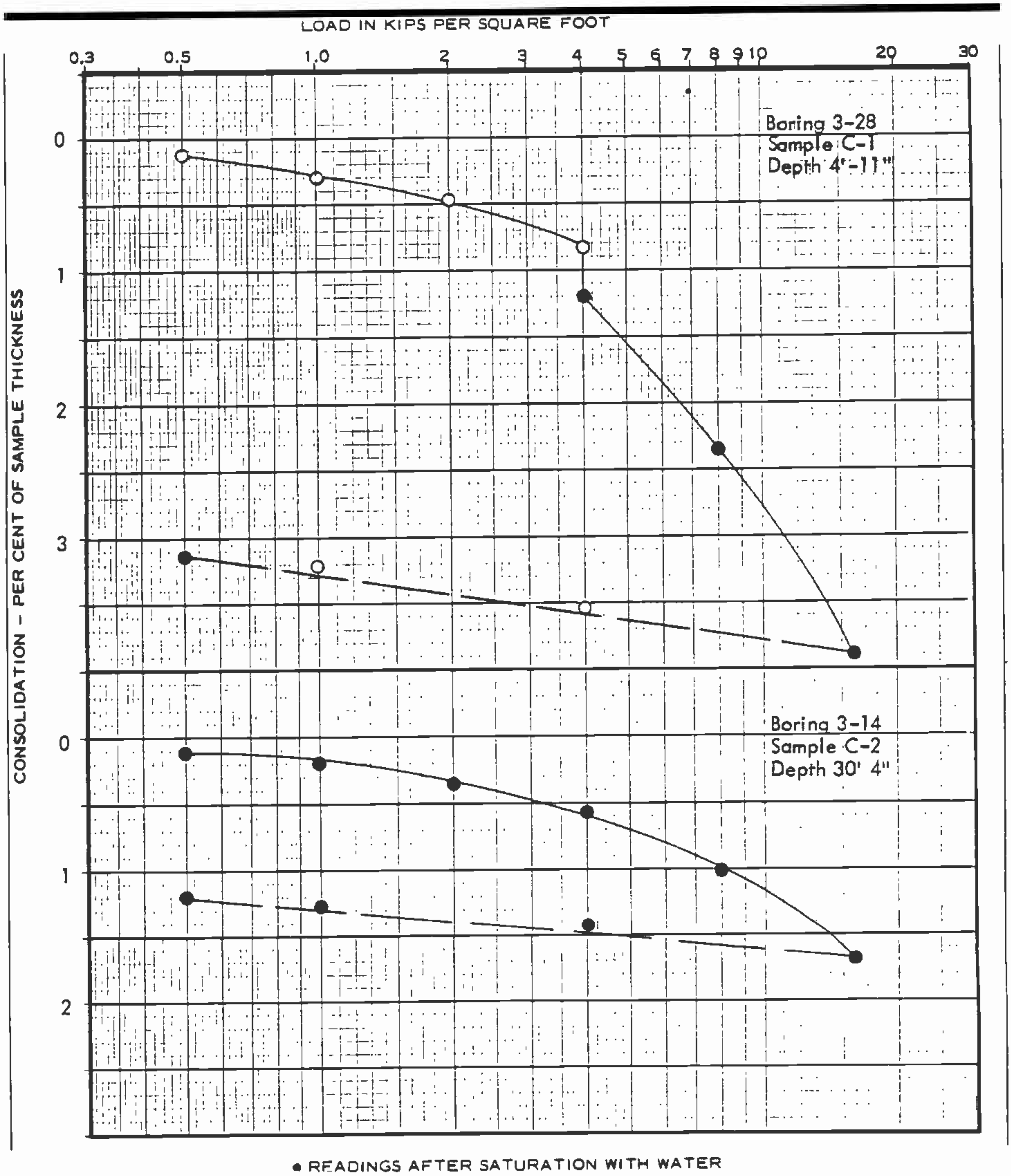

CONSOLIDATION TESTS

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METRO RAIL PROJECT

Project No
83-1140-11

Figure No.
F-33

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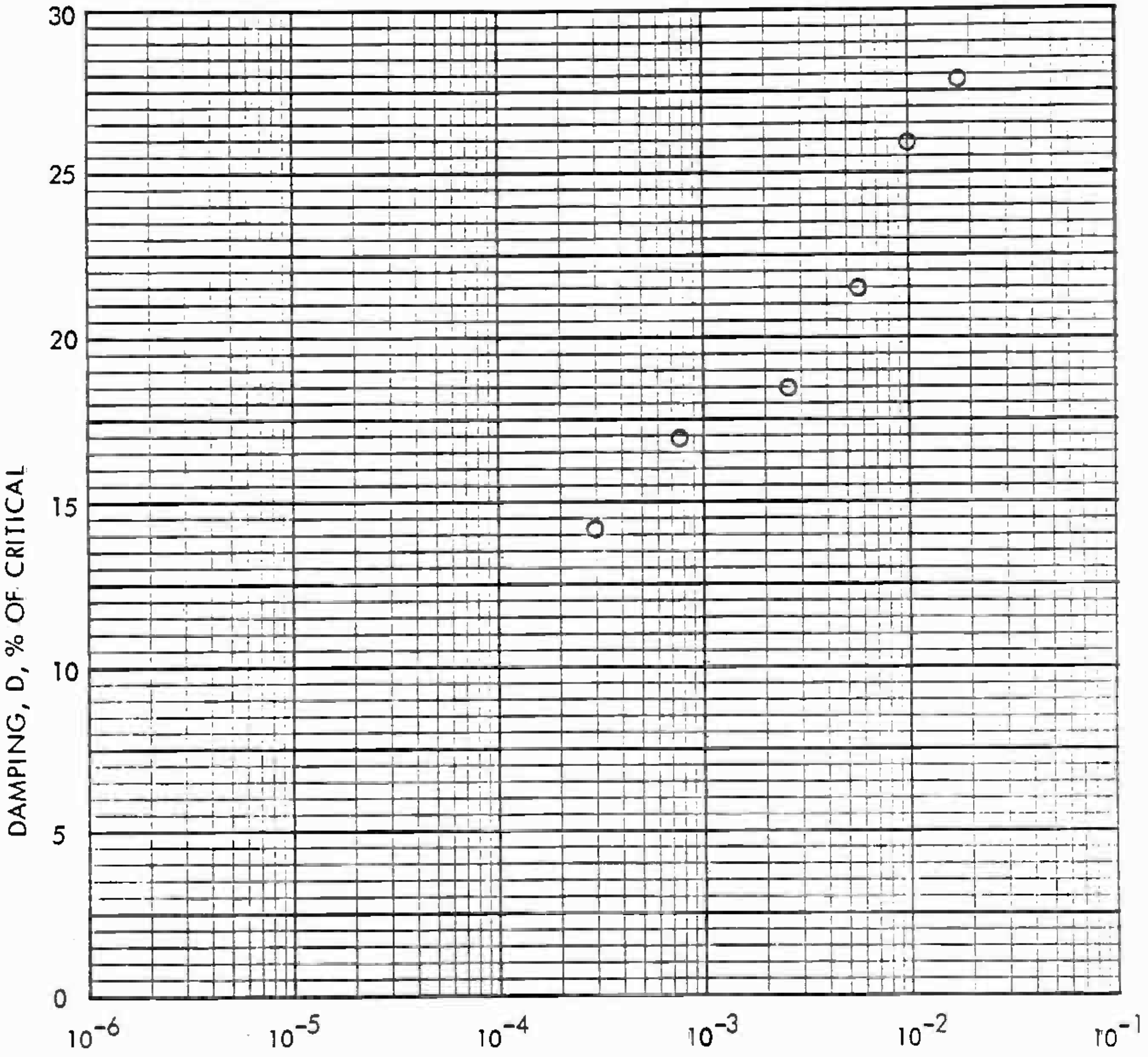
STRAIN DEPENDENT DAMPING



SHEAR STRAIN, Y

BORING	SAMPLE	DEPTH(FT)	γ_d(PCF)	w_o(%)	$\bar{\sigma}_c$(PSI)
2	S-2	28	73	38	30

Sample Description: Clayey Siltstone

DYNAMIC TRIAXIAL TEST

DESIGN UNIT A100
Southern California Rapid Transit District
METRO RAIL PROJECT

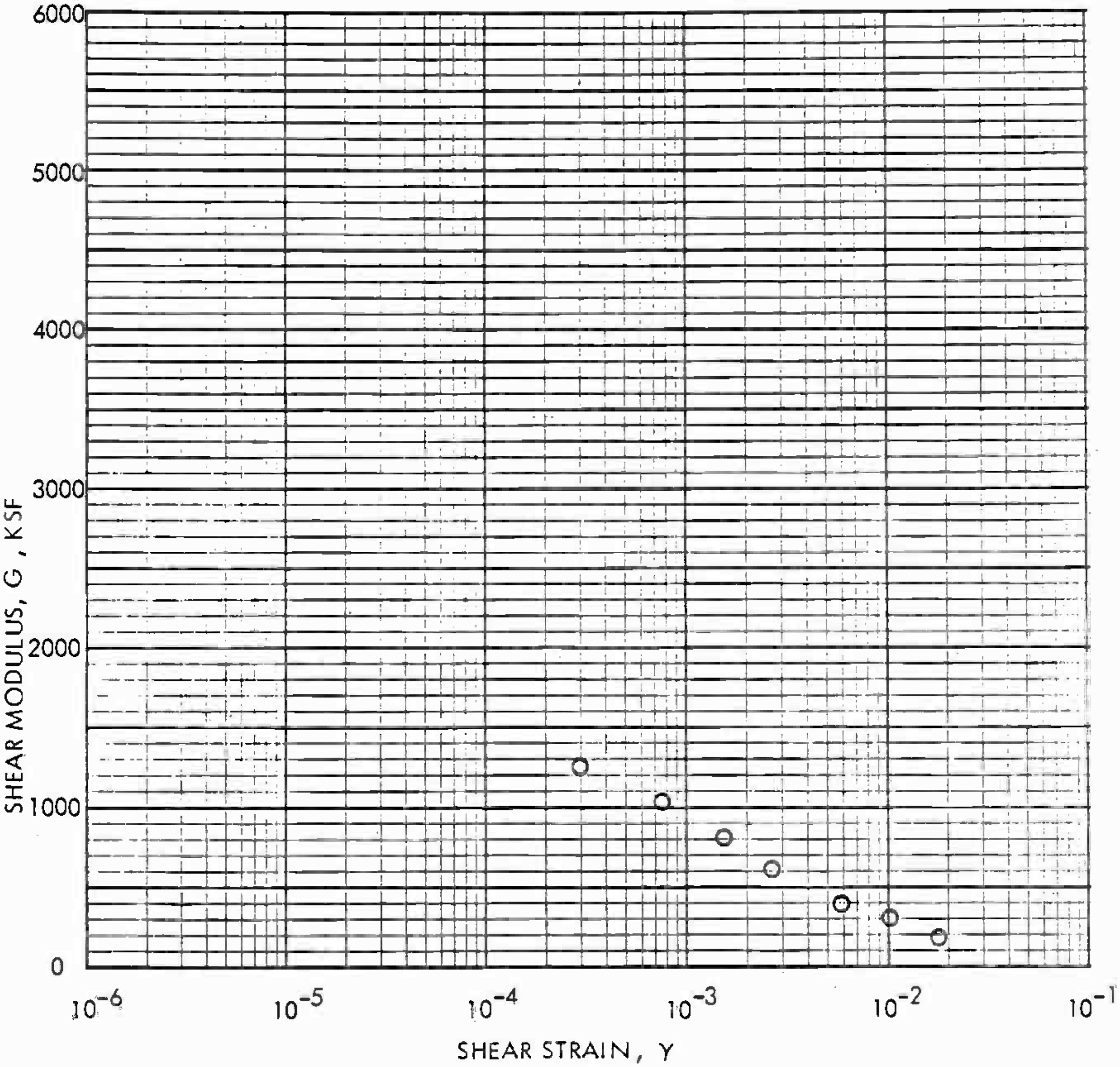
Project No.
83-1140-11

Converse Consultants Geotechnical Consultants

Figure No.
F-34

Approved for publication ______ by ______

STRAIN DEPENDENT DYNAMIC SHEAR MODULUS



BORING	SAMPLE	DEPTH(FT)	γd(PCF)	w_o(%)	$\bar{\sigma}_c$(PSI)
2	S-2	28	73	38	30

Sample Description: Clayey Siltstone

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DYNAMIC TRIAXIAL TEST

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Southern California Rapid Transit District
METRO RAIL PROJ CT

Project No.
83-1140-11

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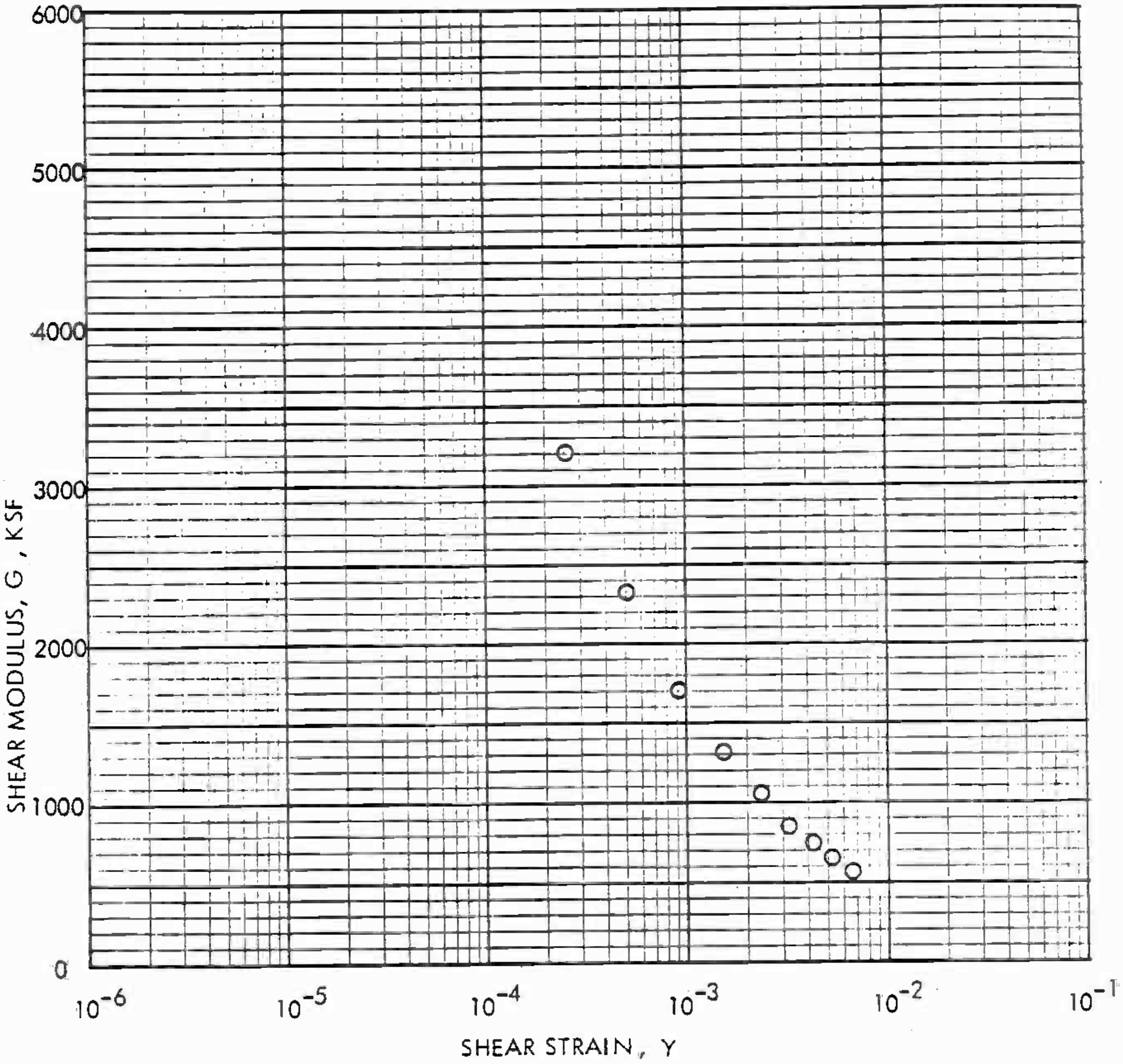
Figure No.
F-35

STRAIN DEPENDENT DYNAMIC SHEAR MODULUS



BORING	SAMPLE	DEPTH(FT)	γ_d(PCF)	w_o(%)	$\bar{\sigma}_c$(PSI)
3	S-3	140	89	30	30

Sample Description: Dark Gray Silty Claystone

DYNAMIC TRIAXIAL TEST

DESIGN UNIT A100
Southern California Rapid Transit District
METRO RAIL PROJECT

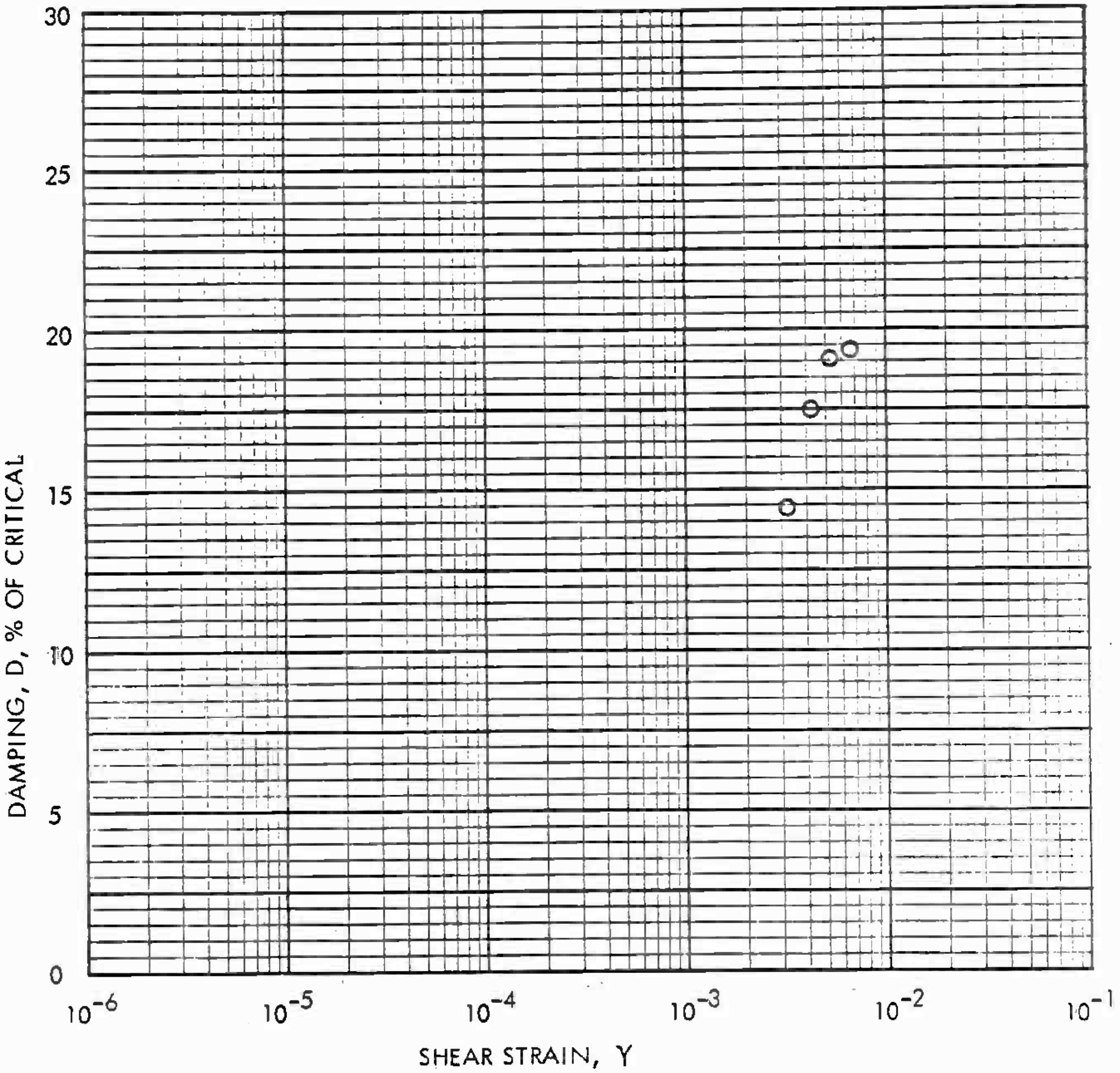
Project No.
83-1140-11

Converse Consultants Geotechnical Consultants

Figure No.
F-36

Approved for publication ______ by ______

STRAIN DEPENDENT DAMPING



BORING	SAMPLE	DEPTH(FT)	γ_d(PCF)	w_o(%)	$\bar{\sigma}_c$(PSI)
3	S-3	140	89	30	30

Sample Description: Dark Gray Silty Claystone

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DYNAMIC TRIAXIAL TEST

DESIGN UNIT A100
Southern California Rapid Transit District
METRO RAIL PROJECT

Project No
83-1140-11

Figure No.
F-37

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Appendix G

Technical Considerations

APPENDIX G TECHNICAL CONSIDERATIONS

G.1 SHORING PRACTICES IN THE LOS ANGELES AREA

G.1.1 General

Deep excavations for building basements in the Los Angeles area are commonly supported with soldier piles with tieback anchors. Although not excavated into the type of dense sands and gravels anticipated at the Union Station site, three case studies of deep tieback excavations in the Los Angeles area are summarized below. These examples are not to be construed to mean that similar installations will be equally successful for the cut-and-cover portions of Design Unit A100.

G.1.2 Atlantic Richfield Project (Nelson, 1973)

This project involved three separate shored excavations up to 112 feet in depth in the siltstones of the Fernando Formation. The project is located just north of Boring CEG-9 (1-1/2 blocks), and the proposed location of the Flower Street Station. Key elements of the design and construction included:

- Basic subsurface material was a soft siltstone with a confined compressive strength in the range of 5 to 10 ksf. It contained some very hard layers, seldom more than 2 feet thick. All materials were excavated without ripping, using conventional equipment. Up to 32 feet of silty and sandy alluvium overlaid the siltstone.
- Volume of water inflow was small and excavations were described as typically dry.
- Shoring system consisted of steel, wide flange (WF) soldier piles set in pre-drilled holes, backfilled with structural concrete in the "toe" and a lean concrete mix above. The soldier pile spacing was typically 6 feet.
- Tieback anchors consisted of both belled and high-capacity friction anchors.
- On the side of one of the excavations a 0.66H:1V (horizontal:vertical) unsupported cut, 110 feet in height, was excavated and sprayed with an asphalt emulsion to prevent drying and erosion.
- Timber lagging was not used between the soldier piles in the siltstone unit. However, an asphalt emulsion spray and wire mesh welded to the piles was used.
- The garage excavation (when 65 feet deep) survived the February 9, 1971 San Fernando earthquake (6.4 Richter magnitude) without detectable movement. The excavation is about 20 miles from the epicenter and experienced an acceleration of about 0.1g. The shoring system at the plaza, using belled anchors, moved laterally an average of about 4 inches toward the excavation at the tops of the piles, and surface subsidence was on the order of 1 inch; surface cracks developed on the street, but there was no structural damage to adjacent buildings. Subsequent shoring used high capacity friction anchors and reportedly moved laterally less than 2 inches.

G.1.3 Century City Theme Towers (Crandall, 1977)

This project involved a shored excavation between 70 and 110 feet deep in Old Alluvium deposits. Immediately adjacent to the excavation (about 20 feet away), there was a bridge structure supported on piles 60 feet below the ground surface. The project is located about one mile west of Boring CEG-20 and the proposed location of the Fairfax Avenue Station. Key elements of the design and construction included:

- Basic subsurface materials were stiff clays and dense silty sands and sands. The permanent ground water table was below the level of the excavation, although minor seeps from perched ground water were encountered.
- Shoring system consisted of steel WF soldier piles placed in 36-inch diameter drilled holes spaced 6 feet on center.
- As the excavation proceeded, pneumatic concrete was placed incrementally in horizontal strips to create the finished exterior wall. The concrete which was shot against the earth acted as the lagging between soldier piles.
- Tieback anchors consisted of high-capacity 12- and 16-inch diameter friction anchors.
- Actual load imposed on the wall by the adjacent bridge was computed and added to the design wall pressures as a triangular pressure distribution.
- Maximum horizontal deflection at the top of the wall was 3 inches, while the typical deflection was less than 1 inch. Adjacent to the existing bridge, the deflections were essentially zero, with the tops of most of the soldier piles actually moving into the ground due to the high prestress loads in the anchors.
- Survey of the bridge pile caps indicated practically no movement.

G.1.4 St. Vincent's Hospital (Crandall, 1977)

This project involved a shored excavation up to 70 feet deep into the claystones and siltstones of the Puente Formation. Immediately adjacent to the excavation (about 25 feet away) was an existing 8-story hospital building with one basement level supported on spread footings. The project is located about 1/3 mile north of Boring CEG-11 and the proposed location of the Alvarado Street Station. Key elements of the design and construction included:

- Basic subsurface materials were shale and sandstone, with a bedding dip to the south at angles ranging from 20° to 40°. Although the permanent ground water level was below the excavation level, perched zones of significant water seepage were encountered.
- Shoring system consisted of steel WF soldier piles placed in 20-inch diameter drilled holes spaced at 6 feet on center.

- Tieback anchors consisted of high-capacity friction anchors.
- Theoretical load imposed on the wall by the adjacent building was computed and added to the design wall pressure. The existing building was not underpinned; thus, the shoring system was relied upon to support the existing building loads.
- Shoring performed well, with maximum lateral wall deflection of about 1 inch and typical deflections less than 1/4 inch. There was no measurable movement of the reference points on the existing building.

G.1.5 Design Lateral Load Practices

Table G-1 summarizes the design lateral loads used for nine shored excavations in the general site vicinity. Based on these projects, the average equivalent uniform pressure used for excavations in alluvium is 15.6H-psf (H = depth of the excavation). For excavations in the Puente or Fernando the average value used is 14.5H-psf.

According to Terzaghi and Peck's rules, the design pressure in granular soils would be equal to 0.65 times the active earth pressure. Assuming a friction angle of 37-degrees, the equivalent design pressure should equal about 22H-psf. For hard clays, the recommended value ranges from 0.15-.30 (equivalent rectangular distribution) times the soils unit weight or at least 18H-psf.

TABLE G-1

SHORING LOADS IN LOS ANGELES AREA

PROJECT LOCATION	EXCAVATION DEPTH (ft)	SOIL CONDITIONS	ACTUAL DESIGN PRESSURE (P)
Broadway Plaza Near 7th/Flower Station	15 to 30	Fill over Alluvium Sands	19.0H
500 South Hill	25	Fill over Sands & Gravel	22.0H
Tishman Building Wilshire/Normandie Station	25	Alluvium-Clays, Sand, Silt	19.0H
Equitable Life Wilshire/Mariposa Avenues	55	Alluvium Sand/Siltstone	20.0H
Arco Flower St./5th to 6th	70 to 90	Alluvium over Claystone	16.0H
Century City	70 to 110	Alluvium-Clays & Sands	18.0H
St. Vincent's Hospital Near 3rd & Alvarado	70	Thin Alluvium over Puente	15.0H
Oxford Plaza Near 7th/Flower	40	Fill & Alluvium over Siltstone	21.0H
Bank Building* 2nd & San Pedro	40	Alluvium (including Sand & Gravel over Siltstone)	20H

* Considerable caving problems were encountered installing tiebacks in dry gravelly deposits in one section of excavation.

Note:

1. All shoring systems were soldier piles.
2. All pressure diagrams were trapezoidal.
3. Equivalent pressure equals a uniform rectangular distribution.

G.2 SEISMICALLY INDUCED EARTH PRESSURES (Temporary Structures)

The Allowable Building Code stress increases for seismic loading (33%) translates into an allowable uniform seismic earth pressure on the temporary shoring of about magnitude 6H. This earth pressure corresponds to a seismic coefficient (K_h) of about 0.15g and a peak ground acceleration of about 0.23g (using the recommended procedures). Data from Part I Seismological Investigation indicates the 0.23g peak acceleration to have a probability of exceedance less than 5% during an average two-year period (a reasonable construction period). The average recurrence of this ground motion level was indicated to be about 100 to 150 years. Based on consideration of the above, the 6H uniform seismic pressure was recommended for design of the temporary wall (see Figure 6-3).

G.3 LIQUEFACTION ANALYSES

G.3.1 Introduction

The procedures used in this study to evaluate liquefaction potential are based mainly on field measurements during the investigation and the performance of similar soils during previous earthquakes. The field observations made at the Yard and Shops site during the 1983 and the previous geotechnical investigation (1981 Geotechnical Investigation Report) that were used to establish the liquefaction potential of the various soils include:

- Standard Penetration Test (SPT) resistance
- Shear wave velocity measurements
- Observed behavior of soil in the large diameter borehole.

In addition to the field observations listed above, gradations of the soils obtained from the field were compared with gradations of materials which have liquefied during past earthquakes and which are considered most susceptible to liquefaction in laboratory tests.

Each of the field observations (and comparisons) is described in the following text. It should be noted that the observations which have been made in the field only provide a basis upon which to judge the liquefaction potential of the various soils. Our conclusions regarding the liquefaction potential of the soils are generally supported by these observations. However, our conclusions are also based on engineering judgement.

G.3.2 Standard Penetration Resistance

The use of the Standard Penetration Test (SPT) in estimating the liquefaction potential of saturated cohesionless soil deposits has been the topic of many previous investigations. Results of these investigations have recently been summarized by Seed et al (1983). Basically, the method utilizes empirical relationships which have been developed from a comprehensive collection of SPT blow count data obtained from sites where evidence of liquefaction or no liquefaction was known to have taken place during past earthquakes. Empirical relationships that have been recently proposed by Seed et al (1983) are shown in Figure G-1.

While results of the Standard Penetration Test have been generally accepted as a good index upon which to estimate the liquefaction potential of saturated sand deposits, it should be noted that the SPT results cannot be utilized to evaluate the liquefaction potential of soils containing gravels, cobbles or boulders. Since much of the Young Alluvium which underlay the Yard and Shops site contains gravel and cobbles, the SPT blow counts recorded in the soils cannot be specifically relied upon. However, for those soils which did not include significant percentages of gravel-sized particles (namely the finer gravelly sand units within the main gravelly sand unit and the lower sands), SPT blow count data were utilized along with the relationships shown in Figure G-1. In general, the SPT blow count measurements taken in the non-gravelly Young Alluvium are around 100 blows per foot indicating that these soils are generally dense to very dense. These blow counts along with the relationship shown in Figure G-1 suggest that liquefaction of the soil deposits during strong earthquake ground shaking would be highly unlikely.

G.3.3 Shear Wave Velocity Measurements

Cross-hole measurements used for the determination of seismic wave velocities along the proposed SCRTD Metro Rail Project alignment were performed as part of the initial 1981 geotechnical investigation. One of the cross-hole surveys was performed within 1,000 feet of the center of the proposed Union Station site in Boring CEG-5. Shear wave velocities measured in the Young Alluvium (approximately the upper 85 feet of the borehole) range between 950±50 fps to 1,400±90 fps. Most of the shear wave velocities measured, however, were about 1,100 fps.

While shear wave velocity in the past has not been as widely accepted as SPT blow count data for estimating the liquefaction potential of a soil deposit, it has received recent attention (Seed et al, 1983). Figure G-1 suggests that liquefaction will never occur during any earthquake if the shear wave velocity in the upper 50 feet of soil exceeds 1,200 fps. Since the shear wave velocities measured close to the Yard and Shops site are approximately 1,100 fps, this is an indication that liquefaction at the site would probably be unlikely.

G.3.4 Observed Soil Behavior

Observations made during the field exploration program may also provide information which may be useful in determining whether or not the saturated cohesionless soils found in the Yard and Shops area are susceptible to liquefaction. The logging of the large diameter boring 6A, and 3-31 through 3-33 allowed the following observations to be made:

- The soils between the ground surface and a depth of 12 feet belled and caved during logging, and between a depth of 15 and 22 feet the soils only experienced slight caving, and between a depth of 22 and 55 feet the hole did not cave.
- Boring 3-31 experienced caving at 15 feet. Boring 3-32 experienced caving between 7 and 20 feet (depth of boring drilled), and Boring 3-33 experienced caving and belling between depths 9 and 17 feet.

These observations are somewhat remarkable in that the soils in these boreholes were generally granular. Seepage was noted only in Boring 6A at a depth of about 18 feet.

If the soils which make up the Young Alluvium were in fact loose (and therefore more susceptible to liquefaction) one would have expected excessive caving of the walls of the boreholes. Instead, the walls stood unsupported (even with ground water seepage) suggesting that the soils are at least medium dense. It is our judgement that the behavior of the soils in these boreholes are generally indicative of a soil that would not have a tendency to liquefy during strong earthquake ground shaking.

G.3.5 Gradational Characteristics

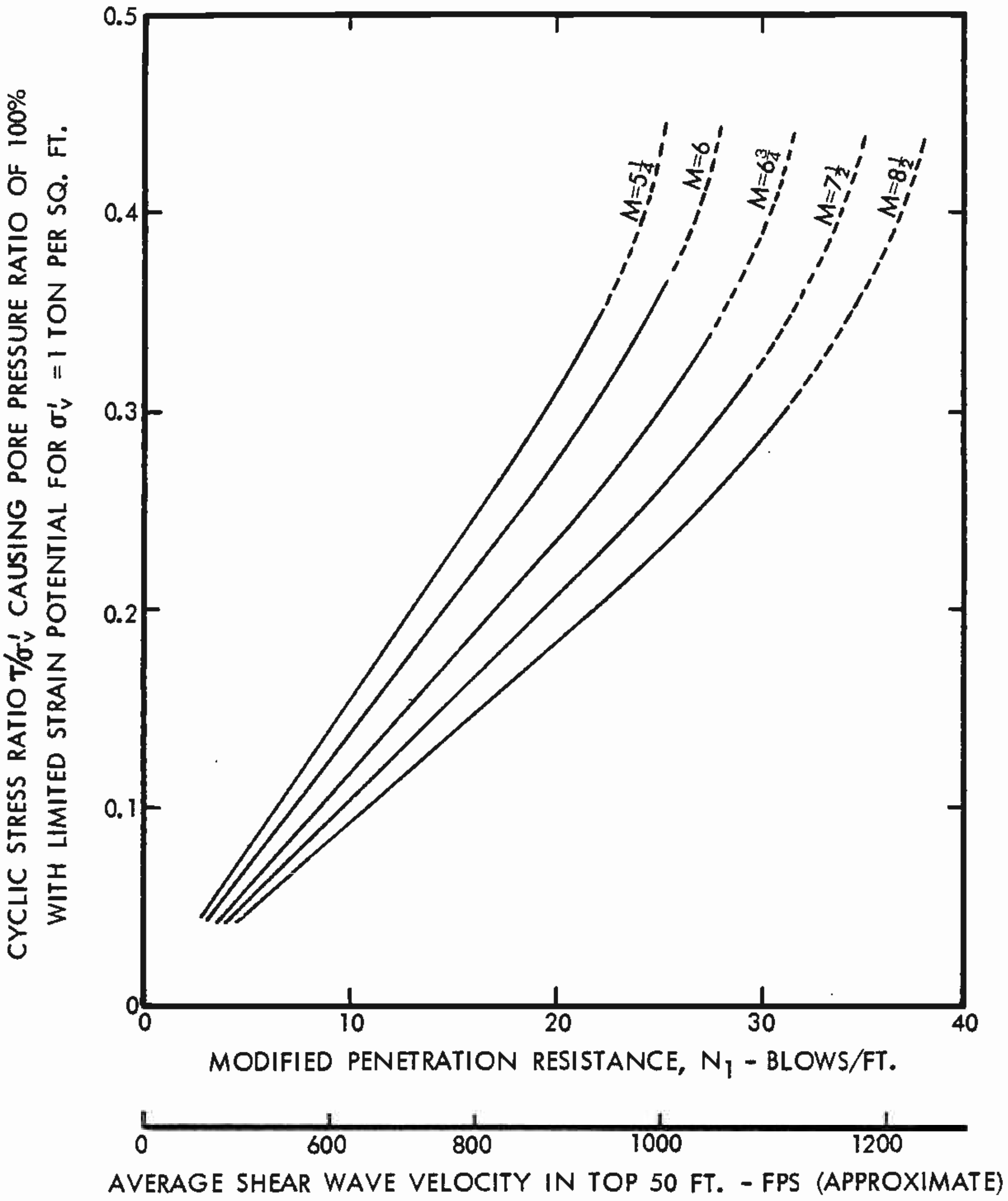
Another factor which may be considered in evaluating the liquefaction potential of a soil is the gradation characteristics of the material. A compilation of the ranges of gradational characteristics of soils which have liquefied during past earthquakes and/or are considered most susceptible to liquefaction in the laboratory is shown in Figure G-2. The ranges shown in this figure have been complied by Lee and Fitton (1968), Seed and Idriss (1967), Kishida (1969), and Youd (1982) and appear to indicate that the soils types most susceptible to liquefaction consist of primarily poorly grade silty sands and sandy silts. It is important to note that all the gradation ranges shown in Figure G-2 have less than 10 percent by weight clay size particles (i.e., particles less than 0.002 mm) suggesting that clayey (cohesive) soils have a low liquefaction potential. Gradation characteristics typical of gravels and gravelly soils are also absent from Figure G-2 suggesting, in part, that these types of soils may not be capable of developing high excess pore pressure either because they are capable of draining rapidly during the cyclic loading or because these types of materials are usually more efficiently packed (i.e., denser) in situ than soils that consist of uniformly-sized particles. While the liquefaction potential of a soil is dependent on many factors other than gradation (such as the relative density of the soil, the intensity and duration of cyclic loading, among others), comparisons of the gradation characteristics of a soil with those ranges shown in Figure G-2 provides a useful guide in establishing the liquefaction potential of a soil.

The gradation characteristics of the various soils which comprise the Young Alluvium were compiled from laboratory tests performed during the 1983 and the previous 1981 investigations. The comparisons of the gradations with the ranges of gradations of the "liquefiable" sandy soils shown in Figure G-2 are presented in Figure G-3.

This figure indicated that five of the 11 samples tested fall within the range of gradations of soils considered "susceptible" to liquefaction. However, it should be noted that three of these samples were obtained near the "lower" sand unit which had high SPT blow counts. The remaining two samples were obtained from samples taken from the gravelly sands. The comparisons shown in Figure G-3 indicates that, on the basis of gradation alone, there appears to be some soils present at the site which may be considered liquefiable.

G.3.6 Conclusions

Based on the above considerations and comparisons, it is our judgement that the Young Alluvium soil deposits found at the Yard and Shops site would not be subject to liquefaction during strong ground shaking produced at the site by the postulated earthquake motions.



(after Seed, 1983)

CORRELATION BETWEEN PENETRATION RESISTANCE AND FIELD LIQUEFACTION BEHAVIOR OF SANDS FOR LEVEL GROUND CONDITIONS

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83-1140

Figure No.
G-1

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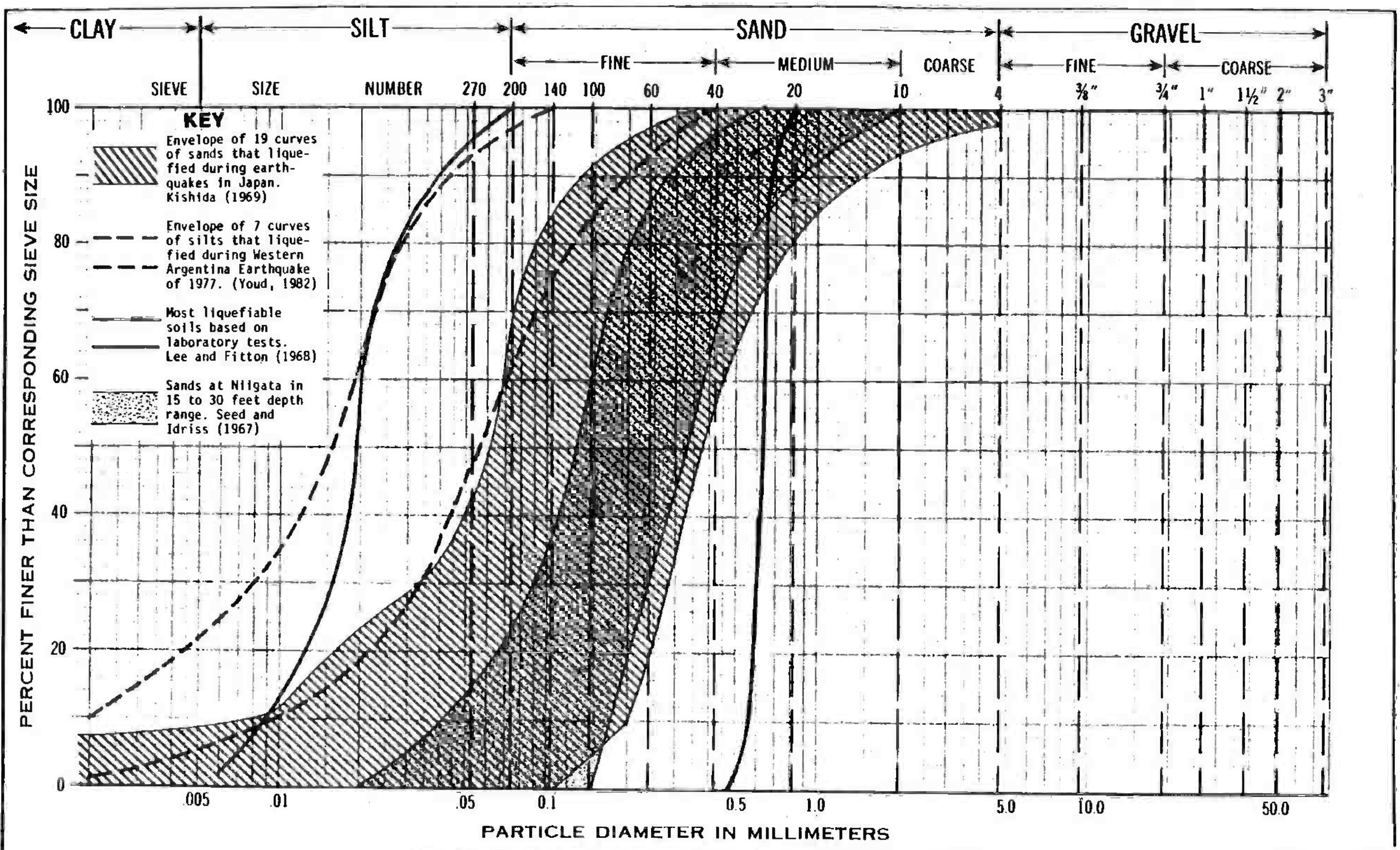

GRADATIONS OF SOILS CONSIDERED SUSCEPTIBLE TO LIQUEFACTION

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83-1101

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Figure No.
G-2

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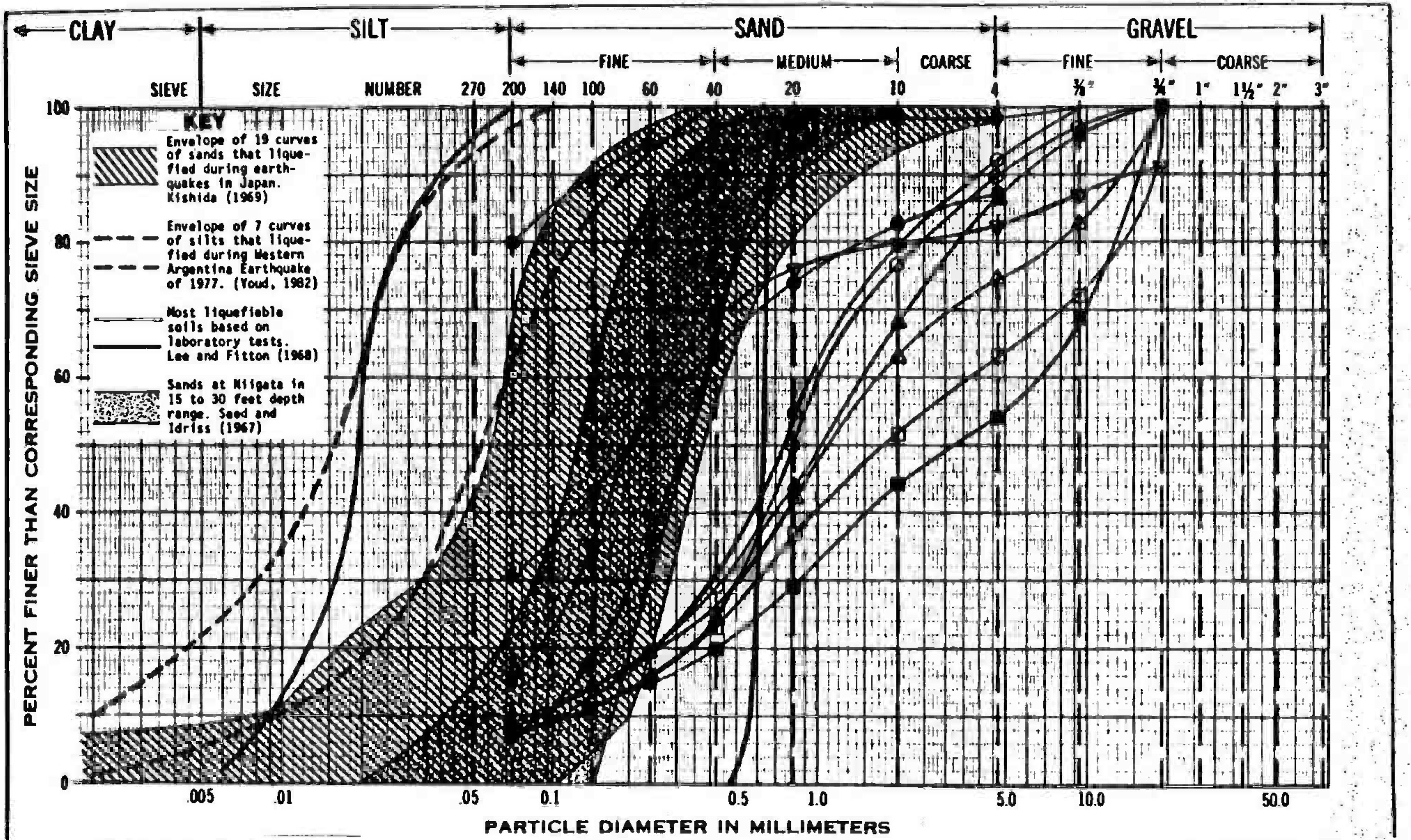

Symbol	Boring	Sample	Depth
○	3-1	J-3	23'-24.5'
●	3-2	C-2	15'1"-15'6"
□	3-3	J-3	22'6"
■	3-4	J-2	16'5"
△	3-6	J-4	26'0"
▲	3-7	J-3	16'-16'5"
⬡	3-8	J-3	16'7"
⬢	3-9	C-5	25'8"
▽	3-11	J-1	6'-6'5"
▼	3-12	C-1	10'7"-11'0"

GRAIN-SIZE DISTRIBUTION CHART

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83-1140-11

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Geotechnical Engineering
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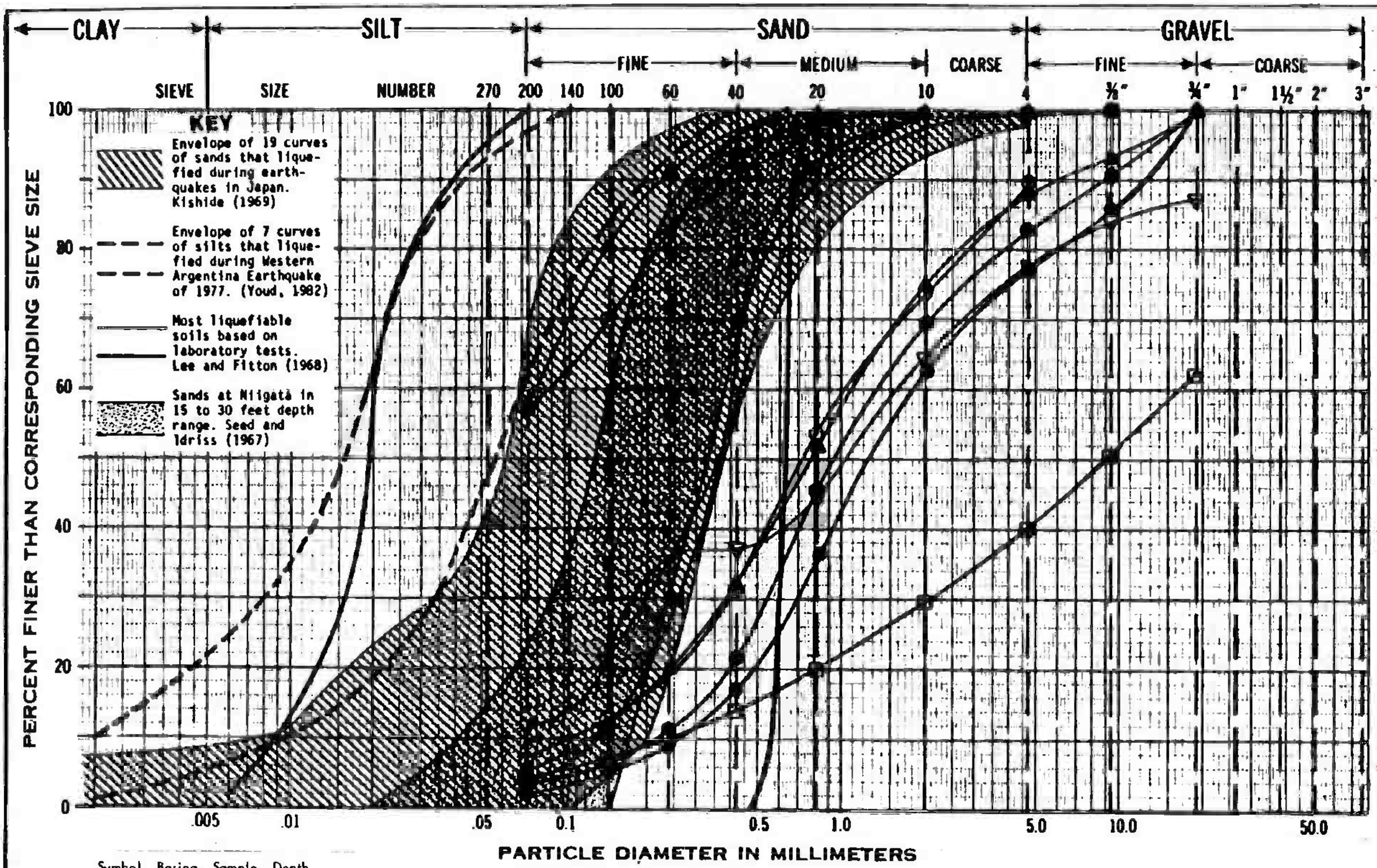
Figure No.
G-3

FORM NO. D-   Approved for publication ________ by ________



Symbol	Boring	Sample	Depth
○	3-13	C-1	10'11"
●	3-14	C-2	30'1"-30'3"
□	3-15	C-4	30'2"-30'6"
■	3-18	C-1	11'0"
△	3-20	J-2	16'6"
▲	3-21	J-1	6'-6.5'
○	3-22	J-1	6'-6.5'
⊛	3-23	C-1	10'8"-10'10"
▽	3-24	C-4	14'6"

GRAIN-SIZE DISTRIBUTION CHART

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83-1140-11

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Figure No.
G-4

Appendix H

Earthwork Recommendations

APPENDIX H - EARTHWORK RECOMMENDATIONS

The following guidelines are recommended for earthwork associated with site development. Recommendations for dewatering and major temporary excavations are presented in the text sections 6.3 and 6.5 respectively.

- Site Preparation (surface structures): Existing vegetation, debris, and soft or loose soils should be stripped from the areas that are to be graded. Soils containing more than 1% by weight of organics may be re-used in planter areas, but should not be used for fill beneath building and paved areas. Organic debris, trash, and rubble should be removed from the site. Subsoil conditions on the site may vary from those encountered in the borings. Therefore, the soils engineer should observe the prepared graded area prior to the placement of fill.

- Minor Construction Excavations: Temporary dry excavations for foundations or utilities may be made vertically to depths up to 5 feet. For deeper dry excavations in existing fill or natural materials up to 15 feet, excavations should be sloped no steeper than 1:1 (horizontal to vertical). Recommendations for major sloped excavations are presented in Section 6.5.

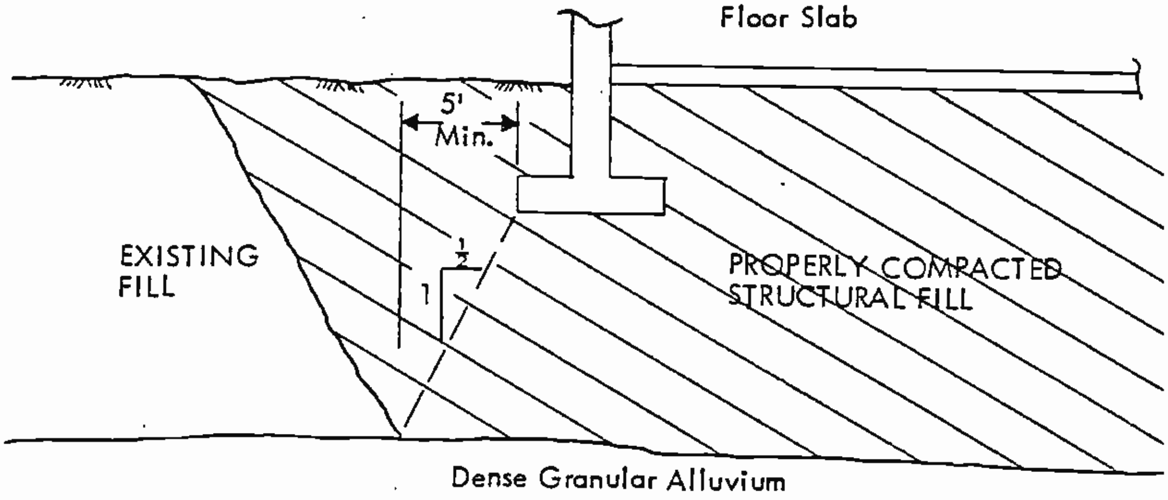
- Structural Fill and Backfill: Where required for support of near surface foundations or where subterranean walls and/or footings require backfilling, excavated onsite granular soils or imported granular soils are suitable for use as structural fill. Loose soil, formwork and debris should be removed prior to backfilling the walls.. Onsite soils or imported granular soils should be placed and compacted in accordance with "Recommended Specifications for Fill Compaction". In deep fill areas or fill areas for support of settlement-sensitive structures, compaction requirements should be increased from the normal 90% to 95% or 100% of the maximum dry density to reduce fill settlement.

  Oversized fill consisting of cobbles and boulders greater than 6 inches but less than 3 feet may be placed in the deeper portions of structural fill at depths greater than 8 feet below rough grade and at least 5 feet (horizontally or vertically) away from existing or proposed structures. All oversized fill materials should be properly windrowed to reduce the potential for voids in the fill after placement. All oversized fill should be placed and compacted in accordance with "Recommended Specifications for Placement of Oversized Material".

  Where space limitations do not allow for conventional backfill compaction operations, special backfill materials and procedures may be required. Sand-cement slurry, pea gravel or other selected backfill can be used in limited space areas. Sand-cement slurry should contain at least 1-1/2 sacks cement per cubic yard. Pea gravel should be placed in a moist condition or should be wetted at the time of placement. Densification should be accomplished by vibratory equipment; e.g., hand-operated mechanical compactor, backhoe mounted hydraulic compactor, or concrete vibrator. Lift thickness should be consistent with the type of compactor used. However, lifts should never exceed 5 feet. A soils engineer experienced in the placement of pea gravel should observe the placement and densification procedures to render an opinion as to the adequate densification of the pea gravel.

If granular backfill or pea gravel is placed in an area of surface drainage, the backfill should be capped with at least 18 inches of relatively impervious type soil; i.e., soils containing at least 40 percent passing the No. 200 sieve.

- Foundation Preparation: Where foundations for near surface appurtenant structures are underlain by existing loose or soft fill soils, or unacceptable fill materials, the existing fill should be excavated and replaced with a zone of properly compacted structural fill. The zone of structural fill should extend to undisturbed dense or stiff natural soils. Horizontal limits of the structural fill zone should extend out from the footing edge a distance equal to 5 feet or 1/2 the depth of the zone beneath the footing whichever is larger. The structural fill should be placed and compacted as recommended under "Structural Fill and Backfill".



- Subgrade Preparation: Concrete slabs-on-grade at the subterranean levels may be supported directly on undisturbed dense alluvium. The subgrade should be proof rolled to detect soft or disturbed areas, and such areas should be excavated and replaced with compacted structural fill. If existing fill soils are encountered in near surface subgrade areas, these materials should be excavated and replaced with properly compacted structural fill. All structural fill for support of slabs or mats should be placed and compacted as recommended under "Structural Fill and Backfill".

- Site Drainage: Adequate positive drainage should be provided away from the surface structures to prevent water from ponding and to reduce percolation of water into the subsoils. A desirable slope for surface drainage is 2% in landscaped areas and 1% in paved areas. Planters and landscaped areas adjacent to the surface structures should be designed to minimize water infiltration into the subsoils.

- Utility Trenches: Buried utility conduits should be bedded and backfilled around the conduit in accordance with the project specifications. Where conduit underlies concrete slabs-on-grade and pavement, the remaining trench backfill above the pipe should be placed and compacted in accordance with "Structural Fill and Backfill".

- Recommended Specifications for Fill Compaction: The following specifications are recommended to provide a basis for quality control during the placement of compacted fill.

1. All areas that are to receive compacted fill shall be observed by the soils engineer prior to the placement of fill.

2. Soil surfaces that will receive compacted fill shall be scarified to a depth of at least 6inches. The scarified soil shall be moisture-conditioned to obtain soil moisture near optimum moisture content. The scarified soil shall be compacted to a minimum relative compaction of 90%. Relative compaction is defined as the ratio of the inplace soil density to the maximum dry density as determined by the ASTM D1557-70 compaction test method.

3. Fill shall be placed in controlled layers the thickness of which is compatible with the type of compaction equipment used. The thickness of the compacted fill layer shall not exceed the maximum allowable thickness of 8 inches. Each layer shall be compacted to a minimum relative compaction of 90%. The field density of the compacted soil shall be determined by the ASTM D1556-64 test method or equivalent.

4. Fill soils shall consist of excavated onsite soils essentially cleaned of organic and deleterious material or imported soils approved by the soils engineer. All imported soil shall be granular and non-expansive or of low expansion potential (plasticity index less than 15%). The soils engineer shall evaluate and/or test the import material for its conformance with the specifications prior to its delivery to the site. The contractor shall notify the soils engineer 72 hours prior to importing the fill to the site. Rocks larger than 6 inches in diameter shall not be used unless they are placed in accordance with "Specifications for Placement of Oversized Material."

5. The soils engineer shall observe the placement of compacted fill and conduct inplace field density tests on the compacted fill to check for adequate moisture content and the required relative compaction. Where less than 90% relative compaction is indicated, additional compactive effort shall be applied and the soil moisture-conditioned as necessary until 90% relative compaction is attained. The contractor shall provide level testing pads for the soils engineer to conduct the field density tests on.

- Recommended Specifications for Placement of Oversized Material: The following specifications are recommended to provide a basis for quality control during placement of oversized material during grading:

1. All areas that are to receive compacted fill shall be observed by the soils engineer prior to the placement of fill.

2. The exposed natural soil surface shall be scarified and compacted to at least the specified density, with adjustment of the moisture content where necessary prior to placement of fill.

3. Non-organic oversized material such as cobbles and boulders shall be placed in windrows in the deep fill areas such that no oversized material lies within 8 feet of the finish grade or 5 feet of existing or proposed structures. The maximum dimension of oversized material shall not exceed 3 feet. Oversized debris shall be placed in windrows approximately 3 feet wide and 2 to 3 feet in height. Windrows should be spaced horizontally at least 10 feet apart.

4. Approved onsite or imported soils shall be placed and compacted in lifts between and around the windrowed material in accordance with applicable specifications for soil fill. The windrows shall be track-rolled with a CAT-D8H dozer or similar equipment and flooded repeatedly with water to the satisfaction of the soils engineer as the soil fill is placed and compacted around the windrows.

5. Soil fill placement and compaction shall continue to a level at least 2 feet above the windrowed material prior to placing additional oversized material. Subsequent windrows shall be staggered between the underlying windrows such that new windrows do not overlie the previous.

6. The soils engineer shall observe the placement, flooding and track-rolling of the windrowed material as well as the placement and compaction of the soil fill around the windrowed material. Where inadequate flooding, track-rolling or compactive effort is indicated, additional effort shall be applied as necessary.