Converse Consultants
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GEOTECHNICAL REPORT

METRO RAIL PROJECT

DESIGN UNIT A275

BY

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

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April 24, 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 A275 prepared in accordance with our Contract No. 503 agreement dated September 30, 1983 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 A275.

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,

Robert M. Pride, Senior Vice President
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RMP:m

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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.

Robert M. Pride
Senior Vice President

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Section 1.0

Executive Summary

1.0 EXECUTIVE SUMMARY

This report presents the results of our geotechnical investigation and engineering analyses for the A275 Design Unit of the Southern California Rapid Transit District's Metro Rail Project in Los Angeles. The A275 Design Unit consists of the Beverly/Fairfax Station and crossover structure having a combined length of 960 feet. The station will be constructed by cut-and-cover methods and will extend in depth up to about 55 feet below the existing ground surface. This report defines the subsurface conditions and provides recommendations for design and construction purposes.

1.1 STATION AND CROSSOVER CONSTRUCTION

The subsurface conditions at the station and crossover site consist of 85 to 90 feet of alluvium, primarily silts, clays, clayey sands and silty sands. Minor amounts of tar were observed occasionally in the alluvial soils generally below depths of about 50 feet. However, this minor amount of tar had no apparent effect on strength and consolidation characteristics of the alluvial soils. Underlying the alluvium, the explorations encountered tar sands of the San Pedro formation which are estimated to be between 25 and 30 feet thick. The San Pedro tar sand is in turn underlain by interbedded siltstone, claystone and sandstone of the Fernando Formation which is also impregnated with tar. Ground water was encountered within the alluvium at depths of 4 to 8 feet below the existing ground surface.

Construction of the station and crossover will consist of an excavation approximately 950 feet long, 60 feet wide, and up to about 55 feet deep. The excavation will be entirely within alluvial soils. Temporary support of the construction excavation will be either flexible or rigid type vertical wall systems with internal bracing or external tieback systems. Successful installation of tiebacks will require certain precautions to maintain the stability of the inclined shafts below ground water elevations. Lateral pressures and other guidelines for design of temporary support systems are provided in the report.

Certain fractions of the alluvium are more pervious than other fractions. Therefore, exterior and/or interior dewatering installations are anticipated to be necessary to control ground water seepage and loss of ground along the excavation faces and to maintain the stability of the bottom of the excavation. Dewatering of the alluvium will result in some surface subsidence which should be confined primarily to an area about 100 feet around the dewatering system and, therefore, is not expected to affect any significant nearby structure.

The alluvial soils expected at the subgrade level will adequately support the permanent reinforced concrete station structure. Design lateral pressures for the permanent structure under varying earth and hydrostatic loading conditions are outlined in the text of the report.

1.2 UNDERPINNING

Guidelines for assessing the need for underpinning of buildings adjacent to the Station construction are discussed in the report. Based on the guidelines presented, it appears that all significant buildings are beyond the zone of influence of the proposed excavation. Detailed analyses to identify and recommend which buildings and/or facilities shall be underpinned will be carried out by the section designer for this Design Unit.

1.3 LIQUEFACTION AND SEISMIC DESIGN

The alluvial soils are predominately clayey in nature with limited zones of granular soils. Based on the index properties of the clayey alluvium, these materials are considered non-liquefiable. Analysis of the SPT results of the limited zones of granular alluvial soils indicate a low probability of liquefaction during the operating design earthquake, but liquefaction during the maximum design earthquake may have a moderate to high probability. However, the granular soil inclusions are of limited extent and generally confined within the matrix of non-liquefiable clayey alluvium. Therefore, it is our opinion that liquefaction of the granular zones will not result in catastrophic changes in the overall dynamic soil loads because the clayey soil matrix is expected to maintain its integrity. The tar content and high SPT values indicate the potential for liquefaction of the San Pedro Sands to be very low.

Design procedures and criteria for underground structures under earthquake loading conditions are defined in the SCRTD report entitled "Guidelines for Seismic Design of Underground Structures" dated 1984. Seismological conditions which may impact the project and the operating and maximum design earthquakes which may be anticipated in the Los Angeles area are described in the SCRTD report entitled "Seismological Investigations and Design Criteria" dated May, 1983. The 1984 report complements and supplements the 1983 report. Site specific static and dynamic properties for materials in design unit A275 are provided in the text of this report.

Section 2.0

Introduction

2.0 INTRODUCTION

This report presents the results of a geotechnical investigation for Design Unit A275, Fairfax/Beverly Station and crossover. The work performed for this report includes borings, laboratory tests, engineering analyses, and the development of recommendations and specifications for design and construction of the station and crossover. This Design Unit is a part of the 18.6-mile long Metro Rail Project (see Drawing 1, Vicinity Map).

Additional geotechnical information on the Metro Rail Project is included in the following reports, some of which may pertain to Design Unit A275.

- "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 RTD in November 1981. This report presents general geologic and geotechnical data for the entire project. The report also comments on tunneling and shoring experience and practices in the Los Angeles area.

- "Seismological Investigation & Design Criteria Metro Rail Project", prepared by Converse Consultants, Lindvall Richter & Associates, Earth Sciences Associates and Geo/Resource Consultants, submitted to RTD 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 by 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.

- "Rapid Transit System Backbone Route", Volume IV, Book 1, 2 and 3, prepared by Kaiser Engineers, June, 1962 for the Los Angeles Metropolitan Transit Authority. This report presents the results of a Test Boring Program for the Wilshire Corridor and logs of borings.

The design concepts discussed in this report are based on the "Final Report for the Development of Milestone 10, CBD to North Hollywood Line Plans, Sheets 4 to 6, dated July 1983; and Preliminary Site Plans, Plans and Sections, Sheets 7 to 12, for Fairfax/Beverly Station, dated May, 1983.

Section 3.0

Site and Project Description

3.0 SITE AND PROJECT DESCRIPTION

The Fairfax/Beverly Station and crossover site is located adjacent to Fairfax Avenue between Beverly Avenue and Third Street. The Station will be located off-street on a north-south axis about 100 feet east of and parallel to Fairfax. The north end of the station is currently a surface parking lot for CBS Television City. Immediately to the south of the station and the crossover is Farmer's Market. Other land use in the area is characterized by retail, commercial, and mixed uses along Fairfax and Beverly, with an immediate shift to residential housing on other streets. The land use west of the station is primarily low-density, single-family housing; to the east are medium and high-density apartments. The existing ground surface along Fairfax Avenue varies from Elevation 190 feet at Beverly Boulevard to Elevation 181 feet at the south end of the crossover.

The Fairfax/Beverly Station and crossover will be a reinforced concrete structure about 950 feet long and 60 feet wide (outside wall dimensions). The station is planned with two entrances, each parallel to Fairfax, one located on the north and the other to the south of the station. A bus turnout lane is proposed on the south side of Beverly adjacent to the station entry. A future parking structure accommodating 1,000 parking spaces will be developed for this location, but only surface parking will be provided initially. The two entries planned for this station will provide access to a mezzanine centered over the length of the platform. Ancillary space will be provided at each end of the station, and a double crossover track will be located at the south end of the station. A traction power substation will be located over the crossover track.

The top of rail varies from about Elevation 140 feet at the north end of the station to about Elevation 137 feet at the south end of the crossover. Assuming the station will be supported on a mat-like foundation, the station area will require an excavation to about Elevation 132 feet. This is approximately 55 feet below the existing grade at the north end of the station, and 50 feet below the existing grade at the south end. After the station and crossover is constructed, about 9 to 14 feet of fill will be placed above the majority of the station box, and up to 28 feet of fill above the crossover structure. Design loads for the subsurface structures were not available at the time of this report.

Section 4.0

Field Exploration and Laboratory Testing

4.0 FIELD EXPLORATION AND LABORATORY TESTING

4.1 GENERAL

The information presented in this report is based primarily on the field and laboratory investigations performed in 1981 and 1983. This information was derived from field reconnaissance, borings, geologic reports and maps, ground water measurements, field geophysical surveys, gas chromatographic measurements, petroleum analyses, ground water quality tests, and laboratory tests on soil and rock samples. References listed at the end of this report were utilized to complement and supplement the more recent information.

4.2 BORINGS

For the A275 investigation, 8 borings were drilled at the station site and in the vicinity of the crossover structure. The borings consist of small diameter rotary wash holes numbered 23-1 through 23-5, and a 36-inch diameter man-size auger boring, 23-B. Rotary wash borings CEG-23 drilled in 1981 and 20-10 drilled in 1983 for Design Unit A250 are also included. The locations of the borings are shown on Drawings 2 and 3, and the logs of the borings from the 1981 and 1983 investigations are provided in Appendix A. Standpipe piezometers were installed in Borings CEG-23 and 20-10, although the CEG-23 piezometer is no longer operable. Installation of piezometers in Borings 23-1 through 23-5 was not allowed by the property owner.

None of the 1962 Kaiser Engineers borings were drilled within the A275 Station site. The closest boring is located four blocks north of the site, near Fairfax High School. Another source of boring information is the U.S. Geological Survey paper, "Geologic Aspects of Tunneling in the Los Angeles Area" (USGS Map No. MF-866, 1977). None of the foundation investigation borings included in the USGS report are shown on our drawings and were not used because they were too shallow for proper interpretation of subsurface conditions along the proposed grade of the Station excavation.

4.3 GEOPHYSICAL MEASUREMENTS

Limited downhole compression wave velocity surveys were performed during the initial 1981 investigation in Boring CEG-23 and at nearby Borings CEG-20 and CEG-24. The CEG-23 boring was drilled on the northwest end of the A275 Station site on Fairfax Avenue. Six seismic refraction lines were also conducted in 1981 in the vicinity of Fairfax High School, located about three blocks north of the Station site. Appendix B summarizes the field survey procedures and the results of the velocity measurements at CEG-23 as well as data obtained from CEG-20, CEG-24. Also presented are seismic refraction survey results obtained at Fairfax High School for reference use.

4.4 GEOTECHNICAL LABORATORY TESTING

The laboratory program developed to test representative soil samples consisted of classification tests, consolidation tests, triaxial compression tests, dynamic triaxial tests, unconfined compression tests, direct shear tests, and permeability tests.

Appendix C summarizes the testing procedures and presents detailed results of the 1983 program and summarizes selected results of the 1981 laboratory program.

4.5 OIL AND GAS ANALYSES

Sulfur and petroleum odors were noted at relatively shallow depths in all the borings drilled in the vicinity of the station site. Strong hydrogen sulfide odors were detected at a depth of 27 feet in the man-size auger Boring 23B drilled at the station site. From 27 feet to the bottom of the hole, there was considerable sulfurous odors, and a gas detector noted explosive limits. Minor amounts of petroleum/tar were observed within the soils samples obtained at depths between about 50 and 85 feet. Below about 85 feet, the tar-impregnated San Pedro sands were encountered. During the 1981 investigation gas chromatography analyses and petroleum tests were performed at Boring CEG-23. The results of the 1981 tests are presented in Appendix D.

The Salt Lake Oil Field is located beneath the proposed Station site. This field was first developed in 1903, and has been long known for its large seeps of heavy oil on the north side of Wilshire Boulevard. Tar, oil and gas are present in the underlying Fernando Formation as well as the overlying San Pedro Formation and alluvial deposits. The possibility exists that the project excavations could encounter abandoned oil well casings.

4.6 WATER QUALITY ANALYSES

Chemical analyses and selected parameters of sampled water obtained in Boring CEG-23 were performed as part of the 1981 geotechnical investigation. An artesian water condition was noted in this boring when it was advanced to a depth of 179 feet. The water which flowed out of the hole the day after its completion was sampled and subsequently analyzed. The chemical analyses and the results of these tests are summarized in Appendix E, which indicate poor water quality. Water from Boring 23B was analyzed during the 1983 investigation. Results of tests at CEG-23 and 23B are presented in Appendix E.

Section 5.0

Subsurface Conditions

5.0 SUBSURFACE CONDITIONS

5.1 GENERAL

During the field programs conducted for this investigation and the 1981 investigation, the contact between the Old and Young Alluvium was difficult to identify since the soils in these two units can be very similar. While the Young and Old Alluvium may be geologically different, our interpretation of the field and laboratory test data suggests that they do not differ significantly from an engineering standpoint. For the purposes of this report, Young and Old Alluvium have not been differentiated and are simply referred to as Alluvium.

Drawings 2 and 4 show generalized subsurface cross sections through the proposed Fairfax/Beverly Station and crossover site. The subsurface profile at the Station and the crossover site consists of approximately 0.5 to 2 feet of fill over fine-grained and granular Alluvium extending to depths of approximately 80 feet. Minor amounts of tar were observed to occur occasionally in the alluvial soils below about 50 feet. Boring CEG-23 drilled in 1981, showed that the alluvium extended down to a depth of 88 feet, where tar-impregnated sand, known as the San Pedro Formation, was encountered to a depth of 115 feet. The tar sands were underlain by weathered Fernando Formation bedrock.

5.2 SUBSOILS

Specific descriptions of the soil materials encountered in the borings drilled at the Station site include:

- Fill: Minor amount of fill soils were encountered below surface pavement in six of the eight borings drilled at the site. Fill depths encountered ranged from 0.5 to 2 feet below the surface. The fill generally consisted of relatively clean sandy or silty clay which was stiff and moist.

- Alluvium: Generally fine-grained alluvial soils were encountered in Borings 23-B, and 23-1 through 23-5 to the total depth drilled (approximately 75 feet). Boring CEG-23 showed that the fine-grained alluvium extended to a depth of 88 feet. The alluvium consisted predominately of sandy clay, silty clay, and clayey silt, with zones of clayey sand, sandy silt and silty sand. The various soil types encountered were observed to be relatively thin layers ranging from 2 or 5 feet thick to up to about 35 feet thick. Some general trends of the soil stratification, i.e. silt/clay mixtures vs. sand/clay mixtures, can be seen on Drawing 4; however, specific layers generally appeared to be discontinuous. Minor amounts of tar in the form of stringers were occasionally observed in the alluvial soils below about 50 feet. Some sulfur and petroleum odors were randomly noticeable in the alluvium at depths ranging from about 5 feet to the bottom of the borings (75 feet). Sampling resistance, SPT results and laboratory test results indicate that these soils are generally stiff to hard and have low compressibility.

- San Pedro (Tar) Sand: San Pedro (Tar) Sands encountered below the alluvium in Borings 20-10 and CEG-23 were typical for this formation. Generally, the formation consisted of tar-impregnated medium to fine sand with occasional gravelly sand or silty sand lenses. The total stratum thickness generally ranged from 25 to 30 feet thick. Sampling resistance and SPT results in the tar sands were high.

5.3 BEDROCK

Only two of the eight borings drilled at and adjacent to the site (Borings CEG-23 and 20-10) penetrated into the Fernando Formation bedrock underlying the alluvium and the San Pedro (tar) Sand. Where encountered, the bedrock consisted of claystone or interbedded siltstone and claystone. The bedrock was little weathered to fresh, thinly bedded to massive. Bedding dip was measured in CEG-23 to be approximately 30°. Strike of the bedding could not be determined from the samples obtained. Regional bedding strike is nearly east-west and the dip is north. Sulphur and/or petroleum odors were noted in the bedrock samples from Borings CEG-23 and 20-10.

The San Vicente Fault trace crosses the alignment at about a 45° angle immediately north of the Station site as shown on Drawing 2. As discussed in the 1981 investigation report, the fault location is based on Salt Lake Oil Field data, and is in the Fernando Formation. This fault is not known to be active or potentially active.

5.4 GROUND WATER

Alluvial ground water occurs at depths ranging from about 4 to 8 feet below the surface at the Fairfax/Beverly Station site. Table 5-1 presents ground water levels measured at Borings CEG-23, 23-1 through 23-4 man-size Borehole 23B. Based on the measurements presented on Table 5-1, it appears that the ground water level may slope downward from north to south.

TABLE 5-1
GROUND WATER OBSERVATION WELL DATA*

	GROUND WATER ELEVATION						
BORING	Initial	(Date)	1981	1982 APRIL	1983 FEB.	1983 NOV.	1984 MARCH
CEG-23	178	01-04-81					
20-10							167
23-4						175	
23-3						177	
23-2						179	
23B	181	03-03-83			181		
23			179	178			
23-1						180	

*Rounded to the nearest foot.

5.5 GAS AND PETROLEUM

Gas chromatography tests and petroleum analyses were made at Boring CEG-23. Sulphur and/or petroleum odors from the alluvium and bedrock samples were noted in all borings drilled at the site and its vicinity. In addition tar impregnated samples from the San Pedro Sand in Boring CEG-23 were obtained and examined. Bitumen content tests were performed on representative samples obtained in Borings 23-3 and 23-4. The results of the tests are presented in Appendices C and D.

5.6 ENGINEERING PROPERTIES OF SUBSURFACE MATERIALS

5.6.1 General

For purposes of our engineering evaluations, only the alluvial soils at the Fairfax/Beverly Station site were considered to have a direct impact on engineering design. The San Pedro Sand Formation and Fernando Formation Bedrock were considered to be too deep to affect shoring and permanent wall design. This section includes an engineering description of the alluvial soils and presents engineering parameters used in our analyses (see Table 5-2). These parameters are based on the laboratory test results, field test results, and data from previous investigations.

TABLE 5-2
MATERIAL PROPERTIES SELECTED FOR STATIC DESIGN

MATERIAL PROPERTY	GEOLOGIC UNIT Alluvium
Moist density above ground water (psf)	120
Saturated density (pcf)	125
Effective Strength	
ϕ' (degrees)	30
c' (psf)	300
Total Strength[a]	
ϕ (degrees)	23
c (psf)	800
Average Unconfined Compressive Strength (psf)	4000
Permeability (cm/sec)	10^{-3} to 10^{-6}
Poisson's ratio	0.35
Initial Tangent Modulus (psf)	$225 \cdot \sigma_v'$[b]

[a] The total stress parameters should be used to determine the increase in undrained shear strength with depth.

[b] σ_v' is the effective overburden pressure (psf) equal to effective density times overburden depth. Moist density should be used to determine σ_v' above the water table and submerged density (saturated density minus water density) should be used for the effective density of soils below the water table.

5.6.2 Alluvium

The alluvium consists of interbedded sandy clays, silty clays, clayey silts, clayey sand and silty sands. Standard Penetration Test (SPT) results and laboratory test results indicate that the clayey alluvium is generally stiff to hard, and granular layers are dense to very dense. Minor amounts of tar occur occasionally in alluvial soils generally below depths of 50 feet. However, results of the laboratory tests indicate that the minor amounts of tar had no apparent negative effect on the strength and consolidation characteristics of the alluvial soils.

Since these soils have generally low permeability, both drained (effective) and undrained (total) strength parameters have been developed from results of direct shear and triaxial compression tests. The recommended strength parameters given in Table 5-2 were selected based primarily on the results of tests performed on samples obtained from the Fairfax/Beverly Station site, although strength test results obtained from other nearby design units were also considered.

Young's Modulus or initial tangent modulus were found to be a function of the consolidation pressure. Modulus values for the alluvium were therefore normalized to the consolidation pressure. The normalized values recommended for the alluvium are presented in Table 5-2.

Permeability tests performed on triaxial test samples of alluvium obtained from this and other design units indicate that these soils have permeability ranging from about 10^{-3} to 10^{-8} cm/sec. However, since the soils were found to be interbedded and lenticular, higher permeabilities are recommended for design calculations.

Section 6.0

Geotechnical Evaluation and Design Criteria

6.0 GEOTECHNICAL EVALUATION AND DESIGN CRITERIA

6.1 GENERAL

Construction of the A275 stations and crossover will involve a deep excavation through stiff and dense alluvium to depths of 50 to 60 feet below the ground surface. The proximity of the site to Fairfax Avenue and adjacent development requires that the excavation be shored. High ground water levels at the site will require either preconstruction dewatering or tight shoring with dewatering below the construction excavation. Dense tar sand soils were encountered at depths of about 30 feet below the proposed station subgrade, soils at and above the subgrade level were found to contain only minor amounts of tar and therefore behavior of these soils is expected to be similar to non-tar alluvial soils.

If areal dewatering is performed, our evaluation indicates that some dewatering-related subsidence will likely occur within a few months over an area about 100 feet around the dewatering system. However, differential settlements due to dewatering subsidence are not expected to cause structural distress to nearby structures because such structures are a significant distance (100+ feet) from the excavation.

Considering the site location and lack of significant adjacent structures, underpinning of existing structures is generally not expected to be required at this site. The "Underpinning Report" to be prepared by the Section Designer will provide a detailed evaluation of underpinning needs for specific structures.

Shoring systems considered technically feasible at this site include soldier piles and lagging and slurry wall with either internal bracing or tie-backs. The shoring system will be chosen by the contractor and based on local construction practice we expect that a soldier pile and lagging system will be used.

The permanent station and crossover structure will, in essence, be a concrete box supported on and retaining the surrounding soils. The subgrade condition at the A275 site generally will be dense and stiff alluvium soils and therefore estimated angular distortions are small. Permanent ground water levels must be assumed at or near the ground surface based on the high ground water levels measured.

The following subsections present our further evaluations and recommendations for design and construction of the A275 Station and crossover structure.

6.2 EXCAVATION DEWATERING

6.2.1 General Evaluation

The construction of the Beverly/Fairfax Station and crossover will require an excavation extending 45 to 55 feet below the measured ground water levels and may require areal construction dewatering if tight shoring is not used. As discussed in Section 5.0, the subsurface conditions at the site generally consist of predominately a clay/sand mixture of soils with zones of silt/sand soils and silt soils with minor tar which overlie the deep tar sands. The deep tar sand strata is relatively flat lying and will be about 30 feet below the bottom of excavation (see Drawing 4).

If pre-construction dewatering is not performed, seepage pressures in the alluvium will be high during excavation. Sand/silt soils will be unstable under conditions of high seepage pressures, possibly resulting in flowing ground. Clayey soils probably would not flow, but stability of the clays would be improved by dewatering. Due to the tar content of the tar sands, the permeability of the tar sands is assumed to be about the same as for clay soils. Therefore, the tar sands are not considered to represent a "permeable" layer below the excavation. Considering that no apparent permeable zone was encountered below the excavation level, basal heave should not be a problem at this site. However, if undetected permeable zones exist near the base of the proposed excavation, basal heave or "blow out" could occur if hydrostatic pressures are not relieved.

Due to the mix of alluvial soil types encountered at the site, dewatering characteristics should be expected to vary also. Drawdown within the more granular alluvial zones will probably occur within a few days to weeks; however, complete drawdown within the clayey alluvium may require a few months. A relatively steep drawdown surface is expected within the clayey alluvium and may extend only about 100 feet beyond the excavation. However, if there are continuous granular alluvium zones, the drawdown surface could extend several hundred feet beyond the excavation. Therefore, major variations in the phreatic surface could occur, especially during the early stages of dewatering.

The approximate estimates of drawdown time and area of influence were necessarily based on assumed hydraulic properties and subsurface conditions. Actual hydraulic properties and possible variations in subsurface conditions could significantly alter drawdown characteristics at the sites from those estimated. In our opinion, the best way to evaluate effects of possible subsurface variations and obtain reliable aquifer properties is by a pump test(s) with observation wells (piezometers) in the alluvium where the probable effect of the dewatering on the phreatic surface could be directly assessed. The test well(s) should ideally approximate characteristics of the dewatering wells. The number and locations of observation wells should be based on the known subsurface conditions and locations of areas in which settlement could be critical.

Changes in vertical pressures within the alluvium due to the reduction of buoyant forces due to dewatering are estimated to result in significant surface settlement within the expected one year or greater construction period. Our settlement calculations based on laboratory consolidation tests indicate that total surface settlements due to dewatering would be 1 to 2 inches for 40 feet of drawdown, 3/4 to 1-1/2 inches for 30 feet of drawdown and 1/2 to 3/4 inches for 15 feet of drawdown. Actual total settlements will depend on variations in subsurface conditions and the duration of construction (dewatering). Differential settlements within the steep drawdown zone of the clayey soils may be significant. However, due to the distance of existing structures from the excavations, differential settlements at the structures should be small. Estimated differential settlements are less than 1/4 inch per 100 feet for locations more than 100 feet from the wells.

It will be essential that the dewatering wells be properly designed (and installed) to prevent piping of soil into the wells. Uncontrolled piping into the wells will result in loss of ground (settlement).

As an alternative to dewatering, tight shoring such as slurry wall construction penetrating below the A275 site could provide an effective ground water barrier. Dewatering within and below the site would still be required to control flow into the excavation.

6.2.2 Possible Dewatering System

Local practice in the site vicinity generally has been to use conventional deep well dewatering systems. However, due to the generally low permeability of the onsite soils, water flows are expected to be low and, therefore, a deep well system may not be practical at this site. Dewatering systems which are better suited to low permeability soils include conventional well points and ejector wells. Conventional well points would require a two- or three-level staged dewatering system since the practical maximum lift of well points is only about 20 feet. An ejector well system, although relatively inefficient, would be capable of pumping water the full excavation depth, thereby requiring only one set of wells. Considering this, it is our opinion that an ejector well system would be best suited for site dewatering. A possible dewatering system might consist of the following:

- Closely spaced ejector wells around the perimeter of the excavations pumping from the granular alluvium zones where possible. These wells would extend to a few feet below the base of excavation level.
- Supplementary ditch drains and sumps within the excavation to handle localized inflows; e.g. from sand layers.

6.2.3 Criteria for Dewatering Systems

It is understood that the contractor will be responsible for designing, installing, and operating a suitable construction dewatering system subject to review and acceptance by the Metro Rail Construction Manager. The dewatering system should satisfy the following criteria:

- The system should maintain ground water levels low enough to provide stability of the bottom of the excavation at all times during construction.
- To adequately draw down the water table, the dewatering system should be installed and in operation for a sufficient time period prior to excavating below the static ground water level. This period will depend on the pumping rate of the system and the hydraulic characteristics of the site.
- The dewatering system should maintain the ground water levels low enough to prevent piping of the alluvial soils into the excavation. Inflow seepage should be reduced to quantities which can be accommodated by a drain/sump system and which allow excavation and construction to proceed.

- Wells must be designed and developed to eliminate loss of ground from piping of soils near the wells. The well operations should be constantly monitored for evidence of piping.

- The system should operate continuously. Emergency power and backup pumps should be required to ensure continual excavation dewatering.

6.3 STRUCTURE UNDERPINNING CONSIDERATIONS

The need to underpin and the appropriate type of underpinning for specific buildings adjacent to the proposed excavation depend on many factors related to both engineering and economics and cannot be generalized. Thus each structure needs to be evaluated separately. The following discussions and evaluations are presented strictly from an engineering standpoint. Economic considerations are beyond the scope of this investigation. We understand that an "Underpinning Report" which will provide recommendations for underpinning needs will be prepared by the Section Designer for the A275 Design Unit.

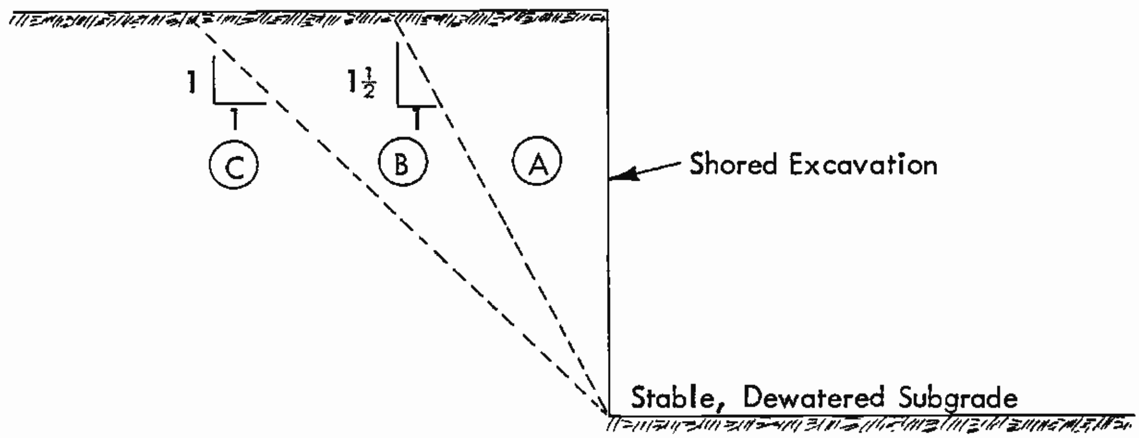
From an engineering standpoint, the need to underpin is evaluated on the basis of expected ground movements and potential for structural damage. Figure 6-1 presents general guidelines for evaluating if a given structure may be within the influence zones of the excavation. Based on Figure 6-1 and the site plan of Drawing 3, all significant buildings appear to be outside the zone of influence of the excavation. Further evaluation of expected ground movements should also be made based upon the type of shoring proposed. Section 6.4.5 discusses the anticipated ground movements in the vicinity of the excavation due to shoring movement.

6.4 TEMPORARY EXCAVATIONS

6.4.1 <u>General</u>

The A275 station and crossover excavation will extend some 50 to 60 feet below the existing ground surface and 45 to 55 feet below the water table and will, therefore, require shoring. There are several currently used shoring methods which include soldier piles and lagging, slurry wall construction and sheet piles. Bracing systems are generally either tieback anchors or internal bracing. We understand that the shoring system will be chosen and designed by the contractor in accordance with specified criteria and subject to the review and acceptance by the Metro Rail Construction Manager.

Effects of the high ground water conditions at the site should be an important consideration in the selection of the shoring system. A discussion of site dewatering requirements and effects of dewatering is presented in Section 6.2. The primary source of ground water flow will be from the granular (silt/sand) soil zones. Caving may occur within the granular soils during excavation for shoring construction. The fine-grained (clay/silt) soils are expected to perform more favorably for construction of the shoring; i.e., less water flow and less tendency for caving.



NOTE: 1.) These guidelines are applicable only for stable ground conditions. Other conditions would require special evaluation.

2.) For structure foundations bearing in zones A, B, or C, the following guidelines are presented:

(A) Special Provisions Required for Important Structures:

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

(B) Generally No Special Provisions Required:

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

(C) No Special Provisions

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UNDERPINNING GUIDELINES

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Figure No. 6-1

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Considering local construction practice, we feel that a soldier pile and lagging shoring system with tiebacks and/or internal bracing is the most likely shoring system to be used at this site. The following discussions and recommendations are, therefore, directed to a soldier pile wall system. However, other shoring systems may be considered by the contractor, and further recommendations can be provided for their design if required.

6.4.2 Soldier Pile Shoring Systems

Soldier piles have been installed in the Los Angeles area in soils similar to those encountered at the proposed A275 Station site. Where granular soils are encountered, caving could be a problem, particularly below the ground water table. The contractor should recognize that caving conditions may be encountered in construction of soldier piles or other drilled shaft elements.

The alluvium at the 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.

6.4.3 Shoring Design Criteria

This section provides design criteria for both conventional and conservative soldier pile shoring systems consisting of soldier piles and wooden lagging supported by tiebacks or internal bracing. The criteria are limited to soldier pile walls. The soldier piles are assumed to consist of steel WF or H-sections installed in predrilled circular shafts. It is assumed that the drilled shaft will be filled with concrete. Thus, for computing the allowable soil support loads, the piles were assumed to have circular concrete sections.

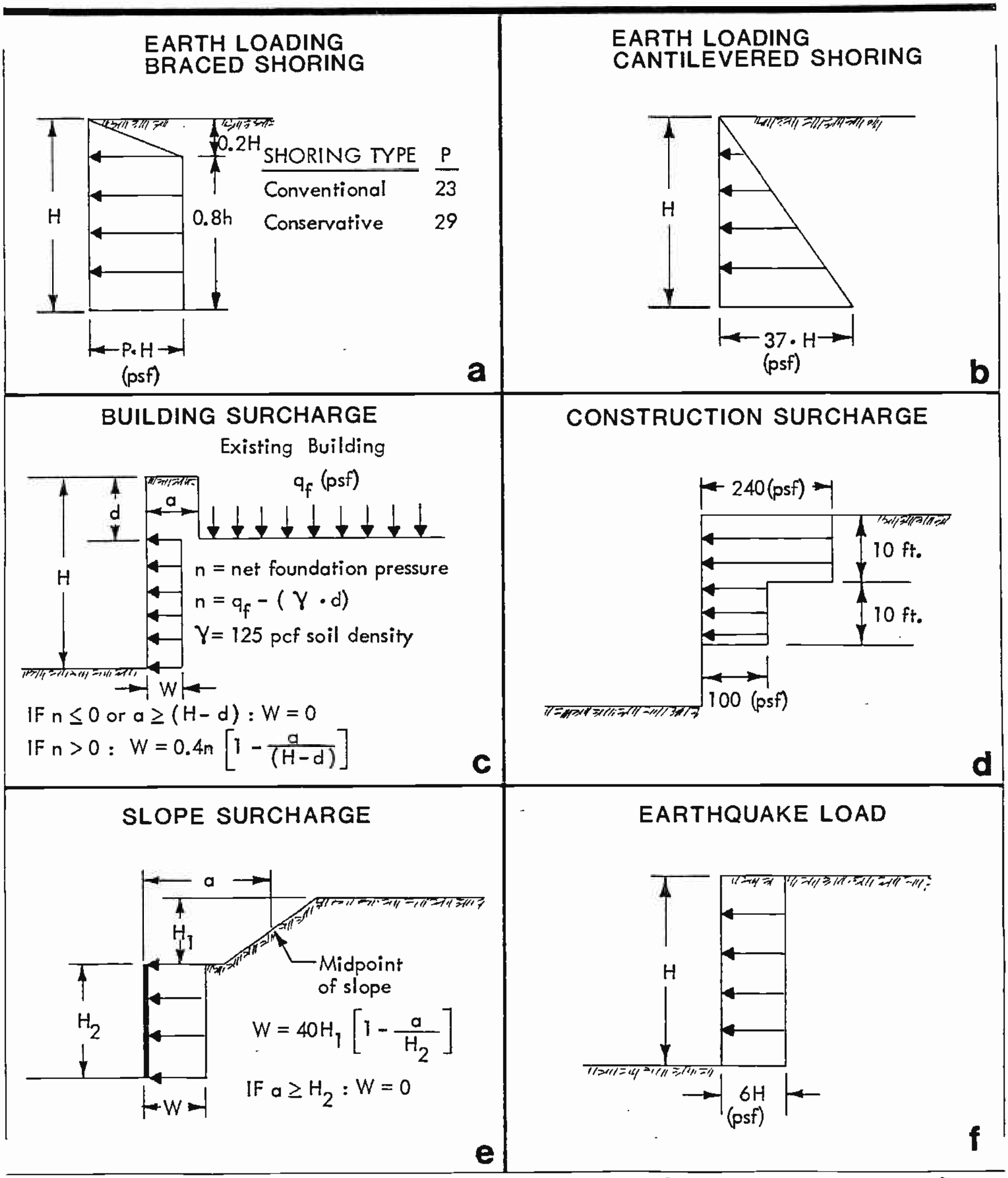
Specific shoring design criteria include:

- Design Wall Pressure: Figures 6-2a and 6-2b present the recommended lateral earth pressure on the temporary shoring walls. Figure 6-2e also includes the case of partial slope cuts. Appendix F.2 provides technical support for the recommended seismic pressures of Figure 6-2f. The full loading diagram above the bottom of excavation should be used to determine the design loads on tieback anchors 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.

- Depth of Pile Embedment: The embedment depth of the soldier pile below the lowest anticipated excavation depth must be sufficient to satisfy both the lateral and vertical loads under static and dynamic loading conditions.

  The required depth of embedment to satisfy vertical loading should be computed based on allowable vertical loads shown on Figure 6-3. Maximum depth of penetration restrictions shown on Figure 6-3 is based on consideration of the depth to the tar sand below the excavation.

  The imposed lateral load on the pile should be computed based on the earth pressure diagrams of Figure 6-2 minus the support from tiebacks or internal bracing. The required depth of embedment to satisfy lateral



LATERAL LOADS ON TEMPORARY SHORING (WITH DEWATERING)

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Figure No 6-2

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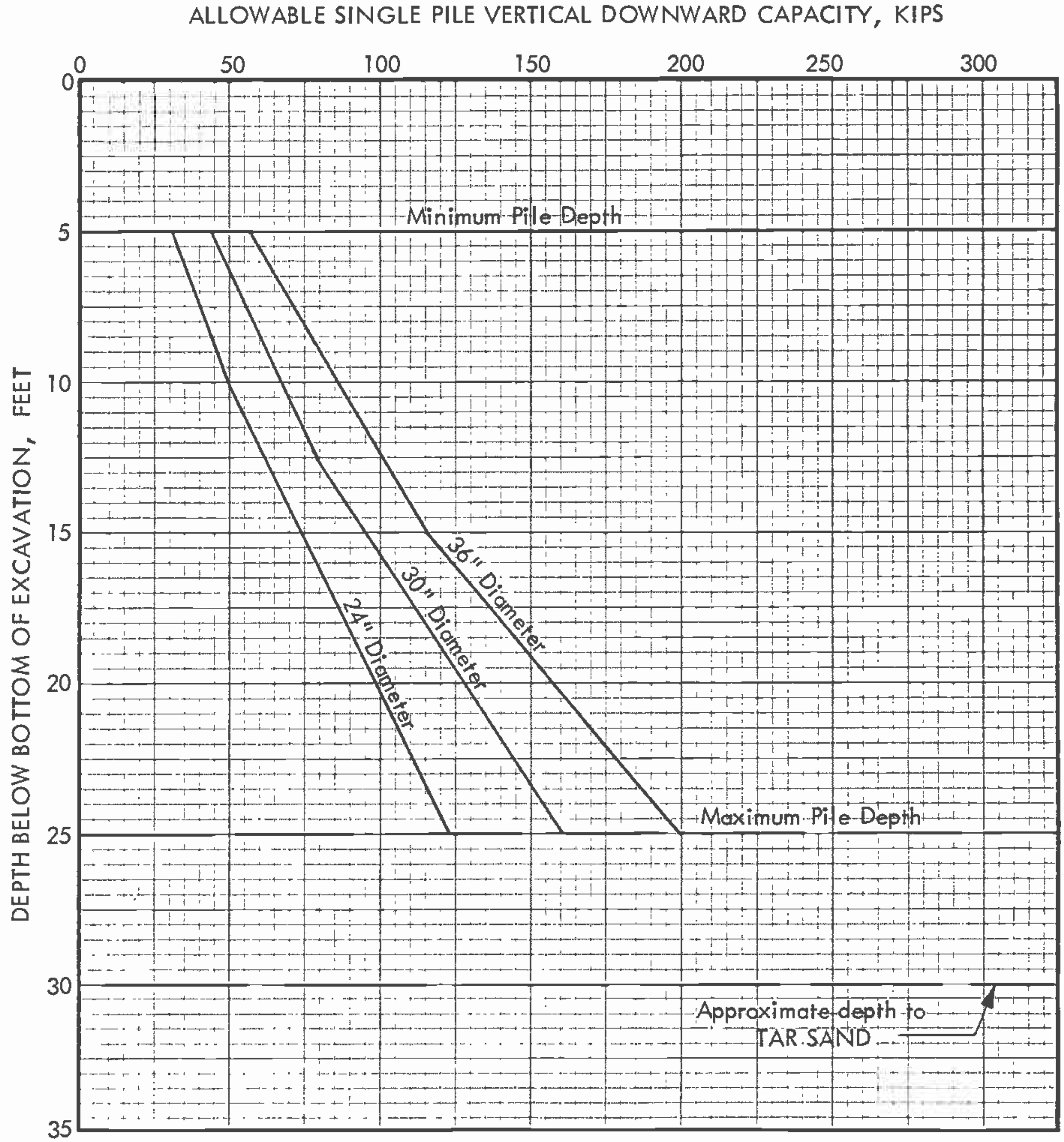

NOTES: 1) For seismic design capacities may be increased 33%.

2) Capacities apply to non-tar fine-grained alluvium.

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VERTICAL CAPACITY OF PILES FOR SHORING

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Figure No. 6-3

loads should be computed based on the net allowable passive resistance (total passive resistance of the soldier pile minus the active earth pressure below the excavation). Due to arching effects, it is recommended that the effective pile diameter be assumed equal to 1.5 pile diameters or half of the pile spacing, whichever is less. Figure 6-4 indicates the recommended method to compute net passive resistance.

° Pile Spacing and Lagging: The optimum pile spacing depends on many factors including soil type, soil loads, member sizes and costs. At the A275 Station site, granular layers may be exposed, and these soils would be subject to ravelling and sloughing. Thus, it is recommended that the pile spacing be limited to about 8 feet and that continuous lagging be placed to minimize ravelling of soils and loss of ground between soldier piles. The contractor should limit the temporarily exposed soil height to less than 3 feet to control ravelling problems, especially in the dewatered zone.

° Excavation Stability: As part of the shoring design, stability calculations should be performed to verify that the shoring/tieback system has an adequate safety factor against deep-seated failure.

6.4.4 Internal Bracing and Tiebacks

6.4.4.1 General: Tiebacks and/or internal bracing may both be suitable to support the temporary shoring wall for the proposed excavation. Tiebacks have the advantage of producing an open excavation which can significantly simplify the excavation procedure and construction of the permanent structure. However, there may be an opportunity to install used pipe and WF sections from other projects as struts and to salvage these for use elsewhere. This often makes the employment of internal bracing more attractive to the contractor than tiebacks. Obtaining permission to install tiebacks under adjacent properties and encountering obstructions from adjacent below grade structures (such as basements) can also affect the economics and feasibility of tiebacks.

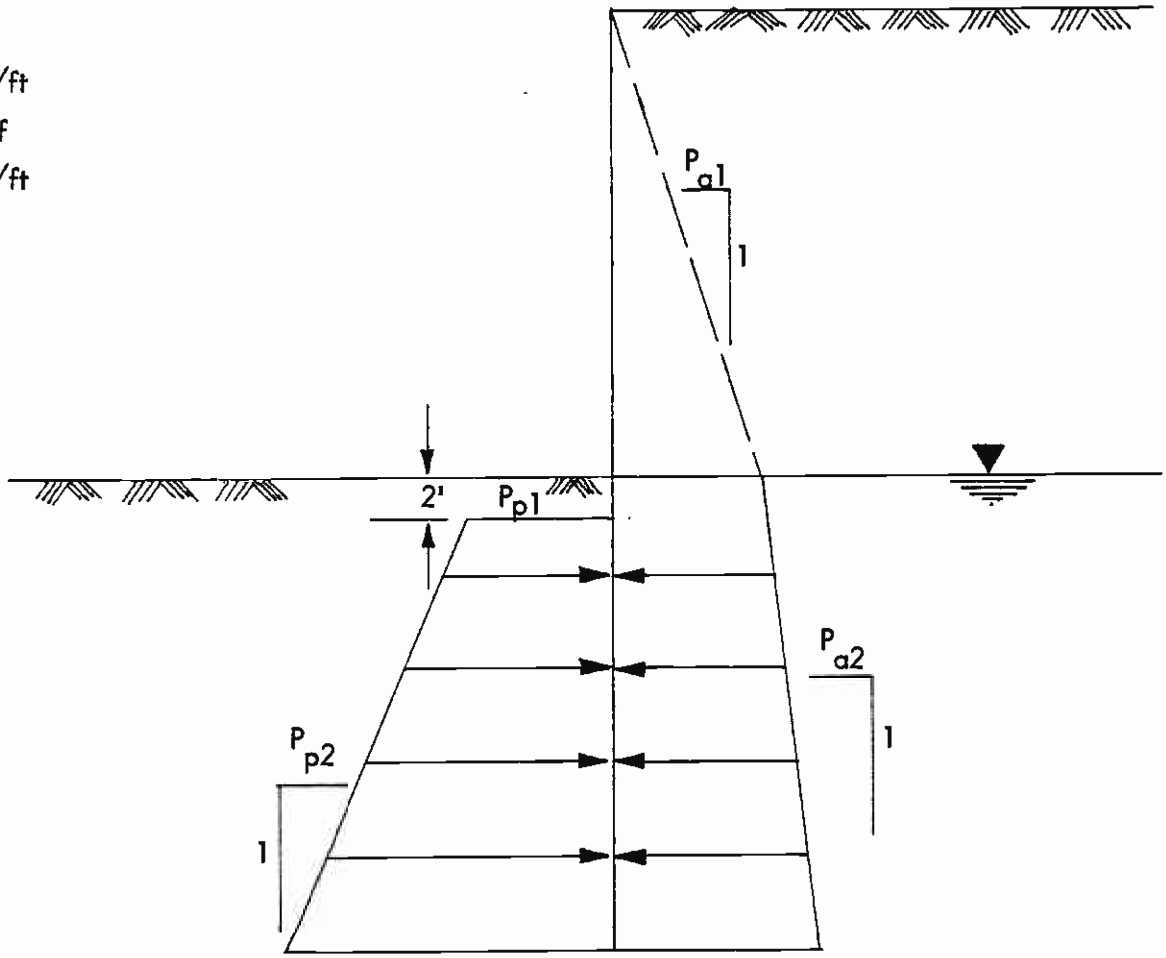
6.4.4.2 Performance: Based on available field data there does not appear to be a significant difference between the maximum ground movements of properly designed and carefully constructed tieback walls or internally braced walls. However, there is a difference in the distribution of the ground movements. Prestressing of both tiebacks and struts is essential to confirm design capacities and minimize ground movements.

6.4.4.3 Internal Bracing: The contractor should not be allowed to extend the excavation an excessive distance below the lowest strut level prior to installing the next strut level. The maximum vertical distance depends on several specific details such as the design of the wall and the allowable ground movement. These details cannot be generalized. However, as a guideline, we recommend consideration of the following maximum allowable vertical distances between struts:

° Conventional Shoring System: 12 feet
° Conservative Shoring System: 8 feet.

Recommended Unit Pressures

P_{a1} = 37 psf
P_{a2} = 19 psf/ft
P_{p1} = 500 psf
P_{p2} = 50 psf/ft



Where: P_p = Total Allowable unit passive pressure

P_a = Unit Active pressure

NOTE:
1.) The site is assumed to be dewatered
2.) Available passive pressure = Total passive - Active
3.) Available passive pressure can be assumed to act on 1.5 pile diameters or ½ the pile spacing whichever is less.
4.) Active pressure shown is for evaluation of available passive pressure. Lateral shoring pressures are presented on Fig. 6-2
5.) Indicated pressures are for soils above the tar sands

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SOLDIER PILE PASSIVE RESISTANCE

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Figure No. 6-4

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In addition, the contractor should not be allowed to extend the excavation more than 3 feet below the designated support level before placing the next level of struts. The contractor may be allowed to excavate a trench within the excavation to facilitate construction operations provided the trench is not less than 15 feet horizontally from the shoring and does not extend more than 6 feet below the designated support level.

To remove slack and limit ground movement, the struts should be preloaded. A preload equal to at least 50% of the design load is normally desirable. The shoring design, preload procedures, and monitoring/maintenance procedures must provide for the effects of temperature changes to maintain the shoring support.

6.4.4.4 Tieback Anchors: There are numerous types of tieback anchors available including large diameter straight shaft friction anchors, belled anchors, high pressure grouted anchors, high pressure re-groutable anchors, and others. Generally, in the Los Angeles area, high capacity straight shaft or belled anchors have been used where construction conditions are favorable.

Tieback anchor capacity can be determined only in the field based on anchor load tests. For estimating purposes, we recommend that the estimated capacity of drilled straight shaft friction anchors be computed based on the following equation:

$$P = \pi DLq$$

Where:

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

The design adhesion value (q) for alluvial soils (above the tar sands) can be determined by:

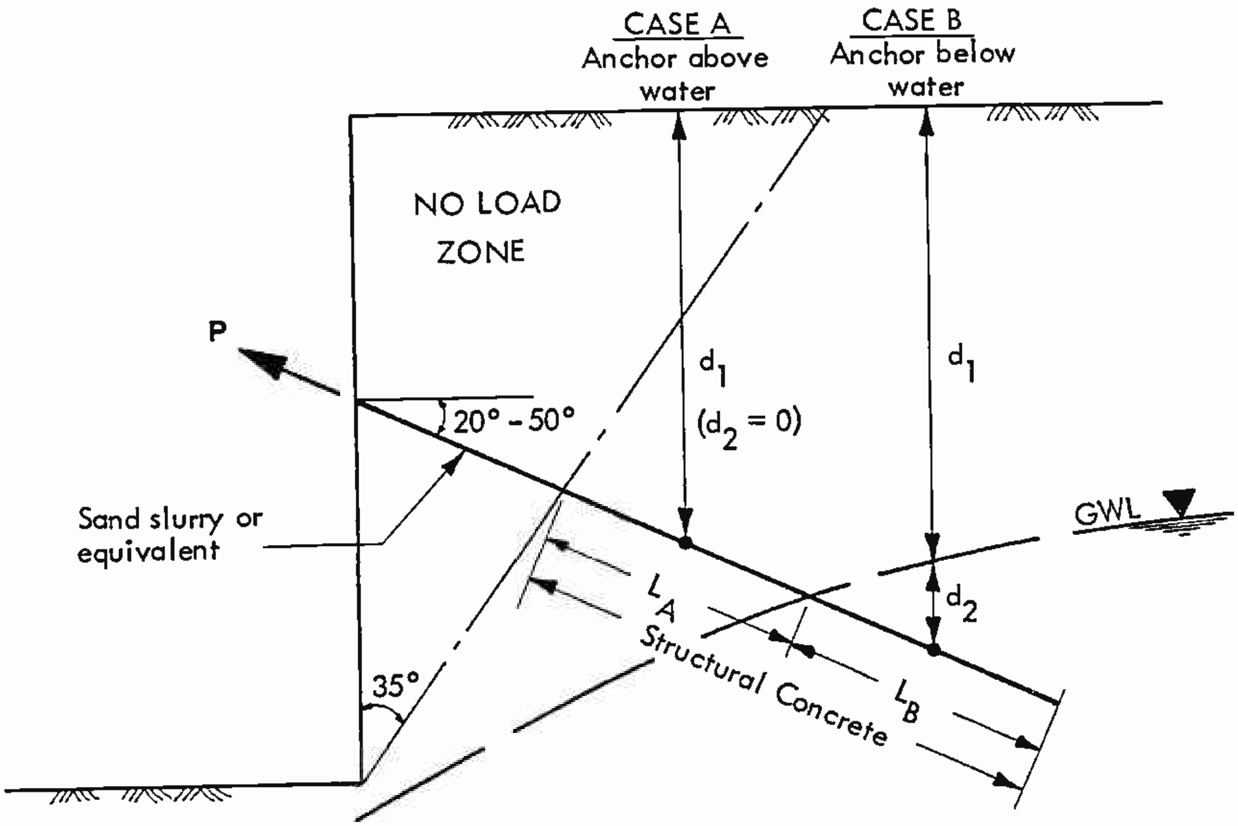
$$q = 20d_1 + 10D_2 < 750 \text{ psf}$$

Where:

d_1 = average depth (in feet) of the non-submerged anchor beyond the no-load zone; measured vertically from the ground surface.

d_2 = average depth (in feet) of the submerged anchor below the ground water level.

Figure 6-5 illustrates guidelines for the design of tieback anchors.



NOTE:

The design adhesion value, q, can be evaluated by

$$q = 20d_1 + 10d_2 \leq 750 \text{ psf (in alluvium)}$$

d = average depth of anchor in feet beyond the no load zone. (d_1 for alluvium above water; d_2 for alluvium below water)

The total anchor capacity can be estimated by:

$$P = \pi D L_A q_A + \pi D L_B q_B$$

See also Section 6.4.4.4

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STRAIGHT SHAFT TIEBACK ANCHOR CAPACITY

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Figure No. 6-5

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Allowable anchor capacity/length relationships for tieback types other than straight shaft friction anchors cannot be generalized. Design parameters for anchors such as high pressure grouted anchors and high pressure regroutable anchors must be based on experience in the field and on the results of test anchors.

For design purposes, it should be assumed that the potential wedge of failure behind the shored excavation is determined by a plane drawn at 35° with the vertical through the bottom of the excavation for alluvial soil conditions. Only the frictional resistance developed beyond the no-load zone should be assumed effective in resisting lateral loads.

The anchors may be installed at angles generally between 20° to 50° below the horizontal. Based on specific site conditions, these limits could be expanded to avoid underground obstructions. Structural concrete should be placed in the lower portion of the anchor up to the limit of the no-load zone. Placement of the anchor grout should be done by pumping the concrete through a tremie or pipe extending to the bottom of the shaft. The anchor shaft between the no-load zone and the face of the shoring must be backfilled with a sand slurry or equivalent after concrete placement. Alternatively, special bond breakers can be applied to the strands or bars in the no-load zone and the entire shaft filled with concrete.

For tieback anchor installations, the contractor should be required to use a method which will minimize loss of ground due to caving. The majority of the anchors should not experience significant caving problems. However, caving from sand layers within the alluvium could occur due to vibration from the drilling equipment and/or ground water effects. Caving problems should be expected where anchors penetrate sands below the water table. Caving not only causes installation problems but could result in surface subsidence and settlement of overlying buildings. To minimize caving, casing could be installed as the hole is advanced but must be pulled as the concrete is poured. Alternatively, the hole could be maintained full of slurry or a hollow stem auger could be used.

It is recommended that each tieback anchor be test loaded to 150% of the design load and then locked off at the design load. At 150% of the design load, the anchor deflection should not exceed 0.1 inches 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 design load the anchor deflections should not exceed 0.15 inches over a 15-minute period. The rate of deflection should consistently decrease during the test period. If the rate of deflection does not decrease the test should not be considered satisfactory.

6.4.5 Anticipated Ground Movements

The ground movements associated with a shored excavation depend on many factors including the contractors' procedures and schedule, and therefore, the distribution and magnitude of ground movements are difficult to predict. Based on shoring performance data for documented excavations combined with our engineering judgement, we estimate that the ground movements associated with properly designed and carefully constructed shoring systems will be as follows:

- Conventional Wall With Tieback Anchors: The maximum horizontal wall deflection will equal about 0.1% to 0.2% of the excavation depth. The maximum horizontal movement should occur near the top of the wall and decrease with depth. The maximum settlement behind the wall should be equal to about 50% to 100% of the maximum horizontal movement and will probably occur at a distance behind the wall equal to about 25% to 50% of the excavation depth.

- Conventional Wall With Internal Bracing: The maximum ground movement will be similar to those anticipated with tiebacks. However, the maximum horizontal movement will probably occur near the bottom of the excavation decreasing to about 25% of the maximum at the surface.

- Conservative Wall With Tiebacks: We believe that the higher design pressure presented for conservative walls will reduce ground movements and limit the maximum horizontal and vertical movements to about 0.1% of the excavation depth.

- Conservative Wall With Internal Bracing Similar to that described above for the conservative tieback supported wall.

6.4.6 Historical Shoring Pressure Diagrams - Los Angeles

Appendix F.1 summarizes the design shoring pressures for nine shoring systems in the Los Angeles vicinity. To our knowledge, there are no data on field measurements of actual lateral soil pressures for shored excavations in the Los Angeles area and, therefore, the design pressures of Appendix F.1 have not been directly verified.

6.5 INSTRUMENTATION OF THE EXCAVATION

In our opinion the proposed A275 Station excavation should be instrumented to reduce liability (by having documentation of performance), to validate design and construction requirements, 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 and structures adjacent to the site prior to construction. This will minimize the risks associated with claims against the owner/contractor. If substantial cracks are noted in the existing structures, they should be measured and periodically re-measured during the construction period.

- <u>Surface Survey Control</u>: It is recommended that several locations around the excavation and on any nearby structures be surveyed prior to any construction activity and then periodically to monitor potential vertical and horizontal movement to the nearest 0.01 feet. In addition, survey markers should be placed at the top of piles spaced no more than every fourth pile or 25 feet, whichever is less.

- <u>Tiltmeters</u>: Tiltmeters are used to monitor the verticality of buildings adjacent to the excavation and can provide a forewarning of distress. Normally ceramic plates are glued to the building walls and read using a portable tiltmeter containing the same type of tilt sensor used in inclinometers. It is recommended that a few tiltmeters be placed on the exterior walls of buildings which are located within the underpinning zone defined on Figure 6-1. Baseline readings should be made prior to all construction activity, and subsequent readings should be made at several excavation/construction stages through the end of construction.

- <u>Inclinometers</u>: It is recommended that several inclinometers be installed and monitored around the station excavation. Inclinometers should be located on each side of the excavation. The casing could be installed within the soldier pile holes or in separate holes immediately adjacent to the shoring wall. Baseline readings of the inclinometers should be made immediately upon installation. Subsequent readings should be made at regular time intervals and/or intervals of excavation progress.

- <u>Heave Monitoring</u>: 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 gages be installed along the longitudinal centerline of the excavation on about 200-foot centers. The devices could consist of conical steel points, installed in a borehole, and monitored with a probing rod that mates with the top of the conical point. The borehole should be filled with a thick colored slurry to maintain an open hole and allow for easy hole location. The top of the points should be at least 2 feet below the bottom of the final excavation to protect them 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 about two-week intervals until the excavation is completed and all heave has stopped.

- <u>Convergence Measurements</u>: We recommend the use of tape extensometers to measure the convergence between points at opposite faces of the excavation during various stages of excavation. These measurements provide inexpensive data to supplement the inclinometer and survey information.

- <u>Measurements of Strut Loads</u>: If internal bracing is used, we recommend that the loads on at least four struts at each support level be monitored periodically during the construction period. These measurements provide data on support loads and a forewarning of load reductions which would result in excessive ground movements.

- 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. The devices should be installed and initial readings should be taken as early as possible. Readings should then be taken and immediately reported 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.

In our opinion, it is important that the installation and measurement of the instrumentation devices be under the direction and control of the Engineer. Experience has shown when the 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 are questionable. By defining Support Work (Contractor) and Specialist Work (Engineer) in the bid documents, RTD could allow the contractor to provide support to the Engineer for installing the instrumentation.

6.6 EXCAVATION HEAVE AND STRUCTURE SETTLEMENT

The proposed A275 excavation will substantially change the ground stresses below and adjacent to the excavation. The proposed 50- to 60-foot excavation will decrease the vertical ground stresses by about 3500 to 4000 psf. Stress reduction caused by the excavation will result in rebound or heave of the alluvium, the tar sands, and bedrock below the excavation. Since the excavation will be open for an extended period, the heave is expected to be completed prior to construction of the station and crossover. The station structures and subsequent backfilling will reload the soil. We estimate that the subgrade load without hydrostatic uplift may range from about 2000 to 4000 psf. Net pressures after water levels have re-established may be as low as 500 to 1000 psf. Such loads will cause the ground to reconsolidate or settle. Thus, even though the weight of the excavated soil may exceed the weight of the final structure, the structure will experience some static ground settlement due to recompression during the construction period of the station.

We estimate that the maximum heave at the center of the excavation will be on the order of 2 to 4 inches. We also believe that the majority of this will occur while the excavation is being made. This estimate is based on computations of elastic shear deformation (elastic rebound) and unit volume changes (consolidation heave) within the alluvium underlying the proposed excavation. Due to the dense and stiff consistency of the alluvium, the majority of the deformation will be elastic rebound.

We computed that the estimated imposed loads from the structure and backfill without hydrostatic uplift may induce settlements on the order of 2 to 4 inches. Settlements due to net loads considering hydrostatic uplift were computed to be about 1/2 to 1 inch. The majority of these settlements will occur during construction. Due to the long, narrow shape of the imposed load, the theoretical differential settlement is relatively small, on the order of 1/2 inch over the width of the structures. This correlates to an angular rotation of only about 1:720. These calculations are based on a uniform foundation bearing pressure which could result only from a uniformly loaded and perfectly flexible structure. We understand that the station will be structurally quite stiff. Thus the actual differential settlement will be less than for the theoretical flexible foundation assumed.

We understand that MRTC has modified the Design Criteria and Standards for underground structures to permit use of a simplifying and 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 simplified uniform pressure approach is left to the discretion of MRTC and Section Designer.

6.7 FOUNDATION SYSTEMS

6.7.1 Main Station

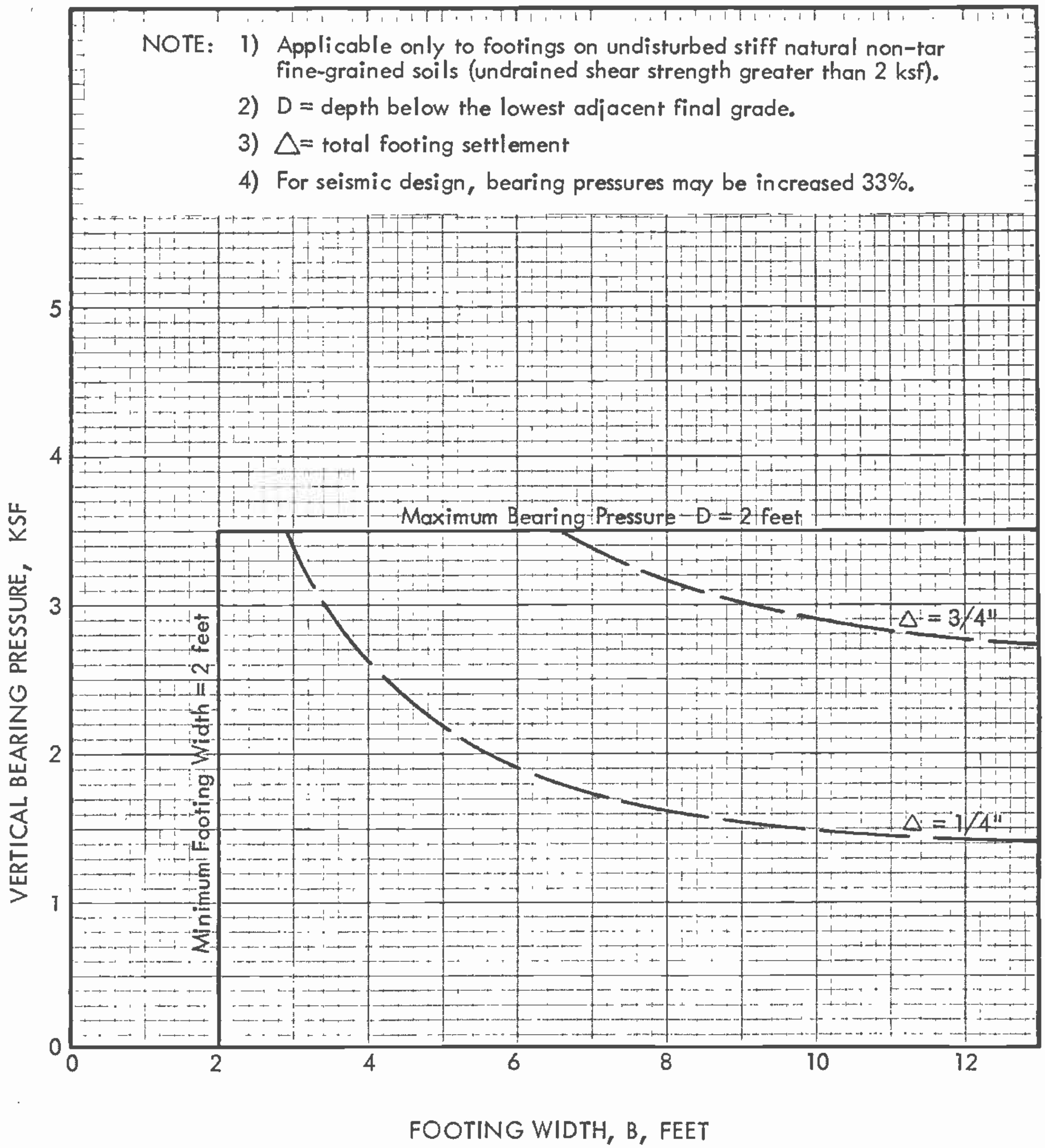
It is understood that the proposed A275 Station will be supported on a thick base slab which will function as a massive mat foundation. We estimate that the net mat foundation bearing pressure will be about 3000 psf. In our opinion the station can be adequately supported on a mat foundation. Section 6.6 presents estimated settlements for the proposed station structure.

6.7.2 Support of Surface Structures

Surface structures can be generally supported on conventional spread footings founded on undisturbed stiff or dense natural soils. If suitable natural soils do not exist at the surface structure site, footings may be founded on a zone of properly compacted structural fill (see Appendix G). Allowable bearing pressures and estimated total settlements of spread footings bearing on the natural alluvium or compacted structural fill can be determined based on Figures 6-6 and 6-7. These figures are based on analytical procedures and experience in the Los Angeles area but are generally conservative due to lack of detailed information on structural loadings and site conditions at the surface structure location. Detailed site specific studies should be performed to provide final design recommendations for specific structures.

All spread footing foundations should be founded at least 2 feet below the lowest adjacent final grade and should be at least 2 feet wide. The bearing values shown on Figures 6-6 and 6-7 are for full dead load and frequently applied live load. For transient loads, including seismic and wind loads, the bearing values can be increased by 33%. Differential settlements between adjacent footings should be estimated as 1/2 of the average total settlements or the difference in the estimated total settlements shown on Figures 6-6 and 6-7, whichever is larger.

For design, resistance to lateral loads on surface structures can be assumed to be provided by passive earth pressure and friction acting on the foundations. An allowable passive pressure of 250 psf/ft may be used for the sides



ALLOWABLE BEARING & SETTLEMENT FOR SPREAD FOOTING ON FINE-GRAINED SOILS

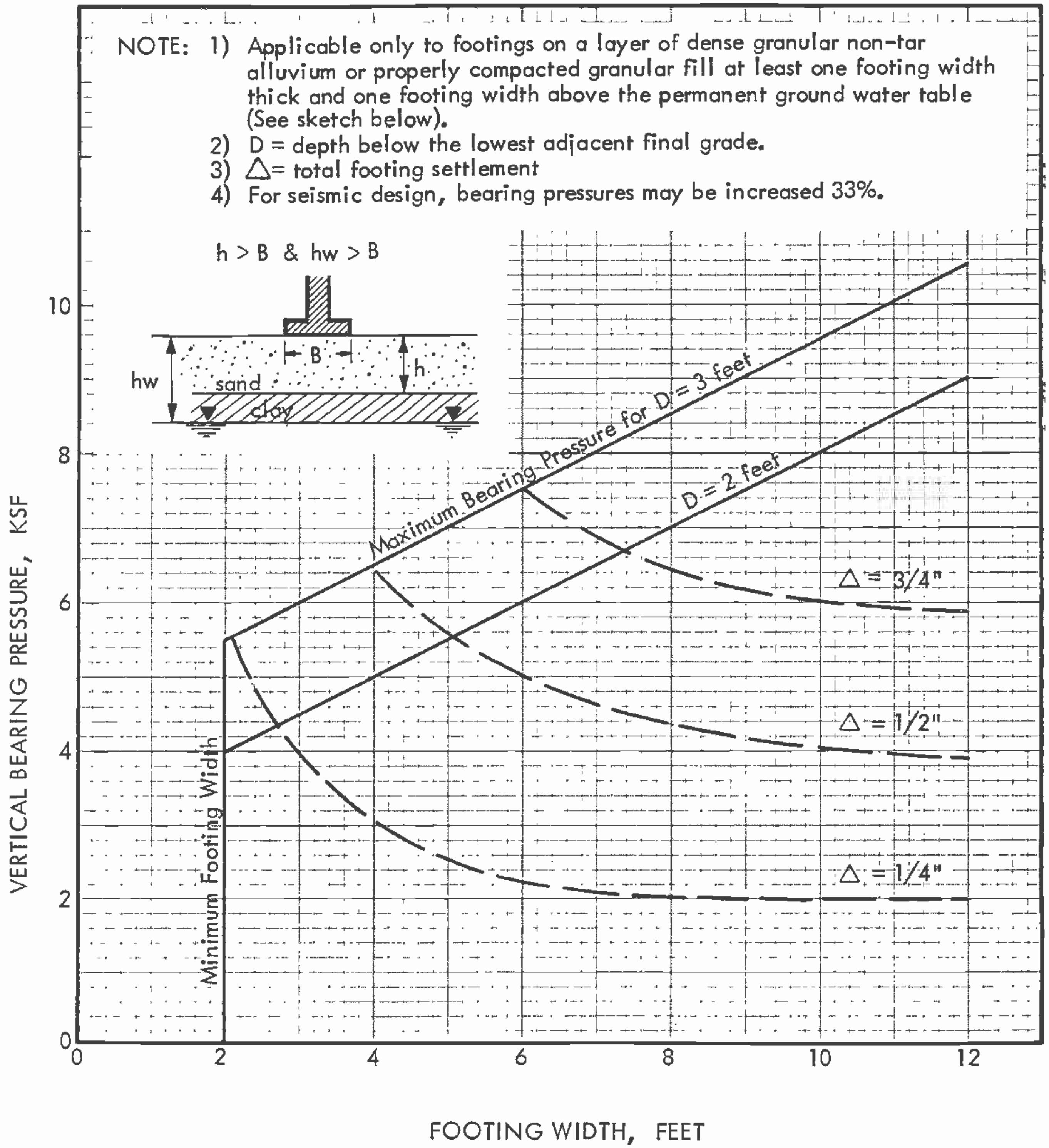
DESIGN UNIT A275
Southern California Rapid Transit District
METRO RAIL PROJECT

Project No. 83-1140

Converse Consultants Geotechnical Engineering and Applied Sciences

Figure No. 6-6

Approved for publication 4/84 by JRD



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SPREAD FOOTING BEARING/SETTLEMENT ON GRANULAR SOILS

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Southern California Rapid Transit District
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Project No. 83-1140

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Figure No. 6-7

of footings poured neat against dense or stiff alluvium or properly compacted fill. Frictional resistance at the base of foundations should be determined using a frictional coefficient of 0.4 with dead load forces.

6.8 PERMANENT GROUND WATER PROVISIONS

We understand that the station will be designed to be water-tight and to resist the full permanent hydrostatic pressures. We recommend that the entire structure be fully waterproofed due to the high design water levels. See Section 6.9.1 for hydrostatic pressure design guideline.

6.9 LOADS ON SLAB AND WALLS

6.9.1 Hydrostatic Pressures

As discussed in Section 5.4, the existing ground water levels as measured in man-size auger 23B near the north end of the station was Elevation 181 to 182 in early February 1983. It is recommended that the long-term design ground water level be assumed to be Elevation 185 at the north end of the station and Elevation 180 at the south end of the crossover structure. Design water levels at intermediate points should be linearly interpolated.

6.9.2 Permanent Static Earth Pressures

Figure 6-8 presents lateral earth pressures recommended for design of permanent subsurface walls.

Vertical earth pressures on the roof should be assumed equal to the full moist and/or saturated weight of overburden soil plus design surcharge loads to be determined by the Section Designer.

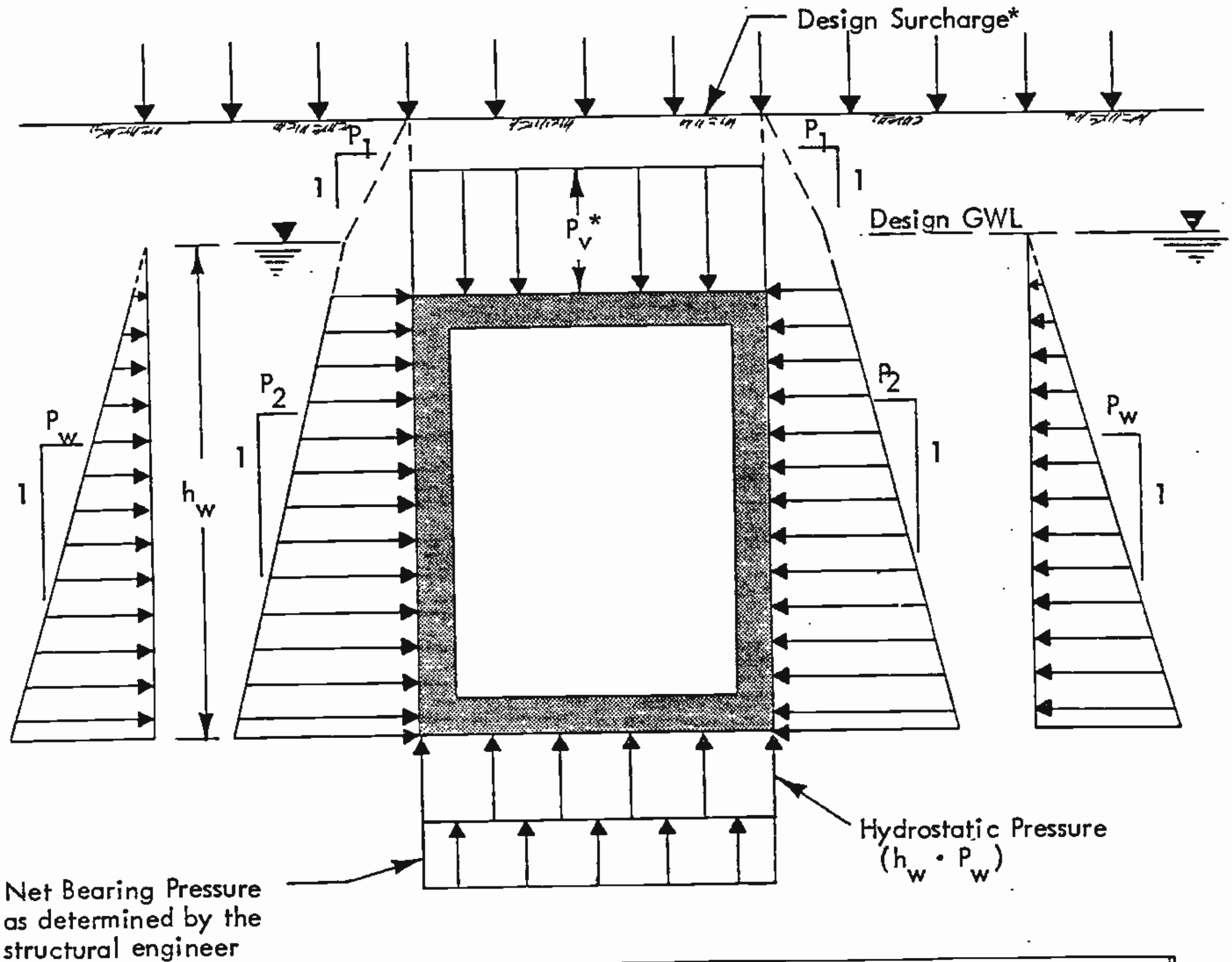
6.9.3 Surcharge Loads

The lateral surcharge loads are identical to those recommended for temporary walls. Procedures for computing these are presented on Figure 6-2. Vertical surcharge loads due to surface traffic, etc. as determined by the Section Designer should also be included in roof design. In addition, consideration should be given to loads imposed by earthmoving equipment during backfill operations.

6.10 SEISMIC CONSIDERATIONS

6.10.1 General

Design procedures and criteria for underground structures under earthquake loading conditions are defined in the Southern California Rapid Transit District (SCRTD) report entitled "Guidelines for Design of Underground Structures", dated March, 1984. Evaluations of the seismological conditions which may impact the project and the probable maximum credible earthquakes, which may be anticipated in the Los Angeles area, are described in the SCRTD report entitled "Seismological Investigation and Design Criteria", dated May, 1983. The 1984 report complements and supplements the 1983 report.



LOADING CONDITION	DESIGN LOAD PARAMETERS				
	P_1 (psf)	P_2 (psf)	P_w (psf)	P_v	GWL
End Construction	37	19	62.4	*	**
Long Term	65	33	62.4	*	***
Side sway †	37/65	19/33	62.4	*	**

* P_v = full overburden pressure (depth x total density) plus design surcharge; distribution and magnitude of design surcharge to be determined by the the section designer.

** Designer should use a GWL (between the base of slab and long term water elevation) which will be critical for the loading condition.

*** Varies linearly from elev. 180 at the south end to elev. 185 at the north end.

† Sidesway condition assumes "End Construction" pressure on one side and "Long Term" pressure on the other.

LOADS ON PERMANENT WALLS

DESIGN UNIT A275
Southern California Rapid Transit District
METRO RAIL PROJECT

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Figure No. 6-8

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6.10.2 Dynamic Material Properties

Dynamic soil parameters required for input into the various types of analyses recommended in the seismic design criteria report are presented in Table 6-1. These include values of dynamic Young's modulus, dynamic constrained modulus, and dynamic shear modulus at low strain levels.

Average values of compression and shear wave velocities based on interpretation of seismic refraction surveys in the general site area as well as downhole and crosshole geophysical surveys performed in Borings CEG-20, 23, 23A and 24 in similar materials during the 1981 investigation are presented at the top of Table 6-1. 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. Computed moduli values for the alluvium are tabulated in Table 6-1.

TABLE 6-1

RECOMMENDED DYNAMIC MATERIAL PROPERTIES FOR USE IN DESIGN

		ALLUVIUM
Average Compression Wave Velocity, V_c (ft/sec)	- moist	2400
	- saturated	5000
Average Shear Wave Velocity, Vs (ft/sec)		1200
*Poisson's Ratio		0.35
Young's Modulus, E, (psi)	- moist	100,000
	- saturated	185,000
Constrained Modulus, E_c, (psi)	- moist	160,000
	- saturated	700,000
Shear Modulus, G_{max}, (psi)		40,000

* For saturated alluvium, use value of 0.45.

The variation of dynamic shear modulus, with shear strain is presented in Figure 6-9 for the various geologic units. Variation of the dynamic shear modulus is expressed as the ratio of the strain compatible modulus (G) to the very low strain modulus (G_{max}). Similar relationships for soil hysteretic damping are presented in Figure 6-10. The modulus and damping curves are based on dynamic laboratory tests performed during our 1981 investigation. Dynamic test results are presented in Vol. II, Appendix H of our 1981 report.

6.10.3 Liquefaction Potential

The generalized subsurface cross section has been described in Section 5.0 and is shown in Drawings 2 and 4. The ground water level at the site is roughly at a depth of about 5 feet below the surface. The soils which are saturated and, therefore, must be evaluated for liquefaction potential include the pockets of granular soils within the matrix of clay soils above the San Pedro Sands and the San Pedro Sands as well.

Our liquefaction evaluation was based on procedures and correlations published by Seed et al (1983) which utilized index soil properties and performance data for soils during previous earthquakes. Field Standard Penetration Tests (SPT), available field geophysical data from CEG-23, and laboratory classification test data were all used in our evaluation of liquefaction potential (see Appendix F).

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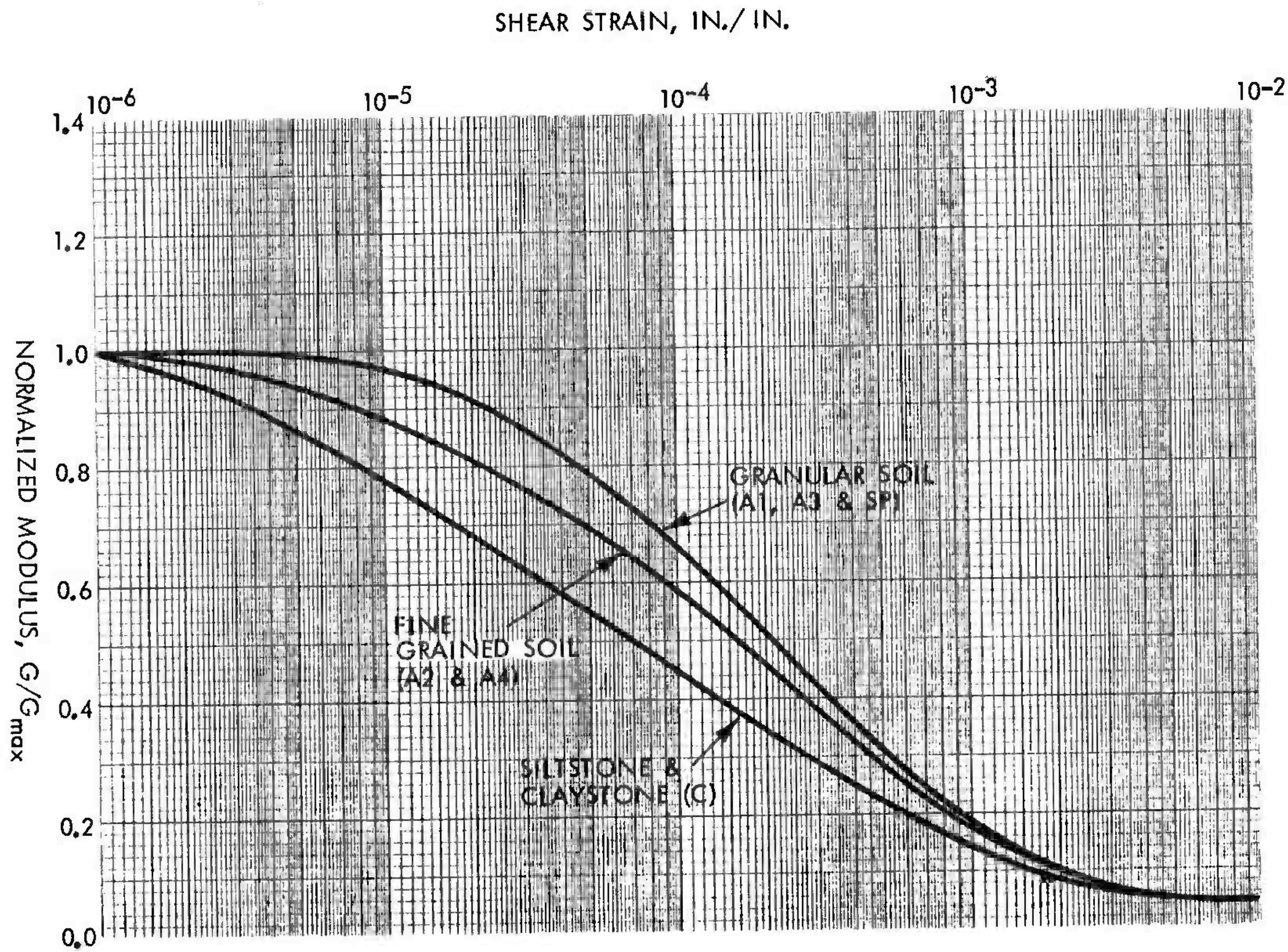

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RECOMMENDED DYNAMIC SHEAR MODULUS RELATIONSHIPS

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Project No. 83-1140

Figure No. 6-9

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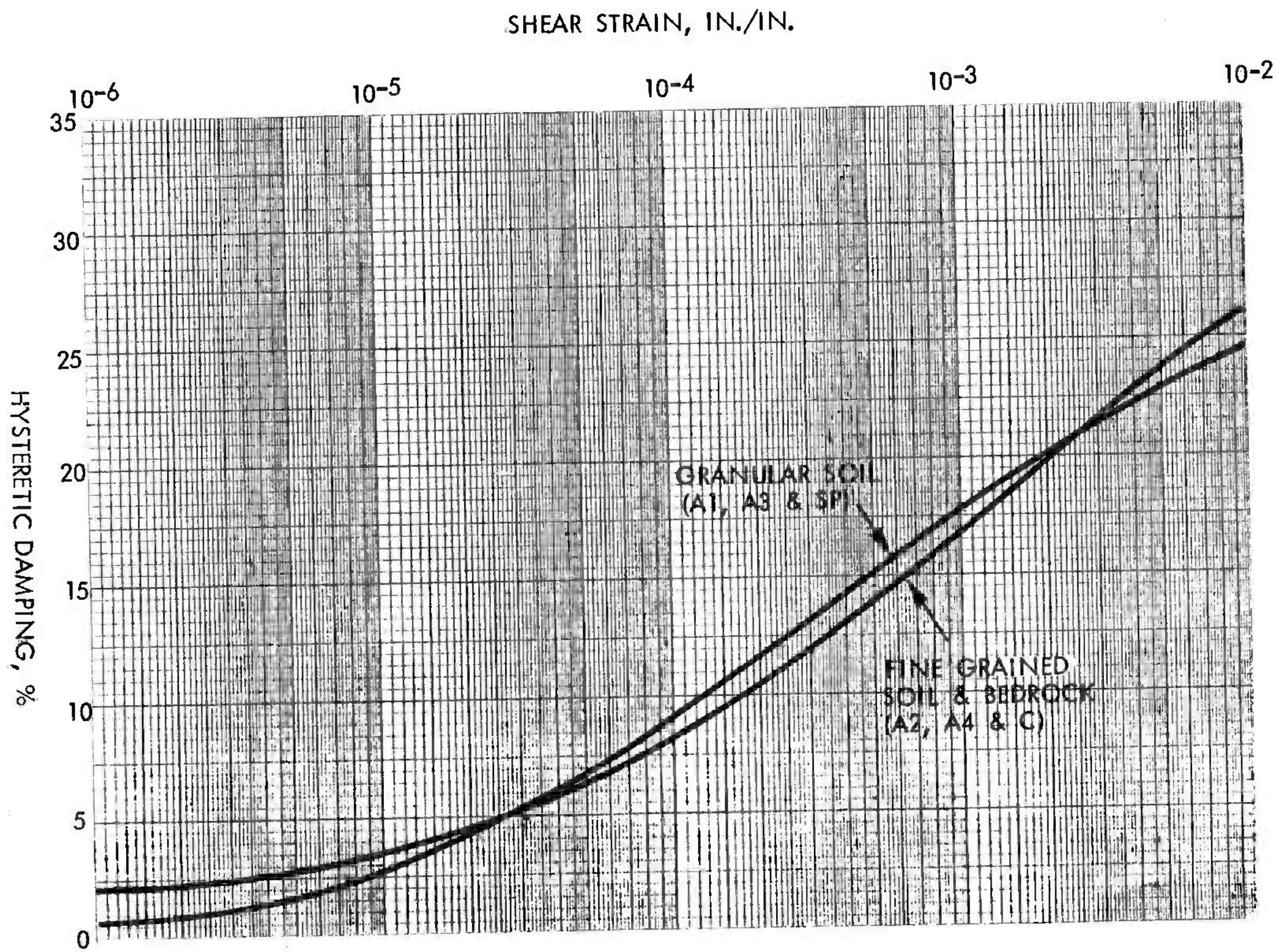

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RECOMMENDED DYNAMIC DAMPING RELATIONSHIPS

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Project No. 83-1140

Figure No. 6-10

Index property tests (Atterberg Limits, moisture content, and grain size distribution) of the clayey alluvium which predominates at this site compared with index properties of clayey soils vulnerable to liquefaction confirmed the onsite clayey soils to be non-liquefiable.

The referenced procedures include correlations of SPT data and liquefaction potential for granular soils. Considering the high SPT values in the San Pedro Sands and the tar content in these materials, the possibility of liquefaction of the San Pedro Sands is judged to be remote. Corrected "N" values (normalized to 2 ksf overburden pressure) for SPT values in saturated granular alluvium zones ranged from 22 to 51 with an average of about 33. Determination of dynamic strength was based on an M6.0 event for the operating design earthquake (ODE) and an M7.0 event for the maximum design earthquake (MDE). The results of the SPT analyses indicated a low potential for liquefaction of the granular lenses during the ODE and a possible moderate to high potential for liquefaction of the granular lenses during the MDE event.

Based on the above, we expect that liquefaction of localized granular soil zones may occur during the MDE event. However, in our opinion, liquefaction of the granular layers within the clayey soil matrix will not result in catastrophic changes in the overall dynamic soil loads on the structure because the clayey soils are expected to maintain their integrity.

6.11 EARTHWORK CRITERIA

Site development is expected to consist primarily of excavation for the subterranean structure but will also include general site preparation, foundation preparation for near surface structures, slab subgrade preparation, and backfill for subterranean walls and footings and utility trenches. Recommendations for major temporary excavations and dewatering are presented in Sections 6.2 and 6.4. Suggested guidelines for site preparation, minor construction excavations, structural fill, foundation preparation, subgrade preparation, site drainage, and utility trench backfill will be presented in Appendix G. Recommended specifications for compaction of fill are also presented in Appendix G. Construction specifications should clearly establish the responsibilities of the contractor for construction safety in accordance with CALOSHA requirements.

Excavated granular alluvium (sand, silty sand and gravelly sand) are considered suitable for re-use as compacted fill, provided it is at a suitable moisture content and can be placed and compacted to the required density. Existing fills and fine-grained soils are not considered suitable because these fine-grained materials will make compaction difficult and could lead to fill settlement problems after construction. If quantities of suitable granular alluvium materials are not sufficient, imported granular soils could be used for fill, subject to approval by the geotechnical engineer.

It should be understood that some settlement of the excavation backfill will occur even if the fill soils are properly placed and compacted. Cracking and/or settlement of pavement on and around the backfilled excavation should be expected to occur for at least the first year following construction. Placement of the final pavement section should be delayed at least one year.

6.12 PAVEMENT DESIGN

Minimum flexible pavement sections for assumed Traffic Index (TI) values of 5.0, 7.0 and 9.0, and a subgrade R-value of 40 were developed using CALTRANS design method. Pavement sections provided below include the recommended thickness of compacted subgrade, base course and asphaltic concrete for the three Traffic Index values.

ASSUMED TRAFFIC INDEX (TI)	THICKNESS (in inches)			
	A.C. with Base Course		Full Depth Asphaltic Concrete	Compacted Subgrade (R ≥40)
	A.C.	Base Course		
5.0	2.0	6.5	4.5	24.0
7.0	3.0	8.5	7.0	36.0
9.0	4.0	11.0	9.5	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 F.

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

6.13 SUPPLEMENTARY GEOTECHNICAL SERVICES

Based on the available data and the current design concepts, the following supplementary geotechnical services may be warranted:

- Supplemental Investigations: Consideration should be given to performing supplemental geotechnical investigations at the sites of proposed peripheral at-grade structures near the station. The purpose of these studies would be to determine site specific subsurface conditions and provide site specific final design recommendations for the peripheral structures.

- Pump Test: It is recommended that a pumping test be performed at the site to evaluate the pumping and dewatering characteristics. The test well should ideally approximate characteristics of the dewatering wells. The number and locations of observation wells should be based on the known subsurface conditions and locations of areas in which settlement could be critical.

- Observation Well Monitoring: Shallow ground water observation wells should be installed at the ends of the station and crossover to be read several times a year until project construction and more frequently during construction if possible. These data will aid in confirming the recommended maximum design ground water levels. They will also provide valuable data to the contractor in determining his construction schedule and procedures.

- Review Final Design Plans and Specifications: A qualified geotechnical engineer should be consulted during the development of the final design concepts and should complete a review of the geotechnical aspects of the plans and specifications.

- Shoring/Dewatering Design Review: Assuming that the shoring and dewatering systems are designed by the contractor, a qualified geotechnical engineer should review the proposed systems in detail including review of engineering computations. This review would not be a certification of the contractor's plan but rather an independent review made with respect to the owner's interests.

- Construction Observations: A qualified geotechnical engineer should 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 the shoring monitoring data and respond to any specific geotechnical problems that occur.

REFERENCES

CHAPMAN, K., Cording, E.J., and Schnabel, H., Jr., 1972, Performance of a Braced Excavation in Granular and Cohesive Soils: ASCE Specialty Conference on Performance of Earth and Earth-Supported Structures, Purdue University, Vol. III, pp. 271-293.

CLOUGH, G.W., 1980, Slurry walls for underground tram facilities: U.S. Department of Transportation Report FHWA-TS-80-221.

CLOUGH, G.W., Buchignani, A.L., 1981, Slurry walls in San Francisco Bay area: ASCE National Conference, New York, p. 81-142.

CORDING, E.J., and O'Rourke, T.D., 1977, Excavation, ground movements, and their influence on buildings: American Soc. of Civil Engineers, Preprint, San Francisco.

CORPS OF ENGINEERS, 1953, Technical Memorandum No. 3-357, March, 1953.

CRANDALL, L.R., and Maljian, P.A., 1977, Use of earth anchors to restrict ground movements: Am. Soc. Civil Engineers, Preprint 2974, p. 1-27.

DEPARTMENT OF THE INTERIOR, Bureau of Reclamation, 1963, Earth Manual.

EVANS, L.T., 1968, Swell and settlement study Equitable Life Building, Los Angeles, California: Report by L.T. Evans, Inc.

GOLDBERG, D.T., Jaworski, W.E., and Gordon, M.D., 1976, Lateral Support Systems and Underpinning: Federal Highway Administration, Offices of Research & Development, Vols. I, II, III.

HARDIN, B.O., 1970, Suggested Methods of test for shear modulus and damping of soils by resonant column: ASTM Special Technical Publication 479.

KISHIDA, H.J. 1969, Characteristics of liquified sand during Mino-Owari Tohnakai and Fukui earthquakes: Soils and Foundations, Japan, Vol. 9, No. 1, March, p. 79-92.

LEE, K.L., and Fitton, J.A., 1968, Factors affecting the cyclic loading strength of soil, vibration effects of earthquakes in soils and foundations: American Society for Testing and Materials, Special Technical Publication 450.

LOS ANGELES COUNTY FLOOD CONTROL DISTRICT, 1976, Hydrologic Report 1974-1975.

MALJIAN, P.A., and Van Beveren, J.F., 1974, Tied-back deep excavations in Los Angeles area: Journal of Constr. Div., ASCE, Vol. 100 CO3, p. 337-356.

MANA, A.I., Clough, G.W., 1981, Prediction of movements for braced cuts in clay: ASCE Geotechnical Journal, June.

MATSUO, H., and O'Hara, S., 1960, Lateral earth pressures and stability of quay walls during earthquakes: Proceedings of Second World Conference on Earthquake Engineering, Tokyo, Japan.

MONOBE, N., and Matsuo, H., 1929, On the determination of earth pressures during earthquakes: Proceedings, World Engineering Conference, Vol. 9, p. 176.

NAVFAC, 1971, Design Manual 7-Soil mechanics, foundations, and earth structures: Department of the Navy, Naval Facilities Engineering Command.

NAVFAC, 1982, Design Manual 7.1-Soil mechanics: Department of the Navy, Naval Facilities Engineering Command, May.

NELSON, J.C., 1973, Earth tiebacks support excavation 112 feet deep in Civil Engineering: Am. Soc. Civil Engineers, Nov. 1973, p. 41-44.

OKABE, S., 1926, General theory of earth pressure: Journal of Japanese Society of Civil Engineers, Vol. 12, No. 1.

PRAKASH, S., 1981, Soil dynamics: McGraw-Hill, New York.

SCHULTZ, M.S., 1981, An empirical investigation into the behavior of diaphragm walls, Masters Thesis, Massachusetts Institute of Technology.

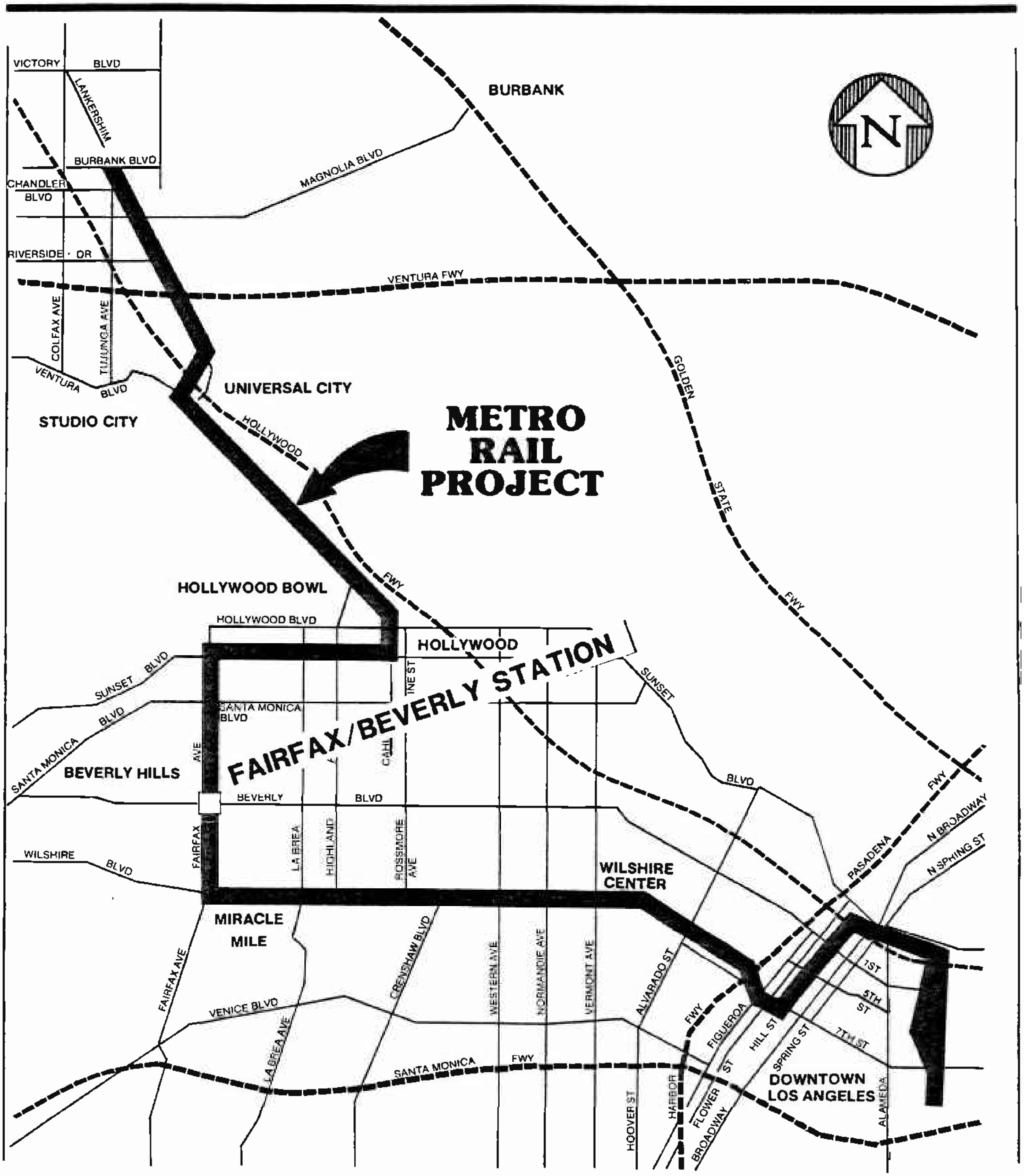
SEED, H.B., Idriss, I., Arango, I., 1983, Evaluation of Liquefaction Potential Using Field Performance Data: ASCE Journal of Geotechnical Division, Vol. 109, No.3, March 1983, p. 458-482.

SEED, H.B, and Whitman, R.V., 1970, Design of earth retaining structures for dynamic loads in Ground and Design of Earth Retaining Structures: ASCE Specialty Conference on Lateral Stresses New York, p. 103-148.

SEED, H.B. and Idriss, I.M., 1967, Analysis of soil liquefaction: Niigata Earthquake Journal of the Soil Mechanics and Foundations Division, ASCE Vol. 93, No. SM3, Proceedings Paper 5233, May, p. 83-108.

WESTERGAARD, H.N., 1933, Water pressures on dams during earthquakes: Transactions, ASCE, p. 418-433.

YOUD, L.T., 1982, U.S. Geological Survey, Menlo Park; personal communication.



VICINITY MAP

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METRO RAIL PROJECT

Project No. **83-1140**

Drawing No. **1**

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Approved for publication 4/84 by [signature]

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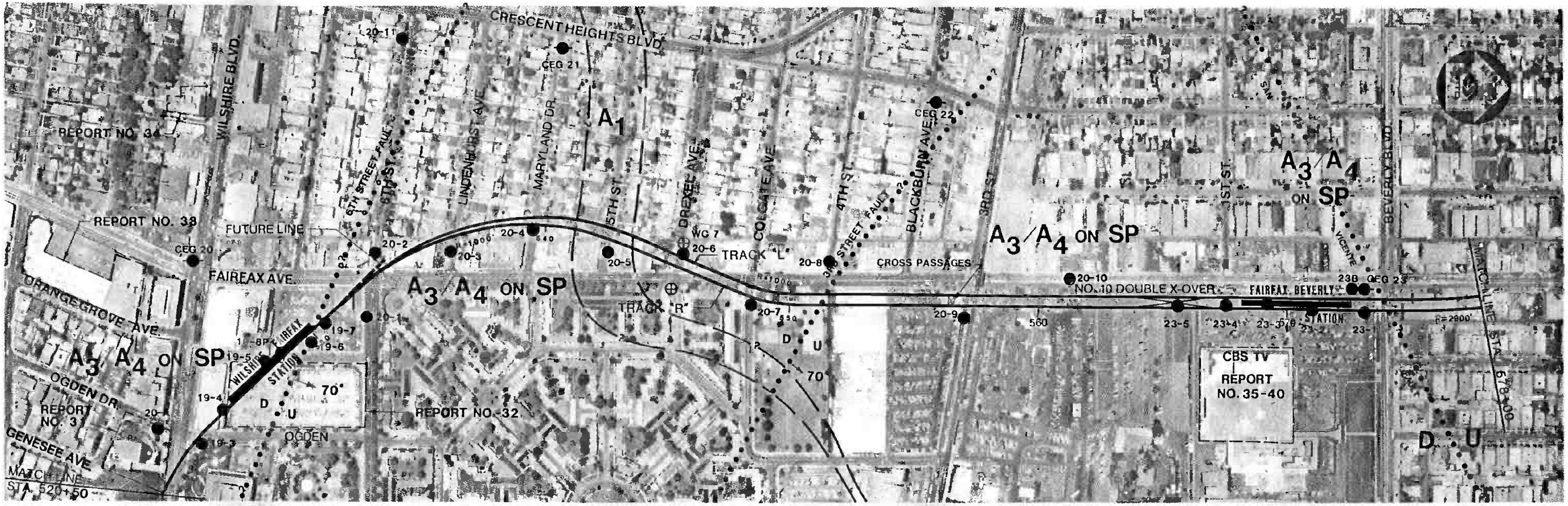
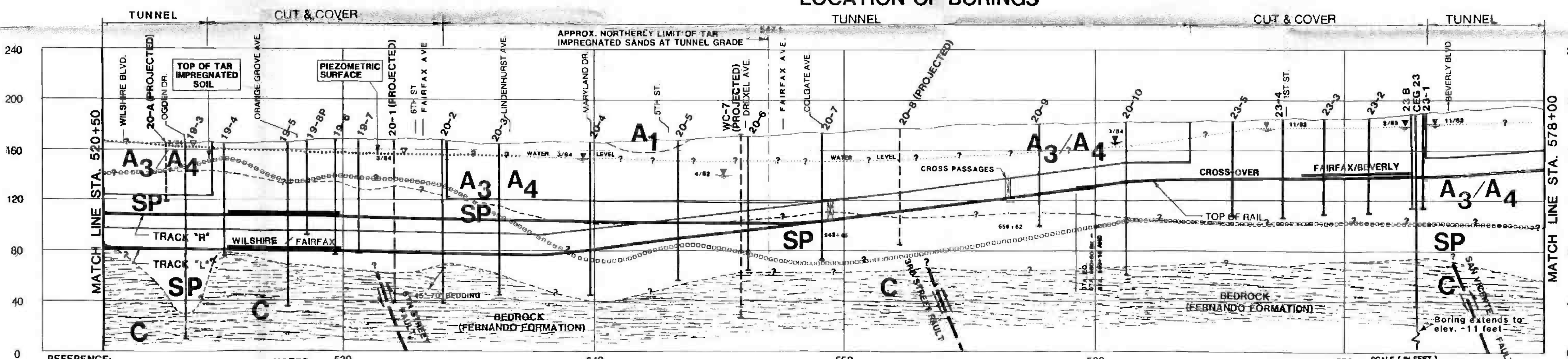


LOCATION OF BORINGS



GEOLOGIC SECTION

REFERENCE:
MILESTONE 10 SHEET 10 OF 21 ALIGNMENT PLAN AND PROFILE STATION 520+50 TO STATION 578+00 DATED MARCH 1983

NOTES:
1.) LOCATION AND GRADE OF TUNNEL AND STATION SUBJECT TO CHANGE.
2.) FOR EXPLANATION OF GEOLOGIC SYMBOLS SEE DRAWING NO. 5.
3.) THIS DRAWING WAS PREPARED AS AN AID IN DEVELOPING DESIGN RECOMMENDATIONS. SUBSURFACE INFORMATION PRESENTED ON DRAWING IS BASED ON INTERPOLATION AND EXTRAPOLATION OF SUBSURFACE DATA BETWEEN AND BEYOND BORING LOCATIONS. ACTUAL CONDITIONS ENCOUNTERED DURING CONSTRUCTION MAY BE DIFFERENT.

REV.	DATE	BY	SUB.	APP.	DESCRIPTION

THE PREPARATION OF THIS DRAWING HAS BEEN FINANCED IN PART THROUGH A GRANT FROM THE U. S. DEPARTMENT OF TRANSPORTATION, URBAN MASS TRANSPORTATION ADMINISTRATION, UNDER THE URBAN MASS TRANSPORTATION ACT OF 1964, AS AMENDED, AND IN PART BY THE TAXES OF THE CITIZENS OF LOS ANGELES COUNTY AND OF THE STATE OF CALIFORNIA

REV.	DATE	BY	SUB.	APP.	DESCRIPTION

DESIGNED BY
DRAWN BY
CHECKED BY
IN CHARGE
DATE

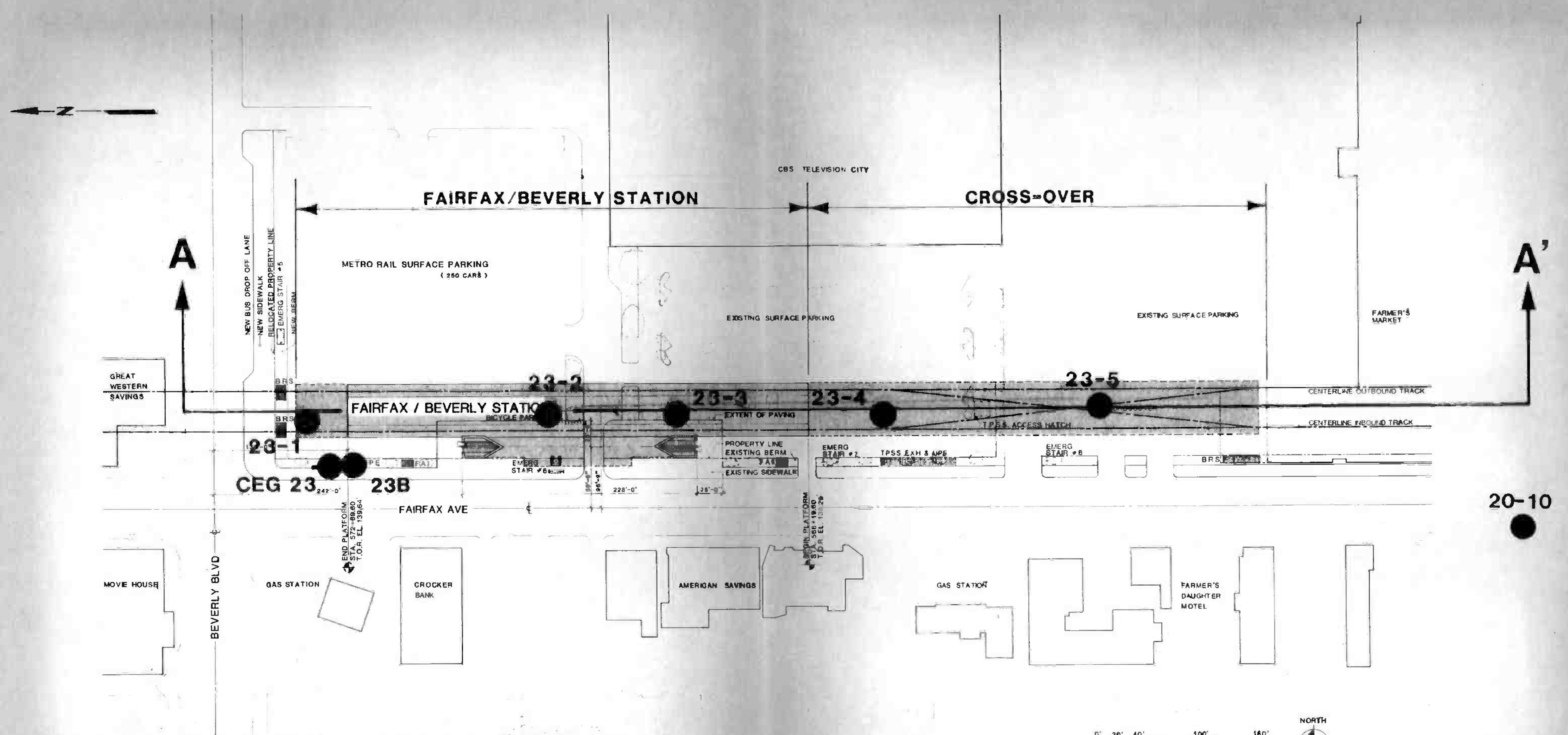
SOUTHERN CALIFORNIA RAPID TRANSIT DISTRICT
METRO RAIL PROJECT
RTD

CCI/ESA/GRC
General Geotechnical Consultants
Submitted R.M. Pride Date 4-13-84

DMJM/PBQD/KE/HWA
A JOINT VENTURE
GENERAL CONSULTANTS
APPROVED

DESIGN UNIT A275 LOCATION OF BORINGS AND GEOLOGIC SECTION

PROJECT NO. 83-1140
DRAWING NO. 2
REV.
SCALE AS SHOWN
SHEET NO.



REF: "PRELIMINARY FAIRFAX/BEVERLY STATION SITE PLAN" DRAWING #A-46, PREPARED BY HARRY WEESE & ASSOCIATES, ORIGINAL SCALE 1" = 40', DATED 1-10-83.

NOTES: 1.) FOR SUBSURFACE SECTION SEE DRAWING NO. 4
2.) FOR EXPLANATION OF SYMBOLS SEE DRAWING NO. 5

LOCATION OF BORINGS

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Southern California Rapid Transit District
METRO RAIL PROJECT

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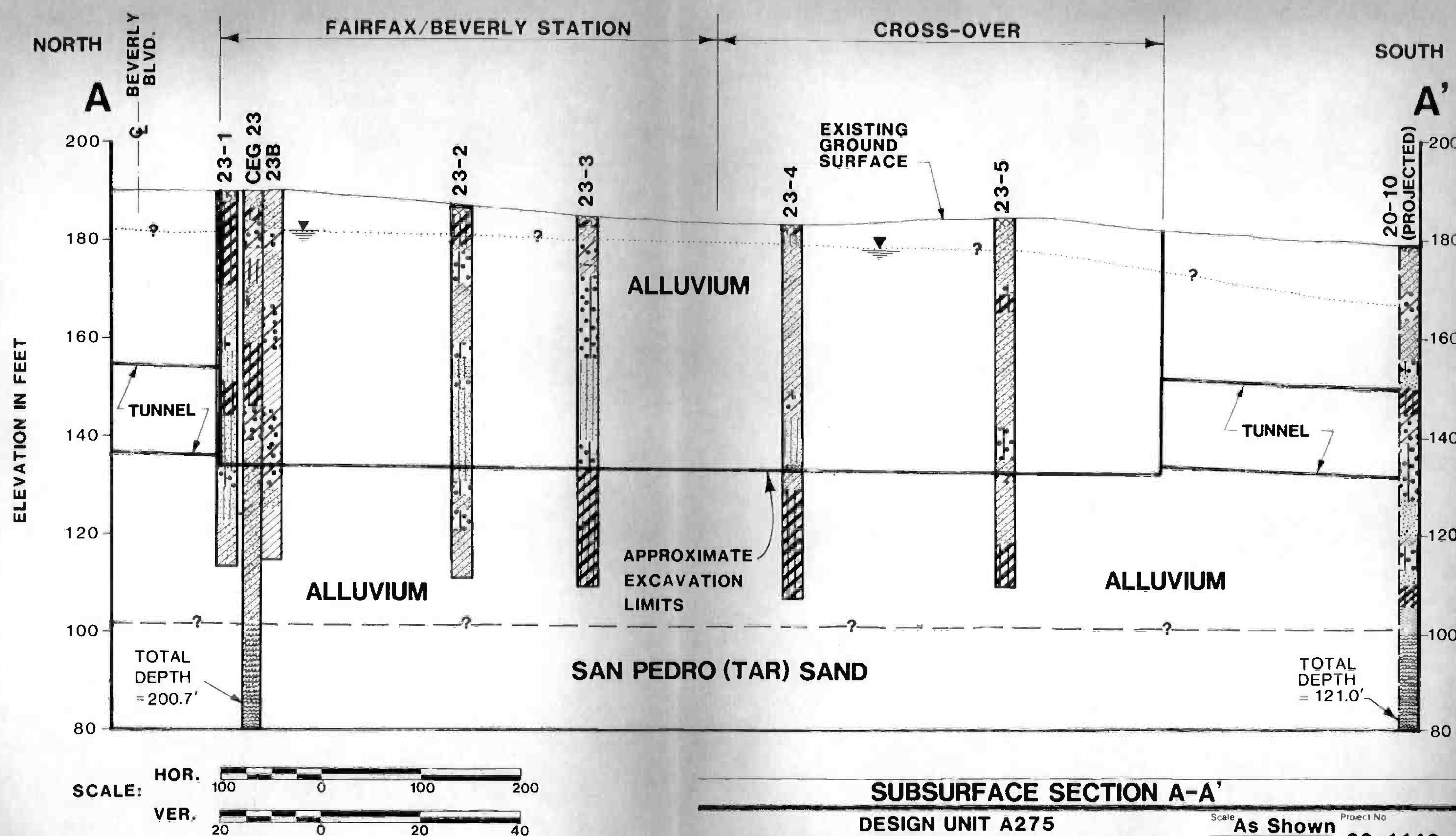
Scale	As Shown
Date	APR., 1984
Prepared by	CSJ
Checked by	RG
Approved By	JAD
Project No	83-1140
Drawing No	3



NOTES: 1.) FOR LOCATION OF SUBSURFACE SECTION A-A' SEE DRAWING NO. 3
2.) FOR EXPLANATION OF SYMBOLS SEE DRAWING NO. 5.

SUBSURFACE SECTION A-A'

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Scale: As Shown	Project No: 83-1140
Date: APR., 1984	Drawing No: 4
Prepared by: CSJ	
Checked by: RG	
Approved By: JAD	

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.

QUATERNARY — 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
- U ●●●●●? D Fault (view in plan): dotted where concealed; queried where inferred; (U) upthrown side, (D) downthrown side
- ? Fault (view in geologic section): 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
- ? Ground 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) Borings projected more than 100′ 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.

3) 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 A275
Southern California Rapid Transit District
METRO RAIL PROJECT

Converse Consultants — Geotechnical Engineering and Applied Sciences

Scale	N/A
Date	APR., 1984
Prepared by	RG
Checked by	JAD
Approved By	HAS
Project No	83-1140
Drawing No	5

Appendix A

Field Exploration

APPENDIX A FIELD EXPLORATION

A.1 GENERAL

Field exploration data presented in this report for Design Unit A275 includes logs of borings drilled for the 1981 Geotechnical Investigation Report, the 1983 borings drilled for this A275 investigation, and a 1984 boring drilled for Design Unit A250. The specific boring logs included are summarized below:

- 1981

  CEG-23

- 1983 - A275

  23B, 23-1 through 23-5

- 1984 - A250

  20-10

Locations of the borings are shown on Drawings 2 through 4. Ground water observation wells (piezometers) were installed in the borings listed in Section 5.4 (Table 5-1). Geophysical downhole surveys were made for the 1981 investigation at Boring CEG-23 within the A275 investigation site, and Boring CEG-20, 23A and 24 for adjacent Design Units A250 and A310. Geophysical crosshole surveys were also carried out at Borings CEG-20 and CEG-24, and a seismic refraction survey was made at Fairfax High School located approximately 2400 feet north of the station site (see Appendix B).

The borings were drilled to depths generally ranging from 75 to 200 feet, and at two locations penetrated through the alluvium into the underlying bedrock. All borings were sampled at regular intervals using the Converse ring sampler, pitcher barrel sampler and the standard split spoon sampler. Sample recovery was generally good in both the siltstone and claystone bedrock and the alluvium.

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 the three firms (CCI/ESA/GRC) 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 logs 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

Most of the drilling was performed by Pitcher Drilling Company of East Palo Alto, California, with Failing 750 and 1500 rotary wash rigs, each operated by a two-man crew. The man-sized auger boring was drilled with bucket auger equipment by A&W Drilling Company of Brea, California.

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 320-pound slip-jar hammer with an 18-inch drop. The Converse sampler was followed with a standard split spoon sample (SPT) driven with a 140-pound hammer with a 30-inch stroke. Where the Fernando Formation was encountered, the borings were generally continuously sampled using a Pitcher Barrel sampler. Converse ring samples were also recovered.

The most common cause for loss of samples or altering the sample interval was when gravel was 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 field 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. (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.) Although particle size distribution estimates were based on volume rather than weight, the field estimates should fall within an acceptable range of accuracy. A description of the Unified Soil Classification Symbols used on the borings logs is presented in Table A-1 below.

TABLE A-1
UNIFIED SOIL CLASSIFICATION SYMBOLS

GRANULAR SOILS

SYMBOL	DESCRIPTION
GW	Well-graded gravels, gravel-sand mixtures, little or no fines
GP	Poorly graded gravels, gravel-sand mixtures, little or no fines
GM	Silty gravels, gravel-sand-silt mixtures
GC	Clayey gravels, gravel-sand-clay mixtures
SW	Well-graded sands, gravelly sands, little or no fines
SP	Poorly graded sands, gravelly sands, little or no fines
SM	Silty sands, sand-silt mixtures
SC	Clayey sands, sand-clay mixtures

FINE-GRAINED SOILS

SYMBOL	DESCRIPTION
ML	Inorganic silts and very fine sands, rock flour, silty or clayey fine sands, or clayey silts with slight plasticity
CL	Inorganic clays of low to medium plasticity, gravelly clays, sandy clays, silty clays, lean clays
OL	Organic silts and organic silty clays of low plasticity
MH	Inorganic silts, micaceous or diatomaceous fine sandy or silty soils, elastic silts
CH	Inorganic clays of high plasticity, fat clays
OH	Organic clays or medium to high plasticity, organic silts
Pt	Peat and other highly organic soils

Table A-2 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-2 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-3.

A.4 PIEZOMETER INSTALLATION

Piezometers were installed in borings CEG-23 and 20-10 located either at or in the vicinity of the Fairfax/Beverly Station site. Procedures for piezometer installation were as follows:

A 2-inch diameter plastic ABS pipe was installed in the boring. At least the lower 20 feet of the ABS pipe was perforated, and the annulus of the boring around the perforated portion of the pipe was backfilled with a coarse sand/pea gravel aggregate. Concrete/bentonite slurry was used to backfill around the non-perforated portion of the pipe 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 piezometers are presented in Section 5.4 of the text.

TABLE A-3 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.

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 23

Proj: DESIGN UNIT A275 **Date Drilled** 12/31/80 - 1/4/81 **Ground Elev.** 188'

Drill Rig Failing 1500 **Logged By** L. Schoeberlein **Total Depth** 200.7'

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

DEPTH	USCS	MATERIAL CLASSIFICATION	SAMPLE	BLOWS (6")	DRILL MODE	REMARKS
0		0.0-0.5 CONCRETE				
	CL	0.5-1.5 CLAY: grayish black; trace of fine sand; moist			AD	
2	CL	ALLUVIUM 1.5-3.5 SANDY CLAY: brownish black; moist				augered to 10'
4	CL	3.5-6.2 SILTY CLAY: medium bluish grey; stiff; moist				
			J-1	4 5 12	SS	1.5/1.5 recovery
6	SC/ CL	6.2-12.0 CLAYEY SAND/SANDY CLAY: light greenish grey			AD	
8						ground water at 9.5'
10			C-1		DR	1.0/1.0 recovery 12/31/80 1/2/81
					RD	drilling with 4 7/8" drag bit
12	CL	12.0-14.0 CLAY: greenish grey; stiff				
14	ML	14.0-19.0 CLAYEY SILT: dark greenish grey; dry to moist; very stiff				
			J-2	0 9 15	SS	1.5/1.5 recovery
16					RD	
18						
	CL	19.0-23.5 SANDY CLAY: dark greenish grey;				
20						

Project DESIGN UNIT A275 Date Drilled 12/31/80-1/4/81 Hole No. 23

DEPTH	USCS	MATERIAL CLASSIFICATION	SAMPLE	BLOWS (6")	DRILL MODE	REMARKS
20	CL	19.0-23.5 SANDY CLAY: continued occasional fine gravel; very stiff; dry to moist	C-2		DR	0.8/1.0 recovery
22					RD	
24	CL	23.5-31.0 SANDY CLAY: dark greenish grey; occasional fine to coarse gravel; stiff; moist				
26			J-3	11 16 25	SS	1.0/1.5 recovery
28					RD	
30			C-3		DR	0.7/1.0 recovery
32	CL	31.0-44.0 SILTY CLAY: dark greenish grey; hard; moist			RD	
34						
36			J-4	9 16 23	SS	1.5/1.5 recovery
38					RD	
40			C-4		DR	
42					RD	
44						

Project DESIGN UNIT A275 Date Drilled 12/31/80 - 1/4/81 Hole No. 23

DEPTH	USCS	MATERIAL CLASSIFICATION	SAMPLE	BLOWS (6")	DRILL MODE	REMARKS
44	SC	44.0-51.0 CLAYEY SAND: dark greenish grey; interbedded with sandy clay; dense; moist to wet			RD	
			J-5	11	SS	1.3/1.5 recovery
				14		
46				17		
					RD	
48						
50			C-5		DR	0.8/1.0 recovery
	CL	51.0-64.0 SANDY CLAY: dark greenish grey; interbedded with clayey sand; very stiff; moist			RD	
52						
54						
			C-6		DR	1.0/1.0 recovery
56					RD	
58						
60		becoming hard	J-6	6	SS	1.1/1.5 recovery
				18		
				25		
					DR	
62						
64	CL	64.0-88.0 SANDY CLAY: greenish black; hard; contains low petroleum content; dry to moist				
			C-7		DR	0.8/1.0 recovery
66			J-7	33	SS	gas test 21% O_2, 0% combustibles
				49		
				43		
68					RD	

Project DESIGN UNIT A275 **Date Drilled** 12/31/80 - 1/2/81 **Hole No.** 23

DEPTH	USCS	MATERIAL CLASSIFICATION	SAMPLE	BLOWS (6")	DRILL MODE	REMARKS
68	CL	64.0-88.0 SANDY CLAY: continued			RD	
70		vertical petroleum streaks	J-8	17 35 40	SS	1.5/1.5 recovery
72					RD	
74						
			J-9	51 46 53	SS	1.5/1.5 recovery
76					RD	
78						
80		6" petroleum rich lens becoming more sandy	J-10	26 45 46	SS	1.5/1.5 recovery
82					RD	
84			C-8		DR	0.8/0.95 recovery
			J-11	30 56	SS	1.0/1.0 recovery gas detector indicates 21% O_2 and 0% combustibles
86					RD	
88	SP	88.0-115.0 TAR SAND: black; fine to medium sand; very dense; petroleum binder				
90			J-12	37 70	SS	0.9/1.0 recovery petroleum sample
92					RD	

Project DESIGN UNIT A275 Date Drilled 12-31-80 Hole No. 23

DEPTH	USCS	MATERIAL CLASSIFICATION	SAMPLE	BLOWS (6")	DRILL MODE	REMARKS
92	SP	88.0-115.0 TAR SAND: (continued)			RD	
94						
			J-13	57	SS	0.5'/0.5' recovery
					RD	
96						
	GP	6" gravel lens				rig chatter
98	SP					
100			J-14	55	SS	
					RD	
	GP	6" gravel lens				rig chatter
102	SP					
		fine sand				
104			C-9		DR	0.8/0.9 recovery
			J-15	84	SS	0.5/0.5 recovery
					RD	
106						
108						
110			J-16	52 50	SS	0.7/0.7 recovery
					RD	gas: 6% combustibles 21% O_2 rig chatter
112	GP					
	SP	112.5 6" gravel lens				
114		114.5 6" gravel lens				
		WEATHERED FERNANDO FORMATION				
	CL	115.-122.0 SILTY CLAYSTONE: greenish black	J-17	28 39	SS	
116						

Sheet 5 of 9

Project DESIGN UNIT A275 Date Drilled 12-31-80/1-4-81 Hole No. 23

DEPTH	USCS	MATERIAL CLASSIFICATION	SAMPLE	BLOWS (6")	DRILL MODE	REMARKS
116	CL	115.0-122.0 SILTY CLAY: (continued)	J-17	50	SS	1.4/1.4 recovery
		very stiff; contains streaks of petroleum rich silt & fine sand; dry to moist			RD	
118						
120	SP	tar sand lens	J-18	58	SS	
	CL				RD	0.5/0.5 recovery
122		FERNANDO FORMATION				
		122.0-140.2 CLAYEY SILTSTONE: olive black to dark greenish grey; poorly cemented; contains streaks and interbeds of fine tar sand				
124			C-10		DR	
		Physical Condition: closely fractured, soft to friable hardness; plastic to friable strength moderate to little weathered	J-19	28 49 50	SS	
126					RD	
128						
		well cemented				gas: 6% combustibles 21% O_2 1-2-81
130		softer (less cement)				1-3-81 gas: 100% combustibles 18% O_2
132						bubbling visible foam 1' from ground surface
			Box 1		PB	changed to 4 7/8 tri-cone
134						1.6/2.8 recovery
						damaged tube drilling through highly cemented zone
136			S-1		PB	
	60°	tar sand				2.2/2.2 recovery
138		siltstone	Box 1 Cont.		PB	gas: 100% combustibles, 18% O_2
140						2.2/2.7 recovery

Project DESIGN UNIT A275 Date Drilled 12-31-80/1-4-81 Hole No. 23

DEPTH	USCS	MATERIAL CLASSIFICATION	SAMPLE	BLOWS (6")	DRILL MODE	REMARKS
140		140.2-200.7 TAR SAND: black; fine sand occasional fine gravel; medium dense to dense; petroleum content varies; siltstone interbeds; becoming dense to very dense and finer with depth, siltstone interbeds as described in 122.0-140.2 interval	Box 1 CONT.		PB	
142						
					PB	143.1 sample removed for petroleum testing
144		144.6 well cemented siltstone concretions				1.5/2.7 recovery
146					PB	
		147.6 well cemented siltstone lens, moderately to well cemented				
148		148 - concretions			PB	2.1/2.2 recovery
150			Box 2			2.5/2.8 recovery
					PB	slow extruding, sample expanding in tube maximum expansion 2-3"
152		interbedded siltstone	S-2			2.7/2.8 recovery
154		153.9 thin siltstone lens	Box 2 Cont.		PB	pocket penetrometer > 4.5 ksf 2-9-81
		154.6 siltstone, slicken sides on most fracture surfaces				2.4/2.8 recovery
156	60°	156.9 very thin cemented zone			PB	2.7/2.7 recovery
158		158.5 clayey siltstone				
					PB	2.5/2.8 recovery
160						0.8' extruded, rest could not be extruded
162		siltstone			PB	1.2/2.7 recovery
164						

Project DESIGN UNIT A275 Date Drilled 12-31-80/1-4-81 Hole No. 23

DEPTH	USCS	MATERIAL CLASSIFICATION	SAMPLE	BLOWS (6")	DRILL MODE	REMARKS
164		140.2-200.7 TAR SAND: (continued)	Box 2 Cont.		PB PB	pocket penetrometer > 4.5 tsf, 2-9-81 2.8/2.8 recovery
166						
168			S-3		PB	2.8/2.8 recovery
170			Box 2 Cont. Box 3		PB	2.8/2:8 recovery sample expanding in tube & bubbling Ryland & Cummings gas testing 1-3-81 1-4-81
172		thinly interbedded siltstone				
174		siltstone with tar sand streaks			PB	pocket penetrometer 4.0 tsf, 2-9-81 2.8/2.8 recovery
176	50°	176-179.5 possible fault gauge moderately cemented, intensely fractured dominantly tar in sample			PB	strong sulfur odor losing circulation 1.7/2.7 recovery
178		tar sand, loose, coarse sand and fine gravel thin, blue green clay lens no tar, fine grained tar sand			PB	2.8/2.8 recovery
180						
182			S-4		PB	1.9/2.8 recovery pocket penetrometer 2.75 tsf 2-9-81 2.7/2.8 recovery
184			Box 3 Cont.		PB	
186						
188			Box 4		PB	

Project DESIGN UNIT A310 Date Drilled 12-31-80 - 1-4-81 Hole No. 23

DEPTH	USCS	MATERIAL CLASSIFICATION	SAMPLE	BLOWS (6")	DRILL MODE	REMARKS
188		140.2-200.7 TAR SAND: cont.	Box 4 (cont)		PB	2.5/2.7 recovery
190		thin gravelly tar and coarse sand lens			PB	2.8/2.8 recovery
192		occasional coarse sand and fine gravel			PB	2.7/2.8 recovery pocket penetrometer 2.75 tsf 2/9/81
194					PB	2.8/2.8 recovery
196						
198			S-5		PB	2.7/2.7 recovery
200						
202		BH 200.7 ft. Terminated hole 1/4/81; downhole geophysical survey (Bruce Auld) completed 1/4/81; E-logs (ESA) completed 1/4/81; site cleaned and piezometer set to 200' for gas monitoring. Moved off site 1/4/81. Water sampled 2/13/81.				
204						
206						
208						
210						
212						

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BORING LOG 23B

Proj: DESIGN UNIT A-275 Date Drilled 2/2/83 Ground Elev. 189.5'

Drill Rig B. Auger Logged By D. Gillette Total Depth 75.0'

Hole Diameter 36" Hammer Weight & Fall N/A

DEPTH	USCS	MATERIAL CLASSIFICATION	SAMPLE	BLOWS (6")	DRILL MODE	REMARKS
0	AF	0.0-0.5 CONCRETE				Observation hole - no samples required
	CH	0.5-2.0 CLAY: grayish black				H_2S odor & gas bubbles coming through sidewalk joints
2	CL	ALLUVIUM 2.0-8.0 SANDY CLAY: brownish black and bluish gray; stiff; moist				
8	SC CL	8.0-12.0 CLAYEY SAND/SANDY CLAY: light greenish gray; moist				groundwater at 8.5' after 21 hours
12	CL	12.0-23.0 SANDY CLAY: greenish gray and dark greenish gray; stiff; moist				
20						

Project DESIGN UNIT A-275 Date Drilled 2/2/83 Hole No. 23B

DEPTH	USCS	MATERIAL CLASSIFICATION	SAMPLE	BLOWS (6")	DRILL MODE	REMARKS
20	CL	12.0-23.0 SANDY CLAY: (continued)				
22						
	SC	23.0-33.0 CLAYEY SAND: dark greenish gray; moist				
24						
26						
28						strong H_2S odor
30						
32						water seep at 32' from northwest side of hole 20.5 gpm (approx.)
34	CH	33.0-44.0 CLAY: dark greenish gray; stiff; moist to wet				
36						
38						
40						40.0-75.0 petroleum in formation
42						
44						

Sheet 2 of 4

Project DESIGN UNIT A-275 Date Drilled 2/2/83 Hole No. 23B

DEPTH	USCS	MATERIAL CLASSIFICATION	SAMPLE	BLOWS (6")	DRILL MODE	REMARKS
44	SC	44.0-52.0 CLAYEY SAND: dark greenish gray; stiff				
46						strong H_2S odor
48						
50						
52	SC/SP	52.0-60.5 CLAYEY SAND/SAND: greenish black and light greenish gray; medium to coarse sand; dense; wet				52.0-62.5 water seeps - 18 gpm (approx.) water rises to 50', 45 min. after drilling to 70'
54						
56						
58						
60	CL	60.5-65.0 SANDY CLAY: greenish black; stiff				
62						
64	CH	65.0-75.0 CLAY: greenish black; very stiff; slightly moist				harder drilling
66						
68						

Sheet 3 of 4

Project DESIGN UNIT A-275 Date Drilled 2/2/83 Hole No. 23B

DEPTH	USCS	MATERIAL CLASSIFICATION	SAMPLE	BLOWS (6")	DRILL MODE	REMARKS
68	CH	65.0-75.0 CLAY: (continued)				strong H_2S odor
70						
72						
74						
76		B.H. 75.0' Terminated hole Special Hole Closure 1. Pea gravel placed from 1' to 50' (hole had caved from 70' to 50' over-night) to act as oil collection sump. 2. Replace concrete on eastside of Fair-fax (sidewalk) per LA City Inspector specifications.				Notes: 1. Water at 50' depth by 11:00 AM 2/2/83 2. Water at 8.5' depth by 7:00 AM 2/3/83 3. Water sample obtain-ed 2/3/83 4. Because of shallow water no down hole inspection was conducted.
78						
80						
82						
84						
86						
88						
90						
92						

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BORING LOG 23-1

Proj: DESIGN UNIT A-275 **Date Drilled** 11/6-7/83 **Ground Elev.** 189'

Drill Rig Failing 750 **Logged By** S. Slaff **Total Depth** 76.5'

Hole Diameter 4 7/8" **Hammer Weight & Fall** SS: 140 lbs @ 30", DR: 320 lbs @ 18

DEPTH	USCS	MATERIAL CLASSIFICATION	SAMPLE	BLOWS (6")	DRILL MODE	REMARKS
0		0.0-0.3 ASPHALT				Drilled 0.0-2.1' with 7" garbage barrel. 6" flight auger from 2.1-5.8'.
	CL	FILL 0.3-2.1 SANDY CLAY with RUBBLE: yellowish brown to brownish black; stiff; dry to moist			(GB)	
2					AD	(GB): garbage barrel
	CL	ALLUVIUM 2.1-4.3 SILTY CLAY: mottled, brownish black and light olive gray; trace of sand; stiff; moist		7	DR	0.9/1.0 recovery
			C-1	15		
4					AD	
	CL	4.3-19.4 SILTY CLAY: greenish gray; trace of sand; very stiff; moist	J-1	6 8 10	SS	1.5/1.5 recovery
6					RD	drilled on with 4 7/8" drag bit
8						groundwater level 11/7/83
		becoming dark greenish gray; petroleum odor	S-1		PB	2.5/2.5 recovery
10			J-2	4 6 6	SS	1.5/1.5 recovery
12					RD	
		becoming mottled, dusky green and pale green		7	DR	1.0/1.0 recovery
			C-2	10		
14					RD	
		mottling decreasing - color is predominately dusky green	J-3	6 9 12	SS	1.5/1.5 recovery
16					RD	
18				10	DR	1.0/1.0 recovery
		becoming more sandy	C-3	15		
20	CL	19.4-30.0 SANDY CLAY: grayish green;			RD	

Project DESIGN UNIT A-275 Date Drilled 11/6-7/83 Hole No. 23-1

DEPTH	USCS	MATERIAL CLASSIFICATION	SAMPLE	BLOWS (6")	DRILL MODE	REMARKS
20	CL	19.4-30.0 SANDY CLAY: (continued) very stiff; sulfurous odor; moist	J-4	6 11 11	SS	1.3/1.5 recovery
22					RD	
			C-4	7 12	DR	1.0/1.0 recovery
24		becoming silty, sand content increasing with depth			RD	slight rig chatter
			J-5	4 6 9	SS	1.5/1.5 recovery
26					RD	
28			S-2		PB	2.4/2.5 recovery
30	SM	30.0-32.6 SILTY SAND: grayish green; medium dense; sulfurous odor; wet	J-6	8 10 13	SS	1.5/1.5 recovery
32					RD	
	ML	32.6-38.0 SANDY SILT: grayish green; very stiff; micaceous; moist	C-5	8 18	DR	1.0/1.0 recovery
34					RD	
			J-7	4 10 14	SS	1.5/1.5 recovery
36	(SC)	becoming clayey				
					RD	
38	CL	38.0-44.0 SILTY CLAY: grayish green; stiff; micaceous; moist	C-6	18 31	DR	1.0/1.0 recovery rig chatter
					RD	
40			J-8	16 24 33	SS	1.5/1.5 recovery
					RD	
42		color change to dusky blue green				
44			C-7	19 32	DR	1.0/1.0 recovery

Project DESIGN UNIT A-275 Date Drilled 11/6-7/83 Hole No. 23-1

DEPTH	USCS	MATERIAL CLASSIFICATION	SAMPLE	BLOWS (6")	DRILL MODE	REMARKS
44	ML	44.0-57.0 SANDY SILT: dusky green			RD	1.5/1.5 recovery
			J-9	6 9 15	SS	11/6/83
46					RD	11/7/83
48			S-3		PB	2.5/2.5 recovery
50	(SM)	50.6-51.0 silty sand lens				
	(SM)	51.2-51.6 silty sand lens	J-10	10 20 25	SS	1.0/1.5 recovery
52					RD	
			C-8	13 25	DR	1.0/1.0 recovery
54		dark greenish gray			RD	
			J-11	6 9 14	SS	1.5/1.5 recovery
56					RD	
	CL	57.0-60.2 SANDY CLAY: grayish green; stiff; occasional fine gravel; wet				
58			C-9	23 30	DR	1.0/1.0 recovery
					RD	
60	ML/SM	60.2-67.5 SANDY SILT/SILTY SAND: dusky green; hard/dense; wet	J-12	12 22 26	SS	1.0/1.5 recovery
62					RD	
	(SM)	63.0-63.8 silty sand lens	C-10	16 17	DR	1.0/1.0 recovery
64		becoming clayey			RD	
		dark greenish gray				
			J-13	12 23 25	SS	1.5/1.5 recovery
66					RD	
68	CL	67.5-76.5 SANDY CLAY: greenish black;				

Project DESIGN UNIT A-275 Date Drilled 11/6-7/83 Hole No. 23-1

DEPTH	USCS	MATERIAL CLASSIFICATION	SAMPLE	BLOWS (6")	DRILL MODE	REMARKS
68	CL	67.5-76.5 SANDY CLAY: (continued) hard; contains petroleum streaks; moist	S-4		PB	2.5/2.5 recovery
70		grayish green	J-14	11 19 24	SS	1.5/1.5 recovery
72					RD	
			C-11	14 28	DR	1.0/1.0 recovery
74					RD	
76			J-15	13 35 50	SS	1.3/1.5 recovery 11/7/83
		B.H. 76.5' Terminated hole				Cleaned and conditioned hole. Tremmied in 5 sack cement grout. Cleaned site. Covered with steel street cover.
78						
80						
82						
84						
86						
88						
90						
92						

Sheet 4 of 4

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BORING LOG 23-2

Proj: DESIGN UNIT A275 | Date Drilled 11/5-6/83 | Ground Elev. 187'

Drill Rig Failing 750 | Logged By S. Slaff | Total Depth 75.9

Hole Diameter 4 7/8" | Hammer Weight & Fall 140 lb, 30" SS., 320 lbs, 18" DR

DEPTH	USCS	MATERIAL CLASSIFICATION	SAMPLE	BLOWS (6")	DRILL MODE	REMARKS
0		0.0-0.4 APSHALT			GB	Drilled 0.0'-0.6' with 7" garbage barrel
	SC	FILL			AD	
	CL	0.4-2.0 CLAYEY SAND/SANDY CLAY: moderate to dark yellowish brown; medium dense to stiff; dry to moist				
2	CL	ALLUVIUM		5	DR	0.8/1.0 recovery
		2.0-5.8 SILTY CLAY: grayish black; with sand and fine gravel; very stiff; moist	C-1	9		
					AD	
4			J-1	4	SS	1.1/1.5 recovery
				5		
				7		
					AD	
6	CL	5.8-8.4 SILTY CLAY: grayish green; stiff moist				
		becoming sandy		4	DR	1.0/1.0 recovery
			C-2	7		ground water entry at 11.0; rose to 8.0 within 5 min.
8					AD	
	CL	8.4-9.6 SANDY CLAY: moderate yellowish brown; soft; moist	J-2	1	SS	1.4/1.5 recovery
	SM	9.6-16.5 SILTY SAND: grayish green; wet below 11'; medium dense; micaceous		2		
10				5		
					AD	
12				11	DR	5" steel surface casing from 0.0-12.2'
		12.7 weak sulfurous odor	C-3	19		1.0/1.0 recovery
					RD	13.0 drilling on with 4 7/8" drag bit
14			J-3	8	SS	1.3/1.5 recovery
	(SW)	becoming sandier		12		
				12		
					RD	
16						
	CL	16.5-28.6 SANDY CLAY: dark greenish gray; very stiff; weak sulfurous odor				
			S-1		PB	2.5/2.5 recovery
18						
20			J-4	8	SS	

Project DESIGN UNIT A275 Date Drilled 11-5-6-83 Hole No. 23-2

DEPTH	USCS	MATERIAL CLASSIFICATION	SAMPLE	BLOWS (6")	DRILL MODE	REMARKS
20	CL	16.5-28.6 SANDY CLAY: continued	J-4	14 21	SS	1.5/1.5 recovery
		increasing sand with depth			RD	
22			C-4	16 26	DR	1.0/1.0 recovery
	(ML)	becoming silty			RD	
24			J-5	9 24 23	SS	0.7/1.5 recovery
26					RD	
			C-5	9 14	DR	1.0/1.0 recovery
28					RD	
	SM	28.6-31.4 SILTY SAND: grayish green; medium dense; occasional thin gravel lenses; wet	J-6	9 14 11	SS	1.5/1.5 recovery
30					RD	
	ML	31.4-35.8 SANDY SILT: grayish green; occasional gravel; very stiff; wet				
32			C-6	11 22	DR	1.0/1.0 recovery
					RD	
34			J-7	7 13 16	SS	1.5/1.5 recovery
					RD	slight rig chatter
36	SM/GM	35.8-38.0 SILTY SAND/GRAVELLY SAND: grayish green				
			S-2		PB	2.0/2.5 recovery
38	ML	38.0-56.6 SANDY SILT: grayish green; very stiff; occasional gravel; wet				
40			J-8	10 11 16	SS	1.2/1.5 recovery
					RD	slight rig chatter
42			C-7	14 29	DR	1.0/1.0 recovery
					RD	
44						

Project DESIGN UNIT A275 Date Drilled 11-5-6-83 Hole No. 23-2

DEPTH	USCS	MATERIAL CLASSIFICATION	SAMPLE	BLOWS (6")	DRILL MODE	REMARKS
44	ML	38.0-56.6 SANDY SILT: continued	J-9	9	SS	1.5/1.5 recovery
				16		
		weak sulfurous odor		23		
46					RD	
				21	DR	1.0/1.0 recovery
	(CL)	zone of coarser sand with clay	C-8	30		
48					RD	
	SM	49.0-50.6 silty sand lens	lost	5	SS	0.0/1.5 recovery
				9		
50				11		
	ML				RD	
52				10	DR	1.0/1.0 recovery
		becoming clayey	C-9	16		
					RD	
54			J-10	6	SS	1.3/1.5 recovery
				10		
				15		
56					RD	
	SM	56.6-59.6 SILTY SAND: grayish green; occasional gravel; medium dense; wet	S-3		PB	2.5/2.5 recovery
58						
60	CL	59.6-62.3 SANDY CLAY: grayish green; very stiff; moist	J-11	8	SS	1.5/1.5 recovery
				10		11/5/83
				14		
					RD	11/6/83
62				18	DR	
	SM	62.3-66.8 SILTY SAND: mottled-olive black and dark greenish gray; low petroleum content; dense; micaceous; moist	C-10	40		
					RD	0.9/1.0 recovery
64			J-12	10	SS	1.2/1.5 recovery
				17		
				22		
66					RD	
				20	DR	0.9/1.0 recovery
68	CL	66.8-75.9 SANDY CLAY: dark yellowish brown; low petroleum content;	C-11	33		

Project DESIGN UNIT A275 Date Drilled 11-5-6-83 Hole No. 23-2

DEPTH	USCS	MATERIAL CLASSIFICATION	SAMPLE	BLOWS (6")	DRILL MODE	REMARKS
68	CL	66.8-75.9 SANDY CLAY: very stiff; becoming hard; moist			RD	
			J-13	7 12 18	SS	0.4/1.5 recovery
70		becoming mottled - moderate yellowish brown, dark yellowish brown, greenish gray; very dusky red			RD	
72			C-12	26 50	DR	0.8/0.8 recovery refusal at 9-1/2"
		becoming more sandy			RD	slight rig chatter
74			J-14	28 50	SS	0.9/0.9 recovery refusal at 11" 11/6/83
76		B.O.H. 75.9 ft Terminated hole				Cleaned and conditioned hole. Tremied 5 sack cement grout into hole; Cleaned site. Placed steel cover over hole. 11/16/83 Removed steel hole cover. Cappped hole with concrete.
78						
80						
82						
84						
86						
88						
90						
92						

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 23-3

Proj: DESIGN UNIT A-275 Date Drilled 11/4/83 Ground Elev. 184.5'
Drill Rig Failing 750 Logged By S. Slaff Total Depth 75.8'
Hole Diameter 4 7/8" Hammer Weight & Fall SS: 140 lbs @ 30", DR: 320 lbs @ 18

DEPTH	USCS	MATERIAL CLASSIFICATION	SAMPLE	BLOWS (6")	DRILL MODE	REMARKS
0		0.0-0.4 ASPHALT			GB	Drilled 0.0-0.7' with 7" garbage barrel.
		FILL				
		ALLUVIUM			AD	Drilled 0.7-6.5' with 6" flight auger.
2	CH	0.6-2.6 SILTY CLAY: dusky yellowish brown; trace of sand; stiff; petroleum odor; moist				
	CL	2.6-4.8 SANDY CLAY: dark yellowish brown; very stiff; petroleum odor		8	DR	1.0/1.0 recovery
4		3.5' color change to pale yellowish brown	C-1	15		
					AD	
	SC	4.8-8.8 CLAYEY SAND: moderate yellowish brown; trace of gravel; stiff; moist		4	SS	1.5/1.5 recovery set 5" steel surface casing from 0.0-6.2', drilling on with 4 7/8" drag bit
6			J-1	5		
				6		
					RD	
8				3	DR	1.0/1.0 recovery
			C-2	5		
	CL/SC	8.8-9.8 SANDY CLAY/CLAYEY SAND: light olive gray; trace of gravel; loose; wet			RD	
10	CL/SC	9.8-12.6 SANDY CLAY/CLAYEY SAND: dark greenish gray; stiff; medium dense; wet		4	SS	1.3/1.5 recovery rig chatter
			J-2	4		
				2		
12		11.0-12.2 gravel lens			RD	
	SM	12.6-29.2 SILTY SAND: dark greenish gray; medium dense; faint petroleum odor; occasional gravel; wet		9	DR	1.0/1.0 recovery
14			C-3	18		
					RD	
				9	SS	1.0/1.5 recovery
16			J-3	14		
				14		
		16.6' thin gravel lens			RD	rig chatter
18				12	DR	1.0/1.0 recovery
			C-4	15		
20					RD	

Project DESIGN UNIT A-275 Date Drilled 11/4/83 Hole No. 23-3

DEPTH	USCS	MATERIAL CLASSIFICATION	SAMPLE	BLOWS (6")	DRILL MODE	REMARKS
20	SM	12.6-29.2 SILTY SAND: (continued)	J-4	10 12 16	SS	0.9/1.5 recovery
22					RD	
24			S-1		PB	1.9/2.5 recovery lost bottom 0.6' due to zone of softer material
26			J-5	19 18 11	SS	1.3/1.5 recovery
		27.2' small gravel lens			RD	rig chatter
28			C-5	7 28	DR	1.0/1.0 recovery
					RD	29.5' drilling harder
30	ML/SM	29.2-46.0 SANDY SILT/SILTY SAND: grayish green; hard; faint sulfurous odor; wet	J-6	7 14 19	SS	1.5/1.5 recovery
32					RD	
	(GM) (SM)	33.3-34.4' sand & gravel lens	C-6	22 32	DR	1.0/1.0 recovery
34					RD	
36			J-7	8 16 22	SS	1.5/1.5 recovery
					RD	
38			C-7	17 27	DR	1.0/1.0 recovery
					RD	
40			J-8	9 22 30	SS	1.5/1.5 recovery
42					RD	
44			PB-2		PB	

Project DESIGN UNIT A-275 Date Drilled 11/4/83 Hole No. 23-3

DEPTH	USCS	MATERIAL CLASSIFICATION	SAMPLE	BLOWS (6")	DRILL MODE	REMARKS
44	ML	29.2-46.0 SANDY SILT: (continued)	S-2		PB	2.0/2.5 recovery
46	SM	46.0-49.6 SILTY SAND: grayish green; dense; occasional fine to coarse gravel; wet	J-9	9 13 25	SS	0.9/1.5 recovery
					RD	
48			C-8	16 19	DR	0.9/1.0 recovery
					RD	
50	SM	49.6-52.0 SILTY SAND: dusky green; petroleum streaks; very dense; wet	J-10	12 24 28	SS	0.8/1.5 recovery
52					RD	
	CL	52.0-75.8 SILTY CLAY: mottled- olive black, light olive gray, and pale green; some sand lens; low petroleum content; hard; wet	C-9	22 38	DR	1.0/1.0 recovery
54					RD	
56			J-11	11 19 18	SS	0.2/1.5 recovery
		color change to dusky brown			RD	
58	(ML)	becoming more sandy and silty	C-10	37 50	DR	0.8/1.0 recovery
					RD	
60			J-12	16 36 47	SS	1.0/1.5 recovery
62					RD	
64			S-3		PB	2.5/2.5 recovery
66			J-13	37 50	SS	0.9/0.9 recovery refusal at 11"
68					RD	petroleum froth floating on mud tub

Project DESIGN UNIT A-275 Date Drilled 11/4/83 Hole No. 23-3

DEPTH	USCS	MATERIAL CLASSIFICATION	SAMPLE	BLOWS (6")	DRILL MODE	REMARKS
68	CL	52.0-75.8 SILTY CLAY: (continued) occasional fine gravel	C-11	55/50	DR	0.75/0.75 recovery
					RD	
70			J-14	66	SS	0.5/0.5 recovery
					RD	
72		color also mottled with grayish green				
			C-12	100	DR	0.5/0.5 recovery
74					RD	
			J-15	36/50	SS	0.8/0.8 recovery
76		B.H. 75.8' Terminated hole				
78						Tremmied 4 sack cement grout into hole. Covered hole with steel cover. 11/8/83 removed steel cover, capped hole with concrete.
80						
82						
84						
86						
88						
90						
92						

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BORING LOG 23-4

Proj: DESIGN UNIT A275 **Date Drilled** 11/3/83 **Ground Elev.** 183.2

Drill Rig Failing 750 **Logged By** S. Slaff **Total Depth** 76.3'

Hole Diameter 4 7/8" **Hammer Weight & Fall** 320 lbs, 18" DR, 140 lbs, 30" SS

DEPTH	USCS	MATERIAL CLASSIFICATION	SAMPLE	BLOWS (6")	DRILL MODE	REMARKS
0		0.0-0.25 ASPAHLT			GB	Drilled 0.0-0.5 with 7" garbage barrel.
	SC	FILL			AD	Drilled 0.5-3.0 with 6" auger.
	OH	ALLUVIUM 0.8-1.8 SILTY CLAY: grayish black; trace of sand; stiff; strong petroluem odor				
2	ML	1.8-3.8 SANDY SILT: mottled - grayish brown, dusky brown, grayish olive green; very stiff; occasional fine gravel; moist; strong petroleum odor	C-1	14 16	DR	1.0/1.0 reocvery
4	CL	3.8-6.6 SANDY CLAY: dusky yellowish brown; stiff; moist	J-1	3 6 6	SS	1.0/1.5 recovery
6					AD	set 5" steel surface casing from 0.0-6.2'. Drilling on with 4 7/8" drag bit.
	CL	6.6-11.0 SANDY CLAY: light olive gray; stiff; moist	C-2	4 8	DR	1.0/1.0 recovery
8					RD	
			J-2	2 3 3	SS	1.4/1.5 recovery
10	GC				RD	
	CL/SC	11.0-34.0 SANDY CLAY/CLAYEY SAND: grayish green; dense; occasional fine to coarse gravel; wet				
12			S-1		PB	2.5/2.5 recovery
14	SC					
	CL/SC		J-3	5 6 9	SS	0.6/1.5 recovery
16					RD	
18			lost	12 23	DR	0.0/1.0 recovery lost sample, rig chatte
	GC	19.2-20.0 gravel lens			RD	
20						

Sheet 1 of 4

Project DESIGN UNIT A275 Date Drilled 11/3/83 Hole No. 23-4

DEPTH	USCS	MATERIAL CLASSIFICATION	SAMPLE	BLOWS (6")	DRILL MODE	REMARKS
20	CL/SC SP	11.0-34.0 SANDY CLAY/CLAYEY SAND: cont. 20.2-22.0 silty sand lens	J-4	6 18 15	SS	0.5/1.0 recovery
					RD	
22	CL/ SC					
				4	DR	0.0/1.0 recovery
			lost	15		
24					RD	
			J-5	7 13 18	SS	1.5/1.5 recovery
26						
		color change to dusky green			RD	
28				17	DR	1.0/1.0 recovery
		becoming silty	C-3	28		
					RD	
30			J-6	5 7 12	SS	1.5/1.5 recovery
32						
			S-2		PB	2.5/2.5 recovery tube damaged by gravel
34	SP	34.0-38.2 CLAYEY SAND: dusky green; very dense; wet				
36		36.0- weak sulfurous odor	J-7	12 24 33	SS	1.5/1.5 recovery
					RD	rig chatter
38	ML/ SM	38.2-49.2 SANDY SILT/SILTY SAND: mottled-dusky green; hard; dense; wet	C-4	20 38	DR	1.0/1.0 recovery
					RD	
40			J-8	11 20 22	SS	1.3/1.5 recovery
42					RD	
44				15	DR	

Project DESIGN UNIT A275 Date Drilled 11-3-83 Hole No. 23-4

DEPTH	USCS	MATERIAL CLASSIFICATION	SAMPLE	BLOWS (6")	DRILL MODE	REMARKS
44	ML/SM	38.2-49.2 SANDY SILT/SILTY SAND: cont.	C-5	30	DR	1.0/1.0 recovery
					RD	
46			J-9	7 15 19	SS	1.5/1.5 recovery
					RD	
48		becoming sandy	C-6	43 50	DR	0.9/0.9 recovery
	SP	49.2-50.0 TAR SAND: very dusky red; some fines; low petroleum content; dense; moist			RD	
50	CL	50.0-54.0 SANDY CLAY: mottled - grayish green with blackish red, very dusky red; grayish brown and dusky brown; hard; with petroleum; moist	J-10	10 16 22	SS	1.5/1.5 recovery
52					RD	
54	CL	54.0-63.8 SILTY CLAY: dark greenish gray; trace of fine sand and gravel; low petroleum content; hard; moist	S-3		PB	1.9/2.5 recovery
56			J-11	11 23 30	SS	1.5/1.5 recovery
58					RD	slow drilling zone 57.0-59.0
			C-7	21 43	DR	1.0/1.0 recovery
60					RD	
			J-12	8 20 30	SS	1.5/1.5 recovery petroleum froth forming on top of mud tub
62					RD	
64	CL	63.8-76.3 SILTY CLAY: light olive gray; trace of sand and petroleum; trace of gravel; hard; moist	C-8	55 50	DR	0.8/0.8 recovery refusal at 10"
					RD	
66		66.0- olive black	J-13	23 50	SS	1.0/1.0 recovery refusal at 11-1/2"
68					RD	

Project DESIGN UNIT A275 Date Drilled 11-3-83 Hole No. 23-4

DEPTH	USCS	MATERIAL CLASSIFICATION	SAMPLE	BLOWS (6")	DRILL MODE	REMARKS
68	CL	63.8-76.3 SILTY CLAY: continued			RD	
			C-9	65	DR	0.5/0.5 recovery
70					RD	
72		strong petroleum odor	PB-4		PB	2.5/2.5 recovery tube damaged by gravel
74					RD	
76			J-14	20 35 50	SS	1.3/1.3 recovery refusal at 16" 11/3/83
		B.O.H. 76.3' Terminated hole.				11/4/83 circulated and conditioned hole. Tremmied grout through drill pipe. Used 5 sacks cement. Covered hole with steel street cover. 11/9/83 removed steel hole cover. Capped with concrete.
78						
80						
82						
84						
86						
88						
90						
92						

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BORING LOG 23-5

Proj: DESIGN UNIT A275 **Date Drilled** 11/2/83 **Ground Elev.** 184'

Drill Rig Failing 750 **Logged By** S. Slaff **Total Depth** 74.9'

Hole Diameter 4 7/8" **Hammer Weight & Fall** 320 lbs, 18" DR, 140 lbs, 30" SS

DEPTH	USCS	MATERIAL CLASSIFICATION	SAMPLE	BLOWS (6")	DRILL MODE	REMARKS
0		0.0-0.2 ASPHALT			GB	Drilled 0.0-0.4' with 7" garbage barrel.
	GM	FILL: dark yellowish brown; sandy gravel, some fines; med. dense, dry to moist			AD	Drilled 0.4-3.0 with 6" auger.
2	CL	ALLUVIUM 1.4-13.6 SANDY CLAY: dark yellowish brown; hard; moist				
				13	DR	1.0/1.0 recovery
4			C-1	25		
					AD	
		4.5-5.4 increasing sand content 4.5 moderate yellowish brown		10	SS	1.5/1.5 recovery
			J-1	17		
6				26		set 5" steel surface casing from 0.0-6.3'.
					AD	Drilling on with 4 7/8" drag bit.
8				16	DR	1.0/1.0 recovery
		becoming very sandy and very stiff	C-2	28		
					RD	
10				7	SS	1.5/1.5 recovery
		10.8-12.0 sandy zone	J-2	11		
				13		
12		12.0-12.5 gravelly zone; moderate yellowish brown to grayish orange			RD	rig chatter
				4	DR	1.0/1.0 recovery
14	SC	13.6-15.2 CLAYEY SAND: moderate yellowish brown; medium dense; moist	C-3	9		
					RD	
				3	SS	1.5/1.5 recovery
16	CL	15.2-19.4 SILTY CLAY: mottled - moderate yellowish brown to very pale orange; trace of sand; stiff; moist	J-3	5		
				8		
					RD	rig chatter
18		mottled with light brown; becoming hard; becoming sandier		18	DR	1.0/1.0 recovery
			C-4	32		
					RD	
20	CL	19.4-42.6 SANDY CLAY: greenish black				

Sheet 1 of 4

Project DESIGN UNIT A275 Date Drilled 11/2/83 Hole No. 23-5

DEPTH	USCS	MATERIAL CLASSIFICATION	SAMPLE	BLOWS (6")	DRILL MODE	REMARKS
20	CL	19.4-42.6 SANDY CLAY: continued hard; occasional fine gravel; moist	J-4	7 18 21	SS	1.5/1.5 recovery
					RD	
22						
		dark greenish gray; becoming less sandy	C-5	21 36	DR	1.0/1.0 recovery
24					RD	
			J-5	11 19 25	SS	1.2/1.5 recovery
	(SP)	25.5-26.4 silty sand lens				
26	CL				RD	
28			C-6	28 42	DR	1.0/1.0 recovery
	(SP)	28.9-29.5 silty sand lens			RD	
30	CL					
		becoming very stiff	J-6	8 15 14	SS	1.5/1.5 recovery
32					RD	
			C-7	26 40	DR	1.0/1.0 recovery
34					RD	
		weak sulfurous odor	J-7	11 13 16	SS	1.5/1.5 recovery
36					RD	
38			S-1		PB	2.5/2.5 recovery
40			J-8	7 9 13	SS	1.5/1.5 recovery
42					RD	
44	SM	42.6-49.0 SILTY SAND: dark greenish gray;				

Project DESIGN UNIT A275 Date Drilled 11/2/83 Hole No. 23-5

DEPTH	USCS	MATERIAL CLASSIFICATION	SAMPLE	BLOWS (6")	DRILL MODE	REMARKS
44	SM	42.6-49.0 SILTY SAND: (continued) medium dense; wet	C-8	26 48	DR	1.0/1.0 recovery
46					RD	
48			lost	9 14 16	SS	0.0/1.5 recovery lost sample probably since check ball did not seat.
	CL	49.0-51.4 SANDY CLAY: dark greenish gray; hard; wet			RD	
50			C-9	21 35	DR	1.0/1.0 recovery
	SC	51.4-54.0 CLAYEY SAND: dark greenish gray; very dense; wet			RD	
52			lost	12 22 28	SS	0.0/1.5 recovery
54	CL	54.0-66.3 SANDY CLAY: dark greenish gray; hard; interbedded thin clayey sand lenses; wet			RD	
56			PB-2		PB	2.5/2.5 recovery
58	(SP)	58.1-58.9 silty sand lens	J-9	16 43 46	SS	1.5/1.5 recovery
					RD	
60			C-10	33 60	DR	1.0/1.0 recovery
62					RD	
		mild sulfurous odor	J-10	12 20 24	SS	1.5/1.5 recovery
64					RD	
66	CH	66.3-74.9 SILTY CLAY: dark greenish gray; trace of sand, gravel and petroleum; hard; moist;	C-11	22 50	DR	0.9/0.9 recovery refusal at 11"
68					RD	

Project DESIGN UNIT A275 Date Drilled 11/2/83 Hole No. 23-5

DEPTH	USCS	MATERIAL CLASSIFICATION	SAMPLE	BLOWS (6")	DRILL MODE	REMARKS
68	CH	66.3-74.9 <u>SILTY CLAY</u>: continued strong petroleum odor	J-11	22 37 50	SS	1.4/1.4 recovery
70					RD	
72			S-3		PB	2.5/2.5 recovery
74			J-12	31 47 50	SS	1.4/1.4 recovery refusal at 17" 11/2/83
76		B.O.H. 74.9' Terminated hole.				Circulated fluid to condition hole. Tremmied in 2 sack cement grout through drill pipe 1' off bottom of hole. Cleaned site, covered hole with steel cover 11/5/83 Removed steel hole cover. Capped hole with concrete.
78						
80						
82						
84						
86						
88						
90						
92						

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BORING LOG 20-10

Proj:	DESIGN UNIT A250	**Date Drilled**	2/11-12/84	**Ground Elev.**	182'
Drill Rig	Failing 1500	**Logged By**	M. Schluter	**Total Depth**	121.0'
Hole Diameter	4 7/8"	**Hammer Weight & Fall**	325 lb @ 18", 140 lb @ 18"		

DEPTH	USCS	MATERIAL CLASSIFICATION	SAMPLE	BLOWS (6")	DRILL MODE	REMARKS
0		0.0-0.7 CONCRETE GUTTER			AD	started drilling @ 1300
		0.7-1.2 GRAVEL BASE				
2	CL	ALLUVIUM 1.2-9.0 SANDY CLAY: moderate brown; moist; soft				
4						
			C-1	8 25	DR	
6					RD	rotary wash
8						
10	SC	9.0-15.0 CLAYEY SAND: dark yellowish brown; loose; moist; trace gravel	J-1	3 5 7	SS	
12					RD	
14						
	CL	15.0-23.5 SANDY CLAY: dark yellowish brown; loose; moist; petrolifer-ous inclusions	C-2	3 5	DR	
16					RD	
18						
20						

Project DESIGN UNIT A250 Date Drilled 2-11/12-84 Hole No. 20-10

DEPTH	USCS	MATERIAL CLASSIFICATION	SAMPLE	BLOWS (6")	DRILL MODE	REMARKS
20	CL	15.0-23.5 SANDY CLAY: continued		5	SS	1.5/1.5 recovery
				10		
			J-2	5		
22					RD	
		increasing sand content				
	SM	23.5-26.0 SILTY SAND: dark greenish gray; medium dense; moist; trace micaceous; $CaCO_3$ infilling				
24						
				15	DR	
			C-3	41		
26	SP	26.0-29.0 SAND: dark greenish gray; poorly graded; medium dense; moist; sub-angular gravel			RD	
28						
	CL	29.0-34.0 SILTY CLAY: greenish black; moist; stiff; trace micaceous				
30				9	SS	1.5/1.5 recovery
				17		
			J-3	23		2-11-84
						2-12-84
32					RD	
34	SC	34.0-40.0 CLAYEY SAND: dark greenish gray; moist; medium dense; well graded; $CaCO_3$ infilling; trace gravel				
				14	DR	
			C-4	32		
36					RD	
38						
40	SM	40.0-52.0 SILTY SAND: dark greenish gray; well graded; moist; medium dense subangular sand grains; trace gravel; trace micaceous		11	SS	1.4/1.5 recovery
			J-4	26		
				34		
42					RD	
44						

Project DESIGN UNIT A250 Date Drilled 2-12-84 Hole No. 20-10

DEPTH	USCS	MATERIAL CLASSIFICATION	SAMPLE	BLOWS (6")	DRILL MODE	REMARKS
44	SM	40.0-52.0 SILTY SAND: continued			RD	
				22	DR	
			C-5	37		
46					RD	
48						
50		slight sulfurous odor		7	SS	1.5/1.5 recovery
			J-5	23		
				31		
52	SW	52.0-59.0 SAND: greenish black; well graded, medium dense to dense; sub-angular grains; trace gravel; slightly micaceous			RD	
54						
				41	DR	
			C-6	63		
56					RD	
58						
	CL	59.0-64.0 SANDY CLAY: dark greenish gray; moist; very stiff; micaceous				
60				26	SS	1.5/1.5 recovery
				38		
			J-6	41		
62					RD	
64	SM	64.0-66.0 SILTY SAND: dark greenish gray; moist; dense; micaceous				
				112	DR	refusal @ 7"
			C-7	50-1		
66	SW	66.0-69.0 SAND: dark greenish gray; well graded; dense; trace gravel			RD	
68						

Project DESIGN UNIT A250 Date Drilled 2-12-84 Hole No. 20-10

DEPTH	USCS	MATERIAL CLASSIFICATION	SAMPLE	BLOWS (6")	DRILL MODE	REMARKS
68	SW	66.0-69.0 SAND: continued			RD	
	CL	69.0-73.5 SILTY CLAY: greenish black; moist; stiff to very stiff; trace micaceous light brown gray - mottling				
70			J-7	11 21 22	SS	1.5/1.5 recovery
72					RD	
74	CL	73.5-79.0 SANDY TAR CLAY: greenish black, brownish black; stiff to very stiff; moist; petroleum odor; slightly sticky				
			C-8	51 70-	DR 5"	refusal @ 11"
76					RD	
78						
	SP/SC	79.0-83.0 TAR SANDS/CLAY TAR SANDS: brownish black; poorly graded; dense to very dense; petroleum odor; sticky				
80			J-8	41 50-	SS 3"	0.1/0.1 recovery refusal @ 8"
82					RD	
	SP	83.0-87.0 TAR SANDS: black; poorly graded; dense to very dense; petroleum odor; sticky				
84			C-9	106 50-	DR 1"	refusal @ 7" 5.5/5.5 ring recovered
					RD	
86						
	SP/SM	87.0-94.0 TAR SANDS/SILT TAR SANDS: black; poorly graded; dense to very dense; petroleum odor; sticky	C-10	83 70-	DR 2"	refusal @ 8"
88					RD	
		micaceous; trace gravel 5-30mm				
90			C-11	90 60	DR	refusal @ 9"
		gravels - cemented zone?			RD	
92						

Project DESIGN UNIT A250 Date Drilled 2-12-84 Hole No. 20-10

DEPTH	USCS	MATERIAL CLASSIFICATION	SAMPLE	BLOWS (6")	DRILL MODE	REMARKS
92	SM	87.0-94.0 TAR SANDS/SILT TAR SANDS: (cont.)			RD	
94	SM/ML	94.0-101.0 SILTY TAR SANDS: black; poorly graded; dense to very dense; petroleum odor; sticky	C-12	105 50-1	DR	refusal @ 7"
					RD	
96						
			C-13	106 50-1	DR	refusal @ 7"
98	ML				RD	
		micaceous; trace gravel 5-25 mm				
100			C-14	138	DR	refusal @ 6" bag 4.9/4.9 rings recovered
	SW/GM	101.0-111.0 GRAVELLY SAND/SANDY GRAVEL: black; well graded; dense to very dense; gravels: subangular to subrounded; 5-25 mm; clean; petroleum odor; sticky			RD	
102						
104			C-15	145	DR	refusal @ 6" bag 4.8/4.8 rings recovered
					RD	Drill rig chatter
106						
			C-16	142	DR	refusal @ 6" 3.5/3.5 rings recovered
108					RD	
110						
	SM	111.0-114.4 GRAVELLY SILTY SAND: black; 5-25 mm; subrounded; dense to very dense; trace to little gravel; petroleum odor; sticky	C-17	131	DR	refusal @ 6" bag 3.5/3.5 rings recovered
112					RD	
114			C-18	76 75-5	DR	refusal @ 11"
		FERNANDO FORMATION 114.4-121.0 SILTSTONE/CLAYSTONE:			RD	
116						

Project DESIGN UNIT A250 Date Drilled 2-12-84 Hole No. 20-10

DEPTH	USCS	MATERIAL CLASSIFICATION	SAMPLE	BLOWS (6")	DRILL MODE	REMARKS
116–121		114.4-121.0 SILTSTONE/CLAYSTONE: cont. dark greenish gray; very stiff to hard; micaceous; shells & shell fragments; massive; moist; slight petroleum odor; gaseous			RD	116.5 - drill rig chatter; cemented layer; 117.0 - Gas Pocket foamed drilling fluid; 80-100% explosive reading on foam; 0% at hole; rapid gas release; 118.0 - rig quieted down
			C-19	40	DR	
				96		
121–140		End of Boring 121.0' tremmied a 4 sac/90 gallon slurry mix into hole				Finished drilling @ 1530

Depth scale: 116, 118, 120, 122, 124, 126, 128, 130, 132, 134, 136, 138, 140

Sheet 6 of 6

Appendix B

Geophysical Exploration

APPENDIX B GEOPHYSICAL EXPLORATION

B.1 DOWNHOLE SURVEY

B.1.1 Summary

Downhole shear wave velocity surveys were performed in Borings CEG-23 for this Design Unit and in Borings CEG-20, 23A and 24 for adjacent Design Units A250 and A310. These adjacent surveys are located some distance from the Fairfax/Beverly Station site but are included to supplement the limited results obtained at Boring 23 since the soil conditions in the alluvium and bedrock do not vary considerably in this area. Measurements were made at 5-foot intervals from the ground surface to depths of 200 feet. A description of the technique and a summary of the results are attached.

B.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.

B.1.3 Data Analysis

For the purpose of illustration, typical wave arrival records from a downhole geophysical survey are reproduced in Figure B-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 B-1 were analyzed to estimate wave travel times and velocities for CEG-20, 23, 23A and 24.

B.1.4 Discussion of Results

Estimated velocity structures are summarized in Table B-1. Velocity estimates are based on selection of linear portions of the downhole arrival time curves (see Figures B-2 through B-5).

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 B-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, the near-surface shear wave velocity was found to be approximately 1200 feet per second. Shear wave velocity estimates at Boring CEG-20 showed an increase with depth to 1180± feet per second. However, at Boring CEG-24, the shear wave velocity decreased from 2570± feet per second between depths of 135 and 175 feet to 1330± feet per second between depths of 175 and 195 feet.

B.2 CROSSHOLE SURVEY

B.2.1 Summary

Crosshole measurements for the determination of seismic wave velocities were performed in Borings CEG-20 and CEG-24 for adjacent Design Units A250 and A310 although none was carried out in the immediate vicinity of Fairfax/Beverly Station. These surveys, although located some distance from the Station site and not part of Design Unit A275, are included in this report due to a lack of data and as soil and bedrock conditions do not vary considerably in this area. The crosshole technique for determining shear wave velocities of in-situ materials was utilized in a three-borehole array. The array consisted of the alignment boring 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 B-2.

B.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. Seismic wave velocity determinations were made at 5-foot intervals from 10 feet below ground surface to a depth of 100 feet (see Figures B-6 through B-9).

B.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.

B.2.4 Discussion of Results

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 B-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 B-1
DOWNHOLE VELOCITIES

BORING No.	DEPTH (ft)	COMPRESSIONAL WAVE					SHEAR WAVE				
		$\bar{V}p$	σp	Ep	Np	Vp*	$\bar{V}s$	σs	Es	Ns	Vs*
20	20- 50	3515	284	176	6	3520±460	1021	209	51	11	1020±260
	50- 75	4849	555	242	26	4849±800	1021	209	51	11	1020±260
	75-190	4849	555	242	26	4849±800	1176	48	59	23	1180±110
23	10-200	4134	323	207	33	4130±530	1828	34	600	4	1830±630
23A	10-188	6103	359	305	37	6110±660	1151	20	56	37	1150±80
24	10-135	2586	277	129	36	2590±410	305	32	65	25	1305±100
	135-175	2938	---	---	11	2940±1500	2569	595	128	9	2570±720
	175-195	2938	---	---	11	2940±1500	1333	97	67	5	1330±160

$\bar{V}p$ = 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.

TABLE B-2
CROSSHOLE VELOCITIES

BORING No.	DEPTH (ft)	COMPRESSIONAL WAVE					SHEAR WAVE				
		$\bar{V}p$	σp	Ep	Np	Vp*	$\bar{V}s$	σs	Es	Ns	Vs*
20	45	4540		450	1	4540±450	1502	11	75	10	1500±90
	50	4297	0	215	6	4300±215	1200	39	65	8	1300±100
	55	3533	167	177	6	3530±340	1266	15	63	11	1270±80
	60	3720	256	186	5	3720±442	1178	16	59	6	1180±80
	65	4404		440	2	4400±440	1087	13	54	6	1090±70
	70	4495	391	225	4	4500±620	1211	25	61	11	1210±90
	75	4209	0	210	4	4210±210	1160	11	58	7	1160±70
	85	4805	169	240	7	4810±410	1177	11	59	9	1180±70
	90	4833	294	242	9	4830±540	1289	53	64	10	1290±120
	95	4877	0	244	2	4880±240	1239	31	62	8	1240±90
	97	4725		470	1	4730±470	1236	37	62	7	1240±100
24	10	2400	98	120	2	2400±220	1272	72	64	8	1270±140
	15	2310	0	115	3	2310±120	1251	39	63	8	1250±100
	20	2288	263	114	4	2290±380	1187	32	59	8	1190±90
	25	----				--------	1413	28	71	12	
	30	2216	13	111	4	2220±120	1276	67	64	8	
	35	2400	0	120	2	2400±120	1352	4	68	12	
	40	----				--------	1273	5	64	8	
	45	2152		220	3	2150±220	1253	41	63	12	
	50	----				--------	1262	10	63	12	
	55						1332	8	67	12	
	60						1295	12	65	12	
	65	2356	103	118		2360±220	1552	43	78	12	
	70	2530	482	127	4	2530±610	1790	36	90	12	
	75	2438	45	122	5	2440±170	1808	47	90	11	
	80	2549	210	127	3	2550±340	1552	43	76	12	
	85	2591	511	130	3	2590±700	1350	78	67	8	
	90						1445	169	72	8	
	95						1725	87	61	10	
	97	2320	270	116	2	2320±340	1267	42	63	10	

$\bar{V}p$ = 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.

B.3 SEISMIC REFRACTION SURVEY

B.3.1 Summary

Six seismic refraction lines were recorded in the vicinity of Fairfax High School during the months of February and March, 1981 at the locations shown on Figure B-13. Although Fairfax High School is located approximately 2400 feet north of the Fairfax/Beverly Station site, the results are included in this report to supplement data obtained for Design Unit A275. The purpose of these lines was to delineate the alluvium/bedrock interface to evaluate evidence for offset along the Santa Monica fault and to supplement information from the exploratory borings.

Seismic readings were recorded in both forward and reverse directions along all lines. Profiles showing subsurface velocity zones were constructed from interpretations of the data, and are presented in Figures B-10 through B-12.

A map showing the locations of the seismic refraction lines is presented on Figure B-13 of this Appendix.

Interpreted results suggest a gently sloping bedrock surface ranging in depth from 105 feet towards the east to 190 feet towards the west. The interpreted ground water table also slopes downward from east to west, ranging from 18 to 42 feet in depth. A few step anomalies were noted in the ground water table (probably associated with interfingering of clay and sand deposits) and one small step anomaly was noted in the alluvium/bedrock interface.

B.3.2 Detailed Description

Seismic refraction Lines S-45 through S-47 were recorded end to end from the southwest corner to the northeast corner of the Fairfax High schoolyard. Lines S-48 and S-49 were overlapped across the yard from the southeast to the northwest corner, approximately at a right angle to Lines S-45 through S-47. Line S-50 was also located at a right angle to and crossed the path of Line S-46.

As shown on the subsurface velocity profiles of Figures B-10, B-11 and B-12, the area beneath Lines S-45 through S-50 is underlain by low velocity material (930 to 1,090 ft/sec) to depths of 3 to 8 feet beneath the ground surface. This low velocity zone is underlain by low to medium velocity material (2,030 to 2,500 ft/sec) to depths of 17 to 42 feet where medium velocity material (4,820 to 5,500 ft/sec) is encountered. The medium velocity zone extends to depths of 105 to 190 feet beneath the ground surface and is underlain by high velocity material (7,380 to 10,120 ft/sec) at depth.

The near-surface low velocity zone is interpreted to represent unconsolidated alluvial deposits and fill. The low to medium velocity zone represents more consolidated alluvial deposits, and the medium velocity zone represents saturated alluvial deposits. The high velocity zone at depth is interpreted to represent the dense, San Pedro sand unit. A small vertical step anomaly was observed in the saturation interface and possibly in the alluvium/bedrock velocity interface beneath Line S-48.

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TRACE IDENTIFICATION

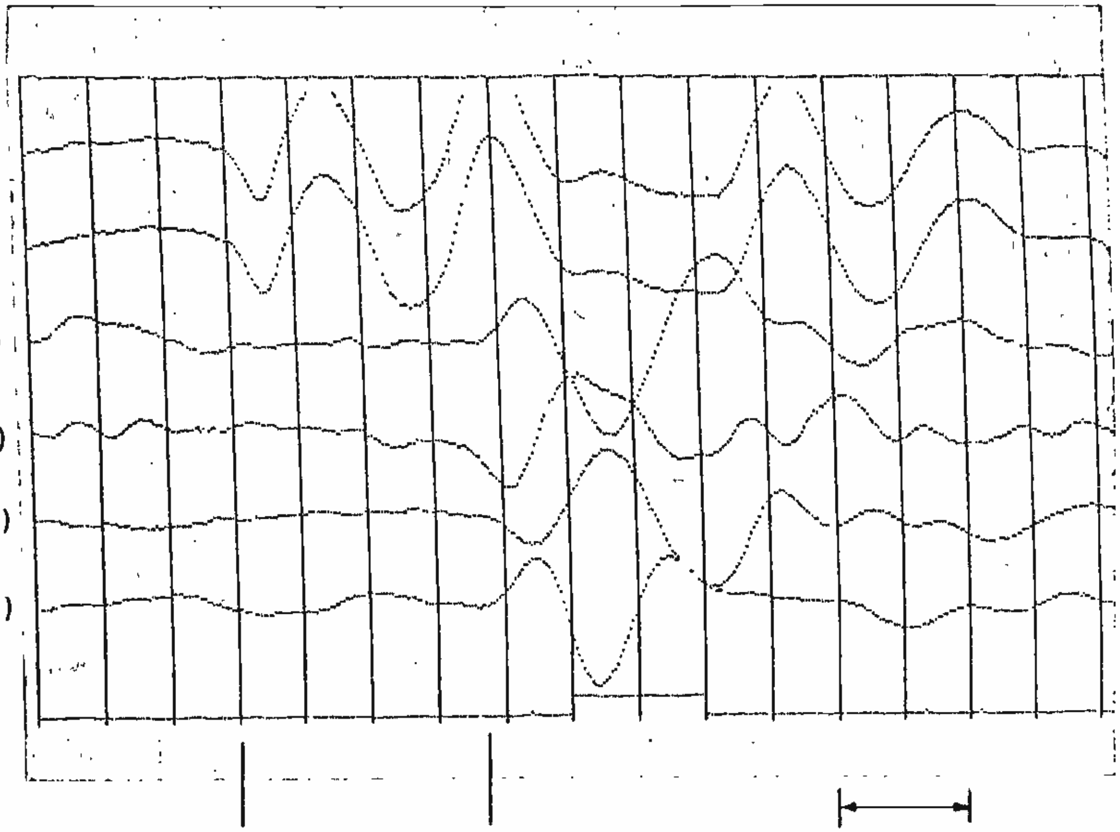
VERTICAL (DOWN)

HORIZONTAL 1 (WEST)

HORIZONTAL 1 (EAST)

HORIZONTAL 2 (WEST)

HORIZONTAL 2 (EAST)



BOREHOLE: 13

DEPTH: 70 FT

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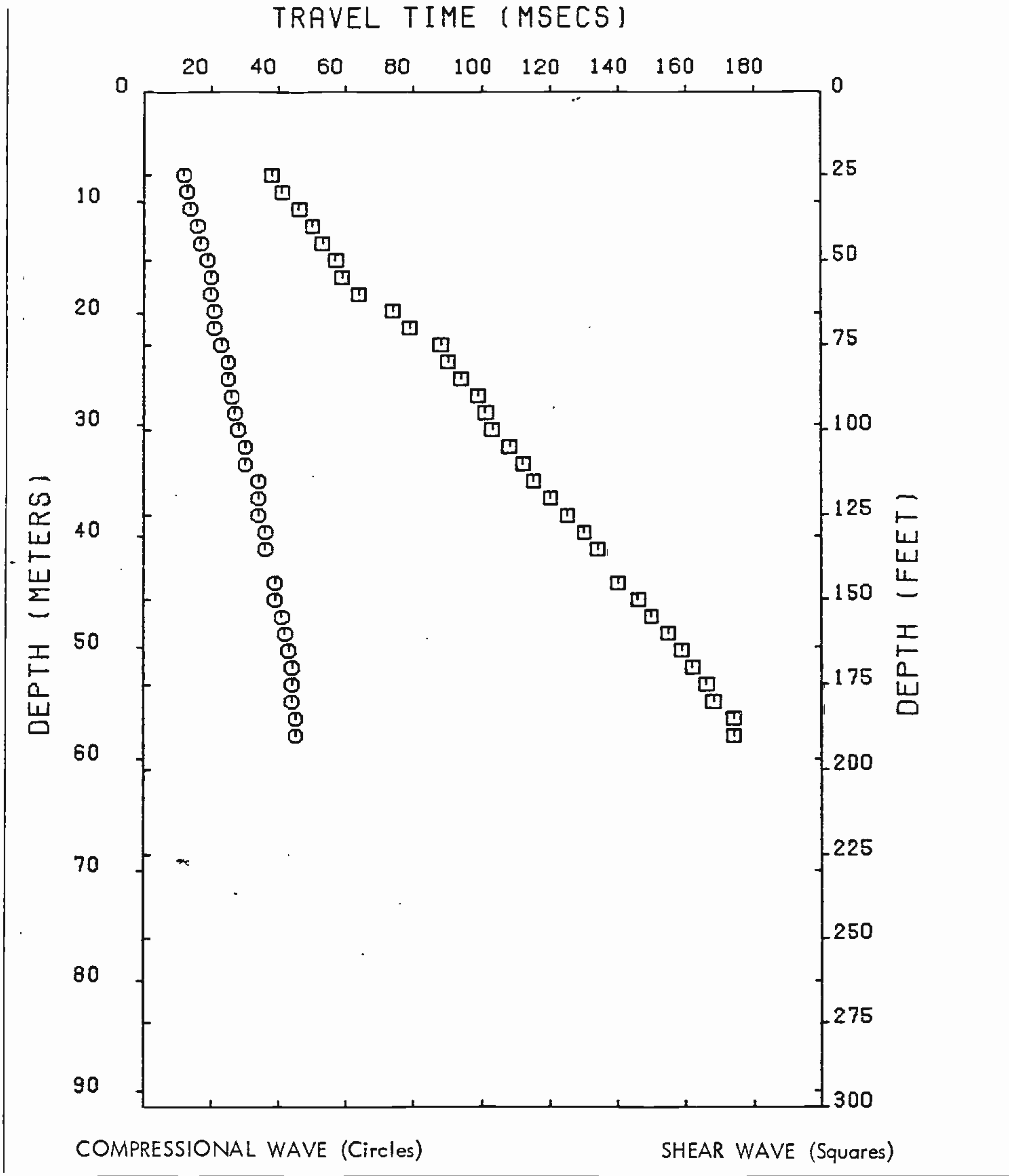
DOWNHOLE SAMPLE RECORD

Project No. 83-1140

Figure No. B-1



DOWNHOLE TRAVEL TIME PROFILE - BORING 20

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Figure No.
B-2

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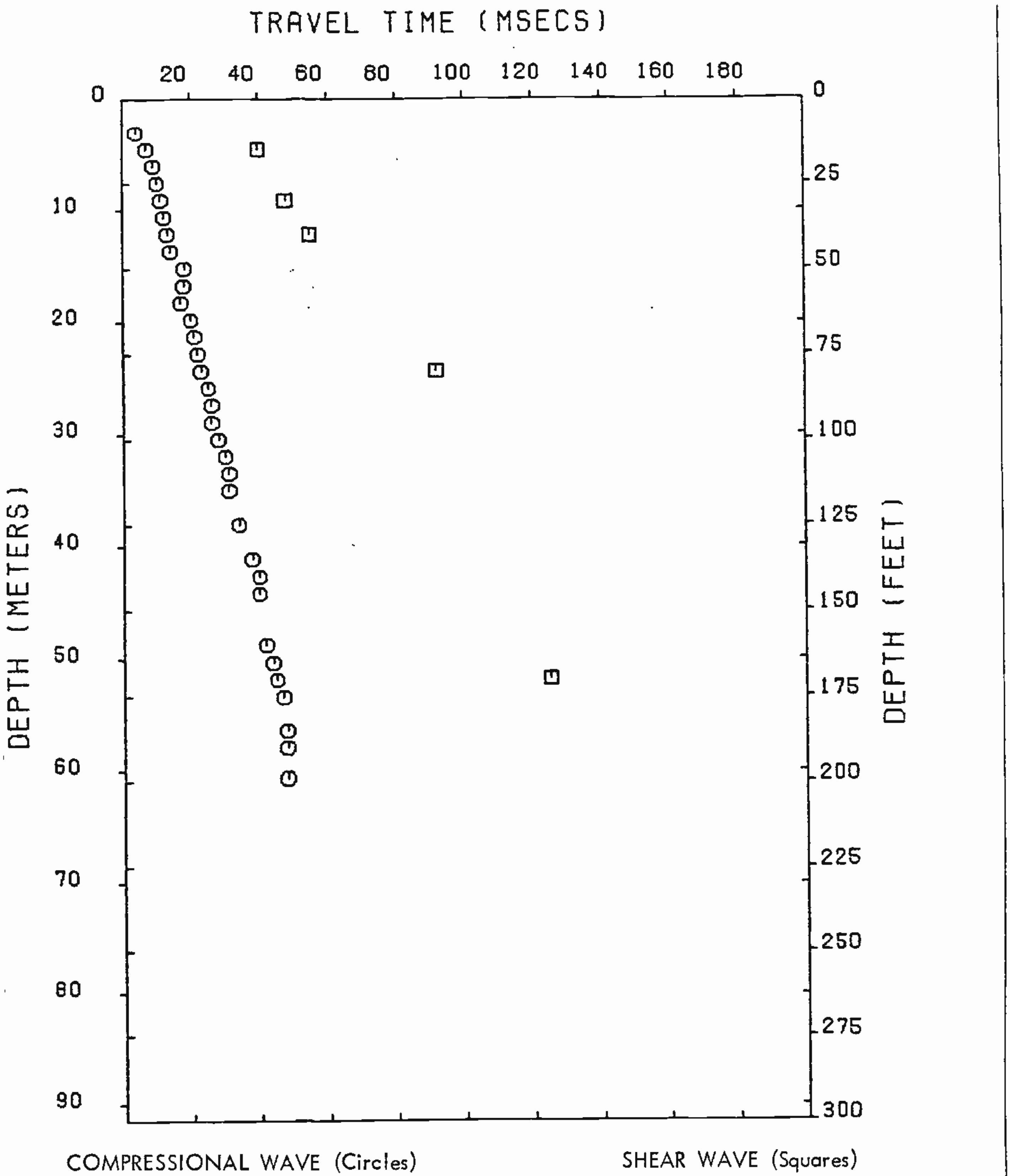

DOWNHOLE TRAVEL TIME PROFILE - BORING 23

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Figure No.
B-3

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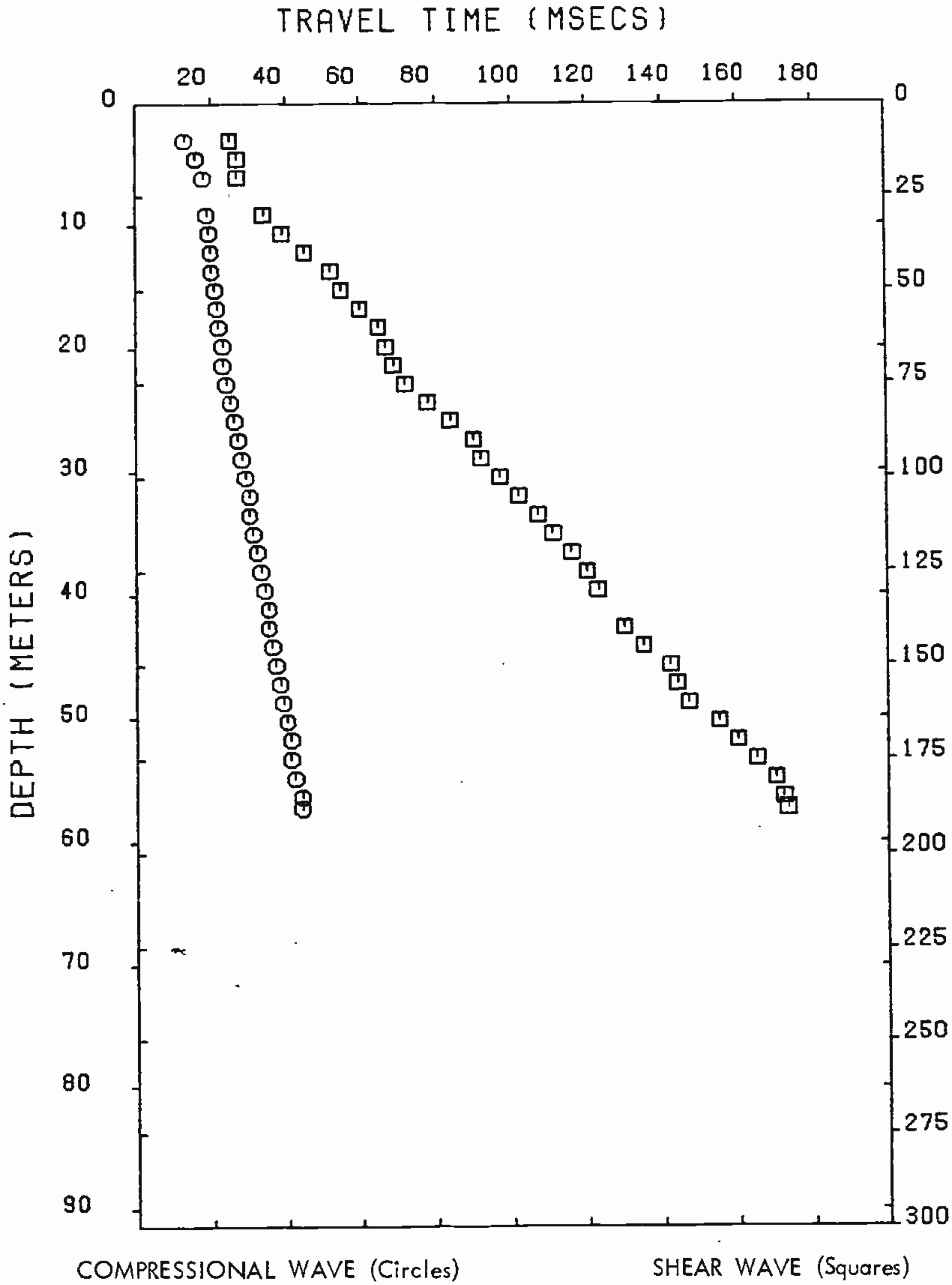

DOWNHOLE TRAVEL TIME PROFILE - BORING 23A

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Figure No. B-4

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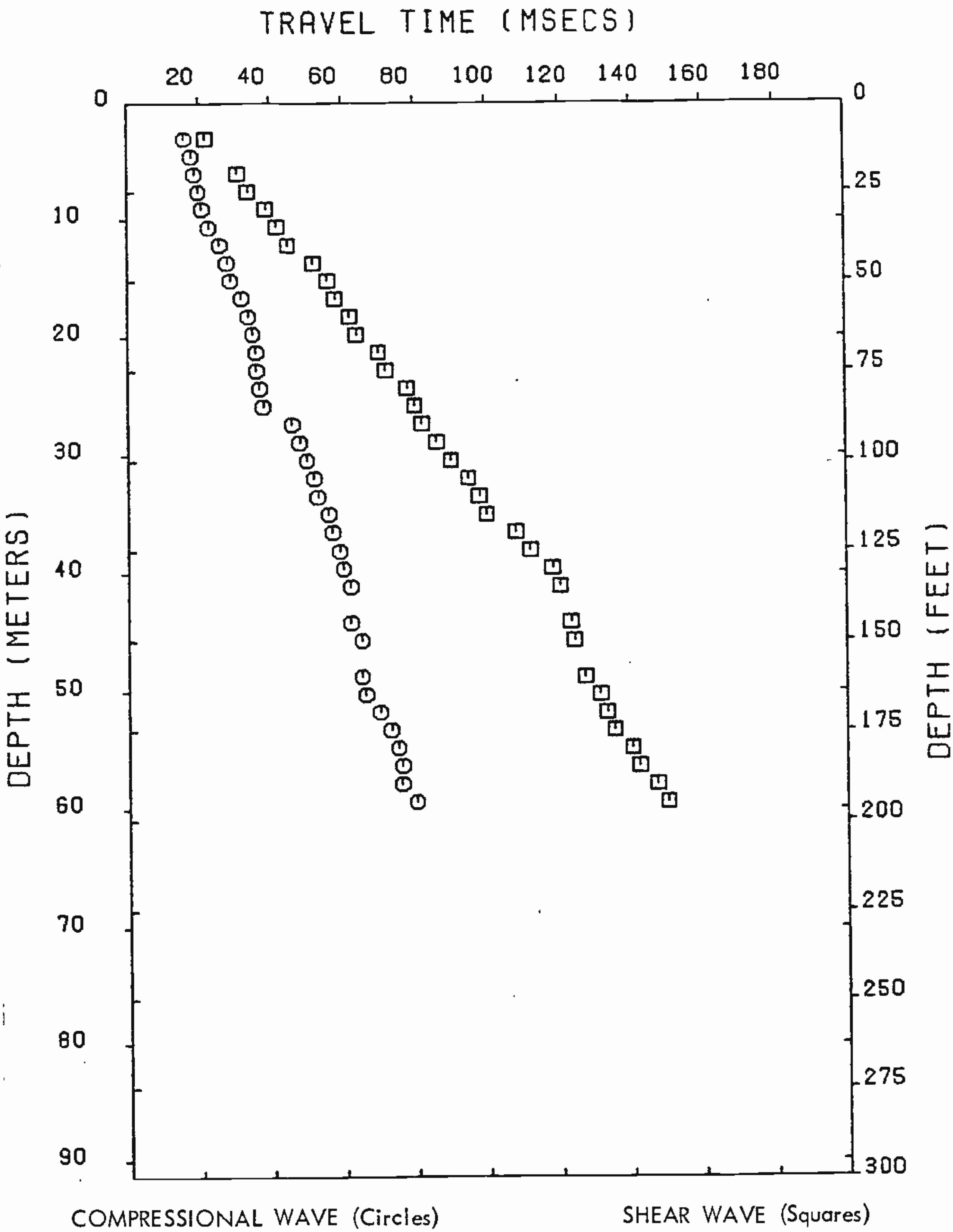

DOWNHOLE TRAVEL TIME PROFILE - BORING 24

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Figure No. B-5

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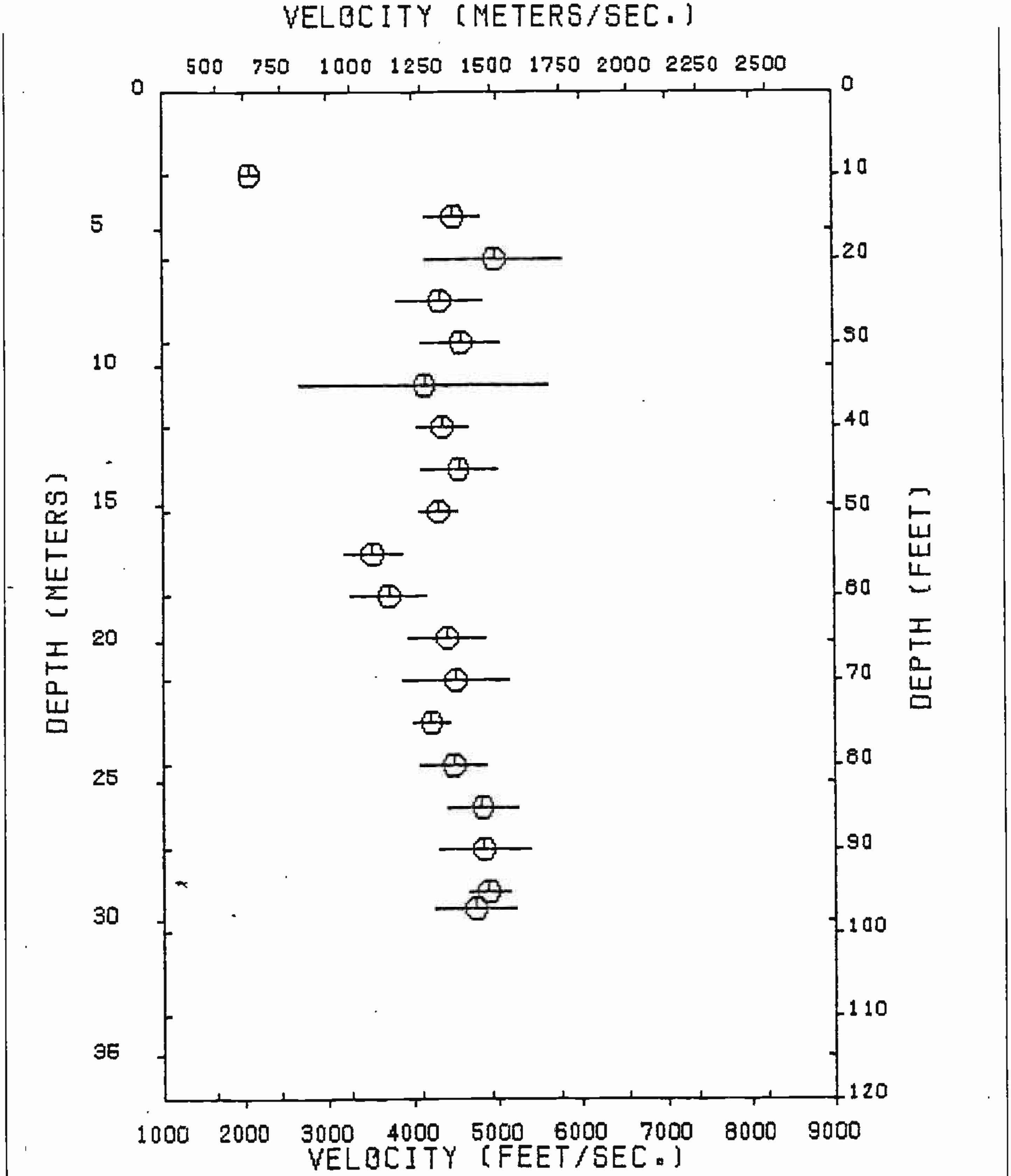

COMPRESIONAL WAVE VELOCITY/DEPTH PROFILE - BORING SITE 20

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Figure No. B-6

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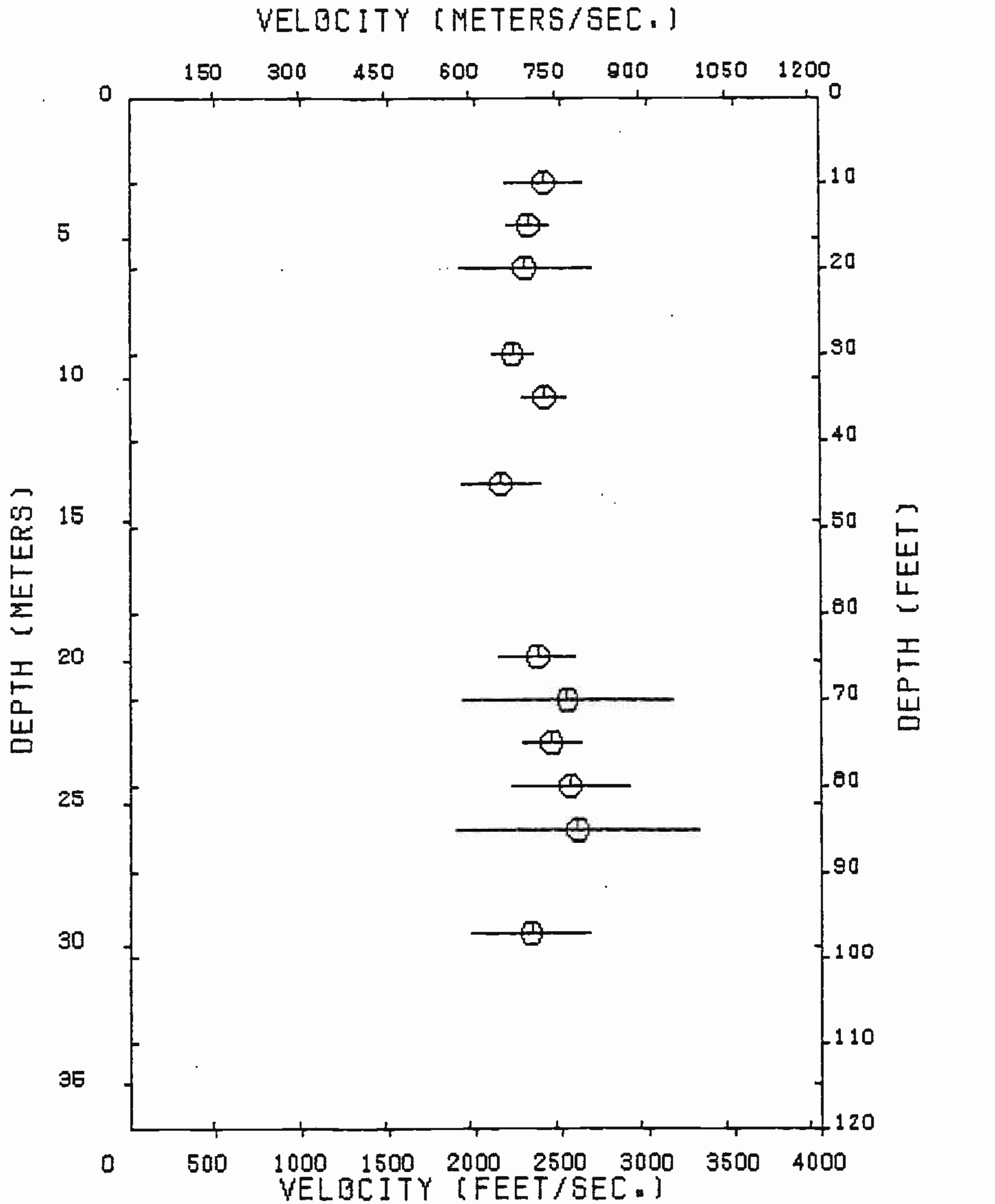

COMPRESSIONAL WAVE VELOCITY/DEPTH PROFILE - BORING SITE 24

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Figure No.
B-7

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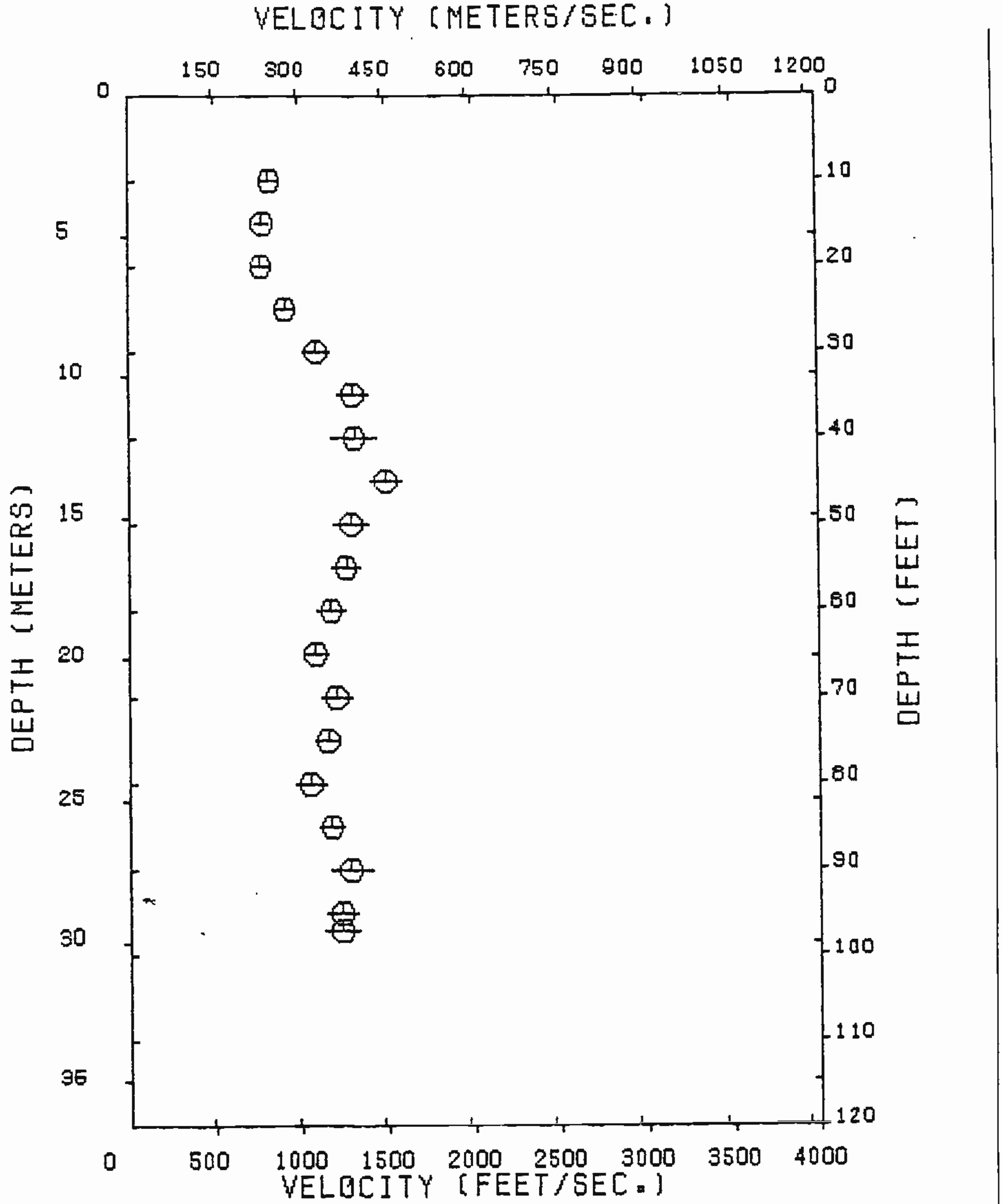

SHEAR WAVE VELOCITY/DEPTH PROFILE - BORING SITE 20

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Figure No. B-8

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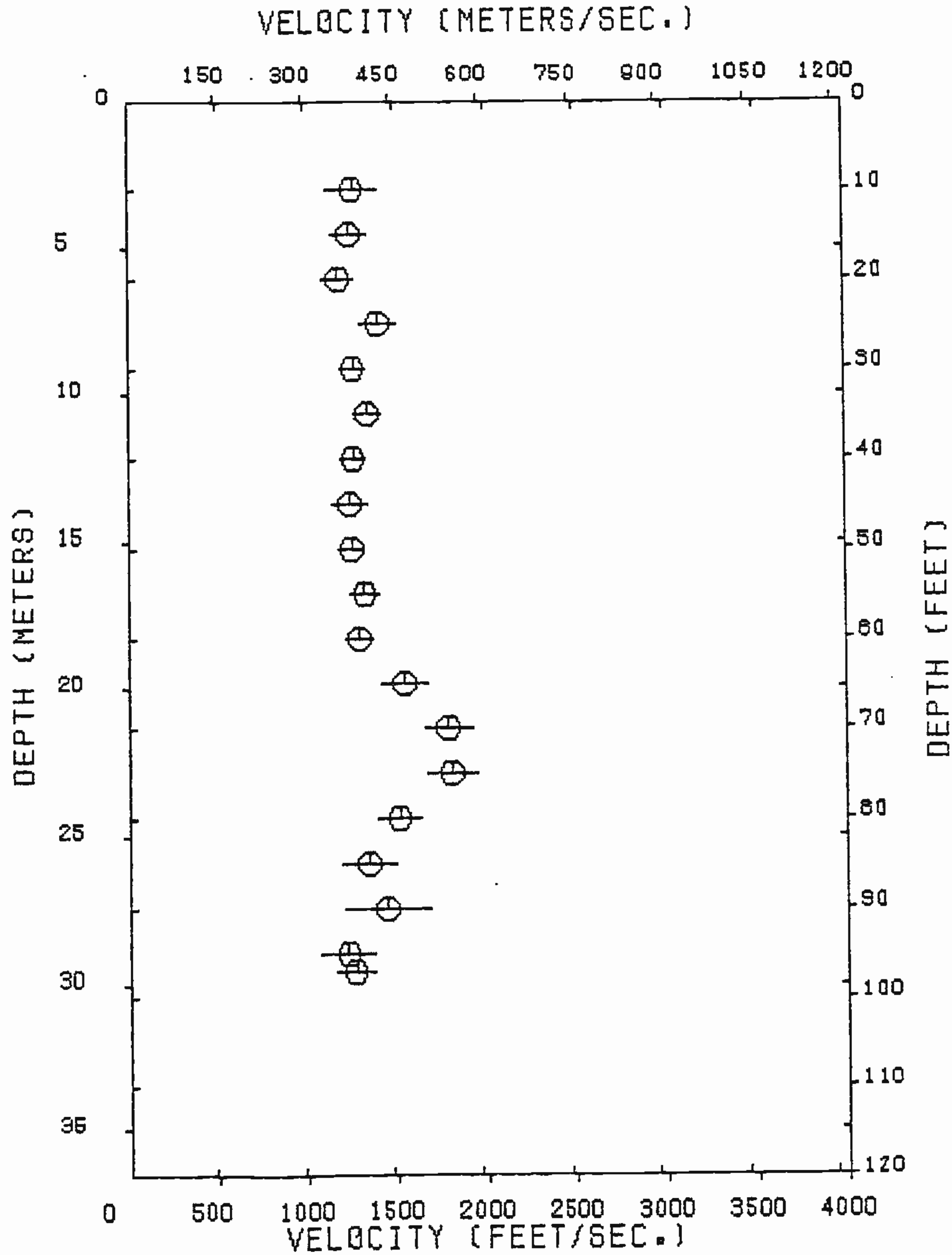

SHEAR WAVE VELOCITY/DEPTH PROFILE - BORING SITE 24

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Figure No. B-9

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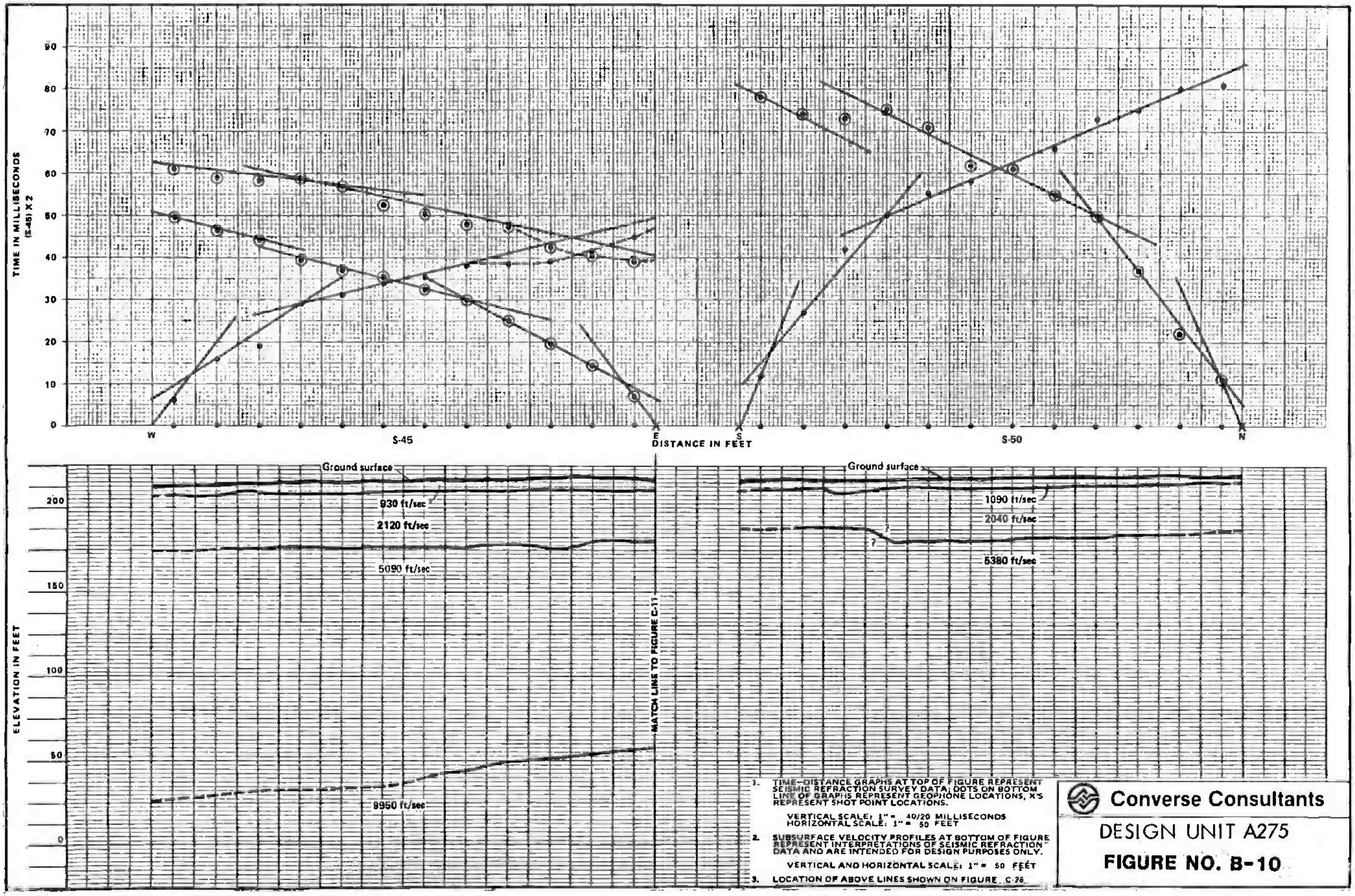

1. TIME–DISTANCE GRAPHS AT TOP OF FIGURE REPRESENT SEISMIC REFRACTION SURVEY DATA; DOTS ON BOTTOM LINE OF GRAPHS REPRESENT GEOPHONE LOCATIONS, X'S REPRESENT SHOT POINT LOCATIONS.

   VERTICAL SCALE: 1" = 40/20 MILLISECONDS
   HORIZONTAL SCALE: 1" = 50 FEET

2. SUBSURFACE VELOCITY PROFILES AT BOTTOM OF FIGURE REPRESENT INTERPRETATIONS OF SEISMIC REFRACTION DATA AND ARE INTENDED FOR DESIGN PURPOSES ONLY.

   VERTICAL AND HORIZONTAL SCALE: 1" = 50 FEET

3. LOCATION OF ABOVE LINES SHOWN ON FIGURE C-26

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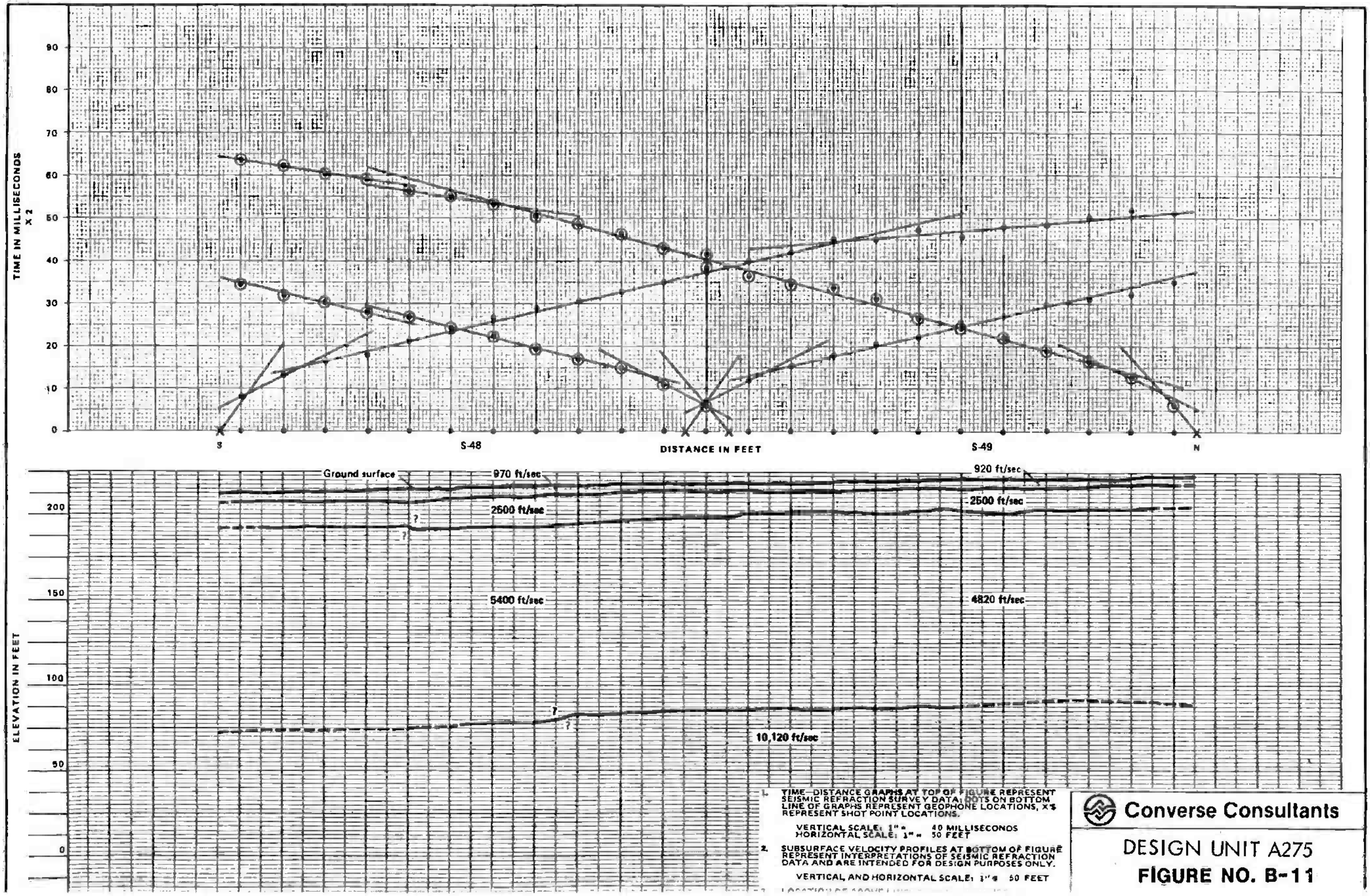
FIGURE NO. B-10



1. TIME—DISTANCE GRAPHS AT TOP OF FIGURE REPRESENT SEISMIC REFRACTION SURVEY DATA; DOTS ON BOTTOM LINE OF GRAPHS REPRESENT GEOPHONE LOCATIONS, X'S REPRESENT SHOT POINT LOCATIONS.

   VERTICAL SCALE: 1" = 40 MILLISECONDS
   HORIZONTAL SCALE: 1" = 50 FEET

2. SUBSURFACE VELOCITY PROFILES AT BOTTOM OF FIGURE REPRESENT INTERPRETATIONS OF SEISMIC REFRACTION DATA AND ARE INTENDED FOR DESIGN PURPOSES ONLY.

   VERTICAL AND HORIZONTAL SCALE: 1" = 50 FEET

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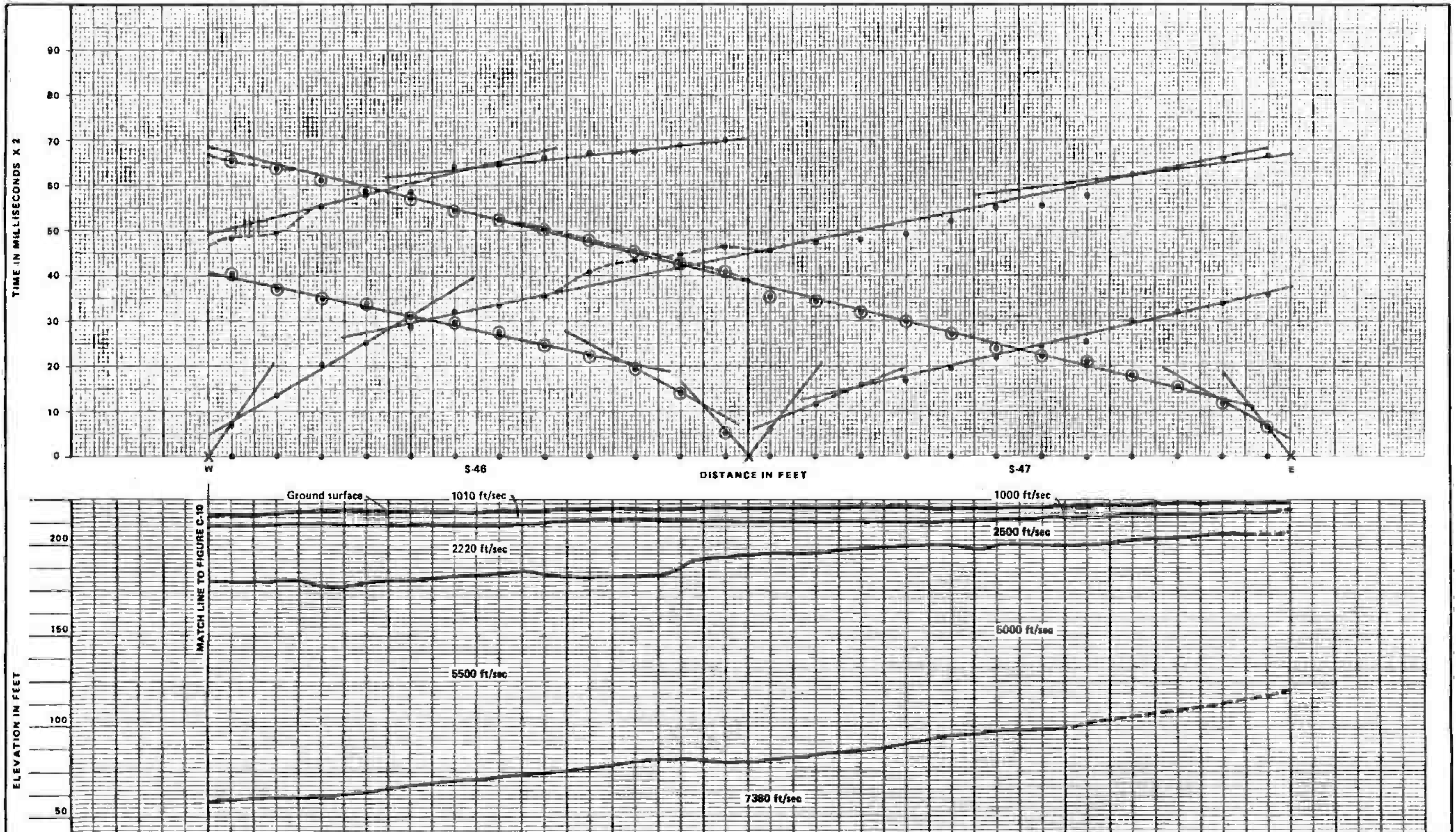
FIGURE NO. B-11



1. TIME—DISTANCE GRAPHS AT TOP OF FIGURE REPRESENT SEISMIC REFRACTION SURVEY DATA; DOTS ON BOTTOM LINE OF GRAPHS REPRESENT GEOPHONE LOCATIONS, X'S REPRESENT SHOT POINT LOCATIONS.

   VERTICAL SCALE: 1" = 40 MILLISECONDS
   HORIZONTAL SCALE: 1" = 50 FEET

2. SUBSURFACE VELOCITY PROFILES AT BOTTOM OF FIGURE REPRESENT INTERPRETATIONS OF SEISMIC REFRACTION DATA AND ARE INTENDED FOR DESIGN PURPOSES ONLY.

   VERTICAL AND HORIZONTAL SCALE: 1" = 50 FEET

3. LOCATION OF ABOVE LINES SHOWN ON FIGURE C-26

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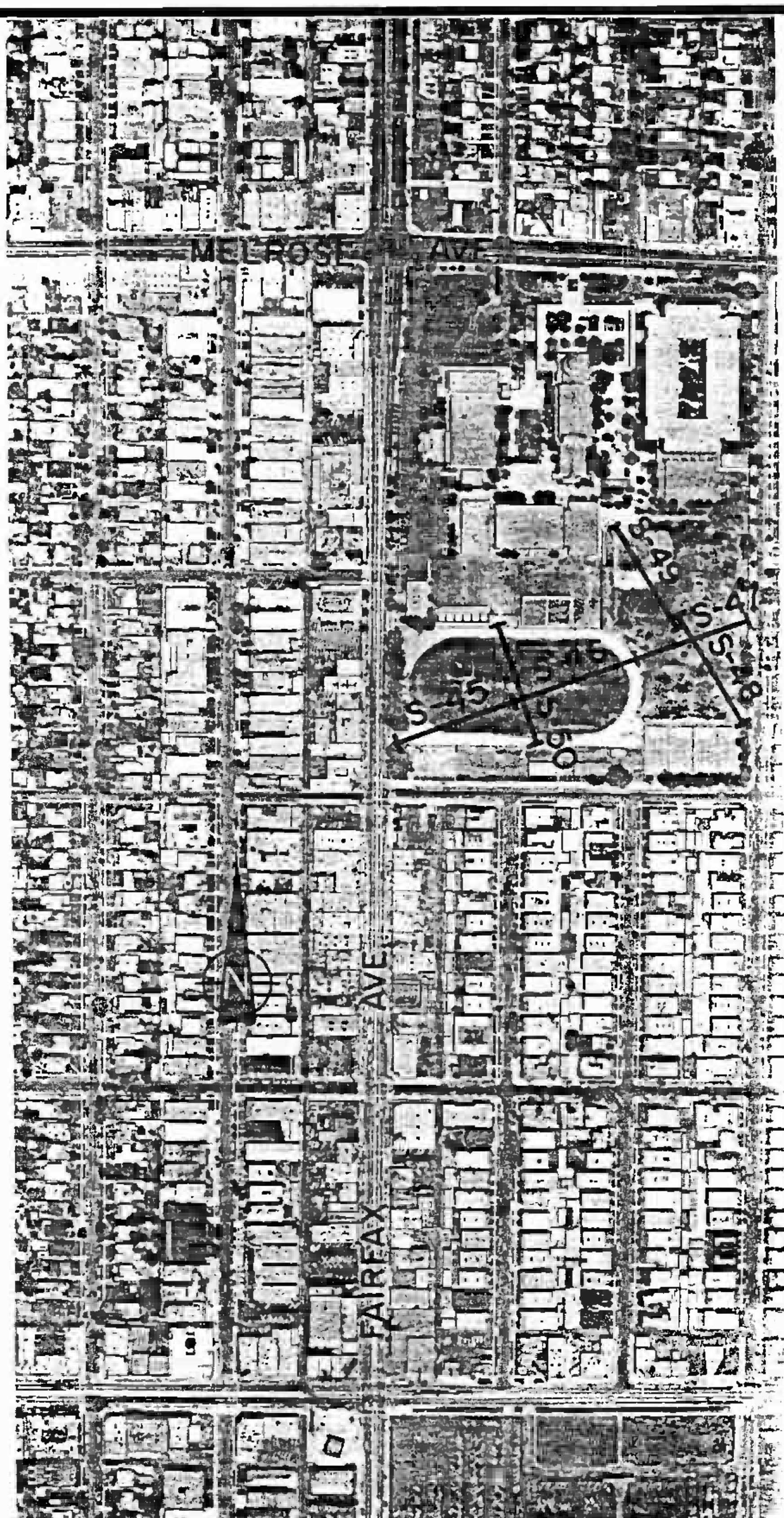
FIGURE NO. B-12



SEISMIC REFRACTION SURVEY - FAIRFAX HIGH SCHOOL

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83-1140

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Figure No.
B-13

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Appendix C

Geotechnical Laboratory Testing

APPENDIX C GEOTECHNICAL LABORATORY TESTING

C.1 INTRODUCTION

This appendix presents laboratory geotechnical tests performed on selected soil and bedrock samples obtained from the borings drilled at or in the vicinity the Fairfax/Beverly site.

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, and dynamic triaxial tests.

The laboratory test data from the present investigation are presented in Table C-1, while data from the 1981 geotechnical investigation are presented in Table C-2. The geologic units listed in these tables are described in Section 5.0 of the report. Figures C-1 through C-4 summarize strength and modulus data for alluvium, granular alluvium at this site.

C.2 INDEX AND IDENTIFICATION

C.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.

C.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 C-5 through C-10.

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.

C.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 C-11 and C-12, and Tables C-1 and C-2.

C.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 C-1 and C-2.

C.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 C.2.4 and the total unit weight. Results of the unit weight tests are presented as dry densities on Tables C-1 and C-2.

C.3 ENGINEERING PROPERTIES: STATIC

C.3.1 Unconfined Compression

Unconfined compression tests were performed on selected samples of alluvium and tar sand from the test borings for the purpose of evaluating the undrained, unconfined shear strength of the various 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 C-1 and C-2.

C.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:

C.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. 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.

Results of the triaxial compression tests are presented on Figures C-13 through C-17.

C.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 C-1 and C-2.

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 C-1 and C-2.

C.3.4 Consolidation

Consolidation tests were performed on selected undisturbed soil samples placed in 1 inch high by 2.42-inch diameter brass rings, or 3-inch diameter Shelby tubes trimmed to a 2.42-inch diameter.

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 C-18 through C-23.

C.3.5 Permeability

Permeability tests were performed on undisturbed specimens selected for testing, or in conjunction with the static 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 C-1 and C-2.

C.4 ENGINEERING PROPERTIES: DYNAMIC

C.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.

C.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.

C.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 frequency of 0.5Hz was used for this test program.

C.4.1.3 Apparatus: The apparatus described below 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.

The pneumatic loading system used for these tests was custom-designed and built for Converse Consultants. The device consists of the four main component groups described below.

- Triaxial Chambers and Cyclic Loading Device: The triaxial chambers are comprised of stainless steel and aluminum cells designed for operating pressures up to 400 psi. (Pressures of up to 160 psi were used for this project.) A pneumatic, double-acting piston, capable of applying both static and cyclic loads, is mounted above the triaxial chamber and connected to the specimen load cap by a low-inertia stainless steel rod. The rod passes through the top of the chamber and is held in place by low friction bushings and pressure seals.

- Control Console: This unit contains the various pressure regulators and reservoir systems for controlling cell pressure, back pressures, and sample saturation and drainage. The controls on the console regulate the wave form, frequency, and magnitude of the static and cyclic axial loads.

- Transducer System and Signal Conditioners: The electronic transducers produce electrical voltages in proportion to the key parameters being measured during the test. Parameters monitored and transducer type employed for this program are:

PARAMETER MONITORED	TRANSDUCER TYPE
Axial displacement	- Linear variable differential transformers (LVDT's) mounted internally to the specimen load caps
Soil pore water pressure	- Unbonded wire resistance strain-gauge-type transducers mounted external to the chamber on sample drainage lines
Axial load	- Bonded resistance strain-gauge-type load cell mounted between double-acting piston and rod connected to specimen load cap

Signal conditioners such as power supplies and variable gain amplifiers are used to excite the transducers and amplify the signals to recordable levels.

° Recording Devices: These include (a) a 4-channel continuous strip chart recorder, thermal pens and heat-sensitive paper, frequency response adequate for frequencies normally employed in cyclic triaxial testing, and (b) a cathode ray oscilloscope.

C.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.

The Dynamic Triaxial test results are shown on Figures C-24 through C-27.

TABLE C-1 LABORATORY TEST DATA

BORING No.	SAMPLE No.	DEPTH (ft)	VISUAL CLASSIFICATION	DRY DENSITY (pcf)	MOISTURE CONTENT (%)	ATTERBERG LIMITS LL	PI	Kv, COEFFICIENT OF PERMEABILITY (cm/sec) (Confining Pressure, psi)	UNCONFINED COMPRESSIVE STRENGTH (ksf)	DIRECT SHEAR STRENGTH ENVELOPE ϕ, deg	c, ksf	ONE-DIMENSIONAL SWELL (% (Normal Load, ksf)	SWELL PRESSURE (ksf)	SIEVE ANALYSIS	HYDROMETER ANALYSIS	OEDOMETER	TRIAXIAL COMPRESSION (Stages)
23-1	1	4	Silty Clay	99	24												
	2	14	Silty Clay	91	31	67	41			13	1.25						
	3	19	Sandy Clay	102	24												
	4	24	Sandy Clay	96	28					24	0.85			X			
	5	34	Sandy Silt	101	24					27	0.75						
	6	39	Silty Clay	95	29									X			
	7	44	Silty Clay	97	30	54	31										
	8	54	Sandy Silt	86	39												
	9	59	Sandy Clay	100	25			2.3×10^{-6}								X	
	10	64	Clayey Sand	93	30									X			X(2)
	11	74	Silty Clay	91	31					39	0.62					X	
23-2	1	3	Sandy Clay	101	20				6.1								
	2	8	Sandy Clay	102	24				3.4								
	3	13	Silty Sand	114	18									X			
	4	23	Sandy Silt	111	19					33	0.60						
	5	28	Sandy Clay	100	23												
	6	33	Sandy Silt	68	31					40	0.25						
	7	43	Sandy Silt	101	25									X	X		X(2)
	8	43	Sandy Clay	107	22			2.0×10^{-5}		30	0.45			X			

TABLE C-1 LABORATORY TEST DATA

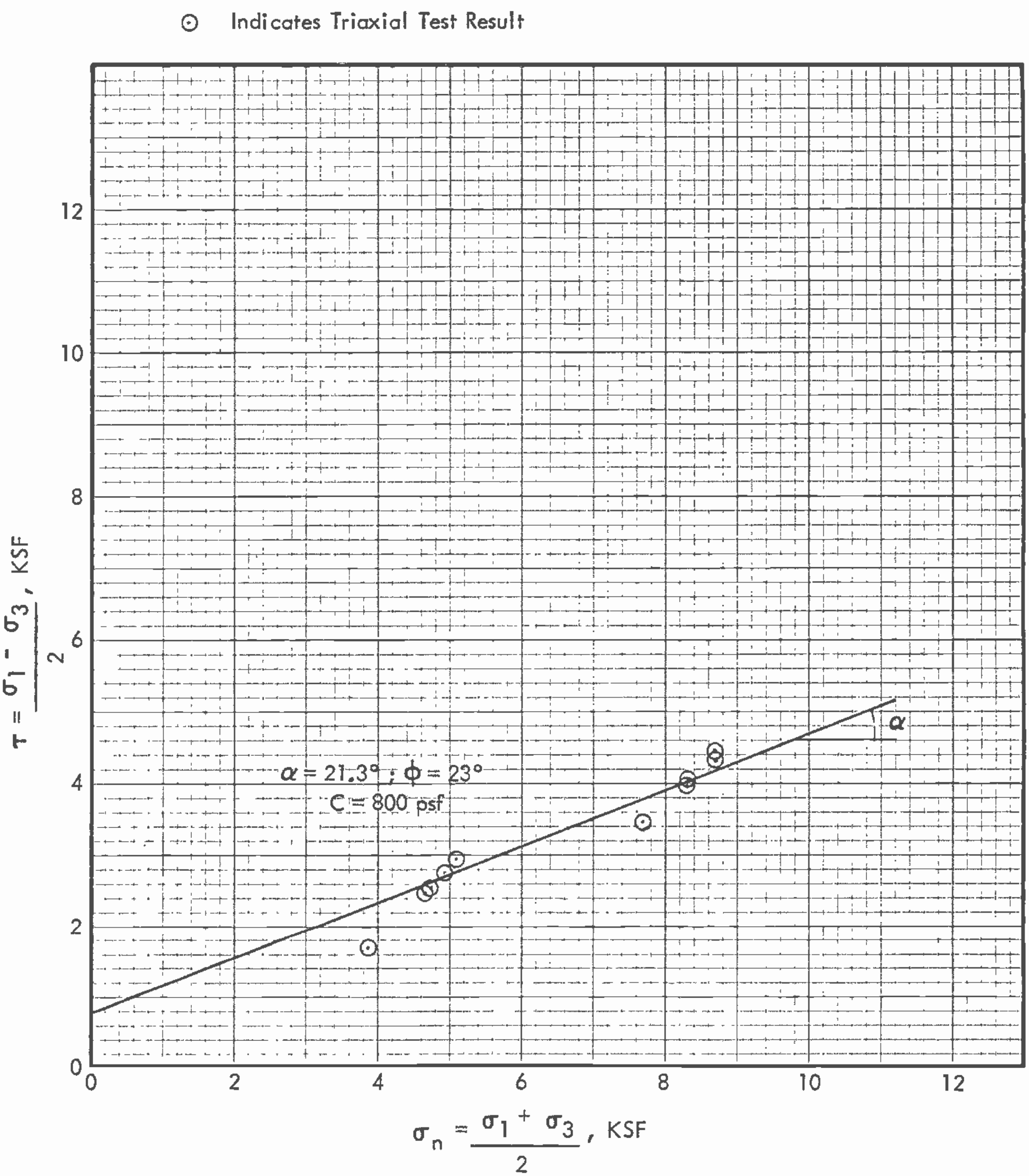
BORING No.	SAMPLE No.	DEPTH (ft)	VISUAL CLASSIFICATION	DRY DENSITY (pcf)	MOISTURE CONTENT (%)	ATTERBERG LIMITS LL	ATTERBERG LIMITS PI	Kv, COEFFICIENT OF PERMEABILITY (cm/sec) (Confining Pressure, psi)	UNCONFINED COMPRESSIVE STRENGTH (ksf)	DIRECT SHEAR STRENGTH ENVELOPE ϕ, deg	DIRECT SHEAR STRENGTH ENVELOPE c, ksf	ONE-DIMENSIONAL SWELL (%) (Normal Load, ksf)	SWELL PRESSURE (ksf)	SIEVE ANALYSIS	HYDROMETER ANALYSIS	OEDOMETER	TRIAXIAL COMPRESSION (Stages)
23-2	9	53	Sandy Clay	94	31											X	
	10	63	Silty Sand	103	19					31	0.35			X			
	11	68	Sandy Clay (tar)	99	21	48	21									X	
	12	73	Sandy Clay (tar)	100	23												
23-3	1	4	Sandy Clay	90	30												
	2	9	Clayey Sand	116	15					30	0.25			X		X	
	3	14	Silty Sand	105	22					31	0.60						
	4	19	Silty Sand	108	14			3.9×10^{-3}		33	0.50						
	5	29	Silty Sand	98	21					24	0.35						
	6	34	Silty Sand	106	20									X			
	7	39	Silty Sand	105	21					28	0.30						
	8	49	Silty Sand	101	22			1.4×10^{-4}						X			
	9	54	Silty Clay	94	26					30	1.00					X	
	10	59	Silt (tar)	110	12												X(2)
	11	69	Silty Clay (tar)	97	22	47	14									X	
	12	74	Silty Clay (tar)	111	17												
23-4	1	3	Sandy Silt	115	7												
	2	8	Sandy Clay	97	28					29	0.67					X	
	3	29	Silty Clay	95	28					15	1.15			X	X		

TABLE C-1 LABORATORY TEST DATA

BORING No.	SAMPLE No.	DEPTH (ft)	VISUAL CLASSIFICATION	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)
23-4	4	39	Silty Sand	98	26			9.0×10^{-5}						X			
	5	44	Silty Sand	97	26					27	0.35						
	6	49	Sand (tar)	110	14												
	7	60	Silty Clay (tar)	100	23	59	32									X	
	8	65	Silty Clay (tar)	109	13												X(2)
	9	69	Silty Clay (tar)	104	16											X	
23-5	1	4	-Disturbed-														
	2	9	Clayey Sand	111	16					39	0.38			X			
	3	14	Clayey Sand	101	23												
	4	19	Sandy Clay	108	19					27	1.0						
	5	24	Sandy Clay	107	20	47	25			33	0.35						
	6	29	Sandy Clay	107	19									X	X		X(2)
	7	34	Sandy Clay	102	24												
	8	45	Silty Sand	107	19			2.7×10^{-4}		33	0.35			X			
	9	51	Sandy Clay	91	30											X	
	10	61	Sandy Clay	111	18					40	0.35						
	11	67	Silty Clay (tar)	89	34	76	51									X	

TABLE C-2 COMPREHENSIVE LIST OF SOILS ENGINEERING PROPERTIES FROM LABORATORY TESTS

BORING No.	SAMPLE No.	DEPTH (ft)	VISUAL CLASSIFICATION	DRY DENSITY (pcf)	MOISTURE CONTENT (%)	ATTERBERG LIMITS LL	ATTERBERG LIMITS PI	PARTICLE SIZE CUMULATIVE % PASSING SIEVE No. 4	40	200	UNCONFINED COMPRESSION STRENGTH (psi)	Kv, COEFFICIENT OF PERMEABILITY (cm/sec)	COEFFICIENT OF CONSOLIDATION C_c (in/in per Log Cycle)	SPECIFIC GRAVITY	POROSITY (n)	UNDRAINED QUICK DIRECT SHEAR ϕ, deg	c, ksf	ONE-DIMENSIONAL SWELL (%) (Normal Load, ksf)	CYCLIC TRIAXIAL (Liquefaction)	DYNAMIC TRIAXIAL (Stress/Strain)	RESONANT COLUMN	TRIAXIAL COMPRESSION
CEG-23	C2	20	Sandy Clay	107	20									2.70	36.3	35.0	1.30					
	C3	30	Sandy Clay			35	14															
	C4	40	Silty Clay	95	31															X		
	C5	50	Clayey Sand	99	25	47	22															Q
	C6	55	Sandy Clay	88	33											30.5	1.44					
	C6	56	Sandy Clay	86	34															X		
	C7	65	Sandy Clay	95	28											35.0	1.20					
	C8	85	Sandy Clay	110	16											29.0	2.64					
	C9	104	Tar Sand	72	3					1.2												



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SUMMARY OF TOTAL STRESS DATA - ALLUVIUM

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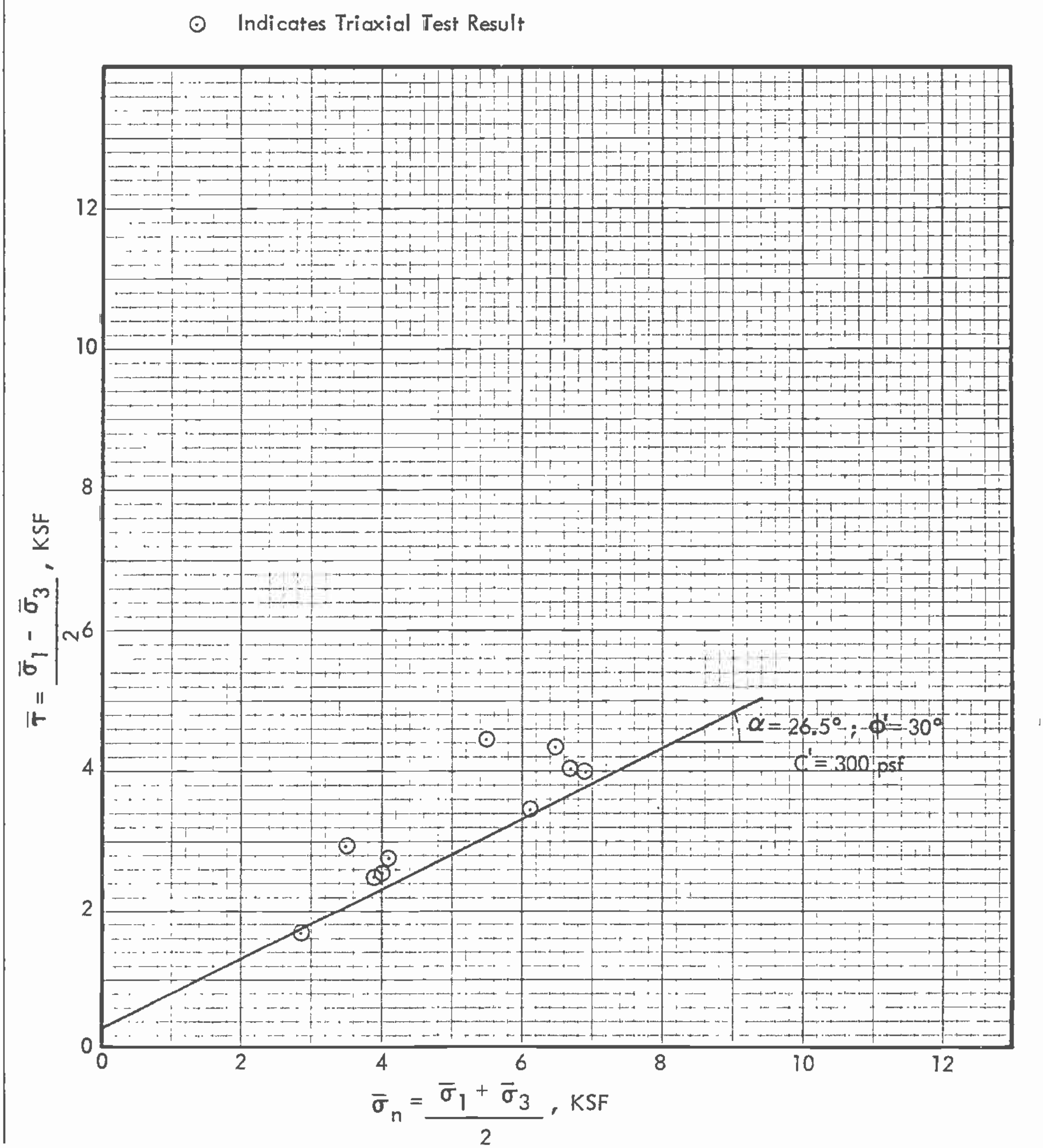
Figure No. C-1



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SUMMARY OF EFFECTIVE STRENGTH DATA - ALLUVIUM

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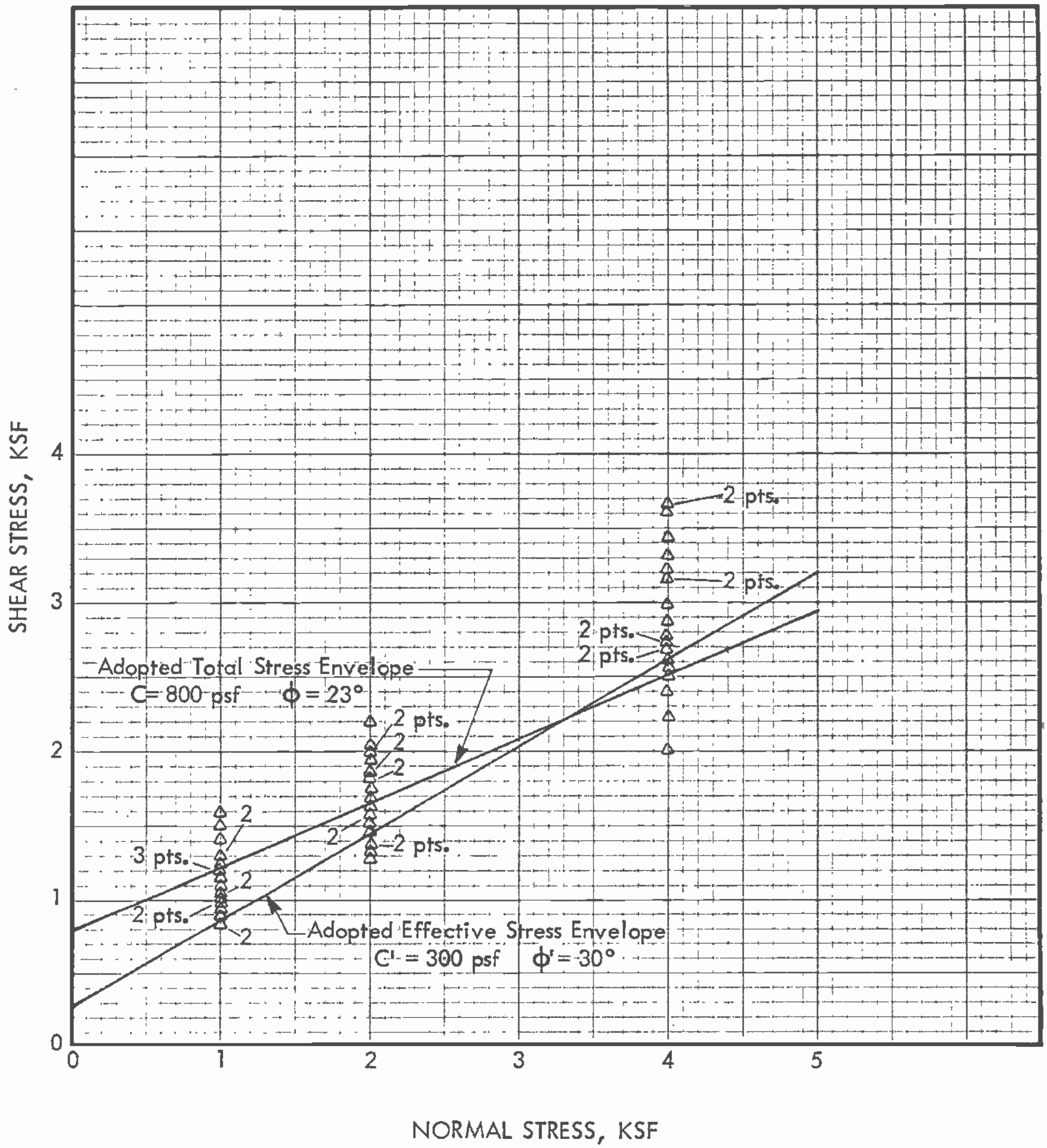
Figure No. C-2



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SUMMARY OF DIRECT SHEAR DATA - ALLUVIUM

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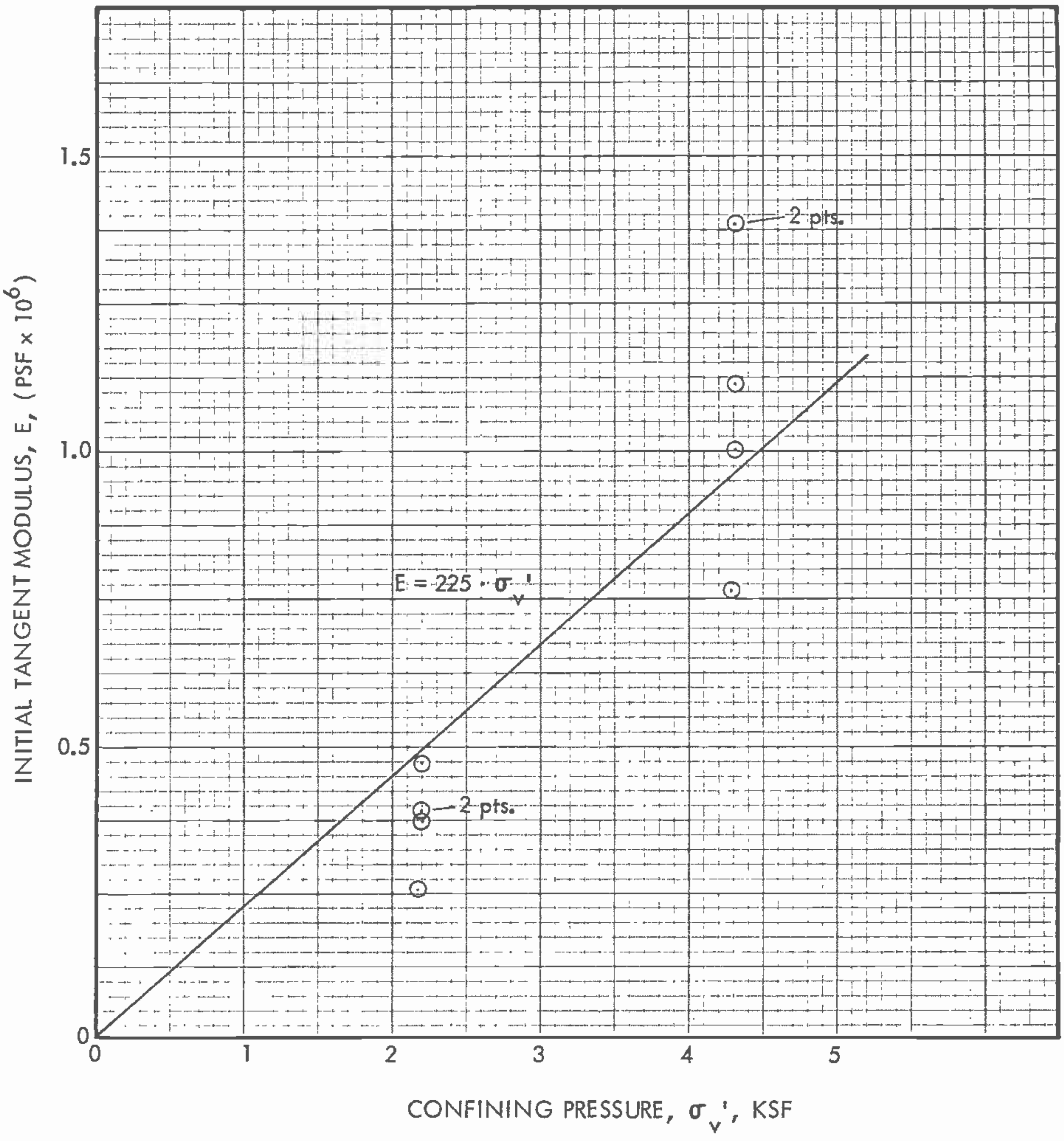
Figure No. C-3



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SUMMARY OF MODULUS DATA - ALLUVIUM

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Figure No. C-4

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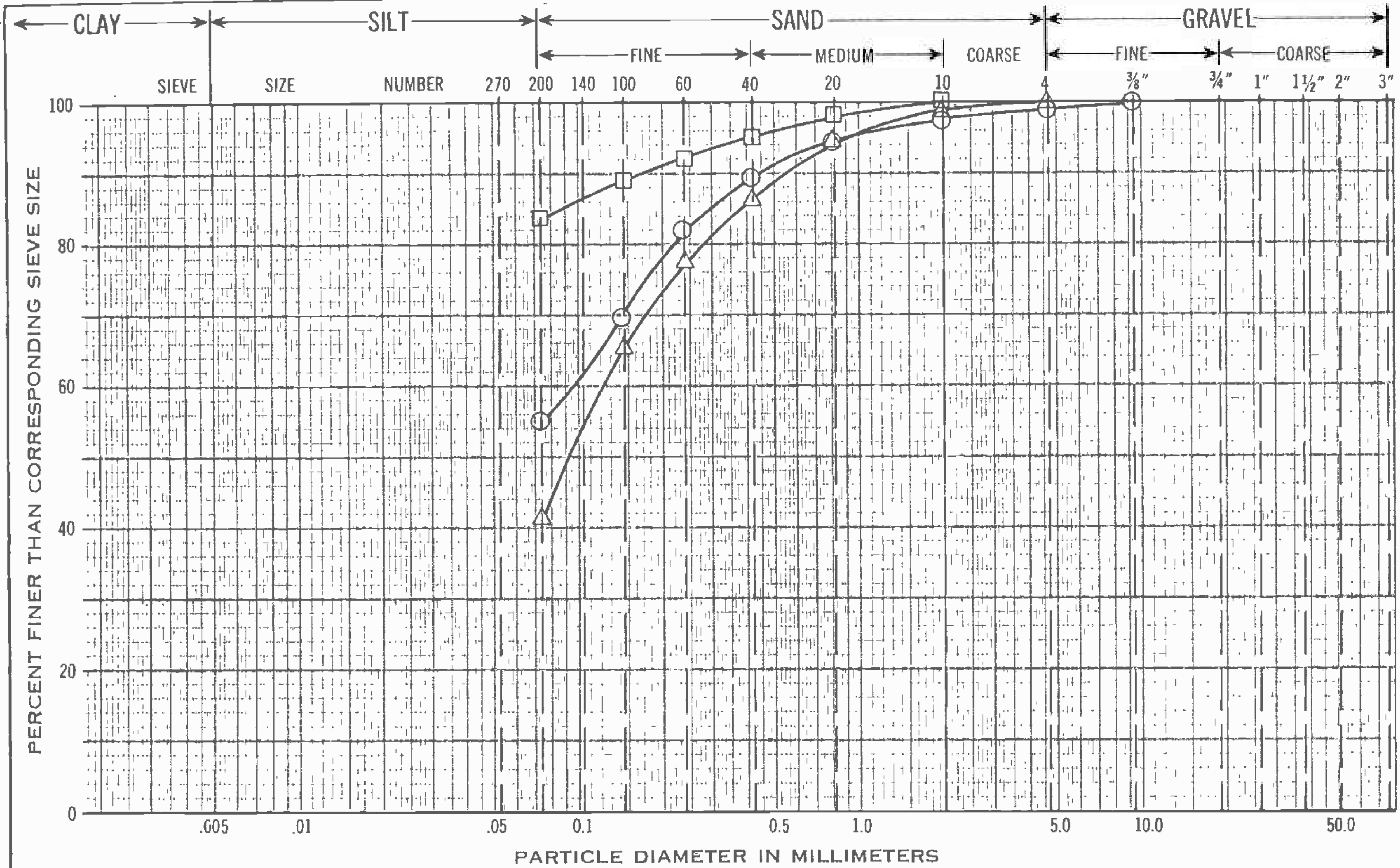

SYMBOL	BORING	SAMPLE	DEPTH
○	23/1	C-4	24'
□	23/1	C-6	39'
△	23/1	C-10	64'

GRAIN-SIZE DISTRIBUTION CHART

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Figure No.
C-5

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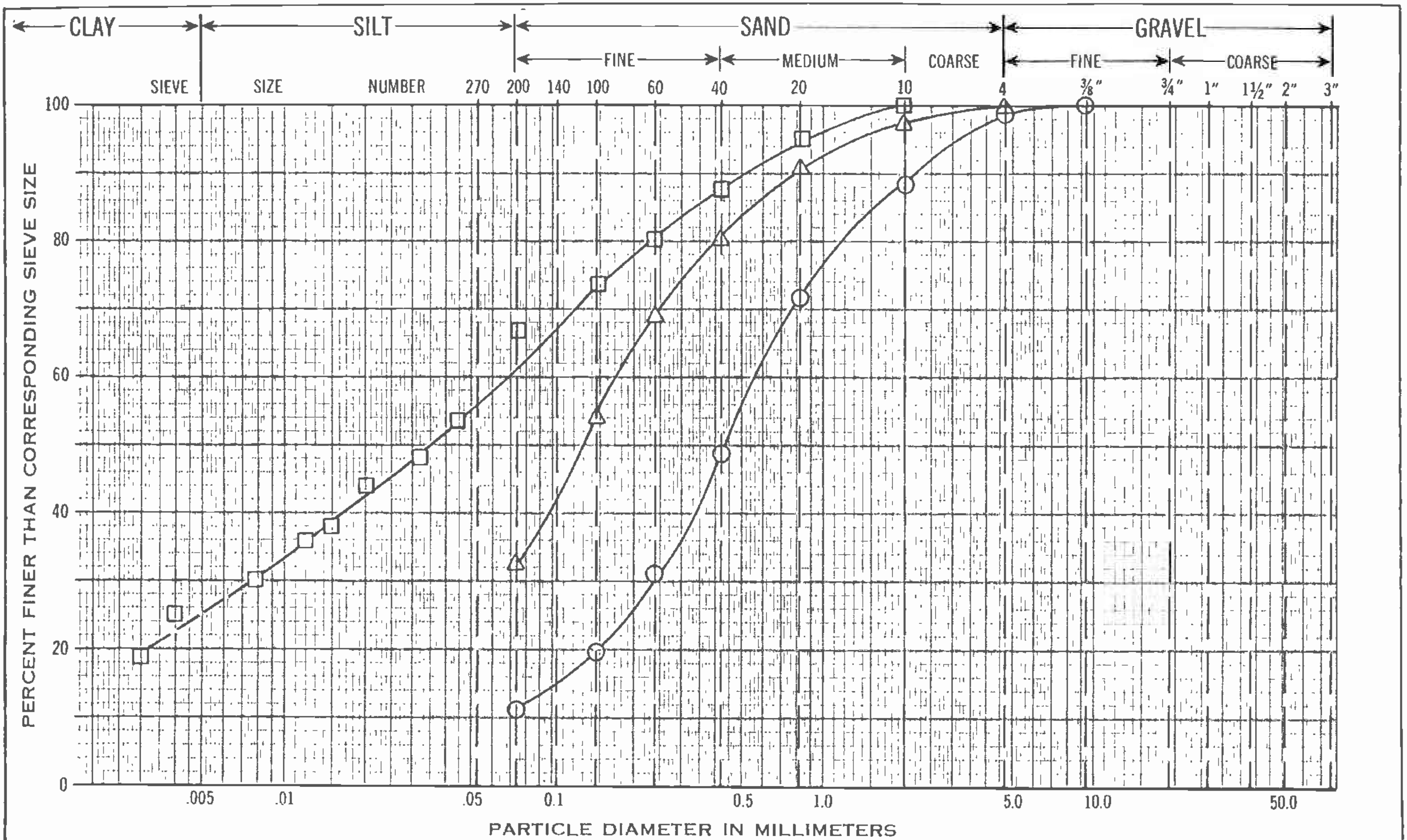

SYMBOL	BORING	SAMPLE	DEPTH
○	23/2	C-3	13'
□	23/2	C-7	42'
△	23/2	C-10	63'

GRAIN-SIZE DISTRIBUTION CHART

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Figure No C-6

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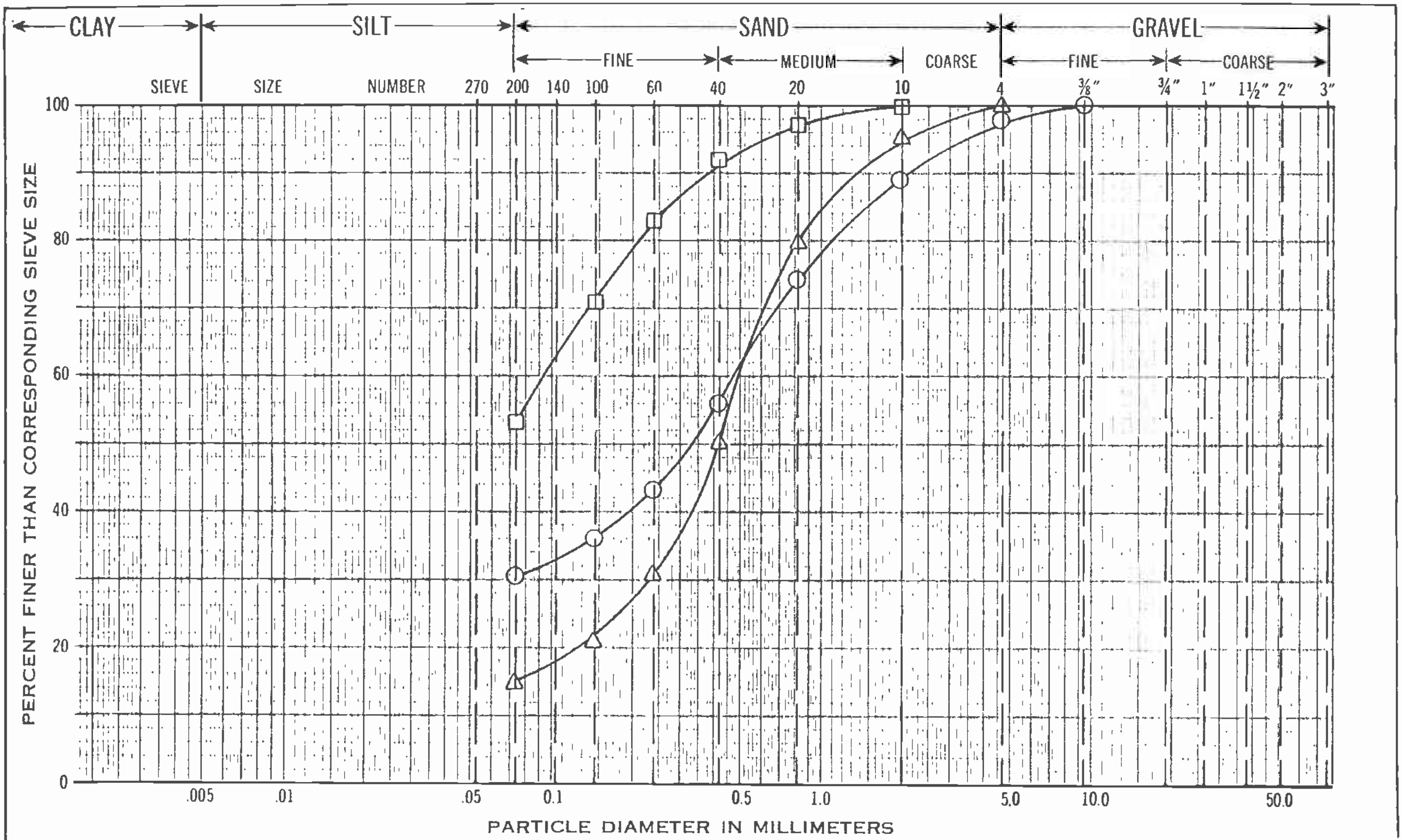

SYMBOL	BORING	SAMPLE	DEPTH
○	23/2	C-8	48'
□	23/4	C-4	39'
△	23/5	C-8	45'

GRAIN-SIZE DISTRIBUTION CHART

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Figure No.
C-7

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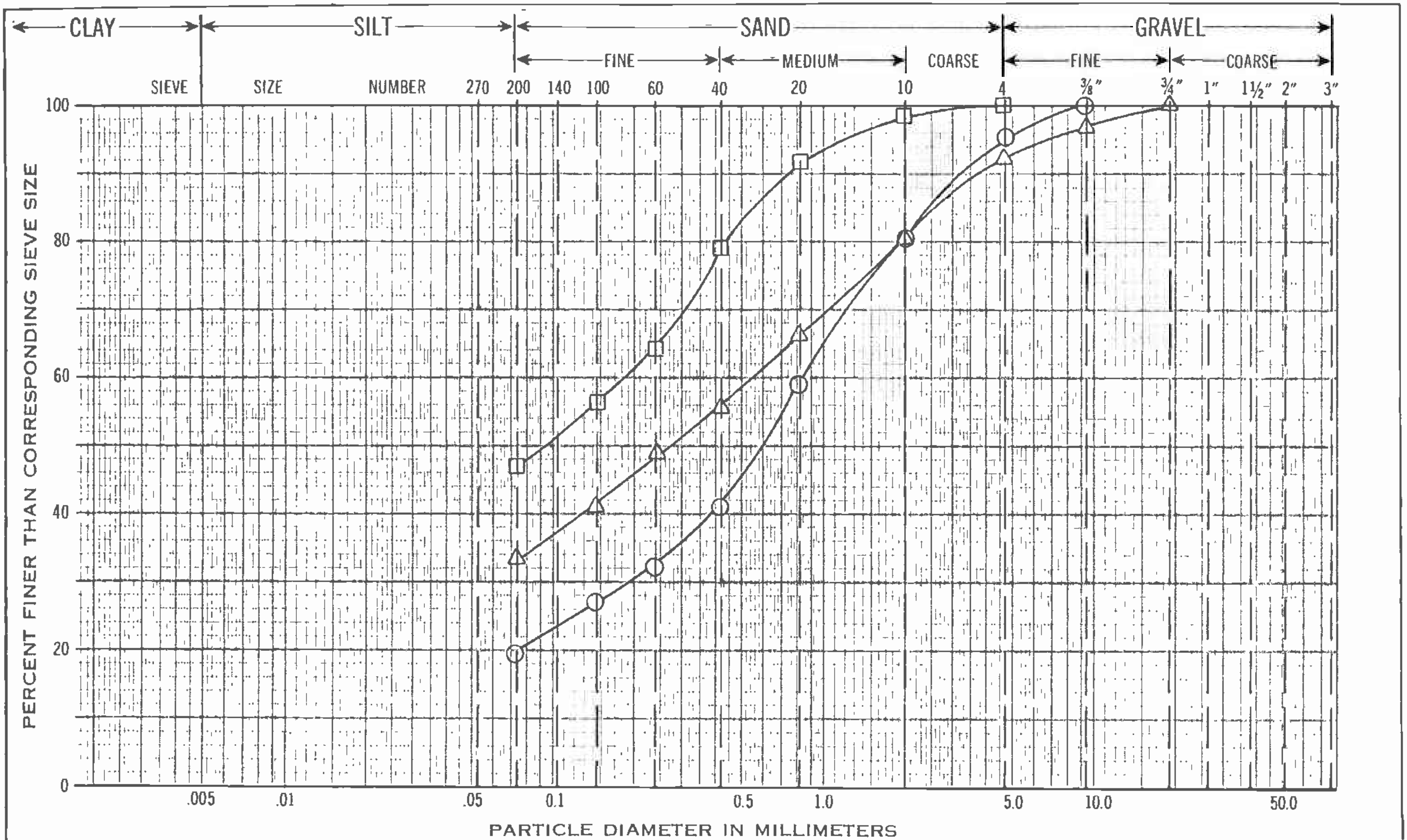

SYMBOL	BORING	SAMPLE	DEPTH
△	23/3	C-2	9'
□	23/3	C-6	34'
○	23/3	C-8	49'

GRAIN-SIZE DISTRIBUTION CHART

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Figure No.
C-8

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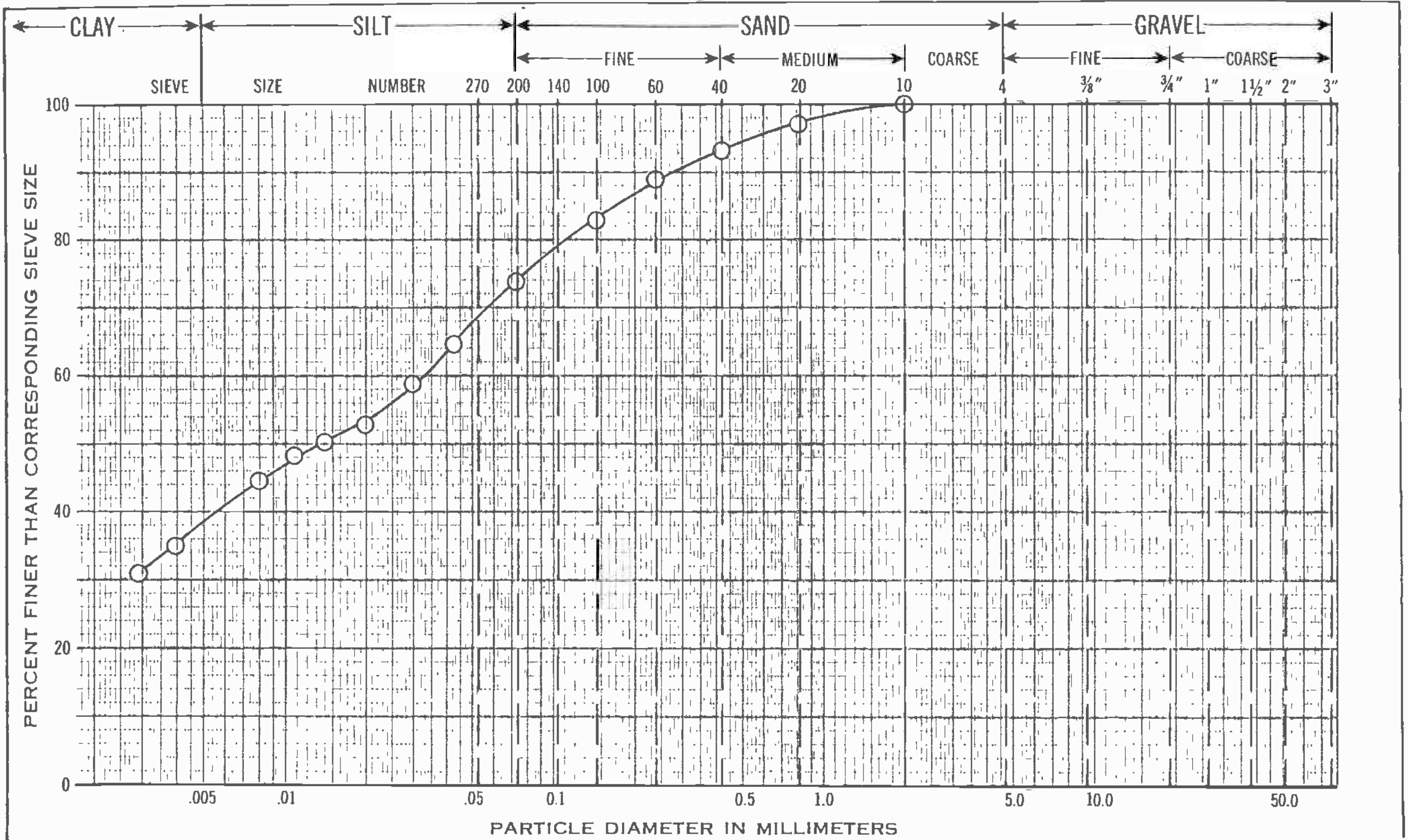

SYMBOL	BORING	SAMPLE	DEPTH
O	23/4	C-3	29'

GRAIN-SIZE DISTRIBUTION CHART

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Figure No. C-9

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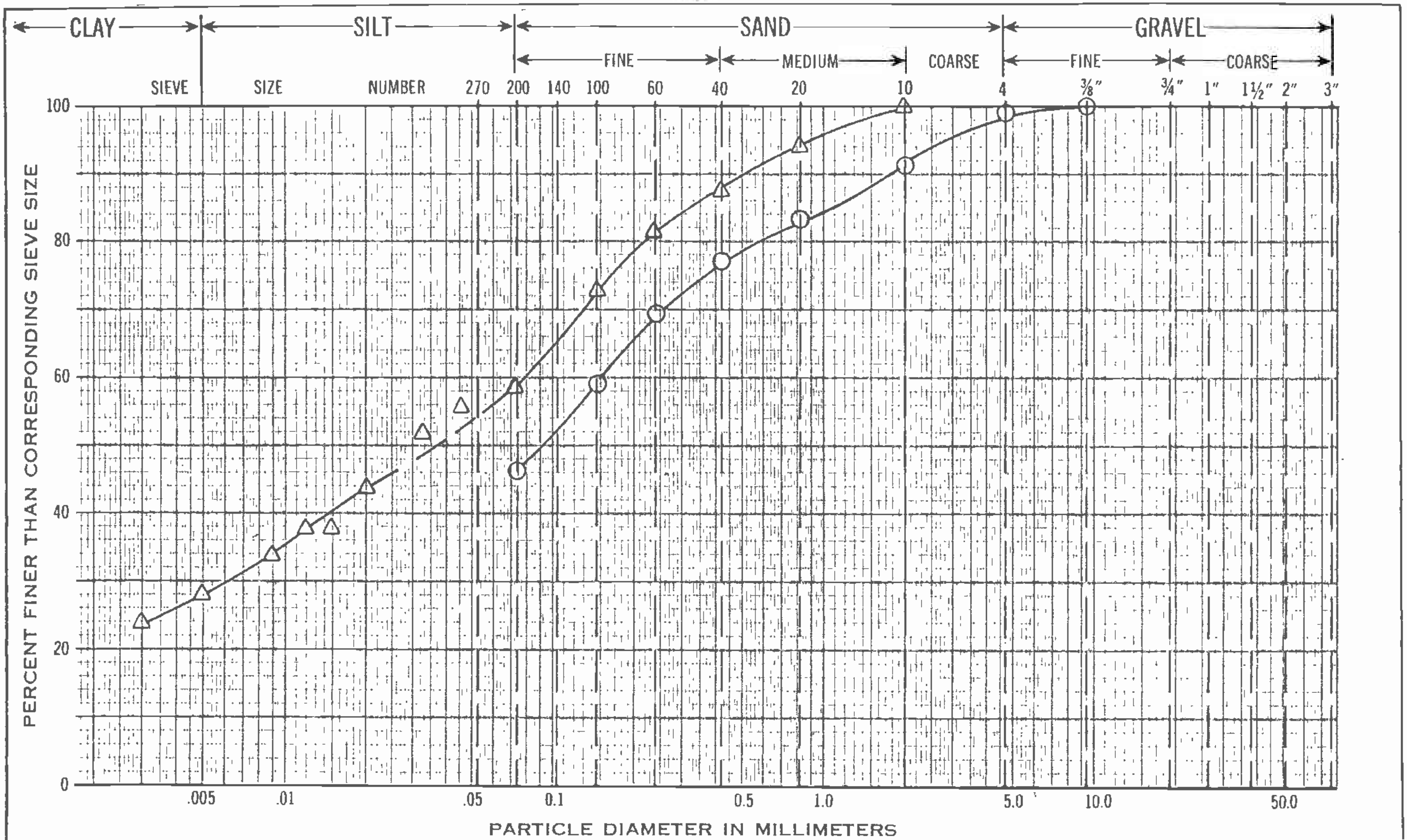

SYMBOL	BORING	SAMPLE	DEPTH
○	23/5	C-2	9'
△	23/5	C-6	29'

GRAIN-SIZE DISTRIBUTION CHART

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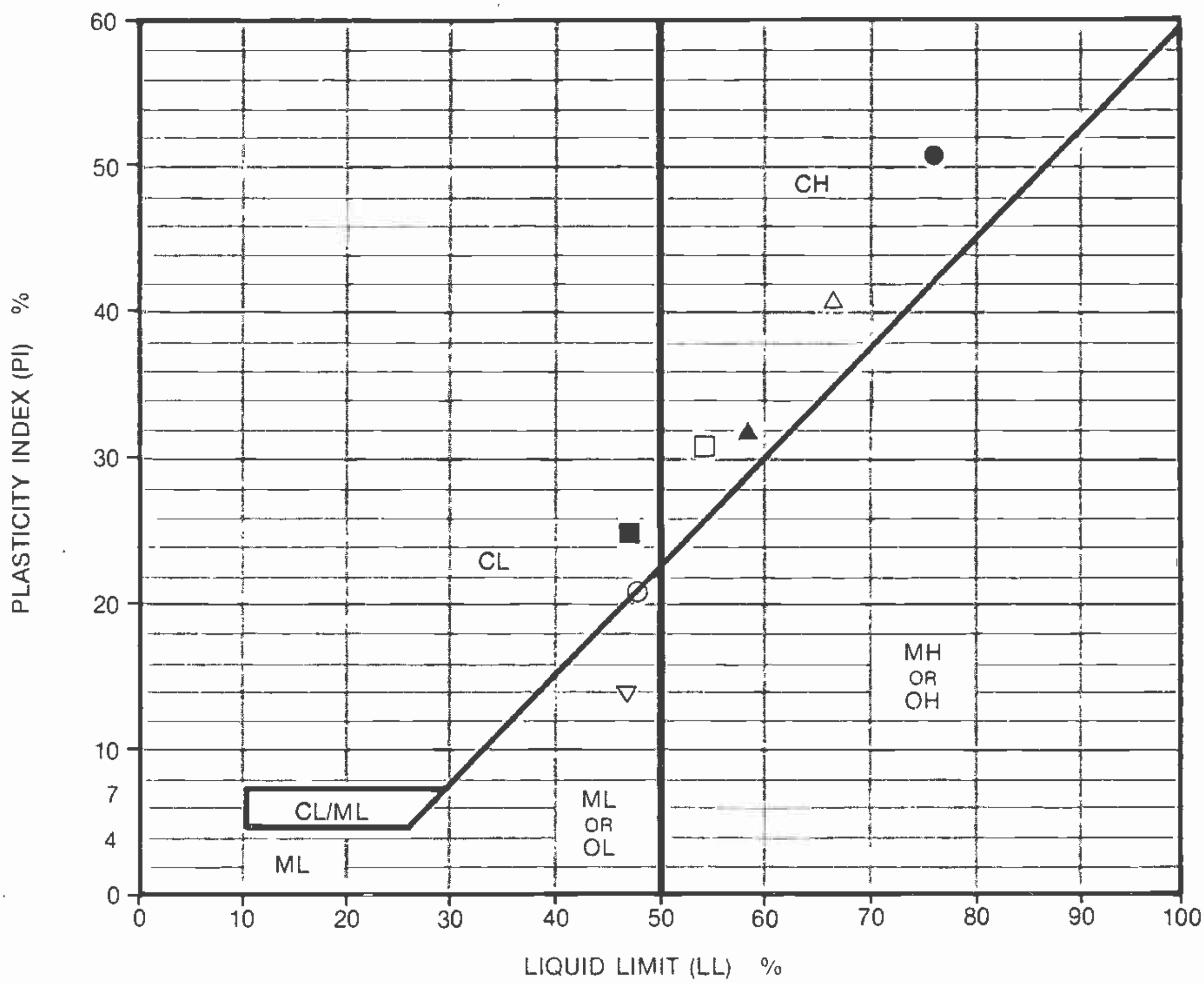
Figure No. C-10



Symbol	Classification and Source	Liquid Limit (%)	Plastic Limit (%)	Plasticity Index (%)	% Passing 200 Seive
△	BH 23-1, Sample C-2, 14'	67	26	41	-
□	BH 23-1, Sample C-7, 44'	54	23	31	-
○	BH 23-2, Sample C-11, 67'	48	27	21	-
▽	BH 23-3, Sample C-11, 68'	47	33	14	-
▲	BH 23-4, Sample C-7, 59'	59	27	32	-
■	BH 23-5, Sample C-5, 24'	47	22	25	-
●	BH 23-5, Sample C-11, 66'	76	25	51	-

PLASTICITY CHART

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Figure No. C-11

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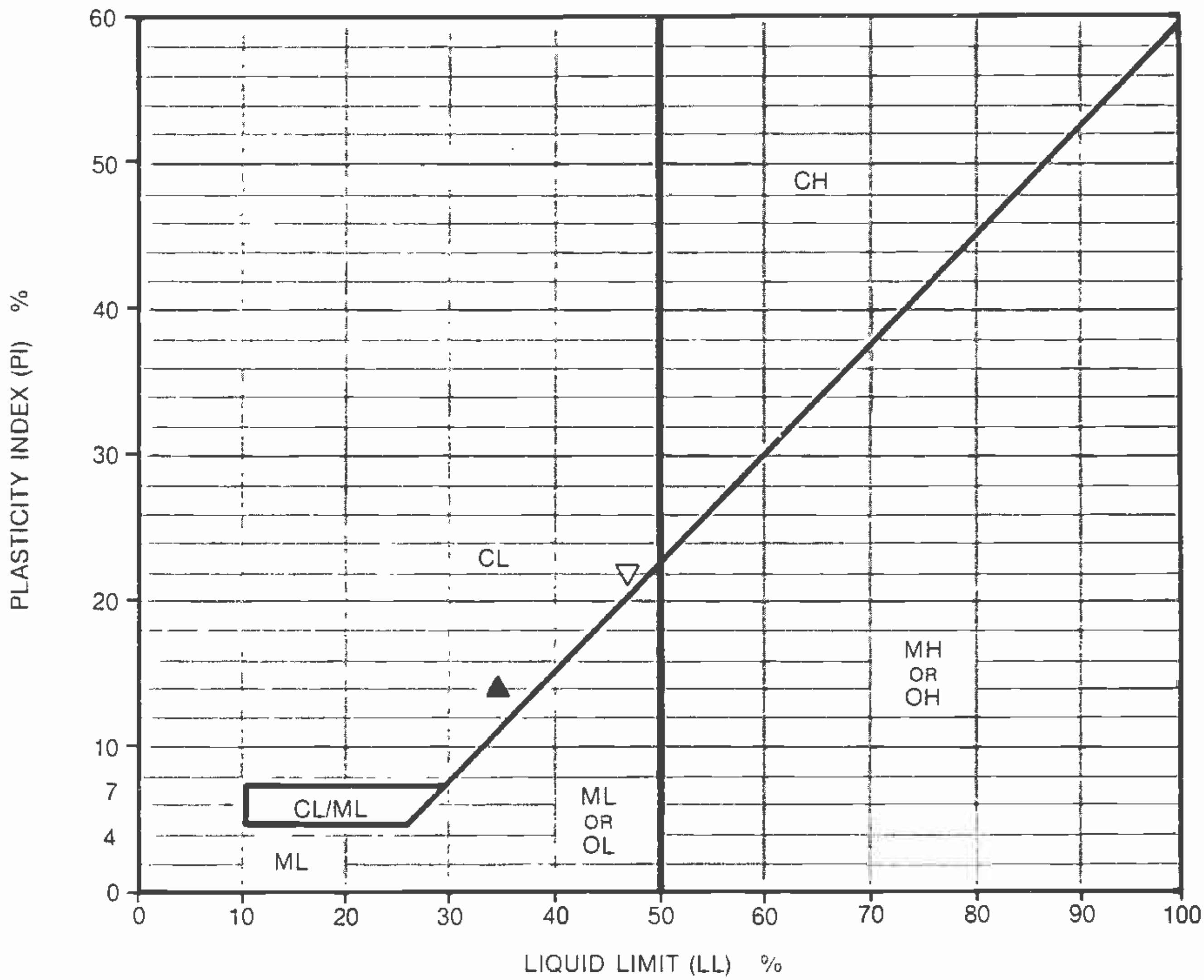

Symbol	Classification and Source	Liquid Limit (%)	Plastic Limit (%)	Plasticity Index (%)	% Passing 200 Seive
▲	BH 23, 31' (CL)	35	21	14	-
▽	BH 23, 50' (CL)	47	25	22	-

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PLASTICITY CHART

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Figure No C-12

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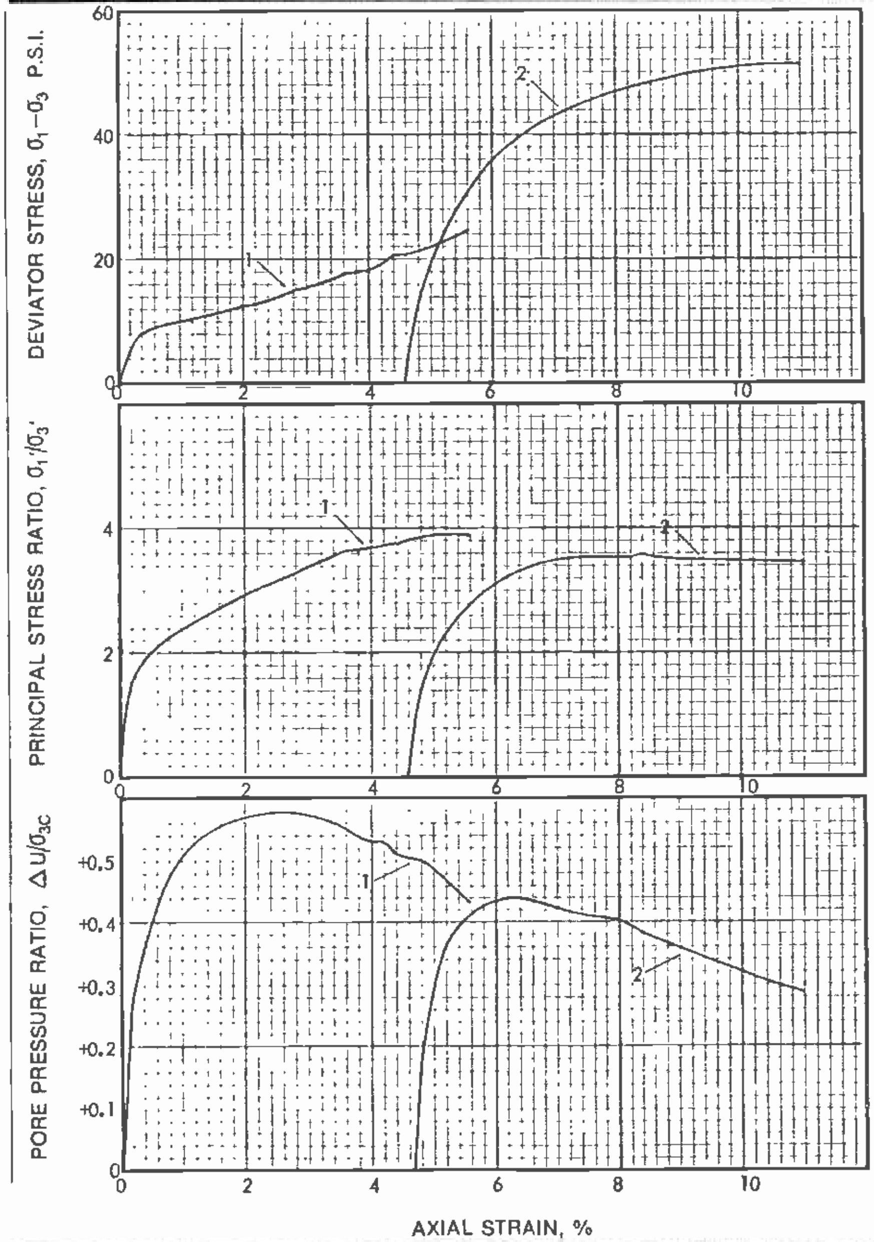

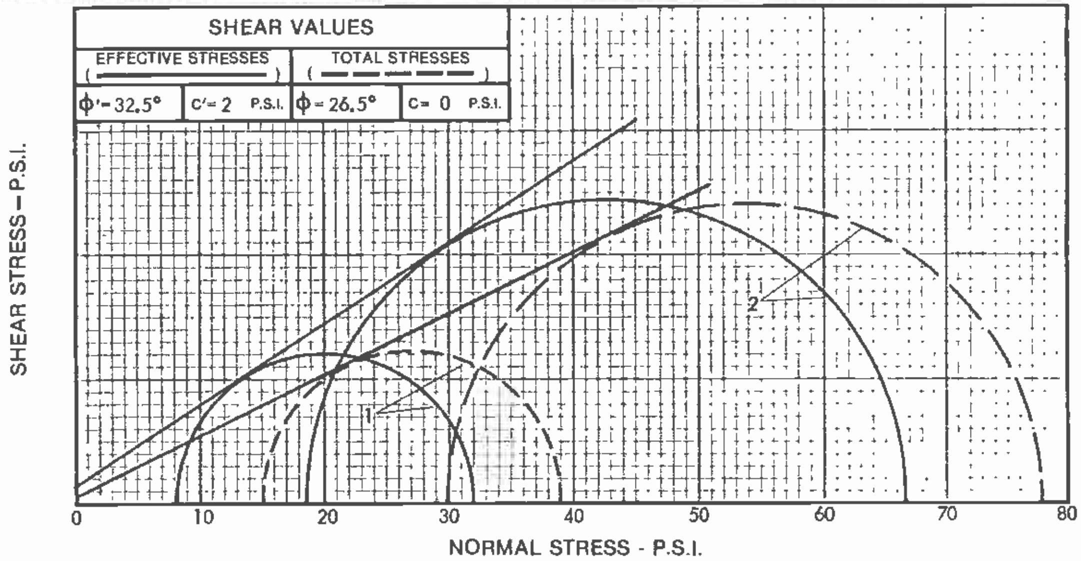

SPECIMEN NUMBER	SPECIMEN LOCATION			INITIAL SPECIMEN DATA					SAMPLE TYPE
	BORING NUMBER	SAMPLE NUMBER	DEPTH (FEET)	SOIL CLASSIFICATION	LENGTH (INCHES)	DIAMETER (INCHES)	DRY DENSITY (P.C.F.)	MOISTURE CONTENT (PERCENT)	
C-10	23/1	C-10	63-64	SM/ML	5.0	2.42	94.7	28.8	5 RING CONVERSE

SPECIMEN NUMBER	SYMBOL	EFFECTIVE CONSOL. PRESSURE σ_{3C} (P.S.I.)	TEST VALUES AT FAILURE (MAXIMUM σ_1'/σ_3')				TEST TYPE
			TOTAL DEVIATOR STRESS $\sigma_1-\sigma_3$ (P.S.I.)	PORE PRESSURE CHANGE ΔU (P.S.I.)	MINOR EFFECTIVE STRESS σ_3' (P.S.I.)	MAJOR EFFECTIVE STRESS σ_1' (P.S.I.)	
C-10	1	15	23.8	6.8	8.2	32.0	TX CUE PROGRESSIVE
C-10	2	30	47.8	11.4	18.6	66.5	

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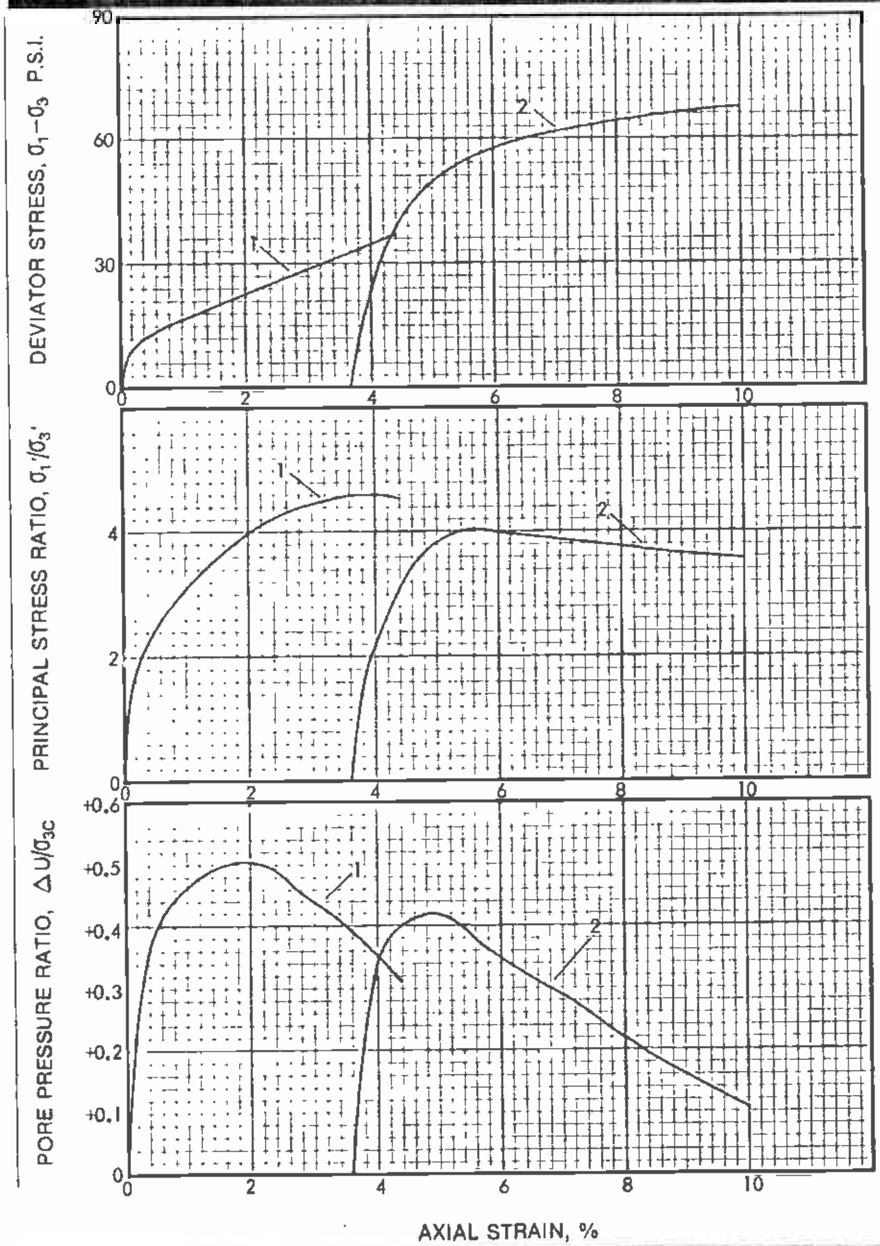
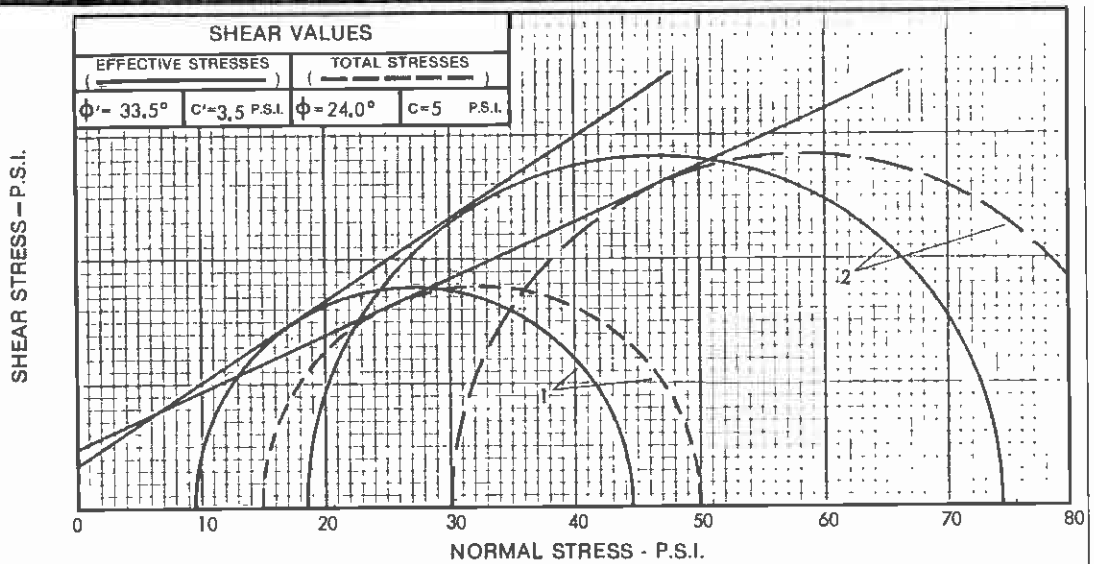
Figure No C-13

Approved for publication 4/14 by JAD





SHEAR VALUES			
EFFECTIVE STRESSES		TOTAL STRESSES	
ϕ'= 33.5°	C'=3.5 P.S.I.	ϕ = 24.0°	C=5 P.S.I.

SPECIMEN NUMBER	SPECIMEN LOCATION			INITIAL SPECIMEN DATA					SAMPLE TYPE
	BORING NUMBER	SAMPLE NUMBER	DEPTH (FEET)	SOIL CLASSIFICATION	LENGTH (INCHES)	DIAMETER (INCHES)	DRY DENSITY (P.C.F.)	MOISTURE CONTENT (PERCENT)	
C-7	23/2	C-7	42 - 43	ML/CL	5.0	2.42	103.9	22.6	5 RING CONVERSE

SPECIMEN NUMBER	SYMBOL	EFFECTIVE CONSOL. PRESSURE σ_{3C} (P.S.I.)	TEST VALUES AT FAILURE (MAXIMUM σ_1'/σ_3')				TEST TYPE
			TOTAL DEVIATOR STRESS σ_1-σ_3 (P.S.I.)	PORE PRESSURE CHANGE ΔU (P.S.I.)	MINOR EFFECTIVE STRESS σ_3' (P.S.I.)	MAJOR EFFECTIVE STRESS σ_1' (P.S.I.)	
C-7	1	15	34.9	5.3	9.7	44.6	TX CUE PROGRESSIVE
C-7	2	30	55.9	11.5	18.5	74.4	

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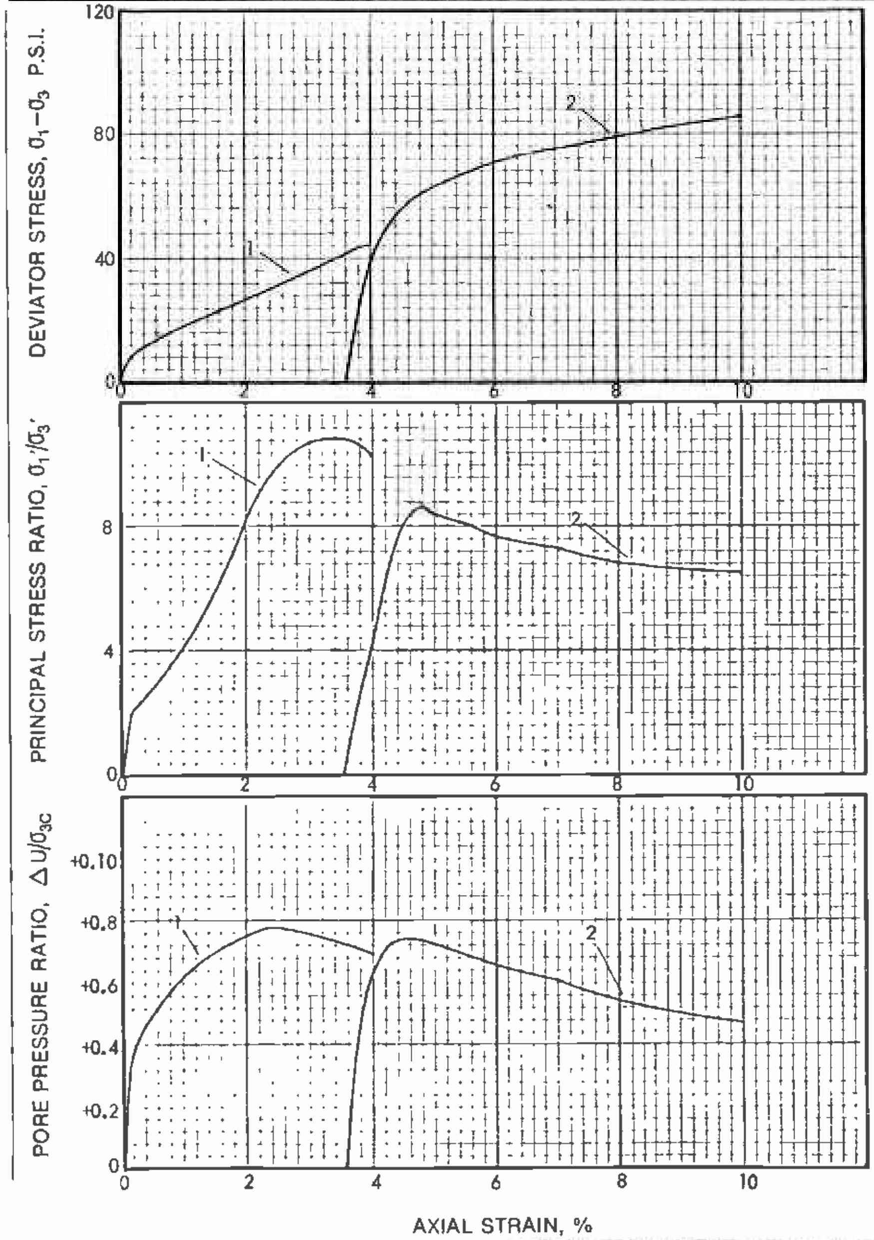
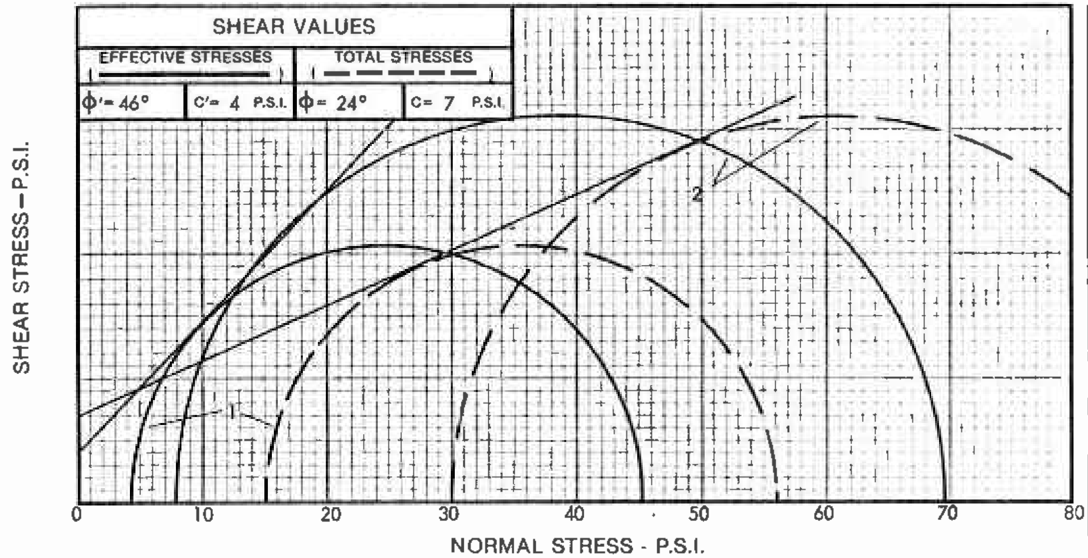
Figure No C-14

Approved for publication 4/84 by [illegible]





SPECIMEN NUMBER	SPECIMEN LOCATION			INITIAL SPECIMEN DATA					SAMPLE TYPE
	BORING NUMBER	SAMPLE NUMBER	DEPTH (FEET)	SOIL CLASSI-FICATION	LENGTH (INCHES)	DIAMETER (INCHES)	DRY DENSITY (P.C.F.)	MOISTURE CONTENT (PERCENT)	
C-10	23/3	C-10	58-59	Tar Silt	5.0	2.42	109.3	13.1	5 RING CONVERSE

SPECIMEN NUMBER	SYMBOL	EFFECTIVE CONSOL. PRESSURE σ_{3C} (P.S.I.)	TEST VALUES AT FAILURE (MAXIMUM σ_1'/σ_3')				TEST TYPE
			TOTAL DEVIATOR STRESS $\sigma_1-\sigma_3$ (P.S.I.)	PORE PRESSURE CHANGE ΔU (P.S.I.)	MINOR EFFECTIVE STRESS σ_3' (P.S.I.)	MAJOR EFFECTIVE STRESS σ_1' (P.S.I.)	
C-10	1	15	41.0	10.8	4.2	45.2	PROGRESSIVE TX CUE
C-10	2	30	61.4	22.0	8.0	69.4	

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Figure No C-15

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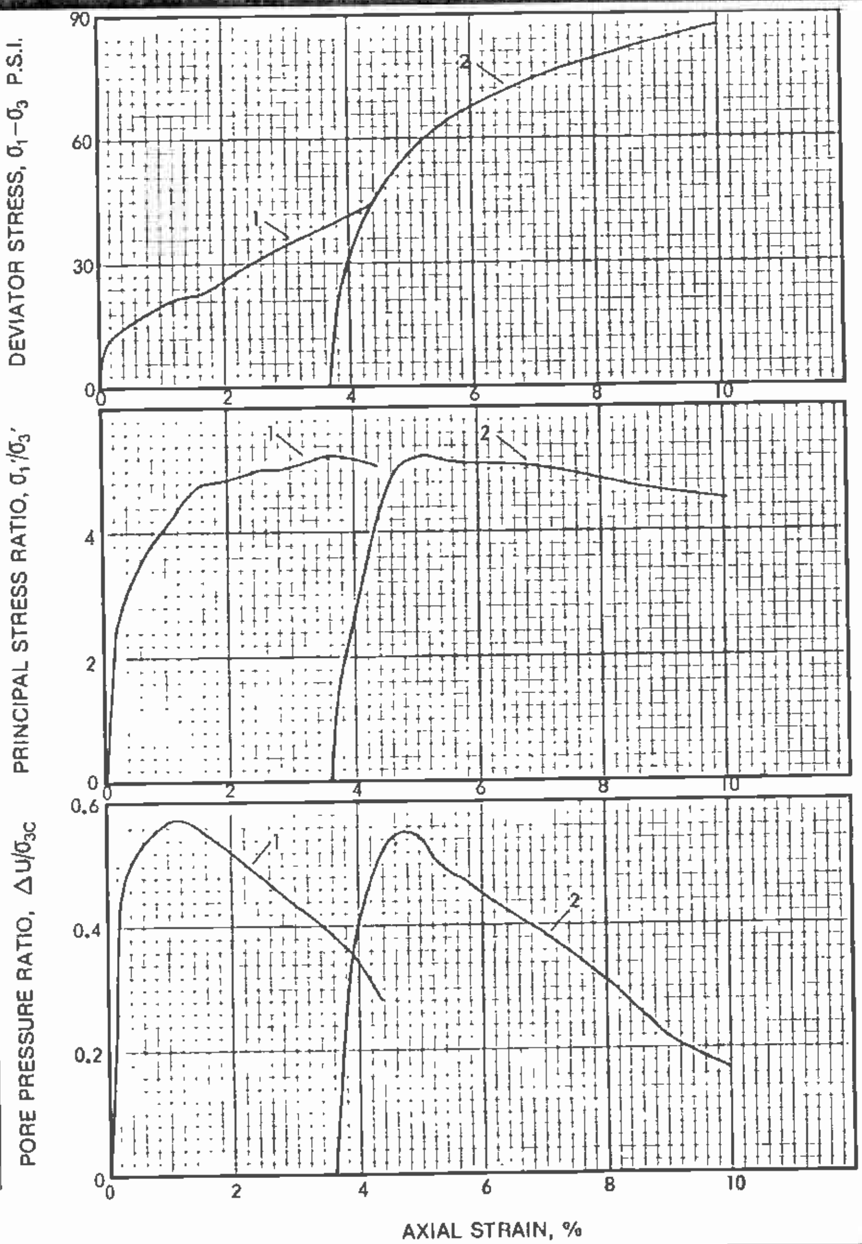

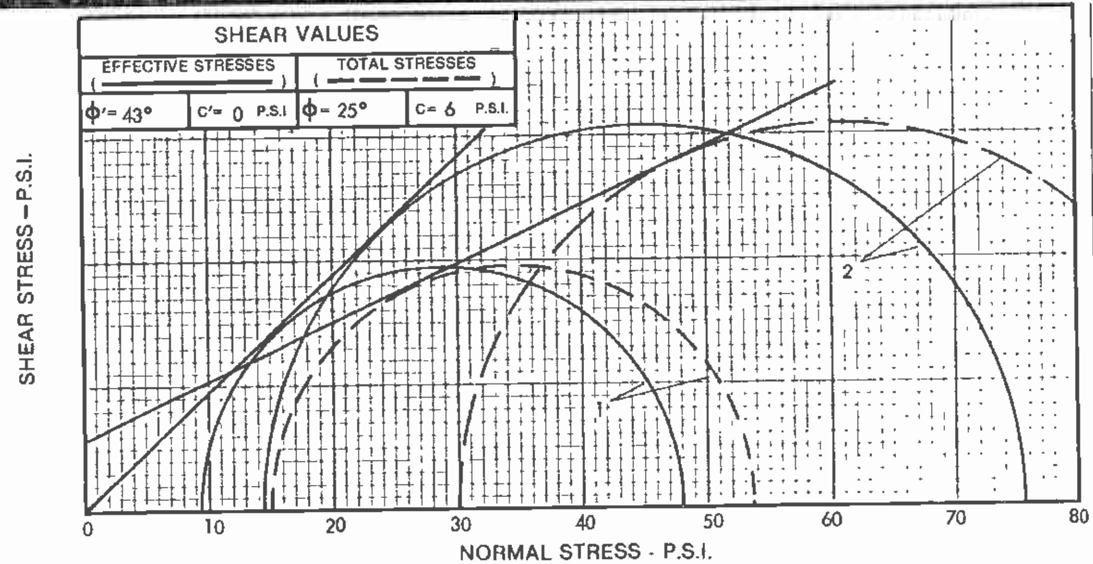

SPECIMEN NUMBER	SPECIMEN LOCATION			INITIAL SPECIMEN DATA					SAMPLE TYPE
	BORING NUMBER	SAMPLE NUMBER	DEPTH (FEET)	SOIL CLASSIFICATION	LENGTH (INCHES)	DIAMETER (INCHES)	DRY DENSITY (P.C.F.)	MOISTURE CONTENT (PERCENT)	
C-8	23/4	C-8	64-65	Tar Silt	5.0	2.42	106.1	17.1	5 RING CONVERSE

SPECIMEN NUMBER	SYMBOL	EFFECTIVE CONSOL PRESSURE σ_{3C} (P.S.I.)	TEST VALUES AT FAILURE (MAXIMUM σ_1'/σ_3')				TEST TYPE
			TOTAL DEVIATOR STRESS $\sigma_1-\sigma_3$ (P.S.I.)	PORE PRESSURE CHANGE ΔU (P.S.I.)	MINOR EFFECTIVE STRESS σ_3' (P.S.I.)	MAJOR EFFECTIVE STRESS σ_1' (P.S.I.)	
C-8	1	15	38.7	5.8	9.2	47.9	PROGRESSIVE CUE
C-8	2	30	61.0	15.5	14.5	75.5	

TRIAXIAL COMPRESSION TESTS

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Figure No C-16

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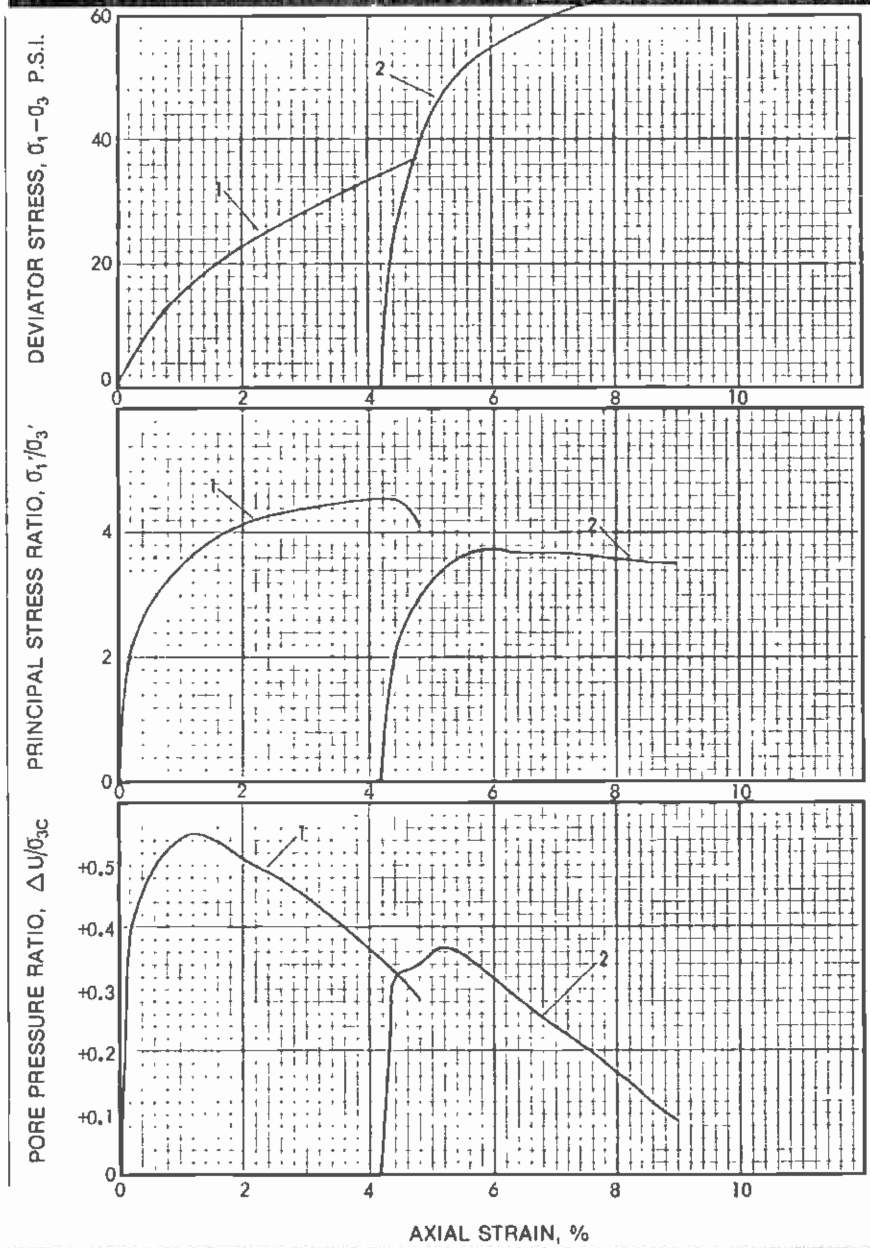

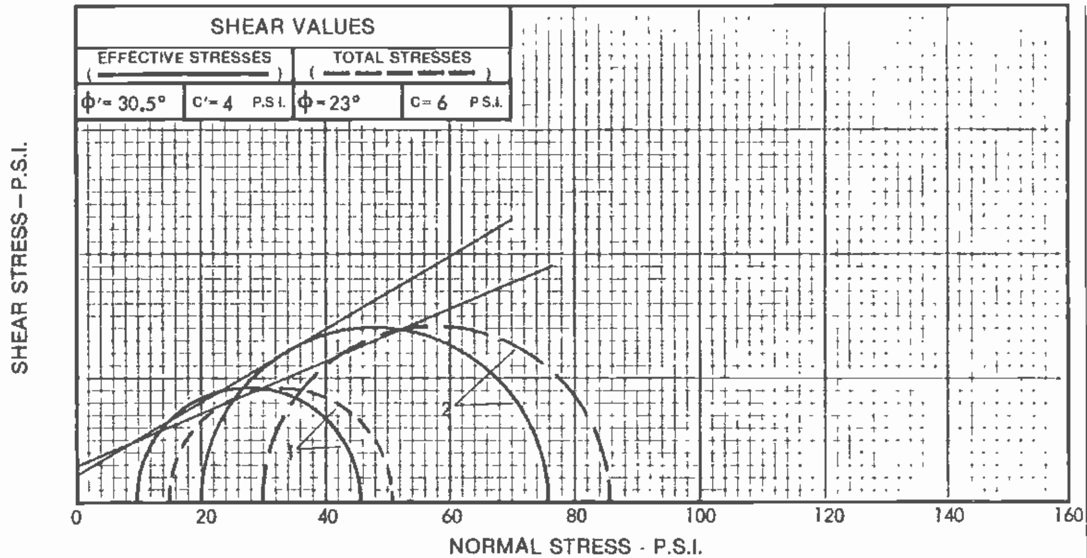

SHEAR VALUES			
EFFECTIVE STRESSES (———)		TOTAL STRESSES (— — —)	
ϕ'= 30.5°	C'= 4 P.S.I.	ϕ= 23°	C= 6 P.S.I.

SPECIMEN NUMBER	SPECIMEN LOCATION			INITIAL SPECIMEN DATA					SAMPLE TYPE
	BORING NUMBER	SAMPLE NUMBER	DEPTH (FEET)	SOIL CLASSIFICATION	LENGTH (INCHES)	DIAMETER (INCHES)	DRY DENSITY (P.C.F.)	MOISTURE CONTENT (PERCENT)	
C-6	23/5	C-6	28 - 29	ML/CL	5.0	2.42	106.7	21.0	5 RING CONVERSE

SPECIMEN NUMBER	SYMBOL	EFFECTIVE CONSOL. PRESSURE σ_{3C} (P.S.I.)	TEST VALUES AT FAILURE (MAXIMUM σ_1'/σ_3')				TEST TYPE
			TOTAL DEVIATOR STRESS $\sigma_1-\sigma_3$ (P.S.I.)	PORE PRESSURE CHANGE ΔU (P.S.I.)	MINOR EFFECTIVE STRESS σ_3' (P.S.I.)	MAJOR EFFECTIVE STRESS σ_1' (P.S.I.)	
C-6	1	15	35.7	5.0	10.0	45.6	TX CUE PROGRESSIVE
C-6	2	30	55.5	9.6	20.4	75.9	

TRIAXIAL COMPRESSION TESTS

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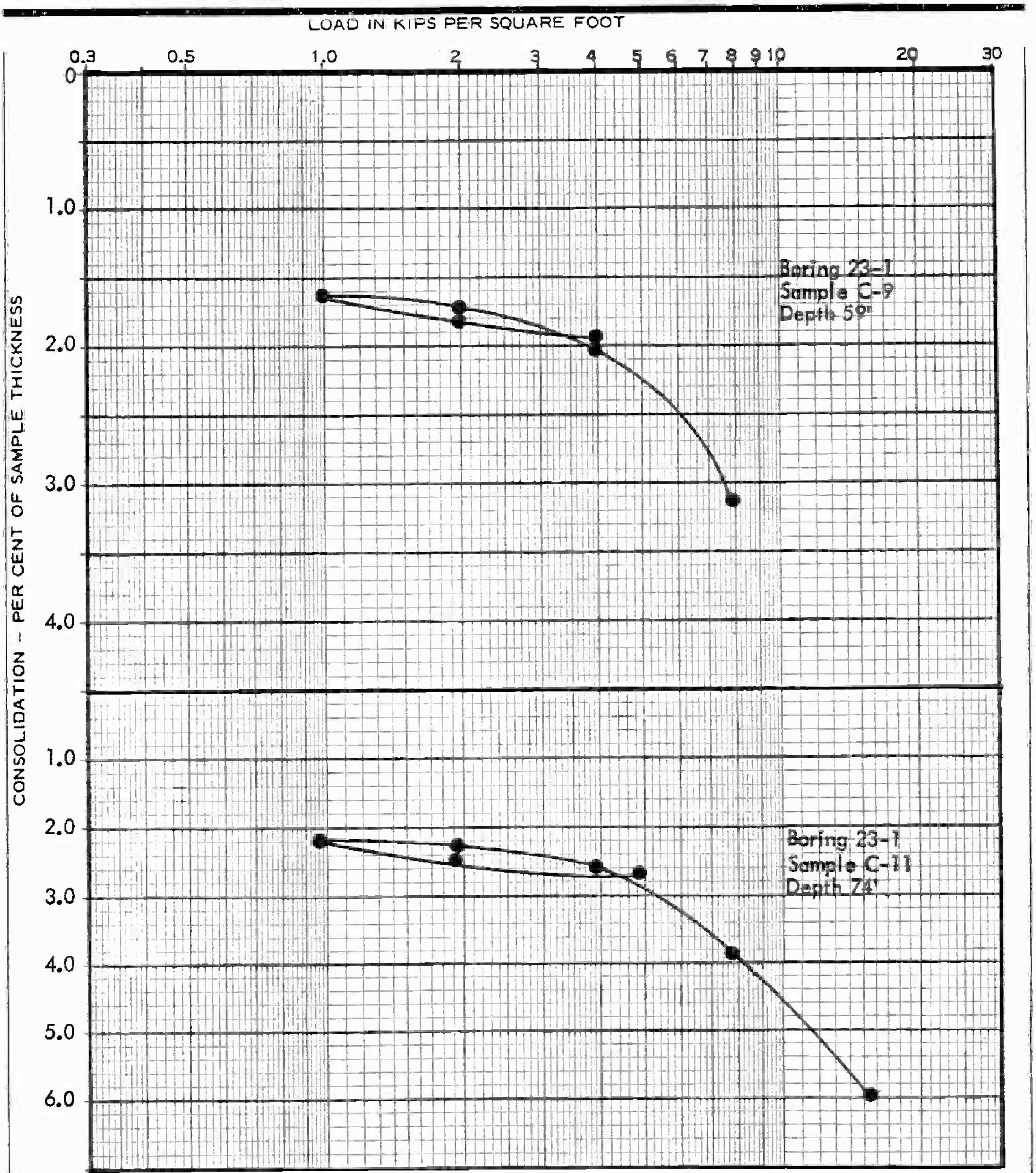
Figure No C-17



• READINGS AFTER SATURATION WITH WATER

CONSOLIDATION TESTS

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83-1140

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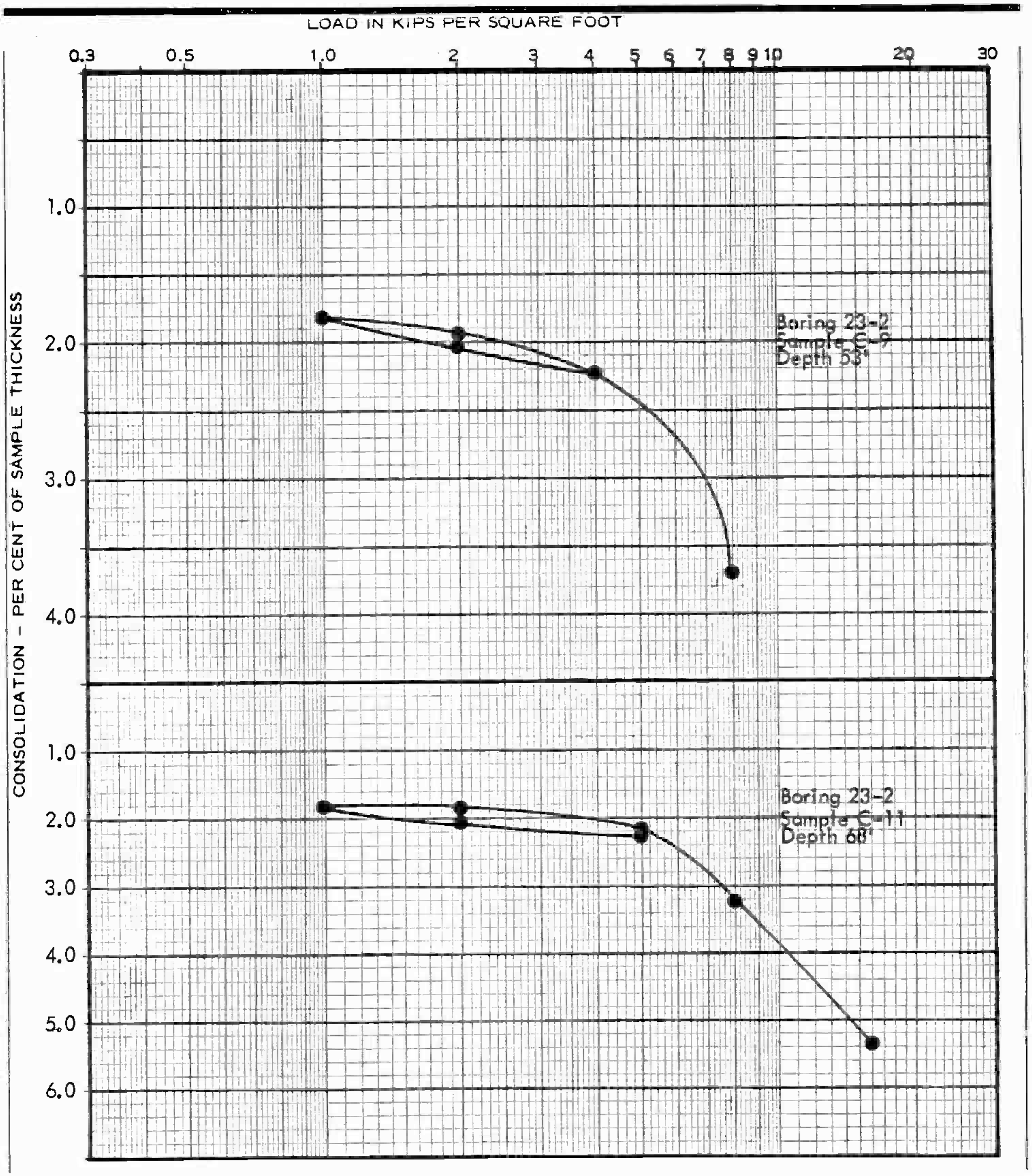
Drawing No.
C-18

Approved for publication 4/84 by JAG



● READINGS AFTER SATURATION WITH WATER

CONSOLIDATION TESTS

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Drawing No. C-19

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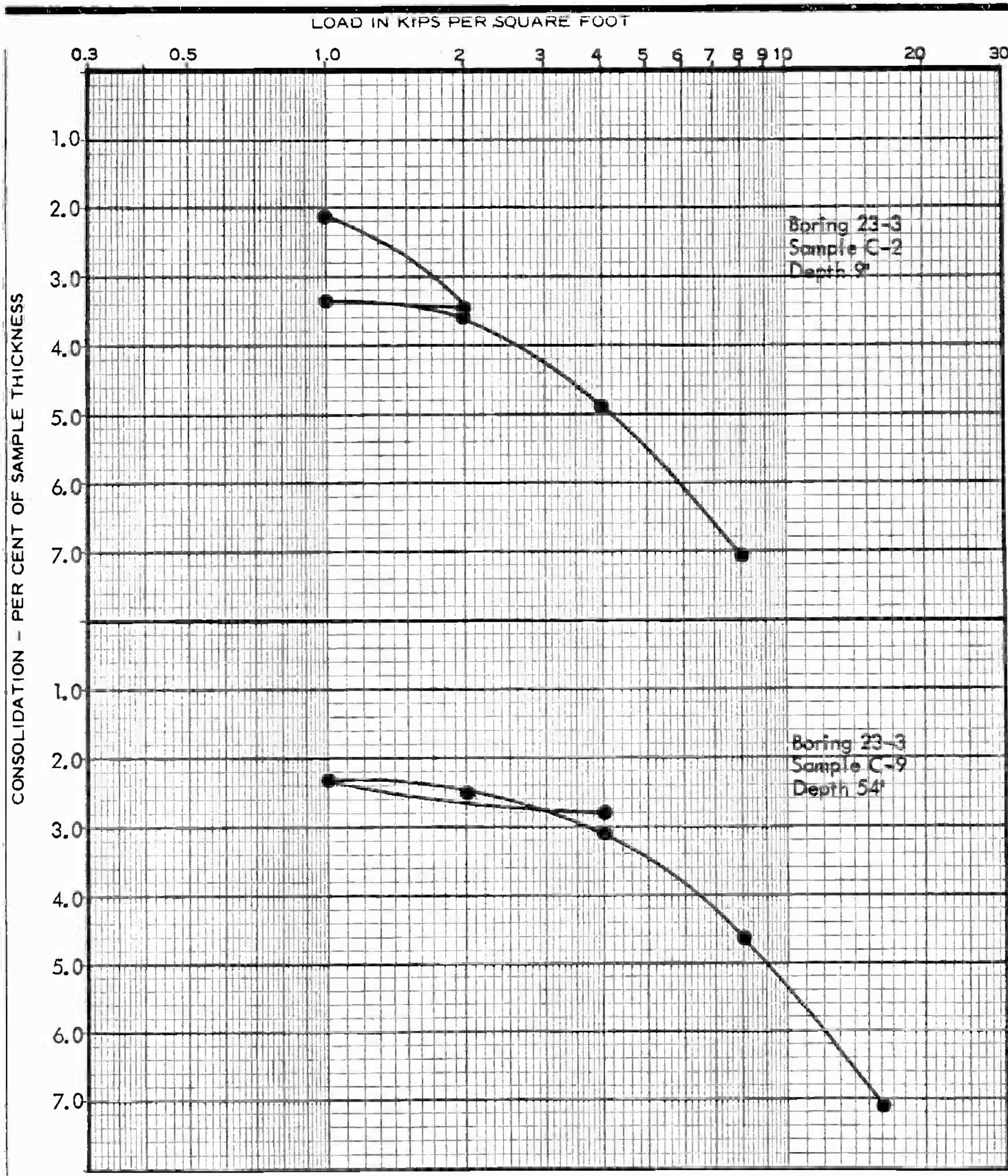

● READINGS AFTER SATURATION WITH WATER

CONSOLIDATION TESTS

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METRO RAIL PROJECT

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Drawing No. C-20

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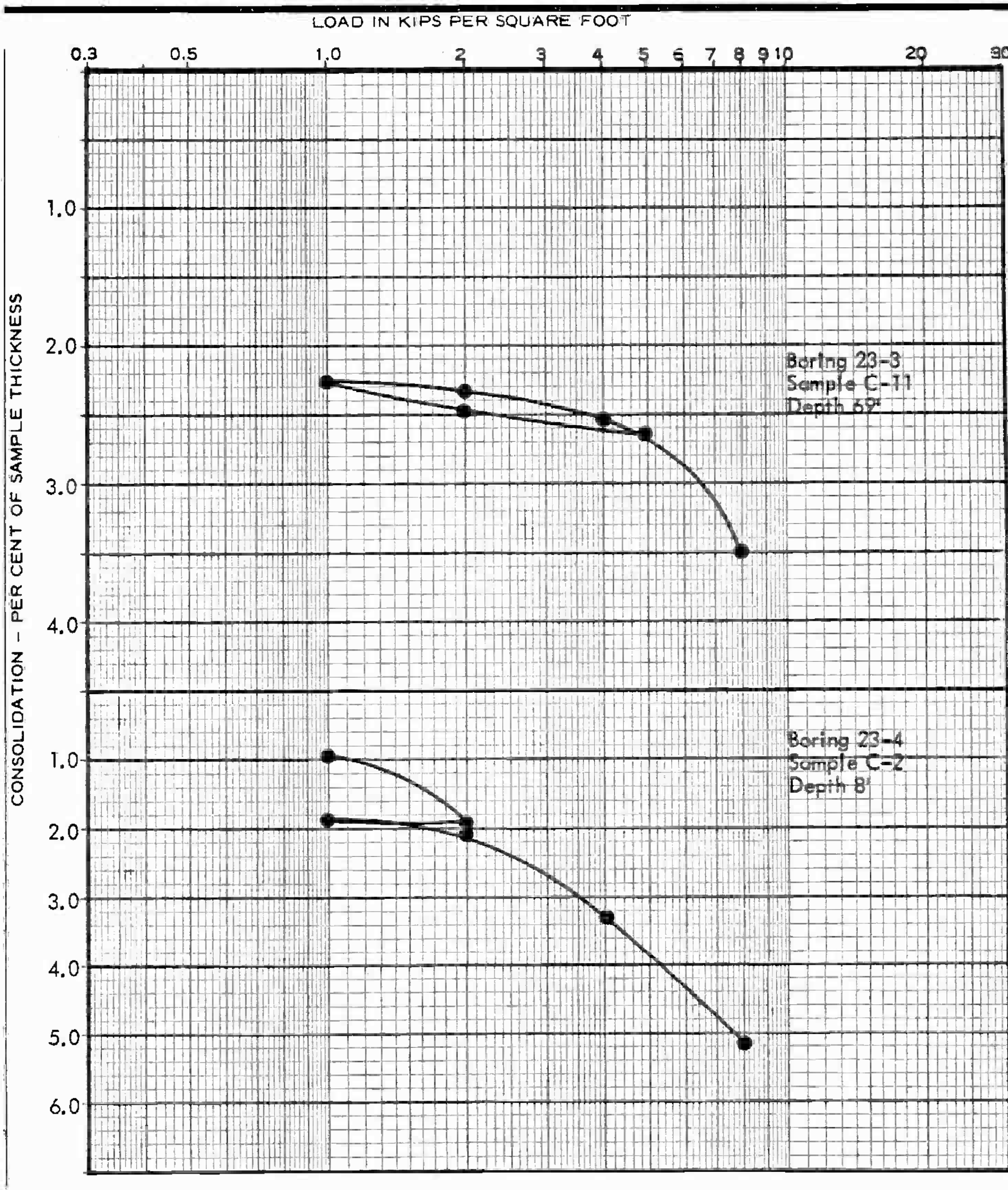

● READINGS AFTER SATURATION WITH WATER

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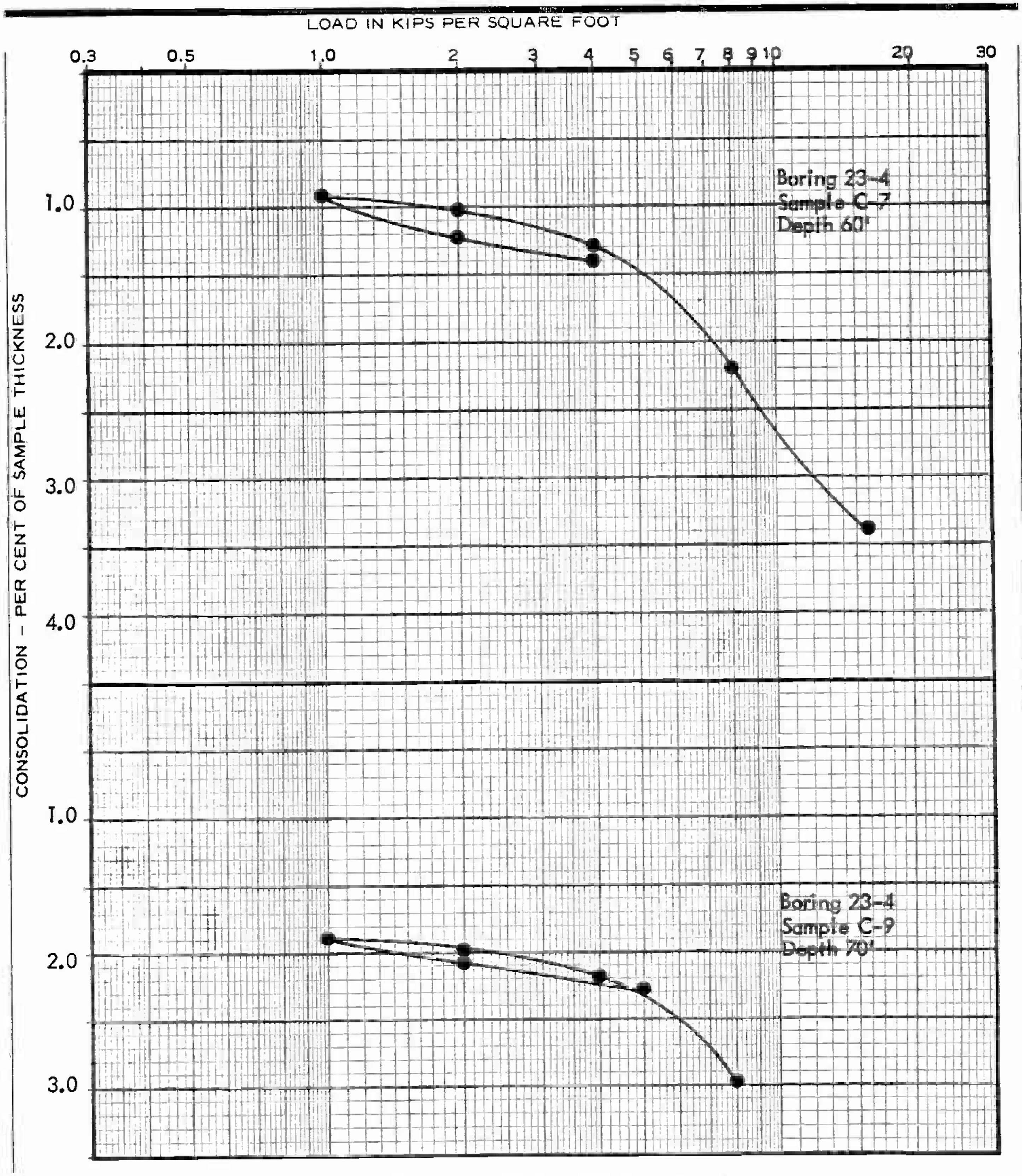
Drawing No. C-21

Approved for publication 4/84 by JAD



● READINGS AFTER SATURATION WITH WATER

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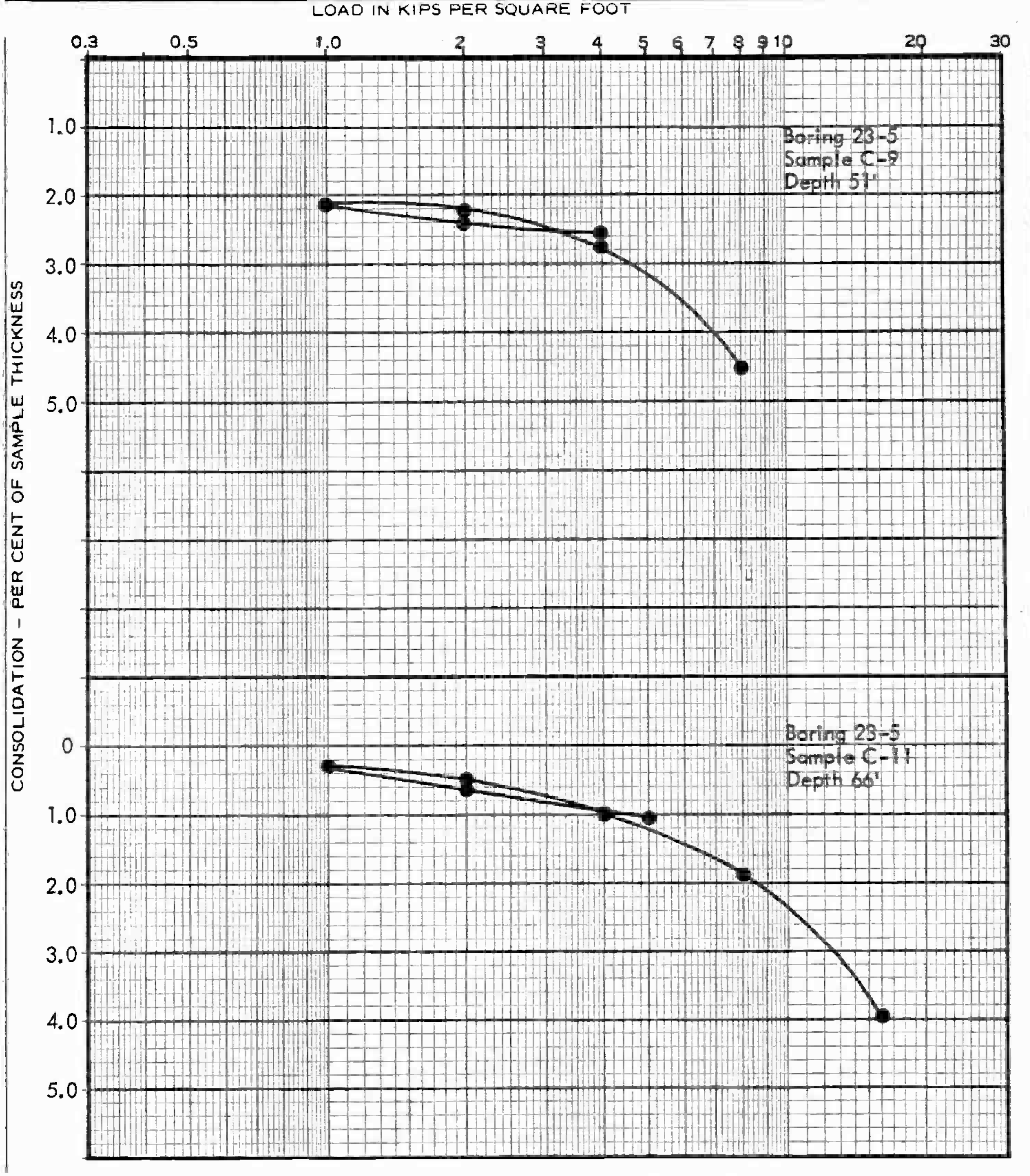
Drawing No. C-22

Approved for publication 4/84 by [illegible]



● READINGS AFTER SATURATION WITH WATER

CONSOLIDATION TESTS

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Drawing No.
C-23

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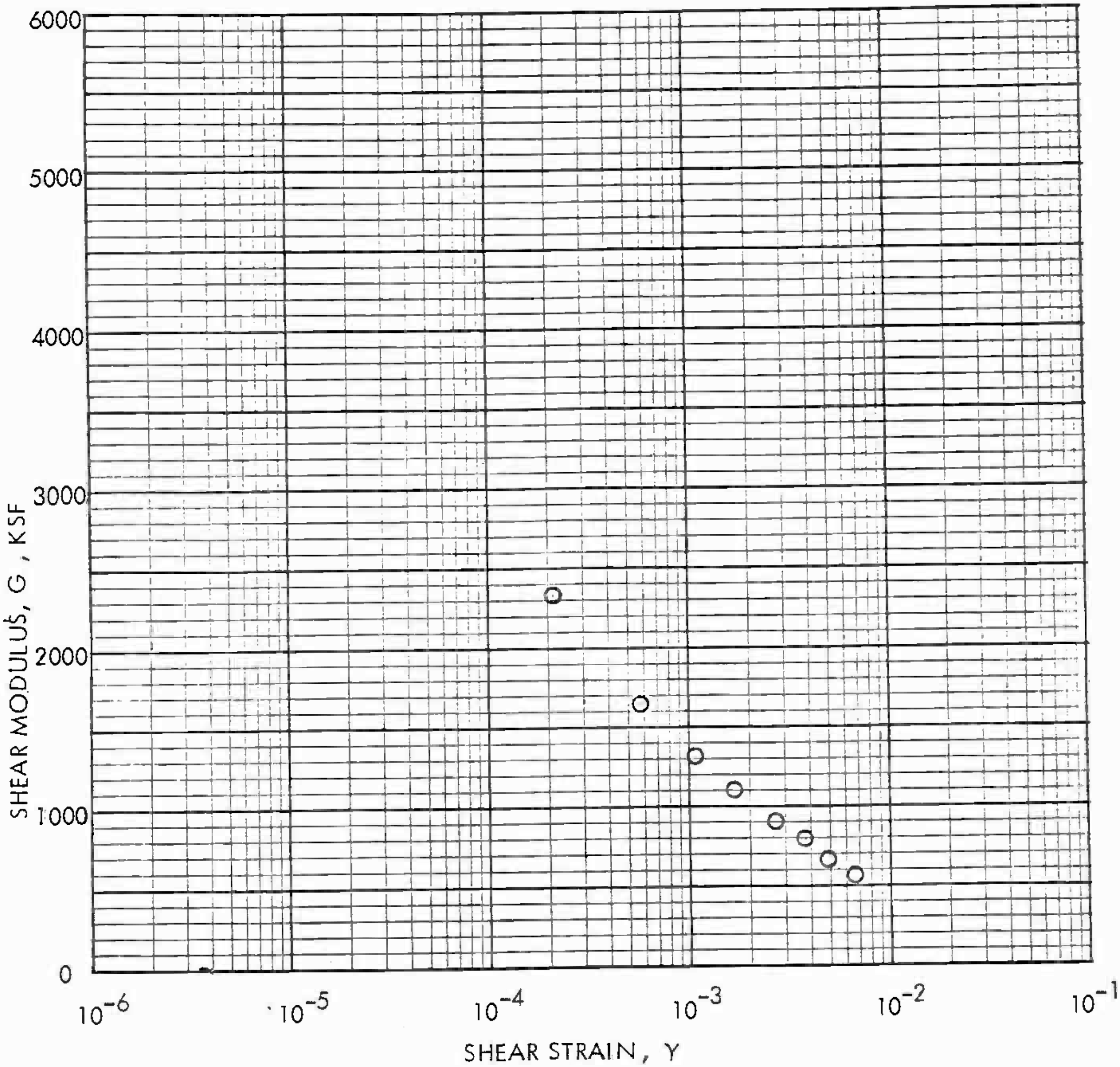
STRAIN DEPENDENT DYNAMIC SHEAR MODULUS



BORING	SAMPLE	DEPTH(FT)	γ_d(PCF)	w_o(%)	$\bar{\sigma}_c$(PSI)
23	C-4	41	100	24	30

Sample Description: Dark Gray Silty Clay; stiff

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DYNAMIC TRIAXIAL TEST

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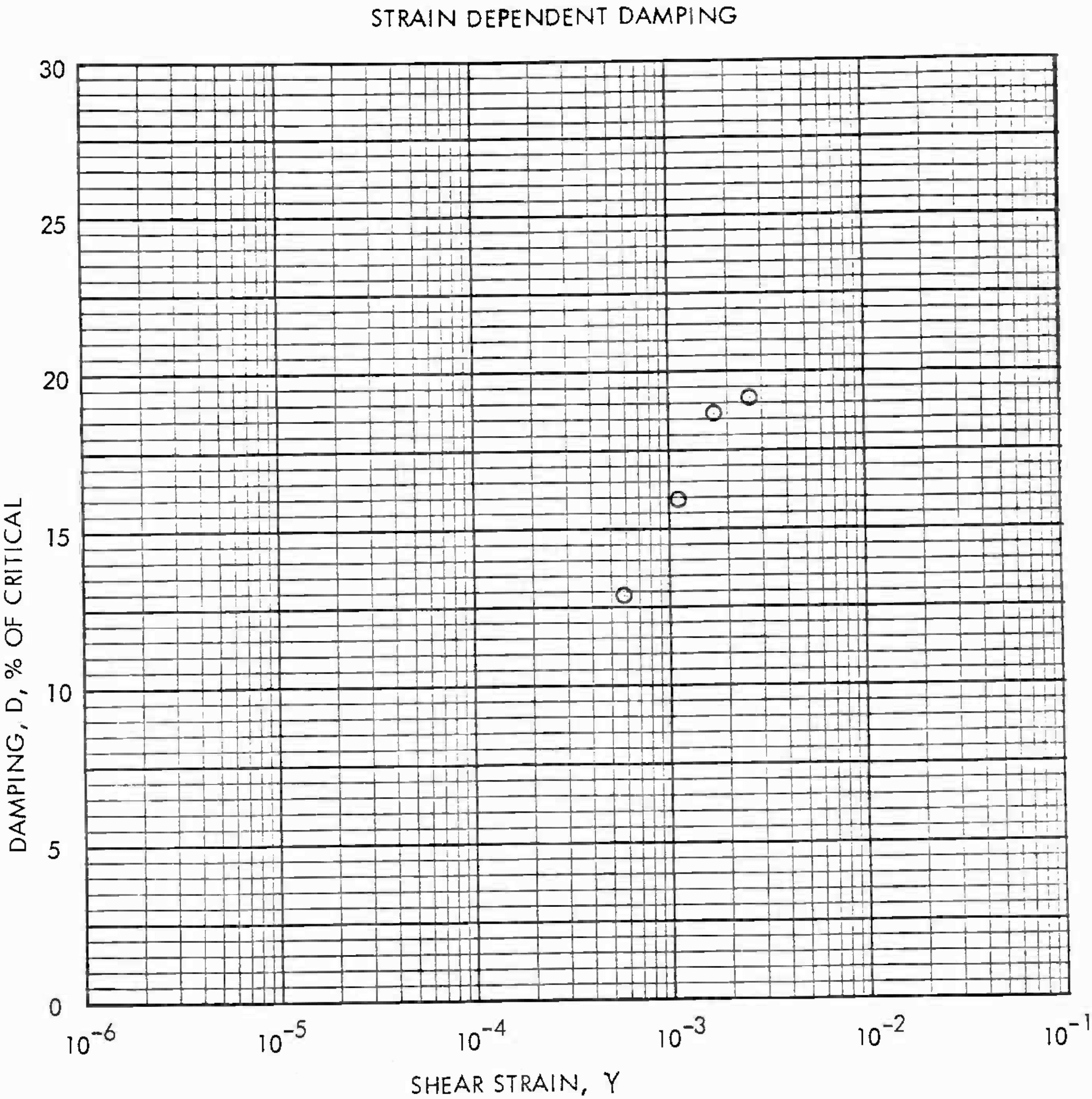
Figure No. C-24



BORING	SAMPLE	DEPTH(FT)	γ_d(PCF)	w_o(%)	$\bar{\sigma}_c$(PSI)
23	C-4	41	100	24	30

Sample Description: Dark Gray Silty Clay; stiff

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DYNAMIC TRIAXIAL TEST

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Figure No. C-25

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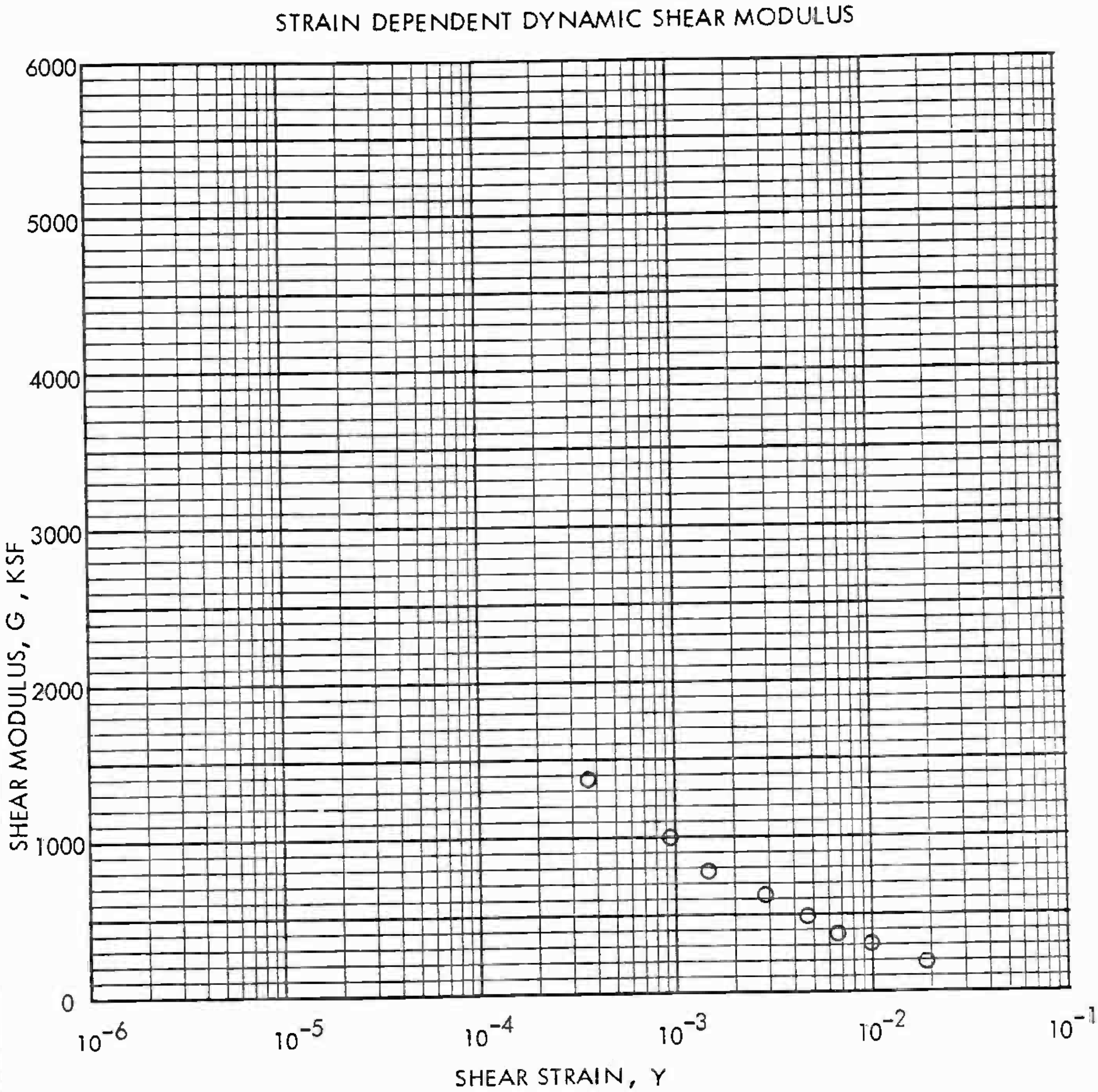

BORING	SAMPLE	DEPTH(FT)	γ_d(PCF)	w_o(%)	$\bar{\sigma}_c$(PSI)	SYMBOL
23	C-6	56	86	34	30	O

Sample Description: Green-Brown, Sandy Clay; firm

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DYNAMIC TRIAXIAL TEST

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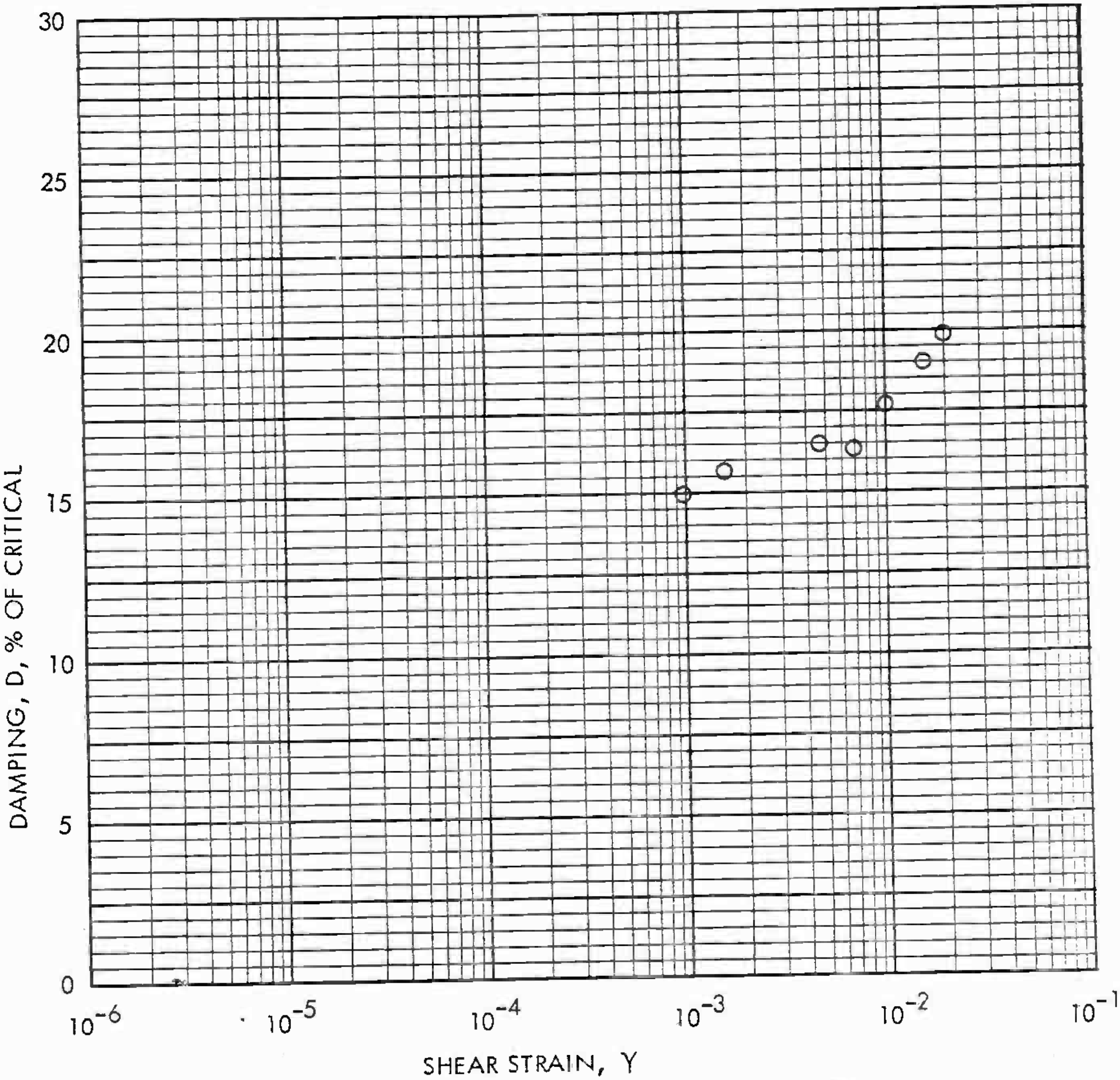
Figure No. C-26

STRAIN DEPENDENT DAMPING



BORING	SAMPLE	DEPTH(FT)	γ_d(PCF)	w_o(%)	$\bar{\sigma}_c$(PSI)	SYMBOL
23	C-6	56	86	34	30	O

Sample Description: Green-Brown, Sandy Clay; firm

DYNAMIC TRIAXIAL TEST

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Figure No. C-27

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Appendix D

Gas Chromatographic and Petroleum Analyses

APPENDIX D GAS CHROMATOGRAPHIC AND PETROLEUM ANALYSES

D.1 INTRODUCTION

Both Gas Chromatographic and Petroleum analyses were performed at Boring CEG-23. Due to the close proximity of the Fairfax/Beverly Station site to the Salt Lake Oil Field, methane and other natural hydrocarbon gases may occur along the proposed station and cross-over site excavation. 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.

Laboratory analyses of petroleum samples were done by Converse's special consultant Mr. Bruce Barron, Strata-Analysts Group. Samples obtained from Boring CEG-23 were tested to identify the concentrations of oil and water and the hydrocarbon content. Identification of hydrocarbons was done using two chromatographic methods: (1) the PTC method, which generally defines compounds in the C_1 to C_8 normal hydrocarbon paraffin series, and (2) the Scot method, which generally defines compounds in the C_8 to C_{18} normal paraffin series. The PTC method could not differentiate the very heavy tar-like hydrocarbons that were present in the sample because the sample was altered.

D.2 FIELD PROGRAM FOR GAS CHROMATOGRAPHIC ANALYSIS

Specific hydrocarbon and non-hydrocarbon gases were collected during the 1981 investigation at shallow depths in Boring CEG-23. 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-23 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-23 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.

Surface and near-surface deposits of petroleum are extremely difficult to analyze because the normal hydrocarbon compounds have been appreciably altered by weathering, bacterial degradation, and contamination due to washing by water. These processes change the characteristics of the original oil. Weathering, water-washing, and/or immaturity are the most commonly accepted reasons for oils of low gravity. Bacterial degradation and/or immaturity commonly result in an absence of normal paraffins. Previous work done by oil companies on other near-surface deposits produced similar results.

No normal traces were found in the other samples, indicating that they contain immature hydrocarbon with many complex aromatic compounds and asphaltenes.

Nevertheless, we were able to group samples that were partially similar in composition (Table D-2). To determine samples that have similar compositional characteristics, the chromatograms were compared to each other and peaks were matched. Only certain peaks matched on some chromatograms; other chromatograms produced no matching peaks. The groupings do not necessarily indicate that samples in the same group came from the same oil field or that the samples in the same group have been subjected to the same developmental history.

D.5 CONCLUSIONS

The known Salt Lake Oil Field is located within the cut and cover box structure area and the chromatogram Boring CEG-23. It's proximity, as mapped, is directly underneath the station and cross-over site. The shallow borings drilled for this investigation did not encounter any of the subsurface gas. However, Boring CEG-23 drilled for the 1981 investigation and Borings 23-1, 23B, 23-2 and 23-3 during the 1983 investigation encountered oil and gas within the samples obtained between depths of 40 and 70 feet below existing ground surface. We may expect to find subsurface gas trapped within the alluvium below the ground water table in the lower portion of box excavation.

Because of the lateral migration of gases below the zones of ground 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.

Samples from Boring CEG-23 indicate immature hydrocarbons containing no normal paraffin compounds. The immature hydrocarbons may be the result of either (1) the immaturity of the oil where the normal paraffins may not have developed, or (2) alteration of the oil that destroyed the normal paraffins.

The hydrocarbons that were tested are very low gravity and could be considered tar. The normal hydrocarbons have not developed because the oil is either immature or has been appreciably altered by (1) weathering, (2) bacterial (biochemical) degradation, and (3) contamination resulting from washing by water. Consequently, the chromatograms of the tested samples could not be matched to chromatographs of standards of normal hydrocarbons. The absence of normal hydrocarbon "signs posts" does not allow a rigorous description of the types or characteristics of deeper petroleum deposits.

Because the petroleum is crude oil, it could be the source of hazardous gases. Any deposit of crude oil must be considered as a potential hazard. Faults, fissures, and similar features exist along the proposed Station and cross-over structure and may be considered as areas for accumulation of the more volatile components of the hydrocarbons.

TABLE D-1 Summary of Data from Gas Chromatograms

GAS	Lower Limit of Flammability* (ppm)	TUNNEL CLASSIFICATION** GASSY 5% LEL (ppm)	EXTRA HAZARDOUS 20% LEL (ppm)	BORING NUMBER 1 Air (ppm)	1 Cased Piezometer Sampled @ 10' Water Level @ 10' (ppm)	2 Air (ppm)	2 Cased Piezometer Sampled @ 18' Water Level @ 20' (ppm)	10 Air (ppm)	10 Cased Piezometer Sampled @ 15' Water Level @ 20' (ppm)	11 Air (ppm)	11 Cased Piezometer Sampled @ 40' Water Level @ 45' (ppm)	19 Air (ppm)	19 Cased Piezometer Sampled @ 9' Water Level @ 10' (ppm)	21 Air (ppm)	21 Refer to notes a, b, c' (ppm)	22 Air (ppm)	22 Alluvium Open Hole Sampled @ 15' Water Level @ 20' (ppm)	23 Air (ppm)	23 Cased Piezometer Sampled @ 12' Water Level @ 15' (ppm)	23A Air (ppm)	23A Cased Piezometer Sampled @ 10' Water Level @ 15' (ppm)
Methane	50,000	2,500	10,000	-	100	-	200	-	-			-	trace	-	a trace b - c -	trace	trace	-	-	-	trace
Ethane	30,000	1,500	6,000	trace†	300	-	300	-	-			-	2,000	100	a trace b trace c trace	100	1,900	-	-	150	350
Propane	21,200	1,060	4,240	-	trace	-	trace	-	-			-	-	-	a trace b - c -	-	-	-	-	trace	-
n-Butane	18,600	930	3,720	-	trace	-	trace	-	-			-	trace	-	a trace b - c -	-	trace	-	trace	trace	-
Isobutane	18,000	900	3,600	-	trace	-	trace	-	-			-	150	-	a trace b - c -	-	trace	-	trace	trace	-
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	trace	-
C_6+	‡	-	-	800	1,000	1,600	1,600	400	1,200			2,000	4,500	300	a 500 b 2,000 c trace	100	1,000	3,500	3,500	2,100	2,500
Nitrogen	-	-	-	772,000	771,000	770,000	770,000	770,000	770,000			770,000	766,000	770,000	a 770,000 b 770,000 c 770,000	770,000	770,000	769,000	763,000	763,000	768,000
Oxygen	-	-	-	200,000	200,000	200,000	200,000	201,000	200,000			200,000	199,000	201,000	a 201,000 b 200,000 c 201,000	201,000	200,000	200,000	200,000	200,000	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	28,000	28,000	28,000			28,000	28,000	28,000	a 28,000 b 27,000 c 28,000	28,000	27,000	27,000	27,000	29,000	27,000
Hydrogen sulfide	43,000	2,175	8,700	trace	trace	-	trace	-	trace			trace	trace	trace	a trace b trace c 100	trace	trace	trace	trace	trace	trace

GAS	THRESHOLD LIMIT VALUES OF SELECTED NON-HYDROCARBON GASES ppm §	ppm §§	BORING NUMBER 1 Air (ppm)	1 Cased Piezometer @ 10' (ppm)	2 Air (ppm)	2 Cased Piezometer @ 18' (ppm)	10 Air (ppm)	10 Cased Piezometer @ 15' Water Level @ 20' (ppm)	11 Air (ppm)	11 Hole (ppm)	19 Air (ppm)	19 Hole (ppm)	21 Air (ppm)	21 Hole (ppm)	22 Air (ppm)	22 Hole (ppm)	23 Air (ppm)	23 Hole (ppm)	23A Air (ppm)	23A 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	28,000	29,000	28,000	28,000			29,000	28,000	28,000	a 28,000 b 27,000 c 28,000	28,000	27,000	27,000	27,000	29,000	29,000
Hydrogen sulfide	10	10, 100; 200	trace	trace	-	trace	-	trace			trace	trace	trace	a trace b trace c 100	trace	trace	trace	trace	trace	trace
Oxygen	#	-	200,000	200,000	200,000	200,000	201,000	200,000			200,000	199,000	201,000	a 201,000 b 201,000 c 201,000	201,000	200,000	200,000	200,000	200,000	200,000

* 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 3, 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 @ 15'
b = Cased 3/4" Piezometer Sampled @ 15'; water @ 16'
c = Cased 2" Piezometer Sampled @ 50', water @ 55'

TABLE D-2 Limits of Flammability

Gas	Formula	Limits of Flammability in Air: Percent by Volume* Lower	Upper	Parts per Million Lower	Upper
Methane	CH_4	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.

ConverseWardDavisDixon
Earth Sciences Associates
Geo/Resource Consultants

Ryland-Cummings, Inc.

Gas Chromatography
Boring No. 23

Date Sampled 1/3/81 Tested by DC

Depth of Sample 12 ft water @ 15 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

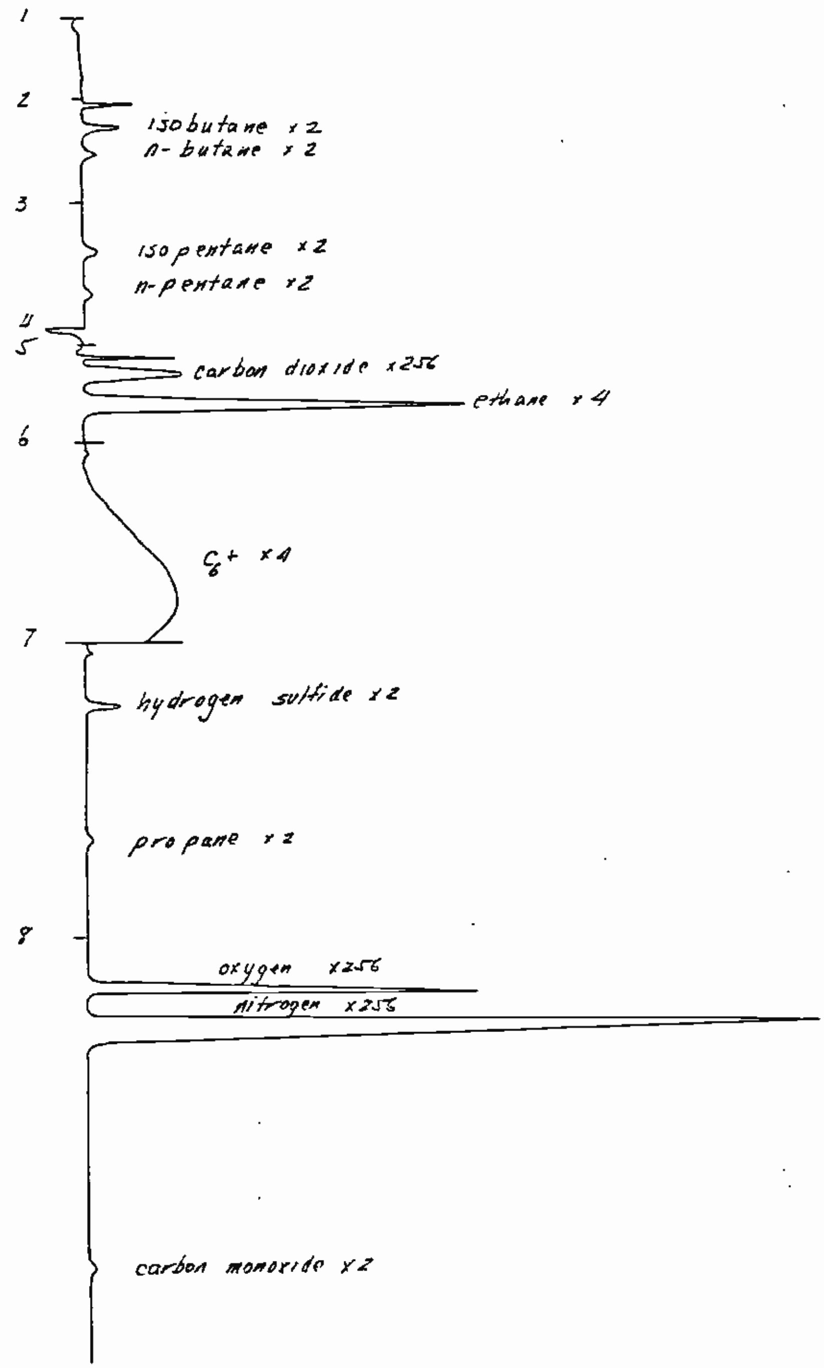


FIGURE D-1

Appendix E

Water Quality Analysis

APPENDIX E WATER QUALITY ANALYSIS

E.1 RESULTS

Water samples were taken from Borings CEG-23 during the 1981 investigation and Borings 23B during the 1983 investigation. 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 future tunneling activities. The chemical constituents tested are attached.

E.2 FIELD PROGRAM

Boring CEG-23 was flushed and established as piezometer. At a later date (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 sample was obtained from Boring 23B by hand bailer. In both cases, the water samples were collected in sterilized one-quart glass containers which were properly identified and marked in the field. The water samples were delivered to both Jacobs Laboratories and Brown and Caldwell Consulting Engineers for testing.

The test results are attached in the following two pages.

Converse Ward Davis Dixon

Lab No. P81-02-142-4

No. Samples : 7
Sampled By : Client
Brought By : Client
Date Received: 2-17-81

Sample labeled: HOLE 23-2"

Conductivity: 1,020 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	1.8	0.09
Magnesium, Mg	43	3.54
Sodium, Na	119	5.18
Potassium, K	3.8	0.10
		Total 8.91
Anions determined:		
Bicarbonate, as HCO_3	595	9.75
Chloride, Cl	74	2.09
Sulfate, SO_4	6	0.12
Fluoride, F	0.3	0.02
Nitrate, as N	0.1	0.01
		Total 11.99
Carbon dioxide, CO_2, Calc.	27	
Hardness, as $CaCO_3$	342	
Silica, SiO_2	44	
Iron, Fe	< 0.01	
Manganese, Mn	< 0.01	
Boron, B	0.22	
Total Dissolved Minerals, (by addition: HCO_3 -> CO_3)	589	

GENERAL MINERAL ANALYSIS*

BROWN AND CALDWELL
CONSULTING ENGINEERS
ANALYTICAL SERVICES DIVISION
373 SOUTH FAIR OAKS AVE.
PASADENA, CA 91105
PHONE (213) 795-7553

Log No. P83-02-105-1
Date Sampled 2/3/83
Date Received
Date Reported

Reported To: Converse Consultants
126 West Del Mar Avenue
Pasadena, CA 91105

Attn: Al Minas

cc.

Laboratory Director

Sample Description 83-1101-21 Hole 23B-8 -8.5'

Anions	Miligrams per liter	Milliequiv. per liter	Determination	Milligrams per liter	Determination	Milligrams per liter
Nitrate Nitrogen (as NO_3)	<0.1	<0.002	Hydroxide Alkalinity (as $CaCO_3$)	0.0		
Chloride	55	1.56	Carbonate Alkalinity (as $CaCO_3$)	0.0		
Sulfate (as SO_4)	11	0.24	Bicarbonate Alkalinity (as $CaCO_3$)	750		
[B]icarbonate (as HCO_3)	910	14.90	Calcium Hardness (as $CaCO_3$)	340		
Carbonate (as CO_3)	0.0	0.0	Magnesium Hardness (as $CaCO_3$)	260		
Total Milliequivalents per Liter		16.84	Total Hardness (as $CaCO_3$)	600		
Cations	**Milligrams per liter**	**Milliequiv. per liter**	Iron			
Sodium	110	4.79	Manganese			
Potassium	3.2	0.08	Copper			
Calcium	140	6.79	Zinc			
Magnesium	63	5.18	Foaming Agents (MBAS)			
Total Milliequivalents per Liter		16.84	Dissolved Residue, Evaporated @ 180°C	853		
			Specific Conductance, micromhos @ 25°C	1360	pH	7.9

*Conforms to Title 22, California Administrative Code (California Domestic Water Quality and Monitoring Regulations)

Appendix F

Technical Considerations

APPENDIX F TECHNICAL CONSIDERATIONS

F.1 SHORING PRACTICES IN THE LOS ANGELES AREA

F.1.1 General

Deep excavations for building basements in the Los Angeles area are commonly supported with soldier piles with tieback anchors. Three case studies involving deep excavations into materials similar to those anticipated at the proposed site are presented below.

F.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, and the proposed location of the 7th/Flower 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.

F.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), 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.

F.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.

F.1.5 Design Lateral Load Practices

Table F-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 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°, the equivalent design pressure should equal about 22H-psf. For hard clays, the recommended value ranges from 0.15 to .30 (equivalent rectangular distribution) times the soils unit weight or at least 18H-psf.

Thus, the local design practices are some 20% less than those indicated by Peck's rules.

TABLE F-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 Street/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.

F.2 SEISMICALLY INDUCED EARTH PRESSURES

The increase in lateral earth pressure due to earthquake forces has usually been taken into consideration by using the Monobe-Okabe method which is based on a modification of Coulomb's limit equilibrium earth pressure theory. This simple pseudo-static method has been applied to the design of retaining structures both in the U.S. and in numerous other countries around the world, mainly because it is simple to use. However, just as the use of the pseudo-static method is not really appropriate for evaluating the seismic stability of earth dams, those same shortcomings are also applicable when using the method to evaluate dynamic lateral pressures.

During an earthquake the inertia forces are cyclic in nature and are constantly changing throughout its duration. It is unrealistic to replace these inertia forces by a single horizontal (and/or vertical) force acting only in one direction. In addition, the selection of an appropriate value of the horizontal seismic coefficient is completely arbitrary. Nevertheless, the pseudo-static method is still being used since it provides a simple means for assessing the additional hazard to stability imposed by earthquake loadings.

Monobe-Okabe originally developed an expression for evaluating the magnitude of the total (static plus dynamic) active earth pressure acting on a rigid retaining wall backfilled with a dry cohesionless soil. The method was developed for dry cohesionless materials and based on the assumptions that:

- The wall yields sufficiently to produce minimum active pressures.
- When the minimum active pressure is attained, a soil wedge behind the wall is at the point of incipient failure, and the maximum shear strength is mobilized along the potential sliding surface.
- The soil behind the wall behaves as a rigid body so that accelerations are uniform throughout the mass.

Monobe-Okabe's method gives only the total force acting on the wall. It does not give the pressure distribution nor its point of application. Their formula for the total active lateral force on the wall, P_{AE}, is as follows:

$$P_{AE} = 1/2\gamma H^2(1-k_v)K_{AE}$$

Where:

$$K_{AE} = \frac{\cos^2(\phi-\theta-\beta)}{\cos\theta\cos^2\beta\cos(\delta+\beta+\theta)\left(1+\sqrt{\frac{\sin(\phi+\delta)\sin(\phi-\theta-i)}{\cos(\delta+\beta+\theta)\cos(i-\beta)}}\right)^2}$$

$\theta = \tan^{-1} \frac{Kh}{1-Kv}$

γ = unit weight of soil

ϕ = angle of internal friction of soil

i = angle of soil slope to horizontal

β = angle of wall slope to vertical

k_h = horizontal earthquake coefficient

K_v = vertical earthquake coefficient

δ = angle of wall friction.

For a horizontal ground surface and a vertical wall,

$$i = \beta = 0$$

The expression for K_{AE} then becomes,

$$KAE = \frac{\cos^2(\phi-\theta-\beta)}{\cos\theta \cos(\delta+\theta)\left(1+\frac{\sqrt{\sin(\theta+\delta)\sin(\phi-\theta)}}{\cos(\theta+\delta)}\right)^2}$$

The seismic component, ΔP_{AE}, of the total lateral load P_{AE} can be determined by the following equation:

$$\Delta P_{AE} = 1/2\ \gamma\ (\text{total})\ H^2\ \Delta K_{AE}$$

Where:

$$\Delta K_{AE} = K_{AE}\ (\text{static+seismic}) - K_{AE}\ (\text{static})$$

Inspection of actual acceleration time histories recorded during strong motion earthquakes indicates that the accelerations are quite variable both in amplitude and with time. For any given acceleration component the values fluctuate significantly during the entire duration of the record. Statistical analyses of the positive and negative peaks do indicate, however, that when one considers the entire record there are generally an equal number of positive and negative peaks of equal intensity. In the past it has been common practice to use the peak value of acceleration recorded during the earthquake as a value of engineering significance. However, this peak value might occur only once during the entire earthquake duration and is usually not representative of the average acceleration which might be established for the entire duration of shaking.

It has been common practice in the past to ignore the effects of the vertical acceleration and to set the value of the vertical earthquake coefficient, k_v, equal to zero when using Monobe-Okabe's equation. This appears reasonable in the "light" of the above discussion since the vertical acceleration will act in upward direction about as often as it will act in the downward direction. It has also been common practice to set the value of the horizontal seismic coefficient, k_h, equal to the peak ground acceleration.

This is extremely conservative since the peak acceleration acts only on the wall for an instant of time. In addition, for a deep excavation the soil mass behind the wall will not move as a rigid body and will have a seismic coefficient significantly less than the peak ground acceleration (analogous to a horizontal seismic coefficient acting on a failure surface for an earth dam).

For evaluating dynamic earth pressures for this study, we recommend that the value of the horizontal seismic coefficient be taken equal to 65% of the peak ground acceleration and that the vertical seismic coefficient, k_v, be set equal to zero.

In a saturated soil medium the change in water pressure during an earthquake has usually been established on the basis of the method of analysis originally developed by Westergaard (1933). His method of analysis was intended to apply to the hydrodynamic forces acting on the fact of a concrete dam during an earthquake. However, it was used by Matsuo and O'Hara (1960) to determine the dynamic water pressure (due to the pore fluid within the soil) acting on quay walls during earthquakes, and has been used by various other engineers for evaluating dynamic water pressures acting on retaining walls backfilled with saturated soil. Unless the soil is extremely porous, it is difficult to visualize that the pore water can actually move in and out quick enough for it to act independently of the surrounding soil media. For most natural soils, the soil and pore water would move together in phase during the duration of the earthquake such that the dynamic pressure on the wall would be due to the combined effect of the soil and water. Thus, the total weight of the saturated soil should be used in calculating dynamic earth pressure values.

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 6-2).

F.3 LIQUEFACTION EVALUATION METHODS

F.3.1 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 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 F-1.

Corrected SPT "N" values (normalized to 2 ksf overburden pressure for 11 SPT tests in saturated granular alluvium ranged from 22 to 51 with an average of about 33. Determination of dynamic strength was based on an M6.0 for the ODE event and an M7.0 for the MDE event. The liquefaction analysis based on Seed et al (1983) indicated the granular soils could withstand the ODE without initial liquefaction. However, the analyses indicated there would be liquefaction of some granular alluvium layers during the MDE event. Therefore, the granular alluvium layers are considered to have a moderate to high liquefaction potential during the MDE.

F.3.2 Shear Wave Velocity Measurements

Crosshole measurements used for the determination of seismic wave velocities along the proposed SCRTD Metro Rail Project tunnel alignment were performed as part of the initial 1981 geotechnical investigation. Downhole and crosshole surveys were performed at Borings CEG-20 and CEG-24 within adjacent Design Units A250 and A310. Average shear wave velocities measured in the Alluvium were about 1200 fps for the crosshole measurements and 1830 fps for the downhole measurements.

While shear wave velocity has not been as widely accepted in the past as SPT blow count data for estimating the liquefaction potential of a soil deposit, it has received some recent attention (Seed et al. 1983). Figure F-1 suggests that liquefaction potential at the site would be low based on the shear wave velocities measured.

F.3.3 Gradation/Plasticity Characteristics

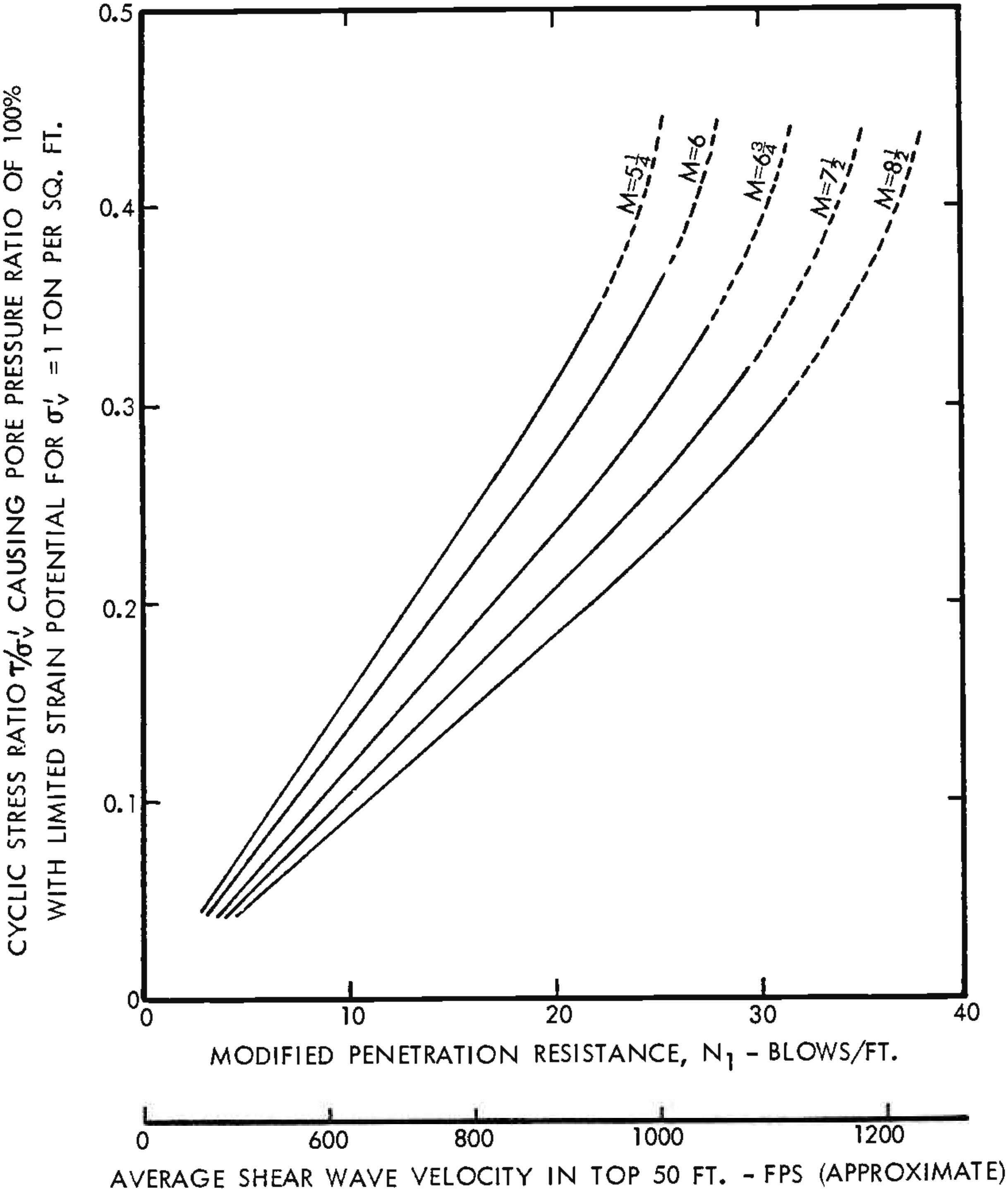
Another factor which may be considered in evaluating the liquefaction potential of a soil is the gradation characteristics of the material. A compilation of the ranges of gradational characteristics of soils which have liquefied during past earthquakes and/or are considered most susceptible to liquefaction in the laboratory is shown in Figures F-2 and F-3. The ranges shown in this figure have been compiled by Lee and Fitton (1968), Seed and Idriss (1967), Kishida (1969), and Youd (1982) and appear to indicate that the soil types most susceptible to liquefaction consist of primarily poorly graded silty sands and sandy silts.

It is important to note that all the gradational ranges shown in Figure F-2 have less than 20% by weight clay size particles (i.e., particles less than 0.005 mm), suggesting that clayey (cohesive) soils have a low liquefaction potential. Seed and Idriss (1983) stated that clayey soils are not vulnerable to significant strength loss during earthquakes if the percentage of particles finer than 0.005 mm is greater than 20 or if the water content is less than 90% of the Liquid Limit. As can be verified by Tables C-1 and C-2 of Appendix C, moisture contents of the clayey soils test are all well below 90% of the Liquid Limit moisture content, thereby indicating the clayey soils to be non-liquefiable.

The gradation characteristics of the various soils which comprise the onsite Alluvium were compiled from laboratory tests performed during this and the previous 1981 investigations. The comparisons of the gradations with the ranges of gradations of the "liquefiable" soils shown in Figure F-2 are presented in Figures F-3 and F-4. Several samples tested fall within the range of gradations of soils considered more "susceptible" to liquefaction are shown on Figures F-3 and F-4.

F.3.4 Conclusions

Based on the above considerations and comparisons, it is our judgement that the fine-grained (clayey) alluvial soil deposits would have low liquefaction potential during ground shaking from both the operating design earthquake (ODE) and the maximum design earthquake (MDE). The layers of granular alluvium within the clay soil matrix have a low potential for liquefaction during the ODE; however, these soils would likely liquefy during the MDE event. In our opinion, liquefaction of the granular alluvium would not result in catastrophic changes in the overall dynamic soil loads on the structure because most of the alluvium is fine-grained and is expected to maintain its integrity during the MDE.



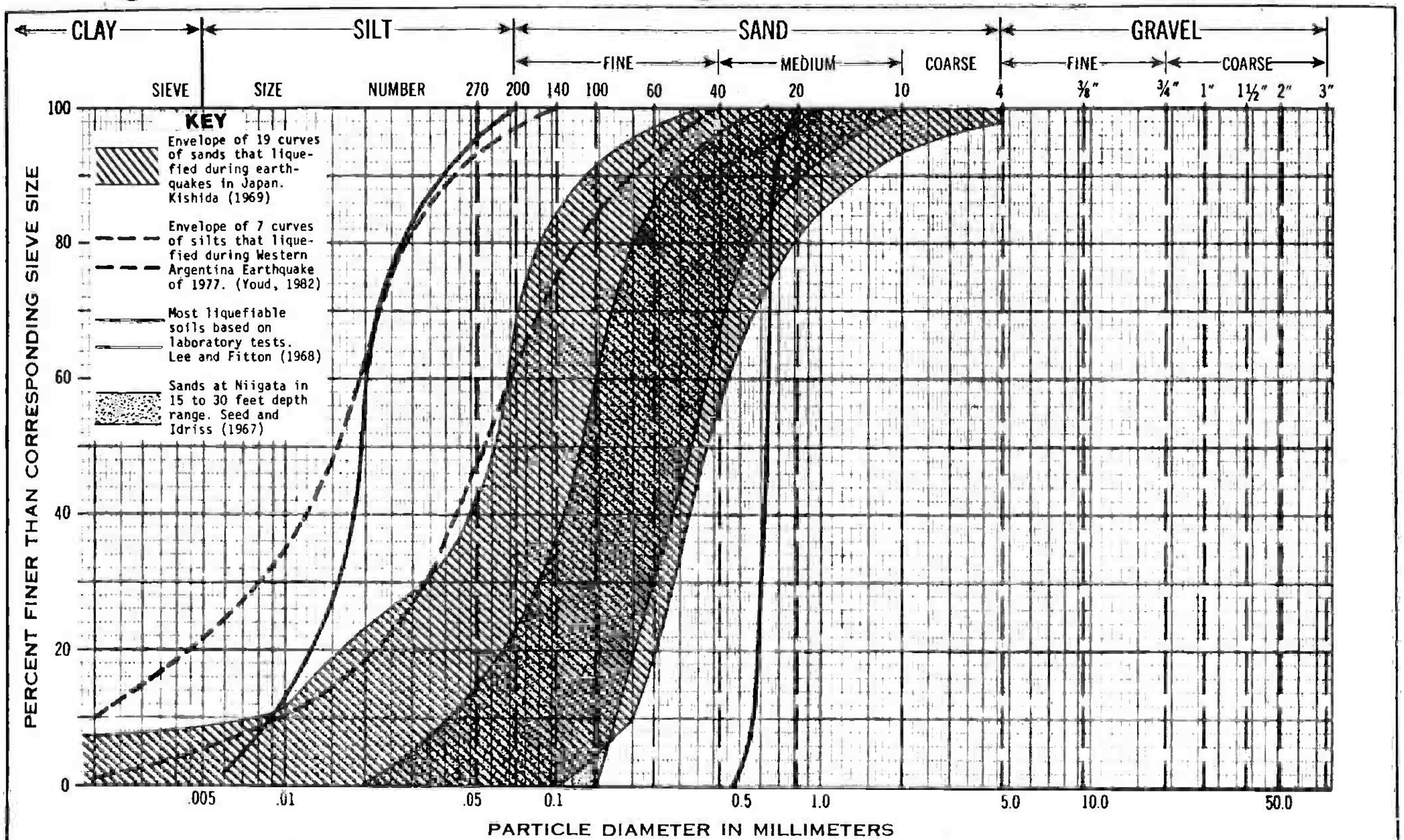
(after Seed, 1983)

CORRELATION BETWEEN PENETRATION RESISTANCE AND FIELD LIQUEFACTION BEHAVIOR OF SANDS FOR LEVEL GROUND CONDITIONS

DESIGN UNIT A275 Southern California Rapid Transit District METRO RAIL PROJECT	Project No. 83-1101
Converse Consultants Geotechnical Engineering and Applied Sciences	Figure No. F-1

Approved for publication 9/5/84 by [signature]

FORM NO. D-3-10 Approved for publication 4/84 by JAD



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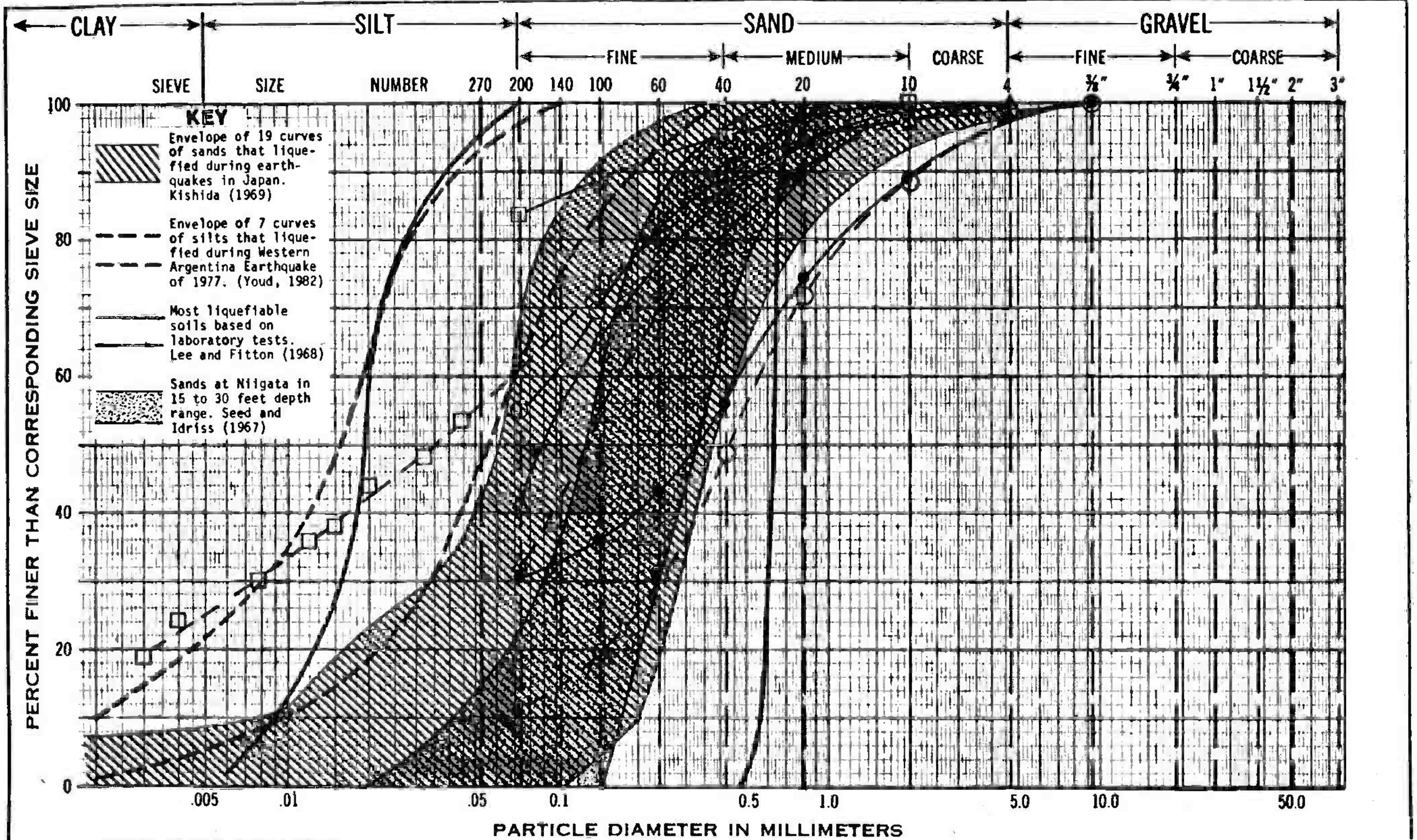
Geotechnical Engineering and Applied Sciences

Figure No. F-2

FORM NO. D-3-10 Approved for publication 4/84 by [illegible]



SYMBOL	BORING	SAMPLE	DEPTH
—○—	23/1	C-4	24'
—□—	23/1	C-6	39'
—△—	23/1	C-10	64'
- -○- -	23/2	C-3	13'
- -□- -	23/2	C-7	42'
- -△- -	23/2	C-10	63'
—●—	23/2	C-8	48'

GRAIN-SIZE DISTRIBUTION CHART

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METRO RAIL PROJECT

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Figure No. F-3

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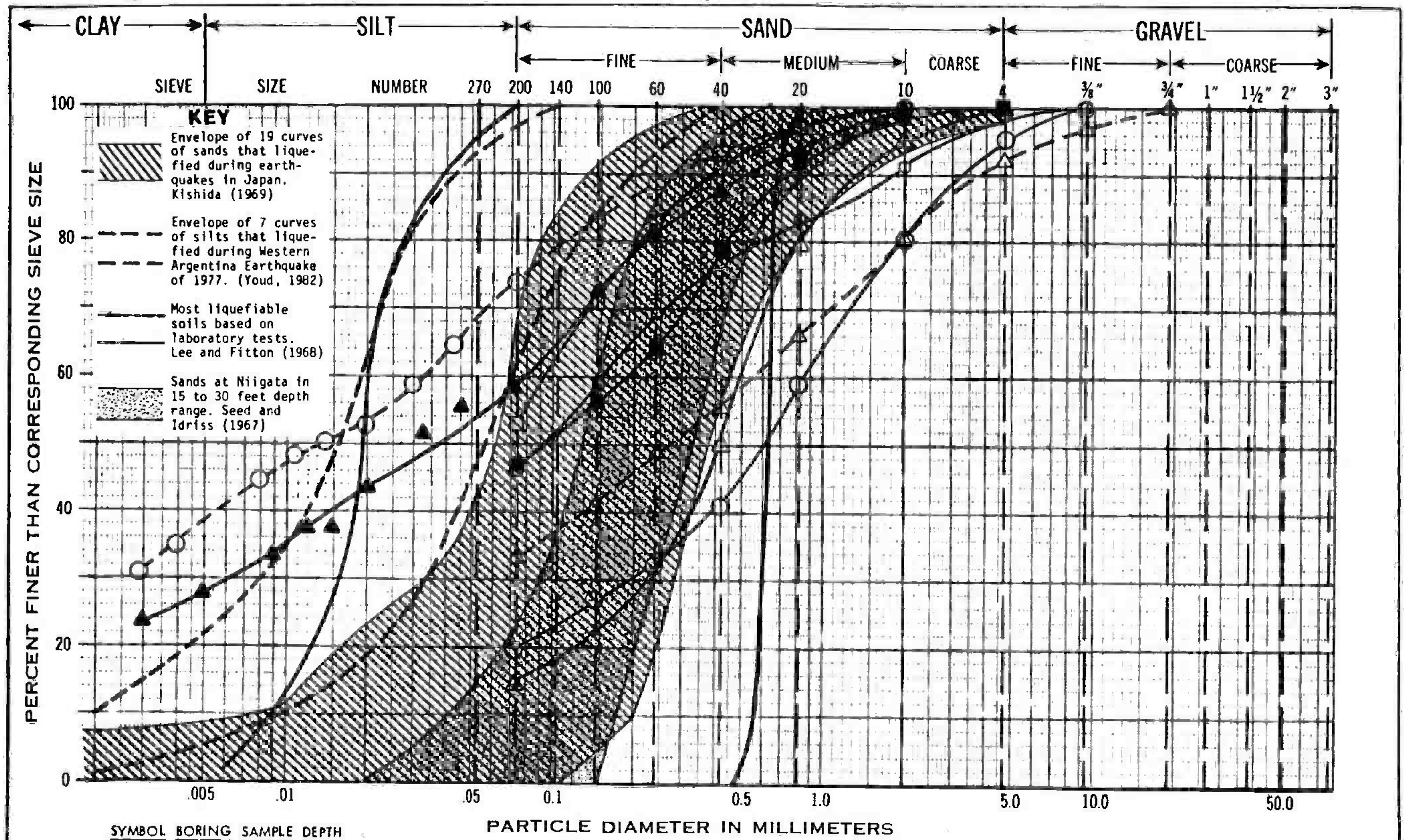

SYMBOL	BORING	SAMPLE	DEPTH
- -△- -	23/3	C-2	9'
—■—	23/3	C-6	34'
—○—	23/3	C-8	49'
—□—	23/4	C-4	39'
- -○- -	23/4	C-3	29'
—△—	23/5	C-8	45'
—○—	23/5	C-2	9'
—▲—	23/5	C-6	29'

GRAIN-SIZE DISTRIBUTION CHART

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Project No.
83-1140

Figure No.
F-4

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Appendix G

Earthwork Recommendations

APPENDIX G 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 6.2 and 6.4, 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 shored excavations are presented in Section 6.4.

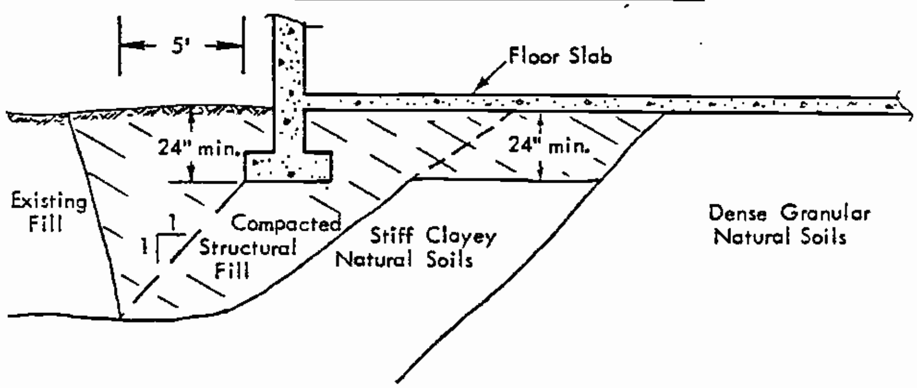
- Structural Fill and Backfill: Where required for support of near surface foundations or where subterranean walls and/or footings require backfilling, excavated onsite granular soils or imported granular soils are suitable for use as structural fill. Loose soil, formwork and debris should be removed 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.

  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., silt-clay soils.

° Foundation Preparation: Where foundations for near surface appurtenant structures are underlain by existing fill soils, 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 (a 1:1 ratio), whichever is larger. The structural fill should be placed and compacted as recommended under "Structural Fill and Backfill".

FOUNDATION/SUBGRADE PREPARATION



° Subgrade Preparation: Concrete slabs-on-grade at the subterranean levels may be supported directly on undisturbed dense materials. The subgrade should be proof rolled to detect soft or disturbed areas, and such areas should be excavated and replaced with structural fill. If existing fill soils are encountered in near surface subgrade areas, these materials should be excavated and replaced with properly compacted granular fill. Where clayey natural soils (near existing grade) are exposed in the subgrade, these soils should be excavated to a depth of 24 inches below the subgrade level and replaced with properly compacted granular fill. Where dense natural granular soils are exposed at slab subgrade, the slab may be supported directly on these soils. 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 broken down.

  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.

Appendix H

Geotechnical Reports

APPENDIX H GEOTECHNICAL REPORTS REFERENCES

REPORT No.	REPORT DATE	LOCATION	CONSULTANT
31	09/30/65	South of Wilshire, between Spaulding & Ogden	L.T. Evans
32	02/23/53	North of Wilshire between Ogden & Orange Grove	L.T. Evans
33	04/30/68	Southeast corner Wilshire/Fairfax	LeRoy Crandall
34	04/16/68	6200 Wilshire	Nilcola
35	01/02/51	CBS - southeast corner Beverly & Fairfax	L.T. Evans
36	04/24/51	CBS - southeast corner Beverly & Fairfax	L.T. Evans
37	12/04/56	CBS - southeast corner Beverly & Fairfax	L.T. Evans
38	08/28/68	CBS - southeast corner Beverly & Fairfax	L.T. Evans
39	04/15/75	CBS - southeast corner Beverly & Fairfax	L.T. Evans
40	10/22/76	CBS - southeast corner Beverly & Genese	L.T. Evans