Southern California Rapid Transit District
METRO RAIL PROJECT

GEOTECHNICAL REPORT

METRO RAIL PROJECT
DESIGN UNIT A165

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Converse Consultants
Earth Sciences Associates
Geo/Resource Consultants

GEOTECHNICAL REPORT

METRO RAIL PROJECT

DESIGN UNIT A165

BY

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

OCTOBER 1983

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

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October 11, 1983 83-1101-73

Southern California Rapid Transit District
Metro Rail Project - WBS 12AAC
425 South Main Street
Los Angeles, California 90013

Attention: Mr. James Crawley
Director of Engineering-Transit Facilities

Gentlemen:

This letter transmits our final geotechnical investigation report for Design Unit A165 7th/Flower Station prepared in accordance with the Contract No. 2256-2 agreement dated January 14, 1983 between Converse Consultants, Inc. and Southern California Rapid Transit District (SCRTD). This report provides geotechnical information and recommendations to be used by design firms in preparing designs for Design Unit A165.

Our study team appreciate the assistance provided by the District's Board of Special Geotechnical Consultants and the Metro Rail Transit Consultants; namely, Dick Proctor, Jack Yaghoubian, Bud Maduke and Bill Armento. We also want to acknowledge the dedicated efforts of each member of the Converse team, especially Bob Plum, Howard Spellman, Julio Valera, Jim Doolittle and Roger Hail.

Respectfully submitted,

Robert M. Pride, Senior Vice President
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RMP:mro

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

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

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Section 1.0

Executive Summary

1.0 EXECUTIVE SUMMARY

This report presents the results of a geotechnical investigation for Project Unit A165 which will consist of the 7th/Flower Station. This facility will be part of the proposed 18-mile long Metro Rail Project in the Los Angeles area. The purpose of the investigation is to provide geotechnical information to be used by design firms in preparing designs for the project. Although this report may be used for construction purposes, it is not intended to provide all the geotechnical information that may be required to construct the project.

The subsurface conditions (see Drawings 3 and 4) consist of Young alluvium (A_1 and A_2) overlying Fernando Formation siltstone bedrock (C). The thickness of the Young alluvium ranges from about 35 feet at the northwest end of the Station to about 50 feet near the southeast end. This variation is related to the alignment being near the margin of the Los Angeles basin. The Young alluvium consisted of an upper 15- to 20-foot thick medium dense/medium stiff fine-grained alluvium (A_2) underlain by 15 to 20 feet of dense granular alluvium (A_1). The bedrock consists primarily of clayey siltstone. In general, the static ground water table occurred within the bedrock. However, locally, areas of perched water occurred within the alluvium.

Construction of the 7th/Flower Station will involve making a 50- to 60-foot deep excavation through the alluvium and into the underlying bedrock. Due to the proximity of several large existing structures, it is our opinion that underpinning and/or construction of a stiff shoring wall will be required. The permanent ground water appears to be relatively deep and occurs within relatively impermeable bedrock. The permanent structure will bear on bedrock and retain alluvium and bedrock.

The geotechnical evaluations and design criteria presented in this report include:

- EXCAVATION DEWATERING: In our opinion, there will generally be only minor ground water inflows into the excavation during construction. Locally, zones of perched ground water may be encountered within the alluvium which will produce temporary increased inflows.

- UNDERPINNING: The report presents general guidelines for assessing the need to consider underpinning. Based on this and the proximity of existing structures, underpinning will probably be required unless "rigid" shoring is used.

- TEMPORARY EXCAVATION SHORING SYSTEM: We understand that the excavation system will be chosen and designed by the contractor in accordance with specified criteria and subject to review and acceptance by the Metro Rail Transit Consultants. Due to the existence of deep basements adjacent to the proposed Station excavation, it is unlikely that tiebacks will be used as the primary shoring support. In our opinion the contractor will most likely propose a shoring system consisting of one of the following systems with internal bracing for lateral support:

... Conventional Soldier Pile Wall: Significant buildings located within the underpinning zone would require underpinning.

... Conservative Soldier Pile Wall: This would consist of a conservatively designed and constructed soldier pile wall which would limit ground movements and may eliminate the need to underpin adjacent buildings. In general, this would consist of using higher design lateral loads and implementing several design and construction procedures intended to reduce ground movements to less than about 3/4 inch.

... Slurry Wall: Installation of a properly designed and constructed slurry wall should eliminate the need to underpin adjacent buildings. This system would also require design and construction procedures to reduce ground movements to less than about 3/4 inch.

Accordingly, the discussions and design criteria presented in the report pertain to these general shoring methods. Other systems may also be appropriate and should be considered by the contractor. The report provides technical support for the concept of the conservative soldier pile wall including a review of the performance of several shoring systems in similar ground conditions.

° EXCAVATION INSTRUMENTATION PROGRAM: In our opinion the proposed excavation should be instrumented. The recommended instrumentation program includes a preconstruction survey, surface survey control, inclinometer measurements, vertical settlement profiles, subgrade heave monitoring, convergence measurements, tieback or strut load monitoring, and testing of slurry consistency (if slurry walls are used). In our opinion it is important that the installation and measurements of the instrumentation devices be under the direction and control of the Engineer.

° FOUNDATIONS: The Station structure can be adequately supported on the bedrock and dense alluvium expected to be exposed at the foundation elevation. The report also presents allowable footing bearing pressures, and estimates of foundation settlements for support of surface structures.

° PERMANENT GROUND WATER PROVISIONS: Selected design ground water levels extend to within about 20 feet of the ground surface at the northwest end of the structure. We understand that the Station will probably be made water tight below the maximum anticipated hydrostatic pressures. An alternative would involve providing an underdrain system around and below the Station. In our opinion, a drainage system would be geotechnically feasible since the ground water inflows and pumping rates are expected to be small.

° LOADS ON SLABS AND WALLS: The report presents recommended lateral design loads on the permanent structures.

° LIQUEFACTION POTENTIAL: Based on existing data, the site is not expected to have an extensive thickness of saturated granular soils, since measured perched water levels were near the bedrock surface. However, liquefaction of granular soils could affect foundation support at the southeast end of the structure as well as lateral wall pressures and shallow entrance structures. Thus, a simplified liquefaction analysis was performed. Based on the results of the analysis, and our engineering judgement, it is our opinion that the site would not be subject to liquefaction during ground shaking produced by the postulated earthquake motions.

Section 2.0

Introduction

2.0 INTRODUCTION

This report presents the results of a geotechnical investigation for Design Unit A165 which will consist of the 7th/Flower Station. This facility will be part of the proposed 18-mile long Metro Rail Project (see Drawing 1, Vicinity Map). The purpose of the investigation is to provide geotechnical information to be used by the design firms in preparing designs of the project. Although this report may be used for construction purposes, it is not intended to provide all the geotechnical information that may be required to construct the project. The work performed for this study included borings, laboratory testing, engineering analyses, and development of geotechnical recommendations.

Additional geotechnical information on the project is included in the following reports:

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

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

- "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 boring 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.

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

The design concepts evaluated in this geotechnical report are based on the "Draft Report for the Development of Milestone 10: Fixed Facilities: dated March 1983 and revised plans A-18 through A-20 dated April 28, 1983. These documents were prepared for SCRTD by Harry Weese & Associates, Tippetts-Abbett-McCarthy-Stratton, Environmental Collaborative, Inc. and Gin Wong Associates.

Section 3.0

Project and Site Description

3.0 PROJECT AND SITE DESCRIPTION

3.1 SITE DESCRIPTION

The proposed 7th/Flower Station structure, as shown on Drawings 1, 2 and 3, will be located within 7th Street, between Hope and Figueroa streets. The Station will extend through the intersection of 7th and Flower streets. Land use in the area includes high-rise office towers, street-level retail and commercial space, department stores, and restaurants. Seventh Street is a major auto, bus and pedestrian artery through the Central Business District. The immediate area contains no undeveloped land, with the exception of the southwest corner of Figueroa and 7th Street. This site is the location of the proposed Pacific Plaza Project which will contain over 3 million square feet of office and commercial/retail space.

Several large existing buildings are located within 10 feet of the proposed Station. We understand that these buildings are supported on spread footings and have relatively deep basement structures. The major structures include:

- The Broadway Plaza Complex: a 33-story office complex and a 24-story hotel with about a 40-foot deep basement.
- The Barker Brothers Building: includes about a 30-foot deep basement.
- The Roosevelt Building: includes about a 25-foot deep basement.
- The Global Marine Building: includes about a 20-foot deep basement.

The surface topography slopes gently to the southeast with the slope increasing somewhat at the east end of the proposed Station. The surface elevation ranges from about Elevation 275 at the northwest end to Elevation 268 at the southeast end. Area vegetation is limited to trees planted in the sidewalk along 7th Street.

3.2 PROPOSED STATION

The proposed Station structure will be about 640 feet long, 65 feet wide and 40 feet high. The Station will include two surface entry structures and an underground entry from the Broadway Plaza Complex. The proposed top of rail is at an elevation of about 224. Assuming that the bottom of slab will be about 4 to 6 feet thick, construction of the Station will require an excavation extending to about Elevation 219. This will be about a 50- to 60-foot deep excavation.

It is understood that the permanent structure will be designed as a rigid reinforced concrete box with one row of interior columns located along the longitudinal centerline of the Station. The roof slab will support about 7 to 15 feet of fill.

Assuming that the 7th Street cannot be closed entirely to vehicular traffic, the planned construction sequence may include:

- Temporarily closing one side of 7th Street;
- Installing a shoring system, and center street decking support piles;
- Excavating some 10 feet and placing a concrete or wooden decking system as a temporary street;
- Moving the traffic to the decked side, installing shoring on the other side and excavating;
- Decking over the second side of the street and completing the excavation;
- Constructing the permanent structure, backfilling, removing the decking, and reconstructing 7th Street.

Section 4.0

Field Exploration and Laboratory Testing

4.0 FIELD EXPLORATION AND LABORATORY TESTING

4.1 GENERAL

The conclusions and recommendations presented in this report are based primarily on the field and laboratory investigations performed in 1983 for this study and those performed in 1981 for the initial Geotechnical Investigation Report. In addition, the subsurface data compiled in 1977 by the USGS and by Kaiser Engineers in 1962 were reviewed. A comprehensive research of all existing boring data in the vicinity of the Project was beyond the scope of this report. Thus there may be additional boring data developed for specific buildings adjacent to the Station which may be reviewed by the designer and contractor.

In general the field and laboratory geotechnical studies included borings, ground water observation wells, field gas measurements, water quality laboratory tests, and soil/rock laboratory tests.

4.2 BORINGS

The 1983 field exploration included four borings (9-1 to 9-4) each drilled to about 85 feet. The 1981 exploration program in this Section included the 200-foot deep Boring CEG-9. In addition, the USGS (Yerkes) identified four borings in the area. Ground water observation wells were installed in Borings CEG-9, 9-1 and 9-4. Observation wells generally consisted of a perforated section within the lower 50 feet of the boring with a gravel backfill placed to the surface seal. Section 5.4 presents a summary of ground water level measurements obtained from the observation wells. Detailed descriptions of the field procedures for both the 1981 and 1983 boring programs are presented in Appendix A.

4.3 WATER QUALITY ANALYSES

Water quality analyses were performed on water samples taken from boring CEG-9. The results are presented in Appendix C. The water was tested for basic cations, anions, conductivity, total dissolved solids, pH, turbidity and boron.

4.4 GEOTECHNICAL LABORATORY TESTING

A laboratory testing program was performed on representative soil and rock samples. These consisted of classification tests, consolidation tests, triaxial compression tests, resonant column tests, unconfined compression tests, direct shear tests, and permeability tests.

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

Section 5.0

Subsurface Conditions

5.0 SUBSURFACE CONDITIONS

5.1 GENERAL

Based on the field and laboratory data presented in Appendices A, B and D, the geologic sequence in the site area (see Drawings 2 and 4) consists of artificial fill, granular Young alluvium (A_1), fine-grained Young alluvium (A_2), Old alluvium (A_3 and A_4) and bedrock of the Fernando Formation (C). The geologic units include:

- Artificial fill: This includes man-made fills placed for construction of existing structures and streets.

- Young alluvium (fine-grained) - A_2: This deposit is also of Holocene age but was deposited in relatively "quiet" water. The unit normally consists of silts and clayey silts. The unit was often found interbedded with the granular alluvium.

- Old alluvium A_3 and A_4: These deposits are of Pleistocene age. At this location these materials consist of sand-clay mixtures ranging from clayey sand to sandy clay. Consistency is dense to very dense and stiff to very stiff.

- Young alluvium (granular) - A_1: This deposit, which is of Holocene age, is primarily sands and gravels deposited in swift-flowing streams. Locally, the unit may contain boulders up to 2 to 3 feet in diameter.

- Fernando Formation - C: The bedrock underlying the proposed Station is Pliocene age and composed of well stratified, gently dipping, weak siltstones and claystones. Local hard beds or nodules ranging from less than 1 inch to more than 3 feet thick may be encountered. It is estimated that these hard zones comprise less than 1% of the Formation.

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

Drawings 2 and 4 show subsurface profiles and cross-sections through the site. The thickness of the Young alluvium ranges from about 35 feet at the northwest side of the Station to about 50 feet at the southeast side. This variation is related to the alignment being near the margin of the Los Angeles basin. In general, the static ground water table was observed to occur within the bedrock. However local areas of perched water may exist within the alluvium.

5.2 SUBSOILS

Specific descriptions of the soil units encountered in the borings include:

- Fill: Although not positively identified in our borings, areas adjacent to existing buildings undoubtedly are underlain by fill within the zone of the basement backfill. It is believed that the fill is probably variable including both granular and fine-grained soils with possible building debris.

- Upper fine-grained alluvium (A_2 and A_4): This upper unit was 15 to 20 feet thick and included interbedded silty sands, clayey sands, and silty clays. The Standard Penetration Resistance during sampling ranged from about 15 to 30 blows per feet. Based on these data and the laboratory densities, we believe that the soil is generally medium dense or medium stiff. Although classified as alluvium in our boring logs, some of the materials encountered may be fill.

- Granular alluvium (A_1 and A_3): All the borings encountered primarily dense sand and gravel below a depth of about 15 to 20 feet and extending to the top of rock. In Boring 9-1, the material was a clayey sand and gravel with as much as 20% fines. In the other borings, the unit was primarily clean sands and gravels. At a depth of about 25 to 35 feet, Borings CEG-9 and 9-3 encountered about 10 feet of interbedded dense/stiff sands, silty sands, sandy silts, and silty clays within the overall sand and gravel formation.

5.3 BEDROCK (C)

The bedrock encountered in the borings consisted predominantly of weak clayey siltstones. The bedrock contained occasional zones of concretions and concentrated shells up to 3 inches thick. Scattered shells and shell fragments occurred throughout the rock. The bedding observed in the boring samples was massive and with observed bedding dipping from about 5-degrees to 30-degrees (to the horizontal). Based on local geologic conditions, it is believed that the bedding strikes east-west and dips to the south.

5.4 GROUND WATER AND GAS CONDITIONS

The site lies within the Los Angeles forebay area hydrologic unit which is part of the Central ground water basin. The term "forebay" refers to a recharge area where substantial infiltration of surface water can occur. Ground water occurs both in the alluvial deposits and within the sedimentary bedrock of the Puente and Fernando Formations. However, in most locations in the forebay area, ground water levels within the bedrock are 50 to 100 feet or more below the bedrock surface. This indicates that ground water within the alluvium is "perched" over the bedrock surface. This conclusion has been verified by deep excavation into the Puente and Fernando Formations in the Los Angeles forebay area. Water can, however, occur in structural discontinuities within the bedrock such as joints, fractures, etc.

Ground water levels measured in Borings CEG-9, 9-1 and 9-4 are presented below. Based on regional ground water data (Los Angeles County Flood Control, 1975), the static, continuous ground water table appears to occur within the bedrock some 50 to 100 feet below the bedrock surface. Ground water levels in Borings CEG-9 and 9-4 were measured slightly below the top of the bedrock surface. However, these piezometers did not include a seal between the alluvium and the bedrock. Thus the readings may reflect some infiltration of perched water from the alluvium to the relatively impermeable bedrock. Ground water measured in Boring 9-1 was some 5 feet above the top of the bedrock within a clayey sand and gravel unit. In our opinion, this ground water represents a "perched" ground water condition caused by infiltration being trapped in the relatively fine-grained clayey sand and gravel unit. Most of the borings drilled for the adjacent Broadway Plaza did not encounter ground water. The borings that did encounter ground water indicated seepage within the alluvium slightly above the bedrock surface. The alluvial interval between the top of the perched water table where it occurs and the top of bedrock, is considered saturated. The interval from the top of bedrock to the permanent ground water level is judged to be near saturation but not submerged.

GROUND WATER ELEVATIONS (ft*)

BORING	3/07/81	6/17/81	4/28/82	4/04/83	4/27/83	6/08/83	9/02/83
CEG-9	216	215	214				216
9-1				245	245	245	244
9-4				204	202	201	205

*Rounded to the nearest foot.

The Union Station Oil Field and Los Angeles City Oil Field are located 3000 feet south and north, respectively, from the alignment. Little is known about these oil fields, but it apparently does produce from bedrock formations at shallow depths. Hydrogen sulfide and hydrocarbon odors were detected (subjective observations without measurements) in all the borings except Boring CEG-9. However, lateral migration from the oil field into the proposed Station excavations is a distinct likelihood, and therefore based on these data it is our opinion that the site should be considered gassy. It is understood that gas monitoring facilities have been installed along the alignment by Engineering Sciences Inc. The report describing the results of the gas monitoring should be consulted by interested parties for a more detailed view of gas conditions in this area.

5.5 ENGINEERING PROPERTIES OF SUBSURFACE MATERIALS

5.5.1 General

For purposes of our engineering evaluations and development of design recommendations, the geologic units at the site were grouped as fill, upper fine-grained alluvium, lower granular alluvium and Fernando Formation bedrock. This section includes engineering descriptions of each geologic unit and, in Table 5-1, presents the engineering parameters used in our analyses. These parameters are based on the laboratory test results (Appendix D), field test results (Appendices A and B), data from previous investigations, and published data of observed and recorded field behavior on recent construction projects. Therefore, the parameters are based on factual data and engineering judgement.

TABLE 5-1
MATERIAL PROPERTIES SELECTED FOR STATIC DESIGN

MATERIAL PROPERTY	FILL	FINE-GRAINED ALLUVIUM	GRANULAR ALLUVIUM	FERNANDO BEDROCK
Moist Density Above Ground Water (pcf)	125	125	125	120
Saturated Density (pcf)	-	-	132	120
Effective Stress Strength				
ϕ' (degrees)	25	-	37	35
c' (psf)	0	-	0	0
Total Stress Strength**				
ϕ (degrees)	-	15	-	10
c (psf)	-	1000	-	5000
Unconfined Compressive Strength (psf)	-	-	-	10,000
Permeability (cm/sec)	-	10^{-3} to 10^{-6}	10^{-2} to 10^{-5}	10^{-6} to 10^{-7}
Initial Tangent Modulus (psf)		3×10^5	$250 \cdot \sigma_v'$*	2×10^6
Poissons Ratio (dry)	-	0.4	0.35	0.35

* 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) was used for the effective density of soils below the water table.

** The total stress strength was used in undrained strength analyses where ϕ=0° with the total stress friction angle used to determine the increase in undrained strength with depth.

5.5.2 Fill

Fill soils are expected to be variable and will probably include soil types such as sands, silts and clays with occasional building debris. Due to the expected variability and possible occurrences of soft/loose zones, relatively conservative strength parameters were used. In general, it was felt that drained (effective strength) properties should be utilized in design since these materials are generally above the water table and only partially saturated.

5.5.3 Alluvium (upper fine-grained)

The upper fine-grained soils were generally classified as silty clay, interbeded with sandy clay, clayey sand and silty sand. Consistency of these materials is considered to be firm to stiff and medium dense to dense based on sampling resistance, density and strength measurements. Compressibility is considered to be low based on the soil stiffness. Direct shear test data obtained for fine-grained soils at this Station and on similar soils from other sections of the Rail Project are presented in Figure D-1. All samples tested were allowed to consolidate prior to rapid shearing. The test conditions were assumed to represent consolidated-undrained strength. Undrained strength was judged to be appropriate and generally conservative for these materials assuming little or no overconsolidation. Based on the estimated average consolidation pressure of the fine-grained alluvial materials at the site, an average undrained strength was selected for use in ϕ=0° analyses.

Elastic constants for these materials were based primarily on published data and engineering judgement.

5.5.4 Alluvium (lower granular)

The lower granular alluvium encountered consisted primarily of silt-sand-gravel mixtures, and these materials were generally classified as clayey sand, gravelly sand and sandy gravel. Standard Penetration Test (SPT) results and laboratory densities indicate the lower sands are dense to very dense. Most of these materials lie above the water table; however, the lower portions of this unit are near or below the water table. Permeability of the sands is expected to vary significantly between the clayey sand materials (10^{-5} cm/sec) and gravelly layers (10^{-2}) which may be encountered.

Strength tests performed on the sand soils included both direct shear and triaxial compression tests. Considering that this unit is primarily high permeability gravels and sands, drained (effective) strength parameters were considered appropriate for static design. Effective strength data for this unit are summarized in Figure D-2 of Appendix D. Figure D-2 also includes test data obtained on similar soils from other Sections of the Rail Project.

Elastic properties for the granular alluvium were based on the laboratory triaxial and consolidation (oedometer) tests combined with published data and engineering judgement. Modulus data on soils samples from this Project Section and similar soil samples from other Rail Sections are summarized in Figure D-3 of Appendix D. The data indicate that the modulus increases linearly with confining pressure. This characteristic is consistent with published data. The modulus value is presented in terms of the effective overburden pressure.

5.5.5 Fernando Formation Bedrock

The Fernando Formation claystone and siltstone was considered to be a very stiff to hard overconsolidated fine-grained soil for the purpose of our engineering evaluations. Based on high stress consolidation tests (Appendix D), maximum past consolidation pressure may be on the order of 100 ksf.

Due to the overconsolidated nature of the bedrock materials and the various loading conditions, both the drained (effective) and undrained (total) strength parameters were considered in developing design recommendations. Strength parameters presented in Table 5-1 were intended to represent the relatively fresh bedrock encountered about 5 feet below the bedrock surface and were based on interpretation of triaxial, unconfined compression and direct shear tests combined with our engineering judgement. Figures D-4 and D-5 in Appendix D summarize both effective stress and total stress laboratory strength data on samples of bedrock obtained from the 7th/Flower Station and other Sections of the Rail Project. The total stress data on Figure D-4 indicate a relatively high undrained friction angle. However, experience and concepts of soil mechanics predict that the undrained strength of the bedrock should approach that of a pure cohesive material. Published data indicate consolidated undrained c/p (cohesion/vertical stress) ratios range from about 0.15 to 0.25 even for normally consolidated low plasticity fine-grained soils. The undrained friction angle of 10-degrees is intended to reflect some increased undrained strength with depth and corresponds to a c/p of about 0.2. Thus these undrained parameters were used in undrained analyses where $\phi=0°$ but undrained strength was assumed to increased with depth.

Bedrock elastic properties were selected based on consideration of field performance data, laboratory test data and published information combined with engineering judgement. Figure D-6 in Appendix D summarizes the bedrock modulus data for samples from this Design Unit as well as samples from other Design Units. For this investigation, the highly overconsolidated bedrock material was considered to have no significant modulus increase within the range of depth affected by the proposed station. The apparent variation of modulus values at low confining pressures indicated by the laboratory data may be due to several factors including the effects of sample disturbance and sample expansion after in situ stresses were removed. Very little data on in situ modulus of the Puente/Fernando formation bedrock is available. Heave monitoring data for an excavation on the order of 50 feet deep at the Equitable Life Building, 3435 Wilshire Boulevard, (Evans, 1968) were obtained and evaluated to determine the average bedrock modulus consistent with the observed heave. The selected constant modulus value presented in Table 5-1 is consistent with the observed bedrock heave and laboratory measurements at higher confining pressures.

Section 6.0

Geotechnical Evaluations and Design Criteria

6.0 GEOTECHNICAL EVALUATIONS AND DESIGN CRITERIA

6.1 GENERAL

In general terms, construction of the 7th/Flower Station will involve making a 50- to 60-foot deep excavation through Recent Alluvium and into soft bedrock. Due to the proximity of several large existing structures, underpinning and/or construction of a stiff shoring wall will be required. The ground water appears to be relatively deep, and occurs within the relatively impermeable bedrock. The permanent reinforced concrete structure will bear on bedrock and retain bedrock and alluvium.

The primary geotechnical considerations at the 7th/Flower site include:

° Design and construction of the temporary shoring system;

° Determining the need for and type of underpinning; this depends strongly on the type of shoring;

° Recommendations for soil and water loads on permanent structures;

° Earthquake design criteria.

6.2 EXCAVATION DEWATERING

As discussed in Section 5.4, the regional ground water table is believed to occur deep within the bedrock below the Alluvium. Boring 9-1 did encounter ground water within a clayey sand and gravel alluvium layer. However, we believe that this does not represent a continuous condition but a zone of perched ground water.

Based on existing data, it is our opinion that there will generally be only minor ground water inflows during construction. We believe that these inflows could be handled with sumps from within the excavation. Considering the generally minor inflows expected, subsidence due to ground water lowering during excavation is expected to be insignificant. Although unlikely, local zones of trapped perched ground water could result in a temporary large inflow of ground water within the alluvium. Use of a slurry wall would eliminate this potential problem.

High, temporary inflows could be a potentially serious problem if soldier piles are used in areas adjacent to existing structures. A possible solution might include installation of wellpoints during the excavation in areas where wet conditions are encountered. The wellpoints could be installed within the excavation between soldier piles to dewater specific zones. Once the excavation extends to the level of the well points, they could be removed. The contractor should submit, in writing, his planned method of resolving this problem should it occur.

As indicated in Section 5.4, hydrogen sulfide and hydrocarbon odors were detected (smelled) in the borings. It is possible that gas production could occur during excavation and dewatering. In addition, water quality results presented in Appendix C should be reviewed by the contractor.

The contractor should be prepared to deal with potential operational and environmental problems associated with ground water quality and/or gas production during excavation and dewatering. Water quality analysis results must be submitted to the Regional Water Quality Control Board for evaluation. A permit may be required prior to discharging water to the storm drain systems.

6.3 UNDERPINNING

6.3.1 General

The need to underpin and the appropriate type of underpinning for specific buildings located adjacent to the proposed excavation depend on many factors which cannot be generalized. Thus, each structure must be evaluated separately.

Figure 6-1 presents guidelines for assessing when underpinning needs to be considered. Based on this figure, underpinning of several existing structures may be required if a conventional shoring wall is installed.

A relatively rigid shoring wall could be constructed which may sufficiently minimize ground movements to eliminate the need to underpin. Section 6.4 presents recommendations for these types of walls which include a slurry wall and a conservative soldier pile wall.

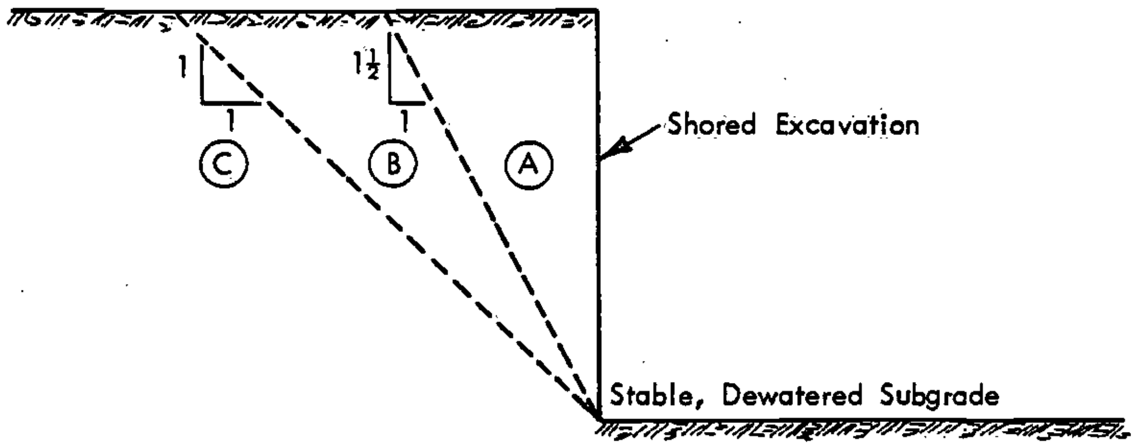
Several methods for underpinning are commonly used in the Los Angeles area. These include jacked piles, slant drilled piers, and hand-dug pit or pier underpinning. Another technique used at BART was the "column pick-up" method which provided a means of jacking up selected columns in the event settlements did occur.

6.3.2 Common Underpinning/Support Methods

Several underpinning/support methods are considered feasible including:

- Jacked Piles: These piles generally consist of H-sections or open end pipe piles 6 to 18 inches in diameter. These sections are normally preferred due to their relatively low volume of displacement which facilitates placement. Open end pipe sections also have the advantage of permitting clean-out to reduce end bearing and shaft resistance during installation. The piles are placed in 4- to 5-foot long sections by jacking against the underpinned footing. Jacked piles are commonly pre-loaded individually to 150% of the design load and then locked off at the design load.

- Slant Drilled Piles: This consists of placing a steel pile in a shaft (generally 12 to 24 inches in diameter) drilled from the side of the foundation. The shaft is drilled at a small angle or slant under the foundation. The actual connection to the foundation is accomplished by excavating a vertical slot below the foundation and placing a steel pile



NOTES: 1.) These guidelines are applicable only for stiff or dense stable ground conditions.
2.) For structure foundations bearing in zones A, B or C, the following guidelines are presented:

ZONE (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.

ZONE (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.

ZONE (C) No Special Provisions

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UNDERPINNING GUIDELINES

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Figure No. 6-1

under the foundation. The connection to the footing can be made by shimming or using "dry pack" concrete. Pre-loading could be accomplished using jacks and shims similar to jacked piles. In weak soils or in ground subject to sloughing this method could result in settlement if there is loss of ground into the drilled hole.

- Hand-dug Pits: This method consists of excavating an approach pit beneath the footing and advancing a square or rectangular shaft, normally 3 to 5 feet wide, down to the bearing stratum. The pier excavation is lagged for the entire depth with the lagging normally left in place. Reinforcement is placed in the shaft, and concrete is tremied in place. Prestressing can be provided by jacking and grout packing.

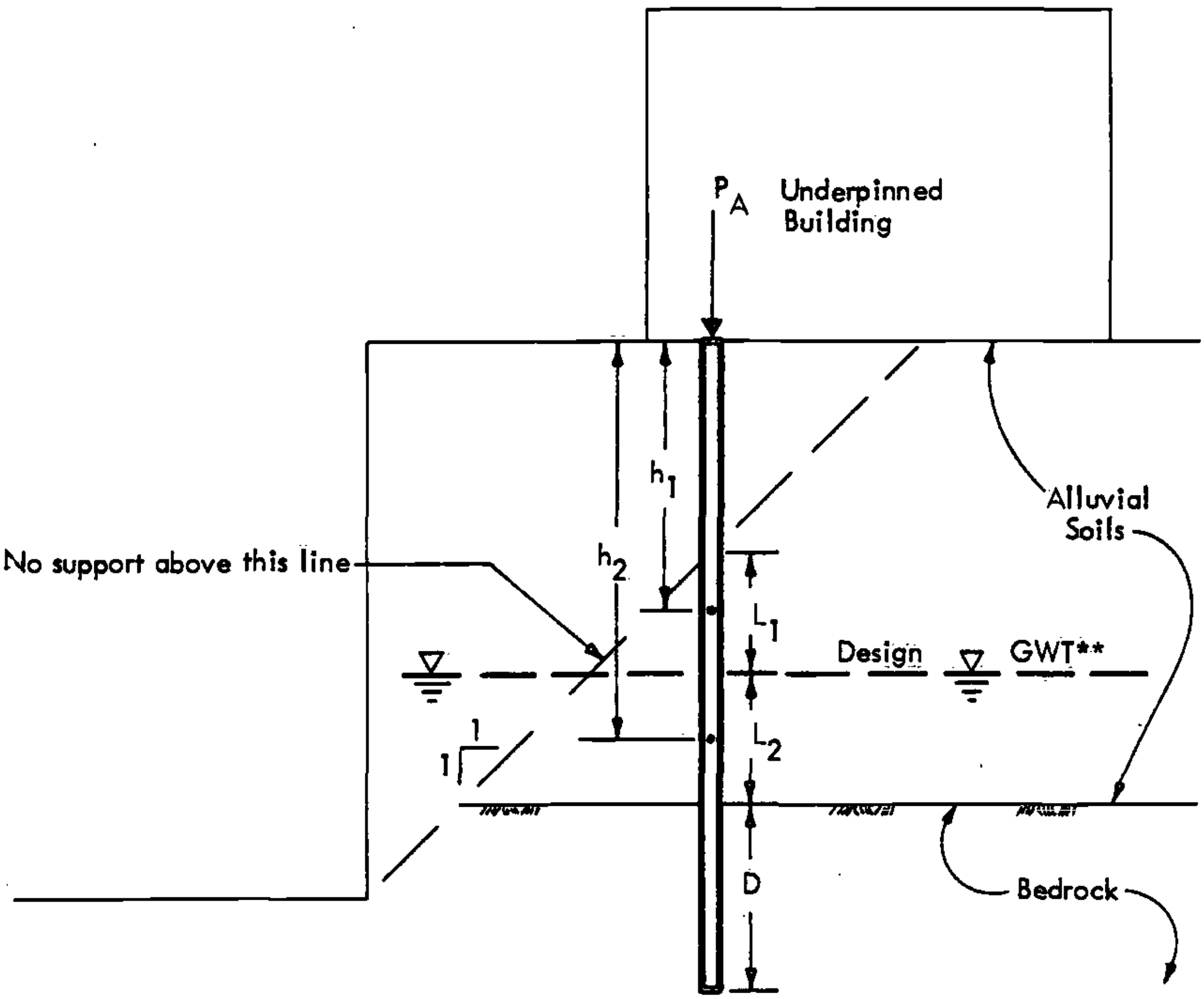
- Column Pick-up: This technique provides a method of releveling specific columns within an adjacent structure without underpinning. A structural break is made between the column or wall and a special collar is used to transmit the load between the footing and the building. If settlement occurs, a jack is used to relevel the column or wall. After completion of the excavation, a permanent connection between the building and foundation is re-established. Since this method does not transfer foundation loads to a lower stratum, both shoring and permanent walls must be designed for surcharge loads imposed by the existing structure.

All of the support methods discussed have their advantages and disadvantages; however, from the structural standpoint the jacked piles have a distinct advantage over slant drilled piles and hand-dug pits since the pile can be easily prestressed. With the other types of underpinning, settlement can occur when the load is transferred from the original foundation to the unloaded underpinning element unless prestressing is implemented.

6.3.3 Design Criteria

Figures 6-3 and 6-4 present geotechnical design criteria for jacked piles and (slant) drilled cast-in-place piles. Figure 6-3 applies to jacked pipe piles which are cleaned out and subsequently concrete filled. Figure 6-4 applies to piles constructed by drilling a vertical shaft to the required bearing depth and filling with concrete (with steel reinforcement or W/H-pile Sections). Figure 6-2 illustrates the procedures for determining the geometry of the support zones required to use Figures 6-3 and 6-4. No support should be allowed within the existing fill soils or within the theoretical zone of influence of the excavation as shown on Figure 6-2.

If jetting (or other methods which remove soil ahead of the pile) is used, no shaft frictional resistance should be allowed for jacked piles. Jetting must not be used for the final 5 feet of penetration to ensure proper bearing. Group action of piles or piers should be considered and an appropriate reduction factor applied to determine the effective group capacity. Appropriate reduction factors are presented in the Los Angeles City Building Code Section 91.2808b.



$$P_A = p_1 \cdot L_1 + p_2 \cdot L_2 + P_r$$

WHERE* : p_1 = average frictional resistance at h_1

p_2 = average frictional resistance at h_2

P_r = allowable bedrock support for penetration D

* See Figures 6-3 and 6-4 for values of p_1, p_2, and P_r

**See Section 6.11 for Design GWT Elevation

UNDERPINNING - DESIGN CAPACITY CRITERIA

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Figure No. 6-2

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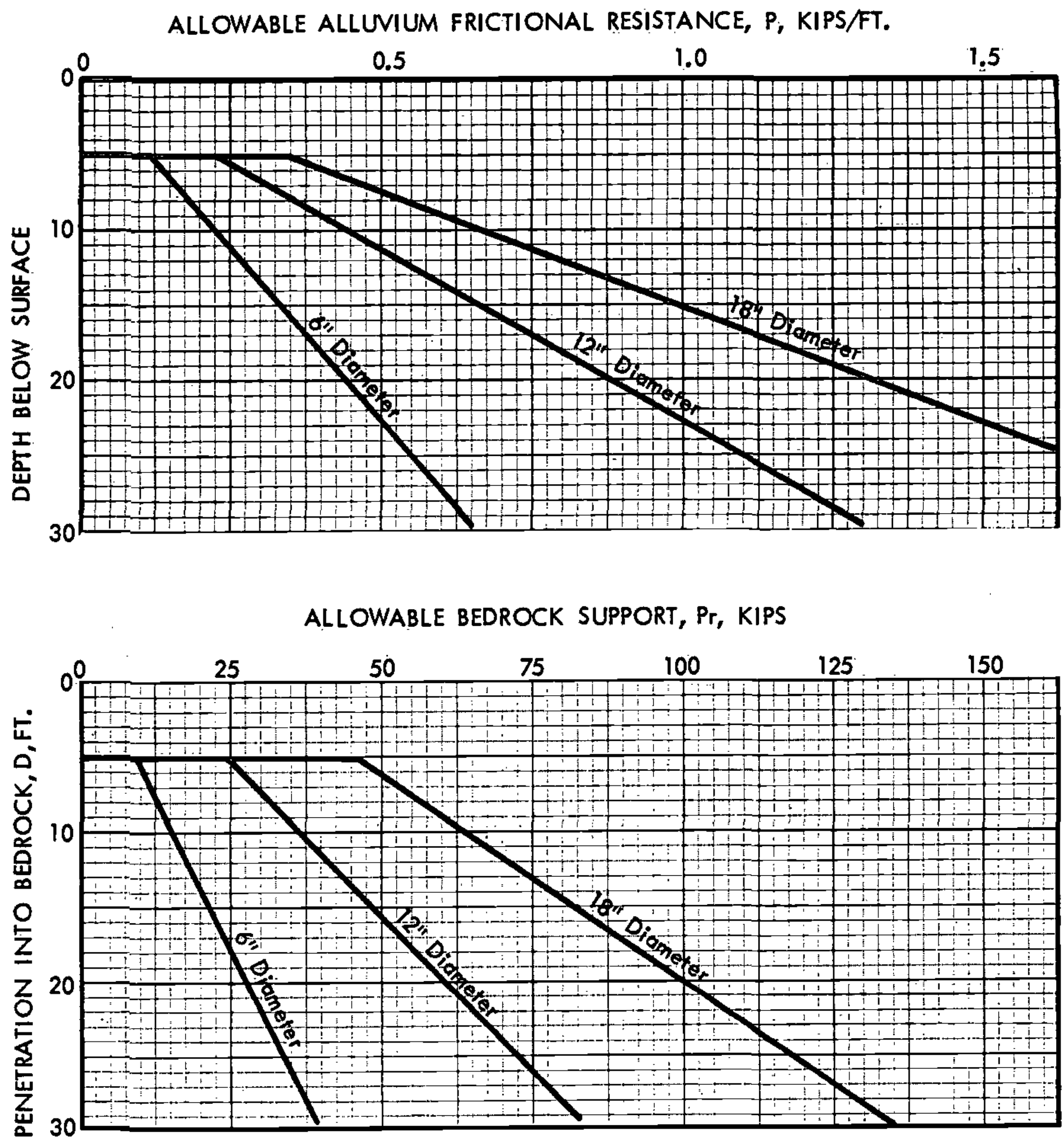

See Figure No. 6-2 for Determination of Total Capacity

UNDERPINNING - JACKED PILE DESIGN PARAMETERS

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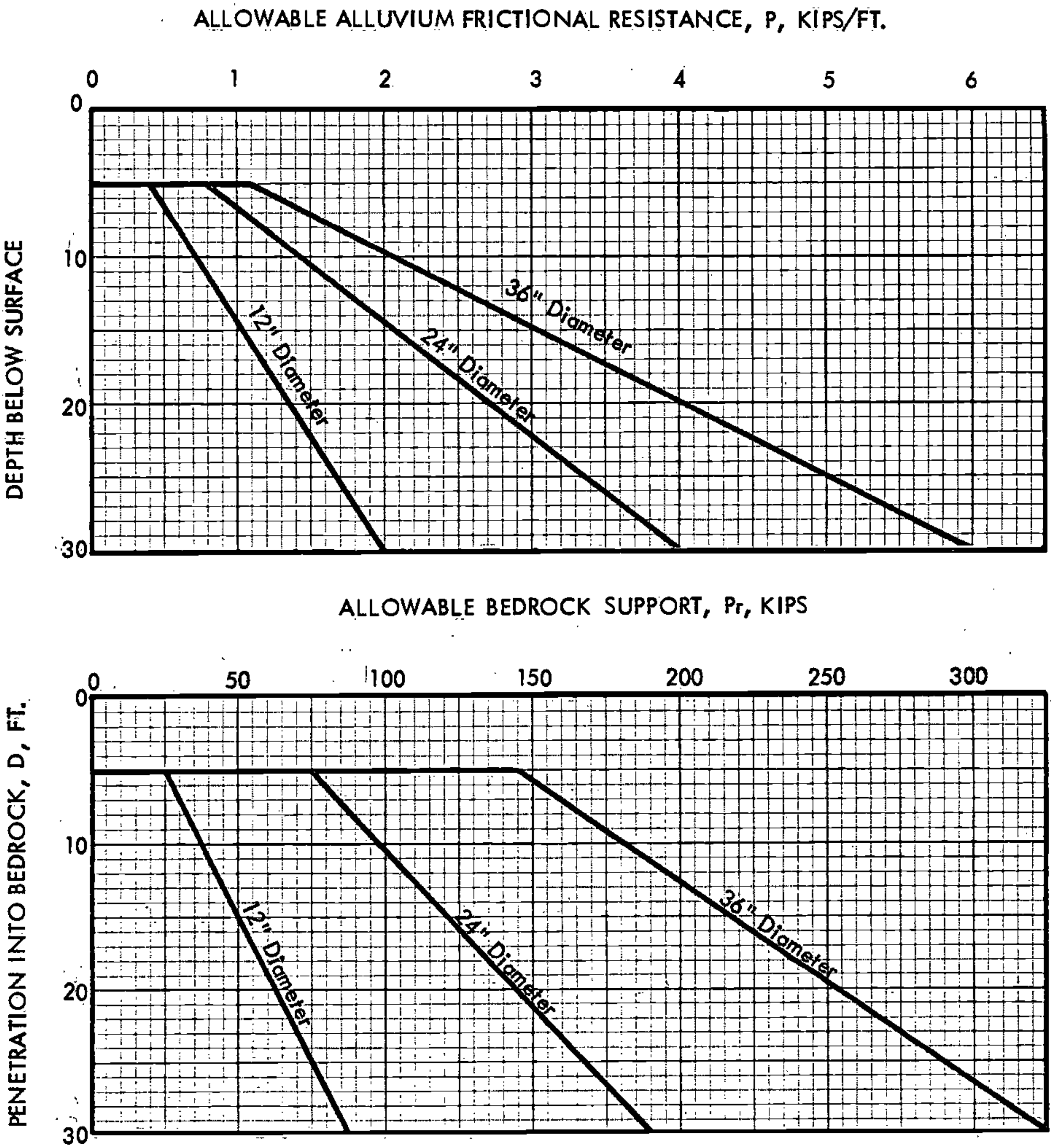
Figure No. 6-3



See Figure No. 6-2 for Determination of Total Capacity

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UNDERPINNING - CAST-IN-PLACE PILE DESIGN PARAMETERS

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Figure No. 6-4

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Total capacity of hand-dug piers should be limited to end bearing only and must extend below the "no support" zone shown on Figure 6-2. All piers are assumed to be 36-inch square or larger in section. For design, an allowable bearing capacity of 12 ksf may be used for piers which bear on the alluvium and penetrate at least 10 feet below the surface. Piers bearing on bedrock may be designed based on 15 ksf for piers which penetrate at least 5 feet into the bedrock. These values apply only if the bearing surface is properly cleaned, observed and approved by a qualified engineer.

Expected lateral ground movements adjacent to the excavation are presented in Section 6.4.6. The capability of the existing structure and underpinning piles to withstand these lateral movements should be evaluated. If necessary, additional lateral restraint could be provided by tieback anchors.

6.3.4 Underpinning Performance

Underpinning is not a guarantee that the structure will be totally free from either settlement or lateral movement. Some movement may occur during the underpinning process and additional movement may occur during the construction of the main excavation. However, movement can be minimized by proper monitoring and maintenance procedures.

6.3.5 Underpinning Instrumentation

Elevation reference points should be established on each foundation element to be underpinned. The points should be monitored on a regular basis consistent with the construction process and may be required daily. Maximum allowable movements should be established for each element by the engineer prior to underpinning. If it appears that these limits may be exceeded, immediate measures should be taken such as restressing underpinning elements, adding more supports, changing the underpinning installation procedures, and/or others.

Where a group of three or more jacked piles is used to underpin a foundation element, load relaxation of previously installed piles can occur. Methods should be implemented to evaluate this problem and restress piles if necessary.

6.4 SHORING SYSTEMS FOR STATION EXCAVATIONS

6.4.1 General

The required excavation for the 7th/Flower Station will extend some 50 to 60 feet below the adjacent street level. As discussed in Section 3.1, there are several existing large buildings located within 10 feet of the required excavation. We understand that these buildings are supported on spread footings on either the bedrock or the Fernando Formation. These buildings have basements extending from about 20 feet (Global Marine Building) to 40 feet (Broadway Plaza) below street level.

The proposed excavation will require shoring due to the space restrictions and to protect the existing adjacent structures. There are several ways to construct the excavation including a shoring system with underpinning of adjacent structures or a "rigid" shoring system which will minimize ground movements and may eliminate the need to underpin. We understand that the excavation 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 Transit Consultants.

In our opinion, the contractor will most likely propose one of the following shoring systems with internal bracing for lateral support:

- Conventional Soldier Pile Wall: Significant buildings located within the underpinning zone (see Figure 6-1) would require underpinning.

- Conservative Soldier Pile Wall: This would consist of a conservatively designed and constructed soldier pile wall which may limit ground movements sufficiently to eliminate the need for underpinning.

- Slurry Wall: Installation of a properly designed and constructed slurry wall may eliminate the need for underpinning.

Accordingly, the discussion and design criteria presented in this section pertain to these general shoring methods. Other shoring support systems may also be appropriate and should be considered by the contractor. We do not believe the contractor will propose driven sheet piles since it would be unfeasible to drive sheets into the dense/stiff soils and bedrock underlying the site.

6.4.2 Current Practice

In the Los Angeles area, deep basement excavations have been constructed with a soldier pile, lagging, and tieback system. Several references are available which summarize these design and construction practices (CWDD, 1981; Nelson, 1973; Crandall and Maljian, 1977; Maljian and Van Beveren, 1974). It is our understanding that adjacent major structures have normally been underpinned if they fall within the 1H:1.5V zone defined on Figure 6-1. However, there have been projects where underpinning was not used, and the existing structures have transferred lateral loads to the shoring system. These have included St. Vincent's Hospital, Century City, a high-rise at 7th and Grand Avenue, and others. Appendix E.1 presents several case studies of soldier pile and tieback shored excavations in the Los Angeles area.

Appendix E.1 also summarizes the design shoring pressures for nine shoring systems in the general project area which have performed adequately. The design pressures presented in this report reflect the local experience.

Rail projects in the District of Columbia, San Francisco, Boston, Atlanta, Baltimore, and New York have involved similar deep shored excavations in close proximity to existing structures. For these projects, both permanent and temporary slurry walls have been used to minimize and/or eliminate the need for underpinning.

6.4.3 Technical Considerations

The function of the shoring at the Station site will be twofold: 1) to provide a safe and stable excavation; and, 2) to limit vertical and lateral ground movements. In Appendix E.2 we discuss the primary factors affecting shoring performance and present data on the design and performance of several shoring systems in the Los Angeles area and in similar ground conditions in Seattle.

As applied to the proposed excavation, several important concepts can be drawn from the discussion presented in Appendix E.2:

- In terms of wall stiffness, a slurry wall may not offer a significant advantage over a more conventional soldier pile wall. This is because the subsurface materials at the site are relatively strong and stiff. The factor of safety against basal failure at the site is estimated to exceed 4.0. As shown in Figure E-1 in Appendix E.2, based on finite element analyses, there is virtually no theoretical difference between the movement of three walls of vastly different stiffness provided the factor of safety against basal failure exceeds about 3.0.

- In our opinion, the data support the concept that it may be feasible to construct a conservatively designed soldier pile wall that limits movements to magnitudes small enough to eliminate the need to underpin. This opinion is based on the high strength and stiffness of the on-site soils and bedrock and the past performance of walls in similar materials (primarily data from Los Angeles and Seattle).

- The primary advantage of a slurry wall would be to minimize potential construction related problems.

6.4.4 Soldier Pile Shoring System

6.4.4.1 General: A soldier pile and lagging system installed in predrilled holes and braced with internal struts or tiebacks is a common method of shoring deep excavations. The soldier piles commonly consist of steel H- or WF-Sections installed in predrilled holes. Below the depth of the excavation the hole is filled with either structural concrete or lean concrete depending upon the vertical load transmitted by the soldier pile. Within the fill and alluvium, support such as wooden lagging would be required between soldier piles to minimize loss of ground. The bedrock may not need to be lagged; however, it is recommended that some surface treatment be applied to control spalling and slaking and to protect workers from falling debris.

In areas where existing structures are located within the 1H:1.5V underpinning zone shown on Figure 6-1, we suggest that two soldier pile alternatives be considered:

- Conservative Soldier Pile Wall
- Conservative Soldier Pile Wall.

Due to the proximity of the adjacent deep basements, tiebacks probably are not a feasible primary support method. Thus the excavation will probably be supported primarily with internal bracing. However, at the ends of the excavation near the tunnel access, tiebacks may be proposed. Section 6.4.6 provides design criteria for both types of lateral support.

6.4.4.2 Conventional Soldier Pile Wall: This alternative involves a conventional soldier pile system and would likely include underpinning of significant structures located within the 1H:1.5V zone indicated on Figure 6-1.

Specific shoring design criteria include:

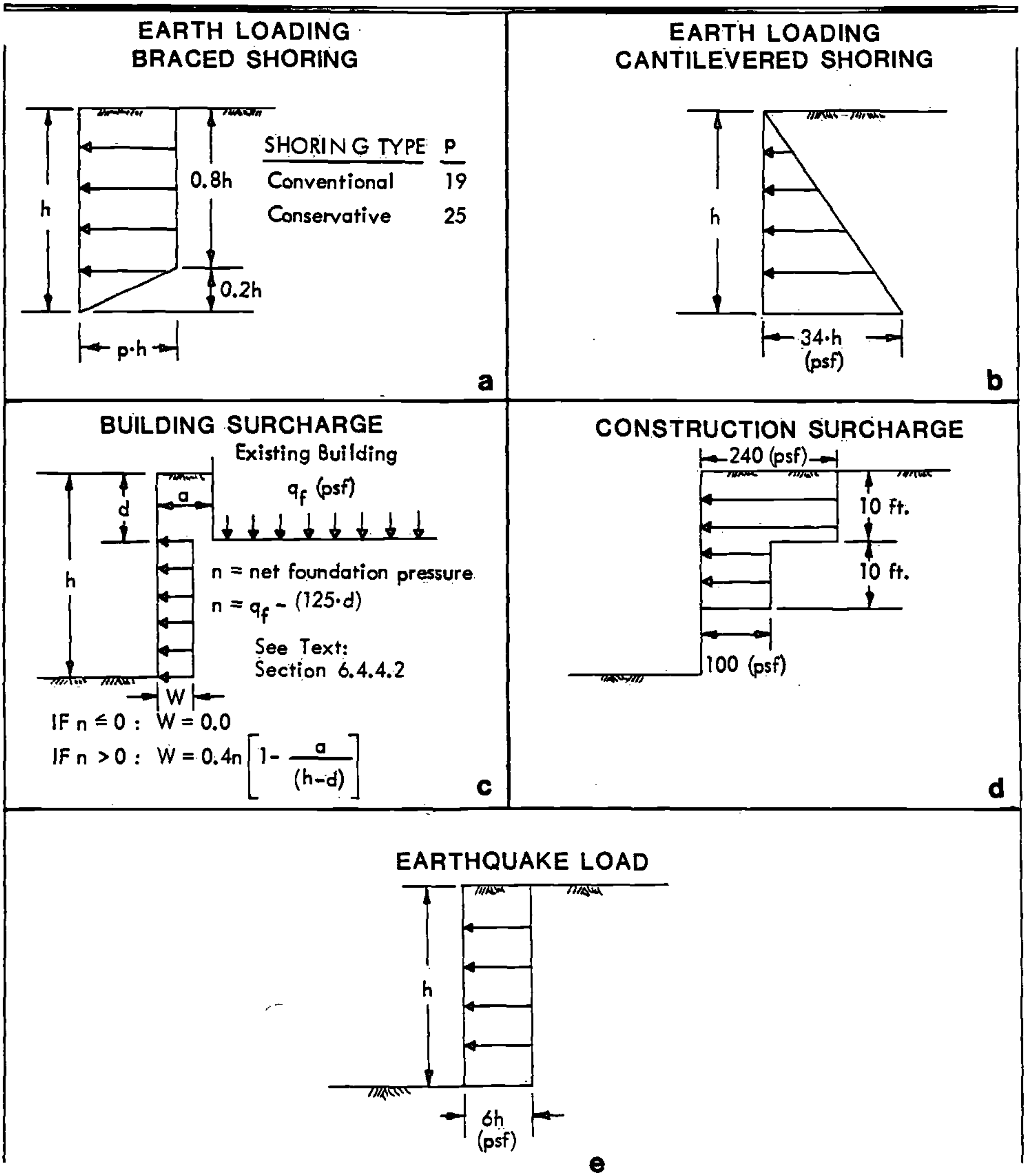
° Design Wall Pressure: Figure 6-5 presents the recommended lateral earth pressure on the temporary walls. This figure includes the additional pressures induced by adjacent structures not underpinned. Appendix E.3 provides technical support for the recommended seismic pressures.

  The proposed shoring wall may be constructed in very close proximity to existing basement walls. We believe that the section of wall above the level of the basement will be subject to reduced lateral load. In addition, a large prestress load in the lateral supports could transmit large loads to the adjacent basement wall and potentially damage the existing structures. Accordingly, we recommend that the design pressures presented in Figure 6-6 be implemented.

  In some cases the adjacent building may be supported on piles or the contractor may be allowed to excavate the soil between the proposed Station excavation and the adjacent building basement. If the adjacent building is supported on piles, additional analyses will be required to determine appropriate shoring pressures. If the soil is excavated it would considerably reduce the height of the shoring wall. We recommend that the wall be designed according to the design earth pressures on Figures 6-5a and 6-5e with h equal to the depth of the cut below the adjacent basement wall. Figure 6-5c can be used to determine the earth pressure due to the building surcharge using the full building load; i.e., not reduced for depth or the 125d factor.

  The full loading diagram should be used to determine the design loads on the internal bracing 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 Soldier Piles: The depth of the soldier pile below the lowest anticipated excavation depth must be sufficient to safely carry both the lateral and vertical loads under static and dynamic loading conditions.



LATERAL LOADS ON TEMPORARY SHORING (WITH DEWATERING)

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Figure No.
6-5

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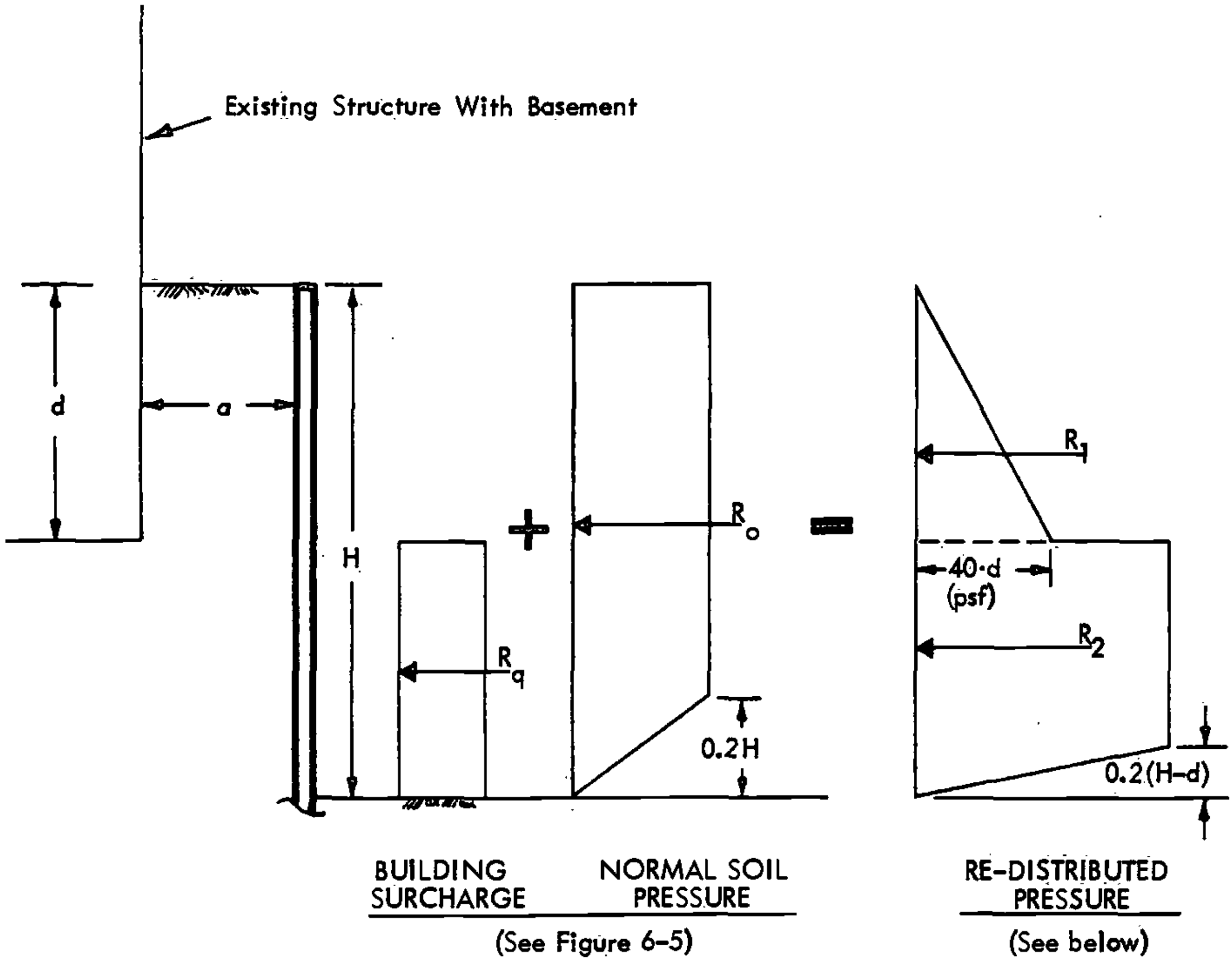

IF $a > d$: USE NORMAL SOIL PLUS SURCHARGE PRESSURES GIVEN IN FIGURE 6-5

IF $a \leq d$: RE-DISTRIBUTE SOIL PRESSURE AS SHOWN ABOVE, SUCH THAT

$$R_2 = (R_o + R_q) - R_1$$

IF THE CONTRACTOR IS ALLOWED TO EXCAVATE ALL THE SOIL BETWEEN THE EXISTING BASEMENT AND THE PROPOSED EXCAVATION, THE ABOVE PRESSURE DIAGRAMS DO NOT APPLY. SEE TEXT SECTION 6.4.4.2 FOR THIS SPECIAL CASE.

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LATERAL LOADS ON SHORING ADJACENT TO EXISTING BUILDINGS

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Figure No. 6-6

The required depth of embedment to satisfy vertical loading can be computed based on allowable vertical loads shown on Figure 6-7.

The imposed lateral load on the embedded portion of the pile should be computed based on the contributing area of the earth pressure diagrams (see Figures 6-5a to 6-5e) below the lowest strut. The allowable passive resistance developed by the pile should be based on the passive pressure minus the active pressure below the bottom of the excavation as shown on Figure 6-8. It should be noted that passive pressure is limited to 6000 psf in the bedrock due to consideration of the undrained strength of these materials. Due to arching effects, the effective pile diameter may be assumed equal to 1.5 the actual diameter or half of the pile spacing, whichever is less.

The required depth to safely carry both the vertical and lateral loads should be computed with the greatest value controlling pile design.

- Pile Spacing and Lagging: The optimum pile spacing depends on many factors including soil loads, member sizes and costs, ability for soils to arch, and other factors. At the 7th/Flower site the upper soils will be granular and may contain pockets of water not fully drained by the construction dewatering system. Thus, it is recommended that the maximum horizontal pile spacing be limited to about 8 feet and that continuous lagging be placed through the fill and alluvium to minimize ravelling of soils and loss of ground between soldier piles. Use of geotextiles and/or limiting temporary exposed soil height should be implemented by the contractor to control ravelling problems.

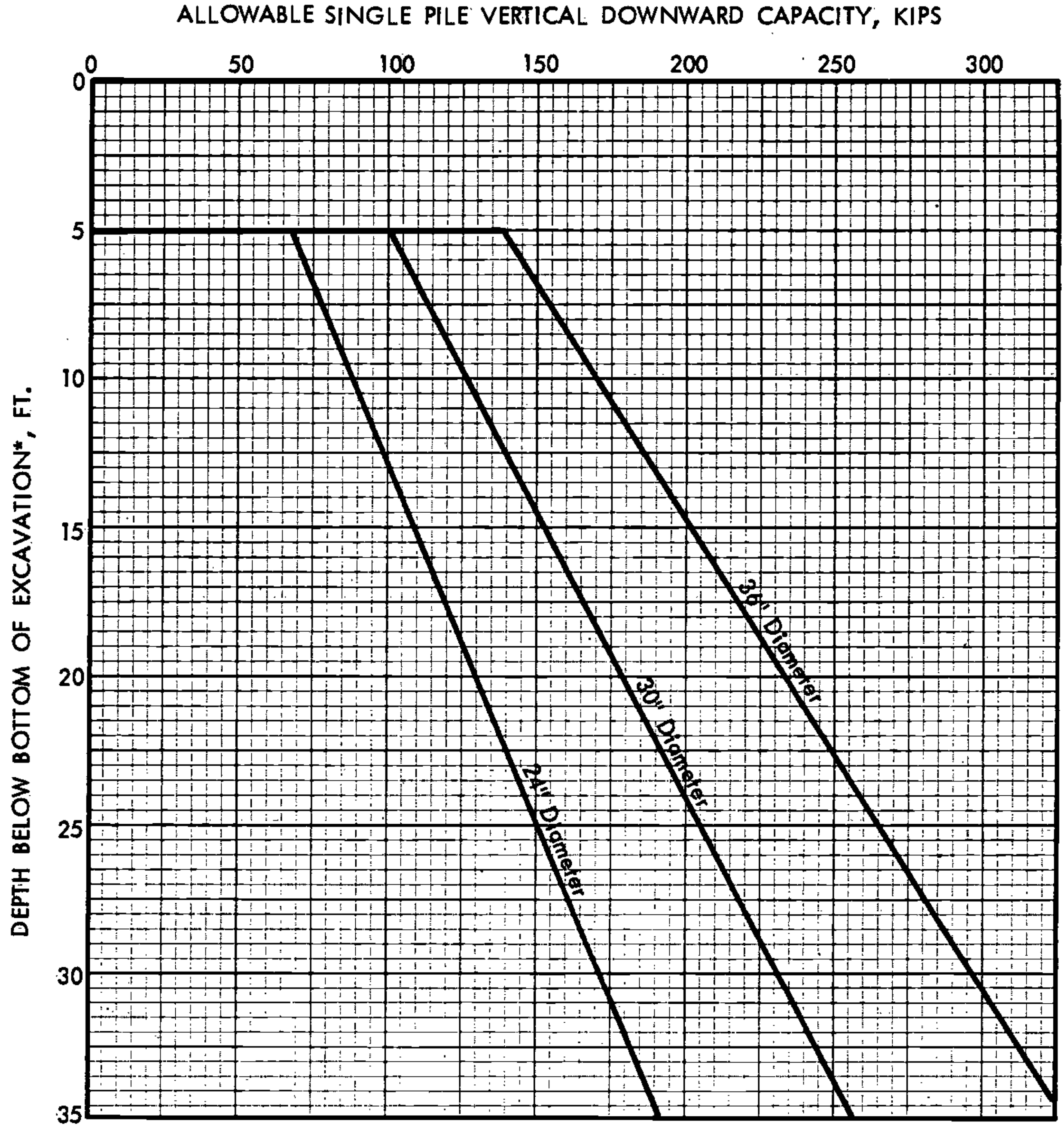
- Support Spacing and Placement: Criteria are presented in Section 6.4.4.3.

- Use of Street Decking Beams: Criteria are presented in Section 6.4.4.3.

- Internal Bracing and Tieback Anchor Design: Design criteria are presented in Section 6.4.6.

6.4.4.3 Conservative Soldier Pile Wall: This alternative involves the installation of a conservatively designed and constructed soldier pile wall which may limit ground movement sufficiently to eliminate the need to underpin.

The decision to implement this alternative would depend on costs and the potential impact of ground movements on the adjacent structures. Underpinning also presents some risks and will result in some movements. We recommend that specific structures which are located within the 1H:1.5V underpinning zone indicated in Figure 6-1 be



*Bottom of Excavation assumed below elevation 219 ft.

NOTES: 1.) Applicable only for drilled shaft piles.
2.) For seismic design, capacities may be increased 33%.

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VERTICAL CAPACITY OF PILES FOR SHORING & DECKING

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Figure No. 6-7

Recommended Unit Pressures

ABOVE THE WATER TABLE:

P_{p1} = 550 psf/ft (for alluvium)
= 450 psf/ft (for bedrock)

P_{p3} = 6000 psf (for bedrock only)

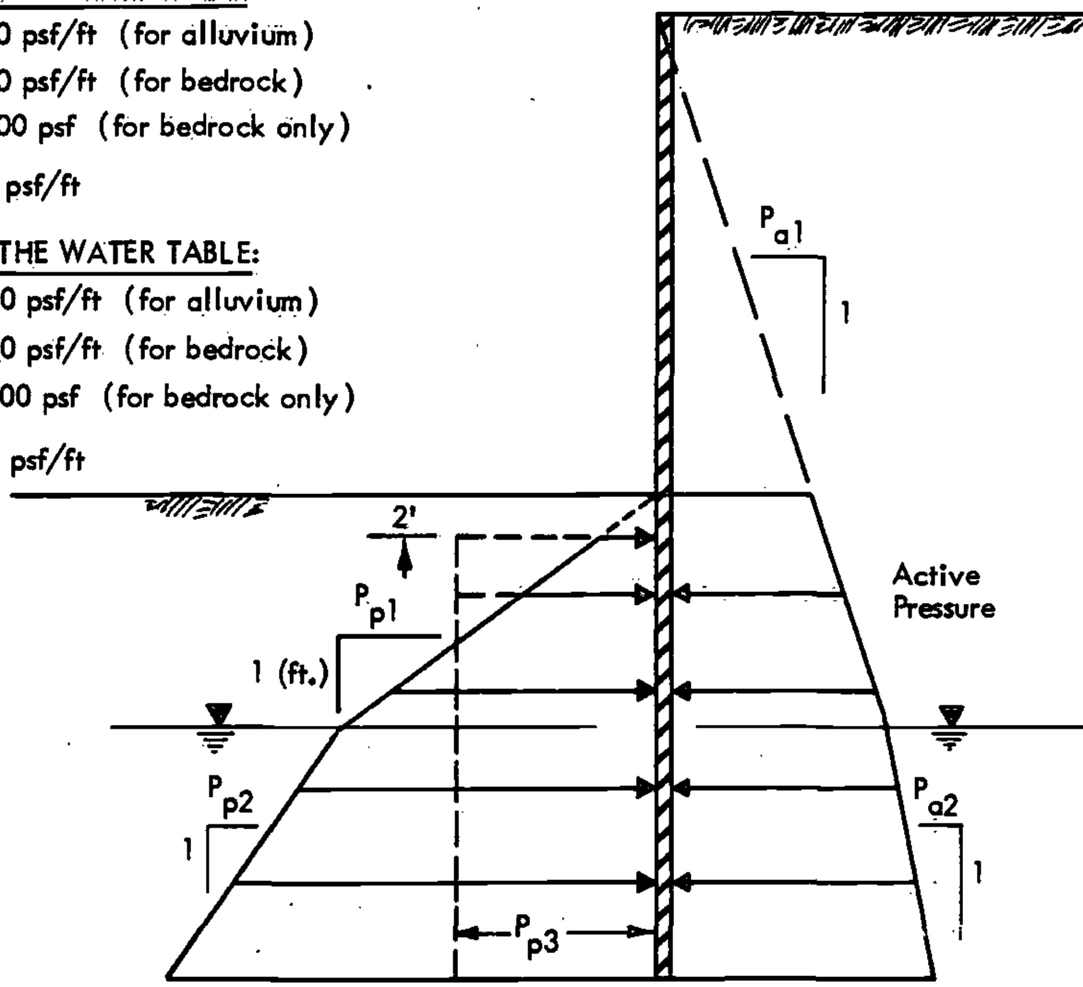
P_{a1} = 34 psf/ft

BELOW THE WATER TABLE:

P_{p2} = 350 psf/ft (for alluvium)
= 450 psf/ft (for bedrock)

P_{p3} = 6000 psf (for bedrock only)

P_{a2} = 15 psf/ft



Where : P_p = Total allowable unit passive pressure.

P_a = Active Pressure.

NOTES:
1.) For bedrock use either triangular (P_{p1} & P_{p2}) or uniform (P_{p3}) pressure distribution whichever gives the lowest resultant force.
2.) Available Passive Pressure = Total Passive − Active
3.) Available passive pressure can be assumed to act on 1½ pile diameters or ½ the pile spacing whichever is less.
4.) Active pressure shown is only for evaluation of available passive pressure. Lateral shoring pressures are presented on Figure 6-5.

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PASSIVE RESISTANCE - SOLDIER PILES

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Figure No. 6-8

Converse Consultants Geotechnical Engineering and Applied Sciences evaluated separately to determine the suitability of the different shoring and underpinning alternatives. The preferences of the City of Los Angeles Building Department and the adjacent property owners may also affect the decision.

Specific shoring design criteria to limit ground movements are presented below. The extent to which some or all of these are applied depends on the specific situation. To avoid duplication, reference is made to Section 6.4.4.2 where appropriate. These criteria should be applied in areas where adjacent significant structures lie within the 1H:1.5V zone underpinning zone as shown on Figure 6-1. The criteria should apply for a minimum distance equal to the structure width plus 30 feet on both sides of the structure. The recommendations include:

- Design Wall Pressure: Figure 6-5 presents the recommended lateral earth pressures. The recommended pressures on braced shoring are about 30% greater than those recommended for the conventional wall with underpinning. This increase is intended to reduce the anticipated ground movements. Appendix E.2 presents a technical bases for this recommendation. Figure 6-6 presents recommended surcharge loads from adjacent structures. Other comments on wall pressures were presented in Section 6.4.4.2.

- Depth of Soldier Piles: Criteria for the depth of the soldier piles is presented in Section 6.4.4.2

- Support Spacing and Placement: The vertical spacing between supports (tiebacks or struts) should not exceed 8 feet. In addition, the contractor should not be allowed to extend the general excavation more than 3 feet below the designated support level before placing struts or tiebacks. The contractor may be allowed to construct a trench within the excavation to facilitate operations provided the trench is more than 15 feet horizontally from the shoring and does not extend more than 6 feet below the designated support level.

- Use of Street Decking Beams: The transverse beams required to support the temporary decking should be used as the upper level of shoring support. The decking should be installed and structurally connected to the wall prior to the excavation proceeding beyond a depth of about 8 feet.

- Pile Spacing and Lagging: To reduce ground movement and minimize the risk of loss of ground between soldier piles, the maximum horizontal spacing of soldier piles should be about 6 feet. Comments on lagging are presented in Section 6.4.4.2

6.4.4.4 Anticipated Ground Movements: The ground movements associated with a shored excavation depend on many factors including the contractor's procedures and schedule. Appendix E.2 presents data on the performance of shoring excavation systems in the Los Angeles area and in similar ground conditions in Seattle.

The distribution and magnitude of ground movement is difficult to predict. Appendix E.2 presents data on ground movement and the factors affecting it. Based on this information and engineering judgement, we believe that the ground movements associated with properly designed and carefully constructed soldier pile walls with internal bracing will be approximately as follows:

- Conventional Wall: The maximum horizontal wall deflections have been observed equal to about 0.1% to 0.2% of the excavation depth. The observed maximum horizontal movement generally occurs near the bottom of the excavation decreasing to about 25% of the maximum at the top. The maximum vertical settlement behind the walls generally is about equal to 50% to 100% of the maximum horizontal movement and occurs at a distance behind the wall equal to about 25% to 50% of the excavation depth.

- Conservative Soldier Pile Walls: We believe that the design and construction procedures presented above in Section 6.4.4.3 will reduce the maximum horizontal and vertical movements to about 0.1% of the excavation depth.

6.4.5 Slurry Walls

A slurry wall installation consists of excavating a narrow trench or slot to full-depth along the temporary wall line in short sections typically 10 to 20 feet long. The excavating is carried out using special digging tools with trench support being provided by the fluid pressure of a carefully controlled bentonite slurry. Trench stability is normally evaluated based on experience and test sections. Once excavated, the usual practice is to lower a reinforcing cage and place tremie concrete which displaces the slurry mixture. Alternately, precast panels can be placed. With precast panels special additives are mixed with the slurry after the panels are placed to produce a stiff clay material between the precast panel and the native ground. The slurry wall technique produces a relatively stiff and relatively water-tight, continuous wall which can provide the temporary excavations support and/or become the permanent wall. As with soldier pile walls, internal bracing or tiebacks are normally used to support the walls during construction.

Slurry walls have been used extensively in Europe and in the United States. Several subway station projects have utilized slurry walls including:

- BART (San Francisco area): Slurry walls were used for temporary support of excavations in difficult ground conditions and/or in close proximity to existing structures where ground movement was critical. The general design concept used the slurry walls to minimize or eliminate the need to underpin.

- MBTA (Boston area): Slurry walls have been used both as temporary shoring only (Davis Square) and in combination as the permanent wall (Harvard Square).

- Baltimore Metro: Slurry walls were used as temporary shoring to eliminate underpinning requirements.

- District of Columbia and Atlanta Systems: Slurry walls have been used at both of these projects.

There are several advantages to slurry walls at the 7th/Flower site including:

- Reduces Ground Movement: A properly designed and constructed slurry wall will generally result in less ground movement than a conventional soldier pile wall. This is probably due to several factors including: greater wall stiffness, better structural continuity and continuous soil support or "tight shoring" (which eliminates loss of ground between wall elements). On many projects slurry walls have been successfully used in close proximity to existing structures, eliminating the need to underpin.

  Use of slurry walls does not, however, eliminate potential problems associated with ground movements. Poor construction procedures, particularly associated with poor mud control and wide wall sections, can result in excessive ground movement. There have also been cases where loss of soil occurred through tieback anchor holes in granular soil below the ground water table.

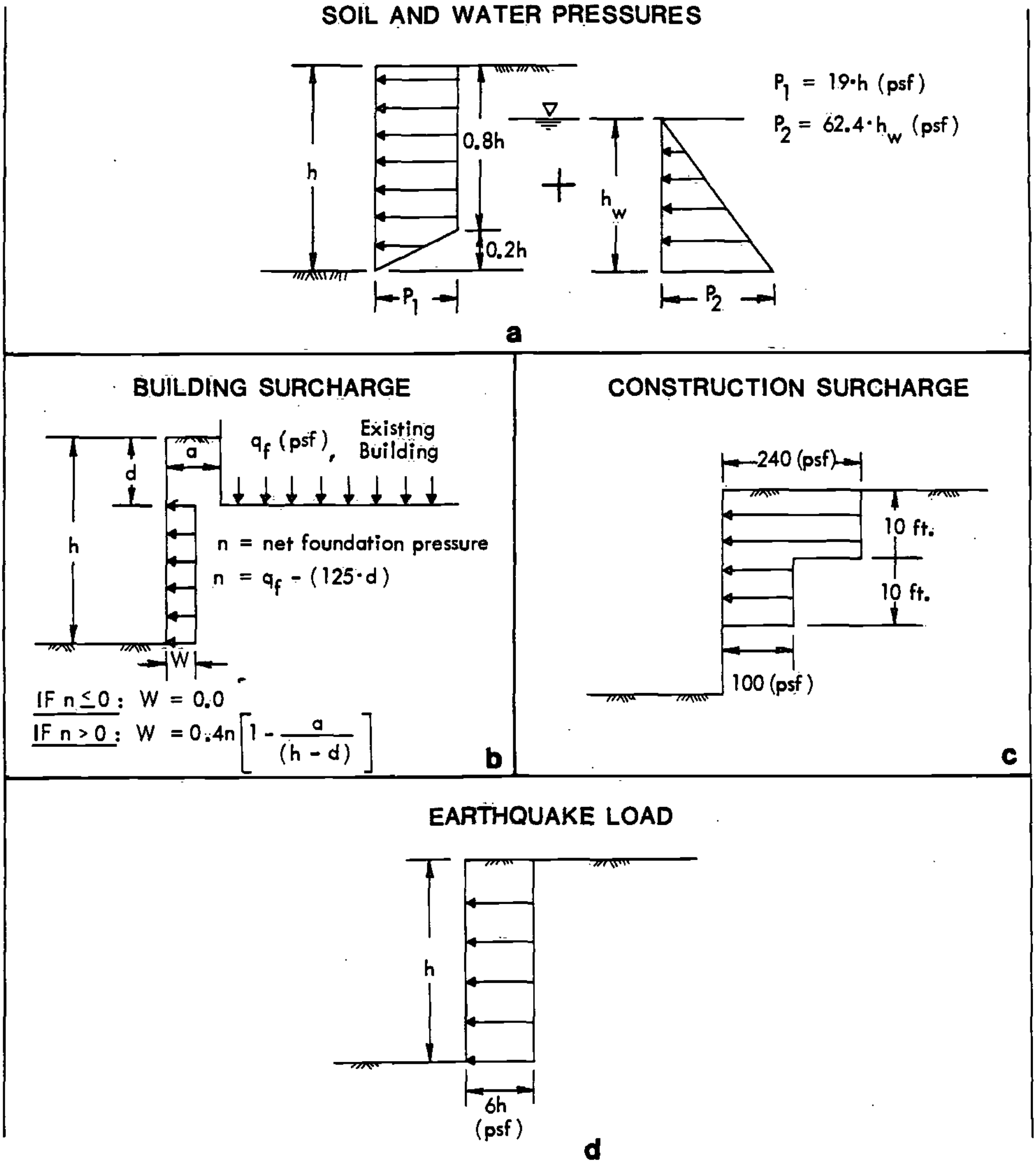
- Use as a Permanent Wall: We understand that the shoring wall will not be used by itself as the permanent wall. However, it could be designed to assist the station exterior walls in resisting lateral loads. All the soil and hydrostatic pressures can be resisted by the slurry wall while the station exterior wall would be designed to resist only the hydrostatic pressure since the slurry wall cannot be assumed to be completely watertight.

- Difficult Ground Conditions: Slurry walls can probably accommodate a wider range of soil and ground water conditions than soldier piles. Problems such as boulders, running ground, and obstructions can present serious problems to a normal soldier pile installation. These conditions are more easily resolved with slurry wall construction and present a lower risk of "lost ground" damage to adjacent structures.

- Cutoff Wall: The slurry wall can be used as a deep ground water cutoff. Ground water inflows are not expected to present a significant problem at the site. However, perched water may be a construction nuisance requiring wells or well points. With a slurry wall, construction dewatering problems should not occur.

Design criteria for slurry walls supporting the temporary excavation include:

- Design Wall Pressure: Figure 6-9 presents the recommended temporary design wall pressures for slurry walls. Since the slurry wall will be essentially water-tight, the wall must be designed to resist the anticipated hydrostatic ground water pressures. Figure 6-9 presents recommended surcharge loads including those from adjacent structures.



LATERAL LOADS ON THE TEMPORARY SLURRY WALL (NO DEWATERING)

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Figure No. 6 - 9

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- Depth of Embedment: The slurry wall must be embedded sufficiently below the maximum depth of the excavation to support applied vertical loads (decking loads and tieback vertical loads) and develop sufficient passive resistance. Figure 6-10 indicates the recommended method to compute passive resistance. Figure 6-11 indicates the allowable vertical loads on slurry walls for different embedment depths. The recommended vertical capacities include both end bearing and side friction (only below the level of the maximum excavation depth).

- Slurry Composition: An unsuitable bentonite slurry may lead to excessive viscosity,flocculation, attendant loss of fluid, and spalling of the excavated face. Some factors which affect the slurry are pH, contamination by salt, iron, calcium or organics. As indicated in Appendix C, the ground water is of relatively poor quality containing dissolved salts, gas and hydrocarbons. The slurry chemistry must accommodate the poor ground water conditions. This might involve the use of special additives and/or saline resistant bentonite.

- Panel Length: Much of the stability of the slurry filled bentonite trench is due to arching. Thus, the stability of the trench and associated ground movements prior to concreting are both related to panel length. We recommend that the panel length generally be limited to 20 feet. Where adjacent existing buildings are founded on the alluvium and are located within the underpinning zone (see Figure 6-1), we recommend that the panel width be limited to 12 feet.

- Panel Location: In areas immediately adjacent to existing footings, a panel section should not extend adjacent to more than half the length of the footing. The intent is to ensure that major isolated exterior footings straddle the wall panels. This would minimize potential movements during the installation phase of the wall.

- Existing Basement Voids: Voids from old basements could be encountered which could lead to loss of slurry. In such areas the voids would have to be filled, the section sealed off, or the top of the slurry section lowered below the void.

We expect that ground movements for a properly designed and constructed slurry wall will be similar to those anticipated for conservatively designed soldier pile walls discussed in Section 6.4.4.4.

6.4.6 Tiebacks and Internal Bracing

6.4.6.1 General: As discussed in Section 6.4.4, internal bracing will probably be used to provide the primary lateral support of the shoring wall. However, tiebacks may be used at the ends of the station to facilitate construction of the tunnel junction. Thus this section includes design criteria for both internal bracing and tiebacks.

Prestressing of both tiebacks and internal bracing is essential to confirm design capacities and minimize ground movements.

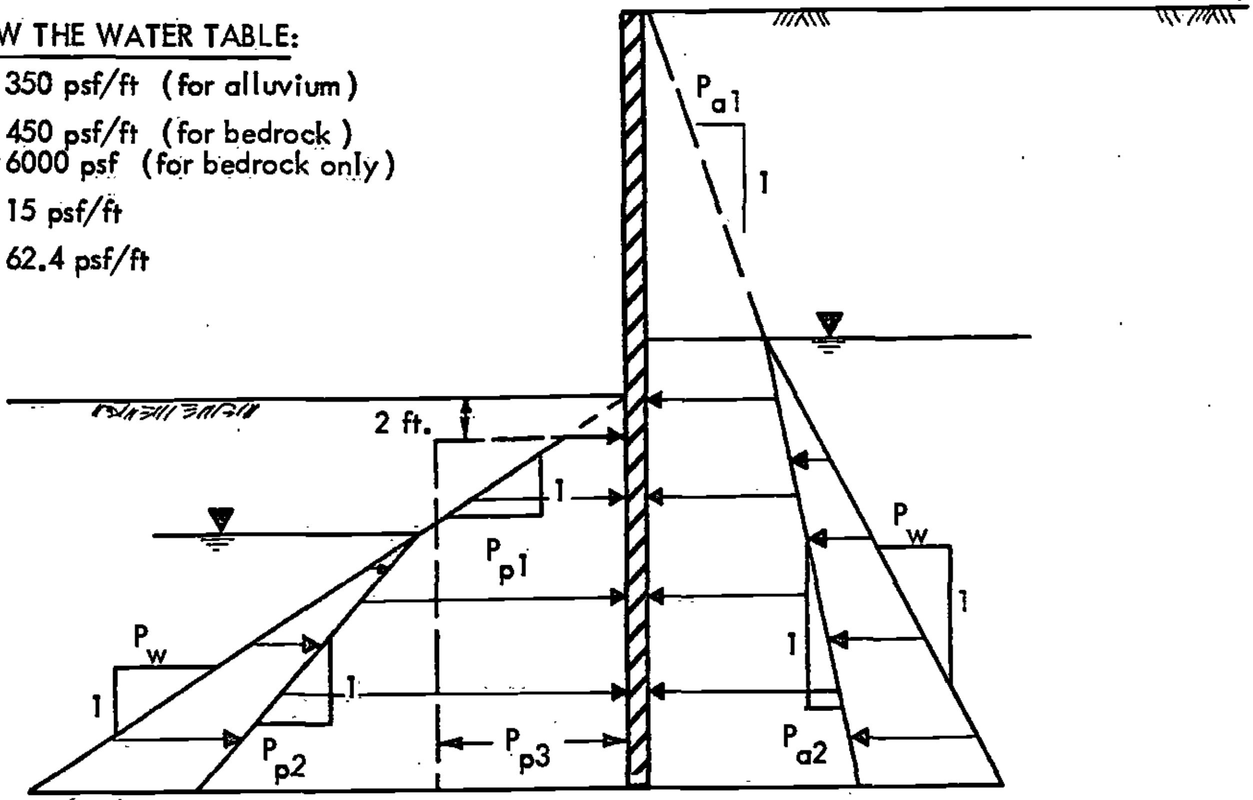
Recommended Unit Pressures

ABOVE THE WATER TABLE:

P_{p1} = 550 psf/ft (for alluvium)
= 450 psf/ft (for bedrock)
P_{p3} = 6000 psf (for bedrock only)
P_{a1} = 34 psf/ft

BELOW THE WATER TABLE:

P_{p2} = 350 psf/ft (for alluvium)
= 450 psf/ft (for bedrock)
P_{p3} = 6000 psf (for bedrock only)
P_{a2} = 15 psf/ft
P_w = 62.4 psf/ft



Where: P_p = Total Allowable Unit Passive Pressure
P_a = Active Pressure
P_w = Hydrostatic Pressure

NOTES: 1.) For bedrock consider both triangular (P_{p1} & P_{p2}) and uniform (P_{p3}) pressure distribution and use the lowest resultant force.
2.) Available Passive Pressure = Total Passive - Active - Hydrostatic diffrential.
3.) Active pressure shown is only for evaluation of available passive pressure. Lateral shoring pressures are presented on Figure 6-5.

PASSIVE RESISTANCE - SLURRY WALL

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Figure No. 6 - 10

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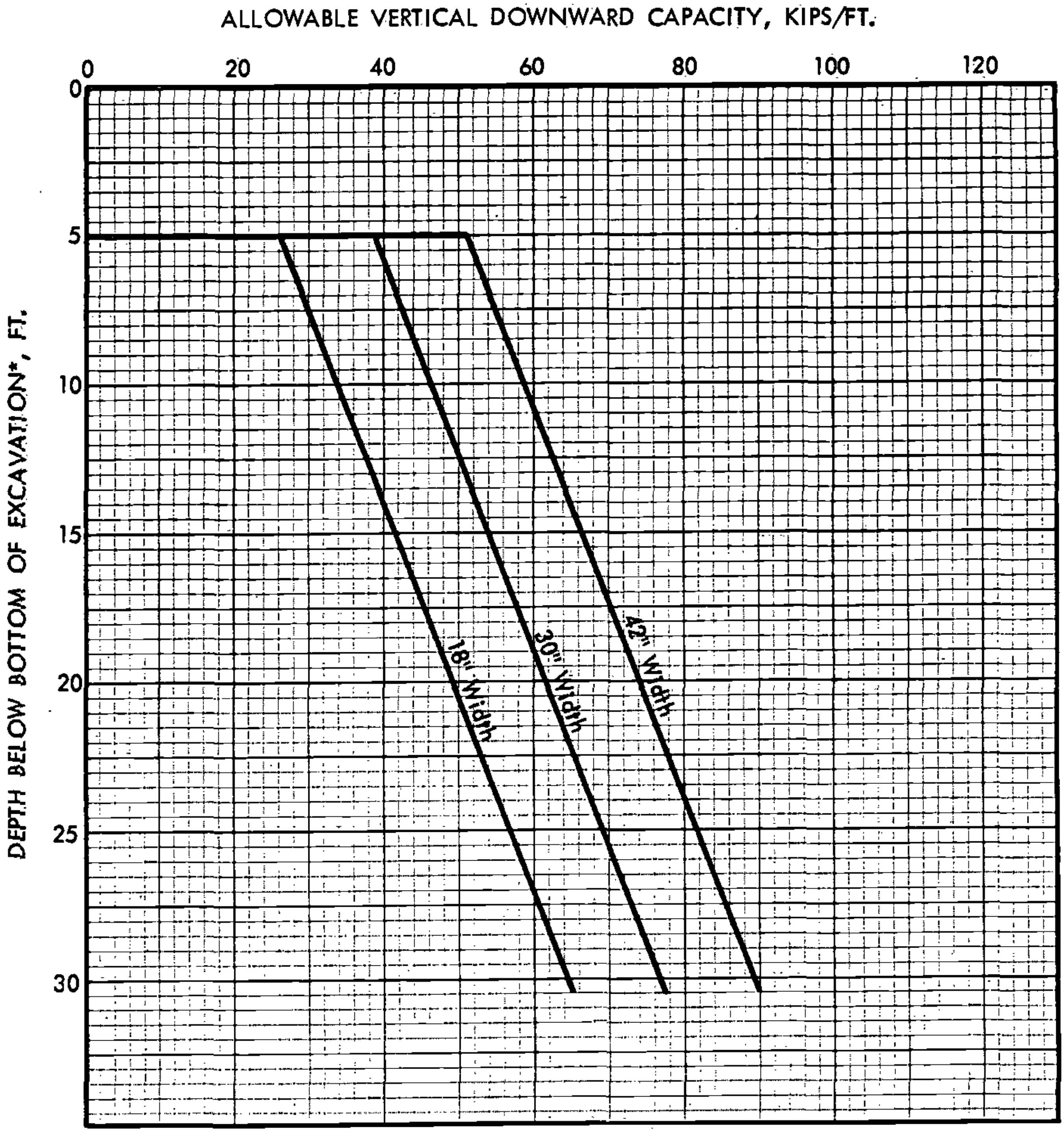

*Bottom of Excavation assumed to be below elevation 219ft.

Note : 1) For seismic Design, capacities may be increased 33%.

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VERTICAL SUPPORT CAPACITY OF SLURRY WALL

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Figure No. 6-11

6.4.6.2 Performance: Based on the 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. Appendix E.2 presents a comparison of the two types of support systems.

6.4.6.3 Internal Bracing: The contractor should not be allowed to extend the excavation an excessive distance below a 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 Soldier Pile Wall: 12 feet
- Conservative Soldier Pile Wall: 8 feet
- Slurry Wall: 12 feet.

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.

To remove slack and limit ground movement, the struts should be preloaded. A preload equal to 50% of the design load is normally desirable. The shoring design and preload procedures must provide for the effects of temperature changes. Several methods should be considered including:

- Varying the preload stress depending on the temperature at the time of installation. The preload stress could be based on developing 50% of the design load at some designated average temperature assuming a non-yielding shoring wall. The assumption of a non-yielding wall to compute temperature-induced stresses is conservative and may warrant refinement to include the estimated soil stiffness (Chapman, 1972).

- Provide a method of minimizing temperature variations such as covering the excavation (street decking), painting the struts with reflective paint, cooling the struts with water, and/or others.

- Provide a method of measuring and adjusting the loads on the struts. The contractor could be required to maintain the struts within a specified stress range. A maximum stress equal to the elastic limit of the strut with a minimum stress equal to 25% of the design load may be appropriate. This method, although technically feasible, may be difficult to perform efficiently in the field.

- Increase the load carrying capacity of the struts (larger members and/or intermediate supports) such that the bracing can safely support the maximum anticipated temperature-induced loads combined with the earth pressure loads.

6.4.6.4 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.

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

$$P = \pi DLq$$

where:

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

The design adhesion value (q) can be assumed equal to:

q = 750 psf (in bedrock)

$q = 20d \leq 1000$ psf (in alluvium)

where:

d = average depth of the anchor beyond the no-load zone; measured vertically from the ground surface.

No resistance should be assumed within the fill.

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

For design purposes, the potential wedge of failure or no-load zone behind the shored excavation is determined by a plane drawn at 45° with the vertical through the bottom of the excavation. Only the frictional resistance developed beyond the no-load zone should be assumed effective in resisting lateral loads. Based on specific site conditions, the extent of the no-load zone may be locally decreased to avoid underground obstructions. Section 6.4.4.3 presents special criteria for the no-load zone for the Conservative Soldier Pile Wall.

The anchors may be installed at angles 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.

The contractor should be required to use a tieback anchor installation method which will minimize loss of ground due to caving. In general, anchors installed entirely within the Fernando Formation should not experience significant caving problems. However, potential caving problems in the overlying fill and alluvium could be a problem particularly for anchors installed under buildings. Uncontrolled 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. This is particularly critical for anchors drilled through fill and alluvium under existing buildings. The contractor should be required to demonstrate adequate procedures to minimize caving before installing anchors below existing structures. Alternative anchor types such as small diameter high pressure anchors or driven anchors could also be proposed.

It is recommended that each tieback anchor be load tested to 150% of the design load and then locked off at the design load. At 150% of the design load, the anchor creep should not exceed 0.1 inch over a 15-minute period. In addition, 5% to 10% of the anchors should be test-loaded to 200% of the design load and then locked off at the design load. At 200% of design load the anchor creep should not exceed 0.15 inches over a 15-minute period.

6.5 SUPPORT OF TEMPORARY DECKING

We understand that, unless the street is closed entirely to vehicular traffic, the temporary street decking will require center support piles. These piles must extend below the maximum proposed excavation level for support. At these depths, the piles would be founded within the soft Fernando bedrock. These materials are suitable for supporting expected pile loads.

Since the shoring contractor will probably install soldier piles to support the excavation, we believe that he may use similar piles to support the center decking. Accordingly, we evaluated the allowable loads on these types of piles for several typical diameters. The recommended allowable design loads are shown on Figure 6-7. These values include both end bearing and shaft friction.

Due to the dense, gravelly composition of the alluvium, we believe that driven piles may be difficult to install and will probably need to be predrilled to achieve the required tip elevation. The pile driving noise may be unacceptable. In addition, driven piles may induce settlements in the soil due to driving vibrations, particularly within the fill layer. Thus, we believe that driven piles would probably not be used. Accordingly, we have not developed design loads for driven piles.

6.6 INSTRUMENTATION OF THE EXCAVATION

In our opinion instrumentation of the excavation is required at the proposed 7th/Flower site. The information obtained will reduce liability (by documenting performance), identify and resolve problems before they become critical, validate design criteria, and aid in the design of future stations. Instrumentation is particularly critical at this site due to the proximity of adjacent structures.

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 reduce the risk associated with claims against the owner/contractor. If substantial cracks are noted in the existing structures, they should be measured and periodically remeasured during the construction period.

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

- Tiltmeters: 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.

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

- Vertical Settlement Profiles: We recommend that four to six devices be installed to monitor the ground settlement pattern with depth around the excavation. There are several methods to obtain these data including a multi-point inductive coil settlement gage and vertical multi-point extensometers. In addition, subsurface vertical and lateral deformation data can be obtained within a single borehole by installing a special inductive coil system around the inclinometer casing.

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

  We recommend that mechanical 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 it from equipment yet allow for easy access should the hole collapse.

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

- Additional Measurements of Strut Loads: 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. There are several methods to obtain these data. A commonly used method involves vibrating wire strain gages mounted on studs welded to the struts. For full measurements of maximum stresses, a minimum of three gages is needed on a pipe strut and four on a wide flange strut. However, two gages are often used to simplify the installation and monitoring effort with acceptable results. There should be a means of measuring the strut temperature at the time of the strain readings.

- Slurry Consistency: As a matter of routine, a slurry wall contractor must test the slurry for consistency and chemistry. This may be particularly critical at the proposed Station site due to the quality of the ground water. Chemicals in the ground water can affect the consistency of the slurry and its ability to form a mudcake. Sections 4.3 and 5.4 present information on the ground water quality.

- <u>Frequency of Readings</u>: 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 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.

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

In our opinion, it is important that the installation and monitoring 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 is questionable.

6.7 EXCAVATION HEAVE AND SETTLEMENT OF MAIN STATION STRUCTURE

The excavation will substantially change the ground stresses below and adjacent to the excavation. The proposed maximum 60-foot excavation will decrease the vertical ground stresses by about 7500 psf. This stress reduction will cause the soils below the excavation to rebound or rise. This response is not due to the occurrence of any swelling type of soils but simply the response to stress unloading. In addition, even with a suitable shoring system, shear stresses will develop tending to cause the soil adjacent to the excavation to heave into the excavation. The net effect will be to cause the bottom of the excavation to heave or deform upward. Since the excavation will be open for an extended period, the heave is expected to be completed prior to constructing the Station. The structure and subsequent backfilling will reload the soil. We estimate that the Station load will be about 4000 to 5000 psf. This load will cause the ground to reconsolidate or settle. Thus, even though the weight of the excavated soil exceeds the weight of the final structure, the structure will cause ground settlement.

We estimate that the maximum heave at the center of the excavation will be on the order of 1½ to 3 inches. We also believe that the majority of this will occur while the soil is being excavated. This estimate is based on computations of elastic shear deformation (elastic rebound) and volume changes within the bedrock underlying the proposed excavation. Due to the consistency of the bedrock, the majority of the heave deformation will be elastic rebound. These values agree well with observed behavior in similar excavations in the Los Angeles area (Evans, 1968).

It was computed that the imposed loads from the structure and backfill will induce settlements on the order of 1 to 2 inches. Due to the long, narrow shape of the imposed load, the theoretical differential settlement in the transverse direction is relatively small, on the order of 1/3 inch over 32.5 feet. This correlates to an angular rotation of only about 1:1200. These calculations are based on a uniform foundation bearing pressure which could only result from a uniformly loaded and perfectly flexible structure. We understand that the Station will be structurally quite stiff. Thus, the actual transverse differential settlement will be less than the theoretical flexible foundation case. Drawings 4 and 5 indicate that the southeast portion of the structure (estimated 140 to 280 feet) will be supported on a wedge of alluvial soils up to 30 feet thick. However, the estimated range of total settlements given above are considered applicable to both the bedrock subgrade and alluvium subgrade portions of the structure. Differential settlements between the northwest (bedrock) and southeast (alluvium) structure areas should not cause significant structural distortion due to the gradual subgrade transition (the bedrock slope is flatter than 12:1). For example, if one inch of differential settlement were to occur between the bedrock subgrade and the maximum depth alluvium subgrade, this would correlate to an angular distortion ratio in the range of 1:1700 to 1:3400. The differential settlements and distribution of the bottom slab bearing pressures could be estimated based on a soil-structure interaction analysis; however, such an analysis is beyond the scope of this study.

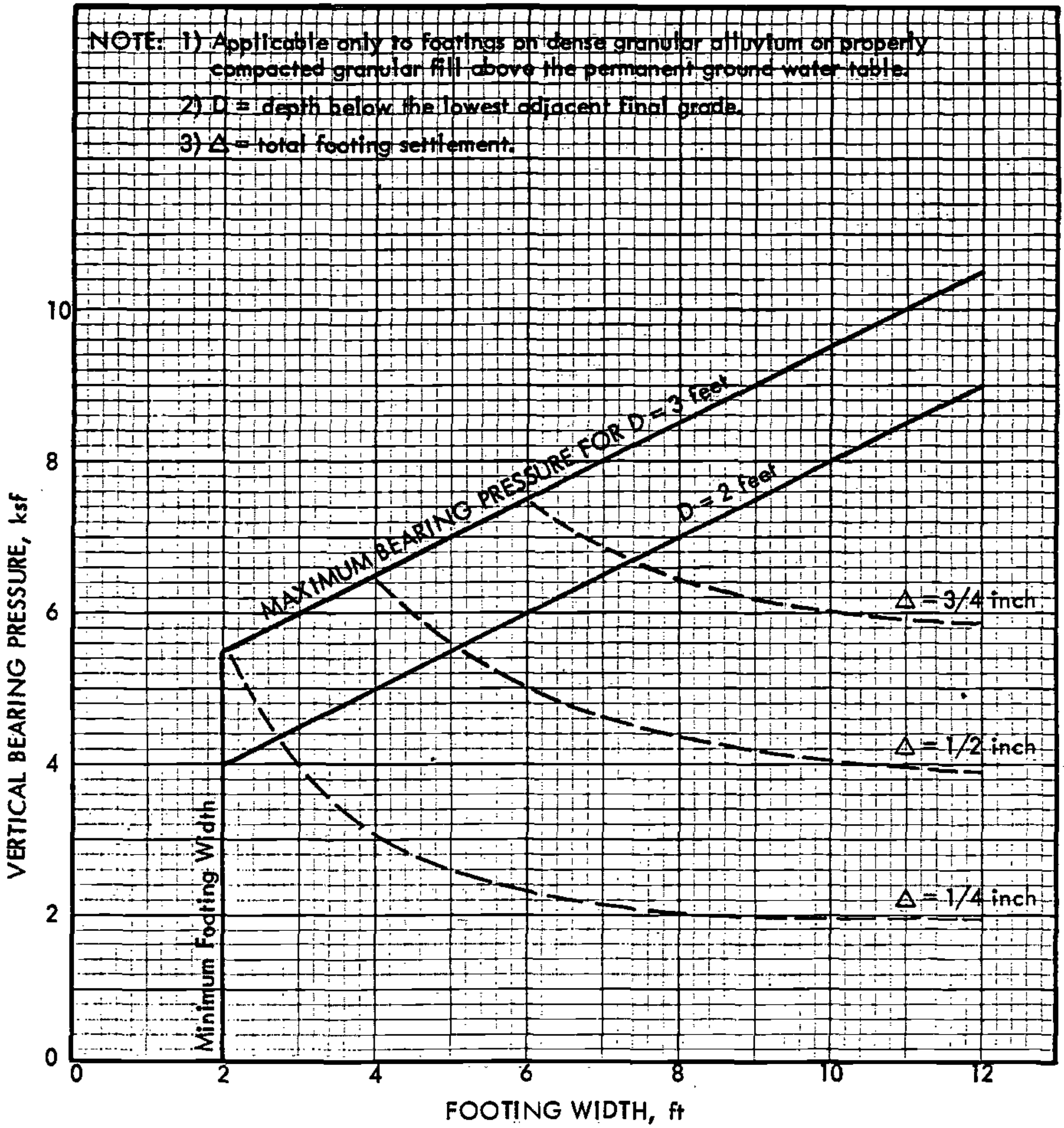
6.8 PERMANENT FOUNDATIONS

6.8.1 Main Station

It is understood that the proposed Station Structure will be supported on a thick base slab which will function as a massive mat foundation. At the proposed foundation level, the main Station mat will bear on the Fernando bedrock formation and dense granular alluvium. We understand that the average foundation bearing pressures for the main Structure will be about 4000 to 5000 psf. In our opinion the proposed mat foundation can adequately support the station on the Fernando bedrock and dense alluvium. Section 6.7 presents a discussion of estimated total and differential settlements for the main Station structure.

6.8.2 Support of Surface Structures

Major surface structures such as a traction power substation and chiller plant should be supported below the the existing fills on the underlying alluvium. Alternatively, the fill may be excavated from below the structure and backfilled with compacted engineered fill for structural support. Generally, it is understood that foundation levels for the surface structures will extend to at least the top of the lower, dense granular alluvium (below about Elevation 255 as shown on Drawings 4 and 5). Figure 6-12 presents recommended maximum bearing pressures and anticipated settlements for footings bearing on either dense, undisturbed granular alluvium or properly compacted structural fill. Lower bearing values should be used for footings bearing on the upper fine-grained alluvium above about Elevation 255. Figure 6-13 presents footing design values for the fine-grained alluvium. These figures are based on



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SPREAD FOOTING DESIGN ON GRANULAR SOILS

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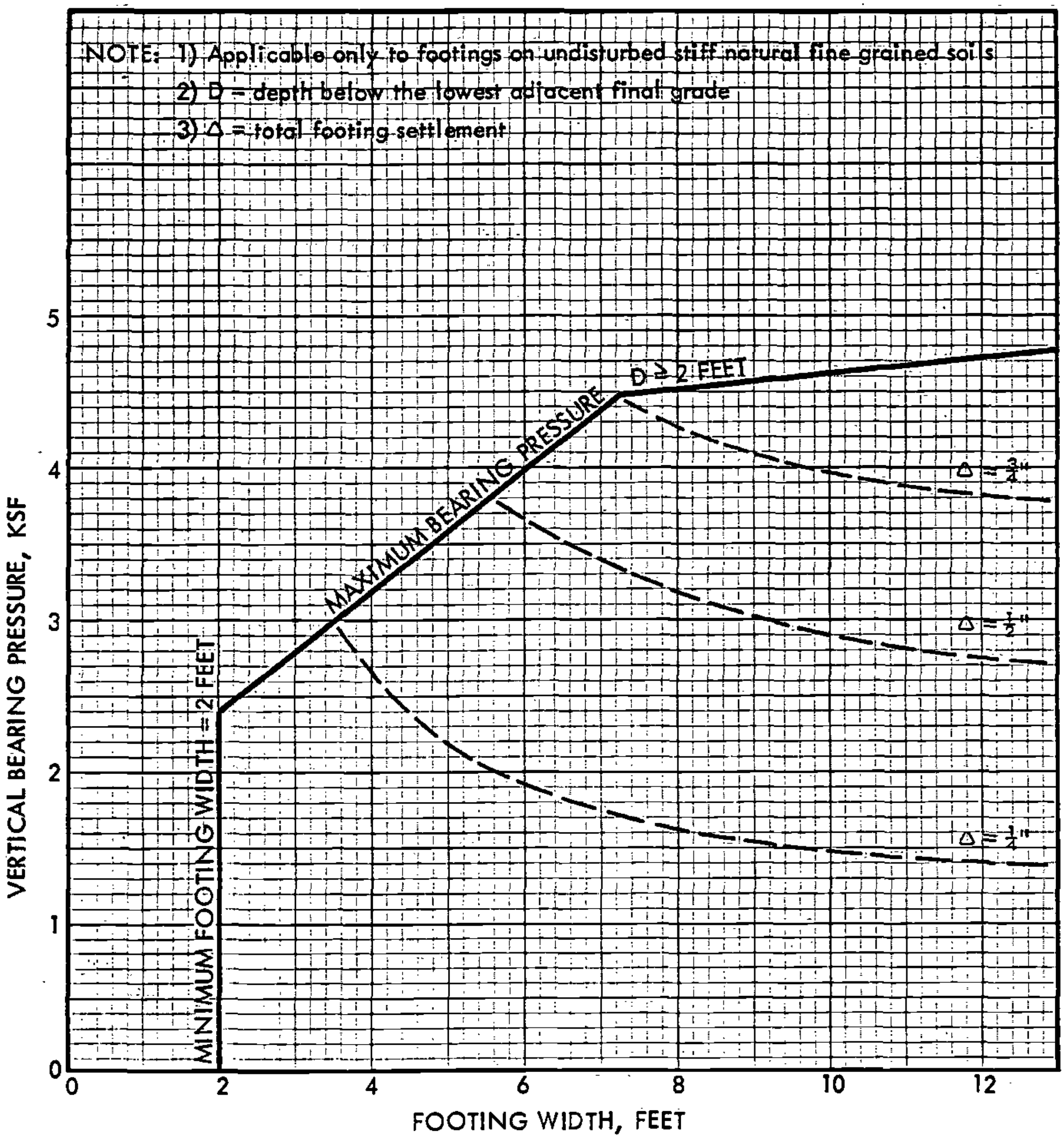
Figure No. 6-12



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SPREAD FOOTING DESIGN ON FINE-GRAINED SOILS

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Figure No. 6-13

analytical procedures and experience in the Los Angeles area. The values shown 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%.

Resistance to lateral loads imposed on footing elements can be assumed to be provided by passive earth pressure and foundation base friction. The allowable passive pressure can be computed based on a fluid weighing 250 psf in the natural dense soils and compacted fill. Base friction can be assumed equal to 0.4 times the vertical dead load. Evaluation of allowable lateral resistance of deep station walls requires a lateral deformation analysis which depends on the height and depth of the wall. The allowable lateral resistance would be less.

6.9 PERMANENT GROUND WATER PROVISIONS

6.9.1 General

As discussed in Section 5.4, the regional ground water table appears to occur deep within the bedrock below the site. However, perched ground water was encountered within the lower granular alluvium.

Once the Station is constructed and the excavation backfilled, the natural perched ground water levels within the alluvium will be re-established. The Station excavation could act as a 'bathtub' and tend to collect ground water. This could occur if the shoring wall provides a perimeter zone of higher permeability due to voids behind the shoring and/or placement of sand filler material.

The permanent ground water condition could be resolved by designing a water-tight Station or by providing for a permanent drainage system. Conventionally, the deep basements in the area have been provided with permanent slab and wall drains draining to sumps. The pumping rates have been small due to the small permanent inflow rates. However, conventional practice for subway stations may be to provide complete water-tight construction and design for the maximum hydrostatic pressures.

We understand that the station will probably be designed to be water-tight and to resist the full hydrostatic pressures. This preference is based on standard subway design practices and a concern that even the best subdrain systems eventually clog. In our opinion, a permanent drainage system which would eliminate high hydrostatic pressures would be geotechnically feasible. This opinion is based on the experience with deep basements in the Los Angeles area and the anticipated small inflows. In fact, under normal conditions, the drainage system would probably be dry.

6.9.2 Complete Water-tight System

The Station could be designed to be water-tight below the level of the maximum anticipated ground water elevation. Thus the permanent structure below this level will need to be designed to resist hydrostatic pressures.

We also recommend that full waterproofing be carried at least 5 feet above the anticipated maximum ground water levels.

6.9.3 Complete Underdrain System

A complete wall and slab underdrain system would eliminate the potential for hydrostatic pressures on the structure and would reduce waterproofing costs. We believe that this system is appropriate since the anticipated inflow and pumping rates will be very small. However, this system may not be appropriate if a slurry wall is used as either a temporary or permanent wall. The details of the drainage system depend on whether the permanent wall is poured directly against the temporary wall or if there is a space between the walls.

If the permanent wall is formed directly against the shoring, a system including special fabric drains and/or perforated pipe drains discharging inside the wall could be used. If the permanent wall is formed away from the shoring, a system of vertical perforated drain lines and collection lines at the base of the walls could be installed. The space between the permanent and shored wall should be backfilled with free draining granular backfill. To prevent piping, a filter fabric (such as Marifi 140S) should be installed against the shoring wall.

The slab underdrain system would include a layer of free draining granular fill drained with a system of perforated pipes discharging to a disposal point. Clean-outs should be provided at selected locations in the drainage system to allow for maintenance.

A more detailed discussion of a complete underdrain system and design criteria can be provided upon request.

6.10 LOADS ON PERMANENT SLAB AND WALLS

6.10.1 Hydrostatic Pressures

As discussed in Section 6.9, there are two design alternatives for control of the ground water: designing a water-tight section; or installing permanent wall and slab drainage.

If the slab and walls are permanently drained, the Station structure does not need to be designed to resist hydrostatic pressures. If drains are not installed, it is recommended that the maximum ground water level be assumed at the elevations given below. These elevations are based on judgement and are intended to provide for possible increase in water levels over the existing measured levels.

LOCATION	ELEVATION (ft)
Northwest end	255
Southeast end	220

The hydrostatic pressures are included on the design pressure diagrams presented on Figure 6-14 for static loading conditions.

6.10.2 Permanent Static Earth Pressures

We recommend that the permanent static lateral earth pressures be based on the anticipated at-rest condition. For this condition, we recommend that the pressure be computed on the basis of an equivalent fluid with a density of 55 pcf above the ground water table and 30 pcf below the ground water table (see Figure 6-14).

The pressures on the roof should be assumed equal to the full weight of the overburden soil plus surcharge.

6.10.3 Surcharge Loads

Lateral surcharge loads from existing buildings not underpinned must be added to the lateral design earth pressure loads. The lateral surcharge loads are identical to those recommended for temporary walls. Procedures for computing these are presented on Figures 6-5 and 6-6. Vertical surcharge loads due to surface traffic, etc. should also be included in roof design. In addition, consideration should be given to loads imposed by earthmoving equipment during backfill operations.

6.10.4 Seismic Wall Pressures

Based on the analysis presented in Appendix E.3, an equivalent rectangular pressure distribution of 8 times the height of the structure is recommended as shown on Figure 6-14.

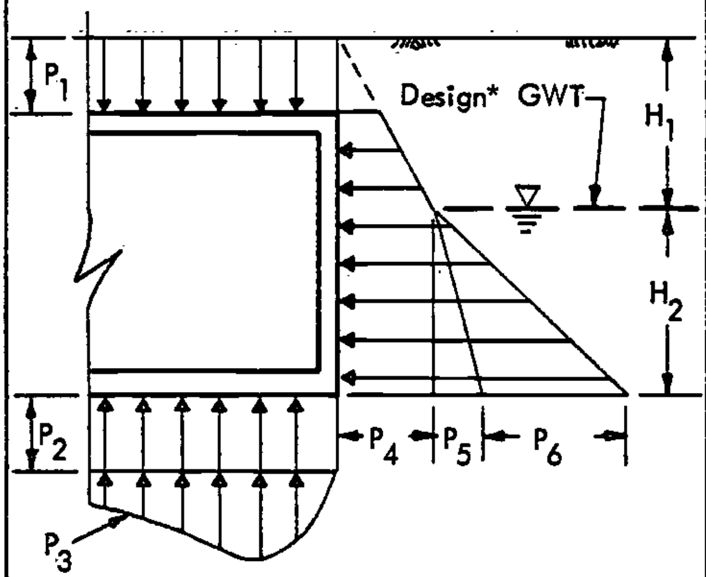
6.11 SEISMIC DESIGN CRITERIA

6.11.1 General

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

For design purposes, two levels of earthquake ground shaking have been designated. The Operating Design Earthquake (ODE) corresponds to the level of ground shaking at which critical items maintain function so that the overall system will continue to operate normally. The Maximum Design Earthquake (MDE) defines the level of ground shaking at which critical items continue the function required to maintain public safety, preventing catastrophic failure and loss of life. Design ground motion parameters for these two earthquakes are presented on Table A-2 and A-3 of Part II, Appendix A, of

STATIC LOADS WITHOUT DRAINAGE



PRESSURE	MAGNITUDE
P_1	Overburden + Surcharge
P_2	$62.4H_2$ (psf)
P_3	** Net Bearing Pressure
P_4	$55H_1$ (psf)
P_5	$30H_2$ (psf)
P_6	$62.4H_2$ (psf)

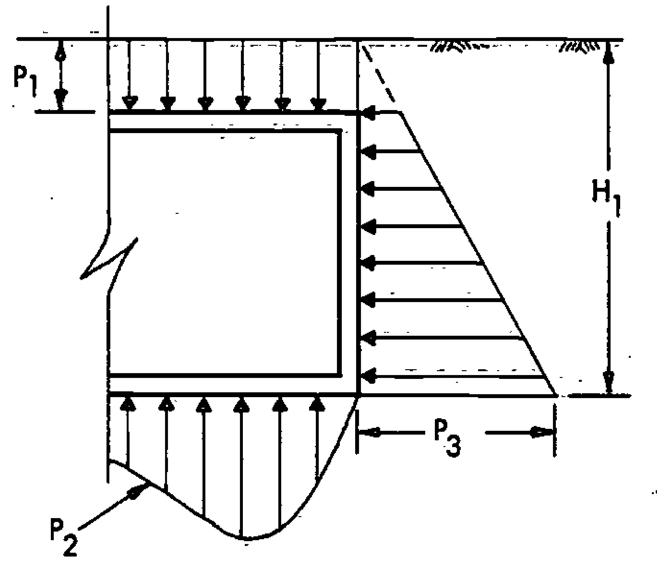
*Design Ground Water Table (GWT) Should Be Based On The Following:

NW end - Elevation 255 ft.
SE end - Elevation 220 ft.

Design water levels at intermediate points may be linearly interpolated.

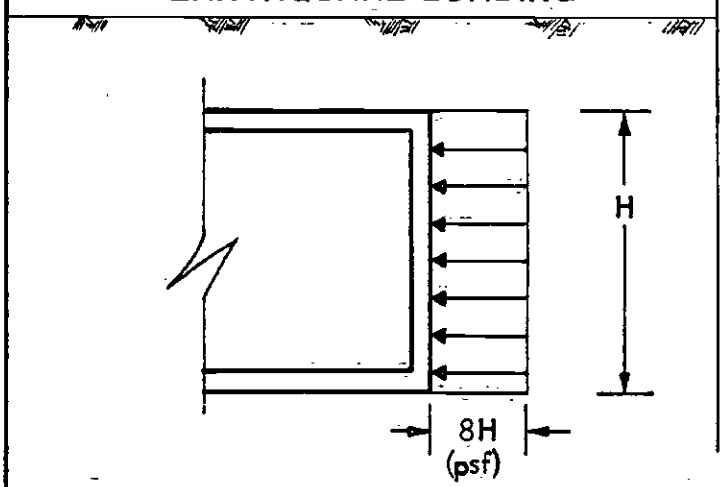
**Distribution and magnitude of bottom slab pressure could be estimated based on soil-structure interaction analysis. Such an analysis is beyond the scope of this study.

STATIC LOADS WITH DRAINAGE



PRESSURE	MAGNITUDE
P_1	Overburden + Surcharge
P_2	** Bearing Pressure
P_3	$55H_1$

EARTHQUAKE LOADING



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LOADS ON PERMANENT WALLS AND SLABS

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METRO RAIL PROJECT

Project No. 83-1101

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Figure No. 6-14

the aforementioned report. Table A-3 gives values of displacements due to fault slip which must be accounted for in design at fault crossings. Design for fault displacement is required only for MDE conditions.

Elastic free field design response spectra for use as input in seismic analysis of structural response are given in Figures A-2 and A-3 or Part II, Appendix A of the seismic design criteria report.

Where time-history type of analysis is to be used, the District will provide appropriate digitized records in the form of computer tapes or decks for ODE and MDE level events.

6.11.2 Dynamic Material Properties

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

Average values of compression and shear wave velocities based on interpretation of a limited downhole geophysical survey performed in Boring CEG-9 and other borings in similar materials during the 1981 investigation (see Appendix C) 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 and Puente bedrock are tabulated in Table 6-1.

TABLE 6-1
RECOMMENDED DYNAMIC MATERIAL PROPERTIES FOR USE IN DESIGN

		FINE-GRAINED ALLUVIUM	GRANULAR ALLUVIUM	PUENTE BEDROCK
Average Compression Wave Velocity, V_c (ft/sec)	- moist	2300	2350	5700
	- saturated		5000	
Average Shear Wave Velocity, Vs (ft/sec)		1100	1300	1300
*Poisson's Ratio		0.40	0.35	0.35
**Young's Modulus, E, (psi)	- moist	70,000	95,000	530,000
	- saturated		210,000	
**Constrained Modulus, E_c, (psi)	- moist	145,000	150,000	850,000
	- saturated		800,000	
**Shear Modulus, G_{max}, (psi)			33,000	45,000

* For saturated Alluvium, use value of 0.45.

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

The variation of dynamic shear modulus, expressed as the ratio of G/G_{max}, with the level of shear strain is presented in Figure 6-15 for the various geologic units. Similar relationships for soil hysteretic damping are presented in Figure 6-16.

6.11.3 Seismic Lateral Earth Pressures

In Section 4.5.4.5 of Part II, Appendix A, of the seismic design criteria report, a discussion of the static and dynamic soil pressures which should be considered in the design of below grade walls to resist lateral earth pressures is presented. The analysis used to calculate seismic loadings on walls is based on the well known Monobe-Okabe formulation as presented in Appendix E.3. This analysis is based on the Monobe-Okabe formulation but also includes various other assumptions based on previous experience and engineering judgement.

Results of the analysis presented in Appendix E.3 indicate that the temporary shoring system should be designed for a uniform seismic lateral earth pressure equal to 6H as shown in Figures 6-5 and 6-9. The permanent wall should be designed for an equivalent uniform lateral earth pressure equal to 8H. This value is based on a peak ground acceleration of 0.30g corresponding to the Operating Design Earthquake (ODE).

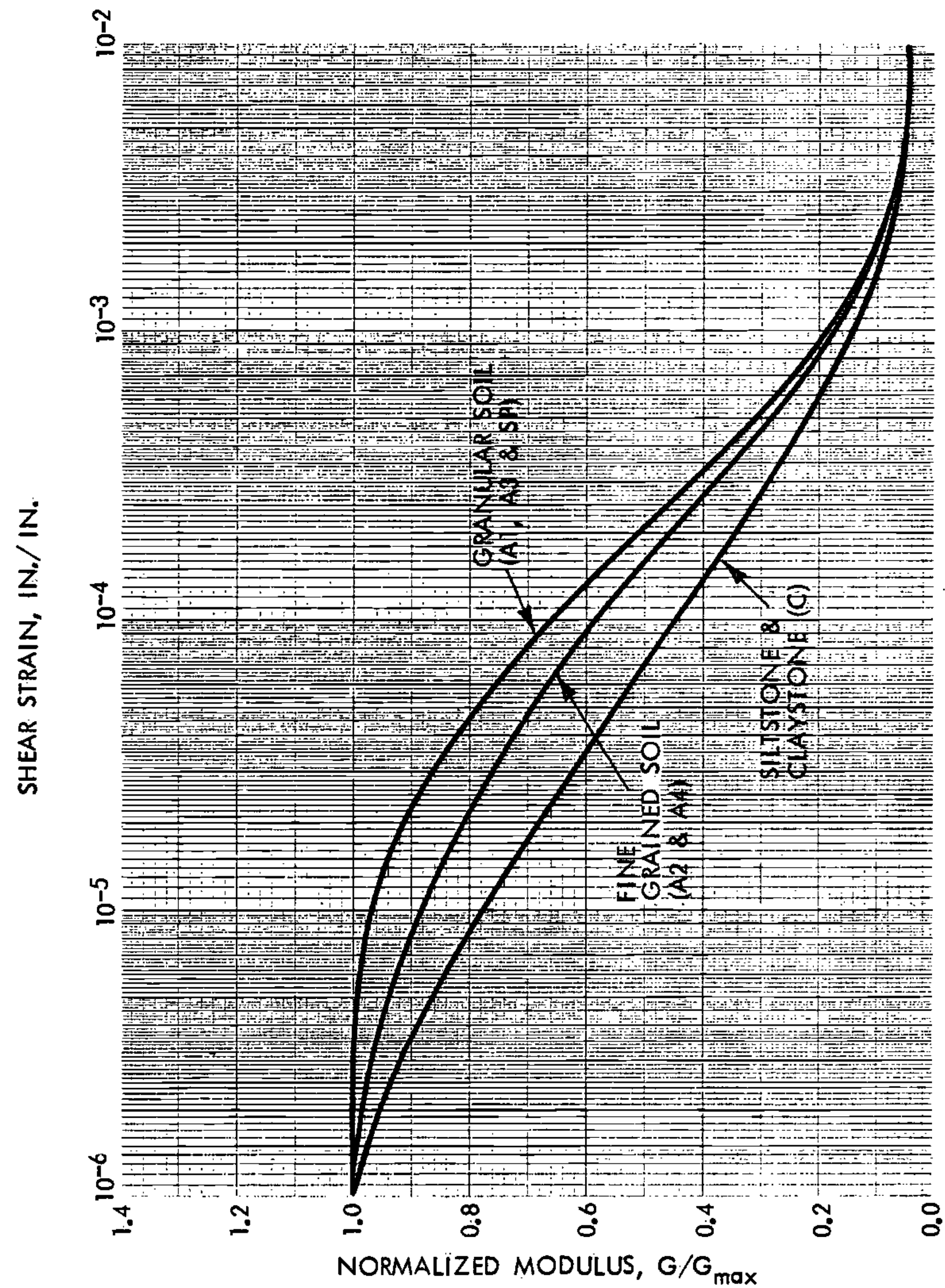
6.11.4 Liquefaction Potential

The 7th/Flower site does not have an extensive thickness of saturated granular alluvium. Locally, such as encountered in Boring 9-1, zones of alluvium may contain perched ground water. Appendix E.4 presents a liquefaction analysis based on the very conservative assumption that the site is underlain by a continuous thickness of saturated alluvium. The analysis indicates that the alluvium has a low risk of liquefaction due to its density and coarse gradation.

Based on the results of the analysis, the fact that the alluvium is not continuously saturated, and engineering judgement, it is our opinion that the alluvium underlying the site would not be subject to liquefaction during strong ground shaking produced by the postulated earthquake motions.

6.12 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. Suggested guidelines for site preparation, minor construction excavations, structural fill, foundation preparation, subgrade preparation, site surface drainage, and utility trench backfill are presented in Appendix F. Recommended specifications for compaction of fill are also presented in Appendix F. Construction specifications should clearly establish the responsibilities of the contractor for construction safety in accordance with CALOSHA requirements.



RECOMMENDED DYNAMIC SHEAR MODULUS RELATIONSHIPS

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METRO RAIL PROJECT

Project No. 83-1101

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Figure No. 6-15

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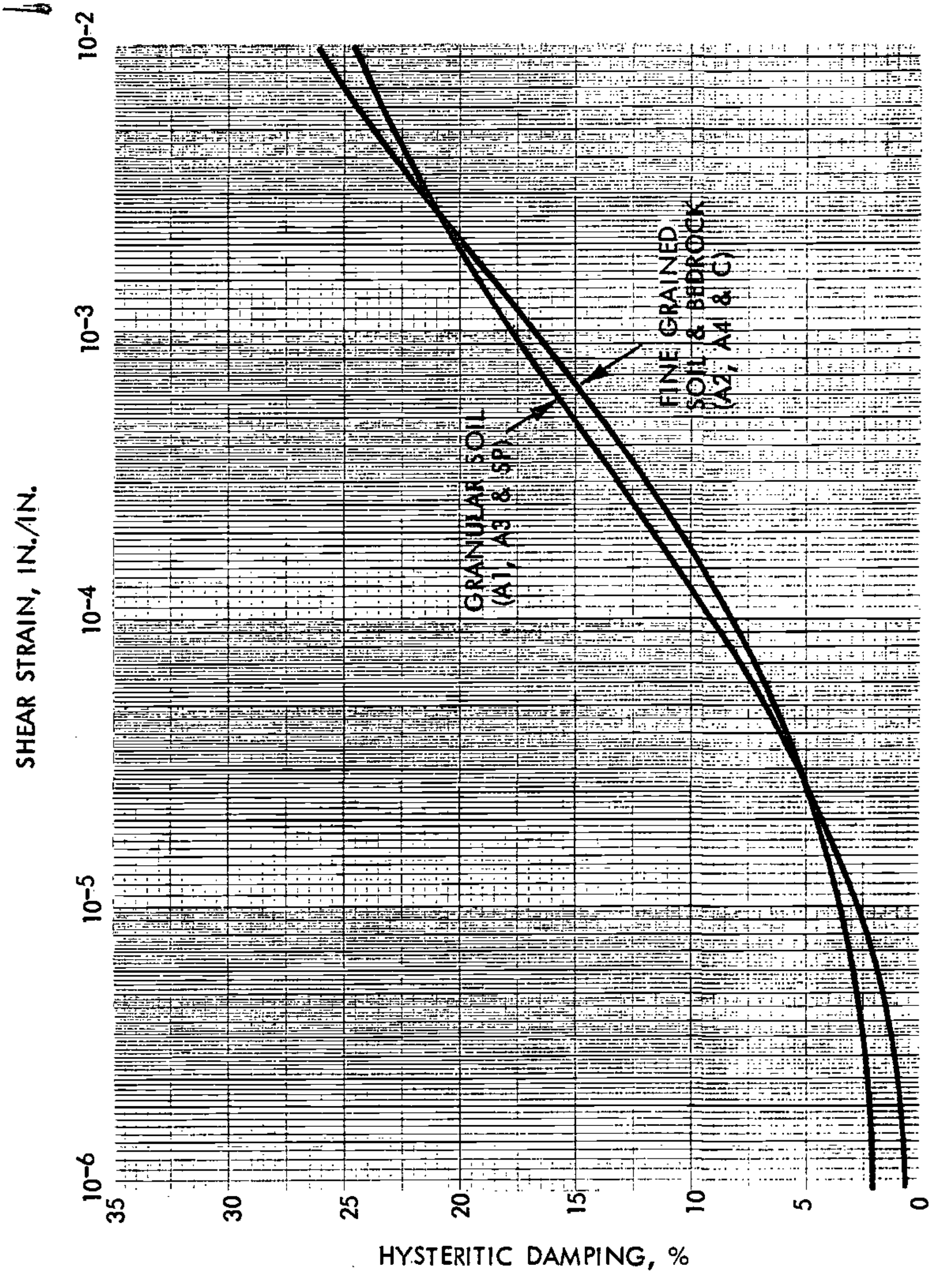

RECOMMENDED DYNAMIC DAMPING RELATIONSHIPS

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Project No.
83-1101

Figure No.
6-16

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Excavated granular alluvium (sand, silty sand, gravelly sand, sandy gravel) 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. The excavated bedrock material is not considered suitable due to its fine-grained nature which will make compaction difficult and could lead to fill settlement problems after construction. If the granular alluvium materials cannot be stockpiled, imported granular soils could be used for fill, subject to approval by the soils engineer.

6.13 SUPPLEMENTARY GEOTECHNICAL SERVICES

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

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

- 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 contract plans and specifications.

- Shoring Plan Review: Assuming that the shoring system is designed by the contractor, a qualified geotechnical engineer should review the proposed system in detail including review of engineering computations. This review is not a certification of the contractor's plan but rather an independent review made with respect to the owner's interests.

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

References

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.

LOS ANGELES COUNTY FLOOD CONTROL DISTRICT, Engineering Geology Section, Materials Engineering Division, 1973, Exploratory drilling and testing for proposed (Sacatella) tunnel sections: Geologic Rept. project 1102, file 364-1102.41, Dec. 26, 1973, 12 p.

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, Production of movements for braced cuts in clay: ASCE Geotechnical Journal, June.

MASTERS, J.A., 1979, Deep basin gas trap, western Canada: Bull. AAPG, Vol. 63, No. 2, p. 152-181.

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.

NEWMAN, S.P., 1975, Analysis of Pumping Test Data from Anisotropic Unconfined Aquifers Considering Delayed Gravity Responses: Water Resources Research, Vol. 11, No. 2, p. 329-342.

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.

PROCTOR, R.J., 1981, Sacatella tunnel oil seeps; personal communication.

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.

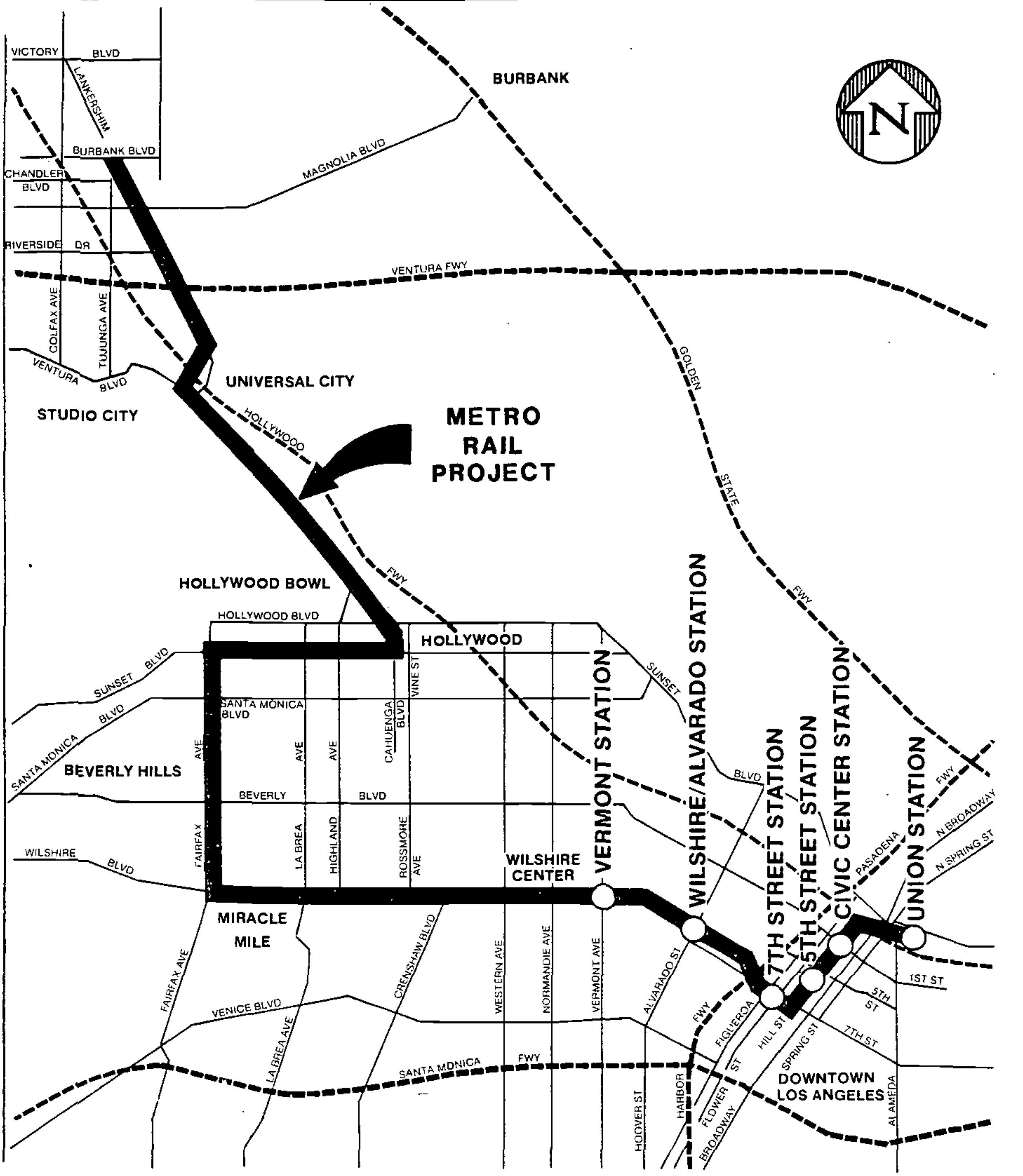
SEED, H.B, and Whitman, R.V., 1970, Design of earth retaining structures for dynamic loads in Ground and Design of Earth Retaining Structures: ASCE Specialty Conference on Lateral Stresses New York, p. 103-148.

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.

WRIGHT, V.L., 1975, Pre-bid geologic proposal (Sacatella Tunnel): Los Angeles County Flood Control District, regional proj. 1102, unit 4, 14 p.

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



VICINITY MAP

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Drawing No.
1

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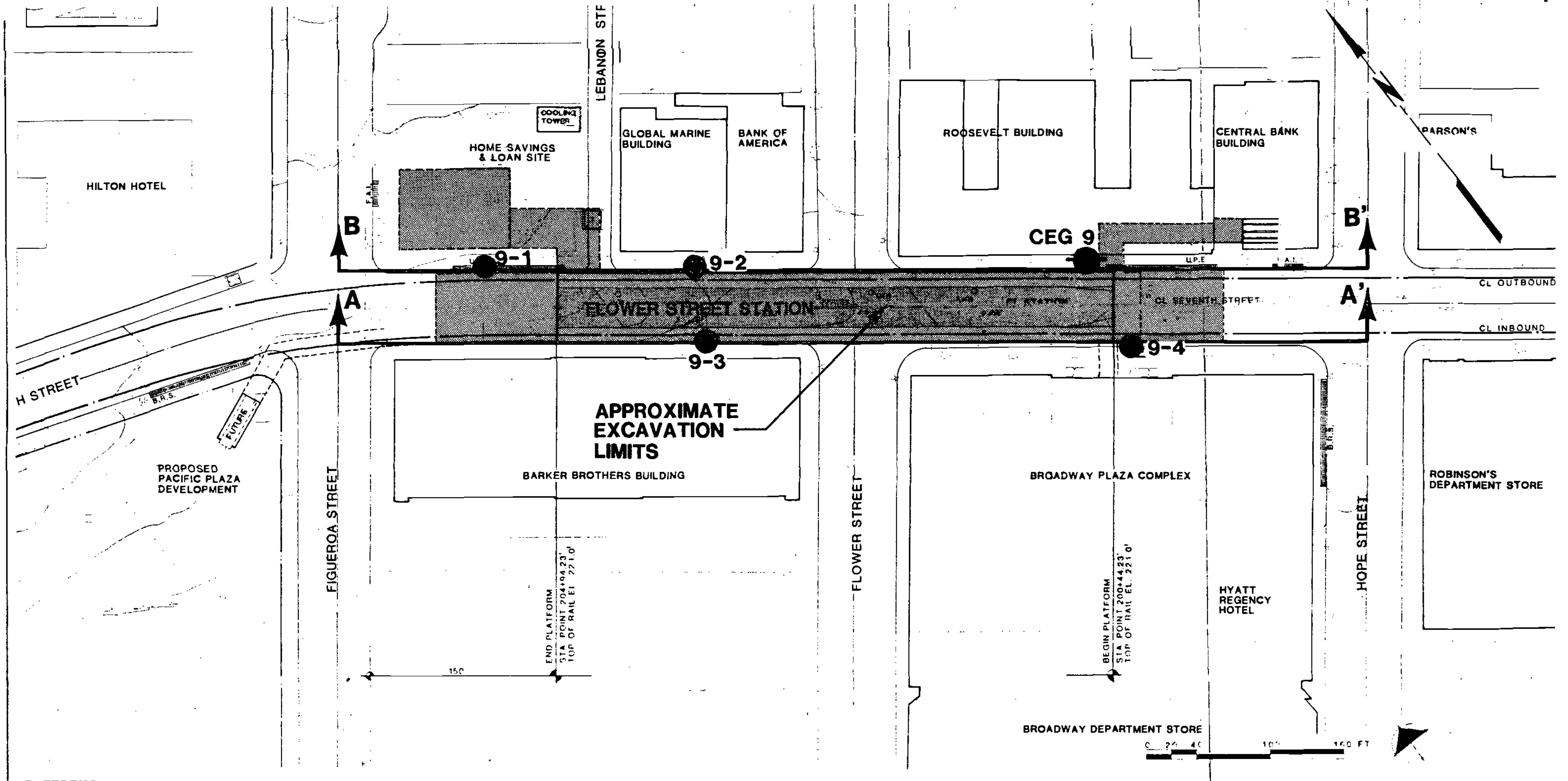

REFERENCE:
"PRELIMINARY 7TH/FLOWER STREET STATION SITE PLAN" PREPARED BY HARRY WEESE & ASSOCIATES, ORIGINAL SCALE 1"=40' REDUCED TO 1"=80'.

NOTES:

1.) FOR SECTIONS A-A' AND B-B' SEE DRAWINGS 4 & 5

2.) FOR EXPLANATION SEE DRAWING NO. 6

LOCATION OF BORINGS

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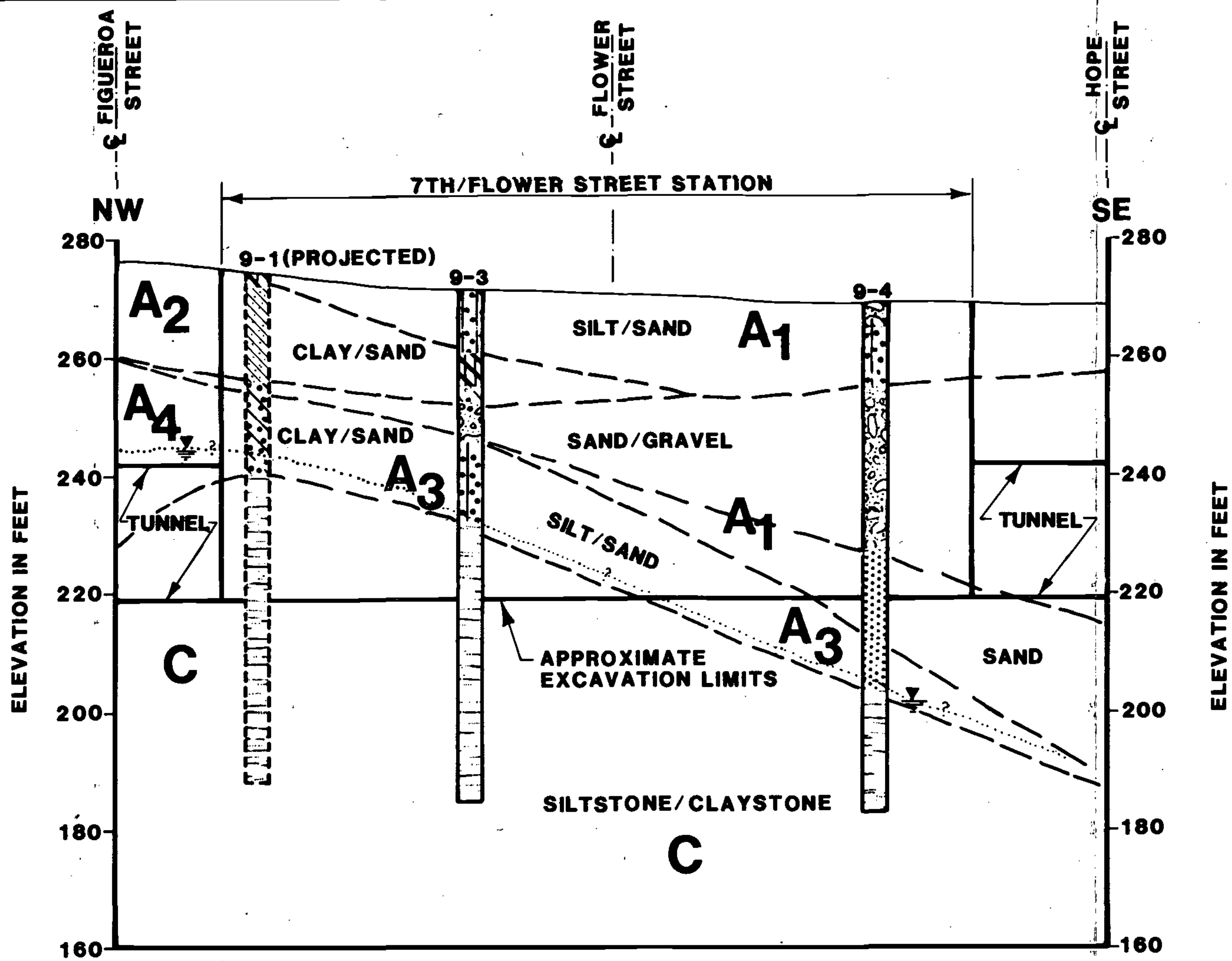
Scale	As Shown
Date	10-11-83
Prepared by	RG
Checked by	HAS
Approved By	RMP
Project No	83-1101
Drawing No	3

BRUNING 40-107



SCALE: HOR. 0 50 100 200 feet

VER. 0 5 10 20 40 feet

NOTES:

1.) FOR SECTION LOCATION SEE DRAWING NO. 3

2.) FOR EXPLANATION OF SYMBOLS SEE DRAWING NO. 6

GEOLOGIC SECTION A-A'

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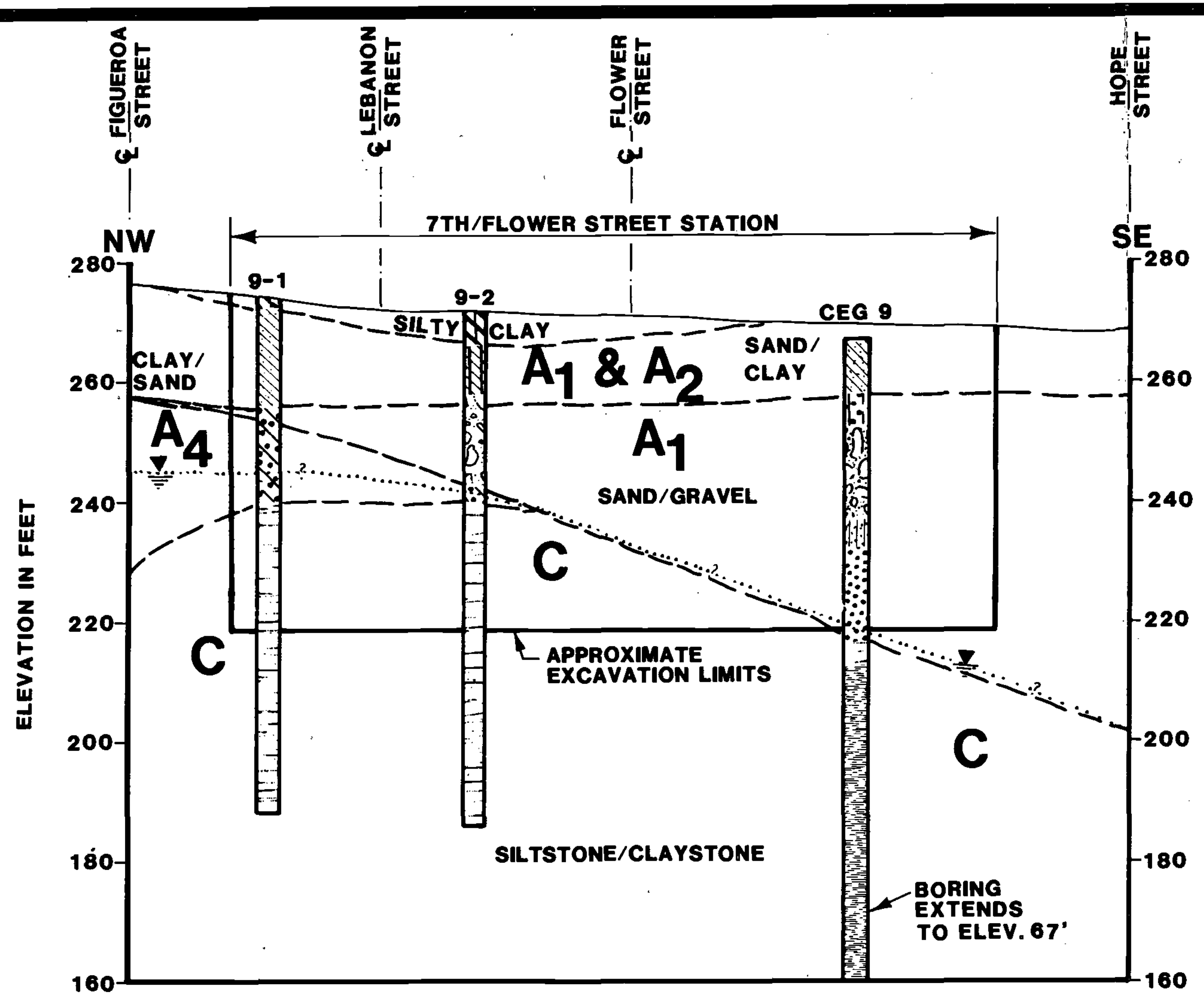
Scale: As Shown	Project No. 83-1101
Date: 10-11-83	
Prepared by: RG	Drawing No.
Checked by: HAS	4
Approved By: RMP	



SCALE: HOR. 0 50 100 200 feet

VER. 0 5 10 20 40 feet

NOTES:
1.) FOR SECTION LOCATION SEE DRAWING NO. 3
2.) FOR EXPLANATION OF SYMBOLS SEE DRAWING NO. 6

GEOLOGIC SECTION B-B'

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Scale	As Shown
Date	10-11-83
Prepared by	RG
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Approved By	RMP
Project No.	83-1101
Drawing No.	5

BRUNING 40-107

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)*

TERTIARY — MIOCENE

3 — Terzaghi Rock Condition Number
— Approximate boundary between Terzaghi numbers

2-5 TOPANGA FORMATION: Conglomerate, sandstone, and siltstone; thickly bedded; primarily hard and strong (Geologic symbol Tt).

1-5 TOPANGA FORMATION: Basalt; intrusive, primarily hard and strong (Geologic symbol Tb).

TERZACHI ROCK CONDITION NUMBERS:*

1 Hard and intact

2 Hard and stratified or schistose

3 Massive, moderately jointed

4 Moderately blocky and seamy

5 Very blocky and seamy (closely jointed)

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

7 Squeezing rock, moderate depth

8 Squeezing rock, great depth

9 Swelling rock

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

SYMBOLS

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

NOTES:

1) The geologic sections are based on interpolation between borings and were prepared as an aid in developing design recommendations. Actual conditions encountered during construction may be different.
2) The locations of the tunnel line and stations are based on the Metro Rail Project, Milestone 10 alignment as of February, 1983.
3) Borings projected more than 200' to the profile line were considered in some of the interpretation of subsurface conditions. However, final interpretation is based on numerous factors and may not reflect the boring logs as presented in Appendix A.
4) Displacements shown along faults are graphic representations. Actual vertical offsets are unknown.

SILT
CLAY
SANDY SILT
SANDY CLAY
CLAYEY SILT
SILTY CLAY
SILTY SAND
CLAYEY SAND
SAND
GRAVELLY SAND
SANDY GRAVEL
GRAVEL
GRAVELLY CLAY
TAR SILT & CLAY
TAR SAND
FILL
SILTSTONE
CLAYSTONE
INTERBEDDED SANDSTONE WITH SILTSTONE OR CLAYSTONE
SANDSTONE
SANDSTONE, CONGLOMERATE
CEMENTED ZONE
META—SANDSTONE
BASALT
BRECCIA
SHEAR ZONE

GEOLOGIC EXPLANATION

DESIGN UNIT A165 Southern California Rapid Transit District METRO RAIL PROJECT	Scale N/A	Project No 83-1101
	Date 10-11-83	
	Prepared by RG	Drawing No
Converse Consultants — Geotechnical Engineering and Applied Sciences	Checked by JAD	6
	Approved By HAS	

BRUNING 40-107

Appendix A
Field Exploration

APPENDIX A FIELD EXPLORATION

A.1 GENERAL

Field exploration data presented in this report for Design Unit A165 includes information from borings drilled for the 1981 Geotechnical Investigation Report and additional borings drilled for this investigation. One boring (CEG-9) was drilled at Design Unit A165 during the 1981 investigation and the log is reproduced in this appendix. Four additional borings (9-1 to 9-4) were drilled in 1983 for this investigation. Locations of the borings are shown on Drawings 2 and 3. Ground water observation wells (piezometers) were installed in borings CEG-9, 9-1 and 9-4. A geophysical downhole survey was made at boring CEG-9 (see Appendix C).

The borings were drilled to depths generally ranging from 85 to 200 feet, and all of the borings penetrated through the alluvium (Units A_1, A_2, A_3 and A_4) into the underlying bedrock (Unit C). All borings were sampled at regular intervals using the Converse ring sampler, pitcher barrel sampler and the standard split spoon sampler. Sample and core recovery was essentially 100% in the siltstone and claystone bedrock (Unit C) but only about 78% in 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 <u>Technical Staff</u>

Members of three firms (CWDD/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 log of the rotary wash cuttings and for sample/core identification, labeling and storage of all samples, and installation of piezometer pipe, gravel pack and bentonite seals.

A.2.2 <u>Drilling Contractor and Equipment</u>

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

A.3 SAMPLING AND LOGGING PROCEDURES

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

A.3.1 Sampling

In the overburden at about 10-foot intervals, the Converse ring sampler was driven using a down-hole 450-pound slip-jar hammer. The Converse sampler was followed with the standard split spoon sample (SPT) driven with a 140-pound hammer with a 30 inch stroke. Where the Puente Formation was encountered, the borings were sampled using a Pitcher-Barrel and Converse ring sampler at 20-foot intervals.

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

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

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

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

A.3.2 Field Classification of Soils

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

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

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

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

A.3.3 Field Description of the Formations

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

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

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

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

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

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

TABLE A-2 Bedrock Description Terms

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

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

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

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

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

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

A.4 PIEZOMETER INSTALLATION

Piezometers were installed in boring CEG-9 in 1981 and two additional piezometers were installed in borings 9-1 and 9-4 in 1983. Procedures for piezometer installation were the following:

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

Converse Consultants

Boring Log 9-1

THIS LOG IS APPLICABLE ONLY AT THIS LOCATION AND TIME. CONDITIONS MAY DIFFER AT OTHER LOCATIONS OR TIME.

PROJECT 83-1101-71 DATE DRILLED 2/20-21/83 HOLE NO. 9-1
LOCATION 7th Street @ Figueroa @ Home Savings GROUND ELEV. 273.5'
DRILLING CONTRACTOR Pitcher LOGGED BY L. Schoebeden DEPTH TO GROUND WATER —
TYPE OF RIG Failing 1500 HOLE DIAMETER 4 7/8" HAMMER WEIGHT AND FALL 140 lb @ 30"
SURFACE CONDITIONS concrete sidewalk TOTAL DEPTH 85 NO. CORE BOXES 0

DEPTH	CLASS.	FIELD DESCRIPTION	SAMPLE	SPT (6")	DRILL MODE	RUN NO.	CORE REC. %	REMARKS
0.0	AC / CL	0.0-0.2 concrete. YOUNG ALLUVIUM: 0.2-2.5 SILTY CLAY: greyish brown (5YR 3/2); 95% moderately plastic fines; 5% fine sand; stiff; moist.			GB			move and set up 3:30 pm drilled through walk w/ garbage barrel encountered signal conduit move and start again 5 p.m.
2.0	CL	2.5-4.7 SANDY CLAY: moderate brown (5YR 3/4); 80% moderately plastic fines; 20% fine sand; stiff; moist.			AO			
4.0	CL	4.7-17.5 SANDY CLAY: moderate brown (5YR 4/4); 75% moderately plastic fines; 25% fine sand; very stiff; moist.	S-1	4, 9, 11	SS			1.5/1.5 100% recovery
6.0					RO			
8.0								2/20/83 2/21/83 H_2O up to casing top at start
			C-1	5, 7	DR			1.0/1.0 100% recovery
10.0					RO			
		contains clayey sand lenses to 2"	S-2	7, 8, 11	SS			0.7/1.5 50% recovery
12.0					RO			
14.0								
16.0				5, 8, 18	SS			0.0/1.5 0 recovery
	SC	17.5-34 CLAYEY GRAVEL & SAND: moderate brown (5YR 4/4); 50% medium to coarse sand; 30% gravel; 20% silty clay; very dense; moist; interbedded with silty clay and silty sand.			RO			rig chatter
18.0		18.0' - silty sand lense 4"		31 / 20-½"	DR / RO			0/0.7 0 recovery
20.0								

SHEET 1 OF 5

PROJECT 83-1101-81 DATE DRILLED 2/20-21/83 HOLE NO. 9-1

DEPTH	CLASS.	FIELD DESCRIPTION	SAMPLE	SPT (6")	DRILL MODE	RUN NO.	CORE REC. %	REMARKS
20.0	SC	17.5 - 34 CLAYEY SAND: interbedded with gravel silty sand, sand, and silty clay. (cont.)			RO			mixed mud 2 sacks
			J-3	8 20 50-3"	SS			0.7/1.3 50% recovery
22.0		22.5 silty clay lense			RO			
24.0		24.5 gravelly zone						
26.0		26.5 silty sand		25-0"	SS RO			refusal
		27.5 3" gravelly zone						
28.0			C-2	125	PR RO			0.5/0.5 100% recovery
30.0			J-4	100-6"	SS RO			0.5/0.5 100% recovery
32.0		33' sandy clay lense						
		FERNANDO FORMATION						
34.0		34-34.5 WEATHERED SILTSTONE: dusky yellow (5Y 6/4)						
		34.5 - 85.0 CLAYEY SILTSTONE: dark olive grey (5Y 3/1); 100% moderately plastic fines; contains mica; massive bedding; concretions or cemented zone @ 36.9' (3"); moist;						
36.0			J-5	7 50-4.5"	SS RO			0.9/0.9 100% recovery
38.0		Physical Condition: little fractured to massive; soft to friable hardness, plastic to friable strength, fresh.	PB-1		PB			2.5/2.5 100% recovery
40.0		39.1 1½" diameter egg shaped concretion smooth on one side lumpy on the other						
42.0		Sample J-6: contains light greenish grey (5G 8/1) sand blebs 1/8" thick 1/4" long and shells sulfur odor	J-6	10 20 30	SS RO			1.5/1.5 100% recovery
44.0								

PROJECT 83-1101-81 DATE DRILLED 2/20-21/83 HOLE NO. 9-1

DEPTH	CLASS.	FIELD DESCRIPTION	SAMPLE	SPT (6")	DRILL MODE	RUN NO.	CORE REC. %	REMARKS
44.0		34.5 - 85.0' CLAYEY SILTSTONE : cont.			RO			
46.0		Sample J-7 : small cemented concretions within shell zone	J-7	10 / 17 / 50-4.5"	SS			1.4/1.4 100% recovery
48.0					RO			
		decrease in shell fragments	C-3	23 / 39	CR			1.0/1.0 100% recovery
50.0					RO			
			J-8	8 / 14 / 24	SS			1.5/1.5 100% recovery
52.0					RO			
54.0								
56.0			J-9	8 / 14 / 23	SS			1.5/1.5 100% recovery
58.0					RO			
			PB-2		PB			2.5/2.5 100% recovery
60.0								
		grading siltier interbeds (2") of claystone within siltstone	J-10	12 / 21 / 32	SS			1.5/1.5 100% recovery
62.0					RO			
64.0								
66.0		odorless	J-11	14 / 25 / 44	SS			1.5/1.5 100% recovery
68.0					SS			

SHEET 3 OF 5

PROJECT 83-1101-81 DATE DRILLED 2/20-21/83 HOLE NO. 9-1

DEPTH	CLASS.	FIELD DESCRIPTION	SAMPLE	SPT (6")	DRILL MODE	RUN NO.	CORE REC. %	REMARKS
68.0		34.5-85' CLAYEY SILTSTONE: cont.			RO			
				40	DR			0.9/0.9
70.0		gasoline odor	C-4	50-5"				100% recovery
					RO			
		Sample J-12: shells and 1/8" x 1/4" lath shaped light greenish grey (5G 8/1) fine sand blebs	J-12	12	SS			1.5/1.5
72.0				28				100% recovery
				32				
					RO			
74.0								
76.0								
				12	SS			1.5/1.5
78.0		becoming slightly less dense	J-13	28				100% recovery
				29				
					RO			
80.0		probable concretion						minor chatter
					PB			2.1/2.3
82.0			PB-3					100% recovery
		high concentration of shells.						refusal of PB.
			J-14	45	SS			1.0/1.0
84.0				50-5.5"				100% recovery
					RO			
86.0		B.H. - 85.0' Terminated hole; drilled out to bottom of spoon and flushed with fresh water; installed 2" ABS; 45-85' slotted and wrapped in filter fabric for top 15' of slotted section						finish drilling 3:30 Converse Sampler driven with 300 lb hammer @ 18" drop; Pitcher samples cut with 200 p.s.i.

SHEET 4 OF 5

SUMMARY BORING NO. 9-1

PROJECT 83-1101-71 STATION HOLE yes DATE DRILLED 2/20-21/83

OVERBURDEN DEPTH (FT.) 0 TO 34.0'.

BEDROCK DEPTH (FT.) 34.0 TO 85 (T.D.).

WATER PRESS. TEST NO; INTERVAL(S) ____ TO ____, ____ TO ____.

GROUND WATER DEPTH (FT.) 29 DATE 4/4/83; ____ DATE ____.

GAS ~~SULFUR~~ GASOLINE; DEPTH FIRST NOTICED 42', DATE 2/21/83

E-LOG NO.

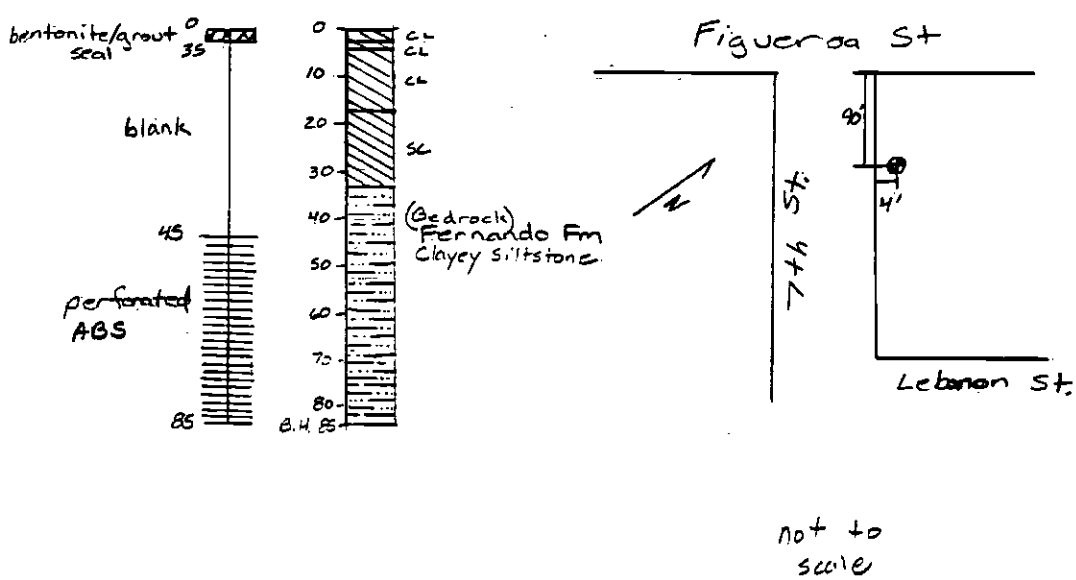
DOWN-HOLE SURVEY NO.

CROSS-HOLE SURVEY NO.

PVC CASING (I.D.): 4" ____ TO ____; 3" ____ TO ____; 2" 0 TO 85'.

GROUND ELEVATION REF. RTD SITE PLAN EL-A-1 (1"=40') Weese & Assoc 2/10/83

SKETCH



SHEET 5 OF 5

Converse Consultants

Boring Log 9-2

THIS LOG IS APPLICABLE ONLY AT THIS LOCATION AND TIME. CONDITIONS MAY DIFFER AT OTHER LOCATIONS OR TIME.

PROJECT 83-1101-71 DATE DRILLED 2/18-19/83 HOLE NO. 9-2
LOCATION 7th between Flower and Figuroa N side GROUND ELEV. 270
DRILLING CONTRACTOR Pitcher LOGGED BY L. Schoeberlein DEPTH TO GROUND WATER —
TYPE OF RIG Failing 1500 HOLE DIAMETER 4 7/8" HAMMER WEIGHT AND FALL 140 lb @ 30"
SURFACE CONDITIONS concrete sidewalk TOTAL DEPTH 84.5 NO. CORE BOXES 0

DEPTH	CLASS.	FIELD DESCRIPTION	SAMPLE	SPT (6")	DRILL MODE	RUN NO.	CORE REC. %	REMARKS
0.0	AC / GC / CL	0.0-0.2 Concrete 0.2-0.4 CLAYEY GRAVEL: YOUNG ALLUVIUM 0.4-5.0 SILTY CLAY: olive black (5Y 2/1); 100% low plasticity fines; stiff; moist.			GB			moved and set up 12:30 pm. 3 attempts to start hole, encountered 2 steel pipes in first 2 holes. Drilling started at 2 pm
2.0								
4.0		gradual color change to greyish brown (5YR 3/2)						
	SC	5.0-13.0 CLAYEY SAND: moderate brown (5YR 3/4); 75% fine sand; 25% moderately plastic fines; dense; moist; upper contact gradational.	J-1	6 12 17	SS			1.5/1.5 100% recovery
6.0					RO			
8.0								
			C-1	5 9	DR			1.0/1.0 100% recovery pocket pen 3.5
10.0		gravel lense 3/8" diameter			RO			
			J-2	8 9 16	SS			1.5/1.5 100% recovery
12.0					RO			
	CL	13.0-14.0 CLAY lense						
14.0	SW	14.0-29.0 GRAVELLY SAND: variegated black, brown and white; 80% medium to coarse angular sand; 18% fine to medium rounded gravel - granitic in composition; 2% silt; very dense; wet.						
16.0								circulation loss mixed mud
			J-3	16 40 50-4"	SS			1.3/1.3 100% recovery
18.0					RO			
20.0								

SHEET 1 OF 5

PROJECT 83-1101-71 DATE DRILLED 2/18-19/83 HOLE NO. 9-2

DEPTH	CLASS.	FIELD DESCRIPTION	SAMPLE	SPT (6")	DRILL MODE	RUN NO.	CORE REC. %	REMARKS
20.0	SW	14.0 - 29.0 GRAVELLY SAND : cont.		43 50-3"	DR RO			0.0/0.8 0 recovery mixed mud
22.0					SS RO			0 drive spoon bouncing 23' drilling smoother
24.0		cobble zone						rig chatter
26.0			J-4	50-4"	SS RO			0.3/0.3 100% recovery 2/18/83 2/19/83 H_2O @ 11' in am no cave in
28.0	SC	29.0 - 31.0 CLAYEY SAND : dusky yellow (5Y 6/4) to moderate yellow (5Y 7/6), moderately plastic.						
30.0		FERNANDO FORMATION 31.0 - 85.0 CLAYEY SILTSTONE : dark greenish grey (5GY 4/1); 100% moderately plastic fines; contains mica; massive bedding.		55-5"	DR RO			0.0/0.2 0% recovery
32.0			J-5	12 14 21	SS RO			1.5/1.5 100% recovery
34.0		Physical Condition: little fracturing to massive; soft to friable hardness. plastic to friable strength; fresh.						
36.0			J-6	11 26 30	SS RO			1.5/1.5 100% recovery
38.0								
40.0			PB-1		PB			pitcher barrel too large for gravel section, hole crooked; added drill collar added to straighten. some chatter 180 p.s.i. 1.1/2.5 recovery
42.0								
44.0				14	SS			

SHEET 2 OF 5

PROJECT 83-1101-71 DATE DRILLED 2/18/83 HOLE NO. 9-2

DEPTH	CLASS.	FIELD DESCRIPTION	SAMPLE	SPT (6")	DRILL MODE	RUN NO.	CORE REC. %	REMARKS
44.0		31.0-85.0 CLAYEY SILTSTONE: cont. sample J-7: contains shells	J-7	24 45	SS			1.5/1.5 100% recovery
					RD			
46.0								
48.0			J-8	11 20 40	SS			1.5/1.5 100% recovery
50.0					RO			
52.0		clay content decreasing	C-2	25 50-5"	DR			1.0/1.0 100% recovery
					RD			
54.0		sample J-9: contains some minor very fine sand lenses; sulfur odor present	J-9	12 20 34	SS			1.5/1.5 100% recovery
56.0					RO			
58.0								
60.0								
62.0			PB-2		PB			2.0/2.5 80% recovery
64.0			J-10	9 17 30	SS			1.5/1.5 100% recovery
66.0					RD			
68.0								

SHEET 3 OF 5

PROJECT 83-1101-71 DATE DRILLED 2/18/83 HOLE NO. 9-2

DEPTH	CLASS.	FIELD DESCRIPTION	SAMPLE	SPT (6")	DRILL MODE	RUN NO.	CORE REC. %	REMARKS
68.0		31.0-85.0 CLAYEY SILTSTONE: cont. 68' gravelly zone strong sulfur odor			RD			rig chatter
			C-3	25 / 50-3"	DR			0.7/0.7 100% recovery
70.0					RD			
72.0		continued massive bedding	J-11	11 / 20 / 31	SS			1.5/1.5 100% recovery
					RD			
74.0								
76.0			J-12	10 / 23 / 37	SS			1.5/1.5 100% recovery
78.0					RO			
80.0								
82.0			PB-3		PB			1.3/2.5
84.0			J-13	13 / 50	SS			1.0/1.0 100% recovery
86.0		B.H. 84.5' Terminated hole: hole grouted to surface and topped off w/ concrete						drilling completed 3:50 pm; Converse sampler driven with 300 lb hammer @ 18" drop; Pitcher samples cut with 200 p.s.i.
88.0								

SUMMARY BORING NO. 9-2

PROJECT 83-1101-71 STATION HOLE yes DATE DRILLED 2/18-19/83.

OVERBURDEN DEPTH (FT.) 0 TO 31'.

BEDROCK DEPTH (FT.) 31 TO 84.5 (T.D.).

WATER PRESS. TEST NO; INTERVAL(S) ____ TO ____, ____ TO ____.

GROUND WATER DEPTH (FT.) 11' DATE 2/19/83 during drilling; ____ DATE ____.

GAS SULFUR; DEPTH FIRST NOTICED 54', DATE 2/19/83.

E-LOG NO.

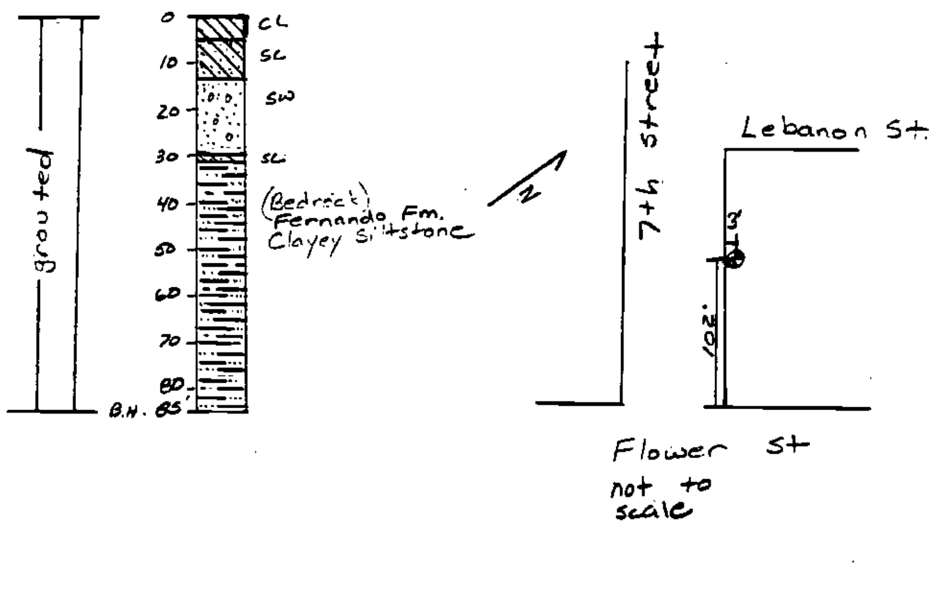
DOWN-HOLE SURVEY NO.

CROSS-HOLE SURVEY NO.

PVC CASING (I.D.): 4" ____ TO ____; 3" ____ TO ____; 2" ____ TO ____.

GROUND ELEVATION REF. RTO SITE PLAN FL-A-1 (1"=40') Weese & Assoc. 2/10/83

SKETCH



Converse Consultants

Boring Log 9-3

THIS LOG IS APPLICABLE ONLY AT THIS LOCATION AND TIME. CONDITIONS MAY DIFFER AT OTHER LOCATIONS OR TIME.

PROJECT 83-1101-71 DATE DRILLED 2/22-23/83 HOLE NO. 9-3
LOCATION 7th Street ~100' W. of Flower GROUND ELEV. 269.0'
DRILLING CONTRACTOR Pitcher LOGGED BY L. Schoeberle DEPTH TO GROUND WATER –
TYPE OF RIG Failing 1500 HOLE DIAMETER 4⅞" HAMMER WEIGHT AND FALL 140 lb @ 30"
SURFACE CONDITIONS Concrete gutter TOTAL DEPTH 85' NO. CORE BOXES 0

DEPTH	CLASS.	FIELD DESCRIPTION	SAMPLE	SPT (6")	DRILL MODE	RUN NO.	CORE REC. %	REMARKS
0.0	AC GC SM	0.0-0.6 Concrete 0.6-0.9 CLAYEY GRAVEL: YOUNG ALLUVIUM 0.9-10.5 SILTY SAND: moderate brown (5YR 4/4). 80% very fine sand, 20% low plasticity fines. moderately dense; moist.			GB			set up 9am start drilling 9:30 Set 8' of 5" steel surface casing
2.0								
4.0								
6.0			J-1	1 5 10	SS RO			1.5/1.5 100% recovery
8.0								
10.0			C-1	5 7	DR RD			1.0/1.0 100% recovery pocket pen 1.0
	CL	10.5-16.0 SILTY CLAY: greyish brown (5YR 3/2), 100% low to moderately plastic fines; stiff to very stiff; moist; inter-bedded with thin (1") sand lenses.						
12.0			J-2	9 12 9	SS RO			1.5/1.5 100% recovery
14.0								
16.0	GC	16.0-18.0 CLAYEY SAND & GRAVEL: mottled brown, black, white; 60% extremely weathered granitic gravel 30% sand. 20% moderately plastic fines; very dense, moist; inter-bedded clay zones in gravels.	J-3	34 50-5"	SS RO			0.9/0.9 100% recovery mixed mud 5 sacks
18.0	SW	18.0-25.0 GRAVELLY SAND:						rig bouncing moved sample up to 20'
20.0		see description next page						

SHEET 1 OF 5

PROJECT 83-1101-71 DATE DRILLED 2/22-23/83 HOLE NO. 9-3

DEPTH	CLASS.	FIELD DESCRIPTION	SAMPLE	SPT (6")	DRILL MODE	RUN NO.	CORE REC. %	REMARKS
20.0	SW	18.0-25' GRAVELLY SAND: black, brown and white varigated; 80% medium to coarse sand, angular, granitic; 20% fine to medium granitic gravel; very dense; moist.		126	DR			0 /0.5 recovery
					RD			losing circulation
22.0								
24.0								
	SM	25.0-38.5 SILTY SAND: moderate yellowish brown (10YR 5/4); 80% very fine sand 20% nonplastic fines. very dense; moist.						
26.0			J-4	21 / 33 / 48	SS			1.5/1.5 100% recovery
					RD			
28.0								
		grading coarser to fine to medium sand w/ less silt	C-2	31 / 50-1"	DR			0.8/0.8 100% recovery pocket pen 2.5
30.0					RD			
		Sample J-5 31' medium sand trace silt 32' clayey sand sandy clay lenses with coarse sand and gravel	J-5	23 / 36 / 50-5"	SS			1.4/1.4 100% recovery
32.0	SC/CL				RD			
	SM	33.5 silty fine sand as above w/ Fe staining mottled w/ dark yellowish orange (10YR 6/6) and dusky yellow (5Y 6/4)						
34.0								
36.0			J-6	16 / 31 / 50-4"	SS			1.3/1.3 100 % recovery
					RD			
38.0		FERNANDO FORMATION						lost a little circulation throughout hole
		38.5-85 CLAYEY SILTSTONE: greenish black (5G 2/1); 100% low to moderately plastic fines; contains mica; massive bedding.	C-3	7 / 13	DR			0.8/1.0 80% recovery
40.0					RD			
			J-7	9 / 14 / 21	SS			1.5/1.5 100% recovery
42.0		Physical Condition: little fractured to massive; soft to friable hardness; plastic to friable strength; fresh.			RD			
44.0								

SHEET 2 OF 5

PROJECT 83-1101-71 DATE DRILLED 2/22-23/83 HOLE NO. 9-3

DEPTH	CLASS.	FIELD DESCRIPTION	SAMPLE	SPT (6")	DRILL MODE	RUN NO.	CORE REC. %	REMARKS
44.0		38.5- 85 CLAYEY SILTSTONE: cont. some massive bedding			RO			lost all circulation mixed mud drove casing to 22'
46.0			J-8	9 17 24	SS			1.5/1.5 100% recovery
48.0					RO			
50.0		contains occasional rounded gravels	PB-1		PB			2.5/2.5 100% recovery
52.0		slight sulfur odor	J-9	6 12 19	SS			1.5/1.5 100% recovery
54.0					RO			
		shells and concretions						rig chatter
56.0		lower clay content	J-10	20 25 28	SS			1.5/1.5 100% recovery
58.0		concretions			RO			rig chatter
		60' sulfur odor shells (small amount)	C-4	18 41	DR			0.9/1.0 90% recovery plastic rings [illegible] of water
60.0					RO			continuous loss of circulation
62.0			J-11	12 24 50-5"	SS			1.4/1.4 100% recovery
					RO			
64.0								rig chatter
66.0		strong sulfur odor, tiny shells	J-12	6 16 36	SS			1.5/1.5 100% recovery
68.0					RO			

PROJECT 83-1101-71 DATE DRILLED 2/22/83 HOLE NO. 9-3

DEPTH	CLASS.	FIELD DESCRIPTION	SAMPLE	SPT (6")	DRILL MODE	RUN NO.	CORE REC. %	REMARKS
68.0		38.5-85' CLAYEY SILTSTONE: cont.			RD			
			PB-2		PB			2.5/2.5 100% recovery
70.0		continued massive bedding						
72.0			J-13	6 / 15 / 36	SS			1.5/1.5 100% recovery
					RD			2/22/83 2/23/83 H_2O @ 31' in am
74.0								
76.0								
78.0			J-14	8 / 25 / 43	SS			1.1/1.5
					RD			
80.0								
82.0			PB-3		PB			1.7/2.5
84.0		Sample J-15; contains scattered shells	J-15	10 / 22 / 46	SS			1.3/1.5
86.0		B.H. 85.0' Terminated hole; grouted to ground surface						finish drilling 7:30 am. Converse sampler driven with 300 lb downhole hammer @ 18" drop; Pitcher samples cut with 200 psi.
88.0								

SHEET 4 OF 5

SUMMARY BORING NO. 9-3

PROJECT 83-1101-71 STATION HOLE yes DATE DRILLED 2/22-23/83

OVERBURDEN DEPTH (FT.) 6' TO 38.5'.

BEDROCK DEPTH (FT.) 38.5' TO 85' (T.D.).

WATER PRESS. TEST NO; INTERVAL(S) ____ TO ____, ____ TO ____.

GROUND WATER DEPTH (FT.) 31' DATE during drilling 2/23/83; ____ DATE ____.

GAS sulfur; DEPTH FIRST NOTICED 52', DATE 2/22/83.

E-LOG NO.

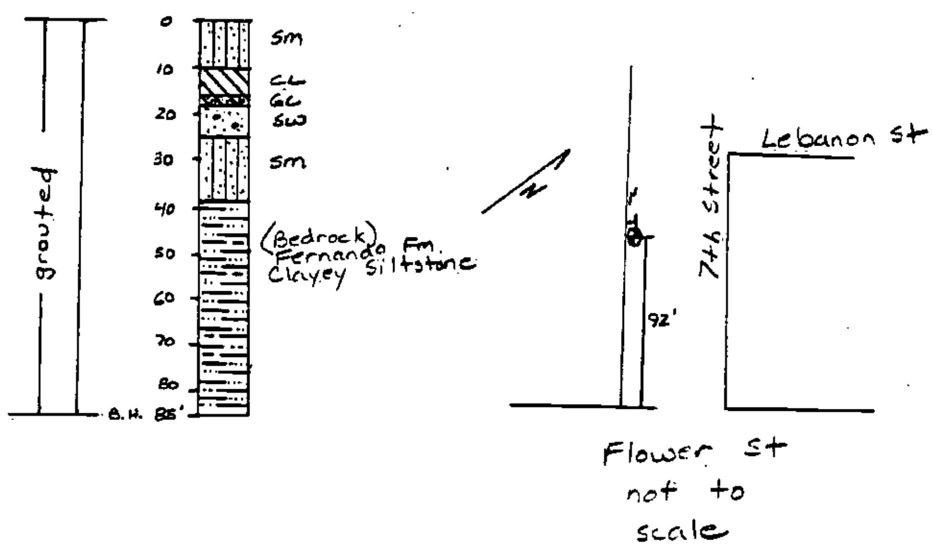
DOWN-HOLE SURVEY NO.

CROSS-HOLE SURVEY NO.

PVC CASING (I.D.): 4" ____ TO ____; 3" ____ TO ____; 2" ____ TO ____.

GROUND ELEVATION REF. RTD SITE PLAN 2/10/83 EL-A-1 (1"=40') Weese & Assoc.

SKETCH



SHEET 5 OF 5

Converse Consultants

Boring Log 9-4

THIS LOG IS APPLICABLE ONLY AT THIS LOCATION AND TIME. CONDITIONS MAY DIFFER AT OTHER LOCATIONS OR TIME.

PROJECT 83-1101-81 DATE DRILLED 2/17-18/83 HOLE NO. -9-4
LOCATION Seventh ~160' N. of Hope GROUND ELEV. 267.5'
DRILLING CONTRACTOR Pitcher LOGGED BY L. Schoeberlein DEPTH TO GROUND WATER ___
TYPE OF RIG Failing HOLE DIAMETER 4 7/8" HAMMER WEIGHT AND FALL 140 lb @ 30"
SURFACE CONDITIONS concrete gutter TOTAL DEPTH 85' NO. CORE BOXES 0

DEPTH	CLASS.	FIELD DESCRIPTION	SAMPLE	SPT (6")	DRILL MODE	RUN NO.	CORE REC. %	REMARKS
0.0	AC	0.0-0.8 concrete			GB			set up 8am start drilling 9am set 5" steel casing
	GW	0.8-2.5 SANDY GRAVEL: olive black (5Y 2/1); 45% gravel, rounded; 35% sand; 10% clay; wet; fill? YOUNG ALLUVIUM						
2.0	CL	2.5'-3.5 SILTY CLAY: olive black (5Y 2/1); 100% moderately plastic fines; wet.			AO			
4.0	SM	3.5-14.0' SILTY SAND: moderate brown (5YR 4/4); 60% very fine sand; 40% non plastic fines; dense; moist.	J-1	6 10 13	SS			1.2/1.5
6.0					RD			
8.0		grading to very fine sand, less dense						
10.0			C-1	4 4	OR			0.8/1.0 80% recovery pocket pen 1.75
					RD			
12.0			J-2	5 8 7	SS			1.2/1.5
					RD			
14.0	GP	14.0'-27.5' SANDY GRAVEL: moderate brown (5YR 4/4); 80% subangular to subround medium gravel, 20% subangular medium sand; very dense; wet; occasional zones w/ minimal binder; gravels consist of dominantly granitics						
16.0			J-3	62	SS			0.5/0.5 100% recovery
					RD			
18.0								
20.0				16 18	OR			0/1.0 0% recovery

PROJECT 83-1101-81 DATE DRILLED 2/17/83 HOLE NO. 9-4

DEPTH	CLASS.	FIELD DESCRIPTION	SAMPLE	SPT (6")	DRILL MODE	RUN NO.	CORE REC. %	REMARKS
20.0	GP	14.0-27.5' SANDY GRAVEL: cont. cobbles to ~4"			RD			mixed mud hole caving 11am-1pm repaired broken hydraulic hose
			J-4	53-4.3				93/0.3 100% recovery
22.0					RD			
24.0								
26.0					SS RD			bouncing no penetration
28.0	SW	27.5-33.5 GRAVELLY SAND: black brown & white; 90% fine to coarse sand, angular to subangular, 8% fine gravel subangular to sub round, 2% nonplastic fines, very dense; wet; granitic in origin						
			C-2	100-5"	DR			0.4/0.4
30.0					RD			
32.0			J-5	63	SS			0.5/0.5
					RD			
34.0	CL	33.5-35.0 SILTY CLAY: greyish brown (5YR 3/2);						
36.0	SP	35.0-41.2 GRAVELLY SAND: black, brown and white. 95% very fine sand 3% rounded medium gravel, granitic, 2% non plastic fines. very dense; moist to wet.						
			J-6	31 50-5"	SS			1.0/1.0
38.0					RD			
40.0			C-3	21 50	DR			1.0/1.0 100% recovery pocket pen 2.25
		41.2-57.2 SAND: pale green (5G ?/2); 99% fine to coarse angular sand; 1% fines, v. dense; moist.			RD			
42.0			J-7	38 50-5"	SS			0.8/0.9
					RD			
44.0								

SHEET 2 OF 5

PROJECT 83-1101-81 DATE DRILLED 2/17-18/83 HOLE NO. 9-4

DEPTH	CLASS.	FIELD DESCRIPTION	SAMPLE	SPT (6")	DRILL MODE	RUN NO.	CORE REC. %	REMARKS
44.0		41.2-57.2 SAND: cont.			RC			
46.0								
			S-8	38 / 50-1"	SS			0.8/0.8 100% recovery
48.0					RD			
50.0		silt content increasing grading to silty sand	C-4	61 / 50-4"	DR			0.8/0.8 100% recovery
					RD			pocket pen 4.25
52.0			S-9	35 / 50-4.5"	SS			0.9/0.9 100% recovery
					RD			
54.0								
		55' thin lense of brown silty sand						
56.0								
		57.2-65.0 SILTY SAND:	S-10	28 / 50	SS			1.0/1.0 100% recovery
58.0		pale green (5G 7/2); 55% very fine sand; 45% nonplastic fines, very dense; moist strong sulfur odor.			RD			
60.0			C-5	35 / 40	DR			1.0/1.0 100% recovery 2/17/83
					RD			2/18/83 clean hole in a.m.
62.0	T <5°	nearly horizontal bedding olive grey (5Y 3/2) sand lenses 1/8" thick	S-11	27 / 56	SS			1.0/1.0 100% recovery
					RC			
64.0		FERNANDO FORMATION						
		65-85 CLAYEY SILTSTONE:						
66.0		dark greenish grey (5GY 4/1); 100% moderately plastic fines; contains mica; massive bedding.						
68.0			S-12	10 / 36	SS			1.4/1.4 100% recovery

SHEET 3 OF 5

PROJECT 83-1101-81 DATE DRILLED 2/17/83 HOLE NO. 9-4

DEPTH	CLASS.	FIELD DESCRIPTION	SAMPLE	SPT (6")	DRILL MODE	RUN NO.	CORE REC. %	REMARKS
68.0		65-85 CLAYEY SILTSTONE: cont.		50-4.5	RD			
70.0		Physical Condition: little fractured to massive; soft to friable hardness; plastic to friable strength; fresh.	PB-1		PB			1.9/2.5
72.0		increase in clay content; contains minor shells and irregular sand blebs (small)	J-13	25 57	SS			1.0/1.0 100% recovery
74.0					RD			
76.0								
78.0			J-14	15 33 50-5"	SS RD			1.4/1.4 100% recovery
80.0								
82.0			PB-2		PB			1.9/2.5
84.0			J-15	15 25 50	SS			1.5/1.5 100% recovery
86.0		B.H. 85.0' Terminated hole set 2" ABS to B.H. Slots 45-85' seal 6' to surface						Converse sampler driven with 300 lb downhole hammer @ 18" drop; Pitcher samples cut with 200 p.s.i.

SHEET 4 OF 5

SUMMARY BORING NO. 9-4

PROJECT 83-1101-71 STATION HOLE yes DATE DRILLED 2/17-18/83.

OVERBURDEN DEPTH (FT.) 0 TO 41.2'.

BEDROCK DEPTH (FT.) 41.2' TO 85' (T.D.).

WATER PRESS. TEST NO; INTERVAL(S) ____ TO ____, ____ TO ____.

GROUND WATER DEPTH (FT.) 64' DATE 4/4/83; ____ DATE ____.

GAS SULFUR; DEPTH FIRST NOTICED 58, DATE 2/17/83.

E-LOG NO.

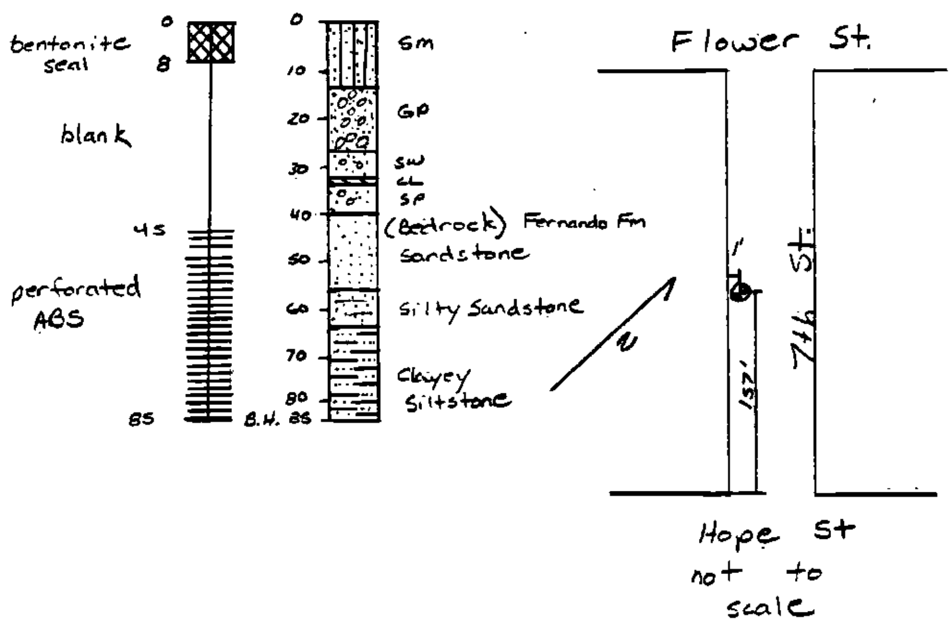
DOWN-HOLE SURVEY NO.

CROSS-HOLE SURVEY NO.

PVC CASING (I.D.): 4" ____ TO ____; 3" ____ TO ____; 2" 0 TO 85'.

GROUND ELEVATION REF. RTD SITE PLAN FL-A-1 (1"=40') Weese & Assoc. 2/10/83

SKETCH



1981 BORING LOGS

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Boring Log 9

THIS LOG IS APPLICABLE ONLY AT THIS LOCATION AND TIME. CONDITIONS MAY DIFFER AT OTHER LOCATIONS OR TIME.

PROJECT SCRTD DATE DRILLED 2-12-81/2-14/81 HOLE NO. 9
LOCATION 7th Street near Flower Street GROUND ELEV. 267'
DRILLING CONTRACTOR Pitcher Drilling Co. LOGGED BY Stephen M. Testa DEPTH TO GROUND WATER —
TYPE OF RIG Failing 1500 HOLE DIAMETER 5" HAMMER WEIGHT AND FALL 140lb - 30in
SURFACE CONDITIONS concrete slab sidewalk TOTAL DEPTH 200.0' NO. CORE BOXES 14

DEPTH	CLASS.	FIELD DESCRIPTION	SAMPLE	SPT (6")	DRILL MODE	RUN NO.	CORE REC. %	REMARKS
0.0	CL	0.0-0.2 CONCRETE; 0.2-9.0 SANDY CLAY: moderate brown (5YR 3/4); plastic fines (80%); very fine sand (20%); moist; slow dilatancy; medium toughness.			AD			Arrived on site at 5:00 AM to setup; started drilling at 7:30; clear day; augered down to 10.0'; 8.5' of casing installed.
2.0								
4.0								hit old clay pipe sewer conduit at 4.0'.
6.0								
8.0								
10.0	SM	9.0-14.0 SILTY SAND: moderate brown (5YR 3/4); non-plastic fines (15%); very fine sand (85%); moist; medium dense; slow to quick dilatancy; slight toughness.	J-1	5 / 6 / 9	SS / RD			SPT at 10.0'; .4/1.5 recovery. 9:30
12.0								
14.0	GP	14.0-29.0 SANDY GRAVEL: mottled; non-plastic fines (<5%); fine to coarse sand (35%); gravel (60%) up to ½" in max. dia.; moist; very dense; dilatancy none; toughness none.	J-2	32 / 50	SS / RD			minor reel chatter at 14.0'. SPT at 15.0'; .5/1.0 recovery.
16.0								
18.0								
20.0								

SHEET 1 OF 10

PROJECT SCRTD DATE DRILLED 2-1?-81 HOLE NO. 9

DEPTH	CLASS.	FIELD DESCRIPTION	SAMPLE	SPT (6")	DRILL MODE	RUN NO.	CORE REC. %	REMARKS	
20.0	GP	14.0-29.0 SANDY GRAVEL: (CONTINUED)		25	DR			(CONTINUED) conventional sample at 20.0', no recovery;	
				50					
		numerous cobbles and boulders.		50/4"	SS			SPT at 21.0', no recovery; moderate to heavy rod chatter to 22.5'; trouble getting back into hole due to cave in.	
22.0					RD				
24.0									
									10:40
26.0									
		clayey gravel (cuttings) moderate yellow brown (10YR 5/4)						moderate to heavy rod chatter at 27.0'; occasional rock from 27.0'.	
28.0									
	ML								
30.0		29.0-34.0 SANDY SILT: dark yellowish brown (10YR 4/2); silt (80%); very fine sand (20%); micaceous, mottled, iron stain stratification; moist; dense; dilatancy slow to quick; slight toughness.	J-3	7	SS			SPT at 30.0', 1.0/1.5 recovery.	12:00
				19					
				21					
32.0					RD				
34.0		color change at 34.0'							
		34.0-38.0 STRATIFIED SAND AND SILTY CLAY: 35.0-35.3 SAND: greenish gray (5G 6/1); non-plastic fines (<5%); fine to coarse sand (95%); moist; grades to light olive brown (10Y 4/2) SILTY CLAY; plastic fines (85%); non-plastic fines (15%); moist; dense; micaceous; grades to grayish olive (10Y 4/2) in color; at 38.0 greenish gray (5G 6/1) SAND similar to that at 36.0'.	J-4	10	SS			SPT at 35.0', 1.2/1.5 recovery.	12:4?
36.0				14					
				20					
					RD				
38.0	SP								
40.0				50	DR			conventional sample at 40.0', no recovery; SPT at 40.5', 1.0/1.5 recovery.	
		38.0-44.5 SAND: dusky yellow (5Y 6/4), non-plastic fines (<5%); fine to medium sand (95%); organics (<1%); stratified (iron stained); moist; very dense; dilatancy quick; toughness none.	J-5	18	SS				
				23					
42.0				50					
					RD				
44.0								SHEET 2 OF 10	

PROJECT SCRTD DATE DRILLED 2-12-81 HOLE NO. 9

DEPTH	CLASS.	FIELD DESCRIPTION	SAMPLE	SPT (6")	DRILL MODE	RUN NO.	CORE REC. %	REMARKS
44.0	SP	38.0-44.5 SAND: (CONTINUED)			RD			(CONTINUED)
	SP	44.5-50.0 GRAVELLY SAND: greenish gray (5G6/1); very dense.	J-6	16 / 25 / 50/4"	SS			SPT at 45.0'; 10/1.3 recovery. [1:20]
46.0		gravelly (fine)			RD			minor rod chatter from 47.0' to 50.0'.
48.0								
50.0		FERNANDO FORMATION 50.0-53.0 SILTSTONE: dusky yellow (5Y6/4), non-plastic fines (90%); very fine sand (10%); moist; very dense; mottled due to variable iron staining.	J-7	19 / 23 / 30	SS			SPT at 50.0'; 1.5/1.5 recovery [1:45]
52.0					RD			
		53.0-54.8 SANDY CLAYSTONE: dusky yellow (5Y6/4) mottled; moist; very dense; gravelly at 54.5.						
54.0			C-1	51 / 50	DR			modified sample at 54.0'; 1.0/1.0 recovery; minor rod chatter at 54.5; smooth drilling at 54.8.
		54.8-200.0 CLAYSTONE: olive gray (5Y3/2) micaceous claystone; subtle compositional banding reflecting slight silt-clay variance during deposition apparent; fossiliferous (bivalves, snails, etc) randomly distributed.			RD			
56.0								
58.0					PB	1		started continuous pitcher barrel sampling from 57.5' [2:00]
60.0	30°	continued. PHYSICAL CONDITION: massive; soft hardness; plastic strength; fresh; 2.5' of core tends to bend when held at both ends (doesn't fall apart)	Box #1				2.0/2.5	200 psi.
62.0		continued.				2	2.5/2.5	pocket penetrometer >4.5 kg./cm.²; gas check - 0.0% LEL.
64.0						3	2.3/2.5	200 psi.
66.0		continued.				4	2.5/2.5	
68.0			Box #2			5		

PROJECT: SCRID DATE DRILLED: 2-12-81 / 2-13-81 HOLE NO. 9

DEPTH	CLASS.	FIELD DESCRIPTION	SAMPLE	SPT (6")	DRILL MODE	RUN NO.	CORE REC. %	REMARKS
68.0		54.8-200.0 CLAYSTONE: (CONTINUED) olive gray (5Y 3/2) micaceous claystone; subtle compositional banding reflecting slight silt-clay variance during deposition apparent; fossiliferous (bivalves, snails, etc.) randomly distributed.	Box #2		PB	5	2.5/2.5	(CONTINUED) very monotonous unit.
70.0		PHYSICAL CONDITION: (CON'T) massive; soft to low hardness; plastic to weak strength; fresh.	S-1			6	2.5/2.5	200psi.
72.0								200psi.
74.0		continued.	Box #2			7	2.5/2.5	200psi.
76.0		continued; very well cemented fine sandstone from 78.0 to 78.9.				8	2.5/2.5	
78.0						9	0/	250psi. 250psi.
80.0		continued.	Box #3			10	1.3/2.5	Stopped drilling at a depth of 81.0' at 4:30PM. 4:30 6:30
82.0		yellowish gray (5Y7/2) silt blebs noted at 82.0'.				11	2.5/2.5	2-13-81 Resumed drilling at 6:30AM; clear day.
84.0			S-2			12	2.5/2.5	200psi.
86.0		continued.				13	2.5/2.5	200psi. pocket penetrometer >4.5 kg/cm².
88.0		continued.	Box #3					200psi.
90.0						14	2.5/2.5	200psi.
92.0			Box #4			15	2.5/2.5	

SHEET 4 OF 10

PROJECT SCRTD DATE DRILLED 2-13-81 HOLE NO. 9

DEPTH	CLASS.	FIELD DESCRIPTION	SAMPLE	SPT (6")	DRILL MODE	RUN NO.	CORE REC. %	REMARKS
92.0		54.8-200.0 CLAYSTONE: (CONTINUED) olive gray (5Y3/2) micaceous clay-stone; subtle compositional banding reflecting slight silt-clay variance during deposition apparent; fossiliferous (bivalves, snails, etc.) randomly distributed. PHYSICAL CONDITION: (CON'T) massive, soft to low hardness; plastic to weak strength; fresh.			AB	15	2.5/2.5	(CONTINUED) 200psi.
94.0			Box #4			16	2.5/2.5	200psi. 8:00 averaging 3runs/hour
96.0						17	2.5/2.5	200psi.
98.0			S-3			18	2.5/2.5	200psi.
100.0		continued.				19	2.5/2.5	200psi.
102.0		continued.						
104.0		continued.	Box #5			20	2.5/2.5	200psi. 9:50
106.0						21	2.5/2.5	200psi.
108.0		continued.				22	2.0/2.0	200psi.
110.0		continued.						
112.0			Box #6			23	2.0/2.5	200psi.
114.0			S-4			24	2.5/2.5	
116.0								

SHEET 5 OF 10

PROJECT: CCRTP DATE DRILLED: 2-13-81 HOLE NO.: 9

DEPTH	CLASS.	FIELD DESCRIPTION	SAMPLE	SPT (6")	DRILL MODE	RUN NO.	CORE REC. %	REMARKS
116.0		54.8-200.0 CLAYSTONE: (CONTINUED) olive gray (5Y3/2) micaceous claystone; subtle compositional banding reflecting silt-clay variance during deposition apparent; fossiliferous (bivalves; snails, etc.) randomly distributed.			NB	25	2.5/2.5	(CONTINUED)
118.0		PHYSICAL CONDITION: (con'd) massive; soft to low hardness; plastic to weak strength; fresh.	Box #6					200 psi. averaging 3 runs/hour.
120.0						26	2.5/2.5	200 psi. 10:45
122.0	10:15					27	2.5/2.5	
		continued.	Box #7					200 psi.
124.0						28	2.5/2.5	pocket penetrometer >4.5 kg/cm²,
		continued.						200 psi.
126.0						29	2.5/2.5	
128.0								200 psi 11:31
130.0			S-5			30	2.5/2.5	
		continued.						200 psi
132.0		wood fragments (<1%) evident at 132.5'.				31	2.5/2.5	
134.0								200 psi.
		wood fragments (<1%) at 135.7'.	Box #8			32	2.5/2.5	
136.0		continued						200 psi. 12.2
						33	2.5/2.5	
138.0								200 psi.
		continued.	Box #9			34	2.5/2.5	
140.0								

SHEET 6 OF 10

PROJECT SCRTD DATE DRILLED 2-13-81 / 2-14-81 HOLE NO. 9

DEPTH	CLASS.	FIELD DESCRIPTION	SAMPLE	SPT (6")	DRILL MODE	RUN NO.	CORE REC. %	REMARKS
140.0		54.8-200.0 CLAYSTONE: (CONTINUED) olive gray (5Y3/2) micaceous claystone; subtle compositional banding reflecting silt-clay variance during deposition apparent; fossiliferous (bivalves, snails, etc) randomly distributed.	Box #9		R2	34	2.5/2.5	(CONTINUED) 200psi
142.0						35	3.0/2.5	200psi
144.0		PHYSICAL CONDITION: (CON'T) massive; soft to low hardness; plastic to weak strength; fresh.	S-6			36	2.5/2.5	
146.0		continued						200psi pocket penetrometer 74.5 kg/cm².
			Box #9			37	2.5/2.5	
148.0	10:31	continued, wood fragments evident.						200psi
150.0		continued.				38	2.5/2.5	200psi
152.0						39	2.5/2.5	
		continued.	Box #10					200psi.
154.0						40	2.5/2.5	
156.0	11:20	grayish yellow (5Y8/4) silt blebs apparent; compositional banding.						200psi
						41	2.5/2.5	
158.0								200psi; gas check 0.0% LEL. (no gas encountered)
160.0			S-7			42	2.5/2.5	
		continued.						200psi
162.0		163.5-164.5 VOLCANIC TUFF: dark greenish gray (5GY4/1); very fine grained, wet.	Box #11			43	2.5/2.5	Stopped drilling at a depth of 163.5' at 4:30 AM 2-14-81 / Resumed drilling at 6:30; clean day.
164.0						44	2.5/	

1:30

2:30

4:20

6:30

SHEET 7 OF 10

PROJECT SCPTD DATE DRILLED 2-14-81 HOLE NO. 9

DEPTH	CLASS.	FIELD DESCRIPTION	SAMPLE	SPT (6")	DRILL MODE	RUN NO.	CORE REC. %	REMARKS	
164.0	A D D	54.8-200.0 CLAYSTONE: (CONTINUED) olive gray (5Y3/2) micaceous clay-stone; subtle compositional banding reflecting silt-clay variance during deposition apparent; fossiliferous (bivalves, snails, etc.) randomly distributed.			AB	44	2.5/2.5	(CONTINUED)	
166.0			Box #11					200 psi. averaging 3 runs/hour.	7:00
168.0		PHYSICAL CONDITION: (CON'T) massive; soft to low hardness; plastic to weak strength; fresh.				45	2.5/2.5	200 psi.	7:16
170.0						46	2.5/2.5		
172.0	10°	continued; grayish yellow (5Y8/4) siltstone lamina (≈? cm. thick) at 171.2 and 173.3'.	Box #12					200 psi.	7:38
174.0						47	2.5/2.5	200 psi.	7:55
176.0		continued.	C-8			48	2.5/2.5	200 psi	8:24
178.0		continued.	Box #12			49	2.5/2.5	200 psi.	
180.0		continued.				50	2.5/2.5	200 psi.	9:07
182.0		continued.	Box #13			51	1.3/2.5	200 psi.	
184.0						52	2.5/2.5		
186.0		continued.						200 psi	
188.0						53	2.5/2.8		

SHEET 8 OF 10

PROJECT SCRTD DATE DRILLED 2-14-81 HOLE NO. 9

DEPTH	CLASS.	FIELD DESCRIPTION	SAMPLE	SPT (6")	DRILL MODE	RUN NO.	CORE REC. %	REMARKS
188.0		54.8-200.0 CLAYSTONE: (continued as previously described) PHYSICAL CONDITION: (continued as previously described).	Box #13		PB	53	28/28	(CONTINUED) 200 psi. 10:10
190.0			S-9			54	28/28	
		continued.						
192.0	10-75°							200 psi.
						55	28/28	
194.0		continued.	Box #14					200 psi.
196.0						56	28/28	
		continued.						200 psi
198.0						57	28/28	Terminated hole at a depth of 200.0' at 11:50 AM. 11:50
200.0	BH							2-15-81 Conducted water pressure test at depth intervals 60.0' to 80.0' and 80.0' to 100.0' from 6:30 AM to 11:30 AM. Installed double piezometers: 200.0' of 2" PVC slotted from 160.0' to 195.0'; 60.0' of 1" PVC slotted from 45.0 to 55.0; bentonite seal from 62.0' to 65.0'; hole back-filled with pea gravel. water sampled 2/23/81

ConverseWardDavisDixon
Earth Sciences Associates
Geo/Resource Consultants

Summary Data
Boring No. 9

SUMMARY BORING NO. 9

PROJECT SCRTD STATION HOLE ______ DATE DRILLED 2-12-81/2-14-81.

OVERBURDEN DEPTH (FT.) 0.0' TO 54.8'.

BEDROCK DEPTH (FT.) 54.8' TO 200.0' (T.D.).

WATER PRESS. TEST YES; INTERVAL(S) 62.0' TO 82.0', 82.0' TO 102.0'.

GROUND WATER DEPTH (FT.) ______ DATE ______; ______ DATE ______.

GAS NC; DEPTH FIRST NOTICED —, DATE —.

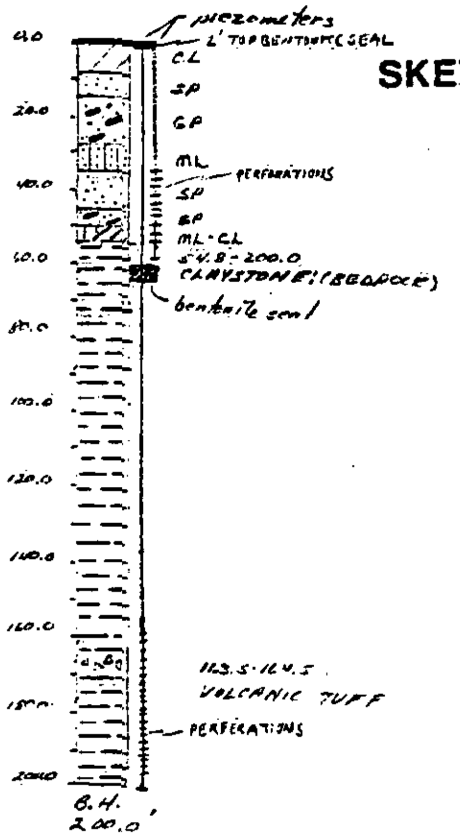
E-LOG YES.

DOWN-HOLE SURVEY YES.

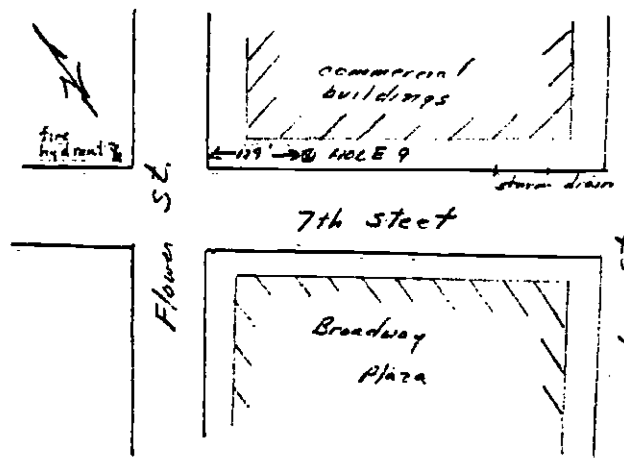
CROSS-HOLE SURVEY NO.

PVC CASING (I.D.): 4" ______ TO ______; ~~3"~~ 1" 0.0 TO 60.0'; 2" 0.0' TO 200.0'.

GROUND ELEVATION REF. 267'.



SKETCH



SHEET 10 OF 10

Appendix B

Downhole Geophysical Survey

APPENDIX B GEOPHYSICAL EXPLORATIONS

B.1 DOWNHOLE SURVEY

B.1.1 Summary

A downhole shear wave velocity survey was performed in Boring CEG-9 in Design Unit A165. Measurements were made at 5-foot intervals from the ground surface to a depth 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 each 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

A typical record from a similar downhole survey is 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. Actual travel times versus depth from Boring CEG-9 are shown in Figure B-2.

B.1.4 Discussion of Results

Estimated velocities are summarized in Table B-1. Velocity estimates are based on selection of linear portions of the downhole arrival time curves.

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^*). V_p^* are the values to be used for studies of the response of these sites. N is the number of data points used for the straight line fit for each velocity estimate.

In general, near-surface shear wave velocity was found to be about 1050 feet per second to depths of 45 feet. The shear velocity estimate increased to about 2500 feet per second for depths of 45 to 200 feet.

TABLE B-1
DOWNHOLE VELOCITIES

Boring No.	Depth (ft)	COMPRESSIONAL WAVE					SHEAR WAVE				
		$\bar{V}p$	σp	Ep	Np	Vp*	$\bar{V}s$	σ_s	Es	Ns	Vs*
9	10-45	2330	163	117	6	2330±280	1044	89	52	5	1040±140
	45-200	4829	188	241	29	4830±430	1534	85	77	31	1530±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

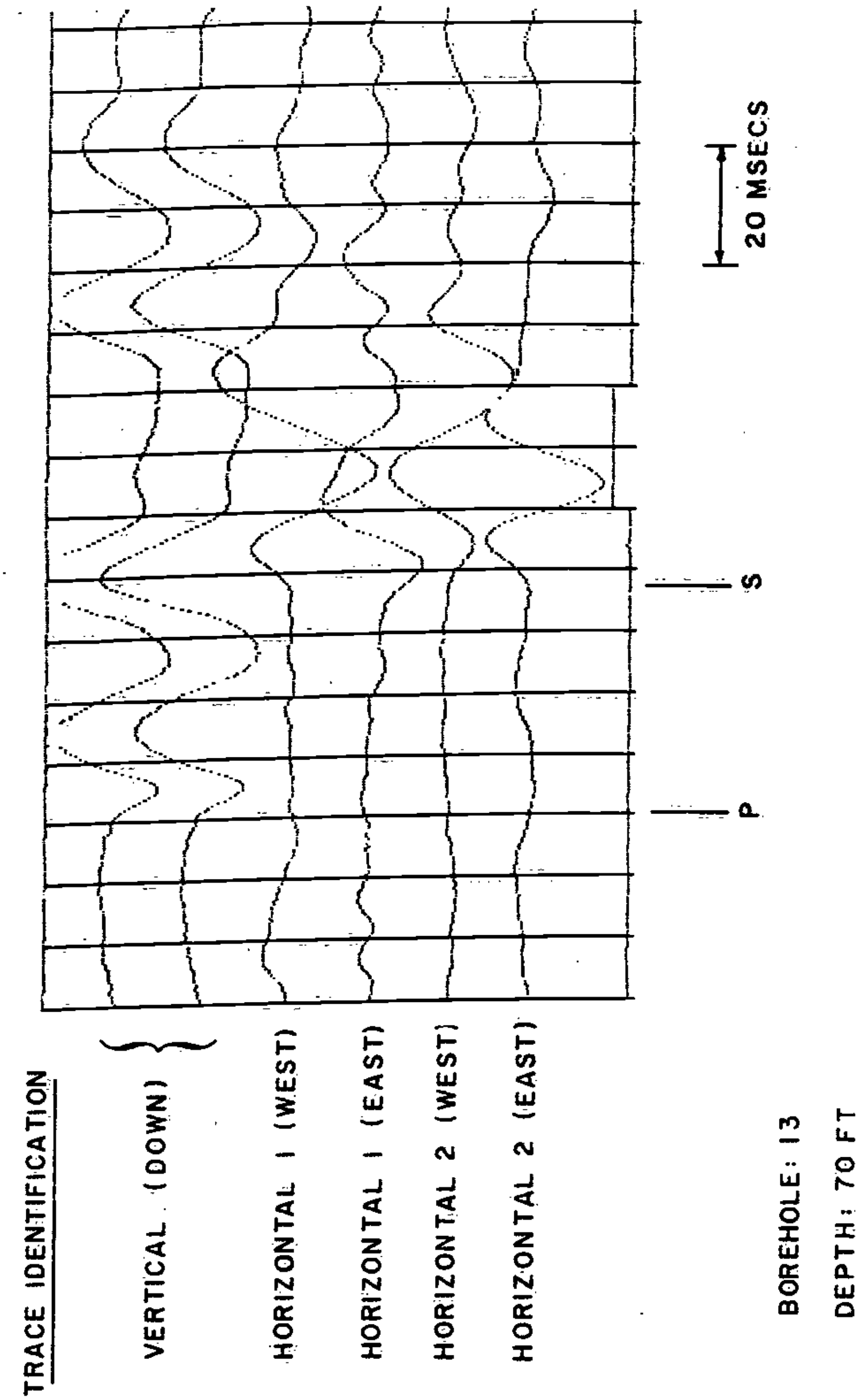
Np = number of points used for straight line fit of compressional wave

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

NOTE: Compression wave velocity of water (approximately 5000 fps) may mask the actual compression wave velocity of the soil structure below the water table.



Approved for publication ______ by ______

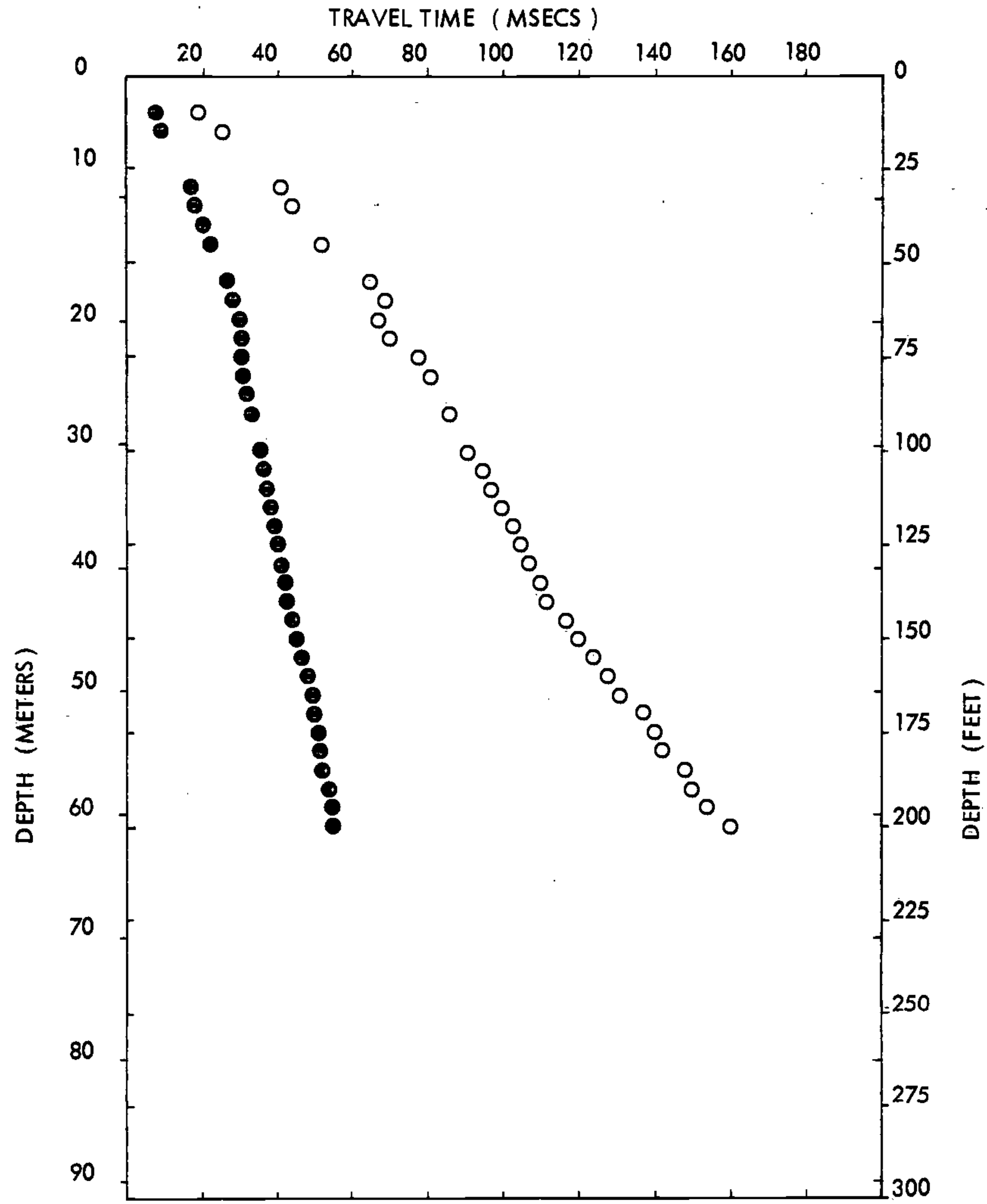
DOWNHOLE SAMPLE RECORD

DESIGN UNIT A 165
Southern California Rapid Transit District
METRO RAIL PROJECT

Project No. 83-1101

Figure No. B-1

Converse Consultants — Geotechnical Engineering and Applied Sciences



COMPRESSIONAL WAVE (Solid Circles)
SHEAR WAVE (Open Circles)

DOWNHOLE TRAVEL TIME PROFILE - BORING CEG 9

DESIGN UNIT A165
Southern California Rapid Transit District
METRO RAIL PROJECT

Project No.
83-1101

Converse Consultants Geotechnical Engineering and Applied Sciences

Figure No.
B - 2

Approved for publication ______ by ______

Appendix C

Water Quality Analysis

APPENDIX C WATER QUALITY ANALYSIS

C.1 RESULTS

Water samples were taken from Boring CEG-9. 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. Chemical constituents tested are attached.

C.2 FIELD PROGRAM

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

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Geo/Resource Consultants

Jacobs Laboratories

Water Quality

April 6, 1981

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

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

Attention: Buzz Spellman

Report of Chemical Analysis

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

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

Respectfully submitted,

William, R. Ray
Manager, Water Laboratory

asl

Converse Ward Davis Dixon

Lab No. P81-03-017-4

No. Samples : 7
Sampled By : Client
Brought By : Client
Date Received: 3-3-81

Sample labeled: HOLE 9-2"

Conductivity: 853 µ mhos/cm

Turbidity: NTU

pH 7.7 @ 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	32	1.60
Magnesium, Mg	7.5	0.62
Sodium, Na	127	5.52
Potassium, K	12	0.31
	Total	8.05
Anions determined:		
Bicarbonate, as HCO_3	202	3.31
Chloride, Cl	101	2.84
Sulfate, SO_4	82	1.71
Fluoride, F	0.7	0.04
Nitrate, as N	0.4	0.02
	Total	7.95
Carbon dioxide, CO_2, Calc.	6	
Hardness, as $CaCO_3$	111	
Silica, SiO_2	20	
Iron, Fe	< 0.01	
Manganese, Mn	< 0.01	
Boron, B	0.74	
Total Dissolved Minerals, (by addition: $HCO_3 \rightarrow CO_3$)	485	

Appendix D
Geotechnical Laboratory Testing

APPENDIX D GEOTECHNICAL LABORATORY TESTING

D.1 INTRODUCTION

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

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

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

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

D.1.1 Data Analysis

The summary of laboratory test results is presented in Table D-1. Figures D-1 through D-6 summarize strength and modulus data for granular alluvium and bedrock at this site and other nearby station sites.

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

$$m_s = a_1 \times a_2 \times \ldots \times a_n)^{1n}$$

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

D.2 INDEX AND IDENTIFICATION

D.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.

D.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 Figure D-7.

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.

D.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 Figure D-8 and Tables D-2 and D-3.

D.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 D-2 and D-3.

D.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 D.2.4 and the total unit weight. Results of the unit weight tests are presented as dry densities on Tables D-2 and D-3.

D.3 ENGINEERING PROPERTIES: STATIC

D.3.1 Unconfined Compression

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

D.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:

D.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. This process was repeated a third time at a still higher confining pressure, and the sample was loaded until failure occurred.

Results of the triaxial compression tests are presented in Figures D-9 through D-11.

D.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 D-2 and D-3.

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

D.3.4 Swell

No swell tests were performed for this design unit.

D.3.5 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 are 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 D-12 through D-16.

D.3.6 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 D-2 and D-3.

D.3.7 Porosity

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

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

w = unit weight of water
d = unit dry weight of water
G = specific gravity of soil solids.

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

D.4 ENGINEERING PROPERTIES: DYNAMIC

D.4.1 Resonant Column

The resonant column test evaluates the shear modulus and damping of soil specimens at shear strains of approximately 10^{-6} to 10^{-4} inches per inch. A solid cylindrical soil specimen is encased in a thin membrane, placed in a pressure cell and subjected to the desired ambient stress conditions. The specimen is caused to vibrate at resonance in torsion by fixing one end and applying sinusoidally varying torque to the free end. The response of the soil specimen is measured using an accelerometer coupled to the free end. Shear modulus and damping values are calculated from the response data.

D.4.1.1 Sample Preparation and Handling

The test apparatus used for this procedure accepts a 1.4-inch diameter by approximately 3.5-inch length specimen. Undisturbed samples were prepared by trimming the 1.4-inch diameter samples from the larger Shelby, Pitcher or Converse ring samples.

D.4.1.2 Test Conditions and Parameters

The resonant column test is considered non-destructive because the shear strain amplitudes are relatively small. Therefore, a single specimen may be used for several tests. For this test program, several of the specimens were tested at confining pressures, ($\sigma 3c$), varying from 15 to 50 psi. Although the apparatus is capable of applying anisotropic consolidation stresses, specimens for this program were consolidated isotropically. The specimens were tested beginning at the lower confining pressures and progressing to the higher confining pressures. At each confining pressure, shear modulus and damping data were obtained at several different values of shear strain within the limiting range of the test apparatus. Damping data were obtained for steady state vibration conditions. A summary of pertinent resonant column test data is presented on Figures D-17 through D-19.

D.4.1.3 Apparatus

The device used in this test program was designed and built by Soil Dynamics Instruments, Inc., of Lexington, Kentucky, and is sometimes referred to as a Hardin Oscillator, after Dr. B.O. Hardin, the designer. Essentially, it consists of the main component groups listed below.

- Pressure Cell and Frame: The unit is aluminum with a transparent plexiglass cylinder designed for maximum operating pressures of approximately 150 psi. The bottom specimen end cap is brass and affixed to the base of the unit.

  Pressure lines and fittings are provided to pressurize the cell and for back pressure or sample drainage, if desired. A pneumatic device is also provided to support the weight of the excitation device during specimen setup.

- Excitation Device: This mechanism consists of a torque-producing apparatus mounted on the underside of a hollow stainless steel cylinder. Its mass is very large in comparison to the test specimen. The driving torque is produced by a system of electromagnetic coils attached to the cylinder and permanent magnets coupled to the top specimen load cap through a system of restoring springs. The device is driven by an audiooscillator having a frequency range of approximately 20 Hz to 40 kHz. Because the device is designed to have a large mass in comparison to the specimen, a lever and weight system supports the weight of the device during the test. A strain gauge load cell is built into the excitation device to monitor the axial load applied to the specimen. In operation, the device applies a sinusoidal torque to the specimen. The driving torque is determined by measuring the voltage drop across a precision resistor in series with the electromagnetic coils.

- Accelerometer and Charge Amplifier: A Columbia Research Labs accelerometer is attached to the excitation device. The accelerometer output is amplified by a charge amplifier, and the system is calibrated to produce output voltage in proportion to the amplitude of angular displacement of the excitation device, and thus of the specimen. Shear strains are calculated from the amplitude of angular displacement.

- Readout Devices: Output voltages produced by the accelerometer, load cell-bridge system, and driving torque are read by a digital multimeter. Resonance of the specimen is determined using a cathode ray oscilloscope connected to display the Lissajous pattern.

D.4.1.4 Data Reduction

Data obtained from the resonant column tests were reduced in accordance with the ASTM "suggested Methods of Test for Shear Modulus and Damping of Soils by the Resonant Column"* using a proprietary computer program developed by Converse Consultants.

*ASTM Special Technical Publication 479.

TABLE D-1 SUMMARY OF LABORATORY TEST RESULTS

	TOTAL DENSITY (pcf)	DRY DENSITY (pcf)	MOISTURE CONTENT (%)	UNCONFINED COMPRESSIVE STRENGTH (ksf)	DIRECT SHEAR TEST PEAK SHEAR STRENGTH ø (deg)	DIRECT SHEAR TEST PEAK SHEAR STRENGTH c (ksf)	TRIAXIAL TEST PEAK UNDRAINED STRENGTH ø (deg)	TRIAXIAL TEST PEAK UNDRAINED STRENGTH c (ksf)	TRIAXIAL TEST PEAK EFFECTIVE STRENGTH ø (deg)	TRIAXIAL TEST PEAK EFFECTIVE STRENGTH c (ksf)	YOUNG MODULUS E/σ_{3c} (ksf/ksf)	PERMEABILITY (cm/sec)
COARSE-GRAINED ALLUVIUM												
NUMBER OF TESTS:	3	3	3	0	6 DS		0		2 TX		1 C + 2 TX	0
HIGH	132	117	32	-	35	.6	-	-	-	-	394	-
LOW	119	90	13	-	31	.4	-	-	-	-	160	-
MEAN	125	105	29	-	-	-	-	-	35	.2	255	-
STANDARD DEVIATION	±6.7	±14	±10	-	-	-	-	-	-	-	-	-
FINE-GRAINED ALLUVIUM												
NUMBER OF TESTS:	3	3	3	0	9 DS		0		0		0	0
HIGH	131	113	20	-	30	.7	-	-	-	-	-	-
LOW	128	106	17	-	27	.3	-	-	-	-	-	-
MEAN	129	109	19	-	28	.5	-	-	-	-	-	-
STANDARD DEVIATION	±1.9	±3.4	±1.9	-	-	-	-	-	-	-	-	-
CLAYSTONE/SILTSTONE												
NUMBER OF TESTS:	11	11	11	13	6 DS		4 TX		5 TX		1 C + 5 TX	2
HIGH	139	123	29	15.4	29	1.4	26	3.3	35	1.5	290	9.7 E-7
LOW	119	93	13	2.6	29	1.2	26	1.4	35	1.0	55	5.1 E-8
MEAN	123	99.6	24.6	10.2	-	-	-	-	35	1.2	195	-
STANDARD DEVIATION	±2.7	±7.5	±4.2	±4.4	-	-	-	-	-	-	-	-

TABLE D-2 LABORATORY TEST DATA

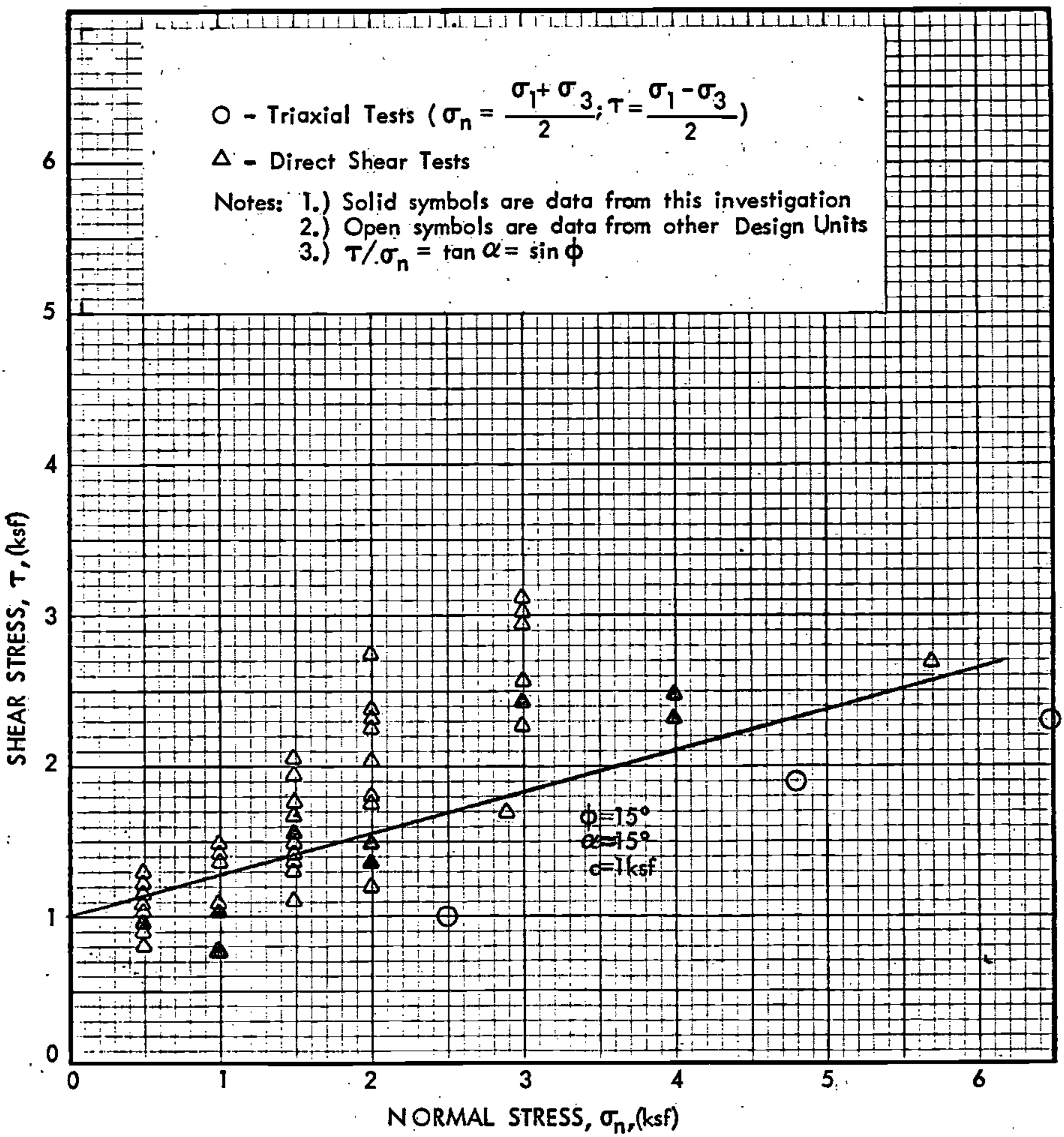
BORING No.	SAMPLE No.	DEPTH (ft)	VISUAL CLASSIFICATION	GEOLOGIC UNIT	DRY DENSITY (pcf)	MOISTURE CONTENT (%)	ATTERBERG LIMITS LL	ATTERBERG LIMITS PI	Kv, COEFFICIENT OF PERMEABILITY (cm/sec) (Confining Pressure, psi)	UNCONFINED COMPRESSIVE STRENGTH (ksf)	DIRECT SHEAR STRENGTH ENVELOPE ϕ, deg	DIRECT SHEAR STRENGTH ENVELOPE c, ksf	ONE-DIMENSIONAL SWELL (%) (Normal Load, ksf)	SWELL PRESSURE (ksf)	SIEVE ANALYSIS	HYDROMETER ANALYSIS	OEDOMETER	TRIAXIAL COMPRESSION
9-1	C-1	10	Silty Clay	A_2	107	20	29	11			24	.65						
	C-2	29	Silty Fine to Coarse Sand	A_3	108	15					36	.62						
	J-4	31	Silty Fine to Coarse Sand	A_3											X			
	PB-1	41	Clayey Siltstone	C	93	28				10.1								
	C-3	50	Clayey Siltstone	C	101	25							0.0 (2.0)				X	
	PB-2	61	Clayey Siltstone	C	97	26	43	11		14.3	--	--						X
	C-4	70	Clayey Siltstone	C	104	19											X	
9-2	C-1	10	Clayey Silt	A_2	113	17					30	.70						
	C-2	53	Clayey Siltstone	C	96	27					31	1.15	0.0 (2.0)				X	
	C-3	70	Clayey Siltstone	C	101	25				11.7							X	
9-3	C-1	10	Silty Fine Sand	A_1	106	20					33	.39						
	C-2	30	Fine to Medium Sand	A_3	117	13			2.3E-6(15)						X			X(2)
	C-3	40	Clayey Siltstone	C	91	32					27	1.35					X	
	PB-1	50	Clayey Siltstone	C	93	27				13.6								
	C-4	60	Clayey Siltstone	C	99	27				10.8							X	
	PB-2	61	Clayey Siltstone	C	97	26												X(2)

TABLE D-2 LABORATORY TEST DATA

BORING No.	SAMPLE No.	DEPTH (ft)	VISUAL CLASSIFICATION	GEOLOGIC UNIT	DRY DENSITY (pcf)	MOISTURE CONTENT (%)	ATTERBERG LIMITS LL	ATTERBERG LIMITS PI	Kv, COEFFICIENT OF PERMEABILITY (cm/sec) (Confining Pressure, psi)	UNCONFINED COMPRESSIVE STRENGTH (ksf)	DIRECT SHEAR STRENGTH ENVELOPE ø, deg	DIRECT SHEAR STRENGTH ENVELOPE c, ksf	ONE-DIMENSIONAL SWELL (%) (Normal Load, ksf)	SWELL PRESSURE (ksf)	SIEVE ANALYSIS	HYDROMETER ANALYSIS	OEDOMETER	TRIAXIAL COMPRESSION
9-4	C-1	10	Silty Sand	A_1														
	C-2	30	Gravelly Sand	A_1	--	--			--						--			
	C-3	41	Sandy Siltstone	C	90	32					30	.20						
	C-4	51	Sandy Siltstone	C	123	13				2.61							X	
	C-5	61	Sandy Siltstone	C	99	23				2.88							X	

TABLE D-3 List of Soils Engineering Properties from 1981 Laboratory Tests

CEG Boring No.	Sample No.	Depth (ft)	Visual Classification	Geologic Unit	Dry Density (pcf)	Moisture Content (%)	Atterberg Limits 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 Cc (in/in per Log Cycle)	Specific Gravity	Porosity (n)	Undrained Quick Direct Shear φ, deg	c, ksf	One-Dimensional Soil Swell % Swell (ksf Normal Load)	Cyclic Triaxial (Liquefaction)	Dynamic Triaxial (Stress/Strain)	Resonant Column Test	Triaxial Compression Test
9	C1	55	Silty claystone	C	95	27																	
	S1	71	Siltstone, massive	C	95	29						61.5											
	S3	99	Silty claystone	C	97	27	44	14	100	100	100		5.1E-8			41.4							cu
	S3	100	Silty claystone	C	98	26																	cu
	S3	100	Silty claystone	C	100	25																	cu
	S3	101	Silty claystone	C	98	27																X	
	S4	114	Sandy siltstone	C	99	26						69.1										X	
	S4	115	Sandy siltstone	C	99	25						87.9										X	
	S4	115	Sandy siltstone	C	97	26							9.7E-7			30.3							
	S6	145	Siltstone, massive	C	100	23						107											
	S7	165	Siltstone, massive	C	100	23						37.6											
	S8	175	Sandy siltstone, massive	C	97	25						97.5											
	S9	190	Siltstone, massive	C	99	25																	



Approved for publication Kom by 10/11/83

SUMMARY OF TOTAL STRENGTH DATA - FINE GRAINED ALLUVIUM

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Project No. 83-1101

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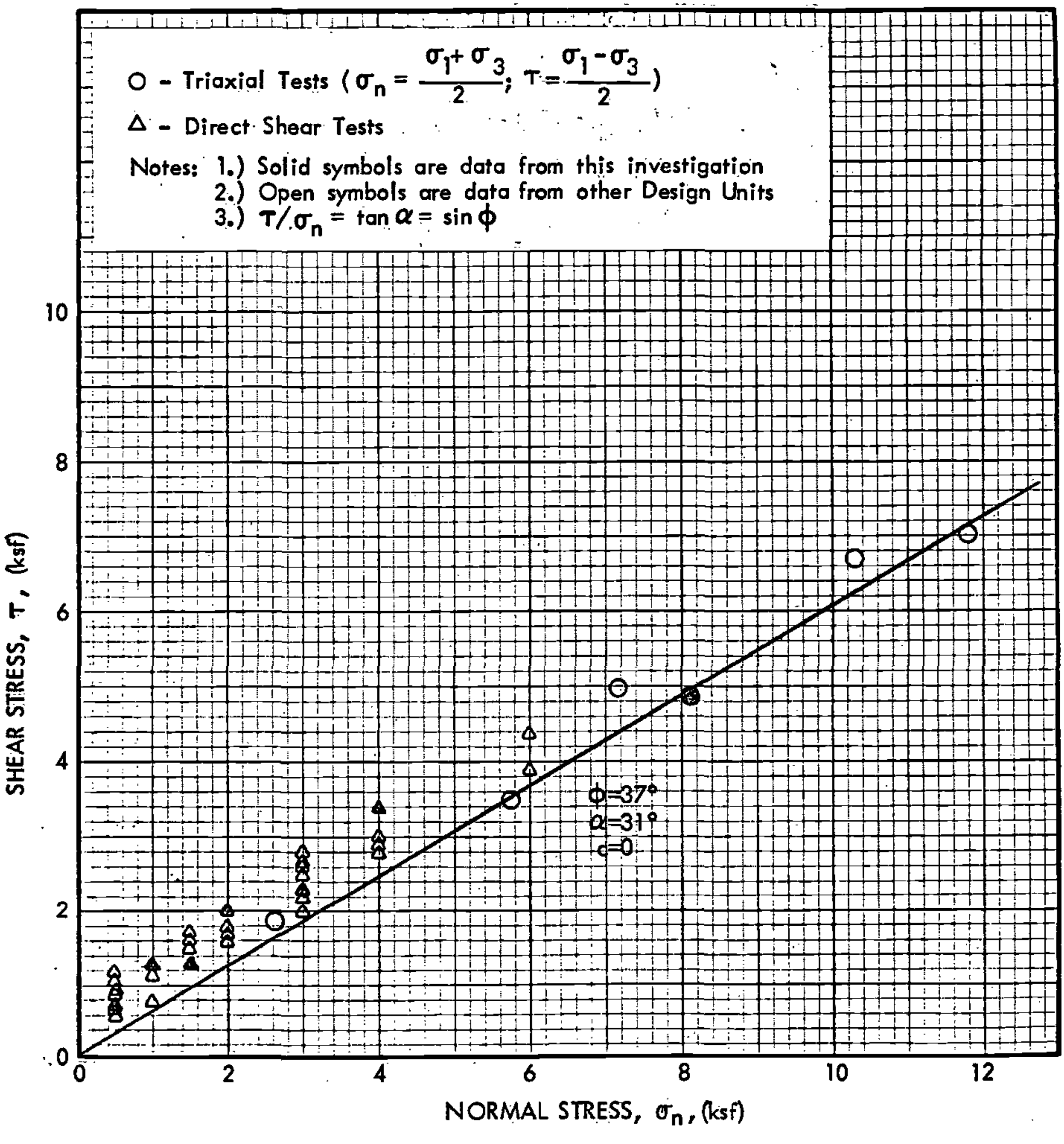
Figure No. D-1



Approved for publication KDM by 10/11/83

SUMMARY OF EFFECTIVE STRENGTH DATA - GRANULAR ALLUVIUM

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METRO RAIL PROJECT

Project No. 83-1101

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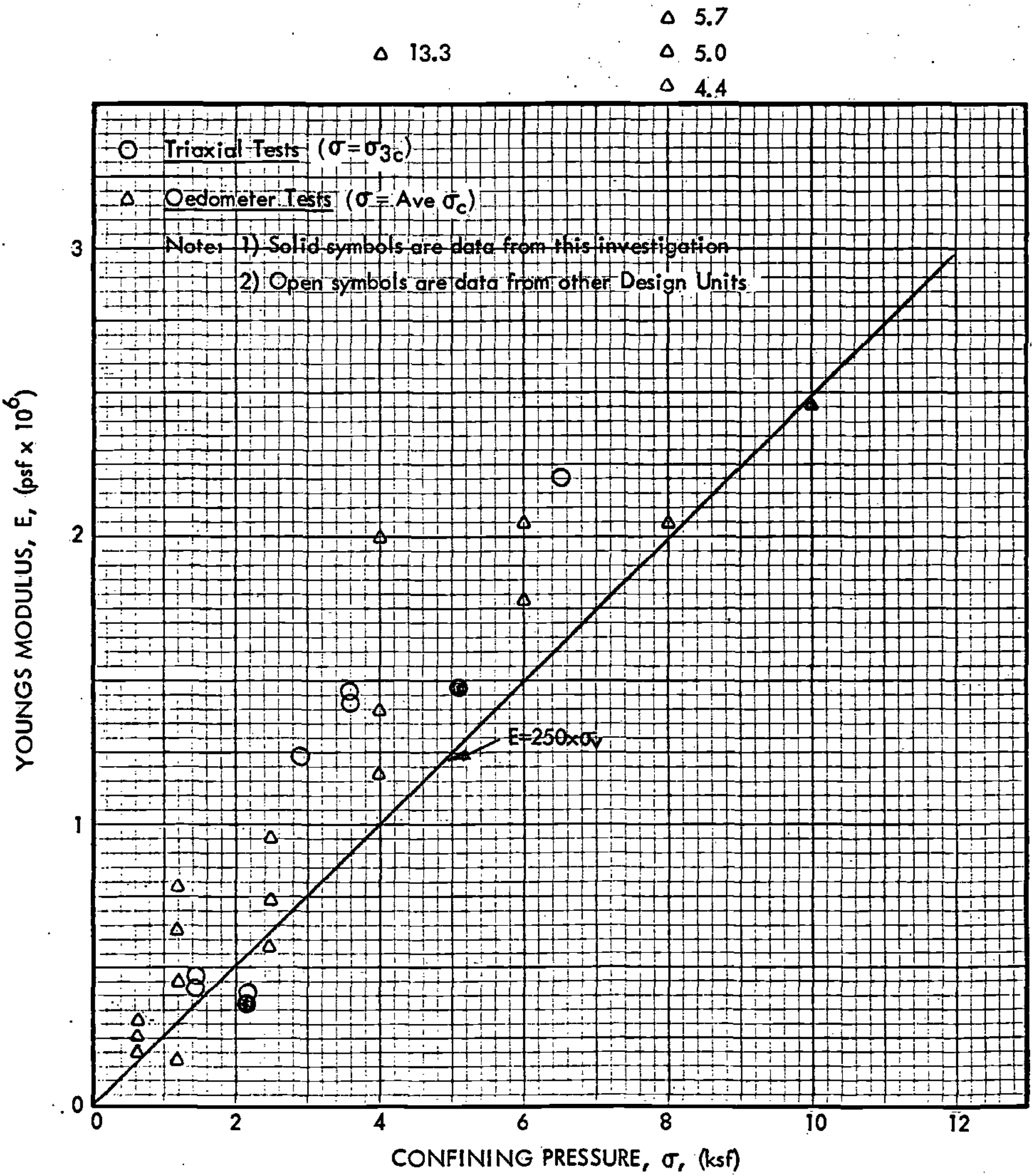
Figure No. D-2



Approved for publication KDM by 10/11/93

SUMMARY OF MODULUS DATA - GRANULAR ALLUVIUM

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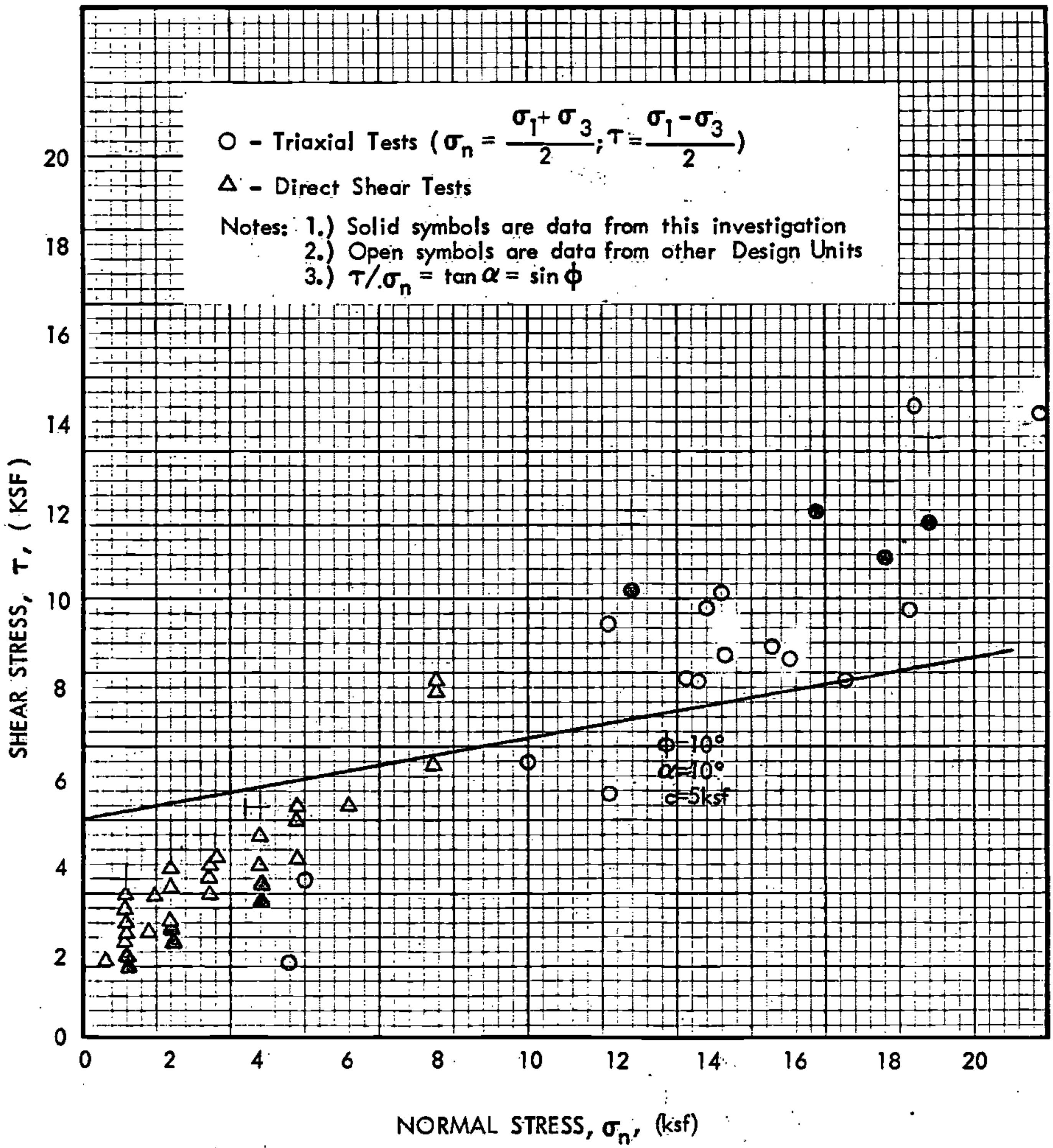
Figure No. D-3



SUMMARY OF TOTAL STRENGTH DATA - BEDROCK

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Figure No. D-4

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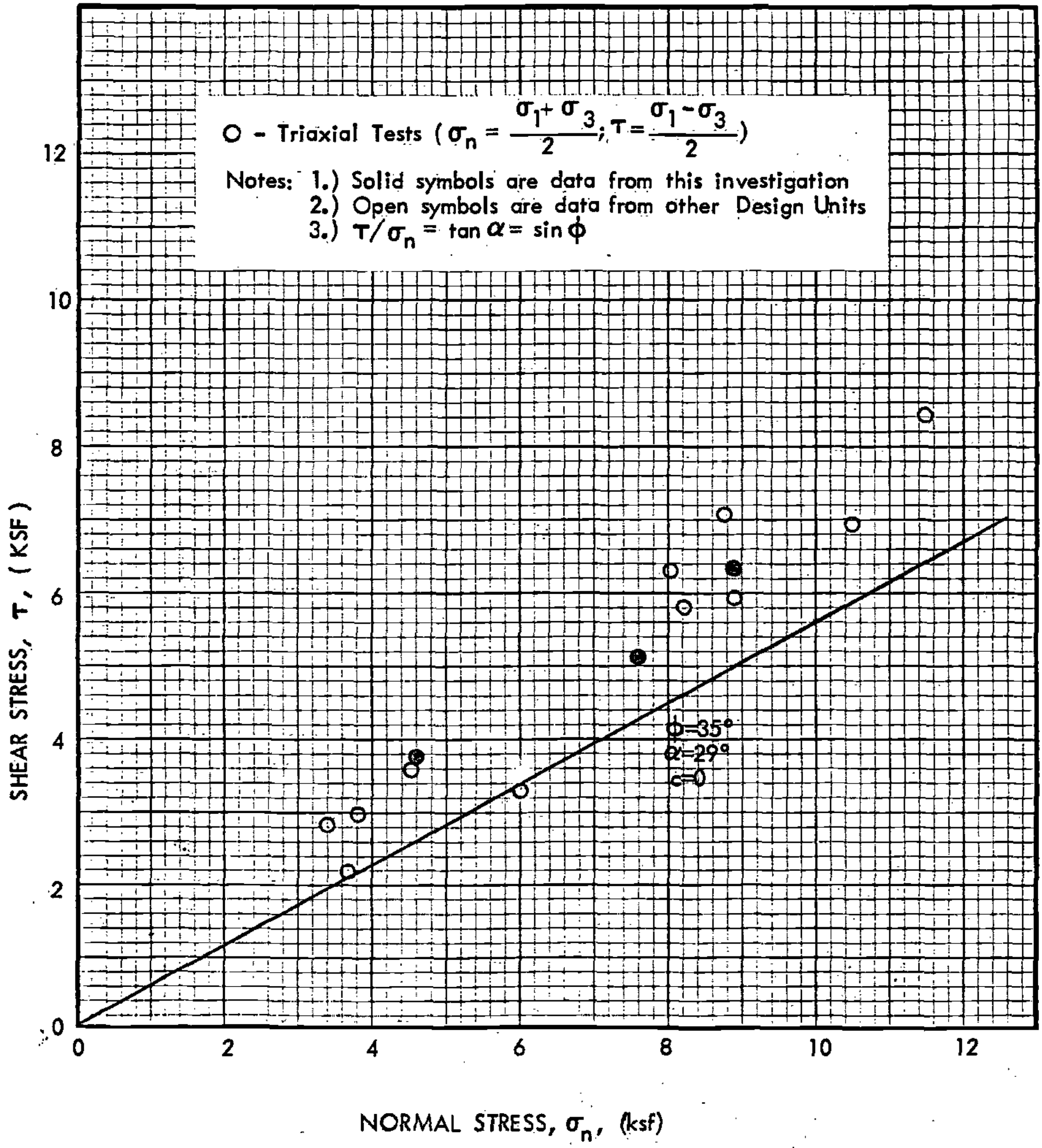

SUMMARY OF EFFECTIVE STRENGTH DATA - BEDROCK

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Figure No. D-5

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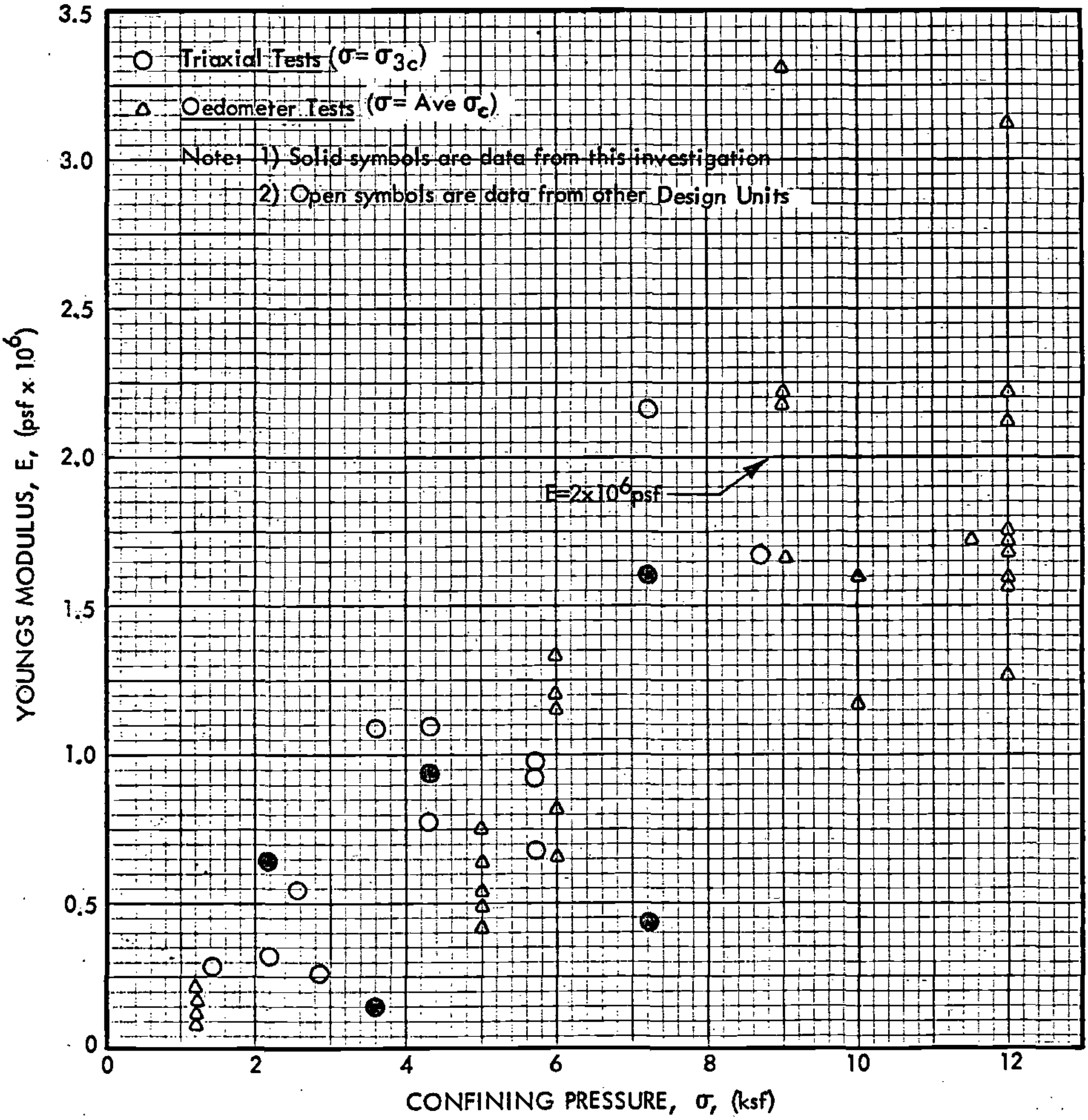

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SUMMARY OF MODULUS DATA - BEDROCK

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Figure No. D-6

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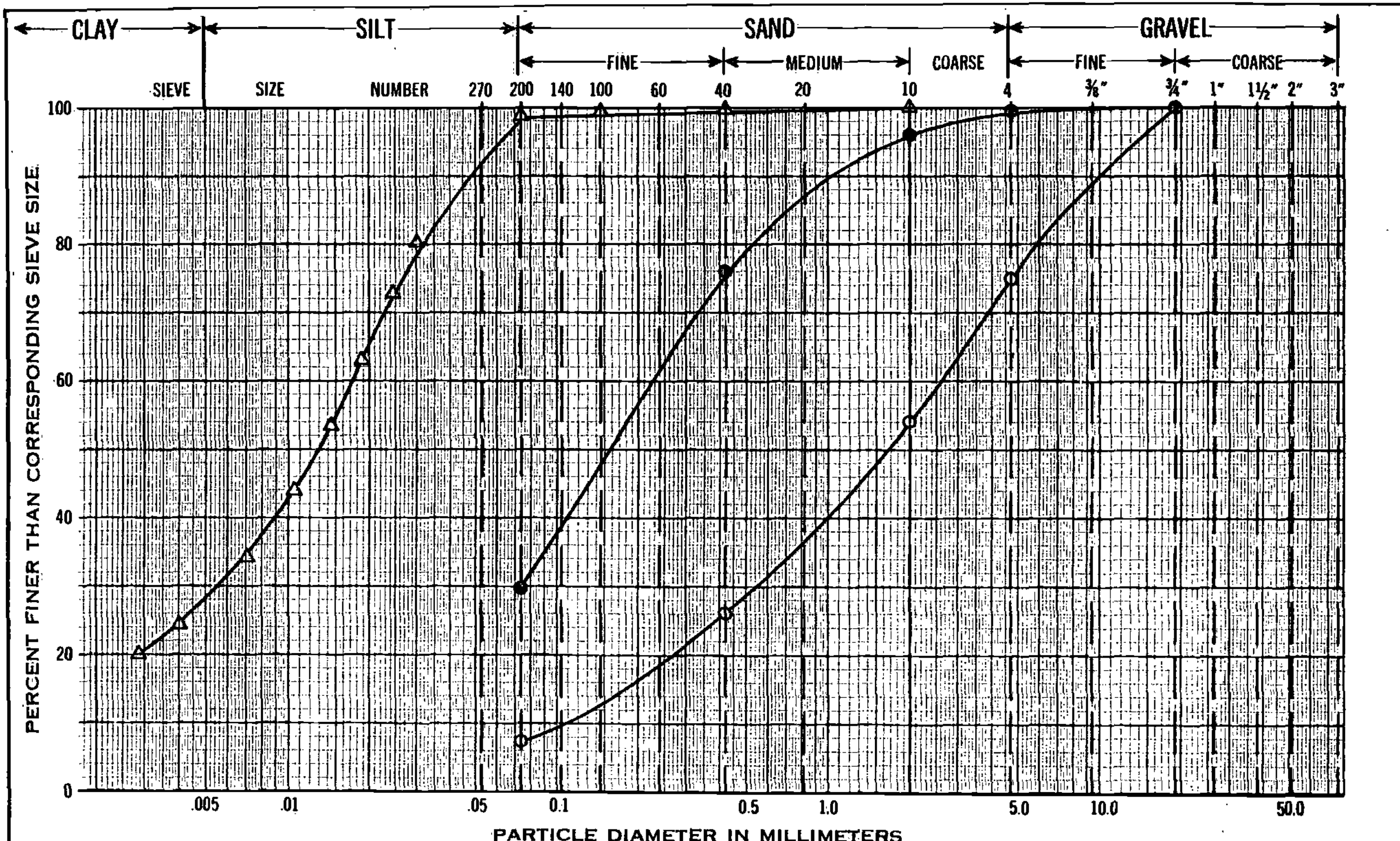

Symbol	Boring	Sample	Depth
○	9-1	J4	31'0" to 31'-6"
●	9-3	C2	29'-3" to 29'-8"
△	9	S-3	99'

GRAIN-SIZE DISTRIBUTION CHART

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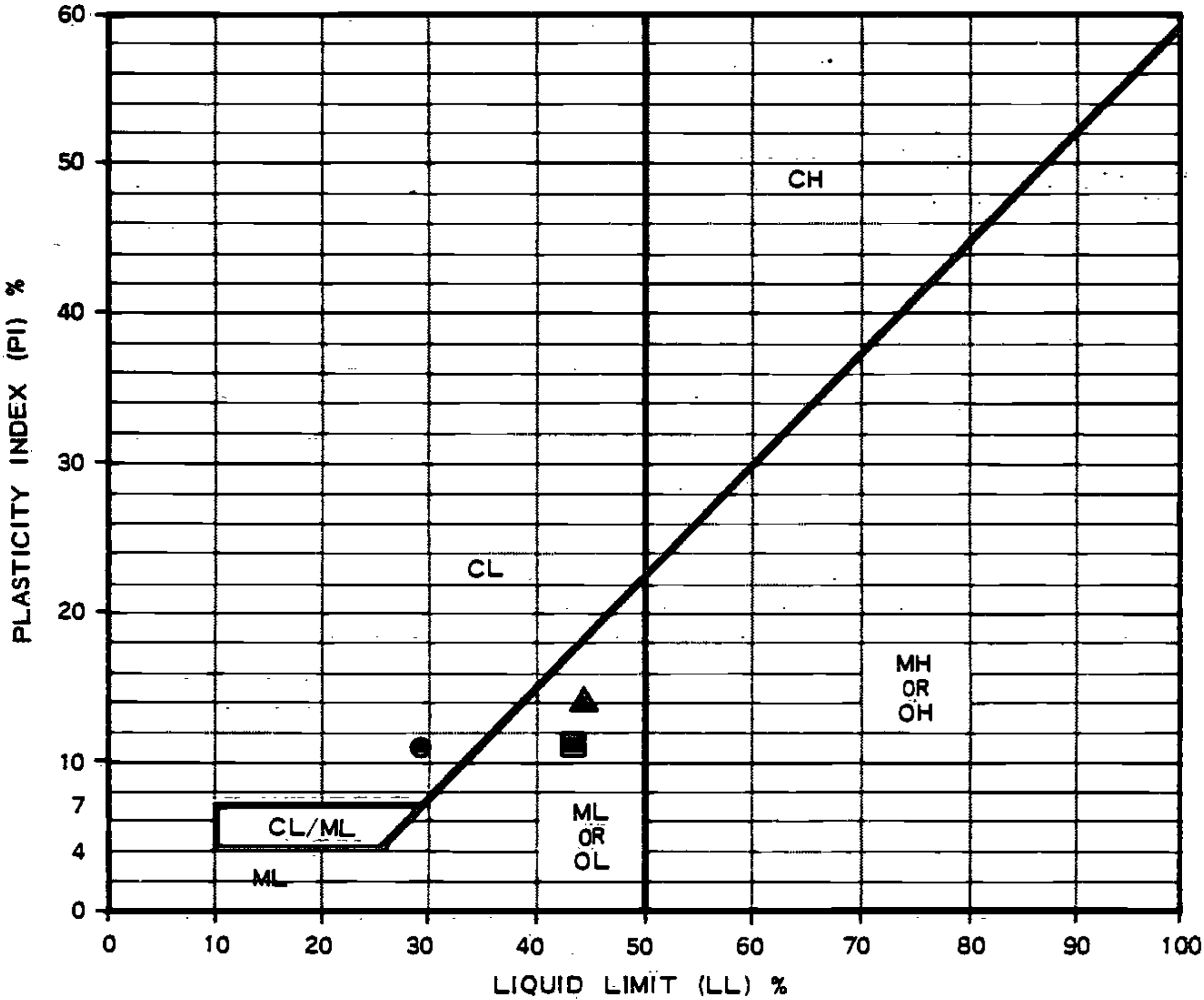
Figure No. D-7



Symbol	Classification and Source	Liquid Limit (%)	Plastic Limit (%)	Plasticity Index (%)	% Passing No. 200 Seive
●	9-1, 10' (CL)	29	18	11	
■	9-1, 61' (ML)	43	32	11	
▲	9, 99' (ML)	44	30	14	

PLASTICITY CHART

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Figure No. D-8

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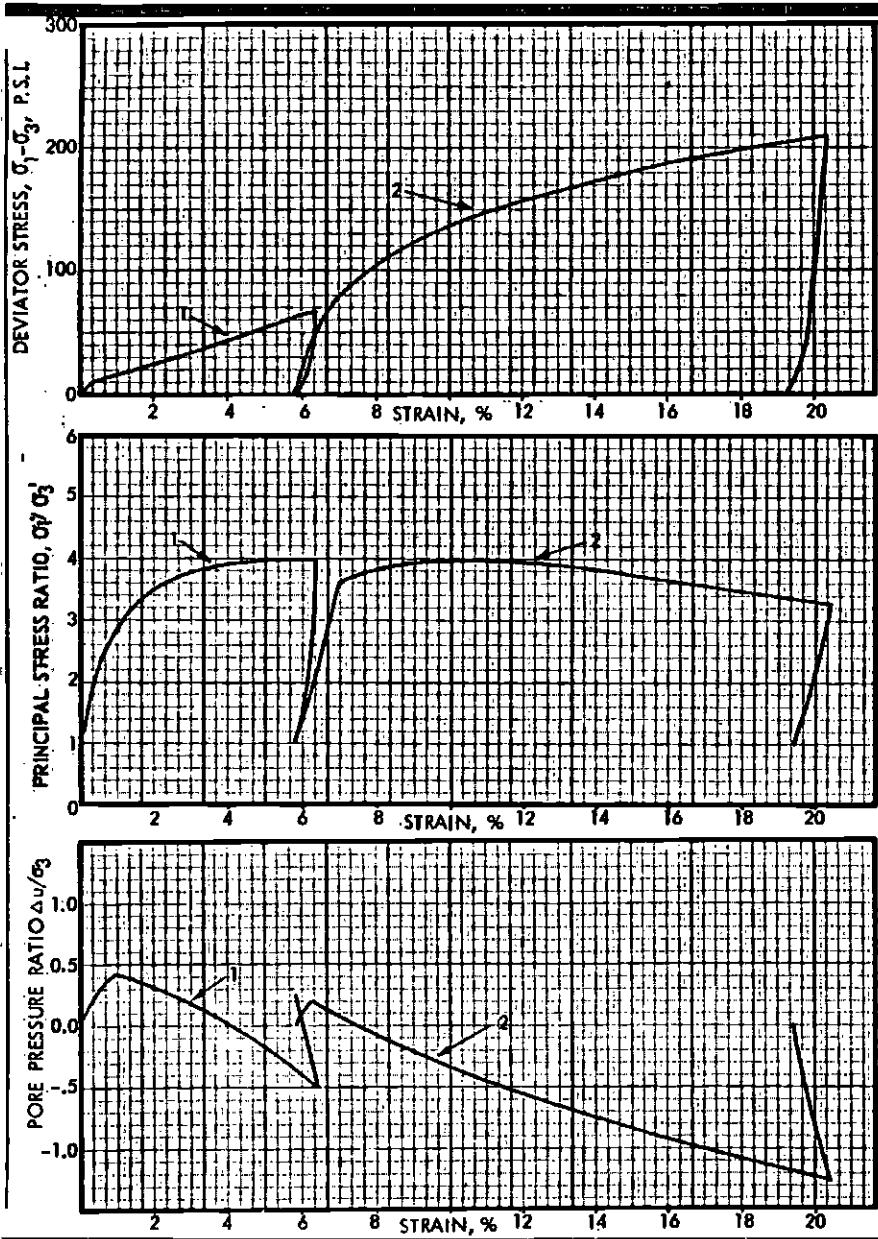

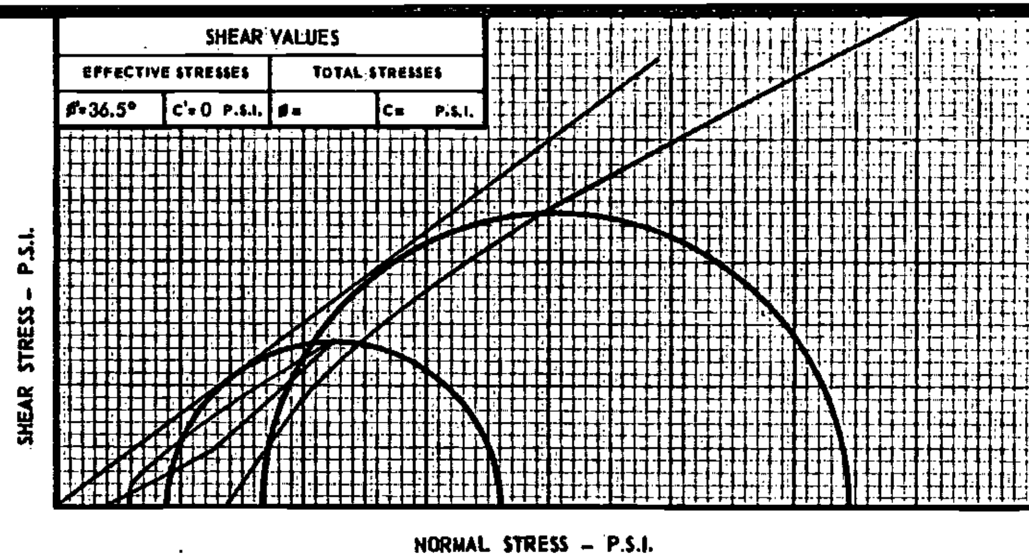

SHEAR VALUES			
EFFECTIVE STRESSES		TOTAL STRESSES	
ϕ'=36.5°	c'=0 P.S.I.	ϕ=	C= P.S.I.

SPECIMEN NUMBER	SPECIMEN LOCATION		INITIAL SPECIMEN DATA					TYPE OF SAMPLE
	BORING NUMBER	DEPTH IN FEET	SOIL CLASSI-FICATION	LENGTH IN INCHES	DIAMETER IN INCHES	DRY DENSITY (P.C.F.)	MOISTURE CONTENT IN PERCENT	
C-2	9-3	29	SM	5.00	2.42	113	17	UNDISTURBED

SYMBOLS	SPECIMEN NUMBER	APPLIED LATERAL PRESSURE σ_3 (P.S.I.)	MAXIMUM DEVIATOR STRESS $\sigma_1-\sigma_3$ (P.S.I.)	TEST VALUES AT FAILURE – MAXIMUM σ_1'/σ_3'		BACK PRESSURE (P.S.I.)	TYPE OF TEST
				EFFECTIVE LATERAL PRESSURE σ_3' (P.S.I.)	MAJOR EFFECTIVE STRESS σ_1' (P.S.I.)		
1	C-2	15	--				CU PROGRESSIVE
2	C-2	35	208	41.9	164.5		CU PROGRESSIVE

TRIAXIAL COMPRESSION TEST

DESIGN UNIT A165
Southern California Rapid Transit District
METRO RAIL PROJECT

Converse Consultants
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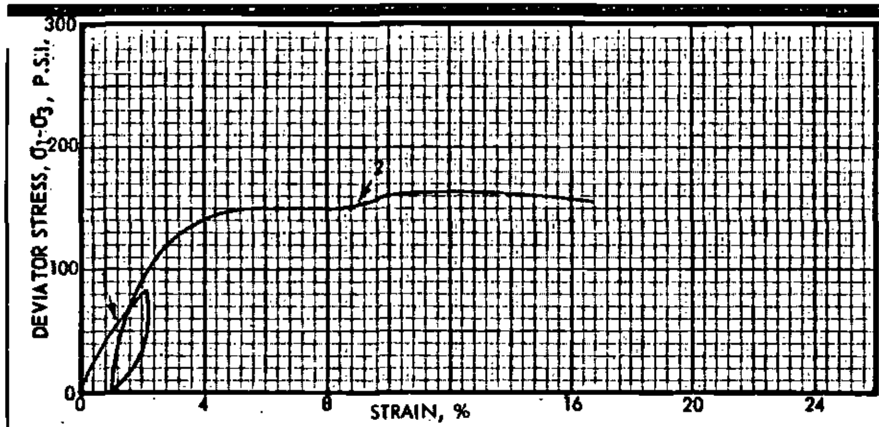
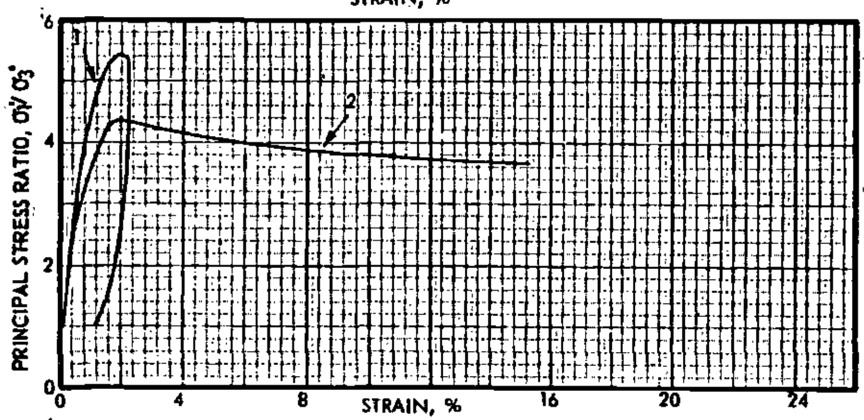
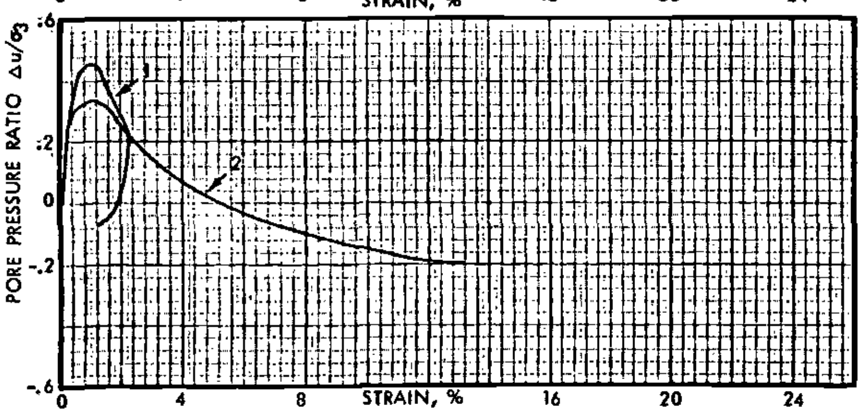
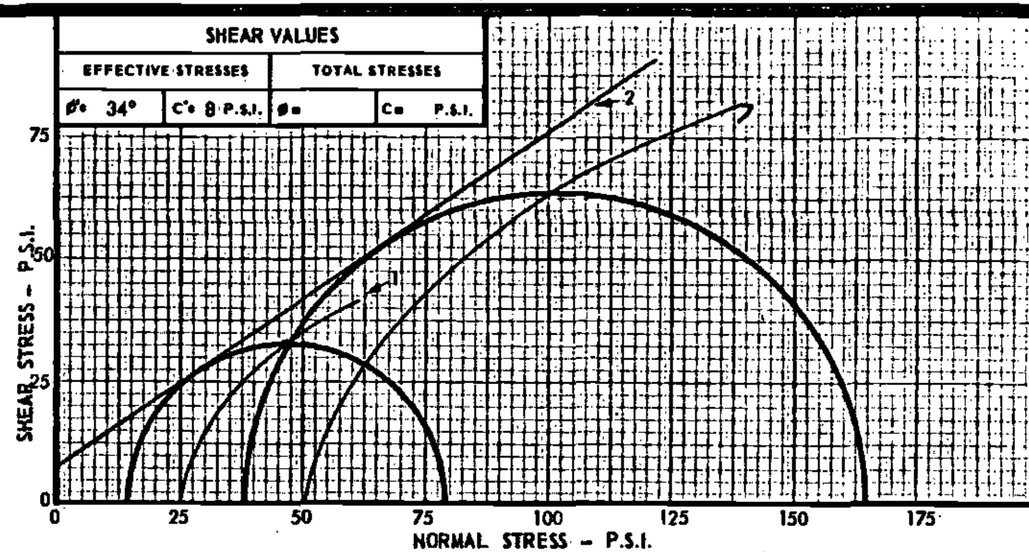
Scale: As Shown	Project No. 83-1101
Date: 10-11-83	
Prepared by: TPZ	Figure No.
Checked by: JDH	D-9
Approved by: JAD	







SHEAR VALUES			
EFFECTIVE STRESSES		TOTAL STRESSES	
Ø= 34°	C= 8 P.S.I.	Ø=	C= P.S.I.



SPECIMEN NUMBER	SPECIMEN LOCATION		INITIAL SPECIMEN DATA						TYPE OF SAMPLE
	BORING NUMBER	DEPTH IN FEET	SOIL CLASSI-FICATION	LENGTH IN INCHES	DIAMETER IN INCHES	DRY DENSITY (P.C.F.)	MOISTURE CONTENT IN PERCENT		
PB-2	9-3	61'	CL	7.04	2.86	97.3	26.3%		PITCHER

SYMBOLS	SPECIMEN NUMBER	APPLIED LATERAL PRESSURE σ_3 (P.S.I.)	MAXIMUM DEVIATOR STRESS $\sigma_1 - \sigma_3$ (P.S.I.)	TEST VALUES AT FAILURE - MAXIMUM σ_1'/σ_3': EFFECTIVE LATERAL PRESSURE σ_3' (P.S.I.)	TEST VALUES AT FAILURE - MAXIMUM σ_1'/σ_3': MAJOR EFFECTIVE STRESS σ_1' (P.S.I.)	BACK PRESSURE (P.S.I.)	TYPE OF TEST
1	PB-2	25	---	17.1	87.7	50	CU PROGRESSIVE
2	PB-2	50	162.9	36.5	159.8	50	CU PROGRESSIVE

TRIAXIAL COMPRESSION TEST

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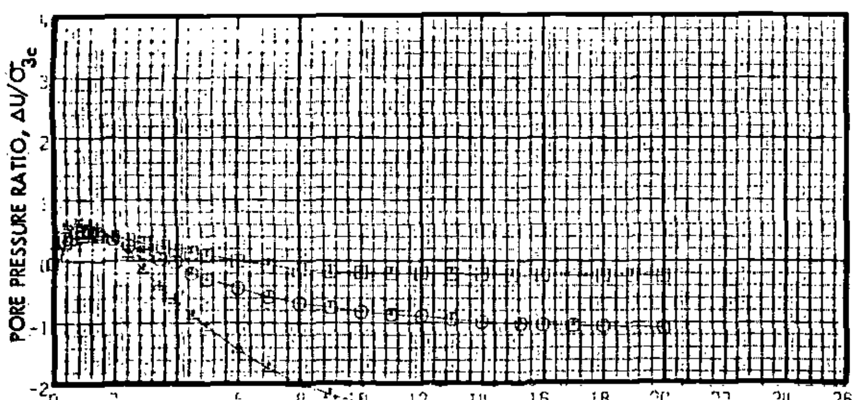
Southern California Rapid Transit District
METRO RAIL PROJECT

Converse Consultants Geotechnical Engineering and Applied Sciences

Scale As Shown
Date 10-11-83
Prepared by RG
Checked by JDH
Approved by JAD

Project No. 83-1101
Figure No. D-10



SHEAR VALUES			
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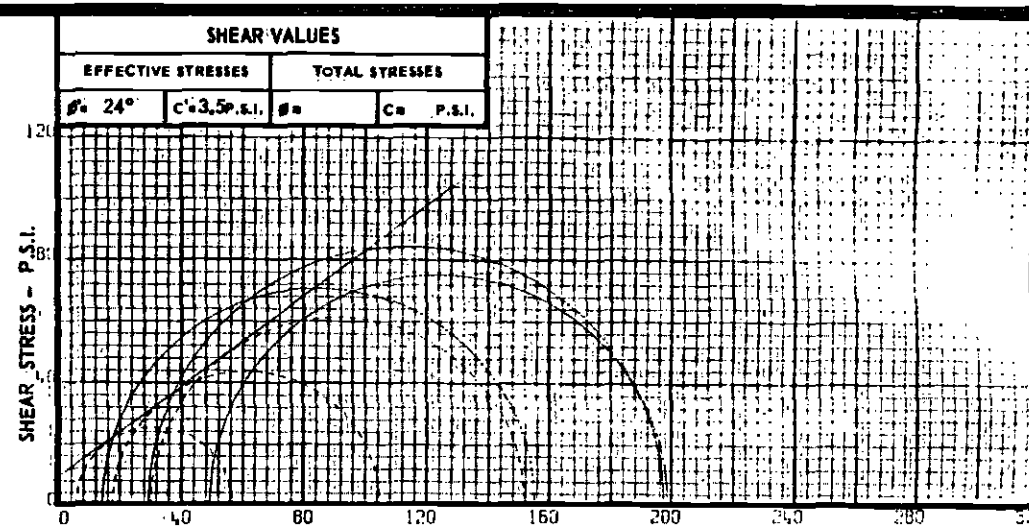

SPECIMEN NUMBER	SPECIMEN LOCATION		INITIAL SPECIMEN DATA					TYPE OF SAMPLE
	BORING NUMBER	DEPTH IN FEET	SOIL CLASSIFICATION	LENGTH IN INCHES	DIAMETER IN INCHES	DRY DENSITY (P.C.F.)	MOISTURE CONTENT IN PERCENT	
S3	9	99	CL	4.97	2.42	97.00	[illegible]	UNDISTURBED
S3	9	100	CL	4.97	2.42	98.40	25.30	UNDISTURBED
S3	9	101	CL	4.98	2.42	100.10	25.40	UNDISTURBED

SYMBOLS	SPECIMEN NUMBER	APPLIED LATERAL PRESSURE σ_3 (P.S.I.)	MAXIMUM DEVIATOR STRESS $\sigma_1 - \sigma_3$ (P.S.I.)	TEST VALUES AT FAILURE – MAXIMUM σ_1'/σ_3'		BACK PRESSURE (P.S.I.)	TYPE OF TEST
				EFFECTIVE LATERAL PRESSURE σ_3' (P.S.I.)	MAJOR EFFECTIVE STRESS σ_1' (P.S.I.)		
	S3	50.0	149.6	30.8	152.9	75.0	CONSOLIDATED UNDRAINED
	S3	30.0	168.3	18.3	105.6	60.0	CONSOLIDATED UNDRAINED
	S3	15.0	141.2	6.3	57.5	50.0	CONSOLIDATED UNDRAINED

TRIAXIAL COMPRESSION TEST

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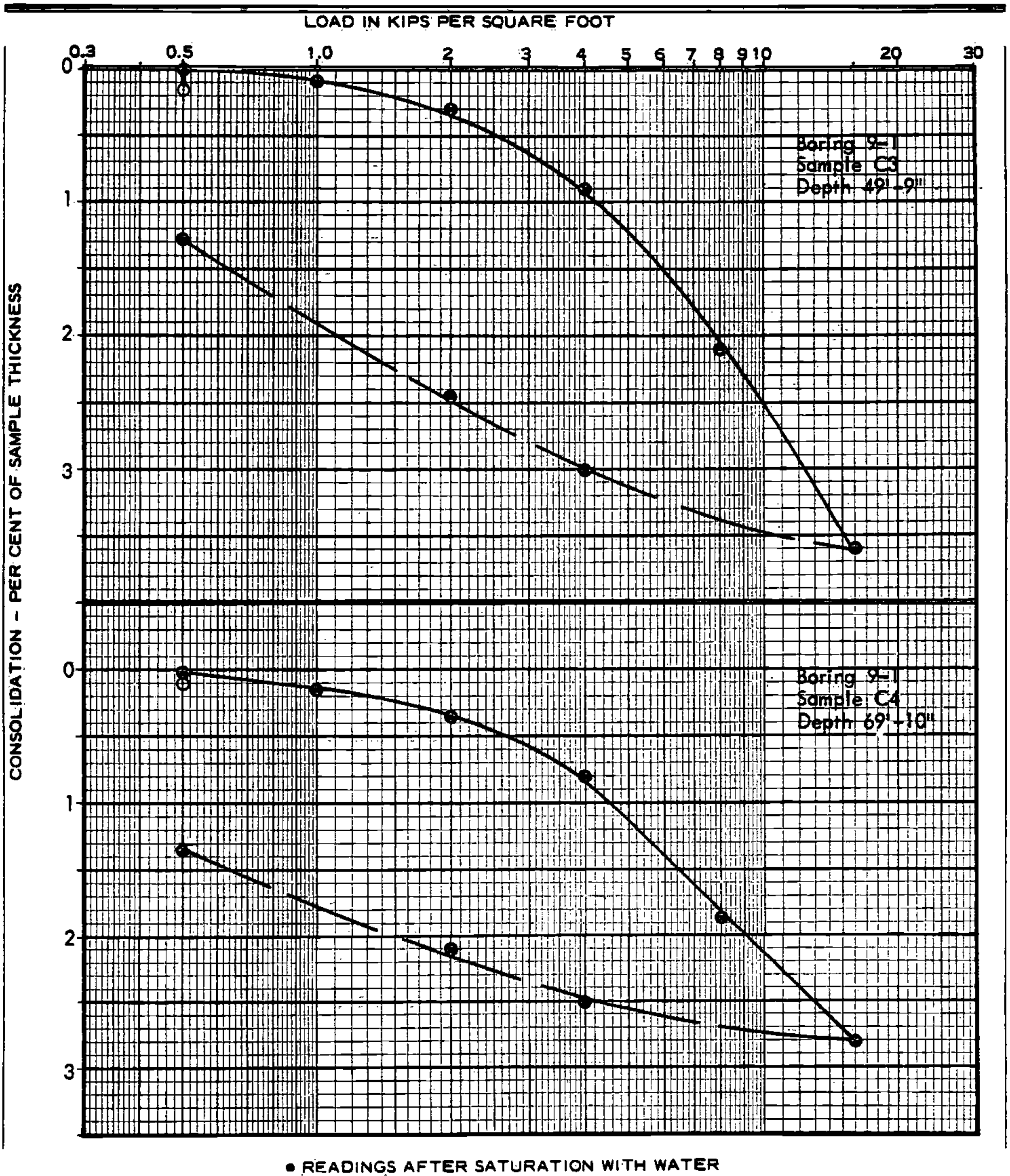
Scale	As Shown	Project No.	83-1101
Date			
Prepared by	TPZ	Drawing No.	
Checked by	JSL		D-11
Approved by	JSL		

FORM NO. D60/77



CONSOLIDATION TESTS

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Figure No.
D - 12

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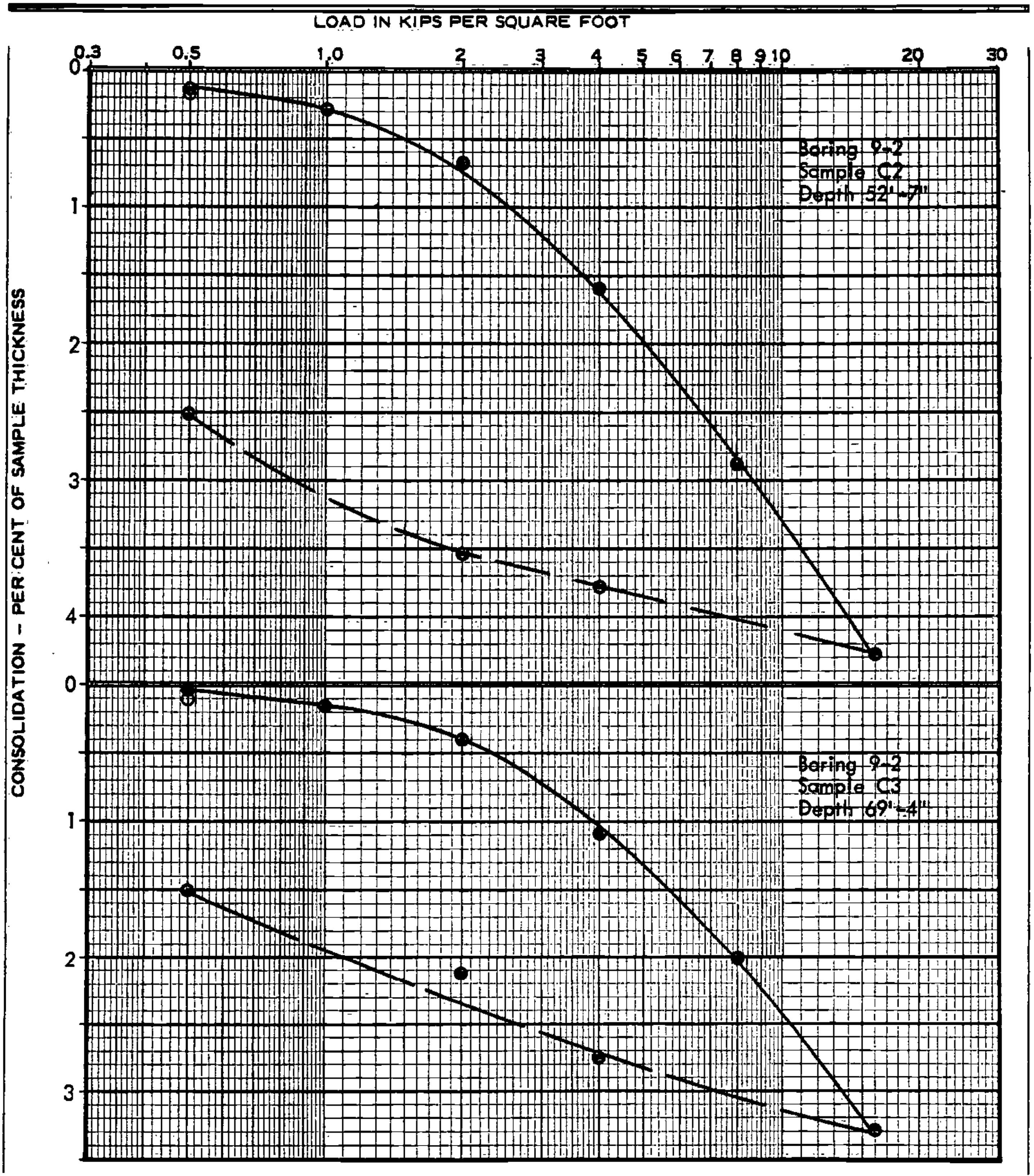

CONSOLIDATION TESTS

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Figure No.
D - 13

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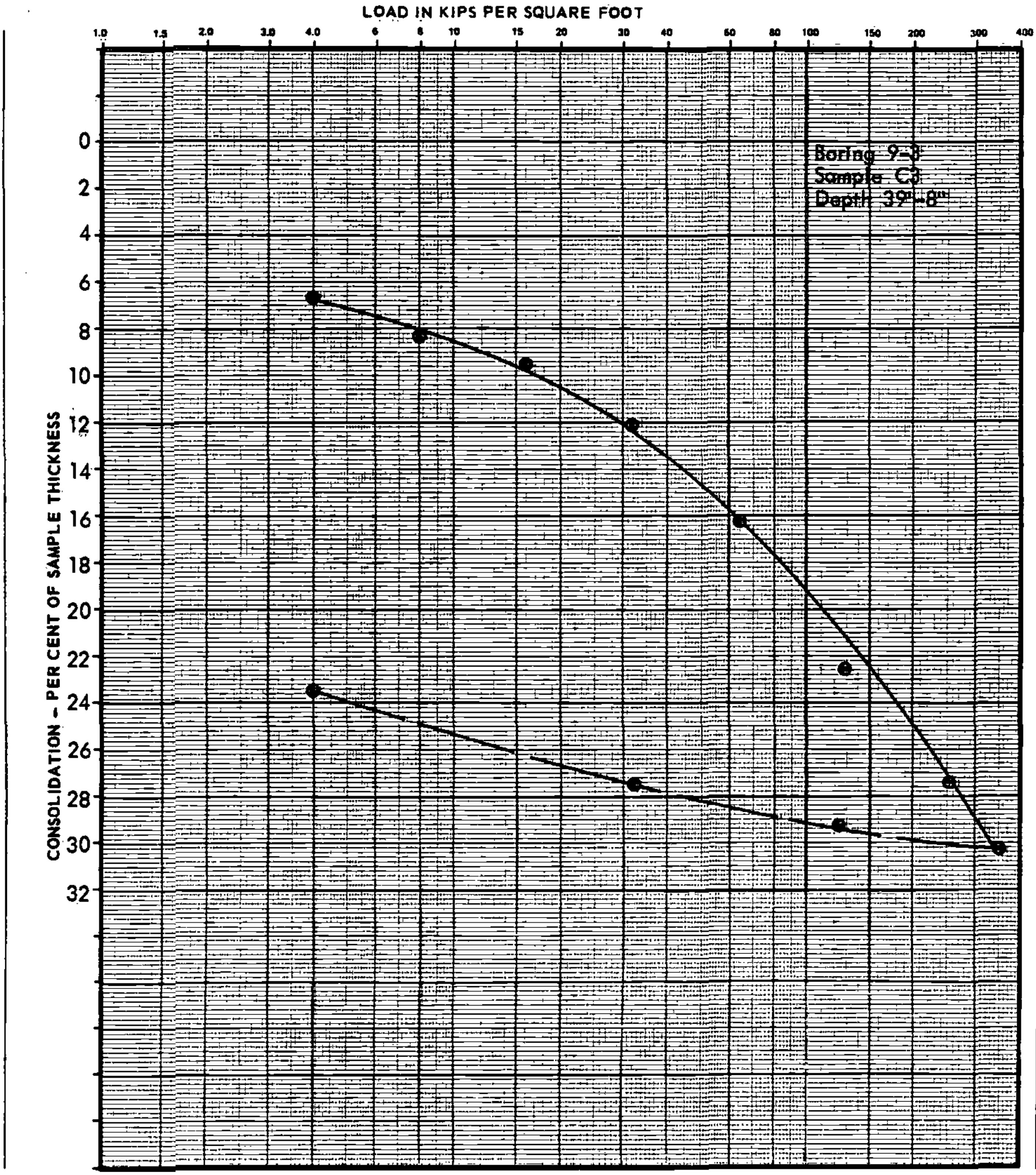

● READINGS AFTER SATURATION WITH WATER

CONSOLIDATION TESTS

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Project No
83-1101

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Figure No.
D-14

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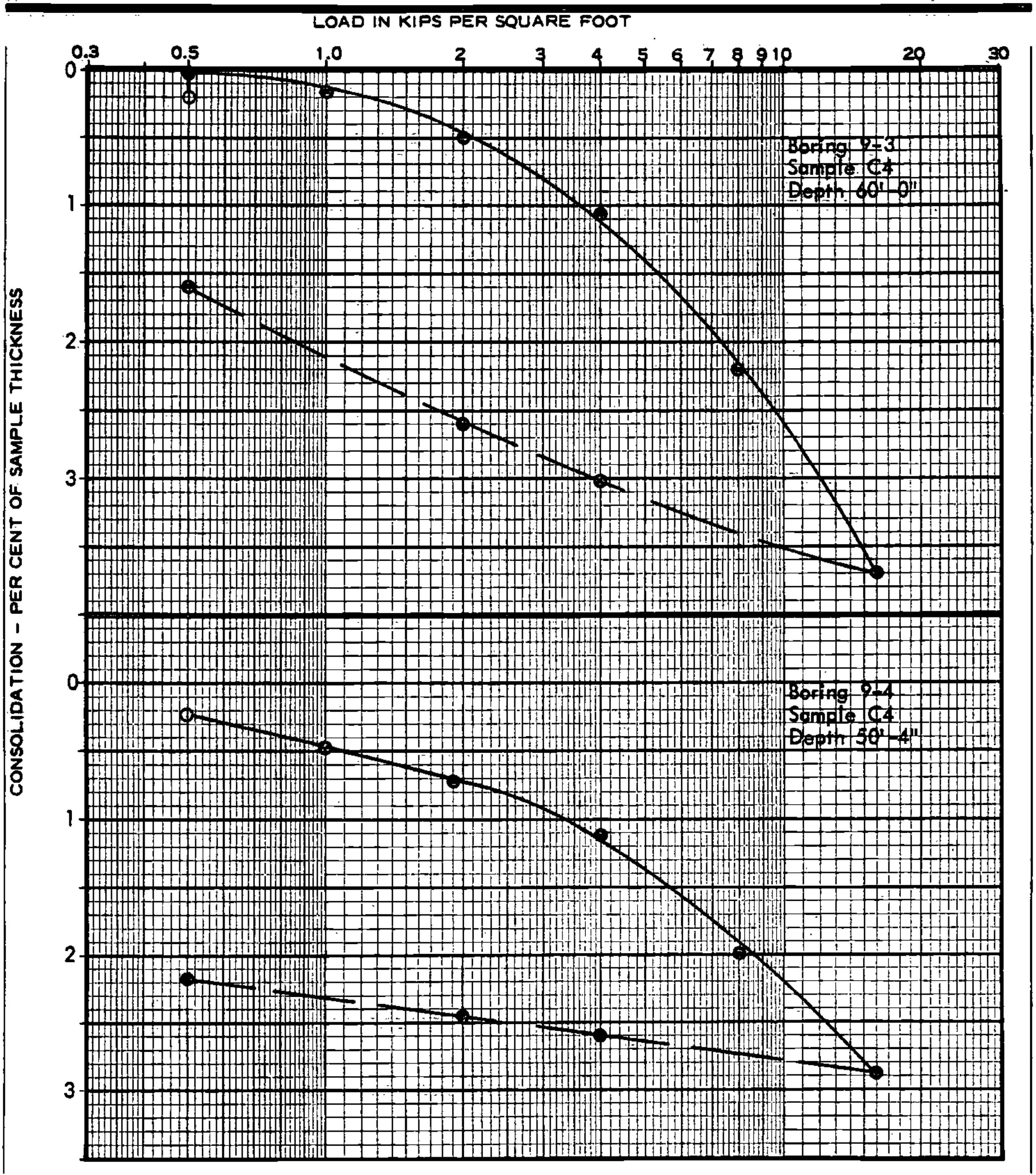

● READINGS AFTER SATURATION WITH WATER

CONSOLIDATION TESTS

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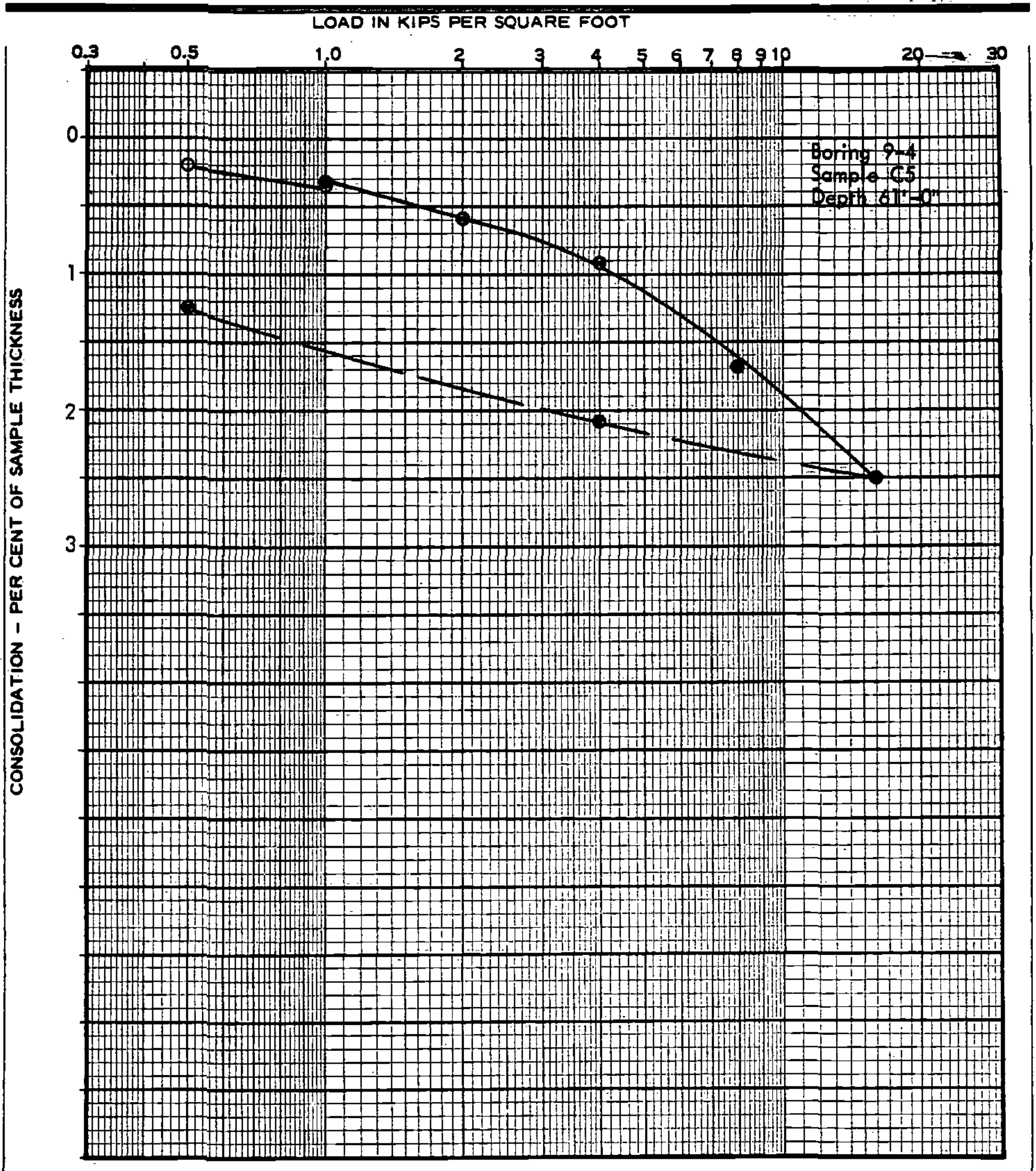
Figure No.
D - 15

Approved for publication ______ by ______



● READINGS AFTER SATURATION WITH WATER

CONSOLIDATION TESTS

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83-1101

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Figure No.
D - 16

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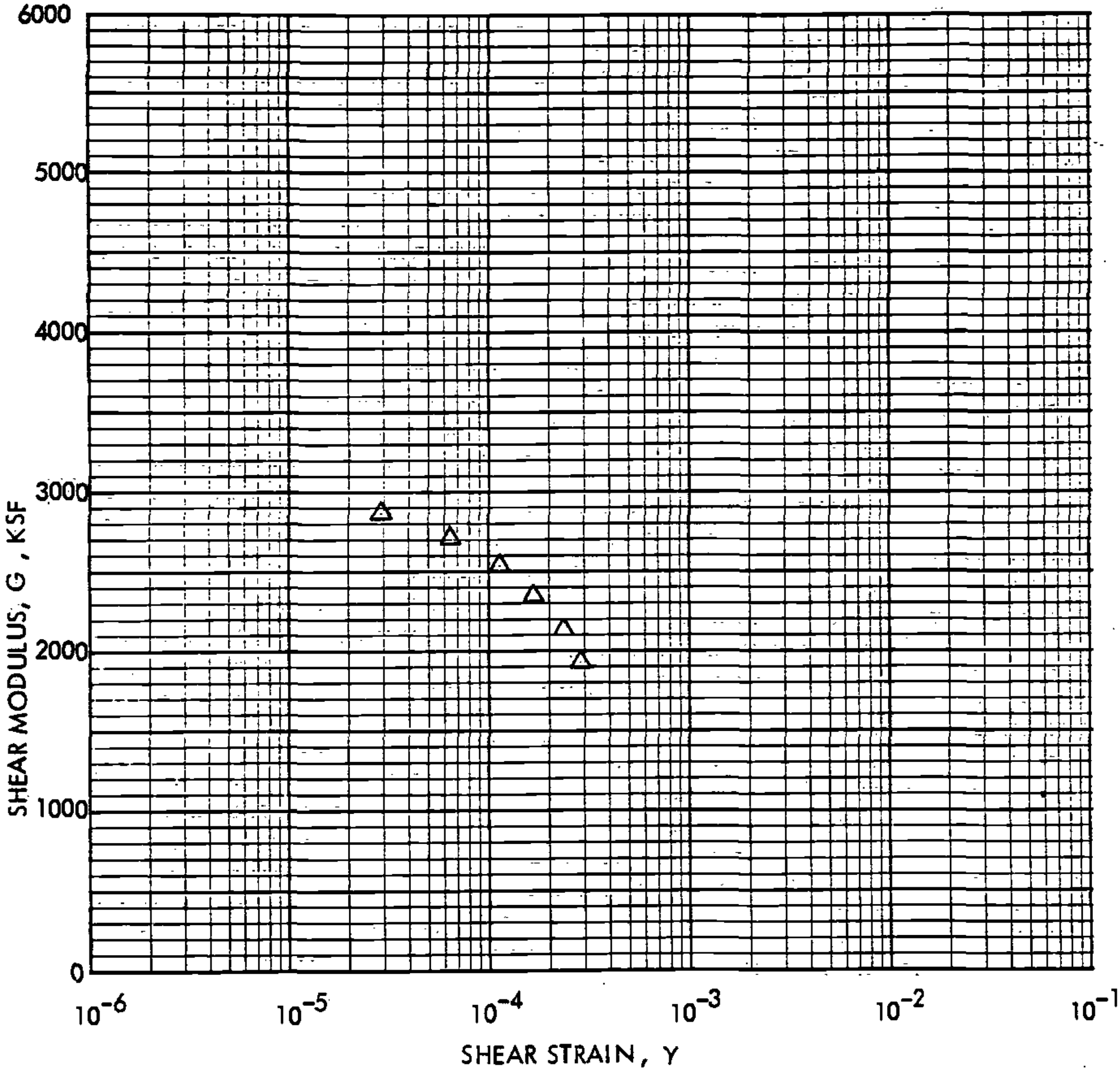
STRAIN DEPENDENT DYNAMIC SHEAR MODULUS



BORING	SAMPLE	DEPTH(FT)	γ_d(PCF)	w_o (%)	$\bar{\sigma}_c$(PSI)	SYMBOL
9	S-3	101	98	27	50	△

Sample Description: Gray Silty Claystone; moist

RESONANT COLUMN TEST

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83-1101

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Figure No.
D-17

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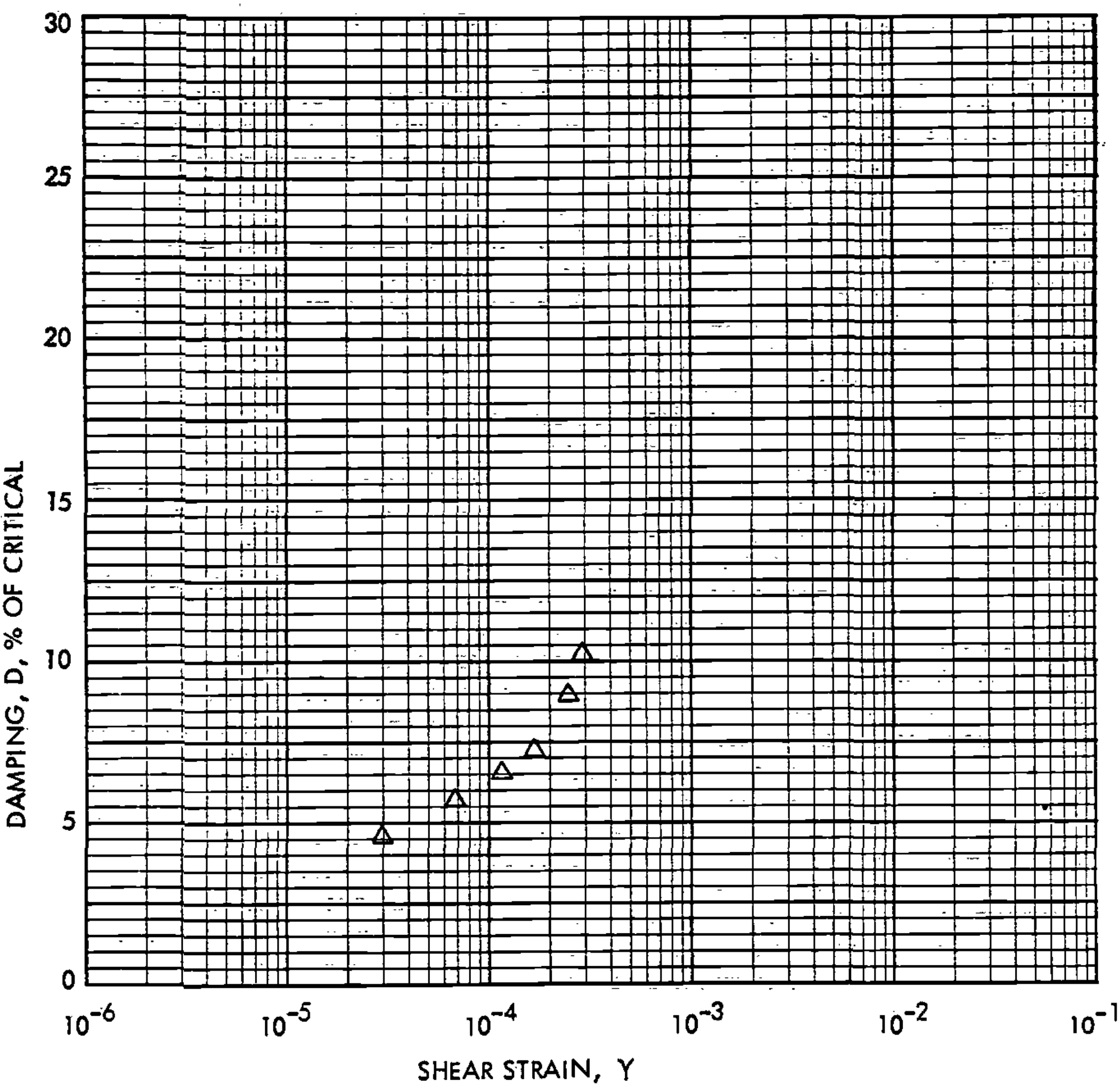
STRAIN DEPENDENT DAMPING



BORING	SAMPLE	DEPTH(FT)	γ_d(PCF)	w_o(%)	$\bar{\sigma}_c$(PSI)	SYMBOL
9	S-3	101	98	27	50	△

Sample Description: Gray Silty Claystone; moist

RESONANT COLUMN TEST

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Project No.
83-1101

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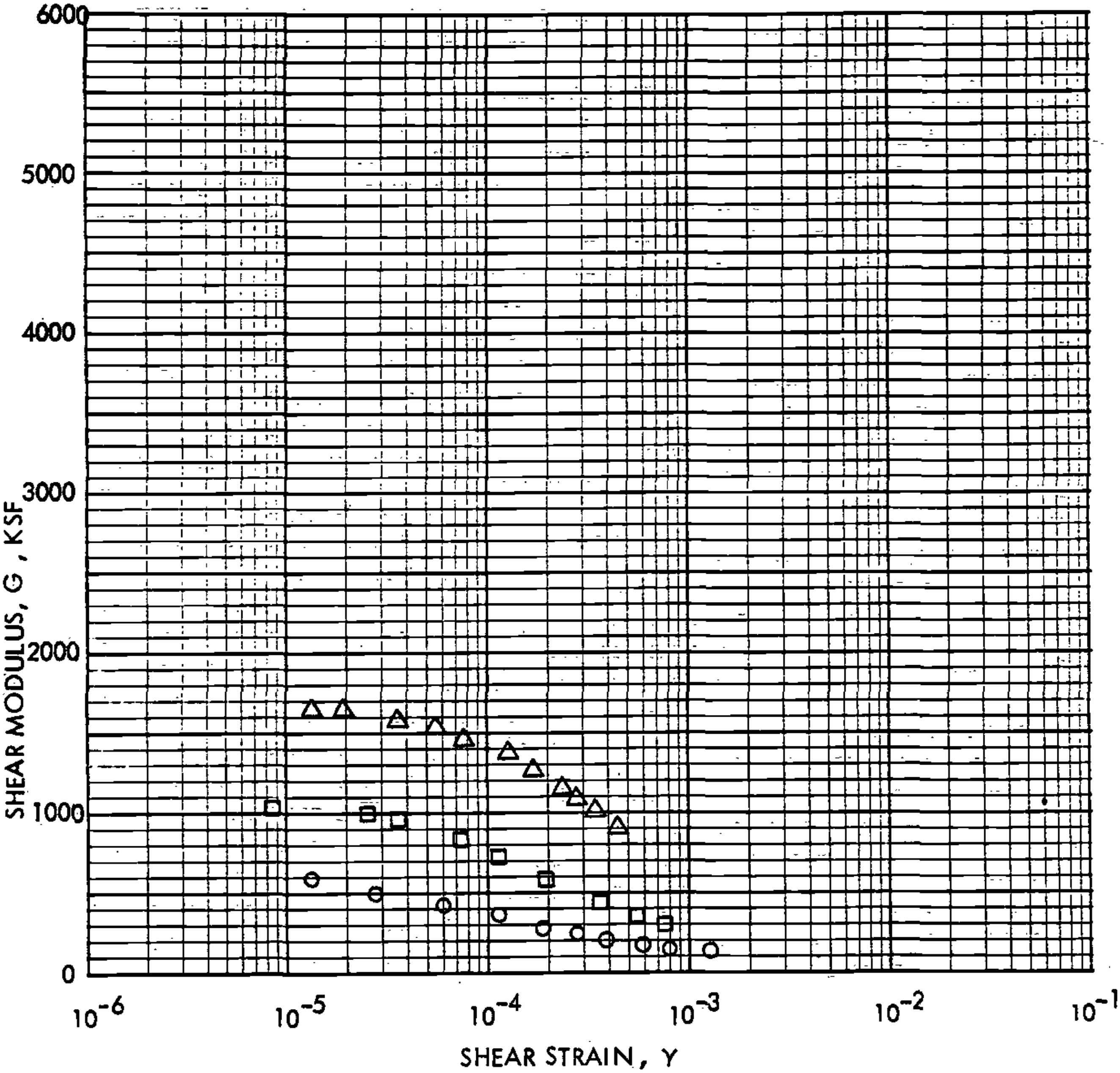
Figure No.
D18

STRAIN DEPENDENT DYNAMIC SHEAR MODULUS



BORING	SAMPLE	DEPTH(FT)	γ_d(PCF)	w_a(%)	$\bar{\sigma}_c$(PSI)	SYMBOL
9	S-4	115	99	25	15	○
					30	□
					50	△

Sample Description: Gray Siltstone, with little fine sand and occasional shell; very moist

RESONANT COLUMN TEST

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Figure No. D-19

Approved for publication ______ by ______

Appendix E

Technical Considerations

APPENDIX E TECHNICAL CONSIDERATIONS

E.1 SHORING PRACTICES IN THE LOS ANGELES AREA

E.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.

The pressures shown on Table E-1 are pressure envelops used to design tieback shoring walls and not actual measured pressures. The tiebacks were pre-stressed and locked off at the computed design loads. The loads imposed on the soil and experienced by the shoring wall were a product of these design loads. Thus the soil and wall loads represent, in a sense, a self fulfilling prophecy. However, use of specific shoring pressures which result in acceptable ground movements are generally considered to be appropriate.

E.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 an 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.

E.1.3 Century City Theme Towers (Crandall, 1977)

This project involved a shored excavation from 70 to 110 feet deep in the 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.

E.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

TABLE E-1

SHORING LOADS IN LOS ANGELES AREA

PROJECT LOCATION	EXCAVATION DEPTH (ft)	SOIL CONDITIONS	ACTUAL DESIGN PRESSURE (P)	EQUIVALENT DESIGN PRESSURE (P')
Broadway Plaza Near 7th/Flower Station	15 to 30	Fill over Alluvium Sands	19.0H	15.2H
500 South Hill	25	Fill over Sands & Gravel	22.0H	17.6H
Tishman Building Near CEG-14	25	Alluvium-Clays, Sand, Silt	19.0H	15.2H
Equitable Life Near CEG-14	55	Alluvium Sand/Siltstone	20.0H	17.5H
Arco Near CEG-9	70 to 90	Alluvium over Claystone	16.0H	12.0H
Century City Near CEG-20	70 to 110	Alluvium-Clays & Sands	18.0H	14.4H
St. Vincent's Hospital Near 3rd & Lankershim	70	Thin Alluvium over Puente	15.0H	12.0H
Oxford Plaza Near 7th/Flower	40	Fill & Alluvium over Siltstone	21.0H	16.8H
Bank Building 2nd & San Pedro	40	Alluvium (including Sand & Gravel over Siltstone)	20H	17.5H

* 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.

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.

E.1.5 Design Lateral Load Practices

Table E-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 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-.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.

E.2 SHORING CONSIDERATIONS

E.2.1 General

The function of the shoring at the Station sites will be twofold: provide a safe and stable excavation; and, minimize ground movements. In this Appendix section we will discuss the primary factors affecting shoring performance. In addition, we will develop the concept that, in the competent soils underlying the Station sites, either slurry walls or a conservatively designed soldier pile wall may limit ground movements sufficiently to eliminate the need for underpinning. Data is presented to support the recommended design procedures and estimated ground movements provided in Section 6.5.

As part of this study, we reviewed data from several soldier pile shoring systems in both the Los Angeles area and the Seattle area. Seattle area data was included since the soil and excavation conditions are believed to be similar to those anticipated for the Stations. Three of the Seattle projects involved deep excavations into hard silty clays which have similar strength and stiffness properties to the Puente and Fernando Formations. Each project was instrumented with survey points on the wall and most included inclinometer data. Table E-2 summarizes the data from these projects and include:

- Soil Conditions
- Excavation Depth
- Wall Stiffness: This is represented by the modulus and moment of inertia of the steel soldier pile section per foot of wall length.
- Support Spacing: This represents the average vertical spacing between tieback supports.
- Preload: This represents the lateral load design used to compute the tieback prestress loads. The value given on the table has been normalized to a uniform pressure distribution for ease of comparing the resultant total design load for each system.
- Movements: Maximum movements at both the top and bottom of the walls are presented if the data was available.
- Stiffness Parameters: The stiffness parameters were computed according to methods proposed by Goldberg, et al. (1976) and Schultz (1983). These parameters represent the total stiffness of the shoring including both the wall section stiffness and support spacing.
- Deformation Mode: The general shape of the wall deformation as inferred from the available data are represented by idealized deformation modes.

The data presented in Table E-2 is discussed in the following sections.

E.2.2 Depth

All other things being equal, the maximum ground movements seem to increase more or less linearly with the excavation depth. Thus the magnitude of maximum ground movement is generally expressed as a percentage of the excavation depth. Typically in the excavations summarized on Table E-2, the maximum movements have ranged from less than 0.1% to about 0.4%. Thus with a 60 foot excavation, the range of maximum movements expected would be on the order of less than 1 inch to about 3 inches.

The depth of the excavation also determines the level of shear stress imposed on the soil outside the excavation. With increasing depth, the soil can be stressed beyond the elastic range going into plastic deformation and eventually approach failure.

E.2.3 Soil Conditions

The soil conditions have a significant effect on the behavior of the shoring. The wall movement decreases with increasing soil strength and stiffness. This is a consequence of two factors: the higher the modulus, the smaller the soil deformations will be in response to the imposed stress changes; and, the stronger the soil the lower the magnitude of plastic yield.

The relationship between wall movement, excavation depth, and soil strength has been well documented for soft to medium clays. The relationship consists of comparing the maximum wall strain (percentage of excavation depth) to the factor of safety against basal heave and/or a stability number. Figure E-1 presents the results of a finite element analysis plotted in this manner (Clough, 1980). Other similar data have been developed by Goldberg, et al., 1976; Mana and Clough, 1981; Cording and O'Rourke, 1977. The excavations presented on Table E-2 were all believed to have a high factor of safety against basal heave.

In addition to the stress-strain behavior of the soil, the soil type affects the risk of loss of ground due to sloughing and piping between wall elements. A granular soil below the water table may have virtually no stand up time and slough into the excavation before the lagging can be placed. Proper construction procedures are essential under these conditions.

E.2.4 Ground Water Conditions

The ground water condition can affect movements if it promotes loss of ground due to sloughing and piping. This problem can be significant in loose granular soils below the ground water table. Slurry walls offer a distinct advantage under these types of conditions. In the extreme case, high seepage forces can lead to development of a quick condition at the base of the excavation resulting in loss of passive soil support.

E.2.5 Type of Shoring

The type of shoring affects the wall movements. In general the stiffer the wall system, the smaller the movements. Goldberg, et al. (1976) have suggested the relationship EI/GwH+4 to describe the overall stiffness of the shoring system. The parameters include: E = wall modulus, I = wall moment of inertial, Gw = 62.4 pcf, and H = vertical strut or tieback spacing. Schultz (1983) has proposed a similar relationship but used H+3 instead of H+4. Thus a soldier pile wall with close support spacing can provide more system stiffness than a slurry wall with large support spacing. The significance of H is related to the observation that much of the wall movement occurred while the excavation is proceeding and before the next lower level of supports are placed. Figures E-1 through E-3 present wall movements as a function of shoring system stiffness based on both field observations and finite element analyses. Data from Table E-1 is plotted on all three figures.

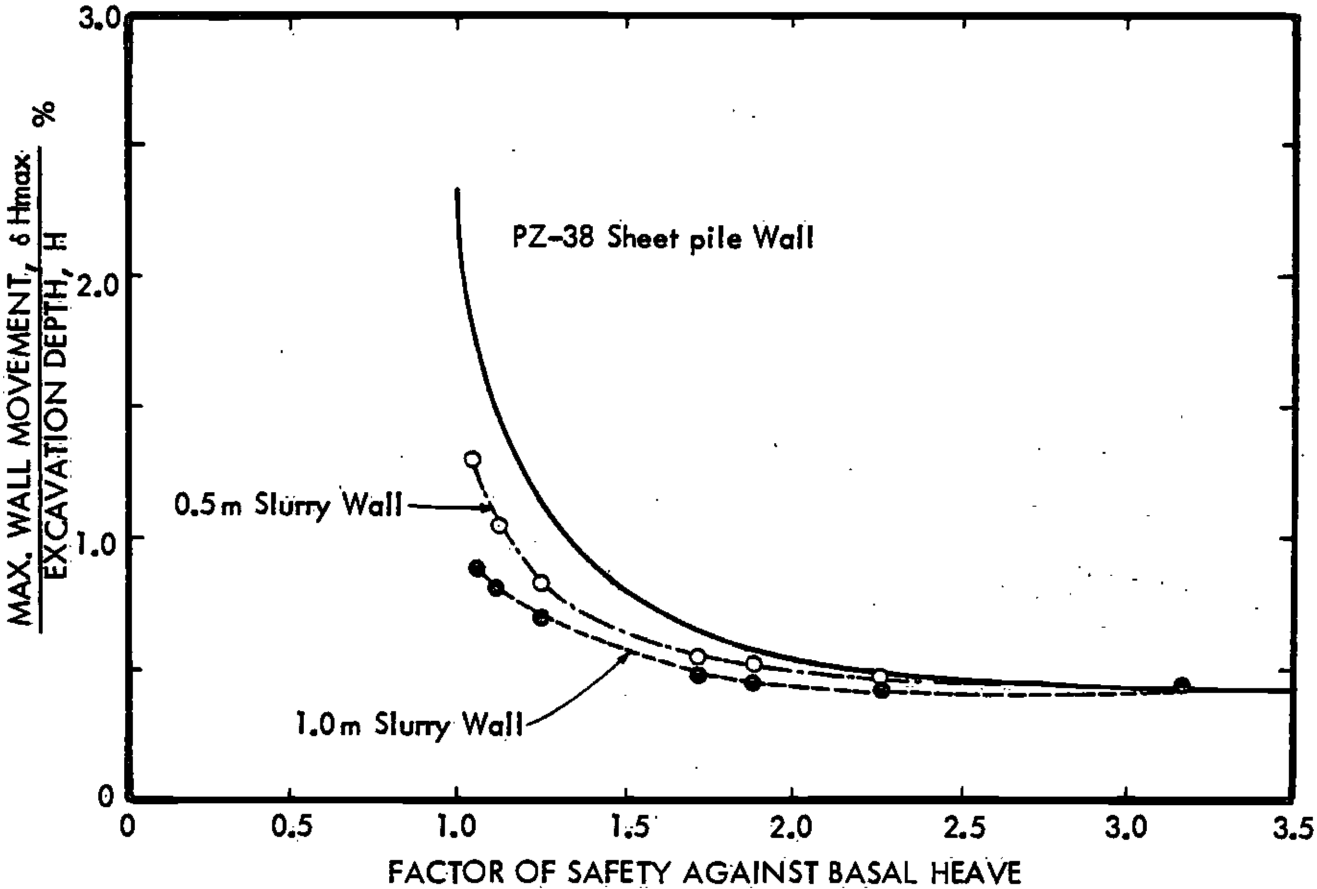
TABLE E-2

SUMMARY OF WALL PERFORMANCE

PROJECT NAME	SOIL CONDITIONS	EX DEPTH D (ft)	MOVEMENT MODE	WALL-EI (10^{-9} in²)	SUPPORT SPACING H (ft)	PRELOAD PRESSURE (psf)	MOVEMENT AT TOP (in)
Arco-Los Angeles	20' Alluvium over Shale	90.00	1	1.15	8.00	14.0D	4.00
Theme-Los Angeles	Old Alluvium, Clay & Sand	110.00	1	0.53	8.00	14.5D	3.00
St. Vincent's-L.A.	Alluvium over Puente	70.00	1	1.00	7.00	12.0D	1.00
Bank Calif.-Seattle	Hard Clays	64.00	2 & 7	1.20	10.00	30.0D	0.50
Columbia Ct.-Seattle	Hard Clay & Tills	120.00	2	4.00	3.50	30.0D	0.10
1st Inter-Seattle	Hard Clays & Tills	80.00	2	7.90	10.00	30.0D	0.10
3rd & Broad-Seattle	Hard Silts & Sands	42.00	2-7	3.80	12.00	24.0D	0.05

NOTES:

- See text for explanation
- Wall EI: Stiffness
- Preload pressure based on equivalent rectangular pressure distribution
- Movement Mode: See Figure E-4



(After Clough, 1980)

Approved for publication 10/4/83 by JAD

WALL STIFFNESS VS WALL MOVEMENTS AND F. S. AGAINST BASAL HEAVE

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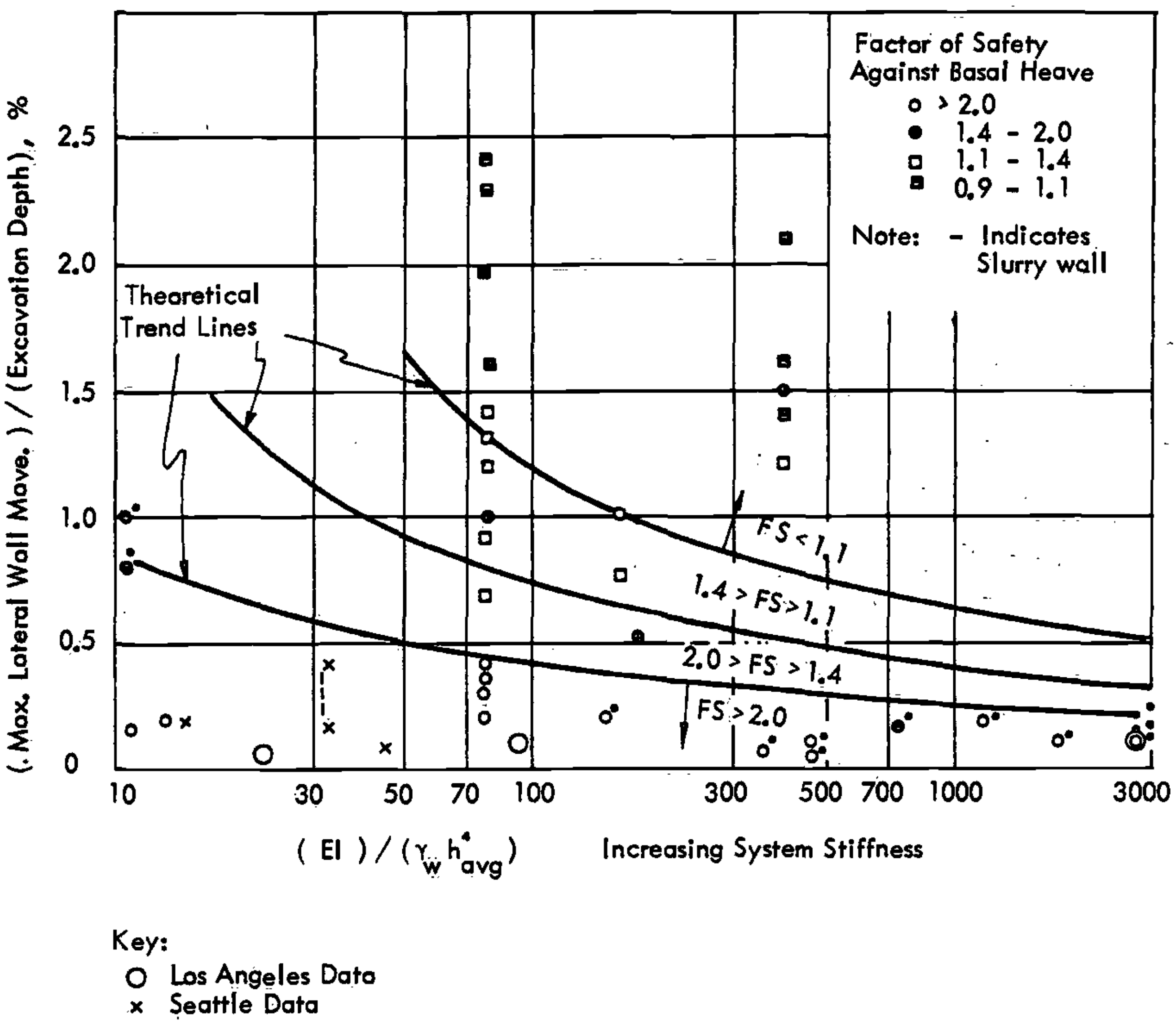
Figure No. E-1



(After Clough, 1981)

FIELD DATA - MAXIMUM WALL MOVEMENT VS BASAL STABILITY AND SYSTEM STIFFNESS

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Figure No.
E - 2

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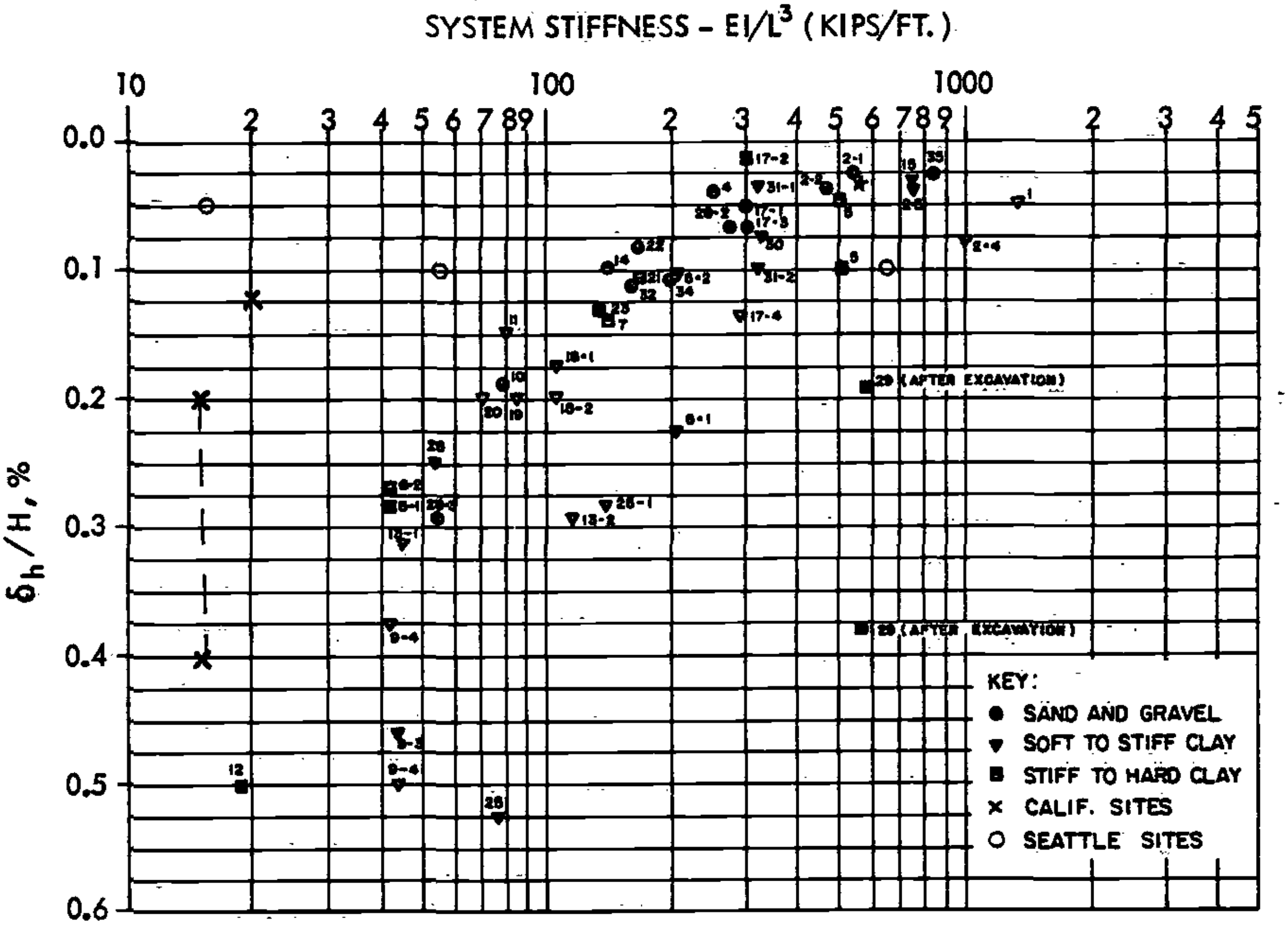

(After Schultz, 1981)

NORMALIZED MAXIMUM LATERAL DISPLACEMENTS VS SYSTEM STIFFNESS

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Figure No. E - 3

The more competent the soil, the less important is the wall system stiffness. This effect is shown on both Figures E-1 and E-2 for clays where the soil strength and stiffness is represented by the factor of safety against basal heave (see Clough, 1980, for definition of factor of safety).

The method of shoring installation can have a significant effect on ground movement. A properly installed slurry wall will cause minimum ground disturbance and provide good contact between the soil and the wall. A soldier pile wall is more prone to ground disturbance and may not provide uniformly good contact between the lagging and the soil.

E.2.6 Tiebacks versus Internal Bracing

For the depth of excavation proposed at the Station site, the shoring will require either internal bracing or tiebacks. Based on the available field data, there does not appear to be a significant difference between the maximum movement of properly designed and constructed internally braced walls and tieback walls. There may, however, be a difference in the distribution of the ground and wall movements. Figure E-4 shows idealized ground movements adjacent to different types of walls. These differences are not always observed as the type of soil, prestress loads and construction details can alter the distribution.

Possible differences between tiebacks and internal bracing include:

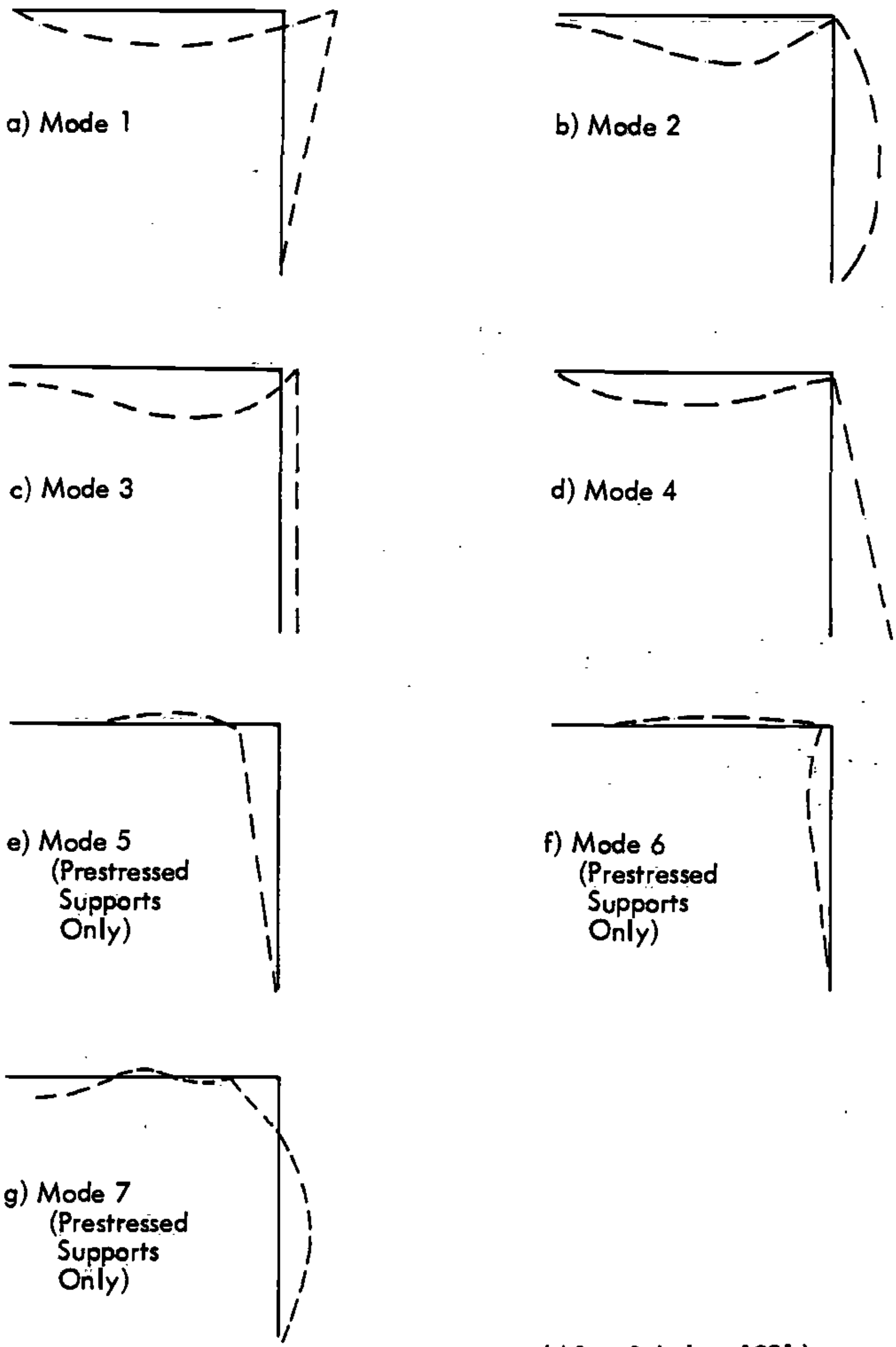
- Tiebacks are typically locked-off at 75% to 100% of the designed loads whereas struts are seldom pre-stressed above 50%. The higher prestress load tends to prestrain the soil and with high prestress loads can actually pull the wall back into the ground. In granular soil, the higher prestress can increase the modulus of the soil mass in the active wedge.

- With tiebacks, there is less incentive for the contractor to excavate a significant distance below the designated support level prior to installing the supports. Thus the tieback wall is less prone to the contractor excavating too far prior to installing supports (which can result in significant ground movement).

- Tiebacks may be subject to creep which could result in additional movements particularly if the excavation is to be open for an extended period. In our opinion, provided the anchors are conservatively designed, creep will not be a problem in the competent soils at the Station sites.

- Tiebacks require installation under adjacent properties. This can sometimes be a problem due to the existence of adjacent underground structures and obtaining easements.

- Struts transmit loads from one side of the excavation to the other side whereas tiebacks transmit the loads back into the soil mass. This difference tends to result in the difference in the distribution of the ground movements shown on Figure E-4.



(After Schultz, 1981)

MODES OF DEFORMATION

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Figure No. E-4

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- Struts are subjected to temperature changes which induce significant stress changes in the members. Temperature effects have little impact on tiebacks since they are buried in the ground.

E.2.7 Prestress Loads

The magnitude of the tieback or strut prestressing appears to affect wall and ground movements. Figures E-5 and E-6 present maximum wall movements for both clays and sands as a function of the prestress loads. The plot is based on work by Clough (1980) with data from Los Angeles and Seattle added. The Seattle movements generally were about 1/3 to 1/2 those measured at the Los Angeles sites. This appears to fit the prestress relationships since the Seattle prestress loads were typically two times the Los Angeles loads.

E.2.8 Surcharge Loads

Adjacent heavy surcharge loads, such as buildings not underpinned, will increase stress in the soil and tend to increase movements. This is somewhat compensated for by designing and prestressing for high earth pressures.

E.2.9 Construction Procedures

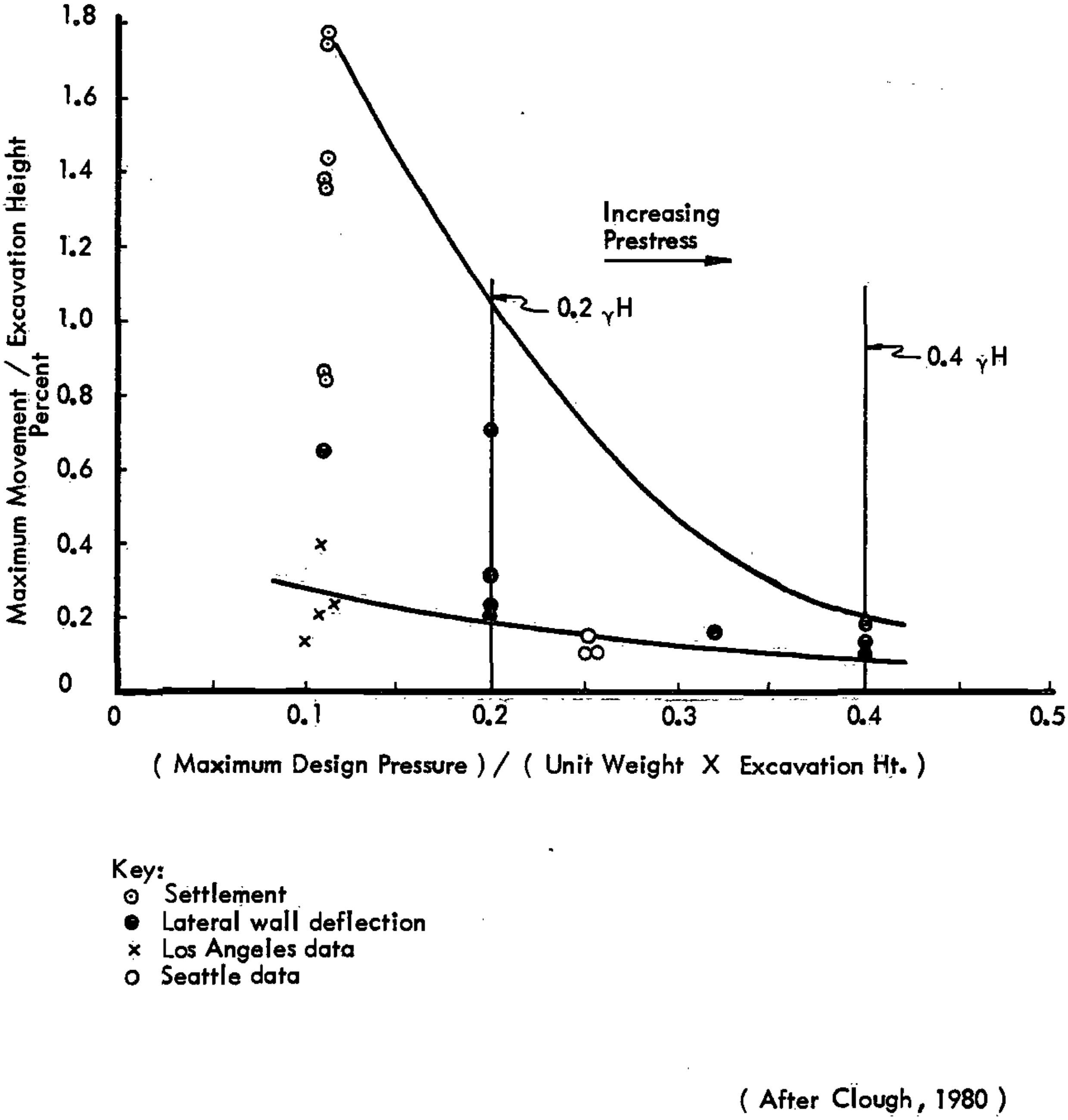
The construction procedures can have a significant effect on the performance of a shoring system. A slurry wall has a distinct advantage in that is is less susceptible to poor workmanship. This is particularly true in poor soil conditions where rapid placement of lagging can be critical with soldier pile walls. Tiebacks also tend to be less prone than internal bracing. With internal bracing problems of prestressing, improper placement of wedges, and over-excavating can result in increased ground and wall movements.

E.2.10 Distribution of Ground Movements

Figure E-4 presents general shapes of the horizontal and vertical movements adjacent to different types of shoring systems.

The lateral movements of the Los Angeles excavations summarized on Table E-2 generally fit Mode 1 with the maximum lateral movement occurring at the top of the walls. The Seattle cases generally fit Mode 2 or Mode 3 with the maximum movement near the base of the excavation. Some of the Seattle cases included wall sections which were pulled into the excavation at the top. We believe that the primary difference between the Los Angeles and Seattle behavior is the higher prestress load used in Seattle. The size of the no-load zone used in Seattle, which extends further into the soil mass, may also contribute to the difference.

Little ground settle data was available for the projects summarized on Table E-2. The following comments are based on compilation of data by Schultz (1983) and Goldberg, et al. (1976). It appears that the maximum vertical movements are generally equal to 50-100% of the maximum lateral movements. To be conservative, the maximum settlement should be assumed equal to the maximum lateral movements. For dense soils the available data indicates that the maximum settlement will generally occur at a distance away from the wall equal



OBSERVED WALL MOVEMENTS VS DESIGN PRESTRESS PRESSURES - CLAY SOILS

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Figure No. E - 5

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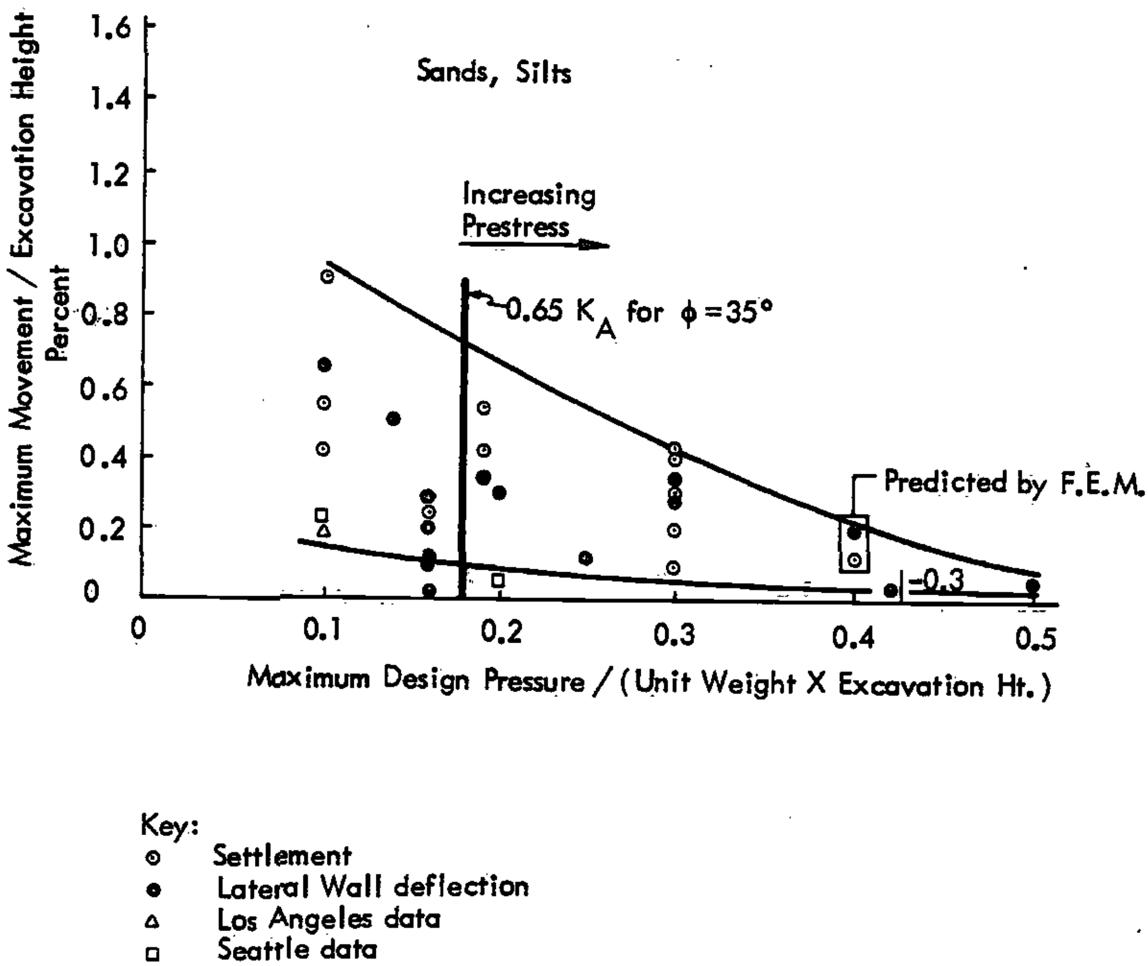

(After Clough, 1980)

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OBSERVED WALL MOVEMENTS VS DESIGN PRESTRESS PRESSURES - SAND SOILS

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Figure No. E-6

to about 50% of the excavation depth. In stiff clays the settlement trough appears to extend away from the wall a horizontal distance equal to about 2 times the excavation depth. In dense sands and gravels, the trough appears to extend a distance equal to about the depth of the excavation.

E.2.11 Conclusions

As applied to the proposed excavations, several important concepts can be drawn from the above discussions:

- In terms of wall stiffness, a slurry wall may not offer a significant advantage over a more conventional soldier pile wall. This is because the soils are relatively strong and stiff. The factor of safety against basal heave at the two sites is estimated to exceed 4.0. As shown in Figure G-1, based on finite element analyses, there is virtually no theoretical difference between the movement of three walls of vastly different stiffness provided the factor of safety exceeds about 3.0.

- In our opinion, the data supports the concept that it may be feasible to construct a conservatively designed soldier pile wall that results in small enough movements to eliminate the need to underpin. This opinion is based on the high strength and stiffness of the soils and the past performance of walls in Los Angeles and Seattle.

- The primary advantage of a slurry wall would be to minimize potential construction related problems.

Technical support for the specific design criteria for conservative soldier pile walls presented in Section 6.5 include:

- WALL PRESSURES: The data shown on Figures E-5 and E-6 support the concept that increased preload will reduce ground movements. The lower line suggested by Clough on the plots appear to agree well with the data from Los Angeles and Seattle. In our opinion the 40% preload increase recommended will reduce ground movements by 25% to 50%.

- USE OF STREET DECKING: In Section 6.5, we recommend supporting the top of the wall with the street decking prior to initiating any significant excavations. Much of the ground movements appear to occur when the shoring is acting as a cantilever. We believe by providing lateral support prior to excavating, the wall movements will be reduced.

- INCREASED NO-LOAD ZONE FOR TIEBACK ANCHORS: The no-load zone typically used in Los Angeles extends from the base of the excavation into the soil mass at a 1:1 slope. The no-load zone in Seattle typically extends horizontally into the soil a distance equal to 1/4 to 1/3 and then upward at a 1:1 slope. We believe by increasing the size of the no-load zone, the lateral loads will be transferred further back into the soil and reduce anchor and subsequent wall movements.

- MINIMUM 8 FOOT SUPPORT SPACING: As discussed in Section E.2.5, the overall stiffness of the shoring is a function of both the wall section stiffness and the vertical support spacing. Without knowing the wall stiffness that the contractor will use, it is not theoretically possible to determine a specific support spacing. The recommended 8-foot spacing was based on judgement and good engineering practice.

- PREDICTED GROUND MOVEMENTS: Based on the available data, we believe that a conservative soldier pile wall designed and constructed in accordance with the criteria presented in Section 6.5.4.3 will perform as follows:

  - Maximum lateral and vertical movements will be equal to 0.1% of the excavation depth.

  - The angular rotation of adjacent buildings caused by ground movements will be about 1:1000. Thus a 60 foot deep excavation would result in about 3/4-inch of settlement over a distance of about 60 feet.

E.3 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:

$$P_{AE} = 12\gamma\ H^2(1-k_v)K_{AE}$$

Where:

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

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

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,

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

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 recommended permanent wall uniform earth pressure (8H) presented in Figure 6-14 gives a seismic load $P_{AE} = 8H^2$. This value of P_{AE} was based on a peak ground acceleration of 0.3g (k_h = 0.2g) corresponding to the Operating Design

Earthquake (ODE). Results of the Seismological Investigation (Part I) indicate the probability of exceedance of 0.3g peak ground acceleration during an average 100-year period is on the order of 20%. This is an average recurrence of about 500 to 1000 years.

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 Figures 6-5 and 6-9).

E.4 LIQUEFACTION ANALYSES

E.4.1 Introduction

The procedures used in this study to evaluate liquefaction potential are based mainly on field observations of the performance of soils during previous earthquakes. The field observations made at the 7th/Flower site during this and the previous geotechnical investigation (1981 Geotechnical Investigation Report) that were used to establish the liquefaction potential of the various soils include:

- Standard Penetration Test (SPT) resistance
- Shear wave velocity measurements
- Observed behavior of soil in the large diameter borehole.

In addition to the field observations listed above, gradations of the soils obtained from the field were compared with gradations of materials which have liquefied during the past earthquakes and which are considered most susceptible to liquefaction in laboratory tests.

Each of the field observations (and comparisons) is described in the following text. It should be noted that the observations which have been made in the field only provide a basis upon which to judge the liquefaction potential of the various soils. Our conclusions regarding the liquefaction potential of the soils are generally supported by these observations. However, our conclusions are also based on engineering judgement.

E.4.2 Standard Penetration Resistance

The use of the Standard Penetration Test (SPT) in estimating the liquefaction potential of saturated cohesionless soil deposits has been the topic of many previous investigations. Results of these investigations have recently been summarized by Seed et al (1983). Basically, the method utilizes empirical relationships which have been developed from a comprehensive collection of SPT blow count data obtained from sites where evidence of liquefaction or no liquefaction was known to have taken place during past earthquakes. Empirical relationships that have been recently proposed by Seed et al (1983) are shown in Figure E-7.

While results of the Standard Penetration Test have been generally accepted as a good index upon which to estimate the liquefaction potential of saturated sand deposits, it should be noted that the SPT results cannot be utilized to evaluate the liquefaction potential of soils containing gravels, cobbles or boulders. Much of the Young Alluvium which underlays the 7th/Flower Station contains gravels, and the SPT blow counts recorded in the soils cannot be specifically relied upon. However, for those granular soils which did not include significant percentages of gravel-sized particles (silty sands and sand units), SPT blow count data were utilized along with the relationships shown in Figure E-7. In general, the SPT blow count measurements taken in the non-gravelly granular Alluvium below a depth of 15 feet are greater than 70 blows per foot, indicating that these soils are generally dense to very dense. These blow counts along with the relationship shown in Figure E-7 suggest that liquefaction of the soil deposits during strong earthquake ground shaking would be highly unlikely.

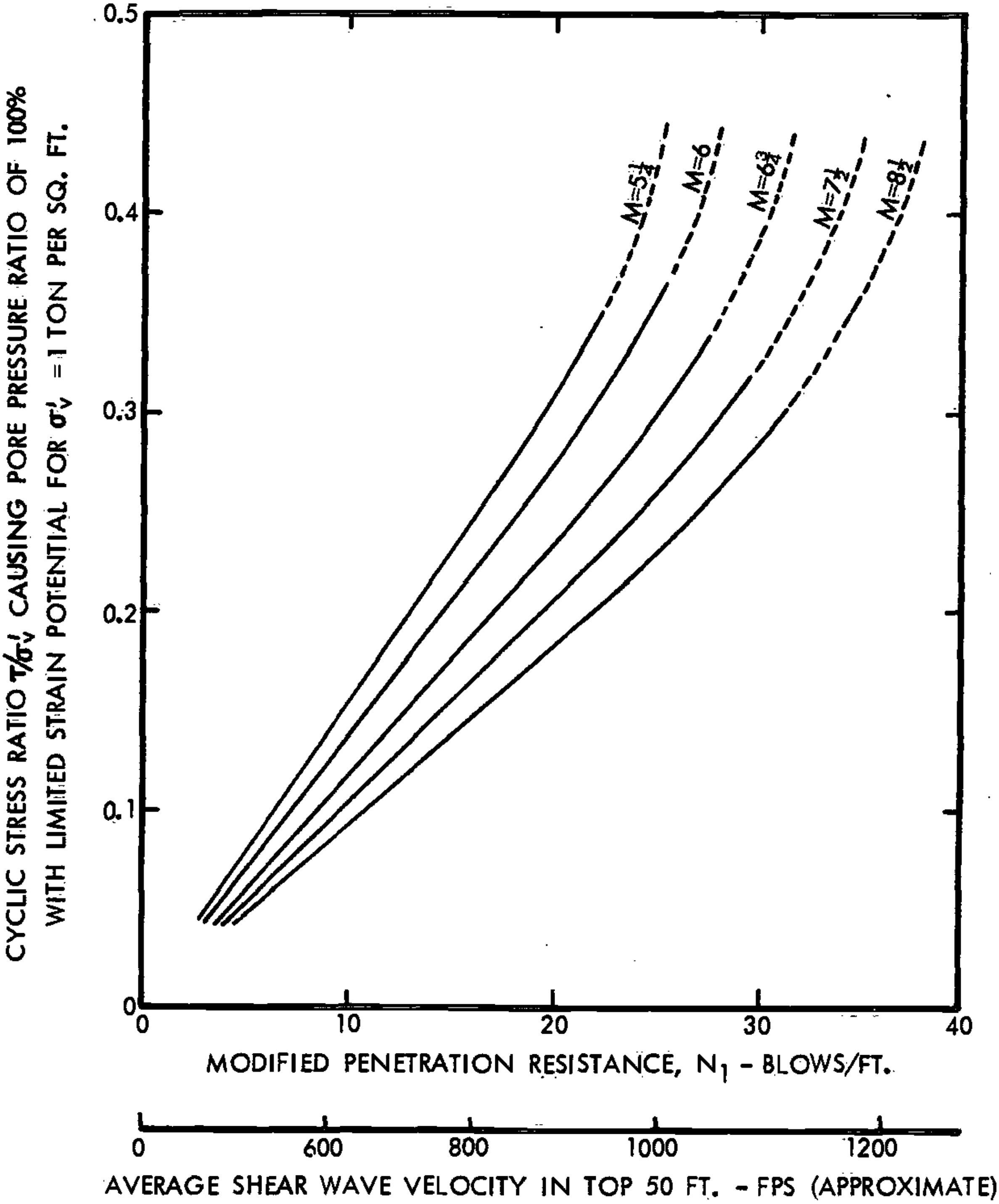
E.4.3 Shear Wave Velocity Measurements

Geophysical 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. One of the downhole surveys was performed at the east end of the proposed 7th/Flower Station site in Boring CEG-9. Shear wave velocities measured in the Young Alluvium (approximately the upper 45 feet of the borehole) were about 1,040$\pm$ 140 fps.

While shear wave velocity in the past has not been as widely accepted as SPT blow count data for estimating the liquefaction potential of a soil deposit, it has received recent attention (Seed et al, 1983). Figure E-7 suggests that liquefaction will never occur during any earthquake if the shear wave velocity in the upper 50 feet of soil exceeds 1,200 fps. Since the shear wave velocities measured close to the 7th/Flower Station site are approximately 1,040 fps, this is an indication that liquefaction at the site would be unlikely.

E.4.4 Gradational Characteristics

Another factor which may be considered in evaluating the liquefaction potential of a soil is the gradation characteristics of the material. A compilation of the ranges of gradational characteristics of soils which have liquefied during past earthquakes and/or are considered most susceptible to liquefaction in the laboratory is shown in Figure E-8. 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 primarily of poorly graded silty sands and sandy silts. It is important to note that all the gradational ranges shown in Figure E-8 have less than 10% by weight clay size particles (i.e., particles less than 0.002 mm) suggesting that clayey (cohesive) soils have a low liquefaction potential. Gradational characteristics typical of gravels and gravelly soils are also absent from Figure E-8 suggesting, in



(after Seed, 1983)

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

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Figure No.
E-7

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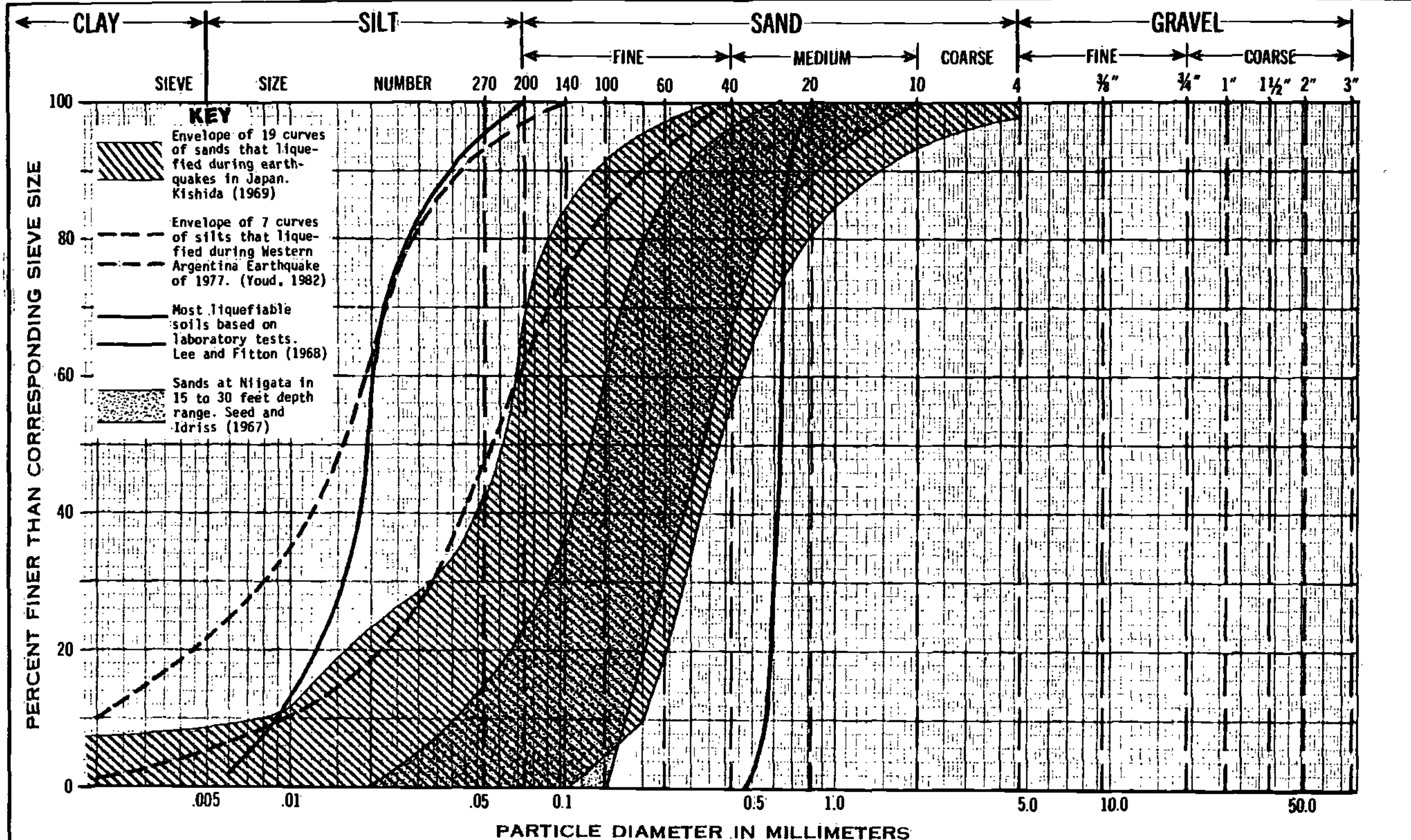

GRADATIONS OF SOILS CONSIDERED SUSCEPTIBLE TO LIQUEFACTION

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Figure No. E - 8

part, that these types of soils may not be capable of developing high excess pore pressures either because they are capable of draining rapidly during the cyclic loading or because these types of materials are usually more efficiently packed (i.e., denser) in situ than soils that consist of uniformly-sized particles. While the liquefaction potential of a soil is dependent on many factors other than gradation (such as the relative density of the soil, the intensity and duration of cyclic loading, among others), comparisons of the gradational characteristics of a soil with those ranges shown in Figure E-8 provide a useful guide in establishing the liquefaction potential of a soil.

The gradational characteristics of two typical soils from the Young Alluvium were determined from laboratory tests performed during this investigation. A comparison of the gradations with the ranges of gradations of "liquefiable" sandy soils shown in Figure E-8 are presented in Figure E-9.

Figure E-9 indicates that the gradation of the silty/clayey sand soil (9-1 at 31'-0" feet) falls within the range of gradations of soils considered "susceptible" to liquefaction. However, it should be noted that the clayey sand soils generally occurred at shallow depths above the water table, and those at or below the water table generally had high SPT blow counts. The gradation of the gravelly sand soil (9-3 at 29'-3" generally falls outside the "susceptible" range. The comparisons shown in Figure E-9 indicate that, on the basis of gradation alone, there appear to be some soils present at the site which may be considered liquefiable if they were below the water level.

E.4.5 Conclusions

Although some silty/clayey sand soils may be considered liquefiable strictly on the basis of gradation, these soils are generally located above ground water levels. Coarse granular soils encountered near or below ground water levels have high SPT blow counts and seismic velocities. Based on the above considerations and comparisons, it is our overall judgement that the Young Alluvium soil deposits found at the 7th/Flower Station site would not be subject to liquefaction during strong ground shaking produced at the site by the postulated earthquake motions.

E.5 LATERAL SURCHARGE PRESSURES ON STATIONS FROM ADJACENT BUILDINGS

E.5.1 General

Unless underpinned, existing buildings in close proximity to the proposed Stations will impose lateral earth pressure loads on both the temporary shoring and the permanent walls. Figures 6-6 and 6-9 presents a simplified method for estimating the imposed lateral pressures from adjacent uniform area loads based on design practices from previous subway station projects. This section discusses the application of this simple method and presents a theoretically more accurate solution which may be appropriate in some situations.

A method for estimating surcharge loads from adjacent line loads is also presented.

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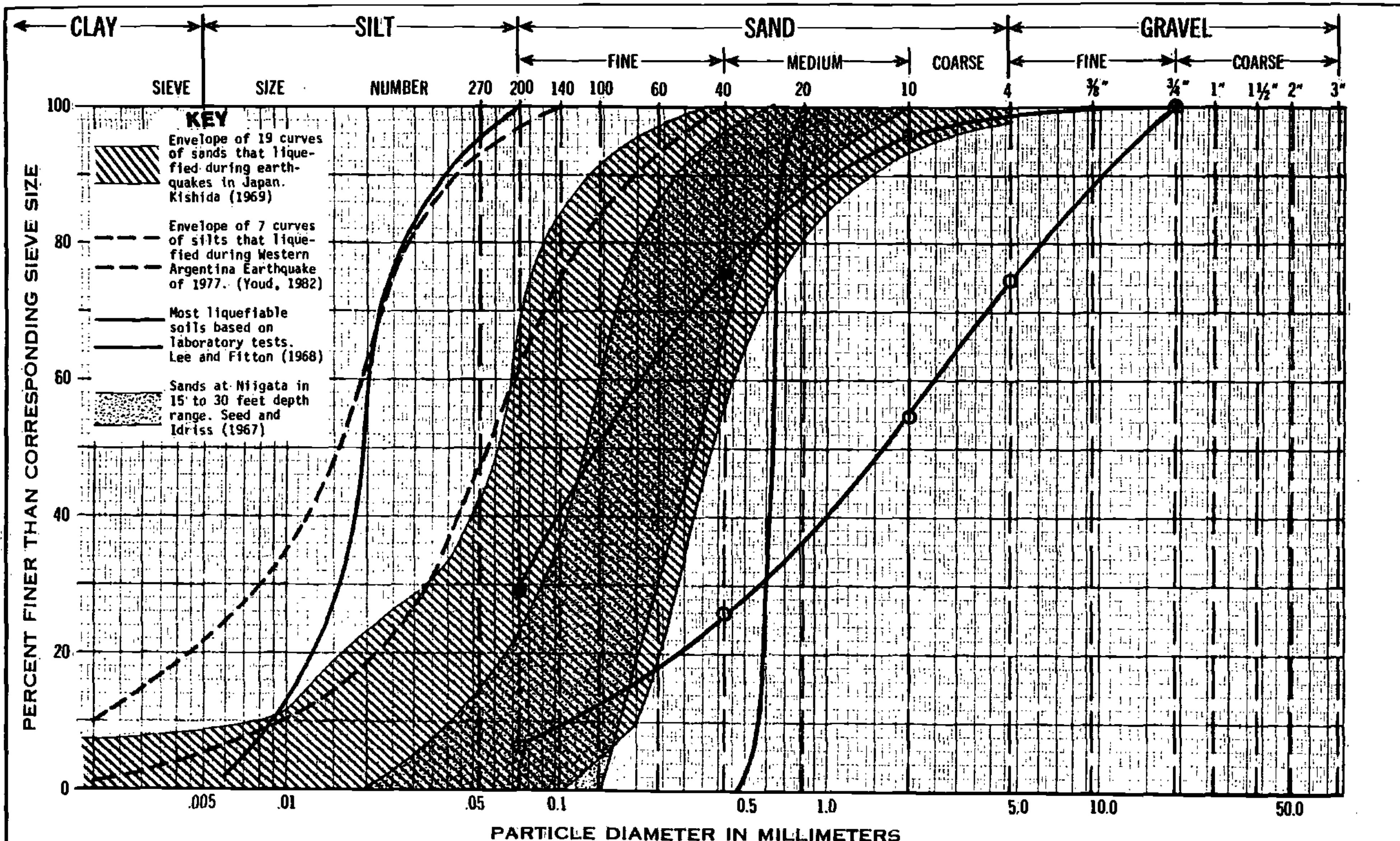

Symbol	Boring	Sample	Depth
O	9-1	J4	31'-0" to 31'-6"
●	9-3	C2	29'-3" to 29'-8"

COMPARISONS OF GRADATIONS

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Figure No. E-9

E.5.2 Uniform Area Loads

The method presented on Figures 6-5 and 6-9 is a function of the building average bearing pressure (q_f), the net building load at the foundation level ($n = q_f - 125 \cdot d$ where d is the foundation depth), and the ratio of the distance from the wall (a) to the excavation depth below the foundation level (h-d). The solution provides a uniform rectangular lateral load distribution W where:

$$W = 0.4n\left[1 - \frac{a}{(h-d)}\right] \text{for } \frac{a}{(h-d)} \text{ from 0 to 1.0}$$

$$\text{for } \frac{a}{(h-d)} > 1 \text{ then } W = 0$$

For a deep basement, the net foundation load (n) may be small or even negative. If negative, the effect of the building should be ignored. We do not recommend applying the surcharge as a negative load to reduce the wall pressures.

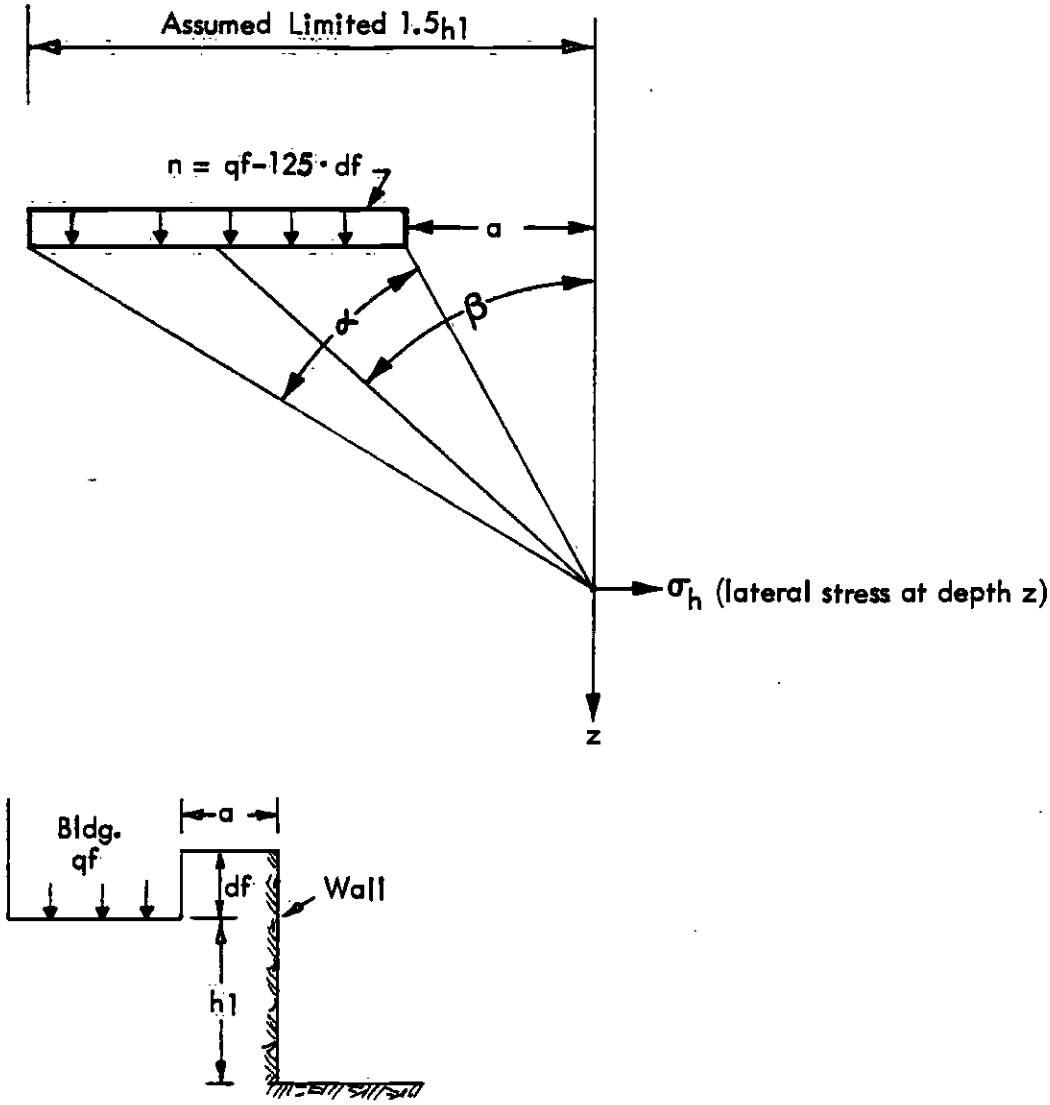
The elastic solution for a strip load equals:

$$\sigma_h = \frac{n}{\Pi}(\alpha - \text{SIN } \alpha \text{ COS } 2\beta)$$

The appropriate parameters are shown on Figure E-10. The solution is affected by loads at considerable distances from the wall. This effect is really only valid for a totally non-yielding wall. In the case of a real shoring wall, some yielding does occur which probably limits the effect of the building loads beyond a certain zone of influence from the wall. In our analysis we have limited the applied building load to a distance away from the wall equal to 1.5 times the depth of the excavation below the foundation level as shown in Figure E-6.

For some conditions, the results of the elastic solution should be doubled to account for the effect of a rigid boundary. In our opinion, since the building loads existed prior to the station walls, the stresses should not be doubled.

Figure E-11 presents the results of the elastic solution. The analysis indicates that the wall pressures are not rectangular but vary with depth. Figure E-12 compares the resultant from the elastic solution with the simplified method presented above and on Figure 6-5. In general, the resultant of the simplified solution compares well with the elastic solution except the cases where the adjacent building is very close to the excavation and at distances beyond about 80% of the excavation depth. The main difference between the two methods is the distribution of pressure. With the building foundation close to the excavation, the simple rectangular solution appears to underestimate the maximum pressure by about 50%.



After NAVFAC DM 7 (1971)

ELASTIC SOLUTION FOR LATERAL PRESSURE FROM UNIFORM BLDG. LOADS

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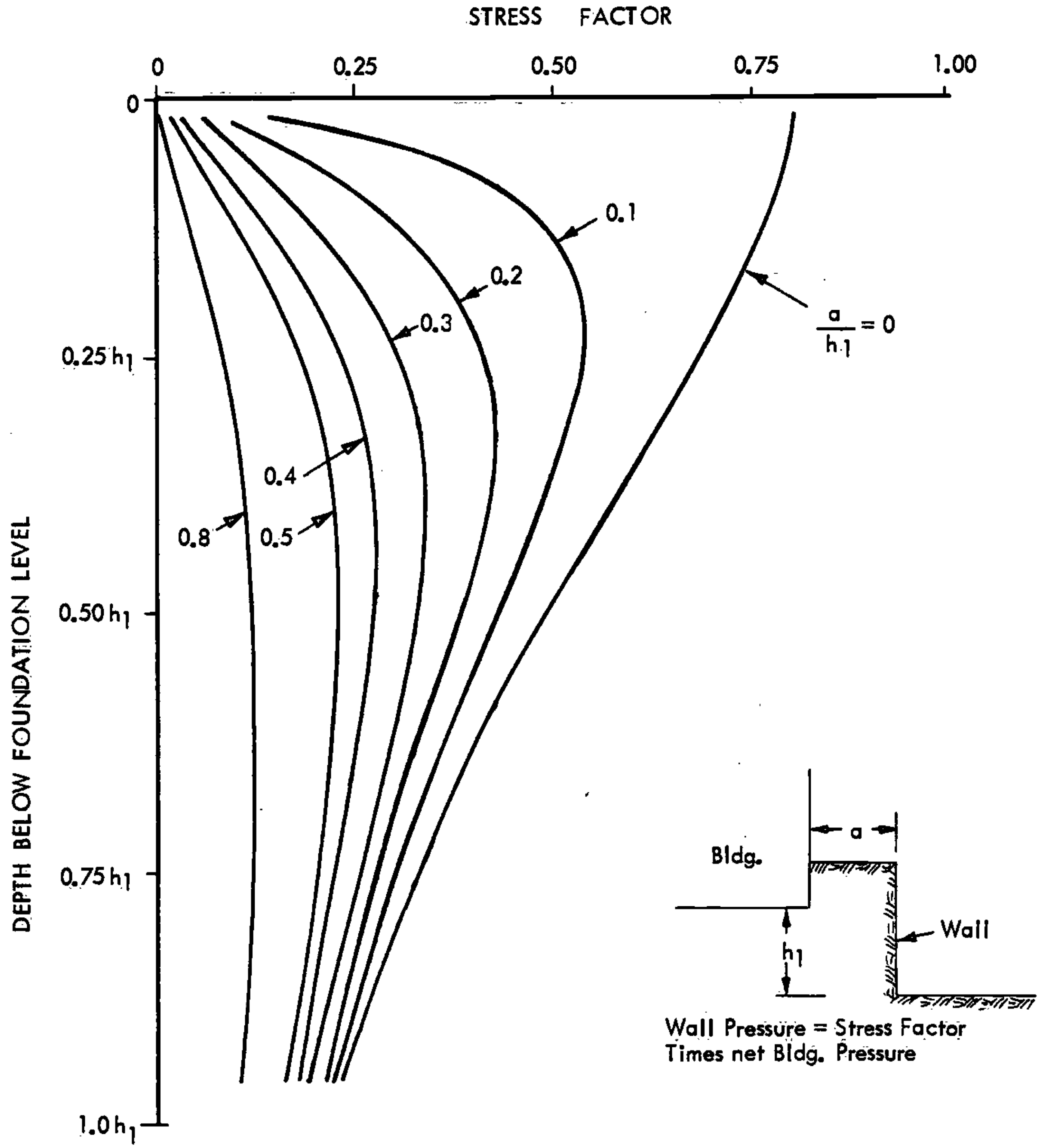
Figure No.
E-10



After NAVFAC DM 7.1 (1982)

LATERAL WALL PRESSURES FROM ADJACENT UNIFORM BUILDING LOAD

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Figure No. E-11

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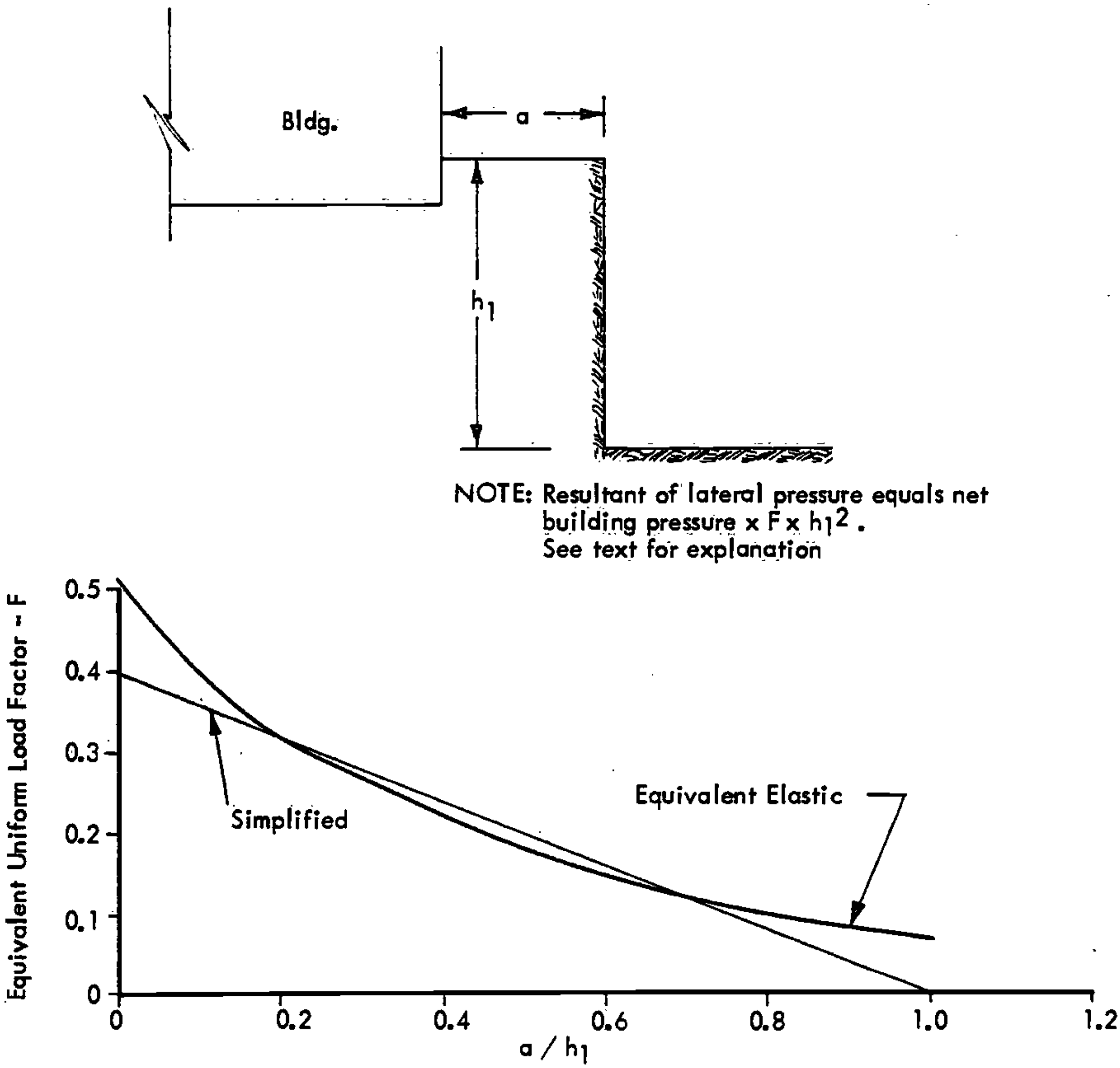

COMPARISON OF SIMPLIFIED & ELASTIC SOLUTION LATERAL SURCHARGE PRESSURE

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Figure No.
E-12

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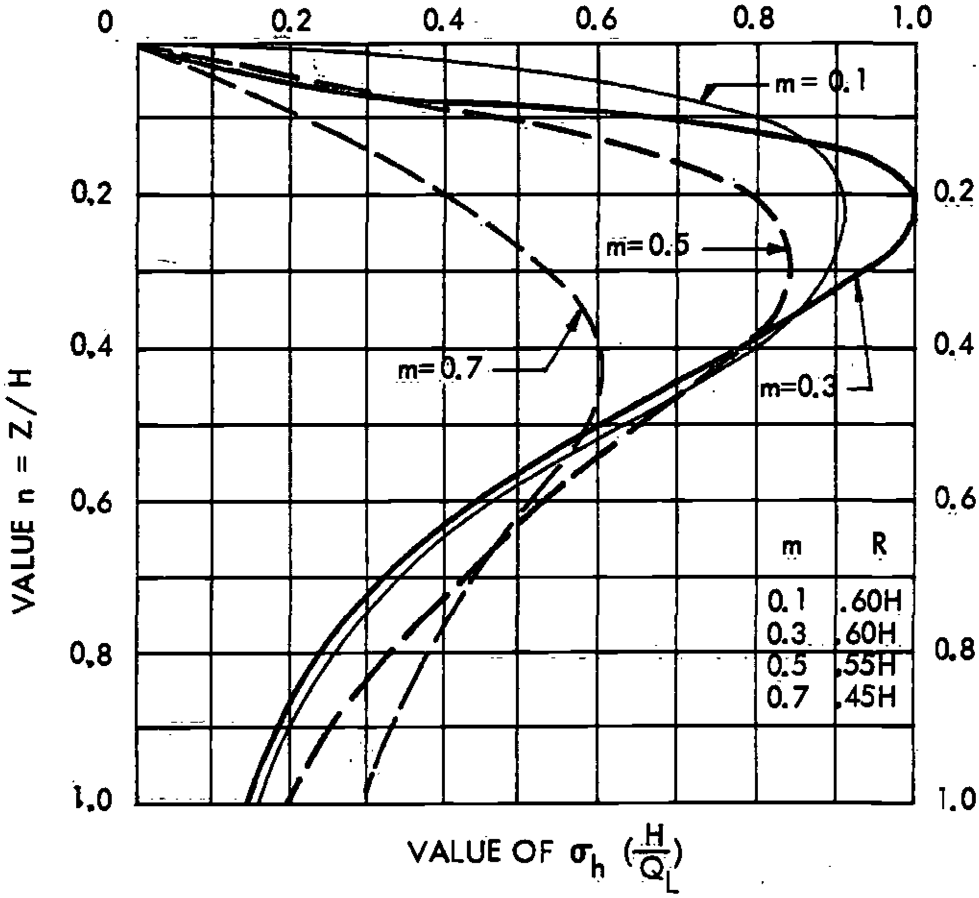
In general, the simplified solution is adequate. However, for major buildings not underpinned and in close proximity to the station, we recommend that the elastic solution be used to ensure that the upper portions of the wall are not underdesigned.

E.5.3 Strip Loads

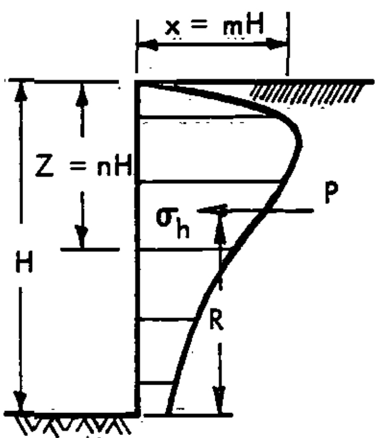
Heavy strip footings not underpinned and adjacent to the station may impose higher lateral stresses than indicated by assuming a simple average area foundation load. Figure E-13 presents a method for estimating lateral pressures from strip loads.

E.5.4 Pressures on Station Roof

Shallow foundation loads in close proximity to the station could impose a vertical surcharge pressure on the station roof. We recommend that a suitable elastic stress method be used to compute imposed vertical roof loads if an imaginary line drawn downward from the base of the building foundation on a 1:1 slope intercepts the station roof. In general this situation will not occur but should be checked.



(a)



RESULTANT $P_h = \frac{0.64\,Q_L}{(m^2 + 1)}$

FOR $m \leq 0.4$:

$$\sigma_h \left(\frac{H}{Q_L}\right) = \frac{0.20\,n}{(0.16 + n^2)^2}$$

$$P_h = 0.55 Q_L$$

FOR $m > 0.4$:

$$\sigma_h \left(\frac{H}{Q_L}\right) = \frac{1.28\,m^2 n}{(m^2 + n^2)^2}$$

(After NAVFAC, 1971)

LATERAL PRESSURES FROM ADJACENT LINE LOAD

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Figure No. E - 13

Appendix F
Earthwork Recommendations

APPENDIX F - EARTHWORK RECOMMENDATIONS

The following guidelines are recommended for earthwork associated with site development.

- Site Preparation (surface structures): Existing vegetation, debris, and soft or loose soils should be stripped from the areas that are to be graded. Soils containing more than 1% by weight of organics may be re-used in planter areas, but should not be used for fill beneath building and paved areas. Organic debris, trash, and rubble should be removed from the site. Subsoil conditions on the site may vary from those encountered in the borings. Therefore, the soils engineer should observe the prepared graded area prior to the placement of fill.

- Minor Construction Excavations: Temporary dry excavations for foundations or utilities may be made vertically to depths up to 5 feet. For deeper dry excavations in existing fill or natural materials up to 15 feet, excavations should be sloped no steeper than 1:1 (horizontal to vertical). Recommendations for major sloped excavations are presented in Section 6.5.

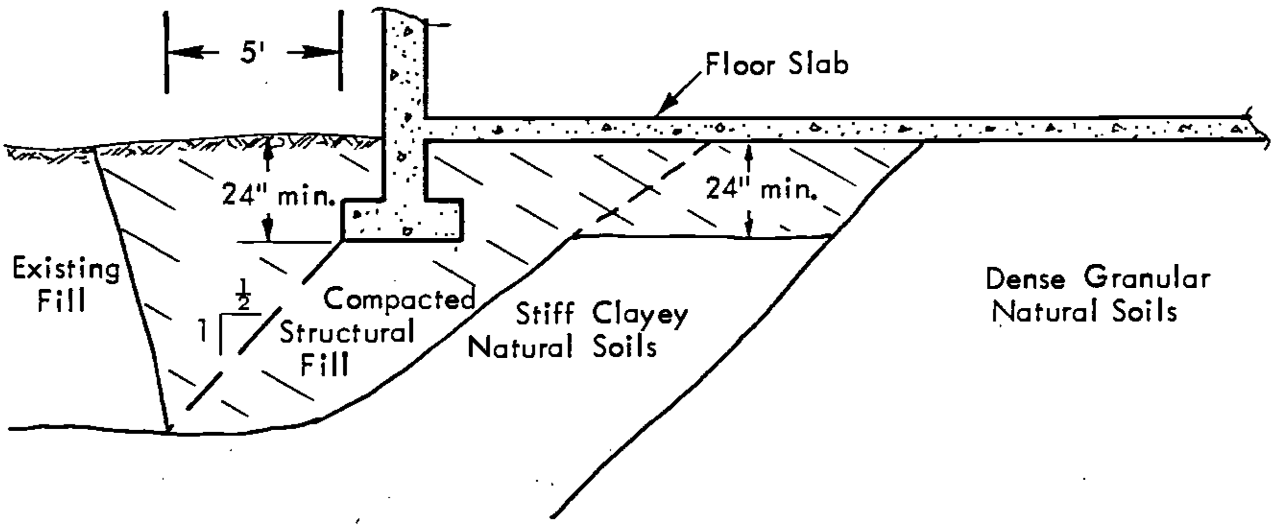
- Structural Fill and Backfill: Where required for support of near surface foundations or where subterranean walls and/or footings require backfilling, excavated onsite granular soils or imported granular soils are suitable for use as structural fill. Loose soil, formwork and debris should be removed prior to backfilling the walls. Onsite soils or imported granular soils should be placed and compacted in accordance with "Recommended Specifications for Fill Compaction". In deep fill areas or fill areas for support of settlement-sensitive structures, compaction requirements could 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., soils containing at least 40 percent passing the No. 200 sieve.

- 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 half the depth of the zone beneath the footing whichever is larger. The structural fill should be placed and compacted as recommended under "Structural Fill and Backfill".

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 structural 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.