GEOTECHNICAL REPORT
FLOWER STREET SUBWAY

PREPARED BY

LeROY CRANDALL AND ASSOCIATES
ADE-85005-8

FOR

SOUTHERN CALIFORNIA RAIL CONSULTANTS
CONTRACT TW 1005

DECEMBER 6, 1985

LeROY CRANDALL AND ASSOCIATES consulting geotechnical engineers, 711 n. alvarado st., los angeles, ca. 90026, (213) 413-3550

December 6, 1985

Southern California Rail Consultants
403 West Eighth Street, Suite 800
Los Angeles, California 90014

Contract No. TW1005
(Our Job No. ADE-85005-8)

Attention: Mr. Simon Zweighaft
Project Director

Gentlemen:

Our "Geotechnical Report, Flower Street Subway, Proposed Long Beach-Los Angeles Rail Transit Project" is herewith submitted.

The scope of the investigation was planned in collaboration with Mr. B. I. Maduke of your staff. The cooperation and guidance provided by Mr. Maduke and others with SCRC are sincerely appreciated.

Respectfully submitted,

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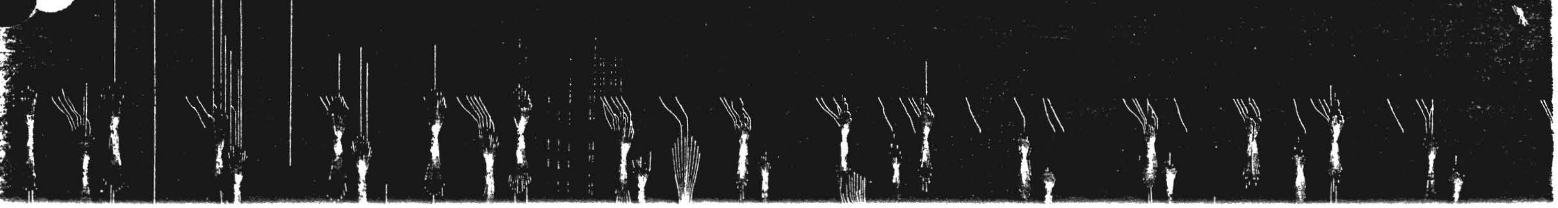


SECTION 1.0:
EXECUTIVE SUMMARY

1.1 INTRODUCTION

This report presents the results of the geotechnical investigation performed for the Flower Street Subway segment of the proposed Long Beach-Los Angeles Rail Transit Project.

The findings and conclusions developed during this geotechnical investigation are presented in this report. The first part of this report contains the text which includes data analyses, interpretative information, conclusions, and recommendations for design. The Appendices that follow the text include information that supports the conclusions and recommendations presented in the main text.

The Flower Street Subway Segment begins at approximate Station 4+55, just south of 6th Street, and extends southerly along Flower Street to just north of 12th Street where the subway will emerge to the surface through a transition U-section at Station 42+30. The subway will have a length of about 3,775 feet.

The proposed subway will extend about 30 to 50 feet below the existing Flower Street grade. Because of the relatively shallow depth, it is expected that cut and cover methods of construction will be used.

One passenger station is planned for this segment. The station will be located beneath the intersection of Flower and 7th Streets. A direct passenger connection is planned at this station with the proposed Southern California Rapid Transit District Metro Rail Project which will run underneath the planned light rail project. The passenger station will necessitate a relocation of the Flower Street sanitary sewer between Wilshire Boulevard and 8th Street.

1.2 DESCRIPTION OF EXPLORATION AND TESTING PROGRAM

Field explorations consisted of subsurface drilling, sampling and testing, piezometer installation, ground water monitoring, and geophysical testing. The laboratory testing program was conducted as the samples were obtained and brought to the laboratory.

A total of 12 borings were drilled for this project to depths of 60 to 80 feet, for a total of 751 lineal feet of drilling. The drilling was performed with rotary wash-type and bucket auger-type drilling equipment. In addition, numerous borings from geotechnical investigations performed for adjacent projects for others have been utilized to complement the geotechnical profile along the subway alignment.

Laboratory tests performed include moisture content and density tests, direct shear tests, consolidation tests, and compaction tests.

1.3 PROJECT GEOLOGY

The proposed subway will pass primarily through Holocene age alluvial soils which are present throughout most of the downtown Los Angeles area. North of 7th Street, the subway will extend into the Pliocene age Fernando Formation, which underlies the alluvium.

The proposed subway is not within a State of California or City of Los Angeles fault study zone. The closest major active or potentially active faults to the subway are the Santa Monica-Hollywood, Raymond, and Newport-Inglewood Faults some 4-1/2, 5-1/2, and 7 miles from the subway, respectively. The San Andreas Fault is some 34 miles from the subway at its closest point.

The probability of liquefaction occurring along the subway segment during a major earthquake is judged to be very low.

1.4 SUBSURFACE CONDITIONS

The soils encountered in the borings consist of existing fill materials and Holocene alluvial deposits. Holocene age alluvial soils were encountered along the entire length of the subway segment. The soils in the borings consist of sand, silty sand, silt, and clay. Varying amounts of gravel and cobbles are present in the sands and silty sands. The uppermost soils are moderately firm to firm but become more firm with increased depth.

Underlying the Holocene alluvial soils is the Pliocene Fernando Formation. The Fernando Formation includes both the Pico and Repetto Formations. This formation consists of siltstone. The surface of the Fernando Formation dips to the south and was not encountered in the borings south of 7th Street. The Fernando Formation siltstone has a consistency of hard to very hard.

1.5 DESIGN CONSIDERATIONS

Design recommendations are presented for foundation support, excavation, shoring, underpinning, dewatering, and backfilling associated with the proposed subway and a related sanitary sewer relocation. The recommendations are based on the results of the field explorations and laboratory tests, the engineering analyses based thereon, and on the geologic and ground motion studies.

The soil and rock materials at the planned foundation level of the subway are good and will offer uniform support of the subway structures.

SECTION 2.0:
INTRODUCTION

2.1 PROJECT LOCATION

This report presents the results of the geotechnical investigation performed along the Flower Street Subway Segment of the proposed Long Beach-Los Angeles Rail Transit Project.

The Long Beach-Los Angeles Rail Transit Project is a conventional light rail system that will extend along a transportation corridor from downtown Long Beach to downtown Los Angeles. The proposed alignment, which is shown on Figure 2-1, System Map, will pass through the cities of Compton and Carson and through the unincorporated areas of Florence-Graham, Willowbrook, and Dominguez Hills in Los Angeles County. The total route will be approximately 22 miles in length, with about 15-1/2 miles of it following the existing Southern Pacific Transportation Company (SPTC) right-of-way (Wilmington and East Long Beach Branches). Much of the project route will essentially be the same as the last line operated by the Pacific Electric Railway's "Red Cars", which ceased operation in 1961. The overall project will be part at grade, part above grade (aerial), and part subway. The location of the Flower Street Subway Segment relative to the downtown Los Angeles alignment is shown on Figure 2-2, Downtown Los Angeles Alignment.

The Flower Street Subway Segment will be under Flower Street in downtown Los Angeles. The northern terminus of the subway segment (and the Long Beach-Los Angeles Rail Transit Project) will be at approximate Station 4+55, which is just south of 6th Street. The subway proceeds southward along Flower Street for a distance of approximately 3,775 feet. There will be a portal structure approximately 100 feet south of 11th Street and the tracks will emerge at street level just north of 12th Street. The alignment and profile are presented on the project drawings, Figures 2-3 through 2-6, Flower Street Subway Segment, Boring Location Plan and Geologic Profile.

2.2 PROJECT DESCRIPTION

The proposed Flower Street Subway Segment will consist of double tracks extending from south of 6th Street to the portal which is north of 12th Street. The planned lower slab of the subway structure will be established at depths of about 30 to 50 feet below the existing Flower Street grade.

Because of the relatively shallow depth of the subway, mining and tunneling techniques will not be too practical. It is anticipated that cut and cover construction methods will be utilized. Excavation for the subway will require installation of a soldier pile shoring system. During construction, decking will be installed at the road surface to allow for flow of traffic to continue.

There will be a passenger station along this segment beneath the intersection of 7th and Flower Streets. A direct passenger connection with the proposed Metro Rail Project Station is planned at this station. The Metro Rail tracks will run underneath and perpendicular to the LRT Alignment at this location.

2.3 PURPOSE OF INVESTIGATION

The purpose of this geotechnical investigation was to evaluate the geotechnical conditions along the proposed alignment with regard to their possible effects on the design and construction of the planned rail transit project.

2.4 SCOPE OF WORK

The scope of work for this investigation included the following:

- Drilling and sampling
- Piezometer installations
- Downhole seismic surveys
- Laboratory testing
- Geologic and seismic studies
- Engineering analyses
- Seismic engineering studies
- Conclusions and recommendations
- Preparation of geotechnical report.

2.5 LIMITATIONS OF INVESTIGATION

Our professional services have been performed using that degree of care and skill ordinarily exercised, under similar circumstances, by reputable geotechnical engineers and geologists practicing in this or similar localities. No other warranty, expressed or implied, is made as to the professional advice included in this report. This report has been prepared for the Southern California Rail Consultants and their design consultants to be used solely in the design of the proposed facilities. The report has not been prepared for use by other parties, and may not contain sufficient information for purposes of other parties or other uses.

2.6 INSPECTION OF BORING SAMPLES

Soil samples recovered from the borings are stored at the laboratory of LeRoy Crandall and Associates, 711 North Alvarado Street, Los Angeles, California 90026.

2.7 PREVIOUS INVESTIGATIONS

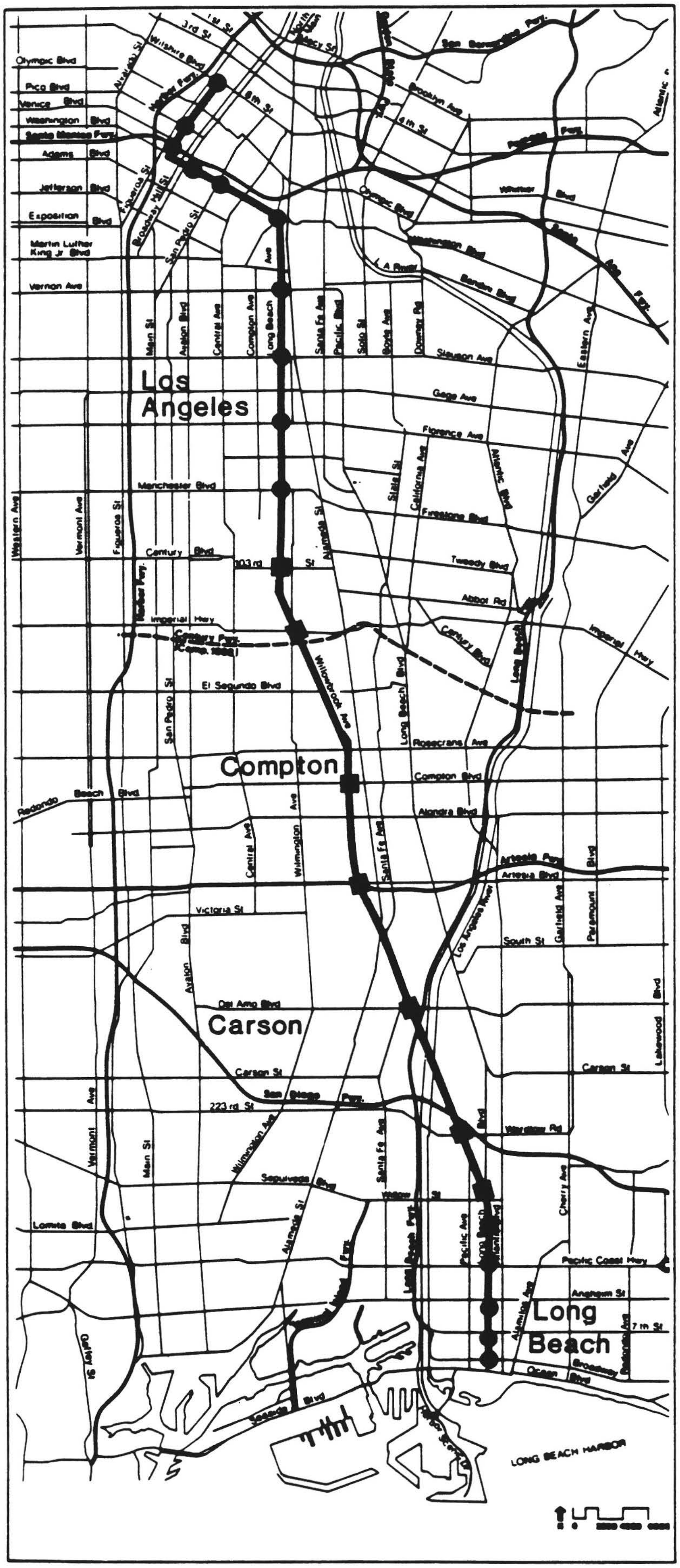
Prior geotechnical investigations performed by our firm for other proposed rail transit projects of which portions extended along alignments similar to that of this project, are covered in the following reports:

- Report of Preliminary Geotechnical Investigation, Proposed Long Beach-Los Angeles Rapid Transit Corridor, for the Southern California Rapid Transit District, dated June 20, 1976.
- Report of Preliminary Foundation Investigation, Proposed Rapid Transit System: Wilshire, San Gabriel Valley, San Fernando Valley, and Long Beach Corridors by the Southern California Rapid Transit District, dated April 26, 1966. (Performed investigation for Joint Venture of Kaiser Engineers and Daniel, Mann, Johnson and Mendenhall.)

Reports covering our prior work for this project are identified as follows:

- Task 1: Library Search, dated March 6, 1985.
- Task 2: Library Search, dated March 8, 1985.
- Task 3: Initial Boring Program, dated March 19, 1985.
- Task 4: Preliminary Budget Estimate and Time Schedule, dated April 18, 1985.
- Task 5: Parameters for Seismic Analysis, Los Angeles River Bridge, dated August 1, 1985.
- Task 6: Geotechnical Report, MC5 SPTC Railroad Relocation, dated November 27, 1985.

- Task 7: Geotechnical Report, LACTC-SPTC Mid Corridor, dated September 23, 1985.
- Task 9: Geotechnical Report, Main Yard and Shops, Aerial Structure, and Los Angeles River Bridge, dated November 20, 1985.
- Task 10: Geotechnical Report, Long Beach Alignment (in progress).
- Task 11: Geotechnical Report, Washington Boulevard Alignment (in progress).
- Preliminary Environmental Risk Assessment and Site Safety Plan prepared by MED-TOX Associates, Inc., dated September 8, 1985.



LEGEND

Station ●

Station with Available Parking ■

Figure 2-1

System Map

Long Beach-Los Angeles
RAIL TRANSIT PROJECT
LOS ANGELES COUNTY TRANSPORTATION COMMISSION

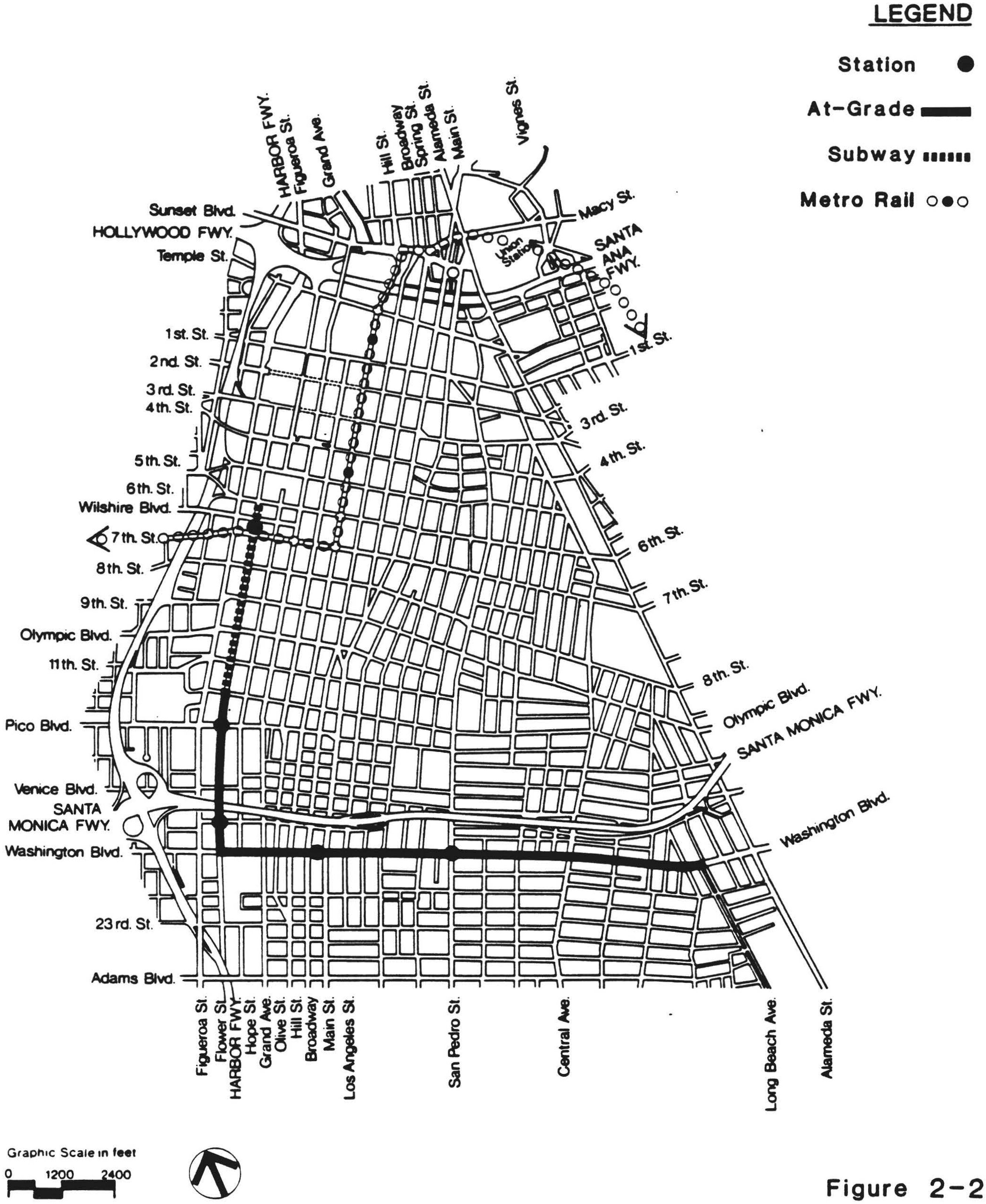
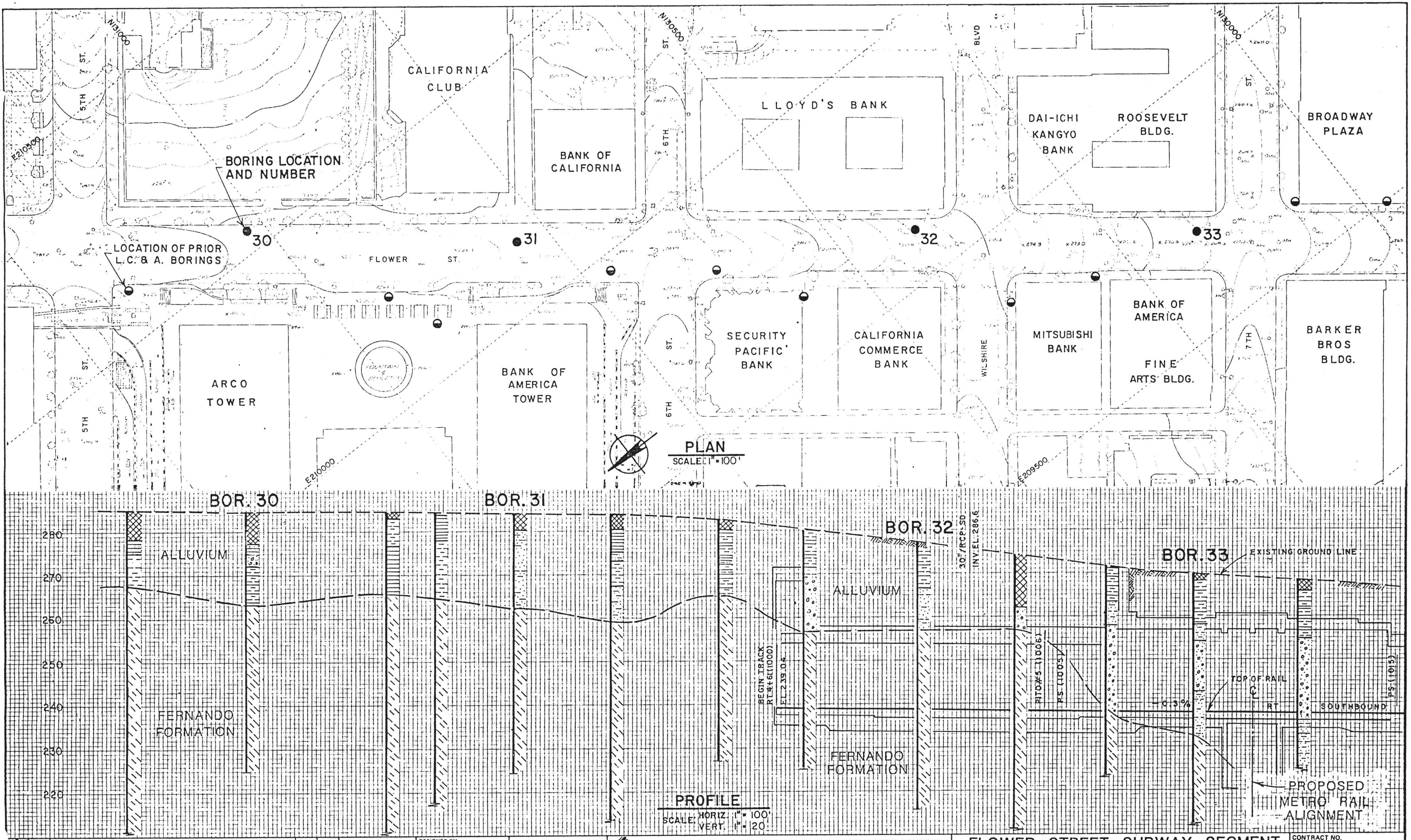


Figure 2-2

Long Beach - Los Angeles
RAIL TRANSIT PROJECT
LOS ANGELES COUNTY TRANSPORTATION COMMISSION

Downtown Los Angeles
Alignment



NOTE: SEE FIGURE 2-7 FOR KEY TO GEOLOGIC SYMBOLS

REV.	DATE	DESCRIPTION	BY SUB	APP.
1	10/8	BORING LOCATIONS AND GEOLOGIC PROFILE ADDED		

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The Long Beach-Los Angeles Rail Transit Project

LeROY CRANDALL AND ASSOCIATES
CONSULTING GEOTECHNICAL ENGINEERS

SUBMITTED: ______________ DATE ______

Southern California Rail Consultants
A Joint Venture of
- Parsons Brinckerhoff Quade & Douglas, Inc
- Kaiser Engineers (California) Corporation
- Daniel, Mann, Johnson & Mendenhall

APPROVED: ______________

FLOWER STREET SUBWAY SEGMENT
BORING LOCATION PLAN AND GEOLOGIC PROFILE
5 TH STREET TO 7 TH STREET

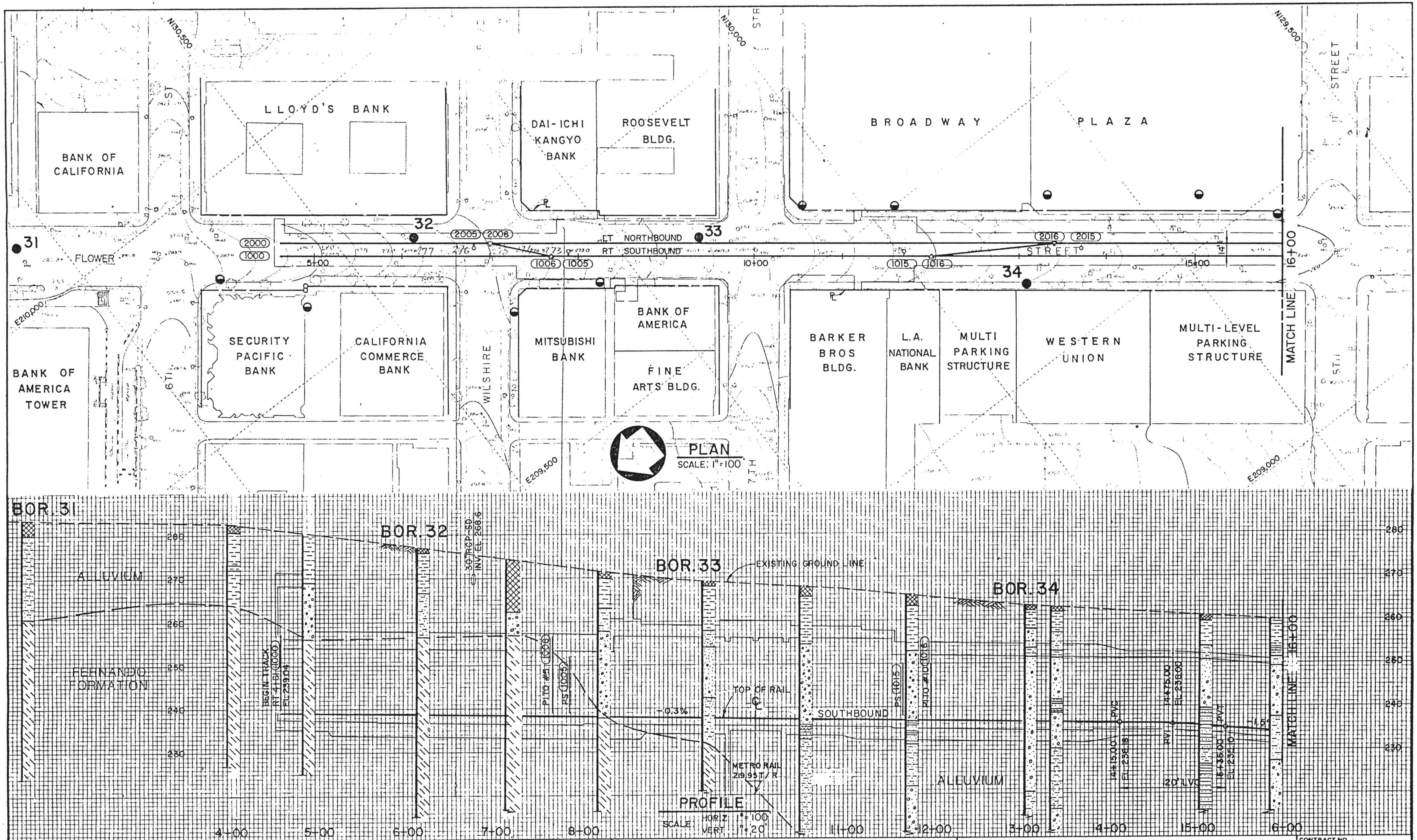
CONTRACT NO.

FIGURE 2-3

REV.

SHEET NO.

SCALE AS SHOWN



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1	10/8	BORING LOCATIONS AND GEOLOGIC PROFILE ADDED	JUB	

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FLOWER STREET SUBWAY SEGMENT
BORING LOCATION PLAN AND GEOLOGIC PROFILE
6 TH STREET TO 8 TH STREET

CONTRACT NO.
FIGURE 2-4
REV.
SHEET NO.
SCALE
AS SHOWN

O.E. MS CHKD. MA

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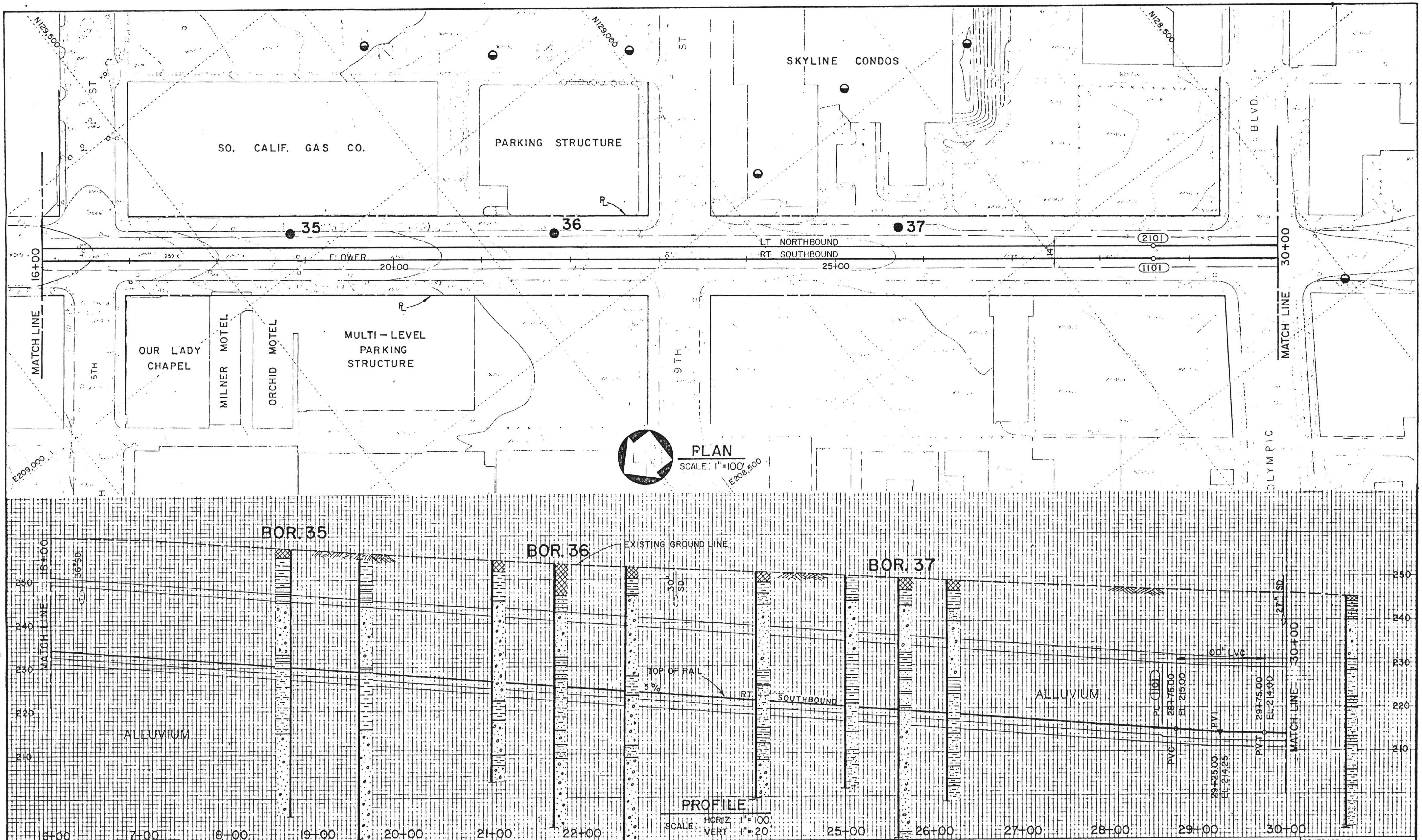

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The Long Beach-Los Angeles Rail Transit Project

LeROY CRANDALL AND ASSOCIATES
CONSULTING GEOTECHNICAL ENGINEERS

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- Daniel, Mann, Johnson, & Mendenhall

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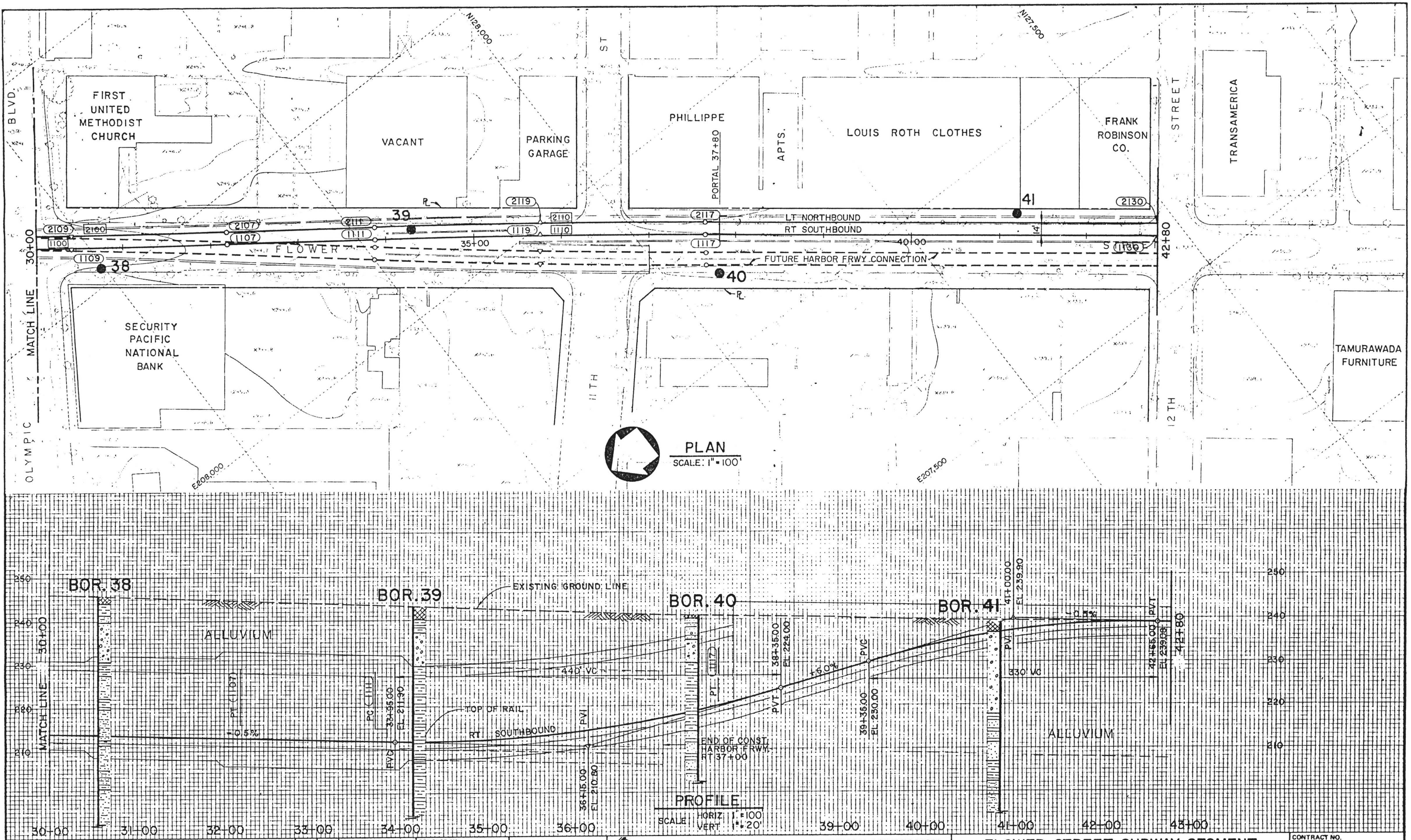
FLOWER STREET SUBWAY SEGMENT

BORING LOCATION PLAN AND GEOLOGIC PROFILE

8 TH STREET TO OLYMPIC BOULEVARD

CONTRACT NO.
FIGURE 2 - 5
REV.
SHEET NO.
SCALE AS SHOWN



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1	10/8	BORING LOCATIONS AND GEOLOGIC PROFILE ADDED		

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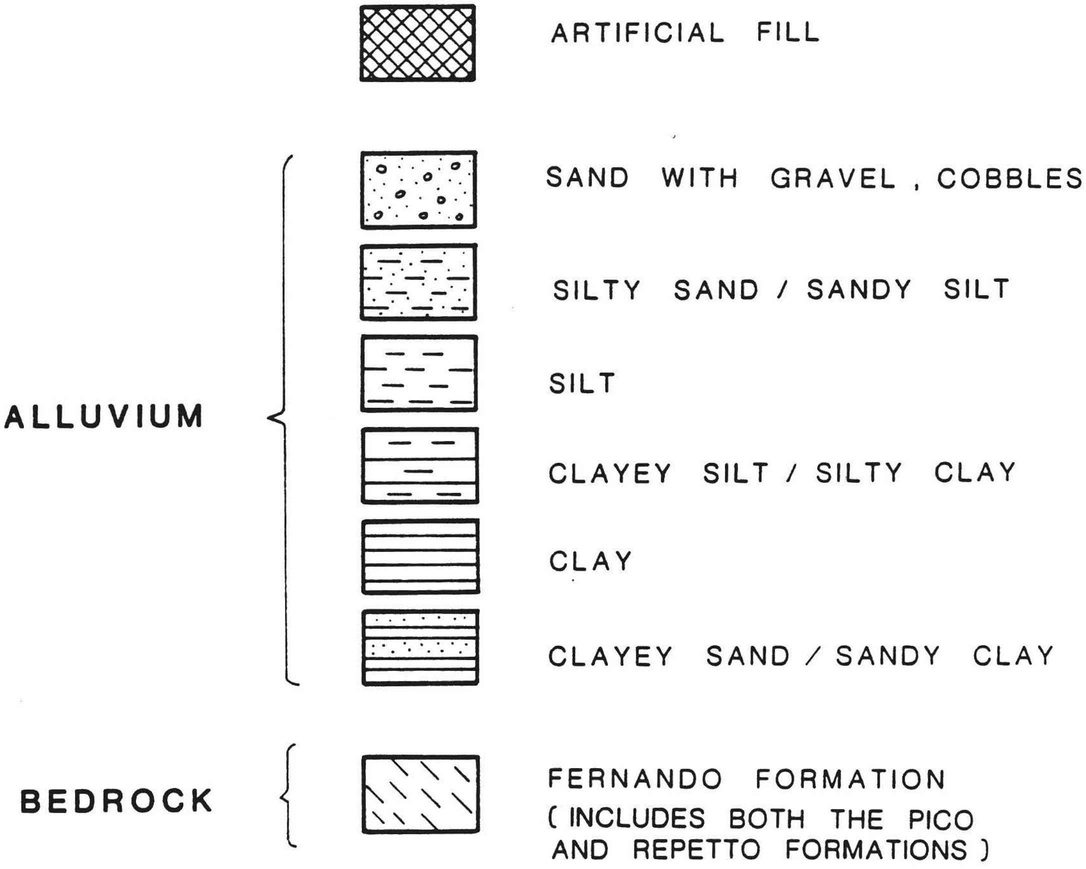
FLOWER STREET SUBWAY SEGMENT
BORING LOCATION PLAN AND GEOLOGIC PROFILE
OLYMPIC BOULEVARD TO 12 TH STREET

CONTRACT NO.
FIGURE 2-6
REV.
SHEET NO.
SCALE AS SHOWN

JOB ADE 85005-8 DATE 10/7/85 DR M.G. W.P. CHKD

FOR

EXPLANATION



KEY TO GEOLOGIC SYMBOLS

LeROY CRANDALL AND ASSOCIATES

FIGURE 2-7

Section 4

Soil Classification and Laboratory Testing

SECTION 3.0:
FIELD EXPLORATION AND LABORATORY TESTING

3.1 FIELD EXPLORATION PROGRAM

The field exploration program was performed in accordance with the scope of work described in our proposal dated June 10, 1985, and which was specified by the SCRC memorandum dated June 5, 1985, the attachments and topographic sheets included therewith.

A detailed description of the drilling exploration program, boring logs, piezometer installations, ground water level monitoring, and downhole seismic surveys is presented in Appendices A through C at the end of this report.

3.1.1 Borings

A total of 12 borings were drilled at the locations shown on Figures 2-3 through 2-6, Boring Location Plan. The locations and depths of borings were initially recommended by SCRC and were modified as necessary to avoid underground utilities.

The logs of the borings are presented in Appendix A. Borings were drilled to depths of 60 to 80 feet, for a total of 751 lineal feet of drilling. The drilling was performed between August 14 and August 24, 1985. A summary of the boring locations and depths is presented in Table 3-1.

Undisturbed samples were obtained with the Crandall sampler at depth intervals of about five feet and at major changes in soil stratigraphy. Pitcher samples were taken in three borings. Detailed description of the field exploration procedures are presented in Appendix A.

3.1.2 Drilling Contractors and Equipment

The drilling was performed with rotary wash-type and bucket auger-type drilling equipment. The rotary wash borings were drilled by Pitcher Drilling Company who utilized a Failing 750 drilling rig operated by a two-man crew. The auger borings were drilled by the C&L Drilling Company, who utilized a bucket-type rig operated by a two-man crew.

Table 3-1
Summary of Borings

Boring Number	Boring Location (Station)	Boring Depth (Ft.)	Drilling Type	Remarks
30	--	60	Rotary Wash	Between 5th St. and 6th St.
31	1+60	60	Bucket	Between 5th St. and 6th St.
32	6+15	61	Bucket	Between 6th St. and Wilshire Blvd.
33	9+40	60	Bucket	Between Wilshire Blvd. and 7th St.
34	13+08	80	Rotary Wash	Between 7th St. and 8th St.
35	18+70	60	Bucket	Between 8th St. and 9th St.
36	21+70	60	Bucket	Between 8th St. and 9th St.
37	25+60	60	Rotary Wash	Between 9th St. and Olympic Blvd.
38	30+55	60	Bucket	Between Olympic Blvd. and 11th St.
39	34+15	70	Rotary Wash	Between Olympic Blvd. and 11th St.
40	37+70	60	Bucket	Between 11th St. and 12th St.
41	41+00	60	Bucket	Between 11th St. and 12th St.

3.1.3 Soil Classification

The soils were classified using the Unified Soil Classification System. The field soil classifications were verified by visual inspection in the laboratory by staff engineers and further verified (as necessary) by laboratory tests.

3.2 PIEZOMETER INSTALLATION

After the completion of drilling Borings 30 and 37, a 2-inch-diameter PVC pipe was installed in each boring for future monitoring of the ground water level. Detailed descriptions of the piezometer installations and observed water levels are presented in Appendix A.

3.3 DOWNHOLE SEISMIC SURVEYS

Downhole seismic surveys were performed at two locations (Borings 30 and 37) to determine the propagation velocities of the compression waves (P waves) and shear waves (S waves). The test procedures and results are presented in Appendix B.

3.4 LABORATORY TESTING PROGRAM

Laboratory tests were performed on selected undisturbed samples and bulk samples. The tests included moisture-density determinations, direct shear tests, consolidation tests. The test procedures and results are presented in Appendix C.

Standard 4.0

Geology

SECTION 4.0:
GEOLOGY

4.1 GENERAL

The proposed Flower Street Subway is located on an alluviated lowland, sometimes referred to as the Coastal Plain (Mendenhall, 1905), within the northwestern part of the Los Angeles basin. The northwestern Los Angeles basin is bounded on the north by the Elysian and Repetto Hills. Geologically, this portion of the basin is situated on the southerly flank of the Elysian Park anticline (Soper and Grant, 1932), the principal structural feature in the area.

The area is underlain by Miocene and Pliocene age sedimentary rocks, exposed within the Elysian and Repetto Hills, which are partly overlain and obscured by non-marine terrace materials and Holocene alluvium within the downtown Los Angeles area. Underlying the Miocene and Pliocene section are pre-Upper Cretaceous basement rocks consisting predominantly of the Santa Monica Slate of Jurassic age (Lamar, 1970).

The relationship of the proposed subway alignment to regional geologic features is shown on Figure 4-1, Regional Geology. The areal geology within the downtown Los Angeles area is depicted on Figure 4-2, Local Geology. The subway is shown in relation to major fault zones and earthquake epicenters on Figure 4-3, Regional Seismicity.

4.2 GEOLOGIC MATERIALS

The uppermost natural soils along the subway alignment are typical of Holocene alluvial deposits that extend throughout most of the downtown Los Angeles area. The Holocene alluvium, within the area of the proposed subway, is underlain by sedimentary units deposited during the Pliocene and Miocene epochs of the Tertiary period. These Tertiary age rocks extend to depths of approximately 10,800 feet in the area and are underlain by rocks of the basement complex (Jurassic age Santa Monica Slate).

A thin cover of artificial fill materials, overlying the alluvium, is present beneath the asphalt paving along some of the subway alignment.

A geologic section along the subway alignment is shown on Figures 2-3 through 2-6; a key to the geologic symbols is shown on Figure 2-7.

4.2.1 Artificial Fill

Fill materials were encountered in five of the exploratory borings excavated for this investigation. As observed, the fill typically consist of dark brown to dark grey clayey sand/sandy clay to sandy silt and clayey silt. Varying amounts of gravel and cobbles are scattered throughout the fill.

The maximum depth of fill encountered in our explorations was 5 feet in Boring 36.

4.2.2 Holocene Alluvium

Holocene alluvial deposits are present along the entire length of the alignment. As noted in the exploratory borings, the alluvium is composed of sand, silty sand, silt, and clay. Varying amounts of gravel and cobbles were encountered in the sands and silty sands; few boulders were also encountered. The relative density and consistency of the uppermost natural alluvial soils varied from compact to firm. The deeper alluvial soils were found to be dense to stiff. The base of the alluvium was encountered at depths as shallow as 20-1/2 feet in borings drilled north of 7th Street. South of 7th Street, the base of the alluvium was not penetrated in the remaining borings.

4.2.3 Pliocene Series

4.2.3.1 Fernando Formation

The Fernando Formation underlies the alluvial materials in the area and was encountered in Borings 30 through 33. This formation is composed of massive marine siltstone units. The Fernando Formation siltstones were hard to very hard. The siltstone varies in color from light brown and brown-grey in the oxidized zone to dark grey in the unoxidized section. Layers of shell fragments occur sporadically within the Fernando Formation.

The Fernando Formation conformably overlies the Puente Formation at depth. Lamar (1970) estimates that the Fernando Formation attains thickness of approximately 700 feet in the downtown Los Angeles area.

4.2.4 Miocene Series

4.2.4.1 Puente Formation

The Puente Formation of the late Miocene epoch underlies the Fernando Formation and is exposed in the hills north of the intersection of Flower Street and Third Street (see Figure 4-2). The Puente Formation is composed of interbedded sandstone, siltstone, and shale. Data from the Union Station Oil Field, located approximately one mile east of the alignment, indicates a maximum thickness of approximately 9,000 feet for the Puente Formation.

4.2.4.2 Topanga Formation

The middle Miocene age Topanga Formation is believed to underlie the Puente Formation beneath the alignment, based upon oil well data in the area. This formation is composed of interbedded marine siltstone, sandstone, and shale and is in unconformable contact with the overlying Puente Formation (Lamar, 1970). It attains an estimated thickness of approximately 1,000 feet in the area.

4.2.5 Basement Rocks

4.2.5.1 Santa Monica Slate

The basement rock underlying the Pliocene and Miocene formations beneath the proposed Flower Station Subway is composed predominantly of metamorphosed slate, phyllite, and schist of the Jurassic age Santa Monica Slate.

4.3 GROUND WATER

The proposed subway alignment is located within the Central Hydrologic Subarea of Los Angeles. The Holocene alluvial deposits beneath the site are generally considered capable of bearing water. The Tertiary Formations are generally considered non-water bearing, although seepage may occur in minor amounts from fractures in the bedrock.

Historic high ground water elevations measured in 1904 near the southerly end of the project indicate that the ground water surface was about Elevation 170 feet, or about 60 feet below ground surface. No early data are available for wells located in the vicinity of the northern portion of the line.

The 1976 ground water contours prepared for the Coastal Plain by the Los Angeles County Flood Control District indicate that the ground water surface slopes to the south-southeast beneath the proposed alignment with ground water elevations typically 40 feet below mean sea level, or approximately 300 feet below ground surface. Light water seepage encountered in previous borings drilled in areas adjacent to the proposed subway and in the downtown Los Angeles area varied from 5 feet to 72 feet below ground surface. In particular, Boring 30, near 5th Street, encountered a water level at 29-1/2 feet on September 6, 1985. The water level in this boring, as well as seepage described for previous borings in the area, generally reflect perched water conditions in the underlying alluvium and water occurring along fractures in the bedrock.

4.4 GEOLOGIC HAZARDS

4.4.1 General

The geologic hazards along the Subway Segment are essentially limited to those caused by earthquakes. The major cause of damage from earthquakes is the result of shaking from earthquake waves. Damage due to actual displacement or fault movement beneath a structure is much less frequent.

4.4.2 Faults

The numerous faults in Southern California include active, potentially active, and inactive faults. The criteria for these major groups, as established by the Association of Engineering Geologists (1973), are presented in Table 4-1. Table 4-2 presents a listing of active faults in Southern California with the anticipated magnitude of a maximum credible earthquake of each fault. Table 4-3 provides a similar listing for potentially active faults.

4.4.2.1 San Andreas Fault

The active San Andreas Fault Zone is California's most prominent structural feature, trending in a generally northwest direction for almost the entire length of the state. In Southern California the San Andreas Fault Zone extends from the Mexican border to the Transverse Mountain Ranges west of Tejon Pass for a length of approximately 200+ miles. Along this segment of the fault zone there is no single traceable fault line; rather, the fault is composed of several branches.

TABLE 4-1

CRITERIA FOR CLASSIFICATION OF FAULTS WITH REGARD TO SEISMIC ACTIVITY

(From Association of Engineering Geologists,
Geology and Earthquake Hazards, 1973)

A. Active Faults: (See Table 4-2)

These faults are those which have shown historical activity. This category includes such faults as the San Andreas, San Jacinto, and Newport-Inglewood.

B. Potentially Active Faults: (See Table 4-3)

These faults are those, based on available data, along which no known historical ground surface ruptures or earthquakes have occurred. These faults, however, show strong indications of geologically recent activity. Potentially active faults can be placed in two subgroups that are based on the boldness or sharpness of their topographic features and the estimates related to recency of activity. These subgroups are:

1. Subgroup One - High Potential
    a. Offsets affecting the Holocene deposits (age less than 10 - 11,000 years).
    b. A ground water barrier or anomaly occurring along the fault within the Holocene deposits.
    c. Earthquake epicenters (generally from small earthquakes occurring close to the fault).
    d. Strong geomorphic expression of fault origin features (e.g. faceted spurs, offset ridges or stream valleys or similar features, especially where Holocene topography appears to have been modified).

2. Subgroup Two - Low Potential

    This subgroup is the same as 1-a, b, or d above, with the exception that the indications of fault movement can be only determined in Pleistocene deposits (less than 1,000,000 years ago).

C. Inactive Faults:

These faults are without recognized Holocene or Pleistocene offset or activity.

TABLE 4-2

MAJOR NAMED FAULTS CONSIDERED TO BE ACTIVE (a) IN SOUTHERN CALIFORNIA

Fault	Date of Latest Major Activity	Maximum Credible Earthquake	Known Fault Length (e) (Miles)
Big Pine	1852	7.5 (b)	47
Coyote Creek	1968	7.2 (c)	50
Elsinore	1910	7.5 (b)	120
Garlock	(d)	7.75(b)	170
Malibu Coast	1973	7.0 (c)	30
Manix	1947	6.25(b)	15
More Ranch	(d)	7.5 (b)	34
Newport-Inglewood	1933	6.5 (f)	25
San Andreas Zone	1857	8.25(b)	200+
San Fernando Zone	1971	6.5 (b)	8
San Jacinto Zone	1968	7.5 (b)	112
Superstition Hills	1951	7.0 (b)	22
White Wolf	1952	7.75(b)	60
Whittier	1929 (?)	7.1 (c)	30

(a) Historic movement (1769 - present).
(b) Greensfelder, C.D.M.G. Map Sheet 23, 1974.
(c) Mark (1977) Length-Magnitude relationship.
(d) Intermittent creep.
(e) Based on Division of Mines & Geology, Preliminary Report 13, 1973.
(f) Raymond Kaiser Engineers, Inc., Report of Subtask 9.6, March 1985.

TABLE 4-3

MAJOR NAMED FAULTS CONSIDERED TO BE POTENTIALLY ACTIVE (a) IN SOUTHERN CALIFORNIA

Fault	Maximum Credible Earthquake	Fault Length (d) (Miles)
Calico-Newberry	7.25 (b)	60
Charnock	6.6 (c)	13
*Chino	6.7 (c)	18
Cucamonga	6.5 (b)	20
*Duarte	6.3 (c)	10
Helendale	7.5 (b)	60
Northridge Hills	6.5 (b)	12
Norwalk	6.4 (c)	20
Oakridge	7.5 (b)	35
*Overland	6.2 (c)	6
Ozena	7.3 (c)	
Palos Verdes	7.0 (b)	30
Pinto Mountain	7.5 (b)	42
Raymond	6.6 (c)	15
San Cayetano	6.75 (c)	
*San Gabriel	7.5 (c)	80
*San Jose	6.5 (c)	17
Santa Cruz Island	7.2 (c)	50
Santa Monica-Hollywood	6.8 (c)	17
Santa Susana	6.5 (b)	10
Santa Ynez	7.5 (b)	100
Sierra Madre	7.5 (b)	55
Sierra Nevada	8.25 (b)	118
*Verdugo	6.8 (c)	12

(a) Pleistocene deposits disrupted.
(b) Greensfelder, C.D.M.G. Map Sheet 23, 1974.
(c) Mark (1977) Length-Magnitude relationship.
(d) Based on Division of Mines & Geology, Preliminary Report 13, 1973.
* Low Potential per A.E.G. definition.

This fault zone is approximately 34 miles north-northeast of the subway alignment, at the nearest point on the fault, and is considered capable of producing an earthquake of magnitude 8 or greater in the Southern California Region.

4.4.2.2 Newport-Inglewood System

The Newport-Inglewood Fault Zone trends northwesterly from Newport Mesa to the Cheviot Hills along the western side of the Los Angeles Basin (Barrows, 1974). This zone is marked by a line of geomorphically young domal hills and mesas formed by the folding and faulting of a thick sequence of sedimentary rocks. The Newport-Inglewood Fault Zone has displayed continuing unrest in numerous small tremors both before and since the magnitude M = 6.3 earthquake in Long Beach on March 10, 1933.

Of the many faults comprising this active zone, the closest to the proposed subway is the Inglewood Fault situated approximately 7 miles to the west-southwest. A maximum credible earthquake of magnitude 6.5 is assigned to the Newport-Inglewood Fault Zone.

4.4.2.3 Santa Monica-Hollywood System

The Santa Monica-Hollywood Fault Zone is the closest potentially active fault to the subway alignment. This fault trends east-west and is situated approximately 4-1/2 miles to the north. All evidence to date indicates that this fault zone has not undergone movement within the Holocene epoch (last 11,000 years). Some geologist believe the Santa Monica-Hollywood Fault Zone is structurally aligned with, and may be contiguous with, the Raymond, Benedict Canyon, and Malibu Coast Faults of similar age, trend, and displacement.

4.4.2.4 Raymond Fault

The Raymond Fault has a known length of 15 miles and traverses from Monrovia Canyon on the east to Arroyo Seco on the west. The fault is a high angle dip-slip reverse fault, with 300 feet of known vertical displacement, juxtaposing Pleistocene deposits north of the fault against Holocene alluvium south of the fault. This fault is approximately 5-1/2 miles north of the subway alignment.

4.4.2.5 Other Active and Potentially Active Faults

Other nearby active faults include the Whittier Fault, located approximately 13 miles east-southeast of the site, the Elsinore Fault, situated 35 miles to the east-southeast, and the San Jacinto Fault Zone located 39 miles to the east-northeast at the closest point.

Other potentially active faults in close proximity to the subway alignment include the Verdugo Fault located approximately 8 miles to the north, the Overland Fault situated approximately 9 miles to the southwest, and the Charnock Fault located 10 miles to the west-southwest.

The proposed subway alignment is not within an established Alquist-Priolo Special Studies Zone nor within a City of Los Angeles Fault Rupture Study Area. The possibility of fault rupture occurring along the alignment is considered remote.

4.4.3 Seismicity

The epicenters of the major recorded earthquakes in Southern California are shown on Figure 4-3. The epicenter of the March 11, 1933 Long Beach earthquake, Richter magnitude 6.3, was located approximately 3-1/2 miles southwest of Newport Beach or about 34 miles southeast of the south end of the subway alignment. This earthquake, although of only moderate magnitude, ranks as one of the major disasters in Southern California. There were 120 fatalites, and damage exceeded $50 million (Iacopi, 1971). The greatest damage was in the coastal cities, particularly Long Beach where many unsuitable buildings had been constructed on artificial fill or saturated alluvium. The majority of the damage was suffered by structures which are now considered substandard construction and/or were located on filled or saturated ground.

The recurrence curve shown on Figure 4-4 indicates the seismicity of the Flower Street Subway area. The recurrence curve was developed based on the seismicity of the area which was determined from a computer search of a magnetic tape catalog of earthquakes. The catalog of earthquakes included those with a Richter magnitude greater than 4 compiled by the California Institute of Technology for the period 1932 to 1981, and those larger earthquakes for the period 1812 to 1931 compiled by Richter and the U.S. National Oceanic and Atmospheric Administration (NOAA).

The computer printout of the seismicity search of the Los Angeles area is presented in Appendix D. The information listed for each earthquake found in the computer printout included date and time in Greenwich Civil Time (GCT), location of the epicenter in latitude and longitude, quality of the epicentral determination (Q), depth in kilometers, and magnitude. Where a depth of 0.0 is given, the solution was based on an assumed 16-kilometer focal depth.

The recurrence curve was developed on the basis of the seismicity of an area having a radius of 100 kilometers. The application of the Poisson probability law to the resulting recurrence curve, as shown on Figure 4-5, Estimated Probability of Earthquake Occurrence, provides an estimate of the probability of earthquake activity that may affect the subway alignment.

4.4.4 Liquefaction and Seismically Induced Settlement

The evaluation of the liquefaction potential of the soils along the alignment involved the estimation of the potential loss of shear strength of the saturated cohesionless soils during earthquakes that may affect the project. The significant factors that may affect liquefaction include the soil types, particle size and gradation, water level, relative density, confining pressure, intensity of shaking, and duration of shaking. Studies indicate that the liquefaction potential is the greatest where the ground water level is shallow and loose fine sands occur within a depth of 40 to 50 feet. The liquefaction potential increases as the ground acceleration and duration of shaking increase.

Based on the depth to ground water, and the nature of the soil conditions beneath the alignment, we see little or no potential for liquefaction occurring along the Flower Street alignment.

Seismically induced differential settlement is not considered a potential problem.

4.4.5 Slope Stability

The proposed Flower Street Subway is located beneath relatively flat ground with typical gradients less than 1% (1 foot vertical per 100 feet horizontal).

The site is not within a City of Los Angeles Slope Stability Study Area. No indicators of slope instability were noted during our site reconnaissance nor is the site in the path of any known or potential landslides. The potential for slope stability problems along the subway alignment following construction is considered negligible.

4.4.6 Tsunamis, Seiches, and Flooding

The Flower Street Subway area is not susceptible to hazards related to tsunamis, seiche, or flooding.

4.4.7 Subsidence

The historic withdrawal of fluids from below ground has been known to cause subsidence. Considerable subsidence has occurred in the Inglewood oil field about 6-1/2 miles southwest of the subway alignment and in the Wilmington oil field about 20 miles to the south. No known subsidence has taken place in the vicinity of the Flower Street alignment. Two small oil fields are situated near the subway alignment. These are the Union Station oil field, approximately one mile east of the alignment, and the Los Angeles Downtown oil field which encroaches into the southwesterly end of the subway alignment near the intersection of Flower Street and 12th Street. The approximate areal extent of these oil fields is depicted on Figure 4-2. These two fields are of such low productivity that there has been no known subsidence in the area.

Subsidence related to ground water withdrawal has occurred in several areas of California, but there has been no evidence of such subsidence in the vicinity of the proposed subway. Due to the lack of significant quantities of ground water beneath the area, future subsidence in the area of the alignment is not anticipated.

4.4.8 Oil and Gas Occurrence

No indications of natural gas were detected in our borings excavated for the subway alignment. The odor of gasoline and traces of tar were noted in Boring 38 at a depth between 28 feet and 29 feet.

The proposed alignment is not within a Methane Potential Risk zone as defined by the City of Los Angeles' Task Force Report for the March 24, 1985 methane gas explosion and fire in the Fairfax area. On October 8, 1985, an HNU Photo-ionizer was used to measure possible methane or methane-like gas concentrations in Borings 30 and 37, which also serve as piezometers. An atmospheric background level of 3 to 5 parts per million (ppm) was measured. The HNU meter indicated levels of 10 and 17 ppm in the Boring 30 and 37, respectively. These levels do not significantly differ from the atmospheric background levels.

The available information from this investigation and our prior experience with deep excavations adjacent to the alignment suggest that gases in hazardous concentrations during and subsequent to construction of this segment of the subway would not be anticipated. Continuing abandonment of oil production and/or future seismic activity may conceivably alter this state of affairs. We are prepared to install gas monitoring wells along the alignment to permit future monitoring of the presence and concentration of gasses.

4.5 REFERENCES

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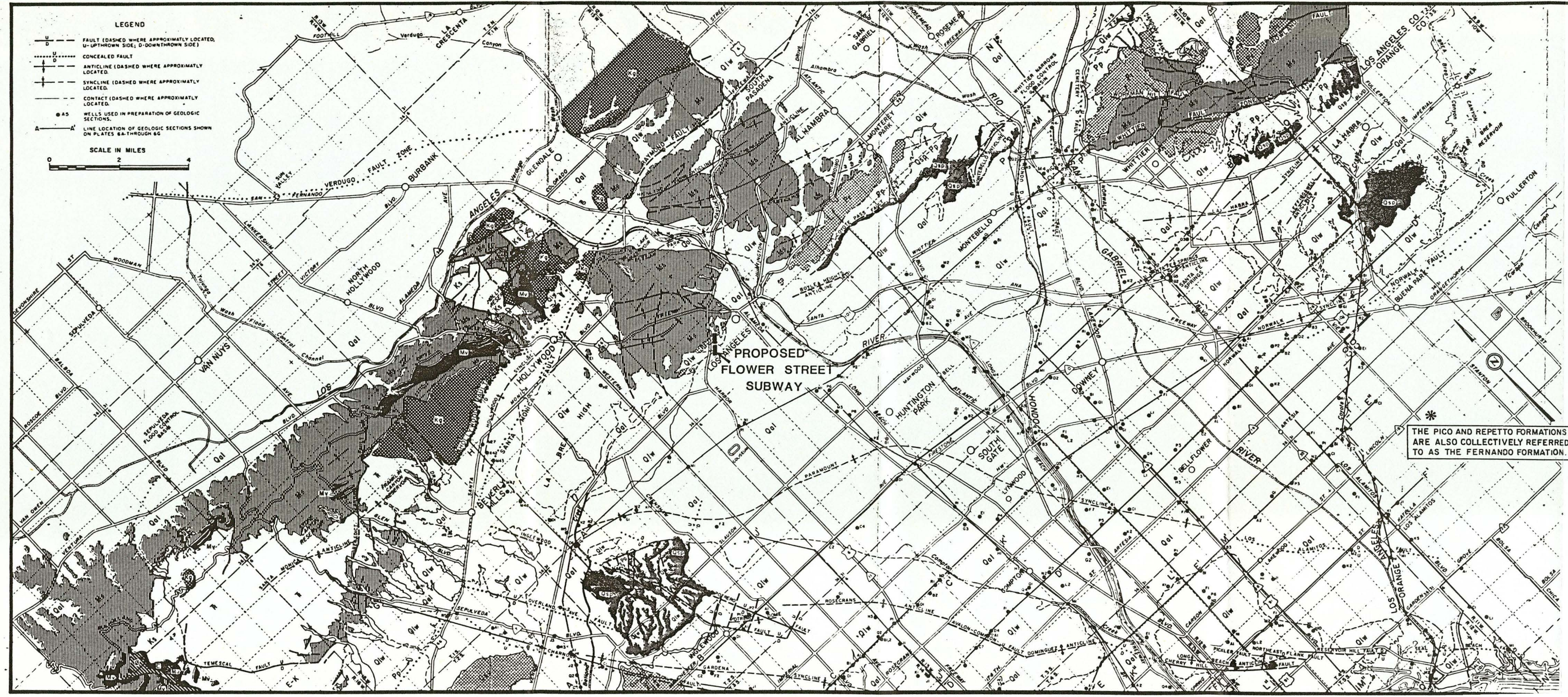

THE PICO AND REPETTO FORMATIONS ARE ALSO COLLECTIVELY REFERRED TO AS THE FERNANDO FORMATION.

LEGEND

SEDIMENTARY ROCKS

QUATERNARY	RECENT		Qal	ALLUVIUM GRAVEL, SAND, SILT AND CLAY
			Qsr	ACTIVE DUNE SAND WHITE OR GREYISH WELL SORTED SAND
	PLEISTOCENE	UPPER	Qso	OLDER DUNE SAND FINE TO MEDIUM SAND WITH SILT, AND GRAVEL LENSES
			Qlw	LAKEWOOD FORMATION (INCLUDES "TERRACE DEPOSITS," "PALOS VERDES SAND," AND "UNNAMED UPPER PLEISTOCENE DEPOSITS") MARINE AND CONTINENTAL GRAVEL, SAND, SANDY SILT, SILT, AND CLAY WITH SHALE PEBBLES
		LOWER	Qsp	SAN PEDRO FORMATION (INCLUDES "LA HABRA CONGLOMERATE" AND PART OF "SAUGUS FORMATION") MARINE AND CONTINENTAL GRAVEL, SAND, SANDY SILT, SILT AND CLAY
			Qsp-Pp	UNDIFFERENTIATED SAN PEDRO FORMATION AND/OR PICO FORMATION MARINE, PARTIALLY CONSOLIDATED GRAVEL, SAND SILT AND CLAY
TERTIARY	PLIOCENE	*	Pp	PICO FORMATION MARINE SAND SILT, AND CLAY INTERBEDDED WITH GRAVEL
		*	Pr	REPETTO FORMATION MARINE SILTSTONE WITH LAYERS OF SANDSTONE AND CONGLOMERATE
	MIOCENE		Ms	(SANTA MONICA MOUNTAINS) MODELO FORMATION MARINE CONGLOMERATIC SANDSTONE, SANDSTONE, AND SHALE TOPANGA FORMATION MARINE CONGLOMERATE, SANDSTONE, AND SHALE (PALOS VERDES HILLS) MONTEREY FORMATION MUDSTONE, DIATOMITE AND SHALE (ELYSIAN HILLS, REPETTO HILLS, AND PUENTE HILLS) PUENTE FORMATION MARINE SILTSTONE, SANDSTONE, SHALE, CONGLOMERATE, LIMESTONE AND TUFF
	OLIGOCENE(?)		Ø	VAQUEROS AND SESPE FORMATIONS CONTINENTAL RED CONGLOMERATE AND SANDSTONE
	EOCENE		E	MARTINEZ FORMATION MARINE CONGLOMERATE, SANDSTONE, SANDY SHALE, AND SHALE
	PALEOCENE(?)		E-K	UNDIVIDED MARTINEZ AND CHICO FORMATIONS
CRETACEOUS	UPPER		Ks	CHICO FORMATION UPPER MARINE MEMBER-HARD CONGLOMERATE, SANDSTONE AND SHALE LOWER CONTINENTAL MEMBER-RED CONGLOMERATE AND SANDSTONE

IGNEOUS AND METAMORPHIC ROCKS

TERTIARY	MIOCENE	Mv	MIDDLE MIOCENE VOLCANIC ROCKS VOLCANIC FLOWS, BRECCIAS, TUFFS, AND INTRUSIVES, CHIEFLY BASALTIC AND ANDESITIC WITH OCCASIONAL ACID ROCKS GENERALLY ASSOCIATED WITH TOPANGA, MODELO, OR PUENTE FORMATIONS
CRETACEOUS	UPPER	Kg	(SANTA MONICA MOUNTAINS) INTRUSIVES OF GRANITE AND GRANODIORITE
JURASSIC		J	(PALOS VERDES HILLS) CATALINA SCHIST COMPARED WITH FRANCISCAN FORMATION OF THE COAST RANGES, VARIED TYPES OF SCHISTOSE ROCKS
TRIASSIC		Tr	SANTA MONICA SLATE GREY TO BLACK SLATE, SPOTTED SLATE, MICA SCHIST WITH QUARTZ VEINS

BASE MAP REFERENCE: CALIFORNIA DEPARTMENT OF WATER RESOURCES, BULLETIN 104, 1961. MODIFIED ACCORDING TO C.D.M.G. GEOLOGIC MAP OF CALIFORNIA, LONG BEACH SHEET, 1962 AND LOS ANGELES SHEET, 1969 AND C.D.M.G. OPEN FILE REPORT 80-10 LA; 1980.

REGIONAL GEOLOGY

LeROY CRANDALL AND ASSOCIATES

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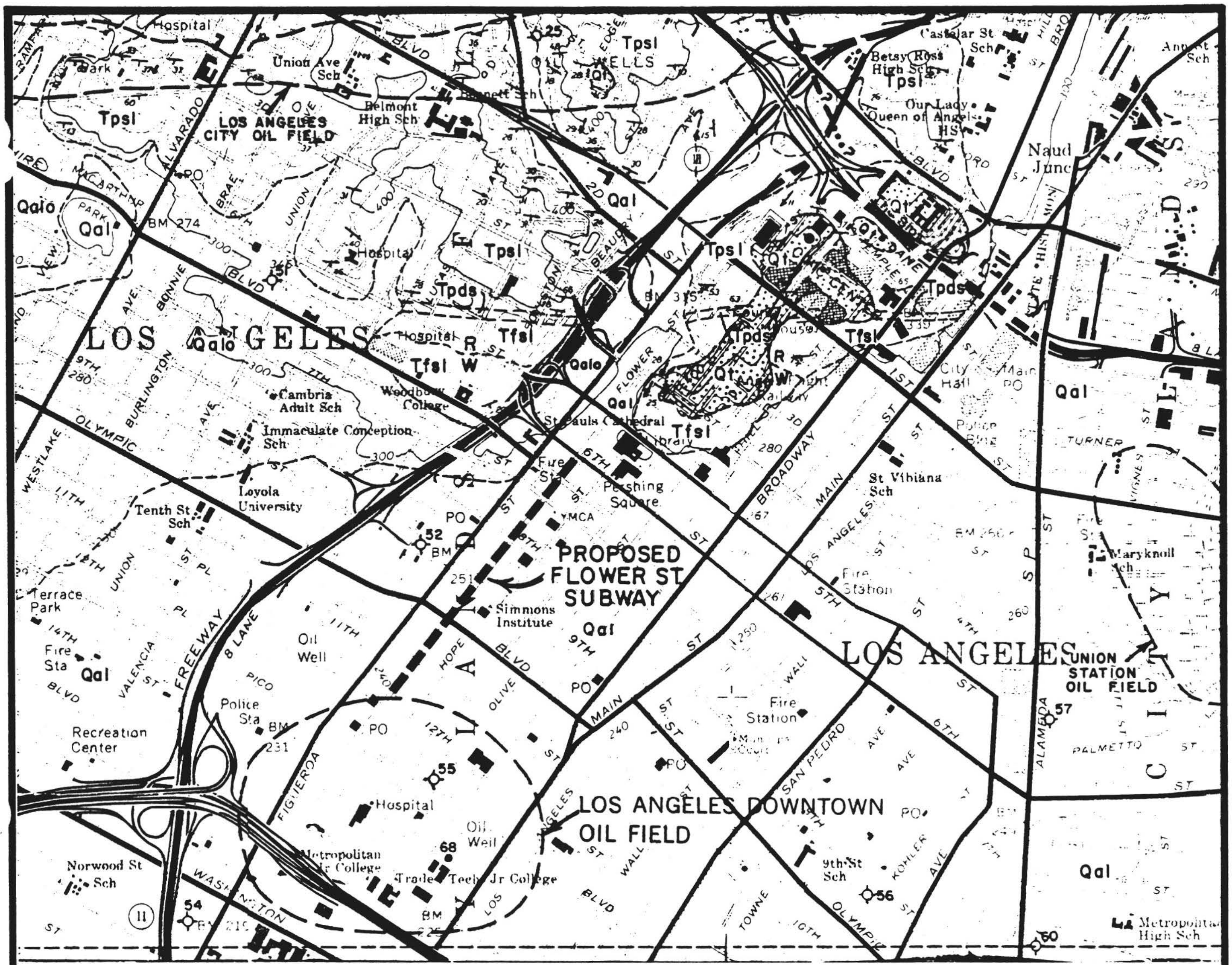
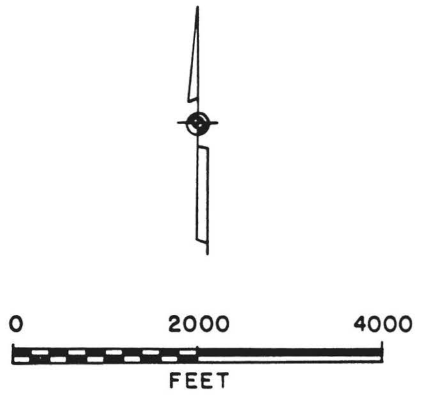


Qal	HOLOCENE ALLUVIUM - Poorly sorted sand, silt and gravel
Qalo	OLDER ALLUVIUM - Silt, sand and gravel. (Includes Qlw)
Qt	TERRACE - Fan and stream deposited sand, gravel and silt.
Tfsl	FERNANDO FORMATION - Massive siltstone.
Tpds	PUENTE FORMATION - Dull white diatomaceous shale.
Tpsl	PUENTE FORMATION - Well bedded siltstone.
------	GEOLOGIC CONTACT - Approximately located.
— —?•••	FAULT - Dashed where approximate, dotted where concealed, queried where uncertain.
⟋31	STRIKE AND DIP OF BEDDING

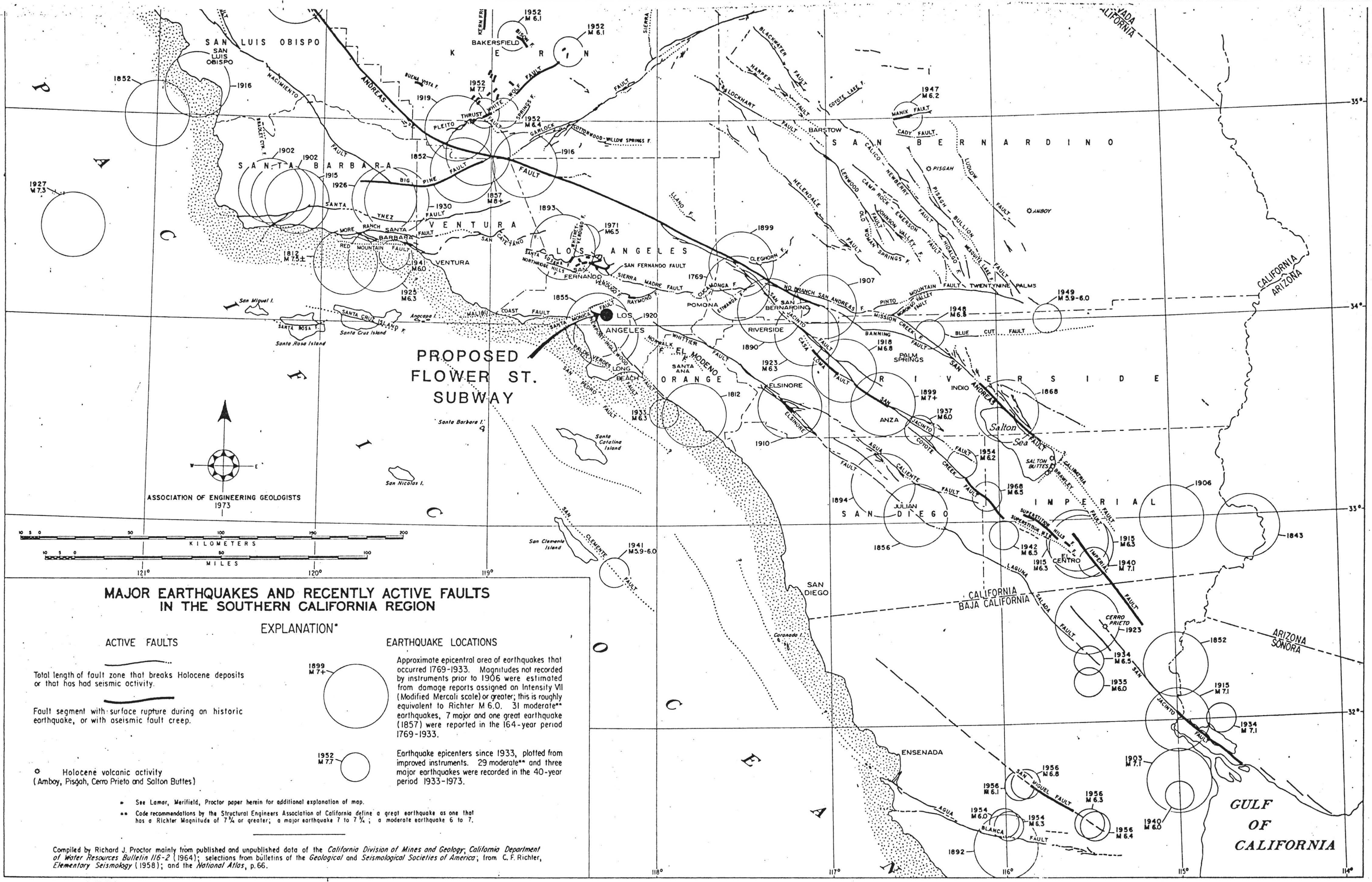
REFERENCE:
Map and geology adapted from California Division Of Mines And Geology, Special Report 101, 1970 - Artificial fill areas not shown.



LOCAL GEOLOGY

LeROY CRANDALL AND ASSOCIATES

FIGURE 4-2



MAJOR EARTHQUAKES AND RECENTLY ACTIVE FAULTS IN THE SOUTHERN CALIFORNIA REGION

EXPLANATION*

ACTIVE FAULTS

Total length of fault zone that breaks Holocene deposits or that has had seismic activity.

Fault segment with surface rupture during an historic earthquake, or with aseismic fault creep.

✿ Holocene volcanic activity
(Amboy, Pisgah, Cerro Prieto and Salton Buttes)

EARTHQUAKE LOCATIONS

1899 M 7+

Approximate epicentral area of earthquakes that occurred 1769-1933. Magnitudes not recorded by instruments prior to 1906 were estimated from damage reports assigned an Intensity VII (Modified Mercali scale) or greater; this is roughly equivalent to Richter M 6.0. 31 moderate** earthquakes, 7 major and one great earthquake (1857) were reported in the 164-year period 1769-1933.

1952 M 7.7

Earthquake epicenters since 1933, plotted from improved instruments. 29 moderate** and three major earthquakes were recorded in the 40-year period 1933-1973.

* See Lamar, Merifield, Proctor paper herein for additional explanation of map.

** Code recommendations by the Structural Engineers Association of California define a great earthquake as one that has a Richter Magnitude of 7 3/4 or greater; a major earthquake 7 to 7 3/4 ; a moderate earthquake 6 to 7.

Compiled by Richard J. Proctor mainly from published and unpublished data of the *California Division of Mines and Geology*; *California Department of Water Resources Bulletin 116-2* (1964); selections from bulletins of the *Geological* and *Seismological Societies of America*; from C. F. Richter, *Elementary Seismology* (1958); and the *National Atlas*, p. 66.

NOTE:
OVERLAND AND CHARNOCK FAULTS NOT SHOWN;
PLEASE SEE FIGURE 4-1.

REGIONAL SEISMICITY

LeROY CRANDALL AND ASSOCIATES

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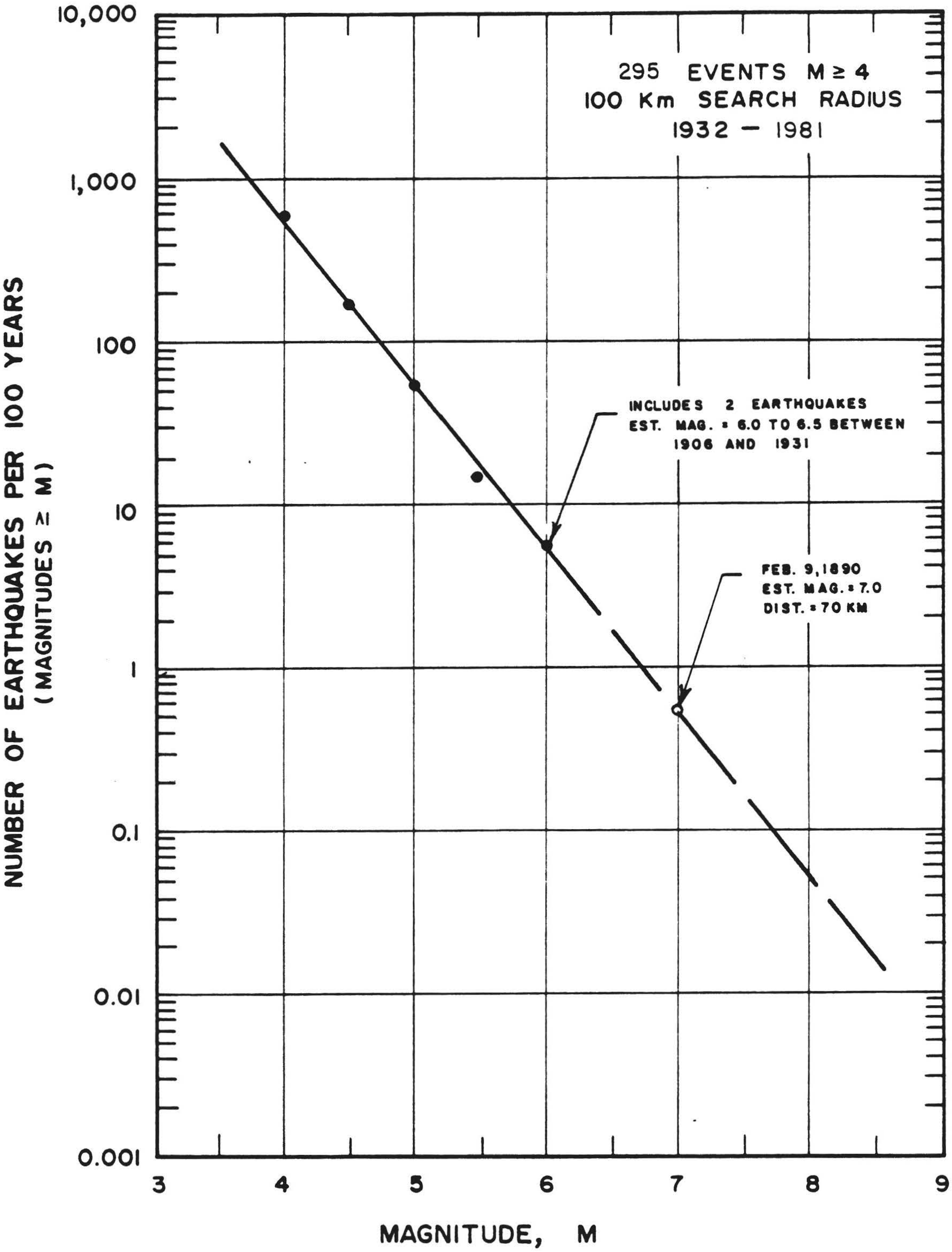

RECURRENCE CURVE

FLOWER STREET SUBWAY

○ REPRESENTS SINGLE EVENT, AND THEREFORE HAS BEEN DISCOUNTED IN PREDICTION.

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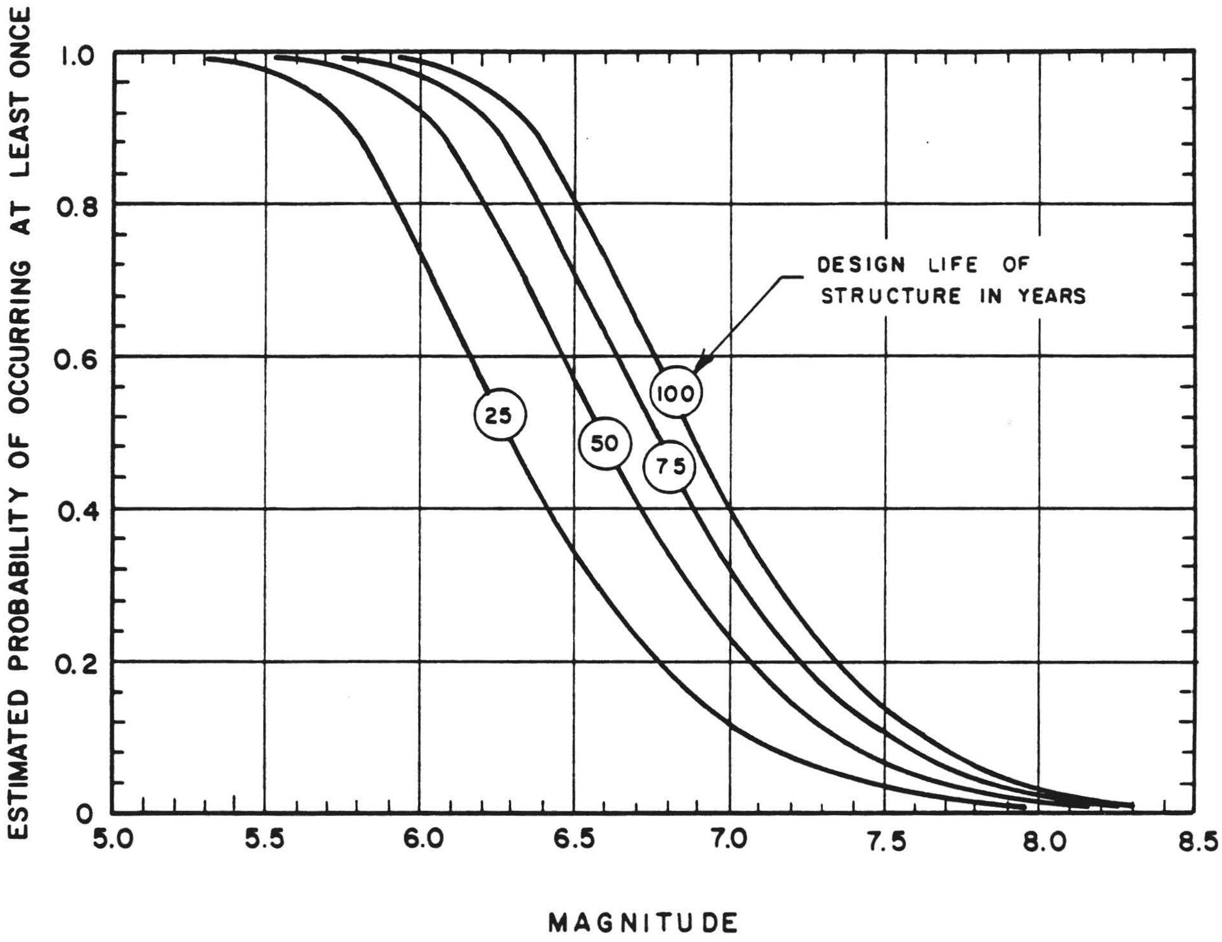
FIGURE 4-4

JOB ADE-85005-8 DATE 9/23/85 DR. JOHN 0 / W.P. CHKD.



ESTIMATED PROBABILITY
OF EARTHQUAKE OCCURRENCE

FLOWER STREET SUBWAY

LeROY CRANDALL AND ASSOCIATES

FIGURE 4-5

[illegible] ations
[illegible] Design Criteria

SECTION 5.0: GEOTECHNICAL EVALUATIONS AND DESIGN CRITERIA

5.1 GENERAL

Construction of the Flower Street Subway will require excavation ranging from about 30 to 50 feet deep. For most of the subway, the excavation will extend into Holocene alluvium. North of 7th Street, the excavation will extend through the Holocene alluvium and into the Fernando Formation siltstone. Because of the depth of excavation, shoring will be required. Because of the proximity of many structures along the alignment of the subway, underpinning and/or shoring designed for surcharge pressures will be required. Ground water was encountered in only one of the borings drilled as part of this investigation. It is conceivable that perched ground water could occur on the surface of the Fernando Formation siltstone. The proposed subway may be supported on the Holocene alluvium or the Fernando Formation siltstone.

5.2 EXCAVATION DEWATERING

As discussed in Section 4.3, the ground water surface within the Holocene alluvial soils is approximately 300 feet below the ground surface. It is doubtful that the ground water elevations will again reach the high levels measured in the early 1900's; those levels were below the lower slab elevation of the proposed subway. Within the Fernando Formation siltstone, there is possibility of water seepage from water perched on the siltstone surface and from fractures within the siltstone. Based on available data, it is our opinion that there will be only minor amounts of ground water encountered during construction. These conditions are more likely to occur within those portions where the excavation would extend into the Fernando Formation siltstone. It is our opinion that water encountered during construction could be collected in sumps within the excavation.

5.3 UNDERPINNING

5.3.1 General

There are many buildings on Flower Street along the alignment of the Flower Street Subway. Because of the proximity of the excavation to these buildings, underpinning may be required if the excavation will remove vertical or lateral support of these buildings. It is not possible to provide specific recommendations for underpinning as the need for underpinning and the appropriate type of underpinning will depend on the relationship between each individual building relative to the proposed subway. Each building will need to be evaluated on an individual basis.

General criteria for determining the need for underpinning are presented on Figure 5-1, Criteria for Determining the Need for Underpinning. Underpinning of the foundations of existing buildings will be required if the foundations are within Zone A. As an alternate to underpinning, it may be possible to design the shoring and subterranean walls of the subway for the lateral surcharge pressures imposed by the adjacent foundations in such a way as to control lateral and vertical movements within acceptable values.

5.3.2 Design Criteria

Underpinning piers should extend below a 45 degree plane drawn upwards from the bottom of the proposed subway. Such underpinning piers carried at least one foot into firm undisturbed natural soils and at least 20 feet below the ground surface may be designed to impose a net dead plus live load of 10,000 pounds per square foot. A one-third increase in the bearing value may be used for wind or seismic loads. The excavation should be observed by a competent geotechnical engineer to verify that the underpinning piers are founded in satisfactory soils.

The downward capacity of slant drilled concrete underpinning piles above a 45 degree plane drawn upward from the bottom of the subway should be neglected in design. The downward capacity below this plane may be determined by using an average friction value of 600 pounds per square foot within the Holocene alluvial soils or 1,000 pounds per square foot within the Fernando Formation siltstone.

5.3.3 Underpinning Performance

Even under the best conditions, an underpinned foundation may still be subject to settlement or lateral movement. This movement may occur during installation of the underpinning or during construction of the subway. The settlement and/or movement can be minimized by proper construction, monitoring, and maintenance.

5.3.4 Underpinning Instrumentation

Some means of monitoring the performance of the underpinning system will be required. The monitoring should consist of periodic surveying of the lateral and vertical locations of the tops of the underpinned footings.

5.4 SHORING SYSTEMS FOR SUBWAY EXCAVATIONS

5.4.1 General

Excavation for the proposed subway will extend typically 30 to 50 feet below the existing Flower Street grade. Because of the lack of space to permit sloped excavations, shoring will be required. Because of the depth of excavation required, internally braced or tied-back shoring will be needed.

The Flower Street Sanitary Sewer will be relocated between Wilshire Boulevard and 8th Street. The relocated sewer will be routed westward from the intersection of Flower Street and Wilshire Boulevard along Wilshire Boulevard, southward along Figueroa Street, and then eastward along 8th Street, where it will reconnect to the existing sewer on Flower Street. The deepest point of the relocated sewer will be approximately 33 feet below grade at the intersection of Wilshire Boulevard and Figueroa Street. No borings were drilled along this new sewer alignment. Nevertheless, we anticipate that the soil conditions along the relocated sewer will be similar to those along the subway alignment. The sewer invert should extend into the Fernando Formation siltstone along Wilshire Boulevard. Along Figueroa Street, there will be a transition from the Fernando Formation siltstone to the Holocene alluvium as the Fernando Formation dips to the south. Along 8th Street, the sewer invert should be totally within the Holocene alluvium. It is expected that conventional excavation and shoring methods may be used for the sewer relocation.

5.4.2 Soldier Pile Shoring System

5.4.2.1 General

The required shoring may consist of steel soldier piles installed in drilled holes, backfilled with concrete, and braced or tied-back with anchors. Where the excavation becomes shallow, particularly near the portal structure north of 12th Street, cantilevered shoring may be used.

5.4.2.2 Lateral Pressures for Internally Braced or Tied-back Shoring

For the design of internally braced or tied-back shoring, we recommend the use of a trapezoidal distribution of lateral earth pressure. For the case where the surface of the retained earth is level, as illustrated in Figure 5-2, the maximum pressure would be equal to 19H in pounds per square foot, where H is the height of the shoring in feet. Where deep basements exist close to the excavation, the lateral pressures may be less and would need to be evaluated when specific details are known.

Where the surface of the retained earth slopes up away from the shoring, or where the shoring is surcharged by an embankment, a greater pressure would be appropriate. Design data could be developed for such cases when the conditions are established.

5.4.2.3 Lateral Pressures for Cantilevered Shoring

For design of cantilevered shoring, a triangular distribution of lateral earth pressure may be used. It may be assumed that the retained soils with a level surface behind the cantilevered shoring will exert a lateral pressure equal to that developed by a fluid with a density of 25 pounds per cubic foot.

5.4.2.4 Lateral Pressures Due to Adjacent Buildings

Adjacent to existing buildings which are within a 1:1 plane drawn upward from the bottom of the subway structure, the shoring should also be designed for any surcharge imposed by the foundations of the adjacent existing building unless the buildings are underpinned.

The magnitude of the lateral surcharge pressures due to adjacent footings will depend on the size and location of the footings relative to the shoring. Each building will need to be evaluated individually. As a guide, the lateral surcharge pressures due to adjacent footings may be estimated according to the criteria shown on Figures 5-3 and 5-4, Lateral Surcharge Pressures Induced by Point Loads and Lateral Surcharge Pressures Induced by Continuous Foundations, respectively.

5.4.2.5 Lateral Pressures Due to Normal Street Traffic

In addition to the recommended earth pressures, the upper 15 feet of shoring should be designed to resist a uniform lateral pressure of 100 pounds per square foot, acting as a result of an assumed 300 pounds per square foot surcharge behind the shoring due to normal street traffic.

5.4.2.6 Lateral Pressures Due to Construction Equipment

The shoring will also need to be designed to support lateral surcharge loads imposed by construction equipment operating adjacent to the shoring. The magnitude of these lateral surcharge loads will depend on the configuration, weight, and orientation of the equipment relative to the shoring; Figures 5-3 and 5-4 may be referred to as a guide in determining the surcharge pressures.

5.4.2.7 Lateral Pressures Due to Earthquakes

In the event of an earthquake occurring during construction of the subway, the shoring walls will be subject to a seismic increment of earth pressure. For design, a pressure equal to that developed by a fluid with density of 10 pounds per cubic foot may be used. However, the distribution of the seismic earth pressure may be taken as an inverted triangle; that is, the maximum pressure would be at the top of the wall. The resultant of the seismic earth pressure would be located at the upper one-third point of the wall.

5.4.2.8 Design of Soldier Piles

As the surface of the Fernando Formation dips to the south along the subway alignment, soldier piles located generally north of Station 9+00 will extend into the Fernando formation; south of Station 9+00, the soldier piles will generally extend into the Holocene alluvial soils.

Where the soldier piles extend into the Fernando Formation, the allowable lateral bearing value (passive value) of the Fernando Formation siltstone below the level of excavation may be assumed to be 1,500 pounds per square foot at the excavated surface, increasing 800 pounds per square foot per foot of depth, up to a maximum of 10,000 pounds per square foot. This would be applicable for soldier piles spaced at least two diameters on center. To develop the full lateral value, provisions should be taken to assure firm contact between the encased soldier piles and the undisturbed siltstone. Structural concrete should be used for the portions of the soldier piles which are below the excavated level; lean-mix concrete may be used above subgrade level.

Where the soldier piles extend into the Holocene alluvium, the allowable lateral bearing value (passive value) of the soils below the level of excavation may be assumed to be 600 pounds per cubic foot. This would be applicable for soldier piles spaced at least two diameters on centers. To develop the full lateral value, provisions should be taken to assure firm contact between the encased soldier piles and the undisturbed soils. Structural concrete should be used for that portion of a soldier pile which is below the excavated level; lean-mix concrete may be used above the subgrade level.

The frictional resistance between the soldier piles and the retained earth may be used in resisting the downward component of the anchor load where tie-back anchors are used. The coefficient of friction between the soldier piles and the retained earth may be taken as 0.4.

The portions of the soldier piles below the excavated level may be used to resist vertical loads. The downward capacities of 24-, 30-, and 36-inch-diameter drilled piles are presented on Figure 5-5, Drilled Pile Capacities. Dead plus live load capacities are shown, a one-third increase may be used when considering seismic loads. The capacities are based on the strength of the soils.

Caving and difficulty in drilling should be expected in installation of the soldier piles where gravelly sands are encountered. Special drilling techniques may be required due to the occasional coarse nature of the overburden.

5.4.2.9 Lagging

Lagging will be required between the soldier piles within the existing fill and Holocene alluvial soils. Lagging may also be required within the Fernando Formation siltstone in zones of water seepage. We believe that lagging may be omitted within the siltstone where the clear spacing between soldier piles is not more than six feet. The exposed siltstone may need to be sprayed with a moisture-retaining substance to prevent slaking. If timber lagging is used, the lagging should be treated if it is to remain in place after completion of subway walls.

The lagging should be designed for the anticipated lateral pressures. However, the pressures on the lagging will be less due to arching of the soils. We recommend that the lagging be designed for the recommended earth pressures but limited to a maximum value of 400 pounds per square foot.

5.4.3 Slurry Walls

The slurry wall technique consists of building the wall in alternating panels. Each panel is typically excavated by clamshell through a surface ditch constantly kept filled with a bentonite slurry to prevent the sidewalls from collapsing. In this particular case, we believe slurry wall construction would require a longer period and more complex construction technology than a conventional soldier pile wall. For these reasons we have not considered slurry wall type construction in further detail.

5.4.4 Internal Bracing and Tie-Back Anchors

5.4.4.1 General

Either internal bracing or tie-back anchors may be used to resist the lateral loads on shoring. In general, it is our opinion that tied-back shoring systems would provide better support than internally braced shoring systems. However, because of the presence of many existing underground utilities and adjacent structures with subterranean levels along the alignment, the installation of tied-back shoring may not be possible in some areas.

5.4.4.2 Internal Bracing

Internal bracing should be installed as the excavation progresses. The bracing at any level should be installed as soon as possible after the excavated level reaches the bracing elevation. The excavation should not extend more than three feet below the bracing elevation prior to installation of those braces.

To limit ground movements, each of the internal braces should be preloaded. The preloading should be at least 50% of the design load. The effects of temperature changes should be incorporated into the design of the braced shoring. Because of the possibility of earthquake loadings, the internal bracing elements should be welded after preloading to have a tensile capacity equal to at least 10% of the design compressive load.

5.4.4.3 Tie-Back Anchors

Tie-back anchors may be used to resist lateral loads. Either friction anchors or belled anchors could be used. However, it has been our experience that friction anchors involve fewer installation problems and provide more uniform support than belled anchors. The presence of gravel and cobbles will cause installation difficulties for the anchors within the more granular soils. Special equipment may be required to drill the anchor holes in such soils.

after the 200% test load has been applied should not exceed 0.25 inch during the 30-minute period. Where satisfactory tests are not achieved on the initial anchors, the anchor diameter and/or length should be increased until satisfactory test results are obtained.

All of the anchors should be pretested to at least 150% of the design load; the total deflection during the tests should not exceed 12 inches. The rate of creep under the 150% test should not exceed 0.1 inch over a 15-minute period. The rate of creep should consistently decrease during the test period; if the rate of deflection does not decrease, the test should not be considered satisfactory.

After a satisfactory test, each anchor should be locked-off at the design load. The locked-off load should be verified by rechecking the load in the anchor. If the locked-off load varies by more than 10% from the design load, the load should be reset until the anchor is locked-off within 10% of the design load.

The installation of the anchors and the testing of the completed anchors should be observed by a competent geotechnical engineer.

5.5 SUPPORT OF TEMPORARY STREET DECKING

It is anticipated that decking will be utilized by the contractor to provide a temporary road surface to maintain traffic along Flower Street during construction of the subway.

The soldier piles used for the shoring system may also be used for support of the temporary street decking. It appears likely that center support piling will be required in some areas, particularly where the excavation will have to be wide, such as at the location of the 7th Street station. The pile capacities presented in Section 5.4.2.8 may be used for design of the support piling.

5.6 SHORING DEFLECTION AND MONITORING

5.6.1 Anticipated Shoring Deflections

It is difficult to accurately predict the amount of horizontal deflection of a shored excavation. It should be realized, however, that some deflection will occur. The horizontal deflection may be as much as one inch at the top of the excavation depending on the nature of the lateral support system and precautions exercised during construction. Precautions may be necessary to limit the magnitude of the deflections to acceptable limits to prevent damage to utilities in the adjacent streets.

5.6.2 Monitoring of the Excavation

Shoring will be required for the entire length of the Flower Street subway and some means of monitoring the performance of the shoring system is recommended. The monitoring should consist of periodic surveying of the lateral and vertical locations of the tops of all the soldier piles and the lateral movement along the entire lengths of selected soldier piles. In addition, selected points on the adjacent existing structures and the street or sidewalk should be monitored.

We suggest that photographs of the adjacent existing buildings be made prior to construction. Any signs of pre-existing distress or damage should be recorded and documented for future reference. The existing buildings should be surveyed and monitored during construction to record any movements.

Some means of periodically checking the load on selected tie-back anchors or internal braces may be necessary, especially when the excavation is near substantial existing structures.

Supplemental instrumentation to monitor the movement of the excavation and adjacent structures could consist of slope inclinometers or tiltmeters.

5.7 EXCAVATION HEAVE

The Fernando Formation siltstones, which will be exposed north of 7th Street, are pre-consolidated; that is, the siltstone has been subjected to a much higher pressure in the past than is currently imposed by the present overburden. Because of this, the siltstone will tend to expand (or rebound) elastically as the existing overburden pressure is reduced during excavation. The heave will occur during excavation and should be completed prior to construction of the subway structure and is expected to be on the order of two inches in the Fernando Formation.

Excavation heave will be about one inch south of 7th Street where the excavation will be entirely within the Holocene alluvial soils.

5.8 FOUNDATION RECOMMENDATIONS

It is our understanding that the proposed subway line structure and station along Flower Street at 7th Street will be supported on a thick base slab which will function as a large mat foundation. North of 7th Street, the subway will be founded in the Fernando Formation siltstone; south of 7th Street, the subway will be founded in the dense Holocene alluvial soils.

The average foundation pressures imposed by the mat foundations are estimated to range from about 2,000 to 2,500 pounds per square foot within the portions to be supported on the Holocene alluvium and about 2,500 to 4,000 pounds per square foot within the portions to be supported on the Fernando Formation siltstone. In our opinion, the subway structures, supported on mat-type foundations, may be adequately supported on either the Fernando Formation siltstone or the dense Holocene alluvium as planned.

The settlement of the subway structures, due to recompression of the elastic heave, is estimated to range between 3/4 inch in the alluvium and 1-1/4 inches in the Fernando Formation siltstone. The differential settlement between the portion supported on the Fernando Formation siltstone and the portion supported on the dense Holocene alluvium will not be significant because of the gradual transition from one material to the other.

5.9 PERMANENT GROUND WATER PROVISIONS

As mentioned earlier in Section 4.3, the ground water levels along the proposed subway alignment are believed to be deep and well below the planned subway construction. However, perched ground water was encountered on occasion in the Holocene alluvium at its interface with the Fernando Formation siltstone. Water could be expected in some fractures in the siltstone.

The proposed subway structure north of Station 8+00 should be designed for possible hydrostatic pressure on the assumption that no drainage of the subway section will be provided. For design, it may be assumed that the water north of Station 8+00 could rise to Elevation 260.

5.10 LOADS ON PERMANENT WALLS AND SLABS

5.10.1 Permanent Static Earth Pressures

The vertical pressure on the roof of the subway structure may be assumed to be equal to the overburden pressure of the soil overlying the roof. A wet unit weight of 125 pounds per cubic foot may be used to compute the overburden pressure. The roof should also be designed for surcharge pressures from any external loads such as traffic.

The walls of the subway should be designed to resist lateral earth pressure. The recommended lateral earth pressure distributions for sidesway and long-term loading conditions are presented on Figure 5-6.

5.10.2 Hydrostatic Pressures

Recommendations for hydrostatic pressures are presented in Section 5.9.

5.10.3 Surcharge Pressures

The subway walls should be designed for lateral surcharge pressures imposed by adjacent buildings within a 1:1 plane drawn upward from the bottom of the subway unless the building foundations are underpinned. As mentioned in Section 5.4.2.7, the magnitude of the lateral surcharge pressures will need to be evaluated on an individual building by building basis.

5.11 SEISMIC DESIGN CRITERIA

5.11.1 Design Earthquake Parameters

The causative faults were selected from the list of faults presented in Tables 4-2 and 4-3 as the most significant faults along which earthquakes are expected to generate motions affecting the subway. Postulated design earthquakes were selected in accordance with the seismic criteria set forth in the "Recommended Lateral Force Requirements and Commentary" by the Structural Engineers Association of California. Those criteria have been interpreted as follows:

- Structures shall resist moderate earthquakes with a low probability of structural damage.

- Structures shall resist major earthquakes, of the intensity of severity of the strongest experienced in California, with a low probability of collapse, but with some structural as well as non-structural damage.

Accordingly, the major and moderate earthquakes were interpreted as the maximum credible earthquake and the maximum probable earthquake, respectively, that may be generated along the causative faults. The maximum credible earthquake constitutes the maximum earthquake that appears to be reasonably capable of occurring under the conditions of the presently known geological framework; the probability of such an earthquake occurring during the lifetime of the subway may be low. The maximum probable earthquake constitutes an earthquake that may be likely to occur during the design life of the subway.

The recurrence curve on Figure 4-4 was developed on the basis of the seismicity of an area having a radius of 100 kilometers. The application of the Poisson probability theory to the resulting recurrence curve, as shown on Figure 4-5, Estimated Probability of Earthquake Occurrence, provides an estimate of the probability of earthquake activity that may affect the site. The probability of at least one occurrence of a 100-year earthquake within the search radius in a time period of 100 years would be approximately 50% to 60%. The probability value is based on the assumption that the seismic risk is equal throughout the search area.

A site dependent procedure was used which is based on a statistical analysis approach consisting of estimating the peak ground motion values (acceleration, velocity, and displacement) anticipated at the site. The ground motion values have been found to vary with the magnitude of earthquake and distance of the site from the source of energy release.

The peak ground accelerations for the postulated design earthquakes are based on the studies by Seed, et al, who developed peak ground acceleration relationships for four broad site classifications: rock, stiff soil, deep cohesionless soil, and soft to medium soil deposits. Based on a review of the results of the boring logs, downhole seismic surveys, and the local geology, the Flower Street subway is judged to be within materials which can be classified as being stiff. This classification is deemed appropriate for the portions of the subway within the Fernando Formation siltstones as well as the dense granular Holocene alluvium.

The peak ground motion values for velocity and displacement are based on relationships developed by Mohraz which relate peak ground velocity and displacement to the peak ground acceleration for four site classifications.

For design purposes, two levels of earthquake ground shaking are to be considered. The Operating Design Earthquake (ODE) corresponds to the level of earthquake at which the subway system will continue to operate normally with no disruption of services; the ODE earthquake has been taken as equivalent to a maximum probable earthquake with a 100-year recurrence interval. The Maximum Design Earthquake (MDE) is considered equivalent to the maximum credible earthquake. The San Andreas, Newport-Inglewood, Santa Monica-Hollywood, and Raymond Faults are considered to be the faults most likely to impact the subway although any of the faults in Southern California could produce significant ground motions. Design ground motion values for the subway are given in Table 5-1, Design Earthquake Parameters, these values are considered applicable to both the siltstone and alluvium.

Table 5-1
Design Earthquake Parameters

Design Earthquake	Acceleration (g)		Velocity (Ft/Sec)		Displacement (Feet)	
	Hor.	Vert.	Hor.	Vert.	Hor.	Vert.
ODE	0.27	0.18	0.9	0.6	0.5	0.3
MDE	0.46	0.31	1.4	0.9	0.7	0.5

5.11.2 Dynamic Material Properties

Shear and compressional wave velocities were determined in two borings (Borings 30 and 37) using the downhole seismic survey method. The results are presented in Appendix B.

Average dynamic properties were derived from the shear and compressional wave velocities and are summarized in Table 5-2. The moduli values correspond to levels of low strain and would need to be adjusted for the design strain level when considering dynamic loading. The variation of shear modulus with shear strain is presented on Figure 5-7.

Table 5-2
Dynamic Material Properties

	Holocene Alluvium	Fernando Formation
Average Shear Wave Velocity (ft/sec)	1,250	1,250
Average Compression Wave Velocity (ft/sec)	3,000	6,000
Poisson's Ratio	0.40	0.48
Modulus of Elasticity (ksf)	1.7×10^4	1.7×10^4
Shear Modulus (ksf)	6.3×10^3	5.8×10^3
Constrained Modulus (ksf)	3.6×10^4	1.5×10^5

5.11.3 Horizontal Shear Deformations

The subway structure will be required to conform to the effects of soil deformations due to earthquake; this is also known as earthquake racking. The estimated horizontal shear deformations for the Operation Design Earthquake and the Maximum Design Earthquake are shown in Figures 5-8 and 5-9. The subway structures should be checked in accordance with the "Supplemental Criteria for Seismic Design of Underground Structures" by Metro Rail Transit Consultants for the Southern California Rapid Transit District, dated June 1984.

5.11.4 Characteristic Site Period

The evaluation of the characteristic site period, Ts, is necessary to determine the coefficient of site-structure resonance, S, in accordance with Section 2312 of the 1982 edition of the Uniform Building Code. The characteristic period of the site was evaluated following the procedures suggested in SEAOC Standard No. 1, Recommended Lateral Force Requirements and Commentary, Seismology Committee, Structural Engineers Association of California, 1980.

The characteristic site period for portions of the subway founded in the Fernando Formation siltstone may be taken as 0.7 to 0.9 seconds. For portions founded in the Holocene alluvium, the characteristic site period may be taken as 3/4 to 1 second. The values nearest to the period of the subway structure should be used in determining the site-structure resonance coefficient, S. The details of the analysis performed to evaluate the site period are presented in Appendix D.

5.11.5 Liquefaction Potential

Water was not encountered in any of the exploratory borings except for Boring 30 where water within the upper alluvial soils was perched over the Fernando Formation siltstone. The alluvial soils encountered along the alignment were dense to very dense. It is our opinion, based on the presence of limited perched ground water and having dense to very dense alluvium, that the probability of liquefaction in the event of an earthquake is very low.

5.12 EARTHWORK RECOMMENDATIONS

5.12.1 Excavation and Slopes

Excavation approximately 30 to 35 feet deep will be required for most of the subway alignment. Excavation up to 50 feet will be required near the north end of the subway. The presence of cobbles and boulders within the Holocene alluvial soils will make excavating along the subway somewhat more troublesome than in finer-grained soils. However, conventional earth-moving equipment may be used. Although some cemented deposits within the Fernando Formation siltstones may be encountered during excavation, we believe that excavation of even the hard deposits can be accomplished using jackhammers. In our opinion, blasting should not be necessary.

Although it is unlikely, where the necessary space is available, temporary unsurcharged excavations up to 10 feet in vertical height may be sloped back at 3/4:1 (horizontal to vertical) within the alluvial soils in lieu of using shoring. With the Fernando Formation siltstone, temporary unsurcharged excavations up to 10 feet in vertical height may be sloped back at 2/3:1 (horizontal to vertical) in lieu of using shoring. All applicable requirements of the California Construction and General Industry Safety Orders, the Occupational Safety and Health Act of 1970, and the Construction Safety Act should be met.

Where sloped embankments are used, the tops of the slopes should be barricaded to keep heavy vehicles and heavy storage loads at least ten feet from the tops of the slopes. If the construction embankments are to be maintained during the rainy season, berms are suggested along the tops of the slopes where necessary to prevent runoff water from entering the excavations and eroding the slope faces. The soils exposed in the cut slopes should be observed during excavation by our personnel so that modifications of the slopes can be made if variations in the soil conditions occur.

5.12.2 Foundation Observation

To verify the presence of the firm soils, all foundation excavations should be cleaned of any loosened soils and subsequently observed by a competent geotechnical engineer. Foundations should be deepened as necessary to reach the firm soils. Required foundation and trench backfill should be mechanically compacted; flooding should not be permitted.

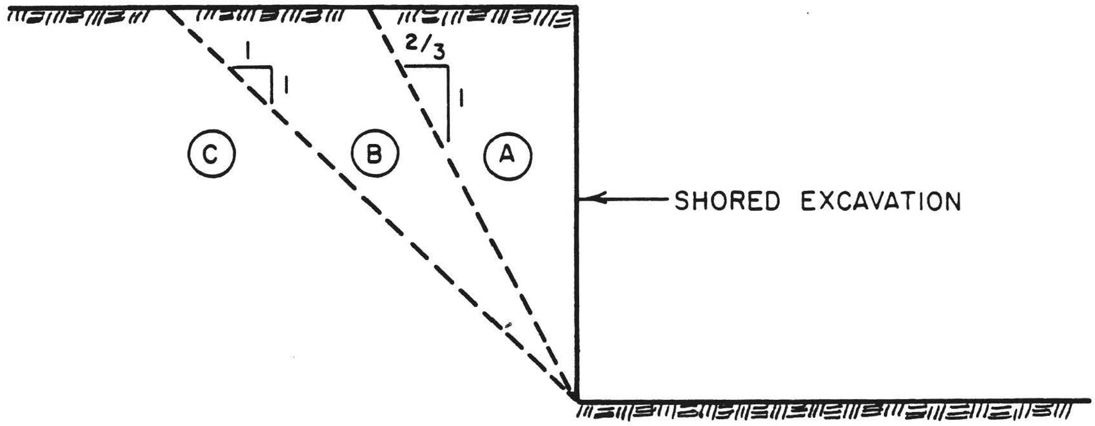
Foundation excavations should be cleaned of loose soils prior to pouring any concrete. The Fernando Formation siltstones may contain occasional hard, cemented layers, and jackhammers or other special equipment may be required to excavate any hard layers which occur within foundations. The foundation excavations should be left slightly uneven if necessary, rather than filling in over-excavated areas with loose or compacted soils.

All applicable requirements of the California Construction and General Industry Safety Orders, the Occupational Safety and Health Act of 1970, and the Construction Safety Act should be met.

5.12.3 Backfill

All required backfill should be mechanically compacted, in layers not more than eight inches thick, to at least 90% of the maximum density obtainable by the ASTM Designation D1557-78 method of compaction. Flooding should not be permitted. Proper compaction of the backfill will be necessary to minimize settlement of the backfill and to minimize settlement of overlying walks and paving. The backfill should be approved for use by a competent geotechnical engineer prior to importing. At least the upper portion of the backfill should consist of relatively impermeable soils to minimize moisture infiltration in the backfill. However, clay soils should not be used because of their expansive nature.

Some settlement of the backfill should be anticipated, and any utilities supported therein should be designed to accept differential settlement, particularly at the points of entry to the subway structure.



ZONE Ⓐ Special Provisions Required for All Structures:

Underpinning or shoring system designed to support lateral surcharge loads from building foundations with acceptably small ground movements.

ZONE Ⓑ Generally, Special Provisions Required for All Structures:

Shoring system designed to support lateral surcharge loads from building foundations with acceptably small ground movements; underpinning not generally needed.

ZONE Ⓒ Generally No Special Provisions

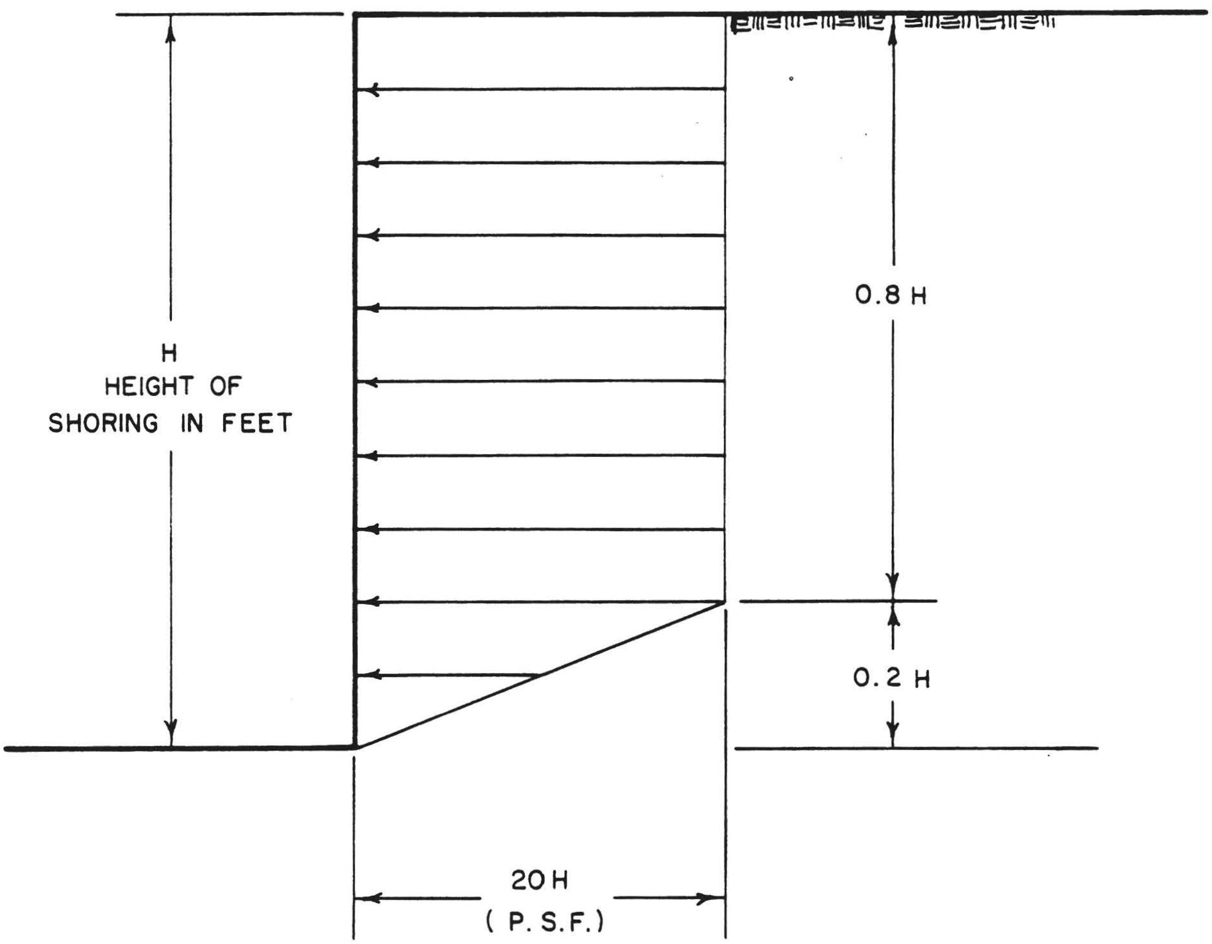
CRITERIA FOR
DETERMINING THE NEED FOR UNDERPINNING

LeROY CRANDALL AND ASSOCIATES

FIGURE 5-1

FOR JOB ADE-85005-B DATE 10/2/85 DR os ML W.P. dmh CHKD

FOR JOB ADE-85005-B DATE 9/30/85 DR ov W.P. CHKD

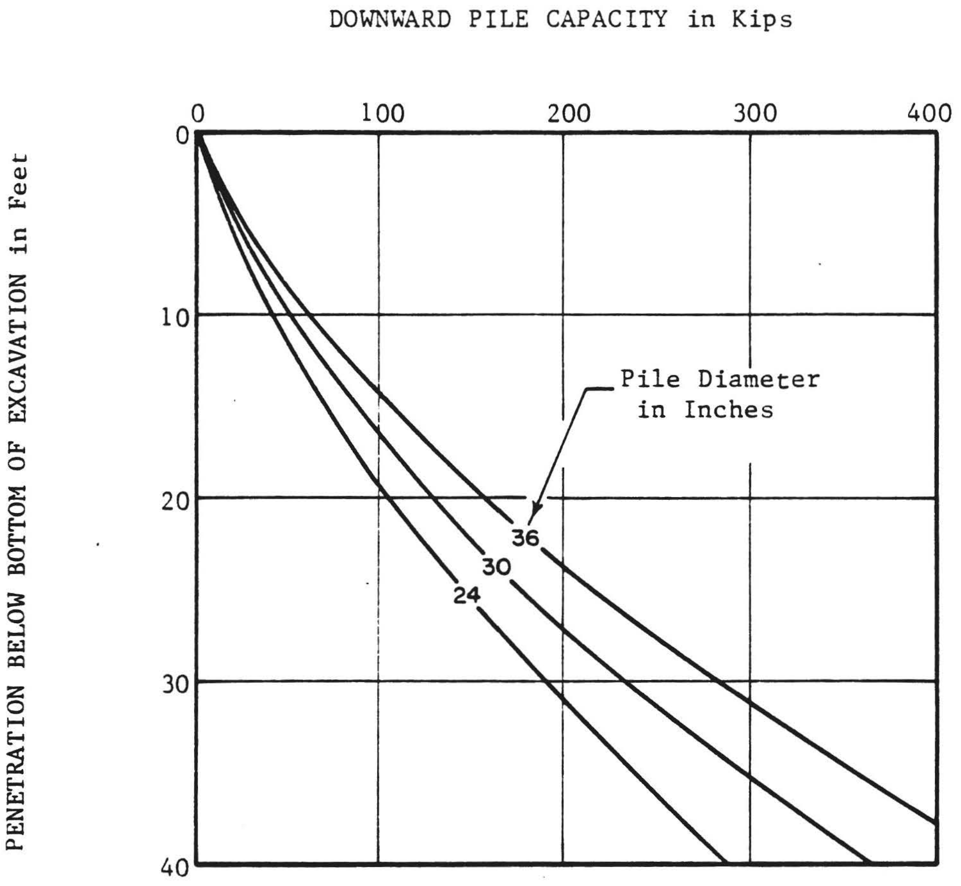


LATERAL EARTH PRESSURE
FOR BRACED OR TIED BACK SHORING

LeROY CRANDALL AND ASSOCIATES

FIGURE 5-2

FOR JOB ADE-85005-8 DATE 10/3/85 DR MS W.P. dmh CHKD



NOTES:

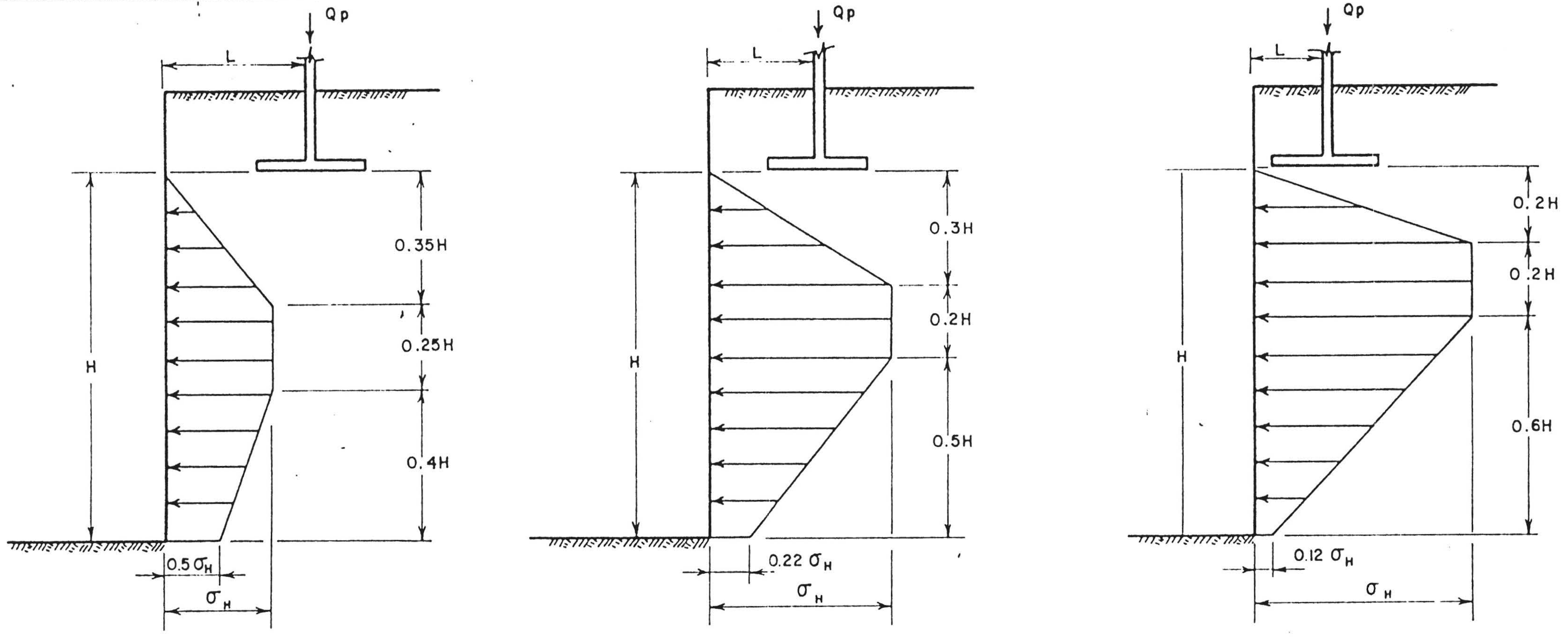
(1) The indicated values refer to the total of dead plus live loads; a one-third increase may be used when considering seismic loads.

(2) Piles in groups should be spaced a minimum of 2½ diameters on centers, and should be drilled and filled alternately with the concrete permitted to set at least 8 hours before drilling an adjacent hole.

(3) The indicated values are based on the strength of the soils; the actual pile capacities may be limited to lesser values by the strength of the piles.

(4) The indicated values apply to both the Fernando Formation and the Holocene Alluvium.

DRILLED PILE CAPACITIES



$0.75\ H \leqslant L < 1.0\ H$

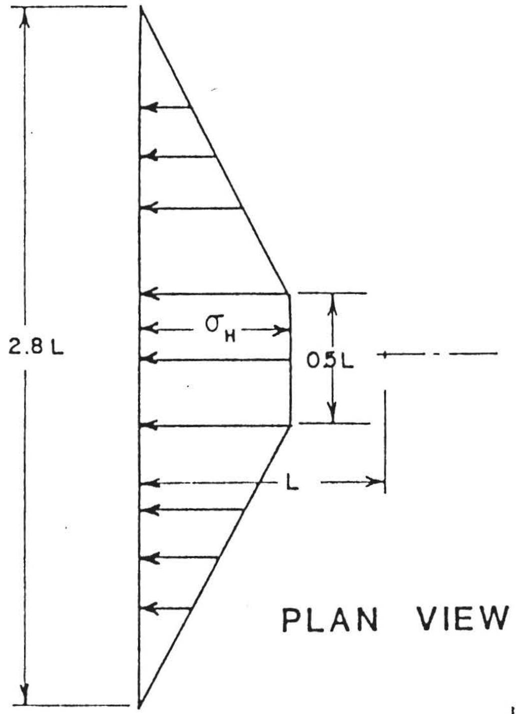
$\sigma_H = 0.5(\frac{Q_p}{H^2})$

$0.5\ H < L < 0.75\ H$

$\sigma_H = 0.9(\frac{Q_p}{H^2})$

$L \leqslant 0.5\ H$

$\sigma_H = 1.7(\frac{Q}{H^2})$



PLAN VIEW

LEGEND:

L - NORMAL DISTANCE IN FEET FROM CENTERLINE OF FOOTING TO FACE OF WALL.

H - VERTICAL DISTANCE IN FEET FROM THE BOTTOM OF FOUNDATION TO THE BOTTOM OF THE EXCAVATION.

σ_H - MAXIMUM HORIZONTAL STRESS IN PSF ACTING NORMAL TO THE PLANE OF THE WALL.

Q_p - COLUMN LOAD IN POUNDS

NOTES:

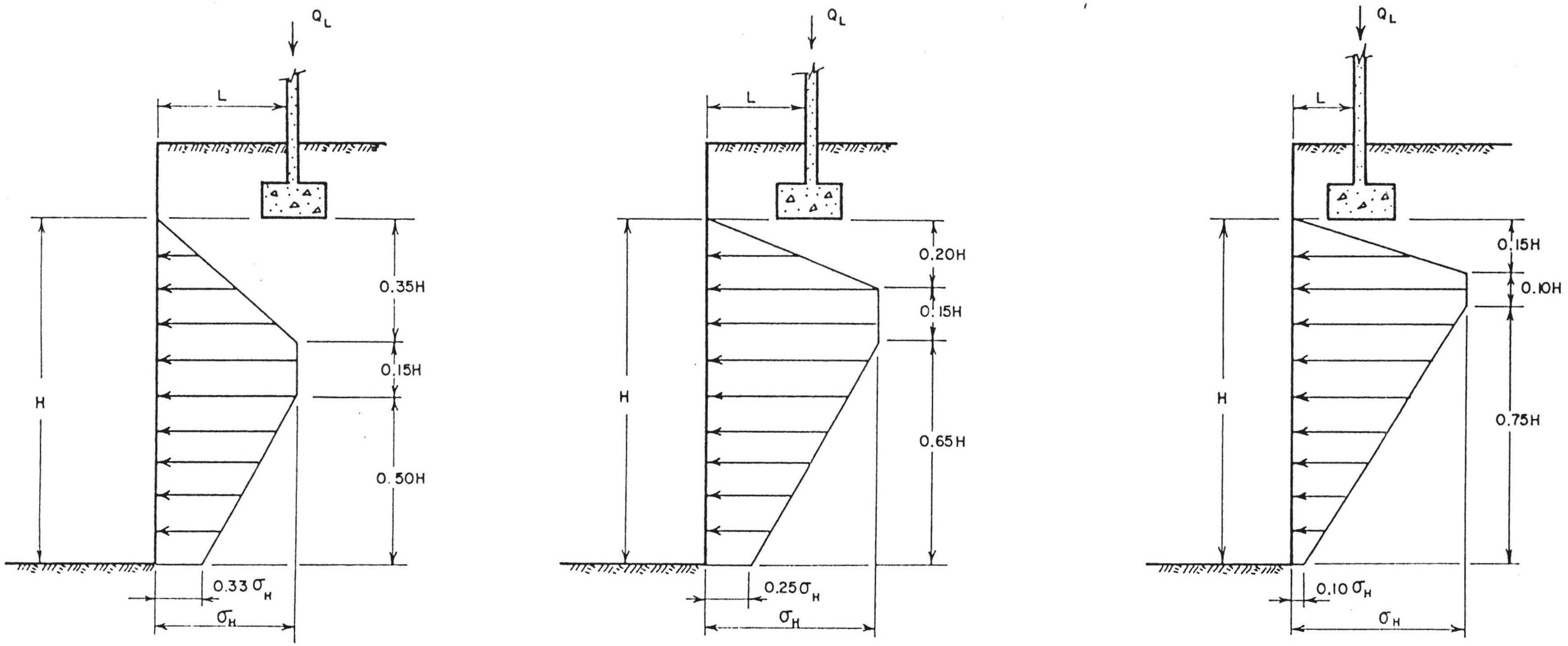
1. PRESSURE DISTRIBUTIONS AS SHOWN IN THIS FIGURE SHOULD BE ADDED TO THE EARTH PRESSURES.
2. WHEN MORE THAN ONE FOUNDATION IS LOCATED WITHIN A DISTANCE H FROM THE EXCAVATION, THE PRESSURES DUE TO EACH FOUNDATION MUST BE SUPERIMPOSED.
3. THE DIAGRAMS SHOWN HERE ARE BASED ON THE BOUSSINESQ ANALYSES FOR LINE LOADS ON ELASTIC FOUNDATIONS, BUT HAVE BEEN SIMPLIFIED TO STRAIGHT LINE DIACRAMS.
4. PRESSURES ARE APPLICABLE BOTH TO SHORING DURING CONSTRUCTION AND TO THE FINISHED STRUCTURES.

LATERAL SURCHARGE PRESSURE INDUCED BY POINT LOADS

LeROY CRANDALL AND ASSOCIATES

FORM I

JOB AUE-83005-8 DATE 11/18/83 DR. M O.E. CHKD.



$0.75 < L < 1.0\ H$

$$\sigma_H = 0.6\left(\frac{Q_L}{H}\right)$$

$0.50\ H < L < 0.75\ H$

$$\sigma_H = 0.85\left(\frac{Q_L}{H}\right)$$

$L \leqslant 0.50\ H$

$$\sigma_H = 1.0\left(\frac{Q_L}{H}\right)$$

LEGEND:

L - NORMAL DISTANCE IN FEET CENTERLINE OF COLUMN TO FACE OF WALL.

H - VERTICAL DISTANCE IN FEET FROM THE BOTTOM OF FOOTING TO THE BOTTOM OF THE EXCAVATION.

σ_H - MAXIMUM HORIZONTAL STRESS IN PSF ACTING NORMAL TO THE PLANE OF THE WALL.

Q_L - FOOTING LOAD IN POUNDS PER LINEAL FOOT

NOTES:

1. PRESSURE DISTRIBUTIONS AS SHOWN IN THIS FICURE SHOULD BE ADDED TO THE EARTH PRESSURES.
2. WHEN MORE THAN ONE FOUNDATION IS LOCATED WITHIN A DISTANCE H FROM THE EXCAVATION, THE PRESSURES DUE TO EACH FOUNDATION MUST BE SUPERIMPOSED.
3. THE DIAGRAMS SHOWN HERE ARE BASED ON THE BOUSSINESQ ANALYSES FOR POINT LOADS ON ELASTIC FOUNDATIONS, BUT SIMPLIFIED TO STRAIGHT LINE DIAGRAMS.
4. PRESSURES ARE APPLICABLE BOTH TO SHORING DURING CONSTRUCTION AND TO THE FINISHED STRUCTURES.

LATERAL SURCHARGE PRESSURE INDUCED BY CONTINUOUS FOUNDATIONS

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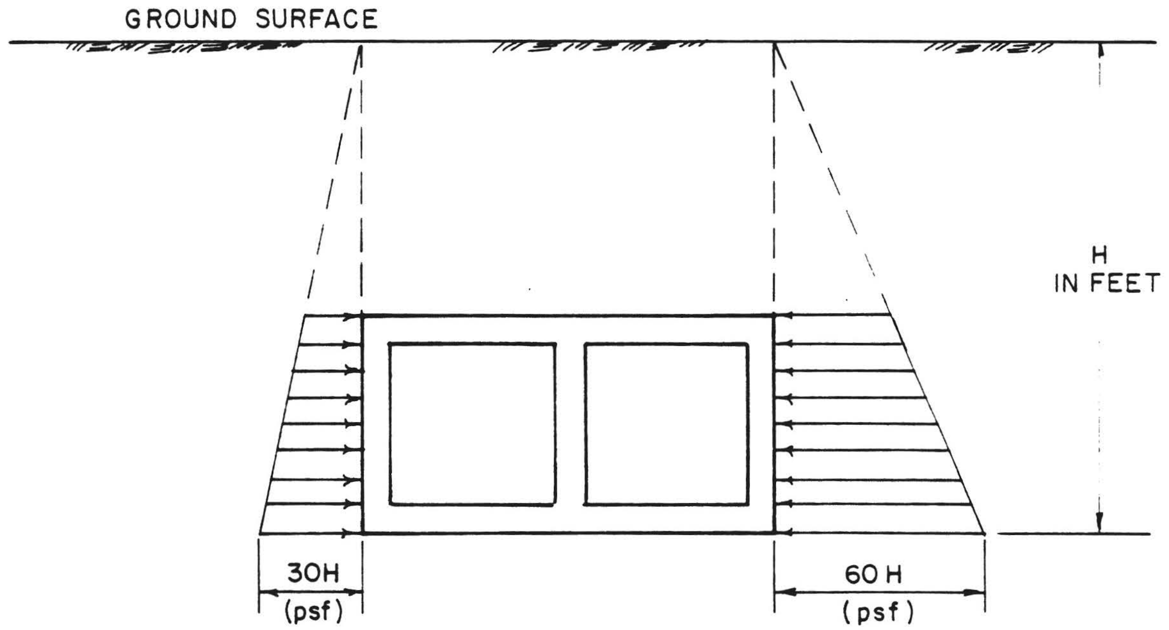
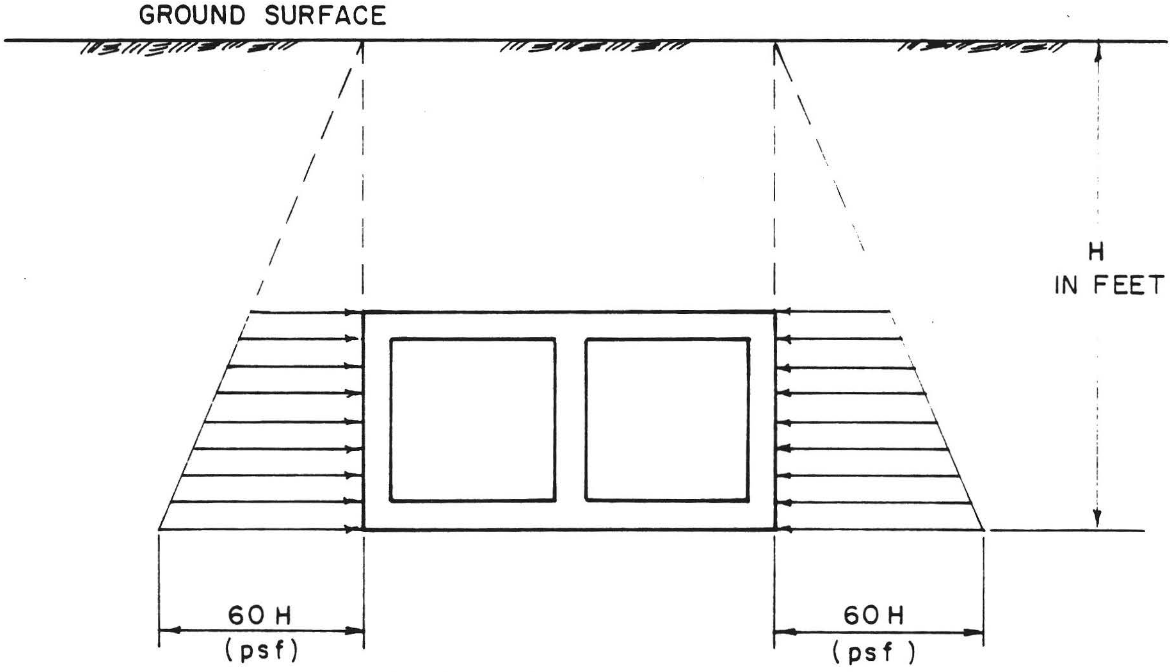


SIDESWAY CONDITION



LONG-TERM LOADING

RECOMMENDED LATERAL EARTH PRESSURE DISTRIBUTIONS

(FOR PERMANENT WALLS)

NOTE: THE SUBWAY STRUCTURE MAY NEED TO BE DESIGNED
FOR POSSIBLE HYDROSTATIC PRESSURES; SEE SECTION 5.9.

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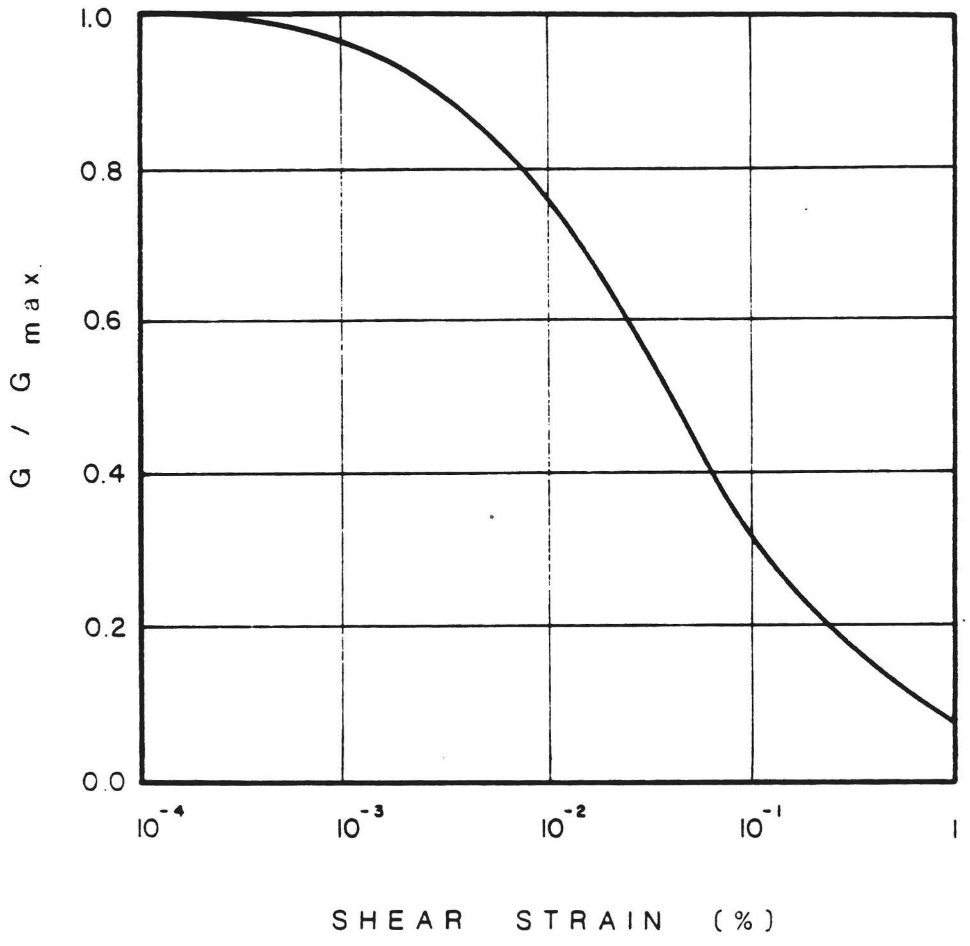
FIGURE 5-6

FOR

JOB ADE-85005-B DATE 11/18/85 DR [illegible] W.P. [illegible] CHKD



VARIATION OF SHEAR MODULUS WITH SHEAR STRAIN

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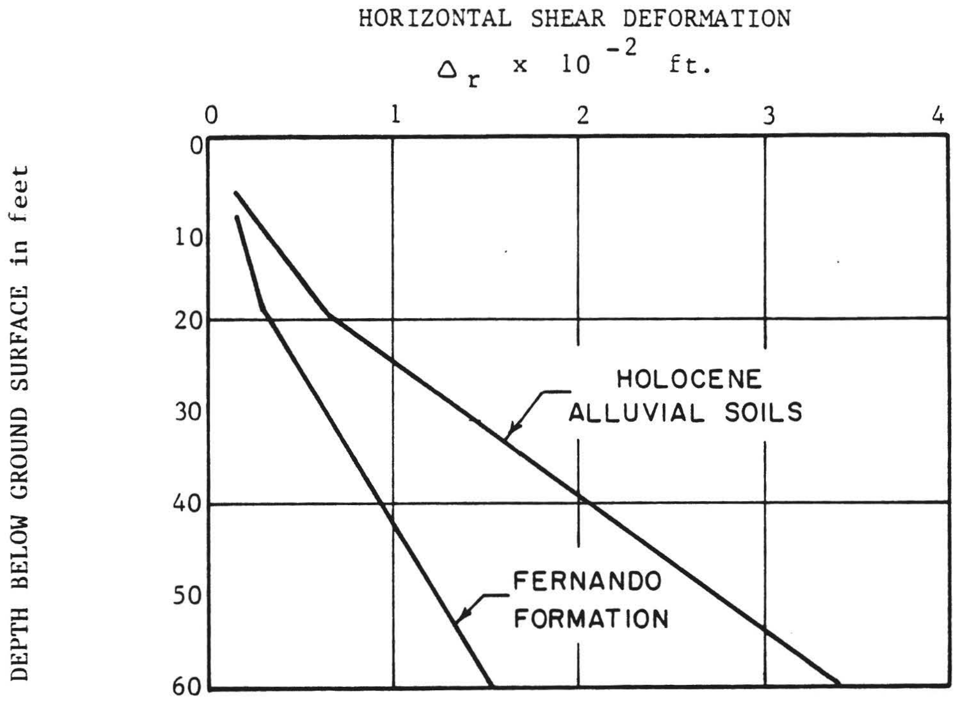
FIGURE 5-7

FOR

JOB ADE·85005-B DATE 11/14/85 DR M.G. W.P. CHKD

FOR JOB ADE 85005-B DATE 10/7/85 DR MS W.P. dmh CHKD



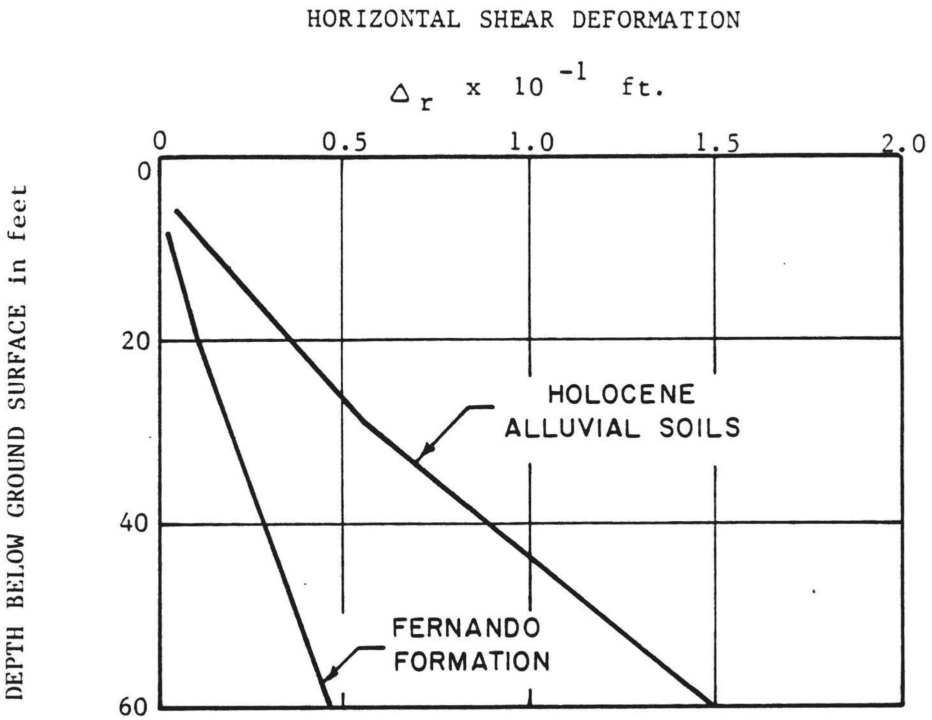
HORIZONTAL SHEAR DEFORMATION

OPERATING DESIGN EARTHQUAKE

(Adapted from "Supplemental Criteria for Seismic Design of Underground Structures" by Metro Rail Transit Consultants for the Southern California Rapid Transit District, June, 1984)

LeROY CRANDALL AND ASSOCIATES

FIGURE 5-8



HORIZONTAL SHEAR DEFORMATION

MAXIMUM DESIGN EARTHQUAKE

(Adapted from "Supplemental Criteria for Seismic Design of Underground Structures" by Metro Rail Transit Consultants for the Southern California Rapid Transit District, June, 1984)

LeROY CRANDALL AND ASSOCIATES

FIGURE 5-9

JOB ADE-85005-B DATE 10/7/85 DR OS : MS W.P. dmh CHKD

FO

Manitoba

[illegible] Program

APPENDIX A:
FIELD EXPLORATION PROGRAM

A.1 SUMMARY

The alignment was explored by drilling a total of 12 exploration borings to depths ranging from 60 to 80 feet, for a total of 751 lineal feet of drilling. The locations of the borings are shown on Figures 2-3 through 2-6 presented in the report. Included in this Appendix are the following:

- Boring Logs
- Unified Soil Classification System
- Key to Boring Logs
- Piezometer Installations and Groundwater Monitoring Data

A.2 METHOD OF DRILLING

The borings were drilled using rotary wash-type and bucket auger-type drilling equipment.

The rotary wash borings were drilled by Pitcher Drilling Company who utilized a Failing 750 drilling rig operated by a two-man crew. The borings were 5 inches in diameter and augered to a depth of 6 feet; 6-inch-diameter steel casing was installed to a depth of six feet. Drilling mud was used in the drilling process below 6 feet. The mud was removed following completion of the drilling to permit measurement of the water level.

The bucket auger borings were drilled by C & L Drilling Company using a bucket-type rig operated by a two-man crew. The bucket borings were 24 inches in diameter. Raveling and/or caving occurred in some of the bucket borings as indicted on the boring logs. Casing or drilling mud was not used to extend these borings to the depths drilled.

Each of the borings was backfilled upon completion of drilling, except for those borings in which piezometers were installed.

A.3 LOGGING AND SAMPLING

The following personnel from LeRoy Crandall and Associates (LC&A) and Geotechnical Consultants Inc. (GCI) participated in the field exploration program:

Robert Chieruzzi - Project Manager, LC&A
Marshall Lew - Assistant Project Manager, LC&A
Mervin Johnson - Principal Engineering Geologist, LC&A
Do Mar - Project Engineer, LC&A
Mike Shahabi - Staff Engineer, LC&A
Wilford Stelts - Field Exploration Manager, LC&A
Gary Cito - Field Exploration Supervisor, LC&A
Theodore Powers - Field Geologist, GCI
James Thurber - Field Geologist, GCI
Amir Matin - Field Geologist, LC&A
Lowell Stelts - Field Technician, LC&A

The borings were logged continuously during the drilling. Undisturbed samples were obtained with the Crandall sampler at depth intervals of about five feet and at major changes in soil stratigraphy. The Crandall sampler is a 3-3/16 inch outside diameter, brass ring lined tube, that is driven with the kelly bar. Bulk samples of the upper soils were obtained to permit the performance of laboratory compaction and California Bearing Ratio tests. Standard penetration tests were performed in the rotary wash borings at depth intervals of approximately ten feet. Pitcher samples were taken in three of the borings.

The logs of the borings are presented on Figures A-1 through A-12; the depths at which undisturbed samples were obtained are indicted to the left of the boring logs. The soils are classified in accordance with the Unified Soil Classification System described on Figure A-13. An explanation of the information presented on the boring logs is presented on Figure A-14, Key to Log of Borings.

A.4 PIEZOMETER INSTALLATION

After the completion of drilling Borings 30 and 37, a 2-inch-diameter PVC pipe was installed in each boring for future monitoring of the ground water level.

The annulus between the pipe and boring walls was filled with gravel. The pipe was perforated along different depth increments. Each piezometer was developed by air-lifting. A summary of the piezometer installations and groundwater monitoring data is presented in Table A-1. Water levels are also presented on the borings logs.

Table A-1
Summary of Piezometer Installations and Groundwater Monitoring Data

Boring Number	Location (Station)	Depth of Pipe (Ft.)	Date Installed	Water Depth (Ft.) 9/6/85	Water Depth (Ft.) 9/17/85
30	--	60	8/17/85	29-1/2	29-1/2
37	25+60	60	8/15/85	NW	NW

NW = No water encountered

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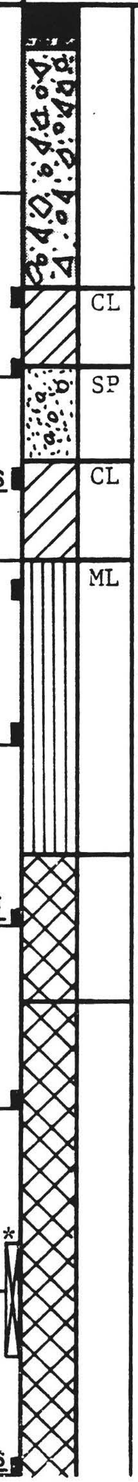
NOTE: THE LOG OF SUBSURFACE CONDITIONS SHOWN HEREON APPLIES ONLY AT THE SPECIFIC BORING LOCATION AND AT THE DATE INDICATED. IT IS NOT WARRANTED TO BE REPRESENTATIVE OF SUBSURFACE CONDITIONS AT OTHER LOCATIONS AND TIMES.

BORING 30

DATE DRILLED: August 17, 1985
EQUIPMENT USED: 5"-Diameter Rotary Wash

ELEVATION 285

ELEVATION (ft.)	DEPTH (ft.)	"N" VALUE STD. PEN. TEST	MOISTURE (% of dry wt.)	DRY DENSITY (lbs./cu. ft.)	DRIVE ENERGY (ft.-kips/ft.)	SAMPLE LOC.
280	5					
			31.7	88	4	
275	10		23.7	102	6	
			24.6	99	3	S
270	15					
			17.1	114	9	
265	20		18.1	111	6	
260	25		30.1	95	18	S
255	30		25.7	99	31	
250	35		PITCHER SAMPLE			*
245	40		24.3	97	48	S



12" Asphaltic Paving - 3" concrete Slab

FILL - TRASH (INCLUDING GRAVEL, ROCK, and PIECES OF BRICK)

SILTY CLAY - dark grey

SAND - fine to medium, about 30% gravel, grey

SILTY CLAY - dark grey

CLAYEY SILT - some Sand, brown

SILTSTONE - weathered, bedded, light brownish-grey

SILTSTONE - massive, dark grey

*INDICATES DEPTH OF PITCHER SAMPLE

Few cemented layers, some shells

(CONTINUED ON FOLLOWING PLATE)

LOG OF BORING

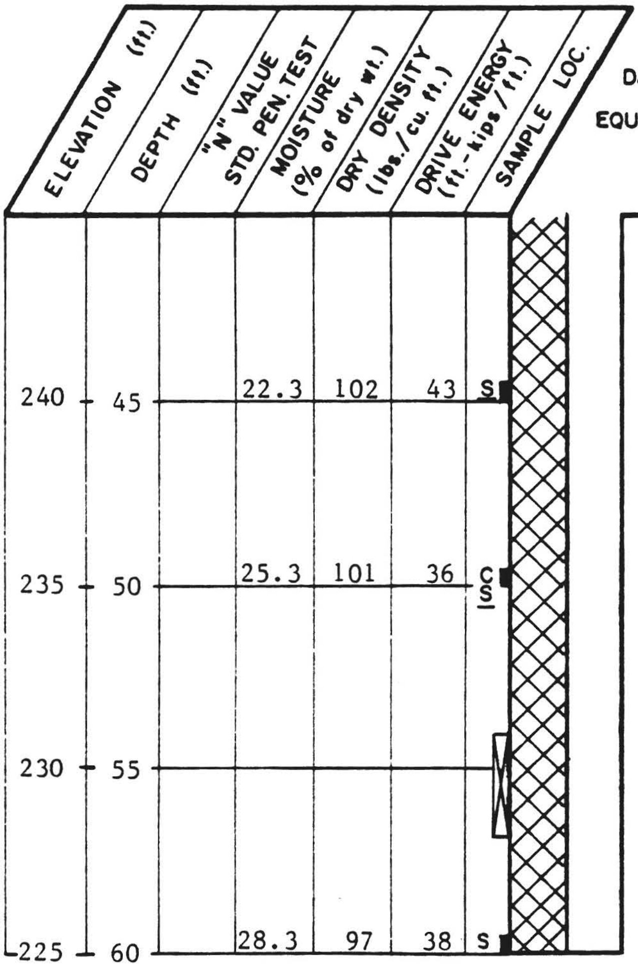
LeROY CRANDALL AND ASSOCIATES

FIGURE A-1a

BORING 30 (CONTINUED)

DATE DRILLED: August 17, 1985

EQUIPMENT USED: 5"-Diameter Rotary Wash



ELEVATION (ft.)	DEPTH (ft.)	"N" VALUE STD. PEN. TEST	MOISTURE (% of dry wt.)	DRY DENSITY (lbs./cu. ft.)	DRIVE ENERGY (ft.-kips/ft.)	SAMPLE LOC.
240	45		22.3	102	43	S
235	50		25.3	101	36	C S
230	55					
225	60		28.3	97	38	S

NOTE: Augered to 6½'. Installed 6½' of 6"-diameter steel casing. Drilling mud used in drilling process below 6½'. Mud removed. Installed 60' of 2"-diameter PVC pipe (perforated at 30' to 40' and 50' to 60'). Water level measured at 29½' on 9/17/85.

NOTE: THE LOG OF SUBSURFACE CONDITIONS SHOWN HEREON APPLIES ONLY AT THE SPECIFIC BORING LOCATION AND AT THE DATE INDICATED. IT IS NOT WARRANTED TO BE REPRESENTATIVE OF SUBSURFACE CONDITIONS AT OTHER LOCATIONS AND TIMES.

LOG OF BORING

LeROY CRANDALL AND ASSOCIATES

FIGURE A-1b

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JOB A-85005-8 DATE 8/28/85 DR. JOHN MS W.P. lmh CHKD

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NOTE: THE LOG OF SUBSURFACE CONDITIONS SHOWN HEREON APPLIES ONLY AT THE SPECIFIC BORING LOCATION AND AT THE DATE INDICATED. IT IS NOT WARRANTED TO BE REPRESENTATIVE OF SUBSURFACE CONDITIONS AT OTHER LOCATIONS AND TIMES.

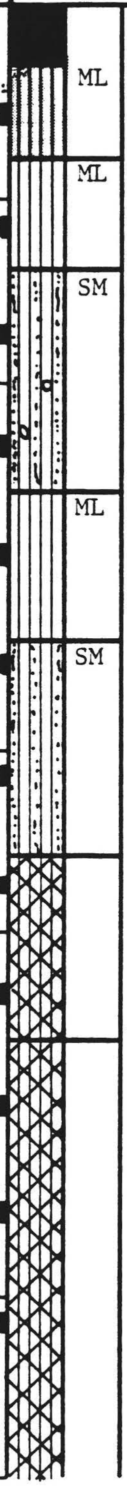
BORING 31

DATE DRILLED: August 24, 1985

EQUIPMENT USED: 24"-Diameter Bucket 0' to 28'
18"-Diameter Bucket below 28'

ELEVATION 284

ELEVATION (ft.)	DEPTH (ft.)	"N" VALUE STD. PEN. TEST	MOISTURE (% of dry wt.)	DRY DENSITY (lbs./cu. ft.)	DRIVE ENERGY (ft.-kips/ft.)	SAMPLE LOC.
			10.3	123	3	
280	5		13.9	118	10	
275			8.6	112	8	
	10		15.5	112	6	S
270	15		20.2	107	5	
265			14.0	108	6	
	20		9.6	120	10	
260			27.4	97	3	S
	25		27.6	96	10	S
255	30		30.0	94	8	
250			26.8	98	8	
	35		28.1	97	9	
245	40					



18" Asphaltic Paving

ML FILL - SANDY SILT - dark greyish-brown

ML SANDY SILT - brown

SM SILTY SAND - fine, few gravel, brown

ML SANDY SILT - brown
Light brownish-grey

SM SILTY SAND - fine, brown

SILTSTONE - weathered, bedded, light brownish-grey

(ENCOUNTERED ANCHOR TIE BACK, SWITCH TO 18"-DIAMETER BUCKET AT 28')

SILTSTONE - massive, dark grey

(ENCOUNTERED ANCHOR TIE BACK)

(CONTINUED ON FOLLOWING PLATE)

LOG OF BORING

LeROY CRANDALL AND ASSOCIATES

FIGURE A-2a

BORING 31 (CONTINUED)

DATE DRILLED: August 24, 1985

EQUIPMENT USED: 24"-Diameter Bucket 0' to 28'
18"-Diameter Bucket below 28'

ELEVATION (ft.)	DEPTH (ft.)	"N" VALUE STD. PEN. TEST	MOISTURE (% of dry wt.)	DRY DENSITY (lbs./cu. ft.)	DRIVE ENERGY (ft.-kips/ft.)	SAMPLE LOC.
			27.4	97	8	
240	45					
			25.4	99	9	S
235	50					
			24.6	100	8	S
230	55		26.8	98	10	C
225	60		26.6	99	12	

Some shells

NOTE: Water not encountered. No caving.
Tie-back anchors from ARCO Plaza construction encountered during drilling.

NOTE: THE LOG OF SUBSURFACE CONDITIONS SHOWN HEREON APPLIES ONLY AT THE SPECIFIC BORING LOCATION AND AT THE DATE INDICATED. IT IS NOT WARRANTED TO BE REPRESENTATIVE OF SUBSURFACE CONDITIONS AT OTHER LOCATIONS AND TIMES.

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LOG OF BORING

LeROY CRANDALL AND ASSOCIATES

FIGURE A-2b

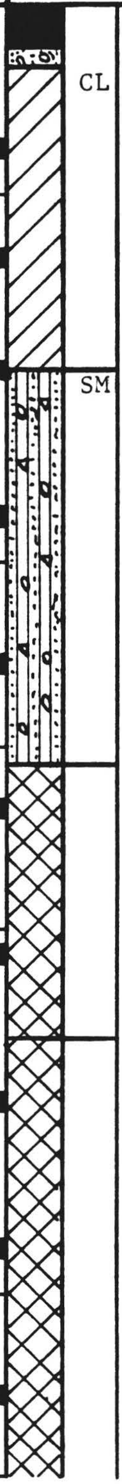
BORING 32

DATE DRILLED: August 17, 1985

EQUIPMENT USED: 24"-Diameter Bucket

ELEVATION 277

ELEVATION (ft.)	DEPTH (ft.)	"N" VALUE STD. PEN. TEST	MOISTURE (% of dry wt.)	DRY DENSITY (lbs./cu. ft.)	DRIVE ENERGY (ft.-kips/ft.)	SAMPLE LOC.
275						
	5		17.1	113	5	■
270			16.0	116	11	■
	10		11.3	113	8	■
265						
			6.6	130	24	S■
	15					
260			9.5	132	18	■
	20					
255			28.9	100	5	■
	25		27.8	97	3	■
250						
	30		28.7	95	8	■
245			28.7	96	7	■
	35					
240			26.3	96	6	S■
	40					



12" Asphaltic Paving - 6" Base Course

SILTY CLAY - brown

SILTY SAND - fine, some gravel, brown

About 15% gravel and cobbles

Fine to medium Sand

SILTSTONE - weathered, bedded, light brownish-grey

SILTSTONE - massive, dark grey

(CONTINUED ON FOLLOWING PLATE)

LOG OF BORING

NOTE: THE LOG OF SUBSURFACE CONDITIONS SHOWN HEREON APPLIES ONLY AT THE SPECIFIC BORING LOCATION AND AT THE DATE INDICATED. IT IS NOT WARRANTED TO BE REPRESENTATIVE OF SUBSURFACE CONDITIONS AT OTHER LOCATIONS AND TIMES.

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FIGURE A-3a

BORING 32 (CONTINUED)

DATE DRILLED: August 17, 1985

EQUIPMENT USED: 24"-Diameter Bucket

ELEVATION (ft.)	DEPTH (ft.)	"N" VALUE STD. PEN. TEST	MOISTURE (% of dry wt.)	DRY DENSITY (lbs./cu. ft.)	DRIVE ENERGY (ft.-kips/ft.)	SAMPLE LOC.
235			29.3	95	5	C ■
	45		25.9	101	6	S ■
230			25.6	101	6	■
	50		25.4	99	7	■
225						
	55		25.3	100	6	■
220						
	60		25.8	100	10	S ■
215						
	65					

NOTE: Water not encountered. Slight raveling from 9½' to 21'.

NOTE: THE LOG OF SUBSURFACE CONDITIONS SHOWN HEREON APPLIES ONLY AT THE SPECIFIC BORING LOCATION AND AT THE DATE INDICATED. IT IS NOT WARRANTED TO BE REPRESENTATIVE OF SUBSURFACE CONDITIONS AT OTHER LOCATIONS AND TIMES.

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LOG OF BORING

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FIGURE A-3b

BORING 33

DATE DRILLED: August 17, 1985

EQUIPMENT USED: 24"-Diameter Bucket

ELEVATION 269

ELEVATION (ft.)	DEPTH (ft.)	"N" VALUE STD. PEN. TEST	MOISTURE (% of dry wt.)	DRY DENSITY (lbs./cu. ft.)	DRIVE ENERGY (ft.-kips/ft.)	SAMPLE LOC.		
								12" Asphaltic Paving
							CL	SILTY CLAY - brown
			15.6	114	5	S		
265								
	5							
			16.1	115	6			
260								
	10		16.7	113	6			
			16.0	115	8		SC	CLAYEY SAND - fine, about 10% gravel, brown
255								
	15						SP	SAND - fine to medium, few gravel, brown
			5.7	117	19			
								Few cobbles
			4.5	132	32		SW	SAND - well graded, about 20% gravel and cobbles (to 10" in size), brown
250								
	20							
								Few boulders (to 16" in size)
245								
	25							
			7.3	115	24			
							ML	SANDY SILT - few gravel, light greyish-brown
			16.6	101	16	S		
240								
	30							
			17.0	101	11	CS		
							SM	SILTY SAND - fine, few gravel, light greyish-brown
235								
	35		22.2	103	19			
								SILTSTONE - weathered, bedded, light brownish-grey
			25.5	100	8			
								SILTSTONE - massive, dark grey
230								
	40							

(CONTINUED ON FOLLOWING PLATE)

LOG OF BORING

NOTE: THE LOG OF SUBSURFACE CONDITIONS SHOWN HEREON APPLIES ONLY AT THE SPECIFIC BORING LOCATION AND AT THE DATE INDICATED. IT IS NOT WARRANTED TO BE REPRESENTATIVE OF SUBSURFACE CONDITIONS AT OTHER LOCATIONS AND TIMES.

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LeROY CRANDALL AND ASSOCIATES

FIGURE A-4a

BORING 33 (CONTINUED)

DATE DRILLED: August 17, 1985

EQUIPMENT USED: 24"-Diameter Bucket

ELEVATION (ft.)	DEPTH (ft.)	"N" VALUE STD. PEN. TEST	MOISTURE (% of dry wt.)	DRY DENSITY (lbs./cu. ft.)	DRIVE ENERGY (ft.-kips/ft.)	SAMPLE LOC.
			25.3	101	8	
225						
	45		27.2	99	10	
220						
	50		26.1	100	10	
215						
	55		24.8	101	12	
210			25.0	101	14	S
	60					

Some shells

NOTE: Water not encountered. Raveling from 16' to 25' (to 3' in diameter).

NOTE: THE LOG OF SUBSURFACE CONDITIONS SHOWN HEREON APPLIES ONLY AT THE SPECIFIC BORING LOCATION AND AT THE DATE INDICATED. IT IS NOT WARRANTED TO BE REPRESENTATIVE OF SUBSURFACE CONDITIONS AT OTHER LOCATIONS AND TIMES.

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LOG OF BORING

LeROY CRANDALL AND ASSOCIATES

FIGURE A-4b

BORING 34

DATE DRILLED: August 14, 1985

EQUIPMENT USED: 5"-Diameter Rotary Wash

ELEVATION 263

ELEVATION (ft.)	DEPTH (ft.)	"N" VALUE STD. PEN. TEST	MOISTURE (% of dry wt.)	DRY DENSITY (lbs./cu. ft.)	DRIVE ENERGY (ft.-kips/ft.)	SAMPLE LOC.
260			15.7	110	14	▮
	5		13.6	118	13	▮
			17.6	111	12	▮
255			19.8	104	8	▮
	10		22.3	102	6	S▮
250						
	15		10.4	129	27	▮
245						
	20		15.9	113	48	▮
240						
	25		18.7	107	48	S▮
235						
	30					
230						
	35		10.7	123	72	▮
225						
	40		12.9	123	48	S▮

	12" Asphaltic Paving
ML	CLAYEY SILT - brown
ML	SANDY SILT - some Clay, brown
ML	CLAYEY SILT - some Sand, brown
SW	SAND - well graded, about 40% gravel and cobbles, greyish-brown
	Layer of Clayey Sand
SP	SAND - fine to medium, few gravel, light brown
SW	SAND - well graded, about 40% of gravel and cobbles, light brown
	Some boulders

(CONTINUED ON FOLLOWING PLATE)

LOG OF BORING

LeROY CRANDALL AND ASSOCIATES

FIGURE A-5a

NOTE: THE LOG OF SUBSURFACE CONDITIONS SHOWN HEREON APPLIES ONLY AT THE SPECIFIC BORING LOCATION AND AT THE DATE INDICATED. IT IS NOT WARRANTED TO BE REPRESENTATIVE OF SUBSURFACE CONDITIONS AT OTHER LOCATIONS AND TIMES.

Form 123 JOB A-85005-8 DATE 8/27/85 DR. JOHN MS W.P. dmh CHKD

BORING 34 (CONTINUED)

DATE DRILLED: August 14, 1985
EQUIPMENT USED: 5"-Diameter Rotary Wash

ELEVATION (ft.)	DEPTH (ft.)	"N" VALUE STD. PEN. TEST	MOISTURE (% of dry wt.)	DRY DENSITY (lbs./cu. ft.)	DRIVE ENERGY (ft.-kips/ft.)	SAMPLE LOC.
220						
	45		16.8	109	48	C S
215						
	50		19.4	98	48	
210						
	55		11.4	109	48	S
205						
	60		10.8	115	72	
200						
	65		18.2	115	48	
195						
	70		17.1	109	48	
190						
	75					
185						
	80					

ML SANDY SILT - some Clay, greyish-brown

SP SAND - fine, light brown

Fine to medium

SW SAND - well graded, about 30% gravel and cobbles, light brown

Layer of Silt

SM SILTY SAND - fine, light brown
Layer of Sand

ML CLAYEY SILT - light grey

NOTE: Augered to 6½'. Installed 6½' of 6"-diameter steel casing. Drilling mud used in drilling process below 6½'. Water level not established.

SW SAND - well graded, large amount of gravel, light brown

NOTE: THE LOG OF SUBSURFACE CONDITIONS SHOWN HEREON APPLIES ONLY AT THE SPECIFIC BORING LOCATION AND AT THE DATE INDICATED. IT IS NOT WARRANTED TO BE REPRESENTATIVE OF SUBSURFACE CONDITIONS AT OTHER LOCATIONS AND TIMES.

JOB A-85005-8 DATE 8/27/85 DR. JOHN W.P. mh CHKD LL

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LOG OF BORING

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Form 124

FIGURE A-5b

BORING 35

DATE DRILLED: August 22, 1985
EQUIPMENT USED: 24"-Diameter Bucket

ELEVATION (ft.)	DEPTH (ft.)	"N" VALUE STD. PEN. TEST	MOISTURE (% of dry wt.)	DRY DENSITY (lbs./cu. ft.)	DRIVE ENERGY (ft.-kips/ft.)	SAMPLE LOC.		ELEVATION 256
255							SC	2" Asphaltic Paving - 6" Concrete Slab FILL - CLAYEY SAND - fine, about 10% gravel, grey
			18.6	112	3		CL	SANDY CLAY - some Silt, dark grey
250	5		16.8	113	5	S		Brown
			15.7	113	11		SC	CLAYEY SAND - fine, about 10% gravel and cobbles, brown
245	10							About 20% gravel and cobbles (to 10" in size), 16" boulder
							SW	SAND - well graded, about 40% gravel, cobbles and boulders (to 24" in size), light brown
240	15		4.1	112	19			
235	20		6.1	128	37			
			16.0	116	10	S	CL	SANDY CLAY - few gravel, brown
230	25		16.5	111	6	SC	ML	SANDY SILT - some Clay, brown
	30		19.6	108	8	S		
225			15.2	117	8			
								Few boulders
220	35		15.3	117	10			Dark brown
			18.0	112	10	C		
	40							

(CONTINUED ON FOLLOWING PLATE)

LOG OF BORING

NOTE: THE LOG OF SUBSURFACE CONDITIONS SHOWN HEREON APPLIES ONLY AT THE SPECIFIC BORING LOCATION AND AT THE DATE INDICATED. IT IS NOT WARRANTED TO BE REPRESENTATIVE OF SUBSURFACE CONDITIONS AT OTHER LOCATIONS AND TIMES.

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FIGURE A-6a

BORING 35 (CONTINUED)

DATE DRILLED: August 22, 1985

EQUIPMENT USED: 24"-Diameter Bucket

ELEVATION (ft.)	DEPTH (ft.)	"N" VALUE STD. PEN. TEST	MOISTURE (% of dry wt.)	DRY DENSITY (lbs./cu. ft.)	DRIVE ENERGY (ft.-kips/ft.)	SAMPLE LOC.		
215							SP	SAND - fine, about 10% gravel, brown
			5.4	107	14	■		
			5.4	115	24	■	SW	SAND - well graded, about 15% gravel, light brown
	45							
210								
			6.0	115	20	■		
	50							
			8.5	122	36	■		
205								
								About 40% gravel
								About 10% gravel
			8.9	120	30	■		
	55							
200								Layer of gravel
	60		7.0	118	25	■		

NOTE: Water not encountered. Heavy caving and raveling from 6' to 25' (to 4' in diameter) and slight raveling below 45' (to 2½' in diameter).

NOTE: THE LOG OF SUBSURFACE CONDITIONS SHOWN HEREON APPLIES ONLY AT THE SPECIFIC BORING LOCATION AND AT THE DATE INDICATED. IT IS NOT WARRANTED TO BE REPRESENTATIVE OF SUBSURFACE CONDITIONS AT OTHER LOCATIONS AND TIMES.

Form 124 JOB A-85005-8 DATE 8/27/85 DR. JOHN MS W.P. mh CHKD

LOG OF BORING

LeROY CRANDALL AND ASSOCIATES

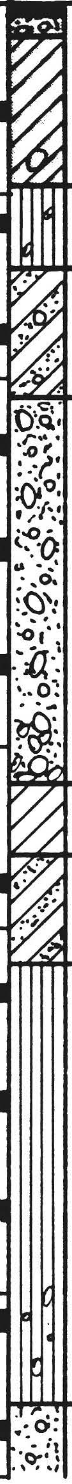
FIGURE A-6b

BORING 36

DATE DRILLED: August 23, 1985

EQUIPMENT USED: 24"-Diameter Bucket

ELEVATION 253



ELEVATION (ft.)	DEPTH (ft.)	"N" VALUE STD. PEN. TEST	MOISTURE (% of dry wt.)	DRY DENSITY (lbs./cu.ft.)	DRIVE ENERGY (ft.-kips/ft.)	SAMPLE LOC.		
								4" Asphaltic Paving - 4" Concrete Slab
							CL	FILL - SANDY CLAY - dark grey
250			20.6	104	2			Brown (ENCOUNTERED ABANDONED 8"-DIAMETER CLAY PIPE AT 4½')
	5							
			19.7	111	3		ML	FILL - SANDY SILT - few gravel, dark greyish-brown
245							SC	CLAYEY SAND - fine, some gravel and cobbles, brown
			11.6	108	11			Cobbles (to 6" in size)
	10							
			2.7	115	32		SW	SAND - well graded, about 10% gravel and cobbles (to 6" in size), greyish-brown
240								
			4.7	128	32			Thin layers of gravel
	15							
235			4.9	126	16			About 30% gravel and cobbles (to 6" in size) Cobbles (to 8" in size) Layer of cobbles (to 10" in size)
	20							
							CL	SILTY CLAY - brown
230								
			10.4	102	10		SC	CLAYEY SAND - fine, brown
	25							
			16.8	112	10	S	ML	SANDY SILT - brown
225								
	30		16.0	113	16	C		
220			13.6	119	13			
	35							
			14.5	116	6	S		Some gravel and cobbles
215			7.5	123	19		SP	SAND - fine to medium, about 10% gravel, brown
	40							

(CONTINUED ON FOLLOWING PLATE)

LOG OF BORING

LeROY CRANDALL AND ASSOCIATES

NOTE: THE LOG OF SUBSURFACE CONDITIONS SHOWN HEREON APPLIES ONLY AT THE SPECIFIC BORING LOCATION AND AT THE DATE INDICATED. IT IS NOT WARRANTED TO BE REPRESENTATIVE OF SUBSURFACE CONDITIONS AT OTHER LOCATIONS AND TIMES.

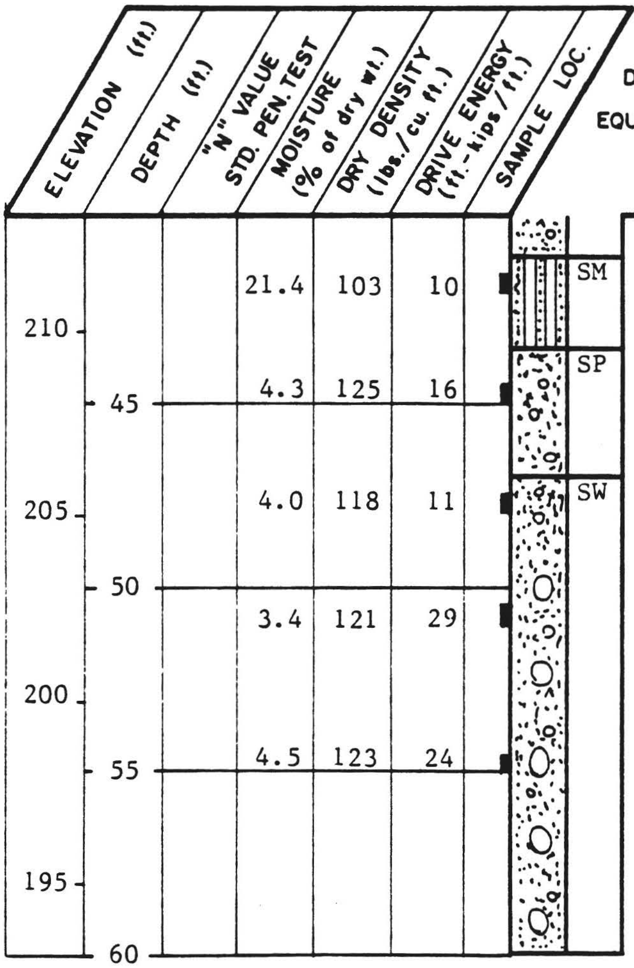
Form 123 JOB A-85005-8 DATE 8/28/85 DR. JOHN MS W.P. dmh CHKD

FIGURE A-7a

BORING 36 (CONTINUED)

DATE DRILLED: August 23, 1985

EQUIPMENT USED: 24"-Diameter Bucket



"N" VALUE STD. PEN. TEST	MOISTURE (% of dry wt.)	DRY DENSITY (lbs./cu. ft.)	DRIVE ENERGY (ft.-kips/ft.)
	21.4	103	10
	4.3	125	16
	4.0	118	11
	3.4	121	29
	4.5	123	24

SM	SILTY SAND - fine, light greyish-brown
SP	SAND - fine, brown
SW	SAND - well graded, about 10% gravel, light brown Thin layer of gravel About 20% gravel

NOTE: Water not encountered. Heavy caving and raveling from 9' to 21' (to 3' in diameter).

NOTE: THE LOG OF SUBSURFACE CONDITIONS SHOWN HEREON APPLIES ONLY AT THE SPECIFIC BORING LOCATION AND AT THE DATE INDICATED. IT IS NOT WARRANTED TO BE REPRESENTATIVE OF SUBSURFACE CONDITIONS AT OTHER LOCATIONS AND TIMES.

Form 124 JOB A-85005-8 DATE 8/28/85 DR JOHN W.P. CHKD

LOG OF BORING

LeROY CRANDALL AND ASSOCIATES

FIGURE A-7b

BORING 38

DATE DRILLED: August 20, 1985

EQUIPMENT USED: 24"-Diameter Bucket

ELEVATION 245

ELEVATION (ft.)	DEPTH (ft.)	"N" VALUE STD. PEN. TEST	MOISTURE (% of dry wt.)	DRY DENSITY (lbs./cu. ft.)	DRIVE ENERGY (ft.-kips/ft.)	SAMPLE LOC.		
								12" Asphaltic Paving - 4" Concrete Slab
			23.6	102	2		CL	SILTY CLAY - some gravel, brown
							SC	CLAYEY SAND - fine to medium, brown
240	5							
			7.9	114	19		SW	SAND - well graded, about 20% gravel and cobbles, brown
			4.9	127	19			
235	10							
			7.3	129	24			12" boulders
230	15		27.6	96	8		ML	CLAYEY SILT - some organic matter, brown
			16.0	115	6	S	ML	SANDY SILT - some organic matter, brown
225	20							
			14.4	119	13			
			12.3	122	8		SC	CLAYEY SAND - fine, brown
220	25							
			16.1	115	16		SM	SILTY SAND - fine, about 10% gravel, brown
215	30		18.5	110	8	SC	ML	SANDY SILT - lenses of tar, strong gasoline odor, dark brown
			12.0	113	12		SC	CLAYEY SAND - fine, light brown
210	35						SM	SILTY SAND - fine, light brown
			8.0	113	10	S		
			9.9	110	14	C		
205	40							

(CONTINUED ON FOLLOWING PLATE)

LOG OF BORING

NOTE: THE LOG OF SUBSURFACE CONDITIONS SHOWN HEREON APPLIES ONLY AT THE SPECIFIC BORING LOCATION AND AT THE DATE INDICATED. IT IS NOT WARRANTED TO BE REPRESENTATIVE OF SUBSURFACE CONDITIONS AT OTHER LOCATIONS AND TIMES.

JOB A-85005-8 DATE 8 / 27/85 DR. JOHN MS W.P. mh CHKD

Form 123

LeROY CRANDALL AND ASSOCIATES

FIGURE A-9a

BORING 38 (CONTINUED)

DATE DRILLED: August 20, 1985
EQUIPMENT USED: 24"-Diameter Bucket

ELEVATION (ft.)	DEPTH (ft.)	"N" VALUE STD. PEN. TEST	MOISTURE (% of dry wt.)	DRY DENSITY (lbs./cu. ft.)	DRIVE ENERGY (ft.-kips/ft.)	SAMPLE LOC.		
			19.3	110	8		CL	SILTY CLAY - dark brown
200	45		14.5	114	12		ML	SANDY SILT - some Clay, brown
			14.0	117	11			
195	50							
			15.9	116	10			
190	55		6.1	122	10			
185	60		12.4	126	18	S		

NOTE: Water not encountered. Raveling from 3' to 15' to (36" in diameter).

NOTE: THE LOG OF SUBSURFACE CONDITIONS SHOWN HEREON APPLIES ONLY AT THE SPECIFIC BORING LOCATION AND AT THE DATE INDICATED. IT IS NOT WARRANTED TO BE REPRESENTATIVE OF SUBSURFACE CONDITIONS AT OTHER LOCATIONS AND TIMES.

Form 124 JOB A-85005-8 DATE 8/27/85 DR. JOHN MS W.P. dmh CHKD

LOG OF BORING

LeROY CRANDALL AND ASSOCIATES

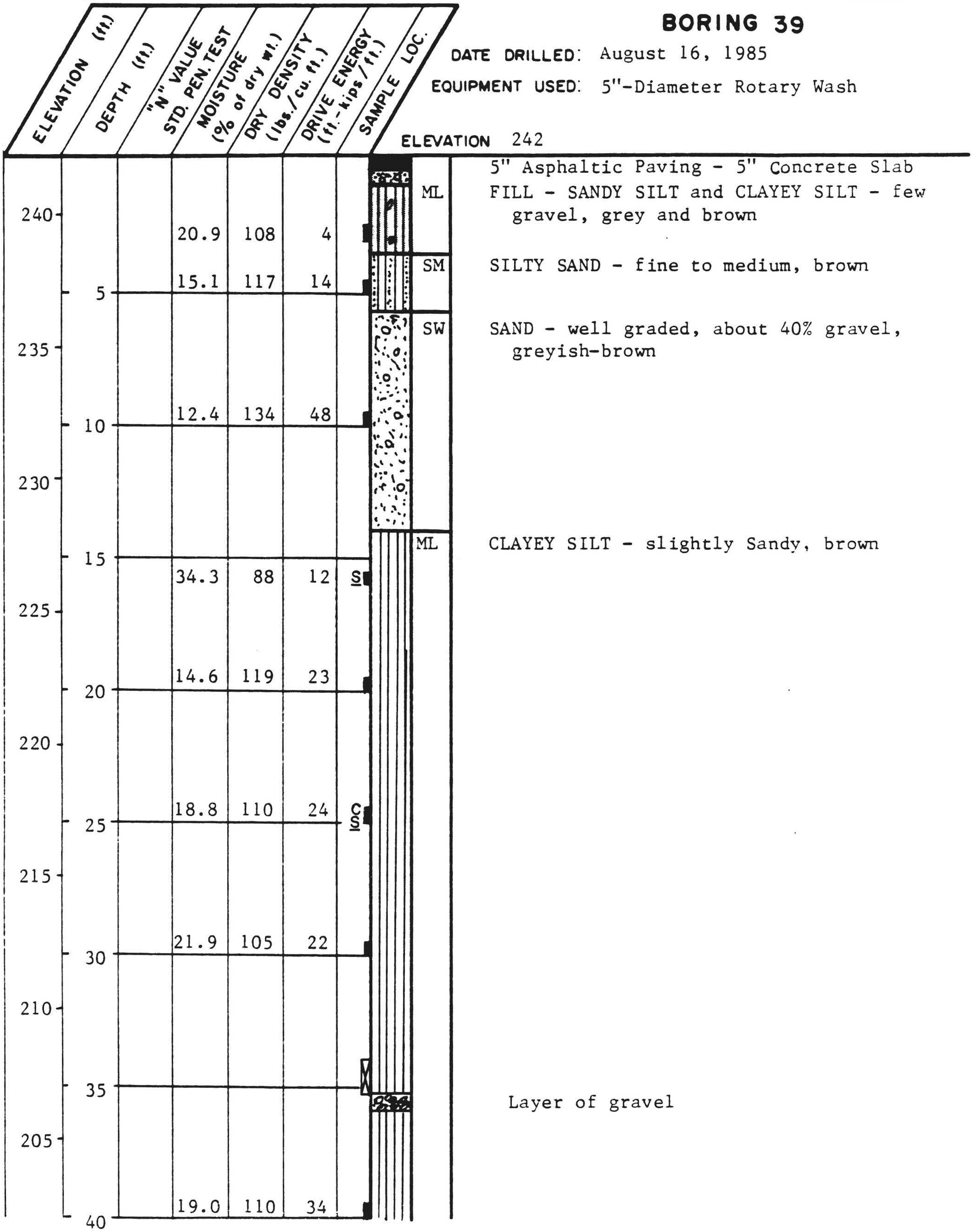
FIGURE A-9b

BORING 39

DATE DRILLED: August 16, 1985

EQUIPMENT USED: 5"-Diameter Rotary Wash

ELEVATION 242



"N" VALUE STD. PEN. TEST	MOISTURE (% of dry wt.)	DRY DENSITY (lbs./cu. ft.)	DRIVE ENERGY (ft.-kips/ft.)	SAMPLE LOC.
	20.9	108	4	
	15.1	117	14	
	12.4	134	48	
	34.3	88	12	S
	14.6	119	23	
	18.8	110	24	CS
	21.9	105	22	
	19.0	110	34	

5" Asphaltic Paving - 5" Concrete Slab

ML FILL - SANDY SILT and CLAYEY SILT - few gravel, grey and brown

SM SILTY SAND - fine to medium, brown

SW SAND - well graded, about 40% gravel, greyish-brown

ML CLAYEY SILT - slightly Sandy, brown

Layer of gravel

(CONTINUED ON FOLLOWING PLATE)

NOTE: THE LOG OF SUBSURFACE CONDITIONS SHOWN HEREON APPLIES ONLY AT THE SPECIFIC BORING LOCATION AND AT THE DATE INDICATED. IT IS NOT WARRANTED TO BE REPRESENTATIVE OF SUBSURFACE CONDITIONS AT OTHER LOCATIONS AND TIMES.

JOB A-85005-8 DATE 8/27/85 DR. JOHN MS W.P. mh CHKD

Form 123

LOG OF BORING

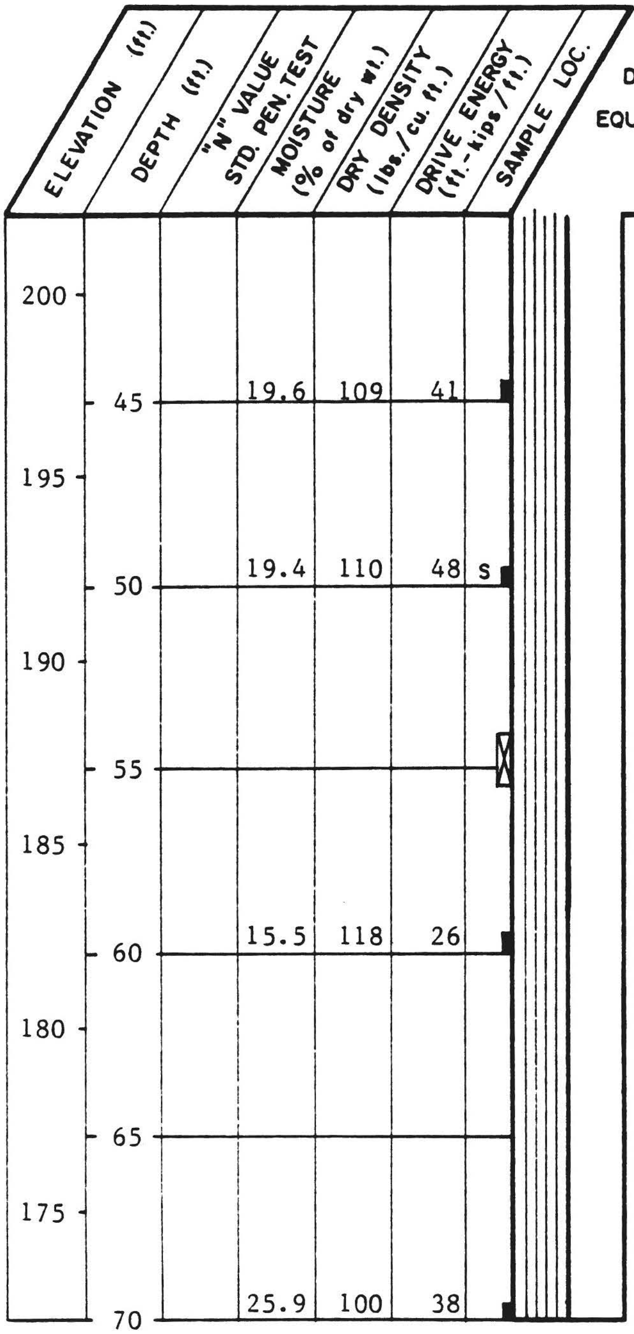
LeROY CRANDALL AND ASSOCIATES

FIGURE A-10a

BORING 39 (CONTINUED)

DATE DRILLED: August 16, 1985

EQUIPMENT USED: 5"-Diameter Rotary Wash



ELEVATION (ft.)	DEPTH (ft.)	"N" VALUE STD. PEN. TEST	MOISTURE (% of dry wt.)	DRY DENSITY (lbs./cu. ft.)	DRIVE ENERGY (ft.-kips/ft.)	SAMPLE LOC.
	45		19.6	109	41	
	50		19.4	110	48	s
	55					
	60		15.5	118	26	
	65					
	70		25.9	100	38	

NOTE: Augered to 6½'. Installed 6½' of 6"-diameter steel casing. Drilling mud used in drilling process below 6½'. Water level not established.

NOTE: THE LOG OF SUBSURFACE CONDITIONS SHOWN HEREON APPLIES ONLY AT THE SPECIFIC BORING LOCATION AND AT THE DATE INDICATED. IT IS NOT WARRANTED TO BE REPRESENTATIVE OF SUBSURFACE CONDITIONS AT OTHER LOCATIONS AND TIMES.

LOG OF BORING

LeROY CRANDALL AND ASSOCIATES

Form 124 JOB A-85005-8 DATE 8/27/85 DR. JOHN IS W.P. dmh CHKD

FIGURE A-10b

BORING 40

DATE DRILLED: August 19, 1985

EQUIPMENT USED: 24"-Diameter Bucket 0' to 31'
18"-Diameter Bucket below 31'

ELEVATION 238

ELEVATION (ft.)	DEPTH (ft.)	"N" VALUE STD. PEN. TEST	MOISTURE (% of dry wt.)	DRY DENSITY (lbs./cu. ft.)	DRIVE ENERGY (ft.-kips/ft.)	SAMPLE LOC.		
								8" Asphaltic Paving
							SC	CLAYEY SAND - fine to medium, about 10% gravel and cobbles, brown
235								
			2.6	118	10	S		Cobbles (to 10" in size)
	5							
			6.2	113	10		SW	SAND - well graded, about 20% gravel and cobbles (to 10" in size), brown
230								
			5.5	134	26			
	10							
							ML	CLAYEY SILT - brown
			34.0	89	8		CL	SILTY CLAY - brown
225								
	15							
			17.8	114	6	C S		
220								
			18.0	114	8			
	20							
			20.2	112	5	S		
215							ML	CLAYEY SILT - some Sand, brown
			18.4	111	6	C		
	25							
							ML	SANDY SILT - some Clay, few gravel, brown
210			15.8	116	14			
	30							
			11.1	125	11		SC	CLAYEY SAND - fine to medium, brown
205								
			11.7	122	10			
	35							
								About 10% gravel
			9.9	117	14	S		
							CL	SANDY CLAY - brown
200								
			21.0	111	10			
	40							

(CONTINUED ON FOLLOWING PLATE)

LOG OF BORING

LeROY CRANDALL AND ASSOCIATES

FIGURE A-11a

NOTE: THE LOG OF SUBSURFACE CONDITIONS SHOWN HEREON APPLIES ONLY AT THE SPECIFIC BORING LOCATION AND AT THE DATE INDICATED. IT IS NOT WARRANTED TO BE REPRESENTATIVE OF SUBSURFACE CONDITIONS AT OTHER LOCATIONS AND TIMES.

Form 123 JOB A-85005-8 DATE 8/27/85 DR. JOHN :S W.P. dmh CHKD

BORING 40 (CONTINUED)

DATE DRILLED: August 19, 1985

EQUIPMENT USED: 24"-Diameter Bucket 0' to 31'
18"-Diameter Bucket below 31'

ELEVATION (ft.)	DEPTH (ft.)	"N" VALUE STD. PEN. TEST	MOISTURE (% of dry wt.)	DRY DENSITY (lbs./cu. ft.)	DRIVE ENERGY (ft.-kips/ft.)	SAMPLE LOC.
195			15.0	118	6	■
	45					
			17.2	117	8	■
190						
			17.2	115	11	■
	50					
			16.5	114	17	■
185						
			15.8	117	12	■
	55					
180						
			19.6	111	10	■
	60					

ML SANDY SILT - brown

SM SILTY SAND - fine, brown

ML SANDY SILT - slightly Clayey, some organic matter, brown

NOTE: Water not encountered. Raveling from 3' to 12' (to 3' in diameter).

NOTE: THE LOG OF SUBSURFACE CONDITIONS SHOWN HEREON APPLIES ONLY AT THE SPECIFIC BORING LOCATION AND AT THE DATE INDICATED. IT IS NOT WARRANTED TO BE REPRESENTATIVE OF SUBSURFACE CONDITIONS AT OTHER LOCATIONS AND TIMES.

Form 124 JOB A-85005-8 DATE 8/27/85 DR JOHN MS W.P. dmh CHKD

LOG OF BORING

LeROY CRANDALL AND ASSOCIATES

FIGURE A-11b

BORING 41

DATE DRILLED: August 16, 1985

EQUIPMENT USED: 24"-Diameter Bucket

ELEVATION 239

ELEVATION (ft.)	DEPTH (ft.)	"N" VALUE STD. PEN. TEST	MOISTURE (% of dry wt.)	DRY DENSITY (lbs./cu. ft.)	DRIVE ENERGY (ft.-kips/ft.)	SAMPLE LOC.
			6.4	128	19	
235	5		2.9	127	19	S
			4.5	126	29	S
230	10		7.7	128	16	
			4.4	129	19	
225	15		6.9	124	26	S
			6.5	131	48	
220	20		9.7	126	19	
215	25		9.3	123	19	S
			15.1	117	12	
210	30		6.4	109	16	
			23.9	102	14	
205	35					

10" Asphaltic Paving - 2" Base Course

SC FILL - CLAYEY SAND - fine, some gravel and cobbles, brown

(ENCOUNTERED 4"-DIAMETER STEEL PIPE AT 3', BORING MOVED 1/2').

SW SAND - well graded, about 20% gravel and cobbles (to 10" in size), light brown

Cobbles (to 12" in size)

About 30% gravel and cobbles (to 8" in size)

SC CLAYEY SAND - fine to medium, some gravel, light brown

SP SAND - fine to medium, about 10% gravel, light brown

CL SILTY CLAY - light brown

(CONTINUED ON FOLLOWING PLATE)

NOTE: THE LOG OF SUBSURFACE CONDITIONS SHOWN HEREON APPLIES ONLY AT THE SPECIFIC BORING LOCATION AND AT THE DATE INDICATED. IT IS NOT WARRANTED TO BE REPRESENTATIVE OF SUBSURFACE CONDITIONS AT OTHER LOCATIONS AND TIMES.

LOG OF BORING

LeROY CRANDALL AND ASSOCIATES

FIGURE A-12a

Form 123 JOB A-85005-8 DATE 8/28/85 DR. JOHN W.P. dmh CHKD

BORING 41 (CONTINUED)

DATE DRILLED: August 16, 1985

EQUIPMENT USED: 24"-Diameter Bucket

ELEVATION (ft.)	DEPTH (ft.)	"N" VALUE STD. PEN. TEST	MOISTURE (% of dry wt.)	DRY DENSITY (lbs./cu. ft.)	DRIVE ENERGY (ft.-kips/ft.)	SAMPLE LOC.		
			16.1	116	10	■	ML	SANDY SILT - some Clay, brown
200			18.0	111	12	■		
	40						CL	SANDY CLAY - light brown
195								
	45		21.4	106	15	■	ML	SANDY SILT - some Clay, traces of organic matter, brown
							SM	SILTY SAND - fine to medium, light brown
190								
	50		16.8	117	19	■	ML	SANDY SILT - brown
								Layer of Clayey Silt, dark brown
185								
	55		16.2	115	10	■		
								Layer of Clayey Sand
180								
	60		20.2	109	13	■	SM	SILTY SAND - fine, light brown

NOTE: Water not encountered. Raveling from 0' to 20' (to 3' in diameter).

NOTE: THE LOG OF SUBSURFACE CONDITIONS SHOWN HEREON APPLIES ONLY AT THE SPECIFIC BORING LOCATION AND AT THE DATE INDICATED. IT IS NOT WARRANTED TO BE REPRESENTATIVE OF SUBSURFACE CONDITIONS AT OTHER LOCATIONS AND TIMES.

JOB A-85005-8 DATE 8/28/85 DR JOHN MS W.P. dmh CHKD DH

Form 124

LOG OF BORING

LeROY CRANDALL AND ASSOCIATES

FIGURE A-12b

MAJOR DIVISIONS			GROUP SYMBOLS	TYPICAL NAMES
COARSE GRAINED SOILS (More than 50% of material is LARGER than No. 200 sieve size)	GRAVELS (More than 50% of coarse fraction is LARGER than the No. 4 sieve size)	CLEAN GRAVELS (Little or no fines)	GW	Well graded gravels, gravel-sand mixtures, little or no fines.
			GP	Poorly graded gravels or gravel-sand mixtures, little or no fines.
		GRAVELS WITH FINES (Appreciable amt. of fines)	GM	Silty gravels, gravel-sand-silt mixtures.
			GC	Clayey gravels, gravel-sand-clay mixtures.
	SANDS (More than 50% of coarse fraction is SMALLER than the No. 4 sieve size)	CLEAN SANDS (Little or no fines)	SW	Well graded sands, gravelly sands, little or no fines.
			SP	Poorly graded sands or gravelly sands, little or no fines.
		SANDS WITH FINES (Appreciable amt. of fines)	SM	Silty sands, sand-silt mixtures.
			SC	Clayey sands, sand-clay mixtures.
FINE GRAINED SOILS (More than 50% of material is SMALLER than No. 200 sieve size)	SILTS AND CLAYS (Liquid limit LESS than 50)		ML	Inorganic silts and very fine sands, rock flour, silty or clayey fine sands or clayey silts with slight plasticity.
			CL	Inorganic clays of low to medium plasticity, gravelly clays, sandy clays, silty clays, lean clays.
			OL	Organic silts and organic silty clays of low plasticity.
	SILTS AND CLAYS (Liquid limit GREATER than 50)		MH	Inorganic silts, micaceous or diatomaceous fine sandy or silty soils, elastic silts.
			CH	Inorganic clays of high plasticity, fat clays.
			OH	Organic clays of medium to high plasticity, organic silts.
HIGHLY ORGANIC SOILS			Pt	Peat and other highly organic soils.

BOUNDARY CLASSIFICATIONS: Soils possessing characteristics of two groups are designated by combinations of group symbols.

PARTICLE SIZE LIMITS

SILT OR CLAY	SAND			GRAVEL		COBBLES	BOULDERS
	FINE	MEDIUM	COARSE	FINE	COARSE		

NO. 200 NO. 40 NO. 10 NO. 4 ¾ in. 3 in. (12 in.)

U. S. STANDARD SIEVE SIZE

UNIFIED SOIL CLASSIFICATION SYSTEM

Reference :
The Unified Soil Classification System, Corps of Engineers, U. S. Army Technical Memorandum No. 3-357, Vol. I, March, 1953. (Revised April, 1960)

LeRoy Crandall and Associates

FIGURE A-13

LC&A SAMPLING: (Sampler Diameter - I.D. = 2.625"; O.D. = 3.188")

12 C S S ▮ Depth at which undisturbed sample taken

S — Direct Shear Test

S — Double Direct Shear Test

C — Consolidation Test

12 — Energy required to drive LC&A sampler 12", in ft. - kips per ft.

BUCKET BORINGS:

Depth Increment	Driving Weight	Stroke
0' to 25'	1,600 lbs.	1'
below 25'	800 lbs.	1'

ROTARY WASH BORINGS:

Driving Weight = 320 lbs.

Stroke = 1½'

STANDARD PENETRATION TEST:

30 ▯ Depth at which test performed

30 — Number of blows required to drive Standard Penetration sampler 12"

CLASSIFICATION SYSTEMS:

Unified Soil Classification Systems

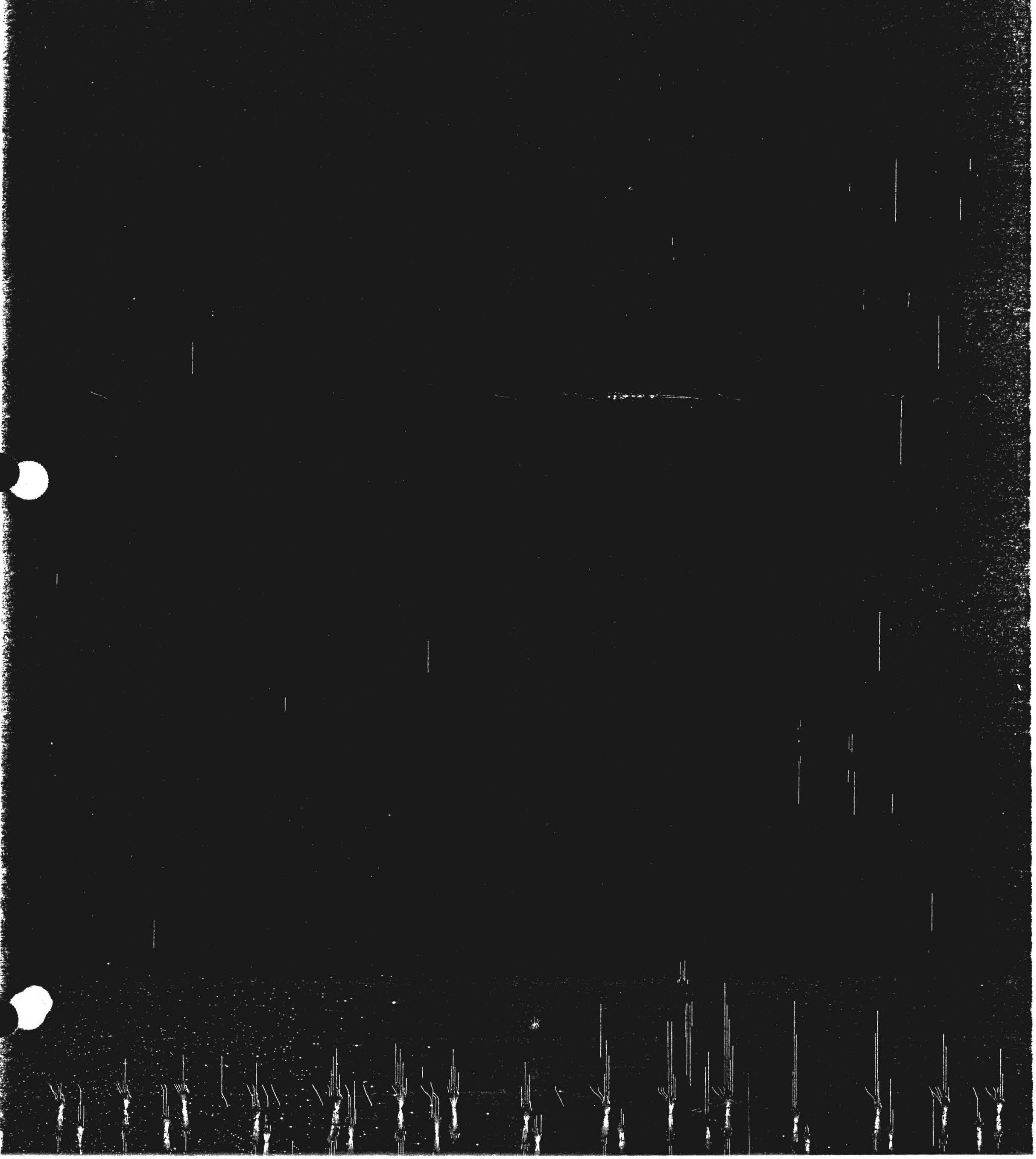
KEY TO LOG OF BORINGS

JOB ADE-85005-8 DATE 10/7/85 DR ____ W.P. ____ CHKD ____

FO

[illegible] 8

[illegible] Survey



APPENDIX B:
DOWNHOLE SEISMIC SURVEY

B.1 SUMMARY

Downhole seismic surveys were performed in Borings 30 and 37. Measurements were made from the ground surface to depths of 60 feet in the two borings.

B.2 PROCEDURE

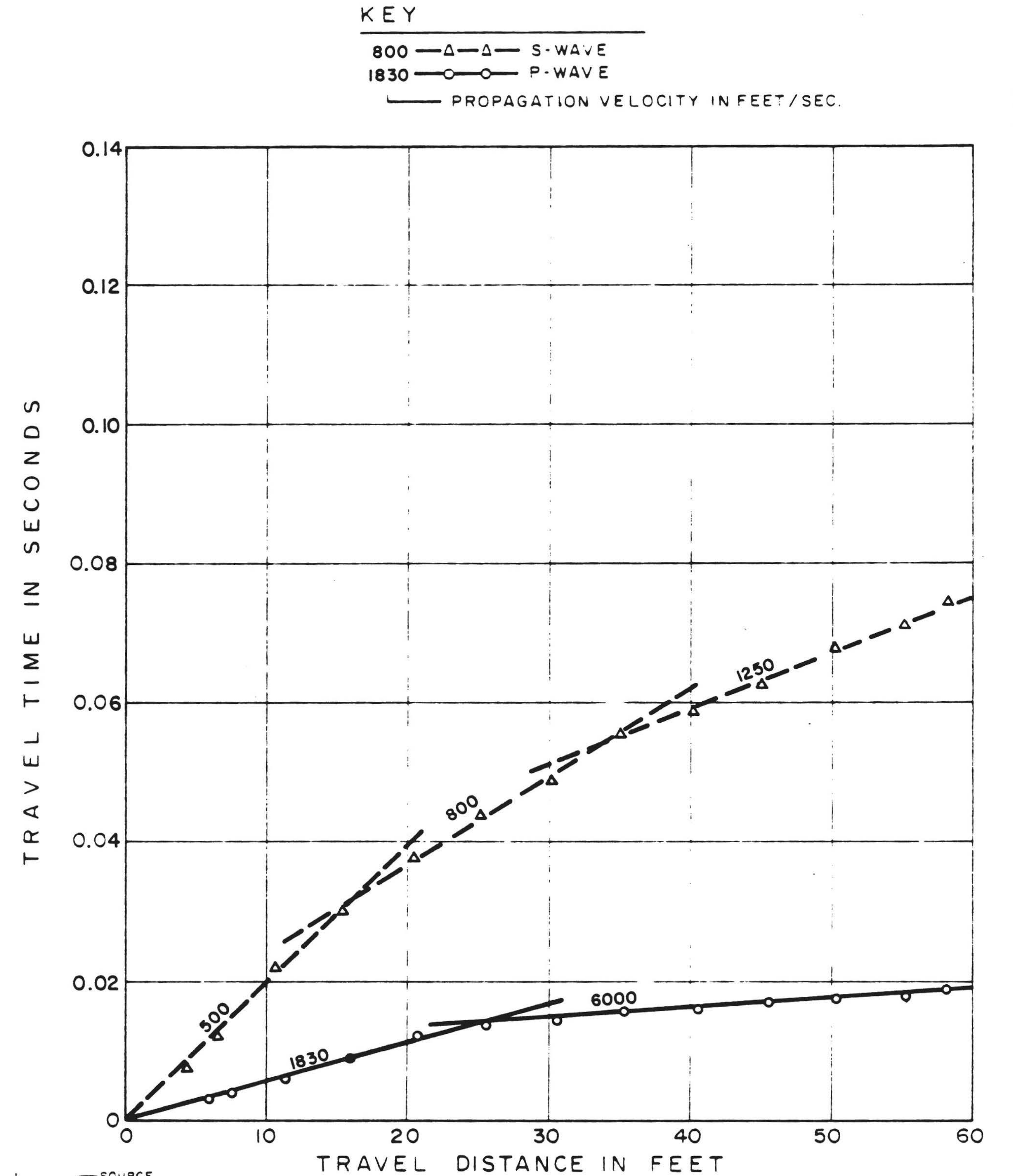
After completion of the drilling of the borings, two-inch-diameter PVC pipe was installed in the borings, and pea gravel backfill was placed around the pipes. Downhole seismic surveys were then performed in the pipes to determine the propagation velocities of the compressional waves (P-waves) and shear waves (S-waves).

A borehole seismometer, connected with cable to an amplifier and recorder, was lowered to the bottom of the pipe. A wooden plank was placed adjacent to the boring and weighted down with the front wheels of a vehicle. The S-waves were generated by horizontally striking the end of the plank with a sledge hammer; the P-waves were generated by vertically striking the top of the plank. The S-waves and P-waves were detected by the three orthogonal geophones of the borehole seismometer. When the measurements were completed at a given depth, the seismometer was raised to a higher level and a new set of measurements was taken.

B.3 ANALYSIS

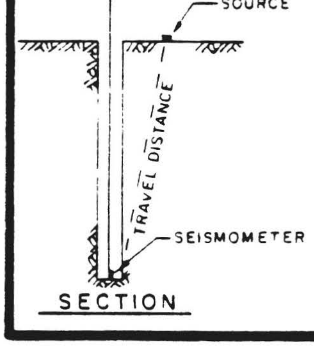
The times of first arrivals of the S-waves and P-waves were determined from the recordings and were plotted versus distance from the source on travel time curves which are presented on Figures B-1 and B-2, Downhole Seismic Survey. The propagation velocities were computed and are presented on Figures B-1 and B-2.

JOB ADE-85005-B DATE 10/3/85 DR [illegible] O [illegible] W.P.____ CHKD [illegible]



DOWNHOLE SEISMIC SURVEY

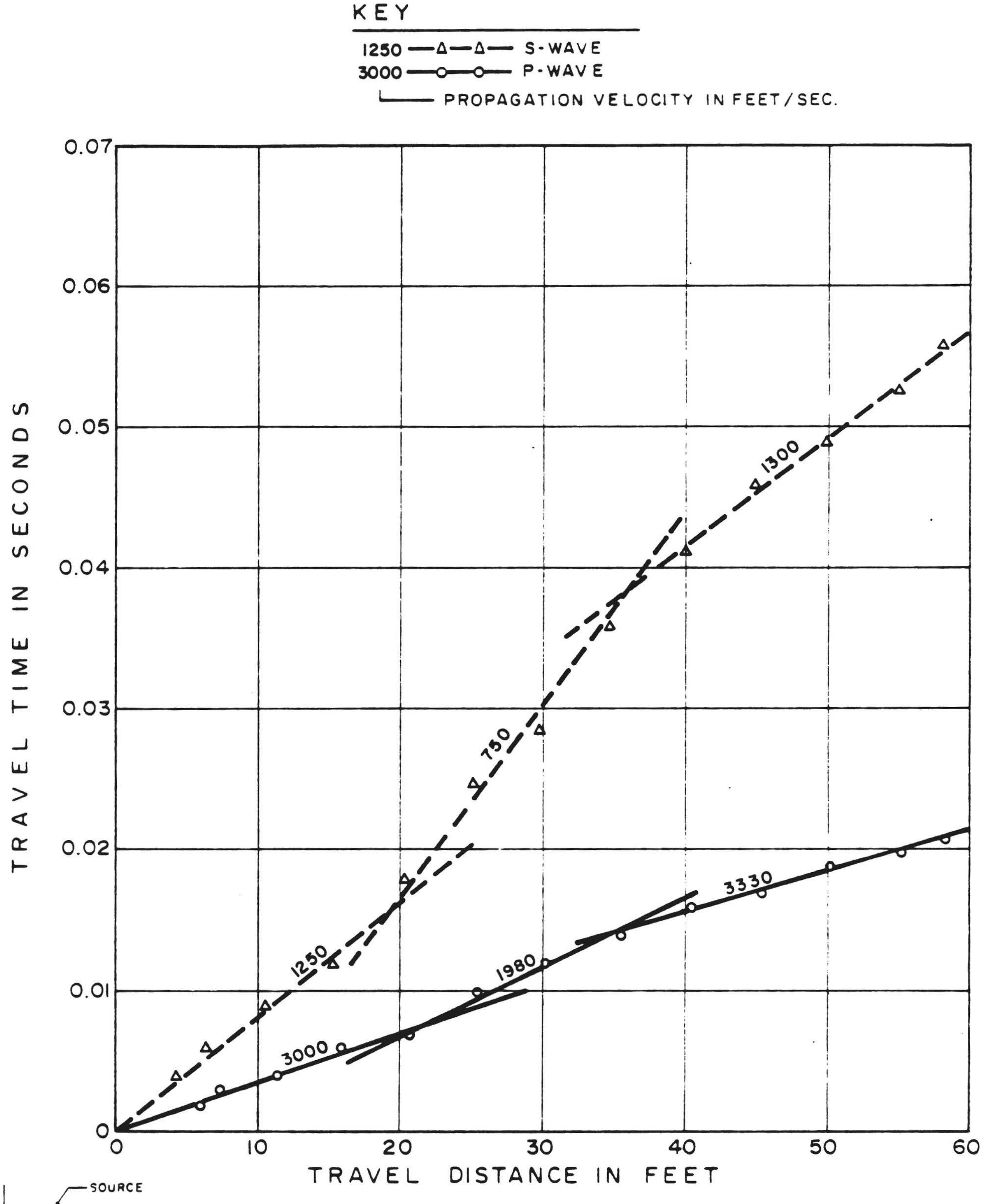
TRAVEL TIME CURVE
(BORING 30)

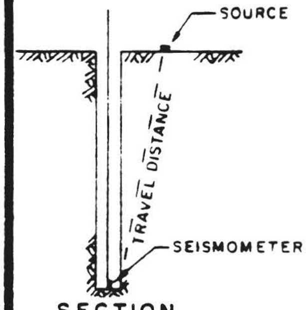


LeROY CRANDALL AND ASSOCIATES

FIGURE B-1

JOB D-85005-B DATE 10/4/85 DR 05 W.P. CHKD





SECTION

DOWNHOLE SEISMIC SURVEY

TRAVEL TIME CURVE

(BORING 37)

LeROY CRANDALL AND ASSOCIATES

FIGURE B-2

APPENDIX C:
LABORATORY TESTING

C.1 INTRODUCTION

The laboratory testing program was directed toward a quantitative determination of the physical properties of the soils along the alignment. Each type of material was thoroughly investigated to determine the significant properties of the materials. All of the laboratory testing was performed within our office in Los Angeles.

C.2 LABORATORY TESTING PROGRAM

The laboratory program included testing of undisturbed samples, as well as tests on bulk materials. The undisturbed samples were placed in plastic bags and stored in sealed cans until ready for use, and the bulk samples were stored in plastic bags. The Pitcher samples were not tested because the samples were disturbed.

The first phase of the testing program consisted of determining the classification of the soils. The primary classifications were made by making a visual inspection. Representative samples were then selected for more specific studies to determine pertinent shear strength and consolidation parameters.

C.3 LABORATORY TESTING PROCEDURES

C.3.1 Moisture Content

Moisture contents were determined by weighing the material at natural moisture content, drying it in an oven at a temperature of about 230°F, weighing the completely oven-dried sample, and calculating the moisture content. Natural water contents were determined on the undisturbed samples shortly after the samples arrived at the laboratory. The results of the tests are presented to the left of the boring logs on Figures A-1 through A-12.

C.3.2 Dry Density

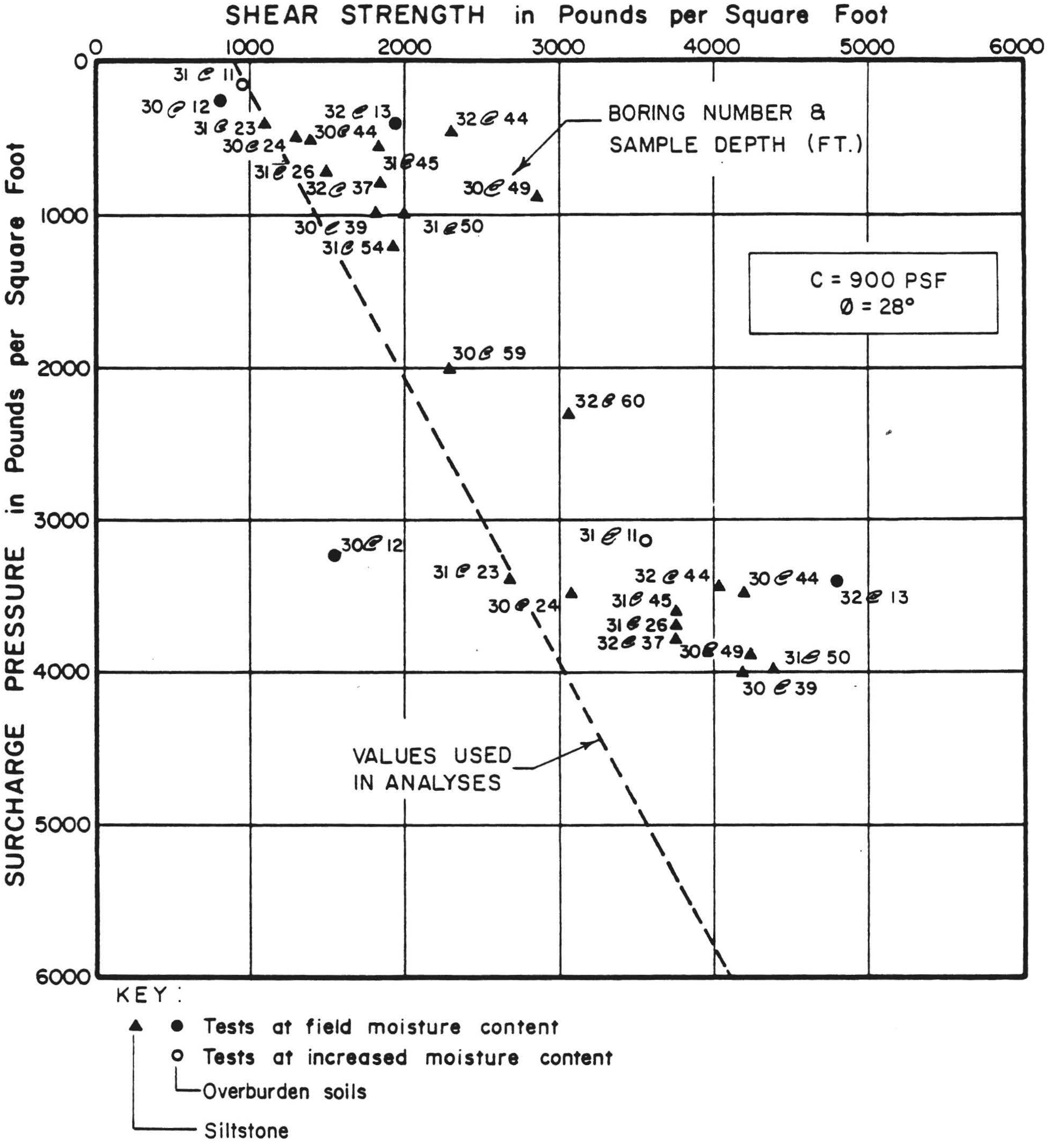
Dry density determinations were obtained by carefully utilizing a ring sample measuring with a known volume of the undisturbed sample, weighing the sample after it had been oven-dried, and calculating the unit weight. Results of the dry density determinations are presented to the left of the boring logs.

C.3.3 Direct Shear Tests

Direct shear tests were performed on selected undisturbed samples. The tests were performed at field and increased moisture contents and at surcharge pressures equal to the existing overburden pressures. Selected samples were tested at an increased surcharge pressure to provide more complete data. All of the samples were tested at a constant strain of 0.05 inches per minute. The yield-point values determined from the direct shear tests are presented on Figures C-1 through C-3.

C.3.4 Consolidation Tests

Undisturbed samples were tested in consolidometers to determine the consolidation characteristics of the soils. Vertical loads were instantaneously applied in increments and the rate of vertical consolidation was measured for each increment. Each load was allowed to consolidate the sample for at least 12 hours before a new increment was added. All the samples were tested at field moisture content. To simulate the effects of the excavation, the samples were loaded, unloaded, and subsequently reloaded. The results of the consolidation tests are presented on Figures C-4 through C-11.

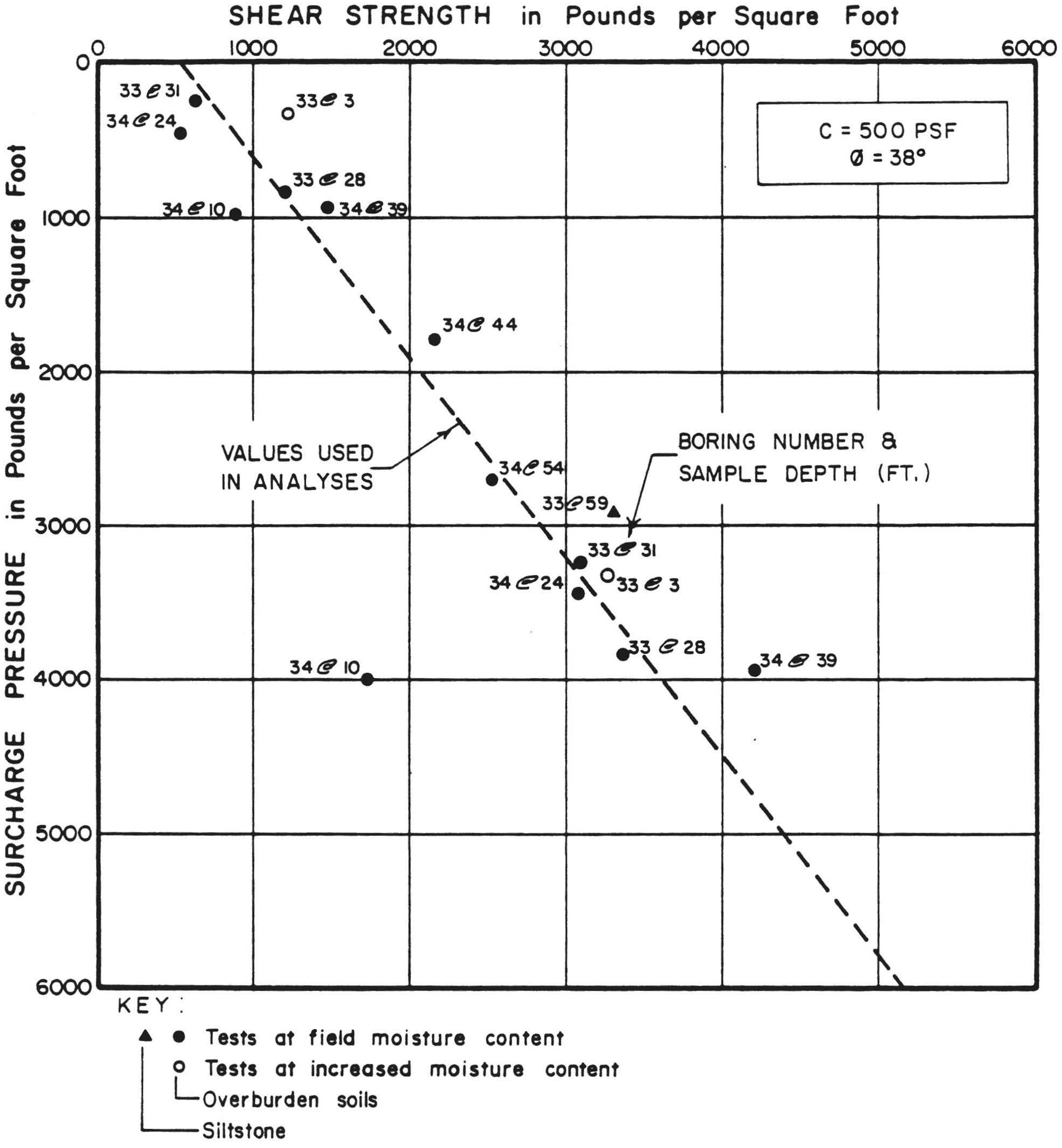


DIRECT SHEAR TEST DATA

(BORINGS 30 THROUGH 32)

LEROY CRANDALL & ASSOCIATES

FIGURE C-1

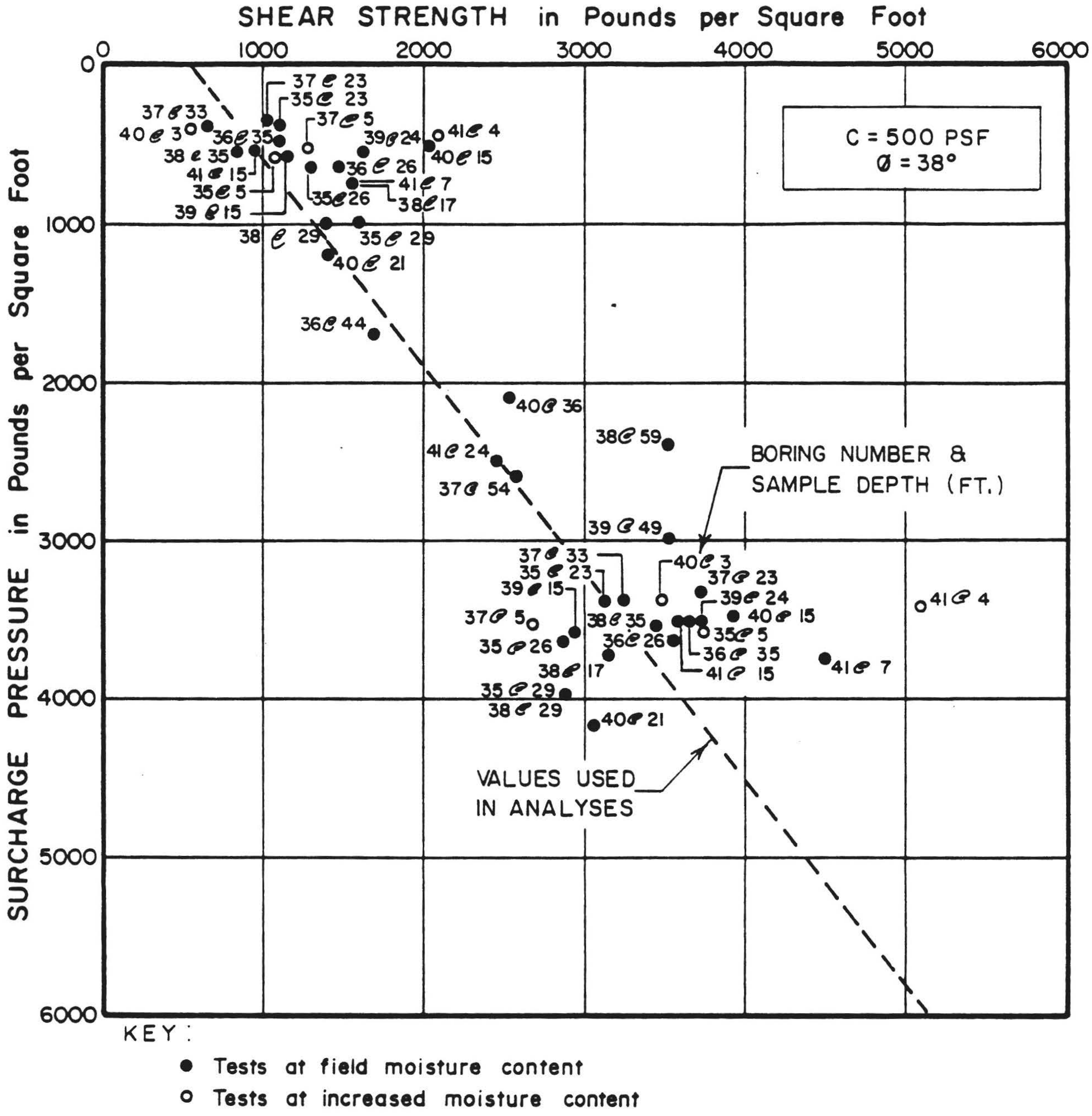


DIRECT SHEAR TEST DATA

(BORINGS 33 AND 34)

LeROY CRANDALL & ASSOCIATES

FIGURE C-2

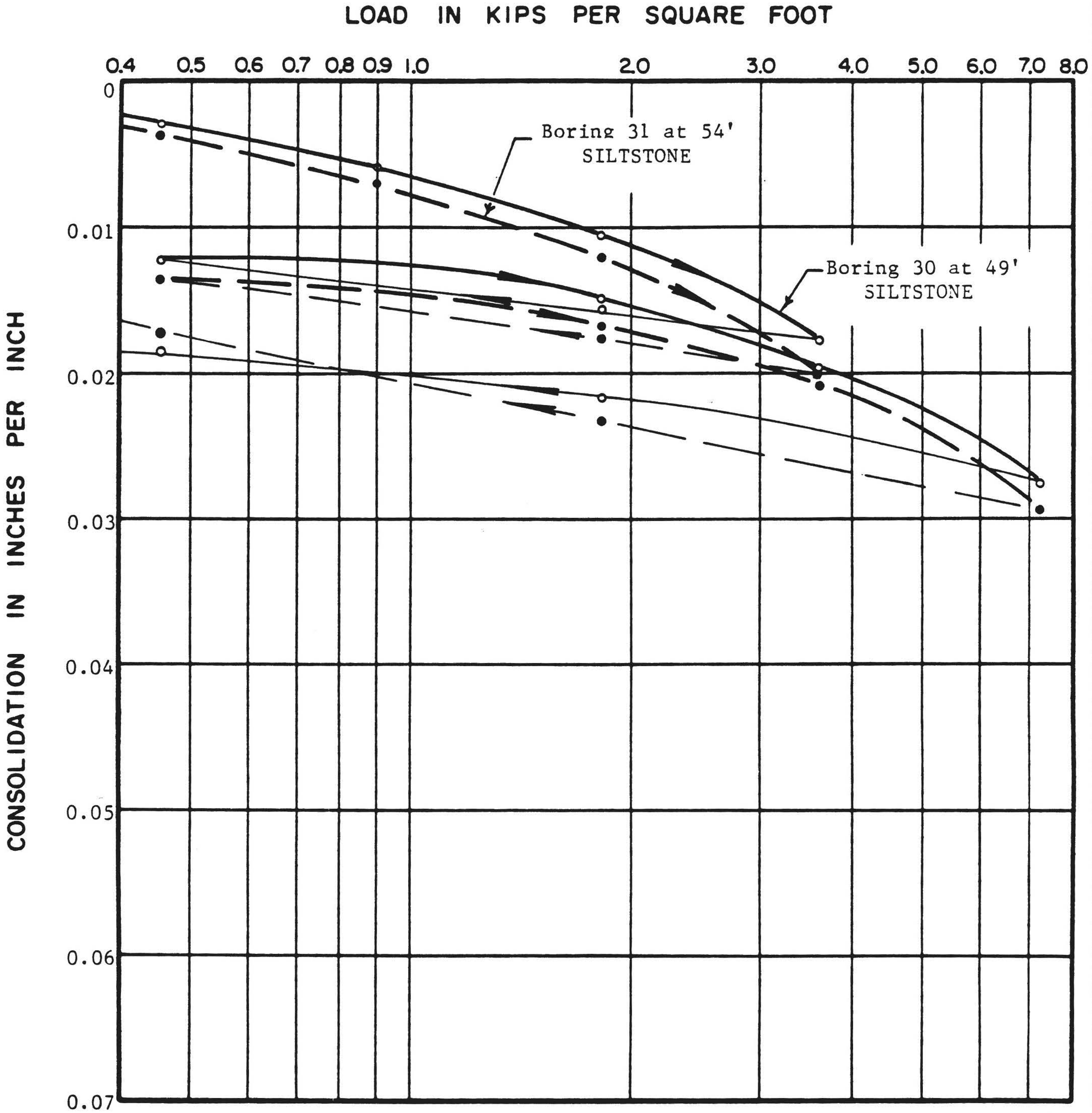


DIRECT SHEAR TEST DATA

(BORINGS 35 THROUGH 41)

LEROY CRANDALL & ASSOCIATES

FIGURE C-3



NOTE: Samples tested at field moisture content.

CONSOLIDATION TEST DATA

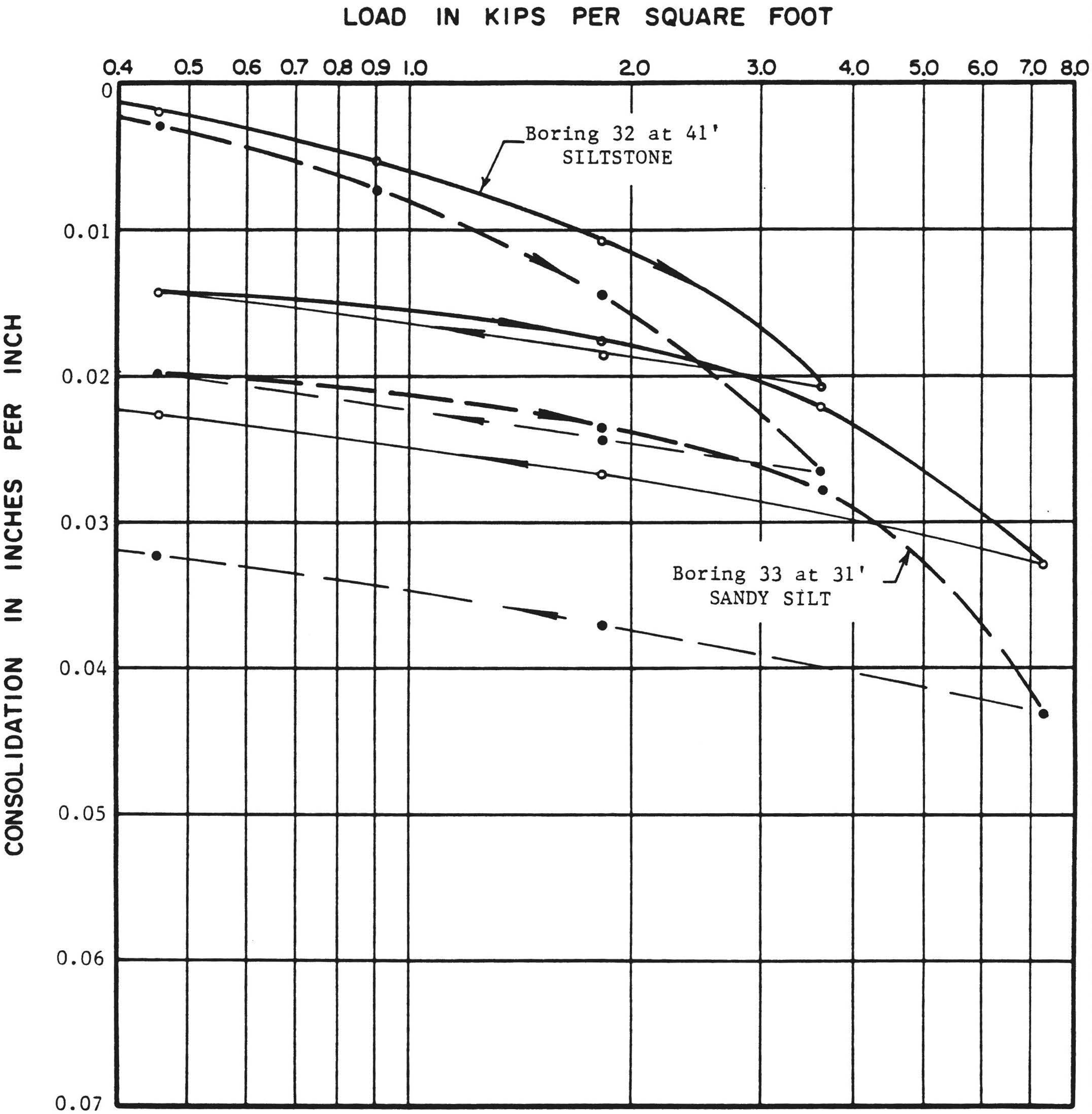
LeROY CRANDALL AND ASSOCIATES

FIGURE C-4

JOB A-85005-8 DATE 9/6/85 DR. os W.P. nh CHKD

FORM 116

JOB *A-85005-B* DATE *9/6/85* DR. *OS* W.P. ____ CHKD ____



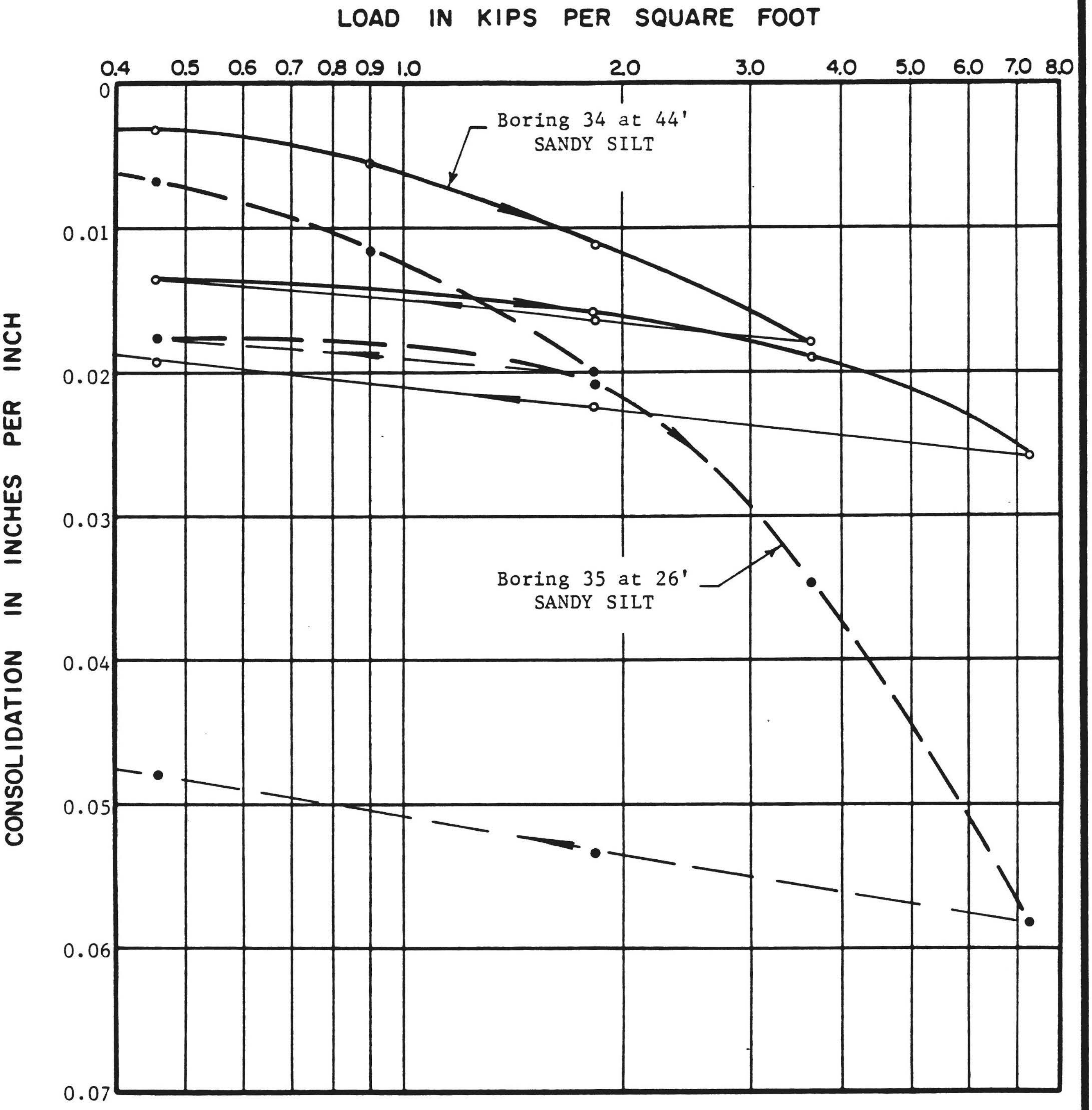
NOTE: Samples tested at field moisture content.

CONSOLIDATION TEST DATA

LeROY CRANDALL AND ASSOCIATES

FIGURE C-5

FORM 116



NOTE: Samples tested at field moisture content.

CONSOLIDATION TEST DATA

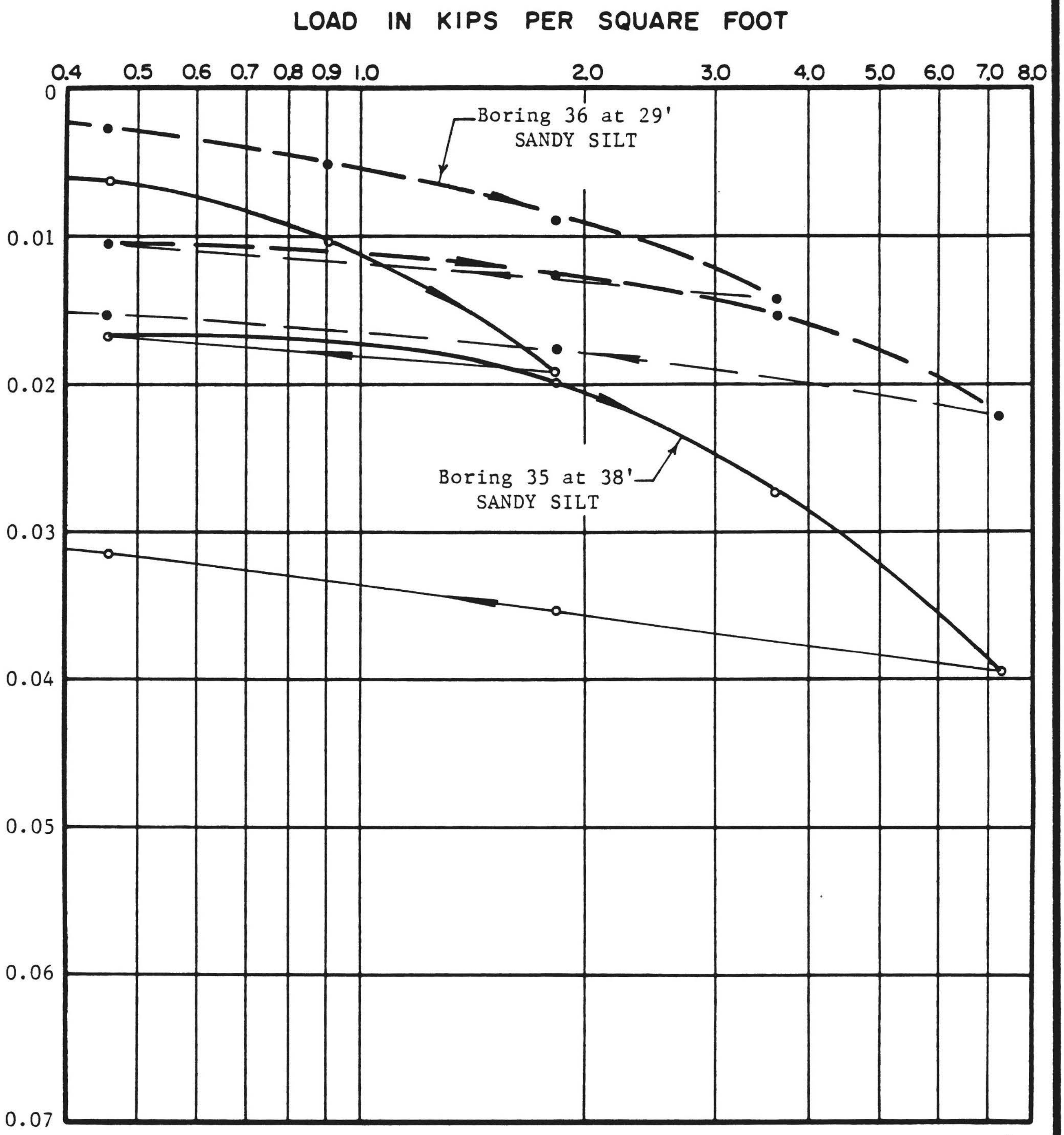
LeROY CRANDALL AND ASSOCIATES

FIGURE C-6

JOB A-85005-8 DATE 9/6/85 DR. OV W.P. MS CHKD
imh

FORM 116

JOB A-85005-8 DATE 9/6/85 DR. ar Ms W.P. dmh CHKD ll



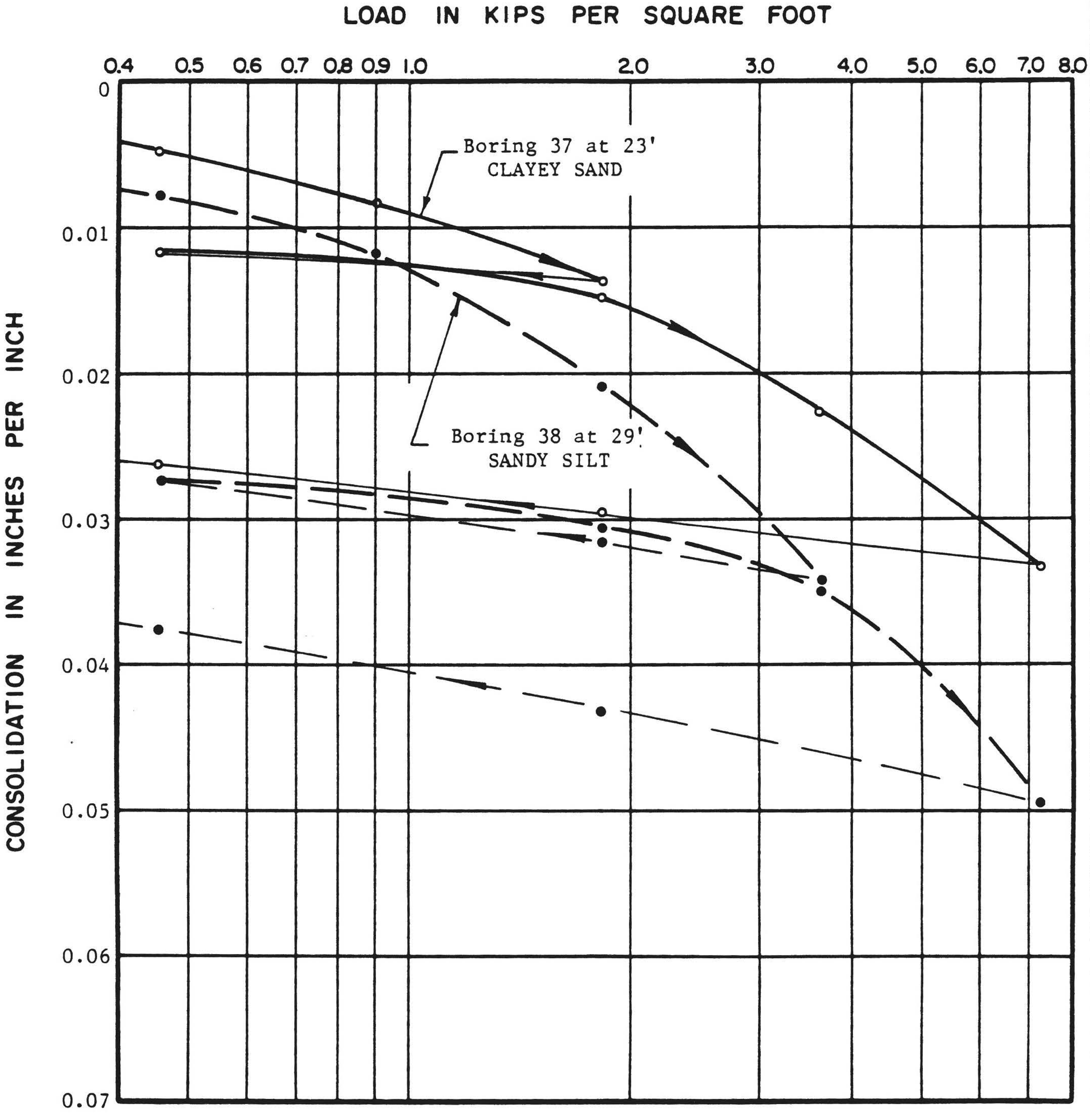
NOTE: Samples tested at field moisture content.

CONSOLIDATION TEST DATA

LeROY CRANDALL AND ASSOCIATES

FIGURE C-7

FORM 116



NOTE: Samples tested at field moisture content.

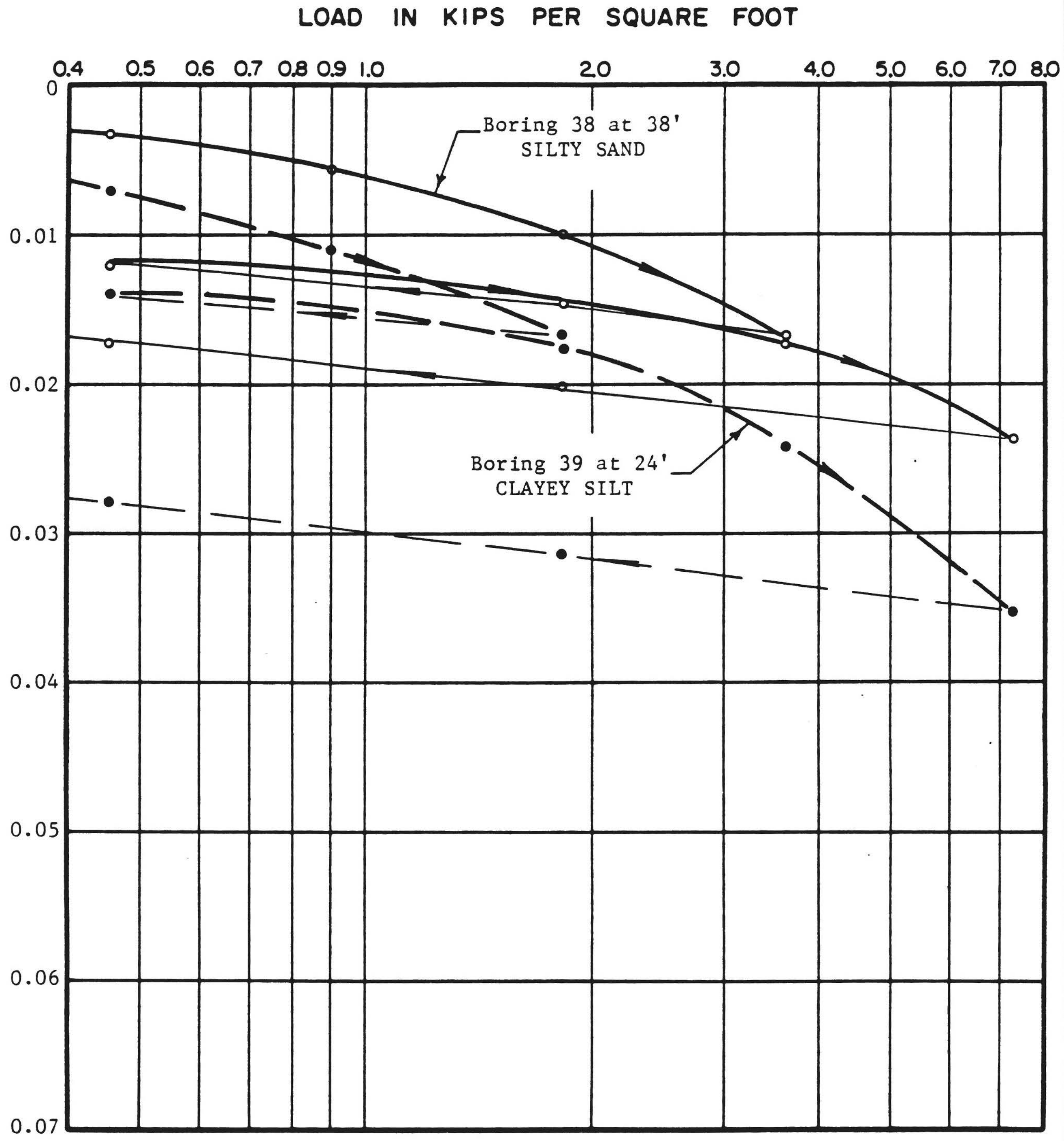
CONSOLIDATION TEST DATA

LeROY CRANDALL AND ASSOCIATES

FIGURE C-8

FORM 116

JOB A-85005-B DATE 9/6/85 DR. or MS W.P. lmh CHKD



NOTE: Samples tested at field moisture content.

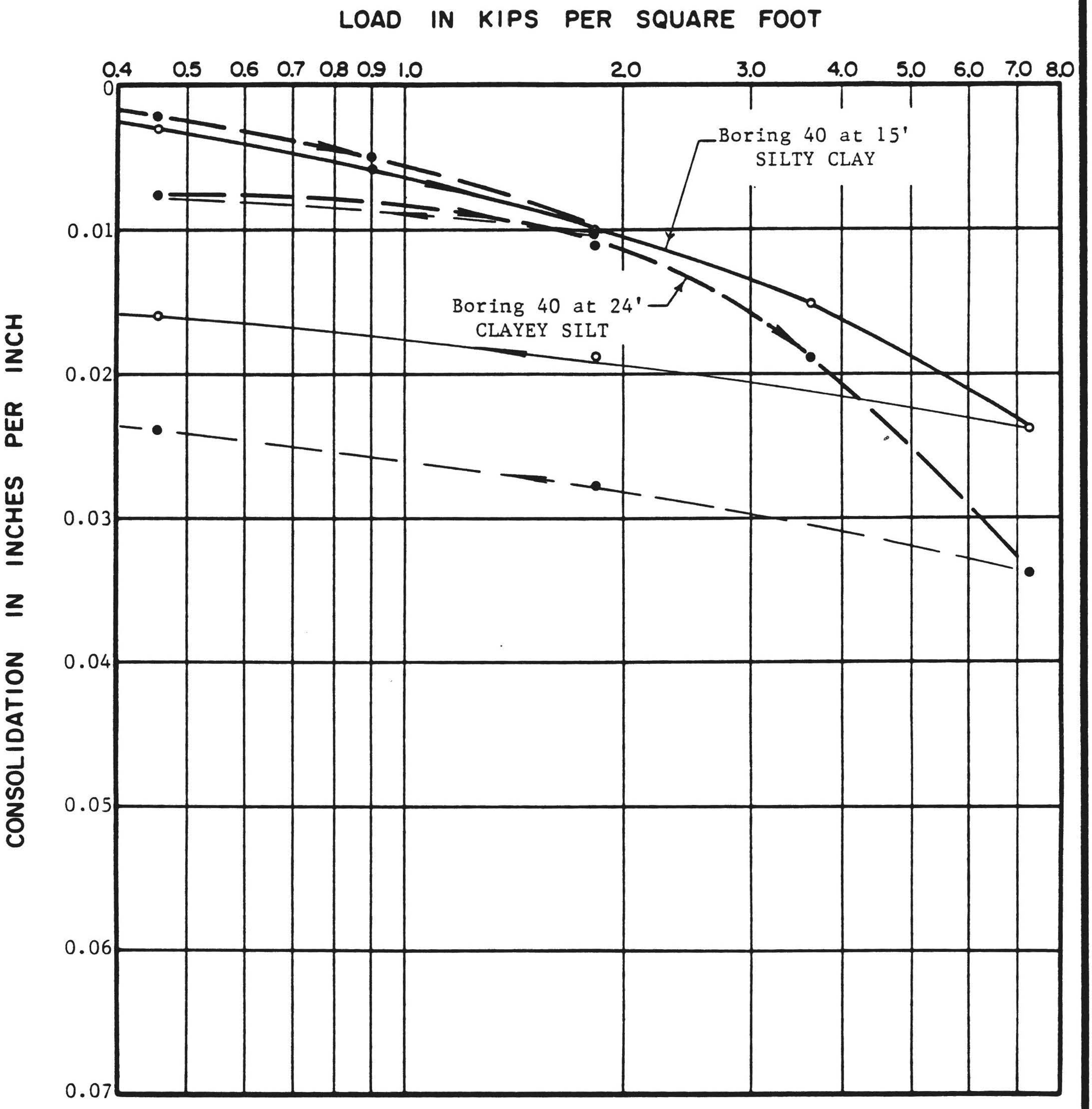
CONSOLIDATION TEST DATA

LeROY CRANDALL AND ASSOCIATES

FIGURE C-9

JOB A-85005-8 DATE 9/6/85 DR. os MS W.P. lmh CHKD

FORM 116



NOTE: Samples tested at field moisture content.

CONSOLIDATION TEST DATA

LeROY CRANDALL AND ASSOCIATES

FIGURE C-10

JOB A-85005B DATE 9/6/85 DR. ov MS W.P. dmh CHKD

FORM 116

Appendix D

[illegible]

APPENDIX D:
SEISMIC DATA

D.1 COMPUTER SEARCH OF RECORDED EARTHQUAKES

The seismicity of the area was determined from a computer search of a magnetic tape catalog of earthquakes. The catalog of earthquakes included those compiled by the California Institute of Technology for the period 1932 to 1981 and those earthquakes for the period 1812 to 1931 compiled by Richter and the U. S. National Oceanic and Atmospheric Administration (NOAA). The computer printout of the earthquakes is presented on Table D. The search for earthquakes that occurred within 100 kilometers of the site indicates that 292 earthquakes of Richter magnitude 4.0 and greater occurred between 1932 and 1981; two earthquakes of magnitude 6.0 or greater occurred between 1906 and 1931; and one earthquake of magnitude 7.0 or greater occurred between 1812 and 1905.

The information listed for each earthquake found in the printout includes date and time in Greenwich Civil Time (GCT), location of the epicenter in latitude and longitude, quality of epicentral determination (Q), depth in kilometers, and magnitude. Where a depth of 0.0 is given, the solution was based on an assumed 16-kilometer focal depth. The explanation of the letter code for the quality factor of the data is presented on the first page of the table.

D.2 SITE PERIOD CALCULATIONS

The evaluation of the characteristic site period, Ts, is necessary to determine the coefficient of site-structure resonance, S, in accordance with Section 2312 of the 1982 edition of the Uniform Building Code. The characteristic periods were evaluated following the procedures suggested in SEAOC Standard No. 1, Recommended Lateral Force Requirements and Commentary, Seismology Committee, Structural Engineers Association of California, 1980.

The site period determination requires the knowledge of the shear wave velocities of the various deposits underlying the site. The shear wave velocity values presented in Appendix B were determined based on the results of downhole seismic surveys. The details and the results of the surveys are presented in Appendix B.

The average shear wave velocities that were utilized in the determination of the site periods are presented on Figures D-1 and D-2, Site Period Determination for geotechnical profiles that are judged to reflect a possible range of depths below the foundation level at which the shear wave velocity is 2,500 feet per second or greater.

Boring No. 30 Location: Flower Street near 5th Street

Characteristic Site Period:

(1) Postulated Geotechnical Profiles Below Ground Surface:

Profile A

Depth Below Ground Surface (Feet)	Layer Thickness (Feet)	Shear Wave Velocity (Ft./Sec.)
0 - 15	15	500
15 - 35	20	800
35 - 60	25	1250
60 - 100	40	1250*
100 - 150	50	1600*
150+	-	2500*

Profile B

Depth Below Ground Surface (Feet)	Layer Thickness (Feet)	Shear Wave Velocity (Ft./Sec.)
0 - 15	15	500
15 - 35	20	800
35 - 60	25	1250
60 - 100	40	1250*
100 - 150	50	1600*
150 - 200	50	2000*
200+	-	2500*

*Extrapolated below 60 feet below ground surface.

(2) Range of Characteristic Site Period:

Ts = 0.7 to 0.9 sec

SITE PERIOD DETERMINATION

FIGURE D-1

Boring No. 37 Location: Flower Street between 9th St. and Olympic Blvd.

Characteristic Site Period:

(1) Postulated Geotechnical Profiles Below Ground Surface:

Profile A

Depth Below Ground Surface (Feet)	Layer Thickness (Feet)	Shear Wave Velocity (Ft./Sec.)
0 - 20	20	1250
20 - 32	12	750
32 - 60	28	1300
60 - 100	40	1300*
100 - 150	50	1600*
150 - 200	50	2000*
200+	-	2500*

Profile B

Depth Below Ground Surface (Feet)	Layer Thickness (Feet)	Shear Wave Velocity (Ft./Sec.)
0 - 20	20	1250
20 - 32	12	750
32 - 60	28	1300
60 - 100	40	1300*
100 - 175	75	1600*
175 - 250	75	2000*
250+	-	2500*

*Extrapolated below 60 feet below ground surface.

(2) Range of Characteristic Site Period:

Ts = 3/4 to 1 sec

SITE PERIOD DETERMINATION

FIGURE D-2

TABLE D
Sheet 1 of 11

LIST OF HISTORIC EARTHQUAKES OF MAGNITUDE 4.0 OR GREATER WITHIN 100 KM OF THE SITE
(CAL TECH DATA 1932-1981)

YEAR	MONTH	DAY	HR	MIN	SEC	LATITUDE	LONGITUDE	Q	DISTANCE	DEPTH	MAGNITUDE
1932	NOV	1	4	45	0	34.00 N	117.25 W	E	93	.0	4.0
1933	MAR	11	1	54	8	33.62 N	117.97 W	A	55	.0	6.3
1933	MAR	11	2	4	0	33.75 N	118.08 W	C	37	.0	4.9
1933	MAR	11	2	5	0	33.75 N	118.08 W	C	37	.0	4.3
1933	MAR	11	2	9	0	33.75 N	118.08 W	C	37	.0	5.0
1933	MAR	11	2	10	0	33.75 N	118.08 W	C	37	.0	4.6
1933	MAR	11	2	11	0	33.75 N	118.08 W	C	37	.0	4.4
1933	MAR	11	2	16	0	33.75 N	118.08 W	C	37	.0	4.8
1933	MAR	11	2	17	0	33.60 N	118.00 W	E	55	.0	4.5
1933	MAR	11	2	22	0	33.75 N	118.08 W	C	37	.0	4.0
1933	MAR	11	2	27	0	33.75 N	118.08 W	C	37	.0	4.6
1933	MAR	11	2	30	0	33.75 N	118.08 W	C	37	.0	5.1
1933	MAR	11	2	31	0	33.60 N	118.00 W	E	55	.0	4.4
1933	MAR	11	2	52	0	33.75 N	118.08 W	C	37	.0	4.0
1933	MAR	11	2	57	0	33.75 N	118.08 W	C	37	.0	4.2
1933	MAR	11	2	58	0	33.75 N	118.08 W	C	37	.0	4.0
1933	MAR	11	2	59	0	33.75 N	118.08 W	C	37	.0	4.6
1933	MAR	11	3	5	0	33.75 N	118.08 W	C	37	.0	4.2
1933	MAR	11	3	9	0	33.75 N	118.08 W	C	37	.0	4.4
1933	MAR	11	3	11	0	33.75 N	118.08 W	C	37	.0	4.2
1933	MAR	11	3	23	0	33.75 N	118.08 W	C	37	.0	5.0
1933	MAR	11	3	36	0	33.75 N	118.08 W	C	37	.0	4.0
1933	MAR	11	3	39	0	33.75 N	118.08 W	C	37	.0	4.0
1933	MAR	11	3	47	0	33.75 N	118.08 W	C	37	.0	4.1
1933	MAR	11	4	36	0	33.75 N	118.08 W	C	37	.0	4.6
1933	MAR	11	4	39	0	33.75 N	118.08 W	C	37	.0	4.9
1933	MAR	11	4	40	0	33.75 N	118.08 W	C	37	.0	4.7
1933	MAR	11	5	10	22	33.70 N	118.07 W	C	43	.0	5.1
1933	MAR	11	5	13	0	33.75 N	118.08 W	C	37	.0	4.7
1933	MAR	11	5	15	0	33.75 N	118.08 W	C	37	.0	4.0
1933	MAR	11	5	18	4	33.57 N	117.98 W	C	59	.0	5.2
1933	MAR	11	5	21	0	33.75 N	118.08 W	C	37	.0	4.4
1933	MAR	11	5	24	0	33.75 N	118.08 W	C	37	.0	4.2
1933	MAR	11	5	53	0	33.75 N	118.08 W	C	37	.0	4.0
1933	MAR	11	5	55	0	33.75 N	118.08 W	C	37	.0	4.0
1933	MAR	11	6	11	0	33.75 N	118.08 W	C	37	.0	4.4
1933	MAR	11	6	18	0	33.75 N	118.08 W	C	37	.0	4.2
1933	MAR	11	6	29	0	33.85 N	118.27 W	C	22	.0	4.4
1933	MAR	11	6	35	0	33.75 N	118.08 W	C	37	.0	4.2
1933	MAR	11	6	58	3	33.68 N	118.05 W	C	45	.0	5.5
1933	MAR	11	7	51	0	33.75 N	118.08 W	C	37	.0	4.2
1933	MAR	11	7	59	0	33.75 N	118.08 W	C	37	.0	4.1
1933	MAR	11	8	8	0	33.75 N	118.08 W	C	37	.0	4.5
1933	MAR	11	8	32	0	33.75 N	118.08 W	C	37	.0	4.2
1933	MAR	11	8	37	0	33.75 N	118.08 W	C	37	.0	4.0
1933	MAR	11	8	54	57	33.70 N	118.07 W	C	43	.0	5.1
1933	MAR	11	9	10	0	33.75 N	118.08 W	C	37	.0	5.1
1933	MAR	11	9	11	0	33.75 N	118.08 W	C	37	.0	4.4
1933	MAR	11	9	26	0	33.75 N	118.08 W	C	37	.0	4.1
1933	MAR	11	10	25	0	33.75 N	118.08 W	C	37	.0	4.0
1933	MAR	11	10	45	0	33.75 N	118.08 W	C	37	.0	4.0

NOTE: Q IS A FACTOR RELATING THE QUALITY OF EPICENTRAL DETERMINATION

A = SPECIALLY INVESTIGATED
B = EPICENTER PROBABLY WITHIN 5 KM, ORIGIN TIME TO NEAREST SECOND
C = EPICENTER PROBABLY WITHIN 15 KM, ORIGIN TIME TO A FEW SECONDS
D = EPICENTER NOT KNOWN WITHIN 15 KM, ROUGH LOCATION
E = EPICENTER ROUGHLY LOCATED, ACCURACY LESS THAN "D"
P = PRELIMINARY

TABLE D
Sheet 2 of 11

YEAR	MONTH	DAY	HR	MIN	SEC	LATITUDE	LONGITUDE	Q	DISTANCE	DEPTH	MAGNITUDE
1933	MAR	11	11	0	0	33.75 N	118.08 W	C	37	.0	4.0
1933	MAR	11	11	4	0	33.75 N	118.13 W	C	35	.0	4.6
1933	MAR	11	11	29	0	33.75 N	118.08 W	C	37	.0	4.0
1933	MAR	11	11	38	0	33.75 N	118.08 W	C	37	.0	4.0
1933	MAR	11	11	41	0	33.75 N	118.08 W	C	37	.0	4.2
1933	MAR	11	11	47	0	33.75 N	118.08 W	C	37	.0	4.4
1933	MAR	11	12	50	0	33.68 N	118.05 W	C	45	.0	4.4
1933	MAR	11	13	50	0	33.73 N	118.10 W	C	38	.0	4.4
1933	MAR	11	13	57	0	33.75 N	118.08 W	C	37	.0	4.0
1933	MAR	11	14	25	0	33.85 N	118.27 W	C	22	.0	5.0
1933	MAR	11	14	47	0	33.73 N	118.10 W	C	38	.0	4.4
1933	MAR	11	14	57	0	33.88 N	118.32 W	C	20	.0	4.9
1933	MAR	11	15	9	0	33.73 N	118.10 W	C	38	.0	4.4
1933	MAR	11	15	47	0	33.75 N	118.08 W	C	37	.0	4.0
1933	MAR	11	16	53	0	33.75 N	118.08 W	C	37	.0	4.8
1933	MAR	11	19	44	0	33.75 N	118.08 W	C	37	.0	4.0
1933	MAR	11	19	56	0	33.75 N	118.08 W	C	37	.0	4.2
1933	MAR	11	22	0	0	33.75 N	118.08 W	C	37	.0	4.4
1933	MAR	11	22	31	0	33.75 N	118.08 W	C	37	.0	4.4
1933	MAR	11	22	32	0	33.75 N	118.08 W	C	37	.0	4.1
1933	MAR	11	22	40	0	33.75 N	118.08 W	C	37	.0	4.4
1933	MAR	11	23	5	0	33.75 N	118.08 W	C	37	.0	4.2
1933	MAR	12	0	27	0	33.75 N	118.08 W	C	37	.0	4.4
1933	MAR	12	0	34	0	33.75 N	118.08 W	C	37	.0	4.0
1933	MAR	12	4	48	0	33.75 N	118.08 W	C	37	.0	4.0
1933	MAR	12	5	46	0	33.75 N	118.08 W	C	37	.0	4.4
1933	MAR	12	6	1	0	33.75 N	118.08 W	C	37	.0	4.2
1933	MAR	12	6	16	0	33.75 N	118.08 W	C	37	.0	4.6
1933	MAR	12	7	40	0	33.75 N	118.08 W	C	37	.0	4.2
1933	MAR	12	8	35	0	33.75 N	118.08 W	C	37	.0	4.2
1933	MAR	12	15	2	0	33.75 N	118.08 W	C	37	.0	4.2
1933	MAR	12	16	51	0	33.75 N	118.08 W	C	37	.0	4.0
1933	MAR	12	17	38	0	33.75 N	118.08 W	C	37	.0	4.5
1933	MAR	12	18	25	0	33.75 N	118.08 W	C	37	.0	4.1
1933	MAR	12	21	28	0	33.75 N	118.08 W	C	37	.0	4.1
1933	MAR	12	23	54	0	33.75 N	118.08 W	C	37	.0	4.5
1933	MAR	13	3	43	0	33.75 N	118.08 W	C	37	.0	4.1
1933	MAR	13	4	32	0	33.75 N	118.08 W	C	37	.0	4.7
1933	MAR	13	6	17	0	33.75 N	118.08 W	C	37	.0	4.0
1933	MAR	13	13	18	28	33.75 N	118.08 W	C	37	.0	5.3
1933	MAR	13	15	32	0	33.75 N	118.08 W	C	37	.0	4.1
1933	MAR	13	19	29	0	33.75 N	118.08 W	C	37	.0	4.2
1933	MAR	14	0	36	0	33.75 N	118.08 W	C	37	.0	4.2
1933	MAR	14	12	19	0	33.75 N	118.08 W	C	37	.0	4.5
1933	MAR	14	19	1	50	33.62 N	118.02 W	C	53	.0	5.1
1933	MAR	14	22	42	0	33.75 N	118.08 W	C	37	.0	4.1
1933	MAR	15	2	8	0	33.75 N	118.08 W	C	37	.0	4.1
1933	MAR	15	4	32	0	33.75 N	118.08 W	C	37	.0	4.1
1933	MAR	15	5	40	0	33.75 N	118.08 W	C	37	.0	4.2
1933	MAR	15	11	13	32	33.62 N	118.02 W	C	53	.0	4.9
1933	MAR	16	14	56	0	33.75 N	118.08 W	C	37	.0	4.0
1933	MAR	16	15	29	0	33.75 N	118.08 W	C	37	.0	4.2
1933	MAR	16	15	30	0	33.75 N	118.08 W	C	37	.0	4.1
1933	MAR	17	16	51	0	33.75 N	118.08 W	C	37	.0	4.1
1933	MAR	18	20	52	0	33.75 N	118.08 W	C	37	.0	4.2
1933	MAR	19	21	23	0	33.75 N	118.08 W	C	37	.0	4.2
1933	MAR	20	13	58	0	33.75 N	118.08 W	C	37	.0	4.1
1933	MAR	21	3	26	0	33.75 N	118.08 W	C	37	.0	4.1
1933	MAR	23	8	40	0	33.75 N	118.08 W	C	37	.0	4.1
1933	MAR	23	18	31	0	33.75 N	118.08 W	C	37	.0	4.1
1933	MAR	25	13	46	0	33.75 N	118.08 W	C	37	.0	4.1
1933	MAR	30	12	25	0	33.75 N	118.08 W	C	37	.0	4.4
1933	MAR	31	10	49	0	33.75 N	118.08 W	C	37	.0	4.1
1933	APR	1	6	42	0	33.75 N	118.08 W	C	37	.0	4.2
1933	APR	2	8	0	0	33.75 N	118.08 W	C	37	.0	4.0

TABLE D
Sheet 3 of 11

YEAR	MONTH	DAY	HR	MIN	SEC	LATITUDE	LONGITUDE	Q	DISTANCE	DEPTH	MAGNITUDE
1933	APR	2	15	36	0	33.75 N	118.08 W	C	37	.0	4.0
1933	MAY	16	20	58	55	33.75 N	118.17 W	C	34	.0	4.0
1933	AUG	4	4	17	48	33.75 N	118.18 W	C	34	.0	4.0
1933	OCT	2	9	10	18	33.78 N	118.13 W	A	32	.0	5.4
1933	OCT	2	13	26	1	33.62 N	118.02 W	C	53	.0	4.0
1933	OCT	25	7	0	46	33.95 N	118.13 W	C	16	.0	4.3
1933	NOV	13	21	28	0	33.87 N	118.20 W	C	21	.0	4.0
1933	NOV	20	10	32	0	33.78 N	118.13 W	B	32	.0	4.0
1934	JAN	9	14	10	0	34.10 N	117.68 W	A	54	.0	4.5
1934	JAN	18	2	14	0	34.10 N	117.68 W	A	54	.0	4.0
1934	JAN	20	21	17	0	33.62 N	118.12 W	B	49	.0	4.5
1934	APR	17	18	33	0	33.57 N	117.98 W	C	59	.0	4.0
1934	OCT	17	9	38	0	33.63 N	118.40 W	B	48	.0	4.0
1934	NOV	16	21	26	0	33.75 N	118.00 W	B	41	.0	4.0
1935	JUN	11	18	10	0	34.72 N	118.97 W	B	99	.0	4.0
1935	JUN	19	11	17	0	33.72 N	117.52 W	B	77	.0	4.0
1935	JUL	13	10	54	17	34.20 N	117.90 W	A	37	.0	4.7
1935	SEP	3	6	47	0	34.03 N	117.32 W	B	87	.0	4.5
1935	DEC	25	17	15	0	33.60 N	118.02 W	B	55	.0	4.5
1936	FEB	23	22	20	43	34.13 N	117.34 W	A	85	.0	4.5
1936	FEB	26	9	33	28	34.14 N	117.34 W	A	86	.0	4.0
1936	AUG	22	5	21	0	33.77 N	117.82 W	B	51	.0	4.0
1936	OCT	29	22	35	36	34.38 N	118.62 W	C	49	.0	4.0
1937	JAN	15	18	35	47	33.56 N	118.06 W	B	57	.0	4.0
1937	MAR	19	1	23	38	34.11 N	117.43 W	A	77	.0	4.0
1937	JUL	7	11	12	0	33.57 N	117.98 W	B	59	.0	4.0
1937	SEP	1	13	48	8	34.21 N	117.53 W	A	70	.0	4.5
1937	SEP	1	16	35	34	34.18 N	117.55 W	A	67	.0	4.5
1938	MAY	21	9	44	0	33.62 N	118.03 W	B	52	.0	4.0
1938	MAY	31	8	34	55	33.70 N	117.51 W	B	79	.0	5.5
1938	JUL	5	18	6	56	33.68 N	117.55 W	A	77	.0	4.5
1938	AUG	6	22	0	56	33.72 N	117.51 W	B	78	.0	4.0
1938	AUG	31	3	18	14	33.76 N	118.25 W	A	32	.0	4.5
1938	NOV	29	19	21	16	33.90 N	118.43 W	A	23	.0	4.0
1938	DEC	7	3	38	0	34.00 N	118.42 W	B	16	.0	4.0
1938	DEC	27	10	9	29	34.13 N	117.52 W	B	69	.0	4.0
1939	APR	3	2	50	45	34.04 N	117.23 W	A	95	.0	4.0
1939	NOV	4	21	41	0	33.77 N	118.12 W	B	34	.0	4.0
1939	NOV	7	18	52	8	34.00 N	117.28 W	A	91	.0	4.7
1939	DEC	27	19	28	49	33.78 N	118.20 W	A	30	.0	4.7
1940	JAN	13	7	49	7	33.78 N	118.13 W	B	32	.0	4.0
1940	FEB	8	16	56	17	33.70 N	118.07 W	B	43	.0	4.0
1940	FEB	11	19	24	10	33.98 N	118.30 W	B	9	.0	4.0
1940	APR	18	18	43	44	34.03 N	117.35 W	A	84	.0	4.4
1940	MAY	18	9	15	12	34.60 N	118.90 W	C	85	.0	4.0
1940	JUN	5	8	27	27	33.83 N	117.40 W	B	83	.0	4.0
1940	JUL	20	4	1	13	33.70 N	118.07 W	B	43	.0	4.0
1940	OCT	11	5	57	12	33.77 N	118.45 W	A	36	.0	4.7
1940	OCT	12	0	24	0	33.78 N	118.42 W	B	33	.0	4.0
1940	OCT	14	20	51	11	33.78 N	118.42 W	B	33	.0	4.0
1940	NOV	1	7	25	3	33.78 N	118.42 W	B	33	.0	4.0
1940	NOV	1	20	0	46	33.63 N	118.20 W	B	47	.0	4.0
1940	NOV	2	2	58	26	33.78 N	118.42 W	B	33	.0	4.0
1941	JAN	30	1	34	47	33.97 N	118.05 W	A	21	.0	4.1
1941	MAR	22	8	22	40	33.52 N	118.10 W	B	61	.0	4.0
1941	MAR	25	23	43	41	34.22 N	117.47 W	B	75	.0	4.0
1941	APR	11	1	20	24	33.95 N	117.58 W	B	64	.0	4.0
1941	OCT	22	6	57	19	33.82 N	118.22 W	A	26	.0	4.9
1941	NOV	14	8	41	36	33.78 N	118.25 W	A	30	.0	5.4
1942	APR	16	7	28	33	33.37 N	118.15 W	C	76	.0	4.0
1942	SEP	3	14	6	1	34.48 N	118.98 W	C	82	.0	4.5
1942	SEP	4	6	34	33	34.48 N	118.98 W	C	82	.0	4.5
1943	APR	6	22	36	24	34.68 N	119.00 W	C	98	.0	4.0

TABLE D
Sheet 4 of 11

YEAR	MONTH	DAY	HR	MIN	SEC	LATITUDE	LONGITUDE	Q	DISTANCE	DEPTH	MAGNITUDE
1943	OCT	24	0	29	21	33.93 N	117.37 W	C	83	.0	4.0
1944	JUN	19	0	3	33	33.87 N	118.22 W	B	20	.0	4.5
1944	JUN	19	3	6	7	33.87 N	118.22 W	C	20	.0	4.4
1946	FEB	24	6	7	52	34.40 N	117.80 W	C	58	.0	4.1
1946	JUN	1	11	6	31	34.42 N	118.83 W	C	67	.0	4.1
1948	MAR	1	8	12	13	34.17 N	117.53 W	B	69	.0	4.7
1948	APR	16	22	26	24	34.02 N	118.97 W	B	66	.0	4.7
1948	OCT	3	2	46	28	34.18 N	117.58 W	A	64	.0	4.0
1950	JAN	11	21	41	35	33.94 N	118.20 W	A	13	.0	4.1
1950	JAN	24	21	56	59	34.67 N	118.83 W	C	87	.0	4.0
1950	FEB	26	0	6	22	34.62 N	119.08 W	C	99	.0	4.7
1951	SEP	22	8	22	39	34.12 N	117.34 W	A	85	.0	4.3
1952	FEB	10	13	50	55	33.58 N	119.18 W	C	100	.0	4.0
1952	FEB	17	12	36	58	34.00 N	117.27 W	A	92	.0	4.5
1952	AUG	23	10	9	7	34.52 N	118.20 W	A	52	.0	5.0
1954	OCT	26	16	22	26	33.73 N	117.47 W	B	81	.0	4.1
1954	NOV	17	23	3	51	34.50 N	119.12 W	B	94	.0	4.4
1955	MAY	15	17	3	26	34.12 N	117.48 W	A	72	.0	4.0
1955	MAY	29	16	43	35	33.99 N	119.06 W	B	74	.0	4.1
1956	JAN	3	0	25	49	33.72 N	117.50 W	B	79	.0	4.7
1956	FEB	7	2	16	57	34.53 N	118.64 W	B	64	.0	4.2
1956	FEB	7	3	16	39	34.59 N	118.61 W	A	68	.0	4.6
1956	MAR	25	3	32	2	33.60 N	119.10 W	A	92	.0	4.2
1957	MAR	18	18	56	28	34.12 N	119.22 W	B	89	.0	4.7
1960	JUN	28	20	0	48	34.12 N	117.47 W	A	73	.0	4.1
1961	OCT	4	2	21	32	33.85 N	117.75 W	B	52	.0	4.1
1961	OCT	20	19	49	51	33.65 N	117.99 W	B	51	.0	4.3
1961	OCT	20	20	7	14	33.66 N	117.98 W	B	50	.0	4.0
1961	OCT	20	21	42	41	33.67 N	117.98 W	B	49	.0	4.0
1961	OCT	20	22	35	34	33.67 N	118.01 W	B	48	.0	4.1
1961	NOV	20	8	53	35	33.68 N	117.99 W	B	48	.0	4.0
1963	SEP	14	3	51	16	33.54 N	118.34 W	B	57	.0	4.2
1964	AUG	30	22	57	37	34.27 N	118.44 W	B	30	.0	4.0
1965	JAN	1	8	4	18	34.14 N	117.52 W	B	69	.0	4.4
1965	APR	15	20	8	33	34.13 N	117.43 W	B	77	.0	4.5
1965	JUL	16	7	46	22	34.48 N	118.52 W	B	53	.0	4.0
1967	JAN	8	7	37	30	33.63 N	118.47 W	B	50	.0	4.0
1967	JAN	8	7	38	5	33.66 N	118.41 W	C	45	.0	4.0
1967	JUN	15	4	58	6	34.00 N	117.97 W	B	27	.0	4.1
1969	FEB	28	4	56	12	34.57 N	118.11 W	A	59	.0	4.3
1969	MAY	5	16	2	10	34.30 N	117.57 W	B	69	.0	4.4
1969	OCT	27	13	16	2	33.55 N	117.81 W	B	69	.0	4.5
1970	SEP	12	14	10	11	34.27 N	117.52 W	A	73	.0	4.1
1970	SEP	12	14	30	53	34.27 N	117.54 W	A	71	.0	5.4
1970	SEP	13	4	47	49	34.28 N	117.55 W	A	70	.0	4.4
1971	FEB	9	14	0	42	34.41 N	118.40 W	B	42	.0	6.4
1971	FEB	9	14	1	8	34.41 N	118.40 W	D	42	.0	5.8
1971	FEB	9	14	1	33	34.41 N	118.40 W	D	42	.0	4.2
1971	FEB	9	14	1	40	34.41 N	118.40 W	D	42	.0	4.1
1971	FEB	9	14	1	50	34.41 N	118.40 W	D	42	.0	4.5
1971	FEB	9	14	1	54	34.41 N	118.40 W	D	42	.0	4.2
1971	FEB	9	14	1	59	34.41 N	118.40 W	D	42	.0	4.1
1971	FEB	9	14	2	3	34.41 N	118.40 W	D	42	.0	4.1
1971	FEB	9	14	2	30	34.41 N	118.40 W	D	42	.0	4.3
1971	FEB	9	14	2	31	34.41 N	118.40 W	D	42	.0	4.7
1971	FEB	9	14	2	44	34.41 N	118.40 W	D	42	.0	5.8
1971	FEB	9	14	3	25	34.41 N	118.40 W	D	42	.0	4.4
1971	FEB	9	14	3	46	34.41 N	118.40 W	D	42	.0	4.1
1971	FEB	9	14	4	7	34.41 N	118.40 W	D	42	.0	4.1
1971	FEB	9	14	4	34	34.41 N	118.40 W	C	42	.0	4.2
1971	FEB	9	14	4	39	34.41 N	118.40 W	D	42	.0	4.1
1971	FEB	9	14	4	44	34.41 N	118.40 W	D	42	.0	4.1
1971	FEB	9	14	4	46	34.41 N	118.40 W	D	42	.0	4.2

TABLE D
Sheet 5 of 11

YEAR	MONTH	DAY	HR	MIN	SEC	LATITUDE	LONGITUDE	Q	DISTANCE	DEPTH	MAGNITUDE
1971	FEB	9	14	5	41	34.41 N	118.40 W	D	42	.0	4.1
1971	FEB	9	14	5	50	34.41 N	118.40 W	D	42	.0	4.1
1971	FEB	9	14	7	10	34.41 N	118.40 W	D	42	.0	4.0
1971	FEB	9	14	7	30	34.41 N	118.40 W	D	42	.0	4.0
1971	FEB	9	14	7	45	34.41 N	118.40 W	D	42	.0	4.5
1971	FEB	9	14	8	4	34.41 N	118.40 W	D	42	.0	4.0
1971	FEB	9	14	8	7	34.41 N	118.40 W	D	42	.0	4.2
1971	FEB	9	14	8	38	34.41 N	118.40 W	D	42	.0	4.5
1971	FEB	9	14	8	53	34.41 N	118.40 W	D	42	.0	4.6
1971	FEB	9	14	10	21	34.36 N	118.31 W	B	35	.0	4.7
1971	FEB	9	14	10	28	34.41 N	118.40 W	D	42	.0	5.3
1971	FEB	9	14	16	13	34.34 N	118.33 W	C	33	.0	4.1
1971	FEB	9	14	19	50	34.36 N	118.41 W	B	37	.0	4.0
1971	FEB	9	14	34	36	34.34 N	118.64 W	C	48	.0	4.9
1971	FEB	9	14	39	18	34.39 N	118.36 W	C	39	.0	4.0
1971	FEB	9	14	40	17	34.43 N	118.40 W	C	44	.0	4.1
1971	FEB	9	14	43	47	34.31 N	118.45 W	B	34	.0	5.2
1971	FEB	9	15	58	21	34.33 N	118.33 W	B	32	.0	4.8
1971	FEB	9	16	19	26	34.46 N	118.43 W	B	48	.0	4.2
1971	FEB	10	3	12	12	34.37 N	118.30 W	B	36	.0	4.0
1971	FEB	10	5	6	36	34.41 N	118.33 W	A	40	.0	4.3
1971	FEB	10	5	18	7	34.43 N	118.41 W	A	44	.0	4.5
1971	FEB	10	11	31	35	34.38 N	118.45 W	A	41	.0	4.2
1971	FEB	10	13	49	54	34.40 N	118.42 W	A	42	.0	4.3
1971	FEB	10	14	35	27	34.36 N	118.49 W	A	40	.0	4.2
1971	FEB	10	17	38	55	34.40 N	118.37 W	A	40	.0	4.2
1971	FEB	10	18	54	42	34.45 N	118.44 W	A	47	.0	4.2
1971	FEB	21	5	50	53	34.40 N	118.44 W	A	42	.0	4.7
1971	FEB	21	7	15	12	34.39 N	118.43 W	A	41	.0	4.5
1971	MAR	7	1	33	41	34.35 N	118.46 W	A	38	.0	4.5
1971	MAR	25	22	54	10	34.36 N	118.47 W	A	39	.0	4.2
1971	MAR	30	8	54	43	34.30 N	118.46 W	A	33	.0	4.1
1971	MAR	31	14	52	23	34.29 N	118.51 W	A	35	.0	4.6
1971	APR	1	15	3	4	34.43 N	118.41 W	A	44	.0	4.1
1971	APR	2	5	40	25	34.28 N	118.53 W	A	36	.0	4.0
1971	APR	15	11	14	32	34.26 N	118.58 W	B	38	.0	4.2
1971	APR	25	14	48	7	34.37 N	118.31 W	B	36	.0	4.0
1971	JUN	21	16	1	8	34.27 N	118.53 W	B	35	.0	4.0
1971	JUN	22	10	41	19	33.75 N	117.48 W	B	79	.0	4.2
1973	FEB	21	14	45	57	34.06 N	119.03 W	B	71	.0	5.9
1974	MAR	9	0	54	32	34.40 N	118.47 W	C	43	.0	4.7
1974	AUG	14	14	45	55	34.43 N	118.37 W	A	43	.0	4.2
1976	JAN	1	17	20	13	33.96 N	117.89 W	A	36	.0	4.2
1976	APR	8	15	21	38	34.35 N	118.66 W	A	50	.0	4.6
1977	AUG	12	2	19	26	34.38 N	118.46 W	B	41	.0	4.5
1977	SEP	24	21	28	24	34.46 N	118.41 W	C	48	.0	4.2
1978	MAY	23	9	16	51	33.91 N	119.17 W	C	85	.0	4.0
1979	JAN	1	23	14	39	33.94 N	118.68 W	B	41	.0	5.0
1979	OCT	17	20	52	37	33.93 N	118.67 W	C	40	.0	4.2
1979	OCT	19	12	22	38	34.21 N	117.53 W	B	70	.0	4.1
1981	SEP	4	15	50	50	33.67 N	119.11 W	C	89	.0	5.3
1981	OCT	23	17	28	17	33.63 N	119.02 W	C	84	.0	4.6
1981	OCT	23	19	15	52	33.64 N	119.06 W	C	87	.0	4.6

**** SEARCH OF EARTHQUAKE DATA FILE 1 ****

* * *

SITE: ADE-85005-8 S.C.R.C. - FLOWER STREET SUBWAY

COORDINATES OF SITE	34.05 N 118.26 W
DISTANCE PER DEGREE	110.9 KM-N 92.3 KM-W
MAGNITUDE LIMITS	4.0 - 8.5
TEMPORAL LIMITS	1932 - 1981
SEARCH RADIUS (KM)	100
NUMBER OF YEARS OF DATA	50
NUMBER OF EARTHQUAKES IN FILE	2789
NUMBER OF EARTHQUAKES IN AREA	295

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***** LEROY CRANDALL AND ASSOCIATES *****

LOS ANGELES

TABLE D
Sheet 7 of 11

LIST OF HISTORIC EARTHQUAKES OF MAGNITUDE 6.0 OR
GREATER WITHIN 100 KM OF THE SITE
(RICHTER DATA 1906-1931)

YEAR	MONTH	DAY	HR	MIN	SEC	LATITUDE	LONGITUDE	Q	DISTANCE	DEPTH	MAGNITUDE
1910	MAY	15	15	47	0	33.70 N	117.40 W	D	88	.0	6.0
1923	JUL	23	7	30	26	34.00 N	117.25 W	D	93	.0	6.3

**** SEARCH OF EARTHQUAKE DATA FILE 2 ****

***

SITE: ADE-85005-8 S.C.R.C. - FLOWER STREET SUBWAY

COORDINATES OF SITE	34.05 N 118.26 W
DISTANCE PER DEGREE	110.9 KM-N 92.3 KM-W
MAGNITUDE LIMITS	6.0 - 8.5
TEMPORAL LIMITS	1906 - 1931
SEARCH RADIUS (KM)	100
NUMBER OF YEARS OF DATA	26
NUMBER OF EARTHQUAKES IN FILE	35
NUMBER OF EARTHQUAKES IN AREA	2

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***** LEROY CRANDALL AND ASSOCIATES *****

LOS ANGELES

TABLE D
Sheet 8 of 11

LIST OF HISTORIC EARTHQUAKES OF MAGNITUDE 7.0 OR GREATER WITHIN 100 KM OF THE SITE
(NOAA/CDMG DATA 1812-1905)

YEAR	MONTH	DAY	HR	MIN	SEC	LATITUDE	LONGITUDE	Q	DISTANCE	DEPTH	MAGNITUDE
1890	FEB	9	4	6	0	34.00 N	117.50 W	D	70	.0	7.0

**** SEARCH OF EARTHQUAKE DATA FILE 3 ****

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SITE: ADE-85005-8 S.C.R.C. - FLOWER STREET SUBWAY

COORDINATES OF SITE	34.05 N 118.26 W
DISTANCE PER DEGREE	110.9 KM-N 92.3 KM-W
MAGNITUDE LIMITS	7.0 - 8.5
TEMPORAL LIMITS	1812 - 1905
SEARCH RADIUS (KM)	100
NUMBER OF YEARS OF DATA	94
NUMBER OF EARTHQUAKES IN FILE	9
NUMBER OF EARTHQUAKES IN AREA	1

***

***** LEROY CRANDALL AND ASSOCIATES *****
LOS ANGELES

TABLE D
Sheet 9 of 11

***** SUMMARY OF EARTHQUAKE SEARCH *****

***

NUMBER OF HISTORIC EARTHQUAKES WITHIN 100 KM RADIUS OF SITE

MAGNITUDE RANGE	NUMBER
4.0 - 4.5	206
4.5 - 5.0	64
5.0 - 5.5	18
5.5 - 6.0	5
6.0 - 6.5	4
6.5 - 7.0	0
7.0 - 7.5	1
7.5 - 8.0	0
8.0 - 8.5	0

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***** LEROY CRANDALL AND ASSOCIATES *****
LOS ANGELES

TABLE D
Sheet 10 of 11

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****  COMPUTATION OF RECURRENCE CURVE  ****
              LOG N = A - BM

                    ***

BIN   MAGNITUDE      RANGE        NO/YR (N)

 1      4.00      4.00 - 8.50     5.92
 2      4.50      4.50 - 8.50     1.80
 3      5.00      5.00 - 8.50     .519
 4      5.50      5.50 - 8.50     .159
 5      6.00      6.00 - 8.50     .585E-01
 6      6.50      6.50 - 8.50     .588E-02 NU
 7      7.00      7.00 - 8.50     .588E-02 NU
 8      7.50      7.50 - 8.50     .000
 9      8.00      8.00 - 8.50     .000

  A = 1.154    B =  .5639     (NORMALIZED)
  A = 4.807    B = 1.0130    SIGMA =   .356E-01

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               LOS ANGELES
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TABLE D
Sheet 11 of 11

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          ***** COMPUTATION OF DESIGN MAGNITUDE *****
                        CONSTANT AREA

                             ***

                  TABLE OF DESIGN MAGNITUDES

RISK      RETURN PERIOD (YEARS)          DESIGN MAGNITUDE

                             DESIGN LIFE (YEARS)
             25     50     75    100      25     50     75    100

 .01 ..    2487   4974   7462   9949 ..  7.96   8.15   8.24   8.29
 .05 ..     487    974   1462   1949 ..  7.37   7.63   7.78   7.88
 .10 ..     237    474    711    949 ..  7.07   7.36   7.52   7.62
 .20 ..     112    224    336    448 ..  6.76   7.05   7.22   7.33
 .30 ..      70    140    210    280 ..  6.56   6.86   7.02   7.14
 .50 ..      36     72    108    144 ..  6.28   6.57   6.75   6.87
 .70 ..      20     41     62     83 ..  6.04   6.34   6.51   6.63
 .90 ..      10     21     32     43 ..  5.77   6.06   6.24   6.36

                MMIN =   4.00     MMAX =  8.50
                MU =     5.69     BETA = 2.332

                             ***

          ***** LEROY CRANDALL AND ASSOCIATES *****
                         LOS ANGELES
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