

R05
S603.00
06/09/93
A2144
\$25.00

VOLUME 1

REPORT OF
GEOTECHNICAL INVESTIGATION
PROPOSED METRO PASADENA LINE
UNION STATION IN LOS ANGELES TO EAST OF
SIERRA MADRE VILLA AVENUE IN PASADENA, CALIFORNIA
FOR THE
LOS ANGELES COUNTY
METROPOLITAN TRANSPORTATION AUTHORITY (MTA)

(L92045.AE4)

JUNE 9, 1993



LAW/CRANDALL, INC.
ENGINEERING AND ENVIRONMENTAL SERVICES
ONE OF THE LAW COMPANIES

VOLUME 1

REPORT OF
GEOTECHNICAL INVESTIGATION
PROPOSED METRO PASADENA LINE
UNION STATION IN LOS ANGELES TO EAST OF
SIERRA MADRE VILLA AVENUE IN PASADENA, CALIFORNIA
FOR THE
LOS ANGELES COUNTY
METROPOLITAN TRANSPORTATION AUTHORITY (MTA)

(L92045.AE4)

JUNE 9, 1993



LAW/CRANDALL, INC.

ENGINEERING AND ENVIRONMENTAL SERVICES

ONE OF THE LAW COMPANIES



LAW/CRANDALL, INC.
ENGINEERING AND ENVIRONMENTAL SERVICES

June 9, 1993

Engineering Management Consultant (EMC)
707 Wilshire Boulevard, Suite 2900
Los Angeles, California 90017

Your File No. R05CA100
L/C Job No. L92045.AE4

Attention: Mr. Erik Collett

Subject: Geotechnical Report
Proposed Metro Pasadena Line
for the Los Angeles County Metropolitan Transportation Authority (MTA)

Ladies/Gentlemen:

We are pleased to submit this report of our geotechnical services for the proposed Metro Pasadena Line. Our authorization to perform this investigation was provided by Mr. Erik Collett in accordance with our proposal dated October 26, 1992. A draft report was submitted on February 26, 1993.

The report is presented in two volumes. Volume 1 contains a description of the geotechnical conditions and the presentation of the earthwork and foundation design data. Volume 2 contains the results of the field explorations, laboratory tests, seismicity evaluation, chemical testing, and corrosion study.

We have discussed the findings and recommendations contained in our draft report with members of your staff. The cooperation and guidance provided by Mr. Bomi Ghadiali and others with EMC are sincerely appreciated. We are available to discuss our recommendations and to provide additional studies or services necessary to complete the project.

June 9, 1993
(L92045.AE4)

We have enjoyed assisting you and look forward to serving as your geotechnical consultant on the remainder of this project and future projects.

Respectfully submitted,

LAW/CRANDALL, INC.

Paul Elliott

Paul Elliott, C.E.G. 1435
Senior Engineering Geologist

J. Kharraz

Jake Kharraz
Principal Engineer

Marshall Lew

Reviewed by: Marshall Lew, Ph.D.
Vice President
Corporate Director of Engineering

metro/MS/mrr
(15 copies submitted)



TABLE OF CONTENTS

Volume 1

	<u>Page No.</u>
SECTION 1. EXECUTIVE SUMMARY	1.1
SECTION 2. INTRODUCTION	2.1
2.1 General	2.1
2.2 Purpose of Investigation	2.2
2.3 Scope of Work	2.2
2.4 Report Structure	2.3
2.5 Limitations and Basis for Recommendations	2.4
SECTION 3. PROJECT DESCRIPTION	3.1
3.1 General	3.1
3.2 Union Station to the Los Angeles River	3.1
3.3 Los Angeles River to Foothill Freeway	3.2
3.4 Foothill Freeway to Sierra Madre Villa Avenue	3.4
Figure 3.1 - System Map	after 3.5
SECTION 4. GEOLOGY	4.1
4.1 Physical Setting	4.1
4.2 Geologic Materials	4.2
4.3 Ground Water	4.5
4.4 Geologic Structure	4.12
4.5 Geologic Hazards	4.13
4.6 Bibliography	4.36
Table 4-1 - Water Level Measurements from Observation Wells	4.10
Table 4-2 - Summary of Ground Water Quality Analyses	after 4.11
Table 4-3 - Summary of Detected Volatile Organic Compounds (VOCs) in Ground Water	after 4.11
Table 4-4 - Criteria for Classification of Faults, with Regard to Seismic Activity	4.14
Table 4-5 - Major Named Faults Considered to be Active in Southern California	4.15
Table 4-6 - Major Named Faults Considered to be Potentially Active in Southern California	4.16
SECTION 5. FIELD EXPLORATIONS AND LABORATORY TESTS	5.1
5.1 Field Exploration Program	5.1
5.2 Piezometer Installation	5.3
5.3 Laboratory Testing Program	5.3
5.4 Soil and Water Corrosivity Testing	5.4
Table 5-1 - Summary of Borings	5.5
SECTION 6. SUBSURFACE CONDITIONS	6.1
6.1 Subsurface Investigation	6.1
6.2 Summary of Soil Conditions	6.1
6.3 Summary of Ground Water Conditions	6.2
SECTION 7. SLOPE STABILITY ANALYSES	7.1



TABLE OF CONTENTS
Volume 1

	<u>Page No.</u>
SECTION 8. RECOMMENDATIONS - AERIAL STRUCTURES	8.1
8.1 General	8.1
8.2 Design Group 1	8.3
Figure 8.1 and 8.2 - Drilled Pile Capacities	
Figure 8.3 - Driven Pile Capacities	
Figure 8.4 - Lateral Deflection Diagrams	
Figure 8.5 - (P-Y) Curves	
Figure 8.6 - Bending Moment Diagram	
Figure 8.7 - Pile Driving Criteria	
8.3 Design Group 2	8.14
Figure 8.8 and 8.9 - Drilled Pile Capacities	
Figure 8.10 - Driven Pile Capacities	
Figure 8.11 - Lateral Deflection Diagrams	
Figure 8.12 - (P-Y) Curves	
Figure 8.13 - Bending Moment Diagram	
Figure 8.14 - Pile Driving Criteria	
8.4 Design Group 3	8.25
Figure 8.15 and 8.16 - Drilled Pile Capacities	
Figure 8.17 - Driven Pile Capacities	
Figure 8.18 - Lateral Deflection Diagrams	
Figure 8.19 - (P-Y) Curves	
Figure 8.20 - Bending Moment Diagram	
Figure 8.21 - Pile Driving Criteria	
SECTION 9. RECOMMENDATIONS - AT-GRADE STRUCTURES/ TRACK BEDS	9.1
9.1 General	9.1
9.2 Foundations	9.2
9.3 Overhead Contact System	9.6
9.4 Tracts on Grade	9.7
9.5 Slabs on Grade	9.7
9.6 Asphaltic Paving	9.8
SECTION 10. RECOMMENDATIONS - SP CORNFIELD YARD AND SHOPS	10.1
10.1 Foundation Support	10.1
10.2 Walls Below Grade	10.3
10.3 Slabs on Grade	10.3
10.4 Paving	10.4
SECTION 11. RECOMMENDATIONS - BELOW-GRADE CONSTRUCTION (DEPRESSED AREAS)	11.1
11.1 General	11.1
11.2 Foundations	11.2
11.3 Excavation and Slopes	11.4
11.4 Shoring	11.5
11.5 Walls Below Grade	11.16
11.6 Subdrain	11.18

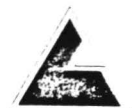


TABLE OF CONTENTS
Volume 1

	<u>Page No.</u>
SECTION 12. RECOMMENDATIONS - MISCELLANEOUS STRUCTURES	12.1
12.1 Retaining Walls	12.1
12.2 Slough Walls	12.5
12.3 Culverts	12.5
12.4 Sign Structures	12.6
SECTION 13. RECOMMENDATIONS - EARTHWORK	13.1
13.1 Site Preparation	13.1
13.2 Excavation and Slopes	13.2
13.3 Compacted Fill	13.2
13.4 Erosion Protection	13.3
13.5 Material for Fill	13.3
13.6 Fill Settlement	13.4
13.7 Grading Observation	13.4

Plates

	<u>Plate No.</u>
Geologic Map and Boring Location Plan	1.1 - 1.32
Geologic Sections	2.1 - 2.10
Local Geologic Map	3.1 - 3.6
Regional Geology	4
Regional Seismicity	5
Recurrence Curve	6
Potential Slope Stability Hazards Map	7
Flood and Inundation Hazards Map	8



VOLUME 1

**REPORT OF
GEOTECHNICAL INVESTIGATION
PROPOSED METRO PASADENA LINE
UNION STATION IN LOS ANGELES TO EAST OF
SIERRA MADRE VILLA AVENUE IN PASADENA, CALIFORNIA
FOR THE
LOS ANGELES COUNTY
METROPOLITAN TRANSPORTATION AUTHORITY (MTA)**



SECTION 1.0

EXECUTIVE SUMMARY

Law/Crandall has performed a geotechnical investigation for the proposed Metro Pasadena Line. The proposed line, which will be about 13.6 miles long, begins at Union Station in Los Angeles, ends east of Sierra Made Villa Avenue in Pasadena, and connects the business centers of Los Angeles and Pasadena. The project will be mostly at-grade and will follow the existing Atchison Topeka & Santa Fe (ATSF) Railroad right-of-way. The project will include an aerial structure from Union Station to north of Chinatown, with an elevated passenger station at Chinatown. The Metro Pasadena Line will also use existing ATSF bridges to be rebuilt or upgraded. There will be at-grade structures (passenger service stations, power substations, traction power stations, and walls). A yard and shops development is planned within the Southern Pacific (SP) Cornfield Property, near the southern end of the alignment.

Field explorations consisted of geologic reconnaissance and mapping and subsurface drilling, logging, and sampling. As part of the geotechnical investigation, 122 borings were drilled. Twenty-nine of these borings were drilled during prior preliminary geotechnical studies. Comprehensive laboratory testing was performed; the laboratory testing program was conducted as the samples were obtained and brought to the laboratory.

Portions of the alignment are locally underlain by artificial fill. Fill soils were encountered in most of the borings and ranged from 1 to 34 feet in depth; the deeper fill soils were encountered in the majority of borings drilled at the top of railroad embankments and along the Foothill Freeway. The overburden natural soils along the alignment consist of topsoil, colluvium, landslide debris, alluvium, and terrace deposits. Bedrock of the Puente and Topanga Formations, consisting of interbedded siltstone and sandstone, was encountered in 34 of the borings at depths ranging from 2 to 97 feet below the existing grade. The Puente Formation bedrock is exposed along the flanks of the Elysian Hills and in the Mt. Washington area. The Topanga Formation bedrock is exposed in the Highland Park, northern Monterey



Hills, and Raymond Hill areas. Except for the upper few feet, the overburden natural soils are generally firm and dense. The bedrock encountered in the borings is firm to very firm. Water seepage was encountered in 16 borings at depths ranging from 5 to 58 feet below the existing grade. Ground water monitoring wells (piezometers) were constructed in 15 of the exploratory borings for future water level readings and sampling. Water levels measured in the wells varied from depths of 7½ to 60 feet along the alignment. In general, the ground water table beneath the alignment varies from about 7½ to over 400 feet below the existing grade.

The major geologic structural features in the project area are the Elysian Park Structure, which is reflected at the surface by the Elysian Park and Repetto Hills, and the Raymond fault, which parallels the Arroyo Seco Channel. The proposed alignment is underlain by geologic units consisting of sedimentary bedrock units of the Topanga and Puente Formations, terrace deposits, older alluvium, landslide debris, alluvium, colluvium, topsoil, and artificial fill. The alignment traverses a known landslide on the north-facing slope, adjacent to Arroyo Seco Park. A separate, smaller landslide exists west of the larger slide.

The geologic hazards along the alignment are limited to those caused by slope instability and earthquakes. A portion of the alignment lies within an established Alquist Priolo Special Studies Zone for fault rupture hazard. This special studies zone has been established along a portion of the Raymond fault and is located along the alignment between Magnolia Street and about 250 feet north of Oak Lawn Avenue in the City of South Pasadena. In our opinion, there is a potential of surface fault rupture occurring beneath the proposed alignment due to the active Raymond fault and the potentially active Eagle Rock-San Rafael fault that cross the proposed alignment. The potential for geologic hazards, such as fault rupture, slope instability, and settlement is considered low for the majority of the alignment. Also, the potential for liquefaction is generally considered to be low.



There are seven areas along the alignment with significant potential for slope stability problems. The potential for slope instability within each area was evaluated, and bedding orientations were found to be generally favorable along most of the alignment. However, in the vicinity of Marmion Way, between Avenue 45 and Museum Drive in the Mt. Washington area, locally adverse bedding planes were observed. Our stability analyses show the existing landslide masses and the slopes to be grossly stable with a factor of safety of more than 1.5. One area, located east of the existing Arroyo Seco Park railroad bridge, was determined to have a factor of safety of less than 1.5 due to the steepness and height of the slope. The slope face is subject to severe erosion and rockfall. It is our opinion that the current slope conditions in this area would present difficulties in continuous operation of the line, especially when there is precipitation. To reduce the hazards associated with this slope, the inclination of the slope could be flattened and/or slough walls constructed. Consideration could also be given to construction of a cover structure in this area over the alignment. Also, an early warning system should be designed for this portion of the alignment to allow stopping operations in the event of a serious slope failure. Another alternative would be to modify the alignment to avoid this area.

The aerial structures, which include bridges to be rebuilt or upgraded and elevated stations, may be supported on drilled cast-in-place concrete piling. Some of the minor and lighter columns of the aerial structures, such as bridge piers and bridge abutments, may be supported on driven piles or small-diameter drilled piles. Spread footings may be a feasible foundation alternative at selected locations. With proper grading, typical light at-grade structures (including passenger service stations, power substations, traction power stations, yard and shop facilities, and retaining walls) could be supported on shallow spread footings established in compacted fill or the underlying natural soils.

The Avenue 26 Station will be located on the fill embankment that extends between the Los Angeles River and Avenue 33. The tracks may be supported on grade at this location. However, supporting the station on the existing fill may result in unacceptable differential



settlement between the tracks and the station platform. Accordingly, we recommend that the station be supported on drilled piles extending through the fill and into the underlying natural soils.

For support of heavier at-grade structures, including heavier passenger service station platforms and major retaining walls, spread footings should extend into the underlying undisturbed firm and dense natural materials. Conventional spread footings may be used where the firm and dense natural materials are exposed at or close to final grade; deep spread footings, straight drilled shafts, and/or drilled piling may be used in the deeper fill areas.

There will be below-grade construction along portions of the line in Highland Park and in Pasadena. Dewatering and a subdrain system will be required for the depressed area at the Highland Park location. Adverse bedding was encountered in the bedrock underlying the area, and east facing shoring and walls below grade where bedrock is encountered should be designed for the adverse bedding. The right-of-way within a portion of the alignment in Pasadena is only 30 feet wide, and the proposed excavation for the depressed area will extend close to some of the various structures adjacent to the alignment. Underpinning of some of the existing footings that are close to the depressed area may be necessary. As an alternative, the shoring and walls below grade could be designed to support the loads imposed by the adjacent existing foundations. The retaining walls for the depressed areas may be supported on spread footings at the excavation levels.



SECTION 2.0 INTRODUCTION

2.1 GENERAL

This report presents the findings, conclusions, and recommendations of a geotechnical investigation performed to assist in the design of the proposed Metro Pasadena Line.

The findings and conclusions developed during this geotechnical investigation are presented in two separate volumes. Volume 1 presents the text that includes data analyses, interpretative information, conclusions, and recommendations. Volume 2 presents the Appendices that include the detailed results of field and laboratory programs, earthquake data base search, chemical testing, and corrosion studies.

We previously performed preliminary geotechnical investigations for:

- the portion of the alignment between Union Station and the Los Angeles River; the results of that investigation were presented in a report dated June 24, 1992 (L92045.AE1).
- the portion of the alignment between the Los Angeles River and the Foothill Freeway (Interstate 210 in Pasadena); the results of that investigation were submitted in a report dated December 4, 1992 (L92045.A3).

The results of the prior studies have been incorporated herein.

The locations of the proposed alignment and our exploration borings are shown on Plates 1.1 through 1.32, Geologic Map and Boring Location Plan.



2.2 PURPOSE OF INVESTIGATION

The purpose of the investigation was to determine and evaluate the geotechnical conditions along the proposed alignment with regard to their possible effects on the design and construction of the planned Metro-Pasadena Line. More specifically, the investigation included the following objectives:

- Evaluate the existing surface and subsurface conditions, including the soils and ground water conditions, along the proposed alignment.
- Define the geologic environment and evaluate geologic/seismic hazards that may affect the project.
- Provide recommendations for design and construction.

2.3 SCOPE OF WORK

The following services were undertaken for the geotechnical investigation:

- Review of our prior preliminary investigations and previous investigations in the immediate vicinity of the alignment and compilation of pertinent data;
- Field exploration program to determine the nature and stratigraphy of the project soils and to obtain bulk and relatively undisturbed samples for laboratory testing;
- Laboratory testing of soil samples to classify the soils and to determine the static soil properties, including moisture and density determinations, direct and triaxial shear, consolidation, compaction and California Bearing Ratio, Expansion Index, grain size, and Atterberg limits;
- Geologic evaluation to define the geologic environment and evaluate the geologic/seismic hazards that may affect the project;
- Chemical testing of water and soil samples;
- Chemical testing to determine the corrosion potential of the water and soils underlying the alignment; and
- Preparation of geotechnical report.



Our LACMTA-certified Disadvantaged Business Enterprise (DBE) subcontractor for the project was Lockwood-Singh and Associates of Los Angeles, California.

2.4 REPORT STRUCTURE

This report, consisting of two volumes, has been divided in to 13 basic sections, and five appendices.

Volume 1

Section 1, Executive Summary: summation of the important aspects of the report

Section 2, Introduction: purpose, scope, report structure, and limitations

Section 3, Project Description: structural features of the project

Section 4, Project Geology: geologic and seismic information and evaluation

Section 5, Field Explorations and Laboratory Tests: field and laboratory work

Section 6, Subsurface Conditions: subsurface conditions along the alignment

Section 7, Slope Stability: a brief description of the analyses performed

Section 8, Recommendations – Aerial Structures

Section 9, Recommendations – At-Grade Structures and Track Beds

Section 10, Recommendations – Southern Pacific (SP) Cornfield Yard and Shops

Section 11, Recommendations – Below-Grade Construction (Depressed Areas)

Section 12, Recommendations – Miscellaneous Structures

Section 13, Recommendations – Earthwork

Volume 2

Appendix A – Field Explorations

Appendix B – Laboratory Tests

Appendix C – Earthquake Data Base Search

Appendix D – Chemical Testing Data

Appendix E – Corrosion Study



2.5 LIMITATIONS AND BASIS FOR RECOMMENDATIONS

Our professional services have been performed using that degree of care and skill ordinarily exercised, under similar circumstances, by reputable geotechnical consultants 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 Engineering Management Consultant (EMC) for evaluation of the proposed Metro Pasadena Line. 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.

The recommendations provided in this report are based upon our understanding of the described project information and on our interpretation of the data collected during the subsurface exploration. We have made our recommendations based upon experience with similar subsurface conditions under similar loading conditions. The recommendations apply to the specific project discussed in this report; therefore, any change in the alignment or project status should be provided to us so that we may review our conclusions and recommendations and make any necessary modifications.



SECTION 3.0

PROJECT DESCRIPTION

3.1 GENERAL

The proposed Metro Pasadena Line begins at Union Station in Los Angeles, ends east of Sierra Madre Villa Avenue in Pasadena, and connects the business centers of Los Angeles and Pasadena. The alignment will be about 13.6 miles long, and may be divided into three segments: Union Station to the Los Angeles River; the Los Angeles River to the Foothill Freeway (Interstate 210); and Foothill Freeway to the east of Sierra Madre Villa Avenue. Upon crossing the Los Angeles River, the alignment follows the Atchison, Topeka, and Santa Fe (ATSF) Railroad right-of-way. It is planned to convert the existing single-track railroad system to a two-track system. The alignment segments are discussed in detail below. The alignment is shown on Figure 3.1, System Map.

3.2 UNION STATION TO THE LOS ANGELES RIVER

This segment of the alignment is about 2.4 miles long. It begins at grade at Platform 1 of Union Station near the Los Angeles Civic Center and extends north to the east side of the Terminal Annex Post Office and near the east side of a 15-foot-high retaining wall. The area of Platform 1 was raised above the surrounding street grade by placing up to 15 feet of fill. Next, the alignment veers to the west outside the rail tracks of Union Station and extends over the retaining wall, where the alignment becomes an aerial structure. The alignment then becomes parallel to Vignes Street adjacent to the parking lot behind the Terminal Annex Post Office building, and continues over the intersection of Main Street, Alpine Street, and Alhambra Avenue, and over Alameda Street. From there it extends to the north and to the northeast over North Spring Street where the elevated Chinatown platform station will be constructed at College and Spring Streets. The alignment continues by descending to grade into the westerly edge of the Southern Pacific (SP) Cornfield Property. After that, the alignment proceeds northerly, eventually paralleling the base of a steep embankment that separates the SP Cornfield Property from North Broadway at the top of the embankment.



The alignment then crosses the Los Angeles River at the existing ATSF Bridge. We understand that the ATSF Bridge that may be replaced is supported on a combination of shallow spread footings and short drilled piles established in the bedrock. A traction power station is planned to the north of the Broadway Bridge.

The following passenger service stations are planned along this segment of the alignment:

- Union Station
- Chinatown Platform Station

A yard and shops development is planned within the Southern Pacific (SP) Cornfield Property. The development will consist of a maintenance building, vehicle maintenance shop, car wash, two traction power stations, Department of Water and Power substation, trash bin, trash compactor, storage area, and paved areas for automobile and truck parking, and for truck loading and unloading. The site plan of the yard and shops development is shown on Plate 1.4.

3.3 LOS ANGELES RIVER TO THE FOOTHILL FREEWAY (INTERSTATE 210)

This segment of the alignment follows the existing Atchison, Topeka and Santa Fe (ATSF) Railroad right-of-way. The length of this segment is about 7.4 miles. After crossing the Los Angeles River the alignment crosses the Mount Washington and Highland Park Districts of Los Angeles, through the City of South Pasadena and into the City of Pasadena where this segment ends at the Foothill Freeway (Interstate 210). Although the alignment crosses over the Pasadena Freeway, Golden State Freeway, Arroyo Seco Channel, and numerous streets, the majority of the alignment will be at grade. The portion of the alignment between the Los Angeles River and Avenue 33 is on fill embankments.

The following passenger service stations are planned along this segment of the alignment:



Avenue 26 Station, Los Angeles	Mission Station, South Pasadena
Marmion Station, Los Angeles	Fillmore Station, Pasadena
Avenue 51 Station, Los Angeles	Del Mar Station, Pasadena
Avenue 57 Station, Los Angeles	Memorial Park Station, Pasadena

A transportation park and ride development may be constructed adjacent to the Del Mar Station.

It is planned to upgrade the following railroad bridges located along this segment of the alignment.

Avenue 19 Bridge	Arroyo Seco Channel and Pasadena Freeway Bridge north of Pasadena Avenue
San Fernando Road Bridge	Arroyo Seco Park and Pasadena Freeway Bridge
Golden State Freeway Bridge	Pasadena Freeway Bridge near Fremont Avenue
Avenue 26 Bridge	

We understand that a depressed grade separation is planned in Highland Park at the intersection of Figueroa Street and Marmion Way, and that excavation up to 30 feet deep will be required.

A 2,000-foot-long depressed area that will extend about 40 feet below the existing grade is planned in Pasadena between Del Mar Boulevard and Holly Street. The excavation for the depressed area will extend close to some of the various structures adjacent to the alignment.

A 2,300-foot-long retaining wall is planned along the hillside in the Arroyo Seco Park area. Another retaining wall is planned between Cawston Street and Orange Grove Avenue, and



a 40-foot-high retaining wall is planned adjacent to Marengo Avenue where a cut in the hillside is planned to provide space for a traction power station.

Numerous sound walls are planned along this segment of the alignment. A 6-foot-high sound wall is planned along the alignment starting just to the south of Avenue 42 and extending to Avenue 58. Another 8-foot-high sound wall starts at the intersection of Marmion Way and Arroyo Drive and extends to the intersection of Arroyo Verde Road and Sycamore Avenue. A third 4-foot-high sound wall is planned between Orange Grove Avenue and Glendon Way, and a fourth 4-foot-high sound wall is planned on both sides of the alignment starting west of Hope Street and extending to the intersection of the alignment with the Pasadena Freeway.

Power substations and traction power stations are planned at the following locations:

Adjacent to Avenue 26	Adjacent to Mission Station
Adjacent to the intersection of Arroyo Seco Avenue and Marmion Way	Near the intersection of Fair Oaks Avenue and Pasadena Freeway
Near the intersection of Avenue 50 and Marmion Way	Adjacent to Del Mar Boulevard Station
Adjacent to Avenue 61	Adjacent to Marengo Avenue
To the west of the intersection of Monterey Road and Lincoln Park	

3.4 FOOTHILL FREEWAY TO SIERRA MADRE VILLA AVENUE

This segment of the alignment also uses the existing ATSF railroad right-of-way in the City of Pasadena; the length of this segment is about 3.8 miles. After the railroad exits the tunnel beneath Marengo Avenue and the eastbound lanes of the Foothill Freeway (Interstate 210), the alignment extends eastward along the median of the Foothill Freeway. The majority of this segment is at grade, except where the freeway crosses over Sierra Madre, San Gabriel, and



Foothill Boulevards, and over Sierra Bonita, Allen, Craig, and Sierra Madre Villa Avenues, and over Altadena Drive.

The following passenger service stations are planned along this segment:

- Lake Station
- Allen Station
- Sierra Madre Villa Station

The existing bridge over Lake Avenue may be widened, and a pedestrian overpass is planned over the Foothill Freeway near the Sierra Madre Villa Avenue Station. Traction power stations are planned adjacent to the alignment at Hill Avenue and Altadena Drive.



SECTION 4.0 GEOLOGY

4.1 PHYSICAL SETTING

The southerly portion of the proposed alignment is located within the Los Angeles Basin at the northerly end of the Peninsular Ranges geomorphic province. This geomorphic province is characterized by elongated northwest-trending mountain ridges separated by straight-sided sediment-filled valleys (Yerkes et al., 1965). This northwest trend is further reflected in the direction of the dominant geologic structural features in the province that include the Elysian Park Structure and the nearby Newport-Inglewood fault zone. At the Raymond fault, the alignment extends north into the Transverse Ranges geomorphic province. In contrast to the Peninsular Ranges geomorphic province, the geologic structures in this geomorphic province trend generally east-west. The Sierra Madre fault zone and the San Rafael-Eagle Rock fault are within the Transverse Ranges geomorphic province.

The southernmost portion of the alignment is located within the eastern Los Angeles coastal plain where the alignment turns north to northeast along the easterly flank of the Elysian Park Hills. Beyond the Elysian Park Hills, the alignment crosses the Los Angeles River and trends northeasterly through the Repetto Hills, parallelling the Arroyo Seco Channel in the Mt. Washington/Highland Park area. Beyond the Repetto Hills, the alignment passes through Raymond Hill and into the western San Gabriel Valley.

The major geologic structural features in the project area are the Elysian Park Structure, which is reflected at the surface by the Elysian Park and Repetto Hills, and the Raymond fault, which parallels the Arroyo Seco Channel.

The geologic conditions along the alignment are shown on Plates 1.1 through 1.32. The subsurface geologic conditions are shown on Plates 2.1 through 2.10, Geologic Sections. The relationship of the rail transit alignment to local and regional geologic features is shown on Plates 3.1 through 3.6, Local Geologic Map, and on Plate 4, Regional Geology. The alignment



is shown in relation to geomorphic provinces, major fault zones, and earthquake epicenters on Plate 5, Regional Seismicity.

4.2 GEOLOGIC MATERIALS

4.2.1 GENERAL

The proposed alignment is underlain by geologic units consisting of sedimentary bedrock units of the Topanga and Puente Formations, terrace deposits, older alluvium, older landslide debris and landslide debris, alluvium, colluvium, topsoil, and artificial fill. The areal extent of the geologic materials along the alignment is shown on Plates 1.1 through 1.32.

4.2.2 TOPANGA FORMATION (Tt)

Marine sedimentary bedrock of the middle Miocene age Topanga Formation is exposed throughout the Highland Park and northern Monterey Hills areas, and in the Raymond Hill area along the central portion of the alignment.

The Topanga Formation in this area has been mapped by Lamar (1970) as the sandstone, siltstone, and conglomerate members. The Topanga Formation was encountered in Borings 19, 69, 70, 71, 72, 83, 84 and 85 and consists of well-bedded, moderately cemented, siltstone and sandstone with a few conglomerate interbeds within the sandstone.

4.2.3 PUENTE FORMATION (Tp)

Marine sedimentary bedrock of the late Miocene age Puente Formation is exposed along the flanks of the Elysian Park Hills and in the Mt. Washington area, adjacent to the southerly half of the alignment.

The Puente Formation in this area has been divided by Lamar (1970) into siltstone, sandstone and shale members; only the sandstone and shale members are located adjacent to the alignment. The materials of the sandstone member consist of light brown, massive to well-bedded sandstone with minor grey and brown interbeds of siltstone and shale. This



member is generally moderately to well cemented and moderately hard. The shale member consists of light brown and light grey, well bedded, poorly to moderately cemented shale and siltstone with minor interbeds of sandstone. The Puente Formation, which occurs along the alignment south of Avenue 49, was encountered in Borings 1, 3, 7 through 11, 13 through 15, 19, 33 through 35, 44, 46, 47, and 56 through 62.

4.2.4 TERRACE DEPOSITS (Qt)

Pleistocene age alluvial terrace deposits were observed along the northerly portion of the alignment in the Pasadena area, north of the Raymond fault. As encountered in our borings, these materials consist of well consolidated sand and gravel, silty sand, and silt with varying amounts of cobbles and boulders.

4.2.5 OLDER ALLUVIUM (Qalo)

The older alluvium underlies the younger alluvium and is exposed at the surface in the southerly portion of the alignment. These materials form a dissected alluvial surface on either side of the Arroyo Seco and the Los Angeles River channels. As encountered in our exploratory borings, the older alluvial materials consist primarily of poorly to moderately consolidated brown to greyish brown sand and gravel, silt, and silty sand with varying amounts of cobbles and boulders. These materials extend to depths of up to 100 feet beneath portions of the alignment.

4.2.6 OLDER LANDSLIDE DEBRIS (Qlso) AND LANDSLIDE DEBRIS (Qls)

The alignment traverses a known landslide on the north-facing slope, east and south of Arroyo Seco Park as shown on Plate 1.14. Borings 72, 73 and 74, drilled within the limits of this landslide, encountered landslide debris consisting of dark brown and grey, fractured, interbedded sandstone and clayey siltstone. Locally, the sandstone and siltstone occurred in large fragments within a silty sand matrix. Fractures within the landslide debris were locally infilled with caliche.



A separate, smaller landslide exists west of the larger slide (Plate 1.13). Boring 71, drilled within the limits of this landslide, encountered landslide debris of similar composition to that encountered in the larger landslide to the east.

4.2.7 ALLUVIUM (Qal)

The younger alluvium has infilled the Los Angeles River and Arroyo Seco channels and their tributaries. These deposits are Holocene in age and directly underlie most of the southern portion of the alignment. The younger alluvium consists of poorly consolidated silt, silty sand, sand, and gravelly sand with some cobbles and boulders. These materials extend to depths of up to about 40 feet beneath the proposed alignment.

4.2.8 COLLUVIUM

Colluvium mantles the higher slopes adjacent to the alignment and was encountered between depths of 12 to 20 feet, between depths of 3 to 8 feet, and between depths of 13 to 19 feet in Borings 46, 70 and 72, respectively. The colluvium consisted of clay, silt, silty sand, and sand and ranged from 5 to 8 feet in thicknesses in our exploratory borings.

4.2.9 TOPSOIL

Topsoil is a natural soil that has developed from in-place weathering of the Holocene and Pleistocene age alluvial deposits, the Pleistocene age terrace deposits, and the bedrock units. Topsoil was locally encountered in our exploratory borings. The topsoil consisted predominantly of silt and clay. The topsoil ranged from 1 to 9½ feet thick, but was typically 1 to 3 feet thick. The thicker topsoil (greater than 6 feet) was encountered in Borings 57, 61, and 62 along Marmion Way.

4.2.10 ARTIFICIAL FILL (af)

Portions of the alignment are locally underlain by artificial fill. Fill soils were encountered in most of the borings ranging from 1 to 34 feet in depth. The shallow fill soils generally consist



of silt, silty sand, clay, and fine to coarse-grained sand with some gravel and cobbles. Pieces of brick, wood, asphaltic paving, glass, and metal debris were locally encountered in the fill.

Deep fill soils were encountered in the majority of the borings drilled at the top of the railroad embankments and along the Pasadena Freeway east of Michigan Avenue. These fill soils generally consist of silty sand, silt, and sand with gravel and cobbles. Only fill soils greater than 5 feet deep are delineated on Plates 1.1 through 1.32.

4.3 GROUND WATER

4.3.1 GENERAL

The proposed alignment traverses the northeastern portion of the Los Angeles Coastal Plain Hydrologic Subunit and the western portion of the Raymond Hydrologic Subunit. Holocene and Pleistocene age alluvial deposits underlie most of the proposed alignment. In the southern portion of the alignment, south of the Los Angeles River to College Street, the alluvial deposits are underlain by bedrock of the Puente Formation at relatively shallow depths. This condition also occurs in the area of Marmion Way between Mt. Washington Drive and Avenue 49. Additionally, Topanga Formation bedrock units underlie the alluvial deposits at shallow depths between the Arroyo Seco Channel (Boring 69) and Arroyo Verde Road (Boring 19). Water within the bedrock typically occurs as localized seepage along joints, bedding, or coarse grained layers within the rock and is not representative of the regional ground water levels.

4.3.2 GROUND WATER OCCURRENCE

4.3.2.1 General

The occurrence of ground water along the proposed alignment has been evaluated using regional ground water data and by direct measurement of water levels encountered in our exploratory borings and observation wells. Previous data include water level measurements by Los Angeles County Department of Public Works (LACDPW) (1934 to 1988), regional water



level information from the U. S. Geological Survey (1977), and water level measurements from previous exploratory borings drilled in the vicinity of the alignment by us and by others.

4.3.2.2 Union Station to Los Angeles River

Water level measurements by the LACDPW indicate the depth to ground water has historically been about 25 to 40 feet beneath the ground surface along the southern portion of the alignment (south of the Los Angeles River to the Union Station). These records also indicate that ground water levels reported by LACDPW are generally more shallow near the Los Angeles River than near Union Station. The highest recorded water levels in the area over the past 58 years were at a depth of about 24 feet in 1980 in Well No. 2772D and 26 feet in 1938 in Well No. 2774F situated 0.4 mile north and 0.4 mile east of the southern portion of the alignment, respectively.

Regional data compiled by the U.S. Geological Survey (Yerkes et al., 1977) indicate ground water levels in 1977 ranged from a depth of 23 feet in the vicinity of the Los Angeles River to a depth of about 40 feet in the vicinity of Union Station. These ground water levels are consistent with LACDPW data.

Water encountered in our exploratory borings and measured in the observation wells, along this portion of the alignment, indicate the depth to ground water ranges from about 19 feet in Boring 33 to 41 feet in Boring 9. Water seepage was encountered in Boring 8 at depths of 14, 19, and 34 feet, and in Borings 10 and 44 at depths of 28 and 10 feet, respectively. Bedrock was encountered at shallow depths in these borings and the water seepage levels are slightly above or below the top of bedrock. Since the bedrock is generally considered non-waterbearing, the water encountered in these borings is believed to represent a localized perched condition.

Borings 7 and 9 were drilled on top of the bluff adjacent to North Broadway. These borings were drilled for slope stability evaluation purposes and are elevated approximately 40 feet



above the proposed alignment. Water encountered in these borings at depths of 32½ and 41, respectively, is also believed to represent perched water conditions.

4.3.2.3 Los Angeles River to Mt. Washington Drive

Along the portion of the alignment between the Los Angeles River and about Mt. Washington Drive, historic ground water levels by the LACDPW have ranged from 30 to 40 feet below the ground surface. The highest recorded ground water levels in the area were measured at depths of about 24 feet in Well No. 2772D (1980) and 38 feet in Well No. 2781 (1971), situated 1/10 mile and ¼ mile northwest of the alignment, respectively.

Ground water in this area was measured in observation wells constructed in Borings 11, 13, 47, 49, 53, and 56 at depths of approximately 44, 21½, 23, 60, 35, and 35½ feet below the ground surface, respectively. These water levels are consistent with regional ground water levels as indicated by LACDPW well data.

4.3.2.4 Mt. Washington Drive to Avenue 49

From approximately Mt. Washington Drive to Avenue 49, significant ground water was not encountered in our exploratory borings drilled to a maximum depth of 40 feet below the ground surface. Along this portion of the alignment, bedrock of the Puente Formation occurs at relatively shallow depths. Minor water seepage was encountered within the bedrock at a depth of 25 feet in Boring 59. This is a localized condition and is not representative of the regional ground water level.

4.3.2.5 Avenue 49 to Arroyo Seco Channel

Along the portion of the alignment from about Avenue 49 to the Arroyo Seco Channel, the alignment is underlain by older alluvial deposits. A review of logs from previous projects in the vicinity indicates the ground water occurred at a depth of about 10 to 25 feet below the ground surface.



Ground water was not encountered in our exploratory borings drilled to a maximum depth of 30 feet below the existing ground surface in this area. However, minor water seepage was encountered in Borings 18 and 66 at depths of 19½ and 24 feet, respectively. This seepage represents localized perched water in the alluvial deposits and does not represent the regional ground water level. Additionally, water was measured in observation wells constructed in Borings 69 and 70 at depths of 7½ and 13 feet, respectively. The water encountered in these borings represents water from surface infiltration perched on the shallow bedrock.

4.3.2.6 Arroyo Seco Channel to Arroyo Verde Road

Between the Arroyo Seco Channel and approximately Arroyo Verde Road, the alignment is underlain by landslide debris and shallow older alluvial deposits over bedrock of the Topanga Formation. Ground water was not encountered in our exploratory borings drilled in this area. However, water seepage was encountered in Boring 72 at depths of 28½, 37 and 48½ feet, and in Boring 73 at depths of 26 and 28 feet and in Boring 74 at a depth of 58 feet. These borings were drilled within the limits of a known landslide. The seepage is believed to be a localized perched condition within the landslide mass and does not represent ground water levels in the area.

4.3.2.7 Arroyo Verde Road to Pasadena Freeway

Between Arroyo Verde Road and the Pasadena Freeway, the alignment is underlain by older alluvial deposits. Regional ground water maps indicate ground water to be approximately 400 to 500 feet below the ground surface. However, seepage was encountered in Boring 77 at a depth of 21½ feet. The seepage represents a localized perched condition and is not representative of the regional ground water level.

4.3.2.8 Pasadena Freeway to Sierra Madre Villa Avenue

At about the Pasadena Freeway near Fremont Avenue, the Raymond fault crosses the alignment. The fault is a known ground water barrier with ground water north of the fault being impeded, resulting in higher water levels on the north side of the fault. North of the



fault, the depth to ground water is approximately 100 feet below the ground surface. Ground water levels south of the fault are between 400 and 500 feet below the ground surface. This portion of the alignment is primarily underlain by Pleistocene age older alluvium and terrace deposits. Recent water level measurements by the LACDPW indicate that the regional water level in the vicinity of the alignment ranges from 140 feet below the ground surface in the vicinity of Raymond Hill to 300 feet below the ground surface in the vicinity of Sierra Madre Villa Avenue and Interstate 210. The highest recorded ground water along this portion of the alignment was at a depth of 99 feet below the ground surface in 1944 in Well No. 4056C, situated approximately 800 feet northwest of the intersection of the alignment and Glenarm Street, and at a depth of 232 feet below the ground surface in 1946 in Well No. 4085A, situated approximately 800 feet south of the intersection of Allen Avenue and Interstate 210, near the northeasterly terminus of the alignment.

Data from our exploratory borings indicate that ground water was not encountered along this portion of the alignment. However, in Borings 29, 118, and 122, slight water seepage was encountered at depths of 18, 19, and 5 feet, respectively. This condition is believed to represent a perched water condition within the alluvial deposits and artificial fill at these locations and is not representative of a regional ground water condition.

4.3.3 WATER LEVEL MEASUREMENTS

Ground water monitoring wells (piezometers) were constructed in 15 of the exploratory borings drilled with rotary wash-drilling equipment to depths of 51 to 101 feet. The locations of the wells were selected to monitor the water levels at the locations where deep foundations are expected along the alignment. The wells were constructed by installing a 2-inch-diameter PVC wall casing in each of the borings. Detailed descriptions of the piezometer installations are presented on the boring logs, and the details of the well construction are presented on Plates A-1.124 through A-1.137 of Appendix A, Volume 2. A summary of the water level measurements and measurement dates in the piezometers is presented in Table 4-1.



Table 4-1
Water Level Measurements From Observation Wells

Boring	Water Level (ft.)	Date Measured
1	40	05/27/92
	39	01/27/92
3	21½	05/05/92
	21½	05/25/92
10	16½	05/05/92
	16½	05/25/92
	18½	12/04/92
	18½	12/30/92
11	43½	09/01/92
	44	09/22/92
	44½	12/04/92
	44½	12/30/92
13	27	09/22/92
	21½	12/04/92
	21½	12/30/92
33	19	02/15/93
34	24½	12/30/92
35	25½	01/20/92
36	29	12/30/92
	29	01/27/93
47	23½	12/30/92
	23	01/27/93
49	60	12/30/92
	60	01/27/93
53	35½	12/30/92
	38	01/08/93
	35	01/27/93
56	35½	12/30/92
69	7½	02/15/93
70	13½	12/30/92
	13	01/27/93



4.3.4 WATER QUALITY

Water quality data collected during the exploration program are summarized in Table 4.2, Summary of Ground Water Quality Analyses, and Table 4.3, Summary of Detected Volatile Organic Compounds (VOCs) in Ground Water, presented on the following pages. The complete analytical results and Chain-of-Custody documentation are presented in Appendix D.

The laboratory results indicate that ground water samples from Borings 11 and 13 had levels of turbidity, settleable solids, total suspended solids (TSS) and Biochemical Oxygen Demand (BOD) in excess of the Regional Water Quality Control Board's (RWQCB) maximum discharge limitations for these constituents. The ground water sample from Boring 13 also had levels of total dissolved solids (TDS), nitrates, sulfate, and lead in excess of the maximum discharge limitations for these constituents. Ground water samples from the remaining borings had various constituents in excess of the maximum discharge limitations, including Boring 36 (turbidity); Boring 47 (turbidity, settleable solids, chromium, and lead); Boring 49 (turbidity and TDS); Boring 53 (turbidity, settleable solids, TDS, TSS, and chromium); Boring 56 (nitrate and TDS); and Boring 70 (sulfide, turbidity, TDS, settleable solids, TSS, chlorides, sulfate, chromium, and lead). However, it is our opinion that the observed turbidity, total suspended solids (TSS), and metals levels would likely decrease with additional development of these wells.

It is our opinion that despite the high detected levels of turbidity and TSS, it is likely that ground water from the vicinity of Borings 11, 36, 47, 49, and 53 could be discharged into a nearby storm drain under an NPDES permit, as the detected constituents discussed above are naturally occurring and the observed concentrations would likely improve with additional development or pumping. Ground water from the vicinity of Borings 13, 56, and 70 could also likely be similarly discharged if a waiver could be obtained or an inexpensive water treatment system (e.g., water softener) could be used.



TABLE 4.2 SUMMARY OF GROUND WATER QUALITY ANALYSIS											
Constituent	Boring 3	Boring 10	Boring 11	Boring 13	Boring 36	Boring 47	Boring 49	Boring 53	Boring 56	Boring 70	Maximum Discharge Limitations*
Oil and Grease	--	--	12 mg/L	6.7 mg/L	<1 mg/L	<1 mg/L	<1 mg/L	--	<1 mg/L	3 mg/L	15 mg/L
Nitrate	--	--	6.4 mg/L	24.5 mg/L	3.9 mg/L	4.1 mg/L	<0.5 mg/L	7.6 mg/L	15 mg/L	<0.05 mg/L	8 mg/L
Sulfide	--	--	<0.1 mg/L	<0.1 mg/L	<0.20 mg/L	<0.20 mg/L	<0.20 mg/L	<0.2 mg/L	<0.20 mg/L	1.3 mg/L	1.0 mg/L
Biochemical Oxygen Demand	--	--	61 mg/L	42 mg/L	2.7 mg/L	1.2 mg/L	2.7 mg/L	2.4 mg/L	1.2 mg/L	7.8 mg/L	30 mg/L
Turbidity, NTU	--	--	860	3,200	370	1,600	98	1,300	43	6,100	75
Phenolics	--	--	<0.05 mg/L	<0.05 mg/L	<0.05 mg/L	<0.05 mg/L	<0.05 mg/L	<0.05 mg/L	<0.05 mg/L	<0.05 mg/L	1.0 mg/L
Total Dissolved Solids (TDS)	--	--	510 mg/L	1,200 mg/L	340 mg/L	940 mg/L	1,100 mg/L	1,000 mg/L	960 mg/L	1,500 mg/L	950 mg/L
Settleable Solids	--	--	25 ml/L	100 ml/L	<0.5 mg/L	130 mg/L	<0.5 mg/L	60 mg/L	<0.5 mg/L	170 mg/L	0.2 mg/L
Total Suspended Solids	--	--	4,200 mg/L	13,000 mg/L	--	--	--	2,200 mg/L	--	2,900 mg/L	75 mg/L
Chlorides	--	--	33 mg/L	99 mg/L	11 mg/L	150 mg/L	110 mg/L	93 mg/L	76 mg/L	180 mg/L	150 mg/L
Sulfate	--	--	210 mg/L	440 mg/L	58 mg/L	260 mg/L	220 mg/L	20 mg/L	190 mg/L	400 mg/L	300 mg/L
Arsenic	<0.002 mg/L	0.002 mg/L	0.013 mg/L	0.011 mg/L	<0.01 mg/L	0.05 mg/L	<0.01 mg/L	0.02 mg/L	<0.01 mg/L	<0.01 mg/L	0.05 mg/L
Cadmium	<0.001 mg/L	<0.001 mg/L	<0.001 mg/L	<0.001 mg/L	<0.01 mg/L	0.01 mg/L	<0.01 mg/L	<0.01 mg/L	<0.01 mg/L	<0.01 mg/L	0.01 mg/L
Chromium	<0.005 mg/L	<0.005 mg/L	0.0089 mg/L	<0.0005 mg/L	0.02 mg/L	0.13 mg/L	<0.01 mg/L	0.08 mg/L	<0.01 mg/L	0.57 mg/L	0.05 mg/L
Copper	<0.02 mg/L	<0.02 mg/L	<0.02 mg/L	0.046 mg/L	0.03 mg/L	0.48 mg/L	0.02 mg/L	0.15 mg/L	<0.02 mg/L	0.89 mg/L	1.0 mg/L
Lead	0.005 mg/L	<0.016 mg/L	<0.023 mg/L	0.31 mg/L	0.02 mg/L	0.09 mg/L	<0.01 mg/L	0.03 mg/L	<0.01 mg/L	0.08 mg/L	0.05 mg/L
Mercury	<0.0005 mg/L	<0.0005 mg/L	<0.0005 mg/L	<0.0005 mg/L	<0.0002 mg/L	0.0014 mg/L	<0.0002 mg/L	<0.0002 mg/L	<0.0002 mg/L	0.0005 mg/L	0.002 mg/L
Selenium	<0.005 mg/L	<0.005 mg/L	<0.0051 mg/L	0.0072 mg/L	<0.01 mg/L	<0.01 mg/L	<0.01 mg/L	<0.01 mg/L	<0.01 mg/L	<0.01 mg/L	0.01 mg/L
Silver	<0.01 mg/L	<0.01 mg/L	<0.01 mg/L	<0.01 mg/L	<0.02 mg/L	<0.02 mg/L	<0.02 mg/L	<0.02 mg/L	<0.02 mg/L	<0.02 mg/L	0.05 mg/L
Zinc	0.29 mg/L	0.05 mg/L	0.046 mg/L	0.23 mg/L	0.11 mg/L	1.1 mg/L	<0.05 mg/L	0.32 mg/L	<0.05 mg/L	2.2 mg/L	5.0 mg/L

Notes: * = Maximum discharge limitations for Los Angeles River from RWQCB General Waste Discharge Requirements Order No. 91-092.

-- = not analyzed.

mg/L = milligrams per liter.

ml/L = milliliters per liter.

TABLE 4.3 SUMMARY OF DETECTED VOLATILE ORGANIC COMPOUNDS (VOCs) IN GROUND WATER										
Constituent	Boring 3	Boring 10	Boring 11	Boring 13	Boring 47	Boring 49	Boring 53	Boring 56	Maximum Discharge Limitations*	MCL**
Benzene	1.0 µg/L	<0.5 µg/L	<0.5 µg/L	<0.5 µg/L	<1 µg/L	<1 µg/L	<1 µg/L	<1 µg/L	1.0 µg/L	1.0 µg/L
Toluene	<0.5 µg/L	0.7 µg/L	<0.5 µg/L	<0.5 µg/L	<1 µg/L	<1 µg/L	<1 µg/L	<1 µg/L	10.0 µg/L	100 µg/L
Xylenes	<1.0 µg/L	1.4 µg/L	<1 µg/L	<1 µg/L	<1 µg/L	<1 µg/L	<1 µg/L	<1 µg/L	10.0 µg/L	1,750 µg/L
1,1,1-Trichloroethane	--	--	0.78 µg/L	<0.5 µg/L	<1 µg/L	<1 µg/L	<1 µg/L	<1 µg/L	--	200 µg/L
Tetrachloroethene	--	--	<0.5 µg/L	<0.5 µg/L	<1 µg/L	8 µg/L	85 µg/L	<1 µg/L	5.0 µg/L	5 µg/L
Chloroform	--	--	1.3 µg/L	<0.5 µg/L	<2.7 µg/L	<1 µg/L	9 µg/L	38 µg/L	--	100 µg/L
Trichloroethene	--	--	0.97 µg/L	<0.5 µg/L	<1 µg/L	25 µg/L	3 µg/L	<1 µg/L	5.0 µg/L	5.0 µg/L
trans 1,2-Dichloroethene	--	--	<0.5 µg/L	<0.5 µg/L	<1 µg/L	6 µg/L	<1 µg/L	<1 µg/L	--	10 µg/L
cis 1,2 Dichloroethene	--	--	3.9 µg/L	<0.5 µg/L	<1 µg/L	190 µg/L	<1 µg/L	<1 µg/L	--	6 µg/L

Notes:

µg/L = Micrograms per liter.

< = Less than laboratory detection limit shown.

-- = not analyzed.

* = Maximum discharge limitations for Los Angeles River from RWQCB General Waste Discharge Requirements Order No. 91-092.

** = California Maximum Contaminant Level for drinking water.

The results further indicate the presence of low concentrations of aromatic hydrocarbons in the water samples from Borings 3 and 10. These compounds were not present in concentrations exceeding established maximum discharge limitations or maximum contaminant levels (MCLs) for these compounds. Low concentrations of several volatile organic compounds (VOCs) were detected in the ground water sample from Boring 13. The compounds detected (1,1,1-trichloroethane, chloroform, trichloroethene, and cis-1,2,-dichloroethene) were not present in concentrations in excess of established maximum discharge limitations or maximum contaminant levels (MCLs) for these compounds. The ground water sample from Boring 49 also contained several VOCs (tetrachloroethene, trichloroethene, and cis-1,2,-dichloroethene) in concentrations exceeding established maximum discharge limitations or maximum contaminant levels (MCLs) for these compounds. The ground water sample from Boring 53 contained tetrachloroethene at a concentration exceeding the established maximum discharge limitation and MCL for this compounds. The VOCs observed, with the exception of chloroform, are industrial solvents that are common ground water pollutants. Ground water from the vicinity of Borings 49 and 53 would likely require treatment to decrease VOC concentrations to acceptable levels prior to discharge under an NPDES permit.

4.4 GEOLOGIC STRUCTURE

The southern half of the alignment lies within the Elysian Park Fold and Thrust Belt defined by Hauksson (1990), which consists of a sequence of northwest-trending folds and faults. The geologic structure in the vicinity of the proposed alignment is shown on Plates 3.1 through 3.6. Along the southern half of the alignment, bedrock of the Puente and Topanga Formation is exposed within the slopes adjacent to the alignment. The bedrock units have been complexly folded and faulted, with most of the folds and faults trending generally west-northwest. Additionally, a system of short northeast-trending faults traverse the bedrock exposures. These faults are typically truncated by, and terminate at their intersection with, the northwest-trending faults. Most of the northwest- and northeast-trending faults are considered



to be inactive, with the exception of the San Rafael-Eagle Rock fault, which is considered potentially active.

4.5 GEOLOGIC HAZARDS

4.5.1 GENERAL

The geologic hazards along the alignment are limited to those caused by slope instability and earthquakes. Earthquake-induced damage is caused mainly by strong ground shaking.

4.5.2 SEISMIC HAZARDS

4.5.2.1 Faults

The numerous faults in Southern California include active, potentially active, and inactive faults. The criteria for these major groups, as established by Slemmons (1979) are presented in Table 4-4. Table 4-5 presents a listing of active faults in Southern California with the distance in miles between the alignment and the nearest point on the fault, and the maximum credible earthquake for the fault. Table 4-6 provides a similar listing for potentially active faults.

A portion of the alignment lies within an established Alquist Priolo Special Studies Zone for fault rupture hazard. This special studies zone has been established along a portion of the Raymond fault and is located along the alignment between Magnolia Street and about 250 feet north of Oak Lawn Avenue. In our opinion, there is a potential for surface fault rupture occurring beneath the proposed alignment due to the active Raymond fault and the potentially active Eagle Rock-San Rafael fault that cross the proposed alignment.



TABLE 4-4
CRITERIA FOR CLASSIFICATION OF FAULTS WITH REGARD TO SEISMIC ACTIVITY
(After D.B. Slemmons, 1979)

Activity Classification and Definition	Criteria		
	Historic	Geologic	Seismologic
<u>Active</u> - a tectonic fault with a history of strong earthquakes or surface faulting, or a fault with a short recurrence interval relative to the life of the planned project. The recurrence interval used to define activity rate may vary according to the consequence of activity.	(1) Surface faulting and associated strong earthquakes. (2) Tectonic fault creep or geodetic evidence of fault displacement or deformation.	(1) Geologically young deposits cut by the fault. (2) Youthful geomorphological features that are characteristic of geologically young displacements along the trace fault. (3) Ground water barriers in geologically young or unconsolidated deposits.	Earthquake epicenter can be assigned with confidence to the fault.
<u>Potentially Active</u> - a tectonic fault without historic surface offset, but with a recurrence interval that could be sufficiently short to be significant to the particular project.	No reliable report of historic surface faulting.	(1) Geomorphic features that are characteristic of active faults, but with subdued, eroded, and discontinuous form. (2) Faults not known to cut or displace youngest alluvial deposits, but offset older Quaternary deposits. (3) Water barriers in older deposits. (4) Geological setting in which the geometry in relation to active or potentially active faults suggests similar degree of activity.	Alignment of some earthquake epicenters along or near fault, but assigned locations have low degree of confidence in location.
<u>Activity Uncertain</u> - a fault with insufficient evidence to define past activity or recurrence interval. The following classifications can be used until the results of additional studies provide definitive evidence.	Available information is insufficient to provide criteria that are sufficiently definitive to establish fault activity. This lack of information may be due to the inactivity of the fault or to lack of investigations needed to provide definitive criteria.		
<u>Tentatively Active</u> - predominant evidence suggests that the fault may be active even though its recurrence interval is very long or poorly defined.	Available information suggests evidence of fault activity, but evidence is not definitive.		
<u>Tentatively Inactive</u> - predominant evidence suggests that the fault is not active.	Available information suggests evidence of fault inactivity, but evidence is not definitive.		
<u>Inactive</u> - a fault along which it can be demonstrated that surface faulting has not occurred in the recent past, and that the requirement interval is long enough not to be of significance to the particular project.	No historic activity.	Geomorphic features characteristic of active fault zones are not present and geological evidence is available to indicate that the fault has not moved in the recent past and recurrence is not likely during a time period considered significant to the site. Should indicate age of last movement: Holocene, Pleistocene, Quaternary, Tertiary, etc.	Not recognized as source of earthquakes.



TABLE 4-5

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

Fault (in alphabetical order)	-	Maximum Credible Earthquake		Distance From Site (Miles)	Direction From Site
Big Pine	7.5	(b)	SS	63	NW
Calico-Newberry	7.25	(b)	SS	88	NE
Cucamonga	6.5	(b)	RO	24	E
Elsinore	7.5	(b)	SS	35	SE
Elysian Park Structure	6.75	(c)	RO	0	--
Garlock	7.75	(b)	SS	60	NNW
Helendale	7.5	(b)	SS	64	NE
Malibu Coast	7.0	(a)	RO	18	WSW
More Ranch	7.5	(b)	SS	87	WNW
Newport-Inglewood	7.0	(b)	SS	8.6	WSW
Raymond	6.9	(a)	RO	0	--
San Andreas	8.25	(b)	SS	24	NE
San Cayetano	7.0	(a)	RO	40	NW
San Fernando	6.5	(b)	RO	13	NW
San Gabriel	7.5	(a)	SS	14	NNW
San Jacinto	7.5	(b)	SS	32	ENE
Sierra Madre	7.5	(a)	RO	2.2	NNE
White Wolf	7.75	(b)	RO	76	NNW
Whittier	7.0	(a)	SS	11.5	SW

(a) Slemmons, 1979

(b) Greensfelder, C.D.M.G. Map Sheet 23, 1974.

(c) Mark, 1977

SS Strike Slip

NO Normal Oblique

RO Reverse Oblique



TABLE 4-6

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

Fault (in alphabetical order)	Maximum Credible Earthquake			Distance From Site (Miles)	Direction From Site
Charnock	6.5	(a)	SS	12	WSW
Chino	7.1	(a)	NO	26	SE
Duarte	6.7	(a)	RO	5.7	E
Eagle Rock - San Rafael	6.6	(a)	RO	0	--
Northridge Hills	6.5	(b)	SS	17	NW
Norwalk	6.7	(a)	RO	14	SE
Oakridge	7.5	(b)	RO	37	NW
Overland	6.0	(a)	SS	10.5	WSW
Palos Verdes	7.0	(b)	SS	19.5	SW
San Jose	6.9	(a)	RO	17	ESE
Santa Cruz Island	7.1	(a)	RO	69	W
Santa Monica-Hollywood	6.9	(a)	RO	3.6	NNW
Santa Susana	6.5	(b)	RO	23	NW
Santa Ynez	7.5	(b)	SS	52	NW
Sierra Nevada (Southern Branch)	8.25	(b)	NO	78	N
Verdugo	7.4	(a)	RO	5.2	N

(a) Slemmons, 1979

(b) Greensfelder, C.D.M.G. Map Sheet 23, 1974

(c) Mark, 1977

SS Strike Slip

NO Normal Oblique

RO Reverse Oblique



Active Faults

Raymond Fault: The alignment crosses a portion of the active Raymond fault at approximately the Pasadena Freeway in South Pasadena. The Raymond fault is a high-angle reverse fault that thrusts basement rocks north of the fault over alluvial sediments south of the fault. It has long been recognized as a ground water barrier in the Pasadena/San Marino area, and numerous geomorphic features along its length (such as fault scarps, sag ponds, springs, and pressure ridges) attest to the fault's activity during the Holocene epoch (last 11,000 years). Eight earthquake events have been recognized to have occurred along the Raymond fault within the last 36,000 years, (Crook et al., 1987). The most recent fault movement, based on actual radiocarbon ages from materials collected in an excavation exposing the fault, occurred sometime between $2,160 \pm 105$ and $1,630 \pm 100$ years before present (Crandall and Associates, 1978; Crook et al., 1987).

Sierra Madre Fault: The active Sierra Madre fault zone is located about 2.2 miles to the north-northeast of the alignment at the segment's northern end. The Sierra Madre fault zone extends approximately 65 miles from San Fernando Pass on the west to Cajon Pass on the east. The fault zone consist of a series of low-angle thrust faults that are believed to have been active at different times in the geologic past (Crook et al., 1987). The 1991 Sierra Madre earthquake and the structural relationship of the active San Fernando fault to the Sierra Madre fault zone indicates that the Sierra Madre fault zone is active. The San Fernando fault zone is considered to be part of the Sierra Madre fault zone.

Newport-Inglewood Fault Zone: The active Newport-Inglewood fault zone is about 8.6 miles to the west-southwest of the alignment at its closest point. This fault zone is composed of a series of discontinuous northwest-trending en echelon faults (including the nearby Potrero and Inglewood faults), extending from the southern edge of the Santa Monica Mountains southeastward to the area offshore of Newport Beach. This zone, commonly referred to as the Newport-Inglewood uplift or zone of deformation, is reflected at the surface by a line of geomorphically young anticlinal hills and mesas formed by the folding and faulting of a thick



sequence of Pleistocene and Tertiary sedimentary rocks (Barrows, 1974). At depth, the fault zone is considered a complex fault system that serves as the boundary between Catalina Schist basement to the west and granitic basement to the east. According to Wissler (1943), the Newport-Inglewood fault zone has been a deformational zone since Miocene time. Stratigraphic evidence indicates recurrent movement during Late Tertiary and Quaternary time. The 1920 Inglewood and 1933 Long Beach earthquakes have been attributed to movement on the Newport-Inglewood fault zone.

San Andreas Fault Zone: The San Andreas fault zone, California's most prominent structural feature, trends in a general northwest direction for almost the entire length of the state. The southern segment, the closest to the alignment, is approximately 280 miles long, and extends from the Mexican border to the Transverse Ranges west of Tejon Pass. The San Andreas fault zone is approximately 24 miles northeast of the alignment at the nearest point. Wallace (1968) estimated the recurrence interval for a magnitude 8.0 earthquake along the total length of the fault to be between 50 and 200 years. More recent data by Sieh (1984) indicate an average earthquake recurrence interval of 140 to 200 years for the local segment of the San Andreas fault. The last major earthquake along the San Andreas fault zone in Southern California was the 1857 Fort Tejon earthquake.

Elysian Park Structure

The 1987 Whittier Narrows earthquake (magnitude 5.9) has been attributed to subsurface thrust faults, which are reflected at the earth's surface by a west-northwest trending anticline known as the Elysian Park Anticline (Lamar, 1970), or the Elysian Park Fold and Thrust Belt (Hauksson, 1990). The axial trace of this fold structure extends approximately 12 miles through the Elysian Park-Repetto Hills from about Silverlake on the west to the Whittier Narrows on the east. The southern portion of the alignment lies within the boundaries of Elysian Park Fold and Thrust Belt as defined by Hauksson (1990). The subsurface faults that create the structure are not exposed at the surface, and do not present a potential surface rupture hazard; however, as demonstrated by the 1987 earthquake and two smaller



earthquakes on June 12, 1989, the faults are a source for future seismic activity. As such, the structure should be considered an "active" feature capable of generating future earthquakes. Based on an approximate length of the axial trace of 12 miles, we have assigned a maximum credible earthquake of magnitude 6.75, using Mark's fault length versus magnitude relationship (1977).

Potentially Active Faults

Eagle Rock-San Rafael Fault Zone: The Eagle Rock-San Rafael fault zone is considered a possible southeasterly extension of the potentially active Verdugo fault zone. Based on geologic mapping by Lamar (1970), Pleistocene age alluvium is not offset by the fault. Recent mapping by Weber et al. (1980) indicated that basement rocks north of the fault have been thrust over conglomerate of the middle Miocene age Topanga Formation and possibly over older alluvium of Pleistocene age. On this basis, the Eagle Rock-San Rafael fault is currently viewed as potentially active (Ziony et al., 1985). The proposed alignment crosses the fault just north of Glenarm Street.

Santa Monica-Hollywood Fault Zone: The potentially active Santa Monica-Hollywood fault zone is located approximately 3.6 miles north-northwest of the alignment (Converse Davis and Associates, 1972). The Santa Monica and Hollywood faults are two distinctly separate structural features. The Hollywood fault lies at the base of the Santa Monica Mountains, approximately 4,000 feet north of the Santa Monica fault. It is separated from the Santa Monica fault to the south by the Hollywood syncline. The two faults are generally poorly defined in the near surface and have been located based on water well, oil well, geophysical data, and recent trenching by Crook et al. (1983). All evidence to date indicates that the Santa Monica fault has not moved within the Holocene epoch. Converse et al. (1981) suggest the Hollywood fault may be active, based on data gathered during investigation for the Metro Red Line Rail Project. These opinions, however, have not been positively verified. Neither the Santa Monica nor Hollywood faults have been zoned as active under the Alquist-Priolo Special Studies Zone program. Currently, the Santa Monica-Hollywood fault zone is



considered potentially active. Some geologists believe the fault zone is structurally aligned with, and may be contiguous with, the Raymond, Benedict Canyon, and Malibu Coast faults, which are of similar age, trend, and displacement (Weber et al., 1980).

Verdugo Fault: The Verdugo fault is located approximately 5.2 miles north of the alignment. This fault forms a partial barrier to ground water movement by having offset upper Pleistocene deposits. The fault, however, has no known effect on Holocene deposits. The main trace of the fault extends beneath the alluvium along a line projected from the southwesterly flank of the Verdugo Mountains to the southerly side of the Pacoima Hills.

Overland Fault: The Overland fault is located about 10.5 miles west-southwest of the alignment. The Overland fault trends northwest and lies between the Charnock and Newport-Inglewood fault zones. The fault extends from the northwest flank of the Baldwin Hills to Santa Monica Boulevard in the vicinity of Overland Avenue. Displacement on the fault is believed to be vertical, with an offset of about 30 feet. Water levels in the Pleistocene age sediments indicate the fault is an effective barrier to ground water movement and that Pleistocene materials have been offset.

Charnock Fault: The potentially active Charnock fault is located 12 miles west-southwest of the alignment at its closest point. The Charnock fault trends northwest-southeast, subparallel to the trend of the Newport-Inglewood fault zone and the Overland fault. Differential water levels occur in the San Pedro Formation across the fault and, therefore, it is concluded that the fault has experienced some movement during early Pleistocene time (approximately 500,000 to 2 million years ago).

4.5.2.2 Seismicity

Ground shaking is caused largely by waves generated by earthquakes. The intensity of the ground shaking generally depends on distance from the causative fault and the response of the



individual geologic unit underlying the site. Generally, the younger and less consolidated the deposit, the greater the intensity of ground shaking within the unit.

A database search of regional seismicity as compiled by the California Institute of Technology was performed. The epicenters of earthquakes with magnitude equal to or greater than 4.0 within a radius of 100 kilometers (62 miles) of the alignment are shown in a table at the end of Appendix C; other pertinent information regarding these earthquakes is also shown in this printout.

The database search indicates that 323 earthquakes of Richter magnitude 4.0 and greater have been recorded within 100 kilometers (62 miles) from the center of the alignment during the period from 1932 to 1991. The earthquake recurrence curve based on that information is presented on Plate 6, Recurrence Curve.

Four earthquakes moderate magnitude have occurred in the metropolitan Los Angeles area within the last 60 years. The earliest of these events was the magnitude 6.3 Long Beach earthquake that occurred on March 11, 1933 (GCT). The epicenter of this earthquake was located about 35 miles south-southeast of the alignment.

The epicenter of the February 9, 1971 San Fernando earthquake of magnitude 6.4 was about 24 miles northwest of the alignment. Surface rupture occurred on various strands of the San Fernando fault zone as a result of this earthquake. The large amounts of damage caused to existing buildings by this earthquake led to adoption of more stringent building codes.

The magnitude 5.9 Whittier Narrows earthquake occurred on October 1, 1987 on a previously unrecognized buried blind thrust fault. The earthquake epicenter was located approximately 6 miles southeast of the alignment. The majority of structural damage resulting from this earthquake occurred in buildings constructed before the building codes developed after the 1971 San Fernando earthquake.



The Sierra Madre earthquake occurred on June 28, 1991 along the Sierra Madre fault zone. The epicenter of the magnitude 5.4 earthquake was in the San Gabriel Mountains, approximately 14 miles north of the site.

Most recently, on June 28, 1992, two earthquakes occurred east of Los Angeles. At 4:58 a.m., a magnitude 7.5 earthquake occurred in the high desert region and is known as the Landers earthquake; the epicenter was located about 100 miles east of the proposed alignment. The second event occurred at 8:04 a.m. near Big Bear Lake and had a magnitude of 6.6; the epicenter was about 78 miles east of the proposed alignment.

The location of the alignment in relation to known active and potentially active faults indicates that the alignment is exposed to greater than normal seismic risk relative to other areas within the Los Angeles County.

4.5.2.3 Liquefaction

The evaluation of the liquefaction potential of the soils along the alignment involved the estimation of the potential loss of shear strength of saturated cohesionless soils during earthquakes. The significant factors that may affect liquefaction include the soil types, particle size and gradation, depth to ground water, 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.

Within the northeasterly half of the alignment, north of Pasadena Avenue, the proposed alignment does not lie within the boundaries of a "potential liquefiable" area as designated by the County of Los Angeles General Plan (1990). The proposed alignment in this area is generally underlain by dense and firm silty sand, silt, clay, and sand, with gravel and cobbles. Additionally, the ground water along this portion of the alignment is generally 100 feet deep



or greater; therefore, the possibility of liquefaction occurring beneath this portion of the alignment is judged to be low.

Most of the southerly half of the alignment is located within either a "very low liquefaction susceptibility area" or a "liquefiable area," as designated by the Los Angeles County Seismic Safety Element (1990). The liquefaction potential along this portion of the alignment was evaluated as six separate areas. Within each area, the geologic and ground water conditions are similar. The liquefaction study areas are designated Areas A through F and are shown on Plates 1.1 through 1.13.

Area A

Area A (Plates 1.1 through 1.4) extends from the southern end of the rail line at Union Station to the Cornfield Yard to the north. This area is underlain by Holocene age alluvium and Pleistocene age alluvium at depth. Ground water beneath this portion of the alignment varies from approximately 21½ to 40 feet beneath the ground surface. Based on sampler blow counts, standard penetration tests, and on laboratory tests, the granular soils encountered in our borings in this area are dense to very dense with relative densities greater than 80%. Accordingly, the potential for liquefaction in this area is judged to be very low.

Area B

Area B (Plate 1.4) extends from the Cornfield Yard to about the North Broadway Bridge. Holocene age alluvial deposits underlain by shallow bedrock of the Puente Formation were encountered in our exploratory borings within Area B. Bedrock was encountered within 12 feet of the ground surface beneath the alignment in this area, and water seepage was generally encountered within the bedrock materials. The bedrock is considered non-waterbearing although water may be present in fractures, along bedding, or within sandy beds as a result of surface infiltration migrating through the bedrock. In Borings 7 and 44, two feet of water was encountered in the lower



alluvial deposits. This water is a localized condition of perched water on top of the less permeable bedrock units as a result of surface infiltration. The bedrock units are very dense and not prone to liquefaction. Additionally, based on sampler blow counts and laboratory tests, the granular alluvial deposits encountered in Borings 7 and 44, below the perched water level, are dense to very dense. Therefore, the potential for liquefaction in this area is judged to be low.

Area C

Area C (Plates 1.5 through 1.8) extends northward from the vicinity of the North Broadway Bridge to the vicinity of the proposed Marmion/Figueroa Station. Within Area C, deep Holocene age alluvium and ground water levels at depths of 21½ to 60 feet beneath the ground surface were encountered in our exploratory borings.

Ground water was generally encountered within the alluvial deposits in this area except within Borings 11, 13 and 47 where seepage was encountered within the bedrock at depth. The bedrock is not prone to liquefaction. As in Area A, sampler blow counts, the results of standard penetration tests, and laboratory test results indicate the granular materials encountered beneath this portion of the alignment are very dense with relative densities greater than 80%. Accordingly, the potential for liquefaction within this area is judged to be very low.

Area D

Area D (Plates 1.8 through 1.10) extends northward along Marmion Way from the vicinity of the Marmion/Figueroa Station to near the Southwest Museum. Holocene age alluvium overlying shallow bedrock units of the Puente Formation (less than 15 feet beneath the ground surface) is typical of the geologic conditions within Area D. Ground water was not encountered in our borings drilled to a maximum depth of 40 feet beneath the existing ground surface. However, localized seepage was encountered within the bedrock at a depth of 25 feet in Boring 59. It is possible for surface water



to infiltrate and become perched on top of the shallow bedrock. However, based on sampler blow counts and laboratory tests, the relative densities of the Holocene and Pleistocene alluvial deposits in this area indicate they are dense to very dense and not prone to liquefaction. Additionally, the bedrock is very dense and not prone to liquefaction. Therefore, the potential for liquefaction beneath the alignment within Area D is judged to be very low.

Area E

Area E is shown on Plate 1.10 and extends from near the Southwest Museum to about Avenue 49. Area E is underlain by Holocene and Pleistocene age alluvial deposits. Within this area, the proposed alignment traverses a deep fill embankment overlying Holocene age alluvium, south of Avenue 49. Based on sampler blow counts and on laboratory tests, the Pleistocene age alluvial deposits are very dense and not prone to liquefaction. Based on the relative densities of similar Holocene alluvium encountered in the vicinity, the potential for liquefaction within these deposits is low.

Area F

Area F is subdivided into two sections, F1 and F2. Area F1 (Plates 1.10 through 1.13) extends from about Avenue 49 to the west side of Arroyo Seco Park. Area F1 is underlain by Pleistocene age alluvial deposits. Ground water in this area is generally at depths of 30 feet or greater. However, water can become perched at shallower depths on less permeable layers within the older alluvium. Ground water was not encountered along this portion of the alignment in our exploratory borings drilled to a maximum depth of 30 feet. However, minor seepage was encountered in Boring 18 at a depth of 19½ feet. Based on sampler blow counts, on standard penetration tests, and on laboratory tests, the Pleistocene age alluvial deposits in this area are dense to very dense and not prone to liquefaction.



Area F2, as depicted on Plate 1.13, includes Arroyo Seco Park, the Pasadena Freeway, and the Arroyo Seco Channel and is underlain by Holocene age alluvium. Borings 69 and 70, located on the west and east banks of the Arroyo Seco Channel, encountered bedrock at depths of 14 and 15½ feet, respectively. Piezometers were installed in both Borings 69 and 70 and water levels were measured at 7½ and 13½ feet, respectively. These water levels represent a localized perch condition and are probably not representative of the regional ground water levels. Based on sampler blow counts, standard penetration tests, and laboratory tests, the Holocene age alluvial deposits overlying the bedrock are dense to very dense and not prone to liquefaction.

4.5.2.4 Seismically-Induced Settlement

Seismic settlement often occurs when loose to medium-dense granular soils densify during ground shaking. If such settlement were uniform beneath a given structure, damage would be minimal. Because of variations in distribution, density, and confining conditions of the soils, however, such settlement is generally non-uniform and can cause serious structural damage. Seismically induced settlement can occur in both dry and partially saturated granular soils, as well as in saturated granular soils. Differential settlement may also be induced by ground failures such as liquefaction, flow slides, and surface ruptures. Generally, differential settlements due to such conditions would be much more severe than those due to densification alone. The granular alluvial soils encountered in our exploratory borings have relative densities greater than 80% and are not in the loose to medium-dense category. Additionally, the bedrock units are not susceptible to settlement. Therefore, the probability of seismically-induced settlement is considered to be low.

4.5.3 NON-SEISMIC HAZARDS

4.5.3.1 Slope Stability

Within the northeasterly half of the alignment, north of Glenarm Street, the ground surface is generally gently sloping. Therefore, the potential for slope stability problems or lurching (movement at right angles to a steep slope during strong ground shaking) is generally low.



The ground surface, from the southerly terminus of the alignment to about Spring Street, slopes gently to the east. Therefore, the potential for slope stability problems or lurching (movement at right angles to steep slope during strong ground shaking) along this portion of the alignment is considered very low. This portion of the alignment is not located within a Slope Stability Study Area as designated by the City of Los Angeles Seismic Safety Plan (1975).

Much of the remainder of the alignment is located within a Slope Stability Study Area as designated by the City of Los Angeles Seismic Safety Plan (1975). Based on our preliminary geotechnical investigation, seven areas with a significant potential for slope stability problems were identified as shown on Plate 7, Potential Slope Stability Hazards Map. The potential for slope instability within each area was evaluated by geologic mapping, downhole logging of selected exploratory borings, aerial photo analysis, and preparation of geologic sections that are presented on Plates 2.1 through 2.10. Slope stability analyses of representative geologic conditions were also performed. A more detailed discussion of slope stability analyses is presented in Section 7.0.

Area 1

Area 1 is within a City of Los Angeles Slope Stability Study Area. An east- to northeast-facing slope occurs adjacent to and along the westerly edge of the alignment (on the west side of the Cornfield Yard). The slope, which is located north of the intersection of the alignment with Spring Street to the Los Angeles River, varies from about 25 to 45 feet high, with a gradient varying from about $1\frac{1}{2}$:1 to $1\frac{1}{4}$:1 (horizontal to vertical) and localized areas as steep as 1:1. As shown on Section A-A' (Plate 2.1), the slope is predominantly underlain by older alluvial materials. However, artificial fill locally underlies portions of the slope, and sedimentary bedrock units of the Puente Formation underlie the slope near the North Broadway Bridge as shown on Sections B-B' and C-C' (Plates 2.2 and 2.3).



The older alluvial deposits exposed in the slope face, as indicated on Section A-A', are horizontally stratified. These materials lack any well-defined planar features or discontinuities (such as bedding or joints) that could act as planes of weakness. This condition is considered favorable for gross stability from a geologic standpoint. However, the slope is prone to surficial instability as evidenced by surficial sloughing and erosion. Such surficial instability is not considered a serious problem and can be mitigated by proper design and construction of a slough wall at the toe of the slope during development. Specific design recommendations for slough walls are presented in Section 12.0 of this report.

The bedding within bedrock materials exposed in the slope and observed in the exploratory borings dip 45 to 64 degrees to the south. Because the bedding dips steeply, the bedding is not unsupported and dips as steep or steeper than the slope face as indicated on Section B-B' and C-C'. This condition is favorable with respect to gross stability.

Area 2

This area is located where the alignment parallels the east side of Marmion Way, between Mt. Washington Drive and Avenue 49. Within Area 2, the existing Atchison, Topeka and Santa Fe (ATSF) tracks are below street grade, with 2:1 (horizontal to vertical) to nearly vertical slopes descending 5 to 15 feet from Marmion Way easterly to the alignment. Retaining walls are planned along the westerly side of the alignment in this area. Additionally, the proposed alignment may be depressed up to 30 feet below the existing ground surface at the Figueroa Street and Marmion Way intersection, and retaining walls are planned on both sides of the alignment in this area.

Artificial fill and older alluvial deposits are exposed in the existing slopes from the intersection of Marmion Way and Figueroa Street to about Museum Drive as indicated in Sections D-D' and F-F' (Plates 2.4 and 2.6). Locally, sandstone and siltstone of the



Puente Formation are also exposed in the slopes adjacent to the alignment from Museum Drive to about Avenue 49, as depicted on Section E-E' (Plate 2.5).

The older alluvial deposits exposed in the existing and proposed slope face are grossly stable from a geologic standpoint because they are horizontally stratified and lack any well-defined planar features or discontinuities that could act as planes of weakness. However, these slopes are prone to surficial sloughing and erosion. This type of surficial instability should not affect the proposed alignment because of the series of retaining walls planned in this area. The retaining walls supporting alluvial deposits will not have to be designed for additional lateral loads due to existing soil conditions.

Sandstone and siltstone of the Puente Formation are locally exposed in the east-facing slope face along the alignment from Museum Drive to Avenue 49. This east-facing slope is expected to expose unsupported bedding planes, with apparent dips of 13 to 32 degrees. The future retaining walls along this portion of the alignment that locally support bedrock will have to be designed for increased lateral forces due to the adverse bedding condition. Specific design recommendations for retaining walls are presented in Section 12.0.

It is possible that bedrock will be exposed in the lower portions of the planned retaining wall excavations in the area of the Marmion/Figueroa Station that will exhibit unsupported components of bedding with apparent dips of 35 degrees out of the east-facing walls. Additionally, adverse bedding components ranging from 8 to 38 degrees could be encountered in the lower wall excavations between Avenue 45 and Museum Drive. The future retaining walls in this area that locally support bedrock will have to be designed for increased lateral forces due to the adverse bedding conditions.

Steep slopes ascend westerly at gradients of $1\frac{3}{4}$:1 (horizontal to vertical) to 1:1 above Marmion Way north of Museum Drive. These slopes exhibit locally unsupported



bedding and surficial instability. However, a series of existing 3- to 6-foot-high slough and retaining walls are situated at the toe of this slope, on the west side of Marmion Way. Additionally, Marmion Way is situated between the alignment and these slopes. Because slough walls are located at the toe of these slopes and because of the significant distance between the alignment and the slope, the surficial instability of this west-facing slope should not affect the proposed alignment and is beyond the scope of this investigation.

Area 3

This area is located between the Arroyo Seco Channel and about 50 feet west of the intersection of Marmion Way and Arroyo Drive. In this area, the alignment parallels a northerly facing steep slope. As shown on Section G-G', Plate 2.7, the slope descends from the alignment about 50 feet toward the Arroyo Seco Channel and ascends up to 180 feet at an overall gradient of about $\frac{3}{4}$:1 (horizontal to vertical). Bedding along the westerly portion of this area dips from 18 to 33 degrees to the southeast. This orientation is favorable for gross stability from a geologic standpoint. However, fractures and joints are present in the bedrock exposed on the slope face, and serious surficial failures and rock falls have occurred in the past.

The lower portion of the slope beneath the proposed alignment, descending approximately 50 feet to the north, is underlain by a relatively small landslide. The landslide configuration, shown on Section G-G', is based on information from downhole logging and topographic expression. Stability calculations indicate this landslide mass is currently stable with a safety factor greater than 1.5.

Area 4

Area 4 is located between Arroyo Verde Road and is approximately 50 feet west of the intersection of Marmion Way and Arroyo Verde Road. Along this area, the alignment traverses the toe of a recently active landslide (1969) which is encompassed by an



ancient landslide. The ancient landslide is approximately 1,250 feet wide and approximately 900 feet long, and ascends approximately 160 feet above the alignment. According to reports from the City of Los Angeles, Department of Public Works, a large portion of the ancient landslide was activated in 1969 as a result of heavy seasonal rains. The landslide is called the "Monterey Road Landslide". Based on geotechnical reports by the City of Los Angeles Street Opening and Widening Division (1969), approximately 12 acres, including about 700 feet along the ATSF Railroad right-of-way, were within the 1969 landslide. The geologic materials involved in the landslide include interbedded siltstone, sandstone and shale of the Miocene age Topanga Formation, colluvium and artificial fill.

According to the City of Los Angeles report (1969), a series of ten dewatering wells, designed to keep the water levels at or near the base of the landslide, were installed within the landslide. The stability of the ancient landslide, and the 1969 landslide, are beyond the scope of this investigation. However, it is our understanding that the landslide has not moved since 1969. Reportedly, the City of Los Angeles is monitoring the landslide on an on-going basis and is responsible for operating and maintaining the dewatering wells.

Existing slopes locally adjacent to the alignment within the landslide debris are typically 1½:1 to 3/4:1 (horizontal to vertical) and range from 50 to 70 feet high. Stability calculations indicate these slopes are grossly stable. However, raveling and surficial slumps are additional slope stability problems in this area. This problem can be mitigated by constructing a slough wall at the toe of the existing slope along this portion of the alignment. Specific design recommendations for slough walls are presented in Section 12.0 in this report.



Area 5

This area is located adjacent to an ascending 75-foot-high slope near where the alignment crosses Fair Oaks Avenue in the vicinity of Raymond Hill. Based on published data, the bedding in this area dips 53 degrees to the west. However, geologic mapping in the area indicates bedding exposed in the slope adjacent to the alignment strikes N49W, and dips 48 degrees to the southwest. This bedding orientation is neutral to the slope face and is considered favorable with respect to gross stability from a geologic standpoint. However, surficial slumps and local raveling are potential slope stability problems in this area. Such surficial instability is not considered a serious problem and can be mitigated by the construction of a slough wall at the toe of the slope. Detailed design recommendations for slough walls are presented in Section 12.0.

As previously stated, there are several areas where retaining walls are planned. A 2,300-foot-long, 10- to 15-foot-high retaining wall is planned along the hillside in the Arroyo Seco Park area. The proposed wall extends from the vicinity of Boring 18 eastward to the westerly abutment of the Arroyo Seco Park Bridge, and from the vicinity of Boring 71, eastward to about Arroyo Verde Road. West of the Arroyo Seco Park Bridge, the proposed retaining wall is expected to support horizontally stratified older alluvial deposits. These materials lack any well-defined planar features or discontinuities (such as bedding or joints) that could act as planes of weakness and are considered grossly stable from a geologic standpoint. Therefore, the future retaining wall, west of the Arroyo Seco Park Bridge, will not have to be designed for additional lateral loads due to existing soil conditions. However, east of the Arroyo Seco Park Bridge, the proposed retaining wall will be underlain by and will support landslide debris. The wall foundation will have to be deepened or a system of piles will be required to support the proposed wall in this area. Specific design recommendations for retaining walls are presented in Section 12.0 of this report.



Another retaining wall is planned between Cawston Street and Orange Grove Avenue, and a 40-foot-high retaining wall is planned adjacent to Marengo Avenue. These future retaining walls will support older alluvial deposits or terrace deposits. These materials are horizontally stratified and lack any well defined planar features or discontinuities (such as bedding or joints) that could act as planes of weakness. Therefore, these walls will not have to be designed for additional lateral loads due to the existing soil conditions.

In addition to these retaining walls, a 2,000-foot-long depressed area, that will extend about 40 feet below the existing grade between Del Mar Boulevard and Holly Street in Pasadena, is planned. This excavation will expose horizontally stratified older alluvial deposits for the entire 40-foot-depth. These materials lack any well-defined planar features or discontinuities (such as bedding or joints) which could act as planes of weakness. Additionally, ground water is very deep and is not anticipated to affect the proposed excavation. These conditions are considered favorable for gross stability from a geologic standpoint.

4.5.3.2 Flooding

Within the limits of the City of Pasadena, the proposed alignment is in a "Zone D" flood hazard area. The Federal Insurance Administration defines "Zone D" as an unstudied area where flood hazards are undetermined but possible. However, according to the Federal Emergency Management Agency and the County of Los Angeles Seismic Safety Element, the Pasadena area is not located in a potential flood hazard zone. The remainder of the alignment is within a "Zone C" flood hazard area, as designated by the Federal Insurance Administration. "Zone C" refers to areas of minimal flood hazard.

4.5.3.3 Dam Inundation and Seiches

According to the revised draft of the County of Los Angeles Seismic Safety Element (1990), the alignment is located within an inundation area as a result of earthquake-induced failure



and seiches (oscillating waves that form in an enclosed body of water) from Sepulveda Dam, Hansen Dam, Elysian Reservoir, Devils Gate Reservoir, and the Highland Reservoir.

The flooding and inundation areas as designated by the County of Los Angeles Seismic Safety Element are shown in relation to the alignment on Plate 8, Flood and Inundation Hazards Map.

These dams, as well as other dams in California, are continually monitored by various governmental agencies (such as the Army Corps of Engineers and the California Division of Safety of Dams) to guard against the threat of dam failure. The possibility of dam failures during an earthquake has been addressed by the California Division of Mines and Geology in the earthquake planning scenarios for a magnitude 8.3 earthquake on the San Andreas fault (Davis, et al., 1982), and a magnitude 7.0 earthquake on the Newport-Inglewood fault (Topozada, et al., 1988). As stated in both reports, catastrophic failure of a major dam as a result of the scenario earthquake is regarded as unlikely. Current design and construction practices, and ongoing programs of review, modification, or total reconstruction of existing dams are intended to ensure that every dam is capable of withstanding the Maximum Credible Earthquake (MCE) that may be generated at the site of the dam.

The Los Angeles City Oil Field is located within approximately 100 feet of the southern portion of the alignment. Additionally, the Union Station Oil Field and the Los Angeles Downtown Oil Field are located ½ mile north and 2 miles northeast of the alignment, respectively. Subsidence associated with petroleum production has been identified in some of the oil fields in the Los Angeles Basin; however, subsidence has not been identified in the Los Angeles City, the Union Station, or the Los Angeles Downtown Oil Fields. Consequently, the potential for future subsidence within the three nearby oil fields is considered low.



4.5.3.5 Oil Wells

Several known wells are located along the southern portion of the proposed alignment, south of the Los Angeles River. According to the State of California, Division of Oil and Gas (CDOG), these wells consist of Texaco, Inc. "Southern Pacific Corehole 1," Ventura Oil Company "Freight Depot Number 1," Ventura Oil Company "Number 1," and Paul F. McKenzie "T-2". Texaco, Inc. "Southern Pacific Corehole 1" is located approximately 50 feet north of the alignment near North Broadway and Solano Avenue. Ventura Oil Company "Freight Depot Number 1" is also located approximately 100 feet south of the alignment near Solano Avenue and North Broadway. Ventura Oil Company "No. 1" is located approximately 100 feet west of the alignment near College Street and North Broadway. Paul F. McKenzie "T-2" is located approximately 150 feet west of the alignment near College Street and North Broadway.

North of the Los Angeles River, the nearest well is the Texaco, Inc. "Core Hole No. 5," an abandoned dry hole located about 250 feet northwest of the alignment at its closed point. Records from CDOG indicate that this well was drilled in April 1945 to a total depth of 2,732 feet. The well was drilled to obtain geological information only, and the requirements of the CDOG pertaining to well abandonment at the time were reportedly fulfilled.

Other unknown wells could be encountered during construction. These wells will need to be properly abandoned in accordance with the current requirements of the CDOG.

4.5.3.6 Methane Gas

Oil wells are located within and in close proximity to the southerly portion of the proposed alignment (south of the Los Angeles River). Therefore, there is a potential for methane and other volatile gases to occur beneath the proposed alignment south of the Los Angeles River.

The presence of volatile organic compounds (VOCs) in the soil samples of the exploratory borings was detected and measured using an organic vapor analyzer (OVA). Methane gas, a volatile organic compound, might have been one of the compounds detected. However, gas sampling was not performed to determine the presence or absence of methane gas.



4.6 BIBLIOGRAPHY

Association of Engineering Geologists, 1973, "Geology and Earthquake Hazards Planners Guide to the Seismic Safety Element," Special Publication.

Association of Engineering Geologists, 1973 "Geology, Seismicity, and Environmental Impact," Special Publication.

Barrows, A. G., 1974, "A Review of the Geology and Earthquake History of the Newport-Inglewood Structural Zone, Southern California," California Division of Mines and Geology Special Report 114.

Bolt, B. A., 1973, "Duration of Strong Ground Motion," in Proceedings, Fifth World Conference on Earthquake Engineering.

California Division of Oil and Gas, 1974, "California Oil and Gas Fields, Vol. II - South Central Coastal and Offshore California" Report No. TR12.

California Division of Oil and Gas, 1986, "Boyle Heights, Los Angeles Downtown and Union Station Oil Fields," Map No. 119.

California Division of Oil and Gas, 1987, "Regional Wildcat Map," No. W1-5.

California Division of Oil and Gas, 1988, "Los Angeles City Oil Field," Map No. 116.

California Department of Water Resources, Bulletin 104, 1961, "Planned Utilization of the Ground Water Basins of the Coastal Plain of Los Angeles County," Appendix, Ground Water Geology.

California Department of Water Resources, Bulletin 116-2, 1964, "Crustal Strain and Fault Movement Investigation."

California Department of Water Resources, 1988, Water Level Data for Southern California on Microfiche.

Converse Consultants, Earth Science Associates, Geo-Resource Consultants, 1981, "Geotechnical Investigation Report, Volume I; Volume II -- Appendices 1 and 2," for Southern California Rapid Transit Metro Rail Project.

Converse Davis and Associates, 1972, "Ground Water Investigation Vicinity of Forest Lawn, Glendale-Los Angeles, California," CDA Project 70-044-EH, Report April 4, 1972, Drawings 1 and 2.



- Crandall, LeRoy and Associates, 1978, "Report of Geologic Studies Related to Raymond Fault Identification, San Marino High School, San Marino, California, for the San Marino Unified School District," E-77186.
- Crandall, LeRoy and Associates, 1983, "Report of Geotechnical Investigation, Proposed Central Maintenance Facility, Macy and Vignes Streets, Los Angeles, California, for the Southern California Rapid Transit District," ADE-83009.
- Crandall, LeRoy and Associates, 1990, "Report of Geotechnical Investigation, Proposed Lippobank Tower and Oriental Marketplace, Hill Street Between Ord and Alpine Streets, Los Angeles, California, for Hip Hing Holdings, Limited," L90255.ADEB.
- Crook, R., Proctor, R. J., and Lindvall, E. I., 1983, "Seismicity of the Santa Monica and Hollywood Faults Determined by Trenching," Technical Report to the U.S. Geological Survey, Contract No. 14-08-001-20253, p. 26.
- Crook, R., Jr., Allen, C. R., Kamb, R. Bayne, C. M., and Proctor, R. J., 1987, "Quaternary Geology and Seismic Hazard of the Sierra Madre and Associated Faults, Western San Gabriel Mountains," in U.S. Geological Survey Professional Paper 1339, Ch. 2, pp. 27-63.
- Davis, J. F., Bennett, J. H., Borchardt, G. A., Kahle, J. E., Rice, S. J., and Silva, M. A., 1982, "Earthquake Planning Scenario for a Magnitude 8.3 Earthquake on the San Andreas Fault in Southern California," California Division of Mines and Geology Special Publication 60.
- Fall, E. W., Rzonca, G. F., and Spellman, H. A., Shleman, R. J., 1987, "Late Quaternary Faulting, Malibu Coast Fault Zone, Malibu, California (abstract)," Association of Engineering Geologists Annual Meeting, October 8-13, Atlanta, Georgia, Abstracts and Program, p. 38.
- Greensfelder, R. W., 1974, "Maximum Credible Rock Acceleration from Earthquakes in California," California Division of Mines and Geology Map Sheet 23.
- Hauksson, E., 1990, "Earthquakes, Faulting, and Stress in the Los Angeles Basin," Journal of Geophysical Research, Vol. 95, No. 1310, Pages 15,365-15,934.
- Jennings, Charles W., et. al., 1975, "Fault Map of California with Locations of Volcanoes, Thermal Springs and Thermal Wells," California Division of Mines and Geology.
- Lamar, D. L., 1970, "Geology of the Elysian Park-Repetto Hills Area, Los Angeles County, California," California Division of Mines and Geology Special Report 101.



- Law/Crandall, Inc., 1991, "Draft Report of Geotechnical Investigation, Proposed Metro Plaza, Macy Street and Vignes Street, Los Angeles, California, for the Catellus Development Corporation," (LC L91291.ADEO).
- Levine · Fricke, 1989a, "Preliminary Environmental and Geotechnical Assessment of Los Angeles Union Passenger Terminal, Los Angeles, California, for the Catellus Development Corporation," Job No. L-F 1646.
- Levine · Fricke, 1989b, "Phase I Soil and Ground Water Investigation, Los Angeles Union Passenger Terminal, Los Angeles Union Passenger Terminal, Los Angeles, California, for the Catellus Development Corporation," Job No. L-F 1977.
- Levine · Fricke, 1990, "Environmental Assessment, Los Angeles Union Passenger Terminal, Los Angeles, California, for the Catellus Development Corporation," Job No. L-F 1977.
- Levine · Fricke, 1991a, "Phase II Soil and Ground Water Investigation, Los Angeles Union Passenger Terminal, Los Angeles, California, for the Catellus Development Corporation," Job No. L-F 1977.
- Levine · Fricke, 1991, "Addition to the Phase II Soil and Ground Water Investigation Report, Los Angeles Union Passenger Terminal, Los Angeles, California, for the Catellus Development Corporation," Job No. L-F 1977.
- Los Angeles, City of, Department of Public Works, 1969, "Monterey Road and Lomitas Drive Landslide Investigation," Bureau of Engineering Street Opening and Widening Division, Geology and Soils Section Report, Work Order No. 01883.
- Los Angeles, City of, 1975, "Seismic Safety Plan."
- Los Angeles, County of, 1975, Draft revision 1990, "Seismic Safety Element."
- Mark, R. K., 1977, "Application of Linear Statistical Models of Earthquake Magnitude Versus Fault Length in Estimating Maximum Expectable Earthquakes," *Geology*, Vol. 5, pp. 464-466.
- Poland, J. F., Garrett and Sinnott, 1959, "Geology, Hydrology, and Chemical Character of Ground Waters in the Torrance-Santa Monica Area, California," U.S. Geological Survey Water Supply Paper ~ 461.
- Sieh, K. E., 1984, "Lateral Offsets and Revised Dates of Large Prehistoric Earthquakes at Pallett Creek, California," *Journal of Geophysical Research*, Vol. 9, pp. 7641-7670.
- Slemmons, D. B., 1979, "Evaluation of Geomorphic Features of Active Faults for Engineering Design and Siting Studies," Association of Engineering Geologists Short Course.



- Toppozada, T. R., Bennett, J. H., Borchardt, G., Saul R., and Davis, J. F., 1988, "Planning Scenario for a Major Earthquake on the Newport-Inglewood Fault Zone," California Division of Mines and Geology, Special Publication 99.
- Wallace, R. E., 1968, "Notes on Stream Channels Offset by San Andreas Fault, Southern Coast Ranges, California," in Dickerson, U. R., and Grantz, A., editors, Proceedings of Conference of Geologic Problems on San Andreas Fault System, Stanford University Publications, Geological Sciences, Vol. IX, p. 6-21.
- Weber, F. H., Bennett, J. H., Chapman, R. H., Chase, G. W., and Saul, R. B., 1980, "Earthquake Hazards Associated with the Verdugo-Eagle Rock and Benedict Canyon Fault Zones, Los Angeles County, California," California Division of Mines and Geology Open File Report 80-1OLA.
- Wissler, S. G., 1943, "Stratigraphic Formations of the Producing Zone of the Los Angeles Basin Oil Fields," California Division of Mines Bulletin 118, Part 2, pp. 210-234.
- Yerkes, R. F., McCulloch, T. H., Schoellhamer, J. E., and Vedder, J. G., 1965, "Geology of the Los Angeles Basin, California," U.S. Geological Survey Professional Paper 420-A.
- Yerkes, R. F., Tinsley, J. C., and Williams, K. M., 1977, "Los Angeles Area Geologic Aspects of Tunneling," U.S. Geological Survey, Map MF-866.
- Zielbauer, E. J., Kues, H. A., Burnham, W. L., and Keene, A. G., 1962, "Coastal Basin Barrier and Replenishment Investigation, Dominguez Gap Barrier and Replenishment Investigation," unpublished report by the Los Angeles County Flood Control District.

SECTION 5.0

FIELD EXPLORATIONS AND LABORATORY TESTS

5.1 FIELD EXPLORATION PROGRAM

Field exploration consisted of subsurface drilling, sampling and testing, piezometer installation, chemical testing, and organic vapor analyzer (OVA) monitoring. Soil samples obtained along the alignment were carefully packaged and transported to our laboratory.

A detailed description of the exploration program is presented in Appendix A (Volume 2). Table 5.1, Summary of Borings, follows the text of Section 5.

5.1.1 BORINGS

The alignment was explored by drilling 122 borings at the locations shown on Plates 1.1 through 1.32. Borings 1 through 29 were drilled for the preliminary studies; the results of the prior studies have been incorporated herein.

The borings were drilled to depths ranging from about 7 to 101 feet below the existing grade, for a total of about 4,600 lineal feet of drilling. The drilling for the comprehensive investigation (Borings 30 through 122) was performed between November 20, 1992 and February 15, 1993. A summary of the boring locations and depths is presented in Table 5.1.

The soils encountered were logged by our field representatives. Both undisturbed and bulk samples were obtained for laboratory inspection and testing. The drilling operations, including soil and bedrock sampling and classification, and monitoring of water levels and volatile organic compounds (VOCs) in the borings, were supervised by a certified engineering geologist. Selected borings were entered by our geologist for downhole logging.

Undisturbed samples were obtained in the bucket borings with the Crandall drive sampler at depth intervals of about 3 to 5 feet and at major changes in soil stratigraphy. Standard penetration tests (SPTs) were performed in the rotary-wash borings. Alternate drive and SPT



sampling was performed at 3-foot intervals in the majority of the borings. The soil samples from our borings were monitored in the field for the possible presence of VOCs. The OVA field measurements are shown on the boring logs. Detailed descriptions of the field exploration procedures are presented in Appendix A (Volume 2).

5.1.2 DRILLING CONTRACTORS AND EQUIPMENT

The drilling was performed with rotary wash-type and bucket auger-type drilling equipment. The borings were drilled by A & W Drilling Company using a Mahew 1000 rotary wash drilling rig and/or a bucket-type rig, and Al-Roy Drilling Company using Mini-Bucket-type rig.

5.1.3 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 project engineers and further verified (as necessary) by laboratory tests.

5.1.4 OVA SAMPLE MONITORING

Soil samples from our borings were monitored for the possible presence of VOCs. The samples were monitored in the field and in our laboratory using Gastechtor Model 1238.

In general, the majority of the samples monitored did not register any elevated OVA readings. However, high OVA levels (greater than 200 ppm) were detected in Borings 6, 34, 35, 53, 56, 60, 61, 73, 74, and 81. Moderate OVA readings (100 to 200 ppm) were detected in a few samples from Borings 7, 30, 37, 39, 42, 44, 53, 56, 59, 71, 73, 74, 87, 105, and 106. Oil stains and odor were noted in Boring 35, and high OVA levels (varying from 1,000 to 3,000 ppm) were detected in this boring between depths of 49 and 74 feet. A reading of 12,500 ppm was taken in Boring 34 in the siltstone at a depth of 70 feet. However, no oil stains or odor were noted in the boring, and the high OVA level may be due to the presence of hydrogen sulfide that occurs naturally in the siltstone. The OVA measurements are shown on the boring logs in Appendix A (Volume 2).



5.2 PIEZOMETER INSTALLATION

After the completion of drilling Borings 1, 3, 10, 11, 13, 33, 34, 35, 36, 47, 49, 53, 56, 69, and 70, a 2-inch-diameter PVC pipe (with lower portion perforated) was installed in each boring for future monitoring of the ground water level and sampling. Detailed descriptions of the piezometer installations and water level measurements are presented in Appendix A (Volume 2), and in Table 4.1 in Section 4.0.

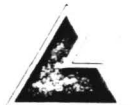
5.3 LABORATORY TESTING PROGRAM

Each soil sample was first visually observed in the laboratory to verify the sample description and classification assigned by the field personnel and by the certified engineering geologist. A laboratory testing program was then developed that would provide the soil parameters required in performing various engineering analyses. The various tests performed include the following:

- Moisture Content/Dry Density
- Direct Shear
- Triaxial Shear
- Consolidation
- Compaction
- California Bearing Ratio
- Expansion Index
- Particle Size Distribution
- Atterberg Limit

The test procedures and results are presented in Appendix B (Volume 2). The field moisture content and dry density of the undisturbed soils samples are shown to the left of the graphic boring logs presented in Appendix A (Volume 2).

Soil samples recovered from the borings and remaining after laboratory testing are stored at the laboratory of the Los Angeles office of Law/Crandall, Inc. located at 200 Citadel Drive, Los Angeles, California 90040.



5.4 SOIL AND WATER CORROSIVITY TESTING

Laboratory tests to evaluate the soil and water corrosivity characteristics were performed at selected locations along the Metro Pasadena Line project alignment. The soil and water corrosion study was performed by M. J. Schiff & Associates under a subcontract.

The soils along the Metro Pasadena Line range from mildly corrosive to severely corrosive. Specific details and recommendations regarding soil and water corrosivity are presented in Appendix E (Volume 2).



TABLE 5.1

SUMMARY OF BORINGS

Boring No.	Plate No.	Boring Depth	Drilling Equipment Type	Location
1	A-1	100	Rotary Wash	Railroad right-of-way at Union Station, north of Macy Street and south of Vignes Street
2	A-2	74½	Rotary Wash	Vignes Street, east of North Main Street
3	A-3	100	Rotary Wash	Intersection of Alameda Street and North Spring Street, south of College
4	A-4	75	Bucket/Rotary Wash	Southwestern edge of Southern Pacific Cornfield Property, east of North Broadway
5	A-5	30	Bucket	Western edge of Southern Pacific Cornfield Property, southeast of North Broadway and Bishops Road
6	A-6	30	Bucket	Western edge of Southern Pacific Cornfield Property, northeast of North Broadway and Savoy Street.
7	A-7	36	Bucket	Western edge of Southern Pacific Cornfield Property at top of bluff, adjacent to North Broadway between Savoy Street and Solano Avenue.
8	A-8	40	Bucket	Western edge of Southern Pacific Cornfield Property, east of North Broadway between Solano Avenue and Casanova Street
9	A-9	50	Bucket	Western edge of Southern Pacific Cornfield Property at top of bluff, adjacent to North Broadway and north of Casanova Street
10	A-10	100	Bucket/Rotary Wash	Southern Pacific Cornfield Property, south of North Broadway Bridge
11	A-11	101	Rotary Wash	ATSF right-of-way, north of Humboldt Street and east of the Los Angeles River
12	A-12	25	Bucket	ATSF right-of-way, west of Avenue 26
13	A-13	51	Rotary Wash	ATSF right-of-way, north of Figueroa Street and east of Marmion Way
14	A-14	20	Bucket	ATSF right-of-way, north of Avenue 45 and east of Marmion Way

(Continued on following page)

TABLE 5.1

SUMMARY OF BORINGS

Boring No.	Plate No.	Boring Depth	Drilling Equipment Type	Location
15	A-15	16	Bucket	ATSF right-of-way, north of Museum Drive and east of Marmion Way
16	A-16	30	Bucket	ATSF right-of-way, east of Avenue 51 and south of Marmion Way
17	A-17	30	Bucket	ATSF right-of-way, between Avenues 57 and 58
18	A-18	30	Bucket	ATSF right-of-way, north of Avenue 61 and east of Figueroa Street
19	A-19	15	Bucket	ATSF right-of-way, at Arroyo Verde Road
20	A-20	30	Bucket	ATSF right-of-way, near Pasadena Avenue and Hawthorne Street
21	A-21	30	Bucket	ATSF right-of-way, south of Hawthorne Street near Adelaine Avenue
22	A-22	30	Bucket	ATSF right-of-way, south of Mission Street and west of Meridian Avenue
23	A-23	30	Bucket	ATSF right-of-way, north of Pasadena Freeway and east of Fremont Avenue
24	A-24	30	Bucket	ATSF right-of-way, north of Oaklawn Avenue Bridge and west of Fair Oaks Avenue
25	A-25	31	Bucket	ATSF right-of-way, south of Glenarm Street
26	A-26	30	Bucket	ATSF right-of-way, north of California Boulevard
27	A-27	40	Bucket	ATSF right-of-way, south of Del Mar Boulevard
28	A-28	60	Bucket	ATSF right-of-way, south of Green Street
29	A-29	40	Bucket	ATSF right-of-way, north of Holly Street
30	A-30	30	Bucket	Union Station Yard, north of 101 Freeway and south of Macy Street
31	A-31	30	Bucket	Union Station Yard, north of 101 Freeway and south of Macy Street
32	A-32	30	Bucket	Union Station Yard, north of Macy Street and south of Vignes Street
33	A-33	75	Rotary Wash	Vignes Street, between North Main Street and Bauchet Street

(Continued on following page)



TABLE 5.1

SUMMARY OF BORINGS

Boring No.	Plate No.	Boring Depth	Drilling Equipment Type	Location
34	A-34	101	Rotary Wash	Northeast corner of Alameda Street and Alpine Street
35	A-35	75	Rotary Wash	North Spring Street, north of College Street
36	A-36	100	Rotary Wash	Southern edge of Southern Pacific Cornfield Property, north of College Street and North Spring Street
37	A-37	35	Bucket	Southern Pacific Cornfield Property, between North Spring Street and North Broadway
38	A-38	35	Bucket	Southern Pacific Cornfield Property, east of North Broadway and Solano Avenue
39	A-39	31	Bucket	Center of Southern Pacific Cornfield Property, east of North Broadway and Solano Avenue
40	A-40	30	Bucket	Eastern edge of Southern Pacific Cornfield Property, near North Spring Street and Mesnager Street
41	A-41	30	Bucket	Eastern edge of Southern Pacific Cornfield Property, near North Spring Street between Mesnager Street and Aurora Street
42	A-42	30	Bucket	Center of Southern Pacific Cornfield Property, near North Spring Street, between Mesnager Street and Aurora Street
43	A-43	30½	Bucket	Southern Pacific Cornfield Property, near North Spring Street and Aurora Street, south of North Broadway Bridge
44	A-44	20	Bucket	Southern Pacific Cornfield Property, south of North Broadway Bridge, between North Spring Street and North Broadway
45	A-45	15	Bucket	Southern Pacific Cornfield Property, north of North Broadway Bridge and west of Los Angeles River
46	A-46	37	Bucket	Elysian Park, north of North Broadway and east of Elysian Park Drive

(Continued on following page)

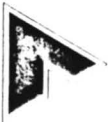


TABLE 5.1

SUMMARY OF BORINGS

Boring No.	Plate No.	Boring Depth	Drilling Equipment Type	Location
47	A-47	101	Rotary Wash	ATSF right-of-way, north of North Broadway Bridge and just west of Los Angeles River
48	A-48	32	Bucket	ATSF right-of-way, east of San Fernando Road
49	A-49	101	Rotary Wash	ATSF right-of-way, west of Golden State Freeway
50	A-50	15	Bucket	DWP Yard, northeast of Avenue 26, between Artesian Street and Lacy Street
51	A-51	15	Bucket	ATSF right-of-way, between Lacy Street and Artesian Street and south of Avenue 33
52	A-52	15	Bucket	ATSF right-of-way, north of Avenue 34
53	A-53	101	Rotary Wash	ATSF right-of-way, south of Pasadena Freeway
54	A-54	14	Bucket	ATSF right-of-way, south of French Avenue and east of Marmion Way
55	A-55	29	Bucket	ATSF right-of-way, near Avenue 37 and Pasadena Avenue
56	A-56	80	Rotary Wash	ATSF right-of-way, west of Pasadena Avenue between Avenue 38 and 39
57	A-57	75	Rotary Wash	ATSF right-of-way, south of Mt. Washington Drive and east of Marmion Way
58	A-58	30	Bucket	Marmion Way, south of Avenue 42
59	A-59	30	Bucket	ATSF right-of-way, east of Marmion Way and north of Avenue 43
60	A-60	20	Bucket	East of Marmion Way and north of Museum Drive
61	A-61	34	Bucket	East of Marmion Way and north of Museum Drive
62	A-62	30	Bucket	East of Marmion Way and south of Shanley Avenue
63	A-63	15	Bucket	ATSF right-of-way, west of Avenue 50 and south of Marmion Way
64	A-64	20	Short Boom Bucket	Marmion Way east of Avenue 53

(Continued on following page)

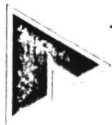


TABLE 5.1

SUMMARY OF BORINGS

Boring No.	Plate No.	Boring Depth	Drilling Equipment Type	Location
65	A-65	20	Bucket	Marmion Way, east of Avenue 55
66	A-66	30	Bucket	ATSF right-of-way, east of Avenue 57 and north of Marmion Way
67	A-67	20	Bucket	ATSF right-of-way, east of Avenue 59 and north of Marmion Way
68	A-68	20	Bucket	ATSF right-of-way, north of Figueroa Street, between Avenue 60 and 61
69	A-69	101	Rotary Wash	Center of Arroyo Seco Park, below ATSF Railroad bridge
70	A-70	76	Rotary Wash	Access Road beneath ATSF right-of-way, Arroyo Seco Park north of Avenue 60 ramp to northbound Pasadena Freeway
71	A-71	47	Bucket	ATSF right-of-way, south of Pasadena Freeway near Avenue 60
72	A-72	50	Bucket	ATSF right-of-way, south of Arroyo Drive and west of Arroyo Verde Road
73	A-73	37	Bucket	ATSF right-of-way, south of Arroyo Drive and west of Arroyo Verde Road
74	A-74	91	Bucket	North of Monterey Road and east of Lomitas Drive
75	A-75	6½	Hand Pit	ATSF right-of-way, between Arroyo Verde Road and Monterey Road
76	A-76	15	Mini Bucket	ATSF right-of-way, south of Pasadena Avenue and east of Arroyo Verde Road
77	A-77	30	Mini Bucket	ATSF right-of-way, south of Hawthorne Street near Cawston Street
78	A-78	20	Bucket	ATSF right-of-way, west of Orange Grove Avenue and south of Hawthorne Street
79	A-79	20	Short Boom Bucket	End of Glendon Court next to ATSF Railroad right-of-way
80	A-80	25	Short Boom Bucket	ATSF right-of-way, south of Hope Street and north of Mission Street
81	A-81	20	Bucket	Magnolia Street, east of Fairview Avenue
82	A-82	61	Rotary Wash	ATSF right-of-way, northeast corner of Fremont Avenue and Grevelia Street

(Continued on following page)

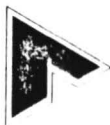


TABLE 5.1

SUMMARY OF BORINGS

Boring No.	Plate No.	Boring Depth	Drilling Equipment Type	Location
83	A-83	30	Bucket	West of Fair Oaks Avenue and north of Oaklawn Avenue Bridge
84	A-84	30	Bucket	ATSF right-of-way, west of Fair Oaks Avenue and south of Columbia Street
85	A-85	20	Bucket	ATSF right-of-way, south of State Street and east of Fair Oaks Avenue
86	A-86	30	Bucket	ATSF right-of-way, south of Fillmore Street and east of Raymond Avenue
87	A-87	30	Bucket	ATSF right-of-way, north of Fillmore Street and east of Raymond Avenue
88	A-88	15	Bucket	ATSF right-of-way, north of Bellevue Drive and west of Arroyo Parkway
89	A-89	30	Bucket	ATSF right-of-way, north of Del Mar Boulevard and west of Arroyo Parkway
90	A-90	60	Bucket	ATSF right-of-way, south of Green Street and west of Arroyo Parkway
91	A-91	61	Rotary Wash	Alley Way, west of Colorado Boulevard and east of Raymond Avenue
92	A-92	60	Rotary Wash	Union Street, between Raymond Avenue and Arroyo Parkway just west of railroad tracks
93	A-93	80	Bucket	Chestnut Street, east of Raymond Avenue
94	A-94	30	Bucket	ATSF right-of-way, north of Walnut Street, and west of Marengo Street and south of 210 Freeway
95	A-95	15	Bucket	ATSF right-of-way, 210 Freeway, east of Los Robles Avenue
96	A-96	15	Bucket	ATSF right-of-way, 210 Freeway, west of El Molino Avenue
97	A-97	15	Bucket	ATSF right-of-way, 210 Freeway, between El Molino Avenue and Lake Avenue
98	A-98	60	Rotary Wash	ATSF right-of-way, 210 Freeway, east of Lake Avenue
99	A-99	25	Bucket	ATSF right-of-way, 210 Freeway, west of Wilson Avenue

(Continued on following page)



TABLE 5.1

SUMMARY OF BORINGS

Boring No.	Plate No.	Boring Depth	Drilling Equipment Type	Location
100	A-100	15	Bucket	ATSF right-of-way, 210 Freeway, east of Wilson Avenue between Mar Vista Avenue and Michigan Avenue
101	A-101	15	Bucket	ATSF right-of-way, 210 Freeway, west of Hill Avenue between Chester Avenue and Holliston Avenue
102	A-102	15	Bucket	Southwest corner of Hill Avenue and Corson Street
103	A-103	15	Bucket	ATSF right-of-way, 210 Freeway, east of Hill Avenue
104	A-104	15	Bucket	ATSF right-of-way, 210 Freeway, west of Sierra Bonita Avenue
105	A-105	15	Bucket	ATSF right-of-way, 210 Freeway, between Sierra Bonita Avenue and Allen Avenue
106	A-106	15	Bucket	ATSF right-of-way, 210 Freeway, east of Allen Avenue
107	A-107	15	Bucket	ATSF right-of-way, 210 Freeway, east of Allen Avenue near Palo Verde Avenue
108	A-108	15	Bucket	ATSF right-of-way, 210 Freeway, west of Craig Avenue near Oak Avenue
109	A-109	40	Bucket	ATSF right-of-way, 210 Freeway, east of Craig Avenue
110	A-110	15	Bucket	ATSF right-of-way, 210 Freeway, east of Craig Avenue near Martello Avenue
111	A-111	15	Bucket	ATSF right-of-way, 210 Freeway, west of Altadena Drive near Carmelo Avenue
112	A-112	15	Bucket	Corson Street, east of Altadena Drive
113	A-113	35	Bucket	ATSF right-of-way, 210 Freeway, east of Altadena Drive
114	A-114	35	Bucket	ATSF right-of-way, 210 Freeway, west of Sierra Madre Boulevard
115	A-115	15	Bucket	ATSF right-of-way, 210 Freeway, east of San Gabriel Boulevard near Daisy Avenue
116	A-116	15	Bucket	ATSF right-of-way, 210 Freeway, east of Sunny Slope Drive

(Continued on following page)

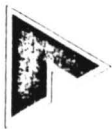


TABLE 5.1

SUMMARY OF BORINGS

Boring No.	Plate No.	Boring Depth	Drilling Equipment Type	Location
117	A-117	40	Bucket	ATSF right-of-way, 210 Freeway, south of Foothill Boulevard
118	A-118	30	Bucket	Titely Avenue, south of Foothill Boulevard
119	A-119	60	Rotary Wash	ATSF right-of-way, 210 Freeway, west Sierra Madre Villa Avenue
120	A-120	35	Bucket	ATSF right-of-way, 210 Freeway, east of Sierra Madre Villa Avenue
121	A-121	15	Bucket	ATSF right-of-way, 210 Freeway, between Sierra Madre Villa Avenue and Rosemead Boulevard near Sycamore Avenue
122	A-122	15	Bucket	ATSF right-of-way, 210 Freeway, west of Rosemead Boulevard



SECTION 6.0

SUBSURFACE CONDITIONS

6.1 SUBSURFACE INVESTIGATION

As discussed in Section 5.0, 122 borings were drilled as part of the geotechnical investigation. Twenty-nine of these borings were drilled for the two preliminary geotechnical studies.

6.2 SUMMARY OF SOIL CONDITIONS

6.2.1 FILL MATERIALS

Fill soils were encountered to depths of about 1 to 22 feet in most of the borings drilled along the portion of the alignment between Union Station in Los Angeles and the Foothill Freeway in Pasadena. Typically, the thickness of the fill soils was generally shallow. The deeper fills occurred in borings drilled at the top of the existing ATSF right-of-way embankment between the Los Angeles River and Avenue 33 and on the slopes located along the alignment. The fill soils generally consist of silt, silty sand, and sand with gravel, and with lesser amounts of clay. These fill soils are not uniformly well compacted and contain some debris.

Within the ATSF right-of-way in the middle of the Foothill Freeway (Interstate 210), compacted fill soils, 10 to 34 feet in depth, were encountered in the borings drilled east of Wilson Avenue; from Wilson Avenue to Rosemead Boulevard, the Foothill Freeway has been constructed on a fill embankment rising above the surrounding grade. These fill soils consist of silty sand, silt, and sand with gravel and cobbles, and were found to be firm and dense with greater than 90% compaction for the majority of the samples. Some of these borings were terminated in the fill as indicated on the boring logs in Appendix A. We understand that these fill soils were compacted under the supervision of CALTRANS. However, a copy of the CALTRANS report covering the placement of the fill was not provided to us.

6.2.2 NATURAL SOILS

The underlying overburden natural soils are topsoil, colluvium, landslide debris, alluvium and terrace deposits consisting of silt, silty sand, sand, and gravel with lesser deposits of clay. Varying amounts of cobbles and boulders were encountered in the sandy deposits. Bedrock



of the Puente and Topanga Formations, consisting of interbedded siltstone and sandstone, was encountered in 33 of the borings at depths ranging from 2 to 97 feet below the existing grade. Except for the upper few feet, the overburden natural soils are generally firm and dense.

Bedrock was encountered at Elevations of 299 to 335 on the south side of the existing ASTF bridge over the Los Angeles River, and at Elevation 308 on the north side of the river. A 1940 U.S. Corps of Engineers' drawing indicates that bedrock elevation varied between Elevation 262 at the west bank and Elevation 271 at the east bank of the river. It is expected that the bedrock will be encountered at the river invert within the limits of the bridge. The bedrock encountered in the borings is firm to very firm. The bedrock beneath the river invert may contain numerous cemented beds.

6.3 SUMMARY OF GROUND WATER CONDITIONS

The exploration borings were drilled using either bucket auger-type or rotary wash-type equipment. Drilling mud was used in the rotary wash borings, making it difficult to determine the water levels in these borings. For water level monitoring and sampling, piezometers were installed in some of the rotary wash borings as was discussed previously.

Table 4.1 in Section 4 summarizes the water level measurements in piezometers installed in the rotary wash borings. Water seepage was encountered in 16 borings at depths ranging from 5 feet to 58 feet below the existing grade; these borings were drilled with bucket equipment along the alignment.



SECTION 7.0 SLOPE STABILITY ANALYSES

Analyses were performed to evaluate the stability of existing slopes along the Metro Pasadena Line for deep-seated and surficial stability.

As stated in Section 4.0, there are seven areas along the Metro Pasadena Line alignment with significant potential for slope stability problems. The potential for slope instability within each area was evaluated during our field investigation by geologic mapping and downhole logging, and ten geologic sections were prepared for slope stability analyses. The sections are presented on Plates 2.1 through 2.10.

Except for Section A-A', which consists of older alluvial deposits underlain by bedrock of the Puente Formation, the sections consist of bedrock materials of the Puente and Topanga Formations. Near Arroyo Seco Park, Sections H-H' and I-I' are underlain by landslides, and there is landslide debris at the toe of the steep portion of Section G-G'. Along most of the alignment, bedding orientations are generally favorable. However, in the vicinity of Marmion Way in Highland Park, locally adverse bedding planes were observed.

The slopes vary in height from 20 to 180 feet, and are as steep as 3/4:1 (horizontal to vertical) at the location of G-G' and as flat as 6:1 at the location of D-D'.

Based on our stability analyses, the existing landslide masses and the slopes, except the slope at Section G-G', are grossly stable with a factor of safety of more than 1.5.

Section G-G', located east of the existing Arroyo Seco Park railroad crossing, was determined to have a factor of safety of less than 1.5 due to the steepness of the slope. As mentioned earlier, although the bedding is favorable, the slope face is subject to severe erosion and rockfall because of adverse joint and fracture geometry, especially during periods of heavy



rainfall. The ATSF Railroad is believed to have a system of slope movement detectors in this area. We are aware that ATSF performs clean-up activities in this area during and after major rainfalls.

It is our opinion that the current slope conditions in this area (slope stability Area 3) would present difficulties in continuous operation of the Metro Pasadena Line, especially when there is precipitation. To reduce the hazards associated with this slope, the inclination of the slope could be flattened and/or slough walls constructed. A system of movement detectors could be installed to provide early warning of slope movement. Consideration could also be given to construction of a cover structure in this area over the alignment. Another alternative would be to modify the alignment to avoid this area. We recommend that additional geotechnical studies be conducted in this area. Interaction with the section designers will be necessary to provide an effective engineering solution to the problems associated with this slope.



SECTION 8.0

RECOMMENDATIONS – AERIAL STRUCTURES

8.1 GENERAL

Design recommendations for structures with similar soil and water conditions have been developed. Basically, the design criteria for the aerial structures fall into three groups, as follows:

Design Group 1

- Aerial Structures - Along Vignes Street, and over the intersection of Main Street, Alpine Street, and Alhambra Avenue, and over Alameda Street (Borings 1, 2, 33 and 34)
- Aerial Structures - North Spring Street and College Street (Borings 3 and 35)
- Elevated Chinatown Platform Station (Borings 4, 36, and 37)

Design Group 2

- Los Angeles River Bridge (Borings 10, 11, 45, 46, and 47)
- Arroyo Seco Park and Pasadena Freeway Bridge (Borings 69 and 70)

Design Group 3

- Avenue 19 Bridge (Boring 48)
- San Fernando Road Bridge (Borings 48 and 49)
- Golden State Freeway Bridge (Borings 49)
- Avenue 26 Bridge (Borings 12 and 50)
- Arroyo Seco Channel and Pasadena Freeway Bridge north of Pasadena Avenue (Borings 52,53, and 54)
- Pasadena Freeway Bridge near Fremont Avenue (Borings 23 and 82)



In addition, Group 3 includes all the existing bridges along the Foothill Freeway.

The aerial structures, which include bridges to be rebuilt or upgraded and elevated stations, may be supported on drilled cast-in-place concrete piling. Drilled piling will result in reduced settlement and will provide good foundation support for resistance of seismic loads and other lateral forces. Either groups of piles or single large-diameter concrete piles may be used. The larger piles may be used for support of the heavier columns of the aerial structure and the smaller size piles could be used to support the lighter columns of the aerial structures near the elevated station in Chinatown. Some of the pile foundations at the stations would be embedded into pile caps to obtain a fixed-head condition to enable the support of greater lateral loads.

Some of the minor and lighter columns of the aerial structures, such as bridge piers and bridge abutments, may be supported on small-diameter drilled piles or on driven piles. Prestressed concrete piles, HP steel piles (H-section), or steel pipe piles may be used.

In summary, the following pile systems are recommended for the various segments:

- Large-Diameter Drilled Piles - Heavy Aerial Structures including Bridges
- Small-Diameter Drilled Piles - Lighter Aerial Structures, Abutments, and Bridge Piers
- Driven Concrete Piles - Abutments, Bridge Piers, and Light Aerial Structures

Spread footings may be a feasible foundation alternative at selected locations within the three groups.

Settlement analyses have been performed for each foundation system for a range of expected foundation loads. The results of the analyses are presented in the following sections.



8.2 DESIGN GROUP 1 (*Aerial Structures – Along Vignes Street, and over the intersection of Main Street, Alpine Street, and Alhambra Avenue, and over Alameda Street; Aerial Structures – over North Spring and College Streets; elevated Chinatown Platform Station*)

8.2.1 SUBSURFACE PROFILE AND SOIL PROPERTIES

Borings 1 through 4 and 33 through 37 were drilled for the aerial structures in this design group. Logs of the borings are presented in Appendix A, Volume 2. Based on the borings, the overburden soils basically consist of silt, silty sand, and sand with gravel and cobbles. Bedrock was encountered at depths varying from 35 to 65 feet below the existing grade. Ground water was measured at depths of 20 to 40 feet below existing grade.

The following soil properties may be used for static design for Design Group 1:

Cohesion	250 psf
Void Ratio	
Granular Soils	0.50
Cohesive Soils	0.75
Bedrock Materials	0.70
Modulus of Subgrade Reaction	
Granular Soils	200 pci
Cohesive Soils and Bedrock Materials	150 pci
Angle of Internal Friction	36 degrees
Moist Unit Weight of Soil	
0 to 20 feet	120 pcf
Below 20 feet	60 pcf

8.2.2 FOUNDATIONS

8.2.2.1 Pile Foundations

Vertical Loads - Drilled Piles: The downward and upward frictional capacities of 16-, 20-, 24-, 60-, 72-, and 84-inch-diameter drilled piles are presented on Figures 8.1 and 8.2, Drilled Pile Capacities, Design Group 1. The capacities of other sizes of piles may be taken as proportional



to the diameter. Dead plus live load capacities are shown; a one-third increase may be used for wind or seismic loads. The dead plus live capacities are allowable capacities; a factor of safety of 2 was used to determine the allowable capacities. If piles in groups are required, the piles should be spaced not less than $2\frac{1}{2}$ diameters on centers so that group action need not be considered. The capacities are based on the strength of the soils; the compressive and tensile strength of the pile section itself should be checked to verify the structural capacity of the pile.

Settlement – Drilled Piles: The settlement of drilled piles will be less than $\frac{1}{2}$ inch for vertical loads between 150 kips and 3,500 kips. For example, the settlement of a 16-inch-diameter pile supporting a vertical load of about 150 kips will be less than $\frac{1}{4}$ inch, and the settlement of an 84-inch-diameter pile supporting a vertical load of about 3,500 kips will be less than $\frac{1}{2}$ inch.

Vertical Loads - Driven Piles: The downward and upward frictional capacities of 12-, 14- and 16-inch-square prestressed concrete piles are presented on Figure 8.3, Driven Pile Capacities, Design Group 1. If batter piles are used, only the vertical projection of the length of the batter pile should be considered in computing the axial pile capacities. Dead plus live load capacities are shown; a one-third increase may be used for wind or seismic loads. The dead plus live capacities are allowable capacities computed with a factor of safety of at least 2. Piles in groups should be spaced not less than $2\frac{1}{2}$ pile widths or 3 feet on centers, whichever is greater, so that group action need not be considered. The capacities are based on the strength of the soils; the compressive and tensile strength of the pile section itself should be checked to verify the structural capacity of the pile.

Settlement – Driven Piles: The settlement of 12-, 14-, and 16-inch-square prestressed concrete piles, supporting vertical loads up to 300 kips, will range from $\frac{1}{4}$ to $\frac{1}{2}$ inch.



Lateral Resistance: The piles may be used to resist lateral loads. The lateral capacity of the piles will depend on the permissible deflection and on the degree of fixity at the top of the pile. The lateral deflection in the piles (in terms of applied load at the top of the pile) is presented on Figures 8.4A through 8.4I, Lateral Deflection Diagrams, Design Group 1. The lateral capacity of a batter pile is equivalent to the horizontal component of the axial pile capacity.

The soil resistance versus pile deflection for the piles is shown on Figures 8.5A through 8.5J, (P-Y) Curves, Design Group 1. These curves were developed using the computer program LPILE distributed by Ensoft, Inc., Austin, Texas. These (P-Y) curves should only be used in LPILE or similar computer programs to determine the behavior of a laterally loaded pile.

The bending moment in the piles due to application of the lateral loads may be distributed in the piles in accordance with Figures 8.6A through 8.6I, Bending Moment Diagrams, Design Group 1.

The lateral capacity and bending moment distributions are based in part on the assumption that any backfill adjacent to the piles will be properly compacted. Furthermore, if the pile locations are predrilled, the backfill in the annular space around the pile should be tight to assure good distribution of load into the surrounding soils. A pile in a pile group of two or more with a pile cap may be considered as a fixed-head pile.

The passive resistance of properly compacted backfill against pile caps may be assumed to be equal to the pressure developed by a fluid with a density of 300 pounds per cubic foot. A one-third increase in the quoted passive values may be used when considering wind or seismic loads.

Ultimate Design: The various design values recommended for piling are for use with loadings determined by a conventional working stress design. If an ultimate design concept is used, the recommended design values may be multiplied by the following factors:



Design Item	Ultimate Design Factor
Axial Capacity of Piles	2.0
Lateral Capacity of Piles	1.5
Passive Resistance	1.25

In no event, however, should the pile lengths be reduced from those required for support of dead plus live loads when using the working stress values.

Installation - Drilled Piling: As previously mentioned, the alignment is underlain by geologic units consisting of artificial fill, topsoil, colluvium, landslide debris, alluvial and terrace deposits, underlain by sedimentary bedrock of the Puente and Topanga Formations. There are significant amounts of sand, gravel, and cobbles within the artificial fill, colluvium, alluvial, and terrace deposits. Caving of boring walls occurred in some of the borings drilled with bucket auger-type drilling equipment. A significant percentage of prior borings drilled by our firm for other projects in the vicinity with bucket auger-type drilling equipment also experienced significant caving.

The bucket auger exploration borings were drilled typically with relatively small-diameter equipment, ranging from 16 to 24 inches in diameter. It should be noted that the caving potential increases as the diameter of the excavation increases. The sandy soils are relatively cohesionless, and may be held together by small capillary tension forces. If allowed to dry, the soils would lose that capillary tension and would be more likely to cave. The use of high speed drilling equipment would also cause disturbance of the soils, increasing the likelihood of caving. The presence of ground water would also increase the likelihood of caving.

It should be anticipated that significant caving, raveling, and sloughing may occur during excavation of the large-diameter drilled piles, particularly below the water, unless positive steps are taken to support the sides of the hole. Special techniques will likely be needed to satisfactorily install the piles. Casing may or may not be required, depending on the field conditions.



Only drilling contractors with demonstrated experience in the installation of drilled cast-in-place large-diameter piles using drilling mud and casing in similar circumstances should be considered for the pile work. The drilling system should be equipped to include the use of casing and drilling mud to prevent shaft caving during drilling and placing of concrete. The equipment should provide for the introduction of drilling mud of sufficient specific gravity to retain the shaft sides, and continuous pumping for retrieval of the mud concurrent with concrete placement. The pressure head of mud in the shaft should be sufficient to maintain a stable bottom during all construction operations. Only environmentally-safe drilling mud should be used.

The contractor should provide the equipment necessary to check the bearing, dimensions, and alignment of each drilled shaft.

Drilling should be performed in a sequence such that no adjacent pile closer than 5 pile diameters on center is drilled sooner than eight hours after placement of the concrete.

Care should be exercised during placement of the reinforcement steel to minimize disturbance and caving of the sides of any uncased portions of the drilled shaft.

Concrete should be pumped through a tremie pipe extending to the bottom of the shaft, with the pipe being at least 8 feet below the surface of the concrete at all times during placement. The discharge end of the tremie should be constructed so that it is watertight against intrusion from the outside and so that it will readily discharge concrete. The tremie pipe should be supported so that it can be raised and lowered rapidly should it become necessary to cut off or retard the flow. Casing should be pulled as the concrete is placed; the bottom of the casing, however, should be maintained at least 3 feet below the surface of the concrete at all times. The discharge pipe should be kept full of concrete during the entire placing operation and should not be removed from the concrete until all of the concrete is placed and fresh concrete



appears at the top of the pile. The volume of concrete pumped into the hole should be recorded and compared to the design volume.

We recommend that continuous observation of the drilling and pouring of the piles be performed by a geotechnical engineer or an engineering geologist licensed in the State of California to verify that the desired diameter and depth are obtained.

Each pile should be tested by a nondestructive testing method to check the pile's integrity. The testing should be performed after at least three days of curing time has elapsed to allow the concrete to harden sufficiently. The nondestructive testing should be undertaken by an experienced and approved independent testing organization.

One of the nondestructive methods that has been successfully used for large diameter piles, is called crosshole sonic logging (CSL). This method measures the time it takes for an ultrasonic pulse to travel from a signal source in one access tube placed within a pile to a receiver in another access tube placed within the same pile. In unreinforced quality concrete, the travel time between the two tubes will be relatively constant, and the CSL test will produce records with good signal amplitude. Longer travel times and lower amplitude/energy signals indicate the presence of irregularities such as poor quality concrete, voids, and soil intrusions. The signal will be completely lost by the receiver for more severe defects such as large voids and soil intrusions.

Steel tubes are recommended because there will be good bonding between steel and concrete, allowing for better transmission of the ultrasonic pulses. Plastic or PVC pipes would be more economical; however, our experience with PVC pipes indicates that certain amounts of debonding occur between the PVC pipe and the concrete after the concrete sets; this is especially the case near the top of the piles. The debonding could lead to erroneous, misleading, or inconclusive readings, thus requiring further testing that may offset any savings in the cost of pipe material. To minimize the debonding, if PVC pipe is used, the tubes



should be filled with water as soon as possible after the concrete is poured, preferably on the same day.

Installation - Driven Piling: All piles should be driven to the predetermined design lengths as shown on Figure 8.3, except as may be modified on the basis of the driving criteria defined on Figure 8.7, Pile Driving Criteria. The pile driving criteria shown on Figure 8.7 may be taken only as a guide. Prior to ordering production piles, we recommend that indicator piles be driven to evaluate the driving resistance. At least three indicator piles should be driven at each abutment and/or bridge pier location. The driving criteria and installation procedures can be modified based on the results of the indicator piles and any necessary adjustments can be made to the design lengths presented on Figure 8.3. The installation of the piling should be observed by a geotechnical engineer licensed in the State of California so that modifications in the driving criteria and the pile lengths can be made if required.

Hard driving will be encountered within the sandy soils and bedrock; predrilling may be required. The diameter of the predrilled hole should not exceed 80% of the pile width. Predrilling may be extended to within 5 feet of the design tip elevation of the pile.

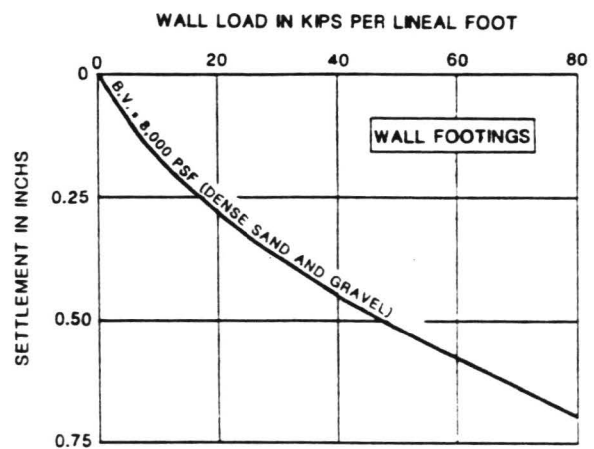
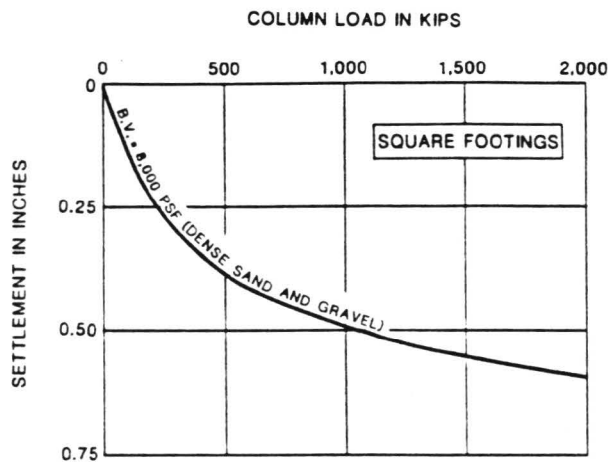
8.2.2.2 Spread Footings

Bearing Value: Spread footings within Design Group 1 may impose bearing pressures up to 8,000 pounds in the dense sand and gravel.

Footings should extend at least 1 foot into the supporting material and should be established at least 3 feet below the lowest adjacent final grade. The depth to the dense sand and gravel may be deeper and govern the footing depth. A one-third increase in the recommended values may be used for wind or seismic loads. The recommended bearing values are net values. The weight of concrete in the footings may be assumed to be 50 pounds per cubic foot; the weight of soil backfill may be neglected when determining the downward load on the footings.



Settlement: The settlement of spread footings established in the dense sand and gravel may be determined using the following graphs.



Lateral Loads: Lateral loads may be resisted by soil friction and by the passive resistance of the soils. A coefficient of friction of 0.5 may be used between footings and the supporting soils. A passive value of 300 pounds per square foot per foot of depth may be used. A one-third increase in the passive values may be used for wind or seismic loads. The frictional resistance and the passive resistance of the soils be combined without reduction in determining the total lateral resistance.



Ultimate Design: The recommended bearing value and lateral load design values are for use with loadings determined by a conventional working stress design. When considering an ultimate design approach, the recommended design values may be multiplied by the following factors:

Design Item	Ultimate Design Factor
Bearing Value	2.5
Passive Pressure	1.5
Coefficient of Friction	1.25

In no event, however, should foundation sizes be less than those required for dead plus live loads when using the working stress design.

Footing Observation: The footing excavations should be observed by a geotechnical engineer or an engineering geologist licensed in the State of California to verify the presence of satisfactory materials at footing design elevations. Footings should be deepened if necessary to extend into satisfactory supporting material.

The foundations should be left slightly uneven if necessary, rather than filling in over-excavated areas with loose or compacted soils. Where it is necessary to deepen a footing below the design depth, the overexcavated portion should be backfilled with concrete. It would be desirable and feasible to pour spread footings neat against the undisturbed soils. Soil backfill should be compacted as recommended in Section 13.0, Earthwork.

Subsurface seepage may be encountered in the footing excavations. A layer of lean concrete may be used to protect the bottom of the excavations and to provide support during construction.



8.2.3 ABUTMENT WALLS

For design of the cantilevered abutment walls, where the backfill is level and properly drained, it may be assumed that the soils will impose an active lateral pressure equal to that developed by a fluid with a density of 35 pounds per cubic foot. For walls with backfill sloped as steep as 2:1 (horizontal to vertical), an equivalent fluid pressure of 53 pounds per cubic foot may be used. An appropriate uniform lateral pressure equal to one-third of the anticipated vertical surcharge within 20 feet of the abutment walls should be included in the wall design.

The above values are for properly drained soils; weep holes should be provided to relieve hydrostatic pressures due to water.

8.2.4 SEISMIC DESIGN DATA

The proposed aerial structures will be designed in accordance with the bridge design criteria of the State of California Department of Transportation (CALTRANS). The geotechnical information needed for the CALTRANS criteria is the depth to rock and the expected peak bedrock acceleration. Based on a review of the soils and geologic data, the thickness of alluvium may be taken as being between 10 to 80 feet for Design Group 1. The expected peak bedrock acceleration is 0.6g, based on the proximity of nearby faults. In addition, the following values may be used in the seismic design of the aerial structures:

Shear Modulus	
0 to 25 feet	2,400 ksf
25 to 75 feet	8,400 ksf
Below 75 feet	14,900 ksf
Poission's Ratio	0.3
Damping Value	0.1

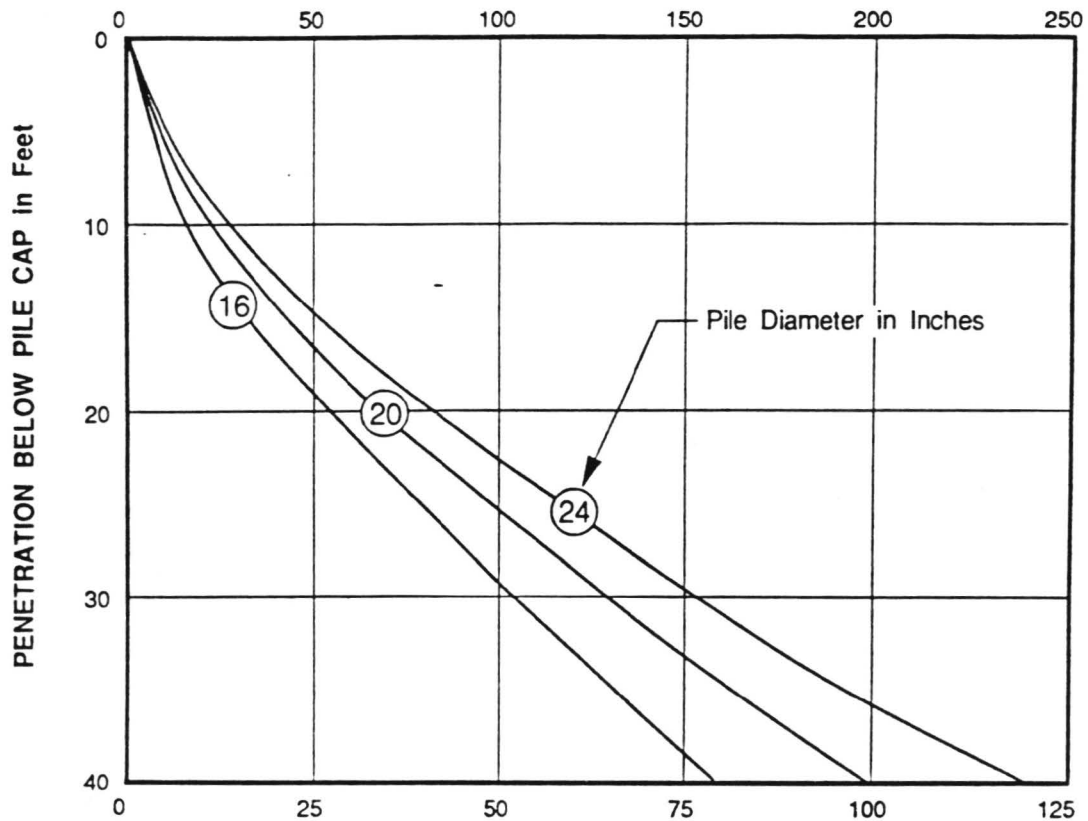


The following figures complete the recommendations for Design Group 1:

Figure No.	Title
8.1 – 8.2	Drilled Pile Capacities, Design Group 1
8.3	Driven Pile Capacities, Design Group 1
8.4A – 8.4I	Lateral Deflection Diagrams, Design Group 1
8.5A – 8.5J	(P-Y) Curves, Design Group 1
8.6A – 8.6I	Bending Moment Diagrams, Design Group 1
8.7	Pile Driving Criteria, Design Group 1



DOWNWARD PILE CAPACITY in Kips



UPWARD PILE CAPACITY in Kips

NOTES :

- (1) The indicated values refer to the total of dead plus live loads; a one-third increase may be used when considering wind or seismic loads.
- (2) Piles in groups should be spaced a minimum of 2-1/2 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.

DRILLED PILE CAPACITIES

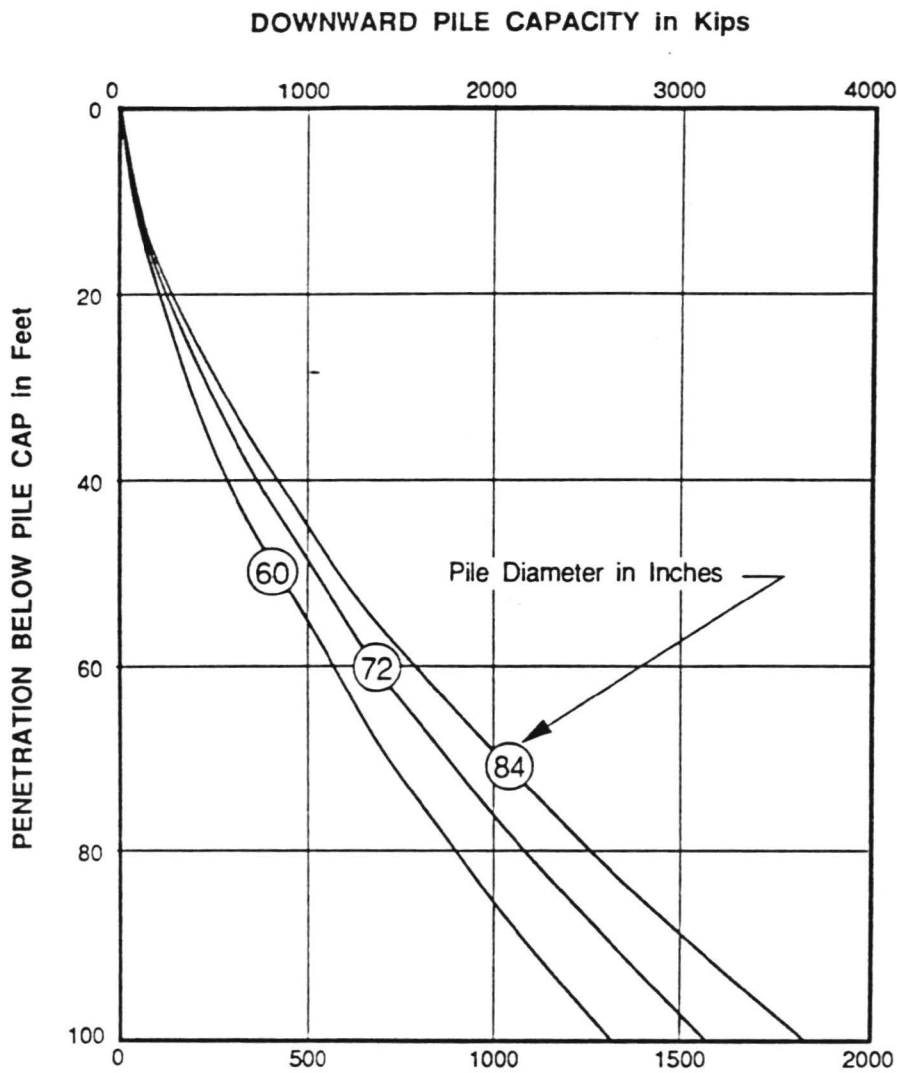
(Design Group 1)

LAW / CRANDALL, INC.



FIGURE 8.1

JOB L92045,AE4 DATE 2/15/93 DR. IK O.E. MS ~~NY~~ CHKD ~~NY~~



NOTES :

- (1) The indicated values refer to the total of dead plus live loads; a one-third increase may be used when considering wind or seismic loads.
- (2) Piles in groups should be spaced a minimum of 2-1/2 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.

DRILLED PILE CAPACITIES

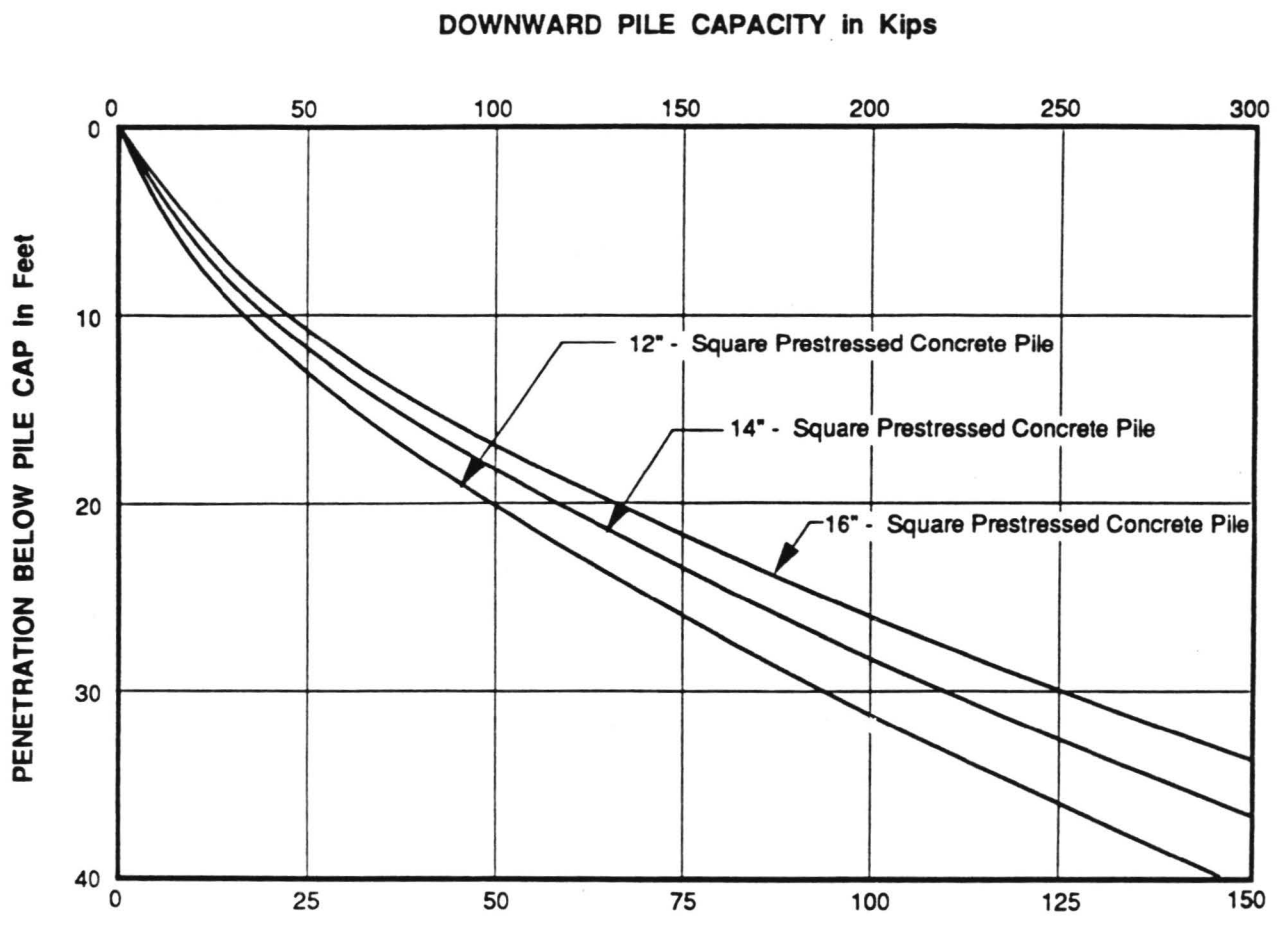
(Design Group 1)

LAW / CRANDALL, INC.



FIGURE 8.2

JOB L92045.AE4 DATE 2/15/93 DR. IK O.E. MS/11/12 CHKD *MS*



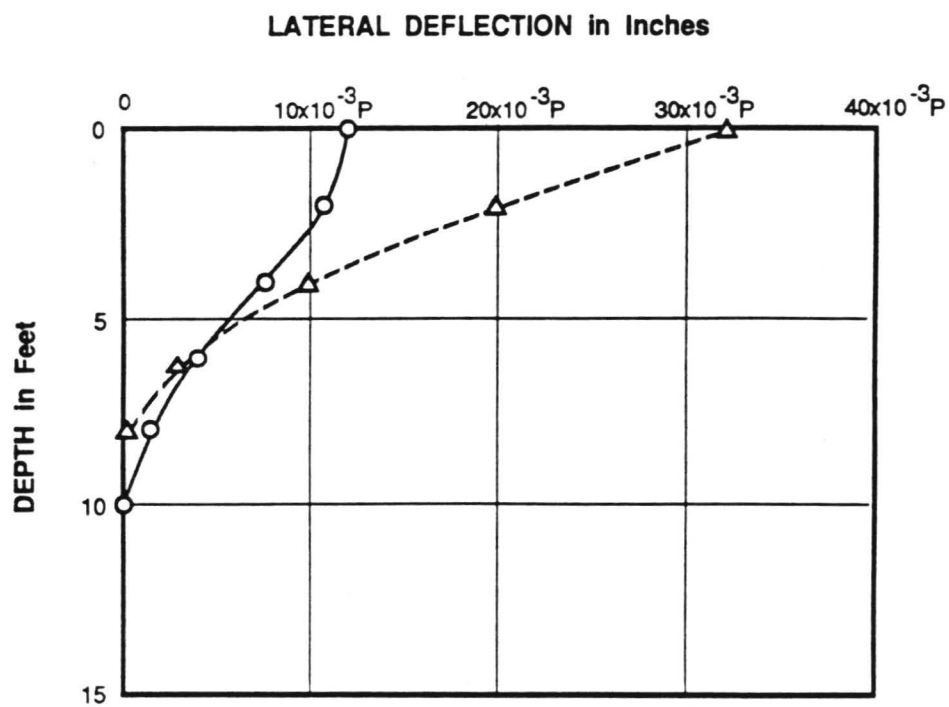
UPWARD PILE CAPACITY in Kips

NOTES :

- (1) The indicated values refer to the total of dead plus live loads; a one-third increase may be used when considering wind or seismic loads.
- (2) Piles in groups should be spaced a minimum of 3 feet on centers.
- (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.

DRIVEN PILE CAPACITIES
(Design Group 1)

JOB L92045.AE4 DATE 4/29/93 DR. *MB* O.E. YK *CK* CHKD



KEY:

○—○ FIXED HEAD PILE

△--△ FREE HEAD PILE

P LATERAL LOAD APPLIED AT TOP OF THE PILE IN KIPS

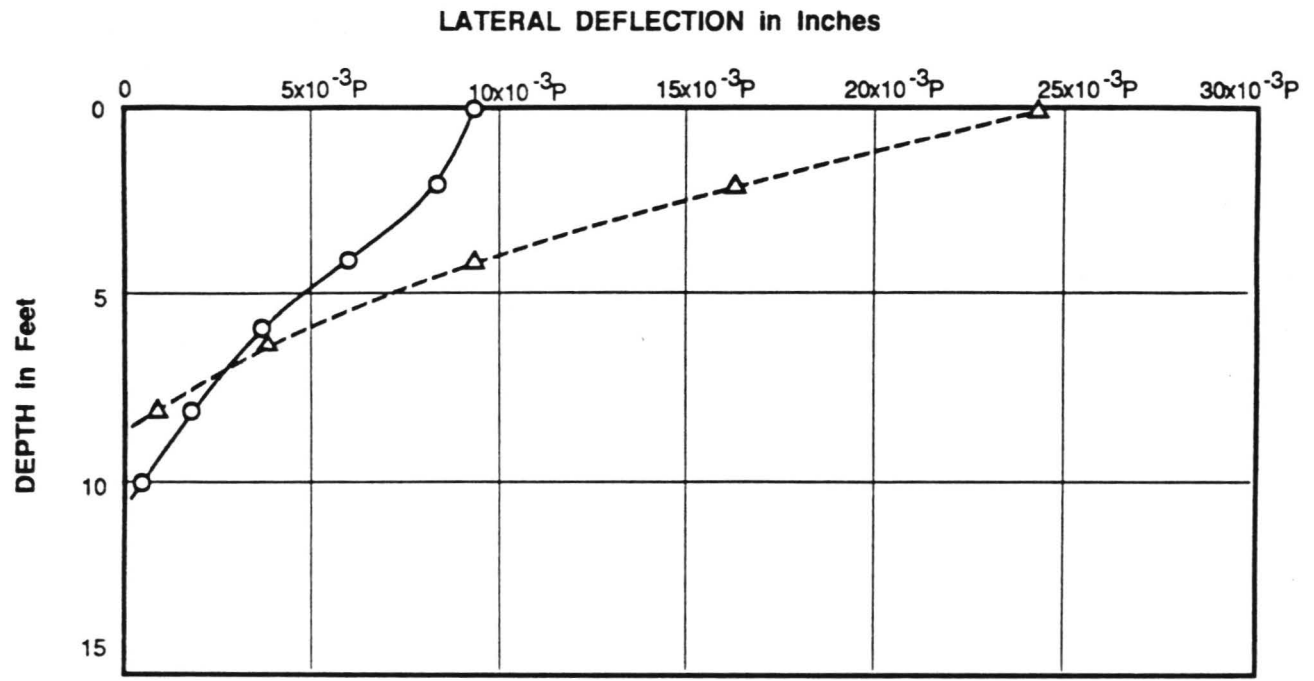
LATERAL DEFLECTION DIAGRAMS
12 - INCH SQUARE PILE
 (Design Group 1)

LAW / CRANDALL, INC.



FIGURE 8.4A

JOB L92045.AE4 DATE 4/29/93 DR. ☒ CHKD *MS*
 O.E. YK ☒



KEY:

- FIXED HEAD PILE
- △--△ FREE HEAD PILE
- P LATERAL LOAD APPLIED AT TOP OF THE PILE IN KIPS

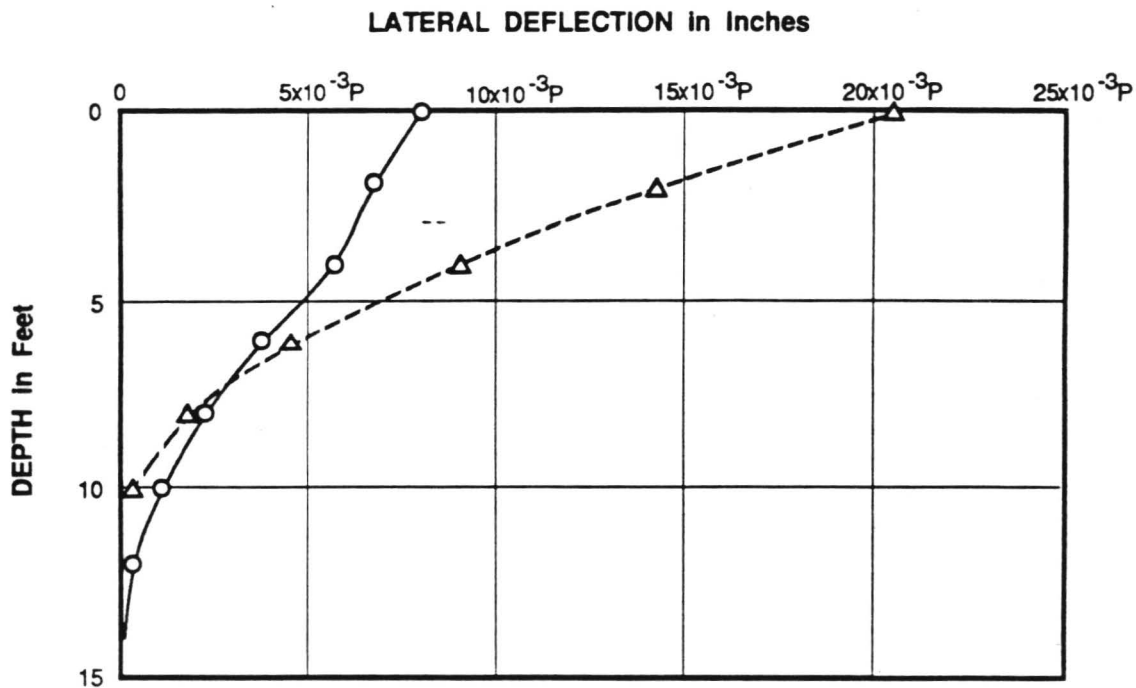
LATERAL DEFLECTION DIAGRAMS
14 - INCH SQUARE PILE
 (Design Group 1)

LAW/CRANDALL, INC.



FIGURE 8.4B

JOB L92045, AE4 DATE 4/29/93 DR. *MS* O.E. YK *MS* CHKD



KEY:

○—○ FIXED HEAD PILE

△--△ FREE HEAD PILE

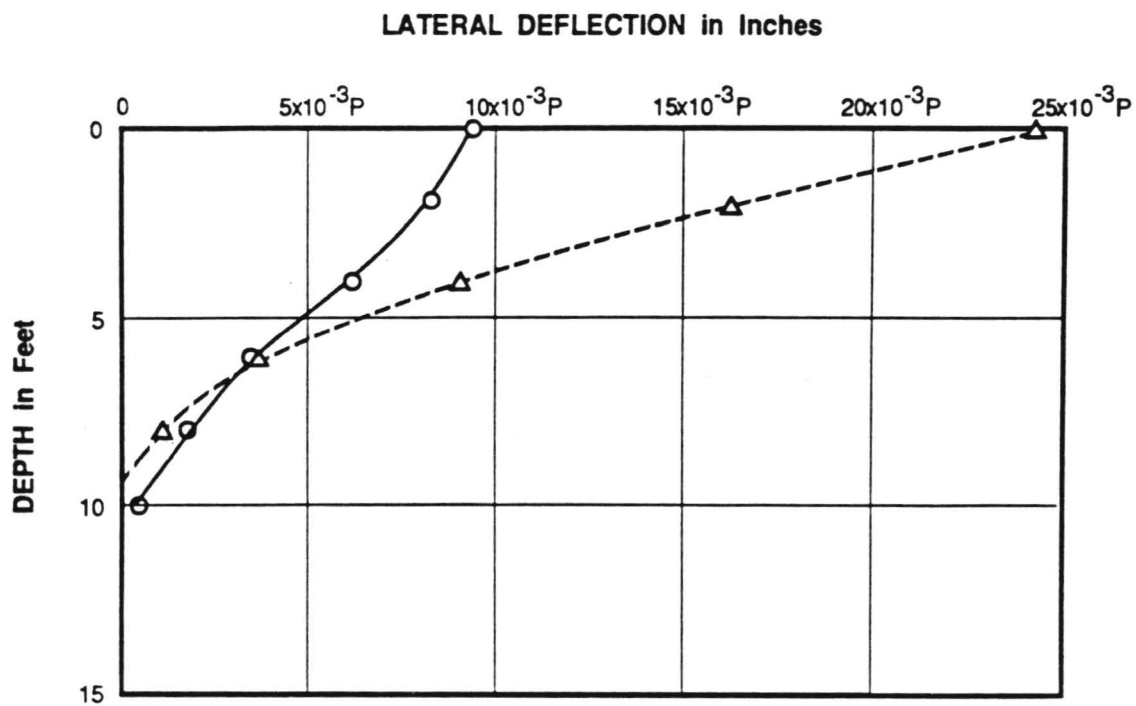
P LATERAL LOAD APPLIED AT TOP OF THE PILE IN KIPS

LATERAL DEFLECTION DIAGRAMS
16 - INCH SQUARE PILE
 (Design Group 1)

LAW / CRANDALL, INC.

FIGURE 8.4C

JOB L92045/AE4 DATE 4/29/93 DR. IK O.E. YK CHKD MS



KEY:

- FIXED HEAD PILE
- △--△ FREE HEAD PILE
- P LATERAL LOAD APPLIED AT TOP OF THE PILE IN KIPS

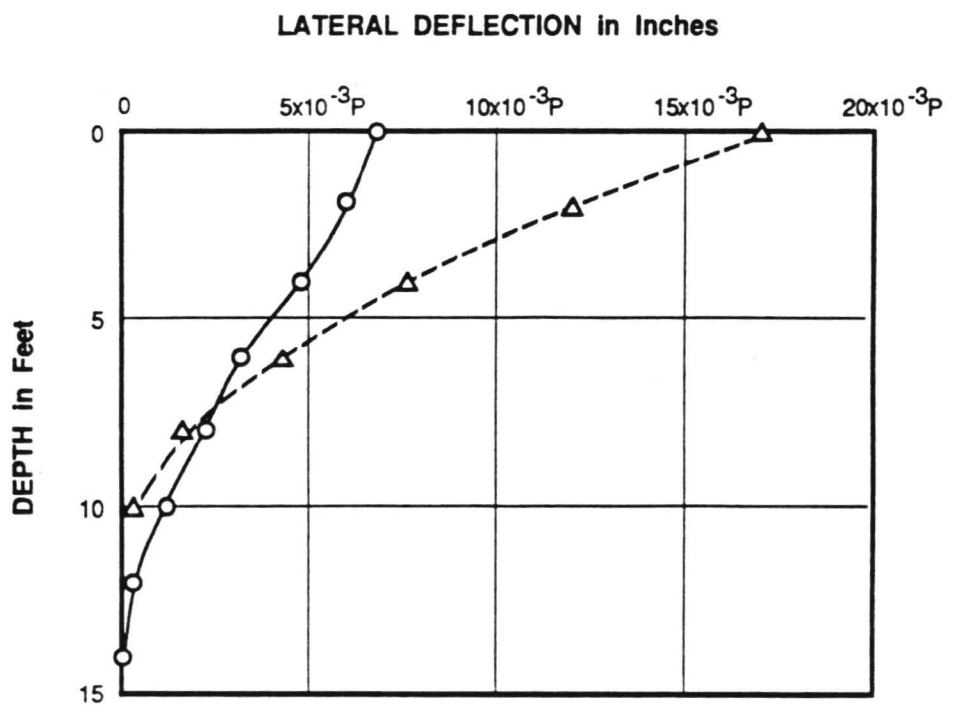
LATERAL DEFLECTION DIAGRAMS
16 - INCH DIAMETER PILE
(Design Group 1)

LAW / CRANDALL, INC.



FIGURE 8.4D

JOB L92045.AE4 DATE 4/29/93 DR. IK O.E. YK 115 CHKD 115



KEY:

○—○ FIXED HEAD PILE

△--△ FREE HEAD PILE

P LATERAL LOAD APPLIED AT TOP OF THE PILE IN KIPS

LATERAL DEFLECTION DIAGRAMS

20 - INCH DIAMETER PILE

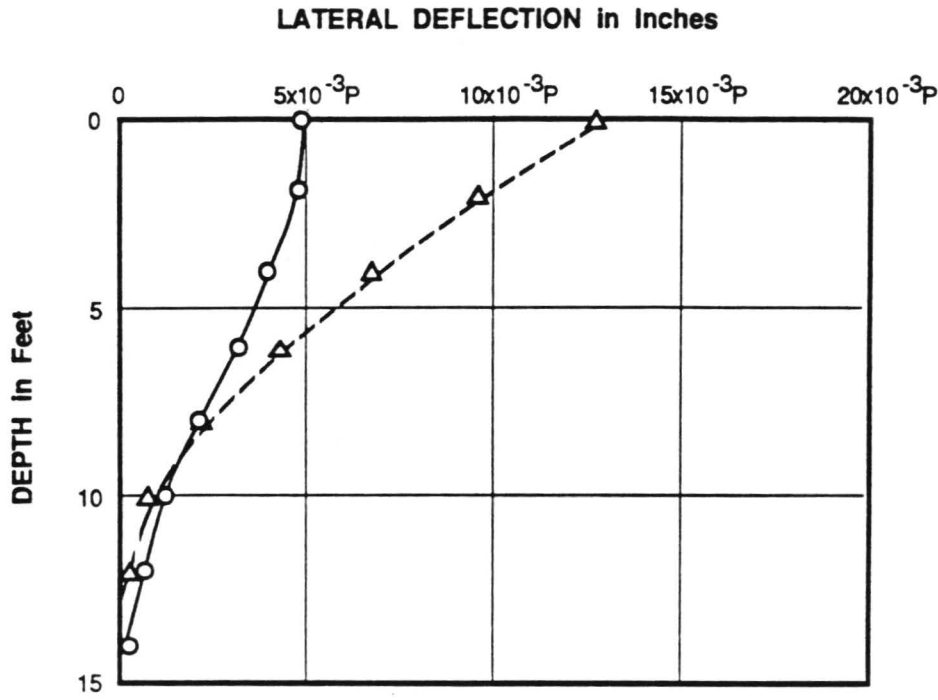
(Design Group 1)

LAW / CRANDALL, INC.



FIGURE 8.4E

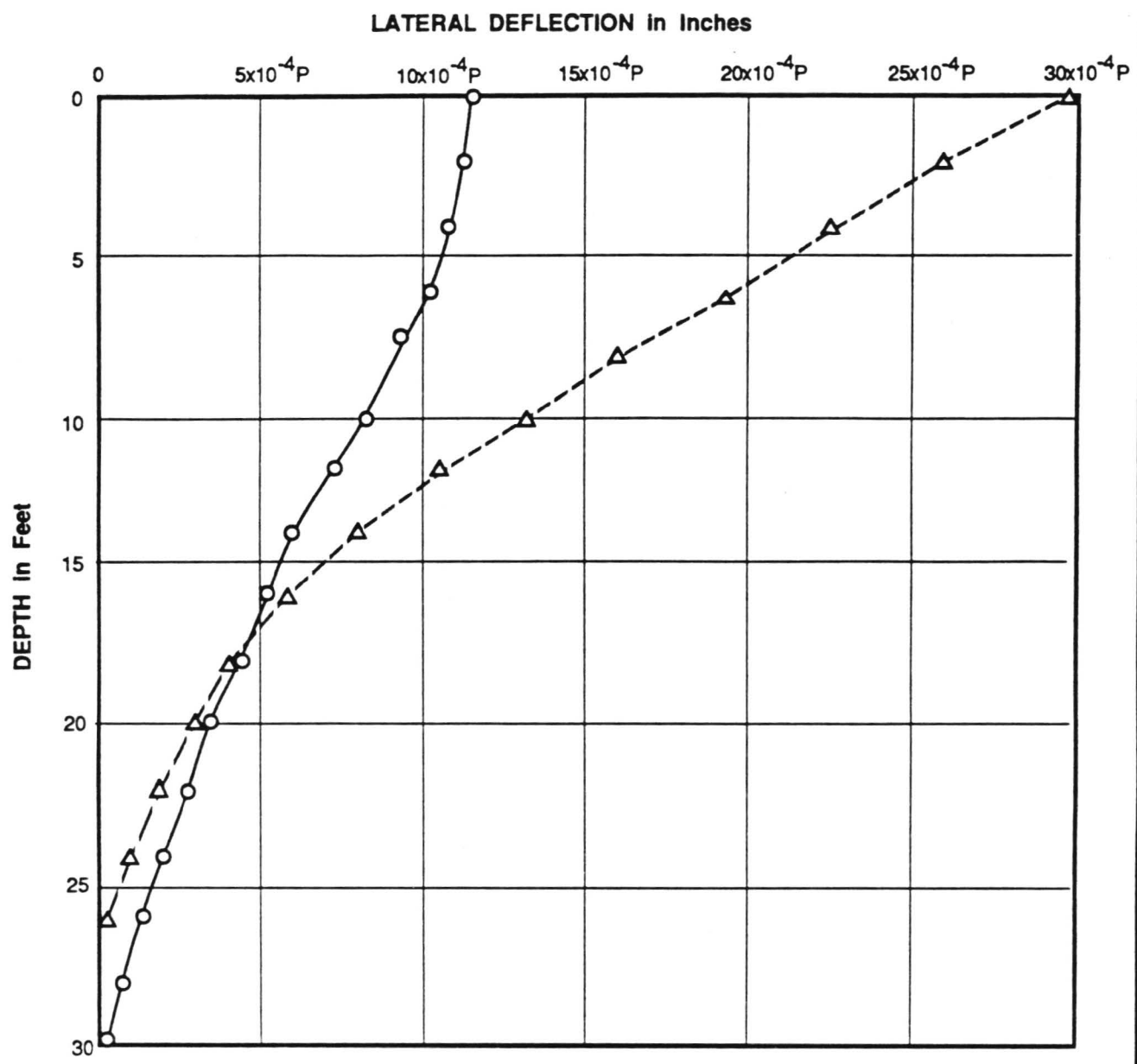
JOB L92045/AE4 DATE 4/29/93 DR. MS CHKD YK O.E. IK



KEY:
 ○—○ FIXED HEAD PILE
 △--△ FREE HEAD PILE
 P LATERAL LOAD APPLIED AT TOP OF THE PILE IN KIPS

LATERAL DEFLECTION DIAGRAMS
 24 - INCH DIAMETER PILE
 (Design Group 1)

JOB L92045.AE4 DATE 4/29/93 DR. *PK* CHKD *PK* YK *PK* O.E. *PK* IK



KEY:

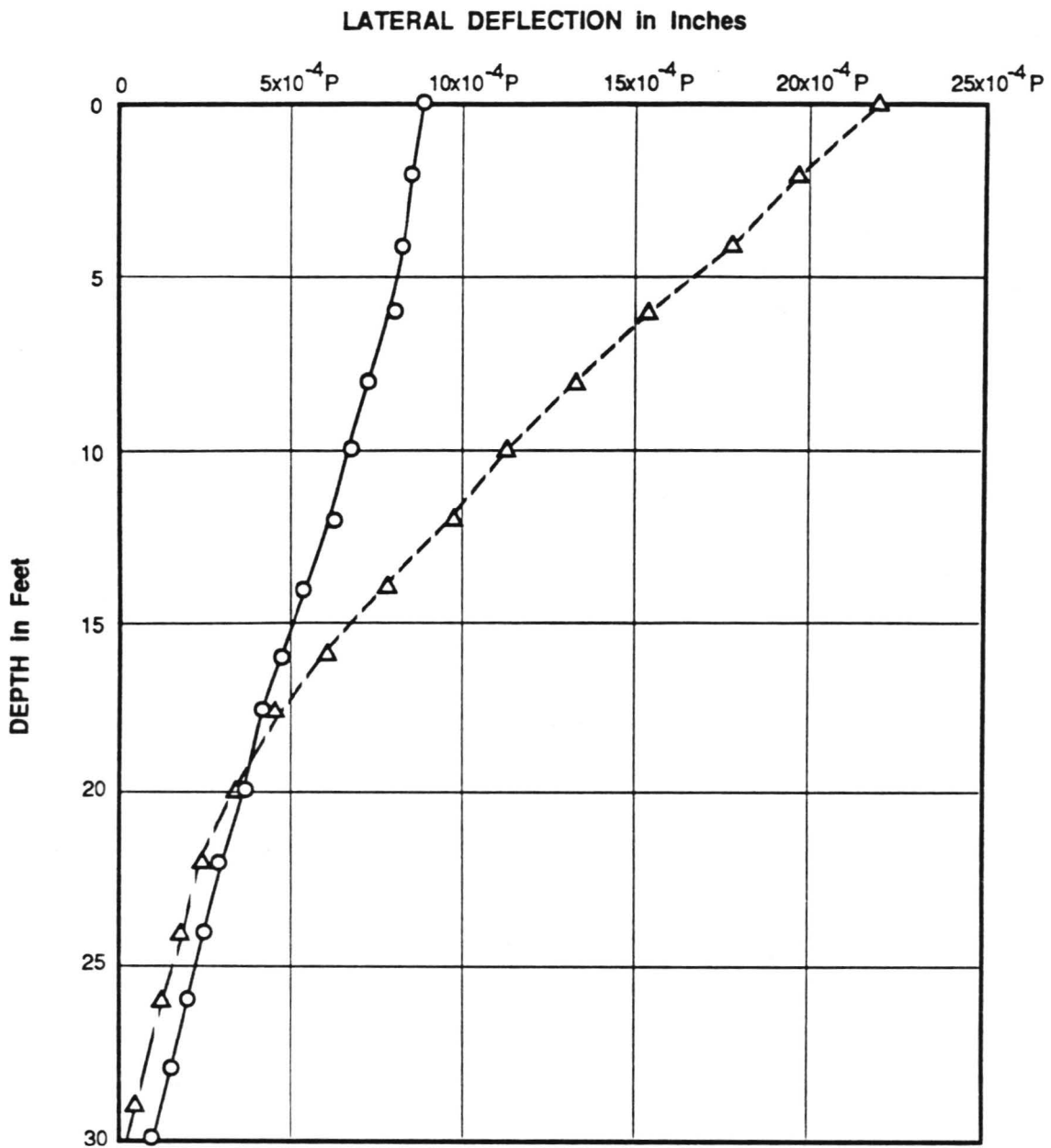
○—○ FIXED HEAD PILE

△--△ FREE HEAD PILE

P LATERAL LOAD APPLIED AT TOP OF THE PILE IN KIPS

LATERAL DEFLECTION DIAGRAMS
60 - INCH DIAMETER PILE
 (Design Group 1)

JOB L92045.AE4 DATE 4/29/93 DR. MS O.E. YK MS CHKD MS



KEY:

- FIXED HEAD PILE
- △--△ FREE HEAD PILE
- P LATERAL LOAD APPLIED AT TOP OF THE PILE IN KIPS

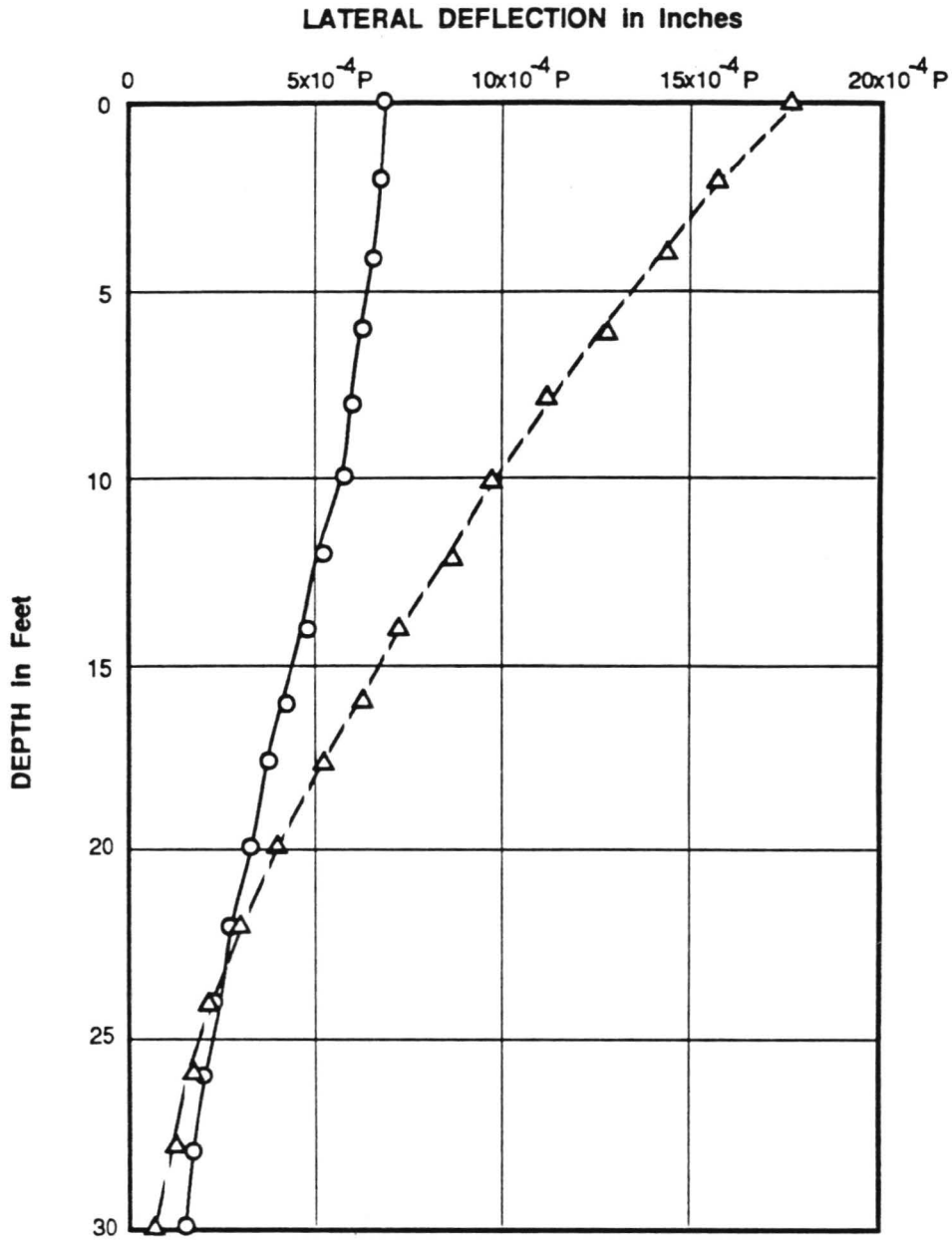
LATERAL DEFLECTION DIAGRAMS
72 - INCH DIAMETER PILE
 (Design Group 1)

LAW / CRANDALL, INC.



FIGURE 8.4H

JOB L92045 AE4 DATE 4/29/93 DR. *MS* O.E. YK *MS* CHKD *MS*



KEY:

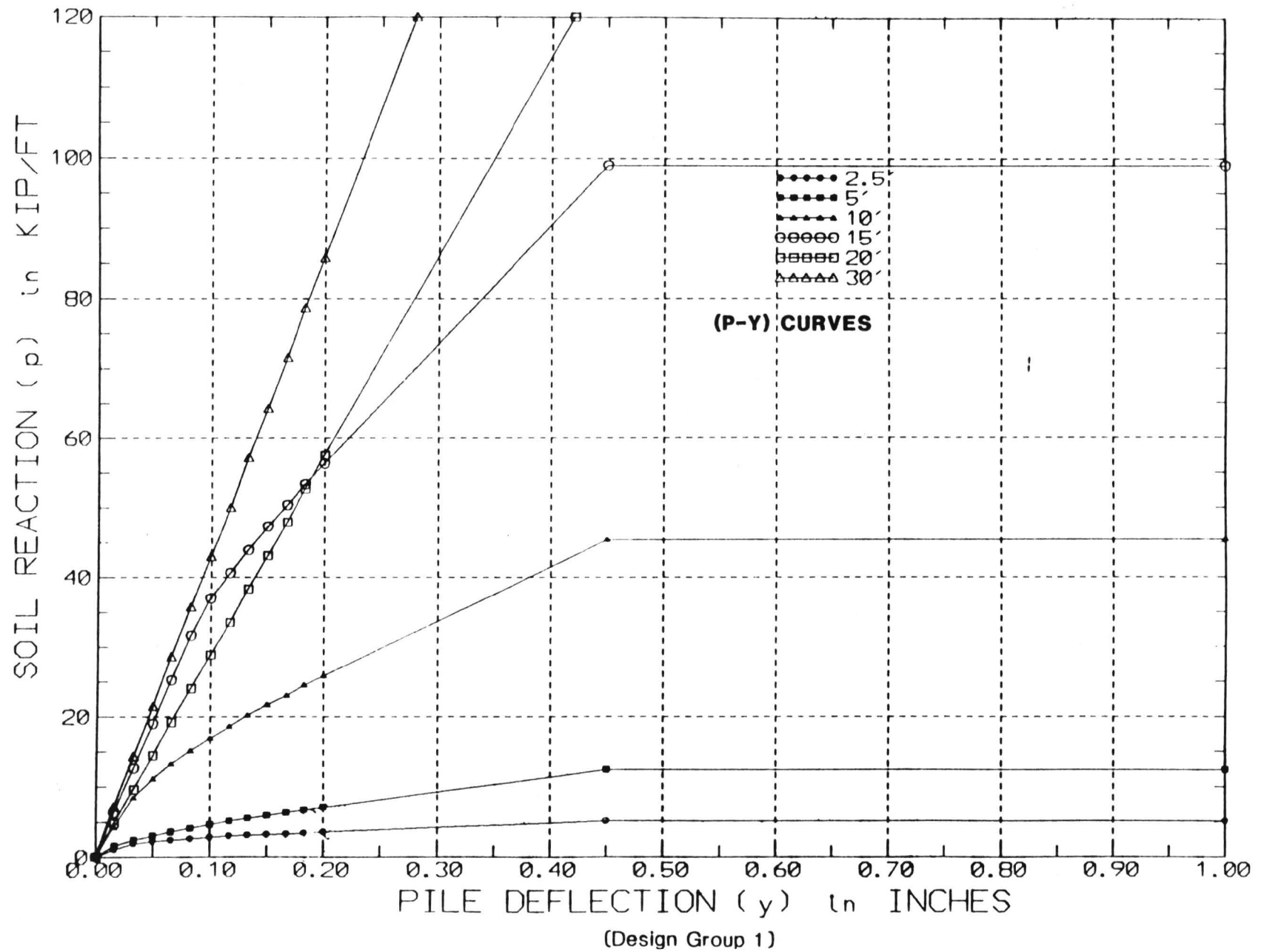
○—○ FIXED HEAD PILE

△--△ FREE HEAD PILE

P LATERAL LOAD APPLIED AT TOP OF THE PILE IN KIPS

LATERAL DEFLECTION DIAGRAMS
84 - INCH DIAMETER PILE
 (Design Group 1)

12 INCH SQUARE PILE

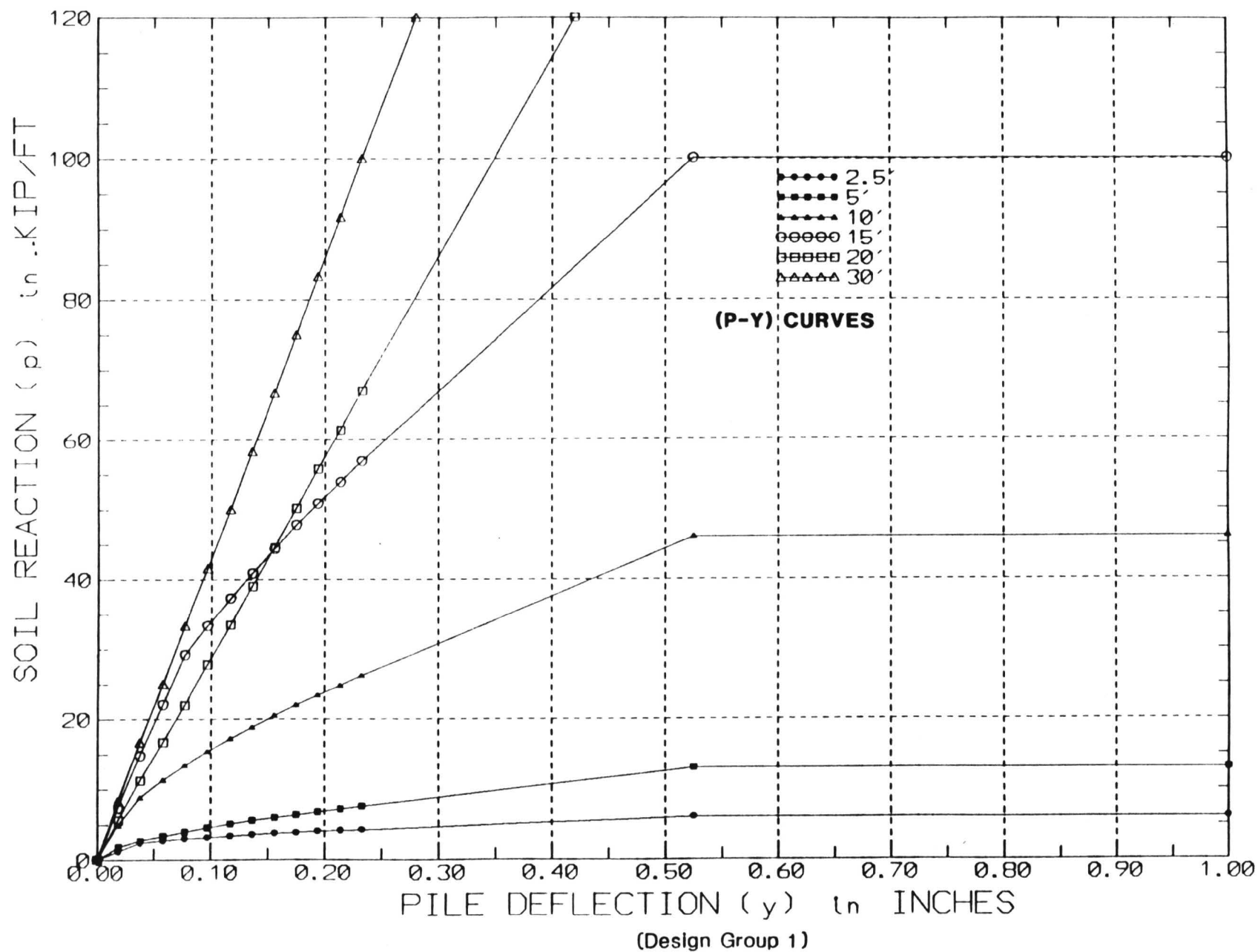


LAW/CRANDALL, INC.



FIGURE 8.5A

14 INCH SQUARE PILE



LAW/CRANDALL, INC.



FIGURE 8.5B

16 INCH SQUARE PILE

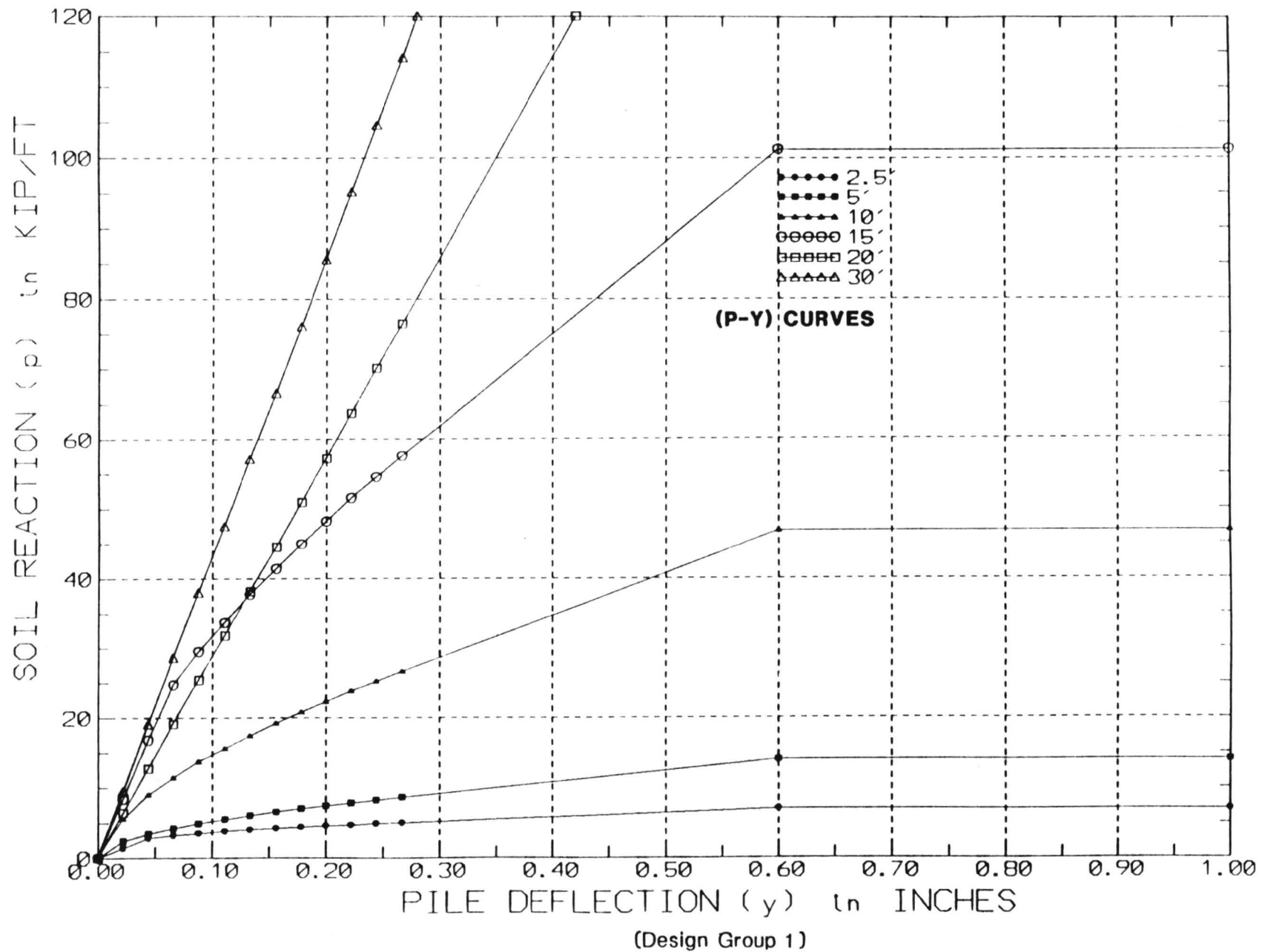
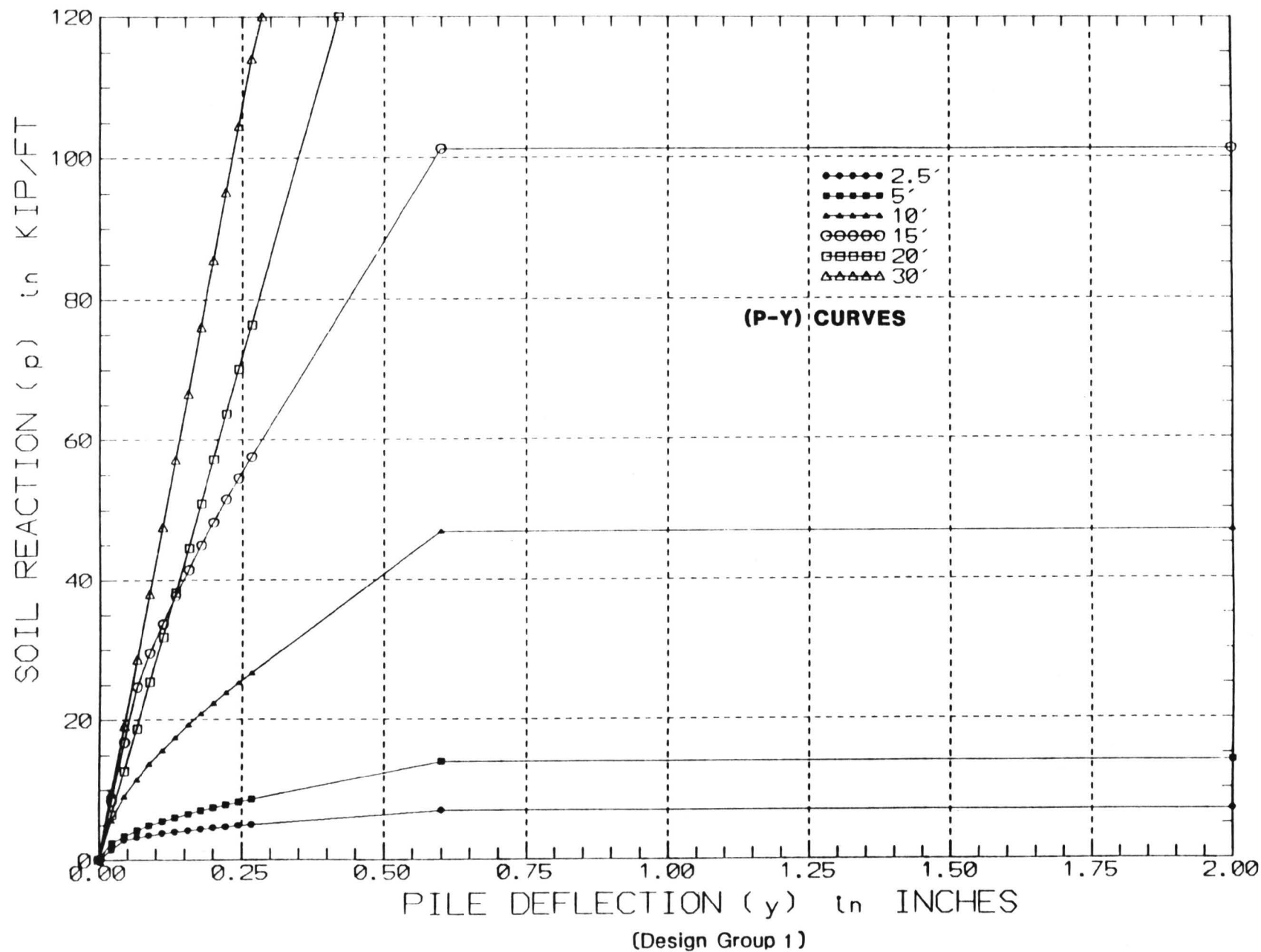


FIGURE 8.5C

LAW/CRANDALL, INC.



16 INCH CIDH PILE

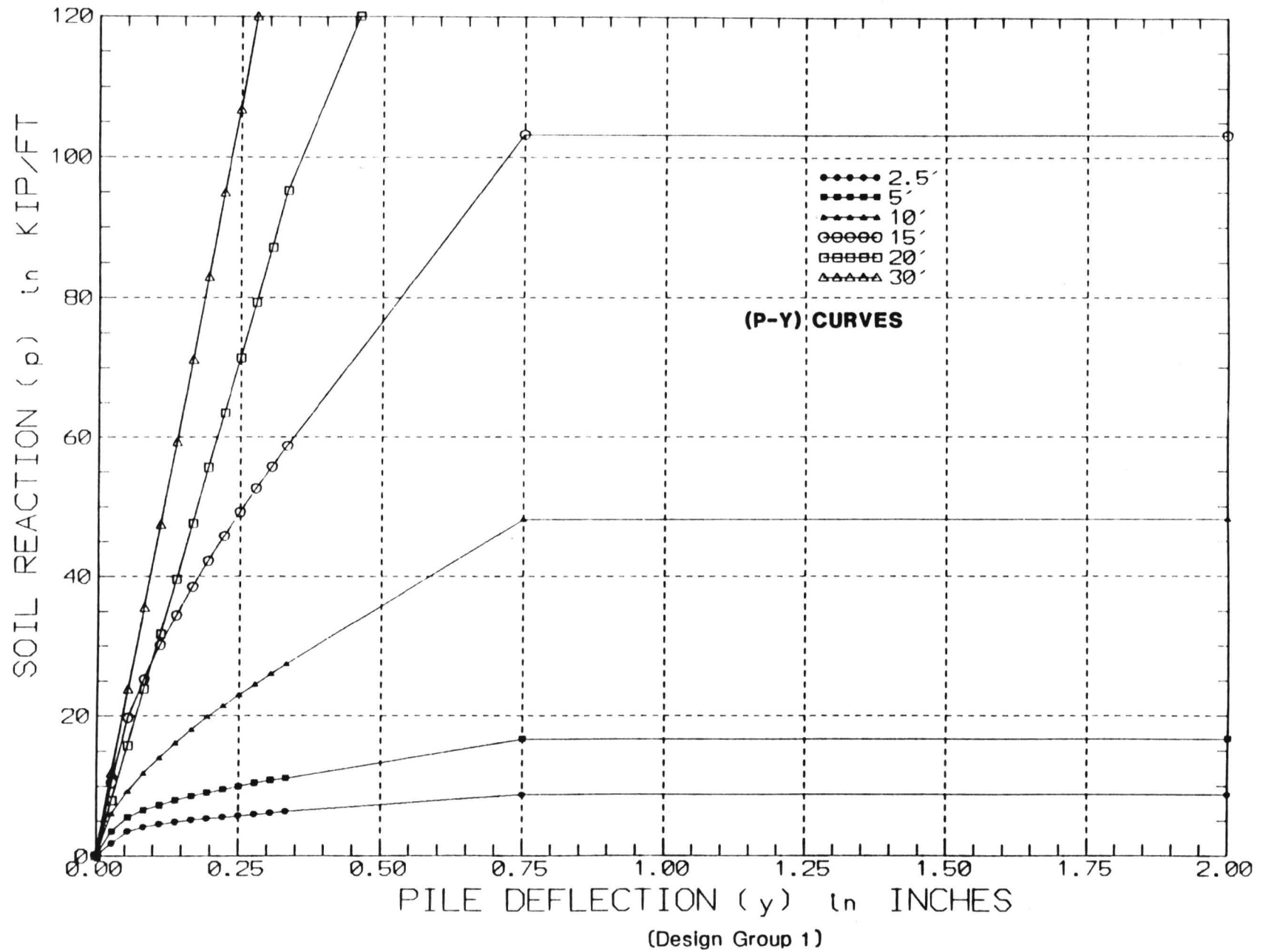


LAW/CRANDALL, INC.



FIGURE 8.5D

20 INCH CIDH PILE

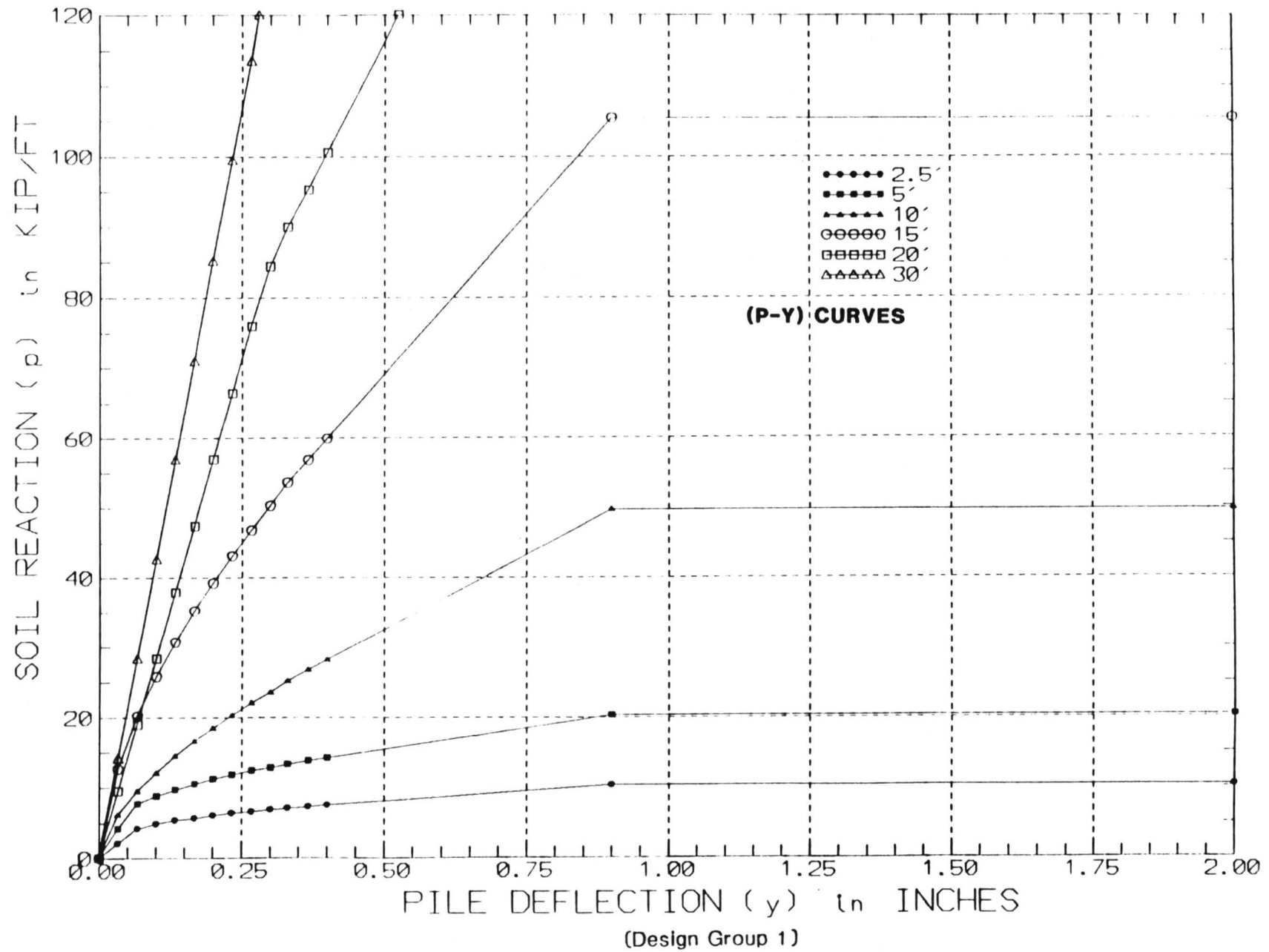


LAW/CRANDALL, INC.



FIGURE 8.5E

24 INCH CIDH PILE

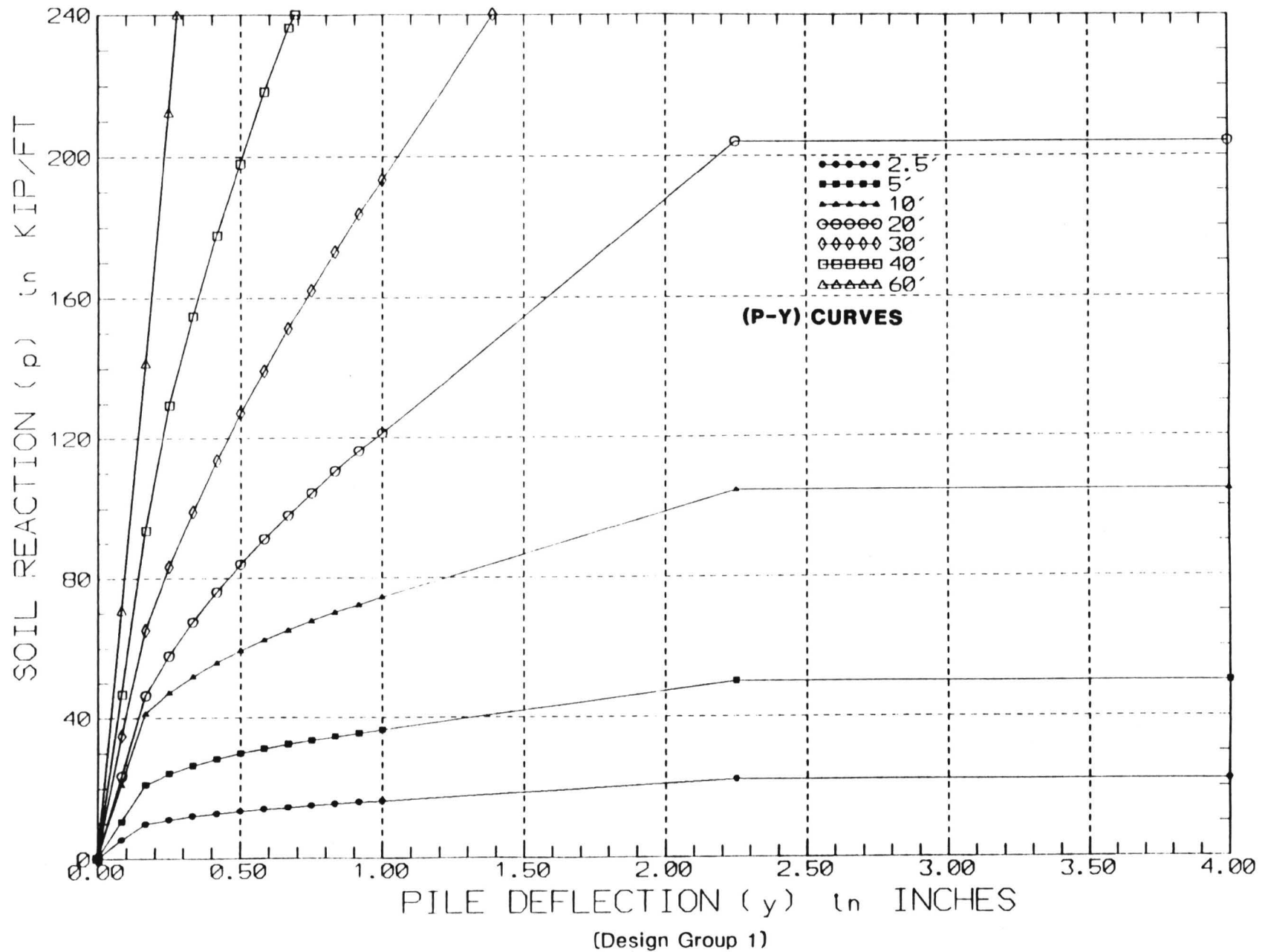


LAW/CRANDALL, INC.



FIGURE 8.5F

60 INCH CIDH PILE

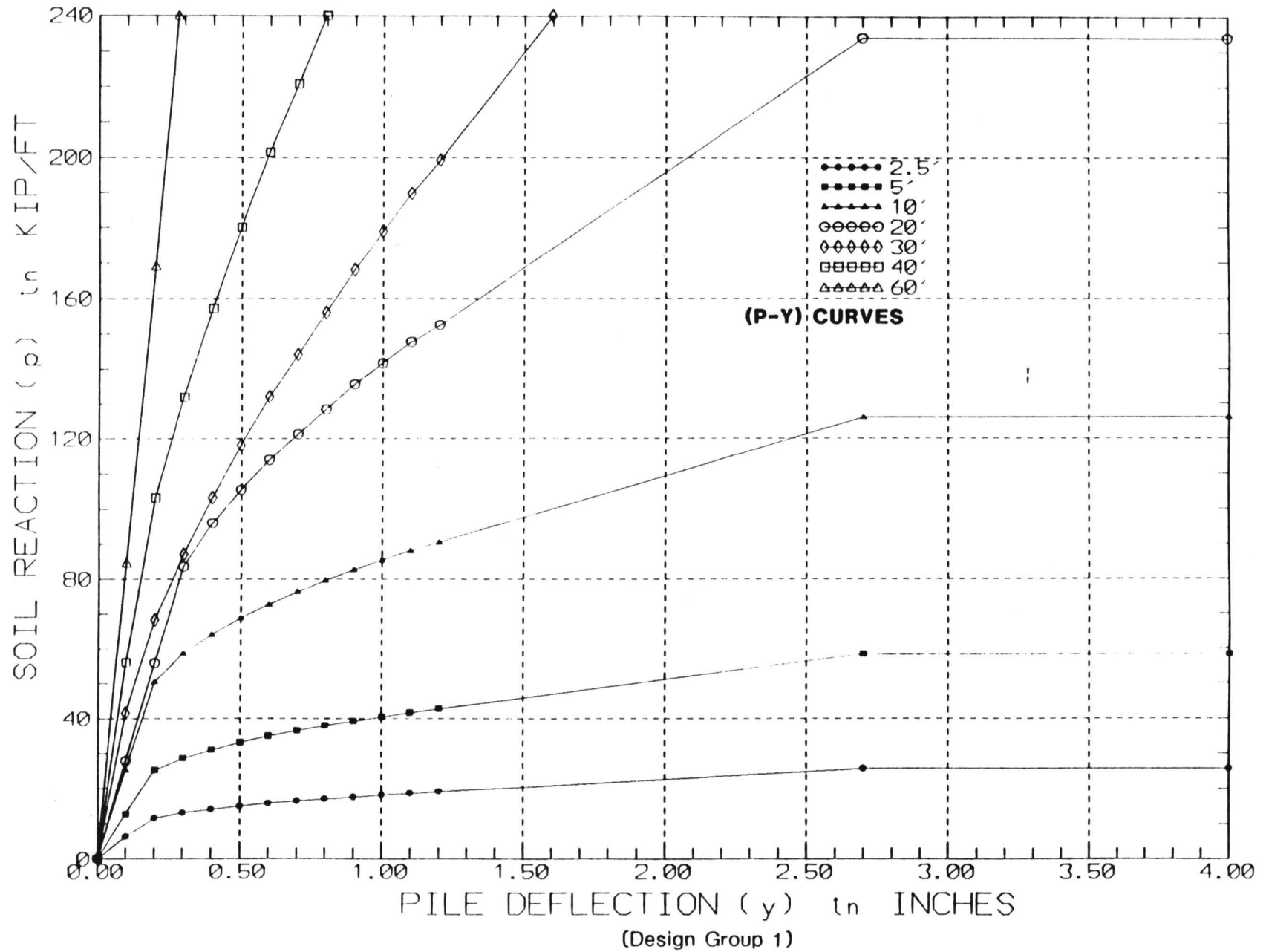


LAW/CRANDALL, INC.



FIGURE 8.5G

72 INCH CIDH PILE

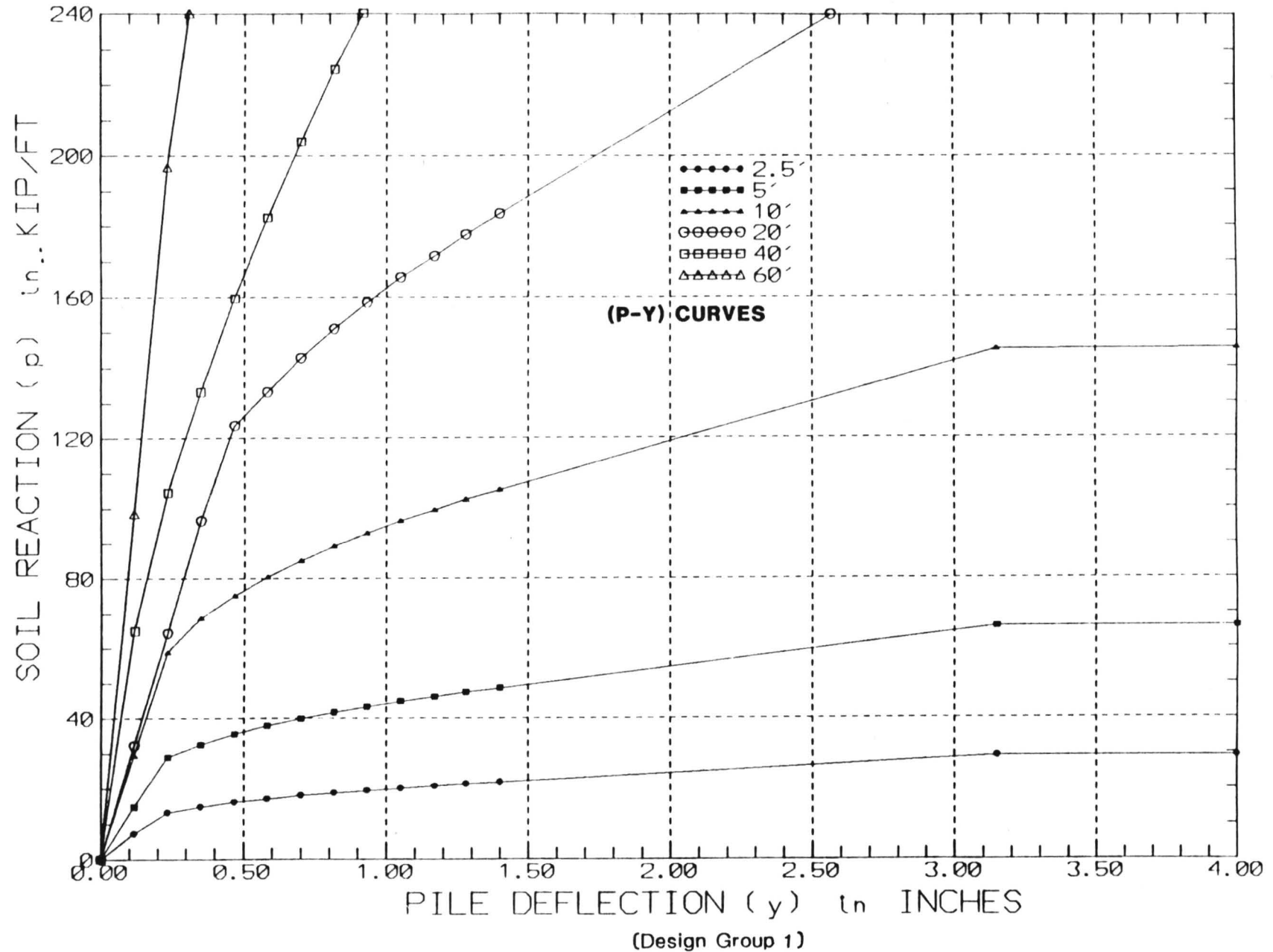


LAW/CRANDALL, INC.

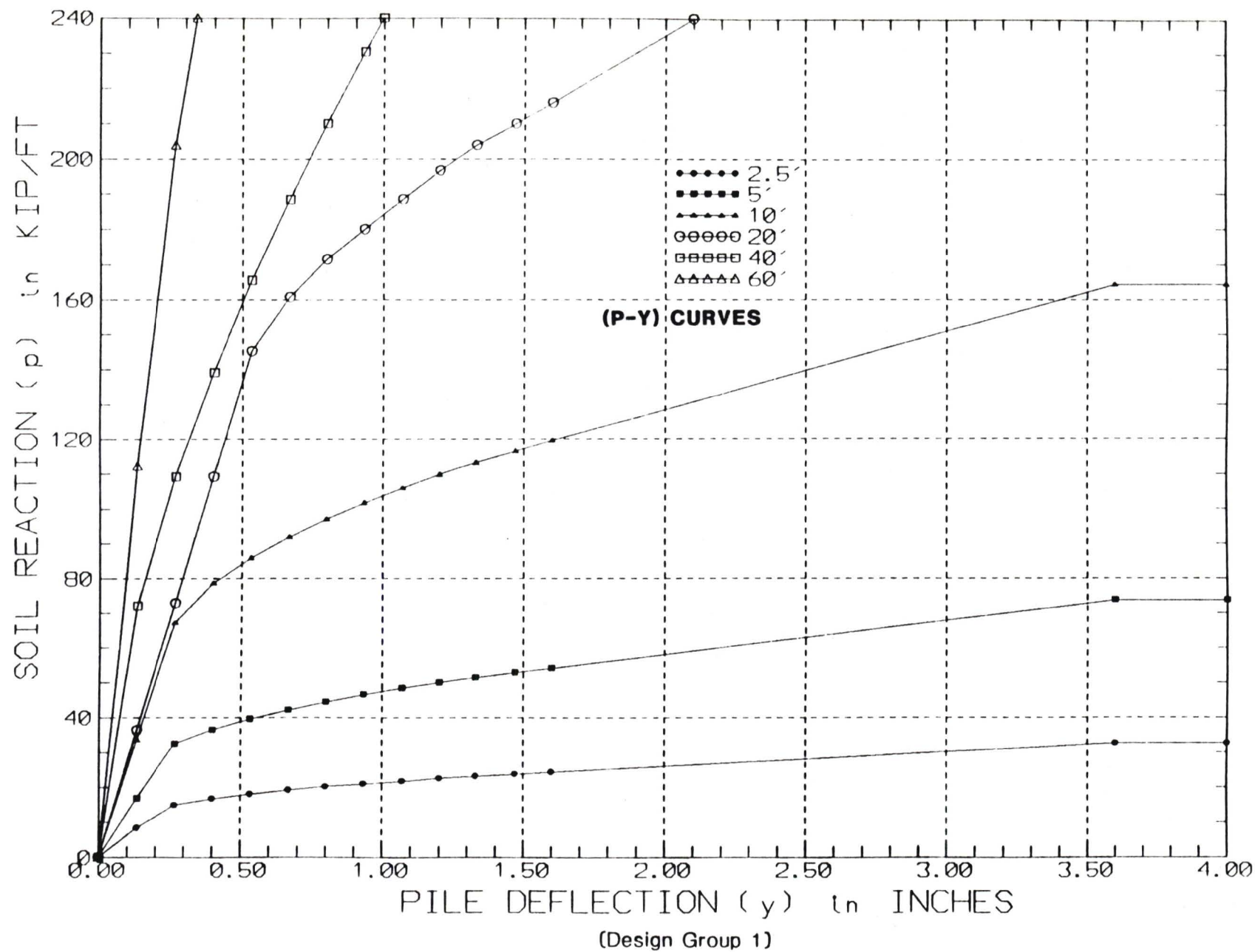


FIGURE 8.5H

84 INCH CIDH PILE



96 INCH CIDH PILE

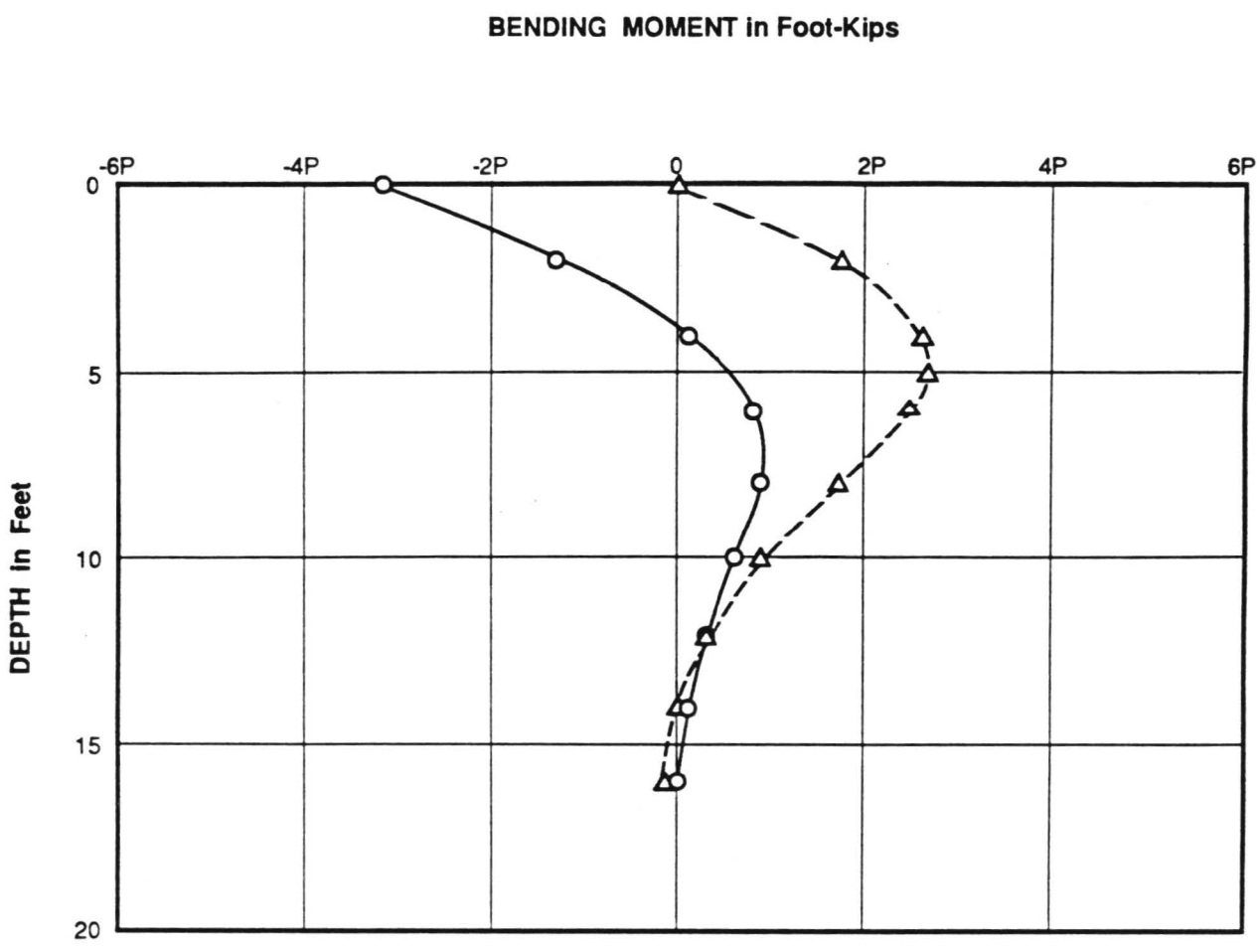


LAW/CRANDALL, INC.



FIGURE 8.5J

JOB L92045, AE4 DATE 2/22/93 DR. IK O.E. MS *MS* CHKD *AB*



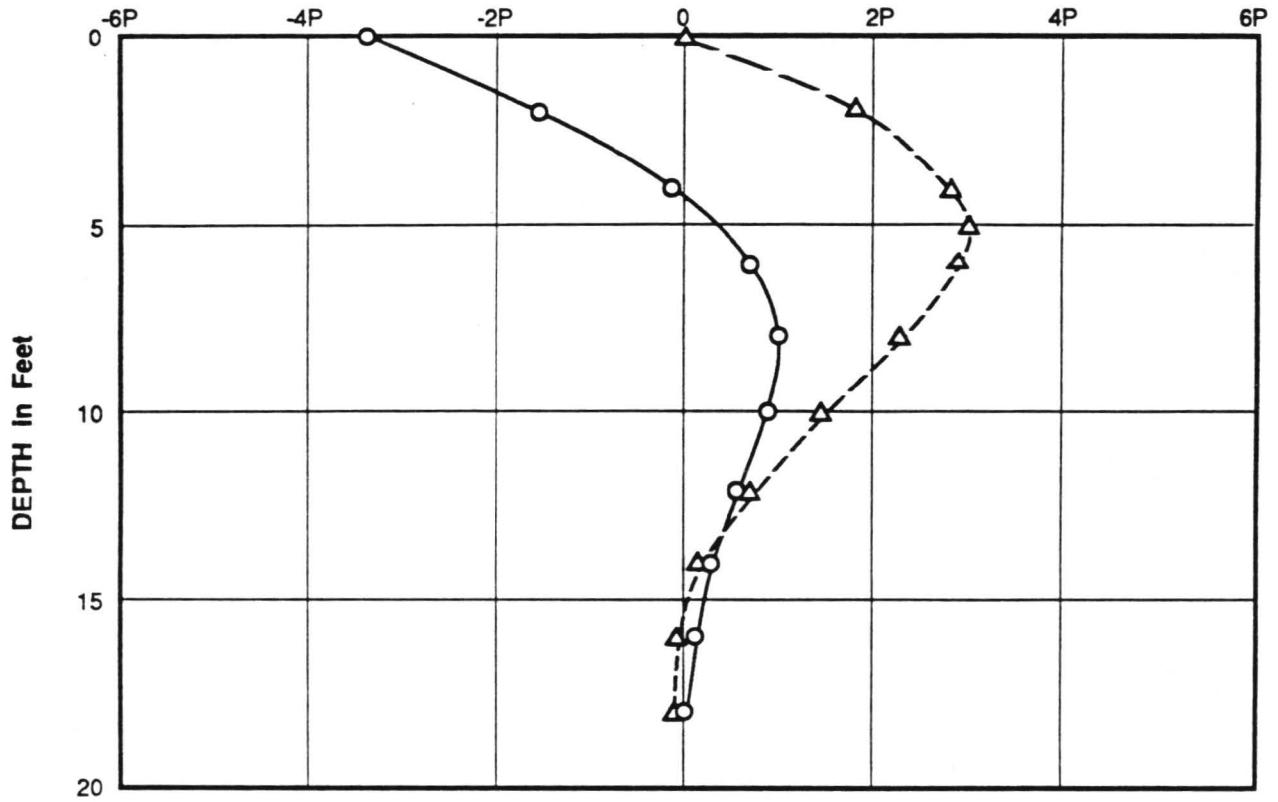
KEY:

○—○ FIXED HEAD PILE
△--△ FREE HEAD PILE
P LATERAL LOAD APPLIED AT TOP OF THE PILE IN KIPS

BENDING MOMENT DIAGRAMS
12 - INCH SQUARE PILE
(Design Group 1)

JOB L92045.AE4 DATE 2/22/93 DR. IK MS CHKD O.E.

BENDING MOMENT in Foot-Kips



KEY:



FIXED HEAD PILE



FREE HEAD PILE

P

LATERAL LOAD APPLIED AT TOP OF THE PILE IN KIPS

BENDING MOMENT DIAGRAMS
14 - INCH SQUARE PILE
(Design Group 1)

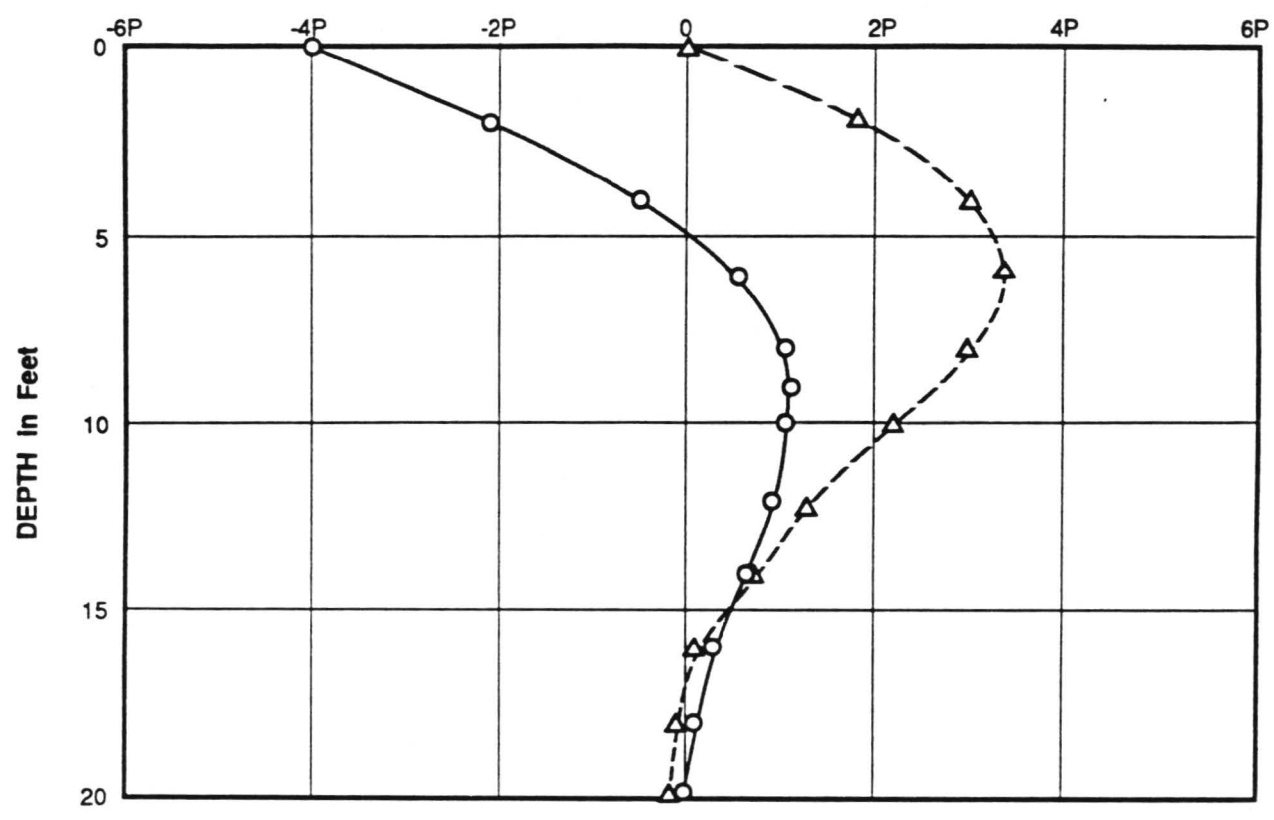
LAW / CRANDALL, INC.



FIGURE 8.6B

JOB L92045.AE4 DATE 2/22/93 DR. Ik MS O.E. MS CHKD

BENDING MOMENT in Foot-Kips

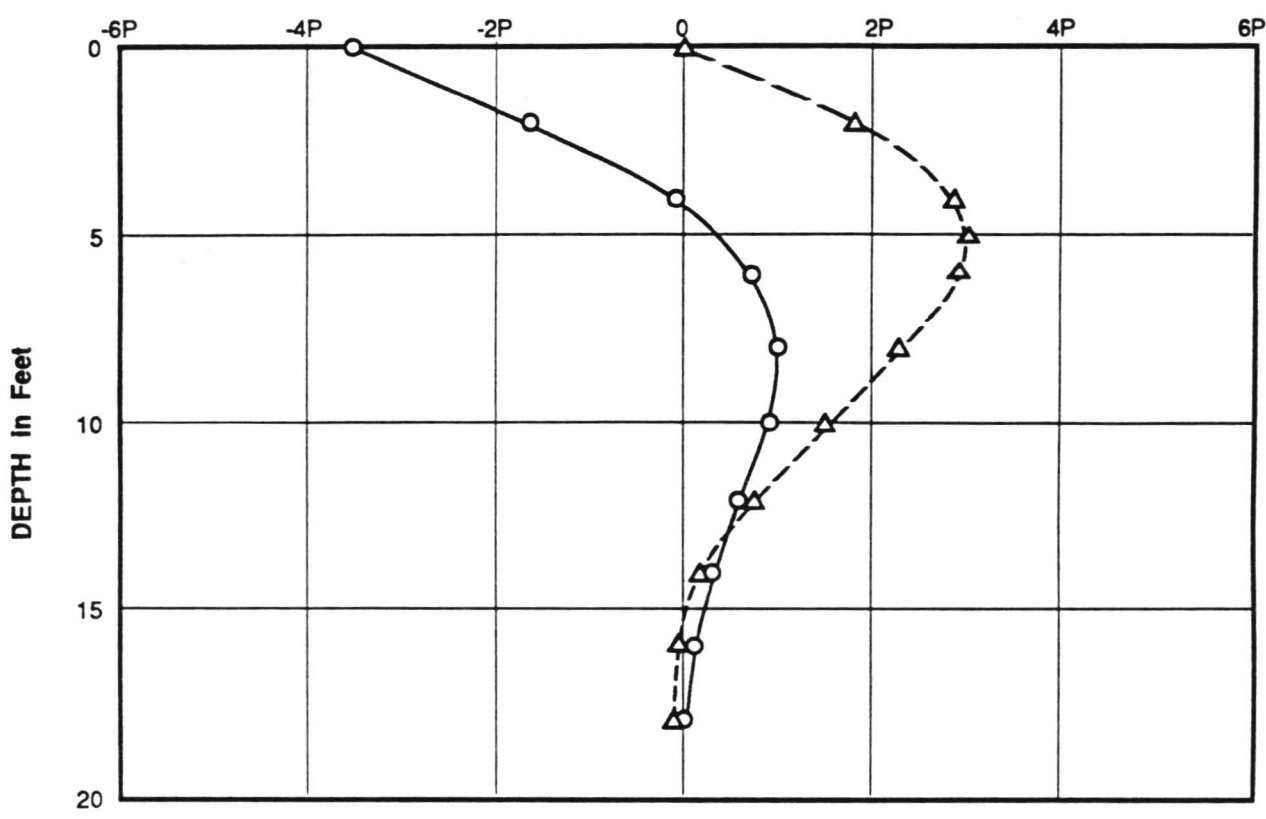


KEY:
 ○—○ FIXED HEAD PILE
 △—△ FREE HEAD PILE
 P LATERAL LOAD APPLIED AT TOP OF THE PILE IN KIPS

BENDING MOMENT DIAGRAMS
 16 - INCH SQUARE PILE
 (Design Group 1)

JOB L92045.AE4 DATE 2/22/93 DR. IK MS CHKD

BENDING MOMENT in Foot-Kips

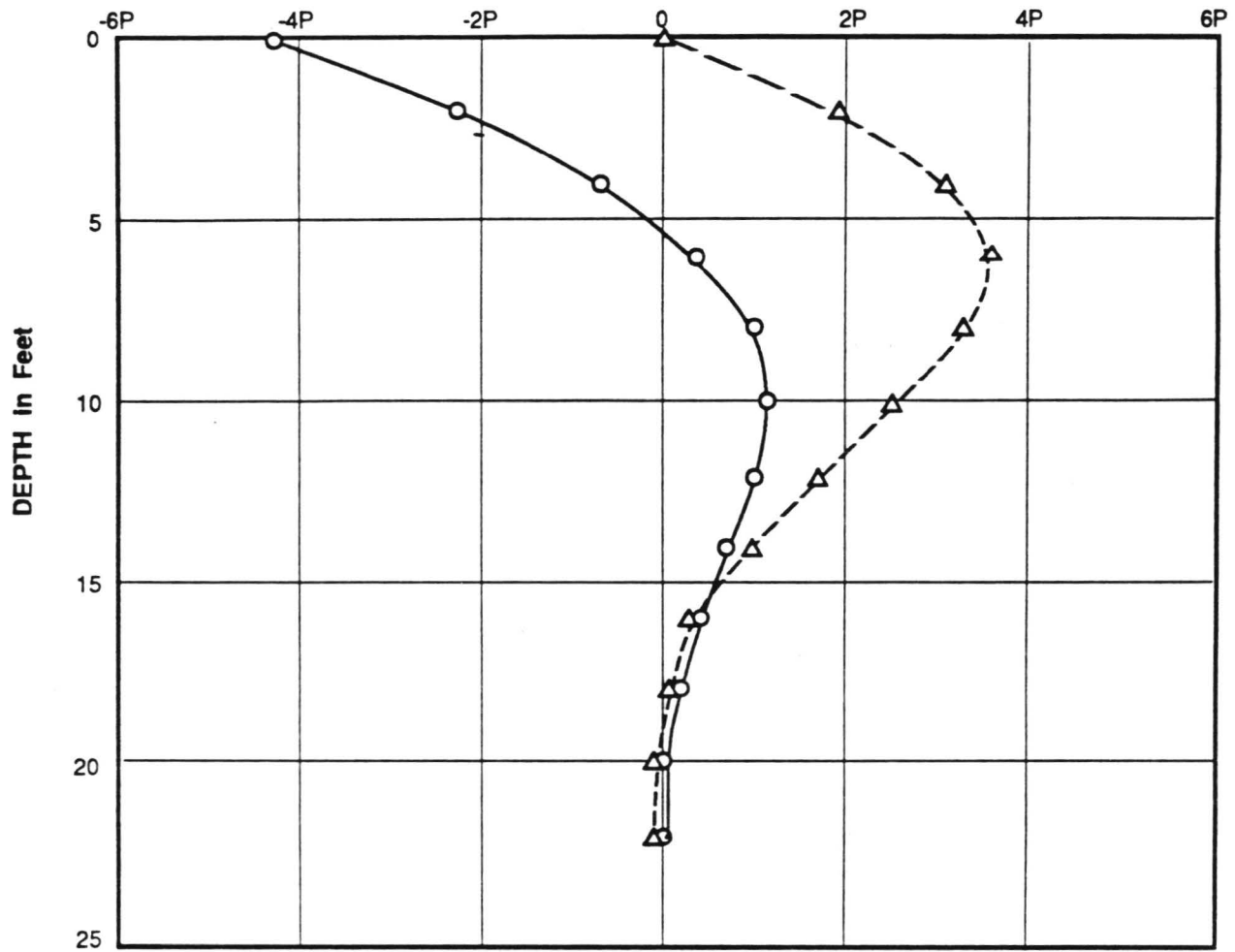


KEY:
 ○—○ FIXED HEAD PILE
 △--△ FREE HEAD PILE
 P LATERAL LOAD APPLIED AT TOP OF THE PILE IN KIPS

BENDING MOMENT DIAGRAMS
16 - INCH DIAMETER PILE
 (Design Group 1)

JOB L92045,AE4 DATE 2/22/93 DR. IK MS CHKD *MS*

BENDING MOMENT in Foot-Kips



KEY:

- FIXED HEAD PILE
- △--△ FREE HEAD PILE
- P LATERAL LOAD APPLIED AT TOP OF THE PILE IN KIPS

BENDING MOMENT DIAGRAMS 20 - INCH DIAMETER PILE

(Design Group 1)

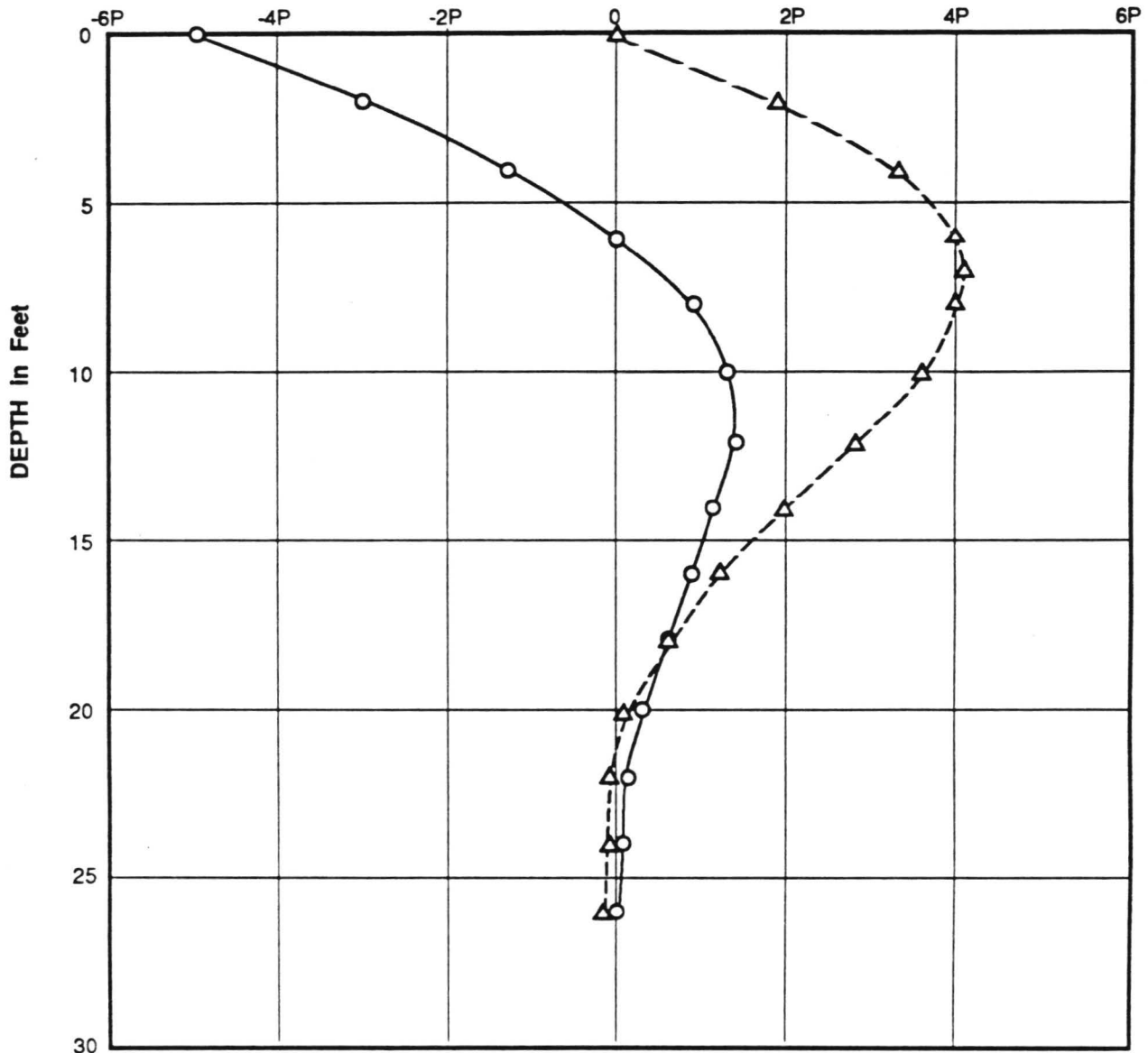
LAW / CRANDALL, INC.



FIGURE 8.6E

JOB L92045.AE4 DATE 2/22/93 DR. JK O.E. MS CHKD *[Signature]*

BENDING MOMENT in Foot-Kips



KEY:

○—○ FIXED HEAD PILE

△--△ FREE HEAD PILE

P LATERAL LOAD APPLIED AT TOP OF THE PILE IN KIPS

BENDING MOMENT DIAGRAMS

24 - INCH DIAMETER PILE

(Design Group 1)

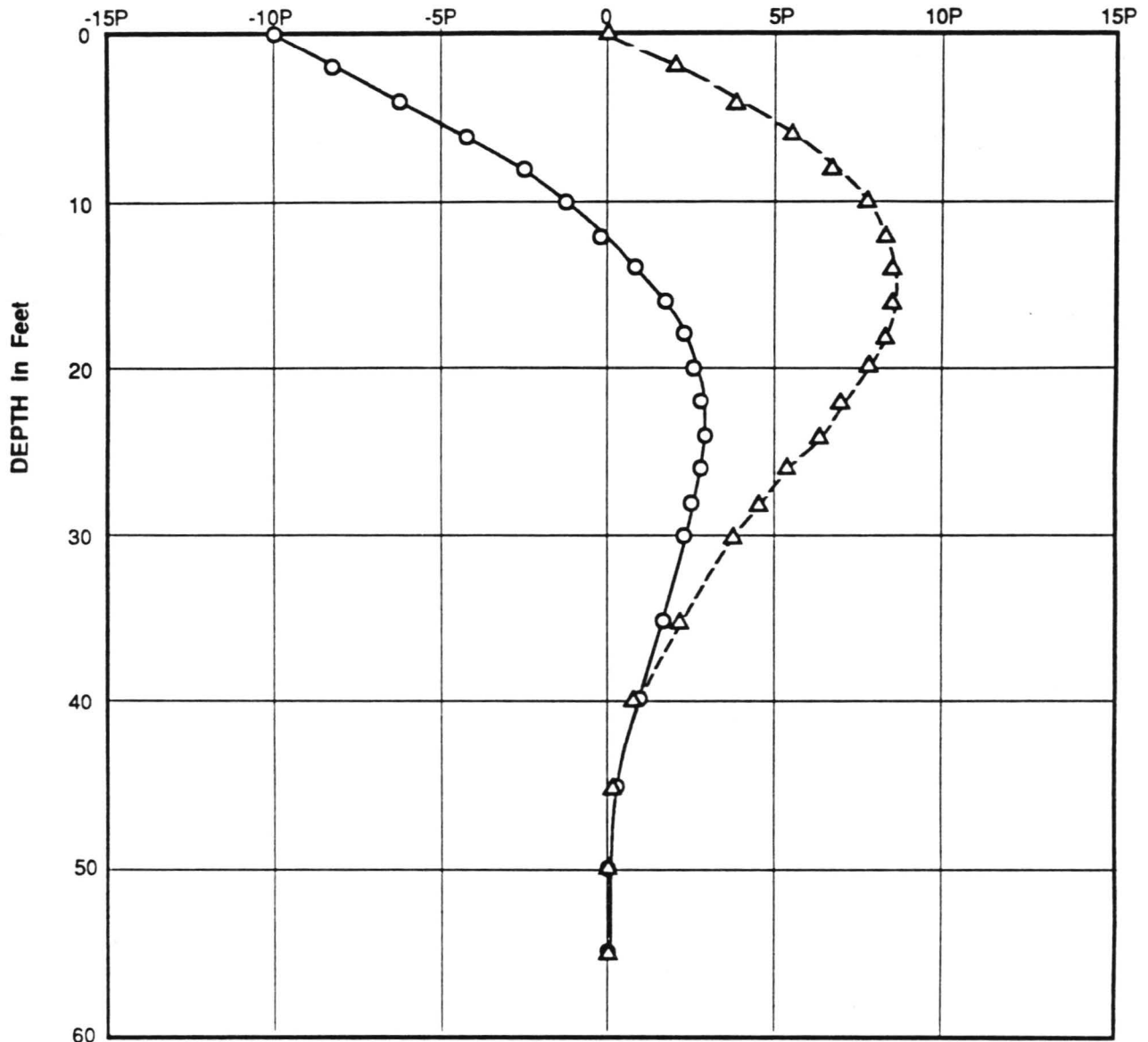
LAW / CRANDALL, INC.



FIGURE 8.6F

JOB L92045.AE4 DATE 2/22/93 DR. IK O.E. MS CHKD

BENDING MOMENT in Foot-Kips



KEY:



FIXED HEAD PILE



FREE HEAD PILE

P

LATERAL LOAD APPLIED AT TOP OF THE PILE IN KIPS

BENDING MOMENT DIAGRAMS

60 - INCH DIAMETER PILE

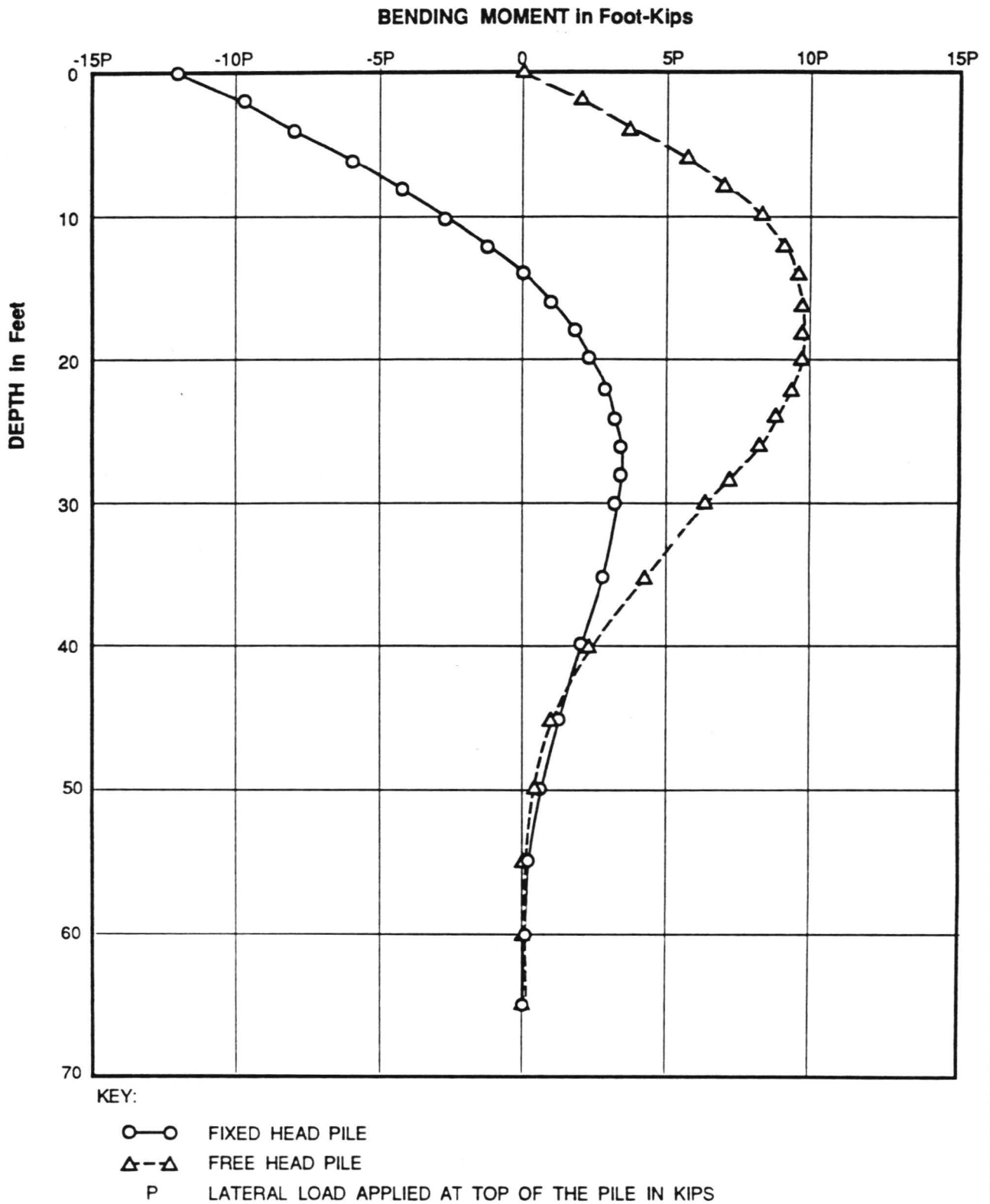
(Design Group 1)

LAW / CRANDALL, INC.



FIGURE 8.6G

JOB L92045, AE4 DATE 2/22/93 DR. IK O.E. MS CHKD *MS*



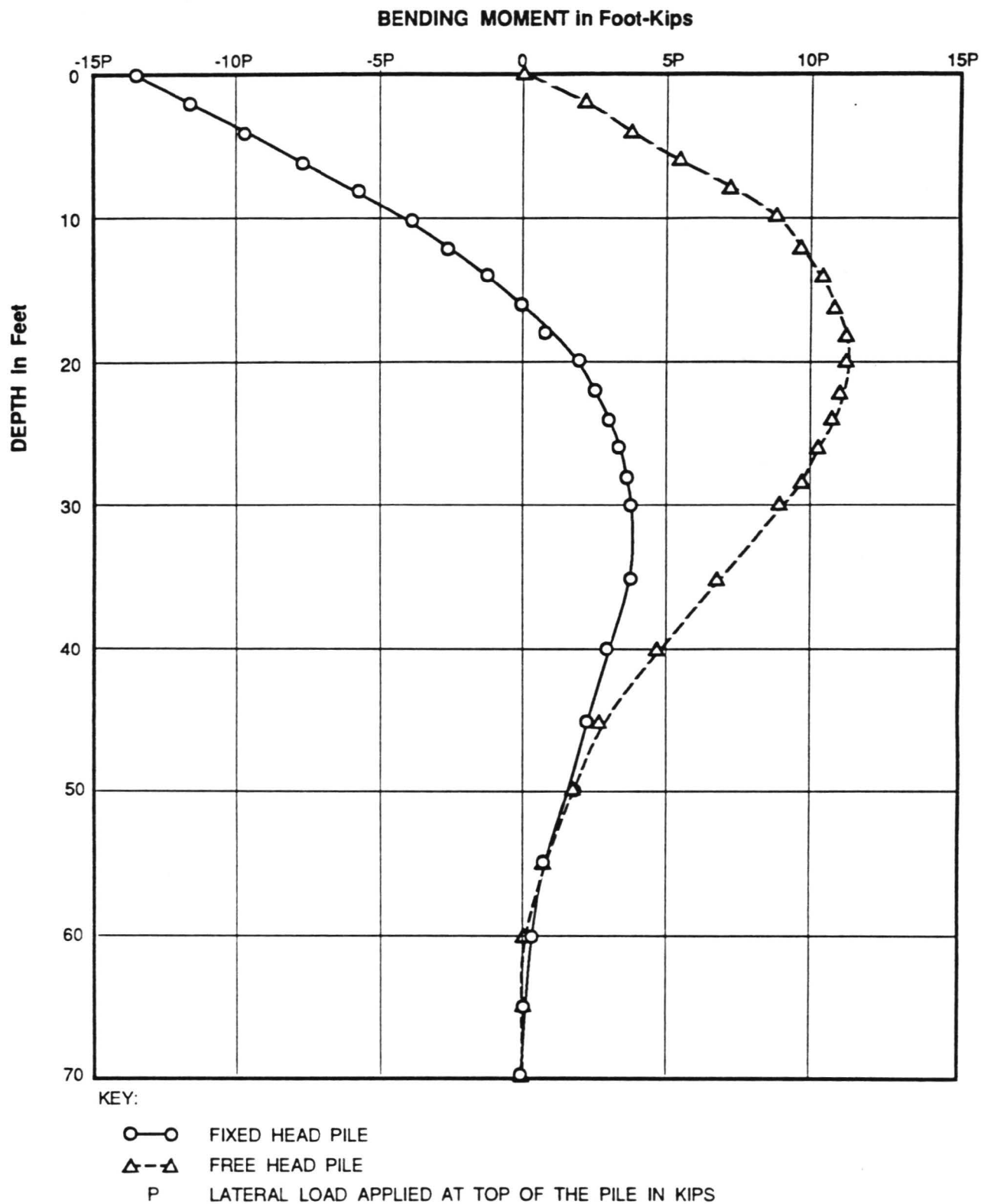
BENDING MOMENT DIAGRAMS
72 - INCH DIAMETER PILE
 (Design Group 1)

LAW / CRANDALL, INC.



FIGURE 8.6H

JOB L92045.AE4 DATE 2/22/93 DR. IK MS CHKD *h* O.E.



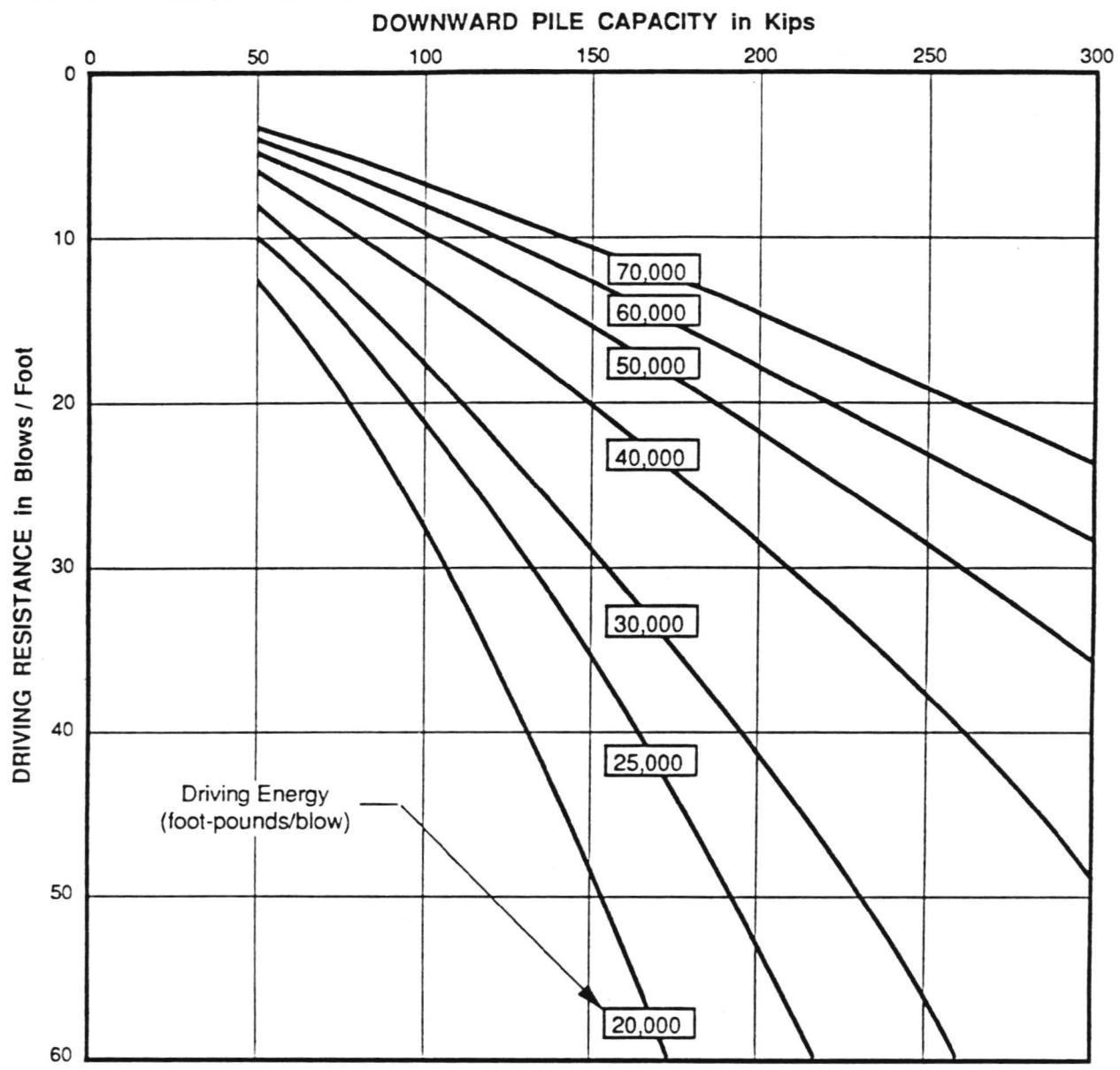
BENDING MOMENT DIAGRAMS
84 - INCH DIAMETER PILE
 (Design Group 1)

LAW / CRANDALL, INC.



FIGURE 8.61

JOB L92045.AE4 DATE 2/23/93 DR. CHKD MS O.E. 1/15



NOTES :

- (1) The above driving resistance should be obtained for the last foot of driving at design pile length. If the driving resistance at the design length is less than above, the piles should be lengthened until the desired driving resistance is obtained.
- (2) As an alternate to lengthening when low driving resistance is obtained, the piles may be allowed to set overnight and the number of blows to drive the pile one inch the following day determined. If the restarting resistance is at least two times the above criteria, the pile may be considered satisfactory.
- (3) If driving resistance of three times the above criteria is encountered within five feet of design length, the pile driving may be stopped.

PILE DRIVING CRITERIA

(Design Group 1)

FIGURE 8.7

8.3 DESIGN GROUP 2 (*Los Angeles River Bridge; Arroyo Seco Park and Pasadena Freeway Bridge*)

8.3.1 SUBSURFACE PROFILE

Borings 10, 11, 45, 46, 47, 69, and 70 were drilled for the aerial structures in this design group. Logs of the borings are presented in Appendix A, Volume 2. Based on the borings, the overburden soils basically consist of silt, silty sand, sand, and gravel with cobbles and occasional boulders. Bedrock was encountered at depths ranging from 7 to 25 feet below the existing ground. Ground water was measured at depths ranging from 15 to 45 feet below grade.

The following soil properties may be used for static design for Design Group 2:

Cohesion 0 to 30 feet below 30 feet	250 psf 1100 psf
Void Ratio Granular Soils Cohesive Soils Bedrock Materials	0.45 0.60 0.55
Modulus of Subgrade Reaction Granular Soils Cohesive Soils and Bedrock Materials	150 pci 200 pci
Angle of Internal Friction 0 to 30 feet below 30 feet	38 degrees 37 degrees
Moist Unit Weight of Soil 0 to 20 feet Below 20 feet	120 pcf 60 pcf



8.3.2 FOUNDATIONS

8.3.2.1 Pile Foundations

Vertical Loads - Drilled Piling: The downward and upward frictional capacities of 16-, 20-, 24-, 60-, 72-, and 84-inch-diameter drilled piles are presented on Figures 8.8 and 8.9, Drilled Pile Capacities, Design Group 2. The capacities of other sizes of piles may be taken as proportional to the diameter. Dead plus live load capacities are shown; a one-third increase may be used for wind or seismic loads. The dead plus live capacities are allowable capacities; a factor of safety of 2 was used to determine the allowable capacities. If piles in groups are required, the piles should be spaced not less than $2\frac{1}{2}$ diameters on centers so that group action need not be considered. The capacities are based on the strength of the soils; the compressive and tensile strength of the pile section itself should be checked to verify the structural capacity of the pile.

Settlement - Drilled Piles: The settlement of drilled piles will be less than $\frac{1}{2}$ inch for vertical loads between 150 and 3,500 kips. For example, the settlement of a 16-inch-diameter pile supporting a vertical load of 150 kips will be less than $\frac{1}{4}$ inch, and the settlement of an 84-inch-diameter pile supporting a vertical load of 3,500 kips will be less than $\frac{1}{2}$ inch.

Vertical Loads - Driven Piles: The downward and upward frictional capacities of 12-, 14-, and 16-inch-square prestressed concrete piles are presented on Figure 8.10, Driven Pile Capacities, Design Group 2. If batter piles are used, only the vertical projection of the length of the batter pile should be considered in computing the pile capacities. Dead plus live load capacities are shown; a one-third increase may be used for wind or seismic loads. The dead plus live capacities are allowable capacities computed with a factor of safety of at least 2. Piles in groups should be spaced not less than $2\frac{1}{2}$ pile widths or 3 feet on centers, whichever is greater, so that group action need not be considered. The capacities are based on the strength of the soils; the compressive and tensile strength of the pile section itself should be checked to verify the structural capacity of the pile.



Settlement – Driven Piles: The settlement of 12-, 14-, and 16-inch-square prestressed concrete piles, supporting vertical loads up to 300 kips, will range from $\frac{1}{4}$ to $\frac{1}{2}$ inch.

Lateral Resistance: The piles may be used to resist lateral loads. The lateral capacity of the piles will depend on the permissible deflection and on the degree of fixity at the top of the pile. The lateral deflection in the piles (in terms of applied load at the top of the pile) is presented on Figures 8.11A through A.11I, Lateral Deflection Diagrams, Design Group 2. The lateral capacity of a batter pile is equivalent to the horizontal component of the downward pile capacity.

The soil resistance versus pile deflection for the piles is shown on Figures 8.12A through 8.12J, (P-Y) Curves, Design Group 2. These curves were developed using the computer program LPILE distributed by Ensoft, Inc., Austin, Texas. These (P-Y) curves should only be used in LPILE or similar computer program to determine the behavior of a laterally loaded pile.

The bending moment in the piles due to application of the lateral loads may be distributed in the piles in accordance with Figures 8.13A through 8.13I, Bending Moment Diagrams, Design Group 2.

The lateral capacity and bending moment distributions are based in part on the assumption that any backfill adjacent to the piles will be properly compacted. Furthermore, if the pile locations are predrilled, the backfill in the annular space around the pile should be tight to assure good distribution of load into the surrounding soils. A pile in a pile group of two or more with a pile cap may be considered as a fixed-head pile.

The passive resistance of properly compacted backfill against pile caps may be assumed to be equal to the pressure developed by a fluid with a density of 300 pounds per cubic foot. The passive resistance of the undisturbed bedrock against pile caps may be assumed to be equal to 1,000 pounds per square foot at the bedrock surface, increasing by 300 pounds per square



foot per foot of depth. A one-third increase in the quoted passive values may be used when considering wind or seismic loads.

Ultimate Design: The various design values recommended for piling are for use with loadings determined by a conventional working stress design. If an ultimate design concept is used, the recommended design values may be multiplied by the following factors:

Design Item	Ultimate Design Factor
Axial Capacity of Piles	2.0
Lateral Capacity of Piles	1.5
Passive Resistance	1.25

In no event, however, should the pile lengths be reduced from those required for support of dead plus live loads when using the working stress values.

Installation - Drilled Piling: As previously mentioned, the alignment is underlain by geologic units consisting of artificial fill, alluvial, and terrace deposits, underlain by sedimentary bedrock of the Puente and Topanga formations. There are significant amounts of sand, gravel, and cobbles within these geologic formations. Caving of boring walls occurred in some of the borings drilled with bucket auger-type drilling equipment. A significant percentage of prior borings drilled by our firm for other projects in the vicinity with bucket auger-type drilling equipment also experienced significant caving.

The bucket auger exploration borings were drilled typically with relatively small-diameter equipment, ranging from 16 to 24 inches in diameter. It should be noted that the caving potential increases as the diameter of the excavation increases. The sandy soils are relatively cohesionless, and may be held together by small capillary tension forces. If allowed to dry, the soils would lose that capillary tension and would be more likely to cave. The use of high speed



drilling equipment would also cause disturbance of the soils, increasing the likelihood of caving. The presence of ground water would also increase the likelihood of caving.

It should be anticipated that significant caving, raveling, and sloughing may occur during excavation of the large-diameter drilled piles in the alluvial soils particularly below the water, unless positive steps are taken to support the sides of the hole. Special techniques will likely be needed to satisfactorily install the piles. Casing may or may not be required, depending on the field conditions. Coring may be required in the bedrock as hard, cemented layers may be encountered.

Only drilling contractors with demonstrated experience in the installation of drilled cast-in-place large-diameter piles using drilling mud and casing in similar circumstances should be considered for the pile work. The drilling system should be equipped to include the use of casing and drilling mud to prevent shaft caving during drilling and placing of concrete. The equipment should provide for the introduction of drilling mud of sufficient specific gravity to retain the shaft sides, and continuous pumping for retrieval of the mud concurrent with concrete placement. The pressure head of mud in the shaft should be sufficient to maintain a stable bottom during all construction operations. Only environmentally safe drilling mud should be used.

The contractor should provide the equipment necessary to check the bearing, dimensions, and alignment of each drilled shaft.

Drilling should be performed in a sequence such that no adjacent pile closer than 5 pile diameters on center is drilled sooner than eight hours after placement of the concrete.

Care should be exercised during placement of the reinforcement steel to minimize disturbance and caving of the sides of any uncased portions of the drilled shaft.



Concrete should be pumped through a tremie pipe extending to the bottom of the shaft, with the pipe being at least 8 feet below the surface of the concrete at all times during placement. The discharge end of the tremie should be constructed so that it is watertight against intrusion from the outside and so that it will readily discharge concrete. The tremie pipe should be supported so that it can be raised and lowered rapidly should it become necessary to cut off or retard the flow. Casing should be pulled as the concrete is placed; the bottom of the casing, however, should be maintained at least 3 feet below the surface of the concrete at all times. The discharge pipe should be kept full of concrete during the entire placing operation and should not be removed from the concrete until all of the concrete is placed and fresh concrete appears at the top of the pile. The volume of concrete pumped into the hole should be recorded and compared to the design volume.

We recommend that continuous observation of the drilling and pouring of the piles be performed by a geotechnical engineer or an engineering geologist licensed in the State of California to verify that the desired diameter and depth are obtained.

Each pile should be tested by a nondestructive testing method to check the pile's integrity. The testing should be performed after at least three days of curing time has elapsed to allow the concrete to harden sufficiently. The nondestructive testing should be undertaken by an experienced and approved independent testing organization.

One of the nondestructive methods that has been successfully used for large diameter piles is called crosshole sonic logging (CSL). This method measures the time it takes for an ultrasonic pulse to travel from a signal source in one access tube placed within a pile to a receiver in another access tube placed within the same pile. In unreinforced quality concrete, the travel time between the two tubes will be relatively constant, and the CSL test will produce records with good signal amplitude. Longer travel times and lower amplitude/energy signals indicate the presence of irregularities such as poor quality concrete, voids, and soil intrusions.



The signal will be completely lost by the receiver for more severe defects such as large voids and soil intrusions.

Steel tubes are recommended because there will be good bonding between steel and concrete, allowing for better transmission of the ultrasonic pulses. Plastic or PVC pipes would be more economical; however, our experience with PVC pipes indicates that certain amounts of debonding occur between the PVC pipe and the concrete after the concrete sets; this is especially the case near the top of the piles. The debonding could lead to erroneous, misleading, or inconclusive readings, thus requiring further testing that may offset any savings in the cost of pipe material. To minimize the debonding, if PVC pipe used, the tubes should be filled with water as soon as possible after the concrete is poured, preferably on the same day.

Installation - Driven Piling: All piles should be driven to the predetermined design lengths as shown on Figure 8.10, except as may be modified on the basis of the driving criteria defined on Figure 8.14, Pile Driving Criteria. The pile driving criteria shown on Figure 8.14 may be taken only as a guide. Prior to ordering production piles, we recommend that indicator piles be driven to evaluate the driving resistance. At least three indicator piles should be driven at each abutment and/or bridge pier location. The driving criteria and installation procedures can be modified based on the results of the indicator piles and any necessary adjustments can be made to the design lengths presented on Figure 8.10. The installation of the piling should be observed by a geotechnical engineer licensed in the State of California so that modifications in the driving criteria and the pile lengths can be made if required.

Hard driving will be encountered within the sandy soils and bedrock and predrilling may be required. The diameter of the predrilling hole should not exceed 80% of the pile diameter. Predrilling may be extended to within 5 feet of the design tip elevation of the pile.

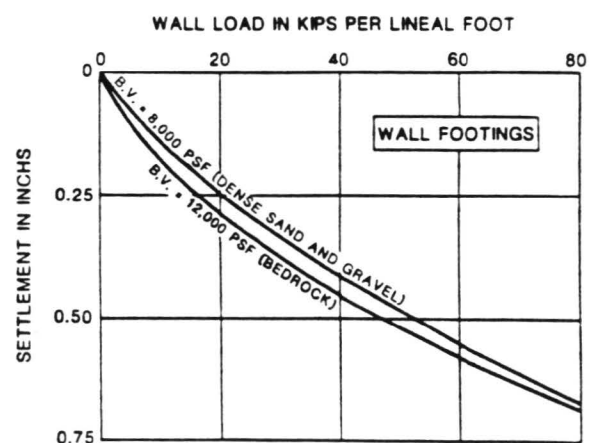
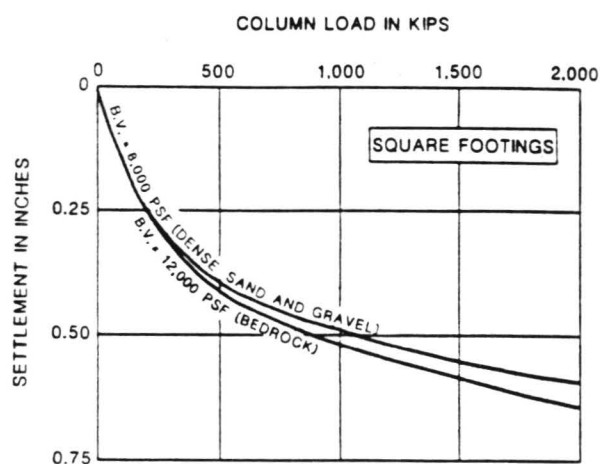


8.3.2.2 Spread Footings

Bearing Value: Spread footings within Design Group 2 may impose bearing pressures up to 8,000 pounds per square foot in the dense sand and gravel, and 12,000 pounds per square foot in the bedrock.

Footings should extend at least 1 foot into the supporting material and should be established at least 3 feet below the lowest adjacent final grade. A one-third increase in the recommended values may be used for wind or seismic loads. The recommended bearing values are net values. The weight of concrete in the footings may be assumed to be 50 pounds per cubic foot, and the weight of soil backfill may be neglected when determining the downward load on the footings.

Settlement: The settlement of spread footings established in the dense sand and gravel, or in the bedrock, may be determined using the following graphs.



Lateral Loads: Lateral loads may be resisted by soil friction and by the passive resistance of the soils and bedrock materials. A coefficient of friction of 0.5 may be used between footings and the supporting materials. The following passive values may be used:

Bedrock	1,000 pounds per square foot at the bedrock surface plus 300 pounds per square foot for each additional foot of depth, up to a maximum of 10,000 pounds per square foot.
Overburden Soils/Compacted Fill	300 pounds per square foot per foot of depth

A one-third increase in the passive values may be used for wind or seismic loads. The frictional resistance and the passive resistance of the soils and bedrock may be combined without reduction in determining the total lateral resistance.

Ultimate Design: The recommended bearing value and lateral load design values are for use with loadings determined by a conventional working stress design. When considering an ultimate design approach, the recommended design values may be multiplied by the following factors:

Design Item	Ultimate Design Factor
Bearing Value	2.5
Passive Pressure	1.5
Coefficient of Friction	1.25

In no event, however, should foundation sizes be less than those required for dead plus live loads when using the working stress design.



Footing Observation: The footing excavations should be observed by a geotechnical engineer or an engineering geologist licensed in the State of California to verify the presence of satisfactory materials at footing design elevations. The exposed bedrock and bedding should be observed by a geologist to confirm that the structural features of the bedrock at the planned foundation locations are favorable for foundation support. Footings should be deepened if necessary to extend into satisfactory supporting material.

The foundations should be left slightly uneven if necessary, rather than filling in overexcavated areas with loose or compacted soils. Where it is necessary to deepen a footing below the design depth, the overexcavated portion should be backfilled with concrete. It would be desirable and feasible to pour spread footings neat against the undisturbed bedrock. Soil backfill should be compacted as recommended in Section 13.0, Earthwork.

Subsurface seepage may be encountered in the footing excavations. A layer of lean concrete may be used to protect the bottom of the excavations and to provide support during construction.

8.3.3 ABUTMENT WALLS

For design of the cantilevered abutment walls, where the backfill is level and properly drained, it may be assumed that the soils will impose an active lateral pressure equal to that developed by a fluid with a density of 35 pounds per cubic foot. For walls with backfill sloped as steep as 2:1 (horizontal to vertical), an equivalent fluid pressure of 53 pounds per cubic foot may be used. An appropriate uniform lateral pressure equal to one-third of the anticipated vertical surcharge within 20 feet of the abutment walls should be included in the wall design.

The above values are for properly drained soils; weep holes should be provided to relieve hydrostatic pressures due to water.



8.3.4 SEISMIC DESIGN DATA

The proposed aerial structures will be designed in accordance with the bridge design criteria of the State of California Department of Transportation (CALTRANS). The geotechnical information needed for the CALTRANS criteria is the depth to rock and the expected peak bedrock acceleration. Based on a review of the soils and geologic data, the thickness of alluvium may be taken as being between 10 to 80 feet for Design Group 2. The expected peak bedrock acceleration is 0.6g, based on the proximity of nearby faults. In addition, the following values may be used in the seismic design of the aerial structures:

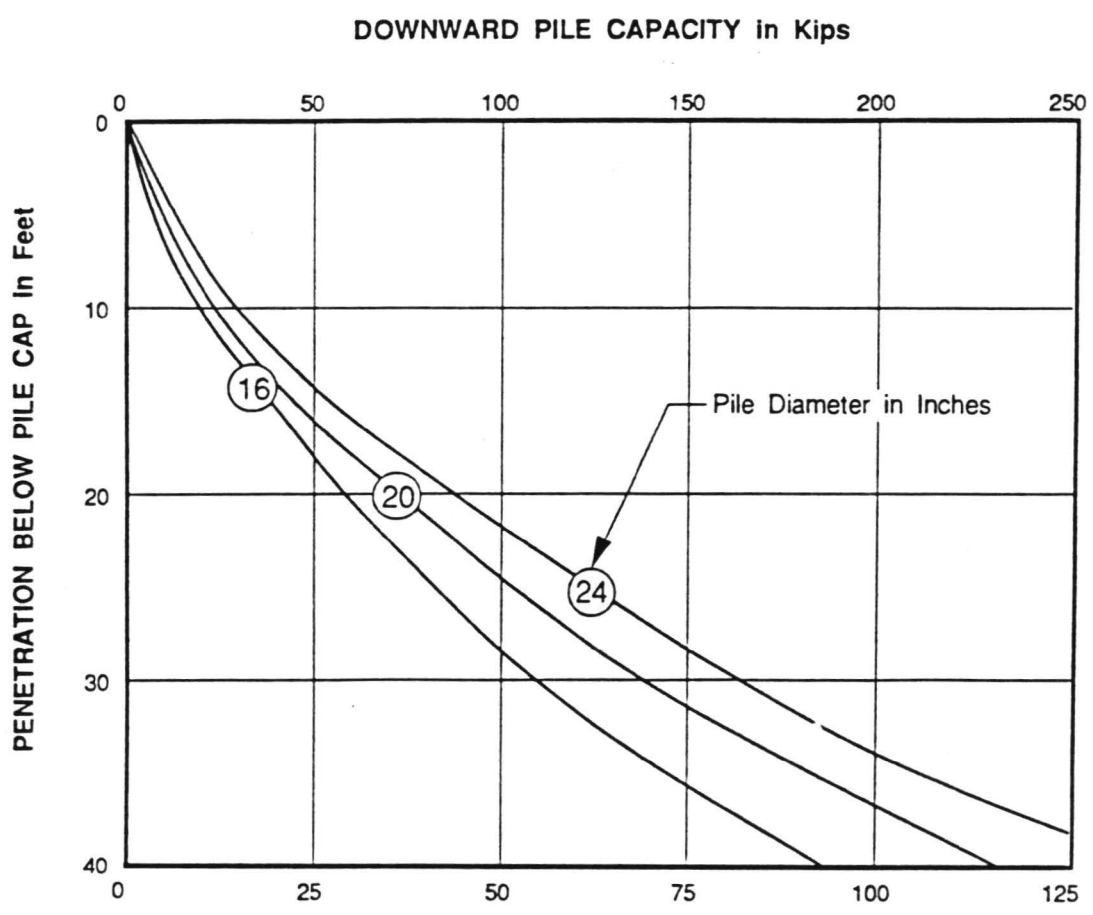
Shear Modulus 0 to 25 feet 25 to 75 feet Below 75 feet	2,400 ksf 8,400 ksf 14,900 ksf
Poisson's Ratio	0.3
Damping Value	0.1

The following figures complete the recommendations for Design Group 2:

Figure No.	Title
8.8 – 8.9	Drilled Pile Capacities, Design Group 2
8.10	Driven Pile Capacities, Design Group 2
8.11A – 8.11I	Lateral Deflection Diagrams, Design Group 2
8.12A – 8.12J	(P-Y) Curves, Design Group 2
8.13A – 8.13I	Bending Moment Diagrams, Design Group 2
8.14	Pile Driving Criteria, Design Group 2



JOB L92045AE4 DATE 2/15/93 DR. MS CHKD MS O.E. MS



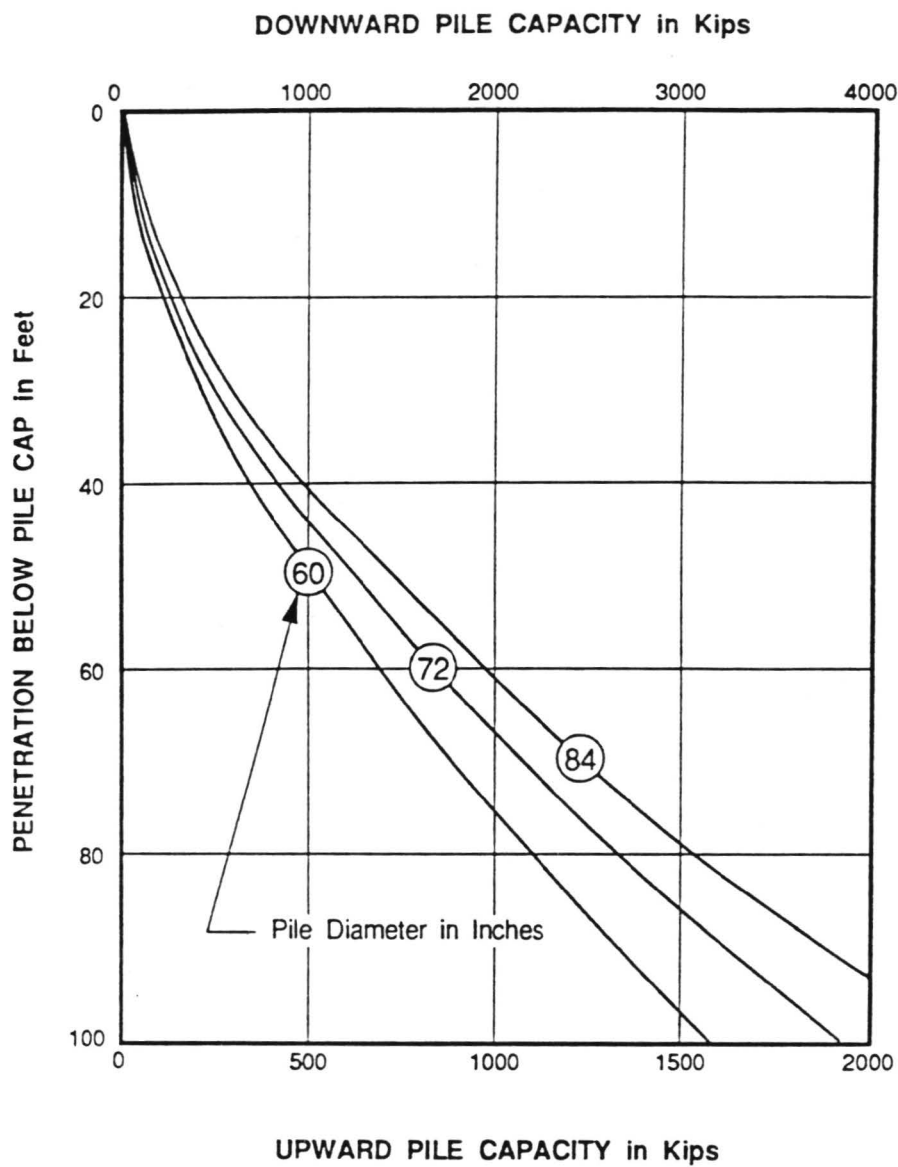
NOTES :

- (1) The indicated values refer to the total of dead plus live loads; a one-third increase may be used when considering wind or seismic loads.
- (2) Piles in groups should be spaced a minimum of 2-1/2 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.

DRILLED PILE CAPACITIES

(Design Group 2)

JOB L92045.AE4 DATE 2/15/93 DR. MS CHKD YLB



NOTES :

- (1) The indicated values refer to the total of dead plus live loads; a one-third increase may be used when considering wind or seismic loads.
- (2) Piles in groups should be spaced a minimum of 2-1/2 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.

DRILLED PILE CAPACITIES

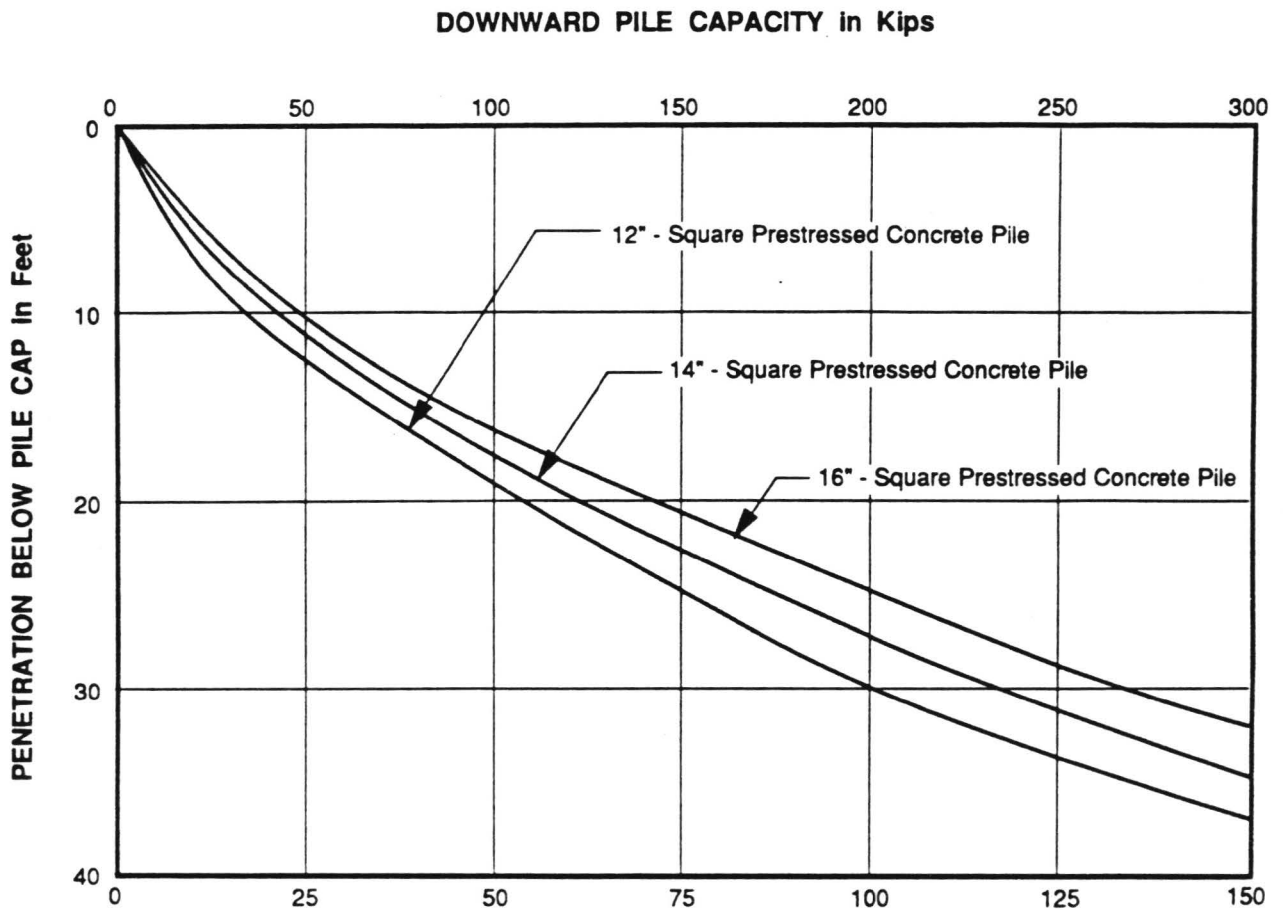
(Design Group 2)

LAW / CRANDALL, INC.



FIGURE 8.9

JOB L92045, AE4 DATE 2/15/93 DR. IK MS CHKD O.E. *MS*



UPWARD PILE CAPACITY in Kips

NOTES :

- (1) The indicated values refer to the total of dead plus live loads; a one-third increase may be used when considering wind or seismic loads.
- (2) Piles in groups should be spaced a minimum of 3 feet on centers.
- (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.

DRIVEN PILE CAPACITIES

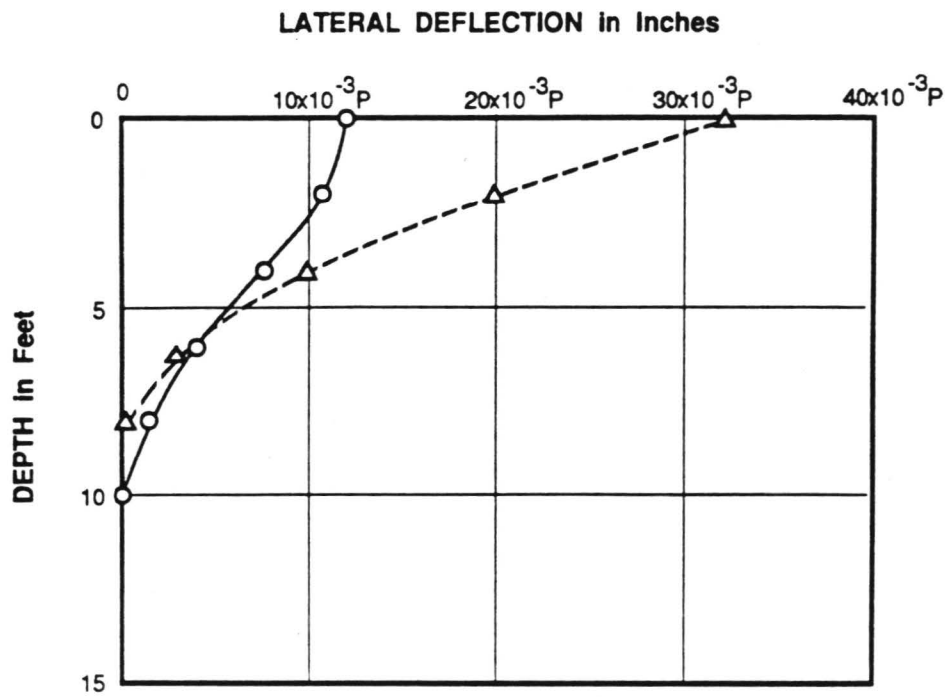
(Design Group 2)

LAW / CRANDALL, INC.



FIGURE 8.10

JOB L92045/AE4 DATE 4/29/93 DR. CHKD 11/13 YK O.E. YK



KEY:

- FIXED HEAD PILE
- △—△ FREE HEAD PILE
- P LATERAL LOAD APPLIED AT TOP OF THE PILE IN KIPS

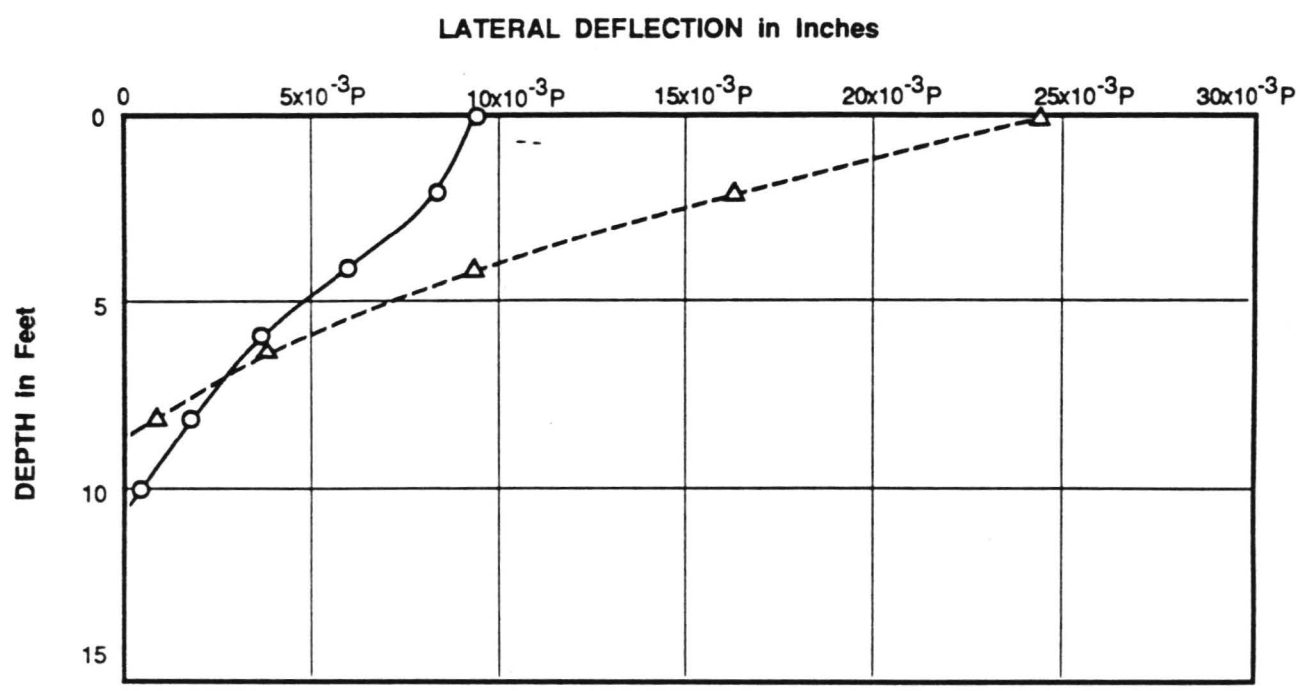
LATERAL DEFLECTION DIAGRAMS
12 - INCH SQUARE PILE
(Design Group 2)

LAW / CRANDALL, INC.



FIGURE 8.11A

JOB L92045.AE4 DATE 4/29/93 DR. *CHKD* YK *MS*



KEY:

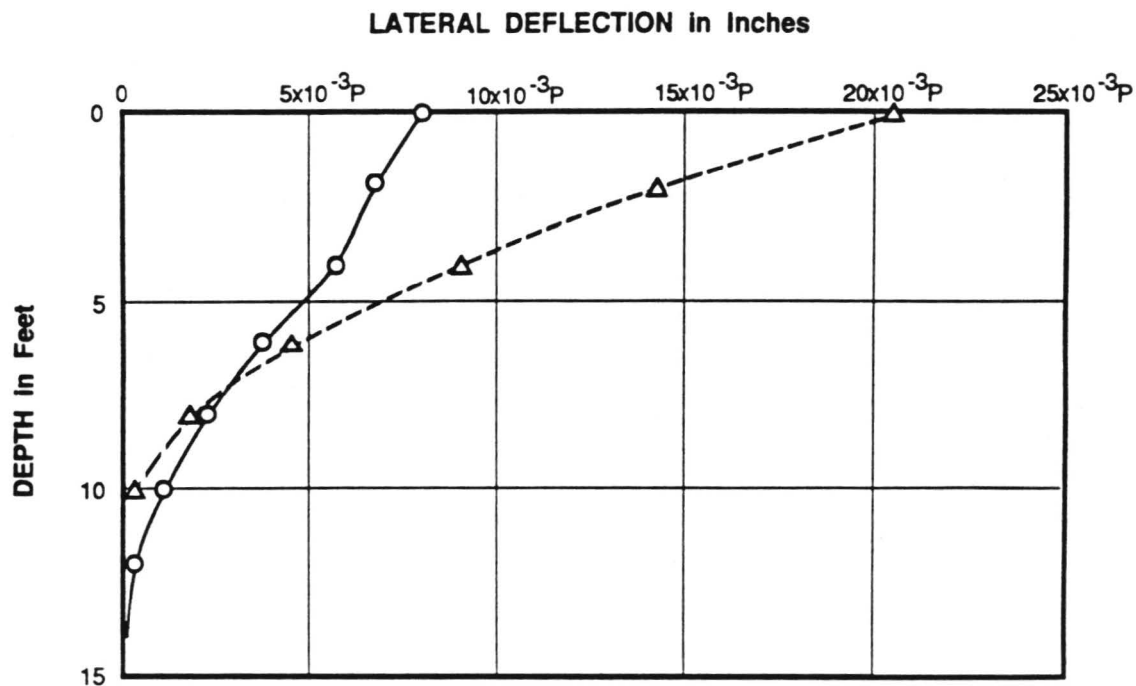
○—○ FIXED HEAD PILE

△--△ FREE HEAD PILE

P LATERAL LOAD APPLIED AT TOP OF THE PILE IN KIPS

LATERAL DEFLECTION DIAGRAMS
14 - INCH SQUARE PILE
 (Design Group 2)

JOB L92045.AE4 DATE 4/29/93 DR. O.E. YK ~~CHKD~~ MS Y6



KEY:

- FIXED HEAD PILE
- △--△ FREE HEAD PILE
- P LATERAL LOAD APPLIED AT TOP OF THE PILE IN KIPS

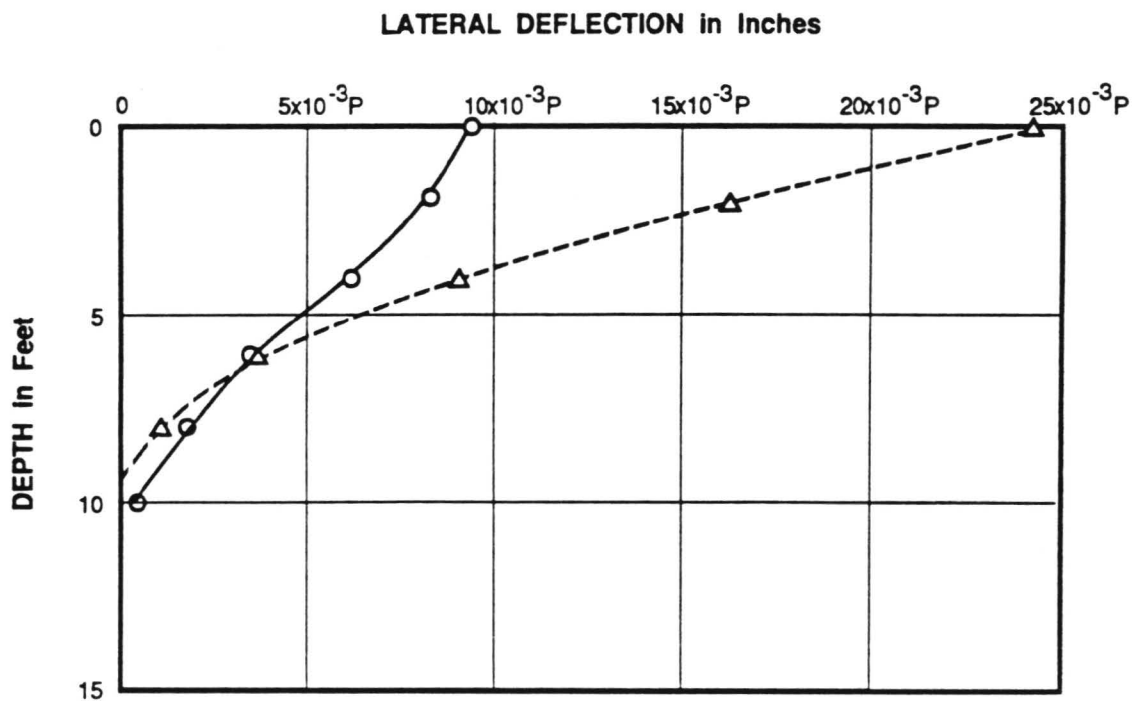
LATERAL DEFLECTION DIAGRAMS
16 - INCH SQUARE PILE
(Design Group 2)

LAW / CRANDALL, INC.



FIGURE 8.11C

MS
 CHKD
 YK
 O.E.
 DR.
 4/29/93
 DATE
 L92045.AE4
 JOB



KEY:

○—○ FIXED HEAD PILE

△--△ FREE HEAD PILE

P LATERAL LOAD APPLIED AT TOP OF THE PILE IN KIPS

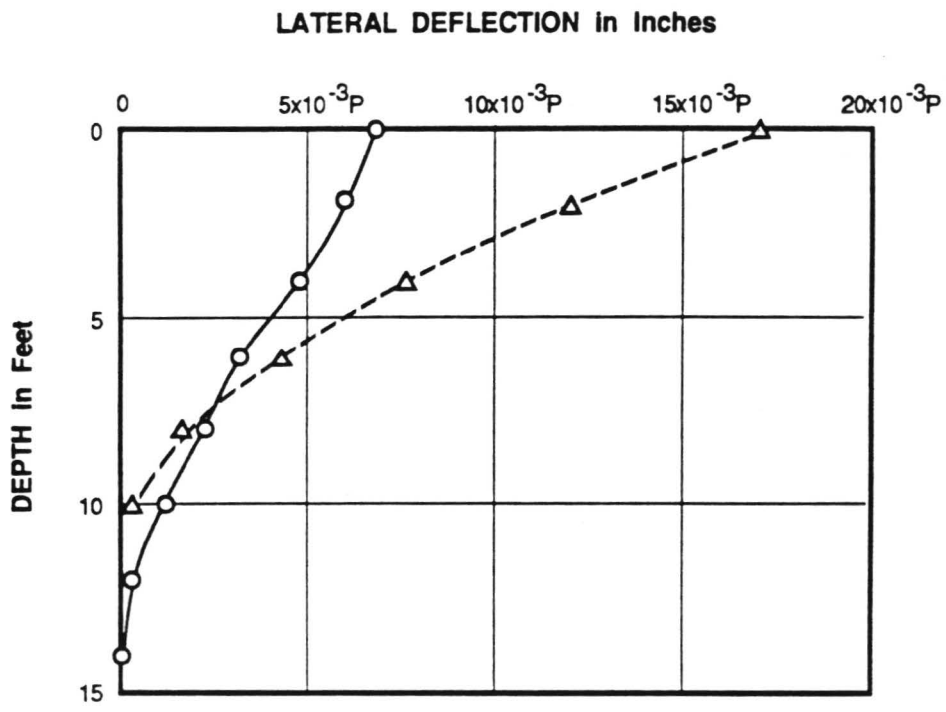
LATERAL DEFLECTION DIAGRAMS
16 - INCH DIAMETER PILE
 (Design Group 2)

LAW / CRANDALL, INC.



FIGURE 8.11D

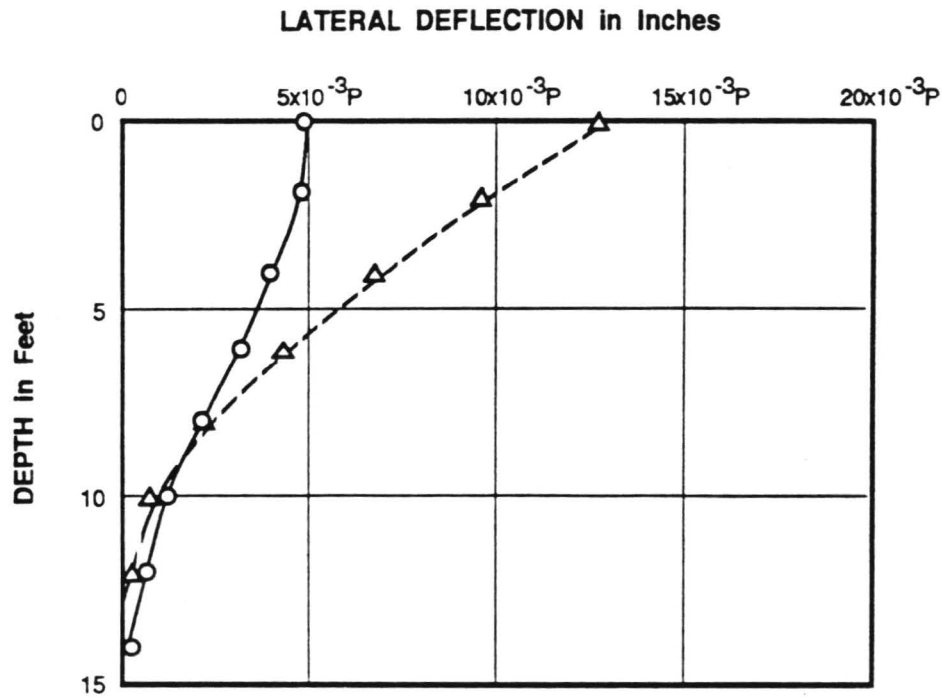
JOB L92045.AE4 DATE 4/29/93 DR. IK O.E. YK ~~CHKD~~ *WJ*



KEY:
 ○—○ FIXED HEAD PILE
 △--△ FREE HEAD PILE
 P LATERAL LOAD APPLIED AT TOP OF THE PILE IN KIPS

LATERAL DEFLECTION DIAGRAMS
 20 - INCH DIAMETER PILE
 (Design Group 2)

JOB L92045.AE4 DATE 4/29/93 DR. *MS* O.E. YK *CHKD* *MS*



KEY:

- FIXED HEAD PILE
- △--△ FREE HEAD PILE
- P LATERAL LOAD APPLIED AT TOP OF THE PILE IN KIPS

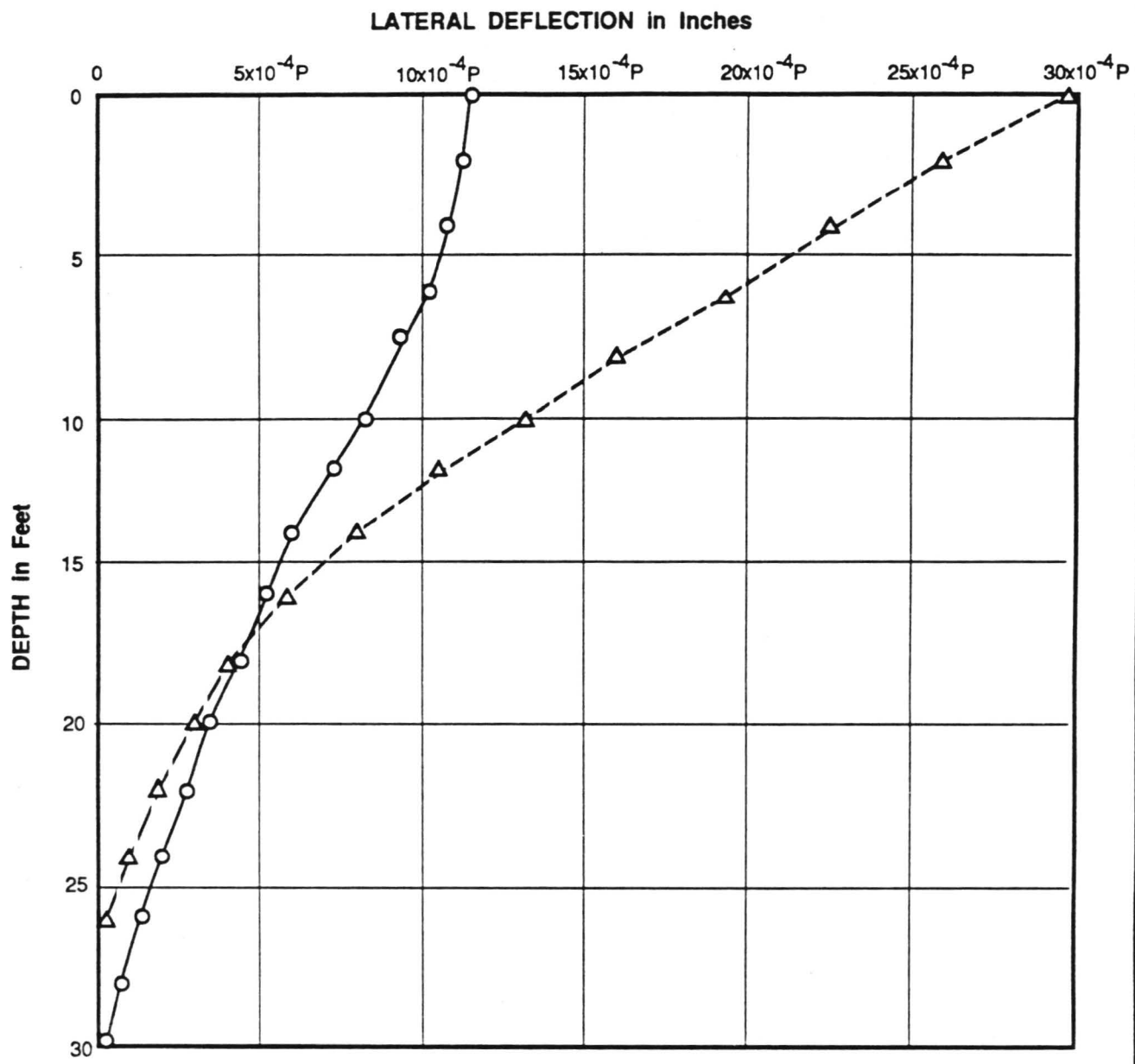
LATERAL DEFLECTION DIAGRAMS
24 - INCH DIAMETER PILE
 (Design Group 2)

LAW / CRANDALL, INC.



FIGURE 8.11F

JOB L92045-AE4
 DATE 4/29/93
 DR. K
 O.E. YK
 CHKD MB



KEY:

- FIXED HEAD PILE
- △--△ FREE HEAD PILE
- P LATERAL LOAD APPLIED AT TOP OF THE PILE IN KIPS

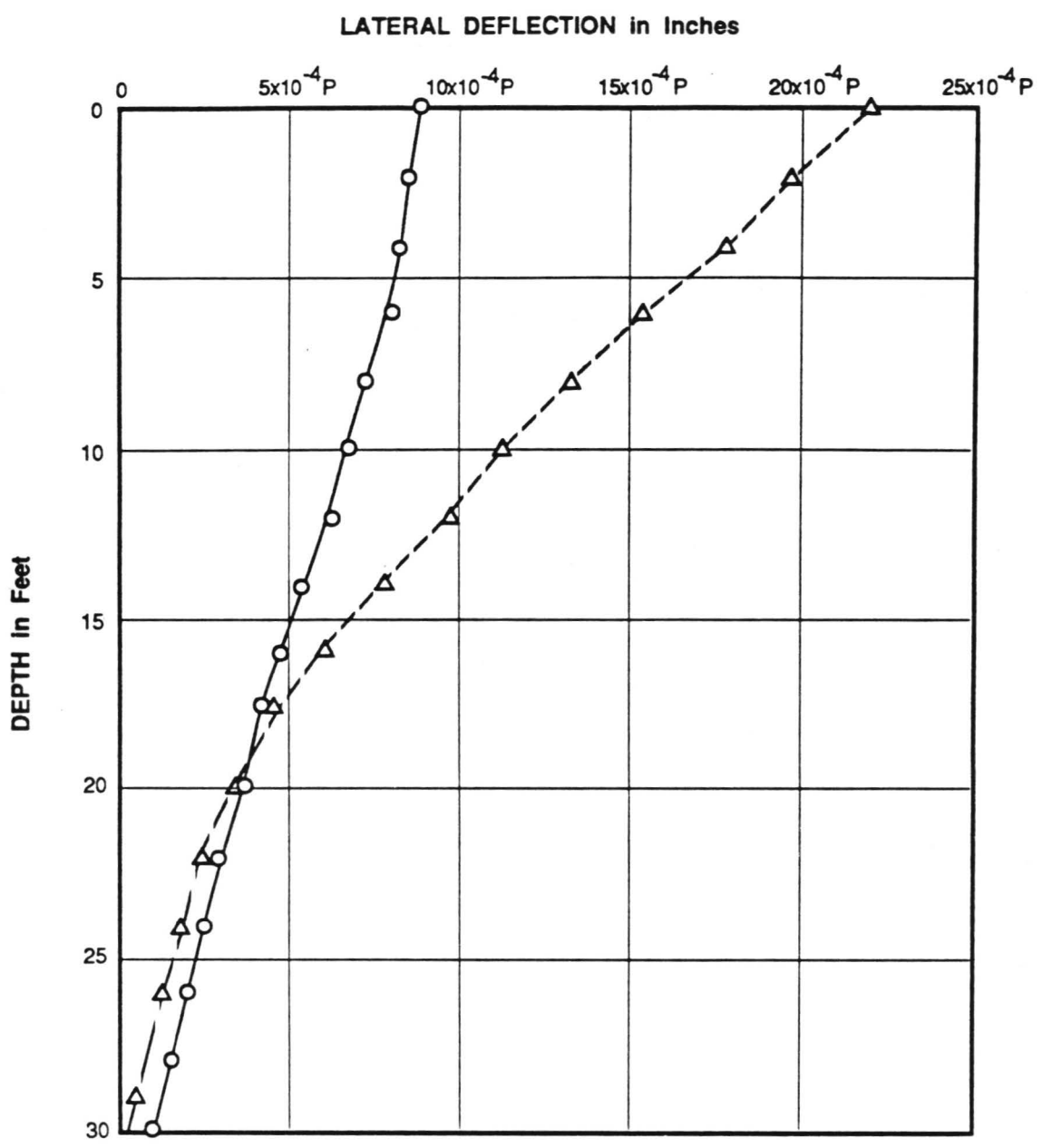
LATERAL DEFLECTION DIAGRAMS
60 - INCH DIAMETER PILE
 (Design Group 2)

LAW / CRANDALL, INC.



FIGURE 8.11G

JOB L92045.AE4 DATE 4/29/93 DR. IK O.E. YK ☒ CHKD MS



KEY:

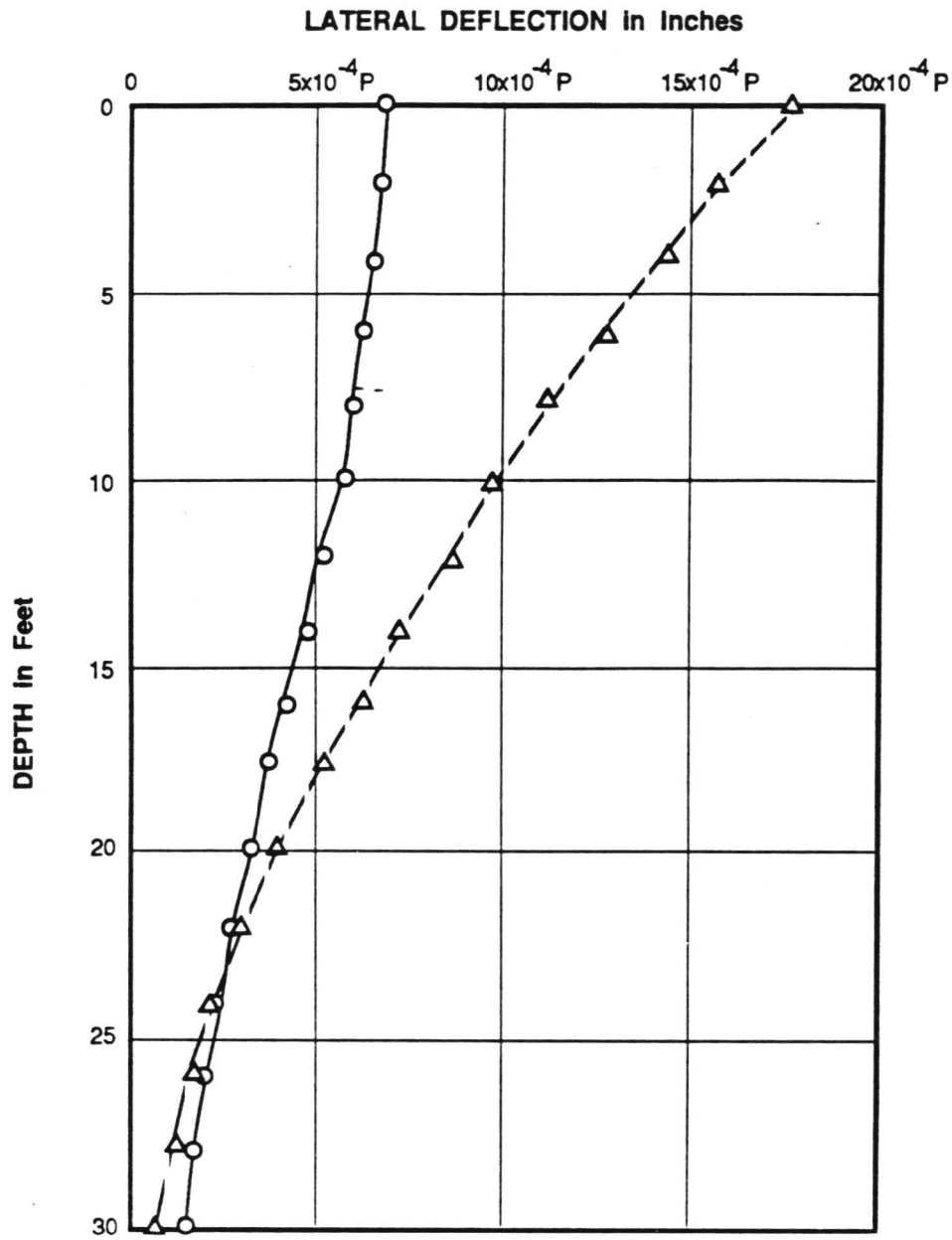
○—○ FIXED HEAD PILE

△--△ FREE HEAD PILE

P LATERAL LOAD APPLIED AT TOP OF THE PILE IN KIPS

LATERAL DEFLECTION DIAGRAMS
72 - INCH DIAMETER PILE
 (Design Group 2)

JOB L92045.AE4 DATE 4/29/93 DR. O.E. YK CHKD *WJ*

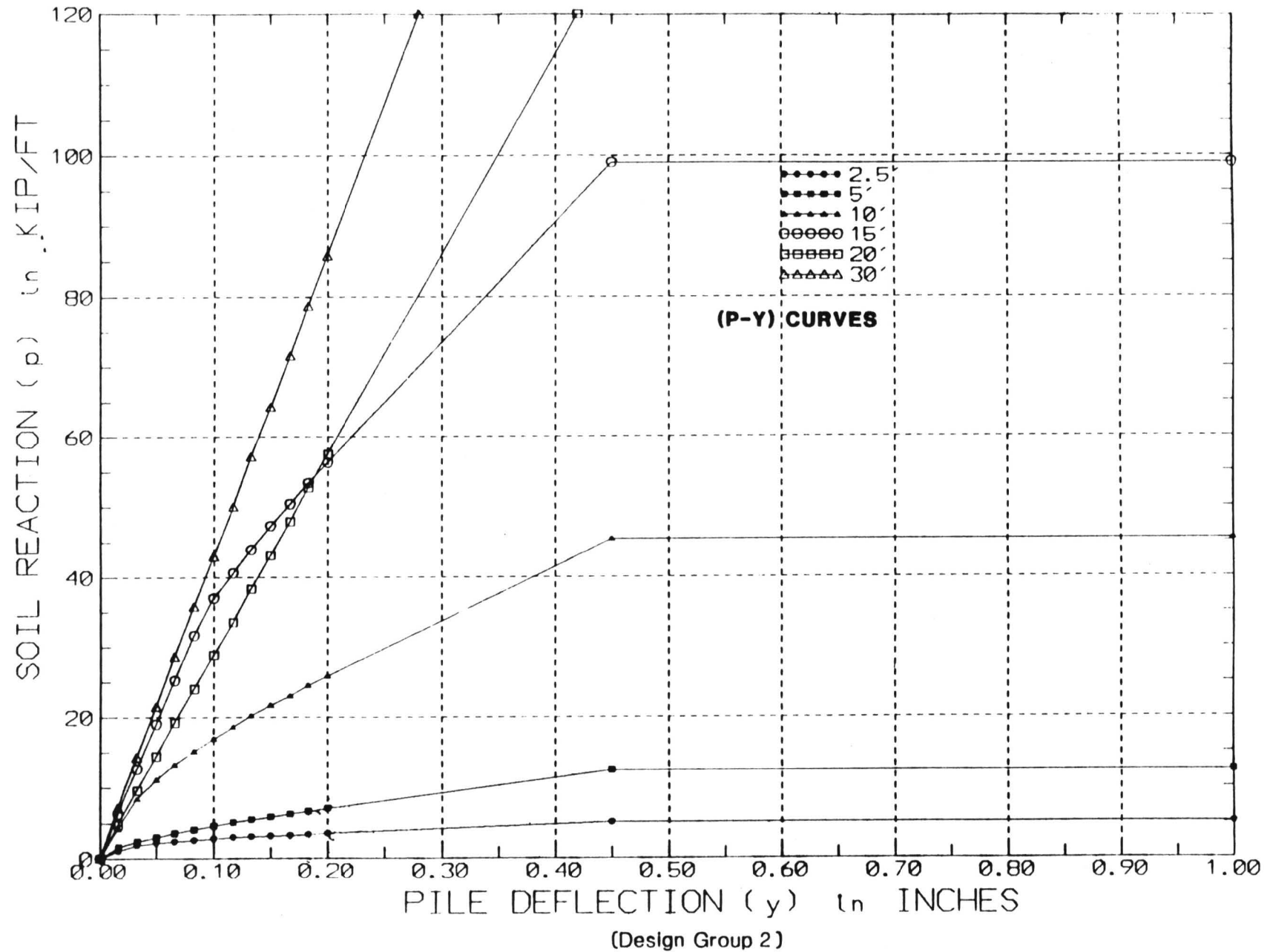


KEY:

- FIXED HEAD PILE
- △--△ FREE HEAD PILE
- P LATERAL LOAD APPLIED AT TOP OF THE PILE IN KIPS

LATERAL DEFLECTION DIAGRAMS
84 - INCH DIAMETER PILE
 (Design Group 2)

12 INCH SQUARE PILE

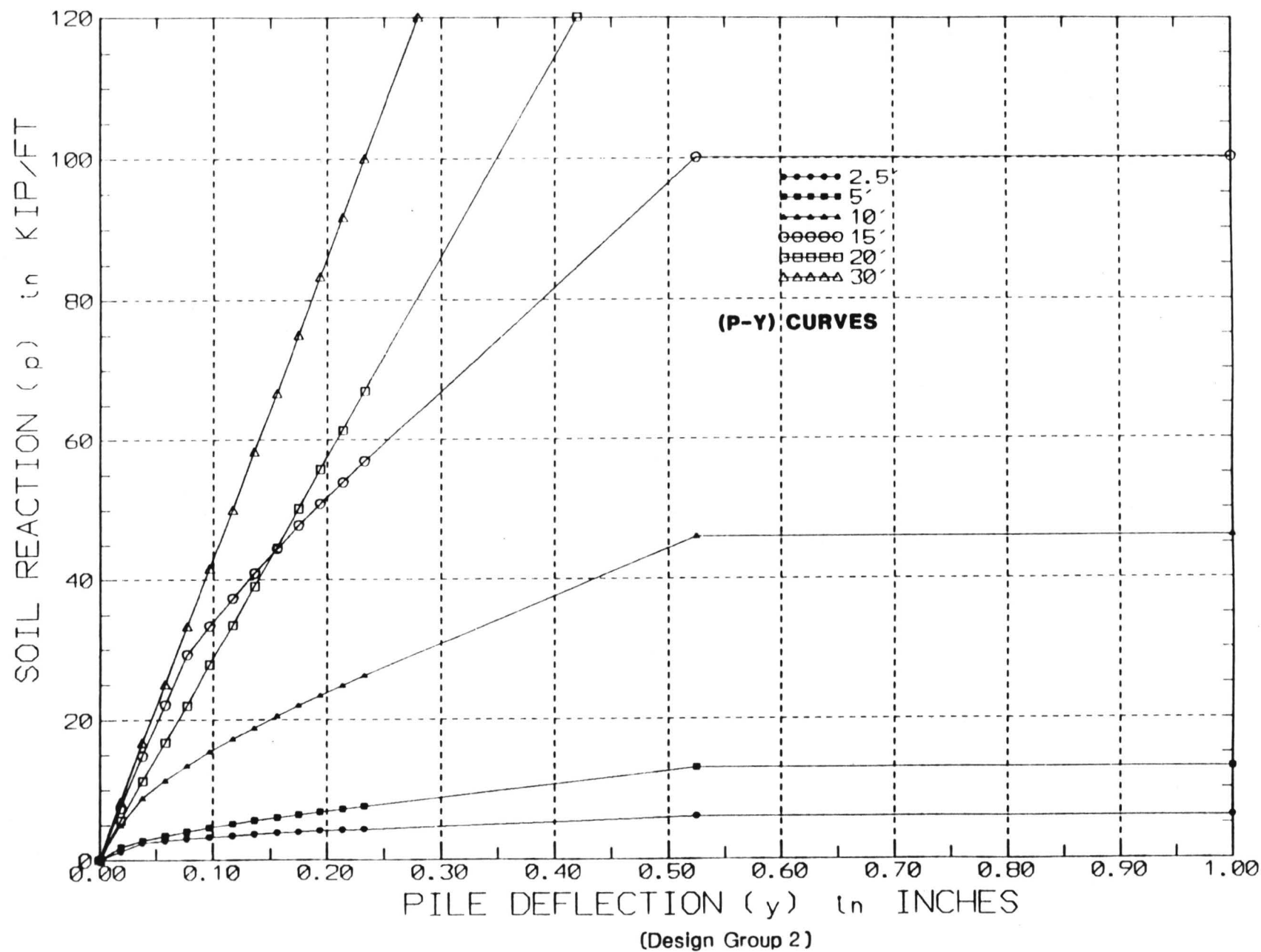


LAW/CRANDALL, INC.



FIGURE 8.12A

14 INCH SQUARE PILE

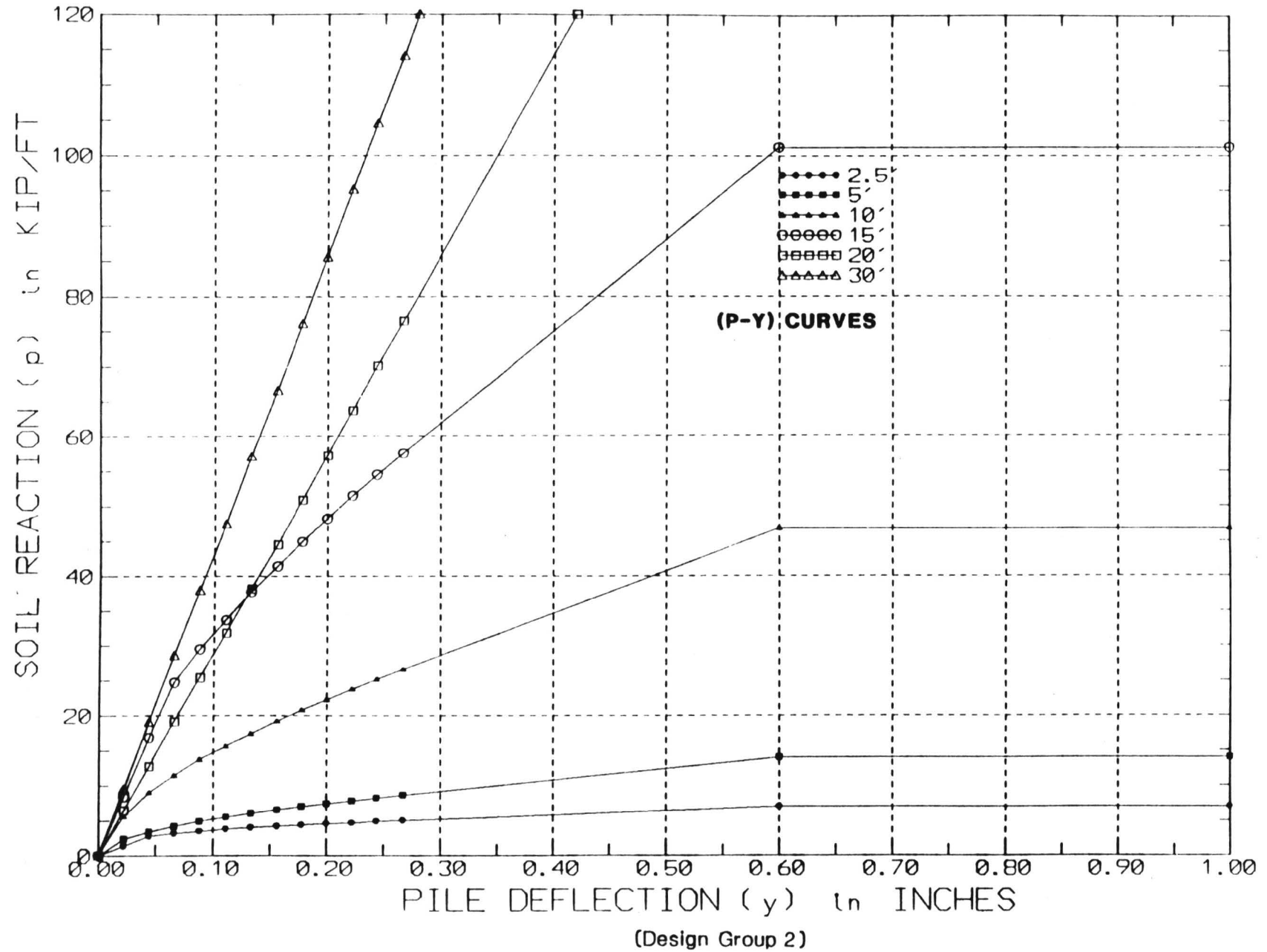


LAW/CRANDALL, INC.



FIGURE 8.12B

16 INCH SQUARE PILE

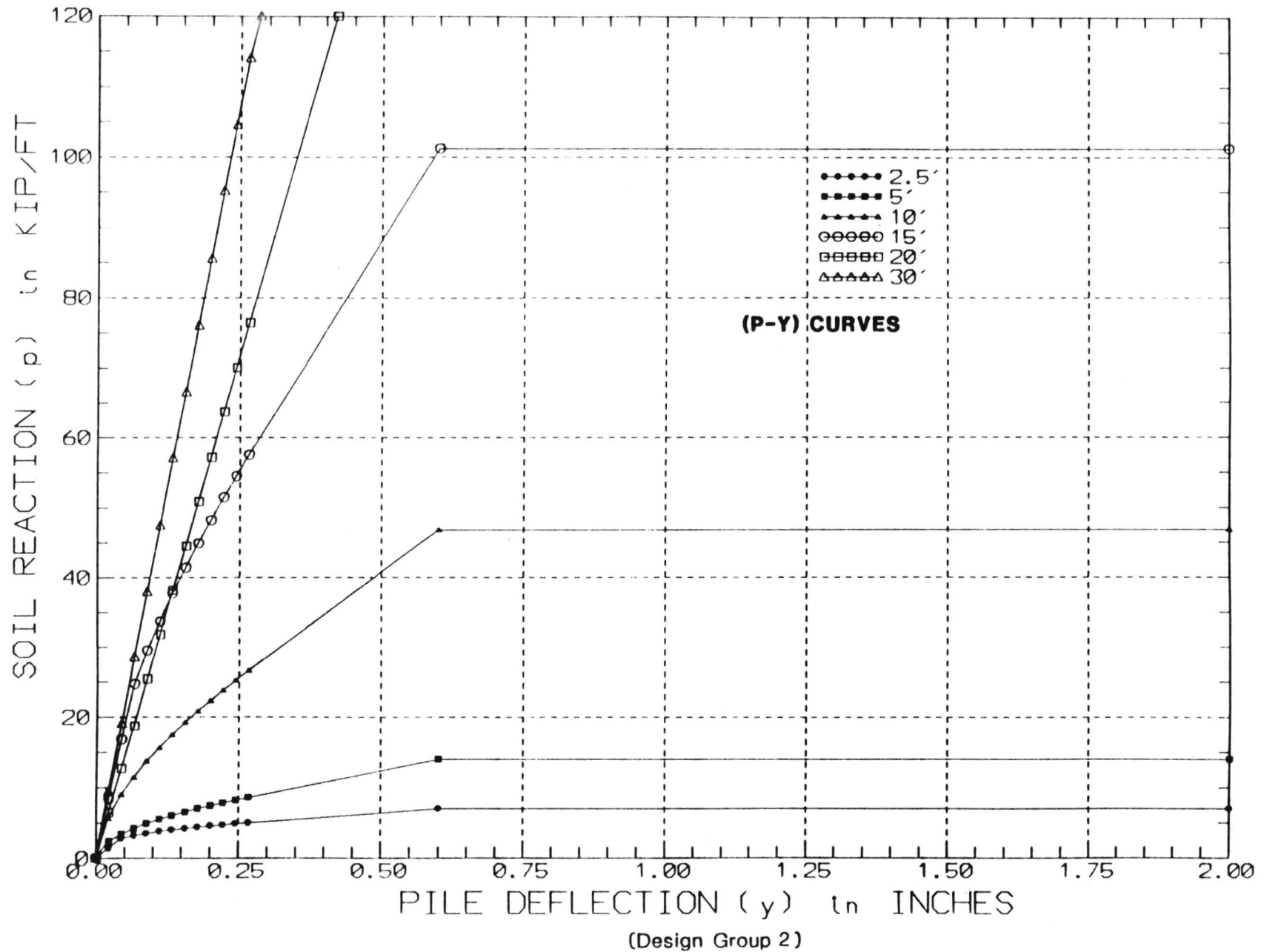


LAW/CRANDALL, INC.



FIGURE 8.12C

16 INCH CIDH PILE



LAW/CRANDALL, INC.



FIGURE 8.12D

20 INCH CIDH PILE

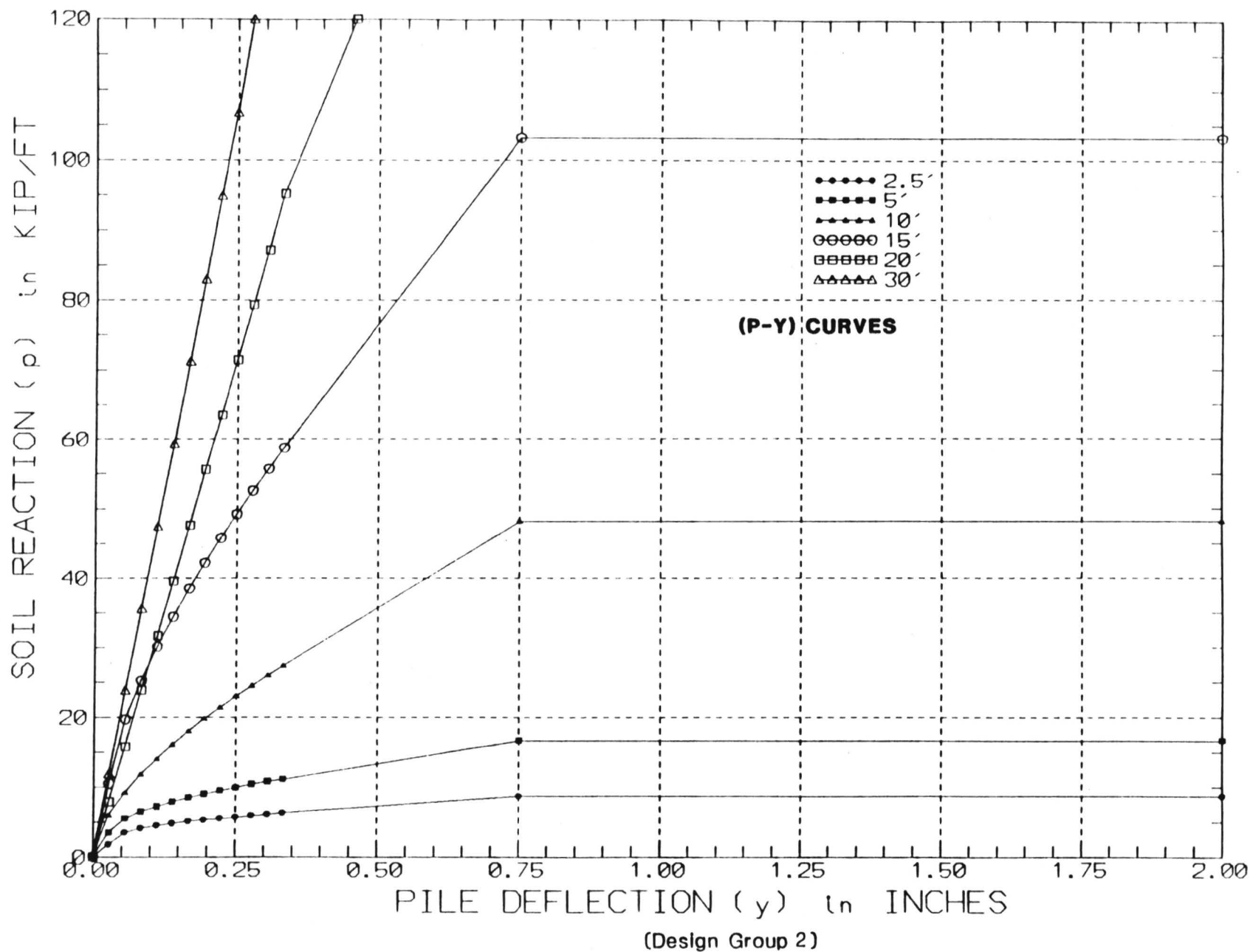
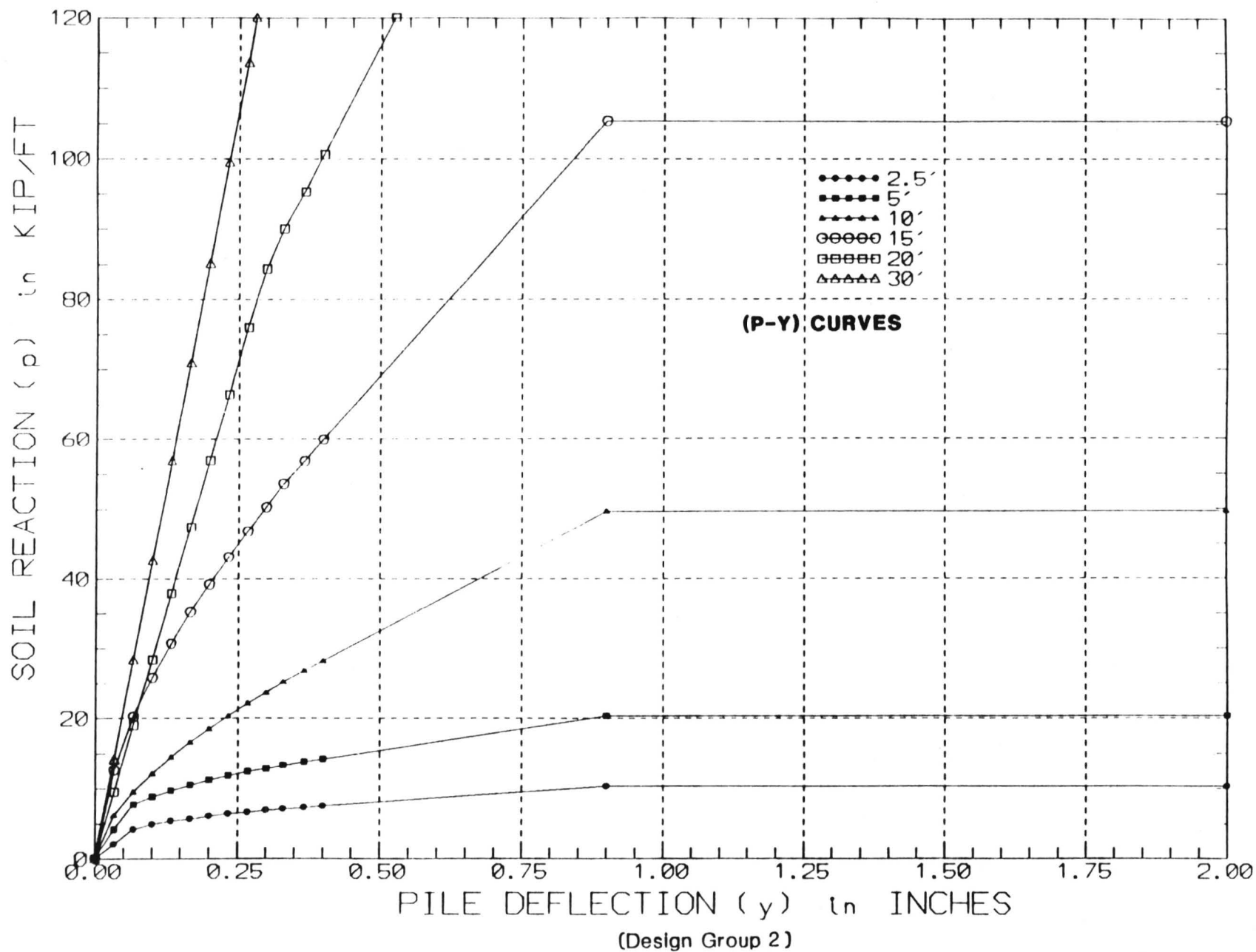


FIGURE 8.12E

LAW/CRANDALL, INC.



24 INCH CIDH PILE

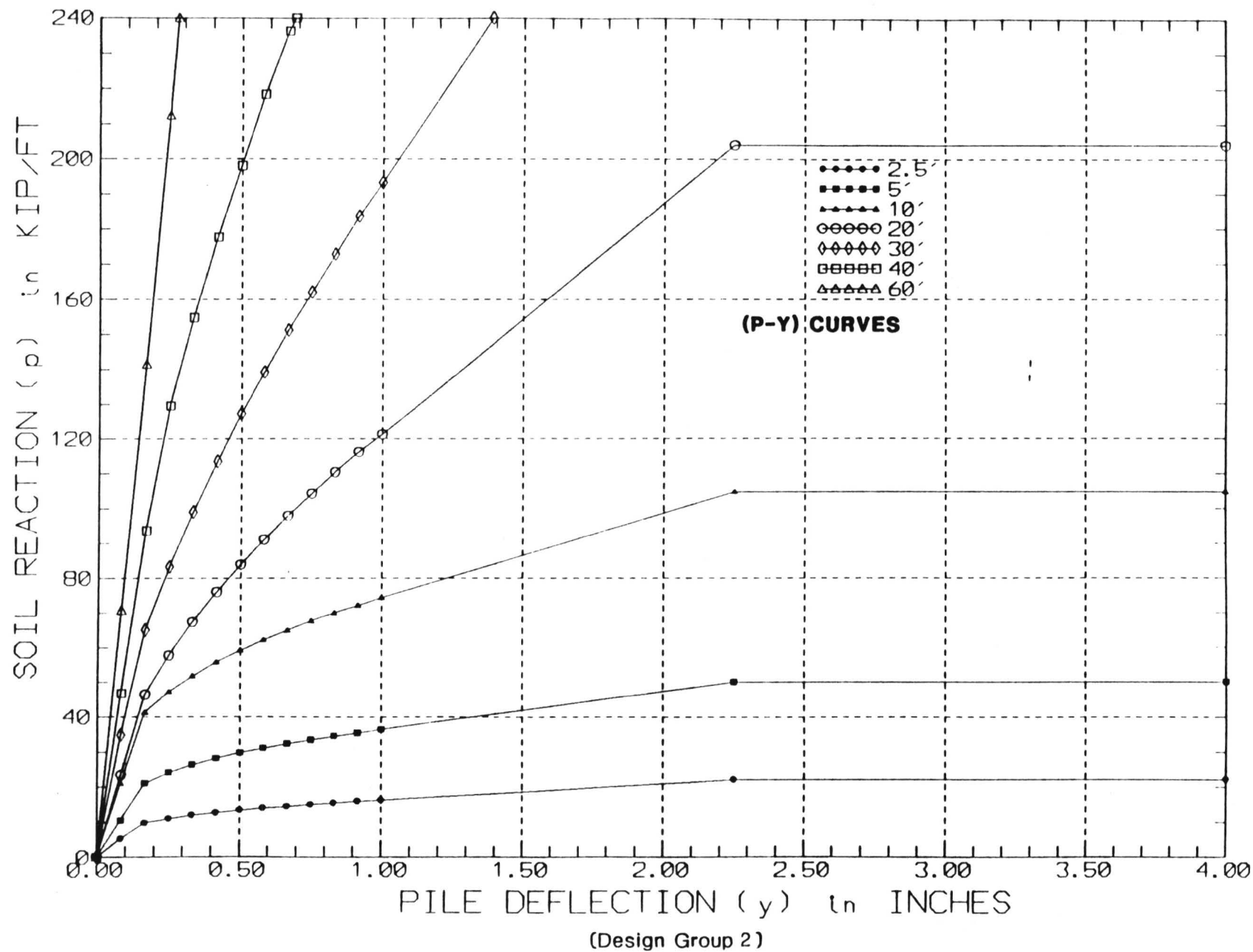


LAW/CRANDALL, INC.



FIGURE 8.12F

60 INCH CIDH PILE

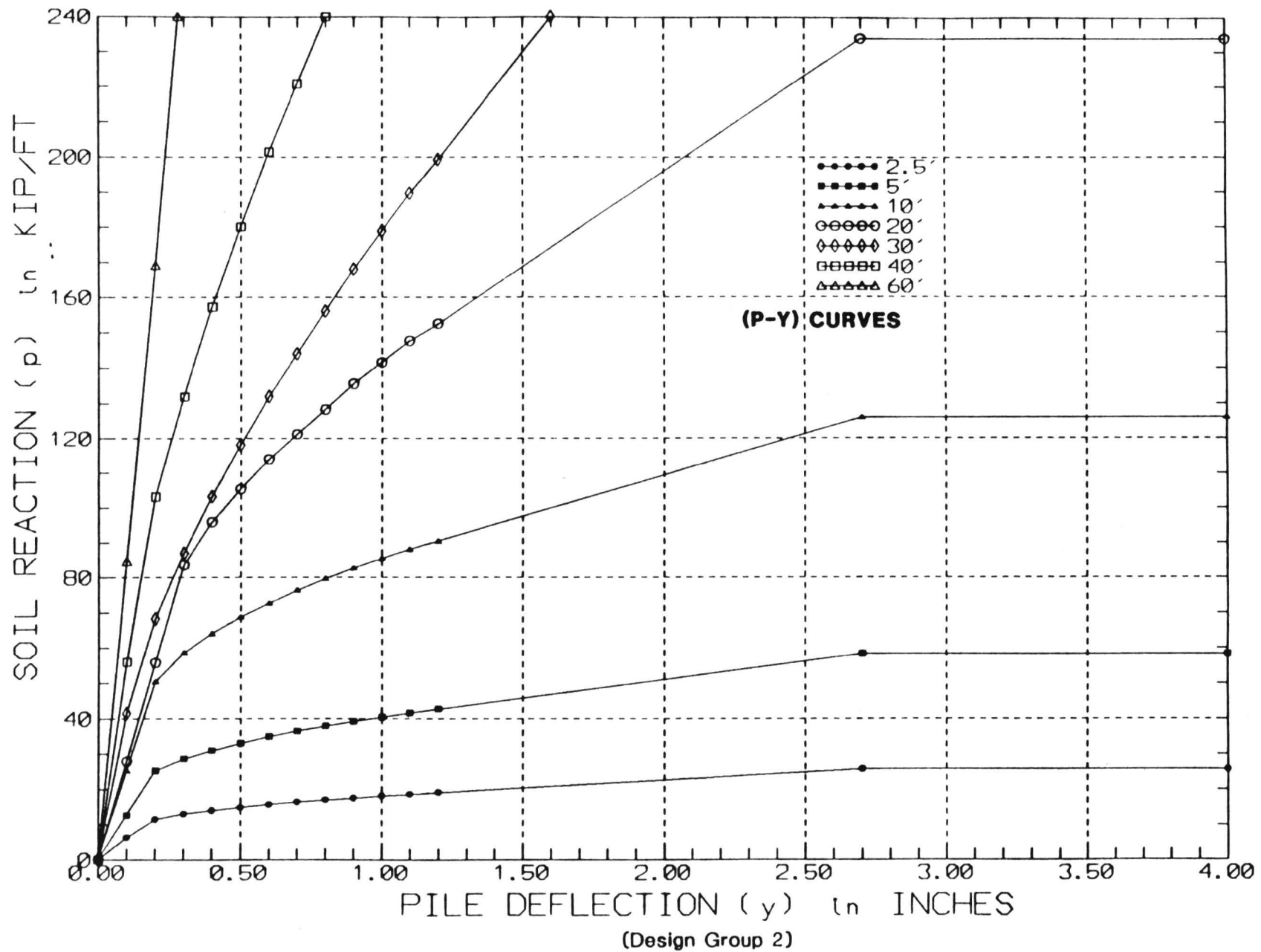


LAW/CRANDALL, INC.



FIGURE 8.12G

72 INCH CIDH PILE

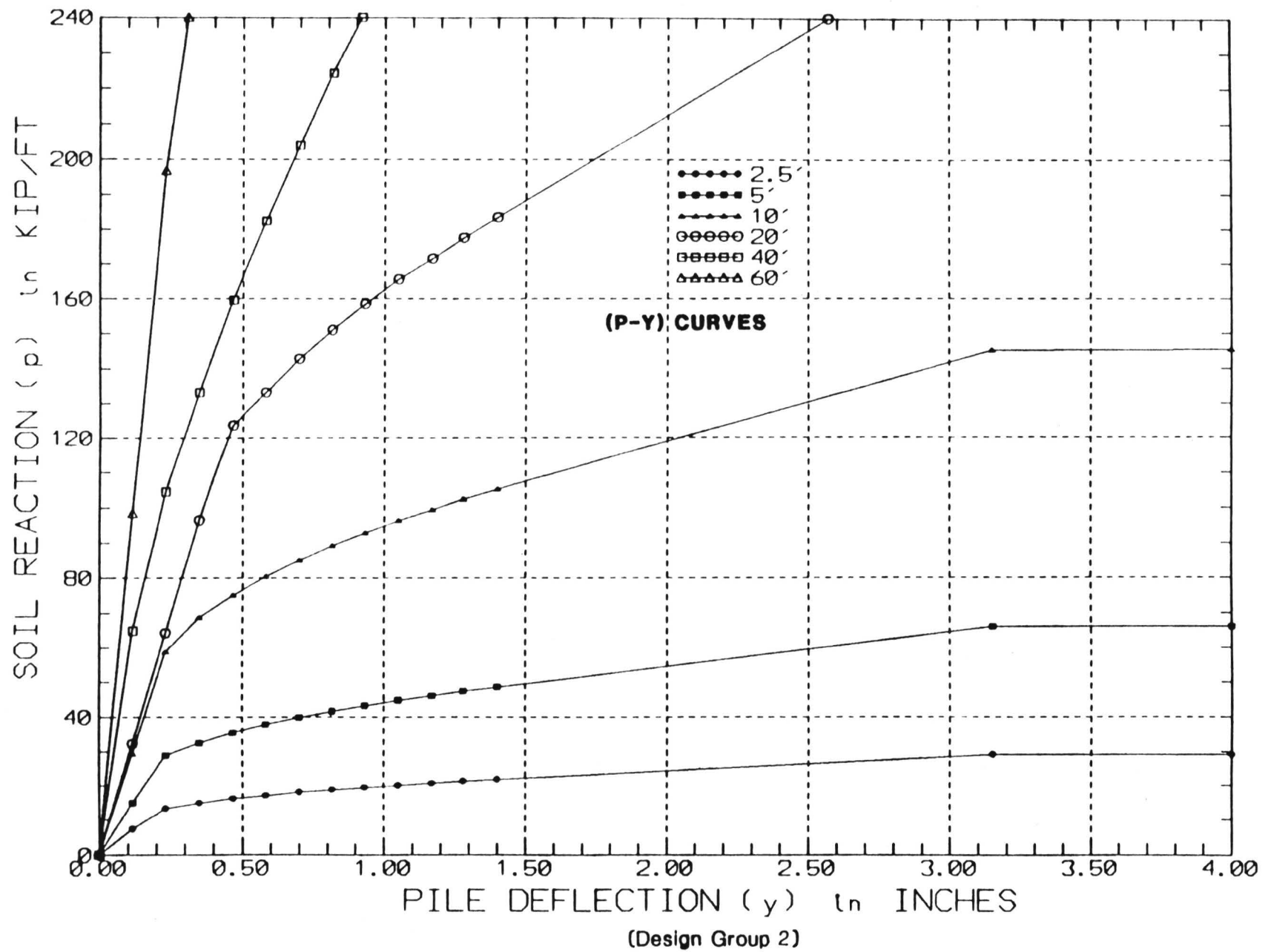


LAW/CRANDALL, INC.



FIGURE 8.12H

84 INCH CIDH PILE



96 INCH CIDH PILE

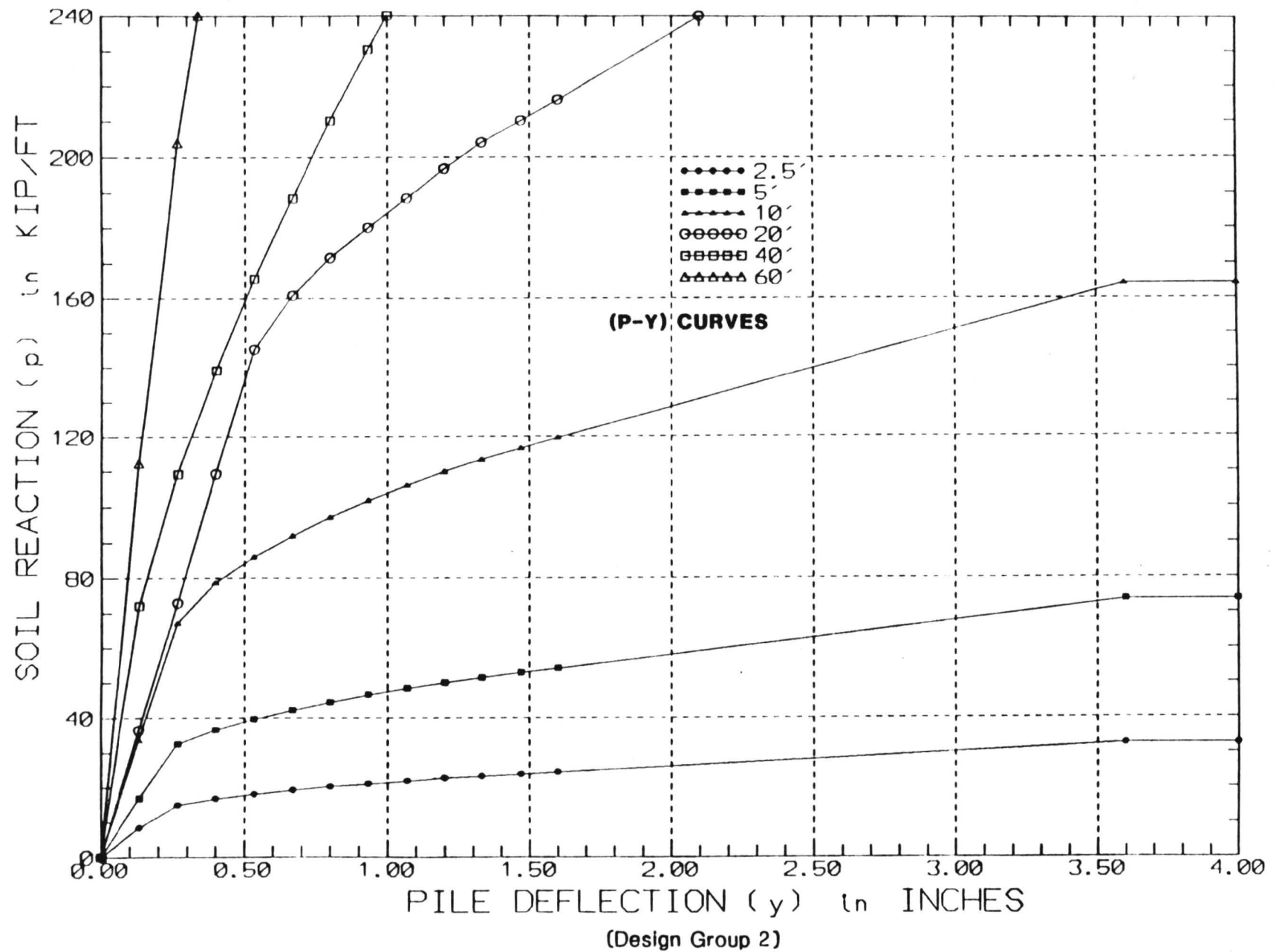


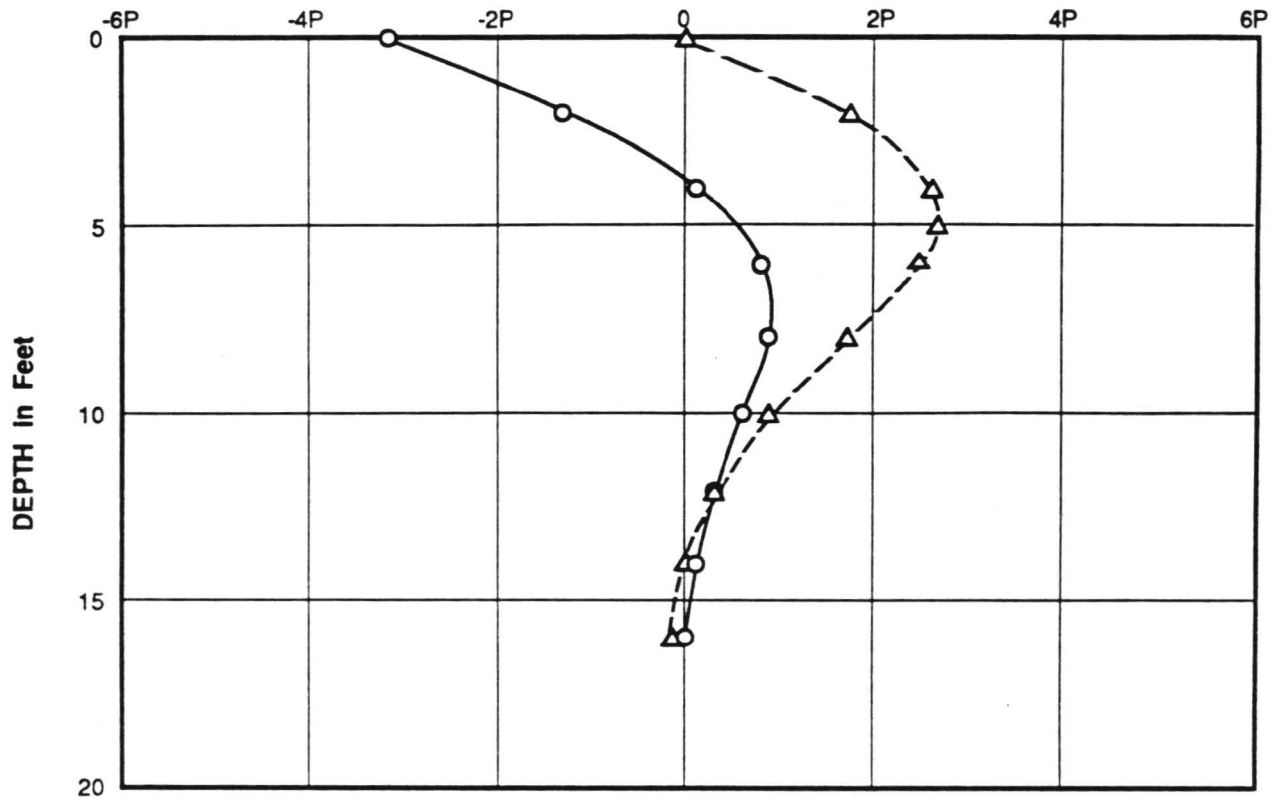
FIGURE 8.12J

LAW/CRANDALL, INC.



JOB L92045.AE4 DATE 2/22/93 DR. IK MS/MS CHKD O.E.

BENDING MOMENT in Foot-Kips



KEY:

- FIXED HEAD PILE
- △--△ FREE HEAD PILE
- P LATERAL LOAD APPLIED AT TOP OF THE PILE IN KIPS

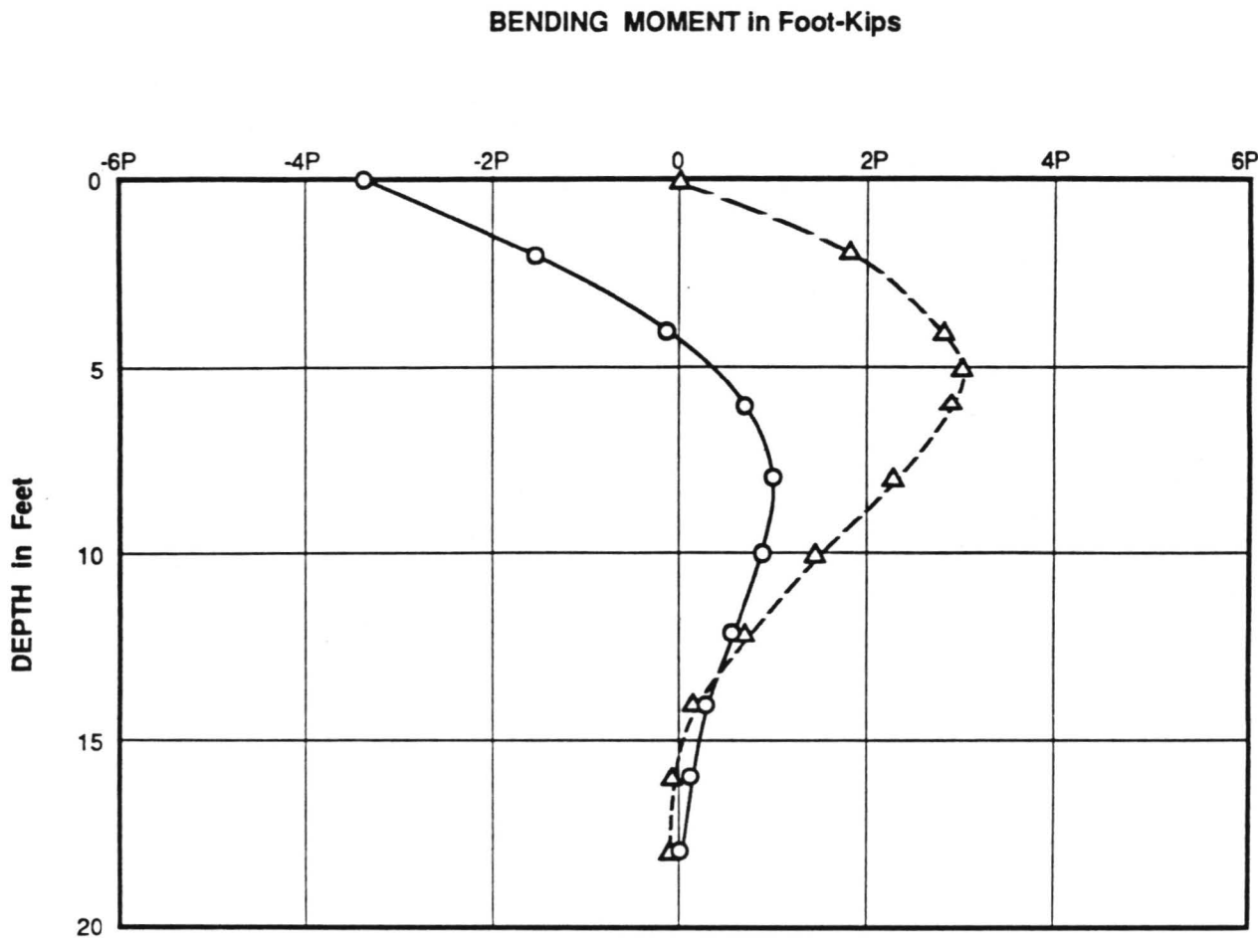
BENDING MOMENT DIAGRAMS 12 - INCH SQUARE PILE (Design Group 2)

LAW / CRANDALL, INC.



FIGURE 8.13A

JOB L92045.AE4 DATE 2/22/93 DR. IK O.E. MS CHKD



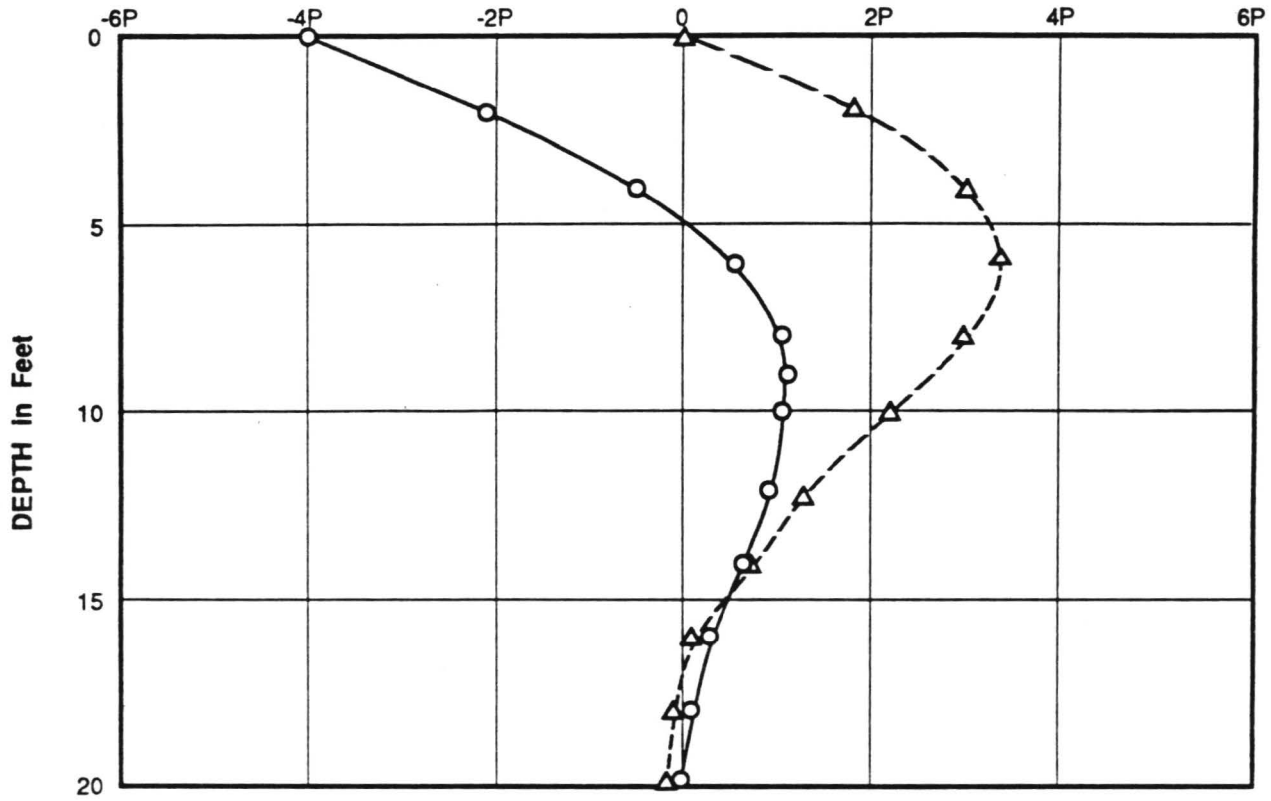
KEY:

- FIXED HEAD PILE
- △--△ FREE HEAD PILE
- P LATERAL LOAD APPLIED AT TOP OF THE PILE IN KIPS

BENDING MOMENT DIAGRAMS
14 - INCH SQUARE PILE
(Design Group 2)

JOB L92045.AE4 DATE 2/22/93 DR. Ik MS CHKD *AB* O.E.

BENDING MOMENT in Foot-Kips



KEY:



FIXED HEAD PILE

FREE HEAD PILE

P

LATERAL LOAD APPLIED AT TOP OF THE PILE IN KIPS

BENDING MOMENT DIAGRAMS
16 - INCH SQUARE PILE
(Design Group 2)

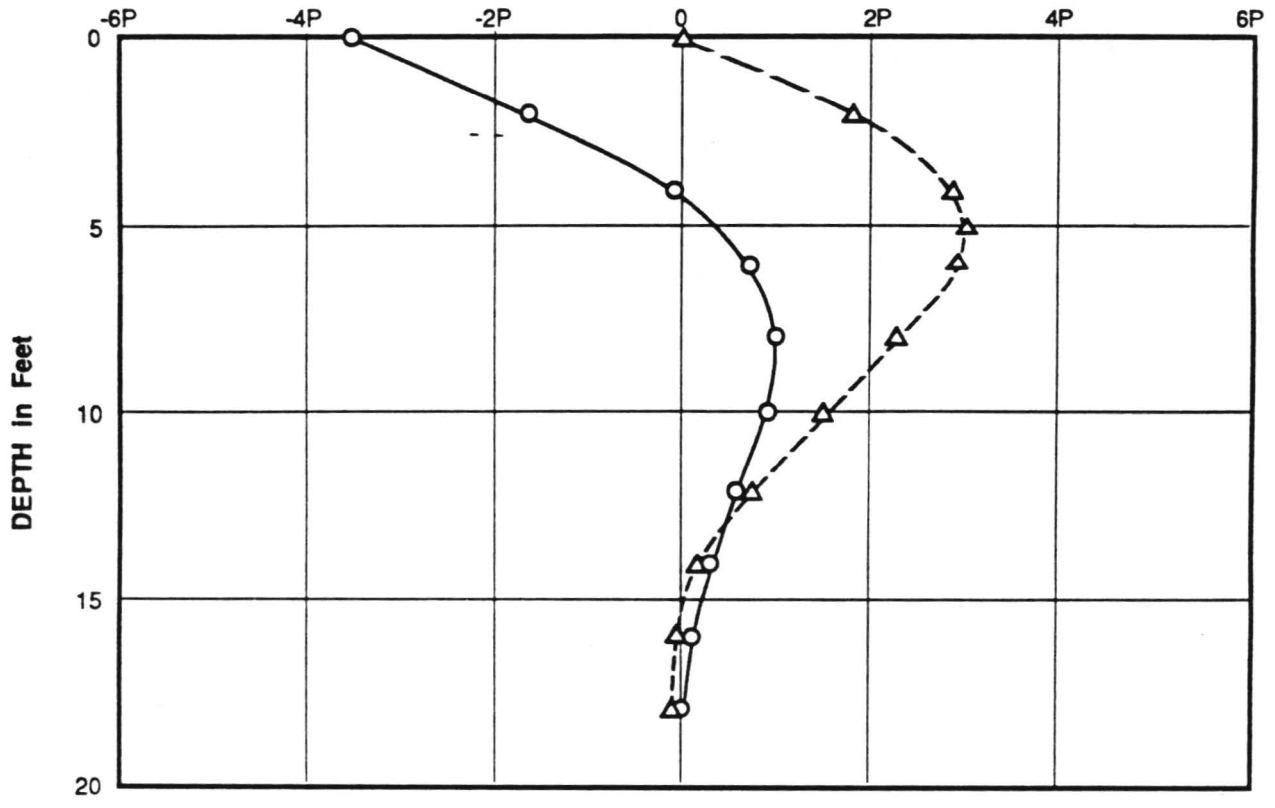
LAW / CRANDALL, INC.



FIGURE 8.13C

JOB L92045.AE4 DATE 2/22/93 DR. IK MS 115 CHKD *fb* O.E.

BENDING MOMENT in Foot-Kips



KEY:

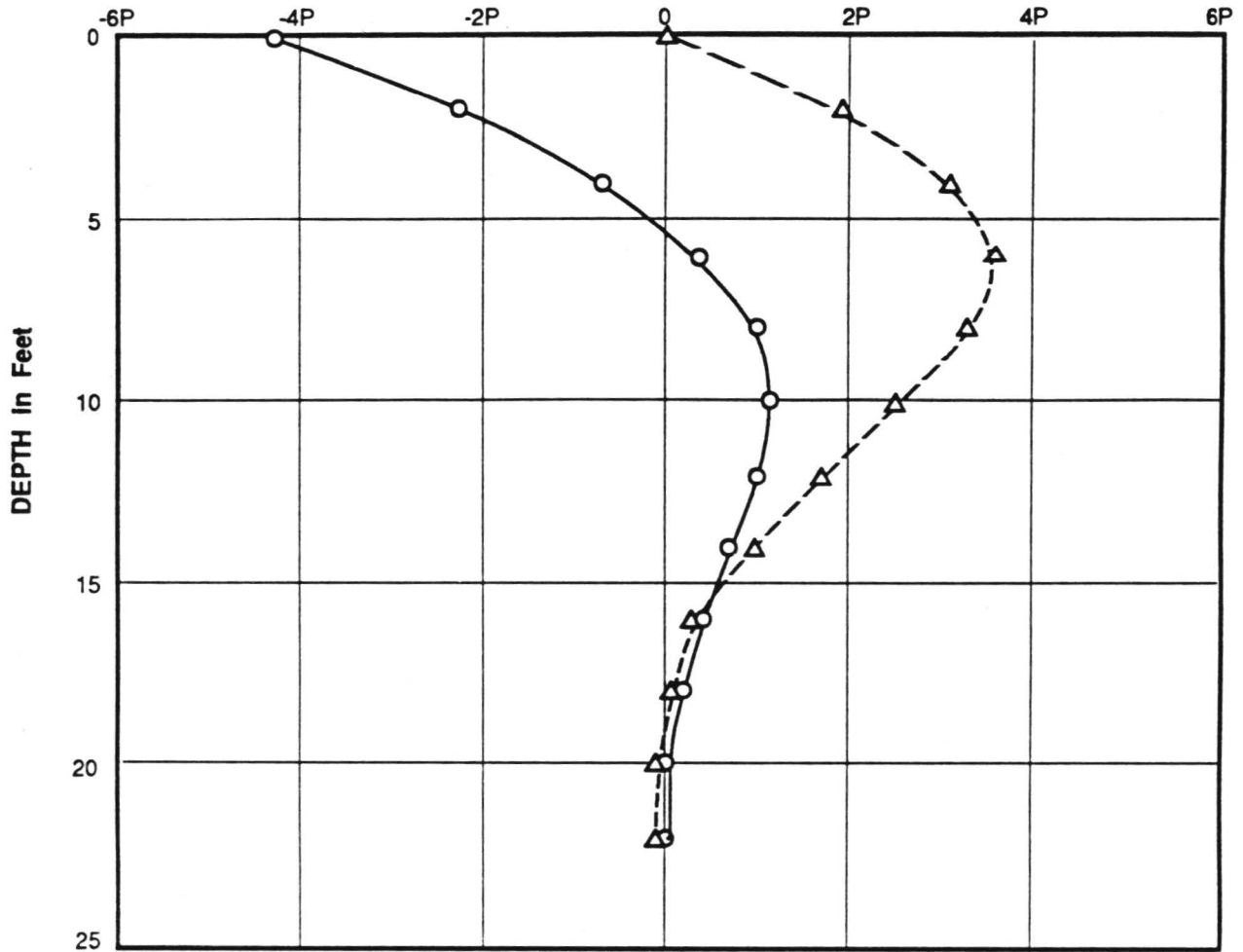
- FIXED HEAD PILE
- △--△ FREE HEAD PILE
- P LATERAL LOAD APPLIED AT TOP OF THE PILE IN KIPS

BENDING MOMENT DIAGRAMS
16 - INCH DIAMETER PILE
 (Design Group 2)



JOB L92045.AE4 DATE 2/22/93 DR. IK MS CHKD O.E.

BENDING MOMENT in Foot-Kips



KEY:

- FIXED HEAD PILE
- △--△ FREE HEAD PILE
- P LATERAL LOAD APPLIED AT TOP OF THE PILE IN KIPS

BENDING MOMENT DIAGRAMS 20 - INCH DIAMETER PILE (Design Group 2)

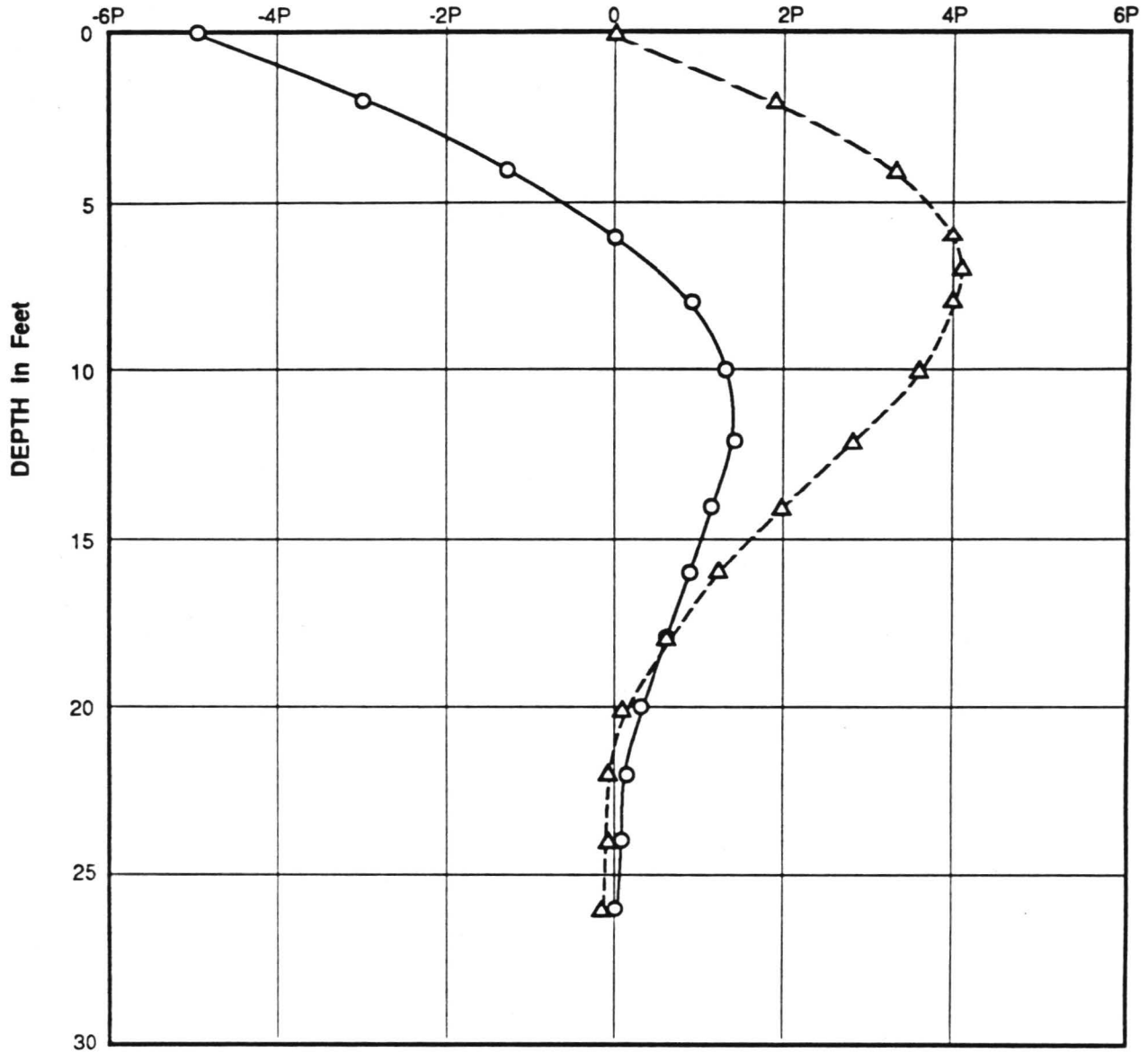
LAW / CRANDALL, INC.



FIGURE 8.13E

JOB L92045.AE4 DATE 2/22/93 DR. IK MS *MS* CHKD *46* O.E.

BENDING MOMENT in Foot-Kips



KEY:

○—○ FIXED HEAD PILE

△--△ FREE HEAD PILE

P LATERAL LOAD APPLIED AT TOP OF THE PILE IN KIPS

BENDING MOMENT DIAGRAMS

24 - INCH DIAMETER PILE

(Design Group 2)

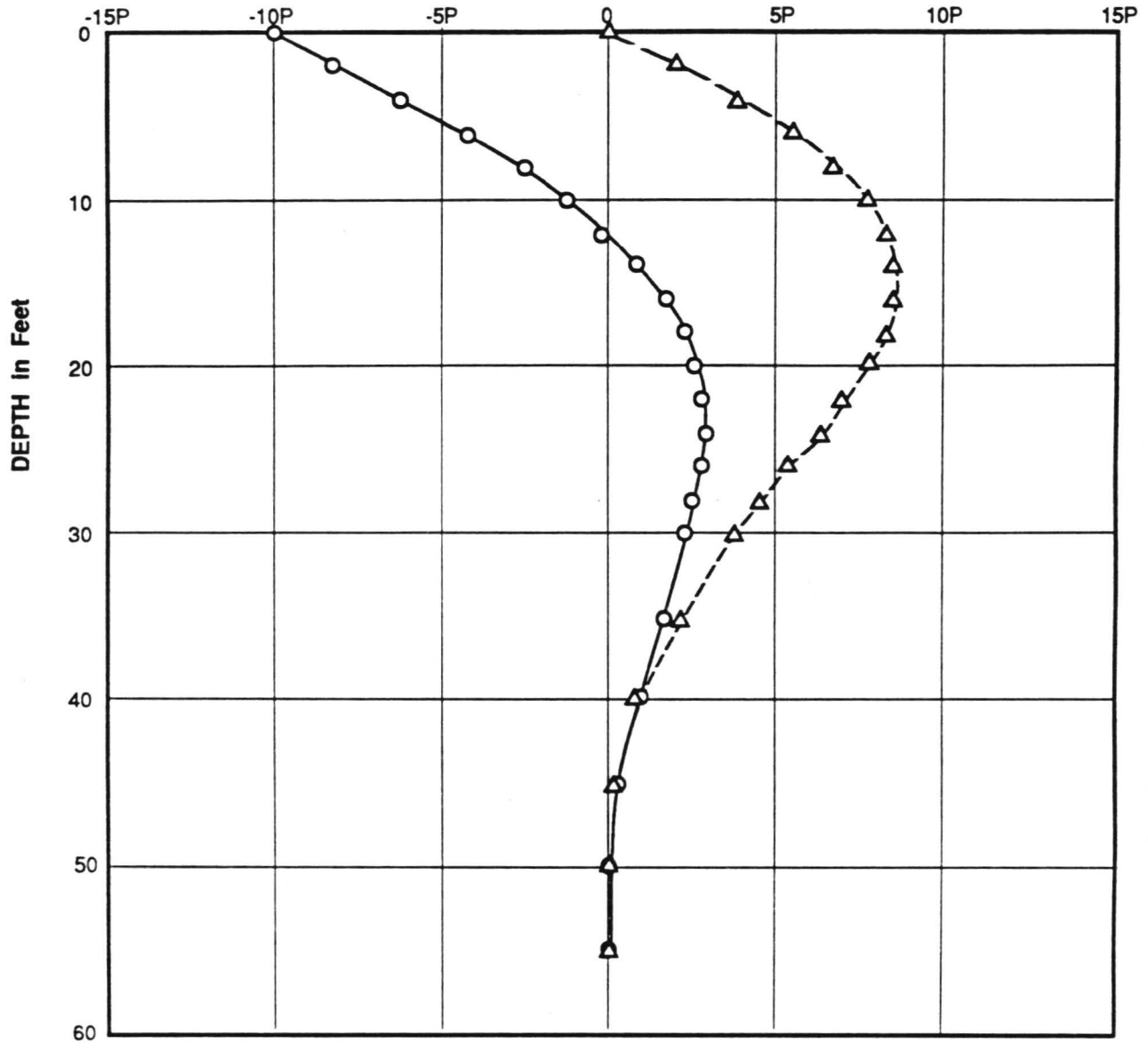
LAW / CRANDALL, INC.



FIGURE 8.13F

JOB L92045.AE4 DATE 2/22/93 DR. IK MS CHKD *MB*

BENDING MOMENT in Foot-Kips



KEY:

- FIXED HEAD PILE
- △--△ FREE HEAD PILE
- P LATERAL LOAD APPLIED AT TOP OF THE PILE IN KIPS

BENDING MOMENT DIAGRAMS

60 - INCH DIAMETER PILE

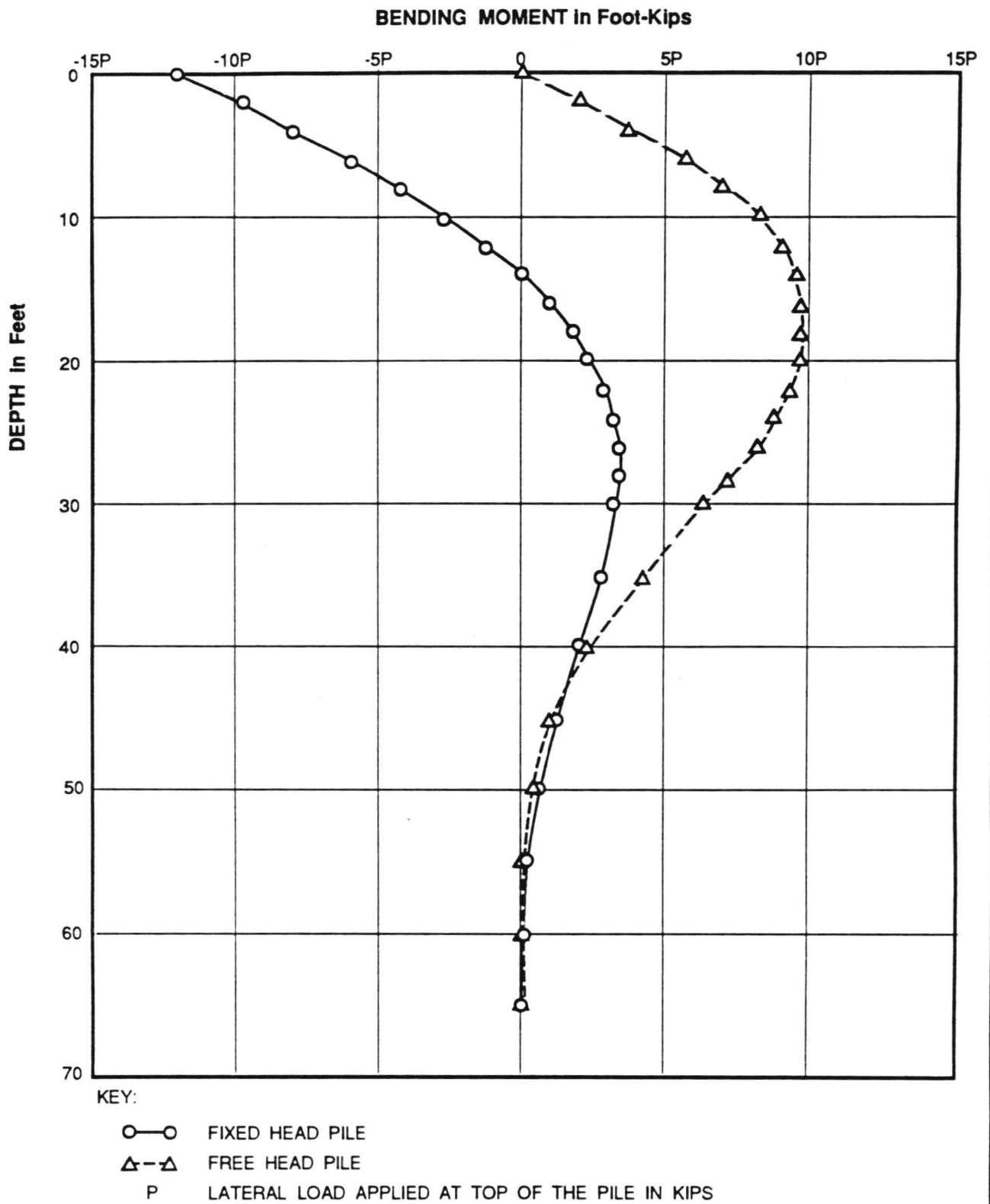
(Design Group 2)

LAW / CRANDALL, INC.



FIGURE 8.13G

JOB L92045.AE4 DATE 2/22/93 DR. IK O.E. MS *MS* CHKD *MS*



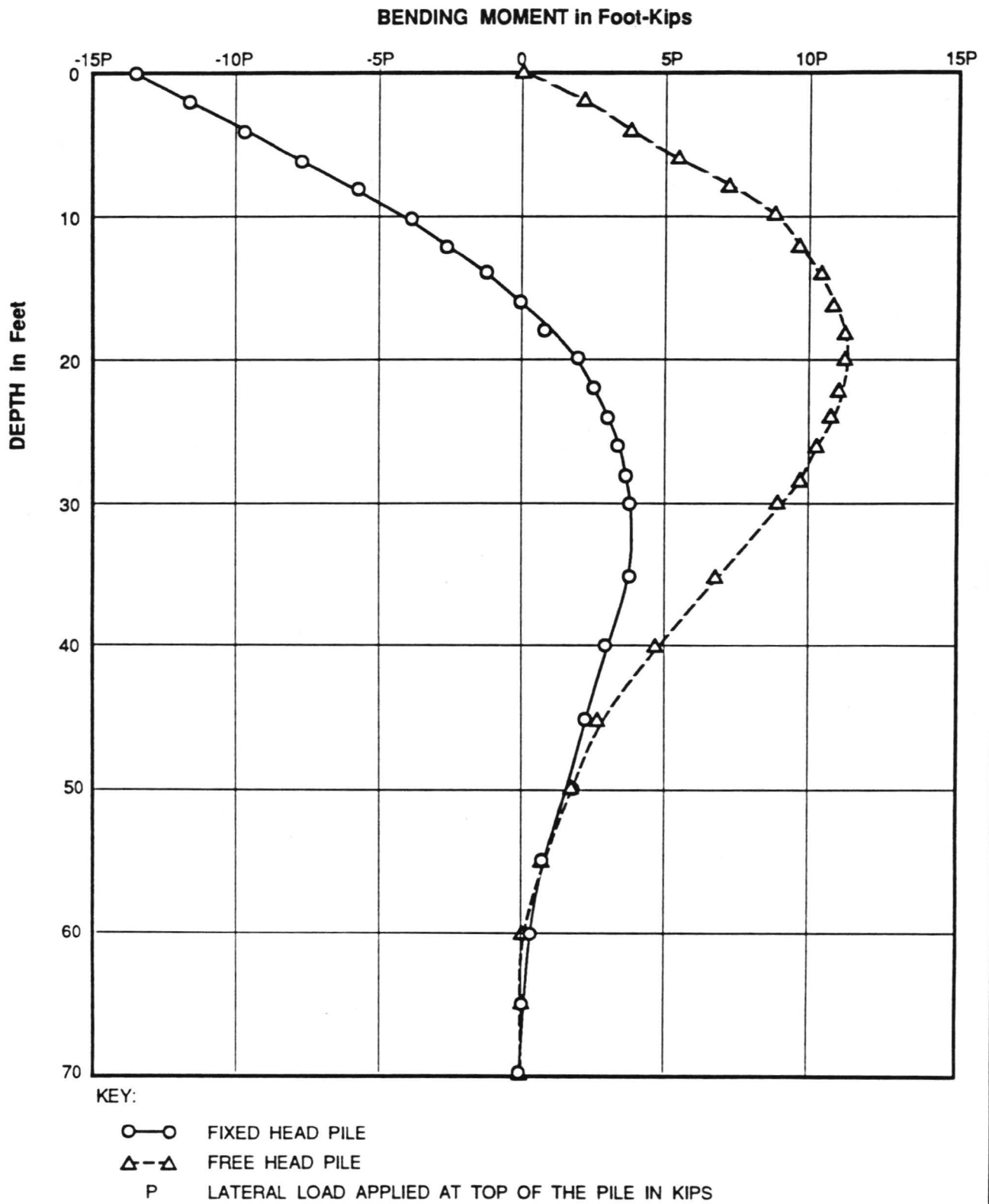
BENDING MOMENT DIAGRAMS
72 - INCH DIAMETER PILE
 (Design Group 2)

LAW / CRANDALL, INC.



FIGURE 8.13H

JOB L92045.AE4 DATE 2/22/93 DR. Ik MS *MS* CHKD *MS*



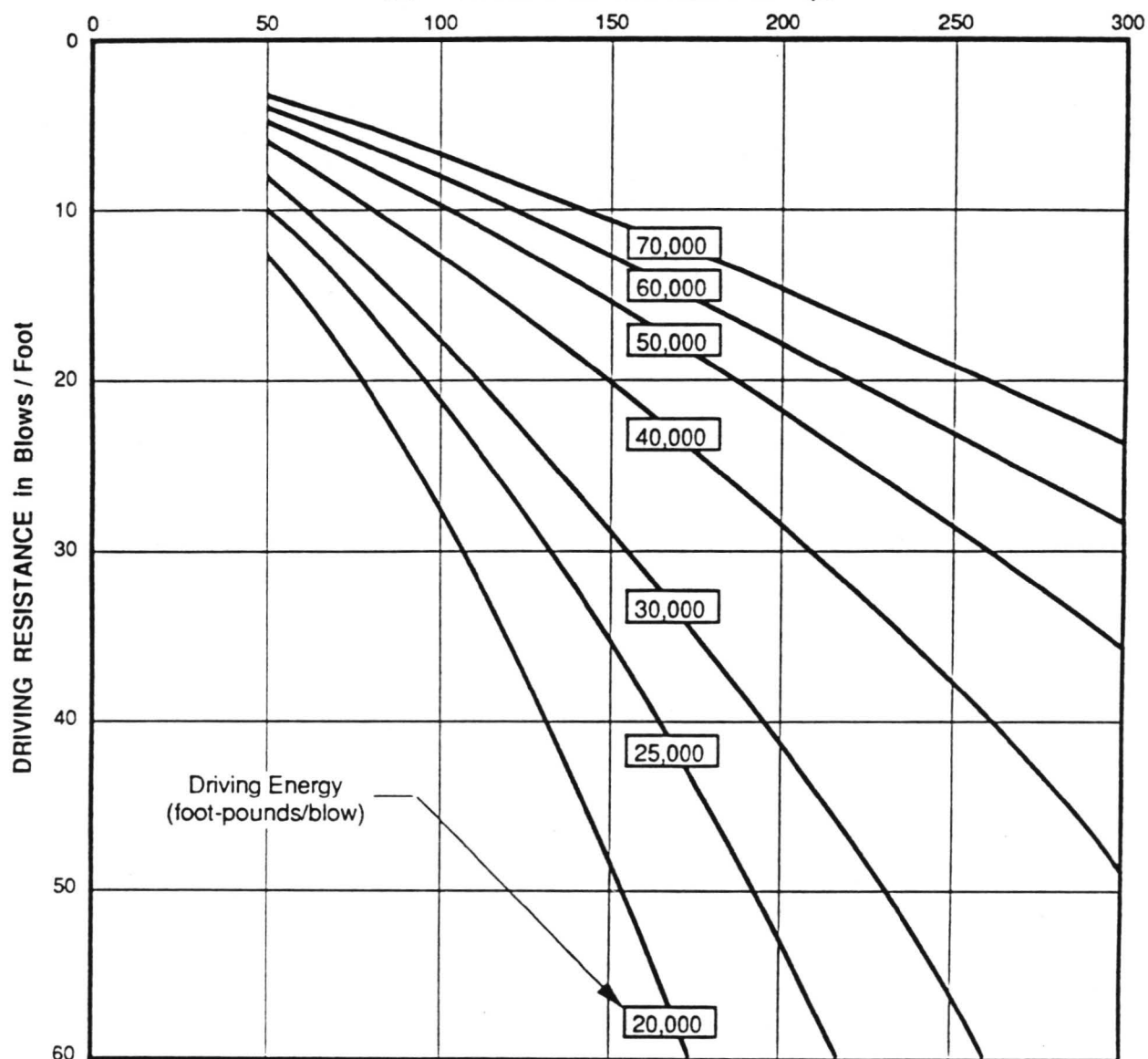
BENDING MOMENT DIAGRAMS
84 - INCH DIAMETER PILE
 (Design Group 2)

LAW / CRANDALL, INC.



FIGURE 8.131

DOWNWARD PILE CAPACITY in Kips



NOTES :

- (1) The above driving resistance should be obtained for the last foot of driving at design pile length. If the driving resistance at the design length is less than above, the piles should be lengthened until the desired driving resistance is obtained.
- (2) As an alternate to lengthening when low driving resistance is obtained, the piles may be allowed to set overnight and the number of blows to drive the pile one inch the following day determined. If the restarting resistance is at least two times the above criteria, the pile may be considered satisfactory.
- (3) If driving resistance of three times the above criteria is encountered within five feet of design length, the pile driving may be stopped.

PILE DRIVING CRITERIA

(Design Group 2)

LAW / CRANDALL, INC.



FIGURE 8.14

11/1/82

CHKD

MS

O.E.

DR.

2/23/93

DATE

192045 AE4

JOB

8.4 DESIGN GROUP 3 (*Avenue 19 Bridge; San Fernando Road Bridge; Golden State Freeway Bridge; Avenue 26 Bridge; Arroyo Seco Channel and Pasadena Freeway Bridge north of Pasadena Avenue; Pasadena Freeway Bridge near Fremont Avenue; bridges along the Foothill Freeway*).

8.4.1 SUBSURFACE PROFILE AND SOIL PROPERTIES

Borings 12, 23, 48, 49, 50, 52 through 54, and 82 were drilled for the aerial structures in this design group. Logs of the borings are presented in Appendix A, Volume 2. Based on the borings, the overburden soils basically consist of silty sand, sand, and gravel with considerable amounts of cobbles and boulders, and lesser silt deposits. Bedrock was encountered at a depth of about 97 feet in Boring 53; bedrock was not encountered in the other borings that extended to depths of up to 101 feet below the existing grade. Ground water was measured at depths of 38 to 60 feet below the existing grade.

The following soil properties may be used for static design for Design Group 3:

Cohesion	100 psf
Void Ratio Granular Soils	0.45
Modulus of Subgrade Reaction Granular Soils	200 pci
Angle of Internal Friction	39 degrees
Moist Unit Weight of Soil 0 to 30 feet Below 30 feet	120 pcf 60 pcf

8.4.2 FOUNDATIONS

8.4.2.1 Pile Foundations

Vertical Loads - Drilled Piling: The downward and upward frictional capacities of 16-, 20-, 24-, 60-, 72-, and 84-inch-diameter drilled piles are presented on Figures 8.15 and 8.16, Drilled Pile Capacities, Design Group 3. The capacities of other sizes of piles may be taken as proportional



to the diameter. Dead plus live load capacities are shown; a one-third increase may be used for wind or seismic loads. The dead plus live capacities are allowable capacities; a factor of safety of 2 was used to determine the allowable capacities. If piles in groups are required, the piles should be spaced not less than $2\frac{1}{2}$ diameters on centers so that group action need not be considered. The capacities are based on the strength of the soils; the compressive and tensile strength of the pile section itself should be checked to verify the structural capacity of the pile.

Settlement – Drilled Piles: The settlement of drilled piles will be less than $\frac{1}{2}$ inch for vertical loads between 150 and 3,500 kips. For example, the settlement of a 16-inch-diameter pile will be less than $\frac{1}{4}$ inch, and the settlement of an 84-inch-diameter pile supporting a vertical load of 3,500 kips will be less than $\frac{1}{2}$ inch. Because of the granular characteristics of the underlying soils, the majority of settlement will occur during construction and immediately after application of structural loads.

Vertical Loads - Driven Piles: The downward and upward frictional capacities of 12-, 14-, and 16-inch-square prestressed concrete piles are presented on Figure 8.17, Driven Pile Capacities, Design Group 3. If batter piles are used, only the vertical projection of the length of the batter pile should be considered in computing the pile capacities. Dead plus live load capacities are shown; a one-third increase may be used for wind or seismic loads. The dead plus live capacities are allowable capacities computed with a factor of safety of at least 2. Piles in groups should be spaced not less than $2\frac{1}{2}$ pile widths or 3 feet on centers, whichever is greater, so that group action need not be considered. The capacities are based on the strength of the soils; the compressive and tensile strength of the pile section itself should be checked to verify the structural capacity of the pile.

Settlement – Driven Piles: The settlement of 12-, 14-, and 16-inch-square prestressed concrete piles, supporting vertical loads up to 300 kips, will be about $\frac{1}{2}$ inch or less. Because of the



granular characteristics of the underlying soils, the majority of the settlement will occur during construction and immediately after application of structural loads.

Lateral Resistance: The piles may be used to resist lateral loads. The lateral capacity of the piles will depend on the permissible deflection and on the degree of fixity at the top of the pile. The lateral deflection in the piles (in terms of the applied load at the top of the pile) is presented on Figures 8.18A through 8.18I, Lateral Deflection Diagrams, Design Group 3. The lateral capacity of a batter pile is equivalent to the horizontal component of the downward pile capacity.

The soil resistance versus pile deflection for the piles is shown on Figures 8.19A through 8.19J, (P-Y) Curves, Design Group 3. These curves were developed using the computer program LPILE distributed by Ensoft, Inc., Austin, Texas. These (P-Y) curves should only be used in LPILE or similar computer program to determine the behavior of a laterally loaded pile.

The bending moment in the piles due to application of the lateral loads may be distributed in the piles in accordance with Figures 8.20A through 8.20I, Bending Moment Diagrams, Design Group 3.

The lateral capacity and bending moment distributions are based in part on the assumption that any backfill adjacent to the piles will be properly compacted. Furthermore, if the pile locations are predrilled, the backfill in the annular space around the pile should be tight to assure good distribution of load into the surrounding soils. A pile in a pile group of two or more with a pile cap may be considered as a fixed-head pile.

The passive resistance of properly compacted backfill against pile caps may be assumed to be equal to the pressure developed by a fluid with a density of 300 pounds per cubic foot. A one-third increase may be used when considering wind or seismic loads.



Ultimate Design: The various design values recommended for piling are for use with loadings determined by a conventional working stress design. If an ultimate design concept is used, the recommended design values may be multiplied by the following factors:

Design Item	Ultimate Design Factor
Axial Capacity of Piles	2.0
Lateral Capacity of Piles	1.5
Passive Resistance	1.25

In no event, however, should the pile lengths be reduced from those required for support of dead plus live loads when using the working stress values.

Installation - Drilled Piling: As previously mentioned, the alignment is underlain by geologic units consisting of artificial fill, alluvial, and terrace deposits, underlain by sedimentary bedrock of the Puente and Topanga formations. There are significant amounts of sand, gravel, and cobbles within these geologic formations. Caving of boring walls occurred in some of the borings drilled with bucket auger-type drilling equipment. A significant percentage of prior borings drilled by our firm for other projects in the vicinity with bucket auger-type drilling equipment also experienced significant caving.

The bucket auger exploration borings were drilled typically with relatively small-diameter equipment, ranging from 16 to 24 inches in diameter. It should be noted that the caving potential increases as the diameter of the excavation increases. The sandy soils are relatively cohesionless, and may be held together by small capillary tension forces. If allowed to dry, the soils would lose that capillary tension and would be more likely to cave. The use of high speed drilling equipment would also cause disturbance of the soils, increasing the likelihood of caving. The presence of ground water would also increase the likelihood of caving.



It should be anticipated that significant caving, raveling, and sloughing may occur during excavation of the large-diameter drilled piles, particularly below the water, unless positive steps are taken to support the sides of the hole. Special techniques will likely be needed to satisfactorily install the piles. Casing may or may not be required, depending on the field conditions.

Only drilling contractors with demonstrated experience in the installation of drilled cast-in-place large-diameter piles using drilling mud and casing in similar circumstances should be considered for the pile work. The drilling system should be equipped to include the use of casing and drilling mud to prevent shaft caving during drilling and placing of concrete. The equipment should provide for the introduction of drilling mud of sufficient specific gravity to retain the shaft sides, and continuous pumping for retrieval of the mud concurrent with concrete placement. The pressure head of mud in the shaft should be sufficient to maintain a stable bottom during all construction operations. Only environmentally safe drilling mud should be used.

The contractor should provide the equipment necessary to check the bearing, dimensions, and alignment of each drilled shaft.

Drilling should be performed in a sequence such that no adjacent pile closer than 5 pile diameters on center is drilled sooner than eight hours after placement of the concrete.

Care should be exercised during placement of the reinforcement steel to minimize disturbance and caving of the sides of any uncased portions of the drilled shaft.

Concrete should be pumped through a tremie pipe extending to the bottom of the shaft, with the pipe being at least 8 feet below the surface of the concrete at all times during placement. The discharge end of the tremie should be constructed so that it is watertight against intrusion from the outside and so that it will readily discharge concrete. The tremie pipe should be



supported so that it can be raised and lowered rapidly should it become necessary to cut off or retard the flow. Casing should be pulled as the concrete is placed; the bottom of the casing, however, should be maintained at least 3 feet below the surface of the concrete at all times. The discharge pipe should be kept full of concrete during the entire placing operation and should not be removed from the concrete until all of the concrete is placed and fresh concrete appears at the top of the pile. The volume of concrete pumped into the hole should be recorded and compared to the design volume.

We recommend continuous observation of the drilling and pouring of the piles be performed by a geotechnical engineer or an engineering geologist licensed in the State of California to verify that the desired diameter and depth are obtained.

Each pile should be tested by a nondestructive testing method to check the pile's integrity. The testing should be performed after at least three days of curing time has elapsed to allow the concrete to harden sufficiently. The nondestructive testing should be undertaken by an experienced and approved independent testing organization.

One of the nondestructive methods that has been successfully used for large diameter piles, is called crosshole sonic logging (CSL). This method measures the time it takes for an ultrasonic pulse to travel from a signal source in one access tube placed within a pile to a receiver in another access tube placed within the same pile. In unreinforced quality concrete, the travel time between the two tubes will be relatively constant, and the CSL test will produce records with good signal amplitude. Longer travel times and lower amplitude/energy signals indicate the presence of irregularities such as poor quality concrete, voids, and soil intrusions. The signal will be completely lost by the receiver for more severe defects such as large voids and soil intrusions.

Steel tubes are recommended because there will be good bonding between steel and concrete, allowing for better transmission of the ultrasonic pulses. Plastic or PVC pipes would be more



economical; however, our experience with PVC pipes indicates that certain amounts of debonding occur between the PVC pipe and the concrete after the concrete sets; this is especially the case near the top of the piles. The debonding could lead to erroneous, misleading, or inconclusive readings, thus requiring further testing that may offset any savings in the cost of pipe material. To minimize the debonding, if PVC pipe is used, the tubes should be filled with water as soon as possible after the concrete is poured, preferably on the same day.

Installation - Driven Piling: All piles should be driven to the predetermined design lengths as shown on Figure 8.17, except as may be modified on the basis of the driving criteria defined on Figure 8.21, Pile Driving Criteria. The pile driving criteria shown on Figure 8.21 may be taken only as a guide. Prior to ordering production piles, we recommend that indicator piles be driven to evaluate the driving resistance. At least three indicator piles should be driven at each abutment and/or bridge pier location. The driving criteria and installation procedures can be modified based on the results of the indicator pile and any necessary adjustments can be made to the design lengths presented on Figure 8.17. The installation of the piling should be observed by a geotechnical engineer licensed in the State of California so that modifications in the driving criteria and the pile lengths can be made if required.

Hard driving may be encountered within the sandy soils and predrilling may be required. The diameter of the predrilled hole should not exceed 80% of the pile diameter. Predrilling may be performed to within 5 feet of the design tip elevation of the pile.

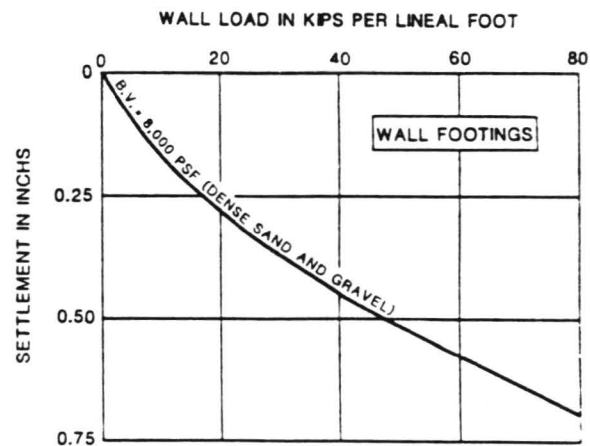
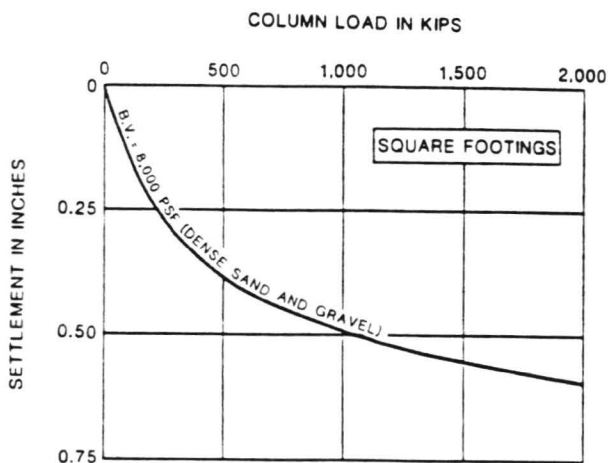


8.4.2.2 Spread Footings

Bearing Value: Spread footings within Design Group 3 may impose bearing pressures up to 8,000 pounds in the dense sand and gravel.

Footings should extend at least 1 foot into the supporting material and should be established at least 3 feet below the adjacent final grade. A one-third increase in the recommended values may be used for wind or seismic loads. The recommended bearing values are net values. The weight of concrete in the footings may be assumed to be 50 pounds per cubic foot, and the weight of soil backfill may be neglected when determining the downward load on the footings.

Settlement: The settlement of spread footings established in the dense sand and gravel may be determined using the following graphs.



Lateral Loads: Lateral loads may be resisted by soil friction and by the passive resistance of the soils. A coefficient of friction of 0.5 may be used between footings and the supporting material. A passive value of 300 pounds per square foot per foot of depth may be used. A one-third increase in the passive value may be used for wind or seismic loads. The frictional resistance and the passive resistance of the soils may be combined without reduction in determining the total lateral resistance.

Ultimate Design: The recommended bearing value and lateral load design values are for use with loadings determined by a conventional working stress design. When considering an ultimate design approach, the recommended design values may be multiplied by the following factors:

Design Item	Ultimate Design Factor
Bearing Value	2.5
Passive Pressure	1.5
Coefficient of Friction	1.25

In no event, however, should foundation sizes be less than those required for dead plus live loads when using the working stress design.

Footing Observation: The footing excavations should be observed by a geotechnical engineer or an engineering geologist licensed in the State of California to verify the presence of satisfactory materials at footing design elevations. Footings should be deepened if necessary to extend into satisfactory supporting material.

The foundations should be left slightly uneven if necessary, rather than filling in overexcavated areas with loose or compacted soils. Where it is necessary to deepen a footing below the design depth, the overexcavated portion should be backfilled with concrete. It would be desirable and feasible to pour spread footings neat against the undisturbed soil. Soil backfill should be compacted as recommended in Section 13.0, Earthwork.



Subsurface seepage may be encountered in the footing excavations. A layer of lean concrete may be used to protect the bottom of the excavations and to provide support during construction.

8.4.3 ABUTMENT WALLS

For design of the cantilevered abutment walls, where the backfill is level and properly drained, it may be assumed that the soils will impose an active lateral pressure equal to that developed by a fluid with a density of 35 pounds per cubic foot. For walls with backfill sloped as steep as 2:1 (horizontal to vertical), an equivalent fluid pressure of 53 pounds per cubic foot may be used. An appropriate uniform lateral pressure equal to one-third of the anticipated vertical surcharge within 20 feet of the abutment walls should be included in the wall design.

The above values are for properly drained soils; weep holes should be provided to relieve hydrostatic pressures due to water.

8.4.4 SEISMIC DESIGN DATA

The proposed aerial structures will be designed in accordance with the bridge design criteria of the State of California Department of Transportation (CALTRANS). The geotechnical information needed for the CALTRANS criteria is the depth to rock and the expected peak bedrock acceleration. Based on a review of the soils and geologic data, the thickness of alluvium may be taken as being between 80 to 150 feet for all the bridges in Design Group 3, except for the Pasadena Freeway Bridge near Fremont Avenue, and for all the bridges along the Foothill Freeway. For these bridges, the thickness of alluvium may be taken as being more than 150 feet. The expected peak bedrock acceleration is 0.6g for all the bridges in this design group within the Cities of Los Angeles and South Pasadena, based on the proximity of the nearby faults. For all bridges within the City of Pasadena along the Foothill Freeway, the expected peak ground acceleration may be taken as 0.65g. In addition, the following values may be used in the seismic design of the aerial structures:



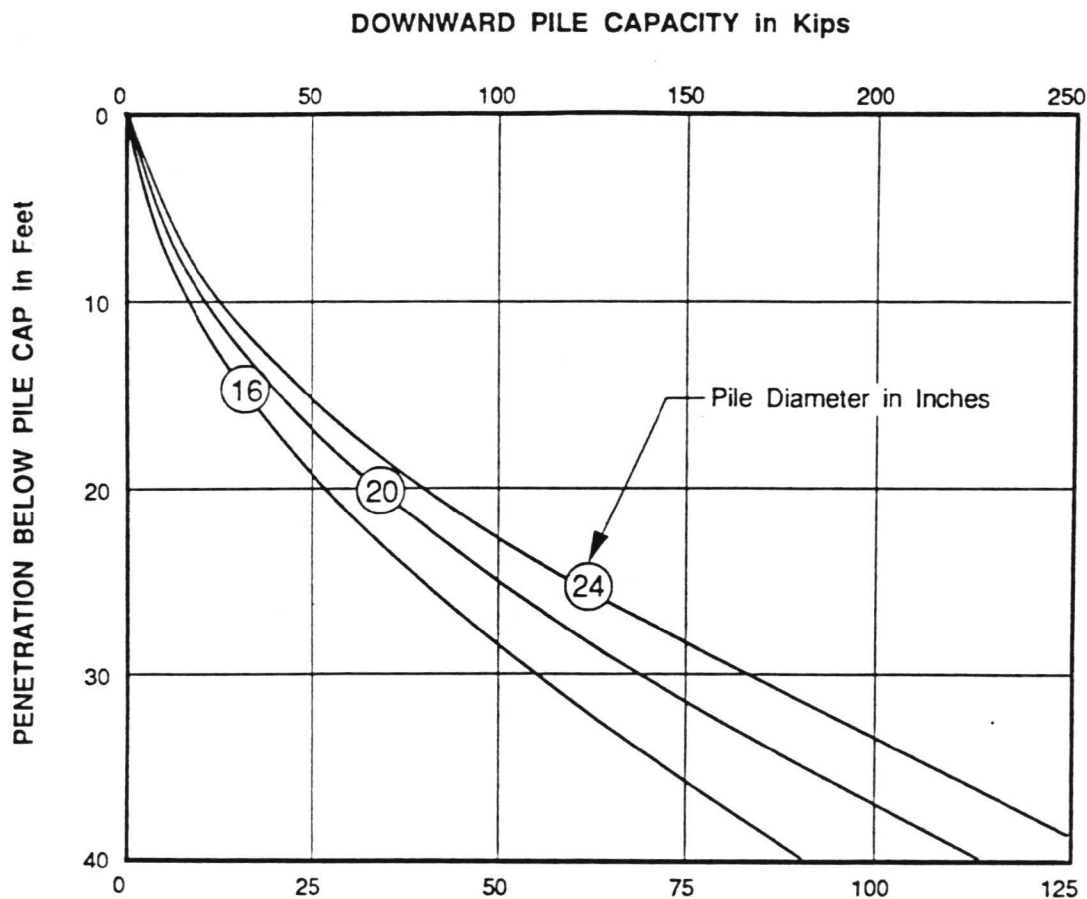
Shear Modulus 0 to 25 feet 25 to 75 feet Below 75 feet	2,400 ksf 8,400 ksf 14,900 ksf
Poisson's Ratio	0.3
Damping Value	0.1

The following figures complete the recommendations for Design Group 3:

Figure No.	Title
8.15 – 8.16	Drilled Pile Capacities, Design Group 3
8.17	Driven Pile Capacities, Design Group 3
8.18A – 8.18I	Lateral Deflection Diagrams, Design Group 3
8.19A – 8.19J	(P-Y) Curves, Design Group 3
8.20 A – 8.20I	Bending Moment Diagrams, Design Group 3
8.21	Pile Driving Criteria, Design Group 3



JOB L92045.AE4 DATE 2/15/93 DR. MS CHKD MS O.E. MS



UPWARD PILE CAPACITY in Kips

NOTES :

- (1) The indicated values refer to the total of dead plus live loads; a one-third increase may be used when considering wind or seismic loads.
- (2) Piles in groups should be spaced a minimum of 2-1/2 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.

DRILLED PILE CAPACITIES

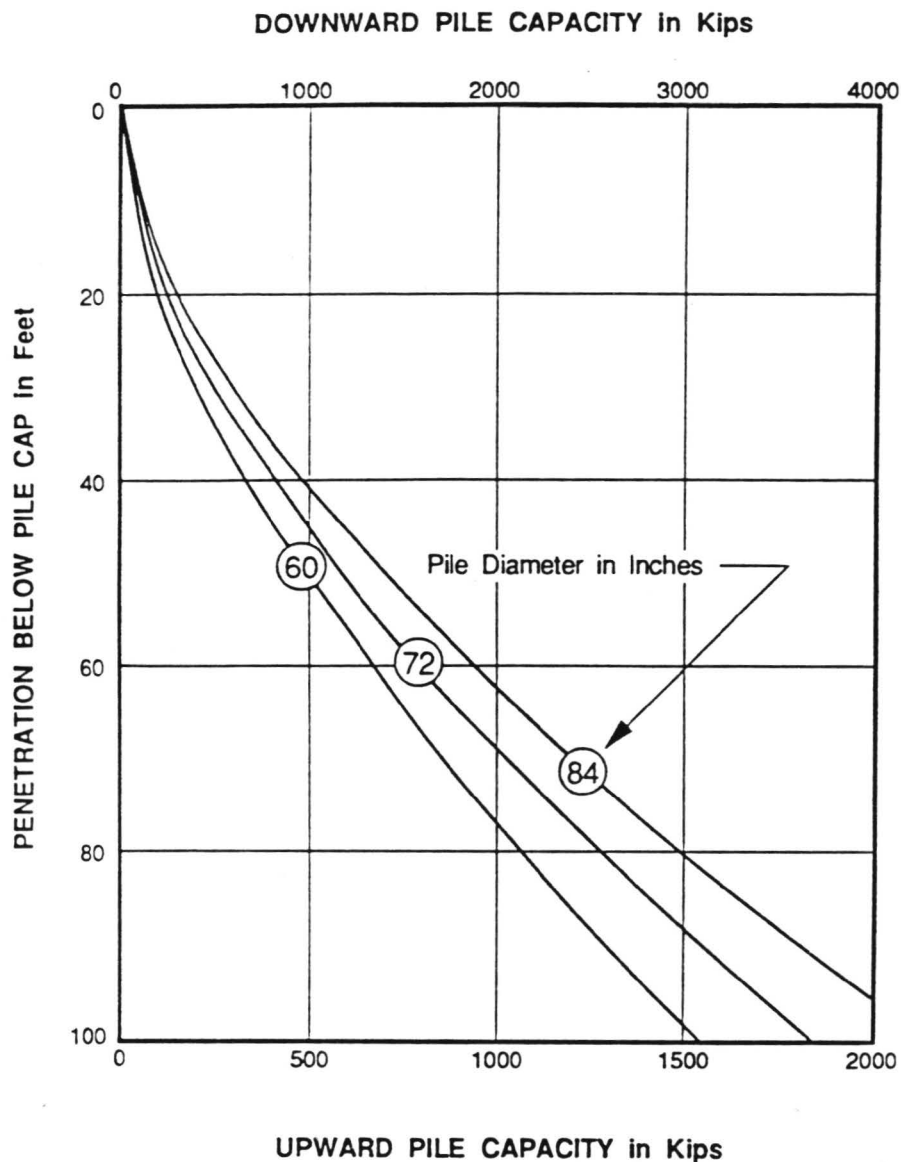
(Design Group 3)

LAW / CRANDALL, INC.



FIGURE 8.15

JOB L92045 AE4 DATE 2/15/93 DR. MS O.E. CHKD



NOTES :

- (1) The indicated values refer to the total of dead plus live loads; a one-third increase may be used when considering wind or seismic loads.
- (2) Piles in groups should be spaced a minimum of 2-1/2 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.

DRILLED PILE CAPACITIES

(Design Group 3)

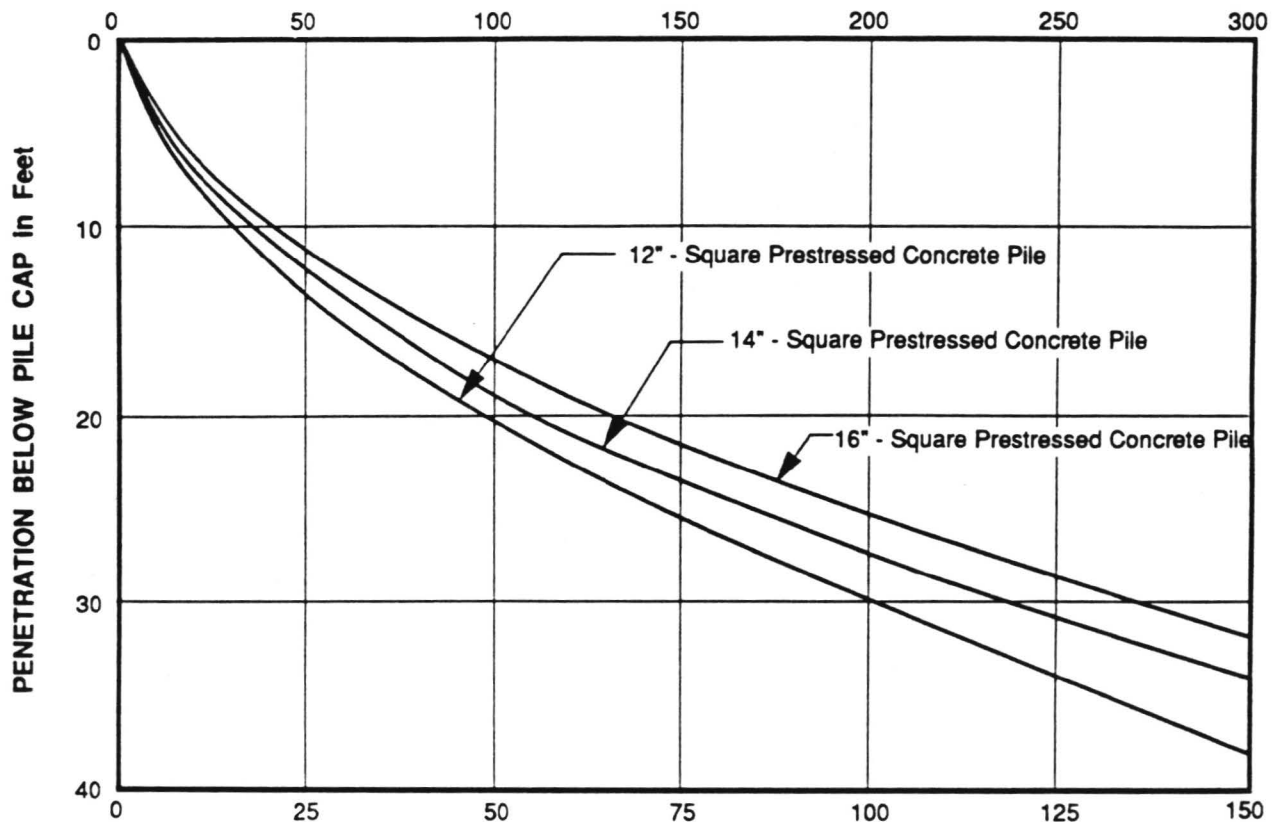
LAW / CRANDALL, INC.



FIGURE 8.16

JOB L92045 AE4 DATE 2/15/93 DR. IK O.E. MS CHKD

DOWNWARD PILE CAPACITY in Kips



UPWARD PILE CAPACITY in Kips

NOTES :

- (1) The indicated values refer to the total of dead plus live loads; a one-third increase may be used when considering wind or seismic loads.
- (2) Piles in groups should be spaced a minimum of 3 feet on centers.
- (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.

DRIVEN PILE CAPACITIES

(Design Group 3)

LAW / CRANDALL, INC.



FIGURE 8.17

MS

CHKD

YK

O.E.

IK

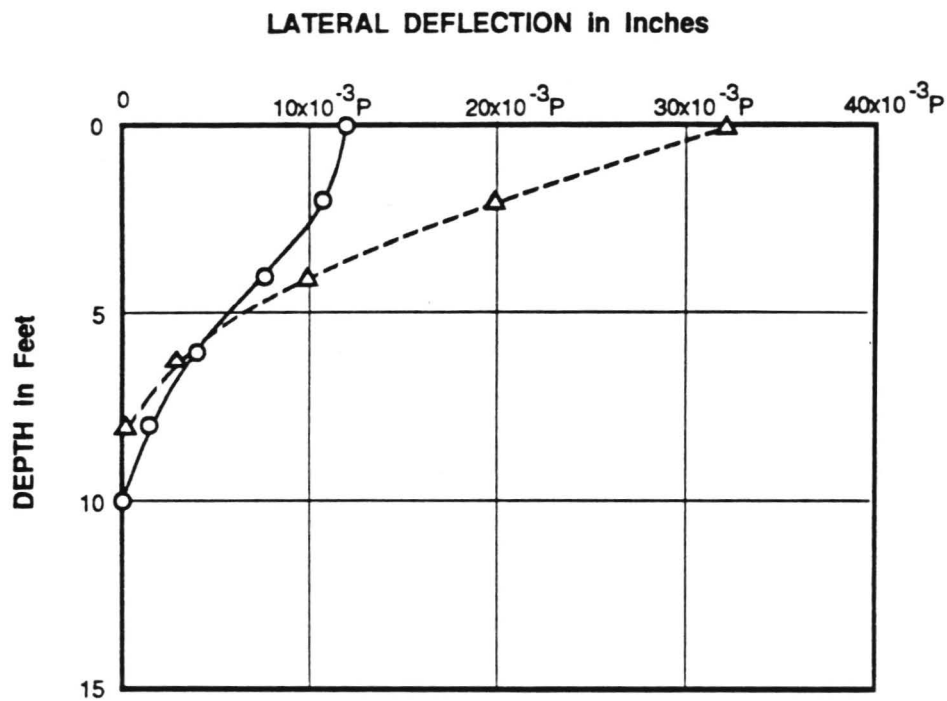
DR.

4/29/93

DATE

L92045.AE4

JOB



KEY:



FIXED HEAD PILE



FREE HEAD PILE

P

LATERAL LOAD APPLIED AT TOP OF THE PILE IN KIPS

LATERAL DEFLECTION DIAGRAMS

12 - INCH SQUARE PILE

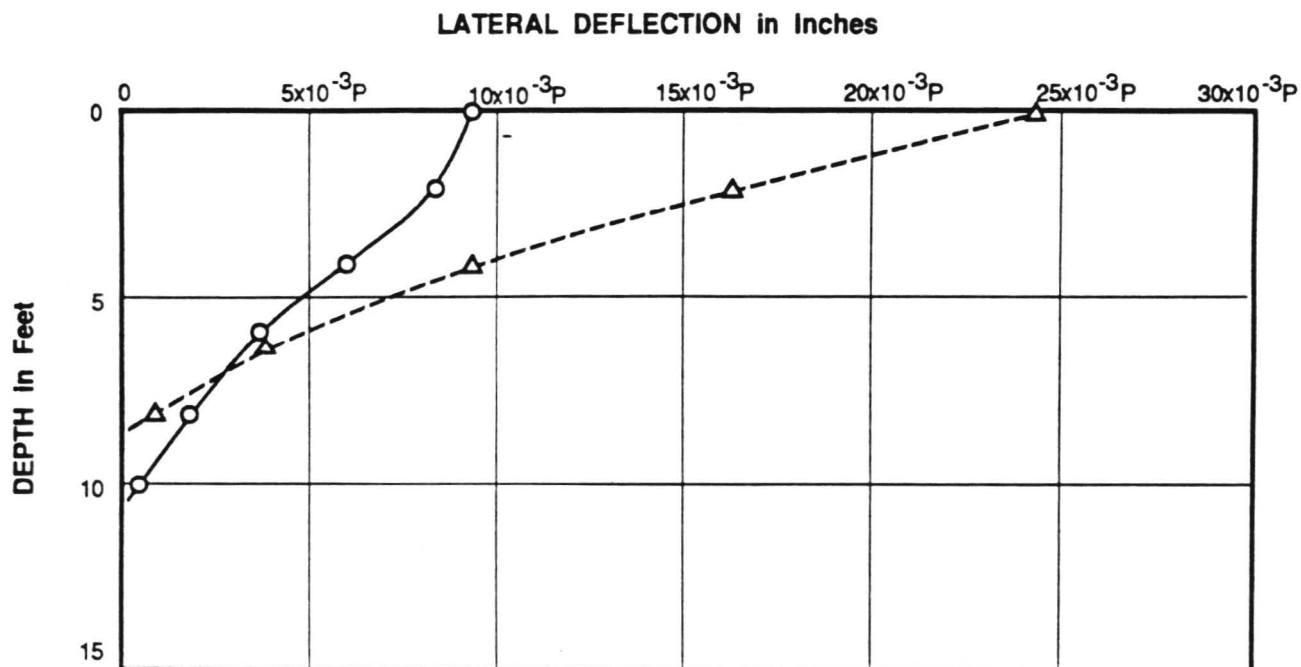
(Design Group 3)

LAW / CRANDALL, INC.



FIGURE 8.18A

145'
CHKD
YK
O.E.
K
DR.
4/29/93
DATE
L92045.AE4
JOB



KEY:

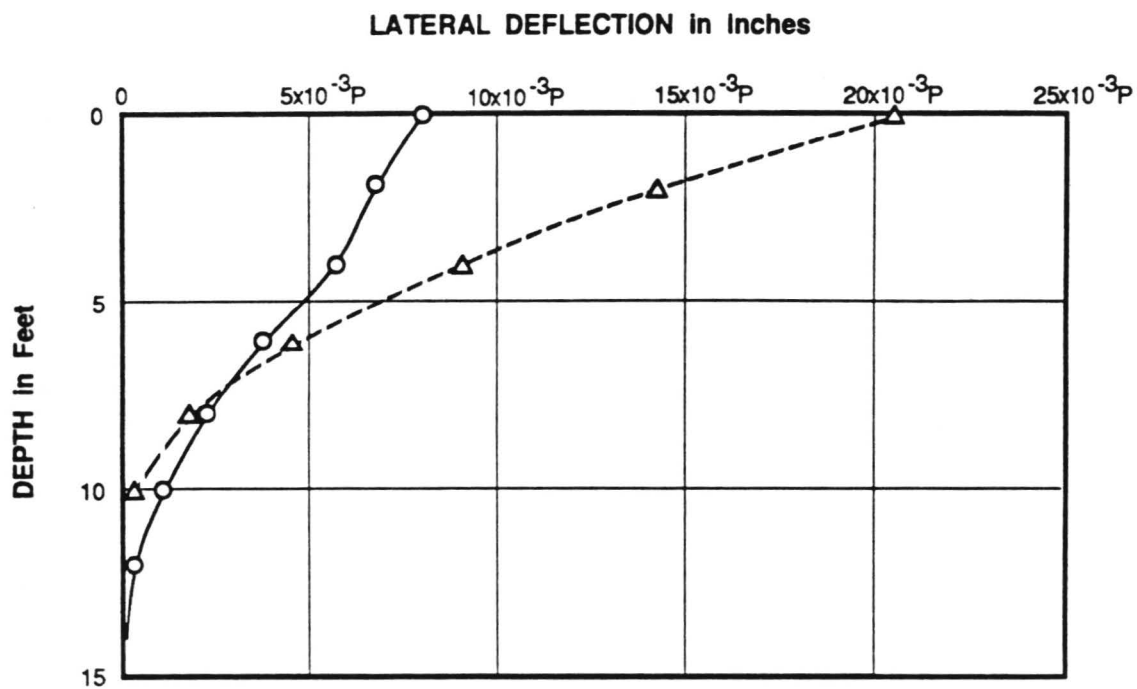
- FIXED HEAD PILE
- △--△ FREE HEAD PILE
- P LATERAL LOAD APPLIED AT TOP OF THE PILE IN KIPS

LATERAL DEFLECTION DIAGRAMS
14 - INCH SQUARE PILE
(Design Group 3)

LAW / CRANDALL, INC.

FIGURE 8.18B

JOB L92045.AE4 DATE 4/29/93 DR. MS O.E. YK CHKD



KEY:

- FIXED HEAD PILE
- △--△ FREE HEAD PILE
- P LATERAL LOAD APPLIED AT TOP OF THE PILE IN KIPS

LATERAL DEFLECTION DIAGRAMS
16 - INCH SQUARE PILE
 (Design Group 3)

LAW / CRANDALL, INC.



FIGURE 8.18C

MS

CHKD

YK

O.E.

IK

DR.

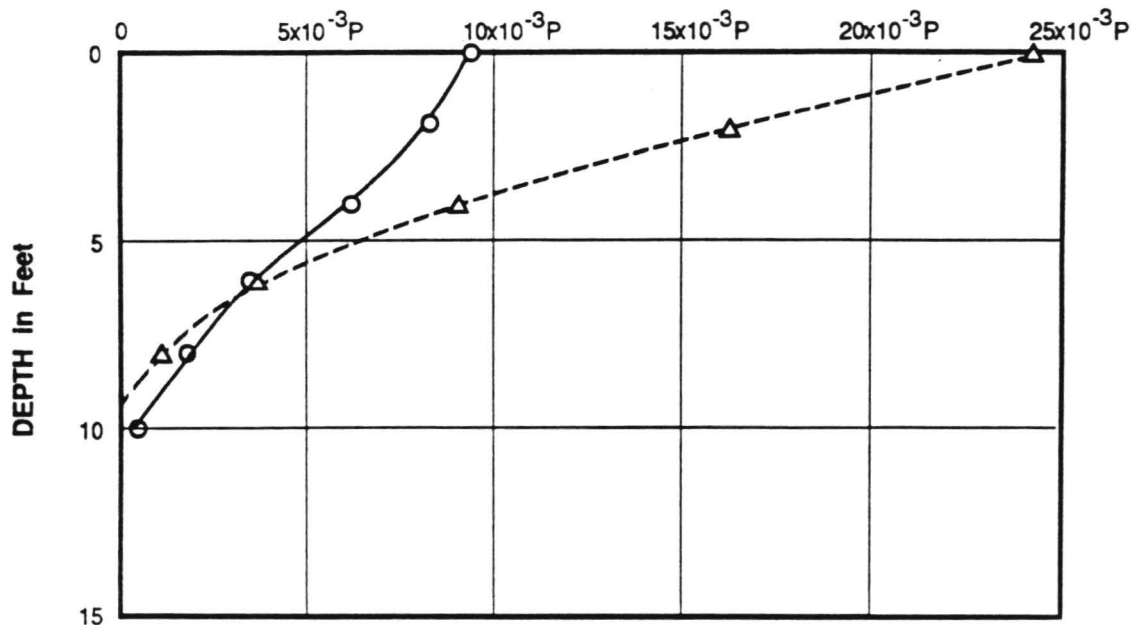
4/29/93

DATE

L92045,AE4

JOB

LATERAL DEFLECTION in Inches



KEY:

- FIXED HEAD PILE
- △--△ FREE HEAD PILE
- P LATERAL LOAD APPLIED AT TOP OF THE PILE IN KIPS

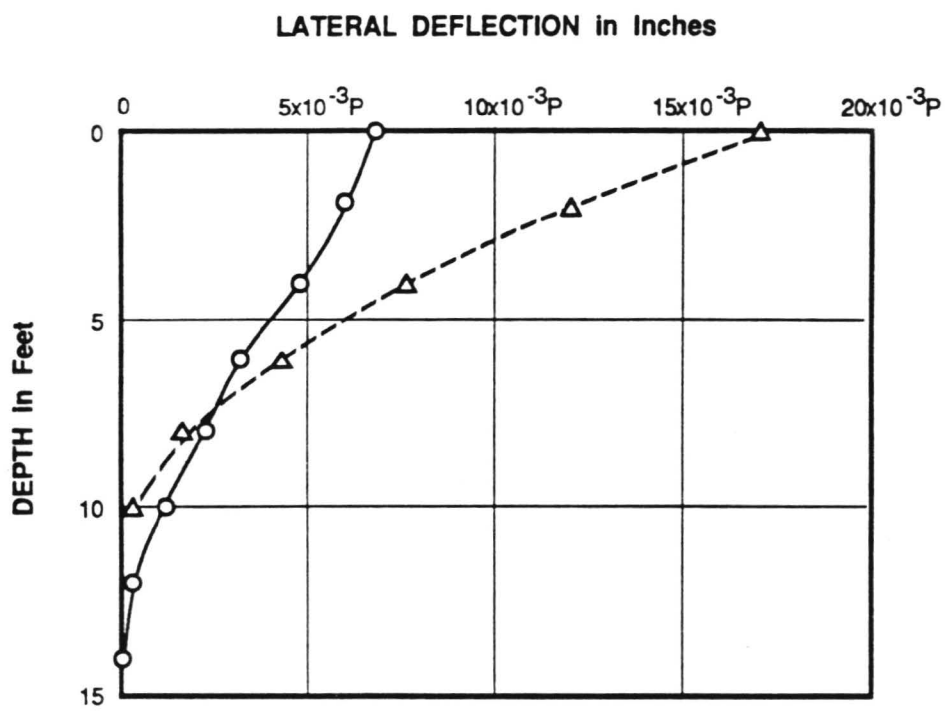
LATERAL DEFLECTION DIAGRAMS 16 - INCH DIAMETER PILE (Design Group 3)

LAW / CRANDALL, INC.



FIGURE 8.18D

JOB L92045.AE4 DATE 4/29/93 DR. K O.E. YK CHKD
HS



KEY:
 ○—○ FIXED HEAD PILE
 △--△ FREE HEAD PILE
 P LATERAL LOAD APPLIED AT TOP OF THE PILE IN KIPS

LATERAL DEFLECTION DIAGRAMS
20 - INCH DIAMETER PILE
 (Design Group 3)

MS

CHKD

YK

O.E.

IK

DR.

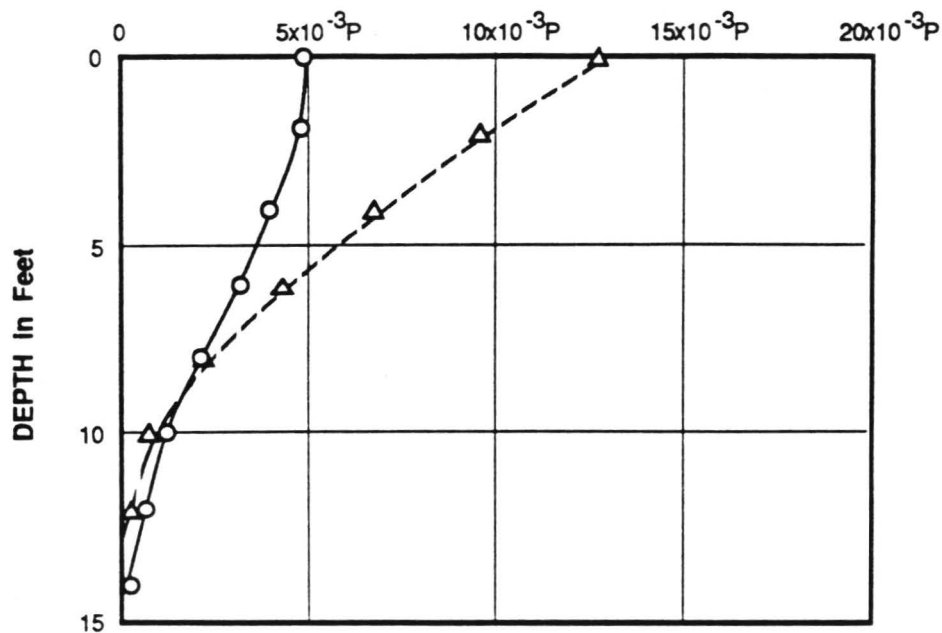
4/29/93

DATE

L92045.AE4

JOB

LATERAL DEFLECTION in Inches



KEY:



FIXED HEAD PILE



FREE HEAD PILE

P

LATERAL LOAD APPLIED AT TOP OF THE PILE IN KIPS

LATERAL DEFLECTION DIAGRAMS

24 - INCH DIAMETER PILE

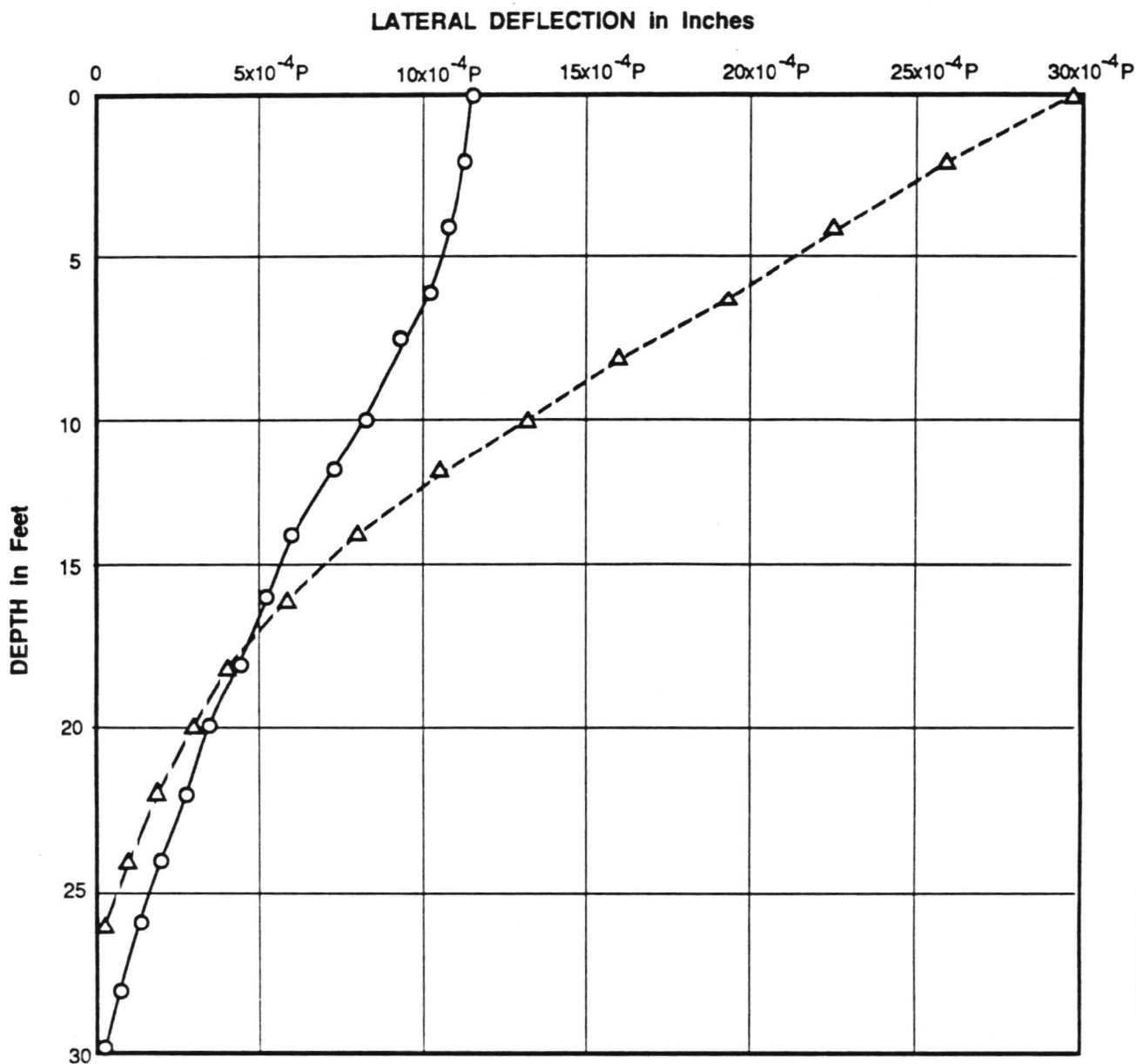
(Design Group 3)

LAW / CRANDALL, INC.



FIGURE 8.18F

JOB L92045.AE4 DATE 4/29/93 DR. IK O.E. YK CHKD *MS*



KEY:

○—○ FIXED HEAD PILE

△--△ FREE HEAD PILE

P LATERAL LOAD APPLIED AT TOP OF THE PILE IN KIPS

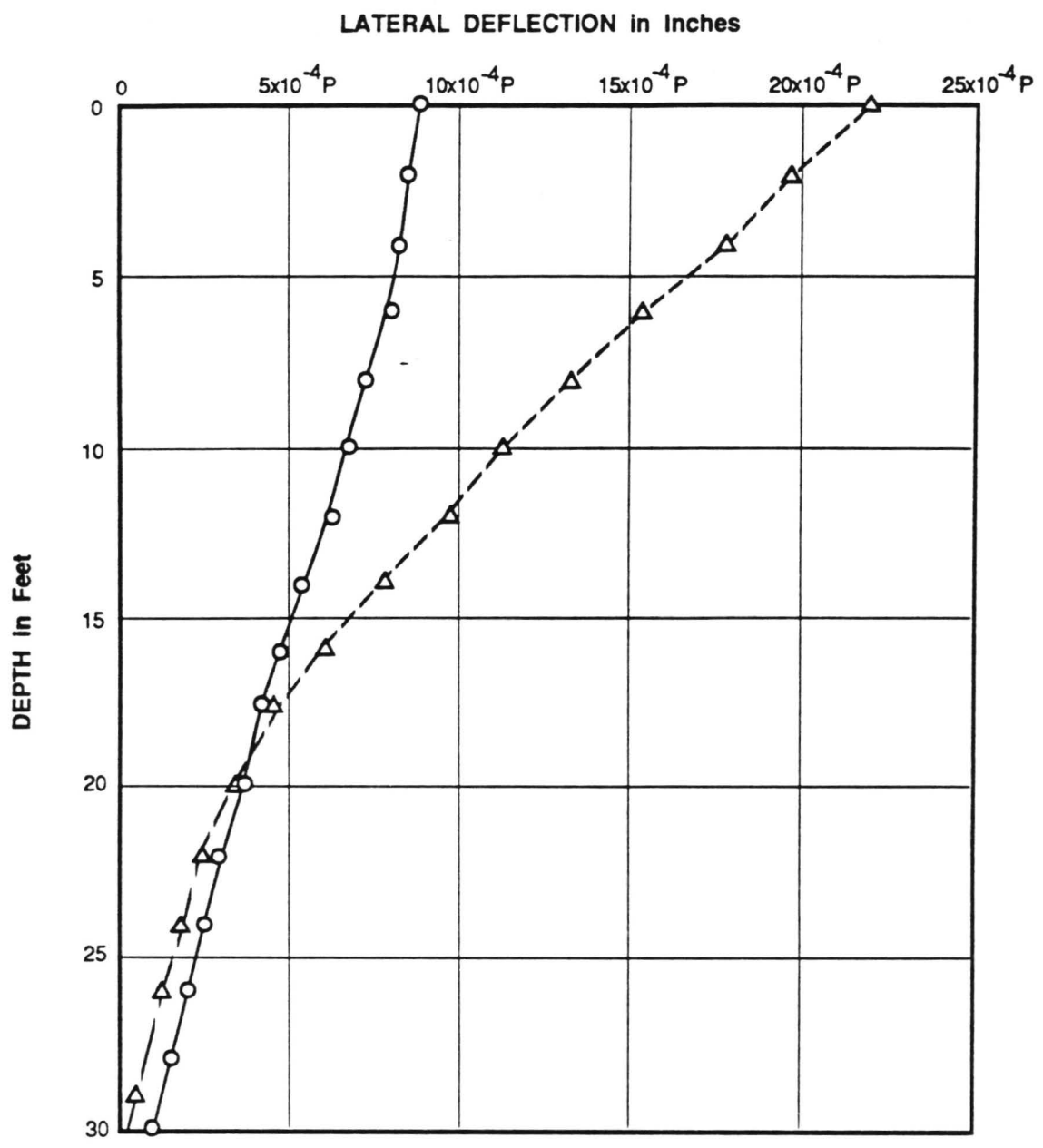
LATERAL DEFLECTION DIAGRAMS
60 - INCH DIAMETER PILE
 (Design Group 3)

LAW / CRANDALL, INC.



FIGURE 8.18G

JOB L92045.AE4 DATE 4/29/93 DR. *IK* O.E. YK *CHKD* *116*

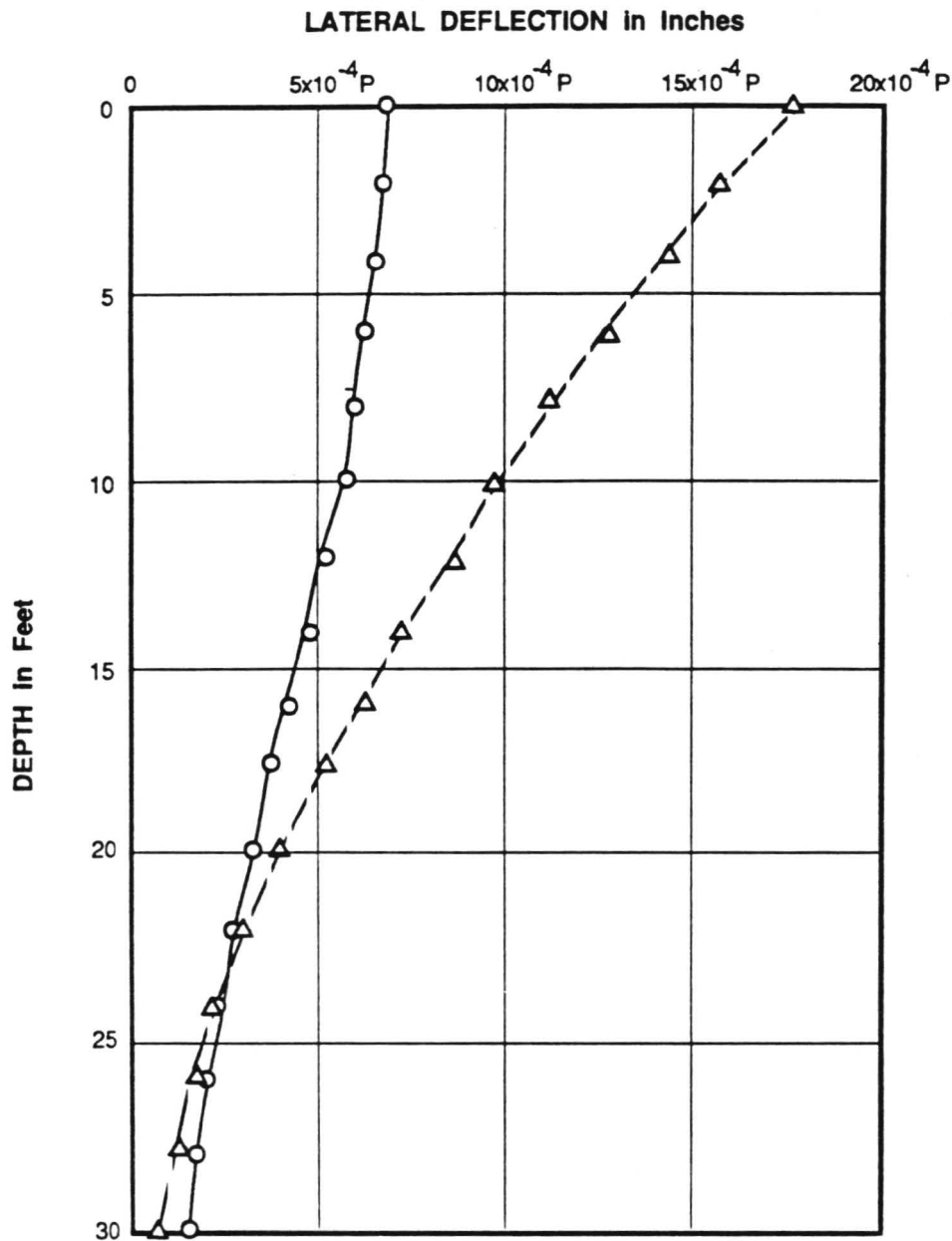


KEY:

○—○ FIXED HEAD PILE
 △--△ FREE HEAD PILE
 P LATERAL LOAD APPLIED AT TOP OF THE PILE IN KIPS

LATERAL DEFLECTION DIAGRAMS
72 - INCH DIAMETER PILE
 (Design Group 3)

JOB L92045.AE4 DATE 4/29/93 DR. IK O.E. YK CHKD MY



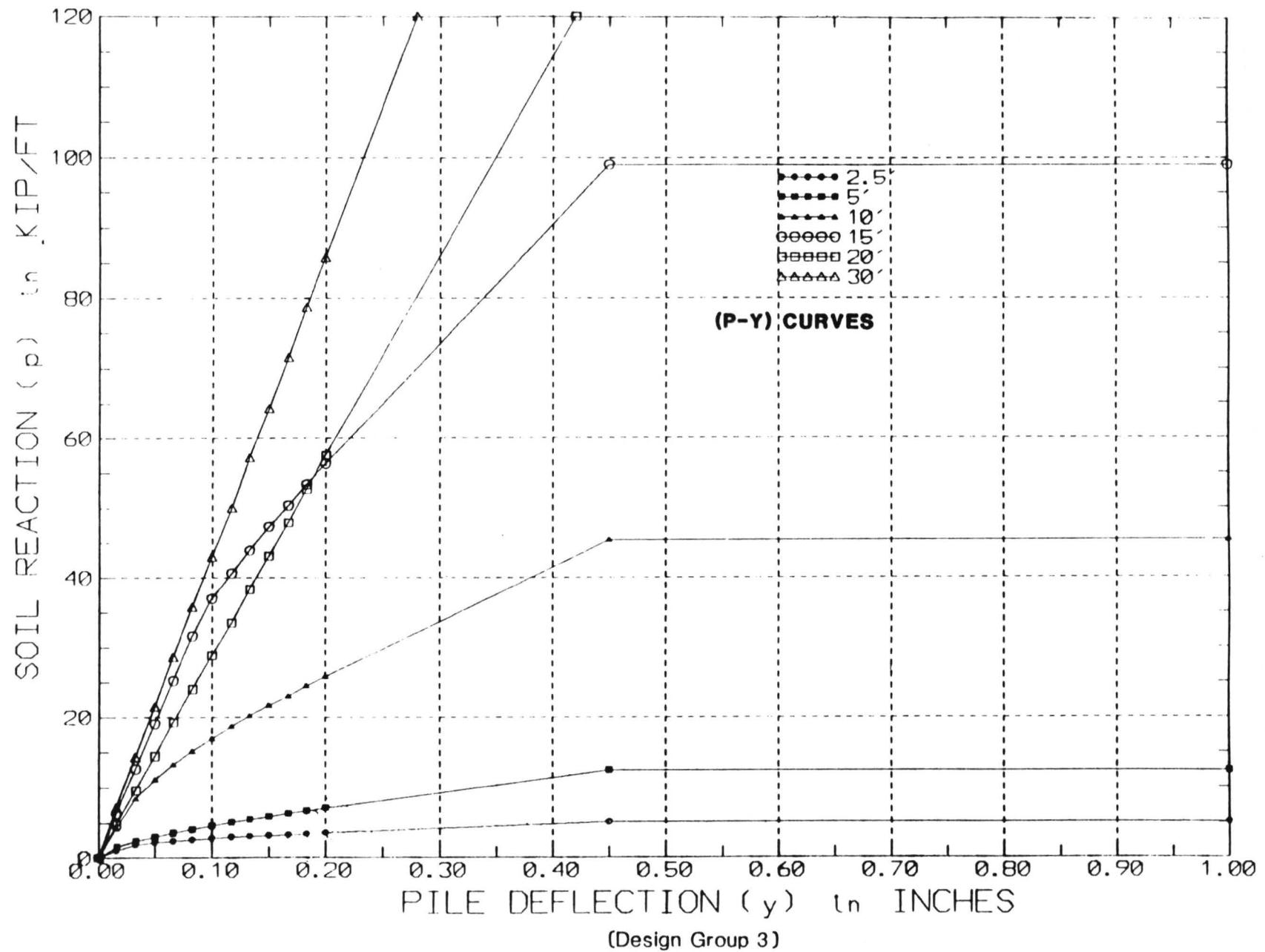
KEY:

- FIXED HEAD PILE
- △--△ FREE HEAD PILE
- P LATERAL LOAD APPLIED AT TOP OF THE PILE IN KIPS

LATERAL DEFLECTION DIAGRAMS
84 - INCH DIAMETER PILE
 (Design Group 3)



12 INCH SQUARE PILE

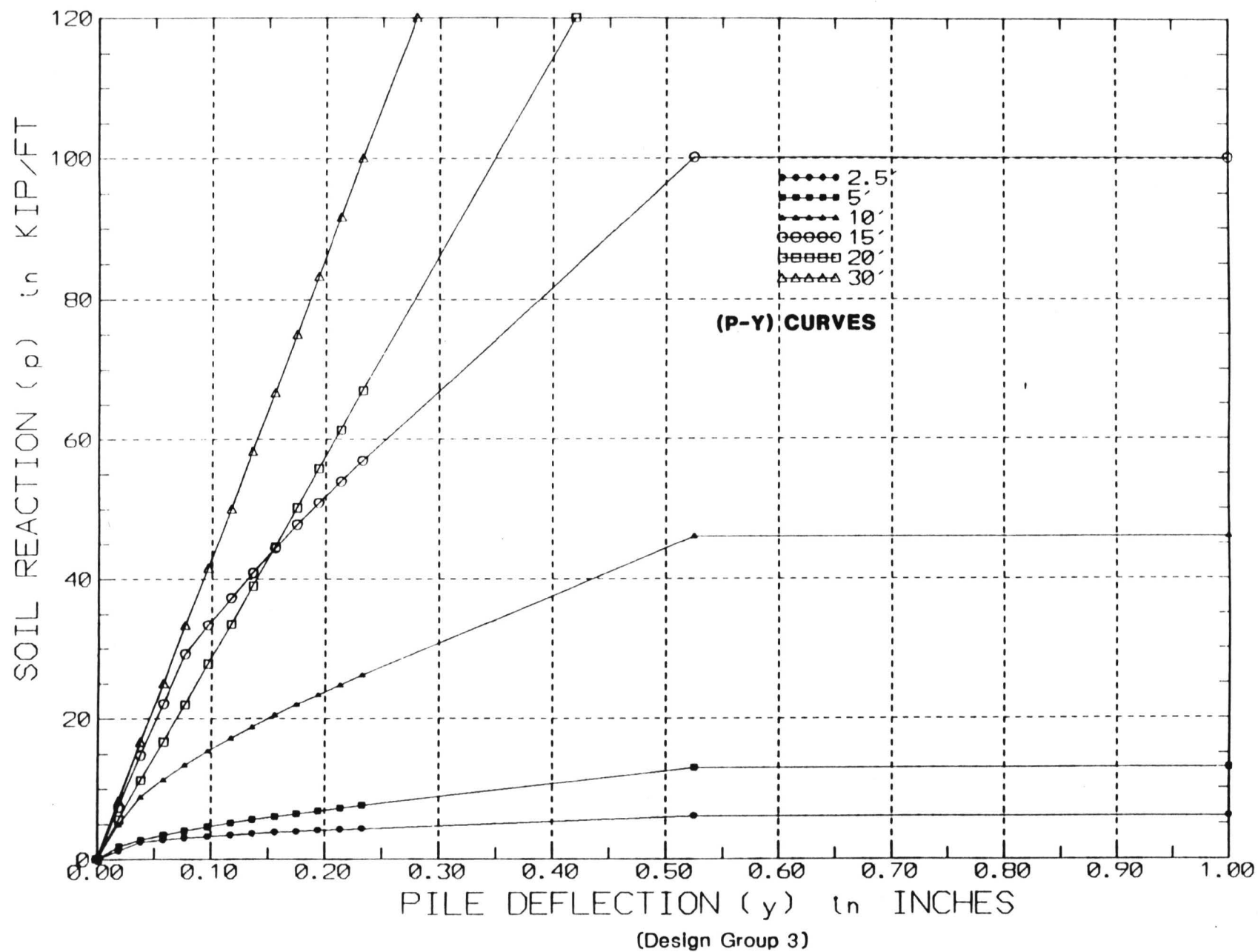


LAW/CRANDALL, INC.



FIGURE 8.19A

14 INCH SQUARE PILE

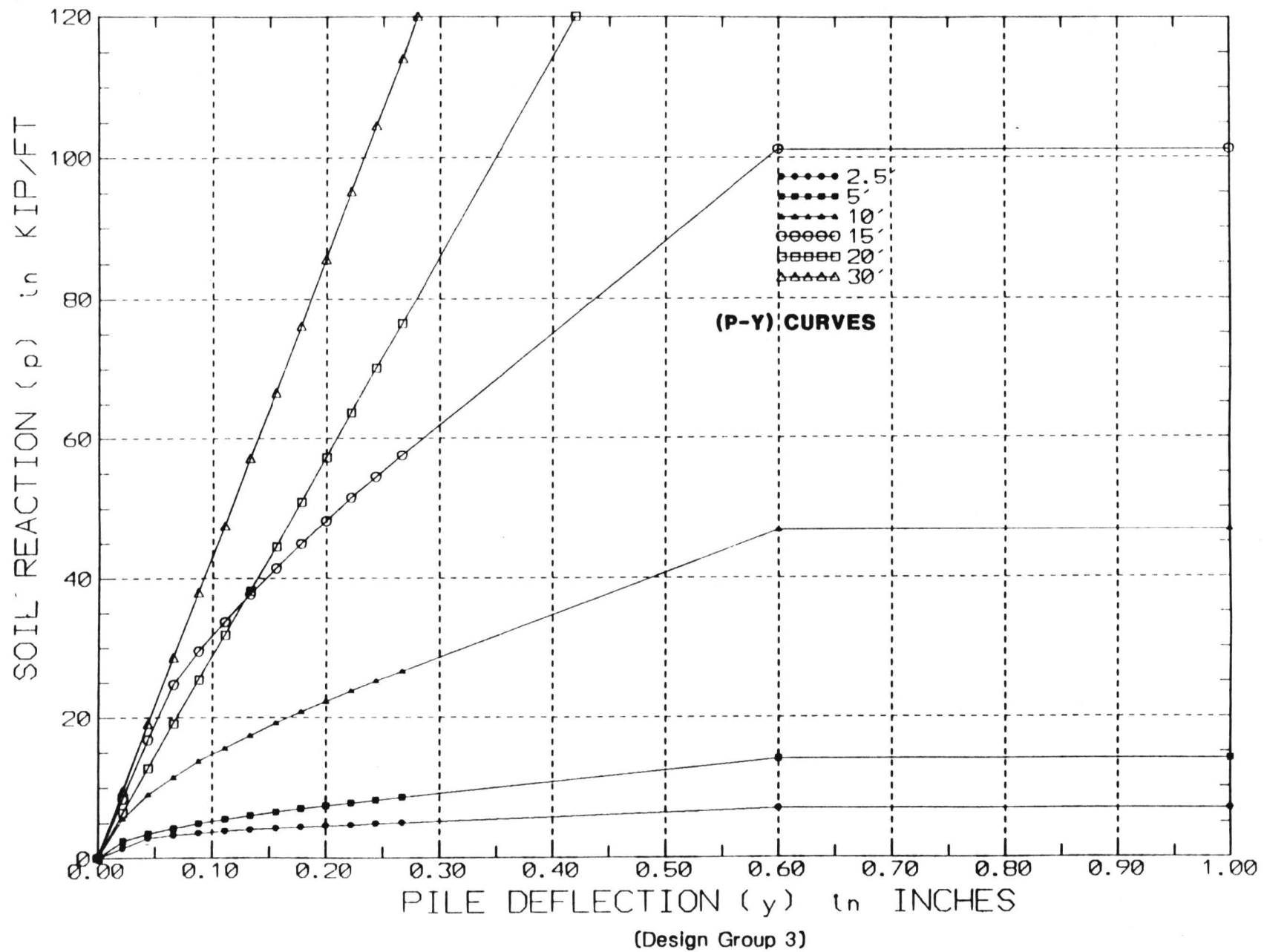


LAW/CRANDALL, INC.



FIGURE 8.19B

16 INCH SQUARE PILE



LAW/CRANDALL, INC.



FIGURE 8.19C

16 INCH CIDH PILE

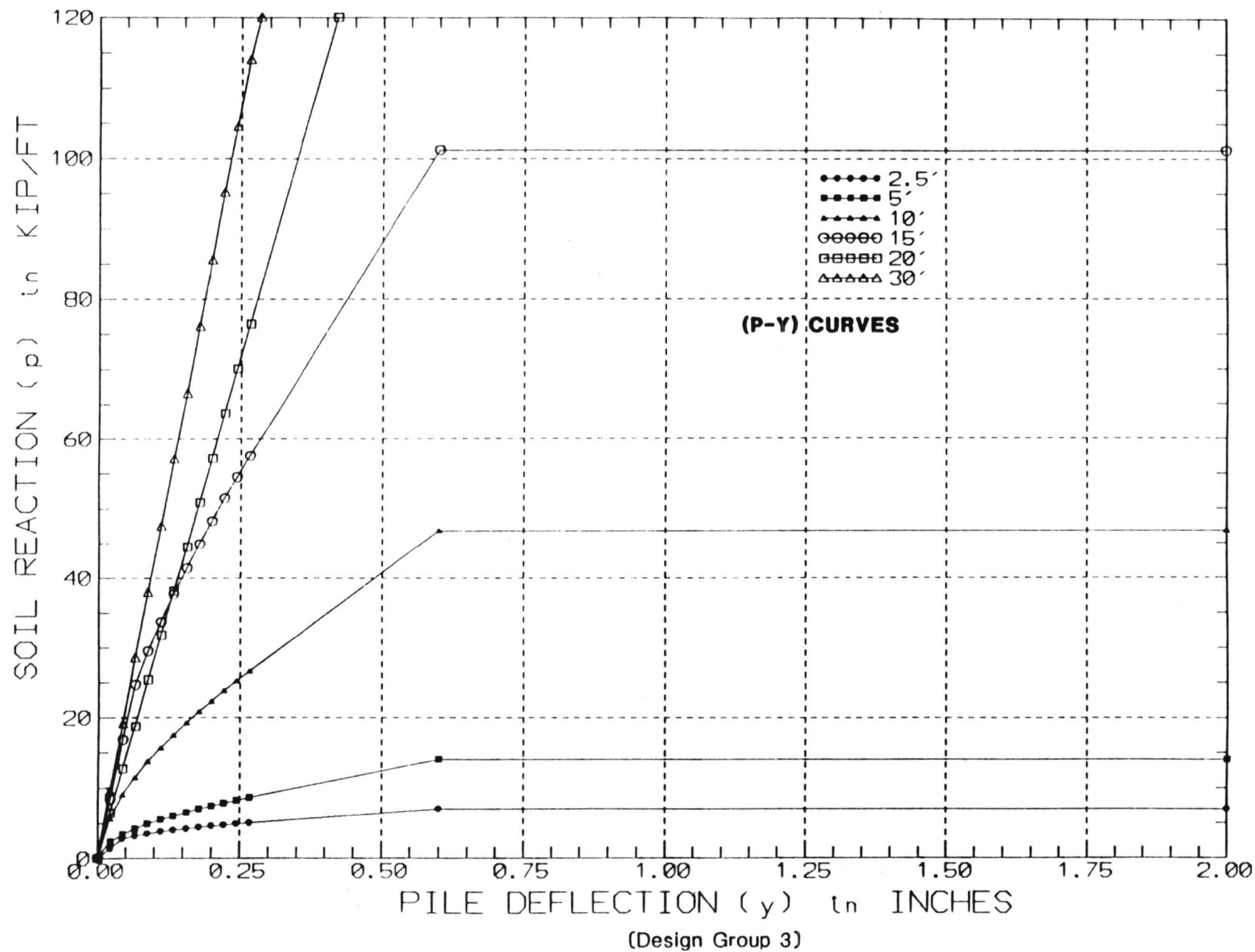


FIGURE 8.19D

LAW/CRANDALL, INC.



20 INCH CIDH PILE

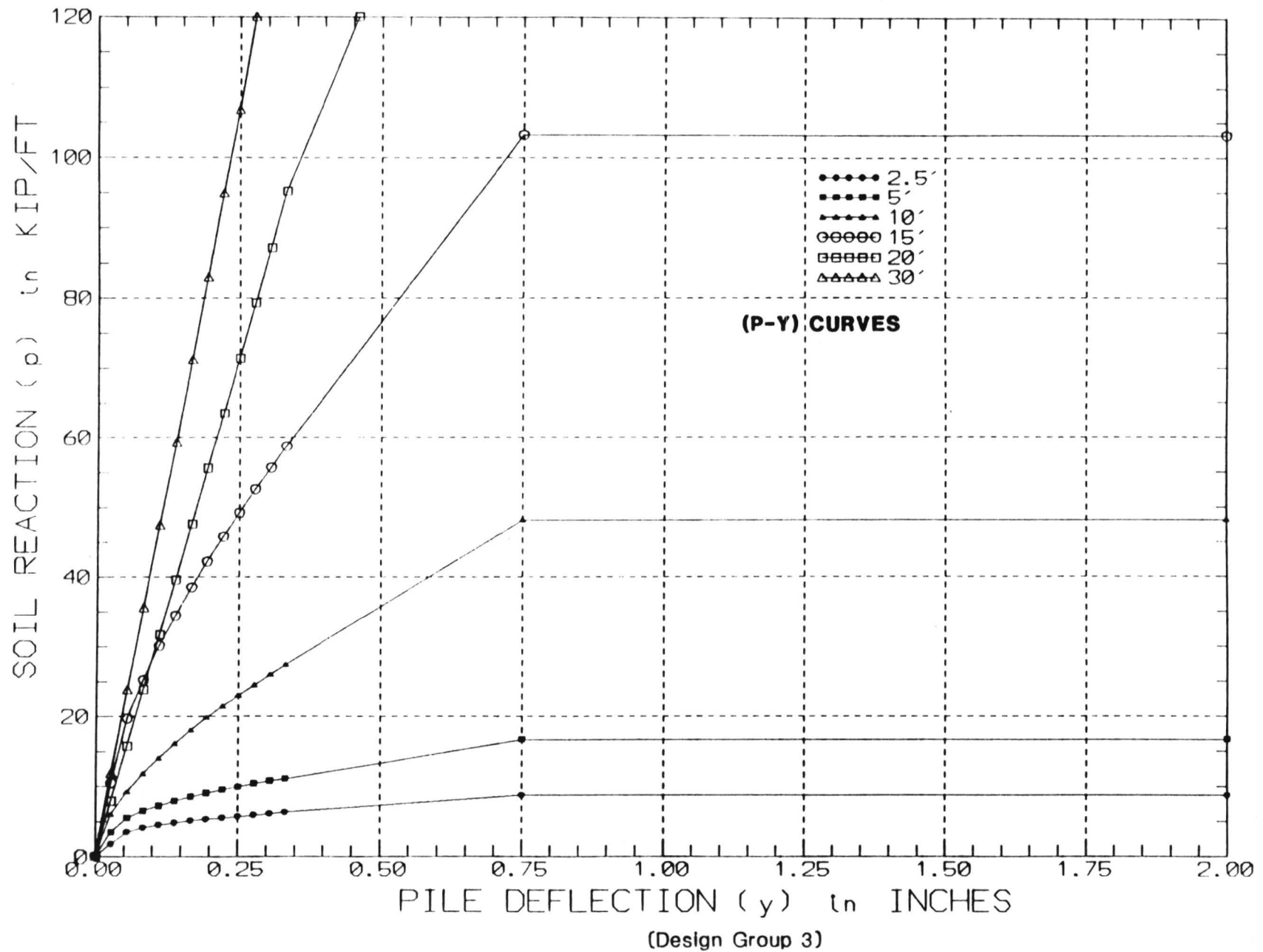
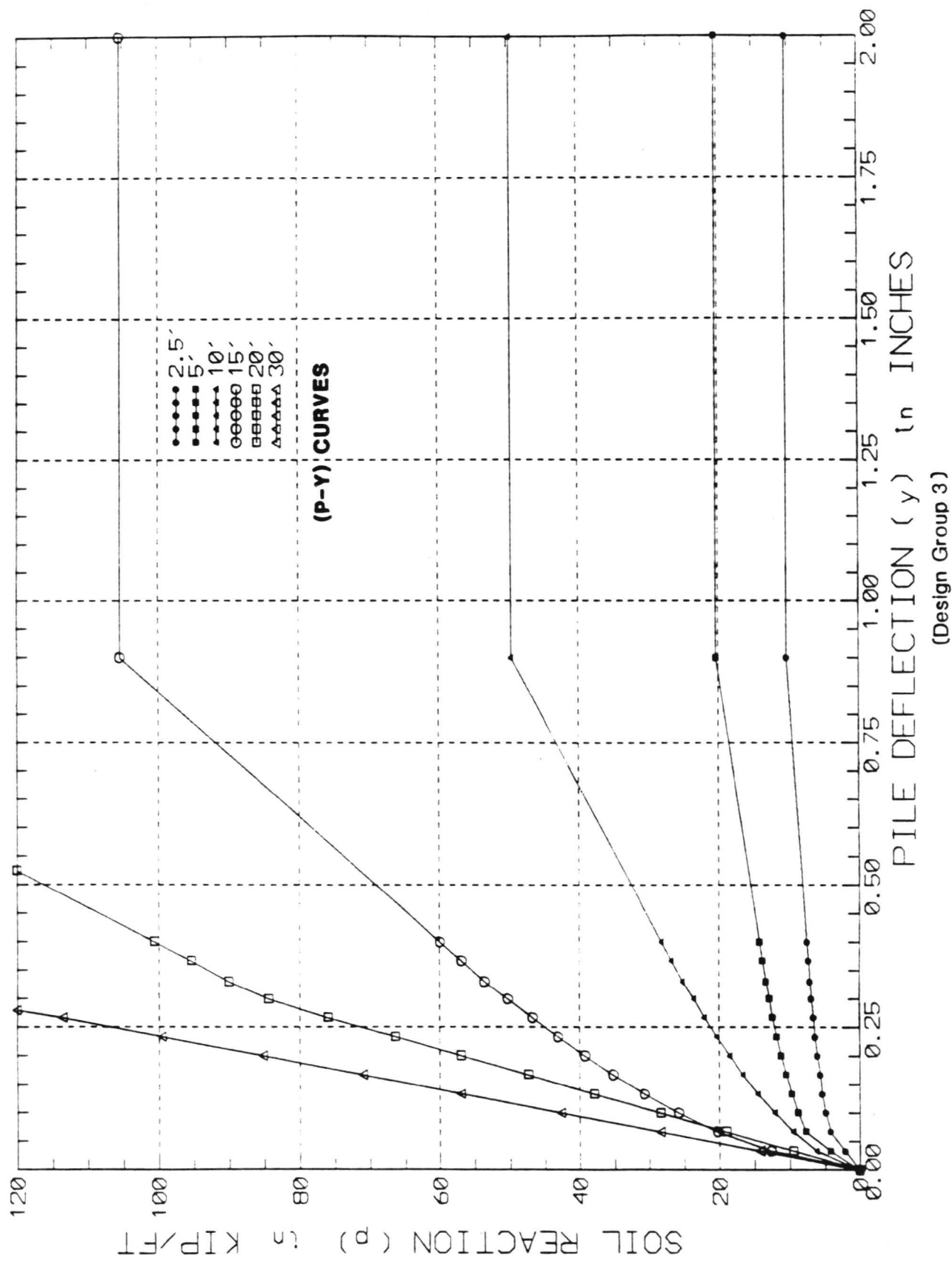


FIGURE 8.19E



24 INCH CIDH PILE



60 INCH CIDH PILE

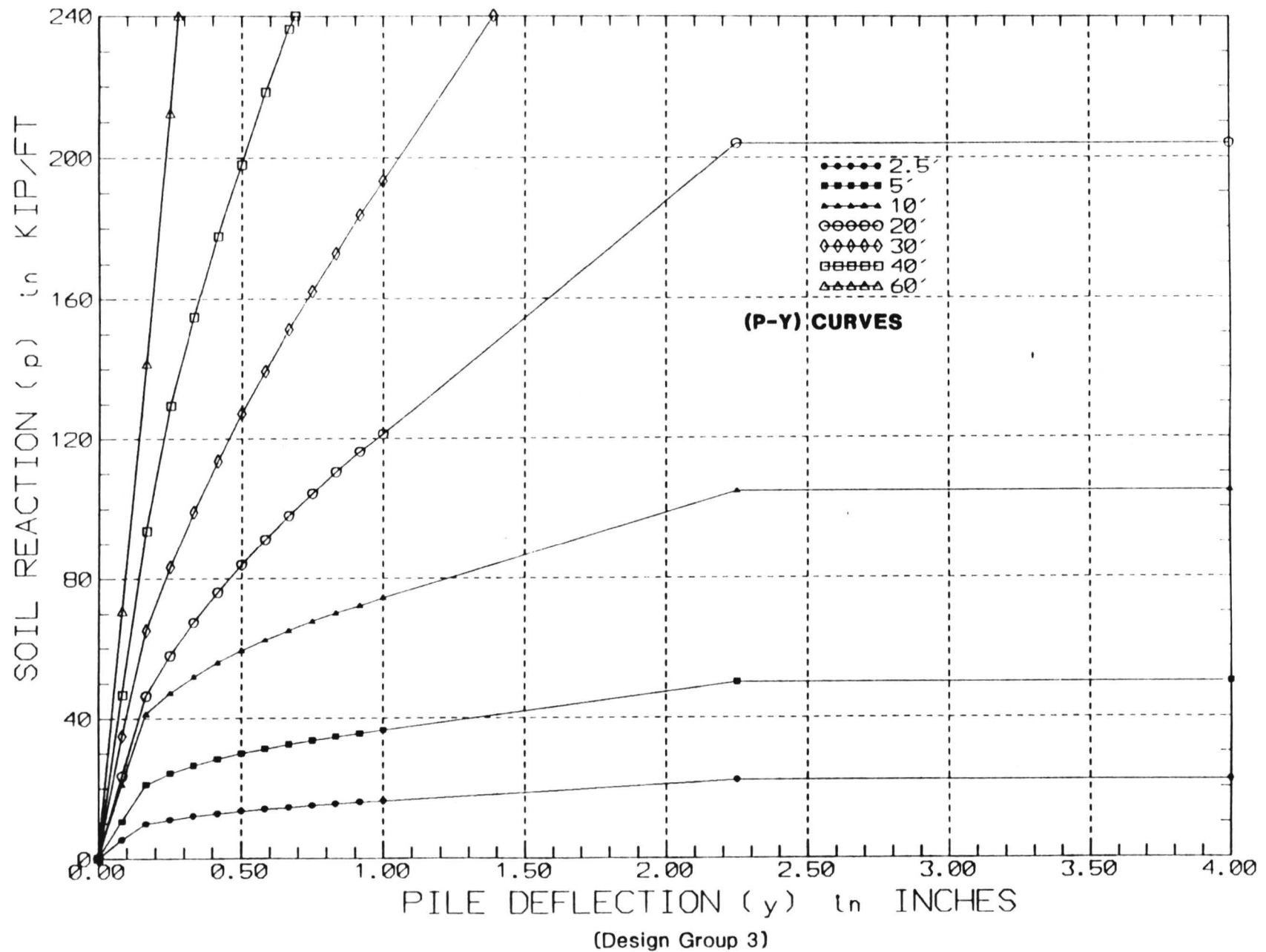
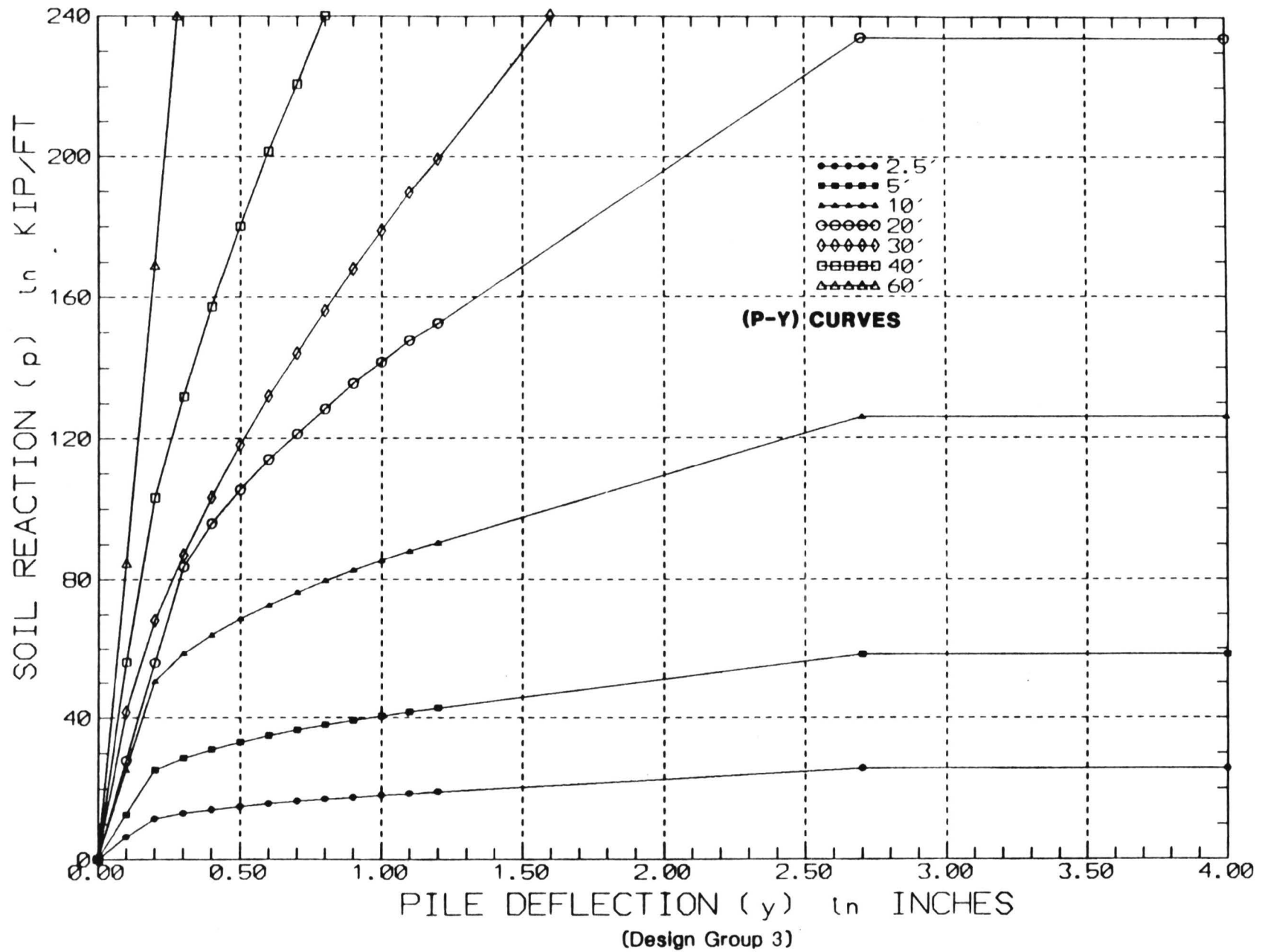


FIGURE 8.19G



72 INCH CIDH PILE



84 INCH CIDH PILE

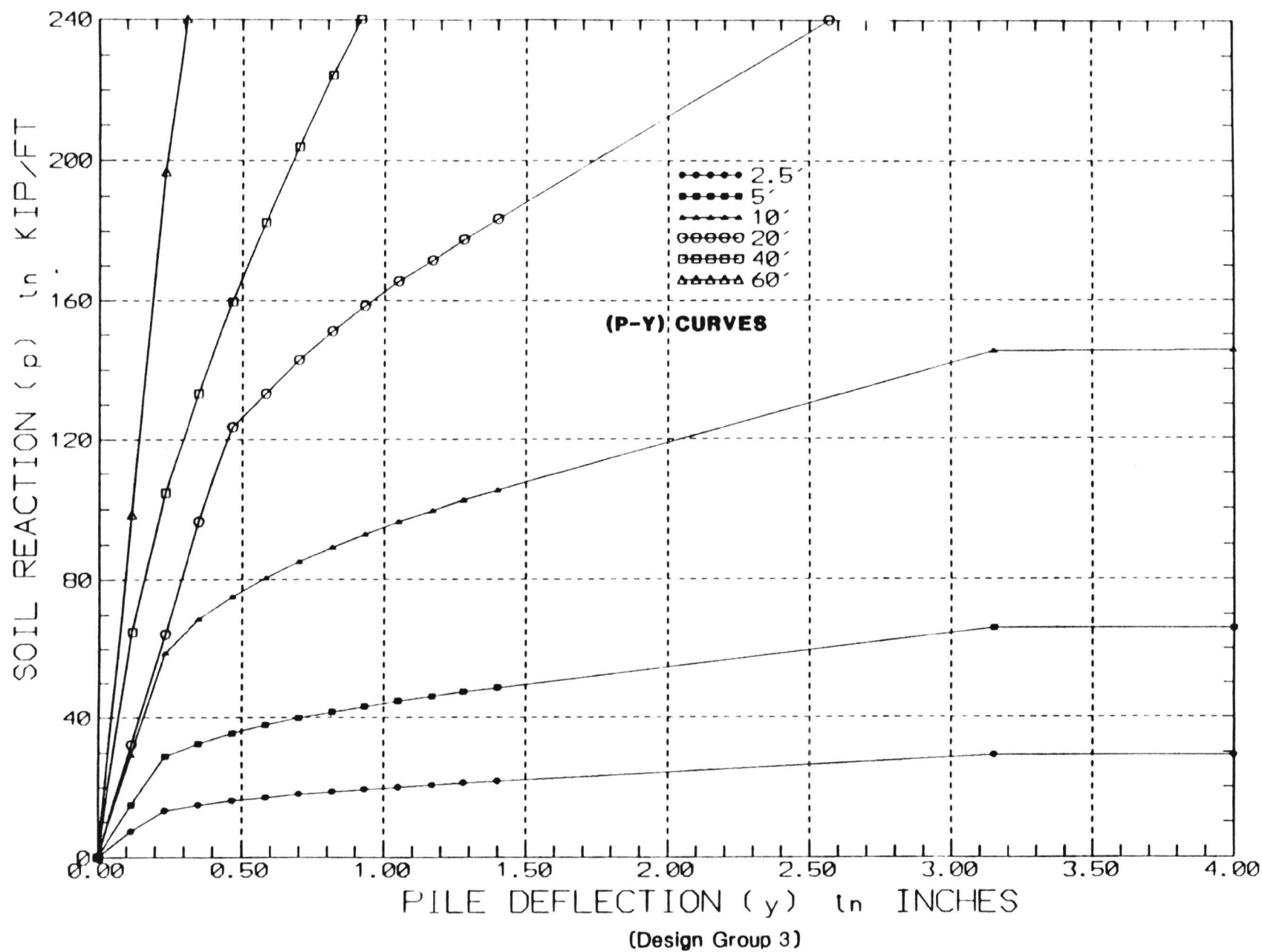
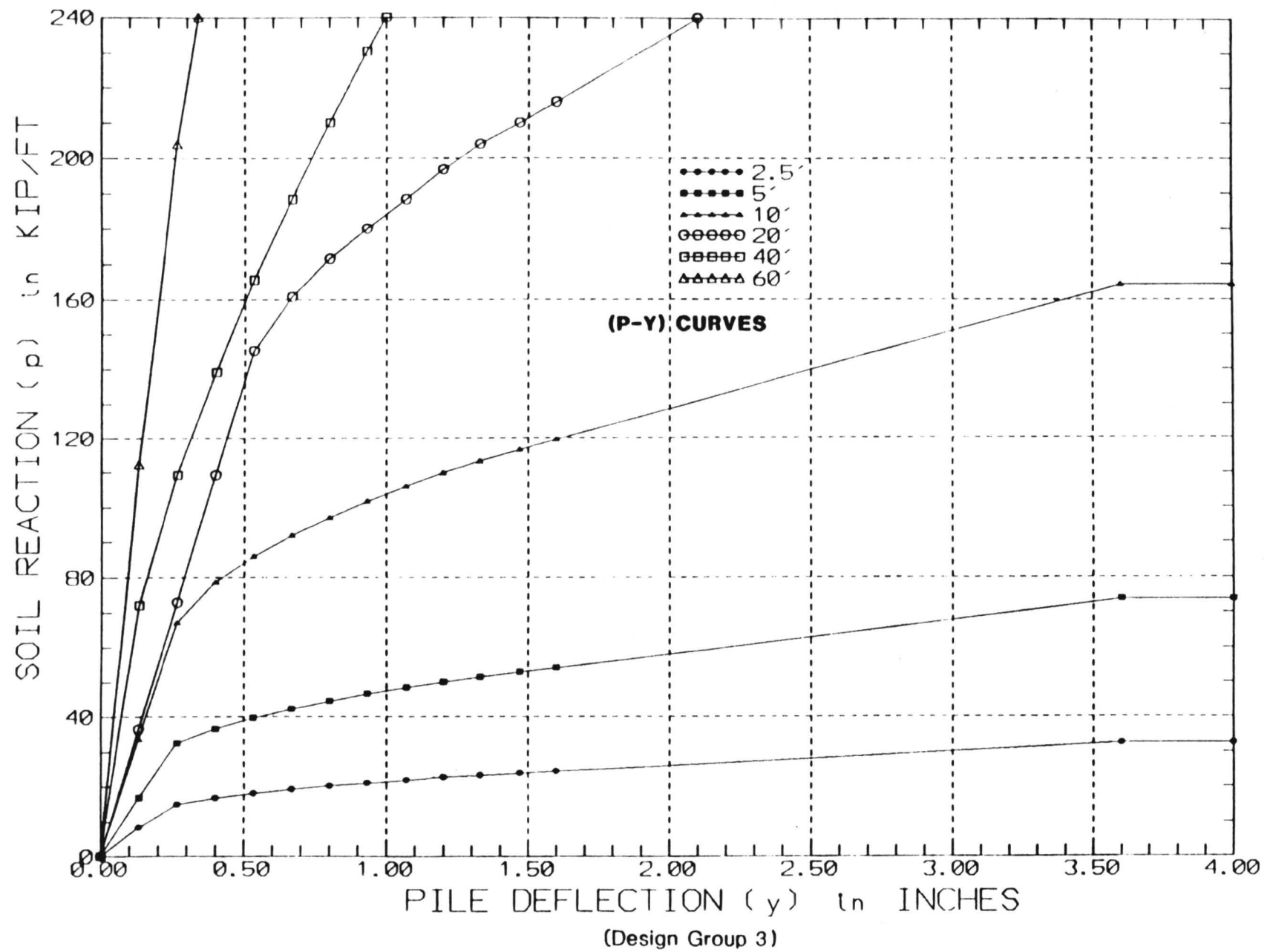


FIGURE 8.191



96 INCH CIDH PILE



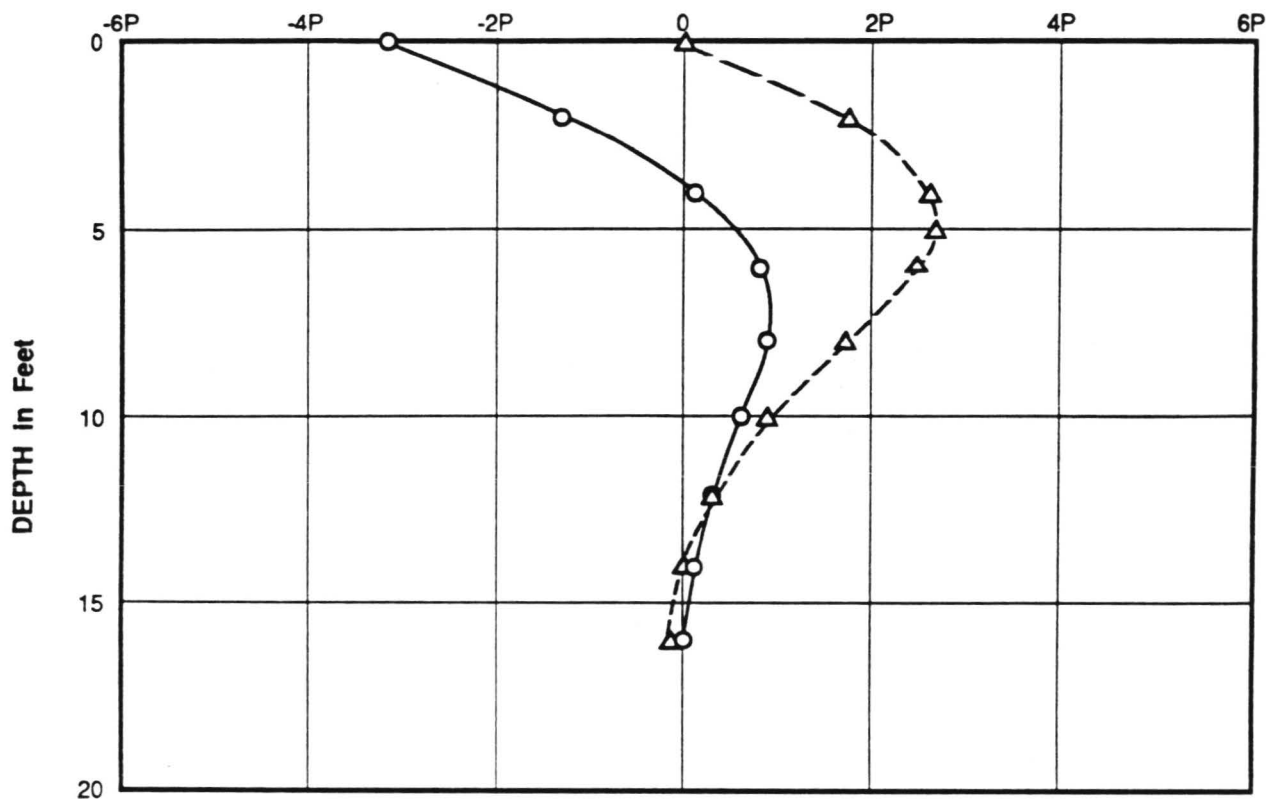
LAW/CRANDALL, INC.



FIGURE 8.19J

JOB L92045.AE4 DATE 2/22/93 DR. Ik O.E. MS *MS* CHKD

BENDING MOMENT in Foot-Kips



KEY:

- FIXED HEAD PILE
- △--△ FREE HEAD PILE
- P LATERAL LOAD APPLIED AT TOP OF THE PILE IN KIPS

BENDING MOMENT DIAGRAMS 12-INCH SQUARE PILE (Design Group 3)

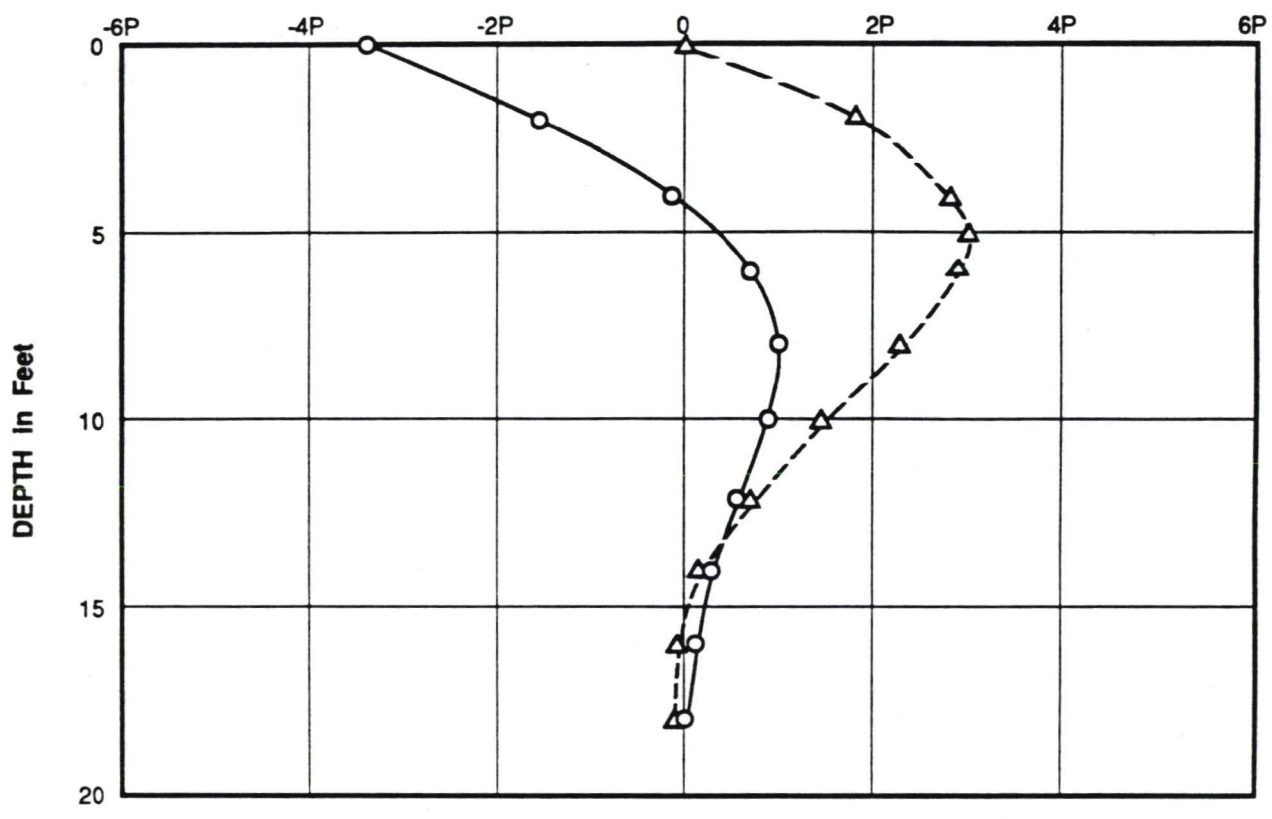
LAW / CRANDALL, INC.



FIGURE 8.20A

JOB L92045.AE4 DATE 2/22/93 DR. IK O.E. MS. CHKD

BENDING MOMENT in Foot-Kips

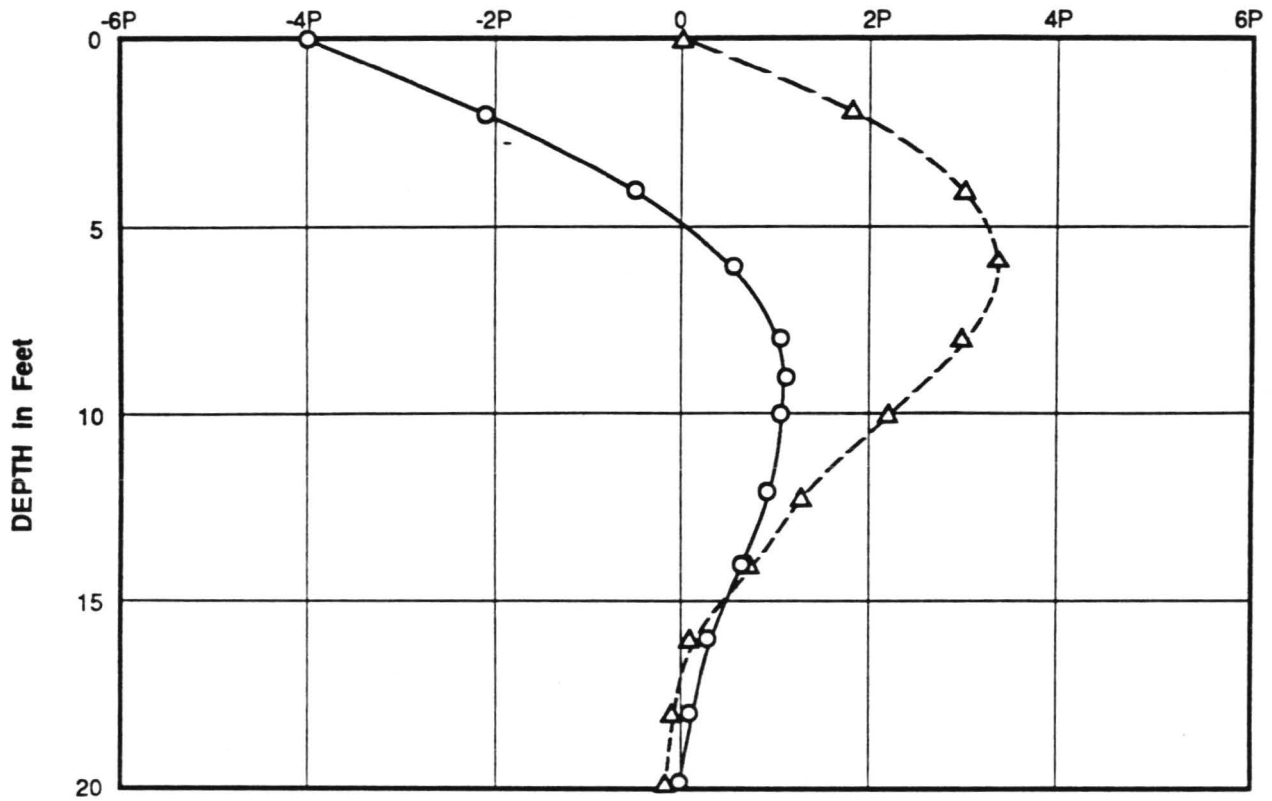


KEY:
 ○—○ FIXED HEAD PILE
 △--△ FREE HEAD PILE
 P LATERAL LOAD APPLIED AT TOP OF THE PILE IN KIPS

BENDING MOMENT DIAGRAMS
 14 - INCH SQUARE PILE
 (Design Group 3)

JOB L92045.AE4 DATE 2/22/93 DR. IK MS CHKD O.E.

BENDING MOMENT in Foot-Kips



KEY:



FIXED HEAD PILE



FREE HEAD PILE

P

LATERAL LOAD APPLIED AT TOP OF THE PILE IN KIPS

BENDING MOMENT DIAGRAMS
16 - INCH SQUARE PILE
(Design Group 3)

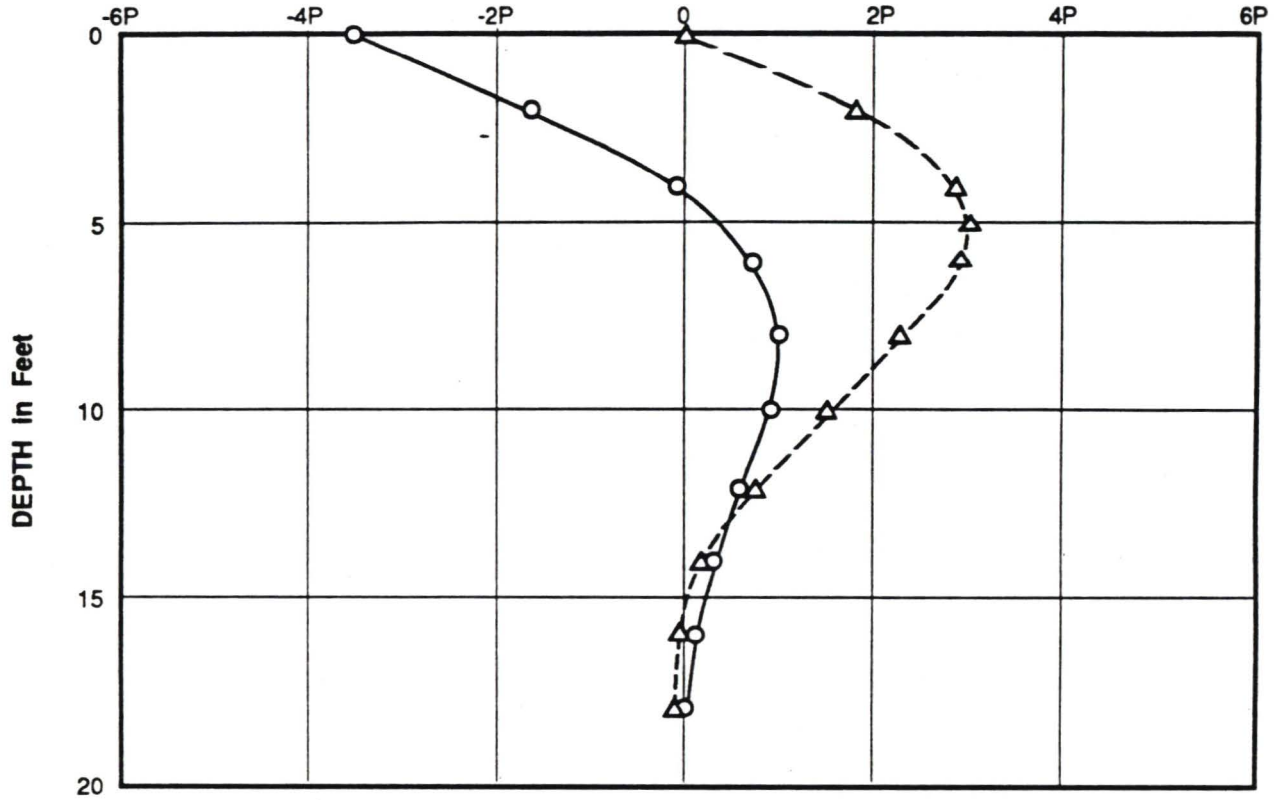
LAW / CRANDALL, INC.



FIGURE 8.20C

JOB L92045-AE4 DATE 2/22/93 DR. IK MS CHKD O.E.

BENDING MOMENT in Foot-Kips



KEY:

- FIXED HEAD PILE
- △--△ FREE HEAD PILE
- P LATERAL LOAD APPLIED AT TOP OF THE PILE IN KIPS

BENDING MOMENT DIAGRAMS 16 - INCH DIAMETER PILE (Design Group 3)

LAW / CRANDALL, INC.



FIGURE 8.20D

82

CHKD

MS

O.E.

IK

DR.

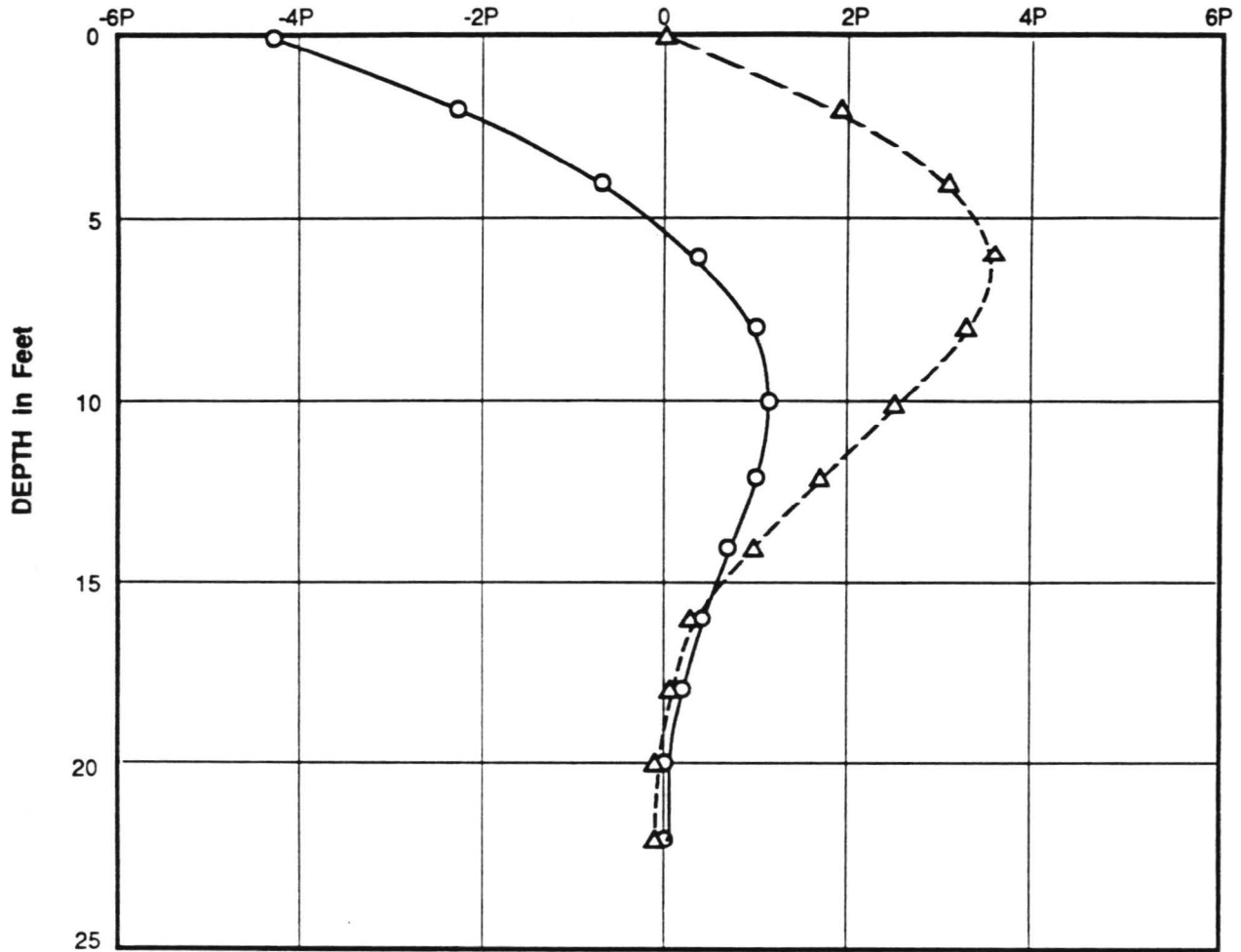
DATE 2/22/93

JOB L92045.AE4

JOB L92045.AE4

JOB

BENDING MOMENT in Foot-Kips



KEY:



FIXED HEAD PILE



FREE HEAD PILE

P

LATERAL LOAD APPLIED AT TOP OF THE PILE IN KIPS

BENDING MOMENT DIAGRAMS

20 - INCH DIAMETER PILE

(Design Group 3)

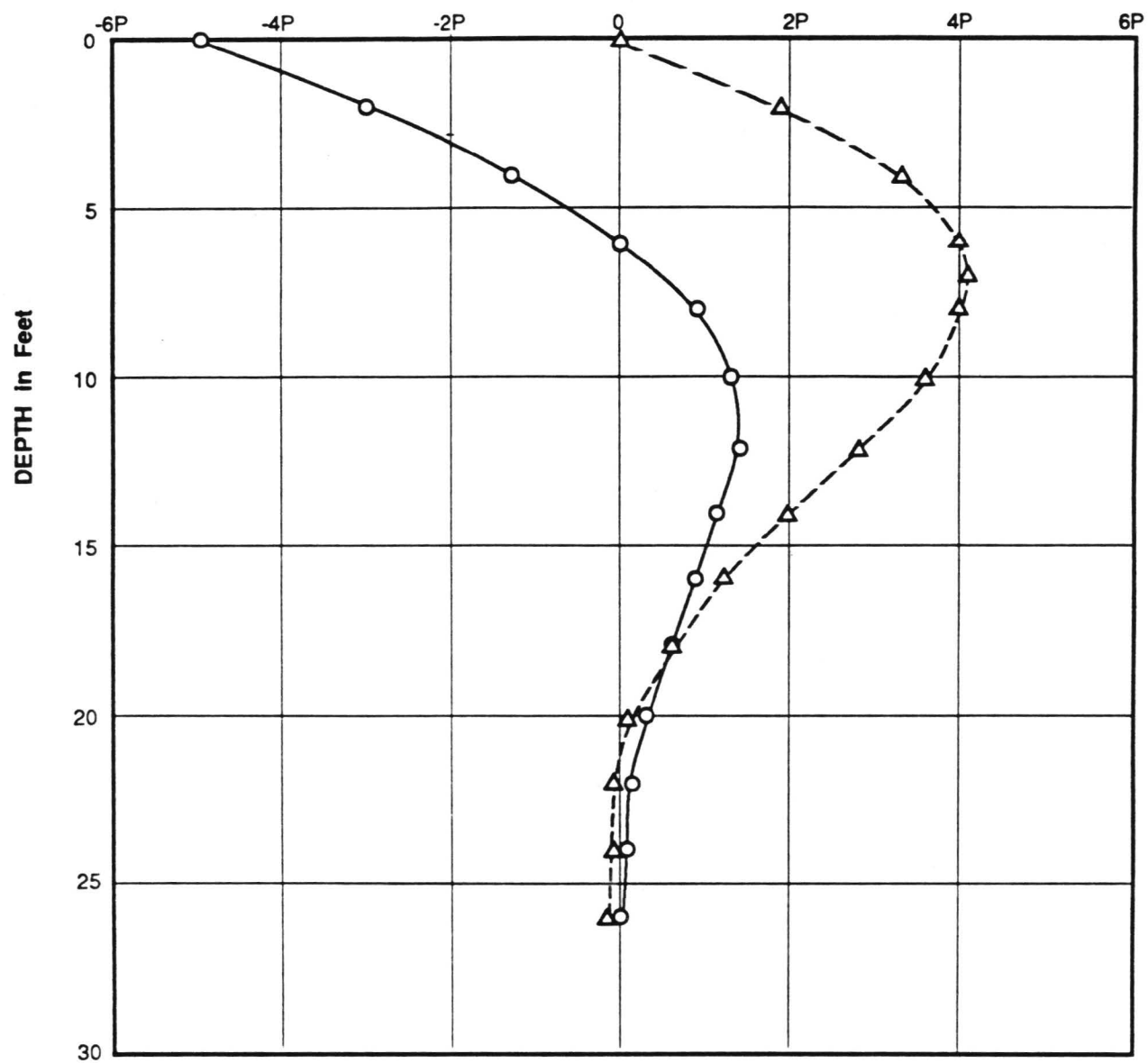
LAW / CRANDALL, INC.



FIGURE 8.20E

JOB L92045.AE4 DATE 2/22/93 DR. Ik O.E. MS CHKD *AK*

BENDING MOMENT in Foot-Kips



KEY:
 ○—○ FIXED HEAD PILE
 △--△ FREE HEAD PILE
 P LATERAL LOAD APPLIED AT TOP OF THE PILE IN KIPS

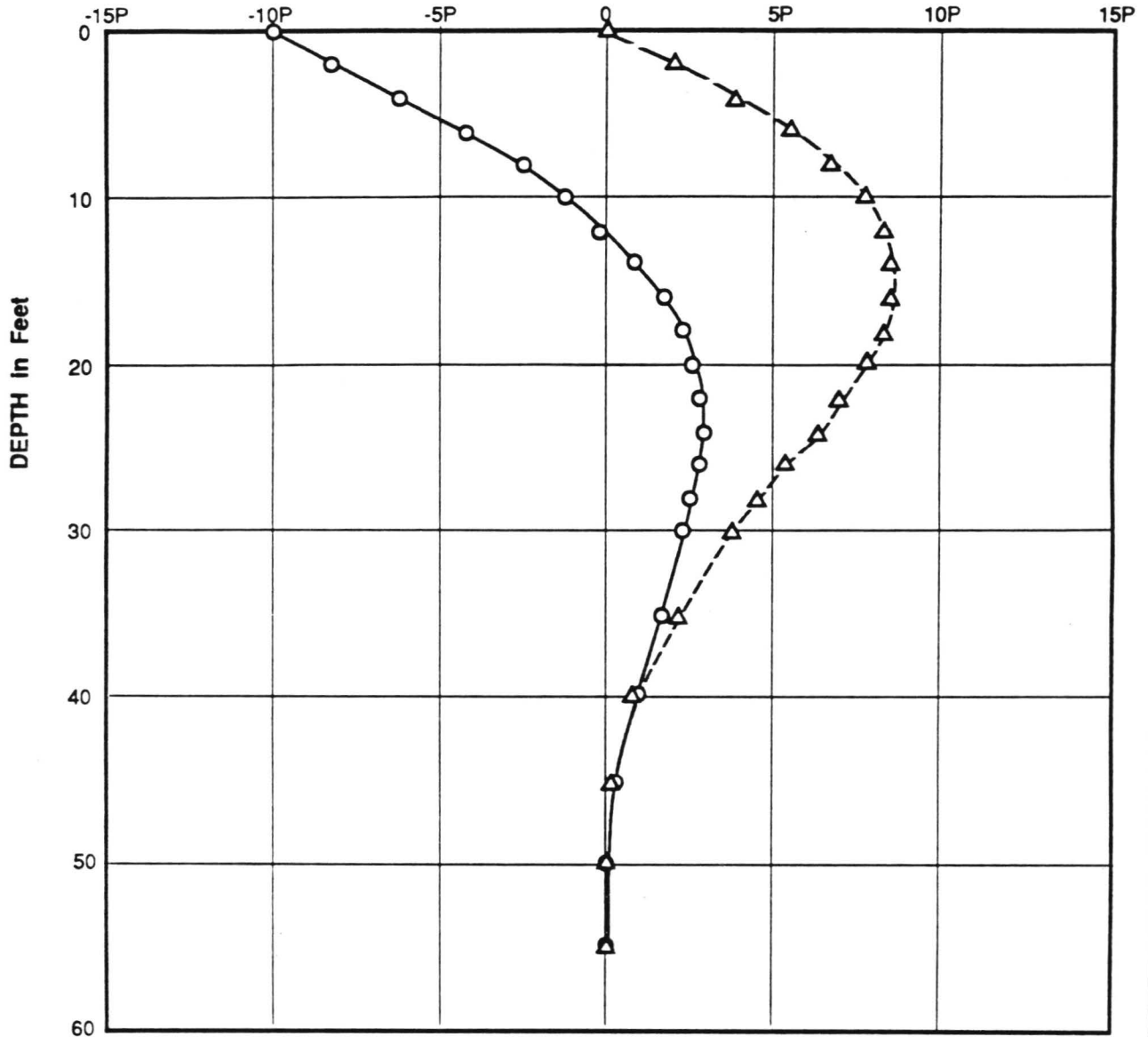
BENDING MOMENT DIAGRAMS
 24 - INCH DIAMETER PILE
 (Design Group 3)

LAW / CRANDALL, INC.



JOB L92045.AE4 DATE 2/22/93 DR. IK MS CHKD *MS* O.E. *MS*

BENDING MOMENT in Foot-Kips



KEY:

- FIXED HEAD PILE
- △—△ FREE HEAD PILE
- P LATERAL LOAD APPLIED AT TOP OF THE PILE IN KIPS

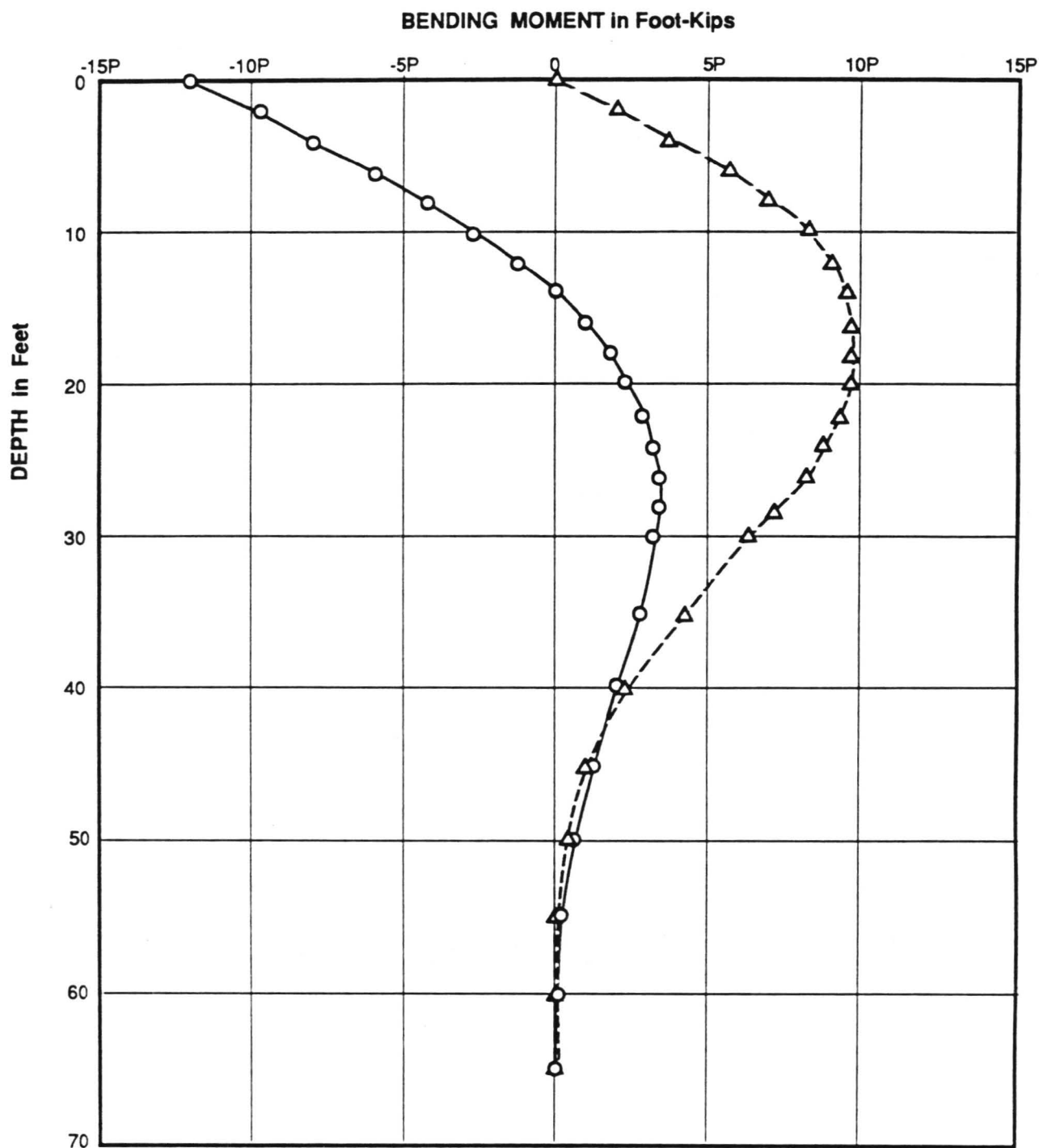
BENDING MOMENT DIAGRAMS 60 - INCH DIAMETER PILE (Design Group 3)

LAW / CRANDALL, INC.



FIGURE 8.20G

JOB L92045 AE4 DATE 2/22/93 DR. IR O.E. MSK CHKD *[Signature]*



KEY:

- FIXED HEAD PILE
- △--△ FREE HEAD PILE
- P LATERAL LOAD APPLIED AT TOP OF THE PILE IN KIPS

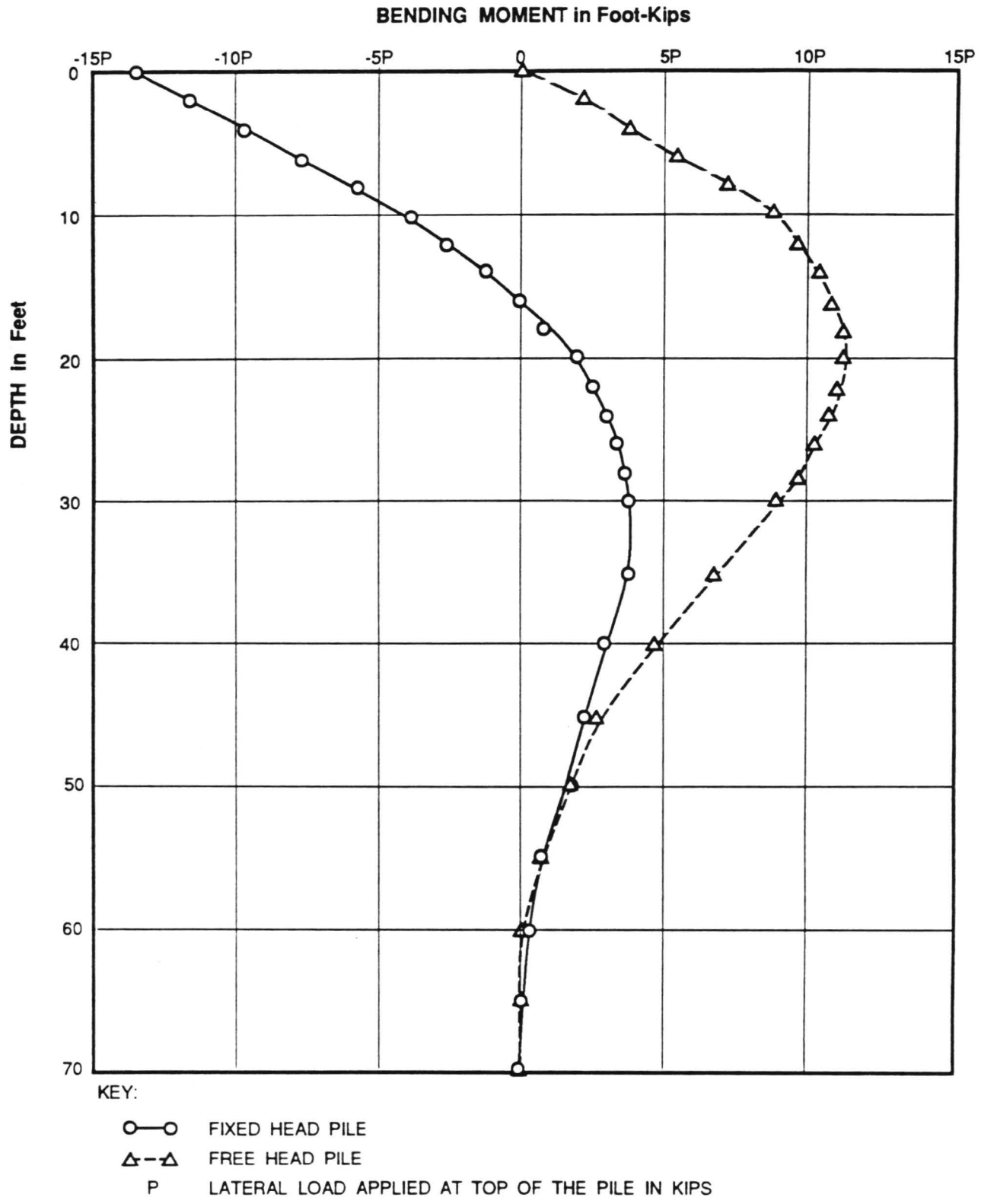
BENDING MOMENT DIAGRAMS
72 - INCH DIAMETER PILE
 (Design Group 3)

LAW / CRANDALL, INC.



FIGURE 8.20H

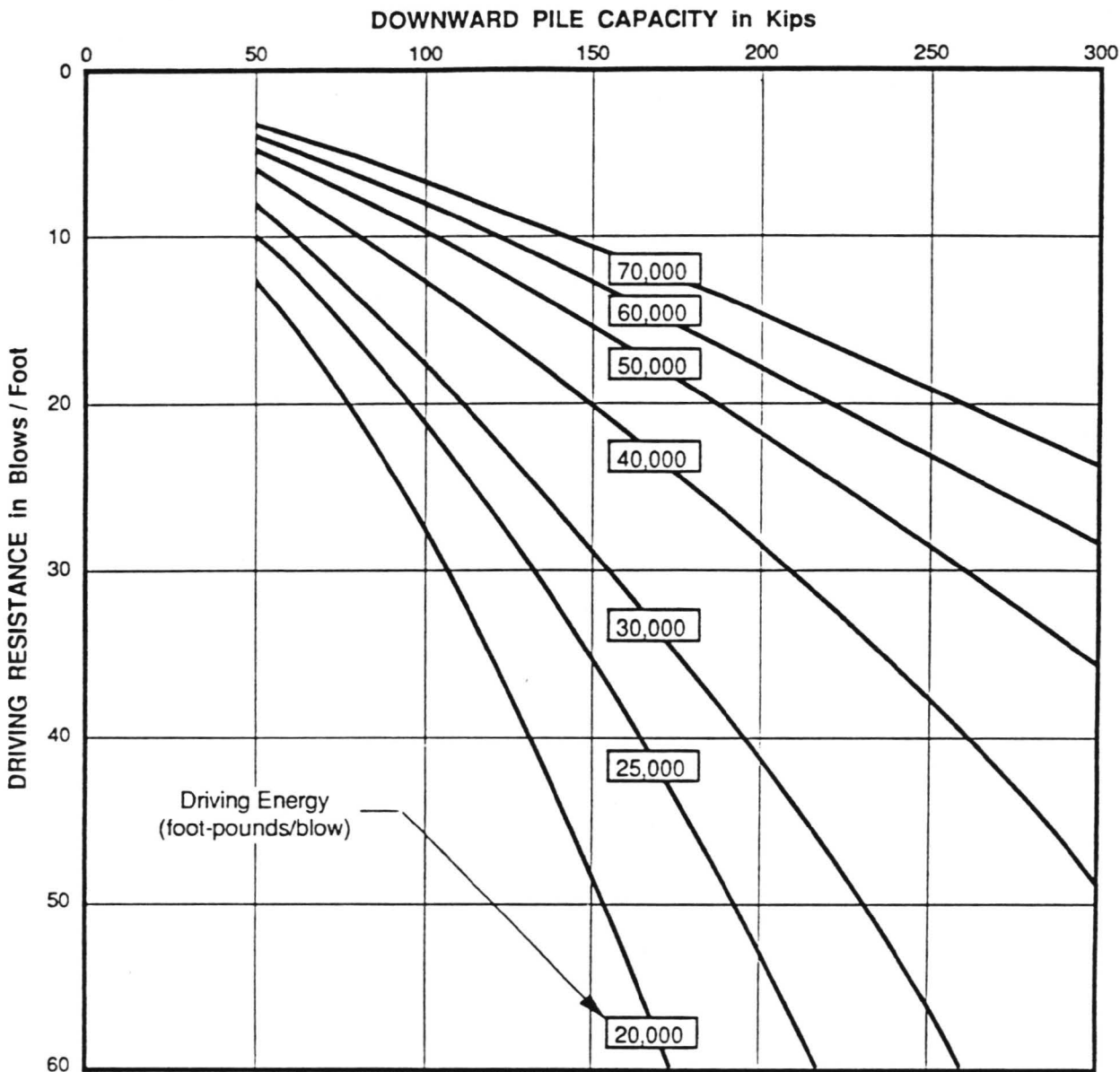
JOB L92045.AE4 DATE 2/22/93 DR. II O.E. MS CHKD *[Signature]*



BENDING MOMENT DIAGRAMS
84 - INCH DIAMETER PILE
 (Design Group 3)

LAW / CRANDALL, INC.

JOB L92045.AE4 DATE 2/23/93 DR. O.E. MS *WJ* CHKD *te2*



NOTES :

- (1) The above driving resistance should be obtained for the last foot of driving at design pile length. If the driving resistance at the design length is less than above, the piles should be lengthened until the desired driving resistance is obtained.
- (2) As an alternate to lengthening when low driving resistance is obtained, the piles may be allowed to set overnight and the number of blows to drive the pile one inch the following day determined. If the restarting resistance is at least two times the above criteria, the pile may be considered satisfactory.
- (3) If driving resistance of three times the above criteria is encountered within five feet of design length, the pile driving may be stopped.

PILE DRIVING CRITERIA

(Design Group 3)

LAW / CRANDALL, INC.



FIGURE 8.21

SECTION 9.0

RECOMMENDATIONS – AT-GRADE STRUCTURES AND TRACK BEDS

9.1 GENERAL

Except along the Foothill Freeway, the existing fill soils encountered along the alignment are not considered suitable for support of the at-grade structures. With proper grading, including excavation and compaction of all existing fill, typical light at-grade structures (including passenger service stations, power substations, traction power stations, and sound walls) could be supported on shallow spread footings established in compacted fill or the underlying natural soils. Minor free-standing walls, slough walls, and auxiliary retaining walls may be supported in the same manner.

To provide uniform support for tracks on grade, the upper fill soils should be excavated to allow the placing of at least 1 foot of properly compacted subgrade fill beneath the subgrade ballast.

The Avenue 26 Station will be located on the fill embankment that extends between the Los Angeles River and Avenue 33. Fill soils, 17 to 23 feet deep, were encountered in the borings drilled through the top of the embankment. The tracks may be supported on grade at this location. However, supporting the station on the existing fill may result in unacceptable differential settlement between the tracks and the station platform. Accordingly, we recommend that the station be supported on drilled piles extending through the fill and into the underlying natural soils.

If the grading recommendations presented in Section 13.0 are followed, light passenger stations and other light structures may be supported on the compacted fill soils of the Foothill Freeway embankment.



Excluding the upper few feet, the natural overburden soils and bedrock beneath the alignment are generally firm and dense. For support of heavier at-grade structures, including heavier passenger service station platforms and major retaining walls, spread footings should extend into the underlying undisturbed firm and dense natural materials. Conventional spread footings may be used where the firm and dense natural materials are exposed at or close to final grade; deep spread footings, straight drilled shafts, and/or drilled piling may be used in the deeper fill areas.

9.2 FOUNDATIONS

9.2.1 SPREAD FOOTINGS

9.2.1.1 Bearing Values

Footings established in either properly compacted fill or the underlying natural soils may be designed to impose a net dead plus live load pressure of 2,500 pounds per square foot. A one-third increase in the bearing value may be used for wind or seismic loads. Footings should extend at least 2 feet below the lowest adjacent grade or floor slab.

Footings designed to be supported on the upper firm and dense natural materials may be designed to impose a maximum dead plus live load pressure of up to 4,000 pounds per square foot. A one-third increase in the bearing value may be used for wind or seismic loads. Such footings should extend at least 1 foot into the firm and dense materials and at least 2 feet below the lowest adjacent grade or floor level.

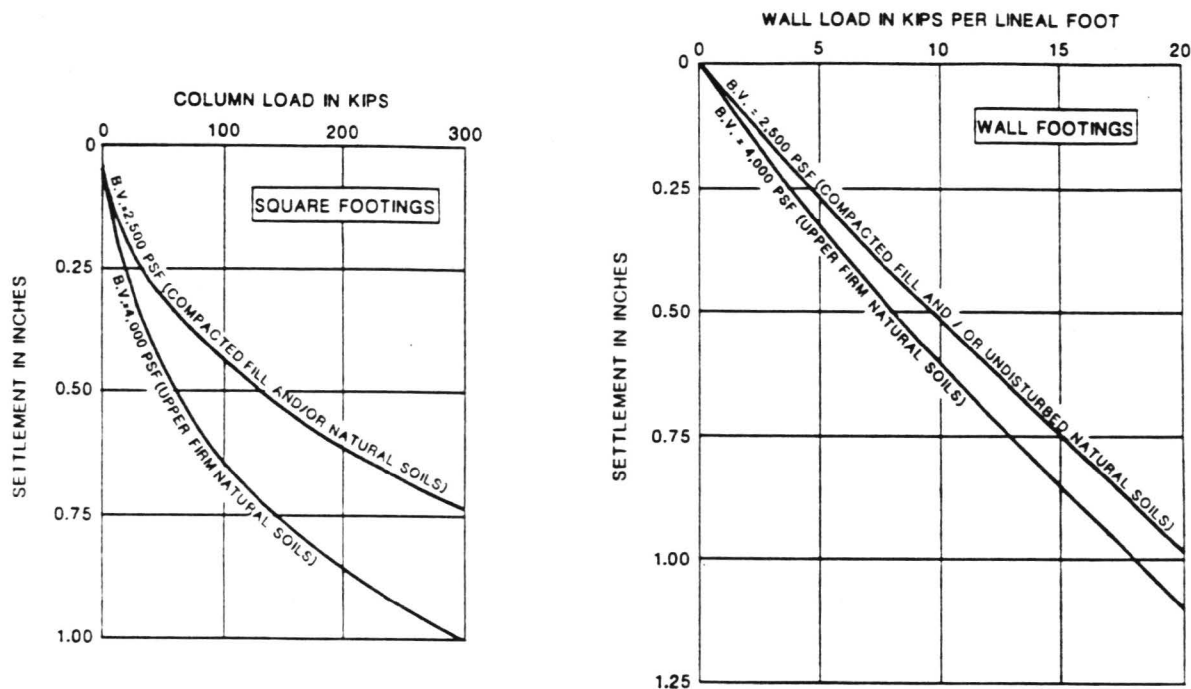
Since the recommended bearing values are net values, the weight of the concrete within the footings may be taken as 50 pounds per cubic foot, and the weight of soil backfill may be neglected when computing the imposed downward foundation loadings.

While the actual bearing value of the fill will depend on the material used and the compaction methods employed, the quoted bearing value should be applicable if acceptable soils are used and are compacted as recommended. The bearing value of the fill should be confirmed after completion of the grading.



9.2.1.2 Settlement

The settlement of the proposed at-grade structures, supported on spread footings in the manner recommended, will depend on the load imposed. The anticipated settlements of footings established in either properly compacted fill or the natural soils, and of spread footings established in the upper firm and dense natural soils, may be determined using the following graphs.



9.2.1.3 Lateral Loads

Lateral loads may be resisted by soil friction and by the passive resistance of the soils. A coefficient of friction of 0.5 may be used between footings or slabs and the supporting soils. The passive resistance of the properly compacted fill or the natural soils against footings may be assumed to be equal to the pressure developed by a fluid with a density of 300 pounds per cubic foot. A one-third increase in the passive value may be used for wind or seismic loads. The frictional resistance and the passive resistance of the soils may be combined without reduction in determining the total lateral resistance.



9.2.1.4 Foundation Observation

To verify the presence of the desired supporting materials at design elevations, all foundation excavations should be observed by a geotechnical engineer or an engineering geologist licensed in the State of California. Foundations should be deepened if necessary to reach satisfactory soils. Footing excavations deeper than 5 feet should be sloped back at 3/4:1 (horizontal to vertical) or shored. Footing excavations should be cleaned of any loosened soils before placing steel or concrete.

All applicable safety requirements, including OSHA requirements, should be met.

Footings should be poured as soon as possible after excavation and cleaning. Footing backfill and utility trench backfill should be mechanically compacted; flooding should not be permitted.

9.2.2 DRILLED PILING

9.2.2.1 Drilled Pile Capacities - Avenue 26 Station

As previously recommended, Avenue 26 Station should be supported on drilled piles extending into the natural soils below the embankment. The downward capacities of a 24-inch-diameter pile are shown in the following table.

Penetration into Natural Soils (Feet)	Downward Pile Capacity (kips)
5	30
10	70
15	110

The uplift capacity may be assumed to be one-half the downward capacity. End-bearing support is neglected, and it is not necessary to hand-clean the bottom before placing concrete.



Any piles in groups should be spaced at least $2\frac{1}{2}$ diameters on centers. If the piles are so spaced, no reduction in the downward capacities of the piles need be considered due to group action.

9.2.2.2 Settlement

The settlement of a 24-inch-diameter pile, supporting a load of 110 kips, will be about 1/4 inch.

9.2.2.3 Lateral Loads

The drilled piles may be used to resist lateral loads. The fill soils adjacent to a single 24-inch-diameter pile, at least 20 feet long, can resist horizontal loads imposed at the top of the pile up to 10,000 pounds. The lateral resistance of other sizes of piles may be assumed to be proportional to the diameter.

To determine the maximum bending moment in a pile, the lateral load imposed at the top of the pile may be multiplied by a moment arm of 5 feet. For design, it may be assumed that the maximum bending moment will occur near the top of the pile and that the moment will decrease to zero at a depth of 20 feet below the pile cap. The lateral capacity and reduction in the bending moment are based in part on the assumption that any required backfill adjacent to the pile caps and grade beams will be properly compacted.

A coefficient of friction of 0.4 may be used between the pile caps and the supporting soils. The passive resistance of the properly compacted fill or natural soils against pile caps and grade beams may be assumed to be equal to the pressure developed by a fluid with a density of 200 pounds per cubic foot. A one-third increase in the quoted passive value may be used for wind or seismic loads.



The resistance of the piles, the passive resistance of the soils against pile caps and grade beams, and the frictional resistance may be combined without reduction in determining the total lateral resistance.

9.2.2.4 Installation

We recommend that continuous observation of the drilling and pouring of the piles be performed by a geotechnical engineer or an engineering geologist licensed in the State of California to verify that the desired diameter and depth are obtained.

Closely spaced piles should be drilled and filled alternately, with the concrete permitted to set at least eight hours before drilling an adjacent hole. Pile excavations should be filled with concrete as soon after drilling and inspection as possible; the holes should not be left open overnight. The concrete should be placed with special equipment so that the concrete is not allowed to fall freely more than 5 feet and to prevent concrete from striking the walls of the excavation.

9.3 OVERHEAD CONTACT SYSTEM

Drilled cast-in-place reinforced concrete pile foundations may be used for support of the overhead contact system where the alignment is at-grade. No exceptional difficulties are expected in the installation of such foundations along the at-grade portions of the rail alignment. Some minor caving should be anticipated. For design, an average friction value of 400 pounds per square foot may be assumed for calculating the downward capacities of drilled piles. A passive pressure value of 500 pounds per cubic foot to a maximum of 5,000 pounds per square foot, may be used to calculate the lateral capacity of the piles if used in an acceptable pole formula, such as that contained in the Uniform Building Code.



9.4 TRACKS ON GRADE

If grading is performed as recommended in Section 13.0, Earthwork, tracks on grade may be supported by the conventional system consisting of ties, ballast, and sub-ballast established on firm natural soils or properly compacted fill.

The tracks may also be supported on concrete ties and reinforced concrete leveling course over an asphaltic concrete course and a subbase. There are other track support options that may be considered, if desired, such as in-place compaction of the subgrade soils, the use of geotextile fabric, soil cement, and others.

The allowable bearing value of the compacted subgrade soils may be assumed to be 20 pounds per square inch.

9.5 SLABS ON GRADE

The undisturbed natural soils and properly compacted fill will offer adequate support to slabs on grade. Any disturbed soils should be excavated and replaced with compacted fill; compaction to at least 90% is recommended. The required thicknesses and reinforcing of the concrete slabs will depend on the imposed loadings as well as the structural characteristics of the concrete. For design of concrete slabs, a modulus of subgrade reaction (k) of 150 pounds per cubic inch may be used.

Construction activities and exposure to the environment can cause deterioration of prepared subgrades. Therefore, we recommend that the condition of the final subgrade soils be checked immediately prior to slab on grade construction and, if necessary, further field density and moisture content tests be performed to determine the suitability of the final prepared subgrade.



9.6 ASPHALTIC PAVING

To provide support for paving, the subgrade soils should be prepared as recommended in Section 13.0, Earthwork. The existing fill soils should be excavated to a depth of at least 1 foot below the planned subgrade level in areas to be paved with asphaltic paving.

To provide data for design of asphaltic paving, California Bearing Ratio (CBR) tests were performed on 13 samples of the upper soils at the locations of passenger service stations, Southern Pacific Cornfield Yard and Shops, and Del Mar. The results are summarized in Table 9.1, Summary of CBR Test Data. The table does not include the results of CBR tests performed on samples from Borings 102 and 112.

TABLE 9.1
 SUMMARY OF CBR TEST DATA

Location	Boring No. and Sample Depth	Soil Type	CBR	
			90%*	95%*
Union Station	31 @ 0' to 4'	Fill - Silty Sand and Sandy Silt	17	36
	32 @ 0' to 2'	Fill - Silty Sand	24	55
Southern Pacific Cornfield Yard and Shops	39 @ 1' to 3'	Fill - Silty Sand	38	84
	44 @ 0' to 2'	Fill - Silty Sand	28	81
Avenue 26 Station	50 @ 1' to 2'	Fill - Silty Sand and Sandy Silt	14	46
Marmion Way Station	55 @ 1' to 2'	Silty Clay	7	16
Avenue 51 Station	63 @ 0' to 2'	Sandy Silt	17	32
Avenue 57 Station	Use data from Boring 63	Sandy Silt	17	32
Mission Street Station	79 @ 0' to 3'	Fill - Silty Sand and Sandy Silt	23	69
Fillmore Street Station	87 @ 1' to 4'	Silty Sand	23	52
Del Mar Station	89 @ 0' to 2'	Fill - Silty Sand	26	66
Lake Avenue Station	106 @ 0' to 2'	Fill - Silty Sand and Sandy Silt	22	47
Allen Avenue Station				
Sierra Madre Villa Station				

* Percent compaction based on ASTM Designation D1557-78 method of compaction.



The required paving thickness and base thickness will depend on the anticipated wheel loads and volume of traffic (Traffic Index). Assumed Traffic Indices and the corresponding paving sections for proposed stations are presented in Table 9.2, Recommended Paving Sections. The paving sections are based on the Flexible Pavement Structural Section Design Guide for California Cities and Counties, Third Edition.

TABLE 9.2
RECOMMENDED PAVING SECTIONS

Stations	Traffic Index	Thickness (Inches) Asphaltic Paving	Thickness (Inches) Base Course
Union Station, Avenue 51, Avenue 57, Mission Street, Fillmore Street, Del Mar Boulevard, Lake Avenue, Allen Avenue, Sierra Madre Villa	5	3	4
	6	3	6
	7	3	7
	8	4	9
Avenue 26	5	3	4
	6	3	6
	7	4	8
	8	5	9
Marmion Way	5	3	6
	6	4	7
	7	5	8
	8	6	12

Recommendations for Southern Pacific Cornfield Yard and Shops are presented separately in Section 10.

Careful inspection is recommended to verify that the recommended or greater thicknesses are achieved and that proper construction procedures are used. We could provide alternative paving sections for different Traffic Index values if desired.



The base course should meet the specifications for Class 2 Aggregate Base as defined in Section 26 of the State of California, Department of Transportation, Standard Specifications, latest edition. Alternatively, the base course could meet the specification for untreated base as defined in Section 200-2 of the latest edition of the Standard Specifications of Public Works Construction. The base course should be compacted to at least 95% of the maximum density.



SECTION 10.0 RECOMMENDATIONS – SOUTHERN PACIFIC (SP) CORNFIELD YARD AND SHOPS

10.1 FOUNDATION SUPPORT

10.1.1 GENERAL

Fill soils, up to about 3 feet in thickness, were encountered in the nine borings (Borings 6, 8, and 38 through 44) drilled within the area of the proposed yard and shops in the Southern Pacific Cornfield Yard. In addition, the demolition and removal of the existing facilities will result in disturbance of the upper soils. The upper natural soils consist primarily of silty sand, sand, and silt. The silty sand and silt soils are moderately firm to firm and the sand deposits are firm and dense.

We recommend that the existing fill soils be excavated and compacted and that all required additional fill soils be properly compacted. If the grading recommendations presented in Section 13.0, Earthwork, are followed, the proposed yard and shops facilities may be supported on spread footings established in the compacted fill and/or natural soils.

The recommendations for Tracks on Grade in Section 9.0 would be applicable to the tracks on grade in the yard and shop areas.

10.1.2 BEARING VALUES

Spread footings established in properly compacted fill and/or the undisturbed natural soils may be designed to impose a net dead plus live load pressure of 2,500 pounds per square foot. A one-third increase in bearing value may be used for wind or seismic loads. Exterior footings should extend at least 2 feet below the adjacent final grade; interior footings should extend at least 2 feet below the top of the adjacent floor slab. Since the recommended bearing value is a net value, the weight of concrete within the foundations may be taken as 50 pounds per cubic foot, and the weight of soil backfill may be neglected when computing the imposed downward foundation loadings.

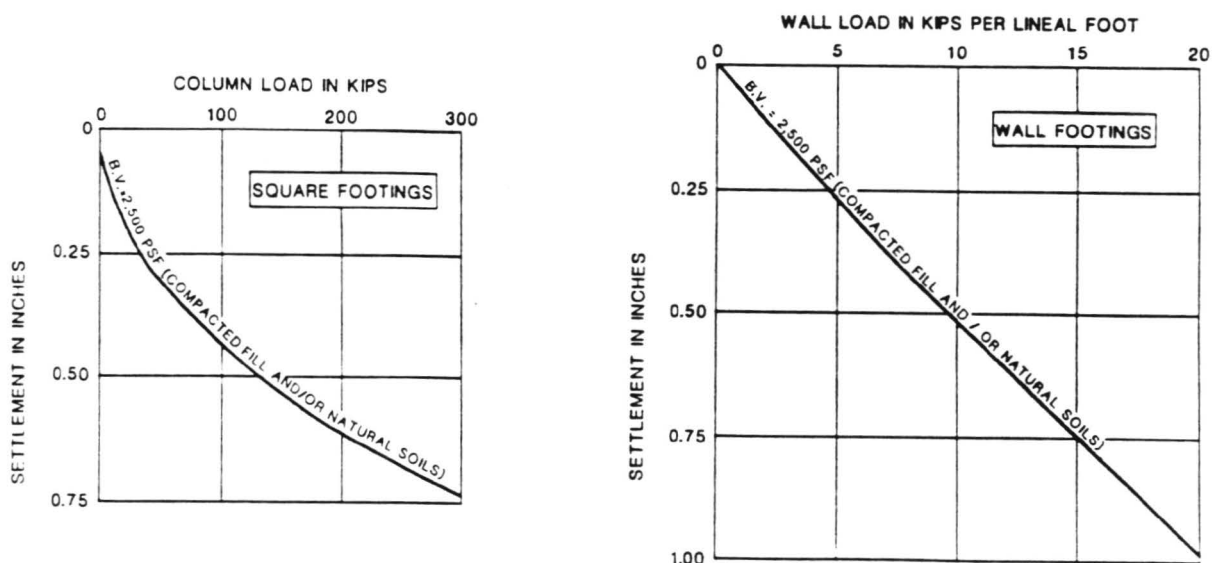


If desired, any loading dock walls, minor retaining walls, and free-standing walls may be supported on shallower footings imposing a lesser bearing value. Footings for such light loads established in properly compacted fill and/or the undisturbed natural soils, and extending at least 1½ feet below the lowest adjacent grade, may be designed to impose a pressure of 1,500 pounds per square foot. The settlement of such minor walls will be less than ½ inch.

While the actual bearing value of the compacted fill will depend on the material used and the compaction methods employed, the quoted bearing values will be applicable if acceptable soils are used and are properly compacted. The bearing value of the compacted fill should be confirmed after completion of the grading.

10.1.3 SETTLEMENT

The settlement of the proposed yard and shops facilities, supported on spread footings in the manner recommended, may be determined using the following graphs.



10.1.4 LATERAL LOADS

Lateral loads may be resisted by soil friction on footings and the floor slabs and by the passive resistance of the soils. A coefficient of friction of 0.5 may be used between footings or slabs and the supporting soils. The passive resistance of the natural soils or properly compacted backfill against footings may be assumed to be equal to the pressure developed by a fluid with a density of 300 pounds per cubic foot. A one-third increase in the passive value may be used for wind or seismic loads. The frictional resistance and the passive resistance of the soils may be combined without reduction in determining the total lateral resistance.

10.1.5 FOOTING OBSERVATION

To verify the presence of satisfactory soils at design elevations, all footing excavations should be observed by a geotechnical engineer or an engineering geologist licensed in the State of California. Footing excavations should be cleaned of any loosened soils before placing steel or concrete.

10.2 WALLS BELOW GRADE

For design of loading dock walls, pit walls, or retaining walls where the surface of the backfill is level, it may be assumed that the soils will exert an active lateral pressure equal to that developed by a fluid with a density of 30 pounds per cubic foot. The walls should also be designed for the appropriate lateral surcharge pressures from adjacent footings, if present.

Backfill adjacent to the walls should be mechanically compacted to at least 90% of the maximum dry density obtainable by the ASTM Designation D1557-78 method of compaction. The recommended lateral earth pressure is for properly drained backfill; weep holes or other means of drainage should be provided. Relatively non-expansive soils should be used for backfill adjacent to the walls.

10.3 SLABS ON GRADE

If the subgrade is prepared as recommended, slabs may be supported on grade. If a floor covering that would be critically affected by moisture, such as vinyl, is to be used, we would



suggest that floor slabs be supported on a four-inch-thick layer of gravel or on an impermeable membrane, such as a capillary break. For design of concrete slabs, a modulus of subgrade reaction (k) of 150 pounds per cubic inch, may be used. A suggested gradation for the gravel layer would be as follows:

Sieve Size	Percent Passing
3/4"	90 - 100
No. 4	0 - 10
No. 100	0 - 3

If the membrane is used, a low-slump concrete should be used to minimize possible curling of the slab. The concrete slab should be allowed to cure thoroughly before placing vinyl or other moisture-sensitive floor covering.

10.4 PAVING

10.4.1 SOIL PROPERTIES FOR PAVEMENT DESIGN

California Bearing Ratio tests were performed on selected samples within the yard and shops site to provide data for pavement design. The results are summarized in Table 10.1, Summary of CBR Test Data.

TABLE 10.1
 SUMMARY OF CBR TEST DATA

Boring No. and Depth	Soil Type	CBR	
		90%*	95%*
39 @ 1' to 3'	Fill - Silty Sand	38	84
44 @ 0' to 2'	Fill - Silty Sand	28	81

* Percent compaction based on ASTM Designation D1557-78 method of compaction



10.4.2 ASPHALTIC PAVING

The required paving and base thicknesses will depend on the anticipated wheel loads and volume of traffic (Traffic Index). Assumed Traffic Indices and the corresponding paving sections are presented in Table 10.2, Recommended Paving Sections. The paving sections are based on the Flexible Pavement Structural Section Design Guide for California Cities and Counties, Third Edition.

TABLE 10.2
RECOMMENDED PAVING SECTIONS

Traffic Index	Asphaltic Paving Thickness (Inches)	Base Course Thickness (Inches)
5	3	4
6	3	6
7	3	7
8	4	9

Careful inspection is recommended to verify that the recommended thicknesses or greater are achieved and that proper construction procedures are used.

10.4.3 BASE COURSE

The base course should meet the specifications for Class 2 Aggregate Base as defined in Section 26 of the State of California, Department of Transportation, Standard Specifications, latest edition. The base course should be compacted to at least 95% of the maximum density obtainable by the ASTM Designation D1557-78 method of compaction.



SECTION 11.0

RECOMMENDATIONS – BELOW-GRADE CONSTRUCTION (DEPRESSED AREAS)

11.1 GENERAL

The below-grade construction will cover the portion of the alignment at the intersection of Figueroa Street and Marmion Way in Highland Park (Borings 13, 56, and 57), and the portion of the alignment between Del Mar Boulevard and Holly Street in Pasadena (Borings 27 through 29 and 89 through 92). Based on a feasibility study report for the below-grade construction in Pasadena, the depressed area will consist of an open cut tunnel with bridges over the tunnel at the street levels. The tunnel will require up to 40 feet of excavation. The feasibility study report, dated October 24, 1991, was prepared by the Metro Rail Transit Consultants for the Rail Construction Corporation.

The soil and water conditions at the two depressed area locations are given below.

Highland Park

Shallow fill soils were encountered within this area. The fill is underlain by overburden natural soils that consist of silt, silty sand, and sand with lesser deposits of clay. The sandy deposits contain considerable amounts of gravel, cobbles, and boulders. Bedrock was encountered at depths ranging from about 15 to 50 feet below the existing grade.

Water was encountered as shallow as 21 feet below the existing grade in Boring 13 drilled near the Figueroa Street and Marmion Way location. Dewatering and a subdrain system will be required for the depressed area at this location. Adverse bedding was encountered in the bedrock underlying the area; the adverse bedding dips to the east at 35 degrees below the horizontal. The east facing shoring and walls below grade where bedrock is encountered should be designed for the adverse bedding at this location.



Pasadena

Fill soils, 1 to 6 feet deep, were encountered in the borings drilled within this area. The fill soils, which are not uniformly well compacted, consist of silt and silty sand and contain some debris. The underlying natural soils consist of sandy silt, silty sand, sand, and gravel with considerable amounts of cobbles. Water was not encountered within the 60-foot depth explored.

The right-of-way within a portion of the alignment (between Del Mar Boulevard and Holly Street) is only 30 feet wide, and the proposed excavation for the depressed area will extend close to the various structures adjacent to the alignment. Underpinning of some of the existing footings that are close to the depressed area may be necessary. As an alternative, the shoring and walls below grade could be designed to support the loads imposed by the adjacent existing foundations.

The retaining walls for the depressed areas may be supported on spread footings.

11.2 FOUNDATIONS

11.2.1 BEARING VALUES

Spread footings may impose bearing pressures up to 12,000 pounds per square foot in the bedrock and up to 8,000 pounds per square foot in the dense sand and gravel at the Highland Park location. For the Pasadena location, spread footings may impose bearing pressures up to 8,000 pounds per square foot in the dense sand and gravel.

Footings should extend at least 1 foot into the supporting material and should be established at least 3 feet below the lowest adjacent final grade. A one-third increase in the recommended values may be used for wind or seismic loads. The recommended bearing values are net values. The weight of concrete in the footings may be assumed to be 50 pounds per cubic foot, and the weight of soil backfill may be neglected when determining the downward load on the footings.



11.2.2 LATERAL LOADS

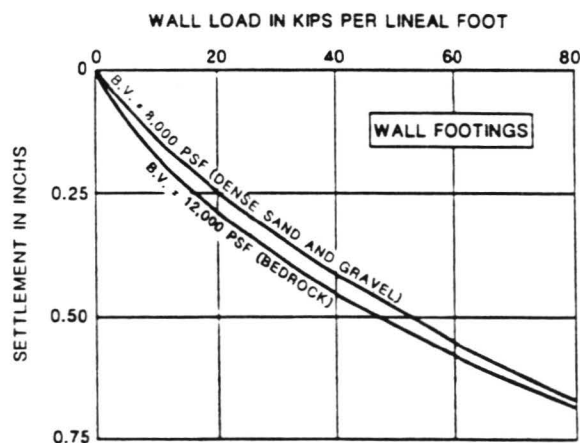
Lateral loads may be resisted by soil friction and by the passive resistance of the soils and bedrock materials. A coefficient of friction of 0.5 may be used between footings and the supporting materials. The following passive values may be used:

Bedrock	1,000 pounds per square foot at the bedrock surface plus 300 pounds per square foot for each additional foot of depth up to a maximum of 10,000 pounds per square foot (Highland Park only)
Overburden Soils Compacted Fill	300 pounds per square foot per foot of depth

A one-third increase in the passive values may be used for wind or seismic loads. The frictional resistance and the passive resistance of the soils and bedrock may be combined without reduction in determining the total lateral resistance.

11.2.3 SETTLEMENT

The settlement of spread footings established in the bedrock or in the dense sand and gravel may be determined using the following graph:



11.2.4 FOOTING OBSERVATION

The footing excavations should be carefully observed to verify the presence of satisfactory materials at footing design elevations. The exposed bedrock and bedding at the Highland Park location should be observed by an engineering geologist, licensed in the State of California, to confirm that the structural features of the bedrock at the planned foundation locations are favorable for foundation support. Footings should be deepened if necessary to extend into satisfactory supporting material.

The foundations should be left slightly uneven if necessary, rather than filling in overexcavated areas with loose or compacted soils. Where it is necessary to deepen a footing below the design depth, the overexcavated portion should be backfilled with concrete.

It would be desirable and feasible to pour spread footings neat against the undisturbed bedrock. Soil backfill should be compacted as recommended in Section 13.0, Earthwork.

Subsurface seepage may be encountered in the footing excavations. A layer of lean concrete may be used to protect the bottom of the excavation and to provide support during construction.

11.3 EXCAVATION AND SLOPES

Where space is not available for sloped cuts, shoring will be required. Shallow excavations are planned at the beginning and at the end of the below-grade construction. Where space is available at these locations, temporary unsurcharged excavations could be sloped back at 1:1. Preliminary data for shoring design are presented in a following section. All applicable safety requirements, including OSHA requirements, should be met.

Where sloped embankments are used, the tops of the slopes should be barricaded to prevent vehicles and storage loads within 5 feet of the tops of the slopes. A greater setback may be necessary when considering heavy vehicles, such as concrete trucks and cranes. If the



temporary 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 excavation and eroding the slope faces. The soils exposed in the cut slopes should be inspected during excavation so that modifications of the slopes can be made if variations in the soil conditions occur, or if adverse seepage conditions develop.

11.4 SHORING

11.4.1 GENERAL

Where there is not sufficient space for sloped embankments, shoring will be required. One method of shoring would consist of steel soldier piles placed in drilled holes, backfilled with concrete, and tied back with drilled-in friction anchors. However, because of space limitations and the granular nature of the subsurface soils, it would be very difficult to install tie-back anchors, and the installation of cantilevered, cross-braced, or internally braced shoring would be more practical. Another method of shoring being considered consists of tangent or secant piles that are cantilevered or cross braced.

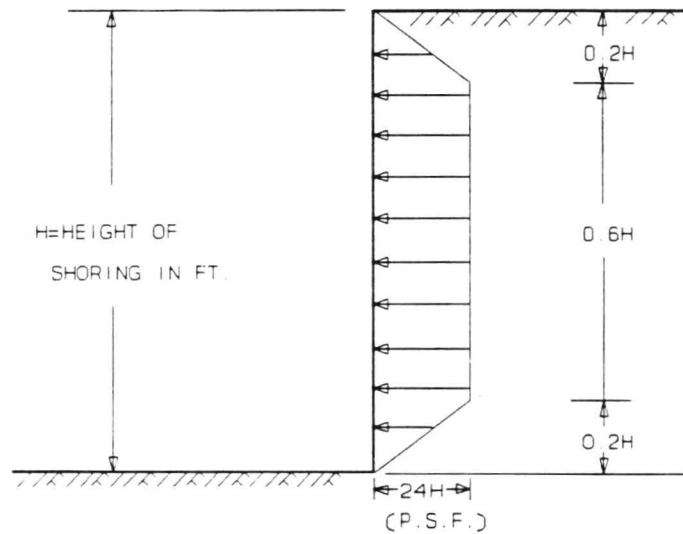
11.4.2 LATERAL PRESSURES

For design of cantilevered shoring, a triangular distribution of lateral earth pressure may be used. It may be assumed that the retained soils with level surface behind the cantilevered shoring will exert an active lateral pressure equal to that developed by a fluid with a density of 33 pounds per cubic foot. For the Highland Park depressed area where there is adverse bedding, the east facing shoring should be designed for an equivalent fluid pressure of 45 pounds per cubic foot.

When considering at rest earth pressure on cantilevered shoring, it may be assumed that the retained soils with level surface behind the shoring will exert a lateral pressure equal to that developed by a fluid with a density of 52 pounds per cubic foot. For the Highland Park depressed area, where there is adverse bedding, the east-facing shoring should be designed for an equivalent fluid pressure of 70 pounds per cubic foot.



For design of cross-braced or internally braced shoring, we recommend the use of a trapezoidal distribution of active earth pressure. The recommended pressure distribution, for the case where the grade is level behind the shoring, is illustrated below with the maximum pressure equal to $24H$ in pounds per square foot, where H is the height of the shoring in feet.



For the Highland Park depressed area where there is adverse bedding, the maximum earth pressure would be $32H$ in pounds per square foot.

When considering "at rest" earth pressure on cross-braced or internally braced shoring, the lateral earth pressure would be similar to that presented above, except that the maximum earth pressure would be $37H$ pounds per square foot. For the Highland Park depressed area, where there is adverse bedding, the maximum earth pressure would be $50H$ in pounds per square foot.



In addition to the recommended earth pressure, the upper 10 feet of shoring adjacent to the streets and traffic areas should be designed to resist a uniform lateral pressure of 100 pounds per square foot, which is a result of an assumed 300 pounds per square foot surcharge behind the shoring due to normal street traffic. If the traffic is kept back at least 10 feet from the shoring, the traffic surcharge may be neglected.

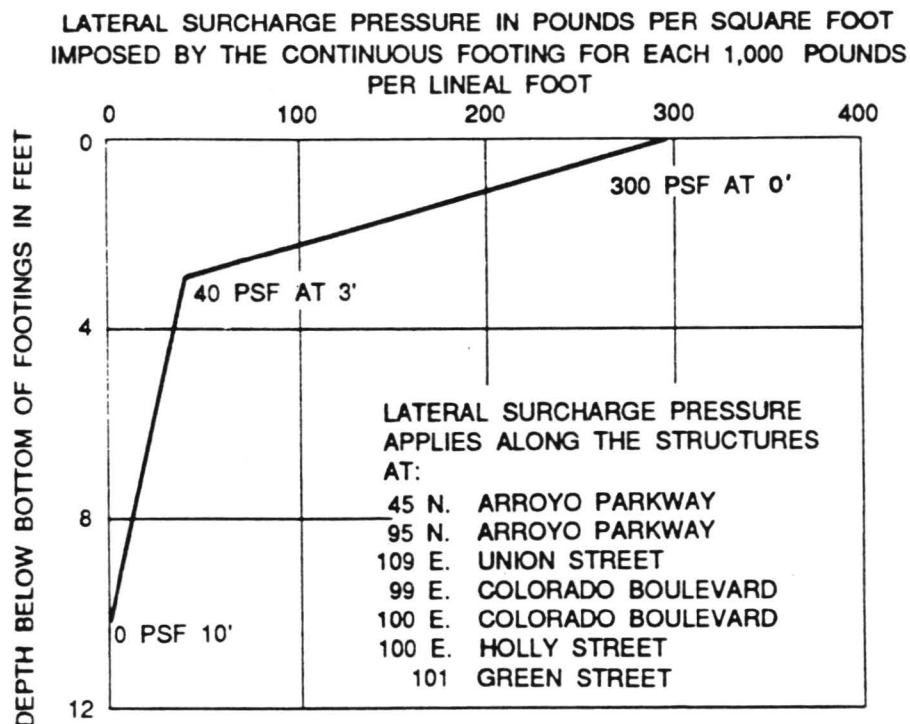
11.4.3 SURCHARGE IMPOSED BY EXISTING STRUCTURES

In addition, the shoring (and walls below grade) should be designed to resist the lateral surcharge pressures imposed by the foundations of the adjacent existing structures within the depressed area in Pasadena. The foundation load information for some of the existing structures located adjacent to the depressed area in Pasadena was provided to us on February 3, 1993 by Mr. Guy Morrow of Degenkolb, Structural Engineers. These structures are discussed below. The lateral surcharge pressures imposed by the continuous footings on the adjacent shoring and walls below grade are discussed on the following pages.



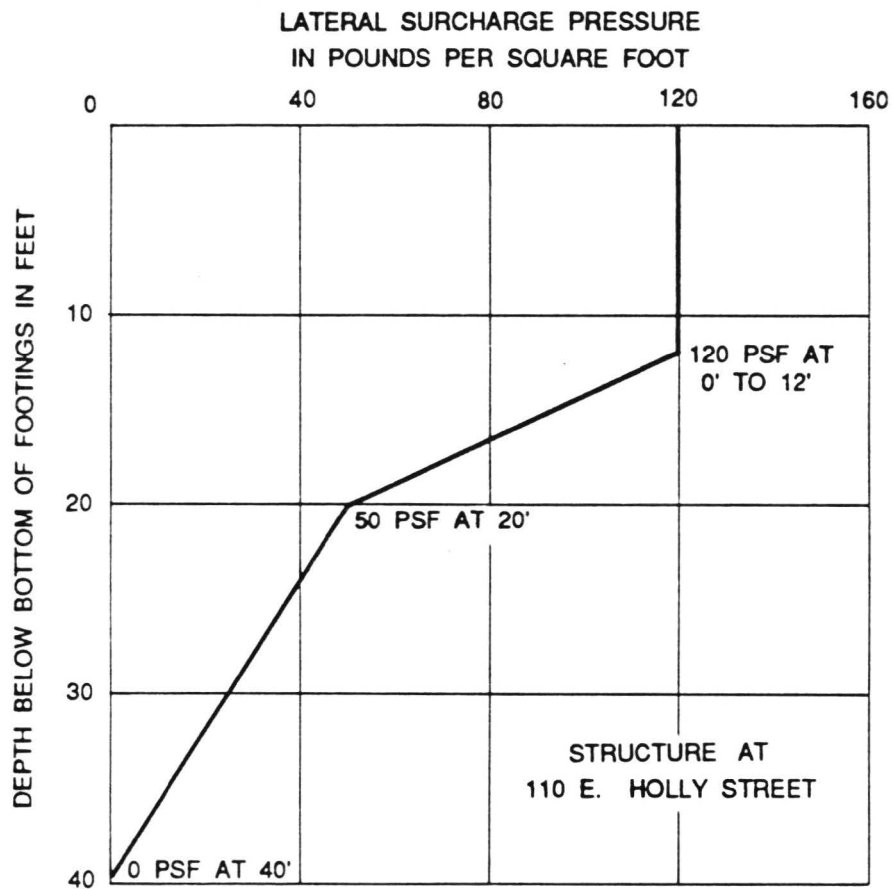
(1) Structures at 45 North Arroyo Parkway; 95 North Arroyo Parkway; 109 East Union Street; 99 East Colorado Boulevard; 100 East Colorado Boulevard; 100 East Holly Street; and 101 Green Street.

We have been advised that each of these structures is supported on 3-foot-wide continuous footings adjacent to the planned excavation. The soil pressures imposed by the footings vary between 1,000 and 10,000 pounds per lineal foot. The pressures shown below are in pounds per square foot for each 1,000 pounds per lineal foot of soil pressure.



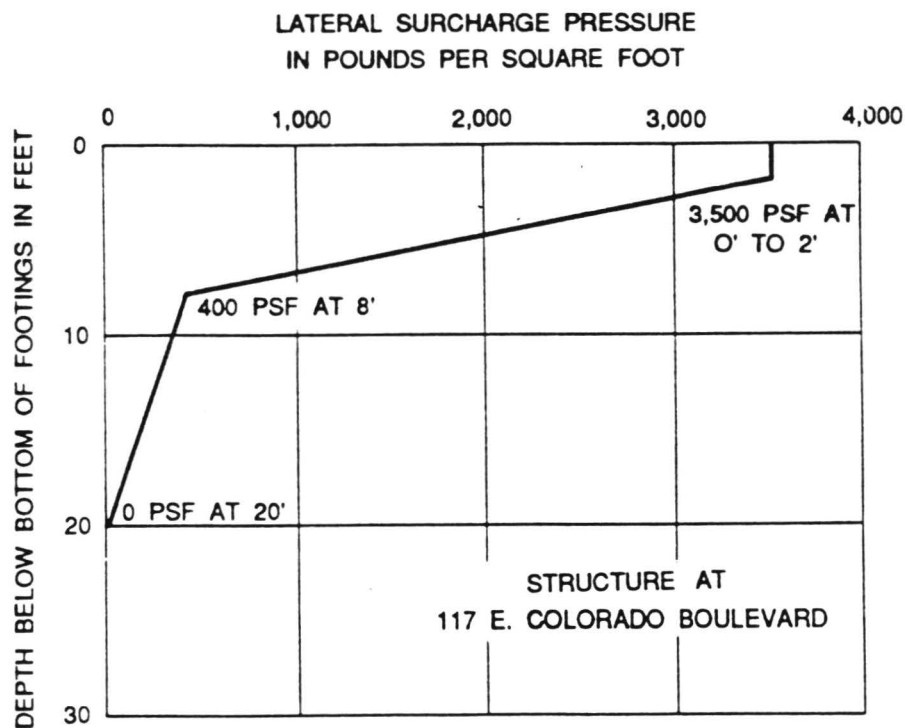
(2) Structure at 110 East Holly Street

We have been advised that the building located at 110 East Holly Street is supported on a 3-foot-wide continuous footing adjacent to the planned excavation. The closest edge of the footing is 11 feet from the excavation; the footing imposes a soil pressure of 4,500 pounds per lineal foot. The lateral surcharge pressures imposed by the structure on the planned shoring and wall are shown below.



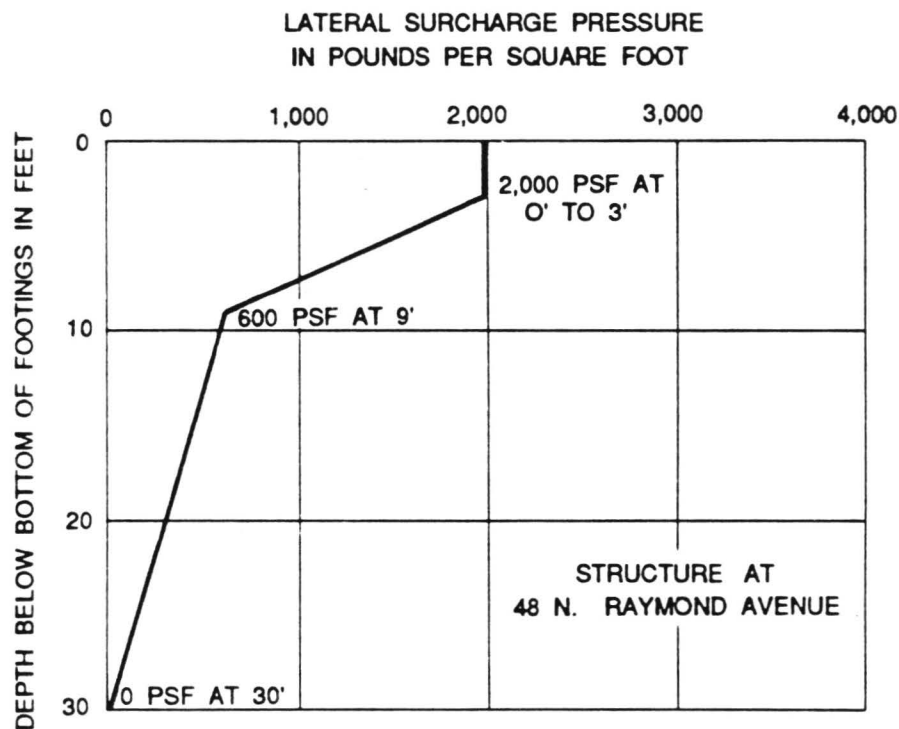
(3) Structure at 117 East Colorado Boulevard

We have been advised that the structure has one basement level and is supported on a combination of square footings and a continuous footing along the planned excavation. The square footings are spaced at 20 feet on centers, and each footing supports a dead plus live column load of 340 kips. The continuous footing is 5 feet wide and imposes a soil pressure of 11,000 pounds per lineal foot. It is assumed that the square footings impose a soil pressure of 6,000 pounds per square foot. The lateral surcharge pressures imposed by the structure are shown below.



(4) Structure at 48 North Raymond Avenue

We understand that the structure has one basement level and is supported on a combination of an exterior continuous footing and interior square footings. The exterior continuous footing along the planned excavation is 5 feet wide and imposes a soil pressure of 14,500 pounds per lineal foot. The interior square footings are 15½ feet square, and each footing supports dead plus live column loads of 600 kips. The footings are spaced 20 to 30 feet on centers. The closest edge of the interior footing is 6 feet from the planned excavation. The lateral surcharge pressure imposed by the footings on the planned excavation is shown below.



11.4.4 DESIGN OF SOLDIER PILES

For the design of soldier piles spaced at least two diameters on centers, the allowable lateral bearing value (passive value) of the overburden soils below the level of excavation may be assumed to be 600 pounds per square foot per foot of depth, up to a maximum of 6,000 pounds per square foot. The bearing value (passive value) of the bedrock materials below the level of excavation may be assumed to be 2,000 pounds per square foot at the bedrock surface increasing by 600 pounds per square foot for each additional foot of depth, up to a maximum of 10,000 pounds per square foot. To develop the full lateral value, provisions should be taken to assure firm contact between the soldier piles and the undisturbed soils. The concrete placed in the soldier pile excavations may be a lean-mix concrete. However, the concrete used in that portion of the soldier pile below the planned excavated level should be of sufficient strength to adequately transfer the imposed loads to the surrounding soils. If the shoring is to be incorporated into the permanent below-grade walls, structural concrete should be used below the excavated level.

Difficulties due to caving, gravel, cobbles, and boulders should be anticipated and special precautions should be taken during installation of the soldier piles. To minimize the amount of caving within the soldier pile shafts, we suggest the use of bucket-type drilling equipment; the drilling speed should be reduced as necessary to minimize vibration and sloughing of the soils.

The coefficient of friction between the soldier piles and the retained earth may be taken as 0.4. (This value is based on the assumption that uniform full bearing will be developed between the steel soldier beams, the lean-mix concrete, and the retained earth). In addition, the soldier piles below the excavated level may be used to resist downward loads. The frictional resistance between the concrete soldier piles and the soils below the excavated level may be taken as equal to 500 pounds per square foot.



11.4.5 DESIGN OF SECANT OR TANGENT PILES

For the design of secant or tangent piles, the allowable lateral bearing value (passive value) of the soils below the level of excavation may be assumed to be 300 pounds per square foot per foot of depth, up to a maximum of 6,000 pounds per square foot.

11.4.6 LAGGING

Because of the sandy nature of the overburden soils and the adverse bedding planes in the bedrock, continuous lagging will be required between the soldier piles. If the spacing between the soldier piles is 6 feet or less, the lagging may be omitted in the bedrock where there is no adverse bedding. As previously stated, adverse bedding, dipping to the east at 35 degrees below the horizontal, was encountered in the bedrock in the borings drilled at the location of the depressed area in Highland Park.

Careful installation of the lagging will be necessary to achieve bearing against the retained earth. Because of the gravel, cobbles, and boulders, it will be difficult to cut a smooth vertical surface at some locations, and back-packing behind the lagging will be required to achieve full bearing. It may be necessary to grout the voids behind the lagging to obtain full contact with the soil.

The soldier piles should be designed for the full anticipated lateral pressure. However, the pressure on the lagging will be less due to arching in the soils. We recommend that the lagging be designed for the recommended earth pressure but limited to a maximum value of 400 pounds per square foot for the overburden soils and a value of 600 pounds per square foot where adverse bedding occurs.



11.4.7 ANCHOR DESIGN

Tie-back friction anchors may be used to resist lateral loads. For design purposes, it may be assumed that the active wedge adjacent to the shoring is defined by a plane drawn at 35 degrees with the vertical through the bottom of the excavation. The anchors should extend at least 20 feet beyond the potential active wedge, and to a greater length if necessary to develop the desired capacities.

The capacities of anchors should be determined by testing of the initial anchors as outlined in a following section. For design purposes, it may be estimated that drilled friction anchors will develop an average friction value of 700 pounds per square foot. Only the frictional resistance developed beyond the active wedge would be effective in resisting lateral loads. If the anchors are spaced at least 6 feet on centers, no reduction in the capacity of the anchors need be considered due to group action.

11.4.8 ANCHOR INSTALLATION

The anchors may be installed at angles of 15 to 40 degrees below the horizontal. Caving of the anchor holes should be anticipated and provisions made to minimize such caving. The anchors should be filled with concrete placed by pumping from the tip out, and the concrete should extend from the tip of the anchor to the active wedge. To minimize caving, we suggest that the portion of the anchor shaft within the active wedge be backfilled with sand before testing the anchor. This portion of the shaft should be filled tightly and flush with the face of the excavation. The sand backfill may contain a small amount of cement to allow the sand to be placed by pumping.

11.4.9 ANCHOR TESTING

At least two of the initial anchors should be selected for a 24-hour 200% test, and ten additional anchors should be selected for quick 200% tests. The purpose of the 200% tests is to verify the friction value assumed in design. The anchors should be tested to develop twice the assumed friction value. 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.

The total deflection during the 24-hour 200% tests should not exceed 12 inches; the anchor deflection should not exceed 0.75 inch during the 24-hour period, measured after the 200% test load is applied. If the anchor movement after the 200% load has been applied for 12 hours is less than 0.5 inch, and the movement over the previous 4 hours has been less than 0.1 inch, the test may be terminated.

For the quick 200% tests, the 200% test load should be maintained for 30 minutes. The total deflection of the anchor during the 200% quick test should not exceed 12 inches; the deflection 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 production 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 for the anchor to be approved for the design loading.

After a satisfactory test, each production 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 geotechnical engineer licensed in the State of California.



11.4.10 DEFLECTION

It is difficult to accurately predict the amount of deflection of a shored embankment. It should be realized, however, that some deflection will occur. We estimate that this deflection could be on the order of 1 inch at the top of the shored embankment. If greater deflection occurs during construction, additional bracing may be necessary to minimize settlement of the utilities in the adjacent streets. If desired to reduce the deflection of the shoring, a greater active pressure could be used in the shoring design.

11.4.11 INTERNAL BRACING

Internal or cross-bracing may be used to brace the soldier piles or the tangent or secant piles. The bracing should be preloaded to about 50% of the design loads.

11.4.12 MONITORING

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.

11.5 WALLS BELOW GRADE

For the design of cantilevered retaining walls below grade, where the backfill is level and properly drained, it may be assumed that the soils will exert an active lateral earth pressure equal to that developed by a fluid with a density of 33 pounds per cubic foot. For the Highland Park depressed area where there is adverse bedding, the east facing walls should be designed for an equivalent fluid pressure of 50 pounds per cubic foot.

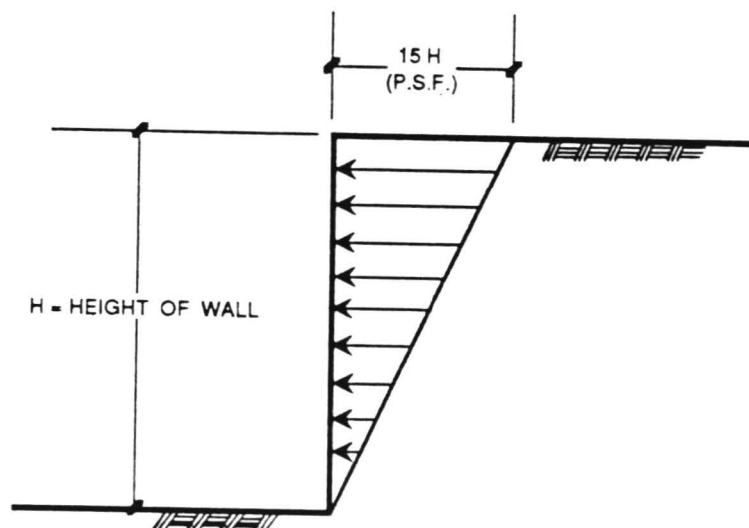
When considering "at rest" earth pressure on cantilevered walls, it may be assumed that drained backfill soils will exert an "at rest" active pressure equal to that developed by a fluid with a density of 52 pounds per cubic foot. For the Highland Park depressed area, where there is adverse bedding, the east-facing wall should be designed for an equivalent fluid pressure of 70 pounds per cubic foot.



We recommend that permanent braced walls below grade be designed to resist a trapezoidal distribution of lateral earth pressure plus the surcharge loading occurring as a result of traffic in the areas with vehicular traffic. The lateral earth pressure on the permanent walls below grade will be $24H$ pounds per square foot, where H is the height of the wall in feet. For the Highland Park depressed area where there is adverse bedding, the maximum earth pressure would be $35H$ in pounds per square foot per cubic foot.

When considering "at rest" earth pressure on permanent braced walls below grade, the lateral earth pressure will be $37H$ pounds per square foot, where H is the height of the wall in feet. For the Highland Park depressed area, where there is adverse bedding, the maximum earth pressure would be $50H$ in pounds per square foot.

In addition to the lateral earth pressure, walls below grade should be designed to resist the lateral seismic loading imposed on the walls during an earthquake. The maximum seismic loading may be assumed as $15H$ in pounds per square foot, where H is the height of the wall in feet. The recommended lateral surcharge pressure distribution is shown below.



In addition to the recommended lateral earth pressure, the upper 10 feet of walls below grade adjacent to any traffic should be designed to resist a lateral pressure of 100 pounds per square foot, acting as a result of an assumed 300 pounds per square foot surcharge behind the wall due to normal traffic. If the traffic is kept back at least 10 feet from the walls, the traffic surcharge may be neglected. Walls below grade should also be designed to resist lateral surcharge pressures imposed by adjacent existing structures along the alignment.

The above recommended values are for properly drained soils; weep holes or backdrains should be provided to relieve hydrostatic pressures due to water. If the soils behind the walls are not properly drained, the walls should be designed to resist the appropriate hydrostatic pressure in addition to the previously stated pressures.

11.6 SUBDRAIN

Water was encountered in the borings above the planned level of excavation at the Figueroa/Marmion intersection in Highland Park; the water is perched on top of the bedrock. There are two alternative procedures that might be followed for the below-grade structure at this location. A permanent subdrain system could be installed beneath the slab below the water elevation to maintain the water below the lower grade, or the slab and the lower portions of the walls could be waterproofed and designed for the possible hydrostatic pressure. The design of the lower slab to resist the possible hydrostatic pressure would require a thorough waterproofing installation and relatively thick slab. Installation of a completely watertight waterproofing system will be difficult. If such a system is desired, we suggest consulting with a contractor experienced in the installation of such systems.

For a subdrain system, a permit from the State of California Regional Water Quality Control Board will have to be obtained to discharge the subdrain water into the storm drain. To obtain such a permit, chemical tests will have to be performed on ground water samples obtained at the site to verify that chemicals or pollutants within the water do not exceed the allowable limits for discharging into the storm drain.



For a subdrain system, we recommend that the slab be underlain by a layer of filter material approximately 1 foot in thickness. (This recommendation could be reviewed based upon the water conditions at the time of construction.) The filter material should be drained by subdrain pipes leading to sump areas equipped with automatic pumping units. We suggest that the filter material meet the requirements of Class 2 Permeable Material as defined in Section 68 of the State of California, Department of Transportation, Standard Specifications, latest edition. (If the Class 2 material is not available, 3/4-inch crushed rock over filter fabric may be used. The crushed rock should have less than 5% passing No. 200 sieve.) The drain lines should consist of perforated pipes placed, with the perforations down, in trenches extending at least 1 foot below the filter material. The trenches should be backfilled with material meeting the requirements of the Class 2 Permeable Material. As an alternative to the Class 2 Permeable Material, 3/4-inch crushed rock underlain by a suitable filter fabric may be used. The drain lines should extend along the walls, and the grade should be crowned to facilitate drainage. A slope of at least 2 inches per 100 feet should be used for the drain lines.

In addition to the drainage system, some means of draining the soils outside the exterior walls will be required. The means of accomplishing drainage outside the walls will depend primarily on the selected method of shoring and the method of constructing the exterior walls. Drainage behind the walls may be provided by strips of Miradrain 6000 (or equivalent). In our opinion, Miradrain 6000 (or equivalent), attached to the lagging or the earth and protected from the concrete placement of the walls, would provide satisfactory drainage. The Miradrain strips may be placed at a depth starting at about 4 feet below the existing grade. The strips should be at least 4 feet wide and placed at 8 feet on centers.

The Miradrain should be connected to weep holes at the bottom of the excavation. The weep holes should consist of solid pipes, and should be spaced at 8 feet on centers. At the connection of the weep holes and the Miradrain, the weep holes should be embedded in 1 cubic foot of free-drainage aggregate. The weep holes could be connected to a solid drainage



pipe placed beneath the edge of the lower slab, or could be connected to the drainage pipes located in the trenches of the subdrain system placed beneath the slab.

Where needed, waterproofing could be placed against the Miradrain. The waterproofing could consist of Volclay™ panels, and an impermeable membrane, or any other equivalent system.

The installed drainage system should be observed by personnel from our firm prior to being backfilled. Inspection of the drainage system may also be required by the reviewing governmental agencies. In addition, we suggest that the design be reviewed after the excavation has been completed. If necessary, the system could be modified as indicated by the observed conditions, including the quantity of water pumped during construction dewatering.

We recommend that the drainage and waterproofing systems be designed by a waterproofing expert and be installed by a contractor experienced in the installation of such systems. We will be happy to provide the designer and the contractor with any additional information needed for the design and installation of the drainage and waterproofing systems.

Water was not encountered in the vicinity of the depressed section in Pasadena. However, to intercept possible water infiltration and to prevent buildup of hydrostatic pressure, a drain could be installed at the base of the walls.



SECTION 12.0

RECOMMENDATIONS - MISCELLANEOUS STRUCTURES

12.1 RETAINING WALLS

12.1.1 GENERAL

In addition to the abutment walls, free-standing retaining walls will be required at various locations; the walls will support both level and sloping grades. This section covers such free-standing retaining walls. The abutment walls should be designed in accordance with the data presented in the section on the bridges associated with the abutment walls. The design data presented in this section are based on the strength of the compacted fill. The data are applicable to the compacted fill, bedrock, or undisturbed natural soils.

12.1.2 FOUNDATIONS

12.1.2.1 Spread Footings

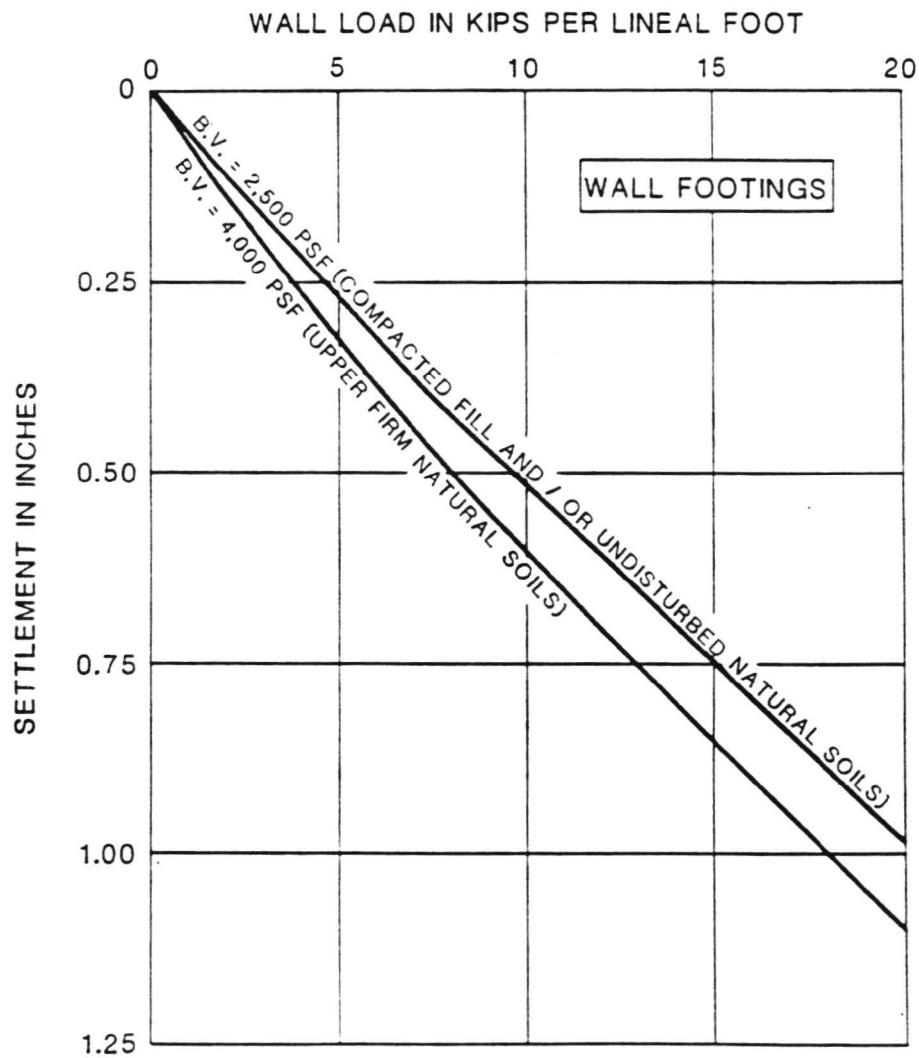
Retaining wall footings established in compacted fill soils and/or undisturbed natural soils may be designed to impose a maximum gross pressure of 2,500 pounds per square foot. Retaining wall footings established in undisturbed upper natural materials may be designed to impose a maximum gross pressure of 4,000 pounds per square foot. Footings should extend at least 2 feet below the lowest adjacent grade. In areas where the retaining walls will transition from soil to bedrock, some differential movement should be anticipated. This differential movement can be accommodated by constructing joints in the wall.

A coefficient of friction of 0.5 may be used between the wall footings and the supporting soils. The passive resistance of the natural soils or properly compacted fill against the footings may be assumed to be equal to the pressure developed by a fluid with a density of 300 pounds per cubic foot.



12.1.2.2 Settlement

The settlement of spread footings established in either properly compacted fill or the underlying natural soils, and of spread footings established in the upper natural materials may be determined using the following graph:



12.1.2.3 Drilled Piles

As an option to the use of spread footings, drilled cast-in-place concrete piles may be used. The downward capacities of a 24-inch-diameter pile are shown in the following table.

Pile Length (feet)	Downward Pile Capacity (kips)
10	30
20	75
30	135
40	210

The uplift capacity may be assumed to be one-half the downward capacity. End-bearing support is neglected, and it is not necessary to hand-clean the bottom before placing concrete.

The stated downward and upward capacities are based on penetration into compacted fill soils and undisturbed natural materials. The capacity developed within uncompacted fill soils, disturbed natural materials, and slide debris should be ignored in computing the pile capacities.

Any piles in groups should be spaced at least $2\frac{1}{2}$ diameters on centers. If the piles are so spaced, no reduction in the downward capacities of the piles need be considered due to group action.

The settlement of a 24-inch-diameter pile, supporting a load of 210 kips, is estimated to be less than $\frac{1}{2}$ inch.

For computing the lateral resistance of the drilled piles, an acceptable pole formula, such as the one in the Uniform Building Code, may be used. When using the pole formula, the lateral resistance of the fill soils may be assumed to be equal to the pressure developed by a fluid



with a density of 600 pounds per cubic foot, up to a maximum of 4,000 pounds per square foot. Required backfill adjacent to pile caps should be properly compacted.

12.1.3 LATERAL EARTH PRESSURES

For design of cantilevered retaining walls where the surface of the backfill is level, it may be assumed that the soils will exert a lateral pressure equal to that developed by a fluid with a density of 30 pounds per cubic foot. Where the surface of the retained earth is sloped at 2:1 (horizontal to vertical), it may be assumed that the soils will impose an active lateral pressure equal to that developed by a fluid with a density of 45 pounds per cubic foot.

The recommended lateral earth pressures are for walls that will not be affected by any adverse bedding adjacent to the wall backfill. We understand that a retaining wall is planned in Highland Park to the north of Marmion Station. The height of the wall is not known at this time. However, adverse bedding planes were encountered in the bedrock in the vicinity of the wall. Adverse bedding, dipping to the east at 8 to 32 degrees below the horizontal, was measured. All east facing retaining walls in this area should be designed to resist an active pressure of 50 pounds per cubic foot.

The above recommended values are for properly drained soils; weep holes or other drainage measures should be provided to prevent the development of hydrostatic pressures due to water. If the soils behind the walls are not properly drained, the walls should be designed to resist the appropriate hydrostatic pressure in addition to the previously stated pressures.

12.1.4 SURCHARGE PRESSURES

In addition to the earth pressure, an appropriate uniform lateral surcharge pressure should be considered due to any roadway traffic adjacent to the walls. This uniform pressure may be assumed to be equal to one-third of the surcharge applied within a distance from the top of the wall equal to the wall height.



12.1.5 BACKFILL

All required soil backfill behind retaining walls should be compacted in accordance with the compaction criteria in Section 13.0. Some settlement of the deep backfill should be allowed for in planning sidewalks and utility connections.

12.2 SLOUGH WALLS

We recommend that slough walls or debris collection structures be installed adjacent to the toes of the slopes located along the Metro Pasadena Line alignment. The slough walls should have a freeboard of at least 5 feet.

The slough walls or debris collection structures should be designed to resist an equivalent fluid pressure of 125 pounds per cubic foot. All soil or debris that may accumulate behind the wall or structure should be cleaned out periodically. Proper provisions for the diversion of surface water should be incorporated in the design.

12.3 CULVERTS

12.3.1 GENERAL

It is anticipated that culverts may be required at various locations. The culverts could be designed in accordance with Caltrans Standards.

12.3.2 SUPPORT

If the subgrade is properly prepared, the natural soils, properly compacted fill, or bedrock will offer adequate support for the culverts. The Caltrans bedding criteria would be applicable to all culvert pipe.

12.3.3 LATERAL PRESSURES

For the design of culvert walls below grade, where the surface of the retained earth is level, we recommend an active lateral earth pressure equal to that developed by a fluid with a density of 30 pounds per cubic foot. The surface of the finished grade above the cuvert should



be considered in computing the lateral surcharge pressure on the culvert walls. In addition, a lateral surcharge pressure should be included in the design for any traffic surcharge.

12.3.4 BACKFILL

All required soil backfill behind retaining walls should be compacted in accordance with the compaction criteria in Section 13.0, Earthwork.

The top of the culvert should be designed for the full weight of the fill soils directly over the culvert plus the anticipated traffic load. For determining the earth load, a moist unit weight of 125 pounds per cubic foot may be used for the overlying fill.

12.4 SIGN STRUCTURES

12.4.1 GENERAL

Sign structures will be required at various locations. The structures could be designed in accordance with Caltrans Standards. It should be feasible to support the signs on drilled cast-in-place concrete piling. The design data presented below are based on an average strength of compacted fill, bedrock, and the natural soils. We recommended a minimum pile diameter of 16 inches and a minimum length of 10 feet.

12.4.2 DRILLED PILES

The downward capacities of a 16-inch-diameter pile are shown in the following table.

Pile Length (feet)	Downward Pile Capacity (kips)
10	20
20	50
30	90
40	140



The uplift capacity may be assumed to be one-half the downward capacity. End-bearing support is neglected; it is not necessary to hand-clean the bottom of the drilled excavations before placing concrete.

For computing the lateral resistance of the drilled piles, an acceptable pole formula, such as the one in the Uniform Building Code, may be used. When using the pole formula, the lateral resistance of the supporting materials may be assumed to be equal to the pressure developed by a fluid with a density of 600 pounds per cubic foot, up to a maximum of 4,000 pounds per square foot. Required backfill adjacent to pile caps should be compacted to at least 90% of the maximum density obtainable by the ASTM Designation D1557-78 method of compaction.



SECTION 13.0

RECOMMENDATIONS - EARTHWORK

13.1 SITE PREPARATION

Prior to starting of grading operations, all affected structures and improvements should be removed, and existing vegetation should be stripped and removed from the site. The material resulting from the stripping operation should not be used in required structural fills.

After clearing the site, any existing fill and any soft or disturbed natural soils should be excavated. We recommend that all of the existing fill material below foundations and slabs be removed. Within areas to be paved, at least the upper 1 foot of existing fill should be excavated; deeper excavation may be required to remove unsuitable deposits. The deep fill within the Interstate 210 Freeway embankment may remain; to provide improved support for the stations, at least the upper 2 to 3 feet of existing fill should be excavated and compacted. To provide uniform support for tracks on grade, the upper fill soils should be excavated to permit placing at least 1 foot of properly compacted subgrade fill beneath the subgrade ballast.

Site preparation, excavation, backfill, fill placement and compaction, construction of embankments and slopes, and selection of fill materials for the proposed Metro Pasadena Line, in general, should be in accordance with the earthwork specifications as defined in Section 19 of the most recent edition of the State of California, Department of Transportation, Standard Specifications.

The following test procedures are used throughout this report and should be used in development of the project specifications:

Maximum Density
In-Place Field Density
Moisture Content
Expansion Index

ASTM D1557-78
ASTM D1557-82
ASTM D2216-90
ASTM D4829-88



13.2 EXCAVATION AND SLOPES

The excavation within the overburden soils and within the underlying bedrock may be made with conventional heavy duty earth-moving equipment.

Permanent cut and fill slopes constructed at 2:1 (horizontal to vertical) will have a satisfactory degree of safety. Permanent cut slopes in bedrock may be constructed at 1½:1, or at the dip of bedding, whichever is flatter. The requirements of the controlling agency will be satisfactory on this site with regard to drainage devices.

Temporary unsurcharged excavations in the existing fill and natural soils may be made at 1:1, and temporary unsurcharged excavations in the sedimentary bedrock may be made at 3/4:1 (horizontal to vertical). Where adverse geologic conditions would be exposed, temporary cuts should be laid back to the angle of the bedding. If the necessary space is not available, shoring and slot cutting may be required.

All applicable safety requirements, including OSHA requirements, should be met.

All cuts, temporary and permanent, should be observed by a competent engineering geologist to verify the anticipated conditions.

13.3 COMPACTED FILL

After excavating as recommended, the exposed soils should be inspected, and further excavation performed if necessary to remove any unsuitable deposits. Next, the exposed soils should be scarified to a depth of 6 inches, brought to about optimum moisture content, and rolled with heavy compaction equipment. The upper 6 inches of exposed natural material should be compacted to at least 90% of the maximum density obtainable by the ASTM Designation D1557-78 method of compaction. All required fill should be placed in horizontal lifts not more than 8 inches in thickness, and compacted to at least 90%. It is recommended that the soils be compacted to a moisture content varying no more than 2% below or above



optimum moisture content. When placing fill on sloping ground, the slope should be benched to allow proper keying of the compacted fill into the underlying bedrock.

To minimize future shallow sloughing, the outer faces of any fill embankments must be firmly compacted as they are constructed. The recommended method of accomplishing this would be to overfill the slope and trim the slope to final grade after completion. Alternatively, the fill slopes could be slope-rolled with a sheepsfoot roller, done at the completion of every 2 feet of fill. Grid rolling at the completion of grading may also be required.

13.4 EROSION PROTECTION

All slopes should be seeded or planted with a proper ground cover as soon as possible. A landscaping expert should be consulted for ground cover recommendations. An adequate sprinkler system should be installed, and all drainage devices and terraces should be completed and maintained to avoid embankment damage from runoff.

13.5 MATERIAL FOR FILL

Except for organic material generated by the stripping operations, the on-site soils will be suitable for use in compacted fills. Any required imported fill should consist of relatively non-expansive soils. Imported fill material should contain sufficient fines (binder material) so the resulting compacted fill will not rut readily under the wheels of construction equipment and will be stable in shallow trenches. Any expansive soil encountered during grading should not be used within the upper 1 foot of subgrade beneath slabs or paving.

The excavation and compaction of the on-site soils and bedrock materials may result in compacted volumes less (shrinkage) or greater (bulking) than the original volume of the in-place materials. In estimating earthwork quantities, it may be assumed that the on-site alluvial and colluvial soils could undergo an average shrinkage of about 10% when compacted to at least 90%.



13.6 FILL SETTLEMENT

The weight of the planned approach fill embankments will result in settlement due to compression of the underlying soils and the embankment fill. In addition, an increase in the moisture content of the fill embankment due to water infiltration could result in secondary fill settlement.

The settlement of the approach fill embankments will be less than 0.1 inch per foot of fill placed, and the majority of the settlement will occur during placement of the fill.

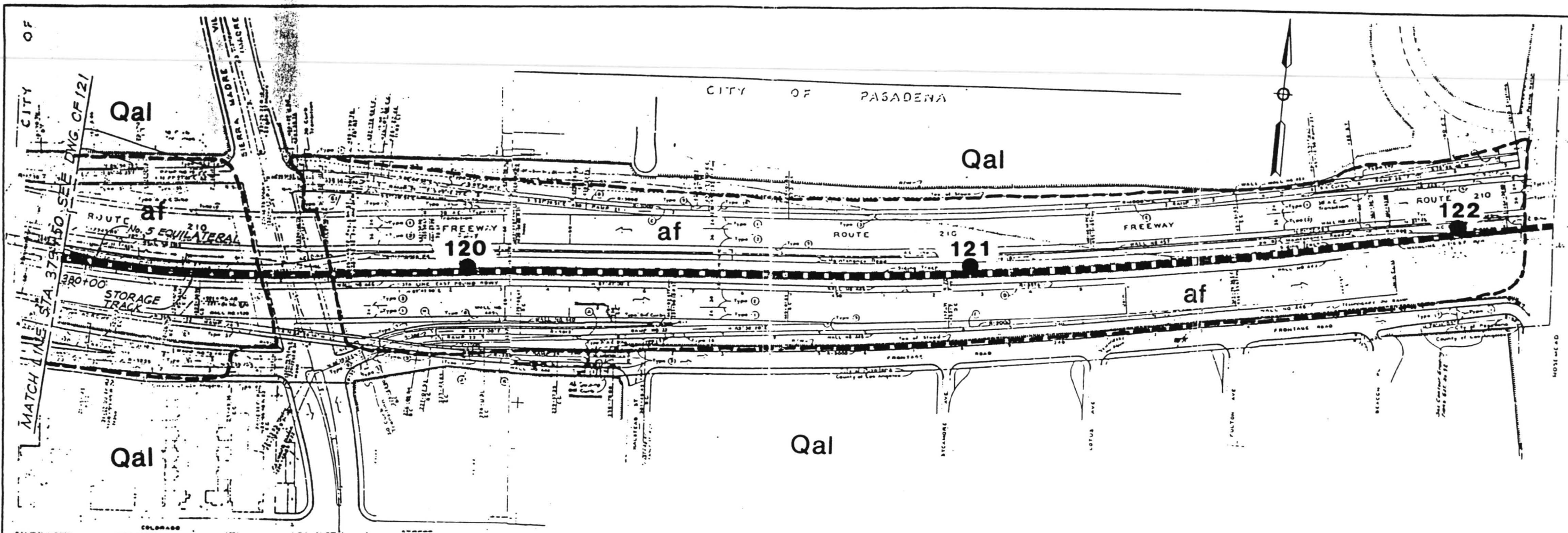
13.7 GRADING OBSERVATION

Observation and testing of the earthwork is considered a continuation of this investigation. Such observation and testing should be performed by a geotechnical engineer licensed in the State of California, after reviewing the results and conclusions of this investigation. The excavation of landslides and all cut slopes should be observed by an engineering geologist licensed in the State of California to verify the conclusions made in this investigation. The compaction of all fills is critical, since such fills will be structural fills used for support of bridge approaches and foundations.

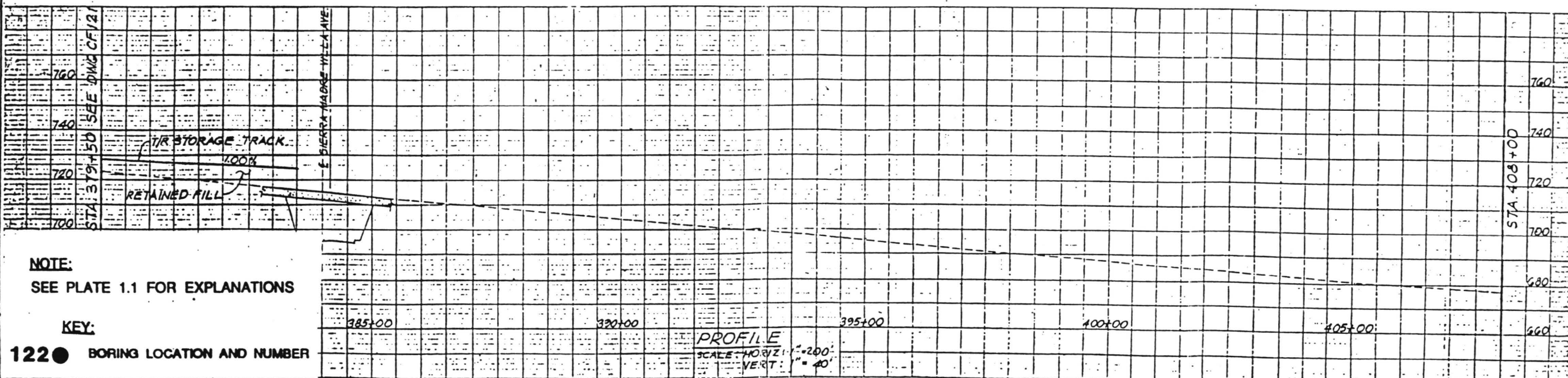


END OF VOLUME 1





PLAN
SCALE: 1"=200'



NOTE:

SEE PLATE 1.1 FOR EXPLANATIONS

KEY:

122 BORING LOCATION AND NUMBER

PROPOSED METRO PASADENA LINE

Designed by

Drawn by M.J.

Checked by

Approved by



LOS ANGELES COUNTY TRANSPORTATION COMMISSION
PASADENA-LOS ANGELES ROUTE REFINEMENT STUDY



LAW / CRANDALL, INC.

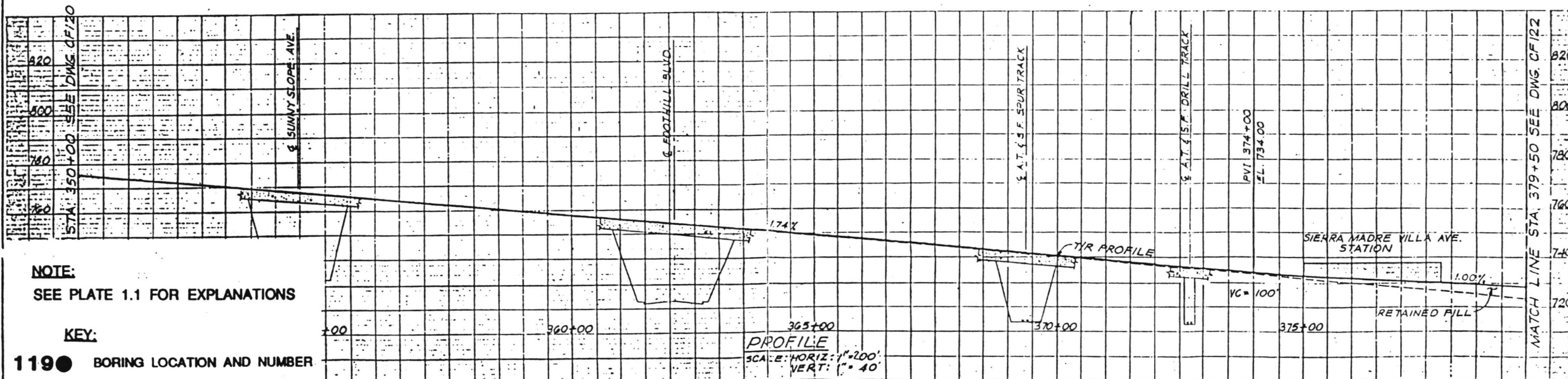
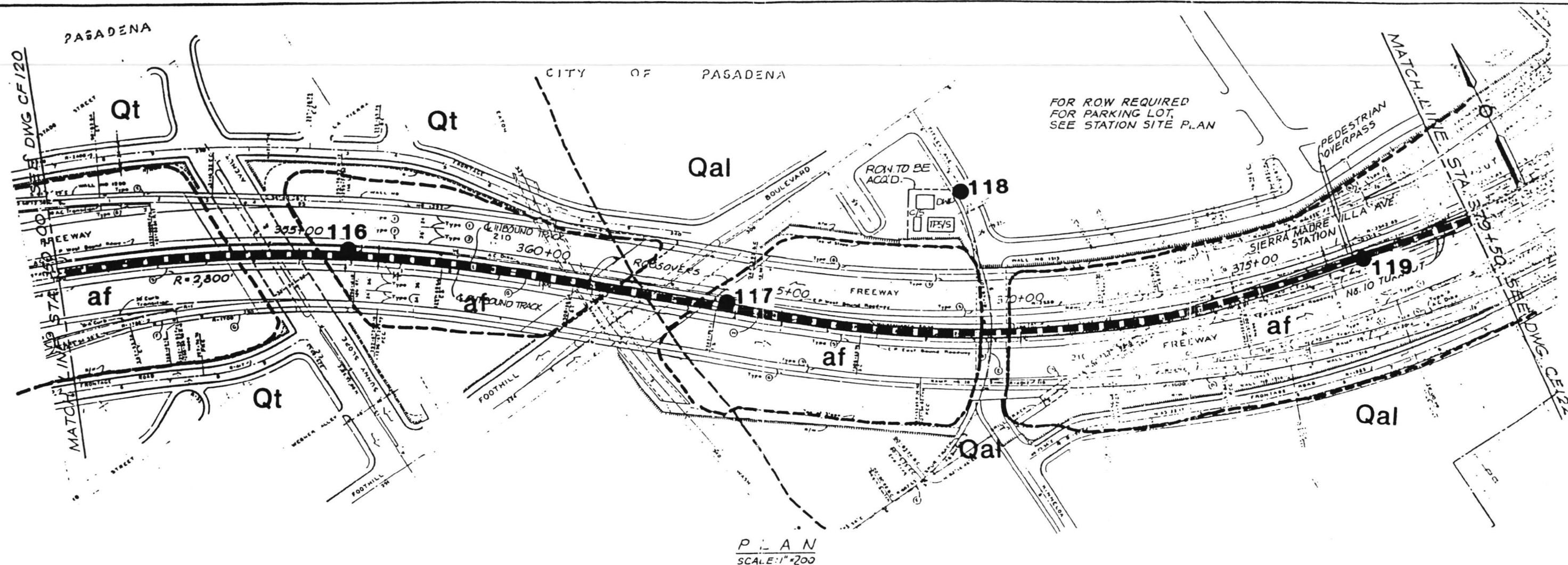
Submitted:

IN ASSOCIATION WITH:
ANIL VERMA ASSOCIATES
ACQUINTAL ANALYSIS ASSOCIATES
BKS ASSOCIATES
MICHAEL BRAHMAN ASSOCIATES, INC.
PGH WONG ENGINEERING, INC.
RALPH STONE AND COMPANY, INC.

GEOLOGIC MAP AND BORING LOCATION PLAN

CONTRACT NO.
DRAWING NO.

SCALE 1"=200' DATE 2/18/93
SHEET NUMBER



PROPOSED METRO PASADENA LINE

Designed by
Drawn by M.N.
Checked by
Approved by

LOS ANGELES COUNTY TRANSPORTATION COMMISSION
PASADENA-LOS ANGELES ROUTE REFINEMENT STUDY

IN ASSOCIATION WITH:
AMR VERMA ASSOCIATES
ACOUSTICAL ANALYSIS ASSOCIATES
DKS ASSOCIATES
MICHAEL BRANDMAN ASSOCIATES, INC.
PGM WONG ENGINEERING, INC.
RALPH STONE AND COMPANY, INC.

Submitted:

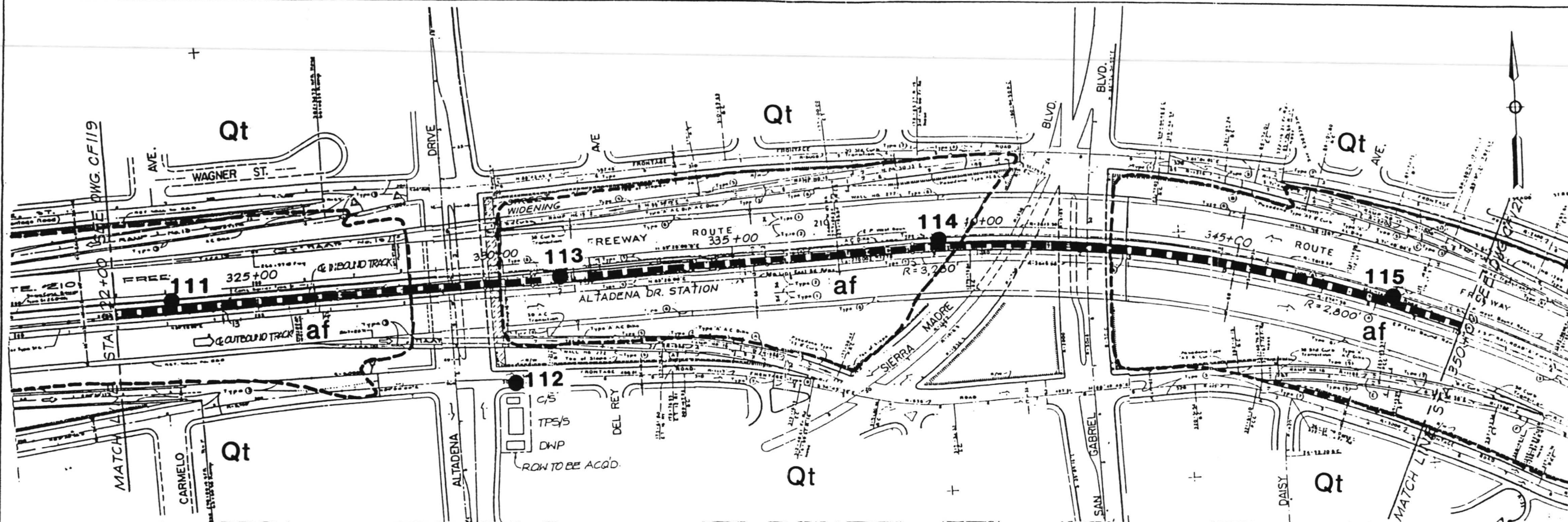
GEOLOGIC MAP AND BORING LOCATION PLAN

CONTRACT NO.
DRAWING NO.

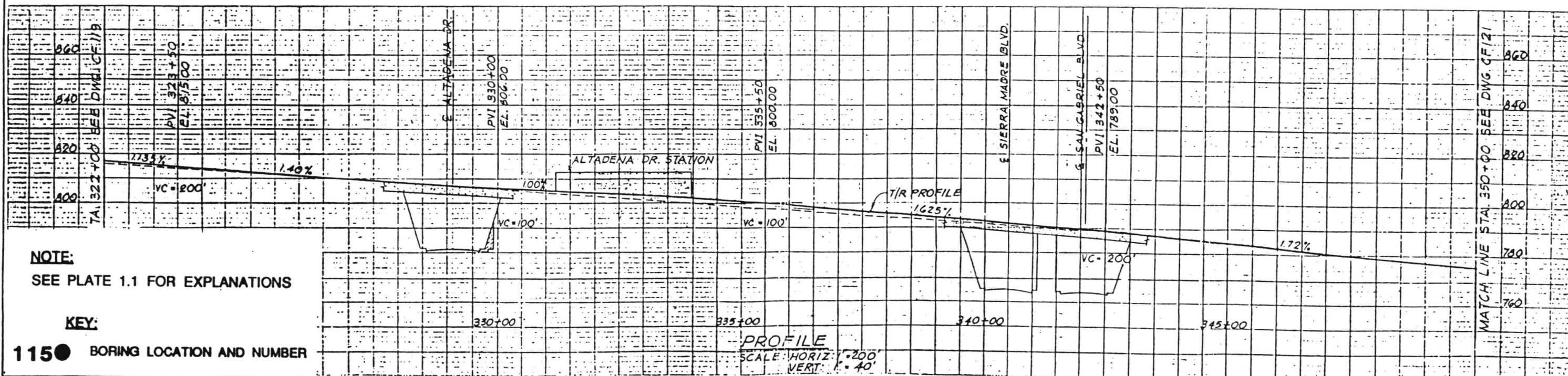
SCALE
1"=200'

DATE
2/18/93

SHEET NUMBER



PLAN
SCALE: 1" = 200'



NOTE:
SEE PLATE 1.1 FOR EXPLANATIONS

KEY:
115 BORING LOCATION AND NUMBER

PROPOSED METRO PASADENA LINE

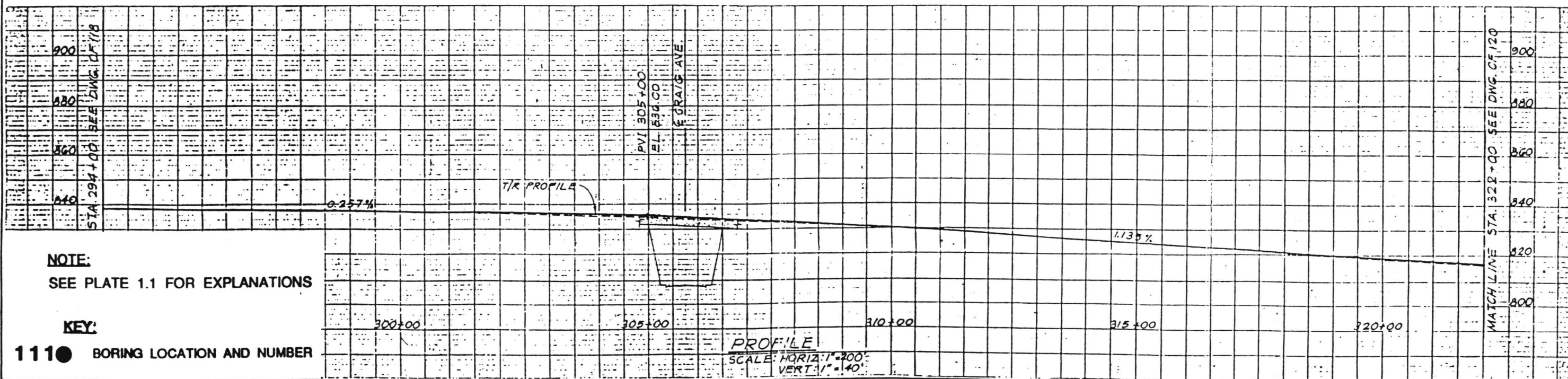
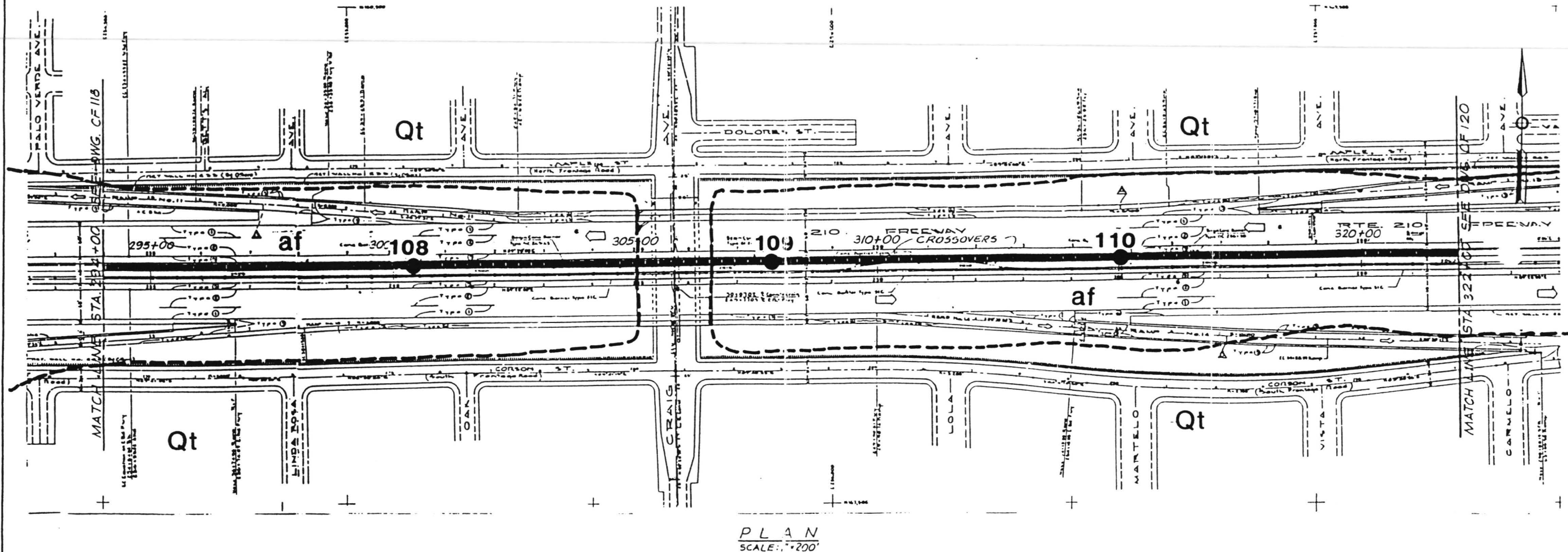
Designed by
Drawn by M.A.
Checked by
Approved by
Submitted:

LOS ANGELES COUNTY TRANSPORTATION COMMISSION
PASADENA-LOS ANGELES ROUTE REFINEMENT STUDY

IN ASSOCIATION WITH:
ANIL VERMA ASSOCIATES
ACUSTICAL ANALYSIS ASSOCIATES
DES ASSOCIATES
MICHAEL BRADSHAW ASSOCIATES, INC.
PEN WORLD ENGINEERING, INC.
RALPH STONE AND COMPANY, INC.

GEOLOGIC MAP AND BORING LOCATION PLAN

CONTRACT NO.
DRAWING NO.
SCALE 1" = 200' DATE 2/18/93
SHEET NUMBER



NOTE:
SEE PLATE 1.1 FOR EXPLANATIONS

KEY:
111 ● BORING LOCATION AND NUMBER

PROPOSED METRO PASADENA LINE

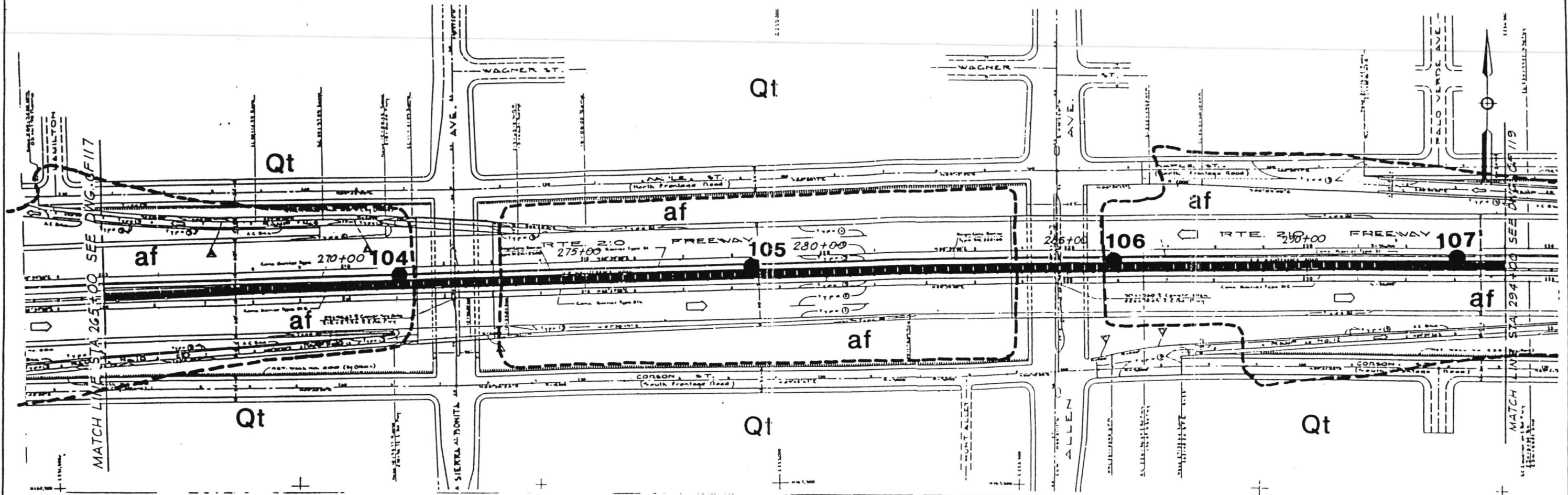
Designed by
Drawn by M.N.
Checked by
Approved by
Submitted by

LOS ANGELES COUNTY TRANSPORTATION COMMISSION
PASADENA-LOS ANGELES ROUTE REFINEMENT STUDY

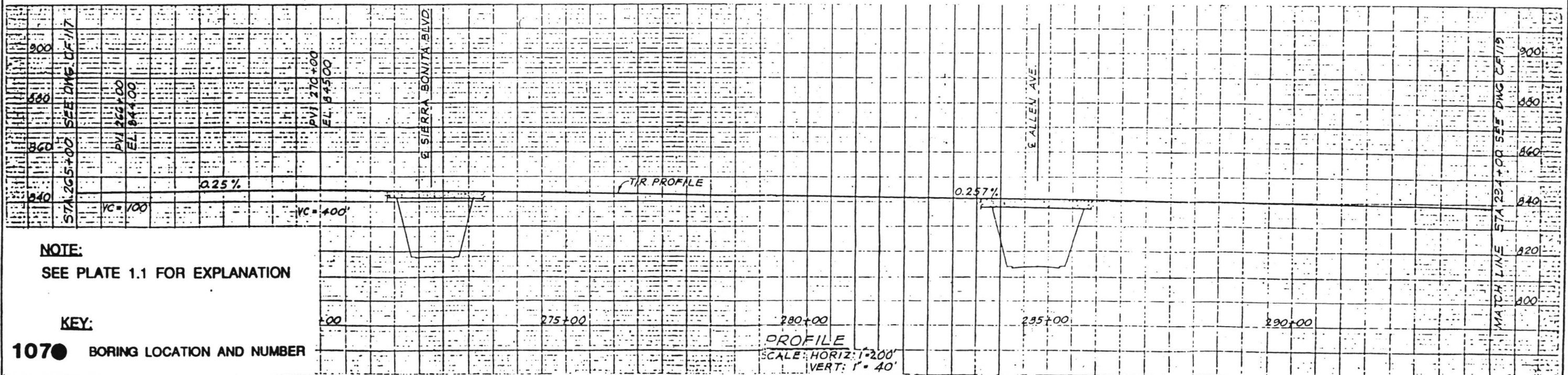
IN ASSOCIATION WITH:
ANIL VERMA ASSOCIATES
ACUSTICAL ANALYSIS ASSOCIATES
BRS ASSOCIATES
MICHAEL BRANDMAN ASSOCIATES, INC.
PCR WORK ENGINEERING, INC.
R. M. STONE AND COMPANY, INC.

GEOLOGIC MAP AND BORING LOCATION PLAN

CONTRACT NO.
DRAWING NO.
SCALE 1" = 200'
DATE 2/18/93
SHEET NUMBER



PLAN
SCALE: 1" = 200'



NOTE:
SEE PLATE 1.1 FOR EXPLANATION

KEY:
107● BORING LOCATION AND NUMBER

PROPOSED METRO PASADENA LINE

Designed by
Drawn by M.J.
Checked by
Approved by

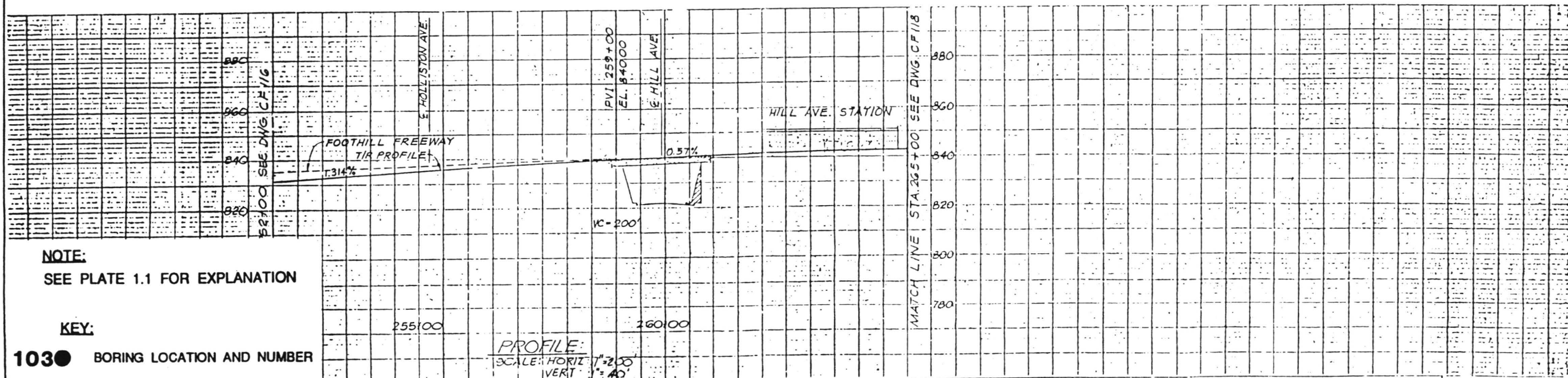
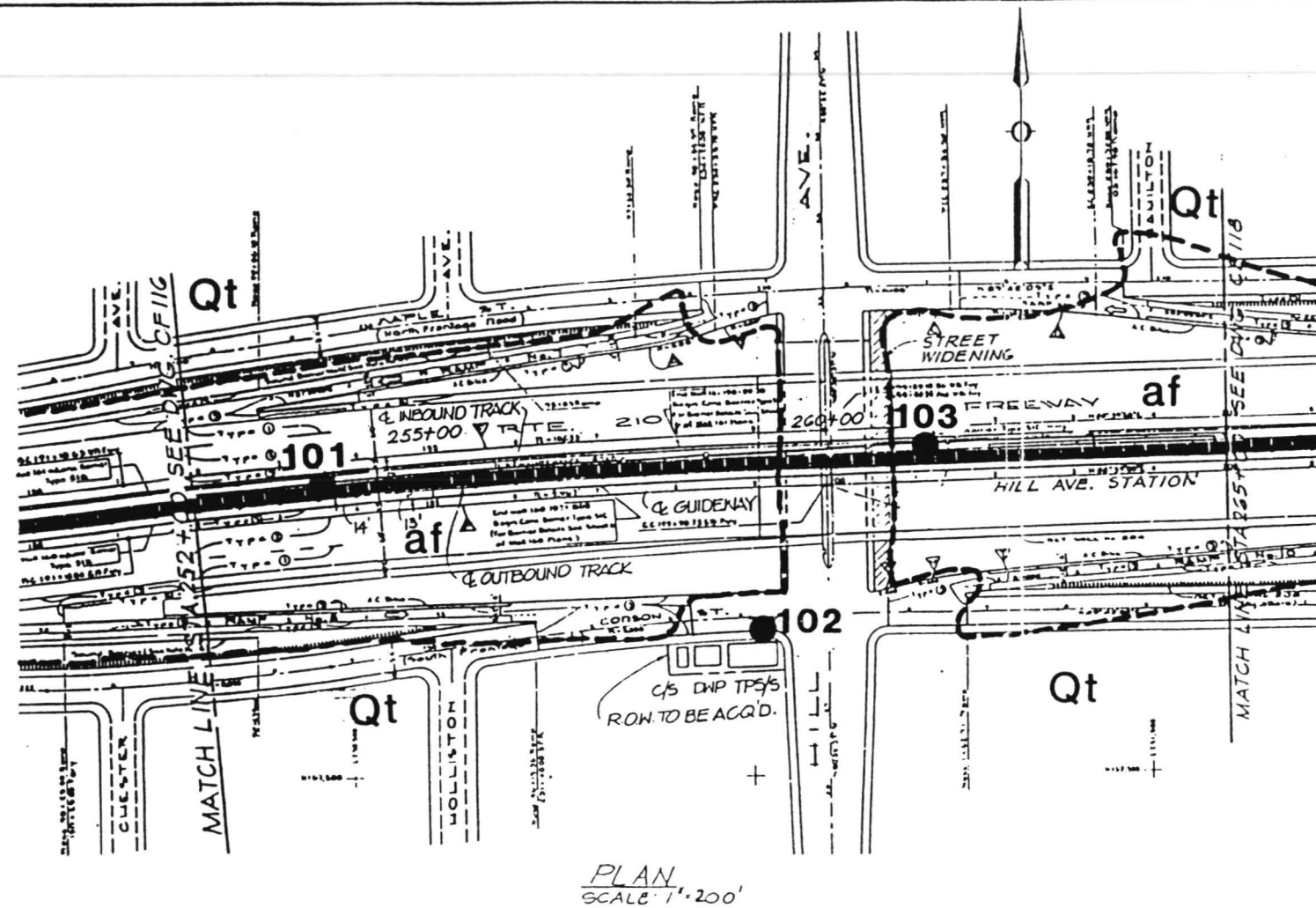


LOS ANGELES COUNTY TRANSPORTATION COMMISSION
PASADENA - LOS ANGELES ROUTE REFINEMENT STUDY

IN ASSOCIATION WITH:
ANIL VERMA ASSOCIATES
ACOUTICAL ANALYSIS ASSOCIATES
OKS ASSOCIATES
MICHAEL BRANDMAN ASSOCIATES, INC.
PEH WONG ENGINEERING, INC.
RALPH STONE AND COMPANY, INC.

GEOLOGIC MAP AND BORING LOCATION PLAN

CONTRACT NO.
DRAWING NO.
SCALE 1" = 200'
DATE 2/18/93
SHEET NUMBER



NOTE:
SEE PLATE 1.1 FOR EXPLANATION

KEY:
103 BORING LOCATION AND NUMBER

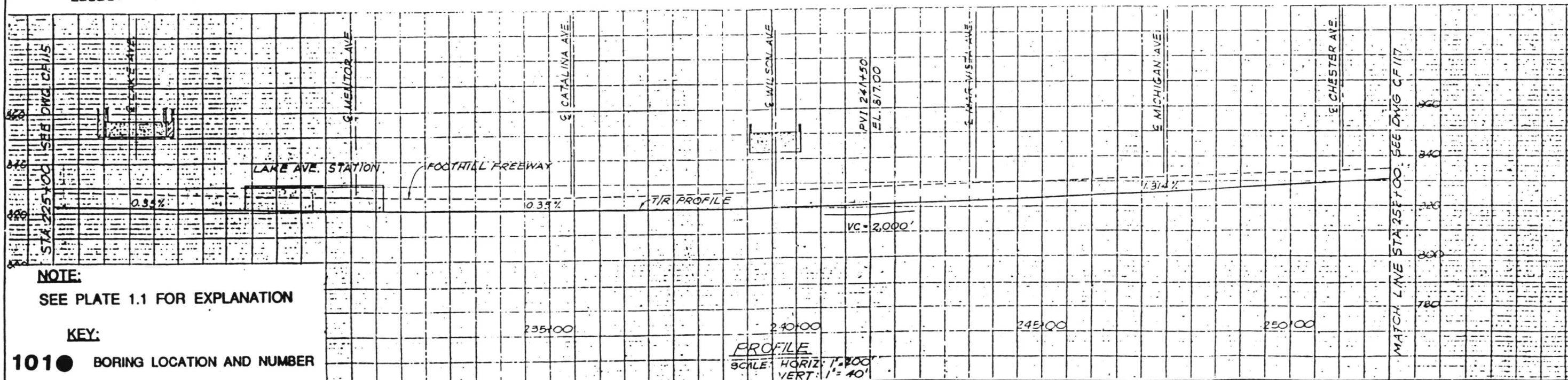
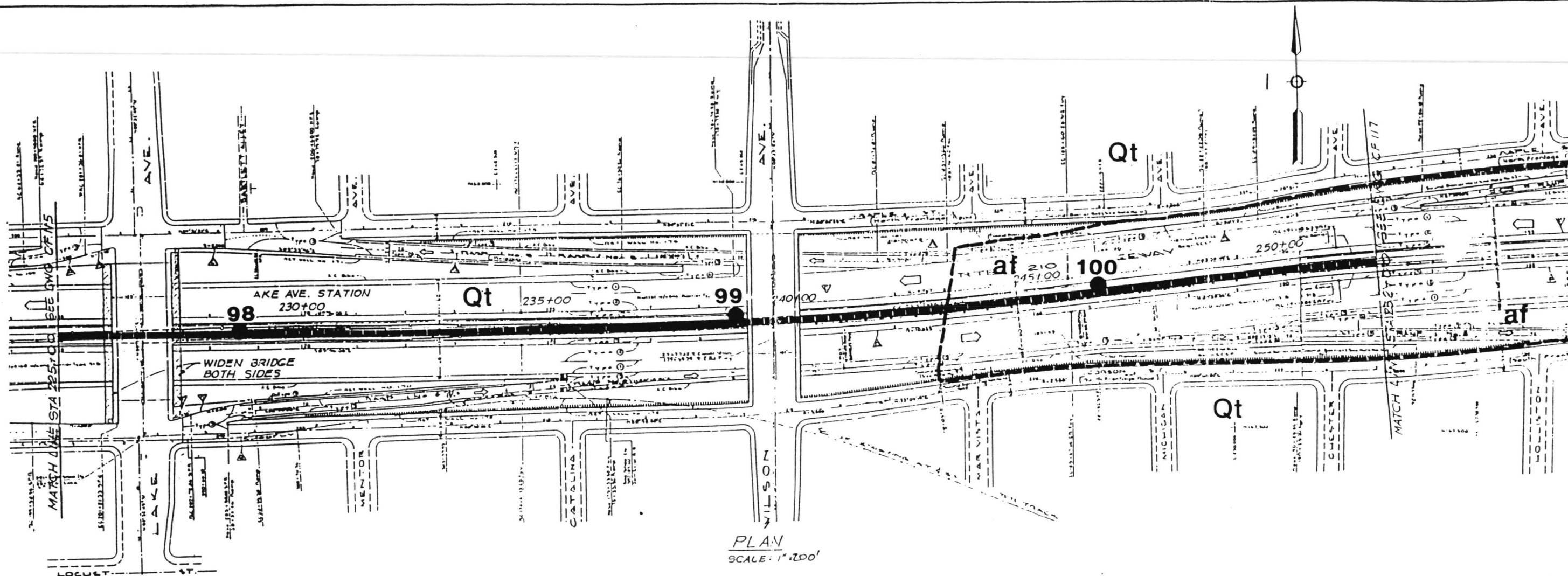
PROPOSED METRO PASADENA LINE

Designed by
Drawn by M.N.
Checked by
Approved by

LOS ANGELES COUNTY TRANSPORTATION COMMISSION
PASADENA - LOS ANGELES ROUTE REFINEMENT STUDY
LAW / CRANDALL, INC.
Submitted:

GEOLOGIC MAP AND BORING LOCATION PLAN

CONTRACT NO.
DRAWING NO.
SCALE 1" = 200' DATE 2/18/93
SHEET NUMBER



NOTE:
SEE PLATE 1.1 FOR EXPLANATION

KEY:
101 ● BORING LOCATION AND NUMBER

PROPOSED METRO PASADENA LINE

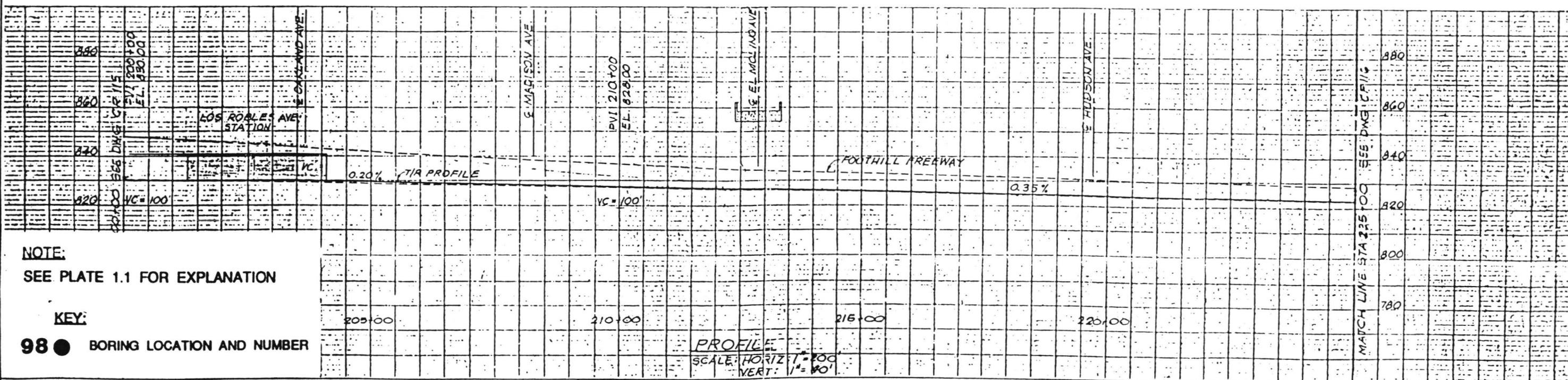
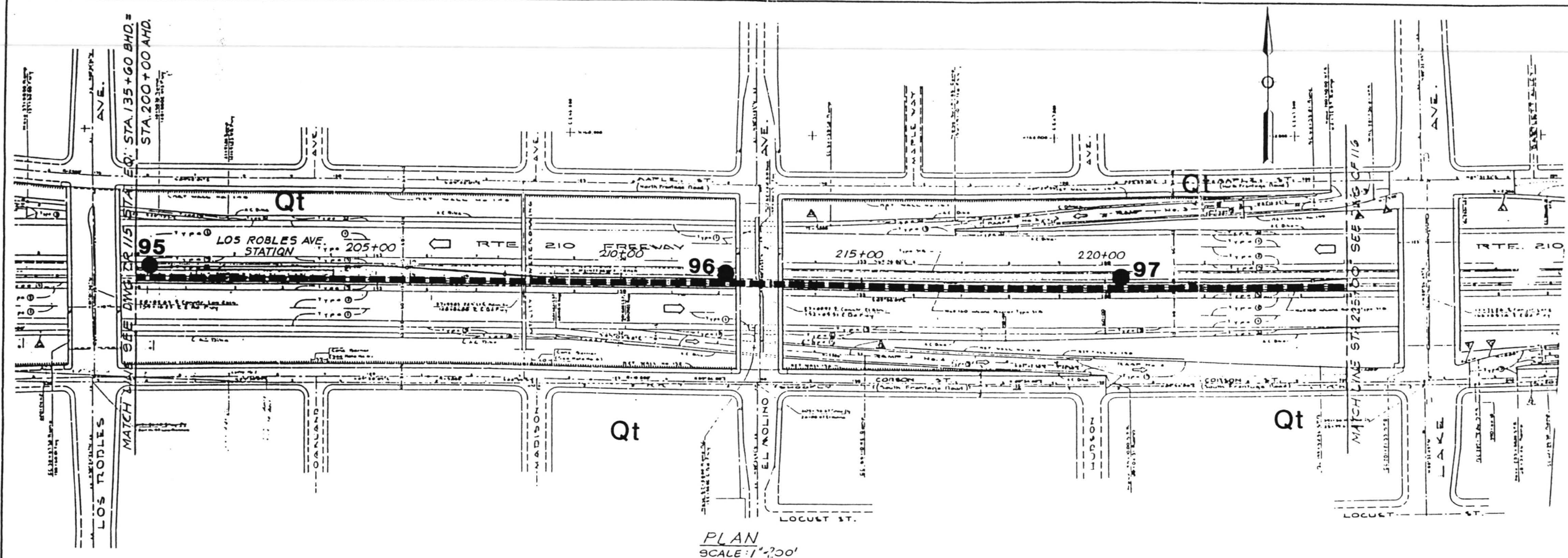
Designed by
Drawn by M.J.
Checked by
Approved by
Submitted:

LOS ANGELES COUNTY TRANSPORTATION COMMISSION
PASADENA - LOS ANGELES ROUTE REFINEMENT STUDY

IN ASSOCIATION WITH:
ANIL VERMA ASSOCIATES
ACQUETICAL ANALYSIS ASSOCIATES
DHS ASSOCIATES
MICHAEL BRANDMAN ASSOCIATES, INC.
PGH WONG ENGINEERING, INC.
RALPH STONE AND COMPANY, INC.

GEOLOGIC MAP AND BORING LOCATION PLAN

CONTRACT NO.
DRAWING NO.
SCALE
1" = 200'
DATE
2/18/93
SHEET NUMBER



NOTE:
SEE PLATE 1.1 FOR EXPLANATION

KEY:
98 ● BORING LOCATION AND NUMBER

PROPOSED METRO PASADENA LINE

Designed by
Drawn by *MJL*
Checked by
Approved by



LOS ANGELES COUNTY TRANSPORTATION COMMISSION
PASADENA - LOS ANGELES ROUTE REFINEMENT STUDY

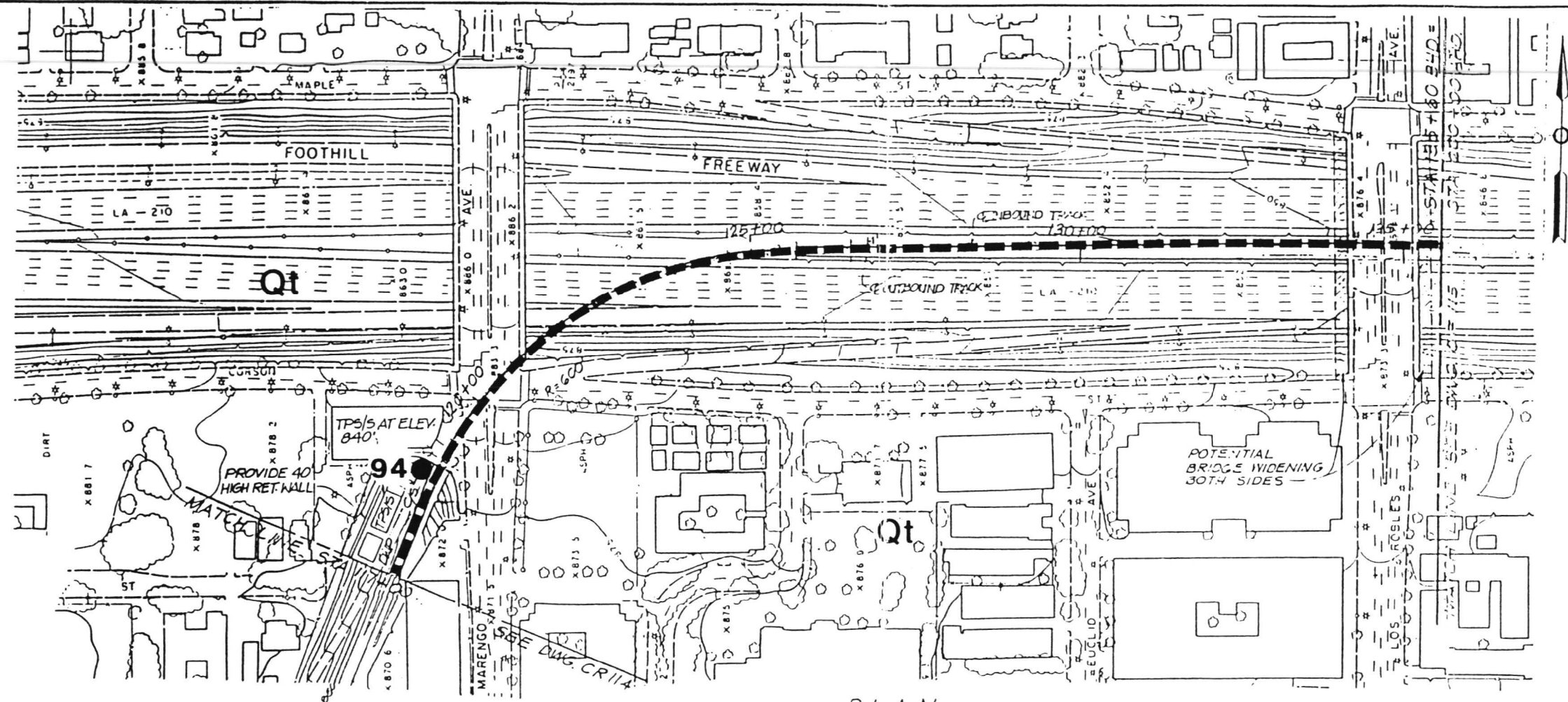


LAW / CRANDALL, INC.

IN ASSOCIATION WITH:
ANIL VERMA ASSOCIATES
ACOUSTICAL ANALYSTS ASSOCIATES
DKS ASSOCIATES
MICHAEL BRANNMAN ASSOCIATES, INC.
PGH WING ENGINEERING, INC.
RALPH STONE AND COMPANY, INC.

GEOLOGIC MAP AND BORING LOCATION PLAN

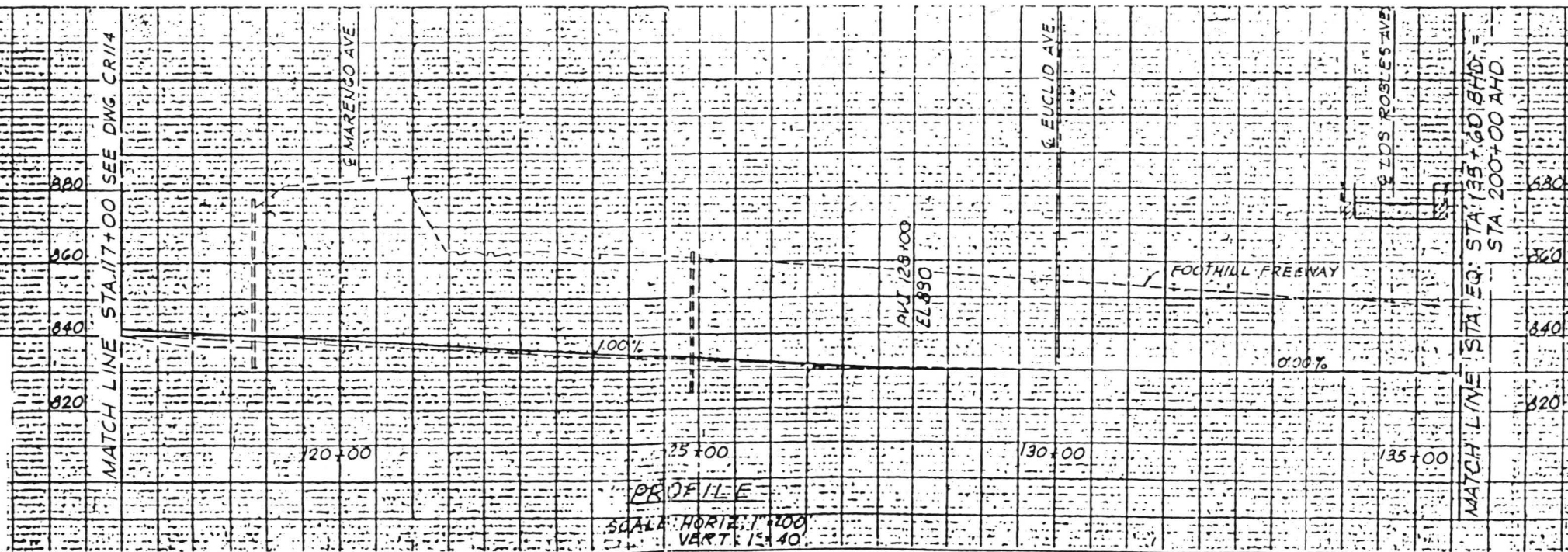
CONTRACT NO.
DRAWING NO.
SCALE 1" = 200' DATE 2/18/93
SHEET NUMBER



PLAN
SCALE: 1"=200'

NOTE:
SEE PLATE 1.1 FOR EXPLANATION

KEY:
94 ● BORING LOCATION AND NUMBER



PROPOSED METRO PASADENA LINE

Designed by
Drawn by M.N.
Checked by
Approved by



LOS ANGELES COUNTY TRANSPORTATION COMMISSION
PASADENA - LOS ANGELES ROUTE REFINEMENT STUDY

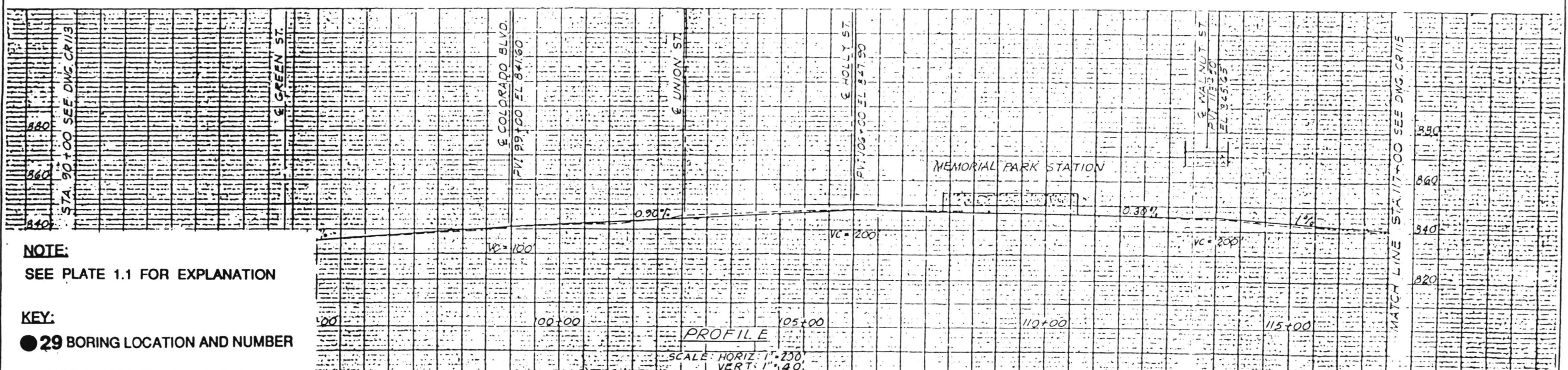
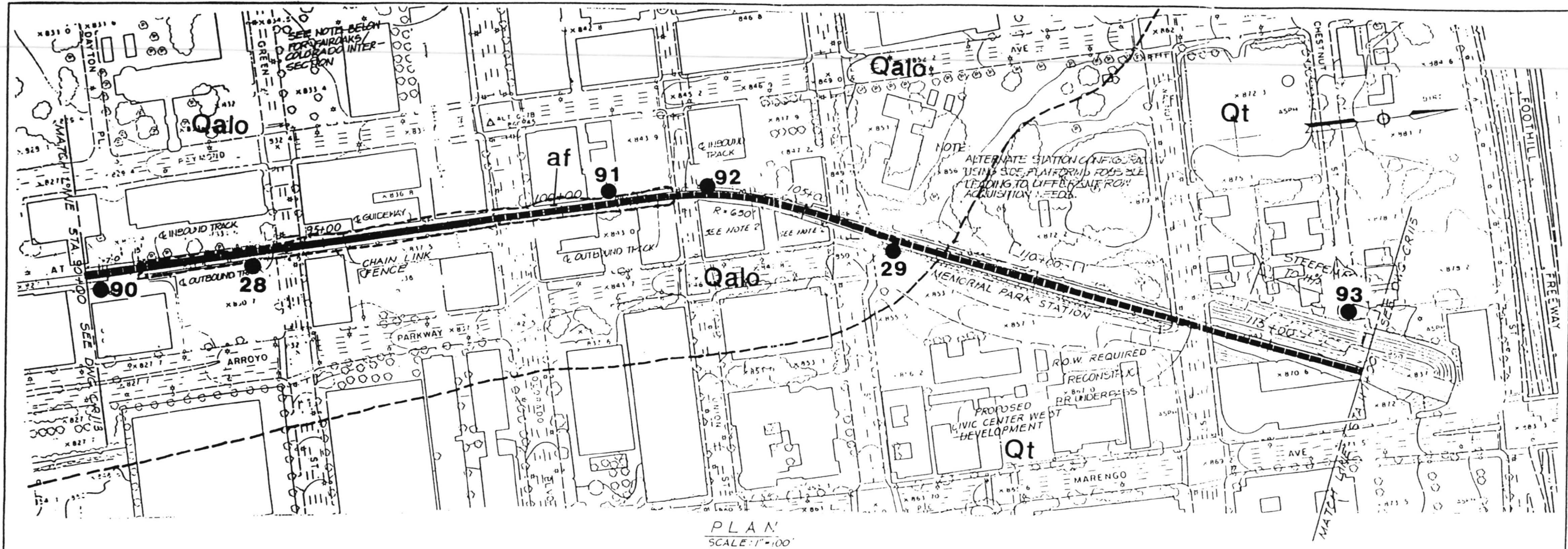
LAW / CRANDALL, INC.
Submitted:

IN ASSOCIATION WITH:
ANIL VERMA ASSOCIATES
ACQUISITION ANALYSIS ASSOCIATES
DKS ASSOCIATES
MICHAEL BRANOMAN ASSOCIATES, INC.
PGM WONG ENGINEERING INC.
RALPH STONE AND COMPANY INC.

GEOLOGIC MAP AND BORING LOCATION PLAN

CONTRACT NO.
DRAWING NO.

SCALE 1" = 200' DATE 2/18/93
SHEET NUMBER

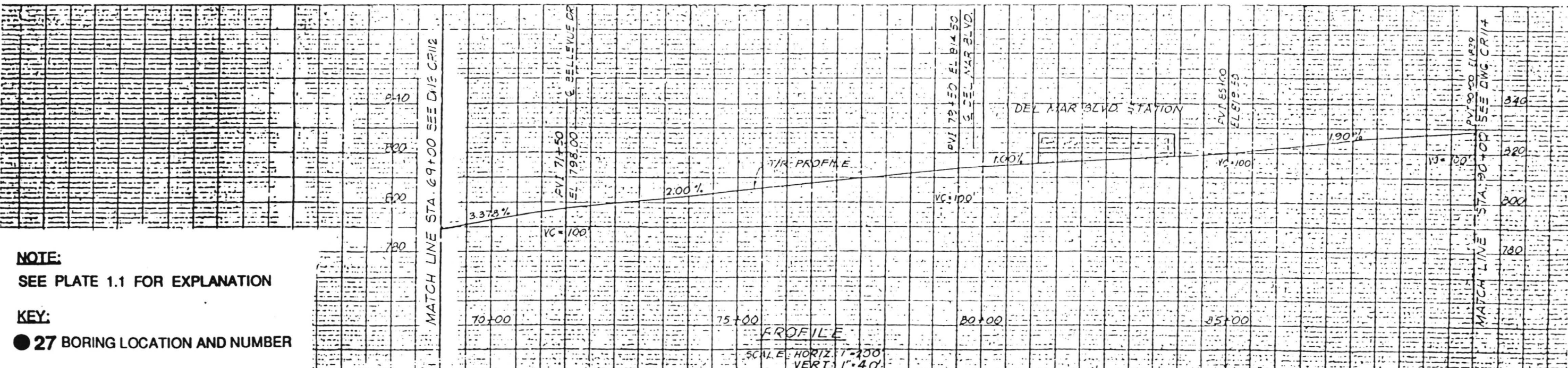
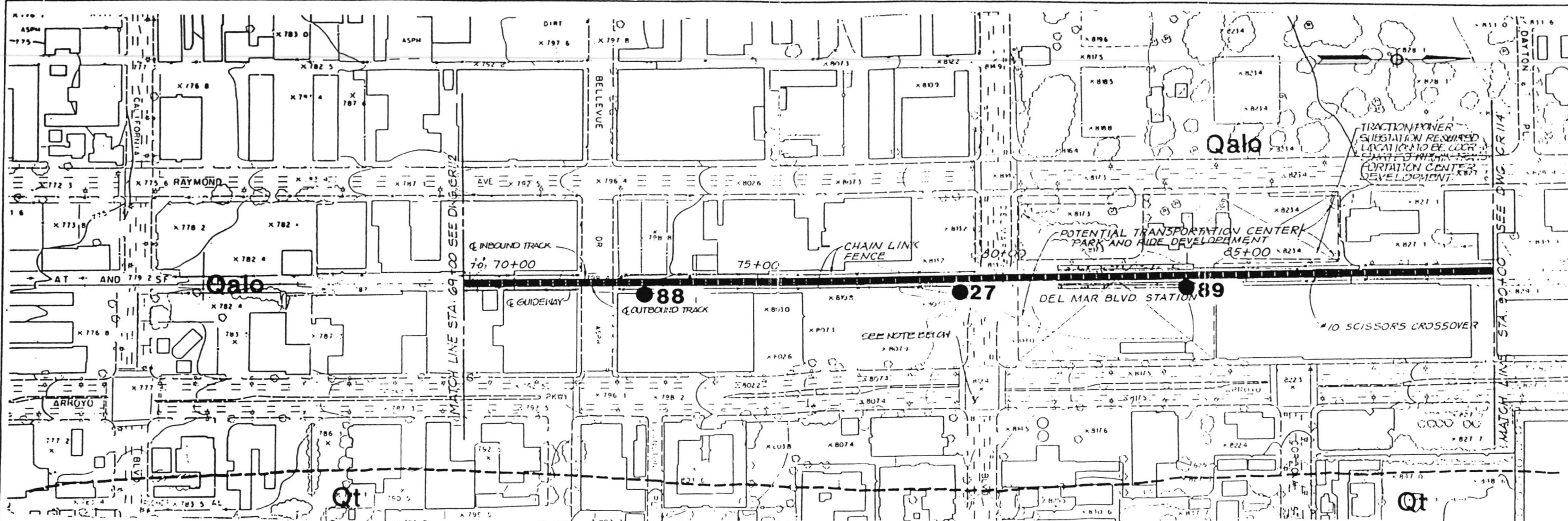


PROPOSED METRO PASADENA LINE

Designed	LOS ANGELES COUNTY TRANSPORTATION COMMISSION
Drawn by	M.N.
Checked by	LAW / CRANDALL, INC.
Approved by	Submitted:

GEOLOGIC MAP AND BORING LOCATION PLAN

CONTRACT NO.	DATE
DRAWING NO.	2/18/93
SCALE	1" = 200'



NOTE:
SEE PLATE 1.1 FOR EXPLANATION

KEY:
● 27 BORING LOCATION AND NUMBER

PROPOSED METRO PASADENA LINE

Designed
Drawn by *M.H.*
Checked by
Approved by
Submitted:

LOS ANGELES COUNTY TRANSPORTATION COMMISSION
PASADENA - LOS ANGELES ROUTE REFINEMENT STUDY

IN ASSOCIATION WITH:
AME VERMA ASSOCIATES
ACQUINTAL ANALYSIS ASSOCIATES
DWS ASSOCIATES
MICHAEL BRANDMAN ASSOCIATES, INC.
PGM WONG ENGINEERING, INC.
RALPH STONE AND COMPANY, INC.

GEOLOGIC MAP AND BORING LOCATION PLAN

CONTRACT NO.
DRAWING NO.

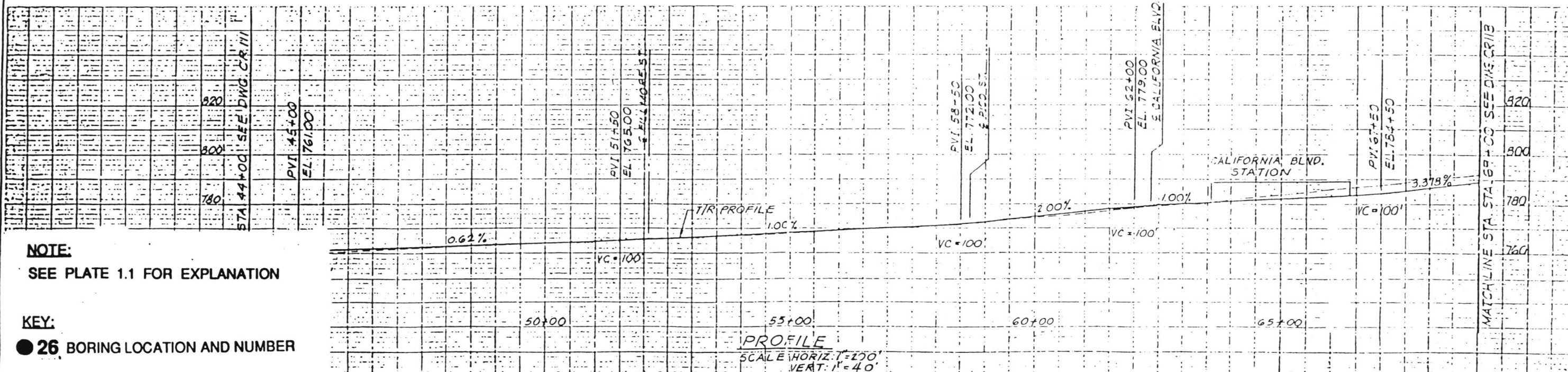
SCALE
1" = 200'

DATE
2/18/93

L92045.AE4 DATE 2/18/93 DR. MOHAMMED CHKD



PLAN
SCALE: 1"=200'



NOTE:
SEE PLATE 1.1 FOR EXPLANATION

KEY:
● 26 BORING LOCATION AND NUMBER

PROPOSED METRO PASADENA LINE

Designed by
Drawn by *M.A.*
Checked by
Approved by
Submitted by

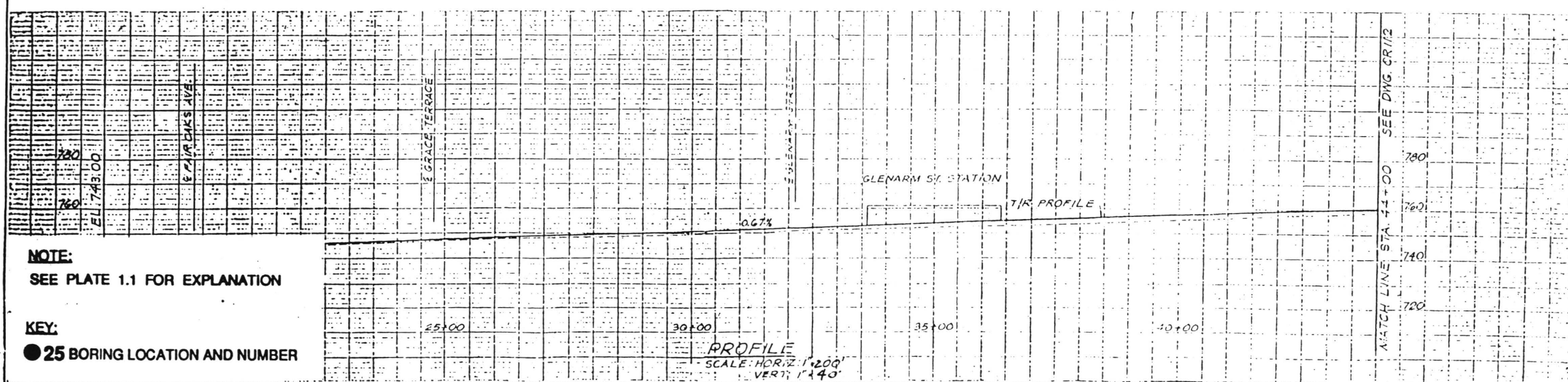
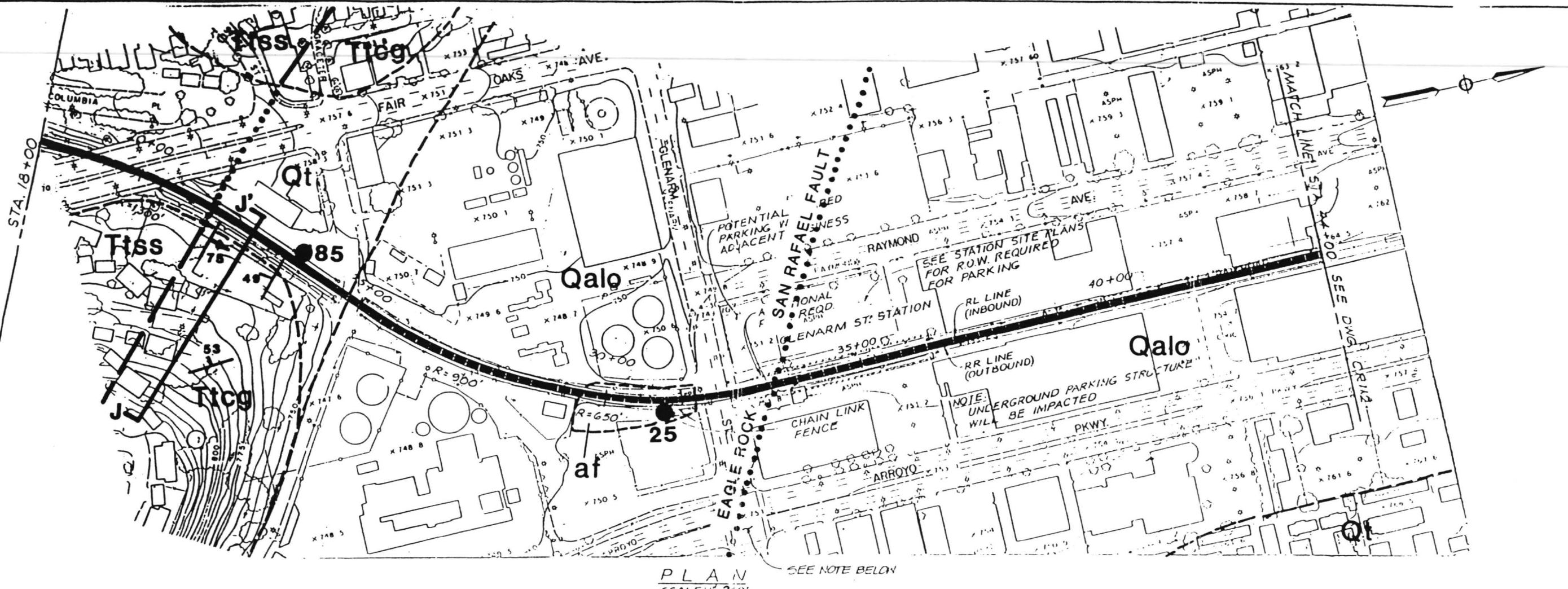
LOS ANGELES COUNTY TRANSPORTATION COMMISSION
PASADENA - LOS ANGELES ROUTE REFINEMENT STUDY

IN ASSOCIATION WITH:
ANIL VERMA ASSOCIATES
ACOUSTICAL ANALYSIS ASSOCIATES
DKS ASSOCIATES
MICHAEL BRANDMAN ASSOCIATES, INC.
PSN WONG ENGINEERING, INC.
RALPH STONE AND COMPANY, INC.

GEOLOGIC MAP AND BORING LOCATION PLAN

CONTRACT NO.
DRAWING NO.

SCALE 1" = 200' DATE 2/18/93

**NOTE:**

SEE PLATE 1.1 FOR EXPLANATION

KEY:

● 25 BORING LOCATION AND NUMBER

PROPOSED METRO PASADENA LINE

Designed

Drawn by *M.H.*

Checked by

Approved by


 LOS ANGELES COUNTY TRANSPORTATION COMMISSION
 PASADENA - LOS ANGELES ROUTE REFINEMENT STUDY

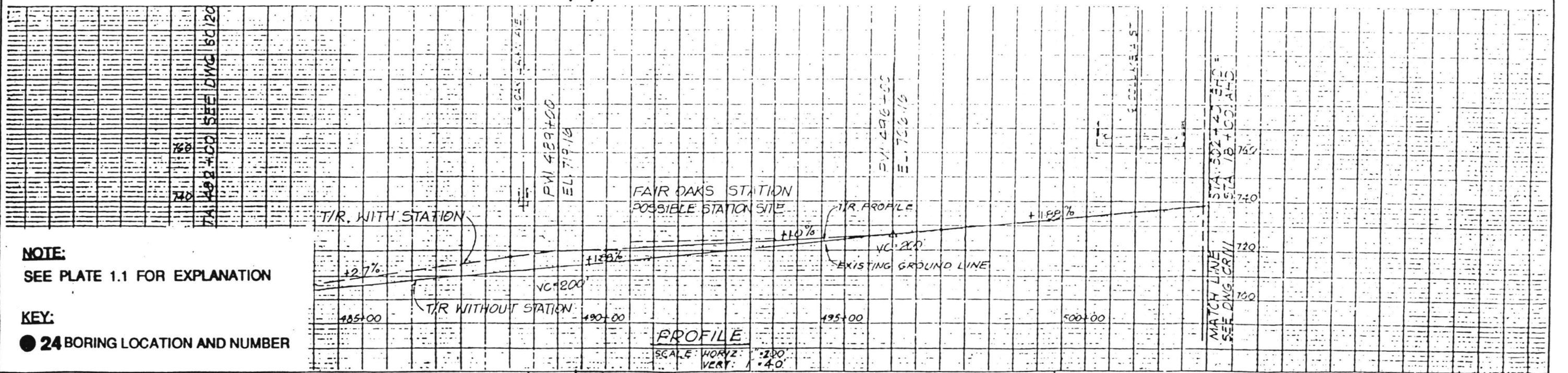
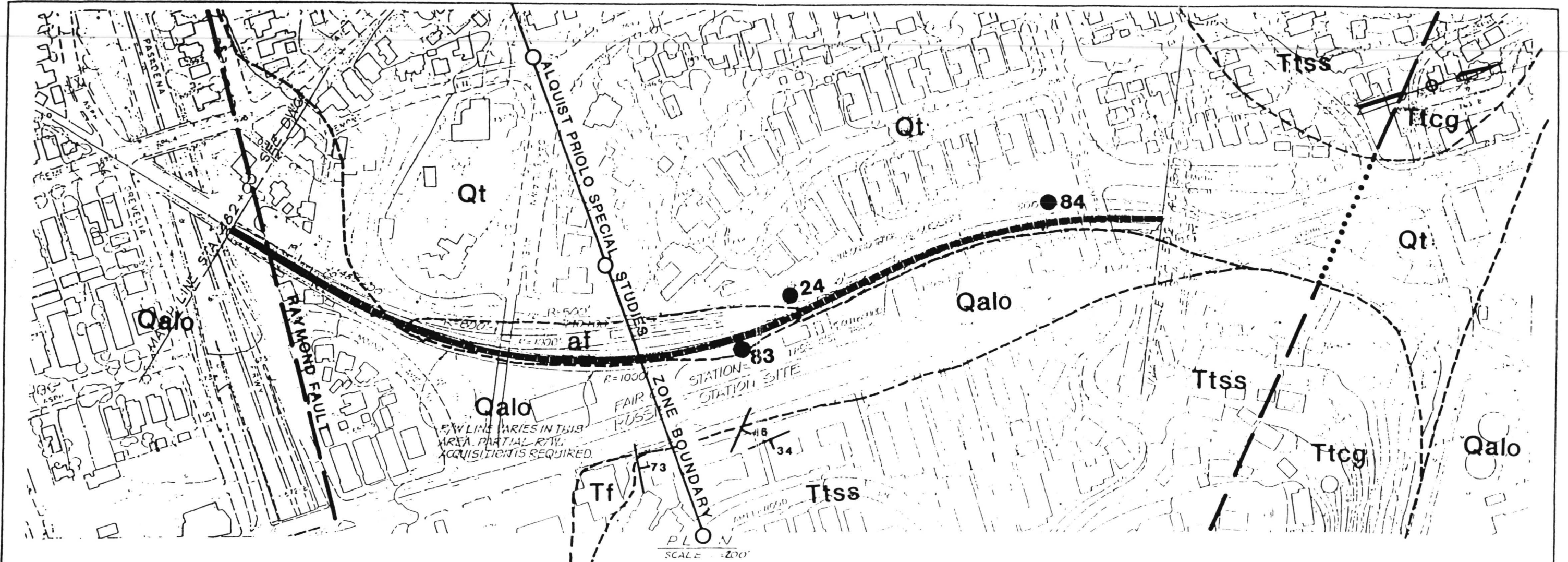

LAW / CRANDALL, INC.

Submitted:

 IN ASSOCIATION WITH:
 ANIL VERMA ASSOCIATES
 ACUSTICAL ANALYSIS ASSOCIATES
 DKS ASSOCIATES
 MICHAEL BRANDMAN ASSOCIATES, INC.
 PCH WONG ENGINEERING, INC.
 RALPH STONE AND COMPANY, INC.

GEOLOGIC MAP AND BORING LOCATION PLAN

CONTRACT NO.
DRAWING NO.SCALE
1" = 200'DATE
2/18/93



NOTE:
SEE PLATE 1.1 FOR EXPLANATION

KEY:
● 24 BORING LOCATION AND NUMBER

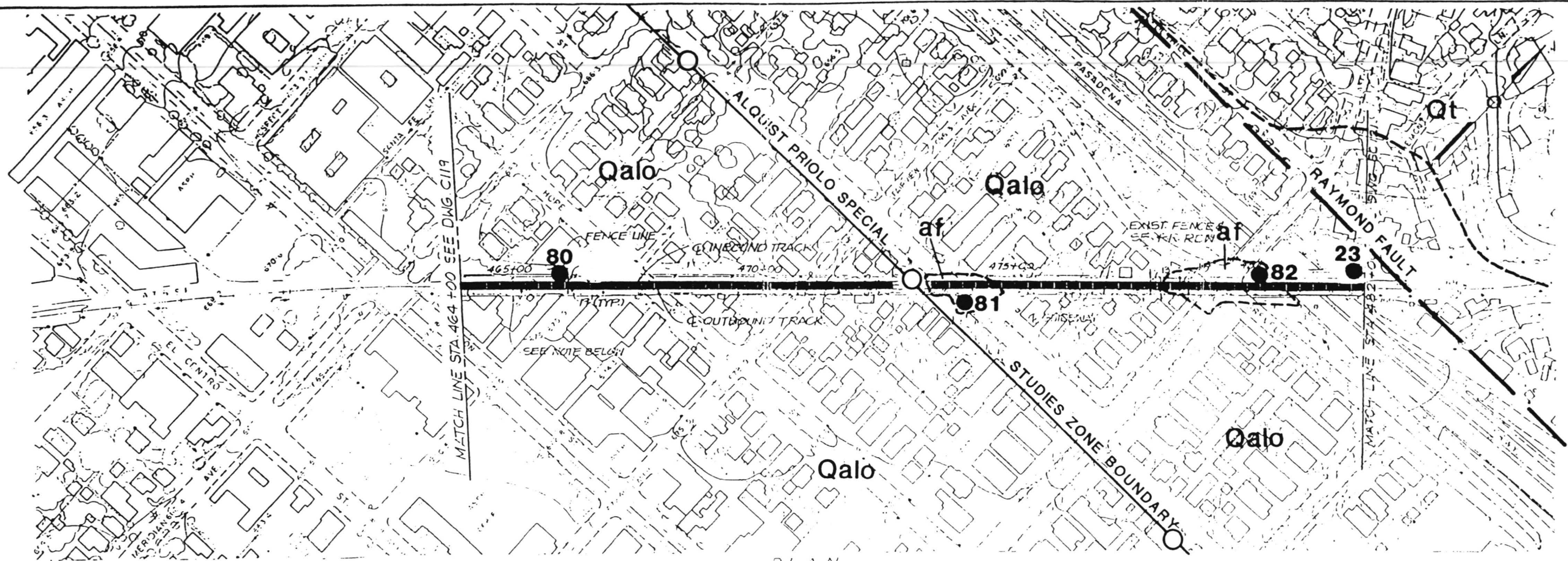
PROPOSED METRO PASADENA LINE

Designed
Drawn by *M.J.*
Checked by
Approved by

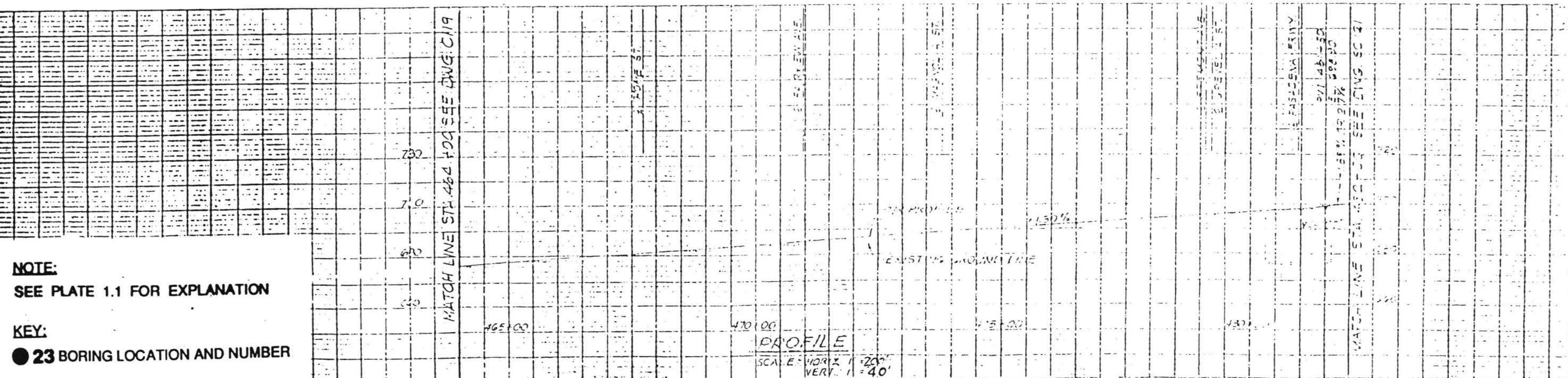
LOS ANGELES COUNTY TRANSPORTATION COMMISSION
PASADENA - LOS ANGELES ROUTE REFINEMENT STUDY
LAW / CRANDALL, INC.
IN ASSOCIATION WITH:
ANIL VERMA ASSOCIATES
ACOUSTICAL ANALYSIS ASSOCIATES
DRS ASSOCIATES
MICHAEL BRANDMAN ASSOCIATES, INC.
PGH WONG ENGINEERING, INC.
RA: PH STONE AND COMPANY, INC.

GEOLOGIC MAP AND BORING LOCATION PLAN

CONTRACT NO.
DRAWING NO.
SCALE 1" = 200'
DATE 2/18/93



PLAN
SCALE 1" = 200'



NOTE:
SEE PLATE 1.1 FOR EXPLANATION

KEY:
● 23 BORING LOCATION AND NUMBER

PROPOSED METRO PASADENA LINE

Designed by
Drawn by M.J.
Checked by
Approved by

LOS ANGELES COUNTY TRANSPORTATION COMMISSION
PASADENA - LOS ANGELES ROUTE REFINEMENT STUDY

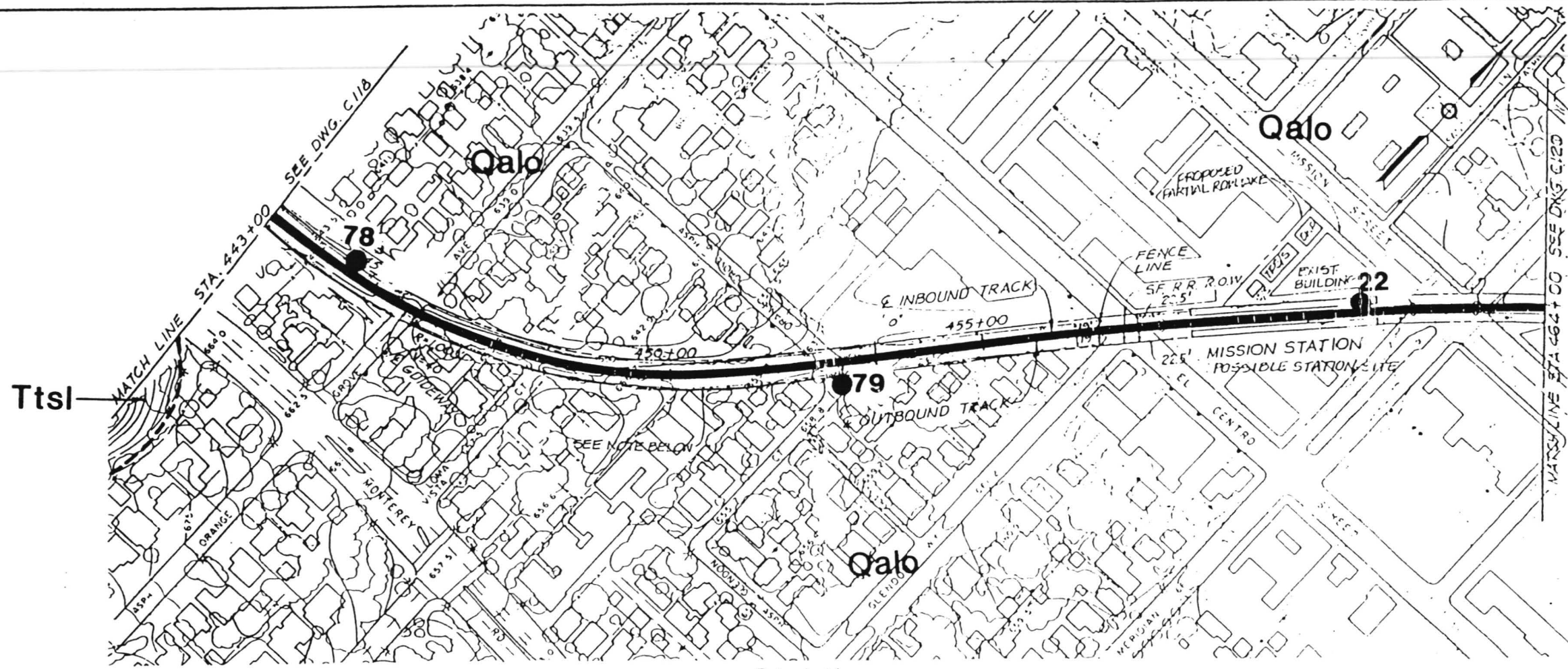
LAW / CRANDALL, INC.
Submitted:

IN ASSOCIATION WITH:
AHL VENNA ASSOCIATES
ACOUSTICAL ANALYSIS ASSOCIATES
DRS ASSOCIATES
MICHAEL BRAHMAN ASSOCIATES INC
PGH WONG ENGINEERING, INC.
RALPH STONE AND COMPANY, INC.

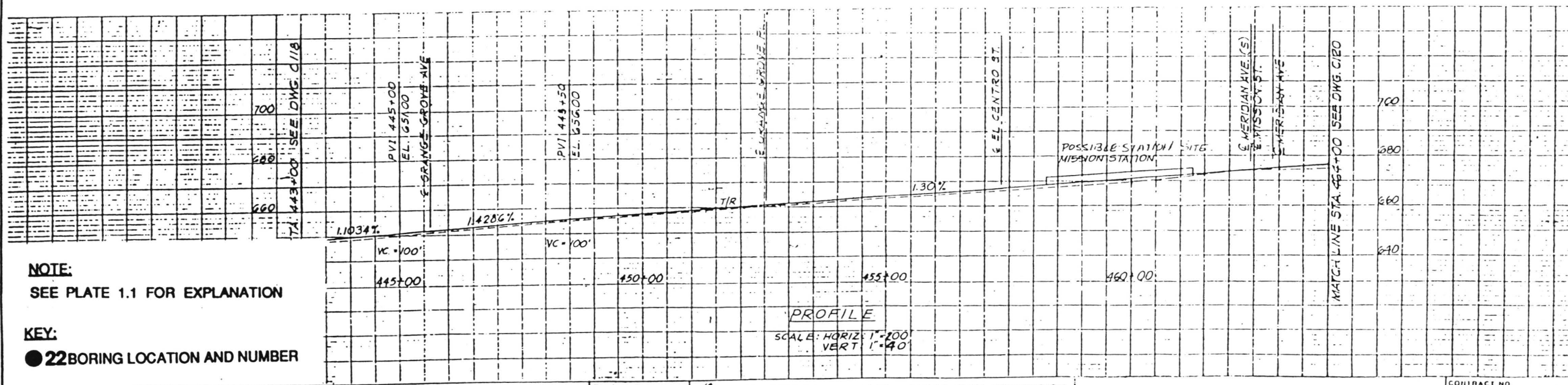
GEOLOGIC MAP AND BORING LOCATION PLAN

CONTRACT NO.
DRAWING NO.

SCALE 1" = 200'
DATE 2/18/93



PLAN
SCALE: 1" = 200'



NOTE:
SEE PLATE 1.1 FOR EXPLANATION

KEY:
● 22 BORING LOCATION AND NUMBER

PROPOSED METRO PASADENA LINE

Designed	LOS ANGELES COUNTY TRANSPORTATION COMMISSION
Drawn by <i>M.H.</i>	PASADENA - LOS ANGELES ROUTE REFINEMENT STUDY
Checked by	IN ASSOCIATION WITH: ANN VERMA ASSOCIATES ACOUSTICAL ANALYSIS ASSOCIATES D&S ASSOCIATES MICHAEL BRANDMAN ASSOCIATES, INC. PGH WONG ENGINEERING, INC. RALPH STONE AND COMPANY, INC.
Approved by	LAW / CRANDALL, INC.
Submitted:	

GEOLOGIC MAP AND BORING LOCATION PLAN

CONTRACT NO.	DATE
DRAWING NO.	2/18/93
SCALE	1" = 200'

CHKD

O.E.

DR

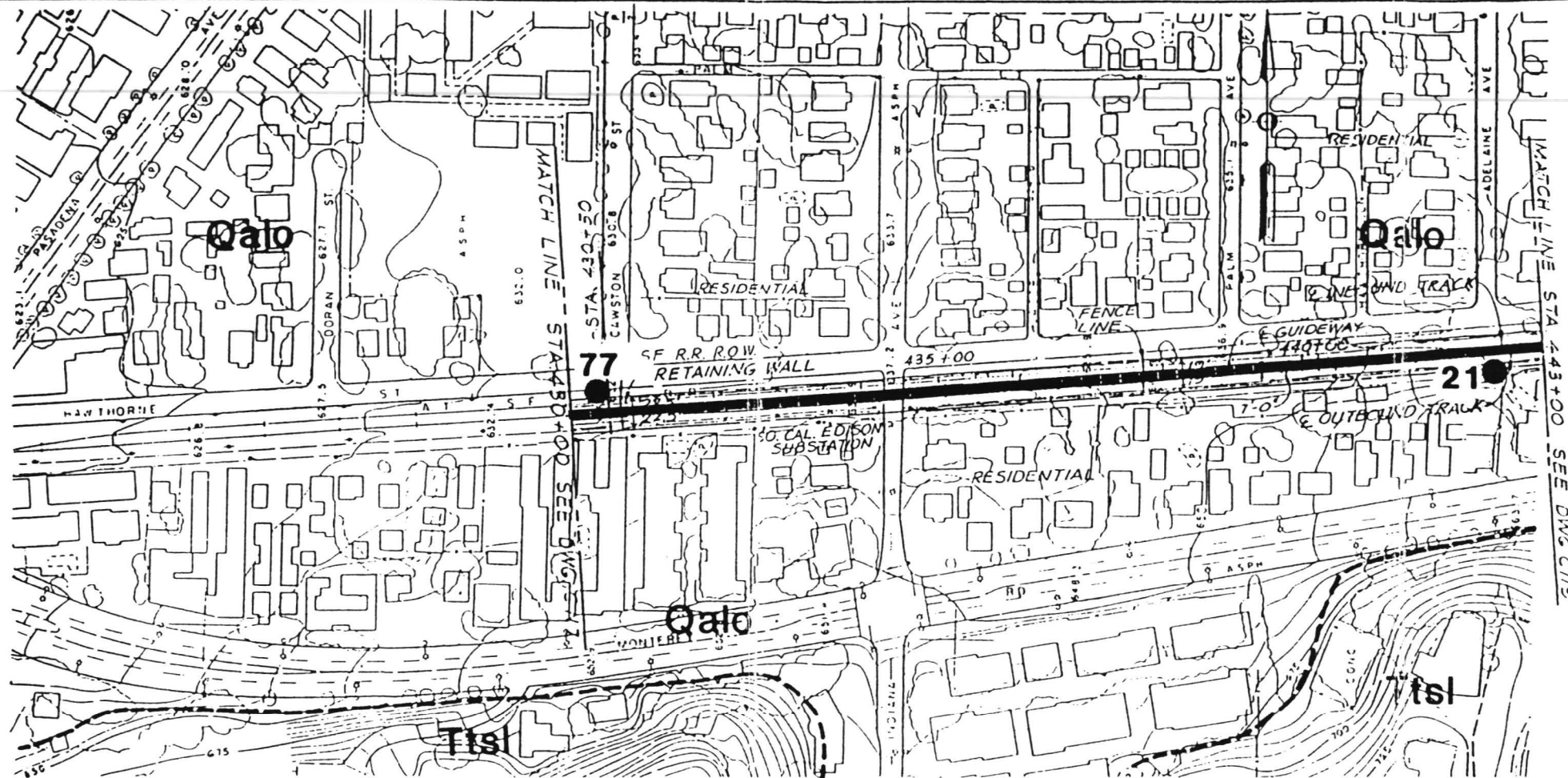
MOHAMMED

DATE

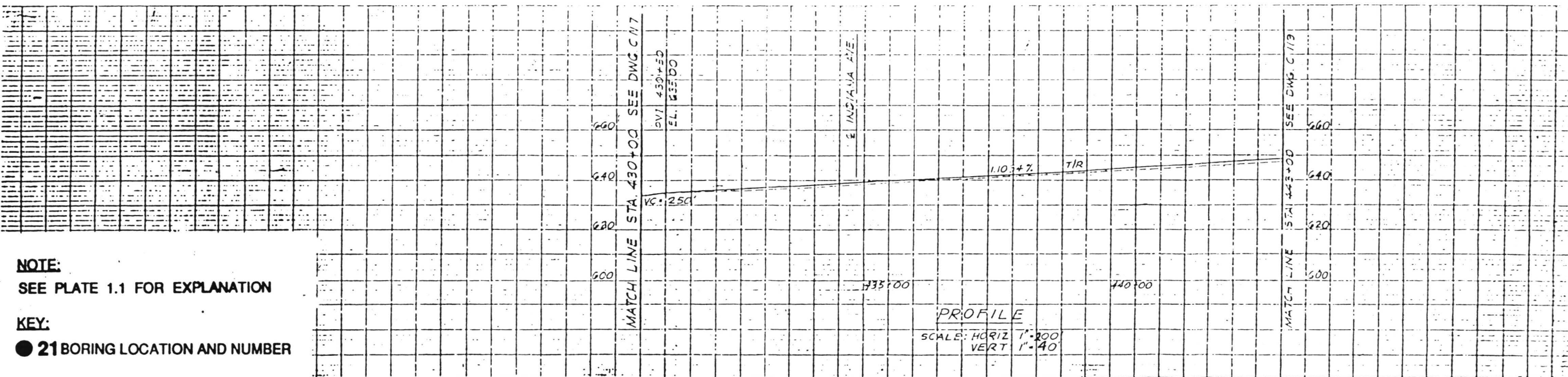
2/18/93

JOB

L92045.AE4



PLAN
SCALE: 1"=200'



NOTE:
SEE PLATE 1.1 FOR EXPLANATION

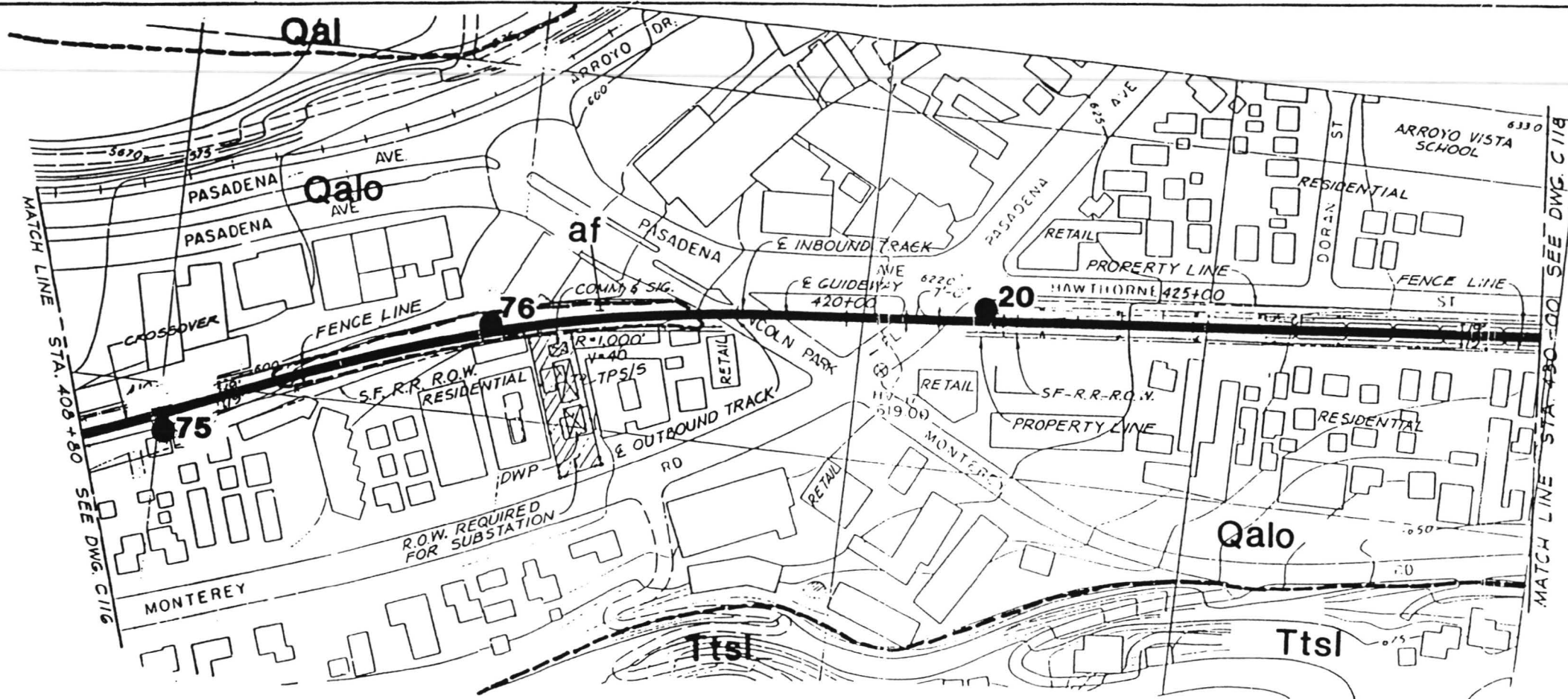
KEY:
● 21 BORING LOCATION AND NUMBER

PROPOSED METRO PASADENA LINE

Designed	LOS ANGELES COUNTY TRANSPORTATION COMMISSION
Drawn by <i>M.N.</i>	PASADENA - LOS ANGELES ROUTE REFINEMENT STUDY
Checked by	LACTC
Approved by	LAW / CRANDALL, INC.
Submitted:	DR. ASSOCIATION WITH AND VEHMA ASSOCIATES ACUSTICAL ANALYSIS ASSOCIATES DAS ASSOCIATES MICHAEL BRANNMAN ASSOCIATES, INC. P.G.H. WONG ENGINEERING, INC. RALPH STONE AND COMPANY, INC.

GEOLOGIC MAP AND BORING LOCATION PLAN

CONTRACT NO.	
DRAWING NO.	
SCALE 1" = 200'	DATE 2/18/93



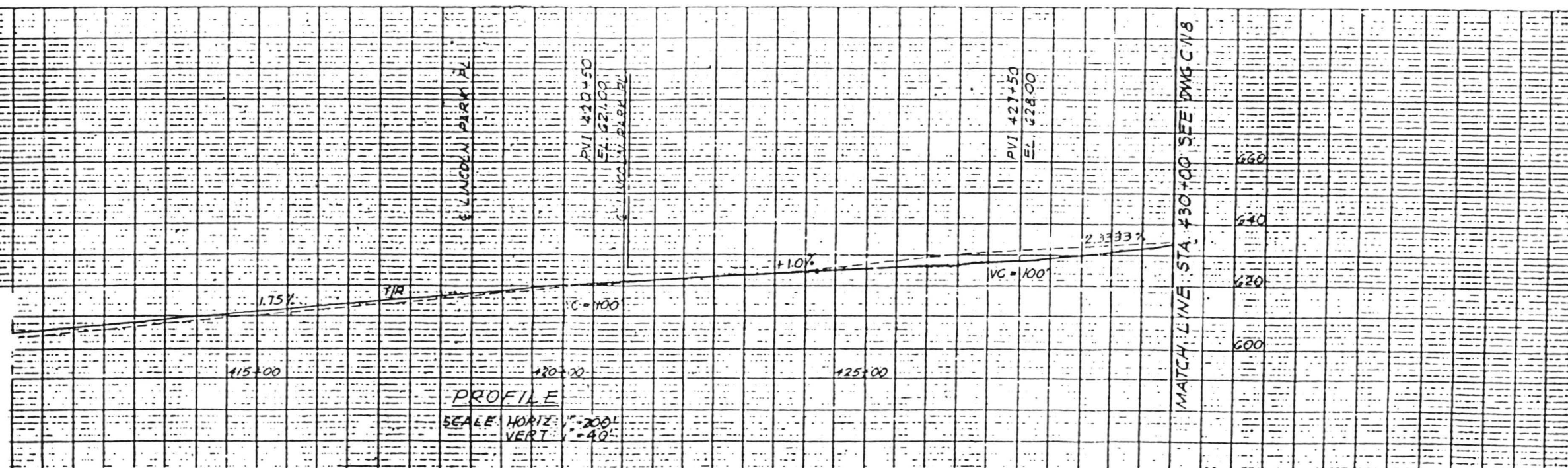
PLAN
SCALE: 1"=200'

NOTE:

SEE PLATE 1.1 FOR EXPLANATION

KEY:

● 20 BORING LOCATION AND NUMBER



PROPOSED METRO PASADENA LINE

Designed
Drawn by **M.H.**
Checked by
Approved by
Submitted:

LOS ANGELES COUNTY TRANSPORTATION COMMISSION
PASADENA - LOS ANGELES ROUTE REFINEMENT STUDY

IN ASSOCIATION WITH:
ANIL VERMA ASSOCIATES
ACOUSTICAL ANALYSIS ASSOCIATES
OKS ASSOCIATES
MICHAEL BRANDMAN ASSOCIATES, INC.
PGH WONG ENGINEERING, INC.
RALPH STONE AND COMPANY, INC.

GEOLOGIC MAP AND BORING LOCATION PLAN

CONTRACT NO.
DRAWING NO.

SCALE
1"=200'

DATE
2/18/93

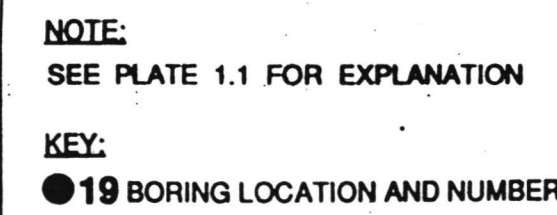
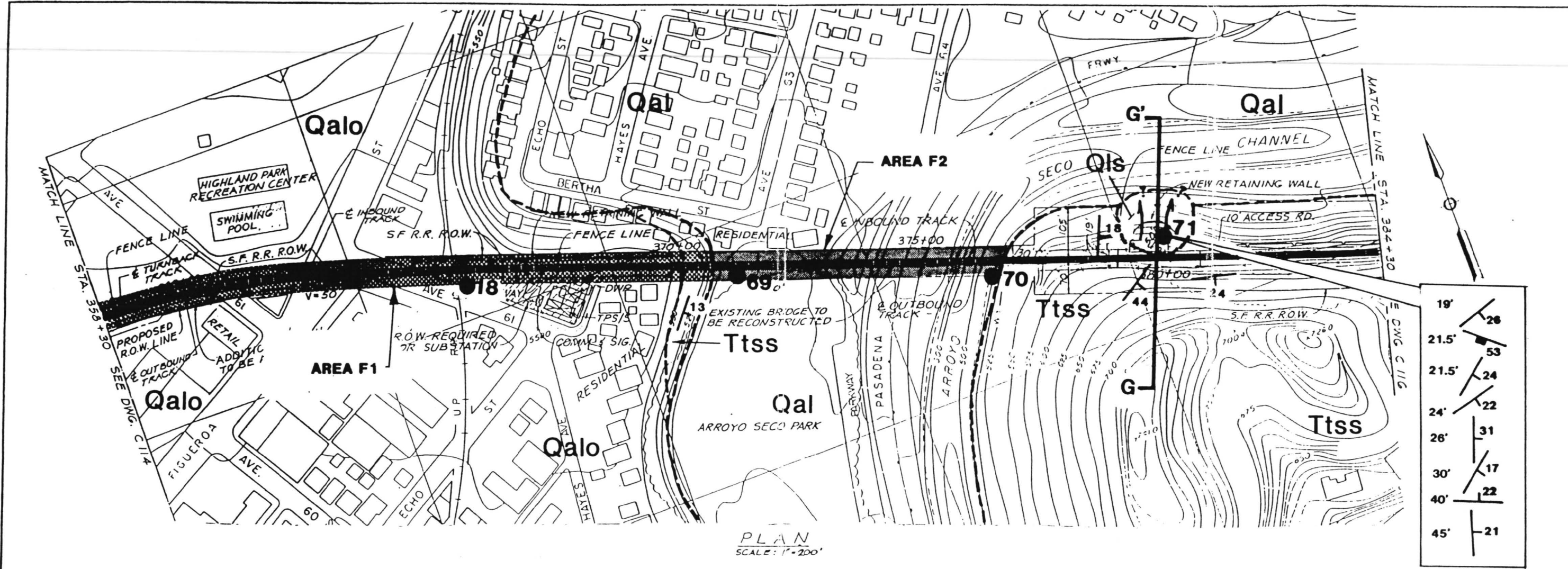
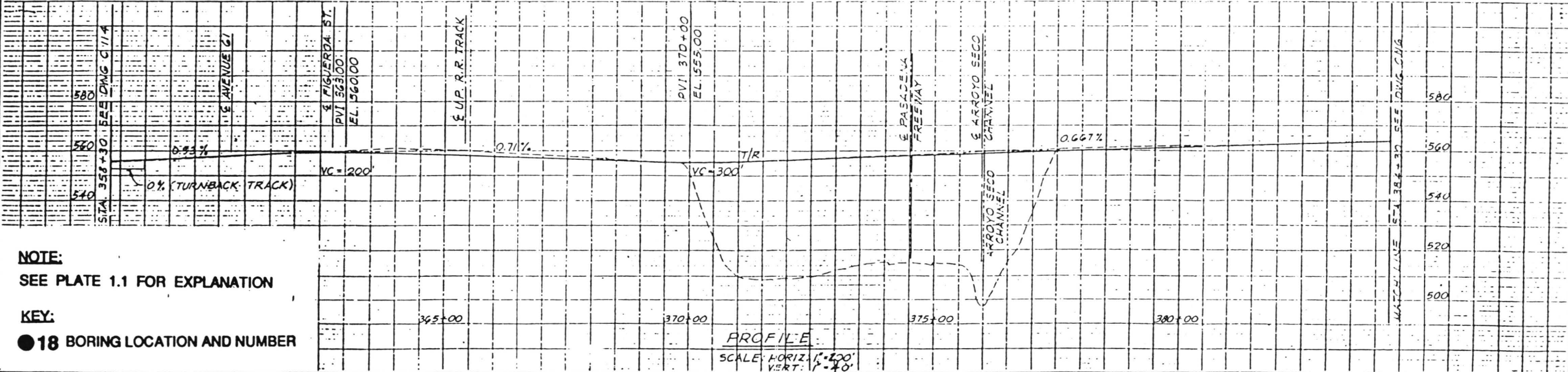


PLATE 1.14

JUL 92045.AE4 DATE 2/18/93 DR MOHAMMED O.E. CHKD



PLAN
SCALE: 1" = 200'



PROFILE
SCALE: HORIZ. 1" = 200'
VERT. 1" = 40'

NOTE:
SEE PLATE 1.1 FOR EXPLANATION

KEY:
● 18 BORING LOCATION AND NUMBER

PROPOSED METRO PASADENA LINE

Designed by
Drawn by *M.U.*
Checked by
Approved by
Submitted by

LOS ANGELES COUNTY TRANSPORTATION COMMISSION
PASADENA - LOS ANGELES ROUTE REFINEMENT STUDY

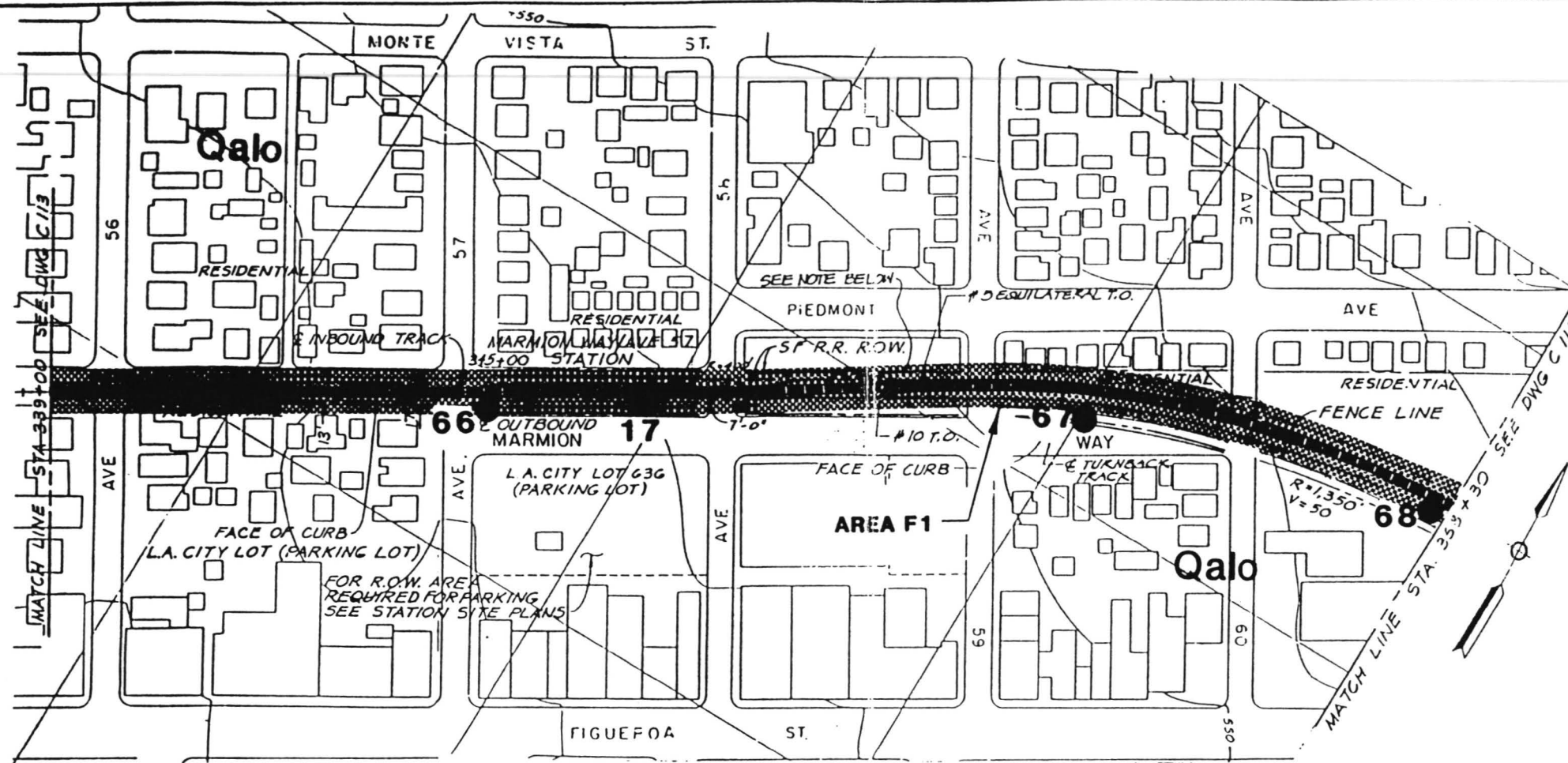
IN ASSOCIATION WITH:
ANIL VERMA ASSOCIATES
ACUSTICAL ANALYSIS ASSOCIATES
DKS ASSOCIATES
MICHAEL BRANDMAN ASSOCIATES, INC.
PCM WONG ENGINEERING, INC.
RALPH STONE AND COMPANY, INC.

LAW / CRANDALL, INC.

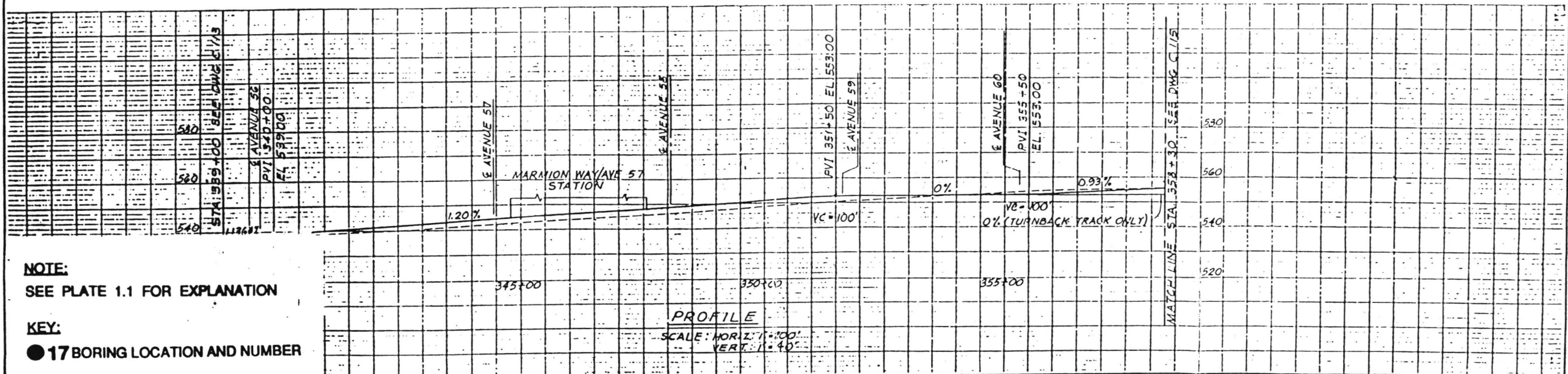
GEOLOGIC MAP AND BORING LOCATION PLAN

CONTRACT NO.
DRAWING NO.

SCALE 1" = 200' DATE 2/18/93



PLAN
SCALE: 1" = 200'



NOTE:
SEE PLATE 1.1 FOR EXPLANATION

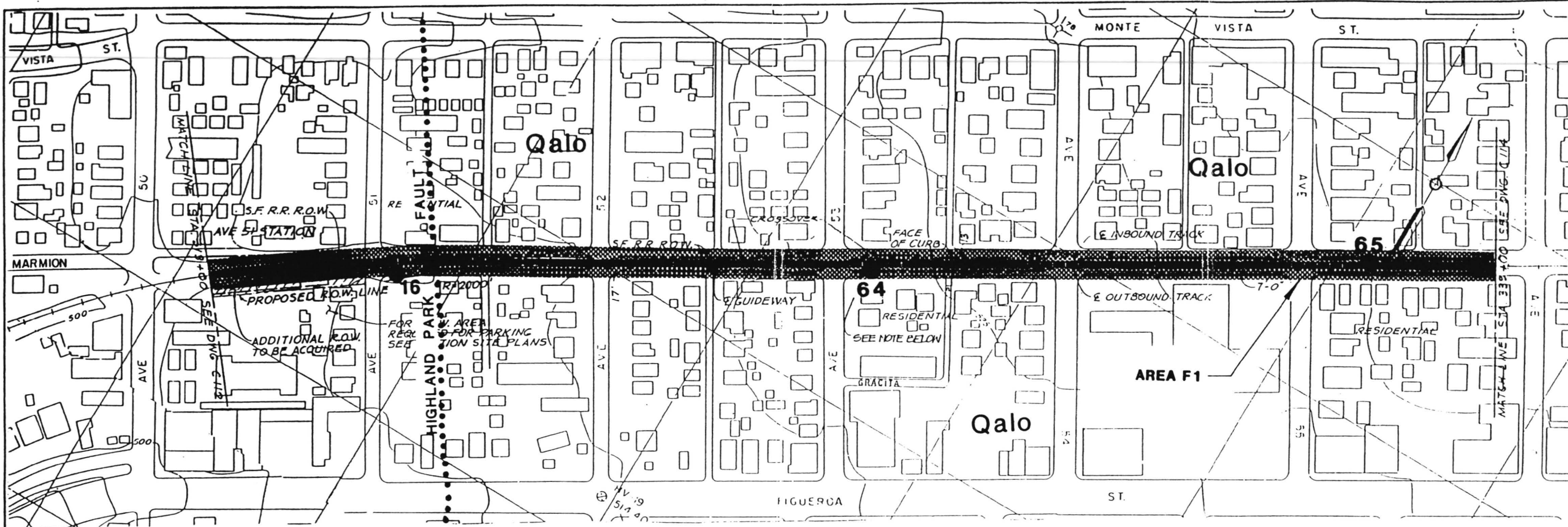
KEY:
● 17 BORING LOCATION AND NUMBER

PROPOSED METRO PASADENA LINE

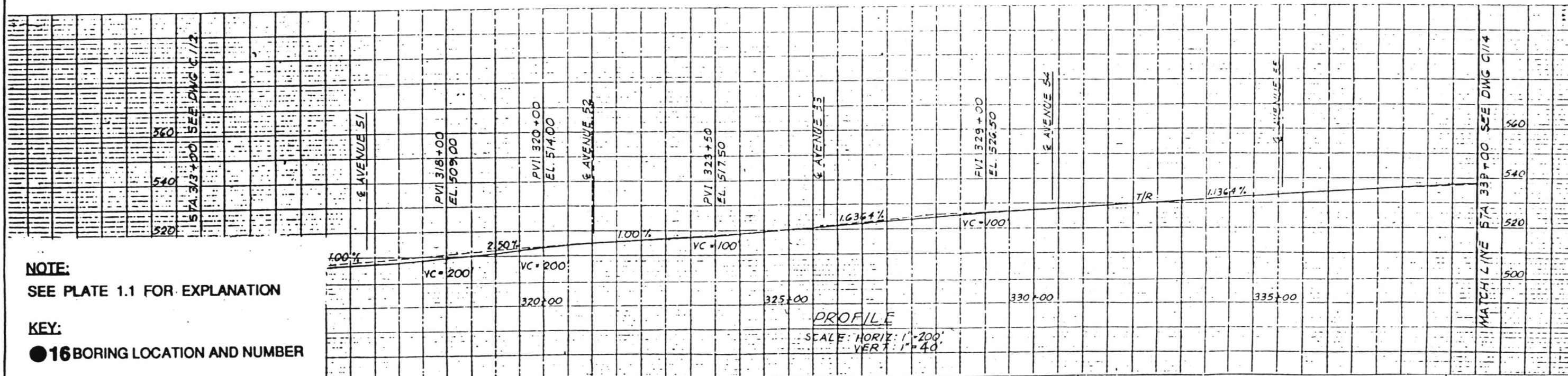
Designed	LOS ANGELES COUNTY TRANSPORTATION COMMISSION
Drawn by M.U.	PASADENA - LOS ANGELES ROUTE REFINEMENT STUDY
Checked by	IN ASSOCIATION WITH: ANIL VERMA ASSOCIATES ACOUSTICAL ANALYTICS ASSOCIATES DWS ASSOCIATES MICHAEL BRANDMAN ASSOCIATES, INC. PGH WONG ENGINEERING, INC. RZ/PH STONE AND COMPANY, INC.
Approved by	Submitted:

GEOLOGIC MAP AND BORING LOCATION PLAN

CONTRACT NO.	DRAWING NO.
SCALE 1" = 200'	DATE 2/18/93



PLAN
SCALE: 1"=200'



NOTE:
SEE PLATE 1.1 FOR EXPLANATION

KEY:
● 16 BORING LOCATION AND NUMBER

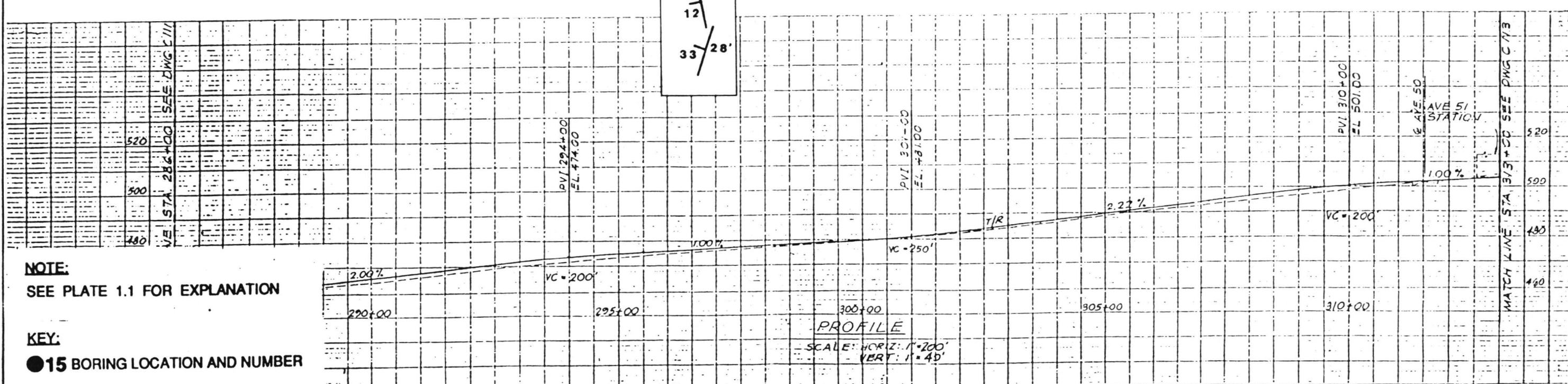
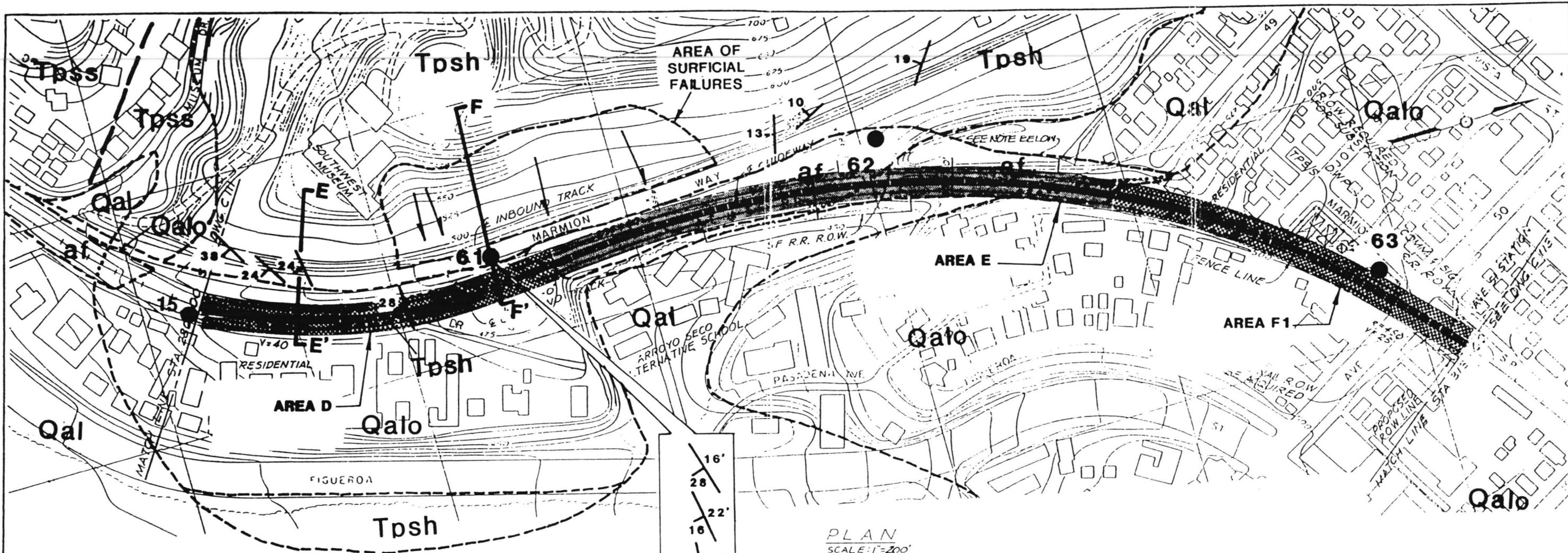
PROPOSED METRO PASADENA LINE

Designed
Drawn by *M.H.*
Checked by
Approved by

LOS ANGELES COUNTY TRANSPORTATION COMMISSION
PASADENA - LOS ANGELES ROUTE REFINEMENT STUDY
LACTC
IN ASSOCIATION WITH:
ANIL VERMA ASSOCIATES
ACOUSTICAL ANALYSIS ASSOCIATES
UKS ASSOCIATES
MICHAEL BRANDMAN ASSOCIATES, INC.
PGH WONG ENGINEERING, INC.
RALPH STONE AND COMPANY, INC.
Submitted:

GEOLOGIC MAP AND BORING LOCATION PLAN

CONTRACT NO.
DRAWING NO.
SCALE 1"=200' DATE 2/18/93



NOTE:
SEE PLATE 1.1 FOR EXPLANATION

KEY:
● 15 BORING LOCATION AND NUMBER

PROPOSED METRO PASADENA LINE

Designed
Drawn by *M.N.*
Checked by
Approved by

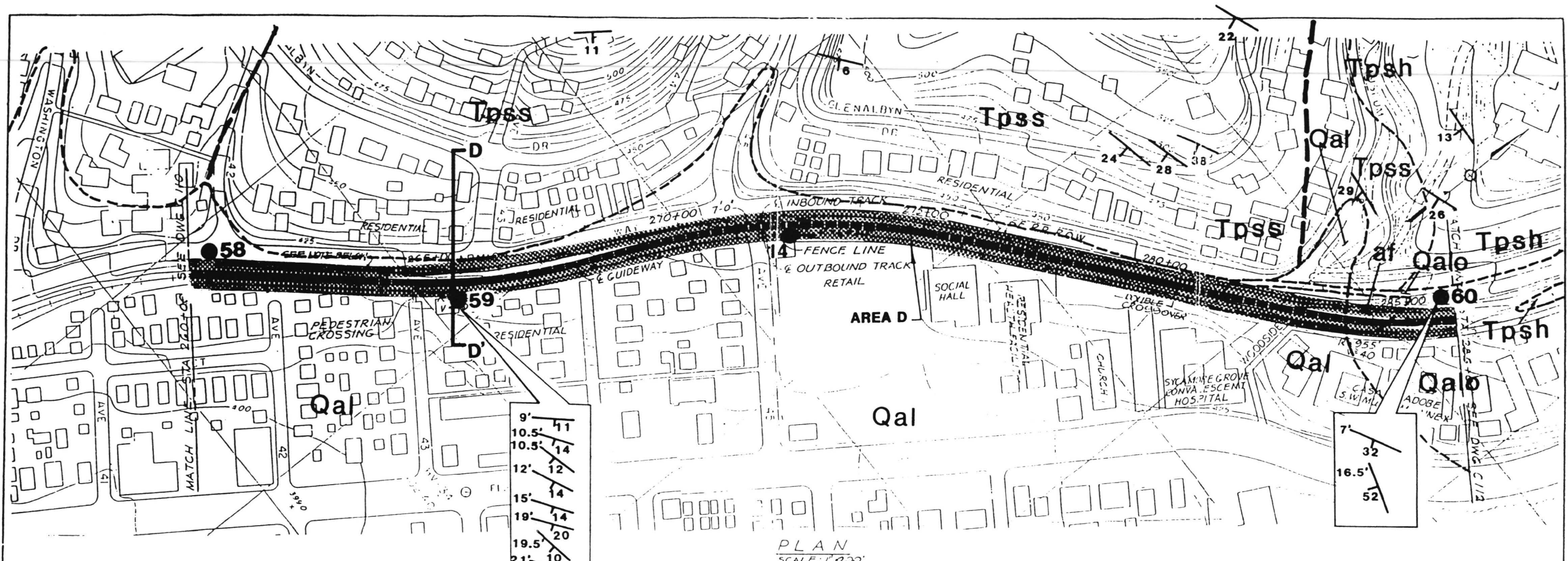


LOS ANGELES COUNTY TRANSPORTATION COMMISSION
PASADENA - LOS ANGELES ROUTE REFINEMENT STUDY
IN ASSOCIATION WITH:
ANH VERMA ASSOCIATES
ACQUISTAL ANALYSIS ASSOCIATES
D&S ASSOCIATES
MICHAEL BRANDMAN ASSOCIATES, INC.
PGH WONG ENGINEERING, INC.
RALPH STONE AND COMPANY, INC.

GEOLOGIC MAP AND BORING LOCATION PLAN

CONTRACT NO.
DRAWING NO.
SCALE 1" = 200'
DATE 2/18/93

JO L92045.AE4 DATE 2/18/93 DR. MOHAMMED O.E. CHKD



NOTE:
SEE PLATE 1.1 FOR EXPLANATION

KEY:
● 14 BORING LOCATION AND NUMBER

PROPOSED METRO PASADENA LINE

Designed
Drawn by *MB/MU*
Checked by
Approved by

LOS ANGELES COUNTY TRANSPORTATION COMMISSION
PASADENA - LOS ANGELES ROUTE REFINEMENT STUDY

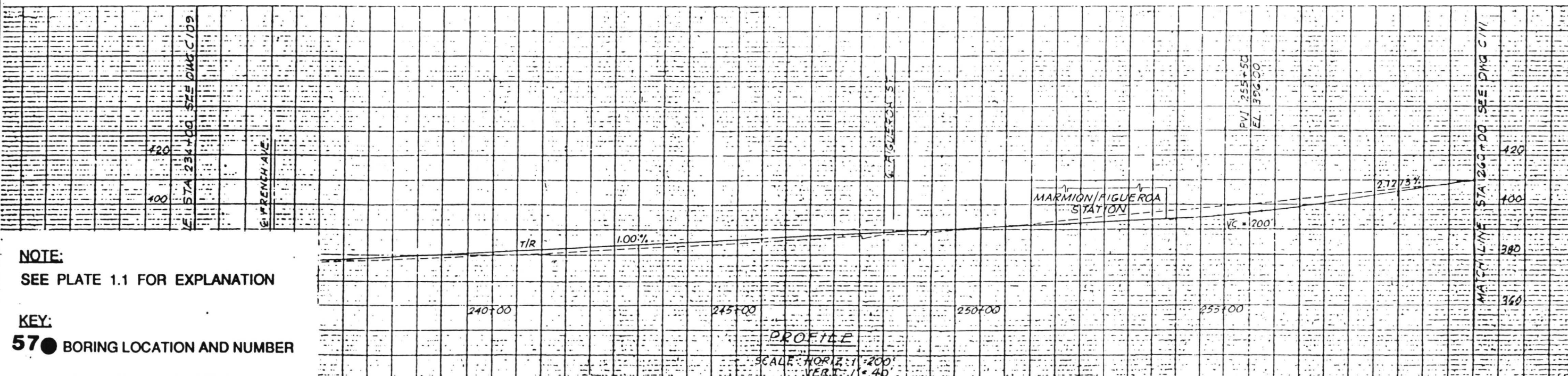
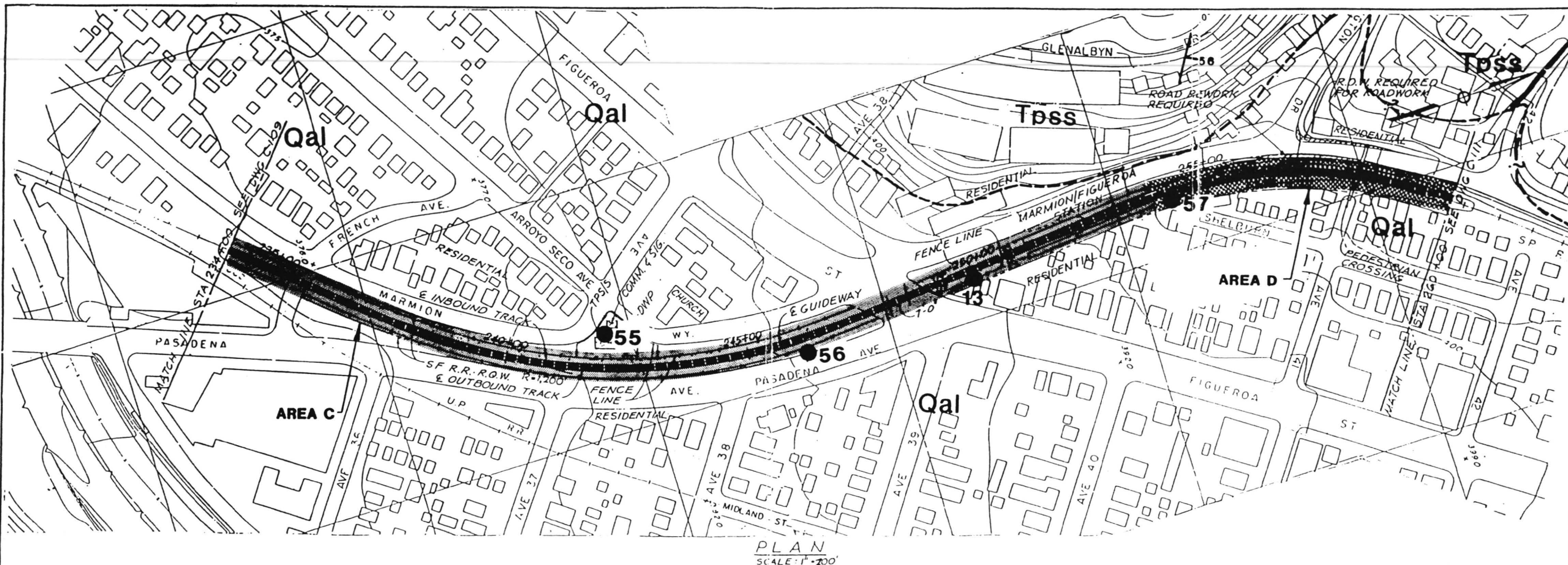
LACTC
LAW / CRANDALL, INC.
Submitted:

IN ASSOCIATION WITH:
ANIL VERMA ASSOCIATES
ACQUINTAL ANALYSIS ASSOCIATES
D&S ASSOCIATES
MICHAEL BRANDMAN ASSOCIATES, INC
PGH WONG ENGINEERING, INC
RALPH STONE AND COMPANY, INC

GEOLOGIC MAP AND BORING LOCATION PLAN

CONTRACT NO.
DRAWING NO.

SCALE 1" = 200'
DATE 2/18/93



NOTE:
SEE PLATE 1.1 FOR EXPLANATION

KEY:
57 ● BORING LOCATION AND NUMBER

PROPOSED METRO PASADENA LINE

Designed

Drawn *M.H.N.*

Checked by

Approved by



LOS ANGELES COUNTY TRANSPORTATION COMMISSION
PASADENA - LOS ANGELES ROUTE REFINEMENT STUDY



LAW / CRANDALL, INC.

Submitted:

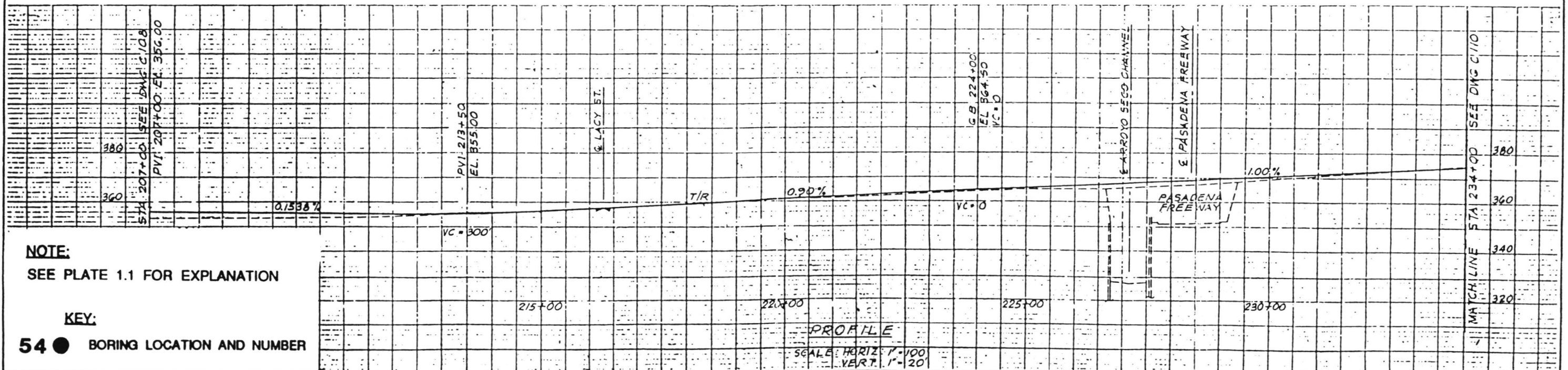
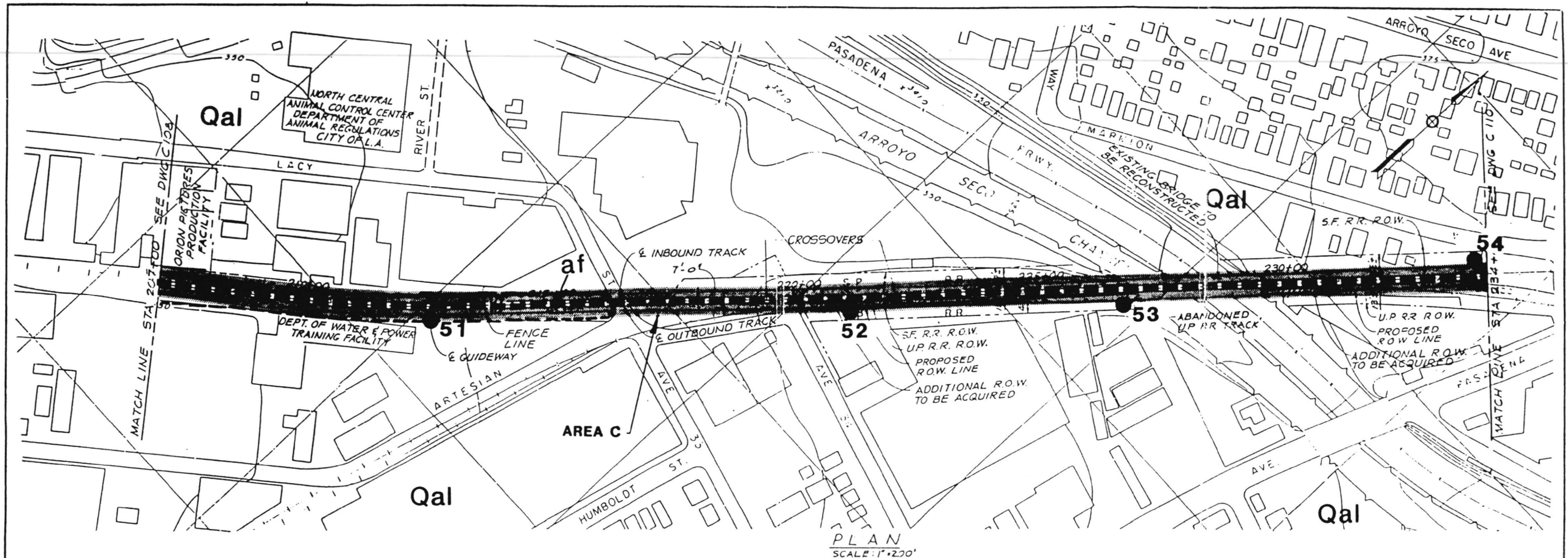
IN ASSOCIATION WITH:
ANIL VERMA ASSOCIATES
ACOUSTICAL ANALYSIS ASSOCIATES
DRS ASSOCIATES
MICHAEL BRANDMAN ASSOCIATES, INC.
PCN WONG ENGINEERING, INC.
RALPH STONE AND COMPANY, INC.

GEOLOGIC MAP AND BORING LOCATION PLAN

CONTRACT NO.
DRAWING NO.

SCALE
1" = 200'

DATE
2/18/93



PROPOSED METRO PASADENA LINE

Designed by
Drawn by *M.A.*
Checked by
Approved by

LOS ANGELES COUNTY TRANSPORTATION COMMISSION
PASADENA - LOS ANGELES ROUTE REFINEMENT STUDY

IN ASSOCIATION WITH:
ANIL VERMA ASSOCIATES
ACOUSTICAL ANALYSIS ASSOCIATES
DKS ASSOCIATES
MICHAEL BRANDMAN ASSOCIATES, INC.
PGH WONG ENGINEERING, INC.
RALPH STONE AND COMPANY, INC.

LACTC
LAW / CRANDALL, INC.
Submitted:

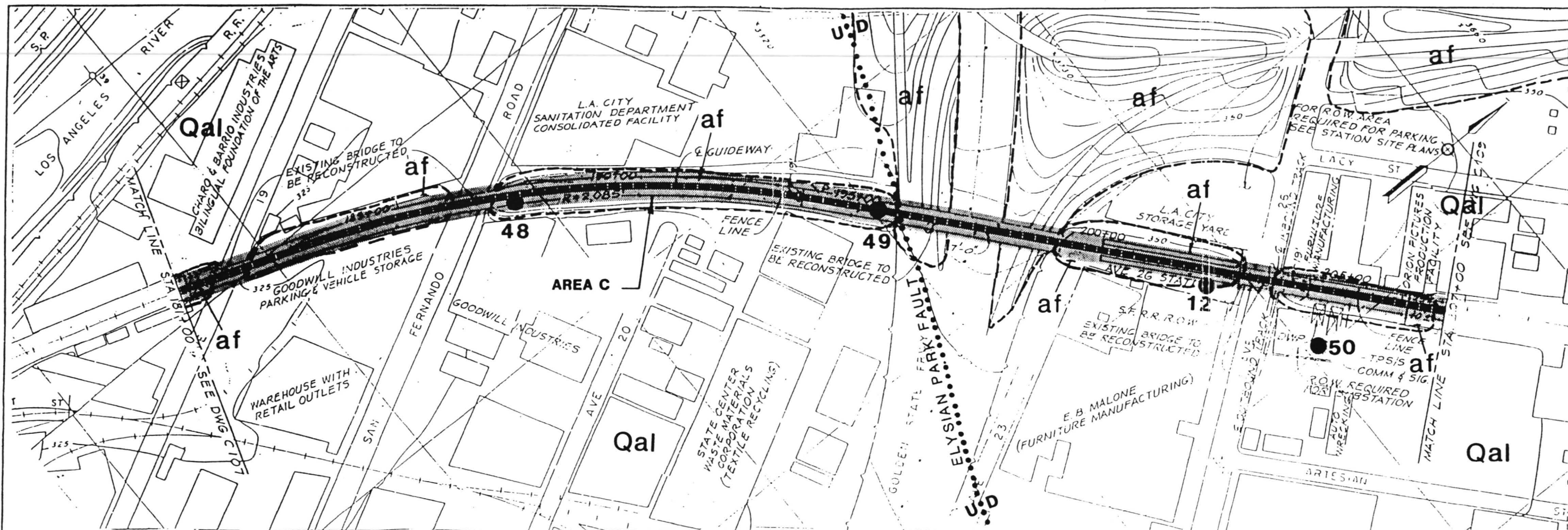
GEOLOGIC MAP AND BORING LOCATION PLAN

CONTRACT NO.
DRAWING NO.
SC 109

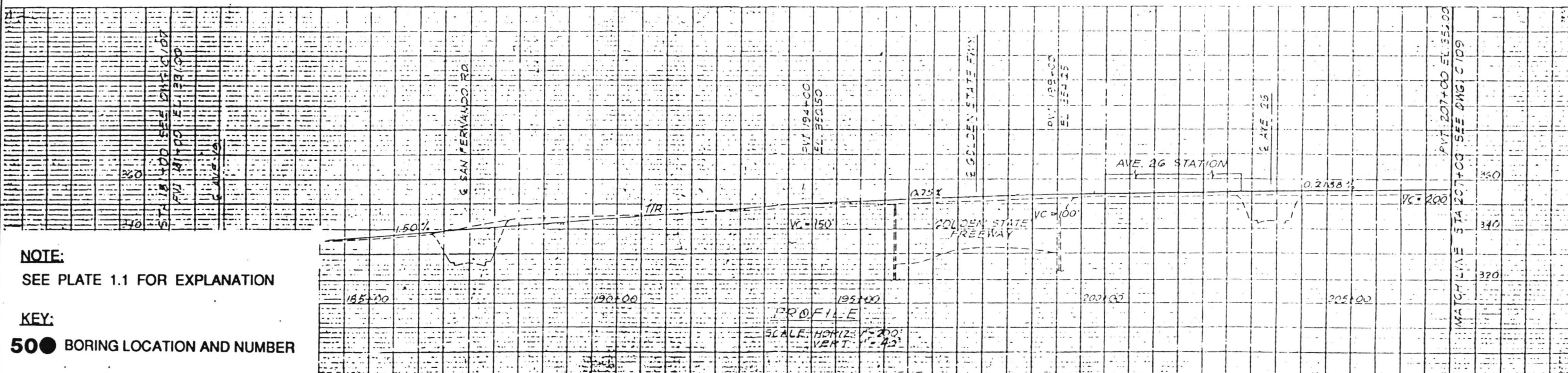
SCALE
1" = 200'

DATE
2/18/93

SHEET NUMBER



PLAN
SCALE: 1"=200'



NOTE:

SEE PLATE 1.1 FOR EXPLANATION

KEY:

50● BORING LOCATION AND NUMBER

PROPOSED METRO PASADENA LINE

Designed

Drawn by *AB/M.N.*

Checked by

Approved by



LOS ANGELES COUNTY TRANSPORTATION COMMISSION
PASADENA-LOS ANGELES ROUTE REFINEMENT STUDY



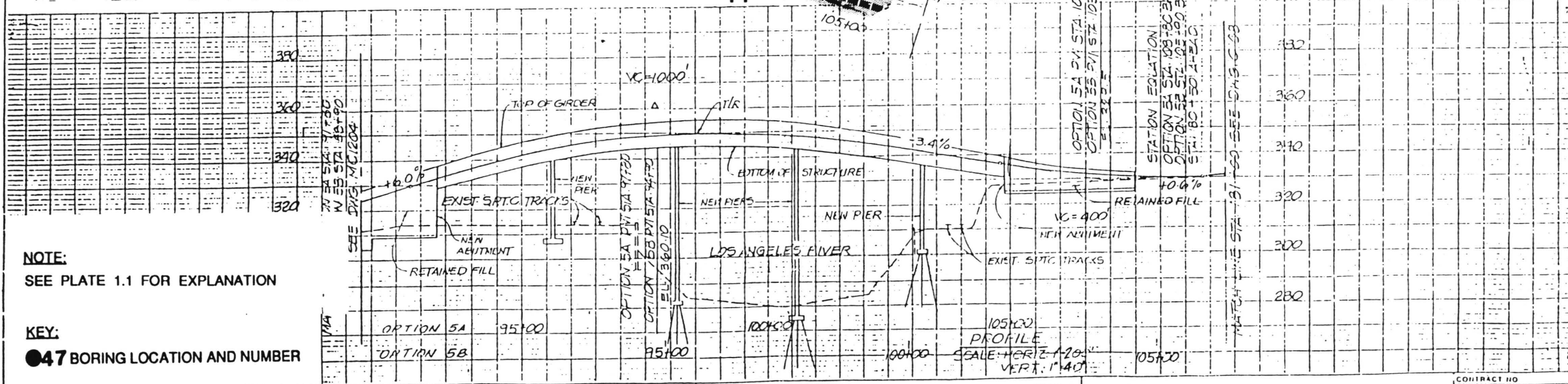
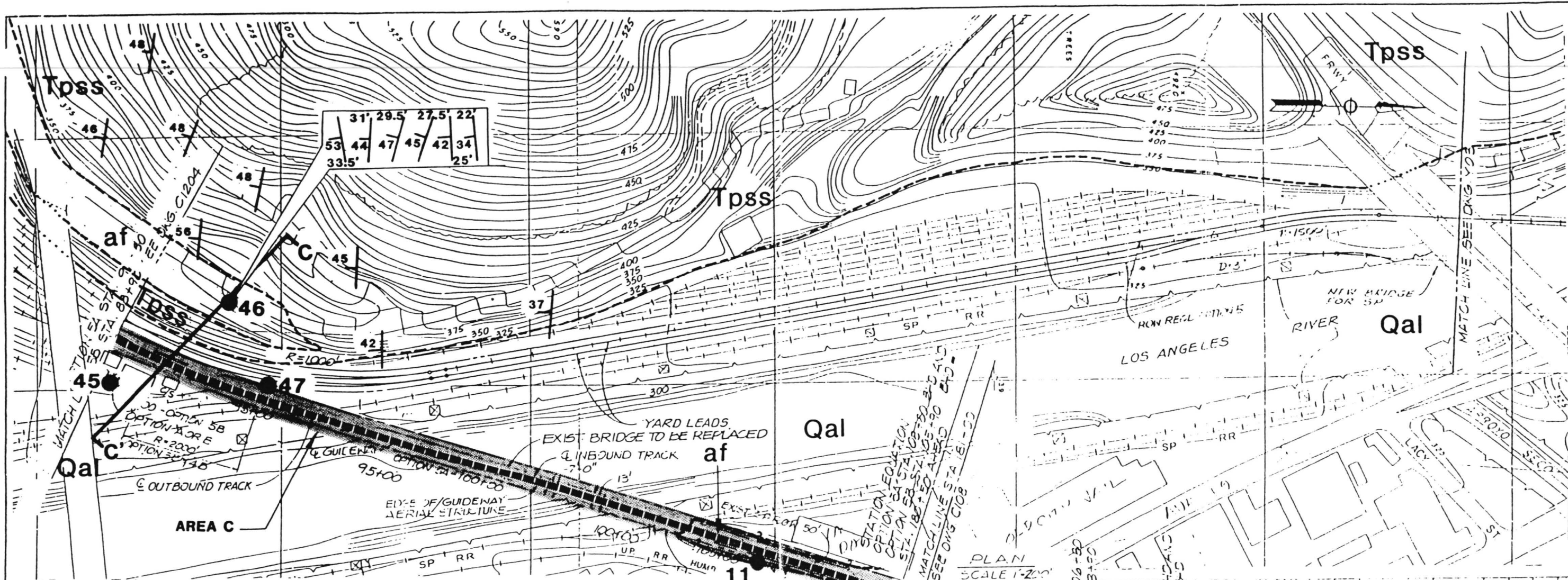
LAW / CRANDALL, INC.

Submitted:

IN ASSOCIATION WITH:
ANIL VERMA ASSOCIATES
ACOUSTICAL ANALYSIS ASSOCIATES
DKS ASSOCIATES
MICHAEL BRANDMAN ASSOCIATES, INC.
PGH WONG ENGINEERING, INC.
RALPH STONE AND COMPANY, INC.

GEOLOGIC MAP AND BORING LOCATION PLAN

CONTRACT NO.
DRAWING NO.SCALE
1" = 200' 2/18/93



NOTE:
SEE PLATE 1.1 FOR EXPLANATION

KEY:
● 47 BORING LOCATION AND NUMBER

PROPOSED METRO PASADENA LINE

Designed
Drawn by *205/M.L.*
Checked by
Approved by
Submitted

LOS ANGELES COUNTY TRANSPORTATION COMMISSION
PASADENA - LOS ANGELES ROUTE REFINEMENT STUDY

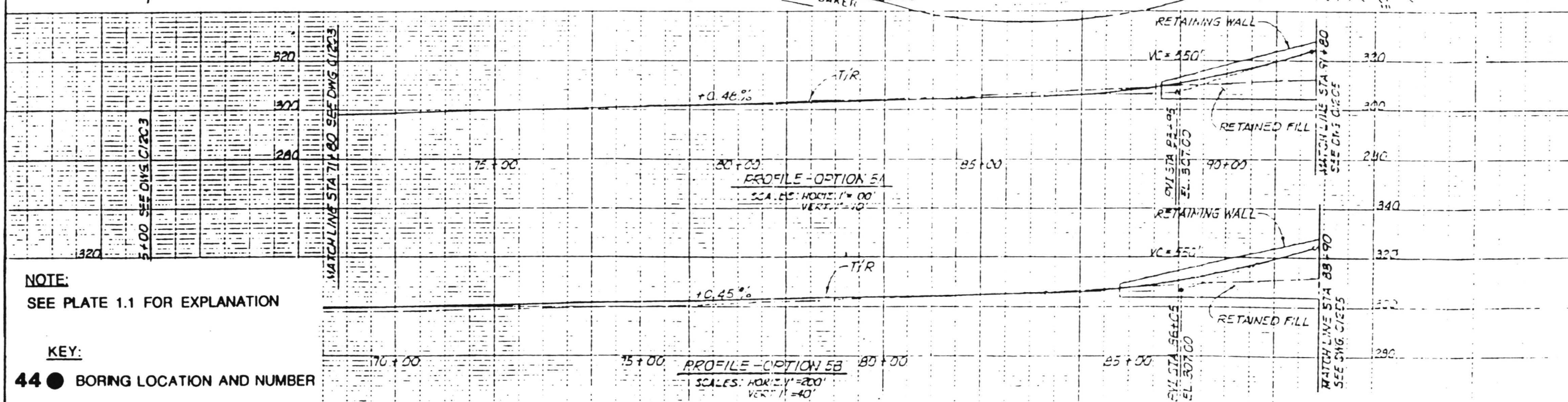
IN ASSOCIATION WITH:
ACOUSTICAL ANALYSIS ASSOCIATES
ANIL VERMA ASSOCIATES
DKS ASSOCIATES
MICHAEL BRANDMAN ASSOCIATES, INC.
PGH WONG ENGINEERING, INC.
RALPH STONE AND COMPANY, INC.

GEOLOGIC MAP AND BORING LOCATION PLAN

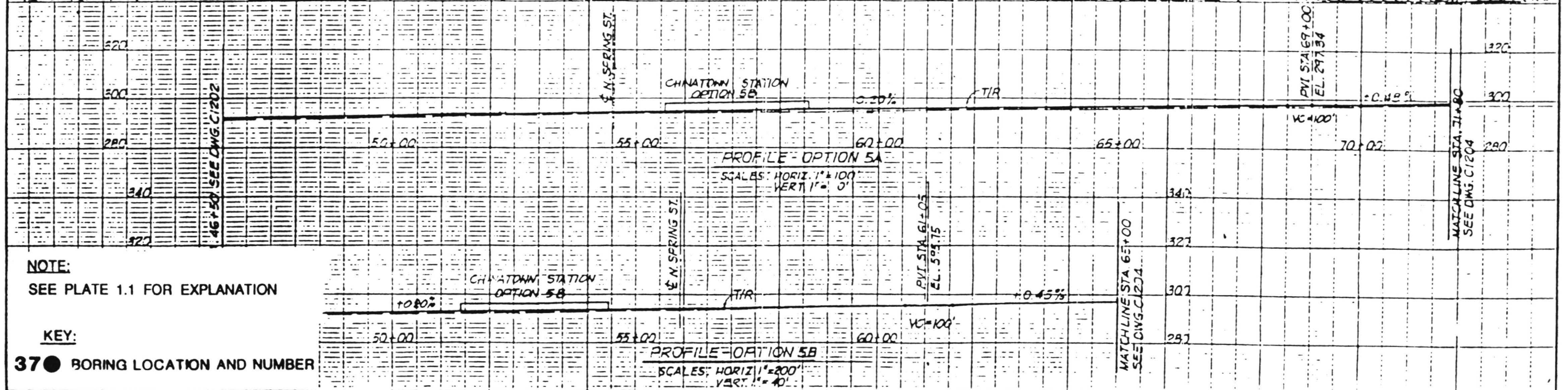
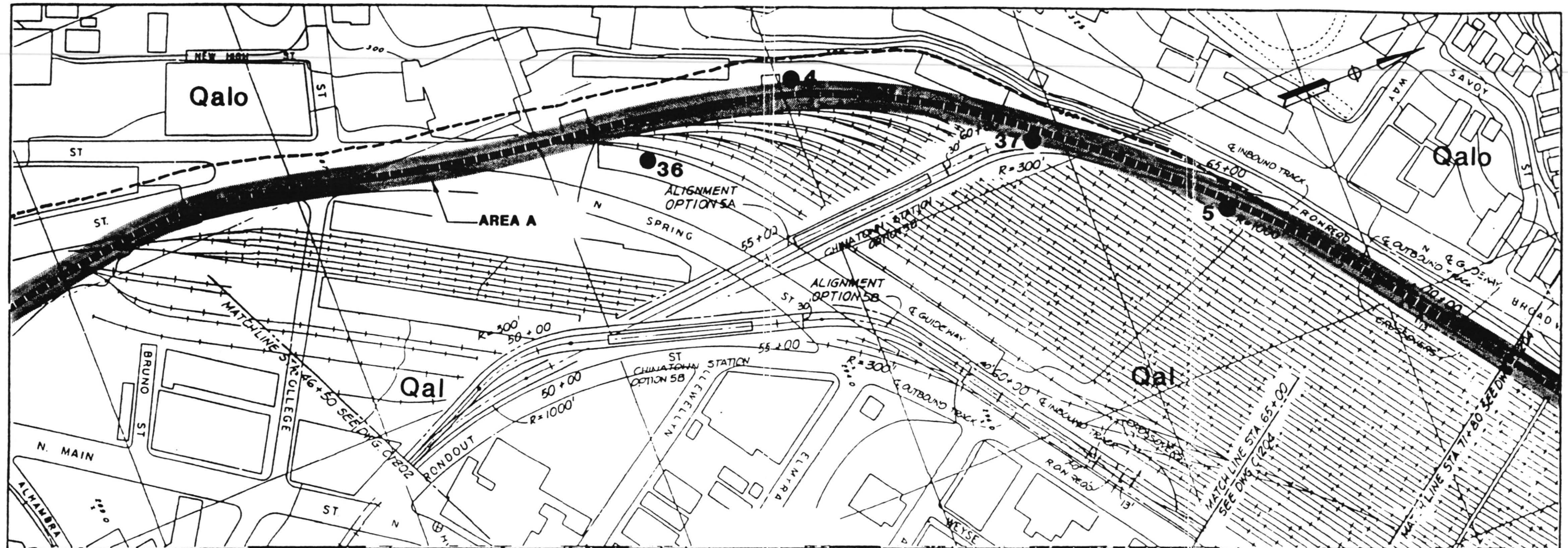
CONTRACT NO.
DRAWING NO.

SCALE
1" = 200'

DATE
2/18/93



CONTRACT NO.	
DRAWING NO.	
SCALE 1" = 200'	DATE 2/18/9
SHEET NUMBER	



NOTE:
SEE PLATE 1.1 FOR EXPLANATION

KEY:
37 ● BORING LOCATION AND NUMBER

PROPOSED METRO PASADENA LINE

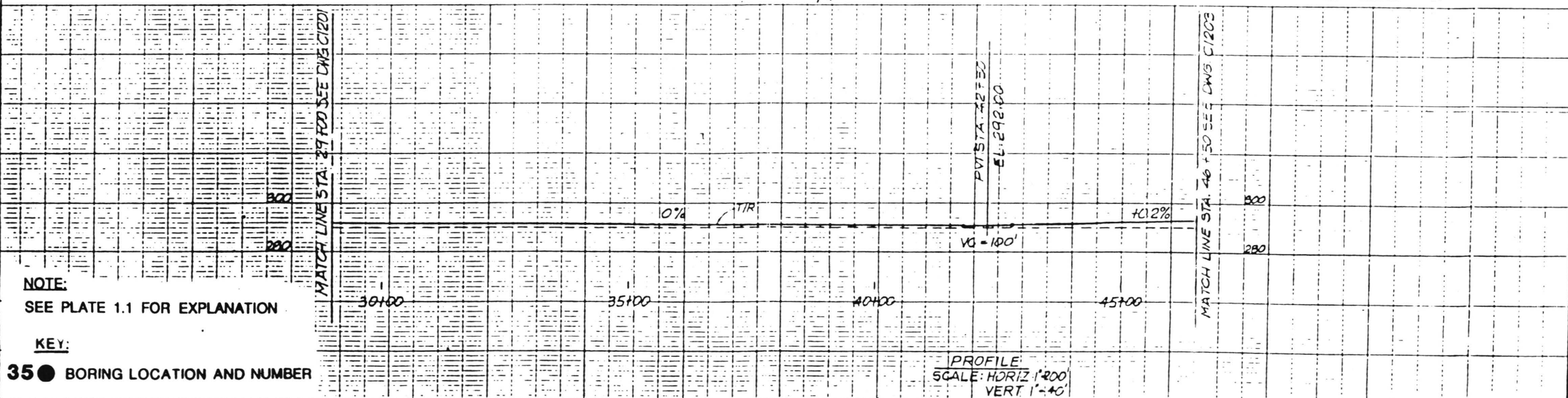
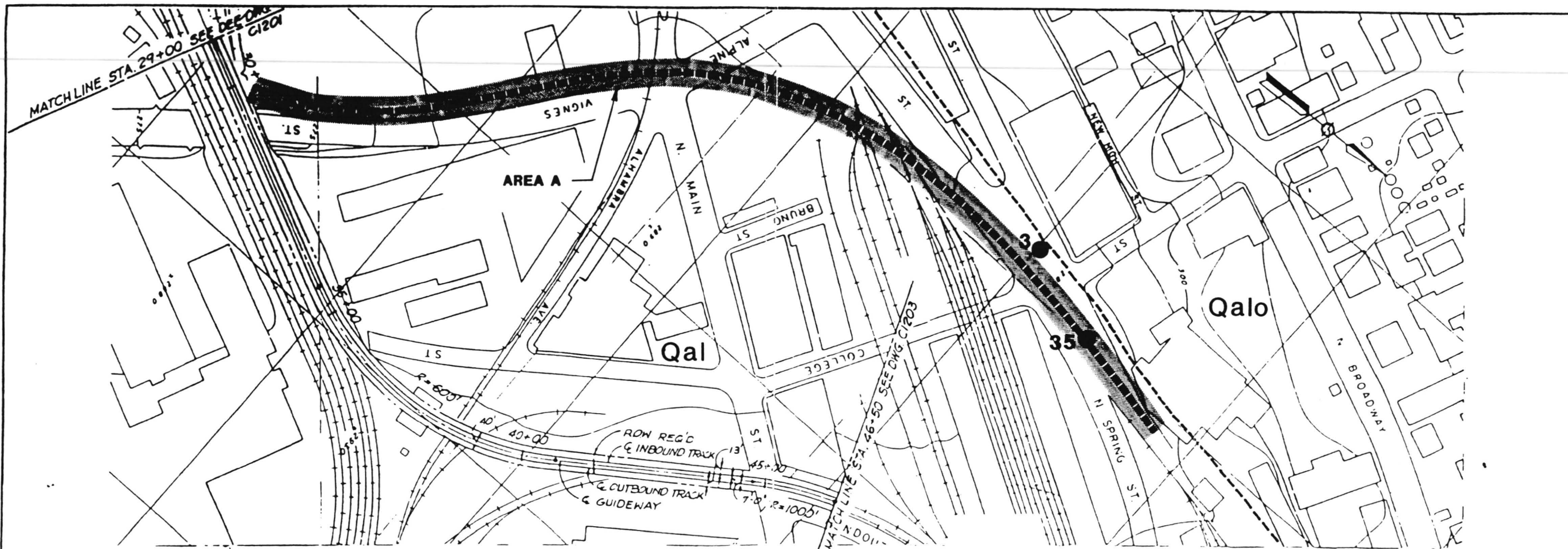
Designed by
Drawn by M.N.
Checked by
Approved by
Submitted:

LOS ANGELES COUNTY TRANSPORTATION COMMISSION
PASADENA - LOS ANGELES ROUTE REFINEMENT STUDY

IN ASSOCIATION WITH:
AMT, VERBA ASSOCIATES
ACOUSTICAL ANALYSIS ASSOCIATES
DES ASSOCIATES
MICHAEL BRANDMAN ASSOCIATES, INC.
PBN WONG ENGINEERING, INC.
RALPH STONE AND COMPANY, INC.

GEOLOGIC MAP AND BORING LOCATION PLAN

CONTRACT NO.
DRAWING NO.
SCALE 1" = 200' DATE 2/18/93
SHEET NUMBER



NOTE:
SEE PLATE 1.1 FOR EXPLANATION

KEY:

35● BORING LOCATION AND NUMBER

PROPOSED METRO PASADENA LINE

Designed by

Drawn by MAI

Checked by

Approved by



LOS ANGELES COUNTY TRANSPORTATION COMMISSION
PASADENA-LOS ANGELES ROUTE REFINEMENT STUDY



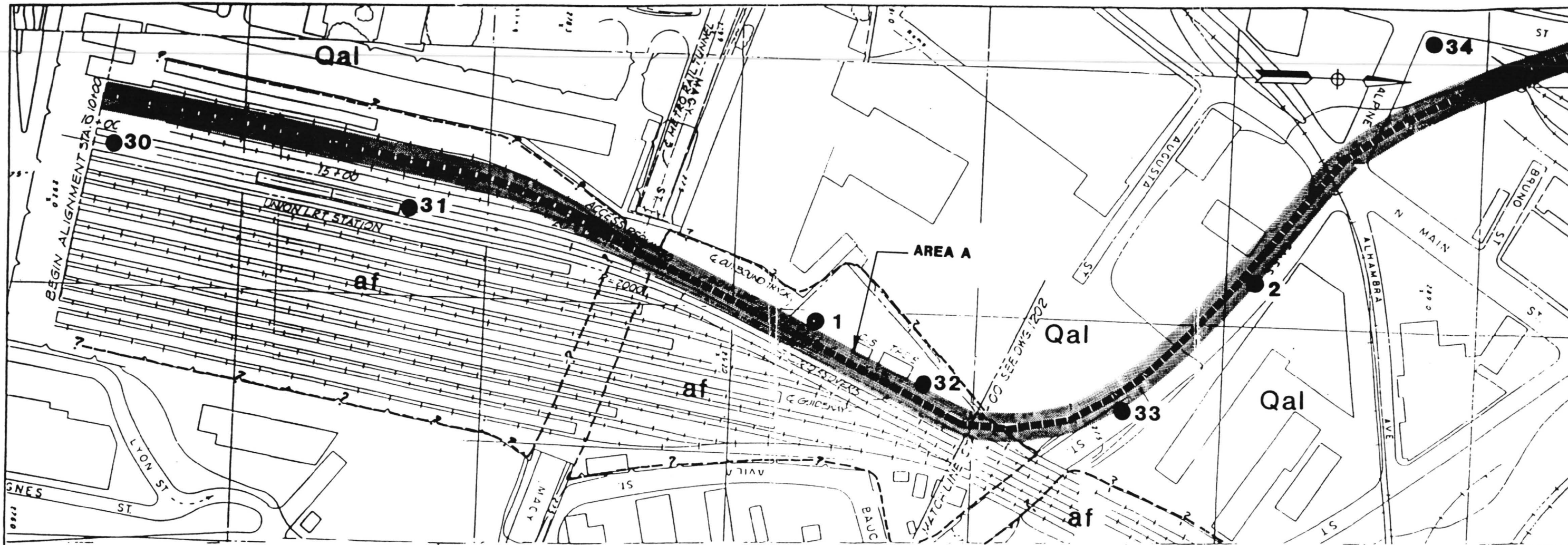
Submitted:

IN ASSOCIATION WITH
ANIL VERMA ASSOCIATES
ACUSTICAL ANALYSIS ASSOCIATES
DHS ASSOCIATES
MICHAEL BRAHMAN ASSOCIATES, INC.
POM WONG ENGINEERING, INC.
RALPH STONE AND COMPANY, INC.

GEOLOGIC MAP AND BORING LOCATION PLAN

CONTRACT NO
DRAWING NO

SCALE	DATE
1" = 200'	2/18/93



EXPLANATION

- | | |
|-------------------------|--|
| af | ARTIFICIAL FILL |
| Qal | ALLUVIUM |
| Qls | LANDSLIDE DEBRIS |
| Qlso | OLDER LANDSLIDE DEBRIS |
| Qalo | OLDER ALLUVIUM |
| Qt | TERRACE DEPOSITS |
| Tf | FERNANDO FORMATION: sandstone and conglomerate |
| Tpsh Tpss | PUENTE FORMATION, Tpsh: shale, Tpss: sandstone |
| Ttss Ttcg | TOPANGA FORMATION: Ttss: sandstone, Ttcg: conglomerate |
- GEOLOGIC CONTACT, dashed where approximately located
queried where inferred

- ? $\frac{D}{U}$... FAULT, dashed where approximately located, queried where inferred or hypothetical, dotted where concealed
- $\frac{64}{53}$ STRIKE AND DIP OF BEDDING
- $\frac{79}{53}$ STRIKE AND DIP OF JOINT
- $\frac{79}{53}$ STRIKE AND DIP OF SHEAR
- LANDSLIDE, arrows indicate direction of movement
- GEOLOGIC SECTION
- PROPOSED ALIGNMENT
- 34 ● BORING LOCATION AND NUMBER

PROPOSED METRO PASADENA LINE

Designed by
Drawn by *M.N.*
Checked by
Approved by

LOS ANGELES COUNTY TRANSPORTATION COMMISSION
PASADENA-LOS ANGELES ROUTE REFINEMENT STUDY

IN ASSOCIATION WITH:
ANN VERMA ASSOCIATES
ACUSTICAL ANALYSIS ASSOCIATES
DHS ASSOCIATES
MICHAEL BRANDMAN ASSOCIATES, INC.
PSH WONG ENGINEERING, INC.
RALPH STONE AND COMPANY, INC.

LAW/CRANDALL, INC.
Submitted:

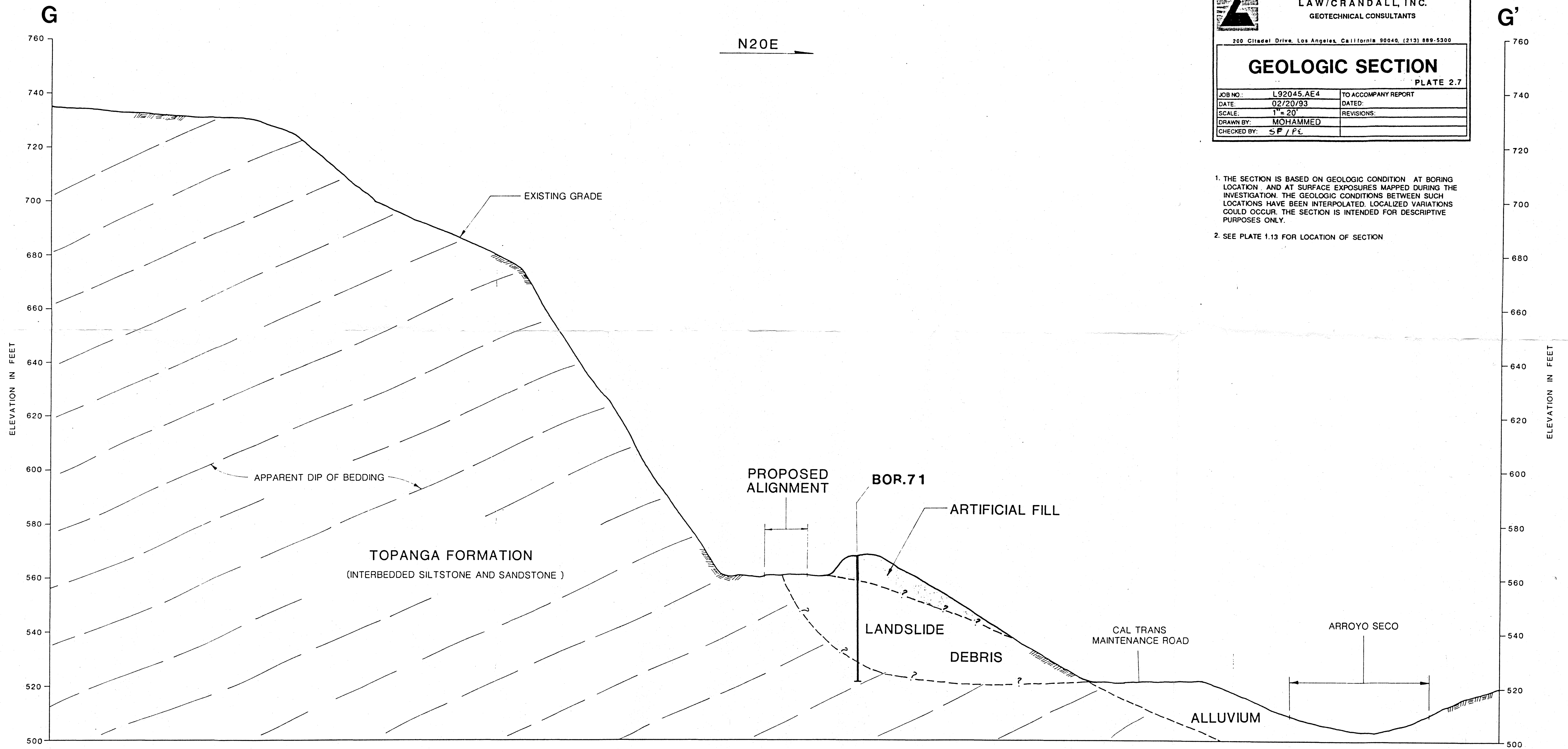
GEOLOGIC MAP AND BORING LOCATION PLAN

CONTRACT NO.
DRAWING NO.

SCALE
1" = 200'

DATE
2/18/93

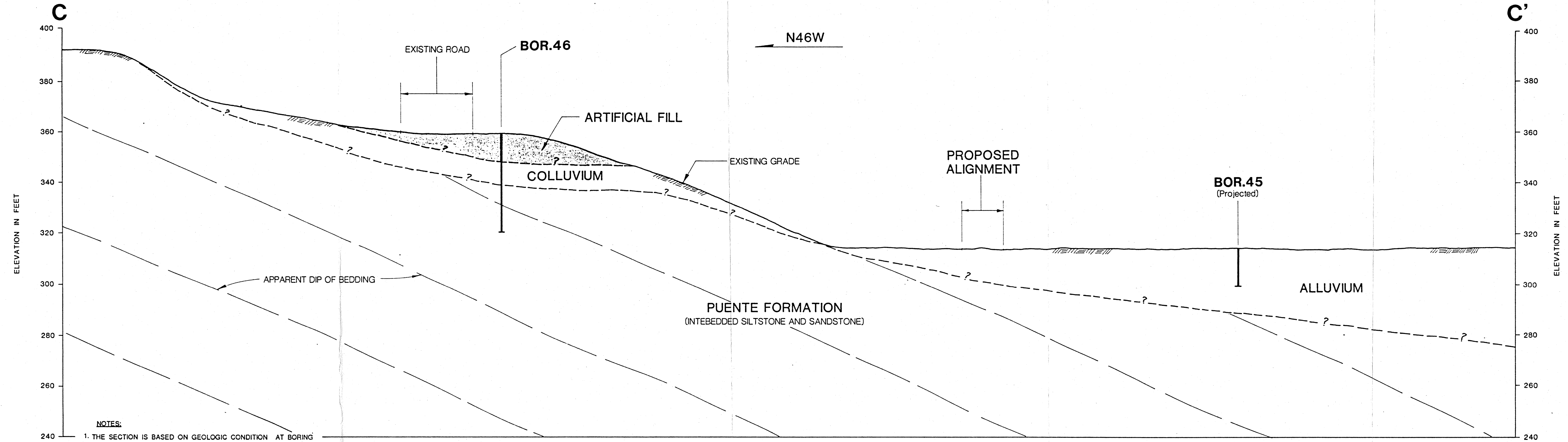
SHEET NUMBER



	
LAW/CRANDALL, INC. GEOTECHNICAL CONSULTANTS	
200 Citadel Drive, Los Angeles, California 90040, (213) 889-5300	
GEOLOGIC SECTION	
PLATE 2.7	
JOB NO.: L92045.AE4	TO ACCOMPANY REPORT
DATE: 02/20/93	DATED:
SCALE: 1" = 20'	REVISIONS:
DRAWN BY: MOHAMMED	
CHECKED BY: SF/PL	

1. THE SECTION IS BASED ON GEOLOGIC CONDITION AT BORING LOCATION, AND AT SURFACE EXPOSURES MAPPED DURING THE INVESTIGATION. THE GEOLOGIC CONDITIONS BETWEEN SUCH LOCATIONS HAVE BEEN INTERPOLATED. LOCALIZED VARIATIONS COULD OCCUR. THE SECTION IS INTENDED FOR DESCRIPTIVE PURPOSES ONLY.
2. SEE PLATE 1.13 FOR LOCATION OF SECTION

04 E4 DATE 1/9 M.L. J.E. 1/2



NOTES:

1. THE SECTION IS BASED ON GEOLOGIC CONDITION AT BORING LOCATION AND AT SURFACE EXPOSURES MAPPED DURING THE INVESTIGATION. THE GEOLOGIC CONDITIONS BETWEEN SUCH LOCATIONS HAVE BEEN INTERPOLATED. LOCALIZED VARIATIONS COULD OCCUR. THE SECTION IS INTENDED FOR DESCRIPTIVE PURPOSES ONLY.

2. SEE PLATE 1.5 FOR LOCATION OF SECTION

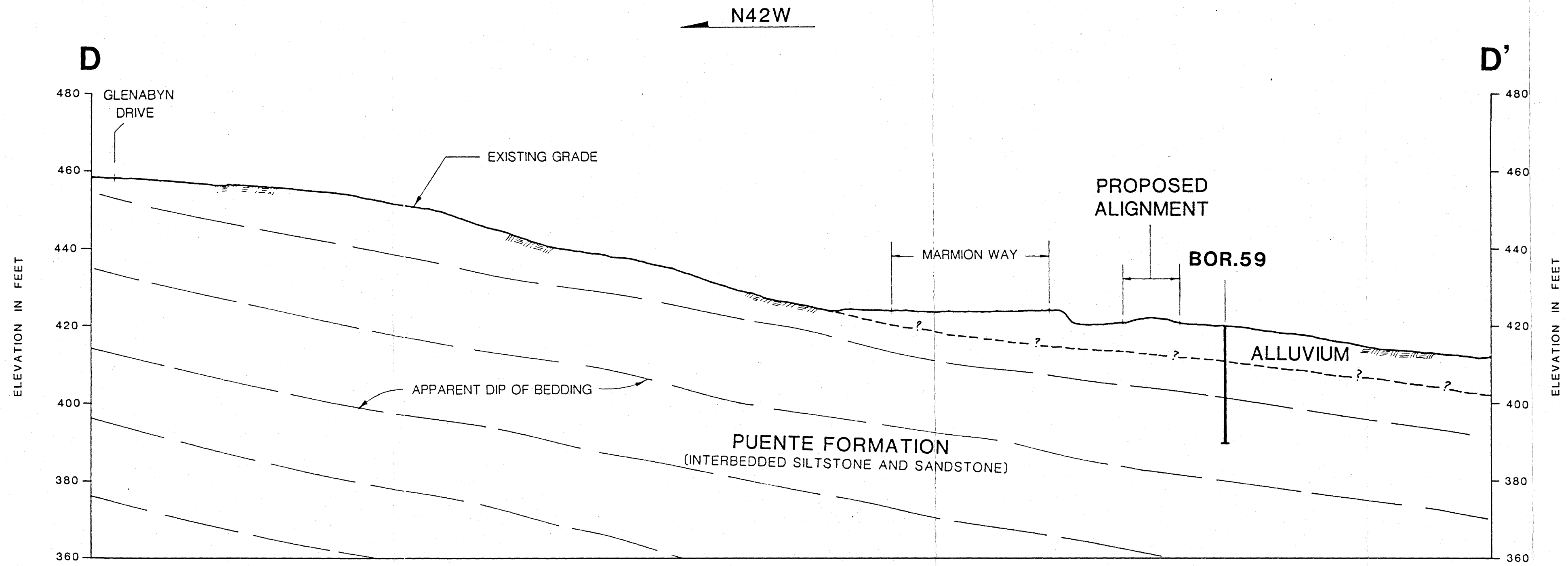
GEOLOGIC SECTION

SCALE 1"=20'

LAW/CRANDALL, INC.



JOB L1042, FILE 4 DATE 02/01/83 DR. - M.H. O.E. - CIRCD. SF/RE



NOTES:

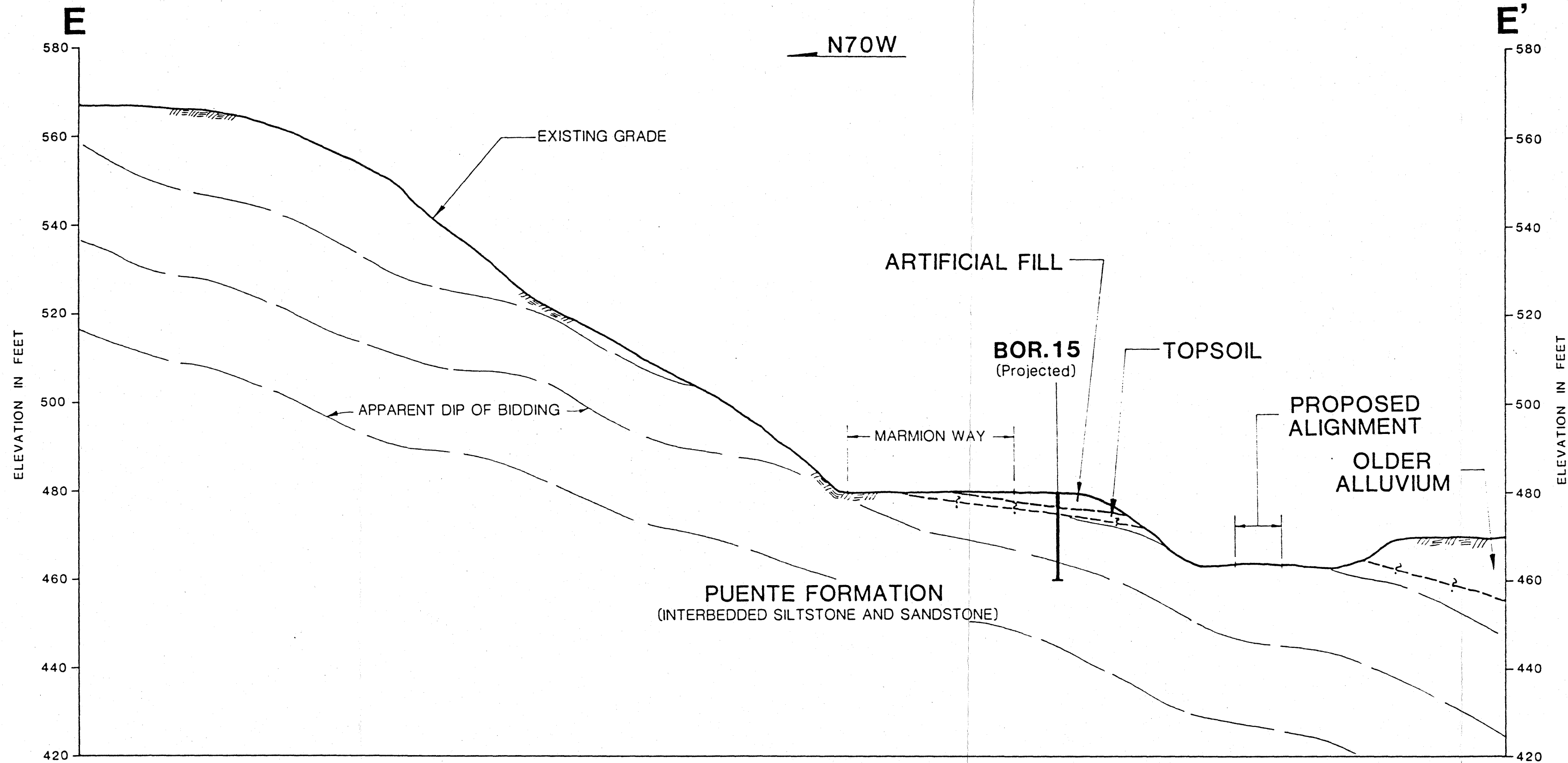
1. THE SECTION IS BASED ON GEOLOGIC CONDITION AT BORING LOCATION AND AT SURFACE EXPOSURES MAPPED DURING THE INVESTIGATION. THE GEOLOGIC CONDITIONS BETWEEN SUCH LOCATIONS HAVE BEEN INTERPOLATED. LOCALIZED VARIATIONS COULD OCCUR. THE SECTION IS INTENDED FOR DESCRIPTIVE PURPOSES ONLY.
2. SEE PLATE 1.9 FOR LOCATION OF SECTION

GEOLOGIC SECTION

SCALE 1"=20'

LAW/GRANDALL, INC.





NOTES:

1. THE SECTION IS BASED ON GEOLOGIC CONDITION AT BORING LOCATION AND AT SURFACE EXPOSURES MAPPED DURING THE INVESTIGATION. THE GEOLOGIC CONDITIONS BETWEEN SUCH LOCATIONS HAVE BEEN INTERPOLATED. LOCALIZED VARIATIONS COULD OCCUR. THE SECTION IS INTENDED FOR DESCRIPTIVE PURPOSES ONLY.
2. SEE PLATE 1.10 FOR LOCATION OF SECTION.

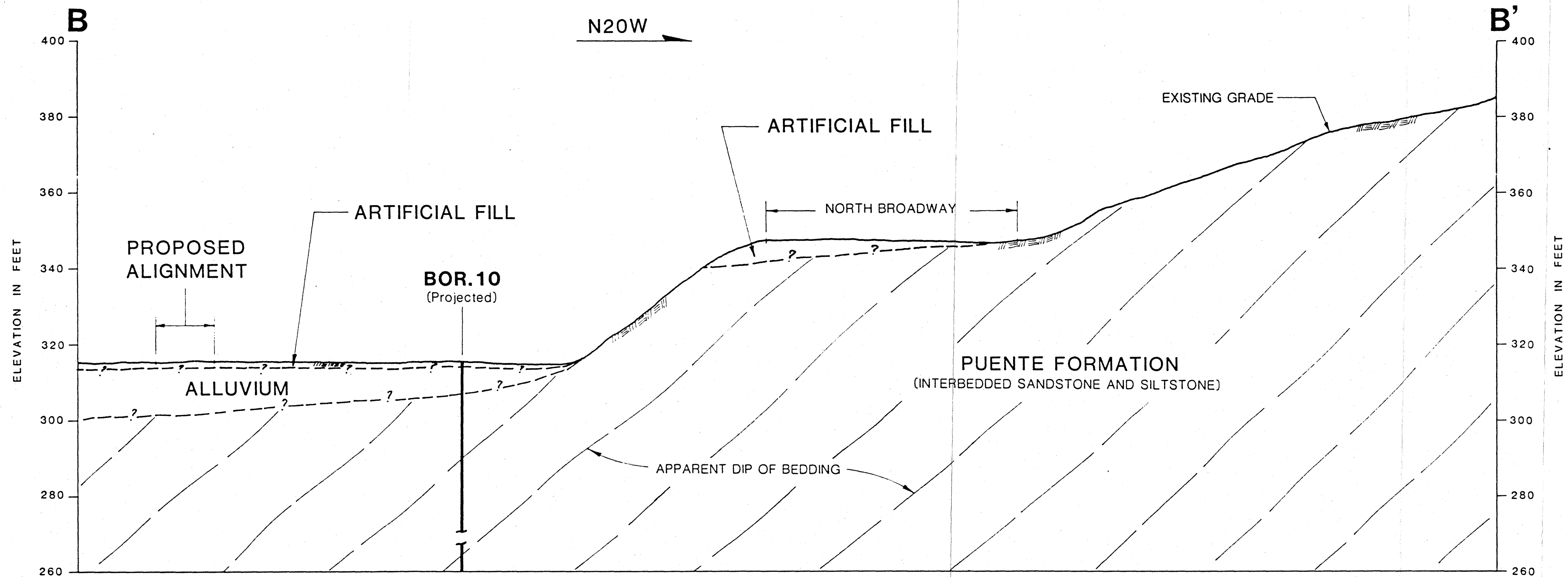
GEOLOGIC SECTION

SCALE 1"=20'

LAW/GRANDALL, INC.



JOB 45.144 DATE 06/18/13 DR. JIN O.E. SP/PA CHKD. SP/YE



NOTES:

1. THE SECTION IS BASED ON GEOLOGIC CONDITION AT BORING LOCATION AND AT SURFACE EXPOSURES MAPPED DURING THE INVESTIGATION. THE GEOLOGIC CONDITIONS BETWEEN SUCH LOCATIONS HAVE BEEN INTERPOLATED. LOCALIZED VARIATIONS COULD OCCUR. THE SECTION IS INTENDED FOR DESCRIPTIVE PURPOSES ONLY.

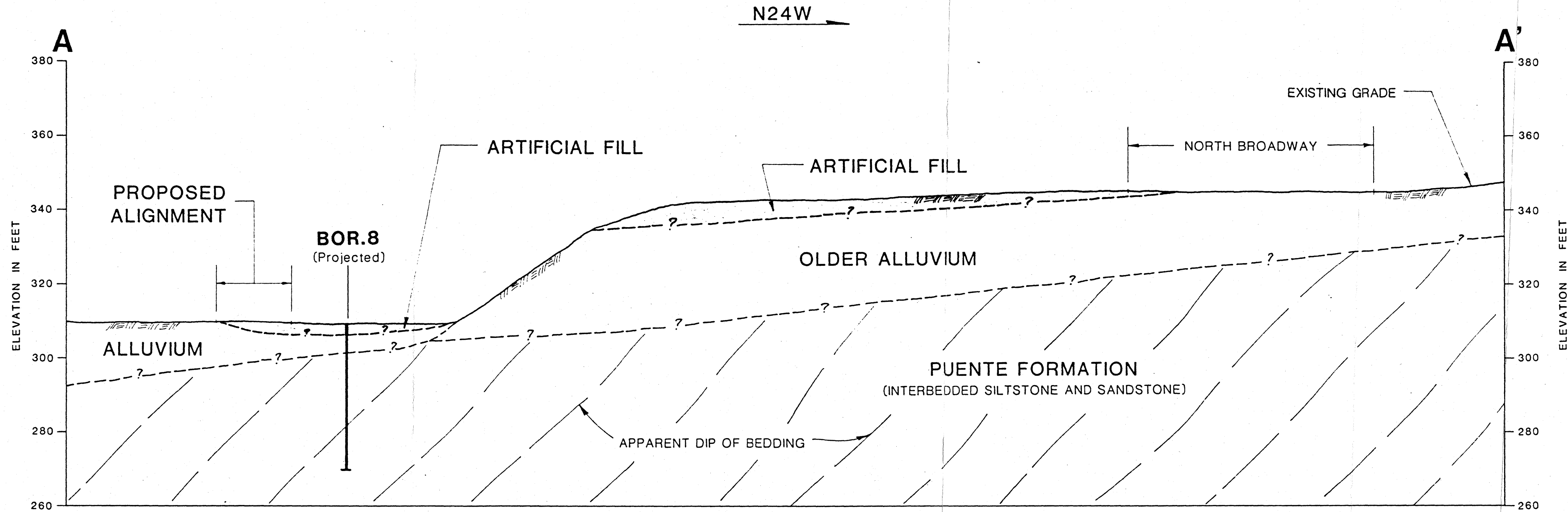
2. SEE PLATE 1.4 FOR LOCATION OF SECTION

GEOLOGIC SECTION

SCALE 1"=20'

LAW/CRANDALL, INC.





NOTES:

1. THE SECTION IS BASED ON GEOLOGIC CONDITION AT BORING LOCATION AND AT SURFACE EXPOSURES MAPPED DURING THE INVESTIGATION. THE GEOLOGIC CONDITIONS BETWEEN SUCH LOCATIONS HAVE BEEN INTERPOLATED. LOCALIZED VARIATIONS COULD OCCUR. THE SECTION IS INTENDED FOR DESCRIPTIVE PURPOSES ONLY.

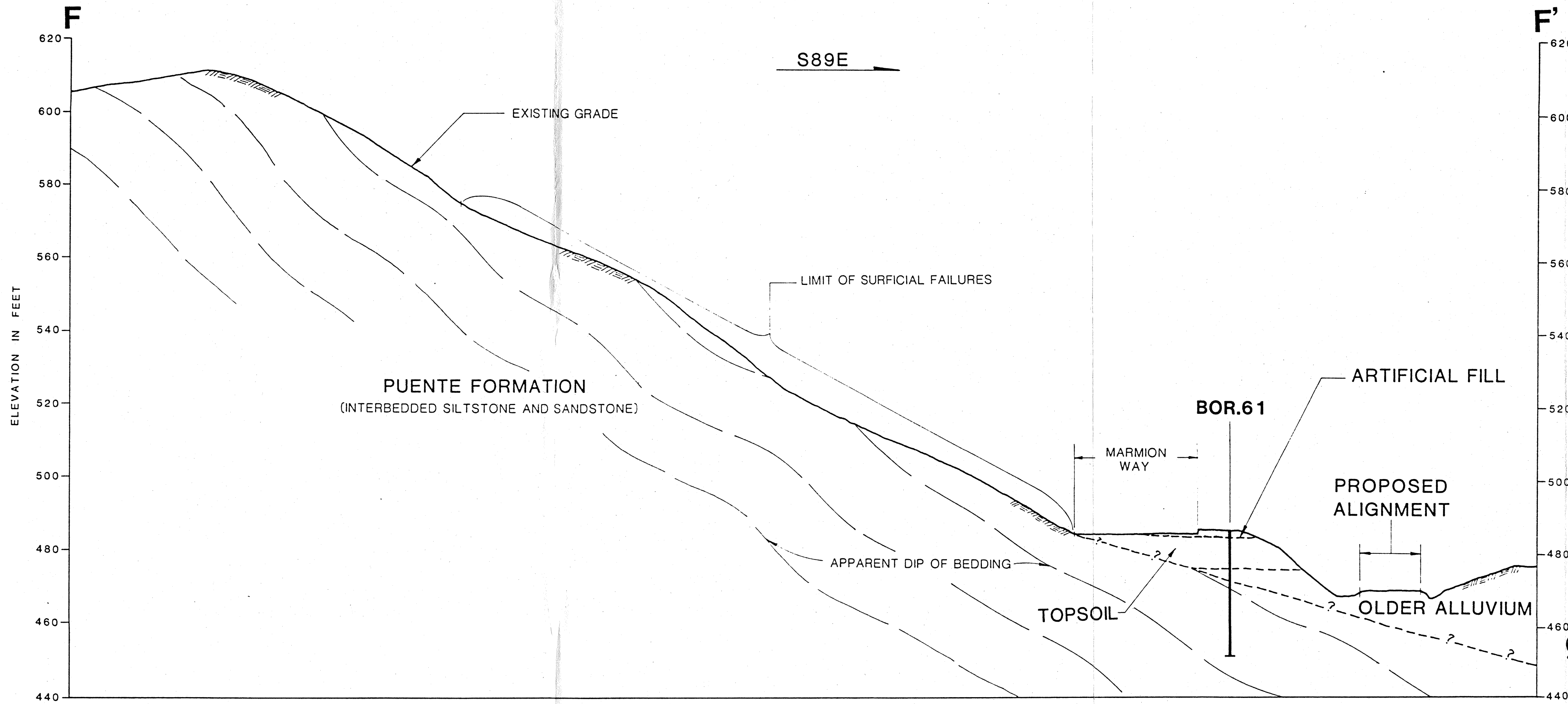
2. SEE PLATE 1.4 FOR LOCATION OF SECTION

GEOLOGIC SECTION

SCALE 1"=20'

LAW/GRANDALL, INC.

JOB 412045, HIGHWAY DATE 04/19/13 DIR. M.U. O.L. 3/1/13 CHKD. 3/



F'

NOTES:

1. THE SECTION IS BASED ON GEOLOGIC CONDITION AT BORING LOCATION AND AT SURFACE EXPOSURES MAPPED DURING THE INVESTIGATION. THE GEOLOGIC CONDITIONS BETWEEN SUCH LOCATIONS HAVE BEEN INTERPOLATED. LOCALIZED VARIATIONS COULD OCCUR. THE SECTION IS INTENDED FOR DESCRIPTIVE PURPOSES ONLY.
2. SEE PLATE 1.10 FOR LOCATION OF SECTION.

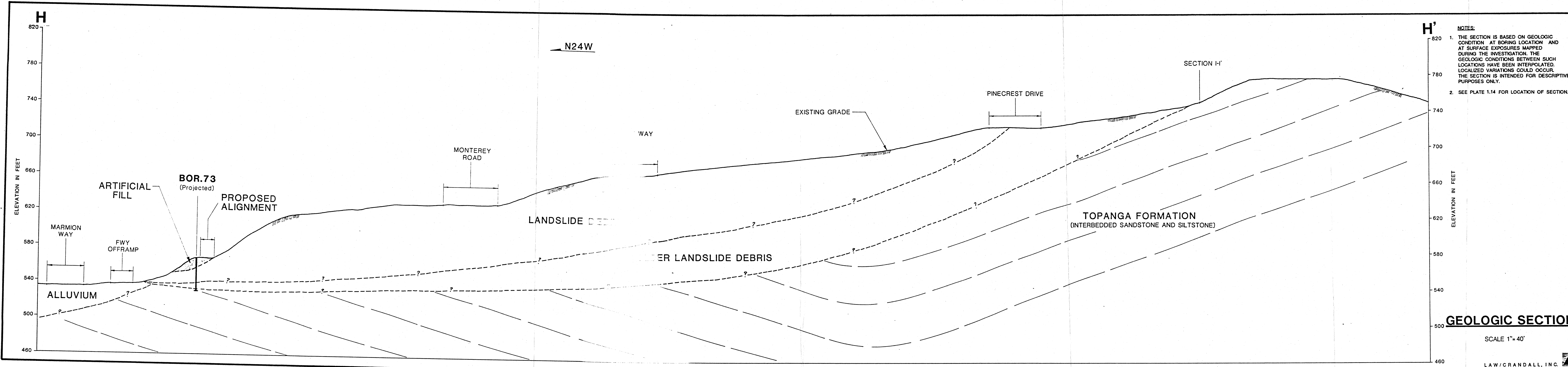
ELEVATION IN FEET

GEOLOGIC SECTION

SCALE 1" = 20'

LAW/CRANDALL, INC.

JOB NO. 04-54 DATE 2/16/83 BY J.E. 4/6/84



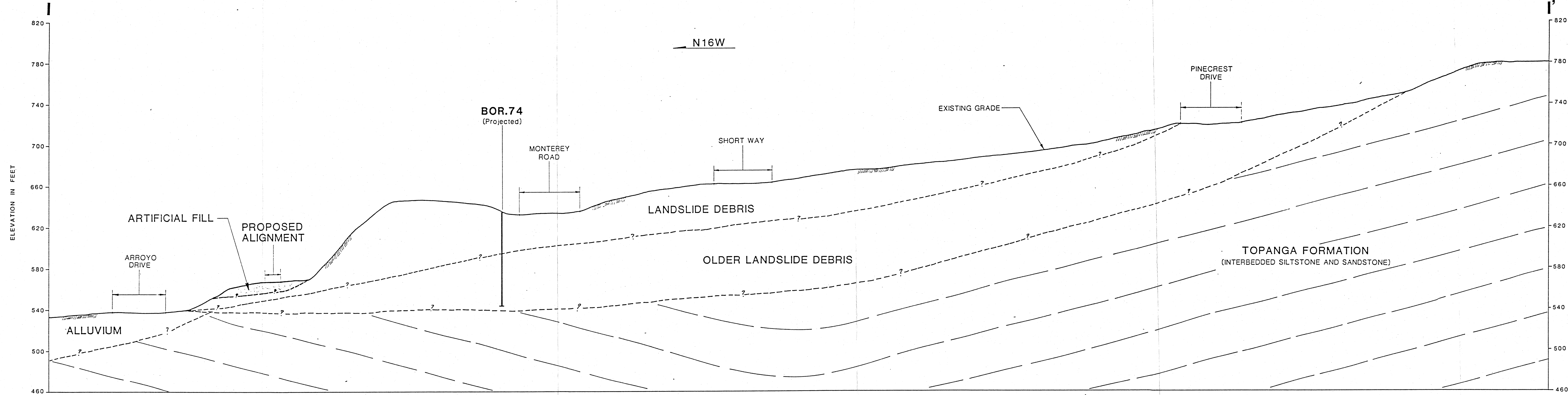
- NOTES:
1. THE SECTION IS BASED ON GEOLOGIC CONDITION AT BORING LOCATION AND AT SURFACE EXPOSURES MAPPED DURING THE INVESTIGATION. THE GEOLOGIC CONDITIONS BETWEEN SUCH LOCATIONS HAVE BEEN INTERPOLATED. LOCALIZED VARIATIONS COULD OCCUR. THE SECTION IS INTENDED FOR DESCRIPTIVE PURPOSES ONLY.
 2. SEE PLATE 1.14 FOR LOCATION OF SECTION.

GEOLOGIC SECTION

SCALE 1"=40'

LAW/CRANDALL, INC.

JOE 744 DATE 12/3/81 SHEET 1 OF 1

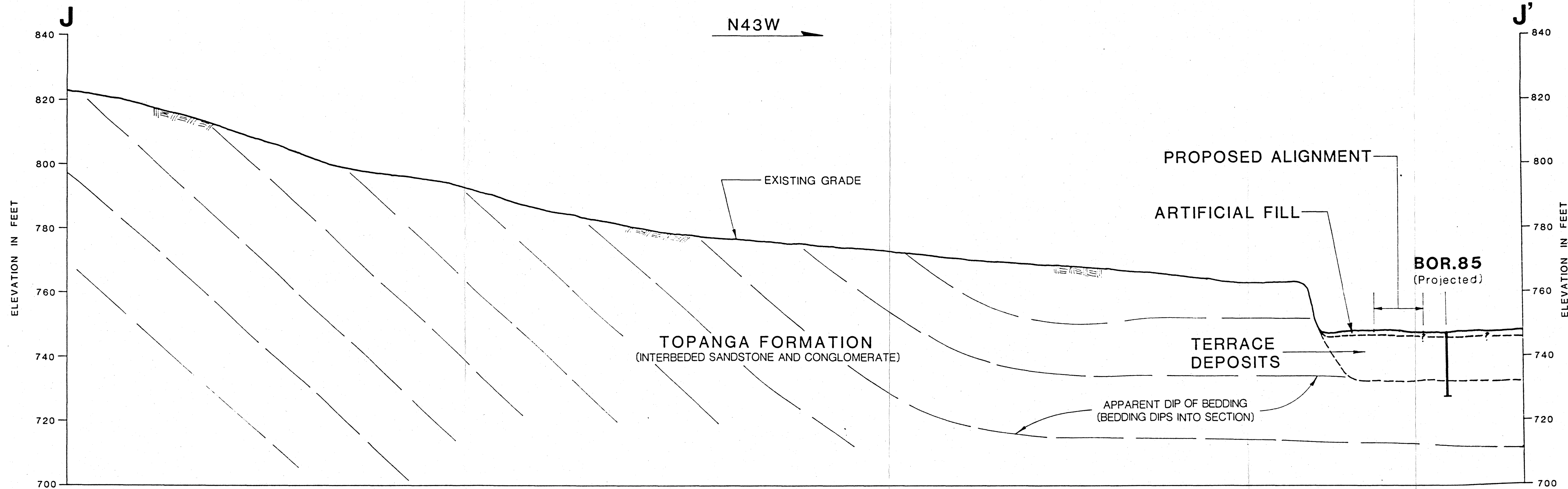


- NOTES:
1. THE SECTION IS BASED ON GEOLOGIC CONDITION AT BORING LOCATION AND AT SURFACE EXPOSURES MAPPED DURING THE INVESTIGATION. THE GEOLOGIC CONDITIONS BETWEEN SUCH LOCATIONS HAVE BEEN INTERPOLATED. LOCALIZED VARIATIONS COULD OCCUR. THE SECTION IS INTENDED FOR DESCRIPTIVE PURPOSES ONLY.
 2. SEE PLATE 1.14 FOR LOCATION OF SECTION.

GEOLOGIC SECTION

SCALE 1" = 40'

JOB L-12042 AE4 DATE 04/18/13 DR. M.N. O.L. SJ/KA CHKD. X



NOTES:

1. THE SECTION IS BASED ON GEOLOGIC CONDITION AT BORING LOCATION AND AT SURFACE EXPOSURES MAPPED DURING THE INVESTIGATION. THE GEOLOGIC CONDITIONS BETWEEN SUCH LOCATIONS HAVE BEEN INTERPOLATED. LOCALIZED VARIATIONS COULD OCCUR. THE SECTION IS INTENDED FOR DESCRIPTIVE PURPOSES ONLY.

2. SEE PLATE 1.20 FOR LOCATION OF SECTION

GEOLOGIC SECTION

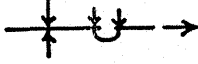
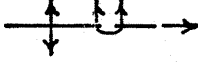
SCALE 1" = 20'

LAW/GRANDALL, INC.

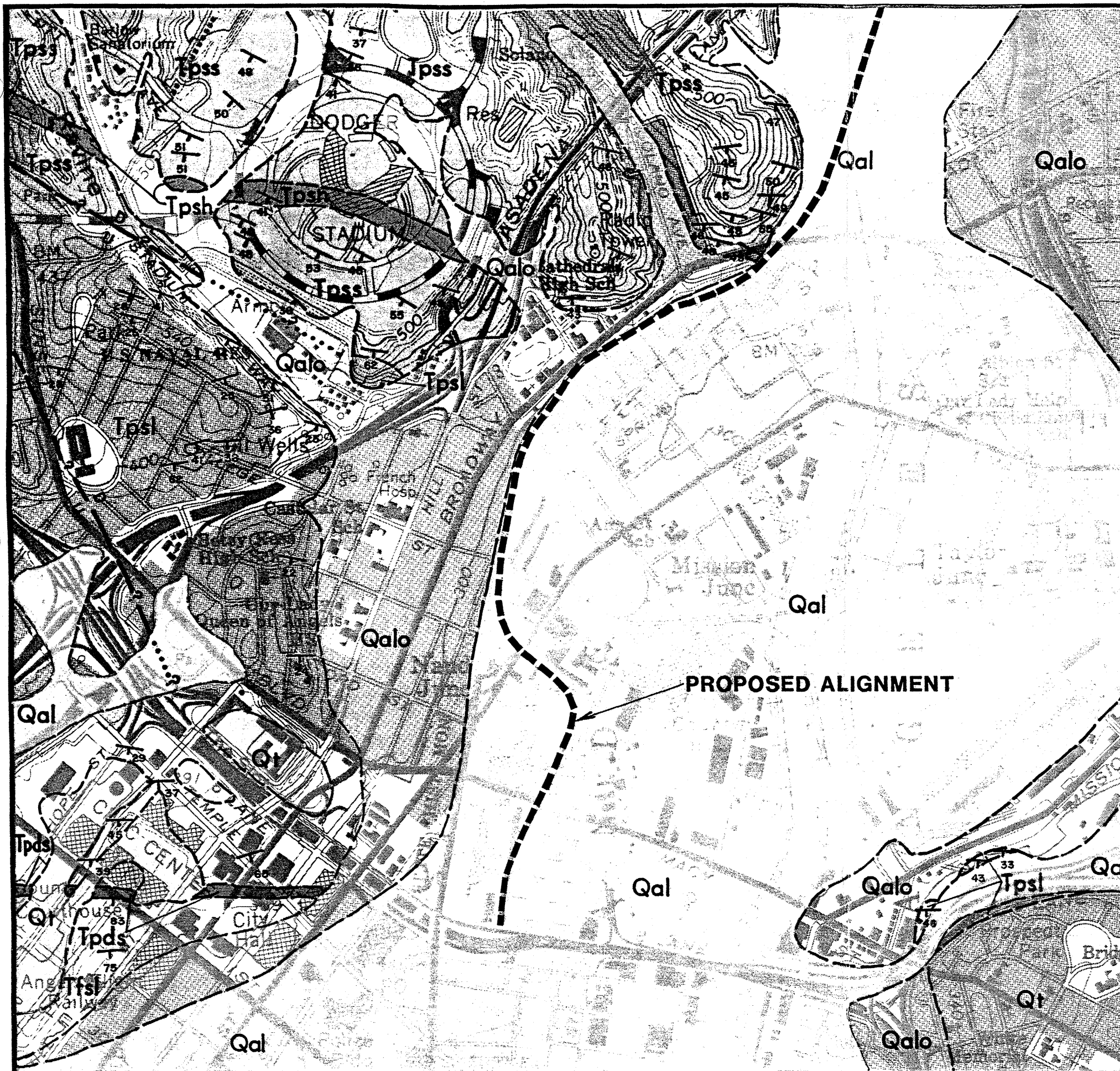


EXPLANATION:

- af** ARTIFICIAL FILL
- Qal** ALLUVIUM
- Qls** LANDSLIDE DEBRIS
- Qalo** OLDER ALLUVIUM
- Qt** TERRACE DEPOSITS
- Tfcg** **Tfss** FERNANDO FORMATION, Tfcg: conglomerate, Tfss: sandstone
- Tpsl** **Tpsh** PUENTE FORMATION, Tpsl: siltstone, Tpsh: shale, Tpbs: diatomaceous shale, Tpss: sandstone
- Ttss** **Ttcg** TOPANGA FORMATION, Ttss: sandstone, Ttcg: conglomerate, **Ttsl** Ttsl: siltstone

-  GEOLOGIC CONTACT, dashed where approximately located queried where inferred
-  FAULT, dashed where approximately located, queried where inferred or hypothetical, dotted where concealed, U: up thrown side, D: down thrown side
-  SYNCLINE, axial planes of folds; arrows indicate dip direction on flanks and plunge; dashed where approximately located
-  ANTICLINE, axial planes of folds; arrows indicate dip direction on flanks and plunge; dashed where approximately located
-  STRIKE AND DIP OF BEDDING
-  STRIKE AND DIP OF OVERTURNED BEDS
-  HORIZONTAL BEDS
-  STRIKE AND DIP OF SHEAR

**LOCAL
GEOLOGIC MAP
EXPLANATION**



REFERENCES:

U.S.G.S. 7.5 MINUTE LOS ANGELES QUADRANGLE
1966, PHOTOREVISED 1981.

GEOLOGIC MAP OF THE ELYSIAN PARK-REPETTO
HILLS AREA, LOS ANGELES COUNTY, CALIFORNIA.
LAMAR 1970 C.D.M.G. SPECIAL REPORT 101.

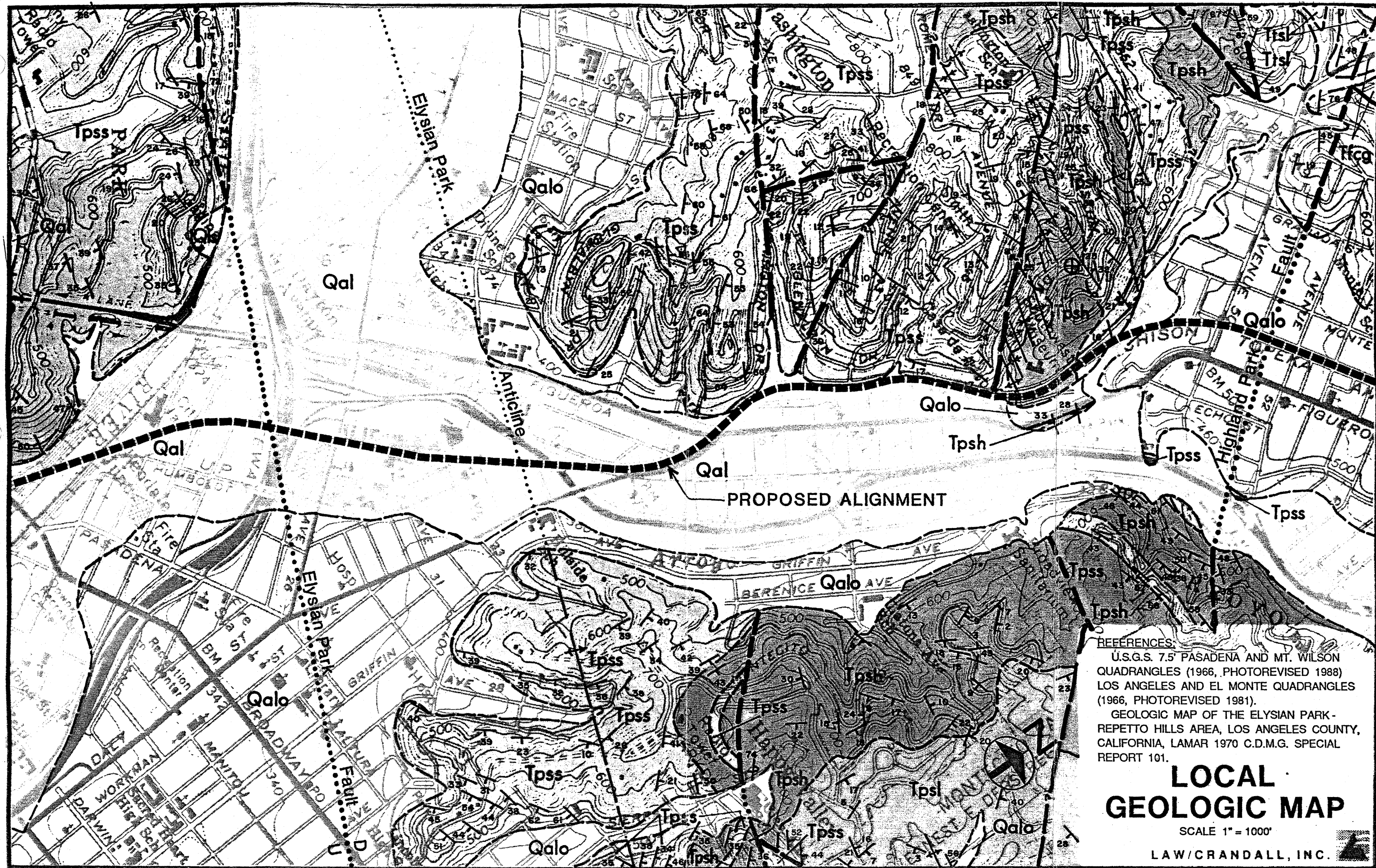


LOCAL GEOLOGIC MAP

SCALE 1"= 1000'

LAW/CRANDALL, INC.



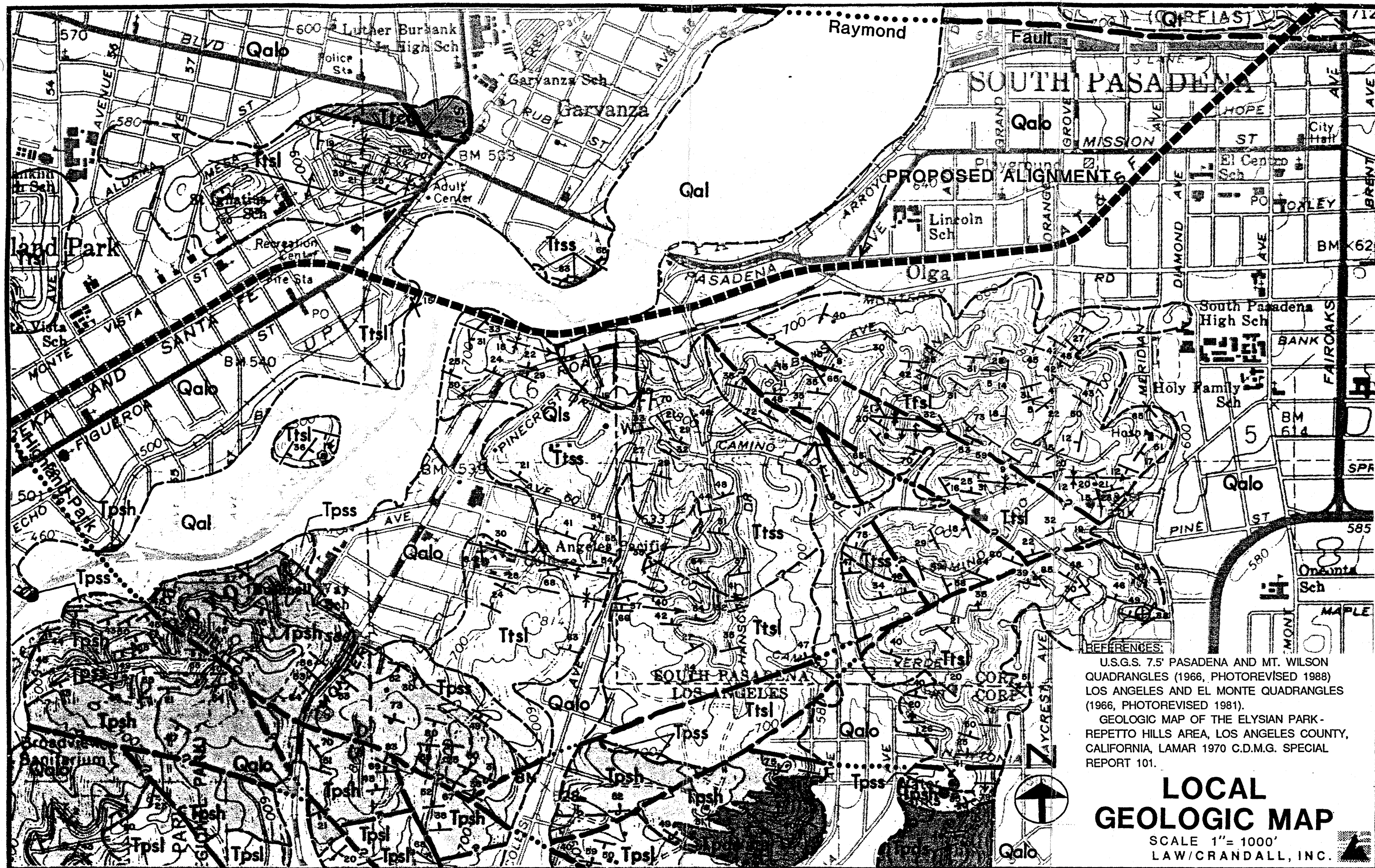


REFERENCES:
U.S.G.S. 7.5' PASADENA AND MT. WILSON
QUADRANGLES (1966, PHOTOREVISED 1988)
LOS ANGELES AND EL MONTE QUADRANGLES
(1966, PHOTOREVISED 1981).
GEOLOGIC MAP OF THE ELYSIAN PARK-
REPETTO HILLS AREA, LOS ANGELES COUNTY,
CALIFORNIA, LAMAR 1970 C.D.M.G. SPECIAL
REPORT 101.

LOCAL GEOLOGIC MAP

SCALE 1" = 1000'

LAW/CRANDALL, INC.



REFERENCES:
 U.S.G.S. 7.5' PASADENA AND MT. WILSON
 QUADRANGLES (1966, PHOTOREVISED 1988)
 LOS ANGELES AND EL MONTE QUADRANGLES
 (1966, PHOTOREVISED 1981).
 GEOLOGIC MAP OF THE ELYSIAN PARK-
 REPETTO HILLS AREA, LOS ANGELES COUNTY,
 CALIFORNIA, LAMAR 1970 C.D.M.G. SPECIAL
 REPORT 101.

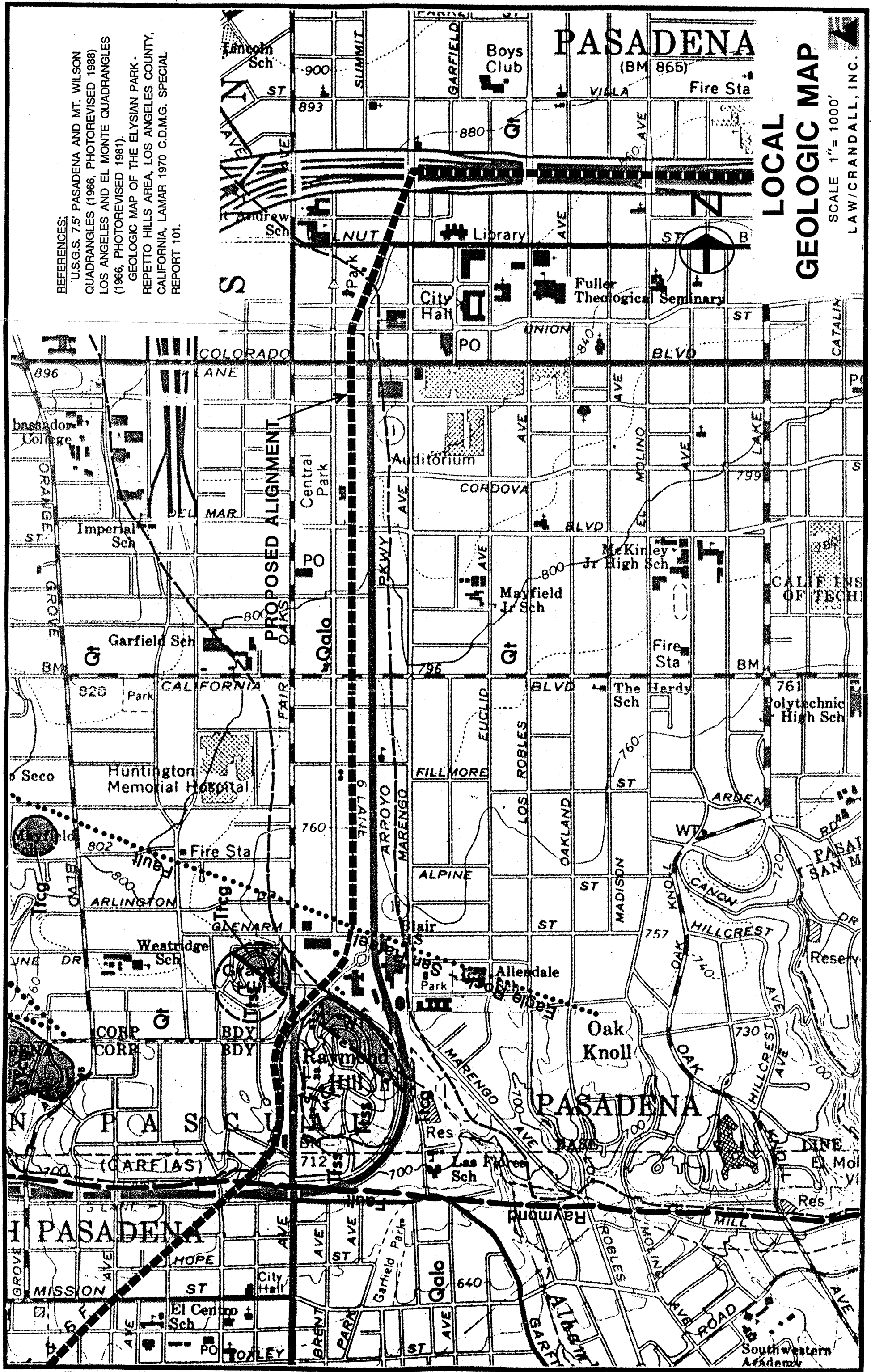
**LOCAL
GEOLOGIC MAP**
 SCALE 1" = 1000'
 LAW/CRANDALL, INC.

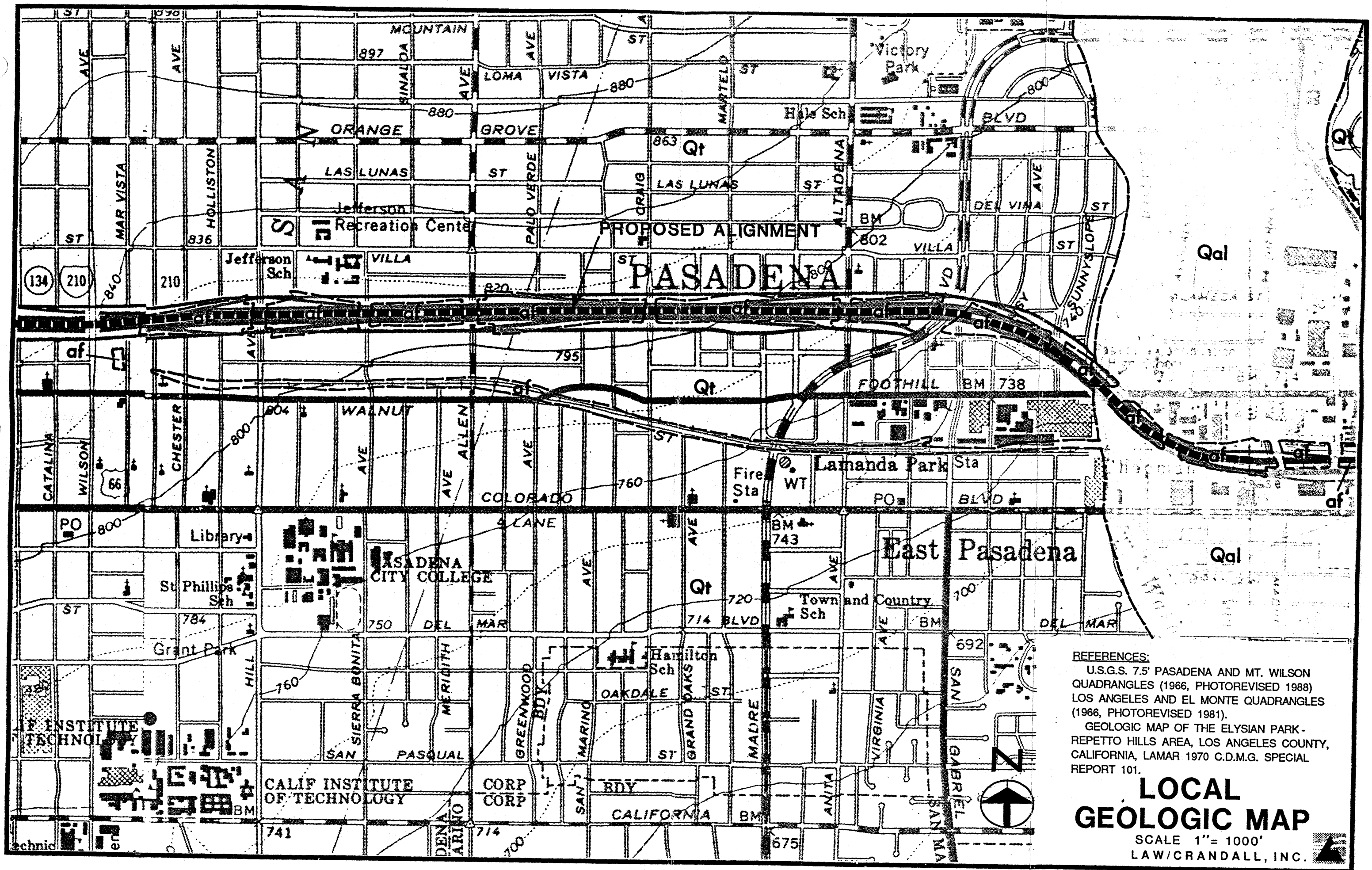
REFERENCES:

U.S.G.S. 7.5' PASADENA AND MT. WILSON
QUADRANGLES (1966, PHOTOREVISED 1988)
LOS ANGELES AND EL MONTE QUADRANGLES
(1966, PHOTOREVISED 1981).
GEOLOGIC MAP OF THE ELYSIAN PARK-
REPETTO HILLS AREA, LOS ANGELES COUNTY,
CALIFORNIA, LAMAR 1970 C.D.M.G. SPECIAL
REPORT 101.

LOCAL
GEOLOGIC MAP

SCALE 1" = 1000'
LAW/CRANDALL, INC.

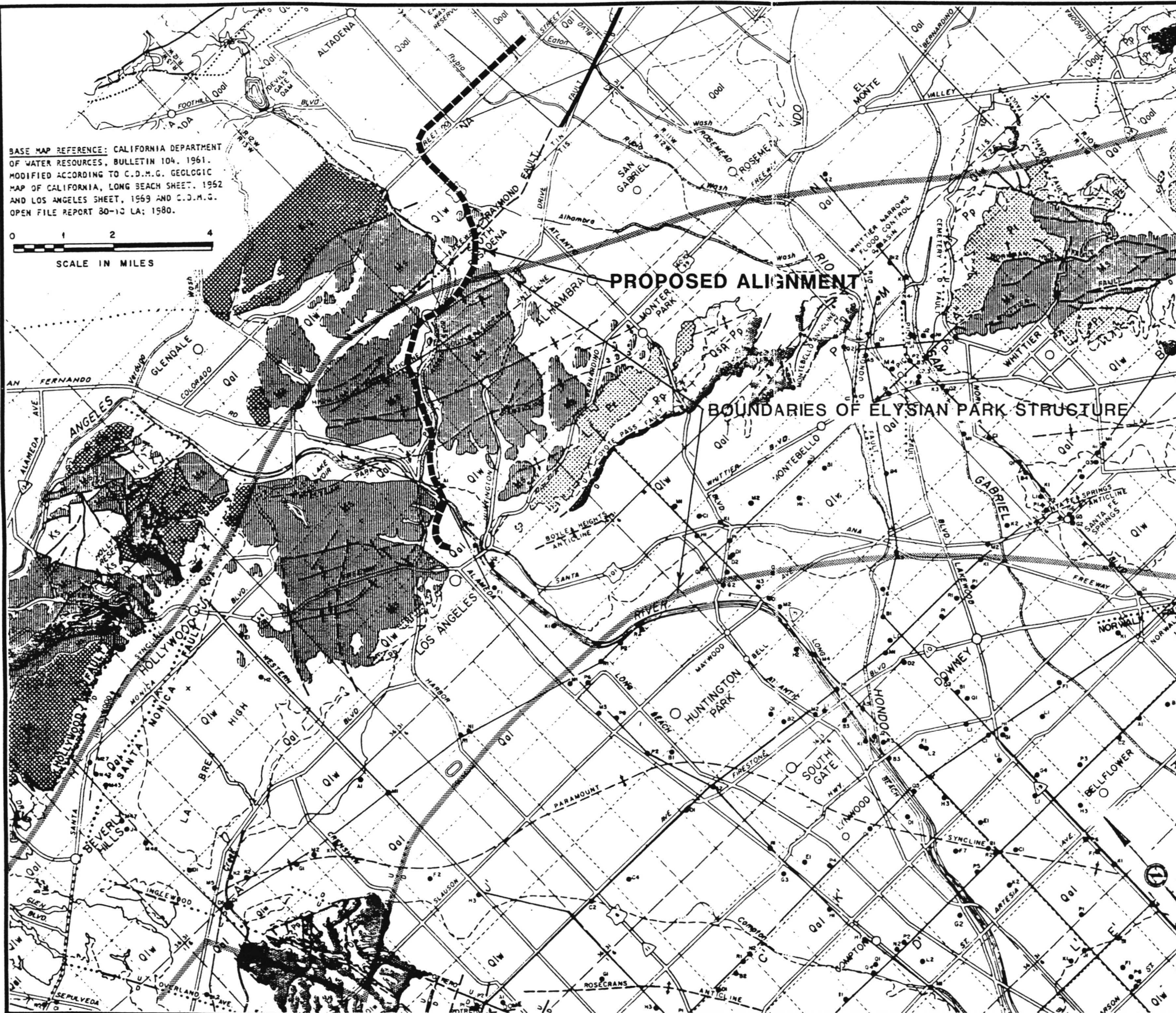




REFERENCES:
 U.S.G.S. 7.5' PASADENA AND MT. WILSON
 QUADRANGLES (1966, PHOTOREVISED 1988)
 LOS ANGELES AND EL MONTE QUADRANGLES
 (1966, PHOTOREVISED 1981).
 GEOLOGIC MAP OF THE ELYSIAN PARK-
 REPETTO HILLS AREA, LOS ANGELES COUNTY,
 CALIFORNIA, LAMAR 1970 C.D.M.G. SPECIAL
 REPORT 101.

**LOCAL
 GEOLOGIC MAP**
 SCALE 1"= 1000'
 LAW/CRANDALL, INC.

BASE MAP REFERENCE: CALIFORNIA DEPARTMENT OF WATER RESOURCES, BULLETIN 104, 1961. MODIFIED ACCORDING TO C.D.W.R. GEOLOGIC MAP OF CALIFORNIA, LONG BEACH SHEET, 1952 AND LOS ANGELES SHEET, 1969 AND C.D.W.R. OPEN FILE REPORT 80-10 LA, 1980.



LEGEND

SEDIMENTARY ROCKS

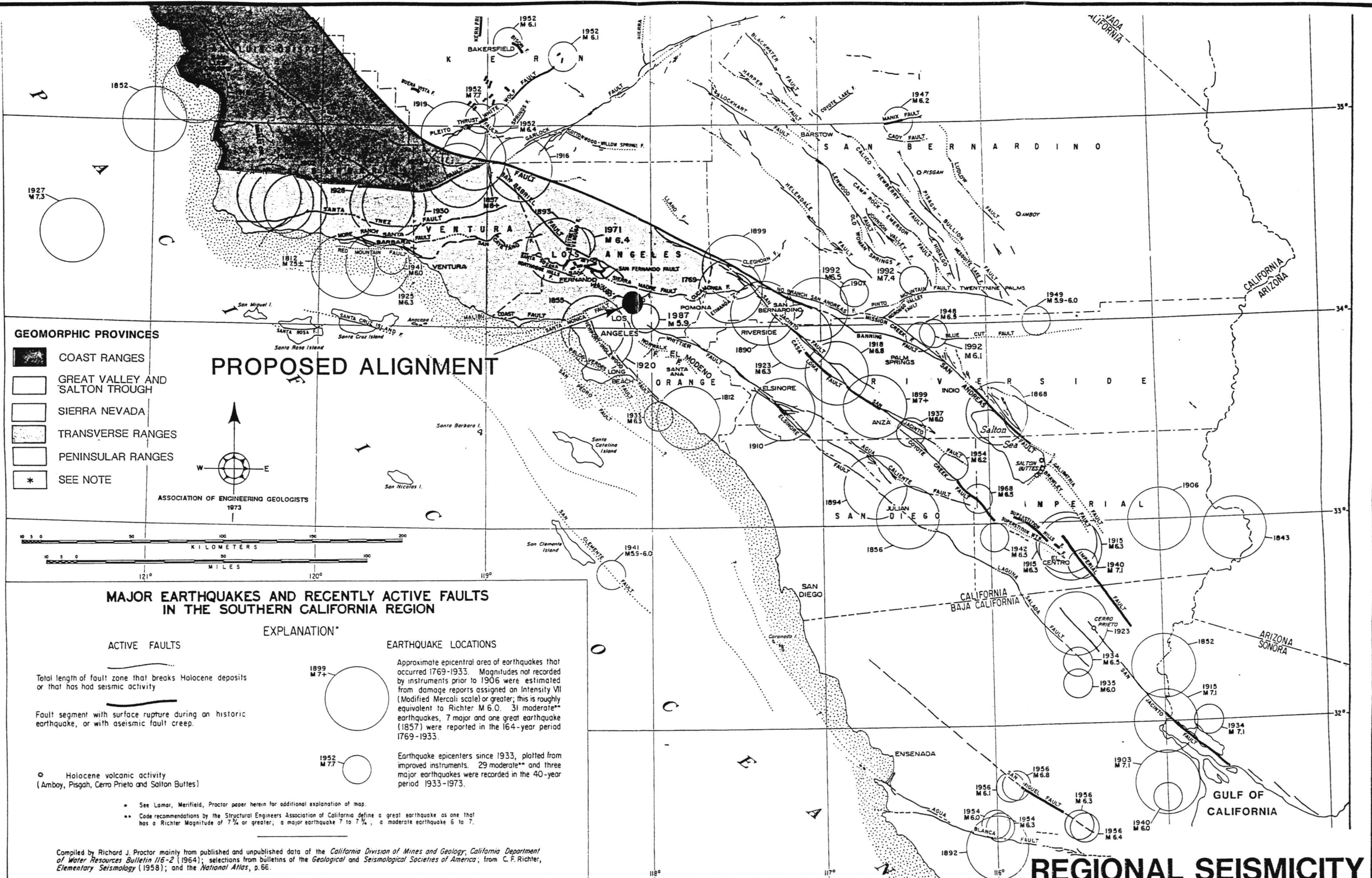
RECENT	Qal	ALLUVIUM GRAVEL, SAND, SILT, AND CLAY
	Qst	ACTIVE DUNE SAND WHITE OR BROWNISH, WELL SORTED SAND
UPPER	Qlw	OLDER DUNE SAND FINE TO MEDIUM SAND WITH SILT, AND GRAVEL LENSES
PLEISTOCENE		LAKEWOOD FORMATION (INCLUDES "TERRACE DEPOSITS", "PALOS VERDES SAND", AND "UNNAMED UPPER PLEISTOCENE DEPOSITS") MARINE AND CONTINENTAL GRAVEL, SAND, SANDY SILT, SILT, AND CLAY WITH SHALE, PEBBLES
LOWER		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
PLIOCENE	Pp	PICO FORMATION MARINE SAND, SILT, AND CLAY INTERBEDDED WITH GRAVEL
		REPETTO FORMATION MARINE SILTSTONE WITH LAYERS OF SANDSTONE AND CONGLOMERATE
		(SANTA MONICA MOUNTAINS) MODELO FORMATION MARINE CONGLOMERATE, SANDSTONE, AND SHALE TOPANGA FORMATION MARINE CONGLOMERATE, SANDSTONE, AND SHALE
MIocene		(PALOS VERDES HILLS) MONTEREY FORMATION MUDSTONE, DIATOMITE, AND SHALE EELYSIAN HILLS, REPETTO HILLS, AND PUENTE HILLS PUENTE FORMATION MARINE SILTSTONE, SANDSTONE, SHALE, CONGLOMERATE, LIMESTONE, AND TUFF
OLIGOCENE(?)		VAQUEROS AND SESPE FORMATIONS CONTINENTAL RED CONGLOMERATE AND SANDSTONE
EOCENE		MARTINEZ FORMATION MARINE CONGLOMERATE, SANDSTONE, SANDY SHALE, AND SHALE
PALEOCENE(?)	E-K	UNDIVIDED MARTINEZ AND CHICO FORMATIONS
UPPER	Ks	CHICO FORMATION UPPER MEMBER - HARD CONGLOMERATE, SANDSTONE, AND SHALE LOWER MEMBER - RED CONGLOMERATE AND SANDSTONE

IGNEOUS AND METAMORPHIC ROCKS

MIocene		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)
UPPER		(SANTA MONICA MOUNTAINS) INTRUSIVES OF GRANITE AND GRANODIORITE
	J	(PALOS VERDES HILLS) CATALINA SCHIST COMPARED WITH FRANCISCAN FORMATION OF THE COAST RANGES. VARIED TYPES OF SCHISTOSE ROCKS
	R	SANTA MONICA SLATE GRAY TO BLACK SLATE, SPOTTED SLATE, UNCA SCHIST WITH QUARTZ VEINS
		FAULT (DASHED WHERE APPROXIMATELY LOCATED, U-UPTHROWN SIDE; D-DOWNTOWN SIDE)
		CONCEALED FAULT
		ANTICLINE (DASHED WHERE APPROXIMATELY LOCATED)
		SYNCLINE (DASHED WHERE APPROXIMATELY LOCATED)
		CONTACT (DASHED WHERE APPROXIMATELY LOCATED)
	AS	WELLS USED IN PREPARATION OF GEOLOGIC SECTIONS.
	A-A'	LINE LOCATION OF GEOLOGIC SECTIONS SHOWN ON PLATES 6A THROUGH 6G.

REGIONAL GEOLOGY

LAW/CRANDALL, INC.



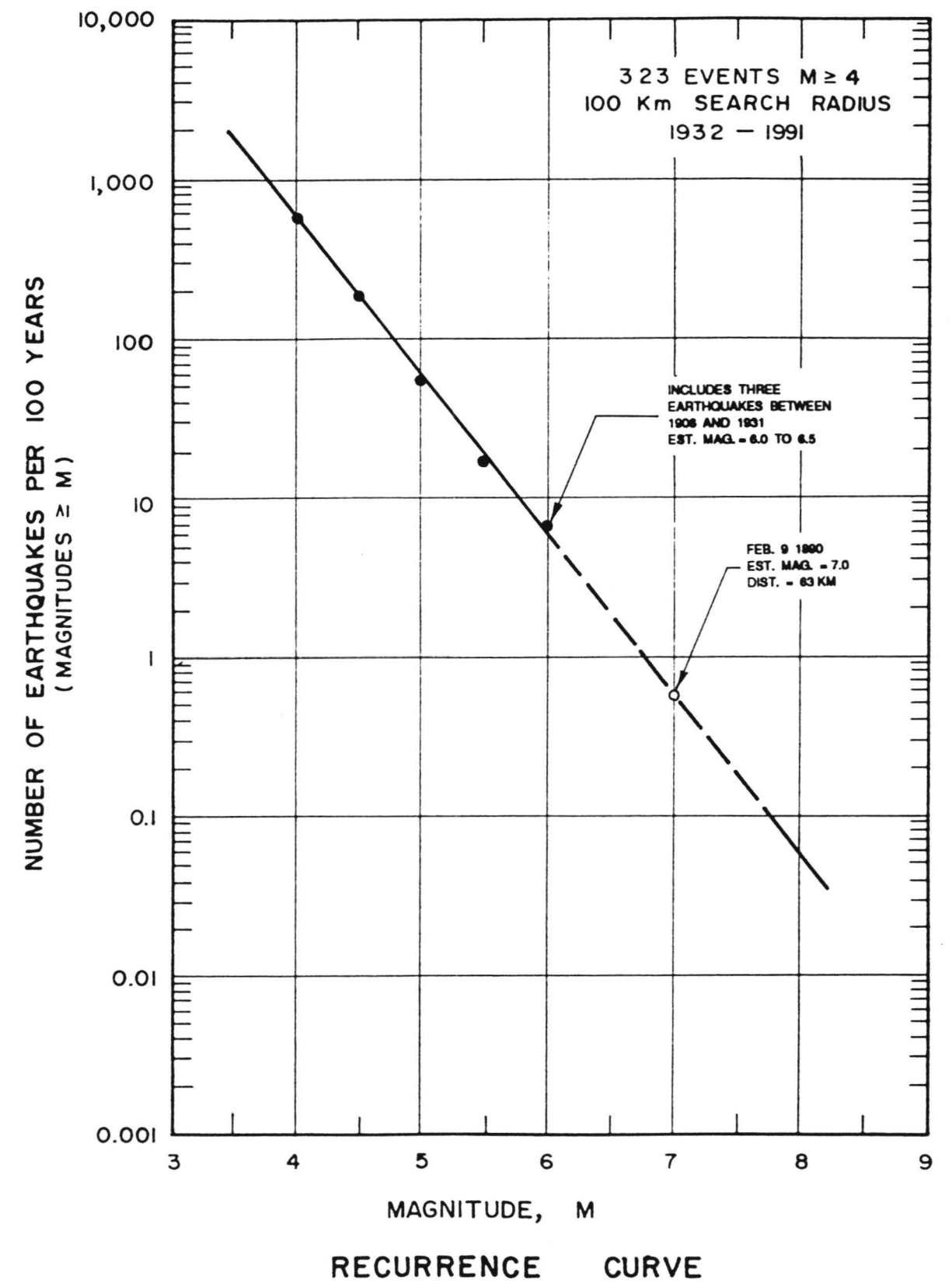
REGIONAL SEISMICITY

(GEOMORPHIC PROVINCES INCLUDED)

LAW/CRANDALL, INC.



JOB L92045.45.4 DATE 2/24/93 W.P. M.N. _____ CHKD _____

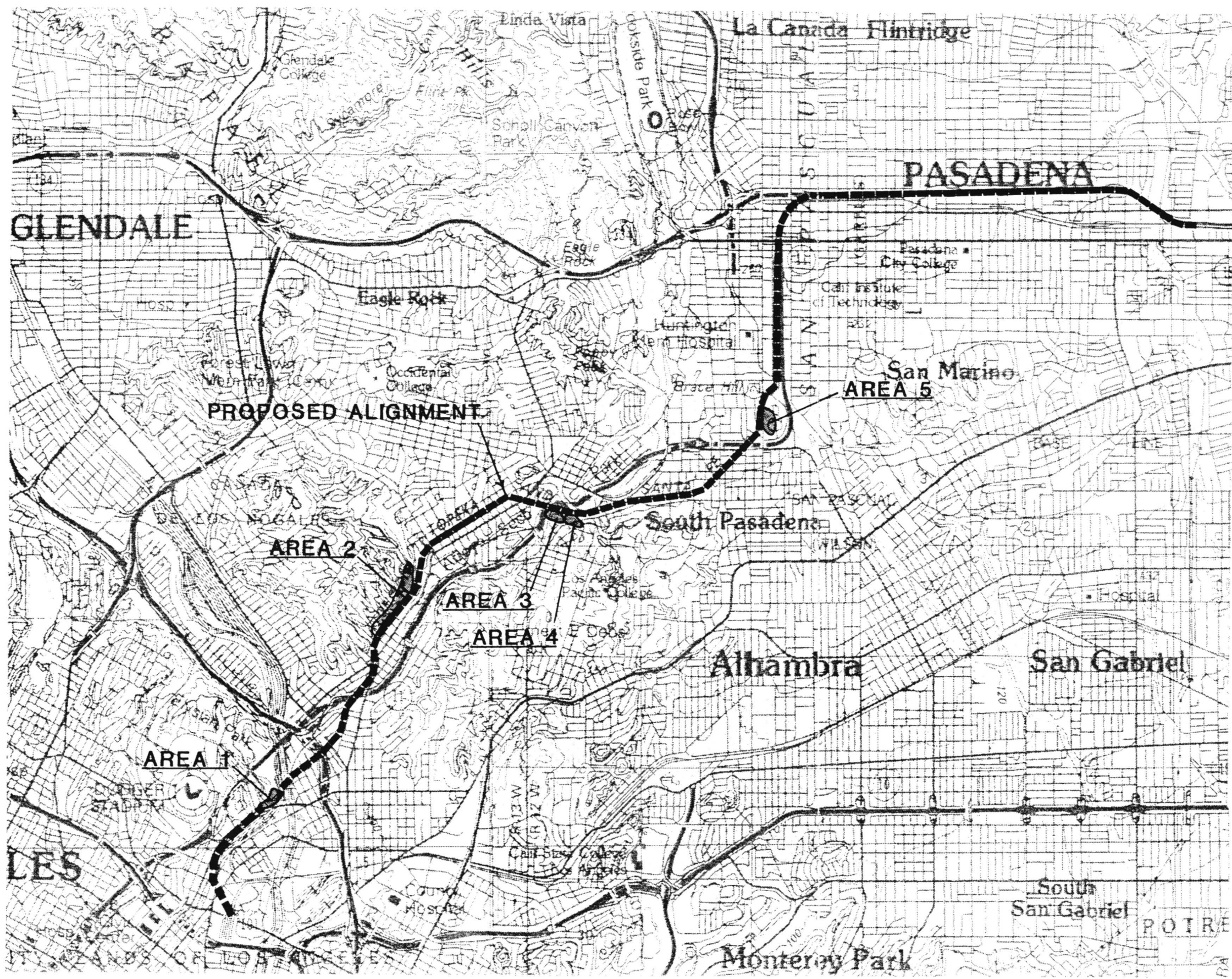


O REPRESENTS SINGLE EVENT, AND THEREFORE
HAS BEEN DISCOUNTED IN PREDICTION.

LAW/CRANDALL, INC.



JOB L92045.AE-4 DATE 02/24/93 DR. JULIANA O.E. GAK CHKD. P1



REFERENCES:

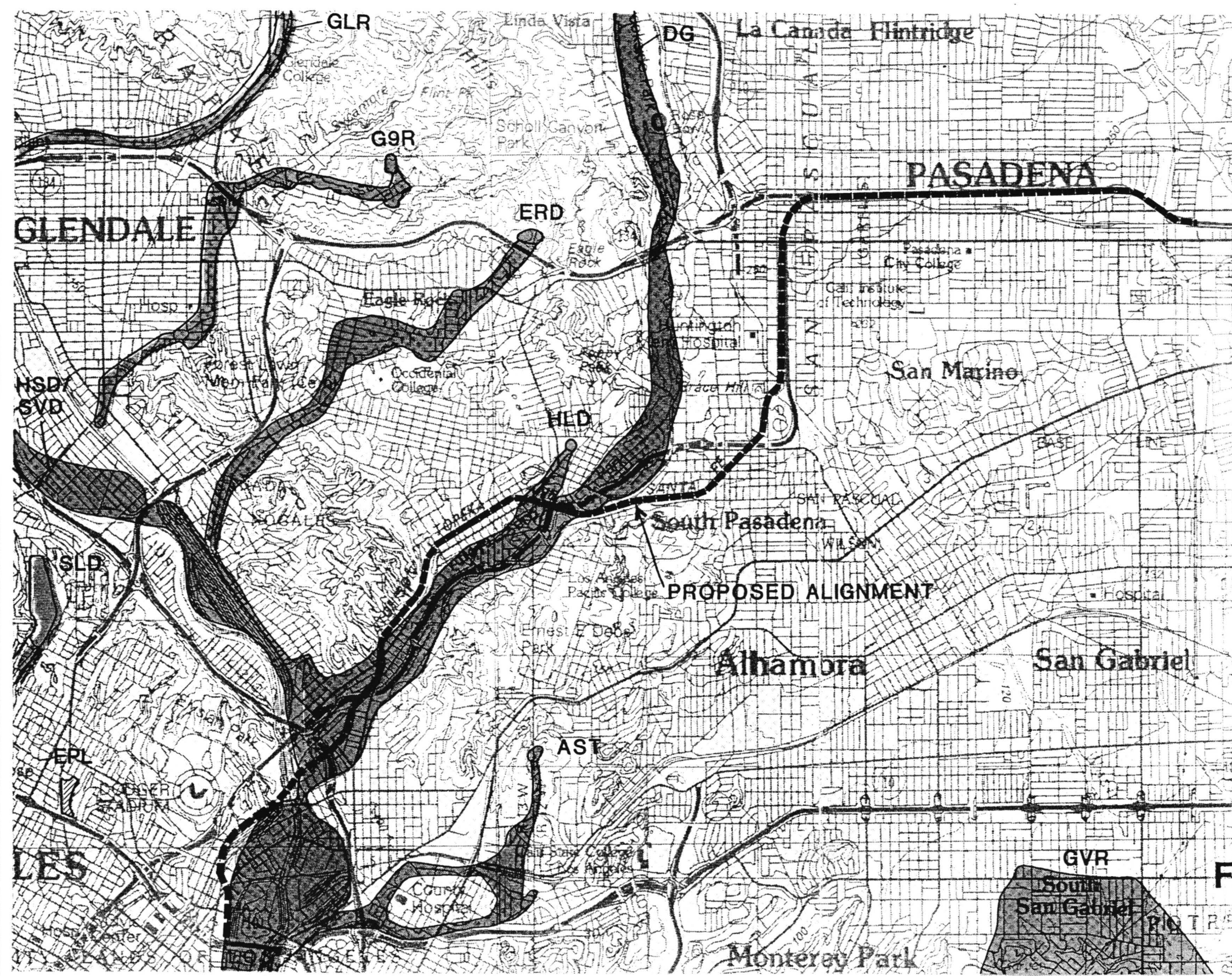
LOS ANGELES COUNTY GENERAL PLAN, DRAFT REVISION 1990, BY LEIGHTON & ASSOCIATES WITH SEDWAY COOKE ASSOCIATES.

POTENTIAL SLOPE STABILITY HAZARDS MAP

SCALE 1" = 5000'

LAW/GRANDALL, INC. 

JOB L92045 AE-4 DATE 02/24/93 DR. JULIANA O.E. GAK CHKD. P2



EXPLANATION:

 DAM INUNDATION HAZARD AREA

RESERVOIRS:

- AST - Ascot Reservoir
- DG - Devils Gate Reservoir
- ERD - Eagle Rock Dam
- GLR - Glorietta East Reservoir
- G9R - Glenoaks 968 Reservoir
- GVR - Garvey Reservoir
- HSD/SVD - Hansen Dam/Sepulveda Basin
- HLD - Highland Reservoir
- SLD - Silver Lake Dam

 FLOOD ZONE
EPL - Echo Park Lake

REFERENCES:

LOS ANGELES COUNTY GENERAL PLAN, DRAFT REVISION 1990, BY LEIGHTON & ASSOCIATES WITH SEDWAY COOKE ASSOCIATES.

FEDERAL INSURANCE ADMINISTRATION, 1988.

FLOOD AND INUNDATION HAZARDS MAP

SCALE 1" = 5000'