

City of San Jose - PBCE – Planning Division - Imaging Index Cover Sheet

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**APPENDIX C**  
**AIR QUALITY METHODS & ASSUMPTIONS**

## APPENDIX C

### AIR QUALITY METHODOLOGY AND ASSUMPTIONS

#### CALINE-4 MODELING

The CALINE-4 model is a fourth-generation line source air quality model that is based on the Gaussian diffusion equation and employs a mixing zone concept to characterize pollutant dispersion over the roadway. Given source strength, meteorology, site geometry and site characteristics, the model predicts pollutant concentrations for receptors located within 150 meters of the roadway. The CALINE-4 model allows roadways to be broken into multiple links that can vary in traffic volume, emission rates, height, width, etc.

A screening-level form of the CALINE-4 program was used to predict concentrations.<sup>1</sup> Normalized concentrations for each roadway size (two lanes, four lanes, etc.) are adjusted for the two-way traffic volume and emission factor. Calculations were made for a receptor at a corner of the intersection, located at the curb. Emission factors were derived from the California Air Resources Board EMFAC7-G computer program based on a 2003 and 2010 vehicle mix.

The screening form of the CALINE-4 model calculates the local contribution of nearby roads to the total concentration. The other contribution is the background level attributed to more distant traffic. The 1-hour background level in 2003 was taken as 10.1 ppm and the 8-hour background concentration was taken as 6.0 ppm. The 1-hour background level in 2010 was taken as 8.7 ppm and the 8-hour background concentration was taken as 5.2 ppm. These backgrounds were estimated using isopleth maps and correction factors developed by the Bay Area Air Quality Management District.

Eight-hour concentrations were obtained from the 1-hour output of the CALINE-4 model using a persistence factor of 0.7.

#### NEW VEHICLE TRAVEL EMISSIONS

Estimates of regional emissions generated by project traffic were made using a program called URBEMIS-2001.<sup>2</sup> URBEMIS-2001 is a program that estimates the emissions that result from various land use development projects. Land use project can include residential uses such as single-family dwelling units, apartments and condominiums, and nonresidential uses such as shopping centers, office buildings, and industrial parks. URBEMIS-2001 contains default values for much of the information needed to calculate emissions. However, project-specific, user-supplied information can also be used when it is available.

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<sup>1</sup> Bay Area Air Quality Management District, BAAQMD CEQA Guidelines, 1996.

<sup>2</sup> San Joaquin Valley Unified Air Pollution Control District, URBEMIS for Windows Computer Program User's Guide, October 2000.

Inputs to the URBEMIS-2001 program include trip generation rates, vehicle mix, average trip length by trip type and average speed. Average trip lengths and vehicle mixes for the Bay Area were used. Average speed for all types of trips was assumed to be 30 mph.

The URBEMIS-2001 run assumed summertime conditions with an ambient temperature of 85 degrees F.

The analysis was carried out assuming project buildout would occur by the year 2004.

**APPENDIX D**  
**GEOTECHNICAL INVESTIGATION**

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**GEOTECHNICAL INVESTIGATION  
NORTH MARKET AND WEST JULIAN SITE  
San Jose, California**

**Legacy Partners  
Foster City, California**

**5 February 2001  
Project No 2737 02**

# **Treadwell&Rollo**

5 February 2001  
Project 2737.02

Mr. Cy Colburn  
Legacy Partners  
4000 E. Third Avenue, 6<sup>th</sup> Floor  
Foster City, California 94404

Subject: Preliminary Geotechnical Investigation  
North Market and West Julian Site  
San Jose, California

Dear Mr. Colburn:

Enclosed are three copies of our preliminary geotechnical report for the proposed North Market and West Julian project site (West Julian Revitalization site) in San Jose, California. The site is east of Route 87 (Guadalupe Parkway), south of the Union Pacific Railroad line, north of West Saint James Street, and west of North Market Street. The work described in this report is in fulfillment of our proposal dated 1 November 2000.

The project will consist of three phases of an office/commercial development proposed by Legacy Partners, three residential development sites, and public improvements. The buildings associated with proposed project may be up to 16 stories high with up to two levels of below-grade parking. This report contains information regarding subsurface conditions, preliminary foundation design criteria, temporary shoring design criteria, and a discussion of dewatering issues. On the basis of the results of this investigation, we conclude the proposed project is feasible. We judge light-weight buildings with no below-grade parking level can be supported on a stiffened shallow foundation system bearing on a layer of compacted fill. Light to moderate weight buildings with below-grade parking levels may be supported on mat foundations. Relatively heavy structures, such as the proposed 400-unit residential building, or moderate weight buildings with no below-grade parking level should be supported on a driven pile foundation. An active dewatering system with a conventional soldier pile and lagging shoring system or a passive dewatering system with a shoring design that utilizes sheetpiles or soil-cement columns are recommended for this site.

Our conclusions and preliminary recommendations are based on limited subsurface exploration and engineering analyses. When individual components of the project are ready for design-level evaluation, we recommend a design-level geotechnical study be performed to further investigate the subsurface conditions at the building and parking structure sites and to develop final design recommendations.



# Treadwell&Rollo

Mr. Cy Colburn  
5 February 2001  
Page 2

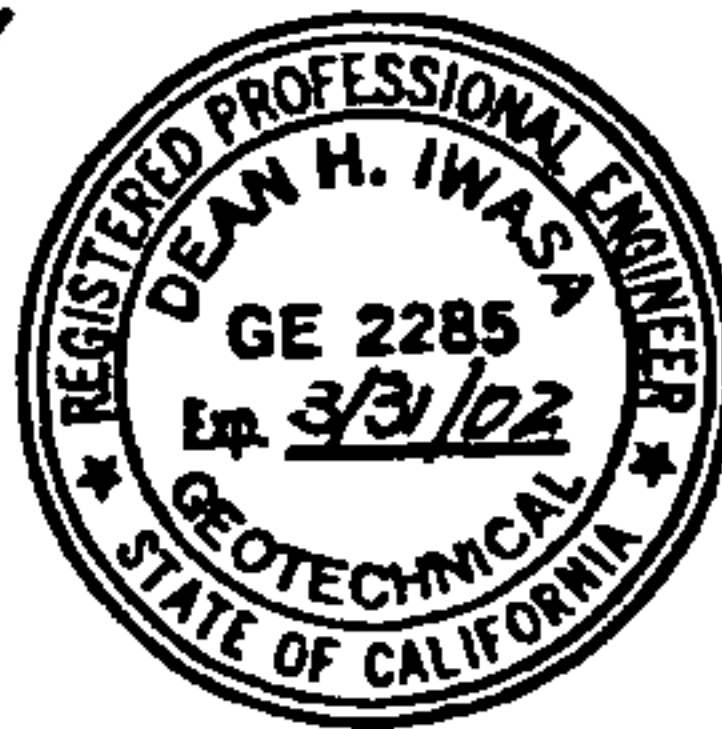
We appreciate the opportunity to work with you on this project. If you have any questions, please call.

Sincerely yours,  
TREADWELL & ROLLO, INC.

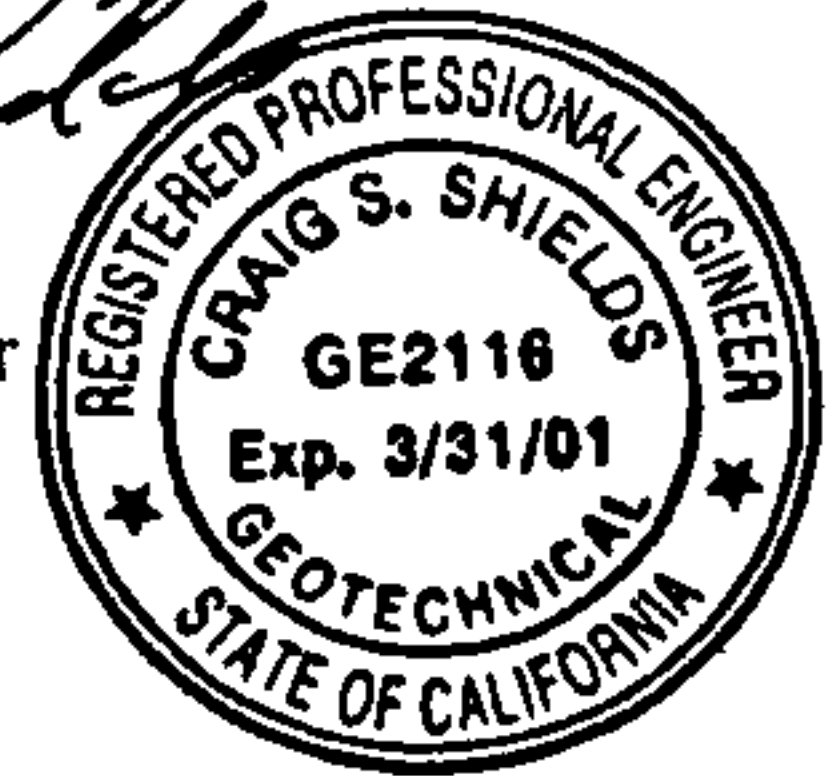


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**GEOTECHNICAL INVESTIGATION  
NORTH MARKET AND WEST JULIAN SITE  
San Jose, California**

**Legacy Partners  
Foster City, California**

**5 February 2001  
Project No. 2737.02**

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DISTRIBUTION

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## **APPENDIX A**

Figures A-1 through A-7	Cone Penetration Test Results CPT-1 and CPT-6
Figure A-8	Classification Chart for Cone Penetration Tests

## **PRELIMINARY GEOTECHNICAL INVESTIGATION NORTH MARKET AND WEST JULIAN STREET SITE San Jose, California**

### **1.0 INTRODUCTION**

This report presents the results of our preliminary geotechnical investigation of the proposed North Market and West Julian Street project site (West Julian Revitalization site) in San Jose. The site is east of Route 87 (Guadalupe Parkway), south of the Union Pacific Railroad line, north of West Saint James Street, and west of North Market Street, as indicated on Figure 1.

The site is currently occupied by several existing structures, asphalt- and concrete-paved areas, open lots, chain-link fences, trees, and existing utility lines. The approximate locations of existing structures, paved areas, and utility lines are indicated on the *Preliminary Improvement Plans* (Sheets 1 and 2), prepared by MacKay & Somps, dated 4 December 2000. Ground surface across the site varies between about Elevation 75 and 83 feet, Mean Sea Level (MSL) datum.

### **2.0 PROJECT DESCRIPTION**

The project will consist of: 1) an office/commercial development proposed by Legacy Partners, 2) residential developments, and 3) public improvements.

#### **2.1 Proposed Office/Commercial Development**

Conceptual plans indicate the office/commercial project proposed by Legacy Partners will be developed in three phases. Each phase will consist of an office/commercial building that extends up to 11 stories high (170 feet above ground) with two levels of below-grade parking (see

Figure 2 for location of each phase). In addition, Phases 1 and 3 will each include the construction of a six-story parking garage with two levels of below-grade parking.

The proposed office/commercial buildings will have plan dimensions that vary between roughly 150 by 280 feet for Phases 1 and 3, and an L-shaped footprint with dimensions of about 260 by 260 feet for Phase 2. The six-story parking garages for Phases 1 and 3 will have plan dimensions of about 200 by 300 feet and 120 by 230 feet, respectively. The structural loads for the buildings and parking structures are not known at this time. However, based on our experience with similar projects, we judge dead plus live column loads will be on the order of 1,000 to 2,000 kips. The excavations for the below-grade parking areas within the proposed office/commercial development will be approximately 25 feet deep. Temporary shoring and dewatering will be required to facilitate the construction of the proposed below-grade levels.

## **2.2 Residential Developments**

As indicated on Figure 2, three areas of the project site are proposed for residential use. The proposed residential building sites along the northwestern and northeastern edges of the project will be developed with up to 50 and 400 residential units, respectively. The 400-unit residential development will also have approximately 4,000 square feet of ground floor commercial space. The third proposed residential building site is southeast of Devine Street and northwest of West Saint James Street. Plans are to develop this site with up to 200 residential units. The design of the residential developments is not complete at this time; however, we understand the proposed residential buildings may have up to two levels of below-grade parking and may be up to 16 stories high. For preliminary design purposes, we have assumed the 50-, 200-, and 400-unit residential buildings will be approximately 2, 8, and 16 stories high, respectively, with dead plus live column loads of about 400, 1,500, and 3,000 kips, respectively. Excavations up to 25 feet deep may be required to accommodate the proposed below-grade parking levels.



## 2.3 Public Improvements

Conceptual plans indicate the proposed public improvements for this project will include a skateboard park at the northern corner of the site, expansion of Pellier Park at the southern end of the site, realignment of West Julian Street, and widening of a segment of Bassett Street between Terraine and San Pedro Streets.

## 3.0 SCOPE OF SERVICES

The scope of our services was outlined in our proposal dated 1 November 2000. Our services consisted of performing six cone penetration tests (CPTs) and engineering analyses to develop conclusions and recommendations regarding:

- soil and groundwater conditions at the project site
- selection of preliminary foundation type(s) for the proposed office/commercial buildings, residential buildings, and parking structures
- preliminary design criteria for the recommended foundation types(s), including vertical and lateral capacities
- estimate of foundation settlement
- geotechnical and geological hazards, including the potential for strong ground shaking and soil liquefaction<sup>1</sup>
- temporary shoring and site dewatering
- 1997 Uniform Building Code (UBC) soil profile type and near source factors.

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<sup>1</sup> Liquefaction is a phenomenon in which saturated, cohesionless soil experiences a temporary loss of strength due to the buildup of excess pore water pressure, especially during cyclic loading such as that induced by earthquakes. Soil most susceptible to liquefaction is loose, clean, saturated, uniformly graded, fine-grained sand and silt of low plasticity that is relatively free of clay.



## 4.0 FIELD INVESTIGATION

Subsurface conditions at the site were explored by performing six CPTs at the approximate locations shown on Figure 2. The CPTs were performed by hydraulically pushing a 1.4-inch-diameter, cone-tipped probe 47 to 80 feet into the ground. CPT-2 was terminated at a depth of 47 feet due to very high penetration resistance encountered during the test. Other CPTs were advanced to a depth of about 80 feet, which was the planned depth of exploration. The CPTs utilize a cone at the end of a probe to measure tip resistance and a sleeve behind the cone tip to measure frictional resistance. A small, porous stone, which lies between the cone tip and the friction sleeve, monitors pore pressures in the soil during the entire depth of probing. Soil data, including tip resistance, frictional resistance, and porewater pressure, were recorded in the field. Accumulated data were processed by computer to provide engineering information for the soil encountered at each location. CPT summaries, which show tip resistance, local friction, approximate strength values, and an interpreted soil profile, are presented in Appendix A. A soil classification chart based on standard electronic cone penetration measurements is also presented in Appendix A.

## 5.0 SUBSURFACE CONDITIONS

Subsurface information from the CPTs performed during this investigation indicate the project site is generally blanketed by 7 to 15 feet of fill consisting of layers of stiff to hard sandy silt, silt, and clay, and medium dense to very dense sand and silty sand. Below the near-surface fill layer, we generally encountered native soil consisting of medium stiff to stiff silt and clay with occasional 1- to 3-foot-thick, interbedded layers of medium dense to dense sand and silty sand. An exception was encountered in CPT-2 at depth of about 22 feet, where an approximately five-foot-thick layer of medium dense silty sand was encountered. The native silt and clay layer extends to a depth of roughly 65 to 80 feet below the ground surface, where a dense to very dense layer of sand was generally encountered.

Based on our review of groundwater monitoring reports prepared by Azure Environmental, dated 31 January, 28 April and 30 July 2000, we understand the shallow groundwater level at the site varies between the depths of 15.0 and 21.9 feet below the ground surface, which correspond to elevations between approximately 55.7 and 62.2 feet, MSL.

## 6.0 REGIONAL SEISMICITY

The major active<sup>2</sup> faults in the area are the San Andreas, Hayward, and Calaveras Faults. These and other active faults of the region are shown on Figure 3. For each of the active faults, the distance and direction from the site and estimated maximum Moment magnitude<sup>3</sup> events are summarized in Table 1.

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<sup>2</sup> Active faults are defined as those exhibiting either surface ruptures, topographic features created by faulting, surface displacements of Holocene (younger than about 11,000 years old) deposits, tectonic creep along fault lines, and/or close proximity to linear concentrations or trends of earthquake epicenters.

<sup>3</sup> Moment magnitude is an energy-based scale and provides a physically meaningful measure of the size of a faulting event. Moment magnitude is directly related to average slip and fault rupture area.

**TABLE 1**  
**Regional Faults and Seismicity**

<b>Fault</b>	<b>Approximate Distance from Site (kilometers)</b>	<b>Direction from Site</b>	<b>Maximum Moment Magnitude</b>
Hayward (Southeast Extension)	9	Northeast	6.4
Monte Vista – Shannon	12	Southwest	6.8
Calaveras (Central segment)	14	Northeast	6.6
Hayward (Total)	14	Northeast	7.1
Calaveras (Northern segment)	14	Northeast	7.0
Hayward (Southern segment)	14	Northeast	6.9
San Andreas (1906 Rupture)	19	Southwest	7.9
San Andreas (Peninsula segment)	19	Southwest	7.2
San Andreas (Santa Cruz Mountain segment)	20	Southwest	7.2
Sargent	23	South	6.8
Zayante-Vergeles	29	South	6.8
Greenville (Southern segment)	36	East	6.9

Since 1800, four major earthquakes have been recorded on the San Andreas Fault. In 1836, an earthquake with an estimated maximum intensity of VII on the Modified Mercalli (MM) scale (Figure 4) occurred east of Monterey Bay on the San Andreas Fault (Toppozada and Borchardt 1998). The estimated Moment magnitude,  $M_w$ , for this earthquake is about 6-1/4. In 1838, an earthquake occurred with an estimated intensity of about VIII-IX (MM), corresponding to a  $M_w$  of about 7-1/2. The San Francisco Earthquake of 1906 caused the most significant damage in the history of the Bay Area in terms of loss of lives and property damage. This earthquake created a surface rupture along the San Andreas Fault from Shelter Cove to San Juan Bautista approximately 430 kilometers (km) in length. It had a maximum intensity of XI (MM), a  $M_w$  of

about 7.9, and was felt over 550 km away in Oregon, Nevada, and Los Angeles. The most recent major earthquake to affect the Bay Area was the Loma Prieta Earthquake of 17 October 1989 with a  $M_w$  of 6.9. The epicenter of this earthquake was in the Santa Cruz Mountains, approximately 34 km from the site.

In 1868, an earthquake with an estimated maximum intensity of X on the MM scale occurred on the southern segment (between San Leandro and Fremont) of the Hayward Fault. The estimated  $M_w$  for the earthquake is 7.0. In 1861, an earthquake of unknown magnitude (probably a  $M_w$  of about 6.5) was reported on the Calaveras Fault. The most recent significant earthquake on this fault was the 1984 Morgan Hill earthquake ( $M_w=6.2$ ).

The southeast extension of the Hayward fault lies about 9 km northeast of the site and is one of the closest active faults to the site. The Monte Vista-Shannon fault is approximately 12 km southwest of the site. Both faults are Type B faults<sup>4</sup> and are capable of generating a magnitude 6.4 and 6.8 earthquake, respectively. They are considered less active than the San Andreas or Hayward faults, which are both Type A faults, mainly due to their limited slip rate (less than 5 mm/yr) and limited maximum magnitude (less than 7.0).

In 1999, the Working Group on California Earthquake Probabilities (WGCEP 1999) at the U.S. Geologic Survey (USGS) predicted a 70 percent probability of a magnitude 6.7 or greater earthquake occurring in the San Francisco Bay Area by the year 2030. More specific estimates of the probabilities for different faults in the Bay Area are presented in Table 2.

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<sup>4</sup> Type A and B faults, as defined in the *California Department of Conservation, Division of Mines and Geology, Maps of Known Active Fault Near-Source Zones in California and Adjacent Portions of Nevada*, dated February 1988.



**TABLE 2**  
**WGCEP (1999) Estimates of 30-Year Probability (2000 to 2030)**  
**for a Magnitude 6.7 or Greater Earthquake**

<b>Fault</b>	<b>Probability (percent)</b>
Calaveras (South segment)	7
Calaveras (North segment)	18
Hayward (South segment)	17
San Andreas (Peninsula segment)	15
San Andreas (Santa Cruz Mtn. segment)	10
Greenville	3

## **7.0 ISSUES, IMPACTS, AND PRELIMINARY MITIGATION MEASURES**

On the basis of our preliminary geotechnical investigation, we conclude the proposed project can be constructed as planned. The primary geotechnical issues that should be addressed during the design of the office/commercial buildings, residential buildings, and parking structures are:

- strong ground shaking and associated geologic hazards, including the potential for soil liquefaction
- selection of an appropriate foundation type(s) for support of the proposed buildings and parking structures
- hydrostatic uplift pressures on the floor slabs and foundations of the below-grade parking levels
- lateral earth and water pressures on temporary shoring and below-grade walls
- dewatering systems.

Our discussion of these and other issues, their potential impacts on the design of the project, and preliminary mitigation measures are presented in the remainder of this report.

## 7.1 Strong Ground Shaking and Geologic Hazards

### 7.1.1 Ground Shaking

Issue 7.1.1A: The site is in the seismically active San Francisco Bay Area.

Impact 7.1.1A: We expect the site will experience strong ground shaking during a major earthquake on one of the nearby active faults.

Mitigation 7.1.1A: Seismic zoning and near-source factors as described in the 1997 Uniform Building Code (UBC) should be used in the seismic design of new structures at the project site. Seismic design criteria are presented in Section 8.6 of this report. Also, if required, a site-specific probabilistic seismic hazard analysis (PSHA) can be performed to develop acceleration response spectra for the seismic design of the proposed office/commercial buildings, residential buildings and parking structures.

### 7.1.2 Geologic Hazards

We evaluated geologic hazards for the site, including the potential for soil liquefaction, ground rupture, lateral spreading<sup>5</sup>, soil densification<sup>6</sup>, and fault rupture.

Issue 7.1.2A: Based on our evaluation, we conclude that a majority of the soil beneath the project site consists of medium stiff to stiff silt and clay and is not susceptible to soil liquefaction and soil densification. However, there are several discontinuous and random 1- to 3-foot-thick

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<sup>5</sup> Lateral spreading is a phenomenon in which surficial soil displaces along a shear zone that has formed within an underlying liquefied layer. Upon reaching mobilization, the surficial blocks are transported downslope or in the direction of a free face by earthquake and gravitational forces.

<sup>6</sup> Soil densification is a phenomenon in which non-saturated, loose cohesionless soil is densified by earthquake vibrations, resulting in ground surface settlement.

layers of saturated, medium dense to dense sand and silty sand and a five-foot-thick layer of medium dense silty sand (see CPT-2 at 22 feet) that are interbedded within the silt and clay layers at the site.

Impact 7.1.2A: During a large earthquake, we judge the saturated and medium dense portions of these sand and silty sand layers are potentially susceptible to soil liquefaction. Liquefaction-induced ground settlement of these sand layers is estimate to be about one inch.

Mitigation 7.1.2A: To mitigate the potential hazard associated with liquefaction-induced settlement, we preliminarily judge that new building foundations and site improvements should be designed to accommodate about one inch of settlement occurring at any location across the site.

Issue 7.1.2B: We evaluated the potential for lateral spreading and concluded the potential is low because the sand layers beneath the site are relatively thin, medium dense to dense, and not laterally continuous and because there is no slope face to flow toward. Likewise, we judge the potential for ground rupture occurring at the site is very low because the potentially liquefiable soil layers are relatively thin in comparison to the overlying non-liquefiable soil layers.

Impact 7.1.2B: No impact on project design.

Mitigation 7.1.2B: No mitigation required.

Issue 7.1.2C: Our review of published geologic maps indicates that no known active faults or fault traces pass through the site. Therefore, we judge the potential for surface rupture occurring at the site is low.

Impact 7.1.2C: No impact on project design.

Mitigation 7.1.2C: No mitigation required.



## 7.2 Foundation Types

Selection of the most appropriate foundation type(s) for support of the proposed buildings and parking structures is based on our preliminary estimates of the weight of the proposed buildings, depth of the proposed below-grade parking levels, and strength and compressibility of the on-site soil. The conceptual design of the 50-, 200-, and 400-unit residential developments is not complete at this time, although we understand these developments may have up to two levels of below-grade parking. The conceptual design of the office/commercial development includes three 11-story office/commercial buildings and two six-story parking structures; each of these buildings and parking structures will have two levels of below-grade parking.

To simplify our engineering analyses and foundation selection process for the proposed residential buildings, office/commercial buildings, and parking structures, we will discuss foundation systems for: 1) residential buildings with no below-grade parking level and 2) residential buildings, office/commercial buildings, and parking structures with 1 to 2 below-grading parking levels. Our discussion of these foundation issues is presented in the following sections.

### 7.2.1 Residential Buildings with No Below-Grade Level

Issue 7.2.1A: The project site is blanketed by 7 to 15 feet of heterogeneous fill consisting of layers of stiff to hard sandy silt, silt, and clay, and medium dense to very dense sand and silty sand.

Impact 7.2.1A: Heterogenous fill does not appear to have been uniformly compacted. We preliminarily estimate that in its existing condition, it could experience about 1 to 2 inches of erratic settlement under the weight of new building loads.

Mitigation 7.2.1A, Part 1: We judge that light residential buildings, such as those associated with the 50-unit development, can be supported in the near-surface fill, provided the proposed shallow foundations are underlain by a layer of uniformly compacted fill and are stiff enough to

span over localized areas of non-support. We preliminarily judge that a stiffened, shallow foundation system, such as interconnected grade beams or a reinforced concrete mat that bears on an 18-inch-thick layer of compacted fill will provide adequate support for light residential buildings. We estimate that a stiffened shallow foundation system bearing on at least an 18-inch-thick layer of compacted fill will settle less than 1/2 inch under static loads.

Mitigation 7.2.1A, Part 2: We judge moderate to heavy residential buildings with no below-grade parking levels, such as those that may be considered for the 200- and 400-unit developments, should be supported on deep foundations that will transfer new building loads below the near-surface heterogeneous fill layer. We preliminarily judge a driven, precast, prestressed concrete pile foundation system will be most appropriate for support of the proposed 200- and 400-unit buildings with no below-grade parking levels. The proposed concrete piles will gain support through a combination of skin friction in medium stiff to stiff silt and clay and through end-bearing in dense to very dense sand that lie beneath the proposed building sites. We preliminarily estimate that pile-supported buildings will experience less than 3/4 inch of total settlement.

## **7.2.2 Buildings and Parking Structures with One or Two Below-Grade Parking Levels**

If the proposed residential buildings are designed with one or two levels of below-grade parking, we estimate that approximately 13- to 25-foot-deep excavations will be required. A 25-foot-deep excavation will be required for the proposed 11-story office/commercial buildings and six-story parking structures. Based on the results of our subsurface exploration program, we judge that medium stiff to stiff silt and clay are likely to be exposed at the bottom of the proposed excavations, except near CPT-2 and CPT-3 where medium dense to dense silty sand may be exposed.

Issue 7.2.2A: During excavation for the below-grade parking level(s), the overburden pressure on the underlying silt and clay layers will be reduced.

Impact 7.2.2A: We expect the bottom of the excavation may heave up to approximately one inch due to soil removal.

Mitigation 7.2.2A: No mitigation required because heave will occur during construction and will not impact the design of the proposed buildings.

Issue 7.2.2B: Medium stiff to stiff silt and clay are likely to be exposed at the bottom of the proposed excavations, except near CPT-2 and CPT-3 where medium dense to dense silty sand is likely to be exposed.

Impact 7.2.2B: We judge the strength and consolidation characteristics of the underlying medium stiff to stiff silt and clay will limit the type of foundation system to be used for support of the proposed residential buildings. Also, medium dense silty sand near CPT-2 and CPT-3 is potentially liquefiable and may result in abrupt ground settlement.

Under the weight of heavy structures, such as the proposed 400-unit residential building, we judge the silt and clay layers at the site may consolidate, resulting in large foundation settlement.

Mitigation 7.2.2B, Part 1: We preliminarily judge that a mat foundation bearing on medium stiff to stiff silt and clay and dense silty sand will be capable of supporting light to moderate weight buildings, such as the 50- and 200-unit residential structures, the 11-story office/commercial buildings, and the six-story parking structures. The settlement of the mat will depend on the weight of the proposed building and depth of the proposed excavation. For light to moderate weight buildings with 1 to 2 levels of below-grade parking, we judge foundation settlement will range between approximately 1/2 and 1-1/2 inches with about half of the settlement occurring during construction of the buildings and parking structures. If medium dense silty sand is exposed at the base of the excavation (possibly near CPT-2 and CPT-3), we preliminarily judge that it should be removed and replaced with lean concrete to reduce the potential for soil liquefaction and liquefaction-induced ground settlement.



Mitigation 7.2.2B, Part 2: As an alternate, the 11-story office/commercial buildings and six-story parking structures can be supported on driven, prestressed, precast concrete piles, as described in Section 7.2.1. We estimate that a pile-supported building will experience less than 1/2 inch of settlement and will not be susceptible to liquefaction-induced settlement.

Mitigation – 7.2.2B, Part 3: The relatively heavy 400-unit residential building may cause the underlying silt and clay to consolidate and settle. Therefore, we preliminarily judge the 400-unit residential structure with one or two levels of below-grade parking should be supported on a pile foundation system, as described in Section 7.2.1. We estimate that a pile-supported building will experience less than 1/2 inch of settlement and will not be susceptible to liquefaction-induced settlement.

### 7.3 Dewatering and Uplift Pressure

Issue 7.3A: The shallow groundwater level was previously measured by Azure Environmental between depths of 15.0 and 21.9 feet below the ground surface, which correspond to elevations between approximately 55.7 and 62.2 feet, MSL.

Impact 7.3A: The proposed shoring system, and below-grade floors and walls will be affected by the hydrostatic pressure of the groundwater.

Mitigation 7.3A: For evaluating hydrostatic pressures on the proposed shoring system, and below-grade floors and walls, a preliminary design groundwater elevation of 65 feet, MSL should be used. This design groundwater level is approximately three feet higher than the highest shallow groundwater level previously measured Azure Environmental. The three-foot difference between the design groundwater level and the measured groundwater level is to account for potential seasonal groundwater level fluctuations that may occur at the site.

Issue 7.3B: The proposed depth of excavation to accommodate two levels of below grade parking is approximately 25 feet below the existing ground surface.

Impact 7.3B: The bottom of the excavation is likely to extend below the groundwater table.

Mitigation 7.3B: To construct the below-grade basement levels, the groundwater will need to be drawn down to a depth of at least three feet below the bottom of the planned excavation and maintained at that elevation until sufficient weight and/or tiedown capacity is available to resist the hydrostatic uplift forces. The groundwater at the site can be drawn down by using either an active dewatering system that pumps the groundwater from the site and surrounding area, or a passive dewatering system that includes a low-permeability cut-off wall comprised of soil-cement columns or sheetpiles, and a series of subdrains, sumps, and pumps to collect and remove water that seeps into the excavation. These proposed dewatering systems are discussed in subsequent sections of this report.

## 7.3.1 Active Dewatering System

Issue 7.3.1A: An active dewatering system typically consists of a series of groundwater extraction wells that are positioned around the perimeter of the site. The wells are screened within the saturated zone and are pumped to draw the groundwater down to a depth of at least three feet below the bottom of the proposed excavation.

Impact 7.3.1A: The pumped groundwater is typically discharged directly to the municipal sewer or storm drain, unless a regulatory agency requires the pumped water to be treated to meet regulatory discharge requirements.

Mitigation 7.3.1A: The dewatering contractor should check and comply with local water discharge requirements.

Issue 7.3.1B: An active dewatering system could pump a large volume of water from the site and if chemical contaminants are present, the water may require treatment and/or disposal.

Impact 7.3.1B: An active dewatering system could be expensive to operator because of the large volume of water to be pumped from the site, especially if the pumped water requires treatment and/or disposal.

Mitigation: 7.3.1B: The volume of water to be removed, treated, and disposed from the site can be reduced by using a passive dewatering system, as discussed in Section 7.3.2.

Issue 7.3.1C: An active dewatering system will lower the groundwater level at the site and surrounding area.

Impact 7.3.1C: Lowering the groundwater level at the site and surrounding area will increase the effective stresses on the underlying soil, resulting in ground settlement that could adversely impact adjacent structures and improvements.

Mitigation 7.3.1C: A design-level geotechnical investigation should be performed for the proposed building and parking structure sites to evaluate the consolidation properties of the underlying silt and clay and to determine the potential for settlement due to site dewatering.

Issue 7.3.1D: An active dewatering system can be used in conjunction with a conventional shoring system consisting of soldier beams and lagging with tieback anchors. Furthermore, the proposed shoring system will not need to be designed to resist hydrostatic pressures because the groundwater at the site and surrounding area will be drawn down by the active dewatering system. Our discussion of temporary shoring systems is presented in Section 7.4.

Impact 7.3.1D: No impact on the design of the dewatering system.

Mitigation 7.3.1D: No mitigation required.

## 7.3.2 Passive Dewatering System with Cut-off Wall

A passive dewatering system with a low-permeability cut-off wall can be used to dewater the proposed excavation. A passive dewatering system typically consists of a network of subdrains, sumps, and pumps that are installed at the bottom of the proposed excavation and are capable of collecting and removing groundwater that may enter the excavation.

Issue 7.3.2A: The performance of a passive dewatering system is sensitive to changes in groundwater level and the depth of the excavation, especially if excavations extend below the foundation subgrade level, such as for elevator pits.

Impact 7.3.2A: Changes in the groundwater level can increase the hydraulic head pressure in underlying soil, possibly resulting in ground heave and soil rupture at the bottom of the proposed excavation. Changes in the depth of excavation can decrease the thickness of low permeability soil layers at the site, resulting in an increase in the rate of groundwater inflow.

Mitigation 7.3.2A: Special dewatering recommendations may be required for excavations that extend below the foundation subgrade level. We should review the excavation and grading plans and evaluate whether special dewatering recommendations are needed.

Issue 7.3.2B: For a passive dewatering system to operate properly, a low-permeability wall around the perimeter of the excavation is required.

Impact 7.3.2B: If a low-permeability wall is not provided around the perimeter of the excavation, groundwater from the surrounding area will flow into the site and will be pumped out by the passive dewatering system. This will increase the volume of water to be pumped from the site and result in a general lowering of the groundwater level in the surrounding area. Therefore, the potentially beneficial effects of: 1) reducing the amount of groundwater to be pumped from the site and 2) protecting adjacent structures from settlement associated with site dewatering will not be achieved. Also, if the bottom of the low-permeability wall is not



sufficiently deep, the bottom of the excavation may heave and the soil subgrade may rupture due to high hydraulic head pressures in the underlying soil.

Mitigation 7.3.2B: We preliminarily judge a low-permeability wall for this site should have an average hydraulic conductivity no greater than  $1 \times 10^{-7}$  feet per minute (fpm). Also, to reduce the potential for bottom heave, we estimate the bottom of the low-permeability wall should extend to a depth of about 15 to 20 feet below the bottom of the excavation. The types of low-permeability barriers that were qualitatively evaluated during this investigation include: 1) a slurry wall, 2) an interlocking sheetpile wall, 3) a mixed-in-place soil-cement wall, and 4) permeation grout to fill the void spaces within highly permeable soil layers.

A slurry wall system consists of excavating a deep trench using a thick soil-bentonite slurry to provide lateral support for the sidewalls of the trench. Once the appropriate trench depth is achieved, a cement and bentonite admixture can be used to fill the trench and create a low-permeability cut-off wall. The disadvantages of this system are the high cost of constructing this type of wall and the potential for movement of the adjacent street improvements during wall construction.

Interlocking sheetpile walls are typically installed by driving individual sheetpiles into the ground using a vibratory hammer. Considering the presence of medium dense to very dense granular soil layers beneath the site, we conclude an interlocking sheetpile system could be difficult to install at the site. Therefore, a test program is warranted to verify that the sheetpiles can be installed to the proper depth.

A series of overlapping, mixed-in-place soil-cement columns can be used to create a low-permeability barrier that restricts the lateral flow of groundwater into the excavation. This system typically consists of using a special auger/mixing tool to drill beneath the ground surface and mix a cement-bentonite fluid into the soil. This system can be made relatively rigid if reinforcing steel is added to the soil-cement columns. If sufficient reinforcing steel is added, the soil-cement column walls can be used to cut off the lateral flow of groundwater and temporarily

retain the soil exposed on the sides of the proposed excavation. In addition, the soil-cement columns are mechanically mixed-in-place using a special auger/mixing tool; therefore, soil-cement columns can be installed even if the site is underlain by layers of dense to very dense sand and gravel. The disadvantages of this system are the cost to install the soil-cement column walls and space requirements. Based on our past experience with soil-cement column walls, we typically find that this system will occupy a 2- to 3-foot-wide space around the perimeter of the site.

Permeation grouts, such as chemical or micro-fine cement grout, can be used to reduce the amount of groundwater entering the proposed excavation. Permeation grouts can be injected into coarse-grained soil to fill the pore space between soil particles; thereby, reducing the overall permeability of the granular soil layers. Considering the potential difficulties associated with injecting grout from the ground surface into underlying layers of sand and gravel, we judge that it could be very difficult to completely cut off the lateral flow of water into the excavation using permeation grouting; however, the rate of groundwater infiltration can be significantly decreased. Therefore, we expect that groundwater may continue to seep into the excavation even after the permeation grouting is completed. The primary advantage of using permeation grouting is the capability of decreasing the flow of water into the excavation at locations that are difficult to access. Disadvantages of using permeation grout are high cost and potential for continued seepage.

Based on our evaluation of various types of low-permeability barriers, we judge that either sheetpiles or soil-cement column walls are the most appropriate system for the proposed project site if a passive dewatering system is used.

## 7.4 Temporary Shoring

Issue 7.4A: Excavations on the order of 13 to 25 feet deep will be required to accommodate 1 to 2 levels of below-grade parking.

Impact 7.4A: If the sides of the excavations are not properly restrained, soil and improvements adjacent to the excavations may experience excessive movements and the existing improvements may be damaged.

Mitigation 7.4A: Several methods of temporary shoring can be used at the project site. The most appropriate shoring system should take into account the requirements for protecting adjacent properties as well as cost. We have qualitatively evaluated the following systems:

- conventional soldier pile and lagging
- sheetpiles
- soil-cement column walls.

A soldier pile and lagging system with tiebacks or internally braces can be used to laterally restrain the sides of the excavation. However, this type of system will not prevent groundwater from flowing into the excavation. Therefore, this system should be used in conjunction with an active dewatering system where the groundwater level is drawn down at the site and surrounding area.

A sheetpile shoring system is relatively flexible, i.e. it can deform measurably as stresses are applied. Therefore, this system may result in horizontal and vertical ground movement adjacent to the shoring system. Also, a sheetpile system may be difficult to install at locations underlain by dense to very dense granular soil layers. A test program should be performed to verify that sheetpiles can be installed to the proper depth.

Mixed-in-place soil-cement column walls can be used at the site to provide a relatively rigid system, capable of significantly limiting lateral deformations and ground subsidence adjacent to the shoring system. This system will likely require tiebacks or internal braces to resist lateral soil and water pressures. This system has an advantage of being capable of cutting off the lateral flow of groundwater into the excavation and reducing the potential for bottom heave. Therefore, this system can be used in conjunction with either an active or passive dewatering system. The

primary disadvantages of this system are cost and space requirements, as discussed in Section 7.3.2.

## 7.5 Excavation and Monitoring

Issue 7.5A: Site excavation and grading equipment will be required during the demolition of existing structures and improvements and during excavation of soil for new below-grade parking levels.

Impact 7.5A: Site and soil conditions can result in difficulties during site demolition and excavation.

Mitigation 7.5A: The soil at the site consists of clay, silt, sand, and gravel that can be excavated with conventional earth-moving equipment, such as loaders and backhoes. Removal of existing on-site improvements, including the concrete foundations of existing buildings will require equipment capable of breaking concrete.

Issue 7.5B: A shoring system will be required to retain the vertical sidewalls of the proposed excavations and limit the potential for lateral and vertical movement of adjacent street improvements.

Impact 7.5B: The shoring system is expected to deform a small amount during construction. The magnitude of shoring movement is difficult to estimate because it depends on many factors, including the type of system used and the contractor's skill in installing the system. For a properly designed and constructed shoring system, we judge lateral movements will be within ordinarily accepted limits of about one inch. If too much movement occurs, existing improvements adjacent to the shoring system may move and become damaged.

Mitigation 7.5B: To reduce the potential for excessive movement of the shoring system, we preliminarily judge the contractor should establish survey points on the shoring, adjacent streets, and sidewalks to monitor the movement of the shoring system and existing improvements during



and immediately after construction. If excessive movement of the survey points is measured, the shoring designer and contractor should be notified immediately to evaluate the condition of the shoring system and determine whether additional support is needed.

## **8.0 PRELIMINARY RECOMMENDATIONS**

Preliminary recommendations for foundation design, temporary shoring, and other geotechnical aspects of this project are presented in the following sections.

### **8.1 Stiffened Shallow Foundation on Compacted Fill Layer**

We preliminarily judge that light residential structures for the 50-unit residential development can be supported on a stiffened, shallow foundation system that bears on at least an 18-inch-thick layer of fill compacted to at least 90 percent relative compaction. A stiffened shallow foundation system may consist of interconnected, concrete strip footings or a reinforced concrete mat. We preliminarily recommend the stiffened foundation system be embedded at least 18 inches below the lowest adjacent grade. We anticipate using an allowable dead plus live load bearing pressure will be between 2,000 and 3,000 pounds per square foot (psf).

Lateral forces can be resisted by a combination of friction along the base and passive resistance against the embedded vertical surfaces of the stiffened shallow foundation system. We preliminarily recommend using a base friction factor of 0.35 times the dead load of the structure. To calculate the passive resistance acting against the sides of the mat, we preliminarily recommend using a triangular pressure distribution of 300 pounds per cubic foot (pcf). These values include a factor of safety of about 1.5.

### **8.2 Mat Foundation**

We preliminarily judge that light to medium weight buildings and parking structures, which include those planned for the 50-unit and 200-unit residential buildings, 11-story office/commercial buildings, and six-story parking structures, each with one or two levels of

below-grade parking, can be supported on a mat foundation that rests on medium stiff to stiff silt and clay and dense sand and silty sand. For buildings and parking structures with two levels of below-grade parking, we preliminarily recommend the mat be designed for a maximum dead plus live load bearing pressure of 2,000 psf. If one level of below-grade parking is used, we preliminarily recommend a maximum dead plus live load bearing pressure of 1,100 psf. These values may be increased by one-third for total loads, including wind or seismic forces. To evaluate the pressure distribution beneath the mat foundation, we preliminarily recommend using a modulus of vertical subgrade reaction of 20 pounds per cubic inch (pci). This value has been scaled to account for the plan dimensions of the mat foundation.

We preliminarily recommend the uplift resistance of the mat be checked using a design groundwater level at Elevation 65 feet. Additional groundwater level measurements should be taken at the end of the rainy season to confirm that this elevation is appropriate for use in the design of the buildings.

Lateral forces can be resisted by a combination of friction along the base and passive resistance against the vertical face of the mat foundation. Friction along the bottom of the foundation should be reduced to account for the presence of waterproofing material at the base of the mat. Frictional resistance will depend on the type of waterproofing used. We preliminarily recommend using a base friction factor of 0.15 and 0.25 for Paraseal and Preprufe membranes, respectively, times the dead load of the structure. If a waterproofing membrane other than Paraseal or Preprufe is used at the site, we should review the material specifications and select an appropriate base friction factor. To calculate the passive resistance acting against the sides of the mat, we preliminarily recommend using a uniform pressure of 2,400 psf. These values include a factor of safety of at least 1.5.

If the weight of the building and mat foundation is not sufficient to overcome the hydrostatic uplift loads, tiedown anchors will be required. Tiedown anchors should consist of high-strength steel bars or tendons embedded in small-diameter, drilled and grouted shafts. The anchors should extend into the medium stiff to stiff clay and silt and medium dense to very dense sand

and gravel beneath the mat and spaced at least four shaft diameters apart or three feet (center to center), whichever is greater. Uplift resistance will be developed through skin friction between the anchor shafts and the surrounding soil. We preliminarily recommend an allowable friction value of 650 psf be used for computing anchor capacities. This value includes a factor of safety of 2.0. Special attention should be given to waterproofing the connections between the tiedown anchors and the mat slab. Because the tiedowns will be permanent, we recommend that all exposed reinforcing steel be adequately protected from corrosion.

### 8.3 Pile Foundation

The proposed 400-unit residential structure and 200-unit residential building with no below-grade parking levels should be supported on driven, 14- or 16-inch-square, prestressed, precast concrete piles. As an alternate foundation system, the 11-story office/commercial buildings and six-story parking structures can also be supported driven concrete piles. The piles will gain support through a combination of skin friction in medium stiff to stiff silt and clay and through end-bearing in dense to very dense sand that lie beneath the proposed building sites. We preliminarily recommend using an allowable skin friction value of 650 psf for computing the compression and tension capacity of the driven piles and an allowable end-bearing value of 10,000 psf. These values may be increased by 1/3 for total loads, including wind or seismic forces. It may be possible to achieve much higher end-bearing support in dense sand and/or gravel layers below the depth of 70 feet. Deep borings should be drilled during the final investigation to determine the thickness and lateral extent of these layers.

Piles should be spaced no closer than three pile widths center to center to avoid reductions to the axial capacities due to group effects.

Lateral load resistance can be mobilized by the individual piles in combination with other foundation elements that are embedded below the ground surface. Lateral resistance of each pile will depend on the stiffness of the pile, the strength of the surrounding soil, allowable deflection of the pile top, and the moment induced in the pile. For preliminary design purposes, we judge



the 14- and 16-inch-square piles should be capable of resisting a lateral load of about 10 kips per pile.

Additional lateral resistance can be developed by passive soil resistance acting against the vertical faces of pile caps and grade beams. An equivalent fluid weight (triangular distribution) of 300 pcf is preliminarily recommended for computing passive resistance against pile caps and grade beams for buildings with no below-grade parking levels. For structures with at least one below-grade level, we preliminarily recommend using a uniform passive pressure (rectangular distribution) of 1,400 psf. These values contain a factor of safety of at least 1.5. Because of the potential for soil disturbance, we recommend the top foot of soil be neglected when computing the embedment required to resist lateral loads unless confined by a concrete slab.

If a pile foundation system is selected, we preliminary recommend the basement floor slab be designed as a structural slab that is capable of resisting the hydrostatic uplift pressure associated with a design groundwater at Elevation 65 feet.

## **8.4 Permanent Basement Wall Design**

Basement walls should be designed to resist both static lateral earth pressures and lateral pressures caused by earthquakes. We preliminarily recommend basement walls at the site be designed for the more critical of the following criteria:

- at-rest equivalent fluid weight of 60 pcf above the design groundwater level (Elevation 65 feet) and 90 pcf below, plus a traffic increment where the wall is adjacent to streets. The traffic increment consists of a uniform (rectangular distribution) lateral pressure of 50 psf applied to the upper 10 feet of the wall.
- active pressure of 35 pcf above the design groundwater elevation, 80 pcf below, plus a seismic increment of 12 times the height of the wall in psf, where the height is in feet.

To protect against moisture migration, below-grade walls should be waterproofed and water stops should be placed at all construction joints.

## 8.5 Site Preparation and Excavation

Medium stiff to stiff clay and silt will generally be exposed at the bottom of the proposed 13- and 25-foot-deep excavations, except near CPT-2 and CPT-3, where a medium dense to dense sand and silty sand are likely to be encountered. If soft clay or silt, or loose sand or gravel is exposed at the bottom of the excavation, it should be removed and the overexcavated area should be backfilled with lean concrete to the planned bottom of the mat foundation. Following excavation for the basement, the soil exposed at the subgrade should be rolled with a small, non-vibratory, smooth-drum roller to produce a smooth, non-yielding surface for application of the waterproofing (if applicable). Alternatively, the subgrade can be cut with a smooth bucket and hand-cleaned.

If waterproofing is required, we preliminarily recommend it be placed directly on the soil subgrade and covered by a mud slab (4- to 6-inch-thick layer of lean concrete). In our experience, waterproofing membranes, such as Paraseal or Preprufe, are typically used for this application. We recommend a waterproofing consultant be retained to determine the most appropriate system for this project. The mud slab should reduce the potential for disturbing the underlying subgrade and should protect the waterproofing material from damage during construction. The mud slab should also provide a firm, smooth surface on which to place the reinforcing steel for the mat foundation or the pile-supported floor slab.

## 8.6 Seismic Design

If the building is designed in accordance with the 1997 UBC, the requirements for Zone 4 should be used as a minimum. Based on the soil conditions at the site, we recommend a soil profile type of  $S_D$ . Near-source factors of 1.04 and 1.04 for  $N_a$  and  $N_v$ , respectively, should be used.

Alternatively, a site-specific probabilistic seismic hazard analysis (PSHA) can be performed to develop acceleration response spectra for the design of new structures at the site.

## **8.7 Temporary Dewatering**

The groundwater should be drawn-down to a depth of at least three feet below the bottom of the proposed excavation and maintained there until sufficient building weight and/or tiedown capacity is available to resist the hydrostatic uplift pressure of the groundwater once it is allowed to rise to its normal elevation.

Either an active or passive dewatering system can be used at the site. Certain issues apply to each system as discussed in Section 7.3. The owner and general contractor should determine which system is most cost-effective for this project.

The dewatering system should be designed and implemented by an experienced dewatering contractor. We should have the opportunity to check the dewatering system design proposed by the contractor prior to installation.

Where dewatering wells or piping penetrate the proposed pile-supported floor slab or mat foundation, special attention should be given to properly sealing and waterproofing the penetrations when they are abandoned or removed from site. Groundwater should be prevented from seeping through the penetrations and into the below-grade parking levels.

## **8.8 Shoring Design**

The proposed excavations for the below-grade parking structures will need to be shored. Recommended lateral pressures for the design of a conventional soldier pile and lagging system with tiebacks are presented on Figure 5. In calculating these design pressures, we assumed an active dewatering system will be used to draw down the groundwater at the site and surrounding area to a depth of at least three feet below the bottom of the excavation. If perched water is encountered during excavation, we assume it will seep through the lagging and thus preventing hydrostatic pressures from acting on the shoring.

Figure 6 presents the recommended lateral pressures for a sheetpile or soil-cement column wall. The pressures presented on Figure 6 assume a passive dewatering system will be used at the site and hydrostatic pressures are allowed to build up against the exterior edge of the shoring.

Irrespective of which shoring system is used, the penetration of the soldier piles or soil-cement columns must be sufficient to insure lateral stability and resist the downward loading of tiebacks. For lateral resistance below the bottom of the excavation, we recommend using a uniform pressure of 1,400 psf to compute passive pressure. Passive pressures can be assumed to act over an area of two soldier pile widths or the embedded face of sheetpile or soil-cement column wall, depending upon which shoring system is selected. The passive pressure value includes a factor of safety of about 1.5. Vertical loads can be resisted by skin friction along the portion of the soldier piles below the excavation. The recommended allowable skin friction value is 500 psf. End bearing should be neglected. This value includes a factor of safety of about 1.5.

The contractor should be responsible for determining the type and size of tiebacks required to resist the pressures presented on Figures 5 and 6. The contractor should determine their lengths based on his/her experience and equipment. The computed bond length should be confirmed by a performance- and proof-testing program under the observation of an engineer with experience in this type of work. If any tieback fails to meet the performance- and proof-testing requirements, additional tiebacks should be added to compensate for the deficiency, as directed by the shoring designer. After testing, the tiebacks should be loaded to the design load (less if specified by the shoring designer) and locked off. The tiebacks should be checked 24 hours after initial lock off to ensure that stress relaxation has not occurred. The bottom of the excavation should not extend more than two feet below a row of unsecured tiebacks.

Because of the critical nature of the shoring for this project, we should review the shoring design prior to bidding of the documents for construction.



## **9.0 FUTURE GEOTECHNICAL INVESTIGATION**

The conclusions and preliminary recommendations presented in this report are the result of limited engineering studies based on our interpretation of the existing geotechnical conditions and available subsurface data. When individual components of the project site are ready for design-level evaluation, a design-level geotechnical study will be required to further evaluate the subsurface conditions at the proposed building and parking structure sites and to develop final design recommendations.



## REFERENCES

California Division of Mines and Geology; *Probabilistic Seismic Hazard Assessment for the State of California*; DMG Open-File Report 96-08; 1996.

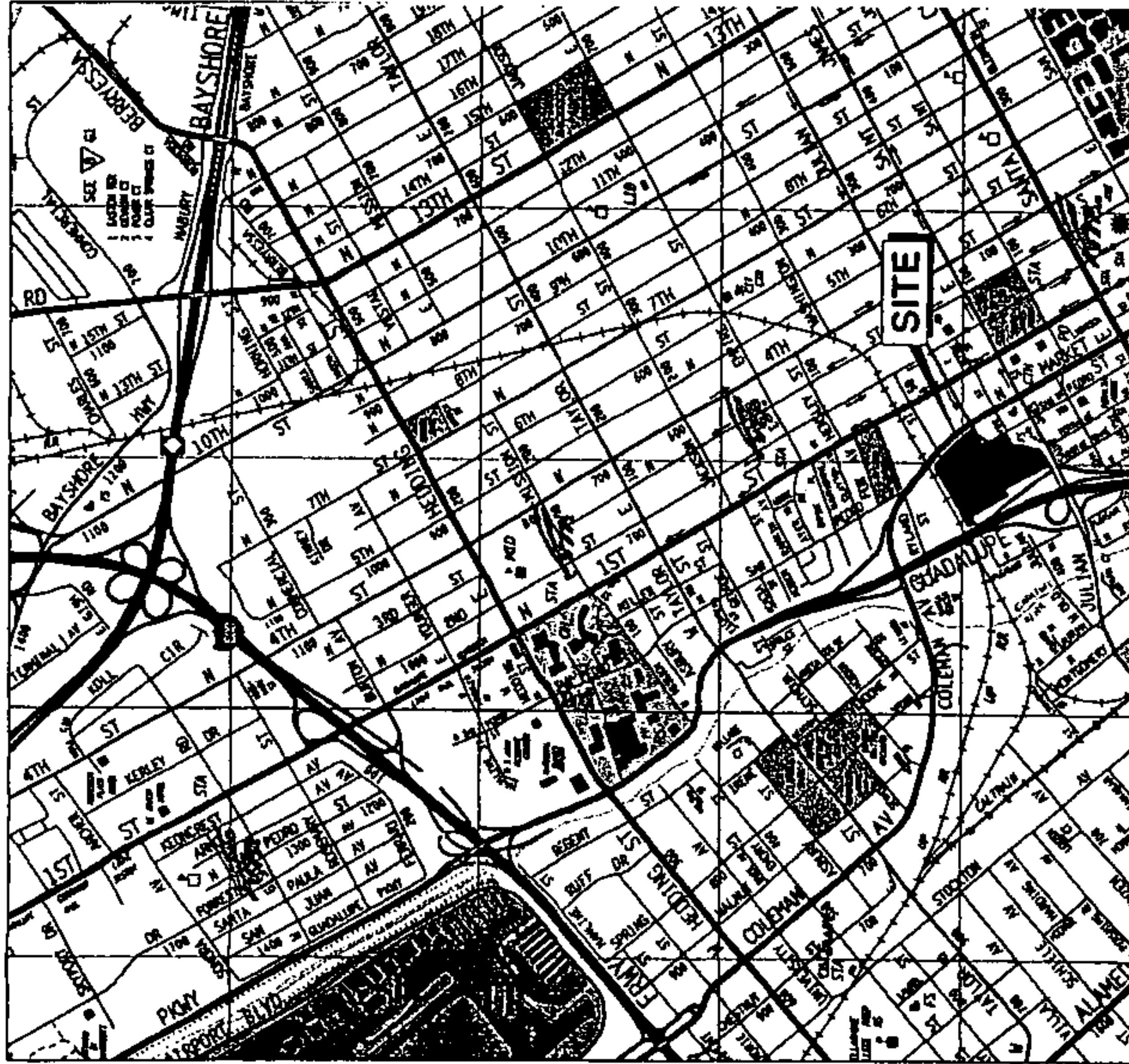
ICBO; *Uniform Building Code, Volume 2, Structural Engineering Design Provisions*; 1997.

Schlocker, Julius; *Geology of the San Francisco North Quadrangle, California*; 1974.

United States Geologic Survey; *Map Showing Areas of Exposed Bedrock, Contours on Bedrock Surface, and Landslides in the San Francisco North Quadrangle, San Francisco and Marin Counties, California*; 1956.

Working Group on California Earthquake Probabilities, *Earthquake Probabilities in the San Francisco Bay Region: 2000 to 2030 — A Summary of Findings*, Open File Report 99-517, 1999.

**FIGURES**



Base map: The Thomas Guide  
Santa Clara County  
1999

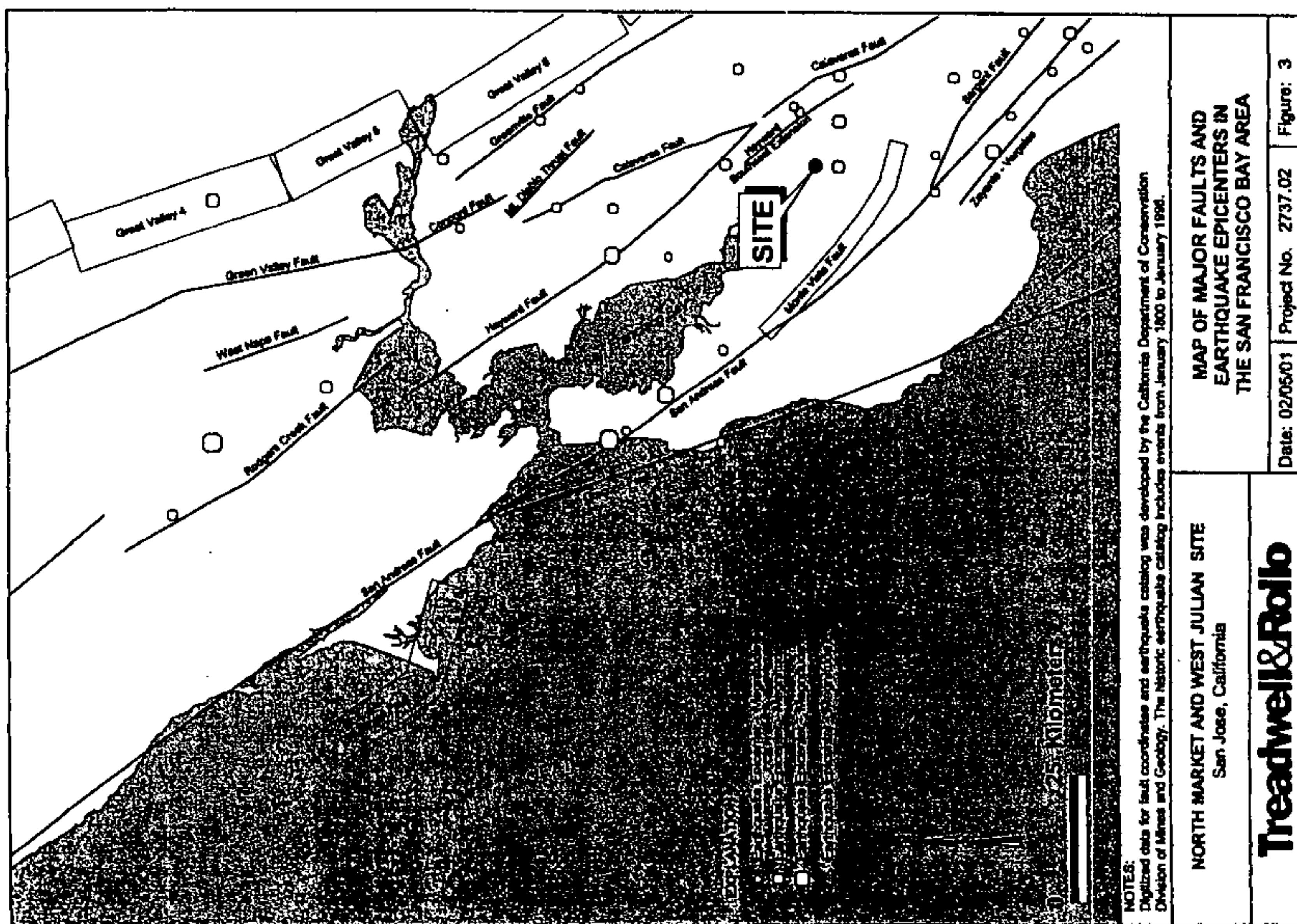
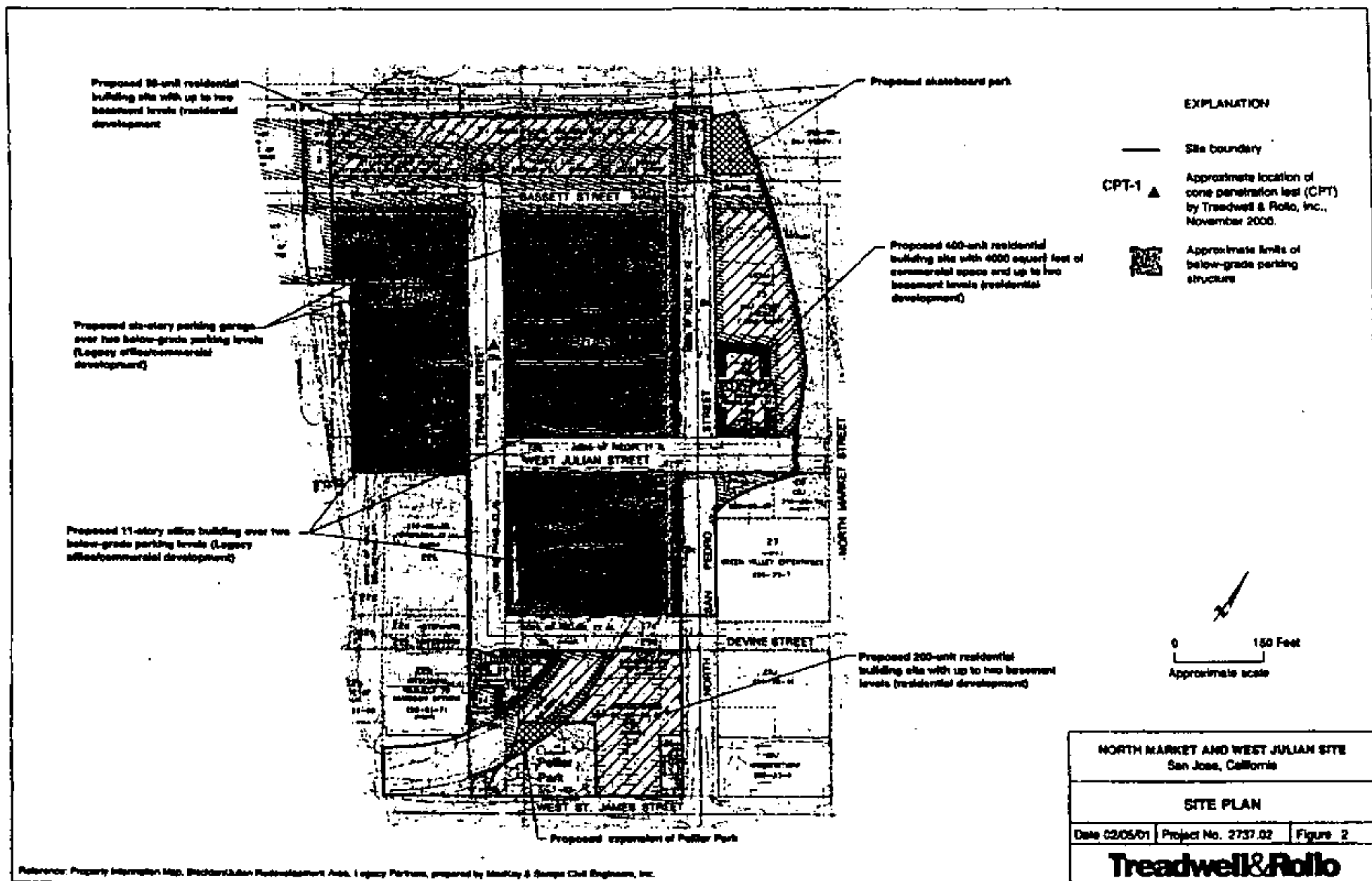
No scale

**NORTH MARKET AND WEST JULIAN SITE**  
San Jose, California

**SITE LOCATION MAP**

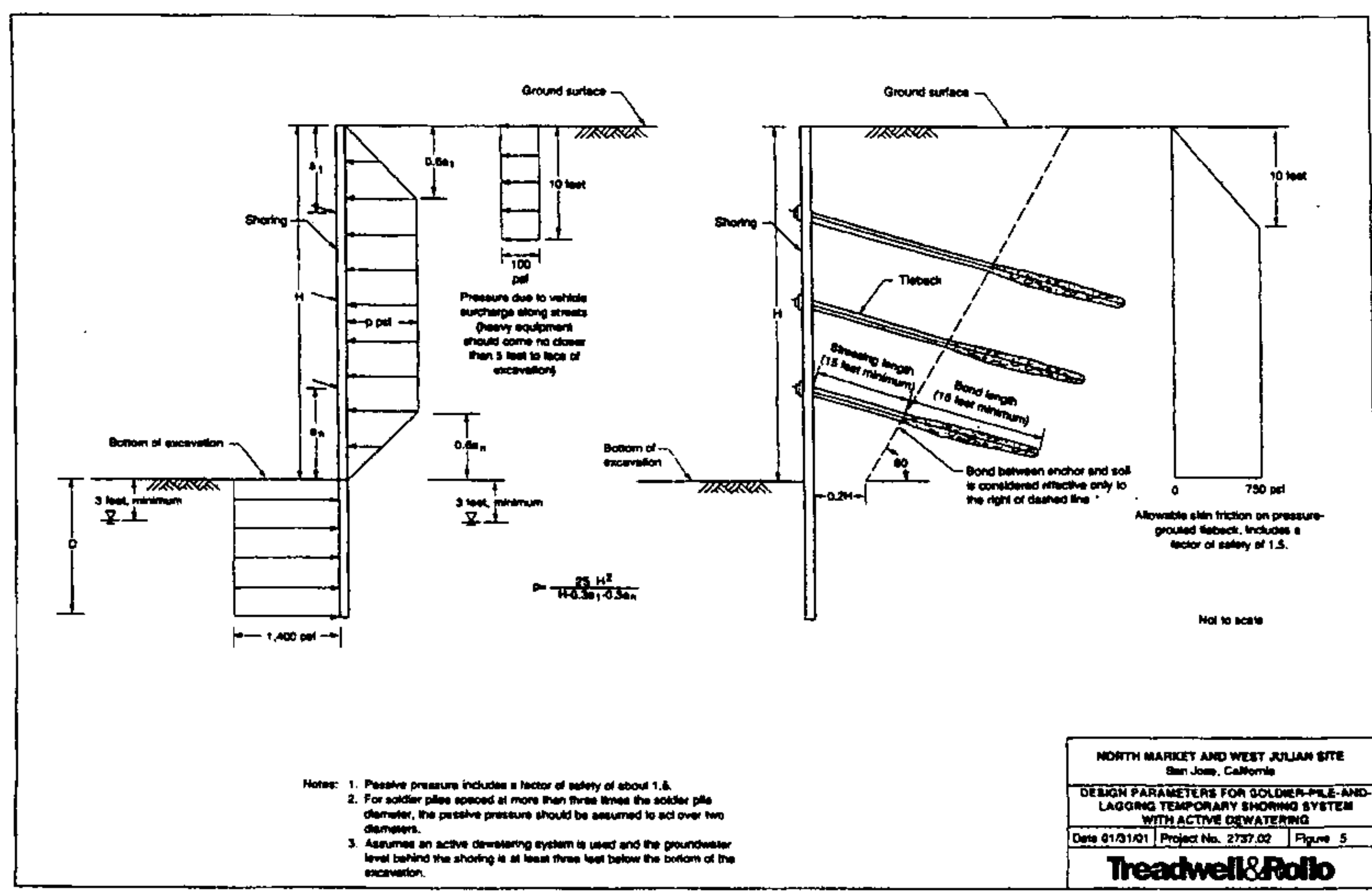
**Treadwell&Rollo**

Date 12/13/00 Project No. 2737.02 Figure 1

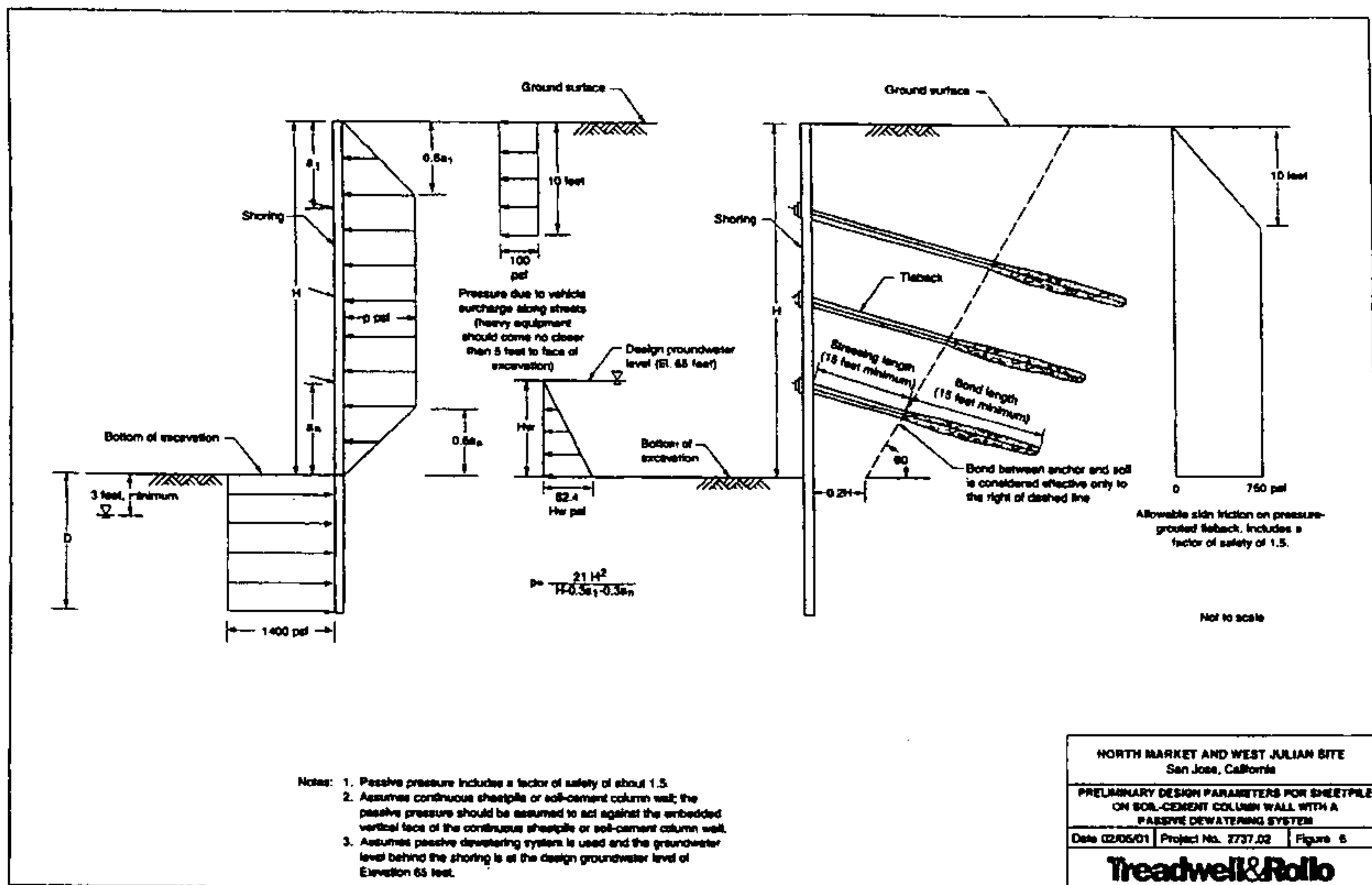




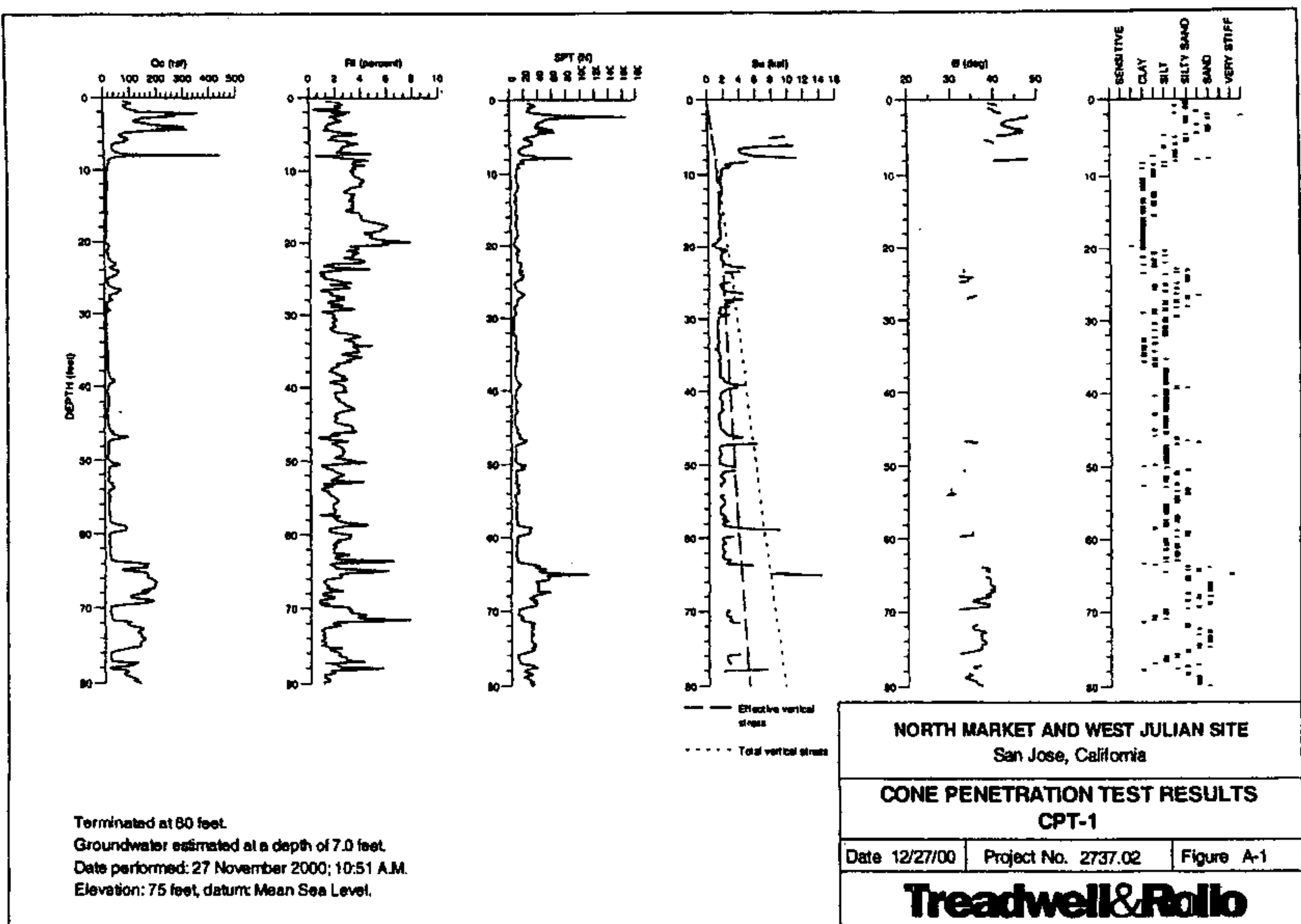
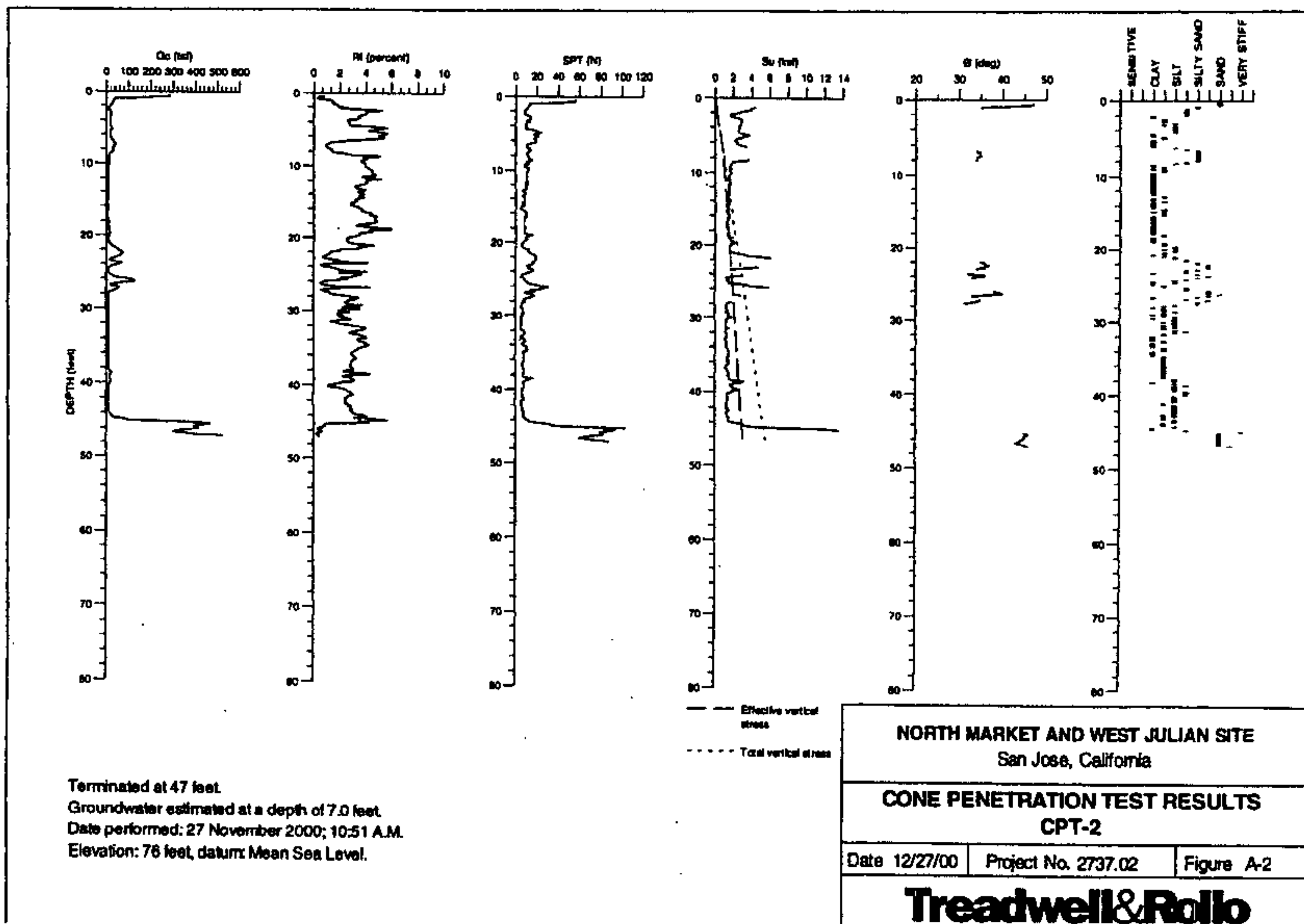
MODIFIED MERCALLI INTENSITY SCALE		
NORTH MARKET AND WEST JULIAN SITE San Jose, California		Date 12/14/00 Project No. 2737.02 Figure 4
Treadwell&Rollo		
I	Not felt by people, except under especially favorable circumstances. However, dizziness or nausea may be experienced. Sometimes birds and animals are uneasy or disturbed. Trees, structures, liquids and bodies of water may sway gently, and doors may swing very slowly.	
II	Felt indoors by a few people, especially on upper floors of multi-story buildings, and by sensitive or nervous persons. As in Grade I, birds and animals are disturbed, and trees, structures, liquids and bodies of water may sway. Hanging objects swing, especially if they are delicately suspended.	
III	Felt indoors by several people, usually as a rapid vibration that may not be recognized as an earthquake at first. Vibration is similar to that of a light, or lightly loaded truck, or heavy trucks some distance away. Duration may be estimated in some cases. Movements may be appreciable on upper levels of tall structures. Standing motor cars may rock slightly.	
IV	Felt indoors by many, outdoors by a few. Awakens a few individuals, particularly light sleepers, but frightens no one except those apprehensive from previous experience. Vibration like that due to passing of heavy, or heavily loaded trucks. Sensation like a heavy body striking building, or the falling of heavy objects inside. Dishes, windows and doors rattle; glassware and crockery clink and clish. Walls and house frames creak, especially if intensity is in the upper range of this grade. Hanging objects often swing. Liquids in open vessels are disturbed slightly. Stationary automobiles rock noticeably.	
V	Felt indoors by practically everyone, outdoors by most people. Direction can often be estimated by those outdoors. Awakens many, or most sleepers. Frightens a few people, with slight excitement; some persons run outdoors. Buildings tremble throughout. Dishes and glassware break to some extent. Windows crack in some cases, but not generally. Vases and small or unstable objects overturn in many instances, and a few tall, hanging objects and doors swing generally or considerably. Pictures knock against walls, or swing out of place. Doors and shutters open or close abruptly. Pendulum clocks stop, or run fast or slow. Small objects move, and furnishings may shift to a slight extent. Small amounts of liquids spill from well-filled open containers. Trees and bushes shake slightly.	
VI	Felt by everyone, indoors and outdoors. Awakens all sleepers. Frightens many people; general excitement, and some persons run outdoors. Persons move unsteadily. Trees and bushes shake slightly to moderately. Liquids are set in strong motion. Small bells in churches and schools ring. Poorly built buildings may be damaged. Plaster falls in small amounts. Other plaster cracks somewhat. Many dishes and glasses, and a few windows break. Kneadables, books and pictures fall. Furniture overturns in many instances. Heavy furnishings move.	
VII	Frightens everyone. General alarm, and everyone runs outdoors. People find it difficult to stand. Persons driving cars notice shaking. Trees and bushes shake moderately to strongly. Waves form on ponds, lakes and streams. Water is muddled. Gravel or sand stream beds cave in. Large church bells ring. Suspended objects quiver. Damage is negligible in buildings of good design and construction; slight to moderate in well-built ordinary buildings; considerable in poorly built or badly designed buildings; adobe houses, old walls (especially where laid up without mortar), spires, etc. Plaster and some stucco fall. Many windows and some furniture break. Loose brickwork and tiles shake down. Weak chimney break at the roofline. Cornices fall from towers and high buildings. Bricks and stones are dislodged. Heavy furniture overturns. Concrete irrigation ditches are considerably damaged.	
VIII	General fright, and alarm approaches panic. Persons driving cars are disturbed. Trees shake strongly, and branches and trunks break off (especially palm trees). Sand and mud erupts in small amounts. Flow of springs and wells is temporarily and sometimes permanently changed. Dry wells run low. Temperatures of spring and well waters varies. Damage slight in brick structures built especially to withstand earthquakes; considerable in ordinary substantial buildings, with some partial collapse; heavy in some wooden houses, with some furniture down. Panels and break away in frame structures. Decayed pilings break off. Walls fall. Solid stone walls crack and break seriously. Wet grounds and steep slopes crack to some extent. Chimneys, columns, monuments and factory stacks and towers twist and fall. Very heavy furniture moves conspicuously or overturns.	
IX	Panic is general. Ground cracks conspicuously. Damage is considerable in masonry structures built especially to withstand earthquakes; great in other masonry buildings - some collapse in large part. Some wood frame houses built especially to withstand earthquakes are thrown out of plumb, others are shifted wholly off foundations. Reservoirs are seriously damaged and underground pipes sometimes break.	
X	Panic is general. Ground, especially when loose and wet, cracks up to widths of several inches. Features up to a yard in width run parallel to canal and stream banks. Landsliding is considerable from river banks and steep coasts. Sand and mud shifts horizontally on beaches and flat land. Water level changes in wells. Water is thrown on banks of canals, lakes, rivers, etc. Dams, dikes, embankments are seriously damaged. Well-built wooden structures and bridges are severely damaged, and some collapse. Dangerous cracks develop in excellent brick walls. Most masonry and frame structures, and their foundations are destroyed. Railroad rails bend slightly. Pipe lines buried in earth tear apart or are crushed endwise. Open cracks and broad wavy beds open in cement pavements and asphalt road surfaces.	
XI	Panic is general. Disturbances in ground are many and widespread, varying with the ground material. Broad fissures, earth slumps, and land slips develop in soft, wet ground. Water charged with sand and mud is ejected in large amounts. Sea waves of significant magnitude may develop. Damage is severe to wood frame structures, especially near shock centers, great to dams, dikes and embankments, even at long distances. Few if any masonry structures remain standing. Supporting piers or pillars of large, well-built bridges are wrecked. Wooden bridges that "give" are less affected. Railroad rails bend greatly and some thrust endwise. Pipe lines buried in earth are put completely out of service.	
XII	Panic is general. Damage is total, and practically all works of construction are damaged greatly or destroyed. Disturbances in the ground are great and varied, and numerous shearing cracks develop. Landslides, rock falls, and slumps in river banks are numerous and extensive. Large rock masses are wrenched loose and torn off. Fault slips develop in firm rock, and horizontal and vertical offset displacements are notable. Water channels, both surface and underground, are disturbed and modified greatly. Lakes are dammed, new waterfalls are produced, rivers are deflected, etc. Surface waves are seen on ground surface. Lines of sight and level are distorted. Objects are thrown upward into the air.	

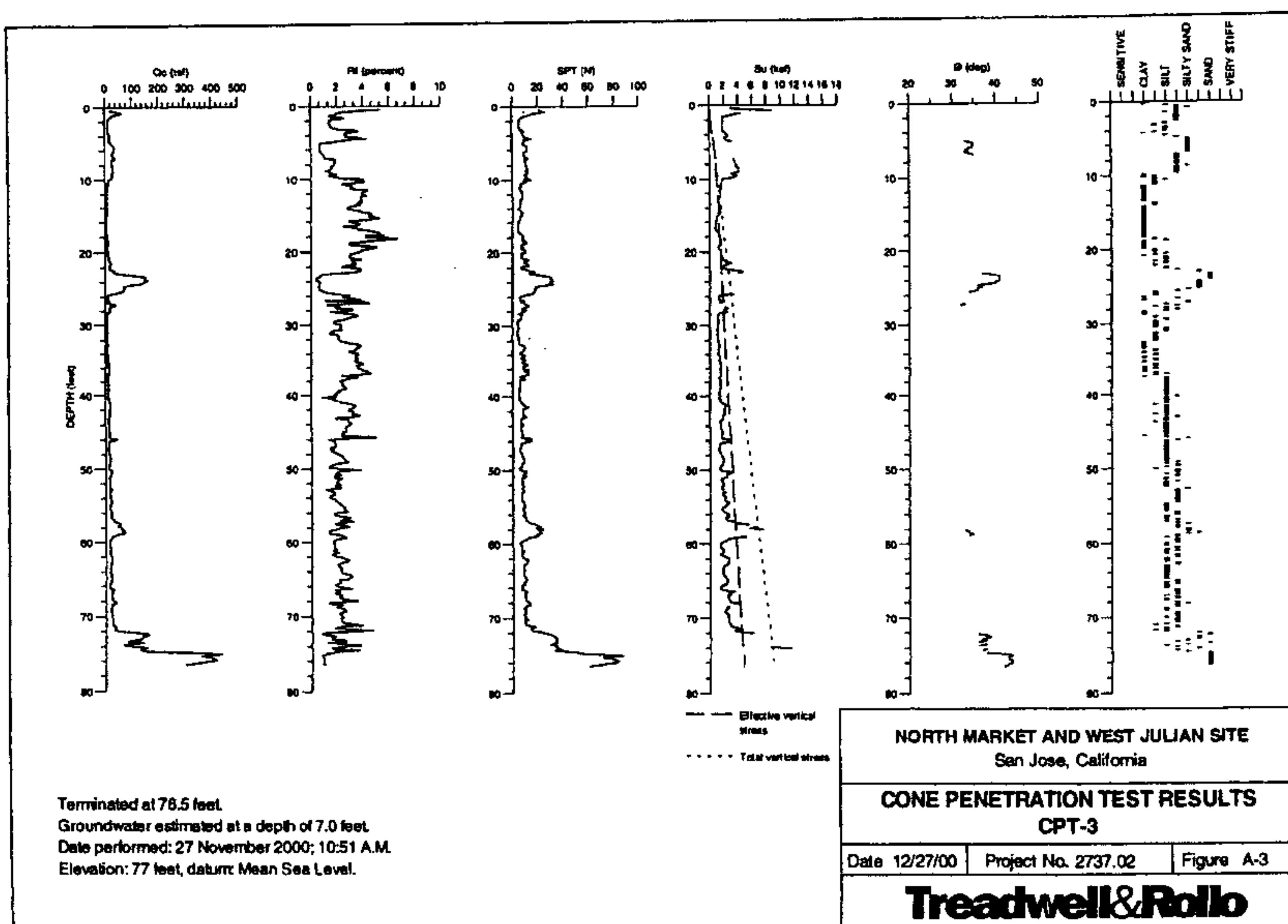
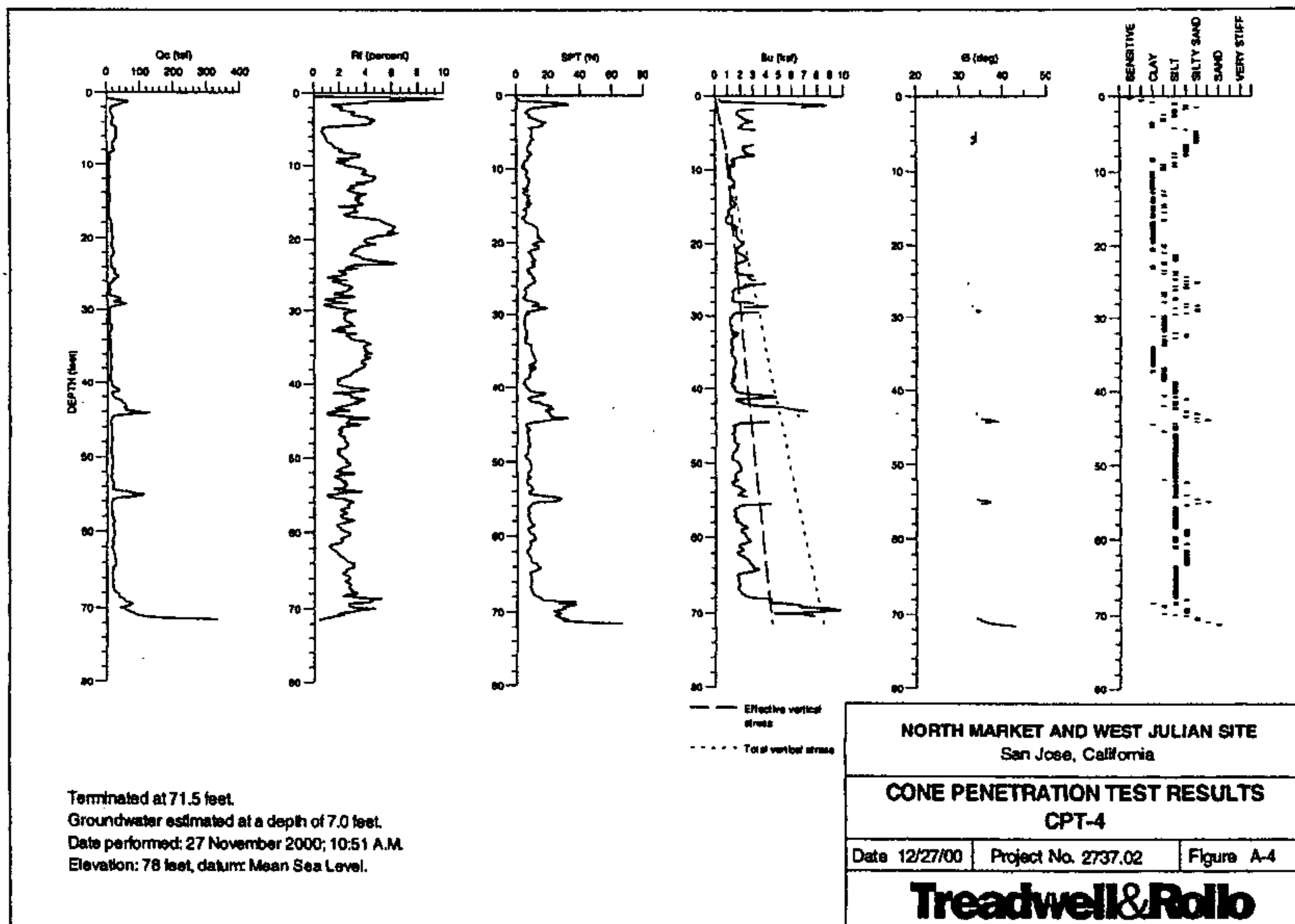


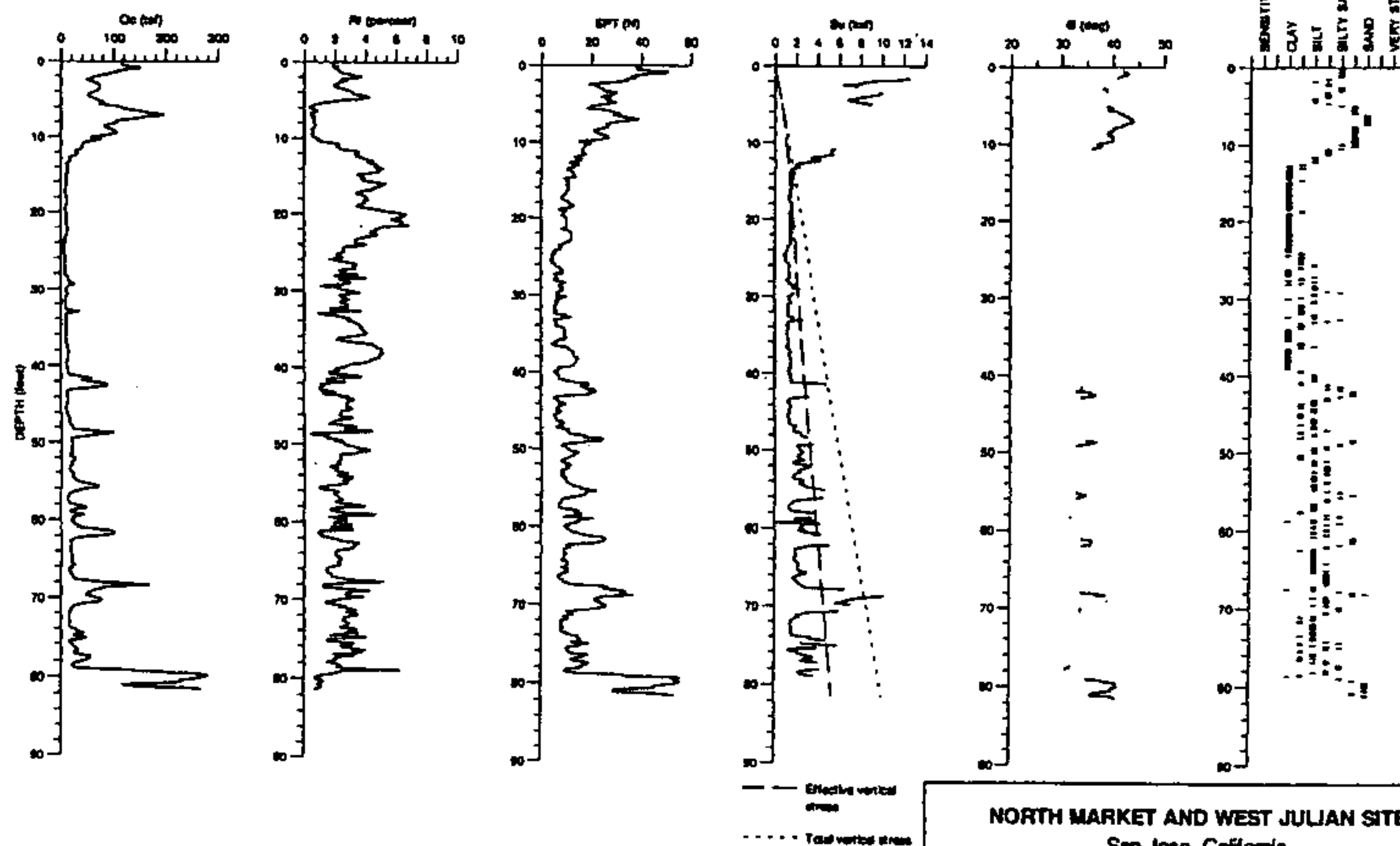
**APPENDIX A**  
**Logs of Cone Penetration Tests**











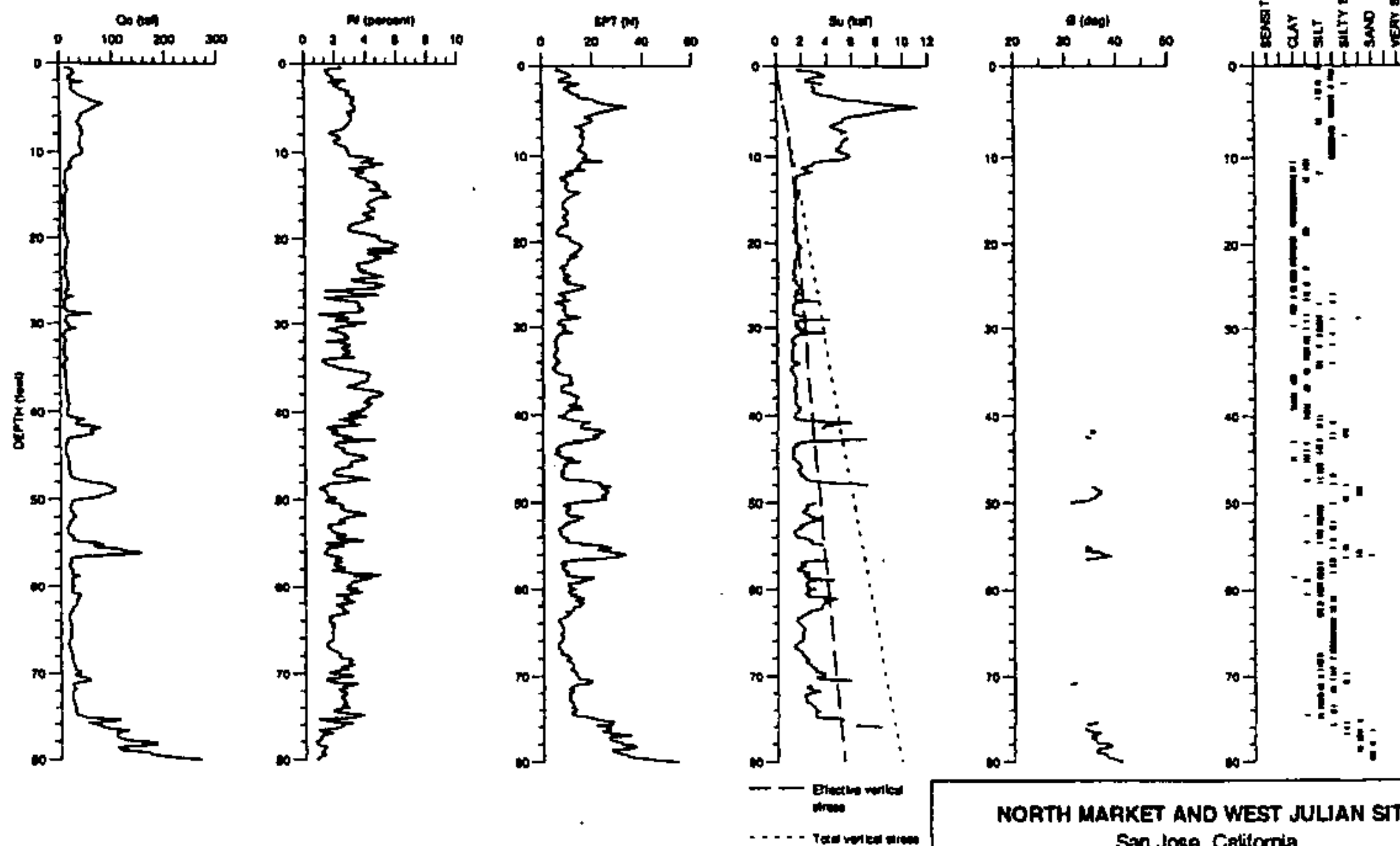
Terminated at 81.5 feet.  
 Groundwater estimated at a depth of 7.0 feet.  
 Date performed: 27 November 2000; 10:51 A.M.  
 Elevation: 78 feet, datum: Mean Sea Level.

NORTH MARKET AND WEST JULIAN SITE  
 San Jose, California

### CONE PENETRATION TEST RESULTS CPT-6

Date 12/27/00 Project No. 2737.02 Figure A-6

**Treadwell&Rollo**



Terminated at 80 feet.  
 Groundwater estimated at a depth of 7.0 feet.  
 Date performed: 27 November 2000; 10:51 A.M.  
 Elevation: 78 feet, datum: Mean Sea Level.

NORTH MARKET AND WEST JULIAN SITE  
 San Jose, California

### CONE PENETRATION TEST RESULTS CPT-5

Date 12/27/00 Project No. 2737.02 Figure A-5

**Treadwell&Rollo**

# Treadwell&Rollo

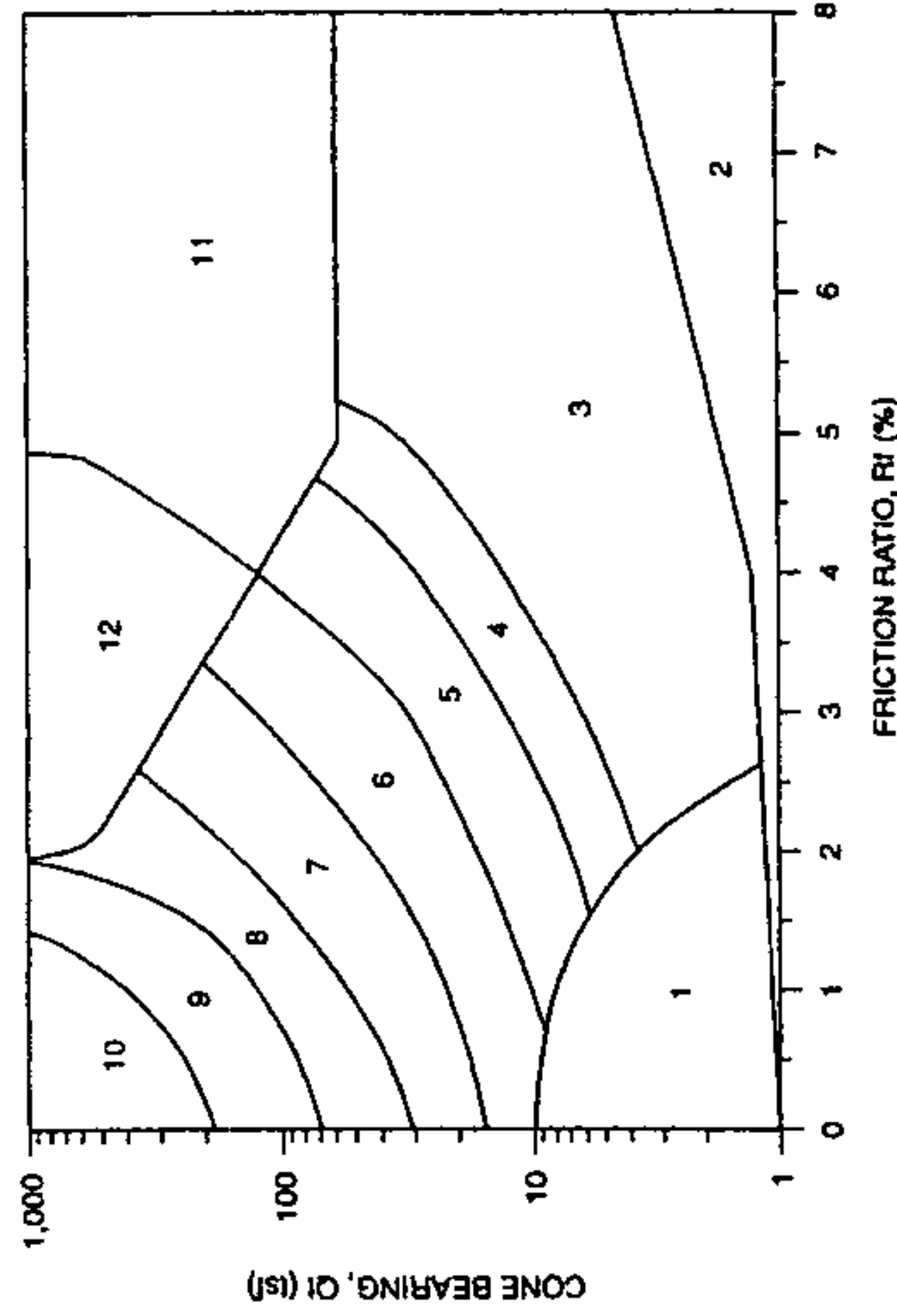
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ZONE	Q/N	SOIL BEHAVIOR TYPE
1	2	Sensitive Fine-Grained
2	1	Organic Material
3	1	CLAY
4	1.5	SILTY CLAY to CLAY
5	2	CLAYEY SILT to SILTY CLAY
6	2.5	SANDY SILT to CLAYEY SILT
7	3	SILTY SAND to SANDY SILT
8	4	SAND to SILTY SAND
9	5	SAND
10	6	GRAVELLY SAND to SAND
11	1	Very Stiff Fine-Grained (*)
12	2	SAND to CLAYEY SAND (*)

(\*) Overconsolidated or Cemented

Q<sub>t</sub> = Tip Bearing

F<sub>s</sub> = Sleeve Friction

R<sub>f</sub> = F<sub>s</sub>/Q<sub>t</sub> x 100 = Friction Ratio

Note: Testing performed in accordance with ASTM D3441.

Reference: Robertson, 1990.

NORTH MARKET AND WEST JULIAN SITE  
San Jose, California

CLASSIFICATION CHART FOR  
CONE PENETRATION TESTS

Treadwell&Rollo

Date 12/27/00 Project No. 2737.02 Figure A-7