Appendix A

PRELIMINARY GEOTECHNICAL INVESTIGATION
MURPHY RANCH ROAD RESIDENTIAL DEVELOPMENT
Milpitas, California

Pardee Residential, LLC
San Diego, California

23 August 2004
Project No. 1270.06

Treadwell&Rollo
Environmental and Geotechnical Consultants

Treadwell&Rollo

23 August 2004
Project 1270.06

Mr. Shon E. Finch
Fairfield Residential, LLC
5510 Morehouse Drive, Suite 200
San Diego, California 92121

Subject: Preliminary Geotechnical Investigation
Murphy Ranch Road Residential Development
Milpitas, California

Dear Mr. Finch:

Enclosed are six copies of our preliminary geotechnical investigation report for the proposed Murphy Ranch Road residential development in Milpitas, California. The site is at the southwest corner of the intersection of Technology Drive and Murphy Ranch Road. This report was prepared in accordance with our proposal dated 9 August 2004.

The site currently consists of 21.7-acre vacant lot with disked soil at the ground surface. We understand plans are to construct multi-family residential buildings at the site; however, the types, number and locations of buildings have not yet been determined. We anticipate the most likely types of construction consist of low-rise, wood-framed residential structures, concrete podium structures with wood-framed construction above, and concrete parking structures.

Treadwell & Rollo drilled 13 borings to depths of 36-1/2 to 61-1/2 feet below the existing ground surface (bgs) in 1992 and 1995 for the previously proposed Octel development. Our interpretation of the subsurface conditions is based entirely on data from the previous investigation. We conclude the site is generally underlain by 8 to 10 feet of soft to medium stiff silt and low-plasticity clay with varying sand content, although this weak surficial layer is 12 to 18 feet thick at two locations. The surficial weak silt and clay is underlain by a combination of stiff to very stiff clay and silt and medium dense to very dense sand with varying silt content to the maximum depth explored. Groundwater was measured at depths between 10-1/2 and 14-1/2 feet bgs during our previous investigations.

The most appropriate foundation types for the proposed buildings will be controlled by the surficial weak silt and clay layer and the presence of potentially liquefiable sandy silt and silty sand layers beneath the site. We preliminarily conclude 3/4 to 2 inches of liquefaction-induced ground surface settlement will occur following a major earthquake on one of the nearby faults, with up to 3/4 inch of differential settlement over a horizontal distance of 50 feet. Foundations for the proposed buildings should be designed to limit differential settlement under static and seismic conditions to acceptable levels. We conclude the proposed buildings can generally be supported on stiffened shallow foundation systems. We preliminarily conclude low-rise

Treadwell & Rollo, Inc. *Environmental & Geotechnical Consultants*
501 14th Street, Third Floor, Oakland, CA 94612
Telephone (510) 874-4500 Facsimile (510) 874-4507

wood-framed structures can be supported on post-tensioned slabs-on-grade (P-T slabs) or well-reinforced mat foundations bearing in the weak surficial silt, and heavier podium or parking structures can be supported on grid foundation systems supported on compacted aggregate pier-reinforced soil.

Preliminary conclusions and recommendations regarding liquefaction hazard mitigation, foundation support, site preparation, slab-on-grade floors, concrete flatwork, and seismic design are presented herein. Prior to final design, the building types, layout, and the estimated column loads should be provided to allow us to plan our final investigation and refine our preliminary conclusions and recommendations based on building-specific information.

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

Sincerely yours,
TREADWELL & ROLLO, INC.

Andrew R. Blaisdell
Civil Engineer

REGISTERED PROFESSIONAL ENGINEER
ANDREW R. BLAISDELL
No. C 65158
Exp. 9/30/05
CIVIL
STATE OF CALIFORNIA

for Craig S. Shields
Geotechnical Engineer

REGISTERED PROFESSIONAL ENGINEER
CRAIG S. SHIELDS
GE2116
Ren. 3/31/05
GEOTECHNICAL
STATE OF CALIFORNIA

12700601.OAK

Enclosure

PRELIMINARY GEOTECHNICAL INVESTIGATION
MURPHY RANCH ROAD RESIDENTIAL DEVELOPMENT
Milpitas, California

Fairfield Residential, LLC
San Diego, California

23 August 2004
Project No. 1270.06

Treadwell&Rollo
Environmental and Geotechnical Consultants

Treadwell&Rollo

TABLE OF CONTENTS

1.0 INTRODUCTION 1

2.0 SCOPE OF SERVICES 1

3.0 FIELD INVESTIGATION AND LABORATORY TESTING 3

4.0 SITE AND SUBSURFACE CONDITIONS 4

5.0 SEISMIC CONSIDERATIONS 5
- 5.1 Regional Seismicity 5
- 5.2 Seismic Hazards 7
  - 5.2.1 Ground Rupture 8
  - 5.2.2 Strong Ground Shaking 8
  - 5.2.3 Cyclic Densification 8
  - 5.2.4 Liquefaction 8
  - 5.2.5 Lateral Spreading 10

6.0 PRELIMINARY CONCLUSIONS AND RECOMMENDATIONS 10
- 6.1 Foundations and Settlement 10
  - 6.1.1 CAP-Supported Footings 12
  - 6.1.2 Mats or Post-Tensioned Slabs-on-Grade 14
- 6.2 Site Preparation and Grading 16
  - 6.2.1 Special Grading Considerations 17
- 6.3 Concrete Slab-on-Grade Floors 19
- 6.4 Asphalt Concrete Pavement 21
- 6.5 Portland Cement Concrete Pavement 23
- 6.6 Concrete Flatwork 23
- 6.7 Seismic Design 23

7.0 ADDITIONAL GEOTECHNICAL SERVICES 24

8.0 LIMITATIONS 25

REFERENCES

FIGURES

APPENDIX A – Logs of Borings and Laboratory Test Results from Previous Investigations

DISTRIBUTION

LIST OF FIGURES

Figure 1	Site Location Map
Figure 2	Site Plan
Figure 3	Map of Major Faults and Earthquakes Epicenters in the San Francisco Bay Area
Figure 4	Modified Mercalli Intensity Scale

Treadwell&Rollo

PRELIMINARY GEOTECHNICAL INVESTIGATION
MURPHY RANCH ROAD RESIDENTIAL DEVELOPMENT
Milpitas, California

1.0 INTRODUCTION

This report presents the results of the preliminary geotechnical investigation performed by Treadwell & Rollo, Inc. for the proposed Murphy Ranch Road residential development in Milpitas, California. The site is at the southwest corner of the intersection of Technology Drive and Murphy Ranch Road, as shown on the Site Location Map, Figure 1.

The project site encompasses an area of 21.7 acres between Murphy Ranch Road and Coyote Creek, south of Highway 237, as shown on the Site Plan, Figure 2. The site is currently a relatively level, vacant disked field that has been used for agriculture in the past. We understand plans are to construct multi-family residential buildings at the site. However, building types, sizes, and locations have not yet been determined. We understand the buildings will likely be low-rise, wood-framed structures, possibly above a one-story concrete podium. All of the buildings will have slab-on-grade floors. It is also possible a concrete parking garage may be constructed on the site. Some of the buildings may have partial or full basements.

2.0 SCOPE OF SERVICES

Our geotechnical report was prepared in accordance with our proposal dated 9 August 2004. The purpose of our preliminary investigation was to evaluate soil and groundwater conditions and provide preliminary geotechnical recommendations for foundation design. We performed a geotechnical investigation for Phases II and III of the Octel Communications Corporation development previously proposed for the subject site and presented the results in our report dated 13 November 1995. We used the results of our previous investigation to develop the preliminary conclusions and recommendations presented herein; no additional subsurface exploration was

performed. Based on the results of our previous field investigation and laboratory testing and our engineering analyses, we have developed preliminary conclusions and recommendations regarding:

- soil and groundwater conditions at the site
- appropriate foundation type(s) for the proposed buildings
- preliminary design criteria for the recommended foundation type(s), including vertical and lateral capacities
- estimated settlement under new fill and foundation loads, including total and differential settlement
- site grading, including criteria for fill quality and compaction
- subgrade preparation for slab-on-grade floors
- flexible (asphalt concrete) and rigid (Portland cement concrete) pavement sections
- site seismicity and seismic hazards, including liquefaction potential
- 2001 California Building Code soil profile type and near-source factors
- construction considerations.

The California State Geological Survey (CGS) has prepared a map titled *State of California Seismic Hazard Zones, Milpitas Quadrangle, Official Map* (Santa Clara Valley portion only), dated 19 April 2004. This map was prepared in accordance with the Seismic Hazards Mapping Act of 1990. The site is within one of the designated liquefaction hazard zones. The CGS has also recommended the content for site investigation reports within seismic hazard zones in the State of California Special Publication (SP) 117, titled *Guidelines for Evaluating and Mitigating Seismic Hazard Zones in California*, dated 13 March 1997. The liquefaction evaluation performed for this investigation was prepared in general accordance with the recommendations presented in SP 117.

3.0 FIELD INVESTIGATION AND LABORATORY TESTING

A total of 13 borings, designated as B-4 through B-6 (drilled in 1992) and B-18 through B-27 (drilled in 1995), were used to evaluate the subsurface conditions at the site. The approximate locations of the borings are shown on Figure 2. The borings were advanced to depths of 36-1/2 to 61-1/2 feet below the existing ground surface (bgs) using truck-mounted rotary-wash drilling equipment. During drilling, our field engineer logged the borings and retrieved representative samples of the soil encountered for further classification and laboratory testing. Logs of the borings as well as a soil classification chart are presented in Appendix A.

Soil samples were obtained using the following samplers:

- Sprague and Henwood (S&H) split-barrel sampler with a 3.0-inch outside diameter and 2.43-inch inside diameter, lined with brass tubes
- Standard Penetration Test (SPT) split-barrel sampler with a 2.0-inch outside diameter and 1.5-inch inside diameter (no liners).

The S&H and SPT samplers were driven with a 140-pound safety hammer (operated with a rope-and-cathead pulley system) falling about 30 inches per drop. The blow counts required to drive the S&H sampler the final 12 inches of an 18-inch drive were converted to SPT N-values using a conversion factor of 0.6. The converted SPT N-values are shown on the boring logs. Where the SPT sampler was used, the actual blow counts are shown on the boring logs. After completion, the borings were backfilled with neat cement grout.

The soil samples were re-examined to confirm field classification and representative samples were tested to determine moisture content, dry density, particle size, Atterberg limits (plasticity), strength, consolidation and shrink-swell properties, and resistance value (R-value). The laboratory results are presented in Appendix A.

4.0 SITE AND SUBSURFACE CONDITIONS

The 21.7-acre site is bordered by Murphy Ranch Road to the east, Coyote Creek to the west, a Hetch Hetchy water easement and the Octel development to the south, and a pump station to the north. The site is currently vacant and appears to have been disked recently. The western edge of the site is bordered by an earthen levee that confines the east side of Coyote Creek. The top of the levee is about 10 feet above the ground surface of the project area.

The results of our previous investigations indicate the site is generally underlain by 8 to 10 feet of soft to medium stiff silt and low-plasticity clay with varying sand content, herein referred to as the weak silt and clay layer. The weak silt and clay layer is thicker at the locations of borings B-20 and B-22, where it extends to depths of 12 and 18 feet bgs, respectively. At a few of our boring locations, the upper layer was stiff to hard. The surficial weak silt and clay was generally dry to moist within a few feet of the ground surface at the time of our previous investigation and moist below three feet bgs. The weak silt and clay deposit is underlain by very stiff to hard clay extending to depths of 18 to 33 feet bgs. At many of the boring locations, the very stiff to hard clay layer grades to stiff to very stiff sandy silt or medium dense silty sand between depths of 18 and 28 feet bgs. The medium dense silty sand and stiff to very stiff silt layers, where encountered, are generally about five feet thick. Beneath the majority of the site, these layers grade to dense to very dense sand with varying silt content, which extends to depths of at least 36-1/2 feet bgs. At the location of boring B-5, sand was not encountered, and stiff to very stiff silt and clay extended to a depth of 61-1/2 feet, the maximum depth explored.

Groundwater was measured at depths ranging from 10-1/2 to 14-1/2 feet bgs in our previous investigations. We anticipate the groundwater level fluctuates depending on seasonal variations, variations in the water level in Coyote Creek, and the amount of groundwater being pumped in the general vicinity.

5.0 SEISMIC CONSIDERATIONS

5.1 Regional Seismicity

The major active faults in the area are the Hayward, Calaveras, and San Andreas Faults. These and other faults of the region are shown on Figure 3. For each of the active faults within 50 km of the project site, the distance from the site and estimated maximum Moment magnitude[1] (California Division of Mines and Geology 1996) event are summarized in Table 1.

TABLE 1
Regional Faults and Seismicity

Fault Segment	Approximate Distance from Site (km)	Direction from Site	Maximum Magnitude
Hayward – South East Extension	5.5	Northeast	6.4
Hayward – Total	9.5	Northeast	7.1
Southern Hayward	9.5	Northeast	6.9
Northern Calaveras	11	East	7.0
Central Calaveras	11	East	6.6
Monte Vista	17	Southwest	6.8
San Andreas – 1906 Rupture	23	Southwest	7.9
San Andreas – Peninsula	23	Southwest	7.2
San Andreas – Santa Cruz Mnts.	28	South	7.2
Sargent	31	South	6.8
Southern Greenville	35	Northeast	6.9
Mount Diablo Thrust	37	North	6.7
Zayante-Vergeles	37	South	6.8
Central Greenville	38	Northeast	6.7
Great Valley – 6	39	Northeast	6.7
Great Valley – 7	40	Northeast	6.7
San Gregorio North	44	Southwest	7.3
Northern Hayward	47	Northwest	6.6
Northern Greenville	48	North	6.6

[1] 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.

Figure 3 also shows the earthquake epicenters for events with magnitude greater than 5.0 from January 1800 through January 1996. 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 in length. It had a maximum intensity of XI (MM), a M_w of about 7.9, and was felt 560 kilometers away in Oregon, Nevada, and Los Angeles. The most recent 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 the earthquake was in the Santa Cruz Mountains, approximately 42 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 ($M_w = 6.2$).

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.

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

Fault	Probability (percent)
Hayward-Rodgers Creek	32
San Andreas	21
Calaveras	18
San Gregorio	10
Concord-Green Valley	6
Greenville	6
Mount Diablo	4

5.2 Seismic Hazards

During a major earthquake on a segment of one of the nearby faults, strong shaking is expected to occur at the project site. Strong shaking during an earthquake can result in ground failure such as that associated with soil liquefaction[2], lateral spreading[3], and cyclic densification[4]. We used the results of the borings to evaluate the potential for these phenomena to occur at the project site. Our evaluation of site seismic hazards was performed in general accordance with the guidelines presented in SP 117.

---

2 Liquefaction is a transformation of soil from a solid to a liquefied state during which saturated soil temporarily loses strength resulting from the buildup of excess pore water pressure, especially during earthquake-induced cyclic loading. Soil susceptible to liquefaction includes loose to medium dense sand and gravel, low-plasticity silt, and some low-plasticity clay deposits.

3 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.

4 Cyclic densification is a phenomenon in which non-saturated, cohesionless soil is compacted by earthquake vibrations, causing ground-surface settlement.

5.2.1 Ground Rupture

Historically, ground surface displacements closely follow the trace of geologically young faults. The site is not within an Earthquake Fault Zone, as defined by the Alquist-Priolo Earthquake Fault Zoning Act, and no known active or potentially active faults exist on the site. We therefore conclude the risk of fault offset at the site from a known active fault is very low. In a seismically active area, the remote possibility exists for future faulting in areas where no faults previously existed; however, we conclude the risk of surface faulting and consequent secondary ground failure from previously unknown faults is also very low.

5.2.2 Strong Ground Shaking

The ground shaking intensity felt at the project site will depend on: 1) the size of the earthquake (magnitude), 2) the distance from the site to the fault source, 3) the directivity (focusing of earthquake energy along the fault in the direction of the rupture), and 4) subsurface conditions. The site is about 5.5 kilometers from the Southeast Extension of the Hayward Fault and about 9.5 kilometers from the main portion of the Hayward Fault. Therefore, the potential exists for a large earthquake to induce strong to very strong ground shaking at the site during the life of the project.

5.2.3 Cyclic Densification

Seismically induced compaction or cyclic densification of non-saturated sand (sand above the groundwater table) caused by earthquake vibrations may result in differential settlement. Based on the limited subsurface data, we believe the silty layers encountered above the groundwater table are sufficiently cohesive that the risk of cyclic densification is very low.

5.2.4 Liquefaction

When saturated, cohesionless soil liquefies, it experiences a temporary loss of shear strength due to a transient rise in excess pore pressure generated by strong ground motion. Flow failure, lateral spreading, differential settlement, loss of bearing strength, ground fissures and sand boils

are evidence of excess pore pressure generation and liquefaction. We evaluated liquefaction potential at the site in accordance with CSP 117 (CDMG, 1992), as described below.

SP 117 states that liquefaction analyses should be performed using subsurface information from rotary wash borings and/or CPTs. Therefore, we used the data from all of our previous borings for our liquefaction evaluation.

Our liquefaction analysis indicates there are layers of stiff to very stiff silt and sandy silt and medium dense silty sand between depths of 18 and 48 feet bgs that will liquefy following a large earthquake on a nearby fault. The results of our borings indicate the potentially liquefiable layers are typically 4 to 6 feet thick; however, none of the layers appear to be vertically or laterally continuous across the entire site.

We used the procedure outlined in the 1997 NCEER report to estimate liquefaction-induced settlement. To estimate the strain potential of any potentially liquefiable layers, we used the method developed by Tokimatsu and Seed (1984), which relates $(N_1)_{60,CS}$ values to strain potential. Using this analysis, we estimate the total liquefaction-induced settlement will generally be on the order of 3/4 to 2 inches at the boring locations. Some of the boring logs do not appear to have layers of potentially liquefiable soil. However, considering the generally low plasticity of the subsurface soil at the site, we anticipate ground surface settlement on the order of at least 1/2 inch should be expected at all locations due to post-liquefaction reconsolidation. Based on the available data, we estimate liquefaction-induced differential settlement will be on the order of 1/2 inch over a horizontal distance of 30 feet.

Some of the data used for our analysis was incomplete with respect to liquefaction analyses, including fines content and/or Atterberg limits information. Additional borings and/or CPTs will be necessary during our final investigation to refine the data used in our preliminary analyses and determine building-specific liquefaction settlement estimates.

5.2.5 Lateral Spreading

Lateral spreading occurs when a continuous layer of soil liquefies at depth and the soil layers above move toward an unsupported face, such as a shoreline slope, or in the direction of a regional slope or gradient. Based on the discontinuous nature and depth of the potentially liquefiable layers, we believe the potential for lateral spreading to occur at the project site is low.

6.0 PRELIMINARY CONCLUSIONS AND RECOMMENDATIONS

On the basis of the subsurface information collected during our previous investigations at the site, we conclude the proposed residential development can be constructed as planned. Preliminary conclusions and recommendations for foundation type and design, site preparation and grading, slab-on-grade floors, concrete flatwork, pavement design, and seismic design are presented in the remainder of the report.

6.1 Foundations and Settlement

We anticipate buildings will generally be constructed with floor slabs within a few feet of existing site grades. Based on the results of our previous investigations, we conclude shallow foundations for the proposed buildings would generally bear on weak silt and be underlain by at least 6 to 8 feet of weak material.

We anticipate the structures associated with the proposed residential development may include relatively light, low-rise, wood-framed structures and heavier structures such as podiums with wood-framed construction above or concrete parking garages. We considered several foundation systems for support of the potential structures, including spread footings with and without reinforced soil, stiffened shallow foundations, and deep foundations. The most appropriate foundation alternatives should address the weak surficial silt and clay as well as potential liquefaction-induced differential settlement. While deep foundations such as driven piles would provide adequate support for buildings at the site, we believe the structural loads for the potential building types will not be large enough to make this foundation alternative economically

feasible. In addition, deep liquefiable deposits would require that the upper 25 to 35 feet of the piles be ignored for frictional capacity. We preliminarily conclude the most appropriate and economical foundation types for buildings at this site consist of stiffened shallow foundations with and/or without reinforced soil, as described below.

We believe the most economical method of foundation support for low-rise wood-framed buildings will consist of stiffened shallow foundations such as post-tensioned slabs-on-grade (P-T slabs) or well-reinforced mat foundations designed using the relatively low bearing pressures presented later in this report. We estimate low-rise wood-framed buildings supported on P-T slabs or mat foundations designed using the preliminary allowable bearing pressures presented later in this report will settle about one inch under dead-plus-live loads, with up to 3/4 inch of differential settlement over a horizontal distance of 30 feet. We estimate each new foot of fill will induce approximately an additional 1/2 inch of settlement; however, this settlement is expected to occur immediately after placement of the fill and will not affect new structures. In addition, post-liquefaction reconsolidation is expected to cause 3/4 to 2 inches of settlement, with up to 1/2 inch of differential settlement over a distance of 30 feet. Therefore, we preliminarily estimate up to three inches of settlement may occur under low-rise wood-framed structures over the life of the project, with up to 1-1/4 inch of differential settlement over a distance of 30 feet.

We anticipate P-T slabs and mat foundations will not be economically feasible to spread column loads on the order of 300 to 700 kips, which would be typical for podium structures or concrete parking garages. Therefore, we preliminarily conclude podium structures or parking garages should be supported on shallow foundations bearing on soil improved by the installation of compacted aggregate piers (CAPs). CAP-supported footings are typically designed to limit foundation settlement to 3/4 inch or less under dead-plus-live loads and 1/2 inch of differential settlement over a distance of 30 feet. However, CAPs will not mitigate the potential differential settlement associated with liquefaction (1/2 inch over a distance of 30 feet) because the liquefiable layers will be below the bottom of the CAPs. We anticipate one inch of differential settlement between adjacent columns may be excessive for conventional shallow foundations.

Therefore, we preliminarily conclude CAP-supported footings should consist of a reinforced grid of footings and grade beams designed to limit differential settlement.

If the proposed development consists of a combination of low-rise wood-framed buildings and podium or parking structures, it may be more economical to support all of the buildings on CAP-supported footings. However, we anticipate the foundation cost savings associated with using CAPs below lightly loaded footings will be less because of the relatively low bearing pressures and the potential differential settlement resulting from post-liquefaction reconsolidation.

As discussed previously, the data used to perform our liquefaction analyses was incomplete in some cases, and the estimated differential settlement resulting from post-liquefaction reconsolidation may change based on additional exploration.

Preliminary recommendations for stiffened shallow foundations and CAP-supported footings are presented in the following subsections.

6.1.1 CAP-Supported Footings

For preliminary design, we recommend CAP-supported footings be bottomed at least 18 inches below the lowest adjacent soil subgrade; however, building code requirements may govern footing depths for buildings taller than three stories. CAPs are typically constructed by drilling a 30- to 36-inch-diameter shaft and replacing the excavated soil with compacted aggregate fill. The aggregate generally consists of clean, open-graded crushed rock below the water table and Class 2 aggregate base above the water table. The aggregate is compacted in approximately 12-inch-thick lifts using a modified hydraulic hammer mounted on an excavator. Compacted aggregate piers develop vertical foundation support through a combination of frictional resistance along the shaft of the pier and improvement of the surrounding soil matrix, allowing use of significantly larger bearing capacities than feasible with the unimproved soil. CAPs can also be designed to resist transient uplift loads by installing steel rods in the center of the pier;

the rod is attached to a flat plate at the base of the pier. Because the foundations are typically designed as footings supported on improved soil, lateral loads are typically resisted through a combination of passive pressure on the face of the footings and friction along the base of the footings. The frictional resistance is typically larger than for a footing supported on unimproved ground because of the presence of the compacted aggregate.

Compacted aggregate piers are typically constructed through a design-build contract. Locally, the most common type of CAPs are Geopiers, installed by the Geopier Foundation Company of Northern California. We anticipate any equivalent system could also be used, although we should review the proposed design prior to incorporation into the project plans.

Because the piers are designed and installed under a design-build contract, we have not analyzed this foundation alternative and cannot provide specific recommendations regarding spacing, costs, vertical and lateral capacities, or estimated settlement. As a minimum, the CAPs should extend through the weak silt and clay layer encountered in our borings and extend at least a few feet into very stiff clay. Therefore, we estimate the CAPs will be about 10 to 20 feet long; the approximate length will be refined once building footprints have been established. We anticipate footings supported on the CAPs can be designed for allowable dead-plus-live load bearing capacities on the order of 5,500 to 6,500 psf, which could significantly reduce the footing dimensions. The design capacity of the CAPs should be verified by at least one load test in compression and one test in tension, if uplift elements are used. We should review the design prior to construction.

As discussed previously, the combination of static settlement and liquefaction-induced settlement is anticipated to be about 2 to 2-3/4 inches of total settlement, with up to one inch of differential settlement over a distance of 30 feet. Therefore, we preliminarily recommend the foundation system be designed as a stiffened grid. For structural design of the stiffened foundation system supported on CAPs, we preliminarily recommend a modulus of subgrade reaction of 140 kips per cubic foot (kcf) be used. This modulus is based on the preliminary allowable bearing pressure for CAP-supported footings and the expected static foundation

settlement. To model the potential differential settlement between columns under static loads, a modulus of 90 kcf should be used beneath the center column of a nine-column (3 by 3) grid and a modulus of 140 kcf should be used beneath the remaining eight columns. To model the potential liquefaction-induced differential settlement between columns, a modulus of 55 kcf should be used beneath the center column of a nine-column (3 by 3) grid and a modulus of 140 kcf should be used beneath the remaining eight columns. Based on our experience with similar foundation systems, we anticipate grade beam-connected footings will be 30 to 36 inches wide for podium or parking structures.

Lateral loads can be resisted by a combination of passive pressure acting on the vertical faces of the footings and friction along the base of the footings, and these can be used in combination without reduction. Passive resistance may be calculated using an equivalent fluid weight of 250 pcf. The upper one foot of soil should be ignored unless it is confined by slabs or pavement. Frictional resistance for CAP-supported footings will be determined by the CAP designer, but for estimating purposes, we anticipate a friction coefficient of 0.4 may be used..

6.1.2 Mats or Post-Tensioned Slabs-on-Grade

We preliminarily recommend low-rise, wood-framed be supported on stiffened shallow foundations consisting of well-reinforced concrete mats or P-T slabs. The edges of the foundation should be thickened such that the foundation edge is bottomed at least six inches below the adjacent exterior grade or three inches below the bottom of the vapor retarder, whichever is lower. We preliminarily conclude that both the mats and P-T slabs will be on the order of 12 to 15 inches thick. We preliminarily recommend the maximum bearing pressure beneath mats and P-T slabs not exceed 1,000 to 1,500 psf under dead-plus-live load conditions. The allowable bearing pressures may be increased by one-third for total loads, including wind and seismic. We anticipate the average bearing pressures beneath low-rise wood-framed structures will be significantly less than the allowable bearing pressures presented above.

For design of the P-T slabs, we have developed design parameters consistent with the recommendations developed by the Post Tensioning Institute [PTI] (1996). For general design of the P-T slabs, we recommend the parameters presented in Table 3 be used.

TABLE 3
General P-T Slab Design Parameters

Parameter	Value
Thornwaite Moisture Index	-10
Edge moisture variation distance Edge lift center lift	 3.0 feet 5.0 feet
Percentage clay	40%
Predominant mineral	Interstratified
Depth to constant soil suction	5 feet
Constant soil suction	3.5 pF
Moisture velocity	0.5 in/month
Soil differential movement	See Below

The soil differential movement is controlled by the amount of differential settlement expected rather than the potential for seasonal differential movement. Therefore, we recommend the slabs be designed for the edge-lift condition using special "compressible soil" design equations specified in Chapter 6 of the PTI manual. For this procedure, we preliminarily recommend total and differential settlement values of 3 and 1-1/4 inches be used for design, respectively.

If conventionally reinforced mats are used in lieu of P-T slabs, they should be evaluated to check they can span an unsupported area 15 feet in diameter in the interior of the building and to cantilever a distance of five feet along the edges and 7.5 feet at the corners. This evaluation for "areas of non-support" is purely empirical and is not intended to model actual site performance; the purpose of the evaluation is to establish foundation stiffness to control differential movement.

Lateral loads for either foundation type can be resisted by a combination of passive pressure acting on the vertical faces of the footings and friction along the base of the mat or P-T slab. Passive resistance may be calculated using an equivalent fluid weight of 250 pcf. The upper one foot of soil should be ignored unless it is confined by slabs or pavement. Frictional resistance should be computed using a base friction coefficient of 0.25 where the P-T slab is directly in contact with the native soil. Where the P-T slab is underlain by a vapor retarder, a base friction coefficient of 0.15 should be used. These values include a factor of safety of about 1.5.

6.2 Site Preparation and Grading

Prior to grading, the site should be cleared of vegetation and soil containing greater than four percent organic materials by dry weight of soil. The site has been disked in the past, and based on our experience at the adjacent site, we anticipate the upper 8 to 10 inches of disked soil will contain excessive organics and will require removal to meet these requirements. Stripped materials should be removed from the site or stockpiled for later use in landscaped areas, if approved by the architect. Existing underground utilities, if any, should be removed to the property lines or service connections and properly capped or plugged with concrete. Where existing utility lines will not interfere with the proposed construction, they may be abandoned in-place provided the lines are filled with lean concrete or cement grout to the property line.

In cut and fill areas, the exposed ground surface should be scarified to a depth of at least 12 inches, moisture-conditioned to near optimum moisture content, and compacted to at least 90 percent relative compaction[5]. The soil subgrade should be kept moist until it is covered by fill or concrete.

---

[5] Relative compaction refers to the in-place dry density of soil expressed as a percentage of the maximum dry density of the same material, as determined by the ASTM D1557-00 laboratory compaction procedure.

Fill should consist of on-site or imported soil that is free of organic matter, contains no rocks or lumps larger than three inches in greatest dimension, has a liquid limit less than 40 and plasticity index less than 12, and is approved by the geotechnical engineer. We anticipate on-site soil will generally meet the fill requirements. Fill should be placed in lifts not exceeding eight inches in loose thickness, moisture-conditioned to near optimum moisture content, and compacted to at least 90 percent relative compaction.

In pavement areas (Portland cement concrete or asphalt concrete) that will receive vehicular traffic, the upper six inches of the subgrade should be compacted to at least 95 percent relative compaction to achieve a firm, unyielding subgrade. The soil subgrade should be kept moist until it is covered by aggregate base.

The Geotechnical Engineer should approve all sources of engineered fill at least three days before use at the site. The grading subcontractor should provide analytical test results or other suitable environmental documentation indicating the imported fill is free of hazardous materials at least three days before use at the site. If this data is not available, up to two weeks should be allowed to perform analytical testing on the proposed import material. If the on-site material is to be exported, analytical testing of the soil may be required by the party or parties receiving the soil.

Backfill for utility trenches and other excavations is also considered fill, and it should be compacted according to the recommendations provided above. If imported or existing clean sand or gravel is used as backfill, however, it should be compacted to at least 90 percent relative compaction if the fill thickness is less than three feet and at least 95 percent relative compaction if the fill thickness is more than three feet. Jetting of trench backfill is not permitted.

6.2.1 Special Grading Considerations

Based on our experience with the surficial weak silt, the soil will be weak and difficult to compact when wet. We anticipate this condition will exist where in-place recompaction is

required of native materials deeper than two feet bgs. Therefore, if it is anticipated that significant excavations deeper than two feet will be performed (i.e., partial basements), we anticipate subgrade stabilization will be required. For smaller areas, subgrade stabilization may consist of overexcavation of the upper 12 to 24 inches of weak, wet soil, placement of a geotextile tensile fabric (Mirafi 500X or equivalent) over the bottom and sides of the excavation, and placement and compaction of Class 2 aggregate base to fill the void and stabilize the excavation.

If stabilization is required over larger areas, it may be more cost-effective to treat the upper 12 to 18 inches of unstable subgrade with lime or cement. Based on the low plasticity of the surficial silt, we anticipate treatment with cement or a lime-cement mixture would be more appropriate. The cement reacts with the soil to create strong physical bonds in the soil, resulting in increased strength. Cement treatment is typically performed by a specialty contractor using specialized equipment to fully blend the chemical additive to full width and depth. Moisture conditioning is performed during the treatment process, and compaction is performed with large sheepsfoot-type compaction equipment.

Grading in areas where weak, wet silt is within a couple feet of the existing ground surface is expected to be difficult. Pumping and rutting of the ground surface under construction traffic wheel loads may result in uneven, weak subgrade soil that is not suitable for support of the proposed improvements. Because pumping and soft subgrade conditions are often exacerbated by heavy, rubber-tired construction equipment, such as scrapers, we have developed recommendations to help reduce the amount of subgrade repair required at the site.

We recommend heavy, rubber-tired grading equipment, such as scrapers, motor graders, and articulated trucks, not be allowed on the building pads. Backhoes, skip loaders, and other lightweight (less than 25,000 pounds), rubber-tired grading equipment are acceptable for use on the building pads. Other rubber-tired equipment, such as Skytraks, Gradalls, water trucks, and concrete trucks, can be used around the perimeter of the buildings (i.e., driveways, landscape areas, walkways, etc.), but also should not be allowed on the building pads. Compaction within

the building pads should be performed with lightweight (less than 20,000 pounds) compaction equipment, such as a Caterpillar (CAT) CP-232C or CAT CP-433C, or equivalent.

We recommend grading for this project be performed during the summer and fall months to avoid saturation of the near-surface soil. We believe this will result in significantly more efficient grading operations and less potential for extra work associated with mitigation of weak subgrade conditions.

6.3 Concrete Slab-on-Grade Floors

The soil subgrade beneath concrete slab-on-grade floors and P-T slabs or mat foundations should be prepared in accordance with the recommendations in Section 6.2 of this report. Where the slab will be subject to vehicular traffic, we recommend the upper six inches of soil subgrade be compacted to at least 95 percent relative compaction. The soil subgrade in areas that will not receive vehicular traffic should be compacted to at least 90 percent relative compaction. If the previously compacted soil subgrade is disturbed during foundation and utility excavation, the subgrade should be rerolled to provide a firm, unyielding surface prior to placement of the capillary break material.

Where water vapor transmission through the floor slab is undesirable (e.g., where floor covering will be placed), we recommend a capillary moisture break and a water vapor retarder be installed beneath the floor. A capillary moisture break and vapor retarder are generally not required below parking slabs-on-grade because there is sufficient air circulation to limit condensation of moisture on the slab surface. As an alternative to the capillary moisture break/vapor retarder described below, six inches of Class 2 aggregate base compacted to at least 95 percent relative compaction should be placed beneath the parking structure slab where the slab will receive vehicular traffic.

A capillary moisture break consists of at least four inches of clean, free-draining gravel or crushed rock. The vapor retarder should meet the requirements for Class C vapor retarders stated

in ASTM E1745-97. The vapor retarder should be placed in accordance with the requirements of ASTM E1643-98. These requirements include overlapping seams by six inches, taping seams, and sealing penetrations in the vapor retarder. The vapor retarder should be covered with two inches of sand to aid in curing the concrete and to protect the vapor retarder during slab construction. The particle size of the gravel/crushed rock and sand should meet the gradation requirements presented in Table 4.

TABLE 4
Gradation Requirements for Capillary Moisture Break

Sieve Size	Percentage Passing Sieve
Gravel or Crushed Rock	
1 inch	90 – 100
3/4 inch	30 – 100
1/2 inch	5 – 25
3/8 inch	0 – 6
Sand	
No. 4	100
No. 200	0 – 5

The sand overlying the membrane should be moist at the time concrete is placed; however, it should not contain free water. Excess water trapped in the sand could eventually be transmitted as vapor through the slab. If rain is forecast prior to pouring the slab, the sand should be covered with plastic sheeting to avoid wetting. If the sand becomes wet, concrete should not be placed until the sand has been dried or replaced.

Concrete mixes with high water/cement (w/c) ratios result in excess water in the concrete, which increases the cure time and results in excessive vapor transmission through the slab. Therefore, concrete for the floor slab should have a low w/c ratio - less than 0.50. If approved by the

project structural engineer, the sand can be eliminated and the concrete can be placed directly over the vapor retarder, provided the w/c ratio of the concrete does not exceed 0.45 and water is not added in the field. If necessary, workability should be increased by adding plasticizers. In addition, the slab should be properly cured.

The current standard-of-practice in the northern California area does not include placing a gravel capillary break beneath P-T slab foundations. Although we believe this practice increases the risk of unacceptable levels of water vapor transmission through the slabs, it is our understanding it has been used successfully for at least 10 years. If the gravel is not used beneath P-T slabs, a vapor retarder meeting the requirements for Class B vapor retarders, overlain by two inches of sand meeting the gradation requirements presented in Table 4, should be used in lieu of a Class C vapor retarder.

In some cases, it is cost effective to remove both the sand and gravel from the capillary break and vapor retarder system used beneath P-T slabs. If it is desired to remove these materials from the underslab system, we recommend a Type A vapor retarder be used in lieu of a Type C vapor retarder. Type A vapor retarders have a lower inherent permeance rating and are less prone to accidental puncture. Examples of Type A vapor retarders are Stego-wrap 15-mil sheeting and Moiststop Ultra A polyolefin sheeting, although any equivalent system can be used. We should observe the condition of the membrane prior to placement of the sand and prior to placement of concrete to check that holes and other penetrations in the membrane have been minimized.

Before the floor covering is placed, the contractor should check that the concrete surface and the moisture emission levels (if emission testing is required) meet the manufacturer's requirements.

6.4 Asphalt Concrete Pavement

The State of California flexible pavement design method was used to develop the recommended asphalt concrete pavement sections. Laboratory test results indicate the near-surface sandy silt has an R-value of 26. However, because of the weak soil conditions encountered during grading for the adjacent Octel site and difficulties in establishing a firm compacted subgrade, we used a reduced R-value of 10 for design of new pavements.

For our calculations, we assumed a Traffic Index (TI) of 4.5 for parking stalls and 5.5 for driveways and drive aisles; these TIs should be confirmed by the project civil engineer. Table 5 presents our recommendations for asphalt pavement sections.

TABLE 5
Pavement Section Design

TI	Asphaltic Concrete (inches)[6]	Class 2 Aggregate Base R = 78 (inches)[7]
4.5	2.5	9.0
5.5	3.0	11.0

Pavement components should conform to the current Caltrans Standard Specifications.
In pavement areas, the upper six inches of subgrade should be compacted to at least 95 percent relative compaction to provide a firm, nonyielding subgrade. The Class 2 aggregate base should be moisture-conditioned to near optimum moisture content and compacted to at least 95 percent relative compaction. The soil subgrade should be kept moist until it is covered by aggregate base.

---

6 Minimum asphalt thickness shall be 2.5 inches.
7 Minimum aggregate base thickness shall be 6 inches.

6.5 Portland Cement Concrete Pavement

Concrete pavement design is based on a maximum single-axle load of 20,000 pounds and a maximum tandem axle of 32,000 pounds. The recommended rigid pavement section for these axle loads is six inches of Portland cement concrete over six inches of Class 2 aggregate base assuming the truck traffic is light to moderate.

The modulus of rupture of the concrete should be at least 500 pounds per square inch (psi) at 28 days. Contraction joints should be constructed at a 15-foot spacing. Where the outer edge of a concrete pavement meets asphalt pavement, the concrete slab should be thickened by 50 percent at a taper not to exceed a slope of 1 in 10. Concrete pavements subject to garbage truck traffic should be at least seven inches thick and reinforced with a minimum of No. 4 bars at 16 inches on center in both directions. Recommendations for subgrade preparation and aggregate base compaction for concrete pavement are the same as those we have described above for asphalt pavement.

6.6 Concrete Flatwork

In areas to receive concrete patios, sidewalks or other flatwork, the native soil subgrade should be scarified to a depth of at least 12 inches, moisture-conditioned to near optimum moisture content, and compacted to at least 90 percent relative compaction. On-site exterior concrete flatwork and sidewalks should be underlain by at least four inches of Class 2 aggregate base. The aggregate base should extend at least six inches beyond the slab edges, and should be moisture-conditioned to near optimum moisture content and compacted to at least 90 percent relative compaction. Public sidewalks should be constructed in accordance with the City of Milpitas requirements.

6.7 Seismic Design

The proposed buildings should be designed to the appropriate seismic codes. The project site is about 5.5 kilometers from the Southeast Extension of the Hayward Fault, defined as a type B

fault in Chapter 16 of the 2001 California Building Code (CBC), and it is about 9.5 kilometers from the main trace of the Hayward Fault, defined as a Type A fault in the 2001 CBC. The type A portion of the Hayward Fault controls the seismic design. Hence, for design in accordance with the 2001 CBC seismic code, we recommend the following:

- Seismic Zone Factor 4
- Soil Profile S_D
- Near Source Factors N_a and N_v of 1.02 and 1.24, respectively.

Per the 2001 CBC, the site could be classified as soil type S_F because of the presence of liquefiable soil beneath the site. Soil type S_F requires a site-specific evaluation to be performed. However, for buildings less than about eight stories tall, short-period, high-frequency motion typically controls for earthquake structural design, and the S_D soil profile type provides the most conservative design parameters for these earthquake motions. Therefore, because we do not anticipate any new structures taller than eight stories, we preliminarily conclude soil profile S_D is appropriate and conservative for the site. This will be confirmed during the final investigation.

7.0 ADDITIONAL GEOTECHNICAL SERVICES

The recommendations provided in this report are preliminary in nature as specific development plans are not available at this time and only limited subsurface exploration was performed for this investigation. A final geotechnical investigation, including additional subsurface exploration and laboratory testing, should be performed prior to final design. Information required for us to plan our final investigation includes building footprint locations, building loads, estimated finished floor elevations, and allowable settlement tolerances. Once the development plans have been provided and additional geotechnical testing has been performed, we can provide final recommendations for foundation design.

8.0 LIMITATIONS

The conclusions and recommendations presented in this report result from limited engineering studies based on our interpretation of the existing geotechnical conditions and available subsurface data. Actual subsurface conditions may vary. If any variations or unforeseen conditions are encountered during construction, or if the proposed construction will differ from that which is described in this report, Treadwell & Rollo should be notified so that supplemental recommendations can be made.

REFERENCES

California Division of Mines and Geology (1966), Probabilistic seismic hazard assessment for the State of California, DMG Open-File Report 96-08.

California Geological Survey (2004), State of California Seismic Hazard Zones, Milpitas Quadrangle, Official Map (Santa Clara County Portion Only), 19 April 2004.

International Conference of Building Officials (1997), Uniform Building Code, Volume 2 – Structural Engineering Design Provisions, Chapter 16.

National Center for Earthquake Engineering Research (NCEER) (1997). *Proceedings of the NCEER Workshop on Evaluation of Liquefaction Resistance of Soils.* Technical Reports NCEER-97-002.

Post Tensioning Institute (1996), *Design and Construction of Post-Tensioned Slabs-on-Ground,* Second Edition.

Seed, R.B. et. al. (2003), Recent Advances in Soil Liquefaction Engineering: A Unified and Consistent Framework, 26th Annual ASCE Los Angeles Geotechnical Spring Seminar, Keynote Presentation, H.M.S. Queen Mary, Long Beach, California, 30 April 2003.

Southern California Earthquake Center, University of Southern California (1999). *Recommended Procedures for Implementation of DMG.* Special Publication 117. Guidelines for Analyzing and Mitigating Liquefaction in California.

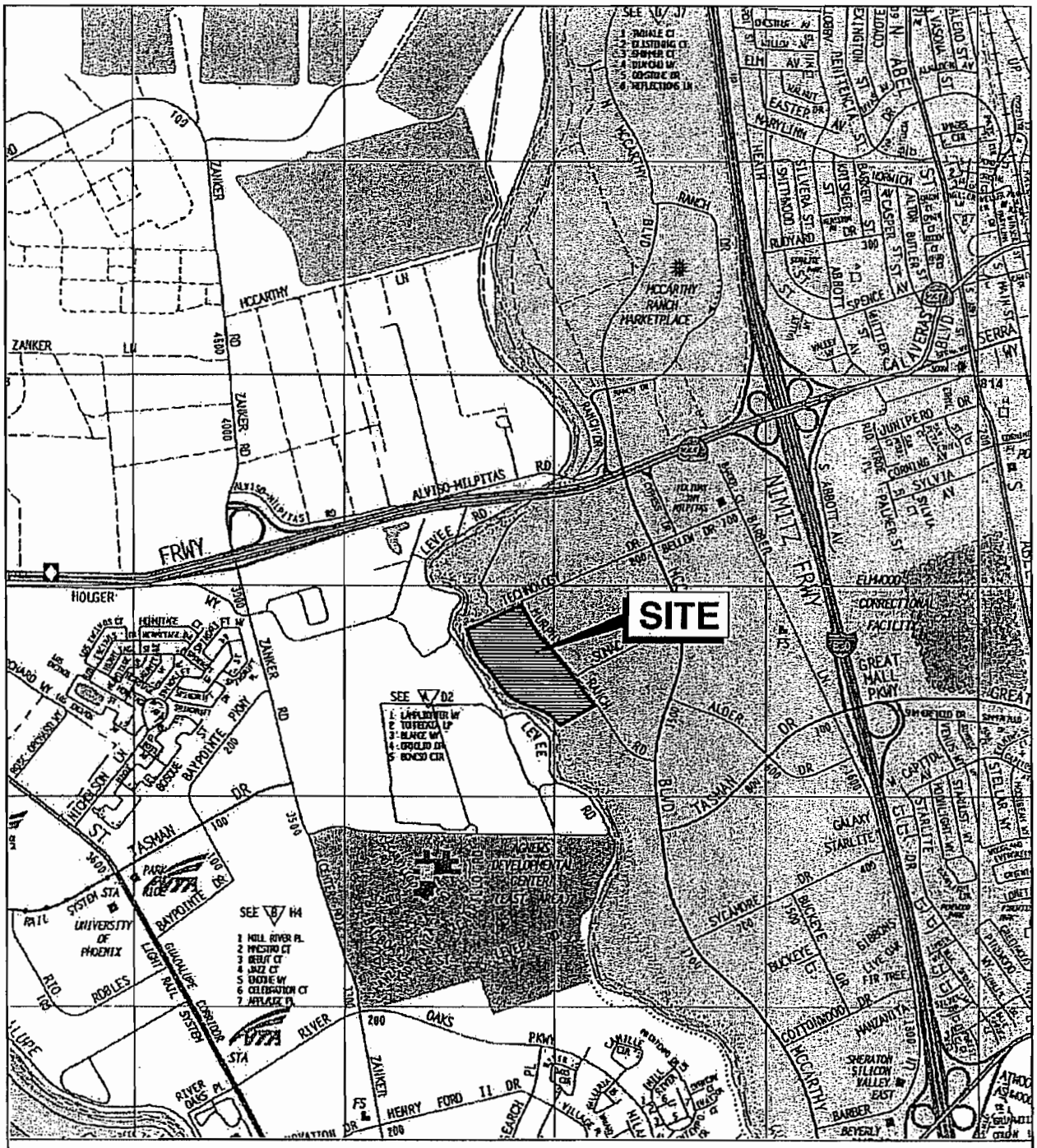
Toppozada, T.R. and Borchardt G. (1998). "Re-evaluation of the 1936 "Hayward Fault" and the 1838 San Andreas Fault Earthquakes." *Bulletin of Seismological Society of America*, 88(1), 140-159.

Treadwell & Rollo, Inc. (1995). *Geotechnical Investigation, Octel Communications Corporation, Corporate Headquarters Facility – Phases II and III, Milpitas, California*, 13 November 1995.

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

Treadwell&Rollo

FIGURES



Base map: The Thomas Guide
Santa Clara County
1999

0 1/4 1/2 Mile

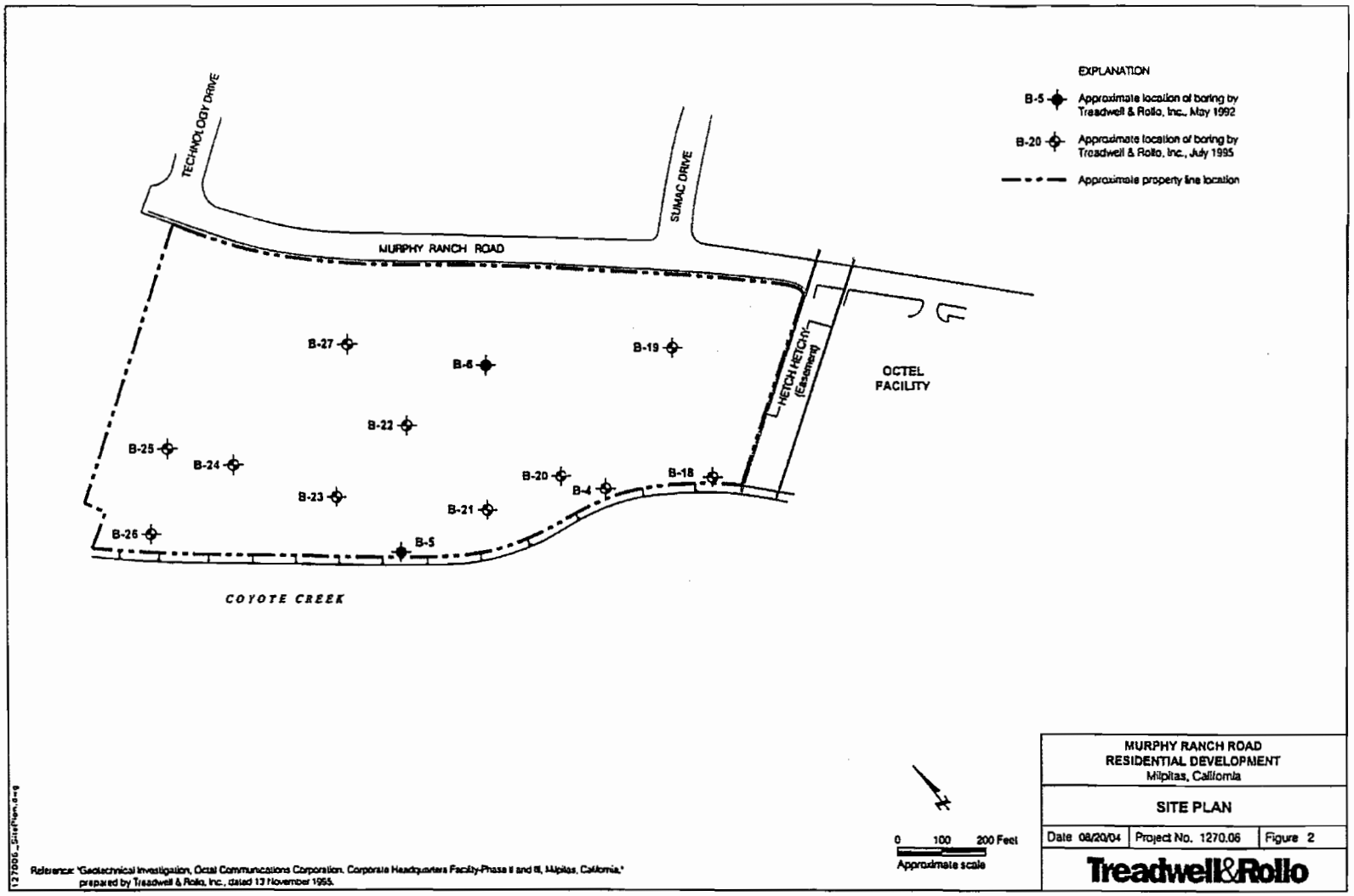
Approximate scale

MURPHY RANCH ROAD
RESIDENTIAL DEVELOPMENT
Milpitas, California

Treadwell&Rollo

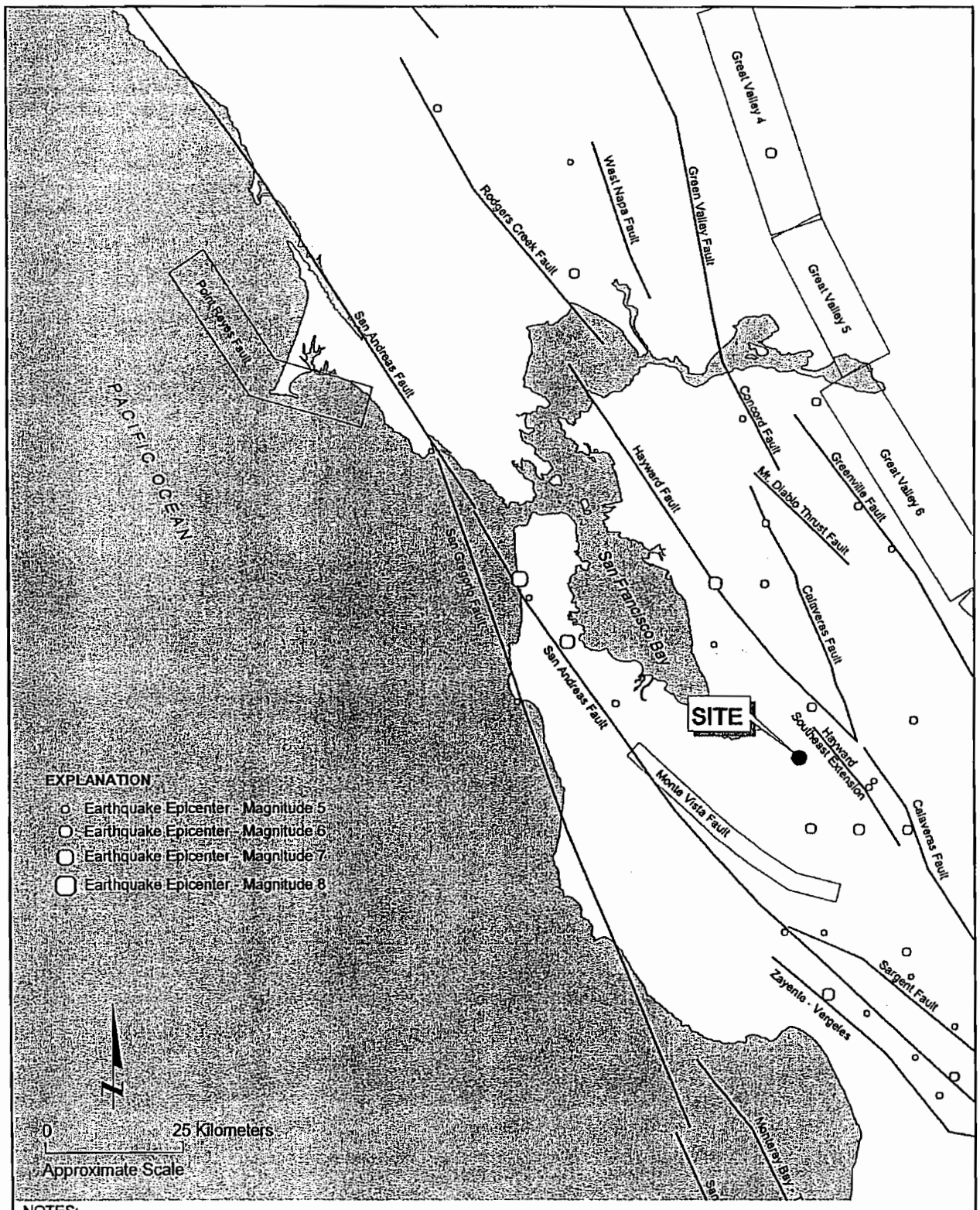
SITE LOCATION MAP

Date 08/20/04	Project No. 1270.06	Figure 1



Reference: "Geotechnical Investigation, Octel Communications Corporation, Corporate Headquarters Facility-Phase II and III, Milpitas, California," prepared by Treadwell & Rollo, Inc., dated 13 November 1995.

MURPHY RANCH ROAD RESIDENTIAL DEVELOPMENT Milpitas, California		
SITE PLAN		
Date 08/20/04	Project No. 1270.06	Figure 2
Treadwell&Rollo		



NOTES:
Digitized data for fault coordinates and earthquake catalog was developed by the California Department of Conservation Division of Mines and Geology. The historic earthquake catalog includes events from January 1800 to December 2000.

MURPHY RANCH ROAD RESIDENTIAL DEVELOPMENT Milpitas, California	MAP OF MAJOR FAULTS AND EARTHQUAKE EPICENTERS IN THE SAN FRANCISCO BAY AREA
Treadwell&Rollo	Date: 08/20/04 | Project No. 1270.06 | Figure: 3

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, 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 trucks, 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 clash. 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 fall. 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. Knickknacks, 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 muddied. Gravel or sand stream banks 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. Loosened brickwork and tiles shake down. Weak chimneys 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 renew flow. 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 tumbling down. Panel walls 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; fissures 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 folds 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 surfaces. Lines of sight and level are distorted. Objects are thrown upward into the air.

MURPHY RANCH ROAD RESIDENTIAL DEVELOPMENT Milpitas, California	MODIFIED MERCALLI INTENSITY SCALE
Treadwell&Rollo	Date: 08/20/04 | Project No. 1270.06 | Figure: 4

Treadwell&Rollo

APPENDIX A

Logs of Borings and Laboratory Test Results from Previous Investigations

PROJECT:	OCTEL COMMUNICATIONS PHASES II AND III Milpitas, California	Log of Boring B-4	PAGE 1 OF 2

Boring location: See Figure 2

Logged by E. Oman

Date started: 5/12/92 | Date finished: 5/13/92

Drilling method: Rotary wash

Hammer weight: 140 lbs. | Drop: 30 inches

Sampler: 3.0-inch O.D. split-barrel; Standard Penetration Test

DEPTH (feet)	SAMPLES: Sample No.	SAMPLES: Sample	SAMPLES: Blows/foot [1]	LITHOLOGY	MATERIAL DESCRIPTION (Ground Surface Elevation: 22 feet [2])	STRENGTH DATA: Type of Strength Test	STRENGTH DATA: Test Surcharge Pressure Lbs/Sq Ft	STRENGTH DATA: Fines %	STRENGTH DATA: Shear Strength Lbs/Sq Ft	MOISTURE-DENSITY DATA: Natural Moisture Content	MOISTURE-DENSITY DATA: Dry Density Lbs/Cu Ft
1				ML	SANDY SILT (ML) brown, medium stiff, dry						
2											
3					CLAY (CL) brown and gray, soft, moist						
4					PI = 8, LL = 30						
5	1	☒	4					91			
6											
7											
8											
9											
10	2	■	19		very stiff, moist Consolidation Test, see Figure A-39					24.2	99
11											
12											
13											
14				CL							
15	3	■	16		very stiff, saturated	TxUU	990		2,400	20	110
16											
17											
18											
19											
20	4	■	21								
21											
22											
23											
24											
25	5	■	18								
26											
27					increasing silt content						
28											
29											
30	6	■	8							31	93

PROJECT: **OCTEL COMMUNICATIONS PHASES II AND III**
Milpitas, California

Log of Boring B-4

DEPTH (feet)	SAMPLES: Sample No.	SAMPLES: Sample	SAMPLES: Blows/foot [1]	LITHOLOGY	MATERIAL DESCRIPTION	STRENGTH DATA: Type of Strength Test	STRENGTH DATA: Test Surcharge Pressure Lbs/Sq Ft	Fines %	Shear Strength Lbs/Sq Ft	MOISTURE-DENSITY DATA: Natural Moisture Content	MOISTURE-DENSITY DATA: Dry Density Lbs/Cu Ft
				CL	CLAY (CL) (continued)						
31					Boring terminated at a depth of 30.5 feet.						
32					Boring backfilled with cement/bentonite grout.						
33					Depth to groundwater obscurred by drilling method.						
34					[1] Blow counts converted to equivalent SPT N-values.						
35					[2] Elevations based on Octel Site Survey, prepared by Mark Thomas & Company, Inc., dated 6/2/92.						
36											
37											
38											
39											
40											
41											
42											
43											
44											
45											
46											
47											
48											
49											
50											
51											
52											
53											
54											
55											
56											
57											
58											
59											
60											

PROJECT:	OCTEL COMMUNICATIONS PHASES II AND III Milpitas, California	Log of Boring B-5	PAGE 1 OF 3

Boring location: See Figure 2

Logged by E. Oman

Date started: 5/13/92 | Date finished: 5/13/92

Drilling method: Rotary wash

Hammer weight: 140 lbs. | Drop: 30 inches

Sampler: 3.0-inch O.D. split-barrel; Standard Penetration Test

DEPTH (feet)	SAMPLES: Sample No.	SAMPLES: Sample	SAMPLES: Blows/foot[1]	LITHOLOGY	MATERIAL DESCRIPTION Ground Surface Elevation: 22 feet[2]	STRENGTH DATA: Type of Strength Test	STRENGTH DATA: Test Surcharge Pressure Lbs/Sq Ft	STRENGTH DATA: Fines %	STRENGTH DATA: Shear Strength Lbs/Sq Ft	MOISTURE-DENSITY DATA: Natural Moisture Content	MOISTURE-DENSITY DATA: Dry Density Lbs/Cu Ft
1–2	1	X	4	ML	SILT with SAND (ML) brown, soft, moist PI = 7, LL = 27			85			
4–5	2	X	7	CL	SANDY CLAY with SILT (CL) brown and gray, medium stiff, moist						
10	3	■	19		very stiff, moist					25	102
15	4	■	29		very stiff, saturated						
20	5	■	10		stiff, saturated					29	95
25	6	■	20		very stiff, saturated						
30	7	■	16							21	108

PROJECT: **OCTEL COMMUNICATIONS PHASES II AND III**
Milpitas, California

Log of Boring B-5

DEPTH (feet)	SAMPLES: Sample No.	SAMPLES: Sample	SAMPLES: Blows/foot 1	LITHOLOGY	MATERIAL DESCRIPTION	STRENGTH DATA: Type of Strength Test	STRENGTH DATA: Test Surcharge Pressure Lbs/Sq Ft	DATA: Fines %	DATA: Shear Strength Lbs/Sq Ft	MOISTURE-DENSITY DATA: Natural Moisture Content	MOISTURE-DENSITY DATA: Dry Density Lbs/Cu Ft
31				CL	SANDY CLAY (CL) brown and gray, very stiff, saturated						
32											
33											
34											
35	8	■	14		stiff, saturated						
36											
37											
38											
39											
40	9	■	21		very stiff, saturated						
41											
42											
43				ML	SANDY SILT (ML) brown and gray, medium dense, saturated PI = 1, LL = 22						
44											
45	10	■	16					61			
46											
47											
48				CL	SANDY CLAY (CL) brown and gray, very stiff, saturated						
49											
50	11	■	25							25	103
51											
52											
53											
54											
55	12	■	23								
56											
57											
58											
59											
60											

PROJECT: **OCTEL COMMUNICATIONS PHASES II AND III**
Milpitas, California

Log of Boring B-5

DEPTH (feet)	SAMPLES: Sample No.	SAMPLES: Sample	SAMPLES: Blows/foot[1]	LITHOLOGY	MATERIAL DESCRIPTION	STRENGTH DATA: Type of Strength Test	STRENGTH DATA: Test Surcharge Pressure Lbs/Sq Ft	Fines %	Shear Strength Lbs/Sq Ft	MOISTURE-DENSITY DATA: Natural Moisture Content	MOISTURE-DENSITY DATA: Dry Density Lbs/Cu Ft
	13		39	CL	SANDY CLAY (CL) (continued)						
61					Boring terminated at a depth of 60.5 feet.						
62					Boring backfilled with cement/bentonite grout.						
63					Depth to groundwater obscurred by drilling method.						
64					[1] Blow counts converted to equivalent SPT N-values.						
65					[2] Elevations based on Octel Site Survey, prepared by Mark Thomas & Company, Inc., dated 6/2/92.						
66											
67											
68											
69											
70											
71											
72											
73											
74											
75											
76											
77											
78											
79											
80											
81											
82											
83											
84											
85											
86											
87											
88											
89											
90											

PROJECT:	OCTEL COMMUNICATIONS PHASES II AND III Milpitas, California	**Log of Boring B-6**	PAGE 1 OF 2

Boring location: See Figure 2		Logged by E. Oman
Date started: 5/13/92	Date finished: 5/13/92	
Drilling method: Rotary wash		
Hammer weight: 140 lbs.	Drop: 30 inches	
Sampler: 3.0-inch O.D. split-barrel; Standard Penetration Test		

DEPTH (feet)	SAMPLES: Sample No.	SAMPLES: Sample	SAMPLES: Blows/foot[1]	LITHOLOGY	MATERIAL DESCRIPTION Ground Surface Elevation: 22 feet[2]	STRENGTH DATA: Type of Strength Test	STRENGTH DATA: Test Surcharge Pressure Lbs/Sq Ft	STRENGTH DATA: Fines %	STRENGTH DATA: Shear Strength Lbs/Sq Ft	MOISTURE-DENSITY DATA: Natural Moisture Content	MOISTURE-DENSITY DATA: Dry Density Lbs/Cu Ft
1–2	1		4	ML	SANDY SILT (ML) brown, medium stiff, moist			71			
5	2		6	ML	PI = 10, LL = 29 increasing clay content					20	100
10	3		20	CL	SANDY CLAY (CL) brown and gray, very stiff, moist					24	103
16	4		16	CL	very stiff, saturated	TxUU	990		1,200	27	99
20	5		14	CL	stiff, saturated						
25	6		18	CL	very stiff, saturated PI= 19, LL = 39						
29–30	7		19	ML	SANDY SILT (ML) brown and gray, medium dense, saturated PI = 0, LL = 23			64			

PROJECT: **OCTEL COMMUNICATIONS PHASES II AND III**
Milpitas, California

Log of Boring B-6

DEPTH (feet)	SAMPLES: Sample No.	SAMPLES: Sample	SAMPLES: Blows/foot [1]	LITHOLOGY	MATERIAL DESCRIPTION	STRENGTH DATA: Type of Strength Test	STRENGTH DATA: Test Surcharge Pressure Lbs/Sq Ft	Fines %	Shear Strength Lbs/Sq Ft	MOISTURE-DENSITY DATA: Natural Moisture Content	MOISTURE-DENSITY DATA: Dry Density Lbs/Cu Ft
				ML	SANDY SILT (ML) (continued)						
31					Boring terminated at a depth of 30.5 feet.						
32					Boring backfilled with cement/bentonite grout.						
					Depth to groundwater obscurred by drilling method.						
33											
34					[1] Blow counts converted to equivalent SPT N-values.						
35											
36					[2] Elevations based on Octel Site Survey, prepared by Mark Thomas & Company, Inc., dated 6/2/92.						
37											
38											
39											
40											
41											
42											
43											
44											
45											
46											
47											
48											
49											
50											
51											
52											
53											
54											
55											
56											
57											
58											
59											
60											

PROJECT: **OCTEL COMMUNICATIONS PHASES II AND III**
Milpitas, California

Log of Boring B-18

Boring location: See Figure 2
Logged by M. Oman
Date started: 7/26/95 | Date finished: 7/26/95
Drilling method: Rotary wash
Hammer weight: 140 lbs. | Drop: 30 inches
Sampler: 3.0-inch O.D. split-barrel; Standard Penetration Test

DEPTH (feet)	SAMPLES: Sample No.	SAMPLES: Sample	SAMPLES: Blows/foot[1]	LITHOLOGY	MATERIAL DESCRIPTION (Ground Surface Elevation: 21.4 feet[2])	STRENGTH DATA: Type of Strength Test	STRENGTH DATA: Test Surcharge Pressure Lbs/Sq Ft	STRENGTH DATA: Fines %	STRENGTH DATA: Shear Strength Lbs/Sq Ft	MOISTURE-DENSITY DATA: Natural Moisture Content	MOISTURE-DENSITY DATA: Dry Density Lbs/Cu Ft
1				ML	SANDY SILT (ML) brown, stiff, moist						
2											
3	1	■	13								
4											
5											
6	2	■	6		Shrink-Swell Test, see Figure A-41 medium stiff					30.3	89
7				CL	CLAY (CL) gray-brown, hard, wet						
8											
9											
10	3	■	40							25.7	101
11											
12											
13											
14											
15											
16	4	■	49								
17											
18				ML	SANDY SILT (ML) mottled orange, brown, and gray, stiff, wet						
19											
20											
21	5	■	9								
22				CL	SANDY CLAY (CL) mottled orange, brown, and gray, very stiff, wet						
23											
24											
25											
26	6	■	16								
27											
28											
29											
30											

PROJECT: **OCTEL COMMUNICATIONS PHASES II AND III**
Milpitas, California

Log of Boring B-18

DEPTH (feet)	SAMPLES: Sample No.	SAMPLES: Sample	SAMPLES: Blows/foot [1]	LITHOLOGY	MATERIAL DESCRIPTION	STRENGTH DATA: Type of Strength Test	STRENGTH DATA: Test Surcharge Pressure Lbs/Sq Ft	Fines %	Shear Strength Lbs/Sq Ft	MOISTURE-DENSITY DATA: Natural Moisture Content	MOISTURE-DENSITY DATA: Dry Density Lbs/Cu Ft
31	7		11	CL	CLAY (CL) gray-brown, stiff, wet						
32											
33											
34											
35				SM	SILTY SAND (SM) dark gray, medium dense, wet						
36	8		16								
37					Boring terminated at a depth of 36.5 feet. Boring backfilled with cement/bentonite grout. Depth to groundwater obscurred by drilling method.						
38											
39											
40					[1] Blow counts converted to equivalent SPT N-values.						
41											
42					[2] Elevations interpolated from topographic map, prepared by Aerometric Surveys, dated 16 May 1995.						
43											
44											
45											
46											
47											
48											
49											
50											
51											
52											
53											
54											
55											
56											
57											
58											
59											
60											

PROJECT:	OCTEL COMMUNICATIONS PHASES II AND III Milpitas, California	**Log of Boring B-19**	PAGE 1 OF 2

Boring location: See Figure 2		Logged by M. Oman
Date started: 7/27/95	Date finished: 7/28/95	
Drilling method: Rotary wash		
Hammer weight: 140 lbs.	Drop: 30 inches	
Sampler: 3.0-inch O.D. split-barrel; Standard Penetration Test		

DEPTH (feet)	SAMPLES: Sample No.	SAMPLES: Sample	SAMPLES: Blows/foot [1]	LITHOLOGY	MATERIAL DESCRIPTION Ground Surface Elevation: 21.2 feet [2]	STRENGTH DATA: Type of Strength Test	STRENGTH DATA: Test Surcharge Pressure Lbs/Sq Ft	STRENGTH DATA: Fines %	STRENGTH DATA: Shear Strength Lbs/Sq Ft	MOISTURE-DENSITY DATA: Natural Moisture Content	MOISTURE-DENSITY DATA: Dry Density Lbs/Cu Ft
1				ML	SANDY SILT (ML) brown, medium stiff, moist						
2											
3	1	■	5								
4											
5				CL	SANDY CLAY (CL) brown, medium stiff, wet						
6	2	■	7							26.2	99
7											
8											
9											
10											
11	3	■	39								
12											
13											
14					▽ 7/28/95 (unstabilized)						
15											
16	4	■	48								
17											
18											
19											
20											
21	5	■	31								
22											
23											
24											
25											
26	6	■	10								
27											
28											
29											
30											

PROJECT: **OCTEL COMMUNICATIONS PHASES II AND III**
Milpitas, California

Log of Boring B-19

DEPTH (feet)	SAMPLES: Sample No.	SAMPLES: Sample	SAMPLES: Blows/foot[1]	LITHOLOGY	MATERIAL DESCRIPTION	STRENGTH DATA: Type of Strength Test	STRENGTH DATA: Test Surcharge Pressure Lbs/Sq Ft	DATA: Fines %	DATA: Shear Strength Lbs/Sq Ft	MOISTURE-DENSITY DATA: Natural Moisture Content	MOISTURE-DENSITY DATA: Dry Density Lbs/Cu Ft
31	7		11	CL	SANDY CLAY (CL) (continued)						
32				SM	SILTY SAND (SM) brown and gray, dense, wet						
33											
34											
35											
36	8		30								
37					Boring terminated at a depth of 36.5 feet. Boring backfilled with cement/bentonite grout. Unstabilized groundwater level measured at a depth of 14.0 feet.						
38											
39											
40					[1] Blow counts converted to equivalent SPT N-values.						
41											
42					[2] Elevations interpolated from topographic map, prepared by Aerometric Surveys, dated 16 May 1995.						
43											
44											
45											
46											
47											
48											
49											
50											
51											
52											
53											
54											
55											
56											
57											
58											
59											
60											

PROJECT:	OCTEL COMMUNICATIONS PHASES II AND III Milpitas, California	Log of Boring B-20	PAGE 1 OF 2

Boring location: See Figure 2

Logged by M. Oman

Date started: 7/26/95 | Date finished: 7/26/95

Drilling method: Rotary wash

Hammer weight: 140 lbs. | Drop: 30 inches

Sampler: 3.0-inch O.D. split-barrel; Standard Penetration Test

DEPTH (feet)	SAMPLES: Sample No.	SAMPLES: Sample	SAMPLES: Blows/foot [1]	LITHOLOGY	MATERIAL DESCRIPTION (Ground Surface Elevation: 20.4 feet [2])	STRENGTH DATA: Type of Strength Test	STRENGTH DATA: Test Surcharge Pressure Lbs/Sq Ft	STRENGTH DATA: Fines %	STRENGTH DATA: Shear Strength Lbs/Sq Ft	MOISTURE-DENSITY DATA: Natural Moisture Content	MOISTURE-DENSITY DATA: Dry Density Lbs/Cu Ft
1–9	1, 2		9, 5	ML	SANDY SILT (ML) brown, stiff, moist Direct Shear Test, see Figure A-36 c = 250 psf, Ø = 22.5°						
10–11	3		5	CL	CLAY (CL) brown, soft to medium stiff, moist	TxUU	750		420	28.7	85
12–16	4		20	CL	SANDY CLAY (CL) mottled orange and brown, very stiff, wet						
17–26	5, 6		28, 22	CL	CLAY (CL) brown-gray, very stiff, wet						
27–30											

PROJECT: **OCTEL COMMUNICATIONS PHASES II AND III**
Milpitas, California

Log of Boring B-20

DEPTH (feet)	SAMPLES: Sample No.	SAMPLES: Sample	SAMPLES: Blows/foot[1]	LITHOLOGY	MATERIAL DESCRIPTION	STRENGTH DATA: Type of Strength Test	STRENGTH DATA: Test Surcharge Pressure Lbs/Sq Ft	DATA: Fines %	DATA: Shear Strength Lbs/Sq Ft	MOISTURE-DENSITY DATA: Natural Moisture Content	MOISTURE-DENSITY DATA: Dry Density Lbs/Cu Ft
	7		15	CL	SANDY CLAY (CL) (continued)						
31	8			SM	SILTY SAND (SM) dark gray, medium dense, wet						
32											
33											
34											
35			20	SP	SAND (SP) dark gray, medium dense, wet						
36	9										
37											
38				SM	SILTY SAND (SM) dark gray, very dense, wet						
39											
40											
41											
42											
43											
44											
45											
46	10		77								
47					Boring terminated at a depth of 46.5 feet. Boring backfilled with cement/bentonite grout. Depth to groundwater obscurred by drilling method.						
48											
49					[1] Blow counts converted to equivalent SPT N-values.						
50											
51					[2] Elevations interpolated from topographic map, prepared by Aerometric Surveys, dated 16 May 1995.						
52											
53											
54											
55											
56											
57											
58											
59											
60											

PROJECT:	OCTEL COMMUNICATIONS PHASES II AND III Milpitas, California	Log of Boring B-21	PAGE 1 OF 2

Boring location: See Figure 2

Logged by M. Oman

Date started: 7/26/95 | Date finished: 7/27/95

Drilling method: Rotary wash

Hammer weight: 140 lbs. | Drop: 30 inches

Sampler: 3.0-inch O.D. split-barrel; Standard Penetration Test

DEPTH (feet)	SAMPLES: Sample No.	SAMPLES: Sample	SAMPLES: Blows/foot[1]	LITHOLOGY	MATERIAL DESCRIPTION (Ground Surface Elevation: 20.5 feet[2])	STRENGTH DATA: Type of Strength Test	STRENGTH DATA: Test Surcharge Pressure Lbs/Sq Ft	STRENGTH DATA: Fines %	STRENGTH DATA: Shear Strength Lbs/Sq Ft	MOISTURE-DENSITY DATA: Natural Moisture Content	MOISTURE-DENSITY DATA: Dry Density Lbs/Cu Ft
1					SILT (ML) brown, medium stiff, dry to moist						
2	1	■	7	ML	Particle Size Analysis Test, see Figure A-31			90.5			
3											
4					CLAY (CL) brown, medium stiff, moist						
5											
6	2	■	6							26.7	97
7											
8											
9											
10											
11	3	■	33	CL	very stiff						
12					▽ 7/27/95 (unstabilized)						
13											
14											
15	4	■	32								
16											
17											
18											
19											
20	5	■	35		very stiff to hard						
21											
22											
23											
24				CL							
25											
26	6	■	20		very stiff						
27											
28											
29											
30											

PROJECT: **OCTEL COMMUNICATIONS PHASES II AND III**
Milpitas, California

Log of Boring B-21

DEPTH (feet)	SAMPLES: Sample No.	SAMPLES: Sample	SAMPLES: Blows/foot[1]	LITHOLOGY	MATERIAL DESCRIPTION	STRENGTH DATA: Type of Strength Test	STRENGTH DATA: Test Surcharge Pressure Lbs/Sq Ft	Fines %	Shear Strength Lbs/Sq Ft	MOISTURE-DENSITY DATA: Natural Moisture Content	MOISTURE-DENSITY DATA: Dry Density Lbs/Cu Ft
31	7		20	CL	CLAY (CL) (continued)						
32											
33											
34				SM	SILTY SAND (SM) brown, medium dense, wet						
35											
36	8		22								
37					Boring terminated at a depth of 36.5 feet. Boring backfilled with cement/bentonite grout. Unstabilized groundwater level measured at a depth of 12 feet.						
38											
39											
40					[1] Blow counts converted to equivalent SPT N-values.						
41					[2] Elevations interpolated from topographic map, prepared by Aerometric Surveys, dated 16 May 1995.						
42											
43											
44											
45											
46											
47											
48											
49											
50											
51											
52											
53											
54											
55											
56											
57											
58											
59											
60											

PROJECT:	OCTEL COMMUNICATIONS PHASES II AND III Milpitas, California	**Log of Boring B-22**	PAGE 1 OF 2

Boring location: See Figure 2

Logged by M. Oman

Date started: 7/26/95 | Date finished: 7/26/95

Drilling method: Rotary wash

Hammer weight: 140 lbs. | Drop: 30 inches

Sampler: 3.0-inch O.D. split-barrel; Standard Penetration Test

DEPTH (feet)	SAMPLES: Sample No.	SAMPLES: Sample	SAMPLES: Blows/foot[1]	LITHOLOGY	MATERIAL DESCRIPTION Ground Surface Elevation: 19.9 feet[2]	STRENGTH DATA: Type of Strength Test	STRENGTH DATA: Test Surcharge Pressure Lbs/Sq Ft	STRENGTH DATA: Fines %	STRENGTH DATA: Shear Strength Lbs/Sq Ft	MOISTURE-DENSITY DATA: Natural Moisture Content	MOISTURE-DENSITY DATA: Dry Density Lbs/Cu Ft
1				ML	SANDY SILT (ML) brown, medium stiff, moist						
2	1	■	6								
3											
4											
5					Shrink-Swell Test, see Figure A-42						
6	2	■	5							25.3	96
7				CL	SANDY CLAY (CL) gray-brown, soft, wet						
8											
9											
10											
11	3	■	3			TxUU	750		550	29.4	93
12											
13											
14											
15											
16	4	■	3		Consolidation Test, see Figure A-40						
17											
18											
19											
20											
21	5	■	21		very stiff						
22											
23				SM	SILTY SAND (SM) brown, medium dense, wet						
24											
25											
26	6	■	11								
27											
28											
29											
30											

PROJECT: **OCTEL COMMUNICATIONS PHASES II AND III**
Milpitas, California

Log of Boring B-22

DEPTH (feet)	SAMPLES: Sample No.	SAMPLES: Sample	SAMPLES: Blows/foot [1]	LITHOLOGY	MATERIAL DESCRIPTION	STRENGTH DATA: Type of Strength Test	STRENGTH DATA: Test Surcharge Pressure Lbs/Sq Ft	Fines %	Shear Strength Lbs/Sq Ft	MOISTURE-DENSITY DATA: Natural Moisture Content	MOISTURE-DENSITY DATA: Dry Density Lbs/Cu Ft
31	7		34	SM	SILTY SAND (SM) (continued)						
32											
33				SP-SM	SAND with SILT (SP - SM) brown, dense, wet						
34											
35											
36	8		45								
37					Boring terminated at a depth of 36.5 feet. Boring backfilled with cement/bentonite grout. Depth to groundwater obscurred by drilling method. [1] Blow counts converted to equivalent SPT N-values. [2] Elevations interpolated from topographic map, prepared by Aerometric Surveys, dated 16 May 1995.						
38											
39											
40											
41											
42											
43											
44											
45											
46											
47											
48											
49											
50											
51											
52											
53											
54											
55											
56											
57											
58											
59											
60											

PROJECT:	OCTEL COMMUNICATIONS PHASES II AND III Milpitas, California	Log of Boring B-23	PAGE 1 OF 2

Boring location: See Figure 2		Logged by M. Oman
Date started: 7/27/95	Date finished: 7/27/95	
Drilling method: Rotary wash		
Hammer weight: 140 lbs.	Drop: 30 inches	
Sampler: 3.0-inch O.D. split-barrel; Standard Penetration Test		

DEPTH (feet)	SAMPLES: Sample No.	SAMPLES: Sample	SAMPLES: Blows/foot[1]	LITHOLOGY	MATERIAL DESCRIPTION (Ground Surface Elevation: 20.0 feet[2])	STRENGTH: Type of Strength Test	STRENGTH: Test Surcharge Pressure Lbs/Sq Ft	DATA: Fines %	DATA: Shear Strength Lbs/Sq Ft	MOISTURE-DENSITY DATA: Natural Moisture Content	MOISTURE-DENSITY DATA: Dry Density Lbs/Cu Ft
1				ML	SANDY SILT (ML) brown, medium stiff, dry to moist						
2											
3	1	■	5		Direct Shear Test, see Figure A-37 c = 570 psf, Ø = 19.5°						
4				CL	CLAY (CL) brown, medium stiff, moist						
5											
6	-	●	7								
7											
8											
9											
10	2	■	40		hard						
11											
12											
13											
14											
15											
16	3	■	32		mottled orange and brown						
17											
18											
19											
20											
21	4	■	14		stiff						
22											
23											
24				ML	SANDY SILT (ML) mottled gray and brown, stiff, wet						
25											
26	5	■	10								
27											
28											
29				SM	SILTY SAND (SM) mottled gray and brown, dense, wet						
30											

Treadwell&Rollo	Project No. 1270.05	Figure A-18

PROJECT: **OCTEL COMMUNICATIONS PHASES II AND III**
Milpitas, California

Log of Boring B-23

DEPTH (feet)	SAMPLES: Sample No.	SAMPLES: Sample	SAMPLES: Blows/foot[1]	LITHOLOGY	MATERIAL DESCRIPTION	STRENGTH DATA: Type of Strength Test	STRENGTH DATA: Test Surcharge Pressure Lbs/Sq Ft	STRENGTH DATA: Fines %	STRENGTH DATA: Shear Strength Lbs/Sq Ft	MOISTURE-DENSITY DATA: Natural Moisture Content	MOISTURE-DENSITY DATA: Dry Density Lbs/Cu Ft
31	6	X	31		SILTY SAND (SM) (continued)						
32											
33				SM							
34											
35											
36	7	X	42	SP-SM	SAND with SILT (SP - SM) gray and brown, dense, wet						
37					Boring terminated at a depth of 36.5 feet. Boring backfilled with cement/bentonite grout. Depth to groundwater obscurred by drilling method.						
38											
39					[1] Blow counts converted to equivalent SPT N-values.						
40											
41					[2] Elevations interpolated from topographic map, prepared by Aerometric Surveys, dated 16 May 1995.						
42											
43											
44											
45											
46											
47											
48											
49											
50											
51											
52											
53											
54											
55											
56											
57											
58											
59											
60											

PROJECT:	OCTEL COMMUNICATIONS PHASES II AND III Milpitas, California	Log of Boring B-24	PAGE 1 OF 2

Boring location: See Figure 2

Logged by M. Oman

Date started: 7/28/95 | Date finished: 7/28/95

Drilling method: Rotary wash

Hammer weight: 140 lbs. | Drop: 30 inches

Sampler: 3.0-Inch O.D. split-barrel; Standard Penetration Test

DEPTH (feet)	SAMPLES: Sample No.	SAMPLES: Sample	SAMPLES: Blows/foot[1]	LITHOLOGY	MATERIAL DESCRIPTION (Ground Surface Elevation: 19.5 feet[2])	STRENGTH: Type of Strength Test	STRENGTH: Test Surcharge Pressure Lbs/Sq Ft	DATA: Fines %	DATA: Shear Strength Lbs/Sq Ft	MOISTURE-DENSITY DATA: Natural Moisture Content	MOISTURE-DENSITY DATA: Dry Density Lbs/Cu Ft
1–4	1	■	8	ML	SANDY SILT (ML) brown, medium stiff to stiff, moist						
5–6	2	■	4	CL	CLAY (CL) brown, soft, wet					31.5	91
10–11	3	■	32	CL	very stiff						
15–16	4	■	31	CL							
21	5	■	12	CL	stiff						
25–26	6	■	7	ML	SANDY SILT (ML) brown, medium stiff, wet						
29–30				SW	SAND with GRAVEL (SW) gray-brown, dense, wet						

Treadwell&Rollo | Project No. 1270.05 | Figure A-20

PROJECT: **OCTEL COMMUNICATIONS PHASES II AND III**
Milpitas, California

Log of Boring B-24

DEPTH (feet)	SAMPLES: Sample No.	SAMPLES: Sample	SAMPLES: Blows/foot [1]	LITHOLOGY	MATERIAL DESCRIPTION	STRENGTH DATA: Type of Strength Test	STRENGTH DATA: Test Surcharge Pressure Lbs/Sq Ft	Fines %	Shear Strength Lbs/Sq Ft	MOISTURE-DENSITY DATA: Natural Moisture Content	MOISTURE-DENSITY DATA: Dry Density Lbs/Cu Ft
31	7	X	39	SW	SAND with GRAVEL (SW) (continued)						
32											
33											
34											
35											
36	8	X	59								
37					Boring terminated at a depth of 36.5 feet. Boring backfilled with cement/bentonite grout. Depth to groundwater obscurred by drilling method.						
38											
39					[1] Blow counts converted to equivalent SPT N-values.						
40											
41					[2] Elevations interpolated from topographic map, prepared by Aerometric Surveys, dated 16 May 1995.						
42											
43											
44											
45											
46											
47											
48											
49											
50											
51											
52											
53											
54											
55											
56											
57											
58											
59											
60											

PROJECT:	OCTEL COMMUNICATIONS PHASES II AND III Milpitas, California	# Log of Boring B-25	PAGE 1 OF 2

Boring location: See Figure 2		Logged by M. Oman
Date started: 7/28/95	Date finished: 7/28/95	
Drilling method: Rotary wash		
Hammer weight: 140 lbs.	Drop: 30 inches	
Sampler: 3.0-inch O.D. split-barrel; Standard Penetration Test		

DEPTH (feet)	SAMPLES: Sample No.	SAMPLES: Sample	SAMPLES: Blows/foot[1]	LITHOLOGY	MATERIAL DESCRIPTION (Ground Surface Elevation: 19.4 feet[2])	STRENGTH DATA: Type of Strength Test	STRENGTH DATA: Test Surcharge Pressure Lbs/Sq Ft	STRENGTH DATA: Fines %	STRENGTH DATA: Shear Strength Lbs/Sq Ft	MOISTURE-DENSITY DATA: Natural Moisture Content	MOISTURE-DENSITY DATA: Dry Density Lbs/Cu Ft
1				ML	SANDY SILT (ML) brown, medium stiff to stiff, moist						
2											
3	1	■	6		Shrink-Swell Test, see Figure A-43					22.3	98
4											
5					Direct Shear Test, see Figure A-38						
6	2	■	3		c = 50 psf, Ø = 32.5° soft						
7				CL	CLAY (CL) brown, very stiff, moist						
8											
9											
10	3	■	23								
11											
12											
13											
14											
15											
16	4	■	20		mottled gray and brown						
17											
18											
19											
20											
21	5	■	10		stiff						
22											
23											
24				ML	SANDY SILT (ML) brown, stiff, wet						
25											
26	6	■	13								
27											
28											
29				SW	SAND with GRAVEL (SW) gray, dense, wet						
30											

PROJECT: **OCTEL COMMUNICATIONS PHASES II AND III** Milpitas, California

Log of Boring B-25

DEPTH (feet)	SAMPLES: Sample No.	SAMPLES: Sample	SAMPLES: Blows/foot [1]	LITHOLOGY	MATERIAL DESCRIPTION	STRENGTH DATA: Type of Strength Test	STRENGTH DATA: Test Surcharge Pressure Lbs/Sq Ft	DATA: Fines %	DATA: Shear Strength Lbs/Sq Ft	MOISTURE-DENSITY DATA: Natural Moisture Content	MOISTURE-DENSITY DATA: Dry Density Lbs/Cu Ft
31	7	X	40	SW	SAND with GRAVEL (SW) (continued)						
32											
33											
34											
35											
36	8	X	39								
37					Boring terminated at a depth of 36.5 feet. Boring backfilled with cement/bentonite grout. Depth to groundwater obscurred by drilling method. [1] Blow counts converted to equivalent SPT N-values. [2] Elevations interpolated from topographic map, prepared by Aerometric Surveys, dated 16 May 1995.						
38											
39											
40											
41											
42											
43											
44											
45											
46											
47											
48											
49											
50											
51											
52											
53											
54											
55											
56											
57											
58											
59											
60											

PROJECT:	OCTEL COMMUNICATIONS PHASES II AND III Milpitas, California	Log of Boring B-26	PAGE 1 OF 2

Boring location: See Figure 2		Logged by M. Oman
Date started: 7/28/95	Date finished: 7/28/95	
Drilling method: Rotary wash		
Hammer weight: 140 lbs.	Drop: 30 inches	
Sampler: 3.0-Inch O.D. split-barrel; Standard Penetration Test		

DEPTH (feet)	SAMPLES: Sample No.	SAMPLES: Sample	SAMPLES: Blows/foot [1]	LITHOLOGY	MATERIAL DESCRIPTION (Ground Surface Elevation: 20.0 feet [2])	STRENGTH DATA: Type of Strength Test	STRENGTH DATA: Test Surcharge Pressure Lbs/Sq Ft	DATA: Fines %	DATA: Shear Strength Lbs/Sq Ft	MOISTURE-DENSITY DATA: Natural Moisture Content	MOISTURE-DENSITY DATA: Dry Density Lbs/Cu Ft
1–4	1	■	66 / 11"	ML	SANDY SILT (ML) brown, hard, moist						
5–14	2, 3	■	9, 10	ML-CL	SILT (ML - CL) brown, medium stiff to stiff, moist; Particle Size Analysis Test, see Figure A-31			93.6			
15–30	4, 5, 6	■	48, 22, 16	CL	SANDY CLAY (CL) gray and brown, hard, wet; very stiff; stiff to very stiff						

PROJECT: **OCTEL COMMUNICATIONS PHASES II AND III**
Milpitas, California

Log of Boring B-26

DEPTH (feet)	SAMPLES: Sample No.	SAMPLES: Sample	SAMPLES: Blows/foot[1]	LITHOLOGY	MATERIAL DESCRIPTION	STRENGTH DATA: Type of Strength Test	STRENGTH DATA: Test Surcharge Pressure Lbs/Sq Ft	STRENGTH DATA: Fines %	STRENGTH DATA: Shear Strength Lbs/Sq Ft	MOISTURE-DENSITY DATA: Natural Moisture Content	MOISTURE-DENSITY DATA: Dry Density Lbs/Cu Ft
31	7	■	22	CL	SANDY CLAY (CL) (continued)						
32											
33											
34											
35				SW	SAND with GRAVEL (SW) gray-brown, very dense, wet						
36	8	☒	60								
37					Boring terminated at a depth of 36.5 feet. Boring backfilled with cement/bentonite grout. Depth to groundwater obscurred by drilling method. [1] Blow counts converted to equivalent SPT N-values. [2] Elevations interpolated from topographic map, prepared by Aerometric Surveys, dated 16 May 1995.						
38											
39											
40											
41											
42											
43											
44											
45											
46											
47											
48											
49											
50											
51											
52											
53											
54											
55											
56											
57											
58											
59											
60											

PROJECT:	OCTEL COMMUNICATIONS PHASES II AND III Milpitas, California	Log of Boring B-27	PAGE 1 OF 2

Boring location: See Figure 2

Logged by M. Oman

Date started: 7/27/95 | Date finished: 7/27/95

Drilling method: Rotary wash

Hammer weight: 140 lbs. | Drop: 30 inches

Sampler: 3.0-inch O.D. split-barrel; Standard Penetration Test

DEPTH (feet)	SAMPLES: Sample No.	SAMPLES: Sample	SAMPLES: Blows/foot[1]	LITHOLOGY	MATERIAL DESCRIPTION (Ground Surface Elevation: 19.6 feet[2])	STRENGTH DATA: Type of Strength Test	STRENGTH DATA: Test Surcharge Pressure Lbs/Sq Ft	Fines %	Shear Strength Lbs/Sq Ft	MOISTURE-DENSITY DATA: Natural Moisture Content	MOISTURE-DENSITY DATA: Dry Density Lbs/Cu Ft
1–4	1	■	8	ML	SANDY SILT (ML) brown, medium stiff, moist						
5–8	2	■	7	CL-ML	CLAY (CL-ML) brown, soft to medium stiff, wet					25.1	101
9–11	3	■	44	CL	SANDY CLAY (CL) brown-gray, hard, wet						
12–16	4	■	41	CL							
17–21	5	■	28	CL	very stiff						
22–26	6	■	16	CL	SANDY CLAY (CL) mottled orange, brown, and gray, very stiff, wet						
27–30				SM	SILTY SAND (SM) mottled orange-brown, medium dense, wet						

PROJECT: **OCTEL COMMUNICATIONS PHASES II AND III**
Milpitas, California

Log of Boring B-27

DEPTH (feet)	SAMPLES: Sample No.	SAMPLES: Sample	SAMPLES: Blows/foot[1]	LITHOLOGY	MATERIAL DESCRIPTION	STRENGTH: Type of Strength Test	STRENGTH: Test Surcharge Pressure Lbs/Sq Ft	DATA: Fines %	DATA: Shear Strength Lbs/Sq Ft	MOISTURE-DENSITY DATA: Natural Moisture Content	MOISTURE-DENSITY DATA: Dry Density Lbs/Cu Ft
31	7		23	SM	SILTY SAND (SM) (continued)						
32											
33											
34											
35				SW	SAND (SW) gray, dense, wet						
36	8		46								
37					Boring terminated at a depth of 36.5 feet. Boring backfilled with cement/bentonite grout. Depth to groundwater obscurred by drilling method.						
38											
39					[1] Blow counts converted to equivalent SPT N-values.						
40											
41					[2] Elevations interpolated from topographic map, prepared by Aerometric Surveys, dated 16 May 1995.						
42											
43											
44											
45											
46											
47											
48											
49											
50											
51											
52											
53											
54											
55											
56											
57											
58											
59											
60											

Unified Soil Classification System

Major Divisions		Symbols	Typical Names
Coarse-Grained Soils (more than half of soil > no. 200 sieve size)	Gravels (More than half of coarse fraction > no. 4 sieve size)	GW	Well-graded gravels or gravel-sand mixtures, little or no fines
		GP	Poorly-graded gravels or gravel-sand mixtures, little or no fines
		GM	Silty gravels, gravel-sand-silt mixtures
		GC	Clayey gravels, gravel-sand-clay mixtures
	Sands (More than half of coarse fraction < no. 4 sieve size)	SW	Well-graded sands or gravelly sands, little or no fines
		SP	Poorly-graded sands or gravelly sands, little or no fines
		SM	Silty sands, sand-silt mixtures
		SC	Clayey sands, sand-clay mixtures
Fine-Grained Soils (more than half of soil < no. 200 sieve size)	Silts and Clays LL = < 50	ML	Inorganic silts and very fine sands, rock flour, silty fine sands or clayey silts with slight plasticity
		CL	Inorganic clays of low to medium plasticity, gravelly clays, sandy clays, lean clays
		OL	Organic silts and organic silty clays of low plasticity
	Silts and Clays LL = > 50	MH	Inorganic silts, micaceous or diatomaceous fine sandy or silty soils, elastic silts
		CH	Inorganic clays of high plasticity, fat clays
		OH	Organic clays of high plasticity, organic silty clays, organic silts
Highly Organic Soils		Pt	Peat and other highly organic soils

Grain Size Chart

Classification	Range of Grain Sizes	
	U.S. Standard Sieve Size	Grain Size in Millimeters
Boulders	Above 12"	Above 305
Cobbles	12" to 3"	305 to 76.2
Gravel	3" to No. 4	76.2 to 4.76
coarse	3" to 3/4"	76.2 to 19.1
fine	3/4" to No. 4	19.1 to 4.76
Sand	No. 4 to No. 200	4.76 to 0.074
coarse	No. 4 to No. 10	4.76 to 2.00
medium	No. 10 to No. 40	2.00 to 0.420
fine	No. 40 to No. 200	0.420 to 0.074
Silt and Clay	Below No. 200	Below 0.074

Sample Designation

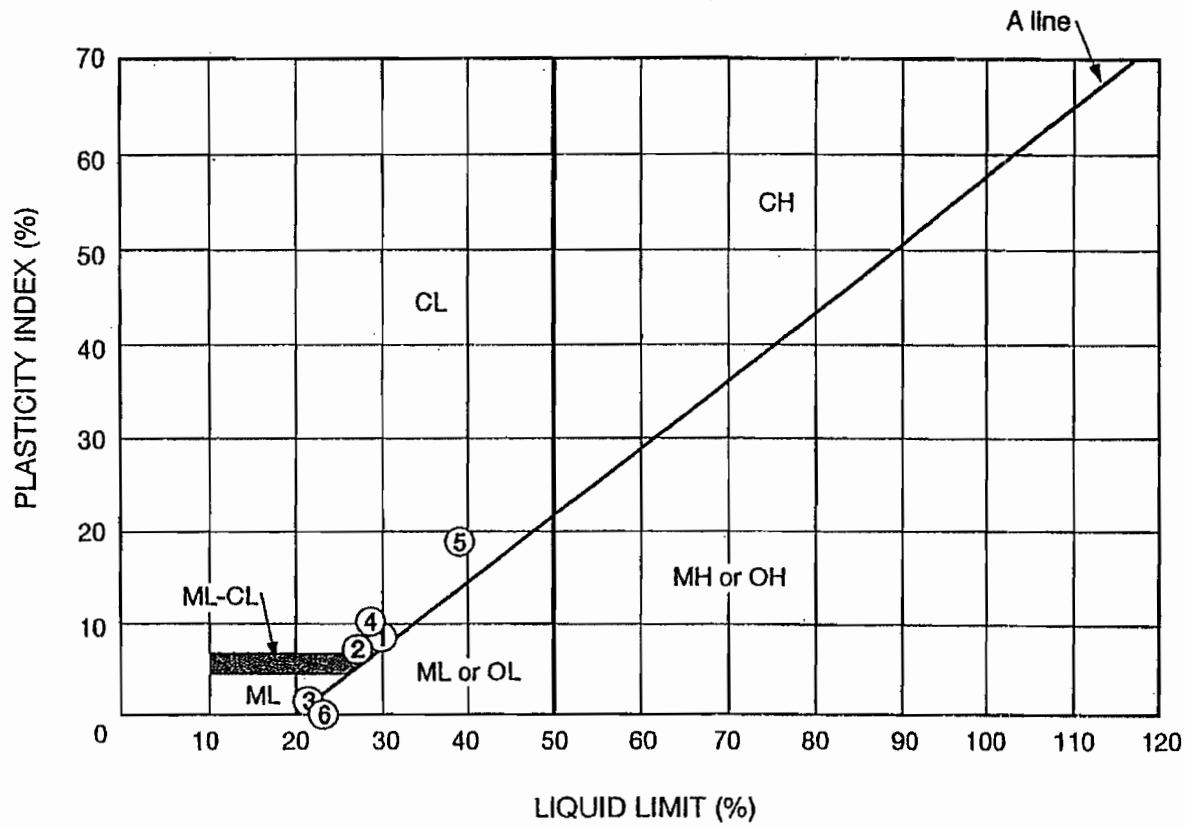
- Sample driven with Sprague & Henwood (3-inch outside diameter) sampler. Darkened area indicates sample obtained
- Classification sample taken with Standard Penetration Test (2-inch outside diameter) sampler
- Undisturbed sample taken with 3-inch outside diameter Shelby tube
- Attempted sample with no recovery

OCTEL COMMUNICATIONS
PHASES II AND III
Milpitas, California

Treadwell&Rollo

CLASSIFICATION CHART

Project No. 1270.05 | Figure A-28



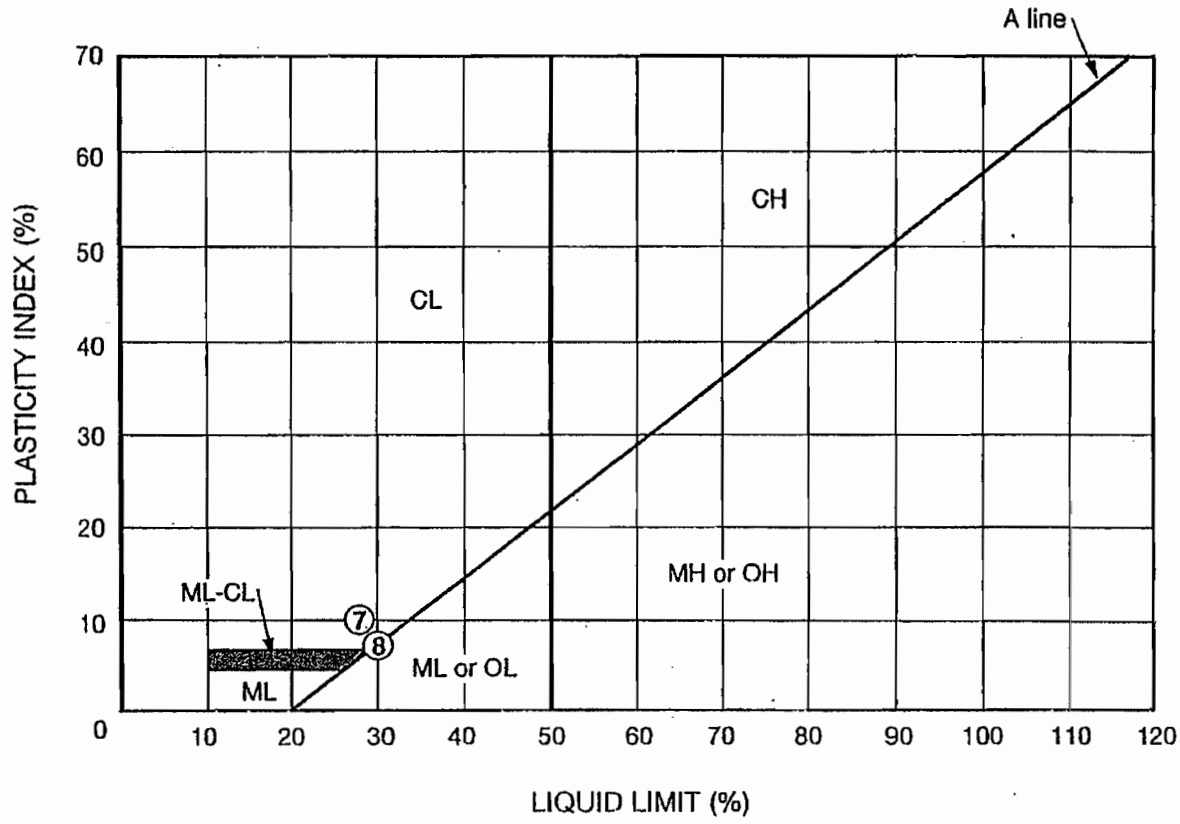
Symbol	Source	Classification	Natural M.C. (%)	Liquid Limit (%)	Plasticity Index (%)	% Passing #200 Sieve
①	Boring B-4 @ 4.0 feet	CLAY (CL) brown and gray	—	30	8	91
②	Boring B-5 @ 1.0 feet	SILT with SAND (ML) brown	—	27	7	85
③	Boring B-5 @ 44.5 feet	SANDY SILT (ML) brown and gray	—	22	1	61
④	Boring B-6 @ 4.5 feet	SANDY CLAY (CL) brown	—	29	10	—
⑤	Boring B-6 @ 24.5 feet	SANDY CLAY (CL) brown and gray	—	39	19	—
⑥	Boring B-6 @ 29.5 feet	SANDY SILT (ML) brown and gray	—	23	0	54

OCTEL COMMUNICATIONS
PHASES II AND III
Milpitas, California

Treadwell&Rollo

PLASTICITY CHART

Project No. 1270.05 | Figure A-29



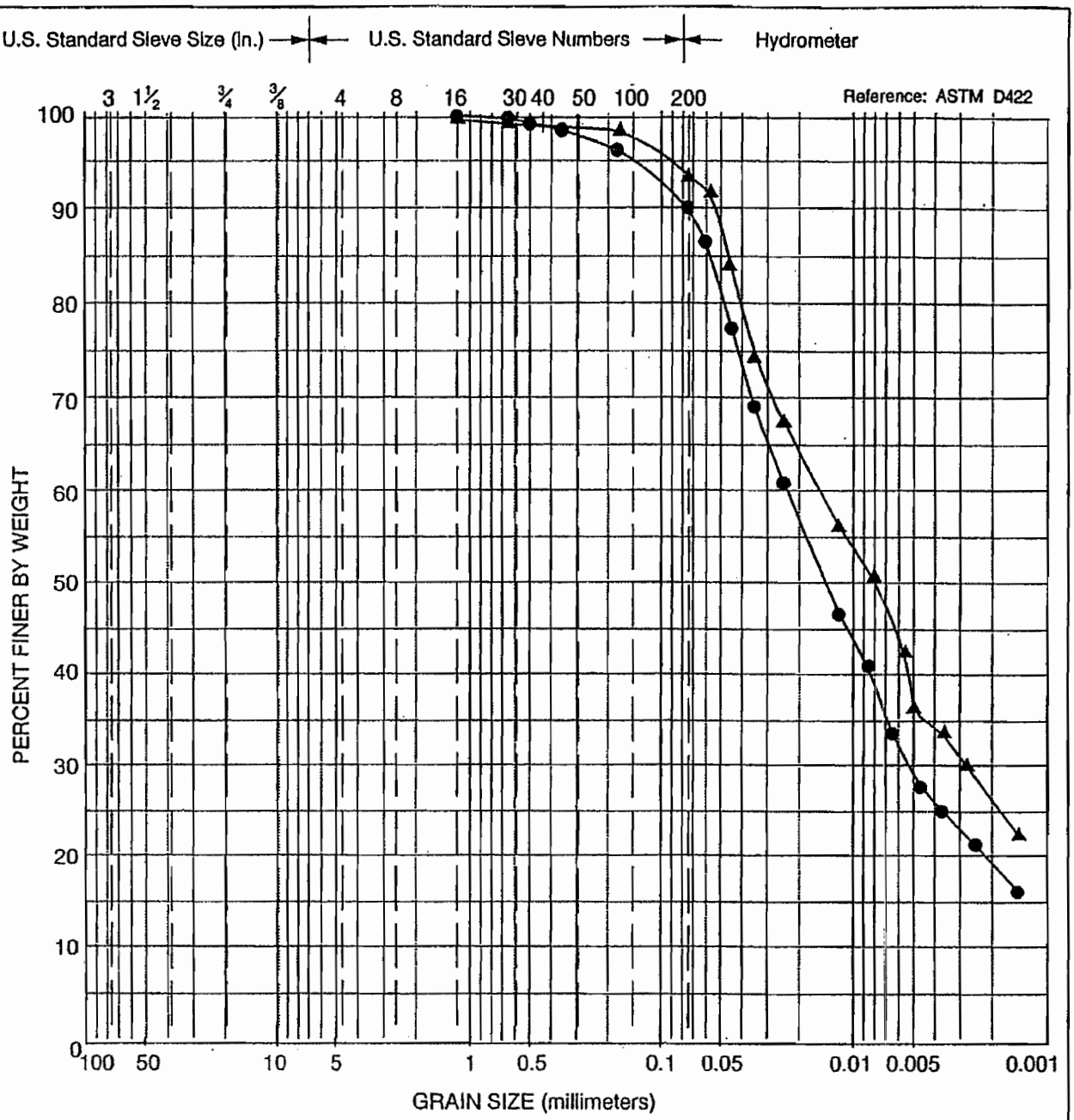
Symbol	Source	Classification	Natural M.C. (%)	Liquid Limit (%)	Plasticity Index (%)	% Passing #200 Sieve
7	Shallow bulk sample (composite)	CLAY (CL) brown	—	28	10	
8	Shallow bulk sample (composite with 4% lime)	SILT (ML - CL) brown	—	30	7	

OCTEL COMMUNICATIONS
PHASES II AND III
Milpitas, California

Treadwell&Rollo

PLASTICITY CHART

Project No. 1270.05 | Figure A-30



Cobbles	Coarse	Fine	Coarse	Medium	Fine	Silt or Clay
	Gravel		Sand			

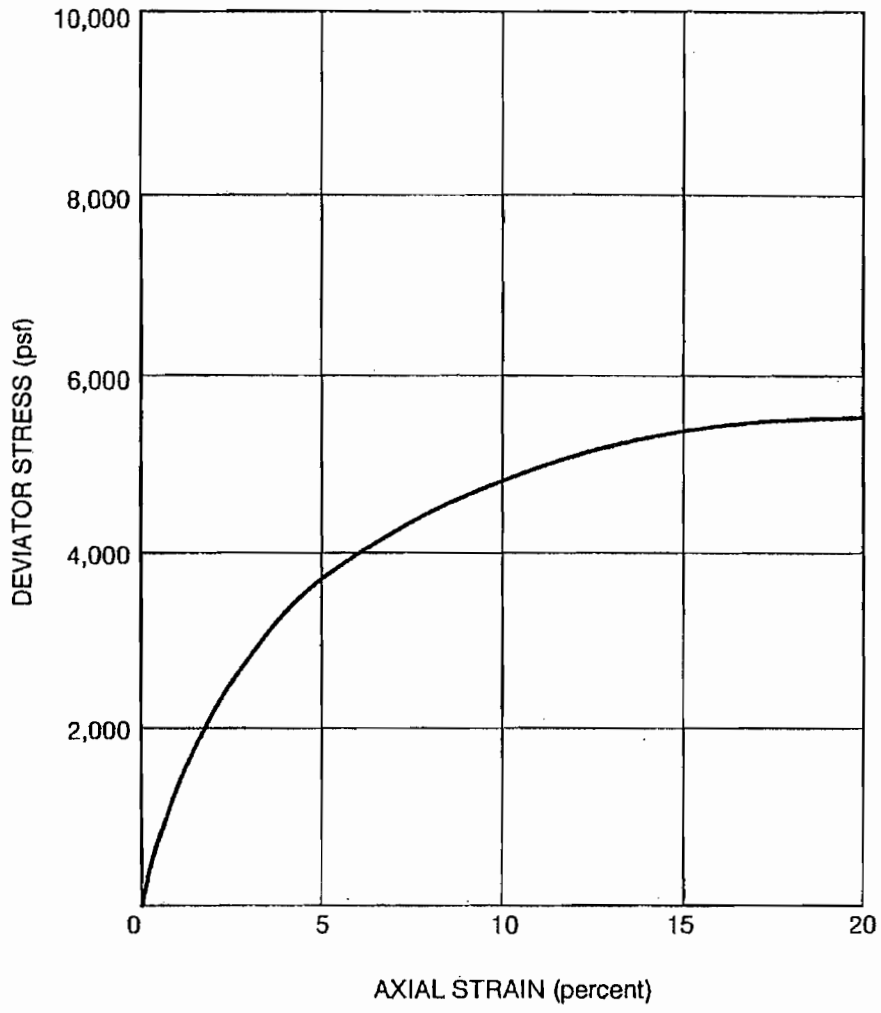
Symbol	Sample Source	Classification
●	Boring B-21 @ 1.5 feet	SILT (ML - CL), brown
▲	Boring B-26 @ 10.5 feet	SILT (ML - CL), brown

OCTEL COMMUNICATIONS
PHASES II AND III
Milpitas, California

Treadwell&Rollo

PARTICLE SIZE ANALYSIS

Project No. 1270.05 | Figure A-31



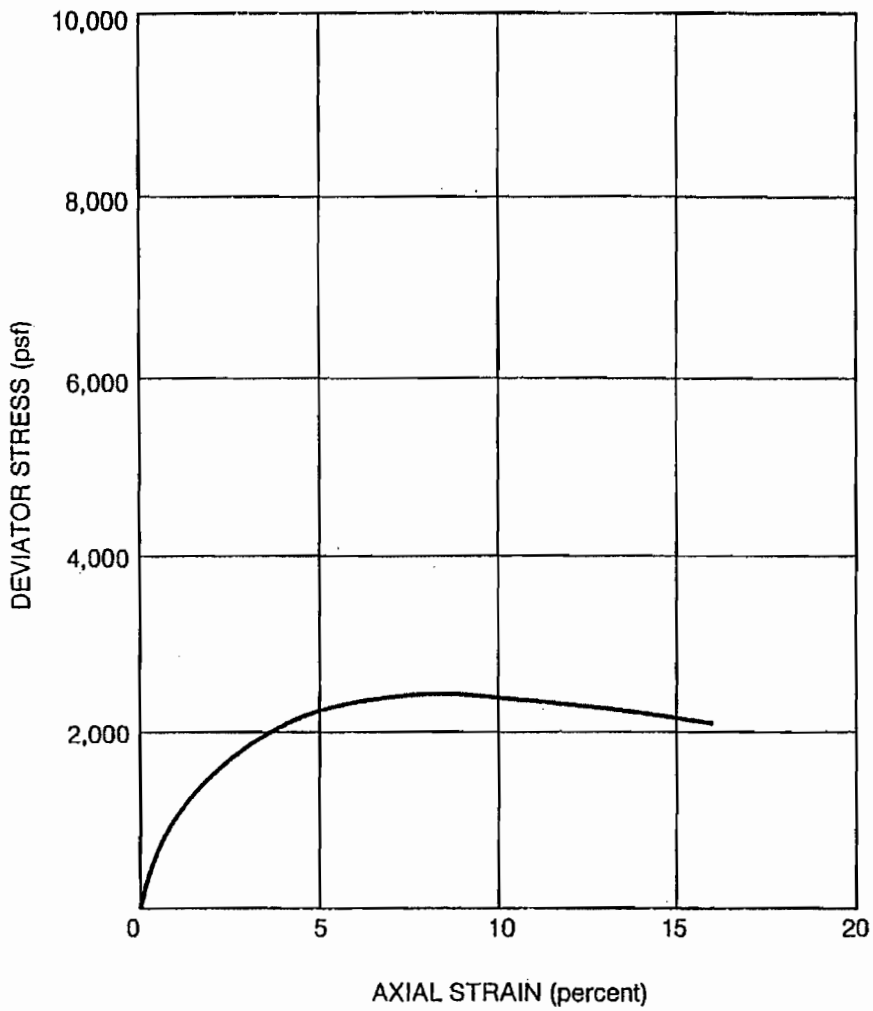
SPECIMEN TYPE Undisturbed		SHEAR STRENGTH	2,400 psf
DIAMETER (in) 2.41	HEIGHT (in) 6.00	STRAIN AT FAILURE	10 %
MOISTURE CONTENT	20 %	CONFINING PRESSURE	990 psf
DRY DENSITY	110 pcf	STRAIN RATE	1.0 %/min
CLASSIFICATION CLAY (CL), brown and gray		SOURCE B-4 @ 14.5 feet	

OCTEL COMMUNICATIONS
PHASES II AND III
Milpitas, California

UNCONSOLIDATED-UNDRAINED TRIAXIAL COMPRESSION TEST

Treadwell&Rollo

Project No. 1270.05 | Figure A-32



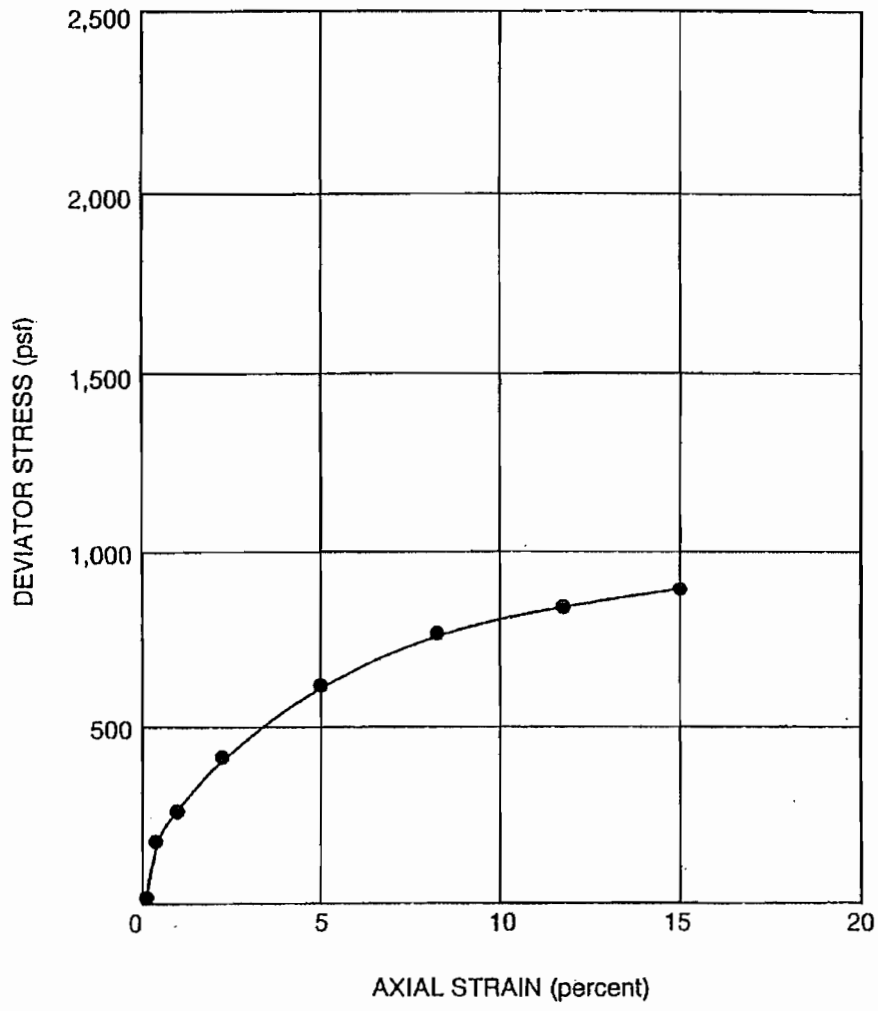
SPECIMEN TYPE	Undisturbed	SHEAR STRENGTH	1,200 psf
DIAMETER (in) 2.41	HEIGHT (in) 5.03	STRAIN AT FAILURE	8 %
MOISTURE CONTENT	27 %	CONFINING PRESSURE	990 psf
DRY DENSITY	99 pcf	STRAIN RATE	1.0 %/min
CLASSIFICATION SANDY CLAY (CL), brown and gray		SOURCE B-6 @ 16.0 feet	

OCTEL COMMUNICATIONS
PHASES II AND III
Milpitas, California

UNCONSOLIDATED-UNDRAINED TRIAXIAL COMPRESSION TEST

Treadwell&Rollo

Project No. 1270.05 | Figure A-33



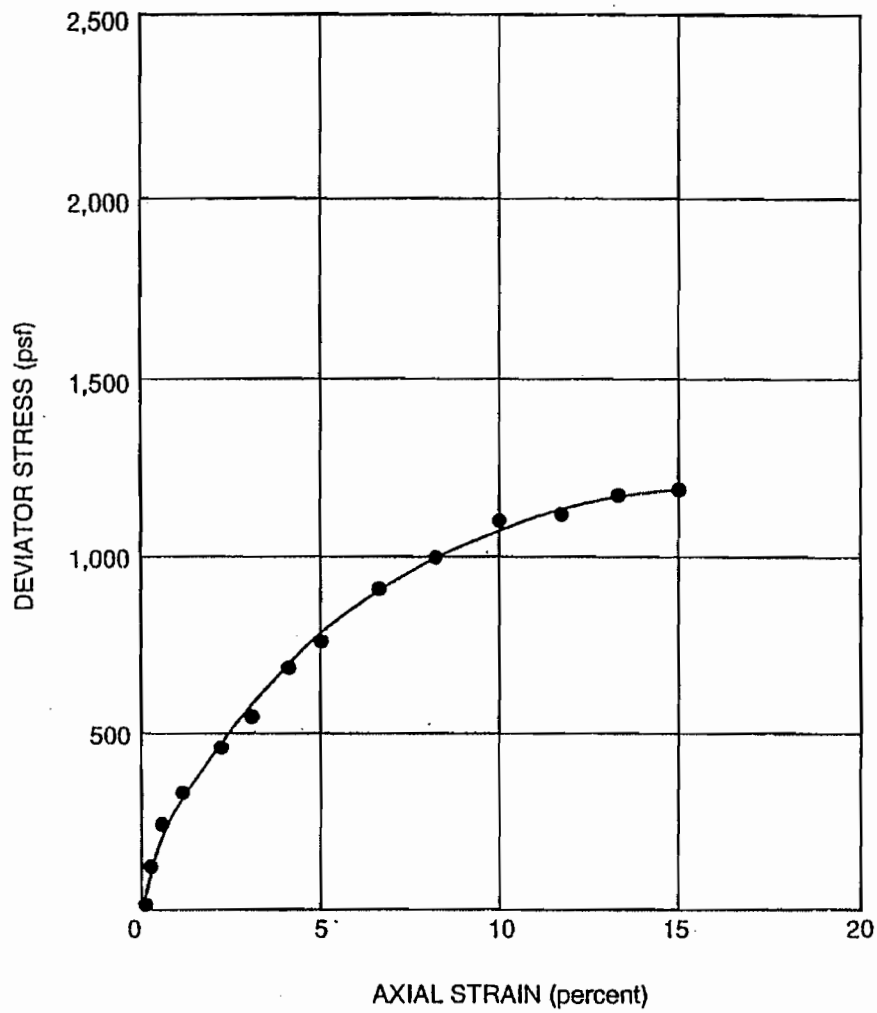
SPECIMEN TYPE	Undisturbed	SHEAR STRENGTH	420 psf
DIAMETER (in) 2.43	HEIGHT (in) 6.00	STRAIN AT FAILURE	10 %
MOISTURE CONTENT	28.7 %	CONFINING PRESSURE	750 psf
DRY DENSITY	85 pcf	STRAIN RATE	1.0 %/min
CLASSIFICATION CLAY (CL), brown		SOURCE B-20 @ 10.5 feet	

OCTEL COMMUNICATIONS
PHASES II AND III
Milpitas, California

UNCONSOLIDATED-UNDRAINED TRIAXIAL COMPRESSION TEST

Treadwell&Rollo

Project No. 1270.05 | Figure A-34



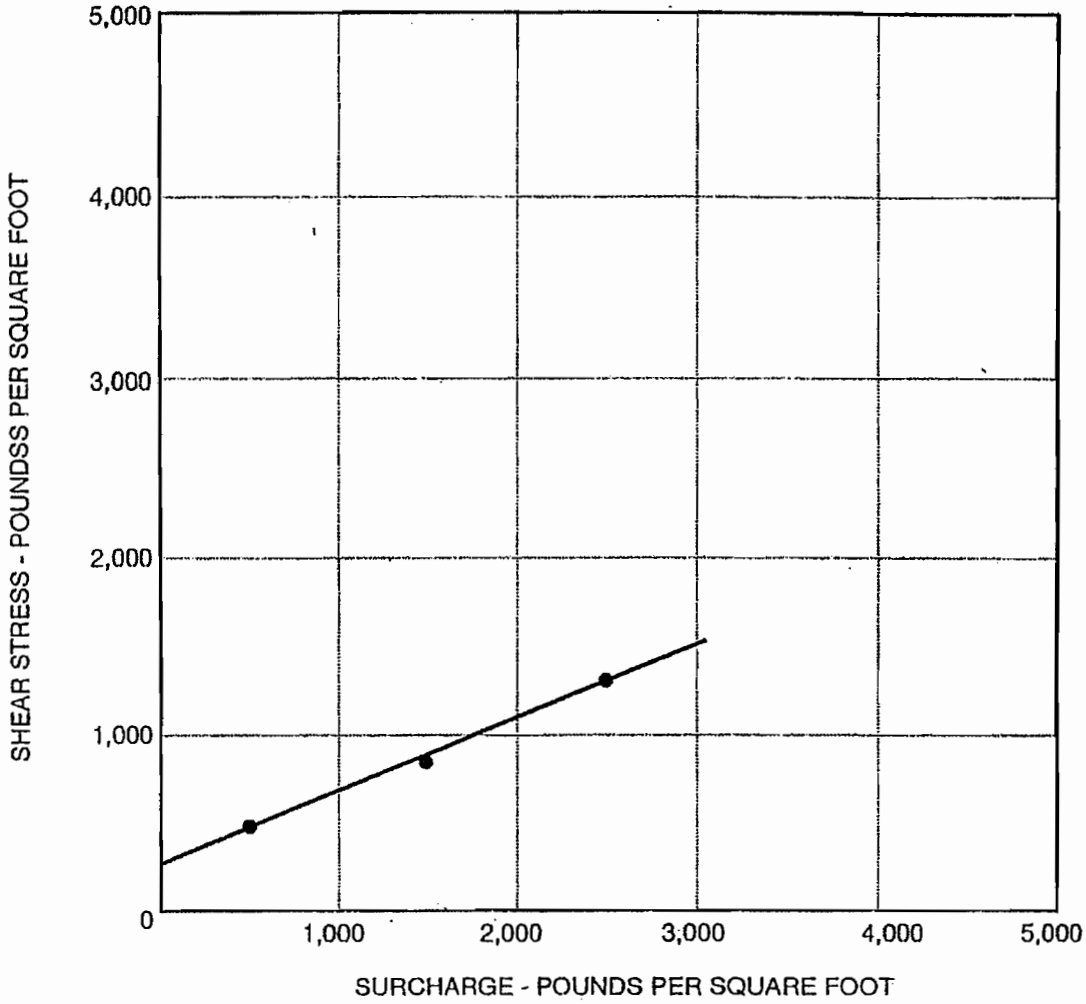
SPECIMEN TYPE Undisturbed		SHEAR STRENGTH	550 psf
DIAMETER (in) 2.42	HEIGHT (in) 6.00	STRAIN AT FAILURE	10 %
MOISTURE CONTENT	29.4 %	CONFINING PRESSURE	750 psf
DRY DENSITY	93 pcf	STRAIN RATE	1.0 %/min
CLASSIFICATION SANDY CLAY (CL), gray-brown		SOURCE B-22 @ 10.5 feet	

OCTEL COMMUNICATIONS
PHASES II AND III
Milpitas, California

UNCONSOLIDATED-UNDRAINED TRIAXIAL COMPRESSION TEST

Treadwell&Rollo

Project No. 1270.05 | Figure A-35



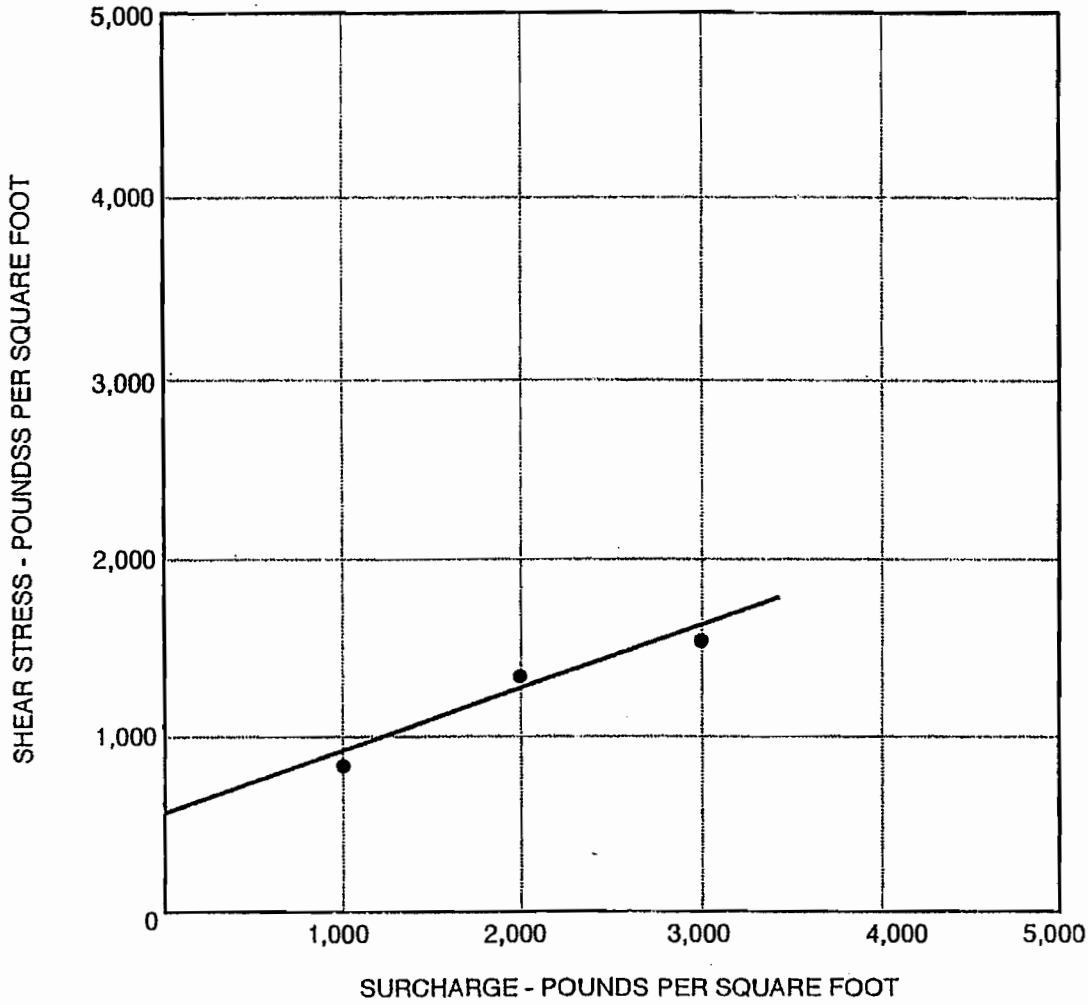
CLASSIFICATION SANDY SILT (ML), brown		SOURCE Boring B-20 @ 5.0 feet	
APPARENT COHESION (PSF) 250		ANGLE OF INTERNAL FRICTION 22.5°	
SPECIMEN NO.	A	B	C
SURCHARGE STRESS (PSF)	500	1,500	2,500
MAXIMUM SHEAR STRESS (PSF)	474	852	1,294
STRAIN RATE (%/MIN)	1.000	1.000	1.000

OCTEL COMMUNICATIONS
PHASES II AND III
Milpitas, California

Treadwell&Rollo

DIRECT SHEAR TEST

Project No. 1270.05 | Figure A-36



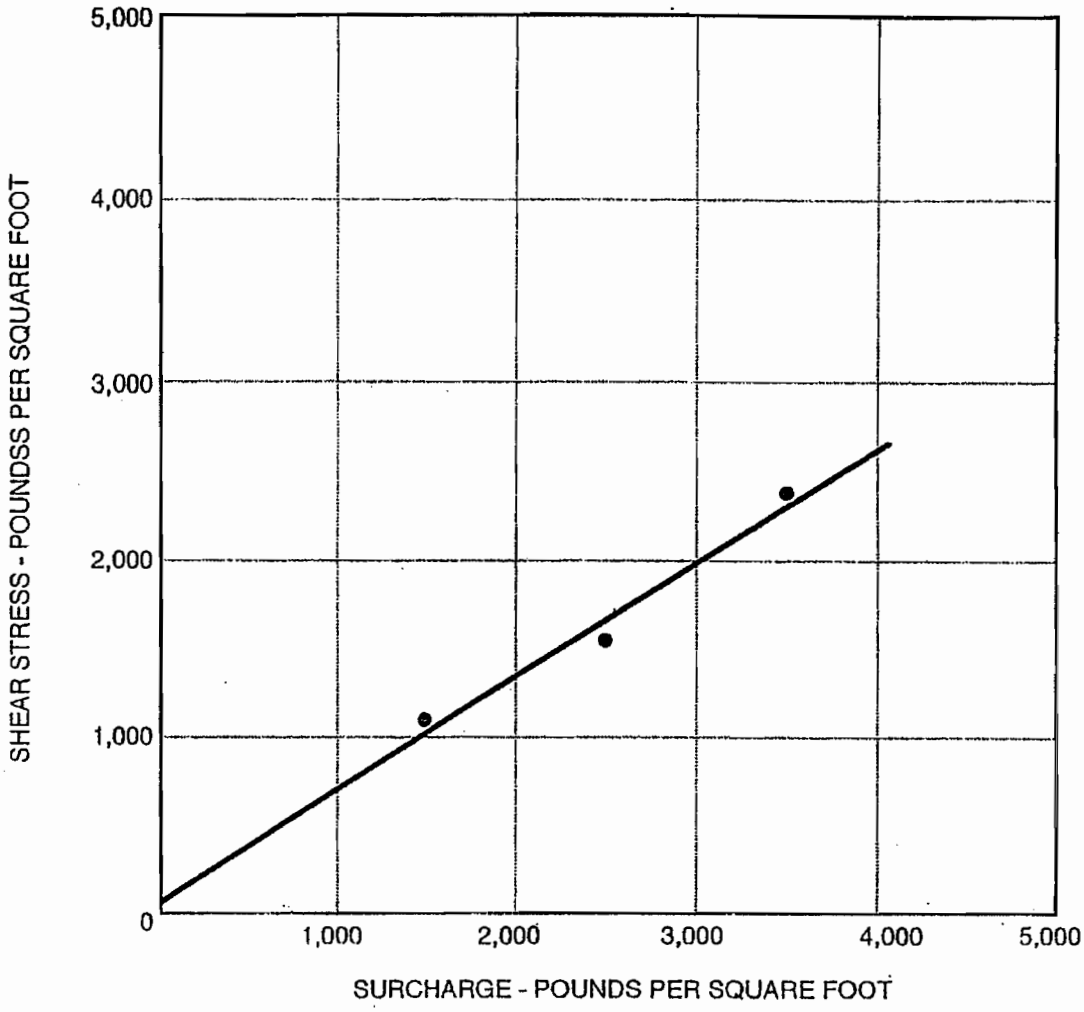
CLASSIFICATION SANDY SILT (ML), brown			SOURCE Boring B-23 @ 2.5 feet
APPARENT COHESION (PSF) 570		ANGLE OF INTERNAL FRICTION 19.5°	
SPECIMEN NO.	A	B	C
SURCHARGE STRESS (PSF)	1,000	2,000	3,000
MAXIMUM SHEAR STRESS (PSF)	821	1,326	1,515
STRAIN RATE (%/MIN)	1.000	1.000	1.000

OCTEL COMMUNICATIONS
PHASES II AND III
Milpitas, California

Treadwell&Rollo

DIRECT SHEAR TEST

Project No. 1270.05 | Figure A-37



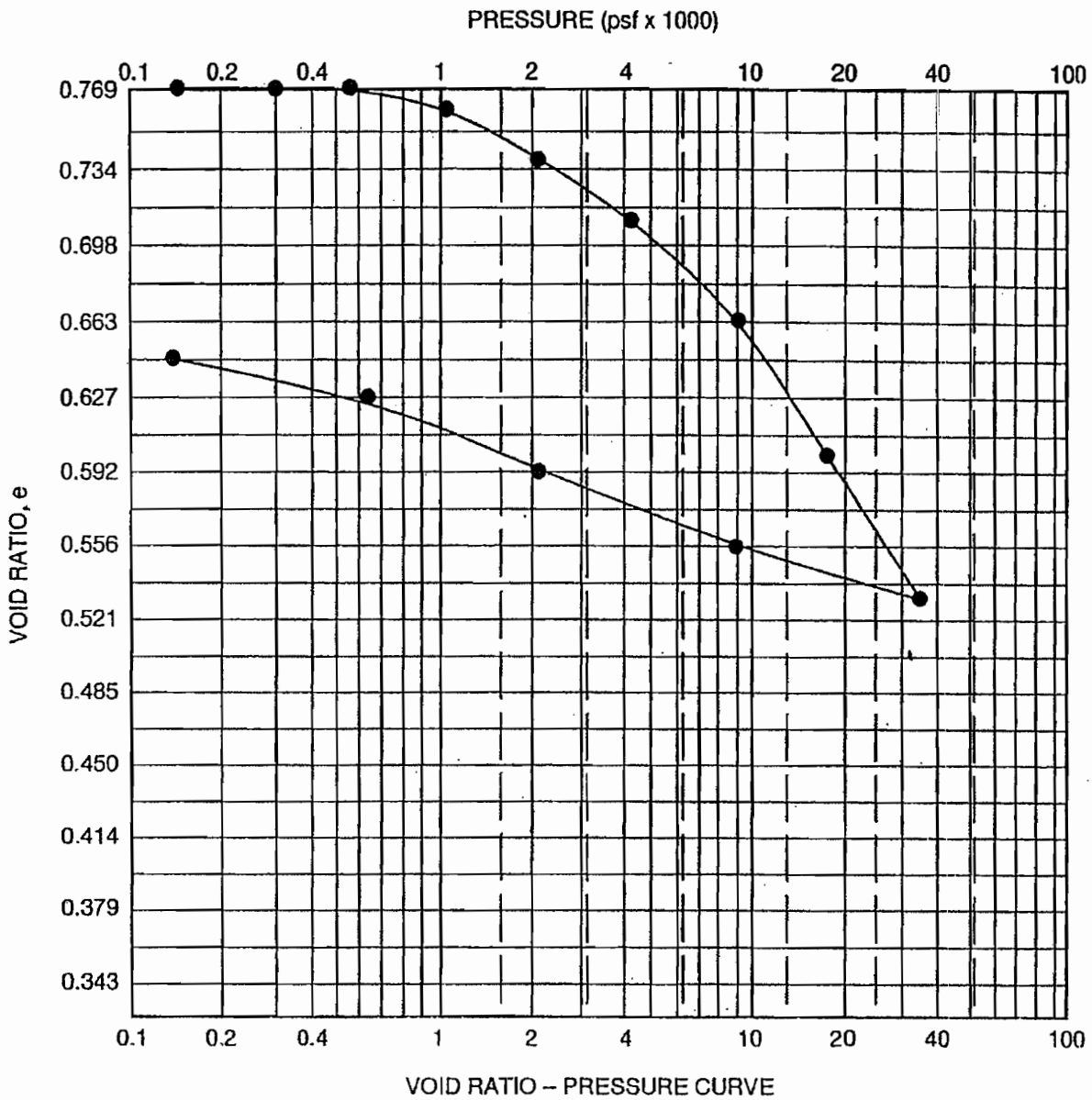
CLASSIFICATION SANDY SILT (ML), brown			SOURCE Boring B-25 @ 5.0 feet
APPARENT COHESION (PSF) 50		ANGLE OF INTERNAL FRICTION 32.5°	
SPECIMEN NO.	A	B	C
SURCHARGE STRESS (PSF)	1,500	2,500	3,500
MAXIMUM SHEAR STRESS (PSF)	1,105	1,547	2,389
STRAIN RATE (%/MIN)	1.000	1.000	1.000

OCTEL COMMUNICATIONS
PHASES II AND III
Milpitas, California

Treadwell&Rollo

DIRECT SHEAR TEST

Project No. 1270.05 | Figure A-38



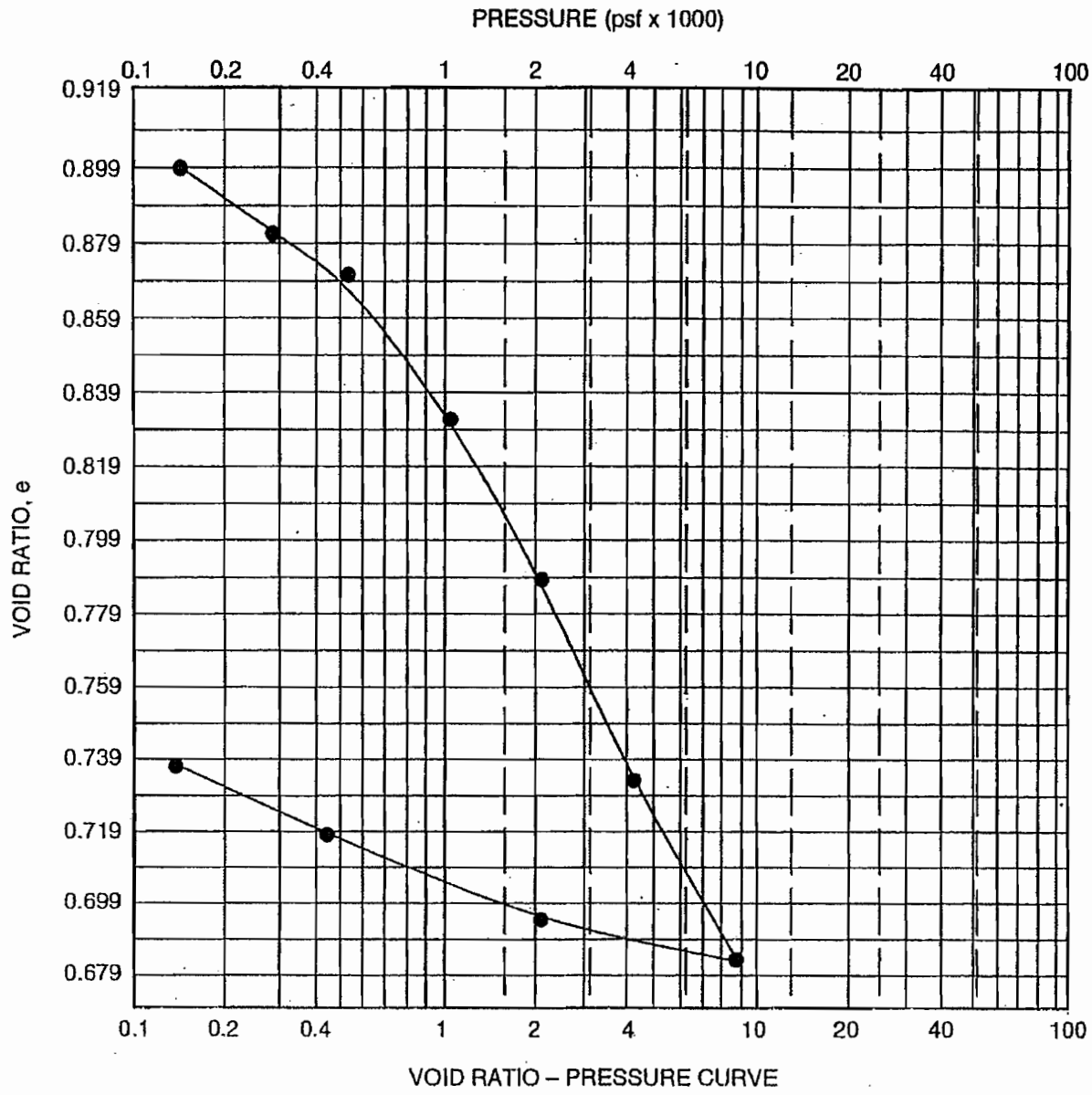
Type of specimen Undisturbed		Condition		Before test		After test	
Diameter (in) 2.37	Height (in) 1.0	Water Content	w_o	24.2 %	w_f	22.9 %	
Overburden Pressure, P_o	1,200 psf	Void Ratio	e_o	0.769	e_f	0.646	
Preconsol. Pressure, P_c	psf	Saturation	S_o	88 %	S_f	100 %	
Compression Ratio, C_{ec}	0.13	Dry Density	γ_d	99 pcf	γ_d	106 pcf	
LL —	PL —	PI —		G_s 2.8 (assumed)			
Classification CLAY (CL), brown		Source B-4 @ 9.5 feet					

OCTEL COMMUNICATIONS
PHASES II AND III
Milpitas, California

Treadwell&Rollo

CONSOLIDATION TEST REPORT

Project No. 1270.05 | Figure A-39



VOID RATIO – PRESSURE CURVE

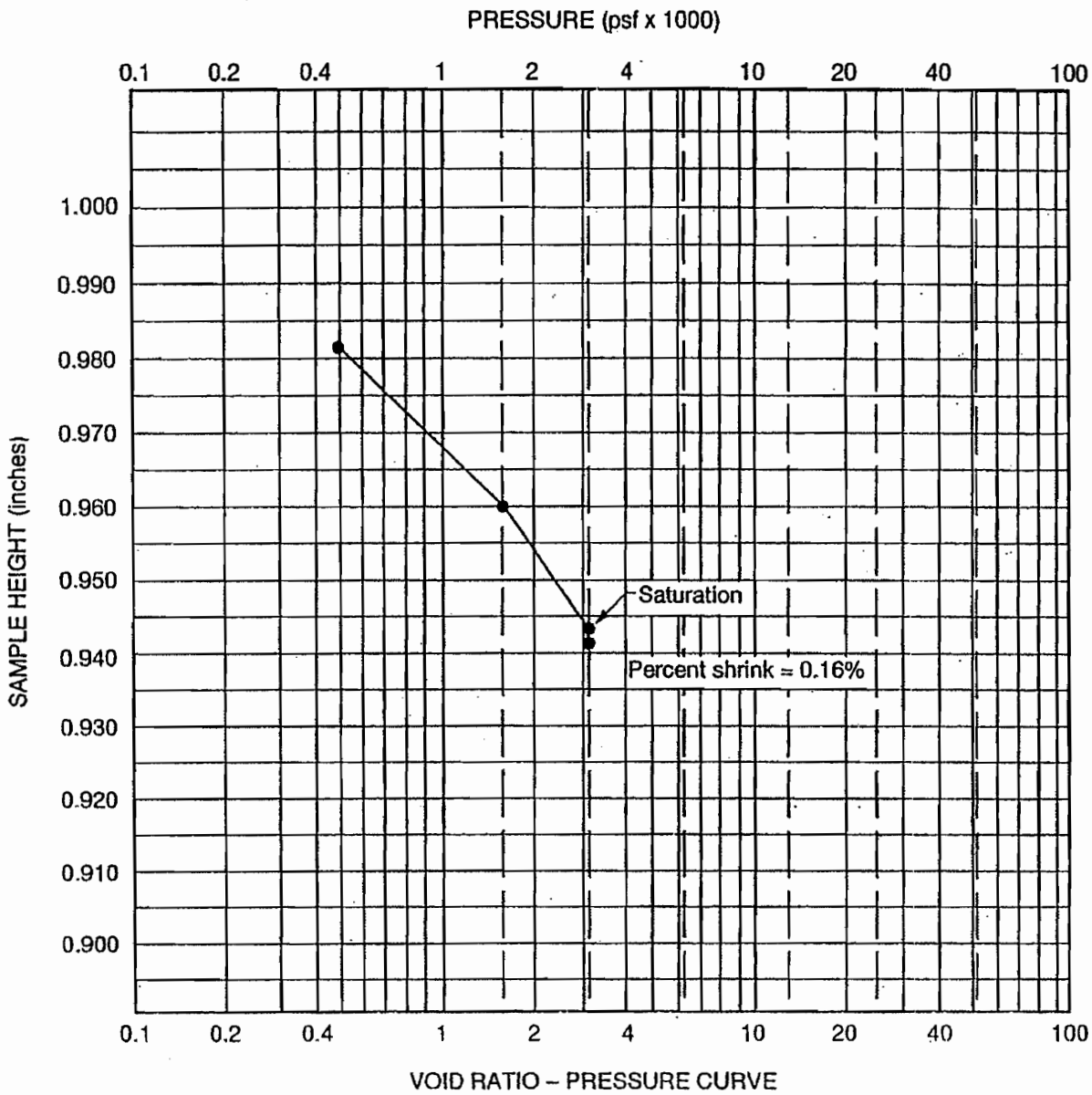
Type of specimen Undisturbed		Condition		Before test		After test	
Diameter (in) 2.37	Height (in) 1.0	Water Content	w_o	33.6 %	w_f	28.0 %	
Overburden Pressure, P_o	1,800 psf	Void Ratio	e_o	0.919	e_f	0.738	
Preconsol. Pressure, P_c	psf	Saturation	S_o	99 %	S_f	100 %	
Compression Ratio, C_{ec}	0.09	Dry Density	γ_d	102 pcf	γ_d	102 pcf	
LL —	PL —	PI —		G_s 2.7 (assumed)			
Classification SANDY CLAY (CL), gray-brown		Source B-22 @ 15.5 feet					

OCTEL COMMUNICATIONS
PHASES II AND III
Milpitas, California

Treadwell&Rollo

CONSOLIDATION TEST REPORT

Project No. 1270.05 | Figure A-40



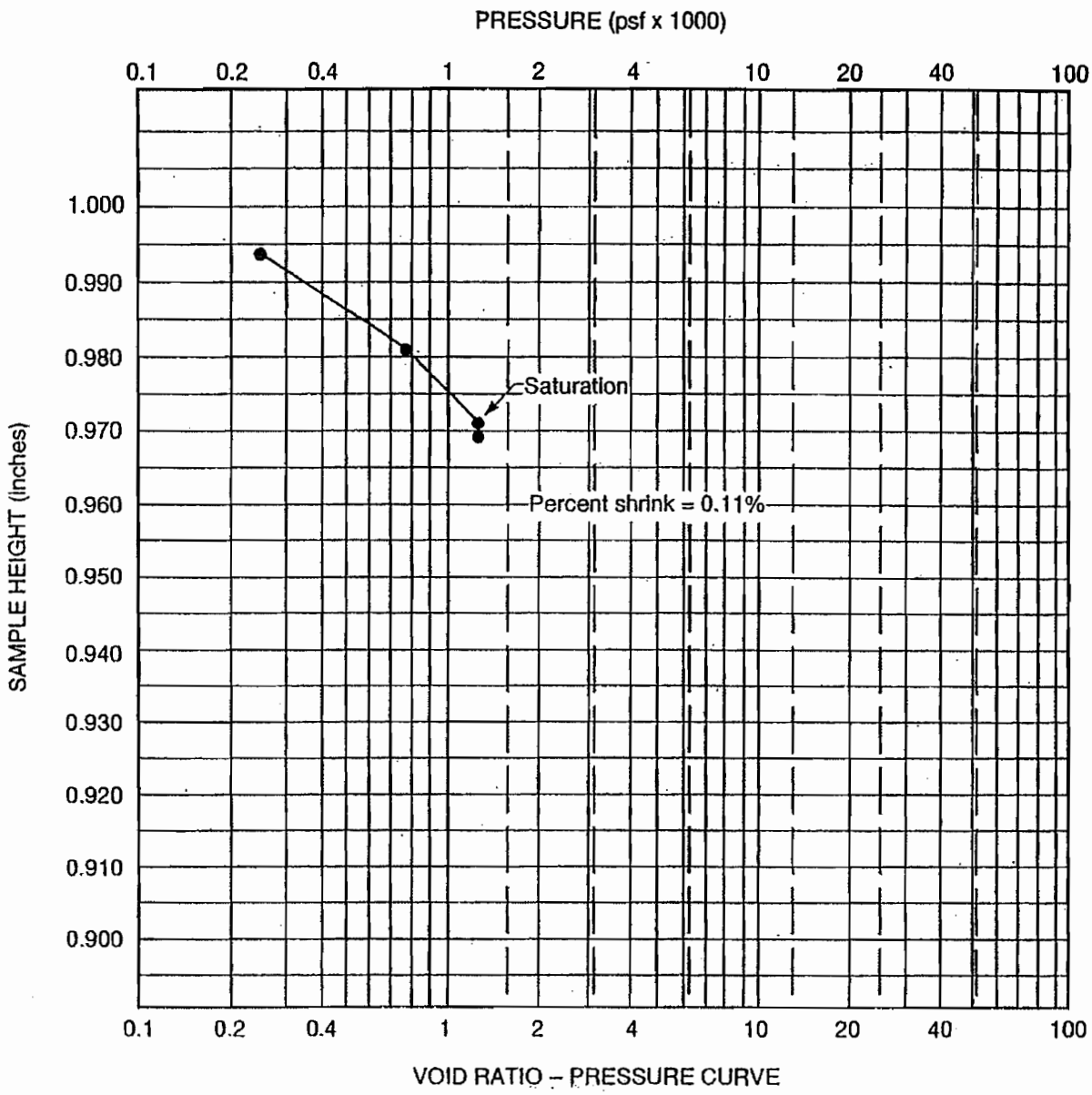
Type of specimen Undisturbed		Condition		Before test		After test	
Diameter (in) 2.42	Height (in) 1.0	Water Content	w_o	30.3 %	w_f	29.8 %	
Overburden Pressure, P_o	690 psf	Void Ratio	e_o	0.89	e_f	0.78	
Maximum Surcharge Pressure	2,500 psf	Saturation	S_o	92 %	S_f	%	
Compression Ratio, C_{ec}		Dry Density	γ_d	89 pcf	γ_d	pcf	
LL —	PL —	PI —		G_s 2.70 (asssumed)			
Classification SANDY SILT (ML), brown		Source B-18 @ 5.5 feet					

OCTEL COMMUNICATIONS
PHASES II AND III
Milpitas, California

SHRINK-SWELL TEST REPORT

Treadwell&Rollo

Project No. 1270.05 | Figure A-41



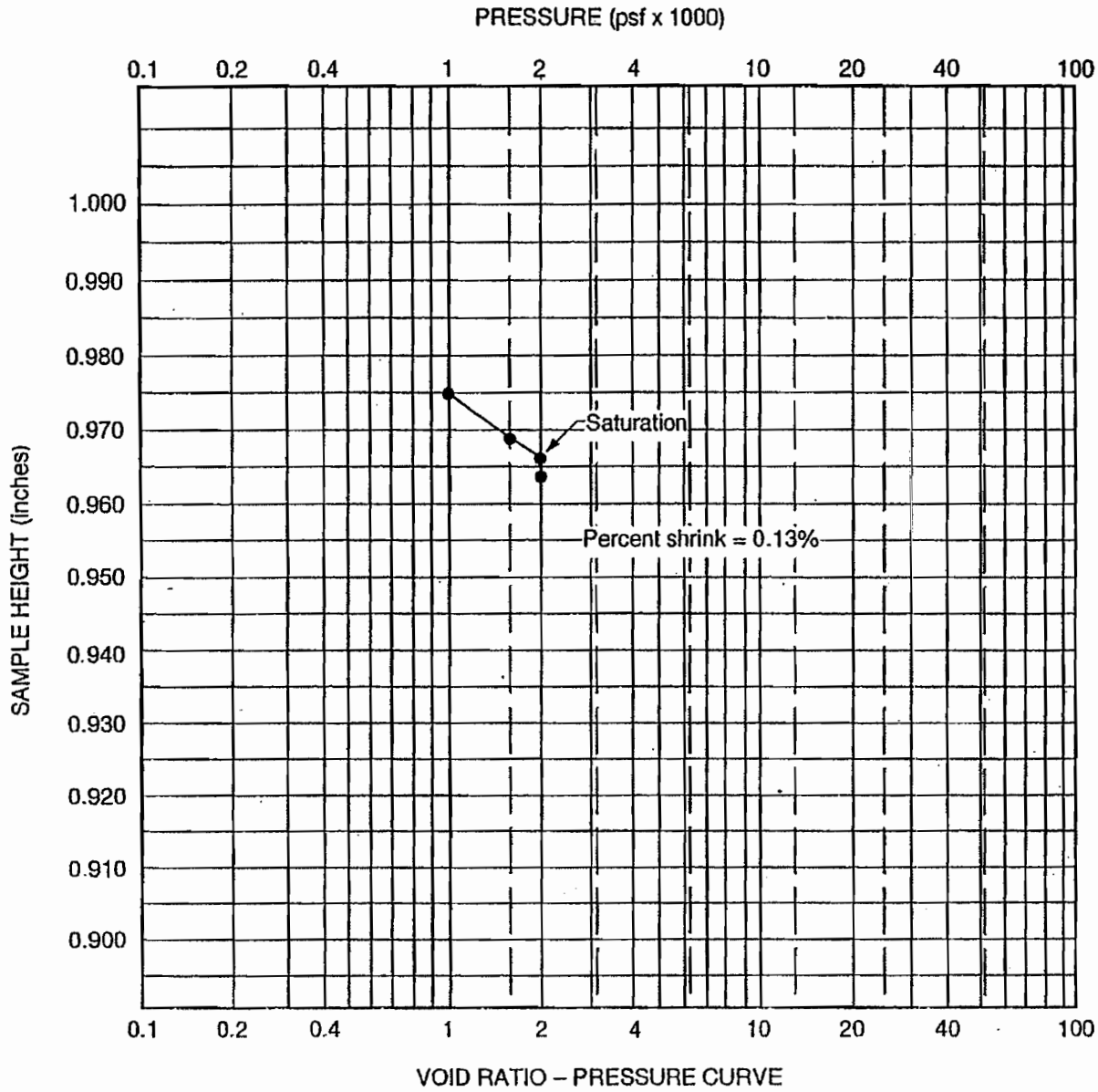
Type of specimen Undisturbed		Condition		Before test		After test
Diameter (in) 2.42	Height (in) 1.0	Water Content	w_o	25.3 %	w_f	26.4 %
Overburden Pressure, P_o	690 psf	Void Ratio	e_o	0.75	e_f	0.69
Maximum Surcharge Pressure	1,250 psf	Saturation	S_o	91 %	S_f	%
Compression Ratio, C_{ec}		Dry Density	γd	96 pcf	γd	pcf
LL —	PL —	PI —		G_s 2.70 (asssumed)		
Classification SANDY SILT (ML), brown			Source B-22 @ 5.5 feet			

OCTEL COMMUNICATIONS
PHASES II AND III
Milpitas, California

SHRINK-SWELL TEST REPORT

Treadwell&Rollo

Project No. 1270.05 | Figure A-42



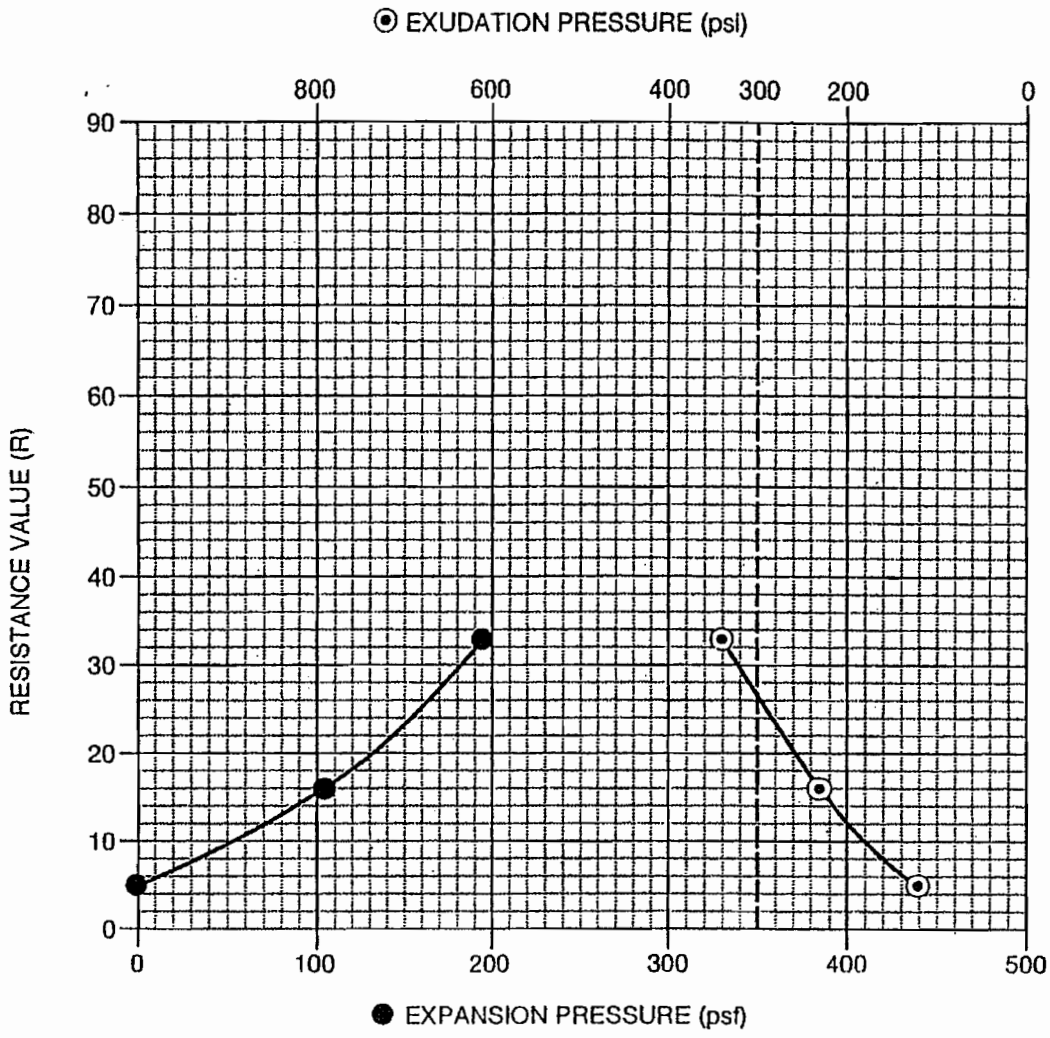
Type of specimen Undisturbed		Condition		Before test		After test	
Diameter (in) 2.41	Height (in) 1.0	Water Content	w_o	22.3 %	w_f	23.1 %	
Overburden Pressure, P_o	310 psf	Void Ratio	e_o	0.72	e_f	0.66	
Maximum Surcharge Pressure	2,000 psf	Saturation	S_o	84 %	S_f	%	
Compression Ratio, C_{ec}		Dry Density	γ_d	98 pcf	γ_d	pcf	
LL —	PL —	PI —		G_s 2.70 (asssumed)			
Classification SANDY SILT (ML), brown		Source B-25 @ 2.5 feet					

OCTEL COMMUNICATIONS
PHASES II AND III
Milpitas, California

Treadwell&Rollo

SHRINK-SWELL TEST REPORT

Project No. 1270.05 | Figure A-43



Specimen No.	A	B	C	
Water Content (%)	18.4	20.8	16.9	
Dry Density (pcf)	109	104	111	
Exudation Pressure (psi)	235	139	343	
Expansion Pressure (psf)	107	0	193	
Resistance Value (R)	16	5	33	

Sample Source	Classification	Sand Equivalent	Expansion Pressure	R Value
Shallow bulk samples (composited)	SANDY SILT (ML - CL) brown		160	26

OCTEL COMMUNICATIONS
PHASES II AND III
Milpitas, California

Treadwell&Rollo

RESISTANCE VALUE TEST DATA

Project No. 1270.05 | Figure A-44

Treadwell&Rollo

DISTRIBUTION

6 copies:	Mr. Shon E. Finch Fairfield Residential, LLC 5510 Morehouse Drive, Suite 200 San Diego, California 92121