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Principals:
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Michael P. Imbriglio
W. Lee Vanderhurst

PRELIMINARY GEOTECHNICAL INVESTIGATION POND 20 - FORMER WESTERN SALT PROPERTY SAN DIEGO, CALIFORNIA

prepared for:

San Diego Unified Port District
P.O. Box 120488
San Diego, California 92110-0488

by

GEOTECHNICS INCORPORATED
Project No. 0298-019-00
Document No. 0-0477

May 23, 2000

(11.1) Geotech

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San Diego Unified Port District
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Attention: Mr. James Trefren

SUBJECT: PRELIMINARY GEOTECHNICAL INVESTIGATION
Pond 20 - Former Western Salt Property
San Diego, California

Dear Mr. Trefren:

We have completed our preliminary geotechnical investigation for the subject Pond 20 project. Our services were completed in accordance with our Agreement for Professional Engineering Services dated April 27, 2000. The results of our investigation are presented in this report, along with our conclusions and preliminary recommendations.

Based on our discussions with you, we understand the specific development is not known at this time, but it may be developed for either commercial, industrial, retail or hotel use. No building locations or grading information is available at this time. Accordingly, our investigation was preliminary in nature and was intended to provide information on the subsurface soil and groundwater conditions and how they may impact the future development. We should conduct a subsequent Geotechnical Investigation once building locations and proposed improvements are known.

We appreciate this opportunity to provide professional services. If you have any questions or require additional services, please do not hesitate to contact us.

GEOTECHNICS INCORPORATED

Robert A. Torres, P.E.
Senior Engineer

John R. Theissen, P.E.
Senior Engineer

Distribution: (4) Addressee

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POND 20 - FORMER WESTERN SALT PROPERTY
SAN DIEGO, CALIFORNIA

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1.0 INTRODUCTION

This report presents the results of our preliminary subsurface investigation for the Pond 20 site, formerly known as the Western Salt Property, located in San Diego, California. The purpose of our investigation was to evaluate the subsurface soil and groundwater conditions at the site, and their impact on future development. The conclusions and recommendations presented in this report are based on our field explorations, laboratory testing, engineering evaluations, and our experience with similar soils and geologic conditions. This report is preliminary and will require a more detailed investigation once building locations and proposed improvements are known.

2.0 SCOPE OF SERVICES

The scope of our services for this project included evaluation of the site geotechnical conditions. The following work was performed as part of this investigation:

- Research and review of the available published geologic maps, topographic maps, stereoscopic photographs, in-house geotechnical reports, and reports by others, pertinent to the site and adjacent properties.

- A geologic reconnaissance of the proposed development area.

- A subsurface exploration consisting of 9 cone penetration test soundings (CPT) and drilling 4 borings to depths ranging between 8 and 40 feet below existing grade. The borings were drilled using a truck-mounted, rotary wash drill rig. Samples were retrieved during drilling using Standard Penetration Test samplers , Shelby tubes, and modified California samplers. The results of our field investigation are presented in Appendix B of this report.

- Laboratory testing on samples collected during drilling. Testing included moisture content and density, particle size analyses, Atterberg limits, consolidation, shear strength, expansion, and sulfate content. The results of our laboratory testing are presented in Appendix C of this report.

- Engineering analysis of field and laboratory data.

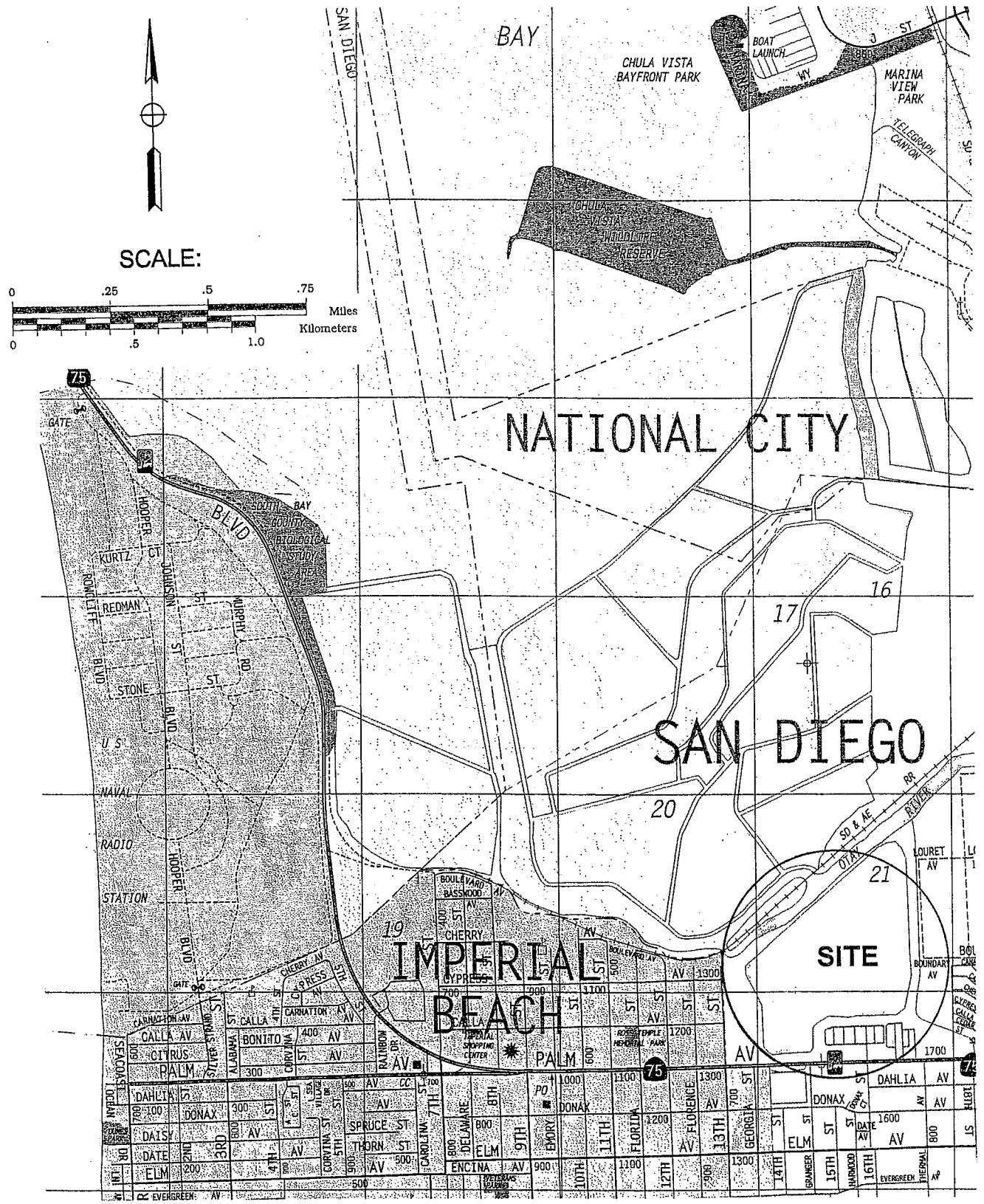
- Preparation of this preliminary report summarizing our findings, conclusions, and preliminary recommendations. This report provides an evaluation of foundation types, including spread footings or deep foundations, recommendations for excavation and grading, evaluation of liquefaction potential and other geologic and seismic hazards, and an estimate of settlement potential due to the addition of fill and structural loads.

3.0 SITE DESCRIPTION

The subject property consists of approximately 95 acres at the southerly terminus of the San Diego Bay. The southerly half of the site will be developed, but the northerly half will remain open space. The subject site is located within the former wetlands of the San Diego Bay and the drainage basin of the Otay River. The site was also formerly used as evaporation ponds for production of salt. The location of the site is shown in Figure 1, Site Location Map. Current site elevations vary from about 8 to 16 feet above mean sea level (MSL). The site layout in relation to our boring and CPT locations is shown on the Site Plan, Figure 2.

4.0 GEOLOGY AND SUBSURFACE CONDITIONS

The subject site is located within the coastal plain section of the Peninsular Range Geomorphic Province of California. The coastal plain consists, in general, of subdued land forms underlain by Cenozoic sedimentary formations. Based on published maps (Kennedy and Siang, 1977) and our subsurface exploration, the subject site is underlain at depth by Quaternary-age sandstone associated with the Bay Point Formation and unnamed near-shore marine sandstone, alluvium, and minor fill soils. The approximate locations of the borings and CPT tests are shown on the Site Plan, Figure 2. Logs describing the subsurface conditions encountered are given in the figures of Appendix B. Generalized descriptions of the materials encountered are presented as follows from oldest to youngest.



Reference: 2000 Thomas Brothers Guide

Geotechnics Incorporated

SITE LOCATION MAP

Project No. 0298-019-00
Document No. 0-0477

FIGURE 1

4.1 Bay Point Formation / Unnamed Near-shore Marine Sandstone

The Bay Point Formation and an unnamed near-shore marine sandstone underly the site at depth. As observed in our borings, this formation consisted generally of gray to reddish brown, silty, fine grained sandstone. Beds of gravel were also observed. The formational materials were observed to be generally dense to very dense and weak to moderately cemented. Drilling refusal was encountered in the Bay Point Formation in all of our CPT's.

4.2 Estuarine Deposits

The estuarine deposits were observed beneath most of the site, in the lower elevations. The maximum depth observed in our borings and CPT's was approximately 40 feet. The estuarine deposits consist of interbedded layers of sand, silt and clay. The soils varied from poorly graded sand (SP), poorly graded sand with silt (SP-SM), silty sand (SM), clayey sand (SC), sandy silt (ML), sandy clay (CL), and high plasticity clay (CH) based on the Unified Soil Classification System.

The estuarine deposits were typically dark brown to dark gray in color. The clays varied from low to high plasticity. The sands were generally fine grained. Standard penetration test (SPT) blow counts and CPT soundings indicated that the sand beds were very loose to medium dense. The clays were generally very soft to soft, with occasional stiff layers. Consolidation testing indicates that the estuarine deposits are compressible and highly susceptible to settlement upon placement of fill or structural loads. The CPT and boring data show bedding consistent with an estuarine depositional environment, with relatively discontinuous layers of sand, silt and clay.

4.3 Fill

The site is bounded by an earthen berm with a variable depth of undocumented fill which is believed to be associated with development of the salt pond. As observed in Boring 2, the fill consists of a dark gray, moist and soft sandy clay (CL) of low to moderate plasticity. The fill soils are also considered highly compressible and susceptible to settlement due to structural or fill loads.

4.4 Groundwater

Groundwater was observed in Borings 1, 2, and 3 drilled over the Estuarine deposits. Groundwater was not observed in Boring 4 where it was drilled in the upper portions of the site within Bay Point Formation. As observed in the borings, the depth to groundwater varied from approximately 2 to 7 feet below grade. However, groundwater was observed at the surface in a majority of the southerly half of the site. The exploratory borings were left open for 30 minutes to one hour to allow the water level to stabilize. The site is situated roughly 8 to 16 feet above mean sea level, and the portion of the Otay River just north and east of the site typically contains surface water. Consequently, it is our opinion that the observed groundwater is a permanent feature associated with the bay and Otay River.

5.0 GEOLOGIC HAZARDS

The subject site is located within an area which has previously been mapped by the City of San Diego Seismic Safety Element as having a high potential for liquefaction. Seismic hazards at the site may be caused by ground shaking during seismic events on nearby or distant active faults. Based on maps used in conjunction with the 1997 Uniform Building Code, the site is located approximately 6.8 miles from the Rose Canyon fault zone. An inferred fault is mapped approximately 1.5 miles east of the site in the City of San Diego Seismic Safety Study. Evidence of past soil failures, landslides, or active faulting on the site was not encountered.

5.1 Ground Rupture

No evidence of active faulting was found during the subsurface investigation, nor have any potentially active faults been mapped within the subject site. Accordingly, ground rupture is not considered to be a significant hazard at the site.

5.2 Liquefaction

Liquefiable soils typically consist of cohesionless sands and silts that are loose to medium dense, and saturated. To liquefy, these soils must be subjected to ground shaking of

sufficient magnitude and duration. The site is underlain by generally loose and soft sandy to silty estuarine deposits and a shallow groundwater table. Based on our preliminary evaluation, the potential for liquefaction to occur is high.

5.3 Subsidence

The subject site is not within an area known for fluid extraction (oil or water), nor is the area known for past cases of subsidence due to fluid removal (Alfors, 1973). It is our opinion that subsidence due to the extraction of fluids is remote. If not mitigated, the potential for significant seismically induced subsidence is anticipated to be high due to the soft to loose nature of the soils underlying the site.

5.4 Landslides and Lateral Spreading

Evidence of ancient landslides or slope instabilities was not observed at the site during our investigation. Accordingly, the potential for landslides to significantly impact the site is considered low. Due to the proximity of the San Diego Bay, the site may be susceptible to lateral spreading associated with a seismic event on a nearby active fault.

5.5 Tsunamis, Seiches, and Earthquake-Induced Flooding

Given the site's relatively low elevation above sea level and short distance from the ocean, damage due to tsunamis (seismically induced waves) is considered moderate to high. Although the occurrence of tsunamis in southern California is relatively infrequent, documented accounts do exist. Houston and Garcia (1974) published predicted run-up heights for coastal Southern California, using 100-year and 500-year events. Predicted run-up heights for the subject site range from approximately 5.3 feet for the 100-year recurrence, and approximately 10.0 feet for the 500-year recurrence. Based the site's elevation, most of the property appears to be above the elevation of the 100-year predicted tsunami runup, with some areas below the predicted 100-year runup elevation. The site may be subject to inundation by a 500-year predicted tsunami runup.

The site is adjacent to a large relatively confined body of water. Accordingly, the potential for seiches (oscillations in a body of water due to earthquake shaking) is considered moderate to high. Flooding after periods of rainfall may occur. The potential for flooding should be evaluated by the project civil engineer.

6.0 CONCLUSIONS

Based on our preliminary investigation and evaluation, it is our opinion that the site could be developed for proposed commercial, retail, industrial or hotel uses. However, mitigation of compressible and liquefiable soils will be necessary to provide suitable support for proposed improvements. From a geotechnical standpoint, the following conditions should be considered in determining site use and development costs:

- The subsurface soils are highly compressible and will consolidate upon placement of fill and structural loads. Areal settlement will occur over a period of time. The amount of settlement and length of time for settlement to occur is proportional to the height of fill placed and depth of compressible soils. Consolidation could be accelerated by the addition of a surcharge fill placed above the planned level of fill.

- Due to their loose relative density and the presence of a high water table, the existing soils are susceptible to liquefaction and will require mitigation prior to building construction.

- Heavy structural loads and basement foundations should be supported on a system of driven piles that extend into the underlying Bay Point Formation materials. One- to two-story structures could be supported on spread footings after completion of the consolidation period.

- Stabilization of the existing surface soils using geotextile and gravel will be necessary prior to placing fill soils.

- The on-site soils have a high potential for corrosion which may affect concrete in contact with the existing soils, ferrous metals, and underground buried utilities.

7.0 PRELIMINARY RECOMMENDATIONS

The following sections presents preliminary recommendations regarding geotechnical aspects of the site which will impact development. The following preliminary recommendations are based on our field explorations, laboratory testing, engineering evaluations, and our experience with similar soils and geologic conditions. This report is preliminary and will require a more detailed investigation after building locations and proposed improvements are known.

7.1 Grading

The site is currently underlain by highly compressible soils and a shallow groundwater table. The use of heavy equipment in close proximity to groundwater will create unstable subgrade conditions. Prior to placing fill, the subgrade may be stabilized using a layer of woven geotextile Type 270 WS, per "Greenbook" specification 213-2. As an alternate, Mirafi 700X geotextile or Tensar BX1100 geogrid, installed in accordance with the manufacturer's requirements, may be used to stabilize the subgrade. Generally, the geotextile or geogrid may be placed directly on the pumping subgrade after removing any large tire rutting. Geotextile laps should be at least 24 inches wide and be tightly secured in accordance with manufacturer's recommendations. A minimum of 2 feet of 1½-inch minus crushed rock should then be placed on top of the geotextile. The crushed rock material should be compacted to achieve a firm and unyielding condition.

Care should be taken during gravel placement to ensure the geotextile is not moved out of position or overstressed. Crushed rock material may be placed in one lift using a track dozer. To avoid overstressing the geotextile, the gravel should be pushed ahead of the dozer from end dumped piles. Geotextile should be stored in areas where damage by delivery trucks or equipment can be prevented.

Imported fill soils should be predominantly granular and have an expansion index of no more than 20 based on UBC Test Method 18-2 or ASTM D4829. The "open space" area north of the site may be used as a source of fill. We recommend that only the upper approximately 2 feet of existing sandy soil be used, and not the underlying clay and silt materials.

All fill and backfill to be placed in association with site construction should be accomplished slightly above optimum moisture content and using equipment that is capable of producing a uniformly compacted lift. The fill should be compacted to at least 95 percent relative compaction as defined by ASTM D 1557. Fill slopes should be constructed at a maximum inclination of 2:1 (horizontal to vertical). Fill slopes should be overbuilt and surcharged with the full height of fill extending above the proposed toe of slopes to consolidate the compressible soils beneath them and improve global stability.

7.2 Areal Settlement

Based on the current site elevations, we estimate the site will be raised approximately 8 to 10 feet to match the street grades along Palm Avenue. For that height of fill our settlement analysis indicates total areal settlement on the order of approximately 12 to 16 inches may occur on the northerly end of the site where the Estuarian soils are up to approximately 40 feet deep. On the southerly end where the Estuarian soils are on the order of approximately 25 feet deep, areal settlement on the order of approximately 8 to 10 inches may occur.

The time for settlement to occur may range between 14 to 16 months, depending on the height of fill and depth of compressible materials. This time period may be reduced to about 8 to 10 months by placing an additional 10-foot surcharge above the planned level of fill. Settlement may be further accelerated by the placement of wick drains spaced uniformly in a grid pattern. During grading, the rate and amount of settlement should be monitored by placing settlement monuments, pneumatic settlement cells, or magnetic extensometers. By monitoring the settlement, one can determine if settlement is completed and when construction can commence.

7.3 Foundations

Heavy loaded structures should be supported on a system of deep, driven piles. We also recommend that basement foundations in close proximity to the soft, compressible soils also be supported on driven piles. The piles may consist of 12-, 16-, or 24-inch square pre-cast concrete piles. For piles extended a minimum of 5 to 10 feet into Bay Point Formation, capacities of up to 150 to 200 kips per pile may be achieved. Pile lengths on the order of 50 to 35 feet in length may be necessary depending on the building locations.

To eliminate frictional downdrag loads, or negative skin friction, the piles should not be installed until areal settlement has taken place. Otherwise, significant loads may develop on the piles causing excessive settlement and/or pile failure. Final pile capacities should be provided by conducting a foundation investigation after building locations are known and after consolidation has occurred.

Spread footings, post-tension slabs, or grade beam foundations may be used for lightly loaded, one- to two- story structures, provided sufficient time has been allowed for areal settlement to occur. A bearing value of up to 2,500 pounds per square foot may be used for preliminary foundation design. We recommend shallow foundations be supported on a minimum 5-foot compacted fill mat placed uniformly beneath proposed structures.

7.4 Excavations Below Grade

Proposed buildings constructed at street level grade could have one level of basement beneath them. Basement levels should be water-proofed and designed for hydrostatic pressure due to possible variations in groundwater levels. Any excavations deeper than one basement level will encounter water seepage, instability, and soil heave at the base. If deep excavations are proposed, they should be properly shored with sheet piling extending to the required depths to prevent basement heave.

7.5 Soil Corrosivity

Soil samples were tested for water-soluble sulfate content to evaluate the general degree of sulfate attack on concrete in contact with soil. The test results are reported in Appendix C in terms of the percentage by weight of the water soluble sulfate in the soil. Based on the results of our test, the on-site soils have a severe degree of sulfate attack on concrete in contact with the soil. This would require that concrete piles be made using Type V cement. In addition, we anticipate the soil is high in salinity and would be highly corrosive to ferrous metals.

8.0 LIMITATIONS OF INVESTIGATION

This investigation was performed using the degree of care and skill ordinarily exercised, under similar circumstances, by reputable geotechnical consultants practicing in this or similar localities. No other warranty, expressed or implied, is made as to the conclusions and professional opinions included in this report.

The samples taken and used for testing and the observations made are believed representative of the project site; however, soil and geologic conditions can vary significantly between borings. As in most projects, conditions revealed by excavation may vary from the preliminary findings. If this occurs, the changed conditions must be evaluated by the geotechnical consultant and additional recommendations made, if warranted.

This report is issued with the understanding that it is the responsibility of the owner, or of his representative, to ensure that the information and recommendations contained herein are brought to the attention of the necessary design consultants for the project and incorporated into the plans, and the necessary steps are taken to see that the contractors carry out such recommendations in the field.

The findings of this report are valid as of the present date. However, changes in the condition of a property can occur with the passage of time, whether due to natural processes or the work of man on this or adjacent properties. In addition, changes in applicable or appropriate standards of practice may occur from legislation or the broadening of knowledge. Accordingly, the findings of this report may be invalidated wholly or partially by changes outside our control. Therefore, this report is subject to review and should not be relied upon after a period of three years.

This report is preliminary and should be followed with a final geotechnical investigation after specific building locations and proposed improvements are determined.

We appreciate this opportunity to be of professional service. Please call if you have any questions or require additional services.

Sincerely,

GEOTECHNICS INCORPORATED

Robert A. Torres, P.E. 43077
Senior Engineer

John R. Theissen, C.E. 28313, G.E. 825
Senior Engineer

APPENDIX A

REFERENCES

Alfors, J.T., Burnett, J.L, Gay, T.E. (1973). Urban Geology: Master Plan for California, California Division of Mines and Geology Notes, Bulletin 198.

American Society for Testing and Materials (1992). *Annual Book of ASTM Standards, Section 4, Construction, Volume 04.08 Soil and Rock; Dimension Stone; Geosynthetics*, ASTM, Philadelphia, PA, 1296 p.

Anderson, J. G. , Rockwell, T. K., Agnew, D. C. (1989). *Past and Possible Future Earthquakes of Significance to the San Diego Region, Earthquake Spectra, Vol. 5, No. 2. pp 299-335.*

California Division of Mines and Geology (1975). *Recommended Guidelines for Determining the Maximum Credible and the Maximum Probable Earthquakes, California Division of Mines and Geology Notes, Number 43.*

California Department of Conservation, Division of Mines and Geology (1992). *Fault Rupture Hazard Zones in California, Alquist-Priolo Special Studies Zone Act of 1972*: California Division of Mines and Geology, Special Publication 42.

California Department of Conservation, Division of Mines and Geology (1998). *Maps of Known Active Fault Near-Source Zones in California and Adjacent Portions of Nevada*, February 1998.

City of San Diego (1995). Seismic Safety Study, Geologic Hazards and Faults, Sheets 5 and 6, plot date: August 21.

Houston, J.R., and Garcia, A.R. (1974). Type 16 Flood Insurance Study: Tsunami Predictions for Pacific Coastal Communities: U. S. Waterways Experiment Station, Technical Report, H-74-3.

International Conference of Building Officials, 1997, Uniform Building Code.

Jennings, C. W. (1975). Fault Map of California, California Division of Mines and Geology, California, Geologic Data Map Series.

Kennedy, M. P., and Siang, S. T. (1977). Geology of National City, Imperial Beach and Otay Mesa Quadrangles, Southern San Diego Metropolitan Area, California: California Division of Mines and Geology, Map Sheet 29.

Ninyo & Moore (1999). *Preliminary Geotechnical Evaluation, South Bay Development Study, Imperial Beach, California,* dated August 29, 1989.

Robertson, P.K. and Campanella, R.G. (1989). *Guidelines for Geotechnical Design Using CPT and CPTU,* University of B.C., Vancouver, B.C., V6T 1Z4.

Seed, H.B., and Idriss, I.M. (1982). *Ground Motions and Soil Liquefaction during Earthquakes*: Berkeley, California, Earthquake Engineering Research Institute, 134p.

APPENDIX B

SUBSURFACE EXPLORATION

The subsurface exploration consisted of nine Cone Penetration Test soundings and four borings. The CPT's were conducted in general accordance with ASTM D5778, and were pushed using a PowerProbe direct push rig anchored with a 6-inch-diameter ground anchor. The borings were drilled using an 8-inch-diameter, rotary wash drill rig using drilling mud to prevent the hole from caving. The CPT's and borings were extended to depths of between approximately 8 and 40 feet below the existing ground surface. The approximate locations of the borings and CPT soundings are shown on Figure 2. Logs describing the subsurface conditions encountered are presented on the following Figures B-1 through B-14. Bulk and relatively undisturbed samples were obtained from the borings at selected intervals.

Bulk samples were sealed in plastic bags, labeled, and returned to the laboratory for testing. Relatively undisturbed samples were collected using thin-walled Shelby tubes in general accordance with ASTM D1587 and a modified California (CAL) sampler . The CAL sampler is a ring-lined tube with an inside diameter of 2 $^3/_8$ inches and an outside diameter of 3 inches. Ring samples were sealed in plastic bags, placed in rigid plastic containers, labeled, and returned to the laboratory for testing. The hammer used to drive the CAL sampler weighed 140 pounds, with a free fall of 30 inches. The number of blows needed to drive the CAL sampler 12 inches is shown on the attached logs. Relatively undisturbed samples of cohesive soils were collected using Shelby tubes using ASTM D 1587-94 as a guideline. The Shelby tubes used for sampling have an outside diameter of 3 inches and wall thickness of about 1/16 inch.

The boring and CPT locations and elevations were established by field surveys conducted by Port personnel. Soil conditions at locations between the borings may be substantially different from those at the specific locations explored. It should be recognized that the passage of time can result in changes in the soil conditions reported in our boring logs.

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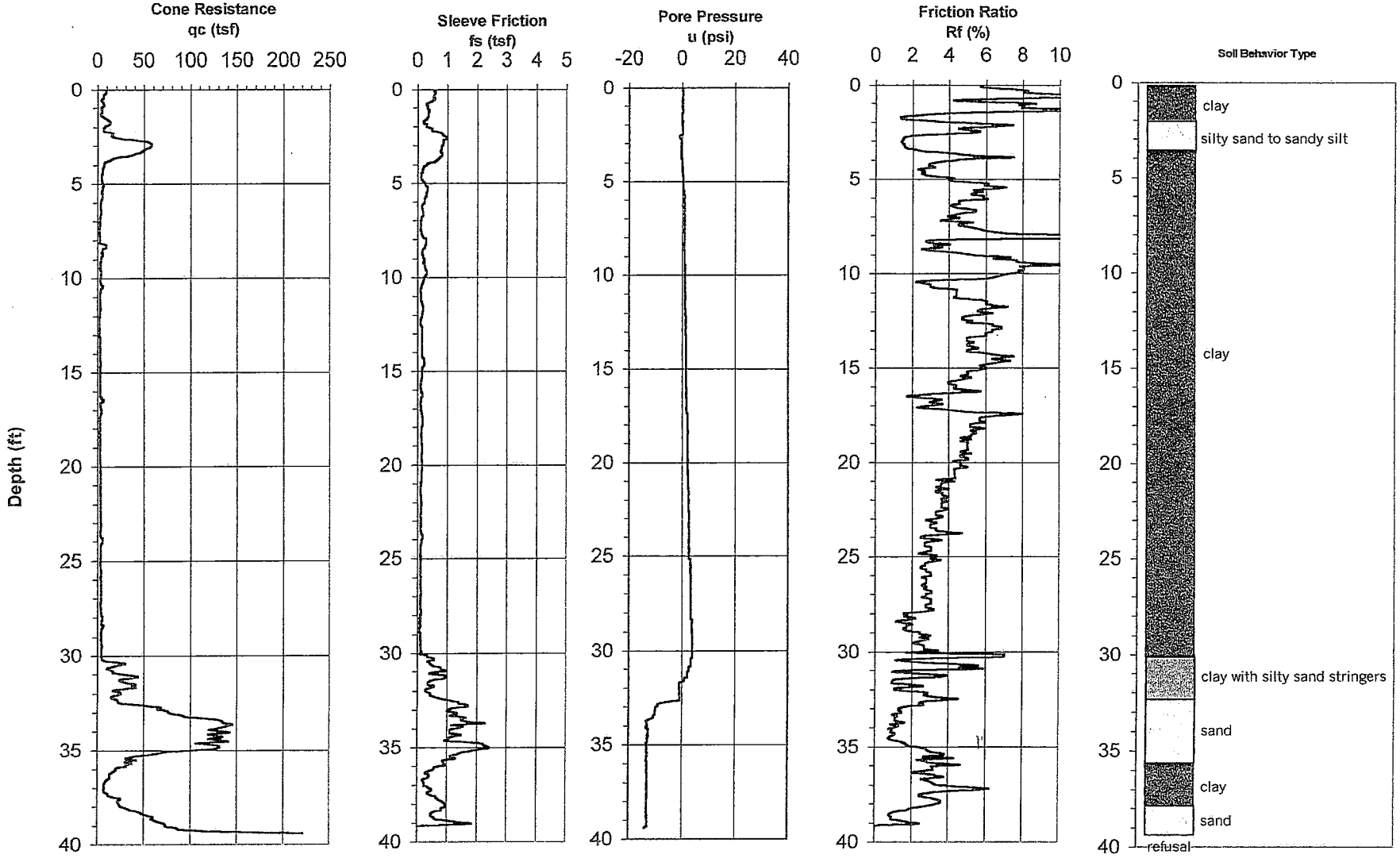
LOG OF CPT-1

Elevation: 14.1' MSL
Date: 4/12/00

Project No. 0298-019-00
Document No. 0-0477

FIGURE B-1



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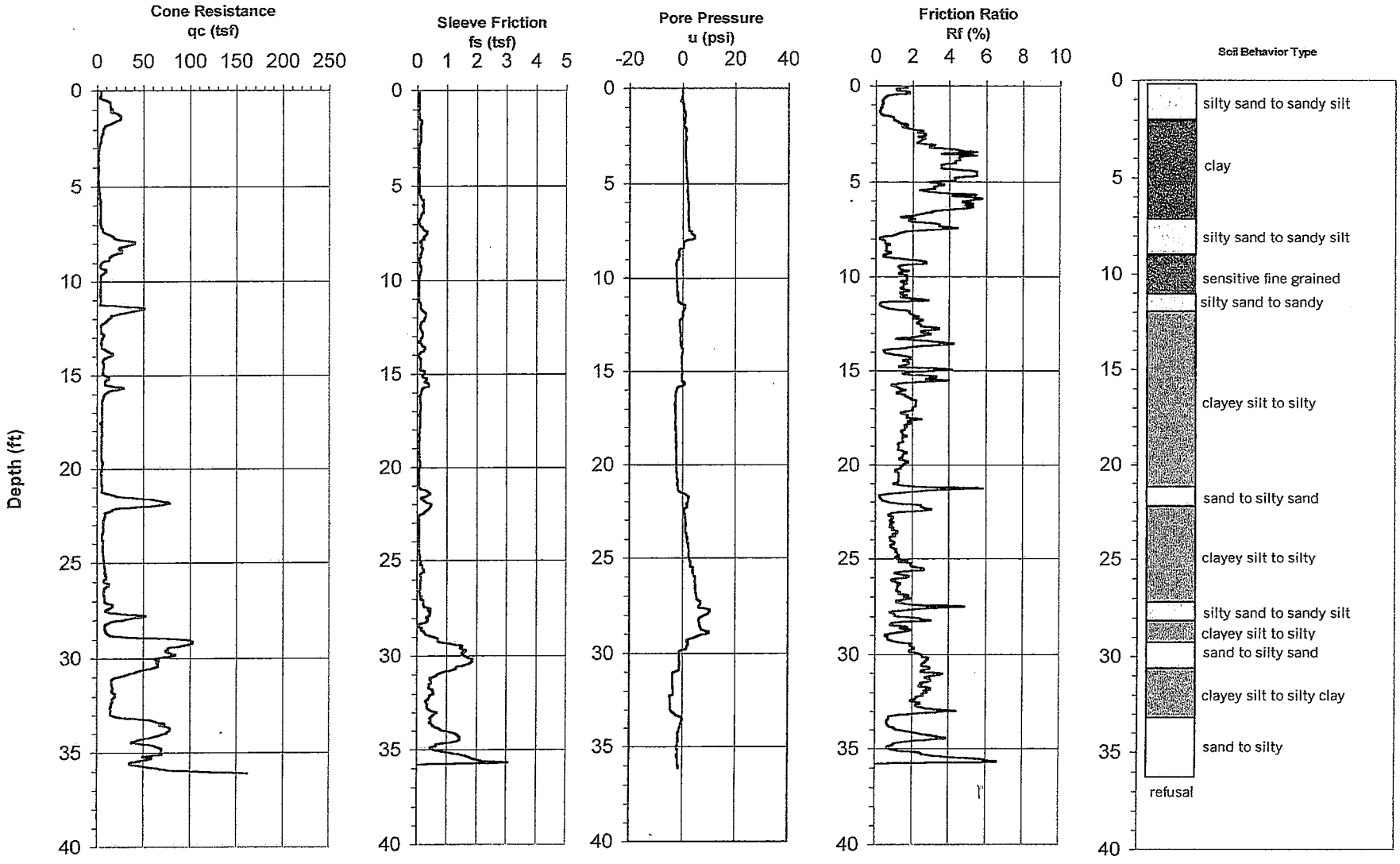
LOG OF CPT-2

Elevation: 8.4' MSL
Date: 4/12/00

Project No. 0298-019-00
Document No. 0-0477

FIGURE B-2



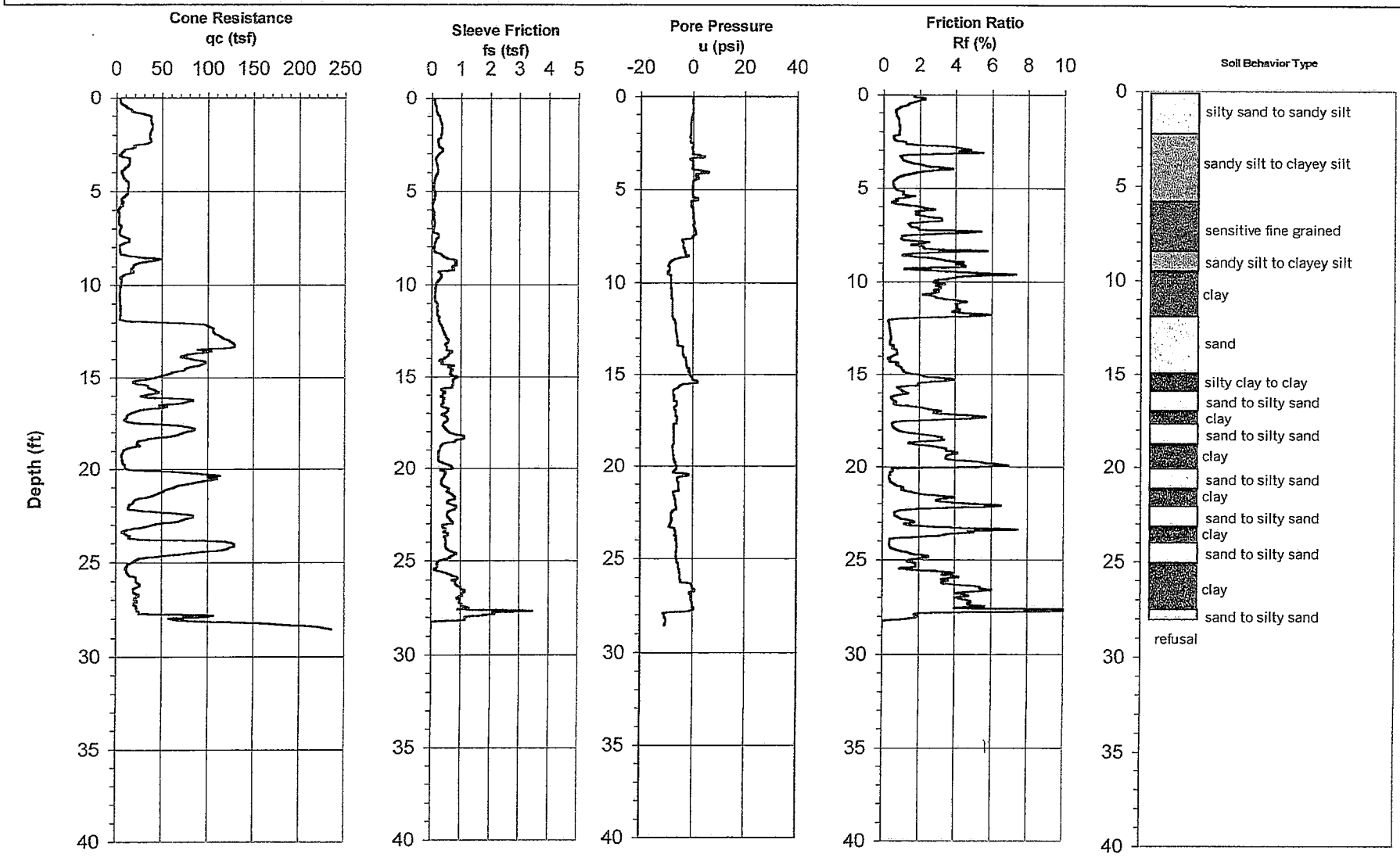
Geotechnics Incorporated

LOG OF CPT-3

Elevation: 11.4' MSL
Date: 4/12/00

Project No. 0298-019-00
Document No. 0-0477

FIGURE B-3



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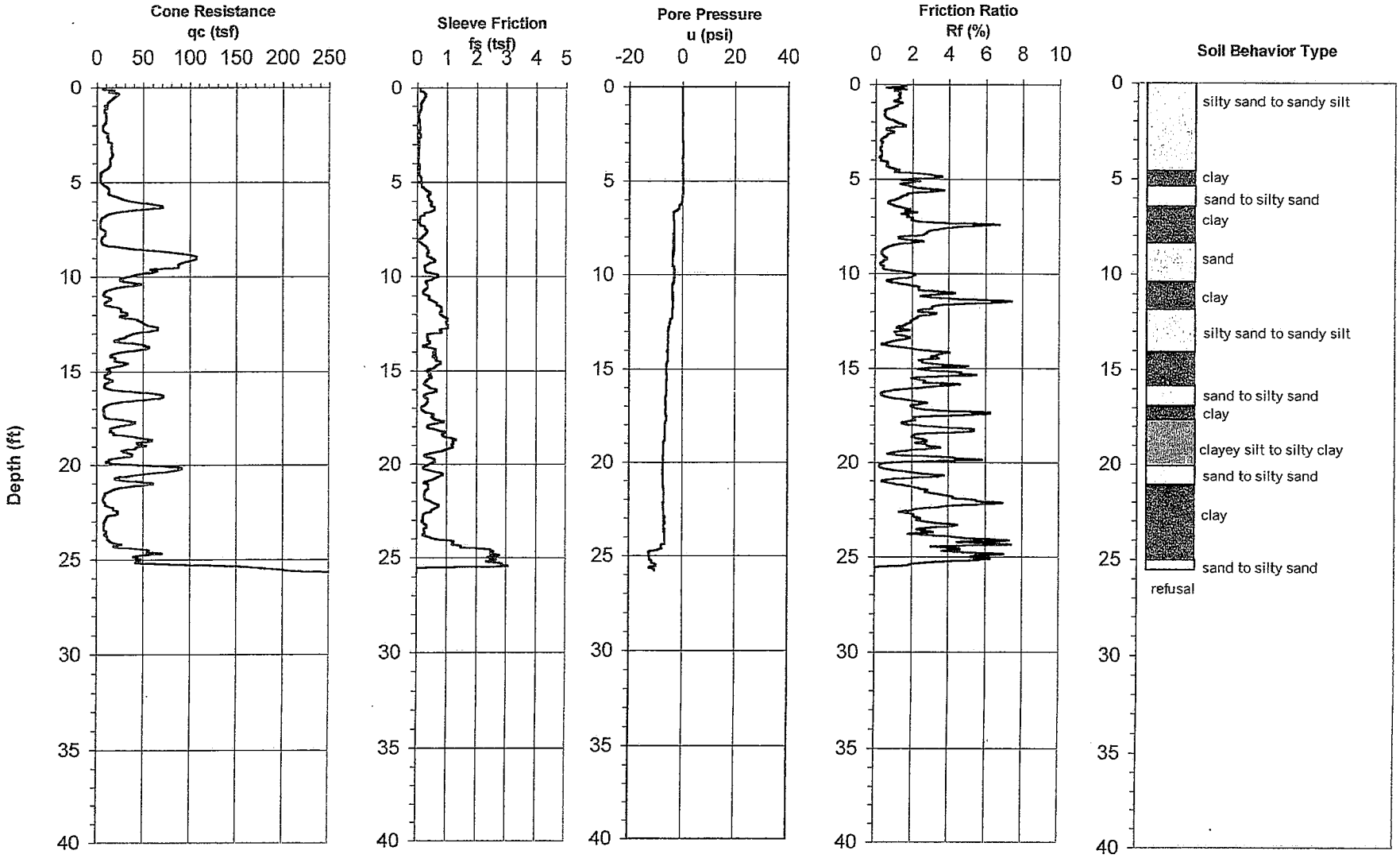
LOG OF CPT-4

Elevation: 10.1' MSL
Date: 4/12/00

Project No. 0298-019-00
Document No. 0-0477

FIGURE B-4



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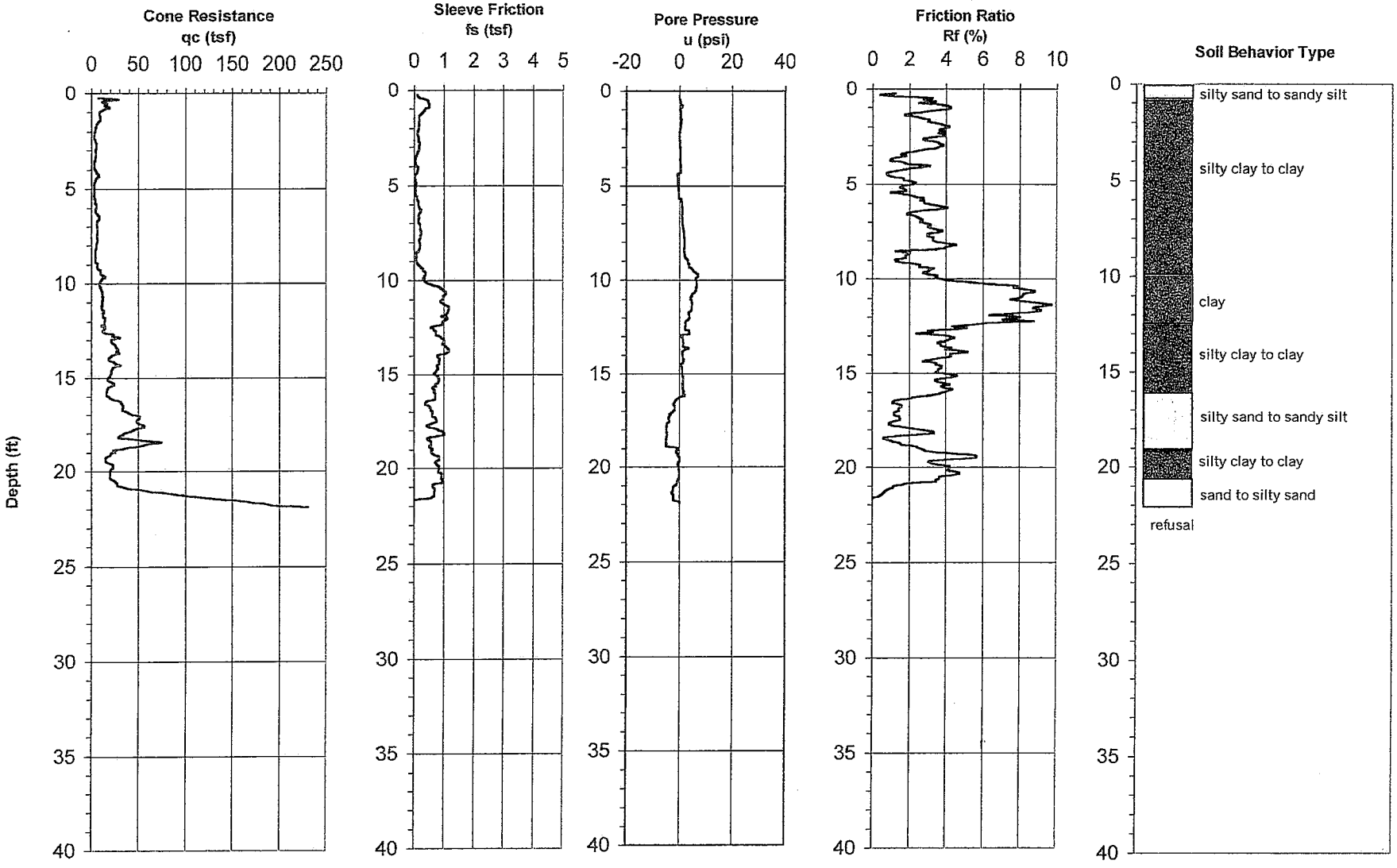
LOG OF CPT-5

Elevation: 8.1' MSL
Date: 4/12/00

Project No. 0298-019-00
Document No. 0-0477

FIGURE B-5



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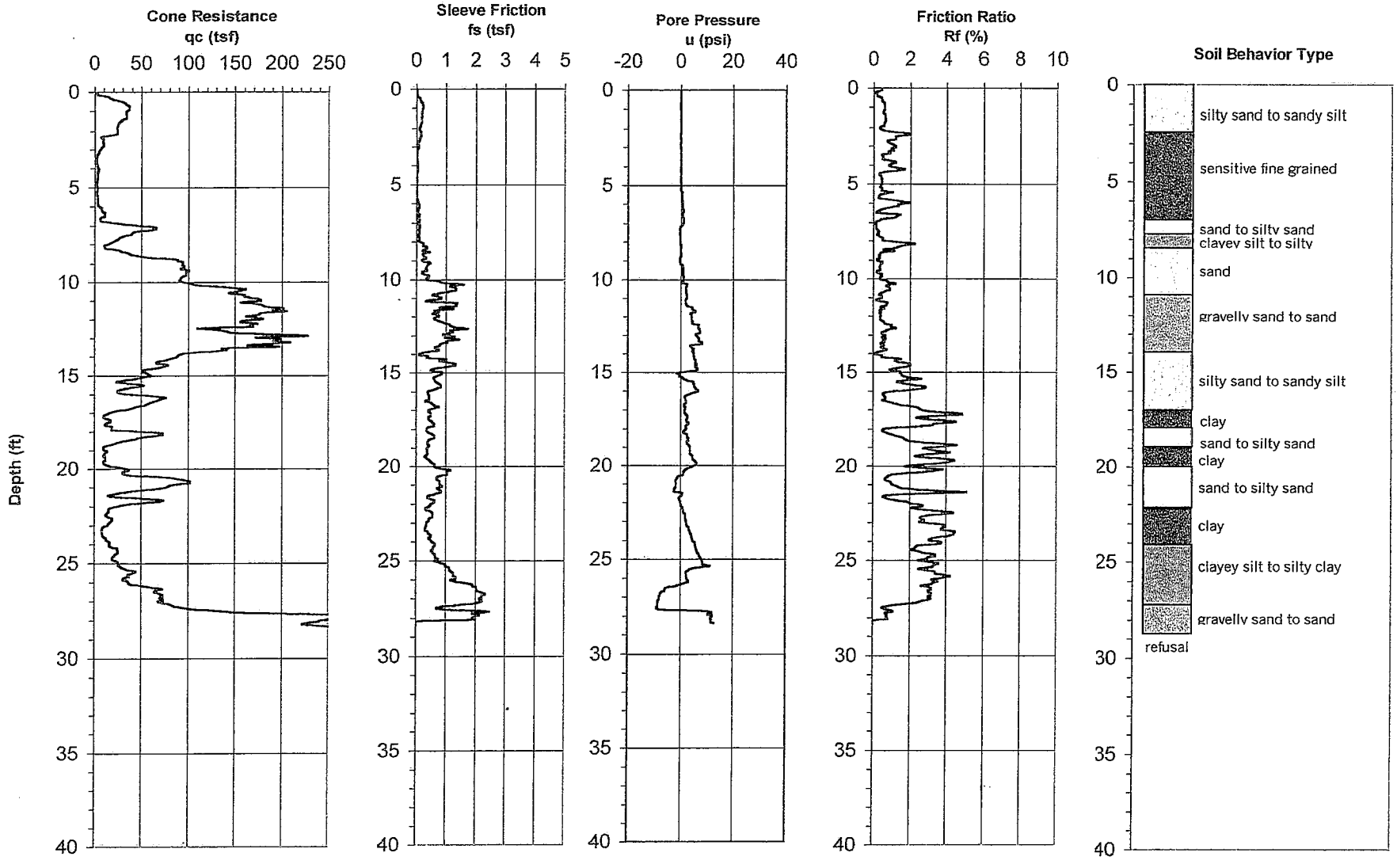
LOG OF CPT-6

Elevation: 9.8' MSL
Date: 4/12/00

Project No. 0298-019-00
Document No. 0-0477

FIGURE B-6



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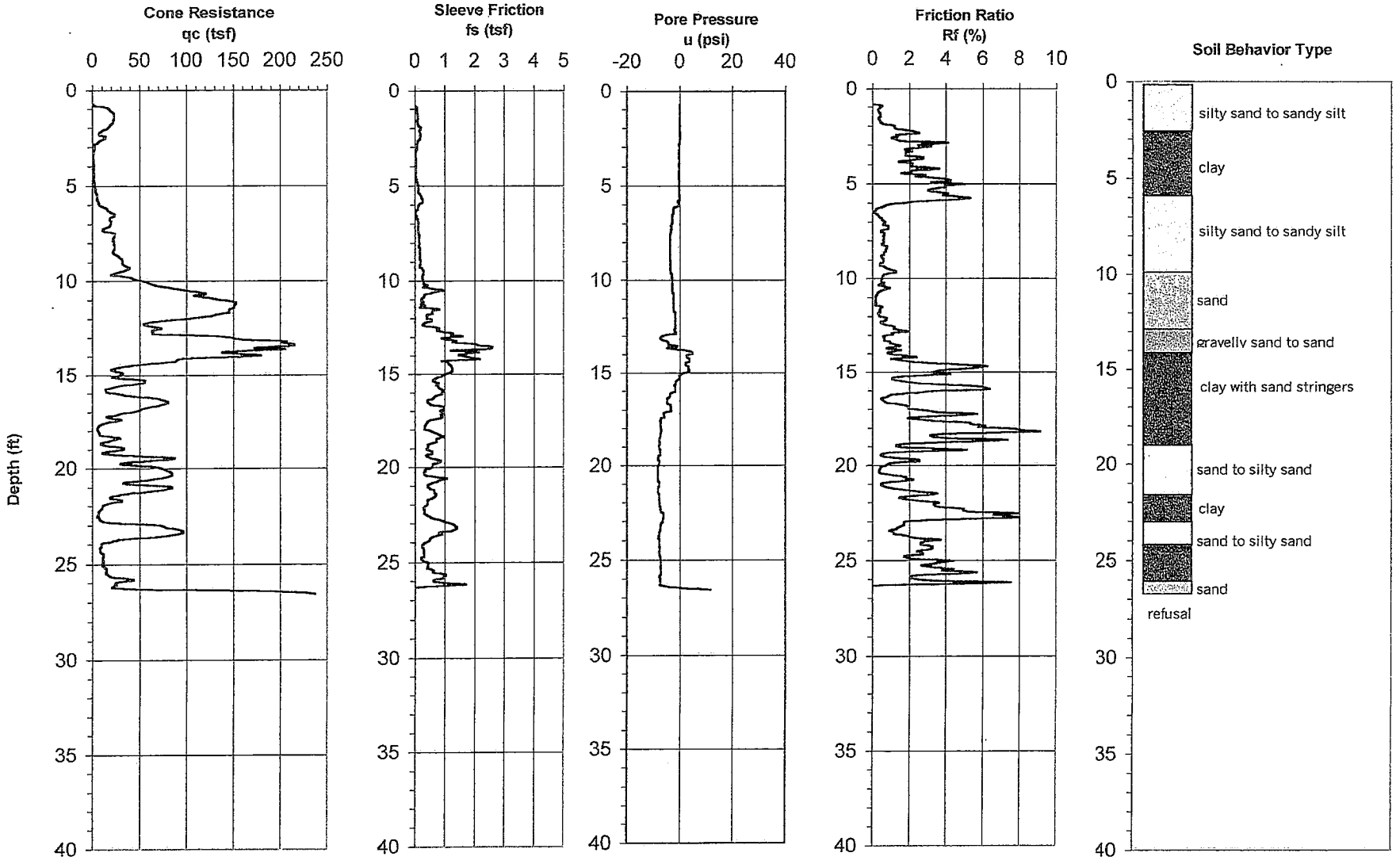
LOG OF CPT-7

Elevation: 10.0' MSL
Date: 4/12/00

Project No. 0298-019-00
Document No. 0-0477

FIGURE B-7



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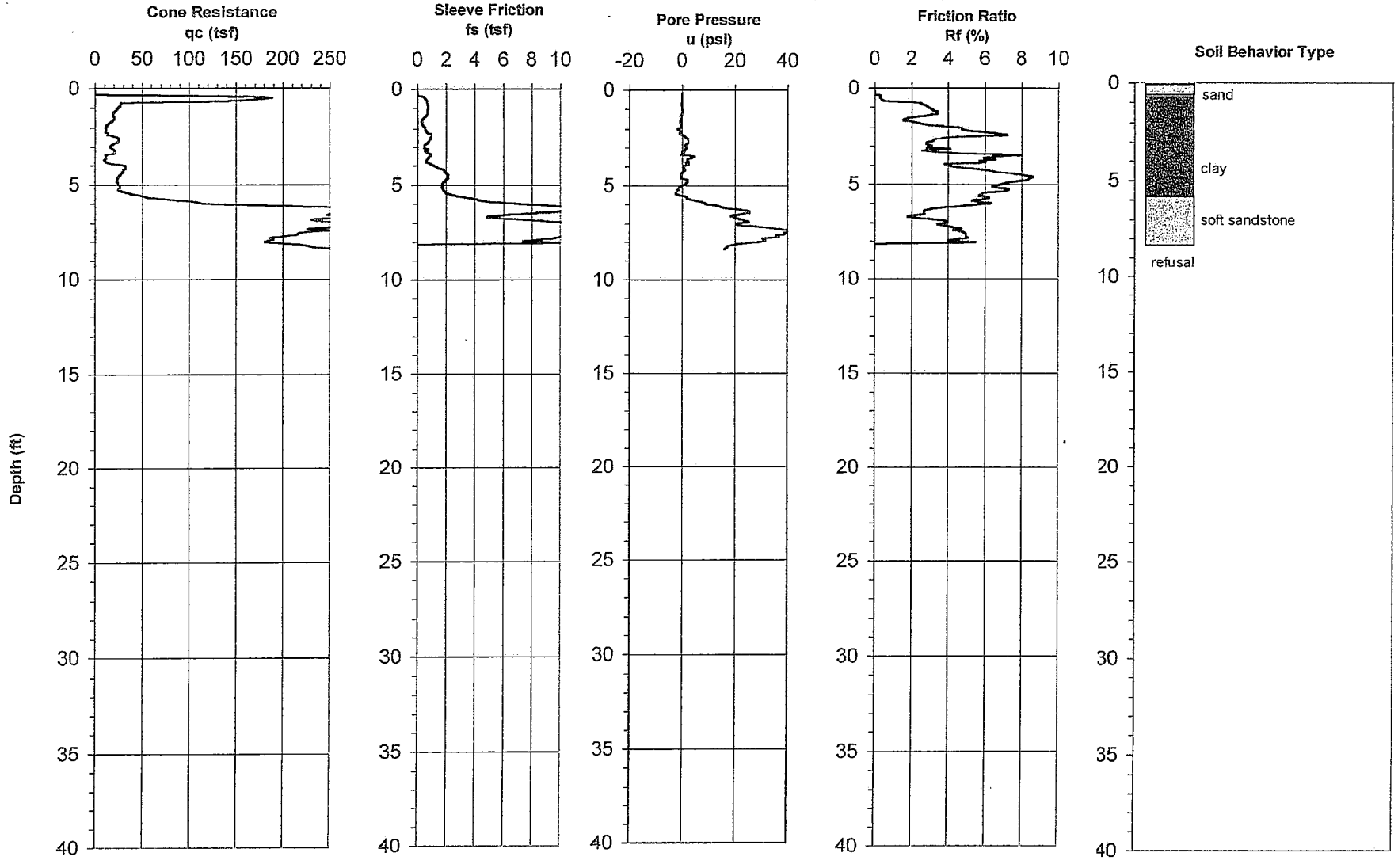
LOG OF CPT-8

Elevation: 15.7' MSL
Date: 4/12/00

Project No. 0298-019-00
Document No. 0-0477

FIGURE B-8



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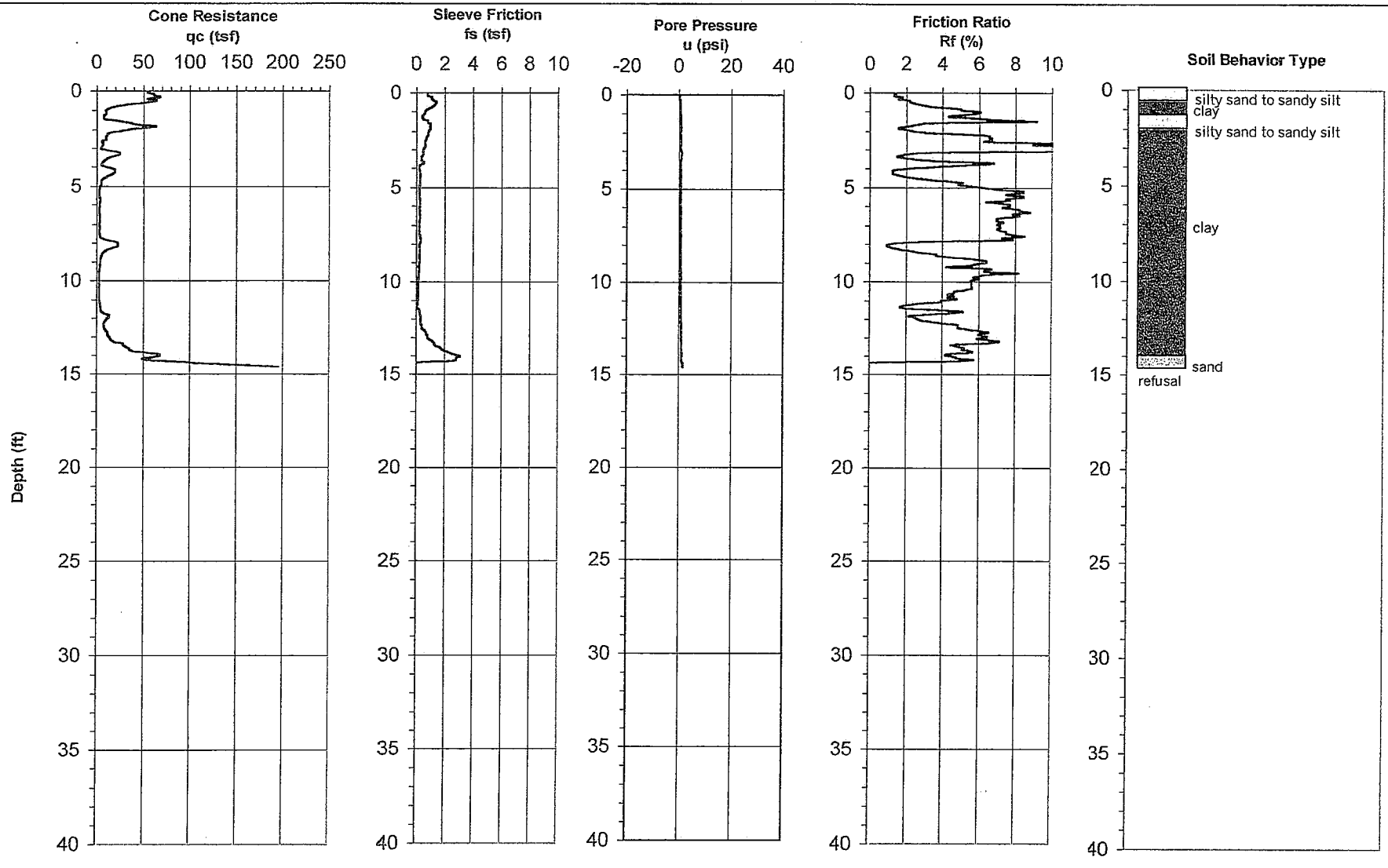
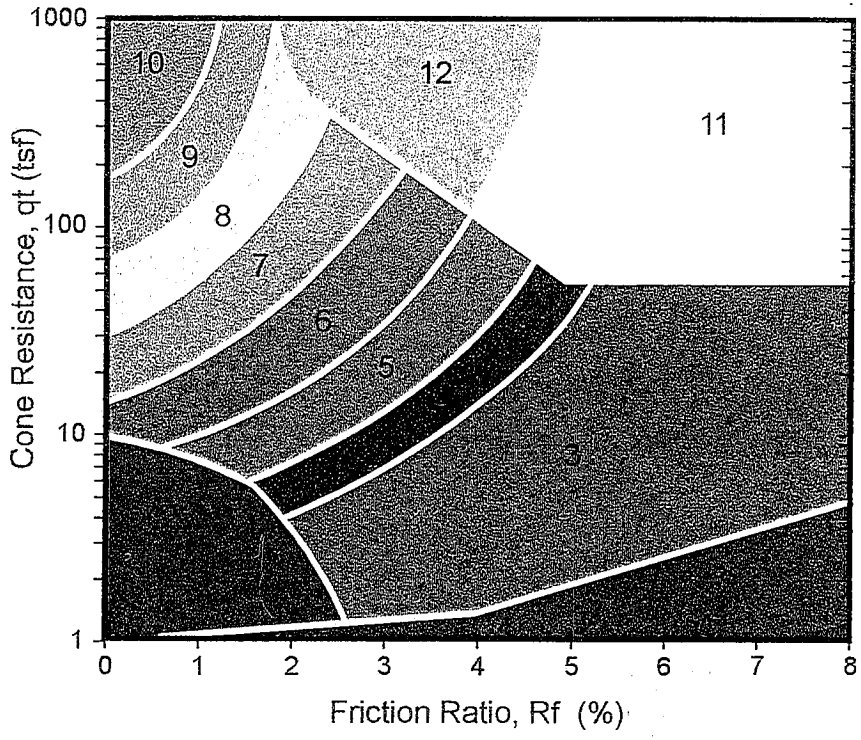
LOG OF CPT-9

Elevation: 14.0' MSL
Date: 4/12/00

Project No. 0298-019-00
Document No. 0-0477

FIGURE B-9





Zone	q_t / N	Soil Behavior Type	UCSCS
1	2	sensitive fine grained	OL-OH
2	1	organic material	Pt-OH
3	1	clay	CH
4	1.5	silty clay to clay	CL-CH
5	2	clayey silt to silty clay	ML-CL
6	2.5	sandy silt to clayey silt	MH-ML
7	3	silty sand to sandy silt	SM-ML
8	4	sand to silty sand	SP-SM
9	5	sand	SP
10	6	gravelly sand to sand	SW-SP
11	1	very stiff fine grained *	CL-MH
12	2	sand to clayey sand *	SP-SC

* overconsolidated or cemented

After Robertson and Campanella, 1988.

Geotechnics Incorporated

CPT CLASSIFICATION CHART

Project No. 0298-019-00
Document No. 0-0477

FIGURE B-10

LOG OF EXPLORATION BORING NO. 1

Logged by: JAA
Method of Drilling: 5"-Rotary Wash
Date Drilled: 4/12/00
Elevation: 11.3' MSL

DEPTH (FT)	BLOWS PER FT	DRIVE SAMPLE	BULK SAMPLE	DENSITY (PCF)	MOISTURE (%)	DESCRIPTION	LAB TESTS
1						ESTUARINE DEPOSITS: Sandy clay (CL), dark brown, moist to wet, fine grained sand, moderately plastic, soft.	
2							
3							
4							
5							
6	PUSH	SHE				No recovery 5 - 7 feet.	
7							
8							
9							
10							
11	PUSH	SHE				No recovery 10 - 12 feet.	
12							
13							
14							
15							
16	15	SPT				Stiff.	
17						Sand with gravels (SP-SM), dark gray, wet, fine grained, loose, nonplastic, subrounded gravels.	
18							
19							
20	8	CAL				No recovery 20 - 21 feet.	
21							
22						Clayey sand (SC), dark gray to dark brown, wet, fine grained, medium dense, low plasticity, subrounded gravels.	
23							
24							
25							
26	10	SPT					
27							
28							
29							
30							

PROJECT NO. 0298-019-00 GEOTECHNICS INCORPORATED FIGURE B-11

LOG OF EXPLORATION BORING NO. 1 (continued)

Logged by: JAA
Date Drilled: 4/12/00
Method of Drilling: 5"-Rotary Wash
Elevation: 11.3' MSL

DEPTH (FT)	BLOWS PER FT	DRIVE SAMPLE	BULK SAMPLE	DENSITY (PCF)	MOISTURE (%)	DESCRIPTION	LAB TESTS
31	102	CR		113	17.9	BAY POINT FORMATION (Qbp+Qn): Silty sandstone with gravel, dark gray to reddish brown, wet, fine grained, very dense, nonplastic, subrounded gravels.	
32						Caving at 32.5'.	
33							
34						Total depth: 32.5 feet.	
35						Groundwater encountered at 2 feet.	
36						Backfilled 4/13/00	
37							
38							
39							
40							
41							
42							
43							
44							
45							
46							
47							
48							
49							
50							
51							
52							
53							
54							
55							
56							
57							
58							
59							
60							

LOG OF EXPLORATION BORING NO. 2

Logged by: JAA
Date Drilled: 4/12/00
Method of Drilling: 5"-Rotary Wash
Elevation: 15.1' MSL

DEPTH (FT)	BLOWS PER FT	DRIVE SAMPLE	BULK SAMPLE	DENSITY (PCF)	MOISTURE (%)	DESCRIPTION	LAB TESTS
1						FILL: Sandy clay (CL), dark gray, moist to wet, fine grained sand, low to moderate plasticity, soft.	
2							
3							
4							
5							
6	2	SPT				ESTUARINE DEPOSITS: Clay (CL-CH), dark brown to dark grayish brown, moist, moderate to high plasticity, soft.	Soluble Sulfate
7							
8							
9							
10							
11	PUSH	SHE		102	54.9		Direct Shear, Consolidation
12							
13							
14							
15							
16	PUSH	SPT				Fat Clay (CH), medium to dark gray, wet, high plasticity, soft. (LL = 76, PI = 45.)	Atterberg Limits, Particle Size Analysis
17							
18							
19							
20	PUSH	SHE				Clay (CL-CH), reddish brown, moist, moderate plasticity, soft.	Soluble Sulfate Direct Shear, Consolidation
21							
22							
23							
24							
25	63	SPT				BAY POINT FORMATION (Qbp+Qn): Silty sandstone, reddish brown, moist to wet, fine grained, very dense, nonplastic.	
26							
27						Total depth: 26.5 feet Groundwater encountered at 5 feet Backfilled 4/13/00	
28							
29							
30							

PROJECT NO. 0298-019-00 **GEOTECHNICS INCORPORATED** FIGURE B-12

LOG OF EXPLORATION BORING NO. 3

Logged by: JAA
Date Drilled: 4/12/00
Method of Drilling: 5"-Rotary Wash
Elevation: 16.0' MSL

DEPTH (FT)	BLOWS PER FT	DRIVE SAMPLE	BULK SAMPLE	DENSITY (PCF)	MOISTURE (%)	DESCRIPTION	LAB TESTS
1						<u>ESTUARINE DEPOSITS:</u> Sandy clay (CL), dark brown, moist, fine grained sand, low plasticity, very stiff, few roots.	
2	25	SPT					
3							
4							
5	4	CAL		125	26.5	Very soft.	Direct Shear, Consolidation
6							
7							
8							
9							
10						Silty sand (SM), dark gray, wet, fine grained, very loose, non-plastic.	Atterberg Limits, Particle Size Analysis
11	4	SPT					
12							
13							
14							
15							
16	PUSH	SHE				No recovery 15 - 17 feet.	
17							
18						Approximate 6 - inch thick layer of subrounded gravels.	
19							
20						<u>BAY POINT FORMATION (Qbp+Qn)</u>: Clayey sandstone with gravels, gray to reddish brown, moist to wet, fine grained, very dense, low plasticity, subrounded gravels.	
21	85	SPT					
22						Total depth: 21.5 feet Groundwater encountered at 7 feet Backfilled 4/12/00	
23							
24							
25							
26							
27							
28							
29							
30							

PROJECT NO. 0298-019-00 **GEOTECHNICS INCORPORATED** FIGURE B-13

LOG OF EXPLORATION BORING NO. 4

Logged by: JAA
Method of Drilling: 5"-Rotary Wash
Date Drilled: 4/12/00
Elevation: 16.4' MSL

DEPTH (FT)	BLOWS PER FT	DRIVE SAMPLE	BULK SAMPLE	DENSITY (PCF)	MOISTURE (%)	DESCRIPTION	LAB TESTS
1						BAY POINT FORMATION (Qbp+Qn): Clayey sandstone, reddish brown, moist, fine grained, very dense, low plasticity.	
2							
3							
4							
5							
6	68	SPT					
7							
8							
9							
10	38	CAL				Medium dense, subrounded gravels.	
11							
12						Total depth: 11 feet Groundwater not encountered Backfilled 4/12/00	
13							
14							
15							
16							
17							
18							
19							
20							
21							
22							
23							
24							
25							
26							
27							
28							
29							
30							

PROJECT NO. 0298-019-00 GEOTECHNICS INCORPORATED FIGURE B-14

APPENDIX C

LABORATORY TESTING

Laboratory testing was conducted in a manner consistent with that level of care and skill ordinarily exercised by members of the profession currently practicing under similar conditions and in the same locality. No other warranty, expressed or implied, is made as to the correctness or serviceability of the test results or the conclusions derived from these tests. Where a specific laboratory test method has been referenced, such as ASTM, Caltrans, or AASHTO, the reference applies only to the specified laboratory test method and not to associated referenced test method(s) or practices, and the test method referenced has been used only as a guidance document for the general performance of the test and not as a "Test Standard." A brief description of the tests performed follows:

Classification: Soils were classified visually according to the Unified Soil Classification System. The soil classifications are shown on the boring logs of Appendix B. Classification was supplemented by laboratory testing in accordance with ASTM D422 and ASTM D4318. Classification of soils from the CPT's were derived using the CPT Classification Chart presented in Figure B-10.

In-Situ Moisture/Density: The in-place moisture content and dry unit weight of selected soil samples were determined using relatively undisturbed samples from the shelby tubes and liner rings of a CAL sampler. The dry unit weight and moisture content are shown on the boring logs in Appendix B.

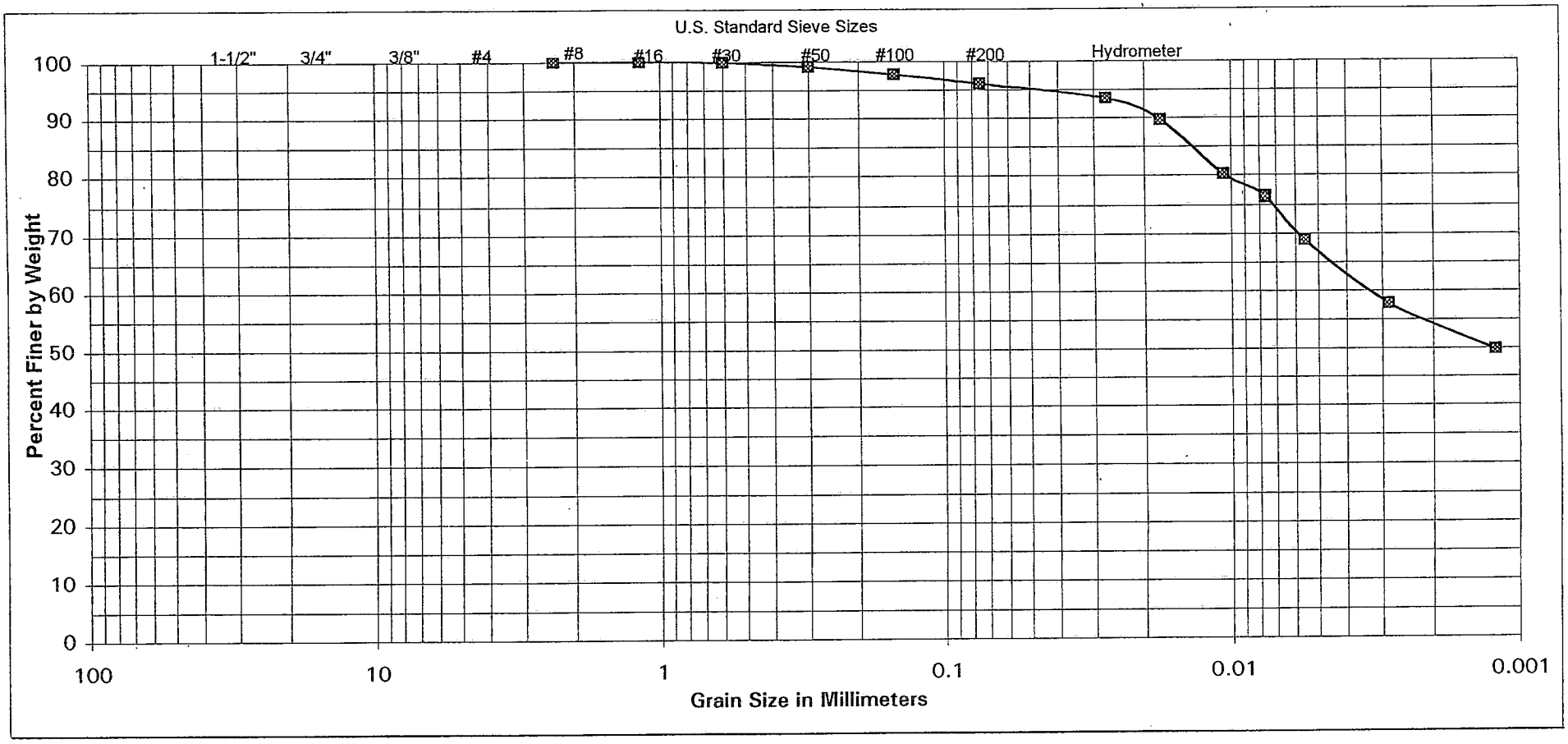
Particle Size Analysis: Particle size analysis was performed in general accordance with the laboratory procedures outlined in ASTM D422. The grain size distribution was used to estimate presumptive soil strength parameters and foundation design criteria. The results are given in Figures C-1.1 and C-1.2.

Atterberg Limits: The Atterberg limits were determined by using ASTM D4318. The results are summarized in Figures C-1.1 and C-1.2.

Sulfate Content: To assess the potential for reactivity with below grade concrete, selected soil samples were tested for water-soluble sulfate content. The water soluble sulfate was extracted under vacuum from the soil using 100:1 and 200:1 (water to dry soil) dilution ratios. The extracted solution was then tested for water soluble sulfate in general accordance with ASTM D516. The results are presented on Figure C-2.

Direct Shear: The shear strengths of selected samples were determined by performing direct shear tests in general accordance with ASTM D3080. Relatively undisturbed Shelby tube samples and samples from the liner rings of a 2 $^{3}/_{8}$-inch ID Modified California sampler were tested. The results of these laboratory tests are summarized in Figures C-3.1 through C-3.3.

Consolidation: Selected relatively undisturbed Shelby tube samples were tested for one-dimensional consolidation in general accordance with ASTM D2435. Consolidation test results are summarized in Figures C-4.1 through C-4.3.



COARSE	FINE	COARSE	MEDIUM	FINE	SILT AND CLAY
GRAVEL		SAND			

SAMPLE	
EXPLORATION NUMBER:	B2
SAMPLE LOCATION:	15' - 16.5'

UNIFIED SOIL CLASSIFICATION:	CH
DESCRIPTION:	FAT CLAY

ATTERBERG LIMITS
LIQUID LIMIT: 76
PLASTIC LIMIT: 31
PLASTICITY INDEX: 45

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SOIL CLASSIFICATION

Project No. 0298-019-00
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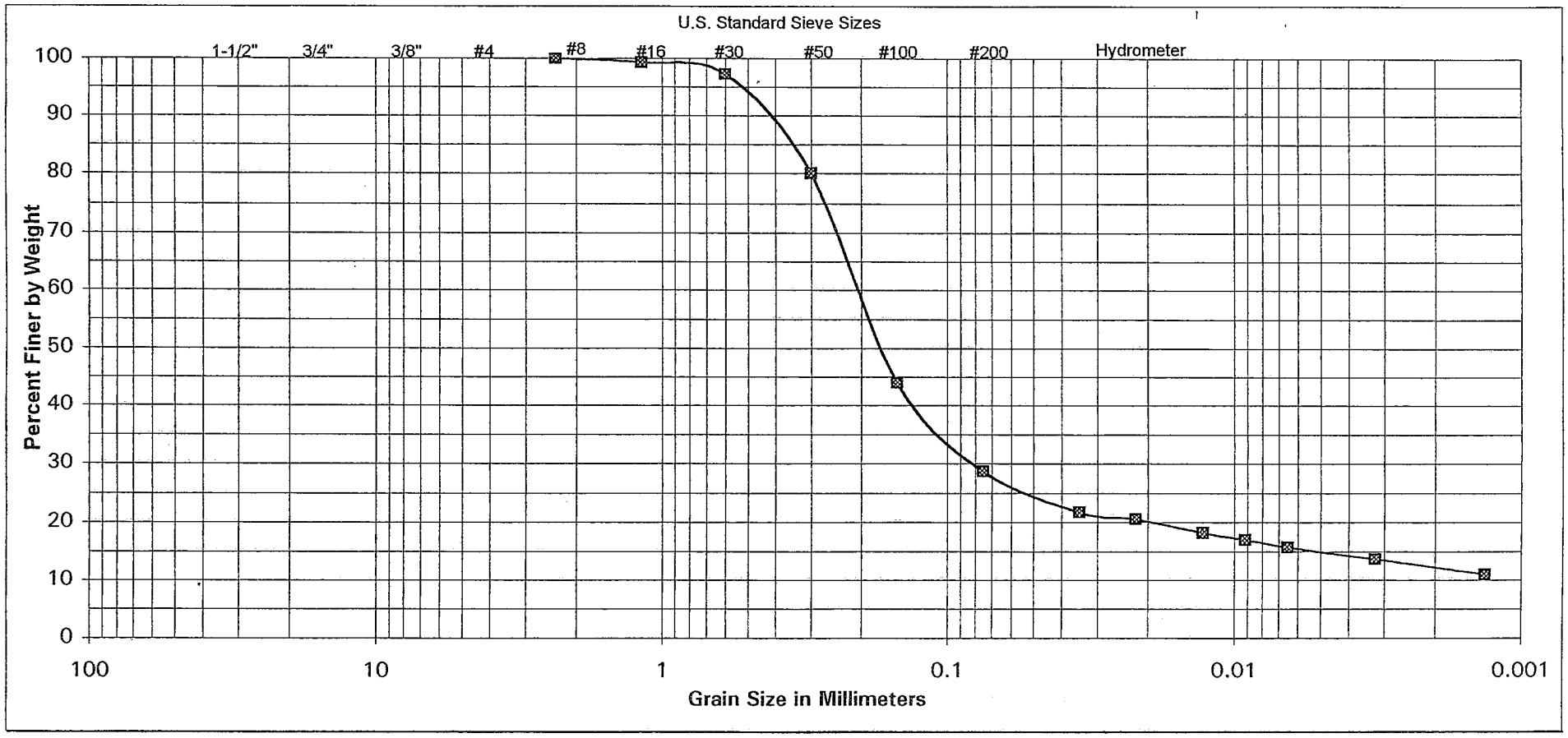
FIGURE C-1.1



COARSE	FINE	COARSE	MEDIUM	FINE	SILT AND
GRAVEL		SAND			CLAY

SAMPLE	
EXPLORATION NUMBER:	B3
SAMPLE LOCATION:	10' - 11.5'

UNIFIED SOIL CLASSIFICATION:	SM
DESCRIPTION:	SILTY SAND

ATTERBERG LIMITS
LIQUID LIMIT: NP
PLASTIC LIMIT: NP
PLASTICITY INDEX: NP

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SOIL CLASSIFICATION

Project No. 0298-019-00
Document No. 0-0477
FIGURE C-1.2

SULFATE, pH, and RESISTIVITY

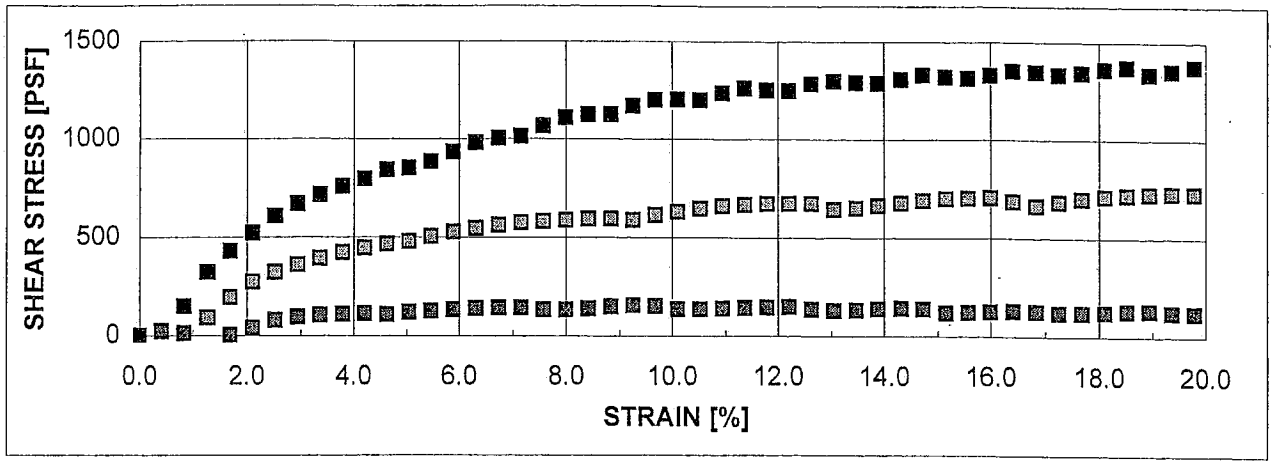
SAMPLE	SULFATE CONTENT (% of Dry Soil Weight) (SMEWW4500S0$_4$E)	pH (CALTRANS 643)	RESISTIVITY (ohm-cm) (CALTRANS 643)	
			As-Received	Saturated
B2 @ 5' - 6½'	0.46	NT	NT	
B2 @ 20' - 22'	0.11	NT	NT	

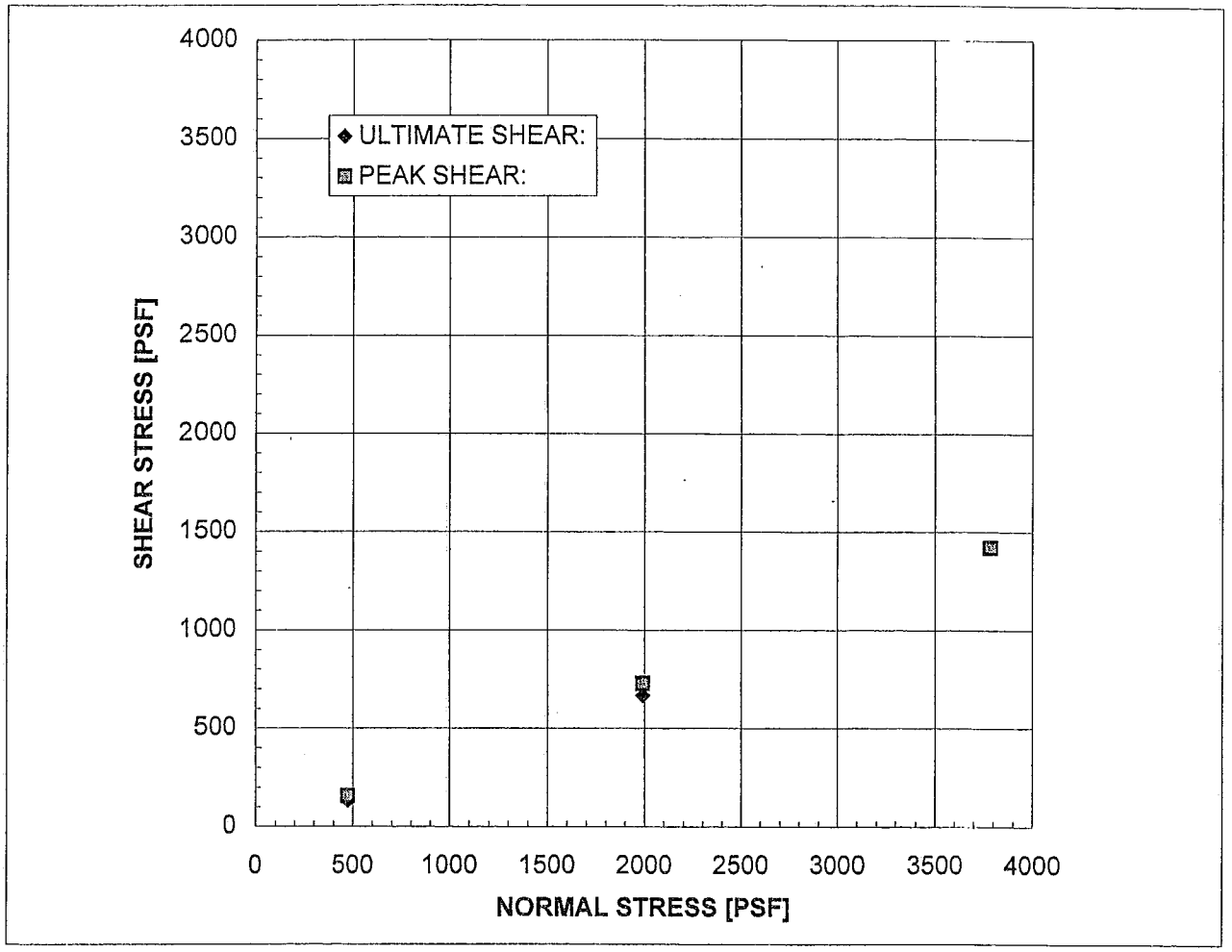
Soil Resistivity in ohm-cm	General Degree of Corrosivity to Ferrous Metal
0 to 1,000	Very Corrosive
1,000 to 2,000	Corrosive
2,000 to 5,000	Moderately Corrosive
5,000 to 10,000	Mildly Corrosive
Greater than 10,000	Slightly Corrosive

Water Soluble Chloride (Cl) Content in % of Dry Soil Weight	General Degree of Corrosivity to Metal
over 0.15%	Severely Corrosive
0.15 % to 0.03%	Corrosive
0.03 % to 0.00%	Negligible

Water Soluble Sulfate (SO_4) Content in % of Dry Soil Weight	General Degree of Reactivity with Concrete
over 2.00 %	Very Severely Reactive
2.00 % to 0.2 %	Severely Reactive
0.20 % to 0.10 %	Moderately Reactive
0.10 % to 0.00%	Negligible

Reference Table 19-A-4 Uniform Building Code





SAMPLE: B2 @ 10' - 12'

ESTUARINE DEPOSITS: Dark brown fat clay (CH)

	PEAK	ULTIMATE
ϕ'	19 °	18 °
C'	0 PSF	0 PSF

STRAIN RATE: 0.0020 IN/MIN
(Sample was consolidated and drained)

	IN-SITU	AS-TESTED
γ_d	66.1 PCF	66.1 PCF
w_c	54.9 %	61.6 %

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DIRECT SHEAR TEST RESULTS

Project No. 0298-019-00
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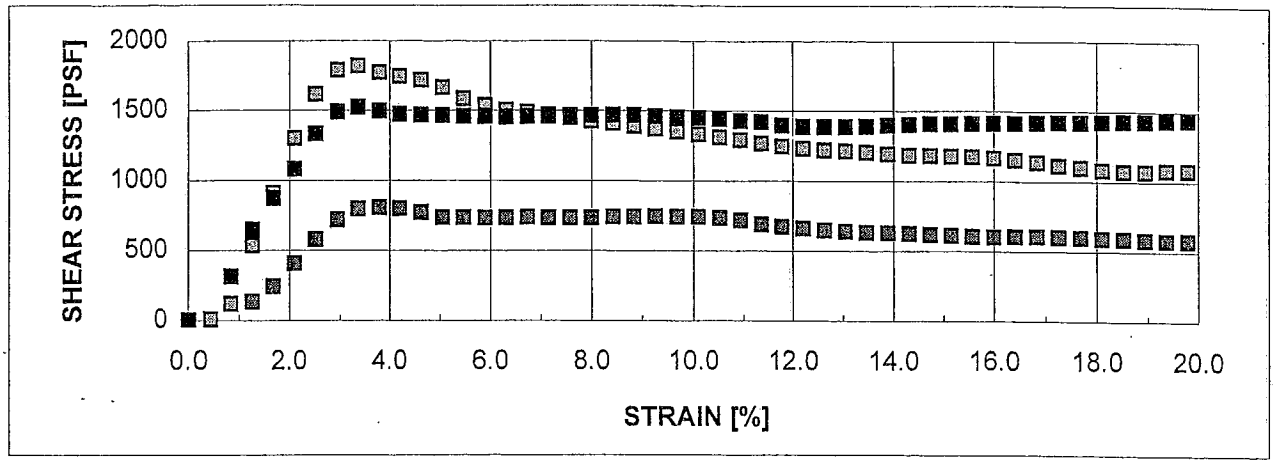
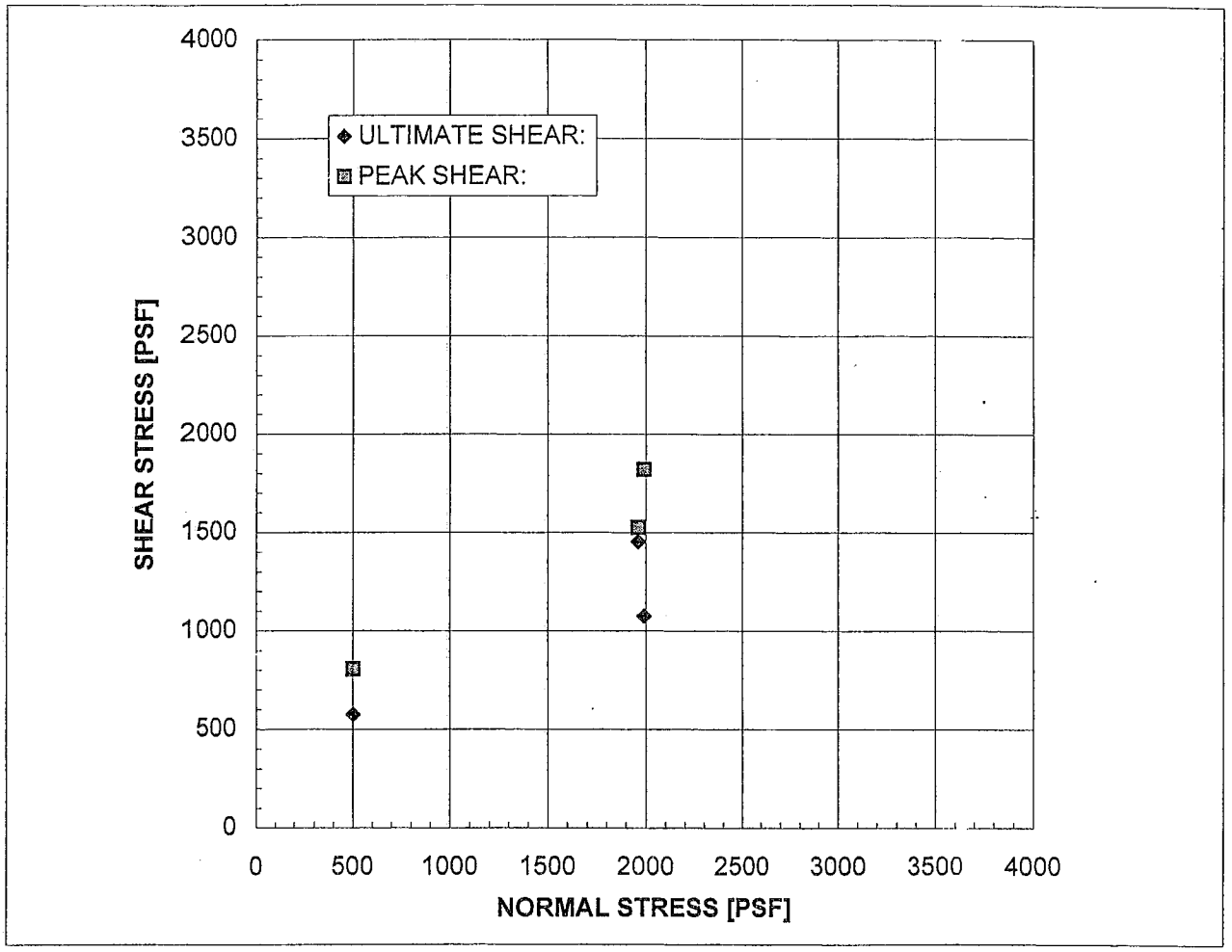
FIGURE C-3.1





SAMPLE: B2 @ 20-1/2' - 21'

ESTUARINE DEPOSITS: Reddish brown clay (CL)

	PEAK	ULTIMATE
ϕ'	27 °	18 °
C'	500 PSF	400 PSF

STRAIN RATE: 0.0100 IN/MIN
(Sample was consolidated and drained)

	IN-SITU	AS-TESTED
γ_d	119.0 PCF	119.0 PCF
w_c	14.5 %	21.1 %

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DIRECT SHEAR TEST RESULTS

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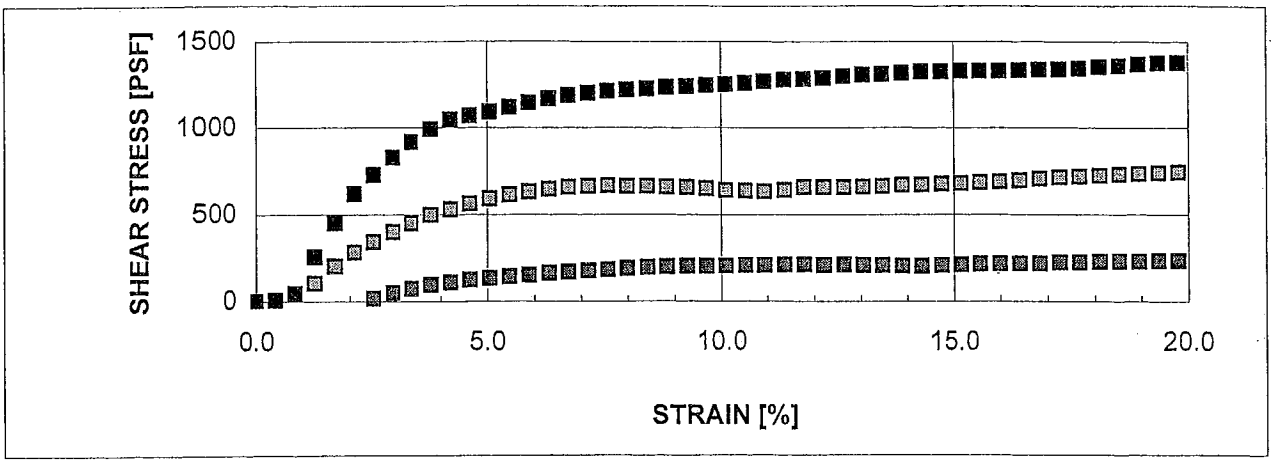
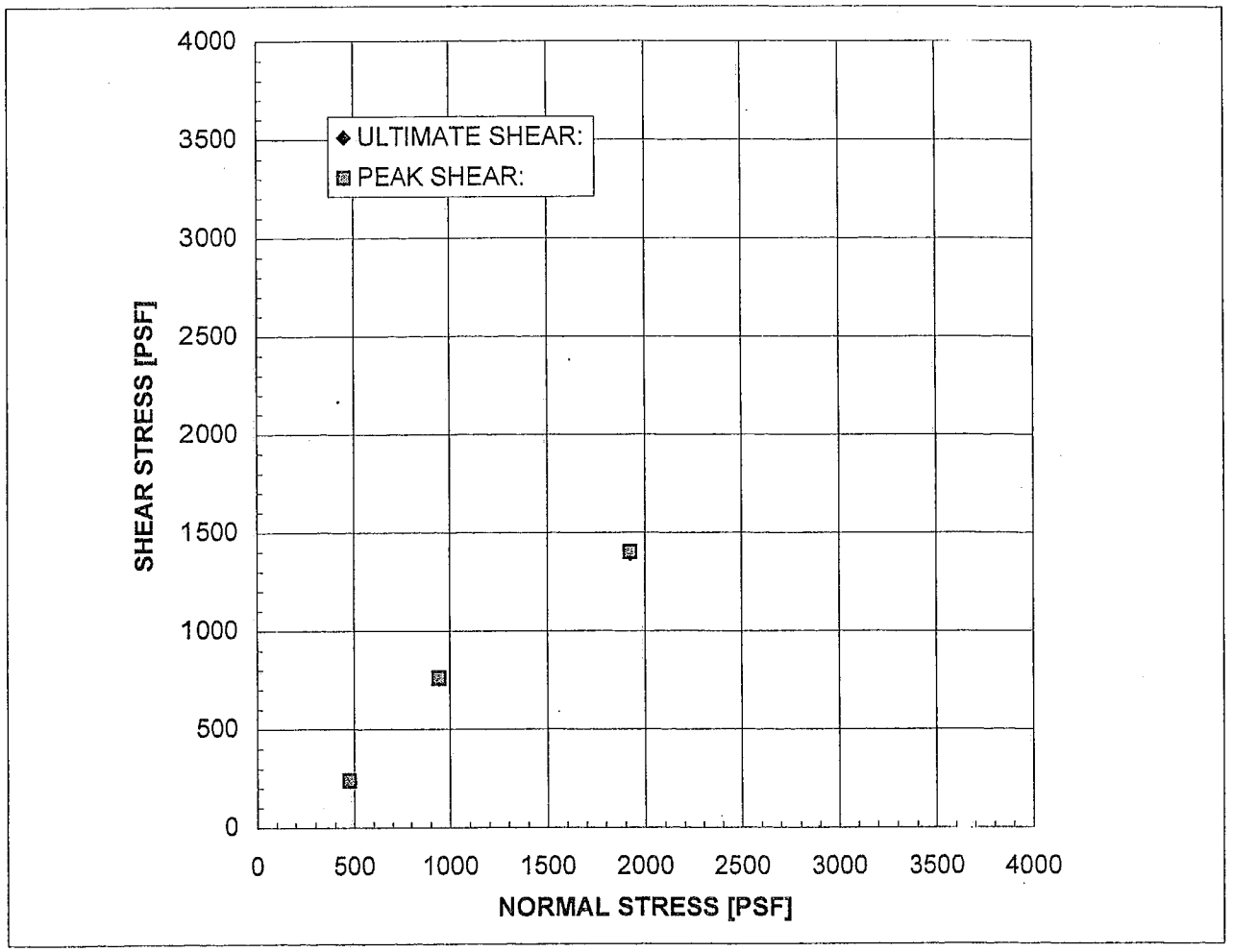
FIGURE C-3.2





SAMPLE: B3 @ 5-1/2' - 6'

ESTUARINE DEPOSITS: Dark greyish brown clay (CL)

	PEAK	ULTIMATE
ϕ'	32 °	31 °
C'	200 PSF	100 PSF

STRAIN RATE: 0.0200 IN/MIN
(Sample was consolidated and drained)

	IN-SITU	AS-TESTED
γ_d	52.6 PCF	52.6 PCF
w_c	24.8 %	28.0 %

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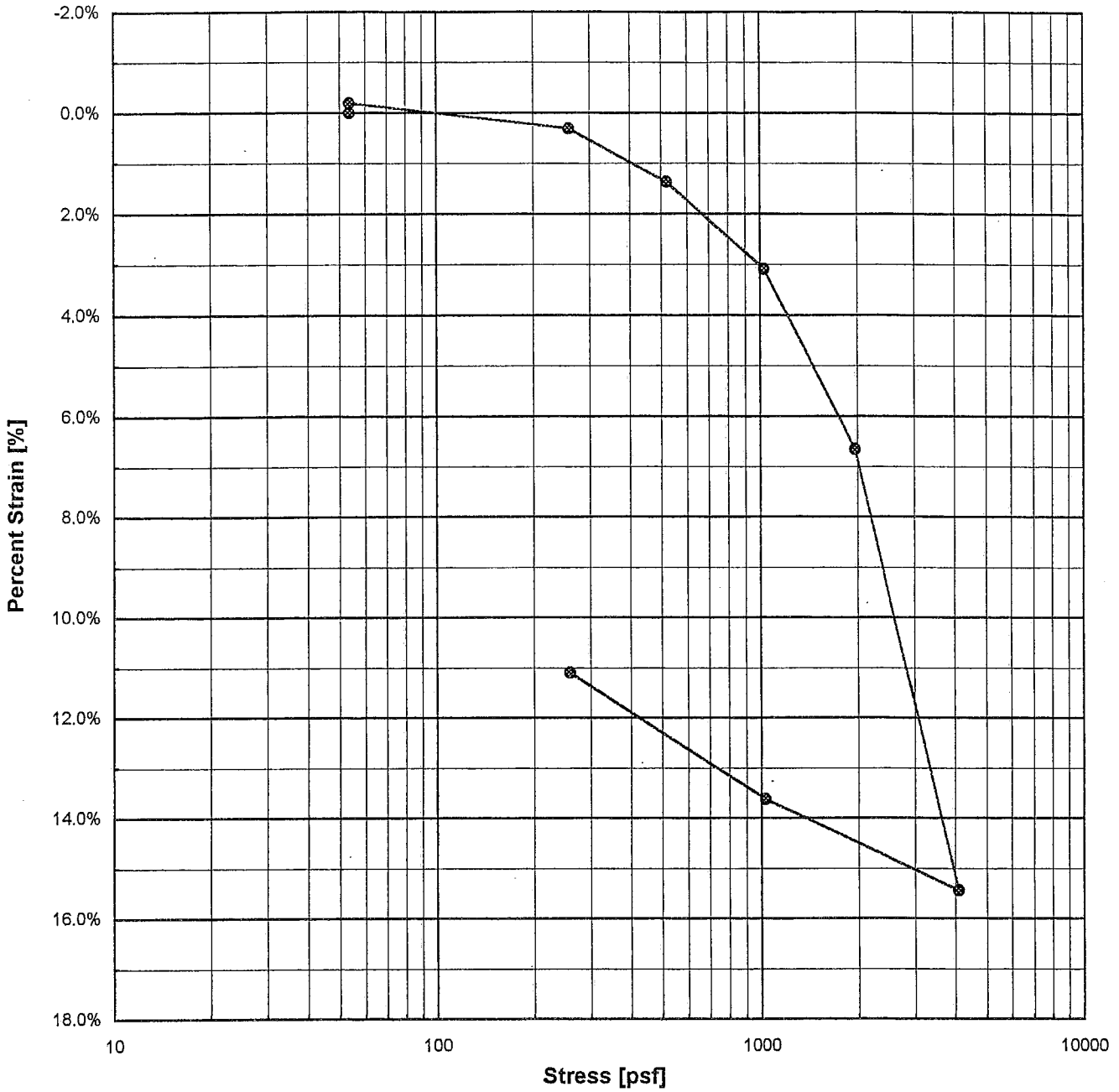
DIRECT SHEAR TEST RESULTS

Project No. 0298-019-00
Document No. 0-0477
FIGURE C-3.3



B2 @ 10' - 12' Water added at 54 psf.

INITIAL	FINAL	
0.9990	0.8868	SAMPLE HEIGHT [IN]
56.7	63.9	DRY DENSITY [PCF]
2.90	2.90	SPECIFIC GRAVITY
2.19	1.83	VOID RATIO
75.5	58.6	WATER CONTENT [%]
99.8	92.6	DEGREE OF SATURATION [%]

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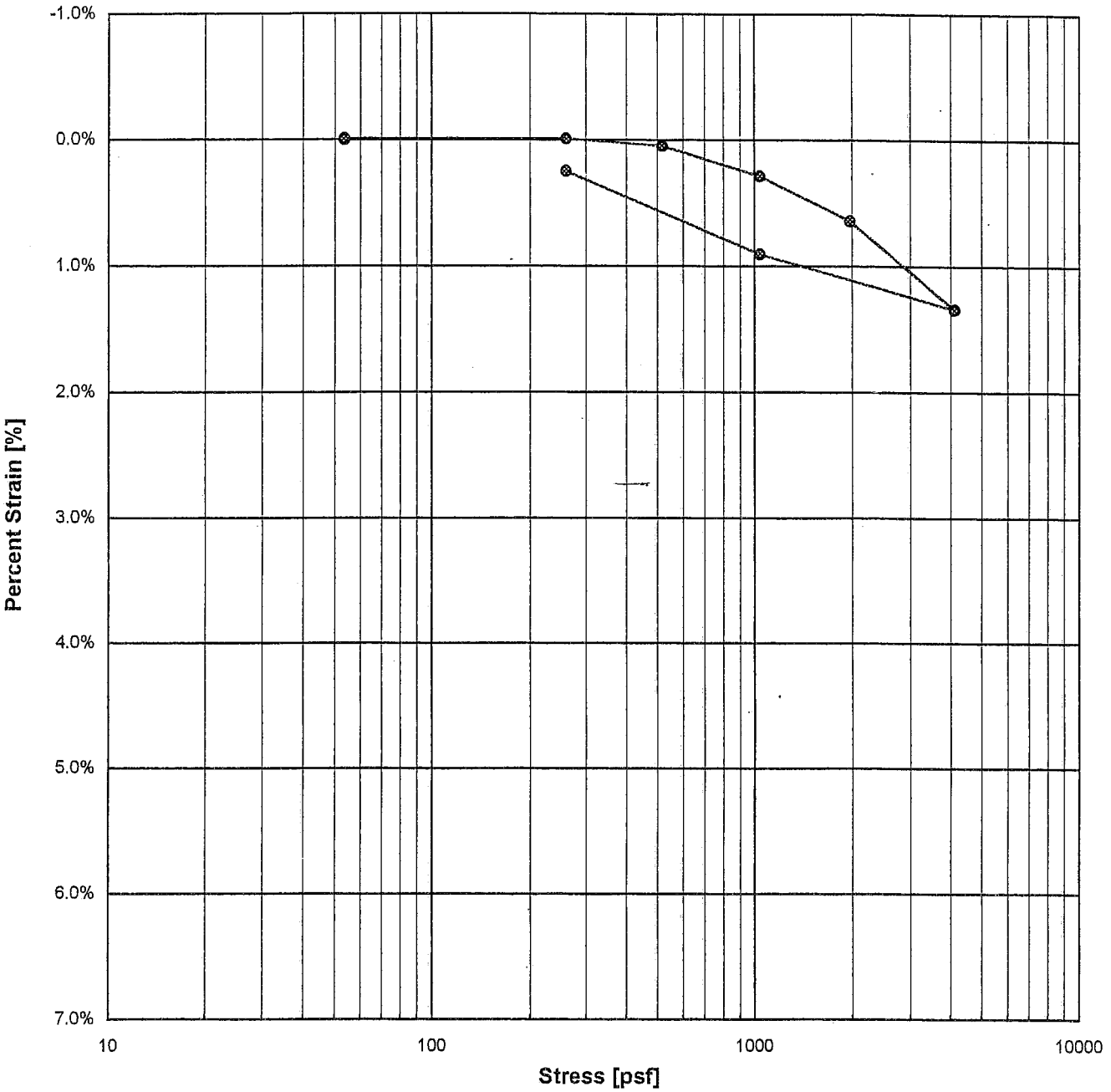
CONSOLIDATION TEST RESULTS

Project No. 0298-019-00
Document No. 0-0477
FIGURE C-4.1



B2 @ 20' - 22' Water added at 54 psf.

INITIAL	FINAL	
0.9990	0.9962	SAMPLE HEIGHT [IN]
118.1	118.4	DRY DENSITY [PCF]
2.80	2.80	SPECIFIC GRAVITY
0.48	0.48	VOID RATIO
15.7	15.9	WATER CONTENT [%]
91.7	93.2	DEGREE OF SATURATION [%]

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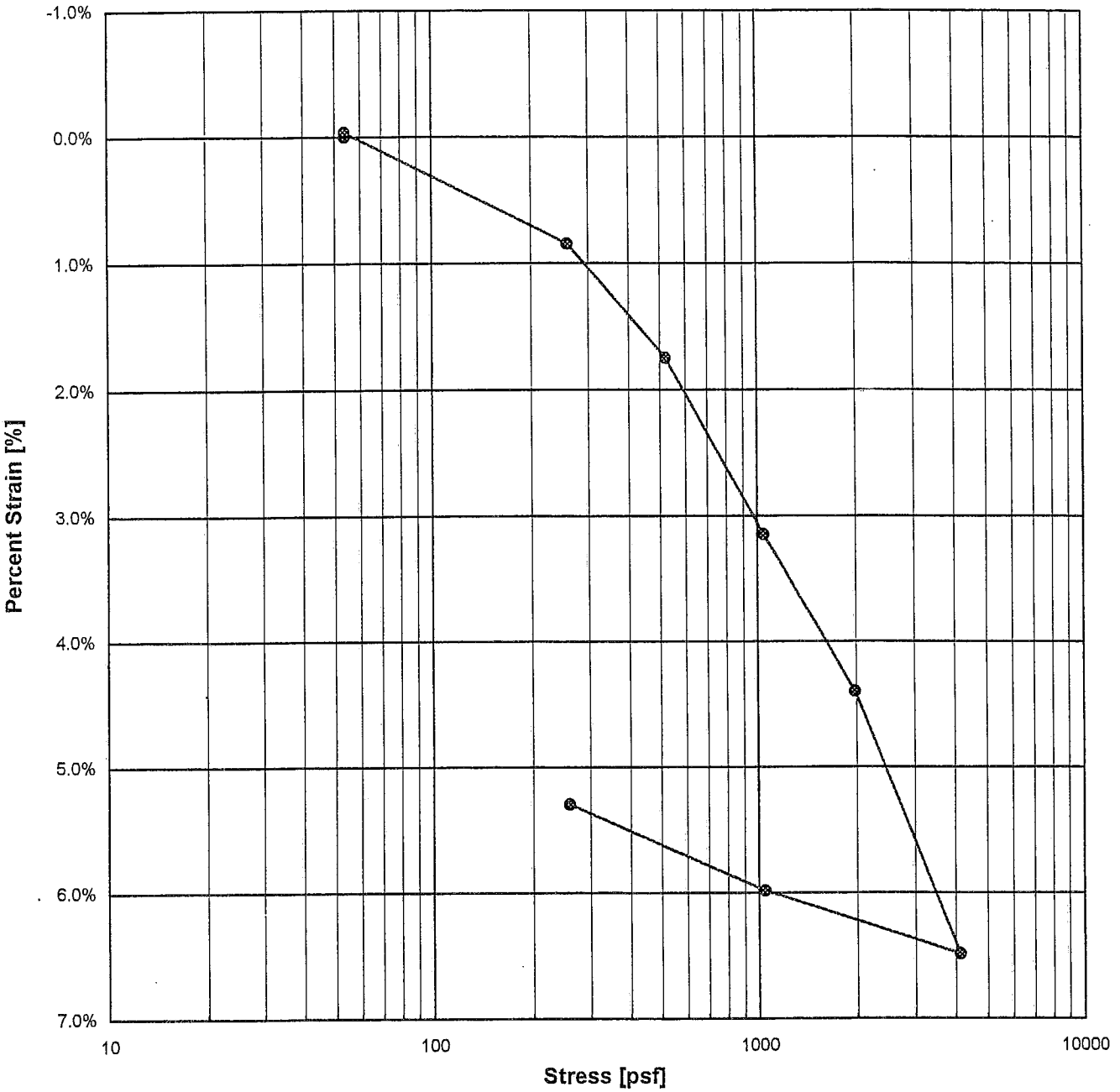
CONSOLIDATION TEST RESULTS

Project No. 0298-019-00
Document No. 0-0477
FIGURE C-4.2



B3 @ 5-1/2' - 6' Water added at 54 psf.

INITIAL	FINAL	
0.9990	0.9458	SAMPLE HEIGHT [IN]
99.2	104.7	DRY DENSITY [PCF]
2.85	2.85	SPECIFIC GRAVITY
0.79	0.70	VOID RATIO
26.5	23.0	WATER CONTENT [%]
95.0	94.0	DEGREE OF SATURATION [%]

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CONSOLIDATION TEST RESULTS

Project No. 0298-019-00
Document No. 0-0477
FIGURE C-4.3