SENSING SYSTEMS FOR MEASURING MECHANICAL PROPERTIES IN GROUND MASSES VOL. 4

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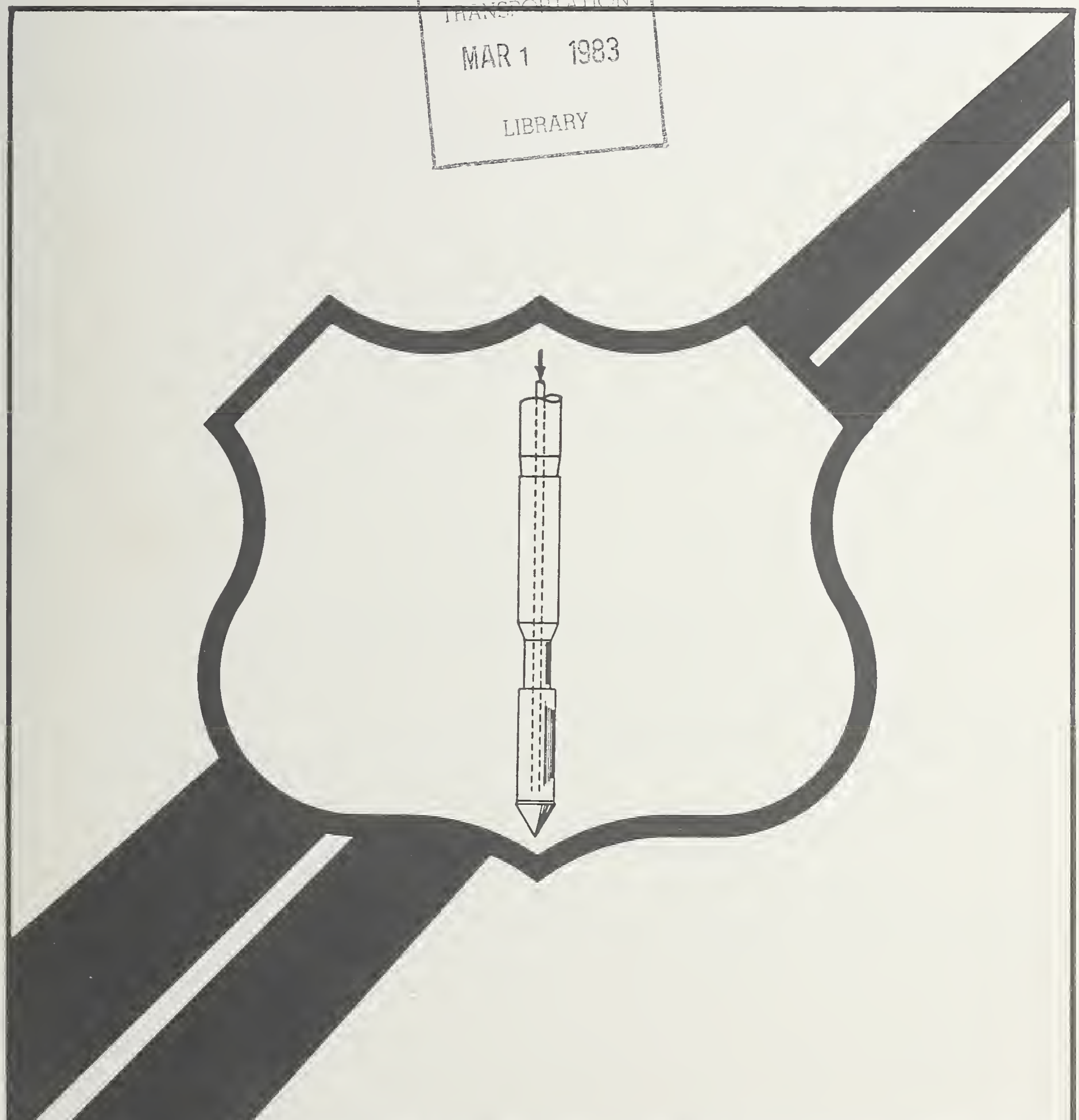

FOREWORD

This report examines the applicability and usefulness of the Begemann static cone penetrometer in site exploration. This is one of five reports on in situ testing devices, prepared by leading geotechnical firms for the Federal Highway Administration (FHWA). Others in this five-volume series are listed on the opposite page.

This evaluation of the Begemann static cone penetrometer is based upon the experience of CH2M-Hill, Inc. The cone has proved to be an accurate, cost effective testing device, especially when used in conjunction with other tests.

This report should serve the needs of geotechnical and structural engineers planning or designing underground structures.

Copies of the report are being distributed by FHWA transmittal memorandum. Additional copies may be obtained from the National Technical Information Service, 5285 Port Royal Road, Springfield, Virginia 22161.

Richard E. Hay

Richard E. Hay, Director
Office of Engineering
and Highway Operations
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16. Abstract

This report describes the performing organization's use of the Begemann type static cone penetrometer as a subsurface soil investigative tool. Applications, methods of interpretation, accuracy and limitations are discussed and five case histories are presented. The case histories are from Oregon and Washington.

The case histories examined make use of the static cone penetrometer data to solve the following types of problems:

Subsurface soil profile identification
Shallow footing design (settlement)
Deep foundation design (capacity)
Construction control

This is the fourth in a series of five reports. The other four volumes are: Volume 1, FHWA/RD-81/109, "Bore Hole Shear, Earth Settlement and Earth Penetrometer Probes;" Volume 2, FHWA/RD-81/110 "Pressuremeter;" Volume 3, FHWA/RD-81/111 "Vane Shear and Cone Piezometer;" and Volume 5, FHWA/RD-81/113 "Dutch Cone Penetrometer Tests-Case Histories".

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CONVERSION OF ENGLISH UNITS TO METRIC

English		Metric
Length		
1 in.	=	25.4 mm
1 ft.	=	0.305 m
Force		
1 lb.	=	4.45 N
1 ton	=	8.9 kN
Pressure		
1 psf	=	47.9 Pa
1 ksf	=	47.9 kPa
1 tsf	=	95.8 kPa
Unit Weight		
1 pcf	=	0.157 kN/m^3

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Chapter 1

INTRODUCTION

PURPOSE OF EVALUATION

The major advances in the state of the art of in situ testing methods in the past few years have resulted in the development of several devices for measuring soil parameters. One such device, the static cone penetrometer, is gaining widespread use and acceptance in this country after many years of successful use in Western Europe.

This report presents, in case history format, the contractor's experience with the static cone penetrometer and the parameters developed as a result of a wide variety of cases ranging from subsurface profile determiniation to construction control of field densification procedures. In addition, analytical techniques and judgmental factors developed by the contractor for using static penetrometer data in engineering solutions are discussed.

SCOPE

General static cone penetrometer experience and methodology are described in Chapters 2 and 3. In Chapters 4 through 8, the contractor's experiences using this device are discussed. The case histories discussed are:

Waste Treatment Facility, Longview, Washington

Vibroflotation Densification Monitoring, Newport, Oregon

Sawmill and Chipping Facility, Aberdeen, Washington

Southeast Harbor Development, Seattle, Washington

Water Treatment Facility, Lake Oswego, Oregon

A summary of the contractor's use of the static cone penetrometer, and general comments and conclusions, are discussed in Chapter 9.

Chapter 2

STATIC CONE PENETROMETER EXPERIENCE

APPLICATIONS

We have used both 10-ton, truck-mounted, and 20-ton, trailer-mounted static cone penetrometers. Both are self-contained hydraulic units that use Begemann cone and Gaudsche Machinefabriek 20-ton hydraulic load cells. The 10-ton unit has been used to depths up to 160 feet in materials with Standard Penetration Test "N" values up to 70 blows per foot. The 20-ton unit has been used to depths up to 230 feet in materials with "N" values up to 90 blows per foot. We have applied the static cone penetrometer data to the following types of analyses:

- Subsurface soil profile identification
- Shallow footing design (settlement)
- Deep foundation design (capacity)
- Construction control

The most frequent application of the penetrometer is as a subsurface investigative tool. Because it is relatively inexpensive, the penetrometer can be used to scope the general subsurface profile and to define zones of material where additional drilling and sampling information should be obtained.

DATA

The primary in situ properties obtained using the static cone penetrometer include:

- Cone point resistance
- Cone friction resistance

From these properties, second order information can be derived. The secondary information includes:

- Soil type
- Consistency

- Compressibility
- Sand relative density
- Internal friction angle (Ø)
- Pile tip and friction capacity

Pile tip and friction capacity most closely resembles first order or primary data, that is, the action of the penetrometer closely resembles that of a pile. Judgment and experience are required, however, to convert the primary cone data into pile design (see Chapter 3). These elements, judgment and experience, differentiate the primary data from the secondary or derived items.

ACCURACY AND LIMITATIONS

The accuracy of the penetrometer is difficult to quantify. In terms of in situ measurement of primary properties, accuracy is very good. The drawback to accurately measuring the two primary properties is that they are not direct input values for determining geotechnical engineering solutions currently practiced in the United States.

The small penetrometer "sampling interval" of 8 inches (20 cm) enables accurate detection of profile changes and/or thin strata of differing materials. The accuracy of delineating subsurface stratification is considerably better with the static cone penetrometer than with the Standard Penetration Test (SPT). This is principally a result of the finer "sampling interval" and reduced subjectivity of the static penetrometer compared with the SPT. As described in Chapter 5, 2- to 12-inch (5- to 30-cm) silt lenses were not detected when initially sampled at 5-foot (1.5-m) intervals with the SPT, but were subsequently detected when the same area was probed using the static penetrometer. The usual SPT sampling procedure consists of driving an 18-inch (46-cm) sampler at 5-foot (1.5-m) intervals, leaving a 3.5-foot (1-m) interval unsampled. The SPT sampling interval can be reduced to almost continuous sampling, but the cost increases in direct proportion to the extra number of samples. Static cone penetrometer detection accuracy is about 100 percent for 12-inch (30-cm) thick lenses; this drops to about 75 percent for 6-inch (15-cm) thick lenses. Accuracy drops off rapidly for lenses thinner than 6 inches.

Our experience with the static cone penetrometer in the Pacific Northwest has involved hydraulically operated rigs that eliminate much of the subjectivity of the operator.

Under certain circumstances (generally rare), dial readings may not stabilize within the sampling interval and hence cone or friction resistances for such intervals will not be obtained. Our experience is that the operator can generally determine when dial variation is associated with gravelly materials based on the vibration and behavior of the machinery.

The accuracy of the second order information is subject to the complexity of the subsurface profile, plus the experience and judgment of the engineer. Most of the second order derivatives are based on statistical correlations with previous experience. If the previous experience is not similar to the project being studied, extrapolation may or may not be accurate. In addition, a complex profile is usually treated by averaging data over selected intervals. The more complex profile leads to more averaging and less accuracy. In alternating lenses of sand and clay, the friction ratio alone might indicate a silty sand. In such cases, a pattern of repetitive cone resistance changes may be used to advantage to detect material changes. We do not rely exclusively on the penetrometer for subsurface investigations, since it does not allow samples to be taken and groundwater measurements to be performed.

SITE LIMITATIONS

Our use of the static cone penetrometer has principally involved recent alluvial materials. The penetrometer is not used for rock exploration. The reliability of the penetrometer in investigating weathered (residual) materials, sensitive clays, caliche, preconsolidated sands, or other materials associated with older geologic horizons is unknown. The penetrometer is unable to penetrate indurated formations. Depending upon the machine used, the penetrometer is usually restricted to material with a SPT blowcount of less than 70 to 90 blows per foot. Our experience indicates that the usefulness of the penetrometer is limited to material containing less than about 45 percent of 1/2-inch or smaller gravel. This depends, however, upon the thickness of the layer to be penetrated and its density. When penetrating soft or squeezing ground, the penetrometer hydraulic system must be able to overcome friction on the rods above the penetrometer. Friction-reducing sleeves are available to reduce the rod friction by cutting an oversized hole. The penetrometer and rods should not be left down an uncompleted hole for any significant amount of time, however, because adhesion or soil freeze could make retrieval difficult.

In addition to soil type limitations, there are some mechanical limitations to using the static cone penetrometer. These include:

- Hydraulic capacity
- Reaction capacity of the rig
- Necessity for rig stability

The hydraulic capacity must be sufficient to overcome 1) point and cone resistance when penetrating a hole, and 2) adhesion plus the weight of the rods when retracting from a hole. Penetrometer rigs found in the Pacific Northwest rely on weight as a reaction against which to push the penetrometer. These rigs are mobile, self-contained units. Water tanks can be filled for added ballast. For ease in setting up, screw anchors are not employed. When working over water, a freestanding barge or fixed casing platform from which to work is necessary. Otherwise, wave action would lead to erratic readings, that is, a portion of the rig weight would cyclically be supported by the penetrometer point and then by the wave tops. We have not used the penetrometer for exploration over water because of the costs involved in setting up a fixed platform.

A specific site problem that has caused difficulties in the past involves soft, miscellaneous fill or miscellaneous fill with voids overlying dense or very stiff materials. As the penetrometer moves through the fill, it can, to some extent, bend its way around the obstructions. When the penetrometer encounters an unavoidable obstacle firm underlying materials, caution must be used to avoid overloading the penetrometer rod system. When too much hydraulic force has been supplied, the rod has buckled and broken.

Chapter 3

STATIC CONE PENETROMETER METHODOLOGY

BACKGROUND

This chapter outlines our approach to the use of the static cone penetrometer. The 1975 subsurface investigation for a wastewater facility in Longview, Washington (described in detail in Chapter 4) was the first project on which we used the static cone penetrometer. Our use of the device followed publication of Static Cone to Compute Static Settlement Over Sand [1]* and The Penetrometer and Soil Exploration [2].

As indicated in Chapter 2, our most frequent use of the cone type penetrometer has been as a primary tool for investigating subsurface soil profiles and for defining zones of material from which additional information should be obtained. Our confidence in the use of the penetrometer has grown because we have repeatedly used both the cone type penetrometer and conventional soil test borings and sampling techniques on the same projects. This use of multiple investigative techniques has afforded many opportunities to compare the data obtained with each method. We have never used the static penetrometer as the sole subsurface investigative tool. Most empirical correlations used by our firm for data interpretation were developed by others.

THE PENETROMETER

We used a Begemann cone penetrometer, which permits local measurement of both point and skin friction resistance. Load readings were taken using a hydraulic load cell. Mechanical operation of the cone is detailed on Figure 1. The testing sequence was as follows:

1. Advance cone 1.6 inches (4 cm) for a point resistance reading.

2. Advance cone and sleeve an additional 1.6 inches (4 cm) for a cone plus sleeve reading.

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*Numbers in brackets refer to references listed at the end of this report.

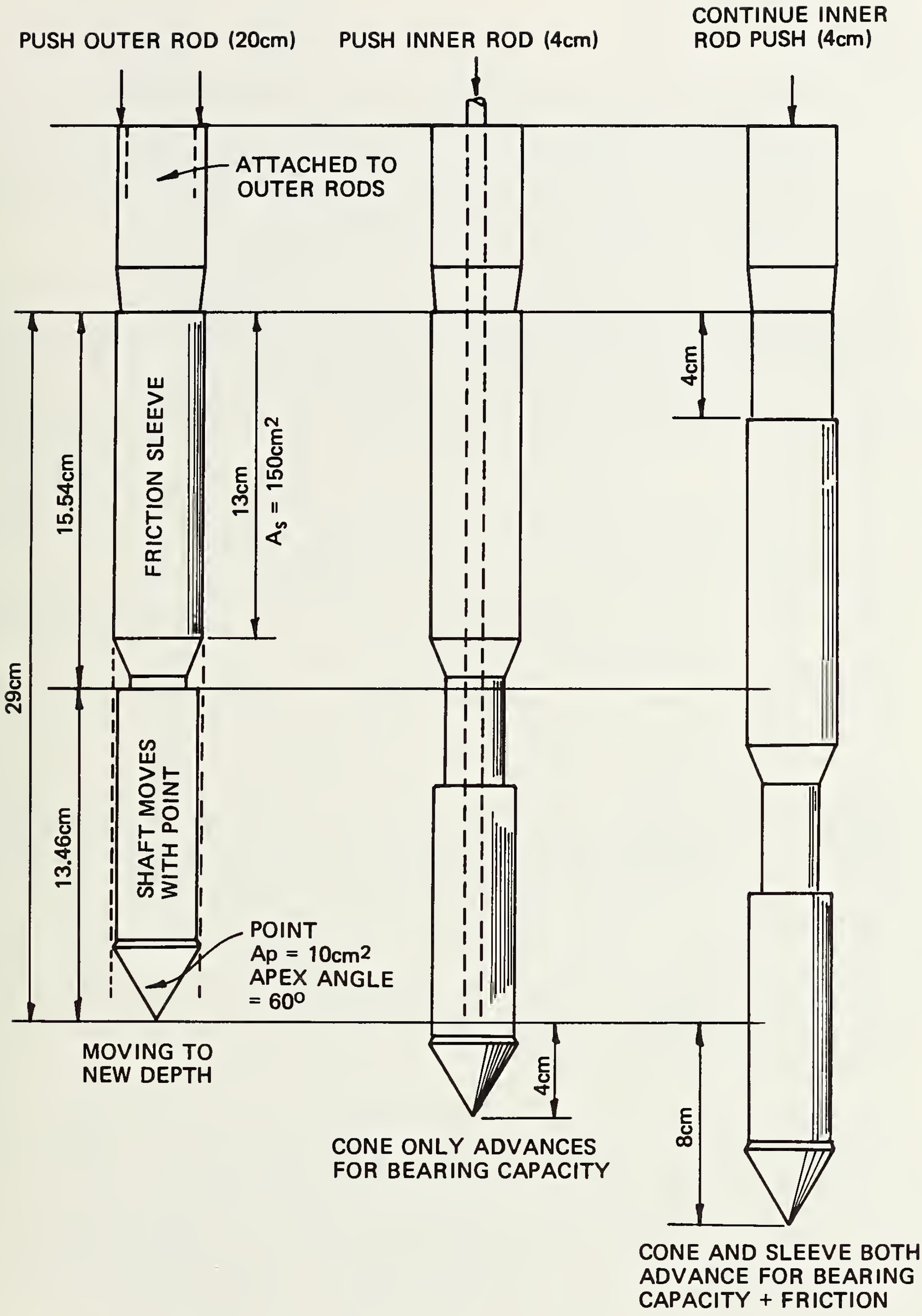


Figure 1. Operation of Begemann cone

3. Push outer rod 8 inches (20 cm) for a total system reading. (Sleeve advances an additional 16 cm and point advances an additional 12 cm, so that the outer rod, sleeve, and point are again in contact).

4. Repeat process starting with step 1.

During its advance, the penetrometer moves at the rate of 1 to 2 centimeters per second. The total system reading has little practical interest other than as a guide to the operator. The operation described above is typical in that readings are repeated on 20 centimeter intervals. Readings could be taken at less than 20 centimeter intervals, but this is not usually done because decreasing the testing interval would increase the relatively low cost of using the static cone penetrometer. For other than shallow footings, a smaller interval does not seem warranted.

The principal data obtained from the penetrometer are:

- Cone point resistance (qc)

- Local sleeve friction resistance (fs)

An important ratio, the "friction ratio," is determined by dividing the local sleeve friction by point resistance. The numerator and denominator must be from the same depth. The sleeve resistance is determined by subtracting the cone resistance from the sum of the cone resistance and the sleeve resistance, assuming the cone reading is constant over each 8 centimeter movement. The friction ratio is determined by dividing the $(n+1)^{th}$ sleeve resistance by the n^{th} point resistance value. No friction ratio is obtained for the first interval sampled.

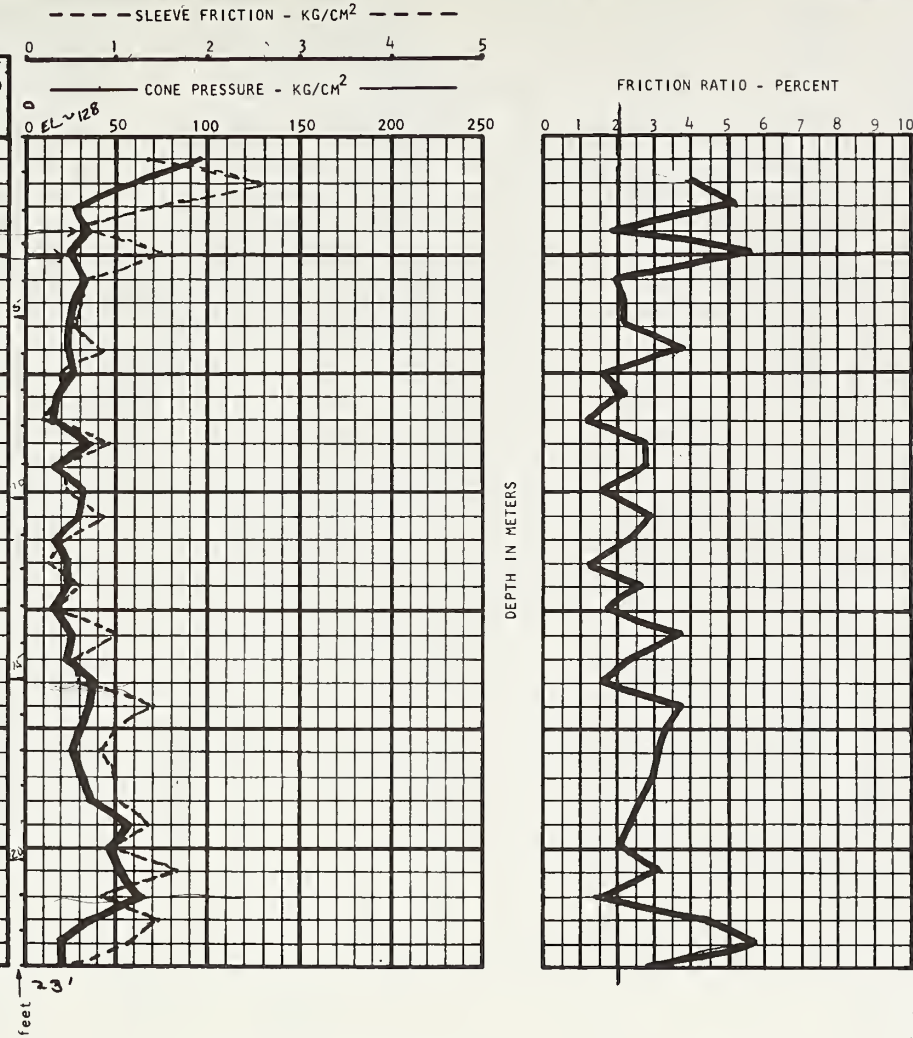
Inspection by our personnel is not required with the static cone penetrometer other than for determining the probe locations and depths. Data are recorded by the subcontractor on a standard log sheet. The subcontractor can plot the point resistance, sleeve friction, and friction ratio with depth on the log sheet and interpret the soil types encountered. Figure 3-2 shows a completed subcontractor's log sheet.

DUTCH CONE LOG & CALCULATION SHEET

CLIENT CH2M (Pd) LOCATION Lake Oswego JOB NO ______ PROBE NO P-1 DATE 6/26/75 SHEET 1 of 3

COMMENTS	NO. RODS	DEPTH IN METERS	DIAL READING TOTAL SYSTEM	CONE ONLY	CONE + SLEEVE	SLEEVE READING	CONE PRESS. — KG/CM²	SLEEVE FRICT. — KG/CM²	FRICT RATIO — %
	2	.2	1200	960	NR	—	97	—	
		0.4	900	590	790	200	60	1.33	4.2
		0.6	820	280	660	380	29	2.53	5.1
		0.8	940	320	540	220	33	1.47	1.8
		1	940	250	340	90	26	.60	5.7
	3	1.2	940	300	520	220	31	1.47	1.9
		1.4	850	260	350	90	27	.60	2.2
		1.6	770	230	320	90	24	.60	2.2
		1.8	660	220	300	80	23	.53	3.8
		2	900	250	380	130	26	.87	1.5
	4	2.2	800	180	240	60	19	.40	2.1
		2.4	830	160	220	60	17	.40	1.2
		2.6	1100	340	370	30	35	.20	2.7
		2.8	930	140	280	140	15	.93	2.7
		3	1200	300	360	60	31	.40	1.5
	5	3.2	1200	290	360	70	30	.47	2.9
		3.4	1200	160	290	130	17	.87	2.4
		3.6	1100	210	270	60	22	.40	1.2
		3.8	1200	220	260	40	23	.27	2.6
RS		4	1100	170	260	90	18	.60	1.8
	6	4.2	1200	250	300	50	26	.33	3.8
		4.4	1400	220	370	150	23	1.00	2.3
		4.6	1400	380	460	80	39	.53	1.5
		4.8	1500	370	460	90	38	.60	3.7
		5	1800	290	520	210	30	1.40	3.3
	7	5.2	1700	270	420	150	28	1.00	3.1
		5.4	1800	320	450	130	33	.87	3.0
		5.6	1700	370	520	150	38	1.00	2.6
		5.8	2000	550	700	150	56	1.00	2.4
		6	1900	470	670	200	48	1.33	2.1
	8	6.2	1900	510	660	150	52	1.00	3.2
		6.4	1900	630	880	250	64	1.67	1.4
		6.6	1700	340	47	130	35	.87	4.4
		6.8	1400	190	420	230	20	1.53	5.7
		7	1500	190	360	170	20	1.13	2.7

NOTE: 1 KG/CM² = 1.024 TON/FT.²



SOIL INTERPRETATION

- Clayey silt (stiff)
- Sandy silt to silty sand w/layers of sand (loose - med dense)
- Silt w/s clay stiff)
- Sandy silt to silty (med dense)
- Clayey silt to silty clay very stiff

COMMENT LEGEND

RS	Rising slowly	D	Dropping
DS	Dropping slowly	DV	Dial variation
R	Rising	SDV	Some dial variation

Figure 2. Dutch cone log and calculation sheet

Upon receipt of the penetrometer data as presented on Figure 2, the following steps are taken to process the data, where appropriate:

- Data check: read the data, checking arithmetic and calculations of friction ratio.
- Classification: interpret subsurface soil types.
- Compressible zones: check for zones of plastic material where normal consolidation might be expected.
- Settlement: estimate structural settlements.
- Piling: estimate pile design capacities.

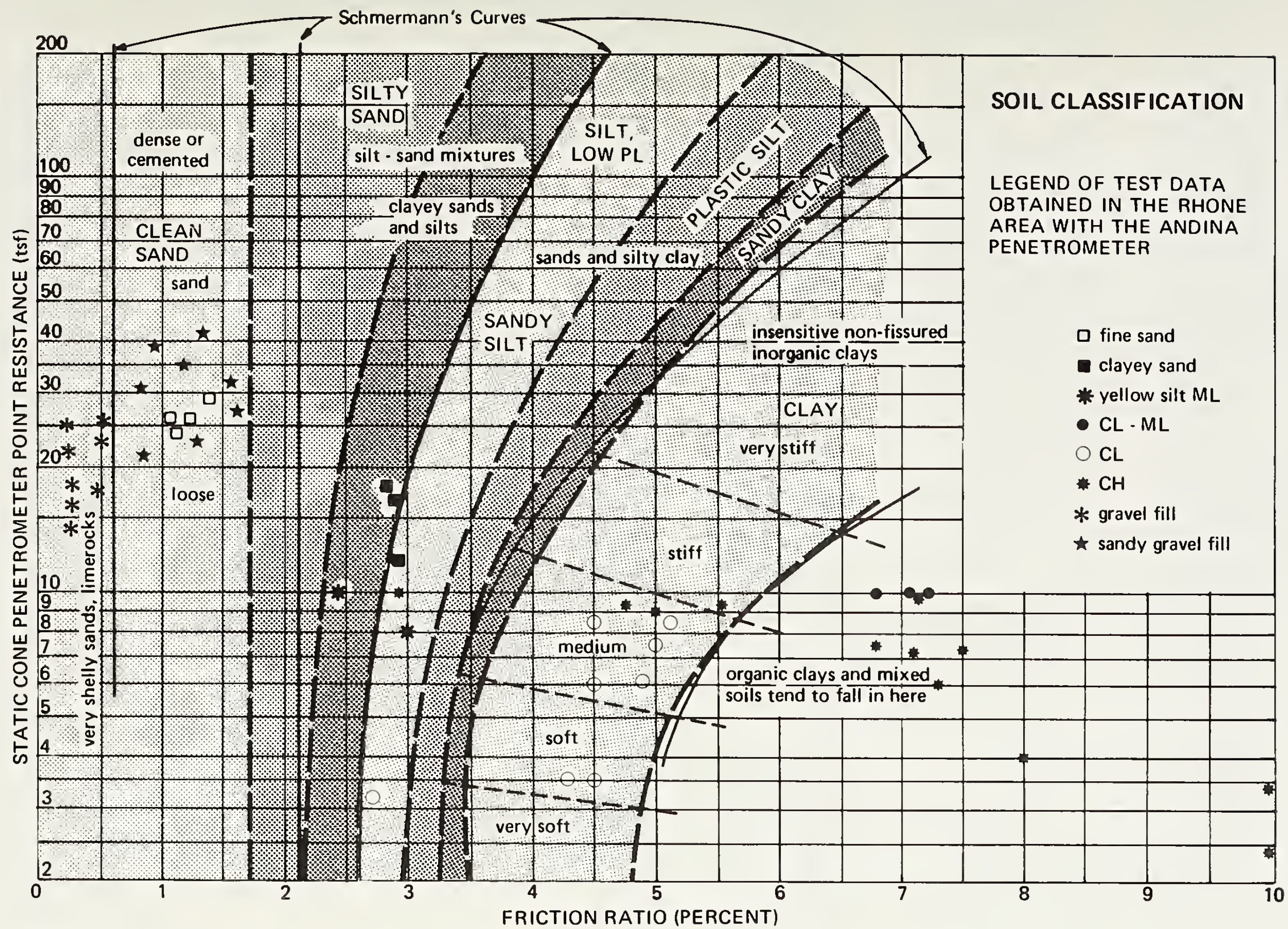
Not all steps are performed for each investigation. A discussion of each step follows.

DATA CHECK

The first step is clerical and constitutes an initial familiarization with the data.

CLASSIFICATION

The general soil type is determined by comparison of the local friction ratio and point resistance with previous correlation studies. Judgment may be used to interpret soil types, particularly borderline soil mixtures such as silty sands and sandy silts. With such mixtures, the silt fraction may determine behavior even though somewhat less than 50 percent silt is present. It is therefore more critical to recognize such soils than to accurately classify them. The correlation chart shown on Figure 3 has proven fairly reliable for the soils encountered in the Pacific Northwest, particularly the Portland, Oregon area. The initial form of Figure 3 was presented by G. Sanglerat [3], but has been modified to correlate with local geology and experience. Clayey sands and sandy clays are not extensive in the Portland area, whereas sands, silts, clays, and sand-silt mixtures are. The sandy clay zone thus occupies a fairly narrow band between the wider silt and clay bands.



NOTE:

Friction ratio values decrease in accuracy with low values of qc and when within a few feet of surface. . . . . 1 tsf = 95.8 kPa

Figure 3. Soil classification chart (1)

For clean sands, the friction ratio alone is generally sufficient for classifying material. For friction ratios greater than 2, both the friction ratio and the cone point resistance (qc) must be considered. Similarly, a one point fluctuation in the friction ratio without a significant change in the point resistance must be considered carefully if a change in the classification is involved. This is particularly true if materials appear the same both above and below the suspect point.

Judgment must determine the importance of accurately defining one point strata. If the decision involves a possible thin, shallow, weak clay stratum at a site otherwise suitable for shallow foundations, perhaps more information is required. If the one point classification would not affect the design, the value of the outcome is more academic than practical.

Once soil strata have been interpreted and classified, some of the in situ material properties may be estimated. Two properties that are fairly easy to determine are relative densities for sand and undrained shear strengths for plastic materials.

Schmertmann Cone Calibration Chart, Figure 4, has been used successfully to determine in situ sand relative densities (Dr). Use of the chart is as follows:

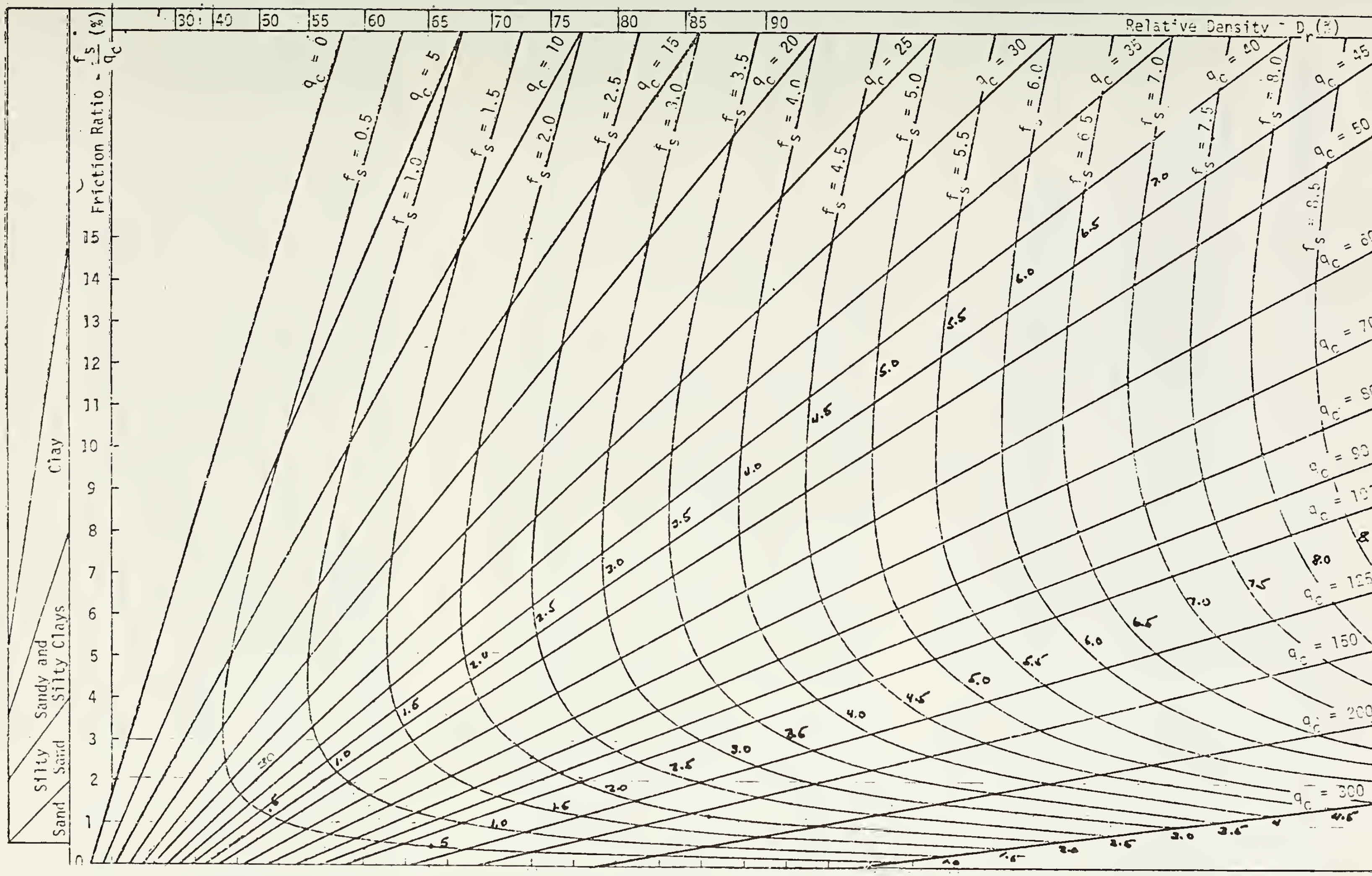
1. For the in situ sand being measured, determine the local sleeve (fs) and point (qc) resistances.

2. Locate the corresponding qc and fs curves on the chart. Some interpolation may be required.

3. Locate the intersection of the selected fs and qc curves.

4. From the intersection (step 3) move vertically upward on the chart and read the in situ relative density.

Use of the static cone penetrometer to measure in situ relative densities and compaction obtained after vibroflotation is described in Chapter 4. Comparisons of in situ densities determined using the static penetrometer versus the standard penetration test (Gibbs and Holtz method) [4] are tabulated in most of the case studies. In the absence of good laboratory test results, particularly in clean sands where undisturbed samples are difficult to obtain, an in situ strength may be estimated by comparing the measured



q_c = POINT RESISTANCE (kg/cm^2)
f_s = SLEEVE FRICTION (kg/cm^2)

Figure 4. Schmertmann cone penetrometer calibration chart

density with published laboratory test results such as those by Wong and Duncan. [5]

In very plastic silts and clays, saturated, undrained, shear strength equals cohesion (C), which remains constant. Empirical correlations described by Sanglerat indicate that for a $\emptyset=0$ analysis, the cohesion may be determined from either the cone point resistance or the sleeve friction.

Several formulas by various researchers for determining C when $\emptyset=0$ are presented by Sanglerat. The ones the authors apply most often, because of their simplicity, are:

$$C = qc/14, \text{ and} \qquad (3\text{-}1)$$

$$C = fs \qquad (3\text{-}2)$$

We believe that the correlations do not provide reliable strength values because $\emptyset=0$ situations are rare in our experience. The presence of a $\emptyset$, other than zero, increases the point resistance. With $\emptyset=0$, the $\emptyset$- and C- resistance components are not distinguishable. The friction jacket measures the adhesion of a remolded soil and is probably a conservative value of the cohesion. The importance of soil fabric on clay strengths is described in detail in an article by Seed and Chan [6]. Some of our attempts to correlate penetrometer data with material strengths are described in the case history discussions. Since our field investigations included test borings and static penetrometer probes, the strengths of plastic silts and clays were determined by laboratory strength tests.

COMPRESSIBLE ZONES

Sanglerat concludes that a point resistance of about 12 kg/cm^2 is the borderline between suitable and unsuitable soils for shallow foundations. In soils with a point resistance less than 12 kg/cm^2, total and differential settlements may be structurally damaging. Another rule-of-thumb applicable to compressible material was suggested by Schmertmann [7]. He suggests that normally consolidated soils can be recognized when the cone point resistance (qc, kg/cm^2) is less than the depth in feet (Z), divided by three. We generally use both tests for selecting stratum for which laboratory consolidation tests may be appropriate.

SETTLEMENT

We have used three different methods to predict settlements using cone data. The methods are referenced by the names of their originators, and include the modified Meyerhoff, Schmertmann, and Terzaghi-Buisman methods. Selection of one method or another depends upon soil type, available data, and type of settlement problem.

For design, the modified Meyerhoff method is most suited to predicting shallow footing settlements in sandy materials. The applicable equation is:

$$q_{ad}(1'') = q_c \left(\frac{B+1}{B}\right)^2 (W^1)\ (Kd)/24.5,\ B>4 \text{ ft}$$

$$= q_c\ (W^1)\ (Kd)/14.6,\ B\ B \leq 4 \text{ ft} \qquad (3\text{-}3)$$

Where:

$q_{ad}(1'')$	=	design footing pressure (KSF) for 1 inch of settlement
q_c	=	penetrometer cone resistance (kg/cm^2)
B	=	least footing dimension (ft)
W^1	=	water reduction factor, which equals 1.0 when water surface is greater than B below footing and 0.5 when water surface is at the footing level
Kd	=	$(1 + D/B) \leq 1.33$, D = footing depth

The equation is simple to apply but can be modified as proposed by Bowles [8]. His evidence indicated Meyerhoff's original equations were too conservative. For preliminary design, building locations, internal layouts, and foundation loads are usually unknown. For shallow footing design, Meyerhoff's predictive equation is useful for the preliminary sizing of footings. Once the building is designed, Schmertmann's method may be used to predict operating settlement.

Schmertmann's method for predicting settlement over sand is described in detail by Schmertmann [9]. The applicable equation is:

$$\rho = C_1 C_2\ \Delta p \sum_{0}^{2B} \left(\frac{Iz}{Es}\right) \Delta Z \qquad (3\text{-}4)$$

Where:

ρ	=	predicted settlement
C_1C_2	=	constants
Δp	=	structural base loading
B	=	structural base dimension
Iz	=	an influence factor
Es	=	soil modulus (a function of depth and cone point resistance)
ΔZ	=	incremental layer thickness

The method was applied in the example set of calculations included in Chapter 8. Once a particular design has been established, the method is very simple to use.

The Terzaghi-Buisman method involves the use of empirical correlations to determine C, the constant of compressibility, from the point resistance qc. Once C is determined, the classical one-dimensional consolidation formula may be used to predict settlement:

$$\frac{\Delta h}{h} = \frac{1}{C} \ln \left(1 + \frac{\Delta p}{po}\right) \qquad (3\text{-}5)$$

Where:

Δh	=	change in layer height
h	=	initial layer height
po	=	initial overburden pressure
Δp	=	change in layer pressure

We have seldom used the method for other than sands because of the difficulty in quantifying the value C. Published [10] correlations between qc and C indicate a range of C equal to 0 to 1.5 times qc/po for sands, but a range of 1 to 8 times the same ratio for silts and clays. Chapter 7 describes application of the method to the prediction of settlement beneath a large sand fill. Extensive correlations among qc, C, and C_c (the compression index) are tabulated in Sanglerat.

PILING

We use the Begemann [11] approach to the design of end bearing pile capacities with static cone penetrometer data. The method is outlined by Sanglerat [12]. The method consists of extrapolating the cone bearing pressure to a bearing pressure that corresponds to the selected pile diameter. The cone pressure is an ultimate or failure pressure, hence the extrapolated pressure is also a failure pressure and must be reduced by a safety factor (F.S.) for design. The extrapolation is as follows:

$$q_{cd} = \frac{1}{2}\,(qc_1 + qc_2) \qquad (3\text{-}6)$$

Where:

q_{cd} = ultimate pressure for the diameter (d) of the pile

qc_1 = average qc, 3.5 x d below the pile base

qc_2 = average qc, M x d above the base (M normally taken as 8 for sands, 1 for very stiff saturated clays)

The method is principally suited for a well defined end bearing stratum where skin friction above the pile base is negligible. A safety factor of 2 to 3 is normally applied (2 to 2.5 when skin friction is neglected, 2.5 to 3 when skin friction is included). The local sleeve resistance is used as the pile adhesion; hence, allowable pile loads are determined as:

$$Q_{allowable} = \frac{1}{F.S.}\,(q_{cd} \times Ap + \sum_{0}^{l} fs \times As) \qquad (3\text{-}7)$$

Where:

Ap = pile tip area

l = pile length

As = gross pile skin area

Chapter 4

CASE HISTORY 1 - WASTEWATER TREATMENT FACILITY LONGVIEW, WASHINGTON

PROJECT DESCRIPTION

In order to upgrade effluent quality from its Longview, Washington pulp mill, Longview Fibre Company constructed a secondary wastewater treatment facility. We performed the geotechnical investigation for the site in early 1975. Site location for the facility is a 23-acre area near the confluence of the Cowlitz and Columbia Rivers. Figure 5 is a site plan and vicinity map. The scope of the investigation included review of previous investigations, subsurface penetrometer probing, soil test drilling and sampling, laboratory testing, and engineering analyses. As part of the analyses, subsurface soil types were profiled and foundation settlements estimated. This subsurface investigation was our first use of the static penetrometer. Plant construction was completed in mid-1977.

AREA GEOLOGY

Area geology has largely been determined by the Columbia River. Quaternary and tertiary silts, sands, and gravels deposited by the Columbia River extend more than 200 feet (61 m) below the project. Quaternary deposition and reworking of the Columbia River sediments is attributable to the Cowlitz River. Meandering by the Cowlitz has disrupted some of the soil profile horizontal continuity.

SUBSURFACE INVESTIGATION

In August 1966, four test borings were completed for the primary clarifier foundation investigation. Subsequently, the clarifier location was changed and post-construction settlement occurred which prompted the thorough subsurface investigation for the secondary facilities.

Subsurface conditions for the secondary treatment facilities were investigated in February and March 1975. Five penetrometer probes and nine soil test borings comprised the field work for the investigation. Logs of the test borings and probes are shown on the subsurface profiles, Figures 6 through 8

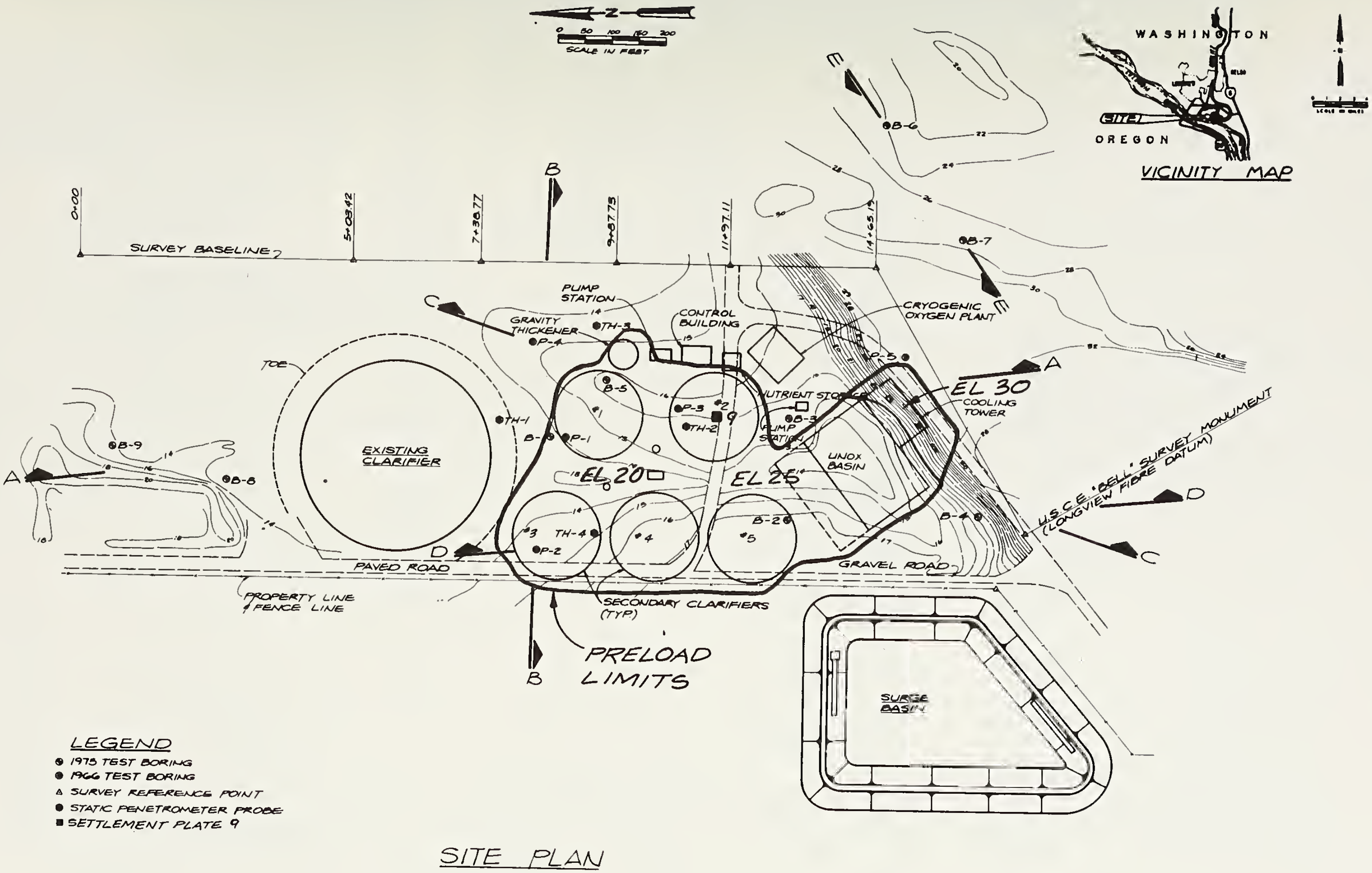
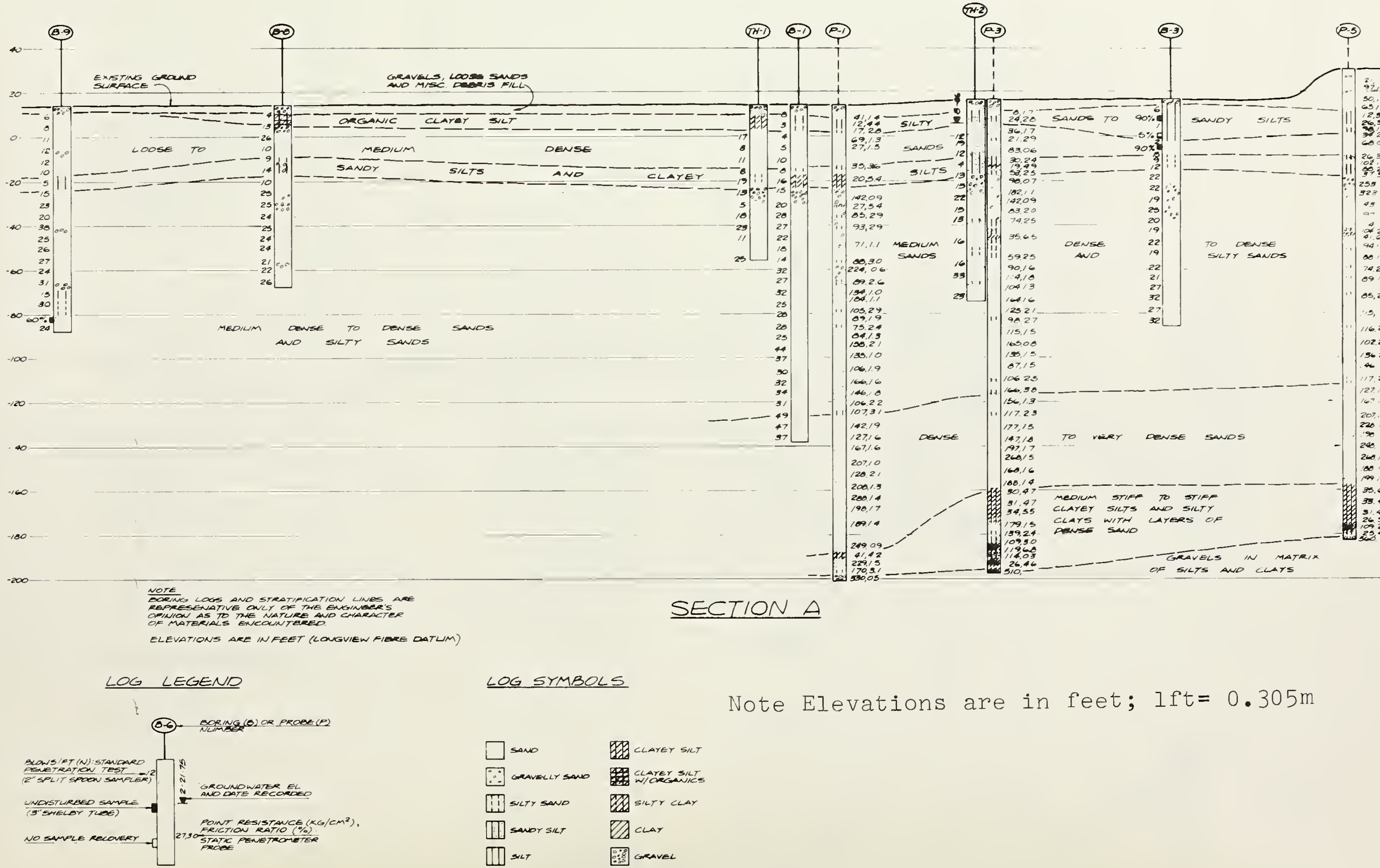


Figure 5. Case history 1 vicinity map and site plan



Note Elevations are in feet; 1ft= 0.305m

Figure 6. Case history 1 subsurface profile A

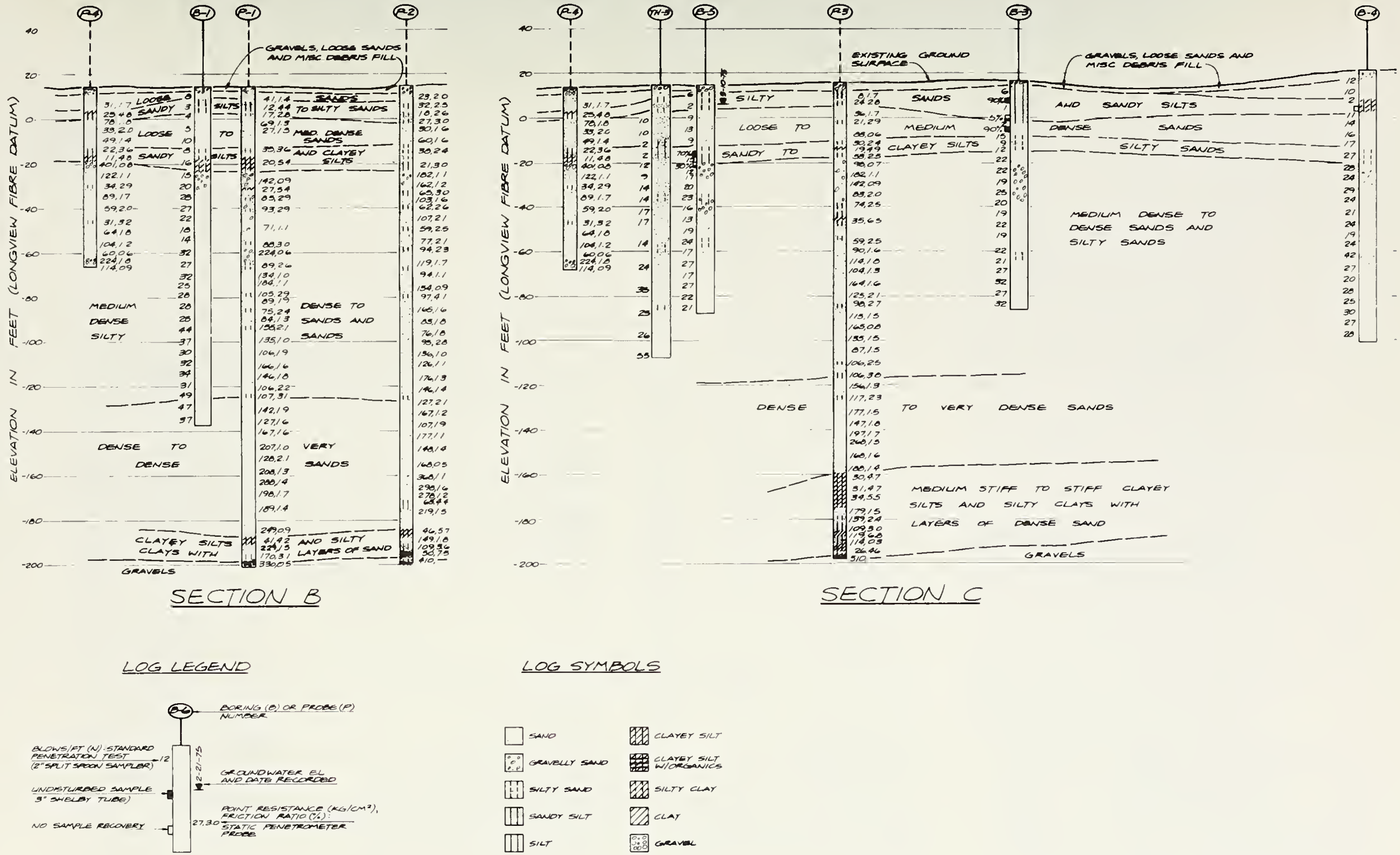


Figure 7. Case history 1 subsurface profiles B and C

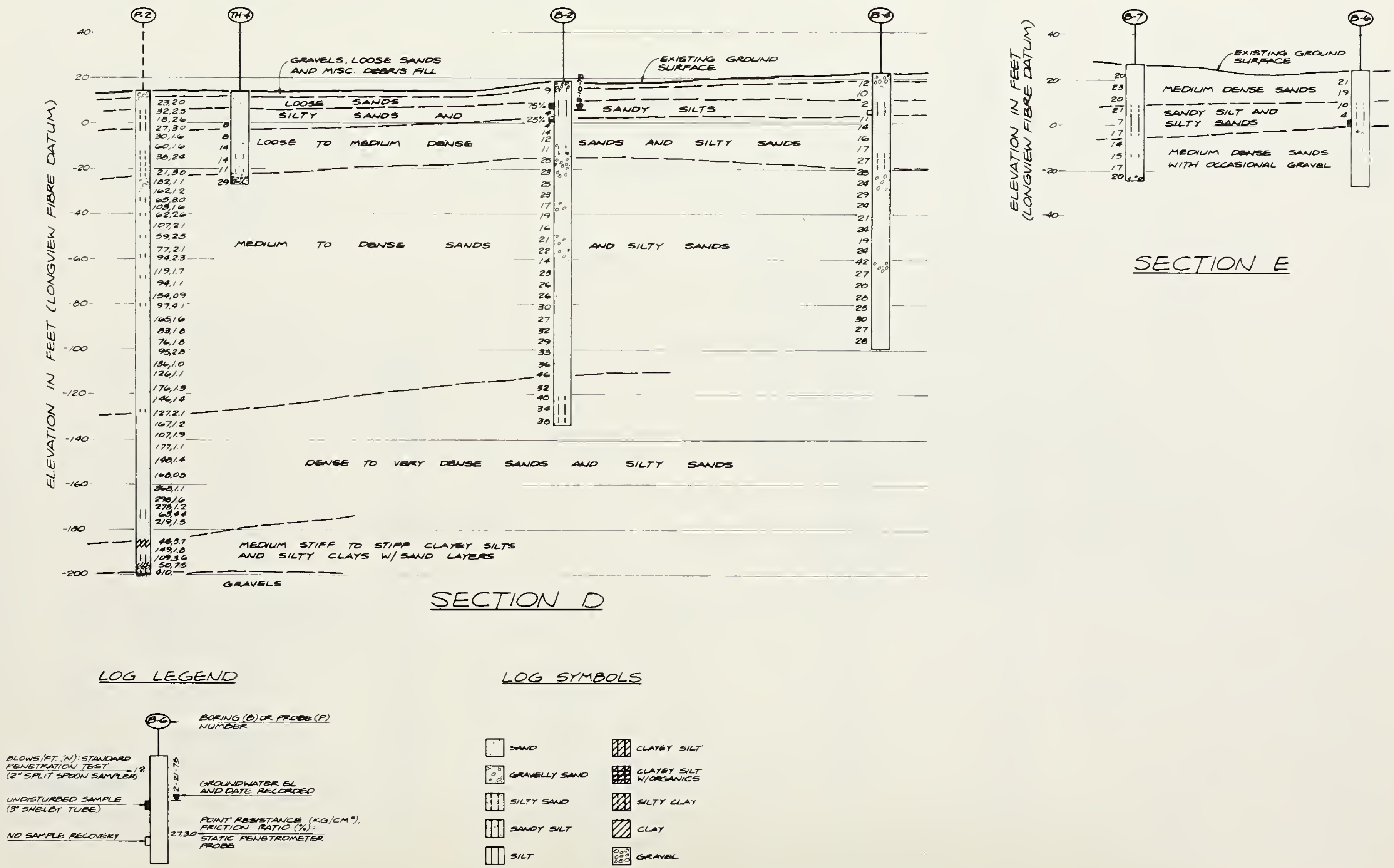


Figure 8. Case history 1 subsurface profiles D and E

Test Borings

Test borings were completed using a CME 55 rotary drill rig. Bentonite drilling mud was used to maintain hole stability. Depths ranged from 50 to 150 feet (15 to 46 m). Representative soil samples were obtained in all borings at minimum intervals of 5 feet (1.5 m). Disturbed samples were taken with a 2-inch (5.1 cm) outer diameter (O.D.) standard penetration test (SPT) sampler. SPT blowcounts, N, are shown on the boring logs. Three-inch (7.62 cm) O.D., thin-walled Shelby tubes were used to recover undisturbed samples. Sample recovery in the test borings was very good--only 3 samples were missed out of 190 sample attempts. Laboratory tests were performed on selected soil samples.

Static Penetrometer

A penetrometer fitted with a Begemann sleeve was used for the static penetrometer probes. Cone plus cone and sleeve readings were repeated at 8-inch (20 cm) intervals. Probe depths ranged from 81 to 214 feet (24.7 to 65.2 m).

Static penetrometer and test boring completion costs are shown in Table 1.

Table 1. Longview Fibre subsurface investigation unit costs* (1975 dollars)

	Excluding Mobilization	Including Mobilization
Static Penetrometer	\$3.00 per foot	\$3.10 per foot
Rotary Drilling	\$6.01 per foot	\$6.07 per foot

* CH2M HILL inspection costs not included.

SOIL PROFILE

The subsurface soil profile is shown on Figures 6 through 8. The stratification lines shown indicate inferred changes in material type, density, and/or consistency. Considerable variation exists in the upper 30 to 40 feet (9.1 to 12.2 m) of the profile. The top 2 to 4 feet (0.6 to 1.2 m) of material consists of miscellaneous fill. Beneath this surface layer, the profile consists of sands, silty sands, and silts. Granular materials range from very loose to medium dense. Cohesive soils vary from sandy and clayey silts to soft organic silt. Plasticity ranges from low to medium. The subsurface cross sections indicate some degree of horizontal continuity over the area. However, in terms of significant engineering characteristics, the upper 30 to 40 feet (9.1 to 12.2 m) cannot be well represented by continuous horizontal layers.

Organic clayey silt was encountered at depths of approximately 5 feet (1.5 m) in the vicinity of the existing clarifier. Sandy to clayey silts, in layers 5 to 20 feet (.5 to 6.1 m) thick, were found throughout the central and southern portions of the site. The less plastic of these silts were within 15 to 20 feet (4.6 to 6.1 m) of ground surface, while those of higher plasticity were at depths of 25 to 30 feet (7.6 to 9.1 m). Non-plastic materials--sands and silty sands--appear to dominate the northwestern part of the project area.

Medium- to fine-grained sand with some silty sand and gravel underlies the variable upper zone (top 30 to 40 feet (9.1 to 12.2 m)) of the soil profile. This sand stratum varies in thickness from 140 to 160 feet (42.7 to 48.8 m) and generally increases in relative density with depth. The upper 90 to 100 feet (27.4 to 30.5 m) may be classified as medium dense to dense. At greater depths, the sand is dense to very dense.

Underlying the thick sand stratum is a layer of medium stiff to stiff clayey silt and silty clay, interspersed with dense sand and silty sand. The layer dips markedly to the north and west from P-3, possibly defining an older river channel.

The stiff silt-clay layer overlies a gravel formation extending to unknown depths. Its top boundary is near elevation -180 mean sea level at P-5, dipping north and east to elevation -195 to -200 mean sea level at P-1, P-2, and P-3. The formation was penetrated 1 to 4 feet (0.4 to 1.2 m), and appears to consist of gravels and coarse sands in a silt-clay matrix. No probes or borings penetrated through the gravels. The deeper gravels are thought to be representative of the Pliocene Troutdale formation.

SOIL CLASSIFICATION

Excellent agreement exists between the penetrometer inferred soil classifications and those determined by the test boring. P-1 and B-1, shown graphically on subsurface Profile A, were located less than 25 feet (7.6 m) apart. Penetrometer inferred classifications were based on the soil classification chart described in Chapter 3. Very good agreement also exists with each method's location of the different soil contacts.

The large size of the project area and the significant number of soil samples obtained prohibits a detailed discussion of soil sample similarities. The interested reader is instead advised to review the subsurface profiles.

Because P-1 and B-1 are very close, their in situ sand relative densities can be conpared, as shown on Figure 4-5. Relative densities in the sands were determined from the SPT "N" values using the criteria of Gibbs and Holtz [13] for wetted sands. Densities were determined from the penetrometer data using the cone calibration chart described in Chapter 3. Both relative density curves have similar shapes. Examination of Figure 9 indicates that the Gibbs and Holtz determined densities are conservative compared to those determined from the penetrometer data. Some of the differences in densities may be attributable to the distance between the probes (approximately 25 feet (7.6 m)) and some to actual density variations. Some of the variation may also be attributed to the parameters discussed by Gibbs and Holtz [14], for example, SPT rod weight, saturated versus partially wetted sands below the water table, and rod whipping. The two estimates of relative density are generally within 30 percent of each other and show a similar trend of density versus depth.

COMPRESSIBLE ZONES

The presence of compressible subsurface zones at the site was known from the 1966 primary clarifier investigation. The four 1966 test holes encountered soft silt strata near the surface. When the location for the clarifier was moved, additional auger holes also detected the silt. The silt was detected in discontinuous strata of variable thicknesses. The existing clarifier was founded on an 8-foot (2.4-m) high fill. To minimize post-construction settlement of the clarifer and structural fill, a 7-foot (2.1-m) preload ring was placed on top of the structural fill over the wall footing area. During the 2-month preload period, settlement of up to 2 inches (5.1 cm) was measured. Maximum post-construction differential settlement of

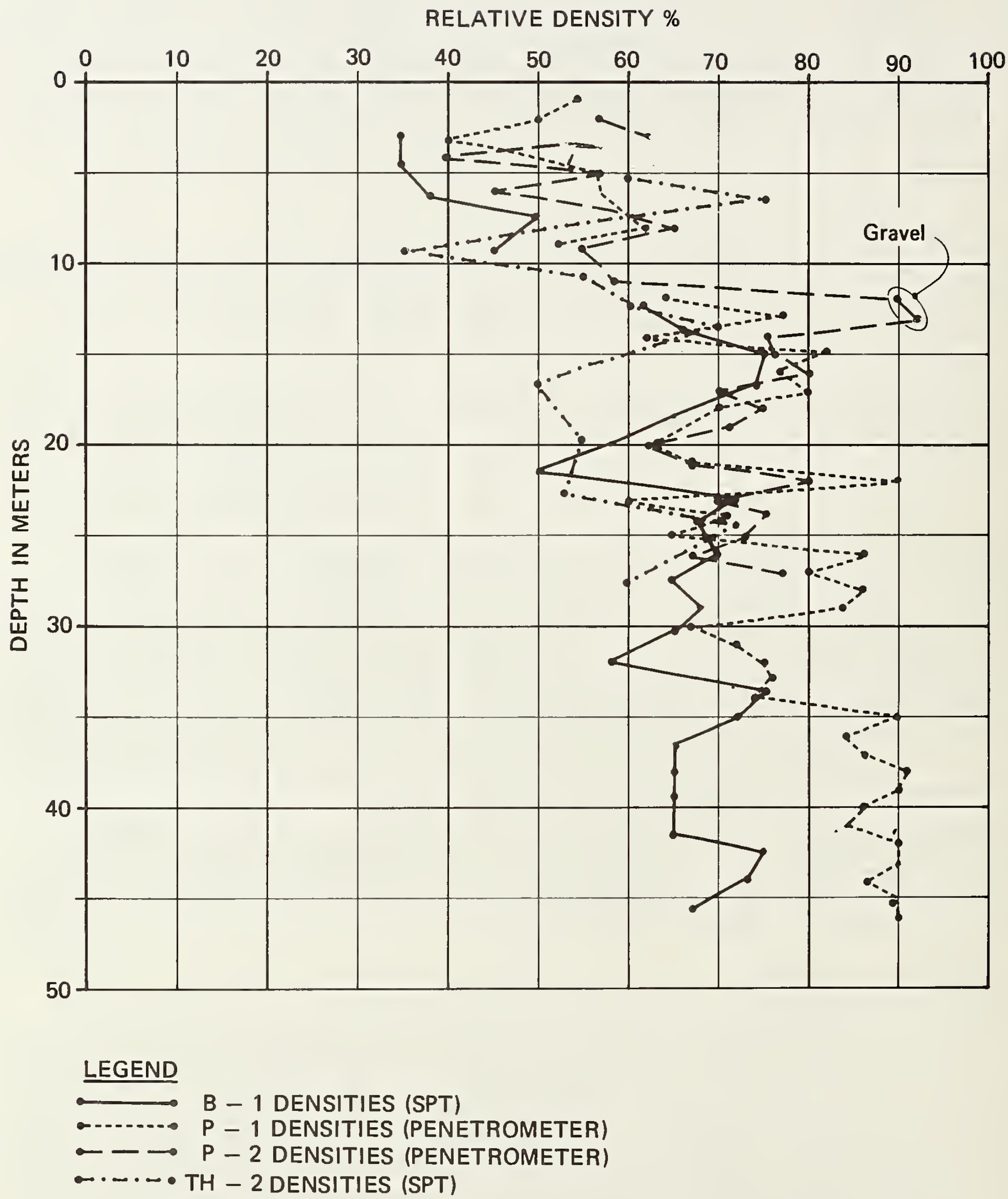


Figure 9. Case history 1 relative density comparison

5/8 inch (1.6 cm) around the primary clarifier of up has been measured across some of the wall joints.

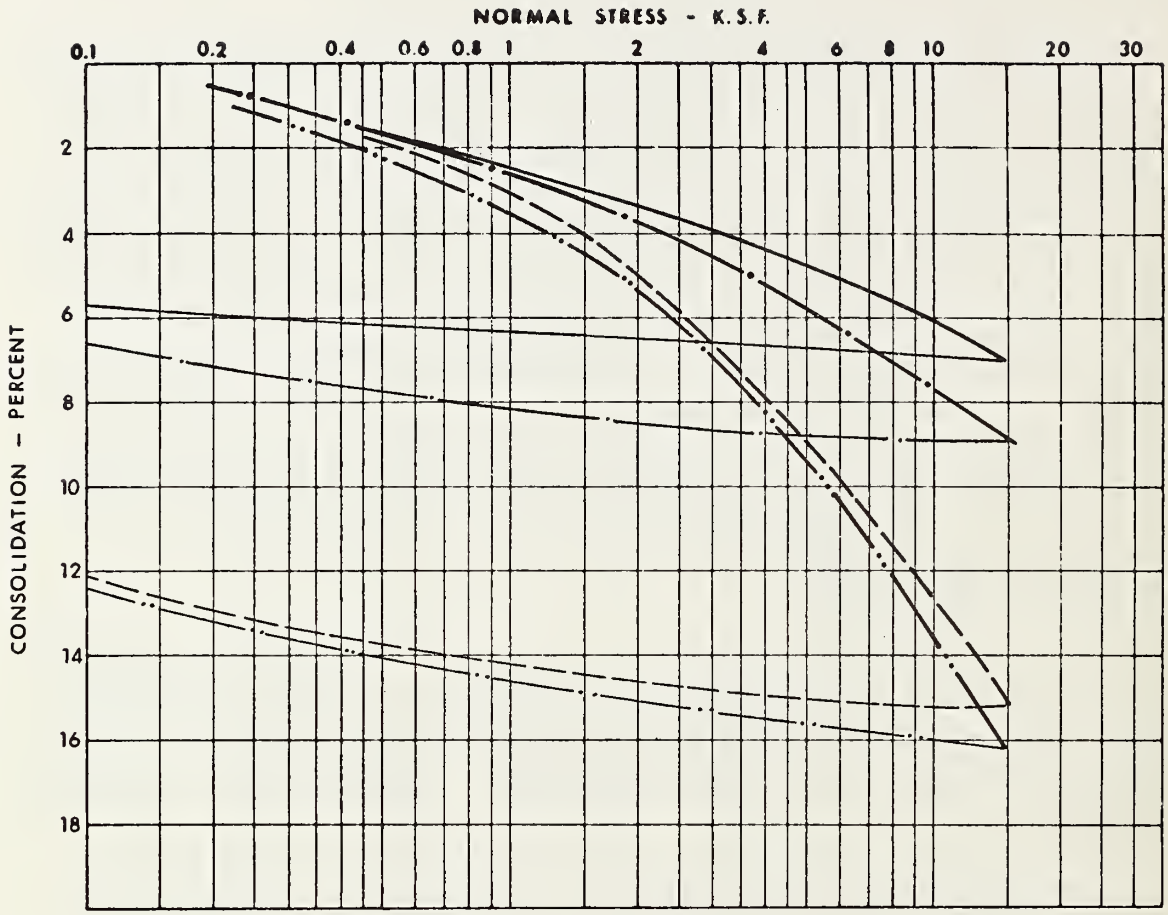
The 1975 subsurface investigation determined that the soft silts encountered beneath the primary clarifier underlay most of the proposed secondary treatment area as well. Application of Sanglerat's 12 kg/cm^2 rule-of-thumb for compressible strata indicated that most of the silt stratum at elevation +5 to +10 would be compressible. Some of the silt and sandy silt between elevation -10 and -20 mean sea level was also suspect. Application of Schmertmann's ($qc < Z/3$) rule-of-thumb also indicated that the deep, medium stiff to stiff silt above the gravels could be normally consolidated. Settlement calculations, as described in the following section, indicated significant plant settlement would occur at the site without some form of treatment. A site preload was therefore recommended and adopted. Post-construction settlement problems have not been detected at the plant in the first six months of its operation.

STRUCTURE SETTLEMENT

Structure settlement was estimated using a combination of consolidation theory and Schmertmann's static penetrometer method. Consolidation theory was used to estimate settlement of silts beneath the site. Schmertmann's method was used to estimate sand settlement. Plots of two consolidation tests for the upper silts are shown on Figure 10

The combined Schmertmann-consolidation methods were used to estimate the preload settlement at settlement plate 9 (SPL-9) as shown on Figure 11 The predicted settlement was 1.5 inches (3.8 cm), whereas the measured settlement at the end of 5 months was 0.8 inch (2.0 cm). Figures 11 and 12 show the analytical model and analysis methods used.

Examination of these figures shows most of the estimated settlement (1.0 inch (2.5 cm)) was due to consolidation of the silts. Based on the variable nature of the silt underlying the site and the fact that no probe or boring was located directly under SPL-9, we felt that the estimated and measured settlements for SPL-9 were in good general agreement.



LEGEND	TEST HOLE	DEPTH	INITIAL MOISTURE CONTENT	DRY DENSITY (PCF)	MATERIAL TYPE
—·—	3	10'	36.9%	83.4	SANDY SILT
———	4	5'	6.9%	80.2	SAND
—··—	3	29'	47.2%	70.4	SILT
– – – –	2	5'	44.0%	80.0	SILT

Note: 1 ksf = 47.9 kPa
1 ft = 0.305 m

Figure 10. Case history 1 consolidation test results

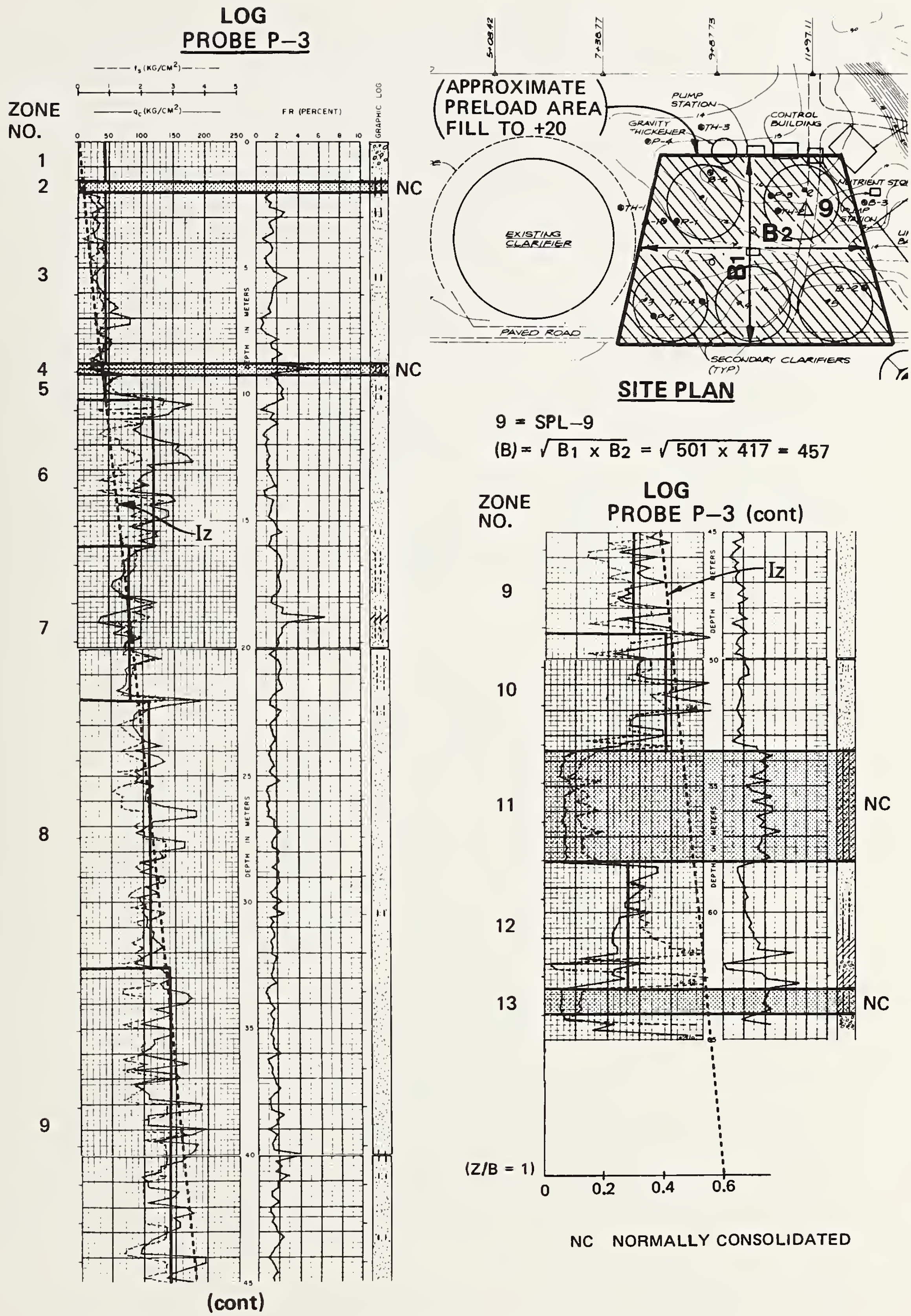


Figure 11. Case history 1 settlement model SPL-9

Assumptions:

1. Soil profile divisible into 13 zones
2. Silt zones with q_c < 12 or < Z/3 normally consolidated (N.C.) with $C_c = 0.15$
3. Schmertmann's method applicable for determining sand settlement
4. Average in situ and preload soil unit weight = 105 pcf
5. Consolidation complete within 5 months
6. Preload height = (+20) - (+16) = 4 ft @ SPL-9
7. Average GWT 3 m below grade
8. Neglect gravel settlement below 64.2 m
9. Preload width B_{avg} appropriate for estimating settlements
10. Stress increase in silts may be determined using Boussinesq stress-influence charts

Silt Consolidation Calculations $\left(\Delta h = \left(\frac{C_c}{1+C_o}\right) h \log \left(\frac{Po+\Delta P}{Po}\right)\right)$

Zone	$\left(\frac{C_c}{1+C_o}\right)$	h(cm)	Po(psf)	ΔPpsf	Δh(cm)
2	0.15	30	551	420	1.107
4	0.15	30	1,888	378	0.357
11	0.15	440	8,489	252	0.839
13	0.15	120	9,564	252	0.203

Δ_{consol} = 2.5 cm

Sand Settlement Calculations $\left(\Delta h = C_1 C_2 \; \Delta P \; \Sigma\left(\frac{Ez}{Es}\right) \Delta z\right)$

$C_1 = 1 - 0.5 \left(\frac{0}{\Delta P}\right) = 1, \; C_2 = 1+(0.2) \log \left(\frac{5/12}{0.1}\right) = 1.12$

Zone	q_c	Es	Δh	Iz	$\left(\frac{Iz}{Es}\right)$ Δh
1	40	80	150	-0-	-0-
3	40	80	680	0.04	0.34
5	50	100	100	0.07	0.07
6	120	240	580	0.11	0.27
7	80	160	600	0.15	0.56
8	110	220	1,060	0.23	1.11
9	140	280	1,640	0.35	2.05
10	190	380	460	0.43	0.52
12	135	270	500	0.51	0.94
				Σ =	5.86

Δ_{sand} = 1.3 cm

∴ Estimated settlement = 3.8 cm = 1.5"
∴ Measured SPL-9 settlement = 0.8"

Note: 1 pcf = 0.157 kN/m^3; 1 ft = 0.305m

Figure 12. Case history 1 settlement model SPL-9 calculations

Chapter 5

CASE HISTORY 2
VIBROFLOTATION DENSIFICATION MONITORING, NEWPORT, OREGON

PROJECT DESCRIPTION

In September of 1974, we issued a report to Northwest Natural Gas Company (NNGC) describing the geotechnical investigation performed for the proposed NNGG Newport Liquified Natural Gas (LNG) Receiving Terminal Facilities. The LNG complex will occupy a 20-acre site on Yaquina Bay, just east of Newport, Oregon. The complex will include a 600-foot (183-m) wharf, a 175-foot (53.4-m) O.D., above-ground, double-wall LNG storage tank, an earth containment dike for the tank, and several operation support buildings. Figure 13 is a site plan for the complex as originally proposed. The subsurface investigation, described below, was initiated prior to construction at the site.

SUBSURFACE INVESTIGATION

A total of 29 soil test borings were completed for the subject investigation. The boring locations are shown on Figure 13. The borings were advanced using a CME 55 drill rig. Steel casing was used during drilling of the offshore borings. Bentonite drilling mud was circulated in the borings to remove cuttings and prevent the holes from caving. Soil samples were attempted in all borings at minimum 5-foot (1.5-m) intervals. Disturbed soil samples were obtained using a Standard Penetration Test (ASTM D1586) split barrel sampler. Undisturbed samples were obtained using a 3-inch (7.6-cm) O.D. Osterberg piston sampler.

Although not encountered at the same elevation in each of the borings, four distinct subsurface zones were defined. They were, proceeding from the surface: (1) a medium to fine sand zone, (2) a silty sand-sandy silt zone, (3) a compressible clayey silt zone, and (4) a well indurated siltstone. The siltstone was representative of the Nye Mudstone. Figure 5-2 is an inferred cross section through the site showing selected boring logs. The upper 10 to 30 feet (3.1 to 9.1 m) of material at the site was dredged material from Yaquina Bay.

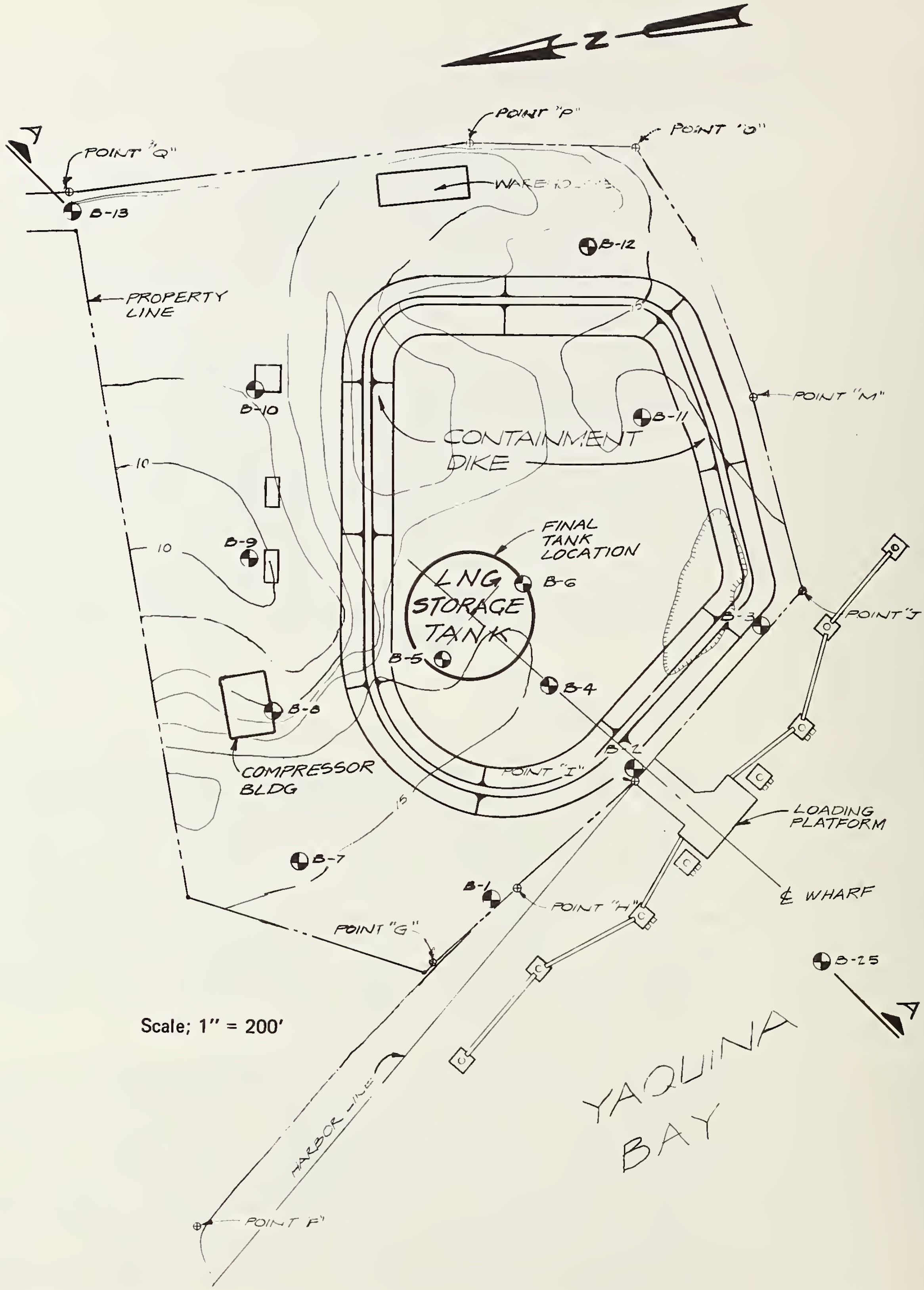


Figure 2, Case history 2 site plan and boring locations

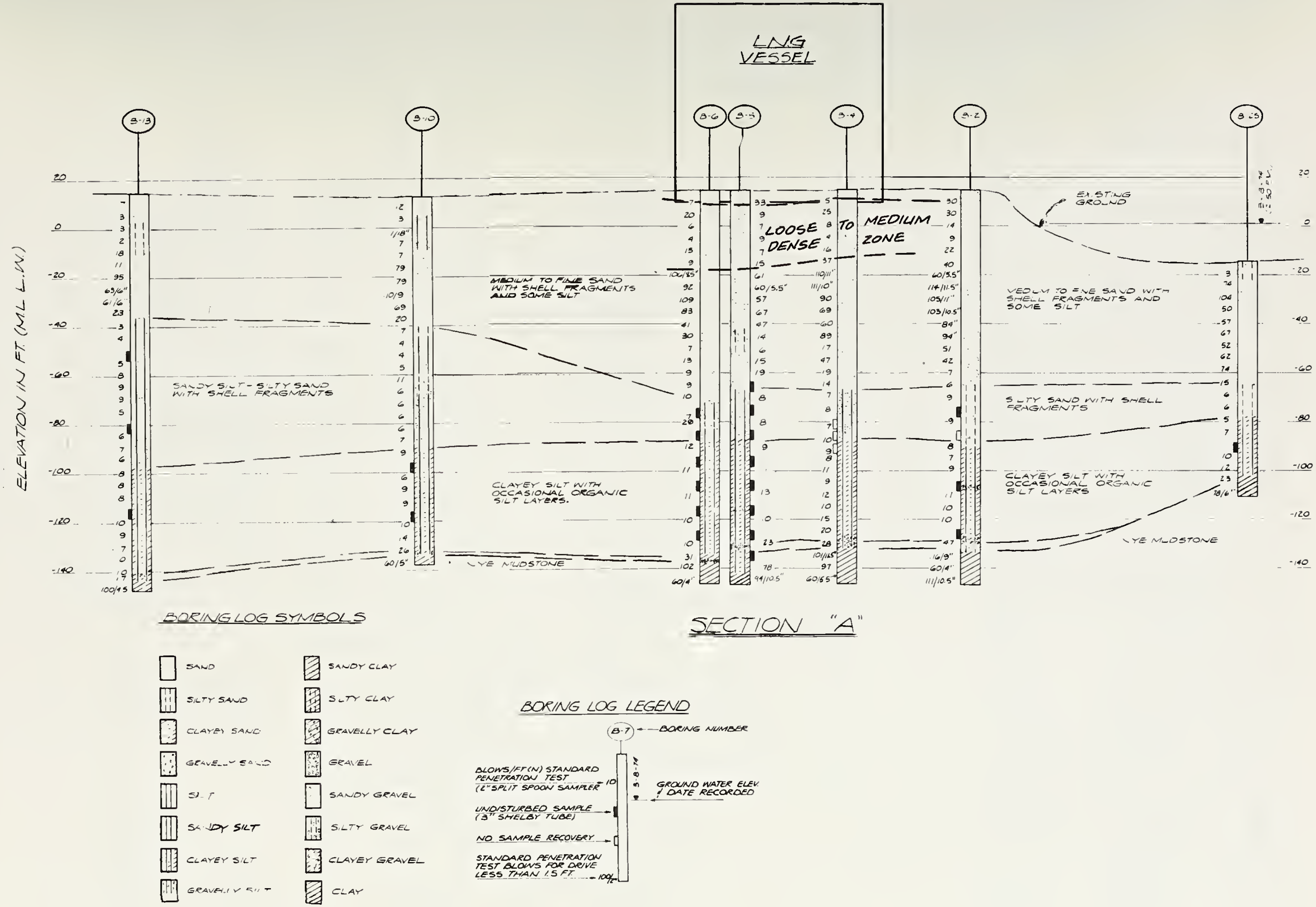


Figure 14. Case hsitory 2 subsurface profile

FOUNDATION DESIGN

Beneath the LNG tank foundation, two zones were determined to require remedial treatment prior to constructing the tank. The two zones were the deep clayey silt and the uppermost 30 feet (9.1 m) loose to medium dense zone within the medium to fine sands.

Consolidation tests performed on selected samples of the deep clayey silt indicated the material was normally consolidated. A sand preload was employed to preconsolidate the site, thereby reducing anticipated LNG tank settlement to acceptable limits.

The second zone, shown on Figure 14, was determined to require treatment for the following reasons:

1. The material could not provide the bearing capacity required for the tank.

2. The loose condition of the material made it susceptible to possible liquifaction during seismic activity.

The liquifaction potential of the upper sands was evaluated using the procedure presented by Seed and Idriss [15] and as shown on Figure 15 In situ densification was recommended for remedying liquefaction problems associated with the upper sands.

VIBROFLOTATION PROGRAM

At the request of NNGC, we investigated and compared in-place densification techniques early in June 1975. Logs of borings B-4, B-5, and B-6 were sent to several contractors, along with the gradation limits shown in Table 2.

Table 2. Medium fine sand gradation limits

Sample Number	Elevation (MLLW)	Percent Passing by Weight U.S. Standard Series Sieve Size #8	#16	#30	#50	#100	#200
B-5, SS-9	-30	100	98	93	67	8	4
B-6, SS-3	-1	100	100	98	71	21	2
B-6, SS-4	-6	100	100	99	98	28	12
B-6, SS-7	-21	100	98	98	77	10	4

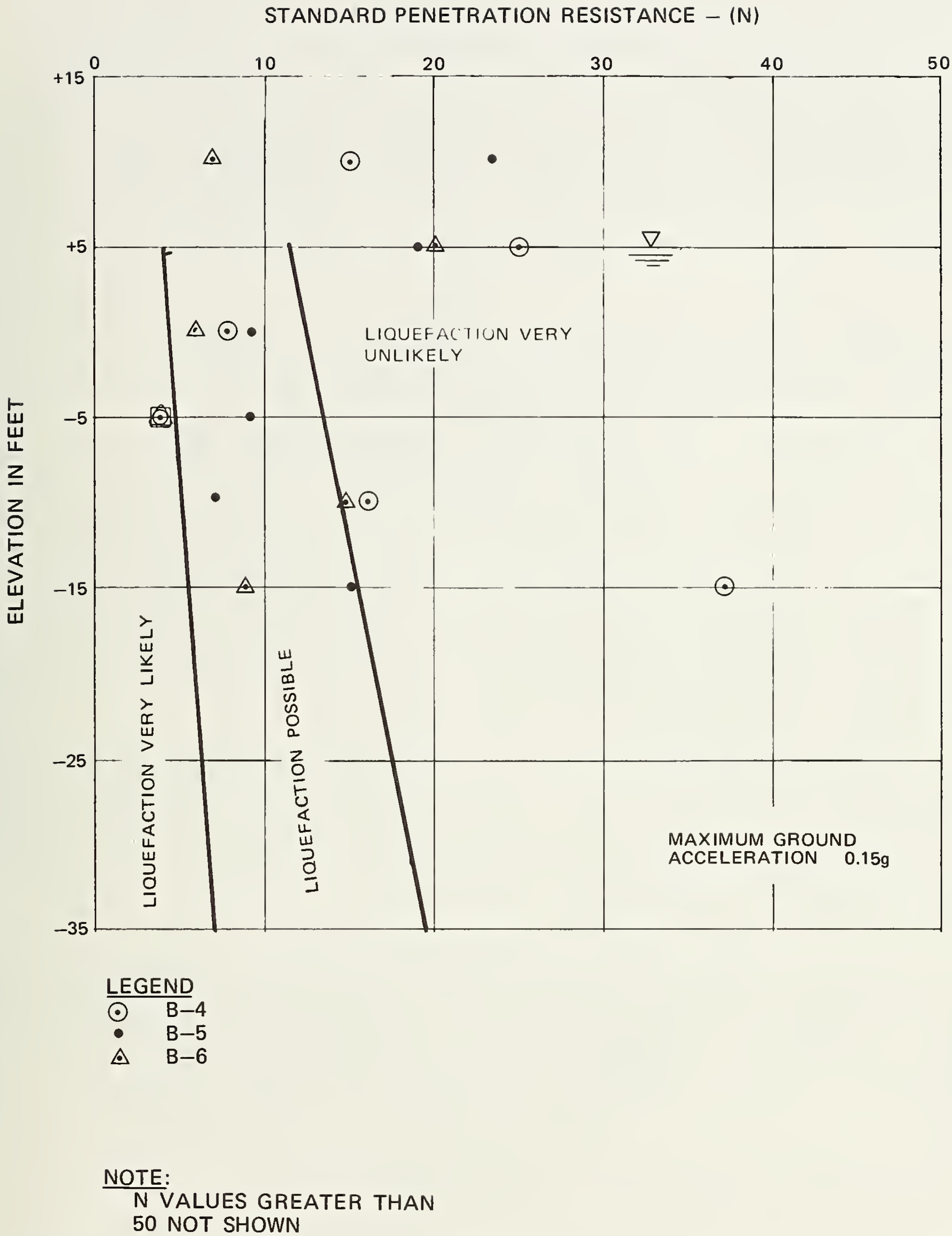


Figure 15. Case history 3 untreated foundation liquifaction potential

All of the upper sand samples visually appeared to be similar, and the four tests tabulated showed very similar gradation limits. The procedure known as "vibroflotation" was judged the most satisfactory. This process employs a probing device called a vibroflot. The vibroflot machine compacts by simultaneous vibration and saturation to move, shake, and "float" soil particles into a dense state. Since the compacted in situ soil occupies a smaller volume, a crater is formed around the vibroflot, and granular backfill material is added from ground surface to compensate for this void.

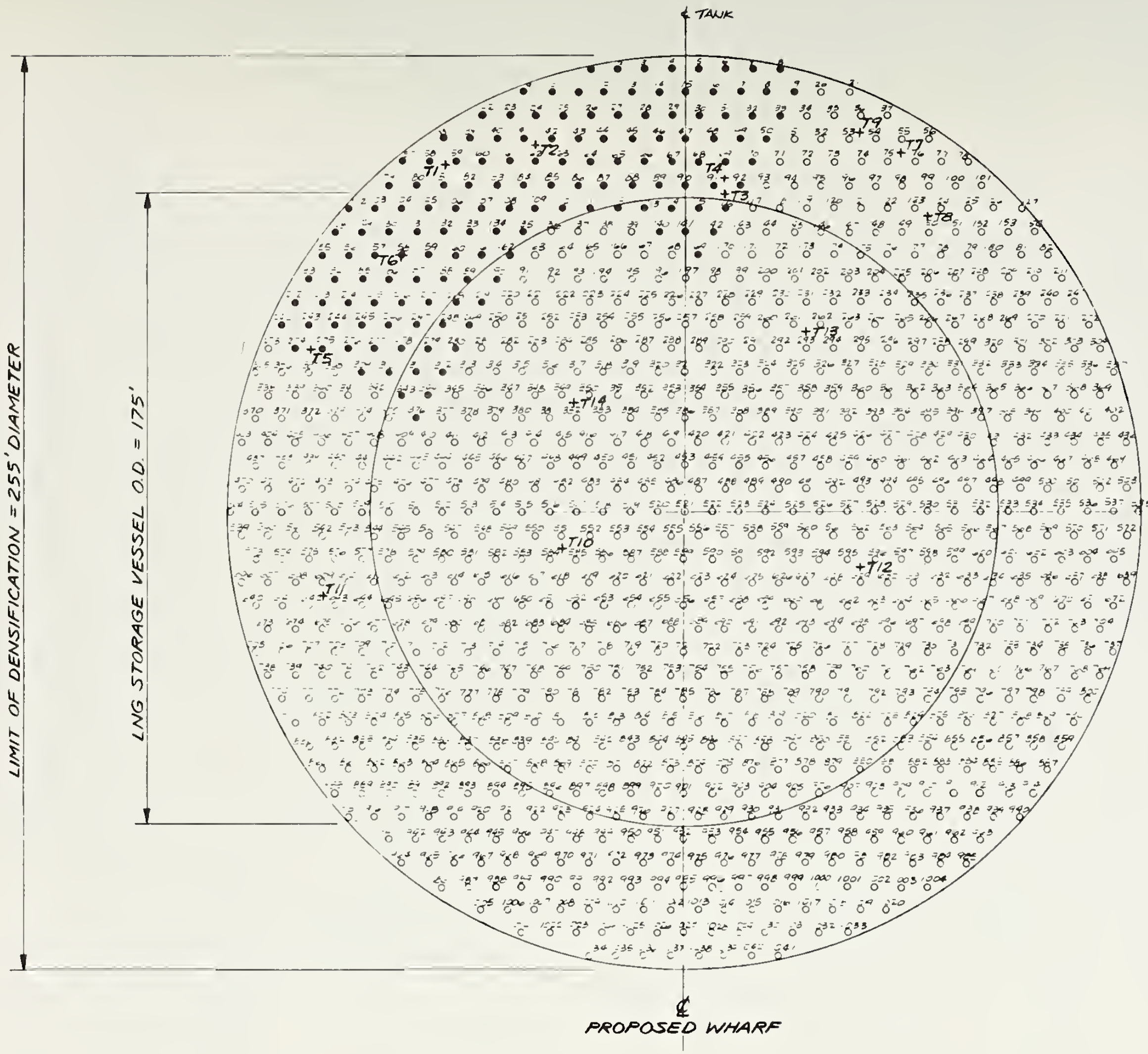
Following our recommendation, NNGC awarded a contract to Vibroflotation Foundation Company (VFC), Pittsburgh, Pennsylvania, the sole United States proprietor for the vibroflotation process, to densify the soil at the LNG tank site. This contract called for a minimum relative density of 70 percent between elevations +12 and -20 (mean low or low water data) to a distance 45 feet (13.7 m) beyond the outside periphery of the tank. The contract stipulated that measurement of the in situ densification was to be done using the static penetrometer. The tank foundation is at elevation +12 (MLLW), so the soil above this level will be disturbed during construction. Field data (Standard Penetration Tests) indicated in situ relative densities using the Gibbs and Holtz [16] criteria already exceeded 70 percent below elevation -20 (MLLW).

Vibroflotation Foundation Company proposed a compaction pattern layout based on a 6.5- by 7.5-foot (2.0-by 2.3-m) triangular grid over the entire 255-foot (77.7 m) (175-foot (53.3-m) outside tank diameter plus 40 feet (12.2 m) each side) diameter area. This required 1,041 separate probes, each 32 feet (9.7 m) deep.

Subsequently, NNGC decided to also densify the upper 15 to 20 feet (4.6 to 6.1 m) of soil at the site of the 60- by 80-foot (18.3- by 24.4-m) compressor building. Using the same triangular grid pattern, 127 separate probes were required for a compacted depth of 20 feet (6.1 m). Figure 16 presents the compaction grid for the storage tank area.

CONSTRUCTION PROCEDURE

The densification work was begun 21 August 1975, using onsite sand to backfill the probes. The initial procedure was fairly standard, and may be described as follows.



LEGEND:

° COMPACTION PROBE - PROCEDURE PRODUCED ACCEPTABLE DENSIFICATION

• COMPACTION PROBE - PROCEDURE PRODUCED UNACCEPTABLE DENSIFICATION; REPROBING REQUIRED

+ STATIC PENETROMETER TEST LOCATION

NOTES:

TOTAL NUMBER OF PROBES = 1041
COMPACTION DEPTH = 32'((+)12→(-)20)

1 ft = 0.305 m

Figure 16. Case history 2 LNG storage tank compaction pattern

1. The vibroflot, a long, slender tube consisting of two parts--the vibrator and follow-up pipe, was suspended vertically from the boom of a crane and positioned over a probe location. (The vibrator is the source of vibratory energy. Its upper compartment houses an electric motor which drives an eccentric located in the lower compartment, developing 20,000 pounds of centrifugal force.)

2. With the motor at full speed, upper and lower jets on the vibroflot were opened full, and the machine was allowed to sink into the soil under its own weight. Water from the lower jets creates a "quick" condition ahead of the vibroflot, permitting almost unimpeded penetration. Penetrating to elevation -20 (MLLW) took 3 to 5 minutes.

3. When the vibroflot reached the required depth of compaction, the entire unit was hoisted from the hole and immediately allowed to free fall back to the bottom. Using this procedure, the hole was purged of debris and clumps of silt.

4. After purging, the water supply to the lower jets was shut off and the pressure reduced to the upper jets. This provides a downward flow of water to aid in compaction and facilitates the continuous feed of backfill from ground surface.

5. The machine was allowed to operate at the base of the hole until the power input to the motor, which has been correlated to soil density, indicated sufficient compaction. By then raising the unit step by step--in lifts of 1 to 3 feet (0.3 to 0.9 m)--and simultaneously backfilling with sand, the entire depth of soil was densified. (The power input to the electric motor is measured by a recording ammeter. As compaction proceeds, the resistance to movement about the vibrator increases until the ammeter "peaks" at the maximum input. The vibroflot is then raised one lift, the power input drops off, and the cycle is repeated.)

FIELD VERIFICATION

Approximately 100 probes had been completed by 22 August 1975 using the above procedure. On this date, a series of six static penetrometer field tests was begun to examine the degree of compaction. Testing was carried out under our supervision. Using this device, data readings were obtained at depth intervals of 8 inches

(20 cm). Data readings were reduced to relative density measurements via use of the cone penetrometer and calibration chart described in Chapter 3. Test locations are shown on Figure 16 With one exception, as discussed below, the tests were near the centroids of the triangular probe patterns--the areas of minimum compaction.

Relative density results of the initial six tests (T1 through T6) are presented on Figure 17. In general, acceptable compaction had been achieved between elevations +12 and -1, and below -17 (MLLW). From elevation -1 to -17 (MLLW), relative density varied from 30 to 60 percent, significantly less than the specified 70 percent.

After the first two of these tests, VFC immediately implemented changes in the densification procedure, substituting crushed rock backfill for the onsite sand.

Twenty-eight compactions were carried out using three different sizes of rock: 3-inch minus, 1-1/2-inch minus, and 3/4-inch minus. All three materials contained fines which inhibited the feeding procedure. Subsequent density tests (T3 and T4) showed no significant improvement over the results of T1 and T2. Another variation on the method was to decrease the withdrawal rate of the vibroflot from 2 feet to 1 foot per minute (0.01 to 0.005 m/sec). It was hoped that transmitting energy to the soil for a longer period of time would increase the compaction. When tested, however, the results (T5) were much the same as for tests 1 through 4.

Penetrometer test T6 was near the edge of a single compaction rather than the pattern centroid, as previously discussed, to see if good densification was, occurring at the probe locations (where compactive effort was greatest). Test data showed the same general pattern as for previous tests, indicating poor compaction in the lower 16-foot (4.9-m) zone.

On 3 September, VFC again altered the procedure. Clean 1-1/2- to 3/4-inch crushed rock was used for backfilling the probes below elevation zero, requiring 8 to 9 cubic yards (6.1 to 6.9 m^3) of rock per probe. Onsite sand was used above this elevation, where test results showed satisfactory densification.

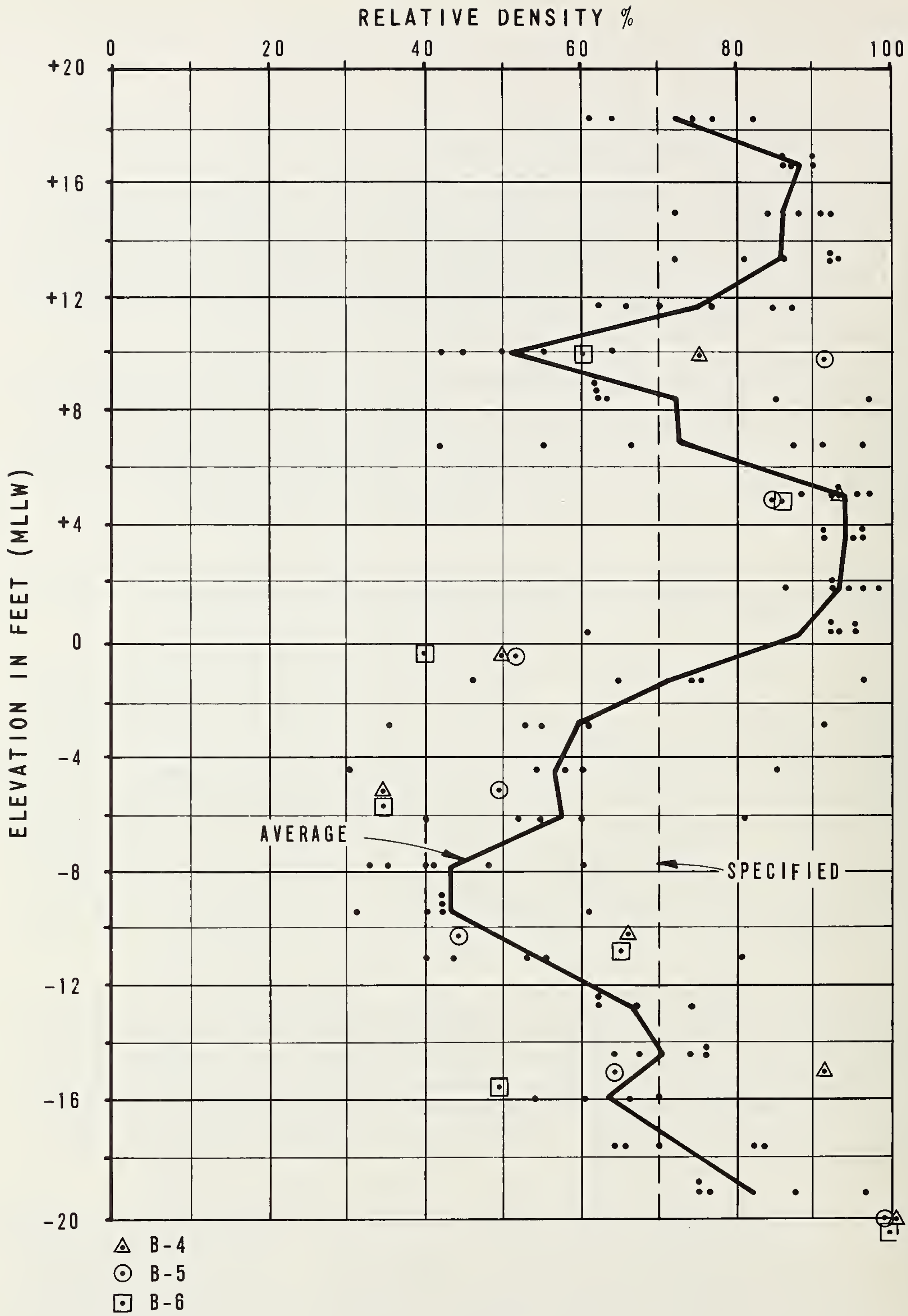


Figure 17. Case history 2 field density test results: T1-T6

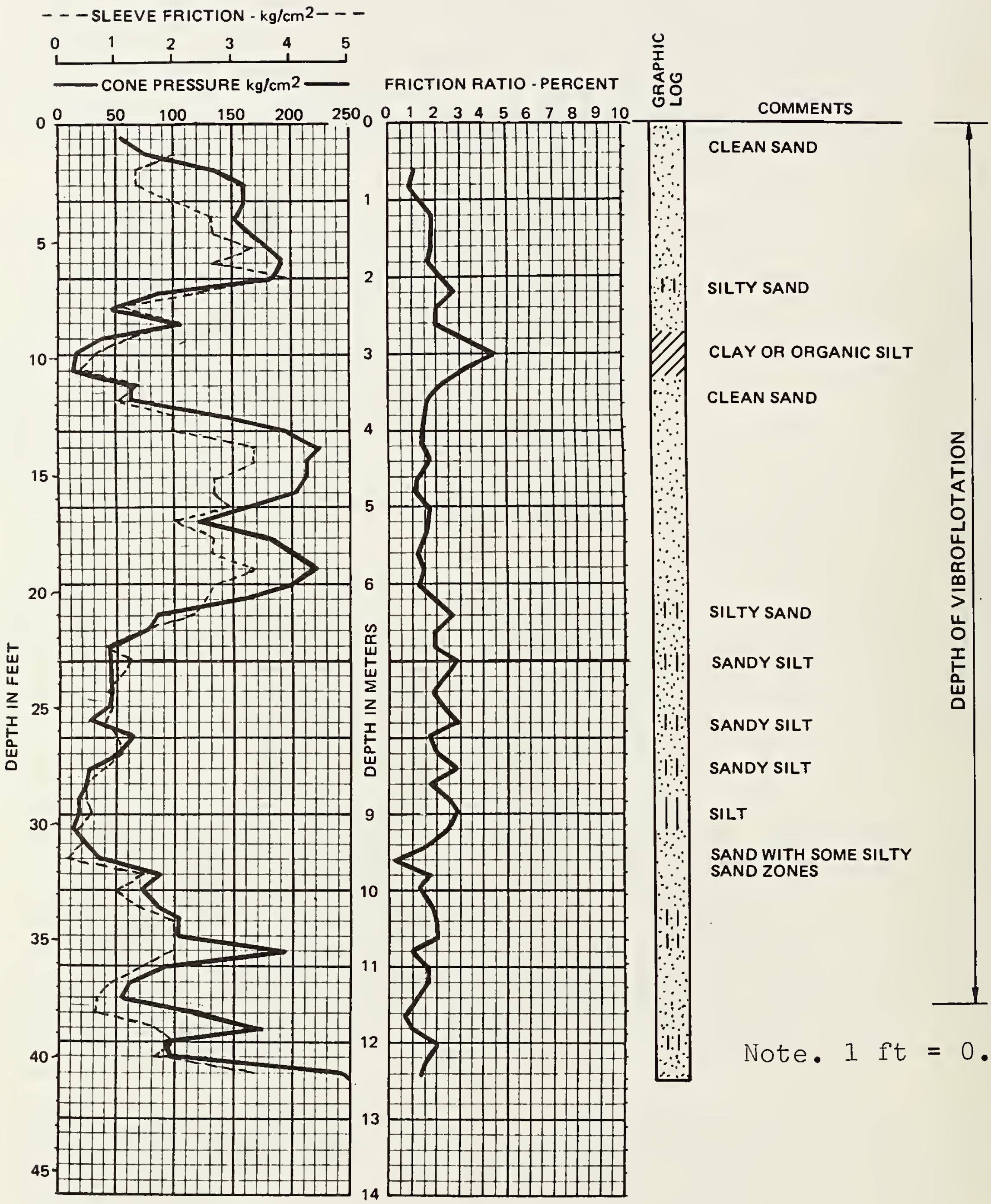
The overall compaction procedure was basically unchanged. The chief modification involved feeding the special rock backfill--rather than sand--into the probe after cutting the water supply to the lower jets. Compaction then proceeded in 1-foot (0.3-m) lifts of crushed rock to about elevation zero, leaving a column of rock in the lower half of each probe. Sand backfill in the upper portion of the probe was also compacted in 1-foot (0.3-m) lifts, allowing the vibroflot to operate at each level for at least 30 seconds.

On 8 September, a second series of density tests was undertaken to evaluate the revised procedure. About 40 separate compactions had been completed with the revised procedure. Locations of the three tests--T7, T8, and T9--are shown on Figure 14 ; results are presented on Figure 19 Relative density still fell below the specified 70 percent at lower depths (roughly between elevations -1 and -12) (MLLW), but the size of this zone was reduced, and average density values were somewhat greater over the entire depth. Use of the clean rock backfill did improve compaction, but still left a 10- to 11-foot (3.0 to 3.4 m) zone not meeting specifications.

Failure to produce desired results with this latest method prompted speculation that soil conditions may differ from those indicated by borings. Small chunks of silt had been washing from the holes during purging, and static penetrometer data had indicated the presence of intermittent lenses of silt and sandy silt at all test locations.

Initially, little attention was paid to the indications of silt layers. The penetrometer showed them to exist throughout the depth of vibroflot penetration, but compaction was difficult for only a portion of this depth. Figure 18 is a typical penetrometer log for the zone requiring densification. Closer examination of the penetrometer test results indicated more of the silt seams at elevations where densification was a problem. The layers were much the same thickness (generally 2 to 12 inches (1 to 30.5 cm), occasionally as much as 18) as at higher elevations, but their occurrence was more frequent in this zone.

These silt seams were not detected during the 1974 geotechnical investigation. Because the layers are intermittent and narrow, recovery of a 6- to 12-inch (15.25- to 30.5-cm) sample at intervals of 5 feet (1.5 m) did not reveal their presence. No samples were obtained within depths of 50 feet (15 m) (approximate elevation -35) at the tank site with more than 12 percent (by weight) silt-clay



Note. 1 ft = 0.305 m

Figure 18. Case history 2 static penetrometer log

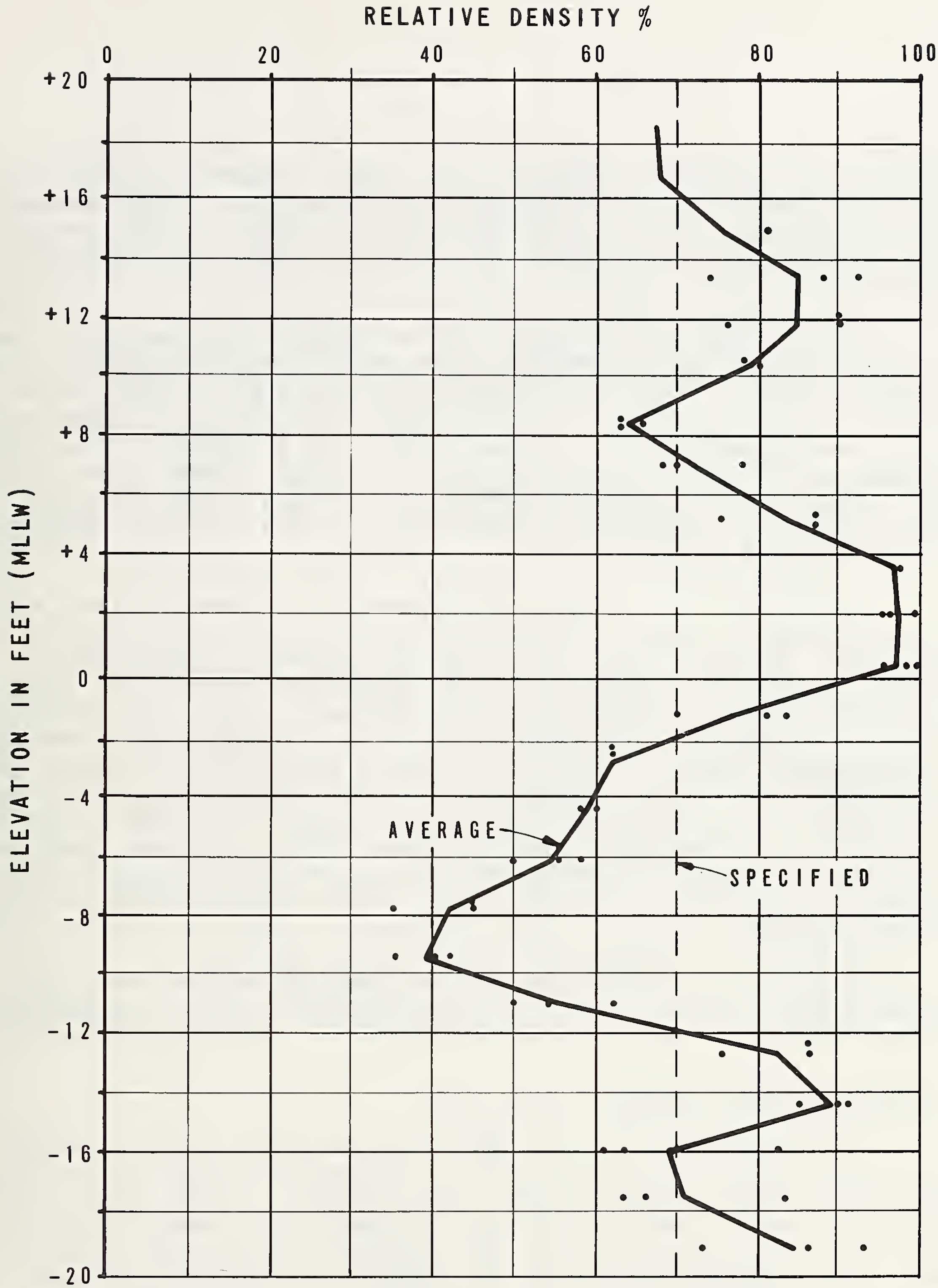


Figure 19. Case history 2 field density test results: T7, T8, T9

content as shown by Table 2. Since vibroflotation is generally effective in granular soils having a combined silt and clay content less than 25 percent, the soil appeared to be well within the range that can be compacted by this process.

To further investigate the now-apparent layering, two auger borings were advanced after the second series of density tests. The holes were drilled with a solid-stem auger, and continually caved. Only two samples could be retrieved from the zone of interest. Both samples consisted primarily of medium to fine sand with small amounts of silt. One, however, contained a 3-inch (7.6-cm) lens of sandy silt, providing some physical evidence of the layering shown by penetrometer data.

Vibrations have little effect on cohesive soils, so vibroflotation does not compact the silt itself. Conversely, however, the cohesive material significantly affects the vibrations from the vibroflot, which are transmitted to the surrounding soil medium. Layers of silt, clay, or organic material dampen these vibrational forces, minimizing vertical transmission and restricting horizontal transmission. Where layers occur more frequently, as below elevation zero, this effect is pronounced and vibratory compaction is substantially limited. In the upper reaches of the soil profile, where silt lenses are less prevalent, the effect appears less important and densification can be accomplished.

Figure 19 represents the extent to which the particular soil profile at the LNG tank site can be densified using in-place vibratory techniques.

DENSIFICATION PROCEDURE REVISIONS

Our recommendation to NNGC was that the revised compaction procedure, as previously described, was satisfying the purposes for which the work was intended, though it did not precisely meet contract specifications. VFC was instructed to complete the job using the approved altered procedure, and the results presented on Figure 19 became the new measure of performance by which the work would be judged. In addition, all probes that had been completed by procedures other than that finally approved would be reprobed by the accepted method. This involved a total of 145 probes, as indicated on Figure 16.

A third, and last, series of density tests was carried out October 1 and 2. Approximately 880 compactions had been completed at the time, including the 145 that had to be redone. Five static penetrometer tests (T10 through T14) were performed, located as shown on Figure 14. Results, as presented on Figure 20 closely matched those shown on Figure 19 even indicating slightly better compaction. The new work was thus deemed acceptable and no further verification of the work with the penetrometer was required because a full time resident engineer was assigned to observe that the accepted procedure was followed.

The standard vibroflotation procedure--backfilling the probes with onsite sand--was used at the compressor building. Since Chicago Bridge & Iron requested densification only to a depth of 15 to 20 feet (4.6 to 6.1 m) beneath the structure, penetration of the "problem zone" was minimal, and rock backfill was not needed.

The densification work at the site of the LNG storage vessel has been completed to the intent of the specifications, that is, to increase the bearing capacity and to decrease the likelihood of foundation liquefaction. The static cone penetrometer was a very useful tool for measuring the in situ densities. The penetrometer readings were obtained rapidly and just as rapidly converted to values of relative density. The penetrometer detected the thin silt lenses described that had not been detected in the initial investigation, which had used interval sampling. The knowledge of the silt presence led to both a hypothesis as to why the densities were not being improved by the vibroflotation process and an amended procedure in which clean, crushed rock was substituted for onsite sand in the vibroflotation process.

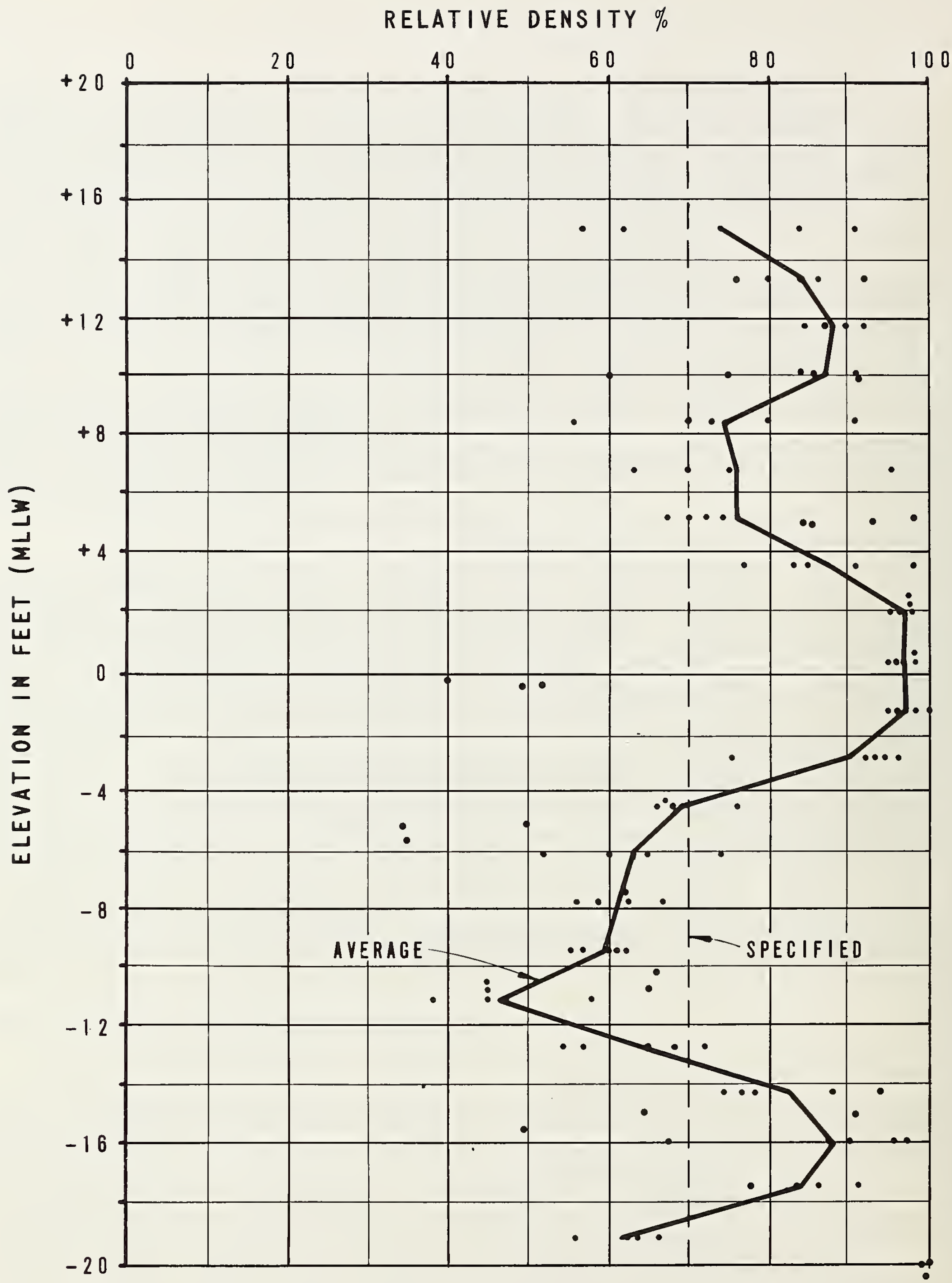


Figure 20. Case history 2 field density test results: T10-T14

Chapter 6

CASE HISTORY 3 - SAWMILL AND CHIPPING FACILITY ABERDEEN, WASHINGTON

PROJECT DESCRIPTION

Boise Cascade Corporation proposed building a sawmill and chipping facility near Aberdeen, Washington. The facility would include a 24,000 square foot (2,230 m^2) building. The proposed site is used by Boise Cascade as a log storage and scaling yard. The site is flat, approximately 11 feet above mean sea level. The town of Aberdeen fronts on Grays Harbor, midway down the Washington State coast. The project location is shown on Figure 21.

Three static penetrometer probes and one rotary-drilled test boring were used to investigate subsurface conditions at the proposed site. The investigation identified subsurface soil types and a pile-bearing stratum approximately 100 feet (30.5 m) deep.

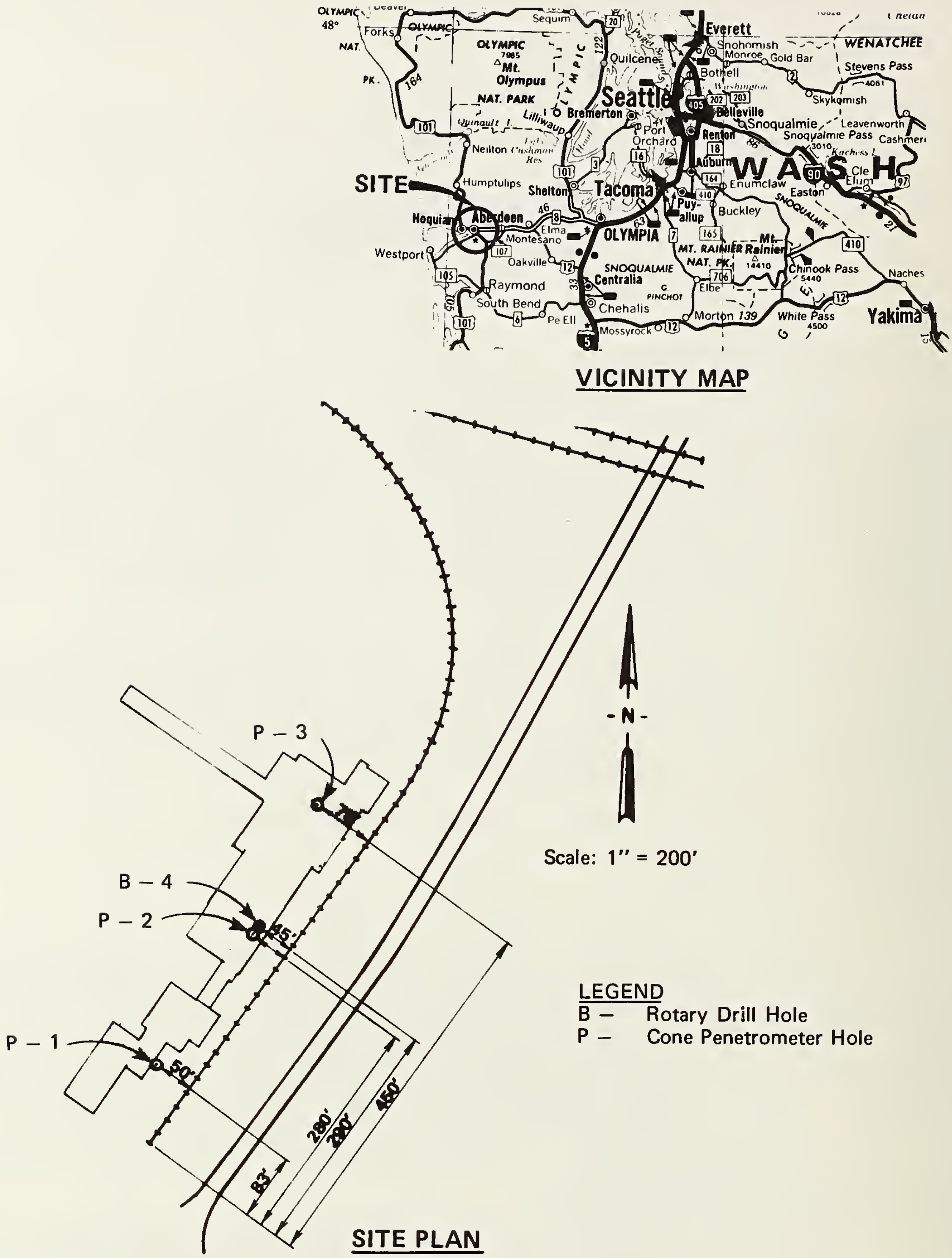
The facility has not been constructed as of this date (1978).

AREA GEOLOGY

Grays Harbor is a shallow coastal estuary opening to the Pacific Ocean. The harbor was formed by a downwarping (basining) of coastal sandstones and siltstones in the Miocene age. Following the downwarping, the basin was partially filled with sands and gravels of Pleistocene Age. Recent silts, sands, and clays overly the Pleistocene sediments. The recent alluvium was deposited by the Chehalis River and other small streams that empty into Grays Harbor.

SUBSURFACE INVESTIGATION

Subsurface conditions at the site were investigated in May 1975. Locations of the three penetrometer probes and test boring are shown on Figure 21. The three penetrometer logs were very similar. Figure 22 shows the log of probe 2. Figure 23 shows the graphic logs of the probes and soil boring.



NOTES:

1. Location of proposed saw mill taken from Boise Cascade Corp. SAW MILL PLOT PLAN NO. 3 dated 21 March 1975.
2. P – 2 and B – 4 Located ten (10) feet apart.

Figure 21. Case history 3 site plan and location of test holes

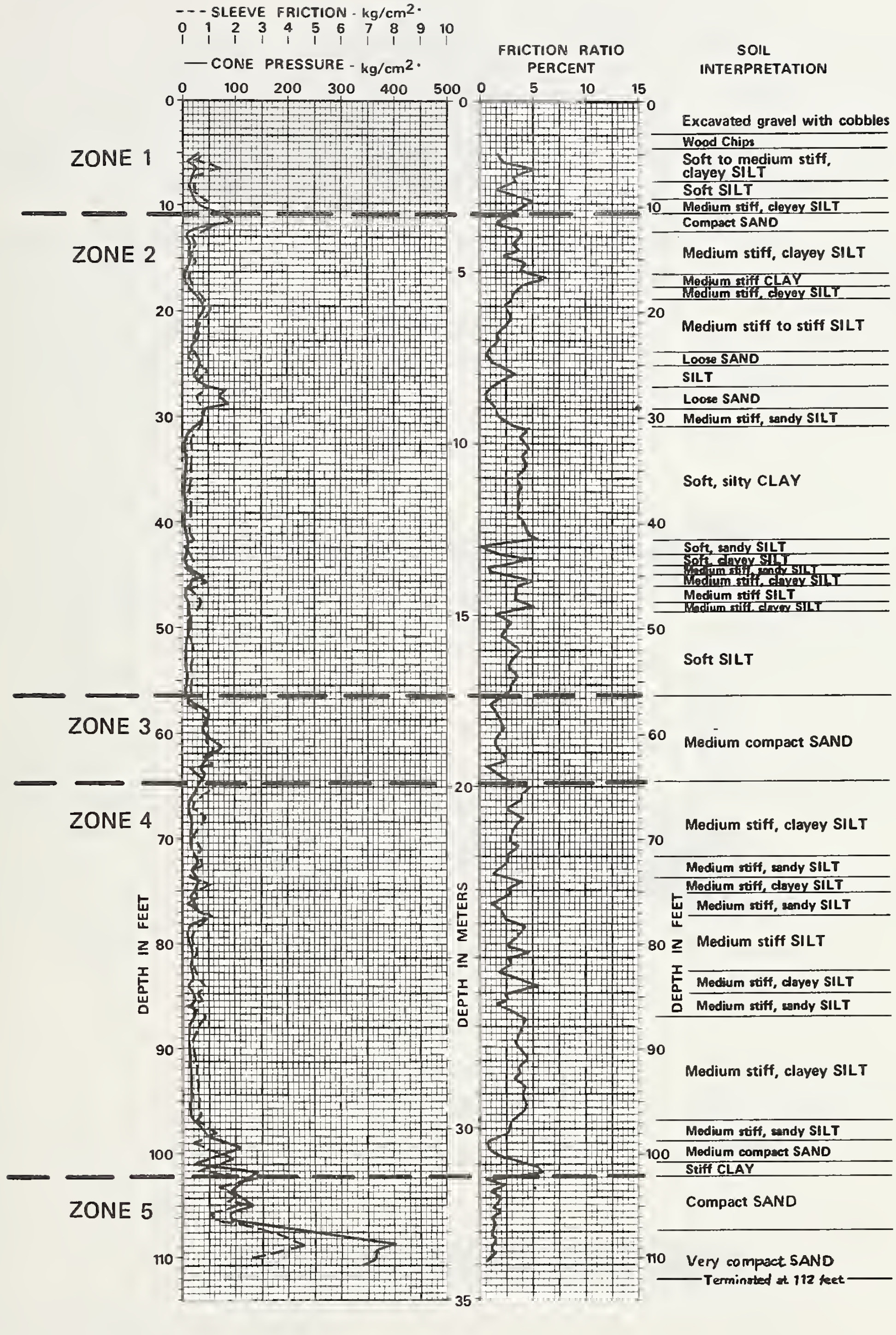


Figure 22. Case history 3 static penetrometer log hole P-2

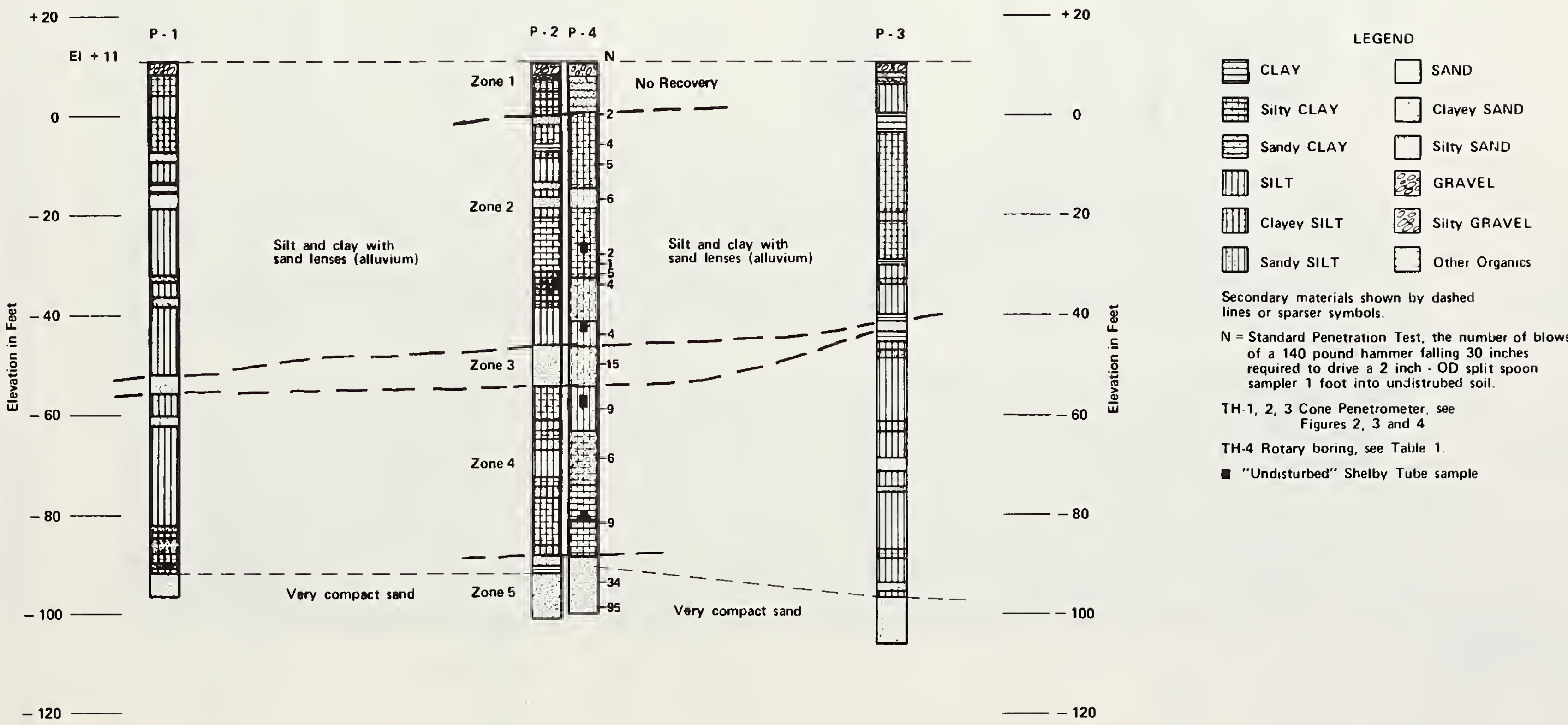


Figure 23. Case history 3 subsurface profile

Test Boring

The test boring was completed using a CME 55 rotary drill rig. Undisturbed soil samples were obtained using 3-inch (7.6 cm) O.D. Shelby tube samplers. Disturbed samples were obtained using a standard penetration test (SPT) sampler. SPT blowcounts (N) versus depth are shown on Figure 3. No correction was applied to the N value.

The sampling interval in the test boring was approximately 5 feet (1.5 m) to a depth of 50 feet (15 m); thereafter, the interval was roughly 10 feet (3 m). Soil samples were visually classified in the field. No laboratory tests were performed on the soil samples. Below a depth of about 50 feet (15 m) (elevation -40 feet) the contacts between material type were based on changes in the drilling; hence, changes in strata are approximate.

Static Penetrometer

A Begemann sleeve penetrometer was used for the penetrometer probes. Cone and sleeve readings were taken at 8-inch (20 cm) intervals. The complete P-2 log is shown on Figure 6-2. Probe P-2 and boring B-4 were roughly 10 feet (3.0 m) apart. Since this was one of the first uses of the penetrometer by our firm our initial attempt was to distinguish each material change, even if that change was indicated by only one peak on the friction ratio curve. The soil interpretation of P-2 would likely appear much simpler if performed today.

Static penetrometer and test boring completion costs are shown in Table 3.

Table 3. Aberdeen subsurface investigation unit costs* (1975 dollars)

	Excluding Mobilization	Including Mobilization
Static Penetrometer	$3.00 per foot	$3.76
Rotary Drilling	6.14	8.65

Inspection costs not included.

SOIL PROFILE

The subsurface soils encountered are consistent with the local and regional geologic history. The top 3 to 5 feet (0.9 to 1.5 m) of material consist of gravel and wood chip fill. The upper fill is underlain by approximately 100 feet (30.5 m) of loose to medium sand and soft silt and clay. The upper 100 feet (30.5 m) of material are representative of recent sediments in Grays Harbor. Horizontal and vertical composition varies in the recent sediments. At 105 to 110 feet (32.0 to 33.5 m) deep, a dense to very dense sand stratum was encountered. The deep sand is believed to be of Pleistocene age. It was encountered at a fairly consistent depth in each of the subsurface probes. Ground water was measured 3 feet (0.9 m) below the surface of the test boring and is expected to vary with the local tides.

Because it was a preliminary study, the scope of the investigation was limited. As mentioned, no laboratory tests were performed. Boise Cascade was of the opinion that pile foundations would be required and thus was primarily interested in locating a bearing stratum.

SOIL CLASSIFICATION

Because of the proximity of P-2 and B-4, comparisons of subsurface interpretation between the static penetrometer and the test boring are possible. Both interpretations show a complex profile in the upper 100 feet (30.5 m). In spite of their complexity, they also show many similarities. The five zones outlined on Figures 22 and 23 were selected for comparison. These zones were selected specifically for this penetrometer report. The following similarities were noted:

- Zone 1: Surface gravels underlain by mixed sands and organic material.
- Zone 2: Soft silts and clays with some sands and/or silty sands. SPT "N" average is about 3.5. Penetrometer qc averages less than 10 kg/cm^2, sleeve friction (fs) averages about 0.4 kg/cm^2.
- Zone 3: Medium dense sand. SPT "N" averages 15. Penetrometer qc averages 50 kg/cm^2, fs averages 1 kg/cm^2. The friction ratio is about 2 to 2.5, indicating some silt.

- Zone 4: Medium stiff silts and clays with occasional sandy areas. SPT "N" averages 8. Penetrometer qc averages 15 kg/cm^2, fs averages 0.5 kg/cm^2.

- Zone 5: Dense sands grading to very dense sands. SPT "N" jumps from 34 to 95. Penetrometer jumps from 100 kg/cm^2 to over 350 kg/cm^2. Friction ratio is 2 or less.

In addition to similarities in material types, there is very good agreement of the zone contact depths. Were this preliminary analysis to be repeated today, the scale of resolution on the logs would likely correspond to the five zones described.

A comparison of in situ strength and density measurements can be made from available study data. Torvane strength measurements were performed in the field on each of the four Shelby tube samples. At the same sample depth, one can compare the sleeve friction and point resistance estimates for cohesion (C) described in Chapter 3 with the torvane readings. The strength comparisons are shown in Table 4.

Generally poor agreement exists among the various estimates of C shown in Table 4. Columns (6) and (8) agree within about 35 percent, perhaps sufficient for a preliminary analysis. Column (9) is consistently higher than (8), however, which suggests that the poor correlation is a result of Ø-C strength equation for the subsurface soils. Extrapolation of the C approximation equations based on the Ø=0 assumption is apparently not suited for this site.

Estimated relative densities in the Zone 3 and 5 sands are shown in Table 5. Estimates are based on data from P-2 and B-4 only. The B-4 density is estimated using the method of Gibbs and Holtz for partially wetted sands [17]. The P-2 density is estimated using the penetrometer calibration chart described in Chapter 3. Good agreement is evident from the data shown in Table 5.

Table 4. Undrained strength parameters
Aberdeen subsurface investigation

(1) Sample Depth	(2) "N" Following	(3) Description	(4) Visual Unified Soil Classification	(5) Penetrometer Classification	(6) Torvane C (tsf)	(7) Following SPT "N" C Approx.	(8) fs
36'	2	Soft Plastic Silt	ML	CL	0.13	0.25	0.28
54'	4	Soft Sandy Silt	ML	ML	0.20	0.50	0.28
69'	9	Medium Sandy Silt	ML	ML	0.28	1.0	0.60
92'	9	Medium Silty Clay	CL	ML	0.19	1.0	0.48

Note: 1 ft = 0.305 m
1 tsf = 95.8 kPa

Table 5. Sand relative densities
Aberdeen subsurface investigation

Sand	Relative Density (%) SPT	Relative Density (%) Penetrometer
Zone 3	50	58
Zone 5 (upper)	68	72
Zone 5 (lower)	98	97

COMPRESSIBLE ZONES

Because of the organic content (wood chips, etc.) and the random nature of the surface fill (Zone 1), this material was judged unsuitable for foundation support.

Both the test boring and the probes indicated the upper 45 feet of recent alluvium (Zone 2) was weak and susceptible to consolidation settlement. The average Zone 2 N value was about 3.5, but several 1 and 2 blowcounts were recorded.

The low N values for Zone 2 indicate local, very soft consistencies. The penetrometer qc average for Zone 2 was 10 kg/cm^2, slightly less than the cutoff of 12 kg/cm^2 that Sanglerat [18] uses to distinguish weak foundation strata. Results of applying Schmertmann's [19] quick test for normal consolidation ($qc < Z/3$) also suggest the Zone 2 material below 30 feet (9.1 m) is subject to normal consolidation.

PILING

The sand 55 to 65 feet (16.8 to 19.8 m) deep (Zone 3) was judged inadequate for supporting end bearing piles. The stratum is thin as observed in P-1 and P-3, and therefore punching failure could be a problem. In addition, Zone 4 material below the sand could settle because of normal consolidation. Based solely on blowcounts, the clays and silts in Zone 4 should have a medium to stiff consistency. Results of applying Schmertmann's quick test, however, suggests normal consolidation. In addition, the Zone 4 average qc of 15 kg/cm^2 is not much greater than the Sanglerat cutoff of 12 kg/cm^2. Therefore, for a preliminary analysis, Zones 1 through 4 did not appear to be bearing strata.

A minimum penetration of 10 feet (3.0 m) into the Zone 5 sands was obtained with the exception of P-1. In P-1, a qc of over 400 kg/cm^2 was reached 8 feet (2.4 m) into the sands. Based on the geologic history of the area and the high resistance in the deep sand stratum, CH2M HILL was confident that a bearing stratum had been located in Zone 5 and deeper penetration was not required.

The above discussion outlines how the penetrometer and one test boring were used to interpret the subsurface profile. Our preliminary report recommended point bearing piles founded in the deep sand stratum. Since this was a preliminary report, a detailed analysis of specific pile capacities was not performed. With 10 to 20 feet (3.0 to 6.0 m) penetration below the Zone 5 contact, however, it was felt that steel H or pipe piles could develop up to 12,000 psi (845 kg/cm^2) on the net steel section. Prestressed concrete piles could also be considered. The 10- to 20-foot (3.0- to 6.0-m) penetration corresponds to the thickness of the dense stratum (approximately 5 feet (1.5 m)) plus an embedment into the very dense sands of 6 to 8 pile diameters (d). A 12 to 14-inch (30.5- to 35.6-cm) d was assumed. For H or pipe piles, the gross area, not the net steel area, would determine d.

Chapter 7

CASE HISTORY 4
SOUTHEAST HARBOR DEVELOPMENT, SEATTLE, WASHINGTON

PROJECT DESCRIPTION

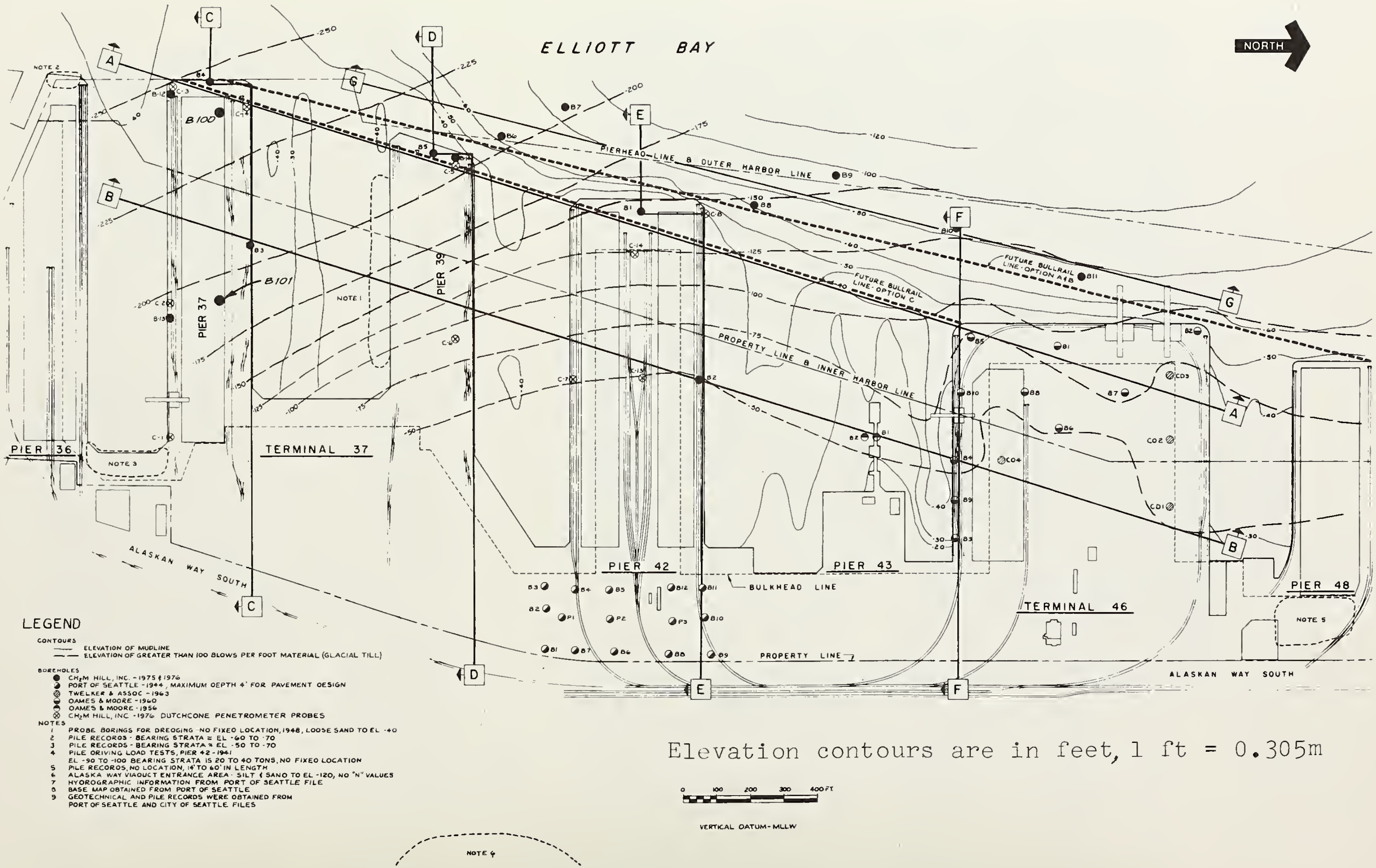
Static penetrometer probes were used to supplement test boring information for the Port of Seattle southeast harbor development geotechnical investigation. The harbor development is located in southeast Elliott Bay between pier 37 and terminal 46. Figure 24 shows the site plan and subsurface test locations.

The proposed project included the removal of piers 42, 39, and a portion of 37 and the construction of cargo storage areas and apron structures. Terminal 46 will remain unchanged. The fill and pile-supported apron structure will eventually extend from pier 37 to terminal 46. Both the piles and the apron will be constructed of prestressed concrete. Fill will be placed from mudline to approximately elevation +15 feet MLLW. The mudline elevation between the piers is approximately -35 (MLLW) and decreases rapidly from the end of the piers into Elliott Bay. The tidal fluctuation of Elliott Bay ranges from elevation -2 to +12 (MLLW). The approximate elevation of the pier decks is +19 feet (MLLW).

AREA GEOLOGY

The project is located at the northeast corner of a delta formed by the Duwamish River in Elliott Bay. The Duwamish River valley was probably initiated in the Pliocene epoch by uplift of the Cascade Mountains and the formation of major drainage systems in the Puget Sound lowland trough.

During the Pleistocene epoch, climatic changes brought on three or four glaciations. In the earlier glaciation, the Duwamish Valley was cut deep into the depositional plain of the Puget Sound lowland. The last glaciation, the Vashon glacier (15,000 to 13,500 years BP) further deepened north-south-trending valleys and plastered till on the ground surface. With the initial retreat of the Vashon glacier, meltwater carved a valley in the till surface. After further glacial retreat, the marine waters invaded Puget Sound, and the Duwamish Valley became an embayment of Puget Sound. The Cedar and Green Rivers filled the Duwamish



Elevation contours are in feet, 1 ft = 0.305m

Figure 24. Case history 4 site plan

embayment with eroded materials to form the present delta at Elliott Bay. Delta deposits are nonuniform interbedded layers of granular and fine material.

SUBSURFACE INVESTIGATION

Static cone penetrometer probes and test borings were used to obtain subsurface information at the site between November 1975 and February 1976. Both investigative methods were performed by independent firms under subcontract to CH2M HILL. The locations of the probes and borings are shown on Figure 24.

Ten penetrometer probes (C-1 - 8, C-13, C-14) were made from the piers with a 10-ton, truck-mounted static cone penetrometer. The probes were advanced through 3-3/8-inch (8.6 cm) inside diameter, hollow stem augers hung from the pier deck to the mudline. The augers were placed using a conventional mobile B-61 drill rig. Discontinuous soundings were made with a static Dutch cone penetrometer (Goudsche Machinefabriek) fitted with a Begemann side friction sleeve. Readings of point resistance (qc) and point resistance plus sleeve friction (qc + fs) were made every 20 centimeters. An example of a cone log, C-3, is found on Figure 7-4. Plots of fs, qc, and friction ratio fs/qc (FR) have been made. Interpreted penetrometer logs can be found on sections A and B, Figures 25 and 26 , respectively.

Eight test boreholes (B-1 - 5, B-12 - 14) were drilled through the piers with a truck-mounted, B61 hollow-stem auger drill. Six borings (B-6 - 11) were drilled offshore from a barge. In each location, the augers were placed inside a 9-inch (22.8-cm) inside diameter steel casing that was hung from the pier or barge to the mudline. Split-spoon (disturbed) samples were taken and standard penetration tests were performed at approximately 5-foot or 10-foot (1.5- or 3-m) intervals in accordance with ASTM D 1586. Undisturbed samples were taken with a thinwall Shelby tube at selected depths in accordance with ASTM D 1587.

Soil test data for a boring, B-1, are illustrated on Figure 28 and interpreted boring logs are plotted on sections A and B (Figures 25 and 26).

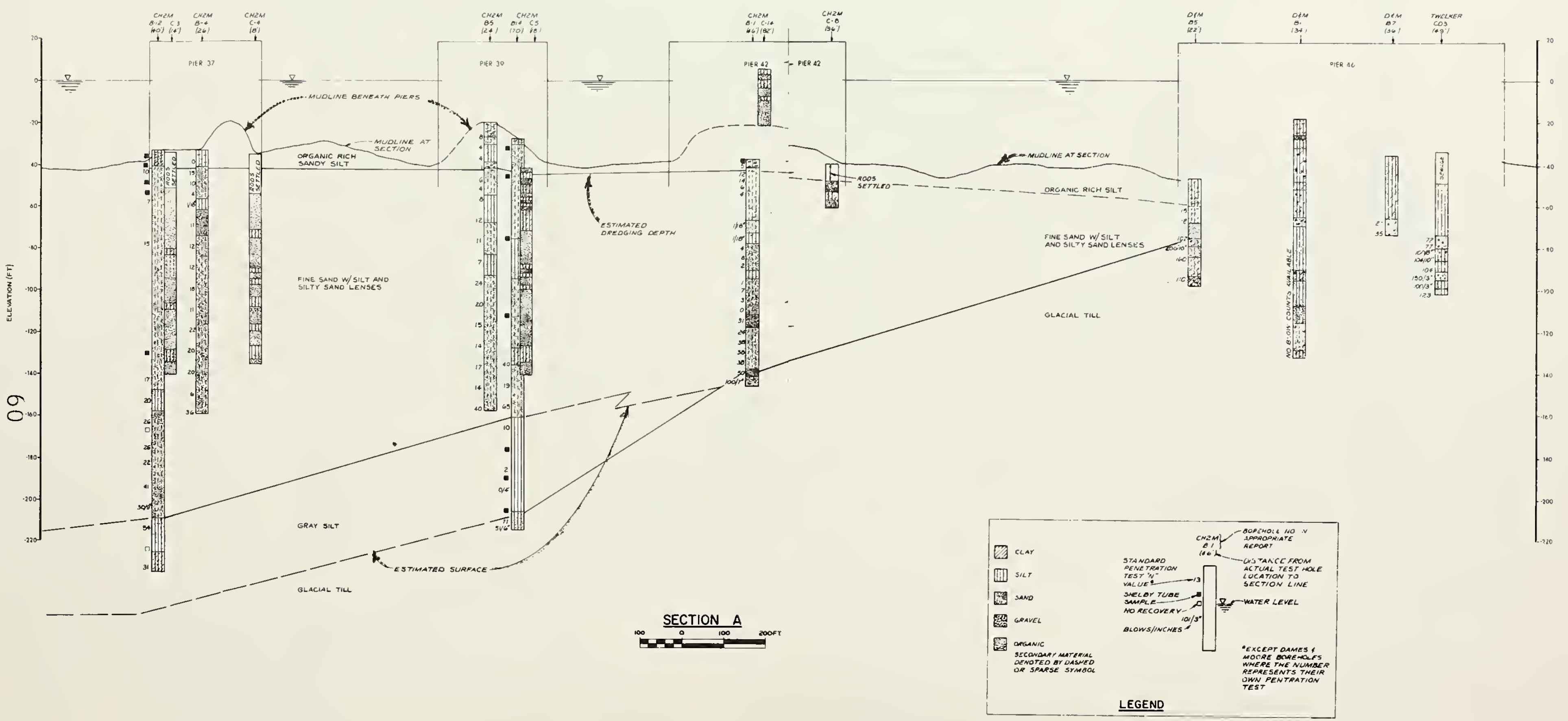


Figure 25. Case history 4, subsurface profile A

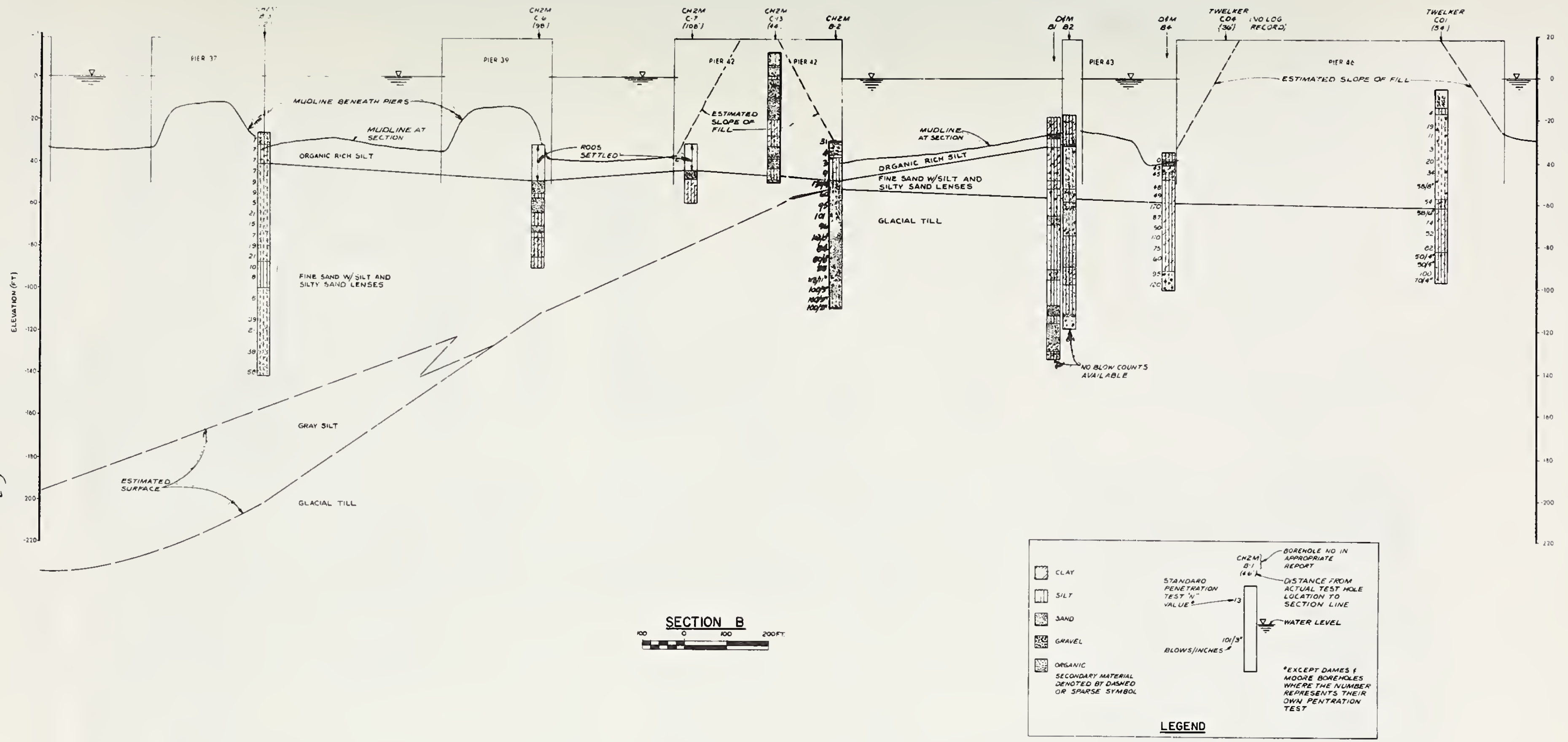


Figure 26. Case history 4, subsurface profile B

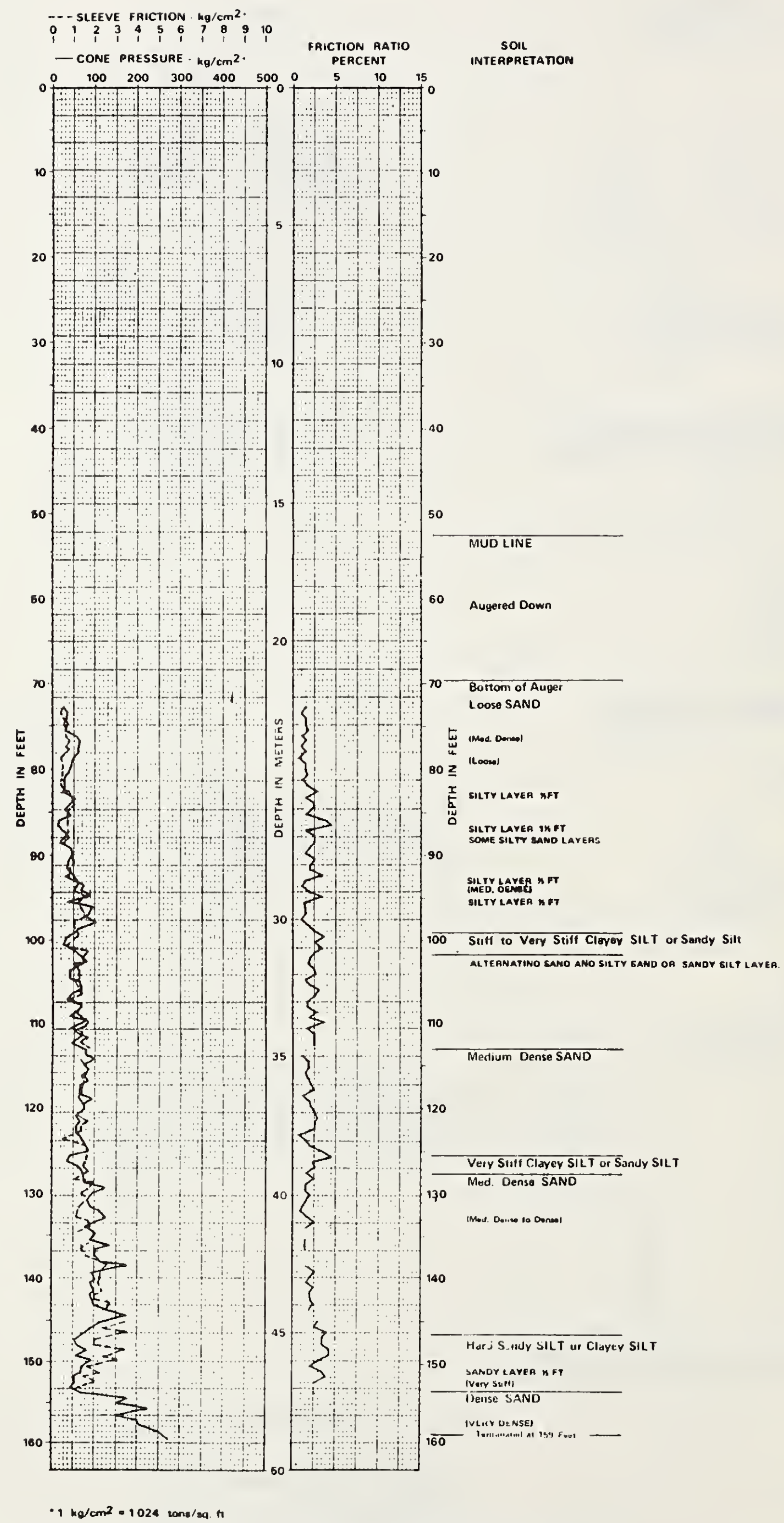


Figure 27. Case history 4 static penetrometer log probe C-3

SAMPLE NUMBER	DEPTH (FEET)	DESCRIPTION OF MATERIAL	CLASSIFICATION	"N" (BL/FT)	MOISTURE PL (%)	MOISTURE w (%)	MOISTURE LL (%)	γ_d (LB/FT³)	STRENGTH q_u (K/FT²)	STRENGTH c (K/FT²)	STRENGTH ϕ (DEGREE)	CONSOLIDATION p_c (K/FT²)	CONSOLIDATION C_c	CONSOLIDATION C_v (FT²/YR)
ST1	2-4	HIGHLY ORG SILTY FINE SAND	SM			93		64						
		W/SHELL FRAGMENTS												
SS2	4-5.5	HIGHLY ORGANIC SILT		5										
SS3	7-8.5	HIGHLY ORGANIC SILT		12										
SS4	12-13.5	ORG SILTY FN SD W/WOOD FRAGS		4										
SS5	17-18.5	ORG SILTY FN SD W/WOOD FRAGS		6										
SS6	22-23.5	HIGHLY ORG FINE GRAVELLY SAND		4										
		W/WOOD FRAGMENTS (DEBRIS)												
SS7A	32-33.5	SL ORG SILTY MED-FINE SAND		1/18"										
SS7B		ORG SILTY FINE SAND												
SS8	37-38.5	ORG FINE SANDY SILT		1/18"										
SS9	42-43.5	SILTY FINE SAND		4										
SS10	47-48.5	(LENSES) FN SDY SILT/FN SAND		8										
SS11	52-53.5	(LENSES) FN SDY SILT/FN SAND		2										
SS12	57-58.5	FINE SANDY SILT		1										
SS13	62-63.5	SL CLAYEY SILTY FINE SAND		7										
SS14	67-68.5	SL CLAYEY SILTY VERY FN SAND		3										
SS15	72-73.5	SL CLAYEY SILTY VERY FN SAND		0										
SS16	77-78.5	SILTY FN SD W/SOME FN GRAVEL		31										
SS17	82-83.5	SILTY MEDIUM-FINE SAND		24										
SS18	87-88.5	SILTY FINE SAND		36										
SS19	92-93.5	COARSE SAND-SILTY FINE SAND		38										
SS20	97-98.5	SILTY MEDIUM-FINE SAND		38										
SS21A	102-103	MEDIUM SAND		50										
SS21B	103-103.5	COARSE SAND W/GRAVEL												
SS22	107-108	NO RECOVERY		100/7"										

PROJECT PORT OF SEATTLE — STATION & OFFSET ______

BORING NO B-1 — GROUND ELEVATION -37 MLLW

SOIL TEST DATA

Note: 1 ft = 0.305m; 1 lb/ft= 0.157 KN/m^3; 1 k/ft^2 = 47.9 kPa

Figure 28. Case history soil data boring B-1

Static penetrometer and test boring completion costs are shown in Table 6.

Table 6. Southeast harbor development subsurface investigation unit costs* (1976 dollars)

	Excluding Mobilization	Including Mobilization
Static Penetrometer (through piers)	$ 4.20 per foot	$ 5.12 per foot
Hollow Stem Auger Drilling (through piers)	$11.06 per foot	$11.59 per foot
Hollow Stem Auger Drilling (over water)	$16.92 per foot	$42.66 per foot

*Inspection costs not included

SOIL PROFILE

The soil profile along the outer end of the piers (section A, Figure 25) shows the upper 10 to 15 feet (3.0 to 4.5 m) of material is a soft silt high in organic matter. Below this is a layer that varies in thickness and is composed of fine sand interbedded with lenses of silt and silty fine sand. Near terminal 46 this sand layer is 20 feet (6.0 m) thick; at pier 37 the layer is 160 feet (48.8 m) thick. In the southern end of the site a 45-foot (13.7-m) thick silt layer lies between the sand and the underlying glacial till. In the northern portion of the site the sand lies directly on the glacial till.

The soil profile for the center portion of the piers (section B, Figure 26) is similar to the profile for the end of the pier, but the thicknesses of the layers are slightly less.

Static penetrometer measurements were recorded and interpreted by a subcontractor and presented to CH2M HILL. The borings were visually classified in the field by one of our engineering geologists or engineers. In the type of soil found at the site area, there is a continuous gradation from sandy silt to silty sand. In several locations (see Figures 25, 26, 27, a static penetrometer probe and a boring were advanced next to each other. A difference in the logs occurred in almost every case. The difference can be attributed to a combination of the following: the sampling interval in the borings was 5 feet (1.5 m) and large gaps occurred between the samples because cuttings did not reach the pier level and the material is layered and variable.

ENGINEERING ANALYSES

A slope stability analysis, a pile capacity analysis, and a settlement analysis were performed for the geotechnical investigation.

Stability Analysis

A preliminary stability analysis was performed for the dike section at bullrail option C, midway between piers 37 and 39. The soil profile chosen for analysis is shown on Figure 29. The following is a discussion of each layer in the model.

Fine Sand with Silt and Silty Sand Lenses. The angle of internal friction for the sandy material was estimated using results from standard penetration tests (SPT), static penetrometer probes, and laboratory direct shear tests. Using relationships that correlate SPT and static penetrometer results with relative density, it was estimated that the fine sandy material had a relative density between 30 and 40 percent. For these results, a friction angle of 30 degrees was estimated. Laboratory shear tests from borings B6-ST3, B7-ST6, and B14-ST3 indicate angles of 32.5 degrees, 35 degrees, and 34 degrees. A value of 30 degrees was used in the analysis; because the strength conditions within the alluvial materials can be variable, ϕ=30 degrees is a conservative but reasonable angle to use for analysis.

Fill. One cone penetrometer test (C-13) was extended through the fill in the center of pier 42. This indicated relative densities of 40 to 50 percent. For clean, well-graded material at 40- to 50-percent relative density, a friction angle of 32 degrees was conservatively estimated. This assumes the material was dumped into place and no mechanical compaction was performed.

The stability of the proposed perimeter dike section was analyzed using the simplified Bishop method. The results for three different sections are shown on

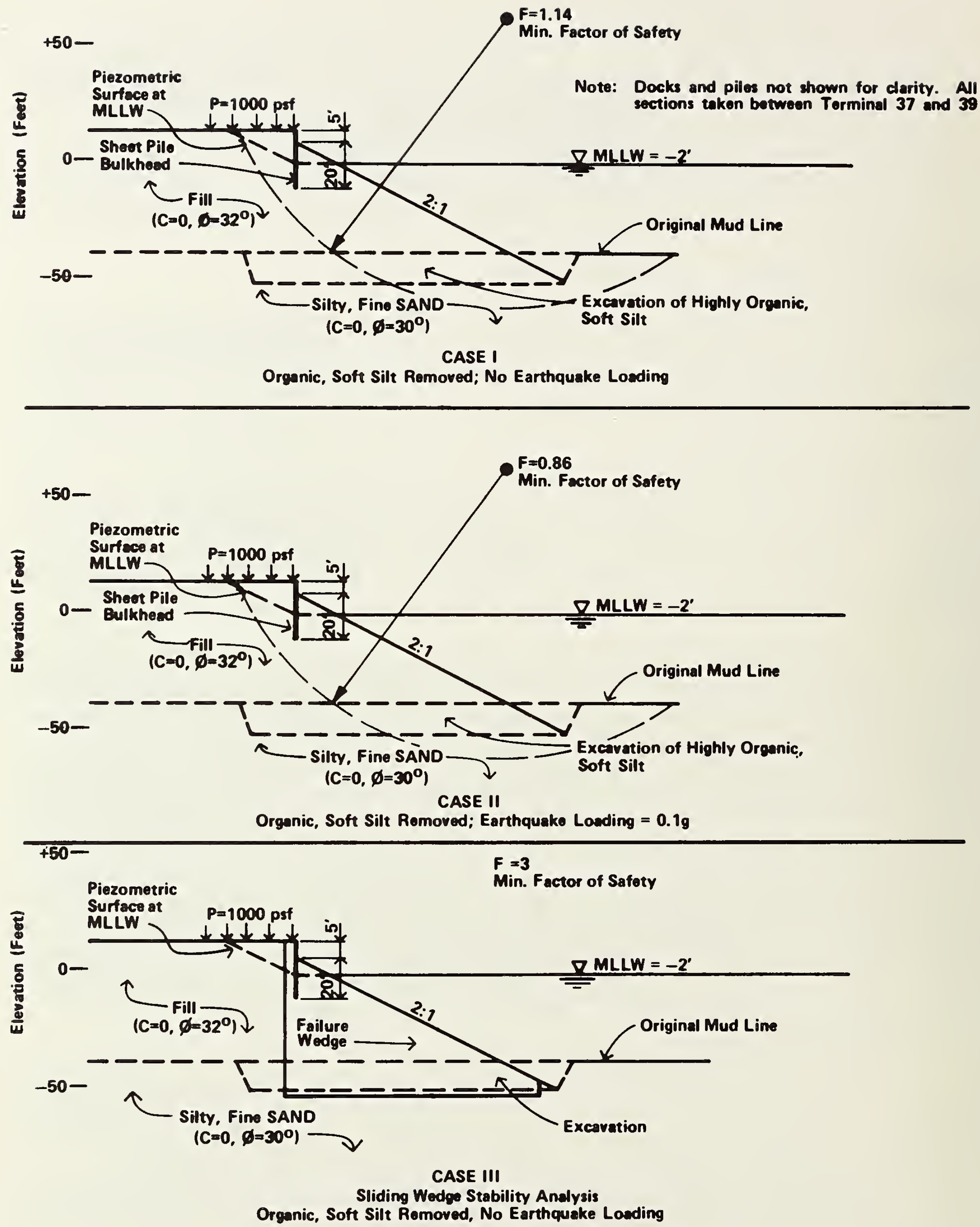


Figure 29. Case history 4 slope stability analysis

Figure 29. The minimum factor of safety (FS) was determined to be 1.14 with no earthquake loading (case I). With the addition of 0.1 g horizontal earthquake loading (case II), the FS was reduced to 0.86. It was assumed in these two cases that all the organic-rich soft silt was removed from beneath the fill.

When a sliding wedge analysis (case III) was performed to consider the possibility of horizontal movement along the foundation material, an FS of 3 was obtained.

Pile Capacity Analysis

The glacial till is found at the project area between -75 feet and -225 feet (MLLW). Contours of the till are shown on Figure 24.

Maximum practical total length of a pile is about 140 to 150 feet. End-bearing piles can be used when the till is above elevation -135, and friction piles will be necessary when the till elevation is below -135 feet. Design criteria for friction piles specified that 16-1/2-inch (42-cm) prestressed octagonal piles with a design load of 117 tons per pile be used for the pier and apron. Using data from boring and static penetrometer probes, static analyses were performed to develop a relationship between friction pile embedment length and ultimate pile capacity.

Using relationships between static penetrometer data, N values, and relative density, strength relationships were developed for piers 37 to 39 and piers 39 to 42. These relationships are shown in Tables 7 and 8 respectively.

Table 7 Piers 37 to 39
strength parameters

Depth Below Mudline (feet)(m)	Relative Density (percent)	Friction Angle (degrees)
20 (6.0)	40-45	32
50-60 (15.2-18.3)	55-65	34
70-90 (21.3-27.4)	65-70	36

Table 8 Piers 39 to 42 strength parameters

Depth Below Mudline (feet)(m)	Relative Density (percent)	Friction Angle (degrees)
15 (4.5)	30-35	30
30 (9.1)	40	31
60 (18.3)	50-55	33
90 (27.4)	60-65	35

The predictive equation for ultimate capacity of friction piles using static penetrometer data was calculated using equation 3-7. Ultimate capacity was calculated at five Dutch cone locations, C-1 through C-5.

A static analysis using a modified Nordlund equation was applied to the profiles assuming no fill above mudline (i.e., toe of the dike). The equation is:

$$Q_{ult} = \alpha N'qL_t A + pLf\gamma \frac{(Lf+x)}{2} K\tan\delta \qquad (7\text{-}1)$$

Where:

Q_{ult} = Ultimate pile capacity, kips

$\delta N'q$ = Bearing capacity factor corrected for depth

L_t = Total length of pile below ground surface, feet

γ = Effective unit weight of soil, kips/cu ft

A = Area of pile tip, square feet

p = Pile perimeter, feet

L_f = Length of embedded portion of pile, feet

x = Cased length below ground surface (if any), feet

K = Lateral earth pressure coefficient

δ = Average angle of friction between pile and soil

Other analysis of the middle one-third of the dike with 20 to 40 feet (6.1 to 12.2 m) of fill (Ø=32 degrees from probe C-5 through fill) and the top two-thirds of the dike at the shoulder with 60 to 80 feet (18.3 to 24.4 m) of fill were performed.

Equation 7-1 was applied to generalized soil profiles generated from static penetrometer data and verified by soil boring N values, while equation 3-7 was applied to specific soil profiles generated from five static penetrometer locations.

Comparisons between results from equation 7-1 and equation 3-7 are presented on Figures 30 and 31. The plots of embedment length versus ultimate pile capacity differ slightly for piers 37 and 39 and piers 39 and 42 because of differing soil conditions.

Settlement Analysis

Soil conditions determined by field investigations indicate that settlement of the interbedded sand and soft silt layers can be expected at terminal 37 and inside the option C bullrail location when the fill is placed.

The generalized soil profile from piers 37 to 42 is shown on Figure 32 The analysis assumes that the fill will be placed adjacent to the north side of the concrete-pile-supported section of pier 37. The fill will slope beneath the pier, and the applied load of the fill will decrease from the north side to the south side of the pier. The analysis also assumes that bottom dump barges will be used and that the organic-rich soft silt will be displaced and mixed with the fill. This 10-foot (3.0-m) thick layer is then added to the thickness of the fill. The remainder of the soil column is separated into layers according to material type: the upper 95 to 120 feet (29.0 to 36.6 m) is fine sand interbedded with silt and silty sand lenses; below that is a 45-foot (13.7-m) thick silt layer directly above the till. The settlement parameters assigned to these layers or a portion of these layers are also given on Figure 32.

The soil profile between pier 42 and terminal 46 is shown on Figure 33. This soil profile is similar to that for piers 37 to 42 except in the thickness of compressible material and the absence of a deep, thick silt layer. Because bottom dump barges were considered in the construction analysis, the upper 10 feet (3.0 m) of organic-rich

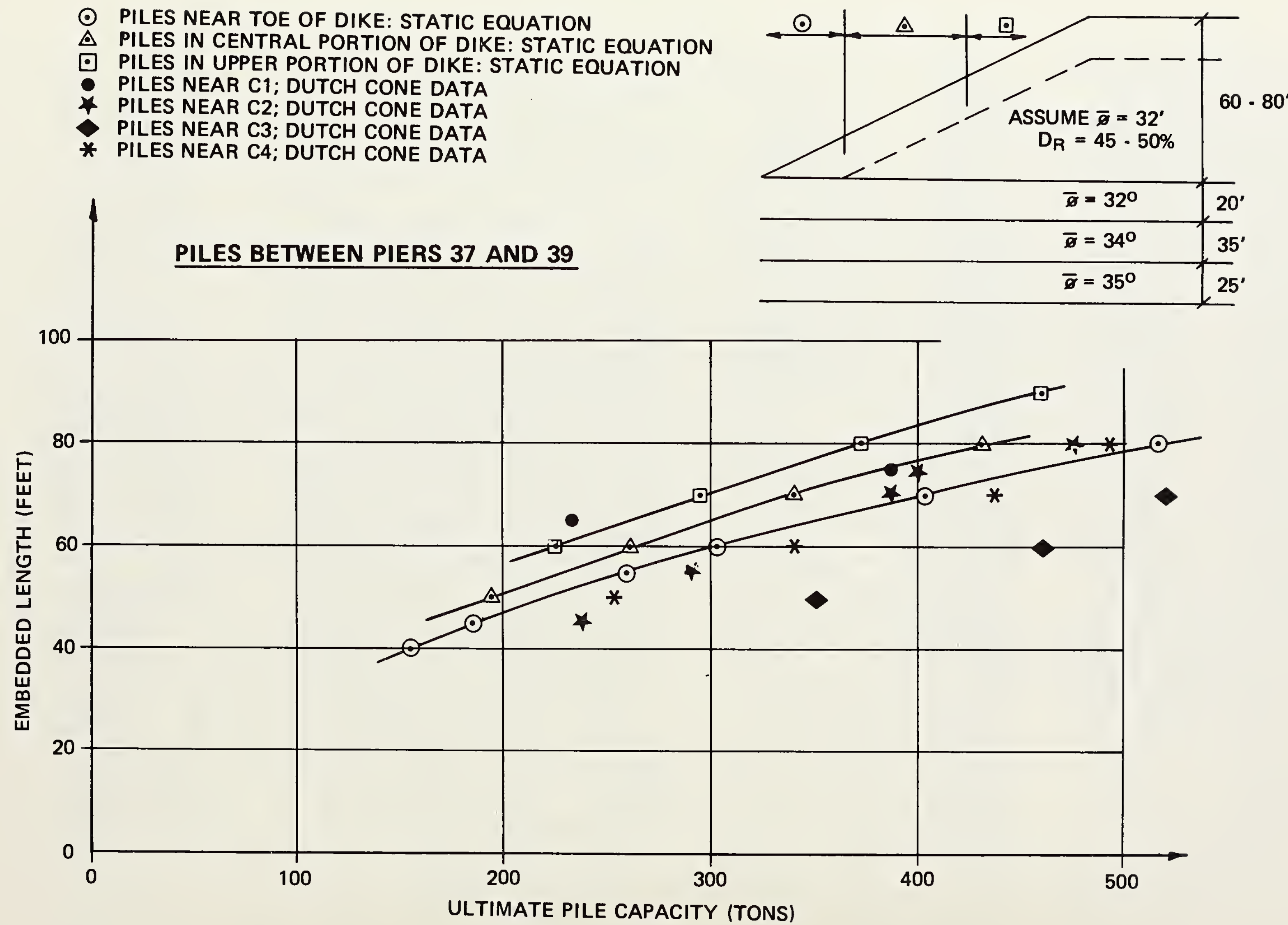


Figure 30. Case history 4 pile capacity piers 37 to 39

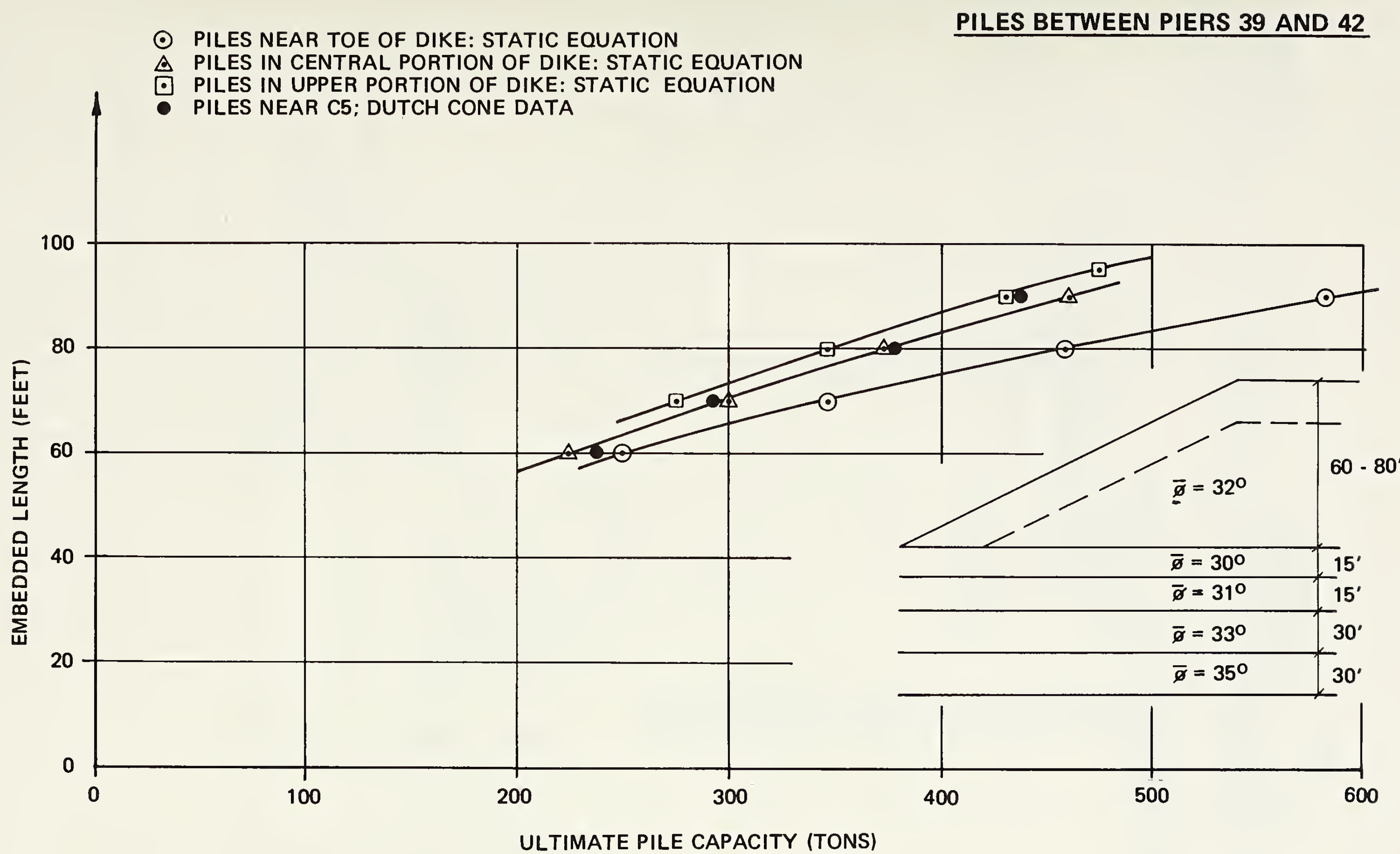
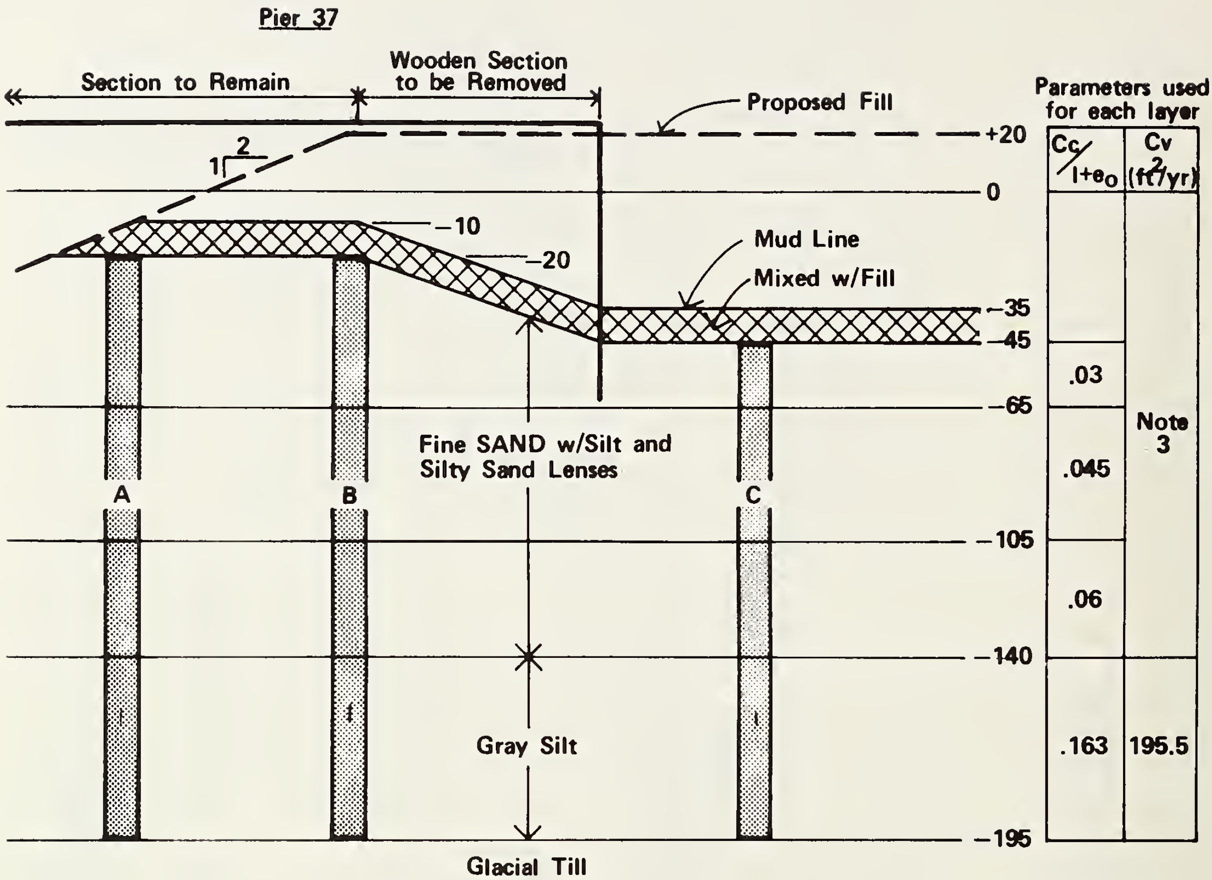


Figure 31. Case history 4 pile capacity piers 39 to 42



Results of Analysis

Analysis Location	Settlement (feet)			Time for 90% Settlement (years)	
	Sand	Silt	Total	Sand	Silt
A	.38	.32	.7	≈ 0.5 to 1	≈ 9
B	1.12	.78	1.9		
C	1.8	1.4	3.2		

Notes:

1 Unit Weight of Soils
Fill = .057 ksf
Subsurface = .055 ksf

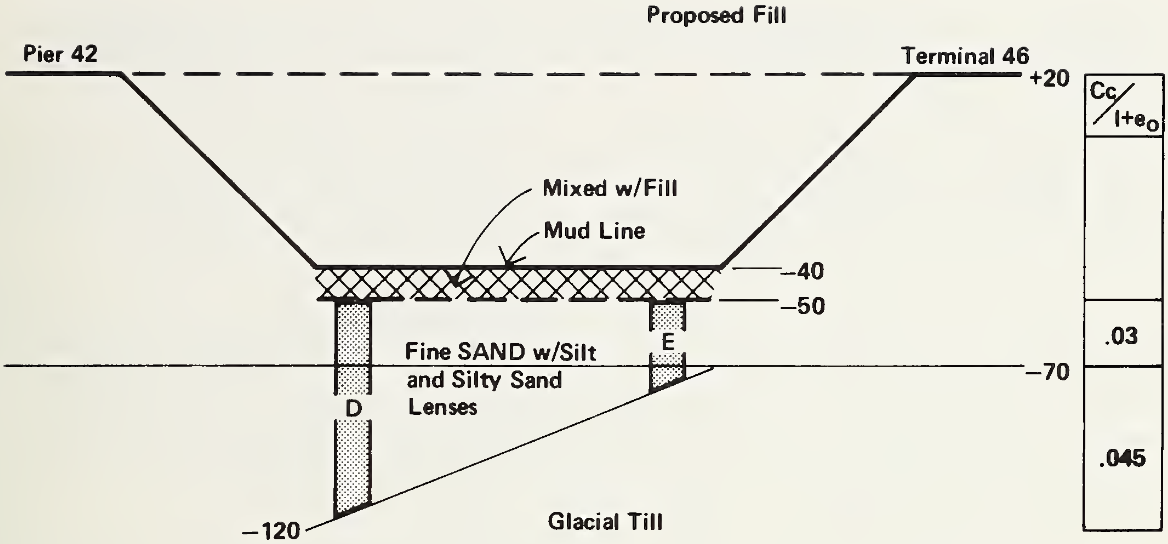
2 Mixed surface layer was added to height of fill and not figured into total settlement

3 Settlement rate of sand was estimated from construction experience and records

4 Not to scale

5. 1 ft = 0.305m; 1 ksf = 47.9 kPa

Figure 32. Case history 4 settlement analysis piers 37 to 42



Note: Not to scale

Results of Analysis

Analysis Location	Total Settlement (feet)	Time for 90% Consolidation (years)
D	1.3	≈.5 to 1
E	0.4	

Figure 33. Case history 4 settlement analysis piers 42 to 46

soft silt is assumed to be mixed with the fill. The settlement parameters used in the analysis are also given on Figure 33.

The settlement parameters (compression index and consolidation coefficient) were developed from a combination of laboratory consolidation test results and static penetrometer values. Settlement data are listed in Table 9.

Four consolidation tests (B13-ST13 and ST16 and B14-ST9 and ST11) in the gray silt layer yielded consistent results that also correlated with the results of one static penetrometer probe (C-2) that reached similar depths. Therefore, these laboratory values were averaged and used for the compression index (C_c) and consolidation coefficient (C_v) of the gray silt layer. (See Table 9)

Two consolidation tests were performed in the layer of sand interbedded with silt and silty sand lenses. Both samples (B12-ST9 and B14-ST3) contained a high percentage of silt. As indicated by the static penetrometer and other test boring samples, a large percentage of the sand layer contained less silt than these laboratory samples. Therefore, it is assumed that the data from the static penetrometer probes (C-1 - 5) were representative of the entire sandy layer, and that these results were chosen for use in the settlement analysis of the sand layer interbedded with silt and silty sand lenses. Example calculations for C_c are found on Figure 34 Construction experience was used to estimate a settlement rate C_v for this fine sand layer.

The amount and time rate of settlement for 90-percent consolidation for five typical analysis locations are given on Figures 32 and 33.

Table 9 Settlement data from laboratory consolidation tests and static penetrometer data

Borehole or Cone Penetrometer No.; Sample No.; and Elevation (ft)			Material Description	$C_c/(1+e_o)$ Lab Tests	$C_c/(1+e_o)$ Cone Penetrometer*	C_v (ft^2/yr) Lab Tests
B-13	ST-13	-144	Gray silt	.104		182.7
B-13	ST-16	-183	Gray silt	.163 avg = .163		175.5 avg = 195.5
B-14	ST-9	-175	Gray silt	.169		262.7
B-14	ST-11	-188	Gray silt	.206		161.3
C-2		-183	Silt very stiff		.181	
B-12	ST-9	-128	Gray silty fine sand	.091		490.3
B-14	ST-3	-47	Silt with fine sand	.097		445.6
C-1		-85	Sand loose		.031	
		-74	Sand medium dense		.037	
C-2		-60	Sand loose		.031	
		-88	Sand medium dense		.047	
		-128	Sand medium dense		.064	
C-3		-63	Sand loose		.028	
		-91	Sand medium dense		.046	
		-118	Sand medium dense		.047	
C-4		-58	Sand loose		.020	
		-78	Sand medium dense		.035	
		-115	Sand medium dense		.064	
C-5		-78	Sand loose		.049	
		-106	Sand medium dense		.071	
		-120	Sand medium dense		.052	

* This calculation is based on the interrelationship between C and C_c of the Terzaghi - Buisman settlement formulas and the Buisman hypothesis, i. e.

$$C = 2.3[(1+E_o)/C_c] \text{ and } C = \alpha(q_c/o_v).$$

(See chapter 11 of Reference 2).

Key:

- C_c = Compression index
- C_v = Consolidation coefficient
- e_o = Initial void ratio
- σv = Effective overburden pressure at depth where q_c was measured
- q_c = Cone resistance
- α = <1 for very dense sands
  = 1 for medium dense sands
  = 1.5 for loose sand

Estimate values for compressibility constant of sandy soils from dutch cone data. Use theory of Buisman.

$$C = \alpha \frac{qc}{\bar{\sigma}_v} \qquad \text{(Sanglerat, Section 112)}$$

Where $\alpha \le 1$ for v. dense sands
$\quad\; = 1$ for med. dense sands
$\quad\; = 1.5$ for loose sands

} Sanglerat, Table 32

$\bar{\sigma}_v$ = effective overburden pressure @ depth where qc measured

After estimating values for C, relationship exists between C and $C_c/1+e_o$

$$\frac{C_c}{1+e_o} = \frac{2.3\ \sigma_v}{\alpha\ q_c} \qquad \text{(Sanglerat, eq 45)}$$

and
$$\frac{C_c}{1+e_o} = \frac{2.3\ \sigma_v}{\alpha\ q_c}$$

Use Following Data in Analysis

Material	δ_b (Kg/cm^3)
Organic silt	4.6×10^{-4}
Loose sand	3.4×10^{-4}
Med. dense sand	8.9×10^{-4}
Dense sand	1.0×10^{-4}
Silt/sandy silt	7.8×10^{-4}

D_R of Sand (%)	α
≤ 35	1.50
40	1.4
45	1.3
50	1.25
55	1.2
60	1.1
≥ 65	1.0

Cone Log C1 (4.6 M Organic Silt) (Typical)

Upper Loose Sand (6.1 M)

$D_R = 40\%$
$\alpha = 1.4$
$q_c = 25$

$$\bar{\sigma}_v = 460\ (4.6 \times 10^{-4}) + 305\ (7.4 \times 10^{-4})$$
$$= .21 + .26 = .47\ \text{Kg/cm}^2$$
$$C = 1.4\ (\tfrac{.25}{.47}) = 75$$
$$\frac{C_c}{1+e_o} = \frac{2.3}{75} = .031$$

Med. Dense Sand (5.5 M)

$D_R = 55\%$
$\alpha = 1.2$
$q_c = 50$

$$\bar{\sigma}_v = .47 + .26 + 275\ (3.9 \times 15^{-4})$$
$$= .47 + .26 + .245 = .975\ \text{Kg/cm}^2$$
$$\frac{C_c}{1+e_o} = \frac{2.3\ \bar{\sigma}_v}{\alpha\ q_c} = \frac{2.3(.975)}{1.2\ (50)} = .037$$

SUMMARY FOR SANDY SOILS

Location	Elevation (ft)	$\frac{C_c}{1+e_o}$
C1	-55	.031
	-74	.037
C2	-60	.031
	-88	.047
	-128	.064
C3	-63	.028
	-91	.040
	-118	.047
C4	-58	.020
	-78	.035
	-115	.064
C5	-78	.049
	-106	.071
	-120	.052

Averages:

-55 to -75	20'	$\frac{C_c}{1+e_o} = .03$
-75 to -100	25'	$\frac{C_c}{1+e_o} = .045$
-100 to -130	30'	$\frac{C_c}{1+e_o} = .06$

Figure 34. Case history 4 settlement calculations

Chapter 8

CASE HISTORY 5
WATER TREATMENT FACILITY, LAKE OSWEGO, OREGON

PROJECT DESCRIPTION

The static penetrometer was used in conjunction with soil test borings for the subject investigation. The investigation was performed in the summer of 1975. Figure 35 shows the expansion plan and the subsurface test locations. The static penetrometer data was used for determining subsurface soil types and for estimating settlement of the planned structures. In 1976, a local bond issue for the expansion was defeated and the project has not been constructed. In spite of the lack of performance data with which to compare the design assumptions, this case history affords a good opportunity to review the penetrometer as a subsurface investigative tool.

Approximate dimensions of the filter and settling basin expansion structures are shown on Figure 35. The expansion units were to share common footings with existing structures. Planned construction was reinforced concrete. The steps in the geotechnical analyses were fairly similar for both the filter and settling basin units; therefore, this case history will focus on one unit, the filter addition. Figure 36 is a cross section through the filter. The structure was to be partially below grade (3 to 8 feet (0.9 to 2.4 m)) and partially above grade (8 to 10 feet (2.4 to 3.0 m)). Normal operating water surfaces would be at the elevations shown. Estimated net operating loadings for various locations beneath the structure are tabulated on Figure 36.

AREA GEOLOGY

The project site is south of Portland on a terrace overlooking the Willamette River. Surface sediments in the area have been mapped [20] as upper Pleistocene lacustrine sediments, predominantly medium and fine sands. The sediments overlay basalts of the mid-Miocene, Columbia River Basalt series. Evidence indicates the sediments were deposited in a vast lake that simultaneously occupied the lower Columbia, Tualatin, and Willamette River valleys. Origin of the sediments (Trimble, [21]) lies with the

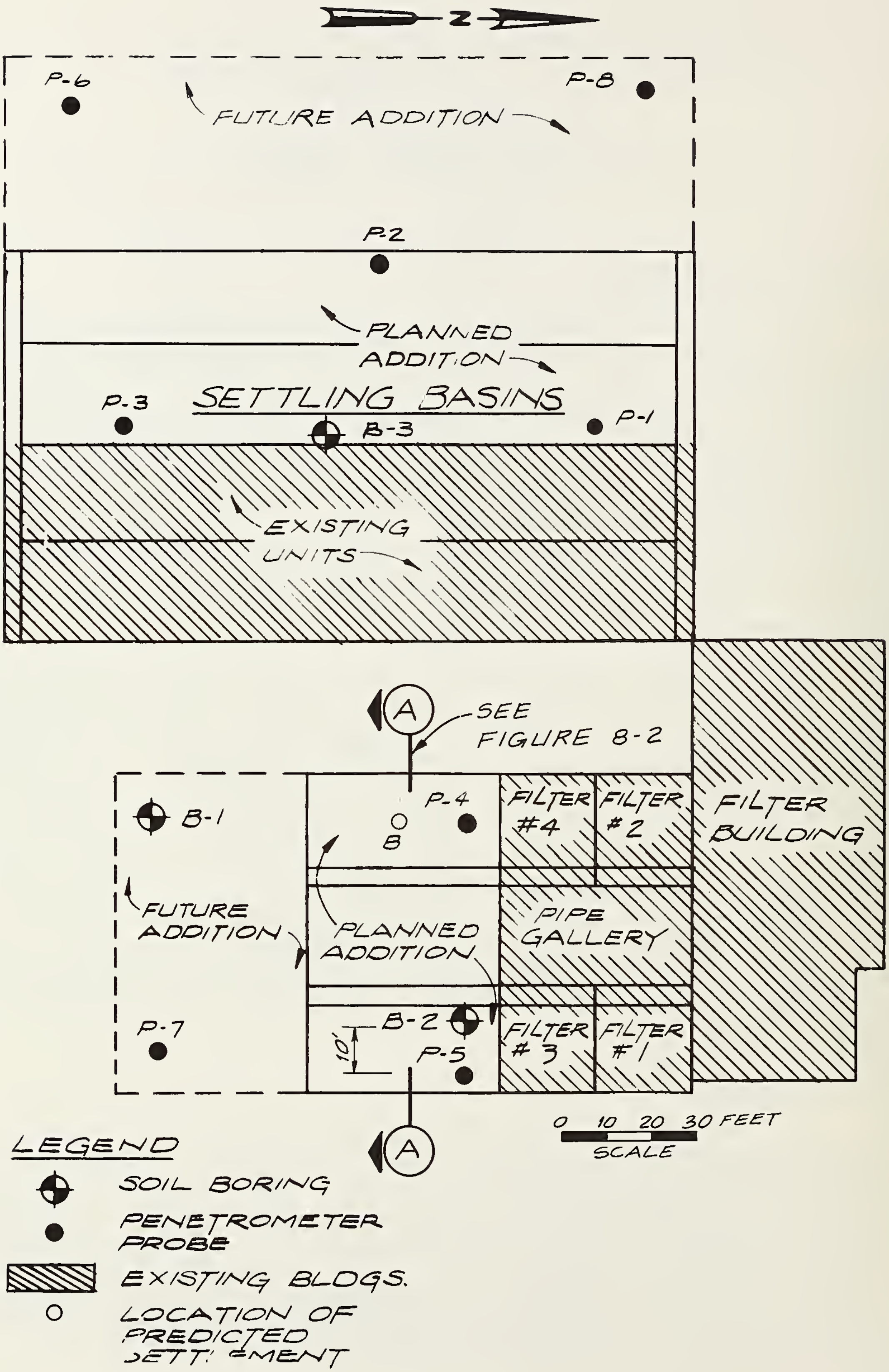
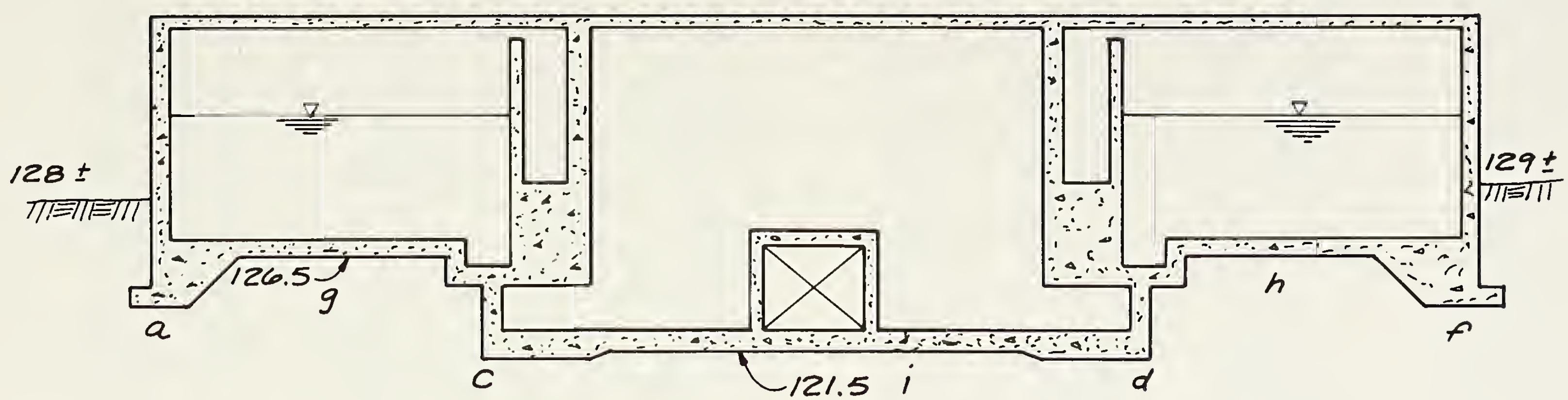


Figure 35. Case history 5 site plan



SECTION A
(FROM FIGURE 1)

NET OPERATING LOADS

LOCATION	STRESS (PSF)
a	1520
c	1210
d	1210
f	1520
g	870
h	870
i	462

Note: 1 psf = 47.9 Pa

Figure 36. Case history 5 filter complex cross section

ancient "Glacial Lake Missoula" of Idaho and Montana. As massive flood waters exited from the Columbia River gorge east of Camas, coarse sands and gravels were the first to be deposited. Fine sands and silts were spread and deposited westward into the lower Willamette and Tualatin River valleys.

SUBSURFACE INVESTIGATION

As described, both the static cone penetrometer and soil test borings were used for the investigation. Soil borings were continuously sampled with 3-inch, outside diameter Shelby tube samplers pushed 2 feet, followed immediately by driving, 18 inches, a Standard Penetration Test split barrel sampler. The borings were drilled with a CME 55 drill rig using a drilling slurry to maintain hole stability. The static penetrometer probes were advanced using a Begemann type cone. Both investigative methods were performed by independent firms subcontracted to CH2M HILL. The expansion site is fairly level with an average elevation of +128 feet mean sea level. Graphic logs of the borings and penetrometer probes are shown on Figure 37. The ground water surface as measured in boring B-1 was 28 feet (8.5 m) below the surface at elevation +101 feet mean sea level.

Test Borings

Graphic logs of the test borings are shown on Figure 37. Sample recovery (length of sample recovered/ length sampled) was excellent with the SPT sampler. A sample was returned with each drive. Sample recovery was very good with the Shelby tube samplers; only 9 of 28 sample attempts had less than 90 percent recovery. Of these, four had less than 30 percent recovery and one had no recovery. The SPT blowcounts shown on Figure 37 were determined by summing the blowcount for the last 12 inches (30.5 cm) of driving. No correction was applied. Shelby tube samples were logged by inspecting the lower end of each sample, however, in doing so, it must be assumed that the lowest inch of the sample is representatived of the entire 24 inches (61.0 cm). Thin lenses of material cannot be detected within the Shelby unless they are very near the bottom or the tube is cut open. This is seldom done.

Soil samples from the test borings were visually classified during the drilling operations. Laboratory tests consisting of Atterberg limits, one dimensional consolidation, unit weight, and moisture content were performed on selected soil samples. Visual classifications were reviewed in the laboratory.

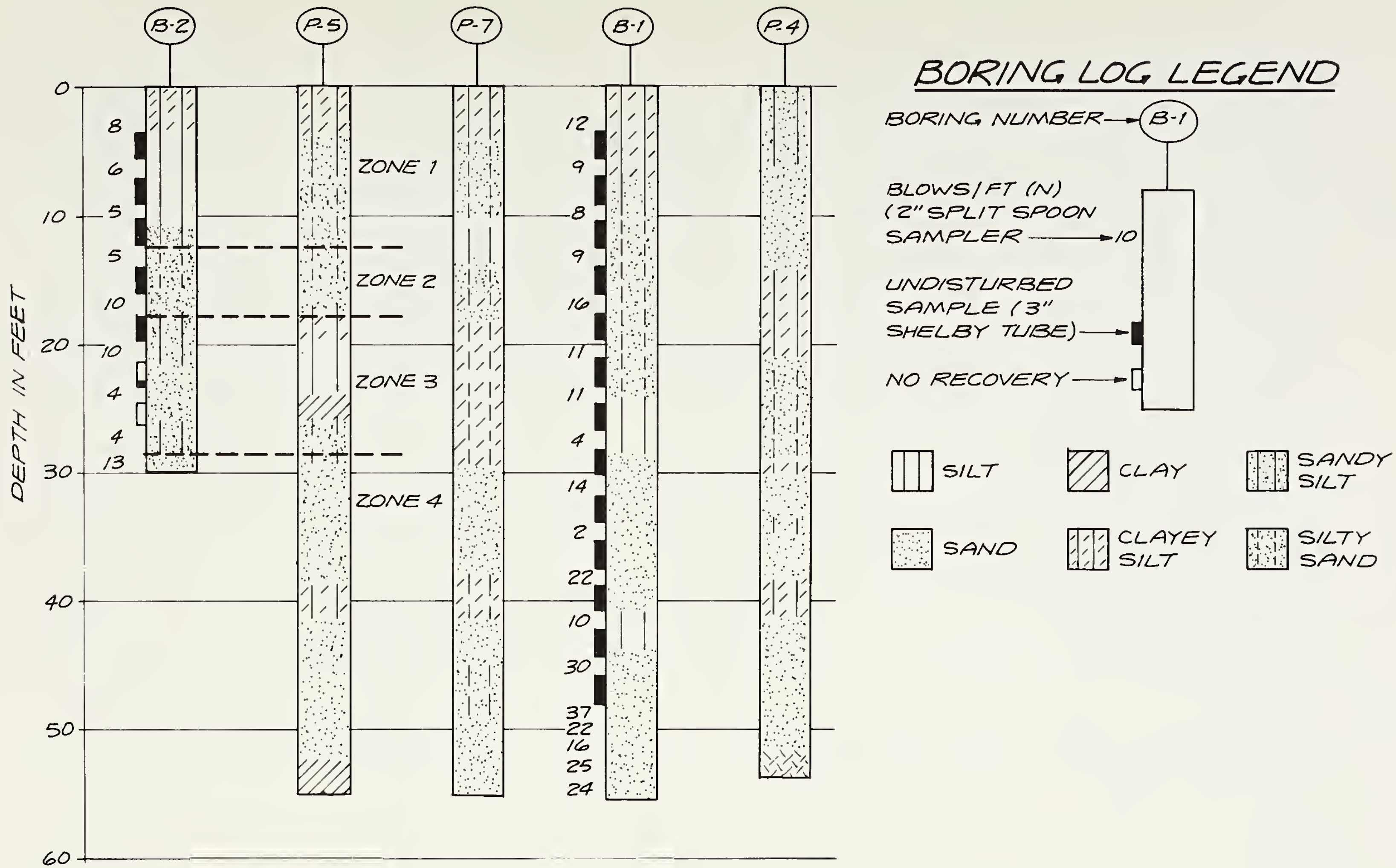


Figure 37. case history 5 filter complex subsurface profile

Static Penetrometer

The subsurface penetrometer logs were all very similar for this site. The graphic log of P-5 is shown on Figure 8-3 with the other probes and borings. The data plot of the point resistance (qc), sleeve friction (fs), and friction ratio for probe P-5 is shown on Figure 38.

The static penetrometer probe and test boring completion costs are shown in Table 10.

Table 10. Lake Oswego water treatment facility subsurface investigation unit costs* (1975 dollars)

	Excluding Mobilization (per foot)
Static Penetrometer	$2.50
Rotary Drilling	$8.26

*CH2M HILL inspection costs not included.

SOIL PROFILE

Subsurface soils encountered are consistent with the reported geology of the area. Visual examination of the soil samples revealed fine grained sands and low to medium plastic silts. The subsurface soils are representative of the fine grained sand and silt phase of the area lacustrine sediments. The deepest test boring (B-1) did not penetrate through the surface sediments. A hard layer, impenetrable with the static penetrometer, was encountered at a consistent depth beneath the site (!155 feet (47.2 m)) and was interpreted as being the underlying basalt surface.

SOIL CLASSIFICATION

On Figure 37 soil classification based on static cone penetrometer data (P-5) is compared to the classification obtained using test boring B-2. The two test locations were approximately 10 feet (3.0 m) apart. Soil classification from the penetrometer data was based on the soil classification chart for Northwest soils described in Chapter 3. Four zones are indicated in which the general correlation of the logs is quite good. In P-5, two zones at 8 and 11 feet

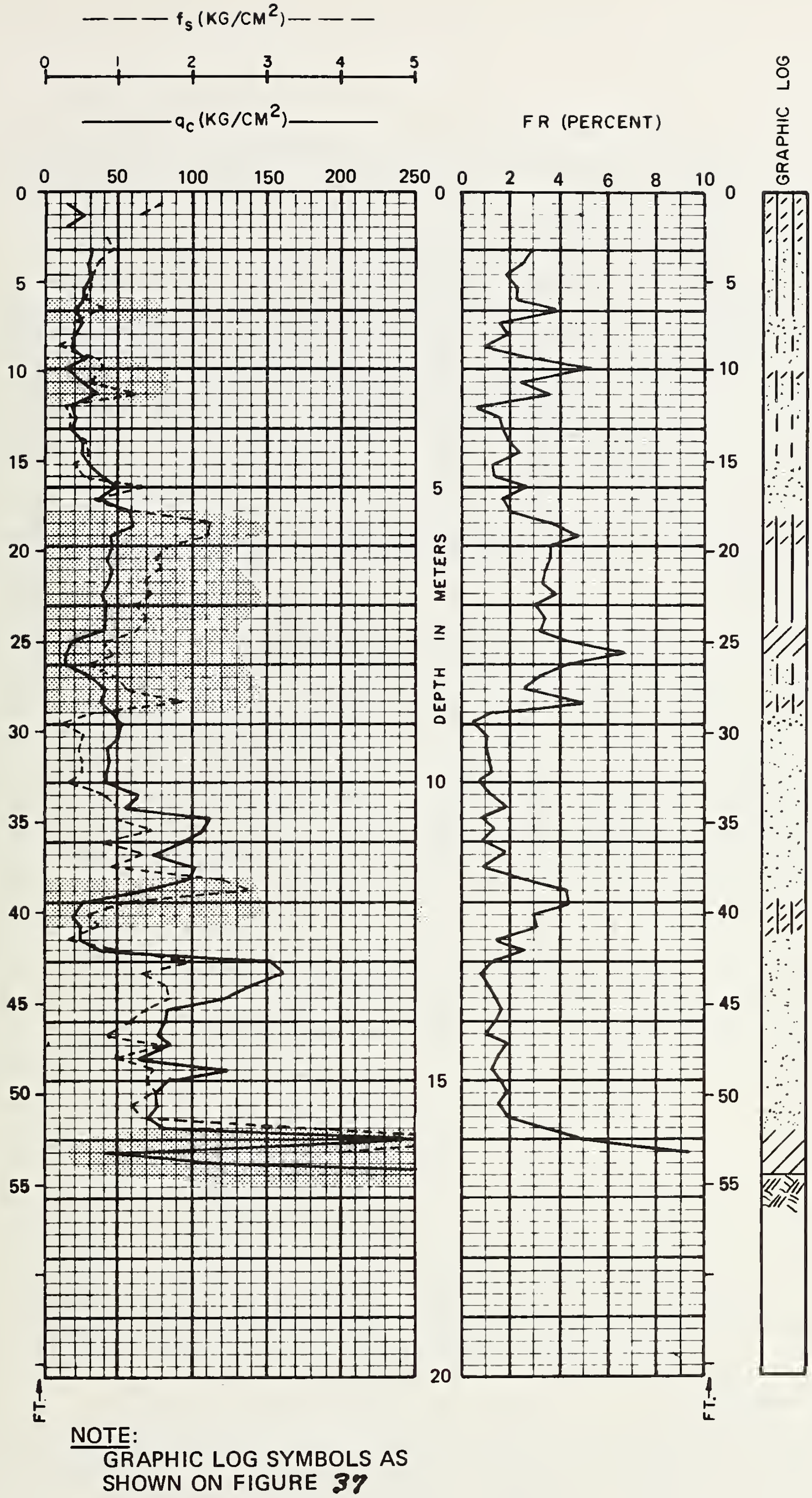


Figure 38. Case history 5 static penetrometer log, probe P-5

(2.4 and 3.4 m) were classified as silty sands, whereas in B-2, at the same depths, the materials were visually classified as low plastic and non-plastic sandy silts. Review of the non-plastic sandy silt in the laboratory indicated approximately 30 percent sand in the material. In terms of engineering behavior, little difference would be expected for either the silty sand or the sandy silt. In Zone 2, the correlation is excellent between the two logs. In Zone 3, the correlation is only moderate. However, in Zone 3, B-2, one of the three Shelby tubes had only 10 percent recovery (2.4 inches (6.1 cm)), and one had no recovery. If better recovery had been obtained, a closer correlation might exist. Clean sand was encountered in the last SPT in B-2 and at the same depth in P-5. In general, soil classification agreement was very good between the test boring and penetrometer logs.

Sand relative densities were determined using both SPT blowcounts and static penetrometer data. The relative densities were determined for comparison purposes only. They were not determined for the original investigation and are reported here only for sands at comparable depths. Blowcount relative densities were estimated using Gibbs and Holtz's [22] criteria for wetted sands. The penetrometer densities were estimated using the Schmertmann Cone Penetrometer Calibration Chart. The densities are shown in Table 11.

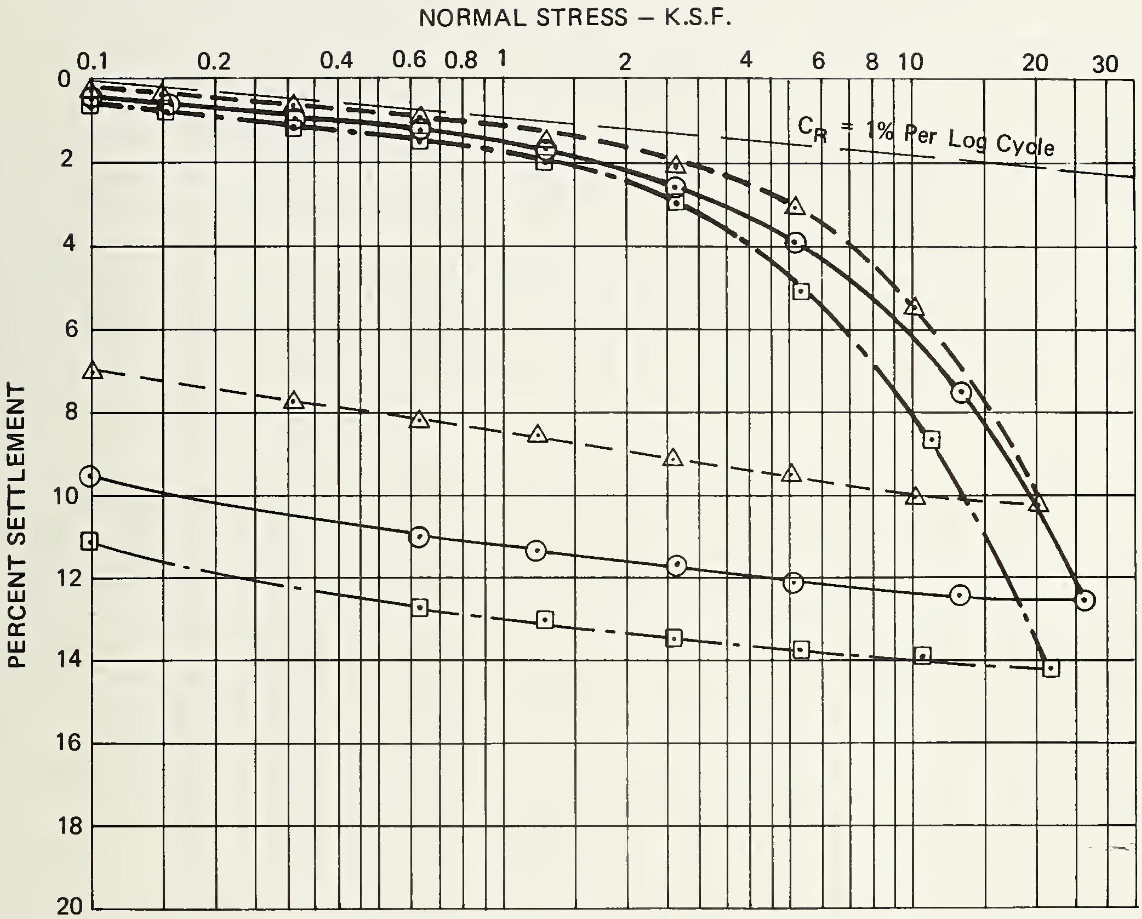
Table 11 SPT and penetrometer determined relative densities (Dr), Lake Oswego water treatment facility

	Dr(%)	
Depth (H)	SPT	Cone Penetrometer
16	50	48
29	50	50

Although only two points were sampled, agreement is very good.

COMPRESSIBLE ZONES

Three consolidation tests were performed as part of the subject laboratory test program. The results of those tests are shown on Figure 39. The existing overburden and reconstructed preconsolidation pressure (Casagrande approach)



Note: 1 ft = 0.305 m, 1 ksf = 47.9 kPa

SYMBOL	SAMPLE	DEPTH	P_o (KSF)	P_c (KSF)	O.C.R.
—⊙—	B – 1, ST – 7	24.5' – 26.5'	2.86	9	3.1
– –△– –	B – 2, ST – 1,	3.5' – 5.5'	0.45	7.5	16.7
–·–⊡–·–	B – 3, ST – 2,	7.0' – 9.0'	0.90	6	6.7

Figure 39. Case history 5 consolidation test results

are tabulated for each sample. All three samples indicate overconsolidated materials.

An independent check for normally consolidated silts and clays was made by examining the static penetrometer data.

None of the silt or clay zones penetrated for the subject investigation had a recorded qc less than one-third the depth (Z/3); therefore, normal consolidation behavior would be unlikely according to Schmertmann's criteria. In addition, the magnitude of the imposed stress plus existing overburden stress nowhere appeared to exceed the preconsolidation stress (Pc). The static penetrometer data thus agreed with the laboratory tests indicating overconsolidated materials.

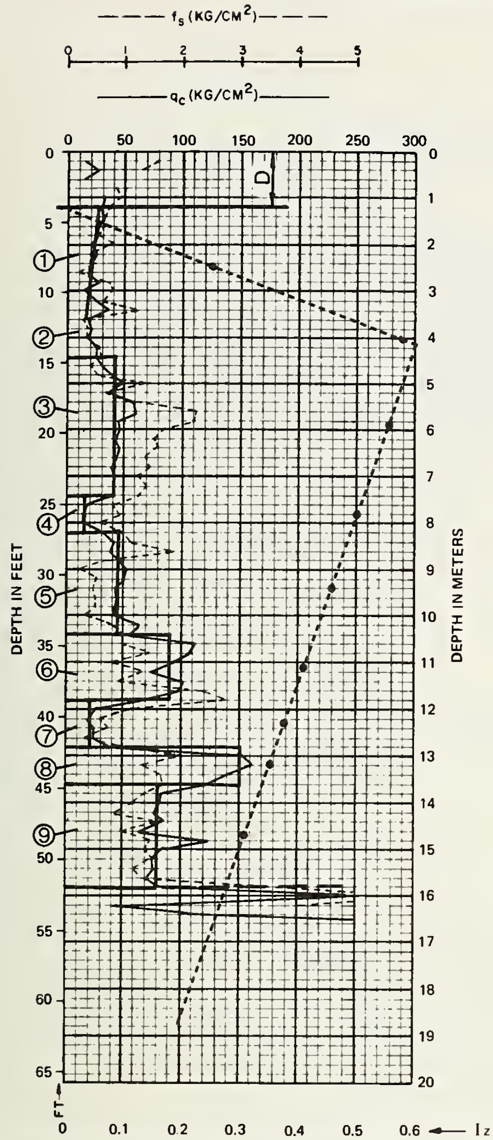
SETTLEMENT

Building settlement was estimated independently using static penetrometer data and consolidation test results. Static penetrometer data were used to estimate settlement following the method outlined by Schmertmann [23]. To use the latter method, it was necessary to assume a rigid block, uniform stress distribution beneath the filter addition. Considering that the roof, walls, and floor were to be connected, the rigid model should be a fair assumption. Foundation level bearing stress was approximated using net operating loads given on Figure 36. The penetrometer settlement calculations were based on the log of P-5. The P-5 log is very similar to the other filter area probes and therefore was not averaged with the other logs. Settlement computations using the penetrometer data are shown on Figure 40.

The consolidation tests (see Figure 39) indicate fairly uniform behavior of subsurface material. The minimum reconstructed Pc value was 6 KSF (287 N/m^2). The sum of the existing overburden stress plus the next stress increase beneath the filter addition (ΔP) results in a total stress less than 5 KSF (239 N/m^2); the material therefore would behave as overconsolidated. The reloading compression indices for the three samples tested all have roughly the same value (1 percent per log cycle). This value was used to estimate the consolidation settlements shown on Figure 41.

The settlement estimated using Schmertmann's method was more than that estimated using the consolidation method. Both values, 0.65 inch (1.65 cm) and 0.48 inch (1.22 cm),

STATIC PENETROMETER SETTLEMENT ESTIMATE



Structure: Filter Addition

-Assume rigid, uniformly loaded surface

-B = least width = 40' (12.2m)

-Average embedment = D = (128-124) = 4'

$-P_{avg} \sim 1{,}000$ psf $\rightarrow \Delta P = 1{,}000 - 4 \times 112 = 552$ psf

$= \frac{.552}{2.044} = .27$ Kg/cm^2

Settlement Estimate Steps

1. Use P-5 Profile @ Left (O.C. Mat'l)
2. B = 40' (Both filters & gallery) D = 4'
3. $\gamma_{soil(m)} = 112$ pcf, $P_o = 4 \times 112 = 448$ psf
4. 2B = 80' → Extends beyond RR @ 52' -layers shown @ left
5. See table for computations
6. Use $E_s = 2xq_c$
7. 0-2B I_z Strain influence indicated @ left. Ignore settlement below 52' (rock).

Layer (1)	Δz (cm) (2)	q_c (Kg/cm^2) (3)	E_s (Kg/cm^2) (4)	Z_{C_L} (cm) (5)	I_z (6)	$(\frac{I_z}{E_s}) \Delta z$ (7)
1	240	25	50	120	0.25	1.2
2	80	20	40	280	0.57	1.14
3	300	40	80	470	0.55	2.06
4	80	15	30	660	0.51	1.36
5	220	45	90	810	0.46	1.12
6	140	90	180	990	0.41	0.32
7	100	20	40	1,110	0.37	0.93
8	80	150	300	1,200	0.35	0.09
9	220	80	160	1,350	0.31	0.43
Total						8.65

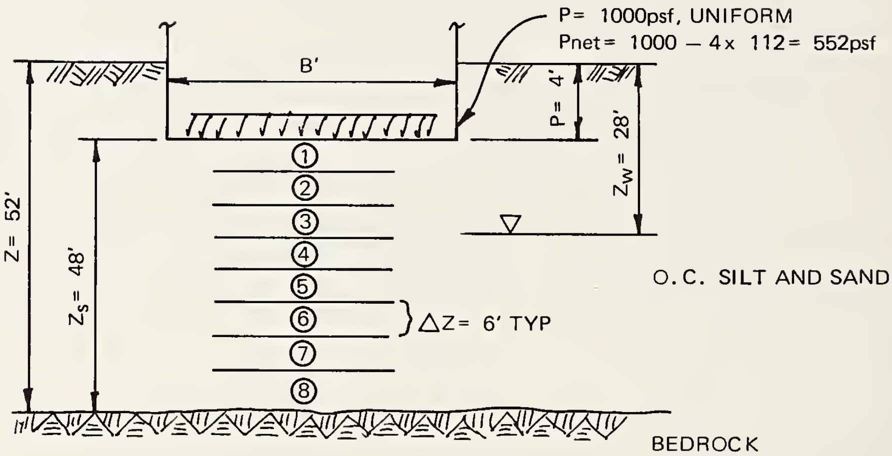
$C_1 = 1 - .5\ (\frac{448}{552}) = 0.59,\ C_2\ (1\ yr) = 1.2,\ \Delta P = 0.27$ Kg/cm^2

$\rightarrow \rho = C_1 C_2\ \Delta P \sum_{o}^{1b} (\frac{Iz}{Es})\ \Delta z = (.59)(1.2)(.27)(8.65) = 1.65$ cm $=$ 0.65 inch

Figure 40. Case history 5 static penetrometer settlement model and calculations

CONSOLIDATION SETTLEMENT ESTIMATE

Soil Model



Assumptions

1. No bedrock settlement
2. Eight equal soil layers sufficient to estimate consolidation settlement
3. Material uniformily overconsolidated with C_r = 1% per log cycle (see Figure 8)
4. Boussinesq stress distribution applicable with B = $\sqrt{40x70}$ = 53' for a square FND @ C_L.
5. Consolidation settlements follow consolidation model, i.e.:

$$\text{Settlement} = \sum_{n=1}^{8} \Delta s_n = \Sigma C_r\ (\Delta z)\ \log_{10}\left(\frac{Po+\Delta P}{Po}\right)$$

Computations

Layer (1)	Δz (ft) (2)	Z_{C_L} (ft) (3)	Po (psf) (4)	I_B (Boussinesq) (5)	ΔP (psf) (6)	ΔS (H) (7)
1	6	3	784	1.00	552	.0139
2	6	9	1,456	.93	513	.0079
3	6	15	2,128	.90	497	.0055
4	6	21	2,800	.80	442	.0038
5	6	27	3,286	.70	386	.0029
6	6	33	3,586	.60	331	.0023
7	6	39	3,886	.55	304	.0020
8	6	45	4,186	.42	232	.0014
Total						.0397

∴ Settlement = .0397 ft = 0.48 inches

Figure 41. Case history 5 consolidation settlement model and calculations

were very close, however. Both methods indicated an acceptable level of settlement, that is, based on the rough approximation to the structural loadings, our firm believes that both methods suggest a reasonable, non-damaging settlement for the structure. The Schmertmann method of estimating settlements is simpler than the consolidation approach and is believed to be conservative for this site.

Chapter 9

SUMMARY AND CONCLUSIONS

While the static cone penetrometer has been in use in Western Europe for many years, it has only recently become an accepted exploratory tool in the United States. European experience with the device is widespread, and has created a confidence in the device that has allowed development of design parameters directly from penetrometer data.

We have used the static cone penetrometer for a variety of geotechnical purposes since early 1975. Our uses of the static penetrometer have included:

- Subsurface investigations
- Soil classification
- Settlement analyses
- Pile capacity analyses

We have used 10-ton, truck-mounted and 20-ton, trailer-mounted static cone penetrometers. Both are self-contained hydraulic units that use the Begemann type cone and Gaudsche Machine Fabriek 20-ton hydraulic load cells. The 10-ton unit has been used to depths approaching 160 feet (48.8 m) and the 20-ton unit has been used to depths approaching 230 feet (70.1 m).

Information from the cone penetrometer data obtained in subsurface investigations includes estimates of soil type and compressibility. Through local experience on jobs in which both soil borings and cone penetrometers have been used, we have modified soil classification correlation charts presented by others. Compressible zones may be detected by observing the cone point resistance versus depth. we use both Sanglerat's "Less than 12 kg/cm^2" and Schmertmann's "qc < Z/3" rules-of-thumb to detect zones of suspect compressibility. Sand relative densities may also be estimated from the static cone penetrometer data.

Estimates of foundation structural performance, such as shallow footing or pile loading capacities, may also be inferred from static cone penetrometer data. Schmertmann's method of predicting shallow footing settlements over sands and sandy silts has proven very useful. Schmertmann's method uses the cone qc resistance to estimate the soil elastic modulus.

We use the Begemann approach to the design of end bearing pile capacities when static penetrometer data are available. The method consists of extrapolating the cone bearing pressure to a bearing pressure that corresponds to the selected pile diameter. The cone pressure is an ultimate or failure pressure; hence, the extrapolated pressure is also a failure pressure and must be reduced by a safety factor for design.

Selected case histories in which we have used the cone penetrometer are summarized below.

1. Geotechnical Investigation - Longview Fibre Company, Secondary Wastewater Treatment Facility, Longview, Washington. Backhoe pits, soil test borings, and static penetrometer probes were used in the investigation. Probes extended 215 feet (65.5 m) to deep gravels. Penetrometer data were used to estimate strength and inplace density, and to determine parameters for settlement analysis of granular soils.

2. Monitoring of Inplace Densification - Northwest Natural Gas Company, Liquefied Natural Gas Facility, Newport, Oregon. Penetrometer probes were used for field control to examine densification obtained to depths of 40 feet (12.2 m) by vibroflotation. Probe data defined zones of unacceptable compaction and led to an explanation for the difficulty: thin (2-inch to 12-inch (5-cm to 30.5-cm)) silt lenses not detected by conventional rotary drill soil test borings. Based on the probe results, an amended procedure in which clean crushed rock was substituted for on-site sand in the vibroflotation process was adopted.

3. Geotechnical Investigation - Proposed Sawmill and Chipping Facility, Aberdeen, Washington. Subsurface conditions were investigated with static penetrometer probes, and one test hole drilled with a rotary drill. Subsurface conditions consisted of 3 to 5 feet (0.9 to 1.5 m) of gravel and wood chip fill underlain by approximately 100 feet (30.5 m) of weak, compressible soil layers composed predominately of silt with scattered layers

and lenses of clay and sand. Pile foundation estimates were based upon the findings of the subsurface investigation.

4. Geotechnical Investigation, Port of Seattle proposed southeast harbor development. The subsurface investigation included test borings to obtain samples of subsurface materials and static penetrometer probes to supplement the boring information. Subsurface conditions consisted of 10 to 15 feet (3.0 to 4.6 m) of soft silt high in organic matter, overlaying a 20- to 160-foot (6.1- to 48.8-m) thick layer of fine sand with lenses of silt. Pile capacities and settlement predictions were made for the site based on the penetrometer data.

5. Geotechnical Investigation, City of Lake Oswego, Oregon, water treatment facility expansion. Penetrometer probes and soil test borings were used to examine surface conditions in silts and sandy silts to a depth of 50 feet (15.2 m). Penetrometer data were used to estimate settlement behavior of a 40- by 70-foot (12.2-m by 21.2-m) filter complex adjacent to an existing filter complex.

Advantages of the static cone penetrometer include:

- Static cone penetrometer costs are between one-third to one-half the cost of drilled test borings.

- We have probed up to 400 feet in an 8-hour shift, about three times faster than conventional boring techniques.

- The "sampling" or testing interval is small (20 cm), enabling very accurate subsurface profiling and allowing identification of layers as thin as 6 inches in some cases.

- Data obtained from probing can be used directly in design and analysis, particularly of pile capacity and settlement in sands.

- Static cone penetrometer probes can be used to develop the most cost-effective and timely field exploration program by cutting down the number of conventional borings and identifying zones of concern that require concentrated sampling and testing.

Disadvantages of the static penetrometer include:

- Samples of subsurface materials cannot be obtained.
- The static penetrometer does not provide indications of ground water conditions.
- Interpretation of probe data in "Ø=C" soils requires considerable subjectivity.
- The device is limited as to the type of materials in which it can be used.

The static penetrometer is not recommended as the sole tool for subsurface geotechnical investigations. It does allow the geotechnical engineer to develop subsurface information more economically than with conventional soil test borings alone (provided cone penetrometer mobilization costs are not prohibitive).

Local experience is advised in developing or using static cone penetrometer data for soil classification. Either the Sanglerat or Schmertmann method outlined in Chapter 3 can readily be applied. As a tool for detecting normally consolidated zones in the subsurface profile.

The methods we employ to reduce static cone penetrometer data as described in Chapter 3 were selected based upon ease of application and accuracy of results. The case histories discussed in this report support the continued use of the static cone penetrometer as a subsurface investigative tool.

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FEDERALLY COORDINATED PROGRAM (FCP) OF HIGHWAY RESEARCH AND DEVELOPMENT

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The diagonal double stripe on the cover of this report represents a highway and is color-coded to identify the FCP category that the report falls under. A red stripe is used for category 1, dark blue for category 2, light blue for category 3, brown for category 4, gray for category 5, green for categories 6 and 7, and an orange stripe identifies category 0.

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