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Celebrating 50 Years

September 27, 1996

116184.G3

Washington State Department of Transportation
Headquarters Materials Laboratory
PO Box 167
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Attn: Mr. James G. Cuthbertson, P.E.:

Subject: WSDOT Agreement Y-6050
Task Assignment No. AC
SR 5, Port of Tacoma I/C to 272nd Stage 4 HOV
Revised Final Geotechnical Report

Attached you will find 15 copies of the revised Final Geotechnical Report for the subject project. Fifteen copies of the original final version were submitted on July 1, 1996. Since that time we have discovered an error in the datum shift from the original plans at NGVD29 to the current design at NAVD88. This error caused most of the elevations cited in the text, tables, and figures of the July 1 report to be roughly 2.1 meters lower than the correct elevations referenced to NAVD88. We have also updated the figures in this report to reflect stationing per the revised preliminary bridge plans dated August, 1996.

Please destroy all copies of the earlier report, dated June 1996 on the cover, and replace them with this revised version. We apologize for this inconvenience and thank you for your cooperation.

CH2M HILL

Karen J. Dawson, P.E.
Project Engineer

SEA/s4cov.doc
Enclosure

# SR 5
# Port of Tacoma Interchange to 272nd
# Stage 4 HOV
# Geotechnical Report

Prepared for

Washington State Department Of Transportation
Materials Branch

SEPTEMBER 1996

CH2M HILL

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# Part 1

# General Information

CHAPTER 1

# Introduction

## Purpose and Scope

The purpose of this report is to summarize subsurface information and to provide geotechnical recommendations for foundations for three bridges and two walls that are part of the SR 5, Port of Tacoma I/C to 272nd, Stage 4 HOV project. The structures included in this project are:

- Bridge 5/503E and 5/503W (SR 18 Overcrossing)
- Bridge 5/504E and 5/504W (SE 336th Overcrossing)
- Bridge 5/506E (Military Road Overcrossing)
- Wall 2 (E-S Ramp at the SR 18 Interchange)
- Wall 3 (DR3 Line, or NB on-ramp at S. 320th St.)

The locations of the five structures are shown in the vicinity/geologic map on Figure 1-1 at the end of this chapter.

The scope of work included:

- A review of published geologic literature, two existing bridge foundation reports (Hong West and Associates, 1992; Washington State Department of Transportation, 1995), and subsurface information obtained by the Washington State Department of Transportation (WSDOT)
- A reconnaissance of the sites
- Characterization of the engineering properties of the subsurface materials at each site
- Analyses to determine foundation design parameters for reasonable foundation alternatives at each site
- Preparation of this summary report

This report is divided into two parts. The first part describes the overall project, the subsurface exploration, and area-wide geologic conditions and seismicity. These topics are common to all five structure sites. In part two, each chapter contains descriptions of the proposed widening, the site conditions, subsurface conditions, analyses, results, and recommendations for a specific structure.

## Background

The project is located on State Route 5 (SR 5) between Tacoma and Federal Way, Washington. The area is in King County, Township 21N., Range 4E., Sections 21, 16, and 9. In this area, SR 5 generally has four or five lanes in each direction separated by a 15 to 20 m (50 to 70 ft.) median. Most of the existing SR 5 structures in this area were constructed in the late

1950s. The proposed improvements typically include one additional lane in each direction for high occupancy vehicles (HOV), additional shoulder width to meet current safety standards, and replacement of the existing curb and bridge railing with new Jersey-type barriers.

Subsurface explorations were planned and performed by the Washington State Department of Transportation (WSDOT) and Hong West and Associates (HWA). CH2M HILL has been supplied with the data from these explorations in order to develop foundation recommendations and prepare this report. A structural consultant will use the information in this report to prepare contract plans and specifications, in accordance with standard WSDOT methods.

A geotechnical report was prepared by HWA (1992) to provide foundation recommendations for Bridges 5/504E, 5/506E, 5/507E, 5/508E, and 5/509E. Because of new subsurface information provided for the adjacent West (or Southbound) bridges, additional site information (about the presence of buried utilities, etc.), and repackaging of the construction contracts, CH2M HILL has been asked to review the information collected by HWA and provide new geotechnical design recommendations.

A geotechnical report was prepared by WSDOT Headquarters Materials Branch (1995) beyond the project limits that provided foundation recommendations for Bridge 5/506W and several other structures to the north. Although this bridge is not part of this scope of work, subsurface information from this report was used to determine the subsurface profile and materials properties at Bridge 5/506E, which is within the scope of this report.

SR 5 stationing is being converted to the metric system, and the plans and specifications for this project will be prepared in metric units. To facilitate the conversion from English to metric units, the results and recommendations in this report are presented in dual units. However, no attempt has been made to modify existing data.

## Area Description

The project is located in an upland area in the southeastern Puget Sound. Most of the area was initially graded for construction of the State Route. Scattered light industrial development is located adjacent to SR 5.

## Authorization

This work, designated Port of Tacoma I/C to 272nd Stage 4 HOV, was authorized by Task Assignment AC, dated February 27, 1996. This task order is part of the on-call services contract, covered by the terms of Consultant Agreement Y-6050 between CH2M HILL and WSDOT.

Task Assignment AC also authorizes CH2M HILL to perform work on three other bridges (5/458, 5/462, and 5/464) and one wall (Wall 1 at S. 375 St.). This work is covered in a separate report.

# Limitations

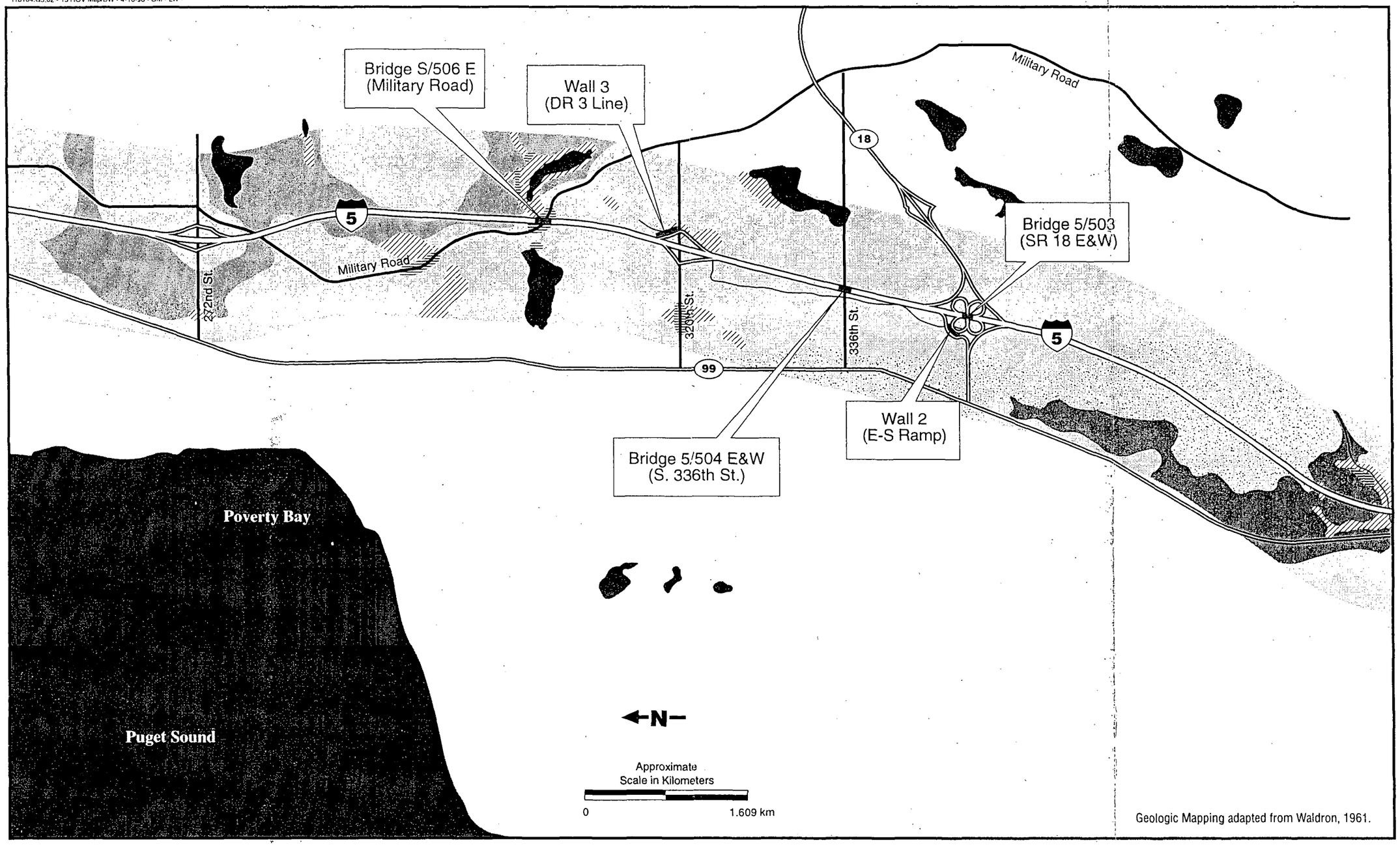
This report has been prepared exclusively for WSDOT and its contractors and consultants for specific application to the proposed improvements to Bridges 5/503E&W, 5/504E&W, and 5/506E and Walls 2 and 3 in accordance with locally accepted geotechnical engineering practice. No other warranty, expressed or implied, is made.

The recommendations contained in this report are based on boring logs, laboratory test results, drawings, and geotechnical reports provided by WSDOT (WSDOT, 1995; Hong West and Associates, 1992). Soil borings indicate subsurface conditions only at the specific locations at the time of sampling. They do not necessarily reflect strata variations that may exist between such locations at a different time. If subsurface conditions different from those described in this report are noted during construction, CH2M HILL should be notified to inspect the area and determine if reevaluation of geotechnical recommendations is necessary.

The recommendations in this report are based on the proposed widening discussed in each of the chapters in Part 2 of this report. If the nature or location of widening changes, the recommendations contained in this report should not be considered valid unless CH2M HILL reviews the changes and verifies or modifies the recommendations in writing. CH2M HILL is not responsible for any claims, damages, or liability associated with the subsurface data presented in this report or reuse of engineering analyses without the express written authorization.

Foundation performance will be dependent, to a large degree, on construction quality control. Therefore, we recommend that a qualified geotechnical engineer observe excavations, foundation preparation, shaft construction, and backfill operations. A qualified observer is necessary to determine if soil conditions differ from those we assumed in forming our recommendations and to ensure that the contractor's construction methods do not alter those conditions.

116184.G3.02 • 15 HOV Map/BW • 4-10-96 • GM • LW




Artificial fill

Water

Creek

Qsr, Proglacial stratified drift;
*Chiefly sand and pebble to cobble gravel. Undifferentiated recessional outwash.*

Qic, Glaciolacustrine deposits;
*Chiefly silt and clay*

Qpm, Lacustrine deposits;
*Organic and mineral sediments deposited chiefly in closed depressions. Thickness ranges from 3 to at least 25 feet.*

Artificial Fill

Qgt, Ground moraine deposits;
*Chiefly compact unoxidized till. Stipple pattern indicates discontinuous mantle of sand and gravel*

Qss, Salmon Springs Drift;
*Chiefly oxidized sand and pebble to cobble gravel; locally includes beds of very fine sand, silt and clay, till, and some nonglacial sediments*

FIGURE 1-1
**Geological and Vicinity Map**
Geotechnical Report SR-5
Port of Tacoma I/C to 272nd
Stage 4 HOV

# Part 2

## Site Specific Information

CHAPTER 2

# Technical Data

## Field Exploration

Test boring information was collected by three different groups. The locations of the borings are summarized in Table 2-1. The boring designations (e.g., H-1-58, B-11-92, H-13-94) indicate the driller and the year in which it was drilled. The methods used to locate the earliest (1958) borings are unknown. The more recent borings were located by measuring to a known point (e.g., the corner of a bridge) with a tape and hand level, while others were located with standard surveying equipment and methods. Surveyed borings are indicated in Table 2-1 by elevations carried out to two decimal places. The boring logs are attached behind each of the site-specific chapters in Part 2. A description of the methods used in each of the four explorations is given below.

The standard penetration test (SPT) was used to collect disturbed samples from all borings. The SPT is a standard test, described in ASTM D1586. The results are presented as three series of blows per 6 inches and as the sum of blows for the last foot driven. The number of blows required to drive the sampler the last foot is called the SPT N value. The test is standard for English units, therefore all recorded N values remain in feet. Where the sampler cannot be driven 6 inches in less than 50 blows, the results are recorded as the number of blows per penetration in inches.

### Original Design Borings

Five borings were drilled by WSDOT for the original design in 1958. The only logs available for these borings are the simplified graphical logs provided on the structural design drawings. The equipment and techniques used for these borings is unknown. The boring logs are in English units.

### Hong West and Associates Widening Borings

Four borings, two at Bridge 5/504E and two at Bridge 5/506E, were drilled by HWA in 1992. Borings BH-4-92, BH-5-92, and BH-8-92 were drilled with a truck-mounted CME 75 owned and operated by Gregory Drilling. These borings were drilled with 102 mm (4 in.) inside diameter (I.D.) hollow-stem augers (HSA) and an automatic trip hammer. Boring B-11-92 was drilled by Soil Sampling Service using a track-mounted HSA rig with a rope-and-cathead system for sample driving.

The boring logs developed by HWA are in English units. They have not been converted to dual units. The soil samples extracted from the borings were examined, classified, and logged in the field by HWA personnel.

### WSDOT Widening Borings

Fifteen borings were drilled by WSDOT crews in late 1994 and early 1995. These borings included three drilled at Bridge 5/506W. Although this bridge is not part of this project, the logs provide additional site information. Subsurface data for Bridge 5/506W also appear in the Stage 3 HOV Foundation Recommendations Report (WSDOT, 1995). The borings were logged by WSDOT personnel. The logs are in both metric and English units.

A skid rig, track-mounted rig, and truck-mounted rig were used. The skid rig is a CME 45 that can operate either 102 mm (4 in.) I.D. HSA or mud rotary with a casing advancer. HW and HQ casing can be used with the casing advancer. With both HSA and mud rotary methods, disturbed samples were collected at 0.61 m (2 ft.) to 1.52 m (5 ft.) intervals by the SPT, conducted in accordance with ASTM D1586, except that sample liners were not used. A Dietrich automatic hammer was used to drive the 0.62 kN (140 lb.) hammer.

The track rig is a CME 375 with the same capabilities and hammer as the skid rig. The truck-mounted rig is a CME 55, equipped with 76 mm (3 in.) I.D. HSA.

In late 1995, three additional borings (H-36-95 through H-38-95) were drilled at Bridge 5/506W to more fully define the variable conditions evident there. The details of the WSDOT drilling rig and drilling methods used for these borings is unknown.

## Laboratory Testing

HWA and WSDOT conducted laboratory testing on selected soil samples to provide classification information. Laboratory testing was limited to grain-size analyses (ASTM D422) and in-situ moisture content (ASTM D2216).

All samples collected by HWA were tested for in-situ moisture content and the results reported on the boring logs. Five samples collected from the HWA borings were tested in their Lynwood laboratory. Fifty-seven in-situ moisture content and grain-size analyses were conducted on samples from the WSDOT borings at their Materials Laboratory. All grain-size curves and the in-situ moisture content results from the WSDOT borings are attached at the end of each site-specific chapter in Part 2.

## Available Historical Information

Some or all of the sheets from the original (1959) bridge design drawings are available. These drawings contain existing footing elevations, original ground contours, and in most cases, the allowable bearing pressures for the existing footings. This information has been reproduced in the bridge layouts and profiles for each site in Part 2. The original contract drawings should be consulted for additional information.

| Table 2-1. I 5 HOV Stage 4 Test Boring Information | | | | |
|---|---|---|---|---|
| | | | | |
| | | | | |
| Note: Vertical Datum = N.A.D. 83/91 | | | | |
| | | | | |
| Bridge | Boring | Station | Offset | Elevation |
| | | (m) | (m) | (m) |
| 5/503E&W | H-13-94 | LW 228+629 | 15.8 LT | 108.5 |
| | H-14-94 | LE 228+644 | 15.8 RT | 109.5 |
| | H-15-94 | LE 228+653 | 17.0 LT | 106.5 |
| | H-16-94 | LW 228+697.5 | 15.2 LT | 109.4 |
| | H-17-94 | LE 228+706.5 | 16.8 RT | 110.0 |
| | H-8-58 | LW 228+699 | 15.5 LT | 102.9 |
| | | | | |
| 5/504E&W | H-18-94 | LW 229+889 | 9.0 LT | 115.0 |
| | H-19-94 | LW 229+927 | 11.9 RT | 110.0 |
| | H-20-95 | LW 229+941.5 | 9.1 LT | 116.0 |
| | BH-4-92 | LE 229+959 | 11.3 LT | 119.0 |
| | BH-11-92 | LE 229+891 | 11.6 LT | 119.0 |
| | H-9-58 | LE 229+957.5 | 0.6 LT | 113.6 |
| | H-10-58 | LW 229+895 | CL | 115.0 |
| | | | | |
| 5/506E&W | H-21-95 | LW 232+937.61 | 8.99 LT | 134.29 |
| | H-22-94 | LW 232+947.52 | 8.69 RT | 125.39 |
| | H-23-95 | LW 233+00.63 | 9.90 LT | 132.80 |
| | H-36-95 | LW 232+923.51 | 10.55 RT | 133.50 |
| | H-37-95 | LW 232+990.41 | 11.16 RT | 131.00 |
| | H-38-95 | LW 232+984.80 | 10.15 LT | 125.18 |
| | BH-5-92 | LE 232+981 | 13.1 LT | 131.0 |
| | BH-8-92 | LE 232+916 | 11.6 LT | 133.0 |
| | H-1-58 | LE 232+975.5 | 4.3 RT | 121.4 |
| | | | | |
| Wall 2 | H-6-95 | E-S 0+160 | 10.9 LT | 101.89 |
| | H-7-95 | E-S 0+280 | 11.2 LT | 106.92 |
| | | | | |
| Wall 3 | H-8-95 | DR3 0+045 | 8 RT | 136.95 |
| | H-9-95 | DR3 0+147 | 7.04 RT | 134.66 |
| | H-10-95 | DR3 0+265 | 8.0 RT | 133.05 |

Substitute COM624 Form for Bridges 5/503 E&W

Vertical Datum = NAVD88

| Bridge | Pier | Assumed Top of Shaft Elevation | | Bottom of Unit 1 (Fill) Elevation | | Bottom of Unit 2 (Unsaturated, Disturbed/ Weathered Till) Elevation | | Bottom of Unit 3 (Unsaturated, Very Dense Till) Elevation | | Bottom of Unit 4 (Saturated, Very Dense Till) Elevation | |
|---|---|---|---|---|---|---|---|---|---|---|---|
| | | (m) | (ft) | (m) | (ft) | (m) | (ft) | (m) | (ft) | (m) | (ft) |
| 5/503W | 1W | 106.6 | 349.6 | 100.5 | 329.6 | N/A | N/A | N/A | N/A | TOE | TOE |
| 5/503W | 1E | 106.6 | 349.6 | 103.8 | 340.6 | 102.9 | 337.6 | 100.5 | 329.6 | TOE | TOE |
| 5/503W | 2W | 102.9 | 337.6 | 100.5 | 329.6 | N/A | N/A | N/A | N/A | TOE | TOE |
| 5/503W | 2E | 102.9 | 337.6 | N/A | N/A | N/A | N/A | 100.5 | 329.6 | TOE | TOE |
| 5/503W | 3E&W | 102.9 | 337.6 | N/A | N/A | N/A | N/A | 100.5 | 329.6 | TOE | TOE |
| 5/503W | 4E&W | 102.9 | 337.6 | N/A | N/A | N/A | N/A | 100.5 | 329.6 | TOE | TOE |
| 5/503W | 5W | 107.3 | 352.1 | 103.8 | 340.6 | 102.9 | 337.6 | 100.5 | 329.6 | TOE | TOE |
| 5/503W | 5E | 107.3 | 352.1 | 106.3 | 348.6 | 102.9 | 337.6 | 100.5 | 329.6 | TOE | TOE |
| 5/503E | all abuts | 107.5 | 352.6 | N/A | N/A | 102.3 | 335.6 | 100.5 | 329.6 | TOE | TOE |
| 5/503E | all interior | 102.3 | 335.6 | N/A | N/A | N/A | N/A | 100.5 | 329.6 | TOE | TOE |

| Soil Unit | Type | Effective Unit Weight | | Cohesion | | Strain at 50% | Internal Angle of Friction | Modulus of Subgrade Rxn | |
|---|---|---|---|---|---|---|---|---|---|
| | | ($kN/m^3$) | (pcf) | (kPa) | (ksf) | | (degrees) | ($MN/m^3$) | (pci) |
| 1 | Sand (Pier 1, 2) | 19.6 | 125 | | | | 33 | 24 | 90 |
| 1 | Sand (Pier 5) | 19.6 | 125 | | | | 35 | 37 | 135 |
| 2 | Stiff clay above water | 20.4 | 130 | 290 | 6 | 0.006 | | | |
| 3 | Stiff clay above water | 20.4 | 130 | 480 | 10 | 0.005 | | | |
| 4 | Stiff clay below water | 11.0 | 70 | 480 | 10 | 0.004 | | 540 | 2000 |

Note: N/A indicates that the layer is absent at that pier

# LOG OF TEST BORING

Washington State Department of Transportation

HOLE No. H-13-94

PROJECT Fife to Military Road Stage 3 — Job No. L-1922

Bridge 5/503 — S.R. 5

Station LW 228+629 — Offset 15.8 m Lt. — C.S. 2719

Equipment — Casing 4" Augers to 48' — Ground El 108.5 m

Method of Boring Hollow Core Augers

Start Date December 5, 1994 — Completion Date December 5, 1994 — Sheet 1 of 3

| Depth (ft) | Meters (m) | Profile | Standard Penetration Blows/ft (10 20 30 40) | SPT Blows/6" (N) | Sample Type | Sample No. (Tube No.) | Lab Tests | Description of Material | Groundwater | Instrument |
|---|---|---|---|---|---|---|---|---|---|---|
| | | | | 2, 3, 7, 4 (10) | | D-1 | | Silty SAND with gravel, loose, brown, moist, disrupted. Retained 1.1 ft. | | |
| | 1 | | | 6, 4, 3, 3 (7) | | D-2 | GS MC | SM, M.C. = 10% Silty SAND with gravel, loose, gray, moist, disrupted. Retained 0.8 ft. | | |
| 5 | | | | | | | | | | |
| | 2 | | | 2, 4, 4, 3 (8) | | D-3 | | Silty SAND with gravel, loose, gray, moist, disrupted. Retained 0.7 ft. | | |
| | | | | 5, 5, 5, 6 (10) | | D-4 | | Silty SAND with gravel, loose, gray, moist, disrupted. Retained 0.2 ft. | | |
| 10 | 3 | | | | | | | | | |
| | | | | 7, 9, 13, 17 (22) | | D-5 | | Silty SAND with gravel, medium dense, gray, moist, disrupted. Retained 0.2 ft. | | |
| | 4 | | | 1, 6, 11, 12 (17) | | D-6 | | Silty SAND with gravel, medium dense, gray, moist, disrupted. Retained 1.2 ft. | | |
| 15 | | | | | | | | | | |
| | 5 | | | | | | | | | |
| | | | | 8, 10, 17, 4 (17) | | D-7 | | Silty SAND with gravel, medium dense, gray, moist, disrupted. Retained 1.0 ft. | | |
| | 6 | | | | | | | | | |
| 20 | | | | | | | | | | |

# LOG OF TEST BORING

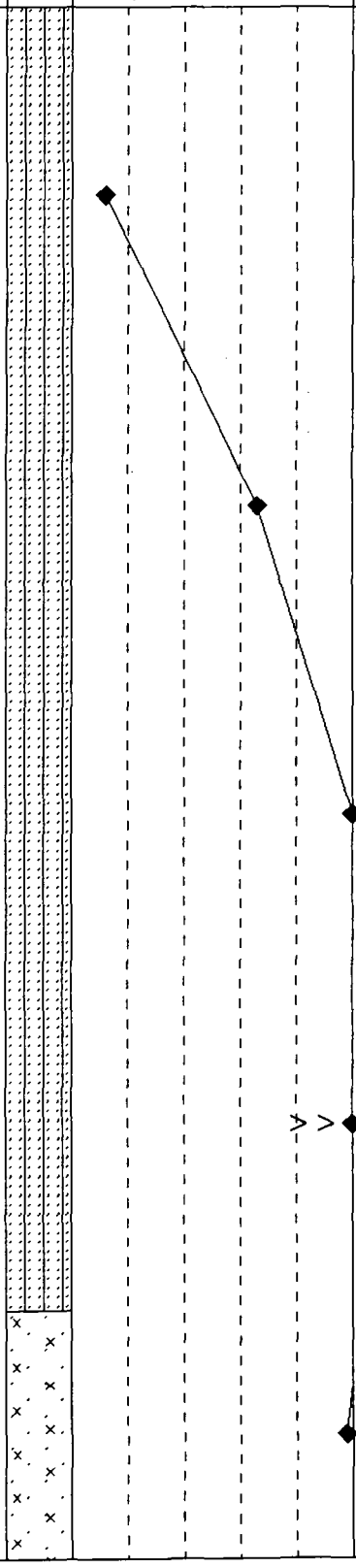
Washington State Department of Transportation

HOLE No. **H-13-94**

Sheet 2 of 3

PROJECT **Fife to Military Road Stage 3**

Job No. L-1922




| Depth (ft) | Meters (m) | SPT Blows/6" (N) | Sample Type | Sample No. (Tube No.) | Lab Tests | Description of Material | Groundwater | Instrument |
|---|---|---|---|---|---|---|---|---|
| 25 | 7 | 1<br>2<br>4<br>45<br>(6) | | D-8 | GS<br>MC | SM, M.C. = 15%<br>Silty SAND with gravel and fir needles, loose, brown, moist, disrupted. Contact point between fill and original ground.<br>Retained 0.9 ft. | | |
| 30 | 8<br>9 | 26<br>33<br>(33/6") | | D-9 | GS<br>MC | SM, M.C. = 5%<br>Silty SAND with gravel, very dense, gray, moist, homogeneous.<br>Retained 1.0 ft. | | |
| 35 | 10<br>11 | 15<br>50<br>(50/6") | | D-10 | | Silty SAND with gravel, very dense, gray, moist, homogeneous.<br>Retained 0.9 ft. | | |
| 40 | 12 | 21<br>25<br>30<br>(55) | | D-11 | | Silty SAND with gravel, very dense, gray, moist, homogeneous.<br>Retained 0.9 ft. | | |
| 45 | 13 | 9<br>20<br>29<br>(49) | | D-12 | GS<br>MC | ML, M.C. = 14%<br>Sandy SILT with gravel, hard, gray, moist, homogeneous.<br>Retained 1.5 ft. | | |

# LOG OF TEST BORING

Washington State Department of Transportation

HOLE No. **H-13-94**

Sheet **3** of **3**

PROJECT **Fife to Military Road Stage 3**

Job No. **L-1922**

| Depth (ft) | Meters (m) | Profile | Standard Penetration Blows/ft (10 20 30 40) | SPT Blows/6" (N) | Sample Type | Sample No. (Tube No.) | Lab Tests | Description of Material | Groundwater | Instrument |
|---|---|---|---|---|---|---|---|---|---|---|
| | 14 | | | | | | | | | |
| | | | | 12<br>17<br>37<br>(54) | | D-13 | | Sandy SILT with gravel, hard, gray, moist, *homogeneous.*<br>Retained 1.4 ft. | | |
| | 15 | | | | | | | | | |
| 50 | | | | | | | | | | |
| | 16 | | | 17<br>50<br>(50/6") | | D-14 | | Sandy SILT with gravel, very hard, gray, moist, homogeneous.<br>Retained 1.0 ft. | | |
| 55 | | | | | | | | | | |
| | 17 | | | | | | | | | |
| | | | | 25<br>38<br>(38/6") | | D-15 | GS<br>MC | SM, M.C. = 7%<br>Silty SAND with gravel, very dense, gray, moist, homogeneous.<br>Retained 1.1 ft. | | |
| | 18 | | | | | | | | | |
| 60 | | | | | | | | End of test hole boring at 59.0 ft. below ground elevation. | | |
| | | | | | | | | This is a summary Log of Test Boring. Soil/Rock descriptions are derived | | |
| | 19 | | | | | | | | | |
| 65 | | | | | | | | | | |
| | 20 | | | | | | | | | |
| | 21 | | | | | | | | | |
| 70 | | | | | | | | | | |

# LOG OF TEST BORING

Washington State Department of Transportation

HOLE No. H-14-94

PROJECT Fife to Military Road Stage 3 — Job No. L-1922

Bridge 5/503 — S.R. 5

Station LE 228+644 — Offset 15.8 m Rt. — C.S. 2719

Equipment — Casing HQ - 35' — Ground El 109.5 m

Method of Boring Wet Rotary

Start Date November 9, 1994 — Completion Date November 9, 1994 — Sheet 1 of 2

| Depth (ft) | Meters (m) | Profile | Standard Penetration Blows/ft (10 20 30 40) | SPT Blows/6" (N) | Sample Type | Sample No. (Tube No.) | Lab Tests | Description of Material | Groundwater | Instrument |
|---|---|---|---|---|---|---|---|---|---|---|
| | | | | | | | | Surface is poorly graded SAND with gravel and small cobbles. | | |
| | 1 | | | 16 13 10 9 (23) | | D-1 | GS MC | GM, M.C.=3% Silty GRAVEL with sand, subangular, dense, dark brown, moist, homogeneous. Driving on large gravel. Retained 0.3 ft. | | |
| 5 | | | | 11 26 41 (67) | | D-2 | GS MC | SM, M.C.=7% Silty SAND with 3 inch or smaller layers of gravel with sand. Very dense, light gray to tan, moist to dry, layered. Retained 1.5 ft. | | |
| | 2 | | | | | | | | | |
| | | | | 12 27 29 (56) | | D-3 | GS MC | SM, M.C.=10% Silty SAND with gravel, gravel is angular to subangular, very dense, light brown to light gray, layered. Retained 1.0 ft. | | |
| 10 | 3 | | | 46 50/2 (50/2") | | D-4 | | Silty SAND with gravel, very dense, tan, traces of gray, moist to dry, layered. Retained 0.7 ft. | | |
| | | | | 51/6" (51/6") | | D-5 | | Silty SAND with gravel, gravel is angular to subangular, very dense, light gray, moist, homogeneous. Poorly cemented. Retained 0.5 ft. | | |
| | 4 | | | | | | | | | |
| 15 | | | | 62/6" (62/6") | | D-6 | GS MC | SM, M.C.=12% Silty SAND with gravel, subangular, very dense, moist, homogeneous. Slightly cemented. Retained 0.5 ft. | | |
| | 5 | | | | | | | | | |
| | 6 | | | | | | | | | |
| 20 | | | | | | | | | | |

# LOG OF TEST BORING

Washington State Department of Transportation

HOLE No. **H-14-94**

Sheet 2 of 2

PROJECT **Fife to Military Road Stage 3**

Job No. **L-1922**

| Depth (ft) | Meters (m) | Profile | Standard Penetration Blows/ft (10 20 30 40) | SPT Blows/6" (N) | Sample Type | Sample No. (Tube No.) | Lab Tests | Description of Material | Groundwater | Instrument |
|---|---|---|---|---|---|---|---|---|---|---|
| | | | | 67/6" (67/6") | | D-7 | | Silty SAND with gravel, gravel is subangular, very dense, olive gray to gray, moist, homogeneous. Retained 0.3 ft. | | |
| | 7 | | | | | | | | | |
| 25 | | | | 50/6" (50/6") | | D-8 | | Very dense, driving on a rock. Recovered 0.2 ft. piece of rock. | | |
| | 8 | | | | | | | Core run - silty SAND with gravel, subangular to subrounded, approximately 30% gravel. | | |
| | 9 | | | | | | | | | |
| 30 | | | > > | 56/6" (56/6") | | D-9 | GS MC | GM, M.C. = 8% Silty GRAVEL with sand, subangular, very dense, light olive gray, moist, homogeneous. Retained 0.3 ft. | | |
| | 10 | | | | | | | | | |
| 35 | | | > > | 60/0" (60/0") | | D-10 | | Driving on rock. No recovery. | | |
| | 11 | | | | | | | End of test hole boring at 36.0 ft. below ground elevation. This is a summary Log of Test Boring. Soil/Rock descriptions are derived from visual field identifications and laboratory test data. | | |
| | 12 | | | | | | | | | |
| 40 | | | | | | | | | | |
| | 13 | | | | | | | | | |
| 45 | | | | | | | | | | |

# LOG OF TEST BORING

Washington State Department of Transportation

HOLE No. H-15-94

PROJECT Fife to Military Road Stage 3 — Job No. L-1922

Bridge 5/503 — S.R. 5

Station LE 228+653 — Offset 17.0 LT — C.S. 2719

Equipment — Casing HW X 14', HQ X 31.5' — Ground El 104.5 m

Method of Boring Wet Rotary

Start Date December 7, 1994 — Completion Date December 8, 1994 — Sheet 1 of 2

| Depth (ft) | Meters (m) | Profile | Standard Penetration Blows/ft (10 20 30 40) | SPT Blows/6" (N) | Sample Type | Sample No. (Tube No.) | Lab Tests | Description of Material | Groundwater | Instrument |
|---|---|---|---|---|---|---|---|---|---|---|
| | 1 | | | 12 31 50/5 (81/ | | D-1 | GS MC | GM, M.C. = 13% Silty GRAVEL with sand, very dense, gray, moist. Retained 0.8 ft. | | |
| 5 | | | | 43 50/5 (50/5") | | D-2 | | Silty GRAVEL with sand, rounded to angular, very dense, gray, moist, homogeneous. Retained 0.5 ft. | | |
| | 2 | | | 40 50/4 (50/4") | | D-3 | | Silty GRAVEL with sand, rounded to angular, very dense, gray, moist, homogeneous. Retained 0.5 ft. | | |
| | | | | 50/1" (50/1") | | D-4 | | No recovery. | | |
| 10 | 3 | | | 36 54/6 (54/6") | | D-5 | GS MC | GM, M.C. = 15% Silty GRAVEL with sand, very dense, gray, moist, homogeneous. Retained 0.8 ft. | | |
| | 4 | | | 50/4" (50/4") | | D-6 | | Silty GRAVEL with sand, very dense, gray, moist, homogeneous. Retained 0.4 ft. | | |
| 15 | 5 | | | 21 27 50/3 (50/ | | D-7 | GS MC | SM, M.C. = 13% Silty SAND with gravel, very dense, gray, moist, homogeneous. Retained 0.8 ft. | | |
| 20 | 6 | | | | | | | | | |

# LOG OF TEST BORING

Washington State Department of Transportation

HOLE No. **H-15-94**

Sheet 2 of 2

PROJECT **Fife to Military Road Stage 3**

Job No. **L-1922**

| Depth (ft) | Meters (m) | Profile | Standard Penetration Blows/ft (10 20 30 40) | SPT Blows/6" (N) | Sample Type | Sample No. (Tube No.) | Lab Tests | Description of Material | Groundwater | Instrument |
|---|---|---|---|---|---|---|---|---|---|---|
| | | | | 50/5" (50/5") | | D-8 | | Silty SAND with gravel, very dense, gray, moist, homogeneous. Retained 0.4 ft. | | |
| | 7 | | | | | | | | | |
| 25 | | | | | | | | | | |
| | 8 | | > > | 14 27 31 (58) | | D-9 | GS MC | SM, M.C. = 10% Silty SAND with gravel, very dense, gray, moist, homogeneous. Retained 1.0 ft. | | |
| | 9 | | | | | | | | | |
| 30 | | | | | | | | | | |
| | | | > > | 30 41 49 (90) | | D-10 | GS MC | SM, M.C. = 12% Silty SAND with gravel, very dense, gray, moist, homogeneous. Retained 1.0 ft. | | |
| | 10 | | | | | | | | | |
| 35 | | | | | | | | End of test hole boring at 33.5 ft. below ground elevation. This is a summary Log of Test Boring. Soil/Rock descriptions are derived from visual field identifications and laboratory test data. | | |
| | 11 | | | | | | | | | |
| | 12 | | | | | | | | | |
| 40 | | | | | | | | | | |
| | 13 | | | | | | | | | |
| 45 | | | | | | | | | | |

# LOG OF TEST BORING

Washington State Department of Transportation

HOLE No. H-16-94

PROJECT Fife to Military Road Stage 3 — Job No. L-1922

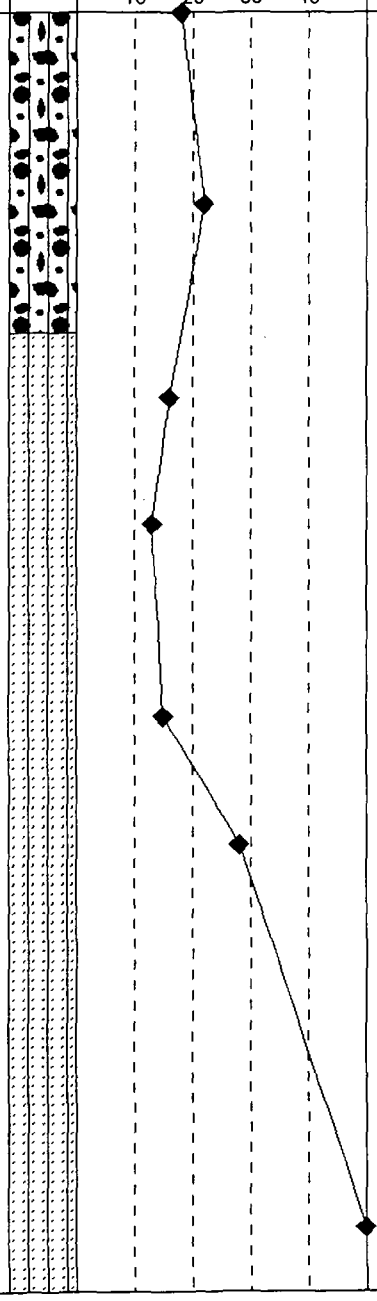
Bridge 5/503 — S.R. 5

Station LW 228+697.5 — Offset 15.2 m Lt. — C.S. 2719

Equipment — Casing 4" Augers to 48' — Ground El 109.4 m

Method of Boring Hollow Core Augers

Start Date December 1, 1994 — Completion Date December 1, 1994 — Sheet 1 of 3




| Depth (ft) | Meters (m) | SPT Blows/6" (N) | Sample Type | Sample No. (Tube No.) | Lab Tests | Description of Material | Groundwater | Instrument |
|---|---|---|---|---|---|---|---|---|
| | | 5<br>10<br>8<br>8<br>(18) | | D-1 | | Poorly graded GRAVEL with sand, medium dense, brown, disrupted. (Fill).<br>Retained 0.5 ft. | | |
| | 1 | 15<br>10<br>12<br>11<br>(22 | | D-2 | GS<br>MC | GM, M.C.=5%<br>Silty GRAVEL with sand, medium dense, gray, moist, homogeneous. Retained 0.8 ft. | | |
| 5 | | | | | | | | |
| | 2 | 6<br>7<br>9<br>12<br>(16) | | D-3 | GS<br>MC | SM, M.C.=6%<br>Silty SAND with gravel, medium dense, gray, moist, homogeneous.<br>Retained 0.9 ft. | | |
| | | 3<br>6<br>7<br>8<br>(13) | | D-4 | GS<br>MC | GM, M.C.=7%<br>Silty GRAVEL with sand silt, medium dense, gray, moist, homogeneous.<br>Retained 1.0 ft. | | |
| 10 | 3 | | | | | | | |
| | | 9<br>7<br>8<br>13<br>(15) | | D-5 | GS<br>MC | SM, M.C.=8%<br>Silty SAND with gravel, medium dense, gray, moist, homogeneous.<br>Retained 0.8 ft. | | |
| | 4 | 13<br>18<br>10<br>11<br>(28 | | D-6 | GS<br>MC | SM, M.C.=10%<br>Silty SAND with gravel, dense, gray, moist, homogeneous.<br>Retained 0.7 ft. | | |
| 15 | 5 | | | | | | | |
| | 6 | 54<br>50/4" | | D-7 | | Silty SAND with gravel, very dense, gray, moist, homogeneous.<br>Retained 0.8 ft. | | |
| 20 | | | | | | | | |

# LOG OF TEST BORING

Washington State Department of Transportation

HOLE No. **H-16-94**

Sheet 2 of 3

PROJECT **Fife to Military Road Stage 3**

Job No. L-1922

| Depth (ft) | Meters (m) | Profile | Standard Penetration Blows/ft (10 20 30 40) | SPT Blows/6" (N) | Sample Type | Sample No. (Tube No.) | Lab Tests | Description of Material | Groundwater | Instrument |
|---|---|---|---|---|---|---|---|---|---|---|
| | | | | (50/4" | | | | | | |
| | 7 | | | 12<br>30<br>50/5"<br>(80 | | D-8 | GS<br>MC | SM, M.C. = 5%<br>Silty SAND with gravel, very dense, gray, moist, homogeneous.<br>Retained 1.2 ft. | | |
| 25 | | | | | | | | | | |
| | 8 | | | | | | | | | |
| | | | | 30<br>50/4"<br>(50/4" | | D-9 | | Silty SAND with gravel, very dense, gray, moist, homogeneous.<br>Retained 0.8 ft. | | |
| | 9 | | | | | | | | | |
| 30 | | | | | | | | | | |
| | 10 | | | 50<br>50/2"<br>(50/2" | | D-10 | GS<br>MC | SM, M.C. = 9%<br>Silty SAND with gravel, very dense, gray, moist, homogeneous.<br>Retained 0.7 ft. | ▽ | |
| 35 | | | | | | | | | | |
| | 11 | | | | | | | | | |
| | | | | 19<br>21<br>40<br>(61) | | D-11 | | Silty SAND with gravel, very dense, gray, moist, homogeneous.<br>Retained 1.5 ft. | | |
| | 12 | | | | | | | | | |
| 40 | | | | | | | | | | |
| | 13 | | | 18<br>52/6<br>(52/6") | | D-12 | | Silty SAND with gravel, very dense, gray, moist, homogeneous.<br>Retained 1.0 ft. | | |
| 45 | | | | | | | | | | |

# LOG OF TEST BORING

Washington State Department of Transportation

HOLE No. **H-16-94**

Sheet **3** of **3**

PROJECT **Fife to Military Road Stage 3**

Job No. **L-1922**

| Depth (ft) | Meters (m) | Profile | Standard Penetration Blows/ft (10 20 30 40) | SPT Blows/6" (N) | Sample Type | Sample No. (Tube No.) | Lab Tests | Description of Material | Groundwater | Instrument |
|---|---|---|---|---|---|---|---|---|---|---|
| | 14 | | > > ◆ | 20<br>43/6<br>(43/6") | ▼ | D-13 | GS<br>MC | SM, M.C. = 10%<br>Silty SAND with gravel, very dense, gray, moist, homogeneous.<br>Retained 1.0 ft. | | |
| | 15 | | | | | | | End of test hole boring at 49.0 ft. below ground elevation.<br><br>This is a summary Log of Test Boring. Soil/Rock descriptions are derived from visual field identifications and laboratory test data. | | |
| 50 | | | | | | | | | | |
| | 16 | | | | | | | | | |
| 55 | | | | | | | | | | |
| | 17 | | | | | | | | | |
| | 18 | | | | | | | | | |
| 60 | | | | | | | | | | |
| | 19 | | | | | | | | | |
| 65 | | | | | | | | | | |
| | 20 | | | | | | | | | |
| | 21 | | | | | | | | | |
| 70 | | | | | | | | | | |

# LOG OF TEST BORING

Washington State Department of Transportation

HOLE No. H-17-94

PROJECT Fife to Military Road Stage 3 — Job No. L-1922

Bridge 5/503 — S.R. 5

Station LE 228+706.5 — Offset 16.8 m Rt. — C.S. 2719

Equipment — Casing 3" Augers to 22' — Ground El 110.0 m

Method of Boring Hollow Core Augers

Start Date October 26, 1994 — Completion Date October 26, 1994 — Sheet 1 of 2

| Depth (ft) | Meters (m) | Profile | Standard Penetration Blows/ft (10 20 30 40) | SPT Blows/6" (N) | Sample Type | Sample No. (Tube No.) | Lab Tests | Description of Material | Groundwater | Instrument |
|---|---|---|---|---|---|---|---|---|---|---|
| | | | | 6<br>10<br>18<br>23<br>(28) | | D-1 | GS<br>MC | SM, M.C.=5%<br>Silty SAND with gravel and root hairs, rounded, dense, brown, dry, disrupted, fill.<br>Retained 1.2 ft. | | |
| | | | | 30<br>42<br>(42/6") | | D-2 | | Silty SAND with gravel, rounded, very dense, gray, dry, homogeneous.<br>Retained 1.0 ft. | | |
| 5 | 1 | | | 53/6"<br>(53/6") | | D-3 | | Silty SAND with gravel, rounded, very dense, gray, dry, homogeneous.<br>Retained 0.5 ft. | | |
| | 2 | | | 83<br>50/2<br>(50/2") | | D-4 | GS<br>MC | GM, M.C.=6%<br>Silty GRAVEL with sand, rounded, very dense, gray, moist, homogeneous.<br>Retained 0.7 ft. | | |
| 10 | 3 | | | 38<br>47<br>(47/6") | | D-5 | | Silty GRAVEL with sand, rounded, very dense, gray, moist, homogeneous.<br>Retained 0.8 ft. | | |
| 15 | 4 | | | | | | | | | |
| | 5 | | | 50/5"<br>(50/5") | | D-6 | | Silty GRAVEL with sand, rounded, very dense, gray, moist, homogeneous.<br>Retained 0.5 ft. | | |
| 20 | 6 | | | | | | | | | |

# LOG OF TEST BORING

Washington State Department of Transportation

HOLE No. H-17-94

Sheet 2 of 2

PROJECT Fife to Military Road Stage 3

Job No. L-1922

| Depth (ft) | Meters (m) | Profile | Standard Penetration Blows/ft (10 20 30 40) | SPT Blows/6" (N) | Sample Type | Sample No. (Tube No.) | Lab Tests | Description of Material | Groundwater | Instrument |
|---|---|---|---|---|---|---|---|---|---|---|
| | | | | 50/2" (50/2") | | D-7 | | Silty GRAVEL with sand, rounded, very dense, gray, moist, homogeneous. Retained 0.5 ft. | | |
| | 7 | | | | | | | End of test hole boring at 22.2 ft. below ground elevation. | | |
| 25 | | | | | | | | This is a summary Log of Test Boring. Soil/Rock descriptions are derived from visual field identifications and laboratory test data. | | |
| | 8 | | | | | | | | | |
| | 9 | | | | | | | | | |
| 30 | | | | | | | | | | |
| | 10 | | | | | | | | | |
| 35 | | | | | | | | | | |
| | 11 | | | | | | | | | |
| | 12 | | | | | | | | | |
| 40 | | | | | | | | | | |
| | 13 | | | | | | | | | |
| 45 | | | | | | | | | | |

Elevation/Depth (ft)

Ground Elev. 102.9 m

Compact Medium Brown Silty All Grades Sand & Gravel With Scattered Boulders. Sand & Gravel: Rounded, Sub-Rounded, And Angular.

Very Dense Partially Cemented Brown-Grey All Grades Silty Sand & Gravel & Scattered Boulders. Sand & Gravel: Rounded, Sub-Rounded, Sub-Angular & Angular.

Cemented - Same As Above.

On Boulder - Blasted @ 13.0'

Silt Lense @ 15.7'

On Boulder - Blasted @ 17.0'

On Boulder - Blasted @ 19.0' & 20.0'

Very Dense Grey All Grades Sand & Fine Gravel. Sand Mostly Angular.

Very Dense Grey Fine & Medium Sand. Mostly Angular.

Total Depth Drilled 30.5'. Drilling Stopped As Cemented Glacial Till & Scattered Boulders Encountered Under A Sand Layer That Corresponds To The Silt Layer From 15.0' To 25.0' At Hole No. 5. No Trace Of Clayey Silt Found On Hole No. 8 To Correspond With Hole No. 5.

Strata In Adjacent Cut Lay Down Horizontally With Erratic Large Granite Boulders Up To 12 ft., Visible At This Location, Scattered On The Ground Surface.

TEST HOLE No. H-8

Sta L' LW 228+699 15.5 m LT

BORING H-8-58

Job No.

Hole No. **H-13-94**

Project

Date **February 15, 1995**

Sheet **1** of **1**

# Laboratory Summary

Washington State Department of Transportation

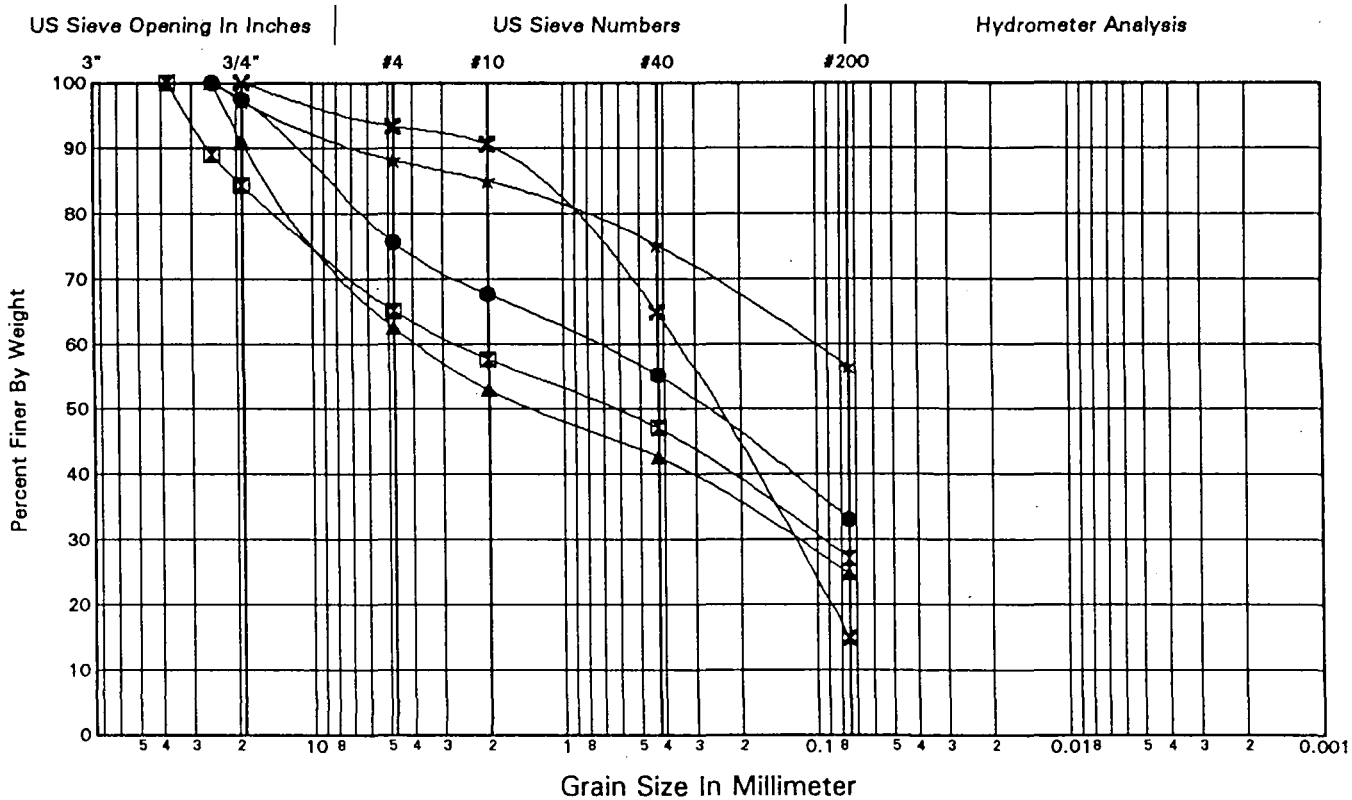
| | Depth (ft) | Depth (m) | Sample No. | USCS | Color | Description | MC% | LL | PL | PI |
|---|---|---|---|---|---|---|---|---|---|---|
| ● | 3.0 | 0.91 | D-2 | SM | GRAYISH BROWN | SILTY SAND with GRAVEL | 10 | NP | NP | NP |
| ⊠ | 23.0 | 7.01 | D-8 | SM | OLIVE | SILTY SAND with GRAVEL | 15 | NP | NP | NP |
| ▲ | 28.0 | 8.53 | D-9 | SM | DARK OLIVE GRAY | SILTY SAND with GRAVEL | 5 | NP | NP | NP |
| ★ | 43.0 | 13.11 | D-12 | ML | PALE OLIVE | SANDY SILT | 14 | NP | NP | NP |
| ✕ | 58.0 | 17.68 | D-15 | SM | OLIVE GRAY | SILTY SAND | 7 | NP | NP | NP |

## GRADATION FRACTIONS

| | %Gravel | %Sand | %Fines | Cu | Cc |
|---|---|---|---|---|---|
| ● | 24.4 | 42.5 | 33.1 | | |
| ⊠ | 34.8 | 38.1 | 27.1 | | |
| ▲ | 37.4 | 37.9 | 24.7 | | |
| ★ | 11.8 | 31.6 | 56.6 | | |
| ✕ | 6.7 | 78.3 | 15.0 | | |

## GRADATION VALUES

| | D60 | D50 | D30 | D20 | D10 |
|---|---|---|---|---|---|
| ● | 0.77 | 0.28 | | | |
| ⊠ | 2.61 | 0.66 | 0.10 | | |
| ▲ | 3.77 | 1.29 | 0.13 | | |
| ★ | 0.10 | | | | |
| ✕ | 0.36 | 0.25 | 0.13 | 0.09 | |




| Gravel | Sand | | | Silt and Clay |
|---|---|---|---|---|
| | Coarse | Medium | Fine | |

Job No.

Hole No. **H-14-94**

Project

Date **February 15, 1995**

Sheet **1** of **1**

# Laboratory Summary

Washington State Department of Transportation

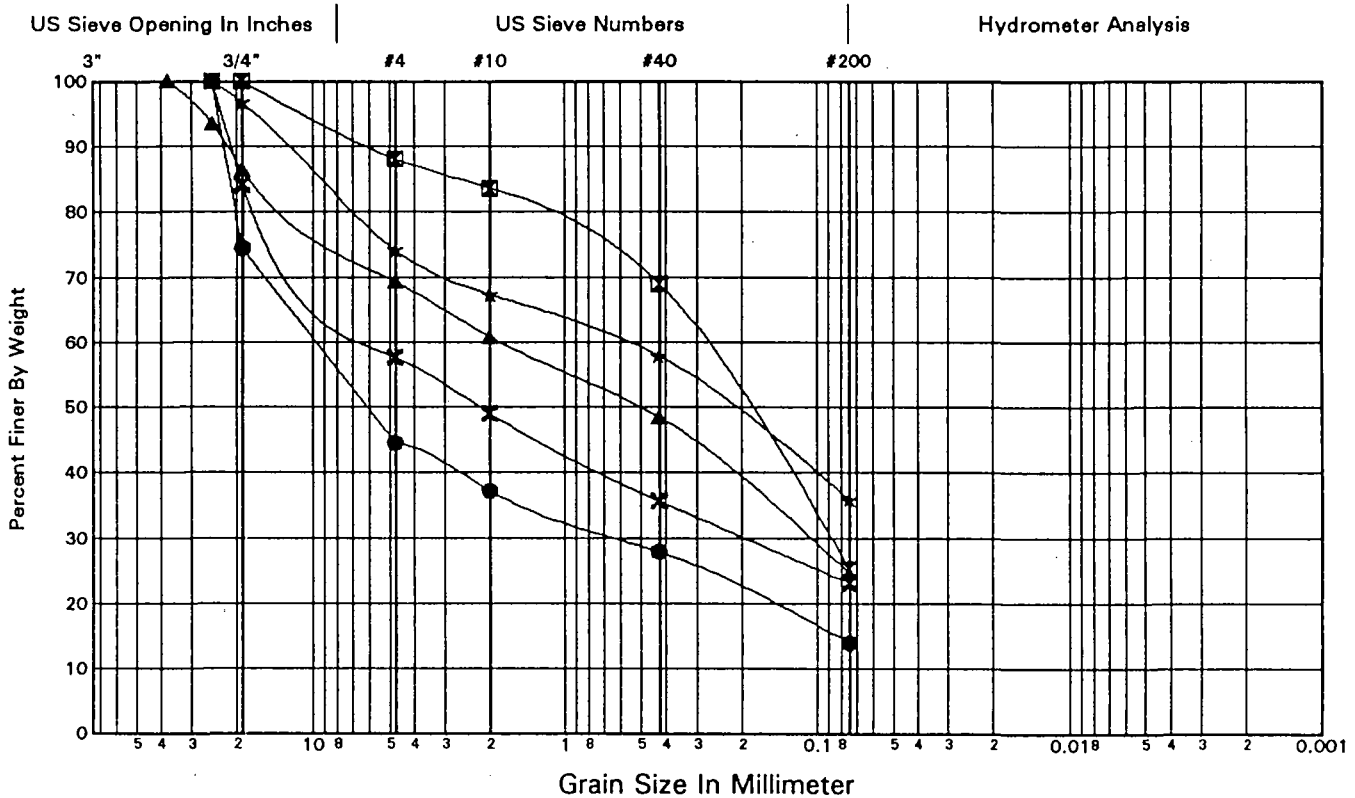
| | Depth (ft) | Depth (m) | Sample No. | USCS | Color | Description | MC% | LL | PL | PI |
|---|---|---|---|---|---|---|---|---|---|---|
| ● | 3.0 | 0.91 | D-1 | GM | LT YELLOWISH BROWN | SILTY GRAVEL with SAND | 3 | NP | NP | NP |
| ⊠ | 5.0 | 1.52 | D-2 | SM | LIGHT OLIVE GRAY | SILTY SAND | 7 | NP | NP | NP |
| ▲ | 8.0 | 2.44 | D-3 | SM | OLIVE GRAY | SILTY SAND with GRAVEL | 10 | NP | NP | NP |
| ★ | 15.0 | 4.57 | D-6 | SM | OLIVE GRAY | SILTY SAND with GRAVEL | 12 | NP | NP | NP |
| ✕ | 30.0 | 9.14 | D-9 | GM | OLIVE GRAY | SILTY GRAVEL with SAND | 8 | NP | NP | NP |

## GRADATION FRACTIONS

| | %Gravel | %Sand | %Fines | Cu | Cc |
|---|---|---|---|---|---|
| ● | 55.5 | 30.5 | 14.0 | | |
| ⊠ | 11.9 | 62.7 | 25.4 | | |
| ▲ | 30.6 | 44.7 | 24.7 | | |
| ★ | 25.8 | 38.3 | 35.9 | | |
| ✕ | 42.3 | 34.5 | 23.2 | | |

## GRADATION VALUES

| | D60 | D50 | D30 | D20 | D10 |
|---|---|---|---|---|---|
| ● | 9.72 | 6.13 | 0.60 | 0.16 | |
| ⊠ | 0.30 | 0.20 | 0.09 | | |
| ▲ | 1.81 | 0.51 | 0.11 | | |
| ★ | 0.60 | 0.23 | | | |
| ✕ | 5.36 | 2.21 | 0.19 | | |




| Gravel | Sand | | | Silt and Clay |
|---|---|---|---|---|
| | Coarse | Medium | Fine | |

Job No.

Hole No. H-15-94

Project

Date February 15, 1995

Sheet 1 of 1

# Laboratory Summary

Washington State Department of Transportation

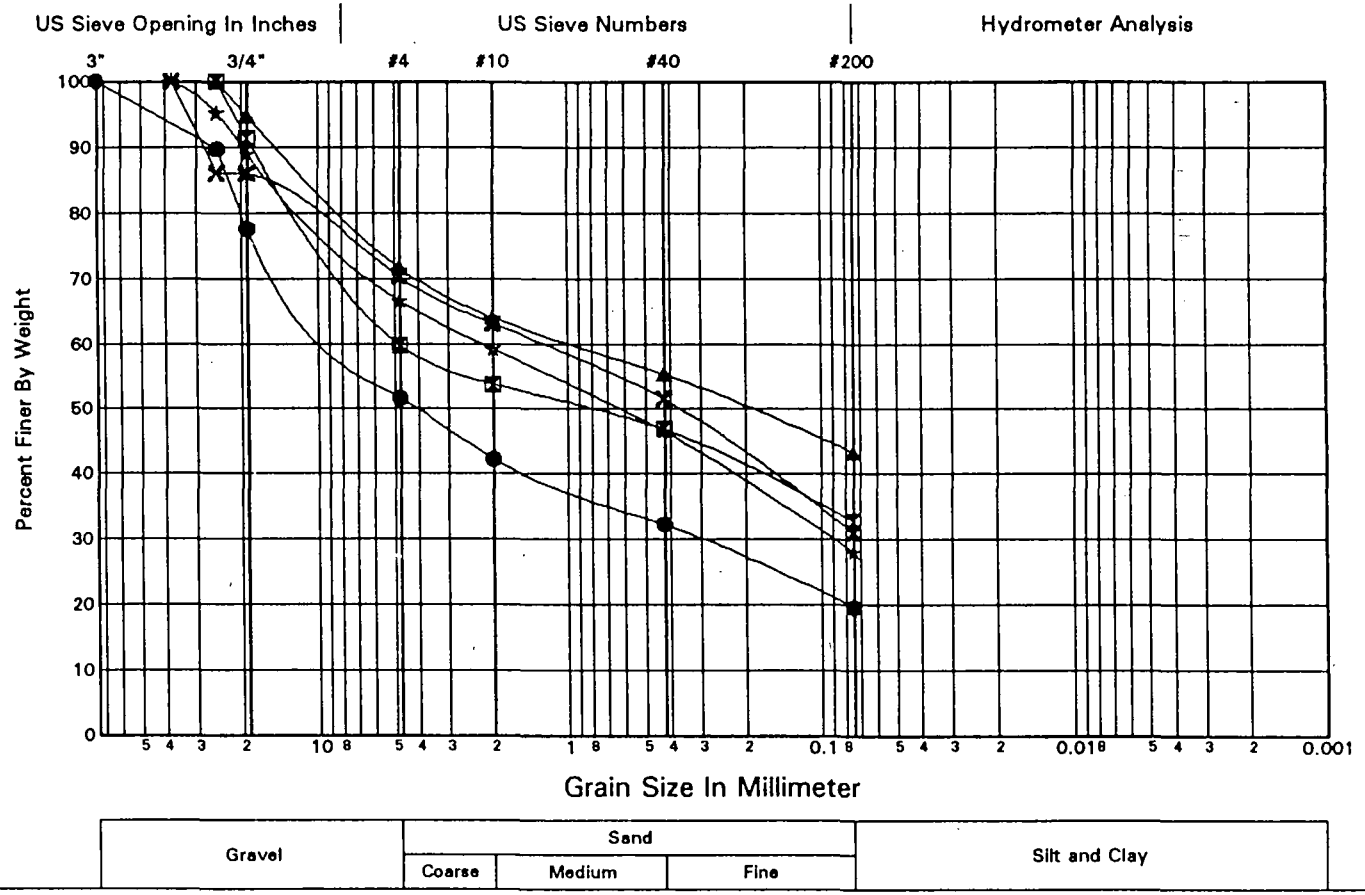
| | Depth (ft) | Depth (m) | Sample No. | USCS | Color | Description | MC% | LL | PL | PI |
|---|---|---|---|---|---|---|---|---|---|---|
| ● | 3.0 | 0.91 | D-1 | GM | OLIVE GRAY | SILTY GRAVEL with SAND | 13 | NP | NP | NP |
| ⊠ | 11.5 | 3.51 | D-5 | GM | P OLIVE TO OLIVE GRY | SILTY GRAVEL with SAND | 15 | NP | NP | NP |
| ▲ | 16.5 | 5.03 | D-7 | SM | LIGHT OLIVE GRAY | SILTY SAND with GRAVEL | 13 | NP | NP | NP |
| ★ | 26.5 | 8.08 | D-9 | SM | OLIVE GRAY | SILTY SAND with GRAVEL | 10 | NP | NP | NP |
| ✕ | 31.5 | 9.60 | D-10 | SM | GRAY | SILTY SAND with GRAVEL | 12 | NP | NP | NP |

## GRADATION FRACTIONS

| | %Gravel | %Sand | %Fines | Cu | Cc |
|---|---|---|---|---|---|
| ● | 48.4 | 32.0 | 19.6 | | |
| ⊠ | 40.1 | 27.2 | 32.7 | | |
| ▲ | 28.3 | 28.5 | 43.2 | | |
| ★ | 33.3 | 38.6 | 28.1 | | |
| ✕ | 29.7 | 39.3 | 31.0 | | |

## GRADATION VALUES

| | D60 | D50 | D30 | D20 | D10 |
|---|---|---|---|---|---|
| ● | 7.43 | 4.10 | 0.31 | 0.08 | |
| ⊠ | 4.77 | 0.86 | | | |
| ▲ | 0.96 | 0.20 | | | |
| ★ | 2.17 | 0.65 | 0.09 | | |
| ✕ | 1.31 | 0.37 | | | |



Job No.

Hole No. **H-16-94**

Project

Date **February 15, 1995**

Sheet **1** of **2**

# Laboratory Summary

Washington State Department of Transportation

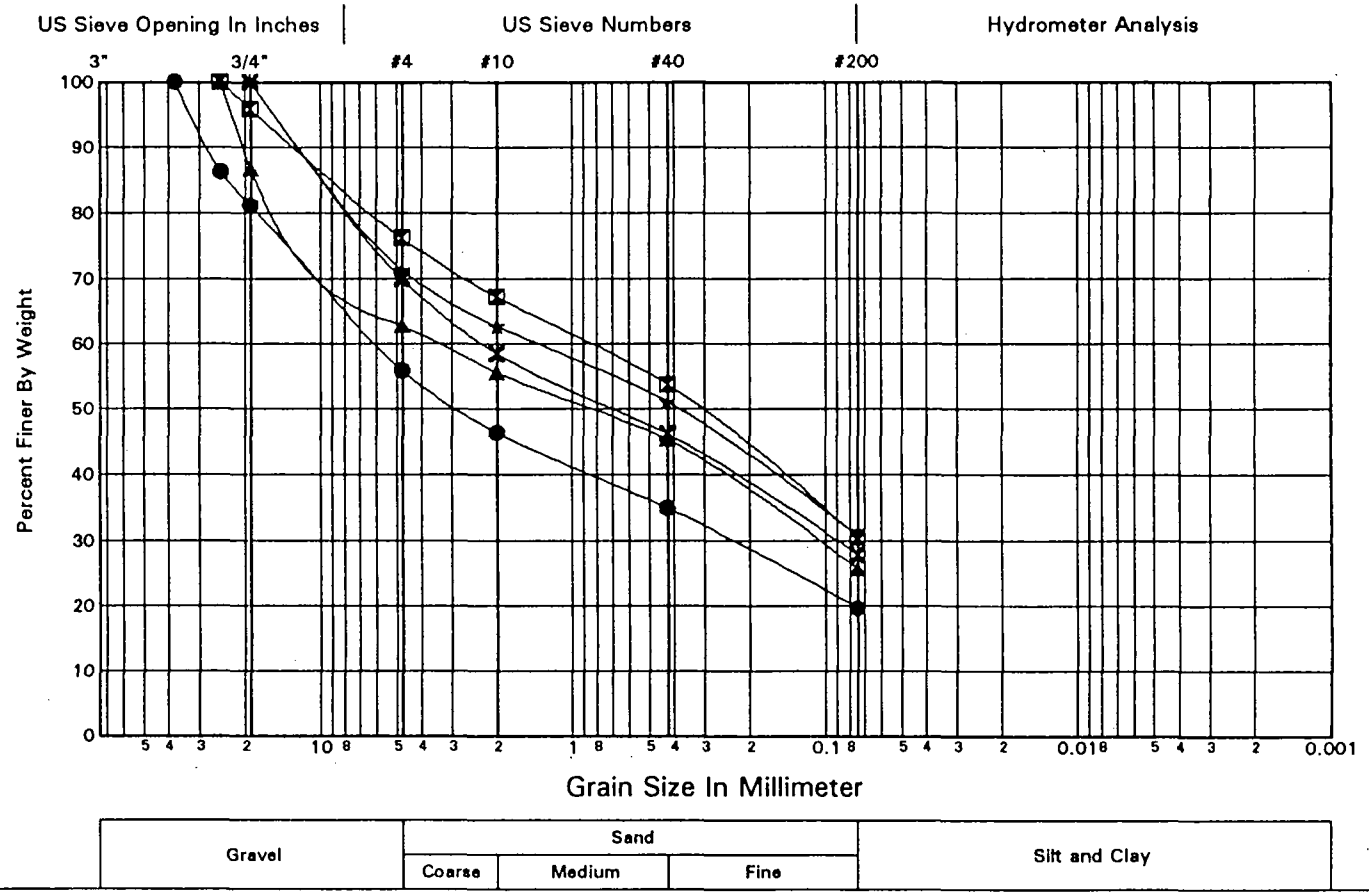
| | Depth (ft) | Depth (m) | Sample No. | USCS | Color | Description | MC% | LL | PL | PI |
|---|---|---|---|---|---|---|---|---|---|---|
| ● | 3.0 | 0.91 | D-2 | GM | OLIVE | SILTY GRAVEL with SAND | 5 | NP | NP | NP |
| ⊠ | 6.0 | 1.83 | D-3 | SM | LIGHT OLIVE BROWN | SILTY SAND with GRAVEL | 6 | NP | NP | NP |
| ▲ | 8.0 | 2.44 | D-4 | GM | OLIVE BROWN | SILTY GRAVEL with SAND | 7 | NP | NP | NP |
| ★ | 11.0 | 3.35 | D-5 | SM | LIGHT OLIVE BROWN | SILTY SAND with GRAVEL | 8 | NP | NP | NP |
| ✕ | 13.0 | 3.96 | D-6 | SM | GRAYISH BROWN | SILTY SAND with GRAVEL | 10 | NP | NP | NP |

## GRADATION FRACTIONS

| | %Gravel | %Sand | %Fines | Cu | Cc |
|---|---|---|---|---|---|
| ● | 44.1 | 36.2 | 19.7 | | |
| ⊠ | 23.8 | 45.6 | 30.6 | | |
| ▲ | 37.3 | 37.0 | 25.7 | | |
| ★ | 28.6 | 40.7 | 30.7 | | |
| ✕ | 30.0 | 42.1 | 27.9 | | |

## GRADATION VALUES

| | D60 | D50 | D30 | D20 | D10 |
|---|---|---|---|---|---|
| ● | 5.95 | 2.78 | 0.24 | 0.08 | |
| ⊠ | 0.87 | 0.32 | | | |
| ▲ | 3.42 | 0.86 | 0.11 | | |
| ★ | 1.40 | 0.39 | | | |
| ✕ | 2.24 | 0.69 | 0.09 | | |



Job No.

Hole No. **H-16-94**

Project

Date **February 15, 1995**

Sheet **2** of **2**

# Laboratory Summary

Washington State Department of Transportation

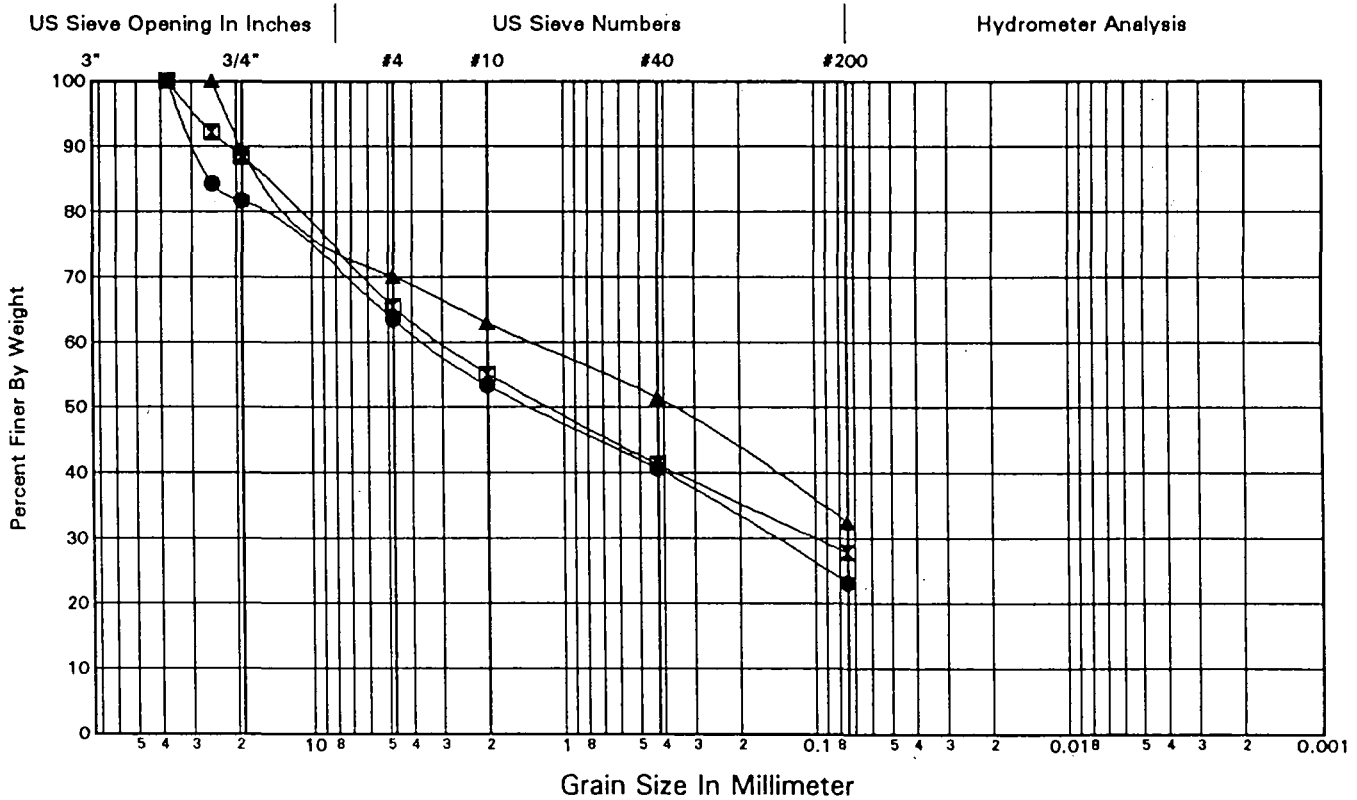
| | Depth (ft) | Depth (m) | Sample No. | USCS | Color | Description | MC% | LL | PL | PI |
|---|---|---|---|---|---|---|---|---|---|---|
| ● | 23.0 | 7.01 | D-8 | SM | OLIVE GRAY | SILTY SAND with GRAVEL | 5 | NP | NP | NP |
| ⊠ | 33.0 | 10.06 | D-10 | SM | LIGHT BROWNISH GRAY | SILTY SAND with GRAVEL | 9 | NP | NP | NP |
| ▲ | 48.0 | 14.63 | D-13 | SM | OLIVE GRAY | SILTY SAND with GRAVEL | 10 | NP | NP | NP |
| | | | | | | | | | | |
| | | | | | | | | | | |

## GRADATION FRACTIONS

| | %Gravel | %Sand | %Fines | Cu | Cc |
|---|---|---|---|---|---|
| ● | 36.5 | 40.3 | 23.2 | | |
| ⊠ | 34.5 | 37.7 | 27.8 | | |
| ▲ | 29.9 | 37.6 | 32.5 | | |
| | | | | | |
| | | | | | |

## GRADATION VALUES

| | D60 | D50 | D30 | D20 | D10 |
|---|---|---|---|---|---|
| ● | 3.53 | 1.34 | 0.15 | | |
| ⊠ | 2.99 | 1.12 | 0.10 | | |
| ▲ | 1.34 | 0.37 | | | |
| | | | | | |
| | | | | | |



Job No. L-1922
Hole No. H-17-94
Project

Date January 4, 1995
Sheet 1 of 1

# Laboratory Summary

Washington State Department of Transportation

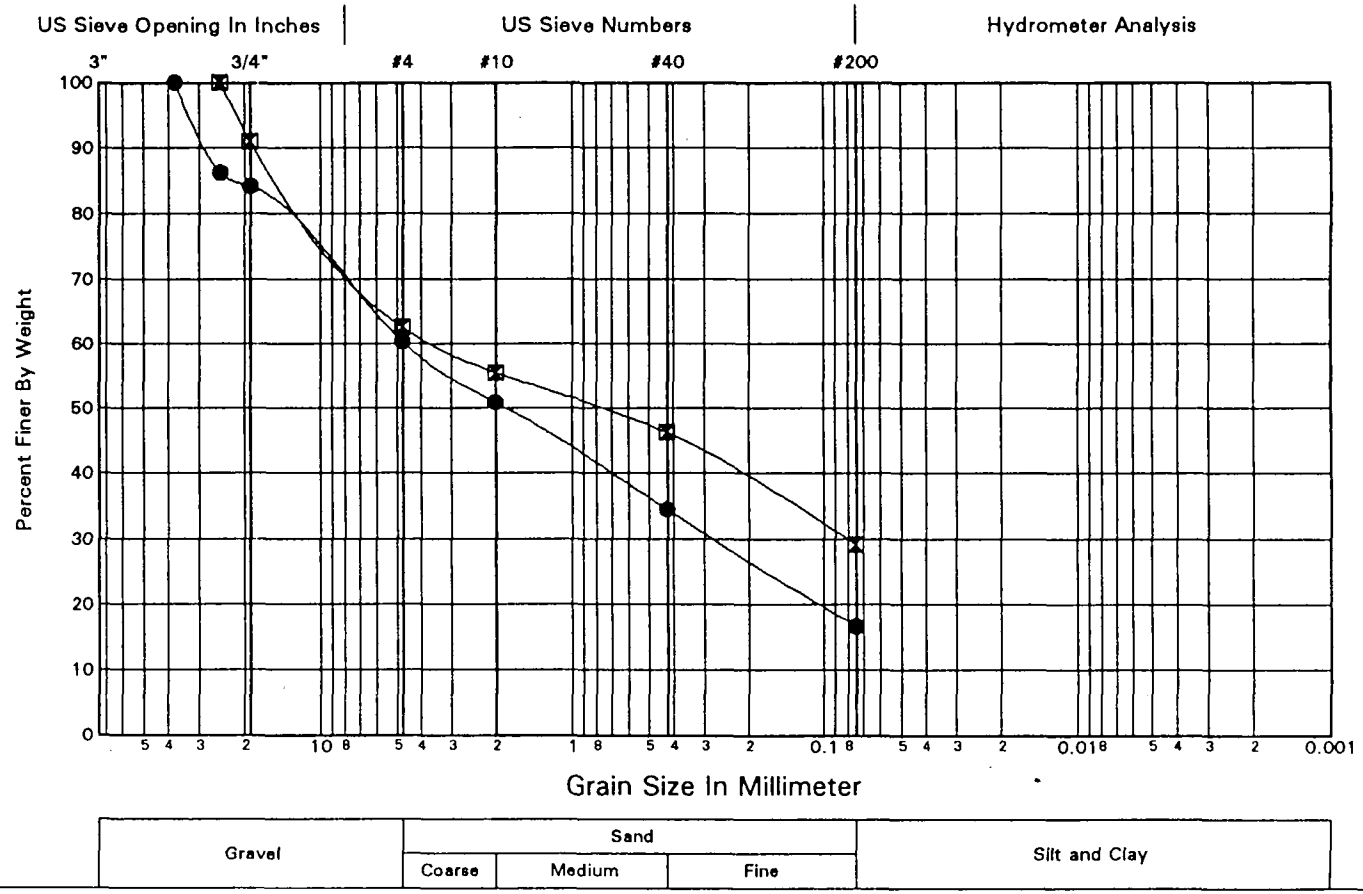
| | Depth (ft) | Depth (m) | Sample No. | USCS | Color | Description | MC% | LL | PL | PI |
|---|---|---|---|---|---|---|---|---|---|---|
| ● | 0.0 | 0.00 | D-1 | SM | OLIVE GRAY | SILTY SAND with GRAVEL w/root hairs | 5 | NP | NP | NP |
| ⊠ | 7.0 | 2.13 | D-4 | GM | LIGHT GRAY | SILTY GRAVEL with SAND | 6 | NP | NP | NP |

## GRADATION FRACTIONS

| | %Gravel | %Sand | %Fines | Cu | Cc |
|---|---|---|---|---|---|
| ● | 39.7 | 43.6 | 16.7 | | |
| ⊠ | 37.5 | 33.3 | 29.2 | | |

## GRADATION VALUES

| | D60 | D50 | D30 | D20 | D10 |
|---|---|---|---|---|---|
| ● | 4.62 | 1.84 | 0.27 | 0.10 | |
| ⊠ | 3.49 | 0.80 | 0.08 | | |



CHAPTER 3

# Area Geology and Seismicity

## Regional Geology

The project area is located in the Puget Sound Lowland, a region of low relief between the Olympic Mountains on the west and the Cascade foothills on the east. The structures are located on a large glacial plain formed by repeated advance and retreat of glaciers over the last 1.5 million years. The glacially deposited materials are estimated to be hundreds of meters thick.

The youngest continental ice sheet, the Vashon Lobe of the Fraser Glaciation, receded approximately 13,000 years ago. Prior to the advance of the Fraser Lobe, the area had been receiving large quantities of continental sediments, deposited as clay, silt, sand, and gravel. Sea level lowering during the advance of the Vashon glaciation caused a period of erosion that removed some or all of the glacial sediments. The Vashon ice sheet, roughly one kilometer thick, over-consolidated the remaining sediments. During the last glacial recession, some glacial debris, as well as the preglacia sediments, were removed. Thus there is a fairly complex series of glacial deposits throughout the project area.

The distribution of surface and near-surface geologic units across the project site, as shown in a published map (Waldron, 1961) is shown in Figure 1-1. The map shows that most of the project area is covered by Vashon till at or near the ground surface. However, the till is commonly overlain by poorly sorted and poorly stratified sand and gravel ranging from a few millimeters to more than 6 meters thick. This material is believed to be glaciaolacustrine or glaciofluvial outwash deposited during the melting of nearly stagnant glacial ice. In many places, the exposures of this recessional outwash are insufficient to delineate its extent, so it is mapped with the till deposit. Geologic mapping also shows small exposures of glacial lacustrine material and artificial fill near some of the bridge and wall locations.

The Vashon till is typically a very dense, non-sorted mixture of clay, silt, sand, gravel, and cobbles that has been over-consolidated by glacial ice. It typically has low permeability, high strength, and low compressibility. The recessional outwash is typically of moderate permeability. Because it has not been over-consolidated by the weight of glacial ice, it is typically medium-dense to dense. The glacial lacustrine materials are commonly hard, of very low permeability, and highly over-consolidated.

Advance outwash, deposited ahead of the advancing ice sheet, typically underlies the Vashon till. In some locations, the till may have been eroded so that advance outwash lies directly beneath recessional outwash.

# Seismicity

## Earthquake Sources

Future earthquakes affecting the site may result from:

- A large, relatively shallow earthquake (magnitude 8+) off the Washington coast on the Cascadia subduction zone
- Deep seismic activity on the subduction zone interface at depths of 40 to 70 kilometers
- Shallow seismic activity in the North American Plate

Records of seismic activity in the Pacific Northwest are limited mainly to the last 100 years and provide information on seismic events of the second and third sources only.

Earthquakes up to magnitude 7.1 have been observed along the subduction zone beneath the Puget Sound area at depths of 40 to 40 kilometers. A maximum magnitude of up to 7.5 is postulated for this source (Algermissen, 1982). The 1949 Olympia magnitude 7.1 earthquake and the 1965 magnitude 6.5 earthquake between Tacoma and Seattle are the two most recent events of this origin. The recurrence interval for these events is thought to range from 35 to 110 years.

The 1872 Lake Chelan earthquake is the largest shallow seismic event on record. It has been assigned a magnitude of 7.4 (Weaver and Shedlock, 1989) and is thought to be the approximate upper limit for shallow earthquakes in the North American Plate.

Noson, et al. (1988), postulated that the Cascadia subduction zone may create an offshore earthquake of magnitude 8+ on a recurrence interval of 300 to 1,000 years. Earthquakes of this magnitude and origin are found in the historic record, but their absence may not be conclusive, considering the short historic record and potentially long recurrence interval. Peterson (1989) reports geologic evidence of shoreline changes in the Pacific Northwest on recurrence interval of about 500 years and proposes that the sea level changes are evidence of tsunami caused by large offshore earthquakes.

## Design Ground Motions

The estimated peak firm-ground acceleration (PGA) from a magnitude 8+ Cascadia subduction zone event at the site could range from 0.15 to 0.35 g (g equals the acceleration of gravity) depending on the assumed earthquake location and the assumed attenuation of the seismic waves as they travel to the site (Youngs and Coopersmith, 1989; Somerville, et al., 1989). Given the estimated recurrence interval of 300 to 1,000 years, the probability of the Cascadia event at the project site in a 50-year design lift is roughly between 5 and 10 percent.

Earthquakes of magnitude 6.5 to 7.5 have a higher probability of occurrence during a 50-year project design life. The firm-ground PGA for these earthquakes ranges between about 0.1 to 0.3 g, depending on the proximity of the earthquake and its attenuation characteristics. In 1988, WSDOT and the Federal Highway Administration (FHWA) funded a seismic zonation study for the state of Washington. The study specifically considered the cumulative probability of ground accelerations resulting from shallow and deep seismic events.

Results of the WSDOT/FHWA study (Higgins, et al., 1988) indicate that the acceleration coefficient for the project area will be approximately 0.25 g. This coefficient has a 10 percent probability of occurrence in a 50-year design life. It is appropriate for firm-ground soil conditions.

Chapter 4
# Appendix

CHAPTER 4

# Bridge 5/503 East and West - SR 18

## Proposed Improvements

The location of the 5/503 bridges within the project area is shown on Figure 1-1. HOV widening of Bridge 5/503 East and West will add 5.38 m (17.7 ft) to the inside lanes (east side of west bridge and west side of east bridge) and 1.69 m (5.5 ft.) to the outside lanes. The location of the widening is shown in Figure 4-1. Figure 4-1 and subsequent figures follow the text at the end of this chapter.

## Existing Structures

The existing bridges are four-span, pre-stressed, concrete girder structures. The roadway width across the structures is 21.6 m (71.0 ft.). The span lengths are 10.7 m (35 ft.) 19.8 m (65 ft.), 19.8 m (65 ft.), and 13.1 m (43 ft) between piers 1 and 2, 2 and 3, 3 and 4, and 4 and 5, respectively.

The original construction plans indicate that the abutments for the east bridge are mostly native material; there is no fill shown at the pier 1 abutment and only 1.2 m (4 ft.) of fill on the west side of the pier 5 abutment. The original grade sloped downward to the west, so that the western bridge abutments required up to 7.9 m (26 ft.) of fill at the western edge of the structure.

The existing bridges are founded on spread footings. The original design bearing pressure is not indicated on the plans. Based on the topography shown in the original 1958 construction drawings, the existing footings were founded at elevations at least 1.2 m (4 ft.) into the native subgrade. The abutments for the east bridge are 3.5 m (11.5 ft.) wide strip footings extending across the entire width of the structure and founded only 2.0 m (6.6 ft.) below the roadway grade. The abutments on the west bridge are founded on spread footings up to 8.1 m (26.5 ft.) below the roadway grade. There is no abutment wall, and the abutment fill has been placed around the piers and graded to the finished 2H:1V abutment slopes.

## Site Information

The site is generally level except for grading changes to accommodate the interchange. The eastern (northbound) SR 5 roadway is roughly 0.8 m (2.5 ft.) higher than the western (southbound) SR 5 roadway. Detention ponds are present in the center of the ramp loops.

The existing SR 5 roadway has five lanes in each direction and a 23.2 m (71 ft.) median. The bridge can be accessed from the shoulders of SR 5. SR 18, beneath the structure, has 3 lanes in each direction. A median barrier separates the eastbound and westbound lanes of SR 18, so that access to the center pier 3 requires a lane closure. Median barriers also bound the outside shoulders of SR 18.

There are no drainage ditches along SR 18 in the vicinity of the structure. The abutment slopes are finished with concrete protection. Runoff from the median ditch of SR 5, the bridge drains, and catch basins along SR 18 is carried in storm drains located just outside the shoulders of SR 18. The storm drains daylight to surface ditches near the edges of the abutment slopes.

# Subsurface Information

One boring was drilled for the original design in 1958 (H-8-58) and five additional borings were drilled in 1994 (H-13-94 through H-17-94). A piezometer was installed in one of the borings (H-15-94). The locations of the borings are shown in Figure 4-1. Boring logs are attached at the end of this section. Laboratory tests consisted of 25 natural moisture contents and 25 grain size analyses. The laboratory test results are also attached at the end of this section.

The native subsurface materials are generally very dense, unsorted, silty sand with gravel or silty gravel with sand. The fines content (percentage finer than the No. 200 sieve) typically ranges between 15 and 35 percent and the natural moisture content is between 5 and 15 percent. The gravel component ranges from subrounded to angular. The materials are fairly consistent between samples and borings except for a 4.5 m (15 ft.) thick layer of sandy silt with gravel in boring H-13-94. The site geology and material descriptions suggest these native materials are glacial till of the Vashon area.

Glacially deposited materials always have the potential for containing cobbles and boulders. Refusal blow counts and poor sample recovery in the till suggest that large gravel, cobbles, or boulders are present. The presence of boulders at the site is further confirmed by references to "blasted boulders" at three depths in the original design boring, H-8-58. The original design boring also noted "erratic large granite boulders up to 3.7 m (12 ft.) in diameter scattered on ground surface".

Although only three samples of the abutment fill material were tested in the laboratory, the test results and visual descriptions indicate that the fill materials have gradations very similar to the native material, however, the SPT N values indicate that they are medium-dense.

Groundwater was measured at depths ranging between 4.0 m (13 ft.) and 1.49 m (4.9 ft.) below the SR 18 roadway, corresponding to approximately elevations 97.9 m (321.3 ft.) and 100.4 m (329.4 ft.). Because of the low permeability of glacial till, groundwater commonly is perched near the surface, on top of the unweathered till. However, there are frequently zones or lenses of more permeable material that carry water through the deposit, therefore design of this structure has been based on the assumption of saturated conditions below elevation 100.4 m (329.4 ft.).

# Methods of Analyses

## Engineering Properties

Engineering properties for the various materials have been estimated at each pier location. The estimates are based on SPT N values, material gradation, correlations to index

properties, and local experience in similar soils. SPT values are highly influenced by gravel, as is present in the subsurface materials. Therefore, the SPT values were not corrected for overburden stress or hammer efficiency.

## Shallow Foundations

Allowable bearing pressures for spread footings were determined from bearing capacity and settlement analyses. Bearing capacity was computed in accordance with Article 4.4 of the AASHTO Standard Specifications for Highway Bridges (1992). Allowable bearing pressures recommended in the next sections are based on a combination of bearing capacity and 25 mm (1 inch) allowable settlement. The subsurface materials are either granular or highly overconsolidated, so settlement is anticipated to be elastic and to occur immediately after the loads are applied.

Immediate settlements were checked by several methods readily calculated by the program CSANDSET (U.S. Army Corps of Engineers, 1990). Where the depth of a relatively uniform layer below each boring was at least two footing widths, elastic settlement was estimated by elastic half-space methods. The elastic half-space methods computed by CSANDSET are variations of the general method described in Article 4.4.7.2.2 of the AASHTO (1993) specifications. Settlement was predicted by the methods described by Schmertmann (1978) where soil layer thickness was less than two footing widths. The Schmertmann method was also used as a check on the infinite half-space methods.

## Drilled Shaft Foundations

Axial capacity of drilled shafts has been calculated in accordance with the methods outlined in the FHWA design manual (Reese and O'Neill, 1988). A factor of safety of 2.5 has been applied to the computed ultimate shaft friction and end-bearing capacities to obtain the allowable capacity for compressive loading. Thus, the allowable axial capacity is:

$Q_{a,\ compression} = (Q_s + Q_b)/2.5$

where $Q_s$ = ultimate skin friction

$Q_b$ = ultimate end-bearing

A factor of safety of 2.5, rather than the commonly used 3.0 has been used because the shafts are anticipated to penetrate granular fill above the water table and very dense, cohesive till below the water table and disturbance to the formation due to shaft construction is expected to be minimal.

For allowable uplift, the factor of safety is 1.5. No reduction in skin friction for cone breakout has been applied. Allowable uplift was computed as:

$Q_{a,\ uplift} = Q_s/1.5 + W$

where W = weight of the shaft

Ultimate skin friction was computed by the Beta method within the fill and by the undrained strength, or alpha method, in the glacial till. Beta is a function of depth and overburden pressure only. At the abutments, the top 1.2 m (4 ft.) of subgrade, representing the depth of the structure, was ignored in computing effective overburden stress. The tops of the abutment shafts were assumed to be 2.4 m (8 ft.) below finished grade. Skin friction in the top and bottom one-diameter length along the shaft was ignored in accordance with the

methods outlined by Reese and O'Neill. Ultimate skin friction within the refusal-blowcount till was set at the limiting value of 260 kPa (5.5 ksf) recommended by Reese and O'Neill.

The design values for skin friction are based on soil-to-concrete contact. Therefore, installation methods must follow WSDOT's master specifications for drilled shaft installation which intended to maintain the integrity of the soil around the shaft and prevent build-up of a filter cake.

An undrained analysis was used to determine the ultimate end-bearing within the till. A limiting value of 3800 kPa (80 ksf) was used for ultimate end-bearing.

Allowable loads were determined by the above methods for several toe elevations. The allowable loads were also checked against a 25 mm (1 in.) maximum settlement criteria. Settlement of shafts bearing in the very dense till were assumed to consist entirely of shaft movement required to mobilize skin friction and end-bearing under working loads.

### Embankment Stability

Recommendations for maximum embankment slopes are based on observations of existing slopes and infinite slope analyses of fill material. The assumed angle of internal friction for new fill is 35 degrees and the moist unit weight is 20 $kN/m^3$ (125 pcf) . Because of the relatively level terrain, no global stability analyses were conducted.

### Lateral Earth Pressures

Active and passive earth pressure coefficients were computed with the Coulomb equation, ignoring wall friction, as recommended by AASHTO. Concrete to soil friction for determining sliding resistance has been estimated as tan (2/3 • phi). A phi angle of 35 degrees has been assumed beneath all footings. A factor of safety of 1.5 was applied to static passive earth pressure coefficients to limit structural movement.

### Seismic Design

Dynamic shear modulus for seismic design of shallow foundations was computed for strains on the order of $10^{-4}$ percent by equations developed by Seed et al. (1986), then adjusted to strains in the range of $10^{-2}$ to $10^{-1}$ percent, similar to those induced by an earthquake, by procedures recommended by these same authors. Poisson's ratio for use in seismic design was estimated from material type and density.

Active earth pressure coefficients for seismic analysis were computed by the Mononabe-Okabe equation, as recommended by AASHTO (1992).

## Recommendations

### Foundation Types

Spread footings are recommended for all bridge piers. Conditions also appear to be suitable for drilled shafts at all piers. Information is provided for 1.52 and 1.83 m (5- and 6-ft.) shaft alternatives, to minimize temporary shoring of the abutments and avoid utilities near the interior piers.

## Soil Properties

The engineering properties used for design are included in Table 4-1. The table references soil types, which refer to numbered soil layers on the subsurface profile of Figure 4-2. Smaller values of Young's Modulus were used for the till in the abutments to account for stress relief and lack of confinement.

**Table 4-1**
**Design soil properties - Bridge 5/503 E&W**

| Soil Type (refer to Fig. 4-2) | Simplified Description | Moist Unit Weight, kN/m³ (pcf) | Saturated Unit Weight, kN/m³ (pcf) | Effective Angle of Internal Friction, degrees | Undrained Cohesive Strength, kPa (ksf) | Young's Modulus, kPa (ksf) |
|---|---|---|---|---|---|---|
| 1 | Southern Abutment Fill | 19.6<br>(125) | -<br>- | 33 | -<br>- | 12,900<br>(270) |
| 2 | Northern Abutment Fill | 19.6<br>(125) | -<br>- | 35 | -<br>- | 15,800<br>(330) |
| 3 | Till on Cut Slope | 20.4<br>(130) | 20.7<br>(132) | -<br>- | 480<br>(10) | 47,900<br>(1,000) |
| 4 | Till Below SR 18 Grade | 20.4<br>(130) | 20.7<br>(132) | -<br>- | 480<br>(10) | 239,400<br>(5,000) |

## Allowable Bearing for Spread Footings

Table 4-2 shows the recommended allowable bearing pressures. In most cases, the design criteria limiting total settlement to one inch, controls the allowable pressure. The allowable bearing pressures in Table 4-2 also greatly reduce the potential for inducing additional settlement of the existing footings. Settlement is anticipated to be elastic, occurring as the load is placed. The maximum elevations in the table represent the shallowest footing elevations based on assumed structural limitations. These elevations were used in the settlement estimates. The allowable bearing pressures are not valid if footing elevations are set above the maximum elevations because settlement will increase.

If footings are located at least one footing width from the face of the embankment slope, the allowable bearing pressures in Table 4-2 will also provide protection from punching shear-type failures.

**Table 4-2**
**Allowable Bearing Pressures For Spread Footings**

| Bridge | Pier | Maximum Elevation, meters (feet) | Allowable Bearing Pressure, kPa (ksf) | Comments |
|---|---|---|---|---|
| 5/503W | 1 west | 106.5 (349.4) | 144 (3) | |
| 5/503W | 1 east | 106.5 (349.4) | 144 (3) | 190 kPa (4 ksf ok if B> 3.7 m (12 ft.) and max. elev. = 104.9 m (342.4 ft.) |
| 5/503W | 2 west & east | 100.7 (330.4) | 718 (15) | match existing |
| 5/503W | 3 west & east | 102.6 (333.4) | 718 (15) | match existing |
| 5/503W | 4 west & east | 100.7 (330.4) | 718 (15) | match existing |
| 5/503W | 5 west & east | 107.2 (351.9) | 239 (5) | |
| 5/503E | 1 west & east | 107.7 (353.6) | 479 (10) | match existing |
| 5/503E | 2 west & east | 100.9 (334.4) | 718 (15) | match existing |
| 5/503E | 3 west & east | 102.2 (335.4) | 718 (15) | match existing |
| 5/503E | 4 west & east | 102.2 (335.4) | 718 (15) | match existing |
| 5/503E | 5 west & east | 108.2 (355.0) | 479 (10) | match existing |

Vertical Datum = NAVD88

## Drilled Shaft Alternative

Figures 4-3 through 4-11 show the allowable capacity in compression and uplift for 1.52 m (5 ft.) and 1.83 m (6 ft.) diameter shafts for various toe elevations. As noted previously, factors of safety of 3.0 and 1.5 were used to compute allowable axial compression and uplift, respectively. Table 4-3 shows the assumed shaft head elevations used to predict capacity for the various toe elevations. If the actual shaft head elevations are significantly lower than these numbers, shaft capacities should be reevaluated. The maximum, or highest, toe elevation provided at each pier location in the table is the maximum elevation permissible to limit settlement to 25 mm (1 in.). The figures should not be interpolated beyond these maximum elevations to shorten shaft lengths for lighter loads.

The skin friction used to compute the allowable loads in the figures is based on the assumption that shaft concrete will be poured directly against relatively undisturbed soil in the shaft walls. Permanent casings should not be used without reducing the recommended capacities shown in the figures. If permanent casing is used, the space between the casing and the surrounding soil must be grouted over the full length of the casing and allowable skin friction must be reduced by one-third.

Temporary casing may be used to maintain a stable excavation during shaft construction. The existing footings appear to be founded in very dense glacial till. Support of excavations into the till below the water table can probably be stabilized with drilling fluid.

Exel tech - pre-cast girders



While the contractor may choose to use temporary casings as a means to protect the existing structure during shaft construction, they should not be required.

**Table 4-3**
**Shaft Head Elevations Assumed for Capacity Analyses**

| Bridge | Pier | Assumed shaft head elevation, meters (feet) |
|---|---|---|
| 5/503W | 1W, 1E | 106.5 (349.4) |
| 5/503W | 2W, 2E, 3W, 3E, 4W, 4E | 102.8 (337.4) |
| 5/503W | 5W, 5E | 107.2 (351.9) |
| 5/503E | 1W, 1E, 5W, 5E | 107.4 (352.4) |
| 5/503E | 2W, 2E, 3W, 3E, 4W, 4E | 102.2 (335.4) |

Vertical Datum = NAVD88

Design soil properties for use in lateral analyses are shown on the attached COM624 forms. Values of initial tangent modulus of subgrade reaction ($k_s$) and strain corresponding to stress at one-half the maximum total-principal-stress difference ($\varepsilon_{50}$) were estimated from published correlations with relative density (Reese and Wang, 1993). Some judgment should be used in applying these numbers.

## Embankment Slopes

Recommended embankment slopes of 2H:1V or flatter are recommended. The new fill should be keyed into the existing fill by cutting benches into the existing embankments, as specified in WSDOT's standard specifications. Concrete slope protection matching the existing slope protection is recommended to keep raveling material beneath the bridges from sloughing onto the SR 18 roadway.

## Lateral Earth Loads

The equivalent static lateral earth loads imposed on abutment walls, columns, and footings should be computed as follows:

$$P_h = 0.5\ (\text{unit weight})(\text{wall height})^2 K$$

Where: K = the appropriate active ($K_a$), passive ($K_p$), or at-rest ($K_o$) static coefficient of lateral earth pressure

A unit weight of 19.6 kN/m$^3$ (125 pcf) should be assumed for backfill materials.

The load $P_h$ will act at one-third the wall height from the base. The following coefficients of lateral earth pressure are recommended.

### Within 2H:1V Sloped Embankments (Piers 1 and 5)

- Passive, $K_p$ = 0.9
- At-rest, $K_o$ = 0.5

At-rest pressures should be used for computing loads on the back of columns and footings within the sloped embankment.

### On Level Ground (Piers 2, 3, and 4)

- Active, $K_a$ = 0.3
- Passive, $K_p$ = 2.5
- At-rest, $K_o$ = 0.5

A factor of safety of 1.5 has been applied to $K_p$ to limit movement. The top 1 meter (3 ft.) of soil should be neglected in computing passive resistance.

Lateral pressures generated by surcharge loads should be added to the lateral loading produced by the backfill. Lateral loading is computed by multiplying the vertical pressure at depth by the appropriate lateral earth pressure coefficient. If the surcharge is uniform, it should be applied uniformly between the top and the base of the wall. If the surcharge is limited in lateral extent, such as a large sign foundation, the lateral load applied to the vertical member will vary with depth. The only known surcharge loads at this time are traffic loads, which should be treated as a 0.6 m (2 ft.) soil surcharge.

A friction factor of 0.4 should be used for computing frictional resistance to sliding.

## Seismic Design

The peak, firm ground acceleration for this area is 0.25 (Higgins et al., 1988). This acceleration has a 10 percent probability of being exceeded in a 50-year design period.

Design response spectra as outlined in the AASHTO guide specifications are appropriate for this site. A Type II soil profile should be used for determining the site coefficient. Liquefaction is not a concern at this site.

A Poisson's ratio of 0.3 is recommended for use in determining soil spring constants for spread footings. Table 4-4 provides recommended values of dynamic shear modulus, G, for strains ranging from $10^{-1}$ to $10^{-2}$ percent for spread footing design.

**Table 4-4**
**Recommendations For Dynamic Shear Modulus**

| Bridge | Pier | G at $10^{-2}$% strain MPa (ksf) | G at $10^{-1}$% strain MPa (ksf) |
|---|---|---|---|
| 5/503W | 1 | 34 (700) | 14 (300) |
| 5/503W | 5 | 38 (800) | 17 (350) |
| 5/503W & 5/503E | 2, 3, 4 | 96 (2000) | 38 (800) |
| 5/503E | 1, 5 | 72 (1500) | 34 (700) |

Lateral earth loads on abutments and footings as a result of seismic loading should be computed as follows:

$$P_{he} = 0.5(\text{unit weight})(\text{wall height})^2 K_{ae}$$

Where $K_a$ = The combined static and seismic active earth pressure coefficient.

~~$K_{ae}$ = 0.65 at Piers 1 and 5 (within sloped embankment)~~ Deleted 5/26/04 dh

$K_{ae}$ = 0.35 at Piers 2, 3, and 4 (on level ground)

The combined static and seismic lateral earth load, $P_{he}$, can be assumed to act at mid-wall height as a result of a uniform pressure distribution.

A static lateral earth pressure coefficient without a factor of safety applied should be used to compute the resistance to combined static and seismic loading. Passive earth pressure coefficients of 0.9 and 2.5 are recommended for the abutment and interior piers, respectively.

## Other Considerations

Allowable bearing pressures for all footings on the east bridge and the interior footings of the west bridge are based on the assumption that the foundations will be founded on very dense, undisturbed glacial till. If this material is not encountered at the design elevations it will be necessary to either:

1. Check the actual design bearing pressure against an estimated allowable pressure for the materials encountered (i.e. consult the geotechnical and structural engineer to verify the adequacy of design), or;
2. Overexcavate until very dense, undisturbed till is encountered and backfill with lean concrete or crushed surfacing base course compacted to 95 percent of maximum density as specified in Section 2-03(14)C, Method C in the WSDOT Standard Specifications.

Footings should have at least 0.6 meters (2 ft.) of cover for frost protection. The subgrade should be firm and free of cobbles and organic material. Where shallow foundations are founded in the undisturbed native material (all piers on the east bridge and the interior piers of the west bridge) the subgrade is anticipated to be sufficiently dense to require no additional preparation. However, the exposed till subgrade will be moisture-sensitive and should be protected from weather and construction traffic. Within the fill the subgrade should be scarified to a minimum depth of 150 mm (6 in.) and recompacted to at least 95 percent of maximum density as specified in Section 2-03(14)C, Method C in the WSDOT Standard Specifications.

Placement of a leveling course of base rock immediately following foundation excavation or preparation is recommended to protect the foundation soil. A 50- to 75-mm (2- to 3-inch) thick, lean concrete slab may be used in lieu of the base rock leveling course.

Gravel Backfill for Walls should be used within 1 m (3 ft.) of the abutment walls. Positive drainage in the form of an Underdrain Pipe surrounded by Gravel Backfill for Drains should also be provided at the base of abutment walls. Gravel Backfill for Foundations should be used for backfill around the interior piers.

Approach slabs may be deleted because of the following geotechnical criteria:

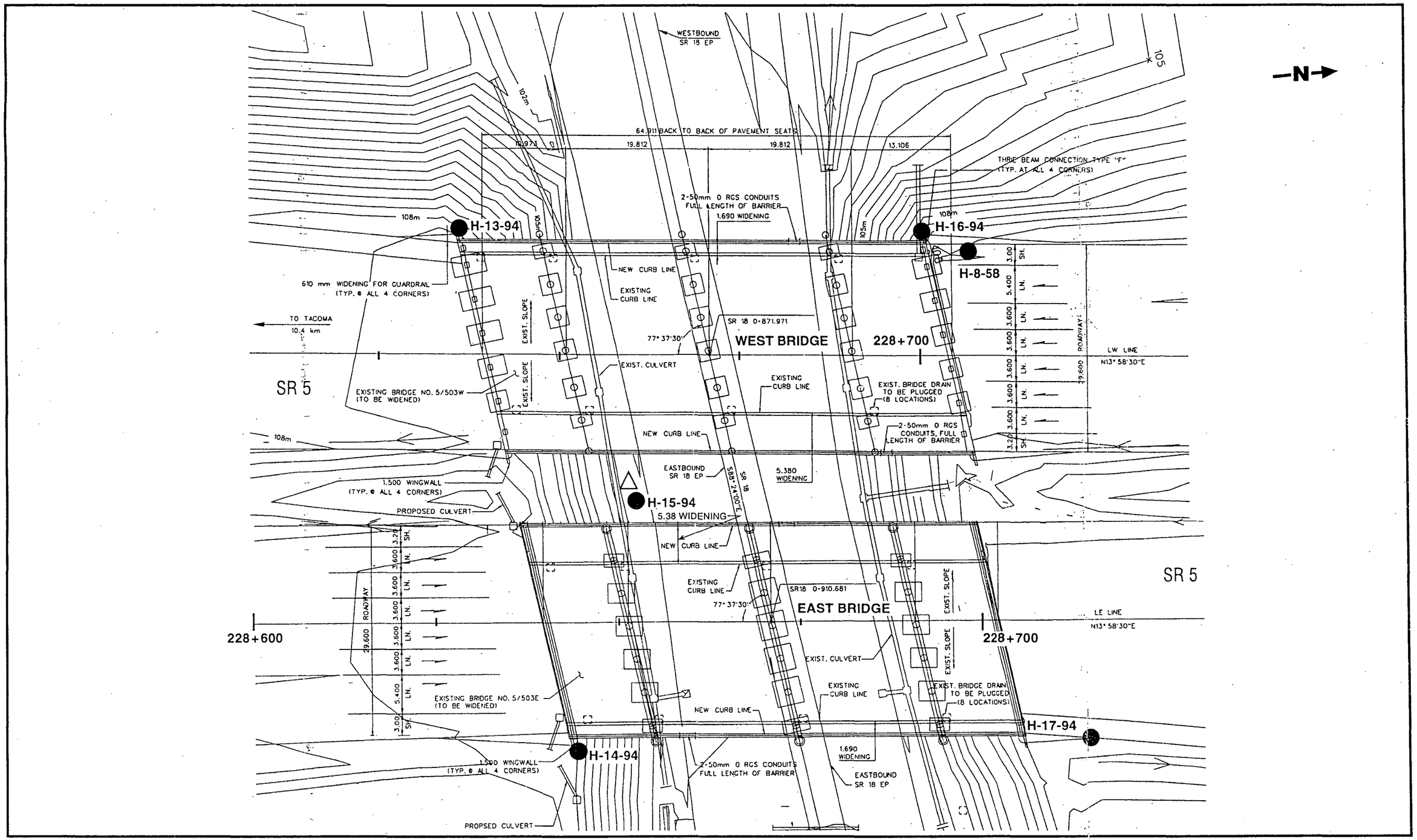
1. Post-construction settlement of the embankment is anticipated to be less than 13 mm (0.5 in.)
2. Creep settlement is not anticipated.

Groundwater has been measured within 150 mm (0.5 ft.) of the bottom of some existing footings. Prolonged rainy periods or heavy runoff could bring the groundwater to higher levels. Perched groundwater is frequently encountered within glacial till and small quantities should be anticipated even above the seasonal water table. If groundwater is encountered, excavations should be dewatered to maintain excavation stability and subgrade strength.

## Advisory Specifications

The contractor should be aware of potentially difficult conditions that may affect drilled shaft construction. Glacially deposited materials, such as those present at this site, frequently contain cobbles and boulders that may impede drilling progress. Test borings had refusal blow counts and poor sample recovery; these are commonly indicators of large gravel, cobbles, or boulders. The original construction boring referenced "blasted boulders" in three locations and noted "erratic granite boulders up to 3.7 m (12 ft.) in diameter" scattered on the ground surface. Drilled shafts will also extend below the measured ground water table. Excavation stabilization measures that may include the use of temporary casing, slurries, and/or application of increased hydraulic pressures to control shaft sidewall stability may be required. The boring logs and geotechnical report should be consulted for subsurface exploration details.

116184.G3.02 • 503 E/W Plan • 6-28-96 • LW




Approximate
Scale in Meters

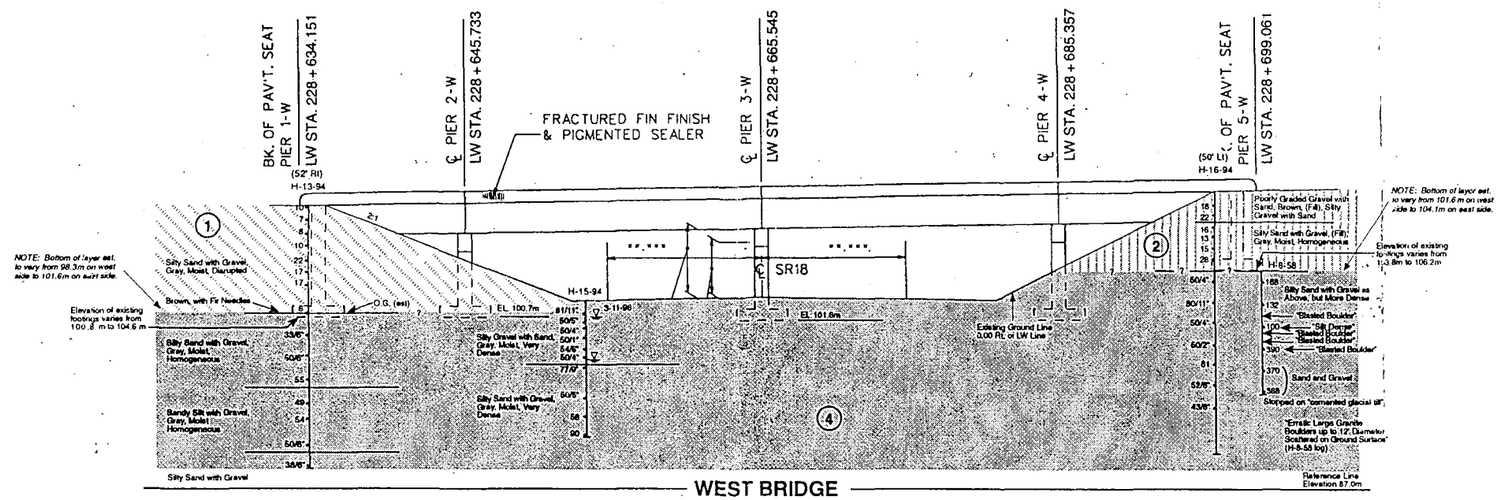
0 20 m

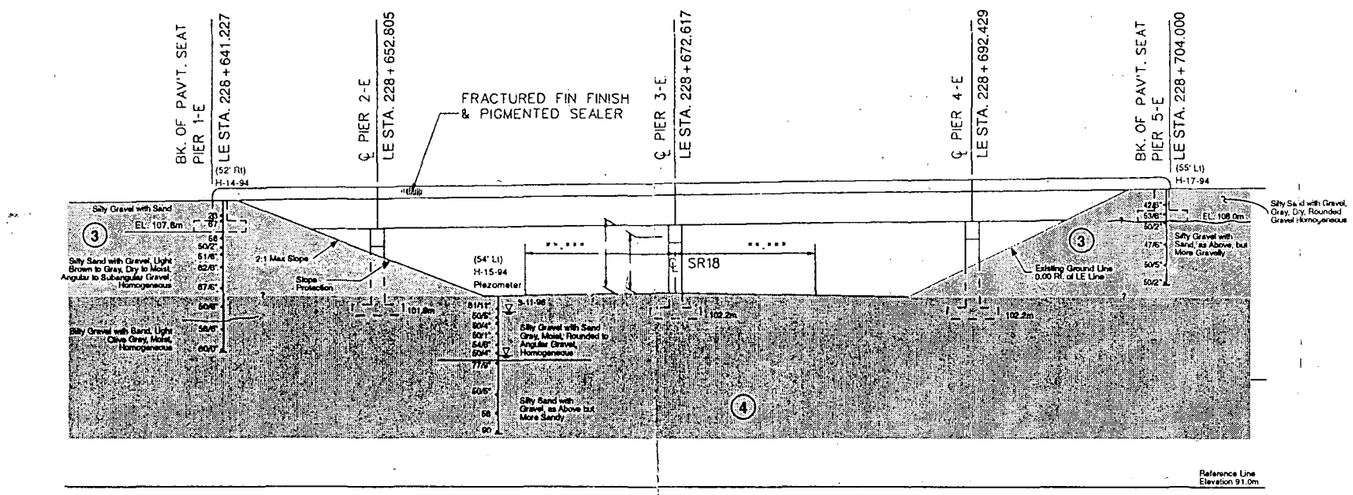
● Test Hole

△ Piezometer

*NOTES:*
1 meter = 3.28 feet
Drawing adapted from Preliminary Bridge Plans dated March 15, 1996.
Final plans may vary.

FIGURE 4-1
**Bridge 5/503 East and West Plan View**
Geotechnical Report SR-5
Port of Tacoma I/C to 272nd
Stage 4 HOV




**LEGEND**

1, 2, 3, 4 — Soil units used to assign design soil properties. See Table 4-1 for listing of assumed properties

Approximate Scale in Meters: 0 – 10 m

*NOTES:*
1 meter = 3.28 feet
Vertical Datum = NAVD88

FIGURE 4-2
**Bridge 5/503 East and West Profile**
Geotechnical Report SR-5
Port of Tacoma I/C to 272nd
Stage 4 HOV

**Bridge 5/503W Pier 1W**

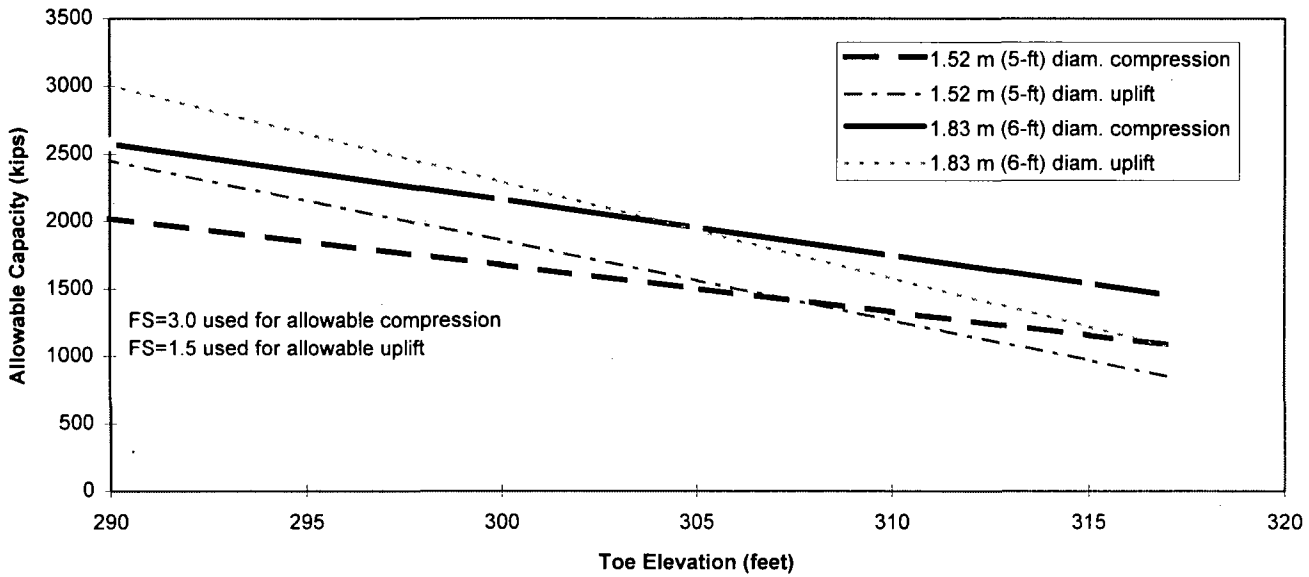



**Figure 4-3**

9/24/96
AXIAL CAPACITY NAVD88 Chart 1

## Bridge 5/503W Pier 1E

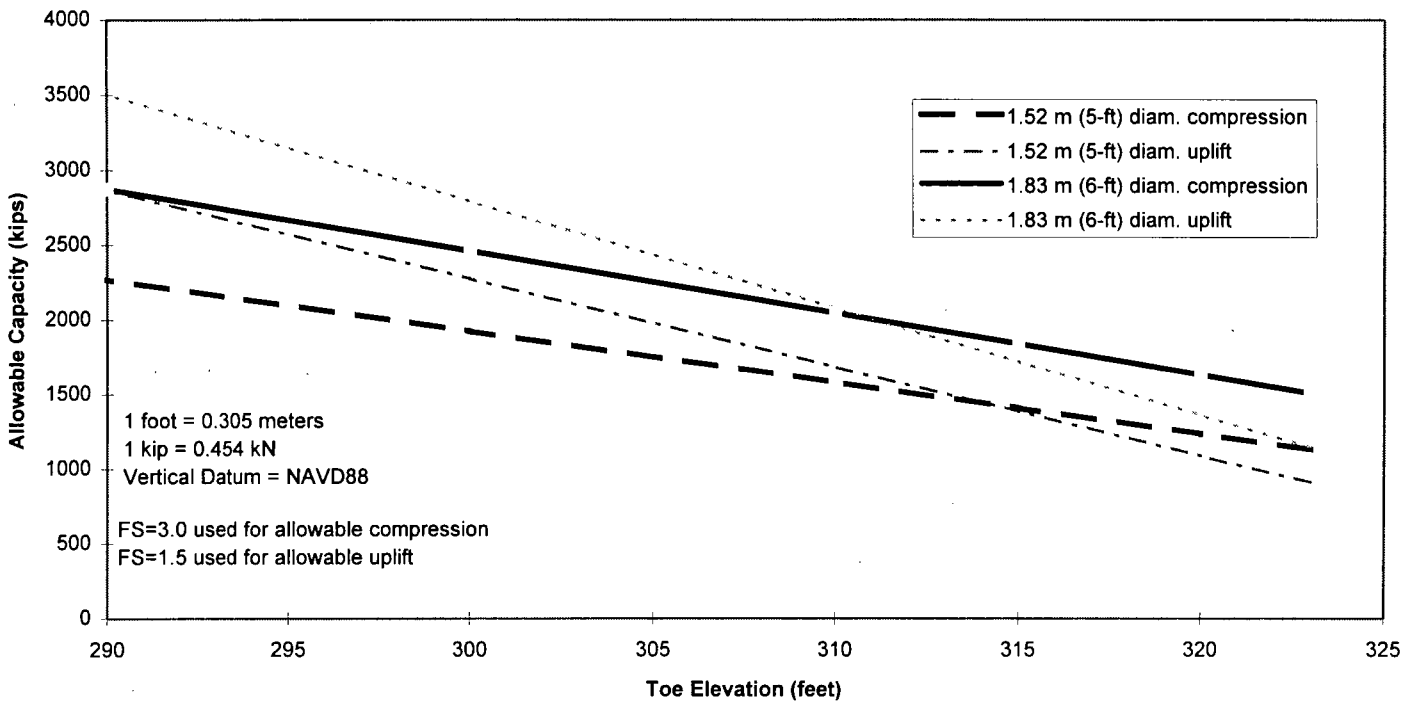



**Figure 4-4**

9/24/96
AXIAL CAPACITY NAVD88 Chart 2

**Bridge 5/503W Pier 2W**

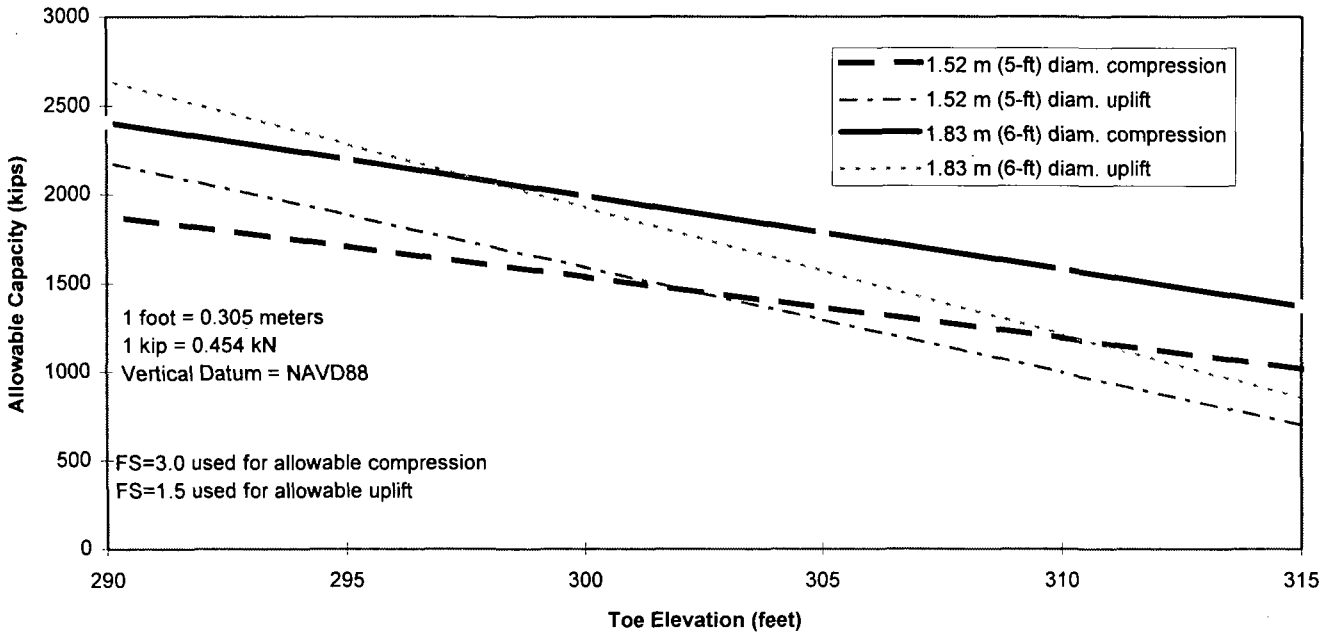



**Figure 4-5**

**Bridge 5/503W Pier 2E**

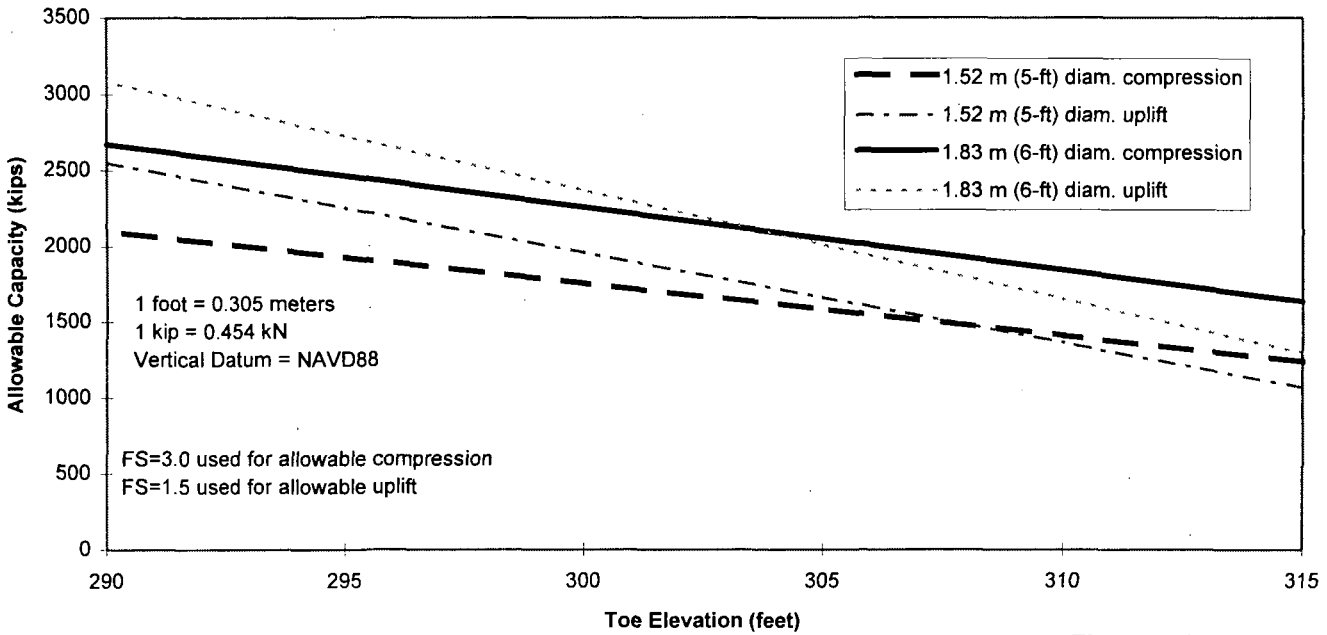



**Figure 4-6**

9/24/96
AXIAL CAPACITY NAVD88 Chart 6

**Bridge 5/503W Piers 3E&W, 4E&W**

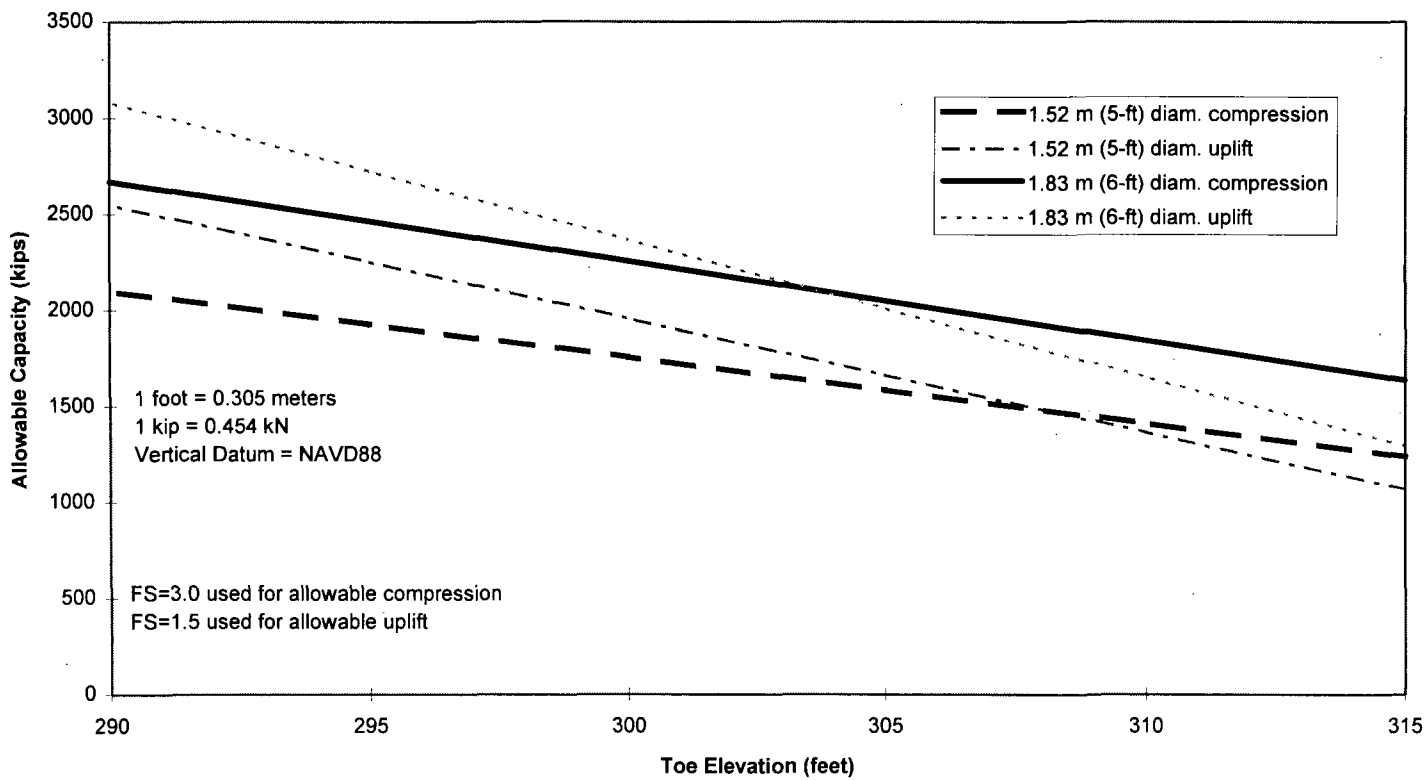



**Figure 4-7**

9/24/96
AXIAL CAPACITY NAVD88 Chart 4

## Bridge 5/503W Pier 5W

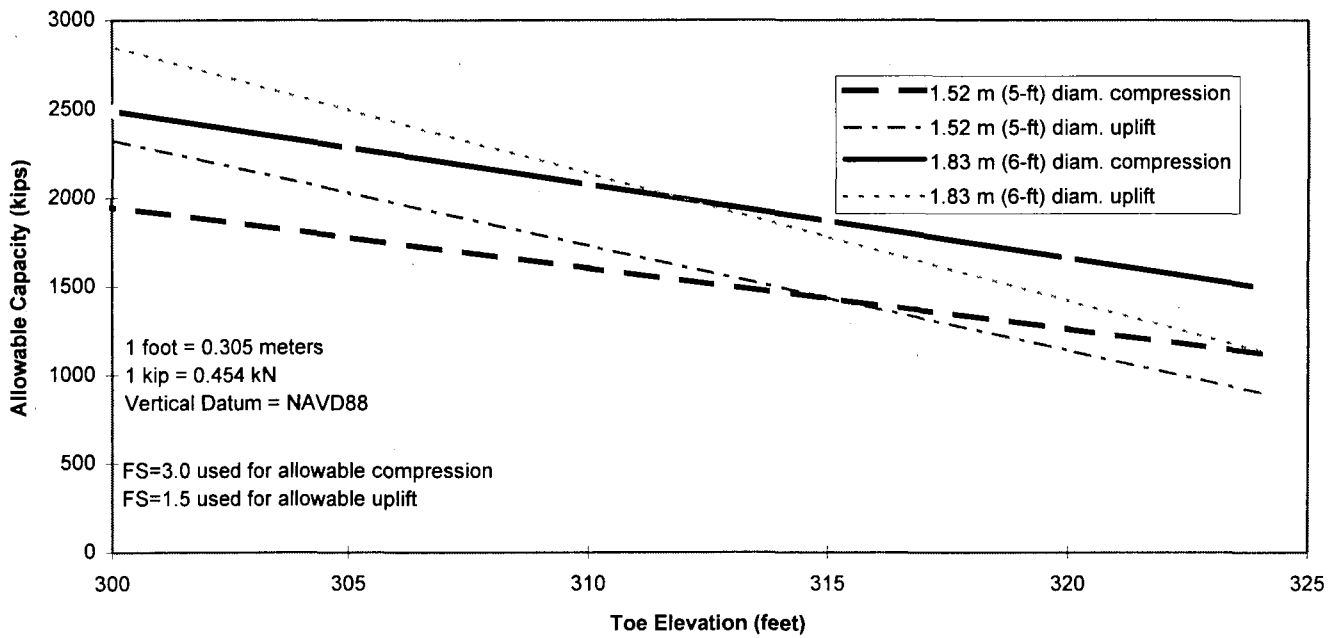



**Figure 4-8**

9/24/96
AXIAL CAPACITY NAVD88 Chart 7

**Bridge 5/503W Pier 5E**

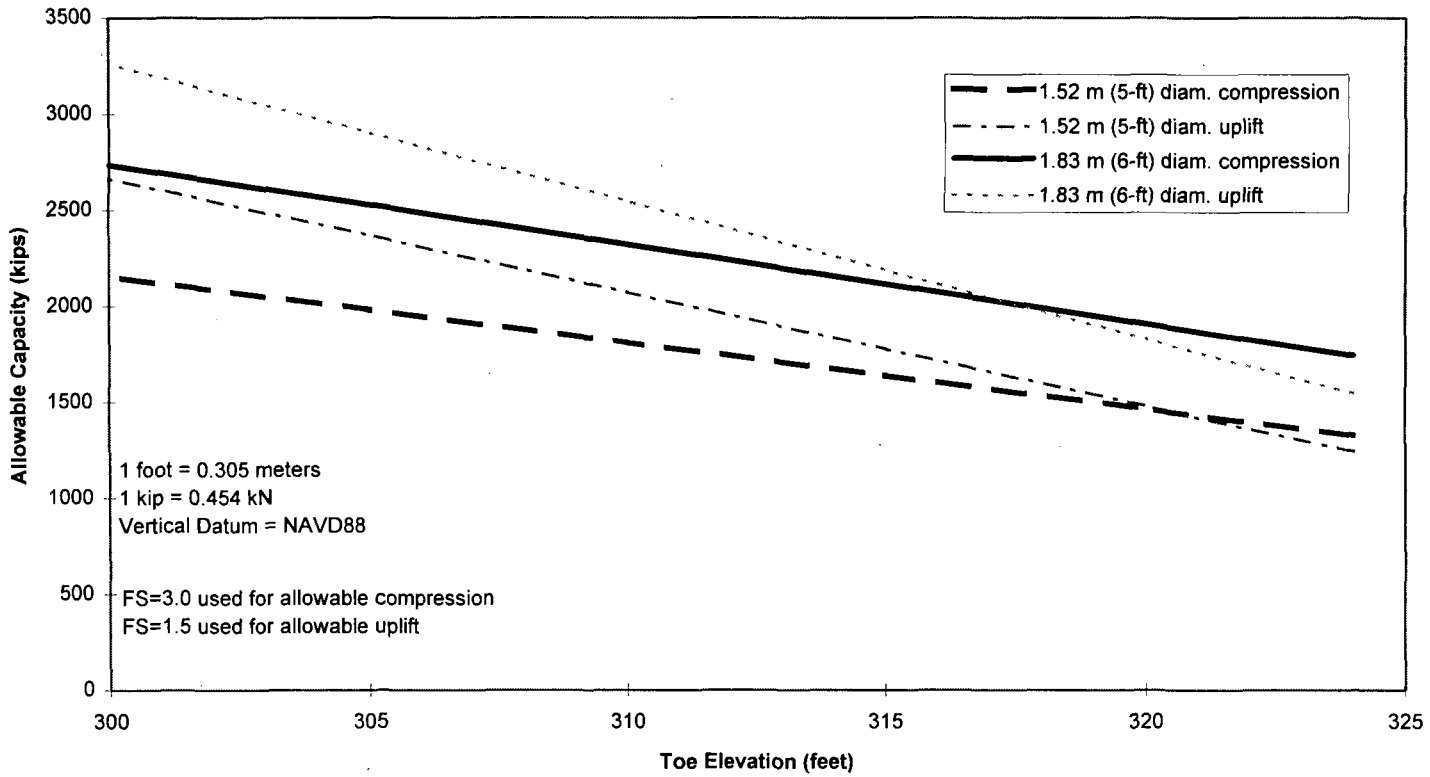



**Figure 4-9**

9/24/96
AXIAL CAPACITY NAVD88 Chart 5

## Bridge 5/503E Abutments (Piers 1 &4)

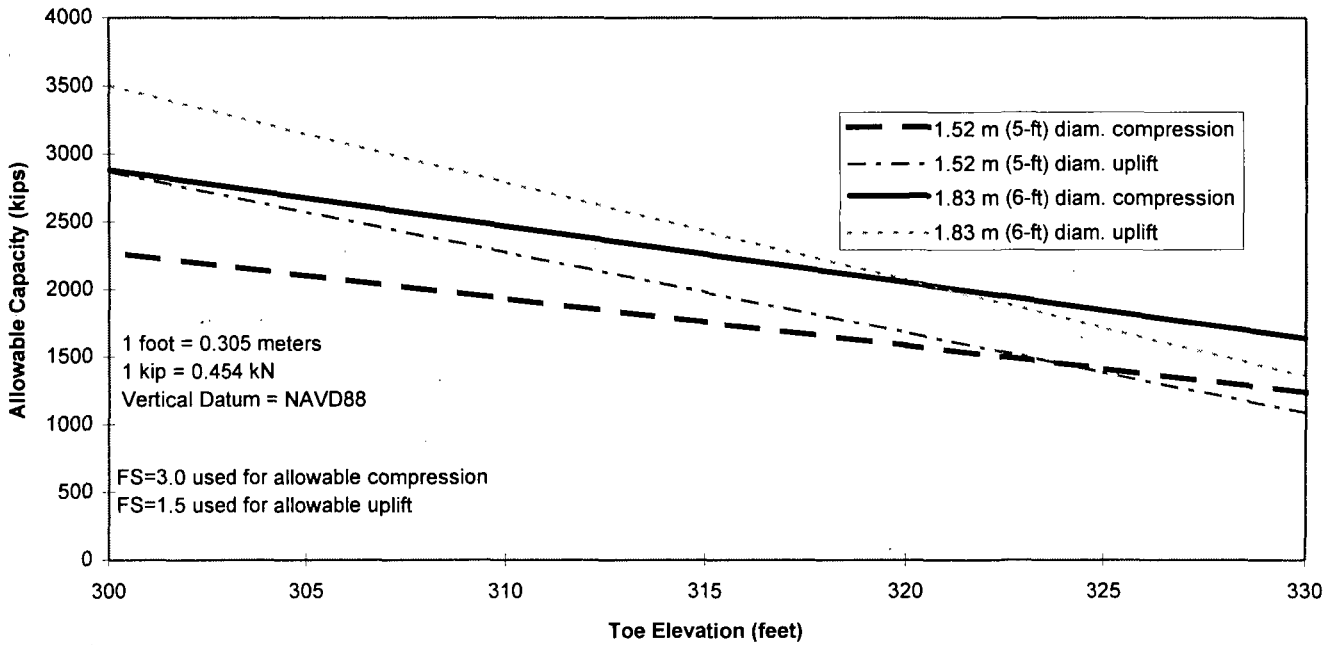



**Figure 4-10**

9/24/96
AXIAL CAPACITY NAVD88 Chart 8

**Bridge 5/503E Interior (Piers 2, 3, & 4)**

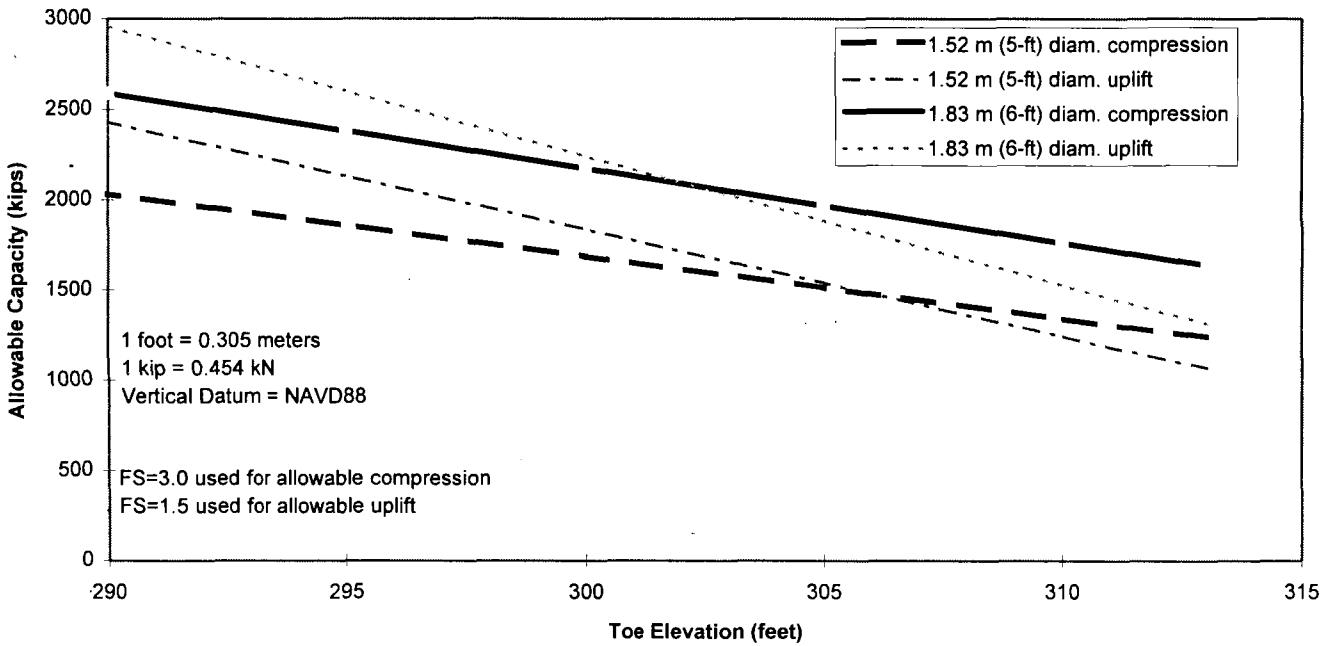



**Figure 4-11**

9/24/96
AXIAL CAPACITY NAVD88 Chart 9

Chapter 5
# Appendix

CHAPTER 5

# Bridge 5/504 East and West - South 336th Street Overcrossing

## Proposed Improvements

HOV widening of Bridge 5/504 East and West will add about 5.48 m (18 ft) to the inside lanes (east side of West bridge and west side of East bridge) and 1.52 m (5 ft.) to the outside lane of the West bridge. All lane widening will occur for the entire full length of the bridges. The location of the proposed widening is shown in Figure 5-1. Figure 5-1 and subsequent figures follow the text at the end of this chapter.

## Existing Structures

The existing bridges are three-span concrete box structures. The roadway width across the structures is 15.9 m (52.0 ft.). The span lengths for the East bridge are 18.2 m (60 ft.), 23.8 m (78 ft.), and 18.2 m (60 ft.) between piers 1 and 2, 2 and 3, and 3 and 4, respectively. For the West bridge, the span lengths are 14.4 m (47 ft.), 18.6 m (61 ft.), and 14.4 m (47 ft.) between piers 1 and 2, 2 and 3, and 3 and 4, respectively.

The existing piers are supported on spread footings founded at the elevations shown in Figure 5-2. From the construction plans provided by WSDOT, the original ground surface sloped slightly downward to the north. Fill thicknesses at the south (Pier 1) and north (Pier 4) abutments of the East bridge are 7 ft. and 27 ft., respectively. For the West bridge, the abutments are on 8 to 12 ft. of fill. Underlying the fill is a medium-dense to very dense native outwash material. The interior piers (Piers 1 and 2) are mostly founded on native material. Except for Pier 4 of East bridge, the existing footings are currently founded into the native advance outwash material.

The elevation of the existing SR 5 roadway at the East and West bridges is 120.1 m (394 ft.) and 117.3 m (385 ft.), respectively. The existing spread footings are 2.4 m (8 ft.) to 3.0 m (10 ft.) wide. The original (1959) construction plans indicate that the footings were designed for an allowable soil bearing pressure of 290 kPa (6 ksf).

## Site Information

Except for grading changes to accommodate the interchange, the project site is located on a fairly level terrain. The existing SR 5 roadways at the bridge consist of five lanes in each direction. From the design drawings provided by WSDOT, the existing ground surface at the East bridge (northbound SR 5) is roughly about 2.7 m (9 ft.) higher than that of the West bridge. Access to Bridge 504 is from S 336th Street beneath the bridge. The S 336th Street roadway has one lane in each direction and has at least 4.6 m (15 ft.) of shoulder on both sides underneath the bridge.

Runoff from the bridge is carried by the 102- to 152-mm (4- to 6-in.) drainage pipes attached to the sides of the existing piers. The pipes are connected to 152- to 203-mm (6- to 8-in.) corrugated metal pipes which daylight into the drainage ditches just outside the shoulders of S 336th Street. A gas pipeline utility sign is located about 3.0 m (10 ft.) south of Pier 2. The gas pipeline is expected to run in a direction parallel to S 336th Street. The existing abutments have no walls such that soil flows between the piers to form the finished 2H:1V abutment slopes.

# Subsurface Information

This section discusses the subsurface condition at the site of Bridge 5/504. The subsurface condition is based on six borings drilled at the site. The locations of these borings are shown in Figure 5-1. One of the borings was drilled during the original design in 1958 (H-9-58) at the north abutment of the East bridge. HWA (1992) drilled two additional borings in 1992 (BH-11-92 and BH-4-92) at the abutments of the East bridge as a part of their geotechnical investigation of the proposed SR 5 widening. At the site of the West bridge, three borings were drilled by WSDOT between December 1994 and January 1995 (H-18-94, H-19-94, and H-20-95). In addition, a piezometer was installed at the location of test boring H-19-94 to allow for periodic monitoring of groundwater conditions at the site. The logs of test borings are attached at the end of this section.

Laboratory soil tests were also conducted to confirm the field visual classification of soils. The tests consisted of 30 natural moisture contents and 15 grain size analyses. Results of these tests are attached at the end of this section.

The south abutment of the East bridge consists of about 2.1 m (7 ft.) of fill which is predominantly silty gravel and crushed rock. The fill is underlain by the very dense native material which is gravely sand with silt. This material was interpreted by HWA as native outwash deposit and generally classified as SP-SM under the Unified Soil Classification System (USCS). The amount of fines (percent passing Sieve No. 200) varies between 5 and 11 percent. SPT blowcounts for the native outwash material averaged roughly 70 blows per 305 mm (1 ft.) and were typically more than 50 blows per 150 mm. (6 inches) of sampler penetration. Groundwater was not encountered during drilling of boring BH-11-92 at the south abutment above elevation 103.3 m (339 ft.).

At the north abutment of the East bridge, the fill material is 8.2 m (27 ft.) deep and consists of medium-dense to dense silty sand to sandy silt with gravel and, possibly, cobbles except for a zone of loose silt between 1.2- and 4.6-m (4- and 15-ft.) below the ground surface. The average SPT N value is 15 over a range of 1 to 33. The native material underlies the abutment fill and is a unit of dense silty gravely fine to coarse sand with SPT N values of about 36 to 70 and fines content of 14 percent. This material extends to the maximum boring depth of 9.6 m (31.5 ft.). Under the USCS classification, this native material is classified as SM. The original test boring (H-9-58) reported a 1 m (3 ft.) thick layer of surficial organic silt. This silt layer was not encountered during the 1992 drilling. The original ground surface was about 113.6 m (372.5 ft.) when H-9-58 was drilled. Underlying the 1 m (3-ft.) layer of organic silt is a 2 m (6 ft.) thick layer of silty, sandy gravel which is in turn underlain by a very dense silty, sandy gravel and gravely sand. Groundwater was not encountered in either test borings drilled at the north abutment.

For the West bridge, three test borings were drilled by WSDOT between December 1994 and January 1995. The depths of the borings range from 10 m (33 ft.) at the location of Pier 3 to 15.4 m (50.5 ft.) at the south abutment. At the south abutment, the soil profile consists of about 2.4 m (8 ft.) thick of fill which is generally medium dense to dense silty sand with gravel (SW-SM) with a moisture content of 5 percent. Approximately 5.5 m (18 ft.) of medium-dense to dense silty gravel with sand underlies the fill. The density, structure, and composition of this material suggests that it is recessional outwash. This layer is underlain by a dense to very dense deposit of silty sand with gravel. The HWA inspector indicated that this latter deposit is advance outwash. SPT N values for the native material range from 15 to 48 and 44 to 81 for the recessional and advance outwash, respectively. The native outwash material is essentially non-plastic with fines content ranging from 7 to 28 percent and moisture content between 5 and 13 percent. Groundwater was not encountered during the drilling operation.

At the location of interior pier 3, the logs of test boring H-19-94 indicate the presence of a very dense silty gravel (GM) with sand and poorly graded gravel with silt and sand (GP-GM) in the upper 4 m (12 ft.) of soil. The fines content for the material ranges from 9 to 39 percent. Underlying this layer is a very dense silty sand with gravel (SM) which grades to poorly graded sand with silt, gravel, and cobble fragments (SP-SM) to the maximum boring depth of 33 ft. The underlying layer contains about 24 percent fines and moisture content of 4 to 19 percent. Groundwater was encountered 3.8 m (12.5 ft.) below the ground surface, corresponding to an approximate elevation of 106.1 m (348.5 ft.).

At the north abutment of the West bridge, the fill is a medium-dense silty sand with gravel with about 19 percent fines. The native outwash material underlying the fill consists of a silty gravel with sand (GM), silty sand with gravel, and well-graded gravel (GW) with sand, silt, and cobble fragments to a maximum boring depth of 12.2 m (40 ft.). Fines content for this deposit ranges between 19 and 24 percent. The material underlying the north abutment fill is similar to the advance outwash encountered at the south abutment. Groundwater was not encountered in the north abutment test boring.

Glacially deposited materials always have the potential for containing cobbles and boulders. Refusal blow counts and poor sample recovery in the advance outwash suggest that large gravel, cobbles, or boulders are present. In some locations where a layer of relatively impermeable advance outwash underlies a coarser material, it is typical that groundwater will perch on top of the impermeable layer. Therefore, perched water may be encountered above the regional groundwater table.

# Methods of Analyses

## Engineering Properties

Engineering properties for the various materials have been estimated at each pier location. The estimates are based on SPT N values, material gradation, published correlations to index properties, and local experience in similar soils. SPT values are believed to be highly influenced by the presence of gravels in the subsurface materials. Therefore, the SPT values were not corrected for overburden stress or hammer efficiency.

## Shallow Foundations

Allowable bearing pressures for spread footings were determined from bearing capacity and settlement analyses. Bearing capacity was computed in accordance with Article 4.4 of the AASHTO Standard Specifications for Highway Bridges (1992). Allowable bearing pressures recommended in the next sections are based on a combination of bearing capacity and 25 mm (1 inch) allowable settlement. The subsurface materials are either granular or highly overconsolidated, so settlement is anticipated to be elastic and to occur immediately after the loads are applied.

Immediate settlements were checked by several methods readily calculated by the program CSANDSET (U.S. Army Corps of Engineers, 1990). Where the depth of a relatively uniform layer below each boring was at least two footing widths, elastic settlement was estimated by elastic half-space methods. The elastic half-space methods computed by CSANDSET are variations of the general method described in Article 4.4.7.2.2 of the AASHTO (1993) specifications. Settlement was predicted by the methods described by Schmertmann (1978) where soil layer thickness was less than two footing widths. The Schmertmann method was also used as a check on the infinite half-space methods.

## Drilled Shaft Foundations

Axial capacity of drilled shafts has been calculated in accordance with the methods outlined in the FHWA design manual (Reese and O'Neill, 1988). A factor of safety of 3.0 has been applied to the computed ultimate shaft friction and end-bearing capacities to obtain the allowable capacity for compressive loading. Thus, the allowable axial capacity was determined from:

$$Q_{a,\,compression} = (Q_s + Q_b)/3.0$$

where $Q_s$ = ultimate skin friction

$Q_b$ = ultimate end-bearing

A factor of safety of 3.0 (rather than 2.5 as used for Bridge 5/503) has been used because the shafts are anticipated to penetrate the dense to very dense glacial outwash below the water table. Disturbance to the formation during shaft construction maybe difficult to control due to the raveling nature of the material. Inspection of the bottom of the excavation prior to concrete placement will be difficult, if not impossible.

For allowable uplift, the factor of safety used is 1.5. No reduction in skin friction for cone breakout has been applied. Allowable uplift was computed as:

$$Q_{a,\,uplift} = Q_s/1.5 + W$$

where: W = weight of the shaft

Ultimate skin friction was computed by the Beta method for both the fill and the outwash. Beta varies non-linearly with depth. At the abutments, the top 1.2 m (4 ft.) of subgrade, representing the depth of the box structure, was ignored in computing the effective overburden stress. The tops of the abutment shafts were assumed to be 2.4 m (8 ft.) below finished grade. Skin friction in the bottom one-diameter length along the shaft was ignored in accordance with the methods outlined by Reese and O'Neill (1988).

The design values for skin friction are based on soil-to-concrete contact. Therefore, installation methods must follow WSDOT's master specifications for drilled shaft installation

which intended to maintain the integrity of the soil around the shaft and prevent build-up of a filter cake. If permanent casing is used, the space between the casing and the surrounding soil must be grouted over the full length of the casing and allowable skin friction must be reduced by one-third.

End-bearing was determined from Reese and O'Neill's recommendations for granular soils. A limiting ultimate end-bearing capacity of 4300 kPa (90 ksf), which is independent of depth, was used. A diameter reduction factor was applied to this limiting value.

Allowable loads were determined by the above methods for several toe elevations or penetration lengths. The allowable loads were also checked against a 25 mm (1 inch) maximum settlement criteria. Settlement of shafts bearing in the very dense outwash was assumed to consist entirely of shaft movement required to mobilize skin friction and end-bearing under working loads.

### Embankment Stability

Recommendations for maximum embankment slopes are based on observations of existing slopes and infinite slope-type analyses of fill material. The assumed angle of internal friction for the new fill is 35 degrees and the moist unit weight is 19.6 $kN/m^3$ (125 pcf). Because of the relatively level terrain, no global stability analyses were conducted.

### Lateral Earth Pressures

Active and passive earth pressure coefficients were computed with the Coulomb equation, ignoring wall friction, as recommended by AASHTO. Concrete to soil friction for determining sliding resistance has been estimated as tan (2/3 • phi). A phi angle of 35 degrees has been assumed beneath all footings. A factor of safety of 1.5 was applied to static passive earth pressure coefficients to limit structural movement.

### Seismic Design

Dynamic shear modulus for seismic design of shallow foundations was computed for small strains on the order of $10^{-4}$ percent by equations developed by Seed et al. (1986). The maximum shear moduli obtained for small strains were then adjusted for strains commonly induced by an earthquake ($10^{-2}$ to $10^{-1}$ percent ) using the charts recommended by the same authors. Poisson's ratio for use in seismic design was estimated from material type and density. Earth pressure coefficients for seismic analysis were computed by the Mononabe-Okabe equations, as recommended by AASHTO (1992).

## Recommendations

### Foundation Types

Spread footings are recommended for all bridge piers. Conditions also appear to be suitable for drilled shafts at Piers 2, 3, and 4. Information is also provided for 1.52 and 1.83 m (5 and 6 ft.) diameter shaft alternatives, to minimize temporary shoring of the abutments and avoid utilities near the interior piers.

## Soil Properties

The engineering properties used for design are included in Table 5-1. The table references soil types, which refer to numbered soil layers on the subsurface profile of Figure 5-2. Young's modulus values to be used in settlement analyses were generally calculated based on published correlations with SPT blowcounts. For deposits with essentially refusal blowcounts, a value of 240 MPa (5,000 ksf) was used for footings on a fairly level ground surface such as those located at the interior piers. For footings on slopes (abutment piers), a value of 48 (1,000 ksf) was adopted to account for stress relief and lack of confinement.

The design groundwater elevation is 107.7 m (353.5 ft.). This elevation is 1.5 m (5 ft.) above the only observed water elevation at the site. Groundwater was observed at elevation 106.1 m (348.5 ft.) in boring H-19-94 during drilling. The piezometer installed in H-19-94 has been destroyed by vandals or an errant vehicle; no post-drilling readings are available. The additional water depth is intended to provide some margin of safety against seasonal variations in the water table and additional conservatism given the lack of data.

**Table 5-1**
**Soil Properties - Bridge 5/50~~5~~4**

| Soil Number (Refer to Figure 5-2) | Simplified Description | Unit Weight (pcf) | | Shear Strength | | Young's Modulus MPa (ksf) |
|---|---|---|---|---|---|---|
| | | γ Moist KN/m³ (pcf) | γ Sat KN/m³ (pcf) | Friction Angle, Ø (degrees) | Cohesion, c kPa (psf) | |
| 1 | Abutment fill | 19.6 (125) | 20.1 (128) | 33 | 0 | 12 (250) |
| 2 | Abutment Advance Outwash | 20.1 (128) | 20.7 (132) | 38 | 0 | 48 (1,000) |
| 3 | Advance Outwash Below 5336 Street Grade | 20.1 (128) | 20.7 (132) | 38 | 0 | 240 (5,000) |
| 4 | Recessional Outwash | 20.1 (128) | 20.7 (132) | 35 | 0 | 24 (500) |

## Allowable Bearing for Spread Footings

Table 5-2 shows the recommended allowable bearing pressures. In most cases, the design criteria limiting total settlement to one inch controls the allowable pressure. The allowable bearing pressures in Table 5-2 also greatly reduce the potential for inducing additional settlement of the existing footings. Settlement is anticipated to be elastic, which will occur as the load is applied. The maximum elevations in the table represent the shallowest footing elevations based on assumed structural limitations. These elevations were used in the settlement estimates. The allowable bearing pressures shown in Table 5-2 are not valid if footing elevations are set above the maximum elevations because settlements will increase and the allowable bearing pressures will decrease.

If footings are located at least one footing width from the face of the embankment slope, the allowable bearing pressures in Table 5-2 will also provide protection from punching shear-type failures.

**Table 5-2**
**Allowable Bearing Pressures for Spread Footings - Bridge 5/504**

| Bridge Location | Pier Number | Maximum Elevation Meters (feet) | Allowable Bearing Pressure kPa (ksf) | Comments |
|---|---|---|---|---|
| 504E | 1 | 116.8 (383.4) | 290 (6) | Match Existing |
| 504E | 2 | 108.3 (355.4) | 290 (6) | Match Existing |
| 504E | 3 | 108.3 (355.4) | 290 (6) | Match Existing |
| 504E | 4 | 115.6 (379.4)<br>112.6 (369.4) | 95 (2)<br>240 (5) | B ≤ 4.9 m (16 ft.) |
| 504W | 1 (West & East) | 112.9 (370.4) | 190 (4) | Match Existing |
| 504W | 2 (West & East) | 107.9 (339.3) ? | 290 (6) | Match Existing |
| 504W | 3 (West & East) | 108.0 (339.3) ? | 290 (6) | Match Existing |
| 504W | 4 (West & East) | 111.7 (366.6) | 290 (6) | Match Existing |

Vertical Datum = NAVD88

## Drilled Shaft Alternative

Figures 5-3 through 5-5 show the allowable capacity in compression and uplift for 1.52 m (5 ft.) and 1.83 m (6 ft.) diameter shafts for various toe elevations and lengths of penetration. As noted previously, factors of safety of 3.0 and 1.5 have been used for axial compression and uplift, respectively. The maximum toe elevation (minimum penetration length) shown at each pier location in the figures is the maximum elevation (minimum penetration length) permissible to limit settlement to 25 mm (1 in.). The figures should not be interpolated beyond these maximum elevations to shorten shaft lengths for lighter loads.

The skin friction used to compute the allowable loads in the figures is based on the assumption that shaft concrete will be poured directly against relatively undisturbed soil in the shaft walls. Permanent casings should not be used without reducing the recommended capacities shown in the figures. If permanent casing is used, the space between the casing and the surrounding soil must be grouted over the full length of the casing and allowable skin friction must be reduced by one-third.

A temporary casing extending down to at least elevation 112.4 m (369 ft.) is recommended at Pier 4 of Bridge 5/504E. The existing Pier 4 foundation is located above approximately 7.0 m (23 ft.) of fill. The use of temporary casing is to safeguard the footing against movement of the fill during construction.

Temporary casing may be used at Piers 2 and 3 to maintain a stable excavation during shaft construction. The existing Pier 2 and 3 footings appear to be founded on very dense glacial outwash. Support of excavations into the outwash below the water table can probably be stabilized with drilling fluid. While the contractor may choose to use temporary casings as

a means to protect the existing structure during shaft construction, they should not be required at Piers 2 and 3.

Design soil properties for use in lateral analyses are shown on the attached COM624 forms. Values of initial tangent modulus of subgrade reaction ($k_s$) and strain corresponding to stress at one-half the maximum total-principal-stress difference ($\varepsilon_{50}$) were estimated from published correlations with relative density (Reese and Wang, 1993). Some judgment should be used in applying these numbers.

## Embankment Slopes

Embankment slopes of 2H:1V or flatter are recommended. The new fill should be keyed into the existing fill by cutting benches into the existing embankment, as specified in WSDOT's standard specifications.

## Lateral Earth Loads

The equivalent static lateral earth loads imposed on abutment walls, columns, and footings should be computed as follows:

$$P_h = 0.5 \text{ (unit weight) (wall height)}^2 K$$

where: K = the appropriate active ($K_a$), passive ($K_p$), or at-rest ($K_o$) static coefficient of lateral earth pressure

A unit weight of 19.6 kN/m$^3$ (125 pcf) should be assumed for backfill materials. The load $P_h$ will act at one-third the wall height from the base. The following coefficients of lateral earth pressure are recommended:

### Within 2H:1V Sloped Embankments (Piers 1 and 4)

- Passive, $K_p$ = 0.9
- At-rest, $K_o$ = 0.5

### On Level Ground (Piers 2 and 3)

- Active, $K_a$ = 0.3
- Passive, $K_p$ = 2.5
- At-rest, $K_o$ = 0.5

At-rest earth pressures should be used for computing loads on the back of columns and footings within the sloped embankment. A factor of safety of 1.5 has been applied to $K_p$ to limit movement. The top 1 m (3 ft.) of soil should be neglected in computing passive resistance.

Lateral pressures generated by surcharge loads should be added to the lateral loading produced by the backfill. Lateral loading is computed by multiplying the vertical pressure at depth by the appropriate lateral earth pressure coefficient. If the surcharge is uniform, it should be applied uniformly between the top and the base of the wall. If the surcharge is limited in lateral extent, such as a large sign foundation, the lateral load applied to the ver-

tical member will vary with depth. The only known surcharge loads at this time are traffic loads, which should be treated as a 0.6 m (2 ft.) soil surcharge. A unit weight of 19.6 $KN/m^3$ (125 pcf) should be used in computing the intensity of surcharge loading. Furthermore, a friction factor of 0.4 should be used for computing frictional resistance to sliding.

## Seismic Design

The peak, firm ground acceleration for this area is 0.25 (Higgins et al., 1988). This acceleration has a 10 percent probability of being exceeded in a 50-year design period.

Design response spectra as outlined in the AASHTO guide specifications are appropriate for this site. A Type II soil profile should be used for determining the site coefficient. Due to the dense to very dense nature of the native outwash material, liquefaction is not a concern at this site.

A Poisson's ratio of 0.3 is recommended for use in determining soil spring constants for spread footings. Table 5-3 provides recommended values of dynamic shear modulus, G, for strains ranging from $10^{-1}$ to $10^{-2}$ percent for spread footing design.

**Table 5-3**
**Recommendations For Dynamic Shear Modulus - Bridge 5/504**

| Bridge | Pier | G at $10^{-2}$% strain MPa (ksf) | G at $10^{-1}$% strain MPa (ksf) |
|---|---|---|---|
| 5/504E | 1 | 67 (1400) | 24 (500) |
| 5/504E | 2 and 3 | 96 (2000) | 38 (800) |
| 5/504E | 4 | 34 (700) | 13 (280) |
| 5/504W | 1 | 37 (780) | 14 (300) |
| 5/504W | 2 and 3 | 96 (2000) | 38 (800) |
| 5/504W | 4 | 67 (1400) | 24 (500) |

Lateral earth loads on abutments and footings as a result of seismic loading should be computed as follows:

$$P_{he} = 0.5(\text{unit weight})(\text{wall height})^2 K_{ae}$$

Where $K_{ae}$ = The combined static and seismic active earth pressure coefficient.

$K_{ae}$ = 0.65 at Piers 1 and 5 (within sloped embankment)

$K_{ae}$ = 0.35 at Piers 2 and 3 (on level ground)

The combined static and seismic lateral earth load, $P_{he}$, can be assumed to act at mid-wall height as a result of a uniform pressure distribution.

A static lateral earth pressure coefficient without a factor of safety applied should be used to compute the resistance to combined static and seismic loading. Passive earth pressure coefficients of 0.9 and 2.5 are recommended for the abutment and interior piers, respectively.

## Other Considerations

Allowable bearing pressures for all interior footings (Piers 2 and 3) of Bridge 5/504 (East and West) are based on the assumption that the foundations will be placed on a very dense, undisturbed outwash deposit. If this material is not encountered at the design elevations it will be necessary to either:

1. Check the actual design bearing pressure against an estimated allowable pressure for the materials encountered (i.e. consult the geotechnical and structural engineer to verify the adequacy of design); or
2. Overexcavate until very dense, undisturbed native outwash deposit is encountered and backfill with lean concrete or crushed surfacing base course compacted to 95 percent of maximum density as specified in Section 2-03.3(14)C, Method C in the WSDOT Standard Specifications.

Footings should have at least 0.6 m (2 ft.) of cover for frost protection. The subgrade should be firm and free of cobbles and organic material. Where shallow foundations are founded in the undisturbed native material, the subgrade is anticipated to be sufficiently dense to require no additional preparation. However, the exposed native outwash subgrade will be moisture sensitive and should be protected from weather and construction traffic. Foundation conditions in the fill beneath the proposed spread footings should be checked by an experienced geotechnical engineer prior to placing a leveling course or concrete. If a soft or loose material is present, overexcavation and replacement with compacted granular material will be necessary. Within the fill the subgrade should be scarified to a minimum depth of 150 mm (6 in.) and re-compacted to at least 95 percent of maximum density as specified in Section 2-03.3(14)C, Method C in the WSDOT Standard Specifications. Placement of a leveling course of base rock immediately following foundation excavation or preparation is recommended to protect the foundation soil.

Gravel Backfill for Walls should be used within 1 m (3 ft.) of the abutment walls. Positive drainage in the form of an Underdrain Pipe surrounded by Gravel Backfill for Drains should also be provided at the base of abutment walls. Gravel Backfill for Foundations should be used for backfill around the interior piers.

Approach slabs may be deleted because of the following geotechnical criteria:

1. Post-construction settlement of the embankment is anticipated to be less than 13 mm (0.5 in.).
2. Creep settlement is not anticipated.

Groundwater is anticipated to rise within about 0.3 m (1 ft.) of the bottom of some existing footings. Prolonged rainy periods or heavy runoff could bring groundwater to higher levels. If groundwater is encountered, excavations should be dewatered to maintain excavation stability and subgrade strength. Surface runoff carried by the roadway ditches will also have to be routed around the construction area.

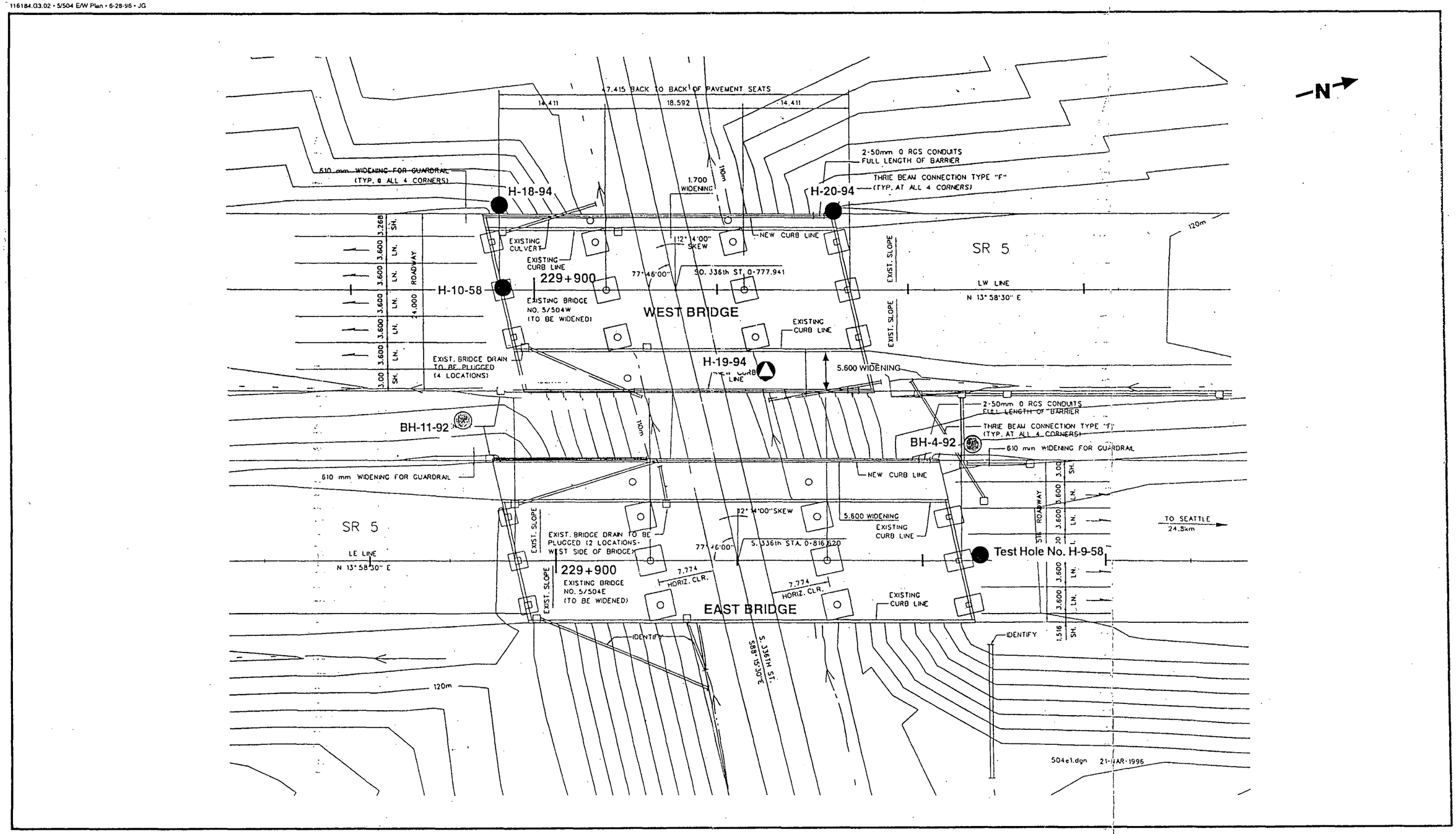
## Advisory Specifications

The contractor should be aware of potentially difficult conditions that may affect drilled shaft construction. Glacially deposited materials, such as those present at this site,

frequently contain cobbles and boulders that may impede drilling progress. Test borings had refusal blow counts and poor sample recovery; these are commonly indicators of large gravel, cobbles, or boulders. Drilled shafts will likely extend below the measured ground water table. Excavation stabilization measures are likely to be required that may include, but are not limited to, the use of temporary casings, slurries, and/or applications of increased hydraulic pressures to control shaft sidewall stability. The boring logs and geotechnical report should be consulted for subsurface exploration details.

As noted in the recommendations section, temporary casing extending down to at least elevation 112.4 m (369 ft.) is required at Pier 4 of Bridge 5/504E. The existing Pier 4 foundation is bearing on approximately 7.0 m (23 ft.) of fill. The use of temporary casing is to safeguard the footing against movement of the fill during construction. Excavation stabilization measures that may include the use of temporary casing, slurries, and/or application of increased hydraulic pressures to control shaft sidewall stability may be required at all pier locations.

116184.03.02 • 5/504 E/W Plan • 6-28-96 • JG




Approximate Scale in Meters

0 10 20 m

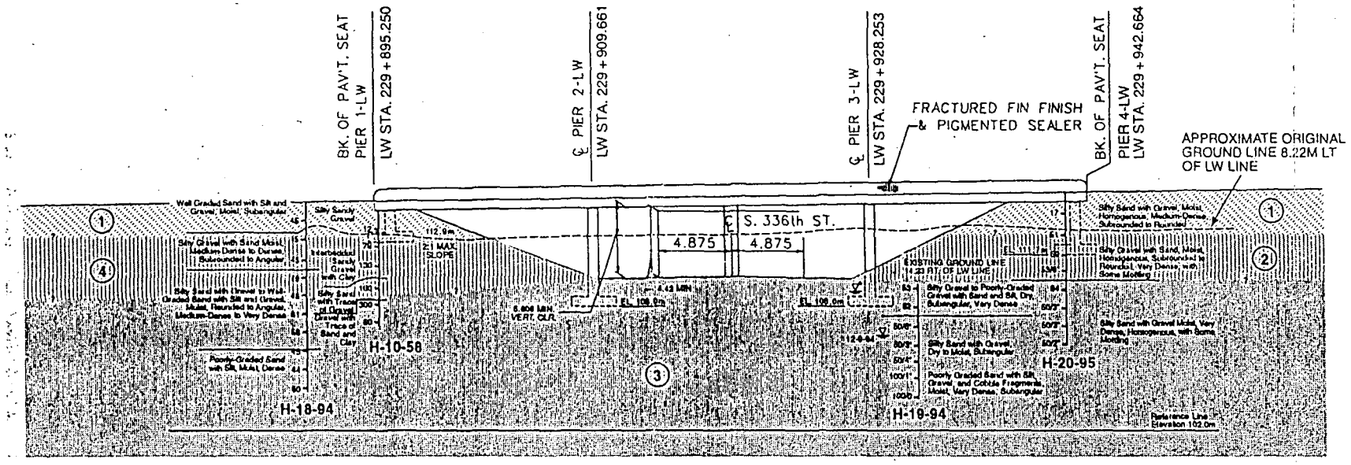
Test Hole

Piezometer

HWA Boring

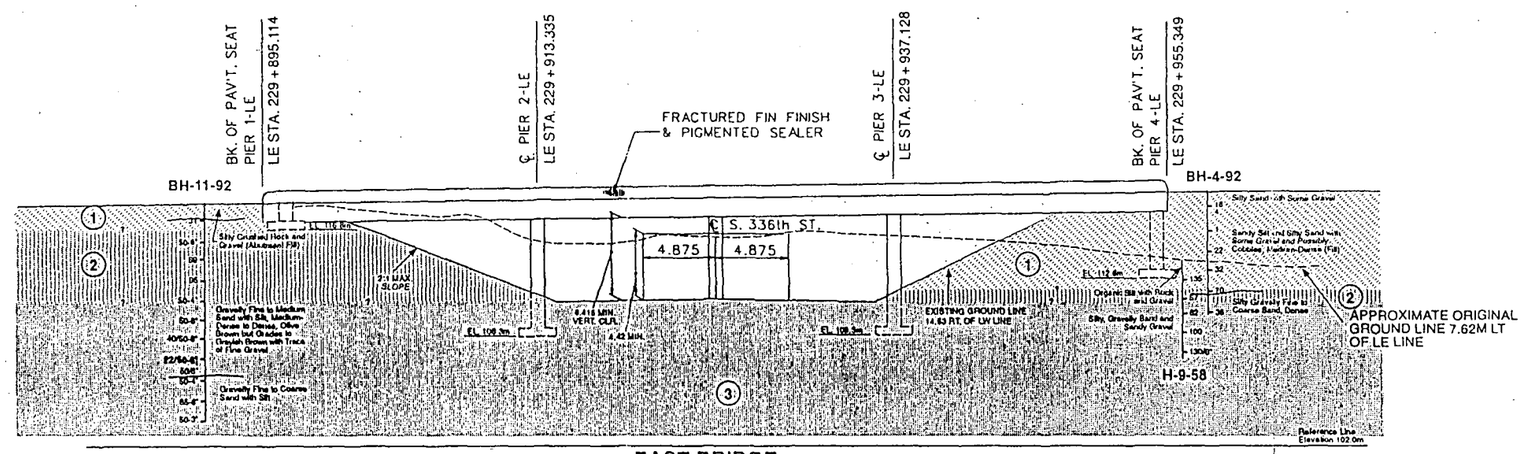
*NOTES:*
1 meter = 3.28 feet
Drawing adapted from Preliminary Bridge Plans dated March 15, 1996.
Final plans may vary.

FIGURE 5-1
**Bridge 5/504 East and West Plan View**
Geotechnical Report SR-5
Port of Tacoma I/C to 272nd
Stage 4 HOV




WEST BRIDGE

* Water observed in ditch 2-96




EAST BRIDGE

**LEGEND**

(1) (2) (3) (4) Soil units used to assign design soil properties. See Table 5-1 for listing of assumed properties

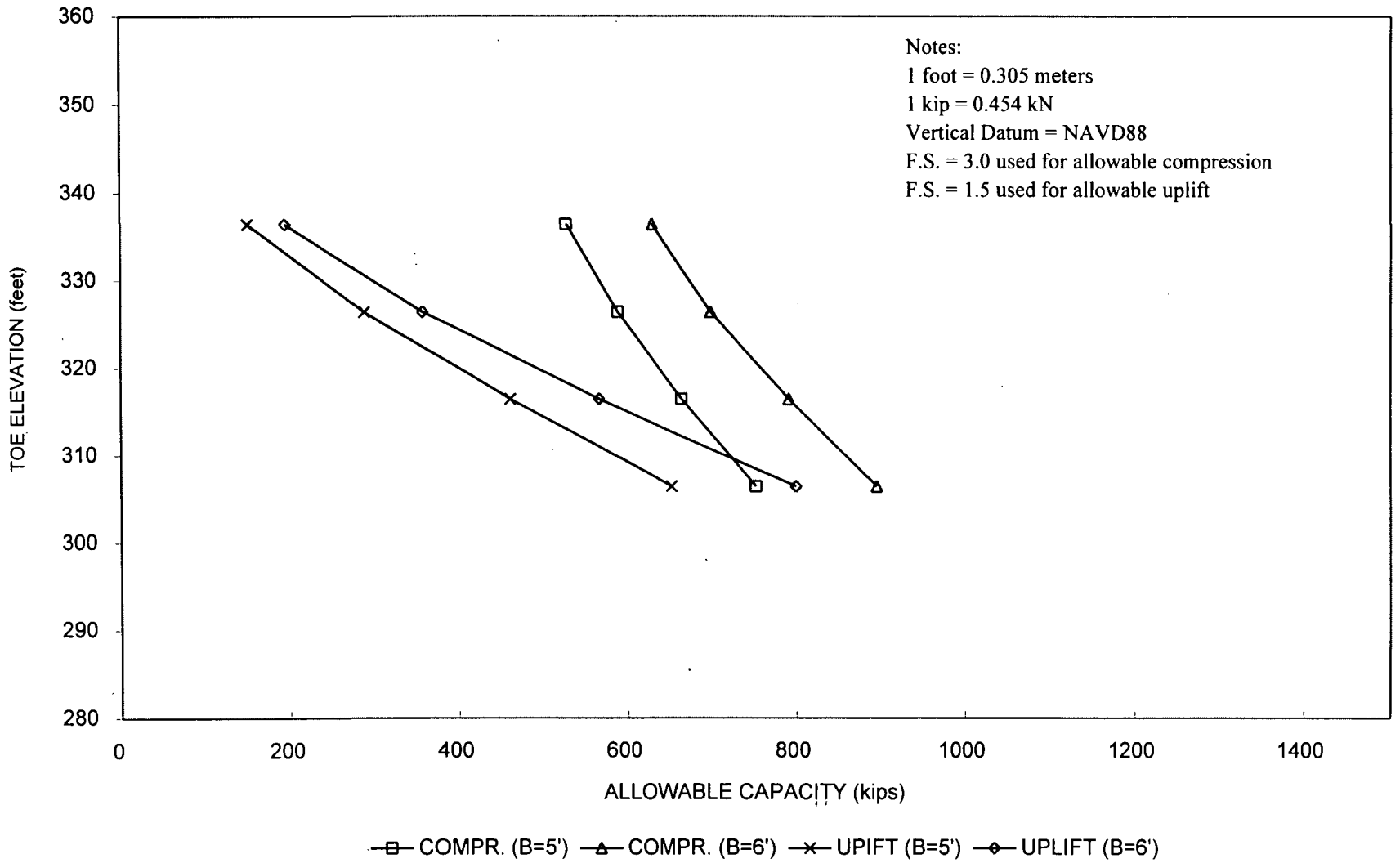
Approximate Scale in Meters

0 — 10 m

*NOTES:*
*1 meter = 3.28 feet*
Vertical Datum = NAVD88

FIGURE 5-2
**Bridge 5/504 East and West Profile**
Geotechnical Report SR-5
Port of Tacoma I/C to 272nd
Stage 4 HOV

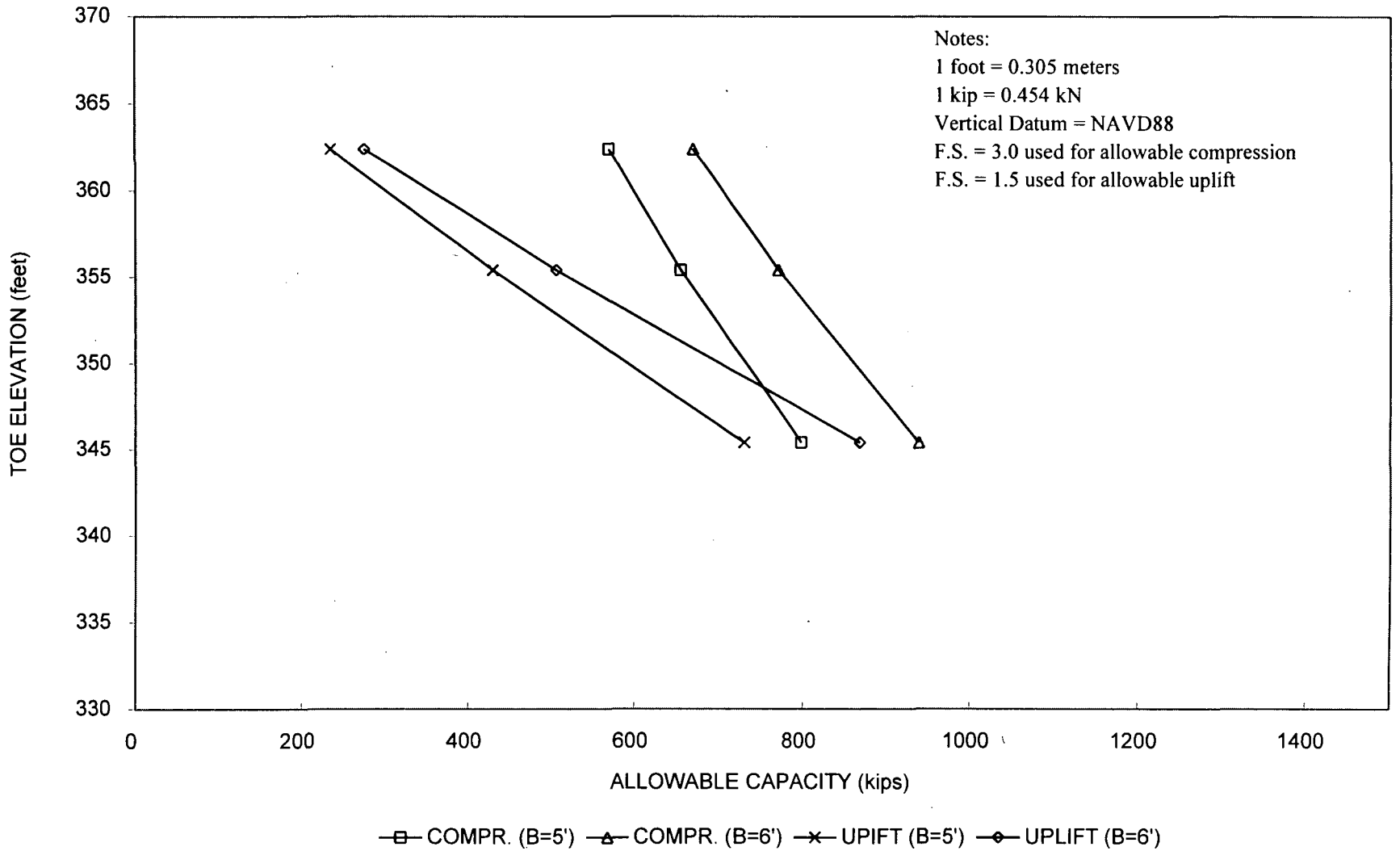
BRIDGE 5/504 (EAST & WEST)
DRILLED SHAFT @ PIERS 2 and 3




9/24/96
504CAP.XLS

FIGURE 5-3

BRIDGE 5/504 (EAST)
DRILLED SHAFT @ PIER 4

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504CAP.XLS

FIGURE 5-4

**L-PILE / COM 624 Soil Data Input**
**Bridge 5 /504E&W**

**Location: Piers 2 and 3**
**No. of Soil Layers :** 1
**Conditions:** **Non-seismic and Seismic**
**Datum NAVD 88** **Reference Elevation :** 111.6 (m) 366 (ft)

| Layer No. | Type of Soil | Depth to Boundry | | | | Effective Unit Weight | | Cohesion | | Int. Angle of Friction | Modulus of Subgrade Reaction (k) | | Strain @ 50% |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| | | Upper | | Lower | | | | | | | | | |
| | | (m) | (ft) | (m) | (ft) | (KN/m*3) | (pcf) | (KPa) | (psf) | (degrees) | (MN/m*3) | (pci) | (ε50) |
| 1 | Sand | 0.0 | 0.0 | Toe | Toe | 11.0 | 70.0 | 0.0 | 0 | 38 | 31.2 | 115 | |
| | | | | | | | | | | | | | |
| | | | | | | | | | | | | | |
| | | | | | | | | | | | | | |
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| | | | | | | | | | | | | | |
| | | | | | | | | | | | | | |
| | | | | | | | | | | | | | |
| | | | | | | | | | | | | | |

Note: Reference elevation = assumed effective top of shaft elevation = top of shaft at abutments
~ 1.5 m below existing ground surface at interior piers

9/24/96
504LPILE.XLS

**L-PILE / COM 624 Soil Data Input**
**Bridge 5 /504E**

**Location: Pier 4**
**No. of Soil Layers :** 2
**Conditions:** **Non-seismic and Seismic**
**Datum NAVD 88** **Reference Elevation :** 119.2 (m) 391 (ft)

| Layer No. | Type of Soil | Depth to Boundry Upper (m) | Depth to Boundry Upper (ft) | Depth to Boundry Lower (m) | Depth to Boundry Lower (ft) | Effective Unit Weight (KN/m*3) | Effective Unit Weight (pcf) | Cohesion (KPa) | Cohesion (psf) | Int. Angle of Friction (degrees) | Modulus of Subgrade Reaction (k) (MN/m*3) | Modulus of Subgrade Reaction (k) (pci) | Strain @ 50% (ε50) |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| 1 | Sand | 0.0 | 0.0 | 10.7 | 35.0 | 19.6 | 125.0 | 0.0 | 0 | 30 | 12.2 | 45 | |
| 2 | Sand | 10.7 | 35.0 | Toe | Toe | 11.0 | 70.0 | 0.0 | 0 | 38 | 31.2 | 115 | |

Note: Reference elevation = assumed effective top of shaft elevation = top of shaft at abutments
~ 1.5 m below existing ground surface at interior piers

9/24/96
504LPILE.XLS

**L-PILE / COM 624 Soil Data Input**
**Bridge 5 /504W**

**Location: Pier 4**
**No. of Soil Layers :** 3
**Conditions:** Non-seismic and Seismic
**Datum NAVD 88** **Reference Elevation :** 116.5 (m) 382 (ft)

| Layer No. | Type of Soil | Depth to Boundry Upper (m) | Depth to Boundry Upper (ft) | Depth to Boundry Lower (m) | Depth to Boundry Lower (ft) | Effective Unit Weight (KN/m*3) | Effective Unit Weight (pcf) | Cohesion (KPa) | Cohesion (psf) | Int. Angle of Friction (degrees) | Modulus of Subgrade Reaction (k) (MN/m*3) | Modulus of Subgrade Reaction (k) (pci) | Strain @ 50% (ε50) |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| 1 | Sand | 0.0 | 0.0 | 4.6 | 15.0 | 19.6 | 125.0 | 0.0 | 0 | 33 | 24.4 | 90 | |
| 2 | Sand | 4.6 | 15.0 | 7.6 | 25.0 | 20.1 | 128.0 | 0.0 | 0 | 38 | 54.3 | 200 | |
| 3 | Sand | 7.6 | 25.0 | Toe | Toe | 11.0 | 70.0 | 0.0 | 0 | 38 | 31.2 | 115 | |

Note: Reference elevation = assumed effective top of shaft elevation = top of shaft at abutments
~ 1.5 m below existing ground surface at interior piers

9/24/96
504LPILE.XLS

# LOG OF TEST BORING

Washington State Department of Transportation

HOLE No. H-18-94

PROJECT Fife to Military Road Stage 3 — Job No. L-1922

Bridge 5/504 — S.R. 5

Station LW 229+889 — Offset 9.0 m Lt. — C.S. 2719

Equipment — Casing 4" OD X 50' Hollow Core — Ground El 115.0 m

Method of Boring Dry Rotary

Start Date December 14, 1994 — Completion Date December 14, 1994 — Sheet 1 of 3

| Depth (ft) | Meters (m) | Profile | Standard Penetration Blows/ft (10 20 30 40) | SPT Blows/6" (N) | Sample Type | Sample No. (Tube No.) | Lab Tests | Description of Material | Groundwater | Instrument |
|---|---|---|---|---|---|---|---|---|---|---|
| | | | | | | | | 0.0 ft. to 4.0 ft. Silty SAND with gravel. (Fill material). | | |
| 5 | 1, 2 | | | 12 22 23 (45) | | D-1 | GS MC | SW-SM, M.C.=5% Well graded SAND with silt and gravel, subangular, dense, dark brown, moist. Retained 0.7 ft. | | |
| 10 | 3 | | | 16 7 8 (15) | | D-2 | GS MC | GM, M.C.=12% Silty GRAVEL with sand, subrounded, medium dense, light brown, moist. Retained 0.8 ft. | | |
| 15 | 4, 5 | | | 17 27 18 (45) | | D-3 | | Silty GRAVEL with sand, subrounded to angular, dense, brown, moist. Retained 0.8 ft. | | |
| 20 | 6 | | | 15 11 | | D-4 | GS MC | SM, M.C.=13% Silty SAND with gravel, angular, medium dense, | | |

# LOG OF TEST BORING

Washington State Department of Transportation

HOLE No. **H-18-94**

Sheet 2 of 3

PROJECT **Fife to Military Road Stage 3**

Job No. L-1922

| Depth (ft) | Meters (m) | Profile | Standard Penetration Blows/ft (10 20 30 40) | SPT Blows/6" (N) | Sample Type | Sample No. (Tube No.) | Lab Tests | Description of Material | Groundwater | Instrument |
|---|---|---|---|---|---|---|---|---|---|---|
| | | | | 8 (19) | | | | brown, moist. Retained 0.8 ft. | | |
| | 7 | | | | | | | | | |
| 25 | | | | 7 28 20 (48) | | D-5 | | Silty SAND with gravel, subrounded, dense, brown, moist. Retained 1.0 ft. | | |
| | 8 | | | | | | | | | |
| 30 | 9 | | | 15 30 51 (81) | | D-6 | GS MC | SW-SM, M.C. = 5% Well graded SAND with silt and gravel, subrounded, dense, brown, moist. Retained 1.0 ft. | | |
| | 10 | | | | | | | | | |
| 35 | | | | 17 24 34 (58) | | D-7 | | Well graded SAND with silt and gravel, rounded to subrounded, very dense, brown, moist. Retained 1.5 ft. | | |
| | 11 | | | | | | | | | |
| | | | | | | | | Material changed at depth 39.0 ft. | | |
| 40 | 12 | | | 18 23 22 (45) | | D-8 | | Poorly graded SAND with silt, dense, brown, moist. Retained 1.5 ft. | | |
| | 13 | | | | | | | | | |
| 45 | | | | 13 19 | | D-9 | GS MC | SP-SM, M.C. = 9% Poorly graded SAND with silt, dense, brown, moist. Retained 1.5 ft. | | |

# LOG OF TEST BORING

Washington State Department of Transportation

HOLE No. **H-18-94**

Sheet 3 of 3

PROJECT **Fife to Military Road Stage 3**

Job No. L-1922

| Depth (ft) | Meters (m) | Profile | Standard Penetration Blows/ft (10 20 30 40) | SPT Blows/6" (N) | Sample Type | Sample No. (Tube No.) | Lab Tests | Description of Material | Groundwater | Instrument |
|---|---|---|---|---|---|---|---|---|---|---|
| | | | | 25 (44) | | | | | | |
| | 14 | | | | | | | | | |
| | 15 | | | 15 22 28 (50) | | D-10 | | Poorly graded SAND with of silt, dense, brown, moist. Retained 1.5 ft. | | |
| 50 | | | | | | | | | | |
| | | | | | | | | End of test hole boring at 50.5 ft. below ground elevation. | | |
| | | | | | | | | This is a summary Log of Test Boring. Soil/Rock descriptions are derived from visual field identifications and laboratory test data. | | |
| | 16 | | | | | | | | | |
| 55 | | | | | | | | | | |
| | 17 | | | | | | | | | |
| | 18 | | | | | | | | | |
| 60 | | | | | | | | | | |
| | 19 | | | | | | | | | |
| 65 | | | | | | | | | | |
| | 20 | | | | | | | | | |
| | 21 | | | | | | | | | |
| 70 | | | | | | | | | | |

# LOG OF TEST BORING

Washington State Department of Transportation

HOLE No. H-19-94

PROJECT Fife to Military Road Stage 3 — Job No. L-1922

Bridge 5/504 — S.R. 5

Station LW 229+927 — Offset 11.9 m Rt. — C.S. 2719

Equipment — Casing — Ground El 110.0 m

Method of Boring Augers

Start Date December 8, 1994 — Completion Date December 9, 1994 — Sheet 1 of 2

| Depth (ft) | Meters (m) | Profile | Standard Penetration Blows/ft (10 20 30 40) | SPT Blows/6" (N) | Sample Type | Sample No. (Tube No.) | Lab Tests | Description of Material | Groundwater | Instrument |
|---|---|---|---|---|---|---|---|---|---|---|
| | 1 | | >> | 18 24 29 (53) | | D-1 | GS MC | GM, M.C.=19% Silty GRAVEL with sand, subangular, very dense, light brown, dry. Residual soil. Retained 0.9 ft. | | |
| 5 | 2 | | >> | 20 25 27 (52) | | D-2 | GS MC | GP-GM, M.C.=4% Poorly graded GRAVEL with silt and sand, subangular, very dense, brown, dry. Residual soil. Retained 1.7 ft. | | |
| 10 | 3 | | | | | | | | | |
| | 4 | | | 23 50 (50/6") | | D-3 | GS MC | SM, M.C.=8% Silty SAND with gravel; subangular, very dense, brown, dry. Residual soil. Retained 0.7 ft. | ▽ | |
| 15 | 5 | | | | | | | | | |
| | | | | 28 50/3 (50/3") | | D-4 | GS MC | SM, M.C.=8% Silty SAND with gravel, subangular, very dense, brown, moist. Residual soil. Retained 0.7 ft. | | |
| 20 | 6 | | | | | | | | | |

# LOG OF TEST BORING

Washington State Department of Transportation

HOLE No. **H-19-94**

Sheet **2** of **2**

PROJECT **Fife to Military Road Stage 3**

Job No. **L-1922**

| Depth (ft) | Meters (m) | Profile | Standard Penetration Blows/ft (10 20 30 40) | SPT Blows/6" (N) | Sample Type | Sample No. (Tube No.) | Lab Tests | Description of Material | Groundwater | Instrument |
|---|---|---|---|---|---|---|---|---|---|---|
| | 7 | | | 50/4" (50/4") | | D-5 | | Poorly graded SAND with silt, gravel and cobble fragments, subangular, very dense, brown, moist. Residual soil. Retained 0.2 ft. | | |
| 25 | 8 | | | | | | | | | |
| | | | > > | 100/1" (100/1") | | D-6 | | Poorly graded SAND with silt, gravel, and cobble fragments, subangular, very dense, brown, moist. Residual soil. Retained 0.1 ft. | | |
| 30 | 9 | | | | | | | | | |
| | 10 | | > > | 100/0" (100/0") | | D-7 | | No recovery. | | |
| 35 | 11 | | | | | | | End of test hole boring at 33.0 ft. below ground elevation. This is a summary Log of Test Boring. Soil/Rock descriptions are derived from visual field identifications and laboratory test data. | | |
| 40 | 12 | | | | | | | | | |
| | 13 | | | | | | | | | |
| 45 | | | | | | | | | | |

# LOG OF TEST BORING

Washington State Department of Transportation

HOLE No. H-20-95

PROJECT Fife to Military Road Stage 3 — Job No. L-1922

Bridge 5/504 — S.R. 5

Station LW 229+941.5 — Offset 9.1 m Lt. — C.S. 2719

Equipment — Casing HQ to 40' — Ground El 116.0 m

Method of Boring Wet Rotary

Start Date January 14, 1995 — Completion Date January 14, 1995 — Sheet 1 of 2

| Depth (ft) | Meters (m) | Profile | Standard Penetration Blows/ft (10 20 30 40) | SPT Blows/6" (N) | Sample Type | Sample No. (Tube No.) | Lab Tests | Description of Material | Groundwater | Instrument |
|---|---|---|---|---|---|---|---|---|---|---|
| 5 | 1, 2 | | | 3 5 12 (17) | | D-1 | GS MC | SM, M.C.=12% Silty SAND with gravel, medium dense, subrounded to rounded, light orange brown, moist, homogeneous, no HCl reaction. Retained 1.1 ft. | | |
| 10 | 3 | | | 21 31 20 (51) | | D-2 | | Silty GRAVEL with sand silt, very dense, subrounded to rounded, light grayish brown, moist, homogeneous, no HCl reaction. Some mottling. Retained 0.4 ft. | | |
| 15 | 4, 5 | | | 49 38 21 (59) | | D-3 | GS MC | GM, M.C.=13% Silty GRAVEL with sand silt, very dense, subrounded to rounded, light grayish brown, moist, homogeneous, no HCl reaction. Some mottling. Retained 1.0 ft. | | |
| 20 | 6 | | | | | | | | | |

# LOG OF TEST BORING

Washington State Department of Transportation

HOLE No. **H-20-95**

Sheet 2 of 2

PROJECT **Fife to Military Road Stage 3**

Job No. **L-1922**

| Depth (ft) | Meters (m) | Profile | Standard Penetration Blows/ft (10 20 30 40) | SPT Blows/6" (N) | Sample Type | Sample No. (Tube No.) | Lab Tests | Description of Material | Groundwater | Instrument |
|---|---|---|---|---|---|---|---|---|---|---|
| | | | | 29<br>53/6<br>(53/6") | | D-4 | | Silty GRAVEL with sand, very dense, subrounded to rounded, light grayish brown, moist, homogeneous, no HCl reaction. Some mottling.<br>Retained 0.4 ft. | | |
| | 7 | | | | | | | | | |
| 25 | | | | 21<br>43<br>41<br>(84) | | D-5 | GS<br>MC | SM, M.C. = 9%<br>Silty SAND with gravel, very dense, subrounded to rounded, light grayish brown, moist, homogeneous, no HCl reaction. Some mottling.<br>Retained 1.0 ft. | | |
| | 8 | | | | | | | | | |
| | 9 | | | | | | | | | |
| 30 | | | | 50/3"<br>(50/3") | | D-6 | | Silty SAND with gravel, very dense, subrounded to rounded, light grayish brown, moist, homogeneous, no HCl reaction. Some mottling.<br>Retained 0.3 ft. | | |
| | 10 | | | | | | | | | |
| 35 | | | | 38<br>50/3<br>(50/3") | | D-7 | | Silty SAND with gravel, very dense, subrounded to rounded, light grayish brown, moist, homogeneous, no HCl reaction. Some mottling.<br>Retained 0.4 ft. | | |
| | 11 | | | | | | | | | |
| | 12 | | | | | | | Well graded GRAVEL with cobble fragments, sand and silt, very dense, subrounded to rounded, light grayish brown, moist, homogeneous, no HCl reaction. Some mottling.<br>Retained 0.2 ft. | | |
| 40 | | | | 50/2"<br>(50/2") | | D-8 | | | | |
| | | | | | | | | *End of test hole boring at 40.2 ft. below ground elevation.* | | |
| | 13 | | | | | | | This is a summary Log of Test Boring. Soil/Rock descriptions are derived from visual field identifications and laboratory test data. | | |
| 45 | | | | | | | | | | |

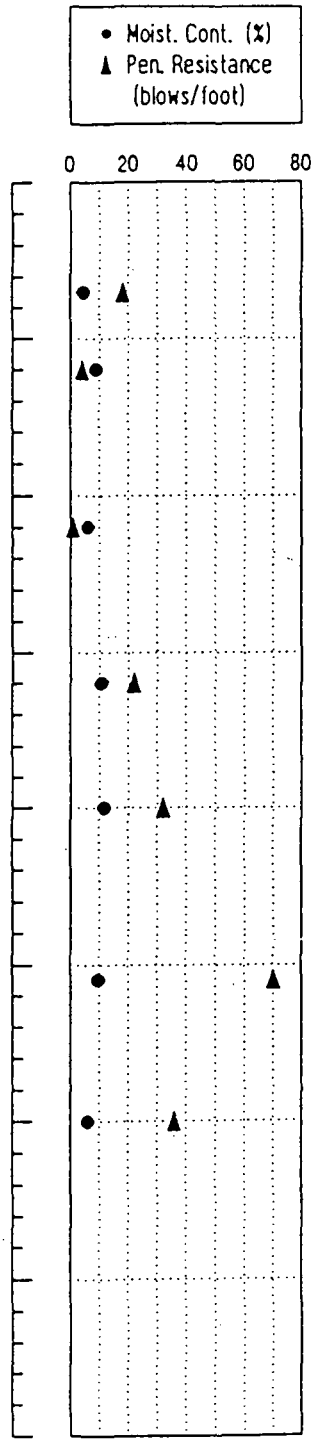
# HONG WEST & ASSOCIATES, INC.

# BORING LOG

DRILLING COMPANY: Gregory Drilling
DRILLING METHOD: Hollow Stem
SAMPLING METHOD: SPT

TOTAL DEPTH: 31.5 Feet
SURFACE ELEVATION: **119.0 m**
MEASURING POINT EL.: Feet

| DEPTH (feet) | SAMPLES | PEN. RESISTANCE (blows/6 inches) | N-VALUE (blows/ft) | MOIST. CONT. (%) | SYMBOL | SOIL CLASS. (USCS) | DESCRIPTION |
|---|---|---|---|---|---|---|---|
| 0 | X | 25/8/10 | 18 | 4.4 | | | Medium dense, brown, silty SAND. Some gravel. Dry. (Fill) |
| 5 | X | 4/2/2 | 4 | 8.6 | | | Very Loose, brown, sandy SILT with gravel. (Fill) |
| 10 | X | 1-18" | 1 | 5.9 | | | |
| 15 | X | 8/9/13 | 22 | 10.5 | | | Medium dense to dense, olive grey and brown, sandy SILT and silty SAND. Some gravel. Possibly cobbles. (Fill) |
| 20 | X | 11/15/17 | 32 | 11.4 | | | |
| 25 | X | 5/20/50 | 70 | 9.6 | | | |
| | | | | | | SM | Dense, olive brown, silty, gravelly, fine to coarse SAND. Damp. (Advance Outwash) |
| 30 | X | 13/16/20 | 36 | 5.9 | | | |
| | | | | | | | End of boring at 31.5 feet. |
| 35 | | | | | | | |
| 40 | | | | | | | |




NOTE: This log of subsurface conditions applies only at the specified location and on the date indicated.

PROJECT: I-5 HOV Lane Widening

BORING: BH-4-92

LOCATION: **LE 229+959 11.3 m Lt.**
DATE COMPLETED: 5/14/92
LOGGED BY: CB

PROJECT NUMBER: 91024-3

PAGE: 1 OF 1

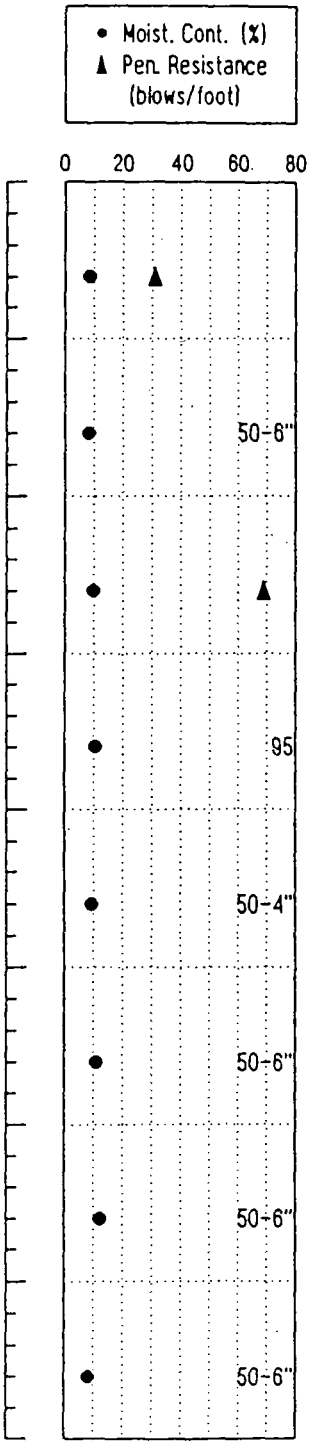
# HONG WEST & ASSOCIATES, INC.

# BORING LOG

DRILLING COMPANY: Soil Sampling Service
DRILLING METHOD: Hollow Stem
SAMPLING METHOD: SPT

TOTAL DEPTH: 54 Feet
SURFACE ELEVATION: **116.5 m**

| DEPTH (feet) | SAMPLES | PEN. RESISTANCE (blows/6 inches) | N-VALUE (blows/ft) | MOIST. CONT. (%) | SYMBOL | SOIL CLASS. (USCS) | DESCRIPTION |
|---|---|---|---|---|---|---|---|
| 0 | | | | | | | Medium dense to dense, brown Silty Crushed Rock, and rounded Gravel, Dry. |
| | ☒ | 16/15/16 | 31 | 8.4 | | SP SM | (Fill) |
| | | | | | | | Dense to very dense, olive brown gravelly SAND with silt. Damp. |
| 5 | | | | | | | (Advance Outwash) |
| | ⊠ | 50 | 50-6" | 8.0 | | | Dense to very dense, olive brown, gravelly, fine to medium SAND. Some silt. Damp to moist. |
| 10 | | | | | | | (Advance Outwash) |
| | ☒ | 38/29/40 | 69 | 9.8 | | | |
| 15 | | | | | | | |
| | ☒ | 30/45/50 | 95 | 10.4 | | | |
| 20 | | | | | | | |
| | ⊠ | 50-4" | 50-4" | 9.2 | | | |
| 25 | | | | | | | |
| | ⊠ | 40/50 | 50-6" | 10.8 | | | |
| 30 | | | | | | | |
| | ⊠ | 22/50 | 50-6" | 12.2 | | | Grades to greyish brown with a trace of fine gravel. |
| 35 | | | | | | | |
| | ⊠ | 50 | 50-6" | 8.0 | | | |
| 40 | | | | | | | |




NOTE: This log of subsurface conditions applies only at the specified location and on the date indicated.

PROJECT: I-5 HOV Lane Widening

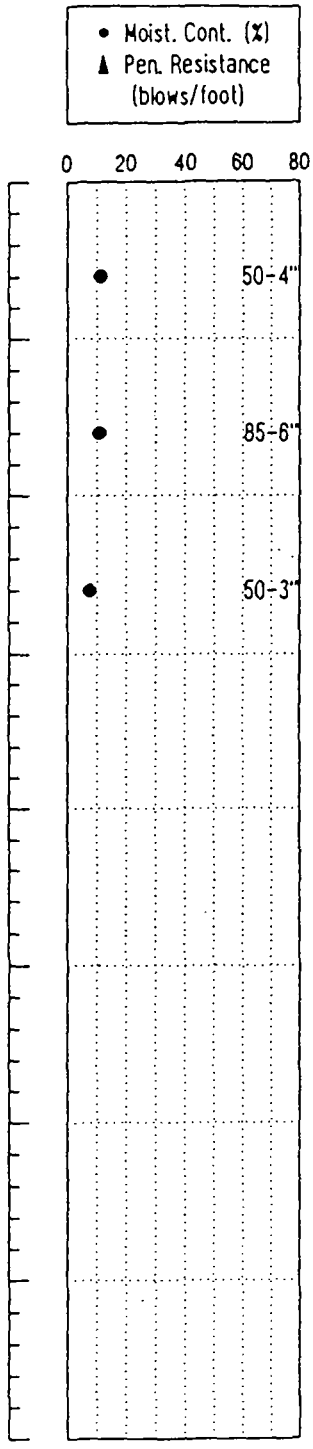
BORING: BH-11-92

LOCATION: **LE 229+891 11.6 m Lt.**
DATE COMPLETED: 8/22/92
LOGGED BY: CB

PROJECT NUMBER: 91024-3

# HONG WEST & ASSOCIATES, INC.
## BORING LOG

| DEPTH (feet) | SAMPLES | PEN. RESISTANCE (blows/6 inches) | N-VALUE (blows/ft) | MOIST. CONT. (%) | SYMBOL | SOIL CLASS. (USCS) | DESCRIPTION |
|---|---|---|---|---|---|---|---|
| 40 | | | | | | SP SM | |
| 42 | ⊠ | 50-4" | 50-4" | 11.2 | | | Grades to very gravelly (Advance Outwash) |
| 45 | | | | | | | |
| 47 | ⊠ | 85 | 85-6" | 10.5 | | | |
| 50 | | | | | | | |
| 52 | ⊠ | 50-3" | 50-3" | 7.7 | | | |
| 53 | | | | | | | End of Boring |
| 55 | | | | | | | |
| 60 | | | | | | | |
| 65 | | | | | | | |
| 70 | | | | | | | |
| 75 | | | | | | | |
| 80 | | | | | | | |




PROJECT: I-5 HOV Lane Widening

BORING: BH-11-92

LOCATION: **LE 229+891 11.6 m Lt.**
DATE COMPLETED: 6/22/92
LOGGED BY: CB

PROJECT NUMBER: 91024-3

PAGE: 2 OF 2

WASHINGTON
STATE HIGHWAY COMMISSION
DEPARTMENT OF HIGHWAYS 336 St Overpass
LOG OF TEST BORING

One Copy with Samples
One Copy to Dist. Engr.
One Copy to Materials Engr.

P. S. H. ~~S. S. H.~~ No. 1 Section Pierce Co. Line to Seattle T.S.E Freeway Job No. L-1666
Hole No. H-9-58 Station LE 229+957.5 Offset 0.6 m Lt. Ground Elev. 111.5 m
Type of Boring Jet Wash - TH-7 Size Casing 9" / 25' 10"
Inspector Mike Malion Date 8-22-58 Sheet No. 1 of 1

| DEPTH | BLOWS PER FT. | PROFILE | SAMPLE TUBE NOS. | DESCRIPTION OF MATERIAL | SAMPLE WEIGHT |
|---|---|---|---|---|---|
| | | | | Brown Organic Material | |
| | | | | Lt. Brown Silt with some inter-mingled broken Rock & Gravel (Altered Till) | |
| 5 | 135 | | 36 53 80 | S-1 Stand. 2" test 4' - 5.5' Gray Silty Sandy (Med) 1"(-) Gravel (Glacial Till - Jar S #1) | |
| 10 | 57 | | 21 36 | S-2 Stand 2" test 9.5' - 10.5' Gray Gravelly Silty Med. Sand. Gray Silty Sandy (Jar S. #2) 1"(-) Gravel Same as S-1 | |
| | | | | Gray Gravelly Silty Med. Sand. | |
| 15 | 62 | | 24 30 32 40 | S-3 Standard 2" test 14' - 16' 3/4"(-) Sandy gravel w/ Lt. Br. Silt Binder (Jar S#3) | |
| 20 | 106 | | 40 66 | S-4 Stand. 2" test 19.5' - 20.5' Same Material | |
| 25 | 100/6" | | 130 | S-5 Stand. 2" test 25' - 25.5' 1 1/2"(-) Gravel w/ trace Sand & Light Brown Clay Binder | |
| | | | | Test Hole Depth 25.5' W.T. 13.0' | |

WASHINGTON
STATE HIGHWAY COMMISSION
DEPARTMENT OF HIGHWAYS

One Copy with Samples
One Copy to Dist. Engr.
One Copy to Materials Engr.

# LOG OF TEST BORING 336th St. Over Pass

S. H. No. 1 Section Pierce Co. Line to Seattle T.S.E. Freeway Job No. L-1666
Hole No. H-10-58 Station LW 229+895 Offset CL Ground Elev. 113.5 m
Type of Boring Jet Wash - TH-7 Size Casing 3" / 31'4"
Inspector Mike Mallon Date 8-25/58 Sheet No. 1 of 1

| DEPTH | BLOWS PER FT. | PROFILE | SAMPLE TUBE NOS. | DESCRIPTION OF MATERIAL | SAMPLE WEIGHT |
|---|---|---|---|---|---|
| | | | | Light Br. Silt w/ Broken Rock & Gravel (Altered fill) | |
| | | | 23 30 78 | S-6 Stand. 2" test. 3.5'-5.0' | |
| 5 | 128 | | | Brownish Grey Silty, Sandy 1"(-) Gravel (Glacial till) Dry, partially cemented | |
| 10 | | | | S-7 Stand. 2" test 10'-11.5' | |
| | 79 | | 40 38 41 | Gravelly, Angular Med. Sand w/ Light Brown Silt Binder | |
| | | | | Gravel as S-6 | |
| 15 | | | | Gravelly, Angular Med. Sand w/ light Brown Silt | |
| | 130 | | 42 36 94 | S-8 Stand. 2" test 15'-16.5 | |
| | | | | 1½"(-) Gravel, Sandy & w/ Binder of Light Brown Clay | |
| 20 | | | | | |
| | 160 | | 70 90 | S-9 Stand. 2" test 20.5'-21.5' | |
| | | | | Light Brown Silty Medium Sand with trace of Gravel | |
| 25 | | | | | |
| | | | | S-10 Stand. 2" test 26.0'-27.0' | |
| | 300 | | 110 190 | 1½"(-) Gravel - trace of sand & Brown Clay | |
| 30 | | | | Light Brown Silty Medium Sand - Trace of Gravel | |
| | | | | S-11 Stand. 2" test 31'-33' | |
| | 60 | | 19 25 35 100 | 1½"(-) Gravel - Trace Sand & Clay | |
| | | | | Test Hole Depth 33' | |
| | | | | Total Casing 31'4" | |
| 35 | | | | W.T. - | |

Job No.

Hole No. **H-18-94**

Project

Date **February 15, 1995**

Sheet **1** of **1**

# Laboratory Summary

Washington State Department of Transportation

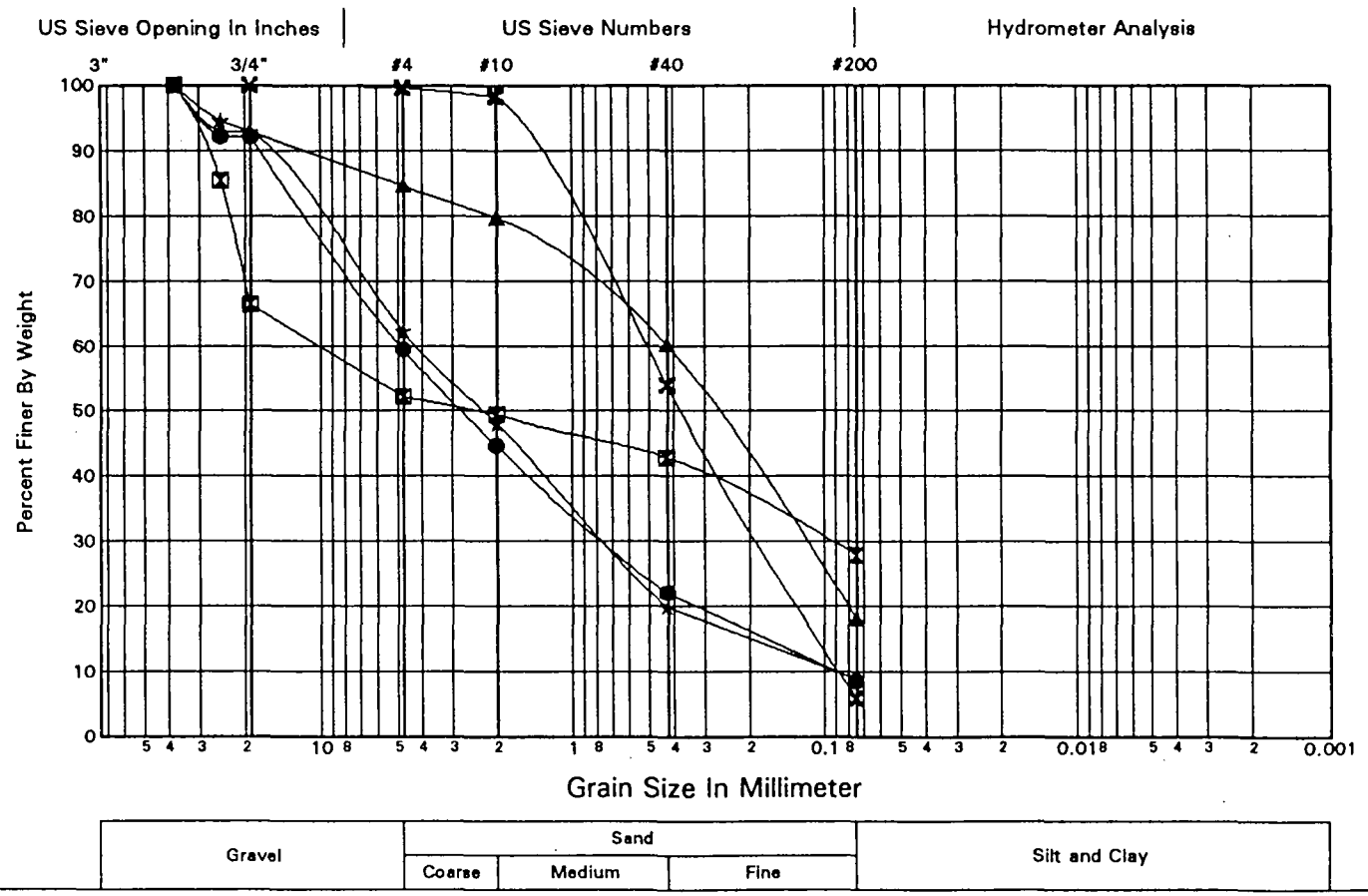
| | Depth (ft) | Depth (m) | Sample No. | USCS | Color | Description | MC% | LL | PL | PI |
|---|---|---|---|---|---|---|---|---|---|---|
| ● | 4.0 | 1.22 | D-1 | SW-SM | VERY DARK GRAY | WELL GRADED SAND with SILT and GRAVEL | 5 | NP | NP | NP |
| ⊠ | 9.0 | 2.74 | D-2 | GM | OLIVE | SILTY GRAVEL with SAND | 12 | NP | NP | NP |
| ▲ | 19.0 | 5.79 | D-4 | SM | OLIVE | SILTY SAND with GRAVEL | 13 | NP | NP | NP |
| ★ | 29.0 | 8.84 | D-6 | SW-SM | OLIVE | WELL GRADED SAND with SILT and GRAVEL | 5 | NP | NP | NP |
| ✕ | 44.0 | 13.41 | D-9 | SP-SM | OLIVE GRAY | POORLY GRADED SAND with SILT | 9 | NP | NP | NP |

## GRADATION FRACTIONS

| | %Gravel | %Sand | %Fines | Cu | Cc |
|---|---|---|---|---|---|
| ● | 40.5 | 50.9 | 8.6 | 1.2 | 54.0 |
| ⊠ | 47.9 | 24.2 | 27.9 | | |
| ▲ | 15.3 | 66.6 | 18.1 | | |
| ★ | 37.7 | 53.5 | 8.8 | 1.5 | 45.7 |
| ✕ | 0.5 | 93.6 | 5.9 | 0.7 | 6.0 |

## GRADATION VALUES

| | D60 | D50 | D30 | D20 | D10 |
|---|---|---|---|---|---|
| ● | 4.85 | 2.74 | 0.73 | 0.32 | 0.09 |
| ⊠ | 10.16 | 2.36 | 0.10 | | |
| ▲ | 0.42 | 0.28 | 0.12 | 0.08 | |
| ★ | 4.13 | 2.26 | 0.74 | 0.43 | 0.09 |
| ✕ | 0.52 | 0.37 | 0.18 | 0.12 | 0.09 |



Job No.

Hole No. **H-19-94**

Project

Date **February 15, 1995**

Sheet **1** of **1**

# Laboratory Summary

Washington State Department of Transportation

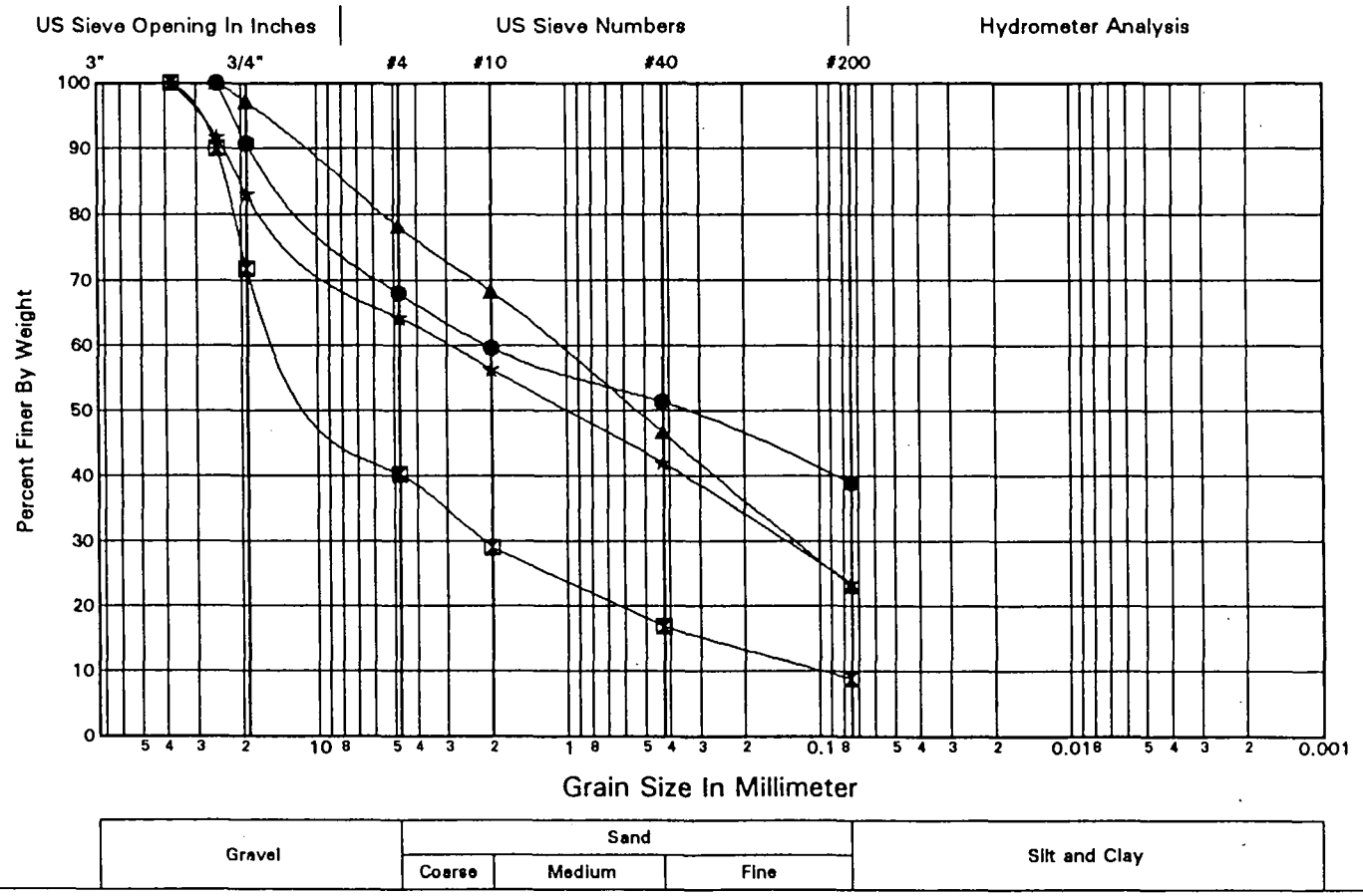
| | Depth (ft) | Depth (m) | Sample No. | USCS | Color | Description | MC% | LL | PL | PI |
|---|---|---|---|---|---|---|---|---|---|---|
| ● | 3.0 | 0.91 | D-1 | GM | OLIVE | SILTY GRAVEL with SAND | 19 | NP | NP | NP |
| ⊠ | 8.0 | 2.44 | D-2 | GP-GM | OLIVE | POORLY GRADED GRAVEL with SILT and SAND | 4 | NP | NP | NP |
| ▲ | 13.0 | 3.96 | D-3 | SM | OLIVE | SILTY SAND with GRAVEL | 8 | NP | NP | NP |
| ★ | 18.0 | 5.49 | D-4 | SM | OLIVE GRAY | SILTY SAND with GRAVEL | 8 | NP | NP | NP |
| | | | | | | | | | | |

## GRADATION FRACTIONS

| | %Gravel | %Sand | %Fines | Cu | Cc |
|---|---|---|---|---|---|
| ● | 32.0 | 29.1 | 38.9 | | |
| ⊠ | 59.8 | 31.4 | 8.8 | 4.2 | 116.8 |
| ▲ | 21.8 | 55.1 | 23.1 | | |
| ★ | 35.7 | 40.8 | 23.5 | | |
| | | | | | |

## GRADATION VALUES

| | D60 | D50 | D30 | D20 | D10 |
|---|---|---|---|---|---|
| ● | 2.08 | 0.35 | | | |
| ⊠ | 11.32 | 7.30 | 2.15 | 0.63 | 0.10 |
| ▲ | 1.10 | 0.54 | 0.12 | | |
| ★ | 2.97 | 1.01 | 0.14 | | |
| | | | | | |



Job No.

Hole No. **H-20-95**

Project

Date **February 15, 1995**

Sheet **1** of **1**

# Laboratory Summary

Washington State Department of Transportation

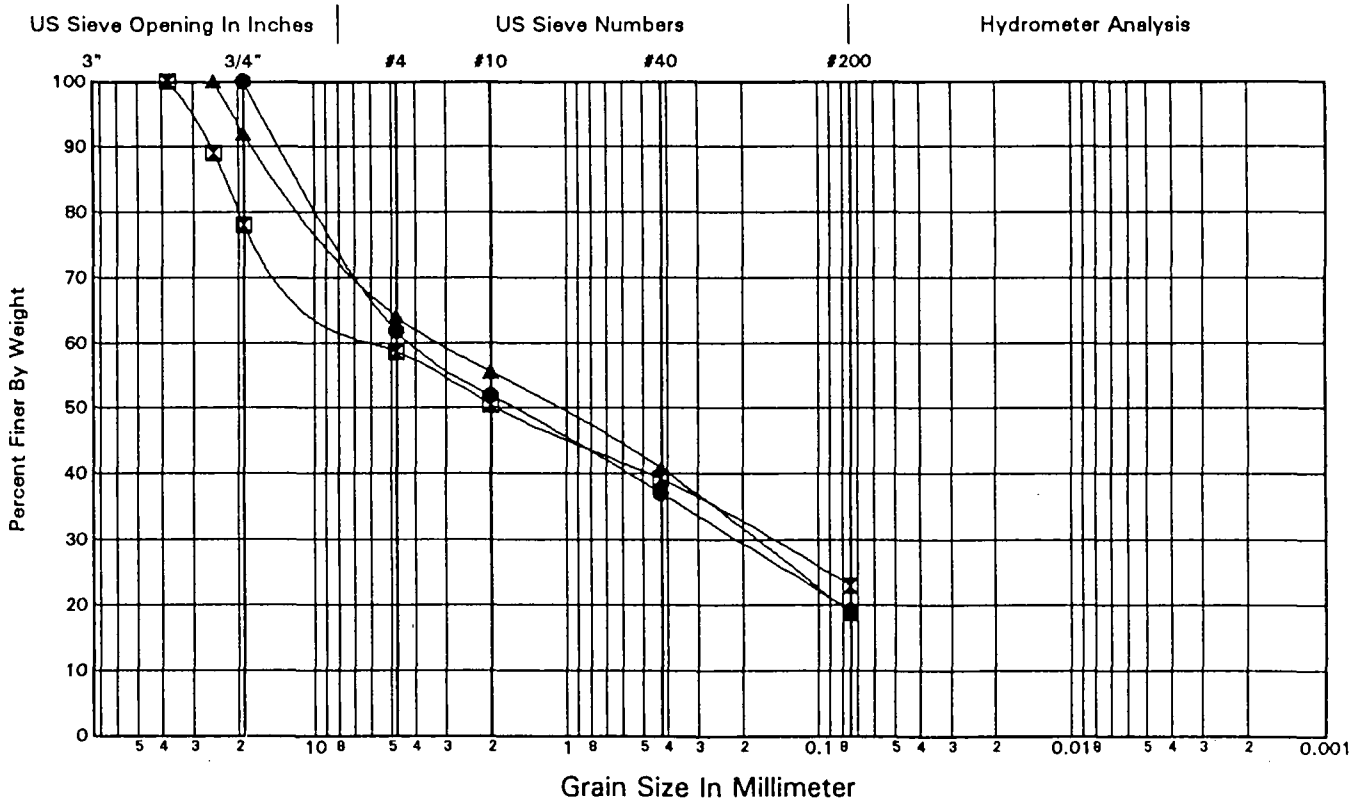
| | Depth (ft) | Depth (m) | Sample No. | USCS | Color | Description | MC% | LL | PL | PI |
|---|---|---|---|---|---|---|---|---|---|---|
| ● | 5.0 | 1.52 | D-1 | SM | OLIVE | SILTY SAND with GRAVEL | 12 | NP | NP | NP |
| ⊠ | 15.0 | 4.57 | D-3 | GM | OLIVE GRAY | SILTY GRAVEL with SAND | 13 | NP | NP | NP |
| ▲ | 25.0 | 7.62 | D-5 | SM | OLIVE | SILTY SAND with GRAVEL | 9 | NP | NP | NP |

## GRADATION FRACTIONS

| | %Gravel | %Sand | %Fines | Cu | Cc |
|---|---|---|---|---|---|
| ● | 38.1 | 42.7 | 19.2 | | |
| ⊠ | 41.3 | 35.7 | 23.0 | | |
| ▲ | 35.9 | 45.4 | 18.7 | | |

## GRADATION VALUES

| | D60 | D50 | D30 | D20 | D10 |
|---|---|---|---|---|---|
| ● | 4.03 | 1.64 | 0.21 | 0.08 | |
| ⊠ | 5.22 | 1.87 | 0.16 | | |
| ▲ | 3.13 | 1.11 | 0.18 | 0.08 | |



# HONG WEST & ASSOCIATES
# GRAIN SIZE DISTRIBUTION

Project: I-5 Bridges

Client: Alpha Engineering Group

Project Number: 91024-3

Date Tested: 5/28/92

Remarks: Olive brown, very gravelly, silty SAND (SM)

Test Hole Number: BH-4-92

Sample Number: 7

Depth: 30-31.5 feet

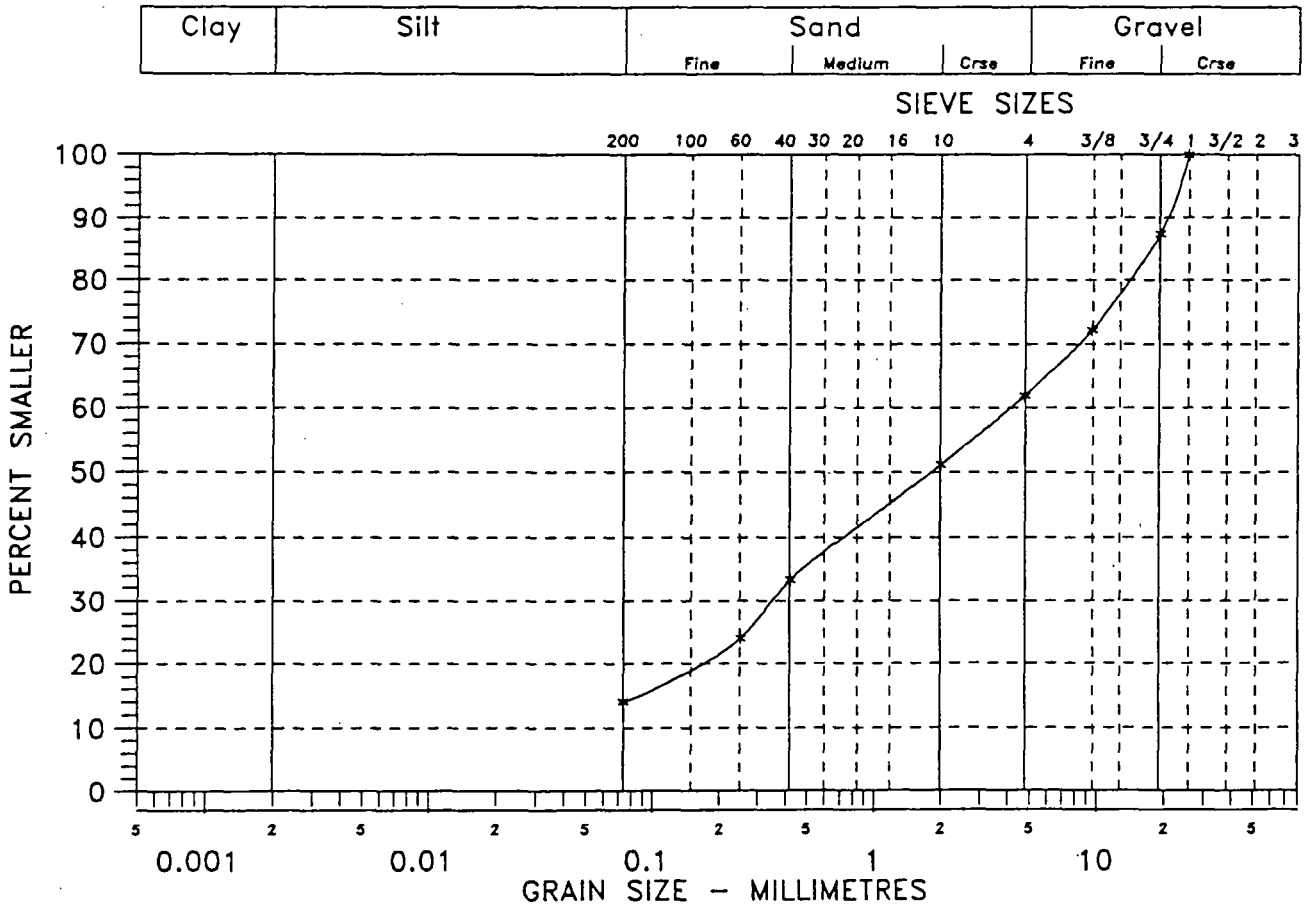
Sample Description:

Gravel: 38.1

Sand: 47.8

Silt: 14.1

Clay:



# HONG WEST & ASSOCIATES
# GRAIN SIZE DISTRIBUTION

Project: I-5 Bridges

Client: Alpha Engineering Group

Project Number: 91024-3

Date Tested: 6/23/92

Remarks: Dark grayish brown, gravelly SAND with some silt (SP-SM)

Test Hole Number: BH-11-92

Sample Number: 3

Depth: 12.5-14.0 feet

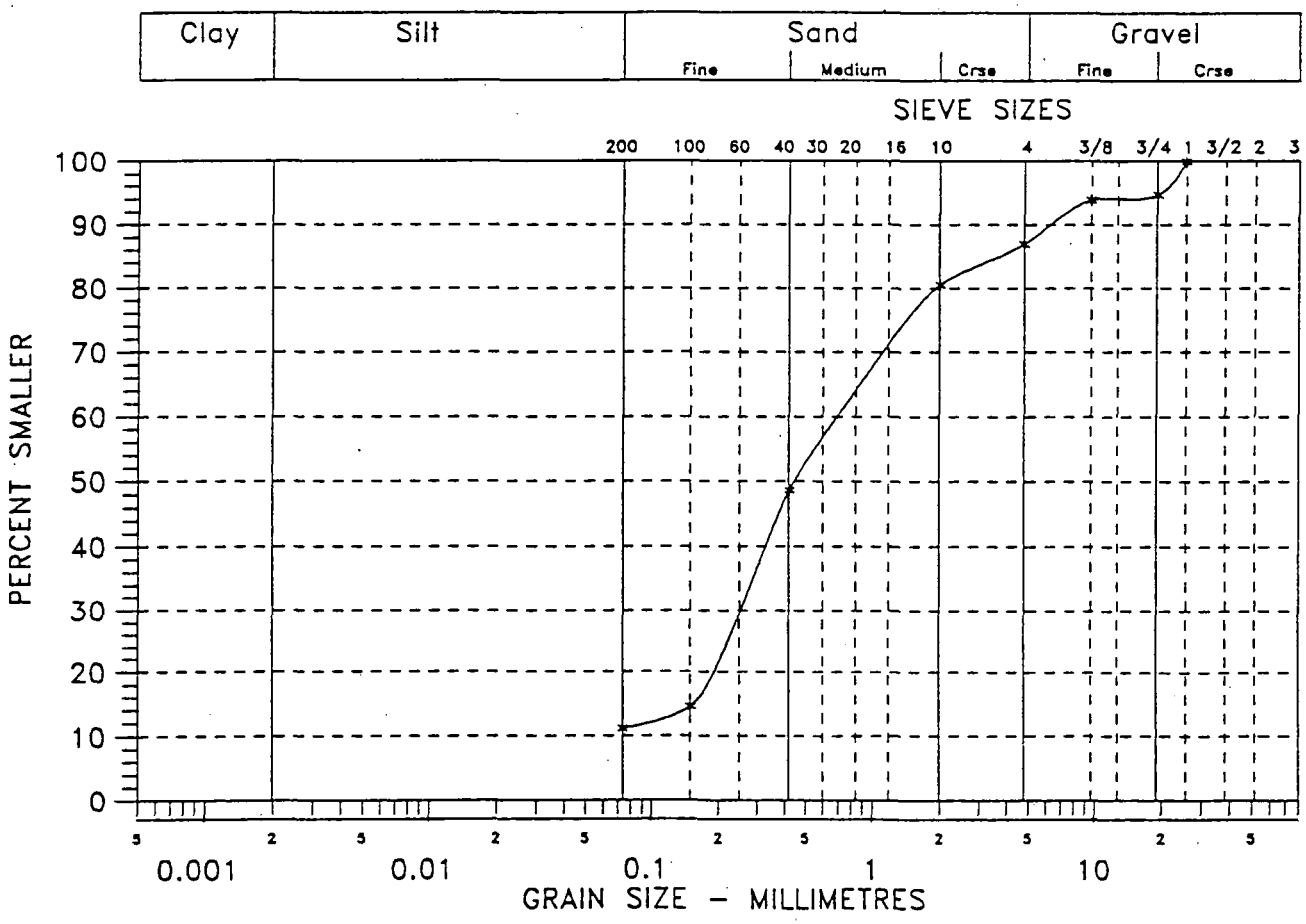
Sample Description:

Gravel: 13.0

Sand: 75.8

Silt: 11.2

Clay:



CHAPTER 6

# Bridge 5/506 East - Military Road

## Proposed Improvements

The location of the bridge is shown on the vicinity map, Figure 1-1. HOV widening of Bridge 5/506 East will add 5.46 m (17.9 ft) to the west side of the existing structure. The location of the widening is shown in Figure 6-1, which along with subsequent figures follows the text at the end of this chapter. Widening of the adjacent Bridge 5/506 West will occur under a separate contract.

## Existing Structures

The existing bridge is a three-span, concrete box structure. The existing structure width is 22.25 m (73 ft.) and the span lengths are 17.83 m (58.5 ft.) and 23.77 m (78.0 ft.) at the abutment and center spans, respectively.

The original construction plans indicate that the existing SR 5 abutments were constructed with fill and that the original ground surface was at approximately the same grade as Military Road.

The existing bridges are founded on spread footings. Based on the topography shown in the original 1958 construction drawings, the existing footings were founded at elevations 0.6 to 1.2 m (2 to 4 ft.) into the native subgrade. Existing abutment loads are transferred to the spread footings by piers over 9.1 m (30 ft.) high. The abutment fill has been placed around the piers and graded to a finished slope of 2H:1V.

## Site Information

The site is gently slopes to a low point in the vicinity of the bridge. There is a sag vertical curve and a horizontal curve in Military Road beneath the east bridge. Two northbound and southbound lanes of SR 5 are at approximately the same elevation, so no sliver fills will be required for abutment widening.

The existing SR 5 roadways have five lanes in each direction and a median roughly 23 m (70 ft.) wide. The bridge can be accessed from the shoulders of SR 5. The bridge crosses over Military Road, a two-lane roadway with paved shoulders. There are guardrails at the edges of Military road and limited level working space outside of the guardrails.

There are several drainage facilities, utilities, and structures beneath the bridge. The south side of Military Road has a drainage ditch that empties into a culvert beneath the western edge of the bridge. A gas line and storm sewer run beneath the southern shoulder of Military Road, on top of or very close to the Pier 2 footings. A gas line, water line, and TV cable parallel the line of footings along Pier 3. Roadway drainage along the north side of Military Road is carried in a storm sewer, also along the line of Pier 3. A metal bin wall roughly

1.5 m (5 ft.) high and located at the edge of the pavement on Military Road retains the southern abutment fill on Bridge 5/506 West. The end of the bin wall is approximately 3.0 m (10 ft.) from the western edge of Bridge 5/506 East. Construction of the foundation for widening of the east bridge will require reconstruction or temporary support of a portion of this wall.

The embankment slopes at the abutments are unprotected. The existing SR 5 pavement is badly rutted and voids of up to 80 mm (3 in.) deep and 150 mm (6 in.) wide have formed at the back of the pavement seat. Rilling, with some channels 150 mm (6 in.) to 300 mm (12 in.) deep, has occurred where runoff from SR 5 has run behind the back of pavement seat.

A spring was observed at the base of a 0.6 m (2 ft.) high gabion wall, approximately 12 m (40 ft.) east of the eastern column of Pier 3 on Bridge 5/506E. The estimated flow at the time of the March 1996 site visit was 4 liters/minute (1 gpm). Drainage ditches in the area flowed with up to 25 mm (1 inch) of water during the site visit. The area around the spring appears to support wetland vegetation. A wetland area with a few millimeters of standing water (in March) is also present to the west of the north abutment of Bridge 5/506W.

## Subsurface Information

At the east bridge, one boring was drilled for the original design in 1958 (H-1-58) and two borings were drilled in 1992 (BH-5-92 and BH-8-92). In 1994 and 1995, six borings were drilled for the adjacent west bridge (H-21-94 through H-23-94 and H-36-96 through H-38-95). Piezometers were installed in two of the borings (H-22-94 and H-23-95). The locations of the borings are shown in Figure 6-1. Boring logs are attached after this section. Laboratory tests consisted of 39 natural moisture contents and 22 grain size analyses. The laboratory test results are also attached at the end of this section.

The subsurface materials, as encountered from the ground surface downward, are described below. Simplified material descriptions, SPT blow counts, and measured water levels are shown on Figure 6-2.

1 Fill is present in both abutments and within the upper few meters at the interior piers. The measured and projected depths of the fill are shown in Figure 6-2. The fill is generally brown to gray-brown, loose to medium-dense, silty sand with gravel and cobbles. The gravel and cobbles are typically subrounded. Water was perched approximately 2.4 m (8 ft.) below the SR 5 roadway grade in B-8-92 in the southern abutment.

2 A layer of gray, medium-stiff to stiff, low plasticity silt was encountered beneath the southern abutment between approximately elevation and 127.6 and 124.4 m (419 and 408 ft.). The silt contains some fine gravel and traces of coarse sand. Groundwater was encountered near the top of the silt unit.

3 Deposits of brown to gray, moist to wet, medium-dense to dense silty sand with gravel and silty gravel with sand underlie the fill and silt. The thickness of this layer varies from 2 to 4m (7 to 13 ft.). The appearance, composition and relative density of this soil suggests that it is recessional drift, consisting of interbedded layers of ablation drift and recessional outwash.

4 The fourth and deepest soils encountered in the borings were similar to the overlying recessional drift, except very dense.

The notes of the drilling inspector suggest that some cobbles may be present in the fill. The largest material visible on the surface of the fill is approximately 100 mm (4 in.) in diameter. Glacially deposited materials always have the potential for containing cobbles and boulders. Refusal blow counts and poor sample recovery in the deeper materials suggest that large gravel, cobbles, or boulders may be present.

Groundwater has been measured at elevations 122.9 m (403.3 ft) and 122.1 m (400.6 ft). Excavations for spread footings at Pier 3, founded at 122.3 m (401.6 ft.), could encounter groundwater. Although the fines content of the glacial drift is relatively high (typically 20 to 30 percent), these types of deposits are commonly of moderate permeability. Occasional layers of relatively clean, highly permeable material are also present. Groundwater perched on silt layers or lenses could be encountered at higher elevations.

# Methods of Analyses

## Engineering Properties

Engineering properties for the various materials have been estimated at each pier location. The estimates are based on SPT N values, material gradation, correlations to index properties, and local experience in similar soils. SPT values are highly influenced gravels, as are present in the subsurface materials at this site. Therefore, the SPT values were not corrected for overburden stress or hammer efficiency.

## Pile Analyses

The axial capacity of pile foundations was determined by the unified pile design approach (Canadian Foundation Engineering Manual, 1985) and the UNIPILE computer program (Goudreault and Fellenius, 1990). The unified pile design approach includes the following assumptions:

- Very small deformations are required to induce downdrag forces on the pile
- Downdrag forces from the soil to the pile are distributed above the neutral plane
- Shaft resistance from the soil to the pile is distributed below the neutral plane
- Settlement is estimated by distributing the pile cap load at a 1H:2V slope to an imaginary load at the neutral plane.

The piles will be primarily end-bearing in the relatively incompressible glacial drift below the fill and silt layers, therefore, settlement is not a controlling factor in design provided the piles are driven to a minimum toe elevation in the glacial drift. However, the unified pile design approach is still appropriate for computing the total stress on the pile (i.e. bridge load plus downdrag load).

Factors of safety of 3.0 for axial compression and 1.5 for uplift were used. The net allowable axial capacity at the head of the pile based on soil resistance was computed as:

$$Q_{a,\ compression} = R_u / 3.0$$

where $R_u$ = sum of ultimate skin resistance and end-bearing

Allowable uplift was computed as:

$Q_{a,\ uplift} = R_s/1.5$
where $R_s$ = ultimate skin resistance

The weight of the pile is typically ignored in axial pile capacity computations.

## Shallow Foundations

Allowable bearing pressure for spread footings was determined from bearing capacity and settlement analyses. Bearing capacity was computed in accordance with Article 4.4 of the AASHTO Standard Specifications for Highway Bridges (1992). Allowable bearing pressures recommended in the next sections are based on a combination of bearing capacity and 25 mm (1 in.) allowable settlement.

Immediate settlements were checked by several methods readily calculated by the program CSANDSET (U.S. Army Corps of Engineers, 1990). In general, elastic settlement as determined by the methods described in Article 4.4.7.2.2 of the AASHTO specifications was used where the depth of a relatively uniform soil layer below each footing was at least two footing widths. Analyses by the methods described by Schmertmann (1978) were used where analysis of a layered soil profile was necessary and as a check on the infinite half-space method described in the AASHTO specifications.

## Drilled Shaft Foundations

Axial capacity of drilled shafts has been calculated in accordance with the methods outlined in the FHWA design manual (Reese and O'Neill, 1988). A factor of safety of 3.0 has been applied to the computed ultimate shaft friction and end-bearing capacities to obtain the allowable capacity for compressive loading. Thus, the allowable axial capacity is:

$Q_{a,\ compression} = (Q_s + Q_b)/3.0$
where $Q_s$ = ultimate skin friction
$Q_b$ = ultimate end-bearing

Shafts will penetrate cohesionless materials below the water table. During construction, it will be difficult to estimate the amount of overexcavation and to determine if the bottom of the excavation is clean. Therefore no reduction in the factor of safety is allowed without integrity or load testing.

For allowable uplift, the factor of safety is 1.5. No reduction in skin friction for cone breakout has been applied. Allowable uplift was computed as:

$Q_{a,\ uplift} = Q_s/1.5 + W$
where W = weight of the shaft.

Ultimate skin friction was computed by the Beta method within the fill and recessional drift (soil units 1, 3, and 4 in Table 6-1 and Figure 6-2.) Ultimate skin friction in the silt (soil unit 2) was computed by the undrained strength, or alpha method. Beta is a function of depth and overburden pressure only. At the abutments, the top 1.2 m (4 ft.) of subgrade, representing the depth of the structure, was ignored in computing effective overburden stress. The heads of the abutment shafts were assumed to be 2.4 m (8 ft.) below finished

grade. Skin friction in the top and bottom one-diameter length along the shaft was ignored in accordance with the methods outlined by Reese and O'Neill.

The design values for skin friction are based on soil-to-concrete contact. Therefore, installation methods must follow WSDOT's master specifications for drilled shaft installation which intended to maintain the integrity of the soil around the shaft and prevent build-up of a filter cake. If permanent casing is used, the space between the casing and the surrounding soil must be grouted over the full length of the casing and allowable skin friction must be reduced by one-third.

End-bearing was determined from Reese and O'Neill's recommendations for granular soils. A limiting ultimate end-bearing capacity of 4,300 kPa (90 ksf) was used. Shaft diameter reduction factors of 0.8 and 0.7 were applied to the uncorrected end-bearing value to determine the ultimate design end-bearing for the 1.52 m (5 ft.) and 1.83 m (6 ft.) diameter shafts, respectively.

Allowable loads were determined by the above methods for several toe elevations. The allowable loads were also checked against a 25 mm (1 in.) maximum settlement criteria. Settlement of shafts bearing in the very dense drift was assumed to consist entirely of shaft movement required to mobilize skin friction and end-bearing under working loads.

## Embankment Stability

Recommendations for maximum embankment slopes are based on observations of existing slopes and infinite slope-type analyses of fill material. The assumed angle of internal friction for new fill is 35 degrees and the moist unit weight is 19.6 $kN/m^3$ (125 pcf) . Because of the relatively level terrain, no global stability analyses were conducted.

## Lateral Earth Pressures

Active and passive earth pressure coefficients were computed with the Coulomb equation, ignoring wall friction, as recommended by AASHTO. Concrete to soil friction for determining sliding resistance has been estimated as tan (2/3 • phi). A phi angle of 35 degrees has been assumed beneath all footings. A factor of safety of 1.5 was applied to static passive earth pressure coefficients to limit structural movement.

## Seismic Design

Dynamic shear modulus for seismic design of shallow foundations was computed for strains on the order of $10^{-4}$ percent by equations developed by Seed et al. (1986), then adjusted to strains in the range of $10^{-2}$ to $10^{-1}$ percent, similar to those induced by an earthquake, by procedures recommended by these same authors. Poisson's ratio for use in seismic design was estimated from material type and density.

Earth pressure coefficients for seismic analysis were computed by the Mononabe-Okabe equations, as recommended by AASHTO (1992).

# Recommendations

## Foundation Types

The recommended foundations are drilled shafts at Piers 1, 2, and 3 and a spread footing at Pier 4. Information for 1.5 and 1.8 m (5 and 6 ft.) diameter shafts, which would minimize disruption of utilities and traffic on Military Road and reduce risks involved with dewatering, are provided. Design parameters for a pile foundation alternative at Pier 1 and spread footing alternatives at Piers 2 and 3 are also provided.

## Soil Properties

The engineering properties used for design are included in Table 6-1. The table references soil types, which refer to numbered soil layers on the subsurface profile of Figure 6-2.

**Table 6-1**
**Design Soil Properties - Bridge 5/506E**

| Soil Type (refer to Fig. 6-2) | Simplified Description | Moist Unit Weight, kN/m$^3$ (pcf) | Saturated Unit Weight, kN/m$^3$ (pcf) | Effective Angle of Internal Friction, degrees | Undrained Cohesive Strength, kPa (ksf) | Young's Modulus, kPa (ksf) |
|---|---|---|---|---|---|---|
| 1 | Loose to medium dense silty sand with gravel (primarily fill) | 19.6 (125) | 20.1 (128) | 33 | - - | 12,000 Interior (250) Interior 19,150 Pier 4 (400) Pier 4 |
| 2 | Stiff, low plasticity silt | 19.2 (122) | 19.6 (125) | - - | 24 (0.5) | - - |
| 3 | Medium dense to dense silty sand with gravel (recessional drift) | 20.4 (130) | 20.7 (132) | 38 | - - | 19,200 (400) |
| 4 | Very dense silty sand with gravel (recessional drift) | 20.4 (130) | 20.7 (132) | 42 | - - | 48,000 (1,000) |

## Pile Foundation (Pier 1)

Both 457 mm (18 in.) and 610 mm (24 in.) closed-end pipe piles were analyzed. Piles will be end-bearing in the recessional drift. Piles must extend through the compressible silt of soil layer 2 to limit settlement. Piles extending at least three diameters into the recessional drift are estimated to have an allowable end-bearing in excess of 8,100 kPa (170 ksf).

If uplift controls the minimum toe elevation of the piles, driving stresses induced by pile driving should be checked, as these may exceed the stresses due to working loads.

Table 6-2 provides allowable axial compression and allowable uplift loads for piles driven to the minimum tip elevation. The values in the table are based on soil strength. The structural capacity of the piles may control design. The allowable loads are net at the butt of the

pile (the weight of the pile is typically neglected in computing the allowable axial compression of piles). Pile spacings of less than three diameters are not recommended.

**Table 6-2**
**Allowable Net Single Pile Loads Based On Soil Strength**

| Pile Diameter, mm (inches) | Minimum Tip Elevation, meters (feet) | Estimated Tip Elevation, meters (feet) | Net Allowable Compression, kN (kips) | Net Allowable Uplift at Minimum Tip Elevation, kN (kips) |
|---|---|---|---|---|
| 457 (18) | 124.0 (407) | 122.6 (400.5) | 1330 (300) | 200 (45) |
| 610 (24) | 124.0 (407) | 123.2 (401) | 1330 (300) | 290 (65) |

Soil properties for lateral analysis are provided for single piles on the attached COM624 forms. The lateral capacity of a pile group may differ from that of a single pile. Group effects occur in lateral loading of piles when the stress field associated with horizontal loading of one pile interacts with an adjacent pile. Generally, a "shadowing" effect occurs, with the trailing pile carrying less load than the leading pile.

The distance at which lateral capacity "shadowing" occurs is subject to continuing discussion within the geotechnical profession. Various reports and textbooks (e.g., NAVFAC, 1986; Poulos and Davis, 1980) introduce efficiency reduction factors if spacing is less than 8 pile diameters. O'Neill et al. (1992) observed a 50 percent reduction for a pile group test at a sand site where piles were spaced at 3 diameters. Other documents (e.g., Lam and Martin, 1986; Reese et al., 1992) suggest little effect at 3 pile diameters.

In an effort to better understand and then resolve this issue of group effects, personal contacts have been made with Professor Hudson Matlock and Professor Lymon Reese, the developers of the p-y approach. Their views were similar: group effects under lateral loading conditions should be negligible when the spacing between piles is at least 4 pile diameters. They did caution that there are limited test data upon which to make any absolute conclusions regarding group effects. They also felt that group effect should be more significant for some soils than others.

On the basis of these discussions, a 20 percent reduction in single-pile lateral capacity is recommended to account for group action when pile spacing is 3 pile diameters.

## Allowable Bearing for Spread Footings (Piers 2, 3 and 4)

Table 6-3 shows the recommended allowable bearing pressures at the three piers with spread footing alternatives. The design criteria limiting total settlement to 25 mm (1 in.) controls the allowable pressure. However, the loose materials anticipated beneath Piers 2 and 3 will result in 38 mm (1.5 in.) of total settlement under working loads of 140 kPa (3 ksf). Settlement at Piers 2 and 3 can be reduced to 25 mm (1 in.) by either:

1. Reducing the allowable bearing pressure to 100 kPa (2 ksf), or
2. Overexcavating to elevation 122.4 m (401. 6 ft.) and replacing the loose materials with gravel borrow or gravel backfill for foundations compacted according to Method C.

Overexcavation will be below the water table, except during the driest periods of the year, and will extend below the base of the existing spread footings. Therefore, temporary support must be provided to protect the stability of the existing footings. Overexcavation is not recommended because of the risk of damaging the existing footings.

Settlement is anticipated to be elastic, occurring as the load is placed. The maximum elevations in the table represent the shallowest footing elevations based on assumed structural limitations. These elevations were used in the settlement estimates. The allowable bearing pressures are not valid if footing elevations are set above the maximum elevations because settlement will increase.

If the footing at Pier 4 is located at least one footing width from the face of the embankment slope, the allowable bearing pressure in Table 6-3 will provide protection from punching shear-type failures. The values in Table 6-3 will also greatly reduce the potential for inducing additional settlement of the existing footings.

**Table 6-3**
**Allowable Bearing Pressures For Spread Footings - Bridge 5/506E**

| Bridge | Pier | Maximum Elevation, m (feet) | Allowable Bearing Pressure, kPa (ksf) | Comments |
|---|---|---|---|---|
| 5/506E | 2 | 124.8 (409.4) | 140 (3) see note! | Anticipate perched groundwater |
| 5/506E | 3 | 122.4 (401.6) | 140 (3) | Anticipate groundwater |
| 5/506E | 4 | 129.8 (425.9) | 240 (5) | |

Note: An allowable bearing pressure of 140 kPa (3 ksf) at Pier 2 will result in 38 mm (1.5 in.) of total settlement. Settlement can be reduced to 25 mm (1 inch) by overexcavating to elevation 122.9 m (401.6 ft.) and backfilling with gravel backfill or gravel backfill for foundations compacted in accordance with Method C.

The original (1959) design drawings indicate that the existing spread footings have an allowable bearing pressure of 335 kPa (7 ksf). The new footing recommendations from Table 6.3 above are 140 kPa (3 ksf) for the interior piers and 240 kPa (5 ksf) for the pier 4 abutment. The discrepancy between allowable bearing in the existing and new piers can be explained as follows:

- Boring H-22-94, the closest boring to Pier 2, indicates loose silty sand within 2.4 m (8 ft.) of the bottom of the footing. Immediate settlement in this material is anticipated to be relatively high and limits the allowable bearing to 140 kPa (3 ksf) with the 25 mm (1 in.) maximum settlement criteria. The 1958 borings used to design the existing footings did not reveal this material and it does not appear at this low elevation in any of the more recent borings.

- The borings nearest to Pier 3 are H-22-94 and BH-5-92. The log of BH-5-92 shows medium dense material at elevations below the Pier 3 footing, a condition differing from H-22-94. There is no boring at Pier 3, so design is based on the conditions indicated by H-22-94, the more conservative approach.

- The existing footing for the abutment Pier 4 is founded in native material at elevation 123.4 m (405 ft.), 12.2 m (40 ft.) below the NB SR 5 roadway grade. Temporary shoring to place a new footing at this elevation would be impractical. The existing fill is more compressible than the native subgrade, hence the lower allowable bearing pressure.

In theory, the footings for Piers 2 and 3 could be lowered or the loose sand overexcavated and replaced to increase the allowable bearing. Neither of these measures is recommended because of the high groundwater. Temporary support of excavations below the water table in granular material can be difficult. Support of the excavations would be critical to the stability of the existing footings. Dewatering also could induce settlement of the silt beneath Pier 1.

## Drilled Shaft Alternative (Piers 1, 2 and 3)

Figures 6-3 and 6-4 show the allowable capacity in compression and uplift for 1.52 m (5 ft.) and 1.83 m (6 ft.) diameter shafts for various toe elevations. As noted previously, factors of safety of 3.0 and 1.5 were used to compute allowable axial compression and uplift, respectively. The maximum, or highest, toe elevation provided at each pier location in the table is the maximum elevation permissible to limit settlement to 25 mm (1 in.). The tables should not be interpolated beyond these maximum elevations to shorten shaft lengths for lighter loads.

The skin friction used to compute the allowable loads in the figures is based on the assumption that shaft concrete will be poured directly against relatively undisturbed soil in the shaft walls. Permanent casings should not be used without reducing the recommended capacities shown in the figures. If permanent casing is used, the space between the casing and the surrounding soil must be grouted over the full length of the casing and allowable skin friction must be reduced by one-third.

Temporary casing should be required for drilled shafts at Piers 1, 2, and 3 to minimize disturbance of loose subgrade soils below the existing spread footings. The temporary casings should extend to at least elevation 123.5, 121.7, and 121.7 m (405.4, 399.4, and 399.4 ft.) at Piers 1, 2, and 3, respectively. The temporary casings are required at these locations and depths because the existing footings are located above loose to medium dense materials that are susceptible to disturbance during excavation of adjacent shafts.

Design soil properties for use in lateral analyses are shown on the attached COM624 forms. Values of initial tangent modulus of subgrade reaction ($k_s$) and strain corresponding to stress at one-half the maximum total-principal-stress difference ($\varepsilon_{50}$) were estimated from published correlations with relative density (Reese and Wang, 1993). Some judgment should be used in applying these numbers.

## Embankment Slopes

Recommended embankment slopes of 2H:1V or flatter are recommended. The new fill should be keyed into the existing fill by cutting benches into the existing fill, as specified in WSDOT's standard provisions.

## Lateral Earth Loads

The equivalent static lateral earth loads imposed on abutment walls, columns, and footings should be computed as follows:

$$P_h = 0.5 \text{ (unit weight) (wall height)}^2 K$$

Where: K = the appropriate active ($K_a$), passive ($K_p$), or at-rest ($K_o$) static coefficient of lateral earth pressure

A unit weight of 19.6 kN/m$^3$ (125 pcf) should be assumed for backfill materials.

The load $P_h$ will act at one-third the wall height from the base. The following coefficients of lateral earth pressure are recommended:

### Within 2H:1V Sloped Embankments (Piers 1 and 4)

- Passive, $K_p$ = 0.9
- At-rest, $K_o$ = 0.5

At-rest pressures should be used for computing loads on the back of columns and footings within the sloped embankment.

### On Level Ground (Piers 2 and 3)

- Active, $K_a$ = 0.3
- Passive, $K_p$ = 2.5
- At-rest, $K_o$ = 0.5

A factor of safety of 1.5 has been applied to $K_p$ to limit movement. The top 1 m (3 ft.) of soil should be neglected in computing passive resistance.

Lateral pressures generated by surcharge loads should be added to the lateral loading produced by the backfill. Lateral loading is computed by multiplying the vertical pressure at depth by the appropriate lateral earth pressure coefficient. If the surcharge is uniform, it should be applied uniformly between the top and the base of the wall. If the surcharge is limited in lateral extent, such as a large sign foundation, the lateral load applied to the vertical member will vary with depth. The only known surcharge loads at this time are traffic loads, which should be treated as a 0.6 m (2 ft.) soil surcharge.

A friction factor of 0.4 should be used for computing frictional resistance to sliding.

## Seismic Design

The peak, firm ground acceleration for this area is 0.25 (Higgins et al., 1988). This acceleration has a 10 percent probability of being exceeded in a 50-year design period.

Design response spectra as outlined in the AASHTO guide specifications are appropriate for this site. A Type II soil profile should be used for determining the site coefficient. Liquefaction is not a concern at this site.

A Poisson's ratio of 0.3 is recommended for use in determining soil spring constants for spread footings. Table 6-4 provides recommended values of dynamic shear modulus, G, for strains ranging from $10^{-1}$ to $10^{-2}$ percent for spread footing design.

**Table 6-4**
**Recommendations For Dynamic Shear Modulus For Spread Footing Design**

| Bridge | Pier | G at $10^{-2}$% strain MPa (ksf) | G at $10^{-1}$% strain MPa (ksf) |
|---|---|---|---|
| 5/506E | 2, 3 | 31 (650) | 14 (300) |
| 5/506E | 4 | 39 (770) | 16 (330) |

Lateral earth loads on abutments and footings as a result of seismic loading should be computed as follows:

$$P_{he} = 0.5(\text{unit weight})(\text{wall height})^2 K_{ae}$$

Where $K_{ae}$ = The combined static and seismic active earth pressure coefficient.

$K_{ae}$ = 0.65 at Piers 1 and 4 (within sloped embankment)

$K_{ae}$ = 0.35 at Piers 2 and 3 (on level ground)

The combined static and seismic lateral earth load, $P_{he}$, can be assumed to act at mid-wall height as a result of a uniform pressure distribution.

A static lateral earth pressure coefficient without a factor of safety applied should be used to compute the resistance to combined static and seismic loading. Passive earth pressure coefficients of 0.9 and 2.5 are recommended for the abutment and interior piers, respectively.

## Other Considerations

Footings should have at least 0.6 m (2 ft.) of cover for frost protection. The subgrade should be firm and free of cobbles and organic material. The subgrade should be scarified to a minimum depth of 150 mm (6 in.) and recompacted to at least 95 percent of maximum density as specified in Section 2-03.3(14)c, Method C, in the WSDOT Standard Specifications. Prior to scarification, the foundation conditions should be checked by an experienced geotechnical engineer . If loose or soft material is present, overexcavation and replacement with compacted granular material will be necessary. Placement of a leveling course of base rock immediately following foundation excavation or preparation is recommended to protect the foundation soil.

Gravel Backfill for Walls should be used within 1 m (3 ft.) of the abutment walls. Positive drainage in the form of an Underdrain Pipe surrounded by Gravel Backfill for Drains should also be provided at the base of abutment walls. Gravel Backfill for Foundations should be used for backfill around the interior piers.

Approach slabs may be deleted because of the following geotechnical criteria:

1. Post-construction settlement of the embankment is anticipated to be less than 13 mm (0.5 in.)

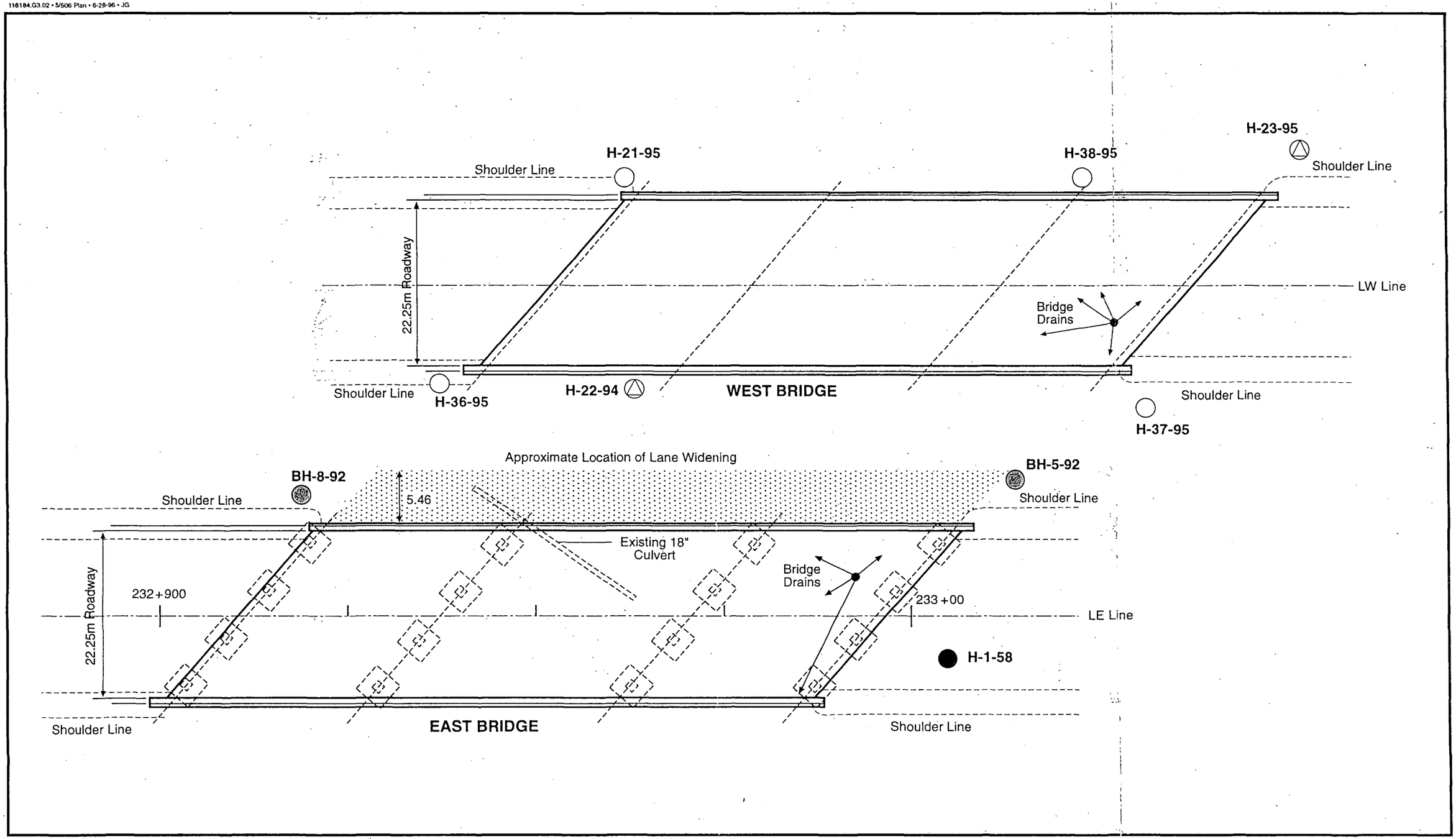
2. Creep settlement is not anticipated.

## Advisory Specifications

The contractor should be aware of potentially difficult conditions that may affect drilled shaft construction. Glacially deposited materials, such as those present at this site, frequently contain cobbles and boulders that may impede drilling progress. Test borings had refusal blow counts and poor sample recovery; these are commonly indicators of large gravel, cobbles, or boulders. Drilled shafts will also extend below the measured ground water table. Excavation stabilization measures that may include the use of temporary casing, slurries, and/or application of increased hydraulic pressures to control shaft sidewall stability will be required. The boring logs and geotechnical report should be consulted for subsurface exploration details.

Temporary casing should be required for drilled shafts at Piers 1, 2, and 3 to minimize disturbance of loose subgrade soils below the existing spread footings. The temporary casings should extend to at least elevation 123.5, 121.7, and 121.7 m (405.4, 399.4, and 399.4 ft.) at Piers 1, 2, and 3, respectively.

116184.G3.02 • 5/506 Plan • 6-28-96 • JG

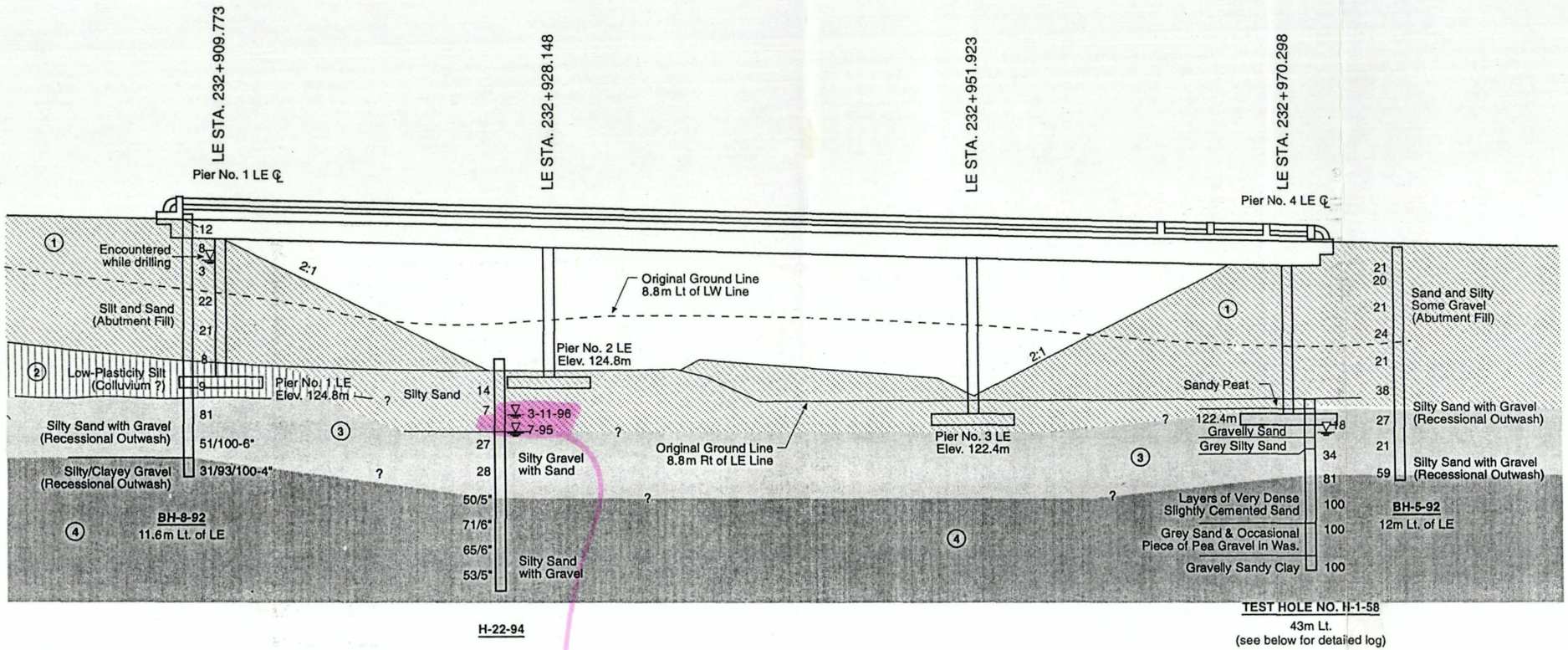



Approximate
Scale in Meters
0 10 m

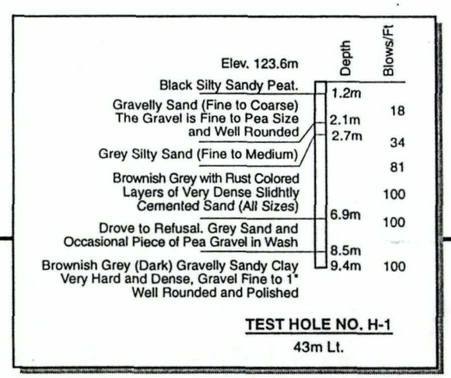
N

Test Hole
HWA Boring
WSDOT Boring
Piezometer

FIGURE 6-1
**Bridge 5/506 East and West Plan View**
Geotechnical Report SR-5
Port of Tacoma I/C to 272nd
Stage 4 HOV




different date than Boring.




Approximate Scale in Meters

0 — 10 m

**LEGEND**

1, 2, 3, 4 — Soil units used to assign design soil properties. See Table 1 for listing of assumed properties

*NOTES:*

Existing spread footings designed for maximum allowable bearing pressure of 335 kPa (3.5 TSF.)

The generalized soil profile shown above was compiled from available subsurface information in the vicinity. It is interpretive in nature; actual soil conditions between borings may vary from those shown

Vertical Datum = NAVD88

1 meter = 3.28 feet

1 inch = 25.4mm

FIGURE 6-2

**Bridge 5/506 East Military Road Profile View**

Geotechnical Report SR-5
Port of Tacoma I/C to 272nd
Stage 4 HOV

## Bridge 5/506E Pier 1

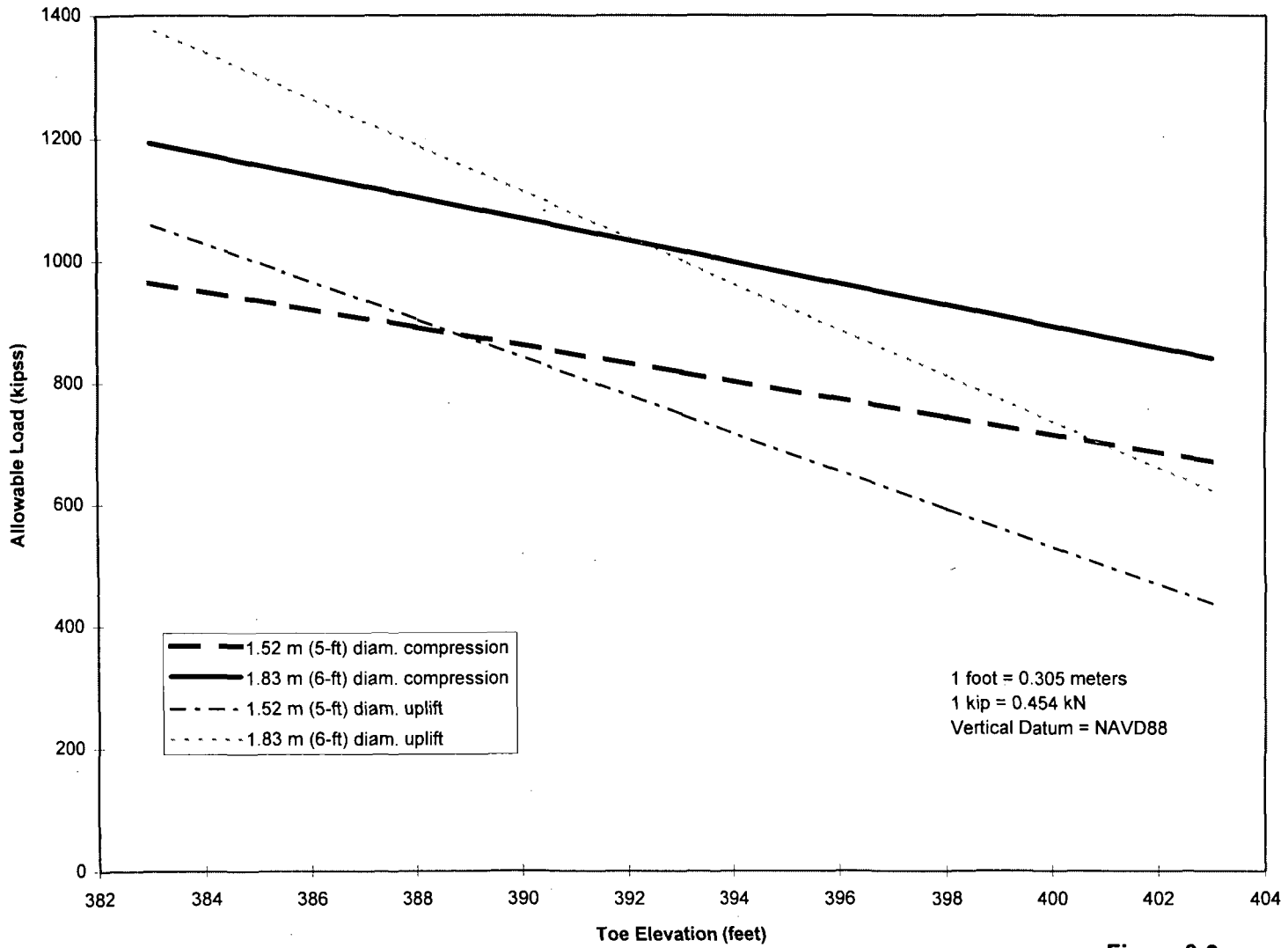



**Figure 6-3**

9/23/96
NAVD88 Chart 2

## Bridge 5/506E Piers 2 & 3

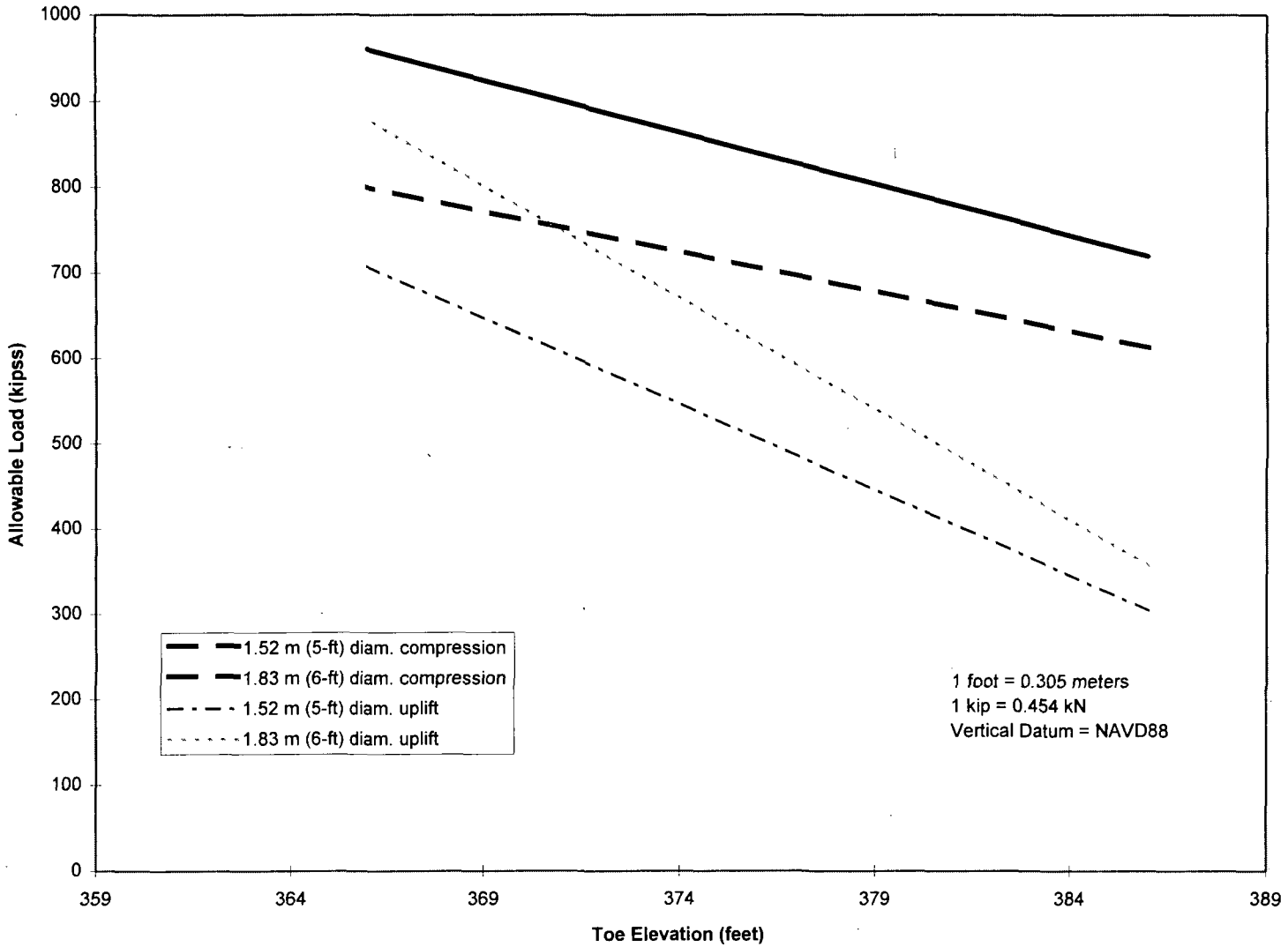



**Figure 6-4**

9/23/96
NAVD88 Chart 1

# Chapter 6
# Appendix

**L-PILE / COM 624 Soil Data Input**
**Bridge 5 /506E**

**Location: Piers 2 and 3**
**No. of Soil Layers :** 4
**Conditions:** **Non-seismic and Seismic**
**Datum NAVD 88** **Reference Elevation :** 123.8 (m) 406 (ft)

| Layer No. | Type of Soil | Depth to Boundry | | | | Effective Unit Weight | | Cohesion | | Int. Angle of Friction | Modulus of Subgrade Reaction (k) | | Strain @ 50% |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| | | Upper | | Lower | | | | | | | | | |
| | | (m) | (ft) | (m) | (ft) | (KN/m*3) | (pcf) | (KPa) | (psf) | (degrees) | (MN/m*3) | (pci) | (ε50) |
| 1 | Sand | 0.0 | 0.0 | 3.0 | 10.0 | 19.6 | 125.0 | 0.0 | 0 | 33 | 24.4 | 90 | |
| 2 | Sand | 3.0 | 10.0 | 3.7 | 12.0 | 9.9 | 63.0 | 0.0 | 0 | 33 | | 60 | |
| 3 | Sand | 3.7 | 12.0 | 8.2 | 27.0 | 11.0 | 70.0 | 0.0 | 0 | 38 | 31.2 | 115 | |
| 4 | Sand | 8.2 | 27.0 | Toe | Toe | 11.0 | 70.0 | 0.0 | 0 | 42 | 46.1 | 170 | |

Note: Reference elevation = assumed effective top of shaft elevation = top of shaft at abutments
~ 1.5 m below existing ground surface at interior piers

9/24/96
506LPILE.XLS

**L-PILE / COM 624 Soil Data Input**
**Bridge 5 /506E**

**Location: Pier 1**
**No. of Soil Layers :** 4
**Conditions:** Non-seismic and Seismic
**Datum NAVD 88** **Reference Elevation :** 131.4 (m) 431 (ft)

| Layer No. | Type of Soil | Depth to Boundry Upper (m) | Depth to Boundry Upper (ft) | Depth to Boundry Lower (m) | Depth to Boundry Lower (ft) | Effective Unit Weight (KN/m*3) | Effective Unit Weight (pcf) | Cohesion (KPa) | Cohesion (psf) | Int. Angle of Friction (degrees) | Modulus of Subgrade Reaction (k) (MN/m*3) | Modulus of Subgrade Reaction (k) (pci) | Strain @ 50% (ε50) |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| 1 | Sand | 0.0 | 0.0 | 5.0 | 16.5 | 19.6 | 125.0 | 0.0 | 0 | 33 | 24.4 | 90 | |
| 2 | Soft Clay | 5.0 | 16.5 | 8.4 | 27.5 | 19.2 | 122.0 | 23.9 | 500 | 0 | | | 0.010 |
| 3 | Sand | 8.4 | 27.5 | 14.8 | 48.5 | 11.0 | 70.0 | 0.0 | 0 | 38 | 31.2 | 115 | |
| 4 | Sand | 14.8 | 48.5 | Toe | Toe | 11.0 | 70.0 | 0.0 | 0 | 42 | 46.1 | 170 | |

Note: Reference elevation = assumed effective top of shaft elevation = top of shaft at abutments
~ 1.5 m below existing ground surface at interior piers

9/24/96
506LPILE.XLS

# LOG OF TEST BORING

Washington State Department of Transportation

HOLE No. H-21-95

PROJECT Pierce County Line to Tukwila Stage 3 — Job No. OL-1863

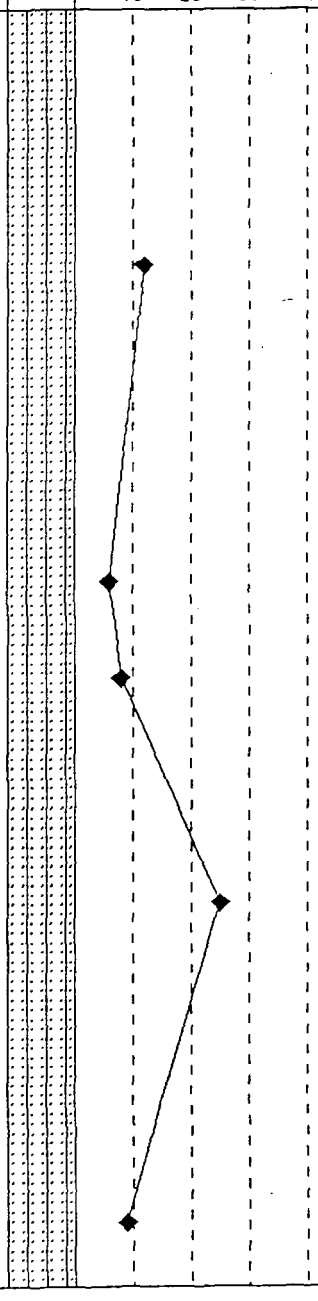
Bridge 5/506 — S.R. 5

Station 232+937.61 — Offset 8.99m Lt. — C.S. 1727

Equipment — Casing — Ground El 440.6 (134.29 m)

Method of Boring Augers

Start Date January 4, 1995 — Completion Date January 4, 1995 — Sheet 1 of 3




| Depth (ft) | Meters (m) | SPT Blows/6" (N) | Sample Type | Sample No. (Tube No.) | Lab Tests | Description of Material | Groundwater | Instrument |
|---|---|---|---|---|---|---|---|---|
| | | | | | | 1 ft. = 0.3048 m. | | |
| 5 | 1 | 3 6 6 (12) | | D-1 | GS MC | SM, M.C. = 8% Silty SAND with gravel, subrounded, medium dense, brownish gray, moist. (Fill). Retained 0.9 ft. | | |
| 10 | 2 3 | 2 3 3 (6) | | D-2 | | Silty SAND with gravel, subrounded, loose, gray, wet. (Fill). Retained 1.0 ft. | | |
| | | 3 4 4 (8) | | D-3 | GS MC | SM, M.C. = 11% Silty SAND with gravel, subrounded, loose, gray, moist. | | |
| 15 | 4 5 | 11 11 14 (25) | | D-4 | GS MC | SM, M.C. = 9% Silty SAND with gravel, subrounded, dense, gray, moist. (Fill). Retained 1.2 ft. | | |
| 20 | 6 | 10 5 | | D-5 | | Silty SAND with gravel, subrounded, loose, gray, wet. (Fill). Retained 0.6 ft. | | |

# LOG OF TEST BORING

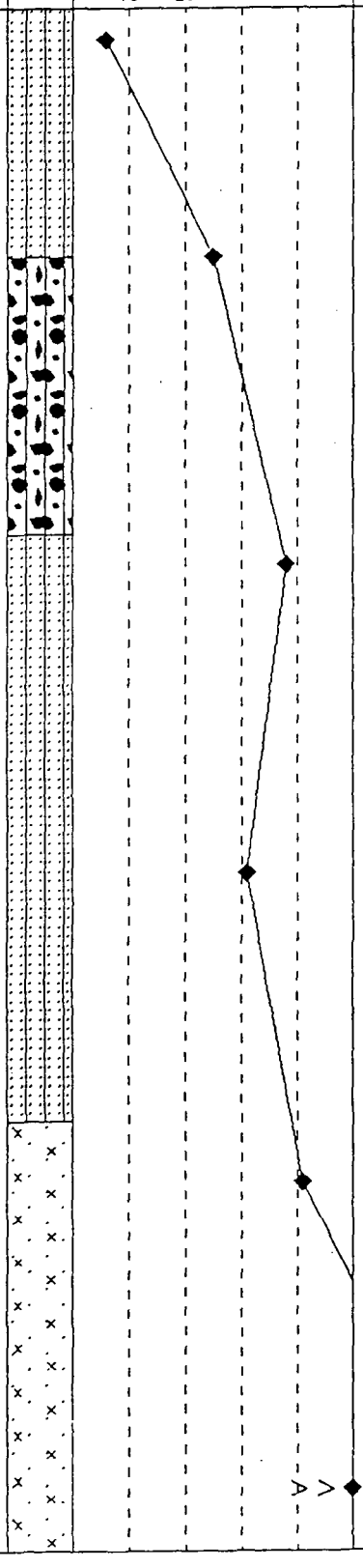
Washington State Department of Transportation

HOLE No. H-21-95

Sheet 2 of 3

PROJECT Pierce County Line to Tukwila Stage 3

Job No. OL-1863




| Depth (ft) | Meters (m) | SPT Blows/6" (N) | Sample Type | Sample No. (Tube No.) | Lab Tests | Description of Material | Groundwater | Instrument |
|---|---|---|---|---|---|---|---|---|
| | | 4 (9) 3 3 3 (6) | | D-6 | GS MC | SM, M.C. = 15% Silty SAND, loose, gray, moist. | | |
| 25 | 7 | 6 10 15 (25) | | D-7 | GS MC | GM, M.C. = 12% Silty GRAVEL with sand, subrounded, dense, gray, wet. *Residual soil.* Retained 1.2 ft. | | |
| 30 | 8 9 | 8 19 19 (38) | | D-8 | GS MC | SM, M.C. = 13% Silty SAND with gravel, subrounded, dense, gray, wet. Residual soil. Retained 0.4 ft. | | |
| 35 | 10 11 | 8 11 20 (31) | | D-9 | | Silty SAND with gravel, subrounded, dense, brown, wet. *Residual soil.* Retained 1.5 ft. | | |
| 40 | 12 13 | 5 12 29 (41) | | D-10 | GS MC | ML, M.C. = 13% Sandy SILT with gravel, subrounded, hard, greenish gray, moist. Residual soil. Retained 1.1 ft. | | |
| 45 | | 5 18 | | D-11 | | Sandy SILT with gravel, subrounded, very hard, greenish gray, moist. Residual soil. | | |

# LOG OF TEST BORING

Washington State Department of Transportation

HOLE No. H-21-95

Sheet 3 of 3

PROJECT Pierce County Line to Tukwila Stage 3

Job No. OL-1863

| Depth (ft) | Meters (m) | Profile | Standard Penetration Blows/ft (10 20 30 40) | SPT Blows/6" (N) | Sample Type | Sample No. (Tube No.) | Lab Tests | Description of Material | Groundwater | Instrument |
|---|---|---|---|---|---|---|---|---|---|---|
| | | | | 50 (68) | | | | Retained 1.5 ft. | | |
| | 14 | | | | | | | | | |
| | 15 | | | 16 27 38 (65) | | D-12 | | Sandy SILT with gravel, subrounded, very hard, greenish gray. Residual soil. Retained 1.2 ft. | | |
| 50 | | | | | | | | | | |
| | | | | | | | | End of test hole boring at 50.5 ft. below ground elevation. | | |
| | 16 | | | | | | | Water table elevation not determined. | | |
| | | | | | | | | This is a summary Log of Test Boring. Soil/Rock descriptions are derived from visual field identifications and laboratory test data. | | |
| 55 | | | | | | | | | | |
| | 17 | | | | | | | | | |
| | 18 | | | | | | | | | |
| 60 | | | | | | | | | | |
| | 19 | | | | | | | | | |
| 65 | | | | | | | | | | |
| | 20 | | | | | | | | | |
| | 21 | | | | | | | | | |
| 70 | | | | | | | | | | |

# LOG OF TEST BORING

Washington State Department of Transportation

HOLE No. H-22-94

PROJECT Pierce County Line to Tukwila Stage 3 — Job No. OL-1863

Bridge 5/506 — S.R. 5

Station 232+947.52 — Offset 8.69m Rt. — C.S. 1727

Equipment — Casing HW 22', HQ 47' — Ground El 411.4 (125.39 m)

Method of Boring Wet Rotary

Start Date November 15, 1994 — Completion Date November 16, 1994 — Sheet 1 of 3

| Depth (ft) | Meters (m) | Profile | Standard Penetration Blows/ft (10 20 30 40) | SPT Blows/6" (N) | Sample Type | Sample No. (Tube No.) | Lab Tests | Description of Material | Groundwater | Instrument |
|---|---|---|---|---|---|---|---|---|---|---|
| | | | | | | | | 1 ft. = 0.3048 m. | | |
| | | | | | | | | Fill. | | |
| | 1 | | | | | | | | | |
| 5 | | | | | | | | | | |
| | 2 | | | 10<br>8<br>6<br>(14) | | D-1 | | Silty SAND with gravel, rounded to angular, medium dense, light brown, dry to moist. Fill<br>Retained 0.3 ft. | | |
| 10 | 3 | | | | | | | | | |
| | | | | 7<br>4<br>3<br>(7) | | D-2 | GS<br>MC | February 23, 1995<br>SM, M.C. = 15%<br>Silty SAND with gravel, rounded to angular, loose, light brown, moist. Fill.<br>Retained 0.7 ft. | | |
| | 4 | | | | | | | | | |
| | | | | | | | | July 31, 1995 | | |
| 15 | | | | | | | | | | |
| | 5 | | | 12<br>15<br>12<br>(27) | | D-3 | GS<br>MC | SM, M.C. = 11%<br>Silty SAND with gravel, angular, dense, gray, moist.<br>Retained 0.7 ft. | | |
| | 6 | | | | | | | | | |
| 20 | | | | | | | | | | |

# LOG OF TEST BORING

Washington State Department of Transportation

HOLE No. H-22-94

Sheet 2 of 3

PROJECT Pierce County Line to Tukwila Stage 3

Job No. OL-1863

| Depth (ft) | Meters (m) | Profile | Standard Penetration Blows/ft (10 20 30 40) | SPT Blows/6" (N) | Sample Type | Sample No. (Tube No.) | Lab Tests | Description of Material | Groundwater | Instrument |
|---|---|---|---|---|---|---|---|---|---|---|
| | 7 | | | 15 15 13 (28) | | D-4 | GS MC | GM, M.C. = 11% Silty GRAVEL with sand, rounded to angular, dense, gray with some orange, moist. Retained 0.5 ft. | | |
| 25 | 8 | | | 20 25 55 (80) | | D-5 | | Silty GRAVEL with sand, rounded to subangular, very dense, gray with rust, moist. Retained 1.0 ft. | | |
| 30 | 9 | | | 40 50/5 (50/5") | | D-6 | GS MC | GM, M.C. = 12% Silty GRAVEL with sand, rounded to angular, very dense, gray, moist. Retained 0.5 ft. | | |
| 35 | 10 | | | | | | | | | |
| | 11 | | | 71/6" (71/6") | | D-7 | | Silty GRAVEL with sand, rounded to angular, very dense, gray, moist. Retained 0.5 ft. | | |
| 40 | 12 | | | | | | | | | |
| | 13 | | | 65/6" (65/6") | | D-8 | GS MC | SM, M.C. = 17% Silty SAND with gravel, subrounded to subangular, very dense, gray, moist. Retained 0.3 ft. | | |
| 45 | | | | | | | | | | |

# LOG OF TEST BORING

Washington State Department of Transportation

HOLE No. H-22-94

Sheet 3 of 3

PROJECT Pierce County Line to Tukwila Stage 3

Job No. OL-1863

| Depth (ft) | Meters (m) | Profile | Standard Penetration Blows/ft (10 20 30 40) | SPT Blows/6" (N) | Sample Type | Sample No. (Tube No.) | Lab Tests | Description of Material | Groundwater | Instrument |
|---|---|---|---|---|---|---|---|---|---|---|
| | 14 | | | | | | | | | |
| | | | > > ◆ | 32<br>53/5<br>(53/5") | ▼▲ | D-9 | GS<br>MC | SM, M.C. = 13%<br>Silty SAND with gravel, subrounded to angular, very hard, moist, gray.<br>Retained 0.8 ft. | | |
| | 15 | | | | | | | End of test hole boring at 47.9 ft. below ground elevation.<br><br>This is a summary Log of Test Boring. Soil/Rock descriptions are derived from visual field identifications and laboratory test data. | | |
| 50 | | | | | | | | | | |
| | 16 | | | | | | | | | |
| 55 | | | | | | | | | | |
| | 17 | | | | | | | | | |
| | 18 | | | | | | | | | |
| 60 | | | | | | | | | | |
| | 19 | | | | | | | | | |
| 65 | | | | | | | | | | |
| | 20 | | | | | | | | | |
| | 21 | | | | | | | | | |
| 70 | | | | | | | | | | |

# LOG OF TEST BORING

Washington State Department of Transportation

HOLE No. H-23-95

PROJECT Pierce County Line to Tukwila Stage 3 — Job No. OL-1863

Bridge 5/506 — S.R. 5

Station 233+008.63 — Offset 9.90m Lt. — C.S. 1727

Equipment — Casing HQ to 50' — Ground El 435.7 (132.80 m)

Method of Boring

Start Date January 17, 1995 — Completion Date January 17, 1995 — Sheet 1 of 3

| Depth (ft) | Meters (m) | Profile | Standard Penetration Blows/ft (10 20 30 40) | SPT Blows/6" (N) | Sample Type | Sample No. (Tube No.) | Lab Tests | Description of Material | Groundwater | Instrument |
|---|---|---|---|---|---|---|---|---|---|---|
| | | | | | | | | 1 ft. = 0.3048 m. | | |
| 5 | 1, 2 | | | 10 8 5 (13) | | D-1 | GS MC | SM, M.C. = 11% Silty SAND with gravel, subrounded, medium dense, light olive brown, moist, homogeneous, no HCl reaction. Retained 0.9 ft. | | |
| 10 | 3, 4 | | | 6 5 6 (11) | | D-2 | | Silty SAND with gravel, subrounded, medium dense, gray, with some mottling, moist, homogeneous, no HCl reaction. Retained 1.0 ft. | | |
| 15 | 5 | | | 21 22 18 (40) | | D-3 | GS MC | SM, M.C. = 10% Silty SAND with gravel, subrounded, dense, gray with some mottling, moist, homogeneous, no HCl reaction. Retained 1.2 ft. | | |
| 20 | 6 | | | | | | | | | |

# LOG OF TEST BORING

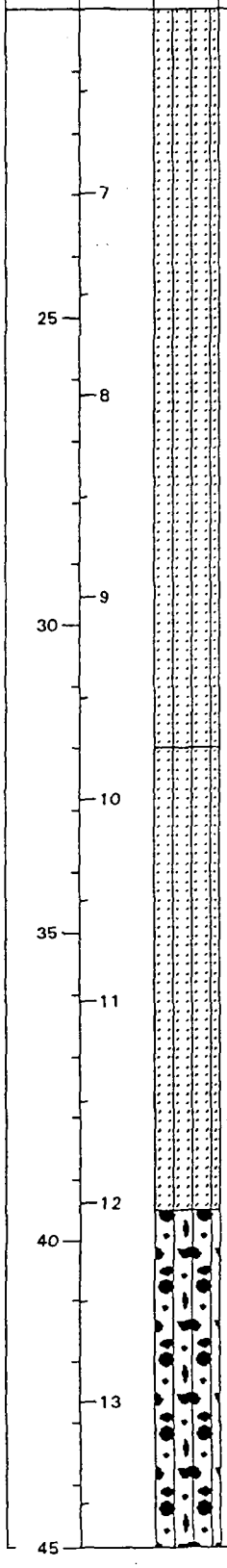
Washington State Department of Transportation

HOLE No. H-23-95

Sheet 2 of 3

PROJECT Pierce County Line to Tukwila Stage 3

Job No. OL-1863



| Depth (ft) | Meters (m) | Standard Penetration Blows/ft (10 20 30 40) | SPT Blows/6" (N) | Sample Type | Sample No. (Tube No.) | Lab Tests | Description of Material | Groundwater | Instrument |
|---|---|---|---|---|---|---|---|---|---|
| | | | 10<br>10<br>9<br>(19) | | D-4 | GS<br>MC | SM, M.C. = 17%<br>Sillty SAND with gravel, subrounded, medium dense, grayish brown, moist, homogeneous (organics present). No HCl reaction.<br>Retained 0.7 ft. | | |
| 25 | 7 | | 13<br>13<br>20<br>(33) | | D-5 | | Silty SAND with gravel, subrounded, dense, light brownish gray, moist, homogeneous, no HCl reaction.<br>Retained 1.0 ft. | | |
| 30 | 8<br>9 | > > | 22<br>24<br>32<br>(56) | | D-6 | | Silty SAND with gravel, subrounded, very dense, light brownish gray, moist, homogeneous, no HCl reaction.<br>Retained 0.8 ft. | | |
| 35 | 10<br>11 | | 29<br>50/4<br>(50/4") | | D-7 | GS<br>MC | SM, M.C. = 18%<br>Silty SAND, subrounded, very dense, light brownish gray with some mottling, wet, homogeneous, no HCl reaction.<br>Retained 0.7 ft. | ▽ | |
| 40 | 12<br>13 | > > | 38<br>40<br>50/3" | | D-8 | GS<br>MC | GW-GM, M.C. = 11%<br>Well graded GRAVEL with silt and sand, angular to subrounded, very dense, pale brown (minor mottling), wet, homogeneous, no HCl reaction.<br>Retained 1.2 ft. | | |
| 45 | | | | | | | | | |

# LOG OF TEST BORING

Washington State Department of Transportation

HOLE No. H-23-95

Sheet 3 of 3

PROJECT Pierce County Line to Tukwila Stage 3

Job No. OL-1863

| Depth (ft) | Meters (m) | Profile | Standard Penetration Blows/ft (10 20 30 40) | SPT Blows/6" (N) | Sample Type | Sample No. (Tube No.) | Lab Tests | Description of Material | Groundwater | Instrument |
|---|---|---|---|---|---|---|---|---|---|---|
| | 14 | | | 24 50/4 (50/4") | | D-9 | GS MC | SM, M.C. = 19% Silty SAND, subangular to subrounded, very dense, pale brown with some mottling, moist, homogeneous, (0.1 ft. thick silty sand layer at 45.4 to 45.5 ft.), no HCl reaction. Retained 0.8 ft. | | |
| | 15 | | | | | | | | | |
| 50 | | | > > | 20 34 48 (82) | | D-10 | GS MC | SP-SM, M.C. = 26% Poorly graded SAND with silt, very dense, brownish gray, (rusty from 50.1 ft. to 50.6 ft.), wet, homogeneous, (one silt layer at 51.3 ft. -- shows liquefaction structure 0.03 ft. thick, bagged as D10-B), no HCl reaction. Retained 1.5 ft. | | |
| | 16 | | | | | | | End of test hole boring at 51.5 ft. below ground elevation. | | |
| 55 | | | | | | | | This is a summary Log of Test Boring. Soil/Rock descriptions are derived from visual field identifications and laboratory test data. | | |
| | 17 | | | | | | | | | |
| | 18 | | | | | | | | | |
| 60 | | | | | | | | | | |
| | 19 | | | | | | | | | |
| 65 | | | | | | | | | | |
| | 20 | | | | | | | | | |
| | 21 | | | | | | | | | |
| 70 | | | | | | | | | | |

# LOG OF TEST BORING

Washington State Department of Transportation

HOLE No. H-36-95

PROJECT Pierce County Line to Tukwila Stage 3 — Job No. OL-1863

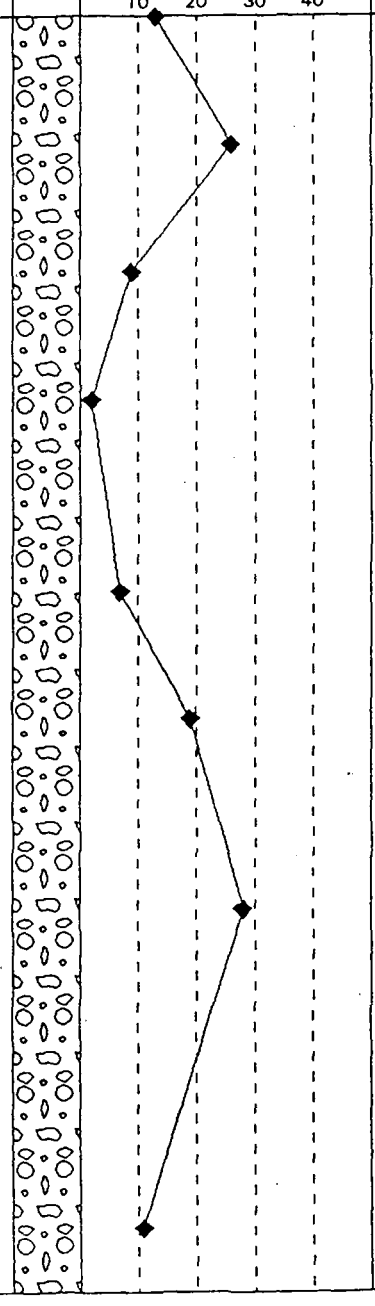
Bridge 5/506 — S.R. 5

Station 232+923.51 — Offset 10.55 m Rt. — C.S. 1727

Equipment — Casing 4" OD X 55' Augers — Ground El 438.0 (133.50 m)

Method of Boring Dry Rotary

Start Date October 7, 1995 — Completion Date October 7, 1995 — Sheet 1 of 3




| Depth (ft) | Meters (m) | SPT Blows/6" (N) | Sample Type | Sample No. (Tube No.) | Lab Tests | Description of Material | Groundwater | Instrument |
|---|---|---|---|---|---|---|---|---|
| | | 3 5 8 18 (13) | | D-1 | | 1 ft. = 0.3048 m. Well graded GRAVEL with sand and silt, rounded to subrounded, medium dense, brown, moist. Fill. Retained 0.9 ft. | | |
| | | 14 14 12 10 (26) | | D-2 | | Well graded GRAVEL with silt and sand, rounded to subrounded, dense, brown, moist. Fill. Retained 1.0 ft. | | |
| 5 | 1 | 5 5 4 2 (9) | | D-3 | | Well graded GRAVEL with silt and sand, rounded to subrounded, loose, brown, moist. Fill. Retained 0.5 ft. | | |
| | 2 | 2 1 1 1 (2) | | D-4 | | Well graded GRAVEL with silt and sand, very loose, brown, moist. Fill or replacement material. Retained 0.5 ft. | | |
| 10 | 3 | 2 3 4 4 (7) | | D-5 | | Well graded GRAVEL with silt and sand, loose, brown to dark gray, moist. Fill. Retained 0.8 ft. | | |
| | | 3 11 8 7 (19) | | D-6 | | Well graded GRAVEL with silt, sand and clay, rounded, medium dense, brown, moist. Fill. Retained 0.9 ft. | | |
| 15 | 4 5 | 15 12 16 (28) | | D-7 | | Well graded GRAVEL with silt and sand, rounded to subrounded, dense, grayish brown, moist. Fill. Retained 1.0 ft. | | |
| 20 | 6 | 11 5 | | D-8 | | Well graded GRAVEL with silt and sand, rounded, medium dense, grayish brown, moist, organics. | | |

# LOG OF TEST BORING

Washington State Department of Transportation

HOLE No. H-36-95

Sheet 2 of 3

PROJECT Pierce County Line to Tukwila Stage 3

Job No. OL-1863

| Depth (ft) | Meters (m) | Profile | Standard Penetration Blows/ft (10 20 30 40) | SPT Blows/6" (N) | Sample Type | Sample No. (Tube No.) | Lab Tests | Description of Material | Groundwater | Instrument |
|---|---|---|---|---|---|---|---|---|---|---|
| | | | | 6 (11) | | | | Retained 1.0 ft. | | |
| | 7 | | | | | | | | | |
| 25 | | | | 2 3 6 (9) | | D-9 | | Silty SAND with gravel, rounded, loose, gray, moist, organics, loosely bonded together with a silt matrix. Retained 0.5 ft. | | |
| | 8 | | | | | | | | | |
| 30 | 9 | | | 6 5 5 (10) | | D-10 | | Silty SAND with gravel, rounded to angular, loose, dark brown, moist, organics. Retained 0.8 ft. | | |
| | 10 | | | | | | | | | |
| 35 | 11 | | | 5 7 7 (14) | | D-11 | | Silty SAND with gravel, rounded to angular, medium dense, gray, moist. Retained 1.2 ft. | | |
| | | | | | | | | October 7, 1995 | ▽ | |
| 40 | 12 | | | 10 12 9 (21) | | D-12 | | Well graded GRAVEL with well graded sand and silt, rounded to subrounded, medium dense, brownish gray, wet. Retained 1.2 ft. | | |
| | 13 | | | | | | | | | |
| 45 | | | > | 13 23 | | D-13 | | Well graded GRAVEL with silt and sand, rounded to subrounded, very dense, brown, moist, moderately | | |

# LOG OF TEST BORING

Washington State Department of Transportation

HOLE No. H-36-95

Sheet 3 of 3

PROJECT Pierce County Line to Tukwila Stage 3

Job No. OL-1863

| Depth (ft) | Meters (m) | Profile | Standard Penetration Blows/ft (10 20 30 40) | SPT Blows/6" (N) | Sample Type | Sample No. (Tube No.) | Lab Tests | Description of Material | Groundwater | Instrument |
|---|---|---|---|---|---|---|---|---|---|---|
| | | | | 34 (57) | | | | bonded with a silt matrix. Retained 1.5 ft. | | |
| | 14 | | | | | | | | | |
| | 15 | | > > | 15 16 45 (61) | | D-14 | | Well graded GRAVEL with silt and sand, very dense, dark gray, moist, loosely bonded with silt. Retained 1.0 ft. | | |
| 50 | | | | | | | | | | |
| | 16 | | | | | | | | | |
| | | | > > | 15 52 60 (112) | | D-15 | | Poorly graded SAND with well graded gravel and silt, rounded to subrounded, very dense, dark gray, wet. Retained 1.5 ft. | | |
| 55 | | | | | | | | | | |
| | 17 | | | | | | | End of test hole boring at 55.5 ft. below ground elevation. | | |
| | | | | | | | | This is a summary Log of Test Boring. Soil/Rock descriptions are derived from visual field identifications. | | |
| | 18 | | | | | | | | | |
| 60 | | | | | | | | | | |
| | 19 | | | | | | | | | |
| 65 | | | | | | | | | | |
| | 20 | | | | | | | | | |
| | 21 | | | | | | | | | |
| 70 | | | | | | | | | | |

# LOG OF TEST BORING

Washington State Department of Transportation

HOLE No. H-37-95

PROJECT Pierce County Line to Tukwila Stage 3 — Job No. OL-1863

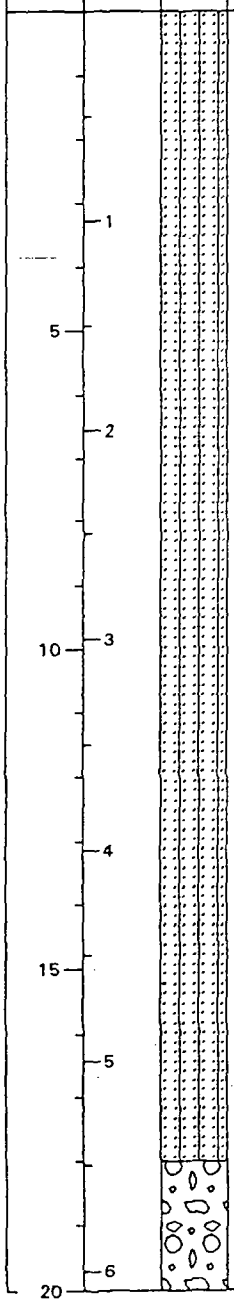
Bridge 5/506 — S.R. 5

Station 232+990.41 — Offset 11.16 m Rt. — C.S. 1727

Equipment — Casing 4" OD X 50' Augers — Ground El 429.8 (131.00 m)

Method of Boring Dry Rotary

Start Date October 7, 1995 — Completion Date October 8, 1995 — Sheet 1 of 3




| SPT Blows/6" (N) | Sample Type | Sample No. (Tube No.) | Lab Tests | Description of Material | Groundwater | Instrument |
|---|---|---|---|---|---|---|
| 4 7 9 15 (16) | | D-1 | | 1 ft. = 0.3048 m. Well graded GRAVEL with silt and sand, rounded, medium dense, brown, moist. Fill. Retained 1.5 ft. | | |
| 15 17 16 12 (33 | | D-2 | | Well graded GRAVEL with well graded sand, silt and wood debris, rounded to angular, dense, brown, moist. Fill. Retained 1.0 ft. | | |
| 5 9 7 12 (16) | | D-3 | | Well graded GRAVEL with silt and sand, rounded to subangular, medium dense, brown, moist. Fill. Retained 1.3 ft. | | |
| 13 12 10 9 (22) | | D-4 | | Well graded GRAVEL with silt and sand, rounded to subangular, medium dense, brown, moist. Fill. Retained 1.3 ft. | | |
| 6 9 20 25 (29) | | D-5 | | Well graded GRAVEL with silt and sand, rounded to subangular, medium dense, brown, moist. Fill. Retained 1.2 ft. | | |
| 17 12 10 11 (22 | | D-6 | | Well graded GRAVEL with silt and sand, rounded to subrounded, medium dense, brown, moist. Fill. Retained 1.2 ft. | | |
| 7 23 13 4 (36) | | D-7 | | Well graded GRAVEL with silt and sand, rounded, dense, brown,moist, loosely bonded together with silt. Fill. Retained 1.1 ft. | | |
| 20 13 | | D-8 | | Well graded GRAVEL with silt and sand, rounded, dense, brownish gray, moist. | | |

# LOG OF TEST BORING

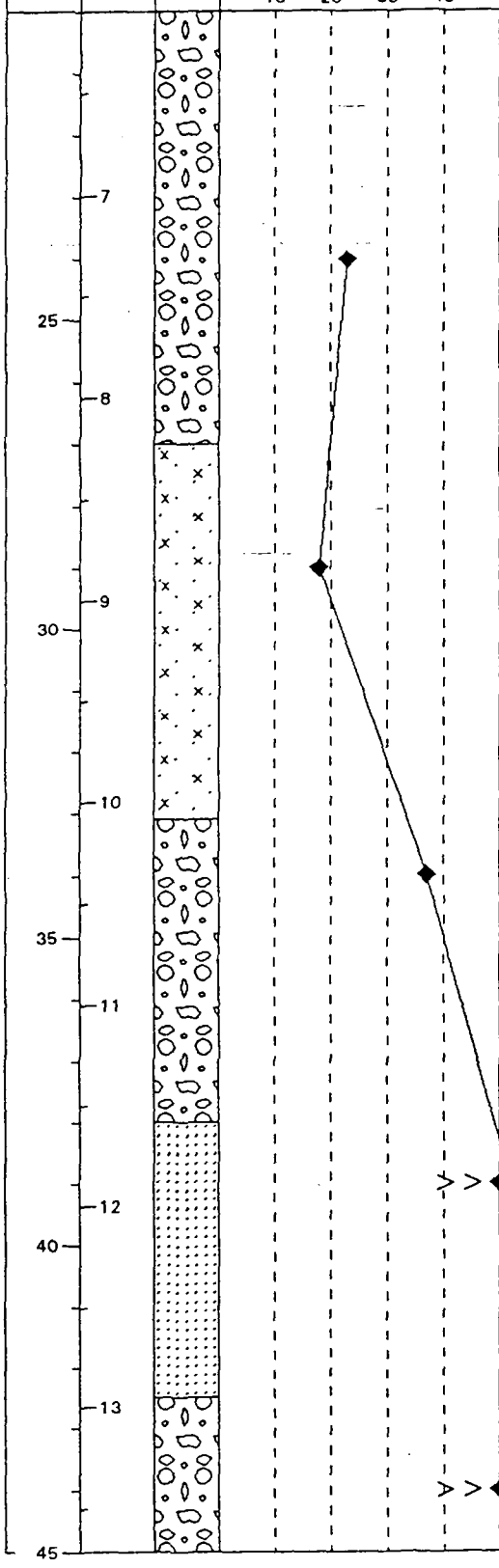
Washington State Department of Transportation

HOLE No. H-37-95

Sheet 2 of 3

PROJECT Pierce County Line to Tukwila Stage 3

Job No. OL-1863




| SPT Blows/6" (N) | Sample Type | Sample No. (Tube No.) | Lab Tests | Description of Material | Groundwater | Instrument |
|---|---|---|---|---|---|---|
| 14 (27) | | | | Retianed 0.9 ft. | | |
| 4 8 15 (23) | | D-9 | | Perched water table at 24.0 ft. Well graded GRAVEL with silt and sand, angular to subrounded, medium dense, dark brown, wet. Retained 0.4 ft. | | |
| 4 7 11 (18) | | D-10 | | Sandy SILT with gravel, angular, medium dense, brown, moist, traces of dark brown to light gray oxidize stains and clay. Retained 0.8 ft. | | |
| 11 19 18 (37) | | D-11 | | October 8, 1995 Well graded GRAVEL with well graded sand and silt, rounded to subrounded, dense, brown, wet. Retained 1.0 ft. | | |
| 6 10 42 (52) | | D-12 | | Poorly graded SAND with silt lenses, very dense, brown, wet. Retained 1.5 ft. | | |
| 46 65 | | D-13 | | WEll graded GRAVEL with well graded sand, rounded to subrounded, very dense, dark gray, | | |

# LOG OF TEST BORING

Washington State Department of Transportation

HOLE No. H-37-95

Sheet 3 of 3

PROJECT Pierce County Line to Tukwila Stage 3

Job No. OL-1863

| Depth (ft) | Meters (m) | Profile | Standard Penetration Blows/ft (10 20 30 40) | SPT Blows/6" (N) | Sample Type | Sample No. (Tube No.) | Lab Tests | Description of Material | Groundwater | Instrument |
|---|---|---|---|---|---|---|---|---|---|---|
| | | | | (65/6") | | | | moist, loosely bonded together with silt. Less than 5% silt. Retained 1.0 ft. | | |
| | 14 | | | | | | | | | |
| | 15 | | > > | 44 100 (100/6") | | D-14 | | Well graded GRAVEL with well graded sand, rounded to subrounded, very dense, dark gray, moist, loosely bonded together with silt, less than 5% silt. Retained 1.0 ft. | | |
| 50 | | | | | | | | End of test hole boring at 50.0 ft. below ground elevation. | | |
| | 16 | | | | | | | This is a summary Log of Test Boring. Soil/Rock descriptions are derived from visual field identifications. | | |
| 55 | | | | | | | | | | |
| | 17 | | | | | | | | | |
| | 18 | | | | | | | | | |
| 60 | | | | | | | | | | |
| | 19 | | | | | | | | | |
| 65 | | | | | | | | | | |
| | 20 | | | | | | | | | |
| | 21 | | | | | | | | | |
| 70 | | | | | | | | | | |

# LOG OF TEST BORING

Washington State Department of Transportation

HOLE No. H-38-95

PROJECT Pierce County Line to Tukwila Stage 3 — Job No. OL-1863

Bridge 5/506 — S.R. 5

Station 232+984.80 — Offset 10.15 m Lt. — C.S. 1727

Equipment CME 45 — Casing — Ground El 410.7 (125.18 m)

Method of Boring

Start Date October 7, 1995 — Completion Date October 7, 1995 — Sheet 1 of 3

| Depth (ft) | Meters (m) | Profile | Standard Penetration Blows/ft (10 20 30 40) | SPT Blows/6" (N) | Sample Type | Sample No. (Tube No.) | Lab Tests | Description of Material | Groundwater |
|---|---|---|---|---|---|---|---|---|---|
| | | | | | | | | 1 ft. = 0.3048 m. | |
| 5 | 1, 2 | | | 15<br>40<br>(40/6")<br>43/6"<br>(43/6") | | D-1<br>D-2 | | Well graded GRAVEL with silt and sand, very dense, brown, moist.<br>Recovered 0.7 ft.<br>Well graded GRAVEL with silt and sand, very dense, brown. | |
| 10 | 3 | | | 13<br>14<br>15<br>(29) | | D-3 | | Well graded GRAVEL with silt and sand, dense, brown, moist.<br>Recovered 0.7 ft. | |
| | | | | 18<br>17<br>17<br>(34) | | D-4 | | Well graded GRAVEL with silt and sand, dense, brown, moist.<br>Recovered 0.7 ft. | |
| 15 | 4, 5 | | | 31<br>26<br>24<br>(50) | | D-5 | | Well graded GRAVEL with silt and sand, very dense, brown-gray, moist.<br>Recovered 0.9 ft. | |
| 20 | 6 | | | | | | | | |

# LOG OF TEST BORING

Washington State Department of Transportation

HOLE No. H-38-95

Sheet 2 of 3

PROJECT Pierce County Line to Tukwila Stage 3

Job No. OL-1863

| Depth (ft) | Meters (m) | Profile | Standard Penetration Blows/ft (10 20 30 40) | SPT Blows/6" (N) | Sample Type | Sample No. (Tube No.) | Lab Tests | Description of Material | Groundwater | Instrument |
|---|---|---|---|---|---|---|---|---|---|---|
| | | | | 50/3" (50/3") | | D-6A | | Well graded GRAVEL with silt and sand, very dense, gray, moist. Recovered 0.3 ft. | | |
| 25 | 7 | | | 24 50 (50/6") | | D-7B | | Gravelly SILT with sand, gravel and sand layers are scattered, very hard, gray, moist. Recovered 1.0 ft. | | |
| 30 | 8 9 | | | 50/6" (50/6") | | D-7 | | Well graded SAND with silt and gravel, very dense, gray, moist. Recovered 0.5 ft. | | |
| 35 | 10 11 | | | 50/2" (50/2") | | D-8 | | Well graded GRAVEL with silt and sand, very dense, gray, moist. Recovered 0.2 ft. | | |
| 40 | 12 13 | | | 26 50/5 (50/5") | | D-9 | | Well graded SAND with silt and gravel, very dense, gray, moist. Recovered 0.9 ft. | | |
| 45 | | | | | | | | | | |

# LOG OF TEST BORING

Washington State Department of Transportation

HOLE No. H-38-95

Sheet 3 of 3

PROJECT Pierce County Line to Tukwila Stage 3

Job No. OL-1863

| Depth (ft) | Meters (m) | Profile | Standard Penetration Blows/ft (10 20 30 40) | SPT Blows/6" (N) | Sample Type | Sample No. (Tube No.) | Lab Tests | Description of Material | Groundwater | Instrument |
|---|---|---|---|---|---|---|---|---|---|---|
| | 14 | | | 36 50/5 (50/5") | | D-10 | | Well graded SAND with silt and gravel, very dense, gray, moist. Recovered 0.4 ft. | | |
| | 15 | | | | | | | | | |
| 50 | | | | 50/5" (50/5") | | D-11 | | Well graded SAND with silt and gravel, very dense, gray, moist. Recovered 0.4 ft. | | |
| | 16 | | | | | | | | | |
| 55 | | | | 50/4" (50/4") | | D-12 | | Well graded SAND with silt and gravel, very dense, gray, moist. Recovered 0.4 ft. | | |
| | 17 | | | | | | | | | |
| | 18 | | | | | | | | | |
| 60 | | | | 50/3" (50/3") | | D-13 | | Well graded SAND with silt and gravel, very dense, gray, moist. Recovered 0.3 ft. | | |
| | 19 | | | | | | | End of test hole boring at 60.3 ft. below ground elevation. Water table elevation not determined. This is a summary Log of Test Boring. Soil/Rock descriptions are derived from visual field identifications. | | |
| 65 | 20 | | | | | | | | | |
| | 21 | | | | | | | | | |
| 70 | | | | | | | | | | |

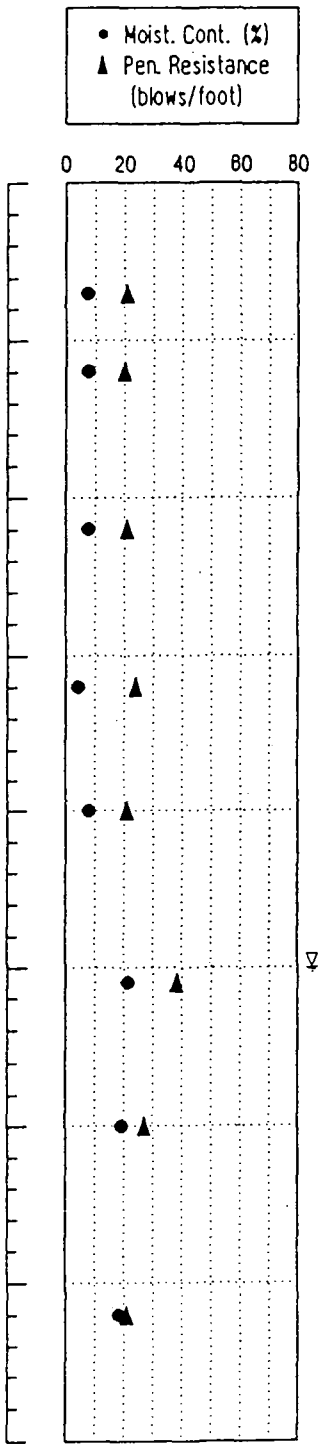
# HONG WEST & ASSOCIATES, INC.

# BORING LOG

DRILLING COMPANY: Gregory Drilling
DRILLING METHOD: Hollow Stem
SAMPLING METHOD: SPT

TOTAL DEPTH: 41.5 Feet
SURFACE ELEVATION: **131.0 m**
MEASURING POINT EL.: Feet

| DEPTH (feet) | SAMPLES | PEN. RESISTANCE (blows/6 inches) | N-VALUE (blows/ft) | MOIST. CONT. (%) | SYMBOL | SOIL CLASS. (USCS) | DESCRIPTION |
|---|---|---|---|---|---|---|---|
| 0 | | | | | | | |
| | X | 14/10/11 | 21 | 7.2 | | | |
| 5 | X | 7/10/10 | 20 | 7.6 | | | Medium dense, olive brown, silty, fine SAND with some gravel. Moist. (Fill) |
| 10 | X | 14/14/7 | 21 | 7.5 | | | |
| | | | | | | | Very stiff, greyish brown, sandy SILT. Some gravel to gravelly. Damp to moist. |
| 15 | X | 7/8/16 | 24 | 4.2 | | | |
| 20 | X | 7/11/10 | 21 | 7.9 | | | (Fill) |
| 25 | X | 8/10/28 | 38 | 21.3 | | SM | Medium dense to dense, grey and brown, silty, fine to medium SAND. Trace to some gravel. Wet. |
| 30 | X | 9/13/14 | 27 | 19.2 | | | (Recessional Outwash) |
| 35 | X | 6/9/12 | 21 | 18.1 | | | |
| 40 | | | | | | | Very dense, greyish brown, silty SAND. Some gravel. Wet. (Recessional Outwash) |




NOTE: This log of subsurface conditions applies only at the specified location and on the date indicated.

PROJECT: I-5 HOV Lane Widening

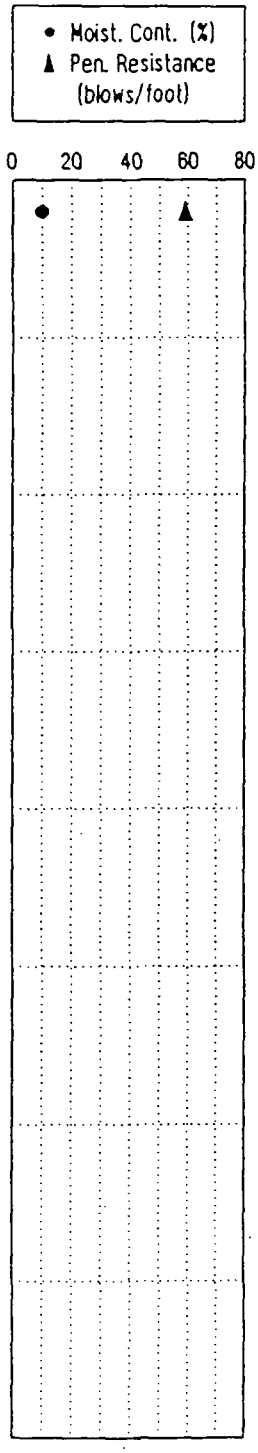
BORING: BH-5-92

LOCATION: **LE 232+981 13.1 m Lt.**
DATE COMPLETED: 5/14/92
LOGGED BY: CB

PROJECT NUMBER: 91024-3

# HONG WEST & ASSOCIATES, INC.
## BORING LOG

| DEPTH (feet) | SAMPLES | PEN. RESISTANCE (blows/6 inches) | N-VALUE (blows/ft) | MOIST. CONT. (%) | SYMBOL | SOIL CLASS. (USCS) | DESCRIPTION |
|---|---|---|---|---|---|---|---|
| 40 | X | 24/33/28 | 59 | 9.9 | | | Very dense, greyish-brown, silty SAND. Some gravel to gravelly. |
| | | | | | | | End of boring at 41.5 feet. |
| 45 | | | | | | | |
| 50 | | | | | | | |
| 55 | | | | | | | |
| 60 | | | | | | | |
| 65 | | | | | | | |
| 70 | | | | | | | |
| 75 | | | | | | | |
| 80 | | | | | | | |




PROJECT: I-5 HOV Lane Widening

BORING: BH-5-92

LOCATION: **LE 232+981 13.1 m Lt.**

DATE COMPLETED: 5/14/92

LOGGED BY: CB

PROJECT NUMBER: 91024-3

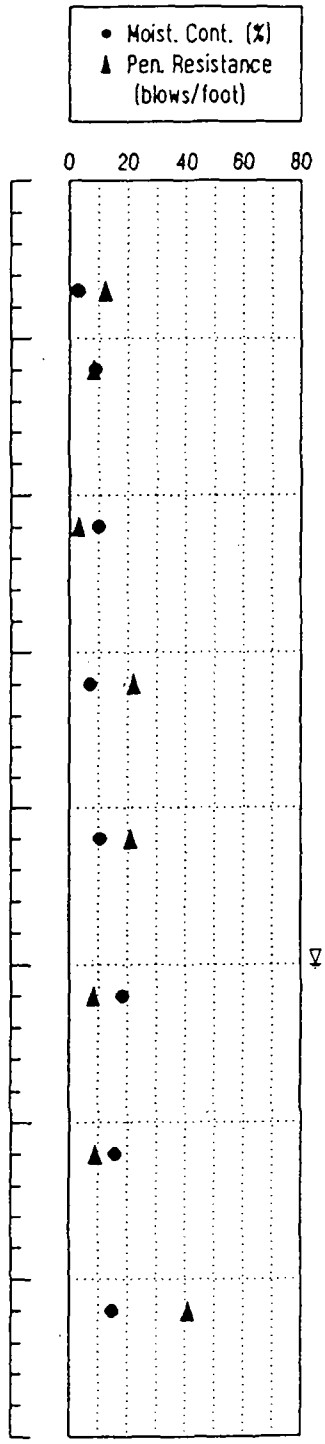
# HONG WEST & ASSOCIATES, INC.

# BORING LOG

DRILLING COMPANY: Gregory Drilling
DRILLING METHOD: Hollow Stem
SAMPLING METHOD: SPT

TOTAL DEPTH: 46.5 Feet
SURFACE ELEVATION: **133.0 m**
MEASURING POINT EL.: Feet

| DEPTH (feet) | SAMPLES | PEN. RESISTANCE (blows/6 inches) | N-VALUE (blows/ft) | MOIST. CONT. (%) | SYMBOL | SOIL CLASS. (USCS) | DESCRIPTION |
|---|---|---|---|---|---|---|---|
| 0 | | | | | | | Loose, brown SILT. Some gravel. Dry to damp. (Fill) |
| | X | 13/3/9 | 12 | 2.9 | | | |
| 5 | X | 6/5/3 | 8 | 8.8 | | | Loose, brown, sandy SILT with some gravel. Moist to wet. (Fill) Probably perched water at 8 feet. |
| 10 | X | 1/1/2 | 3 | 9.7 | | | |
| 15 | X | 5/10/12 | 22 | 7.1 | | | |
| 20 | X | 9/11/10 | 21 | 10.3 | | | Medium dense, dark brown, sandy SILT with some gravel. Trace wood debris. (Fill) |
| 25 | X | 3/3/5 | 8 | 18.2 | | | Medium stiff, grey SILT. Some fine gravel. Trace coarse sand. Low plasticity. |
| 30 | X | 3/4/5 | 9 | 15.7 | | | (Colluvium ?) |
| 35 | X | 9/14/27 | 41 | 14.8 | | SM | Dense to very dense, brownish grey, gravelly/silty SAND. Wet. (Recessional Outwash) |
| 40 | | | | | | | |




NOTE: This log of subsurface conditions applies only at the specified location and on the date indicated.

PROJECT: I-5 HOV Lane Widening

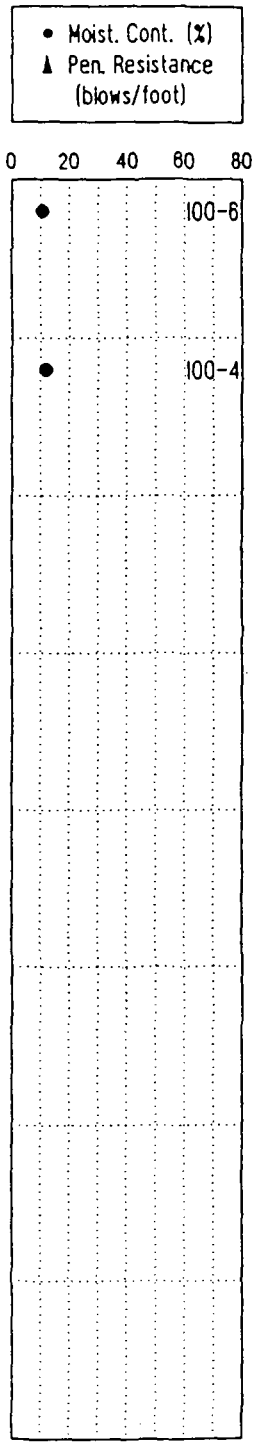
BORING: BH-8-92

LOCATION: **LE 232+916 11.6 m Lt.**
DATE COMPLETED: 5/15/92
LOGGED BY: CB

PROJECT NUMBER: 91024-3

# HONG WEST & ASSOCIATES, INC.
## BORING LOG

| DEPTH (feet) | SAMPLES | PEN. RESISTANCE (blows/6 inches) | N-VALUE (blows/ft) | MOIST. CONT. (%) | SYMBOL | SOIL CLASS. (USCS) | DESCRIPTION |
|---|---|---|---|---|---|---|---|
| 40 | X | 51/100 | 100-6"10.6 | | | SM | Dense to very dense, brownish grey, gravelly/silty SAND. Wet. (Recessional Outwash) |
| 45 | X | 31/93/100-4" | 100-4"11.7 | | | | Very dense, grey, clayey/silty GRAVEL. Damp to moist. |
| | | | | | | | End of boring at 46.5 feet |
| 50 | | | | | | | |
| 55 | | | | | | | |
| 60 | | | | | | | |
| 65 | | | | | | | |
| 70 | | | | | | | |
| 75 | | | | | | | |
| 80 | | | | | | | |




PROJECT: I-5 HOV Lane Widening

BORING: BH-8-92

LOCATION: **LE 232+916 11.6 m Lt.**

DATE COMPLETED: 5/15/92

LOGGED BY: CB

PROJECT NUMBER: 91024-3

Job No. OL-1863 Date January 26, 1995

Hole No. H-21-95 Sheet 1 of 2

Project Pierce Co. Line to Tukwila Stage 3

# Laboratory Summary

Washington State Department of Transportation

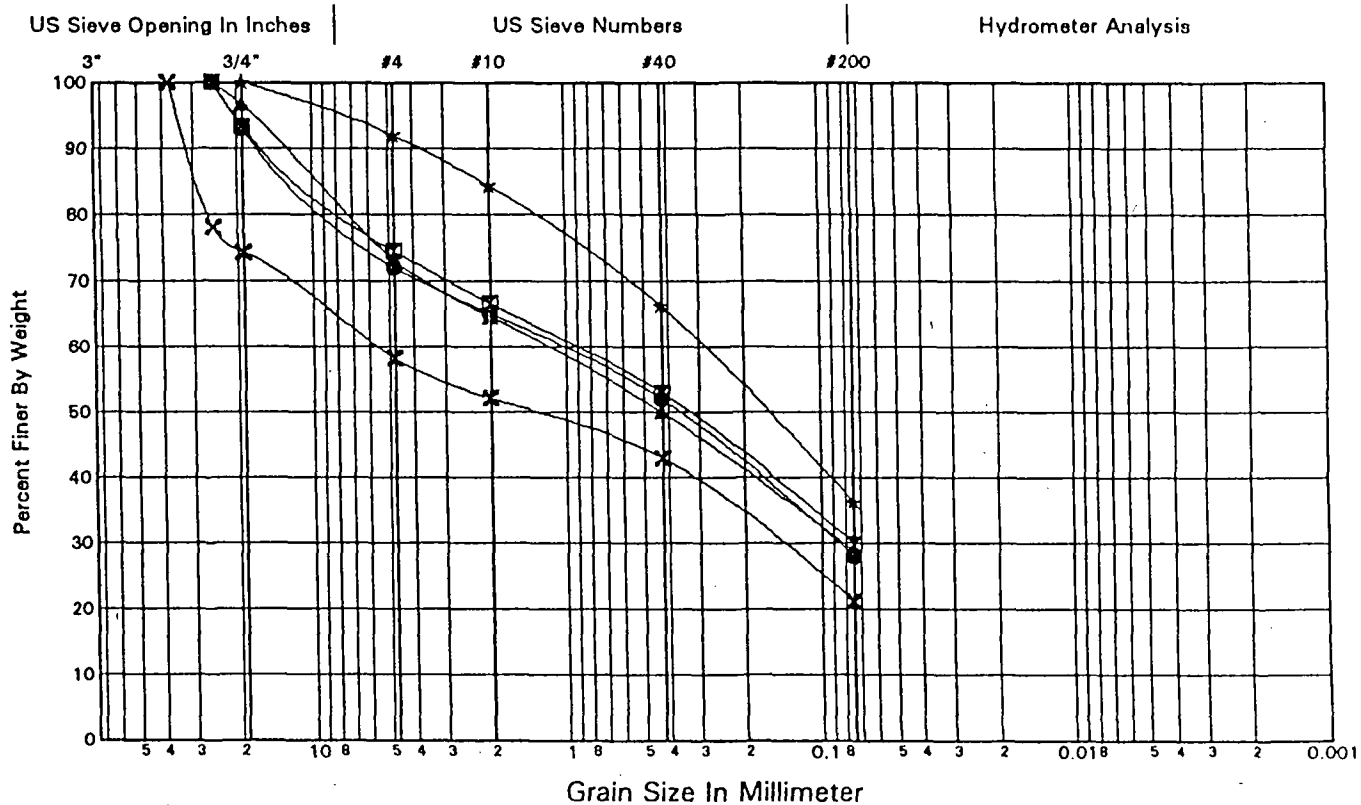
| | Depth (ft) | Depth (m) | Sample No. | USCS | Color | Description | MC% | LL | PL | PI |
|---|---|---|---|---|---|---|---|---|---|---|
| ● | 4.0 | 1.22 | D-1 | SM | OLIVE GRAY | SILTY SAND with GRAVEL | 8 | NP | NP | NP |
| ⊠ | 10.5 | 3.20 | D-3 | SM | GRAY | SILTY SAND with GRAVEL | 11 | NP | NP | NP |
| ▲ | 14.0 | 4.27 | D-4 | SM | OLIVE GRAY | SILTY SAND with GRAVEL | 9 | NP | NP | NP |
| ★ | 20.5 | 6.25 | D-6 | SM | GRAY | SILTY SAND | 15 | NP | NP | NP |
| ✕ | 24.0 | 7.32 | D-7 | GM | LIGHT GRAY | SILTY GRAVEL with SAND | 12 | NP | NP | NP |

## GRADATION FRACTIONS

| | %Gravel | %Sand | %Fines | Cu | Cc |
|---|---|---|---|---|---|
| ● | 28.0 | 44.0 | 28.0 | | |
| ⊠ | 25.5 | 44.6 | 29.9 | | |
| ▲ | 26.8 | 44.4 | 28.8 | | |
| ★ | 8.0 | 55.7 | 36.3 | | |
| ✕ | 41.8 | 37.0 | 21.2 | | |

## GRADATION VALUES

| | D60 | D50 | D30 | D20 | D10 |
|---|---|---|---|---|---|
| ● | 1.09 | 0.37 | 0.09 | | |
| ⊠ | 0.94 | 0.34 | 0.08 | | |
| ▲ | 1.23 | 0.43 | 0.08 | | |
| ★ | 0.30 | 0.17 | | | |
| ✕ | 5.55 | 1.40 | 0.15 | | |



Job No. OL-1863 Date January 26, 1995

Hole No. H-21-95 Sheet 2 of 2

Project Pierce Co. Line to Tukwila Stage 3

# Laboratory Summary

Washington State Department of Transportation

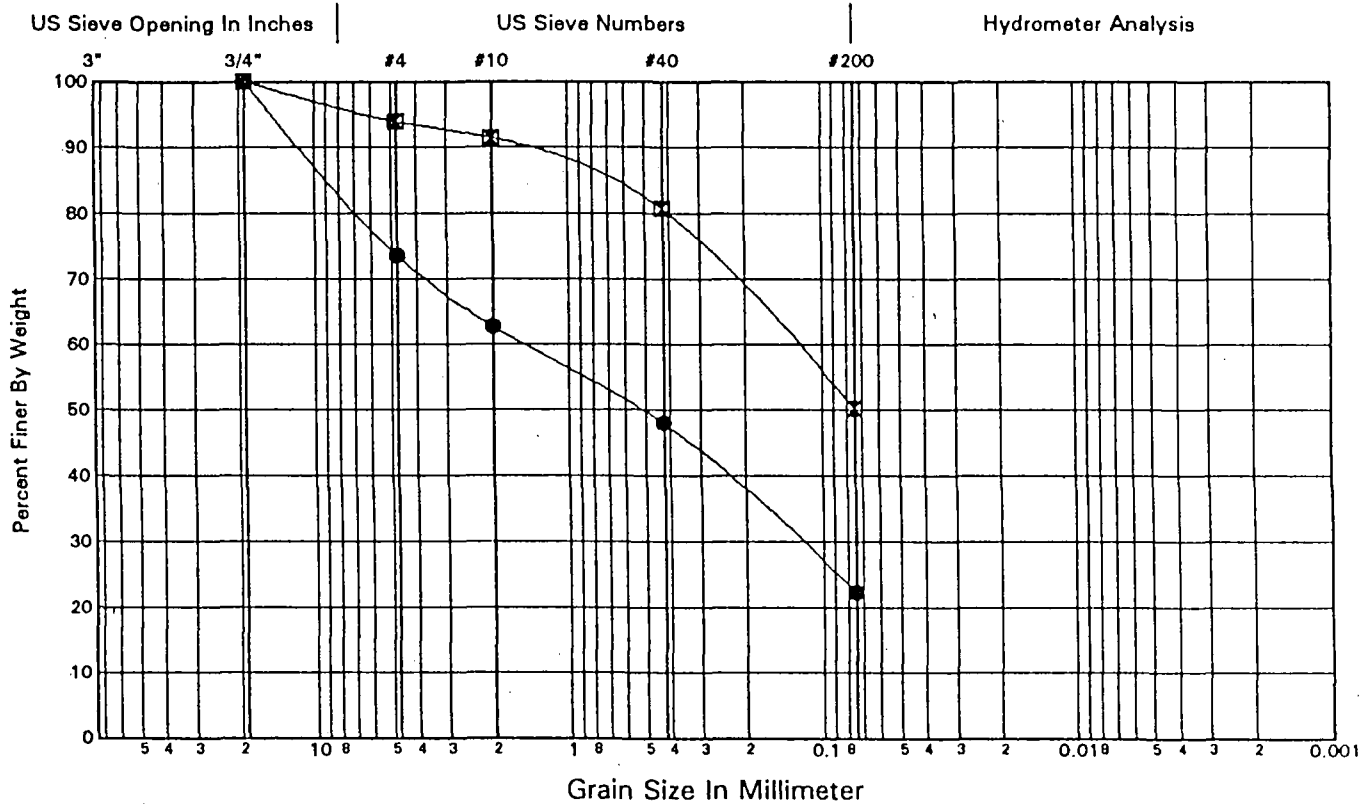
| | Depth (ft) | Depth (m) | Sample No. | USCS | Color | Description | MC% | LL | PL | PI |
|---|---|---|---|---|---|---|---|---|---|---|
| ● | 29.0 | 8.84 | D-8 | SM | OLIVE GRAY | SILTY SAND with GRAVEL | 13 | NP | NP | NP |
| ⊠ | 39.0 | 11.89 | D-10 | ML | DARK GREENISH GRAY | SANDY SILT | 13 | NP | NP | NP |

## GRADATION FRACTIONS

| | %Gravel | %Sand | %Fines | Cu | Cc |
|---|---|---|---|---|---|
| ● | 26.4 | 51.2 | 22.4 | | |
| ⊠ | 6.1 | 43.7 | 50.2 | | |

## GRADATION VALUES

| | D60 | D50 | D30 | D20 | D10 |
|---|---|---|---|---|---|
| ● | 1.51 | 0.52 | 0.13 | | |
| ⊠ | 0.13 | | | | |



Job No. OL-1863 | Date December 23, 1994

Hole No. H-22-94 | Sheet 1 of 2

Project Pierce Co. Line to Tukwila Stage 3

# Laboratory Summary

Washington State Department of Transportation

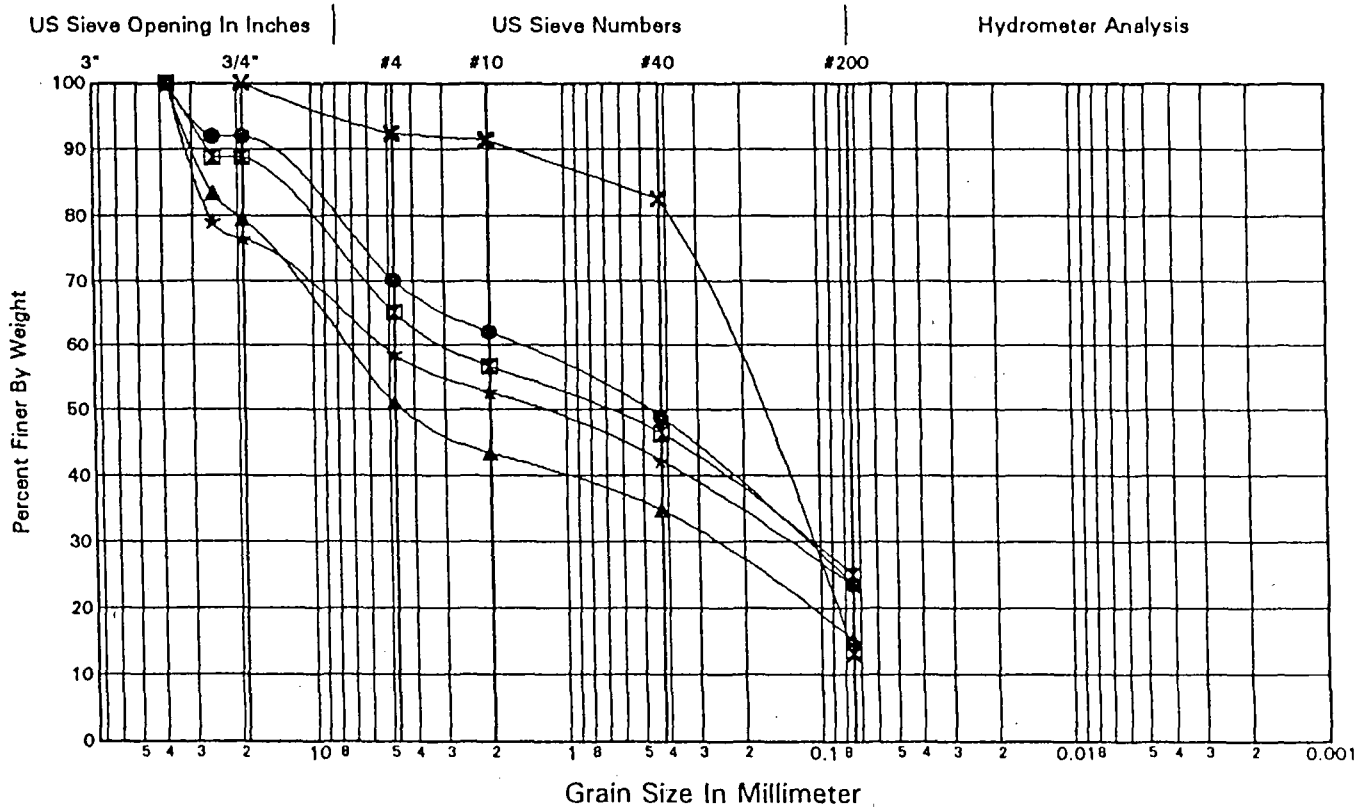
| | Depth (ft) | Depth (m) | Sample No. | USCS | Color | Description | MC% | LL | PL | PI |
|---|---|---|---|---|---|---|---|---|---|---|
| ● | 11.0 | 3.35 | D-2 | SM | OLIVE GRAY | SILTY SAND with GRAVEL | 15 | NP | NP | NP |
| ⊠ | 16.0 | 4.88 | D-3 | SM | OLIVE GRAY | SILTY SAND with GRAVEL | 11 | NP | NP | NP |
| ▲ | 22.0 | 6.71 | D-4 | GM | LIGHT OLIVE GRAY | SILTY GRAVEL with SAND | 11 | NP | NP | NP |
| ★ | 32.0 | 9.75 | D-6 | GM | GRAY | SILTY GRAVEL with SAND | 12 | NP | NP | NP |
| ✕ | 42.0 | 12.80 | D-8 | SM | GRAY | SILTY SAND w/one large gravel piece 49.5 g. | 17 | NP | NP | NP |

## GRADATION FRACTIONS

| | %Gravel | %Sand | %Fines | Cu | Cc |
|---|---|---|---|---|---|
| ● | 29.9 | 46.5 | 23.6 | | |
| ⊠ | 34.9 | 40.1 | 25.0 | | |
| ▲ | 49.0 | 35.7 | 15.3 | | |
| ★ | 41.5 | 35.1 | 23.4 | | |
| ✕ | 7.6 | 79.2 | 13.2 | | |

## GRADATION VALUES

| | D60 | D50 | D30 | D20 | D10 |
|---|---|---|---|---|---|
| ● | 1.58 | 0.48 | 0.12 | | |
| ⊠ | 2.81 | 0.73 | 0.11 | | |
| ▲ | 7.36 | 4.24 | 0.28 | 0.11 | |
| ★ | 5.33 | 1.34 | 0.14 | | |
| ✕ | 0.24 | 0.19 | 0.11 | 0.09 | |



Job No. OL-1863 Date December 23, 1994

Hole No. H-22-94 Sheet 2 of 2

Project Pierce Co. Line to Tukwila Stage 3

# Laboratory Summary

Washington State Department of Transportation

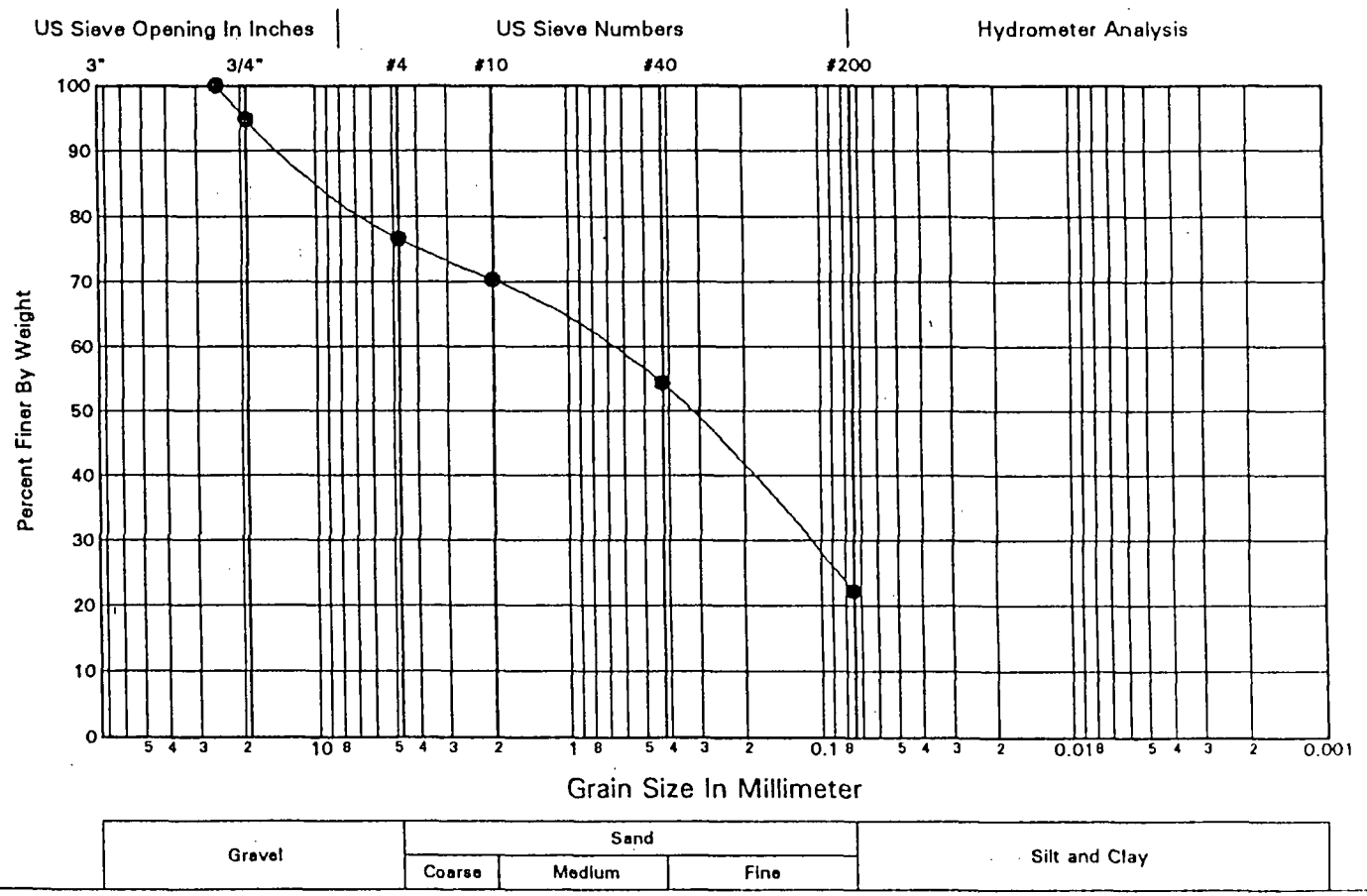
| | Depth (ft) | Depth (m) | Sample No. | USCS | Color | Description | MC% | LL | PL | PI |
|---|---|---|---|---|---|---|---|---|---|---|
| ● | 47.0 | 14.33 | D-9 | SM | DARK GRAY | SILTY SAND with GRAVEL | 13 | NP | NP | NP |

## GRADATION FRACTIONS

| | %Gravel | %Sand | %Fines | Cu | Cc |
|---|---|---|---|---|---|
| ● | 23.4 | 54.3 | 22.3 | | |

## GRADATION VALUES

| | D60 | D50 | D30 | D20 | D10 |
|---|---|---|---|---|---|
| ● | 0.73 | 0.34 | 0.11 | | |



Job No. OL-1863 Date February 3, 1995

Hole No. H-23-95 Sheet 1 of 2

Project Pierce Co. Line to Tukwila Stage 3

# Laboratory Summary

Washington State Department of Transportation

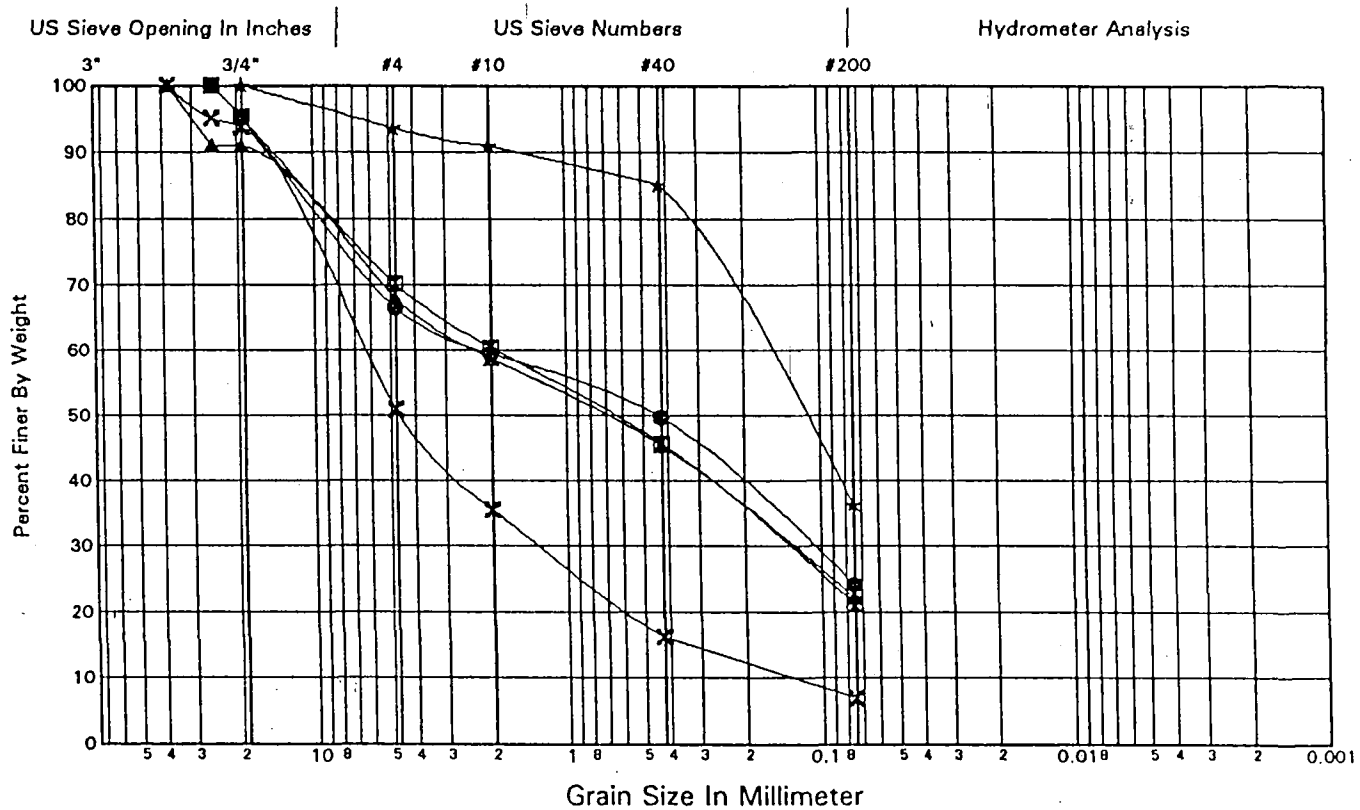
| | Depth (ft) | Depth (m) | Sample No. | USCS | Color | Description | MC% | LL | PL | PI |
|---|---|---|---|---|---|---|---|---|---|---|
| ● | 5.0 | 1.52 | D-1 | SM | OLIVE GRAY | SILTY SAND with GRAVEL | 11 | NP | NP | NP |
| ⊠ | 15.0 | 4.57 | D-3 | SM | LIGHT GRAY | SILTY SAND with GRAVEL | 10 | NP | NP | NP |
| ▲ | 20.0 | 6.10 | D-4 | SM | OLIVE GRAY | SILTY SAND with GRAVEL | 17 | NP | NP | NP |
| ★ | 35.0 | 10.67 | D-7 | SM | OLIVE GRAY | SILTY SAND | 18 | NP | NP | NP |
| ✕ | 40.0 | 12.19 | D-8 | GW-GM | OLIVE | WELL GRADED GRAVEL with SILT and SAND | 11 | NP | NP | NP |

## GRADATION FRACTIONS

| | %Gravel | %Sand | %Fines | Cu | Cc |
|---|---|---|---|---|---|
| ● | 33.5 | 42.3 | 24.2 | | |
| ⊠ | 29.8 | 49.0 | 21.2 | | |
| ▲ | 31.9 | 45.9 | 22.2 | | |
| ★ | 6.4 | 57.3 | 36.3 | | |
| ✕ | 49.1 | 44.0 | 6.9 | 1.9 | 48.0 |

## GRADATION VALUES

| | D60 | D50 | D30 | D20 | D10 |
|---|---|---|---|---|---|
| ● | 2.15 | 0.45 | 0.11 | | |
| ⊠ | 1.92 | 0.66 | 0.14 | | |
| ▲ | 2.25 | 0.73 | 0.13 | | |
| ★ | 0.17 | 0.12 | | | |
| ✕ | 6.38 | 4.52 | 1.28 | 0.57 | 0.13 |



Job No.

Hole No. H-23-95

Project Pierce Co. Line to Tukwila Stage 3

Date February 3, 1995

Sheet 2 of 2

# Laboratory Summary

Washington State Department of Transportation

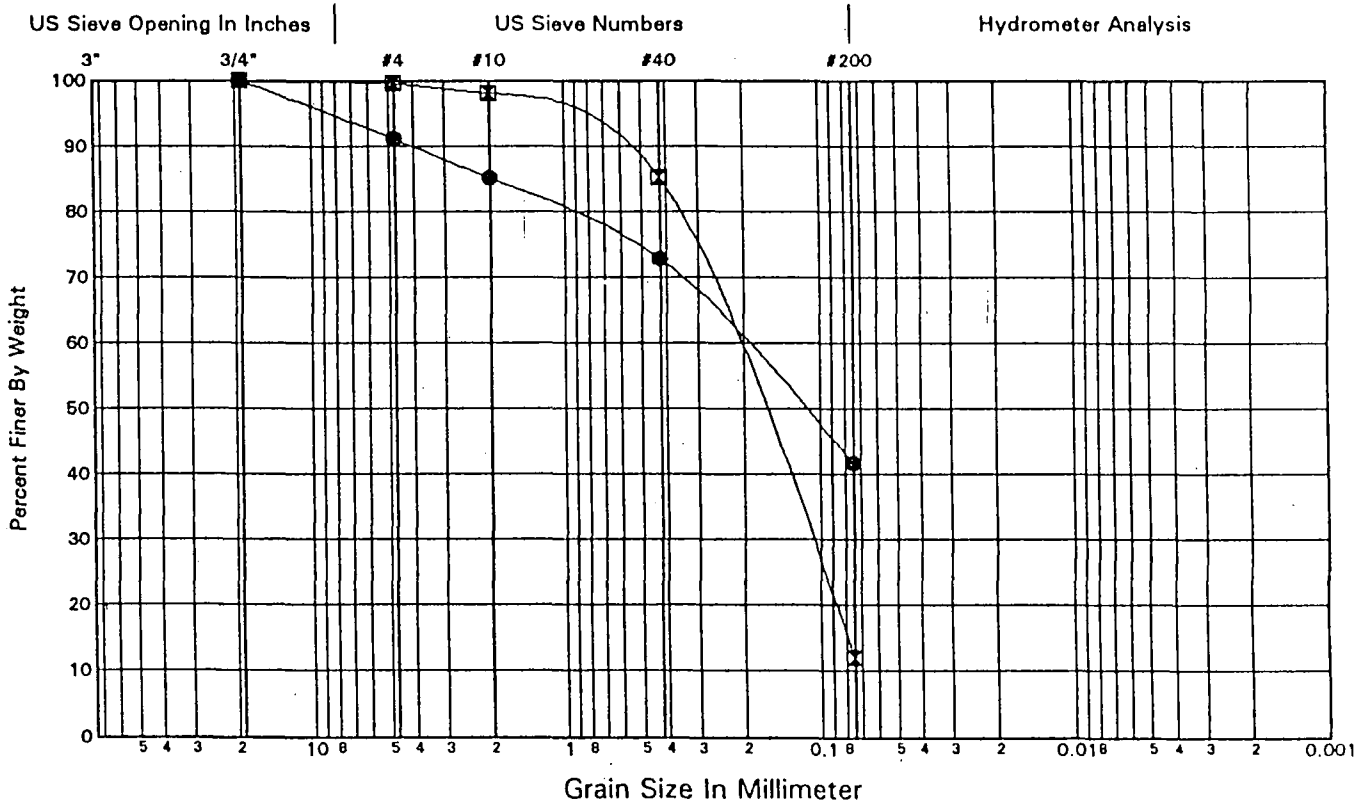
| | Depth (ft) | Depth (m) | Sample No. | USCS | Color | Description | MC% | LL | PL | PI |
|---|---|---|---|---|---|---|---|---|---|---|
| ● | 45.0 | 13.72 | D-9 | SM | OLIVE GRAY | SILTY SAND | 19 | NP | NP | NP |
| ⊠ | 50.0 | 15.24 | D-10A | SP-SM | OLIVE GRAY | POORLY GRADED SAND with SILT | 26 | NP | NP | NP |

## GRADATION FRACTIONS

| | %Gravel | %Sand | %Fines | Cu | Cc |
|---|---|---|---|---|---|
| ● | 8.9 | 49.4 | 41.7 | | |
| ⊠ | 0.5 | 87.5 | 12.0 | 0.8 | 3.3 |

## GRADATION VALUES

| | D60 | D50 | D30 | D20 | D10 |
|---|---|---|---|---|---|
| ● | 0.21 | 0.12 | | | |
| ⊠ | 0.23 | 0.18 | 0.11 | 0.09 | |




| Gravel | Sand | | | Silt and Clay |
|---|---|---|---|---|
| | Coarse | Medium | Fine | |

# HONG WEST & ASSOCIATES
# GRAIN SIZE DISTRIBUTION

Project: I-5 Bridge Improvement

Client: Alpha Engineering

Project Number: 91024-2

Date Tested: 5/28/92

Remarks: Olive brown, silty SAND with some gravel (SM)

Test Hole Number: BH-5-92

Sample Number: 8

Depth: 35-36.5 feet

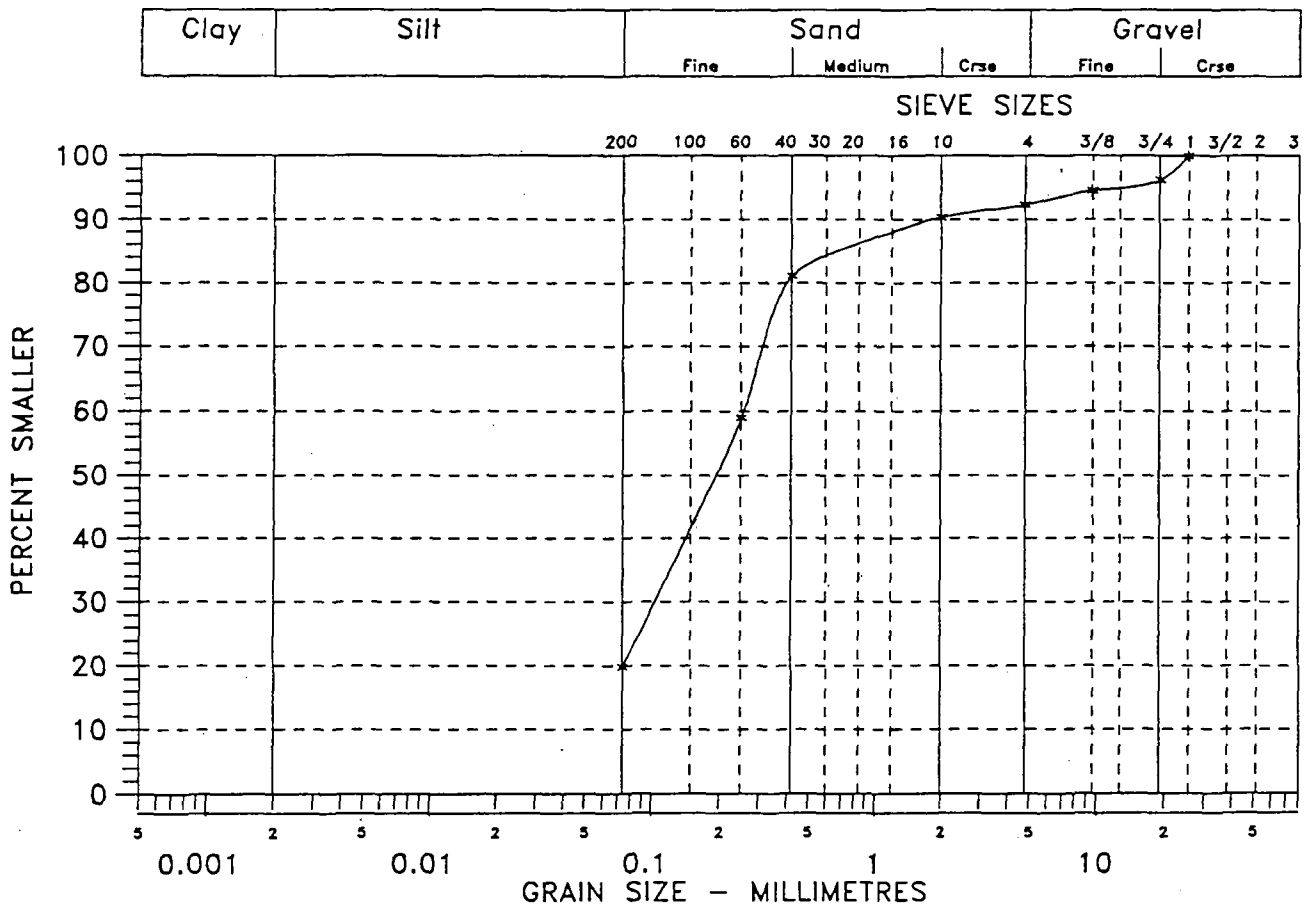
Sample Description:

Gravel: 7.6

Sand: 72.5

Silt: 19.9

Clay:



# HONG WEST & ASSOCIATES
# GRAIN SIZE DISTRIBUTION

Project: I-5 Bridges

Client: Alpha Engineering Group

Project Number: 91024-3

Date Tested: 5/28/92

Remarks: Dark gray, gravelly, silty SAND (SM)

Test Hole Number: BH-8-92

Sample Number: 9

Depth: 40-41.5 feet

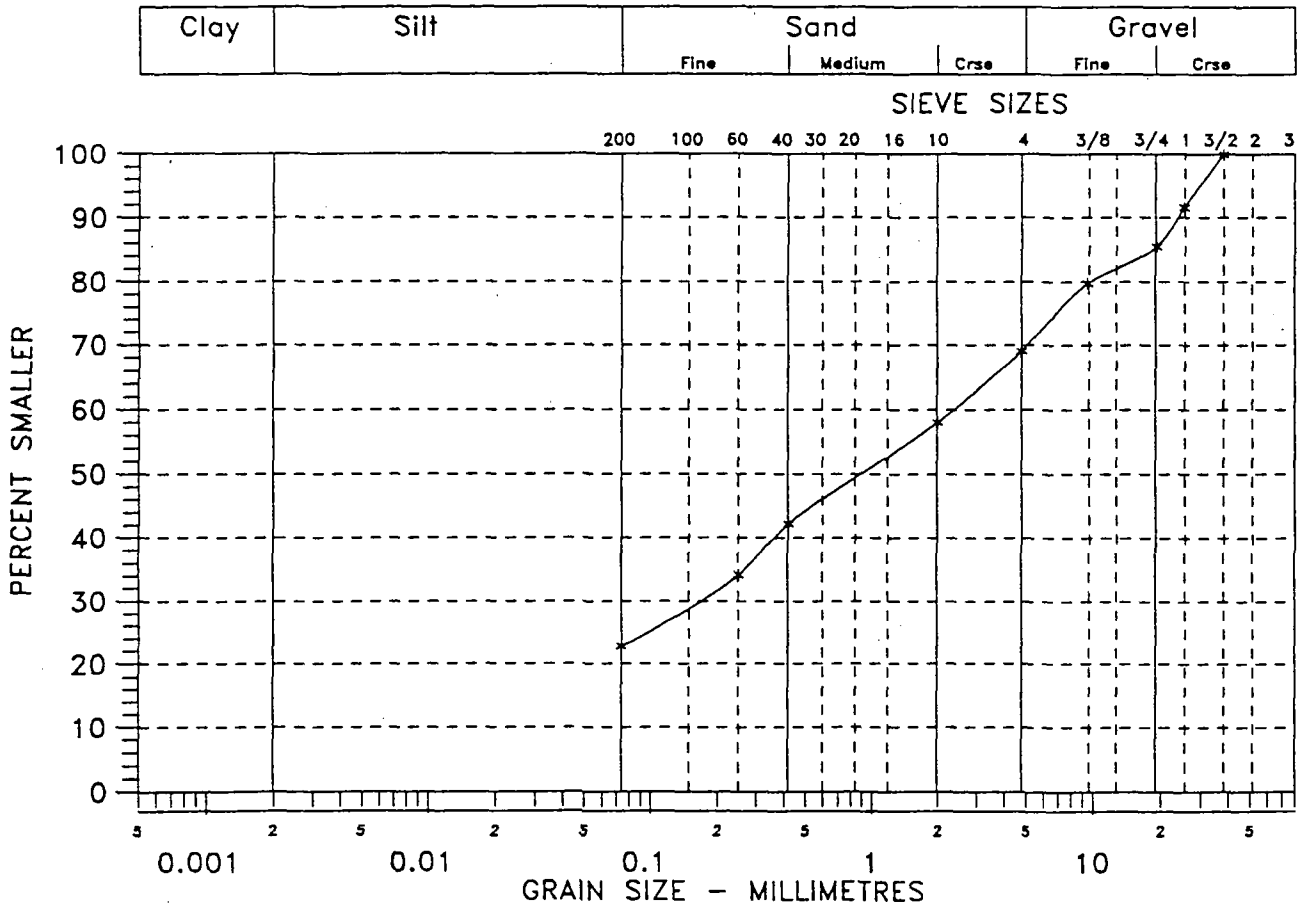
Sample Description:

Gravel: 30.9

Sand: 46.3

Silt: 22.8

Clay:



CHAPTER 7

# Site-Specific Information - Wall 2

## Proposed Improvements

Figure 1-1 shows the location of Wall 2 relative to other structures in the project. Figure 7-1 (the site plan) shows the proposed location of Wall 2 at the SR 18 and SR 5 interchange. The proposed location is on the left side of the existing SR 5 south on-ramp from the westbound lane of SR 18 (E-S Line). Wall cross-sections provided by the WSDOT, indicate that the wall will be located in an existing drainage depression between the E-S Line and the SR 5 south to westbound SR 18 off-ramp (NDL2 Line). The ground surface in front of the wall is relatively level and drains toward the wall. The wall will retain existing road fill as well as new fill required for the roadway widening. Project wall information obtained from the cross-sections is summarized in Table 7-1.

**Table 7-1**
**Wall 2 Information**

| | |
|---|---|
| Wall Stationing | E-S 0+140 to E-S 0+290 |
| Wall Height | 1.9 to 2.8 m (6 to 9 ft.) |
| Bottom of Wall Footing Elevation | 100 to 106 m (328 to 348 ft.) or about 1 m (3.3 ft.) below existing grade |

## Site Description

The proposed wall location is currently used to collect surface water runoff between the SR 5 off-ramp to westbound SR 18 and the SR 5 south on-ramp from westbound SR 18. The slope between the two ramps at the proposed wall location is vegetated with grass. A guard rail and chain link fence also separate the ramps.

## Subsurface Information

Subsurface explorations, designated H-6-95 and H-7-95, were drilled and logged by WSDOT at the locations indicated on Figure 7-1, Site Plan. Detailed logs are provided at the end of this chapter. The borings were drilled from 6 to 9 m (20 to 30 ft.) below existing grade. Grain-size distributions and moisture contents were determined on selected soil samples. Laboratory test results are also attached at the end of this chapter.

Subsurface conditions generally consist of a medium-dense to dense, silty sand with gravel and silty gravel with sand. The groundwater table was not determined.

# Recommendations

Conditions are suitable for the use of either standard concrete cantilever or preapproved proprietary mechanically stabilized earth (MSE) walls. A standard concrete cantilever wall is recommended because of the relatively short wall height, variation of height with length, and limited space for temporarily stockpiling excavated material. The existing slope geometry will require less excavation and backfill for a standard concrete cantilever wall than for an MSE wall.

The following soil parameters are recommended for use in design of the proposed retaining wall.

## Foundation Soils

| | |
|---|---|
| Angle of internal friction: | 36° |
| Unit weight: | 19.6 KN/m$^3$ [125 pounds per cubic foot (pcf)] |
| Allowable soil bearing pressure: | 190 kPa [4000 pounds per square foot (psf)] |
| Estimated elastic settlement: | less than 10 to 15 mm (1/2 inch) |
| Ultimate lateral passive earth pressure: | h*47 KN/m$^3$ (300 pcf), where h = distance from backfill. A factor of safety of 1.5 has been applied to the passive earth pressure to limit wall movement. |

## Retained Fill (Existing Road Fill)

| | |
|---|---|
| Angle of internal friction: | 36° |
| Unit weight: | 19.6 KN/m$^3$ (125 pcf) |

## Proposed Compacted Fill Behind Wall

| | |
|---|---|
| Angle of internal friction: | 36° |
| Unit weight: | 19.6 KN/m$^3$ (125 pcf) |
| Lateral active earth pressure: | h*5.5 KN/m$^3$ (35 pcf) for unrestrained wall plus 7.5 kPa (65 psf) surcharge for traffic loading |

# Other Design Considerations

New surface water runoff control will be required because the existing drainage ditch will be filled in with the construction of the proposed wall. This may require a drainage trench and pipe to discharge to the depression located to the west of Station 0 + 125.

New road fill should be notched into the existing embankment slope. Where the wall height is less than 1 m (3 ft.), the notching can be limited to scarification of the existing fill surface.

Wall fill should consist of a free-draining, compacted structural fill that conforms to the requirements for gravel borrow in Section 9-03.14 of the Standard Specifications. Backfill should be compacted to 95 percent of the maximum density as specified under Compacting

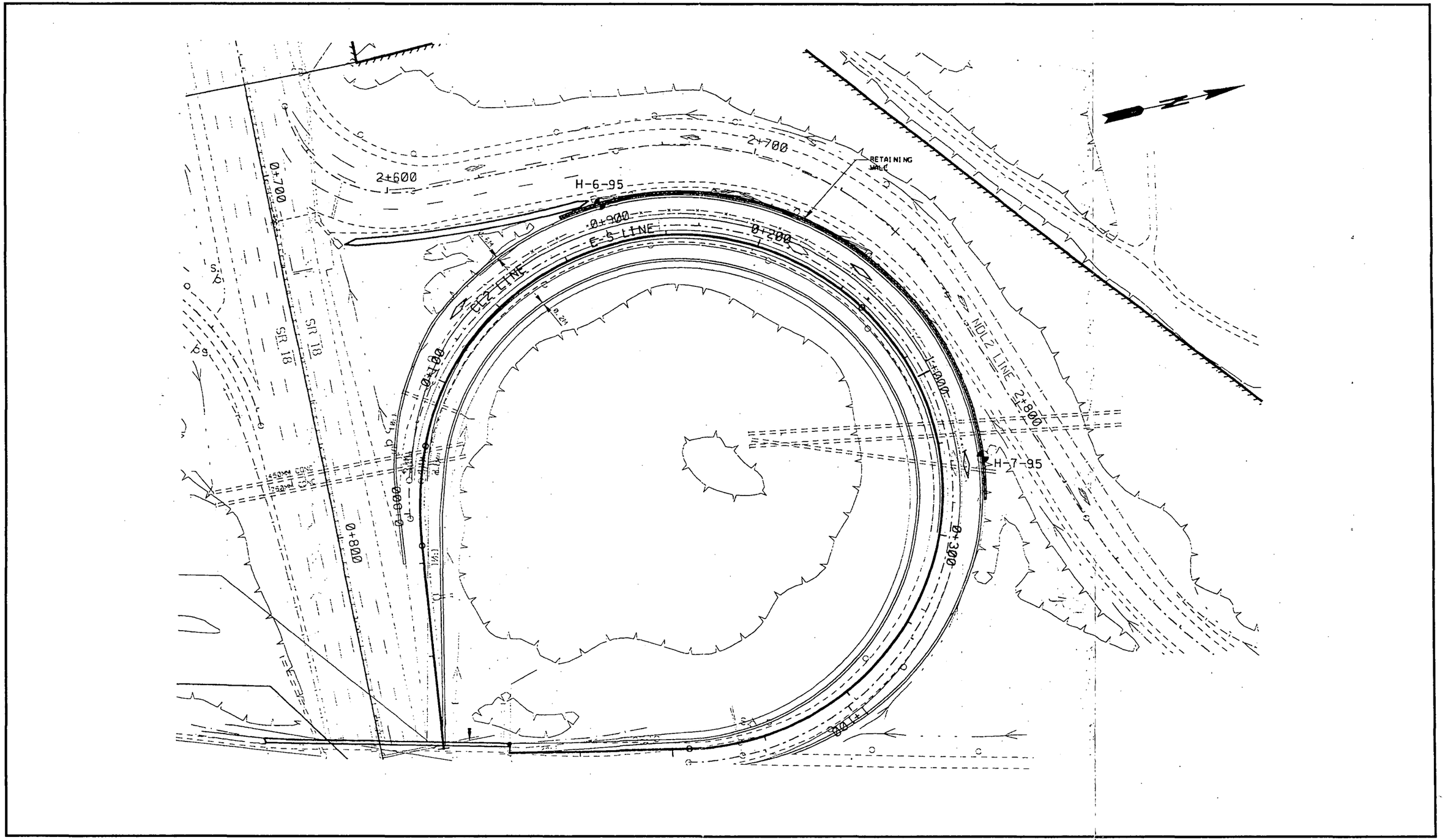
Earth Embankments, Method C, in Section 2-03.3(14)C. The moisture content of the backfill shall meet the requirements of Section 2-03.3(14)C, Method C.

## Construction Considerations

Surface water runoff control may be needed during construction. Open-cut excavations are anticipated for the wall construction. Temporary cuts of 1H:1V are not anticipated to encroach upon the existing traffic lane.

The wall foundation excavations are expected to be in glacial till and fill derived from glacial till. As such, cobbles and boulders may be encountered given the geologic origin of the soil deposit. Any foundation soils found to be unsuitable shall be removed and replaced as directed by the geotechnical engineer.

116184.G3.02 • Wall 2 • 4-4-96 • LW




Approximate
Scale in Meters

0 25 m

FIGURE 7-1
**Wall No. 2 Layout and Boring Locations**
Geotechnical Report SR-5
Port of Tacoma I/C to 272nd
Stage 4 HOV

# Chapter7
# Appendix

# LOG OF TEST BORING

Washington State Department of Transportation

HOLE No. H-6-95

PROJECT Fife to Tukwila HOV Lanes — Job No. OL-1922

Stage 4 Walls — S.R. 5

Station 0+160 — Offset 10.9 m Lt. — C.S. 1727

Equipment — Casing 4" OD X 20' Hollow Core — Ground El 334.3 (101.89 m)

Method of Boring Dry Rotary

Start Date August 28, 1995 — Completion Date August 28, 1995 — Sheet 1 of 2

| Depth (ft) | Meters (m) | Profile | Standard Penetration Blows/ft (10 20 30 40) | SPT Blows/6" (N) | Sample Type | Sample No. (Tube No.) | Lab Tests | Description of Material | Groundwater | Instrument |
|---|---|---|---|---|---|---|---|---|---|---|
| | | | | 3<br>3<br>3<br>(6) | | D-1 | GS<br>MC | 1 ft. = 0.3048 m.<br>Note: Test hole drilled in ditch line.<br>GM, M.C. = 14%<br>Silty GRAVEL with sand, rounded, 1.25 to 0.25 inch, loose brown, moist.<br>Retained 0.9 ft. | | |
| 5 | 1<br>2 | | | 5<br>30<br>57<br>(87) | | D-2 | GS<br>MC | GM, M.C. = 5%<br>Silty GRAVEL with sand, angular to subrounded, 1.25 to 0.25 inch, loose, brown, moist. Outwash.<br>Retained 1.0 ft. | | |
| 10 | 3 | | | 23<br>55/6<br>(55/6") | | D-3 | | Silty GRAVEL with sand, angular to subrounded, very dense, brown, moist. Outwash.<br>Retained 0.5 ft. | | |
| 15 | 4<br>5 | | | 65/6"<br>(65/6") | | D-4 | GS<br>MC | GP-GM, M.C. = 2%<br>Poorly graded GRAVEL with silt and sand, rounded to subrounded, 0.50 to 0.25 inch, very dense, brown, moist. Outwash.<br>Retained 0.3 ft. | | |
| 20 | 6 | | | 36<br>76 | | D-5 | GS<br>MC | GM, M.C. = 5%<br>Silty GRAVEL with sand, rounded to subrounded, | | |

# LOG OF TEST BORING

Washington State Department of Transportation

HOLE No. H-6-95

Sheet 2 of 2

PROJECT Fife to Tukwila HOV Lanes

Job No. OL-1922

| Depth (ft) | Meters (m) | Profile | Standard Penetration Blows/ft (10 20 30 40) | SPT Blows/6" (N) | Sample Type | Sample No. (Tube No.) | Lab Tests | Description of Material | Groundwater | Instrument |
|---|---|---|---|---|---|---|---|---|---|---|
| | | | | 36 (112) | | | | very dense, brown, moist. Retained 1.4 ft. | | |
| | | | | | | | | End of test hole boring at 20.5 ft. below ground elevation. | | |
| | | | | | | | | Water table elevation not determined. | | |
| | 7 | | | | | | | This is a summary Log of Test Boring. Soil/Rock descriptions are derived from visual field identifications and laboratory test data. | | |
| 25 | | | | | | | | | | |
| | 8 | | | | | | | | | |
| | 9 | | | | | | | | | |
| 30 | | | | | | | | | | |
| | 10 | | | | | | | | | |
| 35 | | | | | | | | | | |
| | 11 | | | | | | | | | |
| | 12 | | | | | | | | | |
| 40 | | | | | | | | | | |
| | 13 | | | | | | | | | |
| 45 | | | | | | | | | | |

# LOG OF TEST BORING

Washington State Department of Transportation

HOLE No. H-7-95

PROJECT Fife to Tukwila HOV Lanes — Job No. OL-1922

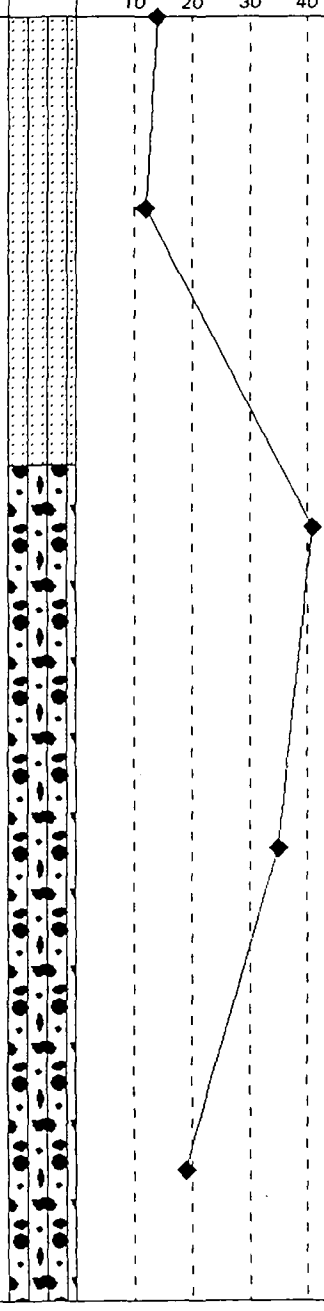
Stage 4 Walls — S.R. 5

Station 0+280 — Offset 11.2 m. Lt. — C.S. 1727

Equipment — Casing 4" OD X 30' Hollow Core — Ground El 350.8 (106.92 m)

Method of Boring Dry Rotary

Start Date August 29, 1995 — Completion Date August 29, 1995 — Sheet 1 of 2




| Depth (ft) | Meters (m) | SPT Blows/6" (N) | Sample Type | Sample No. (Tube No.) | Lab Tests | Description of Material | Groundwater | Instrument |
|---|---|---|---|---|---|---|---|---|
| | | 3<br>6<br>8<br>(14) | | D-1 | GS<br>MC | 1 ft. = 0.3048 m.<br>SM, M.C. = 7%<br>Silty SAND with gravel, rounded to subrounded, 1.50 to 0.25 inch, medium dense, brown, moist.<br>Retained 1.0 ft. | | |
| | 1 | 4<br>5<br>7<br>(12) | | D-2 | | Silty SAND with gravel, rounded to subrounded, medium dense, brown, moist.<br>Retained 0.9 ft. | | |
| 5 | 2 | | | | | | | |
| | | 19<br>19<br>22<br>(41) | | D-3 | GS<br>MC | GM, M.C. = 6%<br>Silty GRAVEL with sand, rounded to subrounded, 1.50 to 0.25 inch, dense, brown, moist.<br>Retained 0.9 ft. | | |
| 10 | 3 | | | | | | | |
| | 4 | 7<br>17<br>18<br>(35) | | D-4 | GS<br>MC | GM, M.C. = 6%<br>Silty GRAVEL with sand, rounded to subrounded, dense, brown, moist.<br>Retained 0.5 ft. | | |
| 15 | 5 | | | | | | | |
| | | 20<br>12<br>7<br>(19) | | D-5 | | Silty GRAVEL with sand and wood particles, rounded to subrounded, medium dense, brown, moist.<br>Retained 0.6 ft. | | |
| 20 | 6 | | | | | | | |

# LOG OF TEST BORING

Washington State Department of Transportation

HOLE No. H-7-95

Sheet 2 of 2

PROJECT Fife to Tukwila HOV Lanes

Job No. OL-1922

| Depth (ft) | Meters (m) | Profile | Standard Penetration Blows/ft (10 20 30 40) | SPT Blows/6" (N) | Sample Type | Sample No. (Tube No.) | Lab Tests | Description of Material | Groundwater | Instrument |
|---|---|---|---|---|---|---|---|---|---|---|
| | 7 | | | 14 8 12 (20) | | D-6 | | Silty GRAVEL with sand and wood particles, rounded to subrounded, 2.0 to 0.25 inch, medium dense, brown, moist. Retained 0.6 ft. | | |
| 25 | 8 | | | | | | | | | |
| | | | > 40 | 12 39 26 (65) | | D-7 | | Silty GRAVEL with sand, rounded to subrounded, 1.5 to 0.25 inch, very dense, brown, moist. Retained 1.0 ft. | | |
| | 9 | | | | | | | | | |
| 30 | | | | | | | | End of the Test Hole Boring at 29.5 ft. below ground elevation. | | |
| | | | | | | | | Water Table Elevation: Not determined. | | |
| | 10 | | | | | | | This is a Summary Log of the Test Hole Boring. Soil/Rock descriptions are derived from visual field identifications and laboratory test data. | | |
| 35 | | | | | | | | | | |
| | 11 | | | | | | | | | |
| | 12 | | | | | | | | | |
| 40 | | | | | | | | | | |
| | 13 | | | | | | | | | |
| 45 | | | | | | | | | | |

Job No. OL-1922 | Date November 2, 1995

Hole No. H-6-95 | Sheet 1 of 1

Project Fife to Tukwila HOV Lanes

# Laboratory Summary

Washington State Department of Transportation

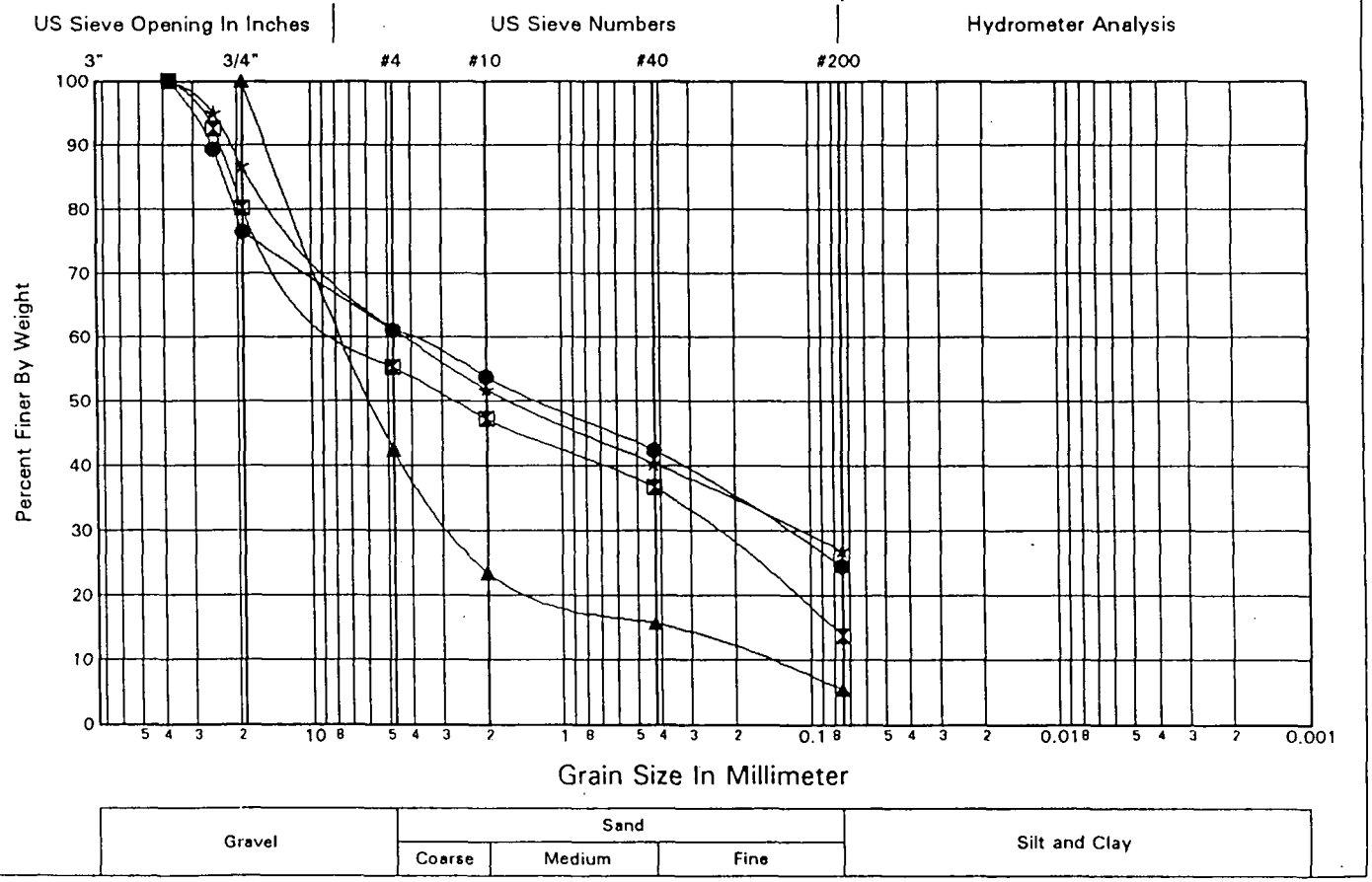
| | Depth (ft) | Depth (m) | Sample No. | USCS | Color | Description | MC% | LL | PL | PI |
|---|---|---|---|---|---|---|---|---|---|---|
| ● | 0.0 | 0.00 | D-1 | GM | Dark grayish brown | SILTY GRAVEL with SAND | 14 | NP | NP | NP |
| ⊠ | 4.0 | 1.22 | D-2 | GM | Olive gray | SILTY GRAVEL with SAND | 5 | NP | NP | NP |
| ▲ | 14.0 | 4.27 | D-4 | GP-GM | Olive gray | POORLY GRADED GRAVEL with SILT and SAND | 2 | NP | NP | NP |
| ★ | 19.0 | 5.79 | D-5 | GM | Olive gray | SILTY GRAVEL with SAND | 5 | NP | NP | NP |
| | | | | | | | | | | |

## GRADATION FRACTIONS

| | %Gravel | %Sand | %Fines | Cu | Cc |
|---|---|---|---|---|---|
| ● | 39.0 | 36.5 | 24.5 | | |
| ⊠ | 44.7 | 41.5 | 13.8 | | |
| ▲ | 57.5 | 37.2 | 5.3 | 6.1 | 44.1 |
| ★ | 38.7 | 34.3 | 27.0 | | |
| | | | | | |

## GRADATION VALUES

| | D60 | D50 | D30 | D20 | D10 |
|---|---|---|---|---|---|
| ● | 4.21 | 1.19 | 0.13 | | |
| ⊠ | 6.17 | 2.70 | 0.26 | 0.12 | |
| ▲ | 7.24 | 5.69 | 2.69 | 1.00 | 0.16 |
| ★ | 4.22 | 1.57 | 0.11 | | |
| | | | | | |



Job No. OL-1922 | Date November 2, 1995

Hole No. H-7-95 | Sheet 1 of 1

Project Fife to Tukwila HOV Lanes

# Laboratory Summary

Washington State Department of Transportation

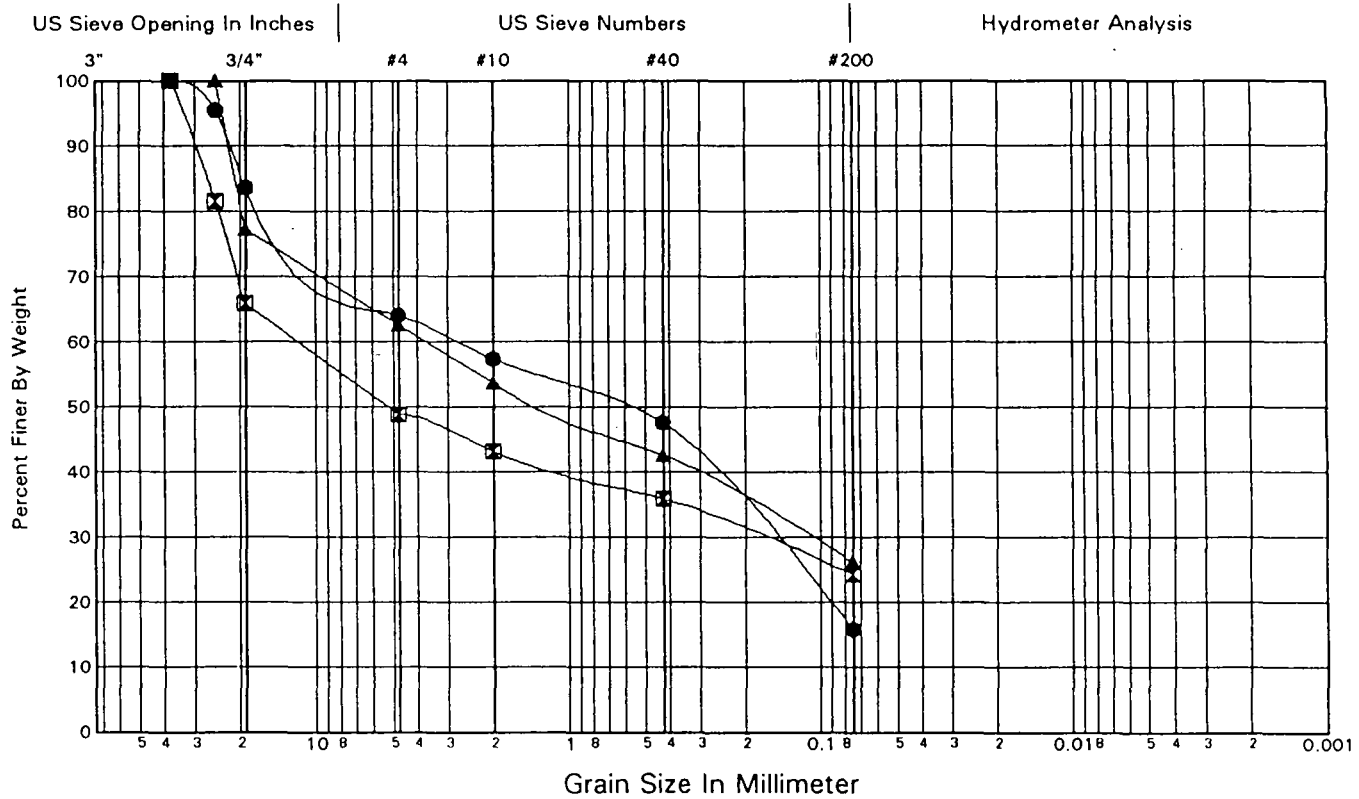
| | Depth (ft) | Depth (m) | Sample No. | USCS | Color | Description | MC% | LL | PL | PI |
|---|---|---|---|---|---|---|---|---|---|---|
| ● | 0.0 | 0.00 | D-1 | SM | Olive gray | SILTY SAND with GRAVEL | 7 | NP | NP | NP |
| ⊠ | 8.0 | 2.44 | D-3 | GM | Olive gray | SILTY GRAVEL with SAND | 6 | NP | NP | NP |
| ▲ | 13.0 | 3.96 | D-4 | GM | Olive gray | SILTY GRAVEL with SAND | 6 | NP | NP | NP |

## GRADATION FRACTIONS

| | %Gravel | %Sand | %Fines | Cu | Cc |
|---|---|---|---|---|---|
| ● | 36.0 | 48.1 | 15.9 | | |
| ⊠ | 51.1 | 24.6 | 24.3 | | |
| ▲ | 37.5 | 36.3 | 26.2 | | |

## GRADATION VALUES

| | D60 | D50 | D30 | D20 | D10 |
|---|---|---|---|---|---|
| ● | 2.83 | 0.62 | 0.16 | 0.09 | |
| ⊠ | 11.81 | 5.20 | 0.18 | | |
| ▲ | 3.71 | 1.18 | 0.11 | | |



CHAPTER 8

# Site-Specific Information - Wall 3

## Proposed Improvements

Figure 1-1 shows the location of Wall 3 relative to the other structures on the project. Figure 8-1, the site plan, shows the proposed location of Wall 3 at the S. 320th Street and SR 5 interchange. The proposed location is on the east side of the existing SR 5 north on-ramp from S. 320th Street (DR3 Line). Wall cross-sections provided by WSDOT indicate that the wall will be located in the existing roadway embankment fill. The wall will retain existing road fill as well as new fill required for the roadway widening. Project wall information obtained from the cross-sections is summarized in Table 8-1.

**Table 8-1**
**Wall 3 Information**

| | |
|---|---|
| Wall Stationing | DR3 0+ 020 to DR3 0 + 270 |
| Wall Height | 1.4 to 3.3m (5 to 11 ft.) |
| Bottom of Wall Footing Elevation | 132 to 139 m (433 to 456 ft.) or about 1 m (3.3 ft.) below existing grade |
| Wall Footing Location | 1.5 m (5 ft.) behind the face of the existing 2H:1V slope |

## Site Description

The proposed wall will be located on the existing 2H:1V roadway embankment fill that has a maximum height of about 10 m (30 ft.). The southern portion of the site is currently vegetated with small, evergreen trees, alder, and brush. The northern portion of the site is cleared and sparsely vegetated with grass.

A stormwater detention pond is located to the west of the roadway embankment. A relatively deep culvert [estimated to be at least 10 m (30 ft.)] below the roadway grade, located near Station 0 + 125 appears to drain the detention pond to a wetland located east of the on-ramp, at the toe of the embankment slope.

Surface runoff collected in the northern end of the grassy swale between DR3 Line and northbound SR 5 does not drain southward to the detention pond, but ponds in the swale. A shallow (less than 1 m (3 ft.) deep) culvert located near DR3 0+260 appears to have once drained this portion of the median to the east. The western end of this culvert appears to be clogged or buried.

# Subsurface Information

Subsurface explorations, designated H-8-95, H-9-95 and H-10-95, were drilled and logged by WSDOT at the locations indicated on Figure 8-1, Site Plan. Detailed logs are provided at the end of this chapter. The borings were drilled from about 6.5- to 9-m (2.0- to 2.7-ft.) below existing grade. Grain-size distributions and moisture contents were determined for selected soil samples. Laboratory test results are presented at the end of this chapter.

Subsurface conditions generally consist of a medium-dense to dense, silty sand with gravel. The groundwater table was not determined at the time of drilling.

# Recommendations

Conditions are suitable for the use of either standard concrete cantilever or preapproved proprietary mechanically stabilized earth (MSE) walls. A standard concrete cantilever wall is recommended because of the relatively short wall height, variation of height with length, and limited space for temporarily stockpiling excavated material. The existing slope geometry will require less excavation and backfill for a standard concrete cantilever wall than for an MSE wall.

The following soil parameters are recommended for use in design of the proposed retaining wall.

## Foundation Soils

| | |
|---|---|
| Angle of internal friction: | 36° |
| Unit weight: | 19.6 KN/m$^3$ [125 pounds per cubic foot (pcf)] |
| Allowable soil bearing pressure: | 190 kPa [4000 pounds per square foot (psf)] |
| Estimated elastic settlement: | less than 10 to 15 mm (1/2 inch) |
| Ultimate lateral passive earth pressure: | h*47 KN/m$^3$ (300 pcf), where h = distance from top of backfill. A factor of safety of 1.5 has been applied to the passive earth pressure coefficient to limit wall movement. |

## Retained Fill (Existing road fill)

| | |
|---|---|
| Angle of internal friction: | 36° |
| Unit weight: | 19.6 KN/m$^3$ [125 pounds per cubic foot (pcf)] |

## Proposed Compacted Fill Behind Wall

| | |
|---|---|
| Angle of internal friction: | 36° |
| Unit weight: | 19.6 KN/m$^3$ [125 pounds per cubic foot (pcf)] |
| Lateral active earth pressure: | h* 5.5 KN/m$^3$ (35 pcf) for unrestrained wall plus 4.5 kPa (65 psf) surcharge for traffic loading |

## Other Design Considerations

Removal and/or relocation of the culvert at Station DR 0+260 will be required due to the proposed wall construction. Surface water runoff control can be re-established by re-grading the area to the west of the on-ramp to promote drainage to an existing stormwater detention pond located to the south.

New road fill should be notched into the existing embankment slope.

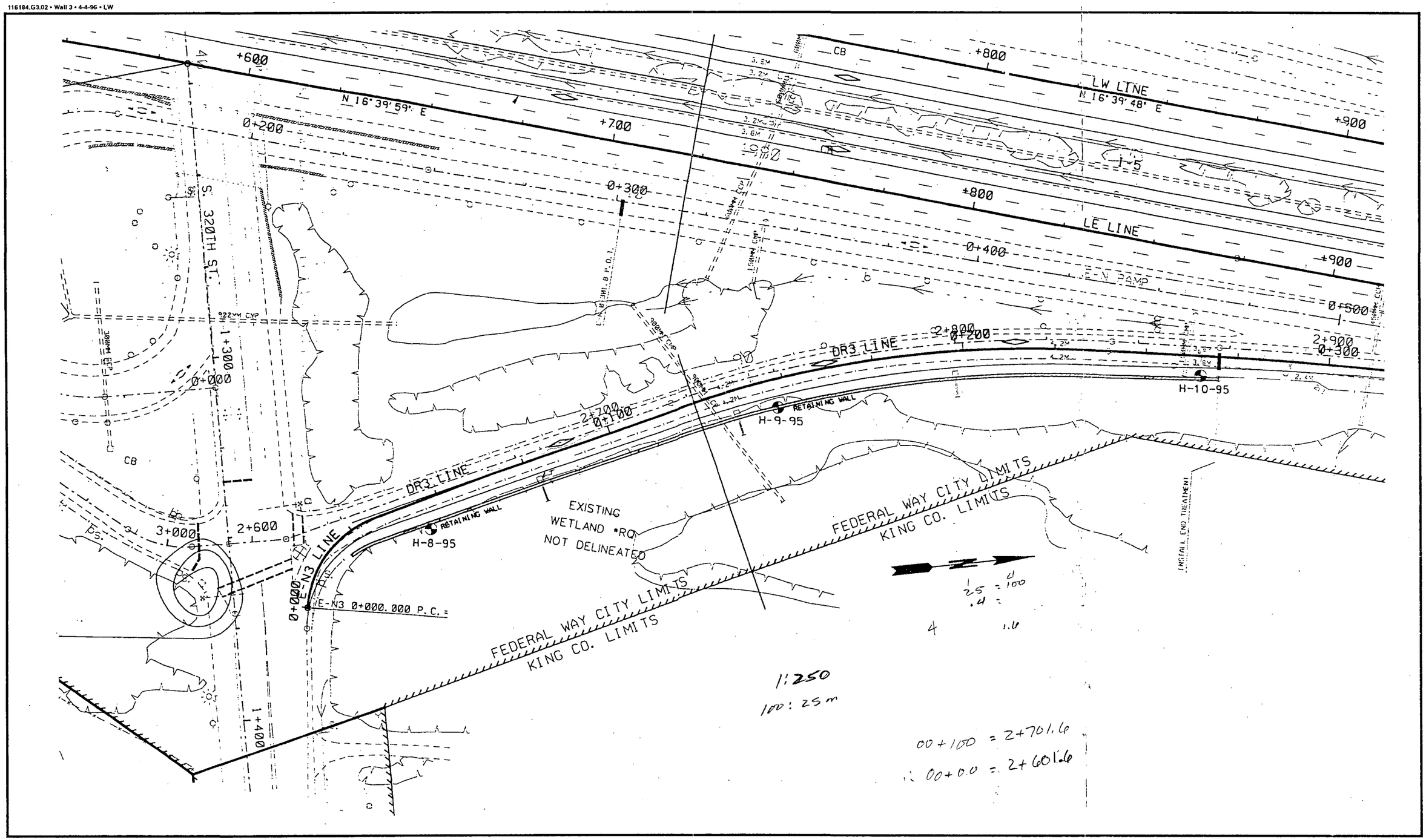
Wall fill should consist of a free-draining, compacted structural fill that conforms to the requirements for gravel borrow in Section 9-03.14 of the Standard Specifications. Backfill should be compacted to 95 percent of the maximum density as specified under Compacting Earth Embankments, Method C, in Section 2-03.3(14)C. The moisture content of the backfill shall meet the requirements of Section 2-03.3(14)C, Method C.

## Construction Considerations

Open-cut excavations are anticipated for the wall construction. Temporary cuts of 1H:1V are not anticipated to encroach upon the existing traffic lane.

Foundation soils that are not firm or that contain organics or debris, shall be removed and replaced as directed by the geotechnical engineer.

116184.G3.02 • Wall 3 • 4-4-96 • LW




Approximate
Scale in Meters

0 25 m

FIGURE 8-1
**Wall No. 3 Layout and Boring Locations**
Geotechnical Report SR-5
Port of Tacoma I/C to 272nd
Stage 4 HOV

# LOG OF TEST BORING

Washington State Department of Transportation

HOLE No. H-8-95

PROJECT Fife to Tukwila HOV Lanes

Job No. OL-1922 (3519)

Stage 4 Walls

S.R. 5

Station DR3 Line 0+045

Offset 8m Rt.

C.S. 1727

Equipment

Casing 20'

Ground El 449.3 (136.95 m)

Method of Boring Wet Rotary

Start Date August 9, 1995

Completion Date August 9, 1995

Sheet 1 of 2

| Depth (ft) | Meters (m) | Profile | Standard Penetration Blows/ft (10 20 30 40) | SPT Blows/6" (N) | Sample Type | Sample No. (Tube No.) | Lab Tests | Description of Material | Groundwater | Instrument |
|---|---|---|---|---|---|---|---|---|---|---|
| | | | | 5<br>50/0"<br>(50/0") | | D-1 | GS<br>MC | 1 ft. = 0.3048 m.<br>SM, M.C. = 1%<br>Silty SAND with gravel and root hairs and scattered fir needles, angular, very dense, light brown, dry, homogeneous. Driving on a cobble.<br>Retained 0.5 ft. | | |
| 5 | 1, 2 | | | 15<br>17<br>17<br>(34)<br>13<br>16<br>18<br>(34) | | D-2<br>D-3 | GS<br>MC<br>PI | GM, M.C. = 11%, PI = NP<br>Silty GRAVEL with sand, subangular, dense, light gray, moist, homogeneous.<br>Retained 0.9 ft.<br>Silty GRAVEL with sand, subangular, dense, light gray, moist, homogeneous.<br>Retained 0.9 ft. | | |
| 10 | 3 | | | 8<br>12<br>16<br>(28)<br>10<br>29<br>21<br>(40) | | D-4<br>D-5 | GS<br>MC<br>PI<br>GS<br>MC<br>PI | SM, M.C. = 14%, PI = NP<br>Silty SAND with gravel, subangular, dense, gray, moist, homogeneous.<br>Retained 0.5 ft.<br>SM, M.C. = 18%, PI = NP<br>Silty SAND with gravel, roots and decayed wood, dense, dark brown, stratified.<br>Retained 0.4 ft. | | |
| 15 | 4, 5 | | | 16<br>16<br>16<br>(32)<br>15<br>17<br>17<br>(34) | | D-6<br>D-7 | GS<br>MC<br>PI | GM, M.C. = 13%, PI = NP<br>Silty GRAVEL with sand, subrounded, dense, gray, moist, homogeneous.<br>Retained 0.8 ft.<br>Silty GRAVEL with sand, subrounded, dense, gray, moist, homogeneous.<br>Retained 0.9 ft. | | |
| 20 | 6 | | | 10<br>13 | | D-8 | | Silty GRAVEL with sand, subrounded, dense, gray, moist, homogeneous. | | |

# LOG OF TEST BORING

Washington State Department of Transportation

HOLE No. H-8-95

Sheet 2 of 2

PROJECT Fife to Tukwila HOV Lanes

Job No. OL-1922

| Depth (ft) | Meters (m) | Profile | Standard Penetration Blows/ft (10 20 30 40) | SPT Blows/6" (N) | Sample Type | Sample No. (Tube No.) | Lab Tests | Description of Material | Groundwater | Instrument |
|---|---|---|---|---|---|---|---|---|---|---|
| | | | | 14 (27) 11 14 14 (28) | | D-9 | | Retained 0.7 ft.<br>Silty GRAVEL with sand, subrounded, dense, gray, moist.<br>Retained 0.6 ft. | | |
| | 7 | | | | | | | End of the Test Hole Boring at 22.0 ft. below ground elevation. | | |
| 25 | | | | | | | | This is a Summary Log of the Test Hole Boring. Soil/Rock descriptions are derived from visual field identifications and laboratory test data. | | |
| | 8 | | | | | | | | | |
| | 9 | | | | | | | | | |
| 30 | | | | | | | | | | |
| | 10 | | | | | | | | | |
| 35 | | | | | | | | | | |
| | 11 | | | | | | | | | |
| | 12 | | | | | | | | | |
| 40 | | | | | | | | | | |
| | 13 | | | | | | | | | |
| 45 | | | | | | | | | | |

# LOG OF TEST BORING

Washington State Department of Transportation

HOLE No. H-9-95

PROJECT Fife to Tukwila HOV Lanes — Job No. OL-1922

Stage 4 Walls — S.R. 5

Station 0+147 — Offset 7.04 m. Rt. — C.S. 1727

Equipment — Casing HQ X 30' — Ground El 441.8 (134.66 m)

Method of Boring Wet Rotary

Start Date September 13, 1995 — Completion Date September 13, 1995 — Sheet 1 of 2

| Depth (ft) | Meters (m) | Profile | Standard Penetration Blows/ft (10 20 30 40) | SPT Blows/6" (N) | Sample Type | Sample No. (Tube No.) | Lab Tests | Description of Material | Groundwater | Instrument |
|---|---|---|---|---|---|---|---|---|---|---|
| | | | | | | | | 1 ft. = 0.3048 m. | | |
| | | | | 14 8 14 (22) | | D-1 | GS MC PI | SM, M.C. = 13%, PI = NP<br>Silty SAND with gravel, less than one inch, subround, very stiff, gray and brown, moist, mottled grey, brown, dark brown.<br>Retained 1.2 ft. | | |
| 5 | 1 | | | 22 25 28 (53) | | D-2 | GS MC PI | SM, M.C. = 9%, PI = NP<br>Silty SAND with gravel, hard, gray, moist, homogeneous.<br>Retained 1.0 ft. | | |
| | 2 | | | 10 11 18 (29) | | D-3 | GS MC PI | GM, M.C. = 8%, PI = NP<br>Silty GRAVEL with sand, less than one inch, subround and subangular, dense, brownish gray, moist, homogeneous, occasional laminae.<br>Retained 1.2 ft. | | |
| 10 | 3 | | | 27 43 30 (73) | | D-4 | | Silty GRAVEL with sand, very dense, brownish gray, moist, homogeneous.<br>Retained 1.0 ft. | | |
| | 4 | | | 10 9 10 (19) | | D-5 | | Silty GRAVEL with sand and organics, less than one inch, subround, medium dense, moist, stratified with one to five inch layers, mottled brown, gray, blue, olive and rust.<br>Retained 1.0 ft. | | |
| 15 | 5 | | | 15 17 22 (39) | | D-6 | | Silty GRAVEL with sand, organics and twigs, dense, greenish gray, moist to wet, mottled, gray, blue, olive and orange.<br>Retained 1.0 ft. | | |
| 20 | 6 | | | | | | | | | |

# LOG OF TEST BORING

Washington State Department of Transportation

HOLE No. H-9-95

Sheet 2 of 2

PROJECT Fife to Tukwila HOV Lanes

Job No. OL-1922

| Depth (ft) | Meters (m) | Profile | Standard Penetration Blows/ft (10 20 30 40) | SPT Blows/6" (N) | Sample Type | Sample No. (Tube No.) | Lab Tests | Description of Material | Groundwater | Instrument |
|---|---|---|---|---|---|---|---|---|---|---|
| | | | | 20 26 14 (40) | | D-7 | | No recovery. | | |
| | 7 | | | | | | | | | |
| | | | | 11 8 9 (17) | | D-8 | | No recovery. | | |
| 25 | | | | | | | | | | |
| | 8 | | | | | | | | | |
| | | | | 1 5 8 (13) | | D-9 | GS MC PI | SM, M.C. = 12%, PI = NP<br>Silty SAND with gravel, wood fragments, and fibrous organic material, dark brown, black and dark brownish gray, medium dense, wet, layered.<br>Retained 1.0 ft. | | |
| | 9 | | | | | | | End of test hole boring at 28.5 ft. below ground elevation. | | |
| 30 | | | | | | | | Water table elevation not determined. | | |
| | | | | | | | | This is a summary Log of Test Boring. Soil/Rock descriptions are derived from visual field identifications and laboratory test data. | | |
| | 10 | | | | | | | | | |
| 35 | | | | | | | | | | |
| | 11 | | | | | | | | | |
| | 12 | | | | | | | | | |
| 40 | | | | | | | | | | |
| | 13 | | | | | | | | | |
| 45 | | | | | | | | | | |

# LOG OF TEST BORING

Washington State Department of Transportation

HOLE No. H-10-95

PROJECT Fife to Tukwila HOV Lanes — Job No. OL-1922

Stage 4 Walls — S.R. 5

Station 0+265 — Offset 8 m. Rt. — C.S. 1727

Equipment — Casing HQ X 25' — Ground El 436.5 (133.05 m)

Method of Boring Wet Rotary

Start Date September 19, 1995 — Completion Date September 19, 1995 — Sheet 1 of 2

| Depth (ft) | Meters (m) | Profile | Standard Penetration Blows/ft (10 20 30 40) | SPT Blows/6" (N) | Sample Type | Sample No. (Tube No.) | Lab Tests | Description of Material | Groundwater | Instrument |
|---|---|---|---|---|---|---|---|---|---|---|
| | | | | | | | | 1 ft. = 0.3048 m. | | |
| | 1 | | | 13 18 14 (32) | | D-1 | GS MC PI | SM, M.C. = 13%, PI = NP Silty SAND with gravel, dense, brownish gray, moist, homogeneous. Retained 1.2 ft. | | |
| 5 | | | | 14 10 7 (17) | | D-2 | | Silty SAND with gravel, medium dense, brownish gray, moist to wet, homogeneous. Retained 0.9 ft. | | |
| | 2 | | | | | | | | | |
| | | | | A B | | U-3 | | Push easy then stop. Sandy SILT with gravel, mottled brown, gray, orange. | | |
| | | | | 12 17 17 (34) | | D-4 | GS MC PI | SM, M.C. = 11%, PI = NP Silty SAND with gravel, dense, moist, brownish gray, stratified, mottled gray, brown, orange. Retained 1.4 ft. | | |
| 10 | 3 | | | 32 32 33 (65) | | D-5 | | Silty SAND with gravel, very dense, light brownish gray, moist, homogeneous. Retained 0.9 ft. | | |
| | 4 | | | 22 29 35 (64) | | D-6 | GS MC PI | SM, M.C. = 11%, PI = NP Silty SAND with gravel, less than one inch, angular, subangular, subround, very dense, light gray, moist, homogeneous. Retained 1.0 ft. | | |
| 15 | | | | | | | | | | |
| | 5 | | | | | | | | | |
| | | | | 27 27 32 (59) | | D-7 | | Silty SAND with gravel, very dense, light gray, moist, homogeneous. Retained 1.2 ft. | | |
| | 6 | | | | | | | | | |
| 20 | | | | | | | | | | |

# LOG OF TEST BORING

Washington State Department of Transportation

HOLE No. H-10-95

Sheet 2 of 2

PROJECT Fife to Tukwila HOV Lanes

Job No. OL-1922

| Depth (ft) | Meters (m) | Profile | Standard Penetration Blows/ft (10 20 30 40) | SPT Blows/6" (N) | Sample Type | Sample No. (Tube No.) | Lab Tests | Description of Material | Groundwater | Instrument |
|---|---|---|---|---|---|---|---|---|---|---|
| | 7 | | > > ◆ | 23 70/6 (70/6") | ▼ | D-8 | | Silty SAND with gravel, very dense, brownish gray, moist, laminated, disturbed bedding. Retained 1.0 ft. | | |
| 25 | | | | | | | | End of test hole boring at 24.0 ft. below ground elevation. | | |
| | 8 | | | | | | | This is a summary Log of Test Boring. Soil/Rock descriptions are derived from visual field identifications and laboratory test data. | | |
| 30 | 9 | | | | | | | | | |
| | 10 | | | | | | | | | |
| 35 | 11 | | | | | | | | | |
| 40 | 12 | | | | | | | | | |
| | 13 | | | | | | | | | |
| 45 | | | | | | | | | | |

Job No. OL-1922 | Date November 2, 1995

Hole No. H-8-95 | Sheet 1 of 1

Project Fife to Tukwila HOV Lanes

# Laboratory Summary

Washington State Department of Transportation

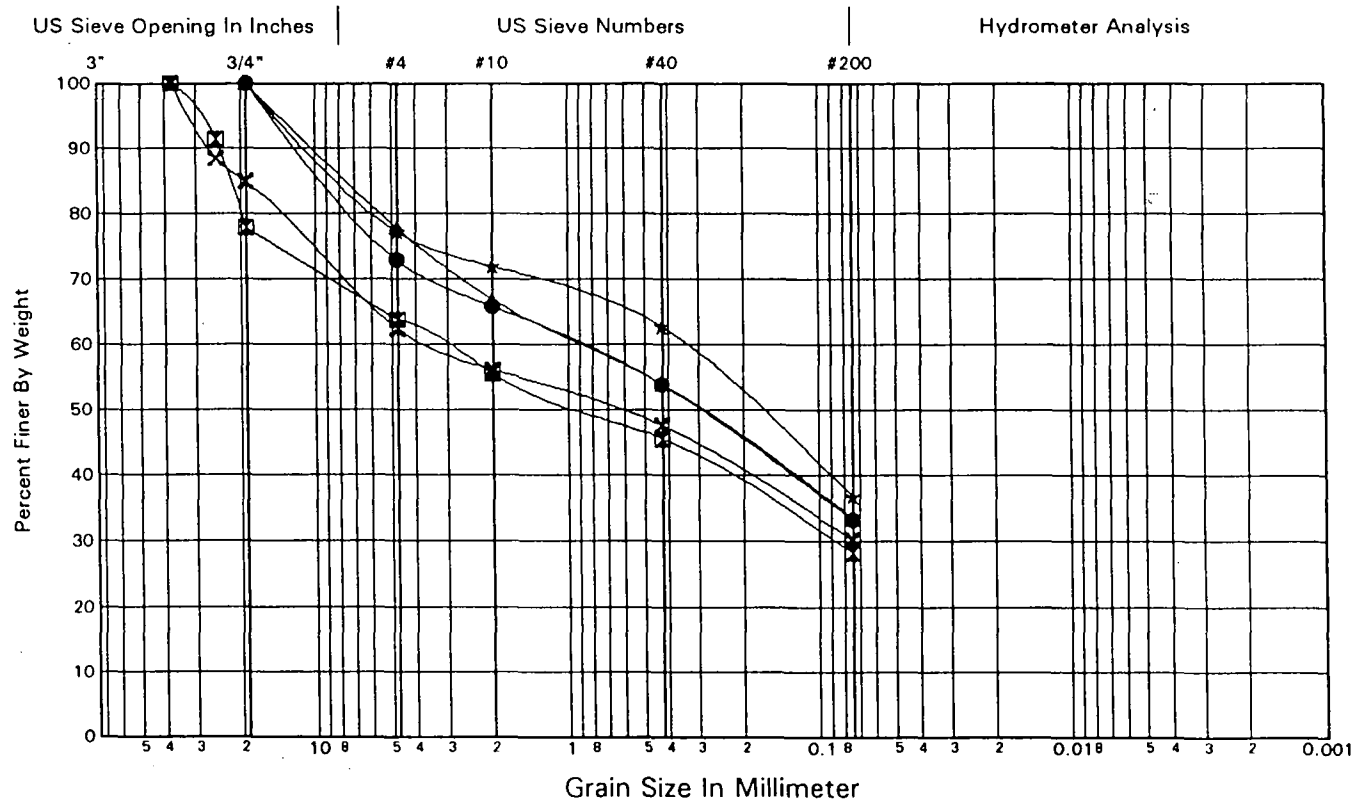
| | Depth (ft) | Depth (m) | Sample No. | USCS | Color | Description | MC% | LL | PL | PI |
|---|---|---|---|---|---|---|---|---|---|---|
| ● | 0.0 | 0.00 | D-1 | SM | Light olive gray | SILTY SAND with GRAVEL and root hairs | 1 | NP | NP | NP |
| ⊠ | 4.0 | 1.22 | D-2 | GM | Light olive gray | SILTY GRAVEL with SAND | 11 | NP | NP | NP |
| ▲ | 9.0 | 2.74 | D-4 | SM | Gray | SILTY SAND with GRAVEL | 14 | NP | NP | NP |
| ★ | 10.5 | 3.20 | D-5 | SM | Olive gray | SILTY SAND with GRAVEL | 18 | NP | NP | NP |
| ✕ | 14.0 | 4.27 | D-6 | GM | Light olive gray | SILTY GRAVEL with SAND | 13 | NP | NP | NP |

## GRADATION FRACTIONS

| | %Gravel | %Sand | %Fines | Cu | Cc |
|---|---|---|---|---|---|
| ● | 27.1 | 39.7 | 33.2 | | |
| ⊠ | 36.2 | 35.6 | 28.2 | | |
| ▲ | 22.2 | 44.2 | 33.6 | | |
| ★ | 22.8 | 40.4 | 36.8 | | |
| ✕ | 37.6 | 32.2 | 30.2 | | |

## GRADATION VALUES

| | D60 | D50 | D30 | D20 | D10 |
|---|---|---|---|---|---|
| ● | 0.94 | 0.31 | | | |
| ⊠ | 3.20 | 0.85 | 0.09 | | |
| ▲ | 0.87 | 0.30 | | | |
| ★ | 0.35 | 0.18 | | | |
| ✕ | 3.42 | 0.66 | | | |



Job No. **OL-1922** Date **November 3, 1995**

Hole No. **H-9-95** Sheet **1** of **1**

Project **Fife to Tukwila HOV Lanes**

# Laboratory Summary

Washington State Department of Transportation

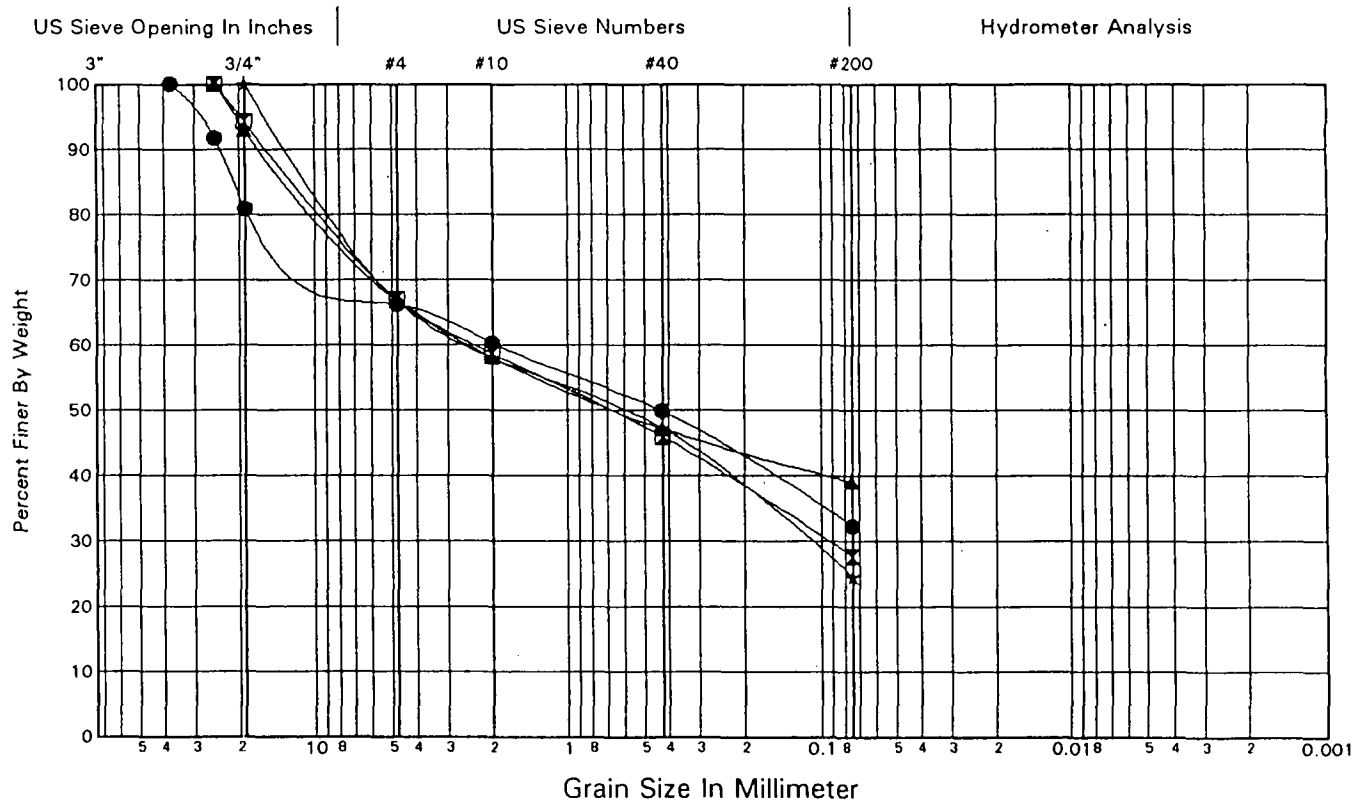
| | Depth (ft) | Depth (m) | Sample No. | USCS | Color | Description | MC% | LL | PL | PI |
|---|---|---|---|---|---|---|---|---|---|---|
| ● | 2.0 | 0.61 | D-1 | SM | Olive gray | SILTY SAND with GRAVEL | 13 | NP | NP | NP |
| ⊠ | 4.0 | 1.22 | D-2 | SM | Gray | SILTY SAND with GRAVEL | 9 | NP | NP | NP |
| ▲ | 7.0 | 2.13 | D-3 | GM | Olive gray | SILTY GRAVEL with SAND | 8 | NP | NP | NP |
| ★ | 27.0 | 8.23 | D-9 | SM | Gray to V. Dk. Gray | SILTY SAND with GRAVEL with wood and fibrous organic material. | 12 | NP | NP | NP |

## GRADATION FRACTIONS

| | %Gravel | %Sand | %Fines | Cu | Cc |
|---|---|---|---|---|---|
| ● | 33.8 | 33.9 | 32.3 | | |
| ⊠ | 32.9 | 39.5 | 27.6 | | |
| ▲ | 33.3 | 27.7 | 39.0 | | |
| ★ | 33.3 | 42.1 | 24.6 | | |

## GRADATION VALUES

| | D60 | D50 | D30 | D20 | D10 |
|---|---|---|---|---|---|
| ● | 1.91 | 0.43 | | | |
| ⊠ | 2.31 | 0.69 | 0.09 | | |
| ▲ | 2.44 | 0.64 | | | |
| ★ | 2.46 | 0.63 | 0.11 | | |




| Gravel | Sand | | | Silt and Clay |
|---|---|---|---|---|
| | Coarse | Medium | Fine | |

Job No. OL-1922 | Date November 2, 1995

Hole No. H-10-95 | Sheet 1 of 1

Project Fife to Tukwila HOV Lanes

# Laboratory Summary

Washington State Department of Transportation

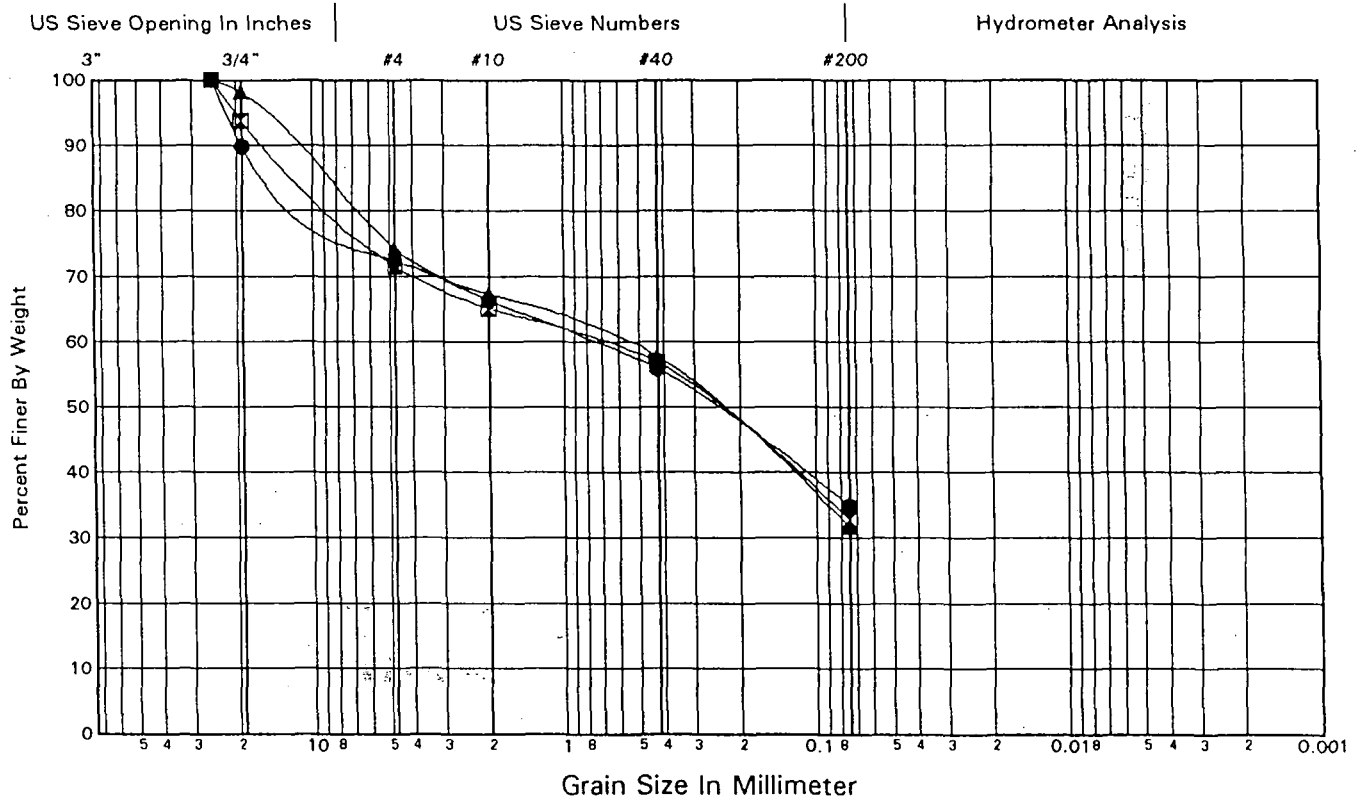
| | Depth (ft) | Depth (m) | Sample No. | USCS | Color | Description | MC% | LL | PL | PI |
|---|---|---|---|---|---|---|---|---|---|---|
| ● | 3.0 | 0.91 | D-1 | SM | Gray | SILTY SAND with GRAVEL | 13 | NP | NP | NP |
| ⊠ | 9.0 | 2.74 | D-4 | SM | Light gray | SILTY SAND with GRAVEL | 11 | NP | NP | NP |
| ▲ | 13.0 | 3.96 | D-6 | SM | Gray | SILTY SAND with GRAVEL | 11 | NP | NP | NP |

## GRADATION FRACTIONS

| | %Gravel | %Sand | %Fines | Cu | Cc |
|---|---|---|---|---|---|
| ● | 27.6 | 37.5 | 34.9 | | |
| ⊠ | 28.4 | 38.9 | 32.7 | | |
| ▲ | 25.7 | 42.7 | 31.6 | | |

## GRADATION VALUES

| | D60 | D50 | D30 | D20 | D10 |
|---|---|---|---|---|---|
| ● | 0.78 | 0.26 | | | |
| ⊠ | 0.76 | 0.26 | | | |
| ▲ | 0.60 | 0.25 | | | |



Chapter 8
# Appendix

CHAPTER 8

# Site-Specific Information - Wall 3

## Proposed Improvements

Figure 1-1 shows the location of Wall 3 relative to the other structures on the project. Figure 8-1, the site plan, shows the proposed location of Wall 3 at the S. 320th Street and SR 5 interchange. The proposed location is on the east side of the existing SR 5 north on-ramp from S. 320th Street (DR3 Line). Wall cross-sections provided by WSDOT indicate that the wall will be located in the existing roadway embankment fill. The wall will retain existing road fill as well as new fill required for the roadway widening. Project wall information obtained from the cross-sections is summarized in Table 8-1.

**Table 8-1**
**Wall 3 Information**

| | |
|---|---|
| Wall Stationing | DR3 0+ 020 to DR3 0 + 270 |
| Wall Height | 1.4 to 3.3m (5 to 11 ft.) |
| Bottom of Wall Footing Elevation | 132 to 139 m (433 to 456 ft.) or about 1 m (3.3 ft.) below existing grade |
| Wall Footing Location | 1.5 m (5 ft.) behind the face of the existing 2H:1V slope |

## Site Description

The proposed wall will be located on the existing 2H:1V roadway embankment fill that has a maximum height of about 10 m (30 ft.). The southern portion of the site is currently vegetated with small, evergreen trees, alder, and brush. The northern portion of the site is cleared and sparsely vegetated with grass.

A stormwater detention pond is located to the west of the roadway embankment. A relatively deep culvert [estimated to be at least 10 m (30 ft.)] below the roadway grade, located near Station 0 + 125 appears to drain the detention pond to a wetland located east of the on-ramp, at the toe of the embankment slope.

Surface runoff collected in the northern end of the grassy swale between DR3 Line and northbound SR 5 does not drain southward to the detention pond, but ponds in the swale. A shallow (less than 1 m (3 ft.) deep) culvert located near DR3 0+260 appears to have once drained this portion of the median to the east. The western end of this culvert appears to be clogged or buried.

# Subsurface Information

Subsurface explorations, designated H-8-95, H-9-95 and H-10-95, were drilled and logged by WSDOT at the locations indicated on Figure 8-1, Site Plan. Detailed logs are provided at the end of this chapter. The borings were drilled from about 6.5- to 9-m (2.0- to 2.7-ft.) below existing grade. Grain-size distributions and moisture contents were determined for selected soil samples. Laboratory test results are presented at the end of this chapter.

Subsurface conditions generally consist of a medium-dense to dense, silty sand with gravel. The groundwater table was not determined at the time of drilling.

# Recommendations

Conditions are suitable for the use of either standard concrete cantilever or preapproved proprietary mechanically stabilized earth (MSE) walls. A standard concrete cantilever wall is recommended because of the relatively short wall height, variation of height with length, and limited space for temporarily stockpiling excavated material. The existing slope geometry will require less excavation and backfill for a standard concrete cantilever wall than for an MSE wall.

The following soil parameters are recommended for use in design of the proposed retaining wall.

## Foundation Soils

| | |
|---|---|
| Angle of internal friction: | 36° |
| Unit weight: | 19.6 KN/m$^3$ [125 pounds per cubic foot (pcf)] |
| Allowable soil bearing pressure: | 190 kPa [4000 pounds per square foot (psf)] |
| Estimated elastic settlement: | less than 10 to 15 mm (1/2 inch) |
| Ultimate lateral passive earth pressure: | h*47 KN/m$^3$ (300 pcf), where h = distance from top of backfill. A factor of safety of 1.5 has been applied to the passive earth pressure coefficient to limit wall movement. |

## Retained Fill (Existing road fill)

| | |
|---|---|
| Angle of internal friction: | 36° |
| Unit weight: | 19.6 KN/m$^3$ [125 pounds per cubic foot (pcf)] |

## Proposed Compacted Fill Behind Wall

| | |
|---|---|
| Angle of internal friction: | 36° |
| Unit weight: | 19.6 KN/m$^3$ [125 pounds per cubic foot (pcf)] |
| Lateral active earth pressure: | h* 5.5 KN/m$^3$ (35 pcf) for unrestrained wall plus 4.5 kPa (65 psf) surcharge for traffic loading |

## Other Design Considerations

Removal and/or relocation of the culvert at Station DR 0+260 will be required due to the proposed wall construction. Surface water runoff control can be re-established by re-grading the area to the west of the on-ramp to promote drainage to an existing stormwater detention pond located to the south.

New road fill should be notched into the existing embankment slope.

Wall fill should consist of a free-draining, compacted structural fill that conforms to the requirements for gravel borrow in Section 9-03.14 of the Standard Specifications. Backfill should be compacted to 95 percent of the maximum density as specified under Compacting Earth Embankments, Method C, in Section 2-03.3(14)C. The moisture content of the backfill shall meet the requirements of Section 2-03.3(14)C, Method C.

## Construction Considerations

Open-cut excavations are anticipated for the wall construction. Temporary cuts of 1H:1V are not anticipated to encroach upon the existing traffic lane.

Foundation soils that are not firm or that contain organics or debris, shall be removed and replaced as directed by the geotechnical engineer.

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