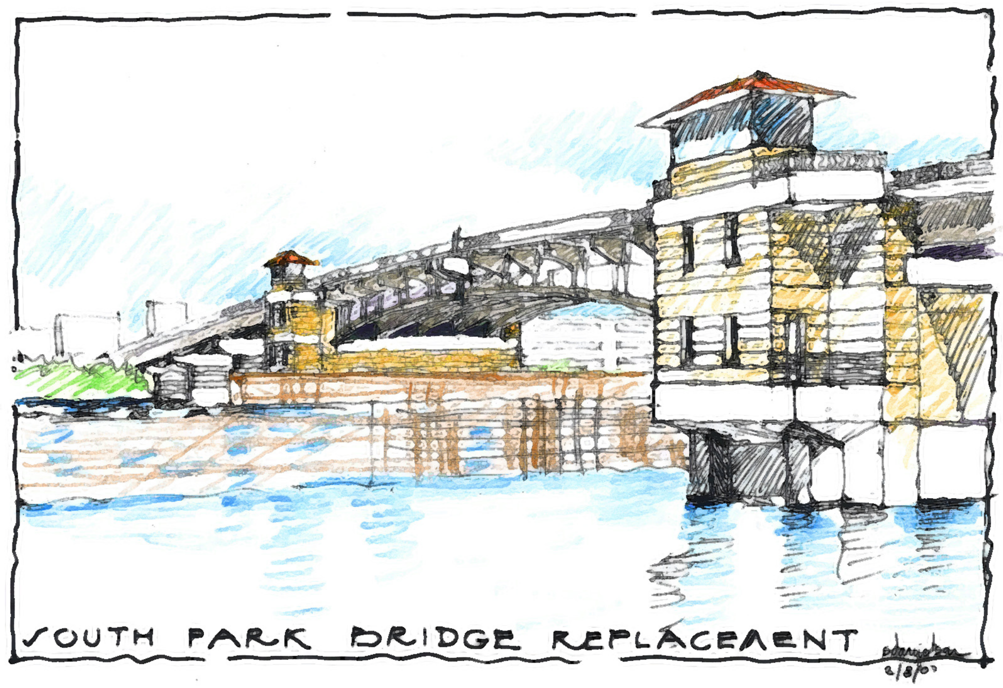
South Park Bridge Replacement
Volume 3: Geotechnical Report



Prepared for: King County

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August 2007

Geotechnical Report
Phase II
South Park Bridge Project
King County, Washington

March 29, 2004

SHANNON & WILSON, INC.

GEOTECHNICAL AND ENVIRONMENTAL CONSULTANTS

At Shannon & Wilson, our mission is to be a progressive, well-managed professional consulting firm in the fields of engineering and applied earth sciences. Our goal is to perform our services with the highest degree of professionalism with due consideration to the best interests of the public, our clients, and our employees.

Submitted To:
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21-1-09584-008

SHANNON & WILSON, INC.
GEOTECHNICAL AND ENVIRONMENTAL CONSULTANTS

50 Years 1954 - 2004

March 30, 2004

Parsons Brinckerhoff
999 Third Avenue, Suite 2200
Seattle, WA 98104

Attn: Mr. Einer Handeland

RE: GEOTECHNICAL REPORT, PHASE II, SOUTH PARK BRIDGE PROJECT, KING COUNTY, WASHINGTON

Enclosed are twenty-five (25) copies of our geotechnical report for Phase II of the South Park Bridge Project. A draft report was submitted to you in November 2003. This report has incorporated the comments on the draft report, which were previously addressed in a letter dated March 16, 2004.

We look forward to the next phase this project. If you have any questions or require further information, please contact me at (206) 632-8020.

Sincerely,

SHANNON & WILSON, INC.

Ming-Jiun (Jim) Wu, P.E.
Senior Vice President

JW/lkd

Enclosures: Geotechnical Report, Phase II, South Park Bridge Project, King County, Washington (25 copies)

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EXECUTIVE SUMMARY

The South Park Bridge has deteriorated significantly in recent years and is being studied for either rehabilitation or replacement. The proposed replacement structures include a new bascule bridge, a mid-level fixed span, and a high-level fixed span. All replacement structures would be located immediately downstream of the existing bridge.

For this current study, eight soil borings were drilled to depths ranging from about 90 to 131 feet below the existing ground surface or mudline. Two borings were drilled over water in the Duwamish Waterway, three borings were drilled on shore for the south approach, and three borings were drilled on shore for the north approach. Two of the on-land borings were fitted with casing for geophysical testing. The geophysical testing consisted of shear wave velocity measurements for seismic site response analyses of the project.

A test pile was also driven and monitored at the north approach of the existing bridge. The test pile was a 24-inch steel pipe pile driven closed-ended to a depth of 74 feet below the ground surface. The monitoring consisted of dynamic load testing, horizontal and vertical settlement of the adjacent ground surface, and ground vibrations during pile driving.

The subsurface exploration encountered fill on shore ranging in thickness between 3 and 12 feet thick. Underlying the fill and mudline of the waterway is alluvial soil consisting of very loose sand with variable amounts of silt and clay. The alluvial soils generally become more dense with increasing depth. The alluvial soils are thickest near the north shore of the proposed replacement bridge crossing. Underlying the alluvial soils is glacially consolidated soil consisting of very stiff to hard clay and silt.

The site is susceptible to liquefaction from the very loose to medium dense sand of the alluvial soil. Liquefaction during a design earthquake having a recurrence interval of 475 years would likely result in 1 to 8 inches of vertical settlement and 3 to 30 feet of lateral spreading towards the waterway. The amount of settlement and lateral spreading generally increases with increasing proximity to the waterway.

To mitigate the liquefaction potential, we recommend that a combination of ground improvement and deep foundations be considered to support the bridge structures. Due to the existing site conditions and possible environmental constraints, we recommend that Earthquake Drains be

considered for the ground improvement along the proposed bridge alignment. Estimated preliminary costs for Earthquake Drains range from $170,000 to $960,000 depending on the bridge alternative and drain spacing. Compaction grouting is a possible alternative, but it is anticipated to cost at least three times greater than Earthquake Drains. The ground improvements would be located to protect the foundation elements from the large earth pressures caused by lateral spreading.

We recommend that a deep foundation system be used to support the bridge structures. The deep foundation systems studied include 24-inch-diameter steel pipe piles and drilled shafts having diameters of 6, 8, and 10 feet. The selected foundation system will be based on proximity to sensitive structures, depth of the foundation element, and constructability. We have provided information to determine axial and lateral capacities of the proposed deep foundations for all bridge alternatives at each boring location.

The new bridge alternative would likely have fill embankments at the north and south approaches. We recommend that a mechanically stabilized earth (MSE) wall system be considered to retain the fill embankments. The MSE wall system has the greatest capacity to tolerate settlements and resist static and seismic loadings within a relatively small footprint.

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GEOTECHNICAL REPORT
PHASE II
SOUTH PARK BRIDGE PROJECT
KING COUNTY, WASHINGTON

1.0 INTRODUCTION

1.1 General

This report presents the results of our field explorations, laboratory testing, and geotechnical engineering studies for the South Park Bridge project located in King County, Washington. The site is located as shown on the Vicinity Map, Figure 1. The existing South Park Bridge has deteriorated significantly in recent years. Based on an alternative development and screening study performed by Parsons Brinckerhoff, Inc. (PB), five project alternatives were selected for further studies. The five proposed project alternatives include a "no action" alternative, a rehabilitation alternative, and three replacement bridge alternatives (bascule, mid-level fixed-span, and high-level fixed-span bridge alternatives). This report addresses geotechnical design of the rehabilitation and replacement bridge alternatives.

This project was authorized by a contract between PB and Shannon & Wilson, Inc. Notice to proceed for this project was received June 23, 2003.

As stated in our revised proposal dated July 21, 2003, our proposed scope to support a preliminary design for the selection of the preferred alternative is presented in the following sections.

1.2 Task 1 – On-Land Field Explorations

The following tasks were completed in support of the six on-land soil borings (borings SB-1, SB-2, SB-3, SB-6, SB-7, and SB-8) and other field explorations:

1. Coordinated the field activities with the project team and obtain utility clearances.

2. Submitted street use permits to drill borings located within City of Seattle and City of Tukwila right-of-ways (ROW).

3. Drilled six geotechnical borings using hollow-stem auger (HSA) drilling technique in approximately the upper 20 feet, then switched to mud rotary drilling technique below. The six borings were drilled to depths ranging from 90.3 to 130.7 feet below the ground surface (bgs). Three borings were drilled south of the Duwamish Waterway (borings SB-6 through SB-8) and the remaining three borings were drilled to the north (borings SB-1 through SB-3).

4. Mud rotary technique was used to complete the borings in order to control the potential disturbance and heave of the saturated sands underlying the project alignment.

5. Collected Standard Penetration Test (SPT) samples from the geotechnical borings every 2.5 feet in the upper 20 feet, and every 5 feet below that. As requested by Mr. Scott Wilbur of Wilbur Consulting, Inc. (Wilbur), three extra samples were collected, below a depth of 50 feet, from each of the geotechnical borings. We understand that these samples were used for environmental testing.

6. Drilled six 4-inch HSA polyvinyl chloride (PVC) wells to an approximate depth of 25 feet bgs, collected SPT samples every 2.5 feet, and provided flush covers. Monitoring of the drilling for these wells and associated testing and analyses were the responsibility and under the supervision of Mr. Scott Wilbur of Wilbur Consulting, Inc.

7. Installed a 3-inch-diameter PVC casing at geotechnical borings SB-2 and SB-6. The casings were used to obtain shear wave velocity measurements (a measure of the soil stiffness) of the site soils by down-hole geophysical procedures. The measurements were used to perform site-specific ground response analyses for the project.

8. Installed a Sondex system attached to a 3-inch-diameter inclinometer casing in boring SB-1. The Sondex system was used to measure settlement of subsurface soils during driving of a test pile described in Task 3.

9. Collected cuttings, mud, and decontamination water into drums from all borings and label them. Boeing handled drums produced on Boeing property located at the south approach. The remaining drums were transported to an area located beneath the south approach of the existing bridge where they were stored until completion of the environmental testing. We understand the costs for disposing contaminated material was addressed by Wilbur.

10. Provided concrete coring services for boring SB-3 drilled on Boeing property, where the parking area consists of approximately 9-inch-thick concrete pavement.

11. Provided traffic control services for borings SB-1, SB-2, SB-6, SB-7, and SB-8 drilled outside Boeing property.

12. Performed down-hole geophysical tests consisting of shear wave velocity measurements in borings SB-2 and SB-6.

1.3 Task 2 – Over-water Field Explorations

The following tasks were performed in support of the two over-water soil borings (borings SB-4 and SB-5):

1. Drilled two geotechnical borings (SB-4 and SB-5) using mud rotary drilling technique (see Item 4 under Task 1) to depths ranging from 101.5 to 126.0 feet below the mudline. One boring was drilled near each of the proposed north and south main-span pier locations for the new bascule and fixed span bridge alternatives. A barge was used to drill both over-water borings. In order to avoid grounding of the barge, drilling of the over-water borings occurred during a period when the low tide did not drop below an elevation needed to float the barge and to keep a minimum distance of 10 feet from the water edge. The drilling occurred on July 22 through 24, 2003, during favorable tidal conditions.

2. Collected SPT samples from the two geotechnical borings every 2.5 feet in the upper 20 feet and every 5 feet below 20 feet. As requested by Mr. Scott Wilbur of Wilbur, three extra samples were collected in the upper 10 feet, and three extra samples below a depth of 50 feet, from each of the over-water borings. We understand that these samples were used for environmental testing.

3. Collected cuttings, mud, and decontamination water into drums from the borings and labeled them. Drums were lifted off the barge using a boom truck placed on the existing bridge. They were then transported to an area located beneath the south approach of the existing bridge where they are stored until environmental testing is complete. We understand the costs for disposing contaminated material was addressed by Wilbur.

4. Provided traffic control services for lifting the drums from the barge.

1.4 Task 3 – Field Monitoring, Engineering Analyses, and Report

The following tasks were completed in support of the field monitoring, analyses, and report preparation:

1. Provided support services to King County and the design team in preparation of the Biological Assessment to submit to U.S. Fish and Wildlife Service (USFWS), and assisted in preparation of Work Plan, Health and Safety Plan, and Sample Analysis Plan for the project.

2. Monitored drilling of and collected samples from geotechnical borings drilled under Task 1.

3. Monitored drilling of and collected samples from geotechnical borings drilled under Task 2.

4. Monitored pile driving and collected vibration and settlement measurements as described under Task 4.

5. Conducted laboratory testing on selected samples retrieved from the field explorations to determine basic index and engineering properties of the subsurface soils, such as static strength and compressibility. The laboratory testing was conducted in general accordance with appropriate American Society for Testing and Materials (ASTM) standards. Tests included determination of natural moisture content, grain size distribution, and plasticity characteristics (Atterberg Limits). The laboratory tests were conducted at Shannon & Wilson, Inc. laboratory in Seattle, Washington.

6. Developed logs for new geotechnical borings (six on-land and two over-water) drilled at the site.

7. Prepared a subsurface profile along the project alignment. The profile is based on results of existing and new field explorations completed at the project site.

8. Performed engineering analysis and evaluation of data derived from existing and new subsurface exploration and laboratory testing programs with respect to the items listed below:

    a. Seismic design considerations related to the proposed replacement bridge structure, including rock/hard soil ground motion and potential seismic hazards.

    b. Site response analyses. We used the computer program ProShake to assess the site soils using one-dimensional equivalent-linear methods. This and similar programs have been widely used to perform site-specific ground response analyses.

    c. Liquefaction susceptibility and mitigation measures.

    d. Suitable foundation types for abutments and the intermediate piers supporting the new bridge. Suitable foundation types include steel pipe piles or drilled shafts.

    e. Allowable axial capacities and settlement estimates of the selected foundations.

    f. LPILE parameters for determining the lateral resistance of the selected deep foundations.

    g. Stability of the Duwamish Waterway Bank.

    h. Lateral earth pressures for retaining wall design.

    i. Stability and settlement of approach embankments.

   j. Construction and erosion control considerations.

9. Prepared a draft report summarizing the results of our geotechnical engineering study including descriptions of surface and subsurface conditions, a site plan showing boring locations and other pertinent features, summary boring logs and cross sections, and laboratory test results.

10. Finalized the geotechnical report after receiving review comments from PB, King County, and other concerned agencies.

11. Provided project management for the geotechnical tasks and participate in three meetings with PB and King County.

Environmental assessment or evaluation regarding the presence or absence of wetlands or hazardous or toxic material in the soil, surface water, groundwater, or air, on or below or around this site were performed by others.

1.5 Task 4 – Test Pile Program

1.5.1 General

Drilled shafts are being considered as foundation support for the rehabilitation and replacement alternatives. However, the potential of site soils being contaminated presented concern about disposal of the spoil produced during drilled shaft installation. Driven piles could be used as an alternative foundation support. Vibrations caused by pile driving could impact existing facilities in the proximity of driving operations. Structures and improvements of particular concern are the existing bridge that should remain in operation if a new bridge is selected as the preferred alternative, Boeing buildings that house sensitive equipment, and underground utilities.

The effects of pile driving (vibration and settlement) were evaluated by completing a test pile program. A test pile, consisting of 24-inch-diameter, ½-inch wall thickness, steel pipe pile, was driven closed-ended. Instrumentation was installed and monitored during the pile driving to obtain information regarding driving-induced vibrations and settlements. In order to optimize the benefit from the test pile program, dynamic testing was also performed on the test pile to obtain information on pile drivability and estimated capacity. After testing was completed, the pile was cut off about 3 feet below existing ground surface, the pile excavation was then filled with lean mix concrete, and the excavated area backfilled with lean mix concrete and covered with an asphalt pavement patch. Excavated material was placed in drums and moved to a temporary storage area beneath the south approach of the existing bridge. The following sections provide a description about our recommended instrumentation for use with the test pile program.

1.5.2 Vibration and Settlement Monitoring

The following tasks were performed to monitor vibration and settlement during driving of the test pile:

1. Vibration monitoring was performed using portable seismographs equipped with internal and external geophones. The seismographs, Blastmate III from Instantel, Inc., have seismic monitoring ranges of up to 10 inches per second (ips), with triggering thresholds down to 0.01 ips. The microphones have decibel ranges from 100 to 142 decibels (dB), and triggering levels starting at 106 dB. During pile driving, one seismograph (and microphone) was placed approximately 50 feet from pile being driven, and the external geophone was located within approximately 100 feet from the pile being driven.

2. Settlement of the subsurface soils was monitored by using the Sondex system attached to an observation well casing in boring SB-1. This method employs steel rings and a magnetic probe to measure vertical settlements within a soil column.

3. Settlement of the ground surface was monitored by installing a horizontal profiler. The horizontal profiler consists of an inclinometer casing, a gravity sensitive probe, a portable readout unit, and a graduated electrical cable linking the probe to the readout unit. The inclinometer casing is a relatively flexible plastic pipe that moves with the soil mass in which it is installed. It has four grooves cut longitudinally into its inside at 90-degree intervals to serve as guides for the probe. The casing is installed horizontally in the ground with one set of the grooves oriented approximately vertical over the entire length of the casing. The probe is aligned in the vertical grooves and incrementally pulled through the casing to measure the angle of each segment of casing with respect to horizontal. The measurement provided a profile of ground settlement out away from pile driving location.

1.5.3 Dynamic Pile Tests

A Pile Driving Analyzer (PDA) was used to perform dynamic load tests during pile driving. With this test, accelerometers and strain transducers were attached to the top of the pile. The instruments measure pile top force and velocity under the dynamic load generated by the impacting ram. Based on the measured signals, the PDA performed calculations to estimate ultimate bearing capacity, pile integrity, hammer performance, and pile stresses during driving. Dynamic testing was performed during continuous driving from a depth of 50 feet below the ground surface to the end of driving, and also during restrike of the pile 24 hours later. In addition, the dynamic test data was used to perform an analysis with Case Pile Wave Analysis Program (CAPWAP). CAPWAP provides a more refined estimate of the pile capacity.

2.0 SITE AND PROJECT DESCRIPTION

2.1 Site and Existing Bridge Description

It should be noted all ground elevations discussed in this report are referenced to the North American Vertical Datum of 1988 (NAVD 88).

The South Park Bridge crosses the Duwamish Waterway near the southern limits of the City of Seattle. The project area is situated in the Duwamish Waterway floodplain, which is bounded by upland areas to the east and west. The ground surface is generally flat lying with elevations between approximately 10 and 20 feet (NAVD 88). Two small rock knobs located near the south end of the project corridor rise upward to approximate elevations of 60 and 100 feet. The lowest point in the project area is the bottom of the Duwamish Waterway at an approximate elevation of –20 feet.

The bridge crosses the Duwamish Waterway in a near north-south orientation. The bridge carries traffic from 16th Avenue S. on the north side to 14th Avenue S. at the southern bridge terminus. Large industrial facilities (Boeing Plant 2) lie to either side of the project corridor across the bridge north of the Duwamish Waterway. South of the bridge is the community of South Park, which has a mix of commercial, industrial, and residential property. Commercial retail and service businesses are located along 14th Avenue S.

The Duwamish Waterway is used for industrial, commercial, and recreational purposes. The bridge is near the upstream limit of heavy industrial uses along the Duwamish Waterway, but it is within the section of the navigation channel maintained by the U.S. Army Corps of Engineers. The existing maximum vertical clearance of the bridge when closed is limited to 32 feet at Mean High Water (MHW), with three to five openings per day on average to accommodate waterway traffic. The existing navigable horizontal clearances is approximately 118 feet at the water level (fender-to-fender), but narrows to 92 feet approximately 115 feet above the water between the open bascule leaves. The depth of the navigation channel is approximately 15 feet at Mean Lower Low-Water (MLLW).

The South Park Bridge was constructed between 1929 and 1931. The existing structure consists of a Scherzer rolling-lift double-leaf bascule movable span. Each side is flanked by 2 truss approach spans and 12 concrete slab approach spans. The overall length of the bridge is approximately 1,045 feet abutment-to-abutment and approximately 1,340 feet in entirety to the grade-match points. The double-leaf bascule movable span has a center-to-center distance between the front bearing points of approximately 190 feet. The roadway consists of four 9.5-foot lanes. The pavement is 38 feet wide with 6-foot-wide sidewalks on both sides.

Reinforced concrete piers founded on timber piling support the bascule span. Two large in-water piers support the counterweights, track supports, and racks for the rolling lift. The attached towers house the operating machinery, electrical equipment, and operator control room.

2.2 Current Bridge Conditions

In spite of substantial on-going maintenance and repairs, the South Park Bridge has suffered considerable deterioration over the past 70 years. In particular, the bascule piers are cracked and unstable resulting in the misalignment of the movable spans. Consequently, the center lock and glide tracks require on-going modifications and adjustments to allow the bridge to operate properly. Long-term, the stability of the entire bridge is at risk because the original piles supporting the north bascule pier were not driven into competent bearing soils, which has resulted in movement of the bridge piers over the decades. The condition of the bridge worsened significantly following the Nisqually Earthquake in February 2001, and it remains vulnerable to future seismic events. A 2002 bridge inspection conducted by King County resulted in an existing condition rating of 6.0 out of a possible score of 100 based on Federal Highway Administration criteria. Reportedly, this was among the lowest ratings given to any bridge structure in the State of Washington in 2002.

2.3 Proposed Bridge Alternatives

2.3.1 General

The project objective is to find the most feasible long-term solution to address the deteriorated condition and increasing seismic vulnerability of the South Park Bridge. As previously described, five alternatives have been identified for the project. The "no action" alternative assumes that the existing bridge would be closed, demolished, and removed at some time in the future. For the rehabilitation alternative, the existing bridge would remain in place. Its structural components would be reinforced or replaced and the equipment used to operate the bridge would be refurbished or replaced. The three replacement bridge alternatives would result in construction of a new bridge. The new bridge for all three replacement alternatives would follow the same horizontal alignment to be located approximately 80 feet, center-to-center, west (downstream) of the existing bridge. The proposed alignment and extent of the replacement alternatives are shown on Figure 2.

2.3.2 No Action Alternative

The "no action" alternative assumes that poor condition of the existing bridge structure would require it to be closed at some time in the future. Deterioration due to use could allow the bridge to continue to operate for the foreseeable future, but at some time in the future, the bridge would need to be closed. From environmental impact studies, the existing bridge is assumed to be closed permanently sometime before 2027 (Shannon & Wilson, Inc., 2003).

However, the bridge could be closed for other reasons than simply deteriorated condition. Another earthquake could cause an unexpected emergency closure of the bridge at any time. The on-going movement of the bridge foundations could eventually cause the moveable spans to become misaligned to the extent that repairs would be infeasible. Also, the cost of maintaining the bridge could become more than King County is willing to expend. Under any of these circumstances, the bridge would be closed.

2.3.3 Rehabilitation Alternative

For the rehabilitation alternative, the existing bridge would remain in place, the structure would be reinforced, and the equipment used to operate the bridge would be refurbished and/or replaced. The bridge would be closed to traffic during reconstruction. Studies have confirmed the existing bridge piers are gradually shifting because the foundation pilings were not originally driven to a sufficient depth and to a competent bearing layer. Although the initial goal was to rehabilitate the existing piers, structural analyses has determined that the existing bascule piers and truss approach span piers must be replaced in order to ensure the long-term (approximately 70 years) integrity of the bridge. If the bascule piers were reconstructed, the longevity of the rehabilitation alternative would be similar to the expected minimum life of a new bridge structure.

The new bascule piers are proposed to be approximately the same size, location, and historic character as the existing piers. To construct the new bascule piers, the bascule leaves and steel approach spans would need to be removed. These elements of the bridge structure would be taken to another site for repair, refurbishment, and/or painting before they are re-installed following the construction of the new piers. The concrete shafts or pilings supporting the foundations of the new piers would extend below the existing pilings to a depth beneath the soft alluvial soils and into firm glacial soils. The removal of the steel truss spans would also allow for replacement of the steel approach piers. The concrete approach spans and bridge abutments would be re-constructed and the bridge deck would be resurfaced. Like the existing bridge, there would be piers both on land and in the water. The first on-land piers would be only an estimated 20 feet from the south shoreline and the closest in-water piers would be approximately 20 feet from the shoreline. The piers on the north shoreline would extend through the existing Boeing dock. The conceptual engineering analysis also determined that the mechanical and electrical systems should be replaced and/or refurbished.

2.3.4 Bascule Bridge Alternative

The bascule bridge alternative would result in the construction of a new movable bridge immediately downriver of the existing bridge. The bridge length would be approximately 1,180 feet from abutment to abutment, not including roadway approaches. Road improvements would extend from a point just south of S. Sullivan Street on the south side of the waterway and north to a point opposite the northeast corner of Boeing Building 2-15. The walls of the bridge abutments would be approximately 200 feet from the shoreline, or approximately the same setback as the existing bridge. With fewer piers than the existing bridge, the first on-land piers of this alternative would be approximately 55 feet from the south shoreline at the shortest distance and the closest in-water piers would be approximately 65 feet from the shoreline. On the north shoreline, the closest in-water piers would be approximately 95 feet from the shoreline and the closest on-land piers would be approximately 30 feet from the shoreline.

Similar to the existing bascule bridge, this bridge profile would be approximately 35 feet above the Duwamish Waterway. The mid-section span would be comprised of two movable leaves that could be raised to open the bridge. The navigation channel would be approximately 125 feet in width (slightly greater than the existing waterway channel). This two-leaf bascule bridge would not impose limitations to the height of waterway users passing the bridge.

2.3.5 Mid-Level Fixed-Span Bridge Alternative

The mid-level fixed-span bridge alternative would result in the construction of a non-movable bridge. The bridge length would be approximately 1,660 feet abutment-to-abutment, not including roadway approaches. The walls of the abutments would be approximately 550 feet from the shoreline of the Duwamish Waterway, or 250 feet further setback than the existing bridge. The closest on-land piers would be approximately 85 feet from the south shoreline and the closest in-water piers would be approximately 100 feet from the south shoreline. On the north shoreline, the closest in-water piers would be approximately 130 feet from the shoreline and the closest on-land piers would be approximately 65 feet from the shoreline. Road improvements would extend slightly south of S. Cloverdale Street and north to a point approximately 320 feet south of East Marginal Way S.

The mid-point of the bridge profile across the Duwamish Waterway would be approximately 65 feet above MHW of the Duwamish Waterway. The horizontal clearance would be approximately 125 feet, or slightly greater than the existing clearance. The vertical clearance, however, would restrict use of some waterway traffic, including some tugs and barges. Most vessels that currently pass the existing bridge would continue to be able to use the navigation channel.

2.3.6 High-Level Fixed-Span Bridge Alternative

The high-level fixed-span bridge alternative is a non-movable bridge. The bridge length would be approximately 2,338 feet abutment-to-abutment, not including roadway approaches. The walls of the abutments would be approximately 900 feet from the shoreline of the Duwamish Waterway, or 650 feet further set back than the existing bridge. The on-land and in-water piers of this alternative are approximately in the same location as proposed for the mid-level fixed-span bridge alternative. Road improvements would extend from S. Trenton Street and continue north to East Marginal Way S. This alternative would require modification of the 16th Avenue S./East Marginal Way S. intersection and of the existing railroad track crossing immediately south of this intersection.

The bridge design would allow for a 100-foot minimum vertical clearance above the MHW of the Duwamish Waterway. The horizontal waterway clearance for the navigation channel would be approximately 125 feet, which is slightly greater than the existing 118-foot clearance (fender-to-fender). The bridge's vertical clearance would not be expected to limit the height of boats and barges currently passing the bridge. However, vessels larger than those currently using the navigation channel might not be able to pass the bridge in the future.

2.4 Previous Studies

In recent history, over 20 engineering studies have been prepared on the South Park Bridge. Starting in 1987, when the bridge was 56 years old, King County contracted for the preparation of a general engineering investigation report to assess the condition of the bridge. In 1991 and 1993, additional studies were completed including a geotechnical study, foundation design report, and a life-cycle cost analysis. This information led King County to undertake a series of studies in 1994 addressing liquefaction risks as well as the condition of the concrete, substructures, approach span joints, and loading rating. In addition, a study was conducted to evaluate potential replacement alternatives for the bridge and another study investigated community issues related to the bridge. Since 1994, King County has recognized that the bridge required either rehabilitation or replacement and has continued to investigate the condition and vulnerabilities of the bridge in an effort to evaluate these options.

Two key engineering studies were conducted that helped to frame the current pursuit to evaluate potential alternatives to rehabilitate or replace the South Park Bridge. A 1994 Sverdrup study evaluated potential design options and a 1999 Entranco study researched and presented the likely steps required to conduct the necessary environmental review of the project alternatives and to complete necessary permitting.

From the previous studies, geotechnical reports for the South Park Bridge either reviewed or incorporated into this report include the following:

- Shannon & Wilson, Inc., 1991, "Geotechnical Report, 16th Avenue South Bridge, Seattle, Washington," July.

- Shannon & Wilson, Inc., 1994, "Liquefaction Evaluation, 16th Avenue South Bridge Approaches, Seattle, Washington," August.

- PanGeo, 2001, "Geotechnical Report, South Park Bridge Seismic Evaluation, King County, Washington," August.
- Shannon & Wilson, Inc., 2002, "Draft Geotechnical Report, South Park Bridge Project, King County, Washington," October.
- Shannon & Wilson, Inc., 2003, "Draft Technical Report, South Park Bridge Project, Geology and Soils," June.

3.0 FIELD EXPLORATIONS

3.1 General

The current field exploration program for the project consisted of six on-land exploratory borings, two over-water exploratory borings, shear wave velocity testing in two borings, pile load testing and monitoring, and assistance with environmental exploration performed by Wilbur. Our field exploration program generally occurred intermittently between June 24 and August 25, 2003. Summaries of our program are presented in the following sections with more detailed information presented in the appendices. Supplemental information was obtained from previous explorations by Shannon & Wilson, Inc. and as provided to us from King County.

The locations of the current exploratory borings, test pile, and selected previous borings are shown at the approximate locations in Figure 3. The locations of the current exploratory borings were determined after drilling by a survey crew working for Lin & Associates.

3.2 On-Land Subsurface Explorations

Six exploratory borings were drilled on land from June 24 through July 16, 2003. Borings SB-1 through SB-3 are located along the north bridge approach and borings SB-6 through SB-8 are located along the south bridge approach. All borings except SB-8 are located along the proposed replacement bridge alignment west of the existing bridge. Boring SB-8 is located along the east shoulder of 14th Avenue S.

The borings were drilled using hollow-stem augers in the upper 20 feet and mud rotary methods below from a truck-mounted drill rig. The depths of the borings ranged from 90.3 to 130.7 feet, for a total footage of about 645 feet. General descriptions of the field explorations and the logs of the borings are presented in Appendix A, Subsurface Explorations.

3.3 Over-Water Subsurface Explorations

Two exploratory borings were drilled over water from July 22 through July 24, 2003. Boring SB-4 is located near the proposed north pier of the replacement bridge alternatives. Boring SB-5 is located near the proposed south pier of the replacement bridge alternatives. The borings were drilled from a barge in the waterway during favorable tidal conditions.

The borings were drilled using mud rotary methods from a truck-mounted drill rig placed on a barge. The depths of borings SB-4 and SB-5 were 126.0 and 101.5 feet, respectively, for a total footage of about 227.5 feet. General descriptions of the field explorations and the logs of the borings are presented in Appendix A, Subsurface Explorations.

3.4 Previous Explorations

Numerous subsurface explorations have been performed in the vicinity of the project alignment north of the bridge, primarily borings performed for the Boeing Company's Plant 2. However, relatively few subsurface explorations have been performed south of the bridge along 14th Avenue S.

We have incorporated selected borings from these previous explorations into this report. The approximate boring locations are shown on Figure 3. The selected borings were from previous Shannon & Wilson, Inc. and Dames & Moore reports. Logs of the borings and related laboratory testing are presented in Appendix B, Previous Subsurface Explorations.

3.5 Shear Wave Velocity Testing

Shear wave velocity testing was performed at borings SB-2 and SB-6 on August 22 and 25, 2003. The shear wave velocity measurements define dynamic soil properties that are used to estimate site response and liquefaction during the design earthquake. The measurements were performed using the "down hole" method where detectors are positioned in the cased borings at regular intervals. The detectors measure compression and shear waves generated in the soil from an energy source located at the surface. Results of the shear wave velocity testing are presented in Appendix C, Downhole Seismic Test.

3.6 Environmental Sampling, Exploration, and Testing

Wilbur is the environmental engineer for the project. At their direction, soil samples were selected for chemical testing by Wilbur at various intervals from the geotechnical borings. Additional drilling for monitoring wells was also performed at Wilbur's direction. The results of the chemical testing, monitoring wells, and environmental recommendations were prepared by Wilbur as a separate report. Potential impacts of environmental contamination, if present, are referred to in this report (e.g., excavations for foundations). The reader is directed to Wilbur's report on the project for information on the potential and extent of soil contamination.

4.0 LABORATORY TESTING

The soil samples obtained from the geotechnical borings were classified, collected in jars and returned to our office for laboratory testing. The laboratory testing program was directed toward determining the index properties of the soils encountered at the site and included visual classification, water content determination, grain-size analysis, and Atterberg limits.

The tests were performed in the Shannon & Wilson, Inc. laboratory in Seattle by an experienced laboratory technician. A more detailed description of the laboratory test methods and summaries of the test results are presented in Appendix D, Geotechnical Laboratory Testing Procedures and Results.

5.0 TEST PILE

A 24-inch-diameter steel pipe pile (test pile) with a wall thickness of ½-inch was driven on July 22, 2003. The pile is located at Station 32+40 approximately 4 feet from boring SB-1. The test pile consists of two 40-foot sections driven to a depth of 71.8 feet using an open-ended diesel hammer having a maximum energy rating of 75,900 foot-pounds. The test pile was driven closed-ended with a 2-inch-thick plate welded to the pile tip. Approximately 24 hours after the initial pile driving, the test pile was re-driven to a depth of about 74 feet.

The purpose of the test pile was to estimate the drivability and vertical capacity of the pile, monitor vibrations during pile driving, and monitor ground settlement adjacent to the test pile. Instrumentation was installed to monitor the test pile for capacity, vibrations, vertical settlement, and horizontal extent of the ground settlement. Pile capacity measurements were performed using dynamic testing to estimate skin resistance and end bearing of the test pile during the initial driving and re-strike. Dynamic measurements were performed by Robert Miner Dynamic Testing, Inc. of Manchester, Washington, and their report is presented in Appendix E: Test Pile Capacity and Instrumentation. Detailed information of the test pile instrumentation program is also presented in Appendix E. A brief summary of the instrumentation and results are presented in the following paragraphs.

During pile driving, vibrations were monitored using a Blastmate III seismograph with geophones. The geophones record peak particle velocities in ips and were arrayed at distances of 50 and 100 feet from the pile location. The maximum velocities recorded at these distances were 0.5 ips and 0.25 ips, respectively. During restrike of the pile, a geophone was placed at about 20 feet from the test pile and recorded velocities between 0.35 and 0.4 ips (with the pile tip about 71 feet below the ground surface).

A Sondex casing was installed in boring SB-1 to monitor vertical displacements of the surrounding soil induced by driving of the adjacent test pile. The Sondex system consists of a plastic casing containing steel rings at discrete intervals. A monitoring probe capable of accurately sensing the steel rings is lowered in the hole to record the intervals in the casing. After pile driving, a vertical displacement of 1 inch was observed in the upper 10 feet of the casing. The vertical displacement decreases with increasing depth to about 55 feet, where negligible displacement was recorded.

The horizontal extent of ground settlements caused by pile driving was monitored to within 6 feet of the test pile. A 100-foot-long casing with internal grooves was secured to the ground surface and oriented radially away from the test pile. A gravity-sensitive probe with wheels running in the grooves was pulled through the casing. The probe measured vertical displacements at 2-foot intervals throughout the casing. Measurements were taken at various intervals before and after test pile driving. The maximum displacement of 0.12 inches occurred closest to the test pile after driving, with negligible displacements observed beyond 12 feet from the test pile.

6.0 HISTORICAL DEVELOPMENT AND GROUND MODIFICATION

The Duwamish Waterway is a channel dredged in the 1910s to straighten the natural meandering course of the Duwamish River (Phelps, 1973). Prior to straightening, the channel of the Duwamish River ran around the north end of the project area. It also may have meandered across most of the flood plain in the recent geologic past. Prior to its filling in the 1910s, an abandoned, former meander channel of the Duwamish existed just south of the existing South Park Bridge, between Dallas Avenue S and S Orr Street. The former channel was likely filled largely with spoils from the dredging of the waterway. Prior to filling, the abandoned channel was a marsh, with much of the original channel depth already filled by natural processes, which includes organic, fine-grained sediments, and wood debris.

7.0 GEOLOGY

The Duwamish Waterway valley is a broad, glacially carved trough bounded by upland areas to the east and west. The valley and upland areas are underlain by glacial and non-glacial sediments deposited during or subsequent to six or more Pleistocene glacial episodes. Underlying these sediments, bedrock exists at variable depths beneath the project area and is exposed at the ground surface at the two rock knobs located near the south end of the project. The bedrock is part of the Blakely Formation of the Oligocene age (35 to 45 million years old) and consists primarily of silty sandstone with some conglomerate.

The most recent glaciation in the Seattle area, termed the Vashon Stade of the Fraser glaciation, ended approximately 13,500 years ago. During the Vashon Stade, the ground was topographically depressed from the weight of ice up to 3,000 feet thick. During the height of the glaciation, sea level was as much as 300 feet lower than the present-day sea level because so much of the earth's water was stored as glacial ice. After retreat of the ice from the area, the ground surface rebounded relatively quickly and was near its present position about 9,000 to 10,000 years ago. As the large ice sheets melted, the sea level rose slowly until it reached the present-day sea level approximately 5,000 years ago. As sea level rose, marine waters filled the trough. As the Duwamish Waterway delta advanced from the south, a thick sequence of estuarine (marine delta) and alluvial (river-deposited) sediments accumulated in the trough.

Glacial deposits underlying the project corridor consist of glaciomarine drift. Glaciomarine drift was deposited in lakes or marine water by a combination of the slow settling of clay and silt particles in quiet waters, and the episodic and variable deposition of clastic debris from melting icebergs. The deposit generally has been overridden by glacial ice and is typically very dense or hard. The upper portions of the glacial deposit may be less dense or hard and may have been reworked since deposition or represent a thin layer of recessional material deposited during recession of the glacial ice. The soils generally consist of poorly graded granular material within a clayey matrix.

The youngest deposits identified within the project area are fill, marsh, alluvium, and estuarine deposits. These recent, non-glacial soils were all deposited during the last 10,000 years. Estuarine deposits consist of fine-grained soils deposited in delta mudflats and floodplains where tidal marine waters are mixed with fresh water from rivers and streams. These deposits consist of very soft to stiff or very loose to medium dense, silty clay to clayey, silty sand. This unit commonly contains scattered organic debris. The estuarine deposits may interfinger with coarser-grained alluvial deposits.

Alluvium is soil deposited by streams and rivers and includes finer-grained overbank deposits. These soils range from very loose to dense and are composed of sand, silty sand, and gravelly sand. The relative density generally increases with depth. A significant thickness of alluvium lies beneath the corridor.

Marsh deposits are fine-grained cohesive soils deposited in poorly drained areas across the historic floodplain. The marsh deposits identified along the project corridor may also include buried topsoil. The marsh deposits consist of very loose to loose silt and sandy silt to very soft to soft, silty clay to clayey silt. These soils commonly contain abundant organic material and may contain peaty material and wood debris.

Fill is soil placed by humans in historic times. Fill can have widely variable properties, depending on the material used as fill and whether the fill was placed in an engineered or non-engineered fashion. The fill within the project area consists of silty sand to gravelly sand with some sandy gravel. Most of the fill is likely to be non-engineered, having been placed hydraulically during dredging of the waterway.

8.0 SUBSURFACE CONDITIONS

8.1 General

The approximate distribution of geologic deposits and subsurface conditions beneath the project area is presented on the Generalized Subsurface Profile A-A' shown on Figure 4.

The project corridor is generally underlain by relatively soft or loose soils that extend to considerable depths in most places. More competent, glacial soils and bedrock underlie these less competent soils. The depth to these more competent soils and rocks varies considerably along the corridor and in the vicinity of the project area. The subsurface conditions at the south end of the project area differ significantly from those along the rest of the corridor, in that competent soils and bedrock exist at relatively shallow depths along 14th Avenue S. south of S. Donovan Street. Bedrock is exposed at two rock knobs located to either side of 14th Avenue S. about 1,200 feet south of the existing bridge.

A brief discussion of subsurface conditions encountered beneath the south approach, bridge crossing, and north approach are discussed in the following sections.

8.2 South Approach

Much, if not almost all, of the area in the vicinity of the project is underlain by fill. Based on the borings, the fill at the south approach is typically about 3 to 12 feet thick. The fill thickness generally increases towards the waterway. Most of the fill was likely placed hydraulically during dredging of the waterway. Therefore, the bulk of the fill is likely to consist of fine to medium sand with variable amounts of silt and layers of silt, sandy silt, and clayey silt to silty clay. Scattered to abundant wood debris, cobbles and boulders, and debris from human activity are also likely to exist. Historical records previously discussed indicate that the South Park area lowlands were filled to about elevation 12 feet (NAVD 88), which is approximately the same as some lower ground west of the bridge along the Duwamish, while 14th Avenue S. was filled to its present elevation of 16 to 17 feet (NAVD 88).

The fill directly overlies fine-grained organic deposits consisting of very soft to medium stiff clayey and/or sandy silt with variable organic material. This upper alluvial deposit is about 6 to 12 feet thick. These deposits may include topsoil because the alluvial deposits were at the ground surface prior to fill placement.

The upper fine-grained alluvial deposits are underlain by a relatively thick deposit of coarse-grained alluvium, which is as thick as 40 feet and extends down to elevations between –15 to -45 feet. Based on the borings, the depth of the alluvium increases towards the waterway. The alluvium generally consists of very loose to dense sand, silty sand, and gravelly sand. The relative density of the granular material generally increases with depth. The alluvium also contains seams and layers of finer-grained material, such as silt, clayey silt to silty clay, and organic deposits. These deposits may be very soft to soft, even at considerable depths.

The alluvium is underlain by a thin layer of cohesive estuarine deposits. The estuarine deposits consist of very soft, clayey, sandy silt to medium dense, slightly clayey sand with some shells and gravel. The estuarine deposits were encountered in the borings at a depth ranging from 35 to 65 feet deep. The thickness of this material is typically less than 5 feet in the south approach area.

The normally consolidated, non-glacial soils described above are underlain by more competent, glacial material that has likely been glacially overridden. The glacial soils consist of very stiff to hard, clayey silt to silty clay with some sand and gravel (glaciomarine drift). This glacial deposit is at least 50 feet thick, but recent borings for the south approach did not penetrate the full thickness of these deposits.

Bedrock was encountered at a depth of about 18 feet in a previous boring located between the two rock knobs at the south end of the alignment (south of boring SB-8). This boring is located on 14th Avenue S. near the intersection of S. Concord Street approximately 1,200 feet south of the existing bridge. Bedrock at relatively shallow depths may also exist along 14th Avenue S. between S. Donovan Street and S. Henderson Street, but was not encountered in the relatively shallow borings located along that portion of the project. The rock encountered in the boring and observed in exposures at the rock knob east of 14th Avenue S. consists of fine-grained, silty sandstone. Some conglomerate has been encountered in borings located away from the alignment. The sandstone likely has unconfined compressive strengths on the order of 1,000 to 7,000 pounds per square inch, based on previous Shannon & Wilson, Inc. projects with similar rock (Shannon & Wilson, Inc., 1999).

8.3 Duwamish Waterway

Subsurface conditions below the waterway are interpreted based on the two over-water borings for the project (borings SB-4 and SB-5) and previous explorations at the site (borings B-1 and B-2). We observed fine-grained organic deposits below the mudline consisting of organic silt, sandy silt to clayey silt with variable organic material on the south side of the waterway channel. These marsh deposits are typically very loose to loose or very soft to soft and were between 15 and 20 feet thick.

The marsh deposits are underlain by alluvium, which is about 35 to 40 feet thick and extends down to elevations of -50 to -70 feet. Based on the borings, the depth of the alluvium is greatest at and just to the north of the waterway. The alluvium generally consists of very loose to dense sand, silty sand, and gravelly sand. The relative density of the granular material generally increases with depth. The alluvium also contains seams and layers of finer-grained material, such as silt, clayey silt to silty clay, and organic deposits. These fine-grained deposits may be very soft to soft even at considerable depths.

The alluvium is underlain by a thin fine-grained estuarine deposit also observed at the south approach. Underlying the thin fine-grained deposit is a thicker estuarine layer consisting of silty gravelly sand and sandy gravel. The thickness of this estuarine layer ranged from about 5 feet at the south approach to about 25 feet on the north side of the waterway. The nature and thickness of this material may differ elsewhere along the corridor.

The normally consolidated, non-glacial soils discussed above are underlain by more competent glacial deposits consisting of glaciomarine drift. This glacial soil consists of very stiff to hard, clayey silt to silty clay with some sand and gravel. The upper portion of some of these deposits, however, is softer or less dense than typical (possibly weathered), and may not provide adequate bearing. This situation exists for the existing north bascule pier where the piles supporting the pier do not quite reach competent glacial deposits with adequate bearing.

8.4 North Approach

Subsurface conditions for the north approach are similar to the south approach with the notable exceptions of thicker alluvial and glacial deposits and the absence of bedrock in the deeper borings and outcrops in the area. Fill was observed in the upper 10 to 15 feet of soil below the ground surface.

Underlying the fill, we observed alluvial deposits consisting of a thin layer (less than 5 feet thick) of soft to medium stiff clayey silt underlain by loose to medium dense silty sand. The relative density of the granular material generally increases with depth. The alluvium also contains seams and layers of finer-grained material, such as silt, clayey silt to silty clay, and organic deposits. The alluvium is approximately 70 feet thick near the waterway decreasing to about 45 feet thick towards the north (boring SB-1).

The alluvial deposits are underlain by estuarine deposits consisting of thin layers of very soft clayey silt over dense to very dense sandy gravel. The thickness of the estuarine deposits is about 20 feet near the waterway to about 5 feet towards the north (boring SB-1).

All recent borings in the north approach area terminate in the glaciomarine drift. As observed in the other areas, the glaciomarine drift consists of very stiff sandy and clayey silt to hard silty clay and clayey silt. The elevations of the top of this bearing layer range from about elevation -90 feet near the waterway to about –50 feet to the north (boring SB-1).

8.5 Groundwater

Groundwater levels along most of the project corridor are within 10 to 12 feet of the ground surface. The groundwater is tidally influenced and fluctuates as much as 11 feet near the waterway. The magnitude of fluctuations decreases away from the waterway.

A previous Shannon & Wilson, Inc. study of the Boeing Plant 2 site (north of the existing bridge) indicated that the river level in this portion of the Duwamish Waterway is typically about 1 foot above the tide level in Puget Sound at Seattle. The study showed that the tidally related fluctuations at groundwater levels near the South Park Bridge are as much as 11 feet. The magnitudes of the fluctuations decrease away from the waterway and are as little as 4 feet where 16th Avenue S. intersects E. Marginal Way S. Similar magnitudes of groundwater fluctuations and a decrease to the south away from the waterway are expected on the south side of the project area.

In addition to the observed tidal fluctuations, groundwater levels can fluctuate in response to seasonal variations, recent rainfall, and other factors. The groundwater levels observed during drilling should be considered approximate.

9.0 EARTHQUAKE ENGINEERING STUDIES

9.1 General

The project area is located in a moderately active tectonic province that has been subjected to numerous earthquakes of low to moderate strength and occasionally to strong shocks during the brief 165-year record history in the Pacific Northwest. Seismicity in the region is attributed primarily to the interaction between the Pacific, Juan de Fuca, and North American plates. The convergence of the Juan de Fuca and North American plates results not only in east-west compressive strain (Lisowski, 1993), but also in dextral shear (right-hand), clockwise rotation, and north-south compression of the crustal blocks that form the leading edge of the North American Plate. It is estimated that the compression rate for these blocks is about 0.03 to 0.04 inch per year, and much of the compression may be occurring within the more fractured, northern Washington block that underlies the Puget Lowland.

From the interaction of these plates and their resulting tectonic stresses, seismologists have identified three different earthquake mechanisms in the region: (1) an interplate or subduction zone, (2) a deep intraslab zone in the subducted Juan de Fuca plate, and (3) a shallow crustal zone. Historical earthquake have been correlated with the latter two zones. A brief description of each zone and the relative hazard posed by earthquakes generated from these sources are given in the following paragraphs.

Research is presently underway regarding large magnitude subduction zone earthquake activity along the Washington and Oregon coasts. Subduction zone earthquakes occur where the North American plate first overrides the Juan de Fuca plate. This initial contact area is called the Cascadia Subduction Zone (CSZ) that runs offshore from Northern California to Vancouver Island. An illustration of the CSZ is presented in Figure 5. While there are no local historic records of CSZ earthquakes, geologic evidence suggests that large earthquakes (e.g., magnitude 8.5 to 9) have occurred in this zone as recently as 300 years ago.

Earthquakes generated from the intraslab zone are likely caused by deformation and breakup of the subducting Juan de Fuca plate beneath the North American plate. The most recent large intraslab event is the magnitude (M_W) 6.8 Nisqually earthquake, which occurred on February 28, 2001, near Olympia. This earthquake (located 35 miles from Seattle and deep below the surface) caused significant damage to the South Park Bridge. Since the earthquake, operation of the moveable span has been less reliable, requiring the bridge to be closed for repairs intermittently for several days. The continuing periodic closure of the bridge for repairs has heightened the awareness of the need for rehabilitation or replacement of the existing bridge. Other recent, large intraslab events include the magnitude (Ms) 7.1 Olympia earthquake of April 13, 1949, and the magnitude (mb) 6.5 Seattle-Tacoma earthquake of April 29, 1965. The intraslab earthquakes appear to occur with the greatest frequency. However, the seismic energy released from the deep, subducting plate (greater than 30 miles) is attenuated at the surface, reducing the severity of the ground shaking in most cases.

Shallow, crustal earthquakes are currently undergoing extensive studies within the region. These earthquakes are usually generated from surface or near-surface faults developed by the tectonic stresses, most of which have only recently been identified. The Seattle Fault, identified between Bainbridge Island and east of Lake Sammamish south of downtown Seattle, is probably the most well known surface fault in the area. The project area is located within the Seattle Fault Zone, which consists of four or more east-west-trending faults (fault splays) that coalesce at a depth to a master, south-dipping fault. This thrust fault zone is approximately 4 to 6 kilometers (km, 2.5 to 3.7 miles) wide with potential splays mapped both to the north and south of the project as shown on Figure 6. In most areas of Seattle and vicinity, fault splay locations have been extrapolated and are not precisely known. A southern splay of the Seattle Fault zone is shown approximately one-half mile south of the existing bridge (Figure 6) on the recently published Geologic Map of Washington – Northwest Quadrant (Dragovich et al., 2002).

Earthquake-induced geologic hazards that may affect any given site include liquefaction and related effects, landsliding, soft-soil ground motion amplification, and surface fault rupture. These earthquake-induced geologic hazards are discussed in the following section.

9.2 Site-Specific Ground Response Analyses for Seismic Design

We understand that the South Park Bridge will be designed in accordance with the 2002 American Association of State Highway and Transportation Officials (AASHTO) Standard Specifications for Highway Bridges. The code prescribes ground motions with a return period of about 475-years (10 percent probability of exceedance in 50 years); however, the code does not provide design response spectra for potentially liquefiable sites. Because of the potential for liquefaction, site-specific ground response analyses were undertaken to develop design response spectra for seismic design.

The site-specific ground response analyses were performed according to the following steps:

1. Develop soil profiles for site response analyses including characterization of shear wave velocity, dynamic soil properties, soil layer thickness, and unit weights.
2. Develop the rock motion design response spectrum using AASHTO (2002) for Soil Profile Type I.
3. Select rock input motions from previous earthquakes for spectral matching to the AASHTO (2002) Soil Profile Type I spectrum.
4. Calculate the free-field surface response using the equivalent-linear program ProShake (EduPro Civil Systems, 1999).
5. Calculate the average soil surface spectrum from the results of (4). Compare with motions recorded from historical earthquakes at nearby sites.
6. Develop a recommended spectrum based on (5).

Details of the analyses are provided in the following sections.

9.2.1 Soil Profile and Dynamic Soil Properties

The subsurface profile used in the analyses was developed primarily from the soil types encountered and downhole shear wave velocity (V_S) measurements made in borings SB-2 and SB-6. The V_S measurements were performed by Geo Recon International, and their report is presented in Appendix C.

The shear wave velocity was measured to a depth of about 109 feet below the ground surface in both borings. Figure 7 shows the measured V_S plotted with depth. Figure 7 also shows V_S measurements in similar geologic units in the Seattle area. Since the borings did not terminate in rock or in rock-like shear wave velocities (2,500 feet/second or greater), the shear wave velocity below the depth of borings SB-2 and SB-6 was estimated based on nearby existing deep borings and V_S measurements in glacial soils from major projects in the Puget Sound region. These projects included the Sound Transit Central Link Light Rail Duwamish Crossing (Shannon & Wilson, Inc., 1999), the King County International Airport Air Traffic Control Tower (Shannon & Wilson, Inc., 2001), and the West Seattle Freeway Bridge (Shannon & Wilson, Inc., 1980). These nearby existing deep borings also provided us with an estimate of the variability in V_S that may be expected at the South Park Bridge site.

The depth of the profiles was extended to rock-like shear wave velocities of 2,500 feet/second. From a bedrock map by Yount et al., 1985, the depth to bedrock at the site is between 164 ft (50m) and 328 ft (100m).

The profiles developed for our analyses are shown in Figures 8a and 8b for borings SB-2 and SB-6, respectively. Although the V_S measured in the two borings are similar, the presence of higher plasticity clay in boring SB-6 required a different soil model for each boring. Each baseline model for V_S was then varied by plus and minus 10 to 20 percent to account for variability in soil properties and geophysical measurements. The V_S of the alluvial soils was varied by 20 percent based on our experience and on the range of V_S measured at other nearby sites. The V_S of the glacial soils was varied by 10 percent since the use of 20 percent of high values of V_S resulted in a larger range of V_S than would be expected at the site. The table below summarizes the characteristics of the six profiles.

Profile Name	V_S Profile Description	Profile Depth (ft)
SB2	Based on measurements in boring SB-2 and middle range of glacial soils in Puget Sound region	220
SB2plus	SB2 times 1.2 in alluvial soils and times 1.1 in glacial soils	180
SB2minus	SB2 divided by 1.2 in alluvial soils and divided by 1.1 in glacial soils	220
SB6	Based on measurements in boring SB-6 and middle range of glacial soils in Puget Sound region	220
SB6plus	SB6 times 1.2 in alluvial soils and times 1.1 in glacial soils	180
SB6minus	SB6 divided by 1.2 in alluvial soils and divided by 1.1 in glacial soils	220

The strain-dependent dynamic soil properties were modeled using published curves including those for sands by the Electric Power Research Institute (EPRI) (1993), clays by Vucetic and Dobry (1991), gravel by Rollins et al. (1998), and rock by EPRI (1993). The appropriate curves for each soil type in the profile were chosen based on soil description, depth, and/or plasticity index (shown on the boring logs and laboratory test results in Appendices A and D, respectively).

9.2.2 Rock Design Spectrum

The design level prescribed in the 2002 AASHTO Standard Specifications is based on ground motions with a 475-year return period. The corresponding design spectrum for rock (Soil Profile Type I) is provided in the code. For comparison, the rock Uniform Hazard Spectrum (UHS) with directivity effects from the 1996/2003 U.S. Geological Survey (USGS) Probabilistic Seismic Hazard Analysis (PSHA) and Somerville et al. (1997) was calculated for ground motions with a 475-year return period. The spectrum for Site Type B (rock) prescribed by the 1997 Uniform Building Code (UBC) was also calculated. The spectra are plotted in Figure 9, and they show that the 2002 AASHTO spectrum for Soil Profile Type I has a higher peak and decreases at a slower rate than the USGS UHS or the UBC spectra.

It should be noted that the 2002 AASHTO spectrum for Soil Profile Type I was modified as shown in Figure 9 for use as a target rock design spectrum for our site response analyses. The modification involved anchoring the peak ground acceleration (PGA) at A = 0.33 and increasing from PGA to 2.5A at a period of 0.15 seconds; the main body of the 2002 AASHTO code shows a spectral acceleration of 2.5 at a period of zero seconds. The modification is based on the rock spectral shape shown in the 1998 Commentary of the 2002 AASHTO Standard Specifications. The resulting spectral shape is more realistic for the purposes of spectral matching of ground motions.

9.2.3 Rock Input Motions

The time histories used for the input motions of the site response analyses were selected after evaluating the deaggregation results of the PSHA performed by the USGS and PSHAs performed recently by Shannon & Wilson, Inc. for local projects. The deaggregation results provide earthquake magnitudes and distances that have the most significant influence on ground motion hazard for a particular return period and structure period.

For the 475-year return period ground motions, the deaggregation results indicate that shallow crustal faults and intraslab events (e.g., Nisqually) are the primary contributors to earthquake hazard at the site at short periods. At longer periods, subduction events on the CSZ also become significant. We understand that the period of the proposed bridge structure is currently unknown and may range from 0.5 to 5 seconds (personal communication, PBQD, 2003). Approximate characteristic magnitudes and distances are 7.0 and 5 km (3.1 miles), respectively, for shallow crustal faults; 6.5 and 50 km (31.1 miles), respectively for intraslab events; and 8.0 and 120 km (74.6 miles), respectively for subduction events. We then searched publicly available ground motion databases for previously recorded earthquake motions with characteristics similar to those identified in the deaggregation.

Four initial motions from previous earthquakes were selected and spectrally matched to the target rock spectrum shown in Figure 10. The program RSPMATCH (Abrahamson, 1997) and BASECOR (Abrahamson, 1994) were used to modify the motions so that their response spectra matched that of the target rock spectrum. The table below summarizes the characteristics of the initial motions.

Earthquake	**Magnitude**	**Station Name**	**Original PGA (g)**	**Source Type**	**Closest Distance (km)**
1992 Landers, California	M_w=7.3	Lucerne (275 comp)	0.72	Shallow crustal	1
1999 Chi-Chi, Taiwan	M_w=7.6	TCU 089 (west comp)	0.33	Shallow crustal	8
2001 Nisqually, Washington	M_w= 6.8	Seward Park (east comp)	0.31	CSZ intraslab	[76] hypocentral
March, 1985 Valparaiso, Chile	M_w=7.8	Valparaiso UFSM (70 comp)	0.18	Subduction zone interplate	[129] hypocentral

Figures 11 through 14 show the acceleration, velocity and displacement time histories before and after spectral matching. Figure 10 shows the spectra of the four motions before and after spectral matching compared to the target rock spectrum (AASHTO Rock Spectrum-Soil Profile Type I).

9.2.4 Results

Using the six soil profiles and the four rock input motions, we used the program ProShake (EduPro Civil Systems, 1999) to calculate the horizontal free-field surface response. The results of the analyses are presented in Figures 15a through 15c and 16. Figure 15a shows that the surface response is highly dependent upon the rock input motion. Figure 15b shows the difference response at SB-2 and SB-6. Figure 15c shows the variation due to use of the best estimate, lower bound, and upper bound V_S profiles for the two borings. As expected, the results in Figure 15c show larger response at short periods when the soil profile is stiffer and larger response at longer periods when the soil profile is softer.

In Figure 16, we compare the results of our site-specific ground response study to the spectrum recommended by AASHTO (2002) for Soil Profile Type III and Soil Profile Type IV. The response spectra of motions recorded at Boeing Field and Harbor Island during the February 2001 Nisqually Earthquake are also presented on Figure 16. Our recommended design response spectrum is also shown on Figure 16 and is based on an envelope of (1) the historical Nisqually earthquake data, (2) the results from the best estimate Vs profile shown as purple lines in Figure 15c, and (3) the mean plus one standard deviation of all of the calculated results.

Our recommended design spectrum shown in Figure 16 assumes the site will liquefy during the design earthquake. As discussed in Section 9.4 of this report, we recommend measures to mitigate the liquefaction potential at the site including Earthquake Drains and compaction grouting. The mitigation measures presented in this subsequent section, if selected, will likely improve the treated soil to at least AASHTO Soil Profile Type IV. Depending on the mitigation method selected, area of improvement, and strength of the improved soil after installation, it may be possible to achieve a Soil Profile Type III design spectrum. This is discussed further in Section 9.4.

9.3 Earthquake-induced Geologic Hazards

Earthquake-induced geologic hazards include landslides, fault rupture, settlement, and liquefaction and its associated effects (loss of shear strength, bearing capacity failure, loss of lateral support, ground oscillation, slumping, and lateral spreading). The principal hazards at the site include liquefaction and its associated effects and to a much lesser extent, fault rupture. The following provides a brief discussion of these hazards.

9.3.1 Landslides

In our opinion, earthquake induced landslide hazards are negligible within the project area due to the relatively flat terrain. Landslides along the riverbank associated with liquefaction and lateral spreading are discussed in the "Liquefaction" section below.

9.3.2 Fault Rupture

The bridge site is located within the Seattle Fault Zone. The fault zone is about 4 to 6 km wide (north-south) consisting of a series of east-west-trending faults. It is postulated that the surface faults coalesce to a master Seattle Fault at depth, which is south-dipping reverse fault. The sense of movement on secondary or antithetic faults within the fault zone may be opposite (i.e., north side up, south side down). Geologic evidence suggests that the most recent earthquake that ruptured the ground surface in the fault zone occurred about 1,100 years ago with nearly 22 feet of permanent vertical displacement across the northern-most fault in the zone. The locations of the nearest mapped faults within the fault zone are shown on Figure 6. As shown on this figure, the nearest mapped strand is approximately 2,500 feet south of the bridge. Future ground rupture within the zone may or may not occur along the existing mapped faults.

While the site is located within the Seattle Fault Zone, the actual risk posed by ground rupture is relatively small. The return period for large earthquakes on the fault that may rupture the ground surface is on the order of thousands of years and much longer than 475-year return period ground motions used in design.

9.3.3 Liquefaction

Soil liquefaction is a phenomena in which pore pressure in loose, saturated, granular soils increases during ground shaking to a level near the initial effective stress, thus resulting in a reduction of shear strength of the soil (a quicksand-like condition). As a result of this reduction in shear strength during liquefaction, ground settlement, lateral spreading (ground movement on very gentle slopes) and landslides may occur. Due to the reduced soil strengths, vertical and lateral foundation restraint may also be significantly reduced.

Liquefaction hazard mapping studies for the greater Seattle-Tacoma region identify alluvium and non-engineered fills as having moderate to high liquefaction susceptibilities (Grant et al., 1992; Palmer, 1992; and Shannon & Wilson, Inc., 1993). Potentially liquefiable alluvial and non-engineered fill soils are present at the site and were observed in the borings.

After the Nisqually earthquake that occurred in February 2001, evidence of liquefaction, such as ground settlement, sand boils, and surface cracking, was observed on the south side of the Duwamish Waterway, along the south approach of the South Park Bridge. The observed ground surface cracks were generally oriented parallel to the waterway bank, which may indicate that some lateral spreading may have occurred as a result of liquefaction. Such lateral spreading could have induced undesirable lateral loads on the existing pile foundations supporting bridge piers and bascules. The Nisqually earthquake also caused significant damage to the bridge structure. The movable span was rendered inoperable and had to be repaired. Two bents along the south approach had to be underpinned and grout had to be injected to fill voids that had developed in the south approach embankment fill.

Liquefaction would affect the behavior of the proposed deep foundations by reducing their vertical and lateral capacity. Recommendations for reduction of deep foundation capacities due to liquefaction are provided in subsequent sections of this report (Section 10.4).

The 1998 Load and Resistance Factor Design (LRFD) Bridge Design Specifications, as outlined by the AASHTO, indicates that the bridge design and evaluation should be based on earthquake ground motions with a 10 percent probability of being exceeded in 50 years (475-year return period). The USGS indicates that for a recurrence interval of 475 years, the site PGA is 0.33g (Frankel et al., 1996). For this study, we used the earthquake ground motions developed as previously discussed in the section "Site-Specific Ground Response Analyses for Seismic Design."

Using the results of our subsurface exploration, laboratory testing, and site-specific ground response analysis, we estimated the effects of liquefaction and liquefaction-related settlement and lateral spreading at each boring (borings SB-1 through SB-8). The liquefaction susceptibility of the soils at South Park Bridge was evaluated using the Seed and Idriss simplified empirical procedure in accordance with NCEER technical report NCEER (1997) for ground motions with a return period of 475 years. Results of the liquefaction analyses performed for the SB-1 through SB-8 are shown on Figures 17 through 24, as plots of factor of safety (FS) against liquefaction versus depth. The results of the analyses indicate that much of non-cohesive

Holocene soils are potentially liquefiable (FS < 1) when subjected to the design level ground motions. Table 1 presents the zones that are likely to liquefy based on our analyses.

The liquefaction potential was measured using blow counts derived from the SPT for each boring. An explanation of the SPT procedure is discussed in Appendix A. The blow counts were corrected for depth, equipment variations, and fines content (percent passing the No. 200 sieve). Using the corrected blow counts, an estimation of the liquefaction potential can be analyzed with correlations between the blow counts and the Cyclic Resistance Ratio (CRR). The CRR indicates the shaking threshold for liquefaction. The CRR is compared with the Cyclic Stress Ratio (CSR), which is the stress induced by the design earthquake described previously. The ratio of the CRR to CSR gives the FS for liquefaction potential. A FS below 1.0 indicates liquefaction is probable for the design earthquake.

As shown on the table and figures, the depth and severity of liquefaction and related phenomenon increases at borings closest to and within the Duwamish Waterway. As shown in Figure 20, liquefaction could occur to a depth of 76 feet (elevation –85 feet) at boring SB-4; however, most of the significant liquefaction to affect the project occurs above about elevation -40 feet at the waterway and above about elevation –20 feet on land. We estimate as much as 8 to 12 inches of liquefaction-induced vertical settlement beneath the mudline at boring SB-4 in the waterway. On shore, we estimate as much as 7 inches of vertical settlement on the south shore (boring SB-7) and 5 inches on the north shore (boring SB-2).

Post-liquefaction settlement was estimated using the simplified method developed by Tokimatsu and Seed (1987) and Ishihara and Yoshimine (1990). Table 1 includes the estimated settlement due to liquefaction. The results of the liquefaction analyses were used in assessing lateral spreading, riverbank stability, and foundation capacities and are discussed in subsequent sections of this report.

Lateral spreading occurs when the ground surface displaces towards a free face (e.g., river bank) during liquefaction. Displacements from lateral spreading depend on the severity of the liquefaction, distance to the free face, and height of the free face. Lateral spreading has produced significant damage to structures at and near a free face, such as bridge abutments and waterfront structures. Lateral spreading at the site would likely result in the ground surface displacing towards the Duwamish Waterway. Using the results of the liquefaction calculations, liquefaction-induced lateral spreading was estimated using the empirical procedure by Youd, Hansen, and Bartlett (2003) for the boring locations located north and south of the waterway. These values are presented in Table 1.

9.4 Remediation

9.4.1 General

Approaches to mitigate the effects of liquefaction on structures include the following:

- Improve the subsurface soils to mitigate the potential occurrence of liquefaction.
- Structurally design to resist displacements and induced lateral loads.
- Increase ductility of the structure to accommodate displacements.
- Avoid the area susceptible to liquefaction and displacement.

We expect that only the first two approaches are feasible options for design of the proposed South Park Bridge rehabilitation/replacement.

Structural design to resist vertical and horizontal displacement and lateral loads typically requires stiff foundation elements, such as piles or drilled shafts, to reduce deflection of the structure to tolerable limits. Site conditions and economical design often result in a combination of deep foundations and ground improvements at a given site.

Soil improvement techniques may include densification by vibration (e.g., deep dynamic compaction and vibratory probe); densification by displacement and reinforcement (e.g., vibro-replacement stone columns and compaction grouting); grouting and admixtures (e.g., jet grouting and deep soil mixing); and vibration and drainage (e.g., Earthquake Drains). Selection of the appropriate ground improvement techniques depends upon a number of factors including the soil type (especially fines content), level of improvement required, area and depth to be improved, proximity of adjacent existing structures, and cost. In the vicinity of the project site, a number of ground improvement projects have used compaction grouting or stone columns. Because of concerns about the effect of vibrations resulting from installation of stone columns on the existing bridge and other adjacent structures, this technique may not be appropriate for this project. Therefore, Earthquake Drains or compaction grouting could be two of the most feasible ground improvement technique for the project site. As discussed in subsequent sections, Earthquake Drains are a relatively recent ground improvement method to mitigate liquefaction and have many economic and installation advantages. Compaction grouting is a more expensive option, but has a longer history of use and therefore more data to support its effectiveness in mitigating liquefaction, particularly lateral spreading.

The ground improvements can be installed before, during, or after foundation construction. Initial estimates of the area and quantities of ground improvements (for both Earthquake Drains and compaction grouting) and a discussion of the recommended improvement techniques are described in more detail in the following sections.

9.4.2 Estimated Area and Quantity of Ground Improvements

Based on the liquefaction potential at the site and resulting vertical settlement and lateral spreading, we have provided an initial estimate of the area and quantities for ground improvement. The estimated areas of ground improvement (i.e., Earthquake Drains or compaction grouting) for the four alternatives are shown on Figures 25 through 28. The estimated quantities for each alternative are summarized in Table 2.

For our initial analyses, we estimated the ground improvement areas and quantities from previous studies of mitigation measures for bridge foundations in the area. This assumes that a block of improved soil is created around the foundation elements to resist lateral spreading. As shown on the figures, the areas of ground improvements are centered on the proposed foundations for each alternative. Changes to the foundation alternatives shown on the figures will affect the estimated areas of improvement. The ground improvement areas were estimated to resist the anticipated lateral spreading at the boring located closest to that particular foundation element.

In addition to the ground improvement around the on-land piers, a buttress of ground improvement is also included along the shoreline to protect foundation elements in the waterway. Assuming that lateral spreading displacements will be perpendicular to the waterway, the buttress is designed to resist displacements where foundation elements are located in the waterway. This is shown graphically on Figure 25, with ground improvement buttress areas for all alternatives shown.

As previously discussed in Section 9.2.4, an improved soil block around foundation elements will improve the recommended site response spectrum shown in Figure 16 to at least

AASHTO Soil Profile Type IV and possibly Soil Profile Type III. Determination of the appropriate AASHTO soil profile will depend on the area, type, and soil strength increase from the selected ground improvement. For preliminary design, we recommend using the Soil Profile Type IV for foundation elements within the improved zones shown on Figures 25 through 28. Foundation elements proposed within the waterway (e.g., the Trunnions shown in Figure 25) do not have ground improvements proposed at this time due to possible environmental concerns. Therefore, the designer should anticipate different responses for the approach and overwater elements depending on the overall structural period of the selected bridge alternative.

The initial analysis performed to estimate the ground improvement areas and quantities uses a simplified procedure that is likely conservative. Once the bridge alternative has been selected, we recommend additional analyses that are dependent on the selected foundation type and likely resistance provided by deep foundation elements to refine the ground improvement areas and quantities. We would anticipate a reduction in the quantity and area of the ground improvements based on a more detailed analysis that considers contribution from the deep foundation. Because of the significant number of variables for this study, we did not perform the detailed analysis.

9.4.3 Earthquake Drains

Earthquake Drains are large-flow capacity vertical drains (typically 3 to 8 inches in diameter) wrapped with a geotextile filter fabric. Typical spacing of Earthquake Drains ranges from about 3 to 5 feet on center. They are typically installed by inserting a tubular steel mandrel containing the drain into the ground, using static force and relatively small vibrations. Once the design depth is reached, the mandrel is withdrawn, leaving the Earthquake Drain in place. While the small vibration during installation provides some densification of the soils surrounding the drain, the primary function of the drain is to provide a path for rapid dissipation of excess pore pressures that may develop in the soil as a result of earthquake ground shaking. For this project, where relatively large vibrations would be detrimental to the existing bridge and other adjacent structures, Earthquake Drains can be installed with drilling equipment or static crowd. Provided that the Earthquake Drains are properly designed and installed to mitigate liquefaction under the design earthquake ground motions, the drains should effectively preclude liquefaction-induced lateral spreading in the improved ground areas and reduce potential settlements.

Earthquake Drains have been installed at several sites in the United States and other countries. To our knowledge, none of these sites have been subjected to strong earthquake shaking that could produce significant liquefaction. Studies using blasting to produce liquefaction have demonstrated significant reductions in pore pressure generation where Earthquake Drains have been installed (Rollins et al., 2003 and 2004). During drain installation, some settlement was observed in the surrounding ground surface, but damaging settlement to structures typically did not extend about 6 feet beyond the drain perimeter.

During liquefaction, excess pore pressure discharges from the drains to the ground surface. Therefore, a reservoir consisting of either a horizontal drainage blanket or layer of drainrock is installed at the surface to collect the water discharging from the drains. The water collected in the reservoir can either be allowed to reinfiltrate into the Earthquake Drains or discharged into the storm drain system. Studies have shown that pore pressure dissipation is more effective when the reservoir layer is placed in close proximity to the static groundwater table. From environmental testing, there is the potential that the excess pore pressure discharged from the Earthquake Drains could be contaminated.

The blasting studies show a reduction in vertical settlement from liquefaction between 30 and 65 percent when compared to areas that were not improved with Earthquake Drains. While not addressed in the studies, the rapid dissipation of excess pore pressure from the Earthquake Drain areas would likely result in little if any lateral spreading.

Currently, no data is available on the performance of Earthquake Drains to resist lateral spreading anticipated for the site. However, case histories and the blasting studies of Earthquake Drain performance during liquefaction suggest lateral spreading would be arrested in the improved areas. We understand studies are currently proposed to address Earthquake Drain effectiveness during lateral spreading. Given the current lack of data, it is our opinion that Earthquake Drains would provide an effective mitigation to lateral spreading based on the liquefaction studies performed to date.

We have provided cost estimates for Earthquake Drain, as shown on Table 2. These estimates include a mobilization fee of $25,000 and drain spacing for 3 and 5 feet. As shown on the table, initial cost estimates range from $150,000 for the 5-foot spacing option of the rehabilitation alternative to $960,000 for the 3-foot spacing option of the high-level fixed-span alternative. As previously described, the costs are dependent on the final area and quantity of required ground improvements. The estimated costs do not include demolition, obstruction, and ROW purchases.

9.4.4 Compaction Grouting

Compaction grouting is the controlled, high-pressure injection of a low-slump grout into a soil to create a bulb of grout that displaces and compacts or densifies the surrounding soils. The grout is injected into the ground typically by installation of (drilling and/or pushing) a series of 3- to 4-inch-diameter steel grout pipes into the ground. The grout pipe is typically installed to the bottom of the zone of soil identified for improvement. Once the grout pipe is installed to the pre-determined depth, it is pulled back up toward the ground surface a few inches to dislodge a sacrificial tip at its end that kept soil from entering the grout pipe during installation. The grout is then pumped under pressure to develop a bulb of grout around the tip of the pipe that displaces and densifies the surrounding soil. The grout pipe is pulled back toward the ground surface as the grouting occurs to provide a continuous grout column. To develop an areal zone of improved ground, compaction grout columns are typically installed in a sequenced, square or diamond pattern about 6 to 10 feet on center.

For comparison with the Earthquake Drain cost estimate previously given, we have estimated the cost of compaction grouting for the high-level fixed-span alternative. Assuming a 6-foot spacing and ground improvements occurring to the depths and areas shown in Table 2 and Figure 28, we estimate the costs of compaction grouting for the high-level fixed-span alternative to be on the order of four million dollars. This cost estimate is significantly more expensive than the $960,000 for Earthquake Drains spaced at 3 feet for the same bridge alternative. We would anticipate similar cost differences between the compaction grouting and Earthquake Drains for other alternatives.

10.0 FOUNDATION DESIGN RECOMMENDATIONS

10.1 General

As discussed previously, the current proposed bridge alignment is underlain by fill, marsh, alluvial, and estuarine deposits. These normally consolidated, non-glacial deposits contain interlayers of very loose to loose and very soft to soft soil that range in thickness from about 40 feet at boring SB-8 to more than 100 feet at boring SB-3. Because loose/soft recent deposits are present, it is our opinion that shallow foundations would not be suitable for support of the proposed bridge rehabilitation/replacement alternatives. Also, shallow foundations would not be suitable for support, because excessive settlements could result from potential liquefaction of the underlying loose granular soils at the site under seismic loading conditions. Deep foundations that extend into the underlying glacial deposits and/or rock are recommended to support the proposed bridge rehabilitation/replacement alternatives. Recommended suitable deep foundation types include driven steel pipe piles and bored drilled shafts. The proposed foundation recommendations for South Park Bridge are similar to recommendations for bridge structures constructed in the Duwamish Waterway within the last 20 years.

Driven steel pipe piles are installed in groups, thus providing redundancy in the foundation system. During installation, driving records and dynamic testing may be used to evaluate the capacity and integrity of the installed piles as was done for the test pile. Disadvantages of driven steel pipe piles include: (1) noise and vibrations resulting from driving operations and (2) corrosive soils could affect the structural integrity of the steel piles. Vibrations may also be detrimental to the existing bridge and/or other adjacent structures and utilities as indicated in our measurements of the test piles. Driving-induced vibrations could be reduced by installing the pipe piles open-end. In addition, a sacrificial thickness could be added to the thickness of the steel pile to address corrosion concerns. The information presented for the test pile (Section 5.0 and Appendix E) may be used to evaluate the potential detrimental effects of pile driving.

Large-diameter drilled shaft can provide large axial and lateral capacity. A single, large-diameter drilled shaft may be used to support an entire column and does not need a foundation cap. Eliminating excavation for a cap would be especially advantageous if contaminated soils are present at the site and/or if a cofferdam would be required for construction of the pile cap. Bored drilled shafts also produce much less vibrations and noise than driven piles. Disadvantages for using drilled shafts include: (1) installation of shafts beyond a depth of about 100 feet becomes increasingly difficult, and possibly cost-prohibitive beyond about 120 feet if casing is not left in place; (2) lack of redundancy if a single shaft is used; (3) construction quality assurance is more critical than that for driven piles for the successful completion of a drilled shaft; and (4) drilling equipment required for installing large-diameter drilled shafts is generally large and heavy, especially if a casing is required for the installation. This means that a large capacity temporary trestle may be required for constructing the piers located within the waterway.

As discussed earlier, based on the subsurface conditions encountered in the reviewed field explorations, Subsurface Profile A-A' (Figure 4) was developed along the proposed new bridge alignment. Based on the subsurface conditions shown on Subsurface Profile A-A' and considering construction impacts on adjacent existing structures and/or utilities, we provide the following foundations recommendations for each of the four alternatives:

10.2 Rehabilitation Alternative

We understand that the existing bascule piers would be replaced with new piers. Foundations for the new bascule piers are proposed to be installed through the existing timber piles supporting the existing piers. We also understand that new foundation support would be installed outside the existing approach-span piers. We recommend that new foundation support for the bascule piers and for the approach-span piers consist of drilled shafts. Prior to installing the new foundations, we recommend that ground improvement, such as Earthquake Drains, be used to increase strength and mitigate liquefaction potential of soils beneath and around the existing foundation. Additional studies would be required to provide design recommendations for ground improvement.

A cofferdam would be required for construction of the foundation cap around the new foundation.

Installation of drilled shafts to significant depth to achieve the required design capacity may be prohibitive. Therefore, driven piles may be necessary if shafts extend beyond a depth of 100 to 120 feet. This would most likely occur along the north shore of the Duwamish Waterway where competent bearing soil is deepest.

10.3 Bascule, Mid-Level Fixed-Span, and High-Level Fixed-Span Alternatives

We recommend that drilled shafts be used as foundations for these alternatives. As discussed earlier, the drilled shafts would provide better axial and lateral capacities.

We also recommend that ground improvement, such as Earthquake Drains, be used to increase strength and mitigate liquefaction potential of soils around and beneath the new foundations. Ground improvement would help reduce the impact of lateral loads resulting from any potential lateral spreading on the new foundation. Additional studies would be required to provide design recommendations for ground improvement.

We understand that a cofferdam would be required for construction of foundation cap around foundation elements supporting the new bascule bridge.

As previously stated, very deep (greater than 120 feet) drilled shafts may not be feasible along the north shore of the Duwamish Waterway and driven piles may be required instead.

For the four rehabilitation/replacement alternatives, a temporary trestle would be used for construction within the river. We recommend that driven closed-end steel pipe piles be used to support the temporary trestle.

Conceptual recommendations are presented in the following sections for 24-inch-diameter steel pipe piles driven closed-end, and 6-, 8-, and 10-foot-diameter drilled shafts.

10.4 Axial Capacity

Capacity of piles/drilled shafts will vary with pile/shaft penetrations, pile/shaft size, and the subsurface conditions. Axial capacity analyses were performed for 24- inch-diameter steel pipe piles driven closed-end, and 6-, 8-, and 10-foot-diameter drilled shafts. This section describes the analysis approach used to estimate the capacities of these piles and drilled shafts and presents the results of the axial capacity analyses.

Axial capacities were evaluated under static and seismic loading conditions. Under static loading conditions, a deep foundation is typically designed to support anticipated dead and live loads. Under seismic loading conditions, a deep foundation is designed to support anticipated dead and seismic loads. In addition to these loads, the deep foundation should be designed to support downdrag forces resulting from potential liquefaction of underlying recent fill and Holocene deposits. The static and seismic axial capacities were determined based on subsurface conditions encountered in the reviewed explorations, relative densities and strengths of the subsurface soils as determined by SPT values (N-values), and our experience in similar soil and project conditions.

Axial capacity analyses were performed using an in-house computer program that determines axial compressive capacity by summing skin friction along the side of the pile/shaft and end bearing at its tip/base. For driven piles, allowable compressive capacity is a summation of allowable skin friction and allowable end bearing. Capacities for the driven piles were also augmented with the dynamic testing and analyses performed for the test pile (Appendix E). For drilled shafts, allowable compressive capacity is a summation of allowable skin friction and mobilized end bearing. Allowable, static skin friction values for pipe piles and drilled shafts were obtained by applying a FS of 2.0 to the estimated ultimate values. Assuming the foundation would be bearing in glacial deposits, a FS of 2.0 was also used to determine the allowable end-bearing values for pipe piles. For drilled shafts, estimated mobilized unit end bearing values of 10 to 15 tons per square foot (tsf) in glacial deposits were used in calculations. Figures 29 through 44 present the estimated static compressive and ultimate uplift capacities versus depth of the driven pipe pile and drilled shaft options for all eight boring locations.

Permanent casing may be required for drilled shafts deeper than about 120 feet, particularly for shafts located between borings SB-3 and SB-4. If permanent casing is used, frictional resistance along the shafts will be reduced by about 25 percent. The reduction is due to the smoother surface of the steel casing against the soil as opposed to the rougher contact between the concrete and soil. Permanent casing will have no effect on the end bearing capacity of the drilled shaft. Based on subsurface conditions encountered, we anticipate casing may be required in the alluvial and estuarine deposits, depending on the installation method used during construction. The frictional capacities of these deposits are relatively small; therefore, we anticipate minor reductions for the static compressive and uplift capacities (typically less than 10 percent) depending on the shaft size and location and the depth and extent of the permanent casing.

The seismic condition accounts for the potential liquefaction of the loose/soft alluvial deposits. Figures 45 through 60 present the estimated seismic compressive capacities versus depth of the pipe pile and drilled shaft options for all eight boring locations. The capacities in Figures 45 through 60 include downdrag forces resulting from potential liquefaction of soils underlying the proposed bridge rehabilitation/replacement alignments.

The seismic capacities presented in Figures 45 through 60 assume liquefaction occurs during the design earthquake. The axial capacities are presented as ultimate values and we recommend an appropriate dynamic FS be applied to values derived from these figures for design. Borings SB-3 and SB-8 have only minor, isolated areas of liquefaction. Therefore, downdrag forces have been neglected from these boring locations. Also, isolated areas of liquefaction below a depth of about 40 feet bgs in the remaining borings have also been ignored.

For areas where ground improvement is proposed (Figures 25 through 28), liquefaction will be mitigated and foundation elements within these areas can be designed using the static capacities shown in Figures 29 through 44 with an appropriate dynamic FS (typically 1.1 or greater). However, for Earthquake Drains, downdrag may still occur from reduced vertical settlement of the surrounding soil.

It should be noted that liquefaction-induced ground settlements and the resulting downdrag forces are likely to develop after the maximum anticipated seismic forces had occurred. Therefore, we recommend that the downdrag forces be applied with post-earthquake loading consisting of the typical service loads under static conditions.

We recommend that the piles/drilled shafts be spaced no closer than three pile/shaft diameters, measured center-to-center. At this spacing, a group reduction factor is not warranted when estimating the group axial capacity.

10.5 Lateral Resistance

As discussed in Section 6.2.5, the soils underlying the project site are susceptible to liquefaction and lateral spreading. The lateral resistance of deep foundations supporting the existing and/or a new bridge depends greatly on the extent of these liquefaction hazards and if ground improvements are implemented to mitigate them.

If ground improvements are accomplished, the lateral capacities at the top of piles/shafts could be between 20 and 40 percent of the foundations' compressive capacities. However, if the ground is not improved, it would be difficult at this stage to estimate the lateral capacity.

Recommended parameters for performing lateral resistance using LPILE under both static and seismic conditions are given for each boring location on Table 3. For areas where ground improvement is proposed, recommended parameters for static loading conditions should be assumed.

10.6 Retaining Walls and Embankment Fills

Because of the limited ROW, retaining walls are under consideration to retain the fills needed for construction of the north and south approaches of the project. Factors affecting the selection of the most appropriate wall type include soil and groundwater conditions, aesthetics, cost, and performance considerations. Suitable wall types also depend on whether the wall is constructed to support a cut or a fill.

Considering the liquefaction potential of soils underlying the project site, we recommend that mechanically stabilized earth (MSE) walls be considered for the proposed approaches. The advantage of using an MSE wall at this location would be that this type of wall is relatively flexible and can tolerate large total and differential settlements.

There are a number of different types of MSE walls that could be constructed at this site. Some of the most common types of proprietary MSE walls are manufactured by Hilfiker Retaining Walls (HRW), SSL, VSL Corporation, and the Reinforced Earth Company. All of these wall systems use metallic inclusions to reinforce the soil and retain the backfill. HRWs and SSL use welded wire mesh while VSL Corporation and the Reinforced Earth Company use horizontal reinforcing strips. Most walls with non-metallic (geosynthetic) inclusions are non-proprietary systems in that there is a choice of several different types of geosynthetics with similar strengths combined with a choice of several different facing systems. In order to tolerate potentially large settlements at the site, we recommend that the propose approach walls consist of wire-meshed wall system, such as Hilfiker Welded Wire Wall from HRW or the MSE Plus Wire System from SSL.

For initial design of proposed MSE walls, we recommend an active lateral earth pressure using an equivalent fluid weight of 35 pounds per cubic foot (pcf) for compacted fill placed behind the wall. This assumes the embankment fill consists of select sand and gravel fill with a unit weight of about 125 pcf and a friction angle of about 34 degrees. The allowable bearing capacity (FS = 2.0) for the walls is 3,000 pounds per square foot (psf) with an equivalent fluid weight of 300 pcf to calculate passive pressure (FS = 1.5) on properly prepared subgrade soils. A surcharge load should be added to the MSE wall to account for vehicular traffic. We suggest an additional two feet of soil be added to the wall height to simulate traffic loading.

Provided that the proposed approach retaining walls are less than 15 feet in height and that any potential damage to these walls would not affect the proposed bridge structure, ground improvement would not be required beneath the proposed approaches. Embankment fill structures less than 15 feet in height would likely result in ground settlement between 3 and 5 inches along the south shore and would be greatest near boring SB-7. The majority of ground settlement would likely be from compression/consolidation of the clayey silt and loose silty sand underlying the existing fill. The clayey silt is significantly thinner or absent on the north shore; however, settlements up to 3 inches are possible. Ground settlement would diminish significantly with increasing distance from the fill edge. The settlement would occur rapidly as the embankment fill is placed and would likely be completed within a month of fill completion.

11.0 CONSTRUCTION CONSIDERATIONS

11.1 General

Based on our review of subsurface conditions along the project alignment, we recommend that the following considerations be evaluated for designing foundation support for the South Park Bridge:

- Fill deposits underlying the proposed alignment may contain varying quantities of debris and obstructions. Such obstructions would cause hard driving conditions for pile installation. They would also present difficulties during installation of drilled shafts.
- The low strength, high sensitivity, and thickness and depth of the estuarine deposits underlying the site could result in necking during installation of drilled shafts. To reduce the potential of necking, a permanent casing may be considered for drilled shafts for these areas. Permanent casing may also be required for drilled shafts greater than about 120 feet.
- Vibration-induced settlements may impose downdrag forces on the timber piles supporting the existing bridge. As previously discussed, piles supporting the north bascule bridge were not driven into competent bearing soils. Vibrations from driving of adjacent new piles may cause additional settlement of the north bascule foundation, which may result in additional distress to the existing bridge. Based on the test pile information, the downdrag force may occur for existing piles located within 10 feet of new driven piles, assuming the new piles are 24-inch-diameter closed-end steel pipe.
- The ground improvement is based on a seismic event with 475-year return period. If a larger event occurs during the bridge lifetime, this could increase lateral forces on the bridge foundations and cause liquefaction at greater depths.
- For driven piles and drilled shafts, the conditions of all adjacent buildings, structures, and existing bridge located within about 150 feet of the construction site should be recorded and monitored before, during, and after the project construction.

Soil and groundwater in some areas at the project site could be contaminated. Contaminated soil and groundwater removed from drilled shaft excavations and/or pile cap construction would require special handling and treatment. In order to prevent vertical migration of contaminated groundwater during installation of drilled shafts, a casing would need to be installed through the contaminated zone. This section provides construction information for the recommended drilled shaft alternative.

11.2 Drilled Shafts

Construction of a drilled shaft requires boring a hole of a specified diameter and depth and then backfilling the hole with reinforced concrete. The selection of equipment and procedures for constructing drilled shafts is a function of the shaft dimensions, the subsoil conditions, and the groundwater characteristics. Consequently, the design and performance of drilled shafts can be significantly influenced by the equipment and construction procedures used to install the shafts. In particular, shaft friction would be impacted by the procedures used for construction and also by method of placement and properties of concrete. Construction procedures and methods are of paramount importance to the success of the drilled shaft installation at this project site.

Drilled shaft contractors who participate on this project should be required to demonstrate that they have suitable equipment for this project, and adequate experience in the construction of drilled shafts with similar subsurface conditions.

11.2.1 Construction

In general, there are three typical methods of installing drilled shafts: the dry method, the casing method, and the wet method. In the dry method of construction, the excavation is normally carried to its full depth without casing or slurry through clay or dry, dense sand where groundwater is not encountered. The casing method is applicable where seepage and/or caving soil conditions are encountered, and a casing can be pushed or driven into an impermeable, firm stratum below the seepage zone or caving soil. The wet method of construction generally involves the use of slurry. The subsurface conditions where the wet method of construction is applicable include any of the conditions described above for the casing method. In instances where heavy seepage and/or caving conditions are encountered and the hole cannot be sealed, the wet method of construction may be the only feasible way to stabilize the shaft walls while drilling is continued. If an impermeable soil zone is not encountered in which to form a seal, or there is a potential for bottom heave or blowout, it would be required to complete the excavation in the wet with slurry.

Alluvial and estuarine deposits observed in all borings above elevations –20 to –80 feet are saturated and would likely result in caving conditions during installation of uncased drilled shafts. The glacial deposits underlying these soils consist of very stiff to hard silt and clay that would likely not require casing. However, zones of more permeable sand and gravel were observed in some borings and are common in glacial deposits. These zones typically are under high seepage pressures where caving may occur.

Because of the very soft/loose deposits encountered at the site and the anticipated penetration depths of the drilled shafts, it is our opinion that installation of drilled shafts for this project could generally proceed using a combination of both the casing and wet methods of construction. We recommend that casing be installed a minimum of 5 feet into the medium dense to very dense alluvial soils, the glacial soils, or to whatever depth necessary to prevent caving and base heave. We also recommend that the casings used for installation of the piers within the waterway be left in place permanently. Slurry would be required within the casing and below it as the excavation proceeds. The use of bentonite slurry, rather than bentonite/soil slurry, is normally preferred for quality control purposes. The slurry column should extend well above the level of the water level. When using slurry to advance an excavation through granular soils, a tool may be used to mix the underlying soils with the overlying slurry to advance the hole. The contractor should be prepared to drill through or remove large logs, debris, and other obstruction that may be present in the soils at the site.

Upon completion of the shaft excavation, the hole is cleaned and the reinforcing steel is installed. In the casing method of construction, the reinforcing steel (typically a rebar cage) is usually placed to the bottom of the hole. The reinforcing steel should therefore be designed to accommodate the structural requirements of the completed shaft, the stability requirements for its placement, and the concrete placement.

After the reinforcing steel is placed, the hole should be filled with concrete. We recommend that the casing be left in place and not withdrawn to avoid disturbing the shaft and to provide additional structural strength, particularly in layers of sand/loose soil.

11.2.2 Monitoring of Drilled Shaft Installations

An experienced and qualified geotechnical engineer who is familiar with the subsurface conditions at the site should monitor installation of drilled shafts. Construction of the shafts using slurry will prevent downhole visual inspection. Inspection and identification of soil mucked from the hole or retrieved from auger flights should be accomplished by an experienced geotechnical engineer/engineering geologist familiar with the project. These observations should be made to confirm that the subsurface conditions encountered during construction are similar to those assumed for design.

In addition, the excavation methods, casing placement, steel reinforcing, and concrete placement operations should be monitored and documented. As a minimum, a report should be prepared for each drilled shaft that includes the criteria recommended in the Drilled Shaft Inspector's Manual prepared by the Deep Foundation Institute.

11.2.3 Integrity Testing

In order to evaluate the integrity of the installed drilled shafts, we recommend that proper tubes be installed in all shafts for crosshole sonic logging (CSL) tests or other appropriate integrity tests. The integrity tests should be performed and analyzed by experienced and qualified personnel.

12.0 LIMITATIONS

This report was prepared for the exclusive use of PB and King County for the rehabilitation or replacement of the South Park Bridge. The report should be provided to reviewing agencies and prospective subcontractors for information based on factual data only and not as a warranty of subsurface conditions, such as those interpreted from the exploration logs and discussions of subsurface conditions included in this report.

The analyses, conclusions, and recommendations contained in this report are based on site conditions as they presently exist. We assume that the exploratory borings made for this project are representative of the subsurface conditions throughout the site; i.e., the subsurface conditions everywhere are not significantly different from those disclosed by the explorations. If conditions different from those described in this report are observed or appear to be present during construction, we should be advised at once so that we can review these conditions and reconsider our recommendations, where necessary. If conditions have changed due to natural causes or construction operations at or near the site, it is recommended that this report be reviewed to determine the applicability of the conclusions and recommendations considering the changed conditions and time lapse.

Within the limitations of the scope, schedule, and budget, the analyses, conclusions, and recommendations presented in this report were prepared in accordance with generally accepted professional geotechnical engineering principles and practice in this area at the time this report was prepared. We make no other warranty, either express or implied.

The scope of our services did not include any environmental assessment or evaluation of hazardous or toxic materials in the soil, surface water, groundwater, or air at the subject site. Limited out-of-scope testing was performed for potential contaminants as described in this report. Shannon & Wilson, Inc., has qualified personnel to assist you with these services should they be necessary.

Shannon & Wilson, Inc., has prepared Appendix F, "Important Information About Your Geotechnical Report," to assist you and others in understanding the use and limitations of our reports.

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TABLE 1
ESTIMATED EFFECTS OF LIQUEFACTION

Boring	Depth of Potentially liquefiable zones[1] (ft)	Estimated liquefaction-induced settlement[2] (in)	Estimated displacement from lateral spreading[3] (ft)
SB-1	13 to 20 (El. 4 to -3)	1.5 to 3	15
SB-2	12 to 26 (El. 0 to -14)	3 to 5	30
SB-3	Minor, localized layers	2 to 4	25
SB-4	12 to 36 (El. -3.5 to -27.5)	8 to 12	-
SB-5	18 to 42 (El. -9 to -33)	5 to 8	-
SB-6	12 to 35 (El. 7 to -16)	4 to 6	20
SB-7	13 to 21 (El. 6 to -2)	5 to 7	10
SB-8	12 to 16 (El. 7 to 3)	1 to 1.5	3

Notes:
1. Liquefaction potential was estimated using the Seed and Idriss' Simplified Procedure (Youd and Idriss, 2001).
2. Liquefaction-induced range of settlements were estimated using simplified methods developed by Tokimatsu and Seed (1987) and Ishihara and Yoshimine (1990).
3. Liquefaction-induced lateral spreading was estimated using an empirical method developed by Youd, Bartlett, and Hansen (2002).

TABLE 2
ESTIMATION OF GROUND IMPROVEMENT FOR SOUTH PARK BRIDGE ALTERNATIVES

Alternative	Improvement Surface Area (sf)	Volume to Improve (cy)	3-foot Spacing		5-foot Spacing	
			Amount (ft)	Estimated Cost	Amount (ft)	Estimated Cost
Rehabilitation Option	30,000	40,000	115,000	$ 400,000	40,000	$ 170,000
New Bascule Option	40,000	1,500,000	160,000	$ 540,000	60,000	$ 220,000
Mid Level Option	65,000	2,225,000	250,000	$ 770,000	90,000	$ 320,000
High Level Option	85,000	2,800,000	310,000	$ 960,000	110,000	$ 390,000

TABLE 3

Recommended Parameters for Development of P-Y Curves Using LPILE^{PLUS}

Location	Boring(s)	Upper Boundary (Depth Below Ground Surface) (feet)	Lower Boundary (Depth Below Ground Surface) (feet)	Soil Type (Soil Type No.)	Effective Unit Weight γ' (pcf)	Cohesion, c (psf)		Friction Angle, ϕ (°)		Modulus of Subgrade Reaction, k (pci)		ε_{50}
						Static/ Cyclic	Seismic	Static/ Cyclic	Seismic	Static/ Cyclic	Seismic	
Sta. 32+40	SB-1	0	13	Sand (4)	115	-	-	29	23	25	20	-
		13	21	Sand (4)	52.6	-	-	29	5	20	2	-
		21	67	Sand (4)	57.6	-	-	36	36	85	85	-
		67	-	Stiff Clay w/o free water (3)	62.6	6,000	6,000	-	-	2,000	2,000	0.004
Sta. 30+60	SB-2	0	12	Sand (4)	115	-	-	30	24	40	32	-
		12	26	Sand (4)	52.6	-	-	30	5	30	3	-
		26	32	Sand (4)	52.6	-	-	32	32	50	50	-
		32	71	Sand (4)	57.6	-	-	36	36	85	85	-
		71	-	Stiff Clay w/o free water (3)	62.6	6,000	6,000	-	-	2,000	2,000	0.004
Sta. 27+50	SB-3	0	13	Sand (4)	115	-	-	30	24	40	32	-
		13	17	Sand (4)	52.6	-	-	30	5	30	3	-
		17	37	Sand (4)	57.6	-	-	34	30	85	70	-
		37	47	Sand (4)	57.6	-	-	32	5	50	5	-
		47	90	Sand (4)	57.6	-	-	36	36	100	100	-
		90	109	Sand (4)	62.6	-	-	40	40	125	125	-
		109	-	Stiff Clay w/o free water (3)	62.6	6,000	6,000	-	-	2,000	2,000	0.004
Sta.25+35	SB-4	0	42	Sand (4)	47.6	-	-	29	5	20	2	-
		42	70	Sand (4)	57.6	-	-	34	34	60	60	-
		70	94	Sand (4)	57.6	-	-	33	33	50	50	-
		94	-	Stiff Clay w/o free water (3)	62.6	5,000	5,000	-	-	2,000	2,000	0.004

TABLE 3

Recommended Parameters for Development of P-Y Curves Using LPILE^{PLUS}

Location	Boring(s)	Upper Boundary (Depth Below Ground Surface) (feet)	Lower Boundary (Depth Below Ground Surface) (feet)	Soil Type (Soil Type No.)	Effective Unit Weight γ' (pcf)	Cohesion, c (psf)		Friction Angle, ϕ (°)		Modulus of Subgrade Reaction, k (pci)		ε_{50}
						Static/ Cyclic	Seismic	Static/ Cyclic	Seismic	Static/ Cyclic	Seismic	
Sta. 23+25	SB-5	0	15	Soft Clay (1)	42.6	150	60	-	-	15	2	0.002
		15	56	Sand (4)	52.6	-	-	31	5	40	4	-
		56	-	Stiff Clay w/o free water (3)	62.6	5,000	5,000	-	-	2,000	2,000	0.004
Sta. 20+00	SB-6	0	13	Sand (4)	115	-	-	29	20	25	20	-
		13	23	Sand (4)	52.6	-	-	29	5	20	2	-
		23	35	Sand (4)	57.6	-	-	34	5	65	7	-
		35	60	Sand (4)	57.6	-	-	34	30	65	45	-
		60	68	Sand (4)	57.6	-	-	34	5	65	7	-
		68	-	Stiff Clay w/o free water (3)	62.6	6,000	6,000	-	-	2,000	2,000	0.004
Sta. 17+40	SB-7	0	13	Sand (4)	115	-	-	29	20	25	20	-
		13	25	Sand (4)	52.6	-	-	28	5	20	2	-
		25	53	Sand (4)	57.6	-	-	36	36	95	95	-
		53	-	Stiff Clay w/o free water (3)	62.6	4,000	4,000	-	-	2,000	2,000	0.004
Sta. 15+20	SB-8	0	12	Sand (4)	115	-	-	31	25	40	30	-
		12	17	Sand (4)	115	-	-	35	5	75	7	-
		17	48	Sand (4)	57.6	-	-	35	35	75	75	-
		48	-	Stiff Clay w/o free water (3)	62.6	6,000	6,000	-	-	2,000	2,000	0.004

Note:

(1) Parameters given above are based on subsurface conditions encountered in indicated boring.

(2) Based on subsurface conditions encountered along alignment, scattered zones of soil may liquefy under earthquake loading. Seismic loading may also result in strength reduction for some cohesive soil layers, mainly soils overlying liquefied zones. Parameters under seismic loading are provided.

(3) Parameters given above do not reflect effect of deep foundation group action. See text regarding recommendations for group action.

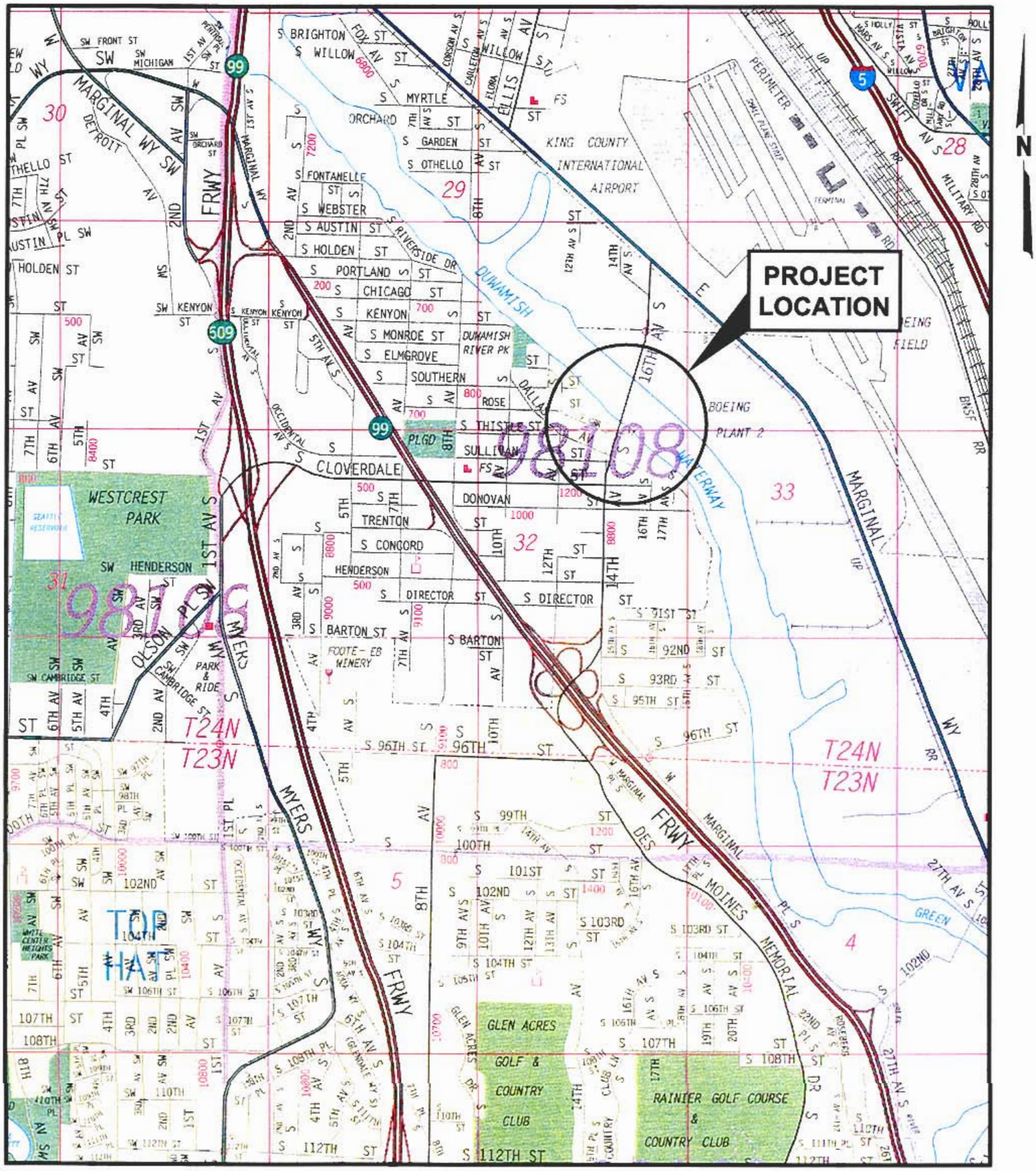
TABLE 4

LPILE^{PLUS} EFFICIENCY FACTORS FOR GROUPS

Spacing	Efficiency Factor, P_m	
	Side	Trailing
1D	0.53	0.58
1.5D	0.67	0.66
2D	0.78	0.73
2.5D	0.89	0.78
3D	0.99	0.83
3.5D	1	0.87
4D	1	0.91
5.5D	1	1.00

Note:

The efficiency factors are based on
recommendations presented in a 1998 ENSOFT



0 1/4 1/2 1

Scale in Miles

South Park Bridge
Seattle, Washington

VICINITY MAP

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FIG. 1

NOTE

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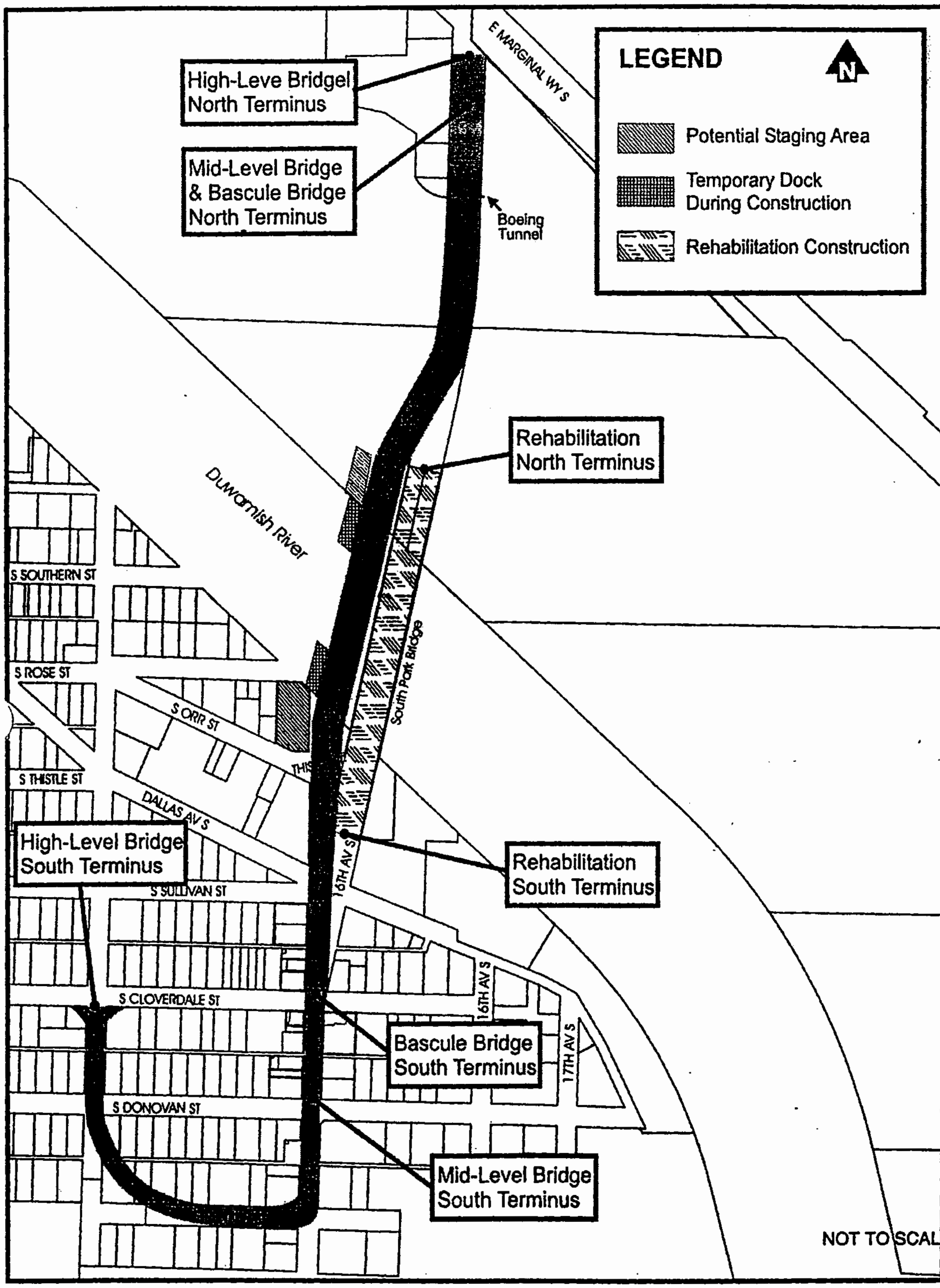

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South Park Bridge
Seattle, Washington

PROPOSED CONSTRUCTION SCHEMES

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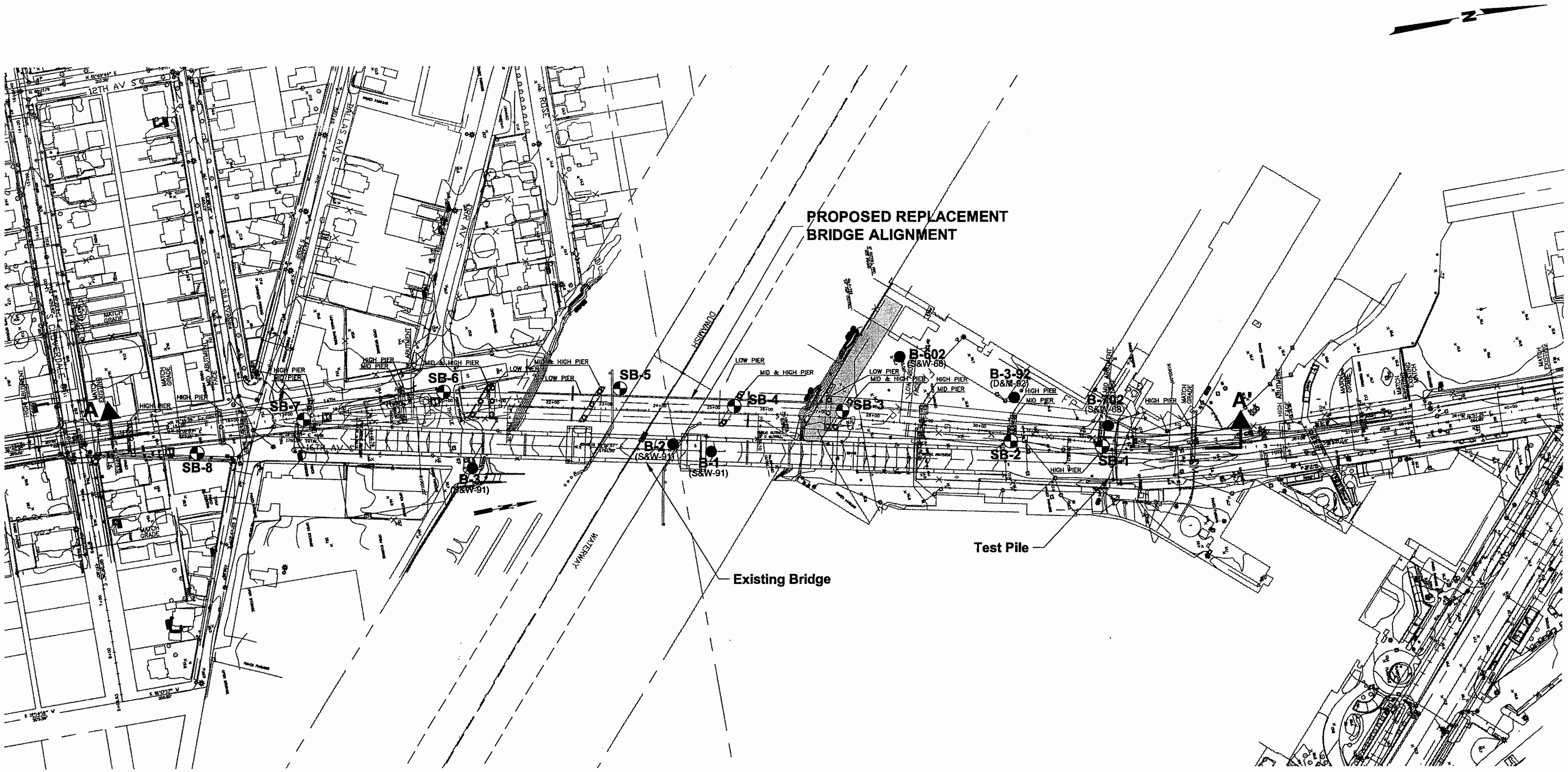
FIG. 2



0 200 400

Scale in Feet

LEGEND

SB-1 Boring Designation and Approximate Location

B-1 (S&W-91) Previous Boring Designation and Approximate Location (Investigator and Year Completed)

A Generalized Subsurface Profile (See Figure 4)

NOTE

Boring locations taken from electronic file provided by Parsons Brinckerhoff, dated 6-20-03.

South Park Bridge
Seattle, Washington

SITE AND EXPLORATION PLAN

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FIG. 3

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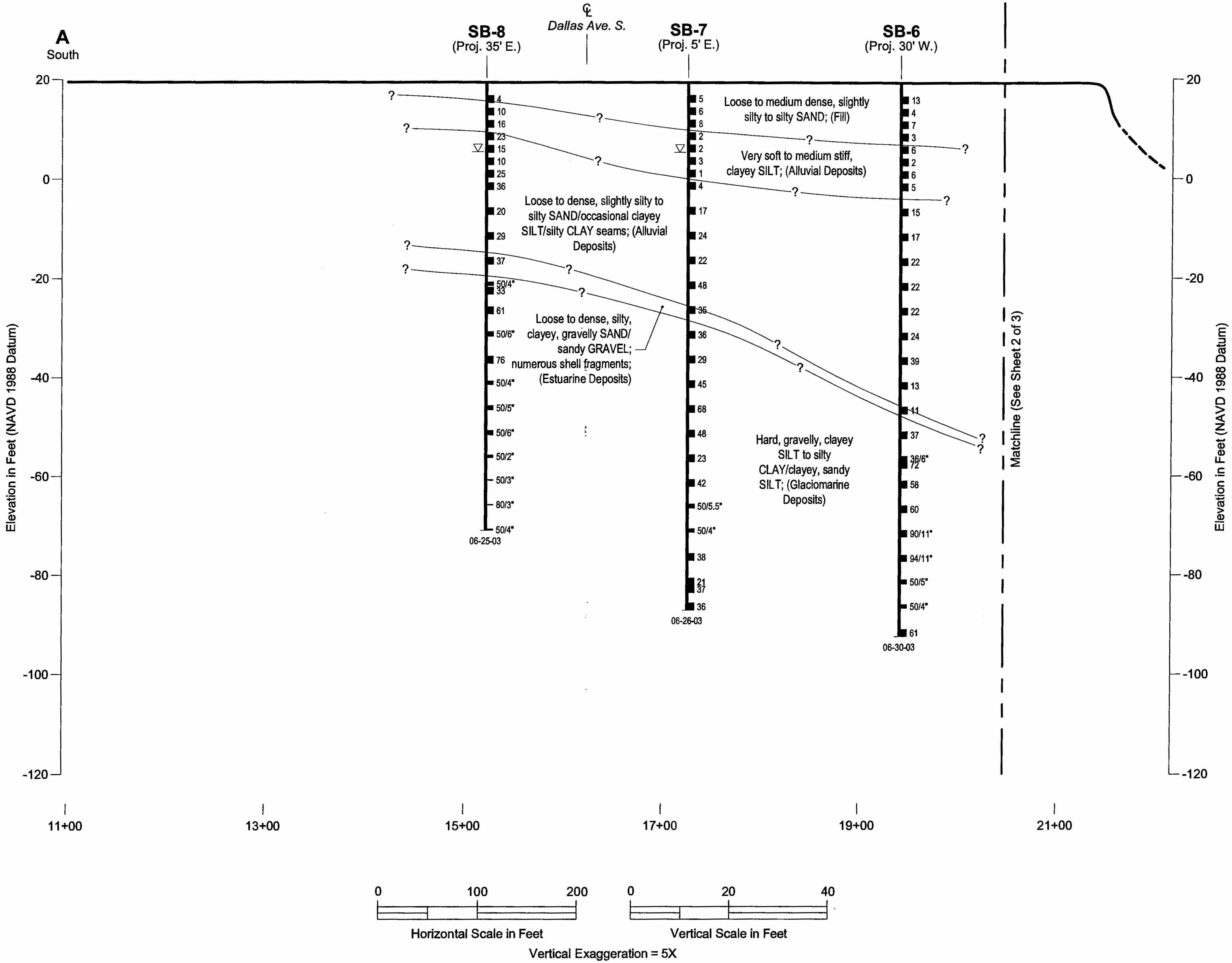

LEGEND

SB-6 — Boring Designation
(Proj. 8' N.) — Projected Distance
8 — Standard Penetration Test Blows/Foot
100/6" — Standard Penetration Test Blows/Inches
12" — Sample Obtained with 3" Dames and Moore Sampler
Groundwater Level Observed
? — ? Approximate Geologic Contact
Bottom of Boring
03-17-03 — Date Completed

NOTES

1. Datum: NAVD (1988).
2. This subsurface profile is generalized from materials observed in soil borings. Variations may exist between profile and actual conditions.

South Park Bridge
Seattle, Washington

GENERALIZED SUBSURFACE PROFILE A-A'

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FIG. 4
Sheet 1 of 3

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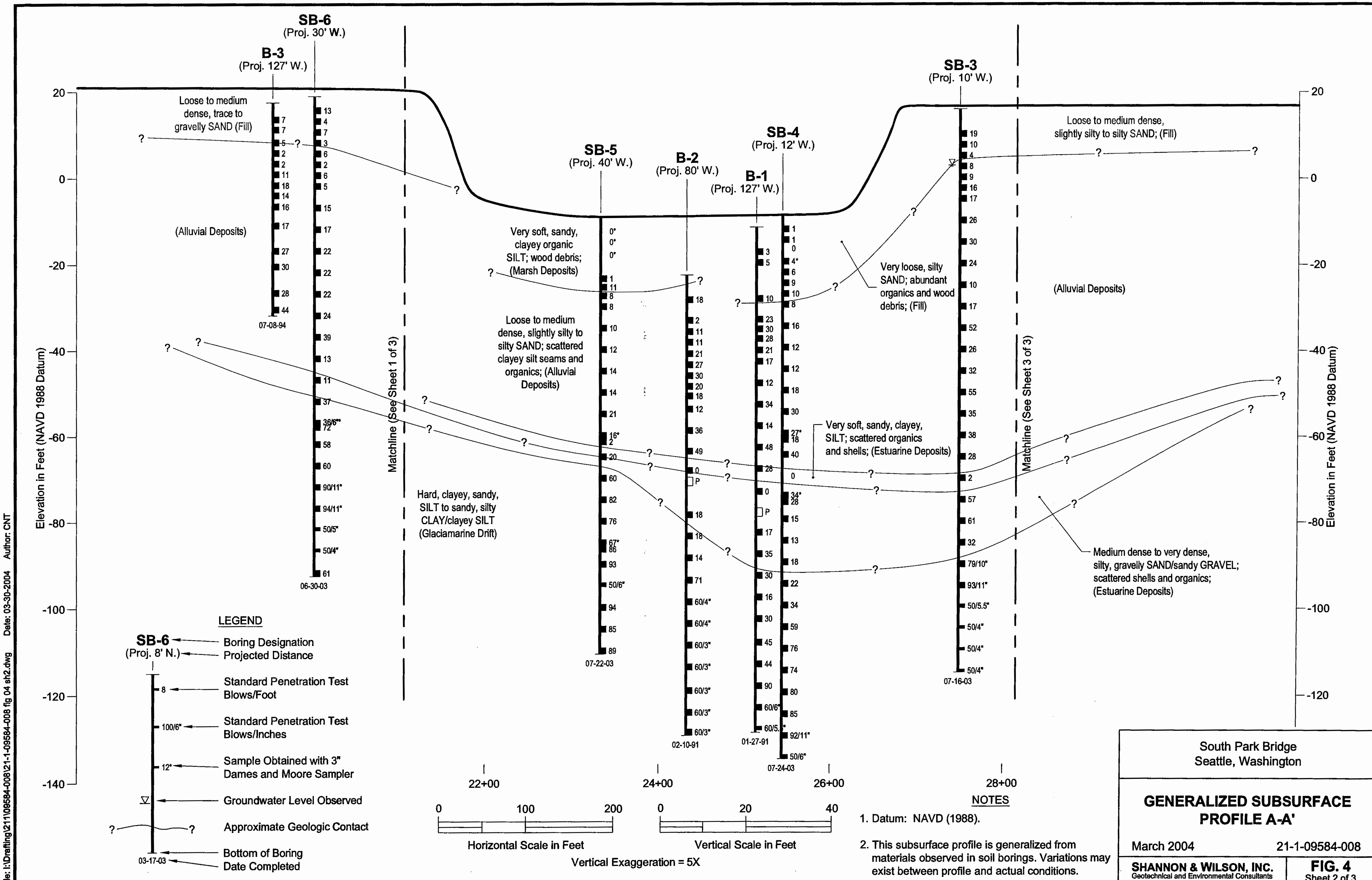

NOTES

1. Datum: NAVD (1988).
2. This subsurface profile is generalized from materials observed in soil borings. Variations may exist between profile and actual conditions.

South Park Bridge
Seattle, Washington

GENERALIZED SUBSURFACE PROFILE A-A'

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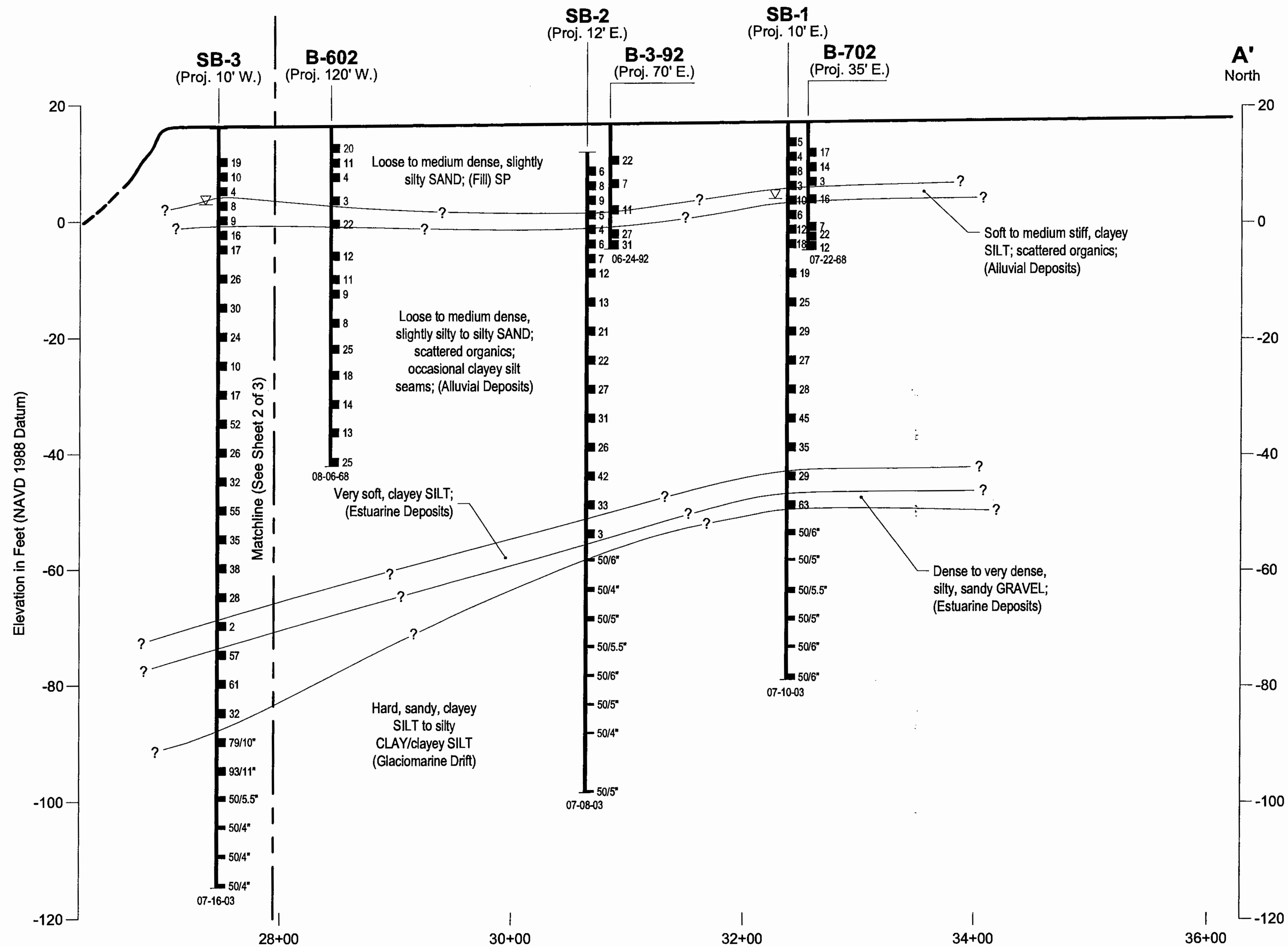
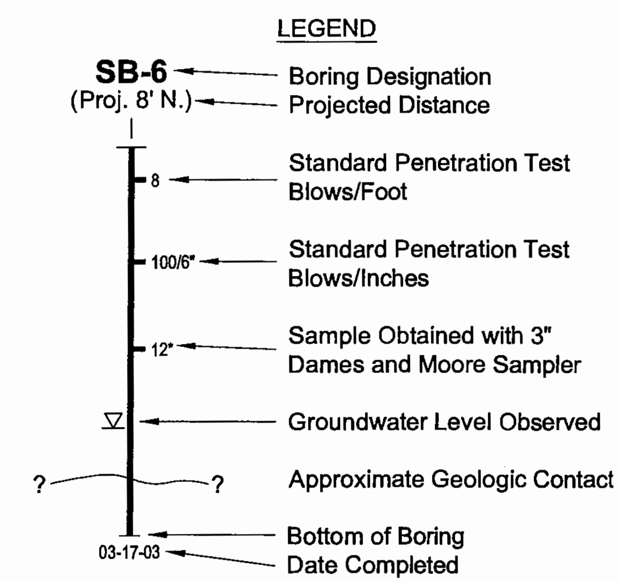
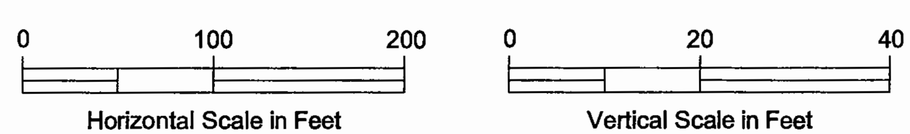
FIG. 4
Sheet 2 of 3







Vertical Exaggeration = 5X

NOTES

1. Datum: NAVD (1988).

2. This subsurface profile is generalized from materials observed in soil borings. Variations may exist between profile and actual conditions.

South Park Bridge
Seattle, Washington

GENERALIZED SUBSURFACE PROFILE A-A'

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FIG. 4
Sheet 3 of 3

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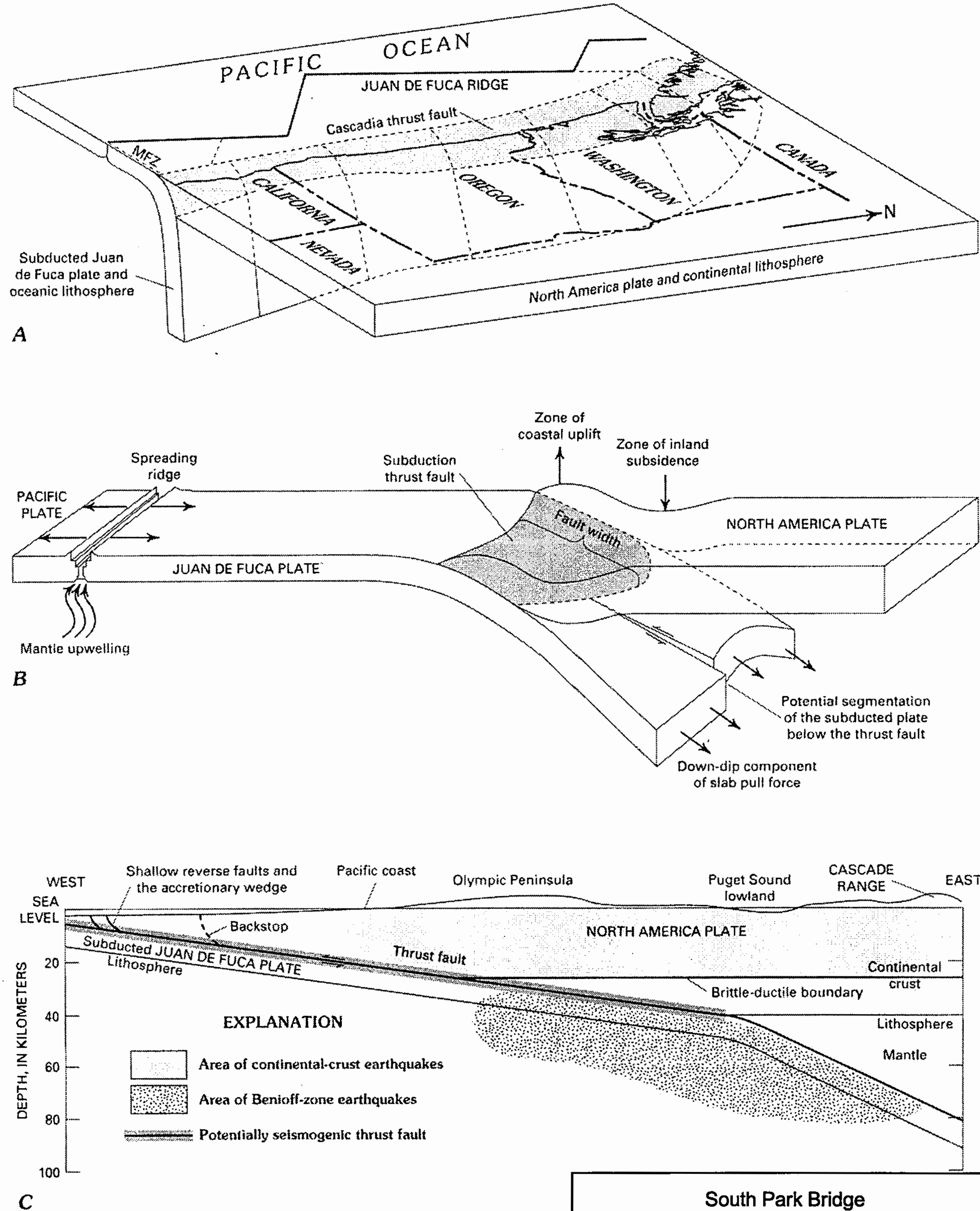

NOTE

Figure adapted from Rogers et. al., "Earthquake Hazards in the Pacific Northwest - An Overview," USGS Professional Paper 1560, dated 1996.

South Park Bridge
Seattle, Washington

REGIONAL MAP OF THE CASCADIA SUBDUCTION ZONE

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FIG. 5

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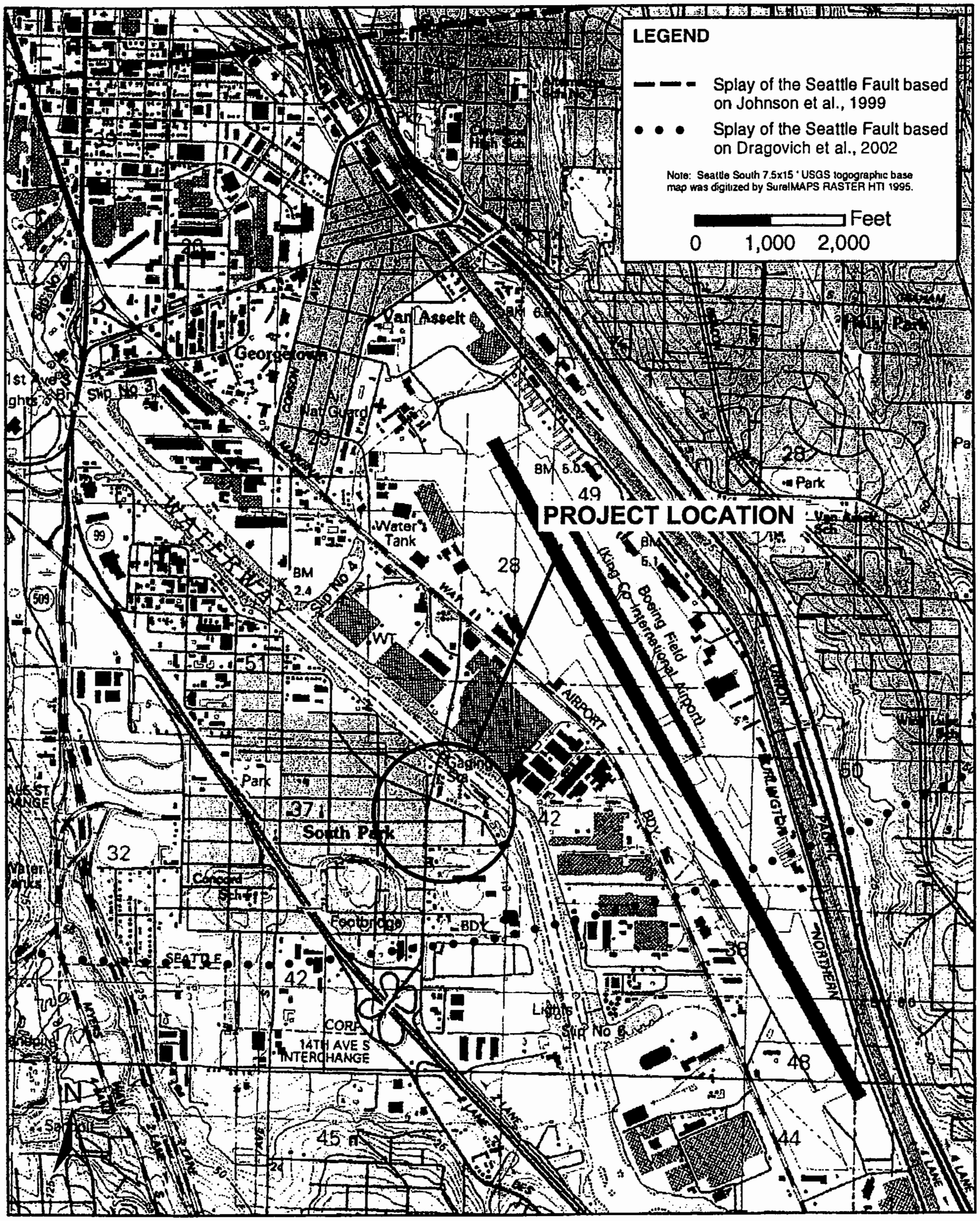

South Park Bridge
Seattle, Washington

FAULT LOCATION MAP

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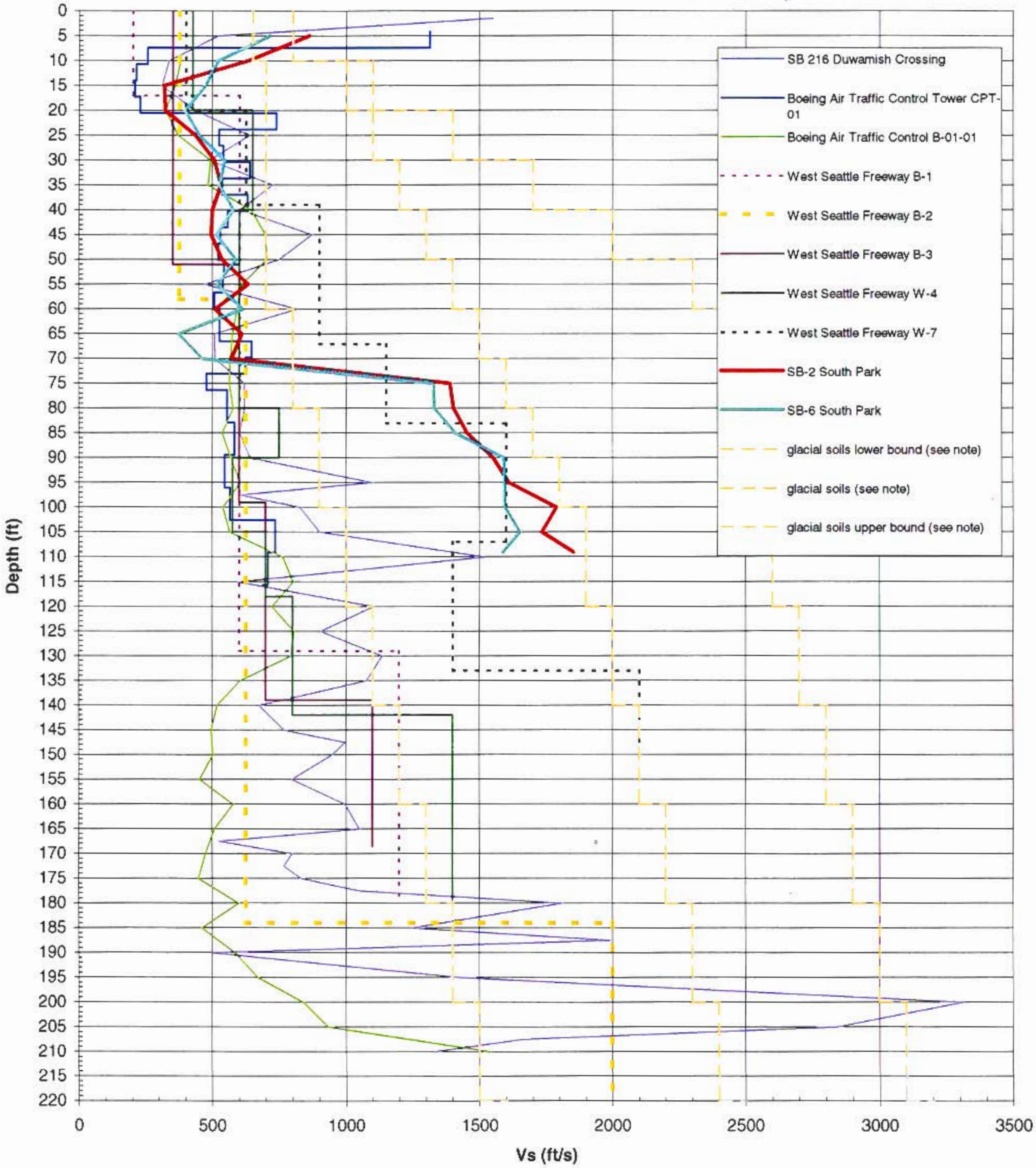
FIG. 6

File: I:\Drafting\21\09584-008\21-1-09584-008 fig 06.dwg Date: 03-30-2004 Author: CNT



Note: Trend of Vs of glacial soils from Shannon & Wilson projects in the Puget Sound.

South Park Bridge
Seattle, Washington

MEASURED SHEAR WAVE VELOCITY vs. DEPTH

March 2004 21-1-09584-008

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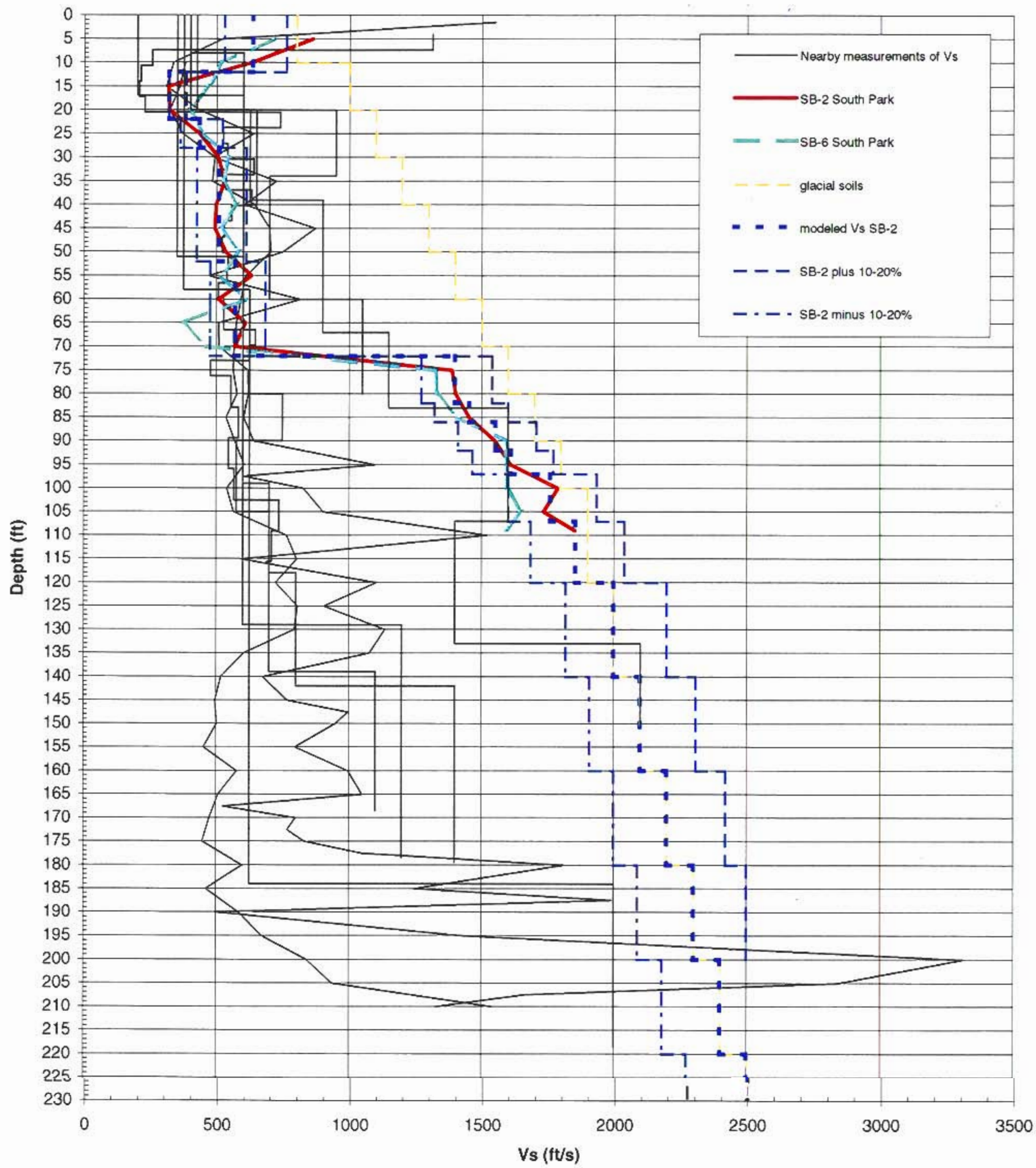
FIG. 7



South Park Bridge
Seattle, Washington

MODELED SHEAR WAVE VELOCITY vs. DEPTH at SB-2

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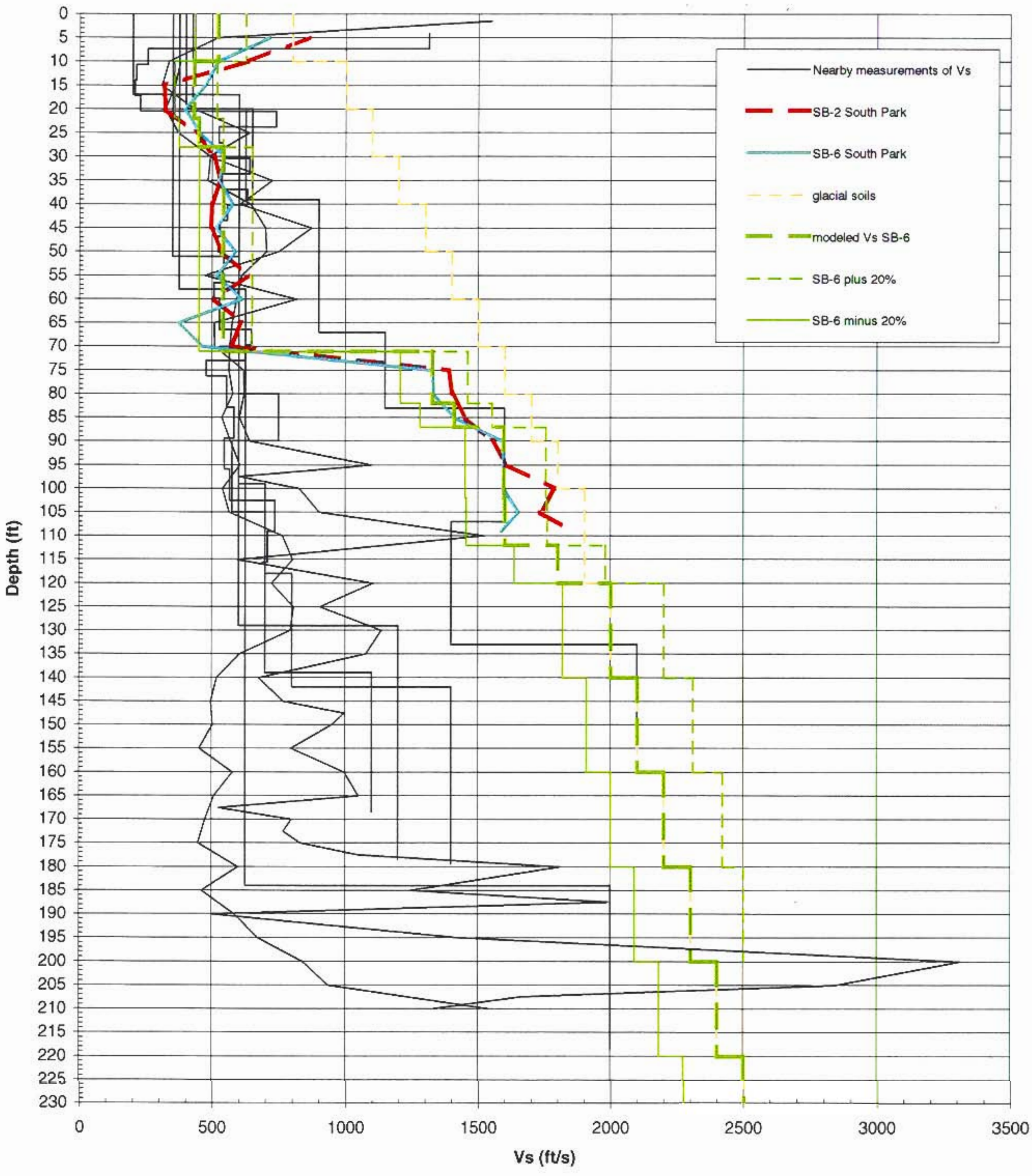
FIG. 8a



South Park Bridge
Seattle, Washington

MODELED SHEAR WAVE VELOCITY vs. DEPTH AT SB-6

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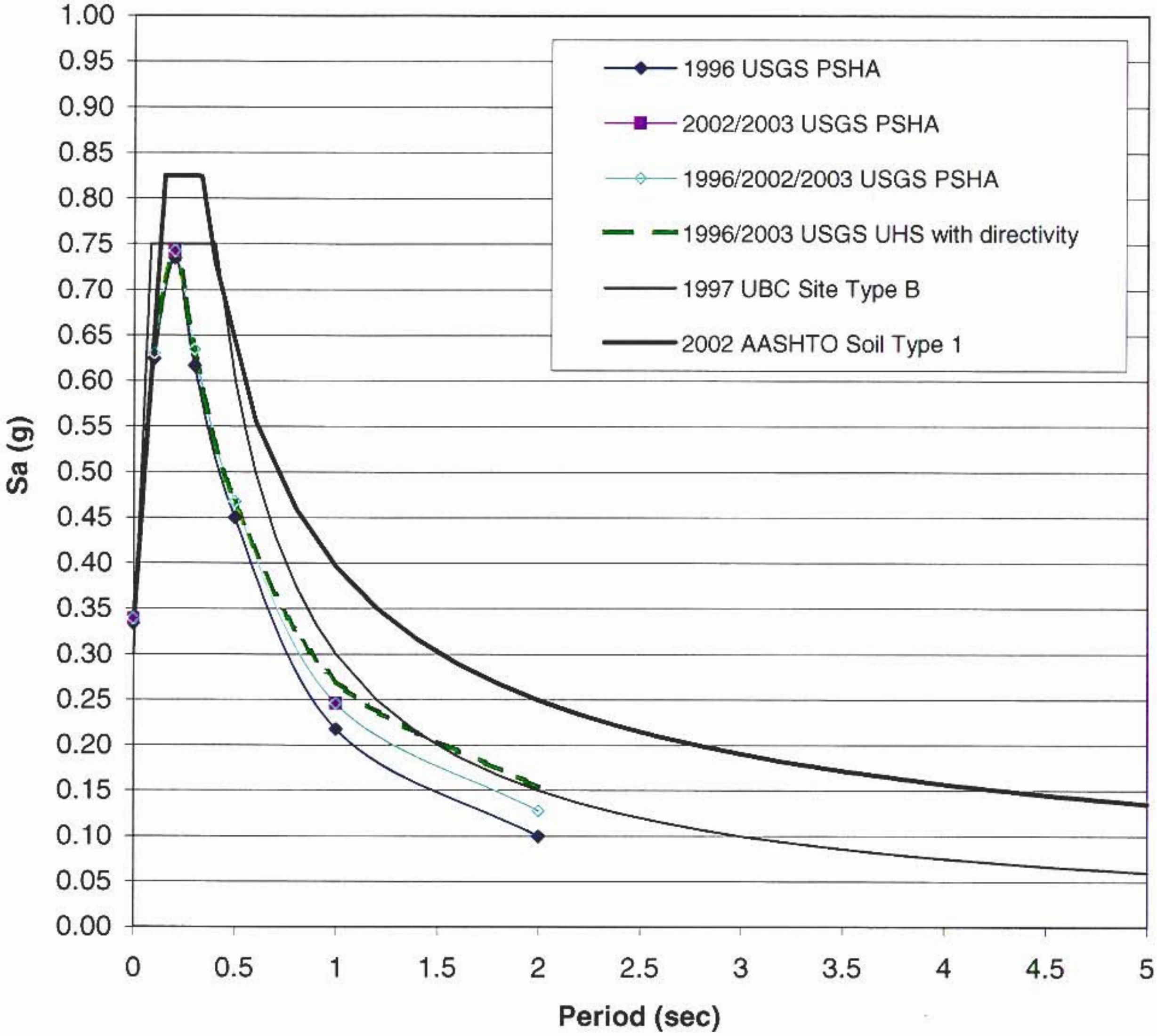
FIG. 8b



South Park Bridge
Seattle, Washington

COMPARISON OF ROCK LEVEL DESIGN RESPONSE SPECTRUM

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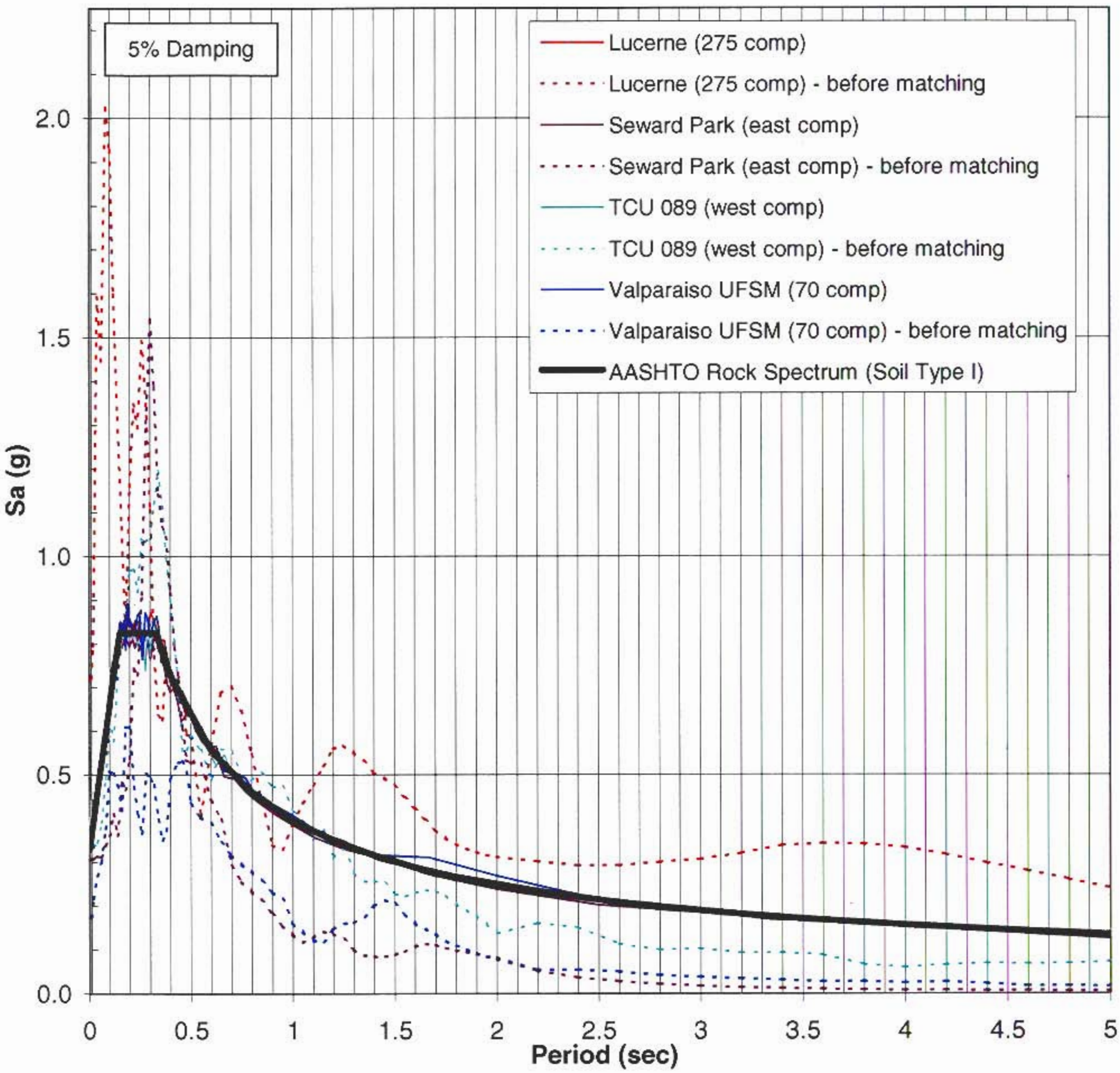
FIG. 9



South Park Bridge
Seattle, Washington

ROCK INPUT MOTIONS SPECTRALLY MATCHED TO AASTHO ROCK SPECTRUM

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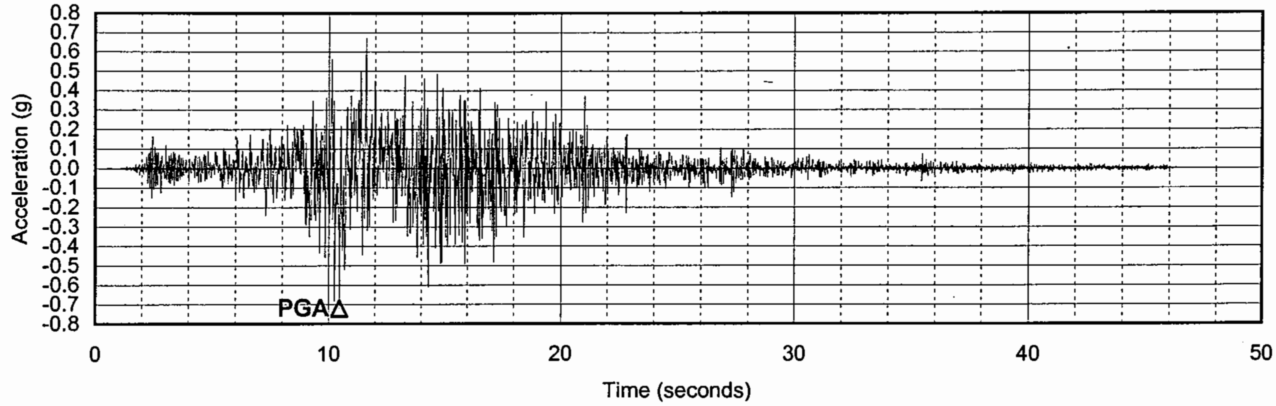
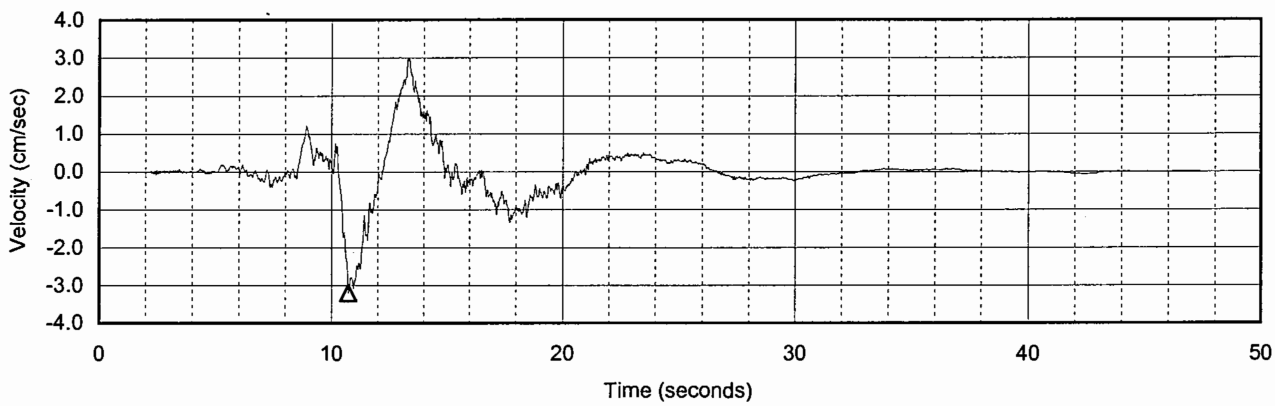
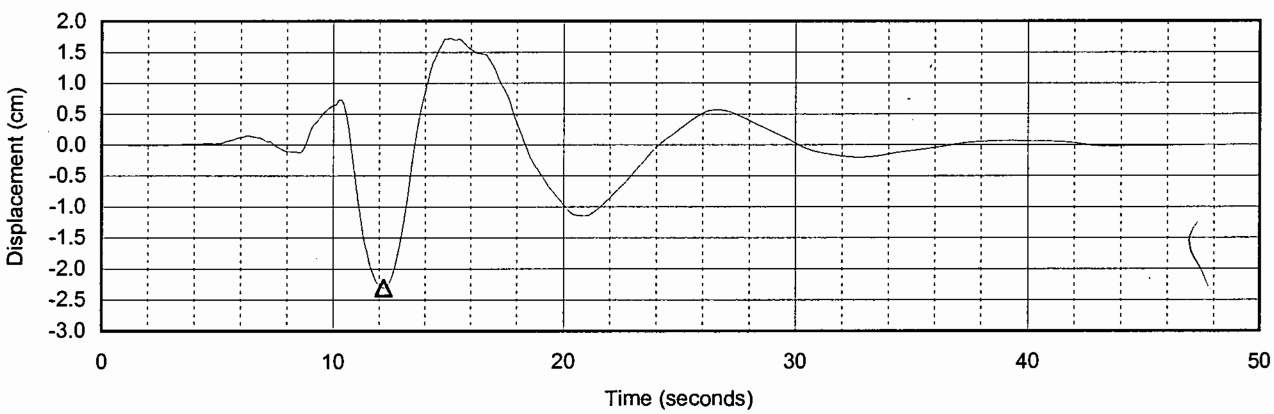
FIG. 10







Peak Values

Acceleration	0.72 g
Velocity	3.2 cm/sec
Displacement	2.3 cm

South Park Bridge
Seattle, Washington

HORIZONTAL SOIL MOTION
Lucerne (275 comp) original data

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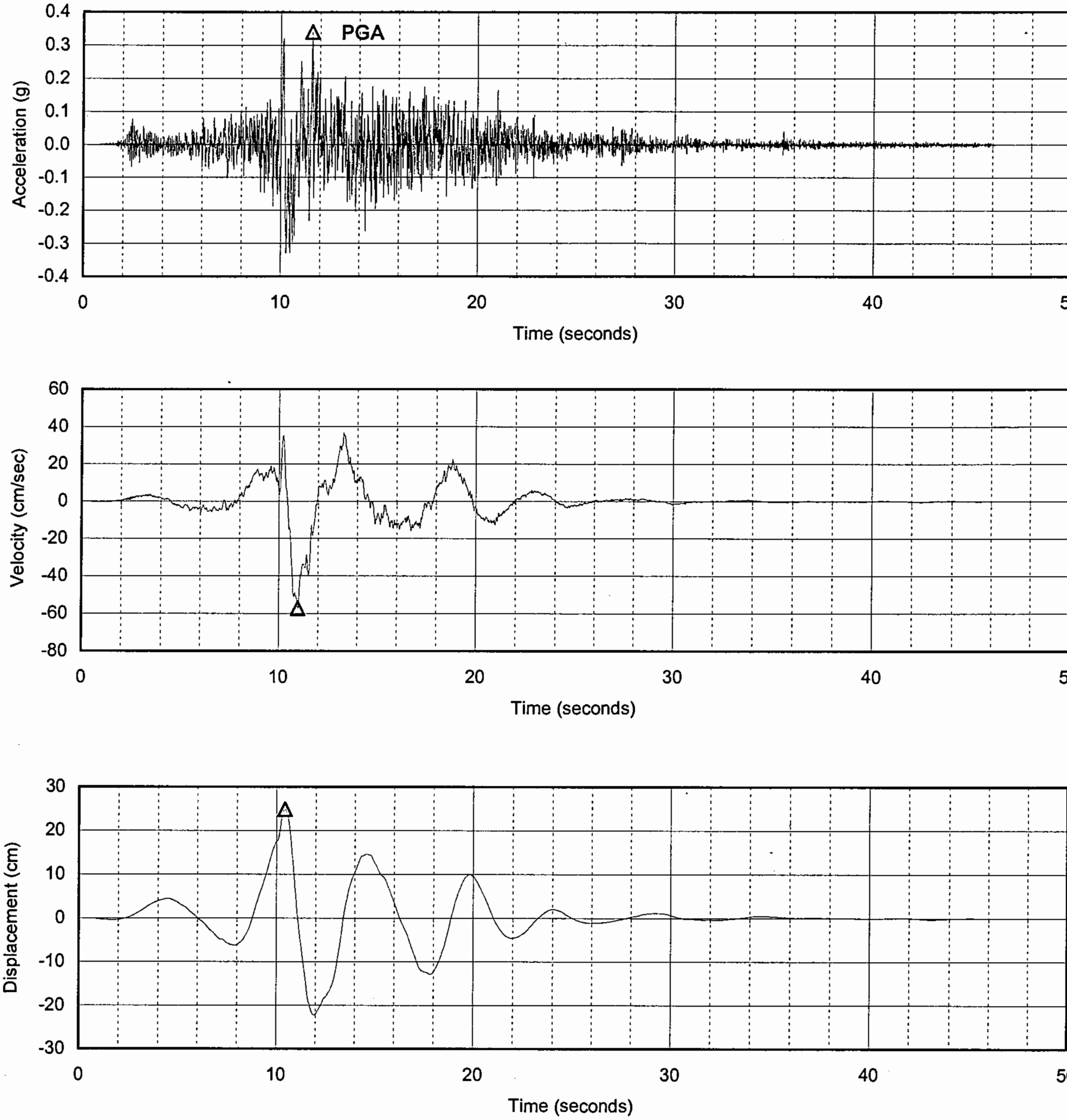
FIG. 11a



5% taper

Peak Values

Acceleration	0.34 g
Velocity	57.3 cm/sec
Displacement	24.9 cm

South Park Bridge
Seattle, Washington

HORIZONTAL SOIL MOTION
Lucerne (275 comp)
5 degree polynomial

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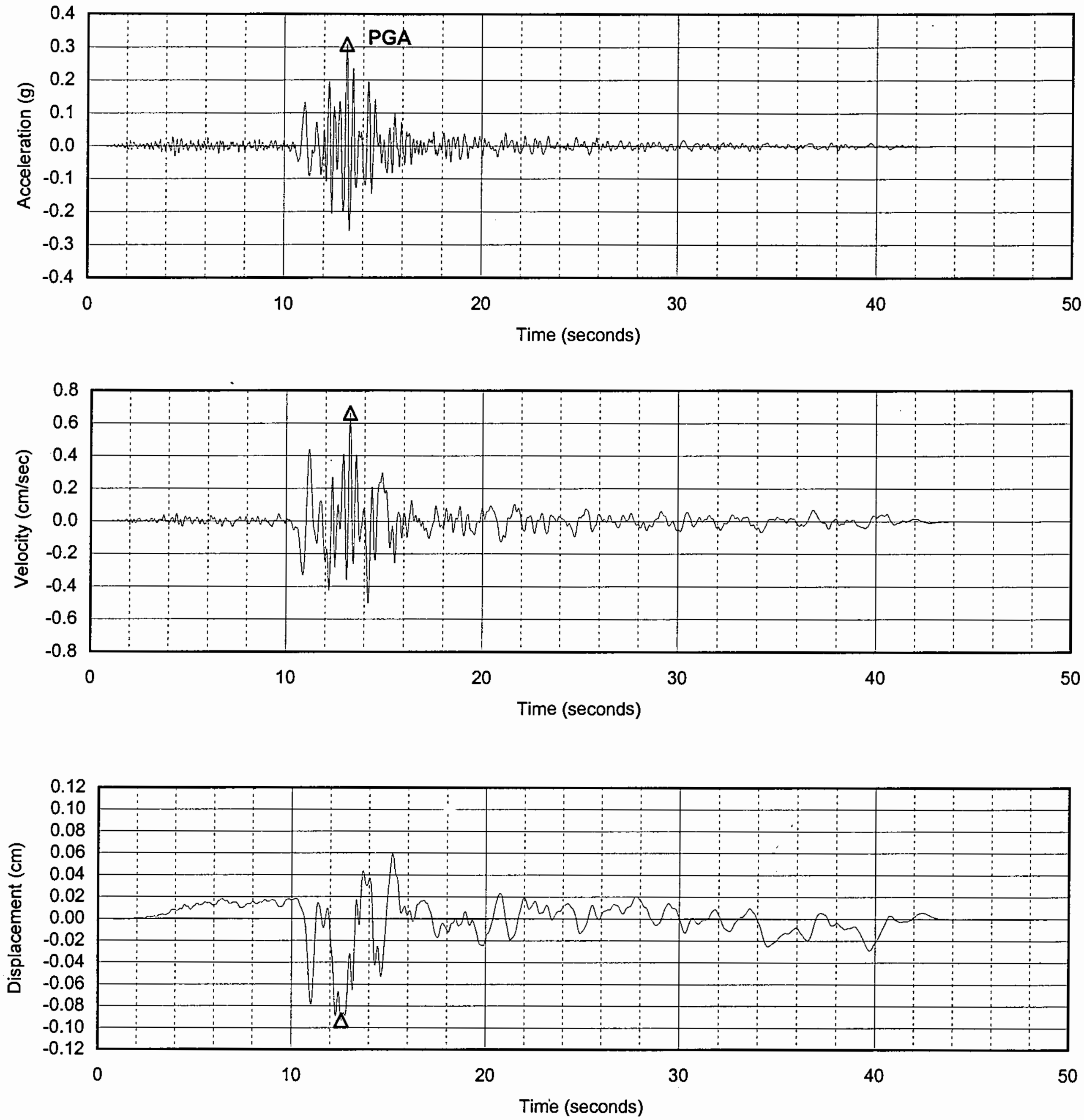
FIG. 11b



Peak Values

Acceleration	0.31 g
Velocity	0.7 cm/sec
Displacement	0.1 cm

South Park Bridge
Seattle, Washington

HORIZONTAL SOIL MOTION
Seward Park (east comp)
original data

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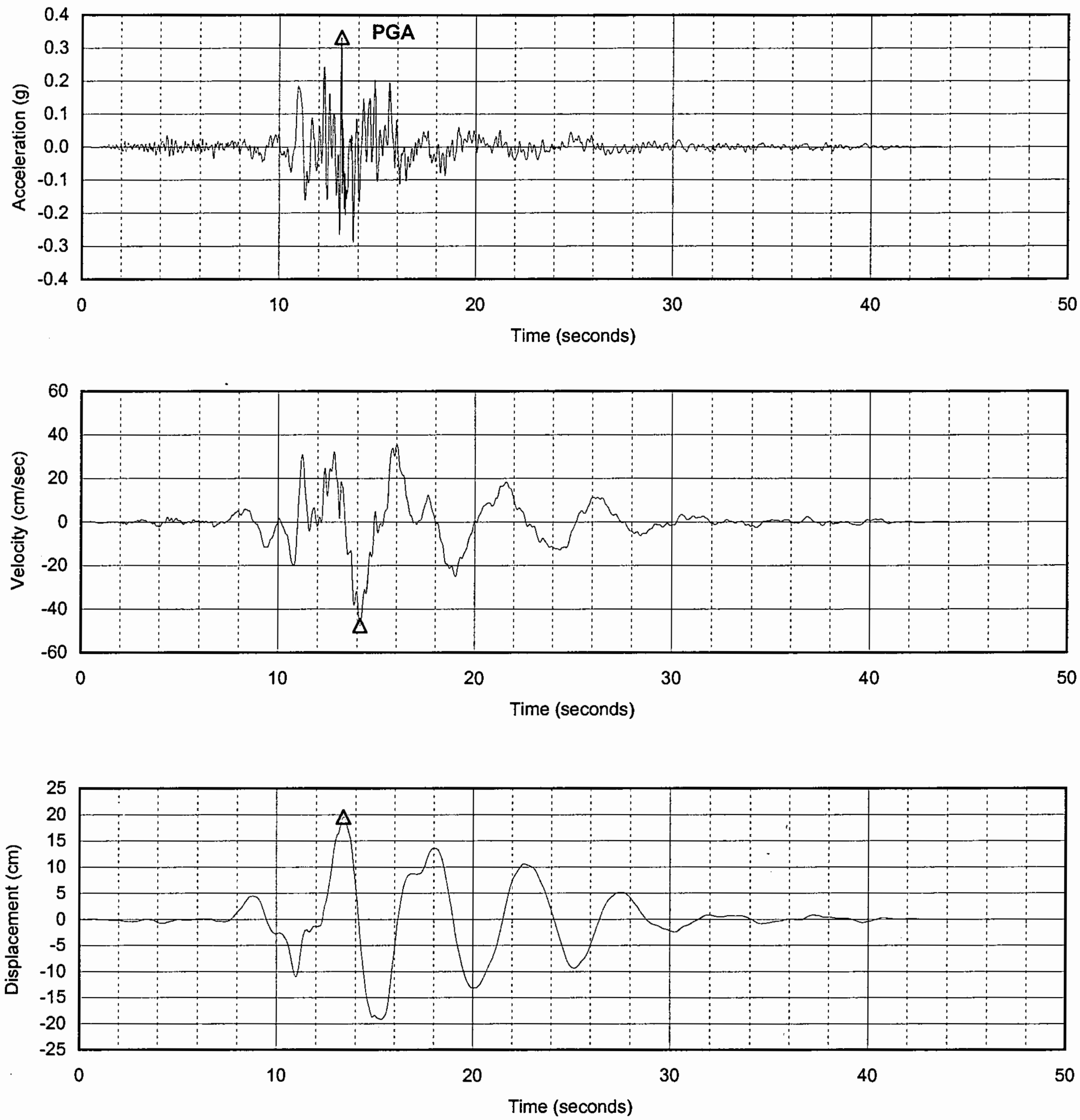
FIG. 12a



5% taper

Peak Values

Acceleration	0.33 g
Velocity	47.6 cm/sec
Displacement	19.6 cm

South Park Bridge
Seattle, Washington

HORIZONTAL SOIL MOTION
Seward Park (east comp)
9 degree polynomial

March 2004 21-1-09584-008

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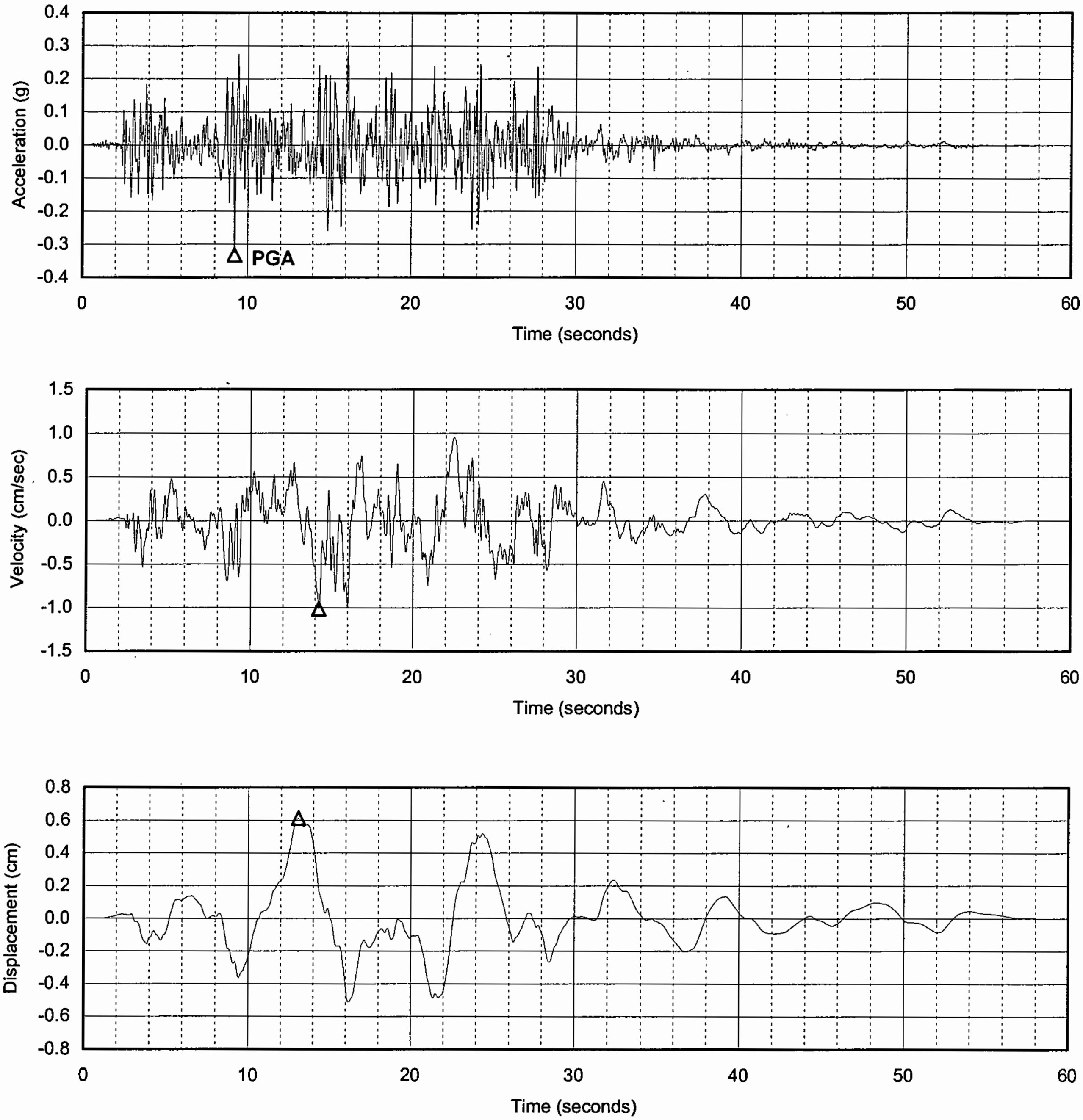
FIG. 12b



Peak Values

Acceleration	0.33 g
Velocity	1.0 cm/sec
Displacement	0.6 cm

South Park Bridge
Seattle, Washington

HORIZONTAL SOIL MOTION
TCU 089 (west comp)
original data

March 2004 21-1-09584-008

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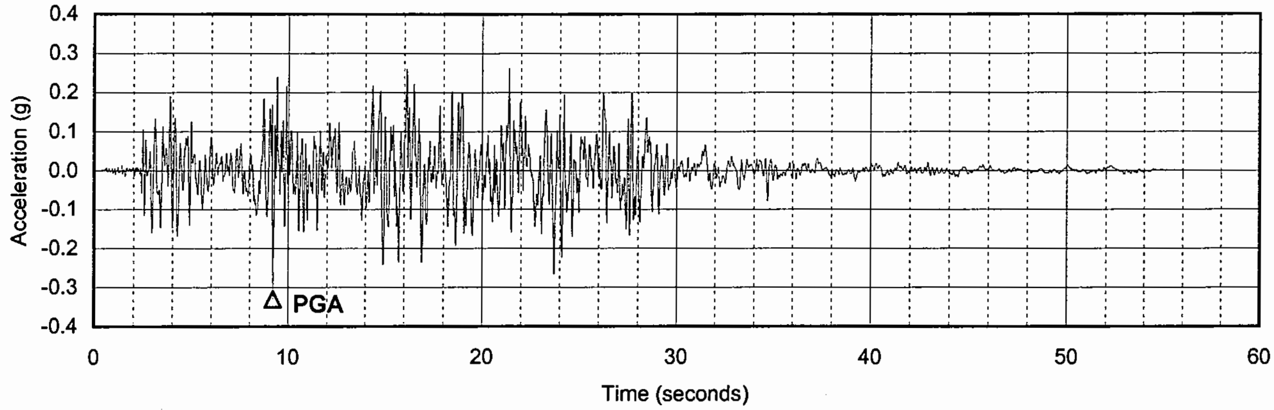
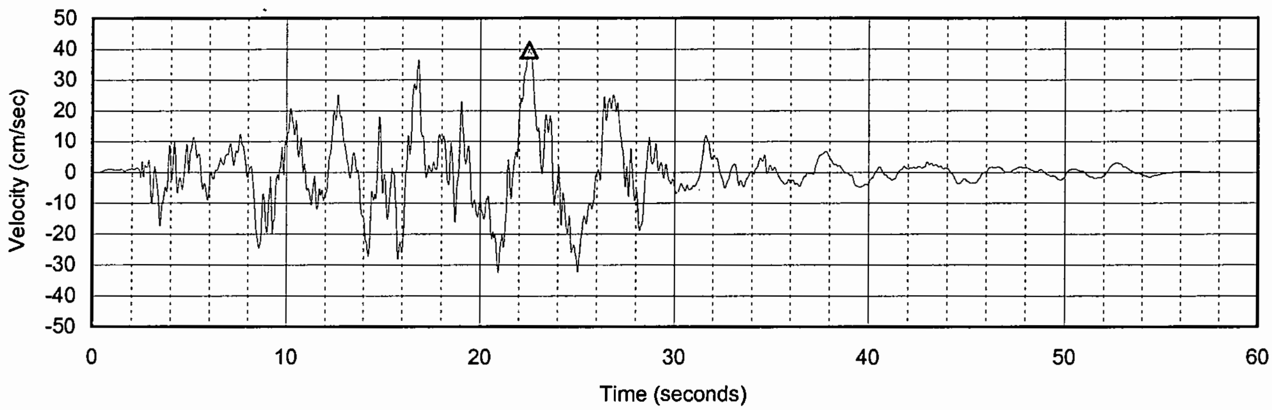
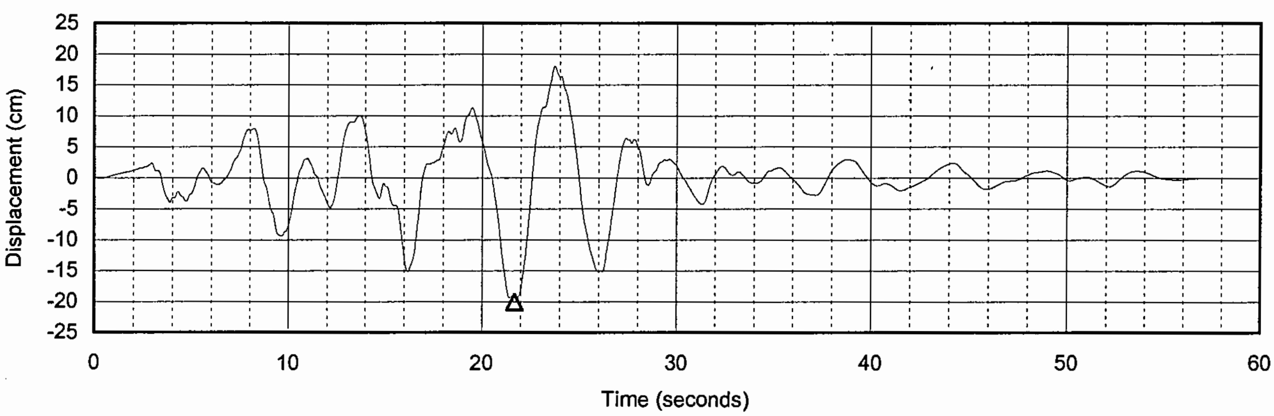
FIG. 13a







5% taper

Peak Values

Acceleration	0.33 g
Velocity	39.7 cm/sec
Displacement	20.1 cm

South Park Bridge
Seattle, Washington

HORIZONTAL SOIL MOTION
TCU 089 (west comp)
9 degree polynomial

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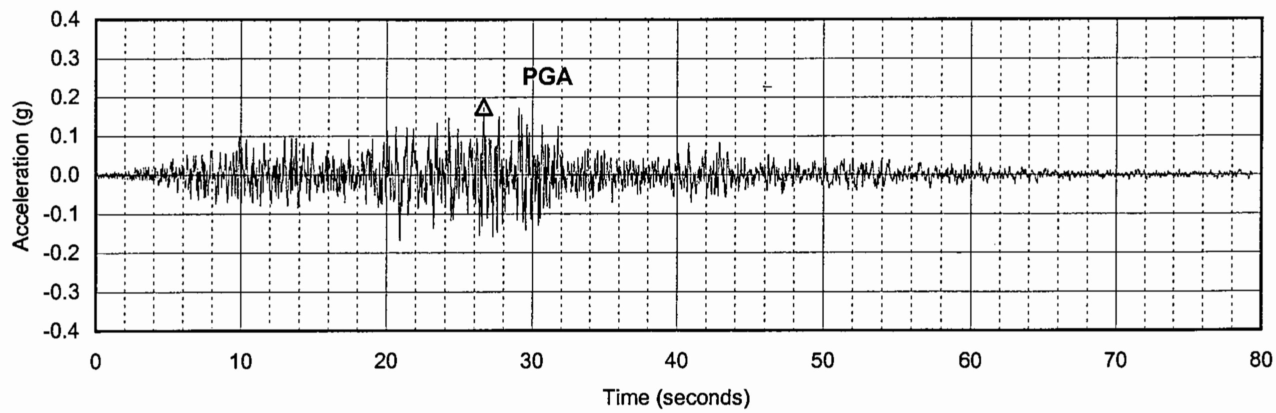
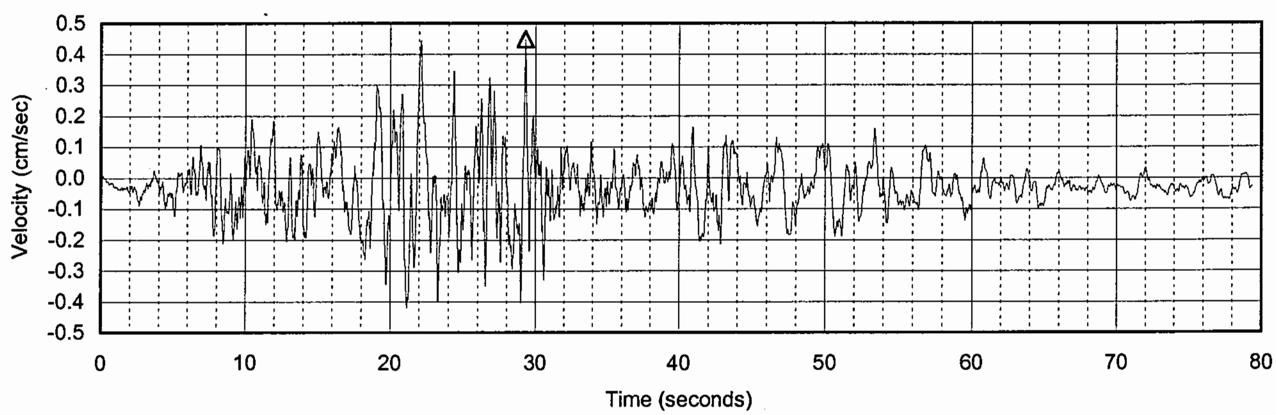
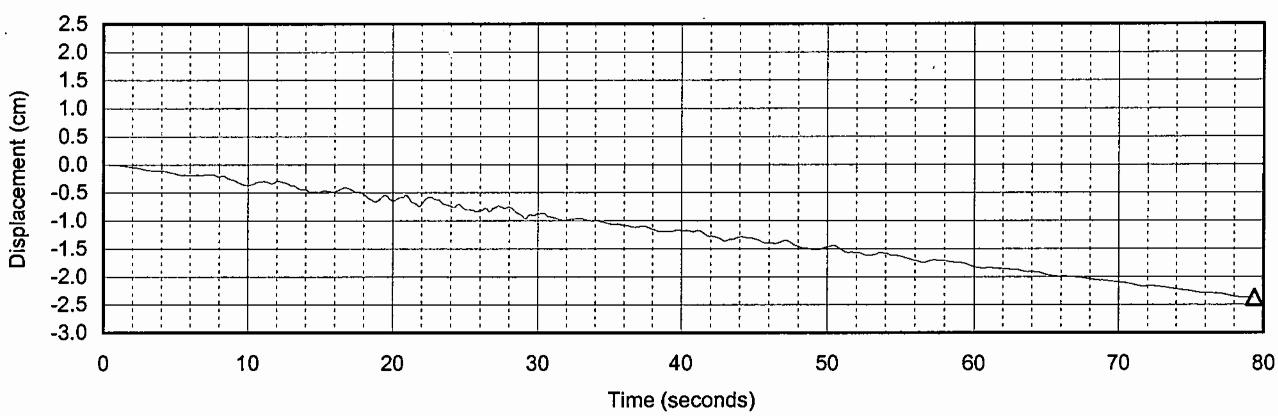
FIG. 13b







Peak Values

Acceleration	0.18 g
Velocity	0.4 cm/sec
Displacement	2.4 cm

South Park Bridge
Seattle, Washington

HORIZONTAL SOIL MOTION
Valparaiso UFSM (70 comp)
original data

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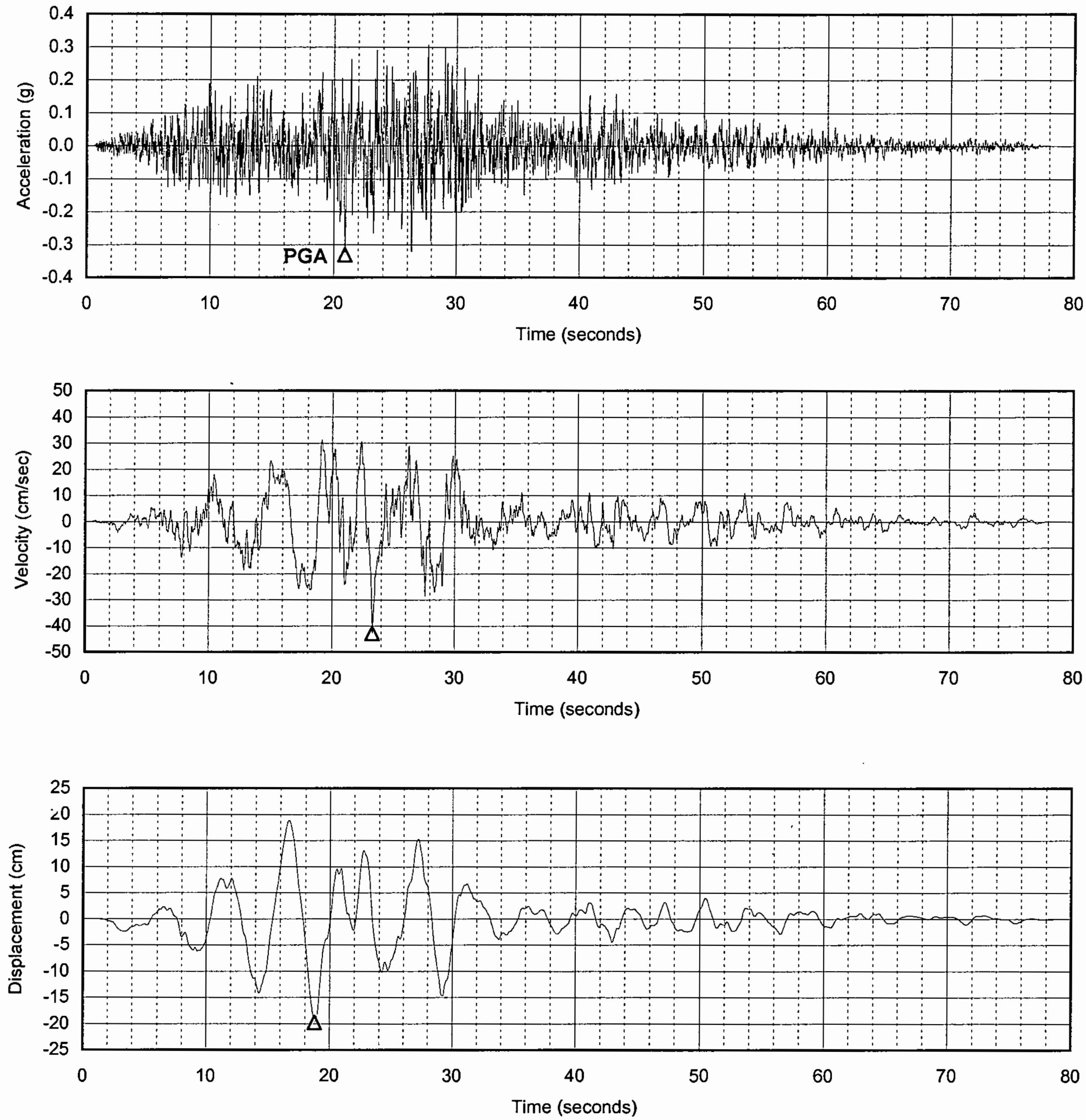
FIG. 14a



5% taper

Peak Values

Acceleration	0.33 g
Velocity	42.8 cm/sec
Displacement	19.8 cm

South Park Bridge
Seattle, Washington

HORIZONTAL SOIL MOTION
Valparaiso UFSM (70 comp)
5 degree polynomial

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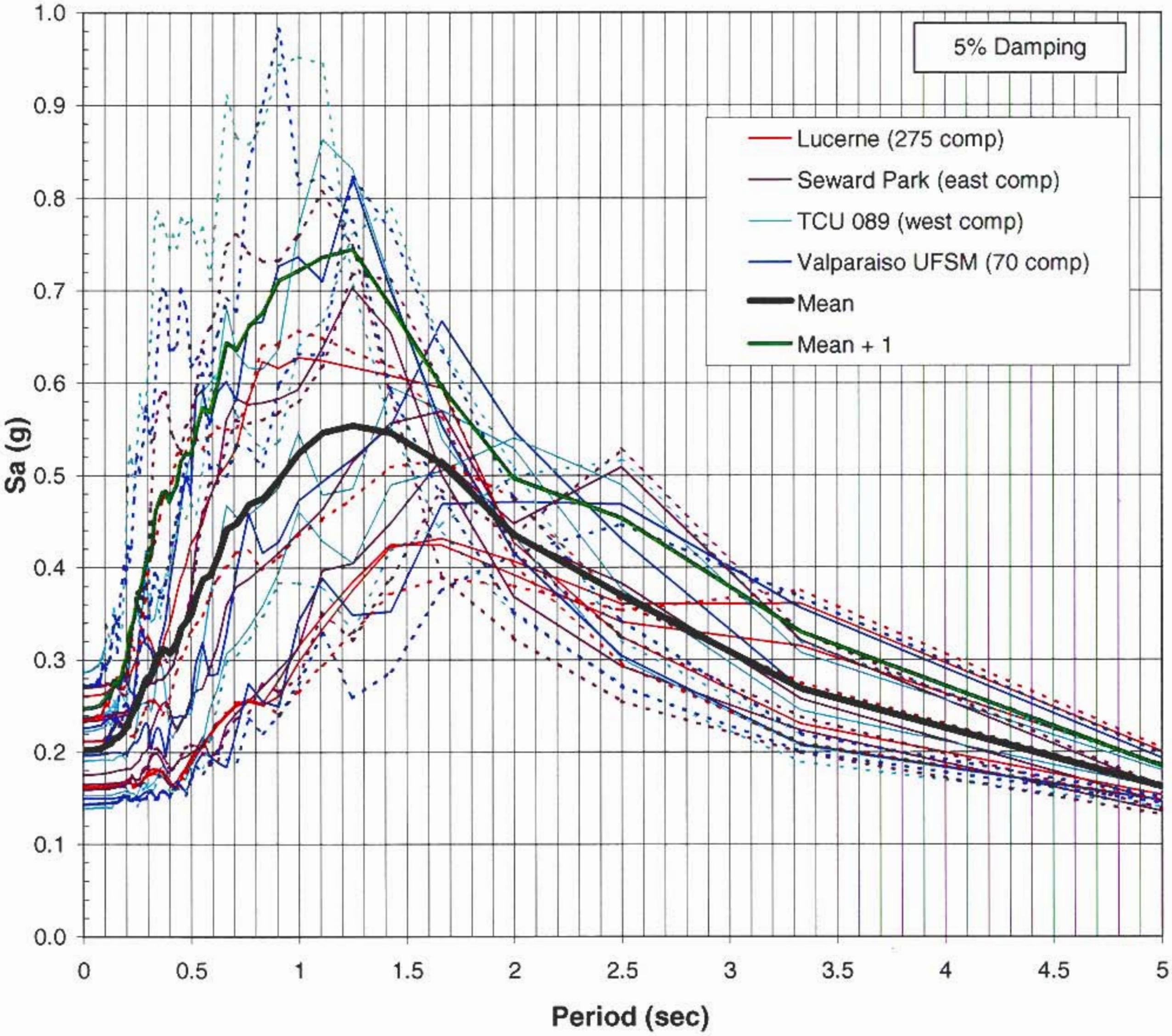
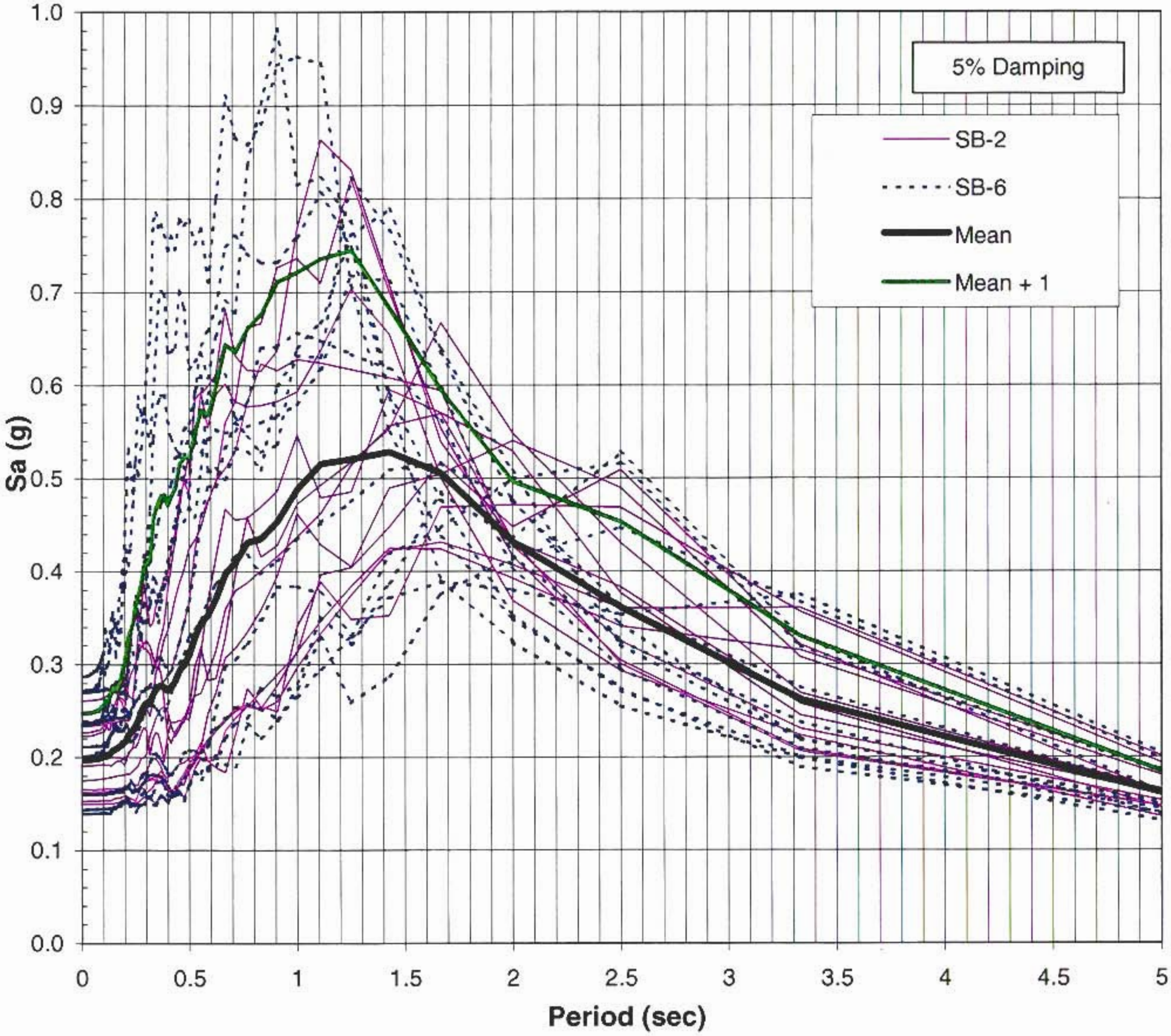
FIG. 14b



NOTES:

1. Response spectra for Borings SB-2 and SB-6 are represented by solid lines and dashed lines, respectively.
2. Response spectra represent horizontal free-field ground motions.

South Park Bridge Seattle, Washington	
SOIL SURFACE RESPONSE SPECTRA EFFECTS OF INPUT MOTIONS	
March 2004	21-1-09584-008
SHANNON & WILSON, INC. Geotechnical and Environmental Consultants	**FIG. 15a**



NOTES:

1. Response spectra for Borings SB-2 and SB-6 are represented by solid lines and dashed lines, respectively.
2. Response spectra represent horizontal free-field ground motions.

South Park Bridge
Seattle, Washington

SOIL SURFACE RESPONSE SPECTRA DIFFERENCE BETWEEN SB-2 AND SB-6

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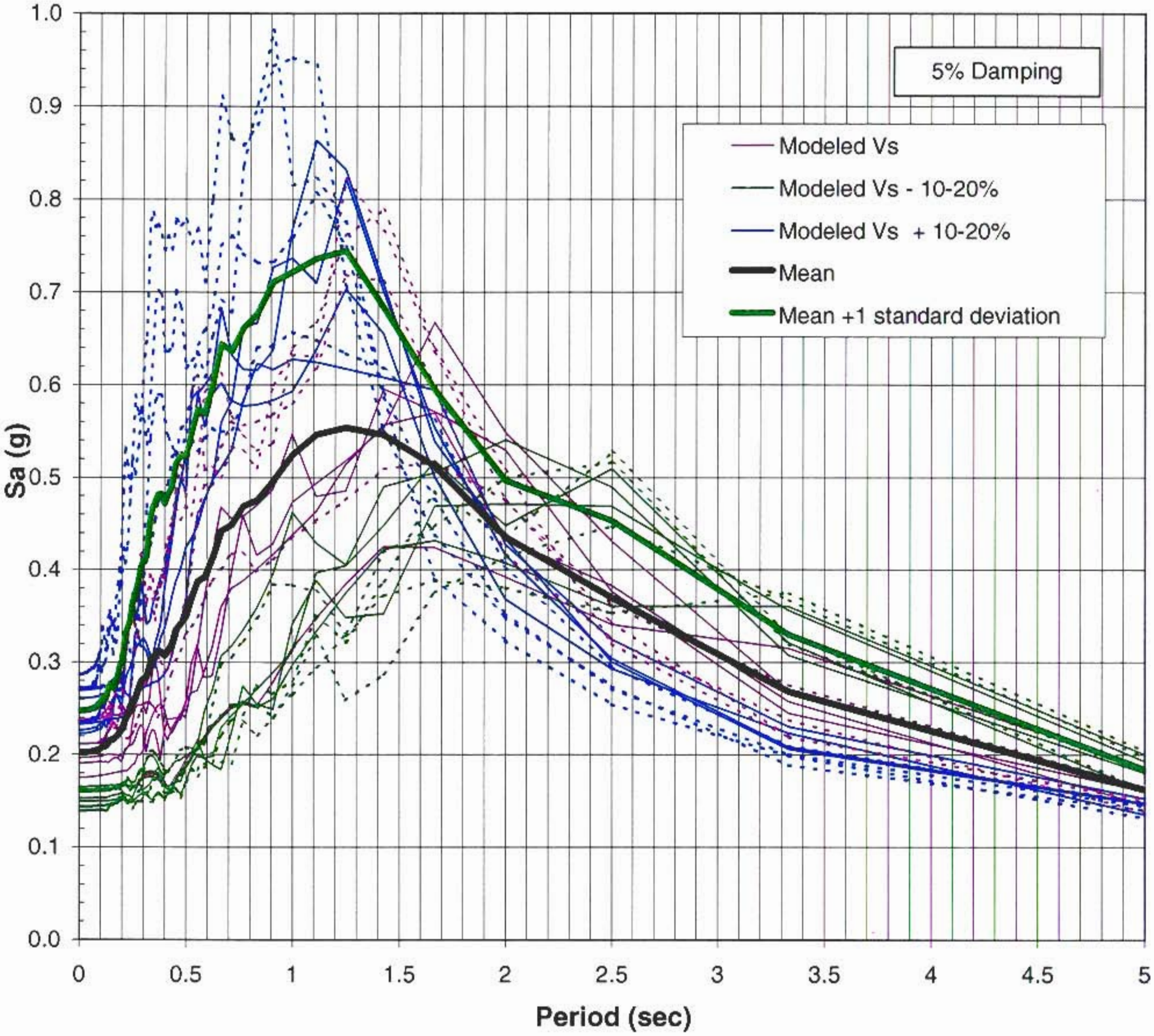
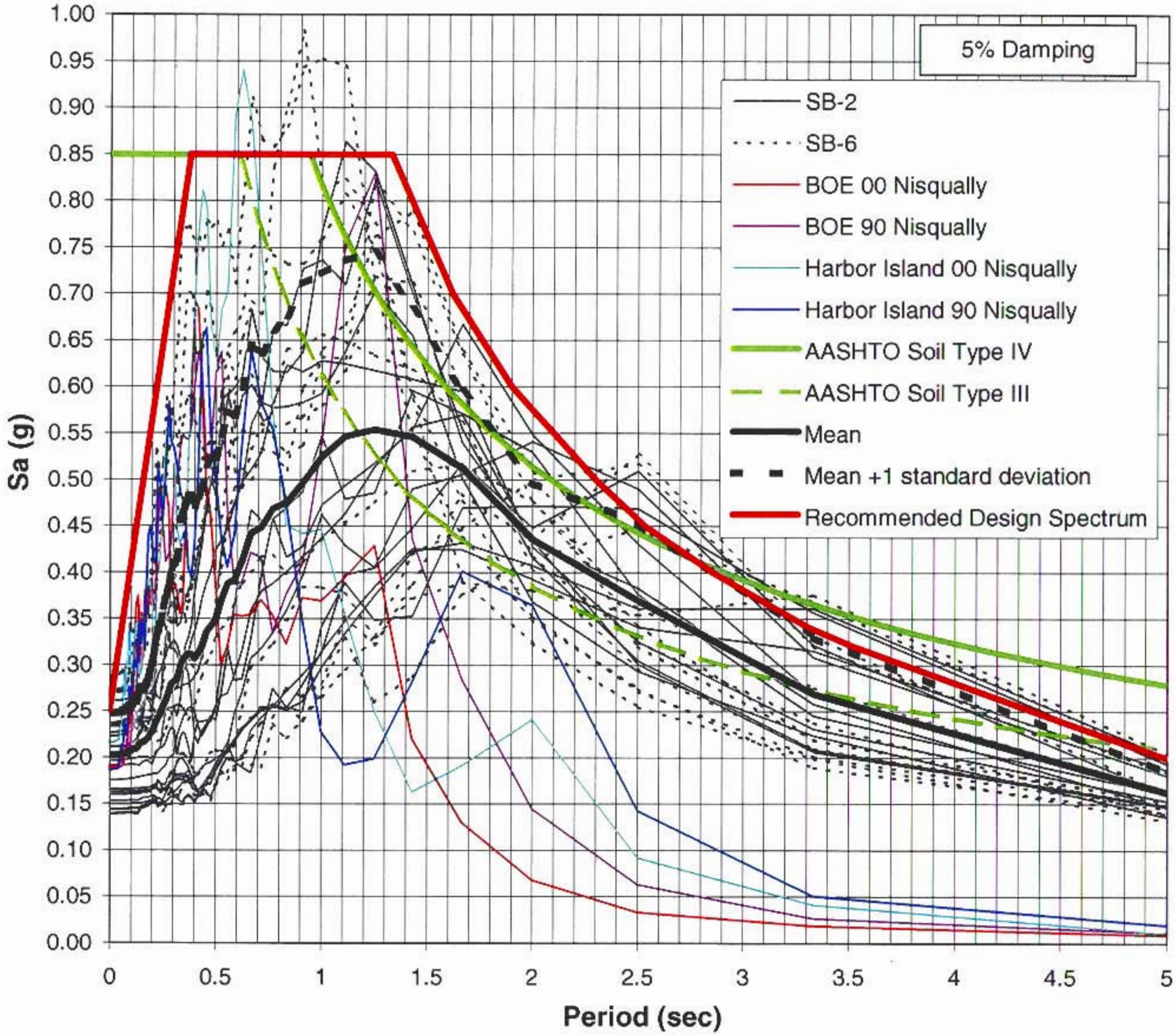
FIG. 15b



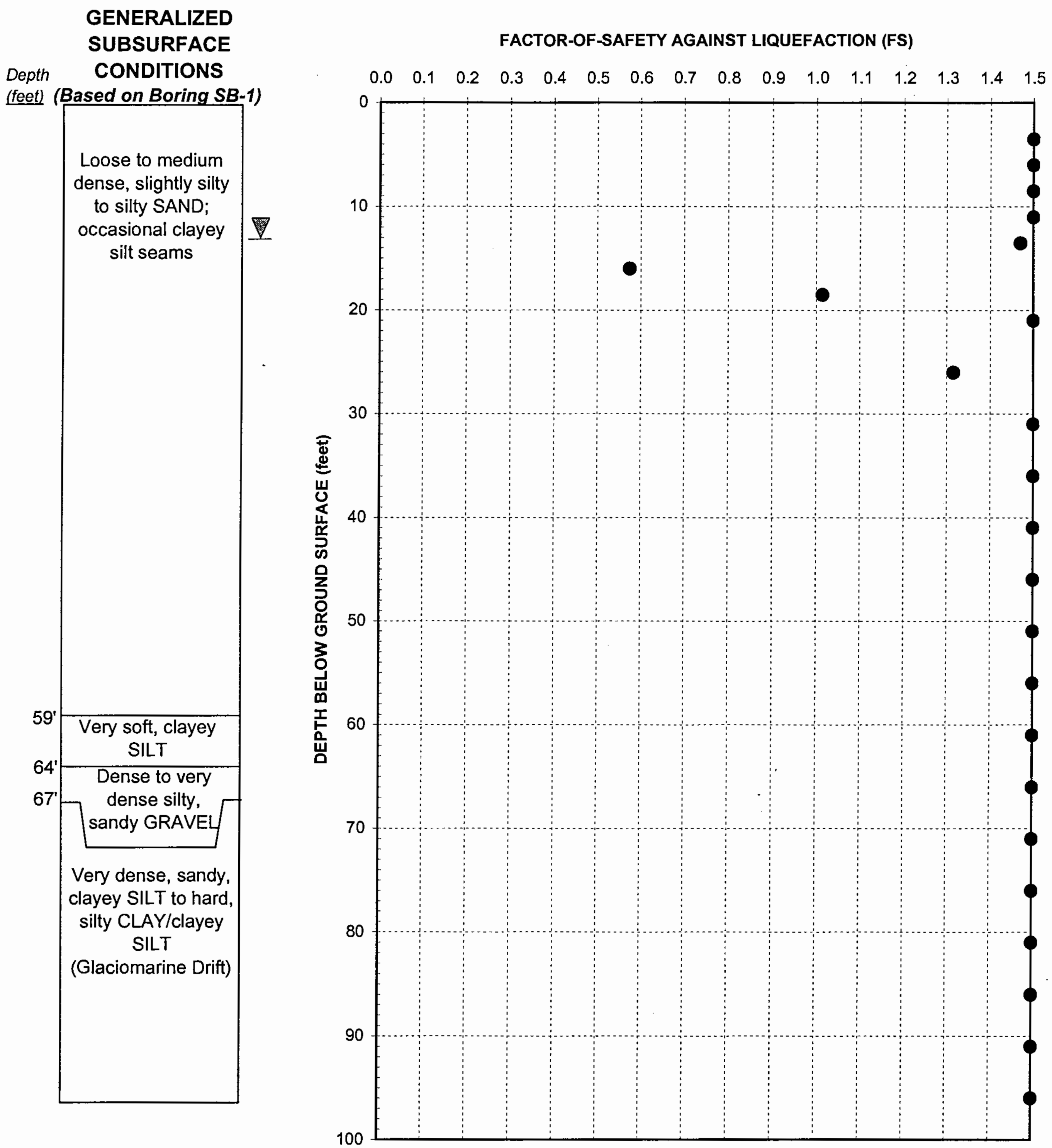
NOTES:

1. Response spectra for Borings SB-2 and SB-6 are represented by solid lines and dashed lines, respectively.
2. Response spectra represent horizontal free-field ground motions.

South Park Bridge Seattle, Washington	
SOIL SURFACE RESPONSE SPECTRA EFFECT OF SHEAR WAVE VELOCITY	
March 2004	21-1-09584-008
SHANNON & WILSON, INC. Geotechnical and Environmental Consultants	**FIG.15c**



South Park Bridge Seattle, Washington	
SOIL SURFACE RESPONSE SPECTRA **RECOMMENDED DESIGN SPECTRUM**	
March 2004	21-1-09584-008
SHANNON & WILSON, INC. Geotechnical and Environmental Consultants	**FIG. 16**



NOTES:

1. *Reference:* Youd, T.L. and Idriss, I.M., 2001, Liquefaction Resistance of Soils: Summary Report from the 1996 NCEER and 1998 NCEER/ NSF Workshops on Evaluation of Liquefaction Resistance of Soils.
2. The analysis was performed using the Cyclic Stress Ratio (CSR) values determined in the site-specific ground response analysis.
3. The liquefaction resistance of a soil is dependent on its density and fines content. The fines content was estimated based on selected grain-size analyses and engineering judgement.

South Park Bridge
Seattle, Washington

RESULTS OF LIQUEFACTION ANALYSES BORING SB-1

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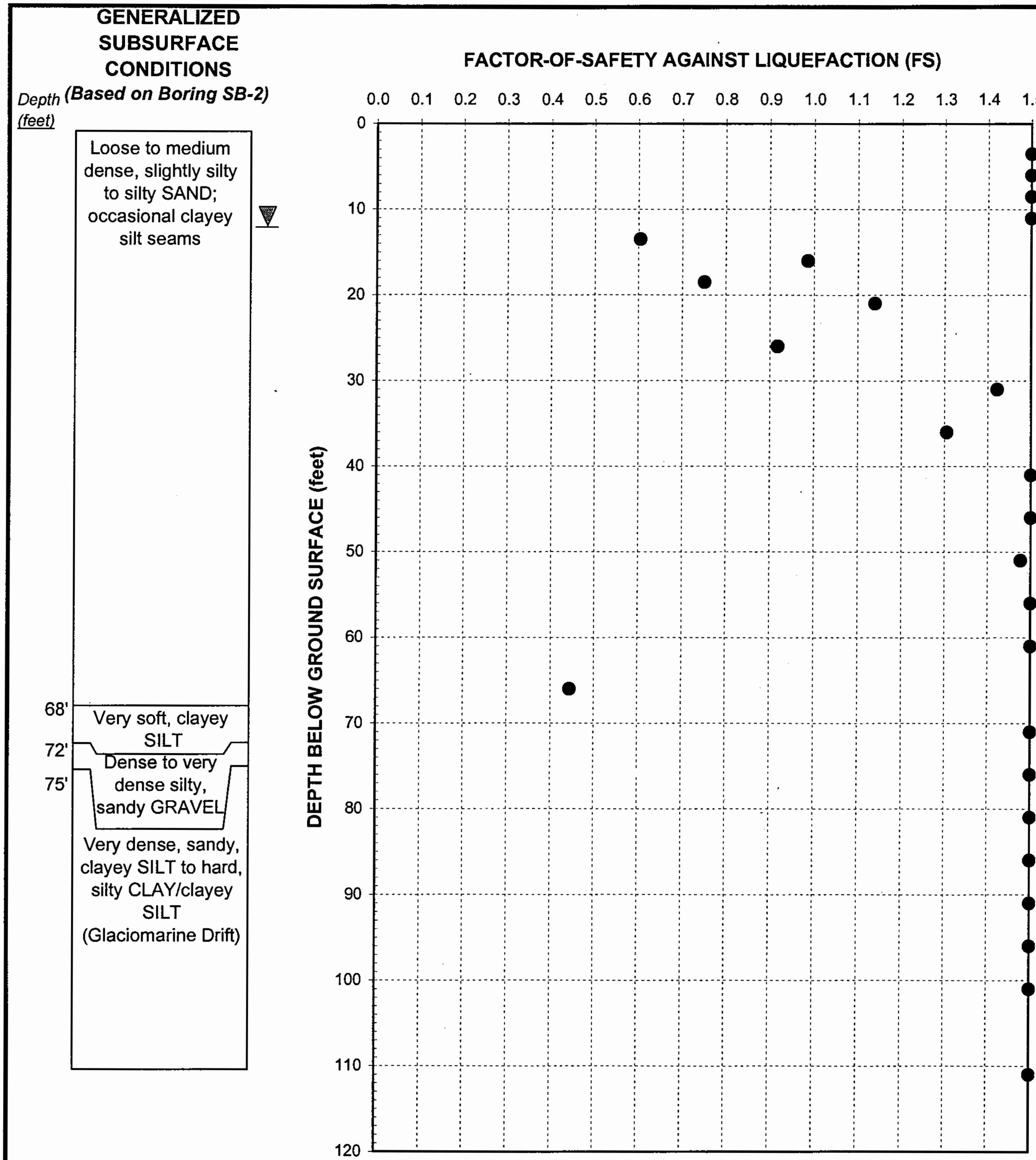
FIG. 17

3/29/2004-LIQspt_SB2_CSR.xls-lmm



NOTES:

1. *Reference:* Youd, T.L. and Idriss, I.M., 2001, Liquefaction Resistance of Soils: Summary Report from the 1996 NCEER and 1998 NCEER/NSF Workshops on Evaluation of Liquefaction Resistance of Soils.
2. The analysis was performed using the Cyclic Stress Ratio (CSR) values determined in the site-specific ground response analysis.
3. The liquefaction resistance of a soil is dependent on its density and fines content. The fines content was estimated based on selected grain-size analyses and engineering judgement.

South Park Bridge
Seattle, Washington

RESULTS OF LIQUEFACTION ANALYSES BORING SB-2

March 2004 21-1-09584-008

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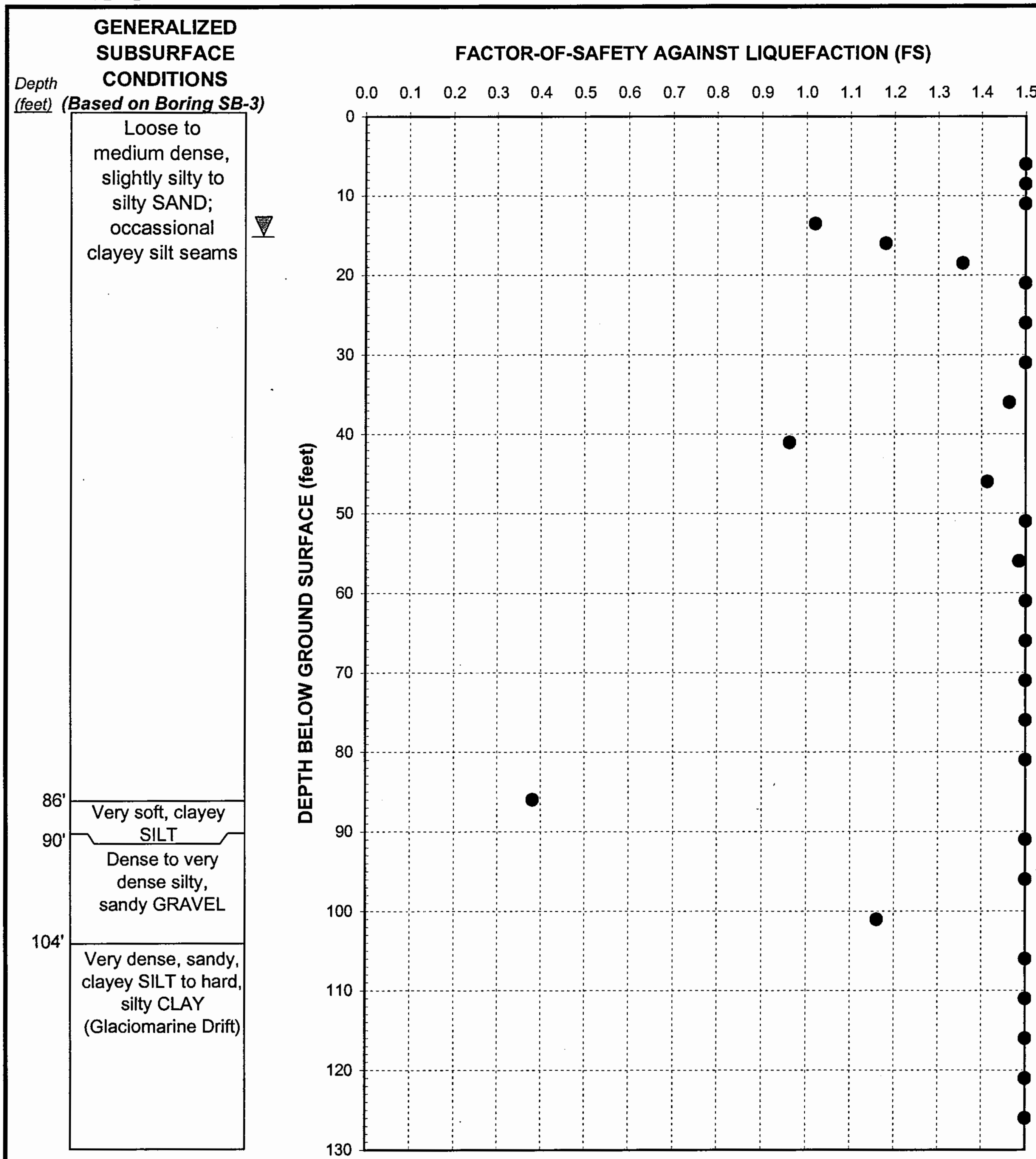
Geotechnical and Environmental Consultants

FIG. 18



NOTES:

1. *Reference:* Youd, T.L. and Idriss, I.M., 2001, Liquefaction Resistance of Soils: Summary Report from the 1996 NCEER and 1998 NCEER/NSF Workshops on Evaluation of Liquefaction Resistance of Soils.
2. The analysis was performed using the Cyclic Stress Ratio (CSR) values determined in the site-specific ground response analysis.
3. The liquefaction resistance of a soil is dependent on its density and fines content. The fines content was estimated based on selected grain-size analyses and engineering judgement.

South Park Bridge
Seattle, Washington

RESULTS OF LIQUEFACTION ANALYSES BORING SB-3

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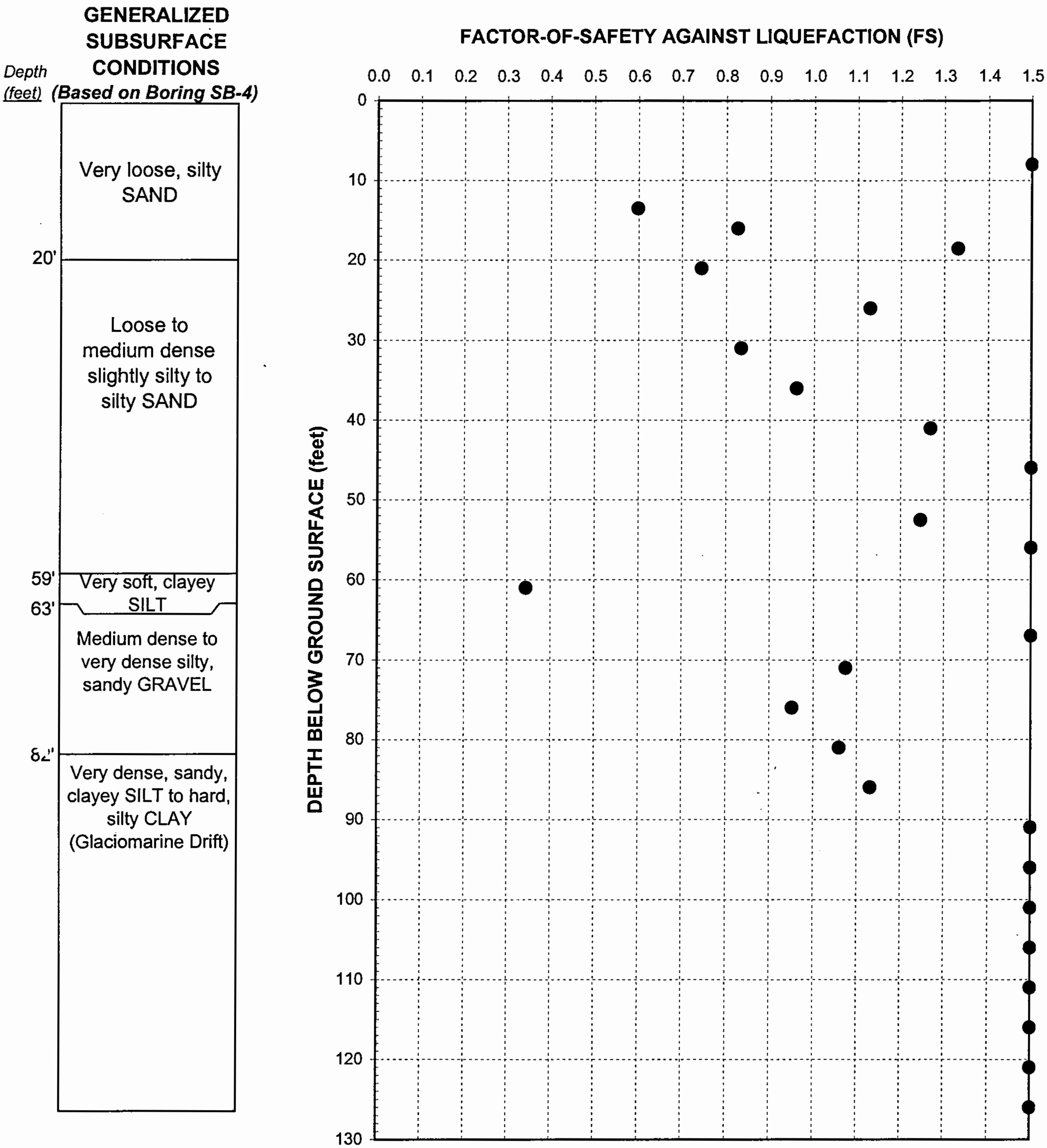
FIG. 19



NOTES:

1. *Reference:* Youd, T.L. and Idriss, I.M., 2001, Liquefaction Resistance of Soils: Summary Report from the 1996 NCEER and 1998 NCEER/ NSF Workshops on Evaluation of Liquefaction Resistance of Soils.
2. The analysis was performed using the Cyclic Stress Ratio (CSR) values determined in the site-specific ground response analysis.
3. The liquefaction resistance of a soil is dependent on its density and fines content. The fines content was estimated based on selected grain-size analyses and engineering judgement.

South Park Bridge
Seattle, Washington

RESULTS OF LIQUEFACTION ANALYSES BORING SB-4

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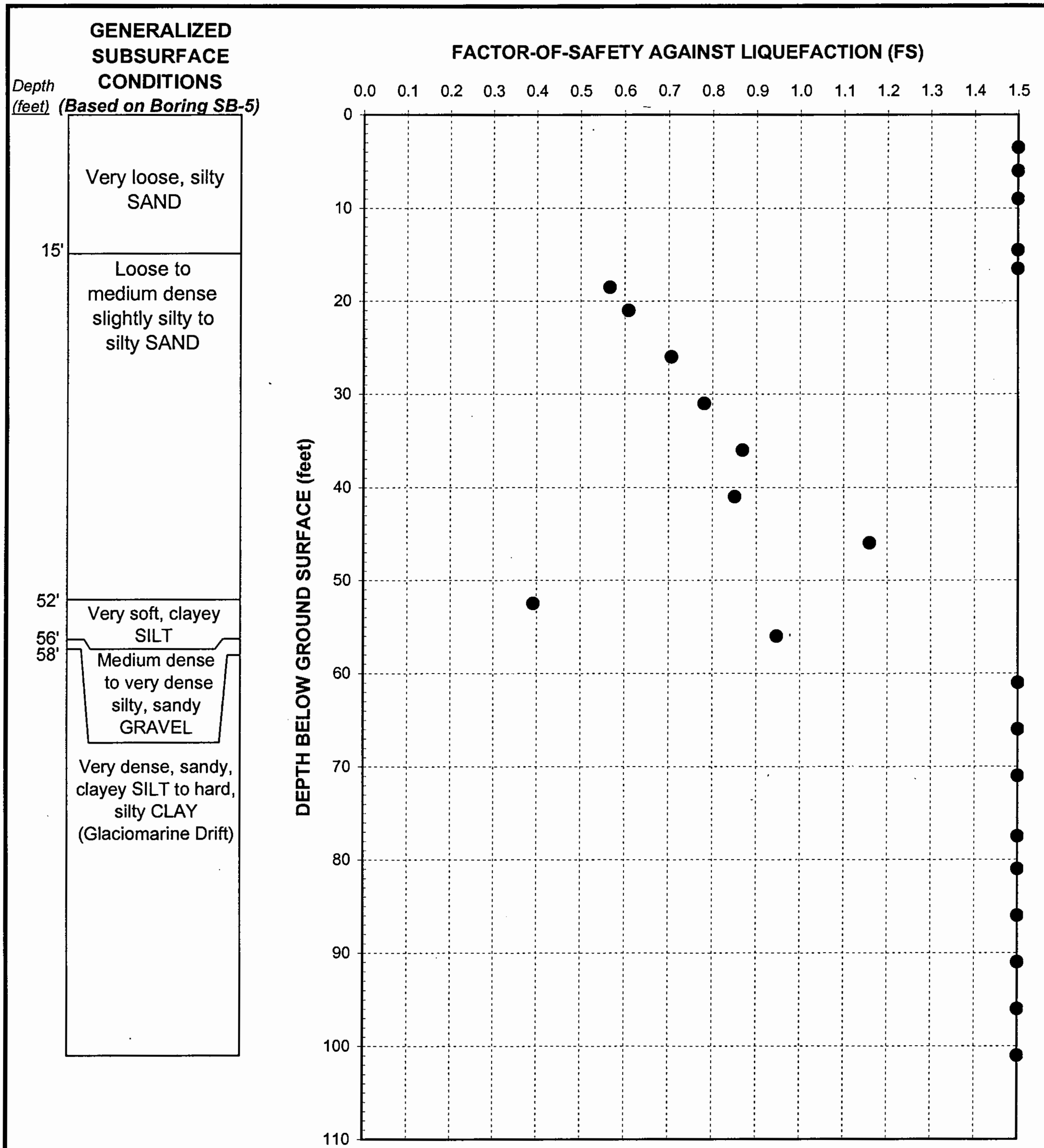
FIG. 20



NOTES:

1. *Reference:* Youd, T.L. and Idriss, I.M., 2001, Liquefaction Resistance of Soils: Summary Report from the 1996 NCEER and 1998 NCEER/ NSF Workshops on Evaluation of Liquefaction Resistance of Soils.
2. The analysis was performed using the Cyclic Stress Ratio (CSR) values determined in the site-specific ground response analysis.
3. The liquefaction resistance of a soil is dependent on its density and fines content. The fines content was estimated based on selected grain-size analyses and engineering judgement.

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Seattle, Washington

RESULTS OF LIQUEFACTION ANALYSES BORING SB-5

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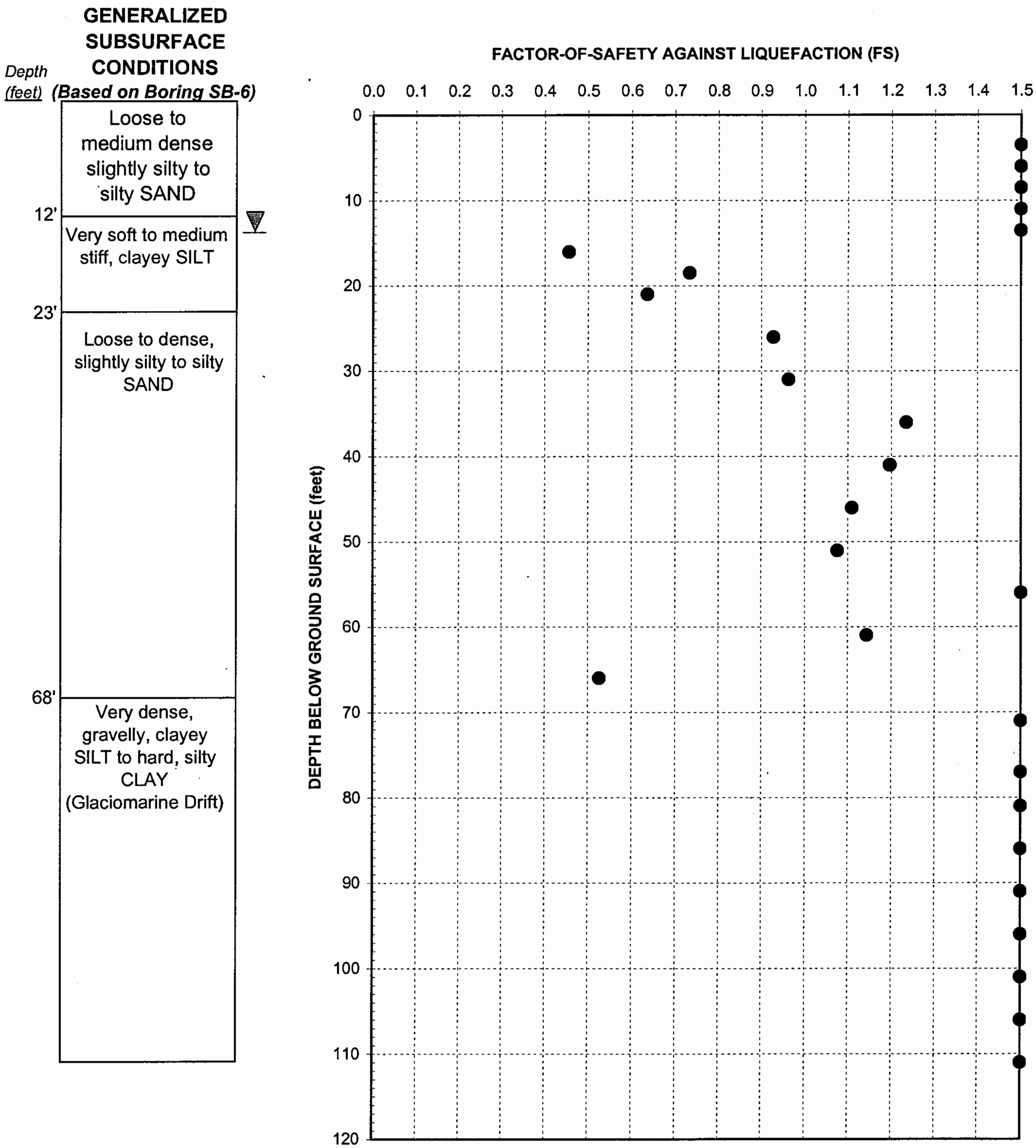
FIG. 21



NOTES:

1. *Reference:* Youd, T.L. and Idriss, I.M., 2001, Liquefaction Resistance of Soils: Summary Report from the 1996 NCEER and 1998 NCEER/ NSF Workshops on Evaluation of Liquefaction Resistance of Soils.
2. The analysis was performed using the Cyclic Stress Ratio (CSR) values determined in the site-specific ground response analysis.
3. The liquefaction resistance of a soil is dependent on its density and fines content. The fines content was estimated based on selected grain-size analyses and engineering judgement.

South Park Bridge
Seattle, Washington

RESULTS OF LIQUEFACTION ANALYSES BORING SB-6

March 2004 | 21-1-09584-008

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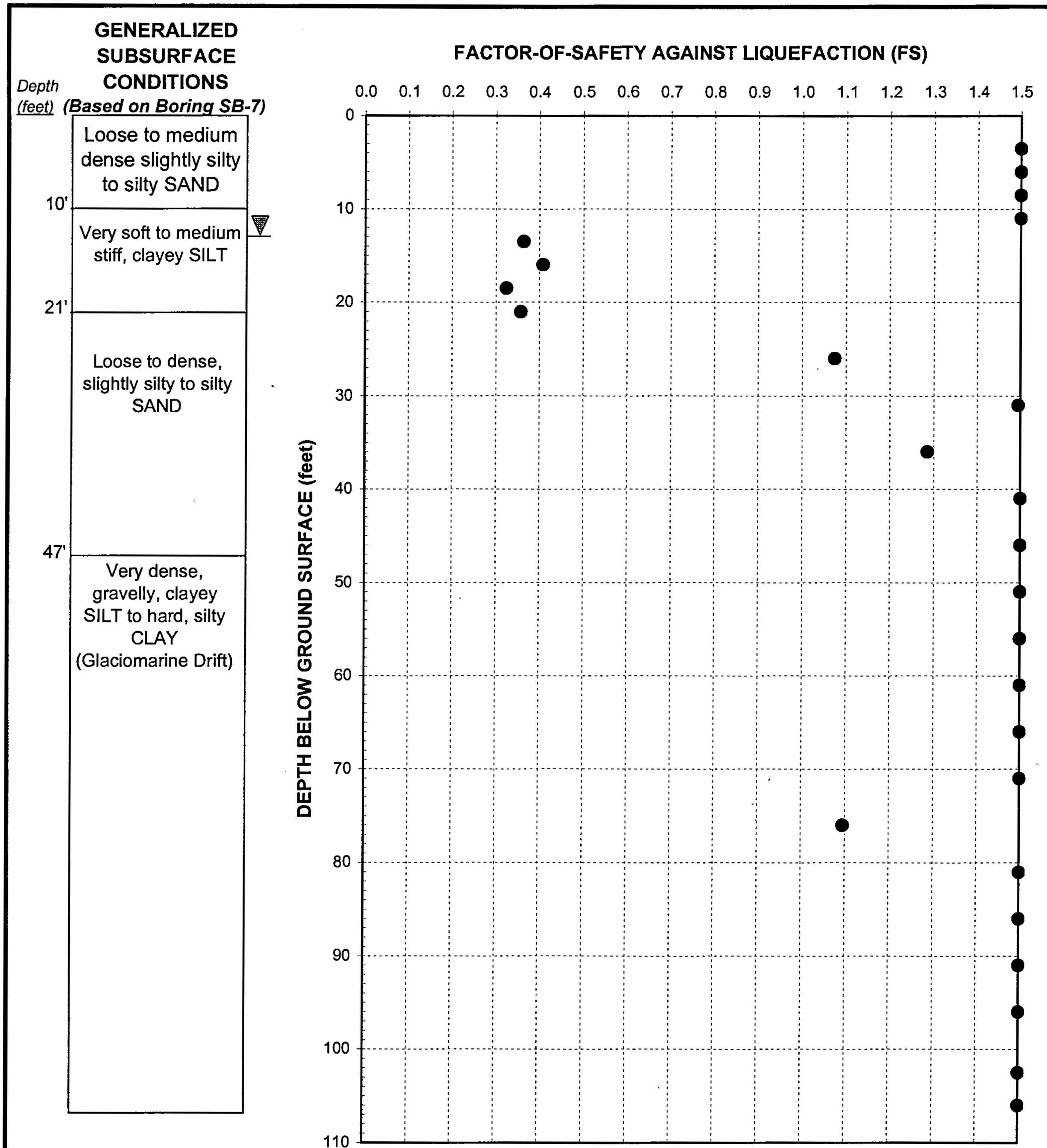
FIG. 22



NOTES:

1. *Reference:* Youd, T.L. and Idriss, I.M., 2001, Liquefaction Resistance of Soils: Summary Report from the 1996 NCEER and 1998 NCEER/ NSF Workshops on Evaluation of Liquefaction Resistance of Soils.
2. The analysis was performed using the Cyclic Stress Ratio (CSR) values determined in the site-specific ground response analysis.
3. The liquefaction resistance of a soil is dependent on its density and fines content. The fines content was estimated based on selected grain-size analyses and engineering judgement.

South Park Bridge
Seattle, Washington

RESULTS OF LIQUEFACTION ANALYSES BORING SB-7

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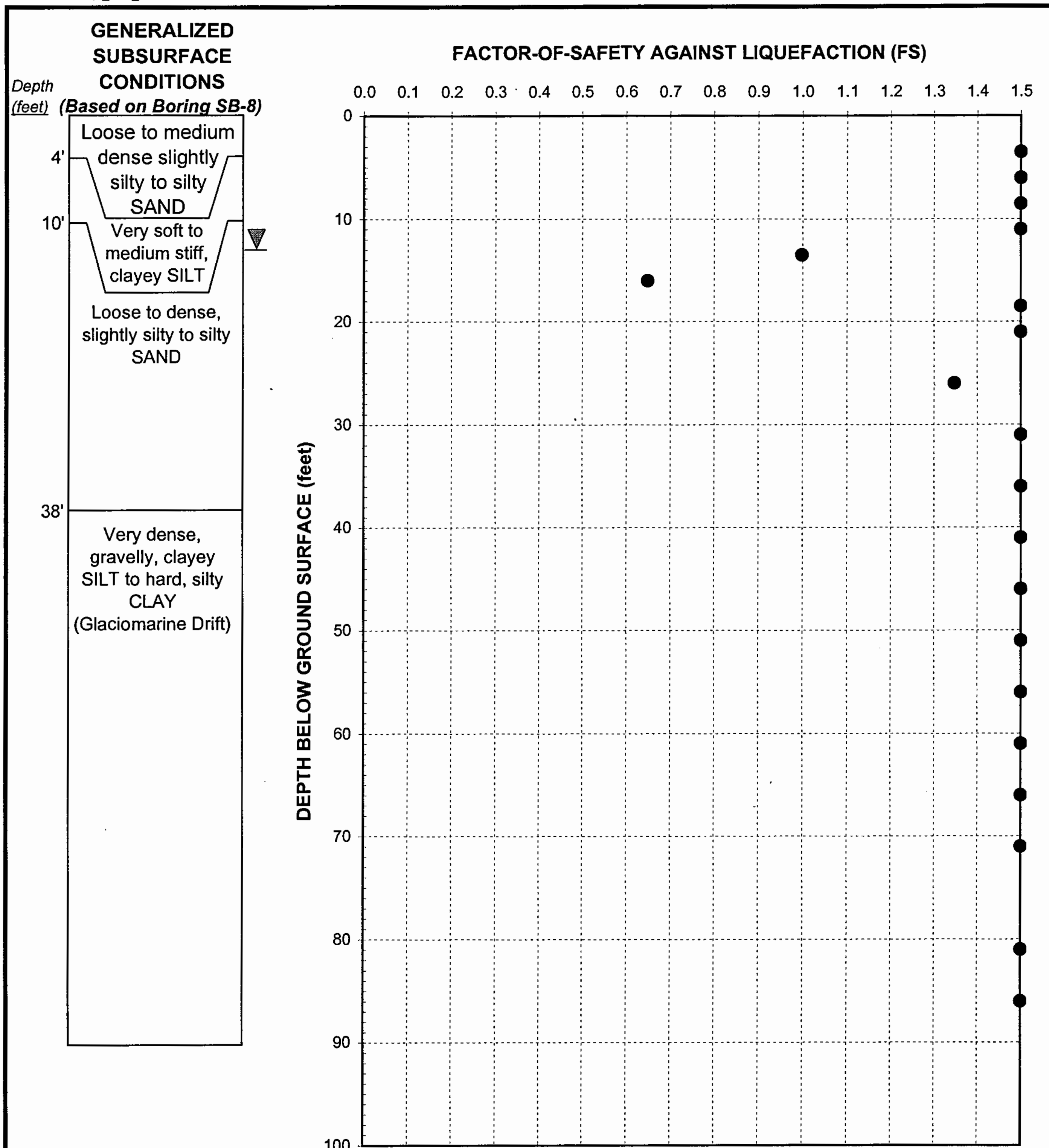
FIG. 23



NOTES:

1. *Reference:* Youd, T.L. and Idriss, I.M., 2001, Liquefaction Resistance of Soils: Summary Report from the 1996 NCEER and 1998 NCEER/ NSF Workshops on Evaluation of Liquefaction Resistance of Soils.
2. The analysis was performed using the Cyclic Stress Ratio (CSR) values determined in the site-specific ground response analysis.
3. The liquefaction resistance of a soil is dependent on its density and fines content. The fines content was estimated based on selected grain-size analyses and engineering judgement.

South Park Bridge
Seattle, Washington

RESULTS OF LIQUEFACTION ANALYSES BORING SB-8

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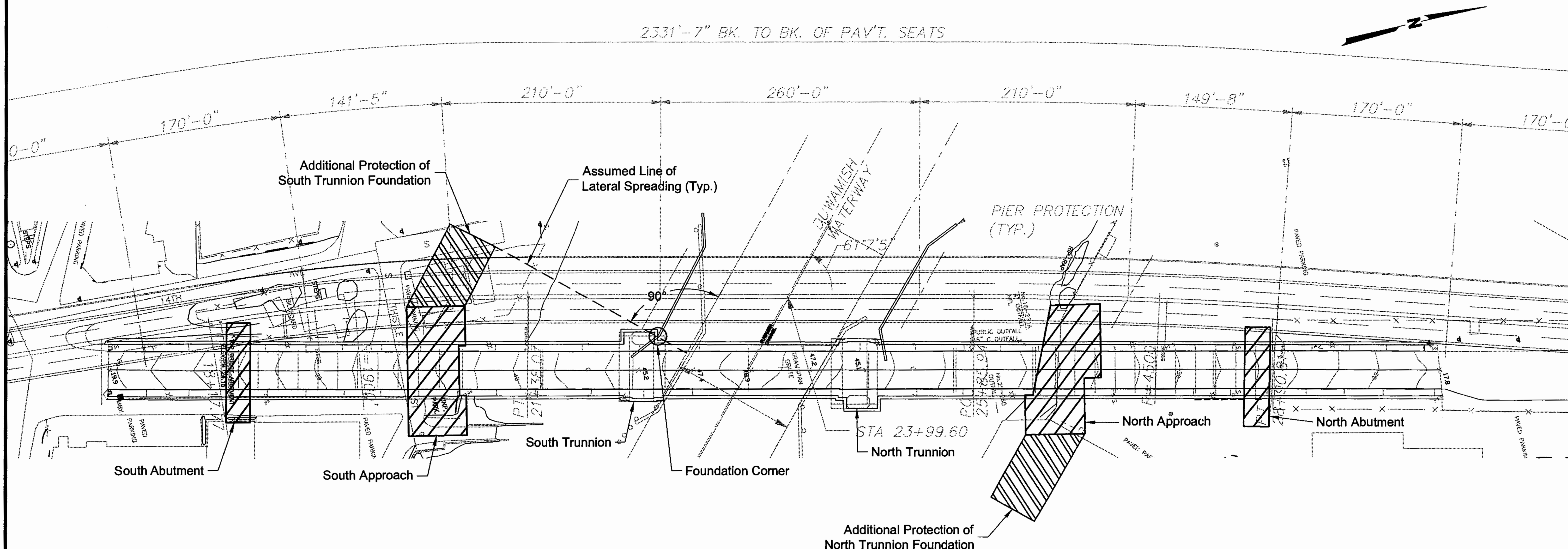
FIG. 24



LEGEND

Estimated Area of Ground Improvement

0 100 200

Scale in Feet

NOTE

Figure adapted from electronic files provided by Parsons Brinkerhoff, Inc., dated 9-26-03.

South Park Bridge Project
Seattle, Washington

PROPOSED GROUND IMPROVEMENT REHABILITATION ALTERNATIVE

March 2004 21-1-09584-008

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FIG. 25

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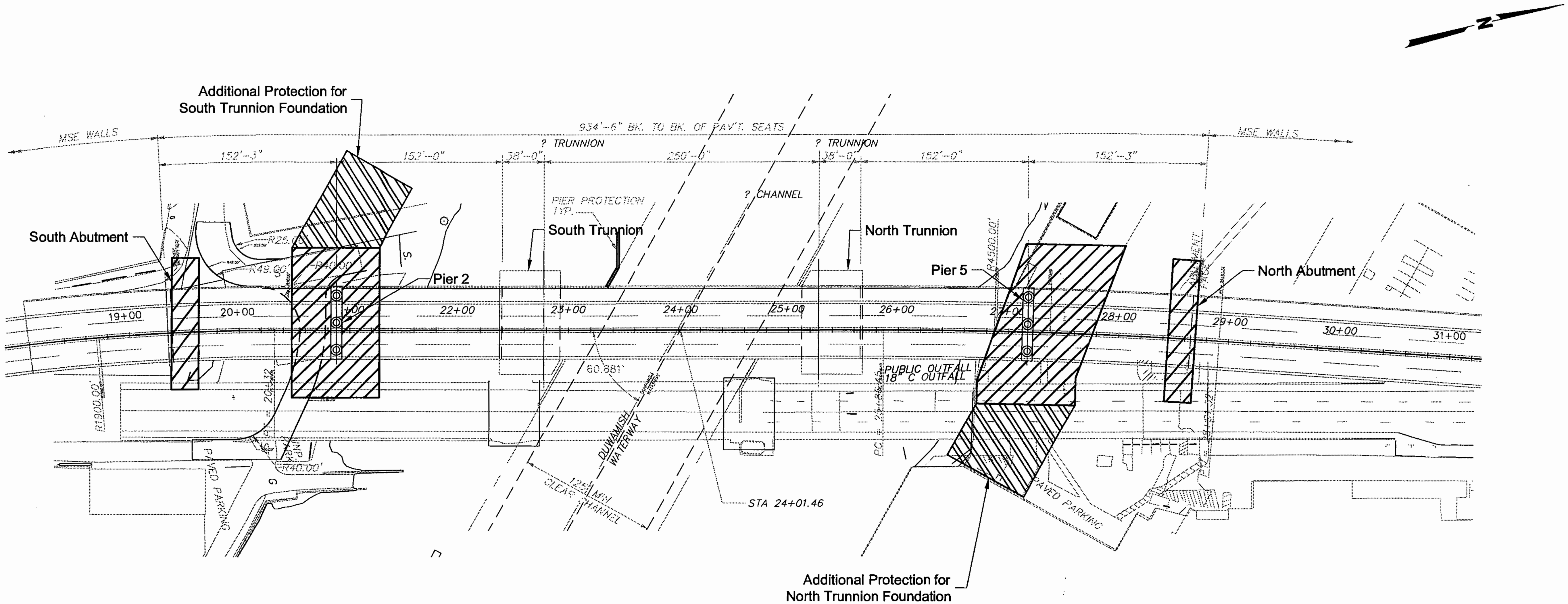

LEGEND

Estimated Area of Ground Improvement

0 100 200

Scale in Feet

NOTE

Figure adapted from electronic files provided by Parsons Brinkerhoff, Inc., dated 9-26-03.

South Park Bridge Project
Seattle, Washington

PROPOSED GROUND IMPROVEMENT
BASCULE BRIDGE ALTERNATIVE

March 2004 21-1-09584-008

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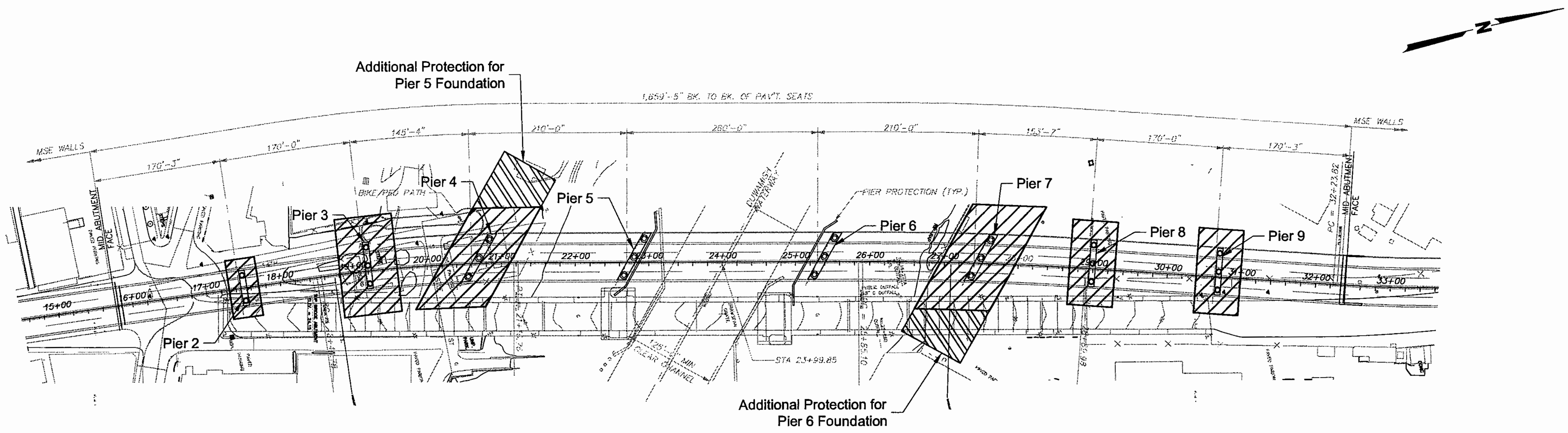
Geotechnical and Environmental Consultants

FIG. 26

File: I:\Drafting\211\09584-008\21-1-09584-008 fig 26.dwg Date: 03-30-2004 Author: CNT



LEGEND

Estimated Area of Ground Improvement

0 150 300

Scale in Feet

NOTE

Figure adapted from electronic files provided by Parsons Brinkerhoff, Inc., dated 9-26-03.

South Park Bridge Project
Seattle, Washington

PROPOSED GROUND IMPROVEMENT MID LEVEL FIXED-SPAN BRIDGE ALTERNATIVE

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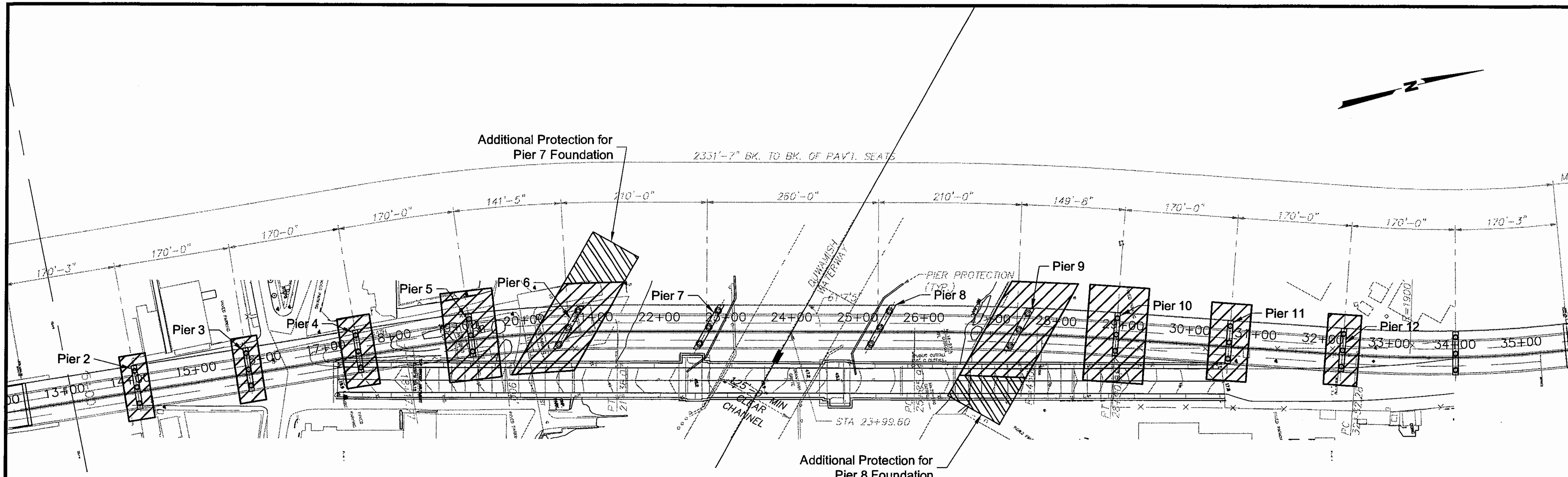
FIG. 27

File: I:\Drafting\211\09584-008\21-1-09584-008 fig 27.dwg Date: 03-30-2004 Author: CNT



LEGEND

Estimated Area of Ground Improvement

0 150 300

Scale in Feet

NOTE

Figure adapted from electronic files provided by Parsons Brinkerhoff, Inc., dated 9-26-03.

South Park Bridge Project
Seattle, Washington

PROPOSED GROUND IMPROVEMENT HIGH LEVEL FIXED-SPAN BRIDGE ALTERNATIVE

March 2004 21-1-09584-008

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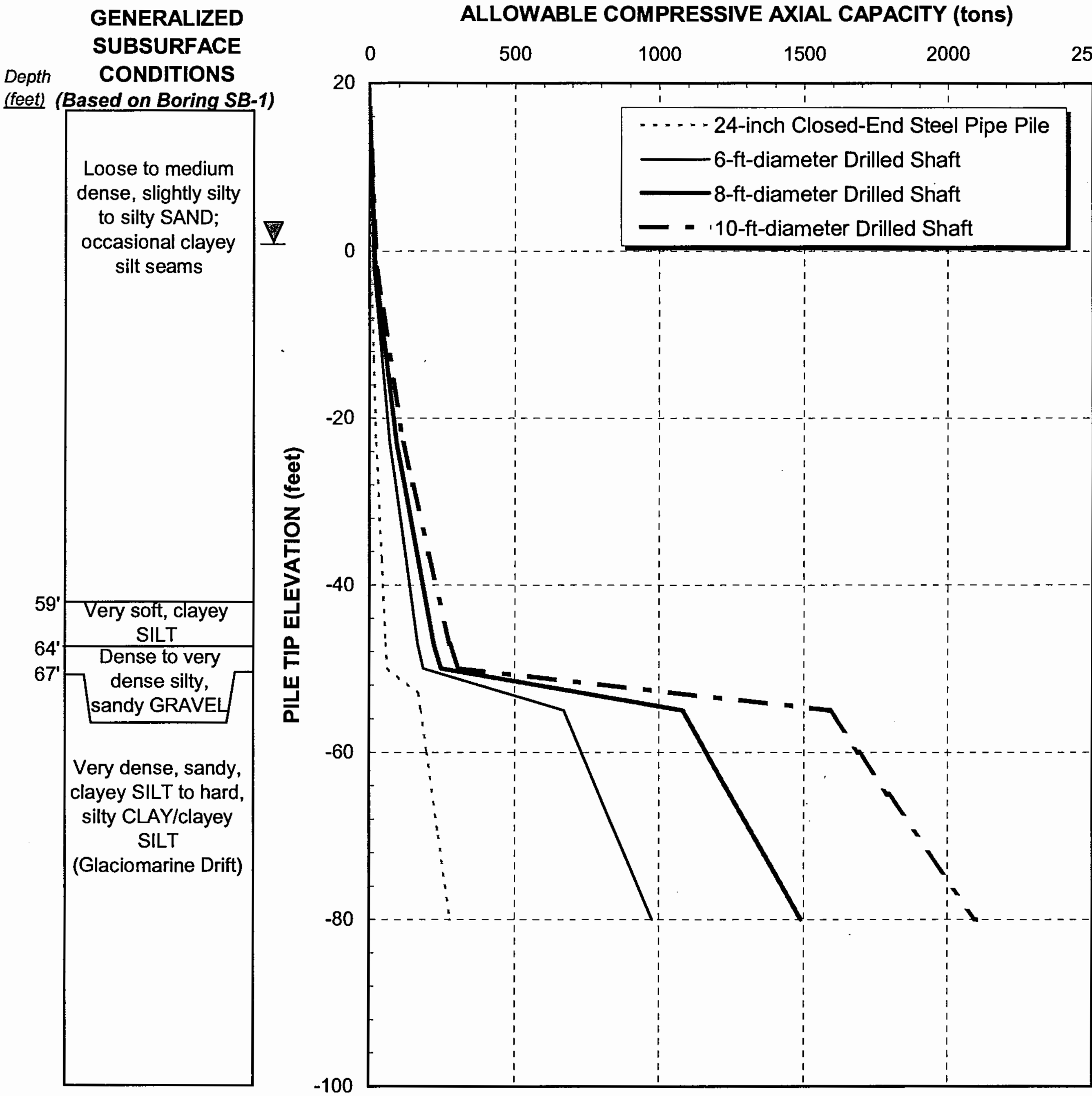
Geotechnical and Environmental Consultants

FIG. 28

File: I:\Drafting\211\09584-008\21-1-09584-008 fig 28.dwg Date: 03-30-2004 Author: CNT



NOTES

1. For driven piles, allowable compressive capacity is a summation of allowable skin friction and allowable end bearing. A factor-of safety of 2 was used for driven piles. For drilled shafts, allowable compressive capacity is a summation of allowable skin friction and mobilized end bearing. A factor-of-safety of 2 was used on the ultimate skin friction values to determine the allowable capacities. The mobilized end bearing was calculated based on an assumed allowable settlement of 1/2 inch at the base of the shaft.
2. Downdrag loads are not included in the capacities shown above.
3. Calculations assume static loading conditions for a single pile/shaft. Pile/shaft group effects are not considered.
4. Full end bearing is not achieved until the pile/shaft penetrates at least 4 to 5 feet into the bearing layer. The effect of underlying soft layers are considered in the analysis.

South Park Bridge Project
Seattle, Washington

ESTIMATED STATIC COMPRESSIVE CAPACITY
Sta. 32+40, Boring SB-1

March 2004 21-1-09584-008

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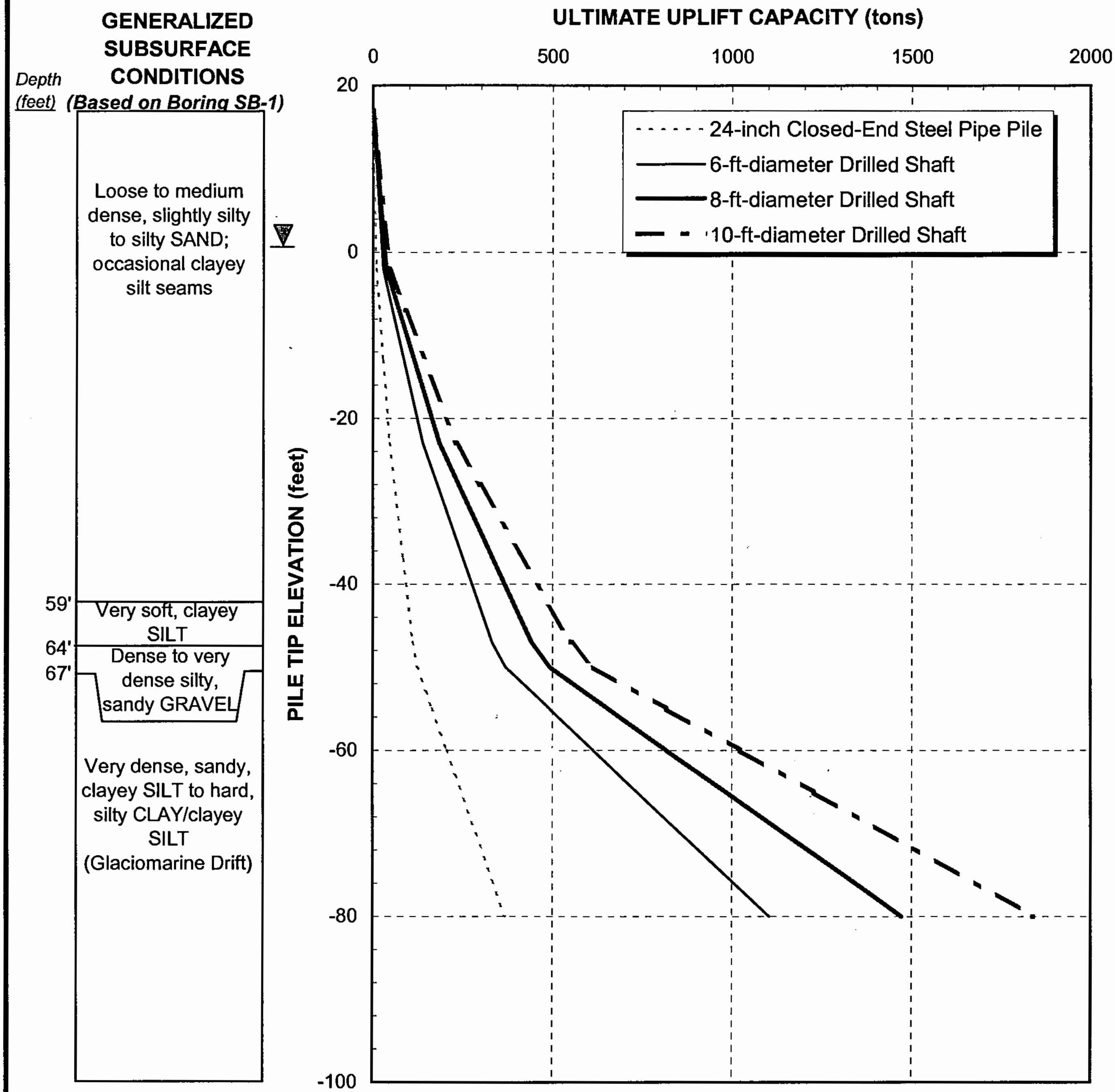
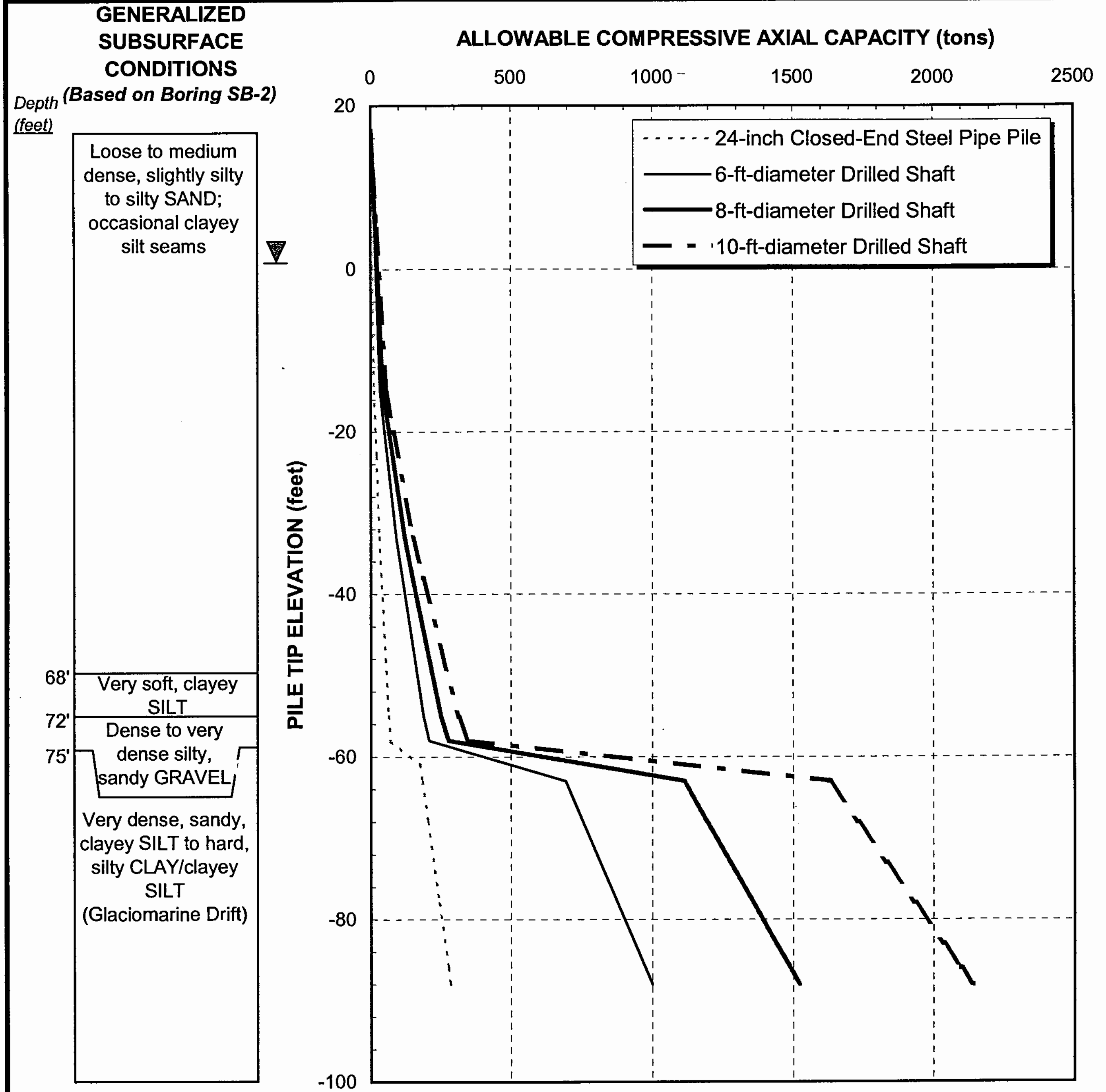
FIG. 29



NOTES

1. Allowable uplift resistance may be obtained by applying the appropriate factor-of safety depending on the design requirements and loading conditions.
2. Downdrag loads are not included in the capacities shown above.
3. Calculations assume static loading conditions for a single pile/shaft. Pile/shaft group effects are not considered.
4. Estimated capacities assume that if casing is used, it will be removed after the shaft installation. If, however, the casing is left in place, grouting should be used to fill all potential voids around the casing and the estimated capacities given above should be re-evaluated.

South Park Bridge Project Seattle, Washington	
ESTIMATED STATIC UPLIFT CAPACITY **Sta. 32+40, Boring SB-1**	
March 2004	21-1-09584-008
SHANNON & WILSON, INC. Geotechnical and Environmental Consultants	**FIG. 30**



NOTES

1. For driven piles, allowable compressive capacity is a summation of allowable skin friction and allowable end bearing. A factor-of safety of 2 was used for driven piles. For drilled shafts, allowable compressive capacity is a summation of allowable skin friction and mobilized end bearing. A factor-of-safety of 2 was used on the ultimate skin friction values to determine the allowable capacities. The mobilized end bearing was calculated based on an assumed allowable settlement of 1/2 inch at the base of the shaft.

2. Downdrag loads are not included in the capacities shown above.

3. Calculations assume static loading conditions for a single pile/shaft. Pile/shaft group effects are not considered.

4. Full end bearing is not achieved until the pile/shaft penetrates at least 4 to 5 feet into the bearing layer. The effect of underlying soft layers are considered in the analysis.

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ESTIMATED STATIC COMPRESSIVE CAPACITY
Sta. 30+60, Boring SB-2

March 2004 21-1-09584-008

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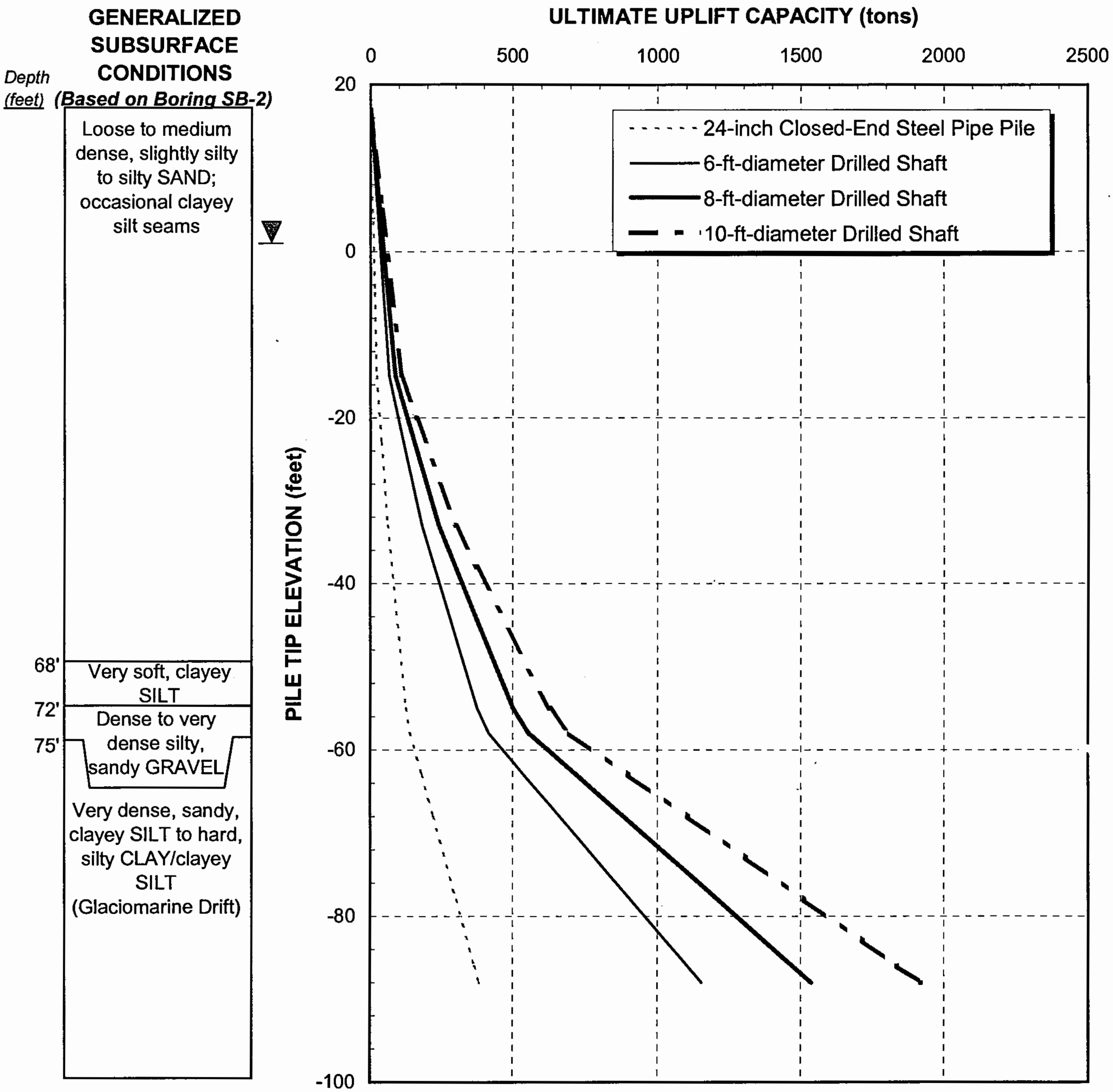
FIG. 31



NOTES

1. Allowable uplift resistance may be obtained by applying the appropriate factor-of safety depending on the design requirements and loading conditions.
2. Downdrag loads are not included in the capacities shown above.
3. Calculations assume static loading conditions for a single pile/shaft. Pile/shaft group effects are not considered.
4. Estimated capacities assume that if casing is used, it will be removed after the shaft installation. If, however, the casing is left in place, grouting should be used to fill all potential voids around the casing and the estimated capacities given above should be re-evaluated.

South Park Bridge Project
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ESTIMATED STATIC UPLIFT CAPACITY
Sta. 30+60, Boring SB-2

March 2004 21-1-09584-008

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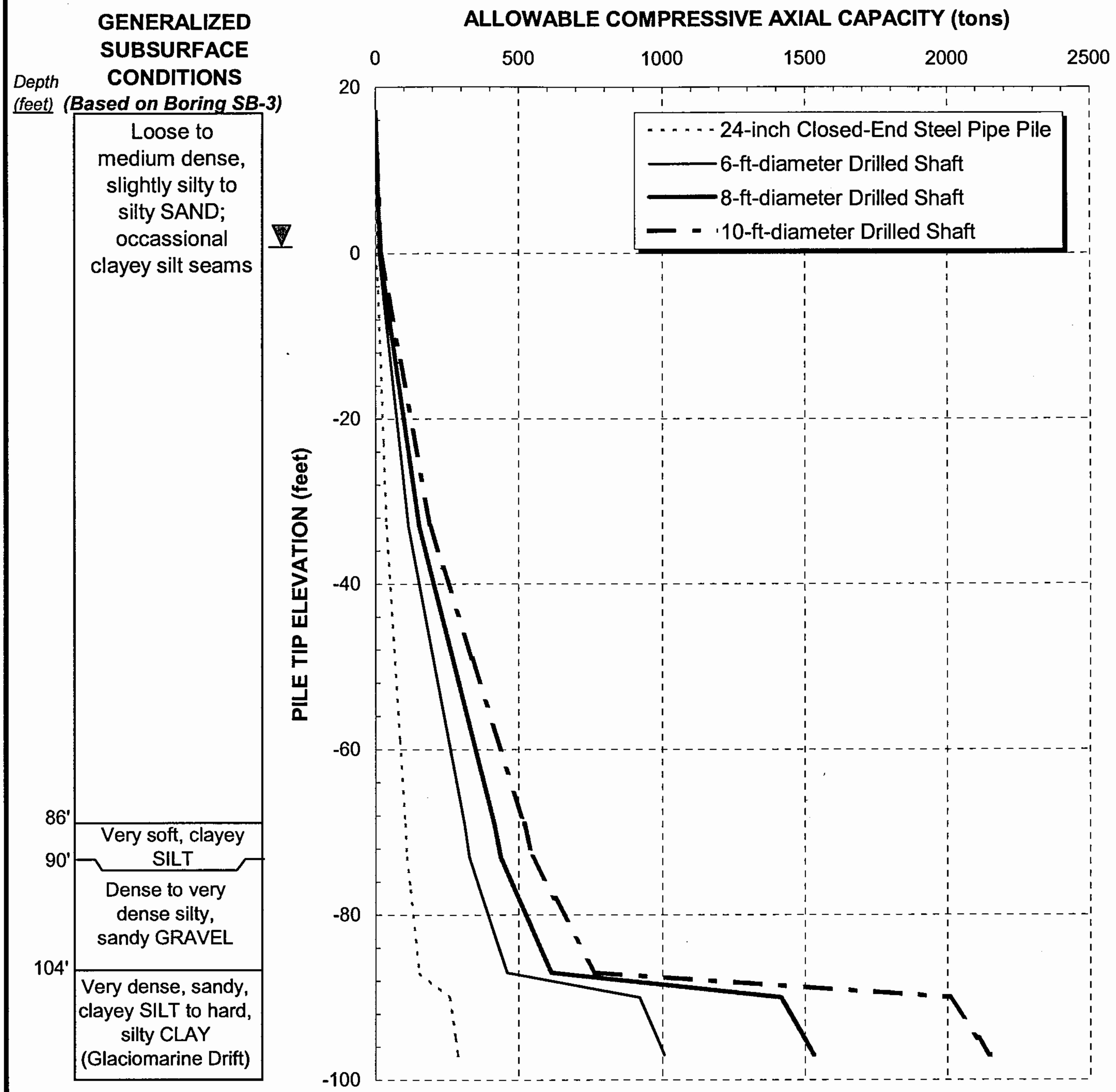
FIG. 32



NOTES

1. For driven piles, allowable compressive capacity is a summation of allowable skin friction and allowable end bearing. A factor-of safety of 2 was used for driven piles. For drilled shafts, allowable compressive capacity is a summation of allowable skin friction and mobilized end bearing. A factor-of-safety of 2 was used on the ultimate skin friction values to determine the allowable capacities. The mobilized end bearing was calculated based on an assumed allowable settlement of 1/2 inch at the base of the shaft.

2. Downdrag loads are not included in the capacities shown above.

3. Calculations assume static loading conditions for a single pile/shaft. Pile/shaft group effects are not considered.

4. Full end bearing is not achieved until the pile/shaft penetrates at least 4 to 5 feet into the bearing layer. The effect of underlying soft layers are considered in the analysis.

South Park Bridge Project
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ESTIMATED STATIC COMPRESSIVE CAPACITY
Sta. 27+50, Boring SB-3

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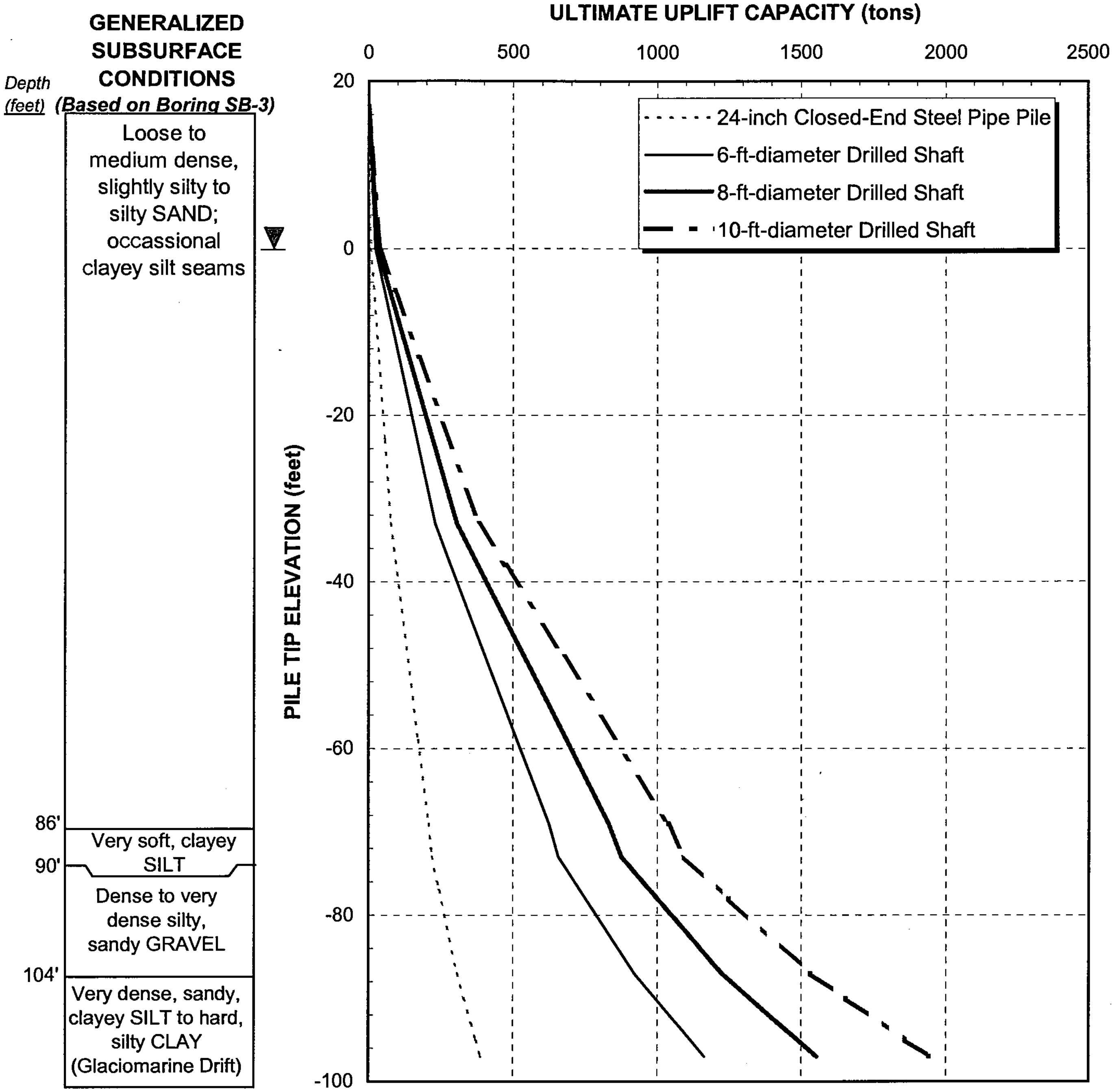
FIG. 33



NOTES

1. Allowable uplift resistance may be obtained by applying the appropriate factor-of safety depending on the design requirements and loading conditions.
2. Downdrag loads are not included in the capacities shown above.
3. Calculations assume static loading conditions for a single pile/shaft. Pile/shaft group effects are not considered.
4. Estimated capacities assume that if casing is used, it will be removed after the shaft installation. If, however, the casing is left in place, grouting should be used to fill all potential voids around the casing and the estimated capacities given above should be re-evaluated.

South Park Bridge Project
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ESTIMATED STATIC UPLIFT CAPACITY
Sta. 27+50, Boring SB-3

March 2004 21-1-09584-008

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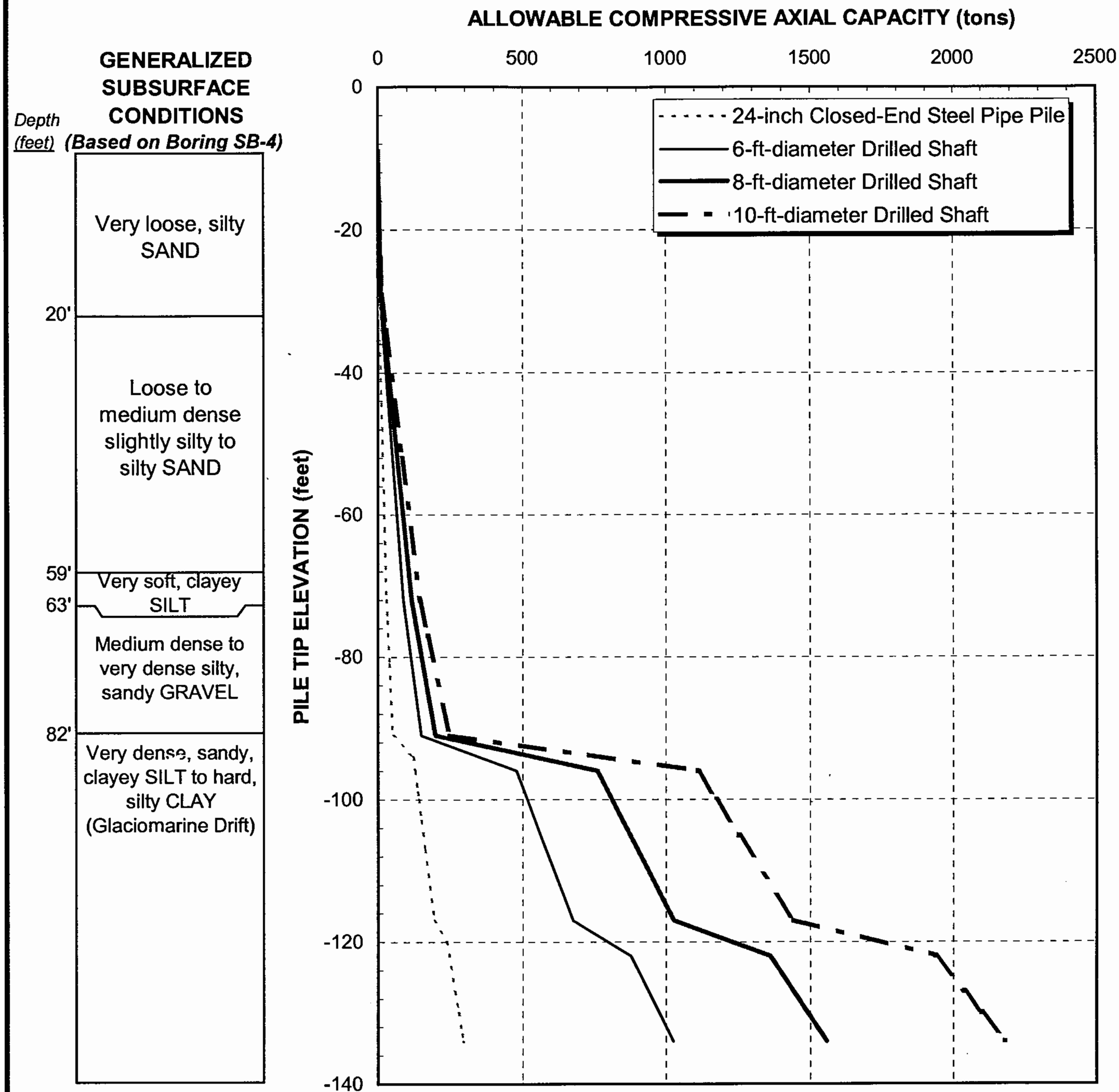
FIG. 34



NOTES

1. For driven piles, allowable compressive capacity is a summation of allowable skin friction and allowable end bearing. A factor-of safety of 2 was used for driven piles. For drilled shafts, allowable compressive capacity is a summation of allowable skin friction and mobilized end bearing. A factor-of-safety of 2 was used on the ultimate skin friction values to determine the allowable capacities. The mobilized end bearing was calculated based on an assumed allowable settlement of 1/2 inch at the base of the shaft.

2. Downdrag loads are not included in the capacities shown above.

3. Calculations assume static loading conditions for a single pile/shaft. Pile/shaft group effects are not considered.

4. Full end bearing is not achieved until the pile/shaft penetrates at least 4 to 5 feet into the bearing layer. The effect of underlying soft layers are considered in the analysis.

South Park Bridge Project
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ESTIMATED STATIC COMPRESSIVE CAPACITY
Sta. 25+45, Boring SB-4

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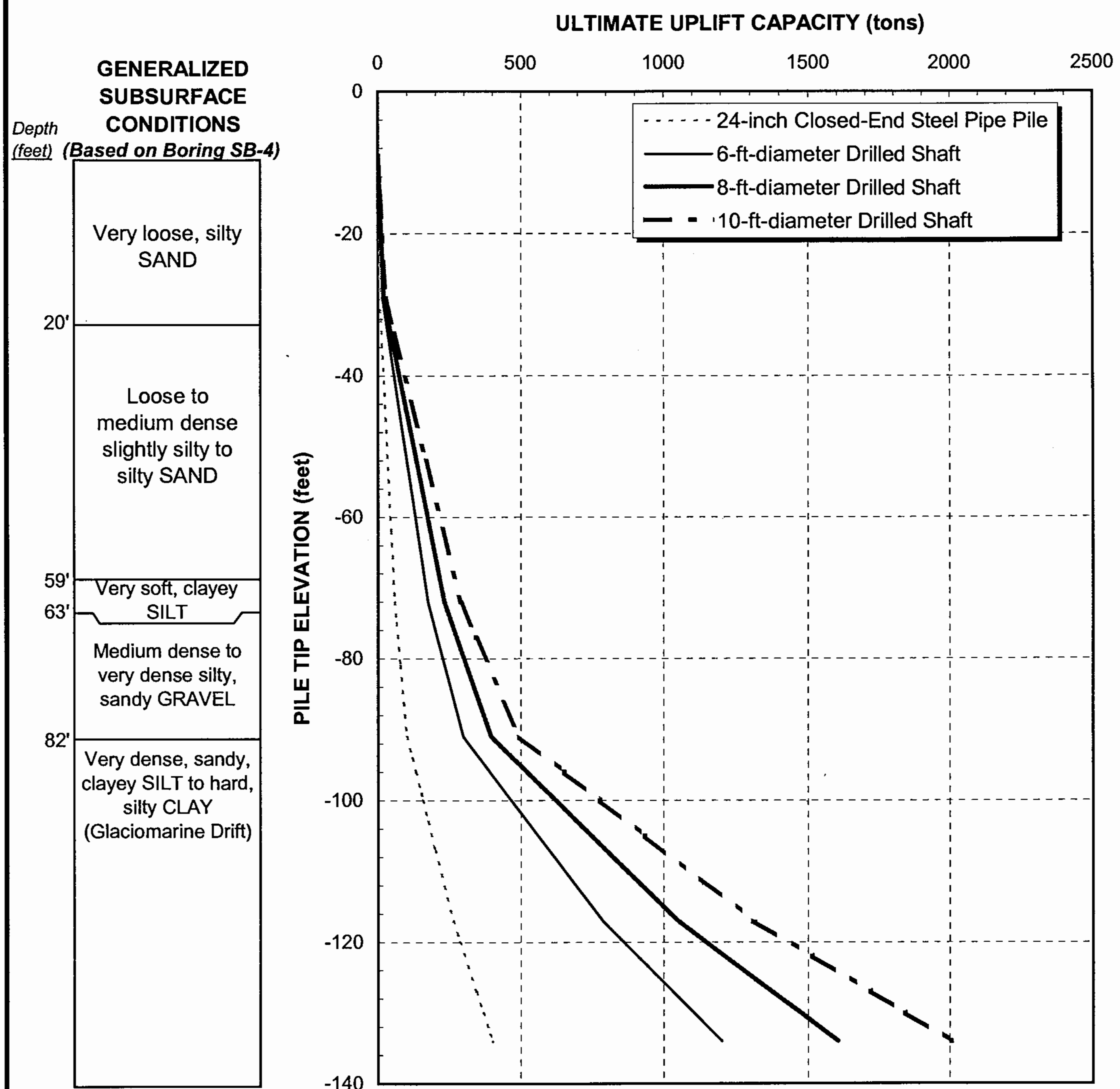
FIG. 35



NOTES

1. Allowable uplift resistance may be obtained by applying the appropriate factor-of safety depending on the design requirements and loading conditions.
2. Downdrag loads are not included in the capacities shown above.
3. Calculations assume static loading conditions for a single pile/shaft. Pile/shaft group effects are not considered.
4. Estimated capacities assume that if casing is used, it will be removed after the shaft installation. If, however, the casing is left in place, grouting should be used to fill all potential voids around the casing and the estimated capacities given above should be re-evaluated.

South Park Bridge Project
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ESTIMATED STATIC UPLIFT CAPACITY
Sta. 25+45, Boring SB-4

March 2004 21-1-09584-008

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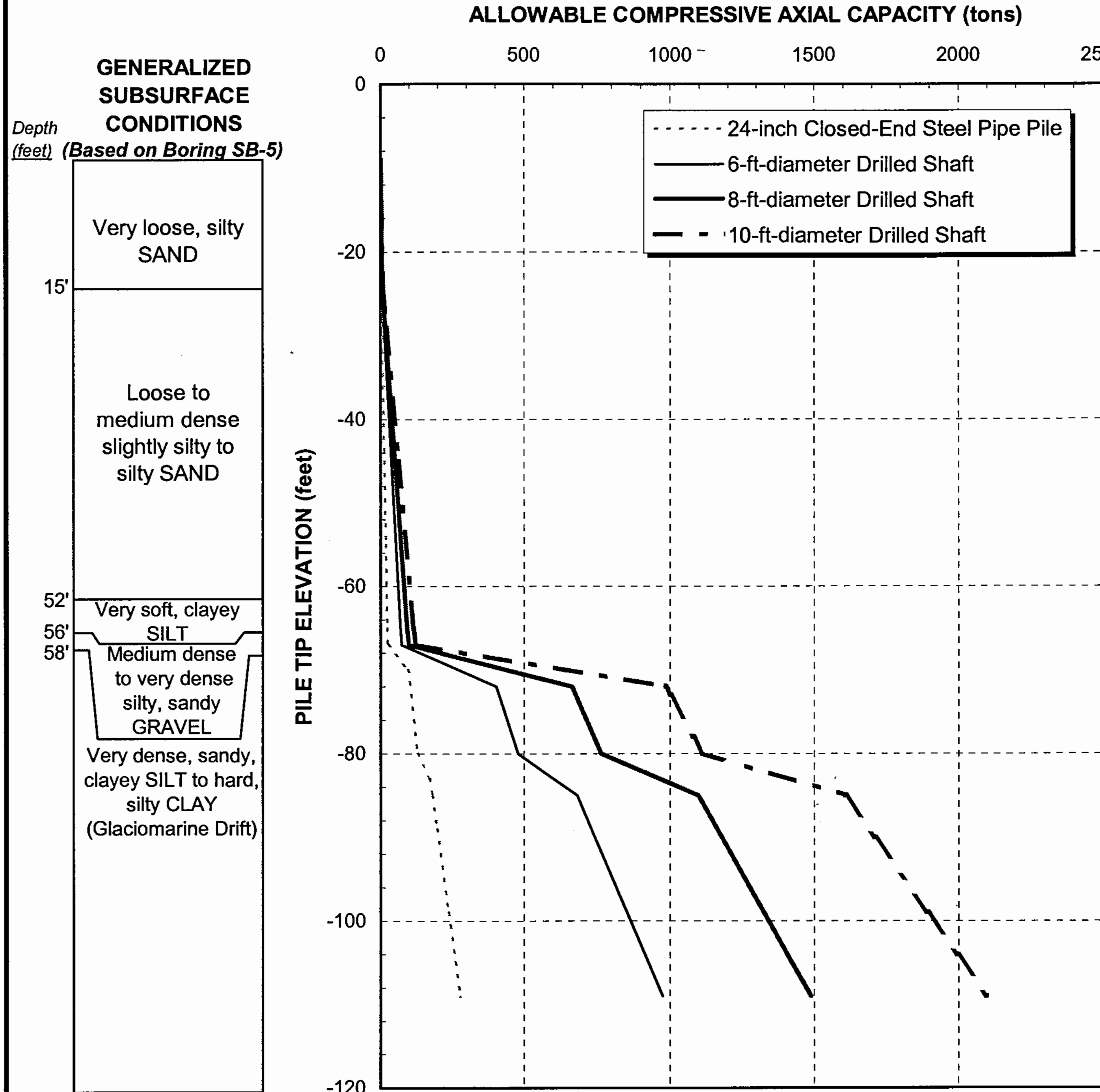
FIG. 36



NOTES

1. For driven piles, allowable compressive capacity is a summation of allowable skin friction and allowable end bearing. A factor-of safety of 2 was used for driven piles. For drilled shafts, allowable compressive capacity is a summation of allowable skin friction and mobilized end bearing. A factor-of-safety of 2 was used on the ultimate skin friction values to determine the allowable capacities. The mobilized end bearing was calculated based on an assumed allowable settlement of 1/2 inch at the base of the shaft.

2. Downdrag loads are not included in the capacities shown above.

3. Calculations assume static loading conditions for a single pile/shaft. Pile/shaft group effects are not considered.

4. Full end bearing is not achieved until the pile/shaft penetrates at least 4 to 5 feet into the bearing layer. The effect of underlying soft layers are considered in the analysis.

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ESTIMATED STATIC COMPRESSIVE CAPACITY
Sta. 23+25, Boring SB-5

March 2004 21-1-09584-008

SHANNON & WILSON, INC.
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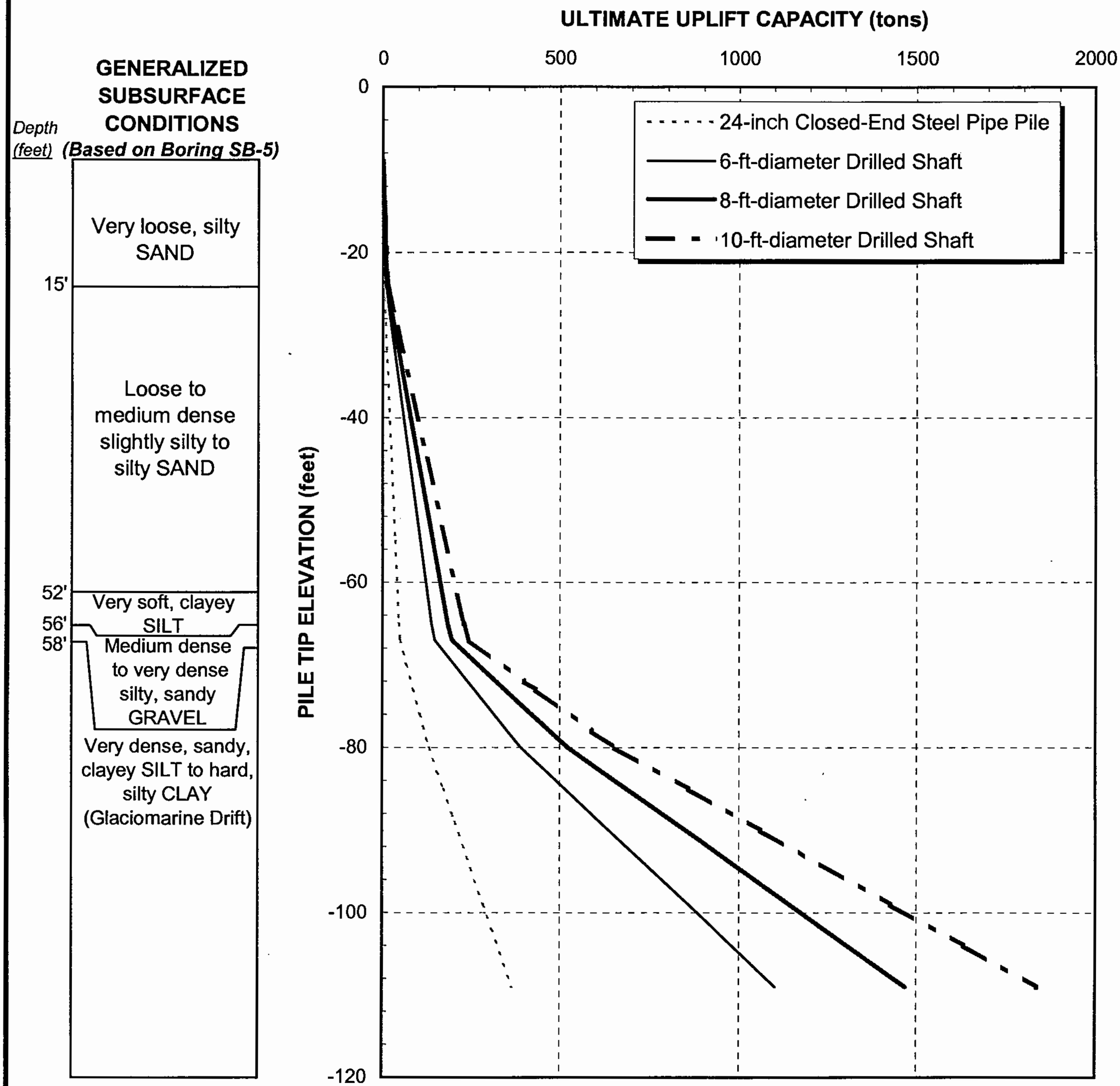
FIG. 37



NOTES

1. Allowable uplift resistance may be obtained by applying the appropriate factor-of safety depending on the design requirements and loading conditions.
2. Downdrag loads are not included in the capacities shown above.
3. Calculations assume static loading conditions for a single pile/shaft. Pile/shaft group effects are not considered.
4. Estimated capacities assume that if casing is used, it will be removed after the shaft installation. If, however, the casing is left in place, grouting should be used to fill all potential voids around the casing and the estimated capacities given above should be re-evaluated.

South Park Bridge Project
Seattle, Washington

ESTIMATED STATIC UPLIFT CAPACITY
Sta. 23+25, Boring SB-5

March 2004 21-1-09584-008

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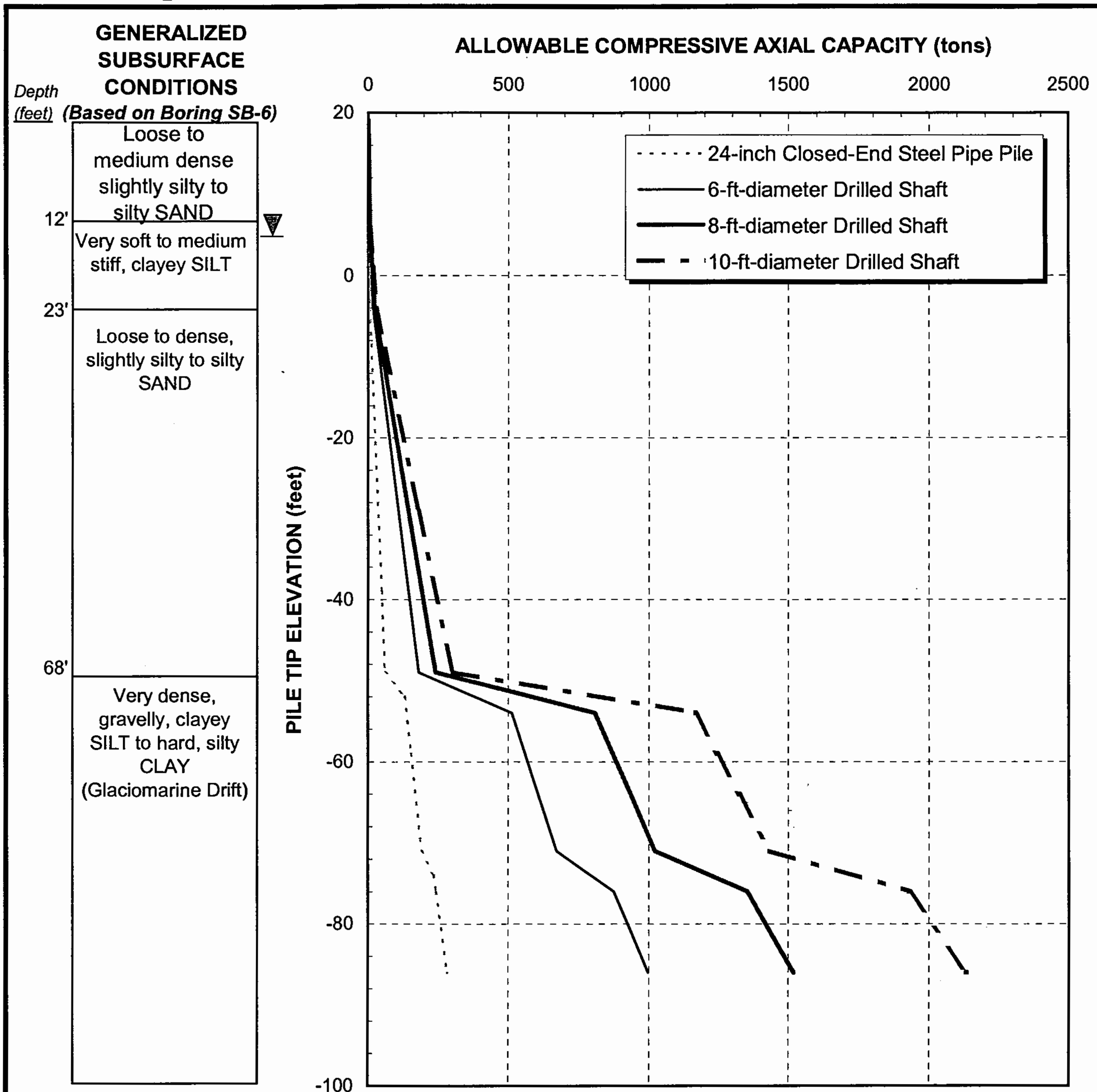
FIG. 38



NOTES

1. For driven piles, allowable compressive capacity is a summation of allowable skin friction and allowable end bearing. A factor-of safety of 2 was used for driven piles. For drilled shafts, allowable compressive capacity is a summation of allowable skin friction and mobilized end bearing. A factor-of-safety of 2 was used on the ultimate skin friction values to determine the allowable capacities. The mobilized end bearing was calculated based on an assumed allowable settlement of 1/2 inch at the base of the shaft.
2. Downdrag loads are not included in the capacities shown above.
3. Calculations assume static loading conditions for a single pile/shaft. Pile/shaft group effects are not considered.
4. Full end bearing is not achieved until the pile/shaft penetrates at least 4 to 5 feet into the bearing layer. The effect of underlying soft layers are considered in the analysis.

South Park Bridge Project
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ESTIMATED STATIC COMPRESSIVE CAPACITY
Sta. 20+00, Boring SB-6

March 2004 21-1-09584-008

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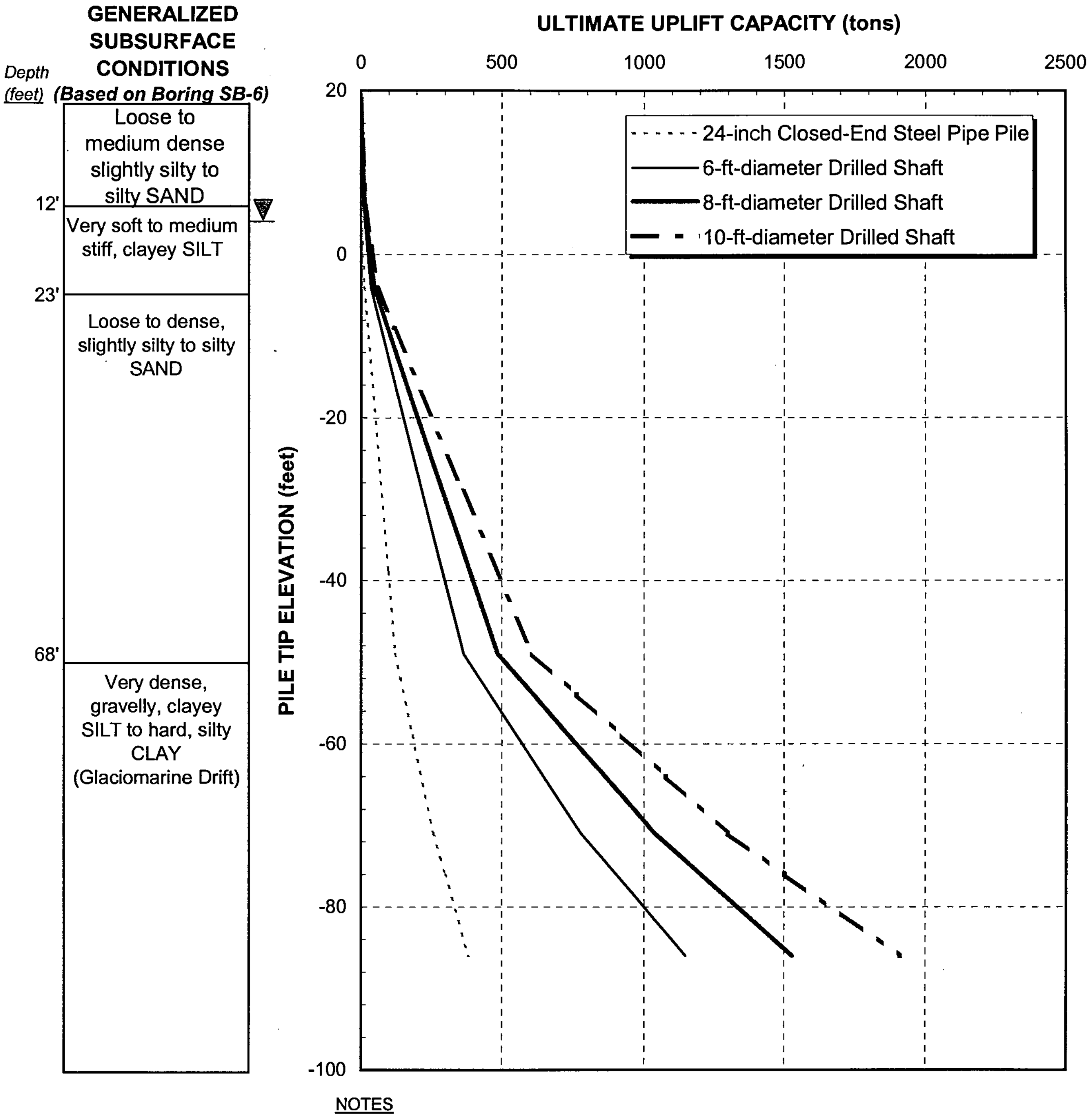
FIG. 39



NOTES

1. Allowable uplift resistance may be obtained by applying the appropriate factor-of safety depending on the design requirements and loading conditions.
2. Downdrag loads are not included in the capacities shown above.
3. Calculations assume static loading conditions for a single pile/shaft. Pile/shaft group effects are not considered.
4. Estimated capacities assume that if casing is used, it will be removed after the shaft installation. If, however, the casing is left in place, grouting should be used to fill all potential voids around the casing and the estimated capacities given above should be re-evaluated.

South Park Bridge Project
Seattle, Washington

ESTIMATED STATIC UPLIFT CAPACITY
Sta. 20+00, Boring SB-6

March 2004 21-1-09584-008

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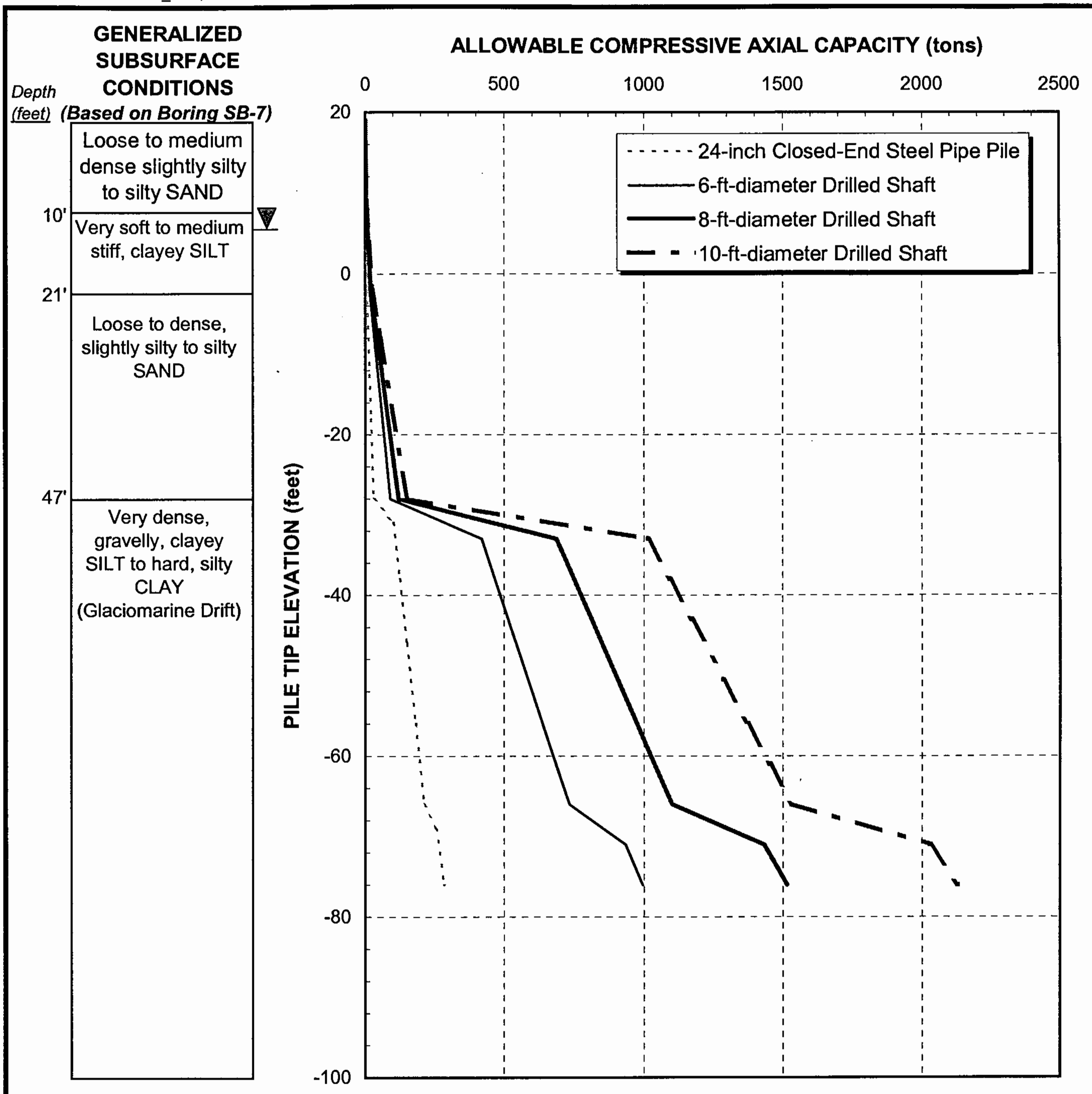
FIG. 40



NOTES

1. For driven piles, allowable compressive capacity is a summation of allowable skin friction and allowable end bearing. A factor-of safety of 2 was used for driven piles. For drilled shafts, allowable compressive capacity is a summation of allowable skin friction and mobilized end bearing. A factor-of-safety of 2 was used on the ultimate skin friction values to determine the allowable capacities. The mobilized end bearing was calculated based on an assumed allowable settlement of 1/2 inch at the base of the shaft.

2. Downdrag loads are not included in the capacities shown above.

3. Calculations assume static loading conditions for a single pile/shaft. Pile/shaft group effects are not considered.

4. Full end bearing is not achieved until the pile/shaft penetrates at least 4 to 5 feet into the bearing layer. The effect of underlying soft layers are considered in the analysis.

South Park Bridge Project
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ESTIMATED STATIC COMPRESSIVE CAPACITY
Sta. 17+40, Boring SB-7

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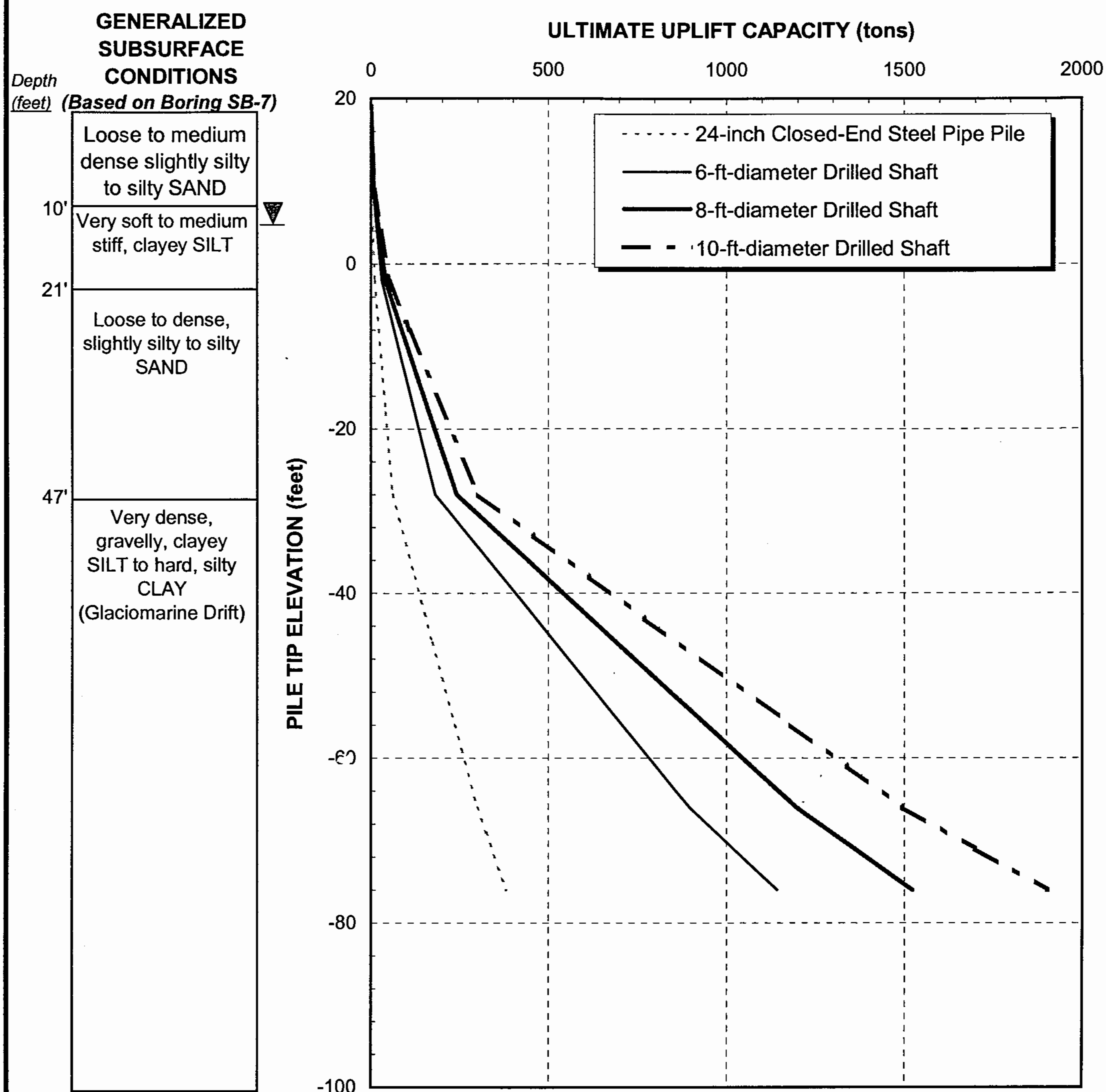
FIG. 41



NOTES

1. Allowable uplift resistance may be obtained by applying the appropriate factor-of safety depending on the design requirements and loading conditions.

2. Downdrag loads are not included in the capacities shown above.

3. Calculations assume static loading conditions for a single pile/shaft. Pile/shaft group effects are not considered.

4. Estimated capacities assume that if casing is used, it will be removed after the shaft installation. If, however, the casing is left in place, grouting should be used to fill all potential voids around the casing and the estimated capacities given above should be re-evaluated.

South Park Bridge Project
Seattle, Washington

ESTIMATED STATIC UPLIFT CAPACITY
Sta. 17+40, Boring SB-7

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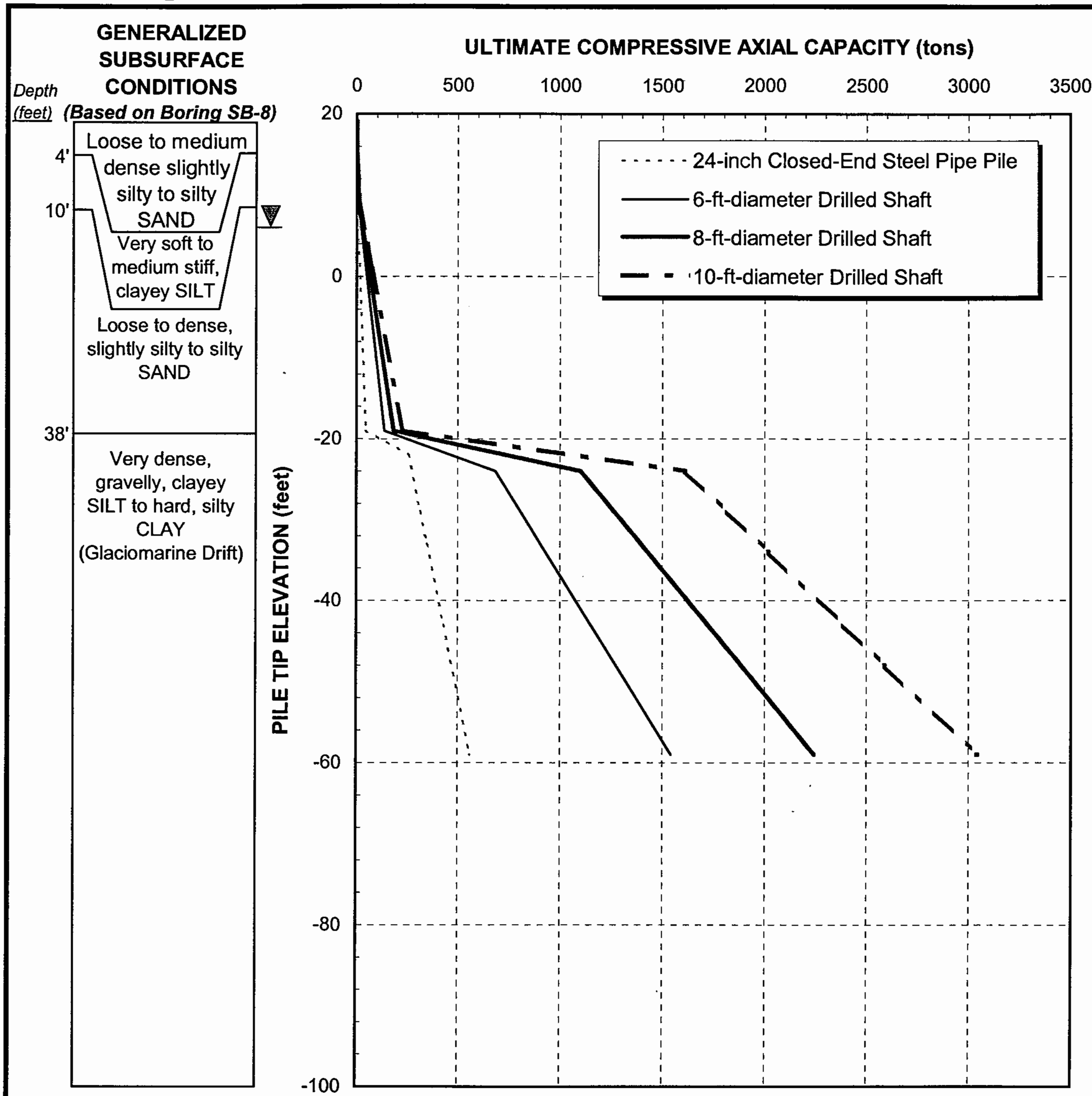
FIG. 42



NOTES

1. For driven piles, allowable compressive capacity is a summation of allowable skin friction and allowable end bearing. A factor-of safety of 2 was used for driven piles. For drilled shafts, allowable compressive capacity is a summation of allowable skin friction and mobilized end bearing. A factor-of-safety of 2 was used on the ultimate skin friction values to determine the allowable capacities. The mobilized end bearing was calculated based on an assumed allowable settlement of 1/2 inch at the base of the shaft.

2. Downdrag loads are not included in the capacities shown above.

3. Calculations assume static loading conditions for a single pile/shaft. Pile/shaft group effects are not considered.

4. Full end bearing is not achieved until the pile/shaft penetrates at least 4 to 5 feet into the bearing layer. The effect of underlying soft layers are considered in the analysis.

South Park Bridge Project
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ESTIMATED STATIC COMPRESSIVE CAPACITY
Sta. 15+20, Boring SB-8

March 2004 21-1-09584-008

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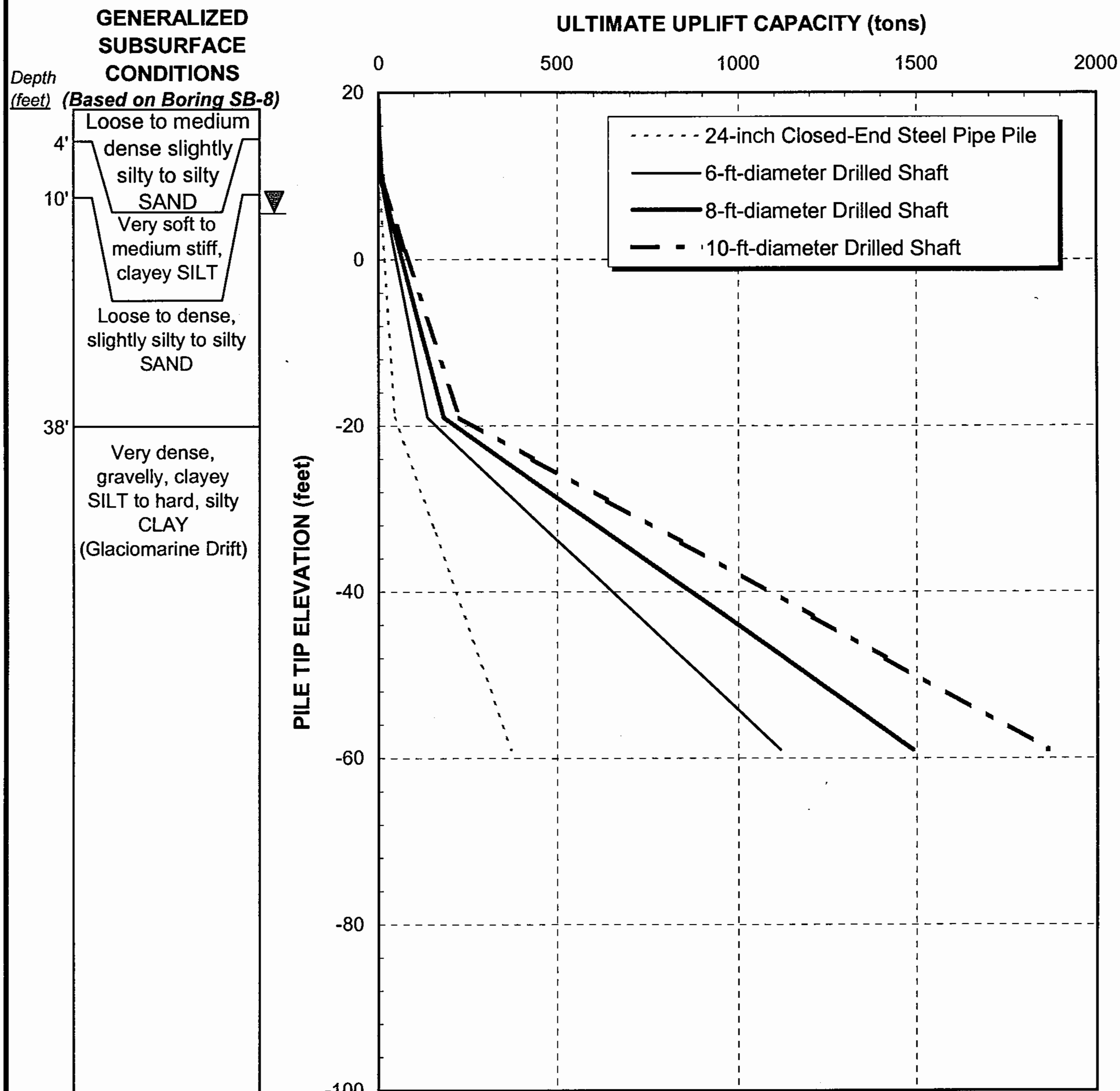
FIG. 43



NOTES

1. Allowable uplift resistance may be obtained by applying the appropriate factor-of safety depending on the design requirements and loading conditions.
2. Downdrag loads are not included in the capacities shown above.
3. Calculations assume static loading conditions for a single pile/shaft. Pile/shaft group effects are not considered.
4. Estimated capacities assume that if casing is used, it will be removed after the shaft installation. If, however, the casing is left in place, grouting should be used to fill all potential voids around the casing and the estimated capacities given above should be re-evaluated.

South Park Bridge Project
Seattle, Washington

ESTIMATED STATIC UPLIFT CAPACITY
Sta. 15+20, Boring SB-8

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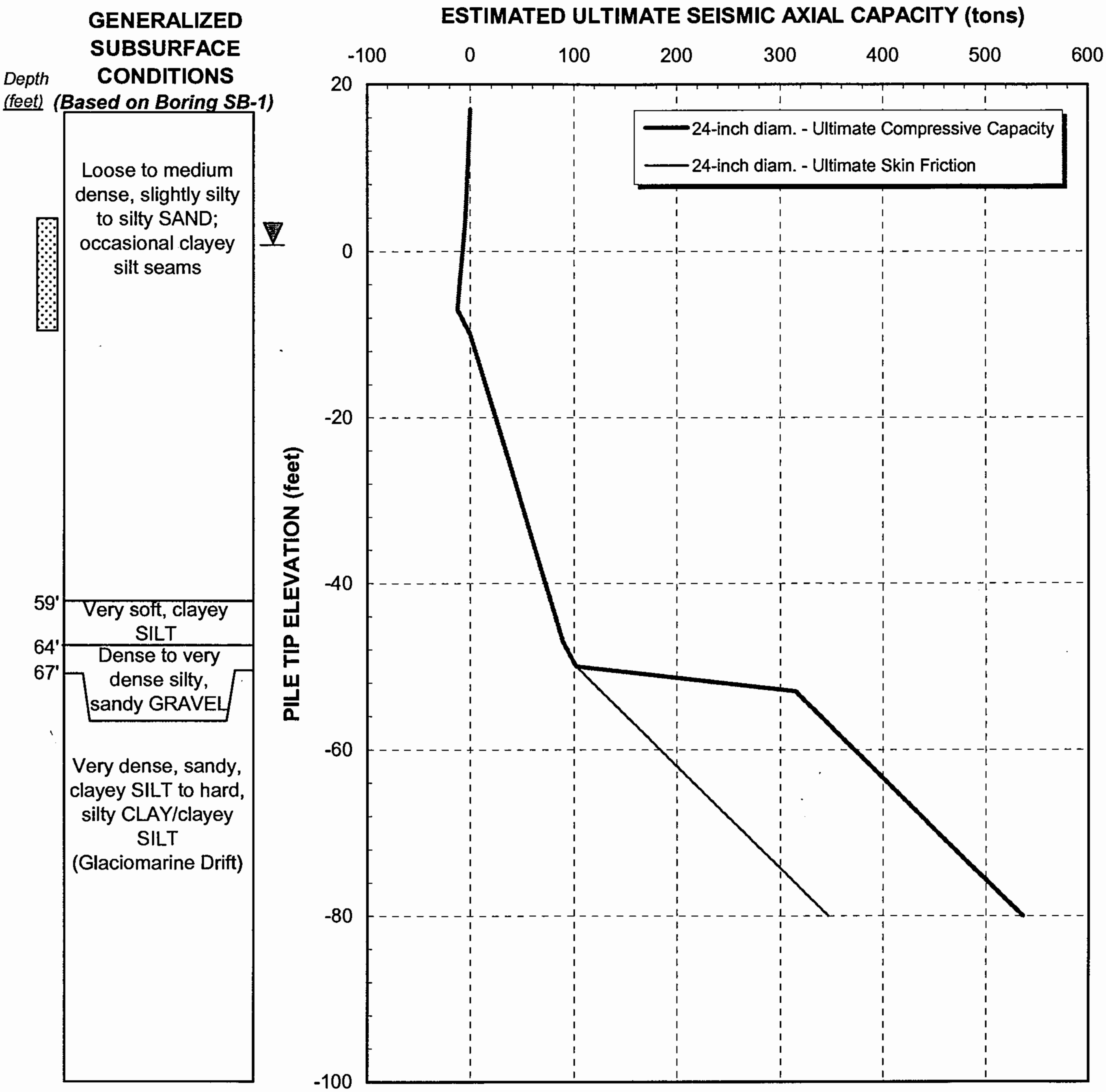
FIG. 44



NOTES

1. Ultimate compressive capacity is a summation of ultimate skin friction and ultimate end bearing. We recommend that an appropriate factor-of-safety be applied to the ultimate values to obtain the allowable design loads, depending on the design requirements and loading conditions.
2. Calculations assume seismic loading conditions for a single pile. Pile group effects are not considered.
3. Downdrag forces are included in the capacities above.
4. The steel pipe piles are assumed to be driven closed-end. A reinforced cutting shoe is recommended to achieve adequate penetrations into glacial deposits.
5. Indicates liquefied soils during the Design Earthquake event. (475-year Return Period)

South Park Bridge Project
Seattle, Washington

ESTIMATED SEISMIC AXIAL CAPACITY
STEEL PIPE PILES
Sta. 32+40, Boring SB-1

March 2004 | 21-1-09584-008

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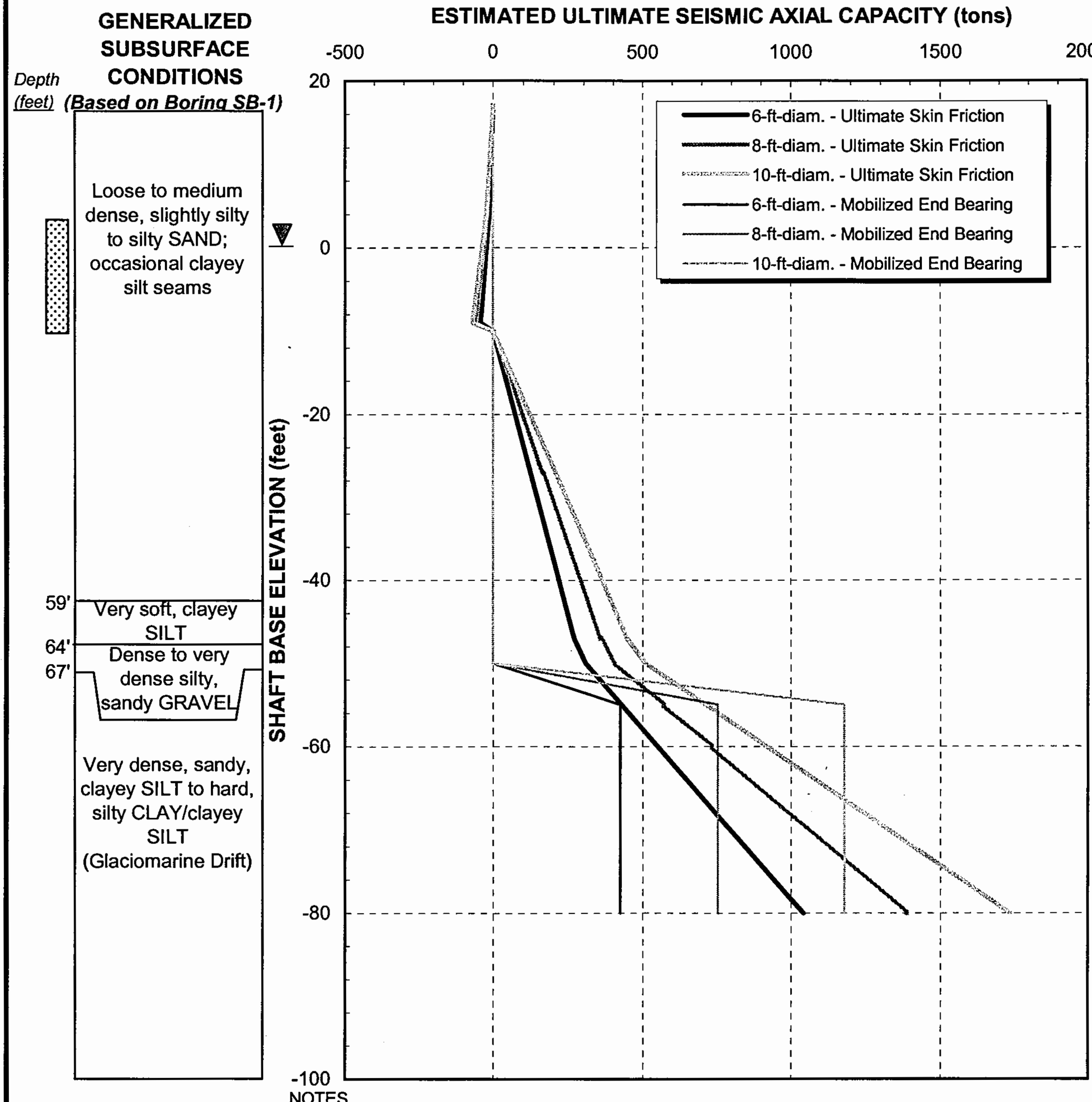
FIG. 45



NOTES

1. Ultimate Compressive Capacity = Ultimate Skin Friction + Mobilized End Bearing
   Allowable Compressive Capacity = Allowable Skin Friction + Mobilized End Bearing
   Allowable Skin Friction = Ultimate Skin Friction / Appropriate Factor of Safety
2. Calculations assume seismic loading conditions for a single shaft. Shaft group effects are not considered.
3. Estimated capacities assume that if casing is used, it will be removed after the shaft installation. If, however, the casing is left in place, grouting should be used to fill all potential voids around the casing and the estimated capacities given above should be re-evaluated.
4. Downdrag forces are included in the capacities above.
5. [pattern] Indicates liquefied soils during the Design Earthquake event. (475-year return period)

South Park Bridge Project
Seattle, Washington

**ESTIMATED SEISMIC AXIAL CAPACITY
DRILLED SHAFTS
Sta. 32+40, Boring SB-1**

March 2004 — 21-1-09584-008

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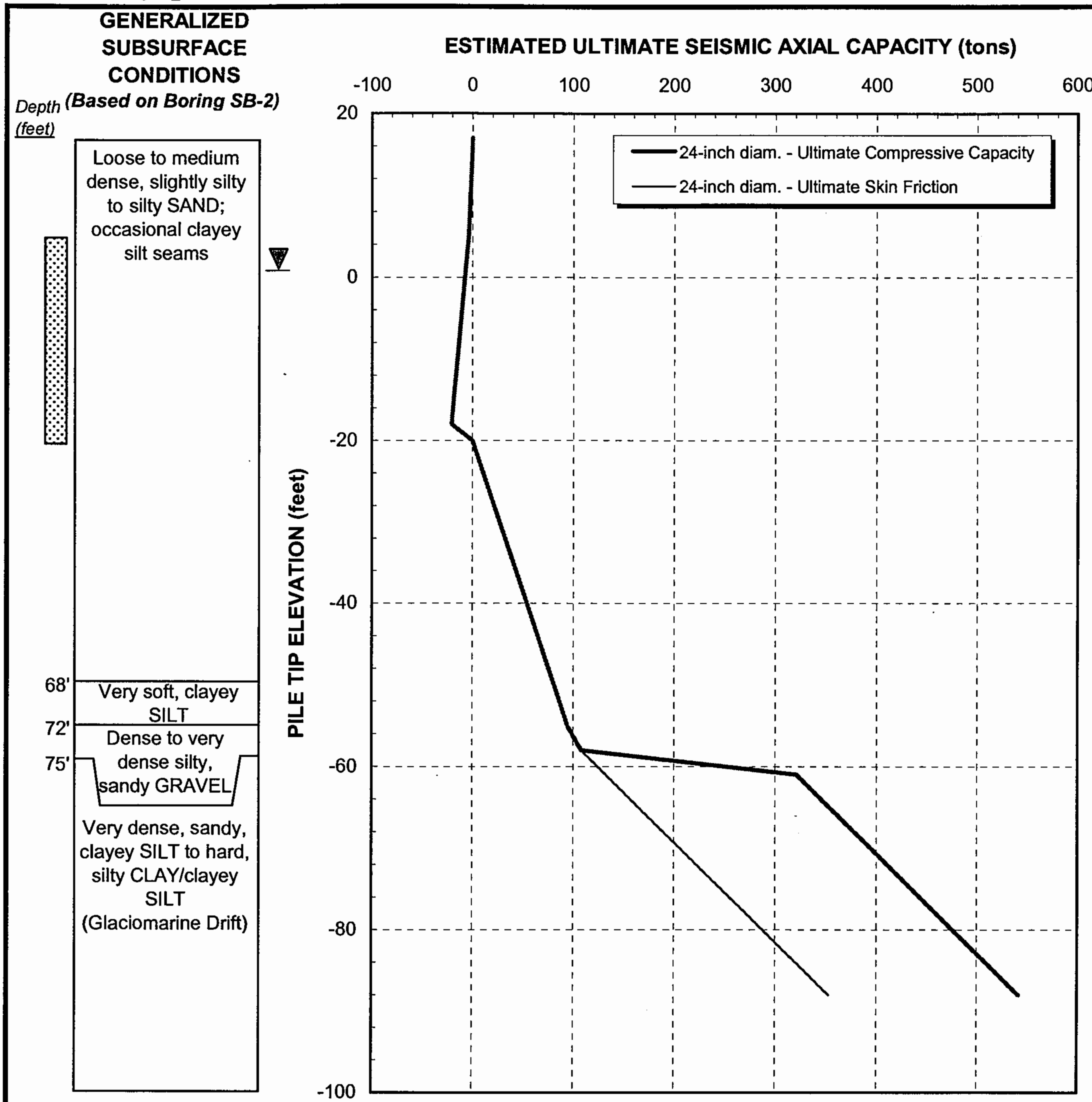
Geotechnical and Environmental Consultants

FIG. 46



NOTES

1. Ultimate compressive capacity is a summation of ultimate skin friction and ultimate end bearing. We recommend that an appropriate factor-of-safety be applied to the ultimate values to obtain the allowable design loads, depending on the design requirements and loading conditions.

2. Calculations assume seismic loading conditions for a single pile. Pile group effects are not considered.

3. Downdrag forces are included in the capacities above.

4. The steel pipe piles are assumed to be driven closed-end. A reinforced cutting shoe is recommended to achieve adequate penetrations into glacial deposits.

5. ⁙ Indicates liquefied soils during the Design Earthquake event. (475-year Return Period)

South Park Bridge Project
Seattle, Washington

ESTIMATED SEISMIC AXIAL CAPACITY
STEEL PIPE PILES
Sta. 30+60, Boring SB-2

March 2004 21-1-09584-008

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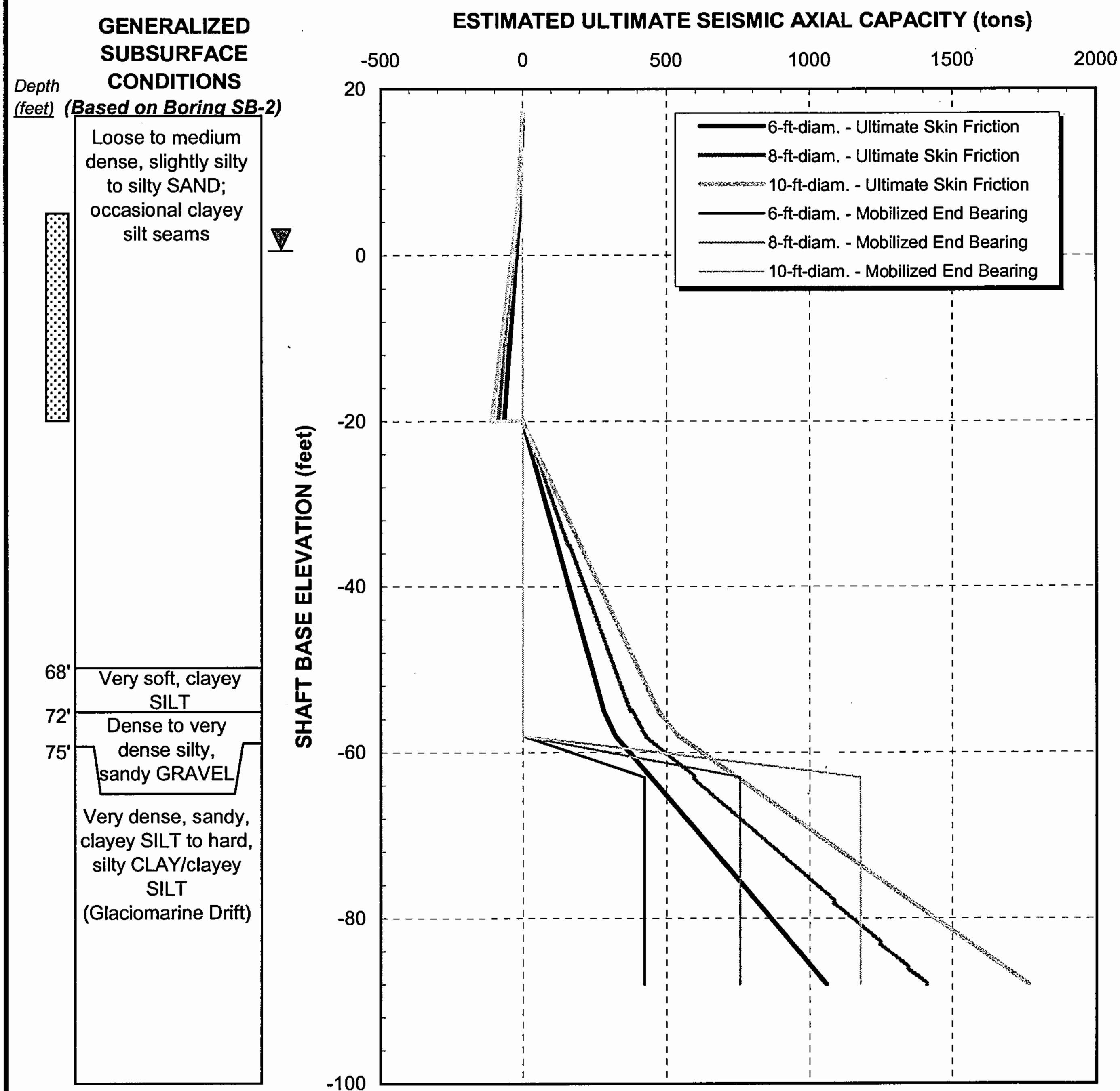
FIG. 47



NOTES

1. Ultimate Compressive Capacity = Ultimate Skin Friction + Mobilized End Bearing
   Allowable Compressive Capacity = Allowable Skin Friction + Mobilized End Bearing
   Allowable Skin Friction = Ultimate Skin Friction / Appropriate Factor of Safety
2. Calculations assume seismic loading conditions for a single shaft. Shaft group effects are not considered.
3. Estimated capacities assume that if casing is used, it will be removed after the shaft installation. If, however, the casing is left in place, grouting should be used to fill all potential voids around the casing and the estimated capacities given above should be re-evaluated.
4. Downdrag forces are included in the capacities above.
5. [dotted pattern] Indicates liquefied soils during the Design Earthquake event. (475-year return period)

South Park Bridge Project
Seattle, Washington

ESTIMATED SEISMIC AXIAL CAPACITY
DRILLED SHAFTS
Sta. 30+60, Boring SB-2

March 2004 21-1-09584-008

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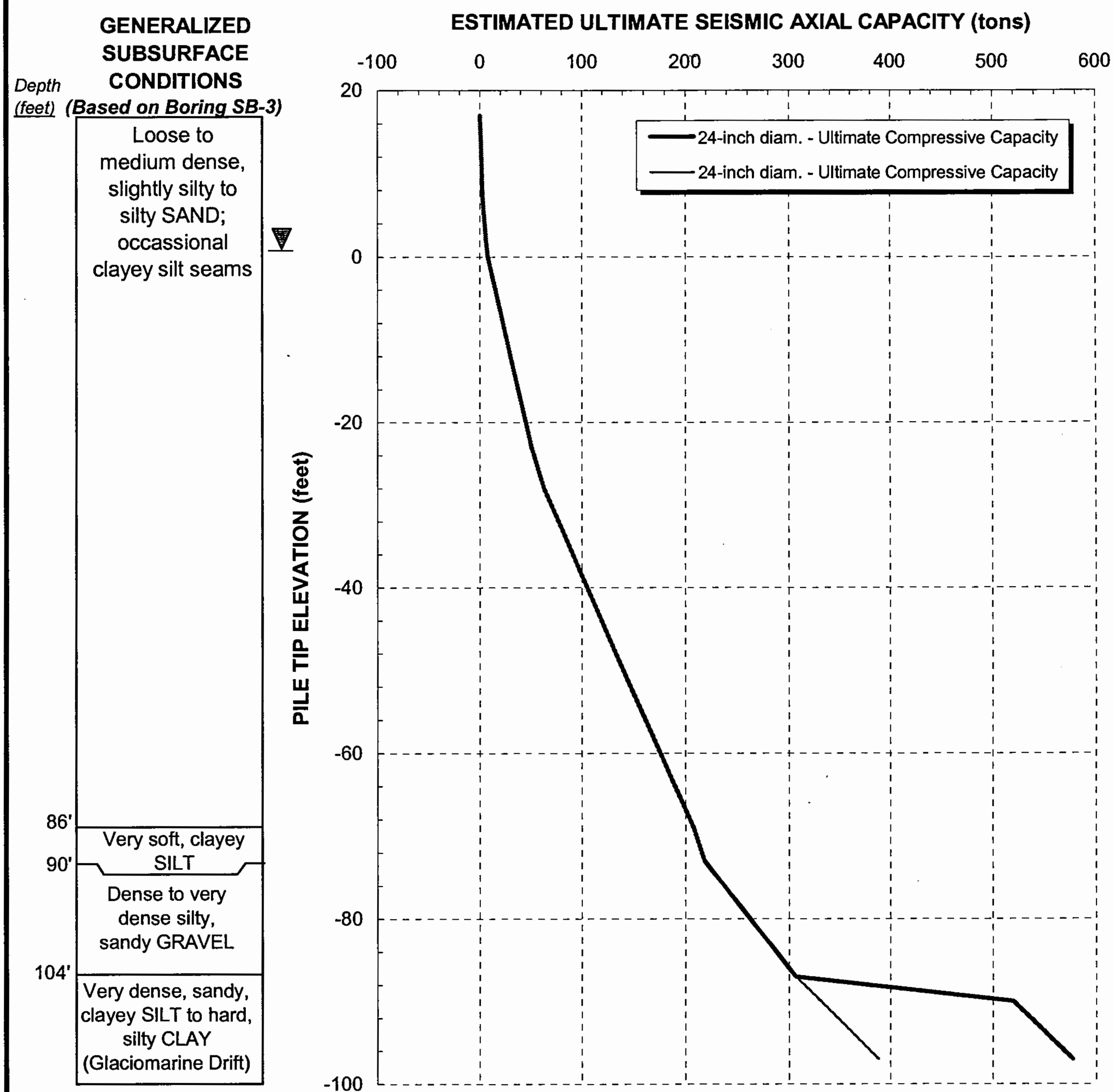
FIG. 48



NOTES

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2. Calculations assume seismic loading conditions for a single pile. Pile group effects are not considered.
3. Downdrag forces are included in the capacities above.
4. The steel pipe piles are assumed to be driven closed-end. A reinforced cutting shoe is recommended to achieve adequate penetrations into glacial deposits.
5. [ ] Indicates liquefied soils during the Design Earthquake event. (475-year Return Period)

South Park Bridge Project
Seattle, Washington

ESTIMATED SEISMIC AXIAL CAPACITY
STEEL PIPE PILES
Sta. 27+50, Boring SB-3

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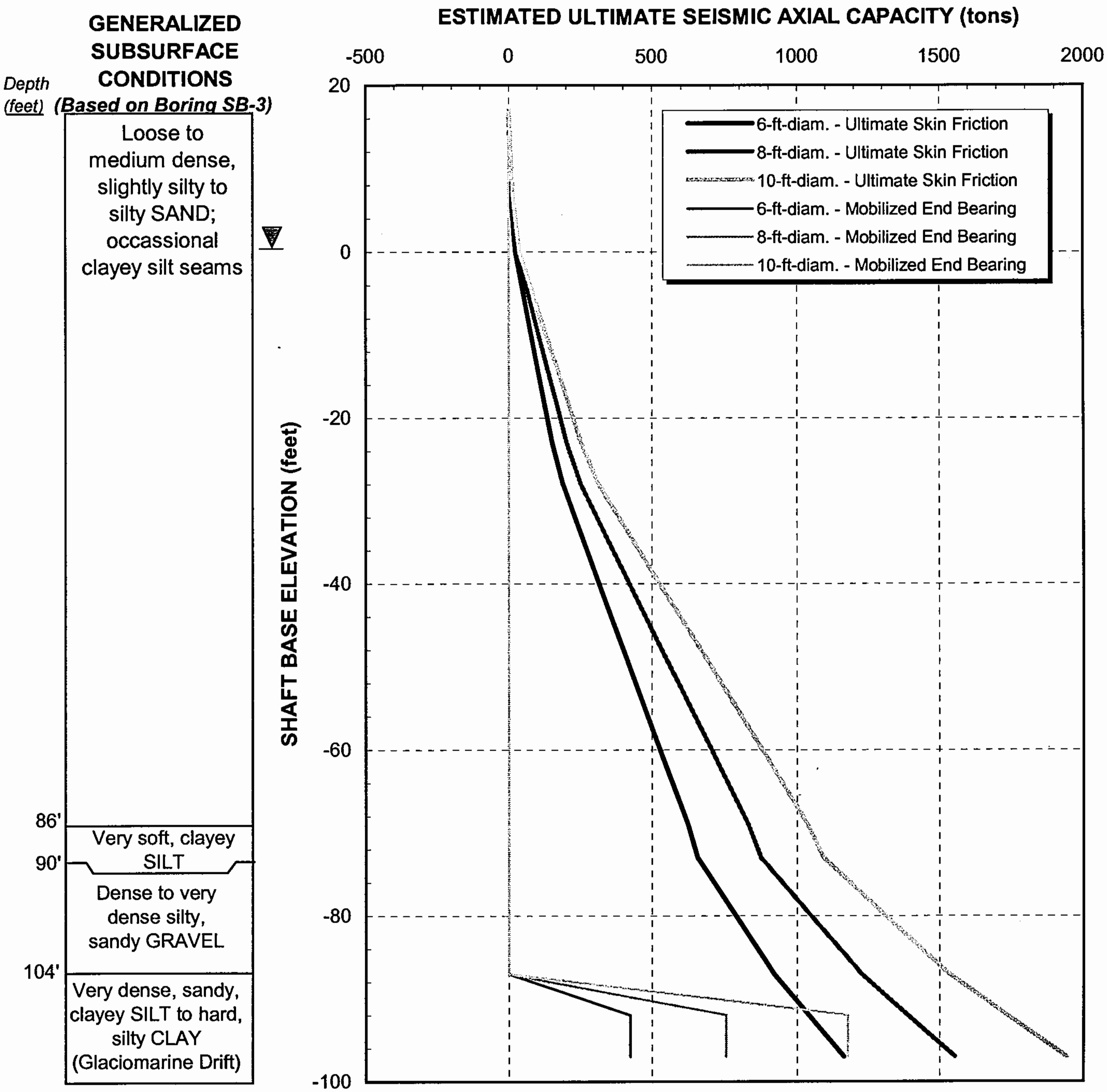
FIG. 49

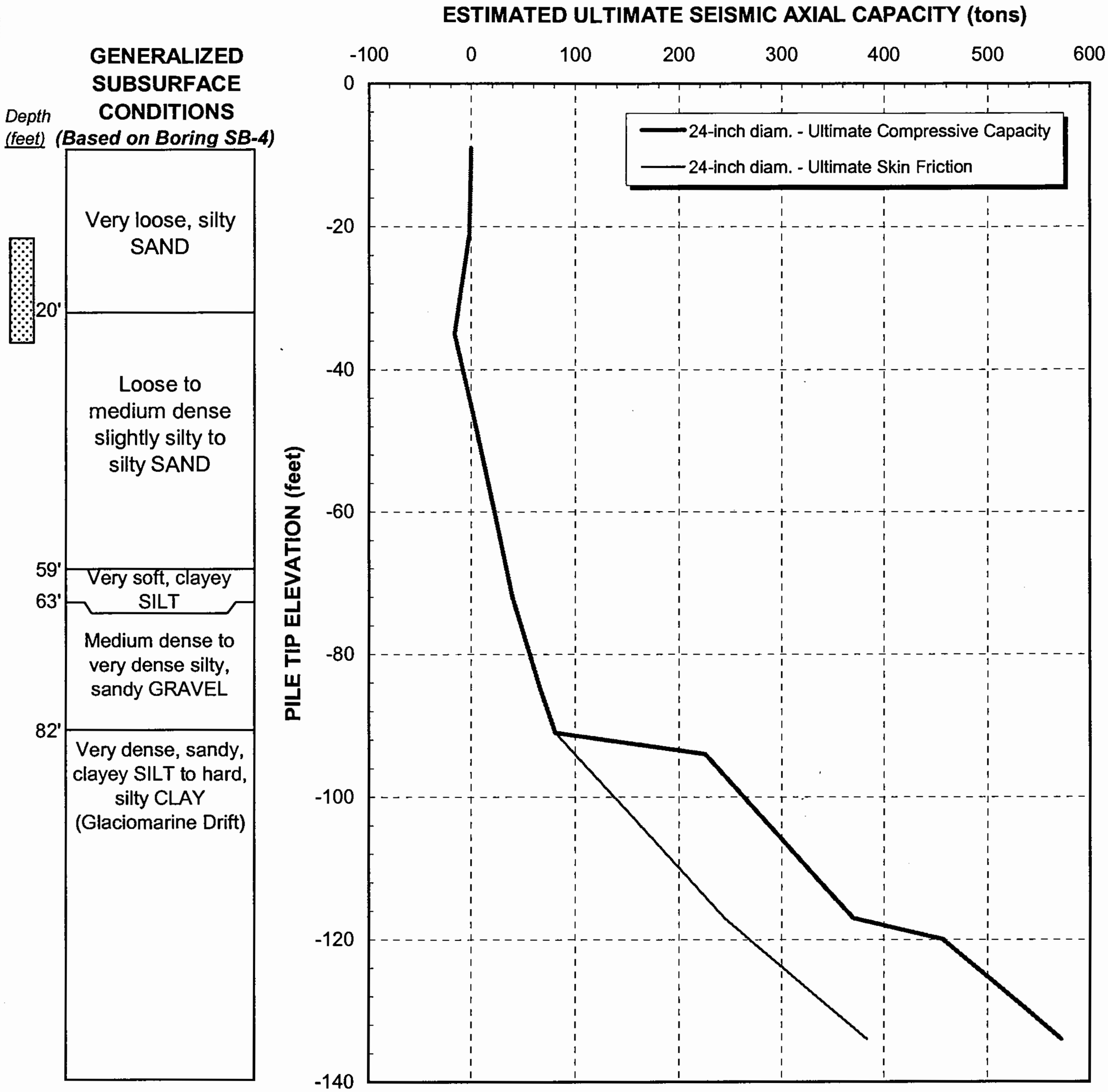
3/30/2004-Downdrag SB_3.xls,Plots-lmm



NOTES

1. Ultimate Compressive Capacity = Ultimate Skin Friction + Mobilized End Bearing
   Allowable Compressive Capacity = Allowable Skin Friction + Mobilized End Bearing
   Allowable Skin Friction = Ultimate Skin Friction / Appropriate Factor of Safety
2. Calculations assume seismic loading conditions for a single shaft. Shaft group effects are not considered.
3. Estimated capacities assume that if casing is used, it will be removed after the shaft installation. If, however, the casing is left in place, grouting should be used to fill all potential voids around the casing and the estimated capacities given above should be re-evaluated.
4. Downdrag forces are included in the capacities above.
5. Indicates liquefied soils during the Design Earthquake event. (475-year return period)

South Park Bridge Project Seattle, Washington	
ESTIMATED SEISMIC AXIAL CAPACITY DRILLED SHAFTS **Sta. 27+50, Boring SB-3**	
March 2004	21-1-09584-008
SHANNON & WILSON, INC. Geotechnical and Environmental Consultants	**FIG. 50**



NOTES

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2. Calculations assume seismic loading conditions for a single pile. Pile group effects are not considered.
3. Downdrag forces are included in the capacities above.
4. The steel pipe piles are assumed to be driven closed-end. A reinforced cutting shoe is recommended to achieve adequate penetrations into glacial deposits.
5. Indicates liquefied soils during the Design Earthquake event. (475-year Return Period)

South Park Bridge Project
Seattle, Washington

ESTIMATED SEISMIC AXIAL CAPACITY
STEEL PIPE PILES
Sta. 25+45, Boring SB-4

March 2004 21-1-09584-008

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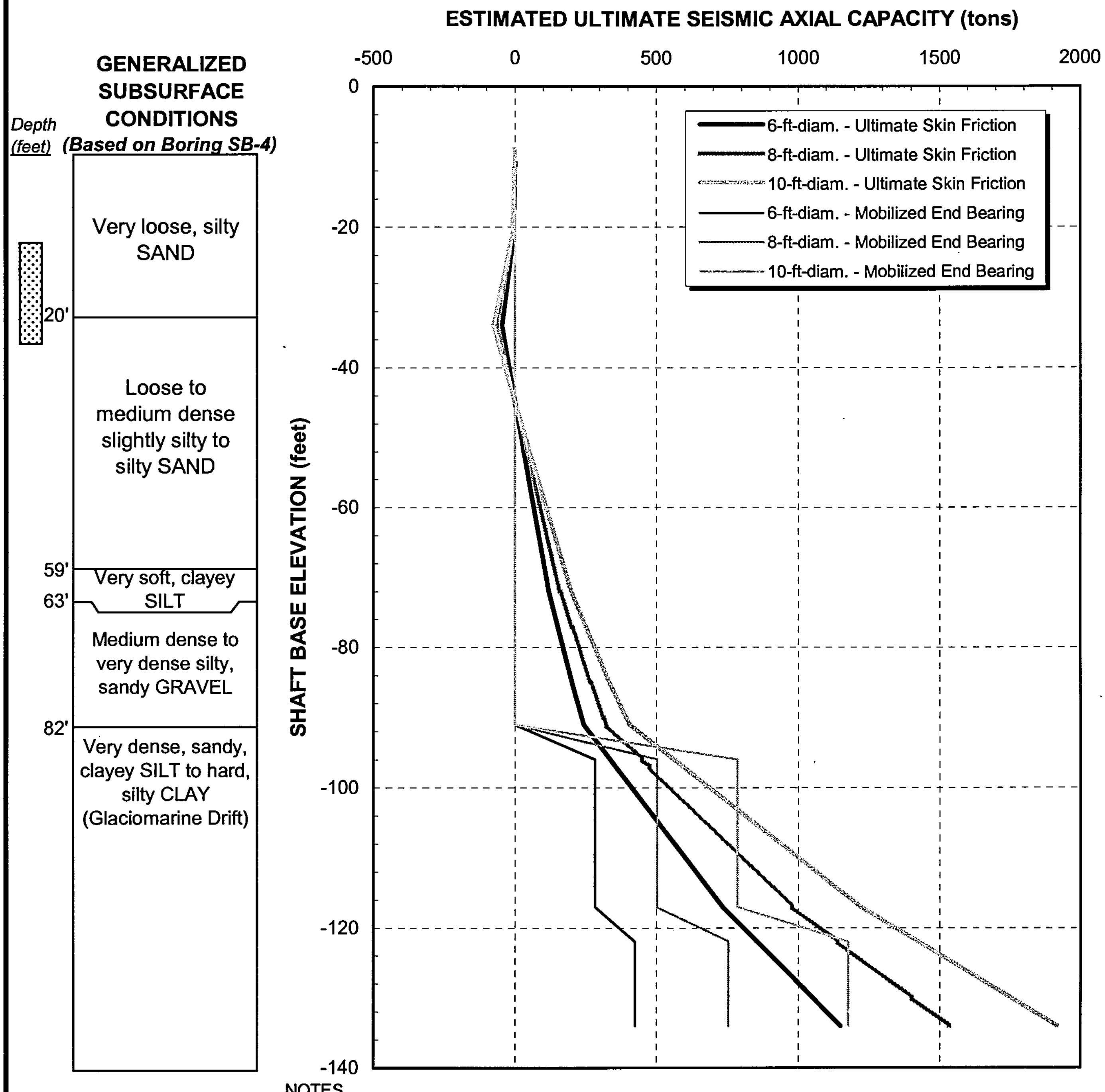
Geotechnical and Environmental Consultants

FIG. 51



NOTES

1. Ultimate Compressive Capacity = Ultimate Skin Friction + Mobilized End Bearing
   Allowable Compressive Capacity = Allowable Skin Friction + Mobilized End Bearing
   Allowable Skin Friction = Ultimate Skin Friction / Appropriate Factor of Safety
2. Calculations assume seismic loading conditions for a single shaft. Shaft group effects are not considered.
3. Estimated capacities assume that if casing is used, it will be removed after the shaft installation. If, however, the casing is left in place, grouting should be used to fill all potential voids around the casing and the estimated capacities given above should be re-evaluated.
4. Downdrag forces are included in the capacities above.
5. Indicates liquefied soils during the Design Earthquake event. (475-year return period)

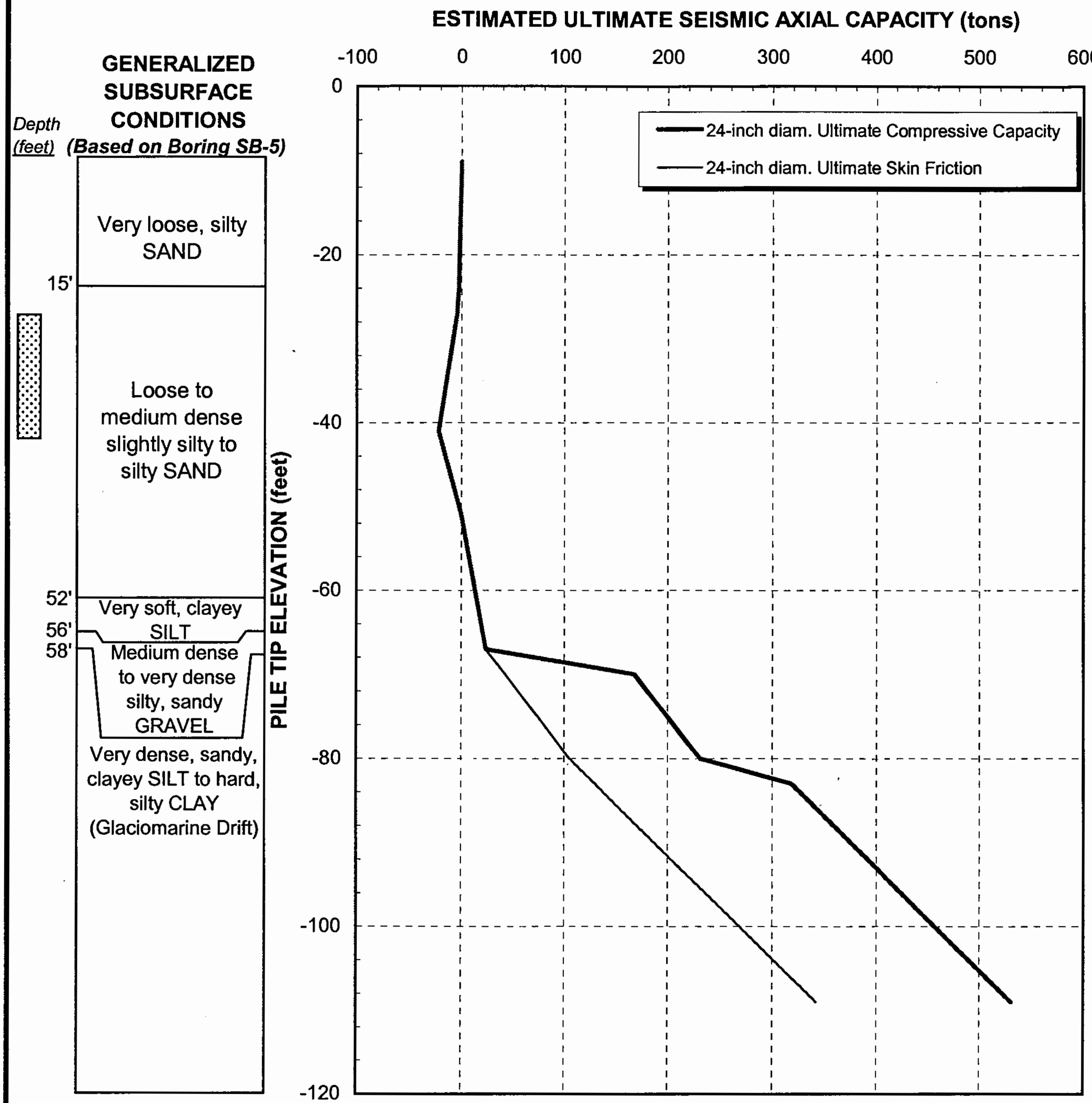
South Park Bridge Project
Seattle, Washington

ESTIMATED SEISMIC AXIAL CAPACITY
DRILLED SHAFTS
Sta. 25+45, Boring SB-4

March 2004 21-1-09584-008

SHANNON & WILSON, INC.
Geotechnical and Environmental Consultants

FIG. 52



NOTES

1. Ultimate compressive capacity is a summation of ultimate skin friction and ultimate end bearing. We recommend that an appropriate factor-of-safety be applied to the ultimate values to obtain the allowable design loads, depending on the design requirements and loading conditions.

2. Calculations assume seismic loading conditions for a single pile. Pile group effects are not considered.

3. Downdrag forces are included in the capacities above.

4. The steel pipe piles are assumed to be driven closed-end. A reinforced cutting shoe is recommended to achieve adequate penetrations into glacial deposits.

5. [dotted box] Indicates liquefied soils during the Design Earthquake event. (475-year Return Period)

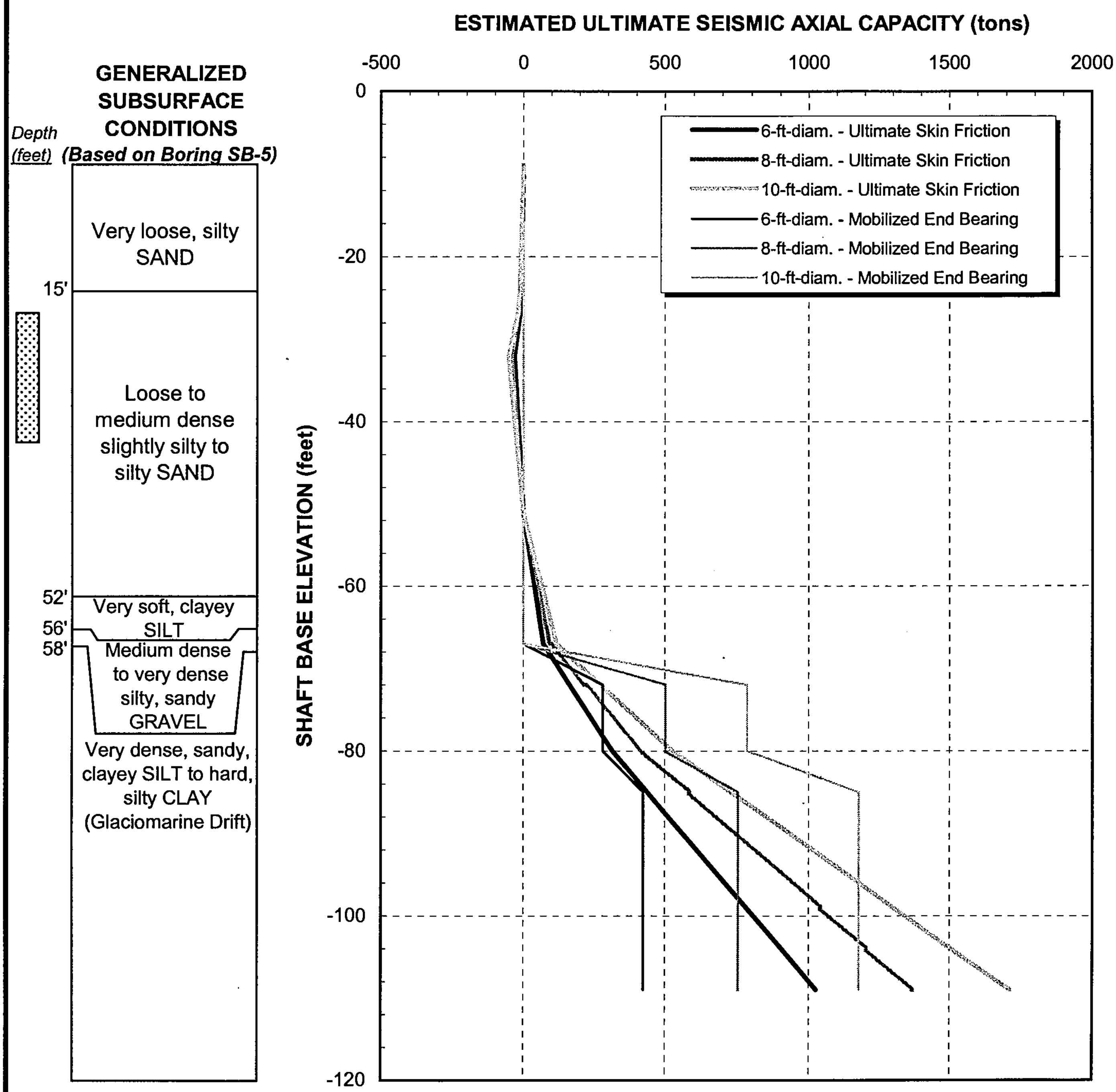
South Park Bridge Project
Seattle, Washington

ESTIMATED SEISMIC AXIAL CAPACITY STEEL PIPE PILES
Sta. 23+25, Boring SB-5

March 2004 21-1-09584-008

SHANNON & WILSON, INC.
Geotechnical and Environmental Consultants

FIG. 53



NOTES

1. Ultimate Compressive Capacity = Ultimate Skin Friction + Mobilized End Bearing
   Allowable Compressive Capacity = Allowable Skin Friction + Mobilized End Bearing
   Allowable Skin Friction = Ultimate Skin Friction / Appropriate Factor of Safety
2. Calculations assume seismic loading conditions for a single shaft. Shaft group effects are not considered.
3. Estimated capacities assume that if casing is used, it will be removed after the shaft installation. If, however, the casing is left in place, grouting should be used to fill all potential voids around the casing and the estimated capacities given above should be re-evaluated.
4. Downdrag forces are included in the capacities above.
5. Indicates liquefied soils during the Design Earthquake event. (475-year return period)

South Park Bridge Project
Seattle, Washington

ESTIMATED SEISMIC AXIAL CAPACITY DRILLED SHAFTS
Sta. 23+25, Boring SB-5

March 2004 | 21-1-09584-008

SHANNON & WILSON, INC.
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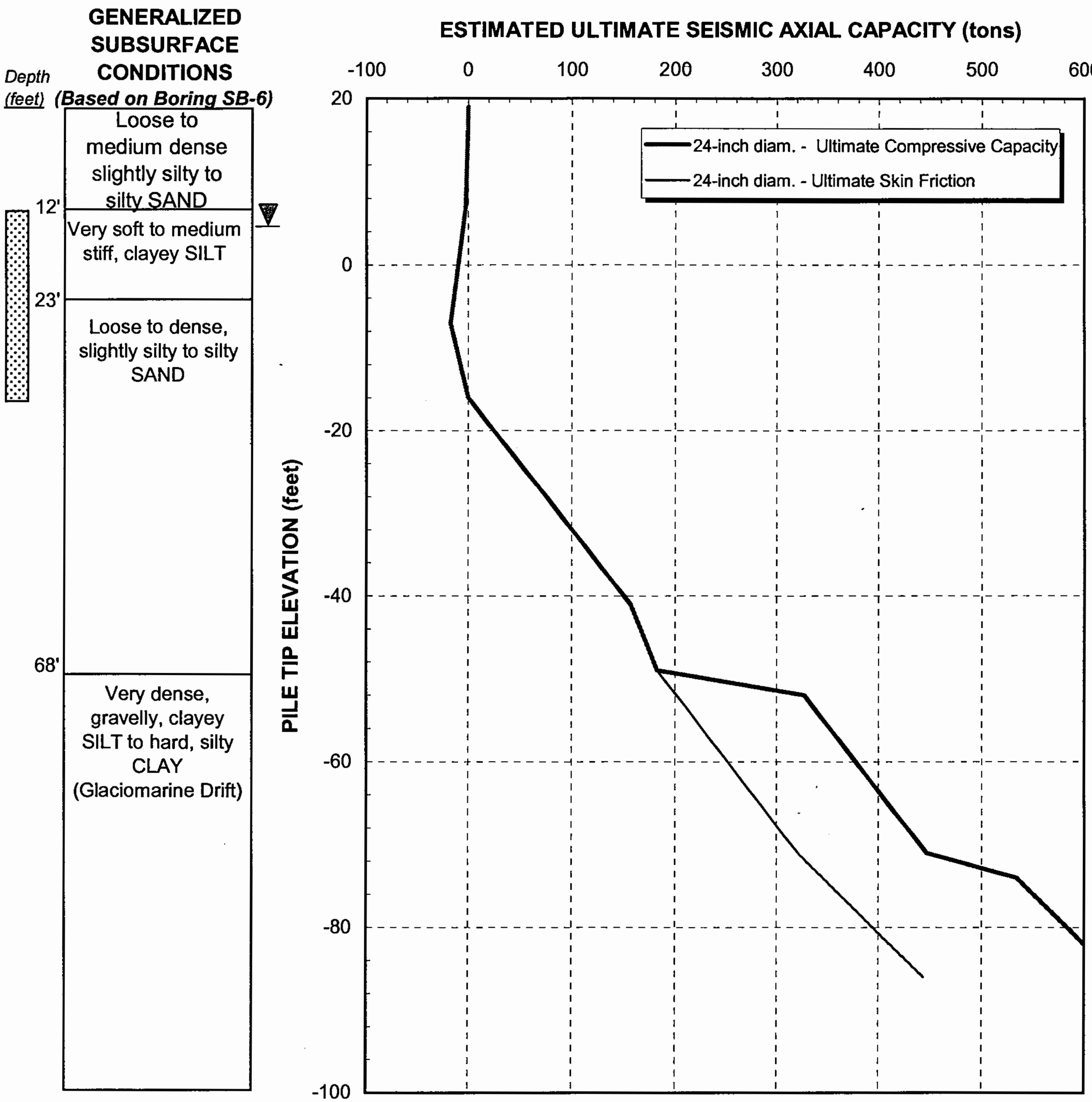
FIG. 54



NOTES

1. Ultimate compressive capacity is a summation of ultimate skin friction and ultimate end bearing. We recommend that an appropriate factor-of-safety be applied to the ultimate values to obtain the allowable design loads, depending on the design requirements and loading conditions.
2. Calculations assume seismic loading conditions for a single pile. Pile group effects are not considered.
3. Downdrag forces are included in the capacities above.
4. The steel pipe piles are assumed to be driven closed-end. A reinforced cutting shoe is recommended to achieve adequate penetrations into glacial deposits.
5. Indicates liquefied soils during the Design Earthquake event. (475-year Return Period)

South Park Bridge Project
Seattle, Washington

ESTIMATED SEISMIC AXIAL CAPACITY
STEEL PIPE PILES
Sta. 20+00, Boring SB-6

March 2004 | 21-1-09584-008

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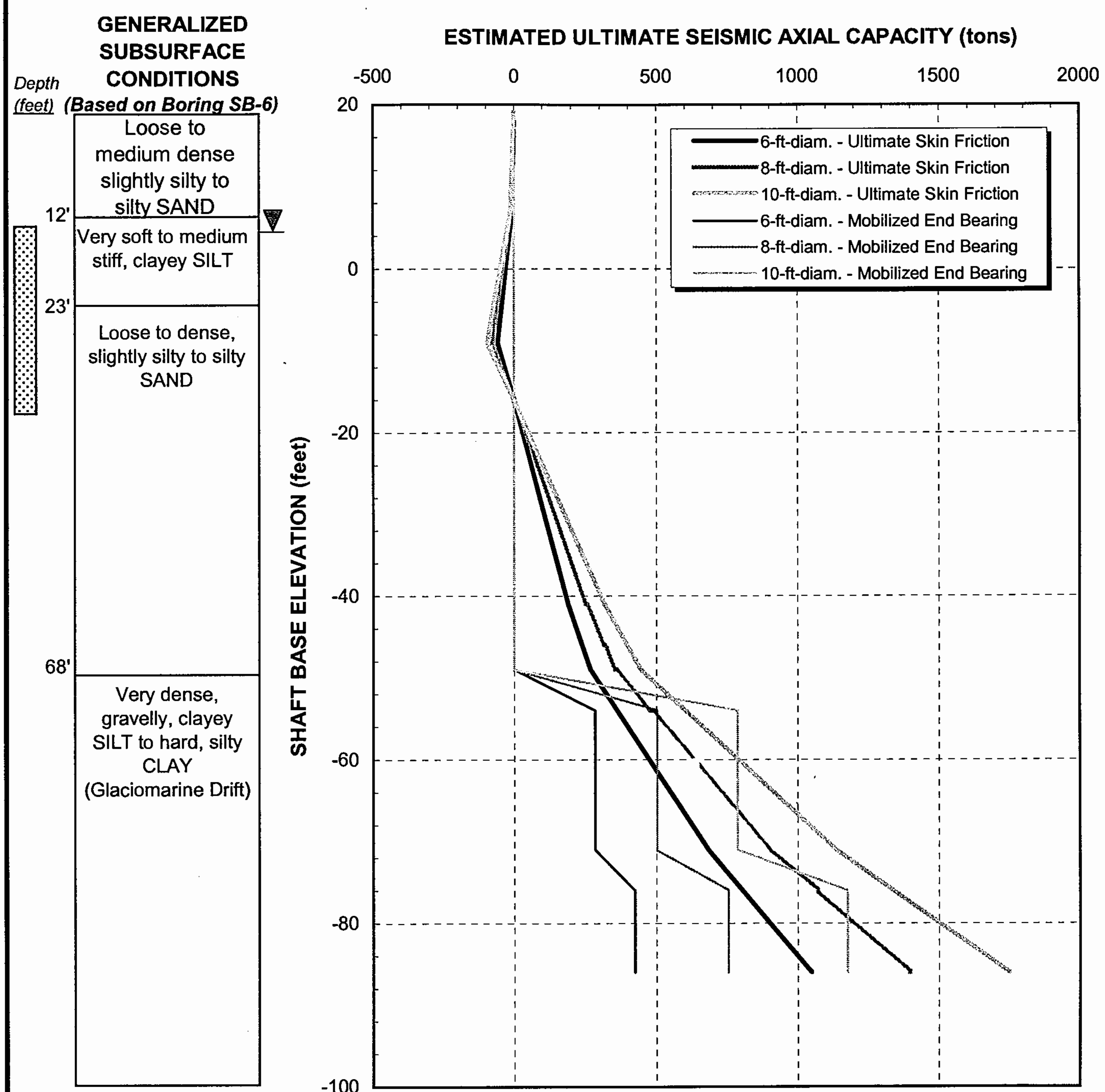
FIG. 55



NOTES

1. Ultimate Compressive Capacity = Ultimate Skin Friction + Mobilized End Bearing
   Allowable Compressive Capacity = Allowable Skin Friction + Mobilized End Bearing
   Allowable Skin Friction = Ultimate Skin Friction / Appropriate Factor of Safety
2. Calculations assume seismic loading conditions for a single shaft. Shaft group effects are not considered.
3. Estimated capacities assume that if casing is used, it will be removed after the shaft installation. If, however, the casing is left in place, grouting should be used to fill all potential voids around the casing and the estimated capacities given above should be re-evaluated.
4. Downdrag forces are included in the capacities above.
5. Indicates liquefied soils during the Design Earthquake event. (475-year return period)

South Park Bridge Project
Seattle, Washington

ESTIMATED SEISMIC AXIAL CAPACITY
DRILLED SHAFTS
Sta. 20+00, Boring SB-6

March 2004 21-1-09584-008

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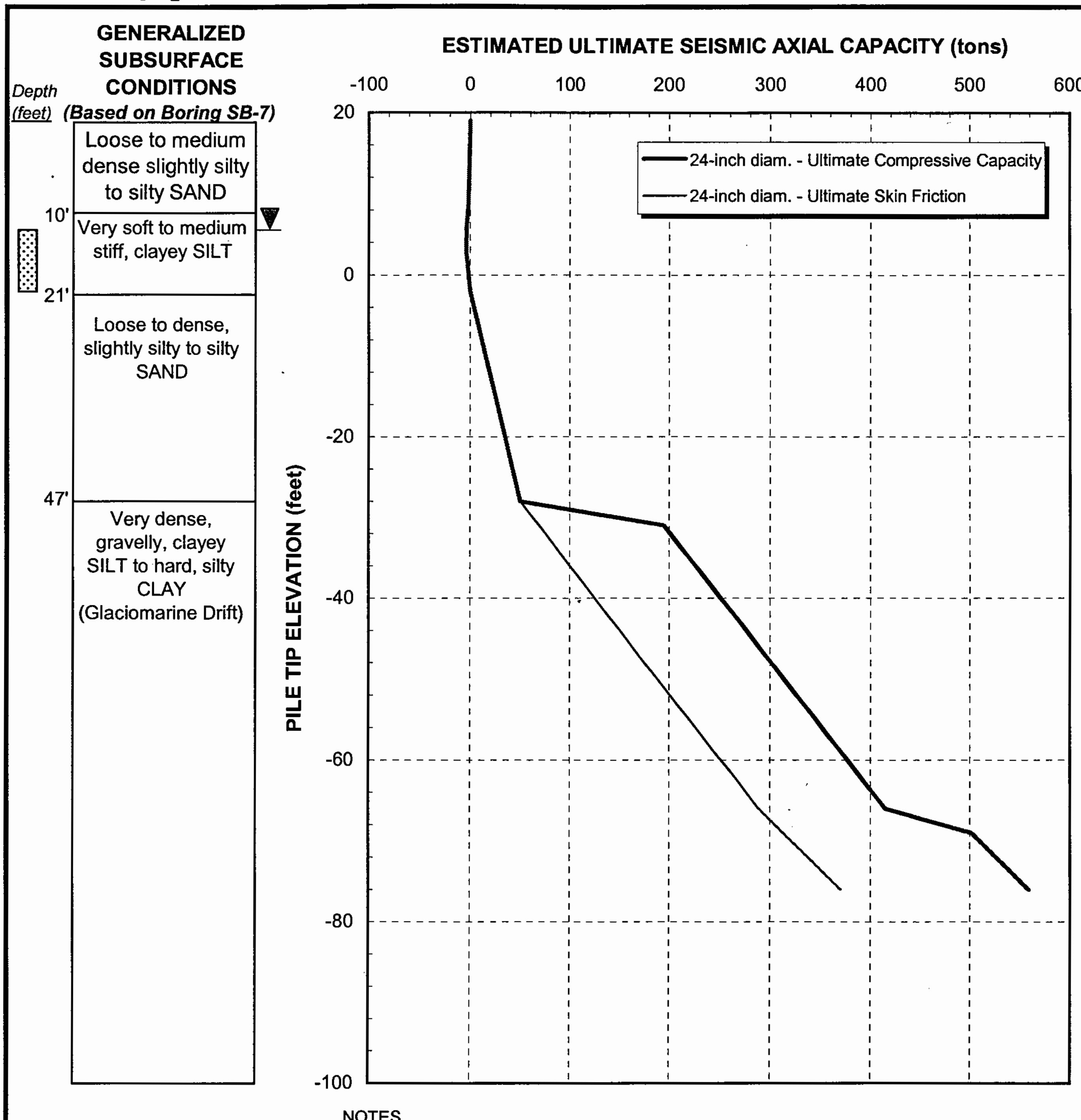
FIG. 56



NOTES

1. Ultimate compressive capacity is a summation of ultimate skin friction and ultimate end bearing. We recommend that an appropriate factor-of-safety be applied to the ultimate values to obtain the allowable design loads, depending on the design requirements and loading conditions.

2. Calculations assume seismic loading conditions for a single pile. Pile group effects are not considered.

3. Downdrag forces are included in the capacities above.

4. The steel pipe piles are assumed to be driven closed-end. A reinforced cutting shoe is recommended to achieve adequate penetrations into glacial deposits.

5. [::::] Indicates liquefied soils during the Design Earthquake event. (475-year Return Period)

South Park Bridge Project
Seattle, Washington

ESTIMATED SEISMIC AXIAL CAPACITY
STEEL PIPE PILES
Sta. 17+40, Boring SB-7

March 2004 21-1-09584-008

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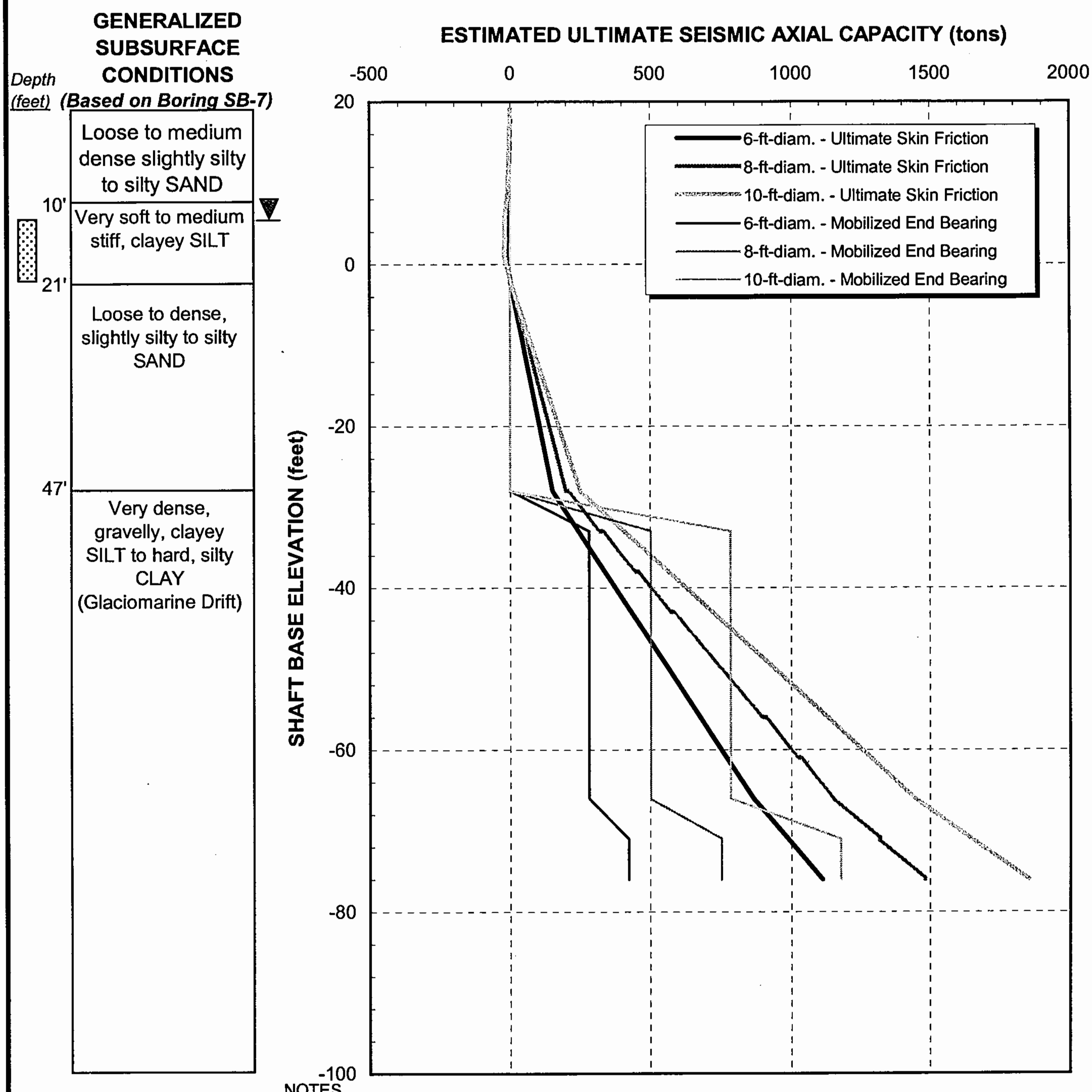
FIG. 57



NOTES

1. Ultimate Compressive Capacity = Ultimate Skin Friction + Mobilized End Bearing
   Allowable Compressive Capacity = Allowable Skin Friction + Mobilized End Bearing
   Allowable Skin Friction = Ultimate Skin Friction / Appropriate Factor of Safety
2. Calculations assume seismic loading conditions for a single shaft. Shaft group effects are not considered.
3. Estimated capacities assume that if casing is used, it will be removed after the shaft installation. If, however, the casing is left in place, grouting should be used to fill all potential voids around the casing and the estimated capacities given above should be re-evaluated.
4. Downdrag forces are included in the capacities above.
5. ░ Indicates liquefied soils during the Design Earthquake event. (475-year return period)

South Park Bridge Project
Seattle, Washington

ESTIMATED SEISMIC AXIAL CAPACITY
DRILLED SHAFTS
Sta. 17+40, Boring SB-7

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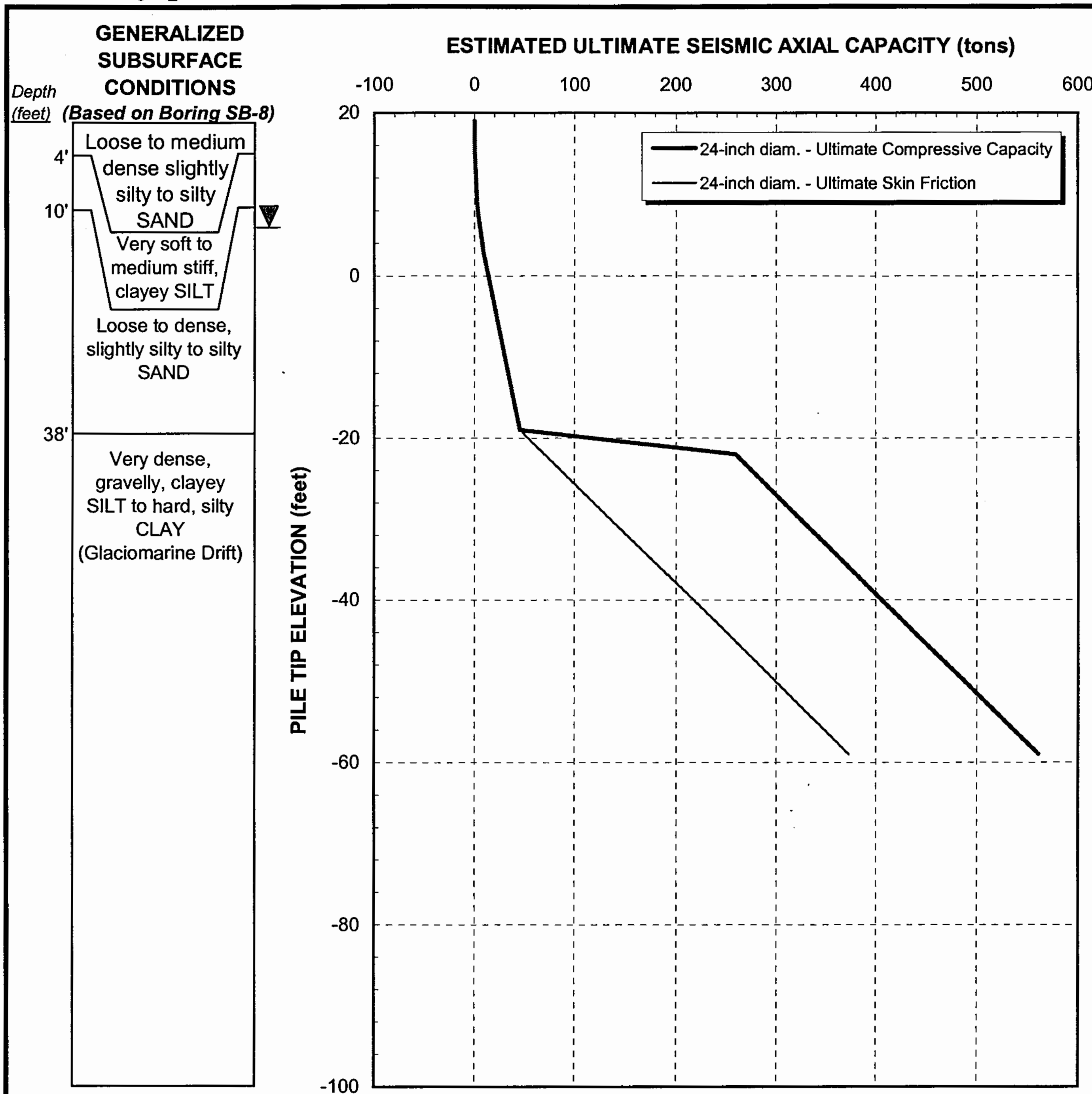
FIG. 58



NOTES

1. Ultimate compressive capacity is a summation of ultimate skin friction and ultimate end bearing. We recommend that an appropriate factor-of-safety be applied to the ultimate values to obtain the allowable design loads, depending on the design requirements and loading conditions.
2. Calculations assume seismic loading conditions for a single pile. Pile group effects are not considered.
3. Downdrag forces are included in the capacities above.
4. The steel pipe piles are assumed to be driven closed-end. A reinforced cutting shoe is recommended to achieve adequate penetrations into glacial deposits.
5. Indicates liquefied soils during the Design Earthquake event. (475-year Return Period)

South Park Bridge Project
Seattle, Washington

ESTIMATED SEISMIC AXIAL CAPACITY
STEEL PIPE PILES
Sta. 15+20, Boring SB-8

March 2004 — 21-1-09584-008

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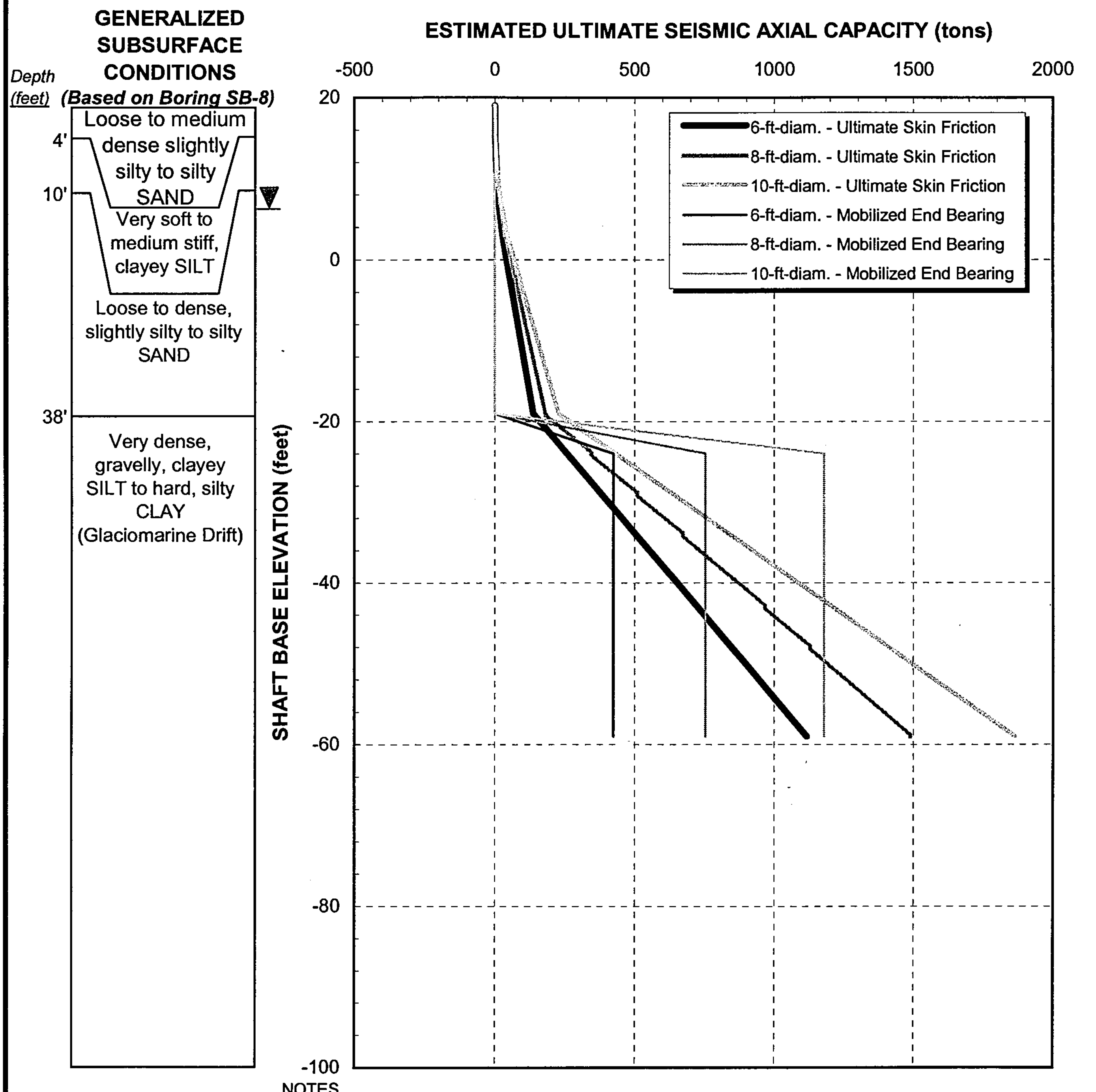
FIG. 59



NOTES

1. Ultimate Compressive Capacity = Ultimate Skin Friction + Mobilized End Bearing
   Allowable Compressive Capacity = Allowable Skin Friction + Mobilized End Bearing
   Allowable Skin Friction = Ultimate Skin Friction / Appropriate Factor of Safety
2. Calculations assume seismic loading conditions for a single shaft. Shaft group effects are not considered.
3. Estimated capacities assume that if casing is used, it will be removed after the shaft installation. If, however, the casing is left in place, grouting should be used to fill all potential voids around the casing and the estimated capacities given above should be re-evaluated.
4. Downdrag forces are included in the capacities above.
5. ⬚ Indicates liquefied soils during the Design Earthquake event. (475-year return period)

South Park Bridge Project
Seattle, Washington

ESTIMATED SEISMIC AXIAL CAPACITY
DRILLED SHAFTS
Sta. 15+20, Boring SB-8

March 2004 21-1-09584-008

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FIG. 60

APPENDIX A

SUBSURFACE EXPLORATIONS

APPENDIX A

SUBSURFACE EXPLORATIONS

TABLE OF CONTENTS

LIST OF FIGURES

APPENDIX A

SUBSURFACE EXPLORATIONS

A.1 GENERAL

Eight soil borings, completed June 24 to July 24, 2002, were drilled for the South Park Bridge project located in King County, Washington. The locations of the borings are shown on the site plan (Figure 3) presented in this report. These locations were determined after drilling by a survey crew under subcontract to Parsons Brinckerhoff (PB).

A.2 BORINGS

Eight borings were drilled, designated SB-1 through SB-8, to evaluate subsurface conditions and develop parameters for engineering studies; six of the borings were drilled on land, and two were drilled over-water. The logs for the borings are presented as Figures A-2 through A-9. The Unified Soil Classification System (USCS), as described on Figure A-1, was used to classify the soils encountered in the borings.

The borings were drilled by Geotech Explorations, Inc., under subcontract to Shannon & Wilson, Inc. The upper 20 feet of the land borings were drilled with a truck-mounted rig using hollow-stem auger drilling techniques. The remaining footage of the land borings and the over-water borings were drilled using mud-rotary drilling techniques. Depths of drilling ranged from 90 to 131 feet below the ground surface or mudline. In general, the mud-rotary drilling procedure consisted of drilling the formation materials and removing the cuttings by circulation of drilling mud. The cuttings were deposited in a settling tank at the ground surface. The drilling mud used was a mixture of water and baroid-zeogel (bentonite). After each boring was completed, the hole was filled with bentonite chips to seal the hole.

Standard Penetration Tests (SPTs) were generally performed in the borings at 2.5-foot intervals in the upper 20 feet and at 5-foot intervals thereafter. The tests were performed in general accordance with the American Society for Testing of Materials (ASTM) Designation: D-1586, Standard Method for Penetration Test and Split-Barrel Sampling of Soils. The SPT consists of driving a 2-inch outside-diameter (O.D.) split-spoon sampler a total distance of 18 inches into the bottom of the boring with a 140-pound hammer falling 30 inches. The number of blows required to cause the last 12 inches of penetration is termed the Standard Penetration Resistance (N-value). When penetration resistances exceeded 50 blows for 6 inches or less of penetration, the test was terminated and the number of blows and the corresponding penetration recorded. The N-values were recorded by an engineer from our firm and plotted on the boring logs. The N-values provide a means for evaluating the relative consistency (stiffness) of cohesive soils and the relative compactness or density of cohesionless (granular) soils. A further description of the N-values and how they relate to soil characteristics is presented on Figure A-1.

The split-spoon sampler used during the penetration testing recovers a disturbed sample of the soil. The samples were field classified and recorded on the logs by our field representative, sealed in jars, and returned to our laboratory for testing.

Shannon & Wilson, Inc. (S&W), uses a soil classification system modified from the Unified Soil Classification System (USCS). Elements of the USCS and other definitions are provided on this and the following page. Soil descriptions are based on visual-manual procedures (ASTM D 2488-93) unless otherwise noted.

S&W CLASSIFICATION OF SOIL CONSTITUENTS

- MAJOR constituents compose more than 50 percent, by weight, of the soil. Major consituents are capitalized (i.e., SAND).
- Minor constituents compose 12 to 50 percent of the soil and precede the major constituents (i.e., silty SAND). Minor constituents preceded by "slightly" compose 5 to 12 percent of the soil (i.e., slightly silty SAND).
- Trace constituents compose 0 to 5 percent of the soil (i.e., slightly silty SAND, trace of gravel).

MOISTURE CONTENT DEFINITIONS

Dry	Absence of moisture, dusty, dry to the touch
Moist	Damp but no visible water
Wet	Visible free water, from below water table

ABBREVIATIONS

ATD	At Time of Drilling
Elev.	Elevation
ft	feet
FeO	Iron Oxide
MgO	Magnesium Oxide
HSA	Hollow Stem Auger
ID	Inside Diameter
in	inches
lbs	pounds
Mon.	Monument cover
N	Blows for last two 6-inch increments
NA	Not applicable or not available
NP	Non plastic
OD	Outside diameter
OVA	Organic vapor analyzer
PID	Photo-ionization detector
ppm	parts per million
PVC	Polyvinyl Chloride
SS	Split spoon sampler
SPT	Standard penetration test
USC	Unified soil classification
WLI	Water level indicator

GRAIN SIZE DEFINITION

DESCRIPTION	SIEVE NUMBER AND/OR SIZE
FINES	< #200 (0.08 mm)
SAND*	
- Fine	#200 to #40 (0.08 to 0.4 mm)
- Medium	#40 to #10 (0.4 to 2 mm)
- Coarse	#10 to #4 (2 to 5 mm)
GRAVEL*	
- Fine	#4 to 3/4 inch (5 to 19 mm)
- Coarse	3/4 to 3 inches (19 to 76 mm)
COBBLES	3 to 12 inches (76 to 305 mm)
BOULDERS	> 12 inches (305 mm)

* Unless otherwise noted, sand and gravel, when present, range from fine to coarse in grain size.

RELATIVE DENSITY / CONSISTENCY

COARSE-GRAINED SOILS		FINE-GRAINED SOILS	
N, SPT, BLOWS/FT.	RELATIVE DENSITY	N, SPT, BLOWS/FT.	RELATIVE CONSISTENCY
0 - 4	Very loose	Under 2	Very soft
4 - 10	Loose	2 - 4	Soft
10 - 30	Medium dense	4 - 8	Medium stiff
30 - 50	Dense	8 - 15	Stiff
Over 50	Very dense	15 - 30	Very stiff
		Over 30	Hard

WELL AND OTHER SYMBOLS

Bent. Cement Grout	Surface Cement Seal
Bentonite Grout	Asphalt or Cap
Bentonite Chips	Slough
Silica Sand	Bedrock
PVC Screen	
Vibrating Wire	

South Park Bridge Project
King County, Washington

SOIL CLASSIFICATION AND LOG KEY

November 2003 21-1-09584-008

SHANNON & WILSON, INC.
Geotechnical and Environmental Consultants

FIG. A-1
Sheet 1 of 2

UNIFIED SOIL CLASSIFICATION SYSTEM (USCS)
(From ASTM D 2487-98 & 2488-93)

MAJOR DIVISIONS			GROUP/GRAPHIC SYMBOL		TYPICAL DESCRIPTION
COARSE-GRAINED SOILS *(more than 50% retained on No. 200 sieve)*	Gravels *(more than 50% of coarse fraction retained on No. 4 sieve)*	Clean Gravels *(less than 5% fines)*	GW		Well-graded gravels, gravels, gravel/sand mixtures, little or no fines
			GP		Poorly graded gravels, gravel-sand mixtures, little or no fines
		Gravels with Fines *(more than 12% fines)*	GM		Silty gravels, gravel-sand-silt mixtures
			GC		Clayey gravels, gravel-sand-clay mixtures
	Sands *(50% or more of coarse fraction passes the No. 4 sieve)*	Clean Sands *(less than 5% fines)*	SW		Well-graded sands, gravelly sands, little or no fines
			SP		Poorly graded sand, gravelly sands, little or no fines
		Sands with Fines *(more than 12% fines)*	SM		Silty sands, sand-silt mixtures
			SC		Clayey sands, sand-clay mixtures
FINE-GRAINED SOILS *(50% or more passes the No. 200 sieve)*	Silts and Clays *(liquid limit less than 50)*	Inorganic	ML		Inorganic silts of low to medium plasticity, rock flour, sandy silts, gravelly silts, or clayey silts with slight plasticity
			CL		Inorganic clays of low to medium plasticity, gravelly clays, sandy clays, silty clays, lean clays
		Organic	OL		Organic silts and organic silty clays of low plasticity
	Silts and Clays *(liquid limit 50 or more)*	Inorganic	MH		Inorganic silts, micaceous or diatomaceous fine sands or silty soils, elastic silt
			CH		Inorganic clays or medium to high plasticity, sandy fat clay, or gravelly fat clay
		Organic	OH		Organic clays of medium to high plasticity, organic silts
HIGHLY-ORGANIC SOILS	Primarily organic matter, dark in color, and organic odor		PT		Peat, humus, swamp soils with high organic content (see ASTM D 4427)

NOTES

1. Dual symbols *(symbols separated by a hyphen, i.e., SP-SM, slightly silty fine SAND)*are used for soils with between 5% and 12% fines or when the liquid limit and plasticity index values plot in the CL-ML area of the plasticity chart.

2. Borderline symbols *(symbols separated by a slash, i.e., CL/ML, silty CLAY/clayey SILT; GW/SW, sandy GRAVEL/gravelly SAND)* indicate that the soil may fall into one of two possible basic groups.

South Park Bridge Project
King County, Washington

SOIL CLASSIFICATION AND LOG KEY

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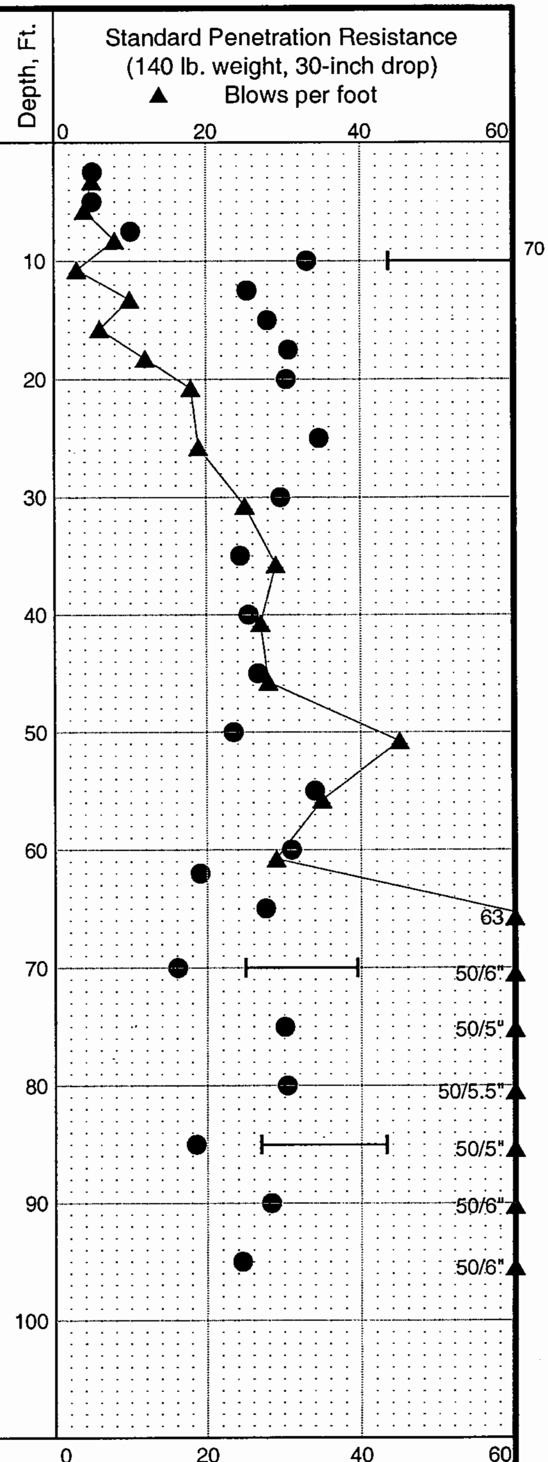
FIG. A-1
Sheet 2 of 2

BORING_CLASS2 21-09584.GPJ SWNEW.GDT 3/30/04

SOIL DESCRIPTION Surface Elevation: Approx. 17.0 Ft.	Depth, Ft.	PID, ppm	Samples
Asphalt.	0.2		
Loose, brown to dark brown, slightly silty, slightly gravelly, fine to medium SAND; moist; (Fill) SP/SP-SM.	10.5	0, 0, 0, 0	1, 2, 3, 4
Soft, dark brown to brown, slightly fine sandy, clayey SILT; moist; scattered organics; (Alluvial Deposits) MH.	12.0	0, 0	5, 6
Loose to medium dense, dark brown to black, trace to slightly silty, fine SAND; wet; occasional clayey silt seams; (Alluvial Deposits) SP.	25.0		7, 8, 9
Medium dense to dense, dark brown to dark gray, trace to slightly silty, fine to medium SAND; wet; trace of coarse sand, massive; (Alluvial Deposits) SP/SP-SM.	60.0		10, 11, 12, 13, 14, 15
Very stiff, dark brown, fine sandy SILT; (Estuarine Deposits) ML.	63.0		16
Very dense, gray, silty, sandy GRAVEL; wet; numerous shell fragments; (Estuarine Deposits) GM.	66.0		18
Hard, gray, silty CLAY; moist; trace of fine sand and gravel; (Glaciomarine Drift) CL.	78.0		19, 20
Hard, gray, clayey SILT; moist; occasional organics, massive; (Glaciomarine Drift) ML.	93.0		21, 22, 23
Very dense, gray, silty fine SAND; wet; scattered mica partings and decayed wood debris; (Glaciomarine Drift) SM.	96.0		24
BOTTOM OF BORING COMPLETED 7/9/2003			

Ground Water: During Drilling ⊻



LEGEND

* Sample Not Recovered
⌶ Standard Penetration Test
⊻ Ground Water Level ATD

● % Water Content

Plastic Limit ├─●─┤ Liquid Limit
Natural Water Content

NOTES

1. The boring was performed using HSA and Rotary Combined drilling methods.
2. The stratification lines represent the approximate boundaries between soil types, and the transition may be gradual.
3. The discussion in the text of this report is necessary for a proper understanding of the nature of the subsurface materials.
4. Groundwater level, if indicated above, is for the date specified and may vary.
5. Refer to KEY for explanation of symbols, codes and definitions.
6. USCS designation is based on visual-manual classification and selected lab testing.

South Park Bridge Project
King County, Washington

LOG OF BORING SB-1

November 2003 21-1-09584-008

SHANNON & WILSON, INC.
Geotechnical and Environmental Consultants

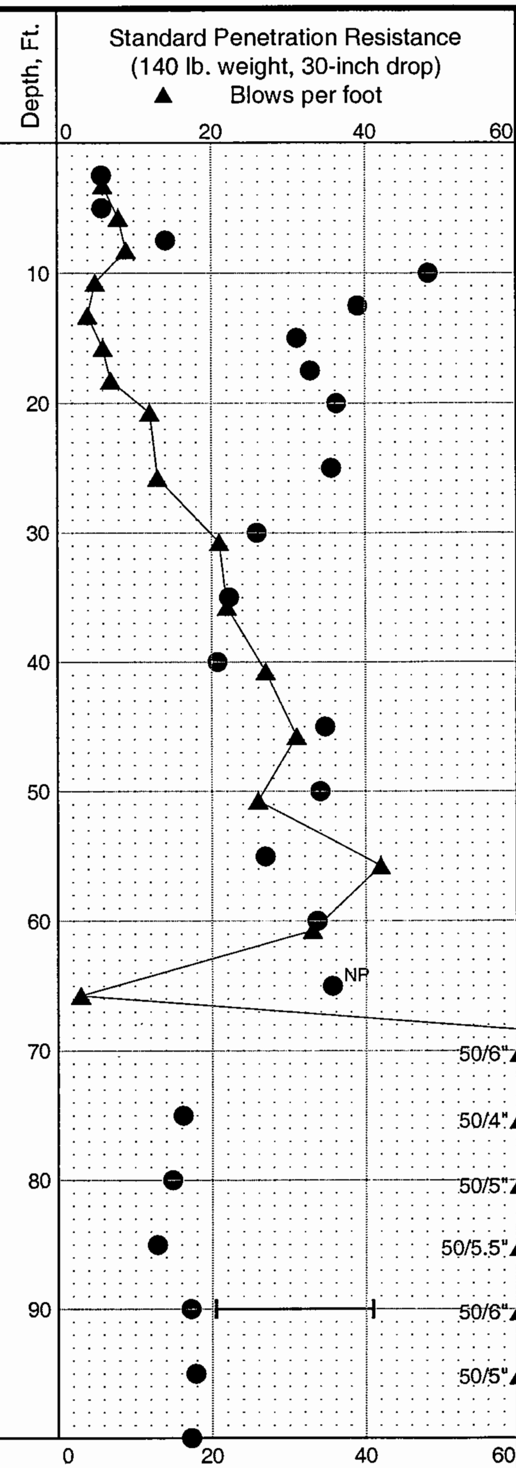
FIG. A-2

MASTER_LOG2 21-09584.GPJ TEMP.GDT 3/30/04 Log: XHL Rev: Typ: EET

SOIL DESCRIPTION Surface Elevation: Approx. 12.0 Ft.	Depth, Ft.
Asphalt.	0.2
Loose, dark brown, trace to slightly gravelly, slightly silty, fine to medium SAND; moist; 3 inches of asphalt at 6.5 feet, medium hydrocarbon odor; (Fill) SP/SP-SM.	8.5
Medium stiff, gray, clayey SILT interbedded with silty fine SAND; moist; scattered fine organics; (Alluvial Deposits) ML/SM.	12.0
Loose, brown-gray, silty fine SAND; wet; occasional fine organics; (Alluvial Deposits) SM.	17.0
Loose to medium dense, dark gray to dark brown, trace to slightly silty, fine to medium SAND; scattered wood debris; (Alluvial Deposits) SP/SP-SM.	42.0
Medium dense to dense, dark gray, slightly silty to silty fine SAND; wet; scattered fine organics; (Alluvial Deposits) SM.	64.0
Soft, dark gray, sandy SILT; wet; scattered shell fragments, trace of gravel; (Estuarine Deposits) ML.	68.0
Dense, gray, silty, sandy GRAVEL; wet; numerous shell fragments; (Estuarine Deposits) GM.	71.0
Very dense, gray, sandy SILT, trace of fine gravel; moist; compacted; (Glaciomarine Drift) ML.	86.0
Hard, gray, slightly sandy to sandy, silty CLAY; moist; occasional gravel, compacted; (Glaciomarine Drift) CL.	

CONTINUED NEXT SHEET

Symbol	PID, ppm	Samples	Ground Water
	0	1	
	0	2	
	0	3	
	0	4	
	0	5	During Drilling ∇
	0	6	
	0	7	
		8	
		9	
		10	
		11	
		12	
		13	
		14	
		15	
		16	
		17	
		18*	
		19	
		20	
		21	
		22	
		23	



LEGEND

- * Sample Not Recovered
- ⌶ Standard Penetration Test
- ∇ Ground Water Level ATD
- ● % Water Content
- Plastic Limit ├─●─┤ Liquid Limit
- Natural Water Content

NOTES

1. The boring was performed using HSA and Rotary Combined drilling methods.
2. The stratification lines represent the approximate boundaries between soil types, and the transition may be gradual.
3. The discussion in the text of this report is necessary for a proper understanding of the nature of the subsurface materials.
4. Groundwater level, if indicated above, is for the date specified and may vary.
5. Refer to KEY for explanation of symbols, codes and definitions.
6. USCS designation is based on visual-manual classification and selected lab testing.

South Park Bridge Project
King County, Washington

LOG OF BORING SB-2

November 2003 21-1-09584-008

SHANNON & WILSON, INC.
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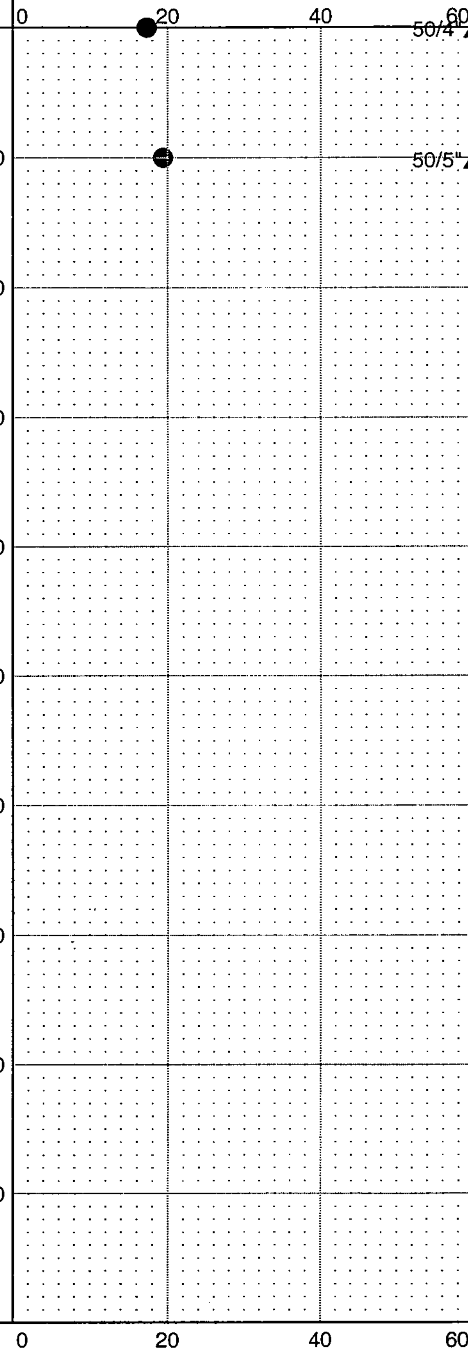
FIG. A-3
Sheet 1 of 2

MASTER_LOG2 21-09584.GPJ TEMP.GDT 3/30/04 Log: XHL Rev: Typ: EET

SOIL DESCRIPTION Surface Elevation: Approx. 12.0 Ft.	Depth, Ft.	Symbol	PID, ppm	Samples	Ground Water	Depth, Ft.
Hard, gray, slightly sandy to sandy, silty CLAY; (Glaciomarine Drift) CL (cont.).				24		
BOTTOM OF BORING COMPLETED 7/8/2003	110.4			25		110

Standard Penetration Resistance
(140 lb. weight, 30-inch drop)
▲ Blows per foot



LEGEND

* Sample Not Recovered

Ground Water Level ATD

Standard Penetration Test

● % Water Content

Plastic Limit ├──●──┤ Liquid Limit
Natural Water Content

NOTES

1. The boring was performed using HSA and Rotary Combined drilling methods.
2. The stratification lines represent the approximate boundaries between soil types, and the transition may be gradual.
3. The discussion in the text of this report is necessary for a proper understanding of the nature of the subsurface materials.
4. Groundwater level, if indicated above, is for the date specified and may vary.
5. Refer to KEY for explanation of symbols, codes and definitions.
6. USCS designation is based on visual-manual classification and selected lab testing.

South Park Bridge Project
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LOG OF BORING SB-2

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FIG. A-3
Sheet 2 of 2

Log: XHL Rev: Typ: EET

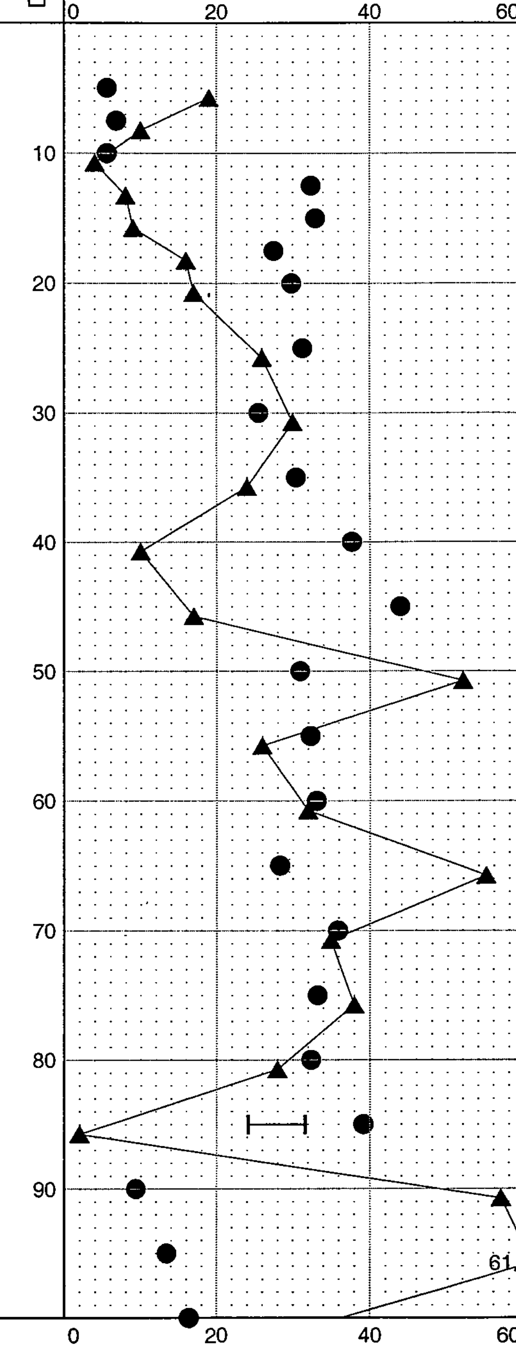
MASTER_LOG2 21-09584.GPJ TEMP.GDT 3/30/04

SOIL DESCRIPTION Surface Elevation: Approx. 16.0 Ft.	Depth, Ft.
Concrete.	0.8
Medium dense, brown, slightly silty fine SAND; moist; scattered gravel and brick debris; (Fill) SP.	6.0
Loose, brown, fine to medium SAND, trace of silt; moist; occasional gravel; (Fill) SP.	12.0
Loose to medium dense, dark brown to dark gray, trace to silty, fine to medium SAND; moist to wet; layers of light brown organic silt, occasional gravel; (Alluvial Deposits) SM/SP.	38.0
Medium dense to very dense, dark gray to brown, silty fine SAND; wet; scattered to abundant organics, numerous wood debris from 45 to 47 feet, massive; (Alluvial Deposits) SM.	80.0
Very stiff, dark gray, fine sandy SILT; wet; (Alluvial Deposits) ML.	85.0
Soft, dark gray, slightly clayey, slightly sandy SILT; wet; scattered shell fragments; (Alluvial Deposits) ML.	90.0
Dense to very dense, gray, silty, gravelly SAND, trace of clay; wet; numerous shell fragments, occasional wood debris;	

Samples	PID, ppm
1	0
2	0
3	0
4	0
5	0
6	0
7	0
8	
9	
10	
11	
12	
13	
14	
15	
16	
17	
18	
19	
20	
21	
22	

Ground Water: During Drilling

Standard Penetration Resistance (140 lb. weight, 30-inch drop) ▲ Blows per foot



CONTINUED NEXT SHEET

LEGEND

- * Sample Not Recovered
- Standard Penetration Test
- Ground Water Level ATD

● % Water Content

Plastic Limit ├──●──┤ Liquid Limit
Natural Water Content

NOTES

1. The boring was performed using HSA and Rotary Combined drilling methods.
2. The stratification lines represent the approximate boundaries between soil types, and the transition may be gradual.
3. The discussion in the text of this report is necessary for a proper understanding of the nature of the subsurface materials.
4. Groundwater level, if indicated above, is for the date specified and may vary.
5. Refer to KEY for explanation of symbols, codes and definitions.
6. USCS designation is based on visual-manual classification and selected lab testing.

South Park Bridge Project
King County, Washington

LOG OF BORING SB-3

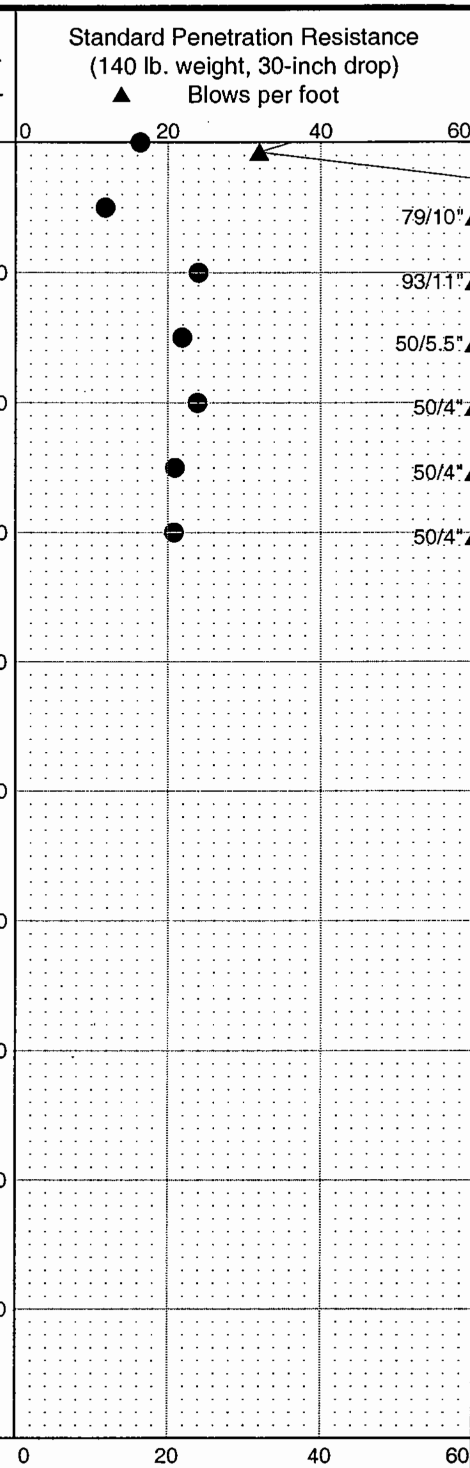
November 2003 21-1-09584-008

SHANNON & WILSON, INC.
Geotechnical and Environmental Consultants

FIG. A-4
Sheet 1 of 2

MASTER_LOG2 21-09584.GPJ TEMP.GDT 3/30/04 Log: XHL Rev: Typ: EET

SOIL DESCRIPTION Surface Elevation: Approx. 16.0 Ft.	Depth, Ft.	Symbol	PID, ppm	Samples	Ground Water	Depth, Ft.
(Estuarine Deposits) SM.				23		
Very dense, brown, sandy, clayey, gravelly SILT; moist; (Glaciomarine Drift) ML.	104.0			24		
Hard, gray, slightly sandy, slightly clayey to clayey SILT; moist; compacted; (Glaciomarine Drift) ML.	109.0			25		110
				26		
				27		120
				28		
BOTTOM OF BORING COMPLETED 7/16/2003	130.7			29		130
						140
						150
						160
						170
						180
						190



LEGEND

* Sample Not Recovered

⌶ Standard Penetration Test

▽ Ground Water Level ATD

● % Water Content

Plastic Limit ├─●─┤ Liquid Limit
Natural Water Content

NOTES

1. The boring was performed using HSA and Rotary Combined drilling methods.
2. The stratification lines represent the approximate boundaries between soil types, and the transition may be gradual.
3. The discussion in the text of this report is necessary for a proper understanding of the nature of the subsurface materials.
4. Groundwater level, if indicated above, is for the date specified and may vary.
5. Refer to KEY for explanation of symbols, codes and definitions.
6. USCS designation is based on visual-manual classification and selected lab testing.

South Park Bridge Project
King County, Washington

LOG OF BORING SB-3

November 2003 21-1-09584-008

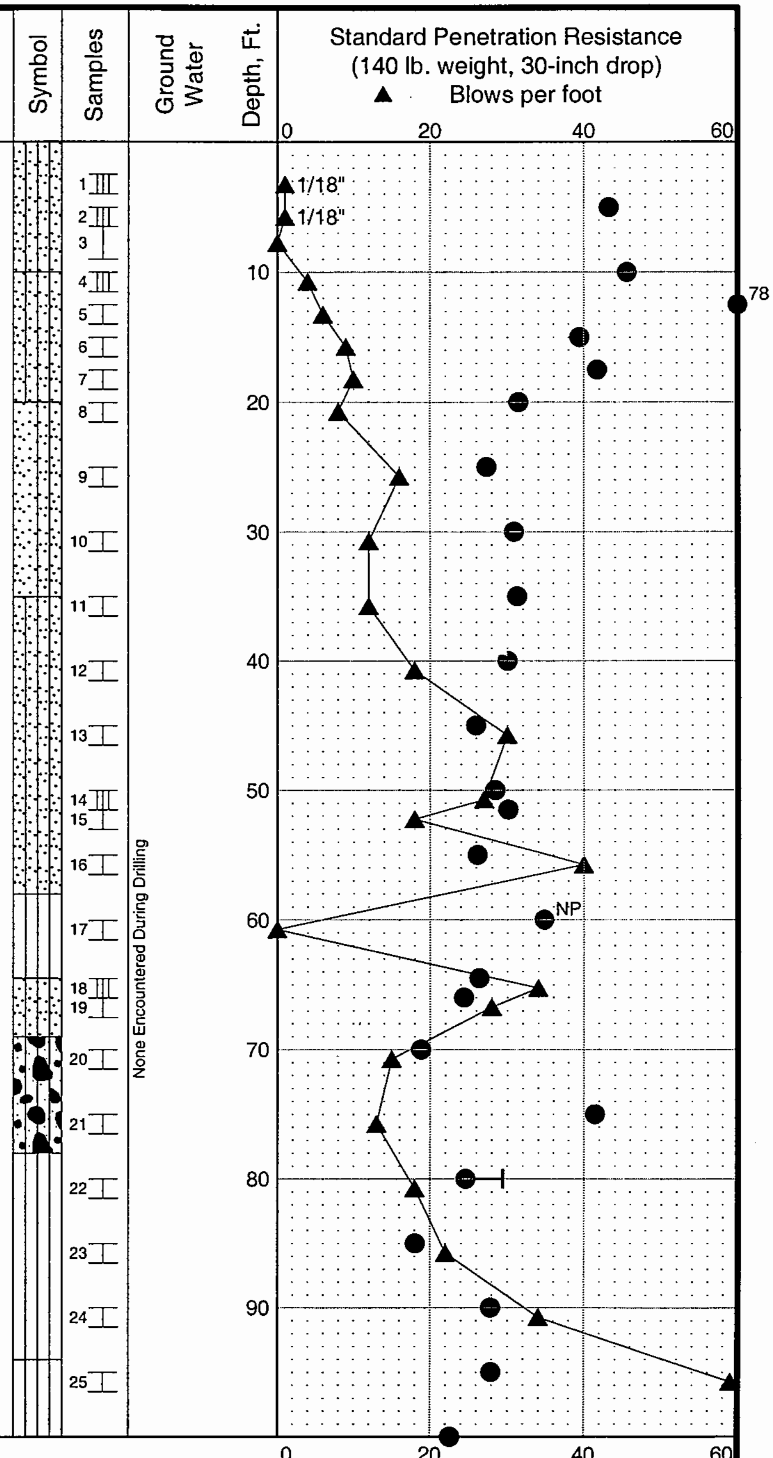
SHANNON & WILSON, INC.
Geotechnical and Environmental Consultants

FIG. A-4
Sheet 2 of 2

MASTER_LOG2 21-09584.GPJ TEMP.GDT 3/30/04 Log: XHL Rev: Typ: EET

SOIL DESCRIPTION Surface Elevation: Approx. 8.5 Ft.	Depth, Ft.
Very loose, black, slightly silty to silty, fine to medium SAND; wet; abundant organic and wood debris; (Fill) SM.	
Loose, dark gray, slightly silty to silty, fine to medium SAND, trace of gravel; wet; scattered to abundant wood debris and organics; (Fill) SM/SP-SM.	10.0
Loose to medium dense, dark gray to brown, trace to slightly silty, fine to medium SAND; wet; trace of organics; (Alluvial Deposits) SP-SM.	20.0
Medium dense, dark brown to dark gray, silty, fine SAND; wet; scattered layers of slightly clayey to clayey silt and fine organics; (Alluvial Deposits) SM.	35.0
Very soft, dark gray, fine sandy SILT, trace of clay; wet; occasional shell fragments; (Estuarine Deposits) ML.	58.0
Medium dense to dense, gray, silty, fine to medium SAND; wet; scattered shell fragments and gravel; (Estuarine Deposits) SM.	64.5
Medium dense, dark brown-gray, slightly clayey, silty gravelly, fine to coarse SAND; wet; scattered shell fragments; (Estuarine Deposits) GM-SM.	69.0
Very stiff to hard, brown-gray, trace to slightly sandy SILT; moist; trace of gravel; (Glaciomarine Drift) ML.	78.0
Hard, gray, trace to slightly sandy, clayey SILT; moist; occasional gravel; compacted;	94.0

CONTINUED NEXT SHEET



LEGEND

* Sample Not Recovered

3" O.D. Split Spoon Sample

Standard Penetration Test

● % Water Content

Plastic Limit ├──●──┤ Liquid Limit

Natural Water Content

NOTES

1. The boring was performed using Mud Rotary drilling methods.
2. The stratification lines represent the approximate boundaries between soil types, and the transition may be gradual.
3. The discussion in the text of this report is necessary for a proper understanding of the nature of the subsurface materials.
4. Groundwater level, if indicated above, is for the date specified and may vary.
5. Refer to KEY for explanation of symbols, codes and definitions.
6. USCS designation is based on visual-manual classification and selected lab testing.

South Park Bridge Project
King County, Washington

LOG OF BORING SB-4

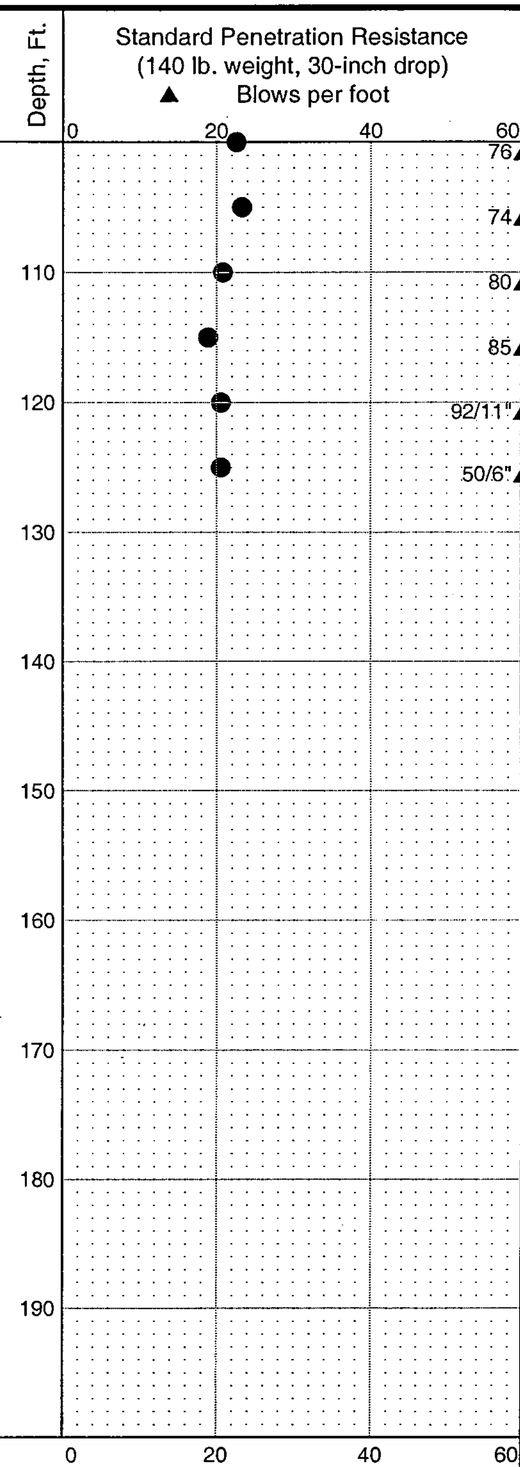
November 2003 21-1-09584-008

SHANNON & WILSON, INC.
Geotechnical and Environmental Consultants

FIG. A-5
Sheet 1 of 2

MASTER_LOG2 21-09584.GPJ TEMP.GDT 3/30/04 Log: XHL Rev: XHL Typ: LKD

SOIL DESCRIPTION Surface Elevation: Approx. 8.5 Ft.	Depth, Ft.	Symbol	Samples	Ground Water
(Glaciomarine Drift) ML.			26 27 28	
Hard, gray, sandy, clayey SILT; moist; scattered gravel; compacted; (Glaciomarine Drift) ML.	115.0		29 30	
BOTTOM OF BORING COMPLETED 7/24/2003	126.0		31	



LEGEND

* Sample Not Recovered
3" O.D. Split Spoon Sample
Standard Penetration Test

NOTES

1. The boring was performed using Mud Rotary drilling methods.
2. The stratification lines represent the approximate boundaries between soil types, and the transition may be gradual.
3. The discussion in the text of this report is necessary for a proper understanding of the nature of the subsurface materials.
4. Groundwater level, if indicated above, is for the date specified and may vary.
5. Refer to KEY for explanation of symbols, codes and definitions.
6. USCS designation is based on visual-manual classification and selected lab testing.

South Park Bridge Project
King County, Washington

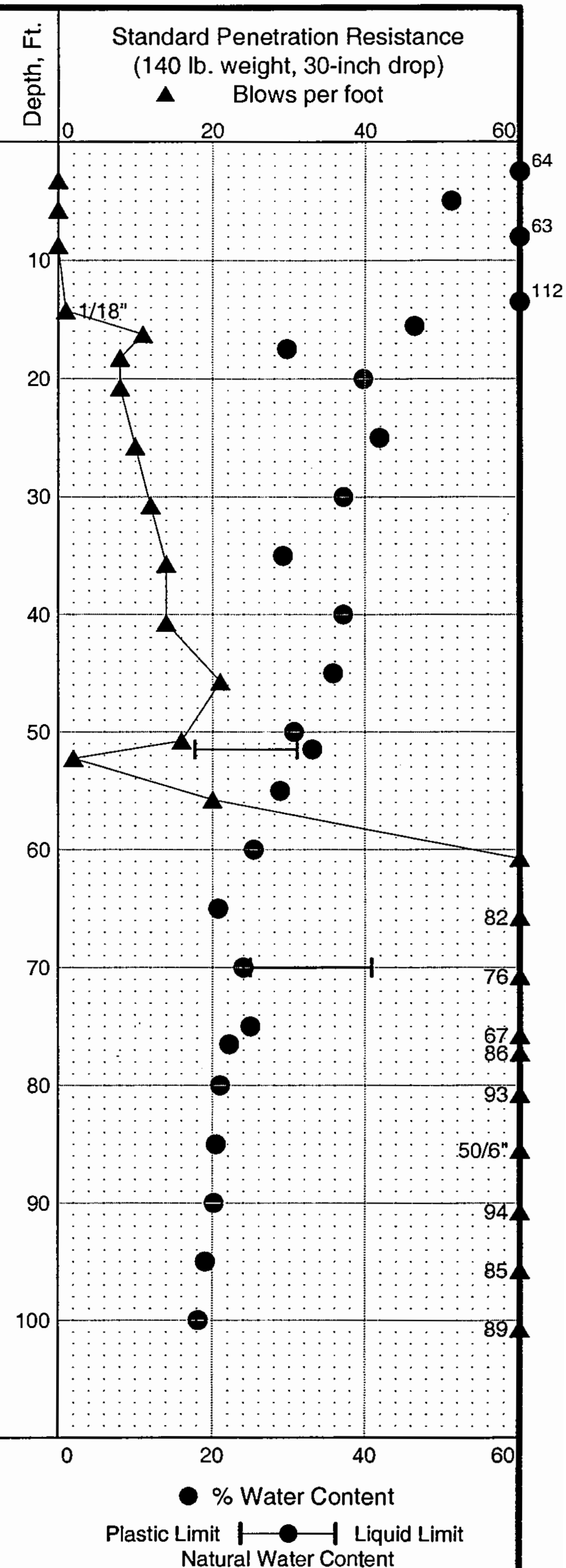
LOG OF BORING SB-4

November 2003 21-1-09584-008

SHANNON & WILSON, INC.
Geotechnical and Environmental Consultants

FIG. A-5
Sheet 2 of 2

SOIL DESCRIPTION Surface Elevation: Approx. 9.0 Ft.	Depth, Ft.	Samples	Ground Water
Very soft, black, sandy, clayey, organic SILT; wet; numerous wood debris at 13 to 16 feet; (Marsh Deposits) OL.		1, 2, 3, 4*, 5	None Encountered During Drilling
Loose to medium dense, dark brown, slightly silty, fine to medium SAND; wet; scattered fine organics; (Alluvial Deposits) SP-SM.	16.0	6, 7	
Medium dense, dark brown to gray, slightly silty to silty, fine SAND; wet; trace of organics, scattered clayey silt seams; (Alluvial Deposits) SP-SM/SM.	25.0	8, 9, 10, 11, 12, 13	
Very soft, gray, slightly sandy to sandy, silty CLAY; wet; scattered organics; some shells at 53 to 55.5 feet; (Estuarine Deposits) CL.	51.5	14	
Medium dense, gray, slightly silty, gravelly SAND; wet; numerous shell fragments; (Estuarine Deposits) SW.	55.5	15	
Hard, gray, trace to slightly sandy, silty CLAY; moist; occasional gravel; compacted; (Glaciomarine Drift) CL.	56.5	16, 17, 18, 19, 20, 21, 22, 23	
Hard, gray,sandy, clayey SILT; moist; occasional gravel; compacted; (Glaciomarine Drift) ML.	92.0	24, 25	
BOTTOM OF BORING COMPLETED 7/23/2003	101.5		



LEGEND

- * Sample Not Recovered
- 3" O.D. Split Spoon Sample
- Standard Penetration Test

NOTES

1. The boring was performed using Mud Rotary drilling methods.
2. The stratification lines represent the approximate boundaries between soil types, and the transition may be gradual.
3. The discussion in the text of this report is necessary for a proper understanding of the nature of the subsurface materials.
4. Groundwater level, if indicated above, is for the date specified and may vary.
5. Refer to KEY for explanation of symbols, codes and definitions.
6. USCS designation is based on visual-manual classification and selected lab testing.

South Park Bridge Project
King County, Washington

LOG OF BORING SB-5

November 2003 21-1-09584-008

SHANNON & WILSON, INC.
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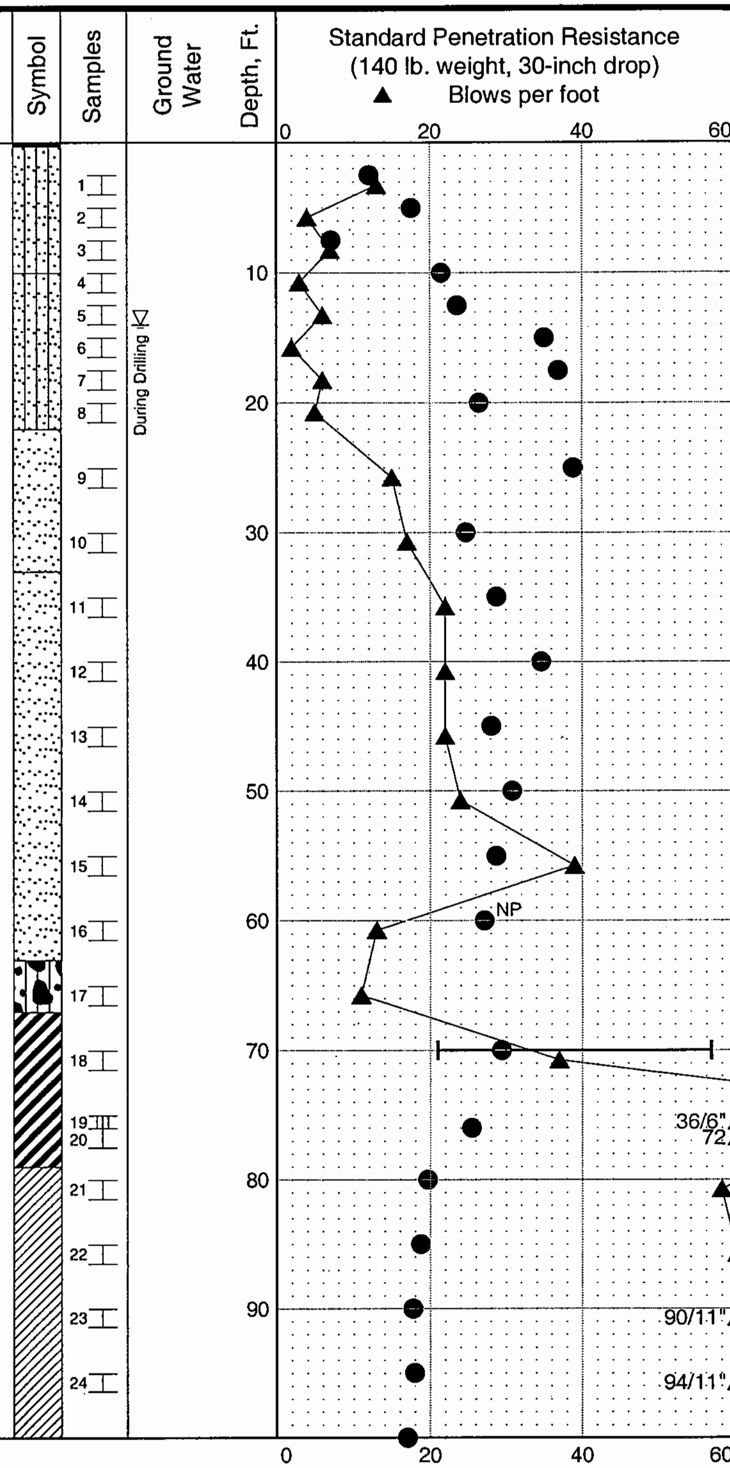
FIG. A-6

Log: XHL Rev: XHL Typ: LKD

MASTER_LOG2 21-09584.GPJ TEMP.GDT 3/30/04

SOIL DESCRIPTION Surface Elevation: Approx. 19.0 Ft.	Depth, Ft.
Asphalt.	0.3
Loose to medium dense, dark brown, trace to gravelly, silty, fine SAND; moist; trace gravel; (Fill) SM.	10.0
Very loose to loose, brown, silty, fine to medium SAND interbedded with fine sandy SILT, trace of gravel; moist to wet; scattered iron oxide stains; (Alluvial Deposit) SM/ML.	22.0
Medium dense, brown to dark gray, trace to slightly silty, fine to medium SAND, trace of gravel; wet; abundant organics and wood debris at 26 to 26.5 feet; (Alluvial Deposits) SP.	33.0
Medium dense to dense, dark gray, slightly silty to silty, fine to medium SAND, trace of fine gravel and clay; wet; occasional fine organics; (Alluvial Deposits) SP/SM.	63.0
Medium dense, gray, silty, sandy GRAVEL; wet; numerous shell fragments; (Estuarine Deposits) GM.	67.0
Hard, gray, silty CLAY; moist; trace of coarse sand and fine gravel; (Glaciomarine Drift) CH.	79.0
Hard, brown-gray, sandy, silty CLAY; moist; scattered coarse sand and gravel; compacted; (Glaciomarine Drift) CL.	

CONTINUED NEXT SHEET



LEGEND

- * Sample Not Recovered
- Standard Penetration Test
- 3" O.D. Split Spoon Sample
- Ground Water Level ATD

NOTES

1. The boring was performed using HSA and Rotary Combined drilling methods.
2. The stratification lines represent the approximate boundaries between soil types, and the transition may be gradual.
3. The discussion in the text of this report is necessary for a proper understanding of the nature of the subsurface materials.
4. Groundwater level, if indicated above, is for the date specified and may vary.
5. Refer to KEY for explanation of symbols, codes and definitions.
6. USCS designation is based on visual-manual classification and selected lab testing.

South Park Bridge Project
King County, Washington

LOG OF BORING SB-6

November 2003 21-1-09584-008

SHANNON & WILSON, INC.
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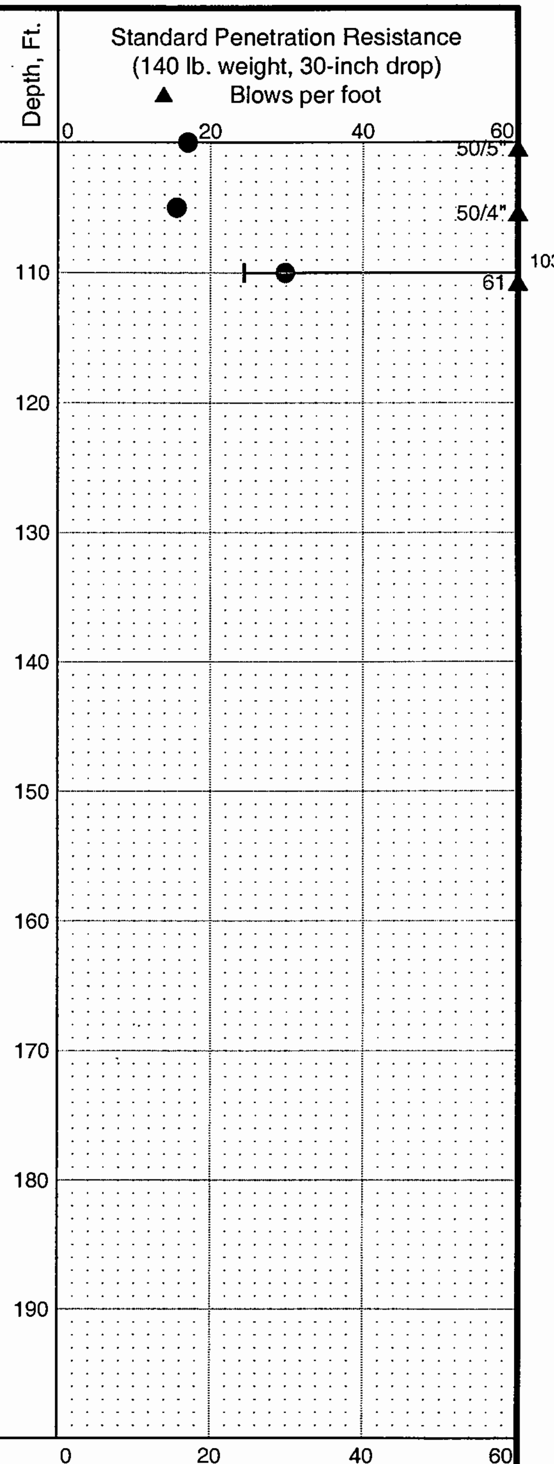
FIG. A-7
Sheet 1 of 2

SOIL DESCRIPTION Surface Elevation: Approx. 19.0 Ft.	Depth, Ft.	Symbol	Samples	Ground Water
			25	
	106.0		26	
Hard, gray, silty CLAY; moist; scattered slickensides; (Glaciomarine Drift) CH.	111.5		27	
BOTTOM OF BORING COMPLETED 6/30/2003				



LEGEND

- * Sample Not Recovered
- Standard Penetration Test
- 3" O.D. Split Spoon Sample
- Ground Water Level ATD

● % Water Content

Plastic Limit ├─●─┤ Liquid Limit
Natural Water Content

NOTES

1. The boring was performed using HSA and Rotary Combined drilling methods.
2. The stratification lines represent the approximate boundaries between soil types, and the transition may be gradual.
3. The discussion in the text of this report is necessary for a proper understanding of the nature of the subsurface materials.
4. Groundwater level, if indicated above, is for the date specified and may vary.
5. Refer to KEY for explanation of symbols, codes and definitions.
6. USCS designation is based on visual-manual classification and selected lab testing.

South Park Bridge Project
King County, Washington

LOG OF BORING SB-6

November 2003 21-1-09584-008

SHANNON & WILSON, INC.
Geotechnical and Environmental Consultants

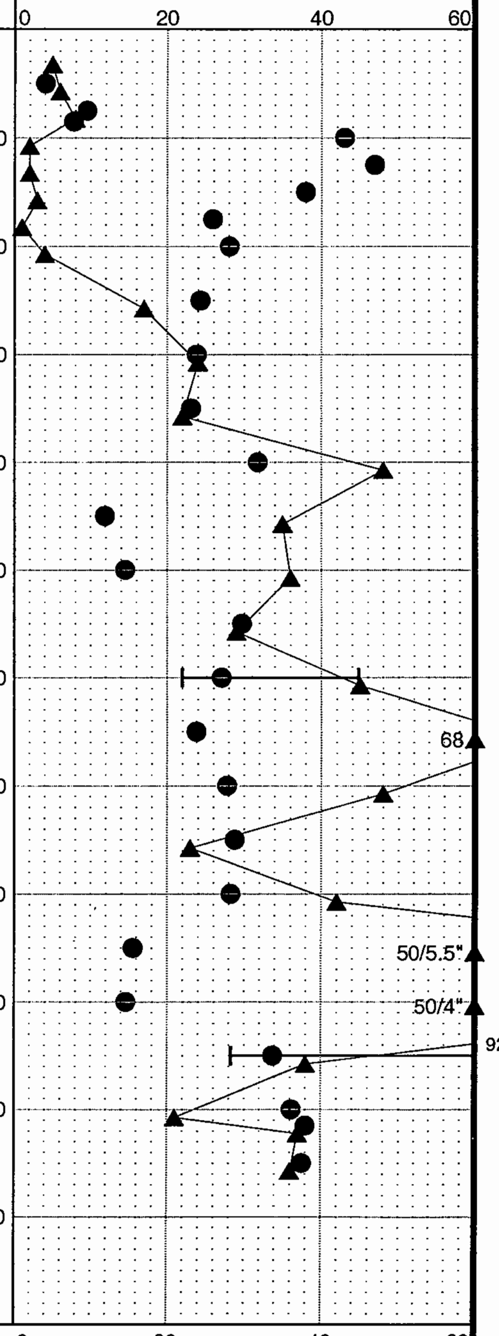
FIG. A-7
Sheet 2 of 2

MASTER_LOG2 21-09584.GPJ TEMP.GDT 3/30/04 Log: XHL Rev: XHL Typ: LKD

SOIL DESCRIPTION Surface Elevation: Approx. 19.0 Ft.	Depth, Ft.
Loose, brown, gravelly, silty SAND; dry to moist; (Fill) SM.	
Very soft, dark brown, clayey SILT; moist to wet; numerous organics; scattered charcoals between 12 to 14 feet; scattered silty fine sand seams; occasional iron-oxide stains; (Alluvial Deposits) ML.	8.5
Loose to medium dense, dark brown SAND; wet; trace of silt; occasional silty clay seams; (Alluvial Deposits) SP/SW.	17.0
Dense, dark gray, silty, fine SAND; wet; occasional fine organics and silt lenses; (Alluvial Deposits) SM.	36.0
Dense, gray, silty, sandy GRAVEL; wet; numerous shell fragments; (Estuarine Deposits) GM.	44.0
Dense, brown, silty, clayey, gravelly SAND; moist; (Estuarine Deposits) SC.	46.0
Hard, gray, silty CLAY; moist; trace of coarse sand and fine gravel; (Glaciomarine Drift) CL.	53.0
Very dense, gray, clayey, sandy SILT; moist; compacted; (Glaciomarine Drift) ML.	84.0
Hard, gray, silty CLAY; moist; locally slickensided; (Glaciomarine Drift) CH.	93.0
BOTTOM OF BORING COMPLETED 6/26/2003	106.5

Symbol | Samples: 1–26 | Ground Water: During Drilling ▽

Standard Penetration Resistance
(140 lb. weight, 30-inch drop)
▲ Blows per foot



LEGEND

- * Sample Not Recovered
- ⌶ Standard Penetration Test
- Ⅲ 3" O.D. Split Spoon Sample
- ▽ Ground Water Level ATD

● % Water Content

Plastic Limit ├──●──┤ Liquid Limit
Natural Water Content

NOTES

1. The boring was performed using HSA and Rotary Combined drilling methods.
2. The stratification lines represent the approximate boundaries between soil types, and the transition may be gradual.
3. The discussion in the text of this report is necessary for a proper understanding of the nature of the subsurface materials.
4. Groundwater level, if indicated above, is for the date specified and may vary.
5. Refer to KEY for explanation of symbols, codes and definitions.
6. USCS designation is based on visual-manual classification and selected lab testing.

South Park Bridge Project
King County, Washington

LOG OF BORING SB-7

November 2003 | 21-1-09584-008

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Geotechnical and Environmental Consultants

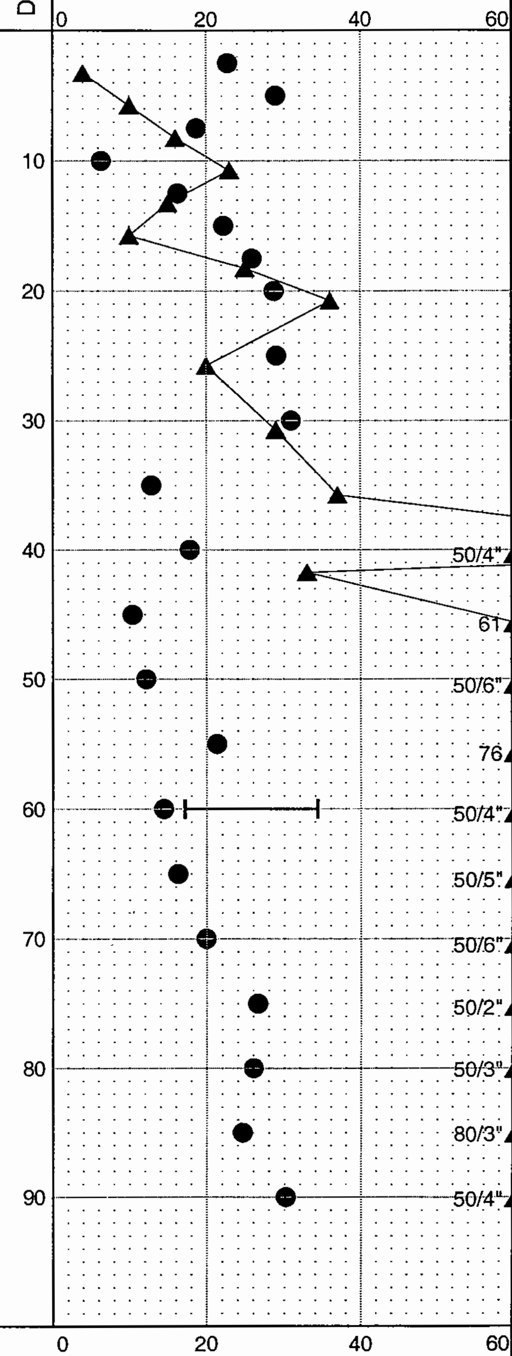
FIG. A-8

Log: XHL Rev: XHL Typ: LKD

MASTER_LOG2 21-09584.GPJ TEMP.GDT 3/30/04

SOIL DESCRIPTION Surface Elevation: Approx. 19.0 Ft.	Depth, Ft.	Samples	Ground Water
Asphalt.	0.1		
Concrete.	0.6	1	
Loose, dark brown, slightly silty, fine SAND; moist; occasional gravel; (Fill) SP.	3.0	2, 3	
Medium stiff to very stiff, light brown to brown, slightly clayey to clayey SILT; moist; interbedded with silty fine sand; (Alluvial Deposits) ML.	10.0	4, 5, 6, 7	During Drilling ⊻
Medium dense to dense, dark brown, trace of silty to slightly silty, fine to medium SAND; moist to wet; (Alluvial Deposits) SP.		8	
Medium dense, dark gray, slightly silty to silty, fine SAND interbedded with slightly clayey, sandy SILT; moist to wet; (Alluvial Deposits) SM-ML.	23.0	9, 10	
Loose, gray, sandy GRAVEL; numerous shell fragments; (Estuarine Deposits) GP.	33.0	11	
Medium dense, brown, gravelly, clayey, silty SAND; moist; (Estuarine Deposits) SC.	36.0	12, 13	
Very dense, brown, gravelly, sandy, clayey SILT; moist; compacted; (Glaciomarine Drift) SM.	40.0	14	
Hard, gray, fine sandy, silty CLAY, trace of coarse sand; moist; compacted; (Glaciomarine Drift) CL.	48.0	15, 16, 17, 18, 19	
Hard, dark brown, sandy SILT; moist; compacted; (Weathered Sandstone); ML.	72.0	20, 21, 22, 23	
BOTTOM OF BORING COMPLETED 6/25/2003	90.3		

Standard Penetration Resistance
(140 lb. weight, 30-inch drop)
▲ Blows per foot



LEGEND

* Sample Not Recovered
⊻ Ground Water Level ATD
Standard Penetration Test
3" O.D. Split Spoon Sample
2.5" O.D. Split Spoon Sample

● % Water Content

Plastic Limit ├─●─┤ Liquid Limit
Natural Water Content

NOTES

1. The boring was performed using HSA and Rotary Combined drilling methods.
2. The stratification lines represent the approximate boundaries between soil types, and the transition may be gradual.
3. The discussion in the text of this report is necessary for a proper understanding of the nature of the subsurface materials.
4. Groundwater level, if indicated above, is for the date specified and may vary.
5. Refer to KEY for explanation of symbols, codes and definitions.
6. USCS designation is based on visual-manual classification and selected lab testing.

South Park Bridge Project
King County, Washington

LOG OF BORING SB-8

November 2003 | 21-1-09584-008

SHANNON & WILSON, INC.
Geotechnical and Environmental Consultants

FIG. A-9

MASTER LOG2 21-09584.GPJ TEMP.GDT 3/30/04 Log: XHL Rev: XHL Typ: LKD

APPENDIX B

PREVIOUS SUBSURFACE EXPLORATIONS

APPENDIX B

PREVIOUS SUBSURFACE EXPLORATIONS

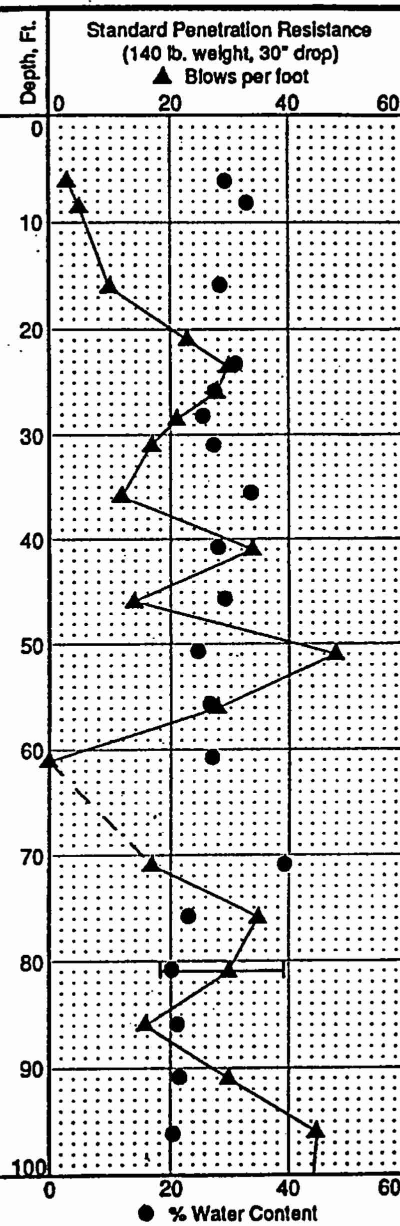
TABLE OF CONTENTS

LIST OF FIGURES

Figure No.

SOIL DESCRIPTION Mud Line Elevation: Approx. -19 Feet (See Fig. 3)	Depth, Ft.	Samples	Ground Water
Very loose to loose, dark gray, slightly silty to silty, fine to medium SAND; wet with some gravel (Organic smell at 7.5')		1, 2	
Medium dense, dark gray, clean to silty, fine to medium SAND; wet with zones of silt	10	3, 4, 5, 6, 7, 8	
Medium dense to dense, dark gray, slightly silty to silty, fine SAND; moist to wet with layers and zones of silt	35	9, 10, 11, 12, 13	
Very soft, gray, slightly clayey, fine to medium sandy SILT; moist with shells and scattered gravel	60	14, 15	
Medium dense, gray, slightly clayey to clayey, silty fine to coarse SAND; moist, with shells and scattered gravel	70	16, 17	
Very Stiff, gray-brown, clayey SILT; moist with scattered coarse sand and a trace of gravel (TILL-LIKE)	80	18, 19	
Hard, gray, clayey SILT; with a trace of coarse sand and gravel (TILL)	90	20, 21	

(Boring log continued)



LEGEND

- 2" O.D. split spoon sample
- 3" O.D. thin-wall sample
- • Sample not recovered
- Impervious seal
- Water level
- Piezometer tip
- P Sample pushed

Atterberg limits:

- Liquid limit
- Natural water content
- Plastic limit

The stratification lines represent the approx. boundaries between soil types, and the transition may be gradual.

16th Avenue South Bridge
Seattle, Washington

LOG OF BORING B-1

February 1991 W-5749-01

SHANNON & WILSON, INC.
Geotechnical Consultants

FIG. A-1a

South Park Bridge
Seattle, Washington

LOG OF BORING B-1

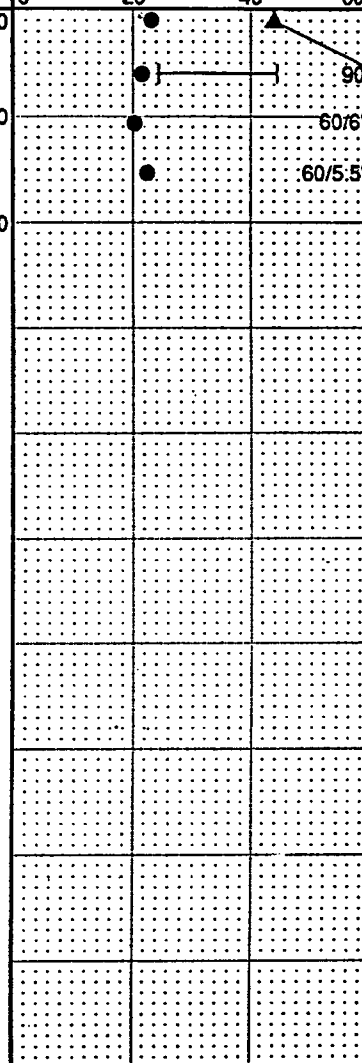
November 2003 21-1-09584-008

SHANNON & WILSON, INC.
Geotechnical and Environmental Consultants

FIG. B-1
Sheet 1 of 2

File: I:\Drafting\21\09584-008\Previous Borings.dwg Date: 11-12-2003 Author: CNT

SOIL DESCRIPTION Mud Line Elevation: Approx. -19 Feet (See Fig. 3)	Depth, Ft.	Samples	Ground Water	Depth, Ft.	Standard Penetration Resistance (140 lb. weight, 30" drop) ▲ Blows per foot 0 20 40 60
Hard, gray, clayey SILT; with a trace of coarse sand and gravel (TILL)	100	22		100	
		23			
		24		110	
	116	25			
BOTTOM OF BORING COMPLETED 1-27-91				120	



LEGEND

- 2" O.D. split spoon sample
- 3" O.D. thin-wall sample
- • Sample not recovered
- Atterberg limits:
  - Liquid limit
  - Natural water content
  - Plastic limit
- Impervious seal
- Water level
- Piezometer tip
- P Sample pushed

The stratification lines represent the approx. boundaries between soil types, and the transition may be gradual.

16th Avenue South Bridge Seattle, Washington	
LOG OF BORING B-1 February 1991 W-5749-01	
SHANNON & WILSON, INC. Geotechnical Consultants	FIG. A-1b

South Park Bridge Seattle, Washington	
LOG OF BORING B-1 November 2003 21-1-09584-008	
SHANNON & WILSON, INC. Geotechnical and Environmental Consultants	**FIG. B-1** Sheet 2 of 2

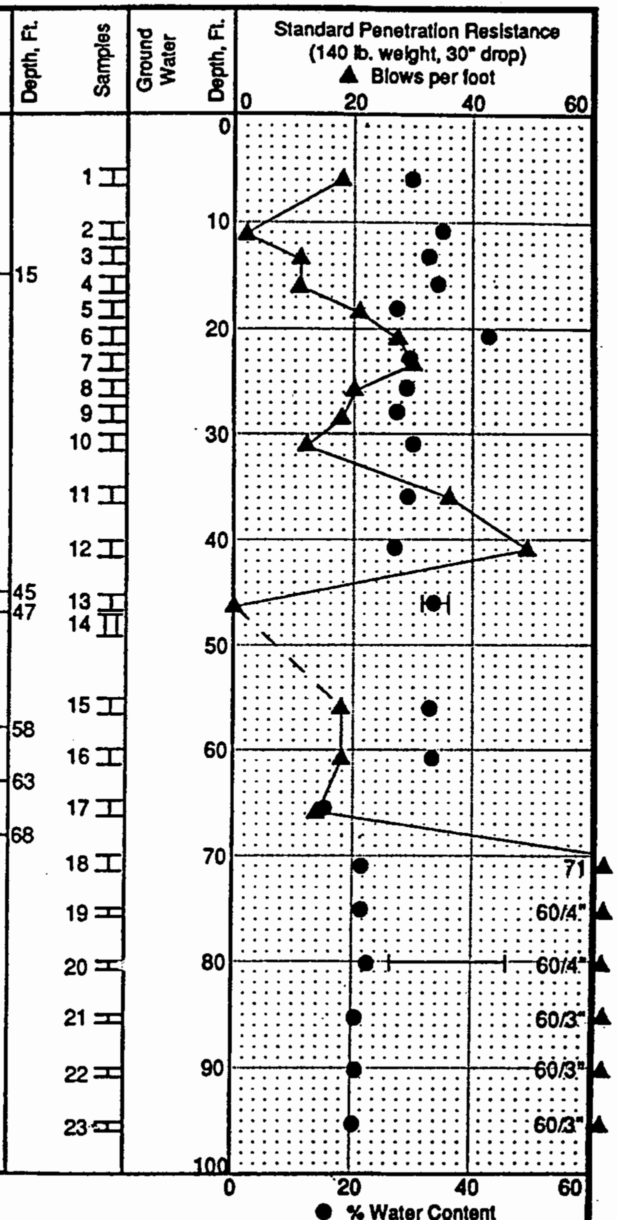
File: I:\Drafting\21\1\09584-008\Previous Borings.dwg Date: 11-12-2003 Author: CNT

SOIL DESCRIPTION

Mud Line Elevation: Approx. -33 Feet (See Fig. 3)

Depth, Ft.	Soil Description
0	Very loose to medium dense, dark gray, clean to silty fine to medium SAND; wet with some shells and organics (organic smell)
15	Medium dense to dense, dark gray, slightly silty to silty, fine SAND; wet with scattered organics and layers and zones of silt
45	Very soft, gray-brown, slightly clayey SILT; moist to wet with a trace of sand and organics
47	Medium dense, gray, slightly clayey, silty fine to coarse SAND; moist to wet with gravel, organics and shells
58	Very stiff, light brown and gray, clayey SILT; moist with laminated layers of: silt, sand and gravel
63	Stiff, gray-brown, gravelly, fine to coarse sandy, clayey SILT; moist (TILL-LIKE)
68	Hard, gray, clayey SILT; dry with a trace of sand (TILL)

(Boring log continued)



LEGEND

- 2" O.D. split spoon sample
- 3" O.D. thin-wall sample
- • Sample not recovered
- Atterberg limits:
  - Liquid limit
  - Natural water content
  - Plastic limit
- Impervious seal
- Water level
- Piezometer tip
- P Sample pushed

The stratification lines represent the approx. boundaries between soil types, and the transition may be gradual.

16th Avenue South Bridge
Seattle, Washington

LOG OF BORING B-2

February 1991 W-5749-01

SHANNON & WILSON, INC.
Geotechnical Consultants

FIG. A-2a

South Park Bridge
Seattle, Washington

LOG OF BORING B-2

November 2003 21-1-09584-008

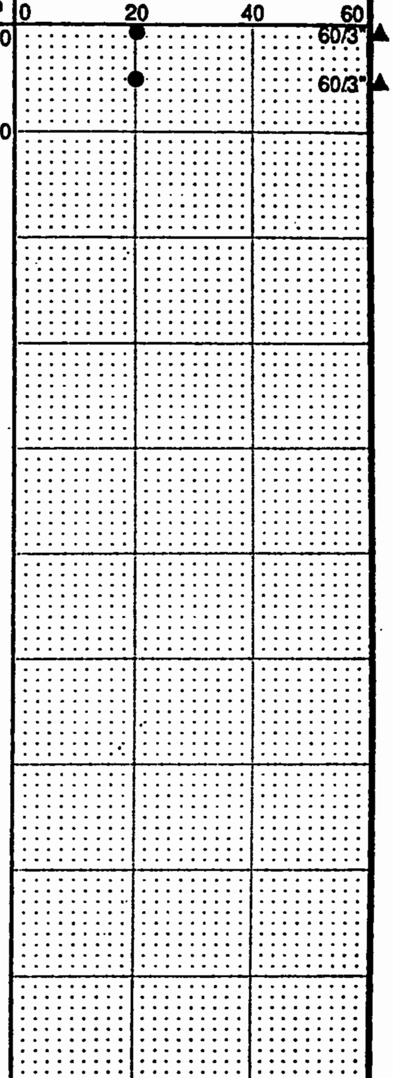
SHANNON & WILSON, INC.
Geotechnical and Environmental Consultants

FIG. B-2
Sheet 1 of 2

File: I:\Drafting\211\09584-008\Previous Borings.dwg Date: 11-12-2003 Author: CNT

SOIL DESCRIPTION Mud Line Elevation: Approx. -33 Feet (See Fig. 3)	Depth, Ft.	Samples	Ground Water	Depth, Ft.
Hard, gray, clayey SILT; dry with a trace of sand (TILL)	100	24		100
	105.7	25		
BOTTOM OF BORING COMPLETED 2-10-91				110

Standard Penetration Resistance
(140 lb. weight, 30" drop)
▲ Blows per foot



LEGEND

- 2" O.D. split spoon sample
- 3" O.D. thin-wall sample
- • Sample not recovered
- Atterberg limits:
  - Liquid limit
  - Natural water content
  - Plastic limit
- Impervious seal
- Water level
- Piezometer tip
- P Sample pushed

The stratification lines represent the approx. boundaries between soil types, and the transition may be gradual.

16th Avenue South Bridge
Seattle, Washington

LOG OF BORING B-2

February 1991 W-5749-01

SHANNON & WILSON, INC.
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FIG. A-2b

South Park Bridge
Seattle, Washington

LOG OF BORING B-2

November 2003 21-1-09584-008

SHANNON & WILSON, INC.
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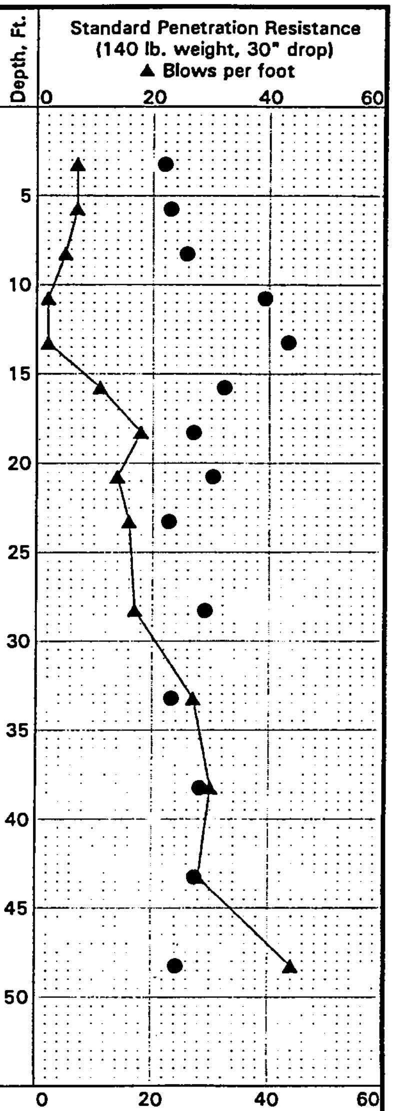
FIG. B-2
Sheet 2 of 2

File: I:\Drafting\211\09584-008\Previous Borings.dwg Date: 11-12-2003 Author: CNT

MASTERLG 8/25/94

MATERIAL DESCRIPTION Surface Elevation: Approx. 7 Feet	Depth, Ft.	Samples
Loose, brown, silty, sandy GRAVEL; moist; (Fill) GM.	1.5	
Loose, brown, slightly silty to silty, fine SAND; moist to wet below 7 feet; (Alluvium) SM.	9.5	1, 2, 3
Very soft, gray-brown SILT, trace of clay and trace of fine sand; wet; (Alluvium) ML.	12.0	4
Very loose to medium dense, gray-brown, silty, fine SAND; wet; scattered iron-stained zones; (Alluvium) SM.	17.5	5, 6
Medium dense, dark gray, clean to slightly silty, fine to medium SAND; wet; (Alluvium) SP.	36.0	7, 8, 9, 10, 11
Medium dense to dense, dark gray, slightly silty, fine SAND; wet; (Alluvium) SP-SM.	49.0	12, 13, 14
BOTTOM OF BORING COMPLETED 7/8/94		



LEGEND

- • Sample Not Recovered
- Ⅰ 2" O.D. Split Spoon Sample
- Ⅱ 3" O.D. Shelby Tube Sample
- Surface Seal
- Annular Sealant
- Piezometer Screen
- Grout
- ▽ Water Level

NOTES

1. The stratification lines represent the approximate boundaries between soil types, and the transition may be gradual.
2. The discussion in the text of this report is necessary for a proper understanding of the nature of subsurface materials.
3. Water level, if indicated above, is for the date specified and may vary.
4. Refer to KEY for explanation of "Symbols" and definitions.
5. USC letter symbol based on visual classification.

16th Avenue South Bridge
Seattle, Washington

LOG OF BORING B-3

July 1994 W-5749-02

SHANNON & WILSON, INC.
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FIG. A-2

South Park Bridge
Seattle, Washington

LOG OF BORING B-3

November 2003 21-1-09584-008

SHANNON & WILSON, INC.
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FIG. B-3

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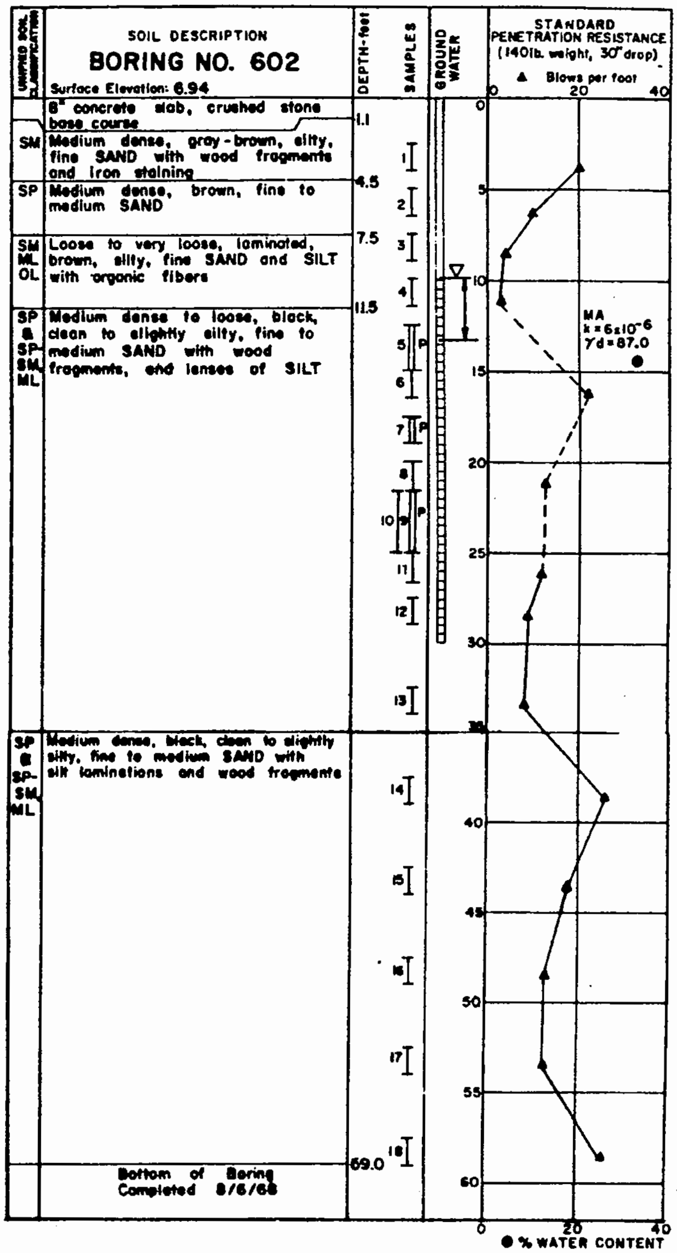

LEGEND

- 2" O.D. split spoon sample
- 3" O.D. thin-wall sample
- P Sampler pushed
- * Sample not recovered
- OBSERVATION WELL
  - Plastic casing
  - R Water level recorder installed
  - Ground water level
  - Fluctuation of ground water
  - Plastic casing with slots
- Impervious seal
- Piezometer tip
- k Coefficient of permeability (cm/sec)
- γd Dry unit weight (pcf)
- MA Grain size
- ● Water content (%)

BOEING PLANT II
PILE INVESTIGATION

BORING LOGS

DECEMBER 15, 1968 W-1547

SHANNON & WILSON
SOIL MECHANICS & FOUNDATION ENGINEERS

South Park Bridge
Seattle, Washington

BOEING PLANT II PILE INVESTIGATION BORING LOGS

November 2003 21-1-09584-008

SHANNON & WILSON, INC.
Geotechnical and Environmental Consultants

FIG. B-4
Sheet 1 of 2

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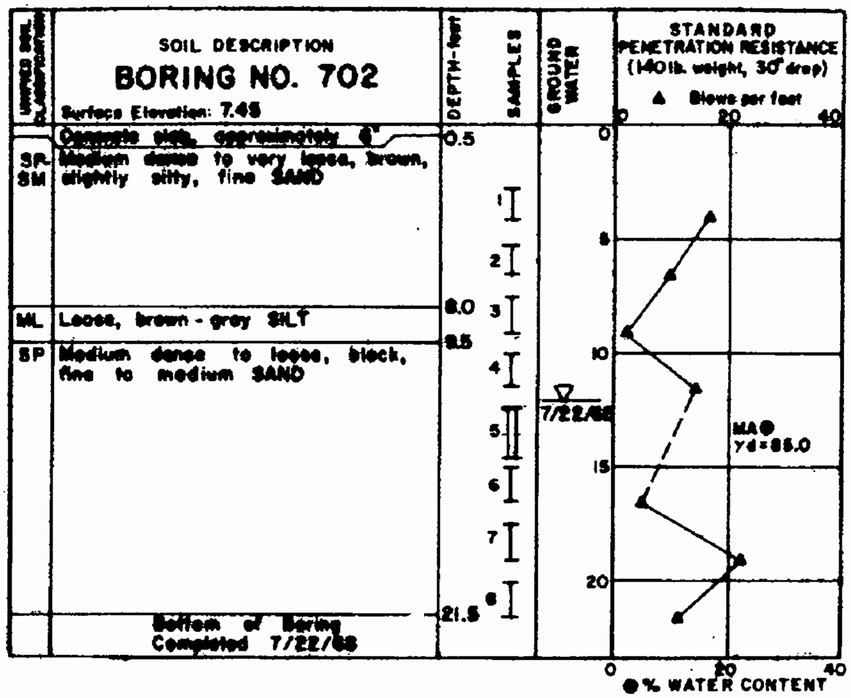

South Park Bridge
Seattle, Washington

BOEING PLANT II PILE INVESTIGATION BORING LOGS

November 2003 | 21-1-09584-008

SHANNON & WILSON, INC.
Geotechnical and Environmental Consultants

FIG. B-4
Sheet 2 of 2

File: I:\Drafting\211\09584-008\Previous Borings.dwg Date: 11-12-2003 Author: CNT

BORING

Surface Elevation: 18.9±

Remarks	Moisture Content (%)	Dry Density (PCF)	Blows per Foot	Sample Type
			89/9"	■
Resistivity =23500ohm-cm			50/5"	□
			22	■
			7	■
48.7% Fines	30.9		11	■
			27	□
	24.3	98	31	■

Symbol	Description
	Concrete slab (8 inches)
GW	Orange sandy gravel (moist) (basecourse)
SP	Gray fine to medium sand, trace silt (moist) (fill) (blow counts unrepresentative due to underlying asphalt)
	Asphaltic concrete layer
GW	Brown sandy gravel (moist) (basecourse) (very dense)
SP	Black fine to medium sand, trace silt (moist) (medium dense) (fill)
ML	Gray mottled silt (moist) (medium stiff)
SP	Black fine to medium sand, trace silt (wet)
SM	Gray silty fine sand (loose)
SP	Black fine to medium sand, trace silt (loose)
	grades to medium dense

Depth (ft): 0, 5, 10, 15, 20, 25, 30, 35

Boring B-3-92 completed at depth of 21.5 feet on 6-24-92.

Groundwater depth measurements:
11.4 feet at 6:30 PM on 6-24-92
11.5 feet at 6:22 PM on 7-28-92

NOTE:
Piezometer installed to 19.2 feet.

LOG OF BORING B-3-92 Dames & Moore

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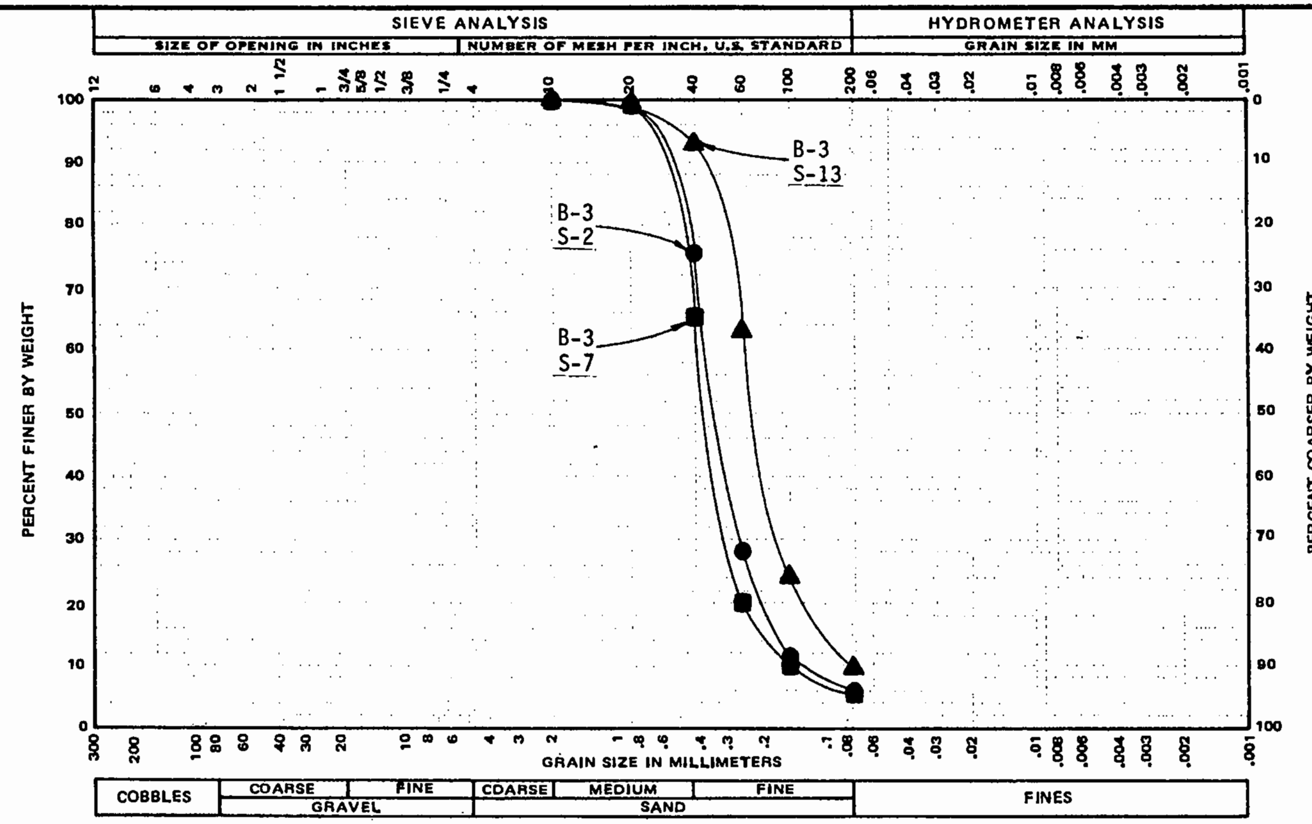
South Park Bridge Seattle, Washington	
LOG OF BORING B-3-92	
November 2003	21-1-09584-008
SHANNON & WILSON, INC. Geotechnical and Environmental Consultants	**FIG. B-5**

File: I:\Drafting\211\09584-008\Previous Borings.dwg Date: 11-12-2003 Author: CNT



SAMPLE NO.	DEPTH-FT.	U.S.C.	CLASSIFICATION	NAT. W.C. %	LL	PL	PI
B-3 S-2	5.0-6.5	SP-SM	Brown, slightly silty SAND.	23.0			
B-3 S-7	17.5-19.0	SP-SM	Brown, slightly silty SAND.	26.9			
B-3 S-13	42.5-44.0	SP-SM	Dark gray, slightly silty SAND.	27.4			

16th Avenue South Bridge
Seattle, Washington

GRAIN SIZE DISTRIBUTION

July 1994 W-5749-02

SHANNON & WILSON, INC.
Geotechnical Consultants

FIG. A-3

South Park Bridge
Seattle, Washington

GRAIN SIZE DISTRIBUTION FOR BORING B-3

November 2003 21-1-09584-008

SHANNON & WILSON, INC.
Geotechnical and Environmental Consultants

FIG. B-6

APPENDIX C

DOWNHOLE SEISMIC TEST

APPENDIX C

DOWNHOLE SEISMIC TEST

REPORT

Report

"Compression and Shear Wave Velocity Measurements, South Park Bridge Project", Submitted by Geo-Recon International, August 29, 2003.

GEO RECON INTERNATIONAL

applied geophysics

August 29, 2003
J03-745

Shannon & Wilson Inc
400 N 34th Street
Seattle, Washington 98103

Compression and Shear Wave Velocity Measurements
South Park Bridge Project, King County, Washington

This report presents the results of the geophysical measurements in Borings SB-2 and SB-6, South Park Bridge Project, King County, Washington. Downhole Compressional and Shear wave velocities for soil dynamic moduli determinations were measured in the borings. The fieldwork was completed on August 22 and 25, 2003.

COMPRESSIONAL AND SHEAR WAVE VELOCITIES

The borings were cased with 3-inch Schedule 40 PVC pipe. The 3-inch casings were grouted in the borehole annulus.

The measured compressional and shear wave velocities are presented in the tables attached to this report, which show the averaged velocities calculated from the interval velocities for the boring, the calculated interval velocities, the interval times, converted downhole time arrivals, the measured time arrivals and depths down the bore hole. When the velocity boundary does not coincide with a measurement depth, the velocity calculation of that point is not accurate from the preceding point of measurement, and the velocity computation between those two points is not included in the velocity average.

Figures 1 and 2 are the time-depth plots for the borings. The plots are the corrected down hole time arrivals of the measured Compressional (P) and Shear (S) wave particle motion, plotted against the depth of measurement. The velocities of the P and S waves are computed from the slopes of the time arrivals on the figures, or as the averaged velocities of the interval velocities. The figures were utilized to determine the depths of the velocity changes in the attached tables and summary presented below.

The summary of the measured P and S wave velocities in the borings are as follows:

Boring SB-2

Depth of Data (feet)	P-wave Velocity (feet/second)	S-wave Velocity (feet/second)	Poisson's Ratio
0 to 8.5	Not Determined	861	
8.5 to 22	5008	320	0.4980
22 to 45	6042	508	0.4964
45 to 70	6356	572	0.4959
70 to 110	7134	1600	0.4735

Boring SB-6

Depth of Data (feet)	P-wave Velocity (feet/second)	S-wave Velocity (feet/second)	Poisson's Ratio
0 to 7	Not Determined	715	
7 to 25	4999	439	0.4961
25 to 60	6629	554	0.4965
60 to 70	6227	417	0.4977
70 to 110	7088	1511	0.4762

Poisson's Ratio is calculated as follows:

$$\mu = \frac{V_p^2 - 2V_s^2}{2(V_p^2 - V_s^2)}$$

Where: μ = Poisson's Ratio
Vp = Compressional Wave Velocity
Vs = Shear Wave Velocity

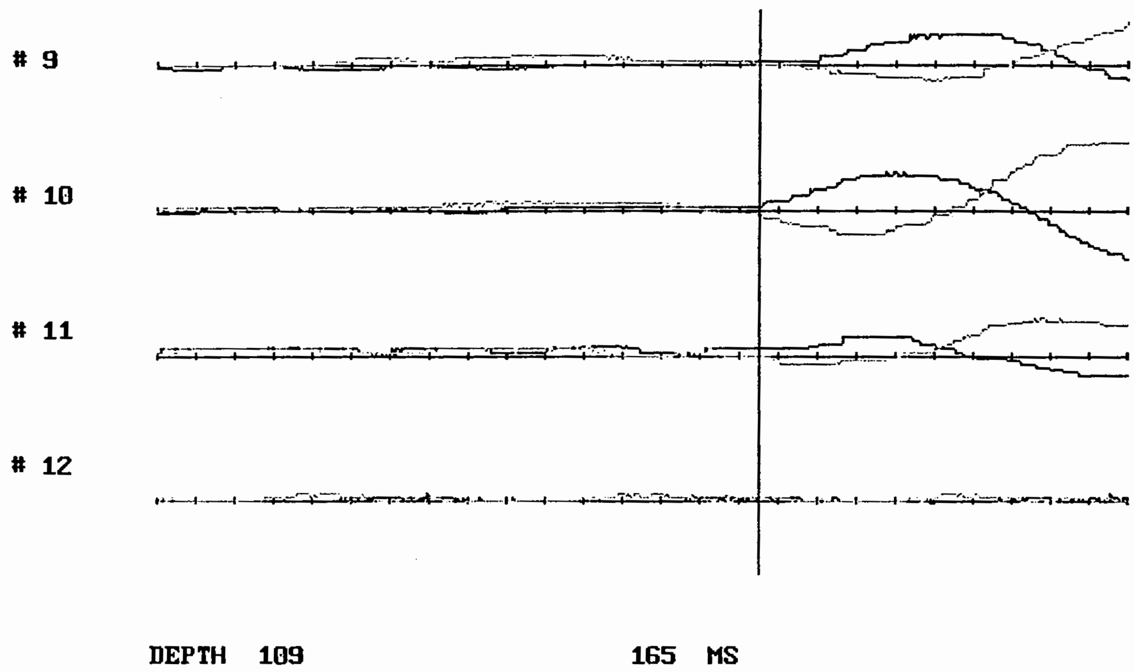
The Compression (P) wave energy was a vertical blow to a metal plate placed on the ground surface, offset from the casing. The zero time of the hammer blow was determined from an impact switch taped to the hammer. Multiple hammer blows were stacked to enhance the energy arrivals.

The Shear (S) Wave energy source was a 6 by 6-inch plank offset from the casing. The front wheels of a vehicle were placed on the top of the plank to provide coupling with the ground. The long direction of the plank was placed tangent to a circle with the radius center at the borehole. An impact switch taped to the handle of the hammer determined zero time.

Two detectors, spaced at a 10-foot interval in the borehole, were used to detect the generated the P and S wave energy. To minimize the effect of the detector spiral as they are lowered down the borehole; each detector package contains four sets of horizontal geophones (8 Hz geophones) placed on axes of 45 degrees. The axis of sensitivity of the geophones is 20 degrees. Utilizing the two detector packages, at least two separate measurements were collected at each data point. The first and final data points, however, are single measurements.

For the S wave data, two recordings were made at each data point. The two separate recordings were made with reversed (polarized) energy inputs utilizing the opposite ends of the plank (blows right and left). The time arrival of the shear wave energy was determined by comparing arrival times and direction of particle motion of the recorded wave motion in the two data sets.

The particle motion of the shear wave energy is polarized and is dependent on the direction of the energy input. On Blow 1, the particle motion is reversed from that produced by Blow 2. The polarization of the energy helps the interpreter to separate S wave arrivals from other energy arrivals. Reversed particle motion, however, can also occur in other ways such as out-of-phase noise, shear energy generated in the boring annulus and casing as tube waves and P to S conversions.



A sample of the Shear wave data made in Boring SB-6 at a depth of 109 ft is shown above. The vertical cursor is at the point picked as the shear wave arrival time, with the arrival time displayed below the cursor. There are 500 samples across the image, with sample number 240 (24 ms milli-Seconds plus a 110 milli-Second time delay) on the left side of the record. The shear wave arrival is shown on Trace 10 at an arrival time 165 milli-Seconds.

The picked arrival times were converted from the "slant distance" travel path to the vertical travel path down the borehole. The "slant distance" travel path is a result of the source to borehole offset. The formula used for the conversion to the 'Corrected Time' vertically down the borehole is:

DH Time = Record Time x [Cos(Arctan (offset/detector depth))]

Borehole drift was not measured in the boring, and no corrections have been applied for possible drift. The velocity changes generally correspond to the logged material changes, so that extreme drift of the borings off of vertical is not expected.

The recording equipment was an EG&G 1225, a 12-channel signal enhancement digital-recording seismograph. The P wave was measured using a 25-millisecond record length and the S wave was measured with 100 millisecond record lengths, with various amounts of delay times to maintain the arrivals within the record length. The sampling rate was 1000 samples per record length; the samples are an 8 bit word. For the P wave records the data was picked with a sampling resolution of 0.025 milliseconds. For the S wave records the data was picked to 0.1 milliseconds. The data was field recorded on a laptop computer in SEG-1 format. Arrival times were picked from a computer screen image of the records.

The information presented in this report is based upon geophysical measurements made by generally accepted methods and field procedures, and our interpretation of these data. The presented information is based upon our best estimate of subsurface conditions considering the geophysical results and all other information available to us. These results are interpretive in nature and are considered to be a reasonably accurate presentation of the existing conditions within the limitations of the method or methods employed.

For Geo-Recon International:

John M Musser Jr

John M. Musser Jr.
Principal Geophysicist

Downhole Compressional and Shear Wave Velocity Measurements

Borehole: SB-2 - South Park Bridge Project
King County, Washington

Shear Wave Data - Interval Velocity Computations

Depth of Data	Recorded Time	Corrected Time	Interval Time	Interval Velocity		Average Velocity
5.0	14.45	5.804	5.804	861		861
	Velocity Change at ~ 8.5 feet					
10.0	20.75	13.683	7.879	635	n/a	
15.0	36.95	29.418	15.735	318		320
20.0	51.70	44.916	15.498	323		
	Velocity Change at ~ 22 feet					
25.0	62.00	56.412	11.496	435	n/a	
30.0	70.90	66.276	9.864	507		
35.0	79.60	75.686	9.410	531		508
40.0	89.10	85.688	10.002	500		
45.0	98.80	95.774	10.087	496		
	Velocity Change at ~ 45 feet					
50.0	107.80	105.103	9.328	536		
55.0	115.40	112.998	7.895	633		
60.0	125.00	122.803	9.805	510		572
65.0	133.00	131.000	8.197	610		
70.0	141.60	139.759	8.758	571		
	Velocity Change at ~ 70 feet					
75.0	145.00	143.353	3.595	1391		
80.0	148.40	146.916	3.562	1404		

Shear Wave Velocity Data - SB-2 Continued

Depth of Data	Recorded Time	Corrected Time	Interval Time	Interval Velocity	Average Velocity
85.0	151.70	150.354	3.438	1454	
90.0	154.80	153.573	3.219	1553	1600
95.0	157.80	156.676	3.103	1611	
100.0	160.50	159.467	2.791	1791	
105.0	163.30	162.346	2.879	1737	
109.0	165.40	164.503	2.157	1855	

Bottom of Casing at 110 feet.

Source to Borehole offset: 11.4 feet.
Casing stickup above ground: 0 feet.
n/a - Not included in Velocity Average.

Velocities in feet per second.
Depths in feet - Times in milli-seconds.
Velocity breaks from Time-Depth Plot.

Downhole Compressional and Shear Wave Velocity Measurements

Borehole: SB-2 - South Park Bridge Project
King County, Washington

Compressional Wave Data - Interval Velocity Computations

Depth of Data	Recorded Time	Corrected Time	Interval Time	Interval Velocity		Average Velocity
5.0	5.175	3.659	3.659	1366	n/a	??
	Velocity Change at ~ 8.5 feet					
10.0	7.475	6.686	3.027	1652	n/a	
15.0	8.100	7.684	0.998	5008		
20.0	8.950	8.683	0.998	5008		5008
	Velocity Change at ~ 22 feet					
25.0	9.800	9.610	0.927	5394	n/a	
30.0	10.600	10.456	0.846	5910		
35.0	11.400	11.285	0.830	6027		6042
40.0	12.200	12.106	0.820	6095		
45.0	13.000	12.920	0.815	6137		
	Velocity Change at ~ 45 feet					
50.0	13.750	13.682	0.761	6568		
55.0	14.500	14.440	0.759	6590		
60.0	15.300	15.247	0.807	6198		6356
65.0	16.100	16.053	0.805	6208		
70.0	16.900	16.857	0.804	6215		
	Velocity Change at ~ 70 feet					
75.0	17.600	17.561	0.704	7103		
80.0	18.300	18.264	0.703	7109		

Compressional Wave Velocity Data - SB-2 Continued

Depth of Data	Recorded Time	Corrected Time	Interval Time	Interval Velocity	Average Velocity
85.0	19.000	18.967	0.703	7114	
90.0	19.700	19.670	0.702	7118	7134
95.0	20.400	20.372	0.702	7121	
100.0	21.100	21.074	0.702	7124	
105.0	21.800	21.775	0.702	7126	
109.0	22.350	22.327	0.551	7257	

Bottom of Casing at 110 feet.

Source to Borehole offset: 5 feet.
Casing stickup above ground: 0 feet.
n/a - Not included in Velocity Average.

Velocities in feet per second.
Depths in feet - Times in milli-seconds.
Velocity Breaks from Time-Depth Plot.

Downhole Compressional and Shear Wave Velocity Measurements

Borehole: SB-6 - South Park Bridge Project
King County, Washington

Shear Wave Data - Interval Velocity Computations

Depth of Data	Recorded Time	Corrected Time	Interval Time	Interval Velocity		Average Velocity
5.0	11.70	6.998	6.998	715		715
	Velocity Change at ~ 7 feet					
10.0	20.00	16.615	9.618	520	n/a	
15.0	29.80	27.209	10.594	472		439
20.0	42.00	39.825	12.616	396		
25.0	52.75	50.952	11.127	449		
	Velocity Change at ~ 25 feet					
30.0	61.60	60.119	9.167	545		
35.0	70.90	69.636	9.517	525		554
40.0	79.40	78.309	8.673	576		
45.0	89.00	88.030	9.721	514		
50.0	97.40	96.537	8.508	588		
55.0	107.00	106.215	9.678	517		
60.0	115.10	114.389	8.174	612		
	Velocity Change at ~ 60 feet					
65.0	128.50	127.823	13.434	372		417
70.0	139.30	138.666	10.844	461		
	Velocity Change at ~ 70 feet					
75.0	143.00	142.433	3.767	1327		
80.0	146.70	146.188	3.755	1331		1511

Shear Wave Velocity Data SB-6 Continued

Depth of Data	Recorded Time	Corrected Time	Interval Time	Interval Velocity	Average Velocity
85.0	150.20	149.736	3.547	1410	
90.0	153.30	152.877	3.141	1592	
95.0	156.40	156.012	3.136	1595	1511
100.0	159.50	159.143	3.131	1597	
105.0	162.50	162.170	3.027	1652	
109.0	165.00	164.689	2.519	1588	

Bottom of Casing at 110 feet.

Source to Borehole offset: 6.7 feet.
Casing stickup above ground: 0 feet.
n/a - Not included in Velocity Average.

Velocities in feet per second.
Depths in feet - Times in milli-seconds.
Velocity breaks from Time-Depth Plot.

Downhole Compressional and Shear Wave Velocity Measurements

Borehole: SB-6 - South Park Bridge Project
King County, Washington

Compressional Wave Data - Interval Velocity Computations

Depth of Data	Recorded Time	Corrected Time	Interval Time	Interval Velocity		Average Velocity
5.0	6.150	4.526	4.526	?	n/a	
	Velocity Change at ~ 7 feet					
10.0	6.325	5.746	1.220	4098		
15.0	7.125	6.812	1.066	4692		4999
20.0	7.975	7.772	0.960	5207		
25.0	8.900	8.753	0.981	5097		
	Velocity Change at ~ 25 feet					
30.0	9.600	9.489	0.736	6793	n/a	
35.0	10.325	10.237	0.748	6686		
40.0	11.075	11.002	0.766	6532		6629
45.0	11.825	11.764	0.761	6568		
50.0	12.575	12.522	0.758	6593		
55.0	13.325	13.279	0.757	6609		
60.0	14.075	14.034	0.755	6621		
	Velocity Change at ~ 60 feet					
65.0	14.900	14.863	0.829	6031		6227
70.0	15.675	15.641	0.778	6423		
	Velocity Change at ~ 70 feet					
75.0	16.400	16.369	0.728	6868		
80.0	17.100	17.072	0.703	7117		

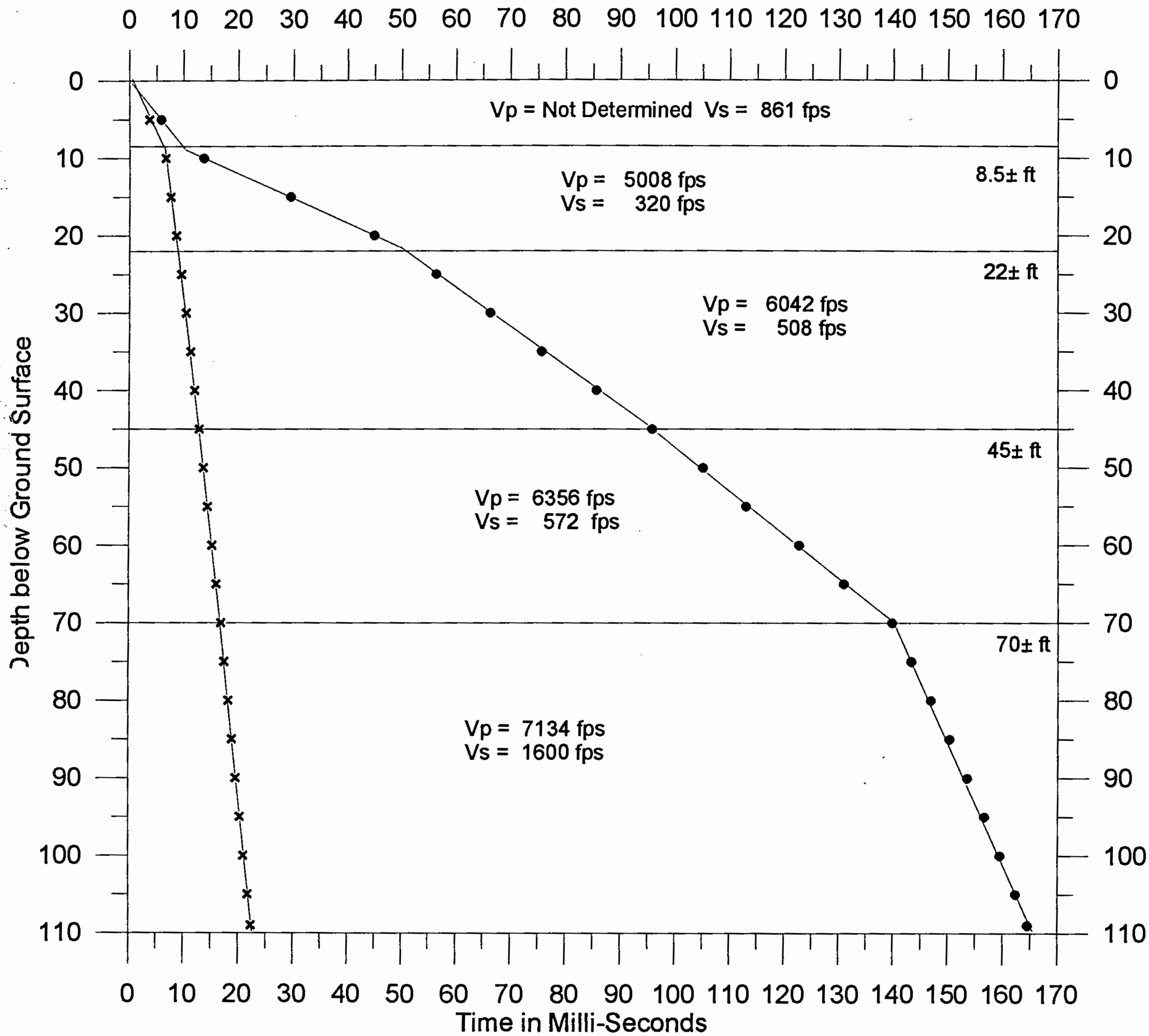
Compressional Wave Data SB-6 Continued

Depth of Data	Recorded Time	Corrected Time	Interval Time	Interval Velocity	Average Velocity
85.0	17.800	17.774	0.702	7121	
90.0	18.500	18.476	0.702	7124	7088
95.0	19.200	19.178	0.702	7126	
100.0	19.900	19.879	0.701	7128	
105.0	20.600	20.580	0.701	7130	
109.0	?				

Bottom of Casing at 110 feet.

Source to Borehole offset: 4.6 feet.
Casing stickup above ground: 0 feet.
n/a - Not included in Velocity Average.

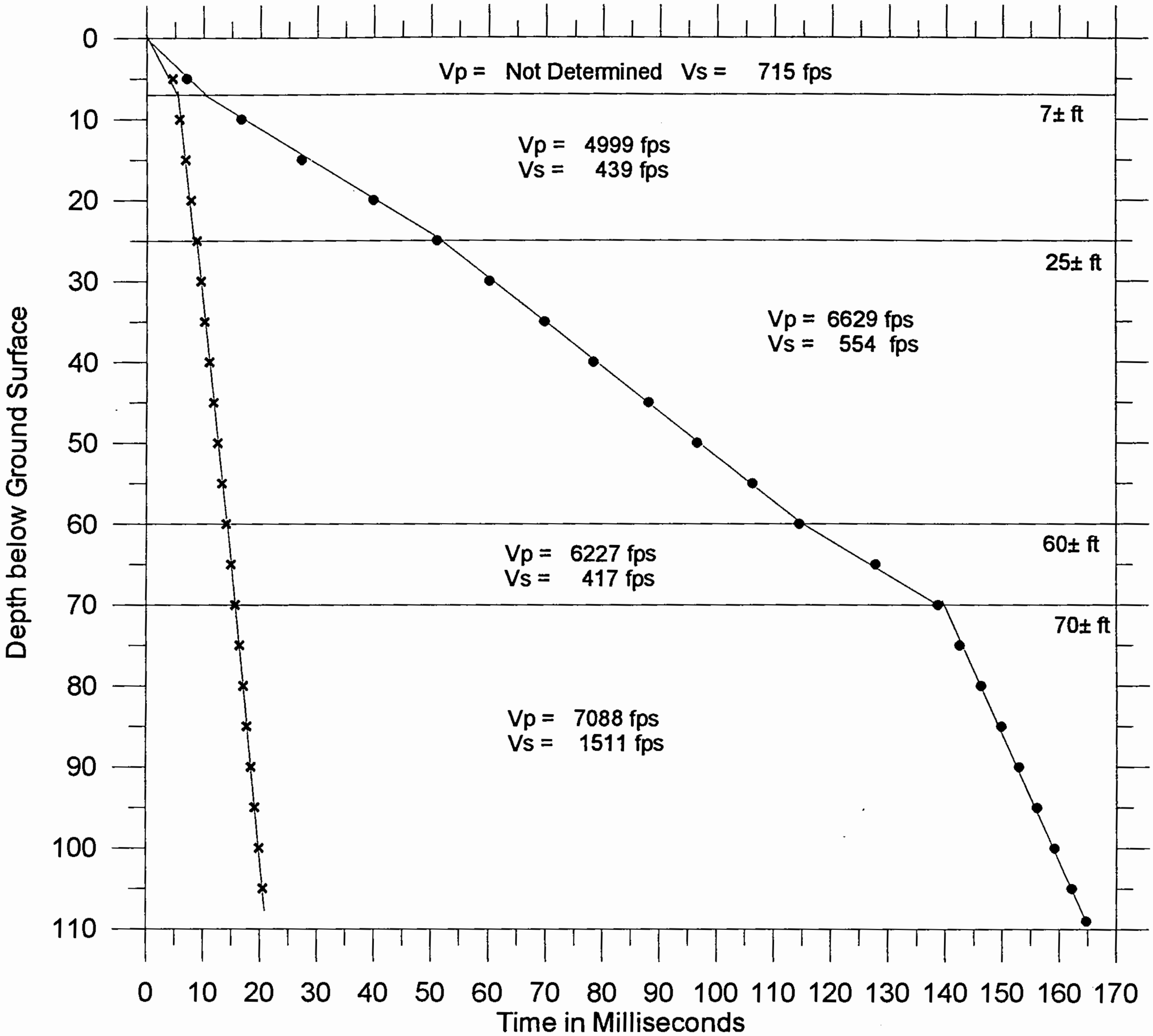
Velocities in feet per second.
Depths in feet - Times in milli-seconds.
Velocity Breaks from Time-Depth Plot.



Boring SB-2 - South Park Project
Shear and Compressional Wave Velocities

- • Shear Wave Arrival
- × Compressional Wave Arrival

Fig 1



Boring SB-6 - South Park Project
Shear and Compressional Wave Velocities

- ● Shear Wave Arrival
- × Compressional Wave Arrival

Fig 2

APPENDIX D

GEOTECHNICAL LABORATORY TESTING PROCEDURES AND RESULTS

APPENDIX D

GEOTECHNICAL LABORATORY TESTING PROCEDURES AND RESULTS

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LIST OF FIGURES

Figure No.

APPENDIX D

GEOTECHNICAL LABORATORY TESTING PROCEDURES AND RESULTS

D.1 INTRODUCTION

This appendix contains descriptions of the procedures and the results of the geotechnical laboratory tests performed on soil samples obtained from the borings performed for the South Park Bridge project. The samples were tested to determine the basic index and physical properties of the foundation soils. The laboratory testing was performed by an engineer or an experienced technician at the Shannon & Wilson, Inc. laboratory in Seattle.

D.2 VISUAL CLASSIFICATION

All of the soil samples recovered from the borings were visually reclassified in our laboratory using a system based on American Society for Testing of Materials (ASTM) Designation: D-2487, Standard Test Method for Classification of Soil for Engineering Purposes, and ASTM Designation: D-2488, Standard Recommended Practice for Description of Soils (Visual-Manual Procedure). This visual classification method allows for convenient and consistent comparison of soils from widespread geographic areas. Using this method, the soils can be classified by using the Unified Soil Classification System (USCS). The individual sample classifications have been incorporated into the boring logs presented in Appendix A. The USCS codes are also shown on Figures D-1 through D-8.

D.3 WATER CONTENT DETERMINATION

The natural water content of selected soil samples recovered from the borings were determined in general accordance with ASTM Designation: D-2216, Standard Method of Laboratory Determination of Water (Moisture) Content of Soil, Rock, and Soil-Aggregate Mixtures. Comparison of natural water content of a soil with its index properties can be useful in characterizing soil unit weight, consistency, compressibility, and strength. The water contents are plotted on the boring logs presented in Appendix A.

D.4 GRAIN-SIZE ANALYSIS

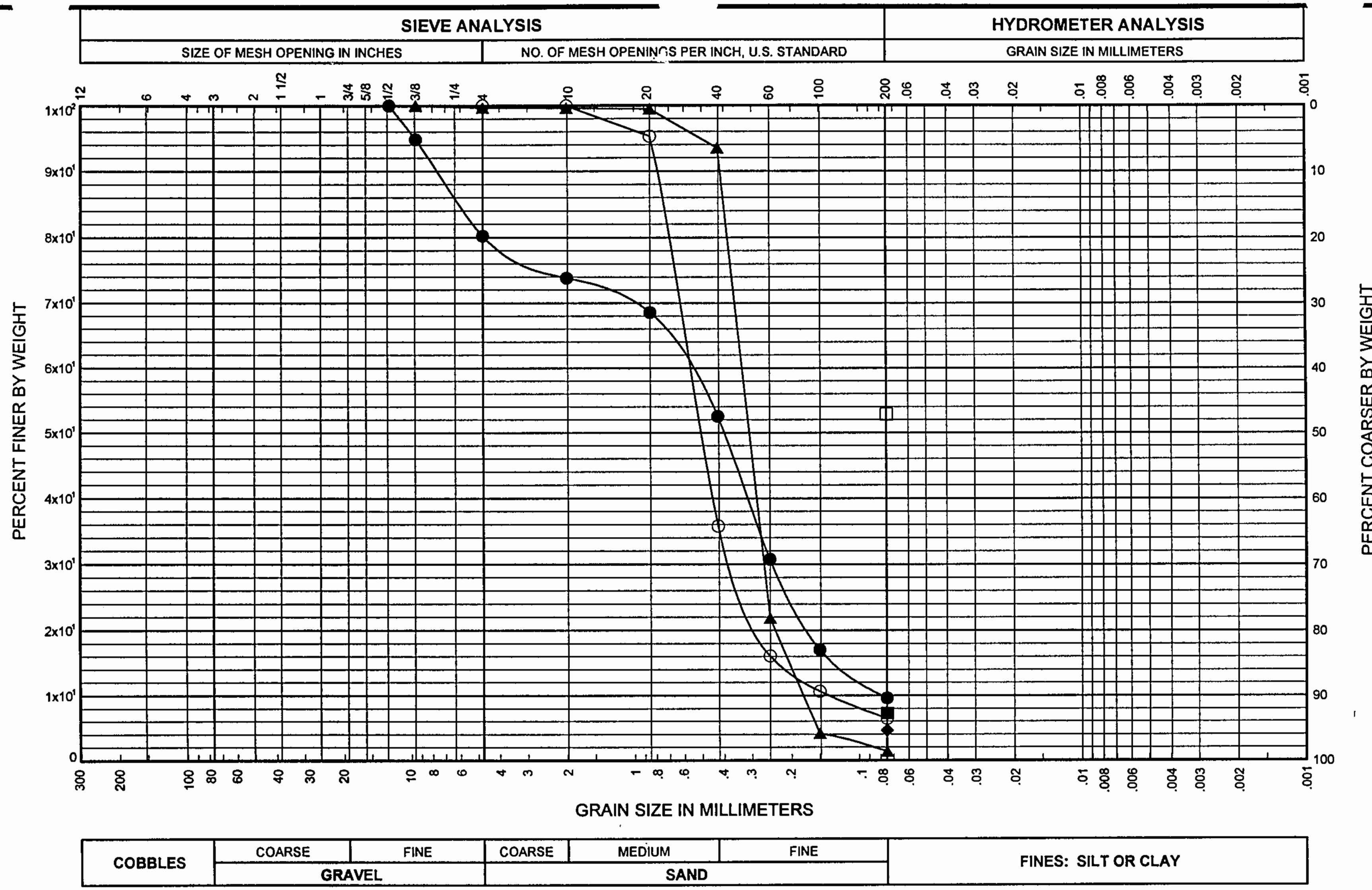
Grain-size analyses were performed on selected samples of granular soils in general accordance with ASTM Designations: D-422, Standard Method for Particle-Size Analysis of Soils and

D-1140, Amount of Material in Soils Finer than the No. 200 (75-µm). Three general procedures to determine the grain size distribution of a soil include sieve analysis, hydrometer analysis, and combined analysis.

Grain size distribution is used to assist in classifying soils and evaluating their liquefaction potential, and to provide correlation with soil properties, including permeability and capillarity. The results of the grain size analyses are plotted on the grain-size distribution curves presented in Figures D-1 and D-8.

D.5 ATTERBERG LIMITS DETERMINATION

The Atterberg limits were determined on selected samples of fine-grained soil obtained in the field explorations in general accordance with ASTM Designation: D-4318, Standard Test Method for Liquid Limit, Plastic Limit, and Plasticity Index of Soils. The Atterberg limits include Liquid Limit (LL), Plastic Limit (PL), and Plasticity Index (PI=LL-PL). They are generally used to assist in classification of soils, indicate soil consistency (when compared with natural water content), and provide correlation to soil properties including compressibility and strength. The results of the Atterberg limits determination are shown graphically on the plasticity chart presented in Figures D-9 through D-11 and shown on the respective boring logs in Appendix A.



BORING AND SAMPLE NO.	DEPTH (feet)	U.S.C.S. SYMBOL	SAMPLE DESCRIPTION	FINES %	NAT. W.C. %	LL %	PL %	PI %
● SB-1, S-1	2.5	SW-SM	Brown, slightly silty, fine gravelly, SAND	9.6	5.1			
■ SB-1, S-6	15.0	SP-SM	Dark brown-gray, slightly silty, fine to medium SAND, trace of fine gravel	7.3	28.0			
▲ SB-1, S-7	17.5	SP	Dark brown, fine SAND, trace of silt	1.3	30.7			
◆ SB-1, S-9	25.0	SP	Dark gray-brown, fine to medium SAND, trace of silt	4.6	34.7			
○ SB-1, S-12	40.0	SP-SM	Dark brown, slightly silty, fine to medium SAND	6.4	25.5			
□ SB-1, S-16	60.0	ML	Dark brown, fine sandy SILT	52.8	31.0			

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GRAIN SIZE DISTRIBUTION
BORING SB-1

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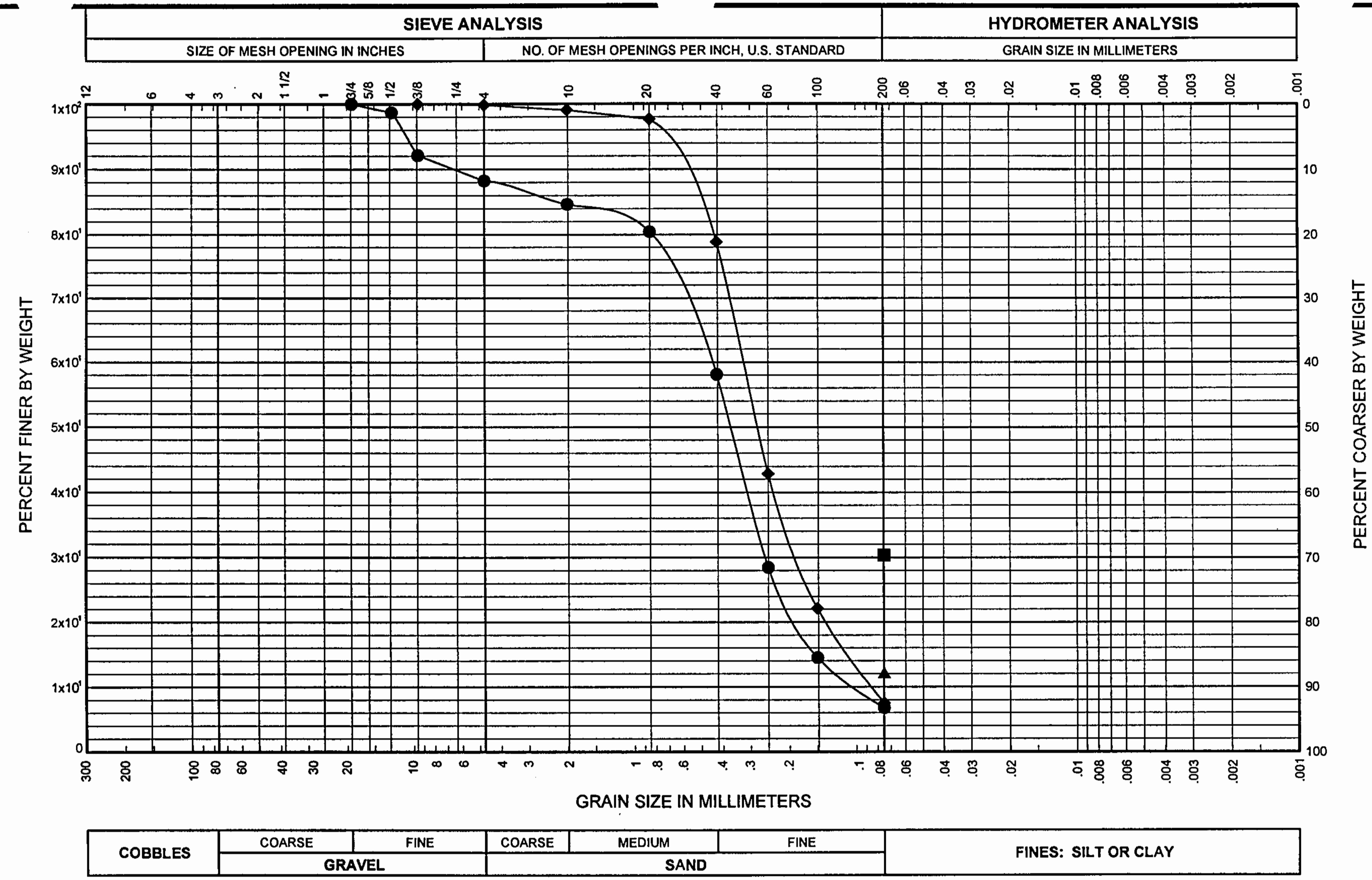
FIG. D-1

FIG. D-1



BORING AND SAMPLE NO.	DEPTH (feet)	U.S.C.S. SYMBOL	SAMPLE DESCRIPTION	FINES %	NAT. W.C. %	LL %	PL %	PI %
● SB-2, S-2	5.0	SP-SM	Dark brown, slightly silty, slightly gravelly, fine to medium SAND; hydrocarbon odor	6.8	5.9			
■ SB-2, S-6	15.0	SM	Dark gray, silty, fine SAND	30.4	31.1			
▲ SB-2, S-7	17.5	SM	Dark gray-brown, silty, fine SAND	12.2	32.9			
◆ SB-2, S-8	20.0	SP-SM	Dark gray, slightly silty, fine to medium SAND; scattered wood fragments	7.5	36.2			

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GRAIN SIZE DISTRIBUTION
BORING SB-2

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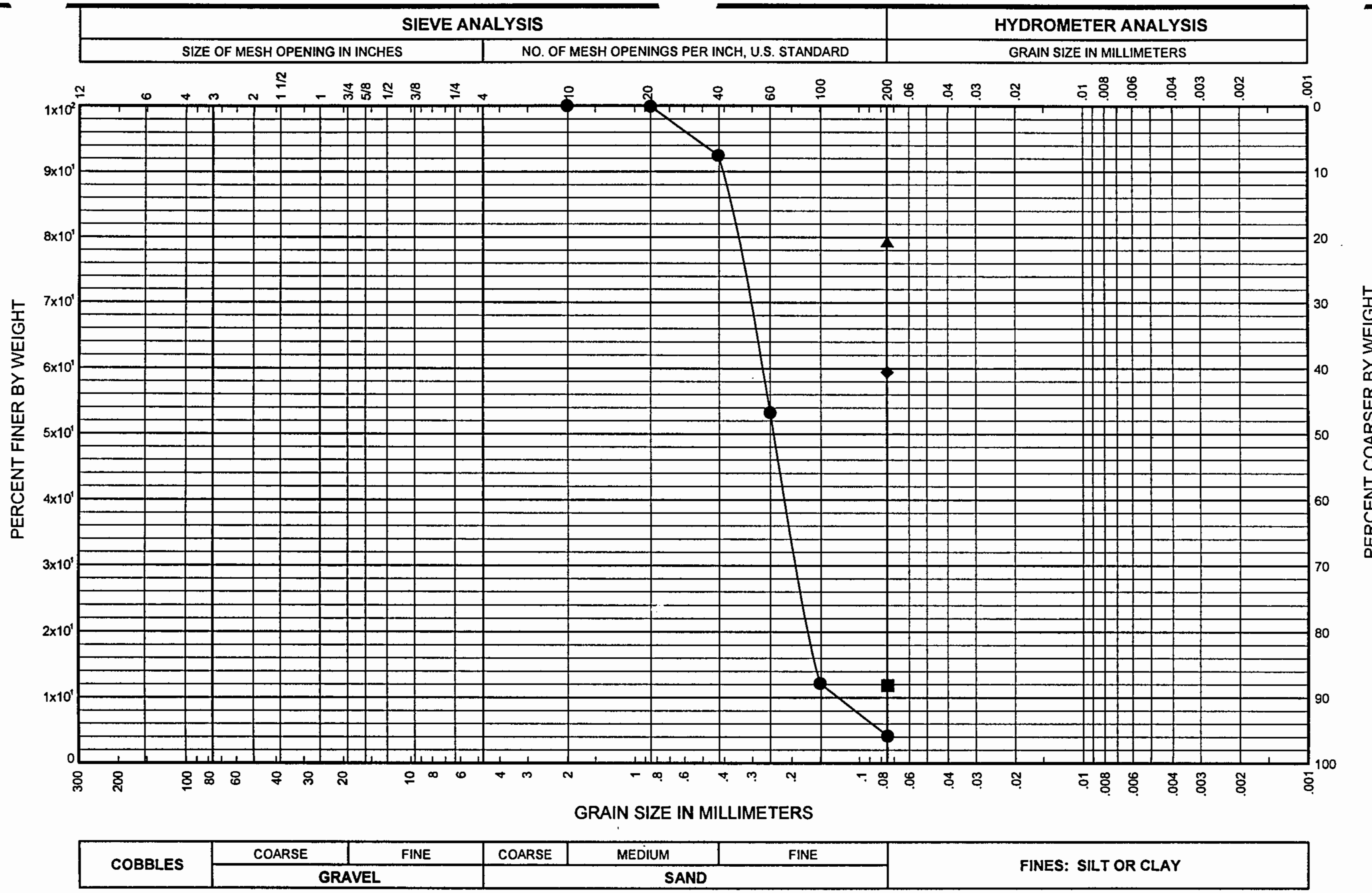
FIG. D-2a

FIG. D-2a



BORING AND SAMPLE NO.	DEPTH (feet)	U.S.C.S. SYMBOL	SAMPLE DESCRIPTION	FINES %	NAT. W.C. %	LL %	PL %	PI %
● SB-2, S-9	25.0	SP	Dark brown, fine SAND, trace of silt	4.2	35.5			
■ SB-2, S-14	50.0	SP-SM	Dark gray, slightly silty, fine SAND	11.9	34.1			
▲ SB-2, S-17	65.0	ML	Dark gray, sandy SILT, trace of fine gravel	79.3	35.5	NP	NP	NP
◆ SB-2, S-20	80.0	ML	Gray, sandy SILT, trace of fine gravel	59.4	14.9			

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GRAIN SIZE DISTRIBUTION BORING SB-2

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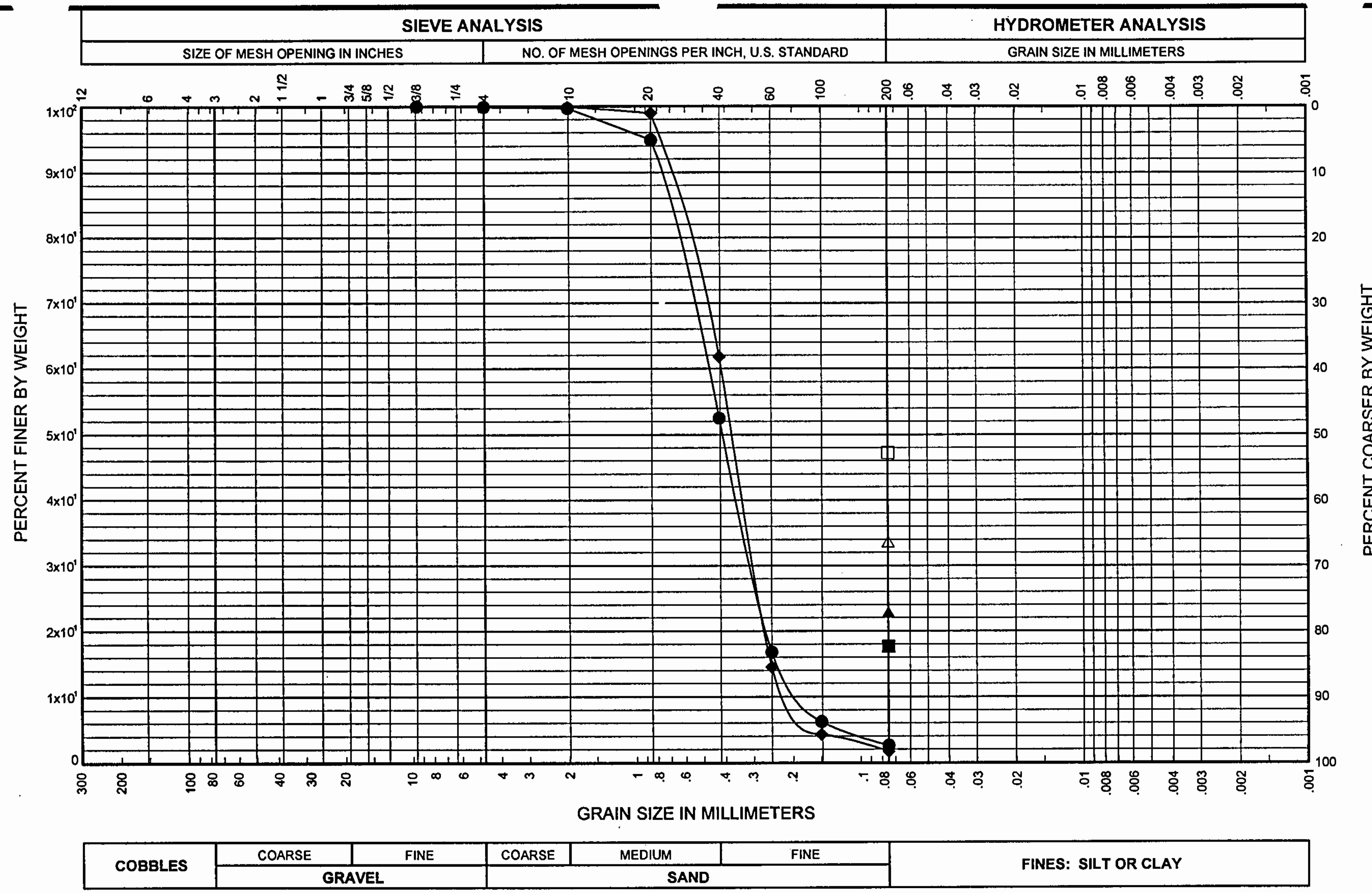
FIG. D-2b

FIG. D-2b



BORING AND SAMPLE NO.	DEPTH (feet)	U.S.C.S. SYMBOL	SAMPLE DESCRIPTION	FINES %	NAT. W.C. %	LL %	PL %	PI %
● SB-3, S-2	7.5	SP	Brown, fine to medium SAND, trace of silt	2.6	6.8			
■ SB-3, S-4	12.5	SM	Dark brown, silty, fine to medium SAND; layers of organic silt	17.7	32.4			
▲ SB-3, S-5	15.0	SM	Dark gray-brown, silty, fine SAND	22.8	33.0			
◆ SB-3, S-6	17.5	SP	Dark gray-brown, fine to medium SAND, trace of silt	1.7	27.5			
○ SB-3, S-7	20.0	SP	Dark gray-brown, fine-medium SAND, trace of silt	2.6	29.9			
□ SB-3, S-11	40.0	SM	Gray, silty, fine SAND	47.2	37.7			
△ SB-3, S-12	45.0	SM	Dark gray-brown, silty, fine SAND; scattered to abundant organics	33.6	43.9			

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GRAIN SIZE DISTRIBUTION
BORING SB-3

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FIG. D-3a

FIG. D-3a

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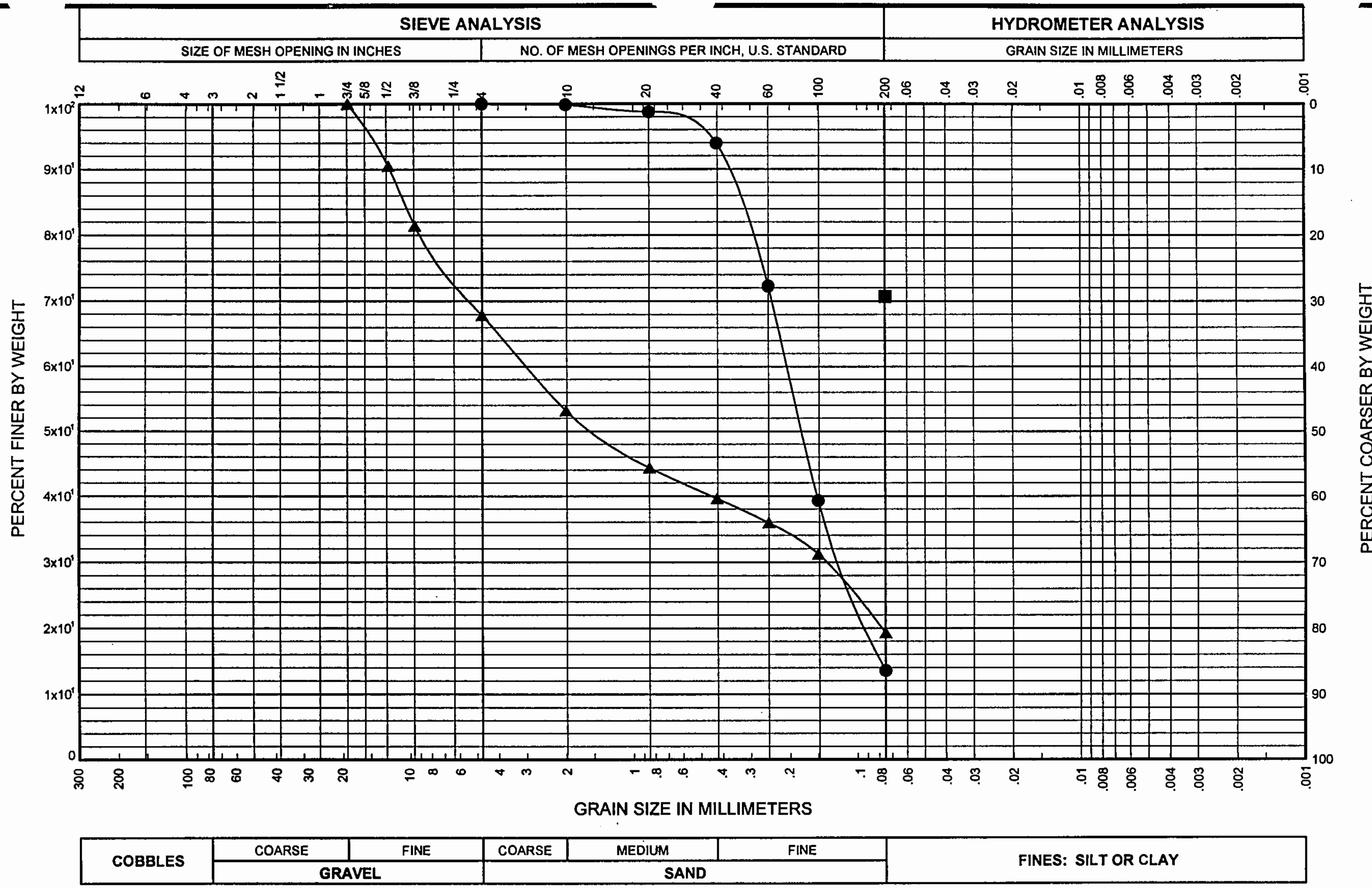

BORING AND SAMPLE NO.	DEPTH (feet)	U.S.C.S. SYMBOL	SAMPLE DESCRIPTION	FINES %	NAT. W.C. %	LL %	PL %	PI %
● SB-3, S-14	55.0	SM	Dark gray-brown, silty, fine SAND	13.6	32.3			
■ SB-3, S-19	80.0	ML	Dark gray, fine sandy SILT	70.7	32.4			
▲ SB-3, S-23	100.0	SM	Dark gray-green, silty, gravelly SAND; abundant shell fragments	19.3	16.4			

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GRAIN SIZE DISTRIBUTION
BORING SB-3

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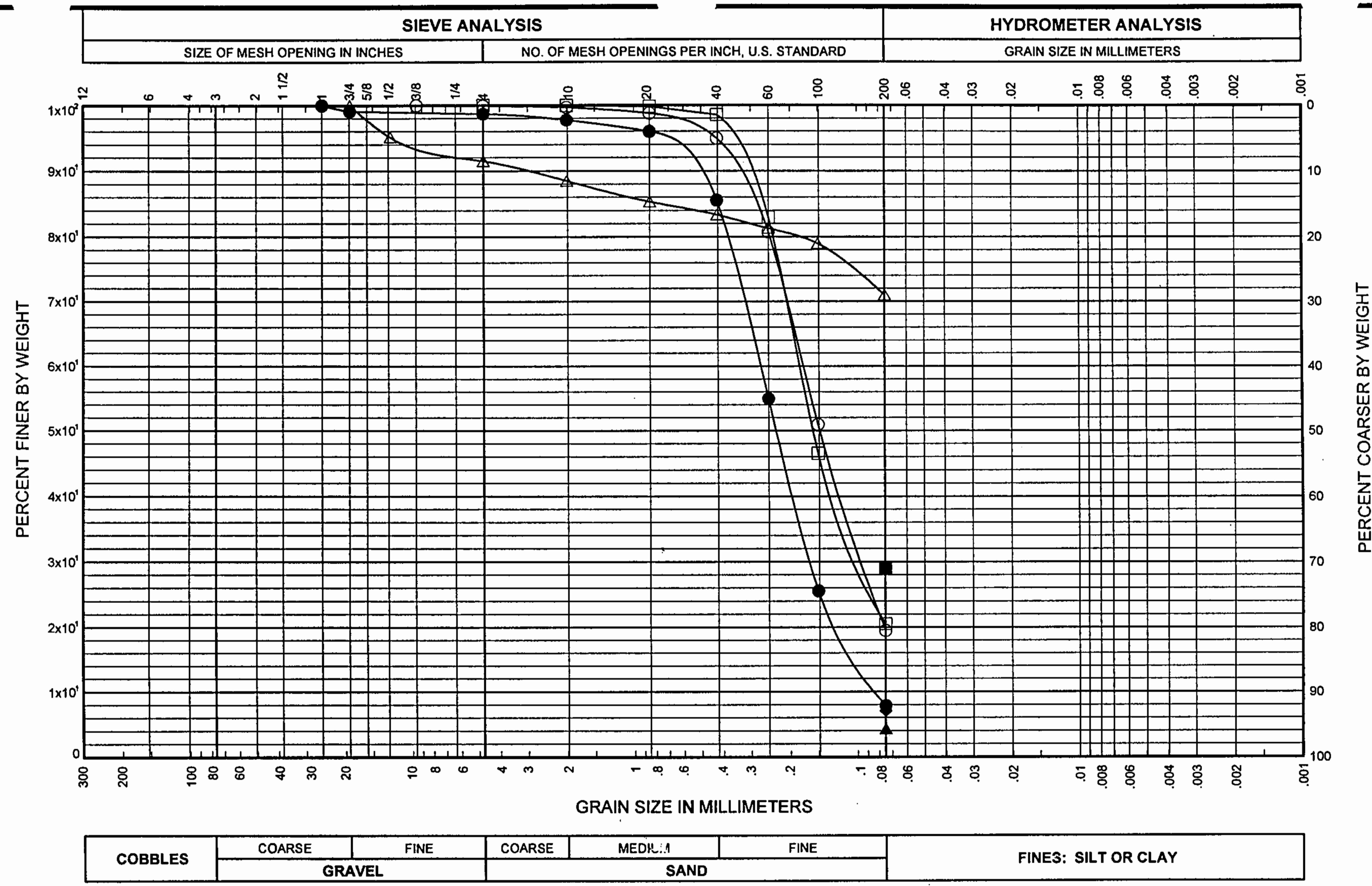
FIG. D-3b

FIG. D-3b



BORING AND SAMPLE NO.	DEPTH (feet)	U.S.C.S. SYMBOL	SAMPLE DESCRIPTION	FINES %	NAT. W.C. %	LL %	PL %	PI %
● SB-4, S-5	12.5	SP-SM	Dark brown, slightly silty, fine to medium SAND, trace of gravel; abundant organics	7.9	77.9			
■ SB-4, S-7	17.5	SM	Dark gray-brown, silty, fine SAND; scattered organics and wood fragments	28.9	41.7			
▲ SB-4, S-8	20.0	SP	Dark brown-gray, fine to medium SAND, trace of silt; scattered organics	4.3	31.4			
◆ SB-4, S-10	30.0	SP-SM	Dark brown-gray, slightly silty, fine to medium SAND; scattered organics	7.1	30.9			
○ SB-4, S-11	35.0	SM	Dark brown, silty, fine SAND; scattered organics	19.5	31.3			
□ SB-4, S-15	51.5	SM	Dark gray, silty, fine SAND; scattered organics	20.5	30.1			
△ SB-4, S-21	75.0	ML	Dark brown-gray, slightly gravelly, sandy SILT; scattered shell fragments	71.0	41.4			

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GRAIN SIZE DISTRIBUTION
BORING SB-4

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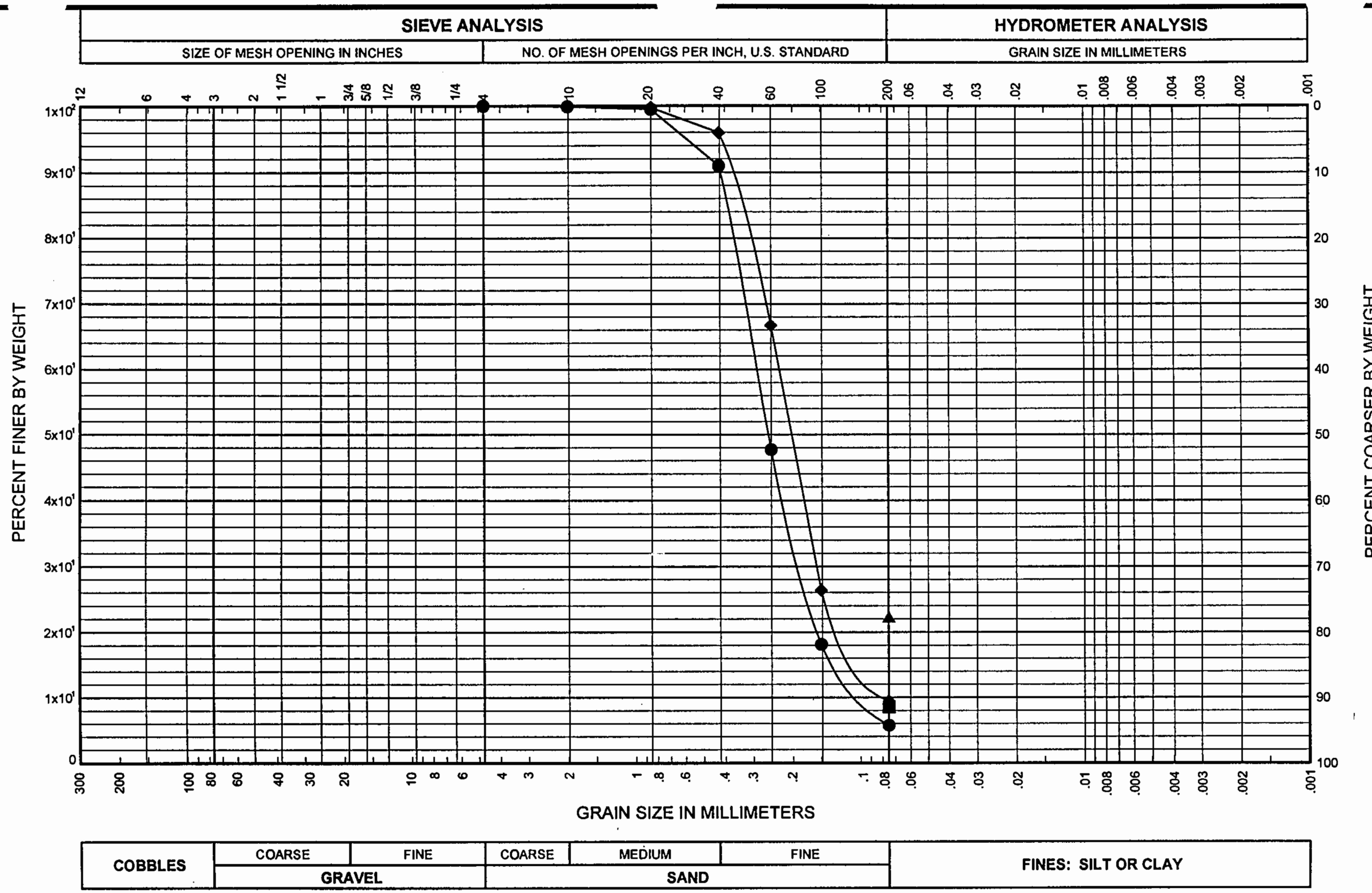
FIG. D-4



BORING AND SAMPLE NO.	DEPTH (feet)	U.S.C.S. SYMBOL	SAMPLE DESCRIPTION	FINES %	NAT. W.C. %	LL %	PL %	PI %
● SB-5, S-6	17.5	SP-SM	Dark brown, slightly silty, fine SAND	5.7	29.7			
■ SB-5, S-8	25.0	SP-SM	Dark gray-brown, slightly silty, fine SAND	8.5	41.9			
▲ SB-5, S-9	30.0	SM	Dark brown, silty, fine SAND; scattered organics	22.3	37.1			
◆ SB-5, S-11	40.0	SP-SM	Dark brown, slightly silty, fine SAND	9.4	37.1			

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GRAIN SIZE DISTRIBUTION
BORING SB-5

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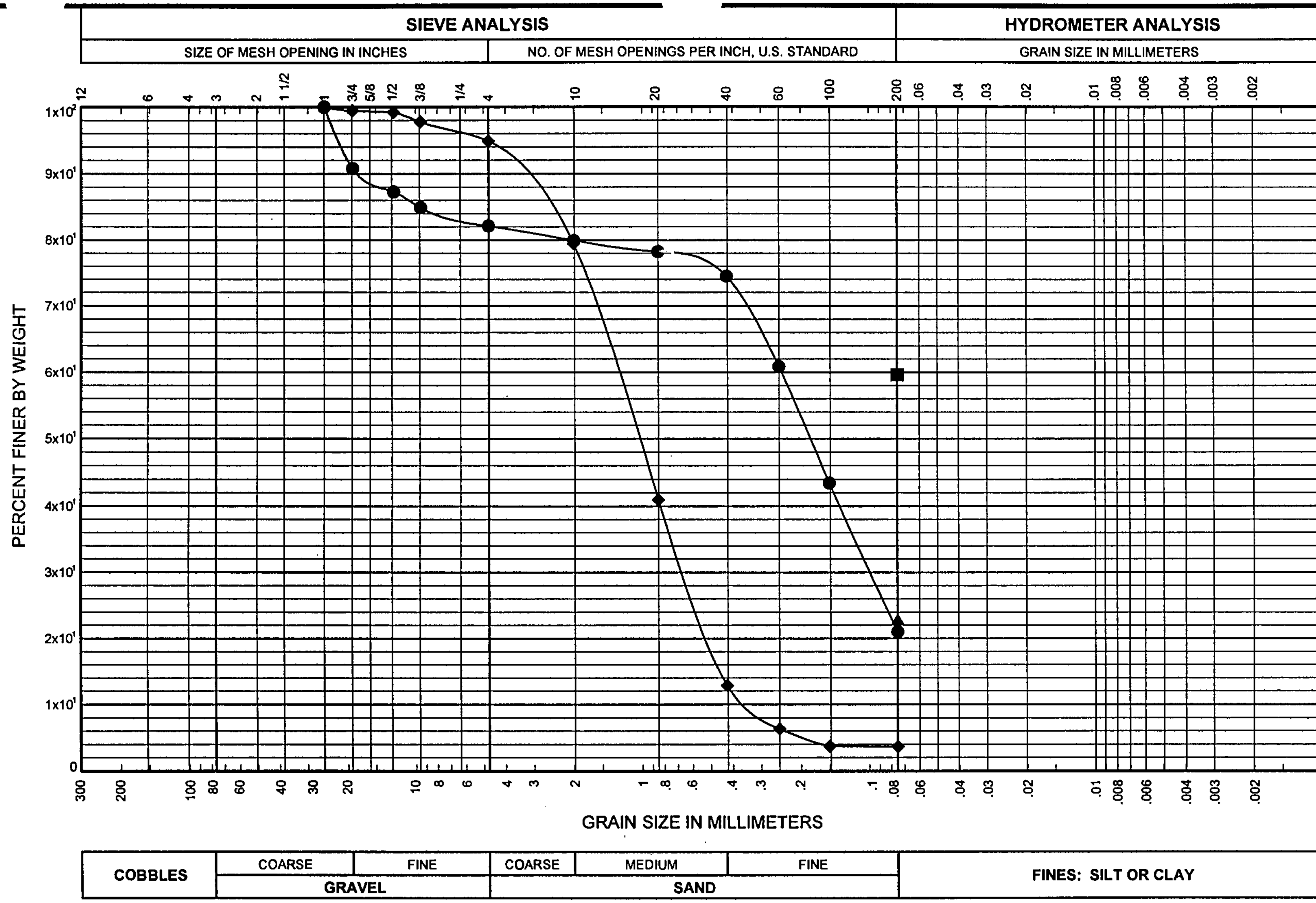
FIG. D-5



BORING AND SAMPLE NO.	DEPTH (feet)	U.S.C.S. SYMBOL	SAMPLE DESCRIPTION	FINES %	NAT. W.C. %	LL %	PL %	PI %
● SB-6, S-1	2.5	SM	Brown, gravelly, silty, fine SAND	21.0	12.1			
■ SB-6, S-6	15.0	ML	Brown, fine sandy SILT; iron oxide stains	59.6	35.1			
▲ SB-6, S-8	20.0	SM	Brown, silty, fine to medium SAND, trace of fine gravel	22.9	26.5			
◆ SB-6, S-9	25.0	SP	Brown, fine to medium SAND, trace of silt and gravel; abundant organics	3.7	38.9			

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GRAIN SIZE DISTRIBUTION
BORING SB-6

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FIG. D-6a

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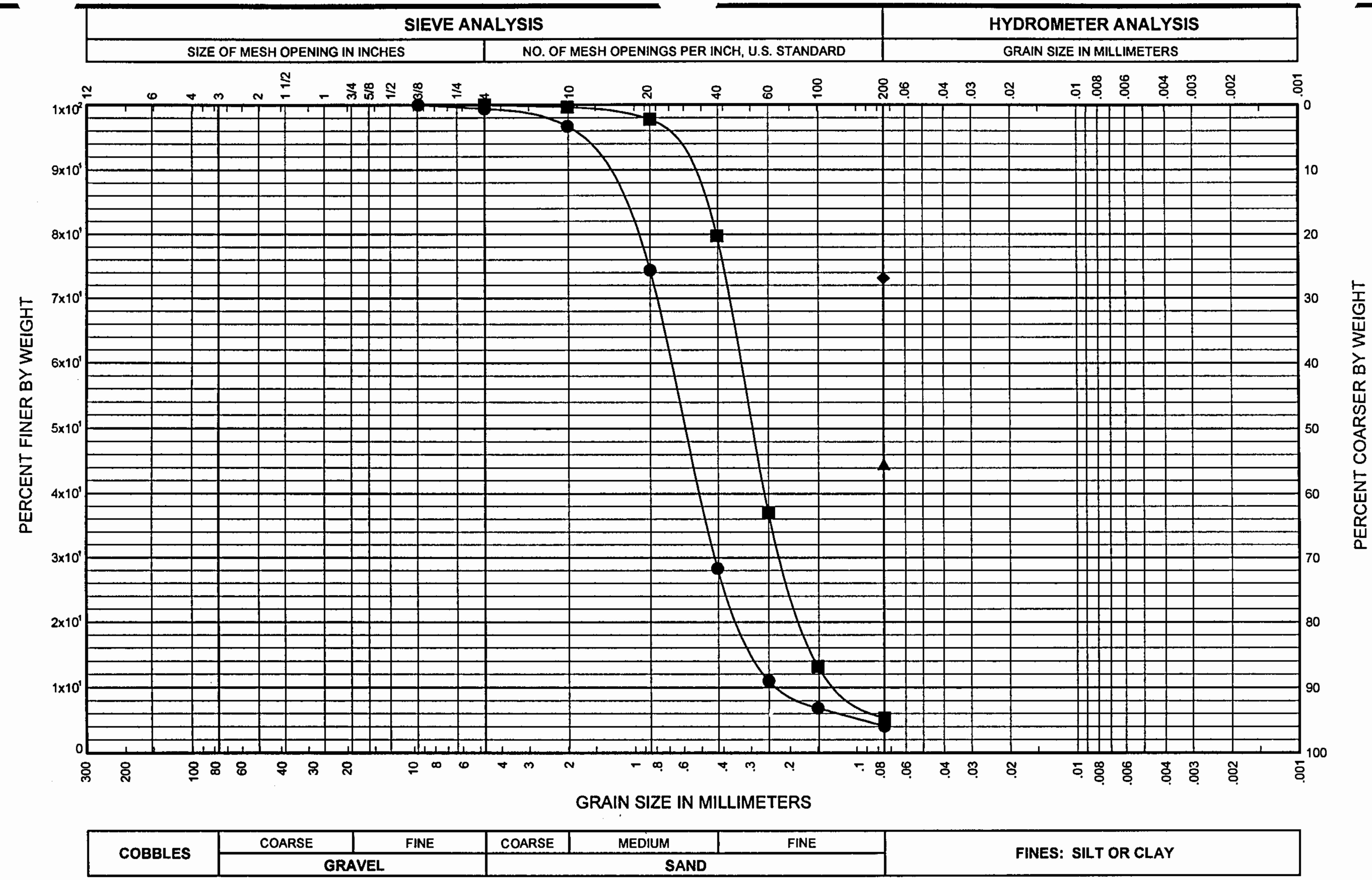

BORING AND SAMPLE NO.	DEPTH (feet)	U.S.C.S. SYMBOL	SAMPLE DESCRIPTION	FINES %	NAT. W.C. %	LL %	PL %	PI %
● SB-6, S-10	30.0	SP	Brown, fine to medium SAND, trace of silt	4.1	24.7			
■ SB-6, S-13	45.0	SP-SM	Dark gray, slightly silty, fine to medium SAND	5.3	28.1			
▲ SB-6, S-16	60.0	SM	Dark gray, silty, fine SAND, trace of fine gravel	44.5	27.2	NP	NP	NP
◆ SB-6, S-21	80.0	CL	Brown-gray, sandy, silty CLAY	73.1	19.7			

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GRAIN SIZE DISTRIBUTION
BORING SB-6

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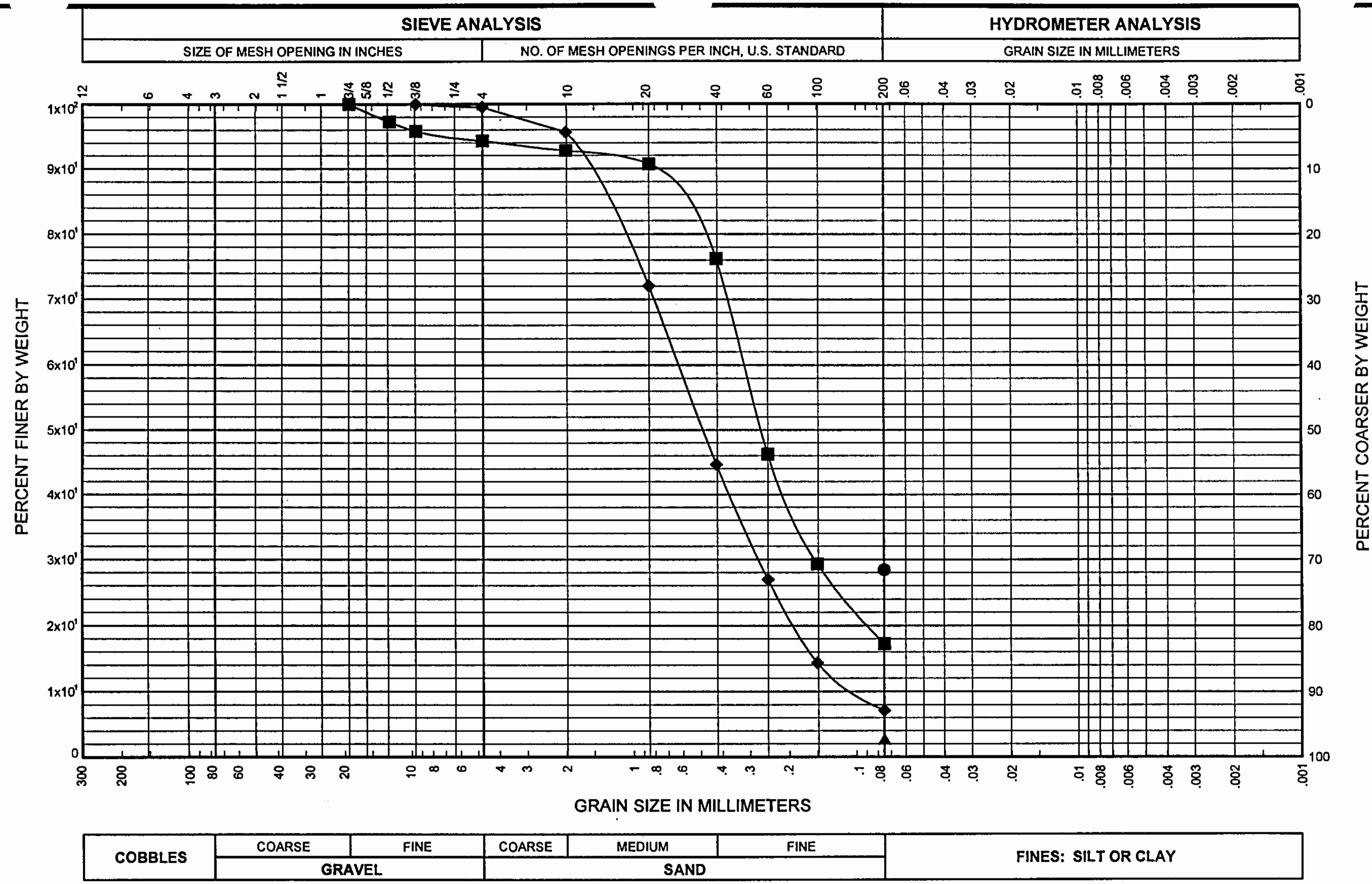
FIG. D-6b

FIG. D-6b



BORING AND SAMPLE NO.	DEPTH (feet)	U.S.C.S. SYMBOL	SAMPLE DESCRIPTION	FINES %	NAT. W.C. %	LL %	PL %	PI %
● SB-7, S-6	15.0	SM	Dark brown, slightly clayey, silty, fine to medium SAND; scattered wood fragments	28.4	38.0			
■ SB-7, S-8	20.0	SM	Dark brown, slightly gravelly, silty, fine to medium SAND; scattered organics	17.2	28.1			
▲ SB-7, S-9	25.0	SP/SW	Dark gray SAND, trace of silt	2.7	24.3			
◆ SB-7, S-11	35.0	SW-SM	Brown, slightly silty SAND	7.1	23.1			

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GRAIN SIZE DISTRIBUTION
BORING SB-7

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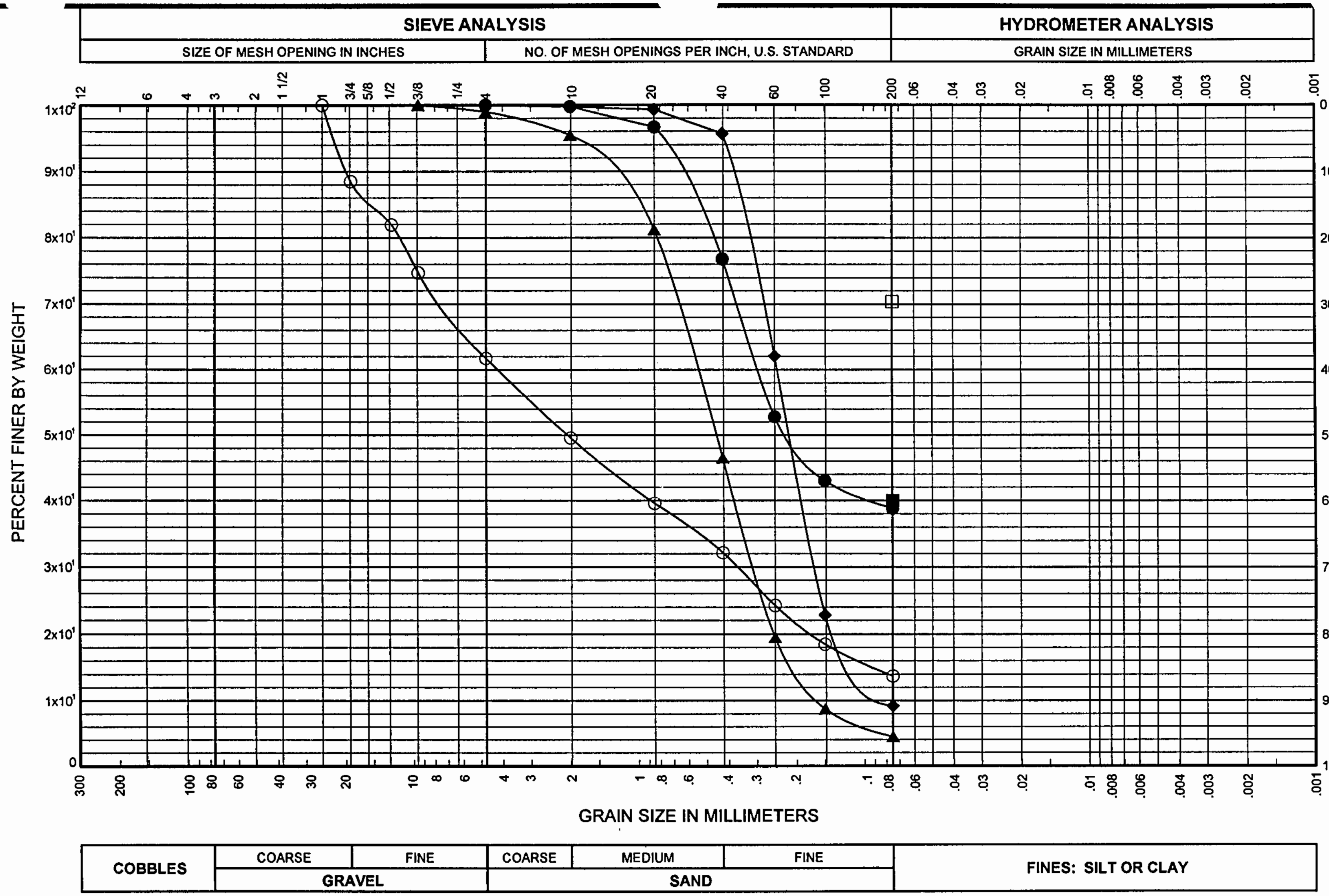
FIG. D-7

GSAMAIN1 21-09584.GPJ SHAN WIL.GDT 11/18/03

FIG. D-7



BORING AND SAMPLE NO.	DEPTH (feet)	U.S.C.S. SYMBOL	SAMPLE DESCRIPTION	FINES %	NAT. W.C. %	LL %	PL %	PI %
● SB-8, S-1	2.5	SM	Dark brown, silty, fine to medium SAND; seams of fine sandy silt	38.8	22.8			
■ SB-8, S-3	7.5	SM	Dark brown-gray, silty, fine SAND	39.9	18.7			
▲ SB-8, S-6	15.0	SP	Dark gray, fine to medium SAND, trace of silt and gravel	4.4	22.3			
◆ SB-8, S-9	25.0	SP-SM	Dark gray, slightly silty, fine SAND; scattered organics	9.2	29.0			
○ SB-8, S-14	45.0	SM	Light brown, slightly clayey, silty, gravelly SAND	13.7	10.4			
□ SB-8, S-17	60.0	CL	Gray, sandy, silty CLAY	70.4	14.4	34	17	17

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GRAIN SIZE DISTRIBUTION BORING SB-8

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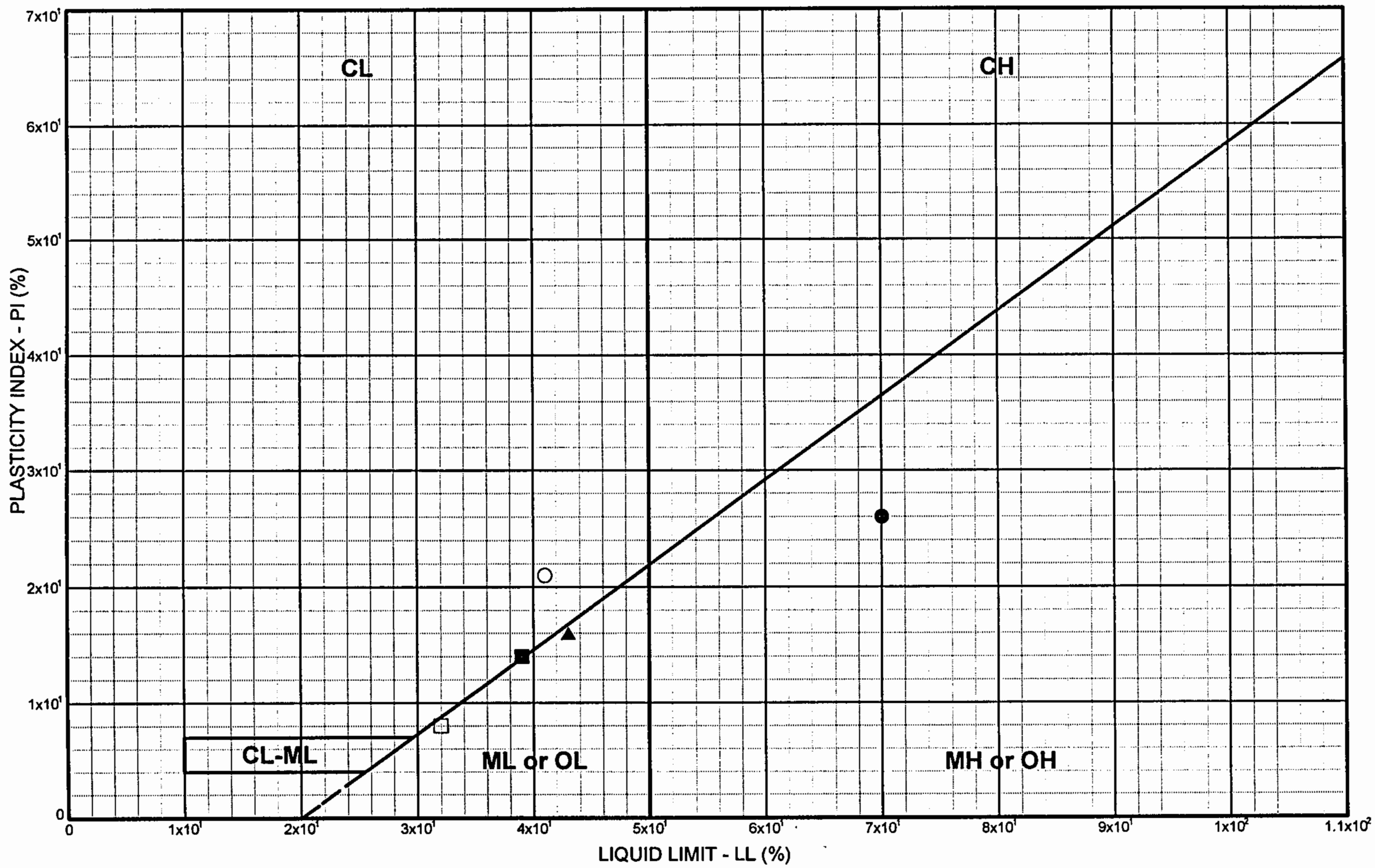
FIG. D-8



LEGEND

CL: Low plasticity inorganic clays; sandy and silty clays

CH: High plasticity inorganic clays

ML or OL: Inorganic and organic silts and clayey silts of low plasticity

MH or OH: Inorganic and organic silts and clayey silts of high plasticity

CL-ML: Silty clays and clayey silts

BORING AND SAMPLE NO.	DEPTH (feet)	U.S.C.S. SYMBOL	SOIL CLASSIFICATION	LL %	PL %	PI %	NAT. W.C. %	PASS. #200, %
● SB-1, S-4	10.0	MH	Dark brown, slightly fine sandy, clayey SILT; scattered organics	70	44	26	33.1	
■ SB-1, S-19	70.0	CL	Gray, silty CLAY, trace of fine sand and gravel	39	25	14	16.1	
▲ SB-1, S-22	85.0	ML	Gray, clayey SILT	43	27	16	18.5	
SB-2, S-17	65.0	ML	Dark gray, sandy SILT, trace of fine gravel	NP	NP	NP	35.5	79.3
○ SB-2, S-22	90.0	CL	Gray, slightly fine sandy, silty CLAY	41	20	21	17.3	
□ SB-3, S-20	85.0	ML	Dark gray, slightly sandy, clayey SILT; scattered shell fragments	32	24	8	39.2	

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PLASTICITY CHART
BORINGS SB-1 THROUGH SB-3

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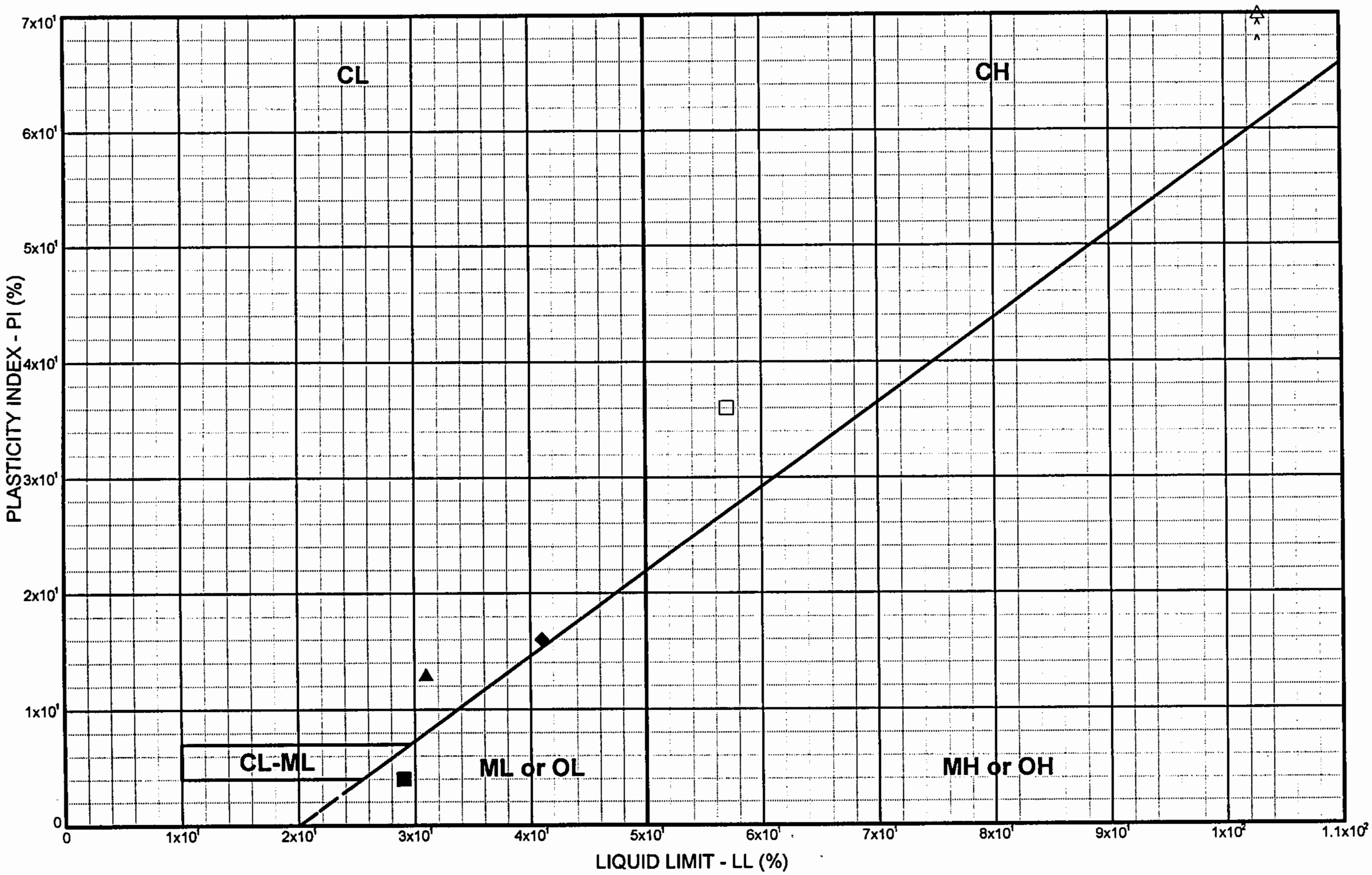
FIG. D-9



LEGEND

CL: Low plasticity inorganic clays; sandy and silty clays

CH: High plasticity inorganic clays

ML or OL: Inorganic and organic silts and clayey silts of low plasticity

MH or OH: Inorganic and organic silts and clayey silts of high plasticity

CL-ML: Silty clays and clayey silts

BORING AND SAMPLE NO.	DEPTH (feet)	U.S.C.S. SYMBOL	SOIL CLASSIFICATION	LL %	PL %	PI %	NAT. W.C. %	PASS. #200, %
SB-4, S-17	60.0	ML	Dark gray, fine sandy SILT	NP	NP	NP	34.8	
■ SB-4, S-22	80.0	ML	Gray, clayey SILT, trace of sand and fine gravel; scattered shell fragments	29	25	4	24.6	
▲ SB-5, S-14	51.5	CL	Dark gray, fine sandy, silty CLAY	31	18	13	33.0	
◆ SB-5, S-18	70.0	CL	Gray, silty CLAY, trace of sand	41	25	16	24.0	
SB-6, S-16	60.0	SM	Dark gray, silty, fine SAND, trace of fine gravel	NP	NP	NP	27.2	44.5
□ SB-6, S-18	70.0	CH	Gray, silty CLAY, trace of coarse sand and fine gravel	57	21	36	29.4	
△ SB-6, S-27	110.0	CH	Gray, silty CLAY	103	24	79	29.9	

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**PLASTICITY CHART
BORINGS SB-4 THROUGH SB-6**

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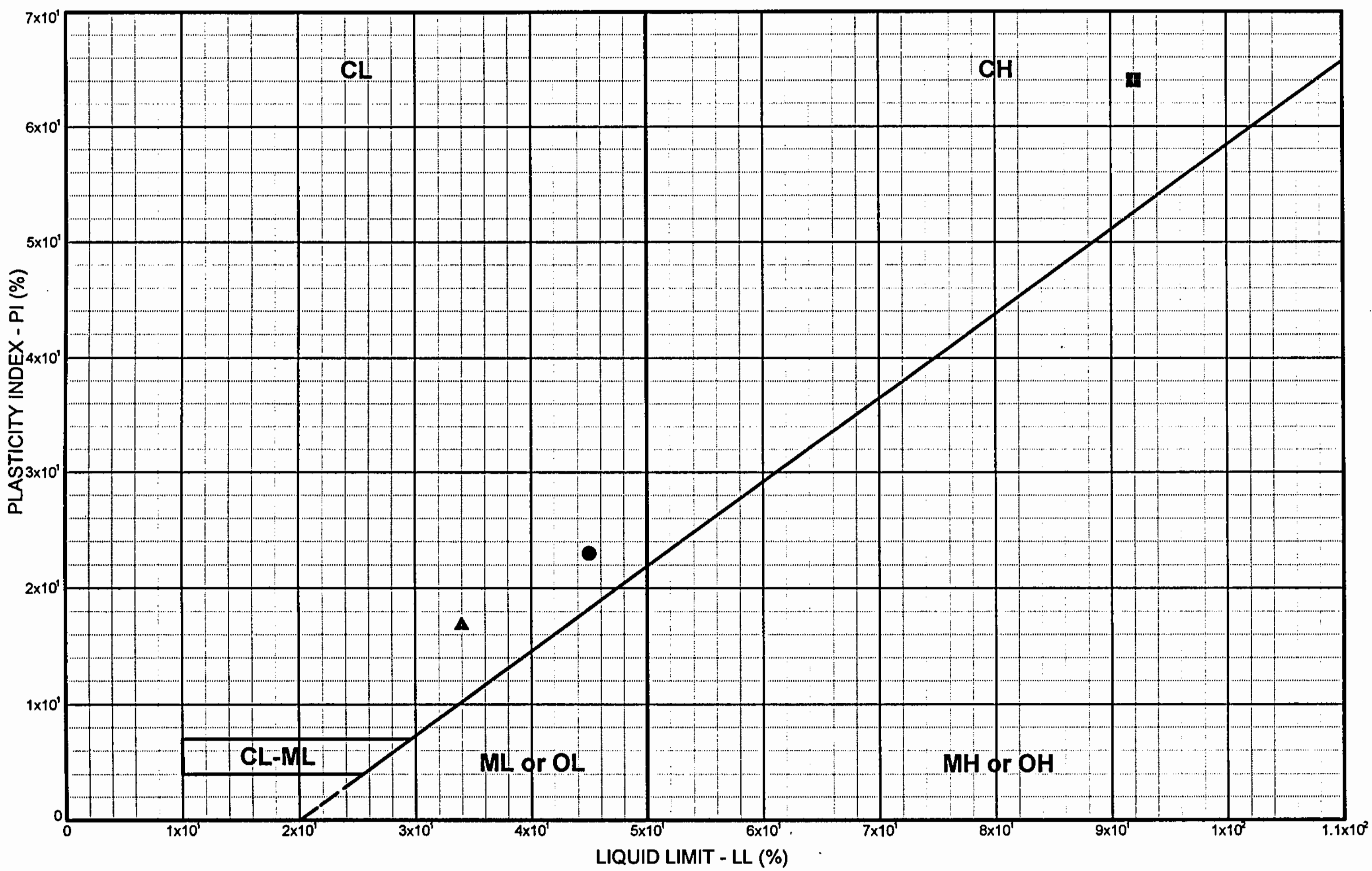
FIG. D-10



LEGEND

CL: Low plasticity inorganic clays; sandy and silty clays

CH: High plasticity inorganic clays

ML or OL: Inorganic and organic silts and clayey silts of low plasticity

MH or OH: Inorganic and organic silts and clayey silts of high plasticity

CL-ML: Silty clays and clayey silts

BORING AND SAMPLE NO.	DEPTH (feet)	U.S.C.S. SYMBOL	SOIL CLASSIFICATION	LL %	PL %	PI %	NAT. W.C. %	PASS. #200, %
● SB-7, S-16	60.0	CL	Gray, silty CLAY, trace of coarse sand and fine gravel	45	22	23	27.1	
■ SB-7, S-23	95.0	CH	Gray, silty CLAY	92	28	64	33.8	
▲ SB-8, S-17	60.0	CL	Gray, sandy, silty CLAY	34	17	17	14.4	70.4

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PLASTICITY CHART
BORINGS SB-7 AND SB-8

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FIG. D-11

APPENDIX E

TEST PILE CAPACITY AND INSTRUMENTATION

APPENDIX E

TEST PILE CAPACITY AND INSTRUMENTATION

TABLE OF CONTENTS

Page

LIST OF FIGURES

Figure No.

ATTACHMENT

APPENDIX E

TEST PILE CAPACITY AND INSTRUMENTATION

E.1 INTRODUCTION

A 24-inch-diameter, ½-inch-thick wall steel pipe pile (test pile) was driven for the South Park Bridge replacement project. The test pile was driven on July 22, 2003, and re-struck 24 hours later on July 23, 2003. Two 40 foot-long sections of pipe were used for this test. The test pile was driven closed-end with a 2-inch-thick steel plate welded to the bottom. The test pile was driven at Station 32+20, approximately 350 feet east of the existing east bridge abutment. The ground surface elevation is approximately 17.0 feet. Boring SB-1 was drilled prior to driving the test pile at Station 32+25. Both the test pile and boring are located in the shoulder on the north side of 16th Avenue South.

Boring SB-1 was drilled to a depth of 96 feet and the boring log is presented in Appendix A. A Sondex casing was installed into the borehole following drilling and soil sampling. Sondex is an instrument system designed to measure ground settlement and is described in the following sections.

E.2 PILE DRIVING

The hammer used to drive the test pile was an open-end and single acting diesel Berminghammer B4505. The maximum rated energy for this hammer is 75,900 foot-pounds per blow. The hammer weighs 17,600 pounds with a ram weight of 6,600 pounds. The hammer has a rated maximum stroke of 11.5 feet. The hammer was operated at about 44 blows per minute at a 7.0-foot stroke. The hammer was mounted in a swinging lead.

The first section was driven to 35 feet deep where driving was stopped and the second section welded on. A total of 80 feet of pile was used for the test. Following welding, the pile was driven to 50 feet deep. At this depth strain gages and accelerometers for dynamic testing were installed near the top of the pile by Robert Miner Dynamic Testing (RMDT). A pile dynamic test was performed using a pile driving analyzer (PDA). Driving was then resumed and the pile driven to 71.8 feet deep.

During driving, time, blows per foot, blows per minute and hammer stroke were recorded. These are shown on the attached Summary of Test Pile TP-1, Figure E-1. A summary of the test pile driving is as follows:

Depth (ft)	Time	Comment
0 to 35	1232 to 1252	Stopped to weld pile sections together.
35 to 50	1507 to 1525	Stopped to install instrument for dynamic testing.
50 to 71.8	1547 to 1612	Drove to substantial resistance.

The test pile was re-driven 24 hours after the initial drive to 71.8 feet. The re-drive consisted of driving the pile two additional feet from 72 to 74 feet deep.

The driving record of the test pile consists of driving resistance in blows per foot (bpf), blows per minute (bpm) and hammer stroke in feet. The Pile Driving Record presents the recorded values for each foot of driving. The results are plotted on the Summary of Test Pile TP-1, Figure E-1. This figure presents a log of the soil encountered in Boring SB-1, a plot of standard penetration resistance (SPT) at sample intervals of 2.5 and 5 feet, the driving resistance in bpf recorded during test pile driving, the range of hammer stroke in feet recorded between 35 and 74 feet, and the measured load distributed along the entire pile length obtained from a PDA and Case Pile Wave Analysis Program (CAPWAP) discussed in the following section.

E.3 DYNAMIC TESTING AND ANALYSIS

During pile driving and re-strike, a PDA was used to take dynamic measurements of the test pile. The PDA uses a transducer attached to the pile to measure strain and acceleration during hammer impact. These measurements are transmitted via cable to a data acquistion unit, which translates them into force and velocity. Using these measurements, calculations can be performed to estimate the pile capacity including pile stress and energy transfer as shown in Figure E-1.

The data collected from the PDA is also utilized by the CAPWAP analysis to calculate static capacity and soil resistance along the pile. The force and velocity measurements from the PDA are iterated using the CAPWAP analysis until signal matching based on site conditions is achieved. Once the analysis has been calibrated to match the site conditions, the capacities for skin friction and end bearing of the test pile can be estimated as shown in Figure E-1.

The PDA and CAPWAP analyses were performed by RMDT. Their report is presented at the end of this appendix section.

E.4 INSTRUMENTATION

E.4.1 Installation of Instrumentation

Boring SB-1 Sondex Casing. Boring SB-1 was drilled at approximate Station 32+25 along the proposed alignment near the north abutment of the bridge. The approximate location of the boring is shown on Figure 3. The layout of the instrumentation relative to the boring is shown on Figure E-2.

The boring was drilled by Geotech Explorations, Inc. under subcontract to Shannon & Wilson, Inc. and completed on July 9, 2003. A discussion of drilling and sampling procedures for this boring is presented in Appendix A of this report. The soil log of this boring is shown on Figure A-2 of Appendix A.

Following the drilling of boring SB-1, a Sondex settlement monitoring system, manufactured by Slope Indicator Company, was installed in the borehole. This system is comprised of a continuous length of corrugated plastic pipe grouted into the borehole with a cement-bentonite grout mix. Stainless steel sensing rings are attached at selected intervals along the corrugated pipe. A plastic pipe is installed within the corrugated pipe to provide access for the monitoring probe. Measurements are obtained by drawing a monitoring probe through the access pipe and recording depths at which the probe senses the metal rings. As the probe transits through the casing, a meter on the gage indicates when the probe is in alignment with a metal ring.

Horizontal Inclinometer Casing. For monitoring surface settlement laterally out from the driven pile, a horizontal profiler system was installed. This system consisted of 5-foot lengths of 3.34-inch outside-diameter (O.D.) relatively flexible plastic inclinometer casing, manufactured by RocTest Inc., coupled together to provide a straight 100-foot-long section of access pipe. The casing has internal grooves cut longitudinally at 90-degree intervals to serve as guides for the probe. The monitoring system, manufactured by Slope Indicator Company, consists of a gravity sensitive probe, a portable readout unit, and a graduated electrical cable that links the probe to the readout unit. Measurements of the system are obtained by pulling the probe through the casing and taking readings at 2-foot intervals throughout the length.

Measurements provide a profile of the casing, and subsequent measurements are compared to the initial to determine vertical displacement.

Prior to installation of the casing along the ground surface, an 18-inch-wide, 100-foot-long strip of asphalt was removed, on the morning of July 22, to provide for direct contact between the casing and the subgrade. The asphalt thickness along this section varied in thickness from 2 to 4 inches. The subgrade material consisted of angular gravel (base coarse) of an undetermined thickness.

The horizontal casing was assembled and placed along the strip of exposed subgrade, after leveling the strip with a shovel. A pull cable was installed in the casing during its assembly. The casing was then secured to the subgrade with 50-pound sand bags at 10-foot intervals along the casing. At 10-foot intervals along the casing, 5 feet from the sand bags, 2-foot-long pieces of rebar were driven into the subgrade on either side of the casing, bent against the casing, and secured with tie wire.

Vibration Monitoring Sensors. For monitoring vibrations resulting from the pile driving, two triaxial geophones were placed along the exposed subgrade. During the initial drive of the pile, the geophones were placed at 50 and 100 feet from the pile. Each geophone was placed on a prepared level surface on the subgrade and then secured in place with a 50-pound sand bag. Signal cables from each of these geophones were routed back to a Blastmate III seismograph, manufactured by Instantel.

E.4.2 Instrumentation Monitoring

On the day before pile driving, three sets of initial readings were obtained for the Sondex settlement casing, in order to provide a satisfactory set of baseline readings. On July 22, two initial readings for the horizontal inclinometer casing were obtained prior to pile driving; one before the leads for the pile were in place and one after. Also prior to driving the pile, background vibrations were monitored to determine level of vibrations created by traffic and adjacent facility activities.

During pile driving, measurements of the horizontal inclinometer casing and the Sondex system could not be performed due to safety concerns. Measurements of the horizontal inclinometer casing were performed with the pile tip at 35-foot depth (during welding of pile sections), at 71-foot depth, at 73-foot depth (after restrike), and two days after restrike. The subsurface measurements from the Sondex were performed with the pile tip at 71-foot depth, at 73-foot depth (after restrike), and two days after restrike. Vibrations were monitored throughout the pile driving, with periodic down time due to equipment problems.

E.4.3 Results of Test Pile Instrumentation

Vibrations. Figure E-3 presents plotted measured peak particle velocity in inches per second (ips) versus pile tip depth for each recorded event. The vibration sensors were located at 50 and 100 feet from the pile, to simulate approximate distances to existing structures from proposed pile locations. During restrike of the pile, one sensor was relocated to within 20 feet of the test pile.

In general, maximum measured vibrations occurred soon after the pile tip had penetrated the fill material (depths of approximately 10 feet). As shown on Figure E-3, vibrations measured at 50 feet from the pile peaked at approximately 0.5 ips, with the pile tip at 15 feet, and then attenuated with depth. Similarly, the sensor located 100 feet from the pile displays a maximum measured peak particle velocity of 0.25 ips, with the pile tip at 15 feet. Sensor problems prohibited the continued monitoring at this location; however, it is expected that the readings would have also exhibited a similar attenuation with depth.

With the sensor located at 20 feet from the pile (and pile tip at 71 feet), vibration events generally were around 0.35 to 0.4 ips. Based on our experience and the data presented herein, we expect that vibrations at 20 and 10 feet distance from the pile driving could be up to 1 ips and 1.5 ips, respectively.

Noise levels for the pile driving activities range from about 110 to 135 dB, with an average 125 dB at a distance of about 20 feet from the pile.

Subsurface Settlement. Figure E-4 presents calculated vertical displacement of several discrete points grouted along a vertical casing located approximately 4 feet from the driven pile. As shown, vertical displacements of up to 1 inch were measured in the upper 10 feet of the casing, corresponding to the loose fill material encountered in the boring. Below that, the vertical displacements decreased with depth, from approximately 0.5-inch at 15-foot depth to negligible at 60-foot depth.

As shown on the figure, up to 0.25 inches of residual settlement (near surface) occurred following the pile driving.

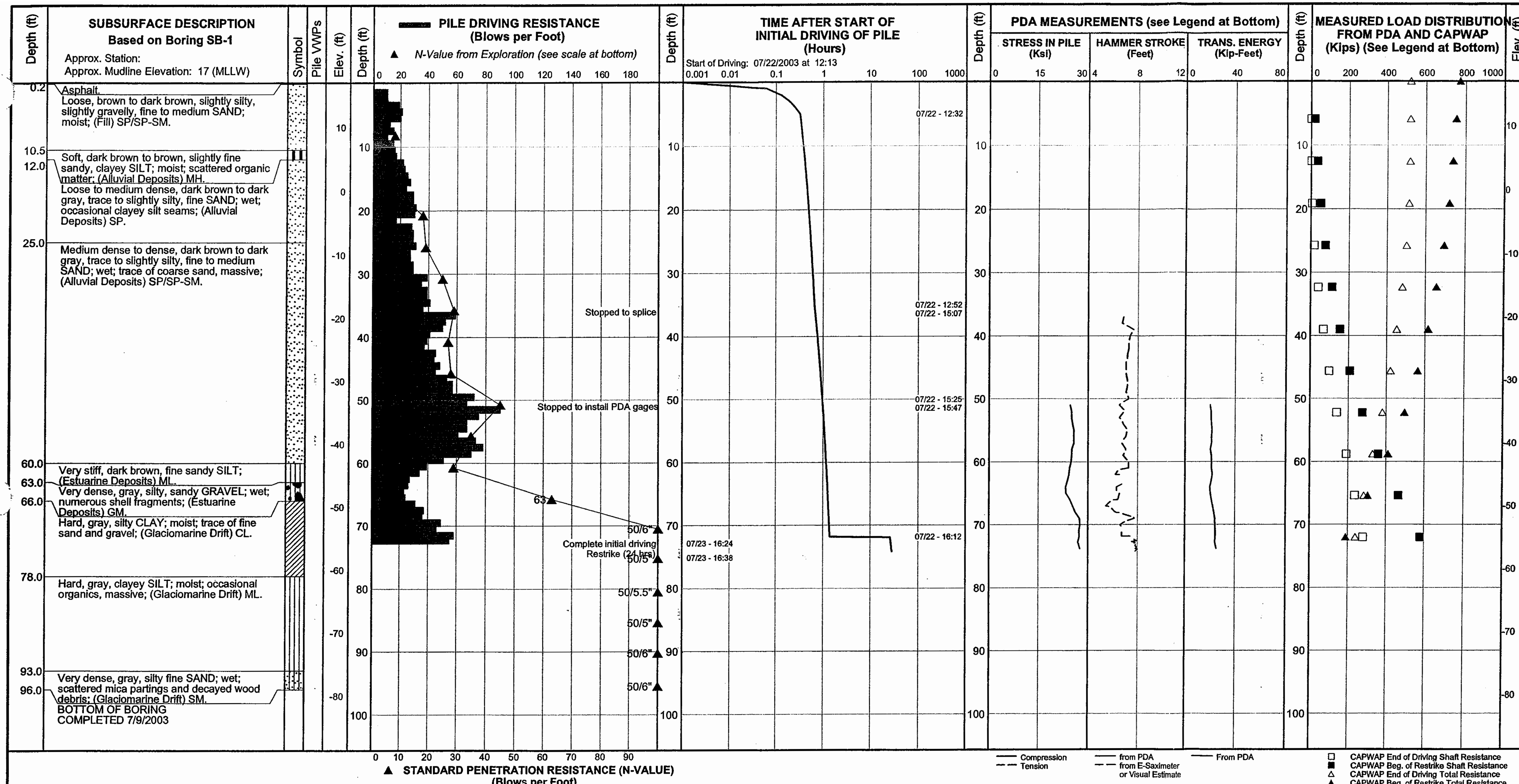
Based on our experience, we anticipate that the subsurface settlements, in close proximity to pile driving, would be increased approximately 50 to 100 percent for pile group installations. Thus, ground settlement within 4 feet of a pile within a pile group could be between 1.5 and 2 inches and will depend on pile location within the group and pile spacing.

Surface Settlement. Figure E-5 presents calculated vertical displacement of a 100-foot-long horizontal casing secured to ground surface, located radially out from the pile. As shown, negligible consistent vertical displacements were observed over most of the casing length. Up to 0.12-inch of settlement of the casing was measured 6 feet from the pile driving. In general, negligible vertical displacements are observed in the casing beyond 12 feet from the pile.

Fluctuations in the plotted data are likely the result of thermal expansion/contraction of the casing, which was exposed to the sun during this period of monitoring.

The magnitude of the settlement at the end of the casing, near the pile, does not reflect the settlement measured in the Sondex casing. One reason for this variation is that the first measurement point in the horizontal casing is 6 feet from the pile, while the Sondex settlement casing is located 4 feet from the pile. In addition, the stiffness of the horizontal plastic casing may have restricted the effectiveness of monitoring the settlement near the end of the casing.

As with the subsurface settlement, we would anticipate increased settlement with placement of pile groups. The extent of the settlement would depend on the locations of the piles.



NOTES:

1. The indicator pile consisted of a 80-foot-long, steel pipe pile, driven closed-end. Two 40-foot-long steel pipe sections of 1/2-inch wall thickness were welded together, with a 2-inch-thick steel plate welded to the tip of the bottom section.
2. The indicator pile was driven with a Berminghammer B4505. The maximum rated energy of this hammer is 75,900 ft/lbs per blow.
3. The test pile was driven by Pile Contractors, Inc. and dynamic testing using a Pile Driving Analyzer (PDA) was performed by Robert Miner Dynamic Testing (RMDT). Subsequent analyses for estimating capacity were performed using the CAPWAP program and are shown on the right-most plot as discrete data points.
4. See the boring log as attached above.
5. Ksi = kips per square inch, Trans. Energy = Maximum energy transmitted by hammer to the top of the pile.

South Park Bridge Project
King County, Washington

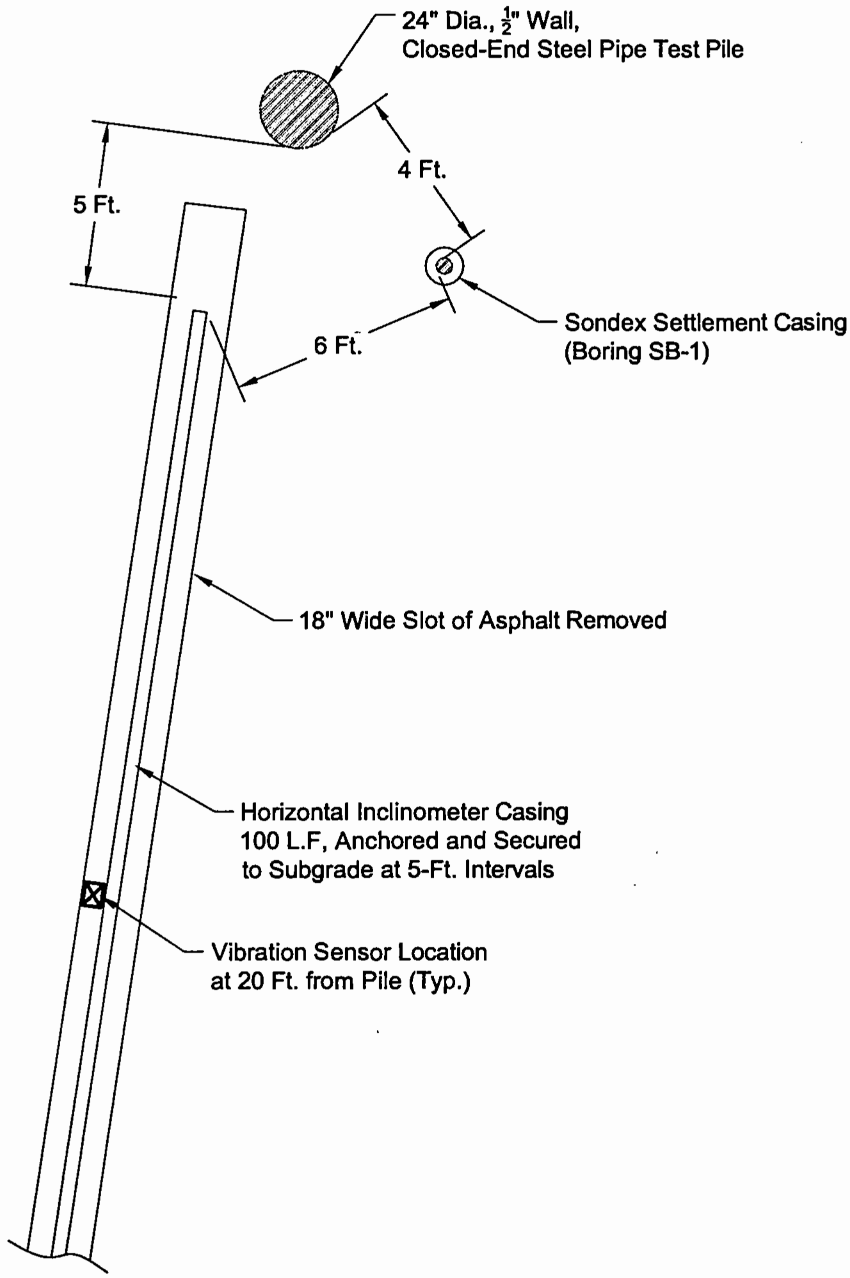
SUMMARY OF TEST PILE TP-1

November 2003 21-1-09584-008

SHANNON & WILSON, INC.
Geotechnical and Environmental Consultants

FIG. E-1

PILE_LOG 21-09317.GPJ GPJ 11/18/03



Not to Scale

South Park Bridge Project
King County, Washington

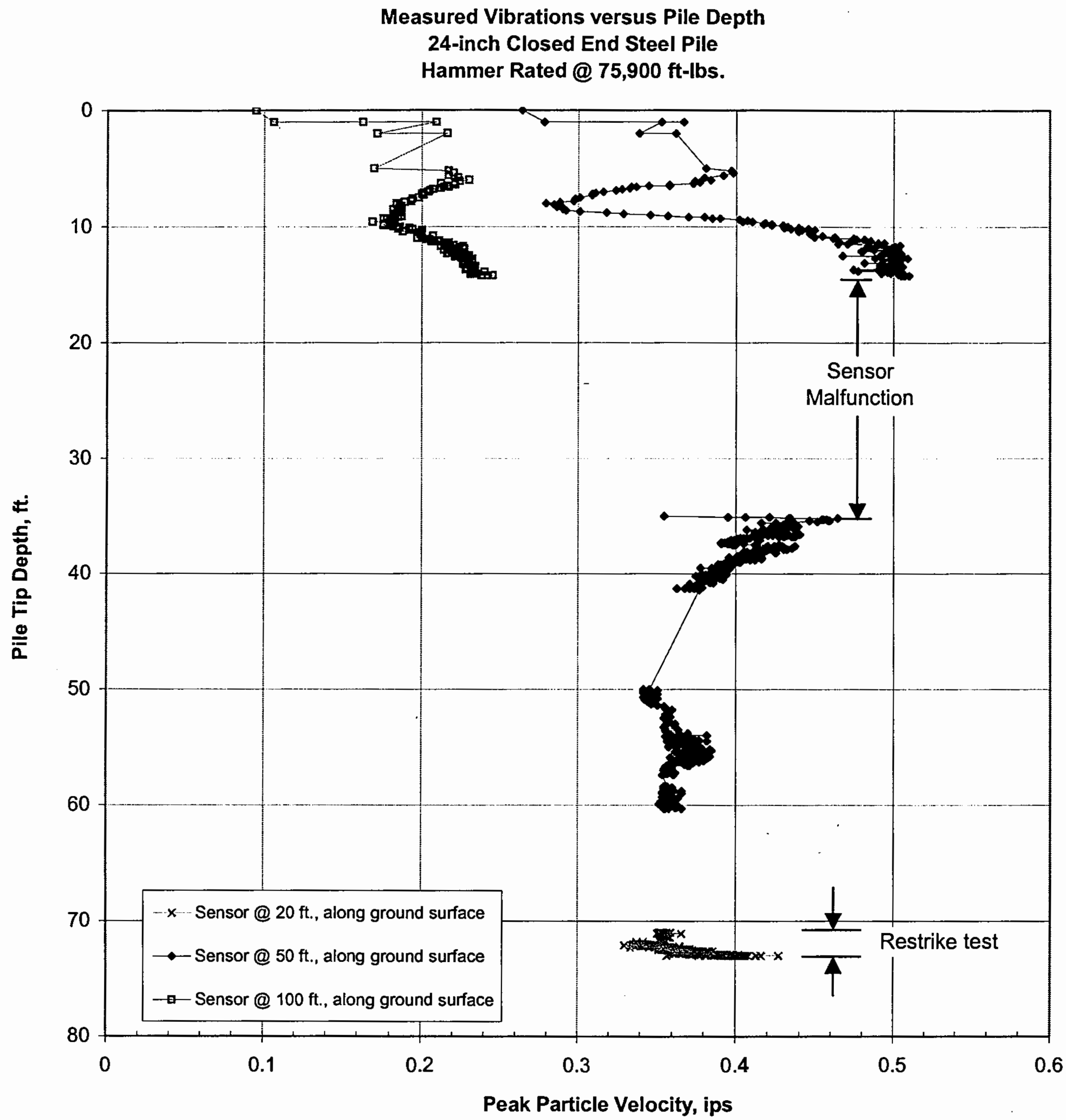
TEST PILE LAYOUT

November 2003 | 21-1-09584-006

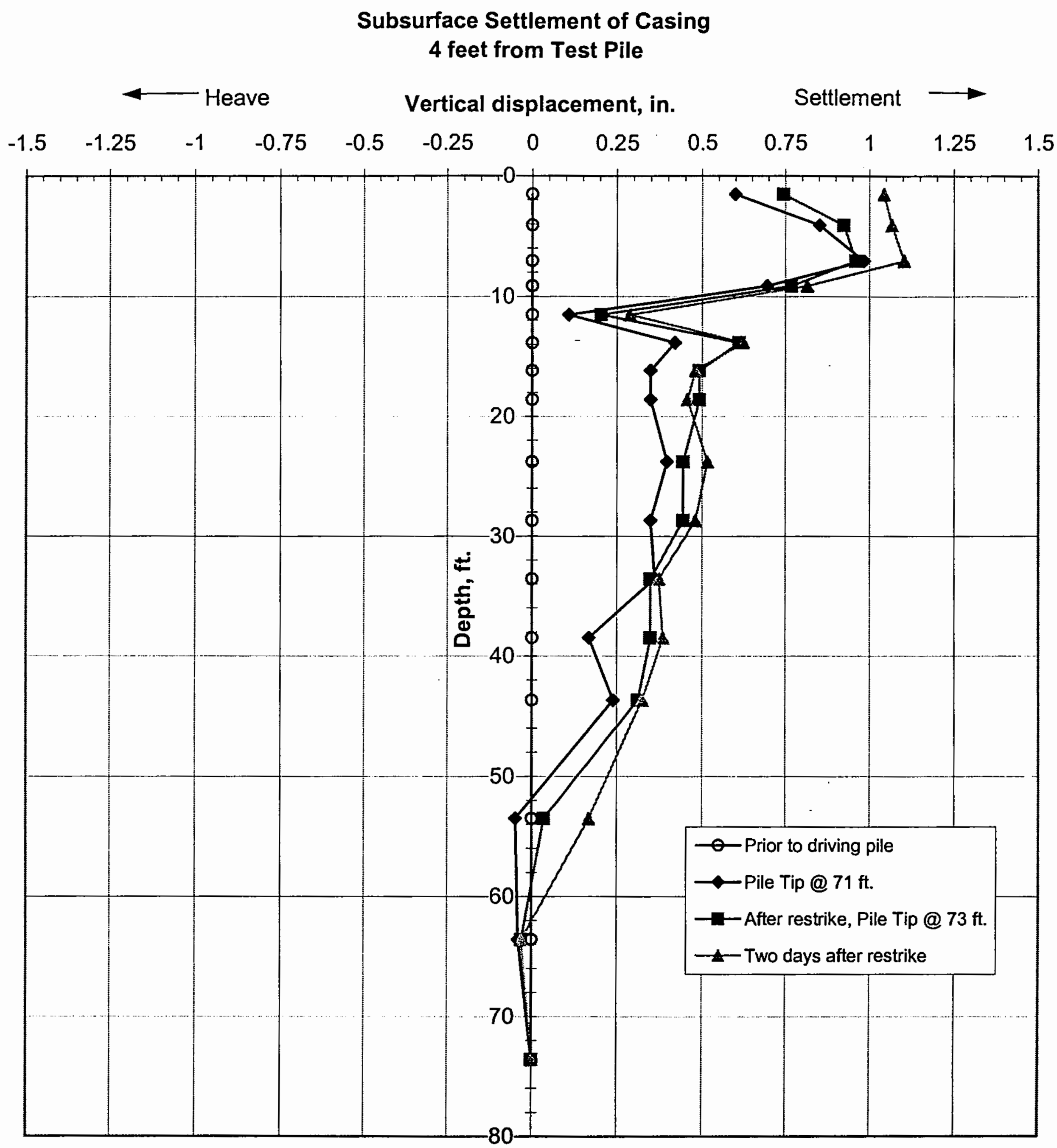
SHANNON & WILSON, INC.
Geotechnical and Environmental Consultants

FIG. E-2

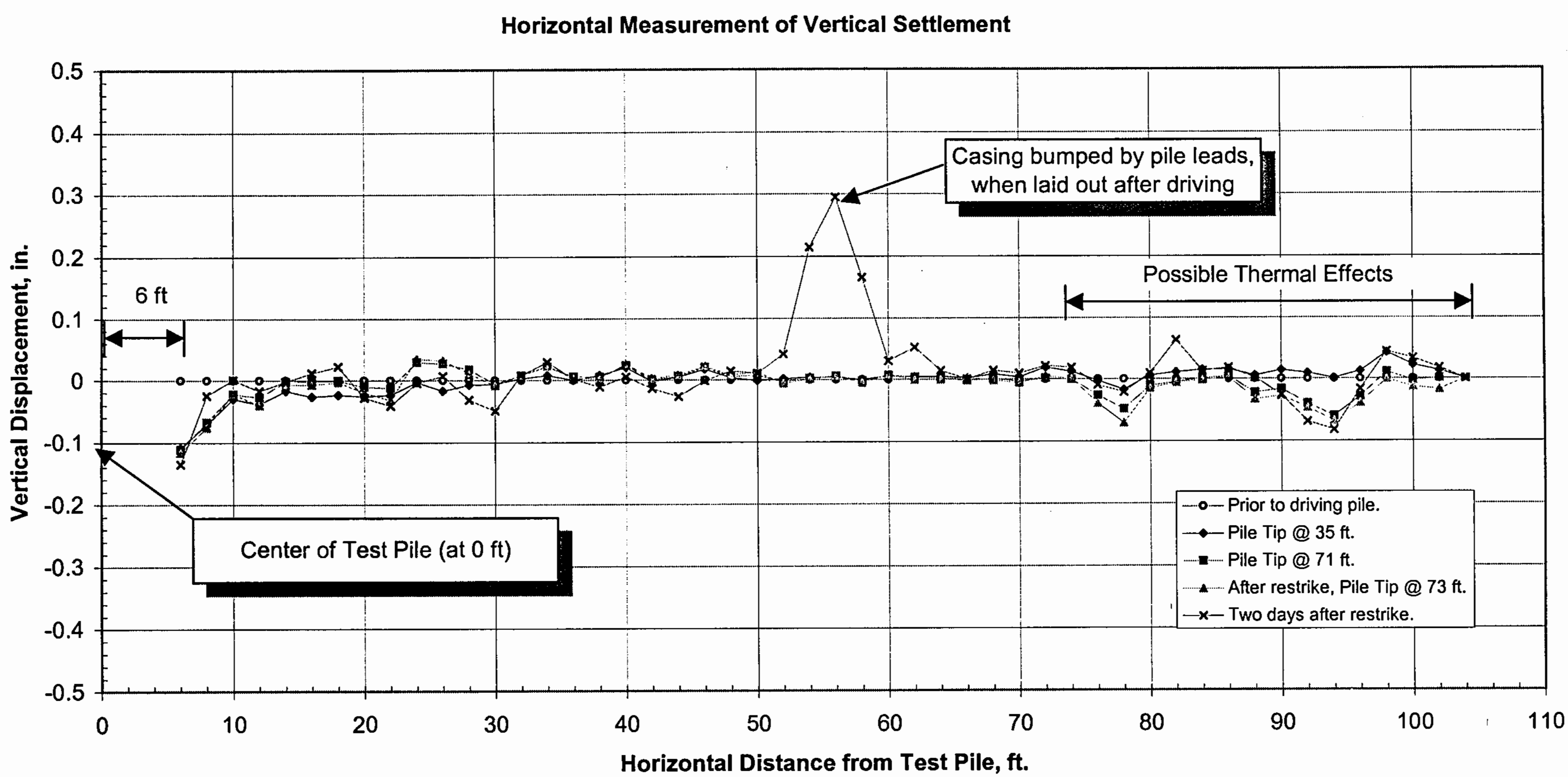
File: I:\Drafting\211\09584-008\21-1-09584-008 fig E-2.dwg Date: 11-12-2003 Author: CNT



South Park Bridge Seattle, Washington	
MEASURED VIBRATIONS VERSUS PILE DEPTH	
November 2003	21-1-09584-006
SHANNON & WILSON, INC. Geotechnical and Environmental Consultants	**FIG. E-3**



South Park Bridge Seattle, Washington	
SUBSURFACE SETTLEMENT ADJACENT TO TEST PILE	
November 2003	21-1-09584-006
SHANNON & WILSON, INC. Geotechnical and Environmental Consultants	**FIG. E-4**



South Park Bridge
Seattle, Washington

HORIZONTAL MEASUREMENT OF VERTICAL SETTLEMENT

November 2003 21-1-09584-006

SHANNON & WILSON, INC.
Geotechnical and Environmental Consultants

FIG. E-5

FIG. E-5

21-1-09584-006

Robert Miner Dynamic Testing, Inc.

Consulting, Dynamic Measurements and Analyses for Deep Foundations

September 27, 2003

Mr. Jim Wu, Ph. D. , P.E.
Shannon and Wilson, Inc.
400 North 34th Street, Suite 100
Seattle WA 98103

Re: Dynamic Pile Measurements and Analyses
PP24"x0.50", Berminghammer B4505, July 22-23, 2003
South Park Bridge Project, Test Pile RMDT Job. No 03F36

Dear Sir:

This report presents results of dynamic measurements and CAPWAP analyses for one Test Pile at the project referenced above. Robert Miner Dynamic Testing, Inc. (RMDT) completed field testing and analysis at the request of Shannon and Wilson, Inc. Field test results include information on hammer transfer energy and pile stresses. CAPWAP analyses results provide information about the soil resistance to axial compressive pile loads. Appendix A contains a description of our methods. Case Method field results and CAPWAP analysis results are in Appendix B and C, respectively.

TEST DETAILS

Test Sequence

RMDT's field testing occurred on July 22, 2003 when the pile was driven to 72 ft penetration, and 24 hours later during a restrike test. For further details on the driving sequence please refer to driving logs or other field observations maintained by Shannon and Wilson, Inc.

Pile

The Test Pile was a 24" OD steel pipe pile with an 0.500" wall thickness. The lower end of the pile was reported to be closed with a 2" thick flat steel plate. During our testing the pile length was 80 ft, and our monitoring sensors were attached to the pile 4 ft below the pile top. RMDT did not observe any mill marking that would indicate the grade of the pile steel.

Hammer

A Berminghammer B4505 single acting diesel hammer drove the Test Pile. The B4505 model is manufactured with a 6.6 kip ram and rated with a maximum stroke of 11.8 ft and an a maximum energy of 77.9 kip-ft.

Instrumentation

Dynamic measurements were made with two strain sensors and two accelerometers bolted to the side of the pile, approximately 4 ft below the pile top. Signals from the sensors were processed and stored by a Pile Driving Analyzer® (PDA). For each hammer blow the PDA displayed the measurements as plots of force and velocity, and computed a variety of results.

Mailing Address: P.O. Box 340, Manchester, WA, 98353, USA **Phone:** 360-871-5480
Location: 2288 Colchester Dr. E., Ste A, Manchester, WA, 98353 **Fax:** 360-871-5483

RMDT's engineer reviewed the measurements and the computed results during and after driving. After the field work ended RMDT completed CAPWAP analyses to evaluate the soil resistance to axial pile movement. Appendix A contains general information on our measurement and analysis methods.

PRESENTATION OF RESULTS

Case Method

In the field, the PDA processed dynamic measurements and computed a variety of results. Table 1 summarizes selected field results for each restrike. The tabulated data includes the approximate penetration resistance (hammer blows per set) as reported to RMDT, computed ram stroke height, STK, energy transferred to the sensor location on the pile, EMX, calculated maximum compressive stress, CSX, and a Case Method estimate of soil resistance. Penetration and penetration resistance data used in this report and in our analyses were provided by Shannon and Wilson, Inc. Figure 1 is a graphical summary of selected results.

Table 1. Summary of Case Method Results

Pile	Test	Blow Count blow/set	Avg. Computed Ram Stroke (STK) ft	Avg. Max. Transfer Energy (EMX) kip-ft	Avg. Max. Compression Stress (CSX) ksi	Case Method Soil Resistance (RX7) kips
TP	End Drive	55/0.8 ft	7.9	24	27.5	490
TP	Restrike	18/ 1 inch	7.7	24	27.0	720

CAPWAP

The CAse Pile Wave Analysis Program (CAPWAP) computes soil resistance forces and their approximate distribution using the force and velocity data recorded in the field during dynamic monitoring. Final CAPWAP results include an evaluation of the soil resistance distribution, pile axial stress as a function of distance below the sensors, soil quake and damping factors, and a simulated static load-set graph. The static load-set graph is based on the CAPWAP calculated static resistance parameters and the elastic compression characteristics of the pile. Table 2 summarizes the CAPWAP results and detailed program output is in Appendix C.

DISCUSSION

Hammer Performance

During final driving (July 22) the average ram stroke and transfer energy were 7.9 ft and 24 kip-ft, respectively. The measured transfer energy may be divided by the hammer's maximum rated energy to compute a Rated Transfer Efficiency of 31 percent. Figure 2 is a summary of Rated Transfer Efficiency for a large number of cases of open end diesel hammers driving steel piles. In that data set the mean Rated Transfer Efficiency is 36.6 percent.

Driving Stresses and Pile Integrity

In routine testing, the PDA uses the average of the signals from the two strain transducers to compute the average maximum axial compression stress at the sensor location, CSX. The PDA also calculates the maximum compressive stress at the sensor location using the largest strain from a single strain transducer, CSI. Table 1 lists CSX values for the end of driving and the start of restrike; on both occasions the average value was less than 28 ksi, and we did not record CSX values greater than 29 ksi at any time.

The PDA computed CSX values apply to the sensor locations, which were approximately 4 ft below the top of the piles. Our CAPWAP analysis computed axial stresses at other locations below the sensor location. The CAPWAP computed stresses were within 1 ksi of the measured stresses and the peak CAPWAP stresses occurred approximately 15 to 35 ft below the pile top. The Case Method CSX values and the CAPWAP computed stresses do not include stresses associated with bending, and do not evaluate local or contact stresses very near the pile top or toe. Comparison of the CSI and CSX value does provide an indication of bending stresses or non-axial stresses at the two sensor locations.

Guidelines for maximum compressive driving stresses are given by the Federal Highway Administration, and may be used as a reference in assessing the stress levels given above. For steel piles, the FHWA recommends that maximum driving stresses be limited to less than 90 percent of the steel yield strength. Assuming that the material strength was at least 35 ksi (ASTM A252 GR 2) the measured CSX stresses were within FHWA recommended limits.

During data acquisition, RMDT evaluated force and velocity records for indications of pile damage below the sensor location. Damage that yields a reduction of axial compressive stiffness during testing would normally be detected provided that it is not too close to the pile toe. We did not observe any evidence of pile damage below the location of our sensor.

Soil Resistance

CAPWAP analysis of data for the end of driving yielded 520 kips of soil resistance, composed of 280 kips of friction and 240 kips of end bearing. We completed two CAPWAP analyses using restrike data. Our analysis for restrike Blow Number 3 yielded 770 kips of resistance composed of 580 kips of friction and 190 kips of end bearing. Our analysis for restrike Blow 11 yielded 700 kips, with 460 kips of friction and 240 kips of end bearing.

In our opinion, the pile displacement, per blow at the start of the restrike, was not sufficient to fully activate the available restrike soil resistance. Thus, the 190 kip computed end bearing for Blow 3 is likely to be a lower bound. It is our opinion that as the restrike progressed, the shaft friction reduced slightly, and the toe displacement became larger such that by restrike Blow 11 the end displacement was sufficient to mobilize an end bearing that was identical to the end bearing from driving. More complete mobilization of the combined end bearing and initial restrike friction would require a hammer impact with larger transfer energy, and equal or greater peak forces. Given the relatively high restrike penetration resistance of 18 blow/inch, it is our opinion that the CAPWAP computed value of 770 kips for the start of restrike is a lower bound value. It is our opinion that for the start of the restrike a more reasonable estimate of the axial compressive ultimate resistance is 820 kips, which value is obtained by combining the early restrike friction with the end bearing from the end of driving.

Table 2. Summary of CAPWAP Results

Pile	Test Type	Computed Soil Resistance, kips		
		Total	Shaft	Toe
TP	End Drive	520	280	240
TP	Restrike, Blow 3	770	580	190
TP	Restrike, Blow 11	700	460	240

Additional Considerations

Various aspects of resistance and loading are usually considered in pile foundation design. In particular, time and pore pressure related increases in shaft friction may occur after testing. In very dense granular soils that dilate during shear, end bearing may decrease with time after driving. Additional resistance and loading aspects include cyclic loading performance, lateral and uplift loading requirements, effective stress changes (due to changes in pore water pressure, excavations, fills or other changes in overburden pressure), settlement from downdrag or underlying weaker layers, the effects of scour or liquefaction on pile capacity, pile group effects, heave of individual piles as a group is completed, strong ground motion, structural design and time dependant changes in pile structural strength. These aspects of foundation design have not been evaluated by RMDT in the interpretation of the dynamic testing results. The foundation designer should determine which, if any of these aspects are applicable to this project, and their impact on the foundation design and construction.

The CAPWAP and Case Method resistance values given in this report are ultimate values for axial compressive resistance at the time of the test, and must be reduced by an appropriate factor of safety to obtain an allowable compressive load.

We appreciate the opportunity to participate on this project. Please do not hesitate to call if you have any questions regarding this report or any aspect of our services on this project.

Very truly yours,

ROBERT F. MINER
STATE OF WASHINGTON
29099
REGISTERED
PROFESSIONAL ENGINEER
EXPIRES 2/27/03

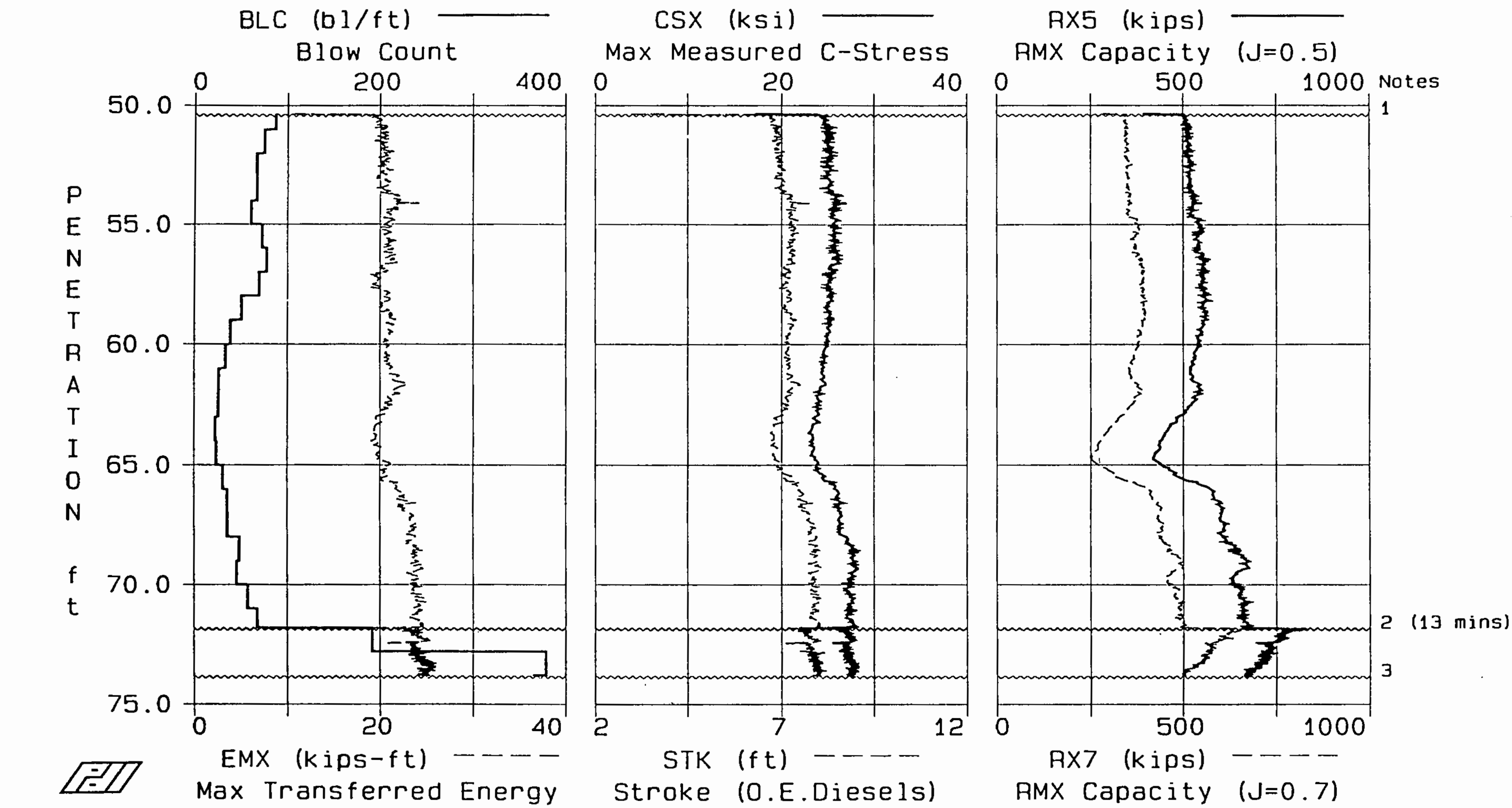
SEPT. 28, 2003

Robert Miner Dynamic Testing, Inc.

Robert F. Miner, P.E.

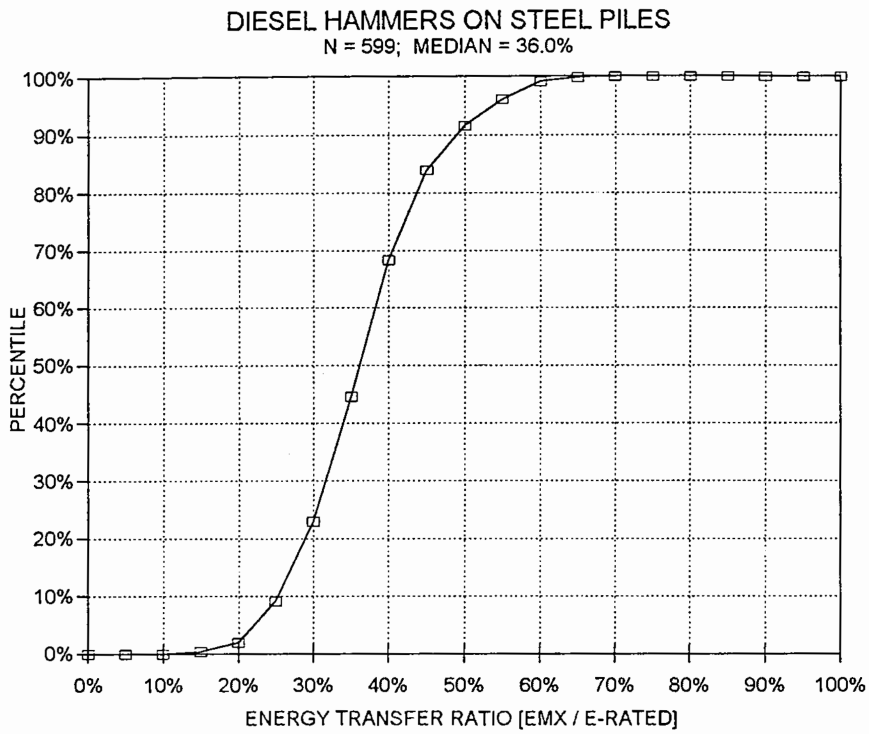
A Miner Dynamic Test 2003-Jul-23

SOUTH PARK BRIDGE, TP, PP24"x0.50", B4505



Notes
1. BEGIN PDA MEASUREMENTS ON TEST PILE NEAR 50 FT, 15:47 22JUL03.
2. STOP NEAR 72FT, 16:12 22JUL03. FRESH-HEAD PILE & RESTRIKE 16:26 23JUL03.
3. END RESTRIKE ON TEST PILE, 16:40 23JUL03.

Figure 1. Summary of Case Method Results, Test Pile, July 22 & 23, 2003.



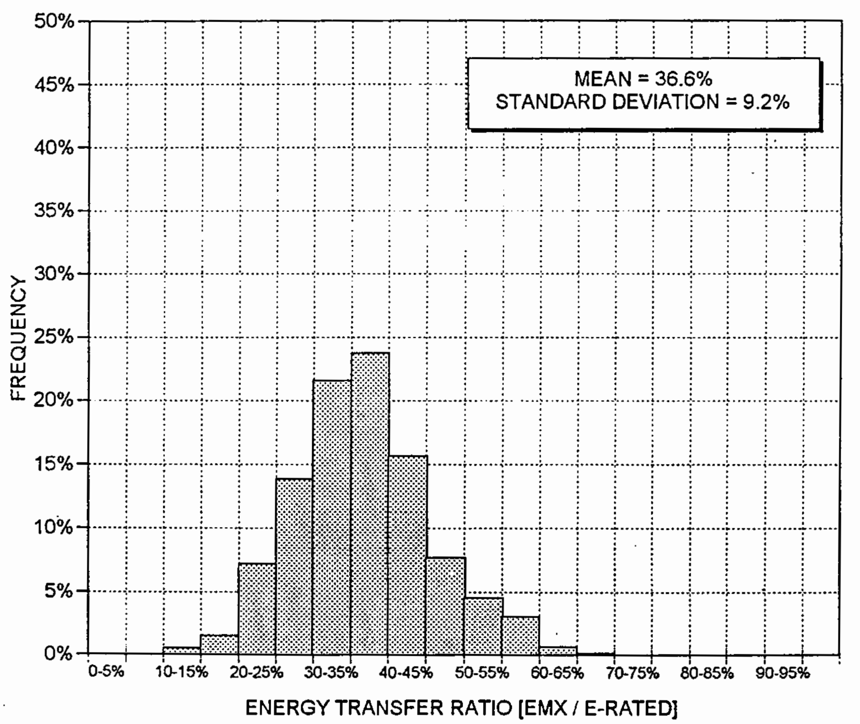


Figure 2. Rated Transfer Efficiency data from many sites with diesel hammers driving steel piles at EOD conditions.

APPENDIX A
AN INTRODUCTION INTO DYNAMIC PILE TESTING METHODS

The following has been written by Goble Rausche Likins and Associates, Inc. and may only be copied with its written permission.

BACKGROUND

Modern procedures of design and construction control require verification of bearing capacity and integrity of deep foundations during preconstruction test programs and also production installation. Dynamic pile testing methods meet this need economically and reliably, and therefore, form an important part of a quality assurance program when deep foundations are executed. Several dynamic pile testing methods exist; they have different benefits and limitations and different requirements for proper execution.

The Case Method of dynamic pile testing, named after the Case Institute of Technology where it was developed between 1964 and 1975, requires that a substantial ram mass (such as that of a pile driving hammer) impacts the pile top such that the pile undergoes at least a small permanent set. The method is therefore also referred to as a "High Strain Method". The Case Method requires dynamic measurements on the pile or shaft under the ram impact and then an evaluation of various quantities based on closed form solutions of the wave equation, a partial differential equation describing the motion of a rod under the effect of an impact. Conveniently, measurements and analyses are done by a single piece of equipment: the Pile Driving Analyzer® (PDA). However, for bearing capacity evaluations an important additional method is CAPWAP® which performs a much more rigorous analysis of the dynamic records than the simpler Case Method.

A related analysis method is the "Wave Equation Analysis" which calculates a relationship between bearing capacity and pile stress and field blow count. The GRLWEAP™ program performs this analysis and provides a complete set of helpful information and input data.

The following description deals primarily with the Case Method or "High Strain Test" Method of pile testing, however, for the sake of completeness, the "Low Strain Test" performed with the Pile Integrity Test™ (PIT), mainly for pile integrity evaluation, will also be described.

RESULTS FROM DYNAMIC TESTING

There are two main objectives of high strain dynamic pile testing:

- *Dynamic Pile Monitoring* and
- *Dynamic Load Testing.*

Dynamic pile monitoring is conducted during the installation of impact driven piles to achieve a safe and economical pile installation. Dynamic load testing, on the other hand, has as its primary goal the assessment of pile bearing capacity. It is applicable to both cast *insitu* piles or drilled shafts and impact driven piles during restrike.

Dynamic Pile Monitoring

During pile installation, the sensors attached to the pile measure pile top force and velocity. A PDA conditions and processes these signals and calculates or evaluates:

- Bearing capacity at the time of testing, including an assessment of shaft resistance development and driving resistance. This information supports formulation of a driving criterion.

- Dynamic pile stresses, axial and averaged over the pile cross section, both tensile and compressive, during pile driving to limit the potential of damage either near the pile top or along its length. Bending stresses can be evaluated at the point of sensor attachment.

- Pile integrity assessment by the PDA is based on the recognition of certain wave reflections from along the pile. If detected early enough, a pile may be saved from complete destruction. On the other hand, once damage is recognized measures can be taken to prevent reoccurrence.

- Hammer performance parameters including the energy transferred to the pile, the hammer speed in blows per minute and the stroke of open ended diesel hammers.

Dynamic Pile Load Testing

Bearing capacity testing of either driven piles or drilled shafts applies the same basic measurement approach of dynamic pile monitoring. However, the test is done independent of the pile installation process and therefore a pile driving hammer or other dynamic loading device may not be available. If a special ram has to be mobilized then its weight should be between 0.8 and 2% of the test load (e.g. between 4 and 10 tons for a 500 ton test load) to assure sufficient soil resistance activation.

For a successful test, it most important that the test is conducted after a <u>sufficient waiting time</u> following pile installation for soil properties approaching their long term condition or concrete to properly set. During testing, PDA results of pile/shaft stresses and transferred energy are used to maintain safe stresses and assure sufficient resistance activation. For safe and sufficient testing of drilled shafts, ram energies are often increased from blow to blow until the test capacity has been activated. On the other hand, restrike tests on driven piles may require a warm hammer so that the very first blow produces a complete resistance activation. Data must be evaluated by CAPWAP for bearing capacity.

After the dynamic load test has been conducted with sufficient energy and safe stresses, the CAPWAP analysis provides the following results:

- <u>Bearing capacity</u> i.e. the mobilized capacity present at the time of testing

- <u>Resistance distribution</u> including shaft resistance and end bearing components

- <u>Stresses in pile or shaft</u> calculated for both the static load application and the dynamic test. These stresses are averages over the cross section and do not include bending effects or nonuniform contact stresses, e.g. when the pile toe is on uneven rock.

- <u>Shaft impedance</u> vs depth; this is an estimate of the shaft shape if it differs substantially from the planned profile

- <u>Dynamic soil parameters</u> for shaft and toe, i.e. damping factors and quakes (related to the dynamic stiffness of the resistance at the pile/soil interface.)

MEASUREMENTS

PDA

The basis for the results calculated by the PDA are pile top strain and acceleration measurements which are converted to force and velocity records, respectively. The PDA conditions, calibrates and displays these signals and immediately computes average pile force and velocity thereby eliminating bending effects. Using closed form Case Method solutions, based on the one-dimensional linear wave equation, the PDA calculates the results described in the analytical solutions section below.

HPA

The ram velocity may be directly obtained using radar technology in the Hammer Performance Analyzer™. For this unit to be applicable, the ram must be visible. The impact velocity results can be automatically processed with a PC or recorded on a strip chart.

Saximeter™

For open end diesel hammers, the time between two impacts indicates the magnitude of the ram fall height or stroke. This information is not only measured and calculated by the PDA but also by the convenient, hand-held Saximeter.

PIT

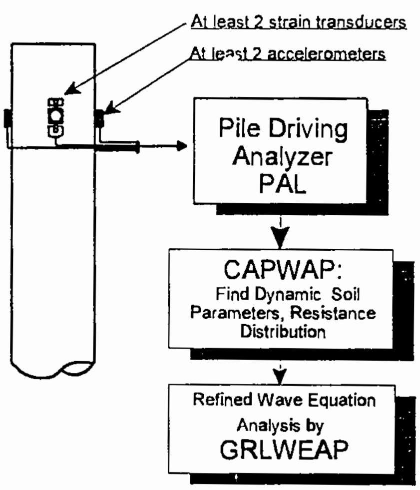
The Pile Integrity Tester™ (PIT) can be used to evaluate defects in concrete piles or shafts which may have occurred during driving or casting. Also timber piles of limited length can be tested in that manner. This so-called "Low Strain Method" or "Pulse-Echo Method" of integrity testing requires only the measurement of acceleration at the pile top. The stress wave producing impact is then generated by a small hand-held hammer and the records interpreted in the time domain. PIT also supports the so-called "Transient Response Method" which requires the additional measurement of the hammer force and an analysis in the frequency domain. This method may also be used to evaluate the unknown length of deep foundations under existing structures.

ANALYTICAL SOLUTIONS
BEARING CAPACITY

Wave Equation

GRL has written the GRLWEAP™ program which calculates a relationship between bearing capacity, pile stress and blow count. This relationship is often called the "bearing graph." Once the blow count is known from pile installation logs, the bearing graph yields the bearing capacity. This approach requires no measurements and therefore can be performed during the design stage of a project, for example for the selection of hammer, cushion and pile size.

After dynamic pile monitoring and/or dynamic load testing has been performed, the "Refined Wave Equation Analysis" or RWEA (see schematic below) is often performed by inputting the PDA and CAPWAP calculated parameters. Then the bearing graph from the RWEA is the basis for a safe and sufficient driving criteria.



Case Method

The Case Method is a closed form solution based on a few simplifying assumptions such as ideal plastic soil behavior and an ideally elastic and uniform pile. Given the measured pile top force F(t) and pile top velocity v(t), the total soil resistance is

$$R(t) = \tfrac{1}{2}\{[F(t) + F(t_2)] + Z[v(t) - v(t_2)]\} \qquad (1)$$

where

t = a point in time after impact
t_2 = time $t + 2L/c$
L = pile length below gages
c = $(E/\rho)^{1/2}$ is the speed of the stress wave
ρ = pile mass density
Z = EA/c is the pile impedance
E = elastic modulus of the pile ($\rho\ c^2$)
A = pile cross sectional area

The total soil resistance consists of a dynamic (R_d) and a static (R_s) component. The static component is therefore

$$R_s(t) = R(t) - R_d(t) \qquad (2)$$

The dynamic component may be computed from a soil damping factor, J, and a pile toe velocity, $v_t(t)$ which is conveniently calculated for the pile toe. Using wave considerations, this approach leads immediately to the dynamic resistance

$$R_d(t) = J[F(t) + Zv(t) - R(t)] \qquad (3)$$

and finally to the static resistance by means of Equation 2.

There are a number of ways in which Eq. 1 through 3 can be evaluated. Most commonly, t_2 is set to that time at which the static resistance becomes maximum. The result is the so-called **RMX** capacity. Damping factors for RMX typically range between 0.5 for coarse grained materials to 1.0 for clays. The **RSP** capacity (this method is most commonly referred to in the literature, yet it is not very frequently used) requires damping factors between 0.1 for sand and 1.0 for clay. Another capacity, **RA2**, determines the capacity at a time when the pile is essentially at rest and thus damping is small; RA2 therefore requires no damping parameter. In any event, the proper Case Method and its associated damping parameter is most conveniently found after a CAPWAP analysis has been performed.

The static resistance calculated by Case Method or CAPWAP is the mobilized resistance at the time of testing. Consideration therefore has to be given to soil setup or relaxation effects and whether or not a sufficient set has been achieved under the test loading that would correspond to a full activation of the ultimate soil resistance.

The PDA also calculates an estimate of shaft resistance as the difference between force and velocity times impedance at the time immediately prior to the return of the stress wave from the pile toe. This shaft resistance is not reduced by damping effects and is therefore called the total shaft resistance **SFT**. A correction for damping effects produces the static shaft resistance estimate, **SFR**.

The Case Method solution is simple enough to be evaluated "in real time," i.e. between hammer blows, using the PDA. It is therefore possible to calculate all relevant results for all hammer blows and plot these results as a function of depth or blow number. This is done in the PDAPLOT program.

CAPWAP

The CAse Pile Wave Analysis Program combines the wave equation pile and soil model with the Case Method measurements. Thus, the solution includes not only the total and static bearing capacity values but also the shaft resistance, end bearing, damping factors and soil stiffnesses. The method iteratively calculates a number of unknowns by signal matching. While it is necessary to make hammer performance assumptions for a GRLWEAP analysis, the CAPWAP program works with the pile top measurements. Furthermore, while GRLWEAP and Case Method require certain assumptions regarding the soil behavior, CAPWAP calculates these soil parameters.

STRESSES

During pile monitoring, it is important that compressive stress maxima at pile top and toe and tensile stress maxima somewhere along the pile be calculated for each hammer blow.

At the pile top (location of sensors) both the maximum compression stress, **CSX**, and the maximum stress from individual strain transducers, **CSI**, are directly obtained from the measurements. Note that CSI is greater than or equal to CSX and the difference between CSI and CSX is a measure of bending in the plane of the strain transducers. Note also that all stresses calculated for locations below the sensors are averaged over the pile cross section and therefore do not include components from either bending or eccentric soil resistance effects.

The PDA calculates the compressive stress at the pile bottom, **CSB**, assuming (a) a uniform pile and (b) that the pile toe force is the maximum value of the total resistance R(t) minus the total shaft resistance, SFT. Again, for this stress estimation uniform resistance force are assumed (e.g. not a sloping rock.)

For concrete piles, the maximum tension stress, **TSX**, is also of great importance. It occurs at some point below the pile top. The maximum tension stress can be computed from the pile top measurements by finding the maximum tension wave (either traveling upward, W_U, or downward, W_d) and reducing it by the minimum compressive wave traveling in opposite direction.

$$W_u = \tfrac{1}{2}[F(t) - Zv(t)] \tag{4}$$

$$W_d = \tfrac{1}{2}[F(t) + Zv(t)] \tag{5}$$

CAPWAP also calculates tensile and compressive stresses along the pile and, in general, more accurately than the PDA. In fact, for non-uniform piles or piles with joints, cracks or other discontinuities, the closed form solutions from the PDA may be in error.

PILE INTEGRITY

High Strain Tests (PDA)

Stress waves in a pile are reflected wherever the pile impedance, $Z = EA/c = \rho cA = A\sqrt{E\,\rho}$, changes. Therefore, the pile impedance is a measure of the quality of the pile material (E, ρ, c) and the size of its cross section (A). The reflected waves arrive at the pile top at a time which is greater the farther away from the pile top the reflection occurs. The magnitude of the change of the upward traveling wave (calculated from the measured force and velocity, Eq. 4) indicates the extent of the cross sectional change. Thus, with β_i (**BTA**) being a relative integrity factor which is unity for no impedance change and zero for the pile end, the following is calculated by the PDA.

$$\beta_i = (1 - \alpha_i)/(1 + \alpha_i) \qquad (6)$$

with

$$\alpha_i = ½(W_{UR} - W_{UD})/(W_{Di} - W_{UR}) \qquad (7)$$

where

W_{UR} is the upward traveling wave at the onset of the reflected wave. It is caused by resistance.

W_{UD} is the upwards traveling wave due to the damage reflection.

W_{Di} is the maximum downward traveling wave due to impact.

It can be shown that this formulation is quite accurate as long as individual reflections from different pile impedance changes have no overlapping effects on the stress wave reflections.

Without rigorous derivation, it has been proposed to consider as slight damage when β is above 0.8 and a serious damage when β is less than 0.6.

Low Strain Tests (PIT)

The pile top is struck with a held hand hammer and the resulting pile top velocity is measured, displayed and interpreted for signs of wave reflections. In general, a comparison of the reflected acceleration leads to a relative measure of extent of damage, again the location of the problem is indicated by the arrival time of the reflection. PIT records can also be interpreted by the β-Method. However, low strain tests do not activate much resistance which simplifies Eq. 7 since W_{UR} is then equal to zero.

For drilled shafts and PIT records that clearly show a toe reflection, an approximate shaft profile can be calculated from low strain records using the PITSTOP program's PROFILE routine.

HAMMER PERFORMANCE

The PDA calculates the energy transferred to the pile top from:

$$E(t) = {}_{o}\int^{t} F(t)v(t)\,dt \qquad (8a)$$

The maximum of the E(t) curve is the most important information for an overall evaluation of the performance of a hammer and driving system. This **EMX** value allows for a classification of the hammer's performance when presented as the rated transfer efficiency, also called energy transfer ratio (**ETR**) or global efficiency

$$e_T = EMX/E_R \qquad (8b)$$

where

E_R is the manufacturer's rated energy value.

Both Saximeter and PDA calculate the stroke (**STK**) of an open end diesel hammer using

$$STK = (g/8)\,T_B^2 - h_L \qquad (9)$$

where

g is the earth's gravitational acceleration,
T_B is the time between two hammer blows,
h_L is a stroke loss value due to gas compression and time losses during impact (usually 0.3 ft or 0.1 m).

DETERMINATION OF WAVE SPEED

An important facet of dynamic pile testing is an assessment of pile material properties. Since in general force is determined from strain by multiplication with elastic modulus, E, and cross sectional area, A, the dynamic elastic modulus has to be determined for pile materials other than steel. In general, the records measured by the PDA clearly indicate a pile toe reflection as long as pile penetration per blow is greater than 1 mm or .04 inches. The time between the onset of the force and velocity records at impact and the onset of the reflection from the toe (usually apparent by a local maximum of the wave up curve) is the so-called wave travel time, T. Dividing 2L (L is here the length of the pile below sensors) by T leads to the stress wave speed in the pile:

$$c = 2L/T \qquad (10)$$

The elastic modulus of the pile material is related to the wave speed according to the linear elastic wave equation theory by

$$E = c^2\rho \tag{11}$$

Since the mass density of the pile material, ρ, is usually well known (an exception is timber for which samples should be weighed), the elastic modulus is easily found from the wave speed. Note, however, that this is a dynamic modulus which is generally higher than the static one and that the wave speed depends to some degree on the strain level of the stress wave. For example, experience shows that the wave speed from PIT is roughly 5% higher than the wave speed observed during a high strain test.

Other Notes:

- If the pile material is nonuniform then the wave speed c, according to Eq. 10, is an average wave speed and does not necessarily reflect the pile material properties of the location where the strain sensors are attached to the pile top. For example, pile driving often causes fine tension cracks some distance below the top of concrete piles. Then the average c is slower than that at the pile top. It is therefore recommended to determine E in the beginning of pile driving and not adjust it when the average c changes.

- If the pile has such a high resistance that there is no clear indication of a toe reflection then the wave speed of the pile material must be determined either by assumption or by taking a sample of the concrete and measuring its wave speed in a simple free column test. Another possibility is to use the proportionality relationship, discussed under "DATA QUALITY CHECKS" to find c as the ratio between the measured velocity and measured strain.

DATA QUALITY CHECKS

Quality data is the first and foremost requirement for accurate dynamic testing results. It is therefore important that the measurement engineer performing PDA or PIT tests has the experience necessary to recognize measurement problems and take appropriate corrective action should problems develop. Fortunately, dynamic pile testing allows for certain data quality checks because two independent measurements are taken that have to conform to certain relationships.

Proportionality

As long as there is only a wave traveling in one direction, as is the case during impact when only a downward traveling wave exists in the pile, force and velocity measured at the pile top are proportional

$$F = v\,Z = v\,(EA/c) \tag{12a}$$

This relationship can also be expressed in terms of stress

$$\sigma = v\,(E/c) \tag{12b}$$

or strain

$$\epsilon = v / c \tag{12c}$$

This means that the early portion of strain times wave speed must be equal to the velocity unless the proportionality is affected by high friction near the pile top or by a pile cross sectional change not far below the sensors. Checking the proportionality is an excellent means of assuring meaningful measurements.

Measurements are always taken at opposite sides of the pile as a means of calculating the average force and velocity in the pile. The velocity on the two sides of the pile is very similar even when high bending exists. Thus, an independent check of the velocity measurements is easy and simple.

Strain measurements may differ greatly between the two sides of the pile when bending exists. It is even possible that tension is measured on one side while very high compression exists on the other side of the pile. In extreme cases, bending might be so high that it leads to a nonlinear stress distribution. The averaging of the two strain signals does then not lead to the average pile force and proportionality will not be achieved.

When testing drilled shafts, measurements of strain may also be affected by local concrete quality variations. It is then often necessary to use four strain transducers spaced at 90 degrees around the pile for an improved strain data quality. The use of four transducers is also recommended for large pile diameters, particularly when it is difficult to mount the sensors at least two pile widths or diameters below the pile top.

LIMITATIONS, ADDITIONAL CONSIDERATIONS

Mobilization of capacity

Estimates of pile capacity from dynamic testing indicate the **mobilized pile capacity at the time of testing**. At very high blow counts (low set per blow), dynamic test methods tend to produce lower bound capacity estimates as not all resistance (particularly at and near the toe) is fully activated.

Time dependent soil resistance effects

Static pile capacity from dynamic method calculations provide an estimate of the axial pile capacity. Increases and decreases in the pile capacity with time typically occur (soil setup/relaxation). Therefore, **<u>restrike testing</u> usually yields a better indication of long term pile capacity than a test at the end of pile driving**. Often a wait period of one or two days between end of driving and restrike is satisfactory for a realistic prediction of pile capacity but this waiting time depends, among other factors, on the permeability of the soil.

(A) Soil setup

Because excess positive pore pressures often develop during pile driving in fine grained soil (clays, silts or even fine sands), the capacity of a pile at the time of driving may often be less than the long term pile capacity. These pore pressures reduce the effective stress acting on the pile thereby reducing the soil resistance to pile penetration, and thus the pile capacity at the time of driving. As these pore pressures dissipate, the soil resistance acting on the pile increases as does the axial pile capacity. This phenomena is routinely called soil setup or soil freeze.

(B) Relaxation

Relaxation (capacity reduction with time) has been observed for piles driven into weathered shale, and may take several days to fully develop. Pile capacity estimates based upon initial driving or short term restrike tests can significantly overpredict long term pile capacity. Therefore, piles driven into shale should be tested after a minimum one week wait either statically or dynamically (with particular emphasis than on the first few blows). Relaxation has also been observed for displacement piles driven into dense saturated silts or fine sands due to a negative pore pressure effect at the pile toe. Again, restrike tests should be used, with great emphasis on early blows.

Capacity results for open pile profiles

Larger diameter open ended pipe piles (or H-piles which do not bear on rock) may behave differently under dynamic and static loading conditions. Under dynamic loads the soil inside the pile or between its flanges may slip and produce internal friction while under static loads the plug may move with the pile, thereby creating end bearing over the full pile cross section. As a result both friction and end bearing components may be different under static and dynamic conditions.

CAPWAP Analysis Results

A portion of the soil resistance calculated on an individual soil segment in a CAPWAP analysis can usually be shifted up or down the shaft one soil segment without significantly altering the match quality. Therefore, use of the CAPWAP resistance distribution for uplift, downdrag, scour, or other geotechnical considerations should be made with an understanding of these analysis limitations.

Stresses

PDA and CAPWAP calculated stresses are average values over the cross section. Additional allowance has to be made for bending or non-uniform contact stresses. To prevent damage it is therefore important to maintain good hammer-pile alignment and to protect the pile toes using appropriate devices or an increased cross sectional area.

In the United States is has become generally acceptable to limit the dynamic installation stresses of driven piles to the following levels:

- 90% of yield strength for steel piles
- 85% of the concrete compressive strength - after subtraction of the effective prestress - for concrete piles in compression
- 100% of effective prestress plus ½ of the concrete's tension strength for prestressed piles in tension

70% of the reinforcement strength for regularly reinforced concrete piles in tension

300% of the static design allowable stress for timber

Note that the dynamic stresses may either be directly measured at the pile top by the PDA or calculated by the PDA for other locations along the pile based on the pile top measurements.

Additional design considerations

Numerous factors have to be considered in pile foundation design. Some of these considerations include

- additional pile loading from downdrag or negative skin friction,
- lateral and uplift loading requirements
- effective stress changes (due to changes in water table, excavations, fills or other changes in overburden),
- long term settlements in general and settlement from underlying weaker layers and/or pile group effects,

These factors have not been evaluated by GRL and have not been considered in the interpretation of the dynamic testing results. The foundation designer should determine if these or any other considerations are applicable to this project and the foundation design.

Wave equation analysis results

The results calculated by the wave equation analysis program depend on a variety of hammer, pile and soil input parameters. Although attempts have been made to base the analysis on the best available information, actual field conditions may vary and therefore stresses and blow counts may differ from the predictions reported. Capacity predictions derived from wave equation analyses should use restrike information. However, because of the uncertainties associated with restrike blow counts and restrike hammer energies, correlations of such results with static test capacities with have often displayed considerable scatter.

As for PDA and CAPWAP, the theory on which GRLWEAP is based is the one-dimensional wave equation. For that reason, stress predictions by the wave equation analysis can only be averages over the pile cross section. Thus, bending stresses or stress concentrations due to non-uniform impact or uneven soil or rock resistance are not considered in these results. Stress maxima calculated by the wave equation are usually subjected to the same limits as those measured directly or calculated from measurements by the PDA.

Appendix B

Summary of Field Results from the Pile Driving Analyzer

Pile: TP
Info: PP24"x0.50"
AR: 36.9 in^2
: 76.0 ft

Proj: SOUTH PARK BRIDGE
SP: 0.493 k/ft^3
WS: 16800 ft/s
EM: 30004 KSI

CSX: Max Measured C-Stress
CSI: Max F1 or F2 C-Stress
EMX: Max Transferred Energy
STK: Stroke (O.E.Diesels)
RP7: RSP Capacity (J=0.7)
BPM: Blows Per Minute
RX5: RMX Capacity (J=0.5)
RX7: RMX Capacity (J=0.7)
RX9: RMX Capacity (J=0.9)

BL# end	bl/ft	depth ft	TY	CSX ksi	CSI ksi	EMX K-ft	STK ft	RP7 kips	BPM bl/min	RX5 kips	RX7 kips	RX9 kips
1	87	50.35	AV	12.03	12.60	4.9	0.00	245	0.0	312	245	203
58	87	51.00	AV	24.72	27.83	19.8	6.77	344	45.2	508	344	272
			SD	0.54	0.79	0.9	0.17	7	0.6	10	7	7
			MX	25.63	28.88	21.4	7.09	360	47.8	524	360	286
133	75	52.00	AV	25.11	28.71	20.5	6.92	347	44.7	514	347	265
			SD	0.32	0.38	0.6	0.10	4	0.3	5	4	8
			MX	25.79	29.37	21.8	7.16	356	45.4	526	356	284
200	67	53.00	AV	25.08	29.19	20.5	6.96	351	44.6	517	351	264
			SD	0.34	0.43	0.5	0.10	6	0.3	6	6	12
			MX	25.90	30.08	21.7	7.19	364	45.3	532	364	291
267	67	54.00	AV	25.40	29.64	20.9	7.07	353	44.2	523	353	265
			SD	0.43	0.42	0.7	0.14	4	0.4	7	4	14
			MX	26.28	30.84	22.4	7.40	361	45.1	536	361	308
328	61	55.00	AV	25.77	29.99	21.4	7.29	358	43.6	530	358	293
			SD	0.38	0.58	0.8	0.13	11	0.4	11	11	11
			MX	27.39	32.92	25.3	7.96	387	44.5	554	387	321
401	73	56.00	AV	25.74	29.87	20.9	7.30	373	43.6	543	373	307
			SD	0.26	0.50	0.5	0.10	9	0.3	9	9	13
			MX	26.23	30.94	22.0	7.51	392	44.3	563	392	336
479	78	57.00	AV	25.76	29.50	21.0	7.23	382	43.8	551	382	305
			SD	0.38	0.39	0.6	0.11	11	0.3	9	11	7
			MX	26.53	30.62	22.4	7.47	403	44.4	573	403	324
549	70	58.00	AV	25.18	28.80	20.0	7.14	391	44.0	553	391	304
			SD	0.30	0.47	0.6	0.11	6	0.3	7	6	15
			MX	25.98	29.91	21.9	7.40	406	45.1	575	406	325
600	51	59.00	AV	25.14	28.54	20.8	7.19	395	43.9	558	395	268
			SD	0.30	0.48	0.6	0.12	6	0.4	8	6	12
			MX	25.98	29.72	22.6	7.58	408	44.6	573	408	300
639	39	60.00	AV	25.01	28.90	21.0	7.21	383	43.8	547	383	256
			SD	0.26	0.37	0.4	0.09	6	0.3	7	6	7
			MX	25.71	29.56	22.0	7.44	395	44.5	563	395	274
73	34	61.00	AV	24.57	29.13	20.9	7.17	366	43.9	530	366	253
			SD	0.23	0.33	0.4	0.08	8	0.2	7	8	6
			MX	24.98	29.70	21.5	7.29	382	44.5	547	382	263

Info: PP24"x0.50"

# nd	bl/ft	depth ft	TY	CSX ksi	CSI ksi	EMX K-ft	STK ft	RP7 kips	BPM bl/min	RX5 kips	RX7 kips	RX9 kips
700	27	62.00	AV	24.38	27.85	21.7	7.25	367	43.7	531	367	252
			SD	0.31	0.49	0.7	0.13	13	0.4	11	13	7
			MX	25.09	28.86	23.6	7.62	394	44.1	558	394	266
726	26	63.00	AV	23.94	27.54	20.8	7.11	359	44.1	519	359	239
			SD	0.26	0.45	0.7	0.13	16	0.4	17	16	8
			MX	24.60	28.29	22.3	7.40	391	44.7	554	391	256
749	23	64.00	AV	23.41	27.17	19.6	6.83	301	45.0	461	301	230
			SD	0.34	0.40	0.4	0.11	16	0.3	15	16	9
			MX	24.28	28.18	20.7	7.12	328	45.7	487	328	245
773	24	65.00	AV	23.48	27.02	20.0	6.84	263	45.0	430	263	222
			SD	0.29	0.34	0.6	0.09	8	0.3	8	8	7
			MX	24.03	27.61	21.3	6.99	273	45.6	439	273	240
804	31	66.00	AV	24.67	27.24	20.9	7.17	333	44.0	501	333	229
			SD	0.76	0.49	0.7	0.20	40	0.6	41	40	12
			MX	25.93	28.07	22.4	7.54	406	44.9	572	406	253
877	36	68.00	AV	26.21	28.19	23.0	7.68	431	42.5	599	431	293
			SD	0.35	0.69	0.8	0.18	12	0.5	12	12	19
			MX	26.93	29.94	24.3	7.96	451	43.7	620	451	330
26	49	69.00	AV	27.49	30.44	23.7	7.89	463	42.0	638	463	314
			SD	0.42	0.50	0.6	0.14	19	0.4	20	19	14
			MX	28.23	31.38	24.8	8.12	507	43.1	683	507	347
972	46	70.00	AV	27.68	30.82	23.9	7.89	474	42.0	650	474	321
			SD	0.36	0.93	0.5	0.13	17	0.3	17	17	15
			MX	28.29	32.38	24.9	8.12	506	42.6	682	506	342
1030	58	71.00	AV	27.33	29.49	24.1	7.89	484	42.0	657	484	324
			SD	0.36	0.78	0.6	0.12	10	0.3	10	10	13
			MX	28.13	31.27	25.9	8.20	514	42.7	690	514	345
1085	68	71.80	AV	27.45	30.78	24.1	7.91	494	41.9	664	494	343
			SD	0.35	0.62	0.6	0.13	8	0.3	9	8	16
DRIVE			MX	28.23	31.68	25.4	8.20	513	42.8	688	513	377
RESTRIKE												
1277	192	72.80	AV	26.97	36.87	23.7	7.73	604	42.4	754	604	454
			SD	0.81	0.98	1.2	0.20	34	0.5	29	34	40
			MX	28.02	38.64	25.8	8.12	720	45.0	845	720	596
1355	379	73.01	AV	27.17	37.23	24.0	7.88	565	42.0	723	565	407
			SD	0.37	0.67	0.8	0.15	11	0.4	14	11	9
			MX	28.02	38.97	26.1	8.29	584	43.1	749	584	423

BL# nd	bl/ft	depth ft	TY	CSX ksi	CSI ksi	EMX K-ft	STK ft	RP3 kips	BPM bl/min	RX5 kips	RX7 kips	RX9 kips
59	379	73.02	AV	27.34	36.80	24.3	7.90	881	42.0	723	565	407
			SD	0.37	0.46	0.8	0.15	15	0.4	12	9	6
			MX	27.80	37.18	25.3	8.08	901	42.4	739	576	414

Pile: TP Proj: SOUTH PARK BRIDGE Pg3
Info: PP24"x0.50"

# / BL#	bl/ft	depth ft	TY	CSX ksi	CSI ksi	EMX K-ft	STK ft	RP3 / RP7 kips	BPM bl/min	RX5 kips	RX7 kips	RX9 kips
ıd / end	bl/ft	ft		ksi	ksi	K-ft	ft	kips	bl/min	kips	kips	kips
1656	379	73.80	AV	27.62	35.52	24.9	7.95	532	41.8	700	532	389
			SD	0.40	0.92	0.8	0.13	23	0.3	20	23	11
			MX	28.75	37.66	26.9	8.37	580	42.7	744	580	420
1667	366	73.83	AV	27.44	34.51	24.7	7.94	504	41.8	675	504	387
			SD	0.40	0.44	0.7	0.14	6	0.4	9	6	11
			MX	27.96	35.09	25.6	8.12	513	42.5	684	513	402

BL#	COMMENTS
1	BEGIN PDA MEASUREMENTS ON TEST PILE NEAR 50 FT, 15:47 22JUL03.
1085	STOP NEAR 72FT, 16:12 22JUL03. FRESH-HEAD PILE & RESTRIKE 16:26 23JUL03.
1341	JC = 0.30
1345	JC = 0.20
1349	JC = 0.50
1667	END RESTRIKE ON TEST PILE, 16:40 23JUL03.

DRIVE TIME SUMMARY (2003-Jul-23 : TPTOT.MDF) DRIVE WAIT
----- minutes ----

BN 1 -> 1085, START 15:47:57 -> 16:12:51 STOP, 24.90 DRIVE JULY 22
(24HR + 13.48)
N 1086 -> 1667, START 16:26:20 -> 16:40:09 STOP, 13.82 RESTRIKE JULY 23

Total Elapsed time 52.20 minutes Total Time 38.72 minutes 13.48
+24 hours
≈ 25 hours.

Appendix C

Results of CAPWAP Analysis

Robert Miner Dynamic Testing, Inc. 31-Jul-2003

SOUTH PARK BRIDGE, Project: 03F36
Pile: TPEND OF DRIVE Blow: 1079 Data: PP24"x0.50", B4505
Collected: 03-07-22 Operator: RMDT--B. Miner CAPWAP(R) Ver. 1997-1

CAPWAP FINAL RESULTS

Total CAPWAP Capacity: 520.0; along Shaft 280.0; at Toe 240.0 kips

Soil Sgmnt No.	Dist. Below Gages ft	Depth Below Grade ft	Ru kips	Force in Pile at Ru kips	Sum of Ru kips	Unit Resist. w. Respect to Depth kips/ft	Unit Resist. w. Respect to Area kips/f2	Smith Damping Factor s/ft	Quake inch
				520.0					
1	9.9	5.9	.7	519.3	.7	.11	.02	.151	.100
2	16.5	12.5	1.2	518.1	1.9	.18	.03	.151	.100
3	23.1	19.1	3.8	514.3	5.7	.58	.09	.151	.100
4	29.7	25.7	11.5	502.8	17.2	1.73	.28	.151	.100
5	36.3	32.3	22.4	480.5	39.6	3.39	.54	.151	.100
6	43.0	39.0	28.0	452.5	67.5	4.23	.67	.151	.100
7	49.6	45.6	31.3	421.2	98.8	4.74	.75	.151	.100
8	56.2	52.2	40.7	380.5	139.6	6.16	.98	.151	.100
9	62.8	58.8	50.9	329.6	190.5	7.70	1.23	.151	.100
10	69.4	65.4	45.3	284.3	235.7	6.85	1.09	.151	.100
11	76.0	72.0	44.2	240.0	280.0	6.69	1.07	.151	.100
Average Skin Values			25.5			3.89	.61	.151	.100
	Toe		240.0				76.45	.030	.380

Soil Model Parameters/Extensions		Skin	Toe
Case Damping Factor		.642	.108
Unloading Quake	(% of loading quake)	20	100
Reloading Level	(% of Ru)	100	100
Resistance Gap (included in Toe Quake)	(inch)		.100
Soil Plug Weight	(kips)		.30
Soil Support Dashpot		1.000	.000
Soil Support Mass	(kips)	4.30	.00

Robert Miner Dynamic Testing, Inc. 31-Jul-2003

SOUTH PARK BRIDGE, Project: 03F36
Pile: TPEND OF DRIVE Blow: 1079 Data: PP24"x0.50", B4505
Collected: 03-07-22 Operator: RMDT--B. Miner CAPWAP(R) Ver. 1997-1

EXTREMA TABLE

Pile Sgmnt No.	Dist. Below Gages ft	max. Force kips	min. Force kips	max. Comp. Stress kips/in2	max. Tension Stress kips/in2	max. Trnsfd. Energy kips-ft	max. Veloc. ft/s	max. Displ. in
1	3.3	1004.8	-110.5	27.231	-2.993	27.22	14.8	.570
2	6.6	1000.2	-84.0	27.106	-2.276	27.11	14.7	.564
4	13.2	997.9	-21.9	27.043	-.594	26.84	14.6	.555
6	19.8	998.6	-20.0	27.061	-.542	26.55	14.5	.543
8	26.4	997.5	-25.3	27.034	-.685	25.89	14.3	.521
11	36.3	997.2	-4.4	27.025	-.120	24.65	13.6	.487
13	43.0	963.8	.0	26.118	.000	22.62	13.0	.472
15	49.6	924.9	.0	25.065	.000	20.47	12.3	.459
18	59.5	813.0	-19.2	22.033	-.519	15.35	11.2	.438
20	66.1	738.6	-1.1	20.017	-.030	11.91	10.7	.423
22	72.7	567.5	.0	15.380	.000	8.71	15.0	.407
23	76.0	385.6	.0	10.451	.000	5.55	15.5	.399
solute	29.7			27.253		(T=	22.2 ms)	
	3.3				-2.993	(T=	30.1 ms)	

CASE METHOD

	J=0.0	J=0.1	J=0.2	J=0.3	J=0.4	J=0.5	J=0.6	J=0.7	J=0.8	J=0.9
RS1	999.	910.	821.	732.	642.	553.	464.	375.	286.	196.
RMX	999.	910.	821.	732.	642.	553.	464.	375.	341.	334.
RSU	1004.	916.	827.	738.	650.	561.	472.	384.	295.	207.

RAU 227. RA2 393.

Current CAPWAP Ru= 520.0; Corresponding J(Rs)= .54; J(Rx)= .54

VMX	VFN	VT1*Z	FT1	FMX	DMX	DFN	EMX	EFN	RLT	REN
14.67	-.19	914.7	976.0	1015.6	.572	.438	27.1	26.3	876.	1200.

PILE PROFILE AND PILE MODEL

Depth ft	Area in2	E-Modulus kips/in2	Spec. Weight kips/ft3	Circumf. ft
.00	36.90	30004.0	.493	6.280
76.00	36.90	30004.0	.493	6.280

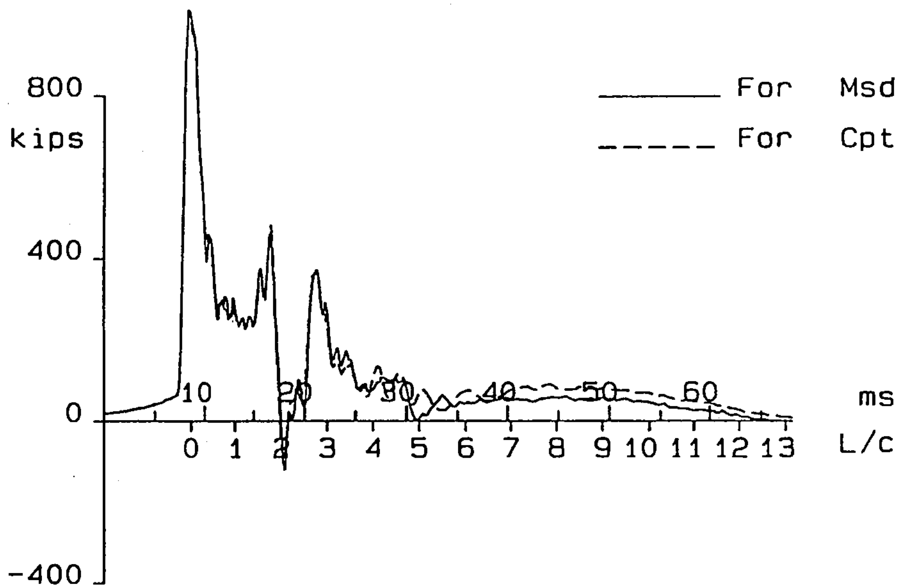
Toe Area 3.140 ft2

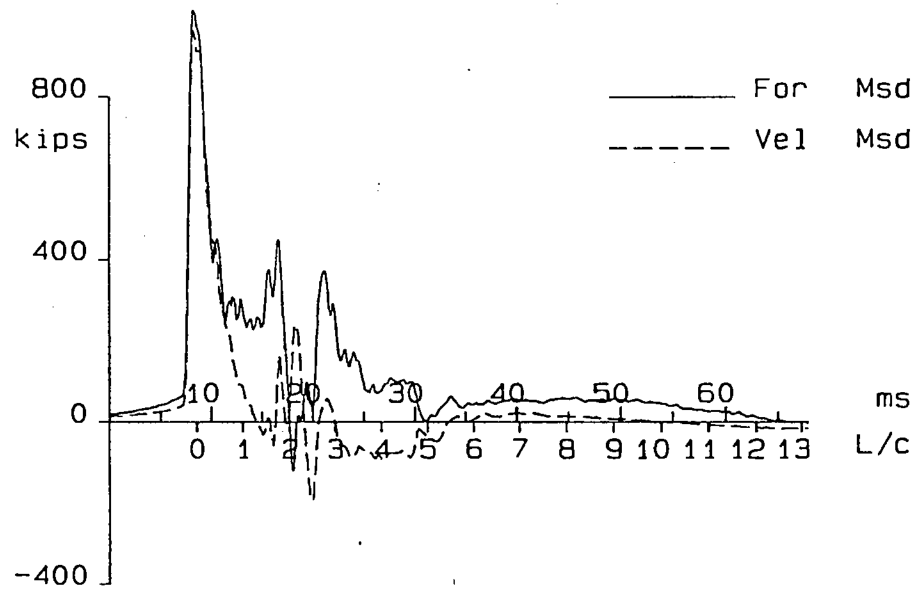
Top Segment Length 3.30 feet, Top Impedance 65.92 kips/ft/s

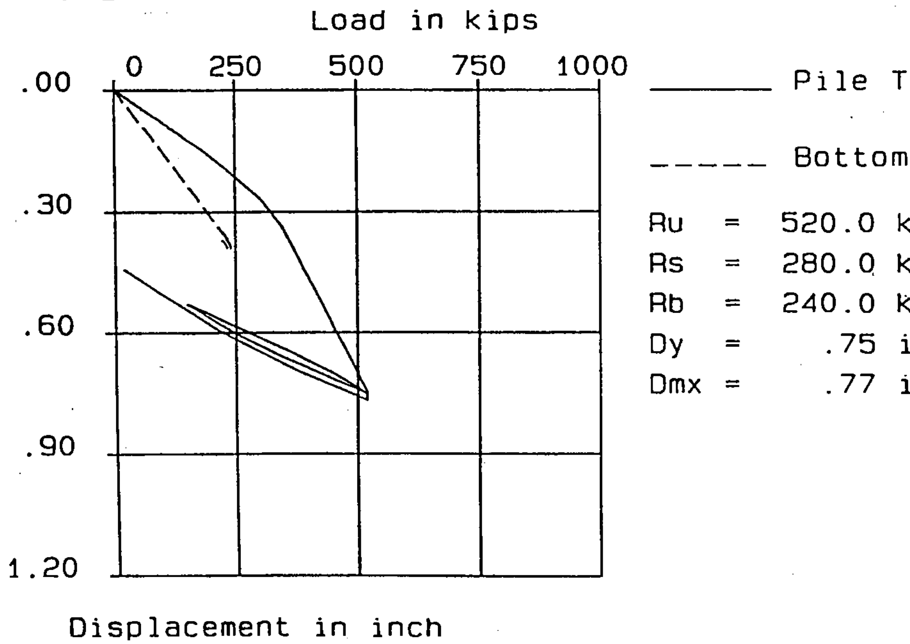
Pile Damping 4.0 %, Time Incr .197 ms, Wave Speed 16794.8 ft/s

SOUTH PARK BRIDGE, TPEND OF DRIVE, BN: 1079 31-Jul-2003

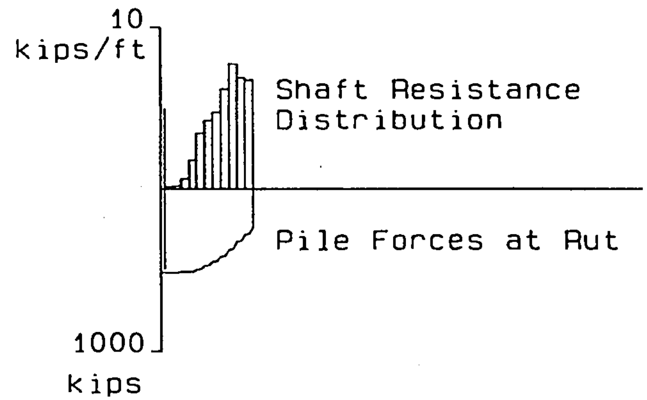
Robert Miner Dynamic Testing, Inc. CAPWAP (R) Version 1997-1







Ru = 520.0 kips
Rs = 280.0 kips
Rb = 240.0 kips
Dy = .75 inch
Dmx = .77 inch



Robert Miner Dynamic Testing, Inc. 31-Jul-2003

SOUTH PARK BRIDGE, Project: 03F36
Pile: TP RESTRIKE B 3 Blow: 3 Data: PP24"x0.50", B4505
Collected: 03-07-23 Operator: RMDT--B. Miner CAPWAP(R) Ver. 1997-1

CAPWAP FINAL RESULTS

Total CAPWAP Capacity: 770.0; along Shaft 580.0; at Toe 190.0 kips

Soil Sgmnt No.	Dist. Below Gages ft	Depth Below Grade ft	Ru kips	Force in Pile at Ru kips	Sum of Ru kips	Unit Resist. w. Respect to Depth kips/ft	Unit Resist. w. Respect to Area kips/f2	Smith Damping Factor s/ft	Quake inch
				770.0					
1	9.9	5.9	19.0	751.0	19.0	2.88	.46	.092	.100
2	16.5	12.5	15.3	735.7	34.3	2.31	.37	.092	.100
3	23.1	19.1	16.6	719.1	50.9	2.51	.40	.092	.100
4	29.7	25.7	26.1	693.0	77.0	3.94	.63	.092	.100
5	36.3	32.3	35.6	657.4	112.6	5.39	.86	.092	.100
6	43.0	39.0	41.8	615.6	154.4	6.32	1.01	.092	.100
7	49.6	45.6	52.0	563.7	206.3	7.87	1.25	.092	.100
8	56.2	52.2	68.5	495.1	274.9	10.37	1.65	.092	.100
9	62.8	58.8	84.1	411.0	359.0	12.73	2.03	.092	.100
10	69.4	65.4	105.8	305.3	464.7	16.00	2.55	.092	.100
11	76.0	72.0	115.3	190.0	580.0	17.44	2.78	.092	.100
Average Skin Values			52.7			8.06	1.27	.092	.100
	Toe		190.0				60.51	.080	.210

Soil Model Parameters/Extensions		Skin	Toe
Case Damping Factor		.806	.231
Unloading Quake	(% of loading quake)	25	10
Reloading Level	(% of Ru)	100	100
Soil Plug Weight	(kips)		.40
Soil Support Dashpot		2.000	.000
Soil Support Mass	(kips)	4.70	.00

Robert Miner Dynamic Testing, Inc. 31-Jul-2003

SOUTH PARK BRIDGE, Project: 03F36
Pile: TP RESTRIKE B 3 Blow: 3 Data: PP24"x0.50", B4505
Collected: 03-07-23 Operator: RMDT--B. Miner CAPWAP(R) Ver. 1997-1

EXTREMA TABLE

Pile Sgmnt No.	Dist. Below Gages ft	max. Force kips	min. Force kips	max. Comp. Stress kips/in2	max. Tension Stress kips/in2	max. Trnsfd. Energy kips-ft	max. Veloc. ft/s	max. Displ. in
1	3.3	874.4	-44.6	23.696	-1.209	23.16	12.3	.481
2	6.6	883.1	-46.9	23.934	-1.271	22.79	12.1	.464
4	13.2	858.9	-33.9	23.277	-.919	20.94	11.9	.427
6	19.8	844.6	-13.5	22.889	-.365	19.77	11.6	.403
8	26.4	834.4	.0	22.612	.000	18.54	11.2	.376
11	36.3	831.3	.0	22.529	.000	16.46	10.4	.327
13	43.0	802.0	.0	21.735	.000	14.63	9.8	.298
15	49.6	771.8	.0	20.917	.000	13.09	9.2	.281
18	59.5	648.7	.0	17.581	.000	9.58	8.1	.252
20	66.1	594.6	.0	16.114	.000	7.57	7.4	.235
22	72.7	464.8	.0	12.595	.000	5.35	8.6	.219
23	76.0	377.1	.0	10.221	.000	3.20	9.0	.213
solute	9.9			24.127		(T=	21.6 ms)	
	6.6				-1.271	(T=	36.6 ms)	

CASE METHOD

	J=0.0	J=0.1	J=0.2	J=0.3	J=0.4	J=0.5	J=0.6	J=0.7	J=0.8	J=0.9
RS1	1056.	994.	932.	870.	808.	746.	684.	622.	560.	498.
RMX	1056.	994.	932.	870.	808.	746.	684.	622.	560.	498.
RSU	1118.	1063.	1007.	952.	896.	840.	785.	729.	673.	618.

RAU 342. RA2 433.

Current CAPWAP Ru= 770.0; Corresponding J(Rs)= .46; J(Rx)= .46

VMX	VFN	VT1*Z	FT1	FMX	DMX	DFN	EMX	EFN	RLT	REN
12.09	.17	782.9	891.8	891.8	.493	.288	23.4	20.2	909.	1250.

PILE PROFILE AND PILE MODEL

Depth ft	Area in2	E-Modulus kips/in2	Spec. Weight kips/ft3	Circumf. ft
.00	36.90	30004.0	.493	6.280
76.00	36.90	30004.0	.493	6.280

Toe Area 3.140 ft2

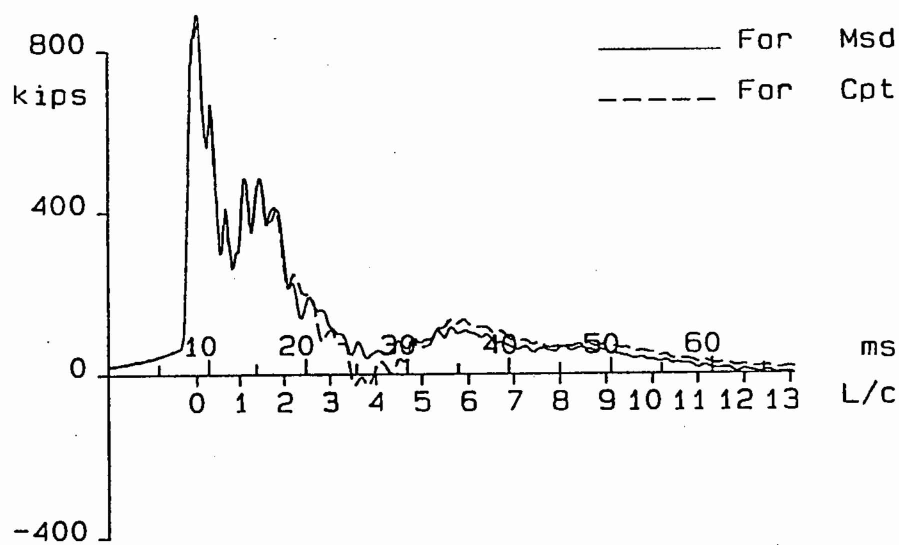
Top Segment Length 3.30 feet, Top Impedance 65.92 kips/ft/s

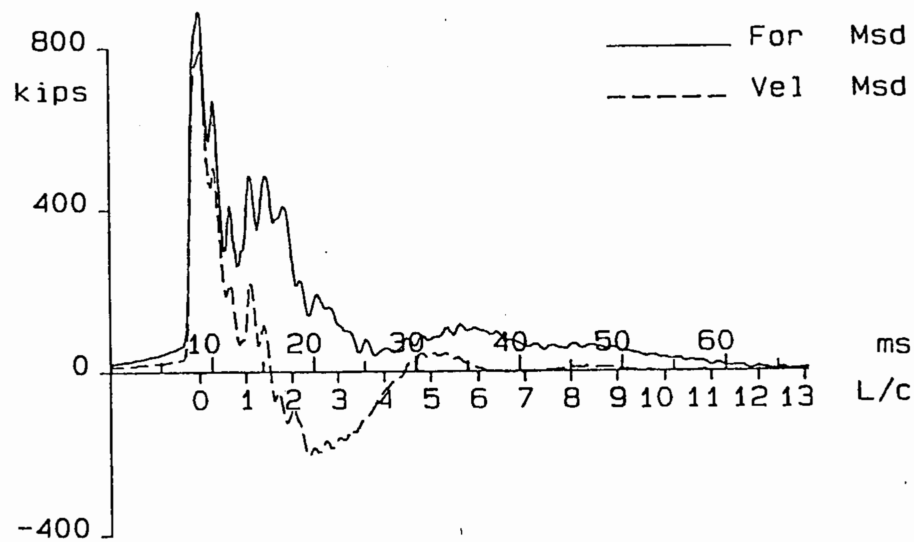
Pile Damping 4.0 %, Time Incr .197 ms, Wave Speed 16794.8 ft/s

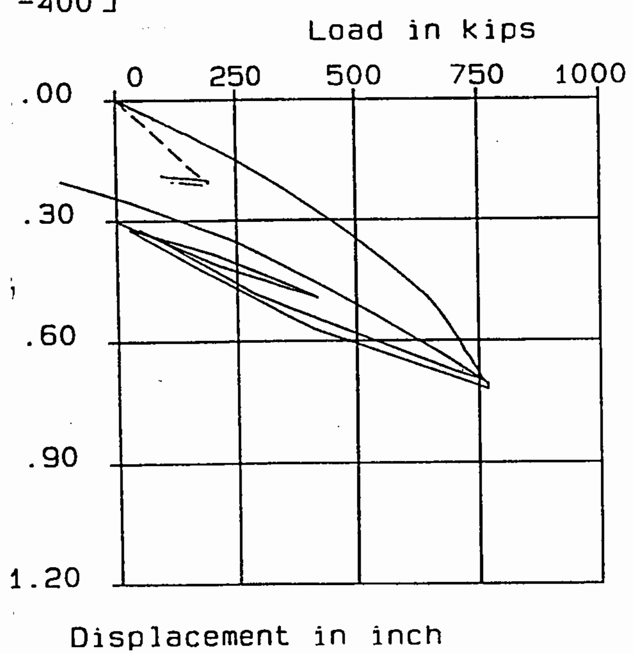
Robert Miner Dynamic Testing, Inc.

SOUTH PARK BRIDGE, TP RESTRIKE B 3, BN: 3
Robert Miner Dynamic Testing, Inc.

31-Jul-2003
CAPWAP (R) Version 1997-1



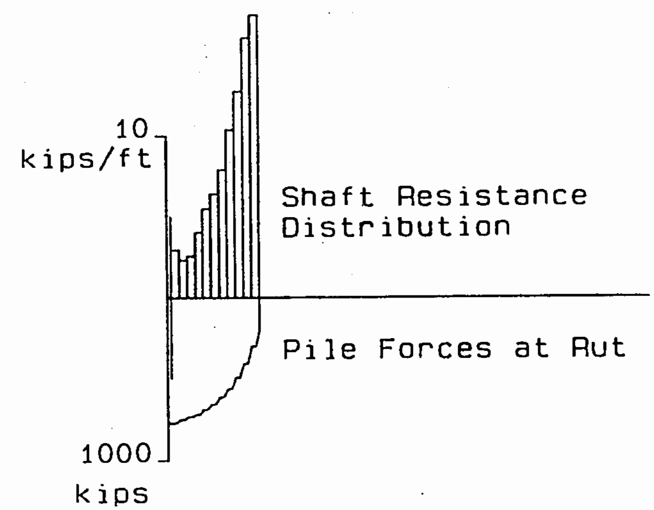




Pile Top

Bottom

Ru = 770.0 kips
Rs = 580.0 kips
Rb = 190.0 kips
Dy = .71 inch
Dmx = .72 inch



Robert Miner Dynamic Testing, Inc.

Robert Miner Dynamic Testing, Inc. 31-Jul-2003

SOUTH PARK BRIDGE, Project: 02F36
Pile: TP RESTRIKE B11 Blow: 11 Data: PP24"x0.50", B4505
Collected: 03-07-23 Operator: RMDT--B. Miner CAPWAP(R) Ver. 1997-1

CAPWAP FINAL RESULTS

Total CAPWAP Capacity: 700.0; along Shaft 460.0; at Toe 240.0 kips

Soil Sgmnt No.	Dist. Below Gages ft	Depth Below Grade ft	Ru kips	Force in Pile at Ru kips	Sum of Ru kips	Unit Resist. w. Respect to Depth kips/ft	Unit Resist. w. Respect to Area kips/f2	Smith Damping Factor s/ft	Quake inch
				700.0					
1	9.9	5.9	12.8	687.2	12.8	1.94	.31	.100	.100
2	16.5	12.5	9.7	677.5	22.5	1.46	.23	.100	.100
3	23.1	19.1	14.2	663.3	36.7	2.15	.34	.100	.100
4	29.7	25.7	25.8	637.5	62.5	3.90	.62	.100	.100
5	36.3	32.3	32.4	605.1	94.9	4.91	.78	.100	.100
6	43.0	39.0	34.5	570.6	129.4	5.22	.83	.100	.100
7	49.6	45.6	42.0	528.6	171.4	6.36	1.01	.100	.100
8	56.2	52.2	54.8	473.8	226.2	8.29	1.32	.100	.100
9	62.8	58.8	65.2	408.6	291.4	9.86	1.57	.100	.100
10	69.4	65.4	77.1	331.5	368.5	11.67	1.86	.100	.100
11	76.0	72.0	91.5	240.0	460.0	13.84	2.20	.100	.100
Average Skin Values			41.8			6.39	1.01	.100	.100
	Toe		240.0				76.43	.060	.230

Soil Model Parameters/Extensions		Skin	Toe
Case Damping Factor		.698	.218
Unloading Quake	(% of loading quake)	12	20
Reloading Level	(% of Ru)	100	100
Unloading Level	(% of Ru)	30	
Soil Plug Weight	(kips)		.40
Soil Support Dashpot		1.500	.000
Soil Support Mass	(kips)	4.30	.00

Robert Miner Dynamic Testing, Inc. 31-Jul-2003

SOUTH PARK BRIDGE, Project: 02F36
Pile: TP RESTRIKE B11 Blow: 11 Data: PP24"x0.50", B4505
Collected: 03-07-23 Operator: RMDT--B. Miner CAPWAP(R) Ver. 1997-1

EXTREMA TABLE

Pile Sgmnt No.	Dist. Below Gages ft	max. Force kips	min. Force kips	max. Comp. Stress kips/in2	max. Tension Stress kips/in2	max. Trnsfd. Energy kips-ft	max. Veloc. ft/s	max. Displ. in
1	3.3	937.6	-2.7	25.408	-.073	25.51	13.5	.505
2	6.6	944.8	-2.8	25.605	-.076	25.23	13.4	.490
4	13.2	927.1	.0	25.124	.000	24.01	13.2	.470
6	19.8	920.7	.0	24.951	.000	23.08	12.9	.448
8	26.4	914.7	.0	24.789	.000	21.90	12.6	.424
11	36.3	907.2	.0	24.585	.000	19.77	11.8	.380
13	43.0	874.4	.0	23.697	.000	17.87	11.3	.355
15	49.6	846.5	.0	22.940	.000	16.15	10.7	.336
18	59.5	734.0	.0	19.891	.000	12.32	9.7	.304
20	66.1	684.3	.0	18.545	.000	10.05	9.0	.284
22	72.7	571.7	.0	15.493	.000	7.62	10.5	.268
23	76.0	450.1	.0	12.197	.000	5.09	11.3	.260
solute	9.9			25.767		(T=	21.2 ms)	
	6.6				-.076	(T=	84.4 ms)	

CASE METHOD

	J=0.0	J=0.1	J=0.2	J=0.3	J=0.4	J=0.5	J=0.6	J=0.7	J=0.8	J=0.9
RS1	911.	836.	762.	688.	614.	540.	466.	392.	317.	243.
RMX	911.	836.	762.	688.	614.	540.	477.	437.	399.	388.
RSU	995.	929.	864.	798.	732.	667.	601.	535.	469.	404.

RAU 124. RA2 479.

Current CAPWAP Ru= 700.0; Corresponding J(Rs)= .28; J(Rx)= .28

VMX	VFN	VT1*Z	FT1	FMX	DMX	DFN	EMX	EFN	RLT	REN
13.20	-.09	801.7	850.1	967.3	.507	.265	25.6	22.9	1052.	1737.

PILE PROFILE AND PILE MODEL

Depth ft	Area in2	E-Modulus kips/in2	Spec. Weight kips/ft3	Circumf. ft
.00	36.90	30004.0	.493	6.280
76.00	36.90	30004.0	.493	6.280

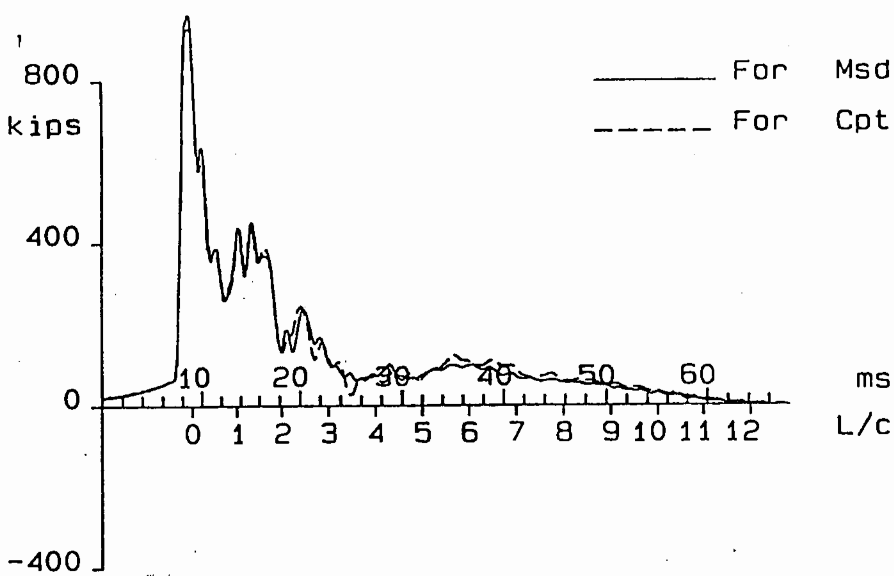
Toe Area 3.140 ft2

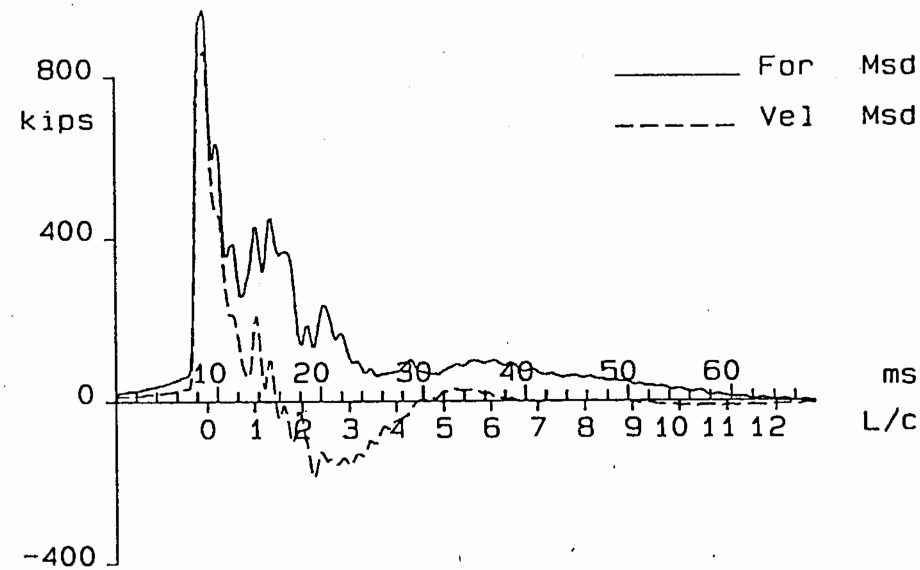
Top Segment Length 3.30 feet, Top Impedance 65.92 kips/ft/s

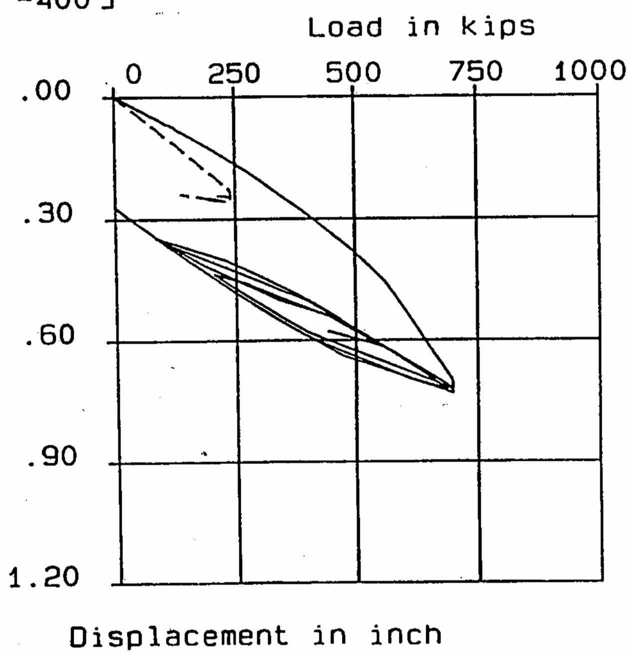
Pile Damping 4.0 %, Time Incr .200 ms, Wave Speed 16794.8 ft/s

SOUTH PARK BRIDGE, TP RESTRIKE B11, BN: 11 31-Jul-2003

Robert Miner Dynamic Testing, Inc. CAPWAP(R) Version 1997-1



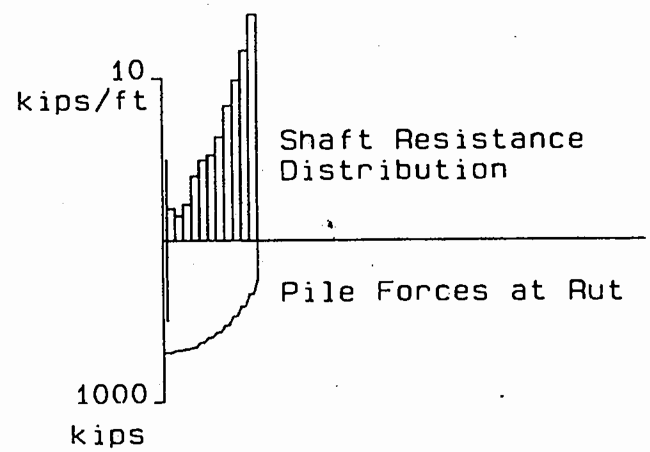




Pile Top

Bottom

Ru = 700.0 kips
Rs = 460.0 kips
Rb = 240.0 kips
Dy = .71 inch
Dmx = .73 inch



APPENDIX F

IMPORTANT INFORMATION ABOUT YOUR GEOTECHNICAL REPORT

Date: March 30, 2004
To: Parsons Brinckerhoff
Seattle, Washington

IMPORTANT INFORMATION ABOUT YOUR GEOTECHNICAL/ENVIRONMENTAL REPORT

CONSULTING SERVICES ARE PERFORMED FOR SPECIFIC PURPOSES AND FOR SPECIFIC CLIENTS.

Consultants prepare reports to meet the specific needs of specific individuals. A report prepared for a civil engineer may not be adequate for a construction contractor or even another civil engineer. Unless indicated otherwise, your consultant prepared your report expressly for you and expressly for the purposes you indicated. No one other than you should apply this report for its intended purpose without first conferring with the consultant. No party should apply this report for any purpose other than that originally contemplated without first conferring with the consultant.

THE CONSULTANT'S REPORT IS BASED ON PROJECT-SPECIFIC FACTORS.

A geotechnical/environmental report is based on a subsurface exploration plan designed to consider a unique set of project-specific factors. Depending on the project, these may include: the general nature of the structure and property involved; its size and configuration; its historical use and practice; the location of the structure on the site and its orientation; other improvements such as access roads, parking lots, and underground utilities; and the additional risk created by scope-of-service limitations imposed by the client. To help avoid costly problems, ask the consultant to evaluate how any factors that change subsequent to the date of the report may affect the recommendations. Unless your consultant indicates otherwise, your report should not be used: (1) when the nature of the proposed project is changed (for example, if an office building will be erected instead of a parking garage, or if a refrigerated warehouse will be built instead of an unrefrigerated one, or chemicals are discovered on or near the site); (2) when the size, elevation, or configuration of the proposed project is altered; (3) when the location or orientation of the proposed project is modified; (4) when there is a change of ownership; or (5) for application to an adjacent site. Consultants cannot accept responsibility for problems that may occur if they are not consulted after factors which were considered in the development of the report have changed.

SUBSURFACE CONDITIONS CAN CHANGE.

Subsurface conditions may be affected as a result of natural processes or human activity. Because a geotechnical/environmental report is based on conditions that existed at the time of subsurface exploration, construction decisions should not be based on a report whose adequacy may have been affected by time. Ask the consultant to advise if additional tests are desirable before construction starts; for example, groundwater conditions commonly vary seasonally.

Construction operations at or adjacent to the site and natural events such as floods, earthquakes, or groundwater fluctuations may also affect subsurface conditions and, thus, the continuing adequacy of a geotechnical/environmental report. The consultant should be kept apprised of any such events, and should be consulted to determine if additional tests are necessary.

MOST RECOMMENDATIONS ARE PROFESSIONAL JUDGMENTS.

Site exploration and testing identifies actual surface and subsurface conditions only at those points where samples are taken. The data were extrapolated by your consultant, who then applied judgment to render an opinion about overall subsurface conditions. The actual interface between materials may be far more gradual or abrupt than your report indicates. Actual conditions in areas not sampled may differ from those predicted in your report. While nothing can be done to prevent such situations, you and your consultant can work together to help reduce their impacts. Retaining your consultant to observe subsurface construction operations can be particularly beneficial in this respect.

A REPORT'S CONCLUSIONS ARE PRELIMINARY.

The conclusions contained in your consultant's report are preliminary because they must be based on the assumption that conditions reve. through selective exploratory sampling are indicative of actual conditions throughout a site. Actual subsurface conditions can be discer only during earthwork; therefore, you should retain your consultant to observe actual conditions and to provide conclusions. Only consultant who prepared the report is fully familiar with the background information needed to determine whether or not the repc recommendations based on those conclusions are valid and whether or not the contractor is abiding by applicable recommendations. consultant who developed your report cannot assume responsibility or liability for the adequacy of the report's recommendations if anot party is retained to observe construction.

THE CONSULTANT'S REPORT IS SUBJECT TO MISINTERPRETATION.

Costly problems can occur when other design professionals develop their plans based on misinterpretation of a geotechnical/environmen report. To help avoid these problems, the consultant should be retained to work with other project design professionals to explain releva geotechnical, geological, hydrogeological, and environmental findings, and to review the adequacy of their plans and specifications relati to these issues.

BORING LOGS AND/OR MONITORING WELL DATA SHOULD NOT BE SEPARATED FROM THE REPORT.

Final boring logs developed by the consultant are based upon interpretation of field logs (assembled by site personnel), field test results, an laboratory and/or office evaluation of field samples and data. Only final boring logs and data are customarily included i geotechnical/environmental reports. These final logs should not, under any circumstances, be redrawn for inclusion in architectural or othe design drawings, because drafters may commit errors or omissions in the transfer process.

To reduce the likelihood of boring log or monitoring well misinterpretation, contractors should be given ready access to the complete geotechnical engineering/environmental report prepared or authorized for their use. If access is provided only to the report prepared for you, you should advise contractors of the report's limitations, assuming that a contractor was not one of the specific persons for whom the report was prepared, and that developing construction cost estimates was not one of the specific purposes for which it was prepared. While a contractor may gain important knowledge from a report prepared for another party, the contractor should discuss the report with your consultant and perform the additional or alternative work believed necessary to obtain the data specifically appropriate for construction cost estimating purposes. Some clients hold the mistaken impression that simply disclaiming responsibility for the accuracy of subsurface information always insulates them from attendant liability. Providing the best available information to contractors helps prevent costly construction problems and the adversarial attitudes that aggravate them to a disproportionate scale.

READ RESPONSIBILITY CLAUSES CLOSELY.

Because geotechnical/environmental engineering is based extensively on judgment and opinion, it is far less exact than other design disciplines. This situation has resulted in wholly unwarranted claims being lodged against consultants. To help prevent this problem, consultants have developed a number of clauses for use in their contracts, reports and other documents. These responsibility clauses are not exculpatory clauses designed to transfer the consultant's liabilities to other parties; rather, they are definitive clauses that identify where the consultant's responsibilities begin and end. Their use helps all parties involved recognize their individual responsibilities and take appropriate action. Some of these definitive clauses are likely to appear in your report, and you are encouraged to read them closely. Your consultant will be pleased to give full and frank answers to your questions.

The preceding paragraphs are based on information provided by the
ASFE/Association of Engineering Firms Practicing in the Geosciences, Silver Spring, Maryland

Supplementary Geotechnical Report

Supplementary Geotechnical Report

Supplemental Geotechnical Report
Phase II
South Park Bridge Project
King County, Washington

June 22, 2007

Submitted To:
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By:
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21-1-09584-008

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SUPPLEMENTAL GEOTECHNICAL REPORT
PHASE II
SOUTH PARK BRIDGE PROJECT
KING COUNTY, WASHINGTON

1.0 INTRODUCTION

Shannon & Wilson has completed and submitted a geotechnical report for the project titled, "Geotechnical Report, Phase II, South Park Bridge Project, King County, Washington," dated March, 29 2004 (2004 geotechnical report). This supplemental report addresses the latest project design changes and presents the results of our additional geotechnical engineering studies and our revised geotechnical recommendations. In particular, this report includes the following design items:

- Free field horizontal and vertical field spectra for 108- and 975-year return period ground motions
- Revised liquefaction analyses for 975-year earthquake ground motions.
- Revised estimated axial compression and uplift capacity plots under static and seismic loading conditions for 24-inch-diameter, closed-end pipe piles; 36-inch-diameter, open-end pipe piles; and 6-, 8-, and 10-foot-diameter drilled shafts, including:
  - Adding axial capacity estimates for 36-inch-diameter, open-end steel pipe piles.
  - Revising end bearing capacity for drilled shafts under seismic loading
  - Providing estimated seismic axial capacities for 975-year earthquake return period.
  - Providing downdrag loads for 975-year earthquake return period.
  - Extending the axial capacity plots to elevation -160 feet.
- Revised LPILE parameters based on residual soil properties under 975-year earthquake return period.
- LPILE parameters for buckling analyses.
- Executive summary of hazardous material study completed by Wilbur Consulting, Inc.
- Additional construction considerations, including ground improvement beneath existing bascule bridge pier foundations.

The geotechnical recommendations presented in this supplemental report supersede those presented in our 2004 geotechnical report. For geotechnical recommendations not addressed in this report, our previous recommendations are still valid as presented in the 2004 geotechnical report.

The additional studies and the preparation of this supplemental report were authorized by a contract between Parsons Brinckerhoff, Inc. (PB) and Shannon & Wilson, Inc. Notice to proceed for this project was received on June 8, 2007.

2.0 EARTHQUAKE ENGINEERING STUDIES

2.1 General

We understand that seismic design of the bridge will be based on a two-level approach, similar to the approach outlined in the Recommended LRFD Guidelines for the Seismic Design of Highway Bridges, Applied Technology Council (ATC), Multidisciplinary Center for Earthquake Engineering Research Design of Highway Bridges, (MCEER), - 49, 2003 (ATC-49). The two earthquake ground motion levels in ATC-49 for design have return periods of 108 and 2,500 years. We understand that for this project, the longer earthquake return period used for design will be 975 years instead of 2,500 years. A return period of 975 years is consistent with the criteria in the Recommended LRFD Guidelines for the Seismic Design of Highway Bridges, prepared for AASHTO by TRC, Imbsen & Associates (TRC/Imbsen, 2006).

2.2 Spectra for 108- and 975-Year Return Periods

Horizontal and vertical free-field response spectra were developed for earthquake ground motions with return periods of 108 and 975 years in general accordance with the procedures outlined in TRC/Imbsen, 2006. Computation of forces used for seismic design for this procedure is based on seismological input and site soil response factors.

The seismological inputs are short period (0.2-second period) spectral acceleration, S_S, and the 1 second period spectral acceleration, S_1 for rock-like conditions, corresponding to two ground motion return period design levels. Based on regional probabilistic seismic hazard analysis (PSHA) studies by the U.S. Geological Survey (USGS) (Frankel et al., 2002) S_S, and S_1 for a 108-year return period are 0.34g and 0.11g, respectively; for 975-year return period, S_S, and S_1 are 1.03g and 0.35g, respectively.

The site soil response factors are based on determination of the Site Classification. Site Classification is based on the measured shear wave velocities within 100 feet of the ground surface. The site average shear wave velocity is 627 feet per second (fps). Sites with average shear wave velocities greater than 600 fps are classified as "D"; sites with average shear wave velocities less than 600 fps are classified as "E." However, Site Class E spectra more closely approximate the results of the ground response analyses performed at this site for the 2004 geotechnical report and 475-year ground motions. Consequently, we recommend that spectra be based on Site Class E shape.

Free-field horizontal spectra corresponding to the recommended S_S and S_1 values and Site Class E are shown in Figure 2. The horizontal 975-year spectrum shown on this figure has been modified to envelope the response measured from the 2001 Nisqually Earthquake nearby on Harbor Island (as was done for the recommended design spectrum provided in the 2004 geotechnical report). The modification consisted of extending the peak constant acceleration portion of the spectrum from a period of approximately 1.0 seconds to 1.1 seconds.

ATC-49 and TRC/Imbsen, 2006 require a site-specific ground motion evaluation of near-fault ground motions if the bridge is located within 10 kilometers of a known active fault. As noted in the 2004 geotechnical report, the site is located within the Seattle Fault Zone, which appears to have a recurrence interval of large earthquakes on the order of a few thousand years. Because of the relatively long recurrence interval for large earthquakes on this fault, the 108-year return period ground motions should not be modified for near fault effects. The PSHAs by the USGS explicitly includes the Seattle Fault, and to this degree, includes near fault effects. Directivity is another "near-fault" effect which varies depending on fault type, and the location of a given site relative to the fault and rupture direction. These effects are not explicitly considered in the USGS PSHA. However, deterministic directivity effects were developed for rupture on the Seattle Fault for the Alaskan Way Viaduct Project, assuming the site is immediately up-dip of the fault. Spectrum amplification factors for average directivity effects ranged from 1.0 (i.e., no amplification) for periods less than or equal to 0.8 seconds, to 1.36 at a period of 5 seconds. For a 975-year ground motion return period, approximately 35 to 40 percent of the hazard is from the Seattle Fault based on the recent USGS PSHA. The period range of interest for the bridge is about 1 to 2 seconds. For the 975-year ground motion return period the spectrum amplification for directivity and proportional to the hazard contribution from the Seattle Fault is between

1.0 and 1.06 for the period range of interest. Because the recommended 975-year spectrum in Figure 2 is already modified for periods greater than 1.0 seconds to envelope the historic Nisqually Earthquake ground motions, no additional modification are required for directivity effects is needed, in our opinion.

Free-field vertical spectra corresponding to the recommended horizontal spectra are also shown in Figure 2. The vertical spectra were developed by multiplying the horizontal spectra by a vertical-to-horizontal spectral ratio. The ratios are for earthquakes consistent with 108- and 975-year return period ground motions and were developed for the nearby Alaskan Way Viaduct & Seawall Replacement Project using published ground motion attenuation relationships.

2.3 Liquefaction Analysis for 975-Year Return Period Ground Motion

The liquefaction potential of the soils encountered at the borings near the proposed bascule piers (SB-4 and SB-5) was evaluated for 975-year ground motions using the same analysis method described in the 2004 geotechnical report. An earthquake magnitude of 6.7 and a conservative peak ground acceleration of 0.45g was used in the liquefaction analyses. A magnitude of 6.7 is consistent with the deaggregation results from the USGS PSHA. The peak ground acceleration corresponds to Site Class D. As previously noted, based on shear wave velocity measurements, the site classifies as Site Class D, but the ground response analyses for the 475-year return period ground motions were approximated better using Site Class E. For the 975-year ground motion levels, the peak ground acceleration for Site Class D is greater than for Site Class E. Therefore, the liquefaction analysis was based on the higher (and more conservative) peak ground acceleration associated with Site Class D.

The results of the liquefaction analyses performed for SB-4 and SB-5 are shown in Figures 3 and 4, respectively, as plots of factor of safety against liquefaction versus depth. We have indicated on these plots where the factor of safety was greater than 1 for the 475-year ground motions but less than on for the 975-year ground motions. The number of points with factors of safety greater than one for the 475-year ground motions but less than 1 for the 975-year ground motions does not significantly affect the characterization of subsoil layers in terms of liquefaction between the 475- and 975-year ground motion levels. Soil shear strength reduction due to liquefaction was considered in developing the foundation capacities subsequently presented in this supplemental report.

3.0 FOUNDATION DESIGN RECOMMENDATIONS

3.1 General

Our 2004 geotechnical report provided foundation recommendations for 24-inch-diameter steel pipe piles driven closed-end, and for 6-, 8-, and 10-foot-diameter drilled shafts. In this report, we add recommendations for 36-inch-diameter steel pipe pile driven open-end. The recommendations provided in this report are based on subsurface conditions encountered in borings SB-4 and SB-5 which were drilled over water in the Duwamish Waterway. As shown in Figure 1, borings SB-4 and SB-5 were drilled near the north and south piers of the replacement bridge alternatives.

For seismic loading conditions, foundation recommendations in our 2004 geotechnical report were based on earthquake ground motions with a 10 percent probability of being exceeded in 50 years (475-year return period). As discussed earlier, the new bridge is proposed to be designed based on a two-level seismic design approach: a 108-year return period for operational level design, and a 975-year return period for "no collapse" level design. This report provides seismic foundation design recommendations based on an earthquake with a 975-year return period.

We understand that the Load and Resistance Factor Design (LRFD) method as defined in AASHTO LRFD Bridge Design Specifications (AASHTO, 2006) would be used to design the bridge foundations. Our recommendations are revised to be based on the LRFD design method.

3.2 Axial Capacity

Axial capacity analyses were performed for 24-inch-diameter steel pipe piles driven closed-end, 36-inch-diameter steel pipe piles driven open-end, and 6-, 8-, and 10-foot-diameter drilled shafts. This section describes the analysis approach used to estimate the capacities of these piles and drilled shafts and presents the results of the axial capacity analyses.

Foundations designed using the LRFD method should be proportioned so that the factored axial resistance provided by the soil is at least equal to the factored axial load applied to the foundations for the service, strength, and extreme event limits states. The factored axial load is defined as the nominal (ultimate) load multiplied by a load factor that would increase that ultimate load. The factored axial resistance is defined as the ultimate resistance provided by the soil multiplied by appropriate resistance factors to account for uncertainties in determination and variability of the actual capacity of the soil. This report provides our estimated unfactored, ultimate resistances and recommended resistance factors for design of the replacement bridge foundations for the strength limit state and for the extreme event limit state. Because of the anticipated high seismic loads, axial resistances for the service limit state are not provided at this early phase of the project design.

Axial capacity analyses were performed using an in-house computer program. They were based on subsurface conditions encountered in borings SB-4 and SB-5, relative densities of the subsoils as determined by SPTs (Standard Penetration Test N-values), and our experience in similar soil and project conditions. Ultimate unit skin friction and unit end bearing values were estimated for each soil unit underlying the project site. These estimated parameters were based on the average N-value within a unit, empirical relationships that relate N-value to soil parameters, results of the laboratory tests, and our experience. Our estimated soil parameters assume that no ground improvement would be implemented at and around the replacement bridge pier locations.

Results of our axial capacity analyses are presented in Figures 5 through 12. These results are presented in terms of plots of unfactored ultimate side resistance and unfactored ultimate base resistance versus pile tip/drilled shaft base elevation. Assumed representative subsurface conditions used in the analyses are also presented in Figures 5 through 12. The ultimate resistances are provided for the strength limit state and for the extreme event limit state. As indicated earlier, resistances are not provided for the service limit state at this phase of the design. Resistance plots for the extreme limit state also present our estimated unfactored downdrag force resulting from potential liquefaction-induced ground settlement. It should be noted that liquefaction-induced ground settlements and the resulting downdrag forces are likely to develop after the maximum anticipated seismic forces had occurred. Therefore, this downdrag force is recommended to be applied with post-earthquake loading consisting of the typical loads under static conditions.

Recommended resistance factors to apply to the unfactored ultimate resistances, and recommended load factor to apply to the unfactored downdrag loads are presented in Table 1. The required pile/drilled shaft penetrations to satisfy the factored loads can be determined from these load and resistance factors and the estimated unfactored ultimate resistance plots and downdrag loads presented in Figures 5 through 12.

3.3 Lateral Resistance

The computer program $LPILE^{PLUS}$ (Reese and Wang, 2006) may be used to generate P-Y curves (load-deflection curves) for the lateral resistance analysis of the replacement bridge foundations and to calculate the magnitude of deflection, shear, and moment along the pile/drilled shaft. Based on the subsurface conditions encountered in borings SB-4 and SB-5, the recommended soil parameters for input into the $LPILE^{PLUS}$ program under static, cyclic, and seismic loading conditions are summarized in Table 2. These parameters can be used to evaluate the lateral resistance of foundations supporting the north and south bascule piers of the proposed replacement bridge. The seismic condition is based on an earthquake with 975-year return period and it assumes reduced strength and/or liquefied conditions, where appropriate. For the liquefied soils, residual shear strengths are recommended for use. These residual shear strengths were estimated based on the results of the liquefaction studies under 975-year return period ground motions.

We understand that the replacement bridge foundations need to be evaluated for buckling under liquefied soil conditions. In order to evaluate the soil resistance against buckling, we recommend that lateral resistance analyses using $LPILE^{PLUS}$ with seismic parameters provided in Table 2 be performed.

The recommended efficiency (reduction) factors due to pile/drilled shaft group action are presented in Table 3. The efficiency factors are based on recommendations presented in a 1998 ENSOFT Seminar (ENSOFT, 1998).

3.4 Load Versus Settlement Curves

Based on the subsurface conditions encountered in borings SB-4 and SB-5, we used the commercial computer programs $APILE^{PLUS}$ (Reese and Wang, 2003a) and SHAFT (Reese and Wang, 2003b), to develop load versus deflection curves for the 24-inch-diameter steel pipe piles and the 8-foot-diameter drilled shafts under static and seismic loading conditions. The load versus deflection curves were developed for side friction in terms of t-z curves at different depths along the foundation embedments. Load versus deflection curves were also developed at the top and tip of the foundations. Data points for the developed load versus settlement curves are provided in Appendix A, Data for Load versus Settlement Curves.

The provided load versus settlement curves as developed by APILE^{PLUS} and SHAFT are based on limited testing. In our opinion, they may not provide realistic representation of soil conditions at the project site and they should be used with reservation. It is our opinion that representative load versus settlement curves should be obtained from actual load test or dynamic testing data at the project site.

4.0 HAZARDOUS MATERIALS

Environmental studies for the South Park Bridge project were performed by Wilbur Consulting, Inc. under subcontract to PB. Executive summaries from the Hazardous Materials Technical Report (Wilbur Consulting, Inc., February 2004) and Executive Summary for Final Preliminary Site Investigation Report for the South Park Bridge Project prepared by Wilbur Consulting, Inc. (July 2004) are presented in Appendix B, Hazardous Materials, to provide a brief background of environmental studies completed to date for the project.

5.0 CONSTRUCTION CONSIDERATIONS

5.1 General

Section 11.0 of our 2004 geotechnical report presents a discussion on construction considerations that should be evaluated for selecting and designing foundation support for the proposed replacement bridge. This section presents additional construction considerations that supplement those presented in the 2004 geotechnical report.

5.2 Construction-induced Vibrations

Construction activities, such as driving piles, installation of cofferdam, and vibrating drilled shaft casings, would cause vibrations that could cause potential damage to existing nearby structures and utilities. Vibration studies indicate that the peak particle velocity (ppv) is one of the parameters that could be used for assessing potential damage to structures and utilities due to vibrations. Based on discussions with King County and in order to minimize the potential for damage to existing bridge due to vibrations caused by construction operations for proposed replacement bridge, ppv at the existing bridge is recommended not to exceed 0.5 inch per second. We recommend that similar vibration criteria be developed for the other existing structures and utilities located in the vicinity of the proposed replacement bridge. The criteria should consider the type and frequency of the vibrations, the structural design and existing condition of the structure, and the vibration effects on people and building contents.

As discussed in Section 5.3, compaction grouting is recommended to be implemented beneath the foundations of the existing bascule bridge piers to minimize the impact of vibration-induced ground settlements on the existing bridge. To further minimize the impact of construction-induced vibrations, we recommend that construction activities causing vibrations not be allowed within 40 feet of the outer limits of the existing bascule pier foundations.

We recommend that particle velocities be measured during construction using a vibration monitor (such as seismograph) at the nearby existing bridge, buildings, utilities, and at any other critical structures within 50 feet of the construction operations. We strongly recommend that existing structural condition surveys be performed for the existing bridge and other facilities located within 50 feet of the construction activities. Documentation should include photographs, videos, sketches, existing crack measurements, and/or written comments.

If there are any cracks on the existing bridge and in walls, floors, and ceilings of other existing structures, we recommend that crack gages be installed on each crack to measure potential changes in crack widths. The aforementioned measurements, existing structural condition surveys, and crack gage installations should be established well in advance of construction so that a set of baseline data can be developed. This information will be invaluable in assessing the need for mitigating measures, as well as resolving potential disputes.

5.3 Compaction Grouting – Existing Bascule Piers

Construction-induced vibrations would likely result in ground settlement and cause downdrag loads on the foundations supporting the existing bridge. It is recommended that ground improvements in the form of compaction grouting be used to minimize further settlement of the timber piles supporting the existing bascule piers and to increase the capacity these piles.

Compaction grouting should be implemented around the timber piles group supporting the existing bridge and beneath their tips into the underlying glacial deposits. The locations of the compaction grout columns should take into account the locations of the cofferdam and the pier foundations for the replacement bridge to avoid potential conflicts.

6.0 LIMITATIONS

This report should be used in conjunction with Shannon & Wilson, Inc. report titled, "Geotechnical Report, Phase II, South Park Bridge Project, King County, Washington," dated March, 29 2004. It was prepared for the exclusive use of Parsons Brinckerhoff, Inc. and King County for the rehabilitation or replacement of the South Park Bridge. The report should be provided to reviewing agencies and prospective subcontractors for information based on factual data only and not as a warranty of subsurface conditions, such as those interpreted from the exploration logs and discussions of subsurface conditions included in this report.

The analyses, conclusions, and recommendations contained in this report are based on site conditions as they presently exist. We assume that the exploratory borings made for this project are representative of the subsurface conditions throughout the site; i.e., the subsurface conditions everywhere are not significantly different from those disclosed by the explorations. If conditions different from those described in this report are observed or appear to be present during construction, we should be advised at once so that we can review these conditions and reconsider our recommendations, where necessary. If conditions have changed due to natural causes or construction operations at or near the site, it is recommended that this report be reviewed to determine the applicability of the conclusions and recommendations considering the changed conditions and time lapse.

Within the limitations of the scope, schedule, and budget, the analyses, conclusions, and recommendations presented in this report were prepared in accordance with generally accepted professional geotechnical engineering principles and practice in this area at the time this report was prepared. We make no other warranty, either express or implied.

The scope of our services did not include any environmental assessment or evaluation of hazardous or toxic materials in the soil, surface water, groundwater, or air at the subject site.

Limited out-of-scope testing was performed for potential contaminants as described in this report. Shannon & Wilson, Inc., has qualified personnel to assist you with these services should they be necessary.

Shannon & Wilson, Inc., has prepared Appendix C, "Important Information About Your Geotechnical Report," to assist you and others in understanding the use and limitations of our reports.

SHANNON & WILSON, INC.

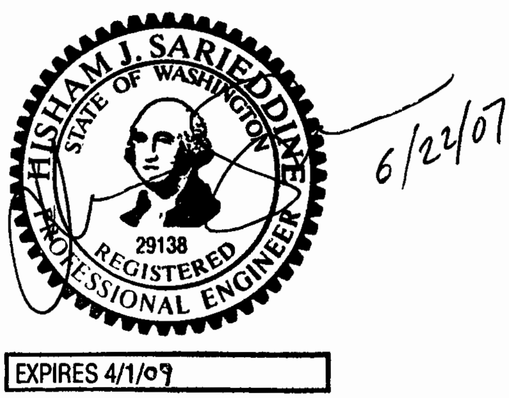

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TABLE 1

Recommended Resistance and Load Factors for Axially Loaded Piles/Drilled Shafts

Resistance or Load Condition	Service Limit State	Strength Limit State	Extreme Event Limit State
Compressive Capacity Resistance Factors			
Base Resistance	1.0	0.5	1.0
Side Resistance	1.0	0.55	1.0
Uplift Capacity Resistance Factors	1.0	0.5	1.0
Downdrag Load Factors	N/A	N/A	1.3

Notes:

1 The loading and resistance factors are based on available subsurface data and our engineering experience and judgement.

2 The downdrag force should be applied with post seismic event loading.

TABLE 2

Recommended Parameters for Development of P-Y Curves Using LPILE^PLUS

Location	Boring	Upper Boundary (Depth Below Ground Surface) (feet)	Lower Boundary (Depth Below Ground Surface) (feet)	Soil Type (Soil Type No.)	Effective Unit Weight γ' (pcf)	Cohesion, c (psf)		Friction Angle, ϕ (°)		Modulus of Subgrade Reaction, k (pci)		ε_{50}
						Static/ Cyclic	Seismic	Static/ Cyclic	Seismic	Static/ Cyclic	Seismic	
Sta.25+35	SB-4	0	20	Sand (4)	47.6	-	-	29	9	20	4	-
		20	82	Sand (4)	57.6	-	-	33	10	60	18	-
		82	94	Sand (4)	57.6	-	-	34	34	70	70	-
		94	-	Stiff Clay w/o free water (3)	62.6	5,000	5,000	-	-	2,000	2,000	0.004
Sta. 23+25	SB-5	0	15	Soft Clay (1)	42.6	150	60	-	-	15	2	0.002
		15	56	Sand (4)	52.6	-	-	31	10	40	12	-
		56	-	Stiff Clay w/o free water (3)	62.6	5,000	5,000	-	-	2,000	2,000	0.004

Note:

(1) Parameters given above are based on subsurface conditions encountered in indicated boring.

(2) Based on subsurface conditions encountered along alignment, the shaded zones of soil in the table may liquefy under earthquake loading. Seismic loading may also result in strength reduction for some cohesive soil layers, mainly soils overlying liquefied zones. Parameters under 975-year return period ground motions are provided.

(3) Parameters given above do not reflect effect of deep foundation group action. See text and Table 3 regarding recommendations for group action.

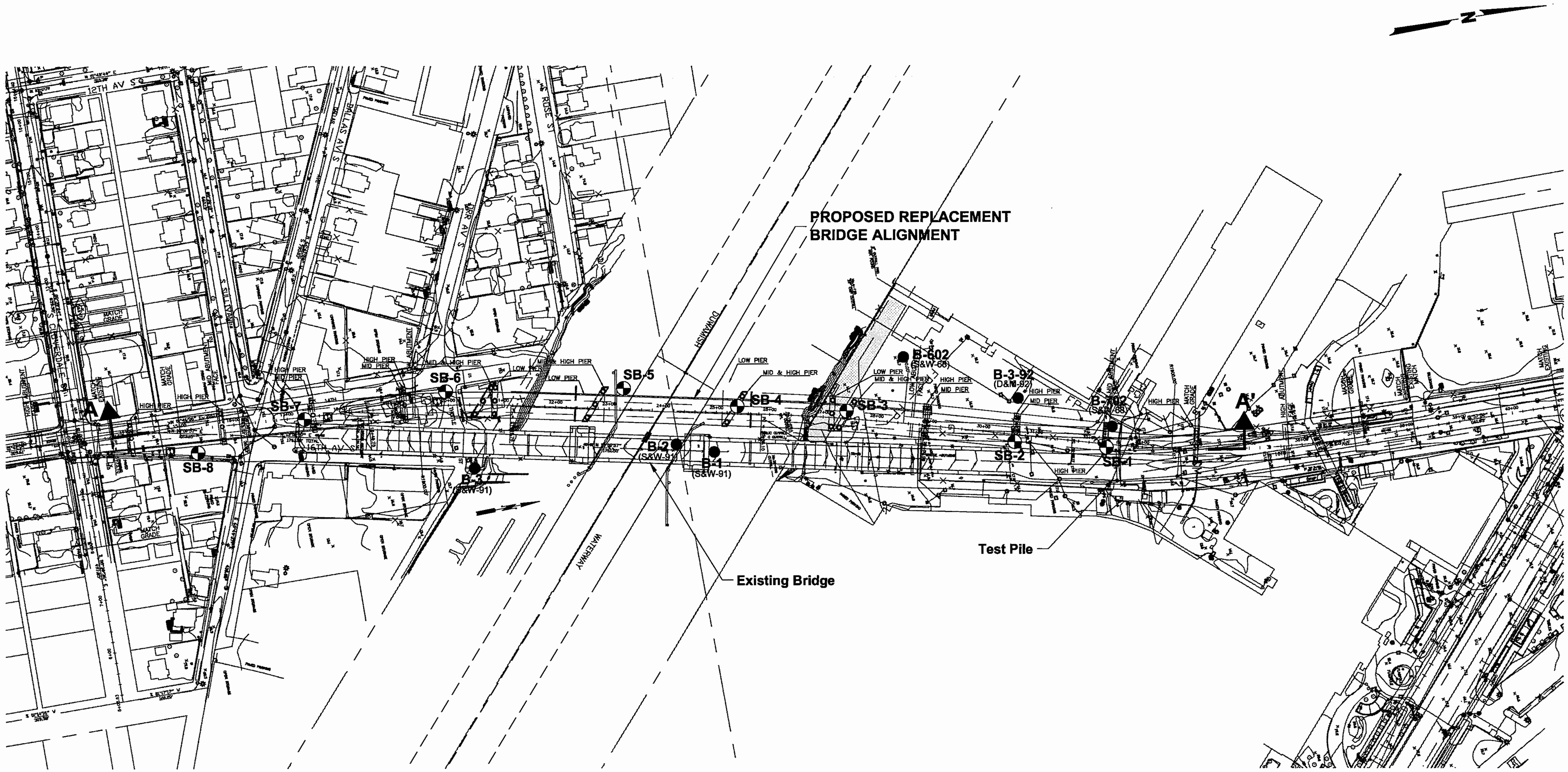
(4) Foundation buckling during liquefaction of subsoils under earthquake loading could be evaluated using LPILE with seismic parameters provided above.

TABLE 3

LPILE^{PLUS} EFFICIENCY FACTORS FOR PILE/ DRILLED SHAFT GROUPS

Spacing	Efficiency Factor, P_m	
	Side	Trailing
1D	0.53	0.58
1.5D	0.67	0.66
2D	0.78	0.73
2.5D	0.89	0.78
3D	0.99	0.83
3.5D	1	0.87
4D	1	0.91
5.5D	1	1.00

Note:
The efficiency factors are based on recommendations presented in a 1998 ENSOFT Seminar (ENSOFT 1998)



0 200 400

Scale in Feet

LEGEND

SB-1 Boring Designation and Approximate Location

B-1 (S&W-91) Previous Boring Designation and Approximate Location (Investigator and Year Completed)

A Generalized Subsurface Profile (See Figure 4)

NOTE

Boring locations taken from electronic file provided by Parsons Brinckerhoff, dated 6-20-03.

South Park Bridge
Seattle, Washington

SITE AND EXPLORATION PLAN

June 2007 | 21-1-09584-101

SHANNON & WILSON, INC.
Geotechnical and Environmental Consultants

FIG. 1

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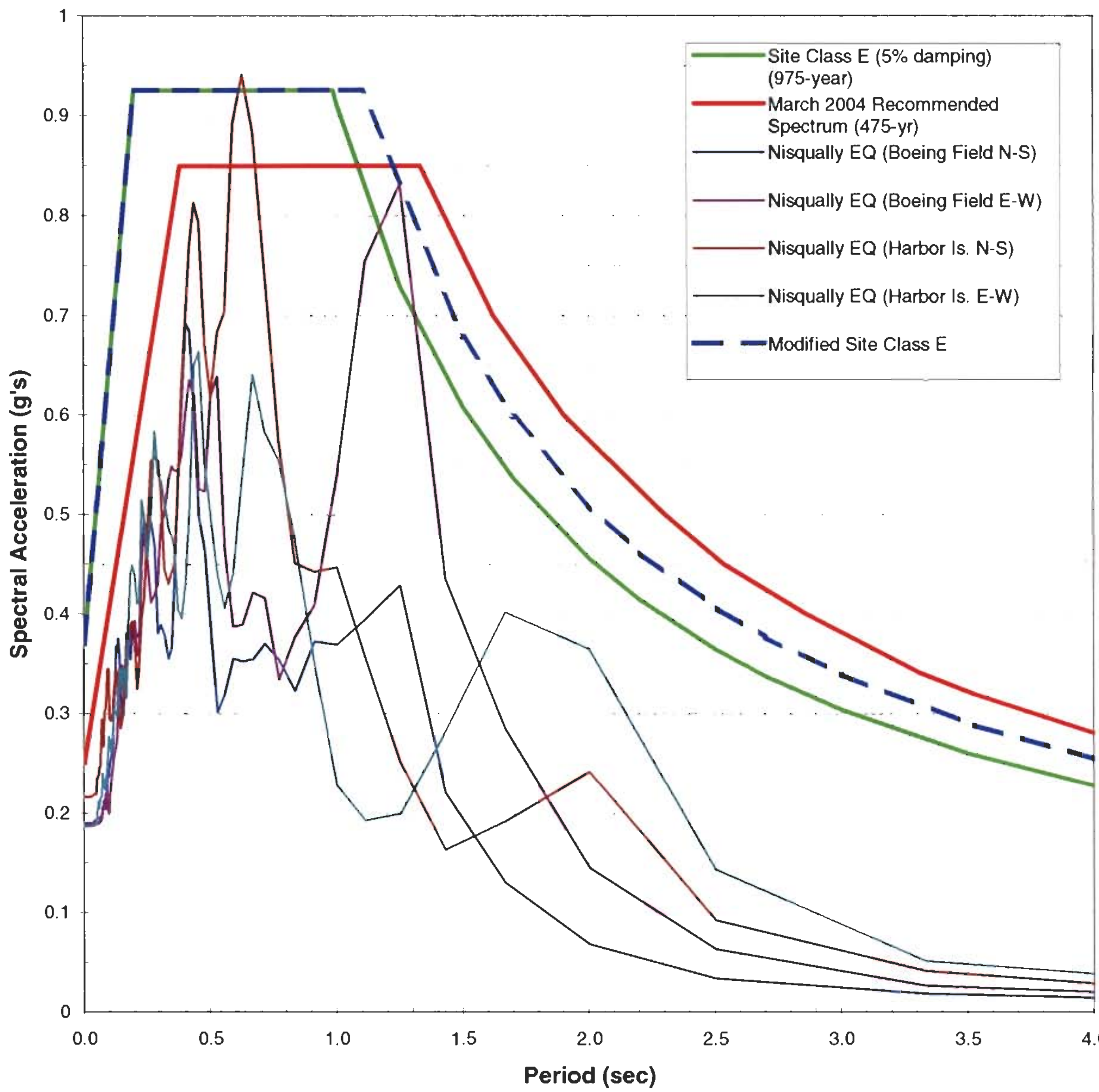

Response Spectrum Parameters[1]

S_S = 1.03 g's	F_a = 0.90
S_1 = 0.35 g's	F_v = 2.64
S_{DS} = 0.93 g's	T_0 = 0.20 sec.
S_{D1} = 0.91 g's	T_S = 0.99 sec.

NOTES

1. Response spectrum parameters are for Site Class E. The Site Class is borderline Site Class D/E. Based on site measurements, the site average shear wave velocity is 627 feet per second (fps). Sites with average shear wave velocities greater than 600 fps are classified as "D"; sites with with average shear wave velocity less than 600 fps are classified as "E."

2. Response spectrum parameters are defined in sections 3.4.1 and 3.4.2 of the Guidelines.

3. Seismic ground shaking hazard in the Recommended 2006 AASHTO LRFD Guildlines are based on 5% probability of exceedance in 50 years (975-year return period).

South Park Bridge
Seattle, Washington

RECOMMENDED 2006 AASHTO LRFD HORIZONTAL RESPONSE SPECTRA SITE CLASS E & NISQUALLY EQ MOTIONS

June 2007 21-1-09584-010

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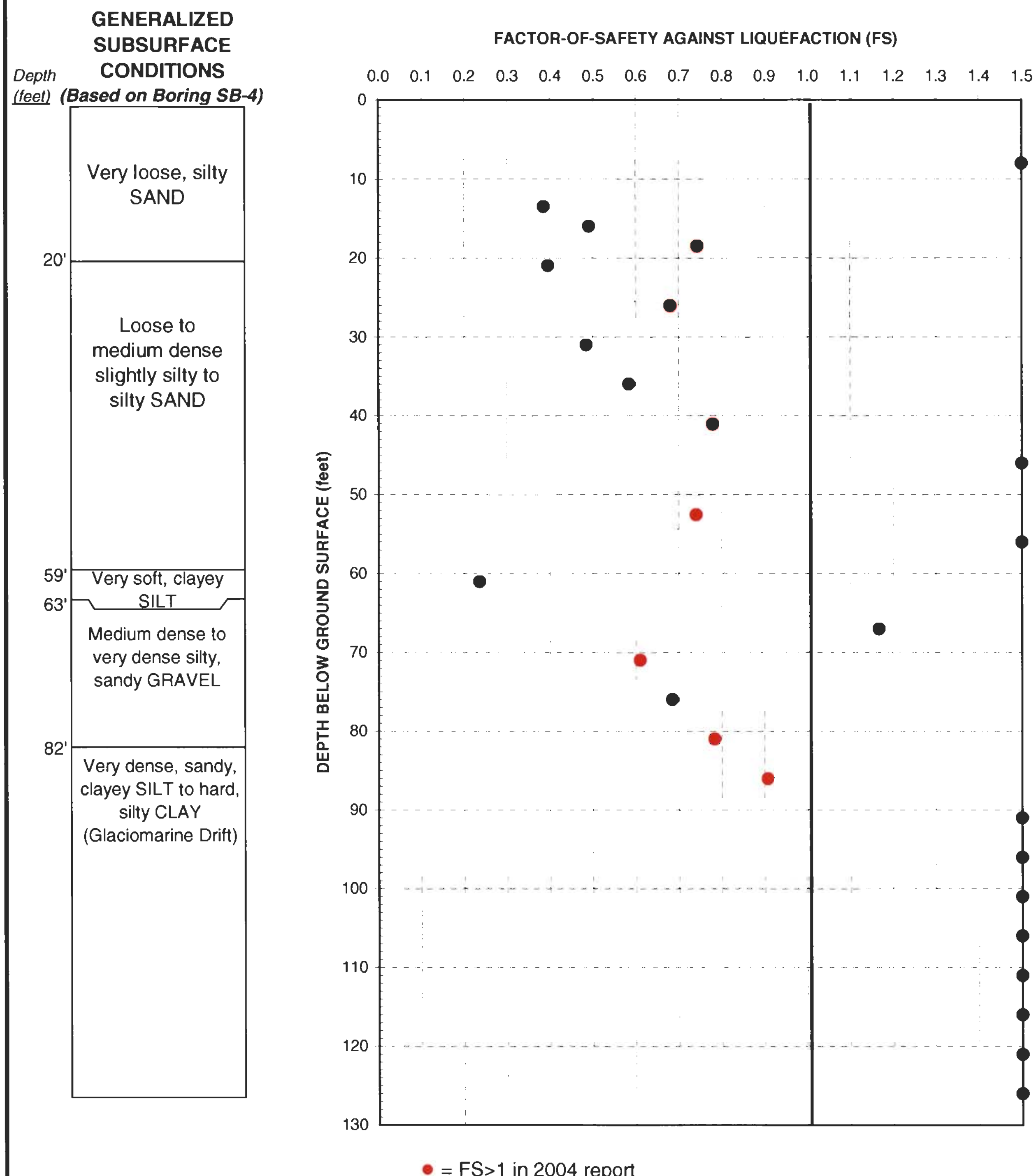
Geotechnical and Environmental Consultants

FIG. 2



NOTES

1. *Reference:* Youd, T.L. and Idriss, I.M., 2001, Liquefaction Resistance of Soils: Summary Report from the 1996 NCEER and 1998 NCEER/ NSF Workshops on Evaluation of Liquefaction Resistance of Soils.
2. The analysis was performed for an earthquake with a magnitude of 6.7 and a peak ground acceleration of 0.45g.
3. The liquefaction resistance of a soil is dependent on its density and fines content. The fines content was estimated based on selected grain-size analyses and engineering judgement.

South Park Bridge
Seattle, Washington

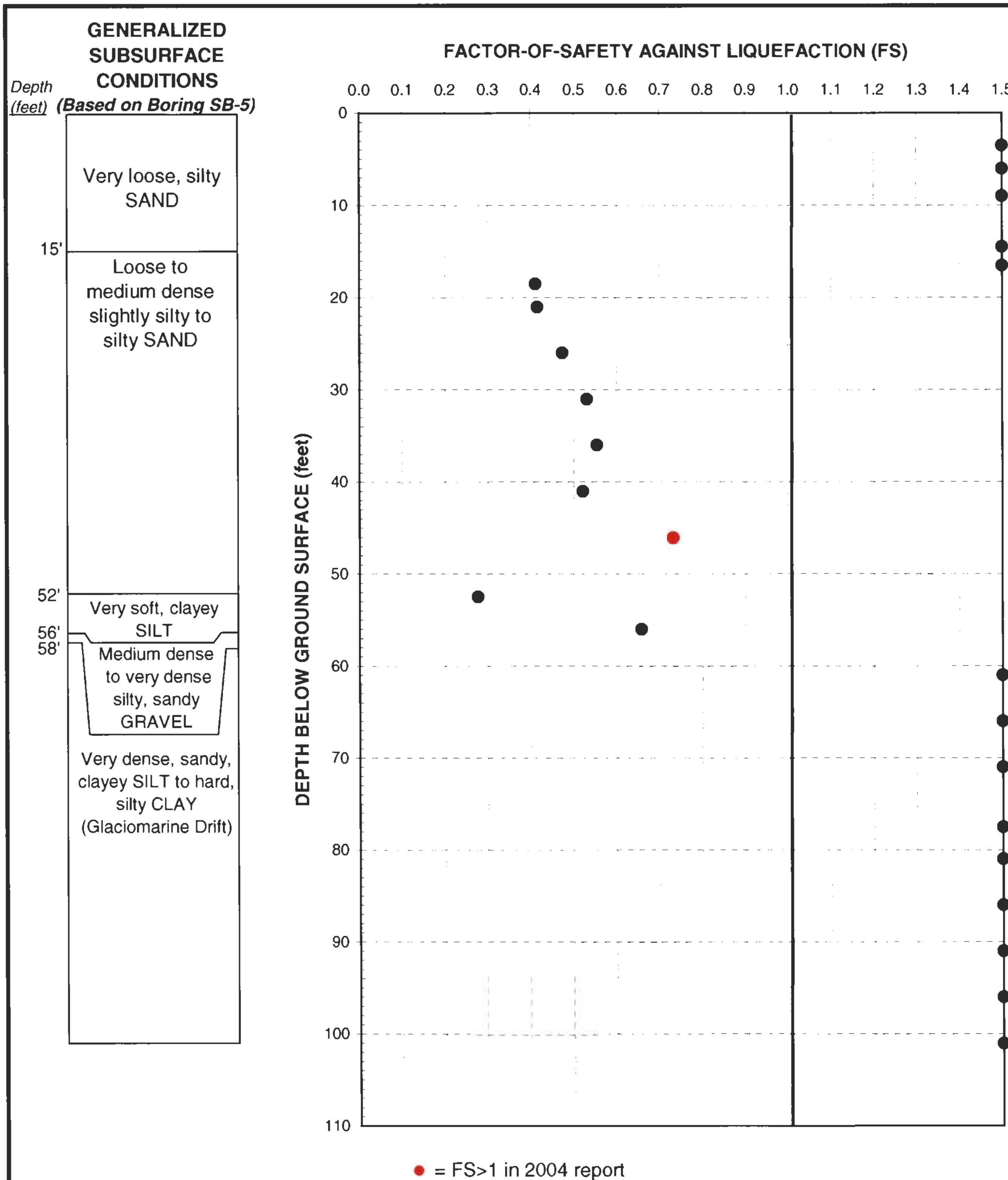
RESULTS OF LIQUEFACTION ANALYSES

BORING SB-4

June 2007 21-1-09584-010

SHANNON & WILSON, INC.
Geotechnical and Environmental Consultants

FIG. 3



NOTES

1. *Reference:* Youd, T.L. and Idriss, I.M., 2001, Liquefaction Resistance of Soils: Summary Report from the 1996 NCEER and 1998 NCEER/NSF Workshops on Evaluation of Liquefaction Resistance of Soils.
2. The analysis was performed for an earthquake with a magnitude of 6.7 and a peak ground acceleration of 0.45g.
3. The liquefaction resistance of a soil is dependent on its density and fines content. The fines content was estimated based on selected grain-size analyses and engineering judgement.

South Park Bridge
Seattle, Washington

RESULTS OF LIQUEFACTION ANALYSES

BORING SB-5

June 2007 21-1-09584-010

SHANNON & WILSON, INC.
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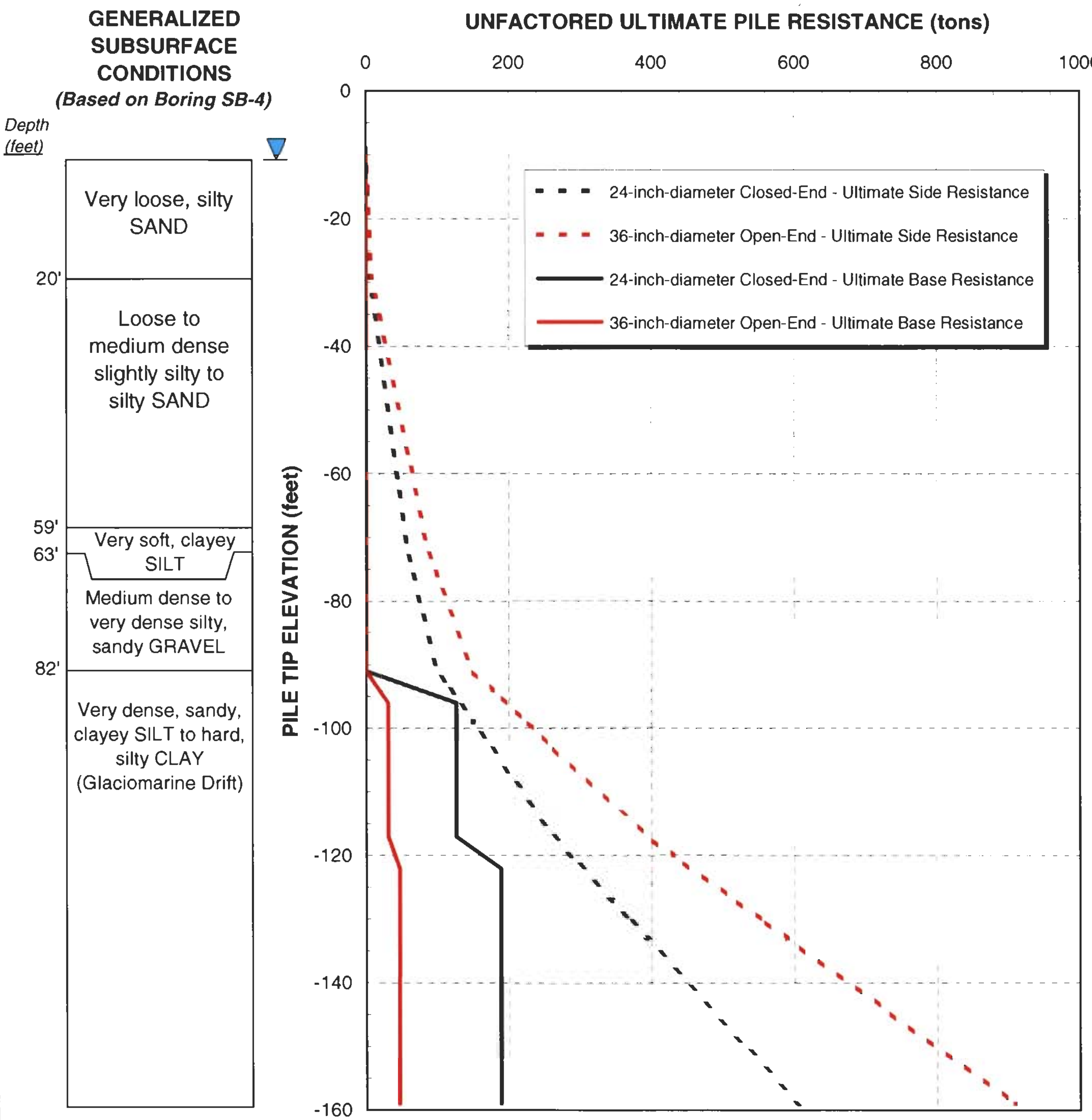
FIG. 4



NOTES

1. Provided unfactored resistances are to be multiplied by the appropriate Resistance Factors (Rf) to determine the factored resistances. See Table 1 for recommended Rf values.
2. Pile group effects are not considered.
3. A conical tip and a reinforced cutting shoe are recommended to achieve adequate penetrations into glacial deposits for the 24- and 36-inch-diameter pipe piles, respectively.
4. Full end bearing is not achieved until the pile/shaft penetrates at least 4 to 5 feet into the bearing layer. The effect of underlying soft layers are considered in the analysis.

South Park Bridge Project
Seattle, Washington

EST. AXIAL CAPACITY - STEEL PIPE PILES
STRENGTH LIMIT
Sta. 25+45, Boring SB-4

June 2007 | 21-1-09584-010

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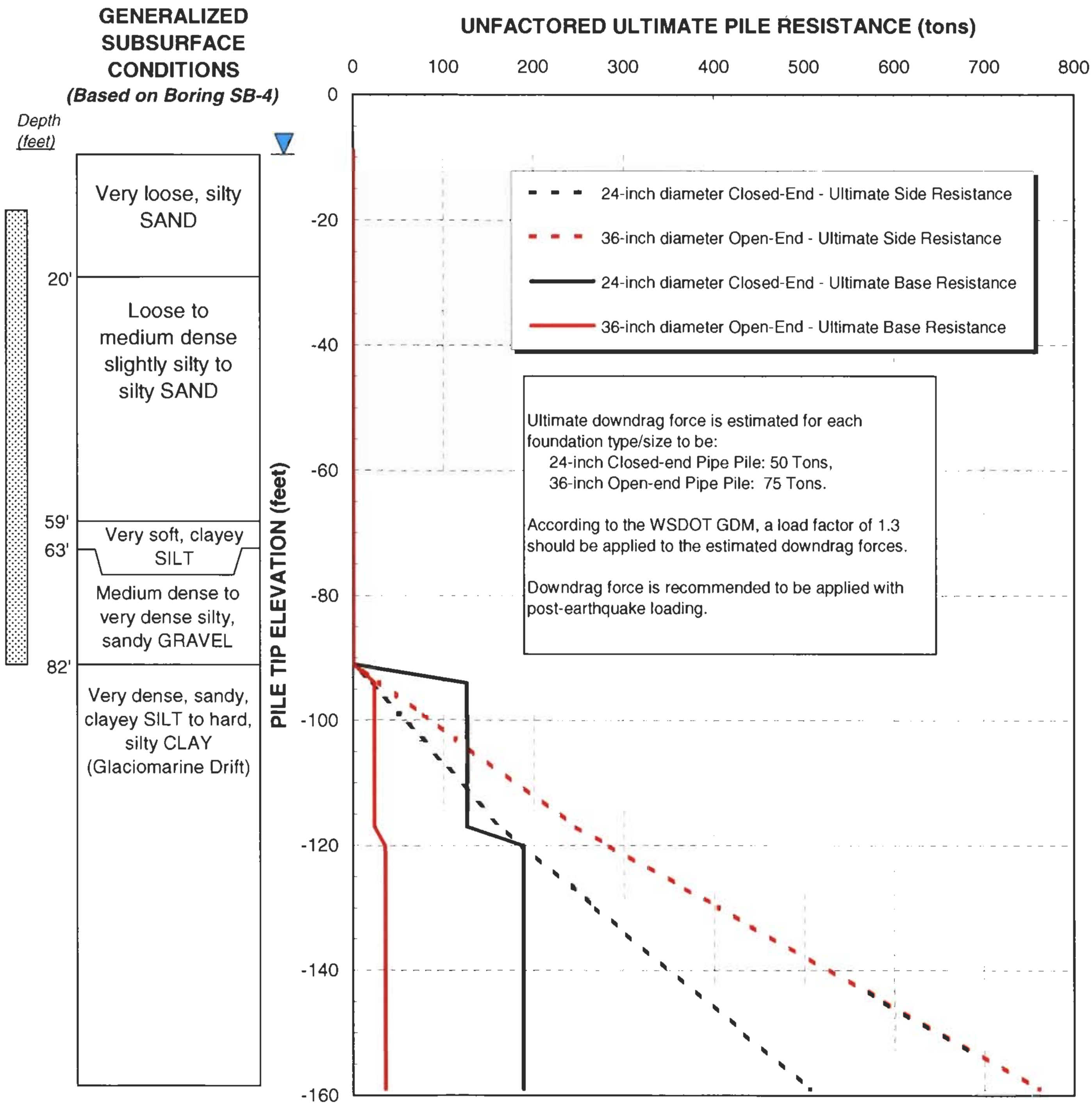
FIG. 5

6/22/2007-Figures-6&8_AC Extreme-Event SB_4(with DD).xls,Plots-lmm/hjs



NOTES

1. Provided unfactored resistances are to be multiplied by the appropriate Resistance Factors (Rf) to determine the factored resistances. See Table 1 for recommended Rf values.
2. Calculations assume seismic loading conditions for a single pile. Pile group effects are not considered.
3. Estimated downdrag forces are shown above.
4. A conical tip and a reinforced cutting shoe are recommended to achieve adequate penetrations into glacial deposits for the 24- and 36-inch-diameter pipe piles, respectively.
5. Indicates liquefied soils during the Design Earthquake event. (975-year Return Period)

South Park Bridge Project
Seattle, Washington

EST. AXIAL CAPACITY - STEEL PIPE PILES
EXTREME EVENT LIMIT
Sta. 25+45, Boring SB-4

June 2007 21-1-09584-010

SHANNON & WILSON, INC.
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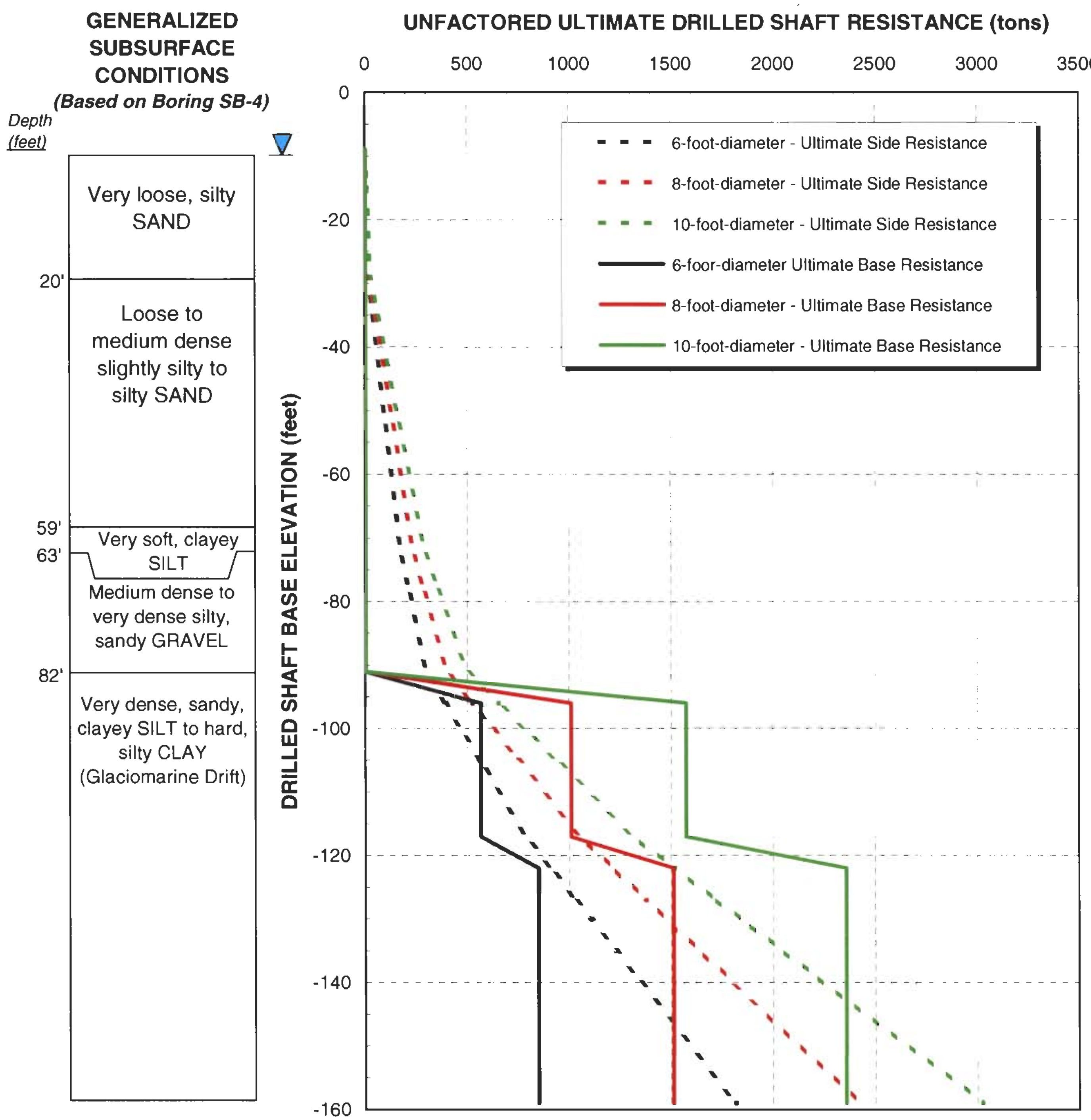
FIG. 6



NOTES

1. Provided unfactored resistances are to be multiplied by the appropriate Resistance Factors (Rf) to determine the factored resistances. See Table 1 for recommended Rf values.
2. Total shaft compressive capacity is a summation of its side and base resistances.
3. Drilled shaft group effects are not considered.
4. Estimated capacities assume that a permanent casing will be left in place for the drilled shaft installation.

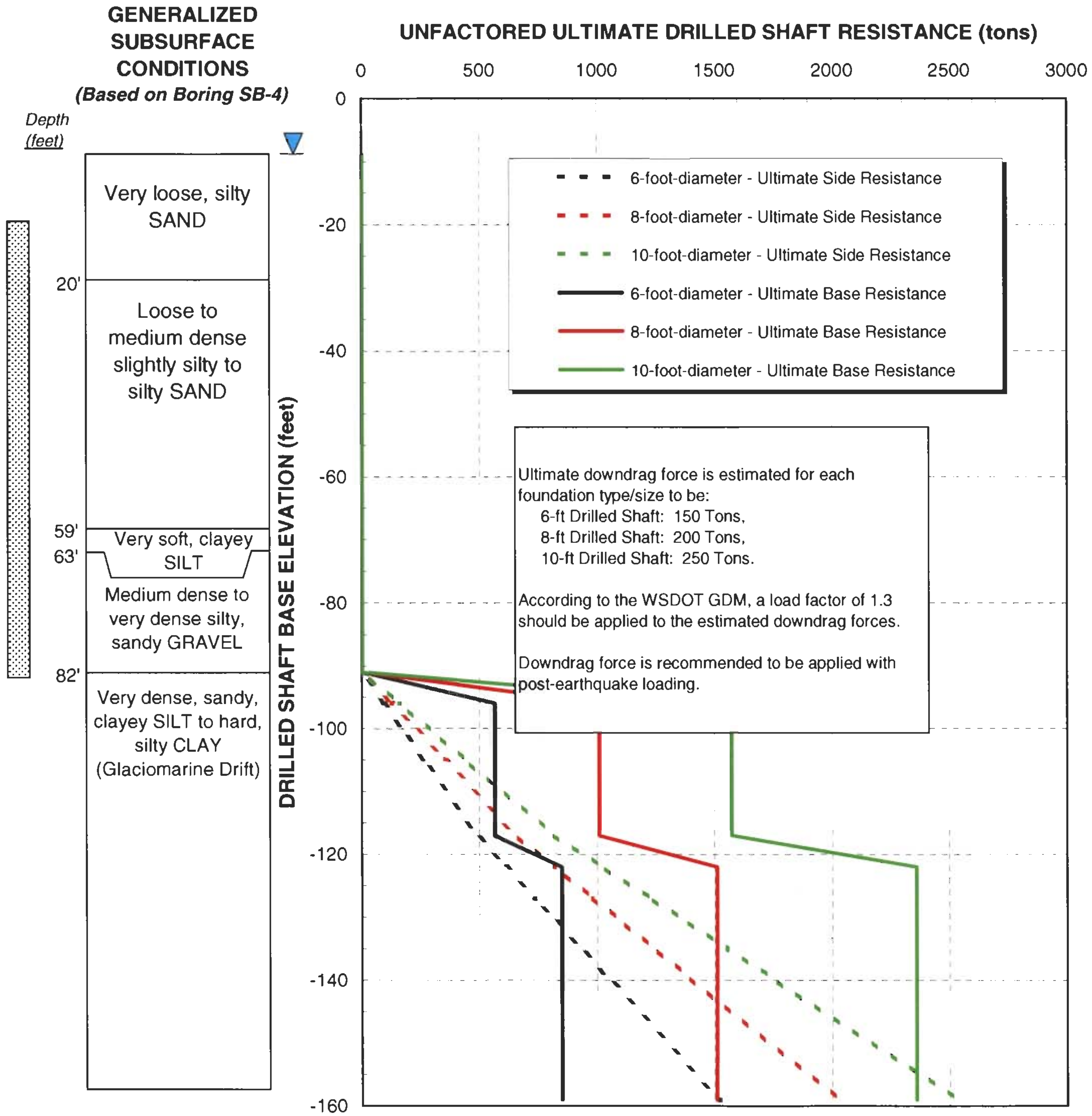
South Park Bridge Project
Seattle, Washington

EST. AXIAL CAPACITY - DRILLED SHAFTS
STRENGTH LIMIT
Sta. 25+45, Boring SB-4

June 2007 21-1-09584-010

SHANNON & WILSON, INC.
Geotechnical and Environmental Consultants

FIG. 7



NOTES

1. Provided unfactored resistances are to be multiplied by the appropriate Resistance Factors (Rf) to determine the factored resistances. See Table 1 for recommended Rf values.
2. Calculations assume seismic loading conditions for a single shaft. Shaft group effects are not considered.
3. Estimated capacities assume that if casing is used, it will be removed after the shaft installation. If, however, the casing is left in place, grouting should be used to fill all potential voids around the casing and the estimated capacities given above should be re-evaluated.
4. Estimated downdrag forces are shown above.
5. Indicates liquefied soils during the Design Earthquake event. (975-year return period)

South Park Bridge Project
Seattle, Washington

EST. AXIAL CAPACITY - DRILLED SHAFTS
EXTREME EVENT LIMIT
Sta. 25+45, Boring SB-4

June 2007 21-1-09584-010

SHANNON & WILSON, INC.
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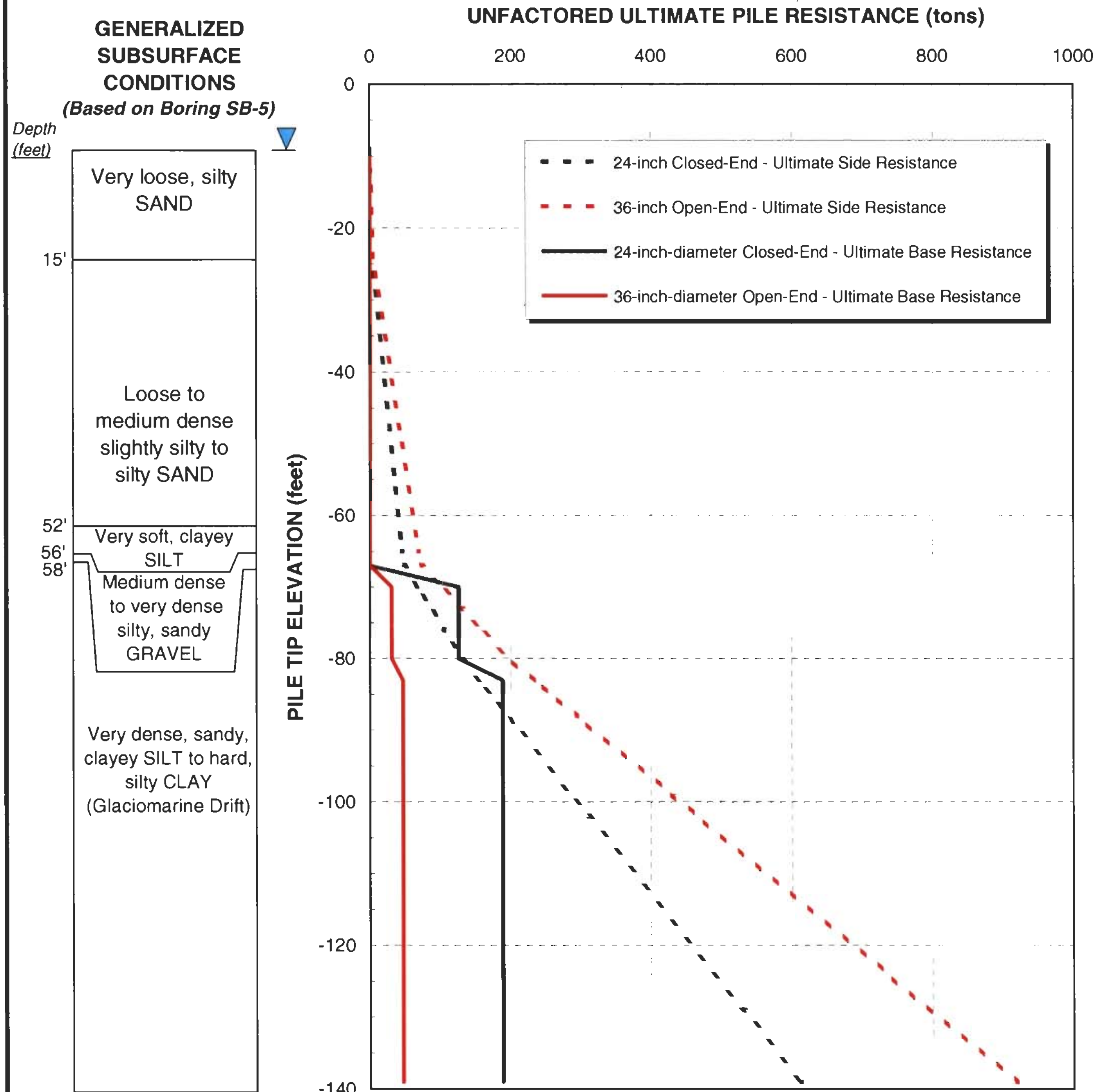
FIG. 8



NOTES

1. Provided unfactored resistances are to be multiplied by the appropriate Resistance Factors (Rf) to determine the factored resistances. See Table 1 for recommended Rf values.
2. Pile group effects are not considered.
3. A conical tip and a reinforced cutting shoe are recommended to achieve adequate penetrations into glacial deposits for the 24- and 36-inch-diameter pipe piles, respectively.
4. Full end bearing is not achieved until the pile/shaft penetrates at least 4 to 5 feet into the bearing layer. The effect of underlying soft layers are considered in the analysis.

South Park Bridge Project
Seattle, Washington

EST. AXIAL CAPACITY - STEEL PIPE PILES
STRENGTH LIMIT
Sta. 23+25, Boring SB-5

June 2007 | 21-1-09584-010

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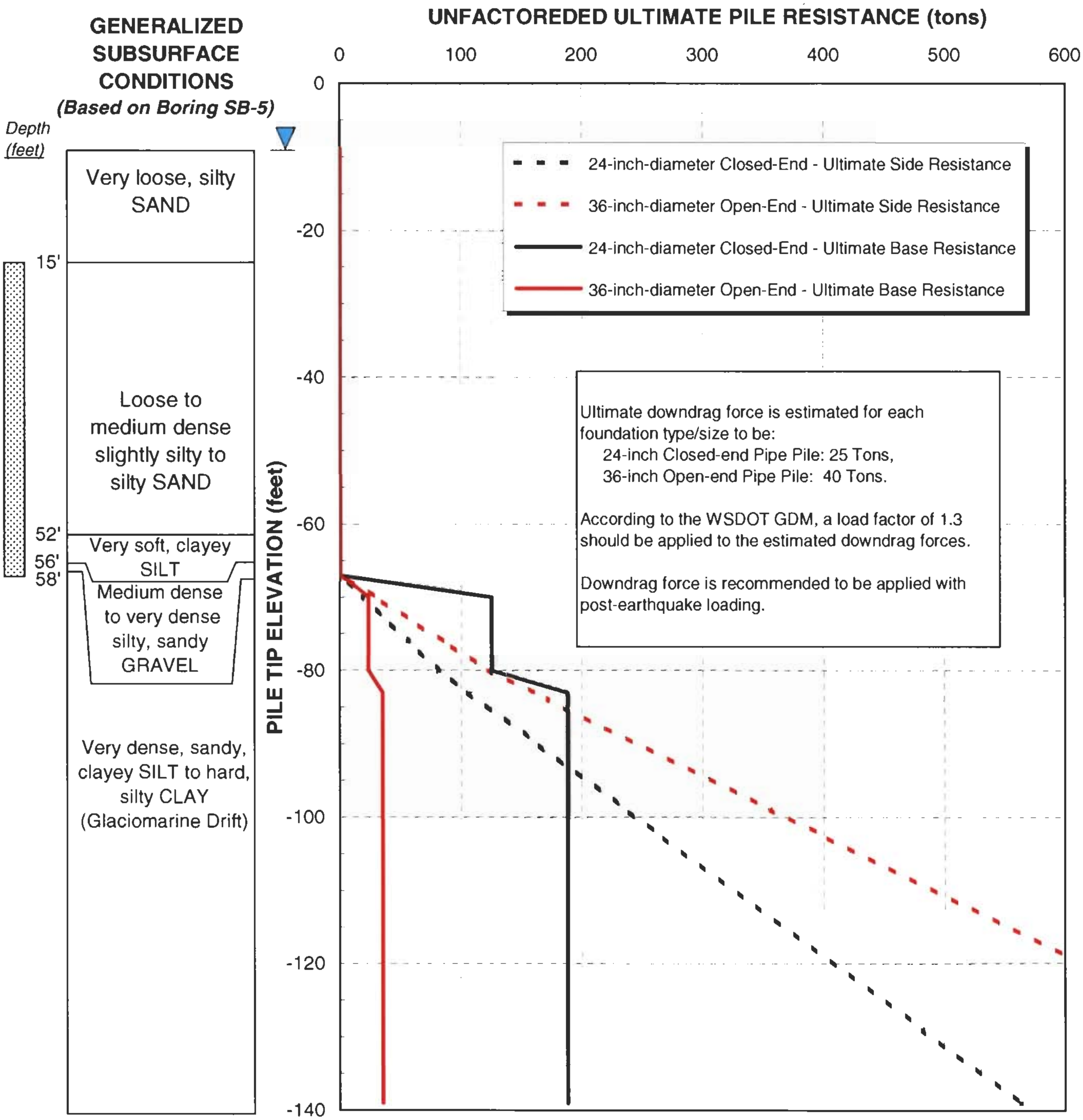
FIG. 9



NOTES

1. Provided unfactored resistances are to be multiplied by the appropriate Resistance Factors (Rf) to determine the factored resistances. See Table 1 for recommended Rf values.
2. Calculations assume seismic loading conditions for a single pile. Pile group effects are not considered.
3. Estimated downdrag forces are shown above.
4. A conical tip and a reinforced cutting shoe are recommended to achieve adequate penetrations into glacial deposits for the 24- and 36-inch-diameter pipe piles, respectively.
5. Indicates liquefied soils during the Design Earthquake event. (975-year Return Period)

South Park Bridge Project
Seattle, Washington

EST. AXIAL CAPACITY - STEEL PIPE PILES
EXTREME EVENT LIMIT
Sta. 23+25, Boring SB-5

June 2007 21-1-09584-010

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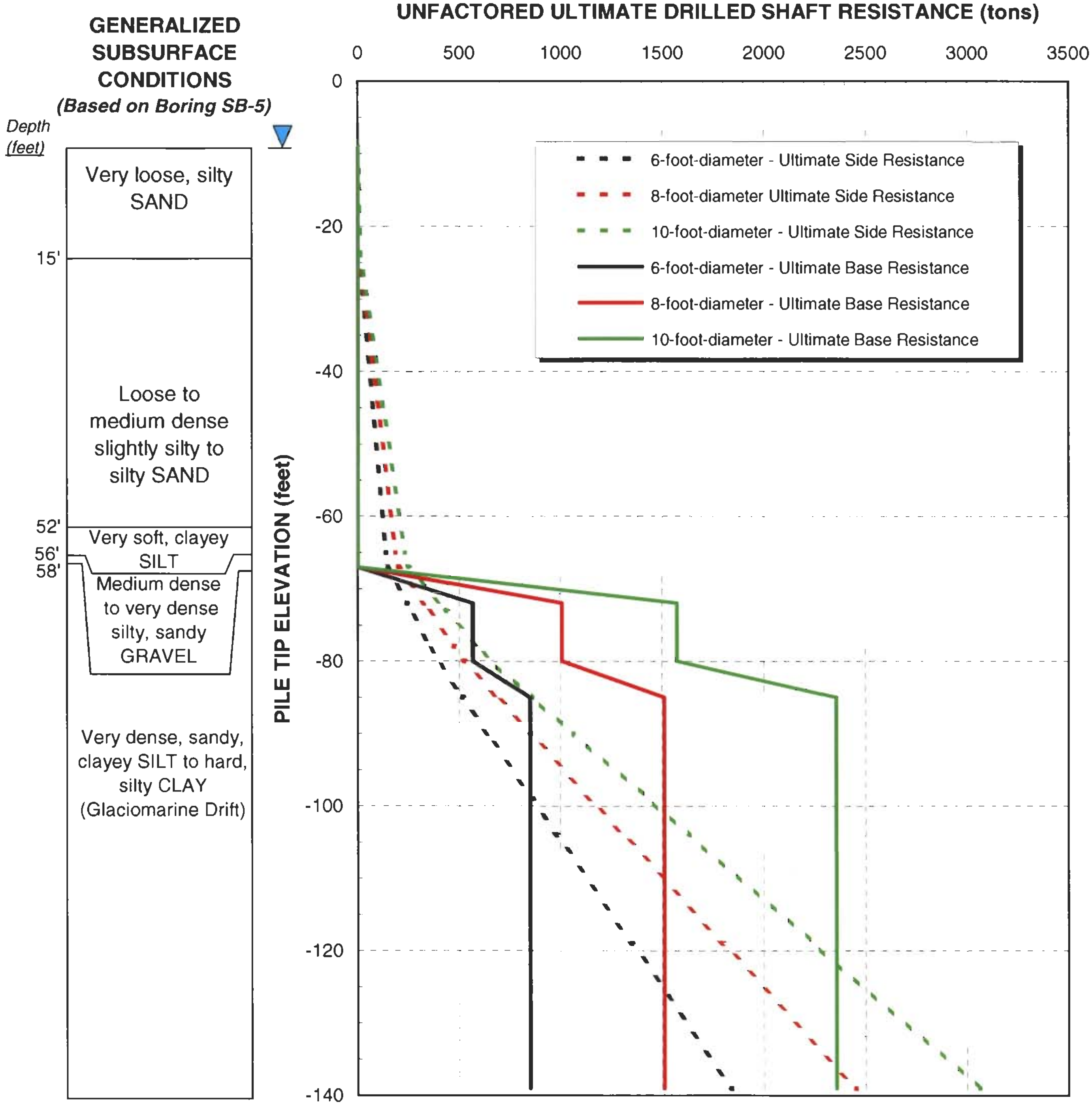
FIG. 10



NOTES

1. Provided unfactored resistances are to be multiplied by the appropriate Resistance Factors (Rf) to determine the factored resistances. See Table 1 for recommended Rf values.
2. Total shaft compressive capacity is a summation of its side and base resistances.
3. Drilled shaft group effects are not considered.
4. Estimated capacities assume that a permanent casing will be left in place for the drilled shaft installation.

South Park Bridge Project
Seattle, Washington

EST. AXIAL CAPACITY - DRILLED SHAFTS
STRENGTH LIMIT
Sta. 23+25, Boring SB-5

June 2007 21-1-09584-010

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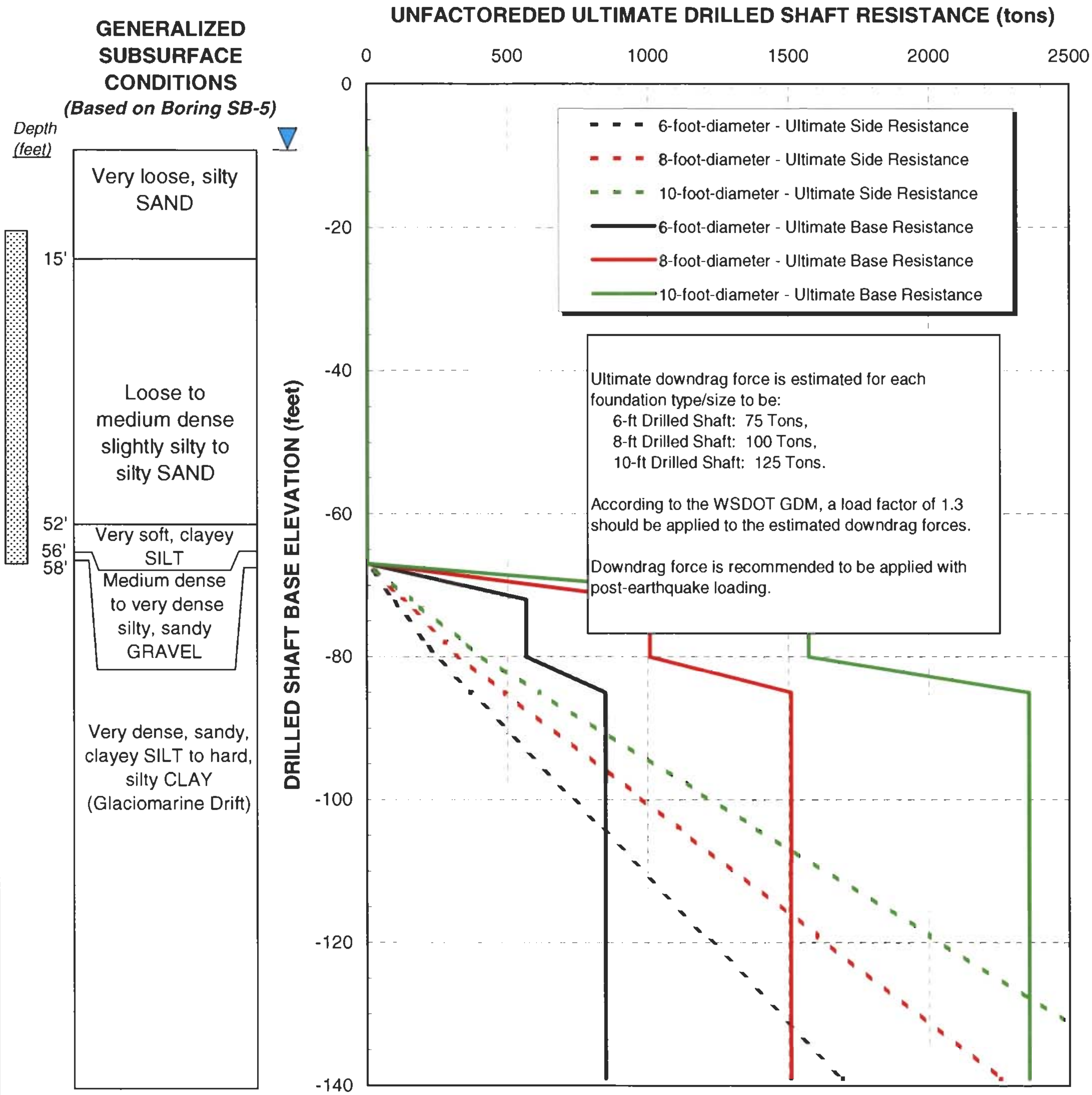
FIG. 11



NOTES

1. Provided unfactored resistances are to be multiplied by the appropriate Resistance Factors (Rf) to determine the factored resistances. See Table 1 for recommended Rf values.
2. Calculations assume seismic loading conditions for a single shaft. Shaft group effects are not considered.
3. Estimated capacities assume that if casing is used, it will be removed after the shaft installation. If, however, the casing is left in place, grouting should be used to fill all potential voids around the casing and the estimated capacities given above should be re-evaluated.
4. Estimated downdrag forces are shown above.
5. Indicates liquefied soils during the Design Earthquake event. (975-year return period)

South Park Bridge Project
Seattle, Washington

EST. AXIAL CAPACITY - DRILLED SHAFTS
EXTREME EVENT LIMIT
Sta. 23+25, Boring SB-5

June 2007 | 21-1-09584-010

SHANNON & WILSON, INC.
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FIG. 12

APPENDIX A

DATA FOR LOAD SETTLEMENT CURVES

APPENDIX A

DATA FOR LOAD SETTLEMENT CURVES

TABLE OF CONTENTS

LIST OF TABLES

Table No.

TABLE A-1

APILE LOAD-SETTLEMENT RESULTS

Boring SB-4, 24-Inch-Diameter Steel Pipe Pile, Closed-end (Static Condition)

T-Z CURVE NO.	DEPTH TO CURVE FT.	LOAD TRANSFER PSI	PILE MOVEMENT IN.
1	0.00		
		0.00	0.00
		0.06	0.01
		0.11	0.02
		0.22	0.04
		0.33	0.06
		0.44	0.08
		0.50	0.09
		0.56	0.10
		0.56	0.50
		0.56	2.00
2	10.03		
		0.00	0.00
		0.06	0.01
		0.11	0.02
		0.22	0.04
		0.33	0.06
		0.44	0.08
		0.50	0.09
		0.56	0.10
		0.56	0.50
		0.56	2.00
3	19.96		
		0.00	0.00
		0.06	0.01
		0.11	0.02
		0.22	0.04
		0.33	0.06
		0.44	0.08
		0.50	0.09
		0.56	0.10
		0.56	0.50
		0.56	2.00
4	20.00		
		0.00	0.00
		0.26	0.01
		0.53	0.02
		1.06	0.04
		1.58	0.06
		2.11	0.08
		2.38	0.09
		2.64	0.10
		2.64	0.50
		2.64	2.00

TOP LOAD KIP	TOP MOVEMENT IN.
6.62	0.01
67.88	0.08
249.00	0.37
369.20	0.62
746.90	1.55
829.10	1.81
930.20	2.48
997.90	3.16
1070.00	4.35

TIP LOAD KIP	TIP MOVEMENT IN.
0.00	0.00
23.56	0.01
47.12	0.02
94.25	0.05
188.50	0.31
282.70	1.01
339.30	1.75
377.00	2.40
377.00	3.60
377.00	4.80

TABLE A-1

APILE LOAD-SETTLEMENT RESULTS

Boring SB-4, 24-Inch-Diameter Steel Pipe Pile, Closed-end (Static Condition)

T-Z CURVE NO.	DEPTH TO CURVE FT.	LOAD TRANSFER PSI	PILE MOVEMENT IN.
5	41.52		
		0.00	0.00
		0.26	0.01
		0.53	0.02
		1.06	0.04
		1.58	0.06
		2.11	0.08
		2.38	0.09
		2.64	0.10
		2.64	0.50
		2.64	2.00
6	62.96		
		0.00	0.00
		0.26	0.01
		0.53	0.02
		1.06	0.04
		1.58	0.06
		2.11	0.08
		2.38	0.09
		2.64	0.10
		2.64	0.50
		2.64	2.00
7	63.00		
		0.00	0.00
		0.49	0.01
		0.97	0.02
		1.94	0.04
		2.92	0.06
		3.89	0.08
		4.38	0.09
		4.86	0.10
		4.86	0.50
		4.86	2.00
8	72.53		
		0.00	0.00
		0.49	0.01
		0.97	0.02
		1.94	0.04
		2.92	0.06
		3.89	0.08
		4.38	0.09
		4.86	0.10
		4.86	0.50
		4.86	2.00

TABLE A-1

APILE LOAD-SETTLEMENT RESULTS

Boring SB-4, 24-Inch-Diameter Steel Pipe Pile, Closed-end (Static Condition)

T-Z CURVE NO.	DEPTH TO CURVE FT.	LOAD TRANSFER PSI	PILE MOVEMENT IN.
9	81.96		
		0.00	0.00
		0.49	0.01
		0.97	0.02
		1.94	0.04
		2.92	0.06
		3.89	0.08
		4.38	0.09
		4.86	0.10
		4.86	0.50
		4.86	2.00
10	82.00		
		0.00	0.00
		1.39	0.01
		2.78	0.02
		5.56	0.04
		8.33	0.06
		11.11	0.08
		12.50	0.09
		13.89	0.10
		13.89	0.50
		13.89	2.00
11	84.53		
		0.00	0.00
		1.39	0.01
		2.78	0.02
		5.56	0.04
		8.33	0.06
		11.11	0.08
		12.50	0.09
		13.89	0.10
		13.89	0.50
		13.89	2.00
12	86.96		
		0.00	0.00
		1.39	0.01
		2.78	0.02
		5.56	0.04
		8.33	0.06
		11.11	0.08
		12.50	0.09
		13.89	0.10
		13.89	0.50
		13.89	2.00

TABLE A-1

APILE LOAD-SETTLEMENT RESULTS

Boring SB-4, 24-Inch-Diameter Steel Pipe Pile, Closed-end (Static Condition)

T-Z CURVE NO.	DEPTH TO CURVE FT.	LOAD TRANSFER PSI	PILE MOVEMENT IN.
13	87.00		
		0.00	0.00
		1.39	0.01
		2.78	0.02
		5.56	0.04
		8.33	0.06
		11.11	0.08
		12.50	0.09
		13.89	0.10
		13.89	0.50
		13.89	2.00
14	97.53		
		0.00	0.00
		1.39	0.01
		2.78	0.02
		5.56	0.04
		8.33	0.06
		11.11	0.08
		12.50	0.09
		13.89	0.10
		13.89	0.50
		13.89	2.00
15	108.00		
		0.00	0.00
		1.39	0.01
		2.78	0.02
		5.56	0.04
		8.33	0.06
		11.11	0.08
		12.50	0.09
		13.89	0.10
		13.89	0.50
		13.89	2.00
16	108.00		
		0.00	0.00
		1.81	0.01
		3.61	0.02
		7.22	0.04
		10.83	0.06
		14.44	0.08
		16.25	0.09
		18.06	0.10
		18.06	0.50
		18.06	2.00

TABLE A-1

APILE LOAD-SETTLEMENT RESULTS

Boring SB-4, 24-Inch-Diameter Steel Pipe Pile, Closed-end (Static Condition)

T-Z CURVE NO.	DEPTH TO CURVE FT.	LOAD TRANSFER PSI	PILE MOVEMENT IN.
17	113.00		
		0.00	0.00
		1.81	0.01
		3.61	0.02
		7.22	0.04
		10.83	0.06
		14.44	0.08
		16.25	0.09
		18.06	0.10
		18.06	0.50
		18.06	2.00
18	118.00		
		0.00	0.00
		1.81	0.01
		3.61	0.02
		7.22	0.04
		10.83	0.06
		14.44	0.08
		16.25	0.09
		18.06	0.10
		18.06	0.50
		18.06	2.00
19	118.00		
		0.00	0.00
		1.81	0.01
		3.61	0.02
		7.22	0.04
		10.83	0.06
		14.44	0.08
		16.25	0.09
		18.06	0.10
		18.06	0.50
		18.06	2.00
20	121.50		
		0.00	0.00
		1.81	0.01
		3.61	0.02
		7.22	0.04
		10.83	0.06
		14.44	0.08
		16.25	0.09
		18.06	0.10
		18.06	0.50
		18.06	2.00

TABLE A-1

APILE LOAD-SETTLEMENT RESULTS

Boring SB-4, 24-Inch-Diameter Steel Pipe Pile, Closed-end (Static Condition)

T-Z CURVE NO.	DEPTH TO CURVE FT.	LOAD TRANSFER PSI	PILE MOVEMENT IN.
21	125.00		
		0.00	0.00
		1.81	0.01
		3.61	0.02
		7.22	0.04
		10.83	0.06
		14.44	0.08
		16.25	0.09
		18.06	0.10
		18.06	0.50
		18.06	2.00

TABLE A-2

APILE LOAD-SETTLEMENT RESULTS

Boring SB-4, 24-Inch-Diameter Steel Pipe Pile, Closed-end (Seismic Condition)

T-Z CURVE NO.	DEPTH TO CURVE FT.	LOAD TRANSFER PSI	PILE MOVEMENT IN.
1	0.00		
		0.00	0.00
		0.00	0.01
		0.00	0.02
		0.00	0.04
		0.00	0.06
		0.00	0.08
		0.00	0.09
		0.00	0.10
		0.00	0.50
		0.00	2.00
2	10.03		
		0.00	0.00
		0.00	0.01
		0.00	0.02
		0.00	0.04
		0.00	0.06
		0.00	0.08
		0.00	0.09
		0.00	0.10
		0.00	0.50
		0.00	2.00
3	19.96		
		0.00	0.00
		0.00	0.01
		0.00	0.02
		0.00	0.04
		0.00	0.06
		0.00	0.08
		0.00	0.09
		0.00	0.10
		0.00	0.50
		0.00	2.00
4	20.00		
		0.00	0.00
		0.00	0.01
		0.00	0.02
		0.00	0.04
		0.00	0.06
		0.00	0.08
		0.00	0.09
		0.00	0.10
		0.00	0.50
		0.00	2.00

TOP LOAD KIP	TOP MOVEMENT IN.
1.69	0.00
17.15	0.04
87.60	0.20
175.20	0.39
549.10	1.32
631.30	1.58
732.40	2.25
800.10	2.93
872.20	4.12

TIP LOAD KIP	TIP MOVEMENT IN.
0.00	0.00
23.56	0.01
47.12	0.02
94.25	0.05
188.50	0.31
282.70	1.01
339.30	1.75
377.00	2.40
377.00	3.60
377.00	4.80

TABLE A-2

APILE LOAD-SETTLEMENT RESULTS

Boring SB-4, 24-Inch-Diameter Steel Pipe Pile, Closed-end (Seismic Condition)

T-Z CURVE NO.	DEPTH TO CURVE FT.	LOAD TRANSFER PSI	PILE MOVEMENT IN.
5	41.52		
		0.00	0.00
		0.00	0.01
		0.00	0.02
		0.00	0.04
		0.00	0.06
		0.00	0.08
		0.00	0.09
		0.00	0.10
		0.00	0.50
		0.00	2.00
6	62.96		
		0.00	0.00
		0.00	0.01
		0.00	0.02
		0.00	0.04
		0.00	0.06
		0.00	0.08
		0.00	0.09
		0.00	0.10
		0.00	0.50
		0.00	2.00
7	63.00		
		0.00	0.00
		0.00	0.01
		0.00	0.02
		0.00	0.04
		0.00	0.06
		0.00	0.08
		0.00	0.09
		0.00	0.10
		0.00	0.50
		0.00	2.00
8	72.53		
		0.00	0.00
		0.00	0.01
		0.00	0.02
		0.00	0.04
		0.00	0.06
		0.00	0.08
		0.00	0.09
		0.00	0.10
		0.00	0.50
		0.00	2.00

TABLE A-2

APILE LOAD-SETTLEMENT RESULTS

Boring SB-4, 24-Inch-Diameter Steel Pipe Pile, Closed-end (Seismic Condition)

T-Z CURVE NO.	DEPTH TO CURVE FT.	LOAD TRANSFER PSI	PILE MOVEMENT IN.
9	81.96		
		0.00	0.00
		0.00	0.01
		0.00	0.02
		0.00	0.04
		0.00	0.06
		0.00	0.08
		0.00	0.09
		0.00	0.10
		0.00	0.50
		0.00	2.00
10	82.00		
		0.00	0.00
		1.39	0.01
		2.78	0.02
		5.56	0.04
		8.33	0.06
		11.11	0.08
		12.50	0.09
		13.89	0.10
		13.89	0.50
		13.89	2.00
11	84.53		
		0.00	0.00
		1.39	0.01
		2.78	0.02
		5.56	0.04
		8.33	0.06
		11.11	0.08
		12.50	0.09
		13.89	0.10
		13.89	0.50
		13.89	2.00
12	86.96		
		0.00	0.00
		1.39	0.01
		2.78	0.02
		5.56	0.04
		8.33	0.06
		11.11	0.08
		12.50	0.09
		13.89	0.10
		13.89	0.50
		13.89	2.00

TABLE A-2

APILE LOAD-SETTLEMENT RESULTS

Boring SB-4, 24-Inch-Diameter Steel Pipe Pile, Closed-end (Seismic Condition)

T-Z CURVE NO.	DEPTH TO CURVE FT.	LOAD TRANSFER PSI	PILE MOVEMENT IN.
13	87.00		
		0.00	0.00
		1.39	0.01
		2.78	0.02
		5.56	0.04
		8.33	0.06
		11.11	0.08
		12.50	0.09
		13.89	0.10
		13.89	0.50
		13.89	2.00
14	97.53		
		0.00	0.00
		1.39	0.01
		2.78	0.02
		5.56	0.04
		8.33	0.06
		11.11	0.08
		12.50	0.09
		13.89	0.10
		13.89	0.50
		13.89	2.00
15	108.00		
		0.00	0.00
		1.39	0.01
		2.78	0.02
		5.56	0.04
		8.33	0.06
		11.11	0.08
		12.50	0.09
		13.89	0.10
		13.89	0.50
		13.89	2.00
16	108.00		
		0.00	0.00
		1.81	0.01
		3.61	0.02
		7.22	0.04
		10.83	0.06
		14.44	0.08
		16.25	0.09
		18.06	0.10
		18.06	0.50
		18.06	2.00

TABLE A-2

APILE LOAD-SETTLEMENT RESULTS

Boring SB-4, 24-Inch-Diameter Steel Pipe Pile, Closed-end (Seismic Condition)

T-Z CURVE NO.	DEPTH TO CURVE FT.	LOAD TRANSFER PSI	PILE MOVEMENT IN.
17	113.00		
		0.00	0.00
		1.81	0.01
		3.61	0.02
		7.22	0.04
		10.83	0.06
		14.44	0.08
		16.25	0.09
		18.06	0.10
		18.06	0.50
		18.06	2.00
18	118.00		
		0.00	0.00
		1.81	0.01
		3.61	0.02
		7.22	0.04
		10.83	0.06
		14.44	0.08
		16.25	0.09
		18.06	0.10
		18.06	0.50
		18.06	2.00
19	118.00		
		0.00	0.00
		1.81	0.01
		3.61	0.02
		7.22	0.04
		10.83	0.06
		14.44	0.08
		16.25	0.09
		18.06	0.10
		18.06	0.50
		18.06	2.00
20	121.50		
		0.00	0.00
		1.81	0.01
		3.61	0.02
		7.22	0.04
		10.83	0.06
		14.44	0.08
		16.25	0.09
		18.06	0.10
		18.06	0.50
		18.06	2.00

TABLE A-2

APILE LOAD-SETTLEMENT RESULTS

Boring SB-4, 24-Inch-Diameter Steel Pipe Pile, Closed-end (Seismic Condition)

T-Z CURVE NO.	DEPTH TO CURVE FT.	LOAD TRANSFER PSI	PILE MOVEMENT IN.
21	125.00		
		0.00	0.00
		1.81	0.01
		3.61	0.02
		7.22	0.04
		10.83	0.06
		14.44	0.08
		16.25	0.09
		18.06	0.10
		18.06	0.50
		18.06	2.00

TABLE A-3

SHAFT LOAD-SETTLEMENT RESULTS

Boring SB-4, 8-foot-Diameter Drilled Shaft (Static Condition)

T-Z CURVE NO.	DEPTH TO CURVE FT.	LOAD TRANSFER PSI	PILE MOVEMENT IN.
1	1.00		
		0.00	0.00
		0.03	0.10
		0.05	0.19
		0.07	0.29
		0.08	0.48
		0.08	0.58
		0.09	0.77
		0.09	1.15
		0.09	1.54
		0.09	9.60
2	5.75		
		0.00	0.00
		0.18	0.10
		0.30	0.19
		0.38	0.29
		0.45	0.48
		0.48	0.58
		0.50	0.77
		0.51	1.15
		0.50	1.54
		0.50	9.60
3	10.50		
		0.00	0.00
		0.19	0.10
		0.32	0.19
		0.40	0.29
		0.48	0.48
		0.51	0.58
		0.53	0.77
		0.54	1.15
		0.53	1.54
		0.53	9.60
4	15.25		
		0.00	0.00
		0.19	0.10
		0.32	0.19
		0.40	0.29
		0.48	0.48
		0.51	0.58
		0.53	0.77
		0.54	1.15
		0.53	1.54
		0.53	9.60

TOP LOAD KIP	TOP MOVEMENT IN.
1.09	0.00
10.89	0.00
27.22	0.00
54.44	0.01
81.64	0.01
108.86	0.01
272.20	0.04
544.60	0.07
816.60	0.11
1060.00	0.14
2036.00	0.34
2740.00	0.62
3090.00	0.89
3278.00	1.15
5286.00	9.86

TIP LOAD KIP	TIP MOVEMENT IN.
0.07	0.00
0.67	0.00
1.68	0.00
3.36	0.01
5.04	0.01
6.72	0.01
16.80	0.03
33.60	0.05
50.40	0.08
67.20	0.10
168.00	0.25
335.60	0.50
498.80	0.75
657.40	1.00
2692.00	9.60

TABLE A-3

SHAFT LOAD-SETTLEMENT RESULTS

Boring SB-4, 8-foot-Diameter Drilled Shaft (Static Condition)

T-Z CURVE NO.	DEPTH TO CURVE FT.	LOAD TRANSFER PSI	PILE MOVEMENT IN.
5	20.00		
		0.00	0.00
		0.19	0.10
		0.32	0.19
		0.40	0.29
		0.48	0.48
		0.51	0.58
		0.53	0.77
		0.54	1.15
		0.53	1.54
		0.53	9.60
6	21.00		
		0.00	0.00
		0.75	0.10
		1.31	0.19
		1.62	0.29
		1.94	0.48
		2.05	0.58
		2.14	0.77
		2.17	1.15
		2.16	1.54
		2.14	9.60
7	30.75		
		0.00	0.00
		0.88	0.10
		1.53	0.19
		1.90	0.29
		2.27	0.48
		2.40	0.58
		2.51	0.77
		2.55	1.15
		2.53	1.54
		2.51	9.60
8	41.50		
		0.00	0.00
		0.88	0.10
		1.53	0.19
		1.90	0.29
		2.27	0.48
		2.40	0.58
		2.51	0.77
		2.55	1.15
		2.53	1.54
		2.51	9.60

TABLE A-3

SHAFT LOAD-SETTLEMENT RESULTS

Boring SB-4, 8-foot-Diameter Drilled Shaft (Static Condition)

T-Z CURVE NO.	DEPTH TO CURVE FT.	LOAD TRANSFER PSI	PILE MOVEMENT IN.
9	52.25		
		0.00	0.00
		0.88	0.10
		1.53	0.19
		1.90	0.29
		2.27	0.48
		2.40	0.58
		2.51	0.77
		2.55	1.15
		2.53	1.54
		2.51	9.60
10	63.00		
		0.00	0.00
		0.88	0.10
		1.53	0.19
		1.90	0.29
		2.27	0.48
		2.40	0.58
		2.51	0.77
		2.55	1.15
		2.53	1.54
		2.51	9.60
11	64.00		
		0.00	0.00
		1.63	0.10
		2.82	0.19
		3.50	0.29
		4.18	0.48
		4.42	0.58
		4.62	0.77
		4.69	1.15
		4.67	1.54
		4.62	9.60
12	67.75		
		0.00	0.00
		1.63	0.10
		2.82	0.19
		3.50	0.29
		4.18	0.48
		4.42	0.58
		4.62	0.77
		4.69	1.15
		4.67	1.54
		4.62	9.60

TABLE A-3

SHAFT LOAD-SETTLEMENT RESULTS

Boring SB-4, 8-foot-Diameter Drilled Shaft (Static Condition)

T-Z CURVE NO.	DEPTH TO CURVE FT.	LOAD TRANSFER PSI	PILE MOVEMENT IN.
13	72.50		
		0.00	0.00
		1.63	0.10
		2.82	0.19
		3.50	0.29
		4.18	0.48
		4.42	0.58
		4.62	0.77
		4.69	1.15
		4.67	1.54
		4.62	9.60
14	77.25		
		0.00	0.00
		1.63	0.10
		2.82	0.19
		3.50	0.29
		4.18	0.48
		4.42	0.58
		4.62	0.77
		4.69	1.15
		4.67	1.54
		4.62	9.60
15	82.00		
		0.00	0.00
		1.63	0.10
		2.82	0.19
		3.50	0.29
		4.18	0.48
		4.42	0.58
		4.62	0.77
		4.69	1.15
		4.67	1.54
		4.62	9.60
16	83.00		
		0.00	0.00
		4.11	0.10
		7.11	0.19
		8.83	0.29
		10.55	0.48
		11.16	0.58
		11.65	0.77
		11.83	1.15
		11.77	1.54
		11.65	9.60

TABLE A-3

SHAFT LOAD-SETTLEMENT RESULTS

Boring SB-4, 8-foot-Diameter Drilled Shaft (Static Condition)

T-Z CURVE NO.	DEPTH TO CURVE FT.	LOAD TRANSFER PSI	PILE MOVEMENT IN.
17	88.50		
		0.00	0.00
		4.41	0.10
		7.64	0.19
		9.48	0.29
		11.32	0.48
		11.98	0.58
		12.50	0.77
		12.70	1.15
		12.64	1.54
		12.50	9.60
18	95.00		
		0.00	0.00
		4.65	0.10
		8.06	0.19
		10.00	0.29
		11.94	0.48
		12.64	0.58
		13.19	0.77
		13.40	1.15
		13.33	1.54
		13.19	9.60
19	101.50		
		0.00	0.00
		4.65	0.10
		8.06	0.19
		10.00	0.29
		11.94	0.48
		12.64	0.58
		13.19	0.77
		13.40	1.15
		13.33	1.54
		13.19	9.60
20	108.00		
		0.00	0.00
		4.65	0.10
		8.06	0.19
		10.00	0.29
		11.94	0.48
		12.64	0.58
		13.19	0.77
		13.40	1.15
		13.33	1.54
		13.19	9.60

TABLE A-3

SHAFT LOAD-SETTLEMENT RESULTS

Boring SB-4, 8-foot-Diameter Drilled Shaft (Static Condition)

T-Z CURVE NO.	DEPTH TO CURVE FT.	LOAD TRANSFER PSI	PILE MOVEMENT IN.
21	109.00		
		0.00	0.00
		6.05	0.10
		10.47	0.19
		13.00	0.29
		15.53	0.48
		16.43	0.58
		17.15	0.77
		17.42	1.15
		17.33	1.54
		17.15	9.60
22	115.00		
		0	0
		6.049	0.096
		10.47	0.192
		13	0.288
		15.53	0.48
		16.43	0.576
		17.15	0.768
		17.42	1.152
		17.33	1.536
		17.15	9.6
23	122.00		
		0	0
		6.049	0.096
		10.47	0.192
		13	0.288
		15.53	0.48
		16.43	0.576
		17.15	0.768
		17.42	1.152
		17.33	1.536
		17.15	9.6
24	129.00		
		0	0
		6.049	0.096
		10.47	0.192
		13	0.288
		15.53	0.48
		16.43	0.576
		17.15	0.768
		17.42	1.152
		17.33	1.536
		17.15	9.6

TABLE A-3

SHAFT LOAD-SETTLEMENT RESULTS

Boring SB-4, 8-foot-Diameter Drilled Shaft (Static Condition)

T-Z CURVE NO.	DEPTH TO CURVE FT.	LOAD TRANSFER PSI	PILE MOVEMENT IN.
25	136.00		
		0	0
		6.049	0.096
		10.47	0.192
		13	0.288
		15.53	0.48
		16.43	0.576
		17.15	0.768
		17.42	1.152
		17.33	1.536
		17.15	9.6

TABLE A-4

SHAFT LOAD-SETTLEMENT RESULTS

Boring SB-4, 8-foot-Diameter Drilled Shaft (Seismic Condition)

T-Z CURVE NO.	DEPTH TO CURVE FT.	LOAD TRANSFER PSI	PILE MOVEMENT IN.
1	1.00		
		0.00	0.00
		0.00	0.10
		0.00	0.19
		0.00	0.29
		0.00	0.48
		0.00	0.58
		0.00	0.77
		0.00	1.15
		0.00	1.54
		0.00	9.60
2	5.75		
		0.00	0.00
		0.00	0.10
		0.01	0.19
		0.01	0.29
		0.01	0.48
		0.01	0.58
		0.01	0.77
		0.01	1.15
		0.01	1.54
		0.01	9.60
3	10.50		
		0.00	0.00
		0.01	0.10
		0.01	0.19
		0.02	0.29
		0.02	0.48
		0.02	0.58
		0.02	0.77
		0.02	1.15
		0.02	1.54
		0.02	9.60
4	15.25		
		0.00	0.00
		0.01	0.10
		0.02	0.19
		0.03	0.29
		0.03	0.48
		0.03	0.58
		0.03	0.77
		0.03	1.15
		0.03	1.54
		0.03	9.60

TOP LOAD KIP	TOP MOVEMENT IN.
0.78	0.00
7.80	0.00
19.50	0.00
39.00	0.01
58.52	0.01
78.02	0.01
195.04	0.03
390.00	0.07
585.40	0.10
768.00	0.14
1506.60	0.32
2076.00	0.60
2386.00	0.87
2566.00	1.13
4582.00	9.84

TIP LOAD KIP	TIP MOVEMENT IN.
0.07	0.00
0.67	0.00
1.68	0.00
3.36	0.01
5.04	0.01
6.72	0.01
16.80	0.03
33.60	0.05
50.40	0.08
67.20	0.10
168.00	0.25
335.60	0.50
498.80	0.75
657.40	1.00
2692.00	9.60

TABLE A-4

SHAFT LOAD-SETTLEMENT RESULTS

Boring SB-4, 8-foot-Diameter Drilled Shaft (Seismic Condition)

T-Z CURVE NO.	DEPTH TO CURVE FT.	LOAD TRANSFER PSI	PILE MOVEMENT IN.
5	20.00		
		0.00	0.00
		0.02	0.10
		0.03	0.19
		0.03	0.29
		0.04	0.48
		0.04	0.58
		0.04	0.77
		0.04	1.15
		0.04	1.54
		0.04	9.60
6	21.00		
		0.00	0.00
		0.02	0.10
		0.03	0.19
		0.04	0.29
		0.04	0.48
		0.05	0.58
		0.05	0.77
		0.05	1.15
		0.05	1.54
		0.05	9.60
7	30.75		
		0.00	0.00
		0.03	0.10
		0.04	0.19
		0.05	0.29
		0.06	0.48
		0.07	0.58
		0.07	0.77
		0.07	1.15
		0.07	1.54
		0.07	9.60
8	41.50		
		0.00	0.00
		0.03	0.10
		0.06	0.19
		0.07	0.29
		0.09	0.48
		0.09	0.58
		0.10	0.77
		0.10	1.15
		0.10	1.54
		0.10	9.60

TABLE A-4

SHAFT LOAD-SETTLEMENT RESULTS

Boring SB-4, 8-foot-Diameter Drilled Shaft (Seismic Condition)

T-Z CURVE NO.	DEPTH TO CURVE FT.	LOAD TRANSFER PSI	PILE MOVEMENT IN.
9	52.25		
		0.00	0.00
		0.04	0.10
		0.08	0.19
		0.09	0.29
		0.11	0.48
		0.12	0.58
		0.12	0.77
		0.13	1.15
		0.13	1.54
		0.12	9.60
10	63.00		
		0.00	0.00
		0.05	0.10
		0.09	0.19
		0.11	0.29
		0.14	0.48
		0.14	0.58
		0.15	0.77
		0.15	1.15
		0.15	1.54
		0.15	9.60
11	64.00		
		0.00	0.00
		0.06	0.10
		0.10	0.19
		0.13	0.29
		0.15	0.48
		0.16	0.58
		0.17	0.77
		0.17	1.15
		0.17	1.54
		0.17	9.60
12	67.75		
		0.00	0.00
		0.06	0.10
		0.11	0.19
		0.13	0.29
		0.16	0.48
		0.17	0.58
		0.18	0.77
		0.18	1.15
		0.18	1.54
		0.18	9.60

TABLE A-4

SHAFT LOAD-SETTLEMENT RESULTS

Boring SB-4, 8-foot-Diameter Drilled Shaft (Seismic Condition)

T-Z CURVE NO.	DEPTH TO CURVE FT.	LOAD TRANSFER PSI	PILE MOVEMENT IN.
13	72.50		
		0.00	0.00
		0.07	0.10
		0.12	0.19
		0.14	0.29
		0.17	0.48
		0.18	0.58
		0.19	0.77
		0.19	1.15
		0.19	1.54
		0.19	9.60
14	77.25		
		0.00	0.00
		0.07	0.10
		0.12	0.19
		0.15	0.29
		0.18	0.48
		0.19	0.58
		0.20	0.77
		0.20	1.15
		0.20	1.54
		0.20	9.60
15	82.00		
		0.00	0.00
		0.08	0.10
		0.13	0.19
		0.16	0.29
		0.19	0.48
		0.20	0.58
		0.21	0.77
		0.22	1.15
		0.22	1.54
		0.21	9.60
16	83.00		
		0.00	0.00
		4.11	0.10
		7.11	0.19
		8.83	0.29
		10.55	0.48
		11.16	0.58
		11.65	0.77
		11.83	1.15
		11.77	1.54
		11.65	9.60

TABLE A-4

SHAFT LOAD-SETTLEMENT RESULTS

Boring SB-4, 8-foot-Diameter Drilled Shaft (Seismic Condition)

T-Z CURVE NO.	DEPTH TO CURVE FT.	LOAD TRANSFER PSI	PILE MOVEMENT IN.
17	88.50		
		0.00	0.00
		4.41	0.10
		7.64	0.19
		9.48	0.29
		11.32	0.48
		11.98	0.58
		12.50	0.77
		12.70	1.15
		12.64	1.54
		12.50	9.60
18	95.00		
		0.00	0.00
		4.65	0.10
		8.06	0.19
		10.00	0.29
		11.94	0.48
		12.64	0.58
		13.19	0.77
		13.40	1.15
		13.33	1.54
		13.19	9.60
19	101.50		
		0.00	0.00
		4.65	0.10
		8.06	0.19
		10.00	0.29
		11.94	0.48
		12.64	0.58
		13.19	0.77
		13.40	1.15
		13.33	1.54
		13.19	9.60
20	108.00		
		0.00	0.00
		4.65	0.10
		8.06	0.19
		10.00	0.29
		11.94	0.48
		12.64	0.58
		13.19	0.77
		13.40	1.15
		13.33	1.54
		13.19	9.60

TABLE A-4

SHAFT LOAD-SETTLEMENT RESULTS

Boring SB-4, 8-foot-Diameter Drilled Shaft (Seismic Condition)

T-Z CURVE NO.	DEPTH TO CURVE FT.	LOAD TRANSFER PSI	PILE MOVEMENT IN.
21	109.00		
		0.00	0.00
		6.05	0.10
		10.47	0.19
		13.00	0.29
		15.53	0.48
		16.43	0.58
		17.15	0.77
		17.42	1.15
		17.33	1.54
		17.15	9.60
22	115.00		
		0	0
		6.049	0.096
		10.47	0.192
		13	0.288
		15.53	0.48
		16.43	0.576
		17.15	0.768
		17.42	1.152
		17.33	1.536
		17.15	9.6
23	122.00		
		0	0
		6.049	0.096
		10.47	0.192
		13	0.288
		15.53	0.48
		16.43	0.576
		17.15	0.768
		17.42	1.152
		17.33	1.536
		17.15	9.6
24	129.00		
		0	0
		6.049	0.096
		10.47	0.192
		13	0.288
		15.53	0.48
		16.43	0.576
		17.15	0.768
		17.42	1.152
		17.33	1.536
		17.15	9.6

TABLE A-4

SHAFT LOAD-SETTLEMENT RESULTS

Boring SB-4, 8-foot-Diameter Drilled Shaft (Seismic Condition)

T-Z CURVE NO.	DEPTH TO CURVE FT.	LOAD TRANSFER PSI	PILE MOVEMENT IN.
25	136.00		
		0	0
		6.049	0.096
		10.47	0.192
		13	0.288
		15.53	0.48
		16.43	0.576
		17.15	0.768
		17.42	1.152
		17.33	1.536
		17.15	9.6

TABLE A-5

APILE LOAD-SETTLEMENT RESULTS

Boring SB-5, 24-Inch-Diameter Steel Pipe Pile, Closed-end (Static Condition)

T-Z CURVE NO.	DEPTH TO CURVE FT.	LOAD TRANSFER PSI	PILE MOVEMENT IN.
1	0.00		
		0.00	0.00
		0.04	0.01
		0.08	0.02
		0.17	0.04
		0.25	0.06
		0.33	0.08
		0.38	0.09
		0.42	0.10
		0.42	0.50
		0.42	2.00
2	7.53		
		0.00	0.00
		0.04	0.01
		0.08	0.02
		0.17	0.04
		0.25	0.06
		0.33	0.08
		0.38	0.09
		0.42	0.10
		0.42	0.50
		0.42	2.00
3	14.96		
		0.00	0.00
		0.04	0.01
		0.08	0.02
		0.17	0.04
		0.25	0.06
		0.33	0.08
		0.38	0.09
		0.42	0.10
		0.42	0.50
		0.42	2.00
4	15.00		
		0.00	0.00
		0.24	0.01
		0.47	0.02
		0.94	0.04
		1.42	0.06
		1.89	0.08
		2.13	0.09
		2.36	0.10
		2.36	0.50
		2.36	2.00

TOP LOAD KIP	TOP MOVEMENT IN.
16.44	0.02
160.60	0.19
486.10	0.74
672.60	1.12
1097.00	2.15
1180.00	2.41
1281.00	3.08
1348.00	3.76
1421.00	4.95

TIP LOAD KIP	TIP MOVEMENT IN.
0.00	0.00
23.56	0.01
47.12	0.02
94.25	0.05
188.50	0.31
282.70	1.01
339.30	1.75
377.00	2.40
377.00	3.60
377.00	4.80

TABLE A-5

APILE LOAD-SETTLEMENT RESULTS

Boring SB-5, 24-Inch-Diameter Steel Pipe Pile, Closed-end (Static Condition)

T-Z CURVE NO.	DEPTH TO CURVE FT.	LOAD TRANSFER PSI	PILE MOVEMENT IN.
5	35.52		
		0.00	0.00
		0.24	0.01
		0.47	0.02
		0.94	0.04
		1.42	0.06
		1.89	0.08
		2.13	0.09
		2.36	0.10
		2.36	0.50
		2.36	2.00
6	55.96		
		0.00	0.00
		0.24	0.01
		0.47	0.02
		0.94	0.04
		1.42	0.06
		1.89	0.08
		2.13	0.09
		2.36	0.10
		2.36	0.50
		2.36	2.00
7	56.00		
		0.00	0.00
		0.33	0.01
		0.67	0.02
		1.33	0.04
		2.00	0.06
		2.67	0.08
		3.00	0.09
		3.33	0.10
		3.33	0.50
		3.33	2.00
8	57.02		
		0.00	0.00
		0.33	0.01
		0.67	0.02
		1.33	0.04
		2.00	0.06
		2.67	0.08
		3.00	0.09
		3.33	0.10
		3.33	0.50
		3.33	2.00

TABLE A-5

APILE LOAD-SETTLEMENT RESULTS

Boring SB-5, 24-Inch-Diameter Steel Pipe Pile, Closed-end (Static Condition)

T-Z CURVE NO.	DEPTH TO CURVE FT.	LOAD TRANSFER PSI	PILE MOVEMENT IN.
9	57.96		
		0.00	0.00
		0.33	0.01
		0.67	0.02
		1.33	0.04
		2.00	0.06
		2.67	0.08
		3.00	0.09
		3.33	0.10
		3.33	0.50
		3.33	2.00
10	58.00		
		0.00	0.00
		1.39	0.01
		2.78	0.02
		5.56	0.04
		8.33	0.06
		11.11	0.08
		12.50	0.09
		13.89	0.10
		13.89	0.50
		13.89	2.00
11	64.53		
		0.00	0.00
		1.39	0.01
		2.78	0.02
		5.56	0.04
		8.33	0.06
		11.11	0.08
		12.50	0.09
		13.89	0.10
		13.89	0.50
		13.89	2.00
12	70.96		
		0.00	0.00
		1.39	0.01
		2.78	0.02
		5.56	0.04
		8.33	0.06
		11.11	0.08
		12.50	0.09
		13.89	0.10
		13.89	0.50
		13.89	2.00

TABLE A-5

APILE LOAD-SETTLEMENT RESULTS

Boring SB-5, 24-Inch-Diameter Steel Pipe Pile, Closed-end (Static Condition)

T-Z CURVE NO.	DEPTH TO CURVE FT.	LOAD TRANSFER PSI	PILE MOVEMENT IN.
13	71.00		
		0.00	0.00
		1.81	0.01
		3.61	0.02
		7.22	0.04
		10.83	0.06
		14.44	0.08
		16.25	0.09
		18.06	0.10
		18.06	0.50
		18.06	2.00
14	73.53		
		0.00	0.00
		1.81	0.01
		3.61	0.02
		7.22	0.04
		10.83	0.06
		14.44	0.08
		16.25	0.09
		18.06	0.10
		18.06	0.50
		18.06	2.00
15	75.96		
		0.00	0.00
		1.81	0.01
		3.61	0.02
		7.22	0.04
		10.83	0.06
		14.44	0.08
		16.25	0.09
		18.06	0.10
		18.06	0.50
		18.06	2.00
16	76.00		
		0.00	0.00
		1.81	0.01
		3.61	0.02
		7.22	0.04
		10.83	0.06
		14.44	0.08
		16.25	0.09
		18.06	0.10
		18.06	0.50
		18.06	2.00

TABLE A-5

APILE LOAD-SETTLEMENT RESULTS

Boring SB-5, 24-Inch-Diameter Steel Pipe Pile, Closed-end (Static Condition)

T-Z CURVE NO.	DEPTH TO CURVE FT.	LOAD TRANSFER PSI	PILE MOVEMENT IN.
17	83.03		
		0.00	0.00
		1.81	0.01
		3.61	0.02
		7.22	0.04
		10.83	0.06
		14.44	0.08
		16.25	0.09
		18.06	0.10
		18.06	0.50
		18.06	2.00
18	89.96		
		0.00	0.00
		1.81	0.01
		3.61	0.02
		7.22	0.04
		10.83	0.06
		14.44	0.08
		16.25	0.09
		18.06	0.10
		18.06	0.50
		18.06	2.00
19	90.00		
		0.00	0.00
		1.81	0.01
		3.61	0.02
		7.22	0.04
		10.83	0.06
		14.44	0.08
		16.25	0.09
		18.06	0.10
		18.06	0.50
		18.06	2.00
20	107.50		
		0.00	0.00
		1.81	0.01
		3.61	0.02
		7.22	0.04
		10.83	0.06
		14.44	0.08
		16.25	0.09
		18.06	0.10
		18.06	0.50
		18.06	2.00

TABLE A-5

APILE LOAD-SETTLEMENT RESULTS

Boring SB-5, 24-Inch-Diameter Steel Pipe Pile, Closed-end (Static Condition)

T-Z CURVE NO.	DEPTH TO CURVE FT.	LOAD TRANSFER PSI	PILE MOVEMENT IN.
21	125.00		
		0.00	0.00
		1.81	0.01
		3.61	0.02
		7.22	0.04
		10.83	0.06
		14.44	0.08
		16.25	0.09
		18.06	0.10
		18.06	0.50
		18.06	2.00

TABLE A-6

APILE LOAD-SETTLEMENT RESULTS

Boring SB-5, 24-Inch-Diameter Steel Pipe Pile, Closed-end (Seismic Condition)

T-Z CURVE NO.	DEPTH TO CURVE FT.	LOAD TRANSFER PSI	PILE MOVEMENT IN.
1	0.00		
		0.00	0.00
		0.00	0.01
		0.00	0.02
		0.00	0.04
		0.00	0.06
		0.00	0.08
		0.00	0.09
		0.00	0.10
		0.00	0.50
		0.00	2.00
2	7.53		
		0.00	0.00
		0.00	0.01
		0.00	0.02
		0.00	0.04
		0.00	0.06
		0.00	0.08
		0.00	0.09
		0.00	0.10
		0.00	0.50
		0.00	2.00
3	14.96		
		0.00	0.00
		0.00	0.01
		0.00	0.02
		0.00	0.04
		0.00	0.06
		0.00	0.08
		0.00	0.09
		0.00	0.10
		0.00	0.50
		0.00	2.00
4	15.00		
		0.00	0.00
		0.00	0.01
		0.00	0.02
		0.00	0.04
		0.00	0.06
		0.00	0.08
		0.00	0.09
		0.00	0.10
		0.00	0.50
		0.00	2.00

TOP LOAD KIP	TOP MOVEMENT IN.
8.16	0.01
84.75	0.14
387.70	0.67
574.20	1.04
998.70	2.07
1081.00	2.34
1182.00	3.00
1250.00	3.69
1322.00	4.88

TIP LOAD KIP	TIP MOVEMENT IN.
0.00	0.00
23.56	0.01
47.12	0.02
94.25	0.05
188.50	0.31
282.70	1.01
339.30	1.75
377.00	2.40
377.00	3.60
377.00	4.80

TABLE A-6

APILE LOAD-SETTLEMENT RESULTS

Boring SB-5, 24-Inch-Diameter Steel Pipe Pile, Closed-end (Seismic Condition)

T-Z CURVE NO.	DEPTH TO CURVE FT.	LOAD TRANSFER PSI	PILE MOVEMENT IN.
5	35.52		
		0.00	0.00
		0.00	0.01
		0.00	0.02
		0.00	0.04
		0.00	0.06
		0.00	0.08
		0.00	0.09
		0.00	0.10
		0.00	0.50
		0.00	2.00
6	55.96		
		0.00	0.00
		0.00	0.01
		0.00	0.02
		0.00	0.04
		0.00	0.06
		0.00	0.08
		0.00	0.09
		0.00	0.10
		0.00	0.50
		0.00	2.00
7	56.00		
		0.00	0.00
		0.00	0.01
		0.00	0.02
		0.00	0.04
		0.00	0.06
		0.00	0.08
		0.00	0.09
		0.00	0.10
		0.00	0.50
		0.00	2.00
8	57.02		
		0.00	0.00
		0.00	0.01
		0.00	0.02
		0.00	0.04
		0.00	0.06
		0.00	0.08
		0.00	0.09
		0.00	0.10
		0.00	0.50
		0.00	2.00

TABLE A-6

APILE LOAD-SETTLEMENT RESULTS

Boring SB-5, 24-Inch-Diameter Steel Pipe Pile, Closed-end (Seismic Condition)

T-Z CURVE NO.	DEPTH TO CURVE FT.	LOAD TRANSFER PSI	PILE MOVEMENT IN.
9	57.96		
		0.00	0.00
		0.00	0.01
		0.00	0.02
		0.00	0.04
		0.00	0.06
		0.00	0.08
		0.00	0.09
		0.00	0.10
		0.00	0.50
		0.00	2.00
10	58.00		
		0.00	0.00
		1.39	0.01
		2.78	0.02
		5.56	0.04
		8.33	0.06
		11.11	0.08
		12.50	0.09
		13.89	0.10
		13.89	0.50
		13.89	2.00
11	64.53		
		0.00	0.00
		1.39	0.01
		2.78	0.02
		5.56	0.04
		8.33	0.06
		11.11	0.08
		12.50	0.09
		13.89	0.10
		13.89	0.50
		13.89	2.00
12	70.96		
		0.00	0.00
		1.39	0.01
		2.78	0.02
		5.56	0.04
		8.33	0.06
		11.11	0.08
		12.50	0.09
		13.89	0.10
		13.89	0.50
		13.89	2.00

TABLE A-6

APILE LOAD-SETTLEMENT RESULTS

Boring SB-5, 24-Inch-Diameter Steel Pipe Pile, Closed-end (Seismic Condition)

T-Z CURVE NO.	DEPTH TO CURVE FT.	LOAD TRANSFER PSI	PILE MOVEMENT IN.
13	71.00		
		0.00	0.00
		1.81	0.01
		3.61	0.02
		7.22	0.04
		10.83	0.06
		14.44	0.08
		16.25	0.09
		18.06	0.10
		18.06	0.50
		18.06	2.00
14	73.53		
		0.00	0.00
		1.81	0.01
		3.61	0.02
		7.22	0.04
		10.83	0.06
		14.44	0.08
		16.25	0.09
		18.06	0.10
		18.06	0.50
		18.06	2.00
15	75.96		
		0.00	0.00
		1.81	0.01
		3.61	0.02
		7.22	0.04
		10.83	0.06
		14.44	0.08
		16.25	0.09
		18.06	0.10
		18.06	0.50
		18.06	2.00
16	76.00		
		0.00	0.00
		1.81	0.01
		3.61	0.02
		7.22	0.04
		10.83	0.06
		14.44	0.08
		16.25	0.09
		18.06	0.10
		18.06	0.50
		18.06	2.00

TABLE A-6

APILE LOAD-SETTLEMENT RESULTS

Boring SB-5, 24-Inch-Diameter Steel Pipe Pile, Closed-end (Seismic Condition)

T-Z CURVE NO.	DEPTH TO CURVE FT.	LOAD TRANSFER PSI	PILE MOVEMENT IN.
17	83.03		
		0.00	0.00
		1.81	0.01
		3.61	0.02
		7.22	0.04
		10.83	0.06
		14.44	0.08
		16.25	0.09
		18.06	0.10
		18.06	0.50
		18.06	2.00
18	89.96		
		0.00	0.00
		1.81	0.01
		3.61	0.02
		7.22	0.04
		10.83	0.06
		14.44	0.08
		16.25	0.09
		18.06	0.10
		18.06	0.50
		18.06	2.00
19	90.00		
		0.00	0.00
		1.81	0.01
		3.61	0.02
		7.22	0.04
		10.83	0.06
		14.44	0.08
		16.25	0.09
		18.06	0.10
		18.06	0.50
		18.06	2.00
20	107.50		
		0.00	0.00
		1.81	0.01
		3.61	0.02
		7.22	0.04
		10.83	0.06
		14.44	0.08
		16.25	0.09
		18.06	0.10
		18.06	0.50
		18.06	2.00

TABLE A-6

APILE LOAD-SETTLEMENT RESULTS

Boring SB-5, 24-Inch-Diameter Steel Pipe Pile, Closed-end (Seismic Condition)

T-Z CURVE NO.	DEPTH TO CURVE FT.	LOAD TRANSFER PSI	PILE MOVEMENT IN.
21	125.00		
		0.00	0.00
		1.81	0.01
		3.61	0.02
		7.22	0.04
		10.83	0.06
		14.44	0.08
		16.25	0.09
		18.06	0.10
		18.06	0.50
		18.06	2.00

TABLE A-7

SHAFT LOAD-SETTLEMENT RESULTS

Boring SB-5, 8-foot-Diameter Drilled Shaft (Static Condition)

T-Z CURVE NO.	DEPTH TO CURVE FT.	LOAD TRANSFER PSI	PILE MOVEMENT IN.
1	1.00		
		0.00	0.00
		0.03	0.10
		0.05	0.19
		0.07	0.29
		0.08	0.48
		0.08	0.58
		0.09	0.77
		0.09	1.15
		0.09	1.54
		0.09	9.60
2	4.50		
		0.00	0.00
		0.13	0.10
		0.23	0.19
		0.28	0.29
		0.34	0.48
		0.36	0.58
		0.37	0.77
		0.38	1.15
		0.38	1.54
		0.37	9.60
3	8.00		
		0.00	0.00
		0.14	0.10
		0.24	0.19
		0.30	0.29
		0.36	0.48
		0.38	0.58
		0.40	0.77
		0.40	1.15
		0.40	1.54
		0.40	9.60
4	11.50		
		0.00	0.00
		0.14	0.10
		0.24	0.19
		0.30	0.29
		0.36	0.48
		0.38	0.58
		0.40	0.77
		0.40	1.15
		0.40	1.54
		0.40	9.60

TOP LOAD KIP	TOP MOVEMENT IN.
1.64	0.00
16.36	0.00
40.90	0.00
81.78	0.01
122.68	0.01
163.58	0.02
409.00	0.04
818.80	0.08
1225.80	0.12
1583.20	0.16
3000.00	0.37
3978.00	0.67
4432.00	0.94
4656.00	1.20
6924.00	9.93

TIP LOAD KIP	TIP MOVEMENT IN.
0.08	0.00
0.76	0.00
1.91	0.00
3.82	0.01
5.73	0.01
7.64	0.01
19.09	0.03
38.18	0.05
57.28	0.08
76.36	0.10
190.92	0.25
381.40	0.50
566.80	0.75
747.00	1.00
3058.00	9.60

TABLE A-7

SHAFT LOAD-SETTLEMENT RESULTS

Boring SB-5, 8-foot-Diameter Drilled Shaft (Static Condition)

T-Z CURVE NO.	DEPTH TO CURVE FT.	LOAD TRANSFER PSI	PILE MOVEMENT IN.
5	15.00		
		0.00	0.00
		0.14	0.10
		0.24	0.19
		0.30	0.29
		0.36	0.48
		0.38	0.58
		0.40	0.77
		0.40	1.15
		0.40	1.54
		0.40	9.60
6	16.00		
		0.00	0.00
		0.58	0.10
		1.00	0.19
		1.24	0.29
		1.48	0.48
		1.56	0.58
		1.63	0.77
		1.66	1.15
		1.65	1.54
		1.63	9.60
7	25.25		
		0.00	0.00
		0.79	0.10
		1.37	0.19
		1.70	0.29
		2.03	0.48
		2.15	0.58
		2.24	0.77
		2.28	1.15
		2.27	1.54
		2.24	9.60
8	35.50		
		0.00	0.00
		0.79	0.10
		1.37	0.19
		1.70	0.29
		2.03	0.48
		2.15	0.58
		2.24	0.77
		2.28	1.15
		2.27	1.54
		2.24	9.60

TABLE A-7

SHAFT LOAD-SETTLEMENT RESULTS

Boring SB-5, 8-foot-Diameter Drilled Shaft (Static Condition)

T-Z CURVE NO.	DEPTH TO CURVE FT.	LOAD TRANSFER PSI	PILE MOVEMENT IN.
9	45.75		
		0.00	0.00
		0.79	0.10
		1.37	0.19
		1.70	0.29
		2.03	0.48
		2.15	0.58
		2.24	0.77
		2.28	1.15
		2.27	1.54
		2.24	9.60
10	56.00		
		0.00	0.00
		0.79	0.10
		1.37	0.19
		1.70	0.29
		2.03	0.48
		2.15	0.58
		2.24	0.77
		2.28	1.15
		2.27	1.54
		2.24	9.60
11	57.00		
		0.00	0.00
		1.12	0.10
		1.93	0.19
		2.40	0.29
		2.87	0.48
		3.03	0.58
		3.17	0.77
		3.22	1.15
		3.20	1.54
		3.17	9.60
12	56.50		
		0.00	0.00
		0.95	0.10
		1.65	0.19
		2.05	0.29
		2.45	0.48
		2.59	0.58
		2.71	0.77
		2.75	1.15
		2.73	1.54
		2.71	9.60

TABLE A-7

SHAFT LOAD-SETTLEMENT RESULTS

Boring SB-5, 8-foot-Diameter Drilled Shaft (Static Condition)

T-Z CURVE NO.	DEPTH TO CURVE FT.	LOAD TRANSFER PSI	PILE MOVEMENT IN.
13	57.00		
		0.00	0.00
		1.12	0.10
		1.93	0.19
		2.40	0.29
		2.87	0.48
		3.03	0.58
		3.17	0.77
		3.22	1.15
		3.20	1.54
		3.17	9.60
14	57.50		
		0.00	0.00
		1.12	0.10
		1.93	0.19
		2.40	0.29
		2.87	0.48
		3.03	0.58
		3.17	0.77
		3.22	1.15
		3.20	1.54
		3.17	9.60
15	58.00		
		0.00	0.00
		1.12	0.10
		1.93	0.19
		2.40	0.29
		2.87	0.48
		3.03	0.58
		3.17	0.77
		3.22	1.15
		3.20	1.54
		3.17	9.60
16	59.00		
		0.00	0.00
		3.90	0.10
		6.75	0.19
		8.38	0.29
		10.01	0.48
		10.59	0.58
		11.06	0.77
		11.23	1.15
		11.17	1.54
		11.06	9.60

TABLE A-7

SHAFT LOAD-SETTLEMENT RESULTS

Boring SB-5, 8-foot-Diameter Drilled Shaft (Static Condition)

T-Z CURVE NO.	DEPTH TO CURVE FT.	LOAD TRANSFER PSI	PILE MOVEMENT IN.
17	61.25		
		0.00	0.00
		4.06	0.10
		7.04	0.19
		8.73	0.29
		10.43	0.48
		11.04	0.58
		11.52	0.77
		11.71	1.15
		11.65	1.54
		11.52	9.60
18	64.50		
		0.00	0.00
		4.30	0.10
		7.45	0.19
		9.25	0.29
		11.04	0.48
		11.69	0.58
		12.20	0.77
		12.39	1.15
		12.33	1.54
		12.20	9.60
19	67.75		
		0.00	0.00
		4.54	0.10
		7.86	0.19
		9.76	0.29
		11.66	0.48
		12.33	0.58
		12.88	0.77
		13.08	1.15
		13.01	1.54
		12.88	9.60
20	71.00		
		0.00	0.00
		4.65	0.10
		8.06	0.19
		10.00	0.29
		11.94	0.48
		12.64	0.58
		13.19	0.77
		13.40	1.15
		13.33	1.54
		13.19	9.60

TABLE A-7

SHAFT LOAD-SETTLEMENT RESULTS

Boring SB-5, 8-foot-Diameter Drilled Shaft (Static Condition)

T-Z CURVE NO.	DEPTH TO CURVE FT.	LOAD TRANSFER PSI	PILE MOVEMENT IN.
21	72.00		
		0.00	0.00
		5.78	0.10
		10.01	0.19
		12.43	0.29
		14.85	0.48
		15.71	0.58
		16.40	0.77
		16.66	1.15
		16.57	1.54
		16.40	9.60
22	87.25		
		0	0
		6.049	0.096
		10.47	0.192
		13	0.288
		15.53	0.48
		16.43	0.576
		17.15	0.768
		17.42	1.152
		17.33	1.536
		17.15	9.6
23	103.50		
		0	0
		6.049	0.096
		10.47	0.192
		13	0.288
		15.53	0.48
		16.43	0.576
		17.15	0.768
		17.42	1.152
		17.33	1.536
		17.15	9.6
24	119.75		
		0	0
		6.049	0.096
		10.47	0.192
		13	0.288
		15.53	0.48
		16.43	0.576
		17.15	0.768
		17.42	1.152
		17.33	1.536
		17.15	9.6

TABLE A-7

SHAFT LOAD-SETTLEMENT RESULTS

Boring SB-5, 8-foot-Diameter Drilled Shaft (Static Condition)

T-Z CURVE NO.	DEPTH TO CURVE FT.	LOAD TRANSFER PSI	PILE MOVEMENT IN.
25	136.00		
		0	0
		6.049	0.096
		10.47	0.192
		13	0.288
		15.53	0.48
		16.43	0.576
		17.15	0.768
		17.42	1.152
		17.33	1.536
		17.15	9.6

TABLE A-8

SHAFT LOAD-SETTLEMENT RESULTS

Boring SB-5, 8-foot-Diameter Drilled Shaft (Seismic Condition)

T-Z CURVE NO.	DEPTH TO CURVE FT.	LOAD TRANSFER PSI	PILE MOVEMENT IN.
1	1.00		
		0.00	0.00
		0.00	0.10
		0.00	0.19
		0.00	0.29
		0.00	0.48
		0.00	0.58
		0.00	0.77
		0.00	1.15
		0.00	1.54
		0.00	9.60
2	4.50		
		0.00	0.00
		0.00	0.10
		0.01	0.19
		0.01	0.29
		0.01	0.48
		0.01	0.58
		0.01	0.77
		0.01	1.15
		0.01	1.54
		0.01	9.60
3	8.00		
		0.00	0.00
		0.01	0.10
		0.01	0.19
		0.01	0.29
		0.02	0.48
		0.02	0.58
		0.02	0.77
		0.02	1.15
		0.02	1.54
		0.02	9.60
4	11.50		
		0.00	0.00
		0.01	0.10
		0.02	0.19
		0.02	0.29
		0.02	0.48
		0.02	0.58
		0.02	0.77
		0.03	1.15
		0.03	1.54
		0.02	9.60

TOP LOAD KIP	TOP MOVEMENT IN.
1.46	0.00
14.59	0.00
36.48	0.00
72.96	0.01
109.44	0.01
145.92	0.02
364.80	0.04
730.40	0.08
1095.80	0.12
1421.40	0.16
2728.00	0.37
3644.00	0.66
4082.00	0.93
4302.00	1.19
6576.00	9.92

TIP LOAD KIP	TIP MOVEMENT IN.
0.08	0.00
0.76	0.00
1.91	0.00
3.82	0.01
5.73	0.01
7.64	0.01
19.09	0.03
38.18	0.05
57.28	0.08
76.36	0.10
190.92	0.25
381.40	0.50
566.80	0.75
747.00	1.00
3058.00	9.60

TABLE A-8

SHAFT LOAD-SETTLEMENT RESULTS

Boring SB-5, 8-foot-Diameter Drilled Shaft (Seismic Condition)

T-Z CURVE NO.	DEPTH TO CURVE FT.	LOAD TRANSFER PSI	PILE MOVEMENT IN.
5	15.00		
		0.00	0.00
		0.01	0.10
		0.02	0.19
		0.02	0.29
		0.03	0.48
		0.03	0.58
		0.03	0.77
		0.03	1.15
		0.03	1.54
		0.03	9.60
6	16.00		
		0.00	0.00
		0.01	0.10
		0.02	0.19
		0.03	0.29
		0.03	0.48
		0.03	0.58
		0.04	0.77
		0.04	1.15
		0.04	1.54
		0.04	9.60
7	25.25		
		0.00	0.00
		0.02	0.10
		0.04	0.19
		0.04	0.29
		0.05	0.48
		0.06	0.58
		0.06	0.77
		0.06	1.15
		0.06	1.54
		0.06	9.60
8	35.50		
		0.00	0.00
		0.03	0.10
		0.05	0.19
		0.06	0.29
		0.08	0.48
		0.08	0.58
		0.08	0.77
		0.09	1.15
		0.09	1.54
		0.08	9.60

TABLE A-8

SHAFT LOAD-SETTLEMENT RESULTS

Boring SB-5, 8-foot-Diameter Drilled Shaft (Seismic Condition)

T-Z CURVE NO.	DEPTH TO CURVE FT.	LOAD TRANSFER PSI	PILE MOVEMENT IN.
9	45.75		
		0.00	0.00
		0.04	0.10
		0.07	0.19
		0.08	0.29
		0.10	0.48
		0.11	0.58
		0.11	0.77
		0.11	1.15
		0.11	1.54
		0.11	9.60
10	56.00		
		0.00	0.00
		0.05	0.10
		0.08	0.19
		0.10	0.29
		0.12	0.48
		0.13	0.58
		0.14	0.77
		0.14	1.15
		0.14	1.54
		0.14	9.60
11	57.00		
		0.00	0.00
		0.05	0.10
		0.08	0.19
		0.10	0.29
		0.12	0.48
		0.13	0.58
		0.13	0.77
		0.13	1.15
		0.13	1.54
		0.13	9.60
12	56.50		
		0.00	0.00
		0.05	0.10
		0.08	0.19
		0.10	0.29
		0.12	0.48
		0.13	0.58
		0.13	0.77
		0.14	1.15
		0.14	1.54
		0.13	9.60

TABLE A-8

SHAFT LOAD-SETTLEMENT RESULTS

Boring SB-5, 8-foot-Diameter Drilled Shaft (Seismic Condition)

T-Z CURVE NO.	DEPTH TO CURVE FT.	LOAD TRANSFER PSI	PILE MOVEMENT IN.
13	57.00		
		0.00	0.00
		0.05	0.10
		0.08	0.19
		0.10	0.29
		0.12	0.48
		0.13	0.58
		0.13	0.77
		0.13	1.15
		0.13	1.54
		0.13	9.60
14	57.50		
		0.00	0.00
		0.05	0.10
		0.08	0.19
		0.10	0.29
		0.12	0.48
		0.13	0.58
		0.13	0.77
		0.14	1.15
		0.13	1.54
		0.13	9.60
15	58.00		
		0.00	0.00
		0.05	0.10
		0.08	0.19
		0.10	0.29
		0.12	0.48
		0.13	0.58
		0.13	0.77
		0.14	1.15
		0.14	1.54
		0.13	9.60
16	59.00		
		0.00	0.00
		3.90	0.10
		6.75	0.19
		8.38	0.29
		10.01	0.48
		10.59	0.58
		11.06	0.77
		11.23	1.15
		11.17	1.54
		11.06	9.60

TABLE A-8

SHAFT LOAD-SETTLEMENT RESULTS

Boring SB-5, 8-foot-Diameter Drilled Shaft (Seismic Condition)

T-Z CURVE NO.	DEPTH TO CURVE FT.	LOAD TRANSFER PSI	PILE MOVEMENT IN.
17	61.25		
		0.00	0.00
		4.06	0.10
		7.04	0.19
		8.73	0.29
		10.43	0.48
		11.04	0.58
		11.52	0.77
		11.71	1.15
		11.65	1.54
		11.52	9.60
18	64.50		
		0.00	0.00
		4.30	0.10
		7.45	0.19
		9.25	0.29
		11.04	0.48
		11.69	0.58
		12.20	0.77
		12.39	1.15
		12.33	1.54
		12.20	9.60
19	67.75		
		0.00	0.00
		4.54	0.10
		7.86	0.19
		9.76	0.29
		11.66	0.48
		12.33	0.58
		12.88	0.77
		13.08	1.15
		13.01	1.54
		12.88	9.60
20	71.00		
		0.00	0.00
		4.65	0.10
		8.06	0.19
		10.00	0.29
		11.94	0.48
		12.64	0.58
		13.19	0.77
		13.40	1.15
		13.33	1.54
		13.19	9.60

TABLE A-8

SHAFT LOAD-SETTLEMENT RESULTS

Boring SB-5, 8-foot-Diameter Drilled Shaft (Seismic Condition)

T-Z CURVE NO.	DEPTH TO CURVE FT.	LOAD TRANSFER PSI	PILE MOVEMENT IN.
21	72.00		
		0.00	0.00
		5.78	0.10
		10.01	0.19
		12.43	0.29
		14.85	0.48
		15.71	0.58
		16.40	0.77
		16.66	1.15
		16.57	1.54
		16.40	9.60
22	87.25		
		0	0
		6.049	0.096
		10.47	0.192
		13	0.288
		15.53	0.48
		16.43	0.576
		17.15	0.768
		17.42	1.152
		17.33	1.536
		17.15	9.6
23	103.50		
		0	0
		6.049	0.096
		10.47	0.192
		13	0.288
		15.53	0.48
		16.43	0.576
		17.15	0.768
		17.42	1.152
		17.33	1.536
		17.15	9.6
24	119.75		
		0	0
		6.049	0.096
		10.47	0.192
		13	0.288
		15.53	0.48
		16.43	0.576
		17.15	0.768
		17.42	1.152
		17.33	1.536
		17.15	9.6

TABLE A-8

SHAFT LOAD-SETTLEMENT RESULTS

Boring SB-5, 8-foot-Diameter Drilled Shaft (Seismic Condition)

T-Z CURVE NO.	DEPTH TO CURVE FT.	LOAD TRANSFER PSI	PILE MOVEMENT IN.
25	136.00		
		0	0
		6.049	0.096
		10.47	0.192
		13	0.288
		15.53	0.48
		16.43	0.576
		17.15	0.768
		17.42	1.152
		17.33	1.536
		17.15	9.6

APPENDIX B

HAZARDOUS MATERIALS

APPENDIX B

HAZARDOUS MATERIALS

TABLE OF CONTENTS

LIST OF REPORTS

Technical Report – Hazardous Materials
South Park Bridge Project

Executive Summary

This report presents information on potential environmental concerns related to properties affected by the South Park Bridge Project. The report describes the hazardous material impacts associated with the five bridge alternatives. The project area extends from the intersection of East Marginal Way S. and 16th Avenue S. to the north to the intersection of 14th Avenue S. and S. Trenton Street to the south to 12th Avenue S. and S. Trenton Street to the west in the City of Seattle. The street name changes from 16th Avenue S. to 14th Avenue S. at the mid-span of the bridge. The South Park Bridge's eastern half is located within the City of Tukwila and its western half is located within King County. The South Park Bridge is a north-south arterial that crosses the Duwamish River. The existing bridge was constructed in the early 1930s, replacing a former wooden bridge crossing the Duwamish River at 16th Avenue S.

To focus analysis on properties that could be affected by the rehabilitation or replacement of the bridge, a site screening process was developed and implemented to identify properties with known or suspected environmental issues. Efforts included historical research on industrial and commercial land use, regulatory agency database lists and file reviews, and a windshield survey of the properties.

In total, 58 sites were reviewed as a part of this *Hazardous Material Technical Report*. Some site numbers are used more than once due to sites that are contained in the same structure as is the case for sites numbered 15, 18, and 24, or a site that was divided into multiple sites as is the case with site number 7 (see Table 1). The Initial Site Assessment (ISA) was used to screen the preliminary proposed alternatives. Forty-three sites were eliminated from further consideration because they were either located downgradient, too far away from the planned right-of-way for the South Park Bridge alternatives, or did not pose significant potential for environmental or construction risks based on the site's reported environmental history.

Table 1. List of Properties Assessed

Map Label	Address Location	Business Name	NPL	RCM	UST	LUST	FINDS	Substantially. Contaminated	Reasonably Predictable	PSI Recommended
1	7700 14th Ave. S.	Former Air Company property	/						X	
2	773316th Ave. S.	Boeing Parking Lot								
3	7745 16th Ave. S.	First InterBank of Kirkland! Vacant Lot								
4	7747 16th Ave. S.	Vacant Lot						X		
5	Boeing Plant 2 North Campus Area	Boeing North Side 16th Ave. S.		X				X		X
6	Boeing Plant 2 Bldg 2-41 Area	Boeing South Side 16th Ave. S.		X				X		
7	Boeing Plant 2 Bldg 2-41 Area	Boeing South Side 16th Ave. S.		X				X		X
A	East Sediment Area Duwamish Waterway	NA	X					X		X
B	West Sediment Area Duwamish Waterway	NA	X					X		X
8	1289 S. Rose St.	South Park Marina						X		X
9	1400 S. Thistle St.	Rem Company								X
10	8456 14th Ave. S.	Boat Repair Yard								X
11	1401 S. Thistle St.	House								X
12	1411 S. Thistle St.	Marine					X		X	
– 13	8510 14th Ave. S.	Tire Factory								
14	845714th Ave. S.	Teriyaki Fast Food to Go								
15	1257 S. Sullivan St.	Pattaya Thai Restaurant								
15	8506 14th Ave. S.	Dry Cleaner								
15	8510 14th Ave. S.	DoIEX							X	

Table 1 . List of Properties Assessed (continued)

Map Label	Address/ Location	Business Name	NPL	RCRA	UST	LUST	FINDS	Substantially Contaminated	Reasonably Predictable	PSI Recommended
16	8507 14th Ave. S.	Discount Market								
17	8514 14th Ave. S.	Buena Visita Travel								
17	8520 14th Ave. S.	Salon Expo				X			X	X
18	8524 14th Ave. S.	Herb's Repair/ Taxi Meters			X	X			X	X
18	8524 14th Ave. S.	Coffee Stand				X			X	X
19	8515 14th Ave. S.	Muy Macho Restaurant								
20	8517 14th Ave. S.	Jalisco Mexican Restaurant								
21	8527 14th Ave. S.	Apartment Building								
22	8601 14th Ave. S.	El Molino Rojo								
23	8601 14th Ave. S.	Mexi-Mart								
24	8611 14th Ave. S.	Vacant Building								
24	8611 14th Ave. S.	Musica & Video								
24	8611 14th Ave. S.	South Park Hall								
25	8600 14th Ave. S.	Napoli Pizzeria								X
26	8613 14th Ave. S.	Vacant Brick Building								
27	8617 14th Ave. S.	Kelly's Tavern								
28	8616 14th Ave. S.	Multi-family Residential								
29	8621 14th Ave. S.	Babia's Sewing				X			X	X
30	8620 14th Ave. S.	Former Gas Station and Dry Cleaner, currently vacant building				X			X	X
31	8701 14th Ave. S.	Sea Mar Community Health Center							X	
32	8700 14th Ave. S.	Former Service Station and Machine Shop, current warehouse				X			X	X
33	8709 14th Ave. S.	Juan Colorado Restaurant				X			X	
34	8721 14th Ave. S.	Parking Lot				X			X	

Table 1 . List of Properties Assessed (continued)

Map	Address/ Label Location	Business Name	NPL	RCRA	UST	LUST	FINDS	Substantially Contaminated	Reasonably Predictable	SI :recommended
35	8721 14th Ave. S.	A.D. Swayne Company				X			X	X
36	8722 14th Ave. S.	Sea Mar Community Health Center								
37	No longer in study area									
38	8801 14th Ave. S.	Medical Dental Office								
39	No longer in study area									
40	No longer in study area									
41	No longer in study area									
42	1057 S. Donovan St.	Residential								
43	1230 S. Trenton St.	Residential								
44	1226 S. Trenton St.	Residential								
45	1220 S. Trenton St.	Residential								
46	1210 S. Trenton St.	Residential								
47	1203 S. Donovan St.	Residential								
48	1207 S. Donovan St.	Residential								
49	8410 Dallas Ave. S.	Spencer Industries				X				X
50	860512t Ave. S.	Residential								

NPA = National Priority List

RCRA = Resource Conservation and Recovery

LUST = Leaking Underground Storage Tank UST

= Underground Storage Tank

FINDS = Facility Index System

The 15 sites retained for detailed analysis from the north to the south along the ROW for each of the alternatives include:

Substantially Contaminated Properties

Based on the investigation of 58 sites, the following is a list of substantially contaminated properties located in the project study area.

- **The Boeing Company (Boeing) Plant** 2: Boeing's Plant 2 is a 107-acre aircraft manufacturing and assembly facility that has been in operation since the mid-1930s. Soil and groundwater contamination beneath the facility and sediment contamination along the plant's shoreline of the Lower Duwamish Waterway (LDW) has been confirmed. In 1994, the Environmental Protection Agency (EPA) and Boeing signed an Administrative Order of Consent requiring that Boeing investigate and perform corrective action at Plant 2 under the Resource Conservation and Recovery Act (RCRA). Boeing is currently investigating and performing corrective action cleanups for soil, groundwater, and sediment at Plant 2 under RCRA, and has begun a corrective-measures study to evaluate and select final cleanup actions for the Plant 2 facility and the sediments in the LDW adjacent to the facility.
- **Sediments within the Lower Duwamish Waterway:** The LDW is listed as a Superfund Site currently in the initial phases of investigation for cleanup of sediments, upland source control areas, and storm sewer drainage basins. The northern shore, Site A, of the LDW for this study is bounded by Boeing's Plant 2 facility, and the southern shore, Site B, is bounded by the South Park neighborhood. For this report, the sediments within the LDW are described as two separate sites.

Reasonably Predictable Properties

Based on the investigation of 58 sites, the following is a list of reasonably predictable properties located in the project study area.

- **1289 S. Rose Street:** At the boat repair yard, ship maintenance and repair activities, as well as hull cleaning and painting, were conducted in the open without surface seal. There is a high probability of soil and stormwater contaminations at the site. Axial photographs show heavy staining on the ground surface prior to placement of asphalt service.
- **1400 S. Thistle Street:** Ship maintenance and repair activities, as well as hull cleaning and painting, were conducted in the open without surface seal. There is a high probability of soil and stormwater contaminations at the site.
- **8456 14th Avenue S.:** The northern portion of this property was used as a boat repair yard. Ship maintenance and repair activities, as well as hull cleaning and painting, were conducted in the open without surface seal. There is a high probability of soil and stormwater contaminations at the site.

- **1401 S. Thistle Street:** This is a house with a junkyard surrounding the residence. Several abandoned vehicles are located on the property. Several hundred plastic containers are stored on-site.

- **8520 14th Avenue S.:** This is a former gasoline service station with a reported leaking underground storage tank site with petroleum contamination.

- **8524 14th Avenue S.:** This is an existing auto repair and service station with a reported leaking underground storage tank site with petroleum contamination.

- **8600 14th Avenue S.:** The Napoli Pizzeria restaurant site is in the Washington Department of Ecology (WADOE) records, but they do not indicate that the company currently participates in dangerous waste activities. Based on a site visit from public areas, there is high potential that Asbestos Containing Materials/Lead Based Paint (ACM/LBP) materials are present in the building structure.

- **8621 14th Avenue S.:** This is a former auto repair shop and service station with a reported leaking underground storage tank site with petroleum contamination. Based on a site visit from public areas, there is high potential that ACM/LBP materials are present in the building structure.

- **8620 14th Avenue S.:** This is a former dry cleaning operation with a long operational history. Chemical containers are stored on the property. Based on a site visit from public areas, there is high potential that ACM/LBP materials are present in the building structure.

- **8700 14th Avenue S.:** This former Chevron service station site has a reported leaking underground storage tank site with petroleum contamination in soil and groundwater.

- **8700 14th Avenue S.:** This is a former machine shop operation which is listed as having released chlorinated solvents into the soil. A warehouse currently occupies the site.

- **8410 Dallas Avenue S.:** The Spencer Industries Incorporated aircraft part manufacturing facility has released chlorinated solvents contaminating the groundwater.

As listed above, two of these sites-Boeing's Plant 2 and the LDW-are considered to be substantially contaminated properties. The first site is under RCRA Corrective Action and the other site has been listed as a Superfund Site. Substantially contaminated properties are typically large or have large volumes of contaminated materials, have a long history of industrial or commercial land use, and have contaminants that are persistent or difficult and expensive to manage.

An additional 12 sites are considered to be reasonably predictable properties. These sites are properties where recognized environmental conditions are known based on existing data, or can be predicted based on site observations, previous experience in similar situations, or by using best professional judgment. These sites are typically small, the contaminants are localized and are relatively non-toxic, or abatement/remediation activities are routine (e.g., asbestos abatement or petroleum hydrocarbon contaminated soil remediation).

All of the structures that are located within the South Park Bridge Project study area were constructed when asbestos containing materials and lead-based paints were commonly used.

Several sites have known and/or suspected impacts to the subsurface media within the project area. Contaminated soil, groundwater, and sediment are expected at the substantially contaminated sites and many of the reasonably predictable properties. Depending upon the structures selected to support bridge structure, it also is possible for contaminated groundwater to be encountered during construction. Examples of expected soil and groundwater contaminants include petroleum products, metals, PCBs, and chlorinated solvents. Surface water impacts are anticipated. Soil erosion and other uncontrolled releases that may occur during construction could negatively impact surface waters. Impacts associated with building materials that contain regulated substances (including asbestos-containing materials and lead-based paint) are also a potential concern for all proposed alternatives.

Table 2 lists properties known to be substantially contaminated and suspected to be contaminated for each of the build alternatives. These properties need to be investigated under the Preliminary Site Investigation (PSI) protocols described in WSDOT's Environmental Procedures Manual M31-11. A PSI for these properties will provide cost data for handling contaminated soil, groundwater, and/or sediments. For each alternative, the PSI will also provide cost projections for realistic scheduling, disposal fees, and design of lay-down areas to handle anticipated contaminated materials. Additional properties that are substantially contaminated and/or suspected to be contaminated adjacent to different bridge alternatives need to be addressed to provide and apply accurate environmental costs to each of the build alternatives.

The No Action Alternative would not require property acquisition, so environmentally impacted properties (sites) would not be encountered. The Rehabilitation Alternative encounters five properties that were found to be substantially and/or suspected to be contaminated. The Bascule Bridge Alternative encounters seven properties that were found to be substantially and/or suspected to be contaminated. The Mid-Level Fixed-Span Bridge Alternative encounters 14 properties that were found to be substantially and/or suspected to be contaminated. The High-Level Fixed-Span Bridge Alternative encounters all 38 of the properties (sites) that were found to be substantially and/or suspected to be contaminated. At the time of developing this document, it is assumed that site 25 will be used as a project laydown area.

A PSI should be conducted on each of the 15 properties listed in Table 2 in order to: 1) develop an accurate assessment of the environmental impacts and costs associated with handling contaminated media for each bridge alternative; 2) determine the best design alternative, accurate construction costs, and any increases in project construction schedules related to environmental impacts and handling contaminated media; and 3) estimate off-site disposal costs and hazardous worker monitoring and/or training costs.

Table 2. Substantially and Suspected Contaminated Sites by Bridge Alternative

Site Number Locations	No Action Alternative	Rehabilitation Alternative	Bascule Bridge Alternative	Mid-Level Fixed-Span Bridge Alternative	**High-Level Fixed-Span Bridge** Alternative
5 & 7 (Boeing)		X	X	X	X
A		X	X	X	X
B		X	X	X	X
8		X	X	X	X
9		X	X	X	X
10		X	X	X	X
11		X	X	X	X
17			X	X	X
18			X	X	X
25		X	X	X	X
29				X	X
30				X	X
32					X
35					X
49		X	X	X	X

Estimated costs for mitigation measures are included in this report. The estimated cost for developing and conducting site-specific preliminary investigations is provided. Limited costs estimates for environmental impacts related to construction activities are provided due to the unavailability of specific contamination concentration data and design information for the project. Unit cost estimates are provided for each of the suspected impacts that may affect King County Department of Transportation (KCDOT) analysis of properties to purchase and/or lease liability, worker safety, and construction activities. The estimates are based on conceptual design, environmental data, and site information gathered during site visits to adjacent public areas.

Proposed mitigation measures include preparing a contaminated media contingency plan that would provide specific guidance for managing contaminated media during construction activities for the selected alternative. The contaminated media contingency plan should address risk-based cleanup and recommend provisions for field screening options, notification requirements, and soil stockpile management. Groundwater mitigation measures include alternatives for construction activities that minimize or avoid intercepting the groundwater table, if possible. Surface water mitigation measures are addressed by way of a Spill Prevention Control and Countermeasure (SPCC) plan. Mitigation measures for demolition debris rely heavily on recycling. Possible impacts related to federal and state Superfund authorities within the project area should be mitigated through early coordination with the EPA and WADOE, respectively.

Final Preliminary Site Investigation Report for the South Park Bridge Project
Wilbur Consulting

Executive Summary

A Preliminary Site Investigation (PSI) was performed for the 14th/16th Avenue South Park Bridge Project to determine if contamination is present in the subsurface soil, groundwater, and sediment near the footprint of the five proposed design alternatives. The PSI focuses on the three proposed bridge replacement alternatives: Bascule, Mid-Level Fixed-Span, and High-Level Fixed-Span. Washington State Department of Transportation (WSDOT) guidelines (2001) were followed in developing and conducting this PSI.

Study Objectives

The PSI was determined to be necessary based on the findings of the *Draft Hazardous Material Technical Report* (Wilbur 2002) completed in 2002. The environmental analysis in that report focused on the 58 properties that would be affected by the proposed replacement alternatives for the South Park Bridge. As a part of that analysis, an Initial Site Assessment (ISA) screening process was conducted based on WSDOT guidelines for identifying properties with known and/or suspected contamination issues.

Of the 58 individual properties assessed under the ISA, 15 sites were identified as having confirmed and documented environmental issues and/or were suspected to have a high probability of environmental risk based on current or past land use.

A PSI is usually conducted on an individual property that needs to be purchased in order to complete a transportation construction project. However, this PSI focused on the eight locations identified as having environmental risks that are located near the footprint of the proposed bridge replacement alternatives (Bascule, Mid-Level Fixed-Span, and High-Level Fixed-Span). Based on the findings of the *Draft Hazardous Material Technical Report* (Wilbur 2002), three sites were found to be substantially contaminated and another twelve were identified as being reasonably predictable to be contaminated. Eight of the fifteen sites were chosen as sample locations because these eight sites would best represent contamination present in the areas of the proposed bridge alternatives. These substantially contaminated and reasonably predictable sites are discussed further in Section 3.1 of the PSI.

WSDOT has adopted the terms *substantially contaminated* and *reasonably predicable* from the Federal Highway Administration (FHWA) environmental guidelines (2001) developed to describe and generally identify the severity of potential environmental risks and to quantify the environmental risks and liabilities. Sites are identified as being either substantially contaminated or reasonably predictable based on the findings of the *Draft Hazardous Materials Technical Report* (Wilbur 2002).

Substantially contaminated sites create more liability either in construction or acquisition than sites identified as reasonably predictable. A substantially contaminated site tends to be a larger site with a long history of heavy industrial operations that handled hazardous materials or has documented releases of hazardous materials to the environment. These sites are usually listed by federal, state, or local regulatory agencies as being contaminated.

Reasonably predictable sites tend to be smaller operations that usually handled or stored less toxic hazardous materials than substantially contaminated sites. Reasonably predictable sites may or may not be listed by federal, state, or local regulatory agencies as being contaminated. Each type of site presents different degrees of environmental risks and liabilities to a transportation construction project. Generally, substantially contaminated sites have the greatest degree of impacts. Sites found to be substantially contaminated or reasonably predictable require a detailed investigation of the nature and extent of contamination in order to quantify the environmental risks to a transportation project's cost and schedule.

Environmental cleanup costs are directly related to the following items: 1) volume of contaminated soil, sediment, and/or groundwater to be removed and disposed of during construction activities; 2) any required dewatering and/or stabilization of excavation material that is contaminated prior to its transport and disposal; 3) the design, permitting, and construction of temporary storage areas to handle contaminated material; and 4) project schedule delays related to permitting and regulatory oversight requirements for the management of contaminated soil, sediment, and groundwater.

Field Work

An extensive work plan was developed for the PSI fieldwork activities to investigate the nature and extent of potential environmental risks in the subsurface soil, groundwater, and sediments near the footprints of the South Park Bridge alternatives. To identify any potential environmental risks that may be present, borings were drilled and sampled at locations where structural support columns for the bridge alternatives would be constructed. The proposed PSI fieldwork activities were outlined in the work plan. Field activities were carried out during a 65-calendar-day period from June 24 to August 7, 2003.

Fifteen borings were drilled into the soil of the upland areas and two borings were drilled into the sediments of the Duwamish Waterway near the footprint of the replacement alternatives. Six of the upland borings were drilled to a depth of 100 feet below the existing ground surface (bgs) by a mud rotary drilling technique to collect geotechnical and soil samples. Seven borings were drilled using a hollow stem auger technique to collect soil samples. However, because of an obstruction, one of these borings was abandoned. The remaining six borings were drilled to a depth of 25 bgs and completed as monitoring wells for the collection of environmental samples. Groundwater samples were collected from each of the monitoring wells. The two in-water borings were drilled to a depth of approximately 100 feet below the water sediment interface in the Duwamish Waterway channel to collect geotechnical and environmental soil samples.

Sampling Activities

The sample analysis plan (SAP) section of the work plan for the PSI outlined the number of soil samples to be collected during the drilling of the environmental and the geotechnical borings. The SAP projected that 84 soil samples would be collected during environmental sampling fieldwork from the six environmental and eight geotechnical borings. The SAP projected that environmental samples were to be collected at 2.5-foot intervals from the environmental borings and from 50-, 75- and 100-foot intervals in the geotechnical borings. Ideally, 60 soil samples from the six environmental borings and 24 soil samples from the eight geotechnical borings were to be collected.

A total of 72 soil samples were collected from the 15 borings and were submitted to the laboratory for analysis. Six sets of groundwater samples were collected from the six environmental borings that were completed as monitoring wells and submitted to the laboratory for analysis. An additional 11 sediment samples were collected from the two borings that were drilled in the water and submitted to the laboratory for analysis.

The actual number of environmental soil samples collected was 54 from the environmental borings and 18 from the geotechnical borings. Several factors led to the reduced number of soil samples collected, including: 1) loss of sampling material when retrieved from the bore hole; 2) saturated fine-grained materials drained out of sampling equipment prevented retrieval below the water table; and 3) asphalt and/or concrete with thickness greater than 2.5 feet at the surface reduced the amount of material available for sampling from some borings.

The loss of sample material during drilling activities is common when collecting environmental samples. Collection equipment used for environmental sampling has a larger diameter barrel than geotechnical sample collection equipment, and this can result in spillage of the sample material when used in a geotechnical well. Loss of sample tends to increase in a boring below the water table because the material is saturated and easily drains out. No samples were taken of asphalt or concrete surface areas, because no soil or groundwater would be present. Groundwater and soil below asphalt and concrete were sampled. Overall, the number of environmental samples collected and analyzed provided a sufficient data set in which to characterize subsurface soils for the presence of hazardous materials in the saturated and unsaturated soil within the project area

Laboratory Results

Soil Results

A total of 17 soil samples from five borings were found to contain compounds with concentrations above Washington State Model Toxics Control Act (MTCA) Method A levels for soil at an unrestricted land use site.

When these soils are encountered during construction, appropriate actions must be taken to store and contain them, prevent stormwater runoff, control fugitive emissions, ensure proper site management, and document disposal to an appropriate regulated landfill. A designated contaminated materials storage area at the construction site is typically used for managing and controlling contaminated materials, thereby addressing these concerns. Construction workers will be required to comply with the Hazardous Worker Training requirements set forth in Code of Federal Regulations 29 CFR 1910.120 and the Washington Administrative Code (WAC) 296-62 Part P in order to work in an area where contaminated soil is encountered.

Groundwater Results

One groundwater set of samples from a single monitoring well exceeded the MTCA Method A levels for groundwater at an unrestricted land use site.

When this groundwater is encountered during bridge construction, the contaminated groundwater must be managed by either discharge to the sanitary sewer or by storing it in onsite storage tanks for ultimate disposal offsite. The management of contaminated groundwater should be documented.

Sediment Results

Six of the eleven sediment samples collected from the upper 10 feet of borings in the Duwamish Waterway channel exceeded U.S. Army Corps of Engineers Puget Sound Dredge Disposal levels for several compounds.

Potential in-water excavation activities of or involving these sediments will require the development and documentation of a detailed sediment management plan. A bio-analysis study may also be required in order to obtain a dredging permit from the U.S. Army Corps of Engineers for the in-water construction activities.

Summary of Study Findings

The soil, groundwater, and sediment from all of the borings were sampled for environmental parameters. The laboratory analysis results and recommendations to mitigate the identified impacts are presented in detail in the body of this report and summarized below.

The PSI confirmed several locations near the footprint of the proposed bridge replacement alternatives where subsurface contamination was found in the soil, groundwater, and sediment. In general, based on the findings, it appears that the larger the footprint of the proposed bridge replacement alternative, the greater and more complex the potential contamination of the soil, groundwater, and sediment will likely be.

Soil located within the north and the south bridge approach areas are also contaminated. Soil contamination was detected in the soil column for all three of the proposed replacement bridge alternatives. The contamination detected is a mix of chemical compounds, located at various depths in the upper 14 feet of the soil column, depending on the location of the boring where the sample was collected.

Groundwater contaminated with chlorinated solvents was encountered in the south bridge approach areas of the proposed alignments of the Mid-Level Fixed-Span and High-Level Fixed-Span bridge alternatives. These chlorinated solvents were more than likely released to the groundwater from local business operations adjacent to the south bridge approach area. Groundwater samples collected within the footprint of the proposed Bascule Bridge Alternative were not contaminated.

Sediment contaminated with polychlorinated biphenyls (PCB) and other chemical compounds were detected within all samples collected in the upper 7 feet of the water-sediment interface on both sides of the Duwamish Waterway. PCB and chemical compounds detected in the samples are likely related to urban and industrial runoff entering the Duwamish Waterway. Sediment samples collected at 50-, 75- and 100-foot depths below the water-sediment interface were found to contain concentrations of metals above regulated levels. The metals contained within these deep sediment samples, however, are naturally occurring elements related to source rock (the source materials from which they were eroded and then deposited) and are not related to human operations.

The environmental risks of each proposed bridge replacement alternative on soil, groundwater, and sediment vary. Environmental risks would likely affect project costs and schedule. Each alternative design is affected by contamination that is present in the soil and sediment, and two alternatives are affected by groundwater contamination. Contaminated soil and sediment will need to be addressed no matter which proposed alternative is selected. Groundwater contamination impacts affect the Mid-Level Fixed-Span and High-Level Fixed-Span bridge alternatives. Contaminated groundwater and soil and sediment impacts will have to be addressed if either the Mid-Level Fixed-Span or the High-Level Fixed-Span bridge alternative design is selected.

Conclusions and Recommendations

WCI's findings in the PSI indicate that the South Park Bridge project area has several sites where the subsurface materials have been contaminated. However, in the samples analyzed, these impacts were not found to exceed federal hazardous waste or Washington State dangerous waste disposal requirements. Nonetheless, some results exceed Washington State Model Toxics Control Act Method A levels and U.S. Army Corps Puget Sound Dredge Disposal levels, and will require special management practices when encountered.

Costs for handling and disposing of these contaminated materials will be much less than they would be if the subsurface materials exceeded federal hazardous waste or Washington State dangerous waste disposal requirements. Estimated costs for handling the contaminated subsurface materials range from $851,000 (Mid-Level Fixed-Span Bridge Alternative) to $1,835,000 (Rehabilitation Alternatives) using 24-inch-steel-pipe piles for construction and $469,000 (Mid-Level Fixed-Span Bridge Alternative) to $631,000 (Rehabilitation Alternatives) using concrete drilled shafts for construction. How these costs were arrived at and costs regarding the proposed Bascule Bridge Alternative and the High-Level Fixed-Span Bridge Alternative are described in this report.

A review of existing aquifer test data for a large-scale dewatering study adjacent to the project site was done as part of the PSI. The total estimated cost for each dewatering system, for a 50-foot-by-50-foot area excavation in which the water table is lowered 8 feet for a period of 74 days, is $762,900. This estimate is based on dewatering system estimated costs of $552,000 and system operational estimated costs of $210,900. Dewatering costs would be in addition to the costs associated with handling and disposal of contaminated materials. Wilbur Consulting, Inc. (WCI) recommends that excavations and/or other construction activities that would require a dewatering system be avoided if at all possible for the South Park Bridge Project. If used, dewatering costs would be incurred for all Build Alternatives.

Based on the analytical results, the following actions are recommended no matter which of the potential project alternatives is selected.

- WCI recommends that a focused environmental investigation should be conducted in the actual support column locations and within utility trenching areas of the approach ramp areas after the final bridge design has been selected. This focused investigation would provide the best data set for subsurface conditions that would actually be encountered during construction.

- WCI recommends that a detailed environmental investigation of the area under the wharf located on The Boeing Company's (Boeing) property along the shore of the Duwamish Waterway should be conducted after selecting a final bridge design alternative. The wharf area has not been investigated in detail at this time. Based on WCI field surveys of the environment under the wharf during in-water sampling activities, it appears that the wharf is built on fill consisting of construction debris and other previously used materials of indeterminate origin. Thus, the potential for environmental and geotechnical problems that may affect bridge construction activities appears to be high. The wharf area was identified as being potentially contaminated in the *Hazardous Materials Technical Report* (Wilbur, 2004), but it was not possible to sample this area during the PSI.

- WCI recommends that a stockpile sampling and analysis plan should be developed to ensure quick processing of unforeseen contaminated materials encountered during bridge construction. Establishment of a process with procedures for sampling, analyzing, and determining disposal requirements for unforeseen contaminated soil is essential for successful construction of this project.

- WCI recommends that non-invasive construction techniques (e.g., push piles, when possible) should be used to limit the volume of excavation material generated during bridge construction. By employing non-invasive construction techniques or techniques that limit the volume of the excavated material generated, smaller amounts of contaminated excavation material will need to be stockpiled, analyzed, transported, and disposed of. This will keep construction costs lower.

- WCI recommends that excavations and/or other construction activities that would require a dewatering system be avoided if at all possible for the South Park Bridge Project.

- WCI recommends that two excavated soil stockpile storage areas should be constructed (one on each side of the Duwamish Waterway) to contain and manage contaminated spoil materials generated during construction. Transportation of contaminated soils around and through the construction site, crossing local jurisdictional boundaries and the Duwamish Waterway, will create unnecessary risks and permitting issues for the project. Contaminated soil generated on the north shore of the Duwamish Waterway should remain on the north shore. This is also true for any contaminated soil generated on the south shore.

References

Federal Highways Administration, 2001, *Environmental Guidebook* FHWA, U.S. Department of Transportation.

Washington Administrative Code, 2001, *Model Toxics Control Act Cleanup Regulation. Chapter 173-340-900.* Olympia, Wash.

Washington State Department of Transportation, 2001, *Environmental Procedures Manual, M31-11,* March.

Wilbur Consulting, Inc., 2002, *South Park Bridge Project Draft Hazardous Materials Technical Report.* Prepared for the King County Department of Transportation, Seattle, Wash.

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APPENDIX C

IMPORTANT INFORMATION ABOUT YOUR GEOTECHNICAL REPORT

IMPORTANT INFORMATION ABOUT YOUR GEOTECHNICAL/ENVIRONMENTAL REPORT

CONSULTING SERVICES ARE PERFORMED FOR SPECIFIC PURPOSES AND FOR SPECIFIC CLIENTS.

Consultants prepare reports to meet the specific needs of specific individuals. A report prepared for a civil engineer may not be adequate for a construction contractor or even another civil engineer. Unless indicated otherwise, your consultant prepared your report expressly for you and expressly for the purposes you indicated. No one other than you should apply this report for its intended purpose without first conferring with the consultant. No party should apply this report for any purpose other than that originally contemplated without first conferring with the consultant.

THE CONSULTANT'S REPORT IS BASED ON PROJECT-SPECIFIC FACTORS.

A geotechnical/environmental report is based on a subsurface exploration plan designed to consider a unique set of project-specific factors. Depending on the project, these may include: the general nature of the structure and property involved; its size and configuration; its historical use and practice; the location of the structure on the site and its orientation; other improvements such as access roads, parking lots, and underground utilities; and the additional risk created by scope-of-service limitations imposed by the client. To help avoid costly problems, ask the consultant to evaluate how any factors that change subsequent to the date of the report may affect the recommendations. Unless your consultant indicates otherwise, your report should not be used: (1) when the nature of the proposed project is changed (for example, if an office building will be erected instead of a parking garage, or if a refrigerated warehouse will be built instead of an unrefrigerated one, or chemicals are discovered on or near the site); (2) when the size, elevation, or configuration of the proposed project is altered; (3) when the location or orientation of the proposed project is modified; (4) when there is a change of ownership; or (5) for application to an adjacent site. Consultants cannot accept responsibility for problems that may occur if they are not consulted after factors which were considered in the development of the report have changed.

SUBSURFACE CONDITIONS CAN CHANGE.

Subsurface conditions may be affected as a result of natural processes or human activity. Because a geotechnical/environmental report is based on conditions that existed at the time of subsurface exploration, construction decisions should not be based on a report whose adequacy may have been affected by time. Ask the consultant to advise if additional tests are desirable before construction starts; for example, groundwater conditions commonly vary seasonally.

Construction operations at or adjacent to the site and natural events such as floods, earthquakes, or groundwater fluctuations may also affect subsurface conditions and, thus, the continuing adequacy of a geotechnical/environmental report. The consultant should be kept apprised of any such events, and should be consulted to determine if additional tests are necessary.

MOST RECOMMENDATIONS ARE PROFESSIONAL JUDGMENTS.

Site exploration and testing identifies actual surface and subsurface conditions only at those points where samples are taken. The data were extrapolated by your consultant, who then applied judgment to render an opinion about overall subsurface conditions. The actual interface between materials may be far more gradual or abrupt than your report indicates. Actual conditions in areas not sampled may differ from those predicted in your report. While nothing can be done to prevent such situations, you and your consultant can work together to help reduce their impacts. Retaining your consultant to observe subsurface construction operations can be particularly beneficial in this respect.

A REPORT'S CONCLUSIONS ARE PRELIMINARY.

The conclusions contained in your consultant's report are preliminary because they must be based on the assumption that conditions revealed through selective exploratory sampling are indicative of actual conditions throughout a site. Actual subsurface conditions can be discerned only during earthwork; therefore, you should retain your consultant to observe actual conditions and to provide conclusions. Only the consultant who prepared the report is fully familiar with the background information needed to determine whether or not the report's recommendations based on those conclusions are valid and whether or not the contractor is abiding by applicable recommendations. The consultant who developed your report cannot assume responsibility or liability for the adequacy of the report's recommendations if another party is retained to observe construction.

THE CONSULTANT'S REPORT IS SUBJECT TO MISINTERPRETATION.

Costly problems can occur when other design professionals develop their plans based on misinterpretation of a geotechnical/environmental report. To help avoid these problems, the consultant should be retained to work with other project design professionals to explain relevant geotechnical, geological, hydrogeological, and environmental findings, and to review the adequacy of their plans and specifications relative to these issues.

BORING LOGS AND/OR MONITORING WELL DATA SHOULD NOT BE SEPARATED FROM THE REPORT.

Final boring logs developed by the consultant are based upon interpretation of field logs (assembled by site personnel), field test results, and laboratory and/or office evaluation of field samples and data. Only final boring logs and data are customarily included in geotechnical/environmental reports. These final logs should not, under any circumstances, be redrawn for inclusion in architectural or other design drawings, because drafters may commit errors or omissions in the transfer process.

To reduce the likelihood of boring log or monitoring well misinterpretation, contractors should be given ready access to the complete geotechnical engineering/environmental report prepared or authorized for their use. If access is provided only to the report prepared for you, you should advise contractors of the report's limitations, assuming that a contractor was not one of the specific persons for whom the report was prepared, and that developing construction cost estimates was not one of the specific purposes for which it was prepared. While a contractor may gain important knowledge from a report prepared for another party, the contractor should discuss the report with your consultant and perform the additional or alternative work believed necessary to obtain the data specifically appropriate for construction cost estimating purposes. Some clients hold the mistaken impression that simply disclaiming responsibility for the accuracy of subsurface information always insulates them from attendant liability. Providing the best available information to contractors helps prevent costly construction problems and the adversarial attitudes that aggravate them to a disproportionate scale.

READ RESPONSIBILITY CLAUSES CLOSELY.

Because geotechnical/environmental engineering is based extensively on judgment and opinion, it is far less exact than other design disciplines. This situation has resulted in wholly unwarranted claims being lodged against consultants. To help prevent this problem, consultants have developed a number of clauses for use in their contracts, reports and other documents. These responsibility clauses are not exculpatory clauses designed to transfer the consultant's liabilities to other parties; rather, they are definitive clauses that identify where the consultant's responsibilities begin and end. Their use helps all parties involved recognize their individual responsibilities and take appropriate action. Some of these definitive clauses are likely to appear in your report, and you are encouraged to read them closely. Your consultant will be pleased to give full and frank answers to your questions.

The preceding paragraphs are based on information provided by the
ASFE/Association of Engineering Firms Practicing in the Geosciences, Silver Spring, Maryland