Draft Geotechnical Report
SR 202 Sammamish River Bridge Project
Type, Size, and Location Study
Woodinville, Washington

August 21, 2007

EXHIBIT 8
PAGE 1 OF 64

Submitted To:
Mr. Rex Meyer
DMJM Harris
10900 NE 8th Street, Suite 750
Bellevue, Washington 98004

RECEIVED
JAN 29 2012
CITY OF WOODINVILLE
DEVELOPMENT SERVICES

By:
Shannon & Wilson, Inc.
400 N 34th Street, Suite 100
Seattle, Washington 98103

21-1-20664-001
DRAFT

EXHIBIT 8
PAGE 2 OF 64

TABLE OF CONTENTS

Page

1.0 INTRODUCTION 1

2.0 SITE AND PROJECT DESCRIPTION 1

3.0 SUBSURFACE EXPLORATIONS 2

4.0 GEOTECHNICAL LABORATORY TESTING 3

5.0 NONDESTRUCTIVE PAVEMENT TESTING AND PAVEMENT DESIGN 3

6.0 GEOLOGY AND SUBSURFACE CONDITIONS 3
- 6.1 Regional Setting 4
- 6.2 Subsurface Conditions 4

7.0 ENGINEERING ANALYSIS AND RECOMMENDATIONS 5
- 7.1 Earthquake Engineering 5
  - 7.1.1 Ground Motions 5
  - 7.1.2 Earthquake Hazards 6
- 7.2 Deep Foundations for Bridge Support 7
  - 7.2.1 Axial Capacity 8
  - 7.2.2 Embankment Fill Effects on Existing and Proposed Foundations 9
  - 7.2.3 Lateral Resistance 10
- 7.3 Embankments 10
  - 7.3.1 Estimated Settlement 11
  - 7.3.2 Permanent and Temporary Slopes 12
  - 7.3.3 Settlement Monitoring 12
- 7.4 Loads on Buried Pipelines 13
- 7.5 Retaining Walls 13
  - 7.5.1 Mechanically Stabilized Earth (MSE) Walls 14
  - 7.5.2 Facings 14
  - 7.5.3 Reinforced and Retained Backfill Materials 14
  - 7.5.4 Drainage Provisions 15

8.0 CONSTRUCTION CONSIDERATIONS 15
- 8.1 Site Preparation 15
- 8.2 Structural Fill Placement and Compaction 16
- 8.3 Suitability of Excavated Material for Use as Fill 16
- 8.4 Excavation and Temporary Groundwater Control 17

EXHIBIT 8
PAGE 3 OF 64

Page

8.5 Wet Weather Earthwork 17
8.6 Driven Pile Installation 18
8.6.1 Pile-driving Equipment 18
8.6.2 Pile-driving Conditions 18
8.6.3 Wave Equation Analysis 19
8.6.4 Test Pile Program 19
8.6.5 Monitoring Pile Driving 20
8.7 Drilled Shaft Installation 20
8.7.1 Overview 20
8.7.2 Methods of Construction 21
8.8 Pile-driving and Shaft Installation Vibrations, Movement Monitoring, and Noise Levels 24
8.9 Plans and Specifications and Construction Observation 25

9.0 RECOMMENDED FUTURE STUDIES 25

10.0 LIMITATIONS 25

11.0 REFERENCES 28

LIST OF TABLES

Table No.

1 Summary of Axial Capacity Analyses
2 Recommended Parameters for Lateral Resistance Analysis Using LPILE^{PLUS}
3 Recommendations for Structural Fill Compaction

SHANNON & WILSON, INC.
EXHIBIT 8
PAGE 4 OF 64

LIST OF FIGURES

Figure No.

1	Vicinity Map
2	Site and Exploration Plan (2 sheets)
3	Generalized Subsurface Profile A-A'
4	Estimated Axial Capacity for 16-inch-dia. CIP Concrete Pile, West Abutment
5	Estimated Axial Capacity for 24-inch-dia. CIP Concrete Pile, West Abutment
6	Estimated Axial Capacity for 4-ft-dia. Drilled Shaft, West Abutment
7	Estimated Axial Capacity for 16-inch-dia. CIP Concrete Pile, East Abutment
8	Estimated Axial Capacity for 24-inch-dia. CIP Concrete Pile, East Abutment
9	Estimated Axial Capacity for 4-ft-dia. Drilled Shaft, East Abutment
10	Loads on Buried Utilities

LIST OF APPENDICES

Appendix

A	Subsurface Explorations
B	Geotechnical Laboratory Test Results
C	Pavement Consultants, Inc. Pavement Widening and Overlaying Report
D	Important Information About Your Geotechnical Report

EXHIBIT 8
PAGE 5 OF 64

DRAFT GEOTECHNICAL REPORT
SR 202 SAMMAMISH RIVER BRIDGE PROJECT
TYPE, SIZE, AND LOCATION STUDY
WOODINVILLE, WASHINGTON

1.0 INTRODUCTION

The State Route (SR) 202 Sammamish River Bridge Project in Woodinville, Washington, will include widening the existing, two-lane vehicular bridge into a four-lane bridge. This draft report presents the results of our subsurface explorations, geotechnical laboratory tests, and engineering studies for the Type, Size, and Location (TS&L) design. Field explorations and laboratory testing were performed to evaluate subsurface conditions. Engineering studies and analyses were conducted to develop design recommendations for deep foundation bridge support and approach embankment construction. Nondestructive pavement tests and pavement widening and overlaying were conducted by a qualified subcontractor.

Our services were performed in general accordance with our proposal dated July 12, 2006, and our additional services proposal for nighttime fieldwork dated February 13, 2007. Notice to proceed with this work was provided on November 30, 2006, by Mr. Aaron Silver of DMJM Harris. E-mail authorization for additional services was provided on February 20, 2007, by Mr. Silver.

2.0 SITE AND PROJECT DESCRIPTION

The project site is shown in the Vicinity Map, Figure 1. The Site and Exploration Plan, Figure 2, shows the existing 33-foot-wide bridge, built in 1963. The bridge has four piers and is about 167 feet long in an east-west direction. The center piers are located near the river's edge. The as-built drawing provided by DMJM Harris shows the bridge is founded on deep foundations. The abutments, designated Bents 1 and 4, are founded on vertical and battered, creosote-treated, timber piles. The interior piers, designated Bents 2 and 3, are founded on 16-inch-diameter, 53-ton, prestressed concrete piles. The timber and concrete pile lengths are unknown.

EXHIBIT 8
PAGE 6 OF 64

As shown in Figure 3, Generalized Subsurface Profile A-A', the ground surface slopes up from the river at about 2.75 Horizontal to 1 Vertical (2.75H:1V) on the west side of the bridge. On the east side, the slope rises at about 1.4H:1V both above and below a level, 13-foot-wide concrete bicycle and pedestrian path. The bicycle/pedestrian path is about halfway up the slope between the river and the bottom of the bridge deck. The Sammamish River level is approximately elevation 21 feet (North American Vertical Datum of 1988 [NAVD88]). The road grade is at about elevations 46 and 42 feet near the east and west abutments, respectively.

Several alternatives are being considered for the proposed bridge widening. The bridge may be widened all to one side or partially on both sides to achieve the desired width. The alternatives include both single and three-span options. According to DMJM Harris, the new piers will carry service loads of about 1,540 to 2,560 kips. The new piers will be founded on either driven, steel, closed-end pipe piles filled with concrete or drilled shafts.

The proposed abutment widening will include placing soil embankment fill to match the existing grades. We understand that the existing bridge must remain in use throughout construction. Based on the proposed embankment fill dimensions and locations, the existing foundations will be impacted by settlement-induced downdrag.

3.0 SUBSURFACE EXPLORATIONS

The field exploration program consisted of drilling two borings, designated B-1 and B-2 (OW). An observation well was installed in boring B-2; therefore, it has the observation well (OW) designation. The borings were located behind the bridge abutments. The explorations were drilled to depths of 120.5 and 100.5 feet in borings B-1 and B-2, respectively.

The Washington State Department of Transportation (WSDOT) drilled three borings in 1963 for the existing bridge design. Those 1963 borings are designated H-1 through H-3; they were drilled to depths ranging from 36 to 102 feet.

Figure 3 presents the generalized subsurface profile along the bridge alignment. The profile incorporates both the 1963 and 2006 boring logs. The elevations of the 1963 WSDOT borings were converted from the U.S. Geological Survey (USGS) datum to NAVD88. The 1963 borings were drilled prior to embankment fill placement for the current bridge.

EXHIBIT 8
PAGE 7 OF 64

The drilling and sampling methodology and procedures as well as the observation well installation and development techniques for borings B-1 and B-2 (OW) are discussed in Appendix A, Subsurface Explorations. The 2007 and 1963 exploration logs are presented in Appendix A as Figures A-2 through A-6. As described in Appendix A, visual and olfactory field screening methods performed during the explorations did not indicate the potential presence of contamination in our subsurface explorations.

4.0 GEOTECHNICAL LABORATORY TESTING

To aid in our engineering analyses, laboratory tests were performed on select soil samples from borings B-1 and B-2 (OW) to determine basic index and engineering properties. The geotechnical laboratory testing was performed in the Shannon & Wilson, Inc. laboratory in Seattle, Washington. The program included visual classification, water content determinations, grain size distributions, an Atterberg Limits determination, and an organic content. Laboratory testing was performed in general accordance with the American Society for Testing and Materials (ASTM, 2007) standard test procedures. The laboratory test results were incorporated into the current exploration log soil descriptions included in Appendix A. Test procedure descriptions and test result summaries are presented in Appendix B.

5.0 NONDESTRUCTIVE PAVEMENT TESTING AND PAVEMENT DESIGN

Pavement Consultants, Inc. (PCI) performed nondestructive falling weight deflectometer (FWD) testing on the existing roadway. The FWD testing was done between the railroad track near Woodinville-Redmond Road to the west and 131st Avenue N.E. to the east. PCI used the field test results, the pavement cross section information obtained from the borings, and the City of Woodinville traffic information to design the proposed widening and overlaying pavements. Their field test results and pavement design recommendations are presented in Appendix C.

6.0 GEOLOGY AND SUBSURFACE CONDITIONS

The Generalized Subsurface Profile A-A' in Figure 3 shows our interpretations of the subsurface conditions and the geologic units.

6.1 Regional Setting

Woodinville lies in the eastern portion of the Puget Lowland, a north-south-trending topographic and structural depression between the Cascade Mountains to the east and the Olympic Mountains to the west. The Lowland is filled with a complexly interstratified sequence of glacial and non-glacial sediments that were deposited during several glacial and interglacial periods over the past 2 million years.

The last glacial ice sheet, known as the Vashon Stade of the Fraser glaciation, receded from the area about 13,500 years ago. Since then, erosion and alluvial processes have removed and reworked some glacial deposits and deposited additional soils over the dense glacial and interglacial material. At the project site, Vashon glacial deposits are absent and recent alluvium and fill deposits directly overlie non-glacial soils laid down during the Olympia interglacial period.

6.2 Subsurface Conditions

Generally, the soils that underlie the project site, from youngest to oldest, consist of varying thicknesses of fill, interbedded Holocene alluvium and lacustrine deposits; and pre-Vashon, non-glacial alluvium and lacustrine deposits. The fill encountered in the borings consists of sand and gravel with varying amounts of silt. In borings B-1 and B-2 (OW), the fill ranges in thickness between 12 and 18 feet.

Underlying the fill is a sequence of interbedded Holocene lacustrine deposits, consisting of clayey silt and silty clay, and Holocene alluvium, consisting of fine to medium sand with varying amounts of silt and gravel. Scattered, thin, organic-rich seams exist within the interbedded recent alluvium and lacustrine units on the west side of the bridge in boring B-1.

Below the base of the Holocene deposits between approximate elevations 5 and 10 feet, interbedded pre-Vashon non-glacial alluvium and pre-Vashon, non-glacial, lacustrine deposits exist to the bottom of the explorations. The alluvium consists of clean to slightly silty, fine to medium sand, and the lacustrine deposit consists of fine sand with scattered, silty clay seams. The pre-Vashon deposits have been overridden by glacial ice and are glacially consolidated.

EXHIBIT 8
PAGE 9 OF 64

As shown in Figure 3, the 1963 borings encountered similar materials, but the blow counts at depth are inconsistent with those observed in the two recent borings. Given the year they were drilled, it is likely that the WSDOT borings were advanced using hollow-stem auger techniques. If water or drilling mud were not added to the inside of the augers while they were drilling through the relatively clean sand layers below groundwater, the sand may have heaved up into the augers because of the imbalanced hydrostatic pressure head. The drillers would then have washed out the soil heave from inside the auger, and then sampled below the auger tip. The heave would result in a looser, disturbed zone beneath the augers where the soil samples/blow counts were taken. The boring log for H-2 indicates that rotary methods ("wash boring") were used through the sand and gravel layer; the log does not say if they used drilling mud or just water. While preparing our subsurface profile, we correlated the soil descriptions from the 1963 borings to the soil units observed in our recent borings. Because our recent borings were drilled with mud-rotary techniques and no signs of heave were noted, we used our recent borings' blow counts to classify the density and consistency of the soil layers.

Groundwater was noted during drilling; however, it is difficult to accurately determine the groundwater level while drilling with mud-rotary techniques. During drilling, groundwater was observed at elevation 29 and 11 feet NAVD88 in borings B-1 and B-2 (OW), respectively. These two groundwater observations are variable; we would anticipate that the groundwater level would be near to or slightly higher than the Sammamish River water level. A groundwater level reading was taken in B-2 (OW) on May 31, 2007, indicated a groundwater elevation of 20 feet.

7.0 ENGINEERING ANALYSIS AND RECOMMENDATIONS

7.1 Earthquake Engineering

7.1.1 Ground Motions

We understand that the bridge will be analyzed in accordance with the 2007 AASHTO LRFD Bridge Design Specifications as outlined by the American Association of State Highway and Transportation Officials (AASHTO). The AASHTO LRFD criteria indicate that bridge design and evaluations should be based on earthquake ground motions with a 10 percent chance of exceedence in 50 years (475-year return period). Current AASHTO maps indicate a peak ground acceleration (PGA) of 0.29g for this return period. These maps, however, do no

EXHIBIT 8
PAGE 10 OF 64

incorporate the probability of movement on the Cascadia Subduction Zone of the coast of Washington indicated by recently discovered geologic evidence. The U.S. Geological Survey (USGS) has produced regional probabilistic seismic hazard ground motion maps for the entire country (Frankel and others, 2002). The 2002 version of the maps have subsequently been incorporated into the Washington State Department of Transportation Bridge Design Manual. The USGS maps indicate that for a recurrence interval of 475 years, the site PGA is 0.30g. Consequently, we recommend that a site PGA of 0.30g be used in the seismic analyses.

The AASHTO LRFD criteria requires special site- and structure-specific studies be performed if the site is located close to an active fault, if long-duration earthquakes are expected in the region, and if the importance of the bridge is such that a longer return period should be considered for design. While the bridge is located close to the Southern Whidbey Island Fault Zone (SWIFZ), the return period for large earthquakes on the fault is on the order of thousands of years and much longer than 475-year return period ground motions used in design. The 2002 USGS maps have incorporated the SWIFZ and all pertinent regional seismic sources, including sources of long-duration earthquakes, into the development of their probabilistic seismic hazard ground motions. The bridge is classified as 'essential' and a design for a 475-yearr return period earthquake ground motion is adequate according to the AASHTO LRFD criteria.

Based on the subsurface conditions reported in the boring logs, we recommend that the site be conservatively classified as AASHTO LRFD Soil Profile Type II with a corresponding site factor of 1.2. While the density of the sand encountered in the borings is generally very dense and could be classified as Profile Type I, the depth to competent rock is approximately 600 feet, according to bedrock maps developed by Shannon & Wilson, Inc. This depth to bedrock is greater than the 200-foot depth specified in the AASHTO LRFD criteria for Soil Profile Type I. Therefore, we recommend that the site be classified as Soil Profile Type II.

7.1.2 Earthquake Hazards

Earthquake induced geologic hazards that may affect a given site include landsliding, fault rupture, settlement, and liquefaction and associated effects (loss of shear strength, bearing capacity failures, loss of lateral support, ground oscillation, lateral spreading, etc.).

EXHIBIT 8
PAGE 11 OF 64

The potential for deep-seated landsliding is negligible due to the lack of significant slopes in the vicinity of the bridge and the relatively competent nature of the soil beneath the bents. Near-surface slumps, slides, and setdowns may occur.

While the site is located about 1 mile from the SWIFZ, the actual risk posed by ground rupture is relatively small. The SWIFZ has been described as a 4- to 7-mile-wide, northwest-trending, northeast-dipping zone with inferred reverse, thrust, and strike-slip (dextral) displacement on different splays within the zone, with a length of at least 50 miles (Johnson and others, 1996; Blakely and others, 2004). Based on studies of coastal marshes on Whidbey Island, Kelsey and others, (2004) postulate a magnitude 6½ to 7 earthquake occurred in this fault zone approximately 2,800 to 3,200 years ago. This and other recently completed fault trenching studies on land near Woodinville suggests that the SWIFZ produced at four events since deglaciation about 16,400 years ago (Sherrod and others, 2005). While the southern splay of the SWIFZ is about 1 mile northwest of the site at its closest approach, the potential effects from fault rupture are low due to the proximity to the site as a return period on the fault on the order of thousands of years and much longer than 475-year return period ground motions used in design.

Liquefaction and related effects appear to pose a low earthquake-induced geologic hazard. Based on the subsurface conditions encountered in boring B-1 and B-2, potentially liquefiable scattered soil zones are present in boring B-1. Potential effects on the foundations from scattered liquefaction zones are not expected to be significant, in our opinion.

7.2 Deep Foundations for Bridge Support

Based on subsurface conditions in recent borings, we recommend deep foundation support for the new bridge structure. Deep foundations such as cast-in-place concrete (CIP) piles and drilled shafts could be used. The CIP piles are typically installed by driving a closed-end steel casing (pipe pile) into the ground and filling it with concrete. In very dense or glacial soils, pile penetration may be limited when driving closed-end pipe piles. Pile penetration can be increased with these pile types if a conical tip or cutting shoe is attached to the pile tip for installation. Alternatively, an open-end pipe could be driven; however, this option would require more penetration and additional axial capacity analyses to determine the pipe pile's load carrying capability.

Drilled shafts generally have a greater axial and lateral load carrying capacity than driven piles; therefore, fewer deep foundation elements would generally be required to support proposed structure. Also, drilled shafts can be installed into the glacially overridden deposits to satisfy the required vertical and lateral resistance.

7.2.1 Axial Capacity

Axial capacity analyses were performed for proposed west and east abutment deep foundations. Deep foundation capacity will vary with foundation penetration and size, subsurface conditions, and installation techniques. A summary of deep foundation types evaluated during our axial capacity analyses is provided in the attached Table 1.

Axial capacity analyses were performed using an in-house computer program. Ultimate unit skin friction and unit end bearing values were estimated for each soil unit encountered in the borings to develop axial capacity estimates. The estimated parameters were based on subsurface conditions encountered in the borings, relative densities of the subsoils as determined by SPTs (N-values), empirical relationships that relate N-value to soil parameters, results of the laboratory tests, and our experience in similar soil and project conditions. Scour was not considered in our analyses.

Results of our axial capacity analyses are presented in figures listed in Table 1. These results are presented in plots of unfactored nominal side resistance and unfactored nominal base resistance versus depth for use in LRFD bridge design. For each drilled shaft case presented, the nominal resistances are provided for the service limit state (at settlement of 0.5 and 1.0 inch), the strength limit state, and the extreme event limit state. For each driven pile case presented, only strength and extreme event limit states are provided because the axial capacity for the service limit state will depend on the settlement of the pile group, which is a function of pile group configuration and pile penetrations. The pile group configurations and penetrations have not been determined yet; therefore, the service limit state was not evaluated during this design phase. Long- and short-term downdrag forces are provided for the west and east abutment cases, respectively, as noted on the capacity figures. Because of the scattered nature of the potential liquefaction in boring B-1, liquefaction and liquefaction-induced downdrag are not included in the capacity plots.

EXHIBIT 8
PAGE 13 OF 64

Recommended resistances factors to be applied to the unfactored nominal resistances are presented by limit state on each figure. Using the recommended unfactored nominal resistances and resistance factors, the required foundation penetrations to satisfy factored loads can be determined. We estimate the maximum closed-end CIP tip penetration would be to about elevation -30 feet.

We recommend that the drilled shafts be designed no closer than four shaft diameters, measured center to center, and driven piles no closer than 2.5 pile diameters, measured center to center. At these spacings, a group reduction factor is not warranted when estimating the group axial capacity of the foundation, assuming that the foundation elements bear in the glacially consolidated soils.

7.2.2 Embankment Fill Effects on Existing and Proposed Foundations

In our opinion, the proposed embankment fill at the roadway level will affect the proposed abutments. We assumed a final configuration with a 2 Horizontal to 1 Vertical (2H:1V) fill slope, and 8- to 12-foot-high, and 10- to 50-foot-wide fill areas depending on the chosen alternative.

Boring B-1, near the west abutment, encountered organic rich soil between depths of about 17 and 20 feet. Boring B-2 (OW), near the east abutment, encountered fill soil with abundant organic fragments to depths of about 20 feet. In general, the organic material ranged from root fragments to organic-rich seams to organic silt to scattered to numerous wood fragments. We anticipate the majority of the settlement would occur within one to three months of fill placement. However, secondary compression in organic silt encountered in boring B-1 could continue for years, but at a decreasing rate. This ongoing settlement could cause downdrag loads on west abutment deep foundations and roadway settlement beyond the bridge abutment. Based on boring B-2 (OW), the organics are in a relatively pervious granular soil layer; therefore, we anticipate that settlement downdrag would be a temporary loading condition at the east abutment. If the full dead and live loads do not act on the east abutment until after the settlement occurs, the downdrag loads should not significantly reduce the pile/shaft capacity. The capacity plot notes include the downdrag effects of fill-induced settlement on the proposed foundations.

EXHIBIT 8
PAGE 14 OF 64

The proposed fill embankments will cause soil settlement that could, in turn, create downdrag on the existing timber piles. The timber pile lengths, diameters, and design capacities are unknown. Given the age of the bridge, it is likely that the timber piles were driven only a few feet into the glacially-overridden soil. We estimate embankment-induced downdrag on the timber piles would be roughly 10 tons.

7.2.3 Lateral Resistance

Lateral loads acting on the structure from earthquake and wind, as well as other loadings, may be resisted by the passive earth pressure against foundation caps and the lateral resistance provided by the deep foundations. The magnitude of lateral resistance developed by a deep foundation depends on the subsurface conditions encountered, the type of deep foundation, spacing, and the moment capacity at the pile cap connection. We recommend ignoring the frictional sliding resistance at the base of the pile foundation cap, because a deep foundation-supported structure may not transmit load directly to the soil beneath the cap.

The computer program LPILE ^{PLUS} (Reese & Wang, 2006) and other similar methods could be used to generate load-deflection (P-Y) curves for the lateral resistance analysis of deep foundations and to calculate the magnitude of deflection, shear, and moment along the pile/shaft. Table 2 presents our LPILE parameter recommendations for both proposed abutments.

We recommend that passive earth pressures developed from compacted granular fill against the foundation caps be estimated using an equivalent fluid weight of 320 pounds per cubic foot (pcf) above groundwater. This value is based on the following assumptions:

- The backfill around the pile cap structure is compacted in accordance with the recommendations for structural fill outlined later in this report.
- The backfill is level for a distance of at least 5 feet from the pile cap.
- The pile cap is embedded at least 2 feet.
- A reduction factor of 1.5 has been applied to the passive earth pressure value above to limit deflections.

7.3 Embankments

The proposed fill embankment construction would include:

EXHIBIT 8
PAGE 15 OF 64

- Clearing and grubbing.
- Removing existing structures and obstructions as necessary.
- Preparing the subgrade.
- Placing and compacting fill.
- Paving roadway.
- Completing other miscellaneous construction details.

Keying the new embankment fill into the existing slopes will be critical to the long-term success of the embankment. Site preparation and earthwork recommendations are provided later in this report.

7.3.1 Estimated Settlement

We estimate that the initial fill-induced ground surface settlements may range from about 2 to 5 inches under the proposed full-height fill embankments. This settlement estimate includes both the settlement of the soil below the embankment and the embankment itself. More settlement could occur if soft soil layers exist beyond the edge of the existing roadway. The subsurface conditions encountered varied between the borings. Therefore, the fill-induced settlement would likely vary along the alignment. We estimate up to about ½ inch of settlement could occur out to a distance of about 5 feet from the proposed embankment toe. We estimate that most of embankment settlement should occur within one to three months of fill placement.

Secondary compression of organic-rich soil beneath the western approach embankment could continue for years. Secondary compression over a 20-year period could result in approximately 1 to 1½ inches of settlement. The secondary compression settlement would decrease over time. If there are organic-rich soils beneath the eastern approach embankment similar to the western side, the secondary compression estimate above would apply.

The embankment settlement would affect the proposed pile and shaft capacities and the existing timber pile capacities as discussed in axial capacity report section. It would also affect the roadway areas beyond the bridge abutments.

EXHIBIT 8
PAGE 16 OF 64

7.3.2 Permanent and Temporary Slopes

We recommend permanent embankment fill slopes be constructed no steeper than 2H:1V. During construction we recommend that the new embankment fill be keyed into the existing slopes.

Temporary cut slopes may be required prior to fill placement to remove organic soil and to key new fills into the slope. Suitable temporary cut slopes would depend on:

- Presence of groundwater.
- Type and density of soils.
- Depth of excavation.
- Surcharge loading adjacent to the excavation such as that from excavated material or construction equipment.
- Time of construction.

For planning purposes, we recommend assuming that temporary cut slopes less than 10 feet high be excavated at no steeper than 1.5H:1V in medium dense, near-surface materials. More specifics for embankment key or step slopes are provided in the construction considerations sections. For temporary cut slopes higher than 10 feet, a stability analysis should be completed to account for soil characteristics, adjacent loads, and groundwater conditions.

If slopes will be open for less than 24 hours, steeper slopes may be possible depending on the soil and water conditions at the site. Consistent with conventional construction practice, temporary excavation slopes should be made the responsibility of the contractor. The contractor is continually at the site and is able to observe the nature and conditions of the subsurface materials encountered, including groundwater, and has responsibility for the methods, sequence, and schedule of construction. If instability is detected, slopes should be flattened or shored. Regardless of the construction method used, all excavation work should be accomplished in compliance with applicable local, state, and federal safety codes.

7.3.3 Settlement Monitoring

We recommend that an instrumentation program be implemented for the embankment construction to monitor slope stability, total settlement, and settlement rates. We recommend an

EXHIBIT 8
PAGE 17 OF 64

instrumentation program that includes settlement plates and reference points. Settlement plates should be installed to monitor vertical ground surface settlement within and adjacent to the proposed embankment footprint. In addition, reference points should be installed to monitor vertical and lateral movements of the embankment. Optical survey methods could be used to monitor vertical and lateral movement, settlement plates, and reference points.

7.4 Loads on Buried Pipelines

Recommended loading on buried utilities from overburden soils (Case [a] and Case [b]) and H-20 live traffic loads (Case [c]) are presented in Figure 10. In our opinion, overburden pressures from both Case (a) and Case (b) for embankment and trench backfill dead loads, respectively, on a conduit may apply. The H-20 live traffic loads shown in Case (c) should be added to the overburden loads to obtain the total load on the project utilities. We recommend using a unit weight of backfill of 125 pcf.

7.5 Retaining Walls

We understand that retaining walls are planned for the project; the locations and heights of these walls are not known at this time, but we assume that they will retain fill. Our recommendations are preliminary and general, and no wall-specific subsurface information was obtained during the TS&L phase.

For fill walls, we preliminarily recommend mechanically stabilized earth (MSE) walls. The use of permanent fill slopes will depend on if there is adequate room within the proposed right-of-way.

The MSE walls would likely require permanent facings. Facing options could include precast concrete fascia panels, cast-in-place concrete, shotcrete, decorative blocks, or other options. Facings can be constructed with patterns, logos, and/or colors to improve the appearance of the wall. The choice of a particular facing is generally a matter of aesthetics.

The following sections provide a brief description of the MSE walls.

EXHIBIT 8
PAGE 18 OF 24

7.5.1 Mechanically Stabilized Earth (MSE) Walls

MSE walls are one of the most cost-effective alternatives for fill retaining walls. It has been our experience that MSE walls generally result in savings in comparison with conventional reinforced concrete retaining walls. The cost of an MSE wall is highly dependent on the facing selection, which can vary from one-fourth to one-third of the total wall cost.

There are a number of different types of MSE walls that could be constructed along the SR 202 alignment. WSDOT has a list of pre-approved proprietary wall systems that is provided in Table 15-D-1 in Appendix 15-D of the WSDOT GDM, M-46-03 (WSDOT, 2005). Some examples from that list are Hilfiker Retaining Walls, Keystone Blocks with earth reinforcing geogrids, and geogrid-reinforced earth with geotextile wrapping at the face. The Hilfiker wall uses metallic welded wire mesh to reinforce the soil and retain the backfill. 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, such as Keystone Blocks. The geosynthetic-to-facing connection is critical to the success of the retaining wall. The base width of MSE walls is typically on the order of 70 percent of the wall height (0.7H).

7.5.2 Facings

Settlement from the fill placement and wall construction is a major consideration in determining the facing type. It has been our experience that Keystone blocks generally withstand up to 2 to 3 inches of differential settlement over a distance of about 200 feet without showing distress. For walls with differential settlements greater than 2 to 3 inches, we recommend using a flexible wall system where the facing is applied after the settlement is substantially completed. Where aesthetics are not a concern and settlement is tolerable, welded wire basket facing would be a viable option.

7.5.3 Reinforced and Retained Backfill Materials

Backfill within the zone of reinforced soil is generally specified to be a granular fill such as gravel borrow per Section 9-03.14 (1), Gravel Borrow, and 9-03.14 (4), Gravel Borrow for Geosynthetic Retaining Wall, of the 2006 WSDOT Standard Specifications. For Hilfiker walls, the backfill in the reinforced zone can be angular gravel and quarry spall material. We

EXHIBIT 8
PAGE 19 OF 64

recommend a maximum particle size of 6 inches. If material smaller than the Hilfiker welded wire mat opening is used, we recommend a hardware cloth (typically a ¼ inch galvanized wire mesh screen) be placed within the facing of the Hilfiker wall to retain the backfill.

7.5.4 Drainage Provisions

The proposed MSE walls would be constructed as fill walls above the surrounding ground surface, the top of the fill would likely be sealed with pavement, and the reinforced and retained fills would be constructed with relatively clean material. However, in sloping ground conditions or in wet weather, groundwater or surface water may be able to periodically build up behind the walls. Therefore, we recommend that a drainage layer be located on the back side of the reinforced soil zone and included in the design and construction of all MSE walls on the project. Drainage material should be in accordance with Section 9-03.12(4) Gravel Backfill for Drains or 9-03-12(2) Gravel Backfill for Walls of the 2006 WSDOT Standard Specifications.

8.0 CONSTRUCTION CONSIDERATIONS

8.1 Site Preparation

We recommend that all topsoils and fill containing significant amounts of roots and organics be excavated from the project site during initial grading and earthwork operations. If necessary, excavation should extend to whatever depth is needed to expose non-organic, granular soils. Areas that will require excavation include the embankment widening. The soil that is excavated could be stockpiled for later use as fill in landscaped areas, but should not be used as structural fill beneath walls and pavements, or as wall backfill.

The excavated area should then be proof-rolled with several passes of a heavy (10-ton or heavier static weight) vibratory roller to compact the surface to a dense, unyielding condition and to a soil density in the upper 12 inches of exposed granular soil of at least 95 percent of the Modified Proctor maximum dry density (ASTM D 1557, Method C or D). If loose and/or wet, spongy soil zones are identified by the proof-rolling process, the soils should be removed and replaced with compacted structural fill, or dried or moistened as required (including scarifying, mixing, and/or aerating), be reworked, and be adequately compacted to the density previously indicated. Where necessary, the excavated soil should be replaced with structural fill to bring the site and

EXHIBIT 8
PAGE 20 OF 64

pavement areas to the desired grades. Structural fill requirements are described later in this report.

In the unlikely event that the fill embankment overexcavation levels are near or lower than the groundwater table, the excavation can be performed in the wet. We recommend placing a layer of structural fill or crushed rock before compacting to raise the grade above the groundwater. Geotextile such as Mirafi 600X or equivalent could be placed over the structural fill or crushed rock to increase basal stability. If the existing roadway is stripped and repaved with new pavement, we recommend proof-rolling the existing pavement subgrade.

8.2 Structural Fill Placement and Compaction

Upon completion of the excavating and subgrade preparation activities, the project site should be filled to desired grades with compacted structural fill. In addition, all fill and backfill placed for walls and beneath pavements, sidewalks, and other areas where settlements are to be minimized, should consist of structural fill.

Structural fill for the above purposes should consist of well-graded sand and gravel consistent with the WSDOT specification for Gravel Borrow, WSDOT Section 9-03.14(1) (WSDOT, 2006) or an approved substitution, but should not have more than 5 percent fines passing the No. 200 Sieve (wet sieve analysis, ASTM D 1140) during wet weather or in wet conditions, and the maximum particle size should be limited to 4 inches. Also, the fines should be nonplastic and the soil should be free of organic matter.

All structural fill should be placed in layers and systematically compacted to a dense, unyielding condition. In general, the thickness of soil layers before compaction should not exceed 8 inches for heavy equipment compactors or 4 inches for hand-operated mechanical compactors. The following densities are recommended in Table 3.

We recommend that all fill placement and compaction be monitored and tested by a qualified geotechnical engineer.

8.3 Suitability of Excavated Material for Use as Fill

Based on the samples retrieved from the subsurface explorations, the near surface, on-site soils appear to be slightly silty to silty; therefore, they may not be suitable for use as structural fill. In

EXHIBIT 8
PAGE 21 OF 64

general, most of the near surface on-site soils likely have fines content (percentage by weight of minus ¾-inch soil fraction passing the No. 200 sieve during wet-sieving) greater than 5 percent and also contain organics. These soils would not be suitable for compaction during wet weather or wet conditions. On-site soils may be reused as landscape fill material provided they are placed during dry weather.

8.4 Excavation and Temporary Groundwater Control

Throughout any excavated areas, the on-site soil contains enough fine-grained material to make it moisture sensitive; therefore, control of surface water and groundwater will be necessary during construction to maintain the integrity of the material and a firm working platform. The contractor should be responsible for groundwater and surface water control within the contract limits. In this regard, sloping, slope protection, ditching, sumps, dewatering, and other measures should direct water away from the roadway to prevent ponding of water.

8.5 Wet Weather Earthwork

In the project area, wet weather generally begins about mid-October and continues through about May, although rainy periods could occur at any time of year. Therefore, it would be advisable to schedule earthwork during the dry weather months of June through September. Most of the soil at the site likely contains sufficient silt to produce an unstable mixture when wet. Such soil is highly susceptible to changes in water content and tends to become unstable and difficult or impossible to proof-roll and compact if the moisture content significantly exceeds the optimum. Performing earthwork during dry weather would reduce these problems and costs associated with rainwater, erosion control, and handling of wet soil. However, should wet weather/wet condition earthwork be unavoidable, the following recommendations are provided:

- The ground surface in and surrounding the construction area should be sloped as much as possible to promote precipitation runoff away from work areas and to prevent water from ponding.
- Work areas or slopes should be covered with plastic. The use of sloping, ditching, sumps, dewatering, and other measures should be employed as necessary to permit proper completion of the work.
- Earthwork should be accomplished in small sections to minimize exposure to wet conditions. That is, each section should be small enough so that the removal of unsuitable soils and placement and compaction of clean structural fill could be

EXHIBIT 8
PAGE 22 OF 64

accomplished on the same day. The size of construction equipment may have to be limited to prevent soil disturbance. It may be necessary to excavate soils with a backhoe, or equivalent, and locate them so that equipment does not pass over the excavated area. Thus, subgrade disturbance caused by equipment traffic would be minimized.

- Fill material should consist of clean, well-graded, pit-run sand and gravel soils, of which not more than 5 percent fines by dry weight passes the No. 200 mesh sieve, based on wet-sieving the fraction passing the ¾-inch mesh sieve.
- No soil should be left uncompacted and exposed to moisture. A smooth-drum vibratory roller, or equivalent, should roll the surface to seal out as much water as possible.
- In-place soil or fill soil that becomes wet and unstable and/or too wet to suitably compact should be removed and replaced with clean, granular structural fill.
- Excavation and placement of structural fill material should be observed on a full-time basis by a geotechnical engineer (or representative) experienced in wet weather/wet condition earthwork to determine that all work is being accomplished in accordance with the project specifications and our recommendations.
- Grading and earthwork should not be accomplished during periods of heavy, continuous rainfall.

We recommend that the above requirements for wet weather/wet condition earthwork be incorporated into the contract specifications.

8.6 Driven Pile Installation

8.6.1 Pile-driving Equipment

A diesel-powered or hydraulic hammer could be used for driving the proposed steel pipe piles. All pile-driving equipment should be designed, constructed, and maintained in a manner suitable for the work to be accomplished for this project. If, in the opinion of the owner, the driving equipment is inadequate or deficient, the owner could direct that it be removed from the job site. All costs for remobilizing, removing, or replacing such equipment should be at the Contractor's expense. The Contractor should furnish the manufacturer's specifications and catalog for the hammer proposed.

8.6.2 Pile-driving Conditions

In our opinion, pile installation for the proposed structure would encounter relatively easy to moderate driving conditions in the medium dense and very stiff deposits, and hard driving

EXHIBIT 8
PAGE 23 OF 64

conditions in the glacially-overridden bearing layer. We recommend a minimum pipe wall thickness of ½ inch to achieve adequate penetration in glacial soils. If soft layers are encountered, they could cause misfire of diesel hammers because of the minimal driving resistance. We recommend that piles be driven with a hammer that allows variable energy settings.

8.6.3 Wave Equation Analysis

To establish driving criteria for pile installation, we recommend that a Wave Equation Analyses for Pile driving (WEAP) be performed. This method allows evaluation of driving stresses so that an appropriate pile-driving hammer size can be selected to obtain the desired pile capacity without damaging the piles. This analysis also provides an estimate of the ultimate pile capacity for a given driving resistance. All piles should be driven to ultimate loads as determined by WEAP.

8.6.4 Test Pile Program

The preliminary recommendations for pile foundations and, in particular, the recommendations for pile penetrations and capacities are based on theoretical and empirical data, subsurface conditions encountered at the site in limited borings, and our engineering judgment and experience. To substantiate our recommendations, we recommend that a test pile program be undertaken. The test pile program could consist of driving indicator piles and performing dynamic pile tests using a Pile-Driving Analyzer (PDA). We recommend a minimum of one pile per pier or abutment be driven as an indicator pile. During indicator pile driving, we recommend that dynamic measurements using a PDA be taken and a Case Pile Wave Analysis Program (CAPWAP) be performed on each test pile. Based on our experience, dynamic pile tests are one of the most cost-effective methods for determining the total ultimate pile capacities and load distributions. Test piles may be used as production piles if they meet the specified installation procedures and requirements.

In addition to the test pile program described above, we also recommend that dynamic measurements be performed on production piles with questionable driving results. Such measurements would help evaluate the integrity of the driven piles and the adequacy of the pile-driving criteria, confirm the pile capacities, adjust the pile-driving equipment if required, and alter pile installation techniques or pile-driving criteria if necessary.

EXHIBIT 8
PAGE 24 OF 64

8.6.5 Monitoring Pile Driving

Shannon & Wilson personnel should observe and evaluate all pile driving by making a continuous driving record of each pile. For this purpose, the Contractor should be required to mark the pile in 1-foot increments. During restrike, additional 1-inch increments between the 1-foot marks would be required.

The pile-driving record would include hammer stroke (diesel hammers), blows per foot, time, date, reasons for delays, and other pertinent information. In addition, the record would include tip elevation, specified criteria, and the initials of inspectors making final acceptance of the pile. The pile-driving records should be reviewed on a daily basis by Shannon & Wilson.

It is often difficult to visually estimate the energy delivered by diesel hammers. During construction, we recommend that the Saximeter, developed by Pile Dynamic, Inc., be used to record blow counts and stroke length that would correlate to driving energy. Use of the Saximeter during pile driving allows for verification that the pile is being driven with the hammer energy needed to develop the required axial capacity.

8.7 Drilled Shaft Installation

8.7.1 Overview

A statement should be included in the contract special provisions alerting the contractor to potential difficulties with groundwater, caving, and gravel when constructing the drilled shafts. Although not encountered in the two site recent explorations, cobbles and boulders are commonly found in glacial soil and could be encountered at this site. The plans should show the assumed soil conditions used for the shaft design.

We anticipate drilling could use conventional soil augering equipment or an oscillator-casing rig. In general, there are three typical methods for installing drilled shafts: (a) the dry method, (b) the casing method, and (c) the wet method. Based on the soil and groundwater conditions, we anticipate the drilled shafts would be installed using the casing method and/or the wet method. For the SR 202 site, casing may be required throughout the drilling depth to maintain the drillhole integrity. If the existing timber piles are to remain in use during drilled shaft installation, casing must be advanced ahead of the excavation to prevent sidewall caving

and pile capacity disturbance. An oscillator casing versus an impact or vibration driven casing may be more effective and involve less risk of caving, ground settlement, and vibration at the bridge. Casing would be used in excavations below the groundwater table and a tremie pipe would be used to place concrete below water. Details about each method are presented in the following section.

An experienced and qualified geotechnical engineer who is familiar with the subsurface conditions of the project area should observe installation of the drilled shafts. A description of the subsurface conditions encountered should be documented, and the excavation methods, steel reinforcing, concrete placement operations, and casing extraction procedures should be noted. At a minimum, a report should be prepared for each drilled shaft that covers the items recommended in the Drilled Shaft Inspector's Manual developed by the Deep Foundations Institute (DFI, 2004).

8.7.2 Methods of Construction

The following paragraphs describe the three typical methods of installing drilled shafts: the dry method, the casing method, and the wet method.

Dry Method. In the dry method of construction, the excavation is normally carried to its full depth without casing or slurry through dry, hard clay or dry, dense sand where groundwater is not encountered. Following cleanout and inspection of the excavation, the contractor places concrete through a drop chute to minimize segregation. We do not anticipate that the dry method will be suitable for this project because of the medium dense soil encountered near the ground surface and the sandy nature of the soil below the groundwater level.

Casing Method. The casing method is applicable where seepage or caving soil conditions are encountered and a casing can be pushed, driven, twisted, or oscillated into a relatively low permeability and/or firm stratum below the caving soil. The hole is generally drilled as in the dry method until caving, squeezing soil, or excessive seepage is encountered, then bentonite slurry is added to the hole. Drilling would then continue until a relatively low permeability layer is encountered. The top of the slurry must be maintained above the groundwater level. Casing is then placed into the shaft and pushed, driven, twisted, or oscillated into an impermeable layer to form a seal. The slurry may be bailed out with a cleanout or mud

bucket and drilling may proceed in the dry. The impermeable, firm stratum must have sufficient thickness to resist hydrostatic pressures below this zone when the shaft is dewatered. For this method to be effective, the casing must be clean and smooth.

EXHIBIT 8
PAGE 26 OF 64

If the soil profile is such that only a thin zone of caving soil exists within the shaft excavation, it may be possible to eliminate use of the slurry as discussed above. For this situation, the casing should be placed into the hole as soon as the caving material is encountered. The casing is then pushed, twisted, or oscillated through the caving zone into an impermeable soil layer below. Excavation may be continued in the dry.

Upon completion of the shaft excavation, the hole is cleaned and the reinforcing steel is installed. For the casing method of construction, the reinforcing steel is usually placed to the bottom of the hole, because it is difficult to keep a partial-length cage in position by a hoist line as the casing is withdrawn. The reinforcing steel should therefore be designed to accommodate both the structural requirements of the completed shaft and the stability requirements for its placement, concrete placement, and casing withdrawal.

After the reinforcing steel is placed, the hole should be filled with concrete. Under no circumstances should the casing be withdrawn until the concrete produces a hydrostatic pressure greater than the groundwater and/or slurry that is sealed by the casing. The casing should be pulled slowly and smoothly so that the concrete flows out of the base of the casing to displace the trapped slurry. All voids or annular spaces that may exist between the casing and the subsurface materials should be filled with concrete during this process.

Improper casing extraction could result in an unacceptable drilled shaft. Casing may tend to adhere to the subsurface soils. Attempts to knock the casing loose take time and may allow the concrete placed in the shaft to set. The concrete may then separate when the casing is pulled, resulting in voids in the shaft. Therefore, the casing should be left in place if the concrete appears to be setting up and extraction becomes difficult. When this situation occurs, frictional resistance would be altered and the load-carrying capacity of the shaft would have to be reevaluated.

The position of the steel reinforcing cages should be maintained when the casing is pulled. As the concrete column is placed in the hole with sufficient head to resist hydrostatic

EXHIBIT 8
PAGE 27 OF 64

forces from the groundwater and/or slurry, downward forces could be exerted on the steel cage. The magnitude of this force will depend on the slump of the concrete, the flow velocity, and volume of reinforcing steel. These forces should be considered in the design of the rebar cage.

The presence of "running" or "caving" formations will require close monitoring of the concrete level during casing extraction. Failure to maintain a positive head of concrete during casing extraction could result in a contaminated mix or presence of voids in the shaft.

Wet Method. The wet method of construction generally involves the use of slurry or water but no casing. 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 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 during drilling. If an impermeable soil zone is not encountered in which to form a seal, or there is a potential for bottom heave or blowout, use of slurry would be required to complete the excavation in the wet.

After the hole is completed to its full depth, the slurry must be processed to meet specifications prior to concrete placement. If there is too much sediment in suspension, material can settle to the bottom of the excavation before concrete is placed, resulting in a soft base. The allowable volume of sediment remaining at the base of the excavation prior to concrete placement would generally depend on the shaft design and the amount of settlement that can be tolerated. For designs where end bearing is high, a clean, firm bottom is required. The American Concrete Institute (ACI 336.3R-93) recommends that in no case should the volume of loose material and spoil at the base of the shaft exceed that which would be required to cover 5 percent of the base area to a maximum depth of 2 inches (ACI, 2006).

In addition to spoil at the base of the shaft, the sediment in suspension could also settle to the top of the concrete column as the pour is progressing. This material could coat the rebar and sidewalls of the shaft, reducing the bond strength. Such issues need to be addressed in the contract specifications and require careful inspection and quality control during shaft construction.

EXHIBIT 8
PAGE 28 OF 64

8.8 Pile-driving and Shaft Installation Vibrations, Movement Monitoring, and Noise Levels

We understand that there are existing utilities near the existing bridge that must be protected. There is a potential for damage to existing utilities if piles are driven or shafts are drilled nearby. We recommend developing and implementing vibration criteria for the existing utilities. The criteria should consider the type and frequency of the vibrations and the existing condition of the lines. Based on a collection of various standards for vibration monitoring, suggested criteria for possible damage resulting from vibrations would be a maximum peak particle velocity (ppv) of 1½ inches per second (ips). Particle velocities can be readily measured during construction using a vibration monitor (seismograph).

We recommend that the first driven piles be installed farthest from vibration-sensitive utilities so that the vibrations can be monitored. The monitoring program could consist of measuring the vibrations at varying distances from the driven pile, such as 5, 10, 20, 30, and 40 feet. In addition, a vibration monitor should be installed directly on the existing lines for the duration of construction activities. This measured vibration data would be used to determine how the vibration levels decrease with distance from the pile, and how they attenuate as compared to the depth of pile installation. The criteria for evaluating the impact of pile installation would then be established based on this initial vibration data. If measured vibrations are at or above a level that could potentially cause damage to the utilities (ppv >1 ips), mitigation measures should be implemented to reduce vibrations.

Various mitigation measures could be utilized. One such mitigation measure consists of predrilling using an auger to loosen soils to about 20 to 25 feet bgs. We do not recommend that the soils be removed, just loosened. The purpose of predrilling is to attempt to ease pile penetration and reduce the vibrations of pile driving; however, vibrations will still occur. We recommend that the vibration monitoring program continue to determine if measured vibrations after predrilling have been reduced below the vibration criteria. Alternatively, while predrilling, a temporary oversized casing could be installed and the soil removed from within the casing. This method would further reduce the vibrations in surrounding soils from pile-driving. The Contractor may be able to propose other mitigation methods.

EXHIBIT 8
PAGE 29 OF 64

An oscillator-type system for drilled shaft installation could be used to mitigate vibrations. The oscillator casing is slowly rotated and pushed into the ground ahead of the soil removal. This system also allows for relatively accurate shaft placement and plumbness.

We recommend that an existing condition survey be performed for critical utilities and other nearby facilities that may be affected by the pile-driving and shaft installation activities. Documentation should include photographs, videos, sketches, and/or written comments. This information will be invaluable in assessing the need for mitigating measures, as well as resolving potential disputes.

Noise levels during pile driving could be unpleasant to humans nearby but are not likely to cause damage. The noise levels could be moderated during pile driving with various damping techniques such as a crane-mounted noise barrier.

8.9 Plans and Specifications and Construction Observation

We recommend that Shannon & Wilson be retained to review the geotechnical aspects of the plans and specifications. We also recommend that we be retained to observe the geotechnical aspects of construction. This observation would allow us to verify the subsurface conditions as they are exposed or encountered during construction and to determine that the work is accomplished in accordance with our recommendations and the project documents.

9.0 RECOMMENDED FUTURE STUDIES

For plans, specifications, and cost estimate (PSE) design, we recommend that additional field explorations be conducted at potential center pier and significant retaining wall locations. Explorations would require additional permitting to drill near the river. We also recommend additional engineering analysis on the chosen alternative's fill embankment settlements, settlement impacts to proposed and existing foundations, and retaining walls.

10.0 LIMITATIONS

This draft report was prepared for the exclusive use of DMJM Harris, the City of Woodinville, and other members of the design team. It should be made available to prospective contractors and/or the contractor for information on factual data only, and not as a warranty of subsurface

EXHIBIT 8
PAGE 30 OF 64

conditions such as those interpreted from the exploration logs and presented in the discussion 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 could review these conditions and reconsider our recommendations, where necessary. If conditions have changed because of natural forces or construction operations at or near the site, we recommend that this report be reviewed to determine the applicability of the conclusions and recommendations considering the changed conditions and time lapse.

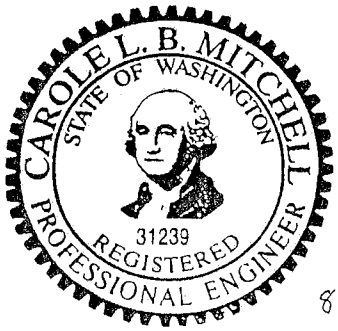
Within the limitations of the scope, schedule and budget, the analyses, conclusions, and recommendations presented in this report were prepared in accordance with generally accepted professional geotechnical engineering principles and practice in this area at the time this report was prepared. We make 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 other than those described in this report. Shannon & Wilson, Inc., has qualified personnel to assist you with these services should they be necessary.

EXHIBIT 8
PAGE 31 OF 64

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

SHANNON & WILSON, INC.



8-21-07

EXPIRES 6/15/ 09

Carole L.B. Mitchell, P.E.
Associate

CKS:WDN:CLBM:TMG/clbm

EXHIBIT 8
PAGE 32 OF 64

11.0 REFERENCES

ADSC: The International Association of Foundation Drilling and Deep Foundations Institute (DFI), 2004, Drilled shaft inspector's manual, second edition: Dallas, Tex.

American Association of State Highway and Transportation Officials (AASHTO), 2006, LRFD Bridge Design Specifications, (2nd ed.): Washington, D. C., 1 v.

American Concrete Institute, 2006, ACI 336.3R-93: Design and construction of drilled piers.

American Society for Testing and Materials (ASTM), 2007, Annual book of standards, Construction, v. 04.08, Soil and rock (I): D 420 – D 5611: West Conshohocken, Pa.

Blakely, R.J., Sherrod, B.L., Wells, R.E., Weaver, C.S., McCormack, D.H., Troost, K.G., Haugerud, R.A., 2004, The Cottage Lake aeromagnetic lineament-A possible onshore extension of the southern Whidbey Island fault, Washington: U.S. Geological Survey Open-File Report 2004-1204, 60 p.

Frankel, A.D. and others, Documentation for the 2002 Update of the National Seismic Hazard Maps: U.S. Geological Survey Open-File Report 02-420, 33p.

Johnson, S.Y., Potter, C.J., Armentrout, J.M., and others, 1996, The Southern Whidbey Island Fault, an active structure in the Puget Lowland, Washington: Geological Society of America Bulletin, v. 108, p. 334-354.

Kelsey, H. M., Sherrod, Brian, Johnson, S. Y., and others, 2004, Land-level changes form a late Holocene earthquake in the northern Puget Lowland, Washington: Geology, v. 32, no. 6, p. 469-472.

Reese, L.C., and Wang, S.T., 2006, Documentation of computer program LPILE^{PLUS}, Version 5.0: Austin, Tex., Ensoft, Inc.

Sherrod, B.L.; Blakely, R.J.; Weaver, Craig; and others, 2005, Holocene fault scarps and shallow magnetic anomalies along the southern Whidbey Island Fault Zone near Woodinville, Washington: U. S. Geological Survey Open File Report 2005-1136, 36 p., available: http://pubs.usgs.gov/of/2005/1136/.

Washington State Department of Transportation (WSDOT) 2006, Bridge design manual (M23-50): Olympia, Wash., Washington State Department of Transportation.

Washington State Department of Transportation (WSDOT) 2005, Geotechnical design manual (M46-03): Olympia, Wash., Washington State Department of Transportation.

Washington State Department of Transportation (WSDOT) and American Public Works Association, 2006, Standard specifications for road, bridge, and municipal construction (M41-10), 2006 – English units: Olympia, Wash., Washington State Department of Transportation.

EXHIBIT 8
PAGE 33 OF 64

TABLE 2

RECOMMENDED PARAMETERS FOR LATERAL RESISTANCE ANALYSIS USING LPILE^{PLUS}

Location and Boring Number	Approximate Depth Below Ground Surface at Boring Location (feet)		Soil Type	Effective Unit Weight, γ' (pcf)	Cohesion, c (psf)	Friction Angle, ϕ (degrees)	Modulus of Subgrade Reaction, k (pci)	Strain at 50% Max Stress, ε_{50}
	Upper Boundary	Lower Boundary						
West Abutment, Boring B-1	0	12	Sand	120	--	32	90	--
	12	15	Stiff Clay with Water	73	3,000	--	1,000	0.005
	15	23	Sand	53	--	30	40	--
	23	38	Stiff Clay with Water	68	2,000	--	560	0.006
	38	50	Sand	58	--	36	120	--
	50	120	Sand	73	--	40	200	--
East Abutment, Boring B-2 (OW)	0	20	Sand	120	--	32	90	--
	20	25	Sand	120	--	36	190	--
	25	33	Sand	58	--	36	110	--
	33	68	Sand	68	--	40	170	--
	68	100	Sand	73	--	40	200	--

NOTES:

1. Parameters given above are for both static and seismic loading conditions. Scattered zones of potentially liquefiable materials were encountered in boring B-1. No potentially liquefiable soil was observed in boring B-2 (OW).
2. The recommended parameters are based on the subsurface conditions and laboratory testing for borings B-1 and B-2 (OW).
3. Pile/shaft group effects are not considered.
4. If applicable, modifications to the p-y curves for sloping ground conditions should be determined in accordance with the LPILE^{PLUS} (2006) manual.
5. We assume that the top of the pile/shaft will be about 8 feet below the top-of-boring elevations.
6. No scour has been included.

pcf = pounds per cubic foot

pci = pounds per cubic inch

psf = pounds per square foot

ε_{50} = strain at one-half the maximum principal stress difference

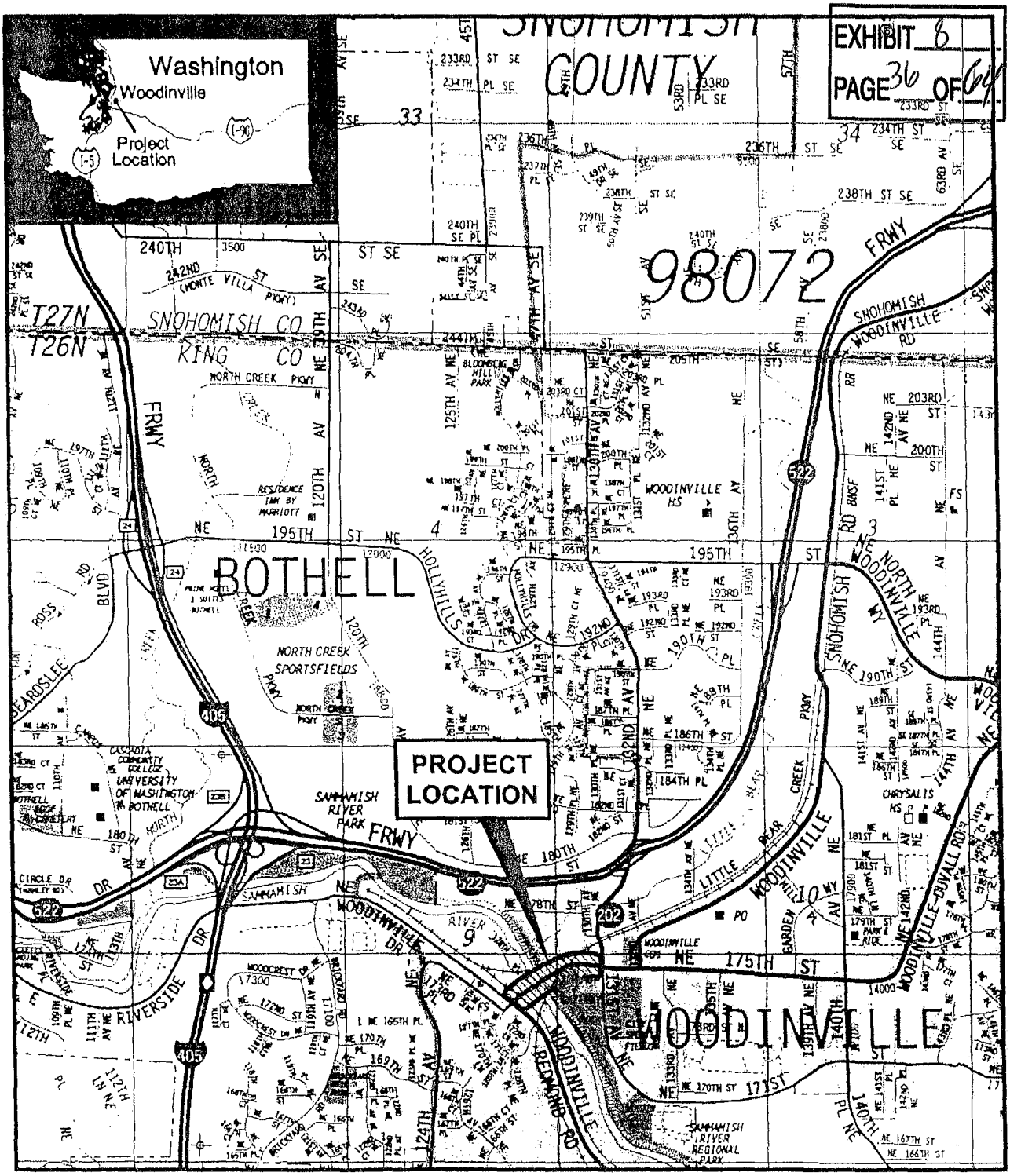
EXHIBIT 8
PAGE 34 OF 69

EXHIBIT 8
PAGE 35 OF 64

TABLE 3
RECOMMENDATIONS FOR STRUCTURAL FILL COMPACTION

Location of Fill	Recommended Minimum Percentage of Modified Proctor Maximum Dry Density*
Beneath Pavement Areas	95
Landscaped Areas/General Fill	90

Note:
*As determined by American Society for Testing and Materials (ASTM) Designation: D 1557-70, Method C or D.



0 1/4 1/2 1

Scale in Miles

NOTE

Reproduced with permission granted by THOMAS BROS. MAPS®. This map is copyrighted by Rand McNally R.L. 06-S-27. It is unlawful to copy or reproduce all or any part thereof, whether for personal use or resale, without permission. All rights reserved.

SR 202 Sammamish River Bridge
Woodinville, Washington

VICINITY MAP

August 2007 | 21-1-20664-001

SHANNON & WILSON, INC.
Geotechnical and Environmental Consultants

FIG. 1

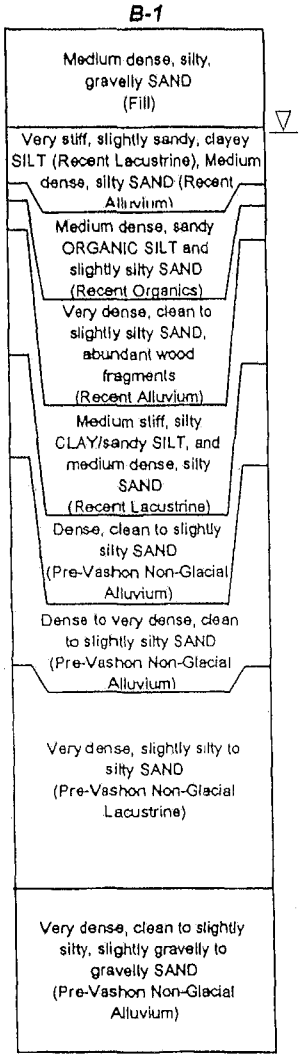
Filename: J:\21\20664-001\21-1-20664-001 Fig 1.dwg Date: 08-20-2007 Login: LR

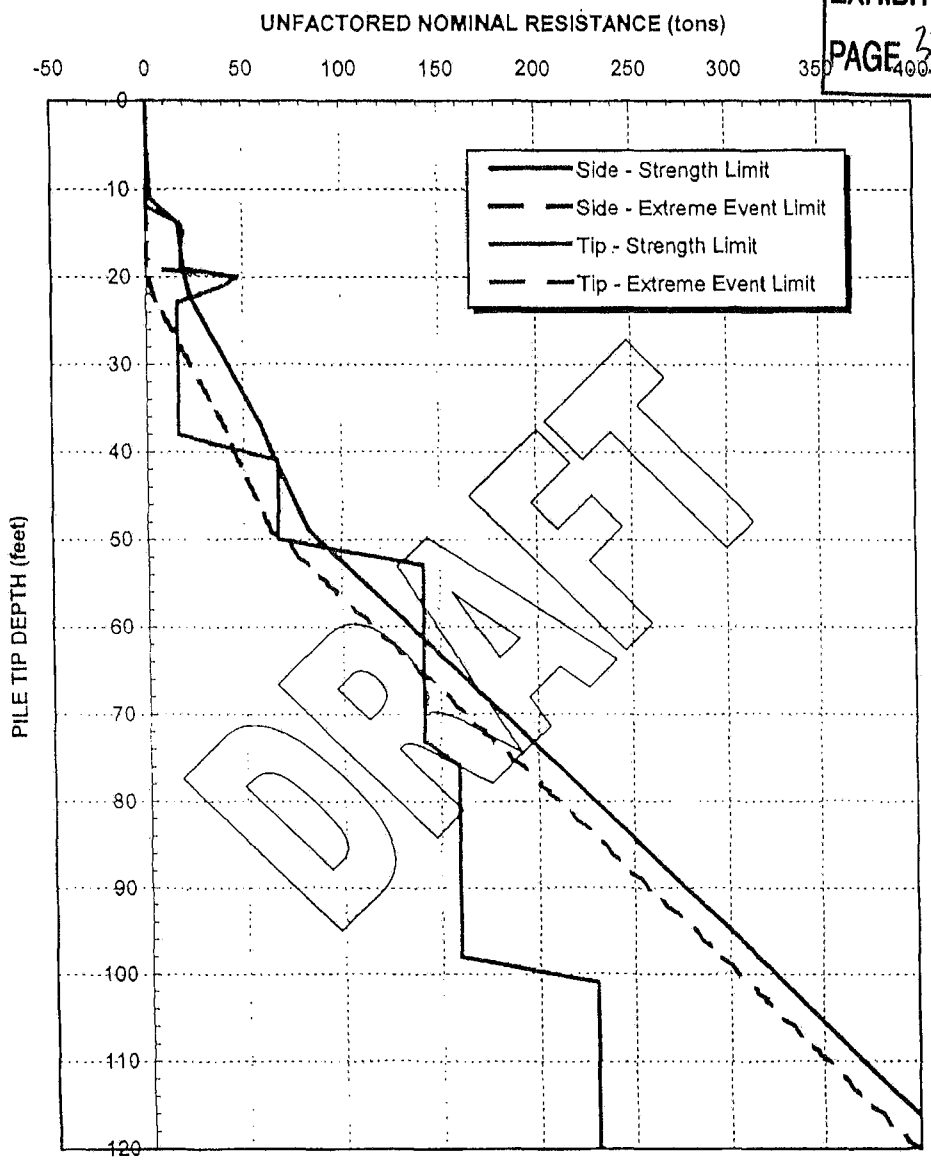
EXHIBIT 8
PAGE 37 of 64

ASSUMED SUBSURFACE PROFILE

Based on Nearby Explorations:

B-1





NOTES:

1. The analyses were performed based on recommendations outlined in the WSDOT Geotechnical Design Manual (GDM) and local experience. The analyses are for a single pile and do not consider group action of piles spaced closer than 2.5 diameters, center to center. Once final pile group sizes and spacings are determined, the axial capacity of the pile group should be re-evaluated.
2. Total pile capacity is a summation of its side and tip resistances. Unfactored nominal resistances shown on plots above are to be multiplied by the appropriate resistance factors (RFs). For strength limit state, RF = 0.45 for both side and tip resistance and RF = 0.35 for uplift resistance. For extreme limit state, RF = 1.0 for both side and tip resistance. Pile uplift capacity can be estimated by using the unfactored side resistance plus 20 tons (to negate the downdrag force) and the appropriate RF listed above.
3. Unfactored downdrag force is estimated to be 20 tons due to liquefaction. Long-term downdrag loads due to potential embankment-induced settlement of organic materials should be included. Per the WSDOT GDM, a load factor of 1.25 is recommended to determine factored downdrag force.
4. The scattered zones of potentially liquefiable soil were not included in the capacity plots. See text for discussion.
5. CIP concrete piles installed by driving a closed-end steel casing will not achieve significant penetration into very dense glacial soils. We estimate the maximum tip penetration would be to about elevation -30 feet. If additional penetration is needed for load capacity requirements, consideration should be given to driving the steel casings open ended. If driving piles open-ended is considered, the axial capacities must be re-evaluated.
6. Typically the strength limit state is used to design the pile depth. After considering the axial and lateral analyses results, a pile group configuration and pile depths are provided. Based on this input, the pile group settlement and service limit case are determined.
7. No scour included. Assumed pile cap at about elevation 37 feet.

SR 202 Sammamish River Bridge
Woodinville, Washington

ESTIMATED AXIAL CAPACITY FOR 16-INCH-DIA. CIP CONCRETE PILE WEST ABUTMENT

August 2007 | 21-1-20664-001

SHANNON & WILSON, INC.
Geotechnical and Environmental Consultants

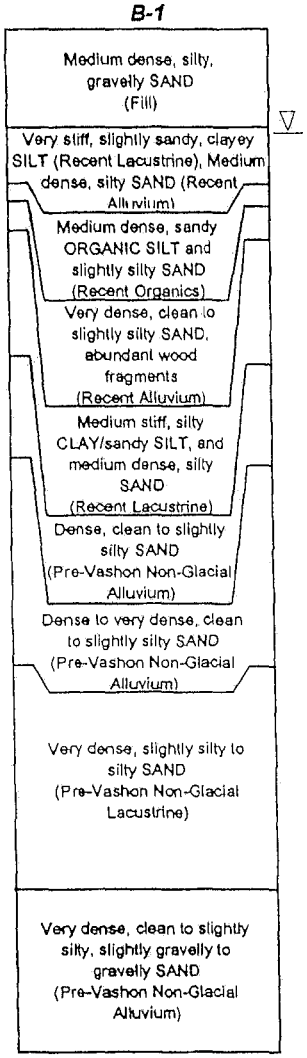
FIG 4

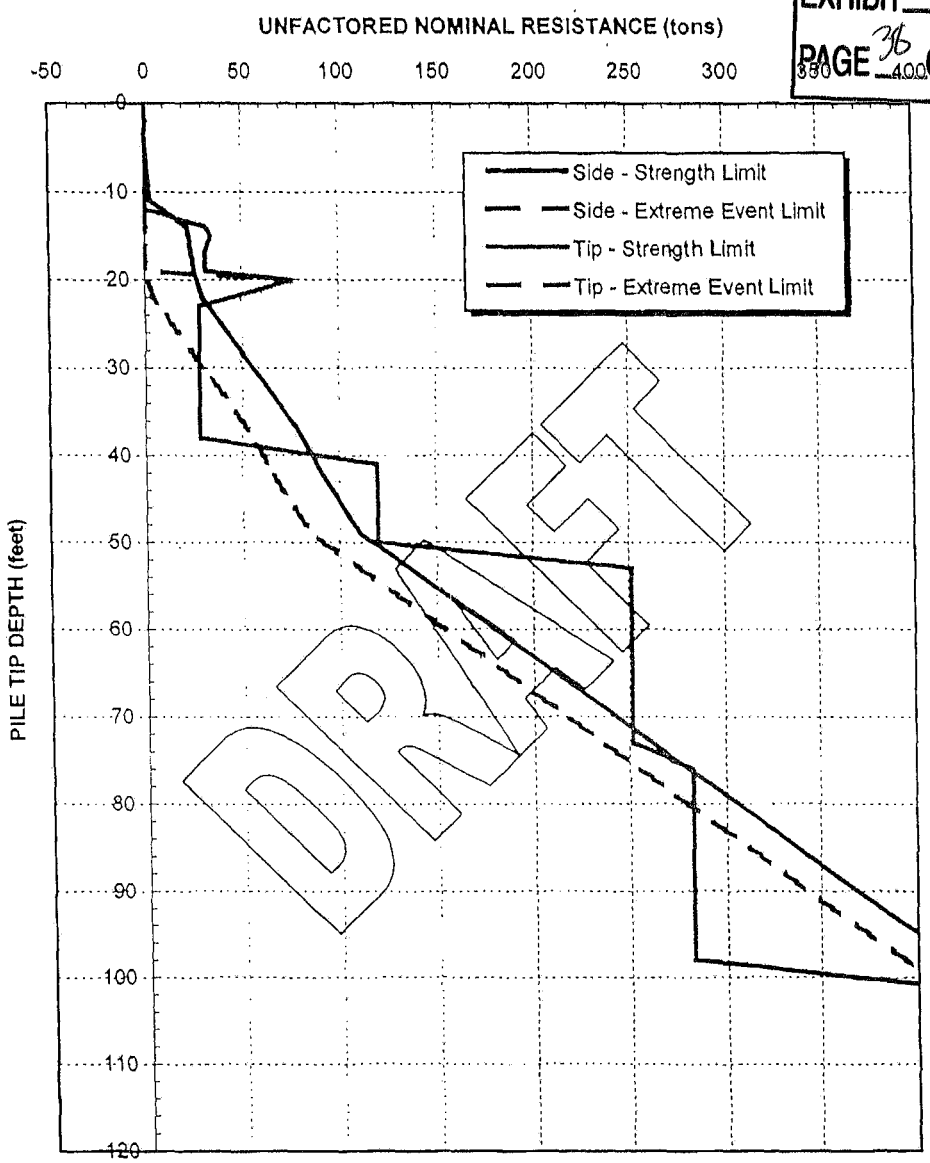
EXHIBIT 8
PAGE 36 OF 44

ASSUMED SUBSURFACE PROFILE

Based on Nearby Explorations:

B-1





NOTES:

1. The analyses were performed based on recommendations outlined in the WSDOT Geotechnical Design Manual (GDM) and local experience. The analyses are for a single pile and do not consider group action of piles spaced closer than 2.5 diameters, center to center. Once final pile group sizes and spacings are determined, the axial capacity of the pile group should be re-evaluated.
2. Total pile capacity is a summation of its side and tip resistances. Unfactored nominal resistances shown on plots above are to be multiplied by the appropriate resistance factors (RFs). For strength limit state, RF = 0.45 for both side and tip resistance and RF = 0.35 for uplift resistance. For extreme limit state, RF = 1.0 for both side and tip resistance. Pile uplift capacity can be estimated by using the unfactored side resistance plus 30 tons (to negate the downdrag force) and the appropriate RF listed above.
3. Unfactored downdrag force is estimated to be 30 tons due to liquefaction. Long-term downdrag loads due to potential embankment-induced settlement of organic materials should be included. Per the WSDOT GDM, a load factor of 1.25 is recommended to determine factored downdrag force.
4. The scattered zones of potentially liquefiable soil were not included in the capacity plots. See text for discussion.
5. CIP concrete piles installed by driving a closed-end steel casing will not achieve significant penetration into very dense glacial soils. We estimate the maximum tip penetration would be to about elevation -30 feet. If additional penetration is needed for load capacity requirements, consideration should be given to driving the steel casings open ended. If driving piles open-ended is considered, the axial capacities must be re-evaluated.
6. Typically the strength limit state is used to design the pile depth. After considering the axial and lateral analyses results, a pile group configuration and pile depths are provided. Based on this input, the pile group settlement and service limit case are determined.
7. No scour included. Assumed pile cap at about elevation 37 feet.

SR 202 Sammamish River Bridge
Woodinville, Washington

ESTIMATED AXIAL CAPACITY FOR 24-INCH-DIA. CIP CONCRETE PILE WEST ABUTMENT

August 2007 | 21-1-20664-001

SHANNON & WILSON, INC.
Geotechnical and Environmental Consultants

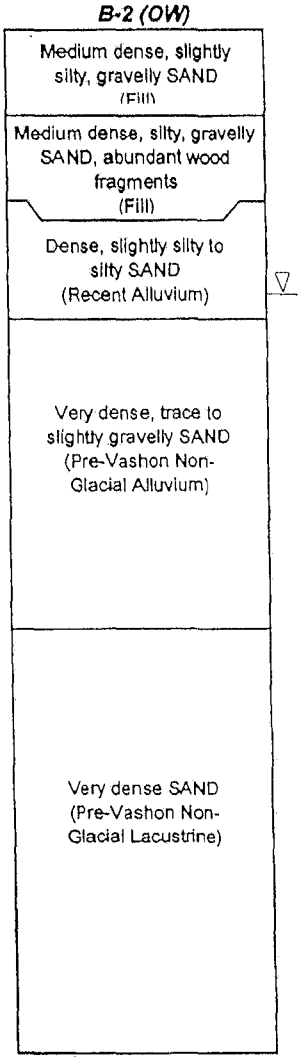
FIG 5

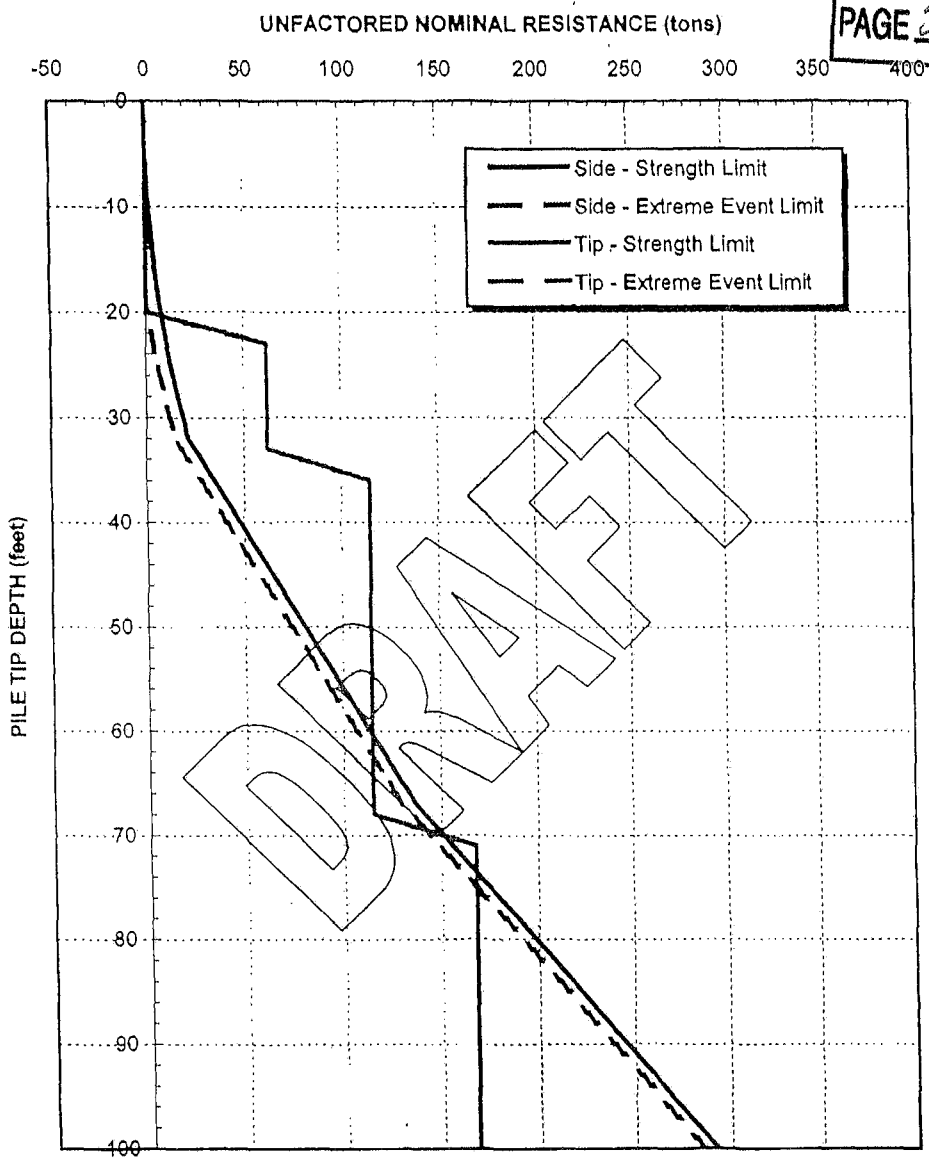
EXHIBIT 8
PAGE 39 of 64

ASSUMED SUBSURFACE PROFILE

Based on Nearby Explorations:

B-2 (OW)





NOTES:

1. The analyses were performed based on recommendations outlined in the WSDOT Geotechnical Design Manual (GDM) and local experience. The analyses are for a single pile and do not consider group action of piles spaced closer than 2.5 diameters, center to center. Once final pile group sizes and spacings are determined, the axial capacity of the pile group should be re-evaluated.
2. Total pile capacity is a summation of its side and tip resistances. Unfactored nominal resistances shown on plots above are to be multiplied by the appropriate resistance factors (RFs). For strength limit state, RF = 0.45 for both side and tip resistance and RF = 0.35 for uplift resistance. For extreme limit state, RF = 1.0 for both side and tip resistance.
3. Unfactored downdrag force is estimated to be 10 tons due to liquefaction. Short-term downdrag loads due to potential embankment-induced settlement of organic materials should be included. Per the WSDOT GDM, a load factor of 1.25 is recommended to determine factored downdrag force.
4. The scattered zones of potentially liquefiable soil were not included in the capacity plots. See text for discussion.
5. CIP concrete piles installed by driving a closed-end steel casing will not achieve significant penetration into very dense glacial soils. We estimate the maximum tip penetration would be to about elevation -30 feet. If additional penetration is needed for load capacity requirements, consideration should be given to driving the steel casings open ended. If driving piles open-ended is considered, the axial capacities must be re-evaluated.
6. Typically the strength limit state is used to design the pile depth. After considering the axial and lateral analyses results, a pile group configuration and pile depths are provided. Based on this input, the pile group settlement and service limit case are determined.
7. No scour is included. Assumed pile cap at about elevation 40 feet.

SR 202 Sammamish River Bridge
Woodinville, Washington

ESTIMATED AXIAL CAPACITY FOR 16-INCH-DIA. CIP CONCRETE PILE EAST ABUTMENT

August 2007 | 21-1-20664-001

SHANNON & WILSON, INC.
Geotechnical and Environmental Consultants

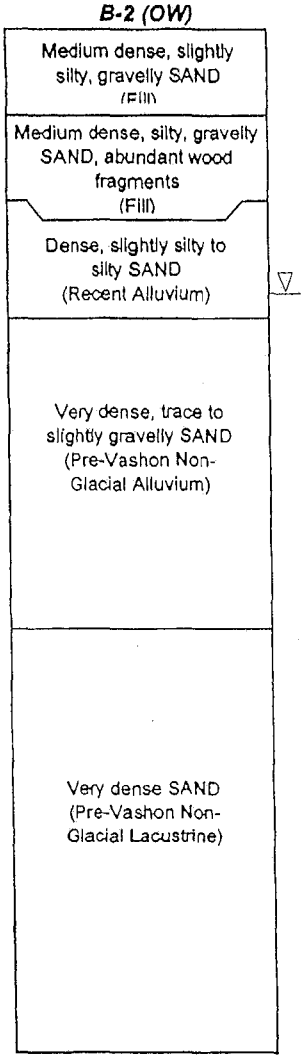
FIG 7

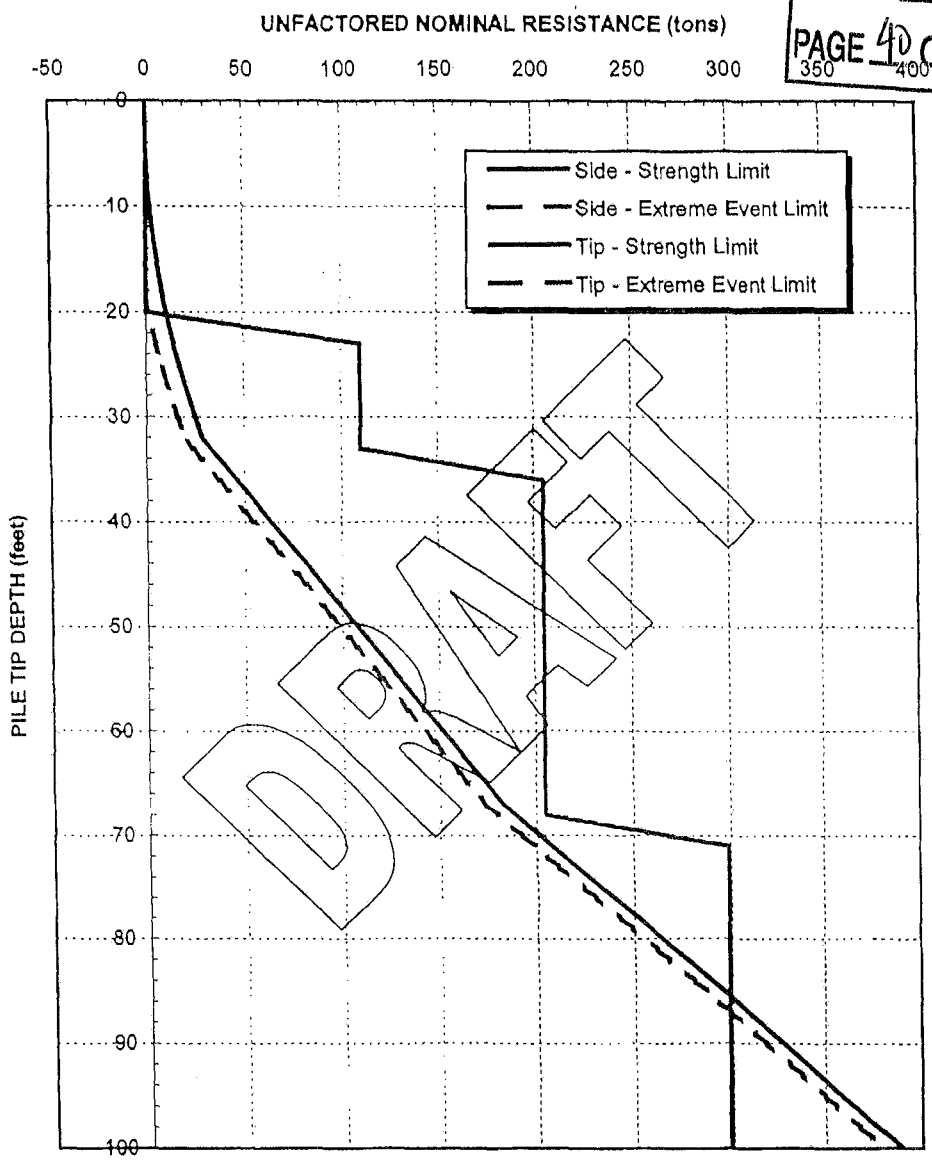
EXHIBIT 8
PAGE 40 OF 64

ASSUMED SUBSURFACE PROFILE

Based on Nearby Explorations:

B-2 (OW)





NOTES:

1. The analyses were performed based on recommendations outlined in the WSDOT Geotechnical Design Manual (GDM) and local experience. The analyses are for a single pile and do not consider group action of piles spaced closer than 2.5 diameters, center to center. Once final pile group sizes and spacings are determined, the axial capacity of the pile group should be re-evaluated.
2. Total pile capacity is a summation of its side and tip resistances. Unfactored nominal resistances shown on plots above are to be multiplied by the appropriate resistance factors (RFs). For strength limit state, RF = 0.45 for both side and tip resistance and RF = 0.35 for uplift resistance. For extreme limit state, RF = 1.0 for both side and tip resistance.
3. Unfactored downdrag force is estimated to be 10 tons due to liquefaction. Short-term downdrag loads due to potential embankment-induced settlement of organic materials should be included. Per the WSDOT GDM, a load factor of 1.25 is recommended to determine factored downdrag force.
4. The scattered zones of potentially liquefiable soil were not included in the capacity plots. See text for discussion.
5. CIP concrete piles installed by driving a closed-end steel casing will not achieve significant penetration into very dense glacial soils. We estimate the maximum tip penetration would be to about elevation -30 feet. If additional penetration is needed for load capacity requirements, consideration should be given to driving the steel casings open ended. If driving piles open-ended is considered, the axial capacities must be re-evaluated.
6. Typically the strength limit state is used to design the pile depth. After considering the axial and lateral analyses results, a pile group configuration and pile depths are provided. Based on this input, the pile group settlement and service limit case are determined.
7. No scour is included. Assumed pile cap at about elevation 40 feet.

SR 202 Sammamish River Bridge
Woodinville, Washington

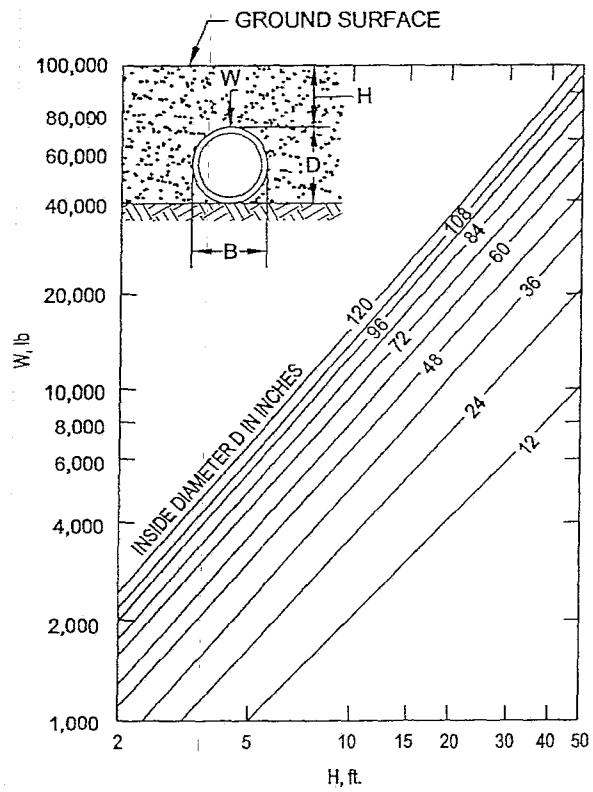
ESTIMATED AXIAL CAPACITY FOR 24-INCH-DIA. CIP CONCRETE PILE EAST ABUTMENT

August 2007 | 21-1-20664-001

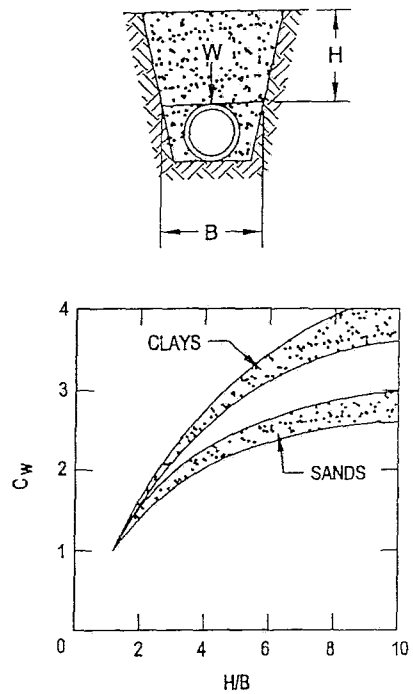
SHANNON & WILSON, INC.
Geotechnical and Environmental Consultants

FIG 8

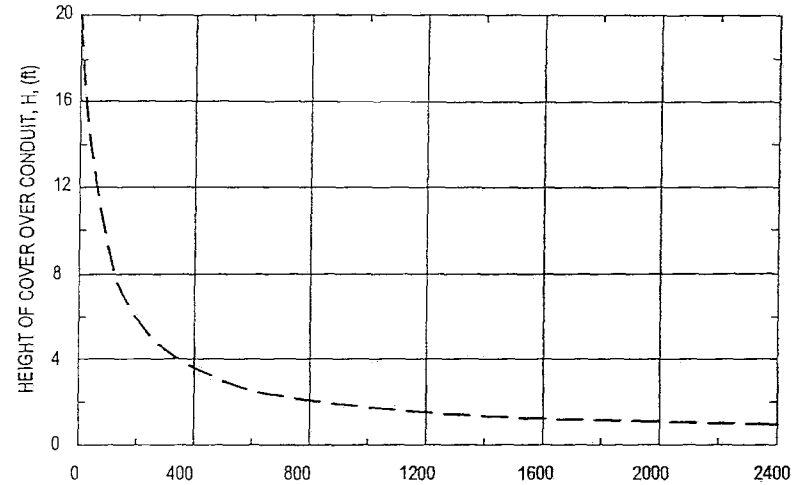
File: J:\211\20664-001\21-1-20664-001 Fig 10.dwg Date: 08-20-2007 Author: LR



(a) EMBANKMENT DEAD LOAD W ON A CONDUIT BURIED IN A SOIL EMBANKMENT.



(b) C_W FOR CONDUIT IN A TRENCH



(c) VERTICAL PRESSURE DUE TO H-20 LIVE LOAD ON CONDUIT (PSF)

NOTES

1. W = Total Dead Load per Unit Length.
2. Embankment dead loads of Case (a) are based on soil unit weight of 100 pcf. For different soil unit weights, adjust the loads proportionately.
3. For trench backfill, Case (b): $W=C_W(\gamma)(B)^2$

   where: γ = Soil Unit Weight.

   B = Trench Width at Top of Pipe Level.
4. Live loads of Case (c) include effect of impact.
5. This figure was reproduced from figures presented in the NAVFAC DM7.

EXHIBIT 8
PAGE 41 OF 62

SR 202 Sammamish River Bridge
Woodinville, Washington

LOADS ON BURIED UTILITIES

August 2007 21-1-20664-001

SHANNON & WILSON, INC.
Geotechnical and Environmental Consultants

FIG. 10

FIG. 10

APPENDIX A

SUBSURFACE EXPLORATIONS

EXHIBIT 8
PAGE 43 OF 64

APPENDIX A

SUBSURFACE EXPLORATIONS

TABLE OF CONTENTS

Page

A.1 GENERAL A-1

A.2 CURRENT BORINGS A-1
- A.2.1 Drilling A-1
- A.2.2 Soil Testing and Sampling A-2
- A.2.3 Observation Well Installation and Development A-2

A.3 FIELD SCREENING METHODOLOGY A-3

A.4 REFERENCE A-3

LIST OF FIGURES

Figure No.

A-1 Soil Classification and Log Key (2 sheets)
A-2 Log of Boring B-1 (3 pages)
A-3 Log of Boring B-2 (OW) (2 pages)
A-4 Log of Boring H-1
A-5 Log of Boring H-2
A-6 Log of Boring H-3

EXHIBIT 8
PAGE 44 OF 64

APPENDIX A

SUBSURFACE EXPLORATIONS

A.1 GENERAL

Two exploratory borings, designated B-1 and B-2 (OW), were performed at night between March 19 and 22, 2007. An observation well was installed in boring B-2; therefore, it has the observation well (OW) designation. The boreholes were drilled to 120.5 and 100.5 feet below the existing grade at borings B-1 and B-2 (OW), respectively. The borings were drilled behind the existing bridge abutments where there were no utility conflicts. The boring locations shown in Figure 2 were surveyed by Roth Hill Engineering. Logs of borings B-1 and B-2 (OW) are presented in Figures A-2 and A-3, respectively. A Soil Classification and Log Key is presented in Figure A-1 as a reference for boring log symbols and information. Three boreholes, designated H-1, H-2, and H-3, were drilled in 1963 by the Washington State Department of Transportation (WSDOT) for the existing bridge design. The 1963 boring logs are presented in Figures A-4 through A-6.

A Shannon & Wilson, Inc. representative was present throughout the field exploration period to observe the drilling and sampling operations, to retrieve representative soil samples for subsequent laboratory testing, and to prepare descriptive field exploration logs. The samples were placed in airtight jars and returned to our laboratory for testing.

A.2 CURRENT BORINGS

The borings were drilled by Boart-Longyear Drilling of Fife, Washington, under subcontract to Shannon & Wilson, Inc.

A.2.1 Drilling

The borings were advanced with a truck-mounted drill rig using mud-rotary drilling methods. Boring B-1 was drilled with a 4⅞-inch-diameter tri-cone bit. Boring B-2 (OW) was drilled with a 6¼-inch-diameter tri-cone bit to allow for the observation well installation. Mud-rotary borings are advanced by circulating thick drilling mud from the rig down through standard 2⅝-inch outside-diameter NX rods to a tri-cone bit at the bottom of the borehole. The drilling mud is a mixture of bentonite powder and water. Cuttings are transported from the

EXHIBIT 8
PAGE 45 OF 64

bottom of the borehole to the surface by drilling mud flowing between the drilling rods and the sides of the borehole. The cuttings are deposited in a settling tank at the ground surface and the mud is recirculated. Boring B-1 was backfilled with bentonite chips after completion of drilling and sampling. Below the base of the observation well screen, boring B-2 (OW) was also backfilled with bentonite chips.

A.2.2 Soil Testing and Sampling

Disturbed samples were obtained in conjunction with the Standard Penetration Test (SPT). SPTs were performed in general accordance with American Society for Testing and Materials (ASTM) Designation: D 1586, Standard Method for Penetration Testing and Split-Barrel Sampling of Soils. SPTs were generally performed every 2.5 feet from the ground surface to a depth of 20 feet below grade and then every 5 feet below 20 feet. The SPT consists of driving a 2-inch outside-diameter, split-spoon sampler a distance of 18 inches into the bottom of the borehole with a 140-pound hammer falling 30 inches. The number of blows required for the last 12 inches of penetration is termed the Standard Penetration Resistance (N-value). These values are plotted at the appropriate boring log depths. Generally, whenever 50 or more blows were required to cause 6 inches or less of penetration, the test was terminated, and the number of blows and the corresponding penetration were recorded. The N-values were plotted on the boring logs presented in Figures A-2 and A-3. This value is an empirical parameter that provides a means for evaluating the relative density, or compactness, of granular soils and the consistency, or stiffness, of cohesive soils.

Soils were classified in general accordance with ASTM Designation: D 2488, Standard Recommended Practice for Description of Soils (Visual-Manual Procedure). The exploration logs presented in Figures A-2 and A-3 represent our interpretation of the contents of the field logs and the results of geotechnical laboratory testing. Figure A-1 presents a key to our classification of the materials encountered.

A.2.3 Observation Well Installation and Development

An observation well was installed in boring B-2 (OW) after the boring was advanced to its total depth and partially backfilled to the desired well bottom depth. The well was constructed using a new, commercially fabricated, threaded, flush-jointed, 2-inch-diameter, Schedule 40 polyvinyl chloride (PVC) screen and riser. The well screen consisted of

EXHIBIT 8
PAGE 46 OF 64

0.02-inch-wide, machine-slotted PVC, with approximately a 9.5-foot screened interval. The top of the well was completed with an expandable locking cap, and a threaded sump was installed at the bottom of the well. Following installation of the casing and screen, a silica sand filter pack was poured into the annular space between the boring wall and the well screen to about 3 feet above the top of the screen. The remaining annulus was filled with bentonite chips to within about 2 feet of the ground surface. The observation well was completed flush to grade by concreting an 8-inch-diameter, flush-mounted, steel monument over the top of the borehole. The screened interval depth is shown in the boring log.

Well development was performed immediately after well installation to improve the hydraulic connection between the aquifer and the screened portion of the observation well. The saturated screened section of the well was developed by bailing to remove water and sediment so that subsequent water level measurements would be representative of the aquifer groundwater conditions. Development equipment consisted of a high-density polyethylene (HDPE) bailer. The bailer was lowered and raised by hand until there was no further observed improvement in water clarity.

A.3 FIELD SCREENING METHODOLOGY

Field screening methods for borings B-1 and B-2 (OW) included visual observations and olfactory observations. These methods were used for worker safety. Visual observations of soil samples and groundwater would include items such as sheen, or gray or black discoloration. Olfactory observations would be recorded when noted. Soil was not intentionally smelled for contamination. No field indication of contamination was observed. No environmental soil samples were collected.

A.4 REFERENCE

American Society for Testing and Materials (ASTM), 2007, Annual book of standards, construction, v. 4.08, soil and rock (I): D 420 – D 5611: West Conshohocken, Pa.

EXHIBIT 8
PAGE 47 of 64

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	

SR 202 Sammamish River Bridge
Woodinville, Washington

SOIL CLASSIFICATION AND LOG KEY

August 2007 | 21-1-20664-001

SHANNON & WILSON, INC.
Geotechnical and Environmental Consultants

FIG. A-1
Sheet 1 of 2

BORING CLASS1 21-20664.GPJ SWNEW.GDT 8/20/07

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)

NOTE: No. 4 size = 5 mm; No. 200 size = 0.075 mm

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.

SR 202 Sammamish River Bridge
Woodinville, Washington

SOIL CLASSIFICATION AND LOG KEY

August 2007 | 21-1-20664-001

SHANNON & WILSON, INC.
Geotechnical and Environmental Consultants

FIG. A-1
Sheet 2 of 2

Total Depth:	120.5 ft.	Northing:		Drilling Method:	Mud Rotary	Hole Diam.:	5 in.
Top Elevation:	42.0 ft.	Easting:		Drilling Company:	Boart Longyear	Rod Diam.:	
Vert. Datum:		Station:		Drill Rig Equipment:		Hammer Type:	Cathead
Horiz. Datum:		Offset:		Other Comments:			

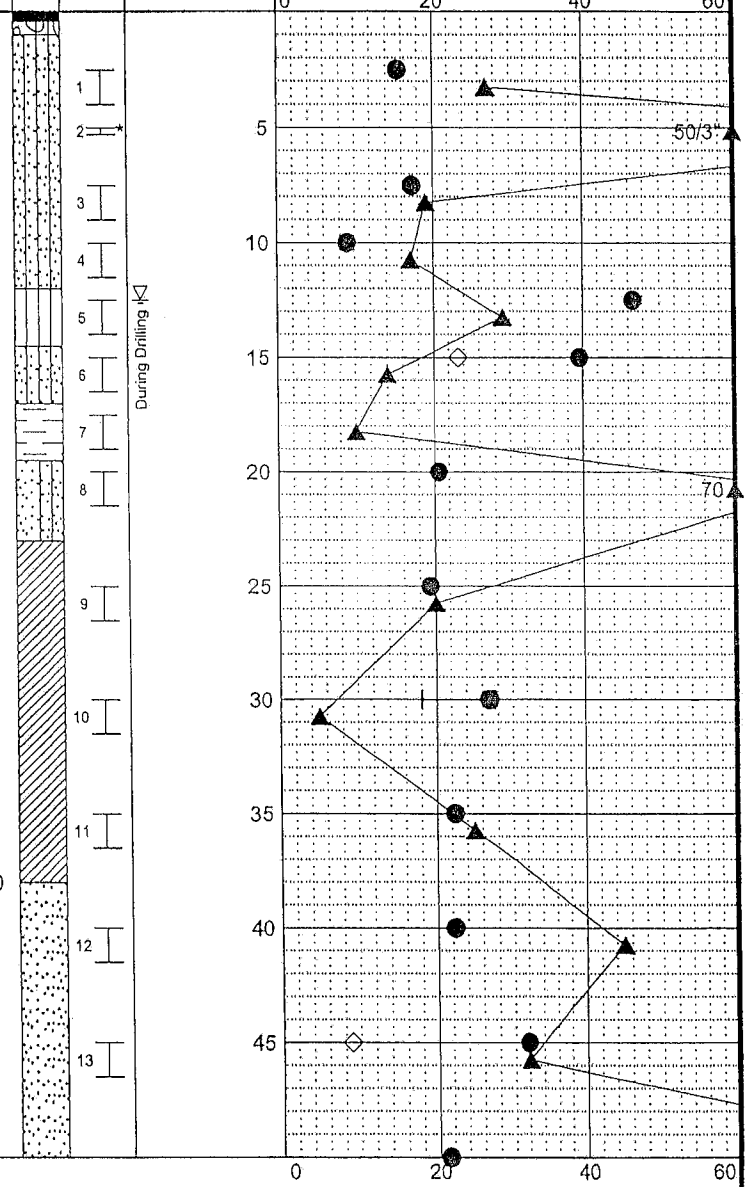
EXHIBIT 8
PAGE 1 of 64

SOIL DESCRIPTION

Refer to the report text for a proper understanding of the subsurface materials and drilling methods. The stratification lines indicated below represent the approximate boundaries between material types, and the transition may be gradual.

Description	Depth, ft.
ASPHALT.	0.4
Crushed 5/8-inch-minus GRAVEL sub-base; moist; (Fill) GP-GM.	1.0
Medium dense and very dense, brown, silty, gravelly SAND to silty, sandy GRAVEL; moist to wet; scattered iron-oxide staining; (Fill) SM/GM.	12.0
Very stiff, gray to brown, slightly fine to medium sandy, clayey SILT; moist; scattered organics, ash; (Recent Lacustrine Deposit) ML.	14.5
Medium dense, gray-brown, silty, fine to medium SAND; wet; bedded, scattered wood fragments; (Recent Alluvium) SM.	17.0
Medium dense, dark brown, interbedded, fine sandy, organic SILT and slightly silty, fine SAND; wet; (Recent Organic Deposit) OL/SP-SM.	19.5
Very dense, dark gray, clean to slightly silty, fine to medium SAND, trace of gravel; wet; abundant wood fragments; (Recent Alluvium) SP-SM/SP.	23.0
Medium stiff, gray, silty CLAY, trace of fine sand; fine sandy SILT, trace of clay; and medium dense, silty, fine to medium SAND; wet; bedded, silty at top and sandy toward bottom; (Recent Lacustrine Deposit) CL/ML/SM.	38.0
Dense and very dense, gray, clean to slightly silty, fine to medium SAND, trace of gravel; wet; locally silty toward top; (Pre-Vashon Non-Glacial Alluvium) SP/SP-SM.	

CONTINUED NEXT SHEET



LEGEND

* Sample Not Recovered

Standard Penetration Test

Ground Water Level ATD

◇ % Fines (<0.075mm)

● % Water Content

Plastic Limit ├─●─┤ Liquid Limit

Natural Water Content

NOTES

1. Refer to KEY for explanation of symbols, codes, abbreviations and definitions.
2. Groundwater level, if indicated above, is for the date specified and may vary.
3. USCS designation is based on visual-manual classification and selected lab testing.

SR 202 Sammamish River Bridge
Woodinville, Washington

LOG OF BORING B-1

August 2007 | 21-1-20664-001

SHANNON & WILSON, INC.
Geotechnical and Environmental Consultants

FIG. A-2
Sheet 1 of 3

REV 1.5

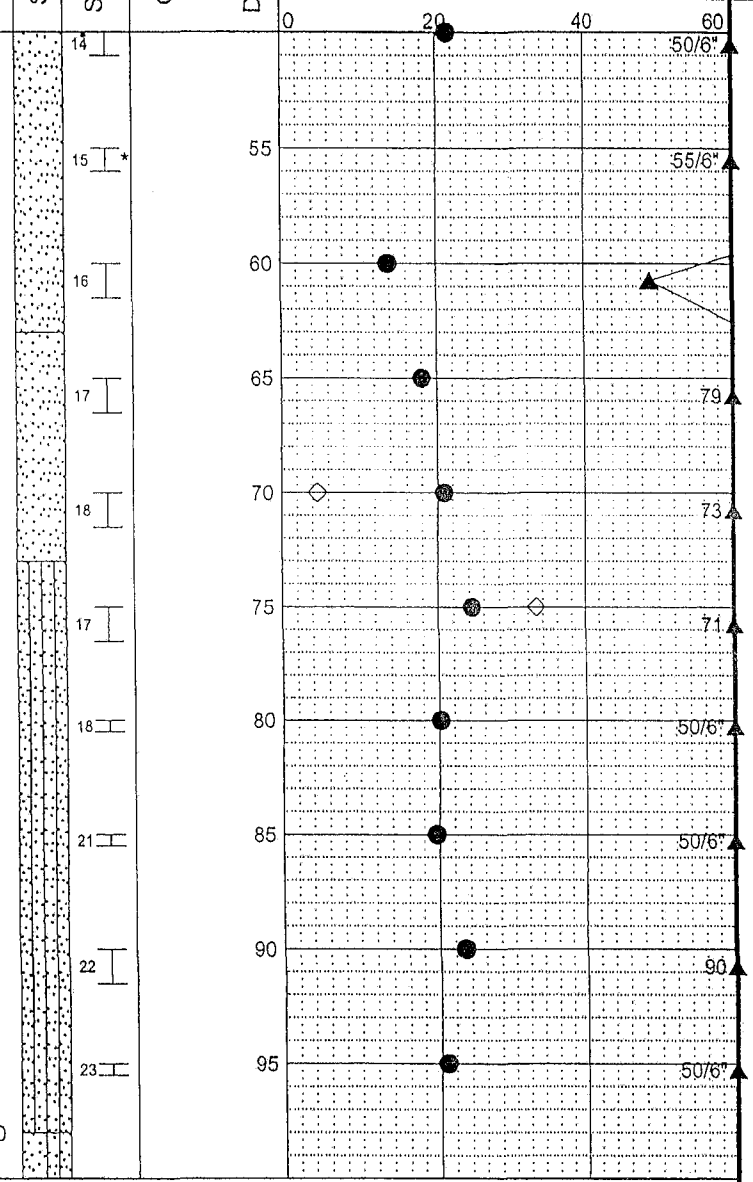
Log: CKS Rev: WDN Typ: EET

MASTER LOG E 21-20664.GPJ 21-20617.GPJ 8/21/07

Total Depth:	120.5 ft.	Northing:		Drilling Method:	Mud Rotary	Hole Diam.:	5 in.
Top Elevation:	42.0 ft.	Easting:		Drilling Company:	Boart Longyear	Rod Diam.:	
Vert. Datum:		Station:		Drill Rig Equipment:		Hammer Type:	Cathead
Horiz. Datum:		Offset:		Other Comments:			

EXHIBIT 8
PAGE 10 OF 24

SOIL DESCRIPTION	Depth, ft.
Refer to the report text for a proper understanding of the subsurface materials and drilling methods. The stratification lines indicated below represent the approximate boundaries between material types, and the transition may be gradual.	
Very dense, gray, fine to medium SAND, trace of silt; wet; (Pre-Vashon Non-Glacial Alluvium) SP.	63.0
Very dense, gray, slightly silty to silty, fine SAND; moist; massive to laminated, abundant very fine organic fragments; scattered clayey silt seams; (Pre-Vashon Non-Glacial Lacustrine Deposit) SM/SP-SM.	73.0
CONTINUED NEXT SHEET	98.0



LEGEND

* Sample Not Recovered
Standard Penetration Test
Ground Water Level ATD

◇ % Fines (<0.075mm)
● % Water Content
Plastic Limit ├─●─┤ Liquid Limit
Natural Water Content

NOTES

1. Refer to KEY for explanation of symbols, codes, abbreviations and definitions.
2. Groundwater level, if indicated above, is for the date specified and may vary.
3. USCS designation is based on visual-manual classification and selected lab testing.

SR 202 Sammamish River Bridge
Woodinville, Washington

LOG OF BORING B-1

August 2007 | 21-1-20664-001

SHANNON & WILSON, INC.
Geotechnical and Environmental Consultants

FIG. A-2
Sheet 2 of 3

MASTER LOG E 21-20664.GPJ 21-20617.GPJ 8/21/07 Log: CKS Rev: WDN Typ: EET

Total Depth:	120.5 ft.	Northing:		Drilling Method:	Mud Rotary	Hole Diam.:	5 In.
Top Elevation:	42.0 ft.	Easting:		Drilling Company:	Boart Longyear	Rod Diam.:	
Vert. Datum:		Station:		Drill Rig Equipment:		Hammer Type:	Cathead
Horiz. Datum:		Offset:		Other Comments:			

EXHIBIT 3
PAGE 5 OF 14

SOIL DESCRIPTION	Depth, ft.	Symbol	Samples	Ground Water	Depth, ft.	PENETRATION RESISTANCE (blows/foot) ▲ Hammer Wt. & Drop: 140 lbs/30 inches
Very dense, gray, clean to slightly silty, slightly gravelly to gravelly, fine to medium SAND; wet; (Pre-Vashon Non-Glacial Alluvium) SP-SM/SP.			24 *			50/5"
			25		105	70/6"
			26 *		110	70/3"
			27		115	70/5"
	120.5		28		120	135/6"
BOTTOM OF BORING COMPLETED 3/20/2007					125	
					130	
					135	
					140	
					145	

0 20 40 60

LEGEND

* Sample Not Recovered
Standard Penetration Test
Ground Water Level ATD

◇ % Fines (<0.075mm)
● % Water Content
Plastic Limit ├─●─┤ Liquid Limit
Natural Water Content

NOTES

1. Refer to KEY for explanation of symbols, codes, abbreviations and definitions.
2. Groundwater level, if indicated above, is for the date specified and may vary.
3. USCS designation is based on visual-manual classification and selected lab testing.

SR 202 Sammamish River Bridge
Woodinville, Washington

LOG OF BORING B-1

August 2007 | 21-1-20664-001

SHANNON & WILSON, INC.
Geotechnical and Environmental Consultants

FIG. A-2
Sheet 3 of 3

Total Depth:	100.5 ft.	Northing:		Drilling Method:	Mud Rotary	Hole Diam.:	6.25 in.
Top Elevation:	45.3 ft.	Easting:		Drilling Company:	Boart Longyear	Rod Diam.:	
Vert. Datum:		Station:		Drill Rig Equipment:		Hammer Type:	Cathead
Horiz. Datum:		Offset:		Other Comments:			

EXHIBIT 8

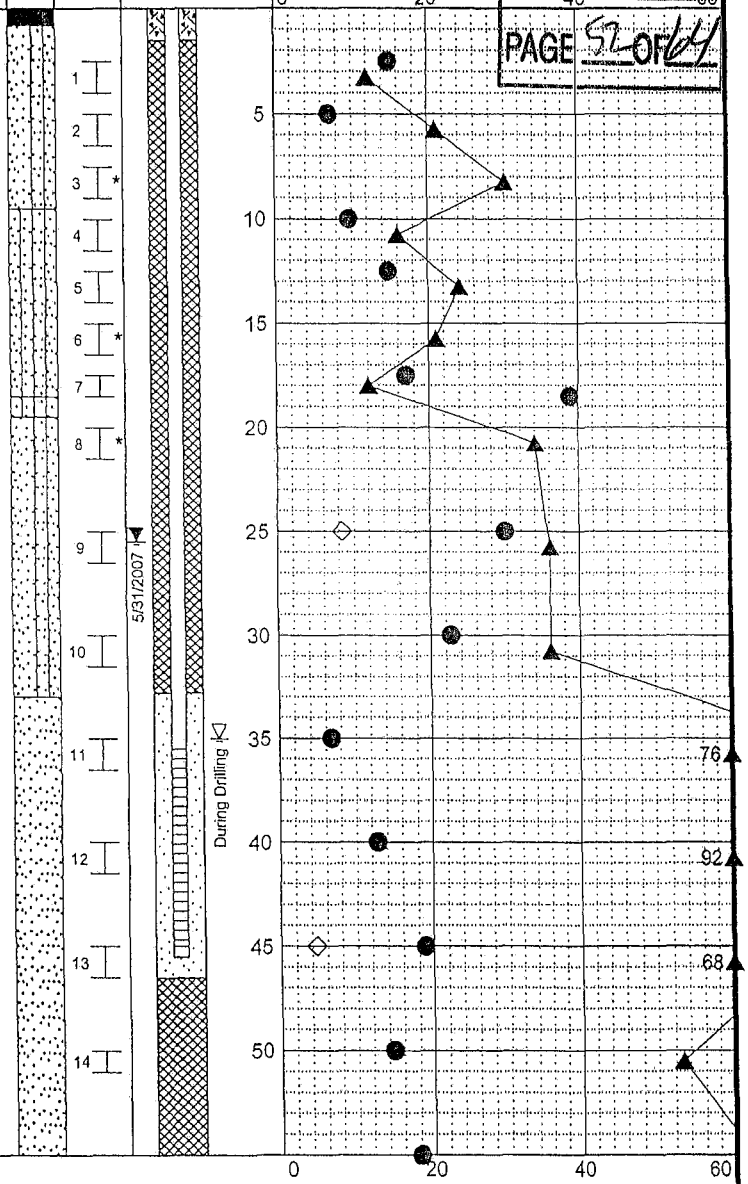
PAGE 52 OF 64

SOIL DESCRIPTION

Refer to the report text for a proper understanding of the subsurface materials and drilling methods. The stratification lines indicated below represent the approximate boundaries between material types, and the transition may be gradual.

Description	Depth, ft.
ASPHALT.	0.8
Medium dense, brown, slightly silty, gravelly SAND; moist; (Fill) SP-SM.	9.5
Medium dense, dark brown, silty, gravelly SAND; moist; locally gravelly at top, abundant wood fragments and slight creosote odor toward bottom; (Fill) SM.	18.5
Medium dense, dark brown, silty SAND, trace of gravel; moist; abundant roots and organic fragments; (Buried Topsoil) SM.	19.5
Dense, gray, slightly silty to silty SAND, trace of gravel and organic silt; wet; interbedded with thin organic-rich seams; (Recent Alluvium) SP-SM/SM.	33.0
Very dense, gray to dark gray, trace of gravel to slightly gravelly SAND, trace of silt to fine to medium SAND, trace of silt; wet; (Pre-Vashon Non-Glacial Alluvium) SP.	

CONTINUED NEXT SHEET



LEGEND

- * Sample Not Recovered
- Standard Penetration Test
- Piezometer Screen and Sand Filter
- Bentonite-Cement Grout
- Bentonite Chips/Pellets
- Bentonite Grout
- Ground Water Level ATD
- Ground Water Level in Well
- ◇ % Fines (<0.075mm)
- ● % Water Content
- Plastic Limit ├─●─┤ Liquid Limit
- Natural Water Content

NOTES

1. Refer to KEY for explanation of symbols, codes, abbreviations and definitions.
2. Groundwater level, if indicated above, is for the date specified and may vary.
3. USCS designation is based on visual-manual classification and selected lab testing.

SR 202 Sammamish River Bridge
Woodinville, Washington

LOG OF BORING B-2 (OW)

August 2007 | 21-1-20664-001

SHANNON & WILSON, INC.
Geotechnical and Environmental Consultants

FIG. A-3
Sheet 1 of 2

EXHIBIT 8
PAGE 53 OF 64

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 to water content), provide correlation to soil properties, and estimate liquefaction potential.

The LL, PL, and PI values determined from the Atterberg Limits test are summarized in Figure B-3. The plasticity chart provides the USCS group symbol, sample description, water content, and percent passing the No. 200 sieve (if a grain size analysis was performed). The results of the Atterberg Limits determination are also shown graphically in the boring B-1 log in Appendix A.

B.5 ORGANIC CONTENT DETERMINATION

The ash (or organic) content of one sample was determined in accordance with ASTM Designation: D 2974, Standard Methods for Moisture, Ash and Organic Matter of Peat and Other Organic Soils. The organic content is presented in the table below.

SUMMARY OF ORGANIC CONTENT

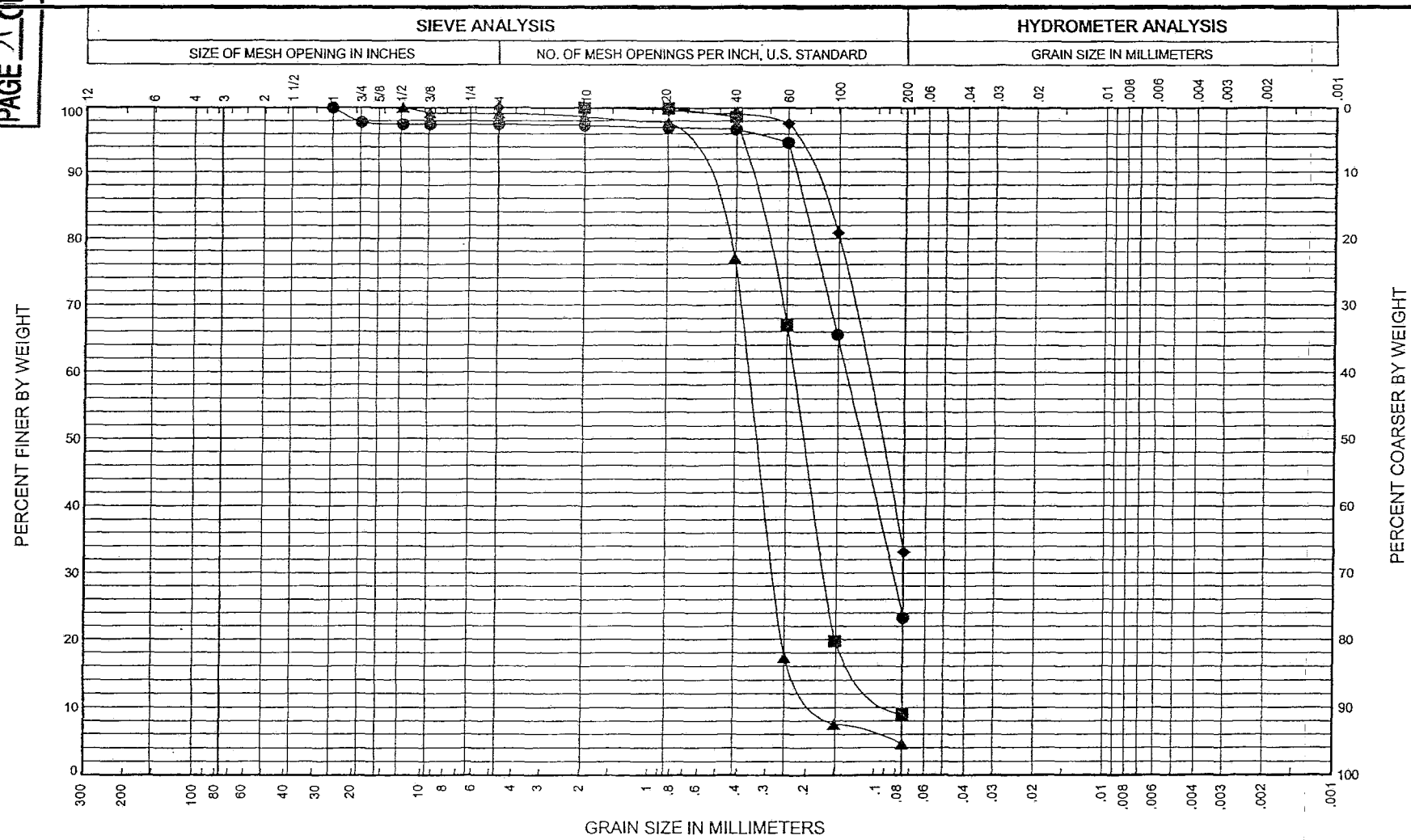
Boring Designation	Sample Number	Sample Depth or Interval (feet)	Unified Soil Classification System Symbol	Organic Content (percent)
B-1	S-7	17.5	OL/SP-SM	8.5

B.6 REFERENCE

American Society for Testing and Materials (ASTM), 2007, Annual book of standards, construction, v. 4.08, soil and rock (I): D 420 – D 5611: West Conshohocken, Pa.

EXHIBIT ___ PAGE ___ OF ___

EXHIBIT 8
PAGE 31 OF 41



COBBLES	GRAVEL: COARSE	GRAVEL: FINE	SAND: COARSE	SAND: MEDIUM	SAND: FINE	FINES: SILT OR CLAY

BORING AND SAMPLE NO.	DEPTH (feet)	U.S.C.S. SYMBOL	SAMPLE DESCRIPTION	FINES %	NAT. W.C. %	LL %	PL %	PI %
● B-1, S-6	15.0	SM	Gray-brown, silty, fine SAND, trace of gravel; scattered wood fragments	23.2	39.6			
■ B-1, S-13	45.0	SP-SM	Gray-brown, slightly silty, fine SAND	8.9	31.8			
▲ B-1, S-18	70.0	SP	Gray SAND, trace of silt and gravel	4.6	20.8			
◆ B-1, S-17	75.0	SM	Gray, silty, fine SAND	33.1	24.3			

SR 202 Sammamish River Bridge
Woodinville, Washington

GRAIN SIZE DISTRIBUTION
BORING B-1

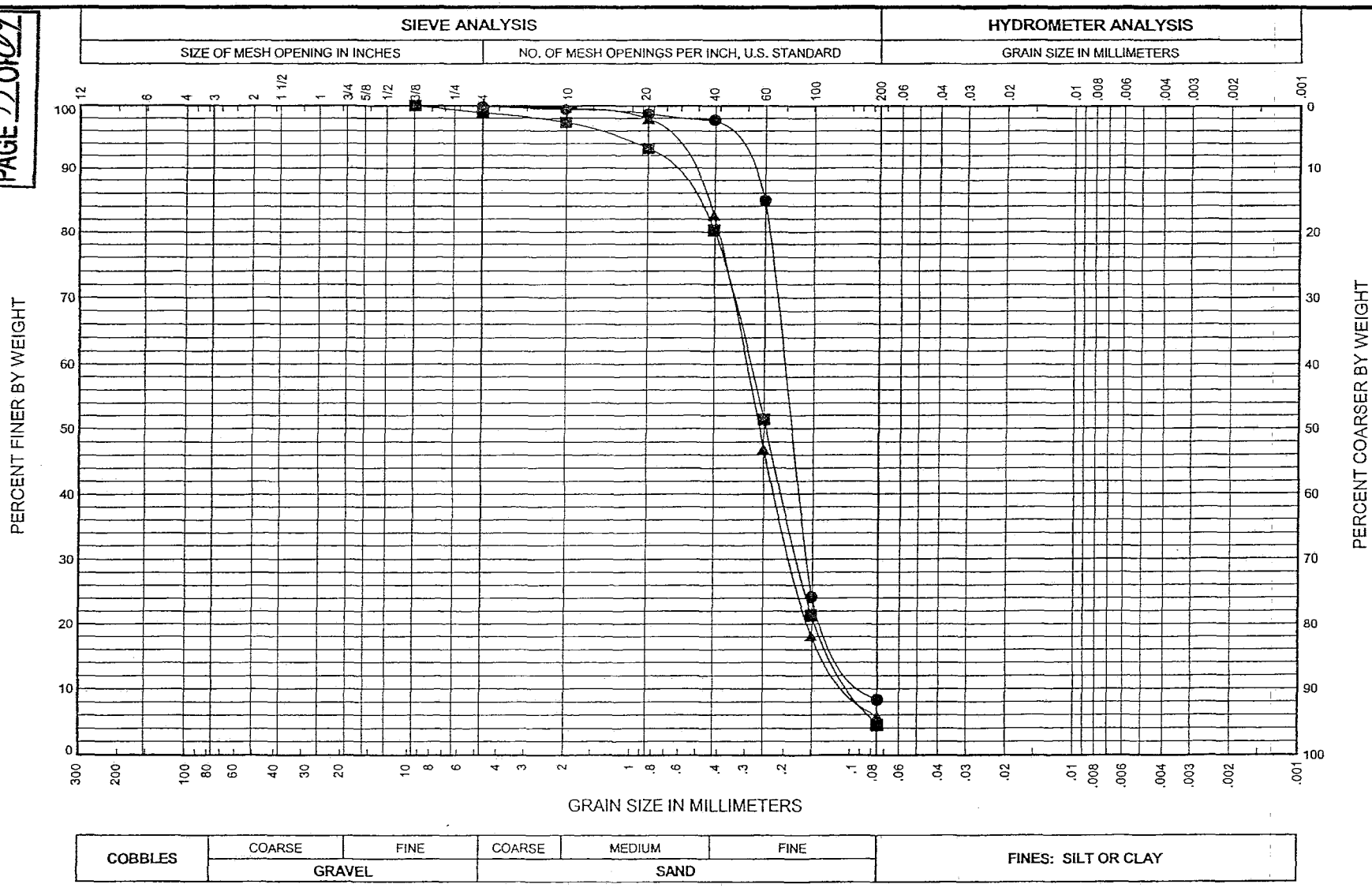
August 2007 | 21-1-20664-001

SHANNON & WILSON, INC.
Geotechnical and Environmental Consultants

FIG. B-1

FIG. B-1

EXHIBIT 8
PAGE 55 OF 64



BORING AND SAMPLE NO.	DEPTH (feet)	U.S.C.S. SYMBOL	SAMPLE DESCRIPTION	FINES %	NAT. W.C. %	LL %	PL %	PI %
● B-2 (OW), S-9	25.0	SP-SM	Gray, slightly silty, fine SAND, trace of gravel; scattered wood fragments	8.3	29.7			
■ B-2 (OW), S-13	45.0	SP	Gray, fine to medium SAND, trace of silt and gravel	4.5	18.8			
▲ B-2 (OW), S-19	75.0	SP-SM	Gray, slightly silty, fine to medium SAND	5.8	19.1			

SR 202 Sammamish River Bridge
Woodinville, Washington

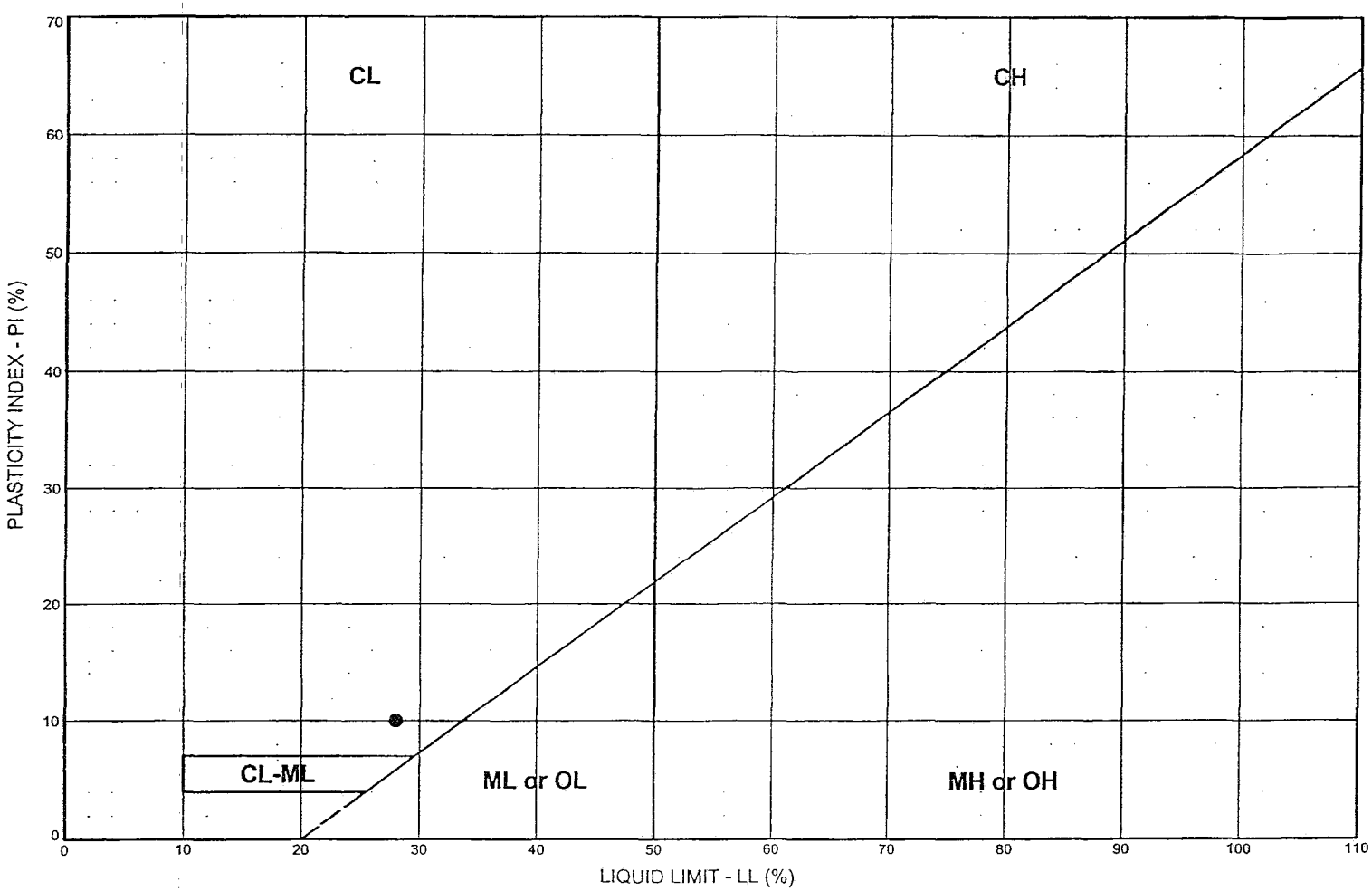
GRAIN SIZE DISTRIBUTION
BORING B-2

August 2007 | 21-1-20664-001

SHANNON & WILSON, INC.
Geotechnical and Environmental Consultants

FIG. B-2

FIG. B-2



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

EXHIBIT 8
PAGE 56 OF 64

BORING AND SAMPLE NO.	DEPTH (feet)	U.S.C.S. SYMBOL	SOIL CLASSIFICATION	LL %	PL %	PI %	NAT. W.C. %	PASS. #200, %
● B-1, S-10	30.0	CL	Gray, slightly fine sandy, silty CLAY	28	18	10	26.9	

SR 202 Sammamish River Bridge
Woodinville, Washington

PLASTICITY CHART

August 2007 21-1-20664-001

SHANNON & WILSON, INC.
Geotechnical and Environmental Consultants

FIG. B-3

FIG. B-3

ATT_MAIN 21-20664.GPJ SHAN WIL.GDT 8/20/07

EXHIBIT 8
PAGE 57 OF 64

APPENDIX C

PAVEMENT CONSULTANTS, INC.
PAVEMENT WIDENING AND OVERLAYING REPORT

Pavement Design Report
SR202 – Widening and Overlaying

As part of the project, pavement thickness designs were required for widening of State Road 202 and thickness of overlay for existing lanes. The pavement design parameter assumptions and resulting pavement thickness designs are provided below.

Assumptions:

Subgrade:
The existing pavements were tested with Falling Weight Deflectometer (FWD) to determine the subgrade strength. The deflection data and the strength of pavement layers including subgrade are shown in Table 1. There were two borings taken to determine the cross-section of existing lanes and the resulting cross-section is shown in Table 2.

Table 2. Cross-Section of Existing Lanes from Borings.

Boring Location	Cross-Section
1- West of bridge	5" Asphalt Concrete 7" Gravel Subbase Subgrade
2 – East of Bridge	9-1/2" Asphalt Concrete Subgrade

The subgrade strengths are expressed in resilient modulus and are shown for each point tested in Table 1. Resilient Modulus values for subgrade indicate subgrade to be of good quality and well compacted.

Traffic:
The City of Woodinville provided the traffic information (Table 3) required to develop the pavement thickness designs. The traffic assumptions are as follows:

- The truck counts shown in Table 3 are representative of the roadway during the design period (20 years) in each direction;
- Truck traffic will grow at a rate of 3% annually;
- Widening lane will use 50% of the truck traffic, and existing outside lane will use the remaining 50% of the truck traffic.

Table 3 shows the truck and car counts and truck traffic is used for determining Equivalent Single Axle Loads (ESALs). Based on the actual counts as shown in Table 3, there are seven percent trucks of the total traffic. WSDOT has

determined ESAL factor of 1.00 to be appropriate for all trucks found in the urban location and this value is used to determine ESALs for 20 years.

Based on the above assumptions and data from Table 3, Design ESALs are 7,258,000.

Subsurface drainage requirements:
It will be advantageous to eliminate trapped water in the widening lane by extending the crushed surfacing base course through the shoulder wherever possible, and by providing a collection system where water is trapped.

Thickness Design:

Using the WSDOT Pavement Guide, Volume 1 and AASHTO Guide for the Design of Pavement Structures, 2002, the following assumptions are used to develop pavement sections for the widening lane and for the overlay thickness:

- Reliability = 95%
- Delta PSI = 1.5
- So = 0.50
- M = 1.0
- Layer coefficient of HMA = 0.44
- Layer coefficient of Crushed Surfacing Base Course = 0.13

Widening Lane Pavement design Recommendation:

0.2' HMA CL ½" PG 64-22
0.4' HMA CL ½" PG 64-22
0.5' (Crushed Surfacing Base Course) CS

Overlay Thickness design for Existing lanes Recommendation:

0.2' HMA CL ½" PG 64-22

Table 1. DEFLECTION DATA AND PAVEMENT MATERIAL PROPERTIES FOR SR202, WOODINVILLE PROJECT

Station (ft.)	Lane	Air Temp. (F)	Load (lbs.)	D0 (mils)	D12 (mils)	D24 (mils)	D36 (mils)	D48 (mils)	D60 (mils)	D72 (mils)	Surface (psi)	Base (psi)	Subgrade (psi)	Comment
0	SB Left	51	22,743	28.84	20.74	13.58	8.89	6.45	5.08	4.17	474,375	46,422	17,728	131th heading S
52	SB Left	51	23,091	25.30	14.40	7.83	4.76	3.58	3.15	2.52	549,062	67,867	31,049	in left lane
128	SB Left	51	23,223	23.96	14.60	8.38	5.43	4.09	3.58	3.15	583,022	67,334	26,022	
200	SB Left	51	23,804	18.06	13.38	9.05	6.02	4.29	3.42	2.79	792,907	75,312	27,833	
250	SB Left	50	23,990	16.17	12.67	8.54	5.75	3.94	3.07	2.48	892,426	80,143	29,459	
368	SB Left	50	24,056	15.50	12.83	9.37	6.49	4.72	3.62	3.03	933,487	79,377	26,138	
										Average:	704,213	69,409	26,371	
										Std. Dev.:	193,619	12,526	4,657	
										X-1SD:	510,594	56,883	21,714	
0	NB Left	53	23,994	16.13	11.77	8.19	5.82	4.33	3.50	2.83	894,747	86,322	29,000	just past the bridge
124	NB Left	53	23,912	16.96	13.38	9.09	6.37	4.53	3.50	2.79	848,266	75,655	26,464	heading north in left
199	NB Left	52	23,819	17.90	13.03	7.99	5.23	3.78	3.03	2.68	800,398	77,410	31,400	lane
263	NB Left	52	24,067	15.39	12.99	9.48	6.61	4.60	3.42	2.68	941,104	78,453	25,684	
301	NB Left	52	23,761	18.49	12.95	8.30	5.67	4.17	3.34	2.75	772,953	77,691	29,584	
401	NB Left	52	23,033	25.89	19.09	12.28	8.46	5.98	4.60	3.70	535,195	51,086	19,207	
										Average:	798,779	74,436	26,890	
										Std. Dev.:	142,871	12,029	4,306	
										X-1SD:	655,908	62,407	22,584	
0	NE	53	24,106	14.99	14.09	12.47	10.90	9.48	6.41	4.64	967,359	72,433	13,450	
101	NE	53	23,905	17.04	13.58	9.21	7.00	5.59	4.37	2.95	844,074	74,535	22,636	
152	NE	53	23,881	17.28	13.28	8.78	6.14	4.60	3.62	3.07	831,728	76,230	27,446	
206	NE	53	23,835	17.75	13.30	8.93	6.37	4.76	3.86	3.31	808,022	75,856	25,436	
268	NE	53	22,886	27.39	18.18	11.96	8.03	5.78	4.72	4.09	502,741	53,287	19,726	starting from the train
438	NE	53	23,978	16.29	14.52	11.57	8.93	6.73	5.12	3.97	885,530	69,901	18,855	tracks heading NE
501	NE	53	23,773	18.38	14.36	10.31	6.93	4.92	3.86	3.07	778,309	70,062	24,217	
606	NE	52	24,048	15.58	12.99	9.60	7.00	5.12	3.94	3.19	928,473	78,390	24,222	
700	NE	53	23,386	22.31	14.64	8.74	5.43	3.74	3.11	2.75	630,596	67,624	28,329	
826	NE	53	22,971	26.52	19.20	12.32	8.22	5.90	4.68	3.90	521,084	50,636	19,705	
902	NE	52	23,327	22.90	18.73	13.42	9.64	6.89	5.31	4.21	612,817	52,718	17,070	
										Average:	755,521	67,425	21,917	
										Std. Dev.:	162,309	10,275	4,576	
										X-1SD:	593,212	57,150	17,341	
0	SW	52	22,212	34.24	25.58	15.54	9.33	5.86	4.60	3.74	390,348	36,759	15,122	
99	SW	53	22,533	30.97	19.60	11.33	7.20	5.23	4.49	3.94	437,762	48,673	20,199	
201	SW	53	23,455	21.60	16.33	10.43	6.77	4.72	3.62	2.99	653,213	60,798	23,803	
366	SW	53	23,347	22.71	15.70	10.43	7.04	4.88	3.58	2.95	618,641	62,943	23,384	just past the first
407	SW	53	24,184	14.21	11.65	8.46	6.22	4.53	3.38	2.68	1,024,233	87,887	27,441	tracks heading SW
451	SW	53	23,254	23.65	16.57	10.15	6.26	4.45	3.46	2.68	591,571	59,417	24,239	
513	SW	53	23,990	16.17	14.36	11.84	9.60	7.52	5.94	4.64	892,426	70,702	16,889	
689	SW	53	23,122	24.99	18.02	11.10	6.89	4.60	3.54	2.83	556,725	54,307	22,050	just past the bridge
750	SW	54	23,726	18.85	14.25	9.64	6.57	4.56	3.34	2.64	757,327	70,505	25,472	
855	SW	53	24,009	15.98	14.28	11.69	9.37	7.24	5.59	4.33	904,146	71,149	17,545	
										Average:	682,639	62,314	21,615	
										Std. Dev.:	208,126	14,078	4,040	
										X-1SD:	474,513	48,236	17,575	

EXHIBIT 8
PAGE 60 OF 64

EXHIBIT 8
PAGE 61 OF 64

Table 3. City of Woodinville
Sammamish Bridge and Road Project (SBRP)
Traffic Counts East of SR 202/127th Place NE

8/2/2007 7:00 am to 9:00 am Counts				
Time	Trucks East	Trucks West	Cars East	Cars West
7:00 AM	7	6	169	56
7:15 AM	12	7	128	74
7:30 AM	4	2	154	74
7:45 AM	9	6	130	80
8:00 AM	5	10	145	65
8:15 AM	7	9	131	84
8:30 AM	8	10	141	82
8:45 AM	8	6	121	97
TOTALS	60	56	1,119	612

8/1/2007 11:00 am to 1:00 pm Counts				
Time	Trucks East	Trucks West	Cars East	Cars West
11:00 AM	19	12	163	110
11:15 AM	12	16	146	126
11:30 AM	11	10	141	163
11:45 AM	15	11	168	138
12:00 PM	13	14	163	126
12:15 PM	11	8	151	120
12:30 PM	9	8	142	154
12:45 PM	17	10	162	147
TOTALS	107	89	1,236	1,084

EXHIBIT 8
PAGE 62 OF 64

APPENDIX D

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