I-90 SNOQUALMIE PASS EAST PROJECT

Phase 1C Roadway Geotechnical Engineering Report (Volume 1 of 2)
2010 Geotechnical Analysis and Reporting

March 2011

Washington State Department of Transportation

U. S. Department of Transportation
Federal Highway Administration

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I-90 Snoqualmie Pass East

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Phase 1C Roadway Geotechnical Engineering Report

2010 Geotechnical Analysis and Reporting

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PHASE 1C ROADWAY GEOTECHNICAL ENGINEERING REPORT

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ACRONYMS

AASHTO	American Association of State Highway and Transportation Officials
ADSC	International Association of Foundation Drilling
AE	eastbound construction centerline (temporary)
AIM	Arthur I. Mears, P.E., Inc.
amsl	above mean sea level
ASD	allowable stress design
ASTM	American Society of Testing and Materials
AW	westbound construction centerline (temporary)
bgs	below ground surface
BP	before present
Burk	Burk GeoConsult LLC
CADD	computer-aided drafting and design
CAVFS	compost-amended vegetated filter strip
CDF	controlled density fill
CEA	connectivity emphasis area
CEVP	cost estimation validation process
cm	centimeter
COBL®	CRUX Oriented Borehole Logging
CRA	cost risk assessment
CRUX	CRUX Subsurface Inc.
CU	consolidated-undrained triaxial compression
DMP	deformation monitoring point
EB	eastbound
EH	lateral earth load
El.	elevation above mean sea level in feet
EPRI	Electric Power Research Institute
ES	earth surcharge load
FEIS	final Environmental Impact Statement
FEMA	Federal Emergency Management Agency
FEU	WSDOT Field Exploration Unit
FHWA	Federal Highway Administration
FPR	factored pullout resistance
F&S	Fisher & Strickler Rock Engineering LLC
GDM	WSDOT Geotechnical Design Manual
GPS	global positioning system
GSI	Geologic Strength Index
GSP	General Special Provision
HDPE	high density polyethylene
HMF	Hyak Maintenance Facility
I-90	Interstate-90
In-Situ	In-Situ Engineering Inc.
kip	kilopound
km	kilometer
KPFF	KPFF Consulting Engineers
ksf	kips per square foot
ksi	kips per square inch

ACRONYMS (continued)

lb	pound
LE	eastbound construction centerline
LL	liquid limit
LRFD	Load and Resistance Factor Design
LW	westbound construction centerline
mm	millimeter
MP	milepost
MPa	megaPascal
MSE	mechanically stabilized earth
N	standard penetration test blow count
NAD	North American Datum
NAVD	North American Vertical Datum
NGA	Northwest Geophysical Associates Inc.
OHWM	ordinary high water mark
OSHA	Occupational Safety and Health Administration
OW	observation well
pcf	pounds per cubic foot
PE	probability of exceedance
PGA	peak ground acceleration; permanent ground anchor
PI	plasticity index
PL	plastic limit
PLT	point load test
PMT	pressuremeter test
PS&E	contract plans, specifications, and estimate
psf	pounds per square foot
psi	pounds per square inch
PVC	polyvinyl chloride
RCB	Resort Creek bridges
RCW	Resort Creek wall
rFS	factor of safety, residual strength analysis
RMR	rock mass rating
RQD	rock quality designation
RRB	Rocky Run bridges
RTK GPS	real-time kinematic global positioning system
SCB	Slide Curve bridge and walls
SCR	WSDOT South Central Region
SCW	Slide Curve median wall
SHANSEP	stress history and normalized soil engineering properties
SNW	soil nail wall
SPT	standard penetration test
SSD	Snowshed
SSE	sign structure
Sta.	station
S&W	Shannon and Wilson
SW1	Structural Wall 1
SW2	Structural Wall 2
TCC	Townsend Creek culvert

ACRONYMS (continued)

TM	technical memorandum
TO	task order
tsf	tons per square foot
UCS	unconfined compressive strength
URS	URS Corporation
USBR	U.S. Bureau of Reclamation
USFS	U.S. Forest Service
USGS	U.S. Geological Survey
WB	westbound
WBS	Work Breakdown Structure
VMS	variable message sign
WISHA	Washington Industrial Safety and Health Act
WSDOT	Washington State Department of Transportation
W&N	Wyllie and Norrish Rock Engineers

1.0 INTRODUCTION

1.1 GENERAL

Washington State Department of Transportation (WSDOT) has identified that the part of Interstate 90 (I-90) on the east side of Snoqualmie Pass between Hyak and Easton (milepost (MP) 55.1 to 70.3) needs improvement. WSDOT has selected the Hyak to Keechelus Dam segment (MP 55.1 to 59.9) as the first phase of the project. The project location is shown in Figure 1.1.

The objective of the project is to improve the roadway by widening and re-aligning the existing highway and constructing or replacing structural elements that do not meet current federal and WSDOT highway standards. The purposes of the improvements are to eliminate or reduce avalanche closures, increase capacity, stabilize slopes, enhance freight mobility, replace pavement, improve sight distance, and address environmental stewardship.

Phase 1 of the project includes multiple subphases:

- 1A: Construction of the detour routes, including structures, and excavation of the Rocky Run Lake Storage Mitigation site. Construction of this phase began in July 2009 and was completed in October 2009.

- 1B: Construction of I-90 from Coal Creek bridge (MP 55.1) to the west end of the proposed snowshed (MP 58.0). Construction of this phase began in April 2010 and is scheduled to be completed in Fall 2013.

- 1C: Construction of the remainder of the project from MP 58.0 to MP 59.9. Construction of this phase is scheduled to begin in Spring 2011 and be completed in Fall 2016.

- 1D/F: Completion of roadside restoration, connectivity emphasis area (CEA) mitigation, offsite stormwater mitigation, fencing, and the Hyak facility. Construction of this phase is scheduled to begin in Spring 2014 and be completed in Fall 2016.

This report presents the results of the roadway engineering part of the 2008-2010 investigation for Phase 1C. The results of the Phase 1B roadway engineering part of the investigation were presented in an earlier report (URS 2009a).

WSDOT contracted URS Corporation (URS) as General Engineering Consultant for the project, under WSDOT Agreement No. Y-9764, dated February 14, 2006. URS is completing a 2007-2010 geotechnical investigation for final design as Task Orders (TO) BS (2007), CJ (2008), DG (2009), and DX (2010) for the field work and TOs CA (2007), CP (2008-2009) and DO (2010) for the analysis and reporting. The geotechnical investigation was conducted in three parts:

- A geotechnical investigation of various roadway structures to provide supplemental design recommendations for the roadway structures (referred to as roadway engineering in this report);

- A rock slope evaluation by Wyllie & Norrish Rock Engineers (W&N) of Redmond, Washington, with support from Fisher & Strickler Rock Engineering LLC (F&S) of Prescott, Arizona, both under subcontract to URS (referred to as rock slope engineering in this report); and

- A materials resource evaluation by Burk GeoConsult LLC (Burk) of Seattle, Washington, with support from F&S, both under subcontract to URS (referred to as materials resource evaluation in this report).

This Phase 1C Roadway Engineering Report was prepared under TOs CP (Work Breakdown Structure (WBS) I90SPE.CP.ENG.07) and DO (WBS E09009B.PE.DS.07). The Phase 1B roadway engineering was prepared under TO CP.

The rock slope and materials resource evaluations were the bases for 3 reports that were submitted as stand-alone documents under separate cover:

- A rock slope engineering report for Phase 1B (W&N 2009a);
- A rock slope engineering report for Phase 1C (W&N 2009b); and
- A material source report (Burk 2009).

These reports were prepared under TO CP, WBS E09009B.PE.DS.07.

1.2 PREVIOUS INVESTIGATIONS

URS previously completed geotechnical investigations in 2006 and 2007 that included:

- A 2006 conceptual geotechnical investigation for preliminary design that included a compilation and review of previous geotechnical investigations in the project area (URS and W&N 2007) and

- A 2007 investigation that developed geotechnical design recommendations for roadway structures, embankments, rock slopes, and materials sources (URS 2008a; W&N 2008; W&N 2009c; Burk 2008).

In addition to the 2007 Roadway Geotechnical Engineering Report (URS 2008a), geotechnical recommendations for three major Phase 1C roadway structures were presented in separate technical memoranda because of their added foundation complexity:

- Technical Memorandum No. 3 (TM3) – Snowshed Replacement, June 2008 (URS 2008b);

- Technical Memorandum No. 4 (TM4) – Structural Wall 1, June 2008 (URS 2008c); and

- Technical Memorandum No. 5 (TM5) – Structural Wall 2, June 2008 (URS 2008d).

Two additional technical memoranda were prepared for Phase 1B structures:

- Technical Memorandum No. 1 (TM1) – Gold Creek bridges, June 2008 (URS 2008e; URS 2009b; URS 2009c; URS 2009d) and

- Technical Memorandum No. 2 (TM2) – Rocky Run bridges, April 2008 (URS 2008f; URS 2009e)

1.3 PURPOSE AND SCOPE

The purpose of the I-90 Snoqualmie Pass East – Hyak to Keechelus Dam project is to provide the following improvements:

- **Eliminate or reduce avalanche closures**: Eliminate or reduce the risk of avalanches and associated road closures, and construct improved avalanche control structures.
- **Increase Capacity**: Improve traffic flow and provide additional capacity for future growth by widening the highway from four to six lanes.
- **Stabilize Slopes**: Stabilize cut slopes to eliminate and/or minimize the risk of rock and debris from unstable slopes reaching the roadway.
- **Enhance Freight Mobility**: Improve freight mobility with another lane in each direction, longer chain-on and chain-off areas, straighter alignment, and new pavement.
- **Replace Pavement**: Replace cracked and deteriorated pavement with new pavement to provide a smoother and more reliable ride.
- **Improve Sight Distance**: Straighten substandard roadway curves to improve sight distance, drivability, and safety for a design speed of 65 miles per hour.
- **Address Environmental Stewardship**: Improve ecological connectivity by increasing the size and number of crossings and habitat connections, and minimize environmental impacts.

These improvements involve widening and re-aligning the existing highway and constructing or replacing various structural elements that do not meet the current federal and WSDOT highway standards.

The purpose of the 2008-2010 roadway geotechnical investigation is to provide geotechnical data, analyses, and recommendations for the design of certain project elements, including the following Phase 1C elements (section numbers refer to this report):

- Snowshed replacement (Section 6);
- Slide Curve bridge and walls (formerly Structural Wall 1) (Section 7);
- Slide Curve median wall (Section 8);
- Slide Curve embankment stabilization (formerly Structural Wall 2) (Section 9);
- Resort Creek wall[1] (Section 10);
- Resort Creek bridges (Section 11);

[1] The proposed Resort Creek wall has been deleted from the project.

- Culverts at the I-90 crossing of Townsend Creek and the Old Sunset Highway Crossing of Resort Creek (Section 12)[2];
- Roadway embankments (Section 13);
- Sign structures (Section 14);
- Stormwater mitigation facilities (Section 15) ; and
- Crossing at MP 59.7 Unnamed Creek (Section 16).

This report also provides the results of the 2008-2010 investigation of offsite stormwater mitigation facilities, which are a part of Phase 1D.

In order to achieve this purpose, URS completed the following activities in consultation with the WSDOT South Central Region (SCR) and Headquarters geotechnical staff:

- Drilled and sampled 101 roadway borings and 28 test pits and retained representative soil samples and all rock cores.
- Monitored groundwater levels in eight observation wells (OWs);
- Completed a geophysical survey of soil stratigraphy near the eastbound (EB) lane in the area of Slide Curve Bridge; Northwest Geophysical Associates (NGA), under contract to WSDOT, completed a geophysical survey on the Slide Curve median wall face alignment between westbound construction centerline (LW) 1403+00 and 1409+00;
- Conducted field and laboratory testing on selected soil and rock samples; and
- Conducted engineering analyses and prepared this roadway geotechnical engineering report.

In addition to the 2008-2009 roadway geotechnical work, URS completed other TO activities that were used to develop the geotechnical conclusions and recommendations. These activities included:

- Cost estimation validation process/cost risk assessment (CEVP/CRA) workshops on September 22 through 25, 2008 and January 26 through 28, 2010;
- Fourteen computer-aided teleconferences between representatives of the WSDOT I-90 Project Office, Bridge and Structures Office, and Geotechnical Division, URS, and KPFF Consulting Engineers (KPFF) on the following dates:
  - November 14 and December 15, 2008;
  - January 13 and 22, February 10 and 23, March 11 and 25, May 28, July 22, August 4, 17, and 28, and October 19 and 21, 2009; and

[2] Townsend Creek culvert is not part of Phase 1C.

  - January 29, May 12, July 13 and 21, August 18, 24, and 31, and September 15 and 20, 2010

- Numerous email exchanges of information with the WSDOT Bridge and Geotechnical Offices between September 2008 and November 2010.

1.4 STRUCTURE DESIGNATIONS

Throughout this report, the roadway structures are designated using a 3-letter abbreviation, as follows.

Roadway Structure	Designation
Snowshed replacement	SSD
Slide Curve bridge and walls	SCB
Slide Curve median wall	SCW
Slide Curve embankment stabilization	SW2
Resort Creek wall	RCW
Resort Creek bridges	RCB
Townsend Creek culvert	TCC
Hyak maintenance facility	HMF
Sign structures	SSE

1.5 REPORT ORGANIZATION

This roadway geotechnical engineering report is organized into 19 sections (Volume 1) and 11 appendices (Volume 2). Sections 1 through 5 present information that applies to the entire project, including:

- Introduction (Section 1);
- Project description (Section 2);
- Field exploration and laboratory testing programs (Section 3);
- Subsurface conditions (Section 4); and
- Geotechnical analysis methods (Section 5).

Geotechnical engineering data, analyses, and recommendations for specific Phase 1C project elements that were investigated during the 2008-2010 geotechnical program are presented in Sections 6 through 16, as listed in Section 1.3.

Section 17 summarizes preliminary submittals in support of the Phase 1C contract plans, specifications, and estimates (PS&E) development. Section 18 describes the limitations of this report. Section 19 presents the references used in developing the report. The appendices include:

- Appendix A – 2008-2010 Boring and Test Pit Logs
- Appendix B – Rock Core and Test Pit Photographs

- Appendix C – Geophysical Surveys
- Appendix D – 2008-2010 Field and Laboratory Testing Data
- Appendix E – Estimation of Rock Mass Strength Properties
- Appendix F – Snowshed Statistical Reliability Analysis
- Appendix G – Preliminary Design Drawings
- Appendix H – Stability and Liquefaction Analysis Graphical Results
- Appendix I – International Association of Foundation Drilling (ADSC) Recommendations
- Appendix J – Slide Curve Median Wall FLAC Model
- Appendix K – Phase 1C PS&E Support

2.0 PROJECT DESCRIPTION

This section presents a project description in four subsections, as follows.

- Site information, including the project location, general information about the existing highway, water level fluctuations in Keechelus Lake (lake levels), and the presence of wetlands;
- The proposed project, including the alignment, roadway structures, embankments, and storm water mitigation facilities;
- The project datum, stationing, and geographic conventions used in this report; and
- Selected design criteria used by WSDOT for the project structures and embankments.

A brief history of road development along the I-90 route by Keechelus Lake is provided in the 2007 Roadway Geotechnical Report (URS 2008a).

Symbols and abbreviations used in the site maps, cross sections, and profiles in this report, as well as general notes applicable to those graphics, are provided in Figure 2.1.

2.1 SITE INFORMATION

2.1.1 Project Location

The project is located east of Snoqualmie Pass in Kittitas County, Washington. Phase 1 extends from Hyak (MP 55.1) to the Keechelus Dam vicinity (MP 59.9), as shown on Figure 1.1. Phase 1C of the project extends from the west end of SSD at MP 58.0 to the end of Phase 1 at MP 59.9.

The general topography within and adjacent to the project area is that of a rugged mountainous region. This part of the highway is surrounded by the following:

- Keechelus Lake along the south of I-90;
- Keechelus Lake Dam at the east end of the lake near MP 61;
- South or west facing slopes between MP 57.64 and MP 59.08 north of I-90, including the area referred to as Slide Curve; and

The ground surface within the project area, in feet above mean sea level (amsl), ranges between elevation (El.) 2450 and El. 2716.

2.1.2 Existing Highway

I-90 is the most heavily traveled east-west highway in Washington. It presently contains four lanes of traffic: two EB lanes and two westbound (WB) lanes. The existing road grade within Phase 1C ranges from El. 2525 to 2540. The proposed EB grade ranges from El. 2525 to 2540, and the proposed WB road grade ranges from El. 2530 to 2580.

2.1.3 Keechelus Lake

Keechelus Dam was constructed at the southern end of Keechelus Lake ten miles northwest of the town of Easton. The lake reservoir and dam are maintained by U.S. Bureau of Reclamation (USBR). The reservoir feeds the Yakima River. A USBR website describes the dam as a 128-foot-high earth fill structure that was completed in 1917. The outlet and control tower were rehabilitated starting in the late 1970s.

The water surface elevation (lake level) typically fluctuates about 60 feet annually, as shown in Figure 2.3. The maximum levels typically occur in June. The level typically begins dropping in mid- to late June and reaches its lowest level in October. Rapid increases of lake level are observed some years from April to June as a result of snowmelt runoff.

In an "average" year, the lake level is above El. 2500 from around mid-April to around mid-July, a period of three months. In the extraordinarily high water year of 1996, the lake level was above El. 2500 from early December until early September, a period of nine months. In the extraordinarily low water year of 2005, the lake level remained below El. 2500 the entire year.

USBR provided WSDOT with lake levels for the 25-year period from 1980 to 2004. The average annual low and average high lake levels for this period are:

- Average Low Lake Level – El. 2439.8
- Average High Lake Level – El. 2511.4

The maximum, minimum, and average lake levels for this period were computed to be the following:

- Maximum Lake Level – El. 2517.8
- Minimum Lake Level – El. 2430.8
- Average Lake Level – El. 2480.1

A WSDOT summary of monthly lake levels statistics and average, median, minimum and maximum lake levels for this 25-year period is shown in Table 2.1. WSDOT identified 1996 and 2005 as the high lake level and low lake level years, respectively. Figure 2.3 compares the lake levels in 1996 (high lake level year), 2005 (low lake level year), and the statistical average year (based on 1981 to 2004 lake level data).

The lake level affects the length of time available to work "in the dry" at any given elevation. Table 2.2 shows the number of weeks the lake level is below various elevations for the high lake level year, average year, and low lake level year.

2.1.4 Wetlands

WSDOT has identified and rated wetlands within the project area. Wetlands are described in the final Environmental Impact Statement (FEIS), with individual wetlands described in the *Wetland/Biology Discipline Report* (Appendix K of the FEIS)(WSDOT and FHWA 2008).

WSDOT has rated project area wetlands based on their functions, using the *Washington State Wetland Rating System for Eastern Washington* (Ecology 2004). The ratings range from

Category I to IV, with Category I considered the most valuable and Category IV considered the least valuable. A few Category I wetlands are located in the study area. These typically contain old-growth or mature conifer trees. Category II wetlands in the study area often are associated with streams. The majority of the wetlands in the study area are Category III and Category IV. Many of the Category IV wetlands in the study area are associated with the ditches that run parallel to the highway.

Major wetlands areas within Phase 1C in the Resort Creek area include riverine wetlands to the north of the highway and lake-fringe wetlands to the south. The riverine wetlands are typically rated as Category II and the lake-fringe wetlands as Category II or III.

2.2 PROPOSED PROJECT

This section describes the proposed alignment, roadway structures, embankments, and storm water mitigation facilities. The purpose of the project is described in Section 1.3. The project location is identified in Section 2.1.1.

The proposed alignments and structures are shown in a series of site plan and boring location maps. Figure 2.1 provides the notes and legend that are used for the site plan and boring location maps, as well as the subsurface cross section and profiles developed for the project. Figures 2.2A through 2.2C present the site plan and boring location maps for Phase 1C. The site plan and boring location maps for Phase 1D are shown in Figures 2.2 D through 2.2F, and in Figures 2.4 and 2.5 for HMF and the Crystal Springs Sno-Park, respectively.

2.2.1 Alignment

The proposed EB and WB centerlines for Phase 1C of the project are shown in Figure 2.2. These figures also indicate station (Sta.) numbers and mileposts. Eastbound and westbound construction centerline (LE and LW) stationing are used for the final roadway construction from the west end of the project to the east end of RCB (LE 1200+00 to 1429+06 and LW 1200+00 to 1428+91). A temporary roadway section will be constructed east of final roadway work. EB and WB temporary construction centerline (AE and AW) stationing are used for the temporary roadway section (AE 100+00 to 124+80 and AW 100+00 to138+55).

The project alignments have been subject to periodic revisions as the highway design has evolved. The March 2010 alignment was used for the cross sections and profiles in this report. The stationings of the borings that are shown on the boring logs are based on the alignment that was current when the boring locations were surveyed.

The roadway was realigned from SCB to RCB in August 2009. Reports and TMs previous to this date refer to older alignments through this section. The primary purpose of the realignment was to eliminate RCW without impacting the wetlands north of the roadway in that area. This was accomplished by shifting the EB alignment up to 40 feet south (toward the lake) east of Slide Curve. The new proposed alignment uses the existing embankment for the WB lanes in that area. This realignment moved SCW toward the rock slopes by as much as 10 feet. A subsequent realignment in March 2010 moved SCW further toward the rock slopes by as much as 5 feet.

The current and superseded alignments are shown in Figure 2.6.

2.2.2 Bridges

The proposed design for Phase 1C includes the construction of the following bridges:

- One EB multi-span bridge (SCB) between SSD and Slide Curve and
- Two (EB and WB) single-span bridges (RCB) crossing Resort Creek.

A summary of the proposed bridges is presented in Table 2.3. The bridge locations are shown in Figures 2.2B and 2.2C. Recommendations for SCB and RCB are provided in Sections 7 and 11, respectively.

Based on the American Association of State Highway and Transportation Officials (AASHTO) Guide Specifications for LRFD Seismic Bridge Design (AASHTO 2009), all project bridges have been classified as "other," that is, as non-critical bridges, for the purposes of seismic design.

2.2.3 Snowshed

The purpose of SSD is to protect both the EB and WB lanes from six avalanche paths. The existing snowshed protects only the WB lane from two main avalanche paths, as described in the Avalanche Mitigation Report, Snowshed Option (URS and Arthur I. Mears, P.E., Inc. (AIM) 2007). I-90 is frequently closed because of active avalanche management and snow removal in the unprotected EB lanes.

SSD will be constructed between LW 1352+50 and 1363+50 and will cover both the EB and WB lanes. The shorter, existing snowshed is approximately 500 feet long between LW 1354+45 and 1359+45. The current conceptual design includes a lakeside permanent geosynthetic wall to support the widened EB lanes.

Geotechnical recommendations for SSD are presented in TM3 (URS 2008b) and in Section 6.

2.2.4 Retaining Walls

Phase 1C includes the construction of 23 walls, including:

- SSD includes five permanent walls (1, 2, 3, 9, and 10) and one temporary wall (Wall 4). Wall 1 is a cast-in-place wall that will support the EB roadway adjacent to the snow catchment between approximately LW 1346+60 and 1352+50. Wall 2 is a gravity block wall that will support the EB embankment adjacent to the generator room. Wall 3 is a geosynthetic wall that will support the EB lanes between approximately LW 1352+50 and 1363+50. Wall 9 includes structural earth walls (Walls 9A, 9B, and 9C) and a counterfort wall (Wall 9D) that will retain the cut slope and avalanche debris adjacent to the west Pier 1 portal tower. Wall 10 is a counterfort wall that will retain avalanche debris adjacent to the east Pier 1 portal tower. Wall 4 is a temporary soil nail wall (SNW) that will support the EB roadway during construction of the Pier 2 substructure and Wall 3.

- SCB includes Walls 5, 6, 7, and 23. Walls 5 and 6 are EB geosynthetic walls that will support the bridge approaches between eastbound construction centerline (LE) 1371+25 and 1372+06 and between LE 1383+58 and 1385+00. Wall 7 is a WB SNW that will support the roadway between LE 1371+72 and 1383+87. Wall 23 is a tied-shaft wall that

will extend Wall 7 to bedrock in an area of deep bedrock between LE 1378+30 and 1381+65.

- SCW (Wall 8) is a 4,570-foot long structural earth wall that will elevate the WB lanes by as much as 50 feet around Slide Curve.

- RCB includes Walls 11 through 18. Walls 11 through 14 are geosynthetic walls that will support the EB approach embankments. Walls 11 through 14 are geosynthetic walls that will support the EB approach embankments. Walls 11 through 14 are structural earth walls that will support the WB approach embankments.

- The MP 59.7 crossing includes Walls 19 through 22, which are gravity block walls that will support the approach embankments.

The wall locations are shown in Figures 2.2A and 2.2B, and the proposed locations, lengths, and heights of the walls are presented in Table 2.4.

Earlier designs included a WB mountain side wall from LW 1420+50 to 1425+85 near the west abutment of the WB RCB. Because of the presence of liquefaction-susceptible silts and sands in this area, stability criteria for the wall under seismic loading could not be achieved without ground stabilization. The wall was replaced with a soil embankment, as described in Sections 10 and 13.

At the SW2 location, a wall was previously considered to stabilize an area where the embankment is experiencing downslope creep and pavement distress. WSDOT is evaluating other options of addressing the creep and pavement distress, including a monitoring and maintenance option and a toe buttress option, as described in TM5 (URS 2008d). An updated analysis of the toe buttress option is presented in Section 9 of this report.

2.2.5 Culverts

The current design for Phase 1C does not include box culverts on mainline I-90. An earlier roadway design included a Resort Creek crossing that consisted of four box culverts. These box culverts have been replaced by EB and WB single-span bridges. The Phase 1C design includes replacement of the box culvert for the Old Sunset Highway crossing of Resort Creek. Recommendations for design of this culvert are presented in Section 12. The location of the culvert is shown in Figure 2.2C.

Geotechnical recommendations for future box culverts at the I-90 crossing of Townsend Creek are presented in Section 12 of this report. The location of the culvert is shown in Figure 2.2D. Geotechnical recommendations for a box culvert at MP 59.7 (Unnamed Creek, also referred to as "Townsend Creek North") were presented in Section 10 of the 2007 Roadway Geotechnical Engineering Report (URS 2008a). The locations of these culverts are shown in Figure 2.2C. The permanent crossings for these two streams will be constructed as part of Phase 2 of the project.

Temporary crossings of Unnamed Creek at MP 59.7 will be constructed as part of Phase 1C. Geotechnical recommendations for these temporary crossings are provided in Section 16.

2.2.6 Embankments

The Phase 1C design includes fill embankments greater than 10 feet high west of Resort Creek from LE 1419+00 to 1425+90 and east of Resort Creek from AE 100+00 to AE 124+80, with a maximum fill height of about 70 feet. The embankments side slopes are planned to be 2H:1V west of Resort Creek and 1.5H:1V east of Resort Creek. Geotechnical recommendations for embankments are presented in Section 13.

2.2.7 Sign Structures

Four cantilever SSEs are planned for Phase 1C at LW 1338+00, LW 1385+00, LW 1421+30, and AE 116+30. Geotechnical recommendations for SSEs are presented in Section 14.

2.2.8 Stormwater Mitigation Facilities

The proposed offsite stormwater mitigation consists of compensatory stormwater treatment at HMF and the Crystal Springs Sno-Park. Runoff from the HMF will be directed into a pretreatment vault, which will discharge into a bioinfiltration swale. Runoff from the Crystal Springs Sno-Park will be directed into compost-amended vegetated filter strips (CAVFSs).

Geotechnical recommendations for stormwater mitigation facilities are presented in Section 15. The HMF and the Crystal Springs Sno-Park offsite mitigation areas are shown in Figures 2.4 and 2.5, respectively.

2.3 PROJECT DATUM AND STATIONING

Surveying for the 2008-2010 geotechnical program was performed by White Shield, Inc. under subcontract to URS. The vertical datum used is the North American Vertical Datum (NAVD) 1988. The horizontal control is based on the Washington Coordinate System, South Zone, and the North American Datum (NAD) 1983/91. State Plane coordinates were converted to the Project Datum for this report. Unless otherwise noted, all elevations and coordinates in this report are referenced to these controls and have been established with an accuracy of ±0.5 feet.

Stationing for the project is established using a baseline of 1200+00 feet at the start of the project at MP 55.1. The stationing of the borings that is shown on the boring logs is based on the stationing that was current at the time the borings were surveyed.

The geographic bearing of the alignment varies through the project area. Because of this, a convention has been adopted in this report to describe the locations of features using the alignment as a reference rather than true geographic directions. Using this convention:

- North is the direction away from Keechelus Lake;
- South is the direction toward Keechelus Lake;
- East is the direction of travel in the EB lane; and
- West is the direction of travel in the WB lane.

2.4 DESIGN CRITERIA FOR STRUCTURES AND EMBANKMENTS

The WSDOT Geotechnical Design Manual (GDM; WSDOT 2010a) requires a minimum factor of safety of 1.5 for global slope stability under static loading for bridges and for walls and reinforced slopes deemed "critical." Critical walls and reinforced slopes are those that support important structures like bridges or other walls. Non-critical walls and reinforced slopes, as well as slopes adjacent to but not directly supporting a structure, are required to have a minimum factor of safety of 1.3 for global slope stability under static loading. For embankments and cut slopes that do not support a structure, the minimum factor of safety under static loading is 1.25.

For seismic design of bridges, including culverts greater than or equal to 20 feet in width, WSDOT has adopted the AASHTO Guide Specifications for LRFD Seismic Bridge Design (AASHTO 2009). This guide specification applies to non-critical bridges, and all project bridges have been classified as non-critical for the purpose of applying the guide specification.

Seismic criteria for global slope stability are not applied to embankments and cut slopes that do not support structures. WSDOT does not mitigate instabilities associated with these cuts and fills due to the high cost of applying such a design policy uniformly to all slopes statewide.

For global stability under seismic loading, structures must be designed to a criterion of "no collapse." For bridges, WSDOT applies this criterion by requiring the abutments and approach fills to have a seismic factor of safety of at least 1.1 and, where liquefaction is predicted under the design earthquake, to meet displacement criteria established by the structural designer.

3.0 FIELD EXPLORATION AND LABORATORY TESTING PROGRAMS

The 2007, 2008, 2009, and 2010 field exploration and laboratory testing programs were conducted under TOs BS (WBS PC-21.06.500.07), CJ (WBS I90PJT.1B.ENG.07), DG, and DX (WBS E09009B.PE.DS.07.01).

3.1 PREVIOUS INVESTIGATIONS

Prior to the 2006 geotechnical investigation, URS reviewed previous geological, geotechnical, and geophysical studies within the project area. URS and W&N supplemented this existing information with a 2006 field exploration program for combined Phase 1B and 1C that consisted of:

- Drilling 38 borings for bridges, walls, stormwater mitigation facilities, and embankments;
- Installing two OWs;
- Installing two inclinometers and two extensometers;
- Installing 15 deformation monitoring points (DMPs);
- Drilling 13 rock slope borings, including 10 at Slide Curve and three at the snowshed; and
- Preparing geologic maps.

The results of the 2006 investigation and the earlier investigations are summarized in the 2006 Conceptual Geotechnical Report (URS and W&N 2007). The subsequent 2007 field exploration program for combined Phase 1B and 1C consisted of:

- Drilling 87 borings for bridges, walls, snowshed, culverts, embankments, and stormwater mitigation facilities;
- Installing two OWs;
- Monitoring two inclinometers, two extensometers, and 15 DMPs at Slide Curve;
- Drilling 43 rock slope borings, including 25 at Slide Curve and 18 at the locations of other rock cuts; and
- Monitoring rock slope movement at Slide Curve (fissure investigation).

The results of the 2007 investigation are presented in the 2007 geotechnical deliverables (URS 2008a, W&N 2008, and W&N 2009c).

Geotechnical information from these earlier investigations and reports is incorporated into the geotechnical analyses and recommendations for the project elements that are presented in Sections 6 through 16 of this report.

3.2 2008-2010 SUBSURFACE EXPLORATION PROGRAM

3.2.1 Exploration Management

The field exploration part of the investigation was performed in three parts:

- Roadway geotechnical exploration conducted by URS and described in this 2008-2010 Roadway Geotechnical Engineering report;
- Rock slopes exploration conducted by W&N and described in the 2008 rock slope geotechnical engineering and material resource evaluation reports (W&N 2009a, W&N 2009b, Burk 2009); and
- Materials resource exploration conducted by Burk and described in the 2008 material resource evaluation report (Burk 2009).

The roadway field exploration was completed under the direction of URS key personnel as follows:

- Task Leader – Cecil Urlich, P.E. (2008-2010)
- Exploration Manager – Brian Hamilton, P.G. (2008) and Kranti Maturi (2009-2010)
- Field Safety Manager – Herb Klug (2008) and Ken Yang, P.E. (2009-2010)
- Geotechnical Engineers – Xiangdong Han, P.E., Ph.D. (2008), Abhijit Bathe, P.E. (2008-2009), Ken Yang, P.E. (2008-2010), Pam Craig, P.E. (2008-2009), and Dave Walker, P.E. (2008)
- Soil Classification – Xiangdong Han, P.E., Ph.D. (2008) and Martin McCabe, P.E., Ph.D. (2009-2010)

The rock slope field exploration was completed under the direction of F&S, W&N, Burk, and URS personnel as follows:

- Task Leader – Norm Norrish, P.E.
- Field Safety Manager – Herb Klug
- Exploration Manager – Balin Strickler, P.G.
- Field Engineering Geologists – Ken Yang, P.E., Ryan Beyer, and Dan Meier, L.G., L.E.G.
- Rock Classification – Robert Burk, Ph.D., L.G., L.E.G.

3.2.2 Scope of Roadway Field Exploration Program

The scope of the roadway geotechnical field exploration program was developed by URS in conjunction with WSDOT, including exploration plan review meetings on March 19 and August 6, 2008, and May 13, 2009. The Phase 1C and 1D field work was conducted by URS, the

WSDOT Field Exploration Unit (FEU), CRUX Subsurface Inc. (CRUX), and NGA and consisted of:

- Drilling 101 borings for the SD, bridges, walls, culverts, embankments, stormwater management facilities, and SSEs;
- Excavating 28 test pits;
- Installing five OWs;
- Monitoring eight OWs;
- Monitoring three inclinometers and 15 DMPs at Slide Curve; and
- Conducting geophysical surveys at SCB and a section of SCW (Wall 8).

The scope of the boring and test pit program is summarized in Table 3.1.

The drilling was conducted from May 6, 2008 to July 27, 2010. Of the 101 borings, 61 were drilled by the WSDOT FEU and 40 were drilled by CRUX. WSDOT FEU drilled 42 land borings and 19 lake borings for a total footage of 3,630 feet. CRUX drilled 39 land borings and one lake boring for a total footage of 1,326 feet.

At SCB, 15 borings were initially planned (SCB-001-08 through SCB-015-08). Borings SCB-003, -008, -012, and -013-08 were deleted and more optimal locations selected based on the incoming boring and geophysical data. Seventeen borings were subsequently added based on the new subsurface information (SCB-016-08 through SCB-32-10).

Borings SCW-013 and -014-08, which were planned to explore lake sediment conditions for evaluation of the stability of SCW, were canceled when it was determined the wall could be founded on bedrock.

Summaries of the 2008-2010 borings and test pits are presented in Tables 3.2 and 3.3, respectively. The geotechnical conditions encountered are described in Section 4. The boring and test pit logs are provided in Appendix A.

In the boring designation, the first three letters indicate the structure (for example, SSD for the snowshed), the three digits between the hyphens (001 to 100) indicate the boring sequence, and the two digits at the end of the boring designation indicate the year of drilling (for example, 08 for 2008). Borings with groundwater observation wells were denoted with (OW) at the end of the boring number.

At several of the SCW locations, the borings are designated SCW-00XA-08. The letter A indicates the boring was drilled close to an existing boring. For example, boring SCW-006A-08 was drilled 19 feet from boring SCW-006-08. This was done to provide boring information closer to the wall face.

Fifteen test pits were excavated for stormwater mitigation facilities and one test pit each was excavated for SSD, SCB, and TCC. The test pit designations for the latter three pits have a three-letter prefix (SSD, SCB, or TCC) that indicates the structure. The test pits for the stormwater

mitigation facilities at HMF and the Crystal Springs Sno-Park have the three-letter prefix HMF and INF, respectively. The designations for the remaining stormwater mitigation facilities pits do not have a site-specific prefix.

Boring depths for the roadway borings were selected based on the type of structure and its dimensions at the boring location. The termination depth was generally determined by URS in collaboration with WSDOT. For retaining walls, if bedrock was not encountered, the boring depth was generally 2.5 times the height of the retaining wall if the material was found to be competent and no soft material or fine sands were encountered. If bedrock was encountered, the boring was terminated after coring 15 feet into bedrock.

For bridge and SSD foundations, if bedrock was not encountered, the boring was advanced through at least 20 feet of material that had standard penetration test (SPT) blow counts greater than 30 (N > 30). If bedrock was encountered, borings were terminated after coring a minimum of 15 feet into bedrock.

For culverts, the termination depth for borings was approximately 30 to 60 feet.

3.2.3 Coordination and Logistics

General locations of the roadway borings were selected by URS in association with the WSDOT Project office, geotechnical division, and FEU staff. WSDOT field-checked the proposed locations, verified access, and obtained the necessary United States Forest Service (USFS) and USBR permits. The final proposed boring locations were determined in the field and staked by WSDOT and URS.

URS field engineers prepared and submitted daily field reports and key photographs to Mr. Urlich, Mr. Hamilton, and Ms. Maturi. Weekly progress reports were prepared by Mr. Hamilton and Ms. Maturi and submitted to WSDOT.

URS geotechnical engineers monitored the roadway drilling activities and performed the following tasks:

- Attended work safety meetings each morning before drilling;
- Confirmed boring locations and photographed them with the drill rig set up;
- Collected, labeled, photographed, and retained the soil samples and rock cores and measured the rock quality designation (RQD) values.
- Documented the drilling program with focus on soil and rock types, and recorded strata changes and the SPT blow-counts;
- Conducted torvane and pocket penetrometer field strength tests on cohesive soil samples;
- Classified soil and rock in accordance with the WSDOT soil and rock classification system described in WSDOT Geotechnical Design Manual M 46-03, dated December 2006;
- Prepared logs of the soil, rock, and groundwater conditions encountered; and

- Prepared daily field reports of exploration activities.

3.2.4 Drilling, Coring, and Sampling Methods

The roadway borings were drilled using the wet rotary method. The drilling fluids were disposed of in a pit at the drilling location that was backfilled with soil at the completion of the boring. The boreholes were supported with steel casing. The sampling procedure consisted of advancing the casing to the desired depth, inserting the sampling device, and collecting the sample from below the bottom of the casing. SPT samples were generally taken at five-foot depth intervals in general accordance with American Society of Testing and Materials (ASTM) D-1586.

The split spoon sampler was driven using the SPT procedure (140-pound hammer falling from a height of 30 inches). The number of hammer blows for each 6 inches of sampler penetration was recorded. The number of blows required to advance the sampler the one-foot interval between 6 and 18 inches of penetration is known as the SPT N-value or blow count, and is a commonly used measure for selecting soil strength parameters.

HQ/NQ core barrels were used between SPT samples, as necessary, where the drilling behavior indicated the boring was being advanced through cobbles, boulders, or bedrock. HQ/NQ core barrels were also used to core bedrock. The core and SPT soil samples were laid out at the site and logged by the field engineers.

The total depth of each boring is shown in Table 3.2. Groundwater OWs, which are discussed in Section 3.2.6, were installed in five boreholes. At locations where instrumentation was not installed, the boreholes were backfilled with bentonite chips.

The soil samples, including samples with cobbles and boulders, were placed in labeled plastic bags and sent to the URS geotechnical laboratory in Seattle for classification checking and laboratory testing. The rock cores were placed in core boxes, and the core boxes were temporarily stored beneath a tarpaulin at HMF for further review and testing. At the end of the exploration program, the core boxes were transported by WSDOT to Tumwater for long-term storage. Photographs of the rock cores are provided in Appendix B.

3.2.5 Sampling Methods for Lake Sediments

Soft lake sediments were encountered in three barge borings (SW2-008-08, SW2-009-08, and SCB-015-08). The lake sediments exploration plan called for undisturbed sampling and in-situ vane shear testing where soft cohesive soils were encountered.

Boulders were encountered at the lake bed surface in SW2-008-08 and SW2-009-08. The presence of the boulders required FEU to use a core barrel to advance the boring. Soft cohesive samples were collected by pushing a 2-inch diameter thin-walled tube. In SCB-015-08, CRUX used a wireline-retrievable drill bit to advance the boring. Soft cohesive samples were collected using a piston sampler and a 3-inch diameter thin-walled tube.

Disturbed material was removed from the ends of the thin-wall tube samples. The tubes were field-cut at the end of the undisturbed material, and plastic caps were taped to the ends of the tubes to maintain sample integrity. The samples were transported to the WSDOT laboratory in Tumwater, where they were extruded from the tubes.

Vane shear testing was not conducted in SW2-008-08 and SW2-009-08 because the inside diameter of the casing used with the core barrel was too small to accept the vane shear apparatus. One vane shear test was attempted in SCB-015-08; however, the test was not valid because of the presence of gravel within the clay matrix.

3.2.6 Groundwater Observation Wells

Groundwater OWs were installed in three 2008-2010 Phase 1C roadway borings (RCB-002-08(OW), RCB-003-08(OW), and SSD-014-09(OW)) and two Phase 1D borings (INF-TP-010-10(OW) and INF-TP-012-10(OW)). OWs had previously been installed in H-24-06(OW), H-38-06(OW), CUL-015-07(OW), and SW2-002-07 (OW). The OWs were installed to collect information including depth to groundwater and seasonal changes in groundwater depth. The OWs were constructed of one-inch-diameter polyvinyl chloride (PVC) pipe with an above-ground protective steel casing with a locking cap.

Following the installation of the wells, groundwater levels were monitored once or twice a month by URS through August 2010 during periods when the wells were not covered by snow. Table 3.4 presents a summary of the Phase 1C and 1D groundwater OWs, and Table 3.5 summarizes the groundwater levels measured through August 2010. Figures 3.1 through 3.5 show the measured groundwater levels from 2006 through 2010 graphically, together with the lake levels, for wells H-24-06(OW), H-38-06(OW), CUL-015-07(OW), RCB-002-08(OW), and RCB-003-08(OW).

SW2-002-07(OW) was destroyed sometime between the 2007 and 2008 monitoring seasons. SSD-014-09(OW) was destroyed during Phase 1B construction activities some time prior to June 2, 2010.

3.2.7 Test Pits

A total of 28 test pits were excavated, including:

- One pit at the proposed location of the temporary SNW (Wall 4) at SSD;
- One pit at the proposed location of the permanent SNW (Wall 7) at SCB;
- One pit at TCC;
- Two pits at the site of a proposed infiltration facility near LW 1470+00;
- Six pits at the offsite stormwater mitigation facility at HMF;
- Six pits at the offsite stormwater mitigation facility at the Crystal Springs Sno-Park; and
- Eleven pits at potential offsite stormwater mitigation facilities west of HMF.

The pits were excavated by WSDOT Hyak maintenance personnel and logged by a URS geologist or engineer. The test pit locations, depths, and dates of excavation are summarized in Table 3.3. Logs of the pits are provided in Appendix A. Photographs of selected representative test pits are provided in Appendix B.

3.2.8 Slide Curve Geotechnical Instrumentation

The following instrumentation has been installed at Slide Curve to monitor embankment movement:

- Fifteen deformation monitoring points (DMPs), which were installed by White Shield, Inc. on July 5, 2006 at 100-foot intervals between approximately LE 1400+10 and 1414+10 on the EB shoulder. The DMPs were used to monitor vertical deformations in the area where pavement had recently been replaced. The DMPs have been monitored from August 2006 to September 2010. The maximum displacement that has been measured is about 0.8 inch downward displacement near LE 1408+00.

- Four inclinometers, (H-1-05(I), H-27-06(I), H-29-06(I), and SW2-006-08(I)). A fifth inclinometer (H-1-97(I)) was installed by WSDOT; however, no readings were taken after 1997 because the inclinometer did not appear to be functioning properly. SW2-006-08(I) was installed about 65 feet upslope of H-1-05(I) by WSDOT during the 2008 geotechnical investigation to a depth of 76 feet. H-27-06(I) and H-29-06(I) were damaged during snow removal activities. They were subsequently repaired, and a baseline reading was taken in June 2009.

- Two extensometers, H-27-06(E) and H-29-06(E), each located approximately five feet from its companion inclinometer, H-27-06(I) and H-29-06(I), respectively. H-27-06(E) was monitored from August 15, 2006 to September 19, 2007, and H-29-06(E) was monitored from July 11, 2006 to September 19, 2007. Neither extensometer showed significant displacement during the monitoring period. They were destroyed in late 2007 during snow removal operations and were not replaced.

- One groundwater OW, (SW2-002-07(OW)). This well was monitored from July 9 to December 12, 2007. It was destroyed during snow removal operations during the winter of 2007-2008 and was not replaced.

The instrumentation locations are shown in Figure 3.6. Currently the DMPs are being monitored by White Shield, and the inclinometers are being monitored by WSDOT. The DMP data are shown graphically in Figure 3.7. Plots of the inclinometer data are provided in Appendix D. The findings from the measurements are discussed in Section 9.4.

Additional information on instrumentation installed for the project, including groundwater OWs, inclinometers, extensometers, and DMPs, is provided in the Instrumentation Plan (URS 2008g).

3.2.9 Boring and Test Pit Surveys

White Shield surveyed the as-drilled locations and elevations of the borings and test pits using a real-time kinematic global positioning system (RTK GPS) with an accuracy of 0.05 feet. White Shield surveyed the ground elevations and, where applicable, the elevations of the top of the outer protective metal casing and the top of the inner PVC casing.

The as-drilled locations of the ten lake borings were surveyed by WSDOT and one lake boring was surveyed by CRUX using a GPS. The water depth was measured using a weighted tape. The mudline elevations of the borings were calculated based on the measured water depths and lake water surface elevations obtained from the USBR Hydromet website.

To confirm the USBR data, White Shield surveyed the lake surface level by GPS on August 3, 2007, and recorded the date and time. The USBR elevation was found to be 0.66 feet higher than the survey elevation. Therefore, URS subtracted 0.66 feet from the USBR-derived mudline elevations to estimate the actual mudline elevations.

3.2.10 Boring and Test Pit Logs

Logs of the 2008 borings and test pits and photographs of the rock cores are presented in Appendix A. A key to the soil and rock classification that explains the terms, abbreviations, and symbols used on the logs precedes the logs in Appendix A. Logs of previous borings are presented in the 2007 Roadway Geotechnical Report (URS 2008a) and the 2006 Conceptual Geotechnical Report (URS and W&N 2007).

The visual descriptions of the SPT samples do not indicate the presence of cobbles and boulders. Where the SPT sampler was used, drilling behavior and high SPT blow counts were used to interpret the presence of cobbles and boulders. Where the core barrel was used in granular soils and where the sample recovery was low, it is possible or even probable that finer-grained material was washed away by the drilling fluid.

The boring logs in part contain URS interpretations of field conditions. The stratigraphic contacts shown on the logs represent approximate boundaries between soil and rock types; actual boundaries may differ.

3.2.11 Geophysical Surveys

Geophysical surveys were conducted at SCB and SCW to better define the depth to bedrock along the bridge and wall alignments. CRUX Oriented Borehole Logging (COBL®) was conducted in selected borings at SSD and SCB to help characterize the bedrock structural fabric. The geophysical survey reports and the COBL® logs are provided in Appendix C.

Slide Curve Bridge and Walls. URS conducted a seismic refraction survey between LE 1371+50 and 1385+30 at SCB. The location of the line was determined by on-site URS project representatives. The seismic line was to follow the alignment of the lake side edge of the bridge and the western and eastern MSE wall faces. However, based on safety concerns associated with the unstable surface materials along the steep slope, partial submersion of the wall alignment under Lake Keechelus at the time of the survey, and poor coupling of the geophones to the blocky fill surface materials, the seismic line was relocated to the right shoulder of the existing EB I-90. The relocated survey line closely follows the proposed SNW (Wall 7) alignment.

The field investigation was completed between April 8 and April 10, 2008, by representatives of the URS Geophysics Group, based in Gaithersburg, Maryland. The seismic refraction survey was conducted using a Geometrics 24 Channel Geode seismograph, with 24 8-Hz P-wave geophones. The geophones were placed along the seismic lines at uniform increments of 10 feet. The seismic pulse was generated by firing a percussion Betsy® Seisgun source at designated shot point locations. At several shot points, impact of a 10-pound sledgehammer against a metal plate was used instead of or in combination with the Seisgun to provide the seismic source. Each seismic spread contained seven shot points. The survey consisted of six adjoining 230 feet long spreads for a total of 1,380 linear feet of surveying.

The results of the geophysical investigation are described in Section 8. A detailed description of the investigation is presented in Appendix C.

Slide Curve Median Wall. A seismic refraction survey was conducted along the wall face between approximately LW 1403+00 and 1409+00 by NGA. The geophysical survey report is provided in Appendix C.

The geophysical survey report also includes results of a survey on the rock slopes east of SSD between LW 1363+50 and 1369+00. The purpose of this survey was to characterize to top of bedrock along the proposed top of cut line for the rock cuts in this interval.

Borehole Logging. Optical and acoustical borehole logging was conducted in seven SSD borings and six SCB borings by CRUX using the COBL® system. The purpose of the logging was to characterize fractures in rock, including frequency, size, and orientation (strike and dip). The boreholes that were logged include:

- SSD-006-09 through SSD-012-09
- SCB-024-09 and SCB-026-09 through SCB-030-09

The COBL® logs are provided in Appendix C.

3.3 FIELD AND LABORATORY TESTING PROGRAM

URS conducted field point load tests (PLTs) on rock samples and soil index tests on selected samples collected from the borings. In addition, WSDOT conducted consolidation and strength tests and associated index tests on selected cohesive lake sediment samples and unconfined compressive strength tests on rock specimens in the WSDOT laboratory. The field and laboratory testing was used to evaluate engineering properties for use in geotechnical analysis and structural design.

The laboratory testing was selected to complement testing conducted during the 2006, 2007, and earlier geotechnical investigations. This section describes the inspection and classification procedures, soil index tests, soil consolidation and strength tests, and rock strength tests conducted on samples collected from the 2008-2010 borings.

3.3.1 Inspection and Classification

Representative portions of the soil samples that were collected during drilling were delivered to the URS soils laboratory, where they were stored for classification checking and testing. The field classification of each sample was reviewed in general accordance with the procedures outlined in the GDM. The field classifications were modified as necessary.

3.3.2 Index Tests

Grain size distribution, Atterberg limits, organic content, and resistivity tests of selected soil samples were conducted in general accordance with the procedures outlined in ASTM test methods D-422, D-4318, D-2974, and G57-06, respectively. The scope of the soil index property testing program included:

- Grain size distribution: 178 tests;
- Atterberg limits: 12 tests;
- Organic content: one test;
- Resistivity and pH: two tests; and
- Water content: 173 tests.

Resistivity and pH tests were conducted by Norton Corrosion Limited.

A summary of the laboratory index testing results is provided in Table 3.6. Laboratory data sheets are provided in Appendix D.

3.3.3 Soil Strength and Consolidation Tests

Consolidated-Undrained Triaxial Compression and Consolidation Tests. Consolidated-undrained triaxial compression (CU) and consolidation tests were conducted by WSDOT on four samples of cohesive lake sediments. The following CU tests were performed:

- two specimens from SW2-008-08 sample U-5;
- three specimens from SW2-009-08 sample S-7;
- three specimens from SCB-015-08 sample S-3; and
- one specimen from SCB-015-08 sample S-5.

Consolidation tests were performed on a single specimen from each of the samples listed above. The strength and consolidation testing results are summarized in Table 3.7. The CU test results for the SCB-015-08 samples were not usable, as described below.

The WSDOT laboratory provided the CU test results to URS in a reduced or partially reduced form, which included cell pressure, effective consolidation stress prior to shear, axial deformation, load and pore pressure during shear, calculated numerical results for stress and strain, and plotted results for stress and strain (Mohr and p-q plots). The WSDOT lab did not provide test results for SCB-015-08 sample S-5. URS found inconsistencies in the CU test results, and this information was transmitted to the laboratory. Revised test results were subsequently provided to URS.

URS reduced the revised CU data and generated p-q plots and p'-q' plots, where the p and q values were taken as those corresponding to the maximum deviator stress. The linear trend line function in Microsoft Excel (least squares best fit) was used to plot the straight line which best fit the data points. These plots are presented in Appendix D. In some cases, it was found that the line intercepted the q-axis at a negative value. Where this occurred, the results were reinterpreted using the linear trend line function, with the additional requirement that the line pass through the origin.

Based on the p-q and p'-q' plots, the values of the total and effective friction angles (φ and φ') and the ratio of the undrained shear strength to the effective overburden pressure (S_u/p') shown in Table 3.7 were calculated and interpreted.

There were some problems associated with the test data for the SCB-015-08 and meaningful results could not be obtained. Constant cell pressure was not maintained in one of the specimens from sample S-3. This yielded the total and effective p-q lines that plotted on top of each other. The data points for the other two specimens were so close together that a meaningful trend line could not be interpreted. Therefore, these test results were not considered useable.

Laboratory data sheets are provided in Appendix D.

Ring Shear Test. A ring shear torsion test was conducted by URS on a sample of clay infilling obtained from rock core sample C-20 from SCB-022-09. The purpose of the test was to measure the undisturbed and residual shear strengths of a representative sample of the bedrock infilling. The results of the ring shear torsion test are summarized in Table 3.8. Laboratory data sheets are provided in Appendix D.

3.3.4 Rock Strength Tests

Unconfined compressive testing was conducted on 84 samples by WSDOT. The results are summarized in Table 3.9

PLTs were performed on 312 rock specimens. The rock specimens were tested in the field using a Geotechnical Consulting and Testing Systems Point Load Tester (Model #PLT-110) in accordance with the procedures in ASTM D 573-02.

The uniaxial compressive strength was estimated based on ASTM test procedure correlations. The results are summarized in Table 3.10. The PLT is considered an index test; therefore the rock strength classifications shown in the boring logs were selected primarily on the basis of RQD and drilling and visual characteristics observed by the field geologist.

3.3.5 In-Situ Pressuremeter Testing

In-situ pressuremeter tests were conducted in three borings at SSD by In-Situ Engineering Inc. (In-Situ). The purpose of the testing was to measure rock elastic properties and, in weak rock, to estimate the unconfined compressive strength.

The In-Situ report, including tabulated results of the testing, is presented in Appendix D.

4.0 SITE CONDITIONS

This section presents an overview of the site geology, soil and rock stratigraphy and engineering properties, the occurrence of boulders, and groundwater conditions. Cross sections and profiles showing the subsurface conditions also are described. Geologic hazards are described in the 2007 Roadway Geotechnical Report (URS 2008a). The results of scour analyses are presented in the Hydraulic Report (Otak 2008a).

More detailed information on site conditions at the locations of structures, embankment fills, and infiltration facilities, including geologic hazards, as appropriate, is presented in the subsequent sections that address these elements. Subsurface profiles for the EB and WB lanes from the beginning of Phase 1C at LE 1351+21 (MP 58.0) to AE 120+97 (MP 59.9) are shown in Figures 4.1 and 4.2, respectively. Symbols, abbreviations, and general notes used in the profiles are provided in Figure 2.1.

4.1 SITE GEOLOGY

This section presents an overview of the geologic setting and the nature and distribution of the surface geologic units in the project area.. This information is based on geologic investigations and mapping conducted for the 2006 geotechnical investigation and was updated based on information gathered during the 2007 geotechnical investigation. A description of the site geology, including geologic maps, is presented in Volume 1 of the 2006 Conceptual Geotechnical Report (URS and W&N 2007). Mapping and analysis of the Slide Curve and Jenkins Knob areas are presented in Volumes 3 through 5 of the 2006 Conceptual Geotechnical Report. In addition, structural geologic mapping investigations of the existing rock cut slopes are presented in a report by Golder (2005) and Chadbourne and Moses (1994). A geologic map of the project area is provided in Figure 4.3.

4.1.1 Geologic Setting

The project is located in the Cascade Range on the western flank of Keechelus Ridge, east of Snoqualmie Pass. The steep shoreline to Keechelus Lake bounds the Project area to the west. The elevation of the area mapped for this Project ranges from about El. 2400 to El. 3800.

The project area geology consists of volcanic and sedimentary rocks of the Ohanapecosh and Naches Formations, which are Oligocene to Eocene in age (Tabor, et al. 2000). The southern area is underlain by the tuff member of Keechelus Lake of the Ohanapecosh Formation and, to a lesser extent, andesite and basalt of the Ohanapecosh Formation. The tuff member of Keechelus Lake is a dacite crystal-vitric tuff and tuff breccia. The northern area is underlain by sedimentary and volcanic rocks (andesite and/or basalt) of the Naches Formation. Late Pleistocene alpine glacial deposits, and post-glacial Holocene alluvium, colluvium, landslide, and lacustrine deposits of Keechelus Lake are locally present.

The overall geologic structure of Keechelus Ridge is a northwest trending anticline with its axis approximately coincident with the ridge crest. An east side down fault is inferred to underlie the unconsolidated deposits of the Gold Creek drainage and northern portion of the lake. It is mapped as being truncated by an Oligocene to Miocene age pluton and the tuff member of lake on the south, and thus does not appear to have been active since the Eocene-Oligocene.

4.1.2 Bedrock Units

The predominant rock unit in the area of the Project is the Ohanapecosh Formation. Four distinct rock types were identified:

- Tuff member of Keechelus Lake (Tolk)
- Lapilli tuff (Tolkw)
- Andesite (Toa)
- Basalt (Tob)

The tuff member of Keechelus Lake (Tolk) is exposed almost continuously in the existing highway cut slope from approximately Sta. 1378+00 to 1410+00 and in the former quarry to the east of Sta. 1410+00. It also outcrops locally on the upper parts of the slope above the highway and along the ridge crest. This rock principally consists of welded dacite crystal-vitric lapilli tuff and tuff breccia.

Non-welded lapilli tuff (Tolkw) is present on the north side of Slide Curve between Sta. 1391+00 and 1396+00 and on the north side of the snowshed. This unit appears to underlie the stronger, less weathered vitric tuff; however, in the existing cut slope it is typically juxtaposed against blocks of the vitric tuff along faults.

Slightly to moderately weathered, moderately strong to strong andesite (Toa) is present between about Stas. 1333+00 and 1378+00. Moderately strong to strong basalt (Tob) is present between Stas.1317+50 and 1333+00.

Minor outcrops of Naches Formation sedimentary and volcanic rocks (Tns) and Mount Catherine Rhyolite are present in the northern portion of the map area outside the area of planned cuts. Therefore, these rock units should not be encountered except in foundation borings north-northwest of Wolfe Creek.

4.1.3 Unconsolidated Deposits

Unconsolidated deposits exposed in the map area include:

- Alpine glacial till (Qag)
- Alluvium (Qa)
- Lacustrine deposits (Ql)
- Colluvium (Qc)
- Landslide debris (Qls)
- Fill (Af)
- Talus (Qt)

Alpine glacial till (Qag), probably from the Vashon stage of the Fraser Glaciation, is exposed in the existing highway cut from north of the snowshed (Sta. 1354+00) to near Sta. 1348+00, and is present on the east side of the highway for most of the area from Sta. 1300+00 to 1317+50 and Sta. 1244+00 to 1285+00.

It consists of predominantly rounded cobbles and small boulders in a matrix of reddish yellow to light brown sand and gravel, typically with a veneer of colluvium. Till is also present locally on

the crest of the ridge capping the bedrock and may exist on some of the upper slopes that are too steep to readily traverse.

Alluvium (Qa) is a mixture of sand, gravel, and cobbles from Gold, Rocky Run, Wolfe, and Resort Creeks that cross the highway. Gold and Resort Creek alluvial deposits are within active creek channels and in deltas exposed in the lake during low water levels. Some Resort Creek alluvium accumulates upstream of the highway in a man-modified retention basin that is a pond during high lake levels. Rocky Run and Wolfe Creek alluvial deposits are in the channels and alluvial fans at the creek mouths.

These creeks also have small deltas in the lake visible at low lake levels. Till, peat, siltstone, and clay beds, interpreted as pre-Vashon non-glacier deposits, are exposed underlying the alluvium and lacustrine deposits near Wolfe Creek.

Lacustrine deposits (Ql) are inter-fingered with an overlay of alluvium (Qa) within the high water boundary of the lake. The lacustrine deposits are primarily comprised of reworked alluvium with silt and clay.

Colluvium (Qc) that is exposed along the existing cut slope consists of soils similar to the alpine glacial (Qag) till but is distinguished primarily by the presence of more sub-angular to angular rock fragments. A veneer of colluvium primarily derived from in place weathering of the rock is also present on the upper slopes and crest of the ridge above the highway. Evidence of soil creep, primarily widespread bent tree trunks, was observed on the steep colluvium covered slopes. Mapping of the glacial and colluvial materials was not performed outside the area of potential impact to the proposed Project or on the steepest slopes.

Landslide debris (Qls) consists of consolidated deposits of primarily colluvium and other mass waste likely derived from down-slope movement. At Slide Curve (Sta. 1396+00 to 1406+50) and the quarry, the material is primarily rock material and rockfill placed during prior work. Rock materials in the quarry area are in the artificial fill/modified land (af/ml) map unit and not delineated separately. Non-rock deposits are a cobble and gravel mixture in silt and sand. Areas mapped as landslides include a head scarp and avalanche chutes, mostly underlain by rock.

Several landslide toes extend to the road; others are on steep slopes above the rock cut and are not readily accessible. The mapped landslides are mostly inferred from aerial photographs and topography. Road cuts could intersect mapped landslides and avalanche chutes from Sta. 1392+00 to 1395+00, 1381+00 to 1381+50, 1374+00 to 1379+00, 1349+00 to 1363+00 and 1341+00 to 1345+00. The primary failure mode appears to be translational with some shallow rotational.

Artificial fill (af) material was placed along most of the project alignment during construction of the existing highway. Most of the exposed fill on the slope below the highway consists of angular to subangular, cobble-to boulder-sized rock generated from the rock cuts. This material was placed directly on the rock by side-casting onto the existing slope to the lake.

Locally, the fill also includes logs, tree stumps, guardrails, trash, rebar, and Portland cement and asphaltic concrete slabs from the highway reconstruction. The deposits currently mapped as fill may also include talus, which are rock fragments accumulated on or at the base of a slope mainly due to gravity.

Talus (Qt) below the highway is similar in nature to the rockfill but may be slightly more weathered and subrounded depending on the age and transport distance and mechanism. A small area of talus was also observed on the upper slope north of the Slide Curve landslide and sub-map scale accumulations are also commonly present at the base of the existing cut slopes.

4.2 STRATIGRAPHY

This section presents an overview of soil and rock units encountered within the Phase 1C area. Site-specific descriptions of the soil units encountered at each structure location are presented in Sections 6 through 15 and in the site-specific technical memoranda (URS 2008b, 2008c, 2008d, 2008e, 2008f).

The soils have been grouped in soil units for geotechnical analyses. The soil units are based primarily on engineering properties and soil classification and, in general, reflect depositional environments as well. Soils were classified using the criteria presented in the GDM.

The stratigraphy and properties of the lake sediments offshore of SCB (URS 2008c), SCW, and SW2 (URS 2008d) have been updated based on the results of lake borings drilled offshore of these locations. The lake sediment stratigraphy and properties were previously based on geophysical surveys (Shannon & Wilson (S&W) 1974). The geophysical surveys identified three soil units called Recent Lake Sediments, Older Lake Sediments, and Morainal Deposits and Glacial Outwash.

The 2008 lake borings encountered Clay/Silt and Sand/Gravel soil units and bedrock. The Sand/Gravel and bedrock were encountered at higher elevations than indicated by the S&W geophysical surveys. The use of the designations Recent Lake Sediments, Older Lake Sediments, and Morainal Deposits and Glacial Outwash has been discontinued.

Soils within the project area have been grouped into seven soil units and one rock unit for geotechnical analysis. These units are described below, generally in order of increasing depth.

- Gravel with Sand/Cobbles/Boulders (Fill)
- Gravel with Sand/Cobbles/Boulders/Rock Fragments (Primarily Colluvium)
- Gravel with Sand/Cobbles (Native Soil)
- Fine Sand and Silt
- Clayey Silt/Silty Clay
- Clay/Silt (formerly Recent Lake Sediments/Older Lake Sediments)
- Sand/Gravel (formerly Older Lake Sediments/ Morainal Deposits and Glacial Outwash)
- Bedrock

The **Gravel with Sand/Cobbles/Boulders (Fill)** unit consists of medium dense to very dense, poorly- to well-graded gravel with coarse to medium sand or with cobbles and boulders. This material is fill placed for construction of the existing road embankment. In areas where the embankment was constructed on a relatively steep slope, such as from the existing snowshed to Slide Curve, the fill thickness is typically greatest beneath the lake side edge of the roadway. The interpreted embankment fill thickness ranges up to 60 feet.

The **Gravel with Sand/Cobbles/Boulders/Rock Fragments (Primarily Colluvium)** unit is a mixture of colluvium, landslide deposits, and glacial deposits. The material consists of a range of particle sizes from silt to boulders. This unit was encountered discontinuously between the west end of Phase 1C and LE 1409+00, including the SSD, SCB, and SW2 locations. Where the unit was encountered, it was found to generally extend continuously from the base of the existing embankment fill to bedrock. Bedrock is typically deeper in areas where this unit is present compared to areas where it is not present.

Beneath the SSD and SCB locations, the interpreted thickness ranges up to 60 feet. At SW2, the interpreted thickness ranges up to 100 feet. Where this unit was encountered, the fine sand and silt and the gravel with sand/cobbles (native soil) units were not encountered.

Historical photographs of the existing snowshed construction show excavation of boulders up to several feet in size, which were placed on the embankment slope. Woody material up to 32 inches thick was encountered in one boring.

In some zones, only small amounts of silt and sand are present. Many zones are characterized by relatively large void spaces that have been loosely infilled, and 100 percent loss of drilling fluid was frequently experienced. SPT blow counts were highly variable, ranging from 4 to greater than 105 blows per foot, with the highest likely caused by cobbles and boulders and the lowest likely occurring in voids between large material that had been infilled with loose soil.

The **Gravel with Sand/Cobbles (Native Soil)** unit consists of medium dense to very dense gravel and sand with cobbles and localized boulders. The unit was encountered from approximately LE 1416+00 to the east end of Phase 1C (AE 120+97). The unit was encountered both above and below the fine sand and silt unit described below between approximately LE 1419+00 and 1430+00. Although they were deposited at different times, the geotechnical engineering properties of these upper and lower subunits differ very little.

The thickness ranges up to approximately 70 feet east of RCB. Where borings were advanced to bedrock, this unit was found to lie directly on top of the bedrock.

The **Fine Sand and Silt** unit generally consists of medium dense silty fine sand, sand, well-graded silty sand with gravel, and sandy silt and medium stiff to stiff silt. The density ranges from loose to dense. Locally the unit contains significant amounts of clay. Much of this soil unit is susceptible to liquefaction. The soil tends to become finer-grained and more plastic with increasing depth.

This unit was encountered beneath the Resort Creek drainage between approximately LE 1419+00 and 1430+00, and is interpreted to pinch out between LE 1430+00 and AE 108+00. It also occurs discontinuously around the east end of Phase 1C at AE 120+97. The maximum thickness encountered was approximately 65 feet near RCB. The thickness generally increases in the lakeward direction.

The **Clayey Silt/Silty Clay** unit consists of localized deposits of stiff to hard clayey silt, silty clay and clay with gravel. This unit includes areas of deposits that have been heavily overconsolidated by glacial action. These overconsolidated deposits were encountered directly on top of bedrock.

The **Clay/Silt** unit consists of very soft to soft, fine-grained lake sediments. In the borings drilled offshore of SCB and SW2, coarse-grained inclusions up to boulder size were locally present in the clay/silt matrix. Measured plasticity indices ranged from 6 to 42.

This unit has been encountered in four borings drilled in the lake.

- H-4-74 drilled offshore of SSD: the unit was encountered at El. 2219 and was present to the total depth of the boring (105 feet; El. 2114);
- SCB-015-08 drilled offshore of SCB: the unit was encountered at El. 2238 and was present to a depth of 33 feet (El. 2205);
- SW2-008-08 drilled offshore of SW2: the unit was encountered at El. 2337 and was present to a depth of 32 feet (El. 2312); and
- SW2-009-08 drilled offshore of SW2: the unit was encountered at El. 2323 and was present to a depth of 56 feet (El. 2284).

In SCB-015-08, SW2-008-08, and SW2-009-08, boulders were encountered in the clay/silt matrix from the surface to a depth of several feet.

The **Sand/Gravel** unit consists of loose to dense sand, sandy gravel, and silty sand. This unit has been encountered in two borings drilled in the lake:

- SW2-008-08 from El. 2312 to the bottom of the boring at El. 2220 and
- SW2-009-08 from El. 2284 to El. 2229.

Bedrock generally consists of fine-grained volcanic rock, including andesite, lapilli tuff, tuff breccia, and basalt. The bedrock surface is typically steeply sloping, and the depth to bedrock varies widely. In areas where the alignment runs parallel to the mountain side, the bedrock surface slopes down toward the lake. The depth to bedrock is generally greater beneath creek valleys and former erosional areas. Beneath the Resort Creek valley, the bedrock is as low as approximately El. 2420, and beneath buried erosional features beneath SSD and SCB, the bedrock is as low as approximately El. 2450 and 2430, respectively.

Based on the results of PLTs, the rock strength ranges from very weak (R1) to very strong (R5). The measured values of uniaxial unconfined compressive strength ranged from 1.2 to 56.1 kips per square inch (ksi). Unconfined compressive strengths measured using unconfined compression and point load testing are summarized in Tables 3.8 and 3.9, respectively.

More detailed descriptions of the rock strength and structural characteristics are provided in subsequent sections for the individual structures.

4.3 BOULDERS

The presence of boulders can affect the types of foundations and construction methods that can be successfully used, in particular where deep foundations are planned (SSD, SCB, and RCB). Boulders can also affect excavations for shallow foundations.

Within the area explored, boulders are most common at the SSD and SCB areas. Boulders were also encountered at SCW, RCW, RCB, TCC, and the MP 59.7 Unnamed Creek Crossing. Boulders are most commonly found in fill used to construct the existing embankments and in the mixed colluvium, talus, and glacial deposits present at SSD, SCB, and SW2. A summary of the numbers of borings where boulders were encountered and the maximum size encountered (where available) is presented in Table 4.1.

There is some uncertainty in the frequency of boulders in the SSD area. The majority of the borings were drilled in 1977 and earlier. These logs often identify boulders without a specific depth or size. Terms such as "rock fragments of all sizes" and "talus" are commonly used. It was also common to advance borings by blasting, rather than by drilling or coring, and the information in the logs is sparser where blasting was used. Logs of borings in the SSD area are presented in TM3 (URS 2008b).

Where sizes were recorded on the logs of borings in the SSD area, the boulders generally ranged in size up to about two feet, with one 6-foot boulder encountered. Historical photographs of the snowshed construction show excavation of boulders larger than 12 feet in size, which were placed on the embankment slope. Woody material was also encountered, and ranged in size up to 32 inches thick in one boring. Boulders to 34 inches in size were encountered in the test pit (SSD-TP-01-08) excavated for the SSD soil nail wall. During Phase 1B construction, boulders up to 12 feet in length were excavated from the cut slope north of the roadway in the vicinity of LW 1347+00.

In addition to boulders, rubble in the form of blocks of asphalt concrete pavement and Portland cement concrete pavement are present on the embankment slopes. Six areas**,** up to 300 feet long each on the embankment slopes from LE 1200+00 to the east end of Phase 1C at AE 120+97, contain asphalt concrete pavement or Portland cement concrete pavement rubble.

4.4 SOIL AND ROCK ENGINEERING PROPERTIES

Soil and rock engineering properties were evaluated using the procedures outlined in GDM Section 5 and LRFD Section 10.4.

4.4.1 Soil Properties

Soil properties were selected based primarily on SPT N-values, grain size analysis, visual classification, drilling behavior, Atterberg Limits, and water content. Properties for the various soil units, including total unit weight, effective friction angle, and cohesion, are summarized in Table 4.2.

For sands, the effective friction angles (φ) were selected from GDM Table 5-1, which relates φ to $N1_{60}$ (the SPT blow count corrected for both overburden pressure and hammer efficiency effects). The blow-counts obtained using an automatic trip hammer, which is 80 percent efficient, were corrected to the standard N-values based on a 60 percent efficient hammer. For the fine sand and silt stratum, the lower end of the range shown in Table 5-1 was used because of the high percentage of fine-grained particles. For predominately gravelly soils, where SPT testing is not reliable, the friction angle was selected from AASHTO LRFD Figure 10.4.6.2.4-1 (AASHTO 2007).

Properties of the cohesive lake sediments were interpreted from the results of consolidation and CU tests and from back-analysis of "known" stability conditions. The CU test results contained some inconsistencies, as described in Section 3.3. As result, the strength properties of the cohesive lake sediments were interpreted primarily using back-analysis of the stability conditions in the Slide Curve area at LE 1407+69, where the existing embankment movement indicates a factor of safety for global stability under static loading of 1.0. This back-analysis is described in Section 9.

The residual shear strength of liquefied soil used for estimating downdrag loads on drilled shafts and evaluating post-earthquake global stability is described in Section 5.4.

4.4.2 Rock Properties

The rock mass strength properties were estimated from the uniaxial unconfined compressive strength of intact rock specimens (q_{ui}) and the rock mass rating (RMR). The rock mass strength properties were used to estimate the tip resistance of drilled shafts, the rock mass unconfined compressive strength (q_u) used in lateral load analysis of drilled shafts, and the bearing resistance of shallow foundations. Side resistance for drilled shafts was estimated using q_{ui} and fracturing effects. The methods used to estimate tip and side resistance for drilled shafts are presented in Section 5.3.

Rock strength was estimated at each of the locations where the structure is founded on rock, and recommended rock strengths to use for design are presented in the individual structure sections of this report. Additional description of the methods used to estimate rock strength is presented in the 2007 Roadway Geotechnical Report (URS 2008a).

4.5 GROUNDWATER CONDITIONS

Groundwater levels have been monitored in nine OWs, with multiple measurements of groundwater levels above the bottom of the well from five of those wells. The locations of the wells are shown in the boring location maps (Figure 2.2); the OWs are indicated by the suffix (OW). Measured groundwater levels are summarized in Table 3.5. The temporal variations of the measured groundwater levels in the five wells with multiple measurements are depicted graphically in Figures 3.1 through 3.5. The groundwater monitoring program is described in Section 3.2.

Because of the presence of snow during the winter and spring, groundwater levels were measured from the beginning of summer through the end of fall. At some well locations, groundwater levels could not be measured throughout this period because the groundwater level dropped below the bottom of the well.

Groundwater levels are influenced by the lake level, water levels in nearby streams, and local and regional groundwater table elevations. In general, groundwater levels are higher during the summer months and lower during the fall months.

Measured groundwater levels in wells placed in the embankment shoulder next to the lake were generally equal to the lake level when the lake level was above the elevation of the embankment toe. Where wells were placed in free-draining material adjacent to a high embankment slope (for example, at SSD, SCB, and SW2), the available data indicate the groundwater level is approximately equal to the lake level year-round.

Groundwater levels in wells placed in the median area or the shoulder opposite the lake were typically higher than and either less affected or not affected by the lake level. The groundwater levels in these wells, as well as in wells placed in the lake side shoulder of relatively short embankment slopes, tended to stabilize during mid to late summer and remain relatively constant throughout the fall.

4.6 SUBSURFACE CROSS SECTIONS AND PROFILES

A total of 23 subsurface cross sections and nine subsurface profiles have been developed for Phase 1C. The cross sections and profiles use the alignments that were current as of March 2010. The methods used to develop the cross sections and profiles are described in the 2007 Roadway Geotechnical Report (URS 2008a). Cross sections were developed at the following locations:

- SSD: 10 cross sections at approximately 100-foot intervals;
- SCB: five cross sections, including one at each abutment or pier location;
- SW2: three cross sections at selected critical or representative locations;
- SCW: five cross sections at selected critical locations;
- RCW: two cross sections at selected representative locations;
- RCB: two cross sections, including one at each abutment location;
- Culverts: two cross sections, consisting of one each at TCC and the MP59.7 Unnamed Creek crossing; and
- Embankments: one cross section at a selected representative location.

All of these cross sections are presented in the individual structure sections of this report.

EB and WB profiles were developed for Phase 1C along the LE and LW lines, respectively. Additional profiles were developed along wall and pier alignments, including:

- SSD Pier 1 and Pier 2 alignments;
- SCB: four profiles, including one along the south edge of the bridge and three along the SNW (Walls 7, 7A, and 7B) alignments;
- SCW (Wall 8) alignment;
- SW2: south edge of EB lane; and
- RCW alignment.

The EB and WB profiles are shown in Figures 4.1 and 4.2, respectively. The cross sections and structure profiles are provided in the subsequent report sections that present the geotechnical recommendations for each structure.

For bridges, the cross sections were developed at the abutment and pier locations. For walls and embankments, cross sections were developed at locations interpreted to be critical for global stability and in consideration of the following factors:

- Wall or embankment height;
- Existing slope height and inclination;
- Presence of nearby borings; and
- Interpreted subsurface conditions.

A total of 21 Phase 1C cross sections were prepared as part of the 2007 geotechnical analyses and reporting program. The lake sediment stratigraphy for all cross sections from LE 1379+90 to 1411+75 was updated based on new information from the lake borings drilled as part of the 2008 field exploration program. Lake sediment stratigraphy was collected at three cross section locations during the 2008 field exploration program:

- SCB 4-4' at LE 1379+90;
- SW2 1-1' at LE 1406+26; and
- SW2 2-2'at LE 1407+69.

Stratigraphic information from these borings was used together with topographic information to interpret and extrapolate the lake sediment stratigraphy for the other cross sections from LE 1379+90 to 1411+75. Because the information was extrapolated over large distances, there is a relatively high degree of uncertainty in the interpreted lake sediment stratigraphy shown in the cross sections.

Geophysical surveys conducted at SCB and at SCW between LW 1403+00 and 1409+00 were used to aid interpretation of bedrock depths in cross sections and profiles for these areas. Reports describing these surveys are provided in Appendix C.

In many areas, the available data was not sufficient to interpret the stratigraphy with reasonable certainty. Where the stratigraphy was interpreted from sparse data, the stratigraphic breaks are marked with a question mark ("?") to indicate significant uncertainty.

5.0 GEOTECHNICAL ANALYSIS METHODS

Geotechnical analyses were conducted following the procedures presented or referenced in the GDM, including the AASHTO LRFD Bridge Design Specifications (AASHTO 2008).

The 2007 Roadway Geotechnical Report (URS 2008a) describes methods and assumptions used to implement or supplement the procedures presented in the GDM and LRFD specifications, including:

- Earthquake engineering;
- Global slope stability analyses;
- Bearing resistance of drilled shafts in rock;
- Bearing resistance of shallow foundations; and
- Wedge analysis of lateral loads imposed on foundation elements by soil that is unstable during design earthquake ground motions.

This section updates and supplements that report. Procedures described in this section include:

- Section 5.1 presents a discussion of site seismicity and provides parameters for structure modeling. This information was previously presented in URS 2008a, but is included here for convenience.
- Section 5.2 presents methods and assumptions used for global stability analysis.
- Section 5.3 described methods used to calculate bearing resistance for rock sockets.
- Section 5.4 describes the selection of residual strength parameters for liquefiable soil for use in downdrag estimates and stability analyses.
- Section 5.5 discusses lateral load analysis of shafts subject to seismic loading.
- Section 5.6 describes methods used to calculate soil displacement due to lateral spread.

Many of the updates to the global stability analysis methods, selection of residual strength parameters, and lateral loads analyses result from updates to Section 6 (Seismic Design) of the GDM, design memoranda from the Bridge and Structures Office, and clarifications provided to URS by WSDOT.

Soil and rock engineering properties that were used in the analyses are described in Section 4.2.2.

5.1 EARTHQUAKE ENGINEERING

This section presents a description of site seismicity, parameters for use in seismic modeling of the project structures, and methods used to assess the effects of flow failures and lateral spread.

Procedures used for assessing liquefaction potential are described in the 2007 Roadway Geotechnical Report.

5.1.1 Seismicity

All project structures have been classified as non-critical structures and will be designed for an earthquake having a 7 percent probability of exceedance (PE) in 75 years (an approximately 975-year recurrence interval, or nominal 1,000-year earthquake). This design earthquake is also used for embankments that support structures and cut slopes that could affect structures if a stability failure occurred. The GDM requires that all non-critical structures be designed for no-collapse based on the design earthquake.

For embankments and culverts less than 20 feet wide, the acceptable risk level is reduced to a 10 percent PE in 50 years (an approximately 475-year recurrence interval, or nominal 500-year earthquake). It is WSDOT policy that seismic instability associated with fills that do not support and would not impact an adjacent structure are not mitigated due to the high cost of applying such a design policy uniformly to all slopes statewide (GDM 6.1.2.1).

Ground motion parameters, including the peak ground acceleration (PGA) for Site Class B rock and the site class amplification factor, were obtained from the software Seismic Design Parameters for 2007 AASHTO Seismic Design Guidelines (Leyendecker, et al. 2007), which is based on Seismic Hazard Maps produced by the USGS. For the project location, the PGA for Site Class B bedrock is 0.28g.

The nominal 1,000-year earthquake magnitude was obtained from the USGS website "2002 Interactive Deaggregations" (USGS 2008). The mean magnitude obtained is equal to 6.34.

5.1.2 Structure Modeling

It is recommended that the general AASHTO seismic procedure be used to define the PGA and acceleration response spectrum. In accordance with AASHTO, for the Project location (47.4 degrees north latitude, 121.4 degrees west longitude), the PGA, 0.2 second spectral acceleration (S_s), and 1.0 second spectral acceleration (S_1) for Class B rock are as follows:

- PGA (Class B rock) = 0.28g
- S_s = 0.638g
- S_1 = 0.208g

Following the procedures in the AASHTO manual, it is recommended that Site Class D be used for the project structures. The structural designer will need to determine the site coefficients for the peak ground acceleration (F_{pga}), short-period range (F_a), and the long-period range (F_v) at each structure using the information above and the Ground Motion Tool in the AASHTO Guide Specification for LRFD Seismic Bridge Design.

The PGA for Class B rock recommended above does not include amplification or damping due to the site soils. In order to assess liquefaction potential, seismic earth pressures, and global stability, the PGA for Class B rock needs to be adjusted for the site soil conditions. It is recommended that Site Class D and an acceleration of 0.35g be used for the assessing liquefaction potential, seismic earth pressures, and global stability.

5.2 GLOBAL SLOPE STABILITY ANALYSIS

This section describes the procedures and assumptions used to evaluate the global stability of permanent structures. Supplemental procedures and assumptions used for individual structures are described in the individual sections, as needed.

Global slope stability was analyzed using the software SLOPE/W by GEO-SLOPE International, Ltd. The program is a two-dimensional model based on the method of limit equilibrium. Driving forces and resisting forces are calculated per foot length of slope using an effective stress analysis. Trial circular and block failure surfaces were simulated using Spencer's Method. In general, URS used the entry and exit mode in SLOPE/W for circular failure surfaces to limit the geometries of the failure surfaces to those considered realistic and relevant. Very shallow, surficial failure surfaces were generally not considered relevant.

SLOPE/W was also used to calculate factors of safety and estimate dimensions needed for mitigation measures for potential slope instability at RCB and RCW. To estimate the magnitude of lateral resistive forces needed from structural elements to achieve global stability under seismic loading at SSD and SCB, URS used a simple wedge analysis, as described in Appendix I of the 2007 Roadway Geotechnical Report.

5.2.1 Minimum Factors of Safety

Factors of safety for global stability under static and seismic loading were calculated using SLOPE/W. Minimum factors of safety are discussed in Section 2.4.

The potential need for mitigation measures was also evaluated where liquefiable soils were present and the factor of safety under seismic loading was greater than 1.1. In this case, the potential soil deformations could impose lateral loads that exceed the capacity of the shafts. This evaluation is described in Section 5.6.

5.2.2 Soil Parameters

The static (fully drained, no excess pore pressures) soil parameters used for analysis of global stability are presented in Section 4.3 and Table 4.2. The strength properties of the cohesive lake sediments were interpreted primarily using back-analysis of the stability conditions at LE 1407+69, where the existing embankment movement indicates a factor of safety for global stability under static loading of 1.0. This back-analysis is described in Section 9. Selection of residual shear strengths used for analysis of stability under seismic loading is described in Section 5.4.

5.2.3 Surcharges

For live load surcharge under static loading, a traffic load surcharge equal to 250 pounds per square foot (psf) and a snow load surcharge equal to 320 psf were considered. It was assumed that these loads would not occur concurrently. For static conditions, the larger snow load surcharge was used.

A traffic load surcharge of 125 psf was included under the seismic loading case. This surcharge is equal to the maximum traffic load of 250 psf multiplied by a load factor of 0.5. The load factor is consistent with Section 3.4.1 of the AASHTO LRFD Bridge Design Specifications (AASHTO

2007). Due to the low probability of the design earthquake occurring concurrently with the maximum snow load, a snow load was not included for seismic conditions.

5.2.4 Stability Under Seismic Loading

For seismic loading, the nominal 1,000-year return interval earthquake was used in the analyses. Global stability under seismic loading was analyzed using a pseudostatic method and, if liquefaction was predicted, a residual strength method. The pseudostatic method was used to represent conditions during shaking. The residual strength method was used to represent conditions after shaking had ceased.

Because long duration earthquakes are not expected in the project area, WSDOT has adopted the approach of assuming for the purpose of calculating global stability factors of safety that no excess pore pressures, and the accompanying strength degradation, will occur during ground shaking. Strength degradation during shaking is considered in calculating potential soil displacements, as described in Section 5.6.

For the pseudostatic method, a horizontal inertial force was applied, which was computed using a horizontal acceleration coefficient (k_h) equal to one-half of the PGA. For the reason described above, fully drained, undegraded (static) shear strength properties were assumed for the analysis.

Stability under seismic loading with potentially liquefiable soils present was evaluated at RCW and RCB. Where liquefaction was predicted, the residual strength analysis assumed that seismic ground motion had ceased. The residual strength method used $k_h = 0$ (that is, no horizontal inertial force) and the residual shear strength of the liquefied soil. The residual shear strength was estimated using the approaches presented in the 2008 GDM seismic design updates, as described in Section 5.4.

5.3 DRILLED SHAFT ROCK SOCKET BEARING RESISTANCE

Drilled shafts will be socketed into rock (rock sockets) at SSD, SCB, and RCB. Drilled shaft rock socket bearing resistances for shaft tip and skin (side) resistance were estimated as a function of rock socket diameter (B) and total depth (D) below the bedrock surface, where D is measured from the downslope edge of the shaft.

The estimates used the layer-average intact rock q_{ui} and RMR estimates for the rock layers with the equations and guidance from AASHTO (2007) Section 10.8.3.5.4 *Estimates of Drilled Shaft Resistance in Rock*. For SSD, where 44 shafts are planned, a reliability analysis was conducted to estimate the layer-average q_{ui}. The reliability analysis is described in Appendix F of this report. At SCB, q_{ui} and RMR were estimated at each pier shaft location. At RCB, q_{ui} and RMR were estimated at each abutment.

The AASHTO methodology is largely based on the 1999 report FHWA-IF-99-025, *Drilled Shafts: Construction Procedures and Design Methods,* by O'Neill and Reese (O'Neill and Reese 1999). This report was used to supplement the AASHTO guidance.

5.3.1 Tip Resistance

Unfactored shaft tip resistance per square foot of shaft tip area (q_p) was calculated for each rock layer using the AASHTO-FHWA methodology with the corresponding layer-average q_{ui} and

RMR estimates. The RMR estimates were used to select corresponding m and s parameters from AASHTO 2007 Table 10.4.6.4-4 as input to AASHTO Equation 10.8.3.5.4c-2 (FHWA 1999, Method 1, Equation B.15), assuming that the rock below the base of the shaft to a depth of 2.0B is jointed and the orientation of the joints is random.

The AASHTO-FHWA procedures use RMR with m and s parameters based on Hoek and Brown (1988). URS knows of no update to the shaft design procedures for Hoek, et al. 2002. In applying the Hoek-Brown (1988) method, it was assumed the rock mass was not disturbed by drilling.

The tip resistance estimates were sensitive to RMR, via corresponding parameters m and s, only at the threshold between the RMR interval corresponding to "poor rock, class No. IV" (RMR = 40 to 21), the RMR interval corresponding to "fair rock, class No. III" (RMR = 41 to 60), and for RMR>60, corresponding to "good rock, class No. II." URS considered the RMR estimates to be rough approximations that do not support excessive refinement. Therefore, a range of credible RMR estimates were compatible with (and would not have changed) the shaft design parameters for tip resistance calculated using the cited method.

Because the resulting tip resistance is a lower bound solution for bearing resistance, the unfactored estimate was doubled so that the factored tip resistance would not be less than the lower-bound estimate. The tip resistance estimates assumed a clean hole, with the shaft in direct contact with the rock surface.

5.3.2 Side Resistance

Unfactored ultimate (maximum) shaft side resistance per square foot of shaft circumferential area (q_s) was calculated for each rock layer using the layer-average q_{ui} as input to AASHTO Equation 10.8.3.5.4b. The estimate for rock fracturing effects was $\alpha_E = 0.5$, based on interpretation of Table 10.8.3.5.4b-1.

For combined skin friction and end bearing, the residual rock strength was used to estimate the side resistance component. Residual shaft side resistance was calculated for each rock layer using the layer-average q_{ui} as input to FHWA (1999) Equations C.33 and C.34 (p.C-28).

The maximum and residual side resistance estimates assumed smooth rock socket sides or that the rock has been drilled using drilling slurry. As AASHTO states, significant additional shaft resistance may be achieved if the borehole is specified to be artificially roughened by grooving (AASHTO 2007 C10.8.3.5.4b). FHWA (1999) Appendix B provides further details (p.B-40 to B-42).

For the service limit state, a portion of the ultimate skin friction in the overlying soil can be assumed to contribute to the total bearing resistance of shafts embedded in rock. Based on an estimated strain on the order of 0.1 percent (0.1 inches) to mobilize full end bearing in rock, it was estimated that on the order of 20 percent of the ultimate skin friction in soil would be mobilized, based on the information presented in AASHTO Fig. 10.8.2.2.2-3.

Under earthquake loading, side resistance in soil is neglected where the shafts are used to stabilize upslope soils (SSD and SCB). Side resistance is also neglected where soils are expected to liquefy under the design earthquake.

5.4 RESIDUAL STRENGTH PARAMETERS AND DOWNDRAG FOR LIQUEFIABLE SOILS

The residual strength properties of potentially liquefiable soils were evaluated for use in analyzing global slope stability and estimating potential downdrag loads on shafts using the updated guidance presented in GDM Sections 6.4.2.5 and 6.5.3.

Global stability under seismic loading was analyzed using two methods to characterize residual strengths: Kramer (2008) and Olson and Stark (2002). For the Kramer method, the 40 percent probability of non-exceedance values were used, which are more conservative (lower) strengths than would be estimated using the 50 percent probability of non-exceedance (or median) values. The Kramer method predicts residual shear strengths that have a non-linear variation with depth (effective overburden pressure), particularly at shallow depths (low effective overburden pressures). This non-linear strength was approximated as a linear variation of strength with depth for stability analyses.

The Olson and Stark method predict residual strengths that increase in direct proportion to the effective overburden pressure. For the Olson and Stark method, the average values were selected for use in stability analyses.

The Kramer method was used for estimating downdrag loads. The 60 percent probability of non-exceedance values were used, which are higher strengths than would be estimated using the median values and result in more conservative (higher) estimated downdrag loads. The non-linear strength was also approximated as a linear variation of strength with depth for estimating downdrag loads.

5.5 LATERAL LOAD ANALYSES FOR SEISMIC LOADING

URS developed supplemental design information for lateral load analyses of the foundations of three structures (SSD, SCB, and RCB) that do not meet global stability criteria under seismic loading. Liquefiable soils are present at RCB and are not present at SSD and SCB. The methods used to develop lateral load analysis input for RCB are described in Section 11.

Lateral earth loads on shafts from upslope foundation soils and surcharges subjected to seismic loading at SSD and SCB were estimated using a wedge analysis. The wedge analysis uses a pseudostatic force equilibrium approach, with the Mononobe-Okabe analysis (Mononobe 1929; Okabe 1926) used to estimate earth pressures on the shafts and resistance provided by the downslope soil during an earthquake. The loads and resistances calculated using the wedge analysis can be used to calculate shears and moments in the shaft at the bedrock surface. These shears and moments can be used as input to lateral load analyses of the rock sockets using DFSAP.

The wedge analysis was described in Appendix I of the 2007 Roadway Geotechnical Engineering Report.

5.6 LATERAL SPREAD DISPLACEMENTS

Potential embankment (free-field) displacements were estimated for bridge abutments underlain by foundation soils that were potentially liquefiable under design earthquake loading. Potentially

liquefiable soils ("affected soils") were assumed to be at their residual undrained shear strength (S_r); non-liquefied soil were assumed to have fully drained shear strength. Methods used to estimate residual strength are described in Section 5.4.

Lateral spread displacement analyses were conducted for embankments having residual factors of safety (rFS) equal to greater than one (rFS≥1.0). Embankments having rFS<1.0 were not analyzed for displacement but assumed to have free-field displacements exceeding three feet. Embankments having rFS<1.0 ordinarily require ground stabilization to limit ground displacement where structures are threatened with potential collapse.

For cases with rFS>1.0, free-field lateral displacements were first calculated assuming no ground improvement. Where ground improvement was considered, the ground improvement was designed to achieve calculated lateral displacements specified by the bridge designer as tolerable under a "no collapse" performance criterion. The following discussion briefly documents the general calculation procedure.

5.6.1 Calculated Free-Field Embankment Displacements Without Ground Improvement

Free-field displacements were calculated by first estimating the yield acceleration (k_y) for the embankment configuration using SLOPE/W. The estimated k_y assumed conditions consistent with post-liquefied soil at its undrained residual strength and non-liquefied soil at fully drained conditions.

Actual yield accelerations are time-dependent. For a given embankment, the yield acceleration is highest at the start of ground shaking, then generally decreases during the time evolution of the ground shaking as excess pore water pressures in affected soils increase, causing the effective soil shear strength to decrease. Yield accelerations reach minimum values as the affected soil liquefies or otherwise experiences maximum excess pore pressures and minimum effective strength. Yield accelerations generally increase thereafter as affected soils experience shear strain and dilate, causing excess pore pressures to decrease as the soil reaches its post-liquefaction undrained residual strength (S_r).

Therefore, because the analysis assumed residual strengths in affected soils, the calculated k_y generally underestimated the time-dependent yield acceleration during that period of earthquake ground shaking when soil pore pressures are increasing and effective strength is decreasing but still greater than the post-liquefaction undrained residual strength (which may be roughly consistent with effective strength excess pore pressures ratios, r_u, in the range of 0.5 to 0.8).

Nominal slope displacements (D) were calculated using the calculated yield acceleration k_y with the Bray and Travasarou procedure (Bray and Travasarou 2007), which also required estimates of the initial fundamental period of the sliding mass (T_s), and the spectral acceleration of the input ground motion at a period of 1.5 times T_s.

Displacements calculated using the Bray and Travasarou procedure are probabilistic. URS reported median estimates, which have a conditional nominal non-exceedance probability of 50%. Displacement estimates that are twice the median estimate have a conditional nominal non-exceedance probability of about 84% (median plus one standard deviation "sigma") according to the Bray and Travasarou procedure.

The resulting displacement estimates were considered to have a somewhat higher than 50% conditional non-exceedance probability because the Bray and Travasarou procedure assumes that the yield acceleration is constant over the duration of the earthquake ground shaking, whereas the true time-dependent yield acceleration is likely to exceed the calculated k_y during all but the later period of ground shaking, as discussed above.

It has been URS' experience with more sophisticated effective-stress time-history analyses that displacements calculated assuming a constant k_y and residual strengths appear, as a rough first approximation, to be about twice the displacements calculated from the full time history of ground shaking and effective strength. This could indicate that the simplified median displacement estimates, discussed above, are closer to an 84% non-exceedance level.

However, because of the numerous theoretical and practical simplifications and approximations required for the displacement analyses, as briefly summarized here, there is significantly more model and parameter uncertainty in the estimates than reflected in the Bray and Travasarou procedure. Thus, though a more refined estimate of the non-exceedance probability would presumably indicate less than 84% (nominal median plus one sigma) and higher than 50% (nominal median), calculated seismic displacements should be viewed appropriately as only an *index* of seismic performance.

Very simply, while URS reported the calculated median slope displacements, calculated displacements should be interpreted as only an imprecise index of seismic performance, subject to significant uncertainty.

5.6.2 Calculated Free-Field Embankment Displacements With Ground Improvement: Compaction Grouting

Free field displacements in situations where ground improvement by compaction grouting has been considered (e.g., at RCB) were analyzed as follows. First, the residual factor of safety rFS was calculated using SLOPE/W assuming the treated zone had a shear strength of 7,200 psf based on an assumed unconfined compressive strength of 1,000 pounds per square inch (psi) for grout columns having an assumed 10% areal or volume replacement in the treated zone. Soil strength had a relatively trivial affect in the treated zone compared to the assumed grout strength and was therefore neglected. Liquefiable soils outside of the treatment zone were assigned their post-liquefaction residual strength, S_r.

The location and width of the grouted zone was varied until a minimum rFS=1.1 was calculated. The associated yield acceleration k_y was then calculated, also using SLOPE/W. Using the calculated k_y, the nominal, median slope displacement was calculated by the Bray and Travasarou procedure. If the calculated displacement exceeded a specified criterion (e.g., one foot of displacement), the width of the treated zone was increased until the displacement criterion was met.

An "unofficial" check calculation of the final, treated configuration was also made assuming that the grout columns lost their effectiveness during earthquake loading (due to subsurface displacements and resulting column discontinuities) or could not otherwise deliver the assumed stabilizing effect of an average 7,200-psf shear strength in the treated zone. For this analysis, the compaction grout treatment was assumed to manifest as an increased resistance to liquefaction through soil densification (i.e., increased $N1_{60}$) so that the treatment zone did not liquefy and had a maximum nominal excess pore pressure ratio of 0.50. No other effect from the grouting was

assumed. The yield acceleration and associated slope displacement were then calculated and evaluated. If the calculated yield acceleration and displacement appeared arguably compatible with the calculated displacement assuming the 7,200-psf shear strength (and no densification effect), the treated zone based on the grouted shear strength assumption was considered acceptable.

6.0 SNOWSHED

Geotechnical design recommendations for the snowshed replacement (SSD) were previously provided in TM3 (URS 2008b). This section presents supplemental geotechnical analyses and recommendations for design of SSD, including:

- Seven additional subsurface cross sections and updates to the three existing sections and the subsurface profiles along Pier 1 and Pier 2 (Section 6.3);
- Updated rock strength parameters (Section 6.3);
- An evaluation of the global stability of SSD for possible failure surfaces through bedrock (Section 6.4);
- Supplemental recommendations for Pier 1, including shallow foundations and the pier wall (Section 6.5);
- Updated recommendations for drilled shaft foundations (Section 6.6);
- Supplemental recommendations for Pier 2 permanent ground anchors (Section 6.7);
- Supplemental recommendations for the Pier 2 geosynthetic wall (Wall 3; Section 6.8)
- Recommendations for the design of the Pier 2 temporary SNW (Wall 4; Section 6.9);
- A discussion of the west portal MSE walls (Walls 9A, 9B, and 9C) and counterfort wall (Wall 9D) (Section 6.10);
- The results of global stability analyses and recommendations for design of Wall 1 (Section 6.11);
- The results of global stability analyses for the east portal wall (Wall 10; Section 6.12);
- Recommendations for design of the transformer building (Section 6.13); and
- Construction considerations (Section 6.14).

Lateral earth pressures for design of a concrete vault that will contain a 30,000-gallon water tank are provided in Appendix K. Information regarding the design of the SSD foundations was presented in computer-aided teleconferences on August 4, 17, and 28, September 23, and October 19, 2009 and May 12, July 13, August 24, 2010.

6.1 PROJECT DESCRIPTION

SSD will be constructed between LW 1352+50 and 1363+50 and will cover both the EB and WB lanes. The existing snowshed is located between LW 1354+45 and 1359+45 and covers only the WB lanes. An aerial photograph of the proposed site and existing snowshed is shown in Figure 6.1. A site plan is provided in Figure 6.2.

Piers 1 and 2 are located on the mountain side and lake side of SSD, respectively. The preliminary design for Pier 1 consists of a tied-back cast-in-place concrete wall founded on shallow foundations. The preliminary design for Pier 2 consists of columns that are spaced 25 feet center-to-center and supported on drilled shafts. The shafts will be tied back using permanent ground anchors installed through the individual shaft caps.

The EB lanes will be shifted as much as 40 feet south toward the lake and supported on a permanent geosynthetic wall (Wall 3). To provide access for construction of the drilled shafts and MSE wall, an access bench will be excavated into the existing embankment slope. To maintain two lanes of traffic in each direction during construction, it will be necessary to oversteepen the cut into the embankment slope. A temporary SNW (Wall 4) will support the oversteepened cut.

Additional walls planned for the SSD vicinity include Walls 1, 9, and 10, which are described in Sections 6.10, 6.11, and 6.12, respectively.

Selected preliminary design drawings are provided in Appendix G.

6.2 FIELD AND LABORATORY INVESTIGATIONS

Subsurface conditions have been explored at the SSD site by drilling 51 borings and excavating one test pit. The explorations are described in TM3 (2007 and earlier borings) and Section 3 (2008-2010 borings and test pit). Boring and test pit information is summarized in Table 6.1. The locations of the borings and test pit are shown in Figure 6.2.

During the 2008-2010 field exploration program, 14 borings (SSD-006-09 through SSD-019-09) were drilled and one test pit (SSD-TP-001-08) was excavated. Seven borings, SSD-006-09 through SSD-012-09, were drilled to explore soil and rock conditions at Pier 2. Eight borings, SSD-013-09 through SSD-019-09, were drilled to characterize soil and rock conditions at Pier 1 and the adjacent Walls 1 and 9. SSD-TP-001-08 was excavated to characterize soil conditions at the temporary SNW (Wall 4). The logs of the borings and test pit excavated in 2008-2010 are provided in Appendix A. The logs of the earlier borings are provided in TM3.

Laboratory and field testing consisted of 25 unconfined compression and 125 PLTs on selected rock specimens. The unconfined compression test and PLT results are summarized in Tables 6.2 and 6.3, respectively.

Pressuremeter testing was conducted by In-Situ from May 31 through June 10, 2009 in three test borings: SSD-006-09, SSD-007-09, and SSD-008-09. A total of 22 tests were conducted. The results are summarized in Table 6.4. The report prepared by In-Situ is presented in Appendix D.

6.3 SUBSURFACE CONDITIONS

Ten subsurface cross sections showing the interpreted subsurface conditions have been developed at SSD. The cross sections are presented in Figures 6.3 through 6.12. Profiles of the interpreted subsurface conditions along Pier 1 and Pier 2 are shown in Figures 6.13 and 6.14, respectively. Profiles of the interpreted subsurface conditions along the LE and LW lines are shown in Figures 4.1A and 4.2A, respectively. The LE line is approximately 17 feet south of the proposed SNW (Wall 4) alignment.

This section summarizes the soil and rock conditions encountered at the SSD site. The soil conditions encountered in the test pit, which was excavated near the proposed SNW alignment, are described in Section 6.9.1.

6.3.1 Soil and Rock Conditions

Two soil units and one rock unit have been identified in the vicinity of SSD and are described as follows.

Unit 1 (Gravel with Sand/Cobbles/Boulders [Fill]) consists of fill placed for construction of the road embankment. Unit 1 consists of medium dense to very dense, poorly- to well-graded gravel with coarse to medium sand or with cobbles and boulders up to six feet in size. Historical photos of the snowshed construction show boulders in excess of 12 feet in size, which were placed on the embankment slope. The thickness of this soil unit ranges up to approximately 20 feet.

Unit 2 (Gravel with Sand/Cobbles/Boulders/Rock Fragments [Primarily Native Soil]) is a mixture of colluvium, landslide deposits, and glacial deposits. The thickness of this unit ranges from approximately 0 to 5 feet along the Pier 1 alignment and from 0 to 56 feet along the Pier 2 alignment. At Pier 2, it is thickest near the center of the proposed pier alignment and thinnest or absent at the west end of the alignment. The material consists of a range of particle sizes from silt to boulders.

Boulders are distributed throughout Unit 2. Boulder sizes generally were not recorded on the logs of borings drilled prior to 1998. Where sizes were recorded, the boulders generally ranged up to about two feet in size, with one six foot boulder encountered. Woody material was occasionally encountered, with a maximum thickness of 32 inches in one boring.

In some zones, only small amounts of silt and sand are present. Many zones are characterized by relatively large void spaces that have been loosely infilled, and 100 percent loss of drilling fluid was frequently experienced. SPT blow counts were highly variable, ranging from 4 to greater than 105 blows per foot, with the highest blow counts likely caused by cobbles and boulders and the lowest blow counts likely occurring where voids between large material had been infilled with loose soil.

Unit 3 (Bedrock) consists of very weak (R1) to very strong (R5) lapilli tuff. Thin-section analyses of rock samples from the 2007 borings drilled along the Pier 1 alignment identified the rock as lapilli tuff. Logs of earlier borings drilled on the lake side of the Pier 1 alignment identify the rock as andesite.

The elevation of the top of bedrock is highly variable both parallel and perpendicular to the SSD alignment, and slopes downward toward the lake and toward the center of SSD. The interpreted bedrock elevations along Piers 1 and 2 are discussed in Section 6.3.2.

The rock strength was interpreted primarily from the results of unconfined compression tests. The rock strength measured in 25 unconfined compression tests ranged from 2.6 to 44.6 ksi. The average measured strength is 19.1 ksi and the standard deviation was 9.2 ksi. Rock strength measured in 125 PLTs ranged from 0.6 to 56.1 ksi. Rock strength is discussed further in Section 6.3.3.

6.3.2 Bedrock Elevations

URS updated the interpreted bedrock elevations presented in TM3 Tables 3 (Pier 1) and 5 (Pier 2) using the additional boring information obtained from the 2009 borings and the March 2010 alignment. The updated information, presented in the form of Pier 1 and 2 subsurface profiles, a Pier 2 bedrock contour map, and ten subsurface cross sections, was provided to WSDOT on May 28, 2010. URS understands that WSDOT intends to have the contractor determine bedrock elevations at the Pier 2 shaft locations after award of the Phase 1C contract.

URS developed a bedrock contour map for the Pier 2 vicinity by triangulation between bedrock elevations at boring locations using computer-aided drafting and design (CADD) software. The contour map is provided in Figure 6.15. The bedrock elevations along Pier 2 are also shown in the subsurface profile, Figure 6.14.

Borings drilled in 1977 could not be used to develop the bedrock contours because inadequate information was available to accurately locate the borings. Corrected locations of other borings drilled in the 1970s were used in the reinterpretation of the bedrock elevations at Pier 2. These corrections were needed because the borings had previously been located using offsets from an incorrect reference line.

Because of the presence of rock cuts, linear interpolation between bedrock elevations measured in boreholes generally could not be used in the Pier 1 vicinity. The following methods were used to interpret the bedrock elevations beneath the Pier 1 centerline, which are shown in Figure 6.13.

- Three new borings (SSD-013-09, SSD-014-09, and SSD-15-09) and new cross sections at LW 1352+66 (SSD 4-4') and 1354+28 (SSD 5-5') were used to interpret the bedrock elevation beneath the Pier 1 alignment between the west end of SSD and the west end of the existing snowshed.

- Beneath the existing snowshed, it was assumed bedrock is present immediately beneath the planned elevations of the existing snowshed footings, as shown in the contract plans for the existing snowshed.

- East of the existing shed, bedrock either outcrops along the Pier 1 centerline or was interpreted to be directly beneath the existing roadway structural fill.

6.3.3 Rock Mass Properties

Rock mass structure and strength properties were evaluated using the boring logs, core photographs, COBL® logs, and strength testing results. The primary results of the evaluation were an assessment of persistent rock discontinuities (joints, faults, flow boundaries, etc.) and rock mass unconfined compressive strength. Rock properties encountered in 15 borings drilled in 2007 and 2009 provided the primary basis for the evaluation. The logs of borings drilled prior to 2007 contain limited rock information.

The eight borings drilled along the Pier 1 alignment in 2007 and 2009 included the information called for in GDM Section 4.3 (weathering state, relative rock strength, discontinuity spacing and condition, core recovery, RQD, and fracture frequency) and rock strength testing using the PLT. Laboratory unconfined compressive strength testing was conducted on selected rock samples from three borings drilled along Pier 1 in 2009.

The seven borings drilled along the Pier 2 alignment in 2009 included the information called for in GDM Section 4.3, PLT, in-situ acoustic and video borehole logging (COBL® logs), in-situ pressuremeter testing (PMT) at selected intervals in three borings, and laboratory unconfined compressive strength testing of selected rock samples.

The assessment of rock discontinuities was conducted by W&N under subcontract to URS. W&N performed a kinematic analysis and identified three discontinuity sets. The work conducted by W&N is documented in a TM that is included in this report in Appendix E. The implications of the results of the W&N study on the global stability of SSD are discussed in the TM and in Section 6.4 below.

Rock mass strength parameters used to develop geotechnical design recommendation in TM3 were based on limited strength data and were calculated using the procedure for the rock mass uniaxial compressive strength, σ_c, as presented in Hoek, et al. (2002).

WSDOT subsequently conducted an analysis of the rock strength properties and concluded in a memorandum that higher rock strength values could be justified for design of the shafts based on the available information (WSDOT 2009c; provided in Appendix E). In particular, WSDOT concluded that the global rock mass strength, σ'_{cm}, as presented in Hoek, et al. (2002), appears to be a more appropriate measure of rock mass strength than the rock mass uniaxial compressive strength.

URS reviewed the WSDOT memorandum and prepared a response memorandum that recommended additional borings and rock strength testing to increase the reliability that the interpreted rock strength values would be representative of the rock conditions at the actual shaft locations (URS 2009g; provided in Appendix E). Based on this recommendation, seven additional borings were advanced into bedrock along the Pier 2 alignment as part of the 2009 geotechnical exploration program. The boring and logging methods, optical and acoustical borehole logging, and in-situ, field, and laboratory rock strength testing are described in Section 3.

Rock mass strength parameters were updated using unconfined compressive strength testing results on rock specimens from the seven Pier 2 borings (SSD-006-09 through SSD-012-09) and further interpretation of the rock mass characteristics, as described in the URS response memorandum. The strength values were interpreted to have a log-normal distribution. URS used a reliability analysis to calculate the intact rock compressive strength and the global rock mass strength (σ'_{cm}) that correspond to a 50% probability that the actual values will be greater than the calculated values for the weakest of the 44 shaft locations. The values of the intact rock compressive strength and the global rock mass strength calculated in this manner were 6.8 and 1.1 ksi, respectively.

A detailed description of the methods used to calculate the intact rock compressive strength and the global rock mass strength is presented in Appendix F.

The results of pressuremeter testing conducted by In-Situ were found to provide limited information with respect to the ultimate rock mass strength properties. In-Situ did not estimate the compressive strength of strong rock. In weak rock with low RQD values such as the tests conducted in SSD-007-09, In-Situ opined in an email dated June 9, 2009 that the rock was

behaving like a granular material and the pressuremeter results may be representative of the rock mass strength.

6.3.4 Groundwater

Groundwater conditions near SSD are described in Section 4.4 of TM3. In TM3, it was concluded that the groundwater level within the existing embankment fill and underlying colluvium is approximately equal to the lake level when the lake level is above El. 2497, which is the elevation of the bottom of the nearest OW (H-24-06(OW)). Because of the coarse, highly permeable nature of the soil in the area, it is anticipated that the groundwater level is also approximately equal to the lake level when the lake level is below El. 2497.

Groundwater elevations in H-24-06(OW) have now been measured through August 2010. With the exception of one anomalous reading in May 2009, the more recent readings also indicate that the groundwater level is approximately equal to the lake level. Groundwater levels in H-24-06(OW) are shown in Figure 3.2.

SSD-014-09(OW) was also completed as a groundwater OW. It was destroyed during construction in spring 2010 before any groundwater measurements were taken.

WSDOT observed seepage conditions in the cut slope immediately west of SSD in late spring/early summer 2009 during expected peak snowmelt/runoff. The overburden was observed to be relatively dry. One exception was a zone of light seepage in the bottom quarter of the slope between approximately LW 1350+00 and 1352+00 (WSDOT 2009d).

6.4 GLOBAL STABILITY

This section describes the results of an evaluation of the global stability of SSD for potential failure surfaces through bedrock. The evaluation was performed by W&N under subcontract to URS. The results are presented in a TM prepared by W&N, which is included in Appendix E. Analyses of global stability for failure surfaces within soil only were previously provided in TM3.

W&N concluded that the potential for the presence of bedrock structure that would adversely affect the global stability of SSD was low. The following is an excerpt from the W&N report.

> "Comparison of the preferred structural orientations with the inferred orientation of the bedrock surface proximal to the proposed SSD indicates minimal potential for planar, wedge or circular modes of overall slope failure. The structural fabric is partially consistent with the prerequisites for a hybrid failure mechanism referred to as bi-planar. Failure by this mechanism is discounted on the basis of the imperfect structural geology and by the fact that persistent, low shear strength surfaces would be required rather than joint discontinuities. On a kinematic basis, therefore, it is concluded that the observed structural geology is not unfavorable for overall rock slope stability proximal to the snowshed."

W&N identified a bedrock discontinuity set that dips at approximately 40 degrees from horizontal. As described above, this discontinuity set would not compromise global stability. However, these data suggest that rock steeper than 40 degrees may be subject to local failure

along this discontinuity set at relatively small loads. It is noted that bedrock steeper than 40 degrees has not been measured to date, but could be encountered during construction.

Global stability evaluations for Walls 1, 4, 9, and 10 are described in Sections 6.10.2, 6.9.2, 6.11, and 6.12, respectively.

6.5 PIER 1 SHALLOW FOUNDATIONS AND RETAINING WALLS

6.5.1 Pier 1 Shallow Foundations

Due to the variability in bedrock excavation in the Pier 1 area, it may be necessary to place the footings on rock in some areas and on soil in other areas. Where the footing will be placed on soil, the thickness of soil should be at least two feet. Rock overexcavation may be needed in some areas to provide the minimum two feet. To accommodate the potential for differential settlement along the longitudinal axis of the pier, it is recommended that a construction joint be placed at the soil to rock transition. Structural fill should be compacted to 95 percent of maximum density.

In the transverse direction, the foundation should be placed entirely on bedrock or soil. As an alternative to rock excavation, the soil may be overexcavated to bedrock and a level concrete or CDF base may be constructed. If the slope of the bedrock exceeds 15 degrees from horizontal, the concrete or CDF base should be either keyed or doweled into the rock to provide sliding resistance.

Bedrock steeper than 40 degrees from horizontal is subject to failure along a discontinuity set at relatively small loads. Should bedrock steeper than 40 degrees from horizontal be present beneath the Pier 1 footings, it is recommended that passive reinforcement be placed that extends through the footing and into bedrock that is flatter than 40 degrees from horizontal. The bedrock inclination can be estimated using the bedrock depths encountered at the Pier 2 shaft excavations and the ground anchor and/or soil nail locations, as well as additional exploratory borings that may be drilled.

Spring parameters for shallow foundation design were estimated using the methodology presented in FEMA 356 - Prestandard and Commentary for the Seismic Rehabilitation of Buildings (FEMA 2000). Because the foundation will be placed directly on rock, the effective shear modulus ratio was selected based on Site Class B.

For foundations supported on the existing coarse-grained soils or properly compacted, suitable fill, foundation springs may be calculated using the following values:

Initial shear modulus, G_0 = 3,400 kips per square foot (ksf)

Effective shear modulus ratio, G/G_0 = 0.98

Poisson's ratio, μ = 0.35

For foundations bearing directly on either rock or a concrete or CDF based placed on rock, foundation springs may be calculated using the following values:

Initial shear modulus, G_0 = 110,000 kips per square foot (ksf)

Effective shear modulus ratio, $G/G_0 = 0.98$

Poisson's ratio, $\mu = 0.25$

6.5.2 Pier 1 Anchored Wall

URS understands that the Pier 1 wall will be tied back using permanent ground anchors bonded in rock. Lateral earth pressures for a cantilever wall were provided in TM3. This section provides recommendations for design of walls tied back by one or two anchors per vertical section, including lateral earth pressures, anchor inclinations, and no-load zones.

Unfactored lateral earth pressures for strength limit state design for footings on soil or rock are presented in Figure 6.16. Lateral earth pressures for extreme event limit state design for footings on soil or rock are unchanged from those presented in TM3, Figure 12. The backfill behind the wall must be fully drained to avoid water and ice pressure against the wall. Figure 6.16 also provides information for calculating hydrostatic pressure loads for a condition of incomplete drainage. Surcharge loads are described in TM3. The unfactored lateral earth pressures were previously provided to WSDOT on July 30, September 29, and October 26, 2010.

For walls that are able to deflect enough for active earth pressure to develop, the active earth pressure coefficient, K_a, may be used to calculate lateral earth load, ES, that results from the avalanche debris surcharge. Otherwise, the at-rest earth pressure coefficient, K_0, should be used to calculate ES.

For strength limit state design, URS recommends load factors equal to 1.35 and 1.5 for lateral earth load (EH) calculated using at-rest and active pressures, respectively. Load factors equal to 1.5 for the earth surcharge load (ES) and 1.0 for hydrostatic pressure (WA) are recommended. A resistance factor for passive earth pressure resistance equal to 0.5 is recommended.

Anchor design should include a no-load zone that extends a minimum of 15 feet from the back face of the Pier 1 wall or 5 feet into bedrock, whichever results in the longer no-load zone. For tremie-grouted anchors, the anchors should be inclined at a minimum of 10° below horizontal.

It is recommended that verification tests be performed along the Pier 2 alignment, as described in Section 6.7.

6.6 PIER 2 DRILLED SHAFT FOUNDATIONS

Pier 2 will be supported by drilled shafts that are socketed into bedrock (rock sockets). The proposed shafts are 8-foot diameter in soil, and reduce to 7-foot or 7.5-foot diameter rock sockets. The tops of the shafts will be tied back with permanent ground anchors (PGAs) through the individual shaft caps to provide additional lateral load capacity. The PGAs will be grouted into bedrock (WSDOT 2009b).

Portal towers will be constructed at each end of Pier 2. Each Pier 2 tower will be supported by four 5-foot diameter shafts that are socketed into rock. A generator room will be constructed adjacent to the eastbound lanes, just outside the west portal. URS understands the generator room will be supported by multiple shafts.

This section provides geotechnical recommendations for bearing resistance and input for lateral load analysis of the Pier 2 shaft foundations that update the recommendations previously provided in TM3. Recommendations for design of the PGAs were also provided in TM3. Supplemental recommendations for PGAs are provided in Section 6.7.

The shaft design recommendations provided in TM3 were based on limited information regarding bedrock elevations and rock strength properties. WSDOT conducted a structural analysis of the shafts using the design recommendations in TM3 and concluded that a shaft foundation could not be designed using the rock strengths and the lateral soil loads for the extreme event limit state that were provided in TM3.

URS and WSDOT collected additional bedrock elevation and rock strength data and conducted further interpretation of intact rock and rock mass strength properties, as described in Section 6.3. These updated bedrock elevations and rock strength properties were used to develop the rock socket design information presented in this section. The updated shaft design information was transmitted to WSDOT on January 29 and February 24, 2010. Shaft design information for the portal towers and generator room was provided to WSDOT on June 4, 2010.

6.6.1 Bearing Resistance

Unfactored bearing resistances for 7-foot and 7.5-foot diameter rock sockets are shown in Figures 6.17 and 6.18, respectively. This information was transmitted to WSDOT on January 29 and November 5, 2010, respectively. Figure 6.17 supersedes TM3 Figure 19. Unfactored bearing resistance for 8-foot diameter drilled shafts in soil is shown in TM3 Figure 16. The methods used to calculate the bearing resistance of shafts in rock are described in Section 5.3 and Appendix F.

Downdrag loads on shafts for the extreme event limit state were updated using the additional bedrock elevation information. The updated loads, which were transmitted to WSDOT on February 24, 2010, are provided in Figure 6.19 and supersede those provided in Table 7 of TM3.

Recommended resistance factors are provided in Table 6.5.

Unfactored bearing resistance for 3-foot, 4-foot, and 5-foot diameter rocket sockets is presented in Figures 6.20 through 6.22 for the west portal tower and generator room and in Figures 6.23 through 6.25 for the east portal tower. It is recommended that bearing resistance in soil be neglected for the portal tower and generator room shafts. Downdrag loads on the west portal tower and generator room shafts for the extreme event limit state are provided in Figure 6.26.

6.6.2 Input for Lateral Load Analyses

Recommendations for lateral load analyses of Pier 2 shafts are provided in Section 7.3 of TM3. This section presents the following updates to TM3 for the Pier 2, east and west portal tower, and generator room shafts:

- DFSAP input
- Lateral soil and rock loads and resistances for the extreme event limit state.

Input for lateral load analyses of the Pier 2, generator room, and portal tower shafts using the computer program DFSAP are provided in Table 6.6. Table 6.6 provides an updated value of the rock mass unconfined compressive strength for the Pier 2 shafts. This information was transmitted to WSDOT on January 29, 2010 and supersedes TM3 Table 8.

For the extreme event limit state, it is recommended that an evaluation of the stability and structural capacity of the shafts be conducted using the soil and rock loads and resistances that would develop on the shafts during the design earthquake, as estimated using a wedge analysis. The soil and rock loads and resistances to use in the evaluation are shown at 50-foot intervals from WB Station 1352+50 to 1363+50 in Figure 6.19. This information supersedes TM3 Figure 20. This information is provided for the tower portal and generator room shafts in Figure 6.26.

URS understands WSDOT is using the computer program GSTRUDL to calculate the shear and moment at the top of rock resulting from the structure loads and the soil and rock loads and resistances provided in Figures 6.19 and 6.26. The calculated shear and moment are used as input to DFSAP for extreme event limit state lateral load analyses.

The lateral loads provided in Figures 6.19 and 6.26 were calculated assuming that soil loads imposed on each shaft as a result of instability during seismic loading result from soil within a width equal to twice the shaft diameter (2B), where B is equal to 8 feet in soil. This information was previously provided to WSDOT on February 24, 2010.

Stability analyses of the slopes downslope from Pier 2 indicated limited displacements on the order of one foot during design earthquake loading. These calculations were interpreted to indicate that the soil centered between the shafts will likely experience limited downslope displacements that will act to relieve some of the lateral stresses on the shafts. This interpretation provides the basis for the assumption that only soil within a 2B width of the shaft centerline would impose lateral soil loads on a shaft during the design earthquake.

TM3 further describes the approach and assumptions used to develop the estimated loads and resistances. The wedge analysis is described in Appendix I of the 2007 Roadway Geotechnical Report (URS 2008a). The earlier results described in TM3 and Appendix I were based on an assumption that the soil load over a width equal to 3B would act on each shaft.

The actual rock strength at any shaft location could exceed the rock strengths provided in Table 6.6 and Figures 6.19 and 6.26. Higher rock strength values will indicate larger shear forces in the shaft near the rock interface. For design of the shafts for shear, the shafts should be analyzed assuming the rock unconfined compressive strength is greater than the unconfined compressive strength of the shaft concrete.

6.7 PIER 2 PERMANENT GROUND ANCHORS

Recommendations for preliminary design of permanent ground anchors in rock were provided in TM3. This section provides supplemental recommendations regarding potential group effects and no-load zones for anchors.

URS understands that the preliminary design calls for anchors with total pullout resistances of up to approximately 2,000 kips per shaft. WSDOT is considering up to four anchors per shaft that

would be placed through the shaft cap. A minimum separation between the anchors within the bond zone in rock of 5 feet is recommended to limit group effects.

URS recommends that anchor pullout resistance within portions of bedrock steeper than 40 degrees be neglected. The no-load zone is illustrated in Figure 6.27. It is noted that bedrock steeper than 40 degrees has not been measured to date, but could be encountered during construction. The no-load zone can be defined using the elevation at which bedrock was encountered during drilling of the anchor and during drilling of the associated shaft.

It is recommended that two verification tests be performed on non-production anchors along the Pier 2 alignment prior to beginning installation of the production anchors. The tests can be used to verify both Pier 1 and Pier 2 PGAs, since the PGAs at both piers will be bonded into the same rock type. Since the demolition of the existing snowshed and start of Pier 1 construction is not planned until Season 4 of the project, conducting both tests at Pier 2 will allow the tests to be completed earlier and with a single mobilization.

The test locations should be within the west and east one-third of the structure, between LW 1354+00 and 1355+00 and between LW 1361+00 and 1362+00. The tests should be performed in conformance with the requirements of GDM Section 15.5.2.3. The tests should be conducted at approximately the same elevations as the production anchors, and the test anchors should be bonded entirely in bedrock. If the tests are conducted after construction of the access bench, the soil nail wall must not be used for reaction.

At the time of this report, the maximum factored design load (FDL) for the Pier 1 anchors was larger than for the Pier 2 anchors. At least one verification test should be conducted to 1.5 times the larger of the maximum Pier 1 and 2 FDLs.

6.8 PIER 2 MSE WALL (WALL 3)

Recommendations for a Pier 2 MSE wall (Wall 3) were provided in TM3. A standard plan geosynthetic wall that incorporates a special footing design will be in the contract documents for Wall 3. The special footing design is needed to increase its clearance from other structural components of SSD, including the Pier 2 shaft caps.

The footing may be designed using an allowable bearing pressure equal to 5 tons per square foot (tsf). For earthquake loading, an allowable bearing pressure equal to 10 tsf may be used. For sliding, a nominal (unfactored) friction coefficient ($\tan\delta$) equal to 0.58 may be used for a footing cast-in-place on compacted, existing coarse-grained soil or structural fill. Using LRFD, the frictional resistance should be multiplied by a resistance factor equal to 0.9 for the strength and extreme event limit states. Using allowable strength design (ASD), a factor of safety no less than 1.5 should be used (Elias, et al. 2001).

The wall embedment below the adjacent finished grade should be at least 3 feet, which is the frost depth, or 5 percent of the wall height, whichever is greater, as described in LRFD C11.10.2.2 for non-abutment walls.

6.9 TEMPORARY SOIL NAIL WALL (WALL 4)

Recommendations for design of a temporary SNW (Wall 4) that will be used to support the roadway during construction of the Pier 2 foundations and Wall 3 were previously provided in TM3 (URS 2008b). This report updates TM3 by providing the following information:

- Soil conditions encountered in a test pit excavated near the wall alignment;
- The results of external stability analyses of the temporary wall during construction;
- Soil parameters for temporary wall design; and
- Avalanche surcharge loads.

A memorandum has been prepared by WSDOT that presents geotechnical design recommendations for Wall 4, including soil nail layout, dimensions, load transfer values, load testing, and monitoring (Allen and Frye 2010).

The construction staging plan as calls for the wall to be built during the first two seasons of Phase 1C construction and to support the detour roadway until construction of the permanent geosynthetic wall (Wall 3) is completed during the third season. Because the wall will be in service for only three years, it was not designed for seismic loading.

Based on preliminary design drawings provided to URS by WSDOT in January 2010, the SNW will be approximately 45 feet north of the Pier 2 centerline and approximately 17 feet north of the LE line. Based on roadway cross sections provided to URS by WSDOT on August 12, 2009, the maximum wall height will be approximately 45 feet at LW 1356+50.

Based on preliminary design drawings obtained from WSDOT on February 17, 2010, the lowest wall elevation will be approximately El. 2487. The wall will be partially submerged during high lake level periods.

6.9.1 Test Pit

Subsurface conditions in the vicinity of the SNW were further explored by excavating a test pit as recommended in GDM Section 15.3.4.2.1. The pit was excavated approximately 50 feet east of the west end of the existing snowshed. The pit location is shown in Figure 6.2.

The test pit was excavated to a depth of 12 feet on June 18, 2008. Test pit SSD-TP-001-08 was 32 feet long at the top and up to 10 feet wide. The pit was inspected again and backfilled, per WSDOT instructions, the following day. A graphical log of the test pit is provided in Appendix A. Photographs of the test pit are presented in Appendix B. A video recording was also made of the pit excavation, and is included as Attachment 1 of the electronic deliverable for this report.

Soil encountered in the test pit consisted of sandy gravel, with numerous cobbles and boulders up to 38 inches in size below a depth of 2.7 feet. Groundwater was not encountered in the excavation.

Some caving of the pit occurred below a depth of about 3 feet. The excavation was terminated at a depth of 12 feet because the amount of caving made it impractical to excavate deeper. The pit

was then backfilled to a depth of 10 feet. No additional caving or sloughing and no groundwater seepage were observed in the 10-foot deep pit after it had remained open for 24 hours. The pit was then backfilled.

6.9.2 Global Stability of Temporary Soil Nail Wall

The factors of safety for external stability of the temporary wall under static loading prior to construction of the permanent Wall 3 were calculated at two locations representing critical combinations of wall height and embankment steepness:

- LW 1354+50 (existing snowshed is not present at this location)
- LW 1357+39 (existing snowshed is present at this location)

It was assumed the reinforced wall zone would have a depth equal to 0.8 times the wall height and would act rigidly (i.e., only failure planes that pass outside of the reinforced zone were evaluated). Stability for failure planes that pass through the reinforced zone (internal stability) will be evaluated by the wall designer.

The existing snowshed is supported by footings that bear on soil beneath the lakeside wall and primarily on rock beneath the mountain side wall. The shed loading on soil was assumed to consist of 10 ksf pressure on a 5-foot wide footing on the lake side, and no load on the mountain side (i.e., the mountainside footing is supported on rock).

At LW 1357+39, it is likely that avalanches will deposit large amounts of snow on the existing EB lanes. Therefore, a correspondingly large snow load was included in the static loading analysis. The static snow load was assumed to consist of a prism that is 15 feet high at the wall face, sloping up to 23 feet high at the south edge of the existing snowshed (Wilbour 2008). The snow unit weight was assumed to be 50 percent of the unit weight of water.

At LW 1354+50, where the existing snowshed is not present and avalanche snow loads during the temporary service life of the soil nail wall are much less likely, a snow load surcharge equal to 320 psf was included.

Factors of safety were calculated for low (El. 2446) and high (El. 2517) water conditions at LW 1354+50 and 1357+39. Groundwater and lake levels were assumed to be equal.

A factor of safety of 1.25 or greater was considered acceptable, consistent with the temporary service life of the wall. The calculated factors of safety were found to be acceptable at both locations. The calculated factors of safety ranged from 1.59 to 1.83. The results of the stability analyses are presented in Table 6.7. Graphical results are presented in Appendix H.1.

6.9.3 Geotechnical Recommendations

The wall should be designed in accordance with Section 15 of the GDM. This section presents recommended soil parameters and surcharges for design of the wall using the computer programs SNAILZ or GOLDNAIL and discusses construction considerations.

Soil Parameters for Temporary Wall Design. Recommended soil parameters for the existing embankment soils to use in design of the temporary wall are shown in Table 6.8. The existing

embankment generally consists of free-draining soil. The groundwater level within the embankment soil may be considered equal to the lake level assuming the wall design incorporates adequate provisions for drainage. Lake levels are discussed in Section 2.1.3.

Avalanche Surcharge Loads. Because the wall will support the roadway for at least one winter season, it will need to be designed to resist avalanche loads. Traffic and seismic loads do not need to be combined with the avalanche loads.

The static snow load should consist of a prism that is 15 feet high at the wall face, sloping up to 23 feet high at the south edge of the existing snowshed (Wilbour 2008). The snow unit weight should be assumed to be 50 percent of the unit weight of water.

The wall will also need to withstand loads from the footings that support the south wall of the existing snowshed. The design should consider the condition where avalanche loads have accumulated on the roof of the shed.

The design of the wall should also consider dynamic loads and shear stresses that could result from the impact of an avalanche on the existing snowshed and roadway. These loads may exceed the static snow loads. The dynamic loads would only need to be considered below avalanches paths. A discussion of avalanche loading, including the locations of avalanches paths, is presented in URS & AIM (2007).

6.10 WALL 1

Wall 1 supports the WB roadway adjacent to a proposed avalanche snow catchment ditch north of the roadway. Wall 1 is up to 15 feet high, and the proposed catchment ditch is 25 feet wide. The north wall of the catchment will be formed by excavating a 0.25H:1V cut into the existing cut slope. URS previously provided geotechnical design recommendations to WSDOT for Wall 1 on September 23, 2010 and information related to the proposed 0.25H:1V cut on August 23, 2010.

6.10.1 Subsurface Conditions

Borings SSD-016-10 and SSD-017-10 were drilled to explore subsurface condition at Wall 1. Subsurface conditions consist of existing embankment fill and colluvium (Units 1 and 2) and bedrock.

Bedrock was encountered at shallow depths along the Wall 1 alignment and slopes down toward the lake. The interpreted rock elevation at the Wall 1 face is approximately 3 feet or less below the planned ditch elevation throughout the Wall 1 alignment. It is above the planned ditch elevation west of approximately LW 1352+00.

Bedrock consists of very weak (R1) to very strong (R5) lapilli tuff. Uniaxial unconfined compressive strengths of rock samples from SSD-016-10 and SSD-017-10 measured using the PLT ranged from 1.1 to 48.1 ksi, with an average of 21.4 ksi and a standard deviation of 17.8 ksi.

6.10.2 Global Stability

URS evaluated the global stability of Wall 1 under static and seismic loading. Factors of safety of 2.2 for static loading and 1.7 for seismic loading were calculated. These factors of safety

exceed the minimum acceptable factors of safety of 1.3 and 1.1 for static and seismic loading, respectively. Graphical representations of the analyses are presented in Appendix H.1.

6.10.3 Geotechnical Recommendations

A standard plan cast-in-place reinforced concrete retaining wall could be used for Wall 1. However, URS understands that WSDOT may elect to design Wall 1 rather than use a standard plan design, because the standard plan wall designs are based on weaker soil conditions than are present at the Wall 1 location. Using a site-specific design could reduce the wall foundation width, which would have the advantage of reducing the quantity of rock excavation required. A counterfort wall is an appropriate wall type for this location because it would require less rock excavation at the wall toe than other cast-in-place wall types.

URS recommends an unfactored bearing resistance equal to 20 ksf for footings placed on level rock or on dense, coarse-grained soil where the depth to rock does not exceed one-half of the wall footing width, although it is better practice to not have foundations bearing across rock-soil transitions, if practical. The total bearing resistance should be calculated using the effective footing width (B'), which is equal to the total footing width (B) minus two times the eccentricity (e) so the B' = B – 2e. Using this bearing resistance, the service limit state settlement should be less than one inch. URS recommends the resistance factors presented in Table 6.9 be used when evaluating the three limit states.

The recommended unfactored lateral earth pressures for Wall 1 design are shown in Figure 6.28 for the strength limit state and Figure 6.29 for the extreme event limit state. For the extreme event limit state, the seismic earth pressure and static earth pressure have been combined as a single uniform pressure in Figure 6.29. A PGA equal to 0.28g for Site Class B was used to calculate the seismic active earth pressure coefficient (K_{ae}) assuming a wall that is free to displace (that is k_h = 0.5PGA = 0.14).

Load factors of 1.5 and 1.0 for lateral earth loads are recommended for the strength and extreme event limit states, respectively.

Sliding and overturning resistance result from the weight of the soil bearing on the wall heel and toe (EV_{heel} and EV_{toe}). For walls embedded three feet or less below the bottom of the adjacent, catchment ditch, it is recommended that passive resistance be neglected. The weight of the soil can be calculated using a soil unit weight equal to 135 pcf. A load factor of 1.0 is recommended for vertical soil loads for evaluation of sliding and overturning.

For a cast-in-place wall footing on a clean, roughened, level rock surface, the unfactored frictional resistance may be computed using a friction coefficient (tanδ) equal to 0.7. For a wall footing cast in place on properly compacted structural fill or existing coarse-grained soil, the unfactored frictional resistance may be computed using a friction coefficient equal to 0.58. The computed sliding resistance must be multiplied by the resistance factors shown in Table 6.9.

It is assumed that the wall backfill will be fully drained so that there are no unbalanced hydrostatic pressures acting against the wall.

6.11 WEST PORTAL WALLS (WALL 9)

Wall 9 consists of a terraced system of walls adjacent to the Pier 1 West Portal, including Walls 9A, 9B, and 9C. The walls are oriented at a 15 degree angle upslope of SSD and support a counterfort retaining wall (Wall 9D) that directs avalanche snow across the roof of the shed.

WSDOT has solicited plans for the MSE walls from interested wall vendors. WSDOT submitted a wall site data package to the responding vendors that included a preliminary TM prepared by URS (URS 2010). In the preliminary TM, URS provided:

- A summary of geotechnical conditions at the wall site;
- An evaluation of and recommendations for global stability of the wall system, including minimum reinforcement lengths to provide global stability;
- Geotechnical parameters for design of the walls; and
- An evaluation of construction considerations, focused on methods of providing temporary shoring of the existing cut slope during wall construction.

WSDOT subsequently requested URS to provide allowable bearing pressures for Wall 9D for rock and soil foundations and to provide recommendations for providing compound stability. URS provided the bearing capacities on November 5, 2010, as follows:

Foundation Material	Allowable Bearing Capacity (tsf)	
	AASHTO Load Group I	AASHTO Load Group VII
Rock	10	13
Soil	3	4

The allowable bearing capacities for walls founded on soil are applicable to both wall sections founded on native soil and those founded on the MSE wall.

Compound failures pass through both the reinforced and unreinforced soil zones. For walls with complex geometries, such as Wall 9, compound stability can be the critical stability condition. URS conducted an analysis of compound stability and developed reinforcement recommendations to provide stability. The analysis and recommendations are described in Appendix H.12. The recommendations are summarized in Figure 6.30.

6.12 WALL 10

The global stability of Wall 10 was evaluated by W&N under subcontract to URS. The following is an excerpt from the TM prepared by W&N, which is provided in Appendix H.11:

> "It is concluded that on the basis of the available structural data the proposed cut slope in the vicinity of Wall 10 should be stable against overall structurally-controlled failure modes (planar, wedge, topple). This assertion does not rule out the potential for a persistent, low-shear strength feature such as a flow boundary that may be discovered during excavation. Accordingly, responsive stabilization contingencies may be required when actual rock conditions are exposed. To

> provide for face stability, stabilization of blocks defined by Set 1 or by the combination of Sets 1 and 3, slope reinforcement will be required."

Appendix H.11 presents recommendations for rock stabilization and design of supplemental rock bolts to provide for lateral stability at Wall 10.

6.13 TRANSFORMER BUILDING

The transformer building is a pre-cast concrete above grade structure with a 16-foot square footprint and a 7-foot wide pre-cast concrete below grade cable vault. The building is located about 10 feet outside of the EB traffic barrier between LE 1349+85 and 1350+02. Based on cross sections provided by WSDOT, the proposed grade at the building site is approximately El. 2535.

The closest boring, H-10-75, is located approximately 50 feet east of the transformer building location. This boring encountered granular fill above El. 2521, fine sandy silt between El. 2521 and 2519.5, and bedrock at approximately El. 2519.5. In this area, bedrock generally becomes shallower towards the west.

Although a deformation analysis was not conducted at the transformer building location, analyses at SSD indicate the potential for significant soil displacement during the design earthquake. WSDOT has determined that the structure can undergo significant displacement under seismic loading (Golbek 2010a). Based on this determination, shallow foundations are suitable for this structure.

Geotechnical recommendations for design of shallow foundations for the transformer room were provided to WSDOT and KPFF on January 5, 2011. URS understands that the bottom of footing elevation will be El. 2530.00. The recommendations assume that the footings are cast in place on medium dense granular fill. Any unsuitable soil encountered in the footing excavation should be overexcavated and replaced with compacted structural fill. The depth to the base of the footings should be at least equal to the frost depth. The footing width, B, should not be less than two feet.

Unfactored bearing resistance values of 14 ksf for the strength and extreme event limit states and 8 ksf for the service limit state are recommended. The bearing resistance may be applied over an effective footing width B', where B' is equal to the footing width, B, minus twice the eccentricity, e, so that $B' = B - 2e$.

Unfactored lateral earth pressures for the strength and extreme event limit states are shown in Figure 6.31. This figure includes lateral earth pressures resulting from the weight of snow that accumulates adjacent to the building (load ES); however, lateral pressures against the building from the snow itself are in addition to the lateral earth pressures. The pressures shown assume that the backfill around the footing is fully drained.

Sliding may be resisted by friction against the bases of the footings and passive resistance. For cast-in-place footings, the unfactored friction resistance may be calculated using a friction coefficient equal to 0.78. Unfactored passive earth pressure is shown in Figure 6.31.

For lateral earth pressures, a load factor equal to 1.5 is recommended for the loads EH and ES. A load factor equal to 1.0 is recommended for EQ. Recommended resistance factors for bearing,

friction, and passive pressure resistance are shown Table 6.9. For the allowable bearing capacity using ASD, the unfactored bearing resistance should be divided by a factor of safety equal to 3.0.

As noted above, the nearest boring is 50 feet east of the transformer building location, and bedrock tends to be shallower towards the west in this area. Consequently, it is possible that bedrock may be encountered in the footing excavation. Because of the potential for differential settlement of footings spanning soil and rock, it is recommended that rock, if encountered, be overexcavated to a depth at least one foot below the bottom of footing elevation and replaced with compacted structural fill.

6.14 CONSTRUCTION CONSIDERATIONS

Construction considerations for SSD were discussed in TM3. This section presents supplemental considerations identified subsequent to TM3.

6.14.1 Recommendations of the ADSC

WSDOT conducted meetings with the ADSC on November 5, 2009 and September 28, 2010. The purpose of the meetings was to solicit input from experienced drilled shaft practitioners regarding the constructability of Pier 2.

URS provided an information package to WSDOT on October 13, 2009 that was subsequently provided to the ADSC for the November 5, 2009 meeting. The ADSC provided their input in memoranda to WSDOT dated November 6, 2009 (Macnab 2009) and October 7, 2010 (Macnab 2010). These memoranda are provided in Appendix I.

6.14.2 Rock Monitoring

Foundation elements, including Pier 1 footings and PGAs and Pier 2 drilled shafts and PGAs, will be designed to develop resistance partly or wholly from bedrock. This report presents sections, profiles, and contours that represent our interpretation of subsurface conditions, including rock elevations, based on data and information from borings, geologic mapping, and drawings for the existing shed. Actual conditions encountered during construction may vary from those shown.

In particular, the configuration of the bedrock surface in the Pier 1 vicinity as shown in the subsurface cross sections must be understood as an interpretation that is based on sparse boring data. Pier 1 is generally in an area of past rock cuts. With the exception of borings near the west and east ends of the proposed shed, there are no borings in the area of the cuts – the closest borings were drilled on the right shoulder of the EB lanes, 60 feet or more from the Pier 1 centerline. Consequently, the location of the south edge of the presumed bench formed by the rock cuts is uncertain. If more precise bedrock information is needed in that area, it would probably be necessary to do additional exploration during construction after traffic has been moved to the detour lanes.

Another possible course of action is to probe the top of bedrock during construction of the temporary soil nail wall. This action would provide bedrock information at an early stage of construction. If steep bedrock dips (steeper than 40°) are identified, the foundation design assumptions should be reviewed in light of this information, including an evaluation of the need for passive localized rock reinforcement.

6.14.3 Pier 2 Monitoring

SSD will extend over an existing embankment with an inclination near the angle of repose. The embankment could be subject to instability under earthquake loading. While it is not practicable to simulate earthquake loading, URS believes it would be prudent to monitor for indications of embankment instability.

The monitoring could be accomplished using slope inclinometers similar to those installed in the Slide Curve embankment. Three to five locations could be monitored immediately downslope from Wall 3 mid way between shafts where the potential for downslope movement will be the greatest. While URS does not expect slope movements under static loading, the inclinometers would also monitor movements that might occur over time.

6.14.4 Permanent Ground Anchors

The centerline of the proposed shaft caps is 15 feet from the edge of the planned construction bench. Limited space will be available for the anchor drilling equipment if the anchors are drilled after construction of the shaft caps.

6.14.5 Soil Nail Wall

Borings and test pits near the wall alignment encountered boulders that were relatively frequent in number and relatively large in size. Historical photos of the snowshed construction show boulders in excess of 12 feet in size. The contractor should provide equipment suitable for drilling in materials containing numerous boulders. The contractor should also be prepared to encounter materials with relatively large voids that have the potential for large grout take. An irregular cut face resulting from protruding boulders and localized caving should be anticipated.

The conditions observed in test pit TP-SSD-001-09 indicate that near-vertical cuts of limited height may be feasible. Soils encountered in the test pit had some apparent cohesion and were able to stand up to a depth of 10 feet for 24 hours. The test pit was of limited length (less than 32 feet long at the bottom), and longer cuts may not stand up over a 10-foot height. Surface water and groundwater could adversely affect the apparent cohesion. Surface water should be directed away from the excavation face. Where nails are planned near the lake level, sufficient time should be allowed for the soil to drain before excavation.

The wall will be partially submerged during high lake level periods, and construction staging will need to consider the lake levels. Lake levels are discussed in Section 2.1.3.

6.14.6 Construction Access Bench Stability

Construction activities, including shaft construction, will need to be conducted from an access bench that is cut into an embankment with side slopes as steep as approximately 1.2H:1V. The planned distance from the center of the shafts to the edge of the bench is 15 feet. The contractor will need to evaluate stability during construction activities. Reaction piles or other measures may be needed for shaft construction. A requirement that the contractor document the method that will be used to provide stability during shaft construction prior to beginning work should be considered.

6.14.7 Shaft Construction

Shafts at SSD will need to be drilled through soil containing large boulders into rock with measured unconfined compressive strengths ranging up to 44.6 ksi. Because of the difficult drilling conditions, an oscillator rig is recommended for shaft construction. The ADSC also recommended use of an oscillator rig (Macnab 2009).

7.0 SLIDE CURVE BRIDGE AND WALLS

7.1 PROJECT DESCRIPTION

The design of Slide Curve bridge and walls (SCB) was completed by URS (geotechnical engineering) and KPFF (structural engineering) under TOs CO for preliminary design and DL for final design. The final design drawings were submitted to WSDOT on January 25, 2011.

SCB extends from approximately LE 1371+10 to 1385+80. At this location, the highway will be shifted up to 65 feet south towards Keechelus Lake. The roadway surface elevation will remain approximately the same, ranging from approximately El. 2539 (LE 1371+00) to El. 2532 (LE 1385+80). The SCB location in relation to the overall project is shown in Figure 2.2B. A plan view and typical cross section are shown in Figures 7.1 and 7.2.

The EB lanes will be supported by a bridge and permanent geosynthetic walls, as shown in Figure 7.1. The bridge will extend from LE 1372+06 to 1383+58, a total length of 1,152 feet. The walls will extend from LE 1371+10 to 1372+06 on the west end (Wall 5) and from LE 1383+58 to 1385+80 on the east end (Wall 6).

The bridge will consist of eight spans ranging in length from 118 to 160 feet. The bridge spans will be supported by abutment Piers 1 and 9 and interior Piers 2 through 8, which are located at LE 1373+24, 1374+43.5, 1376+00, 1377+60, 1379+20, 1380+80, and 1382+36.5. Pier 1 will be founded on a stepped footing. Pier 9 will be supported by two drilled shafts. The interior piers will consist of two columns each, with each column founded on a single shaft. The Pier 9 and interior pier shafts will be 10 feet in diameter in soil, reduced to 9 feet in diameter in rock.

A permanent soil nail wall (SNW) (Wall 7) will be constructed adjacent to the bridge from LE 1371+72 to 1383+87. The purposes of the permanent wall are to support the WB lanes and provide the space for an approximately 60-foot wide construction bench needed to construct the shafts. Temporary SNWs (Walls 7A and 7B) will be constructed from LE 1371+63 to 1373+74 on the west end and LE 1383+14 to 1383+89 on the east end. The temporary walls will be converted to permanent walls.

The WB lanes behind the SNW (Wall 7) are elevated by SCW (Wall 8) between LW 1377+30 and the east end of the SNW. The maximum height of SCW is 18 feet above the proposed adjacent finished grade at the east end of this interval. SCW is further described in Section 8.0.

The maximum height of Wall 7 will be approximately 45 feet. The wall will extend to the construction work bench or bedrock for the entire length of the bridge. The construction work bench typically rests on the bedrock except between LE 1378+31.8 and 1381+63.1. In these intervals, Wall 7 will not extend to bedrock.

Where Wall 7 does not extend to bedrock, a tied-shaft wall (Wall 23) that is socketed into bedrock will be constructed. The purposes of the tied-shaft wall are to provide for global stability of the SNW and WB roadway and isolate the pier shafts from the unstable upslope soils. The tied shafts will be 3 or 5 feet in diameter and socketed into bedrock. The 3-foot diameter shafts will be spaced 6 or 9 feet center-to-center. The 5-foot diameter shafts will be spaced 10 feet center-to-center. The tops of the shafts will be connected by a continuous grade beam. Each shaft will be tied back with two permanent ground anchors.

7.2 BACKGROUND

The design for SCB has undergone substantial evolution during the design process. Conceptual design was completed under TOs BH (URS and KPFF 2007) and BX (URS 2008c; URS 2008i). The conceptual design that was retained was referred to as the MSE/Bedrock+Bridge concept. This concept consisted of an 800-foot-long, five-span EB bridge bounded by EB MSE walls founded on bedrock, with the WB lanes supported by a permanent SNW in the section adjacent to the bridge. The evolution of this conceptual design under TOs BH and BX is described in URS 2008i.

Preliminary design was completed under TO CO. During this phase a design concept was introduced that included half-bridges on each end of the main bridge. This design concept was referred to as the Half Bridge concept.

Final design is being completed under TO DL. During this design phase, the half bridges were deleted and replaced with conventional bridge spans. This design concept is referred to as the Whole Bridge concept. URS and KPFF submitted 90 percent complete design plans on August 16, 2010. The 90 percent complete design plans for the SNWs were prepared as a separate deliverable by URS, which was submitted on August 23, 2010.

The preliminary and final design development included a series of project progress meetings where URS and KPFF reported the design status to representatives of the WSDOT I-90 Project Office, Bridge and Structures Office, and Geotechnical Division. The major decisions and direction related to foundation design that resulted from these meetings are summarized below.

Meeting No. CO.01 – September 25, 2008. It was determined that a pier abutment was the preferred abutment conceptual design. It was also determined that an additional boring was needed to evaluate the stability of the permanent SNW.

Meeting No. CO.02 – January 13, 2009. It was determined that a controlled density filled (CDF) foundation is adequate for support of the MSE wall in those situations where a rock bench cannot be excavated. WSDOT tasked the URS/KPFF design team with comparing the cost of an SNW that extends to bedrock to the cost of a wall that extends to the construction bench and includes a foundation consisting of tied shafts or piles.

Meeting No. CO.03 – February 12, 2009. It was determined that the SNW needed to be founded in bedrock to provide global stability during seismic loading. The lack of stability resulted in the potential for additional lateral loads on the bridge pier shafts. The SNW foundation concept that was agreed upon consisted of piles or shafts socketed in rock and tied back with ground anchors that extend into bedrock.

Meeting No. CO.04 – April 8, 2009. WSDOT tasked the URS/KPFF design team with evaluating (1) shallow foundations on rock at the abutment piers and (2) use of a concrete-filled trench wall foundation wall for the SNW where the depth to bedrock from the based on the SNW is 10 feet or less.

Meeting No. CO.05 – August 4, 2009. It was agreed that the shallow abutment foundation concept should not be retained because (1) the depth to bedrock would result in very large excavations and (2) the potential presence of adverse bedrock structure that could compromise the stability of the abutment.

Meeting No. CO.06 – August 17, 2009. It was determined that supporting the MSE walls on shafts socketed into bedrock (MSE/Shaft conceptual design) where the bedrock elevation is lower than El. 2490 is preferable to supporting the walls directly on bedrock (MSE/Bedrock conceptual design).

Meeting No. CO.07 – September 11, 2009. The purpose of this GoTo Meeting was to present the design and cost comparisons for the MSE/Shaft Wall and Half-Bridge concepts. It was determined that bridge approaches consisting of half-bridges are preferable to approaches consisting of MSE walls (MSE/Shaft concept). The primary drivers for this decision by WSDOT were the lack of proven examples of MSE walls supported by drilled shafts and the higher global stability with the half-bridge concept, even though the estimated construction cost of the half-bridge concept was higher.

Meeting No. CO.08 – October 28, 2009. The URS/KPFF design team presented preliminary design information for the half-bridges at this meeting.

Meeting No. DL.01 – March 29, 2010. Girder constructability and scheduling were discussed at this meeting.

Meeting No. DL.02 – June 10, 2010. The purpose of this meeting was for URS and KPFF to present the Whole Bridge Concept. The current half-bridge conceptual design was first presented, followed by the proposed Whole Bridge design concept and the anticipated construction sequencing. A completed project on WSDOT I-405 was used as an example of a project where a similar design has been successfully executed. The WSDOT Project Office and WSDOT Bridge Group agreed unanimously that the proposed concept should move forward into preliminary design.

Meeting No. DL.03 – June 21, 2010. This GoTo Meeting was a follow-up to the June 10, 2010 meeting. Refinements of the Whole Bridge constructability were presented, including sequence of SNW construction at the west approach. WSDOT Project Office comments regarding our original Whole Bridge concept design were addressed and resolved in this meeting. A spreadsheet was presented that showed how the cost savings of the Whole Bridge was calculated. Action items from this meeting included the following: 1) The projected office set a submittal date of July 8, 2010 for the preliminary design; 2) URS to submit an LOE and SOW for the Whole Bridge preliminary design and final design work; and 3) URS to proceed with final design.

Meeting No. DL.04 – June 30, 2010. Refinements to the Whole Bridge were discussed, including using a stepped foundation for Pier 1 and the necessity for a two-stage construction sequence for the Pier 1. The SNW construction sequence was presented by URS at the west Whole Bridge area. Action items: 1) the WSDOT Project Office would send SNW geometry data; 2) it was agreed by all on the call that the vertical nail spacing would be 2.5 feet and that the alternating nails would extend to bedrock to identify bedrock depth for use during SNW and tied shaft wall construction; and 3) WSDOT Project Office requested the thickness of permanent facing.

Meeting No. DL.05 – July 7, 2010. The proposed design for the Pier 1 abutment was presented, including the bedrock elevations at the foundation location. The Pier 1 construction sequencing and how it related to the temporary and permanent SNW construction was also discussed. The conceptual designs for the temporary and permanent SNW design were agreed upon in the proximity of abutment at Pier 1. Action items: 1) URS would verify the bedrock elevations

assumed as part of the Pier 1 foundation and 2) WSDOT would drill two additional borings in the Pier 1 proximity to verify bedrock depths.

Meeting No. DL.06 – July 7, 2010. This was a follow-up call to meeting DL.05. Continued discussions about the constructability of the Pier 1 abutment were held. It was agreed by those on the call that all temporary shoring for the Pier 1 abutment would be designed by the contractor.

Meeting No. DL.07 – September 8, 2010. The WSDOT Project Office led this GoTo Meeting. These items were discussed, followed by the result:

1) Is it allowable to pour half of the bridge deck for all spans of the bridge and then shift traffic? It was agreed by WSDOT Bridge Office and URS/KPFF that this was feasible.

2) Concerns about placing more girders than are required for stage 1 bridge deck were discussed. It was tentatively assumed that additional girders could be placed beyond the stage 1 bridge deck pour; however, crane movement on the access bench would be restricted.

Meeting No. DL.08 – September 28, 2010. Girder placement, including phasing and equipment positioning, and reinforcement requirements for Walls 5 and 6 were discussed.

URS provided geotechnical design information for the Whole Bridge to KPFF on the following dates:

- Drilled shafts: August 12 and 13, 2010
- Tied shaft wall: June 28 and July 20, 2010
- Shallow abutment foundations: October 8, 2010
- Complete recommendations: October 19, 2010
- Wall 23 anchor lengths to bedrock: January 17, 2011

URS provided geotechnical design information to WSDOT on the following date:

- Approach fill Walls 5 and 6: October 6, 2010

7.3 FIELD EXPLORATION AND LABORATORY TESTING RESULTS

7.3.1 Field Exploration

A total of 40 borings have been drilled near SCB, including:

- Two borings drilled during the 2010 exploration program;
- 12 borings drilled during the 2009 exploration program;
- 14 borings drilled during the 2008 exploration program;
- Seven borings drilled during the 2007 exploration program;
- Two borings drilled during the 2006 exploration program; and
- Three borings drilled during earlier investigations, including one each in 2005 and 1998.

One test pit was excavated during the 2008 exploration program. The boring and test pit locations are shown in Figure 7.1. A summary of the SCB borings and test pit is provided in Table 7.1. Logs of the borings and test pit are provided in Appendix A. Rock core photographs are provided in Appendix B.

The boring depths ranged from 33.0 to 107.5 feet below the existing ground surface. Of the 40 borings, 14 borings were drilled from a barge in Keechelus Lake and the remaining 26 borings were drilled on land using skid or truck-mounted drill rigs.

7.3.2 Visual and Acoustic Borehole Logging

Visual and acoustic borehole logging was conducted in 6 borings (SCB-024-09 and SCB-026-09 through SCB-030-09) by CRUX using the CRUX Oriented Borehole Logging (COBL®) system. The COBL® logs are provided in Appendix C. The use of the COBL® data in the evaluation of rock mass strength properties is described in the memorandum "Slide Curve Bridge, Geologic and Geotechnical Interpretation of Significant Discontinuities," which is provided in Appendix E.

7.3.3 Laboratory Testing

Laboratory testing consisted of index, strength, and consolidation tests on selected samples from the borings drilled at the SCB location. Laboratory data sheets are provided in Appendix D.

Index Tests. Index testing was conducted at the URS laboratory and consisted of grain-size distribution tests on 40 samples, Atterberg limits tests on 4 samples, and water content tests on 42 samples. Index testing results are summarized in Table 7.2.

Strength and Consolidation Tests. Soil strength and consolidation tests were conducted by WSDOT and consisted of CU and consolidation tests on 2 samples of soft lake sediment from boring SCB-015-08. As described in Section 3.3, there were some problems associated with the test data and meaningful results could not be obtained. The consolidation testing results are summarized in Table 3.7.

CU tests were performed at SW2 in lake sediments similar to those encountered near SCB. The test results are discussed in Section 3.3 of this report and are summarized in Table 3.7. It is reasonable to use the strength data obtained from these tests to characterize the material properties required for geotechnical analyses at SCB.

Rock strength tests were conducted by WSDOT and consisted of unconfined compression tests on 51 rock specimens. The unconfined compression test results are summarized in Table 7.3.

Ring shear tests were conducted on one core sample by URS. The purpose of the ring shear tests was to assess the undisturbed and residual shear strength of fine-grained soil infilling that was encountered in some rock samples. The results of the ring shear tests are presented in Table 3.8.

7.3.4 Field Testing

Field testing consisted of 176 PLTs on rock specimens from 139 core runs to measure the uniaxial unconfined compressive strength of the rock. A summary of rock core point load testing results is presented in Table 7.4.

7.3.5 Geophysical Investigation

URS performed a geophysical investigation along the outer edge of the existing EB lane at SCB. The objective of this geophysical investigation was to evaluate the depth to bedrock along the SCB alignment. The investigation consisted of a seismic refraction survey. A summary of the investigation is provided in Section 3.2.10, and detailed descriptions of the methodology used and the results obtained are provided in the geophysical investigation memorandum dated May 20, 2008. This memorandum, which includes a profile showing measured compressional wave velocities along the survey alignment, is presented in Appendix C.

A compressional wave velocity of 6,000 to 8,000 feet per second was interpreted to represent bedrock. Where borings were drilled close to the survey line, the differences between the elevations where rock was encountered in the borings and the interpreted bedrock elevations were five feet or less.

7.4 SITE CONDITIONS

7.4.1 Site Description

The SCB site is located adjacent to the north shore of Keechelus Lake, between SSD and Slide Curve, and adjacent to the southern flank of Keechelus Ridge. The eastern edge of SCB is approximately 1000 feet west of Slide Curve.

The existing I-90 consists of four lanes of traffic: two EB and two WB. The north side of I-90 consists of rock slopes that extend upwards from the highway. Within the SCB site, the south side of the highway consists of an embankment fill slope that extends downward to Keechelus Lake, with a slope varying from approximately 1.2H:1V to 1.8H:1V, and an average slope of approximately 1.3H:1V.

The proposed roadway elevation along the outer edge (lake side) of the SCB alignment ranges from approximately El. 2530 to 2540. The existing ground elevations along this alignment range from approximately El. 2480 to El. 2532.

The water level in Keechelus Lake fluctuates seasonally. The lake level fluctuations are described in Section 2.1.3.

7.4.2 Geologic Setting

Keechelus Lake is located in a U-shaped valley of glacial origin. The highway and slope bounding the highway on the east in the general vicinity of SCB is underlain by volcanic rock of the Keechelus Lake member of the Ohanapecosh Formation (URS, 2007b). Detailed geologic mapping and drilling on the slope by Wyllie and Norrish (2009b) indicates the rock is a meta lapilli dacite tuff with a varying amount of welding and weathering. Three domains were identified based on the degree of welding and weathering and relative rock strength.

The results of drilling along and below the roadway and a geophysical survey indicate bedrock is generally buried beneath colluvium, landslide debris and fill. Fill was placed to construct the existing roadway embankment, and additional rock from rock cuts blasted during construction of I-90 was placed in the embankment slope. Asphalt and concrete fragments removed during past road repair is also present in the fill. Wave action and fluctuating lake levels have reworked the

slope deposits and fill, and deposited a veneer of lacustrine silt, sand, and gravel on the slope between the low and high water levels.

7.4.3 Subsurface Conditions

Seven cross sections and four profiles have been developed that show the interpreted subsurface conditions. The profiles were developed along the outer edge of the SCB alignment (referred to as SCB Structure Profile) and SNW (Walls 7, 7A, and 7B) alignments. Figures 7.3 through 7.9 present the cross sections, and Figures 7.10 through 7.12 present the profiles. Profiles along the LE and LW lines are shown in Figures 4.1 and 4.2, respectively. Figure 2.1 presents the notes and legend for the cross sections and profiles.

The interpreted subsurface soil and rock consist of three soil units and one rock unit, as follows.

Unit 1 (Gravel with Sand/Cobbles/Boulders [Primarily Fill]) consists of fill material that was placed for construction of the existing I-90 roadway embankment. Generally, the material consists of loose to very dense, poorly-graded to well-graded gravel with sand, cobbles, and boulders. Boulders are distributed throughout Unit 1. Boulder sizes generally were not recorded on some of the logs. Where sizes were recorded, the boulders ranged up to about 3.8 feet in size.

The thickness of the unit ranges from approximately 4 to 40 feet along the outer edge of SCB alignment. This layer is the thickest around LE 1380+75 and gradually becomes thinner towards the east and west along the alignment.

Unit 2 (Gravel and Sand with Cobbles/Boulders/Rock Fragments) consists of a mixture of colluvium, talus, landslide deposits and glacial deposits. Generally, the material consists of loose to very dense, typically dense to very dense, gravel or sand with occasional cobbles and boulders. Boulders were encountered in five borings, SCB-005-08, SCB-019-09, SCB-020-09, SCB-022-09 and SCB-025-09. The largest boulder size encountered in this unit was 3.8 feet in SCB-020-09.

The thickness of this unit ranges from 0 to 43 feet along the outer edge of the SCB alignment. This layer is the thickest around LE 1381+25 and gradually becomes thinner towards the east and west along the alignment in general.

Unit 3 (Clay/Silt [formerly Recent Lake Sediments]) consists of very soft to soft, fine-grained soil at and beyond the toe of the embankment slope. Coarse-grained inclusions up to boulder size were present in the clay and silt matrix. This unit was encountered in boring SCB-015-08, which was drilled in the lake, at El. 2237. The thickness of this unit is approximately 35 feet at this boring location. This unit is shown on cross section 4-4' (LE 1379+90).

This unit was also identified during a geophysical investigation conducted by S&W in 1974 (S&W 1974) in the vicinity of SW2. S&W identified a "disturbed area" of very soft lake sediments at the toe of the slope where fill has displaced the sediment and formed a "mud wave" on the lake bottom. This unit corresponds to Unit B in the S&W report.

Golder Associates conducted a geophysical investigation in 1998 in the vicinity of SCB (Golder 1998). Golder also identified lacustrine deposits at and beyond the toe of the embankment slope.

Unit 4 (Bedrock) consists primarily of lapilli tuff of the Naches formation. The elevation of bedrock is highly variable both parallel and perpendicular to the SCB alignment. The interpreted

top of bedrock elevation along the outer edge of SCB alignment ranges from about El. 2420 to El. 2520. The top of bedrock along the alignment is estimated to be at the lowest and highest elevations at approximately LE 1379+90 and 1371+00, respectively. The interpreted location of lowest bedrock is approximately 70 feet east of Pier 6 and 90 feet west of Pier 7.

The RQD of the bedrock ranges from 0 to 100 percent. Bedrock strength ranges from very weak (R1) to very strong rock (R5) based on both field observations and field PLTs. Unconfined compressive strength (UCS) tests were performed on 51 samples. The measured unconfined compressive strengths are tabulated in Table 7.3. The rock strength in the tested samples ranges from 3.7 to 26.8 ksi, which corresponds to moderately weak (R2) to strong (R4) rock.

Some of the poorest-quality bedrock was encountered in SCB-016-08 and SCB-017-08, which are located in the area of deepest bedrock. The RQD of core runs in these borings was typically 0, with a maximum of 15 percent. The rock strength was classified as extremely weak (R0) to moderately weak (R2).

Several clay/silt/sand infillings ranging from very thin to 1-foot thick were encountered within bedrock in several borings. These infillings are further described in Section 7.4.4.

7.4.4 Rock Mass Structure

Soil-filled discontinuities that represent potential planes of weakness in bedrock were encountered in several borings. The thickest soil-filled discontinuities were encountered in the following borings:

- 0.8-foot thick discontinuity at El. 2476 in SCB-022-09 at the west end of the bridge;
- 1-foot thick discontinuity at El. 2465 in SCB-021-07 at the east end of the bridge;
- 0.2 to 0.4-foot thick discontinuity at El. 2431 in SCB-006-08; and
- 0.25 foot thick discontinuity at El. 2450 in SCB-028-09.

URS evaluated the potential for persistent, low-strength, unfavorably-oriented weakness planes that could have adverse effects on the bridge foundations and prepared a TM, dated February 5, 2010, which is presented in Appendix E.4. The evaluation was focused on the east end of the bridge.

The evaluation included visual examination of the rock core photographs, boring logs, and COBL® logs; laboratory strength and index testing of soil from infillings; and development of stereonets of the significant discontinuities.

The TM describes joint sets of potential significance to the performance of the bridge foundations and potential failure mechanisms. URS concluded that, based on the available data, it is unlikely that any of the potentially significant discontinuities encountered in borings near the east end of SCB are persistent across the bridge foundation. The TM also provides rock properties that can be used to incorporate the effects of potentially significant discontinuities in the design process.

7.4.5 Rock Strength Properties

URS interpreted intact rock and rock mass strength properties based on the boring information and rock strength testing. The interpreted rock strength properties were used to develop

recommendations for rock socket bearing and lateral resistance and input for lateral load analyses using DFSAP.

The interpreted rock strength properties at the bridge pier rock socket locations are summarized in Table 7.5.

7.4.6 Bedrock Elevations

URS developed a bedrock contour map for the SCB vicinity using the contouring software EVS by C Tech Development Corporation. The contour map is shown in Figure 7.13.

The following information was used in developing the contour map.

- Topography above the lake level
- Elevations of bedrock encountered in borings;
- Bedrock elevations in rock outcrops below the highway;
- Bedrock elevations estimated from the results of the geophysical survey along the lakeside of the existing highway (Appendix C); and
- Bathymetry and bedrock elevations estimated from the results of an offshore geophysical survey (Golder 1998) below the lake level.

The bedrock contours are an interpolation between the data points from borings and bedrock outcrops and interpreted bedrock elevations at selected points along the geophysical survey lines. The actual bedrock surface is likely to be more irregular and variable than depicted in the areas between data points. The bedrock elevations below the low lake level, which are based on interpretation the geophysical surveys, have greater uncertainty than those in the area above the low lake level, which are based primarily on outcrop and boring data.

7.4.7 Groundwater

Observation wells were not installed in the vicinity of SCB. The nearest OW, H-24-06(OW), is located on the lakeside EB shoulder at LE 1365+92, which is approximately 500 feet west of SCB. H-24-06(OW) was installed on June 13, 2006, with the bottom of the well at approximately El. 2496.

Groundwater levels were monitored from June 2006 to August 2010. Groundwater levels were not measured during the period of late October to late April due to winter conditions at the site.

The measured groundwater levels in H-24-06(OW) were equal to the lake level when the lake level was above the bottom of the well. Based on this available water level information, and considering the high permeability of Units 1 and 2, the groundwater levels along SCB are interpreted to be approximately the same as the fluctuating lake levels. A graphical representation of variations in the groundwater level in H-24-06(OW) and the lake level is provided in Figure 3.2.

7.5 SLOPE STABILITY ANALYSIS

This section describes the interpreted soil and rock parameters used for global slope stability analyses and the results of those analyses. The scope of the stability analyses included:

- An assessment of bedrock slope stability controlled by the structural geology at the west (Pier 1) and east (Pier 9) abutments and interior Pier 7. This assessment was performed by W&N and documented in a TM, which is provided in Appendix H.

- Global stability of the permanent SNW (Wall 7) in areas where the wall is not founded on bedrock, represented by analyses at LE 1379+90 and 1381+50.

- Global stability of the geosynthetic walls supporting the approach fills in areas where the walls are not founded on bedrock.

7.5.1 Stability at West Abutment (Pier 1)

W&N concluded that, based on the available borehole data, rock quality at Pier 1, the westernmost abutment, is better than that observed at Pier 9 (see Section 7.5.3, W&N 2009a, and URS 2010). Given that stability analyses for Pier 9 assuming the lower to moderate rock mass and discontinuity strength estimates yielded favorable margins for global rock mass stability, Pier 1 can reasonably be assumed to be more stable than Pier 9 and detailed stability analyses were not performed.

7.5.2 Stability at East Abutment (Pier 9)

W&N calculated minimum factors of safety of 2.03 and 1.23 for static and seismic loading, respectively. These factors of safety indicate that adequate global stability exists at Pier 9 for potential failure surfaces through rock. A detailed description of the analysis is presented in the W&N TM, which is provided in Appendix H.

7.5.3 Stability at Pier 7

Pier 7 at LE 1380+80 was selected for a stability analysis because of the localized poor quality rock encountered in the vicinity. W&N calculated factors of safety of 1.84 and 1.17 for static and seismic loading, respectively, and noted that engineering practice would typically target a minimum factor of safety value of 2.0 under static loading. A detailed description of the analysis is presented in the W&N TM, which is provided in Appendix H.

The W&N TM noted analysis assumptions that may be conservative and have reduced the calculated factor of safety. URS considers the calculated factor of safety to be adequate, but recommends monitoring the shaft excavation to confirm that conditions more unfavorable than those assumed in the analyses are not encountered. Recommendations for monitoring of the shaft excavation are described in Section 7.7.2.

7.5.4 Stability of Soil Nail Wall

Soil Properties. For the slope stability analyses purposes, Units 1 and 2 were assumed to be free-draining and cohesionless, with effective angles of internal friction of 40 degrees.

Unit 3 is characterized as a normally consolidated (that is, no excess pore pressures), non-free-draining cohesive silts and clays under existing conditions. The geometry and layer thickness of this unit was interpreted based on 2008 geotechnical investigation and a previous geophysical investigation that was performed in the lake within a few hundred feet of the SW2 alignment (S&W 1974). Based on the limited data available, there is uncertainty in the extent of this unit.

For the slope stability analyses, a SHANSEP (stress history and normalized soil engineering properties) approach was used to characterize the undrained (total stress) strength of Unit 3 (Ladd 1974; Kulhawy 1990; Duncan 1989). The SHANSEP approach characterizes undrained shear strength (S_u) in terms of the effective preconsolidation stress (p') using a constant "undrained strength ratio" (S_u/p'). For normally consolidated soil (p') is equivalent to the vertical effective overburden stress (σ'_{vo}).

As discussed in Section 2.2.2, due to non-availability of the strength data information at SCB, it is reasonable to use the strength data obtained from the CU tests performed on Unit 3 at SW2 to determine the material properties required for geotechnical analyses at SCB. A back analysis was performed at SW2 to determine an S_u/p' ratio for Unit 3. An S_u/p' ratio of 0.26 was obtained based on this analyses. The back analysis is discussed in Section 10.0 of this report. The same S_u/p' ratio was used for Unit 3 at SCB to perform the slope stability analyses.

Stability of Soil Nail Wall at LE 1379+90. At LE 1379+90, the global stability analyses presented in TM4 (URS 2008c) were updated using new information on the lake sediments stratigraphy and strength parameters obtained from the 2008 field investigation. This cross section was selected as representative of the most critical combination of wall height and bedrock depth.

Potential failure surfaces starting beneath the base of the SNW (Wall 7) were evaluated. For static loading, the calculated factors of safety were 1.49 (high lake level) and 1.46 (low lake level). For the seismic loading, the calculated pseudostatic factors of safety were 0.81 (high lake level) and 0.84 (low lake level).

For the static cases, the critical failure surface starts at the base of the SNW and passes through Units 1 and 2. For the pseudostatic cases, the critical surface starts at the base of the SNW and passes through Units 1, 2 and 3. The low water and high water cases were critical under static and seismic conditions, respectively.

The analyses indicate that factors of safety for the embankment supporting the proposed SNW are less than minimum required factor of safety of 1.1 under seismic conditions. Table 7.6 presents factors of safety obtained for various cases and the figure numbers of the graphical results for each case. The graphical results are provided in Appendix H.

The analyses were performed assuming that there will not be any soil strength degradation during seismic events. The liquefaction potential for Units 1 and 2 is low; however, Unit 3 has the potential to liquefy and lose strength under seismic events. Therefore, the minimum pseudostatic factors of safety could be lower than those shown in Table 7.6.

The analyses assume the SNW is founded on overburden soils and not on bedrock. The thickness of soil between the base of the SNW and the bedrock surface is greater than 10 feet at this location. The wall should be isolated from the downslope soils so that the stability of the downslope soils does not impact the integrity of the SNW. This could be accomplished by

extending the SNW to bedrock or by providing a wall consisting of tied-back shafts that extends to bedrock in front of the SNW.

Excavation for placing the wall on bedrock would be feasible only during periods when the lake level is lower than the bedrock elevation. The lowest bedrock elevation below the SNW is approximately El. 2465. In an "average" water year (Figure 2.3), the lake level does not drop below El. 2465 until the beginning of September. In a high water year, it is possible that the lake level would remain above El. 2465 the entire construction season. Therefore, a tied-shaft wall that extends to bedrock appears to be a more feasible method of isolating the wall from the unstable downslope soils.

Stability of Soil Nail Wall at LE 1381+50. A pseudostatic analysis of global stability under seismic loading was performed on a second cross section at LE 1381+50. This cross section was selected to verify the stability results obtained at LE 1379+90. Because seismic loading was found to be the critical loading condition at LE 1379+90, an analysis of stability under static loading was not performed.

For a potential failure starting at the base of the SNW (Wall 7), the calculated pseudostatic factors of safety were 0.73 (high lake level) and 0.82 (low lake level). Table 7.6 presents factors of safety obtained for various cases and the figure numbers of the graphical results for each case. The graphical results are provided in Appendix H.

The analysis results verified the results obtained at LE 1379+90 and confirmed the need to provide for wall stability by either extending the SNW to bedrock or constructing a wall consisting of tied-back shafts that extends to bedrock in front of the SNW.

7.5.5 Stability of West Approach Geosynthetic Wall (Wall 5)

Because the permanent geosynthetic wall supporting the west approach fill is 10 feet high or less, WSDOT has determined that the wall only needs to meet the factor of safety criteria for static loading for an embankment (Golbek 2010b). URS analyzed the stability of the west approach wall (Wall 5) at LE 1371+75 for static loading. Factors of safety ranging from 1.25 to 1.36 were calculated. The results of the stability analyses are presented in Table 7.6.

7.5.6 Stability of East Approach Geosynthetic Wall (Wall 6)

URS analyzed the stability of the east approach permanent geosynthetic wall (Wall 6) at LE 1384+00 under static and seismic loading and at LE 1384+50, where the proposed wall height is less than 10 feet, under static loading only. The analyses presented in this section are for a wall that includes extended reinforcement as recommended in Section 7.7.4.

Factors of safety at LE 1384+00 for static loading ranging from 1.25 to 1.30 were calculated. The minimum acceptable factor of safety is 1.3. For seismic loading, the calculated factors of safety ranged from 0.75 to 0.85, which are less than the minimum factor of safety of 1.1 that is typically considered acceptable for a roadway structure. The results of the stability analyses are presented in Table 7.6.

Because the wall is relatively low (maximum height of 18 feet), the WSDOT Geotechnical Office determined that these factors of safety are acceptable assuming that seismic-induced deformations can be limited to one foot or less (Golbek 2010c). URS calculated seismic-induced deformations

ranging from approximately 0.75 to 1.25 feet. These calculated deformations were interpreted to satisfy the need to limit calculated seismic-induced deformations to one foot or less. The results are provided in Table 7.7.

Factors of safety at LE 1384+50 for static loading ranging from 1.2 to 1.31 were calculated, assuming wall reinforcement is used as described in Section 7.7.4. Because the wall is 10 feet high or less at this location, WSDOT has determined that the wall only needs to meet the factor of safety criteria for static loading for an embankment (Golbek 2010b). These factors of safety are considered to meet the minimum acceptable factor of safety of 1.25 for an embankment. The results of the stability analyses are presented in Table 7.6.

7.6 WEDGE ANALYSIS FOR EXTREME EVENT LIMIT STATE

URS used a wedge analysis to estimate the lateral loads imposed on the bridge pier and tied-shaft wall (Wall 23) shafts by the upslope soil and surcharge loads during the design earthquake. The wedge analysis uses a pseudostatic limit equilibrium approach. The wedge analysis focused on the extreme event limit state (earthquake loading), which resulted in the highest lateral forces imposed on the shafts. The strength limit state under static loading was not evaluated. A detailed description of the wedge analysis is presented in Appendix I of the 2007 Roadway Geotechnical Report (URS 2008a).

The wedge analysis assumes movement of a wedge-shaped block of soil upslope of the shafts. For earthquake loading, the driving forces consist of the soil weight, surcharge load, and a horizontal pseudostatic load resulting from the horizontal ground acceleration. The resisting forces consist of friction along the failure surface, any available lateral resistance of the soil and rock downslope of the shafts, and, where structural elements are included, the resistance of those structural elements.

The calculated lateral loads represent the maximum horizontal and vertical resisting forces needed from the shafts to achieve a pseudostatic factor of safety of 1.1 under the design earthquake loading ($k_h = 0.175$). It is noted that, in terms of slope stability, the wedge analysis was used to effectively isolate and analyze the sliding forces and surcharge loading upslope of the shafts.

It was assumed that the full downslope passive resistance would not be mobilized due to the limited deflection of the shafts and the tendency of the downslope soil to move away from the shafts during shaking. Accordingly, the downslope soil resistance for the downslope (outer) pier shaft was analyzed as an active seismic earth pressure and calculated using a Mononobe-Okabe analysis. Downslope soil resistance was neglected for the upslope (inner) pier shaft.

Consistent with a Mononobe-Okabe (1929) approach to modeling seismic earth loads against retaining structures, the net driving force (sum of the driving forces minus the soil frictional resistance) is modeled as a uniform pressure against the upslope face of the shafts, and the downslope soil resistance is modeled as a triangular pressure distribution against the downslope face of the shafts.

7.7 GEOTECHNICAL RECOMMENDATIONS

This section presents the geotechnical recommendations for design for foundations and walls, including:

- Shallow foundations and abutment wall for Pier 1;
- Drilled shaft foundations for Piers 2 through 9;
- Abutment wall for Pier 9;
- Permanent and temporary SNWs (Walls 7, 7A, and 7B);
- A tied-shaft wall (Wall 23) used to provide for global stability beneath certain sections of Wall 7; and
- East and west approach permanent geosynthetic walls (Walls 5 and 6).

7.7.1 Pier 1 Shallow Foundations and Abutment Walls

Shallow foundations on rock are recommended for support of the west bridge abutment (Pier 1). Because of the slope of the bedrock, it will be necessary to construct a stepped footing.

URS recommends an unfactored bearing resistance of 100 ksf for footings on rock be used for all limit states. This bearing resistance was conservatively selected from Table C.10.6.2.6.1-1 of the AASHTO LRFD Bridge Design Specifications as slightly less than the lower end of the ordinary range for very hard, sound basalt. This bearing resistance may be used over an effective footing width (B') that is equal to the total footing width (B) minus two times the eccentricity (e), so that B' = B – 2e. URS recommends the bearing resistance factors provided in Table 7.8.

The recommended unfactored lateral earth pressures to be used for design are shown in Figures 7.14 and 7.15 for the strength and extreme event limit states, respectively. For the strength limit state, URS recommends load factors for EH equal to 1.5 for flexible walls using active earth pressure and 1.35 for rigid walls using at-rest earth pressure. For live load surcharge (LS), a load factor equal to 1.75 is recommended. For the extreme event limit state, load factors of 1.0 and 0.5 are recommended for the earthquake load (EQ) and LS, respectively.

Lateral foundation loads can be resisted by friction along the base of the footing and by passive resistance on the sides of the footings. For footings cast in place on level rock, the unfactored frictional resistance may be computed using a soil unit weight equal to 135 pcf and a coefficient of friction (tan δ) equal to 0.7. This coefficient of friction may be used if the exposed rock is free of any residual soil or rock debris. Passive resistance may be computed using the passive pressure distributions shown in Figures 7.14 and 7.15 for the strength and extreme event limit states, respectively. The computed resistances must be multiplied by the resistance factors for sliding and passive resistance that are provided in Table 7.8.

7.7.2 Bridge Pier Drilled Shafts

This section presents geotechnical recommendations for the design of shaft foundations at Piers 2 through 9, including bearing resistance, downdrag, input for lateral load analyses, and group effects. Bearing resistance charts were previously provided to KPFF on August 12, 2010.

Bearing Resistance. URS evaluated the unfactored bearing and uplift resistance of 9-foot diameter drilled shafts socketed into bedrock (rock sockets) in accordance with the methods presented in the 2008 AASHTO LRFD Bridge Design Specifications. The shaft diameter in soil will be one foot greater than the rock socket diameter. Bearing resistance for the service limit state was calculated assuming an allowable settlement of one inch.

The drilled shafts should have sufficient embedment into the bedrock to provide adequate "kick-out" resistance to lateral loads. A minimum embedment of 10 feet into bedrock is recommended. However, deeper embedment may be needed to develop adequate passive resistance at specific locations.

Figures 7.16 through 7.23 show the unfactored bearing resistance for 9-foot diameter rock sockets at Piers 2 through 9 for the strength, service, and extreme event limit states as follows:

- Figure 7.16: Pier 2 (outer and inner shaft)
- Figure 7.17: Pier 3 (outer and inner shaft) and Pier 4 (inner shaft)
- Figure 7.18: Pier 4 (outer shaft)
- Figure 7.19: Pier 5 (outer and inner shaft)
- Figure 7.20: Pier 6 (outer and inner shaft)
- Figure 7.21: Pier 7 (outer and inner shaft)
- Figure 7.22: Pier 8 (outer and inner shaft)
- Figure 7.23: Pier 9 (outer and inner shaft)

The bearing resistances were calculated for drilled shafts socketed into a sloping bedrock surface. The rock socket depth in Figures 7.16 through 7.23 is measured from the downslope edge of the shaft.

The full skin and tip resistances are not additive. Figures 7.16 through 7.23 show the unfactored bearing resistance for combined skin friction and tip resistance. For this case, the skin friction is calculated using the residual rock shear strength because the movement needed to mobilize full tip resistance exceeds the movement that will cause yielding in skin friction.

If the factored bearing resistance is greater than the factored loads for the strength limit state, settlement under service conditions should not exceed one inch.

The unfactored uplift for the strength, extreme and service limit states may be taken as equal to the unfactored skin friction. The weight of the shaft has not been deducted from the bearing resistance and has not been included in the uplift resistance. It is recommended that the resistance factors provided in Table 7.9 be used when evaluating the various limit states.

Based on a WSDOT design memorandum dated June 28, 2010, group effects for bearing resistance for shafts in rock do not need to be considered if the drilled shafts are spaced at least two shaft diameters on center. A spacing of less than two shaft diameters on center is not recommended.

Downdrag. For the extreme event limit state, the potential exists for negative skin friction, that is, downdrag, to be imposed on the drilled shafts during the design earthquake. The downdrag loads are due to the potential instability of the soil above the bedrock.

Information for calculating the potential downdrag loads (V_s) is presented in Figure 7.24. A load factor equal to 1.0 is recommended for downdrag loads. The downdrag loads should be added directly to the factored structure loads and the skin friction in the soil above bedrock should be neglected when evaluating the shaft capacity required for the extreme event limit state.

Soil Loads and Rock Resistances for Lateral Load Analysis – Extreme Event Limit State. Lateral soil loads and soil and rock resistances for the extreme event limit state were estimated using a wedge analysis, which is described in Section 7.5. The estimated soil loads per foot measured parallel to the alignment are shown in Figure 7.24. The lateral force, L, must be multiplied by three times the shaft center-to-center spacing to obtain the total lateral soil load per shaft. The total load per shaft should be distributed as a uniform pressure from the top of the shaft to the bedrock surface.

Following the 2008 AASHTO LRFD Bridge Design Specifications Figure 3.11.5.6-2, passive resistance in rock is modeled as a uniform pressure from rock elevation at the downslope edge of the shaft to the tip elevation of the shaft that is equal to the total unfactored lateral resistance divided by the rock socket depth. The total lateral resistance per shaft as a function of rock socket depth is shown in Table 7.10 for 9-foot diameter rock sockets. The rock socket depth in Table 7.10 is measured from the downslope edge of the shaft.

Passive resistance in soil may be included for the outer shaft only. The values of passive soil resistance and the distribution of the passive pressure are shown in Figure 7.24.

Load and resistance factors equal to 1.0 are recommended.

Lateral Load Analysis using DFSAP. URS understands that for bridge design purposes KPFF will evaluate the soil and rock response under lateral loading using strain wedge theory and the computer program DFSAP. Geotechnical design parameters and soil and rock stratigraphy for use as input for lateral load analyses using DFSAP are presented in Tables 7.11 and 7.12. The ground surface slope at the SCB shaft location must also be included as model inputs. For static analysis, a snow load surcharge load equal to 320 psf is recommended. For seismic analysis, the snow load may be neglected.

Groundwater elevations may be considered equal to the lake level, with a maximum groundwater elevation of El. 2517. An analysis using the maximum groundwater elevation will result in the softest foundation response.

A resistance factor of 1.0 is recommended for lateral load analysis. Group effects for lateral load analysis can be evaluated with DFSAP using the shaft group option for shaft groups with and without a shaft cap, as appropriate.

Construction Monitoring. Rock conditions at the bridge site are variable, and URS recommends monitoring and documentation of the conditions encountered in the shaft excavations. A URS representative should be present during excavation of the shafts. If a URS representative cannot be present, detailed logs of the excavations should be provided to URS for review.

URS considers the calculated factor of safety (1.84) for global stability at Pier 7 to be adequate, but recommends monitoring the shaft excavation to confirm that conditions more unfavorable than those assumed in the analyses are not encountered. If unfavorable conditions are encountered, global stability should be re-analyzed. If the calculated factors of safety are found to be unacceptable, alternatives for mitigation can be evaluated at that time. Potential mitigation measures include but are not limited to reinforcing the rock mass with permanent ground anchors or micropiles.

7.7.3 Pier 9 Abutment Wall

The recommended unfactored lateral earth pressures to be used for design of the Pier 9 abutment wall are shown in Figures 7.25 and 7.26 for the strength and extreme event limit states, respectively. For the strength limit state, URS recommends EH load factors equal to 1.5 for flexible walls using active earth pressure and 1.35 for rigid walls using at-rest earth pressure. For live load surcharge (LS), a load factor equal to 1.75 is recommended.

For the extreme event limit state, load factors of 1.0 and 0.5 are recommended for EQ and LS, respectively.

7.7.4 Soil Nail Wall

Global stability analyses of the permanent SNW (Wall 7) at LE 1379+90 and 1381+50 indicate that global stability criteria would not be met unless the wall is founded on bedrock. The stability analyses are described in Section 7.5. In areas of deep bedrock between LE 1378+31.8 and 1381+63, the foundation for the wall is provided by a tied-shaft wall (Wall 23) that is socketed into bedrock. Geotechnical design recommendations for the tied-shaft wall are provided in Section 7.7.5.

A major intent of the permanent SNW (Wall 7) and underlying tied-shaft wall (Wall 23) was to limit lateral loads on the downslope bridge pier shafts due to instability in upslope foundation soils above bedrock under design earthquake loading. Consistent with that intent, soil and surcharge loading from *upslope* of the Wall 7-Wall 23 system were not included in the wedge analysis conducted for the bridge pier shafts to determine downslope lateral loads on the shafts to effect a calculated pseudostatic factor of safety (pFS) of 1.1 under design earthquake loading (i.e., pFS=1.1 for k_h=0.175) for upslope loads. That is, the lateral loads on the shafts did not include loads upslope of the Wall 7-Wall 23 system because those loads are effectively carried by the Wall 7-Wall 23 system.

In order for the Wall 7-Wall 23 system to carry those upslope loads, thus shielding the bridge pier shafts, the SNW and tied-shaft wall work together. To achieve this interaction, the tied-shaft wall was designed assuming that the SNW could effectively limit the lateral forces on the TSW determined using a wedge analysis and an assumed 30-degree failure plane (from horizontal) with a pFS=1.1 (k_h=0.175). The wedge analysis for the tied-shaft wall assumed no passive resistance from downslope soils, equivalent to a “free standing” Wall 7-Wall 23 system (conservatively

consistent with instability in the downslope soils under earthquake loading). The SNW was then designed so that it provided a calculated pFS=1.1 when the calculated lateral forces on the tied-shaft wall were included as an external stabilizing (upslope) force in the SNW stability analysis. Thus, based on the Wall 7-Wall 23 system design, the entire wedge of foundations soil above bedrock and surcharge loading upslope of the Wall 7-Wall 23 system was calculated to be stable with a pFS=1.1 for k_h=0.175. The net result was that the Wall 7-Wall 23 system and the pier shafts stabilize the otherwise potentially unstable foundation soils upslope of the bridge piers, resulting in a consistent, calculated pFS=1.1 for the composite system.

Geotechnical Design Recommendations for Permanent Walls. The design for permanent SNW (Wall 7) should be performed in accordance with the FHWA Geotechnical Engineering Circular #7 - Soil Nail Walls (Lazarte, et al. 2003) and the GDM. For the final design of permanent walls, URS recommends ultimate grout bond strength values of 4.0 ksf in soil and 10 ksf in bedrock. These values are the upper end of the range for soil and the lower end of the range for bedrock that are presented in Lazarte, et al. (2003) for colluvium and basalt, respectively, using tremie/gravity grout. For grout pressures of more than 50 psi in soil, the bond strength has been found to be as high as two times the bond strength for tremie/gravity grout. (Elias and Juran 1991).

The upper range value for soil was achieved in verification tests conducted for the Phase 1B overburden SNW. The upper range value has also been achieved in verification tests conducted in the Seattle area.

SNWs are not addressed by the LRFD Bridge Design Specifications. For evaluation of pullout resistance using ASD, minimum factors of safety equal to 2.0 for static loads and 1.5 for seismic loads are recommended. A total unit weight equal to 135 pcf and a friction angle equal to 40 degrees may be used for the final design. Global stability under static loading should be evaluated for each excavation stage, using a minimum factor of safety of 1.2. Global stability under static loading, including crane loads, should also be evaluated for the critical final excavation stage, using a minimum factor of safety of 1.2. Global stability under static and seismic loading should be evaluated for the final wall configuration, using minimum factors of safety of 1.5 and 1.1, respectively.

Soil nails that will be inundated by the lake should include double (Class I) corrosion protection. Single (Class II) corrosion protection may be used for the temporary SNWs (Walls 7A and 7B). All tendons should be protected from coating damage prior to and during installation.

Recommendations for Temporary Walls. The temporary SNWs (Walls 7A and 7B) should be designed using a soil unit weight of 135 pcf and a friction angle of 40 degrees. For evaluation of pullout resistance using ASD, a minimum factor of safety equal to 2.0 for static loads is recommended.

Soil Nail Installation. As discussed in Section 7.4, the subsurface soil conditions at SCB generally consist of gravel with sand, cobbles, and boulders. The wall excavation face will be susceptible to sloughing. Face stability can be improved by installing vertical nails closely behind the proposed wall face before beginning excavation. The nails should be installed into bedrock or a minimum of 2.5 feet below the base of the wall into soil. The wall face stability can be further improved by applying a 2- to 4-inch-thick "flash coat" of shotcrete to the face immediately after excavation. In this case, the nails would be drilled through the flash coat, and the structural shotcrete would be applied later.

The contractor should be prepared for caving conditions during the soil nail installation. The soil nails will likely need to be drilled with a full-length casing.

Wall Drainage. To reduce hydrostatic pressures on the wall facing, it is recommended that prefabricated drainage strips be placed between the soil face and shotcrete facing. At least one vertical strip with a minimum width of 16 inches should be used between all soil nail columns, extending continuously the full wall height.

Soil Nail Testing and Observation. Field testing of soil nails is necessary to confirm design assumptions and construction procedures. The design values must be confirmed in the field by testing the nails in accordance with the WSDOT Standard Specifications and the general special provisions (GSPs).

Because soil nailing requires specialized installation and earthwork techniques to maintain stable conditions during and after construction, URS recommends that a geotechnical engineer or a construction inspector continuously monitor all construction activities. This would include observation and documentation of construction procedures, construction materials, excavation conditions, soil nail testing, and wall deflection.

URS recommends monitoring of potential wall face movement during and after construction. Wall 7 is an SNW that will be vertical and relatively high (45 feet), will be constructed in difficult soil conditions for nailing, and will permanently support the WB lanes. Portions of the wall will be subject to inundation during high lake level periods. These factors support wall monitoring to provide early detection of movements that may occur, and to observe future trends of such movements. URS recommends monitoring during construction and for two years following completion of construction. The post-construction monitoring would capture the potential effects of inundation on the wall.

7.7.5 Tied-Shaft Wall (Wall 23)

A tied-shaft wall (Wall 23) will be used to provide for global stability under seismic loading of the SNW that supports the WB lanes between LE 1378+31.8 to 1381+63.1. Wall 23 will be located immediately in front of the SNW (Wall 7) and will consist of 3-foot and 5-foot diameter shafts that are socketed into bedrock (rock sockets). The 3-foot diameter rock sockets will be spaced 6 or 9 feet center-to-center. The 5-foot diameter rock sockets will be spaced 10 feet center-to-center. The shaft diameter in soil will be one foot greater than the rock socket diameter. The tops of the shafts will be connected by a continuous grade beam. Each shaft will be tied back with two permanent ground anchors.

Geotechnical design recommendations for tiebacks were presented in TM4 (URS 2008c).. This section presents the following supplemental information for extreme event limit state design of Wall 23:

- Rock socket bearing and lateral resistance. Bearing and lateral resistance recommendations were previously provided to KPFF in emails dated June 28 and December 1, 2010, respectively.

- Lateral soil loads on the shafts. This information was previously provided to KPFF in an email dated June 28, 2010. Bedrock elevations were provided to KPFF on July 20, 2010.

This section also presents:

- Estimated distances from the anchor heads to bedrock, for use in calculating anchor tendon lengths. This information was provided to KPFF on January 17, 2011.
- Recommendations for anchor verification tests.

Rock Socket Bearing and Lateral Resistance. Recommended unfactored bearing resistance for design of 3-foot diameter rock sockets west of LE 1379+20 is presented in Figure 7.27. This information is presented for design of 3-foot, 4-foot, and 5-foot diameter rock sockets east of LE 1379+20 in Figures 7.28, 7.29, and 7.30, respectively.

The total lateral resistance per shaft as a function of rock socket depth is shown in Table 7.10. The rock socket depth in Table 7.10 is measured from the downslope edge of the shaft.

Rock resistance factors equal to 1.0 are recommended for lateral and bearing resistance.

Lateral and Downdrag Soil Loads. Recommended lateral soil loads on the tied-shaft wall (Wall 23) for extreme event limit state shaft design are presented in Figure 7.31. This figure presents the loads in kips per horizontal foot of wall, as estimated using a wedge analysis, as a function of the total height from bedrock to the proposed roadway surface including equivalent surcharge load (one foot). For the purpose of calculating the lateral loads, URS understand that KPFF has added five feet to the values of total height presented in Figure 7.31 to account for potential variability in the bedrock elevation.

A wedge failure plane angle, β, of 30 degrees above horizontal was assumed. The loads shown in Figure 7.31 assume that the soil nails extend far enough to effectively allow the soil above the assumed 30 degree failure plane to act as a unit, such that failure on a plane steeper than 30 degrees should not occur.

The total lateral soil load on any shaft can be calculated by determining the lateral force per horizontal foot of wall (L) for the height from bedrock to the proposed roadway including the equivalent surcharge (H) at the shaft location and computing the product of L and the shaft center-to-center spacing (S). As shown in Figure 7.31, this load should be applied as a uniform pressure from the bedrock surface at the shaft centerline to the top of the shaft cap.

Assuming adequate rock socket embedment, the 3-foot or 5-foot diameter shafts, spaced a maximum of three times the shaft diameter (3B) feet on-center, are expected to provide adequate soil arching and soil-load transfer between the shafts to effect slope stabilization between the shafts.

Downdrag loads (V_s) can be calculated from L and B using the information on Figure 7.31 as:

$$V_s = 0.64L\pi(B/2)$$

Soil load factors equal to 1.0 are recommended for lateral and downdrag loads.

Anchor Tendon Lengths. The contractor will need to procure anchor tendons in advance of drilling the anchors holes. This section provides information for estimating the needed lengths of anchor tendons.

The total anchor length consists of the sum of the distance from the anchor head to bedrock, the no-load zone length, and the bonded length. Table 7.13 provides estimated distances from the anchor head to bedrock. It is recommended that a minimum of 10 feet be added to these estimated lengths to account uncertainty in the location of the bedrock surface.

The estimated distances from the anchor head to bedrock were provided to KPFF on January 17, 2011.

PGA Verification Tests. It is recommended that one verification test be performed on a non-production anchor prior to beginning installation of the production anchors. The test location should be near the center of Wall 23, between LE 1379+50 and 1380+50. The test should be performed in conformance with the requirements of GDM Section 15.5.2.3. The test should be conducted at approximately the same elevations as the production anchors, and the test anchor should be bonded entirely in bedrock. If the test is conducted after construction of the access bench, the soil nail wall must not be used for reaction.

7.7.6 Permanent Geosynthetic Walls (Walls 5 and 6)

Permanent geosynthetic walls are planned for the bridges approaches between LE 1371+10 and 1372+06 on the west end (Wall 5) and from LE 1383+58 and 1385+80 on the east end (Wall 6). URS understands that WSDOT will use a standard plan wall system.

Based on the results of global stability analyses (Section 7.5), extended reinforcement is recommended for the east and west approach walls where the walls are founded on soil to provide stability and limit calculated seismic-induced deformation to one foot or less. URS has developed recommendations for extended reinforcement for walls greater than or equal to 10 feet (Case A) and walls less than 10 feet high (Case B). The recommendations apply to both the west (Wall 5) and east (Wall 6) approach walls. The recommended reinforcement options are shown in Figure 7.32. For Case A, the recommended minimum combined long-term tensile strength of the reinforcement layers in the extended reinforcement zone is 15,000 pounds per foot of wall face. To limit lateral earth pressure on the wall facing, each reinforcement wrap at the wall facing must be designed to provide at least 2,500 pounds of pullout resistance per foot of wall face.

Geotechnical recommendations for wall design were provided to WSDOT on October 6, 2010.

Wall reinforcement that is below the high lake level (El. 2518) is subject to inundation and must be designed considering the potential for corrosion.

7.8 CONSTRUCTION CONSIDERATIONS

7.8.1 Drilled Shafts

On the basis of the currently known soil conditions at the SCB site, it is anticipated that a loose to very dense, poorly-graded to well-graded gravel with sand, cobbles, and boulders will be encountered in most drill holes. This soil will likely yield slow, and possibly difficult, drilling rates with a conventional auger. Although the explorations encountered only occasional cobbles or boulders, a higher percentage of such obstructions could exist.

Because of the proposed large shaft diameters and the presence of boulders, it is recommended that an oscillator-type drill rig be used to construct the shafts. The ADSC also recommended use of an oscillator rig (Macnab 2009). Because of the high permeability and granular nature of the soils, it is anticipated that the shafts cannot be excavated using an open-hole method without casing.

Shafts that will be socketed into bedrock will need to penetrate the bedrock surface that has slopes as steep as 35 degrees. The drilling tools may have difficulty getting a "bite" into the sloping rock surface and could be deflected downslope. The contractor should have appropriate drilling tools for this situation.

Due to the anticipated subsurface conditions and the high probability of caving during tieback installation, the tieback anchors should be drilled with a full-length casing. URS recommends that concrete be placed in the drilled tieback anchor holes by tremie methods.

7.8.2 Construction Access Bench Stability

Construction activities, including shaft construction, will need to be conducted from an access bench that is cut into the existing lake side embankment with side slopes of 1.2H:1V to 1.8H:1V, and an average slope of approximately 1.3H:1V. The distance from the center of the shafts to the edge of the bench is as short as five feet. The contractor will need to evaluate stability during construction activities. Reaction piles may be needed for shaft construction. A requirement that the contractor document the method that will be used to provide stability during shaft construction prior to beginning work should be considered.

7.8.3 Lake Level and Winter Conditions

The lake level and winter conditions will need to be considered in construction planning. The variation in lake levels is described in Section 2.1.3. The construction access bench is as low as approximately El. 2480. During an "average" lake level year, the lake level would be below the lowest bench elevation from mid-August through January of the following year. During the "wet" year of 1996, when lake levels were high for long periods, the lake level was below the lowest wall elevation only from mid-September through mid-November.

The estimated construction work window will be a six month period that falls roughly between mid-April and late October. However, the work window could adversely be impacted by abnormally wet weather which would cause lake water levels to rise faster and stay higher than normal.

The lake and groundwater levels will be above the base of many of the foundation elements; therefore, the potential for corrosion must be considered. Elements subject to corrosion, including wall reinforcement, tiebacks, and soil nails will need to be protected against corrosion or made of non-metallic materials.

7.8.4 Recommendations of the ADSC

WSDOT conducted a meeting with the ADSC in April 2008. The purpose of this meeting was to solicit input from experienced drilled shaft practitioners regarding the constructability of walls and shafts at SCB. The ADSC provided their input in a memorandum to WSDOT dated April 28, 2008 (Macnab 2008), which is provided in Appendix I.

8.0 SLIDE CURVE MEDIAN WALL

URS understands that WSDOT will use a pre-approved or proprietary MSE wall for Slide Curve median wall (SCW; Wall 8), and that the wall design, including internal stability and sliding and overturning stability, has been preapproved or will be performed by the wall vendor. This section presents the results of global stability analyses and provides geotechnical recommendations for design of the wall.

8.1 PROJECT DESCRIPTION

Based on roadway cross sections provided to URS by the WSDOT Project Office in March 2010, SCW is a 4,570-foot long MSE wall between approximately LW 1377+30 and 1423+00. The wall extends from approximately 770 feet west of the east end of SCB to approximately 325 feet west of the west end of RCB. The location of the wall is shown in Figure 2.2B.

The wall will retain the WB embankment fill on the south side, and will elevate the WB lanes up to 50 feet above the EB lanes. The wall will mostly be located near the northern edge of the existing WB roadway. The maximum exposed wall height will be 50 feet between approximately LW 1398+00 and 1410+00.

The primary purpose of the WB grade raise is to reduce the height of the rock cuts at Slide Curve. The wall backfill will use much of the excess rock generated by the rock slope cuts for the project.

Bedrock is relatively deep beneath the wall between LW 1403+00 and 1408+00. The maximum estimated depth from existing grade to bedrock beneath the wall face is approximately 18 feet near LW 1405+50. Foundation concepts to provide global stability of the wall in this deep bedrock area were evaluated during meetings between representatives of WSDOT and URS on December 15, 2008 and February 23 and March 11, 2009. Excavation to found the wall directly on bedrock was determined to be the preferred foundation concept. Other foundation concepts for this deep bedrock area that were considered but not retained include:

- A shear wall of drilled shafts;
- Excavation to bedrock and construction of a concrete or CDF base;
- Stabilization of the entire embankment through construction of a soil buttress in the lake at the toe of the embankment;
- Construction of a foundation of micropiles that are socketed into bedrock; and
- Construction of a grouted foundation (solidification of the soil underlying the wall through injection of grout.

8.2 FIELD EXPLORATION AND LABORATORY TESTING RESULTS

A total of 41 borings have been drilled along or near the SCW alignment, including:

- 17 borings drilled during the 2008 exploration program;
- Nine borings drilled during the 2007 exploration program; and
- 15 borings drilled during earlier investigations, including eleven in 2006, two in 2005, and one each in 1998 and 1997. With the exception of three borings near the SW2 location, these borings are all more than 50 feet away from the wall alignment.

The boring locations are shown in Figures 2.2B. A summary of the SCW borings is provided in Table 8.1. Logs of the borings are provided in Appendix A.

In addition to the borings, a geophysical survey was conducted by NGA in November 2009. The purpose of the survey was to determine bedrock elevations near the wall face between LW 1403+00 and 1409+00. The NGA report is provided in Appendix C.

Laboratory testing consisted of 40 particle-size analysis tests, including:

- Sieve analysis tests (ASTM D-422) of eight samples collected from boring SW-002-07(OW), which is located on the existing EB shoulder at SW2 and
- Percent passing the number 200 sieve for 32 samples collected from borings drilled near the wall face.

The testing results are summarized in Table 8.2. Grain-size curves are provided in Appendix D.

PLTs were conducted in the field on 46 rock specimens from 37 core runs from borings located near the proposed wall to measure the uniaxial unconfined compressive strength of the rock. The results of the PLTs are summarized in Table 8.3.

8.3 SITE CONDITIONS

8.3.1 Site Description

The wall site is located between Keechelus Lake and the Slide Curve rock cut slopes. The existing grade along the wall alignment ranges from approximately El. 2527 to El. 2536. The existing roadway embankment slopes down from its crest adjacent to the EB road shoulder to Keechelus Lake at slopes as steep as 1.25H:1V.

During highway construction in October 1957, a rock slide occurred at Slide Curve that resulted in fatalities. The slide was in part a structurally-controlled rock slope instability that occurred when a strong, jointed bedrock mass at the toe of the slope was damaged by over-blasting.

8.3.2 Soil and Rock Conditions

Five cross sections (Figures 8.1 through 8.5) and one profile along the wall alignment (Figure 8.6) have been developed that show the interpreted subsurface conditions. The five cross sections are at LW 1390+75, 1399+00, 1404+17, 1411+66, and 1419+26. An additional three cross sections have also been developed within the wall area for evaluation of the area referred to as SW2. These cross sections are at WB Stations 1407+15, 1408+49, and 1409+37, and are presented as Figures 9.1 through 9.3.

Soil conditions beneath the wall generally consist of gravel and sand with cobbles underlain by bedrock. The bedrock configuration is interpreted to include nearly level benches of variable width near the existing rock cuts adjacent to the existing roadway and generally steeply dipping toward the lake.

The depth to bedrock along the wall face alignment is generally shallow (≤8 feet deep) from LW 1386+00 to 1403+00 and 1408+00 to1420+00. Between approximately LW 1403+00 and 1408+00, the depth to bedrock is greater, with a maximum interpreted depth of approximately 18 feet below existing grade at LW 1405+50. The depth to bedrock along the wall face increases west of LW 1386+00 and east of LW 1420+00.

Two soil units and one rock unit have been identified.

Unit 1 (Gravel with Sand and Cobbles [Primarily Fill]) consists of fill material that was placed for construction of the existing I-90 roadway embankment. The material consists of loose to very dense, poorly-graded to well-graded gravel with sand and cobbles. The thickness of the unit ranges from approximately 3 to 18 feet along the wall alignment. Measured fines contents ranged from 1.1 to 20.7 percent.

Unit 2 (Gravel and Sand with Cobbles/Boulders) consists of a mixture of colluvium, talus, landslide deposits and glacial deposits that directly underlies Unit 1 along the wall alignment between approximately LW 1402+50 and 1409+00. The thickness of Unit 2 encountered within this interval ranges up to 12 feet. Measured fines contents ranged from 2.6 to 12.0 percent.

Scattered boulders were encountered within Units 1 and 2 in nine of the borings drilled in the wall area. The largest boulder encountered was 2.5 feet in size.

Unit 3 (Bedrock) consists of moderately weak (R2) to strong (R4) lapilli tuff. The average uniaxial unconfined compressive strength measured using the PLT on specimens from 11 borings ranged from 8.1 to 39 ksi.

Rock cuts excavated for past road construction are interpreted to be present from approximately LW 1386+00 to 1403+00 and 1408+00 to 1419+00. Between LW 1388+39 and 1402+38, four paired borings (SCW-002 and -002a-08; SCW-004 and -004a-08; SCW-006 and -006a-08; and SCW-008 and -008a-08) have been drilled that were used to interpret the bedrock surface. Each boring pair consists of an upslope boring close to the proposed wall face and a second boring that is 21 to 23 feet downslope. Bedrock was found to be shallow at the upslope boring near the wall face, which indicates the rock surface has been modified by rock cuts beneath most or all of the wall in this interval. At two boring pairs (SCW-002 and -002a-08 and SCW-006 and -006a-08), the average rock slopes from the upslope to the downslope boring were 1.4H:1V and 1.9H:1V. These relatively steep slopes suggest that the edge of the rock ledge formed by the cuts is close to the proposed wall face.

Localized Strata. Two additional soil strata were encountered at the eastern end of the wall alignment, east of approximately LW 1419+50. One stratum consists of medium dense to very dense, poorly- to well-graded gravel with sand and cobbles. This stratum has a maximum thickness of 18 feet. This stratum differs from Unit 2 in that it is likely of an alluvial or glaciofluvial origin.

The second stratum consists of very loose to loose silty sand and poorly graded sand. This unit was encountered at a depth of 53 feet in SCW-13-08 at the east end of the wall and was present to the bottom of the boring at a depth of 70 feet.

Lake Sediments. In addition to the soil and rock units that were encountered along the wall alignment, lake sediments are present beneath the lake, downslope (south) of the wall alignment. These units are potentially significant for the global stability of the wall. The interpreted lake sediments and bedrock stratigraphy is shown in the cross sections, Figures 8.1 through 8.5. The offshore stratigraphy shown in these cross sections was interpreted from three borings drilled offshore of LE 1379+90, 1406+07, and 1407+49.

The uppermost sediment unit consists of very soft to soft silt and clay, typically with embedded rocks up to boulder size. This unit is interpreted to be present at the surface of the lake bed where the lake bed slope flattens. This unit is interpreted to be up to approximately 40 feet thick.

The lower sediment unit consists of loose to dense gravel and sand with cobbles and boulders. This unit is interpreted to be present from the base of the silt and clay unit to bedrock.

8.3.3 Bedrock Elevations

Bedrock elevations at the wall face from LW 1385+00 to 1421+00 were estimated using the wall face (January 2008) profile and bedrock elevations estimated by NGA between LW 1403+00 and 1409+00 (Appendix C). Both the wall face profile and the geophysical survey line were offset from the wall face alignment. The bedrock elevation at the wall face was estimated using the offset and an assumed bedrock slope. The estimated bedrock elevations, offsets, and assumed slopes are summarized in Table 8.4.

Along the wall alignment, the depth to bedrock is interpreted to range from 3 to 8 feet below the existing grade between the west end of the wall at LW 1384+50 and approximately LW 1403+00 and between approximately LW 1408+00 and 1419+00.

The depth to bedrock is greater between approximately LW 1403+00 and 1408+00, which includes the area where the EB embankment has experience movement and pavement distress (the area referred to as “SW2” from LE 1405+86 to 1408+63; LW 1406+91 to 1409+66). A maximum bedrock depth of approximately 18 feet was interpreted within this interval.

East of LW 1419+50, the interpreted bedrock surface slopes down in the direction parallel to the alignment, resulting in a maximum depth to bedrock of greater than 70 feet below existing grade at the east end of the wall.

Perpendicular to the wall face, the bedrock profile is interpreted to consist of nearly level benches formed by past rock cuts and unaltered rock that slopes down toward the lake. As described in Section 8.3.2, bedrock slopes as steep as 1.4H:1V have been encountered. Between approximately LW 1412+00 and 1419+00, rock outcrops are exposed on the lake side of the roadway embankment, indicating that bedrock is relatively flat-lying perpendicular to the wall face.

8.3.4 Groundwater

One groundwater OW is present within the wall area, SW2-002-07(OW). This well is located on the EB shoulder, near the crest of the embankment slope. Groundwater levels measured in this well indicate that the groundwater level is approximately equal to the lake level at this location. This well was monitored from July 9 to December 12, 2007. It was destroyed during snow removal operations during the winter of 2007-2008 and was not replaced.

The drainage paths along the wall alignment are longer than those at SW2-002-07(OW). As a result, groundwater levels along the wall alignment may be affected by local groundwater tables, and may be higher than the lake elevation. Where rock is shallow, a saturated zone may be present on top of the rock during snow melt, heavy precipitation, and/or high lake levels.

8.4 GLOBAL STABILITY

URS evaluated the global stability of SCW at four locations:

- LW 1398+20 (rock failure only);
- LW 1399+00;
- LW 1404+17 (seismic loading only); and
- LW 1407+15

The analyses at LW 1398+20 assumed the wall was founded on bedrock. The analyses at LW 1399+00, 1404+17, and 1407+15 assumed the wall was founded on soil. One purpose of the analyses at these three locations was to evaluate whether it would be necessary to found the wall on rock, or otherwise provide additional structural support, to provide for global stability of the wall.

Global stability was analyzed at LW 1398+20 by W&N (W&N 2009d; Appendix H) for potential failures through bedrock. At this location, the potential exists for the wall to be subject to unbalanced loading from an upslope, potentially unstable rock mass that is present between approximately LW 1397+25 and 1399+25. Bedrock is close to the existing ground surface, and it was assumed the wall would be founded on bedrock.

URS previously evaluated the stability of SCW at LW 1399+00 and 1407+15 (LE 1406+07) and reported the results in the 2007 Roadway Geotechnical Report (URS 2008a) and TM5 (URS 2008d), respectively. Those analyses, as well as the analyses at LW 1404+17, have been updated using the bedrock elevations and lake sediment stratigraphy and strength properties interpreted from the 2008 borings.

The updated bedrock elevations and lake sediment stratigraphy are shown in Figure 8.2 for LW 1399+00, Figure 8.3 for LW 1404+17, and Figure 9.1 for LW 1407+15. The strength of the lake sediments was characterized using a ratio of the undrained shear strength to the effective overburden pressure (S_u/p') equal to 0.26. The strength properties of the lake sediments were interpreted primarily using back-analysis of the stability conditions at LE 1407+49, as described in Section 9.5.

The results of the global stability analyses are presented in Table 8.5. Graphical results are provided in Appendix H.

8.4.1 Global Stability at LW 1398+20

W&N analyzed global stability for conditions both with and without downslope soil resistance. The latter condition represents a "worst case" scenario that could conceivably result from large embankment deformations under seismic loading. Additional description of the analyses is provided in Appendix H.

W&N analyzed four scenarios:

- Failure surface exploits residual failure surface from 1957 (i.e., structural (block) failure), assuming no loss of downslope support from the embankment soil;
- Structural (block) failure, assuming downslope movement of the unstable embankment soil results in a complete loss of support;
- Material failure (circular failure surface), assuming no loss of downslope support; and
- Material failure, assuming complete loss of downslope support.

The soil stratigraphy and shear strengths beneath and downslope of the wall were assumed to be the same as at LW 1399+00. The bedrock was categorized as Domain 1 (blue), Domain 2 (yellow), or Domain 3 (orange).

Domains 1 and 2 are essentially too strong to fail without persistent, low shear strength, adversely-oriented structures. Although the subsurface exploration downslope of the wall was not deep enough to categorically refute the presence of such features, the proximal geology gives no reason to suspect they exist.

Domain 3 was modeled as a Hoek-Brown material with a non-linear shear strength envelope. The shear strength for Domain 3 is similar to the proposed embankment at the operative normal stresses. The shear strength of the pre-existing residual failure surface was characterized as cohesionless with friction angles in the range of 28 to 35 degrees.

All materials except the gravels in front of the wall were assumed to be subjected to a high perched water table condition termed "transient" that is based on continuous monitoring records and site observation of ephemeral springs. The gravels were assumed to respond to a lower water table controlled by the lake level.

Removal of the embankment soils in front of the wall (shown in white) was simulated by assigning zero shear strength and zero unit weight. Thus the failure surfaces still pass through this material but there is no contribution to stability. The soils were assumed to evacuate down to the clayey silt layer. The results show the ten failure surfaces with the lowest factors of safety.

W&N calculated minimum factors of safety of 2.04 under static loading and 1.31 under seismic loading. The lowest factors of safety were calculated for conditions of no downslope soil resistance and no structural reinforcement of the upslope rock mass (i.e., no dowels).

The results show adequate stability under static and pseudostatic (k_h = 0.175g) conditions for all cases (even with the unlikely superposition of a high transient water table and earthquake loading). The structurally-controlled surface shows the lowest stability. The contribution of the dowels is probably reasonable for the structurally controlled failure but could be compromised by the location of the failure surface for the material failure case. It appears that the presence of Domain 2 rock immediately below the face of the wall is critical to preventing the failure surfaces from migrating deeper. In this regard, a high reliance is placed on information from SCW-006A-08.

8.4.2 Global Stability at LW 1399+00

Stability analyses presented in the 2007 Roadway Geotechnical Report for LW 1399+00 indicated factors of safety ranging from 1.08 to 1.81 for static loading and 0.76 to 1.55 for seismic loading for failure surfaces that intersect SCW (Wall 8). The lower and higher factors of safety were calculated using S_u/p' ratios of 0.3 and 0.6, respectively, for "older lake sediments." An S_u/p' ratio of 0.23 was used for "recent lake sediments."

Using the updated interpretation of the bedrock elevation and lake sediment stratigraphy and strength properties, which are described in Section 8.4, factors of safety ranging from 1.08 to 1.15 for static loading and 0.61 to 0.70 for seismic loading were calculated for failure surfaces that intersect SCW (Wall 8).

The soil strength properties used in the analyses are listed in Table 4.2.

8.4.3 Global Stability at LW 1404+17

Analyses at LW 1404+17 were used to characterize stability near the area of deepest bedrock. Only the critical seismic loading case was analyzed. A factor of safety of 1.0 was calculated for seismic loading at low water levels. Factors of safety ranging from 0.78 to 0.88 were calculated for seismic loading at high water levels.

8.4.4 Global Stability at LW 1407+15

The embankment in the area around LW 1407+15 is known to be marginally stable because of its previous movement. The first step in the stability analysis was to calibrate the analysis to the "known" static factor of safety of 1.0 near the outer edge of the EB lane under existing conditions. The calibrated model was then used to calculate factors of safety for the proposed wall. The methods of analysis and results are described in Section 9 (Slide Curve Embankment Stabilization) of this report.

The analyses indicate factors of safety for the proposed wall and embankment that is founded on soil without structural improvements that are less than 1.5 and 1.1 for static and seismic loading, respectively. The calculated factors of safety for stability of the wall under static loading ranged from 1.33 to 1.35. The calculated factors of safety for stability of the wall under seismic loading (pseudostatic analysis) ranged from 0.66 to 0.81.

Stability analyses presented in the TM5 for LW 1407+15 indicated factors of safety ranging from 1.1 to 1.25 for static loading and 0.63 to 0.67 for seismic loading for failure surfaces that intersect SCW. These factors of safety were calculated using S_u/p' ratios of 0.23 and 0.3 for the "recent lake sediments and "older lake sediments," respectively. Although the updated analyses indicate

higher factors of safety, global stability criteria are not achieved for a wall founded on soil without structural improvements.

8.4.5 Interpretation of Results

Global stability analyses at LW 1399+00, 1404+17, and 1407+15 indicate calculated factors of safety for SCW (Wall 8) for failure surfaces in soil that are less than the target factors of safety of 1.5 and 1.1 for static and seismic loading, respectively. The calculated global stability factors of safety at LW 1398+20 for failure surfaces through rock are significantly higher than the targets. These results indicate that global stability criteria will be achieved if the wall is founded on bedrock, but they will not be achieved if the wall is founded on soil without structural improvements in these areas.

8.5 INTERNAL STABILITY

URS evaluated the internal stability of the wall at LW 1399+00 using the finite difference computer program Fast Lagrangian Analysis of Continua (FLAC). At LW 1399+00 the wall has an "inverted" geometry with reinforcement lengths that increase from 40% of the wall height at its base to 100% of the wall height at its top. The overall purpose of the modeling effort was to assess whether a pre-approved wall system could be used for the inverted wall on a rock foundation with the minimum (3 feet) design embedment.

The assessment was made by comparing the wall reinforcement forces predicted using the FLAC model to those predicted by the coherent gravity method, which is the method typically used by wall vendors for design of inextensible reinforcements. If the reinforcement forces predicted by the FLAC model are less than those predicted by the coherent gravity method, it would be expected that a pre-approved wall design would include adequate reinforcement.

The model was used to predict forces in soil reinforcements placed in layers with a vertical separation of 2.5 feet. Both static and seismic loading were modeled. For seismic loading, a pseudostatic analysis was used, assuming a horizontal acceleration coefficient, k_h, equal to one-half of the PGA ($k_h = 0.175$).

The unfactored reinforcement forces predicted by the FLAC model were generally less than those predicted by the coherent gravity method in all reinforcement layers for both static and seismic loading. The maximum unfactored reinforcement forces predicted by the FLAC model were 4.7 and 7.9 kips per foot for static and seismic loading, respectively. The maximum forces were predicted near the base of the wall. The unfactored reinforcement forces predicted by the coherent gravity method in this layer were 8.1 and 9.1 kips per foot for static and seismic loading, respectively. These results indicate that design of the wall using the coherent gravity method should result in reinforcement that is adequate to resist the forces predicted by the FLAC model.

The predicted reinforcement forces are shown in Figures 8.7 and 8.8 for static and seismic loading respectively. A memorandum with a detailed description of the model is presented in Appendix J.

8.6 GEOTECHNICAL RECOMMENDATIONS

This section presents geotechnical recommendations for foundation types and design of pre-approved MSE wall systems. It is recommended that the wall extend below the frost depth (3 feet) or to a depth of 5 percent of the wall height below the adjacent finished grade, whichever is greater. For wall sections that are founded on rock, it is also recommended that the length of the lowest layer of wall reinforcement be a minimum of 40 percent of the wall height.

8.6.1 Wall Foundation Type

Based on the results described in Section 8.4, the roadway embankment between LW 1377+30 and approximately 1412+00 is not stable under seismic loading. The wall will need to be founded on bedrock or otherwise provided with structural support to provide for global stability under seismic loading.

Between LW 1377+30 and 1385+15, the wall is upslope of the proposed permanent SCB SNW (Wall 7). The SNW will be founded on bedrock and will provide the structural support needed for global stability for SCW (Wall 8) in this area. Therefore, SCW (Wall 8) can be founded in existing soils in this interval. The SCB walls must be designed with a surcharge load that represents the weight of SCW.

Between LW 1385+15 and 1386+00, the wall can be founded on soil with a minimum wall embedment of 3 feet below the adjacent finished grade and a minimum base reinforcement length equal to 70 percent of the wall height. As discussed with WSDOT in a project meeting on September 20, 2010, the wall could experience up to one foot of displacement during the design earthquake, based on analyses conducted for the adjacent SCB east approach wall. The analyses for the SCB east approach wall are described in Section 7.5.

Between approximately LW 1386+00 and 1403+00, SCW (Wall 8) is interpreted to be mostly or entirely above shallow, nearly flat-lying bedrock, as described in Section 8.3.2. Where this is the case, the wall could be founded on existing soils where rock is deeper than 3 feet below the adjacent finished grade. Where rock is shallower than 3 feet below the adjacent finished grade, the wall should be founded on leveled bedrock.

Bedrock is relatively deep between approximately LW 1403+00 and 1408+00, ranging from about 8 to 22 feet below the adjacent finished grade. Because the embankment is not stable under seismic loading, the wall should be founded on bedrock. This could be accomplished by excavating bedrock to provide a level bench that extends back from the wall face a distance of at least 40 percent of the wall height. Alternatively, the wall could be placed on a concrete or CDF base that is placed directly on bedrock and has a top width that is at least 40 percent of the wall height.

Between approximately LW 1408+00 and 1412+00, SCW (Wall 8) is interpreted to be partly on a ledge of nearly flat-lying bedrock. On the lake side of this ledge, bedrock slopes down steeply. Because the location of the upper edge of the sloping bedrock with respect to SCW is uncertain, it is recommended that the wall be founded on rock.

Between approximately LW 1412+00 and 1419+00, the wall is interpreted to be entirely above nearly flatlying rock. The wall can be founded on rock or existing soils in this interval. Where

the bedrock depth is less than 3 feet below adjacent finished grade, the wall should be founded on leveled rock.

From approximately LW 1419+00 to the east end of the wall at LW 1423+00, bedrock dips steeply toward the east, and excavation to bedrock would not be practical. In this interval, it is recommended that the wall be founded on existing soils. The wall height is 15 feet or less in this area.

The recommended foundation types are summarized in Table 8.6.

8.6.2 Soil Parameters for Pre-Approved MSE Wall Design

URS understands that overturning, sliding, and internal stability of the pre-approved MSE wall proposed at SCW (Wall 8) will be evaluated by the contractor's designer. Design parameters for inclusion in the GSP titled *Structural Earth Walls* are provided as follows:

Soil Parameters	Wall Backfill	Retained Soil	Foundation Soil
Unit Weight (pcf)	135	135	135
Friction Angle (deg)	40	40	40
Cohesion (psf)	0	0	0

For the service limit state, the wall shall be designed to accommodate a differential settlement of 0.75 inch per 100 feet of wall length.

For the extreme event I limit state, the wall should be designed for a horizontal seismic acceleration coefficient, k_h, of 0.175 g and a vertical seismic acceleration coefficient, k_y, of 0 g.

For evaluation of sliding on a rock foundation, an unfactored (nominal) coefficient of friction, $\tan\delta$, equal to 0.7 may be used. A traffic surcharge of 250 psf should be added when designing the walls.

The wall system should meet the following minimum requirements. Wall design analyses may indicate more stringent requirements.

- The wall should be placed on a level (in direction perpendicular to the wall face) and firm foundation. Walls can be allowed to slope along their length up to 2H:1V, provided the wall base is stepped.

- The base width of the wall where founded on rock should be greater than or equal to 40 percent of the wall height or 8 feet, whichever is greater. The base width of the wall where founded on soil should be greater than or equal to 70 percent of the wall height or 8 feet, whichever is greater.

- The top reinforcing layer should be placed no lower than 2 feet below the top of the wall.

- Wall embedment should be at least 3 feet, which is the frost depth, or 5 percent of the wall height, whichever is greater, as described in LRFD C11.10.2.2 for non-abutment walls.

- Provisions for permanent control of subsurface water behind the wall should consist of a slotted drain pipe embedded in gravel backfill for drains (WSDOT Standard Specification Section 9-03.12(4)).

8.6.3 CDF or Concrete Wall Foundations on Shallow Bedrock

An alternative to excavating bedrock to provide a level foundation on rock is to excavate soil to expose bedrock and construct a level pad of concrete or CDF. This approach is recommended if a base width of greater than 40 percent of the wall height is needed for wall sections founded on rock between LW 1403+00 and 1408+00, unless a wider excavation is approved by a licensed professional engineer or engineering geologist with expertise in rock slope cuts. The stability of the planned rock cuts has not been evaluated for the case where rock excavation for the wall foundation is wider than 40 percent of the wall height.

This section provides parameters for design of a concrete or CDF pad to resist sliding. The wall foundation is assumed to consist of a prism of concrete or CDF placed on a prepared sloping bedrock surface. The upper surface of the prism is assumed to be flat with a minimum width of 0.4H, where H is the wall height. The interpreted bedrock surface slope is shown in the cross sections, Figures 8.1 through 8.5.

Sliding of the CDF or concrete base will be resisted by friction between the base and bedrock and passive soil resistance. Friction coefficients for evaluating sliding of the CDF or concrete base on bedrock and the wall backfill on the CDF or concrete base are provided below.

Condition	Sliding Coefficient	Resistance Factors	
		Strength	Extreme
Concrete or CDF on bedrock	0.7	0.8	0.9
Wall backfill on CDF or bedrock	0.5	0.9	0.9
Wall backfill on concrete	0.45	0.9	0.9

Foundation soil parameters to use to calculate passive resistance are provided in Section 8.5.2. The excavation in front of the base must be backfilled with compacted granular fill to develop full passive resistance.

If adequate sliding resistance for concrete or CDF on bedrock is not provided by the combined friction and passive resistance, dowels may be used to provide additional resistance. The dowels may be designed using a unfactored grout-to-rock bond strength of 36 ksf. A resistance factor of 0.5 is recommended for the grout-to-rock bond strength.

8.6.4 Earthwork

Existing granular soils can be used for wall backfill if they conform to WSDOT Standard Specifications 9-03.14(1) and 9-03.14(4).

The existing gravel and sand soil is considered a Type C soil under Occupational Safety and Health Administration (OSHA) and Washington Industrial Safety and Health Act (WISHA) regulations for trenching and shoring. Temporary cuts above the groundwater table and no greater than 20 feet deep should be no steeper than 1.5H:1V. A steeper slope may be used if designed by a licensed professional engineer. Surface compaction of the slopes may be needed to

prevent localized sloughing. If groundwater seeps from the slope during excavation, the slopes should be flattened to 2H:1V. For open cuts at the site URS recommends:

- No traffic, construction equipment, stockpiles, or building supplies be allowed at the top of the cut slopes within a distance of at least 10 feet from the top of the cut.
- Exposed soil along the slope be protected from surface erosion using waterproof tarps, Visqueen, or flashcoating with shotcrete.
- Construction activities be schedule so that the length of time the temporary cut is left open is reduced to the maximum practical extent.
- Surface water, including stormwater or meltwater runoff, be diverted away from the excavation.

The general condition of the slopes should be observed and evaluated periodically by a geotechnical engineer to confirm adequate stability, particularly immediately after storm events.

8.6.5 Wall Monitoring

URS recommends that WSDOT monitor wall face movement in the vicinity of WB Station 1399+00. The wall in this area was modeled using the computer program FLAC, as described in Section 8.5. The measurements could be used to verify the accuracy of the model results. The monitoring would also provide early detection of movements that may occur, and provide information on future trends of such movements. URS recommends monitoring during construction and for at least one year following completion of construction.

8.7 CONSTRUCTION CONSIDERATIONS

Scattered boulders to 2.5 feet in size were encountered in boring drilled near the wall alignment, and larger boulders may be present. The contractor should have equipment capable of excavating boulders.

URS understands there will be approximately 15 feet available between detour barrier and wall face. Where adequate space is not available to slope a temporary excavation to its base, shoring will be required. The site soils typically consist of gravel with sand, cobbles, and scattered boulders. The type and method of installation of shoring should be appropriate for these soil conditions.

Bedrock elevations may vary significantly over relatively short distances. In some areas, bedrock may be shallower than estimated, resulting in additional rock excavation to achieve the minimum wall embedment. Where rock is deeper than estimated, deeper excavations with longer wall reinforcement or other foundation design modifications may be needed to meet stability requirements.

9.0 SLIDE CURVE EMBANKMENT STABILIZATION

9.1 PROJECT DESCRIPTION

The EB lane of I-90 between LE 1405+86 and 1408+63 near Slide Curve has experienced settlement and pavement distress along an approximately 277-foot long section due to creeping of the embankment toward Keechelus Lake. This section is referred to as SW2 for the purposes of this report. The location of SW2 is shown in Figure 2.2B.

The purpose of this section is to evaluate the feasibility of placing a soil buttress at the toe of the embankment slope to stabilize the slope. The weight of the slope buttress would provide a resisting force that would counteract the downslope force of the unstable soil. The feasibility evaluation consists of slope stability analyses using soil stratigraphy and strength properties that have been revised based on the findings of the 2008 geotechnical investigation.

WSDOT has replaced or patched the pavement several times along this section and has installed instrumentation to monitor ground movement. The following embankment slope stabilization conceptual design options were previously evaluated (URS 2008d; URS 2008i):

- Soldier pile wall with tiebacks;
- Drilled shaft wall with tiebacks;
- Ground anchors; and
- Slope buttress.

Among the slope stabilization design options considered, WSDOT selected the slope buttress option for further evaluation because of its potentially lower cost compared to other stabilization options. An option of continued monitoring and maintenance is also being considered.

Due to the potential environmental impacts and costs associated with the slope buttress option, WSDOT is also considering the option of continued monitoring and maintenance of the EB lanes at SW2 without stabilization. Evaluation of this option is not within the scope of this report.

9.2 FIELD EXPLORATION

A total of 17 borings have been drilled near SW2, including:

- Four borings drilled during the 2008 exploration program;
- Five borings drilled during the 2007 exploration program;
- Five borings drilled during the 2006 exploration program; and
- Three borings drilled during earlier investigations, including one each in 2005, 1998 and 1997.

The boring locations are shown in Figure 2.2B. A summary of the borings is provided in Table 9.1. Logs of the 2008 borings are provided in Appendix A. Logs of previous borings are presented in TM5 (URS 2008d).

The purposes of the 2008 borings were to assess the stratigraphy and the strength and consolidation characteristics of the lake sediments and to assess the bedrock depths beneath the embankment and lake bed.

The boring depths ranged from 20 to 124 feet below the existing ground surface. Borings SW2-007-08 through SW2-009-08, H-30-06 and H-33-06 were drilled from a barge in Keechelus Lake. Most borings, with the exception of SW2-008-08, SW2-002-07 and H-1-97, extended to bedrock.

9.3 LABORATORY AND FIELD TESTING

Laboratory testing consisted of index, strength and consolidation tests on selected samples from the borings drilled at the SW2 location. Laboratory data sheets for 2008 tests are provided in Appendix D of this report and prior to 2008 tests in Appendix B of TM5 (URS 2008d).

9.3.1 Index Tests

Index testing was conducted at the URS laboratory and consisted of grain-size distribution tests on 18 samples, Atterberg limits tests on five samples, an organic content test on one sample and water content tests on 21 samples. Index testing results are summarized in Table 9.2.

9.3.2 Strength and Consolidation Tests

Strength and consolidation tests were conducted by WSDOT and consisted of CU and consolidation tests on two samples. The tests are described in Section 3.3. The testing results are presented in Table 3.7. Interpreted properties are presented in Section 4.3 and Table 4.2.

9.3.3 Rock Strength Testing

Field testing consisted of PLTs on a total of four rock core specimens from lake borings SW2-007-08 and SW2-008-08 to measure the uniaxial unconfined compressive strength of the rock. The rock core PLT results are summarized in Table 3.10.

9.4 SITE CONDITIONS

9.4.1 Site Description

The SW2 site is located at Slide Curve along the north shore of Keechelus Lake. Slide Curve is a section of I-90 that has been prone to rock slope instability above the highway.

Within the bounds of SW2, I-90 consists of two EB and two WB lanes. The north side of highway consists of rock slopes, extending upwards from the highway. The south side of the highway consists of an embankment fill slope that extends downward to Keechelus Lake, with an inclination varying from approximately 1.2H:1V to 1.8H:1V above the gently sloping lake bottom. Portions of the slope are as steep as 1H:1V. The vertical height of the embankment slope is up to approximately 90 feet.

The water level in Keechelus Lake fluctuates seasonally. Keechelus Lake fluctuations are described in Section 2.1.3.

9.4.2 Geologic Setting

The geologic regime and recent construction activities in the vicinity of SW2 contribute to complex subsurface conditions. The bedrock in this area is comprised of lithified pyroclastic flows (typically heterogeneous and massive), which have undergone tectonic uplift resulting in faulting and folding. Subsequent glacial scouring has removed much of the weaker material, leaving a generally open parabolic-shaped valley with localized variability in the bedrock topography. The valley is now occupied by Keechelus Lake.

During highway construction, significant volumes of fill were placed in the SW2 vicinity, likely overlying either bedrock or loose colluvial deposits. Heavy blasting of the bedrock between LE 1406+00 to 1410+00 has significantly altered the topography in the vicinity of Slide Curve.

9.4.3 Subsurface Conditions

Three profiles and three cross sections have been developed that show the interpreted subsurface conditions. Cross sections 1-1' through 3-3' were developed at locations where the combination of slope height and steepness, lake sediments, and depth to bedrock were interpreted to be most critical for global stability, and are presented in Figures 9.1 through 9.3. Profiles were developed along edge of the EB lane and the LE and LW lines, and are presented in Figures 9.4, 4.1B, and 4.2B, respectively. EB stationing is used for all stationing references on all the profiles and cross sections, except for the WB profile. WB stationing is only used for the WB profile. Four soil units and one rock unit were encountered at the SW2 site.

Unit 1 (Gravel with Sand/Cobbles/Boulders [Primarily Fill]) consists of fill material that was placed for construction of the existing I-90 roadway embankment. The material consists of loose to very dense, poorly to well-graded gravel with sand, cobbles, and boulders. Boulders were encountered in SW2-002-07(OW), SW2-004-07, H-32-06, and H-1-05. The boulder size encountered in these borings was not reported in the logs. The thickness of the unit ranges from approximately 27 to 50 feet along edge of the LE line. The unit is thickest at the west end of the pavement distress area at approximately LE 1405+86 and gradually becomes thinner from west to east along the alignment between LE 1405+86 and 1408+63.

Unit 2 (Gravel and Sand with Cobbles/Boulders/Rock Fragments) consists of a mixture of colluvium, talus, landslide deposits and glacial deposits. The material consists of medium dense to very dense gravel with sand, cobbles and boulders. Boulders were encountered in SW2-007-08, SW2-002-07(OW), SW2-004-07, H-30-06, H-16-98, and H-1-97. The thickness of this unit ranges from 0 to 36 feet along edge of the EB lane. This unit is thickest at the west end of the pavement distress area at approximately LE 1405+86 and gradually becomes thinner from west to east along the alignment between LE 1405+86 and 1408+63.

Unit 3 (Clay/Silt [formerly Recent Lake Sediments]) consists of very soft to soft silt, elastic silt or clay at the toe of the embankment slope. Coarse-grained inclusions up to boulder size were encountered in the clay/silt matrix. This unit was encountered in SW2-008-08 and SW2-009-08, which were drilled in the lake, at El. 2337 and El. 2323, respectively. The thickness of this unit is estimated to range from approximately 8 to 62 feet. This unit was also identified during a geophysical investigation conducted by S&W (S&W 1974) in the vicinity of SW2. S&W identified a “disturbed area” of very soft lake sediments at the toe of the slope where fill has displaced the sediment and formed a “mud wave” on the lake bottom (Unit B in the S&W report).

Unit 4 (Gravel/Sand [formerly Older Lake Sediments]) consists of loose to dense sand, sandy gravel and silty sand. This unit underlies Unit 3 at the toe of the embankment slope. In general, the density of this unit is lower and particle size is finer than that of Unit 2. This unit was encountered in SW2-008-08 and SW2-009-08 at El. 2312 and El. 2284, respectively. SW2-008-08 was terminated in this unit. SW2-009-08 was advanced beneath this unit into bedrock. The thickness of this unit is estimated to range from 0 to at least 140 feet as shown on Figure 9.1. This unit was also identified during the S&W geophysical investigation and is considered a part of the "disturbed area" by S&W (Unit C of the S&W report).

Unit 5 (Bedrock) consists of andesite, lapilli tuff, and dacite breccia of the Naches formation. The elevation of bedrock is highly variable both parallel and perpendicular to SW2. The top of bedrock along the edge of the EB lane ranges from about El. 2455 to El. 2500. The RQD of the bedrock ranged from 0 to 100 percent, and typically ranged from 20 to 100 percent. The rock was generally classified in the field as slightly to highly weathered, very weak (R1) to very strong (R5) rock. The bedrock surface was interpreted based on the information from the borings and the bedrock study that was discussed in TM5 (URS 2008d).

9.4.4 Groundwater

One groundwater OW (SW2-002-07(OW)) was installed at SW2 as discussed in Section 3.2.8. The well was monitored from July 9 to December 12, 2007. It was destroyed during snow removal operations during the winter of 2007-2008 and was not replaced.

The OW measurements indicate that the groundwater level is influenced by the lake level. In general, groundwater levels are higher during the summer months and lower during September and October.

9.4.5 Observed Slope Movements

Inclinometers. Four inclinometers, H-1-05(I), H-27-06(I), H-29-06(I) and SW2-006-08(I) were installed at SW2 to monitor the ground movement. A fifth inclinometer, H-1-97(I), was installed but not used because it did not appear to be functioning properly. The inclinometer locations are shown in Figure 3.6. Plots of the data obtained from these inclinometers are provided in Appendix D.

Inclinometer H-1-05 has shown horizontal movement near the soil-bedrock interface, which is approximately 90 feet deep at this location. This inclinometer shows that a downslope movement of about 0.5 to 0.8 inch has occurred above a depth of 90 feet over approximately five years. Readings taken after September 2008 have indicated an accelerated rate of movement above a depth of about 10 feet, with a total movement of about 2.25 inches.

Cumulative movement in the remaining three inclinometers has been 0.25 inch or less.

Deformation Monitoring Points. Fifteen deformation monitoring points (DMP) were installed at a 100-foot spacing between approximately LE 1400+10 and 1414+10 on the EB shoulder around Slide Curve as discussed in Section 3.2.8.

The DMP settlement profile is shown in Figure 3.7. The settlement profile is shaped as a trough, with observed settlements of up to 0.8 inch at LE 1408+00 over the period August 2006 to September 2010. This deformation trend is consistent with the inclinometer H-1-05, which is

offset about 100 feet south from the LE line near LE 1407+60. Upward movements of 0.2 inch have been measured at the easternmost and westernmost DMPs. This movement is greater than the survey accuracy (0.06 inch).

The rate of settlement at LE 1408+00 has been generally consistent over the four-year monitoring period, averaging 0.2 inch per year. A total settlement of 0.2 inch or more has been measured over an approximately 700-foot long zone from LE 1403+00 to 1410+00.

9.5 GLOBAL STABILITY

URS previously evaluated the stability of SW2 at LE 1406+07, 1407+49, and 1408+42 and reported the results in TM5 (URS 2008d). This section describes stability analyses using stratigraphy (bedrock inclination and Unit 3 extent) that was updated based on the 2008 boring results. The embankment at LE 1407+49 was selected as the critical cross section based on the revised stratigraphy.

This section describes the material parameters of the soil units; stability analyses of the existing embankment slope, embankment slope from the base of SCW (Wall 8), and the embankment slope with a slope buttress; and the configuration of the buttress.

Slope stability analyses described in this section consists of the following:

- **Material Parameters and Calibration:** Calibration of the strength parameters for Unit 3 required to achieve a static factor of safety of 1.0 for the existing conditions;
- **Existing Global Stability:** Calculation of static and pseudo-static factors of safety for the existing slope based on the calibrated material parameters;
- **Base of SCW (Wall 8):** Calculation of static and pseudo-static factors of safety of the existing slope from the base of SCW (Wall 8) based on the calibrated material parameters;
- **Proposed Buttress-Long-Term Conditions:** Evaluation of long-term (with pore pressures dissipated) static conditions to determine the size of the buttress; and
- **Proposed Buttress-Short-Term Conditions:** Evaluation of short-term (no pore pressure dissipation) static conditions to evaluate any construction staging limitations.

Table 9.3 presents factors of safety obtained for various cases and the figure numbers of the graphical results for each case. The graphical results are provided in Appendix H.6. Interpretation of these results represents the primary basis of the global slope stability discussion in this section.

9.5.1 Material Parameters and Calibration

Material Parameters. For the slope stability analyses purposes, Units 1, 2, and 4 were assumed to be free-draining and cohesionless, with effective angles of internal friction of 40, 40, and 30 degrees, respectively.

The Unit 5 (bedrock) surface may be a rock failure plane, which may also explain the slope creep observed at SW2. An effective angle of internal friction of 36 degrees was estimated for the interface between the gravels and rock. The interface friction angle was selected considering the relative smoothness of a former rock failure plane.

Unit 3 is characterized as normally consolidated, non-free-draining cohesive silts and clays under existing conditions. The geometry and layer thickness of this unit were interpreted based on the 2008 geotechnical investigation. Based on the limited data available, there is uncertainty in the extent of this unit.

For the slope stability analyses, a SHANSEP approach was used to characterize the undrained (total stress) strength of Unit 3 (Ladd 1974; Kulhawy 1990; Duncan 1989). The SHANSEP approach characterizes S_u in terms of p' using a constant value of the ratio S_u/p'. For normally consolidated soil, p' is equivalent to the vertical effective overburden stress (σ'_{vo}).

Calibration. A back analysis was performed on Unit 3 soils because of uncertainty in the accuracy of the CU lab test results and hence uncertainty in the Su/p' ratio determined based on the CU lab tests. The purpose of the back analysis was to determine an Su/p' ratio for Unit 3 soils that is adequate to achieve a factor of safety of 1.0 for the existing slope at low water for a failure surface that intersects the roadway near the outer EB lane where the pavement distress has occurred. An Su/p' ratio of 0.26 was obtained based on this analysis, which is within the high range of the CU lab test results and is also consistent with Su/p' ratio for normally consolidated soils reported by the Electric Power Research Institute (EPRI) (Kulhawy and Mayne 1990, p. A-4).

Pore Pressure Coefficient. A pore pressure coefficient, B-bar, which is the ratio of excess pore pressure to the increase in major principal stress, was used to calculate the excess pore pressure in Unit 3 soils. In Slope/W, the major principal stress is approximated as overburden stress because in many situations, the major principal stress is near vertical.

For short term cases (no pore pressure dissipation), a B-bar value of one (B-bar = 1.0) was used because excess pore pressures will be generated as a result of placement of the buttress fill. The value of B-bar was selected such that the value of S_u for Unit 3 was the same immediately after as before construction.

For long-term cases (after the pore pressures have dissipated), a B-bar value of zero (B-bar = 0) was used. The value of B-bar was selected such that the value of S_u for Unit 3 increases as a result of the buttress fill after the pore pressures have dissipated.

9.5.2 Existing Global Stability

As discussed in Section 9.5.1, a back analysis was performed for the existing conditions to achieve factors of safety close to 1.0 to be consistent with the slope movement indicated by the pavement distress and the inclinometer measurements. The calculated static factors of safety for the existing embankment slope are 1.00 (low lake level) and 1.05 (high lake level). Under earthquake loading, the calculated pseudostatic factor of safety is 0.64 (low lake level).

In all the cases, the critical failure surface intersects the roadway near the outer EB pavement edge and coincides with a portion of the soil-rock interface. The failure surface passes through Unit 3, which acts as the foundation for the upper slope, and is critical to slope stability at the site.

The low water and high water cases were critical cases under static conditions and seismic conditions, respectively.

9.5.3 Base of SCW (Wall 8)

The calculated static factors of safety were 1.05 (low lake level) and 1.11 (high lake level) for a potential failure of the existing embankment slope starting at the base of SCW (Wall 8). Under earthquake loading, the calculated pseudostatic factors of safety were 0.64 (low lake level).

For all slope stability cases where the potential failure surface starts at the base of the MSE wall, the critical failure surface coincides with a portion of the soil-rock interface and passes through Unit 3. The low water and high water cases were critical cases under static and seismic conditions, respectively.

9.5.4 Proposed Buttress - Long-term Conditions

As discussed in Sections 9.5.2 and 9.5.3, the factors of safety for the existing embankment slope do not meet the WSDOT GDM criterion of a minimum static factor of safety of 1.25 for embankments not supporting structures and indicate a potential for loss of downslope support for Wall 8. Based on the potential loss of downslope support, Wall 8 has been designed to be supported on stable bedrock in the SW2 area.

Based on the analyses results discussed in Sections 9.6.2 and 9.6.3, the lake side embankment will continue to creep toward the lake if no improvements are made to resist the movement. Placement of a buttress at the base of SW2 embankment slope is one method to mitigate slope creep and achieve the minimum required factors of safety.

A series of trial buttress configurations were analyzed for slope stability assuming that SCW (Wall 8) would be founded on bedrock (i.e., not supported by the embankment). A buttress configuration of an approximately 35-foot thick, 210-foot wide bench and 3H:1V outer slope, as shown in Figures H.6.7 and H.6.8, satisfied the minimum factor of safety of 1.25 requirements for an embankment not supporting structures.

For long term cases, the calculated static global factors of safety for the buttress were 1.28 (low lake level) and 1.43 (high lake level). The local static factors of safety for the outer buttress slope were 1.56 (low lake level) and 1.55 (high lake level). The calculated static factors of safety meet the minimum required factors of safety for an embankment not supporting structures. The critical shear surface associated with global stability passes through Unit 3, and could affect the stability of SCW (Wall 8) if the wall is founded on bedrock.

The minimum required static factor of safety for embankments supporting a structure may be achieved if the buttress configuration is enlarged. However, Unit 3 has the potential to liquefy during the design earthquake based on the limited data available. Therefore, the minimum seismic factor of safety would be difficult to achieve because of the liquefaction potential of Unit 3.

Therefore, if the buttress option is selected for mitigating slope creep, the SCW (Wall 8) foundation should be deepened and founded on bedrock in this area to reduce the potential for any instability of the embankment slope to affect the wall.

9.5.5 Proposed Buttress - Short-term Conditions

Construction of the buttress in a single lift was analyzed assuming no dissipation of excess pore pressure generated by addition of the buttress fill. This case was assumed to represent the critical condition for stability of the buttress. It was assumed that there will not be any strength gain in Unit 3 soils as discussed in Section 9.5.1.

For short term cases, the calculated global static factors of safety for the buttress are 1.18 (low lake level) and 1.29 (high lake level). The local static factors of safety for the outer buttress slope are 1.07 (low lake level) and 1.05 (high lake level).

The calculated factors of safety for short term cases are approximately 1.2 to 1.3. It was conservatively assumed in the analyses of these cases that there will not be any strength gain in Unit 3 as a result of placement of buttress fill. In reality, there will be some amount of strength gain and therefore the factors of safety would be higher. Therefore, factors of safety close to 1.2 are considered acceptable for short term cases. Further, for localized instabilities, factors of safety between 1.0 and 1.1 are considered acceptable.

The results indicate that the buttress can be constructed in one stage without endangering buttress stability during or immediately after construction. Using sediment parameters from two consolidation tests (Table 3.7), settlement calculations indicate potential sediment settlement of up to 2.5 feet. This could cause potential deformation in the upslope roadway embankment due to the settlement of the buttress. This potential deformation has not been analyzed in detail.

9.6 CONCLUSIONS AND RECOMMENDATIONS

Stabilization of the Slide Curve embankment using the slope buttress option appears to be feasible. However, the following environmental and maintenance considerations would also have to be adequately addressed before buttress construction could be considered feasible.

Environmental considerations include, but are not limited to, wetland mitigation, water quality, permitting and biological assessment. These considerations would need to be fulfilled for construction of the buttress. The permit requirements may limit the allowable construction season and require water quality monitoring and turbidity controls during construction.

Maintenance considerations include roadway maintenance due to potential deformation in the upslope roadway embankment due to the buttress settlement.

Moreover, the factors of safety that are presented in Sections 9.5.4 and 9.5.5 for the buttress are highly dependent on the lateral and vertical extent and shear strength characteristics of Unit 3, which provides the foundation for the buttress and upper slope below the roadway embankment. The uncertainty in the unit extents will not affect the feasibility of this option. However, it affects the size of the buttress, number of construction stages and settlement of the buttress.

If the slope buttress option is chosen, the width of the buttress for the embankment slope stabilization should be at least 210 feet. The height should be approximately 35 feet, and the top of the buttress should be no lower than El. 2365. The buttress outer slope should be at least as steep as 3H:1V for the unit extents chosen for the analyses presented in Section 9.5. If an outer buttress slope at least as steep as 3H:1V cannot be constructed, additional material should be added to provide adequate buttress weight. The estimated volume needed to construct a 210-foot

wide buttress is approximately 115,000 cubic yards, including an estimated 2.5 feet of consolidation of the underlying lake sediments.

The analyses indicate the slope buttress could be constructed entirely below the elevation of the dam outlet. In this case, the buttress would not affect the useable reservoir storage. Keechelus Lake elevations should be considered in planning the buttress construction. Keechelus Lake elevations are discussed in Section 2.2.3.

Buttress material can be placed from a barge and can be placed in one stage. The material should be relatively clean, self-compacting granular material, such as shot rock. The material should be placed along the longitudinal alignment of the proposed buttress in less than 5-foot thick lifts. A lift should be started only after the previous lift along the entire length of the buttress is completed.

Buttress construction should be carefully and frequently monitored and controlled to assure that stability is not being unacceptably compromised. The monitoring should include, but is not limited to the following:

- Monitoring of the existing DMPs and inclinometers;
- Installation and monitoring of additional DMPs and inclinometers;
- Monitoring of the material placement; and
- Monitoring of the buttress and lake bed sediments movement.

Material placement, buttress movement and lake bed sediment movements beyond the toe of the buttress should be monitored using high-resolution hydrographic surveys. Compromises to buttress stability during or shortly after construction could adversely impact upslope embankments, including an under-construction or completed SCW (Wall 8).

10.0 RESORT CREEK WALL

Resort Creek wall (RCW) was initially proposed to be a 535-foot-long MSE wall between approximately LW 1420+50 and LW 1425+85 near the west abutment of the WB RCB. The wall was to support the north side of the WB embankment. The location of the wall is shown in Figure 2.2C.

However, it was found that RCW would require mitigation measures to provide for stability under earthquake loading. Because construction of the mitigation measures would impact wetlands north of the wall, RCW was eliminated and replaced by a lake side embankment as part of the August 2009 roadway realignment.

This section provides a concise summary of geotechnical analyses conducted by URS prior to elimination of RCW. Geotechnical recommendations for design of the lake side embankment are provided in Section 13.

Prior to the decision to eliminate the wall, several options for mitigation of the instability under earthquake loading were evaluated and discussed at project meetings on February 23 and March 11, 2009. These options included the following.

- Ground improvement using stone columns or compaction grouting;
- Lightweight fill; and
- A shear wall consisting of driven H-piles, pipe piles, or concrete piles.

Global stability analyses for mitigation using compaction grouting are presented in Section 10.4.

10.1 PROJECT DESCRIPTION

Based on the roadway cross sections that were current as of January 19, 2009, the maximum planned wall height was approximately 30 feet. The minimum base of wall elevation was approximately El. 2504 near LW 1422+00. The proposed wall would have raised the existing road surface grade by as much as 15 feet.

The wall would have overlapped with SCW (Wall 8) between LW 1420+50 and 1423+50, where the WB lanes would have been supported by MSE walls on both the north and south sides. Within this section, SCW has a maximum height of approximately 12 feet at the west end of RCW and slopes down to zero height at LW 1423+50.

10.2 FIELD EXPLORATION RESULTS

A total of five borings have been drilled along or near the former RCW alignment, including:

- Three borings drilled during the 2008 exploration program and
- Two borings drilled during the 2007 exploration program

A summary of borings for RCW is provided in Table 10.1. Figure 2.2C shows the locations of the borings. Logs of the borings are provided in Appendix A.

PLTs were conducted on four rock specimens. The test results are provided in Table 10.2.

10.3 SITE CONDITIONS

The former RCW site is located near the mouth of Resort Creek valley. The existing Resort Creek crossing consists of a 7-foot diameter culvert at approximately LW 1423+00. Currently, the roadway design calls for the Resort Creek channel to be relocated east of the former RCW location.

The existing roadway consists of EB and WB lanes that are separated by a narrow median area. The roadway is elevated approximately 25 to 30 feet above the Resort Creek floodplain on the north (wall) side. The surface elevation of the Resort Creek floodplain north of the wall ranges between approximately El. 2494 and El. 2498.

The existing road surface grade at the former RCW site ranges from approximately El. 2525 to El. 2528. At high lake levels, backwater conditions exist in Resort Creek at the former RCW site, and the WB embankment slope is partially submerged. Refer to Section 2.2.3 for a discussion of lake level fluctuations.

The area immediately north of the wall has been classified as a wetland. WSDOT rated this wetland as Category II based on the *Washington State Wetland Rating System for Eastern Washington* (Ecology 2004).

10.3.2 Subsurface Conditions

Two cross sections (LW 1421+99 and 1424+97, Figures 10.1 and 10.2) and one profile along the former RCW alignment (Figure 10.3) have been developed that show the interpreted subsurface conditions. Three soil units and one rock unit have been identified:

Unit 1 (Gravel with Sand/Cobbles/Boulders [Fill]) consists of loose to very dense, poorly- to well-graded gravel with sand, cobbles, and occasional boulders. This material is fill placed for construction of the existing roadway embankment. The thickness of this soil unit along the wall alignment ranges from approximately 10 to 25 feet. Boulders were encountered in two borings:

- RCW-001-08 between approximately El. 2501 and El. 2511 and
- CUL-009-07 at approximately El. 2504

The largest boulder encountered in the borings was 15 inches in size.

Unit 2 (Gravel with Sand/Cobbles [Native Soil]) consists of loose to very dense, poorly- to well-graded gravel with sand and cobbles. Unit 2 was the deepest stratum penetrated by the borings near the wall alignment. The interpreted elevation of the base of this unit ranges between approximately El. 2470 and El. 2485. The unit contains discontinuous layers of medium dense clayey gravel and stiff sandy clay that are up to 14 feet thick.

Unit 3 (Fine Sand and Silt) consists of loose to medium dense silty sand and fine sand, with some gravel. This unit is potentially liquefiable under earthquake loading. The top of the unit was typically encountered between approximately El. 2468 and El. 2485 near the wall alignment, and its thickness typically ranged from 40 to 60 feet. The layer thins toward the west and is interpreted to pinch out near the west end of the wall.

Unit 4 (Bedrock) consists of moderately weak (R2) to strong (R4) meta-welded lapilli tuff. RQD values ranged from 0 to 100 percent, typically ranging from 58 to 88 percent. Unconfined compressive strengths measured for four specimens ranged from 5.5 to 35 ksi. Bedrock was encountered in RCW-001-08, near the west end of the wall, at El. 2474, and at RCB-003-08(OW), near the east end of the wall, at El. 2442. Near the west end of the wall, the top of bedrock is interpreted to rise relatively steeply toward the west, as shown in Figure 10.3.

10.3.3 GROUNDWATER

Groundwater levels were monitored in RCB-003-08(OW), located at the east end of the former RCW site, from May 2008 through October 2008. The measured groundwater levels are tabulated in Table 3.5 and shown graphically in Figure 3.5.

Based on the available data, the groundwater level is approximately equal to the lake level when the lake level is higher than about El. 2490. During lower lake level periods in the fall, the groundwater levels stabilize slightly above El. 2490. No groundwater levels have been measured during low lake level periods in the late fall through spring.

10.3.4 Liquefaction Potential

URS assessed the liquefaction potential of Unit 3 at RCB-003-08(OW), located at the east end of the former RCW alignment, and CUL-008-07, located 175 feet south of the former RCW alignment. The potential for liquefaction was assessed based on the blow counted obtained from the field explorations and the soil type.

Most of Unit 3 was found to be susceptible to liquefaction. Liquefaction-induced settlement of Unit 3 was estimated using the Tokimatsu M-correction procedure (Tokimatsu And Seed 1987). The estimated total settlements were about 5 inches at RCB-003-08(OW) and nearly 12 inches at CUL-008-07. CUL-008-07 was terminated in liquefiable soil at a depth of 50 feet. Additional settlement may be predicted if liquefiable soils are present between 50 and 80 feet depth. The results of the analysis are summarized in Table 10.3. Graphical results are presented in Appendix H.

10.4 GLOBAL STABILITY

URS evaluated the stability of the planned RCW under static and seismic loading for the most critical combination of wall and slope height, which was interpreted to be at LW 1421+99. At this location, a wall approximately 30 feet high was proposed, and the embankment slope below the toe of the wall was approximately 10 feet high.

10.4.1 Method of Analysis

Soil conditions at LW 1421+99 were interpreted from nearby borings and the subsurface profiles and are shown in Figure 10.1. The closest borings, CUL-007-07 and SCW-013-08, are approximately 150 feet east and 100 feet southeast, respectively, of the wall face at LW 1421+99. The soil conditions assumed for the stability analyses consist of gravel with sand (Units 1 and 2) above El. 2475, fine sand and silt (Unit 3) between El. 2420 and 2475, and bedrock below El. 2420.

The sand and silt stratum (Unit 3) is susceptible to liquefaction under the 1,000-year seismic event. Two seismic cases were evaluated:

- Stability during the earthquake (pseudostatic analysis) and
- Stability immediately after the earthquake (residual strength analysis).

The residual shear strength was calculated using the method developed by Kramer (2008).

URS evaluated the effect of the groundwater and lake levels for two cases:

- High groundwater and high lake levels (El. 2517; static loading only) and
- Lower groundwater level (El. 2495) and low lake level (El. 2416).

Additional descriptions of the procedures and assumptions used in the stability analysis are provided in Section 5.2.

10.4.2 Results for Proposed Wall Without Mitigation

For the proposed wall configuration, the calculated factors of safety for the static case were as follows:

- 1.43 for high groundwater and high lake level conditions.
- 1.45 for lower groundwater and low lake level conditions.

The calculated factors of safety for seismic loading using the pseudostatic analysis and the residual strength analysis were 1.16 and 0.98 for the lower groundwater and low lake level conditions.

Graphical results of the stability analyses are provided in Appendix H.

10.4.3 Results for Proposed Wall With Mitigation

For static loading under high water conditions and for seismic loading where liquefaction was predicted, URS evaluated ground improvement using compaction grouting for mitigating instability.

A minimum treated area width of 10 feet was needed to provide minimum factors of safety of 1.5 for static loading and 1.1 for seismic loading, residual strength case. Calculated displacements under seismic loading for treated areas with widths of 10, 20, and 30 feet were 3.5, 1.5, and 0.75 feet, respectively. These results are summarized in Table 10.4. Graphical results are provided in Appendix H.

11.0 RESORT CREEK BRIDGES

11.1 PROJECT DESCRIPTION

Based on preliminary plans received from WSDOT on June 23, 2010, the Resort Creek bridges (RCB) consist of two 145-foot-long single-span bridges, including an EB bridge between LE 1426+11 and 1427+56 and a WB bridge between LW 1426+26 and 1427+71. The roadway elevations are approximately El. 2532 and El. 2531 WB and EB, respectively. The Resort Creek floodplain beneath the bridges is at approximately El. 2500 and El. 2494 WB and EB, respectively. The preliminary plans received from WSDOT are provided in Appendix G.

The bridges will be supported by three 6-foot diameter drilled shafts at each of the four abutments. The shafts will be socketed into rock with 6-foot diameter rock sockets and connected by a shaft cap. The bridge girders will be supported on concrete abutment walls. Preliminary plans provided to URS by WSDOT on July 13, 2010 indicate top of shaft elevations ranging from El. 2500 to 2510 and abutment wall exposed heights below the bridge girders ranging from 5 to 17 feet.

The WB approach fills will be constructed on the existing WB embankment with a grade raise of up to six feet. The EB embankment will be constructed largely over native ground, with new embankment thickness of up to about 40 feet. Preliminary plans provided to URS by WSDOT on June 23, 2010 indicate the approach fills will be retained in the transverse direction adjacent to the abutments by retaining walls (Walls 11 through 18) with exposed heights of up to approximately 20 feet.

Foundations options for RCB were discussed at meetings between WSDOT and URS on January 22 and March 11, 2009 and May 12 and September 15, 2010. Sands and silts that are susceptible to liquefaction under the design earthquake are present beneath the bridge. It was concluded at the January 22 meeting that shallow foundations would be problematic at this location for several reasons:

- The cost and difficulty of shoring the excavations to construct the footings;
- The cost and difficulty of implementing ground improvement that would be needed to stabilize the liquefiable soils; and
- The cost of the large footings and abutment walls that would be needed.

WSDOT requested that URS develop geotechnical design information for a foundation option that uses drilled shafts of sufficient size to withstand the lateral loads, including soil loads from a potential flow failure that could occur if the soils liquefied during seismic loading. Recommendations for shaft design are presented in this section.

WSDOT also requested that URS evaluate ground improvement using compaction grouting. Preliminary layouts for ground improvement were provided to WSDOT on July 14, 2010. Ground improvement was deleted when WSDOT determined it would be more cost effective to design the shafts to withstand the lateral loads potentially resulting from liquefaction.

11.2 FIELD EXPLORATION AND LABORATORY TESTING RESULTS

A total of 17 borings have been drilled at or near the RCB abutments and approach fills, including:

- Five borings drilled in 2010;
- Three borings drilled in 2008;
- Eight borings drilled in 2007; and
- One boring drilled in 2006.

Two of the borings, RCB-002-08(OW) and RCB-003-08(OW), were completed as groundwater OWs.

A summary of borings for RCB is provided in Table 11.1. Figure 11.1 shows the locations of the borings. Logs of the 2008-2010 borings are provided in Appendix A. Logs of the 2006 and 2007 borings are provided in the 2007 Roadway Geotechnical Report (URS 2008a).

Laboratory testing consisted of 38 particle-size analyses, 4 Atterberg Limits tests, 40 moisture content tests, and 8 rock unconfined compressive strength tests. Some particles-size analyses were conducted using composited samples. A total of 47 samples were used in the particle-size analyses. The laboratory soil testing results and the unconfined compressive strength testing results are presented in Tables 11.2 and 11.3, respectively. The particle-size analysis and Atterberg Limits test results are presented graphically in Appendix D.

Field PLTs were conducted on selected rock samples. The PLT results are summarized in Table 11.4.

11.3 SITE CONDITIONS

11.3.1 Site Description

The bridge site will be located at the bottom of the Resort Creek drainage. The existing Resort Creek crossing consists of a 7-foot diameter culvert at approximately LW 1423+00. The new crossing will move the creek channel approximately 400 feet east, which is closer to its historic, more natural location.

On the north side of the highway, the Resort Creek channel was redirected to the west during the original construction of the roadway, then called SR-10, in 1957. Scour below the outlet of the culvert has result in about 25 feet of channel downcutting south of the highway.

The existing roadway consists of EB and WB lanes that are separated by a narrow median area. It is elevated approximately 15 to 25 feet above the floodplain in this area. At high lake levels, backwater conditions exist in Resort Creek at the bridge site, and the embankment slopes are partially submerged. Lake levels are described in Section 2.1.3.

The area in the vicinity of the bridge has been classified as a wetland. WSDOT rated the wetlands north and south of the bridge as Category II and Category III, respectively, based on the *Washington State Wetland Rating System for Eastern Washington* (Ecology 2004).

11.3.2 Subsurface Conditions

Cross sections that show the interpreted subsurface conditions at the west and east abutments are presented in Figures 11.2 and 11.3, respectively. EB and WB subsurface profiles are presented in Figures 4.1B and 4.2B, respectively. Four soil units and one rock unit have been identified:

Unit 1 (Gravel with Sand/Cobbles/Boulders [Fill]) consists of loose to dense, poorly- to well-graded gravel with sand, cobbles, and occasional boulders. This material is fill placed for construction of the existing roadway embankment. The thickness of this soil unit ranges from 25 to 30 feet near the west abutments and 15 to 25 feet near the east abutments. Boulders were encountered in RCB-001-08 at El. 2501 and 2506, in RCB-002-08 at El. 2516, and in RCB-003-08(OW) at El. 2505. The maximum boulder size encountered was 16 inches.

Unit 2 (Gravel with Sand/Cobbles [Native Soil]) consists of medium dense to very dense, poorly- to well-graded gravel, silty gravel with sand and cobbles, and silty sand. The unit was typically encountered at about El. 2500 at the abutment locations. The thickness encountered ranged from 10 to 28 feet.

Unit 3 (Sand and Silt) consists of loose to medium dense silty sand with gravel and silty fine sand. This unit is potentially liquefiable under earthquake loading. Near the proposed west abutments, the top of the unit was encountered between El. 2478 and El. 2483, and its thickness ranged from 36 to 42 feet. It thins toward the east and north. Near the proposed east abutments, the top of the unit was encountered between El. 2483 and El. 2485, and its thickness ranged from 23 to 32 feet.

The soil gradation encountered was finest in CUL-012-07, near the west abutment of the EB bridge. A thin (2 to 6.5-foot thick) layer of soft to medium stiff clayey silt and silty clay was encountered within Unit 3 at the west abutments. The layer was encountered at elevations ranging from El. 2450 to El. 2462.

Unit 4 (Sand and Gravel) consists of very dense sand and gravel with silt and occasional cobbles and boulders. This material is interpreted to be glacial till. It was encountered at elevations ranging from El. 2443 to El. 2460. The layer appears to thin toward the north and was not encountered in RCB-003-08(OW) near the northern roadway edge at the west abutment. The maximum thickness penetrated was 25 feet.

Unit 5 (Bedrock) consists of weak (R1) to moderately strong (R3) andesite. Bedrock was encountered in 6 borings at elevations ranging from El. 2417 to 2445. RQD values ranged from 0 to 100 percent, and were typically 40 percent of higher. The measured unconfined compressive strengths of 8 rock specimens ranged from 1.0 (R1) to 5.6 (R2) ksi, with an average of 2.4 ksi and a standard deviation of 1.4 ksi. PLT results ranged from 0.5 to 17.7 ksi. The unconfined compressive strength and PLT results are summarized in Tables 11.3 and 11.4, respectively.

11.3.3 Groundwater

Groundwater levels were measured from May 2008 through August 2010 in RCB-003-08(OW) near the west abutment and from September 2008 through August 2010 in RCB-002-08(OW) near the east abutment. The measured groundwater levels are tabulated in Table 3.5 and shown graphically in Figures 3.4 and 3.5.

Based on the available data, the groundwater level is approximately equal to the lake level when the lake level is higher than about El. 2490. During lower lake level periods in the fall, the measured groundwater levels are slightly above El. 2490. No groundwater levels have been measured during low lake level periods in the late fall through spring.

11.4 GEOLOGIC HAZARDS

11.4.1 Seismicity and Structure Modeling

Descriptions of site seismicity, ground motion parameters, and parameters for use in seismic modeling of the bridges are provided in Section 5.1.

11.4.2 Scour and Wave Erosion

Scour and scour protection were evaluated by Otak under subcontract to URS. The results and recommendations are provided in Appendix K of the Hydraulic Report (Otak 2008a).

Protection of embankments from erosion by lake waves and wave runup are addressed in the Phase 1B Hydraulic Report (WSDOT 2009a).

11.4.3 Liquefaction and Flow Failure/Lateral Spreading

The potential for liquefaction of Unit 3 under the design earthquake was evaluated using the computer software LiquefyPro (CivilTech 2005) based on the soils encountered in seven borings. Total settlements of less than 1 inch to 8 inches were calculated. The results of the liquefaction analysis are presented in Table 11.5.

Samples with low blow counts were collected from Unit 1 in RCB-002-08. These materials were considered to be non-liquefiable because their coarse gradation, which indicates a high permeability, and relatively short drainage paths suggest that the development of high excess pore pressures would be unlikely.

As described in Section 11.5, factors of safety for global stability under seismic loading, using residual undrained shear strength parameters for the liquefied soil, were less than 1.0 at both abutments, indicating the potential for a flow failure.

11.5 GLOBAL STABILITY

This section presents the methods used and results of global stability analyses for the proposed bridge abutments and approach fill retaining walls.

11.5.1 Methods of Analysis

A general description of the methods used to analyze global stability is presented in Section 5.2. This section presents supplemental information on the methods used at RCB.

As described in Section 11.4, liquefaction of the sand and silt stratum (Unit 3) may occur under the 1,000-year event. Two seismic cases were evaluated:

- Stability during the earthquake (pseudostatic analysis) and

- Stability immediately after the earthquake (residual strength analysis).

For the case of stability during the earthquake, a pseudostatic analysis was conducted using a horizontal acceleration coefficient, k_h, equal to one-half of the PGA ($k_h = 0.175$) and static (fully drained, non-liquefied) soil shear strengths. For the case of stability immediately after the earthquake, static loading (i.e., $k_h = 0$) was used in combination with the residual shear strength for Unit 3. The residual shear strength was selected as described in Section 5.4.

- URS evaluated the effect of the groundwater and lake levels for two cases:
- High groundwater and lake levels (El. 2517) for static loading only and
- A lower groundwater level (El. 2492) combined with a low lake level (El. 2416) for both static and seismic loading.

For live load surcharge, a traffic load surcharge equal to 250 psf and a snow load surcharge equal to 320 psf were considered. It was assumed that these loads would not occur concurrently. For static conditions, the larger snow load surcharge was used. Because of the low probability of the design earthquake occurring concurrently with the maximum snow load, a snow load was not included for seismic conditions. For the traffic load surcharge under seismic conditions, a load factor equal to 0.5 was used, consistent with Section 3.4.1 of the LRFD Bridge Design Specifications, resulting in a surcharge equal to 125 psf.

11.5.2 Results

Abutment Walls. For static loading, factors of safety of 1.13 to 1.24 and 1.50 to 1.71 were calculated for the Pier 1 and 2 abutment walls, respectively. The Pier 1 factors of safety are less than the target factor of safety of 1.5 for static loading.

For seismic loading at Pier 1, factors of safety of 1.03 for the pseudostatic analysis and 0.85 to 0.86 for the residual strength analysis were calculated. For Pier 2, factors of safety of 1.26 for the pseudostatic analysis and 0.86 to 0.87 for the residual strength analysis were calculated.

The residual strength analysis factors of safety indicate the potential for a flow failure and unbalanced lateral loads to be imposed on the shafts due to lack of global stability. The magnitude of the unbalanced loads is discussed in Section 11.6.1.3. The calculated factors of safety are summarized in Table 11.6. Graphical results are provided in Appendix H.8.

Approach Fill Retaining Walls. The calculated factors of safety were equal to or greater than the target factors of safety of 1.3 and 1.1 for static loading and seismic loading, pseudostatic analysis, respectively.

For seismic loading, residual strength analysis, the calculated factors of safety were less than 1.1 for the EB lakeside Walls 11 and 13. The calculated factors of safety ranged from 0.45 to 0.48 for Wall 11 and from 0.63 to 0.72 for Wall 13. These factors of safety are well below 1.0 and indicate a significant potential for a flow failure to occur and unbalanced lateral loads to be imposed on the shafts due to lack of global stability. The magnitude of the unbalanced loads is discussed in Section 11.6.1.3.

For seismic loading, residual strength analysis, the calculated factors of safety were greater than the target factor of safety of 1.1 for the WB hillside Walls 16 and 18. The calculated factors of safety were 1.30 for Wall 16 and 1.85 for Wall 18. Although the factor of safety (1.30) for Wall 16 exceeds the target of 1.1, the potential exists for wall and embankment displacements due to lateral spreading. The factor of safety (1.85) for Wall 18 indicates a limited potential for displacement due to lateral spreading.

The calculated factors of safety are summarized in Table 11.6. Graphical results are provided in Appendix H.8.

11.6 GEOTECHNICAL RECOMMENDATIONS

This section presents geotechnical design recommendations for drilled shafts, abutment and approach fill walls, and earthwork. Shaft and abutment wall design information was previously provided to WSDOT on June 28 and 30, 2010, and July 14 and 22, 2010. This section also provides estimated free-field soil displacements and lateral soil loads imposed on shafts as a result of a potential flow failure in Unit 3 under seismic loading. URS understands that WSDOT will design the bridge shafts to withstand the estimated lateral soil loads, as estimated using a limit equilibrium slope stability analysis, in the longitudinal direction. URS further understands that WSDOT has determined that a flow failure in the transverse direction is unlikely to result in bridge collapse. For this reason, WSDOT has elected to not mitigate potential instabilities in the transverse direction.

Seismic parameters for use in structure modeling are provided in Section 5.1.

11.6.1 Drilled Shafts

This section provides shaft design recommendations for bearing and uplift resistance, downdrag and lost skin friction for the extreme event limit state, and input for lateral load analyses. The shafts should be socketed into bedrock or dense sand and gravel (Unit 4). The approximate elevations of the top of bedrock and Unit 4 can be estimated from Figures 11.2 and 11.3.

11.6.1.1 Bearing and Uplift Resistance

URS evaluated the unfactored (nominal) bearing and uplift resistance of 6-foot, 7-foot, and 8-foot diameter drilled shafts for the abutments in accordance with the methods presented in the 2007 AASHTO LRFD Bridge Design Specifications. Bearing resistance for the service limit state was calculated assuming an allowable settlement equal to 1.0 inch. Lost skin friction and downdrag loads due to potential liquefaction for the extreme event limit state are discussed in Section 11.6.1.2.

The depth to bedrock varies in this area. Because of the variability of the bedrock elevation, two sets of charts have been developed to estimate the shaft bearing resistance.

The first set of charts, Figures 11.4 through 11.9, shows the unfactored skin friction and tip resistance for 6-foot, 7-foot, and 8-foot diameter shafts in soil for the strength, service, and extreme event limit states at Piers 1 (west) and 2 (east), respectively. Tip resistance is provided for the case where bedrock is deep and shafts may be terminated in soil.

If the shafts are socketed into rock, up to 20 percent of the strength limit state skin friction in soil shown in Figures 11.4 through 11.9 may be added to the rock bearing resistance. Because of the potential for liquefaction, soil skin friction and tip resistance in Unit 3 should be neglected for the extreme event limit state. For extreme event limit state design of shafts socketed into rock, up to 20 percent of the skin friction in soil below the base of the potential liquefaction zone shown in Figures 11.4 through 11.9 may be added to the rock bearing resistance.

The second set of charts, Figures 11.10 through 11.12, shows the unfactored skin friction and tip resistance as a function of depth below top of bedrock for 6-foot, 7-foot, and 8-foot diameter shafts in rock for the strength and extreme event limit states. Because the movement needed to mobilize full tip resistance exceeds the movement that will cause yielding in skin friction, the full skin and tip resistances are not additive. Figures 11.10 through 11.12 show the unfactored bearing resistance for combined skin friction and tip resistance. For this case, the skin friction is calculated using the residual rock shear strength.

If the factored bearing resistance is greater than the factored loads for the strength limit state, settlement under service conditions should not exceed one inch.

The unfactored uplift for the strength and service limit states may be taken as equal to the unfactored skin friction. For the extreme event limit state, the unfactored uplift resistance may be taken as equal to the unfactored downdrag load plus the unfactored skin friction in the rock socket. Downdrag loads are discussed in Section 11.6.1.2. Resistance factors for computing uplift resistance are provided in Table 11.7

Note that the weight of the shaft has not been deducted from the bearing resistance in the figures and is not included in the uplift resistance. Skin friction in the shaft-column splice zone (top 10 feet) was neglected. The computed skin friction and uplift resistances assume there is not a permanent casing below the splice zone.

For the extreme event limit state, all skin friction within and above liquefiable zones should be neglected. The base of the liquefiable zone was encountered at approximately El. 2443 at Pier 1 and between El. 2453 and 2460 at Pier 2. This lost skin friction, as well as downdrag loads resulting from settlement of the liquefiable zones, is discussed in the following section.

The shaft center-to-center spacing should not be less than 2B, where B is the shaft diameter. For cohesionless soil (Units 1 through 4), group effects need to be considered where the shaft spacing is less than 4B. Based on a recent WSDOT design memorandum (WSDOT 2010b), bearing resistance reduction factors for group effects for the strength and extreme event limit states depend on whether the shaft group cap is in intimate contact with ground, as follows for a single row of shafts:

- If the shaft group cap is in intimate contact with ground, the reduction factor is 1.0 for a shaft spacing of 2B or greater.

- If the shaft group cap is not in intimate contact with ground, the reduction factor is 0.9 for a shaft spacing of 2B or greater and increases linearly to 1.0 at a shaft spacing of 3B. For a spacing of 3B or greater, the reduction factor is 1.0.

- For service limit state design of shafts in cohesionless soil, the LRFD Bridge Design Specifications, Section 10.8.3.6.3, specify a reduction factor of 0.65 for a center-to-center

shaft spacing of 2.5B. For center-to-center spacing of 4B or greater, the reduction factor is 1.0. Linear interpolation can be used for spacings between 2.5B and 4B.

URS recommends that the resistance factors provided in Table 11.7 be used when evaluating the different limit states.

11.6.1.2 Downdrag Loads and Skin Friction Losses

Downdrag could occur as a result of liquefaction of soils in Unit 3. Significant consolidation of compressible soil layers after shaft construction is not expected, and it is recommended that downdrag loads be assumed to be zero for strength and service limit state design.

Should liquefaction occur, downdrag forces will be imposed on the shafts by all soil above and including the liquefied zone. For the abutment shafts, downdrag loads could be imposed by all soil layers above approximately El. 2440 at Pier 1 and El. 2453 to 2460 at Pier 2. Liquefaction will also result in the loss of bearing resistance from upward skin friction in all soil above and including the liquefied zone. In evaluating downdrag loads and skin friction losses, it was assumed liquefaction would be limited to a depth of 80 feet.

URS evaluated the downdrag loads on drilled shafts for the extreme event limit state based on the guidance presented in the November 2008 updates to Section 6 of the GDM. Downdrag in the liquefied soil zone was estimated using the residual shear strength of the soil. The selection of residual strength parameters for liquefiable soil for use in downdrag estimates in described in Section 5.4.

Estimated downdrag loads and skin friction losses due to liquefaction-induced settlements are provided in Table 11.8. Downdrag loads should be added directly to the factored bridge loads when evaluating the shaft capacity required for the extreme event limit state. A load factor of 1.0 is recommended for downdrag loads.

Skin friction losses must be subtracted from the unfactored skin friction capacity curves for the extreme event limit state (Figures 11.4 through 11.9).

11.6.1.3 Lateral Load Analysis

URS understands that for bridge design purposes WSDOT will evaluate the soil response under lateral loading using the computer programs DFSAP and LPILE. Geotechnical design parameters and soil stratigraphy for use as input for lateral load analyses are presented in Table 11.9. This information was previously provided to WSDOT on July 22, 2010. Scour protection should be implemented as described in the Hydraulic Report to prevent scour around the shafts and shaft cap.

The rock strengths provided in Table 11.9 are based on the global rock mass strength (Hoek, et al. 2002). For LPILE analyses, the available rock response models have not been well verified. Therefore, it is recommended that the Stiff Clay in the Presence of Free Water model be used for rock, using a cohesion value equal to one-half the rock unconfined compressive strength. Because a softer rock response will result in higher bending moments in the shaft and a stiffer response will result in higher shear forces, it is recommended that a range of rock stiffness values be evaluated, as shown in Table 11.9. It is assumed that the extreme event limit state is the critical loading case.

A resistance factor of 1.0 is recommended for lateral load analysis for all limit states.

Group effects for strength limit state design can be evaluated with DFSAP using the shaft group option for both shaft groups with and without a shaft cap. For LPILE analyses of non-liquefied soil, p-multipliers for group effects should be used where the shafts are spaced closer than 5D, where D is the shaft diameter and the spacing is on-center. The p-multipliers for group effects shown in the table below should be used in conjunction with the definitions of loading direction and spacing shown in Figure 8-10 of the GDM. P-multipliers for other spacings can be determined using linear interpolation. A shaft spacing less than 2D should not be used.

Shaft Center-to-Center Spacing in the Direction of Loading	P-multipliers for Group Effects		
	Row 1	Row 2	Row 3 and higher
2D	0.45	0.33	0.25
3D	0.7	0.5	0.35
5D	1.0	0.85	0.7

P-multipliers for liquefied soil are discussed in Section 11.6.1.4.

11.6.1.4 Lateral Load Analysis – Supplemental Recommendations for Extreme Event Limit State Design

For the extreme event limit state, the potentially liquefiable zones should be analyzed as both liquefiable and non-liquefiable to evaluate soft and stiff foundation responses, respectively. An earthquake magnitude equal to 6.33 and a PGA equal to 0.35g should be used to evaluate liquefaction. Measured groundwater elevations range from El. 2492 to 2517. An analysis using the maximum groundwater elevation will result in the softest foundation response.

Shaft Loading Due to Flow Failure or Lateral Spreading. For the case where liquefaction occurs, the potential exists for soil within and above the liquefied zone to displace and impose lateral loads on the shafts. The liquefaction case can be evaluated using the procedure described in the GDM, Section 6.5.4. The GDM provides that either a displacement-based method or a force-based method can be used.

The displacement-based method involves development of a free-field soil displacement profile and reduced soil stiffness properties for the liquefied soil that are used as input to the computer program LPILE. Assuming ground improvement has not been implemented, the free-field soil displacements are estimated to be on the order of three feet or more.

The displacement profile assumes no displacement at the base of the liquefied zone, with the displacement increasing linearly to the maximum amount at the top of the liquefied zone. The displacement in the soil above the liquefied zone is assumed equal to the maximum displacement. Liquefied soil strengths are used in the zone assumed to liquefy, and static soil strengths are used in soil above the liquefied zone. The estimated displacement profiles are summarized in Table 11.10.

For the force-based method, the GDM provides that the lateral load from the liquefied soil can be calculated using either the method specified in the Japanese codes (Japanese method) or a limit equilibrium slope stability analysis. For the limit equilibrium slope stability analysis method, a residual strength analysis is used, as described in Section 11.5. The shafts should be analyzed as

a free-standing bent anchored in rock and non-liquefied soil below the depth of liquefaction (i.e., no soil resistance within and above liquefiable layers).

Lateral soil loads in the longitudinal direction calculated by both of these methods were provided to WSDOT on March 6, 2009 for the case of no ground improvement. The load per 8-foot diameter shaft calculated using the Japanese method was 2,300 to 5,000 kips, depending on passive loading conditions from the crustal layer (including the abutment walls). The load calculated using the limit equilibrium slope stability analysis was approximately 50 kips per foot, or 800 kips per 8-foot diameter shaft, for a limit equilibrium factor of safety equal to 1.1, assuming loads over a width equal to 2B are imposed on each shaft. Loads for other shaft sizes can be calculated assuming the load is directly proportional to the shaft diameter.

Lateral soil loads calculated using the limit equilibrium slope stability analysis method for the transverse direction were provided to WSDOT on September 16, 2010 for the case of no ground improvement. Lateral soil loads were not calculated using the Japanese method. The maximum loads are expected to occur in the EB direction (i.e., toward the lake) at both piers. For Piers 1 and 2, the calculated lateral loads are 125 and 80 kips per foot, respectively, for a factor of safety equal to 1.1. The load per foot should be multiplied by twice the shaft diameter (2B) to obtain the total lateral load. For example, for a 6-foot diameter shaft at Pier 1, the total calculated lateral load would be 1,500 kips.

Using the limit equilibrium slope stability analysis, the calculated force is assumed to be distributed over the foundation in the liquefiable zone as a uniform stress. The calculated loads and graphical presentations for the limit equilibrium slope stability analyses are provided in Table 11.11 and Appendix H.8, respectively.

Interpretation of the limit equilibrium analysis method results should consider that this method does not capture any dynamic or displacement effects of the flow sliding (or lateral spreading). In particular, there is no explicit consideration of passive load imposed by the crustal layer (including the abutment walls), which is included in the Japanese method.

Liquefied Soil Strength Properties. For potentially liquefiable soil, it is recommended that the LPILE program be used with static soil properties (effective friction angle and p-y modulus) reduced to account for the liquefaction effects. The p-y curves generated by LPILE using the liquefied sand model should not be used. The reduced effective friction angle, $\varphi_{reduced}$, was calculated using GDM Section 6.5.1.2, Method 2 as the inverse tangent of the residual undrained shear strength divided by the effective vertical stress at which the residual shear strength was determined:

$$\varphi_{reduced} = \tan^{-1}(S_r/\sigma'_{vo})$$

Where:

S_r = Residual undrained shear strength and

σ'_{vo} = Effective vertical stress (no excess pore pressure)

The recommended S_r is the 40% non-exceedance value (i.e., 60% probably that the strength will exceed the value) of the residual shear strength estimated using the Kramer (2008) method. The reduced static soil properties are provided in Table 11.9.

P-multiplier for LPILE Analysis. For LPILE analyses, it may be assumed that there are no group effects for liquefied soil (p-multiplier for group effects equal to 1.0), unless the force-based method is used. Using the force-based method, the shafts should be analyzed as a free-standing bent anchored in rock and non-liquefied soil below the depth of liquefaction. This can be accomplished by using a p-multiplier equal to 0 for all layers above and including the liquefied zone.

11.6.1.5 Temporary Casing

It is recommend that the temporary casing extend to at least 5 feet below the seasonal low groundwater level or to the base of the gravel and sand layer (Unit 2) that is directly above the liquefiable layer, whichever is lower. The lowest groundwater elevation measured in either well RCB-002-08(OW) or RCB-003-08(OW) is El. 2491. The base of Unit 2 is approximately El. 2480 at Pier 1 and El. 2482 at Pier 2. Based on the above criteria, we recommend temporary casing be placed to at least El. 2480 at Pier 1 and El. 2482 at Pier 2.

11.6.2 Abutment and Approach Fill Walls

URS understands that the abutments will include reinforced concrete retaining walls that are tied into shaft caps at their bases. Lateral earth loads imposed on the abutment walls, wing walls, and curtain walls will depend on whether the wall is free to rotate or translate (i.e., whether the wall is flexible or rigid). Active pressures may be used for flexible walls, and at-rest earth pressures should be used for rigid walls.

Earth pressures for abutment walls that are backfilled prior to construction of the superstructure and walls that are able to displace laterally at least 0.001H, where H is the height of the wall, may be calculated using active earth pressures. Wing walls and curtain walls within a distance from the abutment wall that is equal to the abutment wall height should be considered rigid and designed using at-rest earth pressures.

Figures 11.14 and 11.15 show the recommended earth pressure distributions for the strength and extreme event I limit states, respectively, for flexible and rigid walls. It is recommended that organic soil, if encountered in the wall excavation, be replaced with compacted granular fill. The water pressure may be neglected if the backfill behind the wall is fully drained.

A traffic live load surcharge has been included as specified by the LRFD Bridge Design Specifications. This traffic surcharge is greater than the 250 psf construction equipment surcharge specified by the GDM. Traffic and construction loads would not occur together; thus, the construction surcharge has not been included in the design earth pressure loads.

For the extreme event I limit state, the seismic earth pressure and static earth pressure have been combined as a single uniform pressure in Figure 11.15. A PGA equal to 0.35g and horizontal acceleration coefficients equal to 0.5 times the PGA and 1.5 times the PGA for flexible and rigid walls, respectively, were used to calculate the seismic earth pressure coefficient, K_{ae}. The pressure distribution assumes the backfill behind the wall is drained. It should be noted that additional loads would be imposed on the walls if laterally spreading occurs, as discussed in Section 11.6.1.3.

For the strength limit state, the LRFD Bridge Design Specifications recommend load factors for lateral earth pressure that range from 0.9 to 1.5 for flexible walls and 0.9 to 1.35 for rigid walls.

Load factors for lateral earth loads of 1.5 for flexible walls and 1.35 for rigid walls are recommended. A load factor of 1.0 is recommended for the water pressure component. For the extreme event limit state, a load factor of 1.0 is recommended.

A resistance factor of 0.75 is recommended for passive earth pressure resistance, as specified in the GDM for nongravity cantilever walls.

For the approach fill retaining walls (Walls 11 through 18), URS recommends a wall type that is tolerant of movement, such as a structural earth or geosynthetic wall. Use of one of these wall types would limit damage to the wall and embankment during the design earthquake. URS understands that geosynthetic walls will be used for Walls 11 through 14, where part of the wall will be below the high lake level, and structural earth walls will be used for Walls 15 through 18, where the wall will be entirely above the high lake level.

Design parameters for inclusion in the GSP titled *Structural Earth Walls* are provided as follows:

Soil Parameters	Wall Backfill	Retained Soil	Foundation Soil
Unit Weight (pcf)	135	135	135
Friction Angle (deg)	40	40	40
Cohesion (psf)	0	0	0

For the service limit state, the wall shall be designed to accommodate a differential settlement of 0.75 inch per 100 feet of wall length.

For the extreme event I limit state, the wall should be designed for a horizontal seismic acceleration coefficient, k_h, of 0.175 g and a vertical seismic acceleration coefficient, k_y, of 0 g.

11.6.3 Earthwork

During dry weather, existing granular soils with little organic material can be re-used for general site grading. These soils can be re-used as embankment fill within 100 feet of the abutments if they conform to WSDOT Specification 9-03.14(2) for select borrow, or as backfill behind walls if they conform to WSDOT Specification 9-03.12(2) for wall backfill. To conform to these specifications, the soils may require screening to remove large particles. Soils containing significant amounts of organic material should not be used as structural fill.

Select borrow should be compacted in accordance with the requirements in WSDOT Standard Specification 2-03.3(14)C, Method C. Placement and compaction of wall backfill should conform to WSDOT Specification 2-09.3(1)E, which requires placement of horizontal layers no more than 6 inches thick, with each layer compacted to 95 percent of the maximum density determined by the Compaction Control Test according to WSDOT Specification 2-03.3(14)A.

For erosion protection from waves, the lake side slope of the EB embankment below El. 2523 should be topped with a minimum three-foot thick layer of select rock fill that conforms to the following specifications:

- Maximum particle size: 18 inches diameter
- Minimum of 50 percent by weight greater than 13 inches in diameter

- Less than 20 percent by weight passing the 1 inch square sieve
- Less than 10 percent by weight of the portion that passes the 1 inch square sieve passing the U.S. No. 200 sieve
- Los Angeles Abrasion (500 revolutions): Maximum 50 percent loss
- Washington Degradation value: Minimum 15
- Weathering Category: I to II

WSDOT intends to use rock fill from rock cuts to construct embankments to the extent possible. A materials source report has been prepared that identifies rock cuts where select rock fill can be expected (Burk 2009).

The rock fill should be placed and compacted according to WSDOT Specification 2-03.3(14)A. The new embankment should be keyed into the existing embankment in accordance with WSDOT Standard Specification 2-03.3(14).

The exposed subgrade should be evaluated after site grading is complete. Subgrade preparation should be in accordance with Sections 2-06 and 2-09 of the 2006 WSDOT Standard Specifications.

All materials that are proposed to be imported for use as structural fill should be examined and tested. It will be necessary to monitor site grading and verify soil compaction.

Based on the available OW data, the groundwater level is approximately equal to the lake level when the lake level is higher than about El. 2490. The soils below the groundwater table elevation are generally coarse-grained and highly permeable, and it should be expected that water will rapidly enter any excavation that extends below the water table. Because of the high permeability of the soils, continuous pumping will likely be needed. It should be expected that sheet piles would encounter high driving resistance.

The existing gravel and sand soil is considered a Type C soil under OSHA/WISHA regulations for trenching and shoring. This means that temporary cuts above the groundwater table and no greater than 20 feet deep should be no steeper than 1.5H:1V. A steeper slope may be used if appropriately designed and approved. Surface compaction of the slopes may be needed to control localized sloughing. If groundwater seeps out during the excavation, the slopes should be flattened to 2H:1V. For open cuts at the site, URS recommends the following:

- Do not allow traffic, construction equipment, stockpiles, or building supplies within a distance of at least 10 feet from the top of the cut;
- Protect exposed soil along the slope from surface erosion using waterproof tarps, visqueen, or flash coating with shotcrete;
- Schedule construction activities so that the length of time the temporary cut is left open is reduced to the maximum extent practical; and

- Divert surface water, including storm water or melt water runoff, away from the excavation.

The general condition of the slopes should be observed and evaluated periodically by a geotechnical engineer to confirm adequate stability, particularly immediately following any storm events.

11.7 CONSTRUCTION CONSIDERATIONS

Preliminary plans provided to URS by WSDOT indicate top of shaft elevations as low as El. 2500 to 2510, which is below the average high lake level. As shown in Figure 2.3, the lake level is typically above El. 2500 from mid to late April until mid to late July. In the extreme high water year of 1996, the lake level did not fall below El. 2500 until the end of August.

Shafts should be installed in accordance with WSDOT's Drilled Shaft Special Provisions. Boulders up to 16 inches in size were encountered in Unit 1, but were not common. The contractor should have equipment to excavate and advance casing through boulders. Vibrating the casing has the potential to induce liquefaction and cause settlement of the existing embankments. Therefore, it may be prudent to specify casing advancement using a casing-rotating or oscillating method.

The shaft excavations will need to be cased through the highly-permeable Units 1 and 2 to prevent caving and excessive loss of slurry into the formation. Below Unit 2, it is likely that the shaft excavation could be drilled open hole with slurry. The potential exists for caving and heaving in the sands of Unit 3, and the slurry will need to be monitored in accordance with the Special Provisions to control potential caving and heaving.

Where shafts will be socketed into rock, the elevation of bedrock surface and the lengths of the shafts can be estimated using the boring logs and the information provided in Figures 11.2 and 11.3. However, it should be anticipated that bedrock surface elevations could vary considerably over relatively short distances, and the contractor should have adequate equipment and materials on site, including steel reinforcement cages, to allow for this variability.

If shafts in rock will be designed to achieve the required bearing resistance using tip resistance, the bottom of the shaft excavation must be cleaned in accordance with the Special Provisions such that virtually no loose material is present.

It is recommended that the EB approach fill be overfilled by 2 feet above final grade and cut to final grade after two weeks to reduce the potential for settlement after placement of the pavement. If the monitoring data indicate that significant settlement is still occurring two weeks after completion of fill placement, final grading and paving should be further postponed until the rate of settlement decreases.

Considerations related to water control in excavations are described in Section 11.6.3.

12.0 CULVERTS

This section presents geotechnical recommendations for two box culverts:

- The I-90 crossing of Townsend Creek (TCC) and
- The Old Sunset Highway crossing of Resort Creek.

The Old Sunset Highway crossing of Resort Creek will be part of the Phase 1C contract. TCC will be constructed under a subsequent phase.

12.1 PROJECT DESCRIPTION

TCC is a 25-foot wide box culvert with a 12-foot high opening above the streambed located at approximately LE 1484+23. The proposed culvert is a pre-approved, three-sided, bottomless, concrete box culvert founded on spread footings. The streambed elevations at the upstream and downstream ends are El. 2512.8 and 2499.7, respectively. The proposed culvert is 295 feet long with an average streambed slope of 4.4 percent.

The purpose of the box culvert is to improve channel hydraulics at the crossings and to provide a corridor for wildlife to pass beneath the highway. URS understands that WSDOT will consider the culvert as a bridge for the purpose of design because the width of the culvert exceeds 20 feet.

A preliminary design for the Resort Creek culvert had not been prepared at the time of this report. URS understands that a culvert width of less than 20 feet is planned.

12.2 FIELD EXPLORATION AND LABORATORY TESTING

12.2.1 Townsend Creek Culvert

Three borings were drilled and one test pit was excavated at the TCC location as part of the 2008 exploration program. A summary of the borings and test pit for TCC is provided in Table 12.1. Figure 2.2D shows the locations of the borings and test pit. Logs of the borings and test pit are provided in Appendix A.

Borings TCC-001-08, TCC-002-08, and TCC-003-08 were drilled adjacent to the TCC location near the upstream end, midpoint, and downstream end of the culvert, respectively. Test pit TCC-TP-001-08 was excavated at the mid-point of the TCC location.

Laboratory testing consisted of fines content tests on 13 samples and corrosivity tests on two samples. Corrosivity tests consisted of pH, chlorides, sulfates and resistivity (native and saturated) tests.

Laboratory testing results are presented in Table 12.2. Laboratory data sheets are provided in Appendix D.

12.2.2 Resort Creek Culvert

Two borings were drilled on the existing Old Sunset Highway roadway surface at the Resort Creek culvert location as part of the 2010 exploration program. A summary of the borings is provided in Table 12.1. Figure 2.2C shows the locations of the borings. Logs of the borings are provided in Appendix A.

Laboratory testing consisted of grain size analyses of 11 samples. Laboratory testing results are presented in Table 12.2. Laboratory data sheets are provided in Appendix D.

12.3 SITE CONDITIONS

This section describes the existing site conditions, subsurface conditions, groundwater, and scour at the culvert locations.

12.3.1 Townsend Creek Culvert

Site Description. The existing Townsend Creek crossing is an 8-foot high by 8-foot wide concrete box culvert retro-fitted with a 6.8-foot high by 6.3-foot wide corrugated metal pipe on the upstream end. The downstream end of the culvert is perched approximately 3 feet above the adjacent stream bed. The downstream end has concrete wing walls and head walls, and the upstream end has a flared metal end section.

Subsurface Conditions. One cross-section, TCC 1-1', which is shown on Figure 12.1, was developed to show the interpreted subsurface conditions. EB stationing was used for all stationing references on the cross-section.

The following are the two soil units that were encountered at the TCC site:

Unit 1: Gravel with Sand/Cobbles/Boulders [Primarily Fill] consists of fill material that was placed for construction of the existing I-90 roadway embankment. The material consists of medium dense, poorly to well-graded gravel with sand or silty sand with gravel and occasional cobbles and boulders. The thickness of this unit ranges from approximately 12 to 29 feet. This unit also includes up to 1.5 feet of topsoil that consists of silty sand with organic material. The maximum boulder size encountered was 1.9 feet.

Unit 2: (Gravel and Sand with Cobbles) consists of medium dense to very dense, poorly to well-graded sand with gravel or poorly to well-graded gravel with sand and occasional cobbles. A discontinuous one-foot thick layer of soft clayey sand was encountered in TCC-003-08. The top of this unit was encountered between El. 2510 and 2505. The thickness of this unit ranges from approximately 18 to 30 feet. This unit is the uppermost layer of native soil and underlies Unit 1. The vertical extent of this unit is not known because all the borings were terminated in this unit.

Groundwater. No groundwater monitoring data are available at this location. The measured groundwater level at the completion of excavation of TCC-TP-001-08 on June 18, 2008 was approximately El. 2515. Groundwater levels are influenced by the lake level, nearby stream water levels, and local and regional groundwater levels. In general, groundwater levels are higher during the summer months and lower during the fall months.

Scour. The potential for scour at TCC was evaluated by Otak under subcontract to URS (Otak 2008a). The results of the scour analysis for Townsend Creek are provided in Appendix K of the Draft Hydraulic Report (Otak 2008a).

12.3.2 Resort Creek Culvert

Site Description. The existing Resort Creek crossing consists of a 20-foot by 4-foot concrete box culvert. URS understands that the culvert has a gradient of approximately 8 percent.

Subsurface Conditions. One profile, which is shown on Figure 12.2, was developed to show the interpreted subsurface conditions. The profile is near the centerline of the existing road. EB stationing was used for all stationing references in the profile.

The following are the soil unit and bedrock unit that were encountered at the Resort Creek crossing site:

Unit 1: (Gravel and Sand with Cobbles) consists of medium dense to very dense sand and gravel with cobbles. A boulder was encountered in CUL-022-10 at a depth of approximately 8 feet. The unit thickness encountered in CUL-021-10 and CUL-022-10 is 42 to 43 feet.

Unit 2: (Bedrock) consists of very weak to moderately strong andesite. RQD values ranged from 27 to 66 percent. Bedrock was encountered at El. 2482 and 2483 in the two borings.

Groundwater. No groundwater monitoring wells were installed at this location. The observed groundwater depths during drilling ranged from 22.6 to 23.8 feet below ground surface.

Scour. The potential for scour has not been evaluated for Resort Creek at the Old Sunset Highway.

12.4 GEOTECHNICAL RECOMMENDATIONS

This section presents geotechnical design recommendations for bearing resistance, lateral earth pressure, sliding resistance, and foundation springs for TCC and Resort Creek culvert. Because foundation conditions are similar at the two sites, the design recommendations apply to both sites, except as noted. URS understands that WSDOT will not design the Resort Creek culvert for seismic loading. The recommended ground motion parameters for extreme event limit state design are presented in Section 5.1.

Per the WSDOT Bridge Design Manual, bridge footings may be sloped at inclinations no steeper than 10H:1V (horizontal:vertical). However, a local vendor of pre-fabricated box culverts (Utility Vault) specifies limiting the culvert inclination to no steeper than 20H:1V. The inclination of the TCC is flatter than 20H:1V. The inclination of the Resort Creek culvert has not been determined at the time of this report. URS understands that the inclination of the existing culvert at this location is approximately 12.5H:1V.

Pre-cast footings can be used at the culvert locations. The footing must be embedded below the scour depth or three feet, whichever is deeper. The footing width should not be less than 3 feet.

12.4.1 Bearing Resistance

Figure 12.3 shows the unfactored bearing resistance as a function of the effective footing width for the three limit states, where the effective footing width (B') is equal to the total footing width (B) minus two times the eccentricity (e), so that B' = B - 2e. This bearing resistance is applicable for footings placed on existing coarse-grained soil with a minimum footing embedment of three feet. Any loose soil or soil containing organic material encountered during excavation for the foundations should be removed and replaced with compacted structural fill. The service limit state bearing resistance is based on an allowable settlement of one inch.

URS recommends that the resistance factors presented in Table 12.3 be used when evaluating the three limit states. The unfactored bearing resistance values presented in Figure 12.3 do not incorporate the bearing resistance factors.

12.4.2 Lateral Loads and Sliding Resistance

The side walls of box culverts will respond rigidly to lateral earth loads. Accordingly, lateral earth loads imposed on the culverts were calculated using at-rest earth pressures.

Figures 12.3 and 12.4 show the earth pressure distributions for the strength and extreme event I limit states, respectively, for rigid walls. The water pressure may be neglected if the backfill behind the wall is fully drained.

A live load surcharge has been included as specified by the LRFD Bridge Design Specifications. The live load surcharge may be either a traffic load or a snow load. Typical traffic and snow load surcharges are 250 psf and 320 psf, respectively. Traffic and snow loads would not occur together.

For the extreme event I limit state, the seismic and static earth pressures have been combined as a single uniform pressure in Figure 12.4. A PGA equal to 0.35g and a horizontal acceleration coefficient equal to 1.5 times the PGA were used to calculate the seismic earth pressure coefficient, K_{ae}. The pressure distribution assumes the backfill behind the wall is drained.

A load factor for lateral earth loads of 1.35 for rigid walls is recommended. A load factor of 1.0 is recommended for the water pressure component. For the extreme event limit state, a load factor of 1.0 is recommended.

Lateral foundation loads may be resisted by friction on the bases of the footings and by passive resistance against the sides of the footings and the side walls. For pre-cast footings and cast-in-place footings placed on existing coarse-grained soils or properly compacted, suitable fill, the unfactored frictional resistance may be computed using coefficients of friction (tan δ) of 0.45 and 0.58, respectively. The soil weight may be computed using unit weights of 135 pcf above the water table and 73 pcf below the water table. The computed frictional resistance must be multiplied by the resistance factor for sliding that is provided in Table 12.3.

Passive resistance to sliding may be calculated using the unfactored passive pressure distributions shown in Figure 12.3 for the strength limit state and Figure 12.4 for the extreme event I limit state. Passive resistance was assumed to develop along a vertical soil plane that intersects the outside edge of the footing and not along the wall face. For this reason, it was assumed that no wall friction (Rankine condition) would occur during development of the passive resistance. The

computed passive pressure resistance must be multiplied by the resistance factor for passive pressure that is provided in Table 12.3.

12.4.3 Foundation Springs

Foundation spring parameters for TCC were estimated using the methodology presented in FEMA 356 - Prestandard and Commentary for the Seismic Rehabilitation of Buildings (FEMA 2000). For foundations supported on the existing coarse-grained soils or properly compacted, suitable fill, foundation springs may be calculated using the following values:

Initial shear modulus, G_0 = 3,300 kips per square foot (ksf)

Effective shear modulus ratio, $G/G_0 = 0.6$

Poisson's ratio, $\mu = 0.35$

12.4.4 Global Stability

For global stability, box culverts greater than 20 feet wide need to have the same factors of safety as other structures, such as bridges and walls. Therefore, the box culvert at the Townsend Creek crossing will need to have factors of safety of 1.5 for static conditions and 1.1 for seismic conditions using the 1,000-year return interval earthquake.

As shown in Figures 12.1 and 12.2, the soils beneath and adjacent to the proposed culverts consist of medium dense to very dense, flat-lying gravel with sand. Because of these conditions, the culverts and embankments are expected to be stable under static and seismic conditions and with factors of safety greater than 1.5 and 1.1, respectively.

12.4.5 Earthwork

Material used as fill behind the culvert side walls should conform to WSDOT Specification 9-03.12(2) for wall backfill. Placement and compaction of backfill around the structure should conform to WSDOT Specification 2-09.3(1)E, which requires placement of horizontal layers no more than 6-inches thick, with each layer compacted to 95 percent of the maximum density determined by the Compaction Control Test according to WSDOT Specification 2-03.3(14)D.

Embankments outside of the limits of structure excavation may be constructed with side slopes no steeper than 2H:1V using soil that conforms to WSDOT Specification 9-03.14(3) for common borrow. If the soil contains less than 25 percent, by volume, of particles that are 4 inches in size or larger, the soil should be compacted according to WSDOT Specification 2-03.3(14)C, Method B. If the soil contains 25 percent or more, by volume, of particles that are 4 inches in size or larger, the soil should be compacted according to WSDOT Specification 2-03.3(14)A.

At TCC, the embankment will be subject to erosion by lake waves. To limit wave erosion at TCC, embankments outside of the limits of structure excavation may be constructed with side slopes no steeper than 1.5H:1V using select rock fill that conforms to the following specifications:

- Maximum particle size: 18 inches diameter
- Minimum of 25 percent greater than 4 inches in diameter

- Less than 3 percent by weight passing the U.S. No. 200 sieve
- Los Angeles Abrasion (500 rev): Maximum 50 percent loss
- Washington Degradation Value: Minimum 15
- Weathering Category: I to II
- Jar Slake Index: 5 to 6
- Slake Durability Index: Minimum 85 percent

The select rock fill should be compacted according to WSDOT Specification 2-03.3(14)A.

The exposed subgrade should be evaluated after site grading is complete. Subgrade preparation should be in accordance with Sections 2-06 and 2-09 of the 2006 WSDOT Standard Specifications.

All materials that are proposed to be imported for use as structural fill should be examined and tested. It will be necessary to monitor site grading and verify soil compaction.

The existing gravel and sand soil is considered a Type C soil under OSHA/WISHA regulations for trenching and shoring. This means that temporary cuts above the groundwater table and no greater than 20 feet deep should be no steeper than 1.5H:1V. A steeper slope may be used if designed and approved by a licensed geotechnical engineer. Surface compaction of the slopes may be needed to control localized sloughing. If groundwater seeps out during the excavation, the slopes should be flattened to 2H:1V. For open cuts at the site we recommend:

- No traffic, construction equipment, stockpiles, or building supplies be allowed at the top of the cut slopes within a distance of at least 10 feet from the top of the cut;
- Exposed soil along the slope be protected from surface erosion using waterproof tarps, visqueen, or flash coating with shotcrete;
- Construction activities be scheduled so that the length of time the temporary cut is left open is reduced to the maximum extent practical;
- Surface water, including storm water or melt water runoff, be diverted away from the excavation; and
- The general condition of the slopes be observed and evaluated periodically by a geotechnical engineer to confirm adequate stability, particularly immediately following any storm events.

12.5 CONSTRUCTION CONSIDERATIONS

Groundwater level information is limited at the TCC location. The groundwater levels are generally influenced by the lake level, nearby stream water levels, and local and regional groundwater levels. In general, groundwater levels are higher during the summer months and lower during the fall months.

The lake level and winter conditions will need to be considered in construction planning. The minimum footing elevation will be approximately El. 2496 at the TCC location, assuming the footings are placed four feet below the streambed surface. As discussed in Section 2.1.3, during an "average" year the maximum lake level is El. 2511.4. Therefore, the minimum footing elevation will be 15.4 feet below the average high lake level.

During an "average" lake level year, the lake level would be below the minimum footing elevation from late July through mid-April of the following year. During the "wet" year of 1996, when lake levels were high for long periods, the lake level was below the minimum footing elevation only from early September through November. Lake level information is presented in Section 2.1.3.

When the lake levels are high, it would be difficult to control the rapid groundwater flow through the existing high permeability, coarse-grained soils into shallow foundation excavations. Therefore, it is recommended that footings be placed when the lake levels are at least five feet below the base of the footing elevation. Some seepage into shallow foundation excavations is expected even during the low lake levels. It is anticipated that groundwater seepage can be controlled using sumps and pumps if excavation is done when the lake level is at least five feet below the base of the excavation.

Subsurface explorations indicate the presence of scattered boulders in soils at this location. The contractor should have equipment capable of excavating boulders up to several feet in size.

13.0 EMBANKMENTS

Geotechnical recommendations for construction of embankments with 1.5H:1V and 2H:1V side slopes are presented in the 2007 Roadway Geotechnical Report (URS 2008a). Stability analyses presented in that report for LE 1450+00 (AE 121+03) indicated the need for a toe buttress to achieve a factor of safety of 1.25 under static loading. An additional boring was drilled during the 2009 exploration program to better define subsurface conditions at this location.

Since the 2007 Roadway Geotechnical Report was issued, a proposed realignment completed in March 2010 has added new embankment construction on the lake side from approximately LE 1419+00 to the west end of the RCB approach fill (LE 1424+85). The realignment is depicted in Figure 2.6.

This section presents the following updates to the information in the 2007 report:

- Updated global stability analyses at AE 121+03 using information from a boring drilled in 2009 (EMB-027-09) and

- Global stability analyses at LW 1421+99 (LE 1421+84), representing stability in the section of embankment construction added by the March 2010 realignment.

URS has also developed recommendations for protection of embankment slopes from erosion by lake waves. This information is provided in Appendix A3.12 of the Phase 1B Hydraulic Report (WSDOT 2009a) and is also applicable to Phase 1C.

The RCB approach embankments within 100 feet of the abutments are addressed in Section 11.

WSDOT intends to use rock fill from the rock slope cuts to construct the embankments to the extent feasible. A materials source report has been prepared that identifies rock cuts where select rock fill can be expected (Burk 2009).

13.1 PROJECT DESCRIPTION

Two embankment fills greater than 10 feet in height are planned within Phase 1C. The western fill is located west of RCB between approximately LE 1419+00 and LE 1424+85. In this area, the EB embankment will be extended lakeward approximately 70 feet at the top of the embankment and 115 feet at the embankment toe. This fill has a length of approximately 585 feet and a maximum height of about 70 feet from the top of the embankment to the embankment toe. The proposed embankment side slope is 2H:1V.

The eastern fill is located east of RCB between AE 100+00 and the EB end of project at AE 124+80. This fill has a length of 2,480 feet and a maximum fill thickness of about 30 feet beneath the EB lanes. The proposed embankment side slope is 1.5H:1V.

13.2 FIELD EXPLORATION AND LABORATORY TESTING

A total of 15 borings have been drilled in the new embankment areas, including:

- One boring drilled during the 2009 exploration program (EMB-027-09);

- Seven borings drilled in 2007;
- Four borings drilled in 2006; and
- Three borings drilled in 1998.

Figure 2.2C shows the locations of the borings. A summary of the borings is provided in Table 13.1. The log of EMB-027-09 is provided in Appendix A. Logs of earlier borings are provided in the 2007 Roadway Geotechnical Report.

Index properties, including grain-size distribution (ASTM D-422) and Atterberg Limits (ASTM D-4318), were determined for selected soil samples. The types and total numbers of laboratory tests are as follows:

- Grain-size distribution: six samples
- Atterberg Limits: four samples

The laboratory testing results are summarized in Table 13.2. The laboratory data sheets are provided in Appendix D.

13.3 SUBSURFACE CONDITIONS

13.3.1 Soil and Rock Conditions

URS conducted a reconnaissance of the lakeshore in the embankment fill areas on Sept 29, 2010 with the purpose of identifying soil that is unsuitable for supporting the proposed embankments. No unsuitable soils were found on the ground surface, in shallow soils (less than one foot below ground surface) underlying areas of standing water and ponded sediment on the surface, or in the walls of incised drainage channels.

Subsurface conditions within the new embankment construction areas are generally depicted in the EB subsurface profiles (Figures 4.1B and 4.1C) and five cross sections:

- LW 1419+26 (LE 1418+94)(SCW 5-5'; Figure 8.5);
- LW 1421+99 (LE 1421+84)(RCW 1-1'; Figure 10.1);
- LW 1424+97 (LE 1424+82)(RCW 2-2'; Figure 10.2);
- AE 111+24 (CUL 2-2'; Figure 16.2); and
- AE 121+03 (EMB 3-3'; Figure 13.1).

Three soil units and one rock unit have been identified in the new embankment construction areas.

Unit 1 (Gravel with Sand/Cobbles/Boulders [Fill]) consists of loose to dense, poorly- to well-graded gravel with sand, cobbles, and occasional boulders. This material is fill placed for construction of the existing roadway embankment. The thickness of this soil unit typically ranges from 20 to 30 feet.

Unit 2 (Gravel with Sand/Cobbles [Native Soil]) consists of medium dense to very dense, poorly- to well-graded gravel, silty gravel with sand and cobbles, and silty sand. The thickness encountered ranged from 20 to 60 feet, with the maximum thickness in the RCB area.

Unit 3 (Fine Sand and Silt) consists of loose to medium dense silty sand and fine sand, with some gravel. This unit is compressible and potentially liquefiable under earthquake loading. Its maximum thickness is about 60 feet between approximately LE 1422+00 and 1425+00. It thins to the west and east, pinching out between LE 1419+00 and 1421+00 to the west and between AE 100+00 and 111+00 to the east.

Unit 3 was also encountered at AE 121+03, where it is present directly below Unit 1. Unit 3 thicknesses ranging from 12 to 20 feet were encountered at this location.

Unit 4 (Bedrock) consists of moderately weak (R2) to moderately strong (R3) meta-welded lapilli tuff. The bedrock elevation beneath the proposed embankment fill varies over a wide range. The interpreted bedrock elevation beneath the LE line ranges from approximately El. 2415 to 2480 within the embankment fill areas, as shown in Figure 4.1. The top of bedrock is interpreted to dip down toward the lake.

13.3.2 Groundwater

Groundwater levels were monitored in two wells at RCB (Section 11.3.3) and in H-36-06(OW) (Figure 3.3), which is in the existing median near AE Sta.121+00. Based on the available data, the groundwater level is approximately equal to the lake level when the lake level is higher than about El. 2490 and 2505 at RCB and H-36-06(OW), respectively. During lower lake level periods, the groundwater levels stabilize at approximately these elevations during the fall. No groundwater levels have been measured during low lake level periods in the winter and spring.

The groundwater data suggest that when the lakeshore (embankment fill foundation) is exposed during low lake level periods, the groundwater level will probably be within a few feet of the lakeshore ground surface.

13.4 GEOLOGIC HAZARDS

13.4.1 Wave Erosion and Runup

Wave erosion and runup are addressed in the Phase 1B Hydraulic Report (WSDOT 2009a). The information in that report is also applicable to the Phase 1C embankments.

13.4.2 Liquefaction Potential

A discussion of liquefaction potential is presented in the 2007 Roadway Geotechnical Report. WSDOT does not mitigate instabilities due to design seismic events for embankments that do not support structures because of the high cost of applying such a design policy uniformly to all slopes statewide (GDM 6.1.2.1). Therefore, liquefaction and liquefaction-induced settlement and lateral spread were not quantitatively evaluated for embankments.

13.5 GLOBAL STABILITY

URS evaluated the stability of embankments under static loading at LE 1421+84 and AE 121+03. The stability analysis results are summarized in Table 13.3. Graphic results are provided in Appendix H.

13.5.1 Method of Analysis

For the slope stability analyses purposes, Units 1, 2, 3 were assumed to be free-draining and cohesionless, with effective angles of internal friction of 40, 38, and 30 degrees, respectively. The proposed embankment fill was assumed to be free-draining and cohesionless, with an effective angle of internal friction of 42 degrees. A Mohr-Coulomb model was used to characterize these units in the stability analyses.

URS evaluated the effect of the groundwater and lake levels for two cases at each cross section:

- High groundwater and high lake levels (El. 2517) at each cross section;
- A lower groundwater level (El. 2492) and low lake level at LE 1421+84;
- A lower groundwater level (El. 2506.5) and low lake level at AE 121+03.

For live load surcharge, a snow load surcharge equal to 320 psf was included.

13.5.2 Results

The factor of safety for global stability of an embankment under static loading should be 1.25 or greater (GDM, Section 9.2.3.1). The results of the stability analyses are summarized in Table 13.3.

LE 1421+84. This cross section was used to characterize stability for the proposed embankment fill between the eastern end of Slide Curve and the western end of the west RCB approach fill. The proposed embankment height and fill thickness are up to approximately 70 feet at this location. The proposed embankment side slope is 2H:1V.

Under low and high water conditions, the calculated factors of safety for static loading are 1.27 and 1.52, respectively. These factors of safety meet the criterion for embankments.

AE 121+03. At AE 121+03, the proposed roadway widening on the lake side would be accomplished by placement of a fill prism that would steepened the overall embankment slope. A 12 to 20 foot thick stratum of loose to dense silt/fine sand (Unit 3) is present below the area of proposed embankment construction. The presence of this stratum limits the calculated factor of safety and could adversely affect stability. The measured blow counts in Unit 3 ranged from 4 to 14 blows per foot in EMB-024-07, which was drilled on the existing roadway surface, and from 11 to 40 blows per foot in EMB-027-09, which was drilled at the existing embankment toe.

The location of the calculated failure circle, as shown in Figures H.9.3 and H.9.4, is generally downslope of EMB-024-07 and passes through EMB-027-09. Unit 3 encountered in EMB-027-09 is generally coarser-grained with higher blow counts than in EMB-024-07. To evaluate the effects of the uncertainty in soil parameters, URS evaluated stability assuming effective angles of internal friction of 30 and 34 degrees for Unit 3. An effective angle of internal

friction of 34 degrees for Unit 3 is consistent with the characteristics of the unit encountered in EMB-027-09. For effective angles of internal friction of 30 and 34 degrees for Unit 3, factors of safety of 1.09 and 1.26, respectively, were calculated for the critical low water condition under static loading.

Based on the soil conditions encountered in EMB-027-09, URS considers the calculated factor of safety of 1.26 to be representative of field conditions. It is recommended that the new embankment be observed during construction for any evidence of instability. Should instability be detected, mitigation measures should be undertaken.

Potential mitigation options include construction of a counterberm at the toe of the slope, flattening the slope, and lowering the groundwater table in Unit 3 by providing drainage. URS previously evaluated construction of a counterberm (URS 2008a). It was estimated that a counterberm that is 20 feet wide and 10 feet deep would provide an FS of greater than 1.25, assuming an effective angle of internal friction of 30 degrees for Unit 3.

13.6 GEOTECHNICAL RECOMMENDATIONS

This section presents recommendations for embankment construction. Filter blankets for slope protection, as needed, will be designed by WSDOT.

For 1.5H:1V embankments, URS understands that WSDOT is proposing to use a single select rock fill specification that will also meet both requirements for global stability and wave erosion protection. In this way, fill material production and placement will be simplified. A rock fill with a minimum D_{50} of 13 inches is needed for protection from wave erosion.

Requirements for select rock fill materials and placement for 1.5H:1V embankments were previously provided in the 2007 Roadway Geotechnical Report (URS 2008a). The select rock fill specification, modified for wave erosion protection, includes the following requirements:

- Maximum particle size: 18 inches diameter
- Minimum of 50 percent by weight greater than 13 inches in diameter
- Less than 20 percent by weight passing the 1 inch square sieve
- Less than 10 percent by weight of the portion that passes the 1 inch square sieve passing the U.S. No. 200 sieve
- Los Angeles Abrasion (500 rev): Maximum 50 percent loss
- Washington Degradation value: Minimum 15
- Weathering Category: I to II

The jar slake and slake durability tests previously specified have been determined to be inappropriate for the materials in the project area and have been deleted from the requirements.

Embankments with side slopes no steeper than 2H:1V may be constructed using soil that conforms to WSDOT Specification 9-03.14(3) for common borrow. If the soil contains less than 25 percent, by volume, of particles 4 inches in size or larger, the soil should be compacted according to WSDOT Specification 2-03.3(14)C, Method B. If the soil contains 25 percent or

more, by volume, of particles 4 inches in size or larger, the soil should be compacted according to WSDOT Specification 2-03.3(14)A.

It has been observed that water from Resort Creek and an ephemeral pond is draining through the existing embankment and underlying native soils between approximately LW 1420+80 and 1426+33. WSDOT intends to construct the new embankment in this area so that this drainage pattern is maintained. It is recommended that a minimum 3-foot thick layer of free-draining material be placed at the base of the new embankment. Fill conforming to the select rock fill specification is considered adequate to provide for drainage in this area. If finer-grained material is used to construct subsequent layers, it is recommended that the top of the select rock fill be "choked" to limit the migration of fines into the select rock fill.

WSDOT intends to use rock fill from rock cuts to construct embankments to the extent possible. A materials source report has been prepared that identifies rock cuts where select rock fill can be expected (Burk 2009).

The rock fill should be placed and compacted according to WSDOT Specification 2-03.3(14)A. The new embankment should be keyed into the existing embankment in accordance with WSDOT Standard Specification 2-03.3(14). The exposed subgrade should be evaluated after site grading is complete. Subgrade preparation should be in accordance with WSDOT Standard Specifications 2-06 and 2-09. Embankment fill should not be placed underwater or on frozen ground.

To enable proper compaction where the embankment fill to be placed is too narrow for compaction equipment, either part of the existing embankment should be excavated or the new embankment overbuilt to provide adequate space for equipment to operate. An overbuilt slope would be subsequently cut back to the proposed grade.

Boulders are present on the existing embankment slopes in the Resort Creek vicinity. For proper compaction, boulders larger than 18 inches in size should be removed prior to fill placement.

All materials that are proposed to be imported to the site for use as structural fill should be examined and tested. It will be necessary to monitor the site grading and verify soil compaction. The general condition of the slopes should be observed and evaluated periodically by a geotechnical engineer to confirm adequate stability, particularly immediately following any storm events.

Embankments that do not support structures are not designed for seismic conditions. However, it should be recognized that the potential exists for liquefaction in Unit 3 and associated settlements and instabilities, including localized lateral spreading.

13.7 CONSTRUCTION CONSIDERATIONS

The potential exists for significant post-construction settlement of the embankment under static loading between LE 1419+00 and 1424+85. One to two feet of total (construction plus post-construction) settlement may be experienced in this area. It is recommended that pavement not be placed until settlement is substantially complete. Settlement should be monitored to determine when it is substantially complete and final grading and paving can be undertaken.

Settlement should be monitored using survey hubs that are placed every 50 feet along the EB shoulder. Monitoring should begin as soon as practical after fill placement to establish an early baseline of elevations.

It is expected that most of the settlement would occur during fill placement, and about 90 percent of the remaining settlement would occur within about two weeks after fill placement. The rate of settlement will depend on drainage conditions within the fine-grained layer, which cannot be reliably characterized with the available subsurface information. It is recommended that these areas be overfilled to 2 feet above final grade and cut to final grade after two weeks to reduce the potential for settlement after placement of the pavement. If the monitoring data indicate that significant settlement is still occurring two weeks after completion of fill placement, fill grading and paving should be further postponed until the rate of settlement decreases.

The proposed embankment fill extends as low as El. 2462 in some areas between Slide Curve and RCB. As discussed in Section 2.1.3, during an "average" year the maximum lake level is El. 2511.4. During an "average" year, the lake level is below El. 2462 only from the beginning of September to the beginning of December, as shown in Figure 2.3. Table 2.2 shows the number of weeks the lake level is below various elevations for the high lake level year, average year, and low lake level year.

Since the lake level can rise above the toe of the proposed embankment fill in most areas, it is recommended that construction sequencing be such that the areas where the fill extends to the lowest elevations be constructed during the months when the lake levels are at their lowest. As shown in Figure 2.3, the "average" lowest lake levels are from August to early January. It is cautioned, however, that freezing temperatures can exist during the fall and winter months. Frozen soil should not be used for embankment fill.

14.0 SIGN STRUCTURES

The Phase 1C roadway includes seven proposed sign structures as follows:

- Four cantilever signs at LW 1338+00, LW 1385+00, LW 1421+30, and AE 116+30
- Three double mast arm light standards at LE 1363+00, 1365+33, and 1367+66

The types and locations of the proposed sign structures are summarized in Table 14.1. Preliminary design drawings for the four cantilever signs are provided in Appendix G.

14.1 SUBSURFACE CONDITIONS

Subsurface conditions were explored by drilling one boring, SSE-025-10, at the sign structure location at AE 116+30 and single borings, SSE-026-10 through SSE-028-10, at each of the three light standard locations, as summarized in Table 14.1. An existing nearby boring (RCW-001-08) was used to characterize subsurface conditions at LW 1421+30. Subsurface conditions were not explored at LW 1338+00 and LW 1385+00. LW 1338+00 is within a rock cut area, and LW 1385+00 is within the proposed SCW (Wall 8) fill. The borings are summarized in Table 14.2. Logs of the borings are provided in Appendix A. The boring locations are shown in Figures 2.1A through 2.1C.

Six additional borings were drilled in 2008 and 2009 at proposed sign locations; however, these signs were subsequently deleted or relocated. These borings are also summarized in Table 14.2 and the logs provided in Appendix A.

Laboratory testing included 16 sieve tests. The test results are summarized in Table 14.3.

One soil unit and one rock unit were encountered. Unit 1 consists of medium dense to very dense gravel with sand, silty sand with gravel, or silty gravel with sand. A boulder was encountered in one boring: RCW-001-08 near LW 1421+30 at a depth of 13 feet.

Unit 2 consists of moderately strong (R3) to very strong (R5) lapilli tuff. Bedrock was encountered at shallow depth in the borings for the light standards at LE 1363+00, 1365+33, and 1367+66. Bedrock will also be present at the cantilever sign locations at LW 1338+00, which is within an area of proposed rock cuts, and LW 1385+00. The depth to bedrock at LW 1421+30 and AE 116+30 is greater than 30 feet; therefore, bedrock will be below the bottom of the foundations at these locations. Estimated bedrock elevations are shown in Table 14.1.

14.2 GEOTECHNICAL RECOMMENDATIONS

Geotechnical recommendations for sign structures were developed using the guidelines presented in the GDM, Section 17. Foundation design recommendations were previously provided to WSDOT on July 29, 2010.

URS understands that WSDOT will use a standard plan drilled shaft foundation where feasible. Based on the soil types and SPT N-values, foundations placed in existing soils or compacted structural fill placed for road construction may be designed using a lateral bearing pressure equal to 2,500 psf.

Shallow bedrock is present at the sign structure locations at LW 1338+00, LE 1363+00, LE 1365+33, and LE 1367+66. For footings on rock, a lateral bearing pressure equal to 18,000 psf and an allowable vertical bearing pressure equal to 20,000 psf can be used for all limit states. For shaft foundations in rock, a minimum embedment into competent rock, measured at the downslope face of the shaft, equal to the shaft diameter is recommended. The shaft embedment below ground surface should not be less than five feet.

The GDM (17.2.1) states that the Standard Plan foundation depth should be increased where the centerline of the foundation is less than a distance B from the shoulder of an adjacent slope, where B is the foundation width, when the slope is steeper than 3H:1V, as follows:

- 2H:1V or flatter: add 0.5B to the depth
- 1.5H:1V or flatter: add 1.0B to the depth

14.3 CONSTRUCTION CONSIDERATIONS

The shaft excavation at LW 1385+00 could encounter bedrock that is sloped steeper than 1H:1V. It may be difficult to get the drilling tools to "bite" into the steeply-sloped rock. Consideration should be given to notching the rock at the shaft location before placing the SCW fill. This item of work could be included in the Order of Work document.

Large boulders are present on the existing embankment in the vicinity of the sign locations at LW 1421+30 and AE 116+30. The contractor should be prepared to handle boulders. New fill will be placed on the existing embankment at AE 116+30. Consideration should be given to removing the boulders from the sign location before placing the new fill.

Sign foundations at LW 1421+30 and AE 116+30 will be excavated into the permeable existing embankment fill, and the groundwater level should be expected to be equal to the lake level. If the lake level is above the bottom of the shaft, temporary casing will be needed. Lake levels are discussed in Section 2.1.3.

15.0 STORMWATER MITIGATION FACILITIES

This section provides recommended infiltration rates for use in the design of stormwater mitigation facilities at the HMF and Crystal Springs Sno-Park. This work is planned to be completed under Phase 1D/F.

15.1 PROJECT DESCRIPTION

The purpose of the stormwater mitigation facilities is to provide stormwater runoff treatment by applying stormwater best management practices (BMPs) that will remove pollutants and sediments to the fullest extent practicable before the stormwater is discharged to any of the water bodies.

An equivalent area approach will be used to compensate for the areas where installation of treatment facilities along the roadway is not feasible due to physical constraints. The equivalent area treatment approach allows retrofit of stormwater treatment onto existing off-site impervious surfaces with similar pollutant loading characteristics as the constrained areas.

The current Phase 1D stormwater mitigation plan incorporates the following types of treatment:

- Media filter drains (MFDs) (previously known as ecology embankments);
- Compost amended vegetated filter strips (CAVFSs); and
- Bioinfiltration swales.

Currently, stormwater treatment using bioinfiltration swales is planned at the HMF, and treatment using CAVFSs is planned at the Crystal Springs Sno-Park. In filtration rates are needed for bioinfiltration swales, but are not needed for CAVFSs. Other compensatory offsite impervious areas are currently planned to be treated using MFDs or CAVFSs. MFDs also do not require infiltration rates for design.

The stormwater mitigation plan at the HMF consists of the following:

- West side of the maintenance garage plan and
- East side of the maintenance garage plan.

West Side of the Maintenance Garage Plan. Stormwater from the west side of the maintenance garage will flow into a network of grated inlets and then combine with the flows from the existing inlets prior to entering into a proposed pre-treatment vault for removal of coarse sediment and floatables. Partially treated water from the vault will then be discharged to a proposed bioinfiltration swale that will run parallel to Coal Creek for further treatment. This bioinfiltration swale is referred to as the west swale for the purposes of this report.

East Side of the Maintenance Garage Plan. Stormwater from the east side of the maintenance garage will be conveyed to a pre-treatment vault similar to the one on the west side. Partially treated water from the vault will then be discharged to a proposed bioinfiltration swale, which will be constructed along the edge of the proposed fill area. This swale will discharge to an

armored embankment that will lead down to the lake. This bioinfiltration swale is referred to as east swale for the purposes of this report.

Crystal Springs Sno-Park. Stormwater runoff from the parking area will be routed to four parallel drainage ditches. Runoff water entering will be treated using CAVFSs and routed through a culvert to discharge north of the parking area.

The locations and types of facilities are based on maps that were provided by Otak Inc. and WSDOT to URS on February 16, 2009 and September 21, 2009, respectively, for the HMF and August 10, 2010 for Crystal Springs. These maps are provided in Appendix G.

15.2 FIELD EXPLORATION

Eight borings, HMF-001-08 through HMF-007-08 and HMF-008-09, were drilled and six test pits, TP-003-08, HMF-TP-001-09, HMF-TP-002-09, HMF-TP-003-09, HMF-TP-004-10, and HMF-TP-005-10 were excavated at the HMF during the 2008-2009 exploration programs.

HMF-001-08 through HMF-003-08, HMF-008-09, and HMF-TP-005-10 are located at/near the east swale. HMF-004-08 through HMF-007-08, TP-003-08, HMF-TP-001-09 through HMF-TP-003-09, and HMF-TP-004-10 are located at/near the west swale. The elevation of HMF-TP-004-10 was not surveyed, but was observed to be approximately the same as HMF-TP-002-09 (approximately El. 2534).

Three borings, INF-010-10(OW), INF-012-10(OW), and INF-015-10, were drilled and six test pits, INF-TP-006-10, INF-TP-007-10, INF-TP-009-10, INF-TP-011-10, INF-TP-013-10, and INF-TP-015-10 were excavated at the Crystal Springs Sno-Park. INF-010-10(OW) and INF-012-10(OW) were completed as groundwater OWs.

Figures 2.4 and 2.5 show the locations of the borings and test pits for the HMF and Crystal Springs Sno-Park, respectively. Information on the borings and test pits is provided in Table 15.1, and logs of the borings and test pits are provided in Appendix A.

15.3 LABORATORY TESTING

Laboratory testing consisted of grain-size analyses needed for calculating infiltration rates. Totals of 19 and 14 grain-size analyses were completed at the HMF and Crystal Springs Sno-Park, respectively. Grain-size distribution information is presented in Tables 15.2 and 15.3. The grain-size curves are provided in Appendix D.

15.4 SUBSURFACE CONDITIONS

15.4.1 HMF West Swale

The following two soil units have been identified:

Unit 1 (Sand with Gravel [Fill/Top Soil] consists of loose to medium dense, silty sand with gravel or well graded sand with gravel, occasional wood, and occasional cobbles. Cobbles were encountered in TP-003-08 and the maximum size of the cobbles encountered is 11 inches. The thickness of this unit ranges from 1.5 to 6.0 feet. The thickness typically ranges between 1.5 and

2.5 feet on the northern side of the swale where HMF-TP-001-09 and HMF-TP-002-09 are located and between 3.0 and 6.0 feet on the southern side of the swale where HMF-TP-003-09 and TP-003-08 are located.

Unit 2 (Gravel with Sand/Cobbles/Boulders) consists of medium dense to very dense, silty gravel with sand or poorly to well graded gravel with sand, occasional well graded sand or silty sand with gravel, occasional trace of wood and occasional cobbles and boulders. Cobbles were encountered in all the four test pits. Boulders were encountered in all the test pits, except for HMF-TP-002-09. The maximum size of the cobbles encountered was 11 inches, and the maximum size of the boulders encountered was 16 inches. All of the test pits were terminated in this unit; therefore, its vertical extent is not known.

HMF-TP-004-10 was excavated to assess the depth to groundwater at the West Swale during high water conditions. A groundwater depth of 5.5 feet was measured on June 2, 2010 at the completion of pit excavation, which corresponds to a groundwater elevation of approximately 2528.5.

15.4.2 HMF East Swale

The following three soil units and one rock unit have been identified:

Unit 1 (Gravel with Sand [Fill] consists of very loose to very dense, poorly graded gravel with sand and traces of silt and clay, occasional well graded sand with gravel. The thickness of this unit ranges from 2.3 to 13.0 feet. The thickness of this unit increases from northern side of the swale, where HMF-008-09 to the southern side of the swale, where HMF-003-08 is located.

Unit 2 (Organic Soil) consists of soft to medium stiff, sandy organic soil with trace of fine gravel. This unit was encountered only in HMF-001-08 below Unit 1. The thickness of this unit is approximately 5 feet.

Unit 3 (Silty Sand/Silty Gravel) consists of very loose to very dense, silty gravel with sand or silty sand with or without gravel. This unit was not encountered in HMF-008-09. The thickness of the unit ranges from approximately 2.0 to 9.5 feet.

Unit 4 (Bedrock) consists of very strong (R5) basalt. Bedrock was encountered only in HMF-008-09. Bedrock is estimated to slope down from the northern to the southern side of the swale.

Groundwater was not encountered in HMF-TP-005-10 at the East Swale above the bottom of the pit at a depth of 11 feet on June 2, 2010, which indicates a groundwater level below El. 2534.4 at that time.

15.4.3 Crystal Springs Sno-Park

A single soil unit was encountered at Crystal Springs that consists of medium dense to very dense coarse-grained sands, gravels, silty sands, and silty gravels with occasional cobbles to the maximum depth explored (26 feet bgs).

Groundwater was observed in the test pits and borings at the completion of excavation/drilling in June 2010 and subsequently monitored in two OWs. At the completion of excavation/drilling,

groundwater was observed at elevations ranging from below El. 2411 (dry hole) to 2417.8. Water was observed in INF-TP-015-10 at El. 2443, but this level is not considered to be representative of actual groundwater conditions. Groundwater was not present in OWs INF-TP-010-10 (bottom of well at El. 2419) and INF-TP-012-10 (bottom of well at El. 2411) during monitoring events in July and August 2010. INF-TP-010-10 could not be located in August 2010 and may have been buried during site grading activities.

Two ponds (Ponds 1 and 2) are present within the site, and water levels in these ponds may be indicative of groundwater elevations. The surveyed water elevations in Ponds 1 and 2 on July 1, 2010 were El. 2417.2 and 2421.0, respectively. The lowest vegetation levels at the ponds were about 5 feet above the water levels, which suggests the highest groundwater levels may be approximately El. 2422 in the Pond 1 vicinity and El. 2426 in the Pond 2 vicinity.

15.5 ESTIMATION OF INFILTRATION RATES

Infiltration rates were estimated using the detailed approach described in Section 4-5.3 of the Highway Runoff Manual (HRM). Soil grain-size distribution and groundwater information interpreted from the closest borings and test pits were used to estimate the infiltration rates. In general, the interpreted highest groundwater elevations were used to estimate the infiltration rates.

At the HMF, the highest groundwater elevations were interpreted to be El. 2530 at the West Swale and El. 2534 at the East Swale, based on observed groundwater elevations in HMF-TP-004-10 and HMF-TP-005-10, respectively, on June 2, 2010.

Although groundwater OWs have been installed at the Crystal Springs Sno-Park, the highest groundwater conditions did not occur between the time the wells were installed and the date of this report. Infiltration rates for the Crystal Springs Sno-Park were calculated for various depths to groundwater beneath the base of the infiltration facility. The two groundwater OWs installed in 2010 will be used to assess the high groundwater level.

The soil properties, groundwater and trench elevations, and calculated infiltration rates are presented in Tables 15.2 and 15.3 for the HMF and Crystal Springs Sno-Park sites, respectively. A correction factor for pond size of 1.0 was selected for a facility area of less than two-thirds of an acre. A reduction factor for potential biofouling or siltation of 0.8 was selected, which assumes that potential for siltation and biofouling is low at the facility. The depth of water in the infiltration facility was assumed to be 0.5 feet.

Estimated infiltration rates for the Crystal Springs Sno-Park were previously provided to WSDOT on August 20, 2010.

15.6 DISCUSSION

URS understands that an infiltration rate of at least 2 feet per day (1 inch per hour) is needed for the infiltration facilities to be feasible without the use of an underdrain. Based on soil and groundwater information from nearby borings and base of infiltration facility elevations provided by WSDOT, calculated infiltration rates at the HMF swales generally ranged from 0.5 to 1.7 feet per day. The HRM indicates a minimum separation of 5 feet is needed between the bottom of the infiltration facility and the high groundwater level or a low-permeability layer. This separation can be reduced to 3 feet if judged by the site professional to be adequate to prevent overtopping

and meet the other site selection criteria. Based on the bottom of trench elevations shown in Table 15.2 and the estimated high groundwater level, the infiltration facilities at the HMF appear to meet the required 5-foot separation.

URS understands that WSDOT is currently planning to use CAVFSs at the Crystal Springs Sno-Park, and stormwater infiltration will not be used. However, WSDOT requested that URS calculate infiltration rates. The infiltration rate calculations indicate that the bottom of the infiltration facility generally would need to be at least 5 feet above the estimated high groundwater level shown in Table 15.3 to achieve an infiltration rate of 2 feet per day or higher.

16.0 MILEPOST 59.7 UNNAMED CREEK CROSSING

16.1 PROJECT DESCRIPTION

The existing EB and WB roadways will be widened on the lake side at MP 59.7 Unnamed Creek under Phase 1C. The EB and WB crossings will be designed as permanent and temporary crossings, respectively. Both crossings will be replaced with permanent crossings under Phase 2 of the project.

The design concept for both crossings consists of 25-foot long spans. The EB and WB crossing are supported on 24-inch diameter shafts and micropiles, respectively. The superstructures will consist of voided concrete slabs that are 11.5 feet wide EB and 8 feet wide WB. The EB span extends from AE 111+06 to 111+31 and the WB span from AW 112+30 to 112+55.

The approaches will be supported on the lake side by four walls:

- Wall 19: EB west approach
- Wall 20: EB east approach
- Wall 21: WB west approach
- Wall 22: WB east approach

Walls 19 and 20 are each 20 feet long and up to 12.5 feet high. Walls 21 and 22 are each 10 feet long and up to 7.5 feet high.

This section presents geotechnical recommendations for design of the micropile and shaft foundations and the retaining walls. Geotechnical recommendations for a permanent crossing consisting of a box culvert were presented in Section 10 of the 2007 Roadway Geotechnical Engineering Report (URS 2008a). The stream was referred to as Townsend Creek North in that report.

16.2 FIELD INVESTIGATION

Four borings have been drilled at the crossing: CUL-015-07(OW), CUL-016-07, CUL-019-10, and CUL-020-10. A URS geologist logged CUL-015-07(OW), CUL-016-07, and CUL-020-10. A WSDOT inspector logged CUL-019-10. CUL-015-07(OW) was completed as a groundwater OW.

Figure 16.1 shows the locations of the borings. Boring information is summarized in Table 16.1, and the logs of CUL-019-10 and CUL-020-10 are provided in Appendix A. The logs of CUL-015-07(OW) and CUL-016-07 are provided in the 2007 Roadway Geotechnical Report (URS 2008a).

16.3 SITE DESCRIPTION

The existing crossings consist of 10-foot wide box culverts with wing walls both EB and WB. Hydraulic information for Unnamed Creek is provided in Appendix K of the Draft Hydraulic Report (Otak 2008a). Unnamed Creek is referred to as Townsend Creek North in that report.

16.4 SUBSURFACE CONDITIONS

The interpreted subsurface conditions are depicted in subsurface cross section CUL 2-2' at AE 111+24 (Figure 16.2). Four soil units and bedrock were encountered at the site.

Unit 1 is the existing embankment fill. It consists of loose to very dense gravel with sand and cobbles, and may contain boulders. Unit 1 is interpreted to be up to approximately 8 feet thick at the crossing location.

Unit 2 consists of loose, compressible clayey sand/clayey silt. This unit was encountered only in CUL-016-07 at the south shoulder of the existing EB roadway, where it is present between El. 2512 and 2516.

Unit 3 consists of medium dense to very dense gravel with sand, cobbles, and boulders. Boulders up to at least 3 feet in size were encountered. An 8.5-foot boulder may have been encountered in CUL-019-10.

Unit 4 is glacial till, which consists of very dense clayey sand with gravel and silty gravel. Unit 4 was encountered only in CUL-015-07(OW) at the north shoulder of the existing WB roadway, where it is present between El. 2509 and 2523.

Unit 5 is bedrock, which consists of moderately strong (R3) to strong (R4) andesite. The bedrock dips down toward the lake. It was encountered at El. 2509 in CUL-015-07(OW) and at El. 2494 in CUL-020-10 at the north and south shoulders of the existing WB roadway, respectively.

Groundwater was monitored in CUL-015-07(OW) during April through October 2008, May through July 2009, and June through August 2010. The depth to groundwater ranged from approximately 9 to 12 feet. Groundwater elevations ranged from El. 2524.5 to 2527.5. The groundwater levels are shown graphically in Figure 3.1.

16.5 GEOLOGIC HAZARDS

16.5.1 Earthquake Engineering

URS understand that WSDOT has determined that the WB crossing does not need to be designed for seismic loading because it is a temporary structure. The EB structure is being designed as a permanent structure and must be designed for seismic loading. Descriptions of site seismicity, ground motion parameters, and parameters for use in seismic modeling of the EB crossing are provided in Section 5.1.

16.5.2 Scour

Scour was evaluated by Otak under subcontract to URS. The results of the scour analysis for Unnamed Creek are provided in Appendix K of the Draft Hydraulic Report (Otak 2008a). Unnamed Creek is referred to as Townsend Creek North in that report.

16.6 GLOBAL STABILITY

URS evaluated the global stability of the proposed gravity block approach fill retaining wall under static and seismic loading at the interpreted critical section at AE 111+05. For the static analysis, a snow load equal to 320 psf was included. Stability under static loading was analyzed at both high and low lake and groundwater levels.

For seismic loading, a horizontal inertial force was included. For the permanent EB crossing, the inertial force was calculated using a horizontal acceleration coefficient, k_h, equal to 0.5 times the PGA (i.e., $k_h = 0.175$). A traffic load of 125 psf was included. Stability under seismic loading was analyzed at the interpreted average lake and groundwater levels.

The walls were analyzed for varying wall heights (i.e., varying numbers of blocks) and both with and without a toe berm. The target factors of safety were 1.3 and 1.1 for static and seismic loading, respectively. For static loading and the proposed wall height at AE 111+05 (five blocks), the calculated factors of safety were 1.46 and 1.30 for low and high lake and groundwater levels, respectively. For seismic loading, the calculated factor of safety was 1.09. If a 5-foot wide, 10-foot high toe berm were to be constructed, the calculated factors of safety would increase to 1.51 and 1.23 for static loading (high lake and groundwater levels) and seismic loading, respectively. The results are summarized in Table 16.2. Graphical presentations of the analyses are provided in Appendix H.

16.7 GEOTECHNICAL RECOMMENDATIONS

This section presents geotechnical recommendations for design of foundations consisting of micropile or small-diameter shafts and retaining walls.

16.7.1 Micropile Foundations

A micropile is a small-diameter (typically less than 12 inches), drilled and grouted non-displacement pile that is typically reinforced.

Micropile axial capacities were calculated following the 2008 AASHTO LRFD Bridge Design Specifications, Section 10.9, assuming skin friction only. Axial capacities were developed for micropile types B or D with a bonded diameter of six inches and assumed cased lengths of 5 and 10 feet. The recommended minimum pile spacing is five pile diameters center-to-center (30 inches for a 6-inch diameter micropile).

A grout-to-ground bond strength equal to 5.0 ksf was used to calculate the axial capacity. The grout-to-ground bond strength was selected from the presumptive values presented in Table C10.9.5.3.2-1 of the 2008 AASHTO LRFD Bridge Design Specifications.

The recommended unfactored axial compressive resistance as a function of micropile depth is presented in Table 16.3. Capacities for pile depths not explicitly show in the table may be calculated using linear interpolation. A resistance factor equal to 0.55 is recommended. The factored uplift resistance may be taken as equal to one-half of the factored compressive resistance.

Vertical micropiles should be assumed to have no lateral load capacity. Lateral load resistance should be obtained by battering the micropiles. The lateral resistance of battered micropiles can be computed as the horizontal component of the axial pile compressive capacity.

The micropile capacity must be verified in the field through performance of load test as described in the AASHTO LRFD Bridge Design Specifications, Article 10.9.3.5.4

16.7.2 Shaft Foundations

The EB abutment foundations consist of two 24-inch diameter shafts per abutment. Each abutment has a 14-foot long shaft cap, with the shaft centerlines 3.0 feet from each end of the cap (8.0-foot shaft spacing center-to-center). The elevation of the base of the shaft cap at its lake side edge is approximately El. 2519.

The factored shafts loads in kips per shaft were provided to the URS Seattle office on December 21, 2010, as shown below.

	Factored Shaft Loads, kips per shaft	
Direction	**Service**	**Strength**
Axial	88	127
Lateral, Longitudinal	4.5	6.6
Lateral, Transverse	3.8	3.8

The lateral loads act 1.5 feet above the base of the shaft cap.

Bearing Resistance. Bearing resistance was calculated using the methods provided in the AASHTO LRFD Bridge Design Specifications, Section 10.8.3.5 for the strength and extreme event limit states and Section 10.8.2.2 for the service limit state, assuming an allowable settlement of one inch. The calculated unfactored bearing resistance as a function of depth is shown in Figure 16.3 for a 24-inch diameter shaft.

The skin and tip resistance must be multiplied by the resistance factors for the various limit states that are shown in Table 6.5.

The abutment consists of a single row of shafts spaced 8 feet center-to-center (i.e., a spacing of 4 times the shaft diameter, or 4D). Based on a WSDOT Design Memorandum dated June 26, 2010, the bearing resistance group reduction factor for shafts spaced 4D in cohesionless soils is 1.0.

URS recommends that a temporary casing be used above the groundwater table elevation. At the EB crossing location, the groundwater table is expected to be approximately equal to the lake level, which fluctuates seasonally. In an “average year,” the lake level fluctuates between approximately El. 2450 and 2510.

Lateral Load Analysis. URS evaluated the shaft response to lateral loads using the computer program DFSAP. Because the shaft cap is largely exposed at its lake side end, soil resistance against the cap was neglected.

The assumed shaft properties were as follows:

Diameter: 24 inches
Length: 20 feet
Concrete Compressive Strength: 4.0 ksi
Percent Longitudinal Reinforcement: 1.0
Cover Over Reinforcement: 2.5 inches

The soil properties used in the analysis are summarized in Table 16.4. Free head and fixed head conditions were assumed for the longitudinal and transverse loading analyses, respectively.

Shaft-head deflections of 0.066 inches and 0.025 inches were calculated in the longitudinal and transverse directions, respectively. A maximum moment of 30 kip-feet was calculated in both the longitudinal and transverse directions. Plots of deflection, moment, shear, and line load versus depth are provided in Figures 16.4 through 16.7 for the longitudinal direction and Figures 16.8 through 16.11 for the transverse direction.

16.7.3 Retaining Walls

URS understands that WSDOT is proposing to use a concrete block wall to retain the approach fills for the MP 59.7 Unnamed Creek EB and WB crossings (Walls 19 through 22). Since the maximum wall height is 12.5 feet, a block wall is appropriate for this application. The design must include provisions for drainage of groundwater behind the wall.

The wall design should be consistent with the wall manufacturer's recommendations for maximum wall height and inclination and block orientation. For global stability, the wall should meet the following:

- Minimum wall depth behind the wall face of 2.5 feet in the top 5 feet of the wall;
- Minimum depth of 5 feet between 5 and 10 feet below the top of the wall;
- Minimum depth of 7.5 feet between 10 and 15 feet below the top of the wall; and
- Maximum wall face inclination of 1H:8V.

A layer of loose clayey sand/clayey silt (N = 4 blows per foot) was encountered in boring CUL-016-07 near the EB walls between approximately El. 2511.5 and 2515.5. It is recommended that this material be overexcavated and replaced with properly compacted structural fill.

It is understood that vendor software will be used to design the wall, including bearing resistance, lateral earth pressure and sliding, overturning, and internal stability. Assuming that the loose clayey sand/clayey silt soils are overexcavated and the wall excavation is backfilled with structural fill, the recommended soil properties for use in design of the wall are:

- Internal angle of friction = 40 degrees
- cohesion = 0
- unit weight = 135 pounds per square foot

A live load surcharge equal to 250 psf is recommended for static design of the wall. A load factor for lateral earth pressure loads equal to 1.5 is recommended. Resistance factors recommended for design are provided in Table 16.5.

16.8 CONSTRUCTION CONSIDERATIONS

Boulders up to 8.5 feet in size were encountered in a boring drilled in the existing median area. The contractor will need to consider the presence of boulders in his selection of drilling and shoring methods.

17.0 PHASE 1C PS&E SUPPORT

URS provided on-call responses to WSDOT information requests for development of the Phase 1C plans, specifications, and estimates (PS&E). Most of the information is incorporated into this report in the sections for the various structures. This section documents additional information provided to WSDOT that is not provided elsewhere in this report.

The information was typically transmitted to WSDOT by email. The information provided is attached in Appendix K and includes the following:

Appendix	Structure	Description	Date
K.1.1	SSD	Portal tower loads	March 6, 2009
K.1.2	SSD	Utility room	June 12, 2009
K.1.3	SSD/SCB	Sediment transport	October 6, 2009
K.1.4	SSD (Wall 1)	Catchment ditch cut	August 23, 2010
K.1.5	SSD	Grade separation barrier	October 1, 2010
K.1.6	SSD	Lateral earth pressure for water tank vault	March 4, 2011
K.1.7	SSD	Wall 9D rock elevation and bearing capacity	November 5, 2010
K.2.1	RCB	Flow failure loads on shafts	March 6, 2009
K.3.1	Cut slope	Overburden cut AW 112+70 to 118+15	October 4, 2010

18.0 LIMITATIONS

This report has been prepared by URS to assist WSDOT and URS in the engineering design and construction of the subject project. It should not be used, in part or in whole, for other purposes without contacting the URS geotechnical engineers in the "Acknowledgements" section for a review of the applicability of such reuse. This report should be made available to prospective contractors for their information or factual data only and not as a warranty of ground conditions.

The recommendations contained in this report are based on URS' understanding of the project at the time that the report was written, on site conditions that existed at the time of the field explorations, and on subsurface conditions as interpreted and interpolated from the geotechnical investigations. If significant changes to the nature, configuration, or scope of the project occur during the design process, URS should be consulted to determine the impact of such changes on the recommendations presented in this report.

The site exploration and testing results characterize subsurface conditions only at the locations of subsurface exploration and at the intervals where samples were collected. These data are interpreted by URS, which then renders an opinion regarding the general subsurface conditions. The distribution, continuity, thickness, and characteristics of identified (and unidentified) subsurface materials may vary considerable from that indicated by the subsurface data. Planes of weakness in rock that can strongly affect the strength of the rock mass may not be revealed by the borings.

While this variability cannot be eliminated, URS and WSDOT will need to reduce the impacts of variability on project design, construction, and performance. Periodic observation by geotechnical engineers during construction may be beneficial in this respect.

The recommendations presented in this report assume that surface and subsurface conditions, as observed during field exploration activities, are representative of the site conditions throughout the project area. Actual subsurface conditions can be discovered only during earthwork and construction operations.

19.0 REFERENCES

Allen, T.M. and M.A. Frye. 2010. I-90, MP 55.1 to 59.9, XL-2779. Snowshed to Keechelus Dam Phase 1C Replace Snowshed and Add Lanes. Snowshed Temporary Soil Nail Wall Geotechnical Recommendations. September 7, 2010.

American Association of State Highway and Transportation Officials (AASHTO). 2009. AASHTO Guide Specifications for LRFD Seismic Bridge Design. 1st Edition.

AASHTO. 2007. AASHTO LRFD Bridge Design Specifications. Fourth Edition.

Bartlett, S.F. and T.L. Youd. 1995. Empirical predictions of Liquefaction – Induced Lateral Spread. Journal of Geotechnical Engineering. Vol. 121, No. 4, p. 316-329.

Barton, N.R. and Bandis, S.C. 1990. Review of Predictive Capabilities of JRC-JCS Model in Engineering Practice. In Rock Joints, Proc. International Symposium on Rock Joints. Loen, Norway (eds N. Barton and O. Stephenson), p 603 – 610. A.A. Balkema Publishers, Rotterdam, Netherlands.

Bray, J.D. and T. Travasarou. 2007. Simplified Procedure for Estimating Earthquake-Induced Deviatoric Slope Displacements. ASCE Journal of Geotechnical and Geoenvironmental Engineering. Vol. 133, No. 4, pp. 381-392.

Burk Geoconsult LLC. 2008. Draft Interim Technical Memorandum - Material Source Evaluation. I-90 Snoqualmie Pass 2007 Geotechnical Program. March 2008.

Burk Geoconsult, LLC. 2009. Materials Source Evaluation. I-90 Snoqualmie Pass East Project. 2008 Geotechnical Investigation and Reporting. February 2009.

Chadbourne, D. and L. Moses. 1994. Hyak to Ellensburg Reconnaissance Report. SR90, MP 55.17 to MP 110.00. WSDOT report. 1994.

CivilTech Corporation. 2005. LiquefyPro5.2b Liquefaction and Settlement Analysis Software Manual. October 2005.

Duncan, J., R. Horz, and T. Yang,. 1989. Shear Strength Correlations for Geotechnical Engineering. Virginia Tech Department of Civil Engineering. August 1989.

Elias, V., B. Christopher, and R. Berg. 2001. Mechanically Stabilized Earth Walls and Reinforced Soil Slopes Design and Construction Guidelines. FHWA Report No. FHWA-NHI-00-043. March 2001.

Elias, V. and I. Juran. 1991. Soil Nailing for Stabilization of Highway Slopes and Excavations. Publication FHWA-RD-89-198.

Washington State Department of Ecology (Ecology). 2004. Washington State Wetland Rating System for Eastern Washington. Revised. Ecology Publication No. 04-06-15. August.

Federal Emergency Management Agency (FEMA). 2000. Prestandard and Commentary for the Seismic Rehabilitation of Buildings. FEMA 356. Prepared by the American Society of civil Engineers. November 2000.

Gipner, M. 2009. Information provided to URS by M. Gipner of the I-90 Project Office on February 23, 2009.

Golbek, S. 2010a. Email from Scott Golbek Re: Power Transformer building layout sketch (amended). December 20, 2010.

Golbek, S. 2010b. Email from Scott Golbek to URS Re: Slide Curve Bridge MSE Walls. July 14, 2010.

Golbek, S. 2010c. Email from Scott Golbek to URS Re: Slide Curve Bridge MSE Walls. August 4, 2010.

Golder Associates Inc. 2005. Geotechnical Consultants Summary Geotechnical Report on I-90 Keechelus Lake Alignment (MP 57.5 to MP 59.4). October 31, 2005.

Golder Associates Inc. 1998. Marine Geophysical Investigation of Lake Keechelus Adjacent to SR-90, MP 56.4 and 60.5. December 10, 1998.

Hoek, E, C. Carranza-Torres, and B. Corkum. 2002. Hoek-Brown Criterion – 2002 Edition. Proceedings NARMS-TAC Conference, Toronto, 2002, 1, pp. 267-273.

Hoek, E., and E. Brown. 1988. The Hoek-Brown Failure Criterion – A 1988 Update. Proceedings, 15th Canadian Rock Mechanics Symposium, Toronto, Canada.

Ishihara, K., and Yoshimine, M. 1992. Evaluation of Settlements in Sand Deposits Following Liquefaction During Earthquakes. Soils and Foundations, JSSMFE, Vol. 32, No. 1, March, pp. 173-188.

Kramer, S. 2008. Evaluation of Liquefaction Hazards in Washington State. Prepared for the Washington State Department of Transportation.

Kulhawy, F. and F. Mayne. 1990. Manual on Estimating Soil Properties for Foundations. Electric Power Research Institute (EPRI), EPRI EL-6800, Research Project 1493-6, Final Report 1990, Prepared by Cornell University, Ithaca, New York, August 1990.

Ladd, C. and R. Foote. 1974. New Design Procedure for Stability of Soft Clays. Journal of the Geotechnical Engineering Division, Proceedings of the America Society of Civil Engineers, Vol.100, No. GT7. July 1974.

Lazarte, C., V. Elias, R.D. Espinoza, and P. Sabatini. 2003. Geotechnical Engineering Circular No. 7, Soil Nail Walls. Prepared for the Federal Highway Administration. Report No. FHWA0-IF-03-017. March 2003.

Leyendecker, E., A. Frankel, K. Rukstales. 2007. Seismic Design Parameters for 2007 AASHTO Seismic Design Guidelines.

Macnab, A. 2010. Memorandum to Mo Sheikhizadeh, WSDOT, Re: I-90 Snow Sheds (28 September Review). October 7, 2010.

Macnab, A. 2009. Memorandum to Mo Sheikhizadeh, WSDOT, Re: Snoqualmie Pass Snow Sheds Project Review – 11/5. November 6, 2009.

Macnab, A. 2008. Report of Contractor's Panel of ADSC/WSDOT Task Force. Memorandum to WSDOT Re: I-90 Snoqualmie Pass East Project. April 28, 2008.

Mononobe, N. 1929. Earthquake-Proof Construction of Masonry Dams. *In* Proceedings, World Engineering Conference. Vol. 9, p. 275.

Morrison, K., F. Harrison, J. Collin, A. Dodds, and B. Arndt. 2006. Shored Mechanically Stabilized Earth (SMSE) Wall Systems Design Guidelines. Prepared for the Federal Highway Administration. Report No. FHW-CFL/TD-06-001. February 2006.

Northwest Geophysical Associates. 2009. Draft. Seismic Refraction and MASW Surveys. I-90 Snoqualmie Pass Phase 1C. December 3, 2009.

Okabe, S. 1926. General Theory of Earth Pressure. Journal of the Japanese Society of Civil Engineers. Vol. 12, No. 1.

Olson, S. and T. Stark. 2002. Liquefied Strength Ratio from Liquefaction Flow Failure Case Histories. Canadian Geotechnical Journal, Vol. 39, pp. 629-647.

O'Neill, M. and L. Reese. 1999. Drilled Shafts: Construction Procedures and Design Methods. FHWA-IF-99-025.

Otak, Inc. 2008a. Draft Hydraulic Report. I-90 Snoqualmie Pass East. October 31, 2008.

Seed, R.B. and L.F. Harder. 1990. SPT-based Analysis of Cyclic Pore Pressure Generation and Undrained Residual Strength. Proceedings, H. Bolton Seed Memorial Symposium, University of California, Berkeley. Vol. 2, pp. 351-376.

Shannon & Wilson, Inc. 1974. Geophysical Investigation Snoqualmie Pass Highway Keechelus Lake Vicinity. January 18, 1974.

Stark, T. D. and Olson, S. M.. 1995. Liquefaction Resistance Using CPT and Field Case Histories. Journal of Geotechnical Engineering, ASCE, Vol. 121, No12, pp. 856-869.

Tabor, R.W., V.A. Frizzell Jr., D.B. Booth, and R.B. Waitt. 2000. Geologic Map of the Snoqualmie Pass 30 x 60 Minute Quadrangle, Washington: U.S. Geological Survey, Map I-2538, 57 p., 2000.

Tokimatsu, K. and H.B. Seed. 1987. Evaluation of Settlements in Sand Due to Earthquake Shaking. Journal of Geotechnical Engineering, Vol. 113, No. 8, pp. 861-878.

URS Corporation. 2010. Preliminary Technical Memorandum – Snowshed West Portal MSE Walls. August 25, 2010.

URS Corporation. 2009a. Phase 1B Roadway Geotechnical Report. I-90 Snoqualmie Pass East Project. 2008 Geotechnical Investigation and Reporting. September 2009.

URS Corporation. 2009b. Addendum to Technical Memorandum No. 1 – Gold Creek Bridges. I-90 Snoqualmie Pass East Project. 2008 Geotechnical Investigation and Reporting. February 2009.

URS Corporation. 2009c. Supplement to Technical Memorandum No. 1 – Gold Creek Bridges. I-90 Snoqualmie Pass East Project. 2008 Geotechnical Investigation and Reporting. September 2009.

URS Corporation. 2009d. Supplement 2 to Technical Memorandum No. 1 – Gold Creek Bridges. I-90 Snoqualmie Pass East Project. 2008 Geotechnical Investigation and Reporting. December 2009.

URS Corporation. 2009e. Addendum to Technical Memorandum No. 2 – Rocky Run Bridges. I-90 Snoqualmie Pass East Project. 2008 Geotechnical Investigation and Reporting. February 2009.

URS Corporation. 2009f. Draft Construction Optimization Report - Construction Optimization Workshop. I-90 Snoqualmie Pass East Project. January 2009.

URS Corporation. 2009g. Memorandum. Snowshed Shaft Foundations: Rock Strength Estimates for DFSAP Analysis - I-90 Snoqualmie Pass East. April 29, 2009.

URS Corporation. 2008a. 2007 Roadway Geotechnical Report. I-90 Snoqualmie Pass East Project. 2007 Geotechnical Analysis and Reporting. September 2008.

URS Corporation. 2008b. Final Technical Memorandum No. 3 – Snowshed Replacement. I-90 Snoqualmie Pass East Project. 2007 Geotechnical Analysis and Reporting. July 2008.

URS Corporation. 2008c. Final Technical Memorandum No. 4 – Structural Wall 1. I-90 Snoqualmie Pass East Project. 2007 Geotechnical Analysis and Reporting. July 2008.

URS Corporation. 2008d. Final Technical Memorandum No. 5 – Structural Wall 2. I-90 Snoqualmie Pass East Project. 2007 Geotechnical Analysis and Reporting. July 2008.

URS Corporation. 2008e. Final Technical Memorandum No. 1 – Gold Creek Bridges. I-90 Snoqualmie Pass East Project. 2007 Geotechnical Analysis and Reporting. June 2008.

URS Corporation. 2008f. Final Technical Memorandum No. 2 – Rocky Run Bridge. I-90 Snoqualmie Pass East Project. 2007 Geotechnical Analysis and Reporting. April 2008.

URS Corporation. 2008g. Final Technical Memorandum No. 1 – Instrumentation Plan. I-90 Snoqualmie Pass East Project. 2007 Geotechnical Investigation. June 2008.

URS Corporation. 2008h. Gold Creek MSE Wall: Embankment Alternatives – Discussion of Potential Earthquake Stability Considerations. Memorandum from URS to WSDOT, July 14, 2008.

URS Corporation. 2008i. Additional Considerations - Structural Wall 1 and Structural Wall 2. I-90 Snoqualmie Pass East Project. Retaining Wall Design. December 2008.

URS Corporation and Arthur I. Mears, P.E. Inc. 2007. Avalanche Mitigation Report, Snowshed Option, I-90 Snoqualmie Pass East. December 2007.

URS and KPFF Consulting Engineers. 2008. Addendum to Preliminary Wall Concept Study, I-90 Snoqualmie Pass East. Hyak To Keechelus Dam, Washington. April 2008.

URS Corporation and KPFF Consulting Engineers. 2007. Preliminary Wall Concept Study. I-90 Snoqualmie Pass East. Hyak to Keechelus Dam, Washington. July 11, 2007.

URS Corporation and Wyllie & Norrish Rock Engineers. 2007. 2006 Conceptual Geotechnical Report. I-90 Snoqualmie Pass East. Hyak to Keechelus Dam, Washington. February 12, 2007.

U.S. Army Corps of Engineers (USACE). 2006. HEC-RAS River Analysis System, User's Manual, Version 4.0 Beta, Hydrologic Engineering Center, Davis, CA.

U.S. Department of Transportation (USDOT) Federal Highway Administration (FHWA). 2001. Mechanically Stabilized Earth Walls and Reinforced Soil Slopes Design and Construction Guidelines. Publication No. FHWA-NHI-00-043, NHI Course No. 132042.

U.S. Geological Survey (USGS). 2008. 2002 Interactive Deaggregations. http://eqint.cr.usgs.gov/deaggint/2002/index.php

U.S. Geological Survey. 2007a. http://earthquake.usgs.gov/regional/qfaults/wa/index.php. Accessed November 30, 2007.

U.S. Geological Survey. 2007b. USGS Quaternary Fault and Fold Database for the United States, Wenatchee 1° by 2° Sheet at <http://earthquake.usgs.gov/regional/qfaults/wa/wen.html>

WSDOT. 2010a. Geotechnical Design Manual M 46-03. November 2010.

Washington State Department of Transportation (WSDOT). 2010b. Design Memorandum. Bearing Resistance of Shaft Groups. June 28, 2010.

WSDOT. 2009a. I-90 Hyak to Snowshed Vicinity Add Lanes Hydraulic Report. MP 55.16 to 58.08. I-90 Snoqualmie Pass East. Prepared by Otak, Inc. for WSDOT South Central Region. June 2009.

WSDOT. 2009b. Preliminary Snowshed Cross Section at LW 1355+53. Provided to URS May 2009.

WSDOT. 2009c. Memorandum. I-90 Snoqualmie Pass East – Hyak to Keechelus Dam. Snowshed Shaft Foundations. April 8, 2009.

WSDOT. 2009d. Memorandum. SR-90, MP 57 to 58 Vicinity, XL-2779. Snoqualmie Pass East – Phase 1B. Overburden Thickness and Cut Slope Recommendations. August 31, 2009.

WSDOT. 2007. Hydraulic Manual M 23-03.01. March 2007.

Washington State Department of Transportation and U.S. Federal Highway Administration (FHWA). 2008. Final Environmental Impact Statement and Section 4(f) Evaluation. I-90 Snoqualmie Pass East Project. August 2008.

Wilbour, C. 2008. Personal communication from Craig Wilbour, WSDOT avalanche specialist, to Dave Walker, URS. November 20, 2008.

Wyllie and Norrish Rock Engineers Inc. (W&N). 2009a. Phase 1B 2008 Rock Slope Geotechnical Report. I-90 Snoqualmie Pass East Project. March 2009.

Wyllie and Norrish Rock Engineers Inc. 2009b. Phase 1C 2008 Rock Slope Geotechnical Report. I-90 Snoqualmie Pass East Project. March 2009.

Wyllie and Norrish Rock Engineers Inc. 2009c. Final Report - Slide Curve Rock Slope Design. 2007 Geotechnical Analysis and Reporting. I-90 Snoqualmie Pass East Project. January 2009a.

Wyllie and Norrish Rock Engineers Inc. 2008. Technical Memorandum – Slope Stability Analyses for New Alignment. 2007 Geotechnical Program – Field Services. I-90 Snoqualmie Pass East Project. July 2008.

Youd, L.T., C.M. Hansen and S.F Barlett. 2002. Revised Multilinear Regression Equations for Prediction of Lateral Spread Displacement. Journal of Geotechnical and Geoenvironmental Engineering. Vol. 128, No. 12, p. 1007-1017.

Youd, L. T., I. M. Idriss, I. Arango, G. Castro, J. T. Christian, R. Dobry, W. D. Liam Finn, L. F. Harder Jr., M. E. Hynes, K. Ishihara, J. P. Koester, S. S. C. Liao, W. F. Marcuson III, G. R. Martin, J. K. Mitchell, Y. Moriwaki, M. S. Power, P. K. Robertson, R. B. Seed, and K. H. Stokoe II. 2001. Liquefaction Resistance of Soils: Summary Report from the 1996 NCEER and 1998 NCEER/NSF Workshops on Evaluation of Liquefaction Resistance of Soils. ASCE Journal of Geotechnical and Geoenvironmental Engineering. Vol. 127, No.10, 2001.

TABLES

Table 2.1
Keechelus Lake Reservoir Level Data, 1980 - 2004

Parameter	Month											
	April	**May**	**June**	**July**	**August**	**September**	**October**	**November**	**December**	**January**	**February**	**March**
Average	2497.70	2505.16	2507.36	2499.72	2477.10	2453.74	2448.12	2458.55	2469.51	2476.16	2482.97	2489.12
80th Percentile	2511.67	2514.67	2516.54	2513.64	2495.63	2470.75	2466.02	2475.31	2484.98	2492.77	2500.10	2506.45
Median	2501.34	2506.79	2509.77	2503.03	2478.75	2446.88	2442.40	2455.96	2469.10	2478.90	2483.19	2490.23
High	2517.44	2517.77	2517.67	2517.26	2516.98	2498.83	2492.48	2517.66	2516.44	2508.37	2513.77	2515.93
Low	2451.45	2466.89	2470.87	2445.71	2435.12	2433.33	2430.76	2431.94	2434.07	2438.87	2443.36	2442.78
1st day of month												
Average	2493.32	2502.07	2507.74	2505.89	2491.32	2460.74	2447.69	2451.61	2465.97	2472.97	2480.33	2486.12
Median	2496.52	2504.99	2511.17	2509.00	2497.74	2459.05	2441.47	2446.02	2462.78	2472.40	2481.28	2486.09
High	2516.08	2517.27	2517.62	2517.26	2516.98	2496.96	2478.94	2493.33	2516.44	2503.10	2508.86	2511.42
Low	2451.45	2466.89	2484.95	2470.26	2444.88	2434.97	2431.16	2431.94	2434.07	2438.87	2444.73	2443.24
15th day of month												
Average	2497.65	2504.99	2507.66	2501.24	2479.40	2453.97	2446.97	2458.44	2469.37	2476.07	2482.76	2488.74
Median	2500.51	2505.49	2509.91	2504.32	2485.84	2445.61	2440.34	2453.89	2469.68	2477.94	2483.24	2489.88
High	2516.54	2516.33	2517.41	2517.18	2514.09	2487.28	2485.96	2497.61	2504.45	2504.45	2512.58	2513.18
Low	2454.10	2478.02	2479.42	2461.08	2437.46	2434.26	2430.80	2432.61	2438.80	2440.48	2444.33	2443.60
30th day of month												
Average	2501.84	2507.70	2506.11	2492.01	2463.67	2448.78	2451.28	2465.65	2472.61	2479.71	2486.02	2493.08
Median	2504.05	2511.24	2509.48	2497.81	2462.81	2442.49	2445.76	2461.81	2472.68	2481.10	2485.83	2496.06
High	2517.33	2517.71	2517.28	2517.02	2499.94	2478.51	2492.48	2516.95	2502.64	2508.37	2511.32	2515.93
Low	2466.21	2485.07	2470.87	2445.71	2435.12	2433.33	2431.87	2433.99	2439.02	2444.80	2443.36	2451.04

Average Recorded Pool Elevation 1980-2004: 2480.13 feet
Median Recorded Pool Elevation 1980-2004: 2482.54 feet
Maximum Recorded Pool Elevation 1980-2004: 2517.77 feet
Minimum Recorded Pool Elevation 1980-2004: 2430.76 feet

Notes:
1. Source: Summary table provided to URS by WSDOT. The information in this table was provided to WSDOT by USBR.

Table 2.2
Dry Period Duration for Selected Keechelus Lake Elevations

Elevation (feet)	Duration Lake Level is Below Elevation (weeks)[4]		
	High Lake Level Year[1,4]	Average Year[2]	Low Lake Level Year[3,4]
2450	2	5	10
2460	5	11	17
2470	6	17	21
2480	8	25	29
2490	11	33	34
2500	15	40	52
2510	31	52	52
2520	52	52	52

Notes:
1. Represents 1996 high lake level year
2. Represents 1981-2004 average lake levels
3. Represents 2005 low lake level year
4. High and low lake level information is based on 1981-2005 lake level measurements

Table 2.3
Summary of Proposed Bridges

Bridge	Direction	Begin	End	Length (feet)
Slide Curve Bridge (SCB)	EB	LE 1372+06	LE 1383+58	1,052
Resort Creek Bridges (RCB)	EB	LE 1426+11	LE 1427+56	145
	WB	LW 1426+26	LW 1427+71	145

Table 2.4
Summary of Proposed Retaining Walls

Wall No.	Description	Location	Type	Begin	End	Length (feet)	Approx. Maximum Height (feet)
1	East Shed 1 Catchment Wall	WB	Cast-in-place	LW 1346+60	LW 1352+50	590	15
2	Snowshed Pier 2 – Generator Room	EB	Gravity block	LE 1350+70	LE 1351+13	43	10
3	Snowshed Pier 2	EB	Geosynthetic	LW 1352+71.5 LE 1351+50	LW 1363+50 LE 1362+09	1,059	35
4	Snowshed Pier 2 Access Bench	EB	Soil nail (temporary)	NA	NA	61	45
5	Slide Curve Bridge and Walls	EB	Geosynthetic	LE 1371+10	LE 1372+06	96	8
6		EB	Geosynthetic	LE 1383+58	LE 1385+80	222	18
7		WB	Soil nail	LE 1371+72 W7 10+00	LE 1383+87 W7 22+25	1,225	45
8	Slide Curve Median Wall	WB	Structural Earth	LW 1377+30	LW 1423+00	4,570	50
9	Snowshed West Portal Walls	WB	Structural Earth and Counterfort	LW 1352+10 (approx.)	LW 1352+50	40	65
10	Snowshed East Portal Walls	WB	Counterfort	LW 1363+50 (approx)	LW 1363+60 (approx)	25	65
11-14	Resort Creek Bridge Approaches	EB	Geosynthetic	Varies	Varies	Up to 45	Up to 20
15-18	Resort Creek Bridge Approaches	WB	Structural Earth	Varies	Varies	Up to 22	Less than 15
19,20	MP 59.7 Crossing West and East Approaches	EB	Gravity block	Varies	Varies	20	12.5
21,22	MP 59.7 Crossing West and East Approaches	WB	Gravity block	Varies	Varies	10	7.5
23	Slide Curve Bridge Tied Shaft Wall	WB	Tied shaft	LE 1378+30	LE 1381+65	335	Below grade

Notes
Based on information obtained from the WSDOT Project Office on September 9, 2010
CIP = cast in place
LE = eastbound construction centerline
LW = westbound construction centerline
MSE = mechanically-stabilized earth
NA = not available
TBD = to be determined

Table 3.1
Scope of the 2008-2010 Phase 1C and 1D Boring and Test Pit Program

Structure	No. of Borings	No. of Test Pits
Snowshed	14	1
Slide Curve Bridge and Walls	28	1
Slide Curve Median Wall	17	0
Resort Creek Wall	1	0
Resort Creek Bridges	8	0
Townsend Creek Culvert	3	1
MP 59.7 Unnamed Creek Crossing	2	0
Sign Structures	10	0
Structural Wall 2 (Slide Curve Embankment Stabilization)	4	0
Embankments	1	0
Old Sunset Highway crossing of Resort Creek	2	0
Offsite Stormwater Mitigation	11	25

Table 3.2
Summary of 2008-2010 Phase 1C and 1D Roadway Borings

Boring ID	Date Drilled		Elevation (feet)[1]	Project Datum[2]		Total Depth (feet)
	Started	Completed		Northing (feet)	Easting (feet)	
Phase 1C						
CUL-019-10	5/10/10	5/11/10	2522.7	1,060,708	1,757,696	50.1
CUL-020-10	7/15/10	7/16/10	2530.5	1,060,758	1,757,690	51.0
CUL-021-10	9/22/10	9/22/10	2524.7	1,061,718	1,756,661	50.0
CUL-022-10	9/23/10	9/24/10	2525.2	1,061,729	1,756,697	51.0
EMB-027-09	6/17/09	6/17/09	2479.2	1,059,782	1,758,122	51.0
RCB-001-08	5/29/08	6/4/08	2525.5	1,061,276	1,756,358	96.6
RCB-002-08(OW)	9/4/08	9/4/08	2524.8	1,061,277	1,756,537	76.3
RCB-003-08(OW)	5/31/08	6/1/08	2527.5	1,061,365	1,756,357	101.5
RCB-004-10	5/4/10	5/11/10	2529.3	1,061,362	1,756,517	106.0
RCB-005-10	4/20/10	4/21/10	2495	1,061,216	1,756,352	96.0
RCB-006-10	4/20/10	4/21/10	2495	1,061,199	1,756,514	70.5
RCB-007-10	4/22/10	4/28/10	2525.3	1,061,267	1,756,386	121.0
RCB-008-10	4/21/10	4/27/10	2524.4	1,061,265	1,756,511	109.2
RCW-001-08	5/29/08	5/30/08	2523.4	1,061,379	1,755,836	65.0
SCB-001-09	5/19/09	5/21/09	2497.1	1,064,890	1,754,427	66.0
SCB-002-08	9/22/08	9/25/08	2535.0	1,064,897	1,754,476	61.5
SCB-004-08	6/26/08	6/26/08	2490	1,064,582	1,754,479	51.8
SCB-005-08	9/26/08	9/29/08	2534.1	1,064,744	1,754,517	59.5
SCB-006-08	6/24/08	6/26/08	2505	1,064,424	1,754,500	107.5
SCB-007-08	9/30/08	10/1/08	2532.4	1,064,563	1,754,539	91.5
SCB-009-08	7/22/08	7/22/08	2490	1,064,607	1,754,471	44.0
SCB-010-08	7/16/08	7/17/08	2499	1,064,381	1,754,496	72.5
SCB-011-08	10/1/08	10/2/08	2532.1	1,064,419	1,754,540	92.0
SCB-014-08	9/2/08	9/3/08	2350	1,064,448	1,754,291	49.0
SCB-015-08	11/22/08	11/25/08	2238	1,064,315	1,753,967	59.7
SCB-016-08	8/26/08	8/26/08	2448	1,064,521	1,754,430	65.0
SCB-017-08	8/27/08	8/28/08	2450	1,064,471	1,754,424	70.0
SCB-018-08	9/30/08	10/1/08	2536.5	1,065,001	1,754,444	52.0
SCB-019-09	5/27/09	5/28/09	2503.7	1,065,039	1,754,382	47.0

Table 3.2 (continued)
Summary of 2008-2009 Phase 1C and 1D Roadway Borings

Boring ID	Date Drilled		Elevation (feet)[1]	Project Datum[2]		Total Depth (feet)
	Started	Completed		Northing (feet)	Easting (feet)	
SCB-020-09	5/26/09	5/27/09	2505.3	1,064,254	1,754,489	93.5
SCB-021-08	10/2/08	10/3/08	2533.0	1,064,269	1,754,528	69.5
SCB-022-09	6/12/09	6/14/09	2537.7	1,065,148	1,754,392	71.9
SCB-023-09	5/28/09	5/29/09	2502.7	1,065,146	1,754,347	56
SCB-024-09	6/9/09	6/12/09	2523.4	1,064,160	1,754,491	85.5
SCB-025-09	6/2/09	6/4/09	2536.8	1,065,031	1,754,435	55.6
SCB-026-09	10/9/09	10/11/09	2525.4	1,065,198	1,754,348	66.4
SCB-027-09	10/12/09	10/14/09	2523.3	1,065,099	1,754,390	72.4
SCB-028-09	10/16/09	10/22/09	2499.1	1,064,245	1,754,473	82.5
SCB-029-09	10/23/09	10/24/09	2515.4	1,064,190	1,754,487	73
SCB-030-09	10/24/09	10/25/09	2527.8	1,064,134	1,754,494	88.7
SCB-031-10	7/13/2010	7/13/2010	2539.1	1,065,277	1,754,387	35.0
SCB-032-10	7/14/2010	7/14/2010	2537.8	1,065,283	1,754,330	45.0
SCW-001-08	5/19/08	5/20/08	2533.8	1,064,096	1,754,560	19.0
SCW-002-08	5/6/08	5/7/08	2532.5	1,063,827	1,754,515	38.7
SCW-002A-08	5/28/08	5/28/08	2532.1	1,063,824	1,754,536	13.8
SCW-003-08	5/7/08	5/8/08	2531.7	1,063,588	1,754,474	38.7
SCW-004-08	5/9/08	5/9/08	2531.5	1,063,297	1,754,423	28.7
SCW-004A-08	5/28/08	5/28/08	2531.0	1,063,293	1,754,444	16.0
SCW-005-08	5/6/08	5/7/08	2531.4	1,063,012	1,754,374	39.0
SCW-006-08	5/7/08	5/8/08	2532.6	1,062,768	1,754,363	35.0
SCW-006A-08	5/27/08	5/28/08	2530.4	1,062,768	1,754,386	16.6
SCW-007-08	5/8/08	5/9/08	2533.6	1,062,599	1,754,378	23.2
SCW-008-08	5/9/08	5/12/08	2534.6	1,062,419	1,754,417	24.0
SCW-008A-08	5/27/08	5/27/08	2532.3	1,062,426	1,754,439	18.8
SCW-009-08	5/12/08	5/13/08	2535.6	1,062,238	1,754,481	44.0
SCW-010-08	5/14/08	5/14/08	2533.7	1,061,658	1,754,967	28.7
SCW-011-08	5/14/08	5/14/08	2530.9	1,061,473	1,755,280	18.7
SCW-012-08	5/14/08	5/14/08	2527.2	1,061,381	1,755,587	18.7
SCW-013-08	5/9/08	5/13/08	2525.6	1,061,327	1,756,050	70.2
SSD-006-09	6/2/09	6/4/09	2497.5	1,067,029	1,753,805	79

Table 3.2 (continued)
Summary of 2008-2009 Phase 1C and 1D Roadway Borings

Boring ID	Date Drilled		Elevation (feet)[1]	Project Datum[2]		Total Depth (feet)
	Started	Completed		Northing (feet)	Easting (feet)	
SSD-007-09	5/30/09	6/1/09	2497.0	1,066,796	1,753,908	100
SSD-008-09	6/9/09	6/11/09	2499.6	1,066,554	1,753,987	67
SSD-009-09	6/12/09	6/14/09	2507.2	1,066,916	1,753,869	89
SSD-010-09	6/14/09	6/16/09	2498.9	1,066,679	1,753,954	81
SSD-011-09	6/25/09	6/29/09	2538.1	1,067,246	1,753,717	58
SSD-012-09	6/23/09	6/23/09	2533.4	1,066,243	1,754,078	81
SSD-013-09	11/5/09	11/5/09	2539.6	1,067,289	1,753,828	21.5
SSD-014-09(OW)	11/3/09	11/4/09	2539.4	1,067,227	1,753,865	26
SSD-015-09	11/3/09	11/4/09	2537.9	1,067,145	1,753,906	21.5
SSD-016-10	7/27/10	7/27/10	2542.4	1,067,428	1,753,725	20
SSD-017-10	7/26/10	7/27/10	2540.4	1,067,347	1,753,779	20
SSD-018-10	7/20/10	7/20/10	2630.3	1,067,398	1,753,892	55
SSD-019-10	7/21/10	7/22/10	2634.1	1,067,407	1,753,897	50
SSE-006-08	5/15/08	5/15/08	2539.3	1,065,655	1,754,212	9.0
SSE-008-08	5/15/08	5/15/08	2535.7	1,060,905	1,757,639	8.5
SSE-009-08	5/15/08	5/15/08	2533.5	1,064,103	1,754,598	8.5
SSE-010-08	5/8/08	5/8/08	2533.5	1,064,102	1,754,602	8.5
SSE-011-08	5/14/08	5/14/08	2535.4	1,072,661	1,751,733	13.5
SSE-022-09	6/17/09	6/17/09	2534.0	1,064,145	1,754,609	18
SSE-025-10	4/29/10	4/29/10	2524.7	1,060,276	1,757,906	21.5
SSE-026-10	5/13/10	5/13/10	2534.3	1,066,156	1,754,184	14.5
SSE-027-10	5/12/10	5/12/10	2535.1	1,065,929	1,754,220	9.0
SSE-028-10	5/12/10	5/12/10	2534.6	1,065,705	1,754,279	14.5
SW2-006-08(I)	10/21/08	10/22/08	2541.6	1,061,806	1,754,682	76
SW2-007-08	9/9/08	9/11/08	2402	1,061,810	1,754,416	96.0
SW2-008-08	9/12/08	9/17/08	2344	1,061,661	1,754,196	124.0
SW2-009-08	9/23/08	9/24/08	2340	1,061,521	1,754,312	117.0
TCC-001-08	9/8/08	9/8/08	2526.0	1,057,362	1,760,277	37.3
TCC-002-08	5/11/08	5/11/08	2525.9	1,057,250	1,760,237	40.3
TCC-003-08	5/28/08	5/29/08	2525.3	1,057,165	1,760,170	39.8

Table 3.2 (continued)
Summary of 2008-2009 Phase 1C and 1D Roadway Borings

Boring ID	Date Drilled		Elevation (feet)[1]	Project Datum[2]		Total Depth (feet)
	Started	Completed		Northing (feet)	Easting (feet)	
Phase 1D						
HMF-001-08	5/10/08	5/11/08	2545.3	1,080,146	1,748,786	15.0
HMF-002-08	5/10/08	5/10/08	2545.2	1,080,064	1,748,753	15.3
HMF-003-08	5/11/08	5/11/08	2545.4	1,079,995	1,748,728	16.5
HMF-004-08	5/10/08	5/10/08	2544.4	1,080,224	1,748,482	19.0
HMF-005-08	5/10/08	5/10/08	2545.9	1,080,054	1,748,483	14.3
HMF-006-08	5/10/08	5/10/08	2544.1	1,080,316	1,748,456	15.3
HMF-007-08	5/11/08	5/11/08	2543.1	1,080,345	1,748,400	15.5
HMF-008-09	6/13/09	6/13/09	2546.1	1,080,198	1,748,762	13.3
INF-TP-010-10(OW)	6/22/10	6/22/10	2435.8	1,049,240	1,764,972	16.5
INF-TP-012-10(OW)	6/22/10	6/22/10	2427.9	1,049,049	1,765,449	17.0
INF-TP-015-10	6/21/10	6/21/10	2542.7	1,048,884	1,764,683	26.0

Notes
[1]Vertical datum is the North American Vertical Datum (NAVD) 1988
[2]Horizontal datum is based on the Washington Coordinate System, South Zone, and North American Datum (NAD) 1983/91
OW - indicates groundwater observation well installed

Boring ID abbreviations

CUL: culvert	SSD: snowshed
EMB: embankment	SSE: sign structure
RCB: Resort Creek bridges	SW2: Slide Curve embankment stabilization
RCW: Resort Creek wall	TCC: Townsend Creek culvert
SCB: Slide Curve bridge and walls	HMF: Hyak Maintenance Facility
SCW: Slide Curve median wall	INF: stormwater mitigation facility

Table 3.3
Summary of 2008-2010 Phase 1C and 1D Test Pits

Test Pit ID	Date Drilled		Elevation (feet)[1]	Project Datum[2]		Total Depth (feet)
	Started	Completed		Northing (feet)	Easting (feet)	
Phase 1C						
SSD-TP-001-08	6/18/08	6/18/08	2533.3	1,067,054	1,753,851	12.0
TCC-TP-002-08	6/18/08	6/18/08	2522.5	1,057,226	1,760,219	12.5
SCB-TP-003-08	7/9/08	7/9/08	2536.9	1,065,001	1,754,444	8.0
Phase 1D						
HMF-TP-001-09	5/18/09	5/18/09	2538.7	1,080,327	1,748,358	11.3
HMF-TP-002-09	5/18/09	5/18/09	2533.7	1,080,120	1,748,314	11.5
HMF-TP-003-09	5/18/09	5/18/09	2532.2	1,079,968	1,748,272	11.5
HMF-TP-004-10	6/1/2010	6/1/2010	Not surveyed			10.0
HMF-TP-005-10	6/1/2010	6/1/2010	2545.4	1,080,026	1,748,744	11.0
INF-TP-006-10	6/1/2010	6/1/2010	2437.0	1,049,261	1,764,907	7.5
INF-TP-007-10	6/1/2010	6/1/2010	2429.8	1,049,129	1,765,189	10.5
INF-TP-009-10	6/1/2010	6/1/2010	2424.2	1,048,870	1,765,548	9.7
INF-TP-011-10	6/22/2010	6/22/2010	2431.8	1,048,879	1,765,017	10.0
INF-TP-013-10	6/22/2010	6/22/2010	2429.1	1,048,739	1,765,265	11.0
INF-TP-014-10	6/22/2010	6/22/2010	2425.6	1,048,732	1,765,462	12.0
TP-003-08	6/9/08	6/9/08	2529.5	1,079,833	1,748,294	14.0
TP-005-08	6/11/08	6/11/08	2650.3	1,082,343	1,745,970	5.5
TP-006-08	6/10/08	6/10/08	2633.6	1,081,485	1,746,111	6.0
TP-007-08	6/11/08	6/11/08	2576.9	1,081,042	1,747,196	9.8
TP-009-08	6/11/08	6/11/08	2716.5	1,083,688	1,745,040	4.0
TP-010-08	6/11/08	6/11/08	2779.8	1,085,163	1,744,682	13.0
TP-012-08	6/12/08	6/12/08	2868.4	1,087,242	1,743,464	14.8
TP-013-08	6/12/08	6/12/08	2938.2	1,088,880	1,743,170	14.5
TP-014-08	6/10/08	6/10/08	2951.4	1,089,937	1,743,494	8.0
TP-015-08	6/10/08	6/10/08	2991.0	1,090,247	1,743,240	15.8
TP-017-08	6/12/08	6/12/08	2530.0	1,058,437	1,759,098	9.5
TP-018-08	6/12/08	6/12/08	2523.5	1,058,419	1,759,005	15.0
TP-019-08	6/11/08	6/11/08	2642.9	1,082,109	1,745,991	3.2
TP-020-08	6/10/08	6/10/08	2622.0	1,081,556	1,746,343	7.0

Notes
[1]Vertical datum is the North American Vertical Datum (NAVD) 1988
[2]Horizontal datum is based on the Washington Coordinate System, South Zone, and North American Datum (NAD) 1983/91
[3]The test pit could not be located at the time of the survey

Table 3.4
Observation Well Installation Summary

Boring I.D.	Date Installed	Proposed Structure	Ground El. (feet)	Location		Depth (feet)	Instrumentation [3]	Top of PVC El. (feet)	OW Screen Interval (feet)	Top of Protective Casing El. (feet)
				Northing (feet)	Easting (feet)					
2006										
H-24-06(OW)[1]	6/13/06	Snowshed Bridge	2537.2	1,065,874	1,754,146	60.0	1-inch PVC O.W.	2538.7	15.0-40.0	2539.23[4]
H-38-06(OW)[1]	7/11/06	Retaining Wall	2526.6	1,060,006	1,758,174	30.1	1-inch PVC O.W.	2526.3	18.0-30.0	2526.64[5]
2007										
CUL-015-07(OW)[2]	9/14/07	Infiltration Facility	2536.2	1,060,768	1,757,760	36.5	1-inch PVC O.W.	2537.9	16.0-26.0	2538.17[4]
SW2-002-07 (OW)[2]	7/31/07	Retaining Wall	2541.5	1,061,928	1,754,589	86.0	1-inch PVC O.W.	2542.7	20.0-70.0	2543.23[4]
2008										
RCB-002-08(OW)	9/4/08	Resort Creek Bridges	2524.8	1,061,277	1,756,537	76.3	1-inch PVC O.W.	2524.61	30-50	2524.76[5]
RCB-003-08(OW)	6/1/08	Resort Creek Bridges	2527.5	1,061,365	1,756,357	101.5	1-inch PVC O.W.	2529.64	50-70	2529.90[4]
2009										
SSD-014-09(OW)	11/4/09	Snowshed	2539.4	1,067,227	1,753,865	26.0	1-inch PVC O.W.	2540.4	NA	2540.8
2010										
INF-TP-010-10(OW)	6/22/10	Infiltration Facility	2435.8	1,049,240	1,764,972	16.5	1-inch PVC O.W.	NA	6-16	2435.8[5]
INF-TP-012-10(OW)	6/22/10	Infiltration Facility	2427.9	1,049,049	1,765,449	17.0	1-inch PVC O.W.	NA	5-15	2427.9[5]

Notes
1. Locations and elevations were surveyed by White Shield Inc. from August 1 to 3, 2006 and August 23 to 25, 2006. Northings and eastings are in project datum.
2. Locations and elevations were surveyed by White Shield Inc. between September 13, 2007 and January 9, 2008. Northings and eastings are in project datum.
3. Abbreviations: OW - Observation Well.
4. Stick-up monument protective steel casing was installed.
5. Flush-mount monument protective steel casing was installed.
NA: not available

Table 3.5
Groundwater Monitoring Data

Boring I.D.	Ground Elevation (feet)	Bottom of Well El. (feet)	2006 Groundwater Level Measurements, feet										
			13- to 15-Jun	19- to 21-Jun	22-Jun	26- to 27-Jun	30-Jun	10- to 2-Jul	3-Aug	16-Aug	28-Aug	29-Sep	24-Oct
H-24-06(OW)[1]	2537.2	2496.2	2516.9	2516.5		2513.6		2504.3	Note 3				
H-38-06(OW)[1]	2526.6	2495.6						2506.7	2505.9		2505.6	2505.6	2505.6

Boring I.D.	Ground Elevation (feet)	Bottom of Well El. (feet)	2007 Groundwater Level Measurements, feet											
			3-Jul	23-Jul	9-Aug	22-Aug	6-Sep	17-Sep	1-Oct	15-Oct	29-Oct	12-Nov	27-Nov	12-Dec
H-24-06(OW)[1]	2537.2	2496.2	2510.2	Note 3										
H-38-06(OW)[1]	2526.6	2495.6	2510.7	2506.3	2505.8	2505.7	2505.7	2505.8	2505.8	2506.4	2506.4	2506.5	2506.5	2506.5
CUL-015-07(OW)[2]	2536.2	2510.2	Note 6					Note 5						
SW2-002-07(OW)[2]	2541.5	2470.5	Note 6		2474.1	Note 7								

Boring I.D.	Ground Elevation (feet)	Bottom of Well El. (feet)	2008 Groundwater Level Measurements, feet										
			28-Apr	28-May	13-Jun	27-Jun	14-Jul	28-Jul	19-Aug	28-Aug	13-Sep	28-Sep	29-Oct
H-24-06(OW)[1]	2537.2	2496.2	2496.2	2501.4	2513.0	2516.8	2512.9	2501.3		Note 3		Note 7	
H-38-06(OW)[1]	2526.6	2495.6	2506.5	2506.5	2513.4	2517.1	2513.5	2506.4		2506.2		2505.8	2506.4
CUL-015-07(OW)[2]	2536.2	2510.2	2527.5	2527.5	2526.1	2525.4	2525.1	2524.8		2524.9		2524.5	2524.9
SW2-002-07(OW)[2]	2541.5	2470.5	Well destroyed										
RCB-002-08(OW)[8]	2524.8	2448.5	Note 6								2491.8	2492.1	2491.9
RCB-003-08(OW)[8]	2527.5	2456.5	Note 6	2504.3	2513.0	2516.8	2512.9	2501.3	2491.6	2491.8	2490.8	2491.1	2491.1

Boring I.D.	Ground Elevation (feet)	Bottom of Well El. (feet)	2009 Groundwater Level Measurements, feet		
			26-May	22-Jun	30-Jul
H-24-06(OW)[1]	2537.2	2496.2	2502.0	2515.6	Note 3
H-38-06(OW)[1]	2526.6	2495.6	2506.3	2516.1	2506.0
CUL-015-07(OW)[2]	2536.2	2510.2	2525.7	2525.1	2524.7
RCB-002-08(OW)[8]	2524.8	2448.5	2512.0	2515.9	2492.8
RCB-003-08(OW)[8]	2527.5	2456.5	2514.2	2515.7	2495.5

Table 3.5 (continued)
Groundwater Monitoring Data

Boring I.D.	Ground Elevation (feet)	Bottom of Well El. (feet)	2010 Groundwater Level Measurements, feet		
			2-Jun	16-Jul	19-Aug
H-24-06(OW)[1]	2537.2	2496.2	2504.6	2499.8	Note 3
H-38-06(OW)[1]	2526.6	2495.6	2506.5	2506.4	2505.8
CUL-015-07(OW)[2]	2536.2	2510.2	2525.7	2524.6	2524.7
RCB-002-08(OW)[8]	2524.8	2448.5	2504.8	2500.2	2492.3
RCB-003-08(OW)[8]	2527.5	2456.5	2504.3	2499.6	2490.6
SSD-014-09(OW)	NA	NA	Destroyed by construction		
INF-010-10(OW)	2435.8	2419.3	Note 6	Note 7	Well not found
INF-012-10(OW)	2427.9	2417.9	Note 6	Note 7	Note 7

Notes
1. Ground elevations were surveyed by White Shield Inc. on August 1 to 3, 2006 and August 23 to 25, 2006.
2. Ground elevations were surveyed by White Shield Inc. between September 13, 2007 and January 9, 2008.
3. The measured groundwater elevation was approximately at the bottom of the screen interval. Groundwater probe probably read any sediment/sand trapped at the bottom of the screen. The actual groundwater elevation may be lower than the measured elevation.
4. Approximately 0.5 foot rainwater ponded at ground surface around the stick-up protective casing (well is in approximately 2 to 3 foot deep drainage ditch adjacent to I-90 WB.
5. The water level was not monitored because winter weather prevented access to the well.
6. The well was not installed at the corresponding dates.
7. The water level was below the bottom of the well.
8. Ground elevations were surveyed by White Shield Inc.

Table 3.6
2008-2010 Phase 1C and 1D Laboratory Index Test Results

Boring No. & Sample Information				Water Content (%)	Grain Size Analysis			Atterberg Limits			Organic Content (%)	pH	Resistivity (Ω·cm)
Boring No.	Sample No.	Sample Depth (feet)	USCS/WSDOT Classification		Gravel (%)	Sand (%)	Silt and Clay (%)	LL	PL	PI			
Phase 1C													
CUL-021-10	D-6	15.0	SW		45.0	46.0	9.0						
	D-8	20.0	SM		38.6	48.1	13.3						
	D-10	25.0	SM		43.1	46.8	10.1						
	D-12	30.0	SP		42.5	49.1	8.4						
	D-14	35.0	GM		45.0	44.9	10.1						
	D-16	40.0	SM		14.2	57.2	28.6						
CUL-022-10	D-4	11.0	GW		66.3	30.0	3.7						
	D-6	16.0	SW		45.4	45.6	9.0						
	D-8	21.0	SP		44.7	45.8	9.5						
	D-10	26.0	SP		36.2	54.4	9.4						
	D-15	41.0	SM		13.7	63.3	22.9						
EMB-027-09	D-1	5.0	SM		31.6	56.7	11.7						
	D-2	10.0	SP		34.2	63.8	1.9						
	D-3	15.0	SM		22.9	48.0	29.1						
RCB-001-08	D-4	8.6-10.1	GM	9.8									
	D-6	13.6-15.1	GW										
	D-12	28.6-30.1	SM	27.2	22.5	49.1	28.4						
	D-14	33.6-35.1	SM	21.1	32.5	48.3	19.2						
	D-18	41.3	SM										
	D-20	47.8	SM										
	D-22	50.2	SM	18.9	34.5	53.2	12.3						
	D-25	56.3	SM										
	D-27	62.8	SM	23.4	30.6	43.5	25.9						
	D-29	66.3	SM	27.6	9.7	57.9	32.4						
	D-31	71.3	CL	43.6	0.0		100.0	44	22	22			
	D-33	76.3	SM	19.9	25.6	62.0	12.4						
	D-36	84.3	SM	11.7	25.2	49.0	25.8						
	D-38	86.3	SM										
	D-40	91.7	SM	14.3	34.6	41.9	23.5						
	D-42	96.3	SM										

Table 3.6 (continued)
2008-2010 Laboratory Index Test Results

Boring No. & Sample Information				Water Content (%)	Grain Size Analysis			Atterberg Limits			Organic Content (%)	pH	Resistivity (Ω·cm)
Boring No.	Sample No.	Sample Depth (feet)	USCS/WSDOT Classification		Gravel (%)	Sand (%)	Silt and Clay (%)	LL	PL	PI			
RCB-002-08 (OW)	D-1	4.0-4.9	GW	42.5	76.7	18.6	4.7						
	D-7	20.0-21.5	GM	14.2	56.4	30.0	13.6						
	D-9	25.0-26.5	GM-SM	21.0	46.4	40.3	13.3						
	D-11	30.0-30.5	GP	16.0	56.4	35.0	8.6						
	D-13	35.0-36.5	GP	14.0	64.4	27.5	8.1						
	D-15	40.0-41.5	SM	15.8	35.7	45.8	18.5						
	D-17	45.0-46.5	SM										
	D-23	60.0-61.5	SM	22.0	9.1	64.1	26.8						
	D-25	65.0-66.5	SM										
	D-28	75.0-76.3	SM	8.8	22.2	44.1	33.7						

Table 3.6 (continued)
2008-2010 Laboratory Index Test Results

Boring No. & Sample Information				Water Content (%)	Grain Size Analysis			Atterberg Limits			Organic Content (%)	pH	Resistivity (Ω·cm)
Boring No.	Sample No.	Sample Depth (feet)	USCS/WSDOT Classification		Gravel (%)	Sand (%)	Silt and Clay (%)	LL	PL	PI			
RCB-003-08 (OW)	D-2	9.0-9.3	SM	13.7									
	D-5	15.4-15.9	GM										
	D-13	29.0-30.5	GP	35.8	63.2	28.4	8.4						
	D-15	34.2-35.7	GW	24.8	54.7	36.7	8.6						
	D-18	41.5-42.4	GW										
	D-20	45.6-47.1	SM	26.5	17.3	57.2	25.5						
	D-22	51.5-53.0	CL-ML	26.0	2.6	52.2	45.2	28	21	7			
	D-25	61.3-62.8	SM	23.7	12.1	53.4	34.5						
	D-27	66.5-68.0	CL	32.5	0.0		100.0	41	25	16			
	D-29	71.5-73.0	SM	22.5	15.6	60.7	23.7						
	D-31	76.5-78.0	SM										
RCB-004-08	D-5	51.0-52.5	SM	17.0	18.8	66.6	14.6						
	D-13	71.0-72.5	SM	29.4	69.7		30.3						
SCB-002-08	D-1	5.0-6.5	SW	9.7	45.9	46.1	8.0						
	D-2	11.5-13.0	SW										
	D-11	31.5-33.0	GW	7.0	53.2	38.2	8.6						
	D-13	36.5-37.3	GW										
SCB-004-08	D-2	2.5-4.0	SW	27.6	0.4	94.1	5.5						
	D-8	17.5-19.0	GW	9.3	67.4	29.1	3.5						
	D-10	22.5-22.9	GW										
	D-12	27.5-29.0	SM	16.4	35.1	52.5	12.4						
	D-14	32.5-34.0	SM-GM	13.6	44.3	44.4	11.3						
SCB-005-08	D-1	5.0-6.5	GP	8.5	58.2	33.4	8.4						
	D-2	10.0-10.2	GP										
	D-4	16.5-18.0	GW	8.6	65.0	30.3	4.7						
	D-9	31.5-33.0	GW	6.6	52.5	37.7	9.8						
	D-11	36.0-37.5	SM	9.5	20.6	63.1	16.4						
SCB-006-08	D-2	1.5-3.0	SW	16.4	26.9	67.7	5.4						
	D-4	6.0-7.5	GP	10.9	74.3	22.5	3.3						
	D-6	11.5-11.7	GP										

Table 3.6 (continued)
2008-2010 Laboratory Index Test Results

Boring No. & Sample Information				Water Content (%)	Grain Size Analysis			Atterberg Limits			Organic Content (%)	pH	Resistivity (Ω·cm)
Boring No.	Sample No.	Sample Depth (feet)	USCS/WSDOT Classification		Gravel (%)	Sand (%)	Silt and Clay (%)	LL	PL	PI			
	D-14	41.5-43.0	SW	14.8	43.9	46.5	9.5						
	D-18	51.5-51.9	SM	13.1	30.7	56.8	12.5						
SCB-007-08	D-2	5.0-6.5	GW	7.0	71.9	25.9	2.1						
	D-6 a& b	26.5-28.0	GW	13.0	51.1	41.6	7.3						
	D-13	56.5-57.2	SM	9.6	41.6	46.7	11.6						
SCB-007-08	D-15	61.5-63.0	SM	10.5	37.7	51.0	11.3						
SCB-011-08	D-2	7.0-8.5	GW	6.7	55.2	37.8	7.0						
	D-12	32.0-33.5	GP	11.9	66.0	27.6	6.4						
	D-14	37.0-38.5	GP										
SCB-014-08	D-3	9.5-11.0	GW	5.5	65.2	33.6	1.2						
	D-5	19.5-21.0											
SCB-015-08	D-4	18.0-19.5	MH	36.2				60	34	26			
	S-5	26.0-28.0	MH	52.2				53	30	23			
SCB-021-08	D-2	7.0-8.5	GW	9.5	57.3	39.5	3.2						
	D-5	17.0-18.5	GP	6.5	74.1	21.8	4.1						
	D-7	22.0-23.4	GM	16.7	67.1	18.4	14.5						
	C-18	68.2-68.6	CL-ML	26.1				20	15	5			
SCB-022-09	C-20	61.3-62.0	CL		0.0	8.9	91.1	28	19	9			
SCB-028-09	F-3	50.1-50.3	GM		49.7	23.0	27.2						
	F-4	56.3-56.5	SM		42.1	45.2	12.8						
SCW-001-08	D-2	5.0-5.3	GM	22.8	86.0		14.0						
SCW-002-08	D-1	6.7-8.2	SM	13.3	87.9		12.1						
	D-3	8.8-10.3	SM	8.6	86.0		14.0						
	D-5	13.7-15.1	GW	4.5	98.9		1.1						
SCW-003-08	D-4	13.7-14.2	SM	30.1	79.3		20.7						
	D-7	18.7-20.1	GM	11.1	86.3		13.7						
	D-11	23.7-24.6	SM	9.3	87.2		12.8						
SCW-004-08	D-1	6.0-7.5	GM	11.8	89.7		10.3						
	D-3	8.7-9.5	GM	7.3	89.7		10.3						

Table 3.6 (continued)
2008-2010 Laboratory Index Test Results

Boring No. & Sample Information				Water Content (%)	Grain Size Analysis			Atterberg Limits			Organic Content (%)	pH	Resistivity (Ω·cm)
Boring No.	Sample No.	Sample Depth (feet)	USCS/WSDOT Classification		Gravel (%)	Sand (%)	Silt and Clay (%)	LL	PL	PI			
SCW-004A-08	C-2	2.5	GM	21.7	86.4		13.6						
	D-3	5.0	GM	25.4	87.4		12.6						

Table 3.6 (continued)
2008-2010 Laboratory Index Test Results

Boring No. & Sample Information				Water Content (%)	Grain Size Analysis			Atterberg Limits			Organic Content (%)	pH	Resistivity (Ω·cm)
Boring No.	Sample No.	Sample Depth (feet)	USCS/WSDOT Classification		Gravel (%)	Sand (%)	Silt and Clay (%)	LL	PL	PI			
SCW-006-08	D-3	14.0	GP	12.0	90.3		9.7						
	D-4	19.0-19.8	GM	18.3	85.1		14.9						
	D-5	24.0-25.5	GM	16.5	82.8		16.9						
SCW-006A-08	D-2	2.5	GM	15.7	88.9		11.1						
SCW-006A-08	D-4	5.0	GM	13.3	81.2		18.8						
SCW-007-08	D-1	4.5-5.2	GP	13.7	95.5		4.5						
SCW-008-08	D-1	5.0-6.5	GM	9.1	85.4		14.6						
SCW-008A-08	D-1	2.5	SM	12.9	88.9		11.1						
	D-3	5.0	SP	12.5	91.7		9.3						
SCW-009-08	C-1	2.5-4.0	GM	18.7	88.1		11.9						
	D-4	9.0-10.5	GM	19.0	89.6		10.4						
SCW-012-08	D-1	2.0-2.7	GM	15.0	84.3		15.7						
SCW-013-08	D-1	6.0-7.5	GM	14.5	84.0		16.0						
	D-3	8.7-10.2	GM	17.6	80.3		19.7						
	D-6	13.7-15.2	GM	16.4	84.5		15.5						
	D-8	18.7-20.2	GM	15.0	89.8		10.2						
	D-24	53.7-55.2	SM	28.8	86.8		13.2						
	D-26	58.7-60.2	SP	31.6	92.3		7.7						
	C-27	60.2-60.7	SM	29.6	89.9		10.1						
	D-29	63.7-65.2	SM	22.9	80.8		19.2						
	D-31	68.7-70.2	SM	27.0	85.2		14.8						
	D-2	4.5	GM	17.3	53.1	33.8	13.1						
	D-3	7.0	SP	8.8	40.0	58.8	0.2						
SSE-006-08	D-1	2.5	GW	4.9	73.7	25.6	0.7						
	D-2	4.0	GW	2.0									
SSE-008-08	D-1	2.5	SW	13.9									
	D-3	5.0	SM	29.2	34.9	57.5	7.6						
SSE-009-08	D-1	2.0	SM	11.0	33.0	54.4	13.1						
	D-2	4.5	SC	20.8	30.3	45.1	24.6						
	D-3	7.0	SM	15.3	40.4	45.1	13.5						

Table 3.6 (continued)
2008-2010 Laboratory Index Test Results

Boring No. & Sample Information				Water Content (%)	Grain Size Analysis			Atterberg Limits			Organic Content (%)	pH	Resistivity (Ω·cm)
Boring No.	Sample No.	Sample Depth (feet)	USCS/WSDOT Classification		Gravel (%)	Sand (%)	Silt and Clay (%)	LL	PL	PI			
SSE-010-08	D-1	2.0	GM	16.8	43.7	41.7	14.6						
	D-2	4.5	GW	24.3	54.3	35.9	9.8						
	D-3	7.0	SM	29.4	12.5	66.0	21.5						
SSE-011-08	D-1	2.5	GP	15.1	62.4	30.5	7.1						
	D-3	5.0	GM	17.4	45.6	41.7	12.7						
SSE-018-09	D-2	5.0-6.5	GW	8.7	49.2	43.8	7.0						
SSE-019-09	D-7	16.0-17.5	GW	7.1	58.7	34.6	6.8						
SSE-021-09	D-1	0.0-1.5	SM	3.4	34.9	54.8	10.3						
	D-3	6.5-8.0	GM	8.3	50.7	36.0	13.3						
	D-5	11.5-13.0	SM	9.0	41.4	45.6	13.0						
SSE-022-09	D-2	4.0-5.5	GM	11.1	52.9	36.5	10.6						
SW2-008-08	D-3	9.0-10.5	SW	20.9	4.9	87.0	8.1						
	D-4	19.0-20.5	CL	54.2				49	22	27			
	U-5	24.0-26.5	ML	44.1	0.0	0.4	99.6	30	24	6			
	D-7	34.0-35.5	SM	25.1	1.1	87.5	11.4						
	D-11	54.0-55.5	SW	9.6	41.5	53.9	4.6						
	D-13	64.0-65.5	GW	7.2	57.8	38.2	4.0						
	D-15	74.0-75.5	GW	6.6	50.1	45.0	4.9						
	D-19	94.0-95.5	SM	9.7	19.0	48.8	32.2						
	D-20	104.0-105.5	SP	13.8	6.5	86.7	6.8						
SW2-009-08	D-1	1.0-2.5	SM	20.3	12.6	75.6	11.8						
	D-3a	19.0-20.0	OH	118.3				89	47	42	9.4		
	D-3b	20.0-20.5	SM	13.4	9.4	75.0	15.6						
	D-5	29.0-30.5	SW	15.4	31.9	60.0	8.2						
	D-6	39.0-40.5	CL	47.5				49	23	26			
	S-7	44.0-46.0	CL	40.4	0.0	3.1	96.9	37	21	16			
								32	21	11			
	D-8	54.0-55.5	SM	19.0	1.5	87.9	10.6						
	D-10	64.0-65.5	SP	17.7	0.0	90.1	9.9						
	D-12	76.0-77.5	SM	14.9	17.9	49.7	32.4						
	D-13a	86.0-87.5	SM	19.2	1.0	78.6	20.4						

Table 3.6 (continued)
2008-2010 Laboratory Index Test Results

Boring No. & Sample Information				Water Content (%)	Grain Size Analysis			Atterberg Limits			Organic Content (%)	pH	Resistivity (Ω·cm)
Boring No.	Sample No.	Sample Depth (feet)	USCS/WSDOT Classification		Gravel (%)	Sand (%)	Silt and Clay (%)	LL	PL	PI			
	& b												
	D-15	96.0-97.5	SP	18.9	13.9	81.8	4.3						
TCC-002-08	D-2	7.0-8.5	GP	17.5	92.4		7.6						
	D-5	13.8-15.3	GM	19.7	78.0		22.0						
	D-10	23.8-24.6	GP	14.6	92.4		7.6						
	D-13	28.8-30.3	GP	11.9	90.7		9.3						
	D-18	38.8-40.3	SM	12.0	84.9		15.1						
TCC-003-08	D-2	3.4-4.9	GP	16.1	92.2		7.8						
	D-6	13.4-14.9	GP	37.7	90.9		9.1						
	D-8	18.4-19.9	SM	16.3	81.1		18.9						
	D-10 a	23.4-23.9	SM-SC	56.0	72.0		28.0						
	D-10 b	23.9-24.9	SM-SC	31.8	51.2		48.8						
	D-12	28.4-28.8	SP	23.8	93.6		6.4						
	D-14	33.4-34.1	SM	16.6	85.2		14.8						
	D-17	38.4-39.8	GM	10.3	88.4		11.6						
TCC-TP-002-08	D-1	5.5-5.7	SM									7.5	14,000
	D-2	10.0-10.2	GM									7.8	8,700
Phase 1D													
HMF-001-08	D-2	5.0-6.5	SM	78.1	24.1	55.6	20.3						
	D-4	10.0-11.4	SM-GM	16.4	42.3	42.1	15.6						
HMF-002-08	D-1	2.5-4.0	SM	13.6									
	D-2	5.0-6.5	GM	23.0	49.5	36.5	14.0						
	D-4	10.0-11.5	SM	29.2	34.5	47.1	18.4						
	D-5	14.0-15.3	GP	13.0									
HMF-003-08	D-1	2.5-4.0	GW	11.4	55.2	38.0	6.8						
	D-2	5.0-6.5	GW	11.5									
	D-5	15.0-16.5	SM	17.5	21.4	58.1	20.5						
HMF-004-08	D-4	3.8-5.3	GM	15.6	42.9	40.4	16.7						
	D-6	7.5-10.0	SM	20.2	38.7	40.7	20.6						
	D-8	8.8-10.3	SM	16.5	27.7	50.8	21.5						
	D-12	15.0-16.5	SM	28.7									

Table 3.6 (continued)
2008-2010 Laboratory Index Test Results

Boring No. & Sample Information				Water Content (%)	Grain Size Analysis			Atterberg Limits			Organic Content (%)	pH	Resistivity (Ω·cm)
Boring No.	Sample No.	Sample Depth (feet)	USCS/WSDOT Classification		Gravel (%)	Sand (%)	Silt and Clay (%)	LL	PL	PI			
HMF-005-08	D-1	2.0-2.3	GP	9.6									
	D-2	4.0-5.5	GM	14.1	47.1	38.7	14.2						
	D-3	6.5-8.0	SM	12.7									
	D-4	11.5-13.0	SM	20.7	41.4	48.6	10.0						
	D-5	14.0-14.3	GP	17.0									
HMF-006-08	D-2	1.3-1.6	SM	31.6									
	D-12	13.8-15.3	GW	9.0	63.4	29.6	7.0						
HMF-007-08	D-2	2.5-4.0	GW	10.4	50.9	39.4	9.7						
	D-4	5.0-6.5	GP	6.3									
	D-8	10.0-11.5	GP	33.1									
	D-10	12.5-14.0	SW	18.4	37.6	56.5	5.9						
	D-11	14.0-15.5	GW	9.9	62.1	32.7	5.2						
HMF-TP-001-09	S-1	2.5-3.0	SW	6.5	18.5	72.0	9.6						
	S-2	6.0-6.3	GM	13.5	48.0	40.6	11.3						
	S-3	11.0-11.3	GW	15.0	58.2	36.1	5.7						
HMF-TP-002-09	S-2	6.5-7.0	SM	21.7	35.7	56.6	7.8						
	S-3	10.0-10.5	GP	14.9	50.7	47.5	1.8						
HMF-TP-003-09	S-1	2.5-3.0	SM	8.7	28.5	60.6	10.9						
	S-2	7.0-7.5	SM	25.8	24.6	60.4	14.9						
	S-3	11.0-11.5	GW	3.5	75.4	23.5	1.1						
HMF-TP-004-10	S-1	1.8-2.0	SM		18.3	69.8	11.9						
	S-2	4.2-4.5	SM		24.6	49.8	25.6						
HMF-TP-005-10	S-1	3.3-3.6	SM		13.3	73.7	13.1						
	S-2	7.5-8.0	SW		43.5	47.4	9.1						
	S-3	10.5-11.0	GM		47.5	40.2	12.3						
INF-TP-007-10	S-1	4.0-4.2	GM		47.9	42.0	10.1						
	S-2	6.5-6.7	GM		46.7	43.3	10.0						
INF-TP-009-10	S-2	6.0-6.2	SM		43.3	43.6	13.1						
	S-3	9.5-9.7	GW		62.9	30.6	6.5						
INF-TP-010-10	D-3	10.0-12.0	SW		43.3	48.4	8.3						
INF-TP-011-10	S-2	6.0	GW		54.2	36.3	9.5						

Table 3.6 (continued)
2008-2010 Laboratory Index Test Results

Boring No. & Sample Information				Water Content (%)	Grain Size Analysis			Atterberg Limits			Organic Content (%)	pH	Resistivity (Ω·cm)
Boring No.	Sample No.	Sample Depth (feet)	USCS/WSDOT Classification		Gravel (%)	Sand (%)	Silt and Clay (%)	LL	PL	PI			
	S-3	9.0	SM		42.9	45.7	11.4						
INF-TP-012-10	D-3	10.0-12.0	GM		50.0	38.6	11.4						
INF-TP-013-10	S-1	3.0	GP		71.7	19.5	6.7						
	S-2	6.0	GM		46.6	38.5	14.9						
	S-3	9.0	GP		52.0	40.5	7.5						
INF-TP-014-10	S-3	9.0	SM		33.1	55.7	11.3						
	S-4	12.0	SM		38.4	49.8	11.7						
INF-TP-015-10	D-8/ D-9	25.0-25.7	GW		48.7	41.5	9.8						

Abbreviations
LL = liquid limit
PL = plastic limit
PI = plasticity index
Ω·cm = ohm-centimeter

Table 3.7
Interpreted Soil Strength and Consolidation Test Results

Boring No.	Sample No.	Sample Depth (feet)	USCS/WSDOT Classification	Water Content (%)	Consolidation				Consolidated-Undrained Triaxial Compression		
									Total Strength Parameters		Effective Strength Parameter, φ' (deg)
					σ'_p (psf)	C_c	C_r	e_0	φ (deg)	S_u/p'	
SW2-008-08	U-5	24.0-26.5	ML	52	940	0.28	0.06	1.66	14.4	0.26	34
SW2-009-08	S-7	44.0-46.0	CL	39	1,690	0.19	0.04	1.10	7.4	0.13	28
SCB-015-08	S-3	16.0-18.0	ML	114	640	1.05	0.15	3.09	--[1]	--[1]	--[1]
SCB-015-08	S-5	26.0-28.0	ML/MH	84	1,015	0.70	0.13	2.17	--[1]	--[1]	--[1]

[1]The CU test results for this sample were not usable

Abbreviations

σ'_p - Preconsolidation pressure
C_c – Compression index
C_r - Recompression index
e_0 – Initial void ratio
φ – Friction angle (total)
S_u – Undrained shear strength
p' – Vertical effective overburden stress
φ' - Friction angle (effective)

Table 3.8
Ring Shear Test Results

Boring No.	Sample No.	Sample Depth (feet)	LL (%)	PL (%)	PI (%)	Effective Friction Angle, φ', degrees	Residual Effective Friction Angle, φ_r', degrees
SCB-022-09	C-20	61.25	28	19	9	28	23

Abbreviations

LL = liquid limit
PL = plastic limit
PI = plasticity index

Table 3.9
2008-2010 Rock Unconfined Compressive Strength Test Results

Specimen	Depth (feet)	Compressive Strength (ksi)
RCB-004-10/C-25	105.1-106.0	5.6
RCB-005-10/C-9	83.3-84.2	2.0
RCB-005-10/C-10	89.4-90.1	1.9
RCB-006-10/C-2	56.7-57.7	2.7
RCB-006-10/C-3	60.5-61.2	1.0
RCB-007-10/C-3	114.6-115.3	1.9
RCB-007-10/C-4	120.2-121.0	2.4
RCB-008-10/C-2	98.5-99.2	2.0
SCB-019-09/C-7	34.5-35.1	19.9
SCB-019-09/C-9	42.6-43.2	23.9
SCB-020-09/C-11	52.3-53.0	8.4
SCB-020-09/C-16	67.0-67.7	9.1
SCB-020-09/C-22	90.5-91.5	18.8
SCB-023-09/C-8	40.2-40.7	25.6
SCB-023-09/C-12	52-52.7	26.2
SCB-024-09/C-14	41.75 - 42.3	12.6
SCB-024-09/C-21	77.75 - 78.45	5.1
SCB-026-09/S-1	42.9-43.5	17.2
SCB-026-09/S- 2	43.5-44.2	26.8
SCB-026-09/S- 3	45.0-45.8	19.2
SCB-026-09/S-4	52.2-52.8	13.9
SCB-026-09/S-5	56.6-57.2	24.0
SCB-026-09/S-6	60.3-60.9	19.9
SCB-026-09/S-7	62.1-62.8	22.0
SCB-026-09/S-8	63.3-64.0	20.4
SCB-027-09/S-1	42.6-43.1	13.0
SCB-027-09/S-2	44.1-44.8	15.8
SCB-027-09/S-3	47.9-48.1	14.3
SCB-027-09/S-4	51.7-52.4	17.8
SCB-027-09/S-5	54.8-55.8	12.2
SCB-027-09/S-6	61.6-62.4	11.1
SCB-027-09/S-7	62.9-63.5	14.2
SCB-027-09/S-8	67.4-68.0	8.5
SCB-027-09/S-9	68.4-69.0	5.8
SCB-028-09/S-1	34.9-35.5	15.1
SCB-028-09/S-3	42.0-42.5	13.3
SCB-028-09/S-5	44.0-44.9	7.6
SCB-028-09/S-6	45.6-46.1	13.2
SCB-028-09/S-8	62.5-63.4	22.8
SCB-028-09/S-9	66.5-67.4	18.1
SCB-028-09/S-10	78.5-79.1	23.1
SCB-029-09/S-1	37.7-38.5	11.0
SCB-029-09/S-2	40.8-41.7	19.6
SCB-029-09/S-3	42.7-43.5	17.4
SCB-029-09/S-4	46.0-46.6	14.5
SCB-029-09/S-5	47.8-48.4	16.7
SCB-029-09/S-6	55.3-56.2	19.8

Table 3.9 (continued)
2008-2010 Rock Unconfined Compressive Strength Test Results

Specimen	Depth (feet)	Compressive Strength (ksi)
SCB-029-09/S-7	66.6-67.2	6.7
SCB-030-09/S-1	30.8-31.5	5.8
SCB-030-09/S-2	38.0-38.6	10.4
SCB-030-09/S-3	38.6-39.3	10.9
SCB-030-09/S-4	40.0-40.6	15.1
SCB-030-09/S-5	42.9-43.6	3.7
SCB-030-09/S-6	44.7-45.4	15.6
SCB-030-09/S-7	49.0-49.6	17.9
SCB-030-09/S-8	52.6-53.3	15.6
SCB-030-09/S-9	59.5-60.1	13.0
SCB-030-09/S-10	63.1-63.8	15.2
SCB-030-09/S-11	73.3-73.9	17.4
SSD-006-09/C-17	68.9-69.6	44.6
SSD-006-09/C-19	76.0-76.7	35.3
SSD-007-09/C-17	93.8-94.6	22.8
SSD-007-09/C-18	96.1 - 96.7	27.0
SSD-008-09/C-9	45.3-45.9	16.2
SSD-008-09/C-13	61.3-62.0	23.1
SSD-008-09/C-14	62.7-63.5	22.0
SSD-009-09/C-12	64.2-65	14.1
SSD-009-09/C-13	71.8-72.5	22.1
SSD-009-09/C-15	81.6-82.2	24.8
SSD-010-09/C-12	57.4-58.2	13.8
SSD-010-09/C-13	64.3-65.0	15.6
SSD-010-09/C-15	72.6-73.2	15.8
SSD-010-09/C-16	78.6-79.2	14.9
SSD-011-09/C-6	33.3 - 34.1	10.6
SSD-011-09/C-8	42.6-43.3	17.1
SSD-011-09/C-9	49.6-50.2	13.8
SSD-011-09/C-11	56.9-57.7	18.7
SSD-012-09/C-15	68.3-69.0	9.9
SSD-012-09/C-16	73.7-74.4	13.4
SSD-013-09/S-2	18.6-19.3	16.1
SSD-014-09/S-1	10.0-11.0	5.7
SSD-014-09/S-2	22.9-23.6	2.6
SSD-015-09/S-1	5.8-6.5	27.0
SSD-015-09/S-2	18.0-18.9	29.8

Abbreviations
ksi = kips per square inch

Table 3.10
2008-2010 Rock Point Load Test Results

Boring ID/Sample #	Depth (feet)	Rock Type	Weath. State [1]	Test Type [2]	Dia.[3] (mm)	Width [4] (mm)	De [5] (mm)	qu [6] (ksi)	ISRM [7] Class.	ISRM Description	Failure [8] Type
CUL-020-10/C-10	36.9	Andesite	I	D	60.0	220.0	60.00	16.1	R4	Strong	1
CUL-020-10/C-11	45	Andesite	I	D	60.0	100.0	60.00	13.3	R3	Mod. Strong	1
CUL-020-10/C-12	47	Andesite	I	D	60.0	230.0	60.00	11.2	R3	Mod. Strong	1
CUL-020-10/C-13	47.9	Andesite	I	D	60.0	220.0	60.00	4.8	R2	Mod. Weak	6
CUL-020-10/C-13	49.7	Andesite	I	D	60.0	200.0	60.00	12.8	R3	Mod. Strong	1
EMB-027-09/C-8	32.5	Sandstone	I	D	61	95	61	0.3	R1	Very Weak	6
EMB-027-09/C-9	37.3	Sandstone	I	D	61	82	61	0.3	R1	Very Weak	6
EMB-027-09/C-9	40.5	Sandstone	I	D	61	168	61	10.7	R3	Mod. Strong	1
EMB-027-09/C-10	43.8	Sandstone	I	D	61	210	61	11.1	R3	Mod. Strong	1
EMB-027-09/C-11	47.1	Sandstone	I	D	61	120	61	4.2	R2	Mod. Weak	1
RCB-004-10/C-20	89	Andesite	I	D	60.0	220.0	60.00	7.5	R3	Mod. Strong	6
RCB-004-10/C-21	91.3	Andesite	I	D	60.0	198.0	60.00	8.4	R3	Mod. Strong	1
RCB-004-10/C-22	95.8	Andesite	I	D	60.0	120.0	60.00	2.2	R1	Very Weak	6
RCB-004-10/C-23	102.2	Andesite	I	D	60.0	101.0	60.00	5.5	R2	Mod. Weak	6
RCB-004-10/C-24	104	Andesite	I	D	60.0	96.0	60.00	2.6	R1	Very Weak	6
RCB-005-10/C-9	83.1	Andesite	I	D	60.0	108.0	60.00	5.4	R2	Mod. Weak	1
RCB-005-10/C-9	84.5	Andesite	I	D	60.0	138.0	60.00	3.9	R2	Mod. Weak	1
RCB-005-10/C-10	89.1	Andesite	I	D	60.0	105.0	60.00	4.2	R2	Mod. Weak	1
RCB-005-10/C-10	90.3	Andesite	I	D	60.0	98.0	60.00	3.7	R2	Mod. Weak	1
RCB-005-10/C-11	91.2	Andesite	I	D	60.0	84.0	60.00	1.3	R1	Very Weak	1
RCB-006-10/C-1	53.4	Lapilli Tuff	I	D	60.0	158.0	60.00	1.5	R1	Very Weak	6
RCB-006-10/C-2	56.4	Lapilli Tuff	I	D	60.0	210.0	60.00	2.1	R1	Very Weak	1
RCB-006-10/C-2	57.9	Lapilli Tuff	I	D	60.0	125.0	60.00	1.2	R1	Very Weak	1
RCB-006-10/C-3	60.2	Andesite	I	D	60.0	122.0	60.00	2.2	R1	Very Weak	1
RCB-006-10/C-3	61.9	Andesite	I	D	60.0	108.0	60.00	17.7	R4	Strong	4
RCB-006-10/C-5	70	Andesite	I	D	60.0	155.0	60.00	0.5	R1	Very Weak	1
RCB-007-10/C-1	103.6	Andesite	I	D	60.0	92.0	60.00	5.8	R2	Mod. Weak	1
RCB-007-10/C-2	109	Andesite	I	D	60.0	78.0	60.00	6.1	R2	Mod. Weak	4
RCB-007-10/C-3	114.4	Andesite	I	D	60.0	106.0	60.00	2.6	R1	Very Weak	1
RCB-007-10/C-4	116.4	Andesite	I	D	60.0	220.0	60.00	4.3	R2	Mod. Weak	6
RCB-007-10/C-4	120	Andesite	I	D	60.0	140.0	60.00	5.9	R2	Mod. Weak	1

Table 3.10 (continued)
2008-2010 Rock Point Load Test Results

Boring ID/Sample #	Depth (feet)	Rock Type	Weath. State [1]	Test Type [2]	Dia.[3] (mm)	Width [4] (mm)	De [5] (mm)	qu [6] (ksi)	ISRM [7] Class.	ISRM Description	Failure [8] Type
RCB-008-10/C-1	94	Andesite	I	D	60.0	130.0	60.00	1.7	R1	Very Weak	1
RCB-008-10/C-2	98.3	Andesite	I	D	60.0	72.0	60.00	1.2	R1	Very Weak	1
RCB-008-10/C-2	99.5	Andesite	I	D	60.0	148.0	60.00	3.1	R1	Very Weak	1
RCB-008-10/C-4	106.6	Andesite	I	D	60.0	158.0	60.00	2.6	R1	Very Weak	1
RCB-008-10/C-4	107.9	Andesite	I	D	60.0	147.0	60.00	1.9	R1	Very Weak	1
RCW-001-08/C-21	50	Lapilli Tuff	I	D	61	38	61	35.0	R5	Very strong	1
RCW-001-08/C-22	52	Lapilli Tuff	I	D	61	53	61	5.5	R2	Mod. weak	6
RCW-001-08/C-22	52.4	Lapilli Tuff	I	D	61	31	61	12.1	R3	Mod. strong	1
RCW-001-08/C-24	63.1	Lapilli Tuff	I	D	61	37	61	22.7	R4	Strong	1
SCB-001-09/C-11	39.1	Lapilli Tuff	I	D	45	92	45	2.9	R1	Very Weak	1
SCB-001-09/C-12	45.5	Lapilli Tuff	I	D	45	210	45	8.3	R3	Mod. Strong	1
SCB-001-09/C-14	50.3	Lapilli Tuff	I	D	45	195	45	10.7	R3	Mod. Strong	1
SCB-001-09/C-16	57	Lapilli Tuff	I	D	45	153	45	12.5	R3	Mod. Strong	1
SCB-001-09/C-17	65	Lapilli Tuff	I	D	45	155	45	7.3	R2	Mod. Weak	1
SCB-002-08/C-14	39.5	Lipilli Tuff	I	D	45	152	45	44.3	R5	Very Strong	1
SCB-002-08/C-15	45.8	Lipilli Tuff	I	D	45	152	45	41.1	R5	Very Strong	1
SCB-002-08/C-17	54.0	Lipilli Tuff	I	D	45	152	45	31.5	R5	Very Strong	1
SCB-004-08/C-15	37.1	Lapilli Tuff	I	D	61	62	61	23.0	R4	Strong	1
SCB-004-08/C-16	42.1	Lapilli Tuff	I	D	61	65	61	35.8	R5	Very Strong	1
SCB-004-08/C-18	49.4	Lapilli Tuff	I	D	61	66	61	33.3	R5	Very Strong	1
SCB-005-08/C-21	58.7	Lapilli Tuff	I	D	45	48	45	25.6	R4	Strong	5
SCB-006-08/C-21	62.9	Lapilli Tuff	I	D	61	60	61	11.0	R3	Mod. Strong	1
SCB-006-08/C-23	74.1	Lapilli Tuff	I	D	61	37	61	3.2	R1	Very Weak	1
SCB-006-08/C-26	85.1	Lapilli Tuff	I	D	61	40	61	3.3	R1	Very Weak	1
SCB-006-08/C-27	90.4	Lapilli Tuff	I	D	61	62	61	5.6	R2	Mod. Weak	6
SCB-006-08/C-30	103.1	Lapilli Tuff	I	D	61	30	61	0.4	R1	Very Weak	6
SCB-007-08/C-19	79.8	Lapilli Tuff	I	D	61	135	61	11.2	R3	Mod. Strong	1
SCB-007-08/C-20	81.8	Ash Tuff	I	D	61	65	61	21.0	R4	Strong	1
SCB-007-08/C-20	86.3	Ash Tuff	I	D	61	70	61	28.1	R4	Strong	1

Table 3.10 (continued)
2008-2010 Rock Point Load Test Results

Boring ID/Sample #	Depth (feet)	Rock Type	Weath. State [1]	Test Type [2]	Dia.[3] (mm)	Width [4] (mm)	De [5] (mm)	qu [6] (ksi)	ISRM [7] Class.	ISRM Description	Failure [8] Type
SCB-007-08/C-21	90.5	Lapilli Tuff	I	D	61	75	61	5.2	R2	Mod. weak	6
SCB-009-08/C-7	29.0	Lapilli Tuff	I	D	61	90	61	6.6	R2	Mod. Weak	1
SCB-009-08/C-7	33.1	Lapilli Tuff	I	D	61	56	61	29.7	R5	Very Strong	1
SCB-009-08/C-8	38.3	Lapilli Tuff	I	D	61	83	61	23.8	R4	Strong	1
SCB-009-08/C-9	40.1	Lapilli Tuff	I	D	61	70	61	31.9	R5	Very Strong	1
SCB-009-08/C-9	43.2	Lapilli Tuff	I	D	61	68	61	30.2	R5	Very Strong	1
SCB-010-08/C-12	53.2	Lapilli Tuff	I	D	61	52	61	2.9	R1	Very Weak	6
SCB-010-08/C-12	57.1	Lapilli Tuff	I	D	61	68	61	18.4	R4	Strong Rock	1
SCB-010-08/C-13	58.8	Lapilli Tuff	I	D	61	34	61	11.4	R3	Mod. Strong	1
SCB-010-08/C-14	62.9	Lapilli Tuff	I	D	61	58	61	3.2	R1	Very Weak	1
SCB-010-08/C-15	68.1	Lapilli Tuff	I	D	61	50	61	4.8	R2	Mod. Weak	1
SCB-010-08/C-15	72.2	Lapilli Tuff	I	D	61	52	61	1.3	R1	Very Weak	1
SCB-011-08/C-18	54.5	Lapilli Tuff	III	D	60	120	60	7.2	R2	Mod. Weak	1
SCB-011-08/C-19	59.8	Lapilli Tuff	II	D	60	110	60	2.7	R1	Very Weak	1
SCB-011-08/C-20	62.5	Lapilli Tuff	III	D	60	120	60	17.6	R4	Strong	1
SCB-011-08/C-23	80.5	Lapilli Tuff	III	D	60	150	60	16.2	R4	Strong	1
SCB-011-08/C-25	88.5	Lapilli Tuff	II	D	60	110	60	0.5	R1	Very Weak	1
SCB-014-08/C-10	36.4	Lapilli Tuff	I	D	61	64	61	4.3	R2	Weak Rock	1
SCB-014-08/C-11	41.9	Lapilli Tuff	I	D	61	50	61	1.2	R1	Very Weak	1
SCB-011-08/C-18	54.5	Lapilli Tuff	III	D	60	120	60	7.2	R2	Mod. Weak	1
SCB-015-08/C-7	44.2	Lapilli Tuff	I	D	60	130	60	2.8	R1	Very Weak	1
SCB-015-08/C-8	48.7	Lapilli Tuff	I	D	60	260	60	4.6	R2	Mod. Weak	1
SCB-015-08/C-9	52.8	Lapilli Tuff	I	D	60	180	60	2.6	R1	Very Weak	1
SCB-015-08/C-10	57.1	Lapilli Tuff	I	D	60	270	60	1.9	R1	Very Weak	I
SCB-018-08/C-13	35.0	Lapilli Tuff	I	D	60	150	60	36.3	R5	Very Strong	1
SCB-018-08/C-14	41.5	Lapilli Tuff	I	D	60	160	60	31.8	R5	Very Strong	1
SCB-018-08/C-16	49.3	Lapilli Tuff	I	D	60	170	60	15.9	R4	Strong	1
SCB-019-09/C-7	34.3	Lapilli Tuff	I	D	45	110	45	36.2	R5	Very Strong	1
SCB-019-09/C-8	37.2	Lapilli Tuff	I	D	45	120	45	41.7	R5	Very Strong	1
SCB-019-09/C-8	39.4	Lapilli Tuff	I	D	45	75	45	19.1	R4	Strong	5
SCB-019-09/C-8	42.2	Lapilli Tuff	I	D	45	148	45	25.8	R4	Strong	1

Table 3.10 (continued)
2008-2010 Rock Point Load Test Results

Boring ID/Sample #	Depth (feet)	Rock Type	Weath. State [1]	Test Type [2]	Dia.[3] (mm)	Width [4] (mm)	De [5] (mm)	qu [6] (ksi)	ISRM [7] Class.	ISRM Description	Failure [8] Type
SCB-019-09/C-9	43.5	Lapilli Tuff	I	A	57	45	57	18.6	R4	Strong	1
SCB-019-09/C-9	45.5	Lapilli Tuff	I	D	45	180	45	31.4	R5	Very Strong	1
SCB-020-09/C-11	53.2	Lapilli Tuff	I	D	45	120	45	7.6	R3	Mod. Strong	1
SCB-020-09/C-12	57.3	Lapilli Tuff	I	D	45	223	45	23.0	R4	Strong	1
SCB-020-09/C-15	64.5	Lapilli Tuff	I	D	45	95	45	30.3	R5	Very Strong	1
SCB-020-09/C-17	72.5	Lapilli Tuff	I	D	45	55	45	8.8	R3	Mod. Strong	1
SCB-020-09/C-19	75.6	Lapilli Tuff	I	D	45	85	45	5.8	R2	Mod. Weak	6
SCB-020-09/C-21	83.6	Lapilli Tuff	I	D	45	62	45	26.3	R4	Strong	1
SCB-020-09/C-22	90	Lapilli Tuff	I	D	45	110	45	7.6	R3	Mod. Strong	1
SCB-021-08/C-9	30.2	Lapilli Tuff	I	D	61	45	61	5.0	R2	Mod. Weak	6
SCB-021-08/C-11	39.5	Lapilli Tuff	I	D	61	90	61	11.7	R3	Mod. Strong	1
SCB-021-08/C-13	50.5	Lapilli Tuff	I	D	61	75	61	19.1	R4	Strong	1
SCB-021-08/C-14	52.8	Lapilli Tuff	I	D	61	50	61	2.3	R1	Very Weak	1
SCB-021-08/C-15	59.0	Lapilli Tuff	I	D	61	75	61	10.6	R3	Mod. Strong	1
SCB-021-08/C-17	66.2	Lapilli Tuff	I	D	61	65	61	15.3	R4	Strong	1
SCB-022-09/C-18	55.2	Lapilli Tuff	I	D	61	140	61	18.6	R4	Strong Rock	1
SCB-022-09/C-20	61	Lapilli Tuff	I	D	61	130	61	22.1	R4	Strong Rock	4/1
SCB-022-09/C-21	66.3	Lapilli Tuff	I	D	61	290	61	28.8	R4	Strong Rock	1
SCB-022-09/C-22	71.25	Lapilli Tuff	I	D	61	200	61	10.2	R3	Mod. Strong	1
SCB-023-09/C-8	38.3	Lapilli Tuff	II	D	61	135	61	22.0	R4	Strong	1
SCB-023-09/C-10	43.9	Lapilli Tuff	II	D	45	140	45	36.5	R5	Very Strong	1
SCB-023-09/C-12	52.9	Lapilli Tuff	I	D	45	80	45	12.7	R3	Mod. Strong	1
SCB-024-09/C-13	38.5	Lapilli Tuff	I	D	61	240	61	5.0	R2	Mod. Weak	1
SCB-024-09/C-14	41.15	Lapilli Tuff	I	D	61	90	61	4.1	R2	Mod. Weak	1
SCB-024-09/C-15	46.25	Lapilli Tuff	I	D	61	90	61	0.7	R1	Very Weak	1
SCB-024-09/C-16	52.7	Lapilli Tuff	I	D	61	130	61	3.6	R1	Very Weak	1
SCB-024-09/C-17	56	Lapilli Tuff	I	D	61	190	61	0.7	R1	Very Weak	6
SCB-024-09/C-18	61.75	Lapilli Tuff	I	D	61	155	61	9.6	R3	Mod. Strong	1
SCB-024-09/C-19	66.4	Lapilli Tuff	I	D	61	220	61	1.6	R1	Very Weak	6
SCB-024-09/C-20	70.8	Lapilli Tuff	I	D	61	240	61	7.6	R3	Mod. Strong	1
SCB-024-09/C-20	71.3	Lapilli Tuff	I	D	61	480	61	5.2	R2	Mod. Weak	1
SCB-024-09/C-20	75.25	Lapilli Tuff	I	D	61	?	61	2.9	R1	Very Weak	1
SCB-024-09/C-21	78.75	Lapilli Tuff	I	D	61	160	61	2.9	R1	Very Weak	1

Table 3.10 (continued)
2008-2010 Rock Point Load Test Results

Boring ID/Sample #	Depth (feet)	Rock Type	Weath. State [1]	Test Type [2]	Dia.[3] (mm)	Width [4] (mm)	De [5] (mm)	qu [6] (ksi)	ISRM [7] Class.	ISRM Description	Failure [8] Type
SCB-024-09/C-22	83.75	Lapilli Tuff	I	D	61	100	61	4.7	R2	Mod. Weak	1

Table 3.10 (continued)
2008-2010 Rock Point Load Test Results

Boring ID/Sample #	Depth (feet)	Rock Type	Weath. State [1]	Test Type [2]	Dia.[3] (mm)	Width [4] (mm)	De [5] (mm)	qu [6] (ksi)	ISRM [7] Class.	ISRM Description	Failure [8] Type
SCB-025-09/C-15	44.5	Lapilli Tuff	I	D	61	93	61	28.2	R4	Strong	4
SCB-025-09/C-16	49.6	Lapilli Tuff	I	D	61	90	61	25.1	R4	Strong	1
SCB-025-09/C-18	54.6	Lapilli Tuff	I	D	61	65	61	8.4	R3	Mod. Strong	1
SCB-026-09/C-7	42.8	Lapilli Tuff	I	D	61	210	61	19.8	R4	Strong	1
SCB-026-09/PLT-1	44.7	Lapilli Tuff	I-II	D	61	82	61	45.5	R5	Very Strong	
SCB-026-09/C-8	48.2	Lapilli Tuff	I	D	61	125	61	26.9	R4	Strong	1
SCB-026-09/C-9	51.6	Lapilli Tuff	I	D	61	130	61	19.4	R4	Strong	4
SCB-026-09/C-9	51.9	Lapilli Tuff	I	A	86	61	82	17.8	R4	Strong	2
SCB-026-09/PLT-2	53.4	Lapilli Tuff	I	D	61	95	61	21.8	R4	Strong	
SCB-026-09/C-9	55.7	Lapilli Tuff	II	D	61	123	61	21.1	R4	Strong	1
SCB-026-09/C-10	57.7	Lapilli Tuff	I	D	61	118	61	33.8	R5	Very Strong	1
SCB-026-09/C-11	63.3	Lapilli Tuff	I	D	61	136	61	31.1	R5	Very Strong	1
SCB-027-09/C-9	42.2	Lapilli Tuff	I	D	61	100	61	3.4	R1	Very Weak	1
SCB-027-09/C-11	43.7	Lapilli Tuff	I-II	D	61	102	61	36.7	R5	Very Strong	1
SCB-027-09/PLT-1	44.8	Lapilli Tuff	I-II	D	61	111	61	26.9	R4	Strong	
SCB-027-09/C-12	49.9	Lapilli Tuff	II	D	61	150	61	17.9	R4	Strong	1
SCB-027-09/C-13	54.8	Lapilli Tuff	I-II	D	61	110	61	27.7	R4	Strong	1
SCB-027-09/C-14	57.7	Lapilli Tuff	I	D	61	122	61	23.3	R4	Strong	1
SCB-027-09/C-14	61.1	Lapilli Tuff	II	D	61	65	61	2.9	R1	Very Weak	1
SCB-027-09/C-15	63.8	Lapilli Tuff	I	D	61	160	61	25.5	R4	Strong	1
SCB-027-09/C-15	66.7	Lapilli Tuff	II	D	61	112	61	19.7	R4	Strong	1
SCB-027-09/PLT-2	68	Lapilli Tuff	I-II	D	61	54	61	26.0	R4	Strong	
SCB-027-09/PLT-3	70.5	Lapilli Tuff	I-II	D	61	109	61	20.0	R4	Strong	
SCB-027-09/C-16	71.9	Lapilli Tuff	II	D	61	165	61	7.9	R2	Mod. Weak	1
SCB-028-09/C-5	32.3	Lapilli Tuff	I	D	61	105	61	20.6	R4	Strong	1
SCB-028-09/C-6	33	Lapilli Tuff	II	D	61	131	61	13.0	R3	Mod Strong	1
SCB-028-09/PLT-1	35.5	Lapilli Tuff	I-II	D	61	31	61	22.3	R4	Strong	
SCB-028-09/PLT-2	36.4	Lapilli Tuff	I-II	D	61	78	61	13.9	R3	Mod. Strong	
SCB-028-09/PLT-3	37.8	Lapilli Tuff	I-II	D	61	35	61	20.9	R4	Strong	
SCB-028-09/C-7	38.2	Lapilli Tuff	I	D	61	128	61	2.3	R1	Very Weak	1
SCB-028-09/S-2	39	Lapilli Tuff	I	D	61	43	61	31.1	R5	Very Strong	
SCB-028-09/C-7	40.7	Lapilli Tuff	I	D	61	132	61	3.0	R1	Very Weak	1,6
SCB-028-09/S-4	42.8	Lapilli Tuff	I	D	61	60	61	10.7	R3	Mod. Strong	
SCB-028-09/S-7	46.2	Lapilli Tuff	I	D	61	54	61	24.1	R4	Strong	
SCB-028-09/C-8	47.2	Lapilli Tuff	I	D	61	162	61	11.5	R3	Mod Strong	1

Table 3.10 (continued)
2008-2010 Rock Point Load Test Results

Boring ID/Sample #	Depth (feet)	Rock Type	Weath. State [1]	Test Type [2]	Dia.[3] (mm)	Width [4] (mm)	De [5] (mm)	qu [6] (ksi)	ISRM [7] Class.	ISRM Description	Failure [8] Type
SCB-028-09/PLT-4	48.6	Lapilli Tuff	I	D	61	59	61	41.1	R5	Very Strong	
SCB-028-09/C-10	54.8	Lapilli Tuff	I	D	61	102	61	23.7	R4	Strong	1
SCB-028-09/C-11	56.6	Lapilli Tuff	I	D	61	92	61	6.2	R2	Mod. Weak	1
SCB-028-09/C-13	65.5	Lapilli Tuff	I	D	61	110	61	23.8	R4	Strong	1
SCB-028-09/PLT-5	68.3	Lapilli Tuff	I	D	61	58	61	6.3	R2	Mod. Weak	
SCB-028-09/C-14	69	Lapilli Tuff	I	D	61	103	61	9.0	R3	Mod Strong	1
SCB-028-09/C-15	73.3	Lapilli Tuff	I	D	61	73	61	3.4	R1	Very Weak	1
SCB-028-09/C-15	77.3	Lapilli Tuff	I	D	61	110	61	39.4	R5	Very Strong	1
SCB-028-09/PLT-6	77.7	Lapilli Tuff	I	D	61	47	61	6.5	R2	Mod. Weak	
SCB-028-09/C-16	78.2	Lapilli Tuff	I	A	71	61	74	11.2	R3	Mod Strong	5
SCB-028-09/C-16	81.5	Lapilli Tuff	I	D	61	128	61	2.3	R1	Very Weak	6
SCB-028-09/C-16	82.1	Lapilli Tuff	I	D	61	85	61	27.3	R4	Strong	1
SCB-029-09/C-7	27.2	Lapilli Tuff	I-II	D	61	122	61	2.0	R1	Very Weak	6
SCB-029-09/PLT-1	30.2	Lapilli Tuff	I-II	D	61	41	61	2.8	R1	Very Weak	
SCB-029-09/PLT-2	33	Lapilli Tuff	I-II	D	61	68	61	2.7	R1	Very Weak	
SCB-029-09/C-9	34.2	Lapilli Tuff	II	D	61	118	61	10.5	R3	Mod Strong	1
SCB-029-09/C-10	38.8	Lapilli Tuff	I	D	61	128	61	22.3	R4	Strong	1
SCB-029-09/C-10	42.3	Lapilli Tuff	I	A	61	61	69	3.8	R2	Mod. Weak	6
SCB-029-09/C-10	42.5	Lapilli Tuff	I	D	61	160	61	19.6	R4	Strong	1
SCB-029-09/C-11	46.8	Lapilli Tuff	I	A	61	61	69	9.9	R3	Mod Strong	5
SCB-029-09/C-11	47	Lapilli Tuff	I	D	61	119	61	15.0	R3	Mod Strong	1
SCB-029-09/C-11	47.5	Lapilli Tuff	I	A	61	61	69	23.8	R4	Strong	5
SCB-029-09/C-12	52.5	Lapilli Tuff	I	D	61	156	61	19.7	R4	Strong	1
SCB-029-09/C-13	57.6	Lapilli Tuff	I	D	61	81	61	6.1	R2	Mod. Weak	1
SCB-029-09/C-14	62.6	Lapilli Tuff	I	D	61	82	61	18.2	R4	Strong	1
SCB-029-09/C-15	65.3	Lapilli Tuff	I	D	61	126	61	11.8	R3	Mod Strong	1
SCB-029-09/S-8	68	Lapilli Tuff	I	D	61	64	61	19.7	R4	Strong	
SCB-029-09/C-16	71.3	Lapilli Tuff	I	D	61	188	61	13.5	R3	Mod Strong	1
SCB-030-09/C-4	29.7	Lapilli Tuff	II	D	61	161	61	3.7	R1	Very Weak	1
SCB-030-09/PLT-1	31.4	Lapilli Tuff	II	D	61	49	61	23.7	R4	Strong	
SCB-030-09/PLT-2	31.9	Lapilli Tuff	II	D	61	34	61	11.4	R3	Mod. Strong	
SCB-030-09/C-5	33.4	Lapilli Tuff	II	D	61	121	61	6.3	R2	Mod. Weak	1
SCB-030-09/C-6	37.4	Lapilli Tuff	II	D	61	148	61	8.7	R3	Mod Strong	1
SCB-030-09/C-6	39.4	Lapilli Tuff	I	A	64	61	71	11.3	R3	Mod Strong	2
SCB-030-09/C-7	42	Lapilli Tuff	I	D	61	10	61	23.7	R4	Strong	1
SCB-030-09/C-7	45.6	Lapilli Tuff	II	D	61	102	61	20.4	R4	Strong	1

Table 3.10 (continued)
2008-2010 Rock Point Load Test Results

Boring ID/Sample #	Depth (feet)	Rock Type	Weath. State [1]	Test Type [2]	Dia.[3] (mm)	Width [4] (mm)	De [5] (mm)	qu [6] (ksi)	ISRM [7] Class.	ISRM Description	Failure [8] Type
SCB-030-09/C-8	46.9	Lapilli Tuff	I	D	61	148	61	13.0	R3	Mod Strong	1
SCB-030-09/C-8	51.3	Lapilli Tuff	I	D	61	222	61	16.0	R3	Mod Strong	1
SCB-030-09/C-10	58.4	Lapilli Tuff	I	D	61	138	61	9.9	R3	Mod Strong	1
SCB-030-09/C-10	61.5	Lapilli Tuff	II	D	61	113	61	5.0	R2	Mod. Weak	1
SCB-030-09/C-11	64.7	Lapilli Tuff	I	D	71	152	61	12.9	R3	Mod Strong	1
SCB-030-09/C-12	67.4	Lapilli Tuff	II	D	61	119	61	0.7	R1	Very Weak	6
SCB-030-09/C-13	70.9	Lapilli Tuff	I	D	61	134	61	9.2	R3	Mod Strong	1
SCB-030-09/C-15	78.5	Lapilli Tuff	I	D	61	114	61	11.7	R3	Mod Strong	1
SCB-030-09/C-15	82.5	Lapilli Tuff	II	D	61	68	61	6.1	R2	Mod. Weak	1
SCB-030-09/C-16	87	Lapilli Tuff	I	D	61	108	61	12.8	R3	Mod Strong	1
SCB-031-10/C-2	9.8	Lapilli Tuff	I	D	60.0	80.0	60.00	15.0	R3	Mod. Strong	1
SCB-031-10/C-4	16.3	Lapilli Tuff	I	D	60.0	100.0	60.00	13.1	R3	Mod. Strong	1
SCB-031-10/C-4	17.3	Lapilli Tuff	I	D	60.0	100.0	60.00	35.9	R5	Very Strong	1
SCB-031-10/C-6	27.1	Lapilli Tuff	I	D	60.0	80.0	60.00	26.7	R4	Strong	2
SCB-031-10/C-6	29.3	Lapilli Tuff	I	D	60.0	200.0	60.00	29.2	R5	Very Strong	1
SCB-031-10/C-7	31.3	Lapilli Tuff	II	D	60.0	100.0	60.00	13.1	R3	Mod. Strong	4
SCB-032-10/C-6	26.8	Lapilli Tuff	II	D	60.0	70.0	60.00	30.0	R5	Very Strong	1
SCB-032-10/C-6	28.1	Lapilli Tuff	I-II	D	60.0	73.0	60.00	27.9	R4	Strong	1
SCB-032-10/C-7	32	Lapilli Tuff	I	D	60.0	135.0	60.00	20.3	R4	Strong	1
SCB-032-10/C-7	33.6	Lapilli Tuff	I	D	60.0	210.0	60.00	31.2	R5	Very Strong	1
SCB-032-10/C-9	43.8	Lapilli Tuff	I	D	60.0	138.0	60.00	31.7	R5	Very Strong	4
SCW-001-08/C-2	5.7	Lapilli Tuff	II	D	61	183	61	9.5	R3	Mod. Strong	1
SCW-001-08/C-4	12	Lapilli Tuff	I	D	61	244	61	8.7	R3	Mod. Strong	5
SCW-002-08/C-8	21.9	Lapilli Tuff	III	D	64	85	64	11.4	R3	Mod. Strong	1
SCW-002-08/C-9	24.4	Lapilli Tuff	I	D	64	108	64	21.9	R4	Strong	1
SCW-002-08/C-9	25.1	Lapilli Tuff	I	A	98	64	89	16.0	R4	Strong	5
SCW-002-08/C-9	26.7	Lapilli Tuff	II	D	64	63	64	19.4	R4	Strong	1
SCW-002-08/C-10	32.9	Lapilli Tuff	I	A	108	64	93	11.9	R3	Mod. Strong	5
SCW-002-08/C-10	33.2	Lapilli Tuff	I	D	64	69	64	18.4	R4	Strong	1
SCW-002A-08/C-5	11.5	Lapilli Tuff	III	D	61	244	61	28.3	R4	Strong	
SCW-003-08/C-12	26.3	Lapilli Tuff	II	D	61	59	61	9.9	R3	Mod. Strong	1
SCW-003-08/C-13	30.7	Lapilli Tuff	I	A	68	61	73	17.3	R4	Strong	5

Table 3.10 (continued)
2008-2010 Rock Point Load Test Results

Boring ID/Sample #	Depth (feet)	Rock Type	Weath. State [1]	Test Type [2]	Dia.[3] (mm)	Width [4] (mm)	De [5] (mm)	qu [6] (ksi)	ISRM [7] Class.	ISRM Description	Failure [8] Type
SCW-003-08/C-13	31	Lapilli Tuff	I	D	61	84	61	27.9	R4	Strong	1

Table 3.10 (continued)
2008-2010 Rock Point Load Test Results

Boring ID/Sample #	Depth (feet)	Rock Type	Weath. State [1]	Test Type [2]	Dia.[3] (mm)	Width [4] (mm)	De [5] (mm)	qu [6] (ksi)	ISRM [7] Class.	ISRM Description	Failure [8] Type
SCW-004-08/C-5	10.5	Lapilli Tuff	II	D	61	39	61	6.4	R2	Mod. Weak	1
SCW-004-08/C-7	18.7	Lapilli Tuff	I	D	61	90	61	12.1	R3	Mod. Strong	1
SCW-004-08/C-7	19.4	Lapilli Tuff	II	D	61	50	61	2.1	R1	Very weak	1
SCW-004-08/C-8	23.7	Lapilli Tuff	II	D	61	64	61	13.2	R3	Mod. Strong	1
SCW-004A-08/C-4	7.8	Lapilli Tuff	II	D	61	122	61	20.2	R4	Strong	
SCW-005-08/C-4	17.5	Lapilli Tuff	I	D	64	60	64	37.5	R5	Very strong	1
SCW-005-08/C-6	28.5	Lapilli Tuff	III	D	64	152	64	38.5	R5	Very strong	1
SCW-005-08/C-7	33.6	Lapilli Tuff	II	D	64	91	64	39.4	R5	Very strong	1
SCW-005-08/C-8	38.4	Lapilli Tuff	I	D	64	183	64	40.4	R5	Very strong	1
SCW-006-08/C-1	23.1	Lapilli Tuff	III	D	64	122	64	0.0	R0	Extremely weak	
SCW-006-08/C-2	28.5	Lapilli Tuff	II	D	64	152	64	9.4	R3	Mod. Strong	3
SCW-006-08/C-4	34.5	Lapilli Tuff	I	D	64	152	64	13.0	R3	Mod. Strong	4
SCW-006A-08/C-5	7.4	Lapilli Tuff	IV	D	61	152	61	3.3	R1	Weak	3
SCW-006A-08/C-6	11.9	Lapilli Tuff	III	D	61	213	61	7.1	R2	Mod. Weak	3
SCW-006A-08/C-7	15.7	Lapilli Tuff	IV	D	60	274	60	16.0	R4	Strong	3
SCW-007-08/C-1	7.8	Lapilli Tuff	I	D	63	105	63	33.1	R5	Very Strong	4
SCW-007-08/C-5	17.9	Lapilli Tuff	I	D	61	244	61	27.5	R4	Strong	1
SCW-008-08/C-2	8.5	Lapilli Tuff	III	D	61	165	61	11.5	R3	Mod. Strong	
SCW-008-08/C-4	15	Lapilli Tuff	II	D	61	254	61	11.0	R3	Mod. Strong	
SCW-008A-08/C-5	13	Lapilli Tuff	III	D	61	122	61	23.5	R4	Strong	3
SCW-008A-08/C-6	18.5	Lapilli Tuff	II	D	61	91	61	40.0	R5	Very strong	4
SCW-009-08/C-7	19.2	Lapilli Tuff	I	D	61	244	61	27.0	R4	Strong	
SCW-009-08/C-9	28	Lapilli Tuff	II	D	61	305	61	12.2	R3	Mod. Strong	
SCW-009-08/C-13	40.2	Lapilli Tuff	II	D	61	152	61	29.7	R5	Strong	
SCW-010-08/C-2	6.5	Lapilli Tuff	III	D	61	137	61	11.9	R3	Mod. Strong	
SCW-010-08/C-6	22.9	Lapilli Tuff	I	D	61	244	61	26.7	R4	Strong	
SCW-011-08/C-6	9.4	Lapilli Tuff	I	D	61	95	61	33.7	R5	Very strong	1
SCW-011-08/C-9	17.6	Lapilli Tuff	I	D	61	70	61	26.4	R4	Strong	1

Table 3.10 (continued)
2008-2010 Rock Point Load Test Results

Boring ID/Sample #	Depth (feet)	Rock Type	Weath. State [1]	Test Type [2]	Dia.[3] (mm)	Width [4] (mm)	De [5] (mm)	qu [6] (ksi)	ISRM [7] Class.	ISRM Description	Failure [8] Type
SSD-006-09/C-12	54.7	Lapilli Tuff	I	D	61	195	61	39.2	R5	Very Strong	1
SSD-006-09/C-15	59.8	Lapilli Tuff	I	D	45	118	45	33.0	R5	Very Strong	1
SSD-006-09/C-14	60.3	Lapilli Tuff	I	D	45	113	45	34.2	R5	Very Strong	1
SSD-006-09/C-17	68.7	Lapilli Tuff	I	D	45	100	45	38.4	R5	Very Strong	1
SSD-006-09/C-17	69.9	Lapilli Tuff	I	D	45	142	45	25.9	R4	Strong	1
SSD-006-09/C-18	74.8	Lapilli Tuff	I	D	45	120	45	8.8	R3	Mod. Strong	1
SSD-006-09/C-19	76.9	Lapilli Tuff	I	D	45	156	45	18.4	R4	Strong	1
SSD-007-09/C-12	64.2	Lapilli Tuff	I	D	45	55	45	2.5	R1	Very Weak	1
SSD-007-09/C-12	66.7	Lapilli Tuff	I	D	45	85	45	4.0	R2	Mod. Weak	6
SSD-007-09/C-13	73	Lapilli Tuff	I	D	45	88	45	9.7	R3	Mod. Strong	1
SSD-007-09/C-14	79.5	Lapilli Tuff	I	D	45	102	45	8.5	R3	Mod. Strong	1
SSD-007-09/C-15	84.8	Lapilli Tuff	I	D	45	108	45	5.1	R2	Mod. Weak	1
SSD-007-09/C-16	87.8	Lapilli Tuff	I	D	45	78	45	16.9	R4	Strong	1
SSD-007-09/C-17	93.6	Lapilli Tuff	I	D	45	102	45	10.1	R3	Mod. Strong	1
SSD-007-09/C-18	99.5	Lapilli Tuff	I	D	45	142	45	2.3	R1	Very Weak	1
SSD-008-09/C-9	43.7	Lapilli Tuff	I	D	61	223	61	2.8	R1	Very Weak	1
SSD-008-09/C-9	46.4	Lapilli Tuff	I	D	61	115	61	9.4	R3	Mod. Strong	1
SSD-008-09/C-10	50	Lapilli Tuff	I	D	45	165	45	6.4	R2	Mod. Weak	1
SSD-008-09/C-12	56.4	Lapilli Tuff	I	D	45	190	45	3.8	R2	Mod. Weak	1
SSD-008-09/C-13	61.1	Lapilli Tuff	I	D	45	133	45	6.7	R2	Mod. Weak	1
SSD-008-09/C-14	62.4	Lapilli Tuff	I	D	45	195	45	7.9	R3	Mod. Strong	1
SSD-008-09/C-14	63.8	Lapilli Tuff	I	D	45	152	45	2.6	R1	Very Weak	1
SSD-009-09/C-11	61.7	Lapilli Tuff	I	D	61	153	61	5.7	R2	Mod. Weak	1
SSD-009-09/C-11	63	Lapilli Tuff	I	D	61	85	61	4.4	R2	Mod. Weak	1
SSD-009-09/C-12	68	Lapilli Tuff	I	D	61	150	61	8.9	R3	Mod. Strong	1
SSD-009-09/C-13	71.1	Lapilli Tuff	I	D	61	190	61	9.7	R3	Mod. Strong	1
SSD-009-09/C-13	72.7	Lapilli Tuff	I	D	61	128	61	13.1	R3	Mod. Strong	1
SSD-009-09/C-14	78	Lapilli Tuff	I	D	61	192	61	2.0	R1	Very Weak	1
SSD-009-09/C-15	81	Lapilli Tuff	I	D	61	90	61	13.3	R3	Mod. Strong	1
SSD-009-09/C-15	82.4	Lapilli Tuff	I	D	61	118	61	21.0	R4	Strong	1
SSD-009-09/C-16	86.2	Lapilli Tuff	I	D	61	112	61	13.5	R3	Mod. Strong	1
SSD-010-09/C-12	55.8	Lapilli Tuff	I	D	61	92	61	3.4	R1	Very Weak	1
SSD-010-09/C-12	57.3	Lapilli Tuff	I	D	61	111	61	10.8	R3	Mod. Strong	1

Table 3.10 (continued)
2008-2010 Rock Point Load Test Results

Boring ID/Sample #	Depth (feet)	Rock Type	Weath. State [1]	Test Type [2]	Dia.[3] (mm)	Width [4] (mm)	De [5] (mm)	qu [6] (ksi)	ISRM [7] Class.	ISRM Description	Failure [8] Type
SSD-010-09/C-12	58.5	Lapilli Tuff	I	D	61	162	61	7.6	R3	Mod. Strong	1
SSD-010-09/C-13	64.1	Lapilli Tuff	I	D	61	192	61	17.1	R4	Strong	1
SSD-010-09/C-13	65.2	Lapilli Tuff	I	D	61	120	61	12.8	R3	Mod. Strong	1
SSD-010-09/C-14	66.7	Lapilli Tuff	I	D	61	132	61	9.3	R3	Mod. Strong	1
SSD-010-09/C-15	71.8	Lapilli Tuff	I	D	61	180	61	4.5	R2	Mod. Weak	1
SSD-010-09/C-15	73.5	Lapilli Tuff	I	D	61	170	61	12.9	R3	Mod. Strong	1
SSD-010-09/C-16	78.2	Lapilli Tuff	I	D	61	150	61	6.6	R2	Mod. Weak	1
SSD-010-09/C-16	79.5	Lapilli Tuff	I	D	61	155	61	12.8	R3	Mod. Strong	1
SSD-011-09/C-6	33.7	Lapilli Tuff	II	D	61	210	61	3.1	R1	Very Weak	1
SSD-011-09/C-7	38.8	Lapilli Tuff	II	D	61	172	61	7.1	R2	Mod. Weak	1
SSD-011-09/C-8	42.2	Lapilli Tuff	I	D	61	230	61	7.5	R3	Mod. Strong	1
SSD-011-09/C-8	43.6	Lapilli Tuff	I	D	61	183	61	8.1	R3	Mod. Strong	1
SSD-011-09/C-8	46.2	Lapilli Tuff	I	D	61	176	61	9.6	R3	Mod. Strong	1
SSD-011-09/C-9	49.4	Lapilli Tuff	I	D	61	120	61	8.1	R3	Mod. Strong	1
SSD-011-09/C-10	51.5	Lapilli Tuff	I	D	61	78	61	10.3	R3	Mod. Strong	1
SSD-011-09/C-11	56.7	Lapilli Tuff	I	D	61	114	61	14.8	R3	Mod. Strong	1
SSD-011-09/C-11	57.8	Lapilli Tuff	I	A	76	61	77	11.9	R3	Mod. Strong	1
SSD-012-09/C-9	37	Lapilli Tuff	I	D	61	110	61	15.7	R4	Strong	1
SSD-012-09/C-10	41.2	Lapilli Tuff	II	D	61	86	61	1.6	R1	Very Weak	6
SSD-012-09/C-10	42.3	Lapilli Tuff	II	D	61	122	61	2.4	R1	Very Weak	1
SSD-012-09/C-11	46.2	Lapilli Tuff	II	D	61	100	61	2.1	R1	Very Weak	1
SSD-012-09/C-11	49.1	Lapilli Tuff	II	D	61	98	61	2.2	R1	Very Weak	6
SSD-012-09/C-12	51.5	Lapilli Tuff	II	D	61	198	61	2.9	R1	Very Weak	6
SSD-012-09/C-13	59.5	Lapilli Tuff	II	D	61	120	61	2.5	R1	Very Weak	1
SSD-012-09/C-14	61.4	Lapilli Tuff	II	D	61	72	61	5.1	R2	Mod.Weak	1
SSD-012-09/C-15	68.2	Lapilli Tuff	I	D	61	110	61	15.2	R4	Strong	1
SSD-012-09/C-15	69.7	Lapilli Tuff	I	D	61	88	61	3.3	R1	Very Weak	6
SSD-012-09/C-16	72.3	Lapilli Tuff	I	D	61	178	61	8.1	R3	Mod. Strong	1
SSD-012-09/C-16	74.7	Lapilli Tuff	I	D	61	170	61	5.8	R2	Mod. Weak	1
SSD-012-09/C-17	79.2	Lapilli Tuff	I	D	61	88	61	4.6	R2	Mod. Weak	1
SSD-013-09/S-1	7.5	Lapilli Tuff	I-II	D	61	46	61	9.3	R3	Mod. Strong	
SSD-013-09/C-4	8.3	Lapilli Tuff	II	D	61	120	61	4.4	R2	Mod. Weak	1
SSD-013-09/C-4	8.8	Lapilli Tuff	II	D	61	61	61	0.6	R1	Very Weak	6
SSD-013-09/C-4	9	Lapilli Tuff	II	D	61	136	61	0.9	R1	Very Weak	6
SSD-013-09/C-6	14.2	Lapilli Tuff	I	D	61	86	61	48.8	R5	Very Strong	1

Table 3.10 (continued)
2008-2010 Rock Point Load Test Results

Boring ID/Sample #	Depth (feet)	Rock Type	Weath. State [1]	Test Type [2]	Dia.[3] (mm)	Width [4] (mm)	De [5] (mm)	qu [6] (ksi)	ISRM [7] Class.	ISRM Description	Failure [8] Type
SSD-013-09/C-8	17.7	Lapilli Tuff	I	D	61	116	61	49.6	R5	Very Strong	4
SSD-013-09/C-9	21.2	Lapilli Tuff	I	D	61	178	61	41.1	R5	Very Strong	1
SSD-014-09/C-4	8.9	Lapilli Tuff	I	D	61	108	61	6.3	R2	Mod. Weak	1
SSD-014-09/C-5	11.2	Lapilli Tuff	I	D	61	116	61	4.4	R2	Mod. Weak	1
SSD-014-09/C-6	16.7	Lapilli Tuff	I	D	61	151	61	18.0	R4	Strong	1
SSD-014-09/C-7	22.2	Lapilli Tuff	I	D	61	98	61	3.3	R1	Very Weak	1
SSD-014-09/C-7	24.6	Lapilli Tuff	I	D	61	96	61	10.0	R3	Mod. Strong	1
SSD-015-09/C-4	6.8	Lapilli Tuff	I	D	61	192	61	27.3	R4	Strong	1
SSD-015-09/C-6	11.7	Lapilli Tuff	I	D	61	117	61	29.7	R5	Very Strong	1
SSD-015-09/C-7	18	Lapilli Tuff	I	D	61	112	61	43.5	R5	Very Strong	1
SSD-015-09/C-7	21.3	Lapilli Tuff	I	D	61	152	61	27.0	R4	Strong	1
SSE-022-09/C-5	11.25	Lapilli Tuff	I	D	61	160	61	27.2	R4	Strong Rock	1
SSE-022-09/C-6	16	Lapilli Tuff	I	D	61	180	61	14.4	R3	Mod. Strong	1
SW2-007-08/C-16	88.8	Lapilli Tuff	I	D	45	100	45	37.3	R5	Very Strong	1
SW2-007-08/C-17	95.5	Lapilli Tuff	I	D	45	115	45	37.1	R5	Very Strong	1
SW2-008-08/C-16B	83.1	Lapilli Tuff	I	D	60	127	60	31.2	R5	Very Strong	1
SW2-008-08/C-17	86.8	Lapilli Tuff	I	D	60	125	60	31.3	R5	Very Strong	1

Notes
1. Weathering State: I - Fresh; II - Slightly weathered; III - Moderately weathered; IV - Highly weathered; V - Completely weathered
2. Test Type: A = Axial; D = Diametral
3. Diameter: The loaded side initial dimension
4. Width: For axial and irregular specimens, second dimension value to calculate D_e
5. D_e - Equivalent diameter: for diametral specimens, same as diameter, otherwise it is the square root of 4*Diameter*Width/π
6. Unconfined Compression Strength, q_u: taken as 24.5*$I_{s(50)}$ in MPa for 65 mm core size, and 22* $I_{s(50)}$ in MPa for 45 mm core size based on Table 1 in ASTM D 5731-02
7. ISRM Classification: R0: 0.28 to 1.03 MPa (0.04 to 0.15 ksi); R1: 1.03 to 24.8 MPa (0.15 to 3.6 ksi); R2: 24.8 to 50.3 MPa (3.6 to 7.3 ksi); R3: 50.3 to 103.4 MPa (7.3 to 15 ksi); R4:103.4 to 200 MPa (15 to 29 ksi); R5: > 200 MPa (>29 ksi).
8. Failure Type: 1 - Diametral normal failure; 2 - Axial normal failure; 3 - Block normal failure; 4 - Diametral, chipped failure; 5 - Axial, sheared at angle; 6 - Failed along healed joint

Table 4.1
Boulders Encountered at Project Structure Locations

Structure	No. Borings	No. of Borings with Boulders	Maximum Size Encountered, feet (Note 1)
Snowshed (SSD)	55	30-37 (Note 2)	6 feet; boulders to 12 feet have been placed on embankment slope
Slide Curve bridge and walls (SCB)	40	16	3.8
Slide Curve embankment stabilization (SW2)	17	8	5
Slide Curve median wall (SCW)	17	9	2.5
Resort Creek wall (RCW)	5	3	1.25
Resort Creek bridges (RCB)	12	4	1.33
MP 59.7 Unnamed Cr. Crossing	4	3	8 (Note 3)
Townsend Creek culvert (TCC)	3	1	3.3 (Note 4)

Notes
1. Maximum size of boulders encountered in soil borings as reported on the boring logs. Larger boulders may be encountered during construction.
2. Lower number is number of borings where boulders were explicitly identified in boring log; higher number includes borings where blasting was used but boulders were not explicitly identified.
3. Boulders possibly as large as 8 feet may have been encountered in CUL-019-10, but were not explicitly identified in boring log.
4. Encountered in test pit TCC-TP-01-008

Table 4.2
Summary of Interpreted Soil Properties

Soil Units	Saturated Unit Weights (pcf)	Effective Shear Strength Parameters	
		Cohesion (psf)	Friction Angle (degrees)
Primarily Fill	135	0	40
Gravel with Sand/Cobbles/Boulders/ Rock Fragments (Primarily Colluvium)	135	0	40
Organic Soil	85	50	25
Gravel with Sand/Cobbles (Native Soil)	130	0	38
Fine Sand and Silt	110	0	30
Clay/Silt (Lake Sediments)	100	Note 1	0
Sand/Gravel (Lake Sediments)	115	0	30

Notes
1. The effective shear strength of the Clay/Silt unit was characterized using $\varphi = 0$ and a ratio of average undrained shear strength to effective overburden stress equal to 0.26, that is, $S_u/p' = 0.26$.

Table 6.1
Summary of Snowshed Borings and Test Pits

Boring ID	Date Drilled		Elevation (feet)[1]	Project Datum[2]		Total Depth (feet)
	Started	Completed		Northing (feet)	Easting (feet)	
2008-2010 Borings and Test Pit						
SSD-006-09	6/2/09	6/4/09	2497.5	1,067,029	1,753,805	79
SSD-007-09	5/30/09	6/1/09	2497.0	1,066,796	1,753,908	100
SSD-008-09	6/9/09	6/11/09	2499.6	1,066,554	1,753,987	67
SSD-009-09	6/12/09	6/14/09	2507.2	1,066,916	1,753,869	89
SSD-010-09	6/14/09	6/16/09	2498.9	1,066,679	1,753,954	81
SSD-011-09	6/25/09	6/29/09	2538.1	1,067,246	1,753,717	58
SSD-012-09	6/23/09	6/23/09	2533.4	1,066,243	1,754,078	81
SSD-013-09	11/5/09	11/5/09	2539.6	1,067,289	1,753,828	21.5
SSD-014-09	11/3/09	11/4/09	2539.4	1,067,227	1,753,865	26
SSD-015-09	11/3/09	11/4/09	2537.9	1,067,145	1,753,906	21.5
SSD-016-10	7/27/10	7/27/10	2542.4	1,067,428	1,753,725	20
SSD-017-10	7/26/10	7/27/10	2540.4	1,067,347	1,753,779	20
SSD-018-10	7/20/10	7/20/10	2630.3	1,067,398	1,753,892	55
SSD-019-10	7/21/10	7/22/10	2634.1	1,067,407	1,753,897	50
SSD-TP-001-08	6/18/08	6/18/08	2533.3	1,067,054	1,753,851	12
Earlier Borings						
H-1-73	11/9/1973	11/21/1973	2453.1	1,066,731	1,753,799	97.5
H-2-74	6/13/1974	6/13/1974	2454.0	1,066,756	1,753,851	70
H-3-74	6/20/1974	6/20/1974	2451.5	1,066,918	1,753,773	60
H-4-74	6/27/1974	6/27/1974	2219.0	1,066,612	1,753,129	105
H-5-74	7/1974	7/1974	2453.2	1,066,413	1,753,923	42.5
H-6-74	Not reported	Not reported	2285.5	1,066,634	1,753,486	4
H-7-74	12/4/1974	12/4/1974	2515.6	1,066,447	1,754,019	65.5
H-8-74	12/4/1974	12/4/1974	2528.4	1,067,123	1,753,803	68
H-9-75	6/17/1975	6/17/1975	2518.8	1,067,107	1,753,766	74
H-10-75	6/18/1975	6/18/1975	2536.0	1,067,289	1,753,673	40
H-11-75	6/25/1975	6/25/1975	2519.7	1,067,196	1,753,725	49.5
H-12-75	6/18/1975	6/20/1975	2515.0	1,066,128	1,754,072	40.5
H-16-75	7/25/1975	7/25/1975	2532.0	1,067,251	1,753,737	40.5
H-17-75	10/15/1975	10/20/1975	2528.0	1,067,029	1,753,866	65
H-18-75	10/22/1975	10/22/1975	2527.0	1,066,857	1,753,935	88
H-19-75	10/28/1975	11/7/1975	2528.0	1,066,324	1,754,073	66.5

Table 6.1 (continued)
Summary of Snowshed Borings and Test Pits

Boring ID	Date Drilled		Elevation (feet)[1]	Project Datum[2]		Total Depth (feet)
	Started	Completed		Northing (feet)	Easting (feet)	
H-20-75	11/29/1975	11/29/1975	2510.0	1,067,013	1,753,827	76.3
H-21-75	11/11/1975	12/3/1975	2526.0	1,066,502	1,754,029	70
H-22-75	11/24/1975	11/24/1975	2507.2	1,066,313	1,754,035	68
H-23-75	12/16/1975	12/16/1975	2512.8	1,066,678	1,753,975	83.5
H-24-76	6/1976	6/1976	2479.0	1,066,666	1,753,933	94
H-25-76	6/22/1976	6/30/1976	2488.0	1,066,489	1,753,984	64.5
H-26-76	6/23/1976	6/23/1976	2530.0	1,066,207	1,754,096	22.3
H-27-76	6/29/1976	6/29/1976	2528.9	1,066,190	1,754,168	48.8
H-28-76	6/30/1976	6/30/1976	2530.5	1,066,101	1,754,104	22.7
H-29-76	7/14/1976	7/14/1976	2487.0	1,066,840	1,753,880	94.5
H-1-77	11/4/1977	11/4/1977	2505[3]	1,067,123	1,753,750	35
H-2-77	11/5/1977	11/5/1977	2456[3]	1,066,911	1,753,822	49
H-3-77	11/18/1977	11/18/1977	2455[3]	1,066,753	1,753,871	25
H-4-77	12/9/1977	12/9/1977	2455[3]	1,066,662	1,753,909	33
H-6-98	11/18/1998	11/19/1998	2456.7	1,066,789	1,753,809	89
H-7-98	11/8/1998	11/10/1998	2454.4	1,066,163	1,753,956	54.4
H-14-98	10/30/1998	11/4/1998	2445.7	1,066,664	1,753,652	69.5
H-18-98	12/19/1998	12/19/1998	2466.7	1,066,896	1,753,123	77.5
H-23-06	7/20/2006	7/21/2006	2484.7	1,067,343	1,753,534	20
H-24-06(OW)	6/8/2006	6/13/2006	2537.2	1,065,874	1,754,146	60
SSD-001-07	9/5/2007	9/5/2007	2542.8	1,067,507	1,753,679	30.7
SSD-002-07	8/27/2007	8/27/2007	2566.0	1,067,039	1,753,977	60
SSD-003-07	8/26/2007	8/27/2007	2567.8	1,066,807	1,754,064	60
SSD-004-07	9/5/2007	9/5/2007	2532.0	1,066,523	1,754,105	20.7
SSD-005-07	9/4/2007	9/4/2007	2530.9	1,066,273	1,754,163	30.7

Notes
Logs of 2008-2009 borings and test pits are provided in Appendix A
Logs of earlier borings are provided in Technical Memorandum No. 3 – Snowshed (URS 2008b)
[1]Vertical datum is the North American Vertical Datum (NAVD) 1988
[2]Horizontal datum is based on the Washington Coordinate System, South Zone, and North American Datum (NAD) 1983/91
[3]Estimated from topographic map

Table 6.2
Rock Unconfined Compression Test Results – Snowshed

Specimen	Depth	Compressive Strength (ksi)
SSD-006-09/C-17	68.9-69.6	44.6
SSD-006-09/C-19	76.0-76.7	35.3
SSD-007-09/C-17	93.8-94.6	22.8
SSD-007-09/C-18	96.1 - 96.7	27.0
SSD-008-09/C-9	45.3-45.9	16.2
SSD-008-09/C-13	61.3-62.0	23.1
SSD-008-09/C-14	62.7-63.5	22.0
SSD-009-09/C-12	64.2-65	14.1
SSD-009-09/C-13	71.8-72.5	22.1
SSD-009-09/C-15	81.6-82.2	24.8
SSD-010-09/C-12	57.4-58.2	13.8
SSD-010-09/C-13	64.3-65.0	15.6
SSD-010-09/C-15	72.6-73.2	15.8
SSD-010-09/C-16	78.6-79.2	14.9
SSD-011-09/C-6	33.3 - 34.1	10.6
SSD-011-09/C-8	42.6-43.3	17.1
SSD-011-09/C-9	49.6-50.2	13.8
SSD-011-09/C-11	56.9-57.7	18.7
SSD-012-09/C-15	68.3-69.0	9.9
SSD-012-09/C-16	73.7-74.4	13.4
SSD-013-09/S-2	18.6-19.3	16.1
SSD-014-09/S-1	10.0-11.0	5.7
SSD-014-09/S-2	22.9-23.6	2.6
SSD-015-09/S-1	5.8-6.5	27.0
SSD-015-09/S-2	18.0-18.9	29.8

Abbreviations
ksi = kips per square inch

Table 6.3
Rock Point Load Test Results - Snowshed

Boring ID/ Sample #	Depth (feet)	Rock Type	Weathering State [1]	Test Type [2]	Dia.[3] (mm)	Width [4] (mm)	D_e [5] (mm)	Unconfined Compressive Strength, q_u [6] ksi	ISRM [7] Class.	ISRM Description	Failure [8] Type
H-23-06/C2	9.5	Andesite	I	D	45.0	76.2	45.00	37.3	R5	Very strong	1
H-23-06/C2	9-9.25	Andesite	I	A	76.2	45.0	66.08	23.2	R4	Strong	2
H-23-06/C2	9.51-9.78	Andesite	I	A	82.3	45.0	68.67	20.7	R4	Strong	5
					Average for H-23-06:			**27.1**	**R4**	**Strong**	
H-24-06/C13	47.5	Andesite	III	D	65.0	85.3	65.00	3.2	R1	Very weak	1
H-24-06/C13	47.75	Andesite	III	D	65.0	94.5	65.00	6.2	R2	Moderately weak	1
H-24-06/C13	47.5-47.75	Andesite	III	A	76.2	65.0	79.41	4.8	R2	Moderately weak	5
H-24-06/C15	55-55.25	Andesite	II	A	76.2	65.0	79.41	24.5	R4	Strong	5
					Average for H-24-06:			**9.7**	**R3**	**Moderately strong**	
SSD-002-07/C5	12.3	Lapilli Tuff	I	D	77.0	85.3	77.00	20.2	R4	Strong	6
SSD-002-07/C6	16.2	Lapilli Tuff	II-III	D	78.0	85.3	78.00	1.2	R1	Very weak	1
SSD-002-07/C8	26.6	Lapilli Tuff	I	D	77.0	85.3	77.00	43.4	R5	Very strong	1
SSD-002-07/C11	33	Lapilli Tuff	I	A	73.0	77.0	84.60	25.2	R4	Strong	2
SSD-002-07/C11	33.3	Lapilli Tuff	I	D	77.0	85.3	77.00	29.9	R5	Very strong	4
SSD-002-07/C13	42.5	Lapilli Tuff	I	D	77.0	85.3	77.00	35.4	R5	Very strong	1
SSD-002-07/C13	42.7	Lapilli Tuff	I	A	79.0	77.0	88.01	27.0	R4	Strong	2
SSD-002-07/C15	51.3	Lapilli Tuff	I	A	84.0	77.0	90.75	28.3	R4	Strong rock	5
SSD-002-07/C15	51.8	Lapilli Tuff	I	D	72.0	85.3	72.00	31.7	R5	Very strong	1
SSD-002-07/C16	54.8	Lapilli Tuff	I	D	72.0	85.3	72.00	25.1	R4	Strong	4
					Average for SSD-002-07:			**26.7**	**R4**	**Strong**	
SSD-003-07/C8	25.2	Lapilli Tuff	I	A	85.0	77.0	91.29	10.0	R3	Mod. Strong	5
SSD-003-07/C8	25.7	Lapilli Tuff	I	D	77.0	85.3	77.00	25.0	R4	Strong	1
SSD-003-07/C9	30.5	Lapilli Tuff	I	A	91.0	77.0	94.45	12.7	R3	Mod. Strong	2
SSD-003-07/C10	34.8	Lapilli Tuff	I	D	77.0	85.3	77.00	22.1	R4	Strong	1
SSD-003-07/C11	37.4	Lapilli Tuff	I	A	69.0	77.0	82.25	31.8	R5	Very strong	2

Table 6.3 (continued)
Rock Point Load Test Results - Snowshed

Boring ID/ Sample #	Depth (feet)	Rock Type	Weathering State [1]	Test Type [2]	Dia.[3] (mm)	Width [4] (mm)	D_e [5] (mm)	Unconfined Compressive Strength, q_u [6] ksi	ISRM [7] Class.	ISRM Description	Failure [8] Type
SSD-003-07/C11	37.9	Lapilli Tuff	I	D	69.0	85.3	69.00	19.1	R4	Strong	1
SSD-003-07/C12	46.4	Lapilli Tuff	I	A	64.0	77.0	79.21	18.5	R4	Strong	5
SSD-003-07/C13	48.3	Lapilli Tuff	I	D	76.0	85.3	76.00	24.2	R4	Strong	4
SSD-003-07/C13	49.7	Lapilli Tuff	I	A	76.0	77.0	86.32	10.6	R3	Mod. Strong	5
SSD-003-07/C14	53.8	Lapilli Tuff	I	A	98.0	77.0	98.02	10.8	R3	Mod. Strong	2
SSD-003-07/C14	54.2	Lapilli Tuff	I	A	82.0	77.0	89.66	14.4	R3	Mod. Strong	5
SSD-003-07/C15	59.4	Lapilli Tuff	I	A	63.0	85.3	82.74	22.1	R4	Strong	5
SSD-003-07/C15	59.9	Lapilli Tuff	I	D	63.0	85.3	63.00	56.1	R5	Very strong	1
					Average for SSD-003-07:			**21.3**	**R4**	**Strong**	
SSD-006-09/C-12	54.7	Lapilli Tuff	I	D	60.8	195.0	60.8	39.2	R5	Very Strong	1
SSD-006-09/C-15	59.8	Lapilli Tuff	I	D	45.0	118.0	45.0	33.0	R5	Very Strong	1
SSD-006-09/C-15	60.3	Lapilli Tuff	I	D	45.0	113.0	45.0	34.2	R5	Very Strong	1
SSD-006-09/C-17	68.7	Lapilli Tuff	I	D	45.0	100.0	45.0	38.4	R5	Very Strong	1
SSD-006-09/C-17	69.9	Lapilli Tuff	I	D	45.0	142.0	45.0	25.9	R4	Strong	1
SSD-006-09/C-18	74.8	Lapilli Tuff	I	D	45.0	120.0	45.0	8.8	R3	Mod. Strong	1
SSD-006-09/C-19	76.9	Lapilli Tuff	I	D	45.0	156.0	45.0	18.4	R4	Strong	1
					Average for SSD-006-09:			**28.3**	**R4**	**Strong**	
SSD-007-09/C-12	64.2	Lapilli Tuff	I	D	45.0	55.0	45.0	2.5	R1	Very Weak	1
SSD-007-09/C-12	66.7	Lapilli Tuff	I	D	45.0	85.0	45.0	4.0	R2	Mod. Weak	6
SSD-007-09/C-13	73	Lapilli Tuff	I	D	45.0	88.0	45.0	9.7	R3	Mod. Strong	1
SSD-007-09/C-14	79.5	Lapilli Tuff	I	D	45.0	102.0	45.0	8.5	R3	Mod. Strong	1
SSD-007-09/C-15	84.8	Lapilli Tuff	I	D	45.0	108.0	45.0	5.1	R2	Mod. Weak	1
SSD-007-09/C-16	87.8	Lapilli Tuff	I	D	45.0	78.0	45.0	16.9	R4	Strong	1
SSD-007-09/C-17	93.6	Lapilli Tuff	I	D	45.0	102.0	45.0	10.1	R3	Mod. Strong	1
SSD-007-09/C-18	99.5	Lapilli Tuff	I	D	45.0	142.0	45.0	2.3	R1	Very Weak	1

Table 6.3 (continued)
Rock Point Load Test Results - Snowshed

Boring ID/ Sample #	Depth (feet)	Rock Type	Weathering State [1]	Test Type [2]	Dia.[3] (mm)	Width [4] (mm)	D_e [5] (mm)	Unconfined Compressive Strength, q_u [6] ksi	ISRM [7] Class.	ISRM Description	Failure [8] Type
					Average for SSD-007-09:			**7.4**	**R3**	**Mod. Strong**	
SSD-008-09/C-9	43.7	Lapilli Tuff	I	D	60.8	223.0	60.8	2.8	R1	Very Weak	1
SSD-008-09/C-9	46.4	Lapilli Tuff	I	D	60.8	115.0	60.8	9.4	R3	Mod. Strong	1
SSD-008-09/C-10	50	Lapilli Tuff	I	D	45.0	165.0	45.0	6.4	R2	Mod. Weak	1
SSD-008-09/C-12	56.4	Lapilli Tuff	I	D	45.0	190.0	45.0	3.8	R2	Mod. Weak	1
SSD-008-09/C-13	61.1	Lapilli Tuff	I	D	45.0	133.0	45.0	6.7	R2	Mod. Weak	1
SSD-008-09/C-14	62.4	Lapilli Tuff	I	D	45.0	195.0	45.0	7.9	R3	Mod. Strong	1
SSD-008-09/C-14	63.8	Lapilli Tuff	I	D	45.0	152.0	45.0	2.6	R1	Very Weak	1
					Average for SSD-008-09			**5.7**	**R2**	**Mod. Weak**	
SSD-009-09/C-11	61.7	Lapilli Tuff	I	D	60.8	153.0	60.8	5.7	R2	Mod. Weak	1
SSD-009-09/C-11	63	Lapilli Tuff	I	D	60.8	85.0	60.8	4.4	R2	Mod. Weak	1
SSD-009-09/C-12	68	Lapilli Tuff	I	D	60.8	150.0	60.8	8.9	R3	Mod. Strong	1
SSD-009-09/C-13	71.1	Lapilli Tuff	I	D	60.8	190.0	60.8	9.7	R3	Mod. Strong	1
SSD-009-09/C-13	72.7	Lapilli Tuff	I	D	60.8	128.0	60.8	13.1	R3	Mod. Strong	1
SSD-009-09/C-14	78	Lapilli Tuff	I	D	60.8	192.0	60.8	2.0	R1	Very Weak	1
SSD-009-09/C-15	81	Lapilli Tuff	I	D	60.8	90.0	60.8	13.3	R3	Mod. Strong	1
SSD-009-09/C-15	82.4	Lapilli Tuff	I	D	60.8	118.0	60.8	21.0	R4	Strong	1
SSD-009-09/C-16	86.2	Lapilli Tuff	I	D	60.8	112.0	60.8	13.5	R3	Mod. Strong	1
					Average for SSD-009-09:			**10.2**	**R3**	**Mod. Strong**	
SSD-010-09/C-12	55.8	Lapilli Tuff	I	D	60.8	92.0	60.8	3.4	R1	Very Weak	1
SSD-010-09/C-12	57.3	Lapilli Tuff	I	D	60.8	111.0	60.8	10.8	R3	Mod. Strong	1
SSD-010-09/C-12	58.5	Lapilli Tuff	I	D	60.8	162.0	60.8	7.6	R3	Mod. Strong	1
SSD-010-09/C-13	64.1	Lapilli Tuff	I	D	60.8	192.0	60.8	17.1	R4	Strong	1
SSD-010-09/C-13	65.2	Lapilli Tuff	I	D	60.8	120.0	60.8	12.8	R3	Mod. Strong	1
SSD-010-09/C-14	66.7	Lapilli Tuff	I	D	60.8	132.0	60.8	9.3	R3	Mod. Strong	1

Table 6.3 (continued)
Rock Point Load Test Results - Snowshed

Boring ID/ Sample #	Depth (feet)	Rock Type	Weathering State [1]	Test Type [2]	Dia.[3] (mm)	Width [4] (mm)	D_e [5] (mm)	Unconfined Compressive Strength, q_u [6] ksi	ISRM [7] Class.	ISRM Description	Failure [8] Type
SSD-010-09/C-15	71.8	Lapilli Tuff	I	D	60.8	180.0	60.8	4.5	R2	Mod. Weak	1
SSD-010-09/C-15	73.5	Lapilli Tuff	I	D	60.8	170.0	60.8	12.9	R3	Mod. Strong	1
SSD-010-09/C-16	78.2	Lapilli Tuff	I	D	60.8	150.0	60.8	6.6	R2	Mod. Weak	1
SSD-010-09/C-16	79.5	Lapilli Tuff	I	D	60.8	155.0	60.8	12.8	R3	Mod. Strong	1
					Average for SSD-010-09:			**9.8**	**R3**	**Mod. Strong**	
SSD-011-09/C-6	33.7	Lapilli Tuff	II	D	60.8	210.0	60.8	3.1	R1	Very Weak	1
SSD-011-09/C-7	38.8	Lapilli Tuff	II	D	60.8	172.0	60.8	7.1	R2	Mod. Weak	1
SSD-011-09/C-8	42.2	Lapilli Tuff	I	D	60.8	230.0	60.8	7.5	R3	Mod. Strong	1
SSD-011-09/C-8	43.6	Lapilli Tuff	I	D	60.8	183.0	60.8	8.1	R3	Mod. Strong	1
SSD-011-09/C-8	46.2	Lapilli Tuff	I	D	60.8	176.0	60.8	9.6	R3	Mod. Strong	1
SSD-011-09/C-9	49.4	Lapilli Tuff	I	D	60.8	120.0	60.8	8.1	R3	Mod. Strong	1
SSD-011-09/C-10	51.5	Lapilli Tuff	I	D	60.8	78.0	60.8	10.3	R3	Mod. Strong	1
SSD-011-09/C-11	56.7	Lapilli Tuff	I	D	60.8	114.0	60.8	14.8	R3	Mod. Strong	1
SSD-011-09/C-11	57.8	Lapilli Tuff	I	D	76.0	60.8	76.0	12.0	R3	Mod. Strong	1
					Average for SSD-011-09:			**8.9**	**R3**	**Mod. Strong**	
SSD-012-09/C-9	37	Lapilli Tuff	I	D	60.8	110.0	60.8	15.7	R4	Strong	1
SSD-012-09/C-10	41.2	Lapilli Tuff	II	D	60.8	86.0	60.8	1.6	R1	Very Weak	6
SSD-012-09/C-10	42.3	Lapilli Tuff	II	D	60.8	122.0	60.8	2.4	R1	Very Weak	1
SSD-012-09/C-11	46.2	Lapilli Tuff	II	D	60.8	100.0	60.8	2.1	R1	Very Weak	1
SSD-012-09/C-11	49.1	Lapilli Tuff	II	D	60.8	98.0	60.8	2.2	R1	Very Weak	6
SSD-012-09/C-12	51.5	Lapilli Tuff	II	D	60.8	198.0	60.8	2.9	R1	Very Weak	6
SSD-012-09/C-13	59.5	Lapilli Tuff	II	D	60.8	120.0	60.8	2.5	R1	Very Weak	1
SSD-012-09/C-14	61.4	Lapilli Tuff	II	D	60.8	72.0	60.8	5.1	R2	Mod.Weak	1
SSD-012-09/C-15	68.2	Lapilli Tuff	I	D	60.8	110.0	60.8	15.2	R4	Strong	1
SSD-012-09/C-15	69.7	Lapilli Tuff	I	D	60.8	88.0	60.8	3.3	R1	Very Weak	6

Table 6.3 (continued)
Rock Point Load Test Results - Snowshed

Boring ID/ Sample #	Depth (feet)	Rock Type	Weathering State [1]	Test Type [2]	Dia.[3] (mm)	Width [4] (mm)	D_e [5] (mm)	Unconfined Compressive Strength, q_u [6] ksi	ISRM [7] Class.	ISRM Description	Failure [8] Type
SSD-012-09/C-16	72.3	Lapilli Tuff	I	D	60.8	178.0	60.8	8.1	R3	Mod. Strong	1
SSD-012-09/C-16	74.7	Lapilli Tuff	I	D	60.8	170.0	60.8	5.8	R2	Mod. Weak	1
SSD-012-09/C-17	79.2	Lapilli Tuff	I	D	60.8	88.0	60.8	4.6	R2	Mod. Weak	1
					Average for SSD-012-09:			**5.5**	**R2**	**Mod. Weak**	
SSD-013-09/C-4	8.3	Lapilli Tuff	II	D	61.0	120.0	61.00	4.4	R2	Mod. Weak	1
SSD-013-09/C-4	8.8	Lapilli Tuff	II	D	61.0	61.0	61.00	0.6	R1	Very Weak	6
SSD-013-09/C-4	9	Lapilli Tuff	II	D	61.0	136.0	61.00	0.9	R1	Very Weak	6
SSD-013-09/C-6	14.2	Lapilli Tuff	I	D	61.0	86.0	61.00	48.8	R5	Very Strong	1
SSD-013-09/C-8	17.7	Lapilli Tuff	I	D	61.0	116.0	61.00	49.6	R5	Very Strong	4
SSD-013-09/C-9	21.2	Lapilli Tuff	I	D	61.0	178.0	61.00	41.1	R5	Very Strong	1
					Average for SSD-013-09:			**24.2**	**R4**	**Strong**	
SSD-014-09/C-4	8.9	Lapilli Tuff	I	D	61.0	108.0	61.00	6.3	R2	Mod. Weak	1
SSD-014-09/C-5	11.2	Lapilli Tuff	I	D	61.0	116.0	61.00	4.4	R2	Mod. Weak	1
SSD-014-09/C-6	16.7	Lapilli Tuff	I	D	61.0	151.0	61.00	18.0	R4	Strong	1
SSD-014-09/C-7	22.2	Lapilli Tuff	I	D	61.0	98.0	61.00	3.3	R1	Very Weak	1
SSD-014-09/C-7	24.6	Lapilli Tuff	I	D	61.0	96.0	61.00	10.0	R3	Mod. Strong	1
					Average for SSD-014-09:			**8.4**	**8.3**	**Mod. Strong**	
SSD-015-09/C-4	6.8	Lapilli Tuff	I	D	61.0	192.0	61.00	27.3	R4	Strong	1
SSD-015-09/C-6	11.7	Lapilli Tuff	I	D	61.0	117.0	61.00	29.7	R5	Very Strong	1
SSD-015-09/C-7	18	Lapilli Tuff	I	D	61.0	112.0	61.00	43.5	R5	Very Strong	1
SSD-015-09/C-7	21.3	Lapilli Tuff	I	D	61.0	152.0	61.00	27.0	R4	Strong	1
					Average for SSD-015-09:			**31.9**	**R5**	**Very Strong**	
SSD-016-10/C-2	8.8	Lapilli Tuff	II	D	60.0	100.0	60.00	4.7	R2	Mod. Weak	1
SSD-016-10/C-2	9.9	Lapilli Tuff	II	D	60.0	100.0	60.00	1.1	R1	Very Weak	4
SSD-016-10/C-3	13	Lapilli Tuff	I	D	60.0	125.0	60.00	11.6	R3	Mod. Strong	1

Table 6.3 (continued)
Rock Point Load Test Results - Snowshed

Boring ID/ Sample #	Depth (feet)	Rock Type	Weathering State [1]	Test Type [2]	Dia.[3] (mm)	Width [4] (mm)	D_e [5] (mm)	Unconfined Compressive Strength, q_u [6] ksi	ISRM [7] Class.	ISRM Description	Failure [8] Type
SSD-016-10/C-3	14.3	Lapilli Tuff	I	D	60.0	75.0	60.00	4.8	R2	Mod. Weak	1
SSD-016-10/C-4	19.1	Lapilli Tuff	I	D	60.0	125.0	60.00	7.0	R2	Mod. Weak	1
SSD-016-10/C-4	19.9	Lapilli Tuff	I	D	60.0	75.0	60.00	8.1	R3	Mod. Strong	1
					Average for SSD-016-09:			**6.2**	**R2**	**Mod. Weak**	
SSD-017-10/C-2	5.8	Lapilli Tuff	I	D	60.0	75.0	60.00	48.1	R5	Very Strong	1
SSD-017-10/C-2	7.8	Lapilli Tuff	I	D	60.0	100.0	60.00	40.3	R5	Very Strong	4
SSD-017-10/C-3	10.5	Lapilli Tuff	I	D	60.0	75.0	60.00	17.5	R4	Strong	1
SSD-017-10/C-3	12.2	Lapilli Tuff	I	D	60.0	100.0	60.00	30.0	R5	Very Strong	1
SSD-017-10/C-4	15.4	Lapilli Tuff	I	D	60.0	125.0	60.00	44.5	R5	Very Strong	1
SSD-017-10/C-4	16.5	Lapilli Tuff	I	D	60.0	125.0	60.00	39.6	R5	Very Strong	1
					Average for SSD-017-09:			**36.7**	**R5**	**Very Strong**	
SSD-018-10/C-13	40.2	Lapilli Tuff	I	D	60.0	100.0	60.00	25.8	R4	Strong	1
SSD-018-10/C-13	42.1	Lapilli Tuff	I	D	60.0	75.0	60.00	24.4	R4	Strong	4
SSD-018-10/C-14	48.6	Lapilli Tuff	I	D	60.0	100.0	60.00	20.4	R4	Strong	1
SSD-018-10/C-15	52.8	Lapilli Tuff	I	D	60.0	75.0	60.00	24.5	R4	Strong	1
SSD-018-10/C-15	53.6	Lapilli Tuff	I	D	60.0	100.0	60.00	25.4	R4	Strong	1
					Average for SSD-018-09:			**24.1**	**R4**	**Strong**	

Notes
1. Weathering State: I - Fresh; II - Slightly weathered; III - Moderately weathered; IV - Highly weathered; V - Completely weathered
2. Test Type: A = Axial; D = Diametral
3. Diameter: The loaded side initial dimension
4. Width: For axial and irregular specimens, second dimension value to calculate D_e
5. D_e - Equivalent diameter: for diametral specimens, same as diameter, otherwise it is the square root of 4*Diameter*Width/π
6. Unconfined Compression Strength, q_u: taken as $24.5*I_{s(50)}$ in MPa for 65 mm core size, and $22*I_{s(50)}$ in MPa for 45 mm core size based on Table 1 in ASTM D 5731-02
7. ISRM Classification: R0: 0.28 to 1.03 MPa (0.04 to 0.15 ksi); R1: 1.03 to 24.8 MPa (0.15 to 3.6 ksi); R2: 24.8 to 50.3 MPa (3.6 to 7.3 ksi); R3: 50.3 to 103.4 MPa (7.3 to 15 ksi); R4:103.4 to 200 MPa (15 to 29 ksi); R5: > 200 MPa (>29 ksi).
8. Failure Type: 1 - Diametral normal failure; 2 - Axial normal failure; 3 - Block normal failure; 4 - Diametral, chipped failure; 5 - Axial, sheared at angle; 6 - Failed along healed joint

Table 6.4
Pressuremeter Test Results

Boring ID	Test ID	Depth, feet	Initial Shear Modulus, psi	Unload-Reload Shear Modulus, psi	Limit Pressure, psi
SSD-006-09	1-90-07	66	600,000	>1,500,000	>2,000
	1-90-08	64.5	720,000	>4,500,000	>2,000
	1-90-09	63	1,200,000	>6,000,000	>2,000
	1-90-10	70.5	280,000	>1,500,000	>2,000
	1-90-11	69	360,000	>4,000,000	>2,000
	1-90-12	79	520,000	>2,000,000	>2,000
	1-90-13	77.5	1,000,000	>3,000,000	>2,000
	1-90-14	76	1,000,000	>4,500,000	>2,000
SSD-007-09	I-90-01	77	110,000	600,000	>2,000
	I-90-02	75.5	120,000	650,000	>2,000
	I-90-03	74	200,000	650,000	>2,000
	I-90-04	85	90,000	400,000	>2,000
	I-90-05	83.5	220,000	>1,000,000	>2,000
	I-90-06	82	200,000	750,000	>2,000
SSD-008-09	1-90-15	54	100,000	450,000	>2,000
	1-90-16	52	200,000	900,000	>2,000
	1-90-17	50	200,000	1,000,000	>2,000
	1-90-18	62	130,000	1,000,000	>2,000
	1-90-19	59.5	20,000	300,000	>2,000
	1-90-20	57	60,000	500,000	>2,000
	1-90-21	66.8	110,000	700,000	>2,000
	1-90-22	65	220,000	1,100,000	>2,000

Table 6.5
Drilled Shaft Bearing and Uplift Resistance Factors

Limit State	Resistance Factor, φ		
	Skin Friction	Tip Resistance	Uplift
Strength	0.55	0.5	0.45 (soil) 0.4 (rock)
Service	1.0	1.0	1.0
Extreme Event	1.0	1.0	0.8

Table 6.6
Drilled Shaft Design Parameters for Lateral Load Analysis using DFSAP

Layer (note 1)	Soil Type	Layer Thickness (feet)	Effective Unit Weight (pcf)	Effective Friction Angle (degrees)	Strain, e_{50} (unitless)	Rock Unconfined Compressive Strength (psf) (note 6)	Potential Liquefaction
Pier 2 Shafts							
1	Sand	Varies (note 3)	73 (note 2)	40	0.005	Not used	No
2	Rock	Varies (note 4)	88	Not used	0.004	158,000	No
West Portal Tower and Generator Room							
1	Sand	Varies (note 5)	73 (note 2)	40	0.005	Not used	No
2	Rock	Varies (note 4)	88	Not used	0.004	300,000	No
East Portal Tower							
1	Sand	Varies (note 5)	73 (note 2)	40	0.005	Not used	No
2	Rock	Varies (note 4)	88	Not used	0.004	160,000	No

Notes
1. Soil units 1 and 2 have been combined as a single layer.
2. Above the groundwater table, use a soil unit weight equal to 135 pcf. The groundwater table elevation may be assumed to equal the lake level. The lake level varies from approximately El. 2416 to El. 2517.
3. Thickness of layer can be calculated from the bedrock elevations at the shaft centerline provided in Figure 6.19. The soil layer thickness should be calculated at the downslope face of the shaft, where the bedrock elevation is assumed to be 3 feet lower than at the shaft centerline for an 8-foot diameter shaft.
4. The layer can be assumed to be thick enough to extend below the base of the shaft.
5. Thickness of layer can be calculated from the estimated bedrock elevation provided in Figure 6.26.
6. The value provided is the global rock mass strength, σ'_{cm}, calculated using the method presented in Hoek, et al. (2002). This value differs from and is higher than the rock mass unconfined compressive strength.

Table 6.7
Global Stability Analysis Results for Snowshed Temporary Soil Nail Wall

Direction	Location	Calculated Factor of Safety, Static Loading[1]		Results Figure No.
		High Groundwater and Lake Levels (El. 2517)	Low and Lake Levels (El. 2416)	
Transverse	LW 1354+50	1.60	1.59	H.1.1 – H.1.2
	LW 1357+39	1.63	1.83	H.1.3 – H.1.4

Notes
Included load of 50 kips/foot to represent footing loads beneath south wall of existing snowshed.
[1]An avalanche snow load surcharge of 500 psf at the soil nail wall face increasing to 750 psf at the south wall of existing snowshed was included for static analysis at WB 1357+39. A snow load surcharge of 320 psf was included for static analysis at WB 1354+50.

Table 6.8
Design Parameters for Snowshed Temporary Soil Nail Wall

Soil Type	Unit Weight (pcf)	Friction Angle (degrees)	Cohesion (psf)
Units 1 and 2 (Gravel w/ Sand/Cobbles/Boulders)	135	40	0
Bedrock	135	0	18,000

Notes
pcf = pounds per cubic foot
psf = pounds per square foot

Table 6.9
Resistance Factors for Wall 1 and Transformer Room Foundations

Limit State	Resistance Factor, φ		
	Bearing Resistance	Friction Resistance to Sliding	Passive Pressure Resistance to Sliding
Strength[1]			
Footings on Rock	0.45	0.8	0.5
Footings on Soil	0.45	0.9	Varies[3]
Service	1.0	N/A	N/A
Extreme Event[2]	0.9	0.9	0.9

Notes:
[1] Resistance factors for the strength limit state are recommended in the LRFD Bridge Design Specifications, Table 10.5.5.2.2-1.
[2] Resistance factors for the extreme event limit state are recommended in the GDM, Section 8.10.2.
[3]For Wall 1, it is recommended that passive resistance be neglected. For the transformer room, a resistance factor equal to 0.5 is recommended.

Table 7.1
Summary of Borings and Test Pit for Slide Curve Bridge and Walls

Boring ID	Date Drilled		Elevation (feet)[1]	Project Datum[2]		Total Depth (feet)
	Started	Completed		Northing (feet)	Easting (feet)	
2008-2010 Borings and Test Pit						
SCB-001-09	5/19/09	5/21/09	2497.1	1,064,890	1,754,427	66
SCB-002-08	9/22/08	9/25/08	2535.0	1,064,897	1,754,476	61.5
SCB-004-08	6/26/08	6/26/08	2490	1,064,582	1,754,479	51.8
SCB-005-08	9/26/08	9/29/08	2534.1	1,064,744	1,754,517	59.5
SCB-006-08	6/24/08	6/26/08	2505	1,064,424	1,754,500	107.5
SCB-007-08	9/30/08	10/1/08	2532.4	1,064,563	1,754,539	91.5
SCB-009-08	7/22/08	7/22/08	2490	1,064,607	1,754,471	44.0
SCB-010-08	7/16/08	7/17/08	2499	1,064,381	1,754,496	72.5
SCB-011-08	10/1/08	10/2/08	2532.1	1,064,419	1,754,540	92.0
SCB-014-08	9/2/08	9/3/08	2350	1,064,448	1,754,291	49.0
SCB-015-08	11/22/08	11/25/08	2238	1,064,315	1,753,967	59.7
SCB-016-08	8/26/08	8/26/08	2448	1,064,521	1,754,430	65.0
SCB-017-08	8/27/08	8/28/08	2450	1,064,471	1,754,424	70.0
SCB-018-08	9/30/08	10/1/08	2536.5	1,065,001	1,754,444	52.0
SCB-019-09	5/27/09	5/28/09	2503.7	1,065,039	1,754,382	47
SCB-020-09	5/26/09	5/27/09	2505.3	1,064,254	1,754,489	93.5
SCB-021-08	10/2/08	10/3/08	2533.0	1,064,269	1,754,528	69.5
SCB-022-09	6/12/09	6/14/09	2537.7	1,065,148	1,754,392	71.9
SCB-023-09	5/28/09	5/29/09	2502.7	1,065,146	1,754,347	56
SCB-024-09	6/9/09	6/12/09	2523.4	1,064,160	1,754,491	85.5
SCB-025-09	6/2/09	6/4/09	2536.8	1,065,031	1,754,435	55.6
SCB-026-09	10/9/09	10/11/09	2525.4	1,065,198	1,754,348	66.4
SCB-027-09	10/12/09	10/14/09	2523.3	1,065,099	1,754,390	72.4
SCB-028-09	10/16/09	10/22/09	2499.1	1,064,245	1,754,473	82.5
SCB-029-09	10/23/09	10/24/09	2515.4	1,064,190	1,754,487	73
SCB-030-09	10/24/09	10/25/09	2527.8	1,064,134	1,754,494	88.7
SCB-031-10	7/13/2010	7/13/2010	2539.1	1,065,277	1,754,387	35.0
SCB-032-10	7/14/2010	7/14/2010	2537.8	1,065,283	1,754,330	45.0
SCB-TP-003-08	7/9/08	7/9/08	2536.9	1,065,001	1,754,444	8.0

Table 7.1 (continued)
Summary of Borings and Test Pit for Slide Curve Bridge and Walls

Boring ID	Date Drilled		Elevation (feet)[1]	Project Datum[2]		Total Depth (feet)
	Started	Completed		Northing (feet)	Easting (feet)	
Previous Borings[3]						
SW1-001-07	9/11/2007	9/11/2007	2539.1	1,065,334	1,754,315	32.5
SW1-002-07	9/12/2007	9/13/2007	2536.6	1,065,001	1,754,445	46
SW1-003-07	7/18/2007	7/19/2007	2481.9	1,064,731	1,754,452	40
SW1-004-07	7/14/2007	7/15/2007	2478.4	1,064,505	1,754,474	56.3
SW1-005-07	9/17/2007	9/18/2007	2532.7	1,064,210	1,754,520	41.5
SW1-006-07	6/27/2007	6/27/2007	2531.9	1,063,907	1,754,471	60
SW1-007-07	9/19/2007	9/19/2007	2530.5	1,063,604	1,754,419	60
H-25-06	8/8/2006	8/9/2006	2535.0	1,064,858	1,754,490	64.8
H-26-06	8/7/2006	8/7/2006	2532.7	1,064,088	1,754,503	50.1
H-2-05	4/18/2005	4/19/2005	2510.7	1,064,028	1,754,456	63
H-8-98	11/5/1998	11/6/1998	2461.3	1,064,912	1,754,336	35
H-15-98	11/13/1998	11/13/1998	2445.7	1,064,867	1,754,187	33

Notes
[1]Vertical datum is the North American Vertical Datum (NAVD) 1988.
[2]Horizontal datum is based on the Washington Coordinate System, South Zone, and North American Datum (NAD) 1983/91.
[3]Northing and easting for the previous boring have been updated from State Plane datum (as reported on the log) to the Project datum that was adopted in 2007.

Table 7.2
Laboratory Test Results for Slide Curve Bridge and Walls

Boring No. & Sample Information					Grain Size Analysis			Atterberg Limits		
Boring No.	**Sample No.**	**Sample Depth (feet)**	**USCS/WSDOT Classification**	**Water Content (%)**	**Gravel (%)**	**Sand (%)**	**Silt + Clay (%)**	**LL**	**PL**	**PI**
2008 Laboratory Results										
SCB-002-08	D-1	5.0-6.5	SW	9.7	45.9	46.1	8.0			
	D-2	11.5-13.0								
	D-11	31.5-33.0	GW	7.0	53.2	38.2	8.6			
	D-13	36.5-37.3								
SCB-004-08	D-2	2.5-4.0	SW	27.6	0.4	94.1	5.5			
	D-8	17.5-19.0	GW	9.3	67.4	29.1	3.5			
	D-10	22.5-22.9								
	D-12	27.5-29.0	SM	16.4	35.1	52.5	12.4			
	D-14	32.5-34.0	SM-GM	13.6	44.3	44.4	11.3			
SCB-005-08	D-1	5.0-6.5	GP	8.5	58.2	33.4	8.4			
	D-2	10.0-10.2		8.5						
	D-4	16.5-18.0	GW	8.6	65.0	30.3	4.7			
	D-9	31.5-33.0	GW	6.6	52.5	37.7	9.8			
	D-11	36.0-37.5	SM	9.5	20.6	63.1	16.4			
SCB-006-08	D-2	1.5-3.0	SW	16.4	26.9	67.7	5.4			
	D-4	6.0-7.5	GP	10.9	74.3	22.5	3.3			
	D-6	11.5-11.7	GP							
	D-14	41.5-43.0	SW	14.8	43.9	46.5	9.5			
	D-18	51.5-51.9	SM	13.1	30.7	56.8	12.5			
SCB-007-08	D-2	5.0-6.5	GW	7.0	71.9	25.9	2.1			
	D-6 a & b	26.5-28.0	GW	13.0	51.1	41.6	7.3			
	D-13	56.5-57.2	SM	9.6	41.6	46.7	11.6			
	D-15	61.5-63.0	SM	10.5	37.7	51.0	11.3			

Table 7.2 (continued)
Laboratory Test Results for Slide Curve Bridge and Walls

Boring No. & Sample Information					Grain Size Analysis			Atterberg Limits		
Boring No.	**Sample No.**	**Sample Depth (feet)**	**USCS/WSDOT Classification**	**Water Content (%)**	**Gravel (%)**	**Sand (%)**	**Silt + Clay (%)**	**LL**	**PL**	**PI**
SCB-011-08	D-2	7.0-8.5	GW	6.7	55.2	37.8	7.0			
	D-12	32.0-33.5	GP	11.9	66.0	27.6	6.4			
	D-14	37.0-38.5	GP	11.9	66.0	27.6	6.4			
SCB-014-08	D-3	9.5-11.0	GW	5.5	65.2	33.6	1.2			
	D-5	19.5-21.0								
SCB-015-08	D-4	18.0-19.5	MH	36.2				60	34	26
	S-5	26.0-28.0	MH	52.2				53	30	24
SCB-021-08	D-2	7.0-8.5	GW	9.5	57.3	39.5	3.2			
	D-5	17.0-18.5	GP	6.5	74.1	21.8	4.1			
	D-7	22.0-23.4	GM	16.7	67.1	18.4	14.5			
	C-18	68.2-68.6	CL-ML	26.1				20	15	5
SCB-022-09	C-20	61.25-62.1	CL	21.6	0	8.9	91.1	28	19	9
SCB-028-09	F-3	50.1-50.3	GM	9.6	49.7	23.0	27.2			
	F-4	56.3-56.5	SM	13.0	42.1	45.2	12.8			
Previous Laboratory Results										
SW1-004-07	D-4	11.0-12.5	SW	11.0	6.8	92.5	0.7			
	D-6	16.0-17.5	SP	15.5	22.8	75.3	1.9			
	D-8	21.0-22.5	SP	3.3	22.8	77.1	0.1			
	C-9	22.5-26.0	GP	5.1	61.4	38.4	0.1			
	D-1A	104-105								
	D-1B	105-105.5	GM	14.7	46.5	40.3	13.2			

Abbreviations
LL = liquid limit
PL = plastic limit
PI = plasticity index

Table 7.3
Rock Unconfined Compression Test Results for Slide Curve Bridge and Walls

Specimen	Depth (feet)	Compressive Strength (ksi)
SCB-019-09/C-7	34.5-35.1	19.9
SCB-019-09/C-9	42.6-43.2	23.9
SCB-020-09/C-11	52.3-53.0	8.4
SCB-020-09/C-16	67.0-67.7	9.1
SCB-020-09/C-22	90.5-91.5	18.8
SCB-023-09/C-8	40.2-40.7	25.6
SCB-023-09/C-12	52-52.7	26.2
SCB-024-09/C-14	41.75 - 42.3	12.6
SCB-024-09/C-21	77.75 - 78.45	5.1
SCB-026-09/S-1	42.9-43.5	17.2
SCB-026-09/S- 2	43.5-44.2	26.8
SCB-026-09/S- 3	45.0-45.8	19.2
SCB-026-09/S-4	52.2-52.8	13.9
SCB-026-09/S-5	56.6-57.2	24.0
SCB-026-09/S-6	60.3-60.9	19.9
SCB-026-09/S-7	62.1-62.8	22.0
SCB-026-09/S-8	63.3-64.0	20.4
SCB-027-09/S-1	42.6-43.1	13.0
SCB-027-09/S-2	44.1-44.8	15.8
SCB-027-09/S-3	47.9-48.1	14.3
SCB-027-09/S-4	51.7-52.4	17.8
SCB-027-09/S-5	54.8-55.8	12.2
SCB-027-09/S-6	61.6-62.4	11.1
SCB-027-09/S-7	62.9-63.5	14.2
SCB-027-09/S-8	67.4-68.0	8.5
SCB-027-09/S-9	68.4-69.0	5.8
SCB-028-09/S-1	34.9-35.5	15.1
SCB-028-09/S-3	42.0-42.5	13.3
SCB-028-09/S-5	44.0-44.9	7.6
SCB-028-09/S-6	45.6-46.1	13.2
SCB-028-09/S-8	62.5-63.4	22.8
SCB-028-09/S-9	66.5-67.4	18.1
SCB-028-09/S-10	78.5-79.1	23.1
SCB-029-09/S-1	37.7-38.5	11.0
SCB-029-09/S-2	40.8-41.7	19.6
SCB-029-09/S-3	42.7-43.5	17.4
SCB-029-09/S-4	46.0-46.6	14.5
SCB-029-09/S-5	47.8-48.4	16.7
SCB-029-09/S-6	55.3-56.2	19.8
SCB-029-09/S-7	66.6-67.2	6.7
SCB-030-09/S-1	30.8-31.5	5.8
SCB-030-09/S-2	38.0-38.6	10.4
SCB-030-09/S-3	38.6-39.3	10.9
SCB-030-09/S-4	40.0-40.6	15.1
SCB-030-09/S-5	42.9-43.6	3.7
SCB-030-09/S-6	44.7-45.4	15.6
SCB-030-09/S-7	49.0-49.6	17.9
SCB-030-09/S-8	52.6-53.3	15.6
SCB-030-09/S-9	59.5-60.1	13.0
SCB-030-09/S-10	63.1-63.8	15.2
SCB-030-09/S-11	73.3-73.9	17.4

Abbreviations
ksi = kips per square inch

Table 7.4
Rock Point Load Test Results for Slide Curve Bridge and Walls

Boring ID/Sample #	Depth (feet)	Rock Type	Weath. State [1]	Test Type [2]	Dia.[3] (mm)	Width [4] (mm)	De [5] (mm)	qu [6] (ksi)	ISRM [7] Class.	ISRM Description	Failure [8] Type
2008-2009 Tests											
SCB-001-09/C-11	39.1	Lapilli Tuff	I	D	45	92	45	2.9	R1	Very Weak	1
SCB-001-09/C-12	45.5	Lapilli Tuff	I	D	45	210	45	8.3	R3	Mod. Strong	1
SCB-001-09/C-14	50.3	Lapilli Tuff	I	D	45	195	45	10.7	R3	Mod. Strong	1
SCB-001-09/C-16	57	Lapilli Tuff	I	D	45	153	45	12.5	R3	Mod. Strong	1
SCB-001-09/C-17	65	Lapilli Tuff	I	D	45	155	45	7.3	R2	Mod. Weak	1
SCB-002-08/C-14	39.5	Lapilli Tuff	I	D	44.5	152	44.5	44.3	R5	Very Strong	1
SCB-002-08/C-15	45.8	Lapilli Tuff	I	D	44.5	152	44.5	41.1	R5	Very Strong	1
SCB-002-08/C-17	54.0	Lapilli Tuff	I	D	44.5	152	44.5	31.5	R5	Very Strong	1
SCB-004-08/C-15	37.1	Lapilli Tuff	I	D	61	62	61	23	R4	Strong	1
SCB-004-08/C-16	42.1	Lapilli Tuff	I	D	61	65	61	35.8	R5	Very Strong	1
SCB-004-08/C-18	49.4	Lapilli Tuff	I	D	61	66	61	33.3	R5	Very Strong	1
SCB-005-08/C-21	58.7	Lapilli Tuff	I	D	45	48	45	25.6	R4	Strong	5
SCB-006-08/C-21	62.9	Lapilli Tuff	I	D	61	60	61	11	R3	Mod. Strong	1
SCB-006-08/C-23	74.1	Lapilli Tuff	I	D	61	37	61	3.2	R1	Very Weak	1
SCB-006-08/C-26	85.1	Lapilli Tuff	I	D	61	40	61	3.3	R1	Very Weak	1
SCB-006-08/C-27	90.4	Lapilli Tuff	I	D	61	62	61	5.6	R2	Mod. Weak	6
SCB-006-08/C-30	103.1	Lapilli Tuff	I	D	61	30	61	0.4	R1	Very Weak	6
SCB-007-08/C-19	79.8	Lapilli Tuff	I	D	61	135	61	11.2	R3	Mod. Strong	1
SCB-007-08/C-20	81.8	Ash Tuff	I	D	61	65	61	21	R4	Strong	1
SCB-007-08/C-20	86.3	Ash Tuff	I	D	61	70	61	28.1	R4	Strong	1
SCB-007-08/C-21	90.5	Lapilli Tuff	I	D	61	75	61	5.2	R2	Mod. weak	6
SCB-009-08/C-7	29.0	Lapilli Tuff	I	D	61	90	61	6.6	R2	Mod. Weak	1
SCB-009-08/C-7	33.1	Lapilli Tuff	I	D	61	56	61	29.7	R5	Very Strong	1
SCB-009-08/C-8	38.3	Lapilli Tuff	I	D	61	83	61	23.8	R4	Strong	1
SCB-009-08/C-9	40.1	Lapilli Tuff	I	D	61	70	61	31.9	R5	Very Strong	1
SCB-009-08/C-9	43.2	Lapilli Tuff	I	D	61	68	61	30.2	R5	Very Strong	1

Table 7.4 (continued)
Rock Point Load Test Results for Slide Curve Bridge and Wall

Boring ID/Sample #	Depth (feet)	Rock Type	Weath. State [1]	Test Type [2]	Dia.[3] (mm)	Width [4] (mm)	De [5] (mm)	qu [6] (ksi)	ISRM [7] Class.	ISRM Description	Failure [8] Type
SCB-010-08/C-12	53.2	Lapilli Tuff	I	D	61	52	61	2.9	R1	Very Weak	6
SCB-010-08/C-12	57.1	Lapilli Tuff	I	D	61	68	61	18.4	R4	Strong Rock	1
SCB-010-08/C-13	58.8	Lapilli Tuff	I	D	61	34	61	11.4	R3	Mod. Strong	1
SCB-010-08/C-14	62.9	Lapilli Tuff	I	D	61	58	61	3.2	R1	Very Weak	1
SCB-010-08/C-15	68.1	Lapilli Tuff	I	D	61	50	61	4.8	R2	Mod. Weak	1
SCB-010-08/C-15	72.2	Lapilli Tuff	I	D	61	52	61	1.3	R1	Very Weak	1
SCB-011-08/C-18	54.5	Lapilli Tuff	III	D	60	120	60	7.2	R2	Mod. Weak	1
SCB-011-08/C-19	59.8	Lapilli Tuff	II	D	60	110	60	2.7	R1	Very Weak	1
SCB-011-08/C-20	62.5	Lapilli Tuff	III	D	60	120	60	17.6	R4	Strong	1
SCB-011-08/C-23	80.5	Lapilli Tuff	III	D	60	150	60	16.2	R4	Strong	1
SCB-011-08/C-25	88.5	Lapilli Tuff	II	D	60	110	60	0.5	R1	Very Weak	1
SCB-014-08/C-10	36.4	Lapilli Tuff	I	D	61	64	61	4.3	R2	Weak Rock	1
SCB-014-08/C-11	41.9	Lapilli Tuff	I	D	61	50	61	1.2	R1	Very Weak	1
SCB-015-08/C-7	44.2	Lapilli Tuff	I	D	60	130	60	2.8	R1	Very Weak	1
SCB-015-08/C-8	48.7	Lapilli Tuff	I	D	60	260	60	4.6	R2	Mod. Weak	1
SCB-015-08/C-9	52.8	Lapilli Tuff	I	D	60	180	60	2.6	R1	Very Weak	1
SCB-015-08/C-10	57.1	Lapilli Tuff	I	D	60	270	60	1.9	R1	Very Weak	I
SCB-018-08/C-13	35.0	Lapilli Tuff	I	D	60	150	60	36.3	R5	Very Strong	1
SCB-018-08/C-14	41.5	Lapilli Tuff	I	D	60	160	60	31.8	R5	Very Strong	1
SCB-018-08/C-16	49.3	Lapilli Tuff	I	D	60	170	60	15.9	R4	Strong	1
SCB-019-09/C-7	34.3	Lapilli Tuff	I	D	45	110	45	36.2	R5	Very Strong	1
SCB-019-09/C-8	37.2	Lapilli Tuff	I	D	45	120	45	41.7	R5	Very Strong	1
SCB-019-09/C-8	39.4	Lapilli Tuff	I	D	45	75	45	19.1	R4	Strong	5
SCB-019-09/C-8	42.2	Lapilli Tuff	I	D	45	148	45	25.8	R4	Strong	1
SCB-019-09/C-9	43.5	Lapilli Tuff	I	A	57	45	57.2	18.6	R4	Strong	1
SCB-019-09/C-9	45.5	Lapilli Tuff	I	D	45	180	45	31.4	R5	Very Strong	1
SCB-020-09/C-11	53.2	Lapilli Tuff	I	D	45	120	45	7.6	R3	Mod. Strong	1
SCB-020-09/C-12	57.3	Lapilli Tuff	I	D	45	223	45	23.0	R4	Strong	1

Table 7.4 (continued)
Rock Point Load Test Results for Slide Curve Bridge and Wall

Boring ID/Sample #	Depth (feet)	Rock Type	Weath. State [1]	Test Type [2]	Dia.[3] (mm)	Width [4] (mm)	De [5] (mm)	qu [6] (ksi)	ISRM [7] Class.	ISRM Description	Failure [8] Type
SCB-020-09/C-15	64.5	Lapilli Tuff	I	D	45	95	45	30.3	R5	Very Strong	1
SCB-020-09/C-17	72.5	Lapilli Tuff	I	D	45	55	45	8.8	R3	Mod. Strong	1
SCB-020-09/C-19	75.6	Lapilli Tuff	I	D	45	85	45	5.8	R2	Mod. Weak	6
SCB-020-09/C-21	83.6	Lapilli Tuff	I	D	45	62	45	26.3	R4	Strong	1
SCB-020-09/C-22	90	Lapilli Tuff	I	D	45	110	45	7.6	R3	Mod. Strong	1
SCB-021-08/C-9	30.2	Lapilli Tuff	I	D	61	45	61	5	R2	Mod. Weak	6
SCB-021-08/C-11	39.5	Lapilli Tuff	I	D	61	90	61	11.7	R3	Mod. Strong	1
SCB-021-08/C-13	50.5	Lapilli Tuff	I	D	61	75	61	19.1	R4	Strong	1
SCB-021-08/C-14	52.8	Lapilli Tuff	I	D	61	50	61	2.3	R1	Very Weak	1
SCB-021-08/C-15	59.0	Lapilli Tuff	I	D	61	75	61	10.6	R3	Mod. Strong	1
SCB-021-08/C-17	66.2	Lapilli Tuff	I	D	61	65	61	15.3	R4	Strong	1
SCB-022-09/C-18	55.2	Lapilli Tuff	I	D	61	140	61	18.6	R4	Strong Rock	1
SCB-022-09/C-20	61	Lapilli Tuff	I	D	61	130	61	22.1	R4	Strong Rock	4/1
SCB-022-09/C-21	66.3	Lapilli Tuff	I	D	61	290	61	28.8	R4	Strong Rock	1
SCB-022-09/C-22	71.25	Lapilli Tuff	I	D	61	200	61	10.2	R3	Mod. Strong	1
SCB-023-09/C-8	38.3	Lapilli Tuff	II	D	61	135	61	22.0	R4	Strong	1
SCB-023-09/C-10	43.9	Lapilli Tuff	II	D	45	140	45	36.5	R5	Very Strong	1
SCB-023-09/C-12	52.9	Lapilli Tuff	I	D	45	80	45	12.7	R3	Mod. Strong	1
SCB-024-09/C-13	38.5	Lapilli Tuff	I	D	61	240	61	5.0	R2	Mod. Weak	1
SCB-024-09/C-14	41.15	Lapilli Tuff	I	D	61	90	61	4.1	R2	Mod. Weak	1
SCB-024-09/C-15	46.25	Lapilli Tuff	I	D	61	90	61	0.7	R1	Very Weak	1
SCB-024-09/C-16	52.7	Lapilli Tuff	I	D	61	130	61	3.6	R1	Very Weak	1
SCB-024-09/C-17	56	Lapilli Tuff	I	D	61	190	61	0.7	R1	Very Weak	6
SCB-024-09/C-18	61.75	Lapilli Tuff	I	D	61	155	61	9.6	R3	Mod. Strong	1
SCB-024-09/C-19	66.4	Lapilli Tuff	I	D	61	220	61	1.6	R1	Very Weak	6
SCB-024-09/C-20	70.8	Lapilli Tuff	I	D	61	240	61	7.6	R3	Mod. Strong	1
SCB-024-09/C-20	71.3	Lapilli Tuff	I	D	61	480	61	5.2	R2	Mod. Weak	1
SCB-024-09/C-20	75.25	Lapilli Tuff	I	D	61	--	61	2.9	R1	Very Weak	1
SCB-024-09/C-21	78.75	Lapilli Tuff	I	D	61	160	61	2.9	R1	Very Weak	1
SCB-024-09/C-22	83.75	Lapilli Tuff	I	D	61	100	61	4.7	R2	Mod. Weak	1

Table 7.4 (continued)
Rock Point Load Test Results for Slide Curve Bridge and Wall

Boring ID/Sample #	Depth (feet)	Rock Type	Weath. State [1]	Test Type [2]	Dia.[3] (mm)	Width [4] (mm)	De [5] (mm)	qu [6] (ksi)	ISRM [7] Class.	ISRM Description	Failure [8] Type

Table 7.4 (continued)
Rock Point Load Test Results for Slide Curve Bridge and Wall

Boring ID/Sample #	Depth (feet)	Rock Type	Weath. State [1]	Test Type [2]	Dia.[3] (mm)	Width [4] (mm)	De [5] (mm)	qu [6] (ksi)	ISRM [7] Class.	ISRM Description	Failure [8] Type
SCB-025-09/C-15	44.5	Lapilli Tuff	I	D	61	93	61	28.2	R4	Strong	4
SCB-025-09/C-16	49.6	Lapilli Tuff	I	D	61	90	61	25.1	R4	Strong	1
SCB-025-09/C-18	54.6	Lapilli Tuff	I	D	61	65	61	8.4	R3	Mod. Strong	1
SCB-026-09/C-7	42.8	Lapilli Tuff	I	D	61	210	61	19.8	R4	Strong	1
SCB-026-09/PLT-1	44.7	Lapilli Tuff	I-II	D	61	82	61	45.5	R5	Very Strong	
SCB-026-09/C-8	48.2	Lapilli Tuff	I	D	61	125	61	26.9	R4	Strong	1
SCB-026-09/C-9	51.6	Lapilli Tuff	I	D	61	130	61	19.4	R4	Strong	4
SCB-026-09/C-9	51.9	Lapilli Tuff	I	A	86	61	82	17.8	R4	Strong	2
SCB-026-09/PLT-2	53.4	Lapilli Tuff	I	D	61	95	61	21.8	R4	Strong	
SCB-026-09/C-9	55.7	Lapilli Tuff	II	D	61	123	61	21.1	R4	Strong	1
SCB-026-09/C-10	57.7	Lapilli Tuff	I	D	61	118	61	33.8	R5	Very Strong	1
SCB-026-09/C-11	63.3	Lapilli Tuff	I	D	61	136	61	31.1	R5	Very Strong	1
SCB-027-09/C-9	42.2	Lapilli Tuff	I	D	61	100	61	3.4	R1	Very Weak	1
SCB-027-09/C-11	43.7	Lapilli Tuff	I-II	D	61	102	61	36.7	R5	Very Strong	1
SCB-027-09/PLT-1	44.8	Lapilli Tuff	I-II	D	61	111	61	26.9	R4	Strong	
SCB-027-09/C-12	49.9	Lapilli Tuff	II	D	61	150	61	17.9	R4	Strong	1
SCB-027-09/C-13	54.8	Lapilli Tuff	I-II	D	61	110	61	27.7	R4	Strong	1
SCB-027-09/C-14	57.7	Lapilli Tuff	I	D	61	122	61	23.3	R4	Strong	1
SCB-027-09/C-14	61.1	Lapilli Tuff	II	D	61	65	61	2.9	R1	Very Weak	1
SCB-027-09/C-15	63.8	Lapilli Tuff	I	D	61	160	61	25.5	R4	Strong	1
SCB-027-09/C-15	66.7	Lapilli Tuff	II	D	61	112	61	19.7	R4	Strong	1
SCB-027-09/PLT-2	68	Lapilli Tuff	I-II	D	61	54	61	26.0	R4	Strong	
SCB-027-09/PLT-3	70.5	Lapilli Tuff	I-II	D	61	109	61	20.0	R4	Strong	
SCB-027-09/C-16	71.9	Lapilli Tuff	II	D	61	165	61	7.9	R2	Mod. Weak	1
SCB-028-09/C-5	32.3	Lapilli Tuff	I	D	61	105	61	20.6	R4	Strong	1
SCB-028-09/C-6	33	Lapilli Tuff	II	D	61	131	61	13.0	R3	Mod Strong	1
SCB-028-09/PLT-1	35.5	Lapilli Tuff	I-II	D	61	31	61	22.3	R4	Strong	
SCB-028-09/PLT-2	36.4	Lapilli Tuff	I-II	D	61	78	61	13.9	R3	Mod. Strong	
SCB-028-09/PLT-3	37.8	Lapilli Tuff	I-II	D	61	35	61	20.9	R4	Strong	
SCB-028-09/C-7	38.2	Lapilli Tuff	I	D	61	128	61	2.3	R1	Very Weak	1
SCB-028-09/S-2	39	Lapilli Tuff	I	D	61	43	61	31.1	R5	Very Strong	

Table 7.4 (continued)
Rock Point Load Test Results for Slide Curve Bridge and Wall

Boring ID/Sample #	Depth (feet)	Rock Type	Weath. State [1]	Test Type [2]	Dia.[3] (mm)	Width [4] (mm)	De [5] (mm)	qu [6] (ksi)	ISRM [7] Class.	ISRM Description	Failure [8] Type
SCB-028-09/C-7	40.7	Lapilli Tuff	I	D	61	132	61	3.0	R1	Very Weak	1,6
SCB-028-09/S-4	42.8	Lapilli Tuff	I	D	61	60	61	10.7	R3	Mod. Strong	
SCB-028-09/S-7	46.2	Lapilli Tuff	I	D	61	54	61	24.1	R4	Strong	
SCB-028-09/C-8	47.2	Lapilli Tuff	I	D	61	162	61	11.5	R3	Mod Strong	1
SCB-028-09/PLT-4	48.6	Lapilli Tuff	I	D	61	59	61	41.1	R5	Very Strong	
SCB-028-09/C-10	54.8	Lapilli Tuff	I	D	61	102	61	23.7	R4	Strong	1
SCB-028-09/C-11	56.6	Lapilli Tuff	I	D	61	92	61	6.2	R2	Mod. Weak	1
SCB-028-09/C-13	65.5	Lapilli Tuff	I	D	61	110	61	23.8	R4	Strong	1
SCB-028-09/PLT-5	68.3	Lapilli Tuff	I	D	61	58	61	6.3	R2	Mod. Weak	
SCB-028-09/C-14	69	Lapilli Tuff	I	D	61	103	61	9.0	R3	Mod Strong	1
SCB-028-09/C-15	73.3	Lapilli Tuff	I	D	61	73	61	3.4	R1	Very Weak	1
SCB-028-09/C-15	77.3	Lapilli Tuff	I	D	61	110	61	39.4	R5	Very Strong	1
SCB-028-09/PLT-6	77.7	Lapilli Tuff	I	D	61	47	61	6.5	R2	Mod. Weak	
SCB-028-09/C-16	78.2	Lapilli Tuff	I	A	71	61	74	11.2	R3	Mod Strong	5
SCB-028-09/C-16	81.5	Lapilli Tuff	I	D	61	128	61	2.3	R1	Very Weak	6
SCB-028-09/C-16	82.1	Lapilli Tuff	I	D	61	85	61	27.3	R4	Strong	1
SCB-029-09/C-7	27.2	Lapilli Tuff	I-II	D	61	122	61	2.0	R1	Very Weak	6
SCB-029-09/PLT-1	30.2	Lapilli Tuff	I-II	D	61	41	61	2.8	R1	Very Weak	
SCB-029-09/PLT-2	33	Lapilli Tuff	I-II	D	61	68	61	2.7	R1	Very Weak	
SCB-029-09/C-9	34.2	Lapilli Tuff	II	D	61	118	61	10.5	R3	Mod Strong	1
SCB-029-09/C-10	38.8	Lapilli Tuff	I	D	61	128	61	22.3	R4	Strong	1
SCB-029-09/C-10	42.3	Lapilli Tuff	I	A	61	61	69	3.8	R2	Mod. Weak	6
SCB-029-09/C-10	42.5	Lapilli Tuff	I	D	61	160	61	19.6	R4	Strong	1
SCB-029-09/C-11	46.8	Lapilli Tuff	I	A	61	61	69	9.9	R3	Mod Strong	5
SCB-029-09/C-11	47	Lapilli Tuff	I	D	61	119	61	15.0	R3	Mod Strong	1
SCB-029-09/C-11	47.5	Lapilli Tuff	I	A	61	61	69	23.8	R4	Strong	5
SCB-029-09/C-12	52.5	Lapilli Tuff	I	D	61	156	61	19.7	R4	Strong	1
SCB-029-09/C-13	57.6	Lapilli Tuff	I	D	61	81	61	6.1	R2	Mod. Weak	1
SCB-029-09/C-14	62.6	Lapilli Tuff	I	D	61	82	61	18.2	R4	Strong	1
SCB-029-09/C-15	65.3	Lapilli Tuff	I	D	61	126	61	11.8	R3	Mod Strong	1
SCB-029-09/S-8	68	Lapilli Tuff	I	D	61	64	61	19.7	R4	Strong	
SCB-029-09/C-16	71.3	Lapilli Tuff	I	D	61	188	61	13.5	R3	Mod Strong	1

Table 7.4 (continued)
Rock Point Load Test Results for Slide Curve Bridge and Wall

Boring ID/Sample #	Depth (feet)	Rock Type	Weath. State [1]	Test Type [2]	Dia.[3] (mm)	Width [4] (mm)	De [5] (mm)	qu [6] (ksi)	ISRM [7] Class.	ISRM Description	Failure [8] Type
SCB-030-09/C-4	29.7	Lapilli Tuff	II	D	61	161	61	3.7	R1	Very Weak	1
SCB-030-09/PLT-1	31.4	Lapilli Tuff	II	D	61	49	61	23.7	R4	Strong	
SCB-030-09/PLT-2	31.9	Lapilli Tuff	II	D	61	34	61	11.4	R3	Mod. Strong	
SCB-030-09/C-5	33.4	Lapilli Tuff	II	D	61	121	61	6.3	R2	Mod. Weak	1
SCB-030-09/C-6	37.4	Lapilli Tuff	II	D	61	148	61	8.7	R3	Mod Strong	1
SCB-030-09/C-6	39.4	Lapilli Tuff	I	A	64	61	71	11.3	R3	Mod Strong	2
SCB-030-09/C-7	42	Lapilli Tuff	I	D	61	10	61	23.7	R4	Strong	1
SCB-030-09/C-7	45.6	Lapilli Tuff	II	D	61	102	61	20.4	R4	Strong	1
SCB-030-09/C-8	46.9	Lapilli Tuff	I	D	61	148	61	13.0	R3	Mod Strong	1
SCB-030-09/C-8	51.3	Lapilli Tuff	I	D	61	222	61	16.0	R3	Mod Strong	1
SCB-030-09/C-10	58.4	Lapilli Tuff	I	D	61	138	61	9.9	R3	Mod Strong	1
SCB-030-09/C-10	61.5	Lapilli Tuff	II	D	61	113	61	5.0	R2	Mod. Weak	1
SCB-030-09/C-11	64.7	Lapilli Tuff	I	D	71	152	61	12.9	R3	Mod Strong	1
SCB-030-09/C-12	67.4	Lapilli Tuff	II	D	61	119	61	0.7	R1	Very Weak	6
SCB-030-09/C-13	70.9	Lapilli Tuff	I	D	61	134	61	9.2	R3	Mod Strong	1
SCB-030-09/C-15	78.5	Lapilli Tuff	I	D	61	114	61	11.7	R3	Mod Strong	1
SCB-030-09/C-15	82.5	Lapilli Tuff	II	D	61	68	61	6.1	R2	Mod. Weak	1
SCB-030-09/C-16	87	Lapilli Tuff	I	D	61	108	61	12.8	R3	Mod Strong	1
SCB-031-10/C-2	9.8	Lapilli Tuff	I	D	60.0	80.0	60.00	15.0	R3	Mod. Strong	1
SCB-031-10/C-4	16.3	Lapilli Tuff	I	D	60.0	100.0	60.00	13.1	R3	Mod. Strong	1
SCB-031-10/C-4	17.3	Lapilli Tuff	I	D	60.0	100.0	60.00	35.9	R5	Very Strong	1
SCB-031-10/C-6	27.1	Lapilli Tuff	I	D	60.0	80.0	60.00	26.7	R4	Strong	2
SCB-031-10/C-6	29.3	Lapilli Tuff	I	D	60.0	200.0	60.00	29.2	R5	Very Strong	1
SCB-031-10/C-7	31.3	Lapilli Tuff	II	D	60.0	100.0	60.00	13.1	R3	Mod. Strong	4
SCB-032-10/C-6	26.8	Lapilli Tuff	II	D	60.0	70.0	60.00	30.0	R5	Very Strong	1
SCB-032-10/C-6	28.1	Lapilli Tuff	I-II	D	60.0	73.0	60.00	27.9	R4	Strong	1
SCB-032-10/C-7	32	Lapilli Tuff	I	D	60.0	135.0	60.00	20.3	R4	Strong	1
SCB-032-10/C-7	33.6	Lapilli Tuff	I	D	60.0	210.0	60.00	31.2	R5	Very Strong	1
SCB-032-10/C-9	43.8	Lapilli Tuff	I	D	60.0	138.0	60.00	31.7	R5	Very Strong	4
Earlier Tests											
H-25-06/C16	57.5	Andesite	I	D	45	70.1	45	19.7	R4	Strong	1
H-25-06/C16	59.1	Andesite	I	D	45	64	45	30.3	R5	Very Strong	1
H-25-06/C16	59.1	Andesite	I	A	76.2	45	66.1	16.5	R4	Strong	5

Table 7.4 (continued)
Rock Point Load Test Results for Slide Curve Bridge and Wall

Boring ID/Sample #	Depth (feet)	Rock Type	Weath. State [1]	Test Type [2]	Dia.[3] (mm)	Width [4] (mm)	De [5] (mm)	qu [6] (ksi)	ISRM [7] Class.	ISRM Description	Failure [8] Type
H-26-06/C9	25.9	Dacite Breccia	II	D	65	82.3	65	6	R2	Mod. Weak	1
H-26-06/C9	29.2	Dacite Breccia	II	D	65	94.5	65	10.7	R3	Mod. Strong	1
H-26-06/C10	33.9	Dacite Breccia	II	A	91.4	65	87	16	R4	Strong rock	2
H-26-06/C13	49.1	Dacite Breccia	II	D	65	79.2	65	13.6	R3	Mod. strong	1
H-26-06/C13	49.1	Dacite Breccia	II	A	76.2	65	79.4	10.1	R3	Mod. Strong	2
H-26-06/C13	49.4	Dacite Breccia	II	D	65	79.2	65	18.2	R4	Strong	1

Notes
1. Weathering State: I - Fresh; II - Slightly weathered; III - Moderately weathered; IV - Highly weathered; V - Completely weathered
2. Test Type: A = Axial; D = Diametral
3. Diameter: The loaded side initial dimension
4. Width: For axial and irregular specimens, second dimension value to calculate D_e
5. D_e - Equivalent diameter: for diametral specimens, same as diameter, otherwise it is the square root of 4*Diameter*Width/π
6. Unconfined Compression Strength, q_u: taken as 24.5*$I_{s(50)}$ in MPa for 65 mm core size, and 22* $I_{s(50)}$ in MPa for 45 mm core size based on Table 1 in ASTM D 5731-02
7. ISRM Classification: R0: 0.28 to 1.03 MPa (0.04 to 0.15 ksi); R1: 1.03 to 24.8 MPa (0.15 to 3.6 ksi); R2: 24.8 to 50.3 MPa (3.6 to 7.3 ksi); R3: 50.3 to 103.4 MPa (7.3 to 15 ksi); R4:103.4 to 200 MPa (15 to 29 ksi); R5: > 200 MPa (>29 ksi).
8. Failure Type: 1 - Diametral normal failure; 2 - Axial normal failure; 3 - Block normal failure; 4 - Diametral, chipped failure; 5 - Axial, sheared at angle

Table 7.5
Recommended Rock Properties for Rock Socket Capacity Design

Pier	Rock Depth, feet	Intact Rock UCS, psi	Rock Mass Rating (RMR)	Global Rock Mass Strength, ksf (Note 1)	Notes
2	0-35	18,000	44	404	
3	0-35	20,000	44	449	
4 (inner)	0-35	20,000	44	449	
4 (outer)	0-35	10,000	44	224	
5	0-35	15,000	44	337	
6	0-2	6,000	44	135	
	3-7	11,000	44	247	2
	8-11	21,000	44	471	
	12-15	28,000	44	629	
	16-35	5,000	44	112	
7	0-39	2,000	45	46	2
	40-45	500	45	11	
8	0-13	5,000	44	112	2
	14-45	10,000	44	224	
9	0-35	5,000	32	--	3
	0-35	9,000	44	202	4

Notes
1. Global Rock Mass Strength was used for DFSAP input.
2. Global Rock Mass Strength was assumed constant with depth for DFSAP input. For shafts with Global Rock Mass Strength values that vary with depth, the value on the noted line was used for DFSAP input.
3. Lower (conservative) intact rock UCS and RMR used for Pier 9 bearing resistance (RMR is geometric mean of fair and poor rock quality RMRs).
4. Higher (conservative) intact rock UCS and RMR used for Pier 9 Global Rock Mass Strength for DFSAP input.

Abbreviations
ksf = kips per square foot
psi = pounds per square inch
UCS = unconfined compressive strength

Table 7.6
Global Stability Analysis Results for Walls

LE Station	Structure	Calculated Factor of Safety at Base of Soil Nail Wall				Figure No.
		High Lake Level (El. 2517)		Low Lake Level (El. 2416)		
		Static	Seismic	Static	Seismic	
1379+90	Soil Nail Wall Without Tied Shaft Wall	1.49	**0.81**	1.46	**0.84**	H.2.1 to H.2.4
1381+50	Soil Nail Wall Without Tied Shaft Wall	---	**0.73**	---	**0.82**	H.2.5 to H.2.6
1371+75	West MSE Wall (<10 feet high)	**1.25**	---	1.36	---	H.2.7 to H.2.8
	West MSE Wall (Base of MSE)	**1.19**	---	**1.24**	---	H.2.9 to H.2.10
1384+00[1]	East MSE Wall (>10 feet high)	**1.25**	**0.75**	1.30	**0.85**	H.2.11 to H.2.14
	East MSE Wall (Base of MSE)	1.59	**1.01**	1.43	**0.95**	H.2.15 to H.2.18
1384+50[2]	East MSE Wall (<10 feet high)	**1.24**	---	1.31	---	H.2.19 to H.2.20
	East MSE Wall (Base of MSE)	**1.2**	---	**1.2**	---	H.2.21 to H.2.22

Bold indicates a calculated factor of safety of (1) less than 1.3 for static loading or 1.1 for seismic loading for walls >10 feet high or (2) less than 1.25 for static loading for walls <10 feet high.
[1] Includes Case A reinforcement as shown in Figure 7.32.
[2]Includes Case B reinforcement as shown in Figure 7.32.

Table 7.7
Deformation Analysis Results for East Approach MSE Wall

LE Station	Structure[3]	Yield Acceleration[1]		Estimated Displacement, feet[2]	Figures
		High Lake Level (El. 2517)	Low Lake Level (El. 2416)		
1384+00	East MSE Wall	0.08g	0.11g	~0.75 to 1.25	H.2.23-H.2.24
	East MSE Wall (Base of MSE)	0.175g	0.15g	~0.5	H.2.16 &H.2.25

Notes
[1]Horizontal acceleration that produces a calculated factor of safety of 1.0
[2]Nominal 50% non-exceedance level
[3]Includes Case A reinforcement as shown in Figure 7.32.

Table 7.8
Resistance Factors for Shallow Foundations

Limit State	Resistance Factor, φ		
	Bearing Resistance	Friction Resistance to Sliding	Passive Pressure Resistance to Sliding
Strength[1]	0.45	0.80	0.5
Service	1.0	N/A	N/A
Extreme Event[2]	0.9	0.9	0.9

Notes
[1] Resistance factors for the strength limit state are recommended in the LRFD Bridge Design Specifications, Table 10.5.5.2.2-1.
[2] Resistance factors for the extreme event limit state are recommended in the GDM, Section 8.10.2.

Table 7.9
Drilled Shaft Bearing and Uplift Resistance Factors

Limit State	Resistance Factor, φ		
	Skin Friction	Tip Resistance	Uplift
Strength	0.55	0.5	0.45 (soil) 0.4 (rock)
Service	1.0	1.0	1.0
Extreme Event	1.0	1.0	0.8

Table 7.10
Slide Curve Bridge Rock Socket Lateral Capacity

Rock Socket Depth, feet[1]	Bridge Piers								Tied-Shaft Wall					
									Lateral Capacity, kips[2,3,4]					
	Lateral Capacity, kips, 9-ft Diameter Rock Socket[2,3,4]								Shafts at and West of Pier 6			Shafts East of Pier 6		
	Pier								Shaft Diameter, feet					
	2	3&4I	4O	5	6	7	8	9	3	4	5	3	4	5
1	273	258	213	170	130	109	102	118	50	63	76	41	53	64
2	589	557	459	368	281	235	220	255	120	148	175	100	123	146
3	949	897	739	594	498	378	357	412	185	224	262	177	211	246
4	1,352	1,278	1,054	850	744	540	511	590	313	371	428	272	319	365
5	1,798	1,701	1,403	1,135	1,019	719	684	787	471	549	626	386	445	504
6	2,289	2,167	1,786	1,449	1,323	917	876	1,006	661	759	857	520	592	664
7	2,825	2,676	2,206	1,794	1,658	1,133	1,087	1,246	884	1,003	1,122	675	760	845
8	3,406	3,228	2,660	2,170	2,102	1,367	1,317	1,507	892	1,009	1,126	850	949	1,047
9	4,033	3,824	3,151	2,577	2,588	1,621	1,566	1,790	1,174	1,321	1,468	1,047	1,159	1,271
10	4,705	4,464	3,678	3,015	3,118	1,893	1,836	2,095	1,503	1,680	1,857	1,267	1,393	1,518
11	5,424	5,148	4,241	3,485	3,691	2,185	2,126	2,423	1,880	2,088	2,296	1,509	1,649	1,789
12	6,190	5,878	4,841	3,988	4,360	2,496	2,436	2,773	2,081	2,302	2,524	1,775	1,929	2,084
13	7,003	6,653	5,479	4,524	5,081	2,827	2,768	3,146	2,546	2,805	3,063	2,064	2,233	2,403
14	7,863	7,473	6,154	5,093	5,855	3,177	3,168	3,542	3,070	3,366	3,662	2,378	2,562	2,747
15	8,771	8,340	6,868	5,695	6,683	3,548	3,597	3,962	3,653	3,987	4,322	2,717	2,917	3,117
16	9,728	9,254	7,619	6,332	7,368	3,939	4,055	4,406	10,322	11,169	12,017	3,082	3,297	3,512
17	10,733	10,214	8,409	7,003	8,078	4,350	4,542	4,874	11,737	12,612	13,488	3,473	3,704	3,935
18	11,788	11,223	9,238	7,710	8,812	4,782	5,060	5,366	13,209	14,112	15,015	3,890	4,138	4,385
19	12,892	12,279	10,106	8,451	9,572	5,235	5,608	5,884	14,737	15,668	16,600	4,335	4,599	4,863
20	14,045	13,383	11,014	9,229	10,357	5,709	6,187	6,426	16,323	17,283	18,243	4,808	5,088	5,369
21	15,249	14,536	11,962	10,042	11,168	6,203	6,797	6,994	17,966	18,955	19,944	5,309	5,606	5,903
22	16,504	15,738	12,950	10,892	12,004	6,720	7,438	7,587	19,668	20,686	21,705	5,838	6,153	6,468
23	17,810	16,990	13,978	11,779	12,867	7,258	8,112	8,206	21,429	22,477	23,525	6,397	6,729	7,061
24	19,167	18,291	15,048	12,704	13,756	7,817	8,817	8,851	23,249	24,328	25,406	6,986	7,336	7,686
25	20,575	19,643	16,158	13,666	14,672	8,399	9,555	9,523	25,130	26,239	27,347	7,605	7,973	8,341
26	22,036	21,046	17,310	14,667	15,615	9,002	10,326	10,221	27,072	28,211	29,350	8,255	8,641	9,027
27	23,550	22,500	18,504	15,706	16,585	9,628	11,130	10,947	29,075	30,245	31,415	8,937	9,341	9,746
28	25,116	24,005	19,740	16,784	17,583	10,277	11,968	11,700	31,140	32,341	33,543	9,650	10,073	10,496
29	26,736	25,562	21,019	17,902	18,608	10,948	12,839	12,480	33,267	34,500	35,733	10,395	10,837	11,280
30	28,409	27,172	22,340	19,059	19,662	11,642	13,745	13,288	35,457	36,723	37,988	11,173	11,635	12,096

Table 7.10 (continued)
Slide Curve Bridge Rock Socket Lateral Capacity

Rock Socket Depth, feet[1]	Bridge Piers								Tied-Shaft Wall					
									Lateral Capacity, kips[2,3,4]					
	Lateral Capacity, kips, 9-ft Diameter Rock Socket[2,3,4]								Shafts at and West of Pier 6			Shafts East of Pier 6		
	Pier								Shaft Diameter, feet					
	2	3&4I	4O	5	6	7	8	9	3	4	5	3	4	5
31	30,136	28,834	23,704	20,257	20,744	12,359	14,686	14,125	37,711	39,009	40,306	11,985	12,466	12,947
32	31,918	30,549	25,112	21,495	21,855	13,099	15,661	14,989	40,030	41,360	42,690	12,830	13,331	13,831
33	33,755	32,318	26,563	22,773	22,995	13,862	16,672	15,883	42,412	43,776	45,139	13,709	14,230	14,750
34	35,646	34,140	28,059	24,094	24,164	14,650	17,719	16,805	44,861	46,257	47,653	14,623	15,164	15,705
35	37,593	36,017	29,598	25,456	25,362	15,460	18,801	17,757	47,375	48,805	50,235	15,572	16,134	16,695
36	39,596	37,949	31,183	26,860	26,590	16,295	19,920	18,738	49,955	51,419	52,883	16,557	17,139	17,721
37	41,655	39,935	32,812	28,306	27,849	17,154	21,076	19,749	52,602	54,100	55,598	17,578	18,180	18,783
38	43,771	41,977	34,487	29,796	29,137	18,037	22,269	20,789	55,317	56,850	58,382	18,635	19,259	19,883
39	45,943	44,075	36,207	31,328	30,456	18,945	23,499	21,860	58,100	59,667	61,235	19,729	20,374	21,019
40	48,173	46,229	37,973	32,905	31,806	19,810	24,767	22,962	60,951	62,554	64,156	48,553	50,079	51,605
41	50,461	48,439	39,785	34,525	33,186	20,689	26,073	24,094	63,871	65,509	67,147	51,106	52,653	54,200
42	52,806	50,707	41,643	36,190	34,598	21,583	27,417	25,257	66,861	68,535	70,209	53,703	55,270	56,838
43	55,210	53,031	43,548	37,899	36,042	22,491	28,800	26,452	69,921	71,631	73,341	56,342	57,931	59,519
44	57,673	55,414	45,501	39,653	37,517	23,414	30,221	27,678	73,052	74,798	76,545	59,025	60,634	62,243
45	60,194	57,854	47,500	41,453	39,025	24,352	31,683	28,936	76,254	78,037	79,820	61,750	63,380	65,010

Notes
[1]Measured below downslope rock face
[2]Lateral Capacity Pp Based on AASHTO Figure 3.11.5.6-2, p.3-79, Modified for Depth (D)-variable Sm, Sm(D), making Pp(D).
[3]Rock shear strength Sm based on 2002 Hoek-Brown failure criteria with Sm = τ (tau); $\tau = c' + \sigma N * \tan \Phi'$. 2002 Hoek-Brown tau is generally somewhat higher than tau by 1983-1997 H-B at AASHTO 10.4.6.4-1. For confining stresses expected at SCB and higher intact UCS, 2002 Hoek-Brown tau is less than the H-B rock mass UCS and very much less that the Global RMS. For lower intact UCS (Pier 7), tau can exceed the rock mass UCS, but not the Global RMS.
[4]Hoek-Brown failure criteria from Liang, Yang, and Nusairat "p-y Criterion for Rock Mass," Journal of Geotechnical and Geoenvironmental Engineering, Vol. 135, No. 1, ASCE January 2009 (Liang). Recommended Pp(D) are generally less than the ultimate shaft resistance based on Liang Eq. 13 for deep rock mass failure, which is less than by Liang Eq. 2 for shallow rock mass failure.

Abbreviations
ft=feet
I=inner shaft
O=outer shaft
UCS= unconfined compressive strength

Table 7.11
Design Parameters for Lateral Load Analysis of Bridge Piers Using DFSAP

Soil Layer No.	Top of Layer Elevation (feet)	Soil Type	Soil Layer Thickness (feet)	Soil Effective Unit Weight (pcf)	Soil Effective Friction Angle (degree)	Soil Strain ε_{50} (unitless)	Soil Cohesion (psf)	Rock Unconfined Compressive Strength (psf)	Soil Liquefaction Potential
1	See Figure 7.24	Soil[1]	See Figure 7.23[3]	73	40[2]	0.005	0	N/A	No
2	See Figure 7.24	Bedrock	>300	88	N/A	0.004	0	See Table 7.11	No

[1] Gravel with sand/cobbles/boulders & gravel with sand were combined as one unit
[2] Effective friction angle (drained condition)
[3]Soil layer thickness is equal to the work bench elevation minus the bedrock elevation, as shown in Figure 7.24.

Table 7.12
Global Rock Mass Strength and Ground Slope for Lateral Load Analysis of Bridge Piers Using DFSAP

Pier (LE Sta.)	Shaft	Ground Slope, degrees[1]	Global Rock Mass Strength, psf[2]
2 (1373+24)	Inner	0	404,000
	Outer	38	
3 (1374+43.5)	Inner	0	449,000
	Outer	30	
4 (1376+00)	Inner	0	449,000
	Outer	40	224,000
5 (1377+60)	Inner	0	337,000
	Outer	32	
6 (1379+20)	Inner	0	247,000
	Outer	40	
7 (1380+80)	Inner	0	46,000
	Outer	36	
8 (1382+36.5)	Inner	0	112,000
	Outer	38	
9 (1383+58)	Inner	0	202,000
	Outer	35	

[1]Measured from horizontal
[2]Rock unconfined compressive strength (UCS) based on Hoek et al. 2002 (Equations 16 and 17 p.27 NCHRP Synthesis 360 Rock-Socketed Shafts for Highway Structure Foundations, TRB 2006)

Table 7.13
Estimated Anchor Head to Bedrock Distances for Wall 23 Ground Anchors

Shaft ID	Estimated Anchor Head to Bedrock Distance[1]	Shaft ID	Estimated Anchor Head to Bedrock Distance[1]
S1	10	S19	55
S2	15	S20	55
S3	22	S21	55
S4	25	S22	54
S5	26	S23	51
S6	27	S24	48
S7	28	S25	43
S8	30	S26	38
S9	30	S27	30
S10	34	S28	24
S11	34	S29	21
S12	38	S30	20
S13	44	S31	19
S14	46	S32	18
S15	48	S33	17
S16	51	S34	16
S17	53	S35	14
S18	54	S36	13

Notes
[1]For the purpose of estimating tendon lengths, a minimum of 10 feet should be added to the estimated anchor head to bedrock distance to account for uncertainty in the depth to bedrock.

Table 8.1
Slide Curve Median Wall Borings

Boring ID	Date Drilled		Elevation (feet)[1]	Project Datum[2]		Total Depth (feet)
	Started	Completed		Northing (feet)	Easting (feet)	
2008 Borings						
SCW-001-08	5/19/08	5/20/08	2533.8	1,064,096	1,754,560	19.0
SCW-002-08	5/6/08	5/7/08	2532.5	1,063,827	1,754,515	38.7
SCW-002A-08	5/28/08	5/28/08	2532.1	1,063,824	1,754,536	13.8
SCW-003-08	5/7/08	5/8/08	2531.7	1,063,588	1,754,474	38.7
SCW-004-08	5/9/08	5/9/08	2531.5	1,063,297	1,754,423	28.7
SCW-004A-08	5/28/08	5/28/08	2531.0	1,063,293	1,754,444	16.0
SCW-005-08	5/6/08	5/7/08	2531.4	1,063,012	1,754,374	39.0
SCW-006-08	5/7/08	5/8/08	2532.6	1,062,768	1,754,363	35.0
SCW-006A-08	5/27/08	5/28/08	2530.4	1,062,768	1,754,386	16.6
SCW-007-08	5/8/08	5/9/08	2533.6	1,062,599	1,754,378	23.2
SCW-008-08	5/9/08	5/12/08	2534.6	1,062,419	1,754,417	24.0
SCW-008A-08	5/27/08	5/27/08	2532.3	1,062,426	1,754,439	18.8
SCW-009-08	5/12/08	5/13/08	2535.6	1,062,238	1,754,481	44.0
SCW-010-08	5/14/08	5/14/08	2533.7	1,061,658	1,754,967	28.7
SCW-011-08	5/14/08	5/14/08	2530.9	1,061,473	1,755,280	18.7
SCW-012-08	5/14/08	5/14/08	2527.2	1,061,381	1,755,587	18.7
SCW-013-08	5/9/08	5/13/08	2525.6	1,061,327	1,756,050	70.2
Previous Borings						
CUL-007-07	8/29/2007	8/29/2007	2523.6	1,061,374	1,756,107	40.5
CUL-008-07	7/12/2007	7/13/2007	2513.0	1,061,217	1,756,090	51
SW2-001-07	8/30/2007	9/4/2007	2532.4	1,061,993	1,754,666	26
SW2-002-07 (OW)	7/26/2007	7/31/2007	2541.5	1,061,928	1,754,589	55.5
SW2-003-07	9/5/2007	9/5/2007	2532.6	1,061,883	1,754,749	20
SW2-004-07	9/7/2007	9/10/2007	2543.5	1,061,749	1,754,738	51.5
SW2-005-07	9/6/2007	9/6/2007	2532.7	1,061,796	1,754,829	25
SW1-006-07	6/27/2007	6/27/2007	2531.9	1,063,907	1,754,471	60
SW1-007-07	9/19/2007	9/19/2007	2530.5	1,063,604	1,754,419	60

Table 8.1 (continued)
Slide Curve Median Wall Borings

Boring ID	Date Drilled		Elevation (feet)[1]	Project Datum[2]		Total Depth (feet)
	Started	Completed		Northing (feet)	Easting (feet)	
H-1-97	9/29/1997	9/29/1997	2541.6	1,061,827	1,754,671	56.5
H-16-98	11/15/1998	11/17/1998	2445.7	1,062,177	1,754,158	53
H-1-05	4/20/2005	5/5/2005	2503.4	1,061,761	1,754,626	115
H-2-05	4/18/2005	4/19/2005	2510.7	1,064,028	1,754,456	63
H-26-06	8/7/2006	8/7/2006	2532.7	1,064,088	1,754,502	50.1
H-27-06 (I)	7/18/2006	7/19/2006	2537.4	1,062,766	1,754,307	80.5
H-27-06 (E)	8/4/2006	8/6/2006	2537.2	1,062,777	1,754,306	89.6
H-28-06	8/1/2006	8/2/2006	2462.9	1,062,791	1,754,202	74.2
H-29-06 (I)	6/20/2006	6/22/2006	2540.5	1,062,218	1,754,429	90
H-29-06 (E)	6/27/2006	6/28/2006	2540.7	1,062,209	1,754,433	69.3
H-30-06	7/25/2006	7/26/2006	2475.3	1,062,173	1,754,337	50.4
H-32-06	6/30/2006	7/6/2006	2541.9	1,061,583	1,754,924	50
H-33-06	7/11/2006	7/11/2006	2494.1	1,061,472	1,754,930	26.5
H-34-06	7/7/2006	7/7/2006	2533.6	1,061,278	1,755,667	31.5
H-35-06	7/11/2006	7/11/2006	2497.7	1,061,199	1,755,670	15.5

Notes
Vertical datum is the North American Vertical Datum (NAVD) 1988
Horizontal datum is based on the Washington Coordinate System, South Zone, and North American Datum (NAD) 1983/91

Table 8.2
Slide Curve Median Wall Laboratory Test Results

Boring No. & Sample Information				Water Content (%)	Grain Size Analysis		
Boring No.	Sample No.	Sample Depth (feet)	USCS/WSDOT Classification		Gravel (%)	Sand (%)	Silt/Clay (%)
2008 Testing							
SCW-001-08	D-2	5.0-5.3	GM	22.8	86.0		14.0
SCW-002-08	D-1	6.7-8.2	SM	13.3	87.9		12.1
	D-3	8.8-10.3	SM	8.6	86.0		14.0
	D-5	13.7-15.1	GW	4.5	98.9		1.1
SCW-003-08	D-4	13.7-14.2	SM	30.1	79.3		20.7
	D-7	18.7-20.1	GM	11.1	86.3		13.7
	D-11	23.7-24.6	SM	9.3	87.2		12.8
SCW-004-08	D-1	6.0-7.5	GM	11.8	89.7		10.3
	D-3	8.7-9.5	GM	7.3	89.7		10.3
SCW-004A-08	C-2	2.5	GM	21.7	86.4		13.6
	D-3	5.0	GM	25.4	87.4		12.6
SCW-006-08	D-3	14.0	GP	12.0	90.3		9.7
	D-4	19.0-19.8	GM	18.3	85.1		14.9
	D-5	24.0-25.5	GM	16.5	82.8		16.9
SCW-006A-08	D-2	2.5	GM	15.7	88.9		11.1
	D-4	5.0	GM	13.3	81.2		18.8
SCW-007-08	D-1	4.5-5.2	GP	13.7	95.5		4.5
SCW-008-08	D-1	5.0-6.5	GM	9.1	85.4		14.6
SCW-008A-08	D-1	2.5	SM	12.9	88.9		11.1
	D-3	5.0	SP	12.5	91.7		9.3
SCW-009-08	C-1	2.5-4.0	GM	18.7	88.1		11.9
	D-4	9.0-10.5	GM	19.0	89.6		10.4
SCW-012-08	D-1	2.0-2.7	GM	15.0	84.3		15.7
SCW-013-08	D-1	6.0-7.5	GM	14.5	84.0		16.0
	D-3	8.7-10.2	GM	17.6	80.3		19.7
	D-6	13.7-15.2	GM	16.4	84.5		15.5
	D-8	18.7-20.2	GM	15.0	89.8		10.2
	D-24	53.7-55.2	SM	28.8	86.8		13.2
	D-26	58.7-60.2	SP	31.6	92.3		7.7
	C-27	60.2-60.7	SM	29.6	89.9		10.1
	D-29	63.7-65.2	SM	22.9	80.8		19.2
	D-31	68.7-70.2	SM	27.0	85.2		14.8
Previous Testing							
SW-002-07(OW)	D-2	5.5-7.0	GP	2.8	72.8	23.4	3.8
	D-3	8.0-9.5	GP	2.8	72.8	23.4	3.8
	D-4	10.5-12.0	GP	2.4	79.7	17.7	2.6
	D-5	13.0-14.5	GP	2.4	79.7	17.7	2.6
	D-8	18.0-19.5	GP	2.7	79.5	17.8	2.8
	D-10	20.5-21.5	GP	3.0	65.4	27.6	7.0
	D-12	25.0-26.5	GM	8.2	51.5	36.5	12.0
	D-13	30.0-31.5	GM-SM	10.7	44.4	44.4	11.2

Notes
1. Abbreviations: USCS: Unified Soil Classification System.
2. Material description per WSDOT soil and rock classification system.

Table 8.3
Rock Point Load Test Results for Slide Curve Median Wall

Boring ID/ Sample #	Depth (feet)	Rock Type	Weath. State[1]	Test Type[2]	Dia. (mm)[3]	Width (mm)[4]	De (mm)[5]	Unconfined Compressive Strength, q_u, (ksi)[6]	ISRM Classification[7]	ISRM Description
2008 Tests										
SCW-001-08/C-2	5.7-6.3	Lapilli Tuff	II	D	60.9	182.9	60.90	9.5	R3	Moderately Strong
SCW-001-08/C-4	12.0-12.8	Lapilli Tuff	I	D	60.9	243.8	60.90	8.7	R3	Moderately Strong
SCW-002-08/C-8	21.9-22.4	Lapilli Tuff	III	D	63.5	85.0	63.50	11.4	R3	Moderately strong
SCW-002-08/C-9	24.4-25.1	Lapilli Tuff	I	D	63.5	108.0	63.50	21.9	R4	Strong
SCW-002-08/C-9	25.1-25.4	Lapilli Tuff	I	A	98.0	63.5	89.01	16.0	R4	Strong
SCW-002-08/C-9	26.7-27.1	Lapilli Tuff	II	D	63.5	63.0	63.50	19.4	R4	Strong
SCW-002-08/C-10	32.9-33.2	Lapilli Tuff	I	A	108.0	63.5	93.44	11.9	R3	Moderately strong
SCW-002-08/C-10	33.2-33.6	Lapilli Tuff	I	D	63.5	69.0	63.50	18.4	R4	Strong
SCW-002A-08/C-5	11.5-12.3	Lapilli Tuff	III	D	60.7	243.8	60.70	28.3	R4	Strong
SCW-003-08/C-12	26.3-26.7	Lapilli Tuff	II	D	61.0	59.0	61.00	9.9	R3	Moderately strong
SCW-003-08/C-13	30.7-31	Lapilli Tuff	I	A	68.0	61.0	72.67	17.3	R4	Strong
SCW-003-08/C-13	31-31.6	Lapilli Tuff	I	D	61.0	84.0	61.00	27.9	R4	Strong
SCW-004-08/C-5	10.5-10.8	Lapilli Tuff	II	D	61.0	39.0	61.00	6.4	R2	Moderately weak
SCW-004-08/C-7	18.7-19.3	Lapilli Tuff	I	D	61.0	90.0	61.00	12.1	R3	Moderately strong
SCW-004-08/C-7	19.4-19.7	Lapilli Tuff	II	D	61.0	50.0	61.00	2.1	R1	Very weak
SCW-004-08/C-8	23.7-24.2	Lapilli Tuff	II	D	61.0	64.0	61.00	13.2	R3	Moderately strong
SCW-004A-08/C-4	7.8-8.2	Lapilli Tuff	II	D	60.5	121.9	60.50	20.2	R4	Strong
SCW-005-08/C-4	17.5-17.8	Lapilli Tuff	I	D	63.5	60.0	63.50	37.5	R5	Very strong
SCW-005-08/C-6	28.5-28.9	Lapilli Tuff	III	D	63.5	152.4	63.50	38.5	R5	Very strong
SCW-005-08/C-7	33.6-34.0	Lapilli Tuff	II	D	63.5	91.4	63.50	39.4	R5	Very strong
SCW-005-08/C-8	38.4-39.0	Lapilli Tuff	I	D	63.5	182.9	63.50	40.4	R5	Very strong
SCW-006-08/C-1	23.1-23.5	Lapilli Tuff	III	D	63.5	121.9	63.50	0.0	R0	Extremely Weak
SCW-006-08/C-2	28.5-29.0	Lapilli Tuff	II	D	63.5	152.4	63.50	9.4	R3	Moderately Strong
SCW-006-08/C-4	34.5-35.0	Lapilli Tuff	I	D	63.5	152.4	63.50	13.0	R3	Moderately Strong
SCW-006A-08/C-5	7.4-7.9	Lapilli Tuff	IV	D	60.8	152.4	60.80	3.3	R1	Weak
SCW-006A-08/C-6	11.9-12.6	Lapilli Tuff	III	D	60.7	213.4	60.70	7.1	R2	Moderately Weak
SCW-006A-08/C-7	15.7-16.6	Lapilli Tuff	IV	D	59.5	274.3	59.50	16.0	R4	Strong
SCW-007-08/C-1	7.8-8.1	Lapilli Tuff	I	D	63.2	104.7	63.20	33.1	R5	Very Strong
SCW-007-08/C-5	17.9-18.7	Lapilli Tuff	I	D	61.0	243.8	61.00	27.5	R4	Strong
SCW-008-08/C-2	8.5-9.0	Lapilli Tuff	III	D	60.7	165.1	60.70	11.5	R3	Moderately Strong

Table 8.3 (continued)
Rock Point Load Test Results for Slide Curve Median Wall

Boring ID/ Sample #	Depth (feet)	Rock Type	Weath. State[1]	Test Type[2]	Dia. (mm)[3]	Width (mm)[4]	De (mm)[5]	Unconfined Compressive Strength, q_u, (ksi)[6]	ISRM Classification[7]	ISRM Description
SCW-008-08/C-4	15.0-15.8	Lapilli Tuff	II	D	60.9	254.0	60.90	11.0	R3	Moderately Strong
SCW-008A-08/C-5	13.0-13.4	Lapilli Tuff	III	D	60.6	121.9	60.60	23.5	R4	Strong
SCW-008A-08/C-6	18.5-18.8	Lapilli Tuff	II	D	60.6	91.4	60.60	40.0	R5	Very Strong
SCW-009-08/C-7	19.2-20.0	Lapilli Tuff	I	D	60.8	243.8	60.80	27.0	R4	Strong
SCW-009-08/C-9	28.0-29.0	Lapilli Tuff	II	D	60.8	304.8	60.80	12.2	R3	Moderately Strong
SCW-009-08/C-13	40.2-40.7	Lapilli Tuff	II	D	60.8	152.4	60.80	29.7	R5	Strong
SCW-010-08/C-2	6.5-6.95	Lapilli Tuff	III	D	60.9	137.2	60.90	11.9	R3	Moderately Strong
SCW-010-08/C-6	22.9-23.7	Lapilli Tuff	I	D	60.9	243.8	60.90	26.7	R4	Strong
SCW-011-08/C-6	9.4-10	Lapilli Tuff	I	D	61.0	95.0	61.00	33.7	R5	Very strong
SCW-011-08/C-9	17.6-18	Lapilli Tuff	I	D	61.0	70.0	61.00	26.4	R4	Strong
Previous Tests										
H-26-06/C9	25.9	Dacite Breccia	II	D	65.0	82.3	65.00	6.0	R2	Moderately weak
H-26-06/C9	29.2	Dacite Breccia	II	D	65.0	94.5	65.00	10.7	R3	Moderately strong
H-26-06/C10	33.9-34.2	Dacite Breccia	II	A	91.4	65.0	86.99	16.0	R4	Strong
H-26-06/C13	49.1	Dacite Breccia	II	D	65.0	79.2	65.00	13.6	R3	Moderately strong
H-26-06/C13	49.1-49.35	Dacite Breccia	II	A	76.2	65.0	79.41	10.1	R3	Moderately strong
H-26-06/C13	49.35	Dacite Breccia	II	D	65.0	79.2	65.00	18.2	R4	Strong
							Average:	12.4	R3	Moderately strong

Notes
1. Weathering State: I - Fresh; II - Slightly weathered; III - Moderately weathered; IV - Highly weathered; V - Completely weathered
2. Test Type: A – Axial; D - Diametral
3. Diameter: The loaded side initial dimension
4. Width: For Axial and Irregular Specimens, second dimension value to calculate De
5. D_e - Equivalent Diameter: for Diametral specimens, same as Diameter, otherwise it is the square root of 4*Diameter*Width/π
6. Unconfined Compression Strength. q_u: taken as 24.5*$I_{s(50)}$ in MPa for 65 mm core size, and 22* $I_{s(50)}$ in MPa for 45 mm core size based on Table 1 in ASTM D 5731-02
7. ISRM Classification: R0: 0.28 to 1.03 MPa (0.04 to 0.15 ksi); R1: 1.03 to 24.8 MPa (0.15 to 3.6 ksi); R2: 24.8 to 50.3 MPa (3.6 to 7.3 ksi); R3: 50.3 to 103.4 MPa (7.3 to 15 ksi); R4:103.4 to 200 MPa (15 to 29 ksi); R5: > 200 MPa (>29 ksi).

Table 8.4
Estimated Bedrock Elevations at Slide Curve Median Wall Face

LW Station	Offset Dec 2009 Wall Face from Jan 2008 Wall Face (4)	Assumed Bedrock Slope, V:H	Rock Elevation Difference	Jan 2008 Wall Face Rock Elevation	NGA (2009) Rock Elevation at Seismic Line	Offset NGA Seismic Line from Dec 2009 Wall Face (4)	Estimated Wall Face (Dec 2009) Rock Elevation	Offset Mar 2010 Wall Face from Dec 2009 Wall Face (4)	Offset Mar 2010 Wall Face from NGA line (4)	Estimated Wall Face (Mar 2010) Rock Elevation (1,2,3)
1385+00	-9.0	0.5	-4.5	2523	NA	NA	2518.5	NA	NA	2518.5
1386+00	-8.1	0.5	-4.1	2528.5	NA	NA	2524.4	NA	NA	2524.4
1387+00	-6.8	0.5	-3.4	2527.5	NA	NA	2524.1	NA	NA	2524.1
1388+00	-5.0	0.5	-2.5	2526.5	NA	NA	2524.0	NA	NA	2524.0
1389+00	-2.8	0.5	-1.4	2524.5	NA	NA	2523.1	NA	NA	2523.1
1390+00	-1.0	0.5	-0.5	2522	NA	NA	2521.5	NA	NA	2521.5
1391+00	-1	0.5	-0.5	2520.5	NA	NA	2520.0	NA	NA	2520.0
1392+00	-1	0.5	-0.5	2522.5	NA	NA	2522.0	NA	NA	2522.0
1393+00	-1	0.5	-0.5	2524.5	NA	NA	2524.0	NA	NA	2524.0
1394+00	-1	0.5	-0.5	2525.5	NA	NA	2525.0	NA	NA	2525.0
1395+00	-1	0.5	-0.5	2525.5	NA	NA	2525.0	0.0	NA	2525.0
1396+00	-0.46	0.5	-0.2	2525.5	NA	NA	2525.3	0.4	NA	2525.4
1397+00	2.31	0.33	0.8	2525.5	NA	NA	2526.3	1.1	NA	2526.6
1398+00	4.69	0.33	1.5	2525.5	NA	NA	2527.0	1.8	NA	2527.6
1399+00	6.68	0.33	2.2	2526	NA	NA	2528.2	2.4	NA	2528.0
1400+00	8.24	0.33	2.7	2525.5	NA	NA	2528.2	3.0	NA	2528.0
1401+00	9.39	0.33	3.1	2525.5	NA	NA	2528.6	3.5	NA	2528.0
1402+00	10.11	0.33	3.3	2525.5	NA	NA	2528.8	4.0	NA	2528.0
1403+00	10.39	0.5	Not used	Not used	2524.4	1.0	2523.9	4.4	3.4	2525.5
1403+50	10.37	0.5	Not used	Not used	2519.4	1.6	2518.6	4.5	2.9	2520.4
1404+00	10.25	0.5	Not used	Not used	2516.1	2.3	2515.0	4.7	2.4	2516.9
1404+50	10.01	0.5	Not used	Not used	2514.7	2.9	2513.2	4.9	2.0	2515.4
1405+00	9.67	0.5	Not used	Not used	2515.1	3.5	2513.3	5.0	1.5	2515.6
1405+50	9.21	0.5	Not used	Not used	2514.6	4.2	2512.5	5.1	0.9	2514.9
1406+00	8.66	0.5	Not used	Not used	2515.7	4.8	2513.3	5.2	0.4	2515.8
1406+50	7.99	0.5	Not used	Not used	2517.0	5.5	2514.3	5.3	-0.2	2517.1
1407+00	7.22	0.5	Not used	Not used	2522.3	6.1	2519.3	5.4	-0.7	2522.6
1407+50	6.35	0.5	Not used	Not used	2528.4	6.7	2525.0	5.4	-1.3	2529.1
1408+00	5.37	0.5	Not used	Not used	2530.7	7.4	2527.0	5.4	-2.0	2530.0
1408+50	4.29	0.5	Not used	Not used	2531.8	8.0	2527.8	5.4	-2.6	2530.0

Table 8.4 (continued)
Estimated Bedrock Elevations at SCW Wall Face

LW Station	Offset Dec 2009 Wall Face from Jan 2008 Wall Face (4)	Assumed Bedrock Slope, V:H	Rock Elevation Difference	Jan 2008 Wall Face Rock Elevation	NGA (2009) Rock Elevation at Seismic Line	Offset NGA Seismic Line from Dec 2009 Wall Face (4)	Estimated Wall Face (Dec 2009) Rock Elevation	Offset Mar 2010 Wall Face from Dec 2009 Wall Face (4)	Offset Mar 2010 Wall Face from NGA line (4)	Estimated Wall Face (Mar 2010) Rock Elevation (1,2,3)
1409+00	3.11	0.33	1.0	2528	NA	NA	2529.0	5.4	NA	2530.0
1410+00	0.46	0.33	0.2	2530	NA	NA	2530.0	5.4	NA	2530.0
1411+00	-2.58	0.5	-1.3	2532.5	NA	NA	2530.0	5.3	NA	2530.0
1412+00	-5.98	0	0.0	2534	NA	NA	2530.0	5.1	NA	2530.0
1413+00	-9.73	0	0.0	2528	NA	NA	2528.0	4.8	NA	2528.0
1414+00	-13.81	0	0.0	2526.5	NA	NA	2526.5	4.5	NA	2526.5
1415+00	-18.2	0	0.0	2525	NA	NA	2525.0	4.2	NA	2525.0
1416+00	-22.87	0	0.0	2525	NA	NA	2525.0	3.7	NA	2525.0
1417+00	-27.81	0	0.0	2525	NA	NA	2525.0	3.2	NA	2525.0
1418+00	-32.99	0	0.0	2524.5	NA	NA	2524.5	2.7	NA	2524.5
1419+00	-38.39	0	0.0	2525	NA	NA	2525.0	2.1	NA	2525.0
1420+00	-41.16	0	0.0	2496	NA	NA	2496.0	1.4	NA	2496.0
1421+00	-47	NA	NA	Not used	NA	NA	2473.9	0.7	NA	2473.9
1422+00	-43.58	NA	NA	<2450	NA	NA	<2450	0.05	NA	<2450
1423+00	-37.74	NA	NA	<2450	NA	NA	<2450	-0.7	NA	<2450
1424+00	-31.89	NA	NA	<2450	NA	NA	<2450	NA	NA	<2450

Notes.
1. Highest interpreted bedrock elevation is El. 2528 west of LW 1407+00 and El. 2530 east of LW 1407+00
2. Bedrock elevations were extrapolated from SCW wall face profile (January 2008) from LW 1385+00 to 1402+00 and 1409+00 to 1420+00 and from NGA (2009) estimated bedrock elevations from LW 1403+00 to 1408+50.
3. Bedrock elevations were extrapolated assuming bedrock slopes of 2H:1V and 3H:1V downslope and upslope, respectively, between LW 1385+00 and 1411+00. Between LW 1411+00 and 1420+00, bedrock is interpreted to be flatlying.
4. Positive value indicates upslope.
5. This information was provided to WSDOT on March 25, 2010

Abbreviations
NA = Not applicable
V:H = vertical:horizontal
NGA = Northwest Geophysical Associates

Table 8.5
Global Stability Analysis Results for Slide Curve Median Wall

LW Sta.	Loading Condition	Calculated Factor of Safety				Figure No.
		High Lake Level (El. 2517)		Low Lake Level (El. 2416)		
		Base of MSE	EB Outer Pavement Edge	Base of MSE	EB Outer Pavement Edge	
1399+00	Static	**1.15**	---	**1.08**	---	H.4.1 – H.4.2
	Seismic	**0.61**	---	**0.70**	---	H.4.3 – H4.4
1404+17	Seismic	**0.78 – 0.88**	---	**1.0**	---	H.4.5 – H.4.8
1407+15 (EB 1406+07)	Static	**1.35**	**1.18**	**1.33**	**1.11**	H.4.9 – H.4.12
	Seismic	**0.66**	---	**0.81**	---	H.4.13 – H.4.14
Global Stability for Potential Failures Within Rock						
LW Sta.	**Loading Condition**	**With Downslope Soil Resistance**		**Without Downslope Soil Resistance**		**Figure No.**
		Structural (Block) Failure	**Material (Circular) Failure**	**Structural (Block) Failure**	**Material (Circular) Failure**	
1398+20[1]	Static	2.32 – 2.35	2.67 – 2.77	2.04 – 2.12	2.26 – 2.40	See Appendix H.5
	Seismic	1.47 – 1.50	1.69 – 1.78	1.31 – 1.40	1.44 – 1.56	

Notes
Bold indicates a factor of safety less than 1.5 for static conditions or less than 1.1 for seismic conditions.
For seismic loading, a pseudostatic analysis was used, assuming a horizontal acceleration coefficient equal to 0.175 and full static (drained) shear strength for lake sediments.
[1] The ranges of factors of safety represent without rock dowels (low end of range) and with rock dowels (high end of range)

Table 8.6
Recommended Wall Foundation Types

Station (LW)	Estimated Wall Face (March 2010) Rock Elevation	Recommended Foundation Type	Comments
1377+00	Not estimated	Soil	Global stability is provided by the permanent Slide Curve Bridge (SCB) soil nail wall.
1378+00	Not estimated		
1379+00	Not estimated		
1380+00	Not estimated		
1381+00	Not estimated		
1382+00	Not estimated		
1383+00	Not estimated		
1384+00	Not estimated		
1385+00	2518.5	Soil	Estimated wall displacement approximately one foot or less
1386+00	2524.4	Soil or Rock	The embankment is not stable under seismic loading; however, the wall is mostly or entirely on shallow, nearly flat-lying rock. In some areas, rock excavation may be needed to get 0.1H wall embedment.
1387+00	2524.1		
1388+00	2524.0		
1389+00	2523.1		
1390+00	2521.5		
1391+00	2520.0		
1392+00	2522.0		
1393+00	2524.0		
1394+00	2525.0		
1395+00	2525.0		
1396+00	2525.4		
1397+00	2526.6		
1398+00	2527.6		
1399+00	2528.0		
1400+00	2528.0		
1401+00	2528.0		
1402+00	2528.0		
1403+00	2525.5	Rock	Relatively deep, sloping bedrock in this area. Wall would not meet stability criteria under seismic loading if founded on soil.
1403+50	2520.4		
1404+00	2516.9		
1404+50	2515.4		
1405+00	2515.6		
1405+50	2514.9		
1406+00	2515.8		
1406+50	2517.1		
1407+00	2522.6		
1407+50	2529.1		
1408+00	2530.0	Rock	Rock is generally shallow below wall; however, the available information suggests that steeply sloping bedrock could be present below parts of the wall on the lake side.
1408+50	2530.0		
1409+00	2530.0		
1410+00	2530.0		
1411+00	2530.0		
1412+00	2530.0	Soil or Rock	Bedrock outcrops on S embankment slope indicate shallow, flatlying bedrock. In some areas, rock excavation may be needed to get 0.1H wall embedment.
1413+00	2528.0		
1414+00	2526.5		
1415+00	2525.0		

Table 8.6
Recommended Wall Foundation Types

Station (LW)	Estimated Wall Face (March 2010) Rock Elevation	Recommended Foundation Type	Comments
1416+00	2525.0		
1417+00	2525.0		
1418+00	2524.5		
1419+00	2525.0		
1420+00	2496.0	Soil	Rock foundation not practical due to depth to bedrock. Wall height generally 10 feet or less.
1421+00	2473.9		
1422+00	<2450		
1423+00	<2450		
1424+00	<2450		

Table 9.1
Summary of Borings for Slide Curve Embankment Stabilization

Boring ID	Date Drilled		Elevation (feet)[1]	Project Datum[2]		Total Depth (feet)
	Started	Completed		Northing (feet)	Easting (feet)	
2008 Borings						
SW2-006-08(I)	10/21/2008	10/22/2008	2541.6	1,061,806	1,754,682	76.0
SW2-007-08	9/9/2008	9/11/2008	2402	1,061,810	1,754,416	96.0
SW2-008-08	9/12/2008	9/17/2008	2344	1,061,661	1,754,196	124.0
SW2-009-08	9/23/2008	9/24/2008	2340	1,061,521	1,754,312	117.0
Previous Borings						
H-1-97	9/29/1997	9/29/1997	2541.6	1,061,826	1,754,671	56.5
H-1-05	4/20/2005	5/5/2005	2503.4	1,061,760	1,754,626	115.0
H-27-06(E)	8/4/2006	8/6/2006	2537.2	1,062,604	1,753,970	89.6
H-27-06(I)	7/18/2006	7/19/2006	2537.4	1,062,593	1,753,971	80.5
H-29-06(E)	6/27/2006	6/28/2006	2540.7	1,062,208	1,754,433	69.3
H-29-06(I)	6/20/2006	6/22/2006	2540.5	1,062,217	1,754,429	90.0
SW2-001-07	8/30/2007	9/4/2007	2532.4	1,061,993	1,754,666	26.5
SW2-002-07(OW)	7/26/2007	7/31/2007	2541.5	1,061,928	1,754,589	86.0
SW2-003-07	9/5/2007	9/5/2007	2532.6	1,061,883	1,754,748	20.0
SW2-004-07	9/7/2007	9/10/2007	2543.5	1,061,748	1,754,737	51.5
SW2-005-07	9/6/2007	9/6/2007	2532.7	1,061,795	1,754,829	25.0

Notes
[1]Vertical datum is the North American Vertical Datum (NAVD) 1988
[2]Horizontal datum is based on the Washington Coordinate System, South Zone, and North American Datum (NAD) 1983/91

Table 9.2
Laboratory Index Test Results for Slide Curve Embankment Stabilization

Boring No. & Sample Information				Water Content (%)	Grain Size Analysis			Atterberg Limits			Organic Content (%)
Boring No.	Sample No.	Sample Depth (feet)	USCS/WSDOT Classification[1]		Gravel (%)	Sand (%)	Silt and Clay (%)	LL	PL	PI	
SW2-008-08	D-3	9.0-10.5	SW	20.9	4.9	87.0	8.1				
	D-4	19.0-20.5	CL	54.2				49	22	27	
	U-5	24.0-26.5	ML	44.1	0.0	0.4	99.6	30	24	6	
	D-7	34.0-35.5	SM	25.1	1.1	87.5	11.4				
	D-11	54.0-55.5	SW	9.6	41.5	53.9	4.6				
	D-13	64.0-65.5	GW	7.2	57.8	38.2	4.0				
	D-15	74.0-75.5	GW	6.6	50.1	45.0	4.9				
	D-19	94.0-95.5	SM	9.7	19.0	48.8	32.2				
	D-20	104.0-105.5	SP	13.8	6.5	86.7	6.8				
SW2-009-08	D-1	1.0-2.5	SM	20.3	12.6	75.6	11.8				
	D-3a	19.0-20.0	OH	118.3				89	47	42	9.40
	D-3b	20.0-20.5	SM	13.4	9.4	75.0	15.6				
	D-5	29.0-30.5	SW	15.4	31.9	60.0	8.2				
	D-6	39.0-40.5	CL	47.5				49	23	26	
	S-7	44.0-46.0	CL	40.4	0.0	3.1	96.9	32	10	3	
	D-8	54.0-55.5	SM	19.0	1.5	87.9	10.6				
	D-10	64.0-65.5	SP	17.7	0.0	90.1	9.9				
	D-12	76.0-77.5	SM	14.9	17.9	49.7	32.4				
	D-13a & b	86.0-87.5	SM	19.2	1.0	78.6	20.4				
	D-15	96.0-97.5	SP	18.9	13.9	81.8	4.3				

Notes
1. Material description per WSDOT soil and rock classification system.

Abbreviations
LL = liquid limit
PI = plasticity index
PL = plastic limit
USCS = Unified Soil Classification System

Table 9.3
Global Stability Analysis Results for Slide Curve Embankment Stabilization

LE Sta.	Condition	Calculated Factor of Safety				Figure No.
		High Lake Level (El. 2517)		Low Lake Level (El. 2416)		
		Static	Seismic - Pseudostatic	Static	Seismic - Pseudostatic	
1407+49	Existing	**1.05**	**--**	**1.0**	**0.64**	H.6.1 – H.6.3
	Proposed without soil buttress – failure surface intersects median wall	**1.11**	**--**	**1.05**	**0.64**	H.6.4 – H.6.6
	Proposed with soil buttress	1.43	--	1.28	**--**	H.6.7 – H.6.8
	Proposed with soil buttress (outer buttress slope)	1.55	--	1.56	**--**	H.6.9 – H.6.10
	During soil buttress construction	1.29	--	1.18	**--**	H.6.11 – H.6.12

Notes

Bold indicates a factor of safety less than 1.25 for static conditions (1.2 for static conditions during construction) or less than 1.1 for seismic conditions. For a failure surface that intersects the median wall, **bold** indicates a factor of safety less than 1.5 for static conditions or less than 1.1 for seismic conditions.

For seismic, pseudostatic case, a horizontal acceleration coefficient equal to 0.175 was assumed.

Table 10.1
Summary of Borings for Resort Creek Wall

Boring ID	Date Drilled		Elevation (feet)[1]	Project Datum[2]		Total Depth (feet)
	Started	Completed		Northing (feet)	Easting (feet)	
2008 Borings						
RCW-001-08	5/29/08	5/30/08	2523.4	1,061,379	1,755,836	65.0
SCW-013-08	5/9/08	5/13/08	2525.6	1,061,327	1,756,050	70.2
RCB-003-08(OW)	5/31/08	6/2/08	2527.5	1,061,365	1,756,357	101.5
2007 Borings						
CUL-007-07	8/29/2007	8/29/2007	2523.6	1,061,374	1,756,107	40.5
CUL-009-07	8/28/2007	8/28/2007	2525.6	1,061,367	1,756,250	41.5

Notes
[1]Vertical datum is the North American Vertical Datum (NAVD) 1988
[2]Horizontal datum is based on the Washington Coordinate System, South Zone, and North American Datum (NAD) 1983/91

Table 10.2
Rock Point Load Test Results for Resort Creek Wall

Boring ID/ Sample #	Depth (feet)	Rock Type	Weathering State [1]	Test Type [2]	Dia.[3] (mm)	Width [4] (mm)	D_e [5] (mm)	Unconfined Compressive Strength, q_u [6] ksi	ISRM [7] Class.	ISRM Description	Failure [8] Type
RCW-001-08/C-21	50-50.3	Lapilli Tuff	I	D	60.9	38.0	60.90	35.0	R5	Very strong	1
RCW-001-08/C-22	52-52.4	Lapilli Tuff	I	D	60.9	53.0	60.90	5.5	R2	Moderately weak	6
RCW-001-08/C-22	52.4-52.6	Lapilli Tuff	I	D	60.9	31.0	60.90	12.1	R3	Moderately Strong	1
RCW-001-08/C-24	63.1-63.4	Lapilli Tuff	I	D	60.9	37.0	60.90	22.7	R4	Strong	1

Notes

1. Weathering State: I - Fresh; II - Slightly weathered; III - Moderately weathered; IV - Highly weathered; V - Completely weathered
2. Test Type: A = Axial; D = Diametral
3. Diameter: The loaded side initial dimension
4. Width: For axial and irregular specimens, second dimension value to calculate D_e
5. D_e - Equivalent diameter: for diametral specimens, same as diameter, otherwise it is the square root of 4*Diameter*Width/π
6. Unconfined Compression Strength, q_u: taken as 24.5*$I_{s(50)}$ in MPa for 65 mm core size, and 22* $I_{s(50)}$ in MPa for 45 mm core size based on Table 1 in ASTM D 5731-02
7. ISRM Classification: R0: 0.28 to 1.03 MPa (0.04 to 0.15 ksi); R1: 1.03 to 24.8 MPa (0.15 to 3.6 ksi); R2: 24.8 to 50.3 MPa (3.6 to 7.3 ksi); R3: 50.3 to 103.4 MPa (7.3 to 15 ksi); R4:103.4 to 200 MPa (15 to 29 ksi); R5: > 200 MPa (>29 ksi).
8. Failure Type: 1 - Diametral normal failure; 2 - Axial normal failure; 3 - Block normal failure; 4 - Diametral, chipped failure; 5 - Axial, sheared at angle

Table 10.3
Liquefaction Analysis Results for Resort Creek Wall

Parameter[1]	Borehole	
	CUL-008-07[2]	**RCB-003-08(OW)**
Total thickness of liquefaction zone (feet)	46	28
Depth of liquefied zone (feet)	4-50	44-72
Total thickness of liquefaction zone in Unit 3/thickness of Unit 3 (%)	100	78
Settlement (inches)	10.6	4.5

Notes
[1] Liquefaction calculated assuming a peak ground acceleration (PGA), including site amplification effects, of 0.35g and a magnitude of 6.35 for an earthquake with a return period of 975 years
[2] Boring terminated at a depth of 50 feet in a potentially liquefiable zone. Additional liquefaction-induced settlement could occur below a depth of 50 feet.

Table 10.4
Global Stability Analysis Results for Resort Creek Wall, LW 1421+99

Direction	Treated Zone Width, feet	Calculated Factor of Safety						Results Figure No.
		High Groundwater and Lake Levels (El. 2517)			Low Groundwater (El. 2495) and Lake Levels (El. 2416)			
		Static	Seismic		Static	Seismic		
			Pseudo-static[1]	Residual Strength[2]		Pseudo-static[1]	Residual Strength[2]	
Wall without Stabilization								
Transverse, WB Embankment	0	**1.43**	--	--	**1.45**	1.16	**0.98**	H.7.1 – H.7.4
Wall with Stabilization								
Direction	**Treated Zone Width, feet**	**Calculated Factor of Safety**		**Estimated Displacement, feet[3]**	**Calculated Factor of Safety**		**Estimated Displacement, feet[3]**	**Results Figure No.**
		High Groundwater and Lake Levels (El. 2517)			**Low Groundwater (El. 2495) and Lake Levels (El. 2416)**			
		Static	**Residual Strength[2]**		**Static**	**Residual Strength[2]**		
Transverse, WB Embankment	10	--	--	--	--	1.12	3.5	H.7.5 – H.7.6
	20	--	--	--	--	1.27	1.5	H.7.7 – H.7.8
	30	--	--	--	--	1.44	0.75	H.7.9 – H.7.10

Notes
Bold indicates a factor of safety of less than 1.5 for static loading or less than 1.1 for seismic loading
[1]Used k_h = 0.175 and static (fully drained) shear strength for pseudostatic analysis
[2]Residual strength was estimated using the Kramer (2008) Method
[3]50% non-exceedance level

Table 11.1
Summary of Borings for Resort Creek Bridges

Boring ID	Date Drilled		Elevation (feet)[1]	Project Datum[2]		Total Depth (feet)
	Started	Completed		Northing (feet)	Easting (feet)	
2008 Borings						
RCB-001-08	5/29/08	6/4/08	2525.5	1,061,276	1,756,358	96.6
RCB-002-08(OW)	9/4/08	9/4/08	2524.8	1,061,277	1,756,537	76.3
RCB-003-08(OW)	5/31/08	6/2/08	2527.5	1,061,365	1,756,357	101.5
RCB-004-10	5/4/10	5/11/10	2529.3	1,061,362	1,756,517	106.0
RCB-005-10[4]	4/20/10	4/21/10	2495	1,061,216	1,756,352	96.0
RCB-006-10[4]	4/20/10	4/21/10	2495	1,061,199	1,756,514	70.5
RCB-007-10	4/22/10	4/28/10	2525.3	1,061,267	1,756,386	121.0
RCB-008-10	4/21/10	4/27/10	2524.4	1,061,265	1,756,511	109.2
Previous Borings						
CUL-011-07	8/29/2007	8/29/2007	2527.8	1,061,363	1,756,400	41.5
CUL-012-07	7/12/2007	7/12/2007	2493.8	1,061,192	1,756,377	52
CUL-013-07	8/28/2007	8/28/2007	2529.5	1,061,362	1,756,546	43
CUL-014-07	7/26/2007	7/31/2007	2493.0	1061,171	1,756,525	39.5
CUL-017-07	9/5/2007	9/5/2007	2529.1	1,061,360	1,756,606	41.5
CUL-018-07	9/7/2007	9/10/2007	2495.5	1,061,203	1,756,596	40
EMB-022-07	9/6/2007	9/6/2007	2493.8	1,061,123	1,756,914	32
EMB-023-07	9/6/2007	9/6/2007	2530.4	1,061,339	1,756,905	61
H-36-06[3]	7/20/2006	7/20/2006	2484.1	1,061,120	1,756,427	70.5

Notes
[1]Vertical datum is the North American Vertical Datum (NAVD) 1988
[2]Horizontal datum is based on the Washington Coordinate System, South Zone, and North American Datum (NAD) 1983/91
[3]Northing and easting have been updated from State Plane datum (as reported on the log) to the Project datum that was adopted in 2007.
[4]The drilling site was inundated by the rising lake before survey crews could be mobilized. Northing and easting were measured by WSDOT FEU using GPS. Elevation was interpreted from the topographic map.

Table 11.2
Laboratory Test Results for Resort Creek Bridges

Boring No.	Sample No.	Sample Depth (feet)	USCS Classification[1]	Water Content (%)	Grain Size Analysis			Atterberg Limits		
					Gravel (%)	Sand (%)	Silt + Clay (%)	LL	PL	PI
2008-2010 Laboratory Results										
RCB-001-08	D-4	8.6-10.1	GM-GW	9.8						
	D-6	13.6-15.1								
	D-12	28.6-30.1	SM	27.2	22.5	49.1	28.4			
	D-14	33.6-35.1	SM	21.1	32.5	48.3	19.2			
	D-18	41.3								
	D-20	47.8								
	D-22	50.2	SM	18.9	34.5	53.2	12.3			
	D-25	56.3								
	D-27	62.8	SM	23.4	30.6	43.5	25.9			
	D-29	66.3	SM	27.6	9.7	57.9	32.4			
	D-31	71.3	CL	43.6	0.0		100	44	22	22
	D-33	76.3	SM	19.9	25.6	62.0	12.4			
	D-36	84.3	SM	11.7	25.2	49.0	25.8			
	D-38	86.3								
	D-40	91.7	SM	14.3	34.6	41.9	23.5			
	D-42	96.3								
RCB-002-08 (OW)	D-1	4.0-4.9	GW	42.5	76.7	18.6	4.7			
	D-7	20.0-21.5	GM	14.2	56.4	30.0	13.6			
	D-9	25.0-26.5	GM-SM	21.0	46.4	40.3	13.3			
	D-11	30.0-30.5	GP	16.0	56.4	35.0	8.6			
	D-13	35.0-36.5	GP	14.0	64.4	27.5	8.1			
	D-15	40.0-41.5	SM	15.8	35.7	45.8	18.5			
	D-17	45.0-46.5								
	D-23	60.0-61.5	SM	22.0	9.1	64.1	26.8			
	D-25	65.0-66.5								
	D-28	75.0-76.3	SM	8.8	22.2	44.1	33.7			

Table 11.2 (continued)
Laboratory Test Results for Resort Creek Bridges

Boring No.	Sample No.	Sample Depth (feet)	USCS Classification[1]	Water Content (%)	Grain Size Analysis			Atterberg Limits		
					Gravel (%)	Sand (%)	Silt + Clay (%)	LL	PL	PI
2008-2010 Laboratory Results										
RCB-003-08 (OW)	D-2	9.0-9.3	SM-GM	13.7						
	D-5	15.4-15.9								
	D-13	29.0-30.5	GP	35.8	63.2	28.4	8.4			
	D-15	34.2-35.7	GW	24.8	54.7	36.7	8.6			
	D-18	41.5-42.4								
	D-20	45.6-47.1	SM	26.5	17.3	57.2	25.5			
	D-22	51.5-53.0	CL-ML	26.0	2.6	52.2	45.2	28	21	7
	D-25	61.3-62.8	SM	23.7	12.1	53.4	34.5			
	D-27	66.5-68.0	CL	32.5	0.0		100	41	25	16
	D-29	71.5-73.0	SM	22.5	15.6	60.7	23.7			
	D-31	76.5-78.0								
RCB-004-08	D-5	51.0-52.5	SM	17.0	18.8	66.6	14.6			
	D-13	71.0-72.5	SM	29.4	69.7		30.3			
Previous Laboratory Results										
CUL-012-07	D-1	2-3.5	Well-graded SAND with gravel	14.7	40.2	54.6	5.2			
	D-2	5-6.5	Poorly-graded GRAVEL with sand	11.2	71.4	24.8	3.8			
	D-3	7-8.5	Well-graded GRAVEL with sand	13.5	50.5	41.3	8.1			
	D-4	10-11.5	Poorly-graded GRAVEL with sand	12.5	64.7	29.1	6.1			
	D-5	12-13.5	Silty SAND with gravel	14.1	36.1	51.1	12.8			
	D-6	15-16.5	Silty SAND with gravel	18.1	27.8	58.1	14.1			
	D-7	17-18.5	Silty SAND	34.9	0.3	82.6	17.1			
	D-8	20-21.5	Silty SAND	24.4	9.7	72.9	17.4			
CUL-018-07	D-3	15-16.5	Silty SAND with gravel	10.4	34.1	55.4	10.5			
	D-4	20-21.5	Silty SAND with gravel	12.8	29.4	55.6	15.0			
	D-5	25-26.5	Silty SAND	13.5	9.3	73.2	17.5			
	D-6	30-31.5	Sandy SILT	18.7	0.2	30.2	69.6	24	23	1

Notes
1. Material description per WSDOT soil and rock classification system.
Abbreviations
LL = liquid limit
PI = plasticity index
PL = plastic limit
USCS = Unified Soil Classification System

Table 11.3
Rock Unconfined Compression Test Results for Resort Creek Bridges

Specimen	Depth	Compressive Strength (ksi)
RCB-004-10/C-25	105.1-106.0	5.6
RCB-005-10/C-9	83.3-84.2	2.0
RCB-005-10/C-10	89.4-90.1	1.9
RCB-006-10/C-2	56.7-57.7	2.7
RCB-006-10/C-3	60.5-61.2	1.0
RCB-007-10/C-3	114.6-115.3	1.9
RCB-007-10/C-4	120.2-121.0	2.4
RCB-008-10/C-2	98.5-99.2	2.0

Table 11.4
Rock Point Load Test Results for Resort Creek Bridges

Boring ID/Sample #	Depth (feet)	Rock Type	Weath. State [1]	Test Type [2]	Dia.[3] (mm)	Width [4] (mm)	De [5] (mm)	qu [6] (ksi)	ISRM [7] Class.	ISRM Description	Failure [8] Type
RCB-004-10/C-20	89	Andesite	I	D	60.0	220.0	60.00	7.5	R3	Mod. Strong	6
RCB-004-10/C-21	91.3	Andesite	I	D	60.0	198.0	60.00	8.4	R3	Mod. Strong	1
RCB-004-10/C-22	95.8	Andesite	I	D	60.0	120.0	60.00	2.2	R1	Very Weak	6
RCB-004-10/C-23	102.2	Andesite	I	D	60.0	101.0	60.00	5.5	R2	Mod. Weak	6
RCB-004-10/C-24	104	Andesite	I	D	60.0	96.0	60.00	2.6	R1	Very Weak	6
RCB-005-10/C-9	83.1	Andesite	I	D	60.0	108.0	60.00	5.4	R2	Mod. Weak	1
RCB-005-10/C-9	84.5	Andesite	I	D	60.0	138.0	60.00	3.9	R2	Mod. Weak	1
RCB-005-10/C-10	89.1	Andesite	I	D	60.0	105.0	60.00	4.2	R2	Mod. Weak	1
RCB-005-10/C-10	90.3	Andesite	I	D	60.0	98.0	60.00	3.7	R2	Mod. Weak	1
RCB-005-10/C-11	91.2	Andesite	I	D	60.0	84.0	60.00	1.3	R1	Very Weak	1
RCB-006-10/C-1	53.4	Lapilli Tuff	I	D	60.0	158.0	60.00	1.5	R1	Very Weak	6
RCB-006-10/C-2	56.4	Lapilli Tuff	I	D	60.0	210.0	60.00	2.1	R1	Very Weak	1
RCB-006-10/C-2	57.9	Lapilli Tuff	I	D	60.0	125.0	60.00	1.2	R1	Very Weak	1
RCB-006-10/C-3	60.2	Andesite	I	D	60.0	122.0	60.00	2.2	R1	Very Weak	1
RCB-006-10/C-3	61.9	Andesite	I	D	60.0	108.0	60.00	17.7	R4	Strong	4
RCB-006-10/C-5	70	Andesite	I	D	60.0	155.0	60.00	0.5	R1	Very Weak	1
RCB-007-10/C-1	103.6	Andesite	I	D	60.0	92.0	60.00	5.8	R2	Mod. Weak	1
RCB-007-10/C-2	109	Andesite	I	D	60.0	78.0	60.00	6.1	R2	Mod. Weak	4
RCB-007-10/C-3	114.4	Andesite	I	D	60.0	106.0	60.00	2.6	R1	Very Weak	1
RCB-007-10/C-4	116.4	Andesite	I	D	60.0	220.0	60.00	4.3	R2	Mod. Weak	6
RCB-007-10/C-4	120	Andesite	I	D	60.0	140.0	60.00	5.9	R2	Mod. Weak	1
RCB-008-10/C-1	94	Andesite	I	D	60.0	130.0	60.00	1.7	R1	Very Weak	1
RCB-008-10/C-2	98.3	Andesite	I	D	60.0	72.0	60.00	1.2	R1	Very Weak	1
RCB-008-10/C-2	99.5	Andesite	I	D	60.0	148.0	60.00	3.1	R1	Very Weak	1
RCB-008-10/C-4	106.6	Andesite	I	D	60.0	158.0	60.00	2.6	R1	Very Weak	1
RCB-008-10/C-4	107.9	Andesite	I	D	60.0	147.0	60.00	1.9	R1	Very Weak	1

Table 11.4 (Continued)
Rock Point Load Test Results for Resort Creek Bridges

Notes

1. Weathering State: I - Fresh; II - Slightly weathered; III - Moderately weathered; IV - Highly weathered; V - Completely weathered
2. Test Type: A = Axial; D = Diametral
3. Diameter: The loaded side initial dimension
4. Width: For axial and irregular specimens, second dimension value to calculate D_e
5. D_e - Equivalent diameter: for diametral specimens, same as diameter, otherwise it is the square root of 4*Diameter*Width/π
6. Unconfined Compression Strength, q_u: taken as 24.5*$I_{s(50)}$ in MPa for 65 mm core size, and 22* $I_{s(50)}$ in MPa for 45 mm core size based on Table 1 in ASTM D 5731-02
7. ISRM Classification: R0: 0.28 to 1.03 MPa (0.04 to 0.15 ksi); R1: 1.03 to 24.8 MPa (0.15 to 3.6 ksi); R2: 24.8 to 50.3 MPa (3.6 to 7.3 ksi); R3: 50.3 to 103.4 MPa (7.3 to 15 ksi); R4:103.4 to 200 MPa (15 to 29 ksi); R5: > 200 MPa (>29 ksi).
8. Failure Type: 1 - Diametral normal failure; 2 - Axial normal failure; 3 - Block normal failure; 4 - Diametral, chipped failure; 5 - Axial, sheared at angle; 6 - Failed along healed joint

Table 11.5
Liquefaction Analysis Results for Resort Creek Bridges

Parameter[1]	Borehole						
	West Abutment (Pier 1)			East Abutment (Pier 2)			
	RCB-001-08	RCB-003-08(OW)	CUL-012-07	RCB-002-08(OW)	RCB-004-10[3]	CUL-014-07[2]	CUL-018-07
Total thickness of liquefaction zone (feet)	24	28	41	8	42	19.5	4
Depth of liquefiable zone(s) (feet)[4]	29-35, 48-54, 59-71	44-72	0-3.5, 14.5-52	44.5-46, 59-65.5	15.5-17, 21-30, 41-64, 67.5-76	10-14, 17.5-33	13-17
Total thickness of liquefiable zone in Stratum 3/ thickness of Stratum 3 (%)	69	78	92	27	90	85	15
Settlement (inches)	5.0	4.5	8.0	0.8	6.9	3.4	0.8

Notes
[1]Liquefaction calculated assuming a peak ground acceleration (PGA), including site amplification effects, of 0.35g and a magnitude of 6.35 for an earthquake with a return period of 975 years
[2]Boring terminated in a potentially liquefiable zone
[3]Used soils information from CUL-013-07 for depths of 0 to 44 feet.
[4]Liquefied zones defined as zones where the calculated factor of safety (FS) for liquefaction is less than 1.2, i.e.:
FS=(cyclic resistance ratio)/(cyclic strength ratio)<1.2

Table 11.6
Global Stability Analysis Results for Resort Creek Bridges Without Ground Improvement

Pier	Direction	Wall (location)	Factor of Safety for Static Loading		Factor of Safety for Seismic Loading[1]			Figure Nos.
			Low Water Level[1]	High Water Level[2]	Pseudostatic	Residual Strength		
						Kramer	Olson & Stark	
Pier 1 (West)	Longitudinal	NA	**1.24**	**1.13**	**1.03**	**0.85**	**0.86**	H.8.1-H.8.5
	Transverse – EB	11 (lakeside)	1.65	1.71	1.07	**0.48**	**0.45**	H.8.6-H.8.10
		12 (median)	3.09	2.83	2.30	**--**	**--**	H.8.29-H.8.31
	Transverse – WB	15 (median)	2.71	2.69	2.04	--	--	H.8.32-H.8.34
		16 (hillside)	1.62	1.46	1.19	--	1.30	H.8.11-H.8.14
Pier 2 (East)	Longitudinal	NA	1.71	1.50	1.26	**0.86**	**0.87**	H.8.15-H.8.19
	Transverse –- EB	13 (lakeside)	1.71	1.88	1.08	**0.72**	**0.63**	H.8.20-H.8.24
		14 (median	3.12	3.09	2.36	--	--	H.8.35-H.8.37
	Transverse – WB	17 (median)	3.11	3.11	2.31	--	--	H.8.38-H.8.40
		18 (hillside)	1.48	1.31	1.11	--	1.85	H.8.25-H.8.28

Notes
Bold indicates factors of safety of less than 1.5 for static loading in the longitudinal direction, 1.3 for static loading in the transverse direction, and 1.1 for seismic loading.
[1] Groundwater level at El. 2492 and lake level at El. 2416
[2] Groundwater and lake levels at El. 2517

Table 11.7
Drilled Shaft Bearing and Uplift Resistance Factors

Limit State	Resistance Factor, φ		
	Skin Friction	Tip Resistance	Uplift
Strength	0.55	0.5	0.45 (soil) 0.4 (rock)
Service	1.0	1.0	1.0
Extreme Event	1.0	1.0	0.8

Table 11.8
Downdrag Loads and Skin Friction Losses Due to Liquefaction

Pier	Unfactored Downdrag Load (kips)			Unfactored Skin Friction Loss (kips)		
	6-ft Dia. Shaft	7-ft Dia. Shaft	8-ft Dia. Shaft	6-ft Dia. Shaft	7-ft Dia. Shaft	8-ft Dia. Shaft
1 (West)	1,200	1,400	1,600	1,950	2,270	2,600
2 (East)	940	1,100	1,260	1,490	1,740	1,990

Table 11.9
Design Parameters for Lateral Load Analysis Using DFSAP and LPILE at Resort Creek Bridges

Layer	Top of Layer Elevation	Soil Type	Layer Thickness	Effective Unit Weight	Strength Limit State			Extreme Event Limit State (Note 4)			Cohesion	Rock Unconfined Compressive Strength (Notes 5, 6)
					Effective Friction Angle	Strain, e_{50}	P-y Modulus (Note 6)	Effective Friction Angle	Strain, e_{50}	P-y Modulus (Note 7)		
	feet		feet	pcf	degrees	unitless	pci	degrees	unitless	pci	psf	psf
West Abutment (Pier 1)												
1	Note 1	Sand	Note 1	68 (Note 1)	38	0.005	60	38	0.005	60	--	--
2	2480	Sand	40	48	30	0.005	40	7	0.03	20	--	--
3	2440	Sand	Note 2	68	38	0.005	125	38	0.005	125	--	--
4	Note 2	Rock (Note 5)	Note 3	88	0	0.004	2,000 (soft) 4,000 (stiff)	0	0.004	2,000 (soft) 4,000 (stiff)	--	80,000
East Abutment (Pier 2)												
1	Note 1	Sand	Note 1	68 (Note 1)	38	0.005	60	38	0.005	60	--	--
2	2484	Sand	31	48	30	0.005	40	13	0.03	20	--	--
3	2453	Sand	Note 2	68	38	0.005	125	38	0.005	125	--	--
4	Note 2	Rock (Note 5)	Note 3	88	0	0.004	2,000 (soft) 4,000 (stiff)	0	0.004	2,000 (soft) 4,000 (stiff)	--	80,000

Notes
1. The top of shaft elevation has not been established at this time. The thickness of Layer 1 can be calculated using the top of shaft elevation and the elevation of the top of Layer 2. For Layer 1, the effective unit weight is equal to 68 pcf at and below El. 2517 and 130 pcf above El. 2517.
2. The thickness of Layer 3 and the top elevation of Layer 4 can be estimated for each pier using the top of bedrock elevations shown in cross sections RCB 1-1' (Figure 11.1) and RCB 2-2' (Figure 11.2) for Piers 1 and 2, respectively.
3. The layer can be assumed to be thick enough to extend below the base of the shaft.
4. For potentially liquefiable layers, the residual values of the effective friction angle, strain, and modulus of subgrade reaction are provided. Residual effective friction angles are based on Kramer's residual strength (Sr) method at 40% non-exceedance probability for a deterministic analysis. Sr estimates are based on estimated average $(N1)_{60}$ from borings in the vicinity of each pier. Residual strain e_{50} are nominal values.
5. The value provided is the global rock mass strength, σ'_{cm}, calculated using the method presented in Hoek, et al. (2002). This value differs from and is higher than the rock mass unconfined compressive strength.
6. Layer 4 should be modeled as Stiff Clay with Free Water for LPILE analyses, using a cohesion value equal to one-half the rock unconfined compressive strength.
7. For LPILE analyses, the shafts should be analyzed using both the soft and stiff p-y moduli for Layer 4.

Abbreviations
pcf – pounds per cubic foot
psf – pounds per square foot

Table 11.10
Estimated Free-Field Deformations for LPILE Analyses at Resort Creek Bridges

Elevation, feet	Estimated Free-Field Deformation, feet[1]
Pier 1	
≥2480	3
≤2440	0
Pier 2	
≥2484	3
≤2453	0

Notes
[1]The estimated free-field deformation varies linearly between El. 2440 and 2480 at Pier 1 and between El. 2453 and 2484 at Pier 2.

Table 11.11
Calculated Lateral Forces – Seismic Loading, Residual Strength Case

Abutment	Direction	Calculated Lateral Force, kip/ft[1]	Calculated Factor of Safety	Figures
Pier 1 (West)	Longitudinal	48	1.10	H.8.41
	Transverse – EB	125	1.12	H.8.42
Pier 2 (East)	Transverse – EB	80	1.12	H.8.43

Notes.
Residual strength characterized using Kramer (2008) method
Groundwater level at El. 2492 and lake level at El. 2416
[1]Lateral resisting force, in kips per horizontal foot, needed from foundation to provide a calculated limit equilibrium factor of safety of 1.1 for seismic loading, residual strength case. The lateral force should be modeled as a uniform pressure over the depth of the liquefiable zone.

Table 12.1
Summary of Borings and Test Pit for Culverts

Boring/ Test Pit ID	Date Drilled		Elevation (feet)[1]	Project Datum[2]		Total Depth (feet)
	Started	Completed		Northing (feet)	Easting (feet)	
Townsend Creek Culvert						
TCC-001-08	9/8/08	9/8/08	2526.0	1,057,362	1,760,277	37.3
TCC-002-08	5/11/08	5/11/08	2525.9	1,057,250	1,760,237	40.3
TCC-003-08	5/28/08	5/29/08	2525.3	1,057,165	1,760,170	39.8
TCC-TP-001-08	6/18/08	6/18/08	2522.5	1,057,226	1,760,219	12.5
Resort Creek Culvert on Old Sunset Highway						
CUL-021-10	9/22/10	9/22/10	2524.7	1,061,718	1,756,661	50.0
CUL-022-10	9/23/10	9/24/10	2525.2	1,061,729	1,756,697	51.0

Notes
[1]Vertical datum is the North American Vertical Datum (NAVD) 1988
[2]Horizontal datum is based on the Washington Coordinate System, South Zone, and North American Datum (NAD) 1983/91

Table 12.2
Summary of Laboratory Test Results for Culverts

Boring No. & Sample Information					Grain Size Analysis			Atterberg Limits				Resistivity (Ω·cm)			
Boring No.	Sample No.	Sample Depth (feet)	USCS/WSDOT Classification[1]	Water Content (%)	Gravel (%)	Sand (%)	Silt & Clay (%)	LL	PL	PI	pH	Native	Saturated	Chlorides (ppm)	Sulfates (ppm)
TCC-002-08	D-2	7.0-8.5	GP	17.5	92.4		7.6								
	D-5	13.8-15.3	GM	19.7	78.0		22.0								
	D-10	23.8-24.6	GP	14.6	92.4		7.6								
	D-13	28.8-30.3	GP	11.9	90.7		9.3								
	D-18	38.8-40.3	SM	12.0	84.9		15.1								
TCC-003-08	D-2	3.4-4.9	GP	16.1	92.2		7.8								
	D-6	13.4-14.9	GP	37.7	90.9		9.1								
	D-8	18.4-19.9	SM	16.3	81.1		18.9								
	D-10-A	23.4-23.9	SM/SC	56.0	72.0		28.0								
	D-10-B	23.9-24.9	SM/SC	31.8	51.2		48.8								
	D-12	28.4-28.8	SP	23.8	93.6		6.4								
	D-14	33.4-34.1	SM	16.6	85.2		14.8								
	D-17	38.4-39.8	GM	10.3	88.4		11.6								
TCC-TP-002-08	D-1	5.5-5.7	SM								7.5	14,000	11,000	10.2	20.1
	D-2	10.0-10.2	GM								7.8	8,700	8,300	6.1	13.3
CUL-021-10	D-6	15.0-16.5	SW	10.2	45.0	46.0	9.0								
	D-8	20.0-21.5	SM	10.9	38.6	48.1	13.3								
	D-10	25.0-25.8	SM	14.4	43.1	46.8	10.1								
	D-12	30.0-31.5	SP	11.7	42.5	49.1	8.4								
	D-14	35.0-36.2	GM	10.5	45.0	44.9	10.1								
	D-16	40.0-40.5	SM	10.6	14.2	57.2	28.6								

Table 12.2 (continued)
Summary of Laboratory Test Results for Culverts

Boring No. & Sample Information				Water Content (%)	Grain Size Analysis			Atterberg Limits			pH	Resistivity (Ω·cm)		Chlorides (ppm)	Sulfates (ppm)
Boring No.	Sample No.	Sample Depth (feet)	USCS/WSDOT Classification[1]		Gravel (%)	Sand (%)	Silt & Clay (%)	LL	PL	PI		Native	Saturated		
CUL-022-10	D-4	11.0-12.5	GW	11.5	66.3	30.0	3.7								
	D-6	16.0-17.5	SW	11.3	45.4	45.6	9.0								
	D-8	21-22.5	SP	20.6	44.7	45.8	9.5								
	D-10	26-27.5	SP	14.8	36.2	54.4	9.4								
	D-15	41-41.3	SM	9.8	13.7	63.3	22.9								

Notes
1. Material description per WSDOT soil and rock classification system.

Abbreviations
LL = liquid limit
PI = plasticity index
PL = plastic limit
ppm = parts per million
USCS = Unified Soil Classification System
Ω·cm = ohm-centimeter

Table 12.3
LRFD Shallow Foundation Resistance Factors

Limit State	Resistance Factor, φ		
	Bearing Resistance	**Friction Resistance to Sliding**	**Passive Pressure Resistance to Sliding**
Strength[1]	0.45	0.80	0.5
Service	1.0	N/A	N/A
Extreme Event[2]	0.9	0.9	0.9

Notes
[1] Resistance factors for the strength limit state are recommended in the LRFD Bridge Design Specifications, Table 10.5.5.2.2-1.
[2] Resistance factors for the extreme event limit state are recommended in the GDM, Section 8.10.2.

Table 13.1
Summary of Borings for Embankments

Boring ID	Date Drilled		Elevation (feet[1])	Project Datum[2]		Total Depth (feet)
	Started	Completed		Northing (feet)	Easting (feet)	
2008-2009 Borings						
EMB-027-09	5/10/2008	5/11/2008	2507.9	1,073,697	1,751,260	51.0
Previous Borings						
CUL-008-07	7/12/2007	7/13/2007	2513.0	1,061,217	1,756,090	51.0
CUL-010-07	7/13/2007	7/13/2007	2513.0	1,061,201	1,756,239	56.5
CUL-016-07	9/12/2007	9/12/2007	2524.7	1,060,662	1,757,638	41.5
EMB-022-07	10/9/2007	10/10/2007	2493.8	1,061,123	1,756,914	30.5
EMB-023-07	9/20/2007	9/20/2007	2530.4	1,061,339	1,756,905	60.5
EMB-024-07	10/10/2007	10/11/2007	2524.2	1,059,835	1,758,202	58.0
EMB-025-07	10/15/2007	10/15/2007	2546.5	1,059,978	1,758,334	16.0
H-34-06	7/7/2006	7/7/2006	2533.6	1,061,278	1,755,667	31.5
H-35-06	7/11/2006	7/11/2006	2497.7	1,061,199	1,755,670	15.5
H-37-06	7/11/2006	7/12/2006	2493.9	1,060,895	1,757,275	49.0
H-38-06(OW)	7/11/2006	7/11/2006	2526.6	1,060,006	1,758,174	30.1
H-9-98	10/31/98	11/4/98	2,474.6	1,060,739	1,756,056	110.0
H-10-98	10/27/98	10/29/98	2,483.2	1,060,295	1,757,571	69.5
H-11-98	10/25/1998	10/26/1998	2476.4	1,059,611	1,758,193	74.5

Notes
[1]Vertical datum is the North American Vertical Datum (NAVD) 1988
[2]Horizontal datum is based on the Washington Coordinate System, South Zone, and North American Datum (NAD) 1983/91

Table 13.2
Summary of Laboratory Test Results for Embankments

Boring No. and Sample Information				Water Content (%)	Grain Size Analysis			Atterberg Limits		
Boring No.	Sample No.	Sample Depth (feet)	USCS/WSDOT Classification[1]		Gravel (%)	Sand (%)	Silt/Clay (%)	LL	PL	PI
EMB-027-09	D-1	5-6.5	SM	13.0	31.6	56.7	11.7	--	--	--
	D-2	10-11.5	SP	16.3	34.2	63.8	1.9	--	--	--
H-11-98	D-2	8.0	SM	2.0	26.6	61.3	12.1	NP	NP	NP
	D-6	28.0	SM	1.0	33.0	42.4	24.6	NP	NP	NP
	D-7	33.0	SC-SM	9.0	32.9	33.3	33.7	24	18	6
	D-10	44.5	SC	10.0	4.8	55.1	40.1	34	19	15

Notes
[1]Material description per WSDOT soil and rock classification system.

Abbreviations
LL = liquid limit
NP = non-plastic
PI = plasticity index
PL = plastic limit
USCS = Unified Soil Classification System

Table 13.3
Global Stability Analysis Results for Embankments

Location	Calculated Factor of Safety for Static Loading		Results Figure No.
	High Groundwater and Lake Levels[1]	**Low Groundwater and Lake Levels [2]**	
LE 1421+84	1.52	1.27	H.9.1 - H.9.2
AE 121+03	--	**1.09[3]**	H.9.3
	--	1.26[4]	H.9.4

Notes
Bold indicates a factor of safety less than 1.25.
[1]High groundwater and high lake levels are at El. 2517.
[2]Low groundwater level is El. 2492 at LE 1421+84 and 2506.5 at AE 121+03. Low lake level = El. 2416.
[3] Friction angle for Unit 3 = 30 degrees
[4] Friction angle for Unit 3 = 34 degrees

Table 14.1
Summary of Phase 1C Sign Structures and Subsurface Conditions

Station[1]	Side of Roadway[1,2]	Direction of Traffic[1]	Type of Sign Structure[1]	Boring(s)	Estimated Bedrock Elevation, feet	Depths Boulders Encountered, feet
LW 1338+00	Left	WB	Cantilever	None- within rock cut area	NA	NA
LE 1363+00	Median		Light standard	SSE-026-10	2530.3	None
LE 1365+33	Median		Light standard	SSE-027-10	2533.0	None
LE 1367+66	Median		Light standard	SSE-028-10	2531.2	None
LW 1385+00	Left	WB	Cantilever	None- within fill area	2540	NA
LW 1421+30	Left	WB	Cantilever	RCW-001-08	2473.9	13
AE 116+30	Right	EB	Cantilever	SSE-025-10	Not encountered above El. 2503.2	None

Notes
[1]Sign types and locations are based on an email received from WSDOT on March 25, 2010
[2] Side of roadway is relative to the EB direction of travel
EB = eastbound
LE = EB centerline
LW = WB centerline
NA = not applicable; the sign location is within an area of proposed rock cuts.
WB = westbound

Table 14.2
Summary of Borings for Phase 1C Sign Structures

Boring ID	Date Drilled		Elevation (feet)[1]	Project Datum[2]		Total Depth (feet)
	Started	Completed		Northing (feet)	Easting (feet)	
RCW-001-08	5/29/08	5/30/08	2523.4	1,061,379	1,755,836	65.0
SSE-006-08	5/15/08	5/15/08	2539.3	1,065,655	1,754,212	9.0
SSE-008-08	5/15/08	5/15/08	2535.7	1,060,905	1,757,639	8.5
SSE-009-08	5/15/08	5/15/08	2533.5	1,064,103	1,754,598	8.5
SSE-010-08	5/8/08	5/8/08	2533.5	1,064,102	1,754,602	8.5
SSE-011-08	5/14/08	5/14/08	2535.4	1,072,661	1,751,733	13.5
SSE-022-09	6/17/09	6/17/09	2534.0	1,064,145	1,754,609	18
SSE-025-10	4/29/10	4/29/10	2524.7	1,060,276	1,757,906	21.5
SSE-026-10	5/13/10	5/13/10	2534.3	1,066,156	1,754,184	14.5
SSE-027-10	5/12/10	5/12/10	2535.1	1,065,929	1,754,220	9.0
SSE-028-10	5/12/10	5/12/10	2534.6	1,065,705	1,754,279	14.5

Notes
[1]Vertical datum is the North American Vertical Datum (NAVD) 1988
[2]Horizontal datum is based on the Washington Coordinate System, South Zone, and North American Datum (NAD) 1983/91

Table 14.3
Laboratory Test Results for Sign Structures

Boring No. & Sample Information				Water Content (%)	Grain Size Analysis					
							Silt (%)		Clay (%)	
Boring No.	Sample No.	Sample Depth (feet)	USCS/WSDOT Classification		Gravel (%)	Sand (%)	>0.005 mm	>0.002 mm	<0.005 mm	<0.002 mm
SSE-006-08	D-1	2.5	GW	4.9	73.7	25.6	0.7			
	D-2	4.0	GW	2.0						
SSE-008-08	D-1	2.5	SW	13.9						
	D-3	5.0	SM	29.2	34.9	57.5	7.6			
SSE-009-08	D-1	2.0	SM	11.0	33.0	54.4	13.1			
	D-2	4.5	SC	20.8	30.3	45.1	24.6			
	D-3	7.0	SM	15.3	40.4	45.1	13.5			
SSE-010-08	D-1	2.0	GM	16.8	43.7	41.7	14.6			
	D-2	4.5	GW	24.3	54.3	35.9	9.8			
	D-3	7.0	SM	29.4	12.5	66.0	21.5			
SSE-011-08	D-1	2.5	GP	15.1	62.4	30.5	7.1			
	D-3	5.0	GM	17.4	45.6	41.7	12.7			
SSE-018-09	D-2	5.0-6.5	GW	8.7	49.2	43.8	7.0			
SSE-019-09	D-7	16.0-17.5	GW	7.1	58.7	34.6	6.8			
SSE-021-09	D-1	0.0-1.5	SM	3.4	34.9	54.8	10.3			
	D-3	6.5-8.0	GM	8.3	50.7	36.0	13.3			
	D-5	11.5-13.0	SM	9.0	41.4	45.6	13.0			
SSE-022-09	D-2	4.0-5.5	GM	11.1	52.9	36.5	10.6			

Table 15.1
Summary of Borings and Test Pits for Stormwater Mitigation Facilities

Boring/Test Pit ID	Date Drilled		Elevation (feet) [1]	Project Datum [2]		Total Depth (feet)
	Started	Completed		Northing (feet)	Easting (feet)	
HMF-001-08	5/10/2008	5/11/2008	2545.3	1,080,146	1,748,786	15.0
HMF-002-08	5/10/2008	5/10/2008	2545.2	1,080,064	1,748,753	15.3
HMF-003-08	5/11/2008	5/11/2008	2545.4	1,079,995	1,748,728	16.5
HMF-004-08	5/10/2008	5/10/2008	2544.4	1,080,224	1,748,482	19.0
HMF-005-08	5/10/2008	5/10/2008	2545.9	1,080,054	1,748,483	14.3
HMF-006-08	5/10/2008	5/10/2008	2544.1	1,080,316	1,748,456	15.3
HMF-007-08	5/11/2008	5/11/2008	2543.1	1,080,345	1,748,400	15.5
HMF-008-09	6/9/2008	6/9/2008	2546.1	1,080,198	1,748,762	14.0
TP-003-08	6/9/2008	6/9/2008	2529.5	1,079,832	1,748,294	14.0
HMF-TP-001-09	5/18/2009	5/18/2009	2538.7	1,080,327	1,748,358	11.3
HMF-TP-002-09	5/18/2009	5/18/2009	2533.7	1,080,120	1,748,314	11.5
HMF-TP-003-09	5/18/2009	5/18/2009	2532.2	1,079,968	1,748,272	11.5
HMF-TP-004-10[3]	6/1/2010	6/1/2010	Not surveyed			10.0
HMF-TP-005-10	6/1/2010	6/1/2010	2545.4	1080026.17	1748743.59	11.0
INF-TP-006-10	6/1/2010	6/1/2010	2437.0	1,049,261	1,764,907	7.5
INF-TP-007-10	6/1/2010	6/1/2010	2429.8	1,049,129	1,765,189	10.5
INF-TP-009-10	6/1/2010	6/1/2010	2424.2	1,048,870	1,765,548	9.7
INF-TP-010-10(OW)	6/22/2010	6/22/2010	2435.8	1,049,240	1,764,972	16.5
INF-TP-011-10	6/22/2010	6/22/2010	2431.8	1,048,879	1,765,017	10.0
INF-TP-012-10(OW)	6/22/2010	6/22/2010	2427.9	1,049,049	1,765,449	17.0
INF-TP-013-10	6/22/2010	6/22/2010	2429.1	1,048,739	1,765,265	11.0
INF-TP-014-10	6/22/2010	6/22/2010	2425.6	1,048,732	1,765,462	12.0
INF-TP-015-10	6/21/2010	6/21/2010	2542.7	1,048,884	1,764,683	26.0

Notes
[1]Vertical datum is the North American Vertical Datum (NAVD) 1988.
[2]Horizontal datum is based on the Washington Coordinate System, South Zone, and North American Datum (NAD) 1983/91.
[3]HMF-TP-004-10 could not be located at the time of the survey; located by visual observation from existing features.

Table 15.2
Estimated Infiltration Rates – Hyak Maintenance Facility

Facility Location	Approximate Elevation, feet				Soils Information[1]							Hydraulic Conductivity feet/day[2]	Hydraulic Gradient, unitless[3]	Infiltration Rate, feet/day[4]
	Existing Ground	Proposed Ground	Bottom of Trench	High Groundwater Level[5,6]	Boring ID	Depth, feet	Description	D_{10}, mm	D_{60}, mm	D_{90}, mm	Fines Fraction			
East Swale	2545	2540	2539.3	2534	HMF-001-08	5	Silty SAND with gravel	0.04	0.9	15	0.203	23	0.031	**0.6**
	2545	2540	2539.3	2534	HMF-001-08	10	Silty SAND with gravel or silty GRAVEL with sand.	0.03	5.4	20	0.156	27	0.030	**0.7**
	2545	2540	2539.3	2534	HMF-002-08	5	Silty GRAVEL with sand	0.04	8.8	30	0.14	26	0.030	**0.6**
	2545	2540	2539.3	2534	HMF-002-08	10	Silty SAND with gravel	0.04	2.7	25	0.184	20	0.031	**0.5**
	2545	2540	2539.3	2534	HMF-003-08	2.5	Well graded GRAVEL with sand	0.15	9.0	22	0.068	76	0.027	**1.7**
	2545	2540	2539.3	2534	HMF-003-08	15	Silty SAND with gravel	0.03	1.4	9.5	0.205	26	0.030	**0.6**
	2545	2540	2539.3	2534	HMF-004-08	3.8	Silty SAND with gravel	0.02	5.5	20	0.167	24	0.030	**0.6**
	2545	2540	2539.3	2534	HMF-004-08	7.5	Silty SAND with gravel	0.01	4.3	18	0.206	20	0.031	**0.5**
	2545	2540	2539.3	2534	HMF-004-08	8.8	Silty SAND with gravel	0.01	2.7	15	0.215	20	0.031	**0.5**
	2545	2540	2539.3	2534	HMF-005-08	4	Silty GRAVEL with sand	0.02	6.3	20	0.142	29	0.030	**0.7**
	2545	2540	2539.3	2534	HMF-005-08	11.5	Silty SAND with gravel	0.08	5.0	15	0.1	50	0.028	**1.1**

Table 15.2 (continued)
Estimated Infiltration Rates – Hyak Maintenance Facility

Facility Location	Approximate Elevation, feet				Soils Information[1]							Hydraulic Conductivity feet/day[2]	Hydraulic Gradient, unitless[3] Existing Ground	Infiltration Rate, ft/day[4] Proposed Ground
	Existing Ground	Proposed Ground	Bottom of Trench	High Groundwater Level[5,6]	Boring ID	Depth, feet	Description	D_{10}, mm	D_{60}, mm	D_{90}, mm	Fines Fraction			
West Swale	2538.7	2536	2535	2530	HMF-TP-001-09	2.5	Well graded SAND with gravel	0.08	2.5	6.5	0.096	62	0.026	**1.3**
	2538.7	2536	2535	2530	HMF-TP-001-09	6	Silty GRAVEL with sand	0.04	7.0	25	0.113	32	0.028	**0.7**
	2538.7	2536	2535	2530	HMF-TP-001-09	11	Well graded GRAVEL with sand	0.24	12.9	40	0.057	79	0.026	**1.6**
	2533.7	2536	2535	2530	HMF-TP-002-09	6.5	Silty SAND with gravel	0.14	3.9	20	0.078	62	0.026	**1.3**
	2533.7	2536	2535	2530	HMF-TP-002-09	10	Poorly graded GRAVEL with sand	0.61	7.9	25	0.018	635	0.021	11
	2532.2	2536	2535	2530	HMF-TP-003-09	2.5	Silty SAND with gravel	0.06	3.1	15	0.109	42	0.027	**0.9**
	2532.2	2536	2535	2530	HMF-TP-003-09	7	Silty SAND with gravel	0.03	2.4	12	0.149	32	0.028	**0.7**
	2532.2	2536	2535	2530	HMF-TP-003-09	11	Well graded GRAVEL with sand	1.40	19.3	35	0.011	22,000	0.015	260

Notes
Bold indicates calculated infiltration rate is <2 feet per day
1. Data obtained from laboratory testing of soil samples.
2. Calculated using Equation 4-12 of WSDOT Highway Runoff Manual, $\log_{10}(K_{sat}) = -1.57 + 1.90D_{10}+0.015D_{60}-0.013D_{90}-2.08f_{fines}$ where K_{sat} is the hydraulic conductivity in centimeters per second, D_{10}, D_{60} and D_{90} are grain sizes in mm for which 10, 60 and 90 percent of the sample is finer, and f_{fines} is the fraction of soil (by weight) that passes the number 200 sieve.
3. Calculated based on Equation 4-22 of WSDOT Highway Runoff Manual, $i =((D_{wt} + D_{trench})/78K_{equiv}^{0.05}$, where D_{wt} is the depth from the base of the infiltration facility to the water table in feet and D_{trench} is the depth of water in the facility in feet. Assumed D_{trench} = 0.5 feet (one-half of trench depth) in all the infiltration facilities. Assumed the slope of the trench does not exceed 0.5% and $K_{equiv} = K_{sat}$.
4. Calculated based on Equation 4-23 of WSDOT Highway Runoff Manual, $f = K_{equiv}iCF_{Silt/bio}$, where f is the infiltration rate of water through a unit cross-section in feet per day, K_{equiv} is the hydraulic conductivity in feet per day, and i is the hydraulic gradient (unitless). $CF_{Silt/bio}$ = 0.8 was used, which assumes a low potential for biofouling and a low degree of long-term maintenance/performance monitoring.
5. High water level information was not obtained at the east swale and it was estimated using information from the nearby boring, HMF-004-08. High water level at the west swale was based on the information from the test pits HMF-TP-001-09, HMF-TP-002-09 and HMF-TP-003-09 that were excavated at the swale location.
6. The water table at HMF-TP-001-09 at the west swale location may be periodically above bottom of the trench. An assumed water table one foot below bottom of the trench was used to calculate the infiltration rate. The estimated high water level at that location is approximately El. 2535.7 feet.

Table 15.3
Estimated Infiltration Rates – Crystal Springs Sno-Park

Boring/ Test Pit ID	Approx. Existing Ground Surface Elevation (feet)	Estimated High Groundwater Elevation (feet) (Note 1)	Soil Information (Note 4)						K_{sat} (feet/day) (Note 5)	Infiltration Rate (ft/day) as a Function of Depth (feet) to High Groundwater Below Base of Facility (Notes 2,3)				
			Sample Depth (feet)	Sample Description	D_{10} (mm)	D_{60} (mm)	D_{90} (mm)	Fines Fraction		1	2	3	5	10
INF-TP-007-10	2429.80	2426.0	4.0	Silty GRAVEL with Sand	0.08	7.3	24	0.1	41	**0.5**	**0.9**	**1.2**	**1.9**	3.7
			6.5	Silty GRAVEL with Sand	0.08	7.2	24	0.1	41	**0.5**	**0.9**	**1.2**	**1.9**	3.7
INF-TP-009-10	2424.2	2422.2	6.0	Silty GRAVEL with Sand	0.05	5.8	22	0.13	32	**0.4**	**0.7**	**1.0**	**1.5**	2.9
			9.5	Well-graded GRAVEL with Sand	0.18	20.4	33	0.065	92	**1.1**	**1.9**	2.6	4.2	7.9
INF-010-10(OW)	2435.8	2426.0	10.0	Well-graded SAND with Gravel	0.11	5.7	24	0.083	49	**0.6**	**1.0**	**1.5**	2.3	4.4
INF-TP-011-10	2431.8	2422.2	6.0	Well-graded GRAVEL with Sand	0.09	9.2	23	0.095	49	**0.6**	**1.0**	**1.5**	2.3	4.4
			9.0	Silty SAND with Gravel	0.05	5.3	16	0.11	42	**0.5**	**0.9**	**1.2**	2.0	3.7
INF-012-10(OW)	2427.9	2426.0	10.0	Silty GRAVEL with Sand	0.05	9.5	31	0.11	31	**0.4**	**0.7**	**0.9**	**1.5**	2.8
INF-TP-013-10	2429.1	2422.2	3.0	Poorly-graded GRAVEL with Sand	0.22	56.0	70	0.067	123	**1.5**	2.5	3.5	5.5	10.4
			6.0	Silty GRAVEL with Sand	0.03	8.6	32	0.15	22	**0.3**	**0.5**	**0.7**	**1.1**	2.0
			9.0	Poorly-graded GRAVEL with Sand	0.13	13.2	33	0.075	55	**0.7**	**1.2**	**1.6**	2.5	4.9
INF-TP-014-10	2425.6	2422.2	9.0	Silty SAND with Gravel	0.06	3.0	23	0.11	33	**0.4**	**0.7**	**1.0**	**1.5**	3.0
			12.0	Silty SAND with Gravel	0.06	4.2	27	0.12	29	**0.4**	**0.6**	**0.9**	**1.4**	2.6
INF-015-10	2452.7	2422.2	25.0	Well-graded GRAVEL with Sand	0.08	8.1	23	0.098	45	**0.6**	**0.9**	**1.3**	2.1	4.0

Notes

Bold indicates an estimated infiltration rate of less than 2 feet per day.

1. The high groundwater elevation was interpreted to be 5 feet above the surveyed water elevation on July 1, 2010 in Pond 1 (2417.2) for INF-TP-009, 011, 013, 014, 015-10 and in Pond 2 (2421.0) for INF-TP-007-10, INF-010-10(OW) and INF-012-10(OW).
2. Calculated based on Equation 4-23 of WSDOT Highway Runoff Manual.
3. Assumed depth of water in facility equal to 0.5 feet and a value of $CF_{Silt/bio}$ = 0.8, which assumes a low potential for biofouling and a low degree of long-term maintenance/performance monitoring.
4. D_{10}, D_{60} and D_{90} are grain sizes in mm for which 10, 60 and 90 percent of the sample by weight is finer, and fines fraction is the fraction of soil by weight that passes the number 200 sieve.
5. Calculated based on Equation 4-12 of WSDOT Highway Runoff Manual.

Table 16.1
Summary of Borings for MP 59.7 Unnamed Creek Crossing

Boring ID	Date Drilled		Elevation (feet)[1]	Project Datum[2]		Total Depth (feet)
	Started	Completed		Northing (feet)	Easting (feet)	
CUL-015-07(OW)	9/13/2007	9/13/2007	2536.2	1,060,768	1,757,760	36.5
CUL-016-07	9/12/2007	9/12/2007	2524.7	1,060,662	1,757,638	41.5
CUL-019-10	5/10/2010	5/11/2010	2522.7	1,060,708	1,757,696	51.0
CUL-020-10	7/15/2010	7/16/2010	2530.5	1,060,758	1,757,690	51.0

Notes
[1]Vertical datum is the North American Vertical Datum (NAVD) 1988
[2]Horizontal datum is based on the Washington Coordinate System, South Zone, and North American Datum (NAD) 1983/91

Table 16.2
Global Stability Analysis Results for Wall 20

Station	Toe Berm[1]	No. blocks	Total Wall Height, feet	Exposed Wall Height, feet	Factors of Safety			Figures
					Static, High Water	Static, Low Water	Seismic	
W20 10+05	N	4	10	6.7	1.13	--	0.96	H.10.1 – H.10.2
	N	5	12.5	6.7	1.30	1.46	1.09	H.10.3 – H.10.5
	N	6	15	6.7	1.41	--	1.19	H.10.6 – H.10.7
	Y	4	10	6.7	1.22	--	--	H.10.8
	Y	5	12.5	6.7	1.51	--	1.23	H.10.9 – H.10.10

Notes
[1] Toe berm size: 5 foot wide, 10 foot high with 2H:1V side slopes

Table 16.3
Unfactored Axial Compressive Resistance for 6-inch Diameter Micropiles

Pile Depth, feet[1]	Unfactored Axial Capacity, kips	
	Casing Depth = 5 feet	Casing Depth = 10 feet
0	0	0
5	0	0
10	39	0
15	79	39
20	118	79
25	157	118
30	196	157
35	236	196
40	275	236
45	314	275
50	353	314

Notes
[1]Depth below ground surface

Table 16.4
Design Parameters for Lateral Load Analysis Using DFSAP – MP 59.7 EB Crossing

Layer	Top of Layer Elevation	Soil Type	Layer Thickness (Note 1)	Effective Unit Weight	Effective Friction Angle	Strain, e_{50}	Cohesion	Potential Liquefaction
Units	feet		feet	pcf	degrees	unitless	psf	
1	2520	Sand	4	130	38	0.005	--	No
2	2516	C-φ	4	23	25	0.01	50	No
3	2512	Sand	>20	68	38	0.005	--	No

Notes
1. Groundwater assumed to be at El. 2516.
2. The ground surface slopes are assumed to be -7° in the longitudinal direction and-26° in the transverse direction, where negative values indicate the slope is downward in the direction of applied load.

Table 16.5
Recommended Resistance Factors for Wall Design

Limit State	Resistance Factor, φ		
	Bearing Resistance	Friction Resistance to Sliding	Passive Pressure Resistance to Sliding
Strength	0.45	0.9	0.5
Service	1.0	NA	NA

FIGURES