I-90 Snoqualmie Pass East Project

Phase 1C – Rock Slope Engineering Report
2008 Geotechnical Program

Volume 1 of 2

April 2009

Washington State Department of Transportation

U. S. Department of Transportation
Federal Highway Administration

Title VI

WSDOT ensures full compliance with Title VI of the Civil Rights Act of 1964 by prohibiting discrimination against any person on the basis of race, color, national origin or sex in the provision of benefits and services resulting from its federally assisted programs and activities. For questions regarding WSDOT's Title VI Program, you may contact the Department's Title VI Coordinator at (360) 705-7098.

Americans with Disabilities Act (ADA) Information

If you would like copies of this document in an alternative format – large print, Braille, cassette tape, or on computer disk, please call (360) 705-7097. Persons, who are deaf or hard of hearing, please call the Washington State Telecommunications Relay Service, or Tele-Braille at 7-1-1, Voice 1-800-833-6384, and ask to be connected to (360) 705-7097.

Acknowledgements

I-90 Snoqualmie Pass East
Agreement No. 9764
Task Order CP

Phase 1C - Rock Slope Engineering Report
2008 Geotechnical Analyses and Reporting

Submitted to:
WSDOT
I-90 Project Office

Prepared by:
Wyllie & Norrish Rock Engineers Inc.
17918 NE 27th Street
Redmond, WA 98052

NORMAN I. NORRISH
STATE OF WASHINGTON
37407
REGISTERED
PROFESSIONAL ENGINEER
EXPIRES 8/29/09

Norman I. Norrish, P.E.
Wyllie & Norrish Rock Engineers Inc.
Principal Geologist

Balin Strickler, LG
Fisher & Strickler Rock Engineering, LLC
Principal Geologist

Robert L. Burk, Ph.D., LG, LEG
Burk GeoConsult, LLC
Principal Geologist

TABLE OF CONTENTS
VOLUME 1 OF 2: SECTIONS 1-11, TABLES, AND FIGURES

TABLES

FIGURES

VOLUME 2 OF 2: APPENDICES

ACRONYMS / ABBREVIATIONS

ADAS	Automatic data acquisition system
ASTM	American Society for Testing and Materials
bgs	Below Ground Surface
COBL	Crux Oriented Borehole Logging
CRUX	Crux Subsurface, Inc.
CT	California Test
cy	Cubic yard
DM	WSDOT Design Manual
EB	Eastbound
ft	foot
GSA	Geologic Society of America
I-90	Interstate 90
ISRM	International Society of Rock Mechanics
JRC	Joint Roughness Coefficient
MP	Milepost
MRE	Material Resource Evaluation
MSL	Mean sea level
NAD	New Alignment Design
NAF	New Alignment Feasibility
ODOT	Oregon Department of Transportation
pcf	pounds per cubic foot
PI	Plasticity index
PLT	Point Load Testing
ppm	Parts Per Million
PS&E	Plans, Specifications and Estimates
psf	Pounds per square foot
psi	Pounds per square inch
sf	Square foot
SPT	Standard penetration test
Sta.	Station
TCD	Tri-County Drilling
TDR	Time Domain Reflectometry
TO	Task Order
UCS	Unconfined compressive strength
URS	URS Corporation
VWP	Vibrating Wire Piezometer
WB	Westbound
WSDOT	Washington State Department of Transportation

EXECUTIVE SUMMARY

The primary objective of this work package was to develop design recommendations for the proposed and existing rock cuts for that portion of the Interstate 90 (I-90) Snoqualmie Pass East Project designated as Phase 1C. The design requirements included guidance for slope templates and for stabilization measures necessary to provide margins of stability in keeping with Washington State Department of Transportation (WSDOT) customary practices for interstate construction.

Field investigations included geotechnical drilling, downhole surveys, structural geologic mapping, and the installation of instrumentation for measurement of groundwater and slope displacements. Data from these activities was coupled with laboratory testing as input to detailed slope stability analyses. Observation and historical performance of existing slopes was used to "calibrate" new designs wherever possible.

The new slope cuts for Phase 1C defined the project limits for this study, namely, 1352+50 to 1473+00 westbound (WB). New cut slopes are required for specific intervals of the 12,050-foot alignment. To facilitate design this interval was subdivided into eleven 'Design Sectors' wherein geologic and geometric conditions were reasonably consistent. For Phase 1C these sectors were designated from VIII to XVIII progressing from the Snowshed to the eastern limit of the project. Design recommendations for Phase 1C sectors XIII to XVI pertain to Slide Curve and were previously submitted under separate cover (Wyllie & Norrish, 2009a).

The primary results of the study were to confirm the feasibility of higher cut slopes required by the WSDOT alignment changes. The station interval between the snowshed and Slide Curve will require extensive stabilization due to the poor quality of much of the weathered volcanic rock. Pattern slope reinforcement, shotcrete, slope drape and slope drainage will be necessary to excavate and stabilize ¼H:1V slopes. Extra rock stabilization is required to protect the eastern snowshed portal and associated overpass structure. Rock cuts at the east end of the project offer flexibility as to inclination due to flatter topography above the cuts. Therefore, provided adequate right-of-way is available, the proposed cuts can be designed to flatter inclinations to maximize stability.

Templates for the overburden above the rock cuts are variable dependent on the overburden thickness and the existing topography. Local thick areas of overburden will require extensive back slope cuts.

Successful excavation and long term performance of the proposed rock cut slopes will require that stabilization measures be installed as all cuts are brought down. This will require close coordination between owner and contractor and to some extent will restrict the efficiency of rock excavation.

Rock conditions in volcanic terrains are highly variable and defy accurate characterization, irrespective of drilling and mapping intensity. Provision must be made to make design changes to rock slopes during excavation as actual rock conditions are encountered. Such changes will primarily relate to types and/or frequency of stabilization measures such as rock bolts and shotcrete. Site geotechnical engineering during construction coupled with predictive slope displacement monitoring are recommended to recognize and mitigate slope instability in a timely manner. To this end, extensive documentation of the geologic conditions inherent to the geotechnical design recommendations is embodied in this report.

1.0 INTRODUCTION

WSDOT has identified that the part of 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 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.

WSDOT South Central Region contracted URS Corporation (URS) as General Engineering Consultant for the project, under WSDOT Agreement No. Y-9764, dated February 14, 2006. URS subcontracted Wyllie & Norrish Rock Engineers Inc. to provide conceptual-level, leading to design-level, engineering recommendations pertaining to the new and existing rock slopes for this project. The work by Wyllie & Norrish was performed under URS Master Services Agreement No. 131491-UB, Work Order No. 131491-UB.

Wyllie & Norrish prepared this design report under task order (TO) CP – 2008 Geotechnical Analyses and Reporting. The field exploration supporting the report was performed under TO CJ – 2008 Geotechnical Exploration Program. Other subconsultants to URS worked directly with Wyllie & Norrish and contributed significantly to the rock slope studies, including Fisher & Strickler Rock Engineering LLC, Burk GeoConsult LLC and Haneburg Geoscience.

The report deals with a portion of the project alignment previously referred to as the "New Alignment," so named following refinement and selection of the preferred alignment in early 2007. At that time little geologic and engineering attention had been directed at the rock slopes for the preferred alignment and thus a 2007 work activity was defined for Wyllie & Norrish and termed "New Alignment Feasibility" or NAF.

2.0 SCOPE OF SERVICES

A conceptual level report was submitted by Wyllie & Norrish in July 2008 entitled "Technical Memorandum – Slope Stability Analyses for New Alignment" (Wyllie & Norrish, 2008a). Subsequent to that report the overall Hyak to Keechelus Dam realignment project was reorganized by WSDOT into the Phase 1B and Phase 1C projects to facilitate contracting and construction scheduling. This report deals with the Phase 1C portion of the project, and a previous design report pertaining to Phase 1B was issued as Wyllie & Norrish 2009b under separate cover.

The general scope of the rock slopes project included planning and execution of the site characterization studies, testing, analyses and reporting. The overall objective of the work was to develop recommendations for cut slopes between Station (Sta.) 1305+00 and 1396+50 WB and from 1411+00 WB to the east end of the project (Figure 1). Project activities to support this objective include:

Task Order CJ

1. Develop exploration plan.
2. Perform structural mapping.
3. Perform on-site reconnaissance in support of exploration activity.
4. Provide drilling coordination and inspection.
5. Perform field and laboratory testing.
6. Prepare or update subsurface profiles and cross sections.

Task Order CP

1. Perform rock slope engineering analyses including data compilation, presentation and stability analyses.
2. Prepare geotechnical design criteria for slope design templates.
3. Design rockfall catchment widths in keeping with general State practice.
4. Provide recommendations for construction.
5. Prepare draft and final reports for Phase 1B rock cuts (Sta. 1305+00 to 1352+50).
6. Prepare draft and final reports for Phase 1C rock cuts (Sta. 1352+50 to end of project).
7. Prepare draft and final Material Source Report.
8. Provide project management activities.

As previously described this design report fulfills the reporting requirement for Item 6 above covering Phase 1C. Separate reports will be submitted for Phase 1B and for the Material Source evaluation.

This report is intended as guidance for highway designers in the preparation of Plans, Specifications and Estimates (PS&E). It is strongly recommended that Wyllie & Norrish have the opportunity to review the PS&E documents to ensure accurate incorporation of the geotechnical recommendations herein.

This report is presented in two volumes. For ease of reference the design report (volume 1) has been subdivided into topical sections. Project background and project setting are summarized in Sections 3.0 and 4.0. The field and laboratory programs are discussed in Sections 5.0 and 6.0. In Section 7.0 the salient geological and geotechnical features of each Design Sector for the Phase 1C project are described. The detailed stability analyses and design recommendations are presented in Sections 8.0 and 9.0, respectively. Section 10.0 covers construction issues while Section 11.0 documents the references cited in the report. An extensive database of background information is contained within the Appendices (volume 2). In this regard, it is pointed out that the Phase 1C report has been structured to be self-contained in terms of background data by incorporating data developed from earlier studies. However, data developed for the 1B portion of the alignment and for the Materials Source Report are only partially incorporated and the reader is referred to the companion reports for a comprehensive compilation of that data.

3.0 PROJECT BACKGROUND

As mentioned above, Wyllie & Norrish has conducted extensive rock slope feasibility-level studies of various portions of the overall project alignment since 2006 (Wyllie & Norrish, 2007a, 2007b, 2008a, 2008b, 2009a, 2009b). An important detail for readers of the previous reports is that the station reference was changed from eastbound (EB) for the earlier studies to westbound (WB) for the current design study. To avoid ambiguity, this report uses the convention xxxx+yy WB.

Two areas of the project are often referred to herein by their colloquial names. The Milepost 57 rock cut in the Phase 1B portion of the project is commonly referred to as “Jenkins’ Knob”. Similarly, the MP 59 vicinity is referred to as “Slide Curve” so named for the rock slope failure that occurred in 1957 during the original construction of I-90.

The most pertinent previous studies to the Phase 1C report are the Slide Curve reports (Wyllie & Norrish, 2007a and 2009a) and the New Alignment Feasibility (NAF) memorandum (Wyllie & Norrish, 2008a). While the Phase 1C portion of the overall project includes Slide Curve, the project feasibility studies mandated that rock slope design issues be resolved at that location prior to the remainder of Phase 1C. Accordingly, separate standalone reports were issued for Slide Curve. The NAF memorandum was a feasibility level study that covered the remainder of Phase 1C with the exceptions of Slide Curve and the east end of the project. The drilling and mapping database that formed the basis of the NAF memorandum was consistent with feasibility-level objectives. The current study has expanded the database and the better understanding of geologic conditions has led to localized rock slope design changes from those presented in the NAF memorandum.

Intervals of the alignment within which geologic and geometric slope conditions are relatively consistent have been designated as “Design Sectors”. These sectors are numbered with ascending Roman numerals from the WB limit (Sta. 1305+00) where cut slopes start. Between the NAF memorandum and the current study there has been a slight refinement of these Design Sector station limits and nomenclature as follows:

Table 1
Sector Definition

NAF Report			Phase 1C Report		
Sector No	Sta. Start	Sta. End	Sector No	Sta. Start	Sta. End
Snowshed	1351+75	1363+50	Snowshed	1352+50	1363+50
IX	1363+50	1366+75	IX	1363+50	1366+00
X	1366+75	1376+00	X	1366+00	1375+50
XI	1376+00	1383+00	XI	1375+50	1382+50
XII	1383+00	1396+50	XII	1382+50	1396+50
XIII to XVI	Not covered		XIII to XVI	See Slide Curve Reports	
XVII	Not covered	Not covered	XVII	1442+50	1468+00
XVIII	Not covered	Not covered	XVIII	1468+00	1480+00

The plans, profiles and cross sections depicted in this report are based on overburden templates prepared by WSDOT from geotechnical guidance provided in the Draft 1C report. In some cases their cross sections do not show the overburden template and simply project the rock cut template; or alternatively they show two templates. These inconsistencies reflect the difficulty of matching a consistent template to inconsistent overburden and topographic conditions and while suitable for quantity takeoffs, the templates will have to be adjusted to site-specific conditions during excavation.

For the purposes of this report, directions have been referenced as westbound or eastbound relative to I-90. In the text these directions are often shortened to west and east with the understanding that they are not directly tied to compass direction. Note that the general orientation of I-90 is northwest – southeast through the project zone (Figure 1).

4.0 PROJECT SETTING

4.1 TOPOGRAPHY AND CLIMATE

The project area is situated in the Cascade Mountain Physiographic Province of Washington State on the northeast side of a glaciated valley. The I-90 highway is bordered to the west by Keechelus Lake, to the east by Keechelus Ridge and trends north-northwest through the project area (figure 1). The highway is located at an approximate elevation of 2,550 feet above mean sea level (MSL) with Keechelus Ridge rising to about 4,000 feet MSL. It was necessary to cut into the valley wall to build the highway. Initial topographic modification during construction of the early wagon road and Sunset Highway (1915) was likely limited to minor cut and fill slopes.

Larger cuts and fills were completed during the construction of I-90 commencing in the late 1950's.

Precipitation is highly variable from west to east across the Cascade Mountain Range. A 30+ year average of precipitation data from a monitoring station (approximately 3 miles south-southwest of Jenkins' Knob i.e. Lake Keechelus Station) shows an average yearly rainfall total of approximately 70 inches compared with approximately 105 inches at Snoqualmie Pass. Similarly, snowfall at the Lake Keechelus station ranged from 53 to 452 inches with an average of approximately 220 inches and the Snoqualmie Pass station snowfall ranged from 172 to 800 inches with an average of approximately 460 inches of snowfall (http://www.wrcc.dri.edu/). Variations in temperature are common throughout the winter with rapid snowmelt and rain-on-snow events, which generate large amounts of runoff, occurring frequently throughout the project area.

Mean maximum temperatures ranged from 52.2 degrees (F) at the Lake Keechelus station to 51.0 degrees Fahrenheit at Snoqualmie Pass. Mean minimum temperatures ranged from 31 degrees at Lake Keechelus to 33.2 degrees at Snoqualmie Pass.

4.2 REGIONAL GEOLOGY

Glacial processes dominate the surficial geology of the area at Snoqualmie Pass as evidenced by open parabolic shaped valleys and deposits of alpine till. During the last glaciation, an alpine icecap accumulated in the Snoqualmie Pass area and lobes of ice advanced to the east and west. The valley that now contains Keechelus Lake is one of these glaciated valleys and was dammed in part by glacial deposits and in part by an earthfill structure completed in 1917. Glacial scouring removed many of the previously existing surficial deposits and scoured the rock outcrops. Glaciers take the path of least resistance and the ice in the Keechelus Lake valley was no exception; it followed a fault that trends down the Gold Creek valley into the main part of the valley (Tabor et al, 2000).

Glacially over-steepened topography is frequently subject to landslide activity after the glaciers recede, when the lateral support of the slopes that had been provided by the ice is removed. As much as glacial action scours upstream rock exposures and plucks rock downstream (stoss and lee topography), removes older surficial deposits, and erodes weaker rock, it can also be an effective weathering agent for in-place rock. This weathering is a function of water and ice wedging. The interface between the ice and the valley wall is wet and high-pressure water penetrates discontinuities. Frost weathering is another ongoing phenomenon that affects rocks by ice accumulation at the leading edge of a freezing front. Not only was this process active in glacial periods, but it continues today perhaps at an even greater rate than in the past. Hallet (2006) and Walder and Hallet (1985) among others have shown that freeze-thaw cycles, although widely considered significant to rock weathering are likely not important in nature. However, availability of water and consistent temperatures in 31 to 33 degree Fahrenheit range, lead to freezing fronts that do have the ability to propagate cracks in rock. Optimum temperature conditions are present in the project area, suggesting that rapid discontinuity propagation and weathering related to freezing is a currently active process at Snoqualmie Pass.

The geologic regime for the project site provides the context within which to evaluate geologic features at the scale important to rock slope stability. This is especially true in the Snoqualmie Pass area where the bedrock geology comprises older accreted terranes overlain by younger Tertiary rocks of the Cascade volcanic arc. Eocene interbedded volcanic and nonmarine sedimentary rocks north of Jenkins' Knob and south of Slide Curve (Naches Formation) represent deposition immediately prior to the development of the volcanic arc. These older rocks are overlain by late Eocene and younger Tertiary rocks of the Cascade volcanic arc, including the altered andesite and dacite tuff breccias and volcaniclastic rocks of the Keechelus Lake member of the Ohanapecosh Formation. Ohanapecosh rocks underlie Jenkins' Knob and extend to the south to Slide Curve. Hydrothermal fluids associated with Cascade arc igneous intrusive bodies contributed to the alteration of the Naches and the Ohanapecosh Formations.

Within the Lake Kachess drainage immediately to the east of the project area, the Straight Creek fault is present in the subsurface. The fault, and its probable extensions in Canada and Alaska, is a major strike-slip feature that had an estimated 180-190 kilometers (112-118 miles) of movement during early Tertiary time. By mid-Oligocene, movement on the fault had slowed and the Cascade volcanic arc became well established, flooding the Snoqualmie Pass area with a variety of volcanic rocks and burying the no longer active Straight Creek fault (Tabor et al, 2000).

Intrusive igneous rocks, likely related to magmas associated with the Cascade Arc volcanoes, are also present in the Snoqualmie Pass area. The largest of these intrusive bodies is the Snoqualmie batholith that outcrops over an area of approximately 580 km^2 (224 mi^2) including areas north and west of Snoqualmie Pass (Tabor, et. al., 2000). Hydrothermal fluids associated with these intrusive bodies were likely significant in the alteration of the earlier Tertiary volcanic rocks, such as the Ohanapecosh Formation, that they intruded. In particular, well jointed rocks, those that are highly permeable, or those that have a reactive mineralogy are highly susceptible to hydrothermal alteration and subsequent weathering that can be significant to rock mass stability. Minerals (sericite, zeolite, epidote, and chlorite) and devitrified volcanic glass found in these rocks are consistent with this kind of low-grade metamorphic alteration.

The materials overlying the bedrock (surficial deposits) comprise Quaternary alpine glacial till that is frequently associated with Recent colluvial and alluvial deposits.

5.0 FIELD PROGRAM

5.1 PHASE 1C INVESTIGATION PROGRAM

This section summarizes the methods and objectives of the field program over the last three (3) years and highlights the methods used in 2008. The 2008 field program included a Materials Resource Evaluation (MRE) completed by Burk GeoConsult, LLC (Burk GeoConsult, 2009). Subsurface data obtained from the MRE investigation has been provided to Wyllie & Norrish for inclusion in this report; however, a complete MRE report has been submitted under separate cover.

The field program undertaken for feasibility and design-level engineering evaluation of the Phase 1C project consisted of geologic mapping, assessment of discontinuities and a subsurface investigation and instrumentation program. Field program objectives were as follows:

- Geologic Mapping
    - Generation of a Design Sector map
    - Observation and definition of various engineering domains
    - Review of existing discontinuity data and collection of supplemental data
- Structural Mapping
    - Supplemental discontinuity orientation data to that previously collected by WSDOT and others
    - Acquiring discontinuity data using Sirovision that included “hard to reach” areas
    - Analysis of discontinuity characteristics
- Subsurface Investigation and Instrumentation
    - Investigation of subsurface geologic and structural conditions for rockmass characterization of the observed Design Sectors
    - Assessment of discontinuity conditions in the subsurface
    - Downhole optical and acoustical televiewer surveys of each borehole to obtain supplemental discontinuity data for structural analysis
    - Installation of vibrating wire piezometers (VWP) for long-term monitoring of hydraulic head changes both among boreholes and between separate zones within selected boreholes
    - Installation of time domain reflectometry (TDR) cable for upslope stability monitoring during construction activities

5.2 GEOLOGIC MAPPING

Geologic mapping of the Phase IC area complemented the regional geologic mapping information available from published information, URS regional mapping efforts, and the site-specific information available from WSDOT and work by other consultants. The purpose of geologic mapping activities undertaken was to:

- Develop a sufficient understanding of the geologic framework of the site and the uncertainties with respect to subsurface conditions to provide final recommended borehole locations
- Define the discontinuity types that may affect the feasibility of the project

- Use geologic information on rock characteristics to define engineering domains useful in development of cut slope support requirements

Orthophotographs inscribed with a proposed cut line provided by WSDOT and dated 6/21/06 formed the initial base for mapping completed in 2006 (Wyllie & Norrish 2007a,b). Subsequent alignment changes resulted in the updated maps provided herein. Stereographic vertical aerial photographs examined as part of this work included black and white images taken 9/12/45, 9/4/57, and 10/20/76, and false-color infrared images taken 8/18/83 and 8/25/85. A 1944 orthophotograph of a portion of the project area from the Corps of Engineers was also reviewed.

5.3 STRUCTURAL MAPPING

Detailed structural mapping of exposed outcrops along the I-90 project was completed in various phases between 2006 and 2008. This mapping effort included Sirovision structural mapping and was supplemented with traditional detailed structural mapping. Mapping efforts focused on identifying the number of joint sets, joint orientations, and the engineering characteristics of the discontinuities within the exposed rock mass. All structural mapping utilized a declination value of approximately 17.5 degrees East and the data provided has been presented as referenced to true north.

In support of rock slope design, structural mapping of the existing rock cuts throughout Phase IC was accomplished utilizing 3-D models of select rock exposures created from digital photogrammetry. Siro3D was utilized for 3-D model creation and SiroJoint for structural mapping and analysis. Additional information about the software is available at www.sirovision.com. Haneberg Geosciences of Seattle, Washington was contracted to complete the remote structural mapping. This included collection of photographs in the field, generation of the 3-D models, mapping and analysis of the models and generation of a final report. Sirovision mapping was completed as part of the field investigation in 2006 and 2007 and the reports are provided in Appendix A of this report.

Traditional structural mapping was completed in 2008 along the existing cut slope of I-90 using a Brunton™ by Wyllie & Norrish personnel in general accordance with International Society of Rock Mechanics (ISRM) Suggested Methods for Quantitative Description of Discontinuities in Rock Masses (Brown, 1981) and the FHWA Rock Slopes Manual (FHWA, 1998). Select window mapping was completed at approximately 100-foot intervals along the outcrop. Discontinuities were mapped from road grade to approximately six feet above grade. The mapping was completed to complement the Sirovision mapping, provide information on discontinuity condition (i.e.: aperture, infilling, roughness, JRC) and to validate the mapping completed using the SiroJoint software. Mapping data sheets and a photograph log of the locations are provided in Appendix B.

5.4 SUBSURFACE INVESTIGATION - DRILLING PROGRAM

The objective of the 2008 drilling program was to further characterize the engineering and geologic subsurface conditions observed during the previous investigations. The proposed location, borehole depths and type of instrumentation was developed by Wyllie & Norrish based on the results of the 2006 and 2007 field program, the existing data available, the proposed realignment and the surface geologic mapping activities. Similar to the prior years, the proposed subsurface investigation (Wyllie & Norrish, 2008a) was reviewed and approved by WSDOT and URS. An additional ten (10) rock slope boreholes and four (4) MRE boreholes were advanced during the Phase IC 2008 field program totaling 772.0 feet and 403.3 feet of drilling respectively. The total drilling completed for Phase IC during the 2006 and 2007 field seasons was 149.3 feet and 1,587.5 feet, respectively. Total drilling footage advanced during the three years was approximately 2,910 feet.

Three types of borehole nomenclature are represented in this report. The typical sequence is XXX-000-00. The first identifier are letters with H=hole, RKS=rock slopes and MRE=material resource evaluation. The second identifier provides numbers indicating the number of the borehole in the sequence. The third identifier is a two digit representation of the year the borehole was completed. Therefore, RKS-09-07 would be a rock slopes hole, #9 in the RKS sequence, drilled in 2007.

Table 2 provides a summary of the borehole locations from the 2006, 2007, and 2008 field programs while Figure 2 shows the borehole locations. Figures 3 through 10 provide Design Sector plans and profiles while Figures 11 through 24 provide detailed geologic cross sections of select boreholes.

Table 2 Borehole Locations and Instrumentation

Borehole #	Northing	Easting	Ground Surface Elevation (feet MSL)	WB Station	Offset (feet)	Borehole Total Depth (feet bgs)	Borehole Total Depth (feet MSL)	Vibrating Wire Piezometer Instrument Installation Data			Time Domain Reflectometer Instrument Installation Data (feet bgs)
								VWP Serial Number	Instrument Depth (feet bgs)	Instrument Elevation (feet MSL)	
2006 Feasibility Geotechnical Investigation											
H-110-06	1063054	1754649	2,697.6	1395+84	204'L	149.3	2,548.3	87189	72.2	2625	148
								87177	147.2	2550	
2007 New Alignment Feasibility Geotechnical Investigation											
RKS-08-07	1066520	1754208	2,616.8	1361+07	99'L	105.0	2,511.8	88129	52.2	2565	Not Installed
								92271	99.7	2517	
RKS-09-07	1066268	1754253	2,620.3	1363+62	80'L	120.0	2,500.3	88131	68.7	2552	Not Installed
RKS-10-07	1066134	1754301	2,653.3	1365+04	93'L	140.7	2,512.6	92272	137.7	2516	Not Installed
RKS-11-07	1065626	1754436	2,675.6	1370+29	98'L	145.4	2,530.2	No instruments were installed in this borehole			
RKS-12-07	1065421	1754471	2,648.0	1372+36	81'L	130.4	2,517.6	92273	128.1	2520	Not Installed
RKS-13-07	1064144	1754685	2,633.3	1385+14	85'L	108.5	2,524.8	92058	69.7	2564	Not Installed
								92079	105.7	2528	
RKS-14-07	1063707	1754623	2,624.4	1389+44	68'L	130.0	2,494.4	No instruments were installed in this borehole			
RKS-15-07	1063427	1754588	2,624.6	1392+26	81'L	163.2	2,461.4	88130	83.7	2541	Not Installed
								92269	157.7	2467	
RKS-16-07	1063305	1754573	2,631.6	1393+49	86'L	140.2	2,491.4	No instruments were installed in this borehole			
RKS-17-07	1063170	1754565	2,650.7	1394+83	102'L	114.0	2,536.7	No instruments were installed in this borehole			
RKS-18-07	1065467	1754551	2,745.5	1372+12	170'L	185.0	2,560.5	92268	182.7	2563	185
RKS-19-07	1063056	1754523	2,612.1	1396+03	79'L	105.1	2,507.0	92270	102.3	2510	Not Installed
2008 New Alignment Design Geotechnical Investigation											
RKS-38-08	737773	1426089	2,636.3	1366+98	84'L	119.5	2,516.8	N/A	N/A	N/A	119
RKS-39-08	736141	1426447	2,618.9	1383+52	98'L	101.0	2,517.9	No instruments were installed in this borehole			
RKS-40-08	735964	1426477	2,660.6	1385+23	128'L	139.5	2,521.1	92732	99.2	2561	75
								92734	137.2	2523	
RKS-41-08	735809	1426436	2,628.0	1386+73	96'L	89.5	2,538.5	N/A	N/A	N/A	89
RKS-42-08	732146	1429911	2,566.1	1447+46	94'L	35.5	2,530.6	No instruments were installed in this borehole			
RKS-43-08	731749	1430181	2,562.4	1452+27	97'L	35.0	2,527.4	No instruments were installed in this borehole			
RKS-44-08	730920	1430721	2,574.3	1462+00	109'L	45.0	2,529.3	94879	24.2	2550	Not Installed
								92939	39.7	2535	
RKS-45-08	730537	1430878	2,588.5	1466+02	123'L	50.0	2,538.5	No instruments were installed in this borehole			
RKS-46-08	730096	1431038	2,617.3	1470+87	120'L	85.0	2,532.3	94577	55.2	2562	Not Installed
								92938	83.2	2534	
RKS-47-08	729714	1431275	2,588.4	1475+70	100'L	72.0	2,516.4	No instruments were installed in this borehole			
2008 Material Resource Evaluation											
MRE-05-08	738312	1425968	2,620.6	1361+46	101'L	85.5	2,535.1	No instruments were installed in this borehole			
MRE-06-08	737605	1426141	2,652.9	1368+74	92'L	115.5	2,537.4	No instruments were installed in this borehole			
MRE-07-08	737395	1426182	2,654.8	1370+88	80'L	115.9	2,538.9	No instruments were installed in this borehole			

MRE-08-08	737187	1426229	2,621.6	1373+01	73'L	86.4	2,535.2	No instruments were installed in this borehole

Notes:

1. Survey data based on ground surveys completed by White Shield, Inc in 2006, 2007 and 2008. State Plane Coordinates, Vertical Datum = NAVD 1988
2. Location data provided in reference to local project datum
3. bgs = below ground surface
4. MSL = Mean Sea Level
5. VWP = Vibrating Wire Piezometer
6. TDR = Time Domain Reflectometry

The 2008 New Alignment Design (NAD) and MRE subsurface investigation was completed in one phase of work from September 8, 2008 to November 9, 2008. Because of limited drill rig availability from CRUX Subsurface, Inc. (Crux) and the drilling production schedule, URS subcontracted Tri-County Drilling (TCD) based in San Diego, California to begin the drilling program on the Phase IB project scope. TCD commenced drilling with a 2800 SIMCO skid-mounted drill on September 9' 2008. WSDOT subcontractor, Crux based in Spokane, Washington, mobilized to site on October 7, 2008 with platform-mounted Burley 4500 and Burley 5500 drill rigs. Crux commenced drilling on October 8, 2008 with the Burley 5500 and October 9, 2008 with the Burley 4500. All boreholes were accessed by helicopter with the rig, tooling and instrument installation or backfill supplies flown in to each location. Drilling in 2006 and 2007 has been summarized in previous reports (Wyllie & Norrish, 2007a and Wyllie & Norrish 2008a).

All borings were advanced through the overburden material (when present) using mud rotary drilling methods and either HQ or HWT casing with a retrievable tricone bit or chevron advancer. Rock core drilling was initiated when bedrock was encountered, typically between 0 and 10 feet below ground surface (bgs). Boreholes, RKS-42-08 through RKS-45-08, encountered bedrock increasing in depth from approximately 12 feet (RKS-42-08) to 28 feet (RKS-45-08). Soil sampling was completed (where possible) using the standard penetration test (SPT) in accordance with American Society for Testing and Materials (ASTM) D 1586. Rock core drilling and sampling was completed using HQ (2.4-inch core diameter) triple-tube wire line coring technology.

Rock core and soil was described in accordance with the Geotechnical Design Manual (M 46-03), Chapter 4 (WSDOT, 2005). Discontinuity data were further described in accordance with a modified ISRM Suggested Methods (Brown, 1981). Point load strength index testing (PLT) was completed on the rock core using the ISRM Suggested Method for Determining Point Load Strength (ISRM, 1985). Color was described using the Geologic Society of America (GSA) Rock Color Chart. Rock core was logged, placed in plastic core boxes and photographed prior to being moved to the Hyak WSDOT Maintenance Facility for temporary storage before transportation to WSDOT's Olympia, Washington Materials Laboratory for more permanent storage. Summary boring logs are provided in Appendix C and photographs of the rock core are provided in Appendix D. PLT data summary sheets are provided in Appendix E.

All boreholes drilled for NAD, NAF and the Slide Curve Feasibility and Design investigations were surveyed using downhole optical and acoustical televiewer methods. None of the MRE boreholes advanced within the Phase IC project interval were surveyed by televiewer. CRUX Oriented Borehole Logging (COBL), a division of CRUX, provided all necessary equipment and staff to complete the downhole surveys and process the data. Wyllie & Norrish reviewed and commented on the draft data sets before COBL issued their final report. The report provides orientation data corrected for true north using a declination of 17 degrees east. The televiewer logs and summary tables of the orientation data are provided in Appendix F.

Wyllie & Norrish completed a comprehensive core review of the 2006, 2007 and 2008 drilling programs in Olympia Washington, in December of 2008. The task included visual review of **all** rock core, surface mapping, field and laboratory testing results, core photographs, COBL data, the most recent alignment and cross section data and the 2006 and 2007 reporting efforts. The intent was to synthesize the data from three years of field work in order to develop design sectors with emphasis on key design elements for each of the identified sectors.

5.5 SUBSURFACE INVESTIGATION - INSTRUMENTATION PROGRAM

Vibrating wire piezometers (VWP) and time domain reflectometry (TDR) cable was installed in several of the boreholes. A brief summary of the piezometer and slope monitoring instrumentation is provided in the sections below. Instrumentation installed during the three years of the field program is summarized above in Table 2. The instrument installation procedures, calibration sheets and summary data plots are attached in Appendix G.

5.5.1 Vibrating Wire Piezometers

VWPs allow for collection of piezometric pressure data from a borehole without the standard screened well construction. The instruments are grouted into the borehole with a variable sand pack and a cable extends from the instrument to the top of the borehole. Changes in pore water pressure within the formation are translated through the sand and/or grout column and are reflected as a change in the frequency (hertz) read at the ground surface. The instrument is calibrated to correlate frequency with pressure which can be used to calculate hydraulic head. Six (6) VWPs were installed in three (3) boreholes during the 2008 field investigation and eleven (11) VWPs were installed in eight (8) boreholes in 2007. Two (2) VWPs were installed in H-110-06 in 2006. The location, serial number and depth information for these boreholes is summarized in Table 2.

Advantages to the VWP technology for this project include:

- Ease of installation of multiple VWPs in one borehole
- No traditional well development required
- Isolation of potential hydrogeologic units for monitoring of piezometric pressures within a single borehole

- Increased frequency of data collection
- Remote monitoring capabilities

Depths for the VWP instruments were selected using the rock core logs, observations on the core, review of the core photographs and review of the raw COBL data provided by the optical and acoustical televiewer surveys. For selected boreholes, one VWP was installed at or near the bottom of the boring with a second VWP instrument (if installed) located above a potential hydrologic boundary.

5.5.2 Slope Monitoring Information

TDR cable was installed in H-110-06 in 2006, RKS-18-07 during the 2007 program and in RKS-38-08, RKS-40-08 and RKS-41-08 during the 2008 field program. Each cable was typically installed to the total depth of the borehole, except for RKS-40-08 which had cable installed from ground surface to 75 feet bgs. The primary purpose of the installation was to monitor upslope rock stability during construction activities. Installation will allow for several seasons of baseline data to be obtained prior to the commencement of construction.

TDR cable is a method used for monitoring slope stability. Data collection consists of attaching a TDR cable tester to a coaxial cable grouted in a borehole. An electrical pulse is sent down the coaxial cable and when the pulse encounters a break or deformation in the cable, it is reflected. The reflection shows as a "spike" on the graphical display. The relative magnitude, rate of displacement, and the location of the zone of deformation can be determined. The size of the spike correlates with the magnitude of movement on the ground surface. However, the direction of movement and quantitative data relating to deflection is not obtained. TDR cable installations are summarized in Table 2 above.

6.0 LABORATORY TESTING PROGRAM

Laboratory testing was completed during all three years of the investigation. Test results of rock core samples were used for the following:

- Refinement of the field classification of rock strength
- Development of a correlation ratio between unconfined compressive strength and point load index for comparison to field point load data
- Assessment of friction angles of clay-filled discontinuities
- Refinement of the field classification of rock type, mineral assemblages, and geologic origin
- Evaluation of clay and silt plasticity
- Evaluation of rock and joint infilling for corrosion potential

6.1 ROCK STRENGTH TESTING

Rock core samples collected from the borings were preserved, labeled and stored at the Hyak Maintenance Facility and later at the Headquarters Facility in Tumwater. Selected samples were submitted to Geo Test Unlimited of Nevada City, California during the 2006, 2007 and 2008 field program. Testing included diametral and axial point load tests (ASTM D5731), unconfined compressive strength tests (ASTM D2938) and direct shear tests (ISRM Suggested Methods). The rock strength testing reports are provided in Appendix H.

6.2 CORROSIVITY AND ATTERBERG LIMITS TESTING

Selected material from the rock strength testing program was submitted to Sunland Analytical of Rancho Cordova, California for analysis by pH and Minimum Resistivity (CA DOT Test #643), Sulfate (CA DOT Test #417) and Chloride (CA DOT Test #422). Select samples of fine grained material from joints, faults and/or flow boundaries was collected and submitted to Vector Engineering, Inc of Grass Valley, California for testing by ASTM method D-4318. The corrosion and Atterberg limits testing reports are provided in Appendix H.

6.3 PETROGRAPHIC ANALYSIS

Several rock samples from the 2006, 2007 and 2008 rock slopes and material resource investigation were submitted for petrographic analyses. The analyses were conducted with emphasis on overall rock mineralogy, texture and susceptibility to weathering. Spectrum Petrographics of Vancouver, Washington prepared the thin sections and the analysis was completed in general accordance with ASTM C295 and ISRM Suggested Method for the Petrographic Description of Rocks (Brown, 1981). Spectrum Petrographics of Vancouver, Washington and Dr. Robert Burk (Burk GeoConsult, LLC) contributed to the petrographic work. Dr. Ray Ingersoll, Professor of Geology at University of California at Los Angeles, provided technical review of the work. The Petrographic Report is provided as Appendix H.

6.4 X-RAY DIFFRACTION ANALYSIS

Two (2) rock samples were submitted for X-ray diffraction analyses. The analyses were conducted to identify the mineralogy present in the samples selected with emphasis on the presence or absence of swelling mineralogic clays. The samples were collected, preserved and submitted for analysis by Wyllie & Norrish personnel. K/T GeoServices, Inc of Argyle, Texas prepared and analyzed the samples. The report is provided in Appendix H.

7.0 GEOTECHNICAL DATABASE

This section presents the results from three years of geotechnical investigations by Wyllie & Norrish. The results of the investigation have been complemented by data available from studies completed prior to 2006 both by Wyllie & Norrish, WSDOT and other agencies, consultants and individuals. The information provided in the project geology and project engineering geology with associated figures was similarly reported in the 2007 Slide Curve feasibility report (Wyllie & Norrish, 2007a). Borehole and laboratory testing results for the 2006 and 2007 field programs have also been previously published (Wyllie & Norrish, 2007a and b, 2008a). A brief summary of the 2006, 2007 and 2008 Wyllie & Norrish field investigation objectives is as follows:

- 2006 - Complete a feasibility-level site characterization to support the rock cut design of the proposed alignment between approximately WB stationing 1394+00 to 1411+00.
- 2007 – Complete a feasibility level site characterization for the rock cut design from WB stationing 1352+50 to 1396+50.
- 2008 – Complete a final design level site characterization for the rock cut design from WB stationing 1352+50 to 1396+50 and 1442+00 to 1480+00.

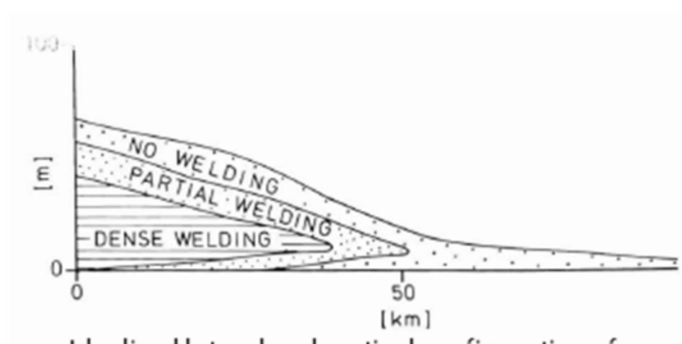
7.1 PREVIOUS STUDIES

The project area has undergone several studies over the past 50 years. In addition to the 2008 field program a list of the previous work considered for this geotechnical database is provided below.

- Ph.D. dissertation by Paul Hammond (1963)
- Geologic mapping by Frizzell and others (1984) and Tabor and others (2000)
- Forensic analyses of the 1957 rockslide at Slide Curve (Ritchie, 1957; 1960; Coombs, 1957; 1958)
- Select slope characterization between MP 55.17 and MP 110.00 by Chadbourne and Moses (1994)
- Discontinuity assessment in relation to the anticline present on Keechelus Ridge completed by Badger (2000)
- Ongoing site visits and site characterization by WSDOT, specifically work by Steve Lowell in 1998 and Doug Anderson in 1999
- Stability analysis and conceptual design parameters for Slide Curve completed by Golder Associates and Wyllie & Norrish (Findley and Norrish, 2005)

7.2 PROJECT GEOLOGY

As summarized in Section 4.2, the geology between Jenkins Knob and Slide Curve comprises pyroclastic flows from the Keechelus Lake member of the Ohanapecosh Formation (Figure 25). In general, pyroclastic flows are extremely heterogeneous in their chemical and depositional characteristics. Frequency of flows, flow thickness and proximity of sequential eruptive centers affect the cooling rates of both individual flow units and successive deposits that form cooling units (Fisher, 1984). The variability within cooling units, viscosity of the deposited mass and overall gas content are all factors that influence the degree of welding (viscous deformation of vitric fragments) affecting the deposit. An idealized representation of welding within a relatively homogeneous and simple cooling unit is shown from Fisher, 1984. The variability in the original (protolith) composition of the tuff, degree of welding and in-situ porosity and



Idealized lateral and vertical configuration of welding zones (Fisher, 1984).

permeability throughout the flow deposits is further complicated area by subsequent low-grade metamorphic alteration.

A design level investigation was completed for the area colloquially known as Slide Curve from WB station 1396+50 to 1411+50 (Wyllie & Norrish 2009a). As part of the Slide Curve investigation Engineering Domain types 1 through 3 were developed to characterize the observed geologic and engineering boundaries within the rockmass which are not bound by linear features or by defined flow units (Wyllie & Norrish, 2007a). The geologic conditions observed at Slide Curve continue to the north to approximately WB station 1382+50 (through Design Sector XII). Therefore the rockmass characteristics and nomenclature has been incorporated into this Phase IC report. A summary of each Domain is provided below.

- ***Domain #1*** – Meta-welded lapilli dacite tuff, fresh, strong to very strong, slight weathering of surface exposure both along rock outcrops and on more significant discontinuity surfaces. Color is typically pale greenish to medium bluish gray and medium gray with slightly olive gray to olive brown discoloration envelopes (< 1 inch) around discontinuities. The contact between Domain #1 and Domain #2 is typically gradational over several feet.

- ***Domain #2*** – Meta lapilli dacite tuff, fresh, medium strong to very strong, slight to moderate weathering of discontinuities. This domain has not been observed at the surface. Color is typically light gray to yellowish gray with light brown to dark yellowish orange envelopes (1 to 12 inches) around discontinuities. The degree of welding is significantly reduced from Domain 1.

- ***Domain #3*** – Meta lapilli dacite tuff, moderately to highly weathered, extremely weak to moderately weak, observed at the surface and in boreholes with color of light brown to dark yellowish orange. The degree of welding is similar to Domain #2. Domain #3 appears to represent the rockmass of Domain #2 after significant weathering (Moderate or greater) occurs.

The southern portion of the project area (WB stationing 1442+00 to 1480+00) was mapped as Naches Formation by Tabor et al., (2000). Rock types encountered included meta-basalts/andesites, meta-lapilli tuffs, silicic volcanics and silty sandstone. The meta-basalt/andesite was exposed in rock cut with the silicic volcanics outcroping adjacent to the south. The remainder of the area is covered in alpine till ranging from several feet to approximately 30 feet. The meta-basalt outcrop forms the topographic high of the area with glacially developed stoss and lee geomorphology observed.

7.3 PROJECT ENGINEERING GEOLOGY

7.3.1 Discontinuities

The term discontinuity refers to any mechanical "break" in a rockmass having zero or reduced tensile strength relative to the surrounding intact rock. It is a collective term that in Phase 1C and elsewhere along the proposed highway realignment includes faults, shear zones, joints, and lava or pyroclastic flow boundaries. All of these features are of potential concern, if they represent surfaces with an orientation and persistence that could allow them to adversely intersect the proposed highway cut slope or if they are sufficiently unpredictable or ubiquitous in nature that for design purposes they can exist anywhere within the cut face. Healed discontinuities were abundant based on field observations and the borehole drilling program, however they have not been considered here because they do not materially affect rockmass strength and are not properly discontinuities. Calcite and probably zeolite mineralization were common bonding agents.

7.3.2 Joints

Related in part to regional anticlinal folding (Badger, 2002), major joints are present throughout the area. Where faults show there has been translational movement in the plane of the fault, joints have moved primarily perpendicular to the joint plane. Joints are of concern especially if they have low shear strength in-filling, such as clays. Where filling materials are not present or are permeable, joints may be open conduits for meteoric water. These open discontinuities frequently receive meteoric waters, and thus may weather more rapidly than those sealed by clay or other rockmass conditions. As weathering proceeds fracture flow may decrease. Primarily systematic joint structure was observed along the project corridor. Many of the prominent joints are persistent and without significant filling materials (Figure 26). Others have from a few tenths of an inch up to two to three inches of clay filling. Typically, no or slight weathering halos of iron oxides (discoloration of rock mass) were observed to represent alteration of the rock by oxygenated ground water. Select rock types were found to be more susceptible to weathering and developed strong weathering halos of iron oxides.

7.3.3 Faults/Shear Zones

Faults and shear zones are discontinuities of particular concern because they are frequently persistent over many hundreds of feet to many miles. Faults without much movement but abundant sheared rock fragments are typically considered shear zones. These zones may be conduits for groundwater, whereas faults with a greater degree of shearing may retard groundwater movement because of the production of finely comminuted rock (gouge). Faults were identified along the alignment in the existing rock cut and the boreholes (Figure 27). Although not a direct focus of this investigation, none of the faults observed were considered active because no evidence was found of displacement of the glacial deposits that partially blanket the area.

7.3.4 Flow Boundaries

Flow boundaries may or may not represent discontinuities depending on the degree of welding that occurs to previous flows, other rock units, or buried soils (paleosols). Absence of evidence of a flow boundary discontinuity may simply mean that the flows were consecutive over short periods and welded to each other. Where sufficient time elapses between flows, paleosols, ash, and vegetative remains may be present. All of those materials were found within the several flows identified along the project corridor (Figure 28). In several cases, water (<<1 gpm) seeped from the exposed flow boundaries when first observed in June 2006. Later in the season, water was less apparent; however, the local presence of algae on the rock face indicated consistency of wet conditions. Flow boundaries reflect the topographic surface over which the pyroclastic and volcanic rocks flow and the new topography created as cooling solidifies materials that are part of the same flow. The geometry of these boundaries, or the presence or lack of materials between flows is not predictable. Thus, from an engineering viewpoint it must be expected that they could occur anywhere within the sequence. Overall orientation of the flows can be extremely variable depending on source area and subsequent tectonic activity. However, within the Phase 1C Slide Curve area there is strong evidence for a southwestern dip direction out of the slope (Figure 29).

7.3.5 Discontinuity Identification

Definitive fault and flow boundary identification based on borehole observations can be difficult. Frequently core observations can suggest but not prove the presence of a fault or flow boundary. In order to clarify the terminology in this report the following definitions are provided.

Possible Fault or Flow Boundary –Significant weathering and/or decrease in rock strength in rock observed adjacent to discontinuity with clay infilling present. Discontinuity surfaces may or may not include polished surfaces. Discontinuity may be filled with comminuted rock or broken zones. While interpretation of the genesis of this feature is uncertain, this discontinuity has the same engineering significance as a fault or flow boundary. Because it could represent a flow boundary, discontinuity persistence may be irregular and less predictable than a fault.

Probable Fault – Observed conditions similar to a “Possible Fault” with confirmed polished surfaces and/or possible slickensided surfaces. Other methods of genesis are not probable and for engineering analyses this will be considered a fault.

Fault – Observed conditions as described for a “Probable Fault” including slickensided surfaces and/or change in rock type across discontinuity. Evidence of offset or interpreted finely comminuted rock fragment (mylonitic) infilling observed.

Flow Boundary – Observed conditions indicative of boundaries including evidence of baked zones, ash, paleosols and/or change in rock type across discontinuity. Flow boundaries have engineering significance based on their irregular and relatively unpredictable geometry compared to fault discontinuities.

7.4 FIELD INVESTIGATION RESULTS

7.4.1 Surface Exploration

Surface structural mapping was completed to support the design effort. Structural mapping included Sirovision mapping and traditional outcrop mapping using a Brunton™ compass. A total of sixty-one (61) three-dimensional Sirovision images from the Phase IC interval were generated and analyzed between 2006 and 2008 with approximately 3200 structural features identified and mapped. The Sirovision reports are provided in Appendix A. Detailed structural mapping was completed in 2008 at twenty-eight (28) locations along the existing rock cut at approximately 100-foot intervals. Approximately 810 structural features were identified and mapped including orientation, persistence, spacing, roughness and infilling among other data. The mapping data sheets and photos of the mapping stations are provided in Appendix B.

7.4.2 Subsurface Exploration

Twenty-three (23) boreholes were advanced as part of the rock slope subsurface investigation and an additional four (4) boreholes were advanced for the MRE study, the results of these boreholes being provided for this report. The borehole locations are shown on Figure 2 and summary table of locations is provided as Table 2 in Section 5.5.

The findings of the subsurface drilling investigation are summarized by borehole in Table 3 which provides a summary by design sector of total core recovery, rock quality designation, rock strength, weathering and fracture frequency with select columns highlighted for interpreted rock core quality. Detailed summary of the design sectors and borehole observations are provided below under Section 7.3.

Table 3 represents borehole data collected over three years of investigation and from boreholes drilled by two different contractors utilizing at least four different types of drill rigs and several different drillers. Additionally there were seven (7) geologists who worked on the project logging the rock core at the drill. Balin Strickler trained and managed all field geologists throughout the program and also completed reviews of all core. Subsurface conditions are interpreted from limited information at widely spaced locations for design purposes only.

Table 3 Summary of Rock Parameters by Design

Design Sector	Borehole	Total Depth (feet)	Depth to Bedrock (feet)	Total Bedrock Footage Drilled	Total Core Recovery (TCR)	Rock Quality Designation (RQD)					Rock Strength [1]			Weathering [1]		Fracture Frequency (# per foot)		
						0 - 25%	25 - 50%	50 - 75%	75 - 90%	90%+	R0 - R1	R2 - R3	R4 - R5	III-VI	I - II	3+	2 - 3	0 - 1
Snowshed	RKS-08-07	105.0	9.2	95.8	100.0%	1%	0%	26%	16%	57%	2%	98%	0%	1%	99%	14%	16%	70%
	MRE-05-08	85.5	8.9	76.6	99.1%	2%	7%	24%	33%	35%	0%	100%	0%	0%	100%	20%	22%	58%
IX	RKS-09-07	120.0	2.6	117.4	100.0%	0%	0%	13%	21%	66%	0%	100%	0%	0%	100%	5%	23%	72%
	RKS-10-07	140.7	3.5	137.2	99.3%	4%	0%	10%	16%	70%	12%	88%	0%	13%	87%	6%	15%	79%
X	RKS-11-07	145.4	1.4	144.0	99.4%	0%	3%	14%	30%	52%	0%	42%	58%	0%	100%	2%	26%	72%
	RKS-12-07	130.4	0.0	130.4	99.3%	0%	0%	15%	42%	43%	1%	41%	59%	2%	98%	7%	25%	69%
	RKS-18-07	185.0	0.0	185.0	99.7%	0%	1%	3%	14%	83%	0%	88%	12%	2%	98%	1%	15%	84%
	RKS-38-08	119.5	6.3	113.2	100.0%	7%	7%	15%	2%	69%	0%	38%	62%	2%	99%	17%	15%	68%
	MRE-06-08	115.5	3.3	112.2	100.0%	2%	11%	27%	37%	23%	0%	90%	10%	0%	100%	19%	31%	50%
	MRE-07-08	115.9	2.8	113.1	99.6%	4%	13%	27%	43%	12%	0%	33%	67%	0%	100%	18%	22%	59%
	MRE-08-08	86.4	6.7	79.7	94.2%	0%	0%	0%	53%	47%	0%	15%	85%	0%	100%	5%	19%	76%
XI	No Boreholes Drilled In This Sector																	
XII	H-110-06	149.2	5.7	143.5	99.8%	0%	1%	9%	17%	72%	2%	1%	97%	3%	97%	3%	13%	85%
	RKS-13-07	108.5	3.0	105.5	97.2%	9%	15%	28%	19%	28%	32%	68%	0%	38%	62%	5%	28%	67%
	RKS-14-07	130.0	2.6	127.4	100.0%	0%	0%	0%	27%	73%	4%	47%	50%	8%	92%	3%	15%	82%
	RKS-15-07	163.2	5.6	157.6	99.7%	3%	15%	3%	19%	60%	15%	53%	32%	23%	77%	3%	15%	82%
	RKS-16-07	140.2	7.0	133.2	99.8%	0%	0%	5%	8%	87%	13%	44%	43%	13%	87%	2%	14%	85%
	RKS-17-07	114.0	6.5	107.5	98.0%	0%	0%	5%	9%	86%	7%	52%	42%	0%	100%	3%	9%	88%
	RKS-19-07	105.1	10.9	94.2	100.0%	0%	0%	18%	16%	66%	5%	95%	0%	12%	88%	7%	22%	71%
	RKS-39-09	101.0	5.0	96.0	93.1%	20%	17%	31%	8%	24%	2%	98%	0%	2%	98%	48%	25%	27%
	RKS-40-08	139.5	3.8	135.7	97.0%	8%	11%	18%	37%	26%	5%	90%	5%	11%	89%	20%	25%	55%
	RKS-41-08	89.5	3.8	85.7	98.1%	11%	14%	31%	35%	10%	5%	95%	0%	25%	75%	28%	30%	42%
XVII	RKS-42-08	35.5	11.5	24.0	100.0%	0%	0%	18%	0%	83%	2%	98%	0%	6%	94%	2%	0%	98%
	RKS-43-08	35.0	19.4	15.6	100.0%	0%	13%	0%	87%	0%	0%	100%	0%	0%	100%	23%	38%	38%
	RKS-44-08	45.0	26.7	18.3	99.5%	0%	0%	0%	69%	31%	0%	100%	0%	0%	100%	0%	56%	44%
	RKS-45-08	50.0	28.1	21.9	100.0%	16%	33%	23%	22%	7%	13%	87%	0%	26%	74%	23%	41%	37%
XVIII	RKS-46-08	85.0	3.4	81.6	100.0%	2%	24%	36%	17%	20%	0%	64%	36%	0%	100%	23%	41%	36%
	RKS-47-08	72.0	12.8	59.2	100.0%	0%	0%	24%	8%	67%	1%	7%	92%	1%	99%	7%	29%	65%
% Value represents good ground conditions						<5%	<15%	--	--	--	<5%	--	--	<5%	--	<10%	--	--
% Value represents moderate ground conditions						5 -10%	15 - 30%	--	--	--	5 - 15%	--	--	5 - 15%	--	10 - 25%	--	--
% Value represents poor ground conditions						>10%	>30%	--	--	--	>15%	--	--	>15%	--	>25%	--	--

Notes:

[1] = Rock strength and Weathering values based on WSDOT Geotechnical Design Manual M 46-03. September, 2005

Percentages are representative of total linear footage of bedrock encountered and do not inlcude overburden

MSL = Mean Sea Level

Sector

All twenty-three (23) boreholes for the rock slopes program were surveyed using optical and acoustical televiewer methods. Approximately 2250 feet of bedrock was surveyed using either one or both of the televiewer tools depending on fluid presence and fluid clarity. Approximately 2200 individual discontinuities were mapped with orientations from the available data.

PLT data was collected in the field during the drilling for all twenty-seven (27) boreholes. 381 qualifying tests were completed on the various rock types. More than 500 tests were initially completed but either significant indentation (>5mm) or failures not passing through the plane of the platens excluded many of the field tests from reporting. Table 4 provides a summary of the testing results by borehole.

Table 4 PLT Summary Results

Design Sector	Borehole	R Value Range		# of Tests	R Value Average[1]	Standard Deviation[1]
		Max	Min			
Snowshed	RKS-08-07	3	1	18	2.5	0.6
	MRE-05-08	4	2	12	2.5	0.7
IX	RKS-09-07	3	1	20	2.3	0.6
	RKS-10-07	4	1	18	2.2	0.9
X	RKS-11-07	5	2	16	3.9	1.1
	RKS-12-07	5	1	14	3.9	1.3
	RKS-18-07	5	1	26	2.8	1.1
	RKS-38-08	5	1	17	3.1	1.4
	MRE-06-08	5	2	23	3.8	0.9
	MRE-07-08	5	4	15	4.7	0.5
	MRE-08-08	5	3	13	4.5	0.7
XI	None	No Available Data				
XII	H-110-06	5	4	27	4.5	0.5
	RKS-13-07	4	1	17	3.0	1.1
	RKS-14-07	5	2	19	3.8	0.9
	RKS-15-07	5	1	16	3.7	0.9
	RKS-16-07	5	1	22	3.5	0.9
	RKS-17-07	5	1	22	3.5	0.9
	RKS-19-07	3	2	13	2.7	0.5
	RKS-39-08	4	1	11	3.0	1.2
	RKS-40-08	4	1	18	2.9	0.8
	RKS-41-08	4	3	7	3.3	0.5
XVII	RKS-42-08	3	3	4	3.0	0.0
	RKS-43-08	2	1	3	1.3	0.6
	RKS-44-08	No Available Data				
	RKS-45-08	3	3	1	3.0	0.0
XVIII	RKS-46-08	5	3	5	4.4	0.9
	RKS-47-08	4	2	4	2.5	1.0

Notes:
1. R value based on WSDOT Design Manual M 46-03
2. Design Sector – Design Sector as defined below in Table 9
3. [1] = Assumes a normal distribution.

7.5 LABORATORY RESULTS

The laboratory testing program included rock and clay samples from both the rock core and outcrops that were collected throughout the three-year program and submitted for testing. Strength testing, corrosion testing, direct shear testing, plasticity index testing and petrographic analysis were completed on these samples. The data reported below for strength testing and petrographic analysis are specific to the Phase IC project interval only. Results for other areas of the project have been provided under separate cover. The corrosion, direct shear and plasticity results provided herein represent all testing completed at the direction of Wyllie & Norrish during the ongoing investigation regardless of location.

7.5.1 Rock Strength Testing

Discrete rock core samples were selected for rock strength characterization. These samples were collected during the drilling effort, preserved in foil and wax immediately after drilling and submitted to the testing laboratory for analysis. The results are summarized on Table 5 and the Laboratory reports are provided in Appendix H.

Table 5 – Summary of Rock Strength Testing Data

Borehole #	Depth (feet bgs)	Sample #	Rock Core Description [A]	Density (pcf)	Axial Corrected Point Load Index (psi)	Diametral Corrected Point Load Index (psi)	Unconfined Compressive Strength (psi)
H-110-06	80.1 - 80.6	102A	Medium gray, I, MWLT	164.0			27,724
	80.6 - 81.3	102B			1,260	1,222	
RKS-08-07	10.0-10.7	08-101A	Medium gray meta-welded lapilli tuff	158.1	364		9,795
RKS-08-07	10.7-11.3	08-101B	Medium gray meta-welded lapilli tuff			321	
RKS-09-07	41.3-41.9	09-102A	Medium gray meta-welded lapilli tuff		262	234	
RKS-09-07	41.9-42.6	09-102B	Medium gray meta-welded lapilli tuff	156.6			10,040
RKS-10-07	74.1-74.8	10-2A	Medium gray meta-welded lapilli tuff	154.6	279		10,674
RKS-10-07	74.8-75.4	10-2B	Medium gray meta-welded lapilli tuff		128/206		
RKS-11-07	64.1-64.8	11-101A	Medium gray meta-welded lapilli tuff	161.3			11,100
RKS-11-07	64.8-65.4	11-101B	Medium gray meta-welded lapilli tuff		254	429	
RKS-12-07	28.9-29.5	101A	Medium gray meta welded lapilli tuff		1235	1365	
RKS-12-07	29.5-30.3	101B	Medium blue gray meta welded lapilli tuff	164			31,257
RKS-12-07	124.0-124.6	103A	Medium gray meta welded lapilli tuff		288[2]	848	
RKS-12-07	124.6-125.3	103B	Medium blue gray meta welded lapilli tuff	164.3			23,952
RKS-13-07	19.0-19.7	102	Medium brown meta welded lapilli tuff	155.7		91[2]	4,120
RKS-13-07	44.1-44.6	104A	Light greenish gray meta welded lapilli tuff	162.6			8,184
RKS-13-07	44.6-45.1	104B	Light greenish gray meta welded lapilli tuff		801	425	
RKS-14-07	78.2-78.8	103A	Light greenish gray meta welded lapilli tuff		222	259	
RKS-14-07	78.8-79.4	103B	Light greenish gray meta welded lapilli tuff	159			5,805
RKS-15-07	20.5-21.1	102A	Brown fractured and altered meta welded lapilli tuff		37	121	
RKS-15-07	21.1-21.8	102B	Medium brown meta welded lapilli tuff	151.6			3,250
RKS-15-07	120.0-120.5	105A	Medium brown meta welded lapilli tuff	159.6			8,287
RKS-15-07	120.5-121.1	105B	Light brownish gray fractured and altered meta welded lapilli tuff		148	366	

RKS-16-07	47.0-47.6	101A	Light brownish gray fractured and altered meta welded lapilli tuff		277	397	
RKS-16-07	47.6-48.2	101B	Medium brown meta welded lapilli tuff	159.3			6,408
RKS-17-07	53.3-54.0	102	Meta welded lapilli tuff, medium brown and blue gray clasts in a light gray matrix	156.2		430	7,767
RKS-18-07	72.0-72.5	102A	Dark gray fractured meta welded lapilli tuff		193	317	
RKS-18-07	72.5-72.9	102B	Dark gray heavily fractured meta welded lapilli tuff	159.4			6,005
RKS-19-07	30.4-31.4	102	Light bluish white friable clasts in light gray, I-II, MWLT	156.4		259	5,959
RKS-19-07	55.6-56.3	103A	Light bluish white friable clasts in light gray, I, MWLT		405	342	
RKS-19-07	56.3-57.0	103B		160.9			9,148
RKS-46-08	50.7-51.2	101	Medium gray meta-basalt		1294	1138	
RKS-46-08	51.2-52.0	102	Medium gray meta-basalt with a major weak diagonal joint about 25 degrees to the core axis that was glued with cyanoacrylate glue.	166.7	1629	1024^{I} 811^{L}	6,063
RKS-47-08	35.8-36.7	102	Medium gray quartzite with light gray discoloration along numerous diagonal joints about 40 degrees to the core axis. The weak fractured end-corner was repaired with cyanoacrylate glue.	161.8	413	286^{w}	2,528

Notes:

^{A} Includes discontinuity and infilling data if present, rock color, weathering (I to VI) and rock type
^{W} Failed along a plane of weakness
^{I} Invalid – Failure did not pass through both points of loading
^{i} Initial Strength Parameter
^{L} Irregular Lump Point Loading
^{AL} Applied Axial Load Exceeded Load Frame Capacity
^{f} Final Strength Parameter
1The shear box rotated and developed box-to-box contact preventing the evaluation of final values
MWLT = Meta-welded lapilli tuff
I to VI = Degree of rock weathering, fresh (I) to residual soil (VI)
bgs = below ground surface
pcf = pounds per cubic foot
psi = pounds per square inch

7.5.2 Corrosion Testing

Select samples of bedrock and discontinuity infilling were collected and submitted for analysis. The results are summarized on Table 6 and the Laboratory reports are provided in Appendix H.

Table 6 – Summary of Corrosion Testing Data

Borehole #	Depth (feet bgs)	Sample #	Rock Core Description	Soil pH	Minimum Resistivity (ohm-cm x1000)	Chloride (ppm)	Sulfate (ppm)
RKS-19-07	33.0-33.7	101	Planar discontinuity filled with 1-5mm thick light bluish white clay in light gray, I-II, MWLT	7.86	0.51	46.2	1105.1
RKS-23-07	60.0-60.8	104	Clay filled joint, II, R3-R2, MWLT	8.54	3.48	6	10.4
RKS-24-07	36.3-36.9	102	5/8-1" wide shear zone with rock flakes and light bluish white clay in blue gray, I, MWLT	8.18	0.64	28.3	293.5
RKS-27-07	25.8-26.7	101	I, R3, Meta-welded lapilli Tuff	8.13	4.56	5.7	2.6
SI-2-07	40.4-41.4	102A	Planar discontinuity filled with 1/8" thick light yellowish brown clay in medium gray to brown gray, I, MWLT	7.98	0.72	43.3	315
	84.6-86.0	105	Planar discontinuity filled with white clay and 1/8-3/8" thick light brown interlacings in blue gray, I, MWLT	7.9	0.86	11.2	252
SI-05-07	66.8-67.3	104	II, R2, MWLT (Clay filled joint-disturbed)	7.7	4.29	8.3	20.1
SI-9-07	27.4-27.7	203	III-IV, R0, Meta-welded lapilli tuff	7.24	4.56	13.9	2.2
SI-10-07	43.4-44.0	104	Planar discontinuity filled with 1/4-5/8" thick bluish white clay and rock fragments in fractured blue gray, I, MWLT	8.1	0.7	27	271.9
SI-12-07	59.9-60.9	102	Clay-filled discontinuity, I-II, R4-R5, MWLT	7.72	3.48	8.1	10.1

Notes:
[A] Includes discontinuity and infilling data if present, rock color, weathering (I to VI) and rock type
MWLT = Meta-welded lapilli tuff
R0 to R5 = Rock strength ranging from extremely weak (R0) to very strong (R5)
I to VI = Degree of rock weathering, fresh (I) to residual soil (VI)
bgs = below ground surface
ppm = parts per million

7.5.3 Direct Shear Testing

Select samples of bedrock and discontinuity infilling were collected and submitted for direct shear testing. The results are summarized on Table 7 and the Laboratory reports are provided in Appendix H.

Table 7
Summary of Direct Shear Testing Data

Borehole #	Depth (feet bgs)	Sample #	Rock Core Description [A]	Type of Test	Shear Intercept (psi)	Friction Angle (degrees)
H-107-06	84.0 - 84.9	103A	Planar discontinuity filled with 3/8" thick soft light brown clay in dark gray, I, MWLT	Discontinuity	1.9	7.8
		103B		Discontinuity	3.6	3.2
		103C		Saw Cut	4.5[i]	30.6[i]
					6.0[f]	39.4[f]
RKS-06-07	80.9-81.6	06-104	Jointed, fresh (I) MWLT, interspersed in tannish gray clay	Discontinuity	1.3[i]	23.2[i]
					2.3[f]	23.0[f]
RKS-11-07	107.3-107.8	11-102	Jointed, fresh (I) MWLT, interspersed with firm brown clay	Discontinuity	4.1[i]	10.1[i]
					4.1[f]	12.3[f]
RKS-13-07	39.0-39.5	103	Joint in medium brownish gray, slightly to moderately weathered (II - III) meta lapilli tuff, light yellowish brown filling <1/8" thick	Discontinuity	4.6[i]	18.07[i]
					1.7[f]	18.5[f]
RKS-19-07	33.0-33.7	101	Planar filled joint in light gray, fresh (I) meta lapilli tuff, thick light bluish white filling	Discontinuity	0.6[i]	19.9[i]
					3.3[f]	15.9[f]
RKS-19-07	56.3-57.0	103B	Saw-cut and ground interface in light gray, fresh (I), meta lapilli tuff with a few light bluish white friable clasts	Saw Cut	0.2[i]	27.6[i]
					1.7[f]	31.5[f]
RKS-22-07	11.7-12.5	101A	Light to medium gray, I-II, MWLT	Saw Cut	NA[1]	28.1[i]
					0.8[f]	33.3[f]
RKS-24-07	36.3-36.9	102	5/8-1" wide shear zone with rock flakes and light bluish white clay in blue gray, I, MWLT	Discontinuity	3.7[i]	14.5[i]
					NA[1]	NA[1]
RKS-29-07	18.8-19.5	102A	Medium greenish gray to medum gray, I, MWLT	Saw Cut	NA[1]	26.7[i]
					0.6[f]	31.1[f]
RKS-30-07	55.1-55.8	103A	Medium blue gray, I, MWLT	Saw Cut	0.1[i]	33.4[i]
					0.8[f]	39.2[f]
SI-2-07	40.4-41.4	102A	Planar discontinuity filled with 1/8" thick light yellowish brown clay in medium gray to brown gray, I, MWLT	Discontinuity	1.1[i]	15.7[i]
					1.7[f]	11.1[f]
SI-2-07	84.6-86.0	105	Planar discontinuity filled with white clay and 1/8-3/8" thick light brown interlacings in blue gray, I, MWLT	Discontinuity	6.6[i]	16.7[i]
					1.2[f]	13.0[f]
SI-10-07	43.4-44.0	104	Planar discontinuity filled with 1/4-5/8" thick light bluish white clay and rock fragments in fractured blue gray, I, MWLT	Discontinuity	3.2[i]	16.9[i]
					1.9[f]	15.7[f]
RKS-39-08	10.5-11.0	101	Slightly rough curved dark brown oxide coated joint in fresh (I) meta-lapilli tuff	Discontinuity	3.2[i]	32.1[i]
					2.7[f]	30.4[f]
RKS-35-08	43.3-44.0	S-2-A	Saw-cut interface in medium gray, fresh (I) meta-basalt.	Saw Cut	0.3[i]	22.4[i]
					1.3[f]	30.1[f]
RKS-35-08	50.4-51.5	S-3-A	Contact between stiff greenish gray slickensided clay and altered meta-basalt.	Discontinuity	1.8[i]	16.3[i]
					3.3[f]	13.6[f]
RKS-47-08	41.3-42.0	103	Saw-cut interface in light to medium gray, fresh (I) quartzite.	Saw Cut	NA (-0.6)[i]	29.4[i]
					0.2[f]	32.9[f]

Notes:
[A] Includes discontinuity and infilling data if present, rock color, weathering (I to VI) and rock type (MWLT)
[i] Initial Strength Parameter
[f] Final Strength Parameter
[1] The shear box rotated and developed box-to-box contact preventing the evaluation of final values
MWLT = Meta-welded lapilli tuff
I to VI = Degree of rock weathering, Fresh to Residual Soil
NA = Not Applicable
NT = Not Tested
bgs = below ground surface
psi = pounds per square inch

7.5.4 Plasticity Testing

Select samples of discontinuity infilling were collected and submitted for Atterberg Limits Testing. The results are summarized on Table 8 and the Laboratory reports are provided in Appendix H.

Table 8 – Summary of Plasticity Testing Data

Boring or Outcrop Location	Sample #	Depth, ft	Moisture Content (%)	Liquid Limit	Plastic Limit	Plasticity Index	Classification
RKS-34-08	S-3-A	68.7-69.4	NT	25	12	13	CL
RKS-35-08	S-3-A	50.4-51.5	NT	46	15	31	CL
RKS-41-08	104	75.5-76.3	NT	28	17	11	CL
RKS42-08	101	26.7-26.7	NT	21	15	6	CL-ML
1325+25	1	NA	24.1	63	49	14	MH&OH
1325+25	1	NA	24.1	62	48	14	MH&OH
1325+25	2	NA	9.8	37	25	12	CL/ML & OL

Notes:
CL – Lean Clay
ML – Low Plasticity Silt
MH – High Plasticity Silt
OH – High Plasticity Organic
OL – Low Plasticity Organic
NA = Not Applicable
NT = Not Tested
bgs = below ground surface
psi = pounds per square inch

7.5.5 Petrographic Analysis

Petrographic analyses were completed in 2007 and 2008 as part of the rock slopes feasibility and the MRE investigations. Twenty (20) samples were collected from boreholes and nine (9) samples were collected from the existing outcrops. Of the twenty-nine (29) samples, fifteen (15) were collected as part of the MRE investigations. A summary of samples by depth and design sector is provided in Table 9 below. The Petrographic Report for Phase IB is provided in Appendix H. The analysis confirmed the rock types as meta-basalts and meta-welded lapilli tuffs of dacitic composition.

7.5.6 X-Ray Diffraction Analyses

X-ray diffraction analyses were completed in 2008 as part of the rock slopes design investigation. Two (2) samples were collected from boreholes RKS-19-07 and MRE-09-08. Testing was completed after swelling was observed in altered pumice fragments present in RKS-19-07 within Domain 2 type rock. A second sample from Domain 1 rock was identified and submitted for comparison. The sample submitted for testing from RKS-19-07 was collected directly from the altered pumice exhibiting the swelling condition while the sample from RKS-09-08 was taken from the rock matrix, the most likely location for the presence of swelling clays within the Domain 1 rock type. Results from the testing show 62% of the sample from RKS-19-07 comprised a mixed-layer Illite/Smectite with 10% Smectite layers. The Smectite clay mineral group is composed of expandable clay minerals, confirming the presence of swelling clays. Only 0.8% of the sample from RKS-09-08 contained the mixed-layer Illite/Smectite group with more than 87% of the rock comprised of quartz and plagioclase. The laboratory report is provided in Appendix H.

7.6 DESIGN SECTOR SUMMARIES

This section incorporates results from the 2006, 2007 and 2008 field investigation to provide concise geologic and engineering summaries for each of the seven (7) design sectors (Sector VIII to XII, XVII, XVIII). Figure 2 provides a plan view of the sector boundaries. Table 9 provides a summary of the various sectors by station and includes the available borehole and surface information collected within each Sector.

Table 9 – Design Sector Summary Table

Design Sector	WB Stationing		2006, 2007 and 2008 Drilling Investigation	Petrographic Analysis		Other Available Data			
	Start	End		Depth (feet bgs)	Collected from rock cut	COBL Logs	PLI Field Testing	Structural Mapping	Sirovision
VIII Snowshed	1352+50	1363+50	RKS-08-07	None	3 Samples	Yes	Yes	3 Locations	8 Locations
			MRE-05-08	54.6		None	Yes		
IX	1363+50	1366+00	RKS-09-07	22.7, 101.2	None	Yes	Yes	3 Locations	6 Locations
			RKS-10-07	None		Yes	Yes		
X	1366+00	1375+50	RKS-11-07	None	3 Samples	Yes	Yes	8 Locations	18 Locations
			RKS-12-07	64.0, 77.7, 129.6		Yes	Yes		
			RKS-18-07	153.7, 168.5		Yes	Yes		
			RKS-38-08	None		Yes	Yes		
			MRE-06-08	72.3		None	Yes		
			MRE-07-08	36.1, 92.2		None	Yes		
			MRE-08-08	50.9		None	Yes		
XI	1375+50	1382+50	None	None	None	None	None	1 Location	4 Locations
XII	1382+50	1396+50	H-110-06	None	None	Yes	Yes	12 Locations	14 Locations
			RKS-13-07	None		Yes	Yes		
			RKS-14-07	None		Yes	Yes		
			RKS-15-07	None		Yes	Yes		
			RKS-16-07	None		Yes	Yes		
			RKS-17-07	None		Yes	Yes		
			RKS-19-07	None		Yes	Yes		
			RKS-39-08	None		Yes	Yes		
			RKS-40-08	None		Yes	Yes		
			RKS-41-08	None		Yes	Yes		
XVII	1442+50	1468+00	RKS-42-08	None	None	Yes	Yes	None	None
			RKS-43-08	None		Yes	Yes		
			RKS-44-08	None		Yes	Yes		
			RKS-45-08	None		Yes	Yes		
XVIII	1468+00	1480+00	RKS-46-08	36.5, 62.8	4 Samples	Yes	Yes	1 Location	11 Locations
			RKS-47-08	52.3		Yes	Yes		

Notes:
WB = West bound
bgs = below ground surface
COBL = Crux Oriented Borehole Logging
PLI = Point Load Index

As described in Section 7.3 the geology of the project area is significant to the engineering characteristics that influence design. Faults and flow boundaries represent significant features that must be considered during the design effort due to their persistence, condition and the unpredictable nature of their orientation. Not only are these features significant to design efforts but accurate identification and mapping of the features is a critical part of the construction excavation activities. Appendix I includes a memorandum that provides a descriptive approach to field identification and analysis of these persistent discontinuities. Table 10 provides a summary of all surface and subsurface features identified from the Design Sectors during the three years of investigation. More detailed analysis of the features is found in the Design Sector Summaries provided below.

Table 10 – Fault and Flow Boundary Summary

Design Sector	Location	Type of Strucutre	Available Orientation(s) [Dip, Dip Direction]	Depth (feet bgs)	Notes:
Snowshed	RKS-08-07	Possible Fault	[09/356]	91	Noted as possible shear on log
	Outcrop	Flow Boundary	--	--	Boundary between Sector Snowshed and XI. No measured data available, general apparent orientation obtained from across highway, dipping at ~45 degrees to the south.
IX	Outcrop	See above comment on flowboundary in outcrop from Sector Snowshed			
	RKS-09-07	Possible Flow Boundary	[38/210]	27	Infilling appears to be laminated.
		Flow Boundary	[27/187], [42/211]	69	Likley intersection of boundary observed in outcrop.
X	RKS-11-07	Possible Flow Boundary	[22/136]	97	Noted on borehole log as possible FB
	RKS-12-07	Possible Flow Boundary	[41/192]	18	B Haneberg calculated best fit above at [42, 151], intersection of RKS-12 at 18 feet bgs
		Possible Flow Boundary	[32/283], [27/284]	27	B Haneberg calculated best fit above at [42, 151], intersection of RKS-12 at 18 feet bgs
	RKS-18-07	Possible Flow Boundary	[38/182]	44	B Haneberg calculated best fit above at [42, 151], intersection of RKS-18 at 44 feet bgs
		Possible Flow Boundary	[50/198]	48	B Haneberg calculated best fit above at [42, 151], intersection of RKS-18 at 44 feet bgs
	MRE-07-08	Probable Healed Fault	[60/???]	46	Healed fault zone, with brecciated rock
	Outcrop	Flow Boundaries	--	--	Several bulbous shaped outcrops upslope from existing cut with significant discontinuities observed. One surveyed and analyzed with orientation of [42,151]
XI	None noted in outcrop				
XII	RKS-39-08	Probable Fault	[49/225]	55	
		Flow Boundary and/or Fault	Various	72 - 94; 97 - 98	
	RKS-40-08	Possible Fault	[37/184], [38/182]	131	
	RKS-41-08	Fault	[38/235]	14	
		Fault	[45/235], [50/164]	33	
		Probable Fault	[58/195], [30/178], [42/211]	36 - 37	
		Fault Zone	[46/203], [46/230]	68 - 82	Orientation from main fault zone at 76 feet, several mylonite shears on either side
	RKS-13-07	Possible Fault	[21/290]	20	
		Probable Fault	[42/244]	39	
		Possible Flow Boundary		67 - 76	Several 'possible flow boundary's' noted on the log but it appears to be hightly weathered rock mass
		Possible Flow Boundary	[62/190]	106	Noted as 'possible flow boundary'
	RKS-14-07	Possible Flow Boundary	[16/146]	68	
		Possible Fault	[12/270]	80	
		Fault	[21/120]	97	
		Possible Flow Boundary	[19/162]	101	Noted on log as flow boundary (?)
		Probable Fault	[20/233]	119	
	RKS-15-07	Fault	[50/291], [37/163], [34/220]	130 - 132	3 faults noted on borehole log through this interval
		Healed Fault	[18/137]	149	More at 156.2 [08, 081], 157.5 [03, 345], 161.2 [15, 044]
	RKS-16-07	Possible Flow Boundary	[69/219]	70	Noted on logs as possible
		Probable Fault	[30/175], [43/161]	133	
	RKS-17-07	Possible Fault	[56/213]	109	Noted on borehole log as possible slicken-sides
	Sirovision Mapping				
	1392+30	Fault	[58/228], [60/224]		
		Fault	[55/245]		
	Outcrop Mapping				
	1392+30	Fault	[60/ 225], [48/220]		
	1387+00	Fault	[52/266]; [55/181]		
	1385+25	Fault	[50/195]; [47/175]		
	1383+75	Flow Boundary	Curved - not reliable		
XVIII	RKS-45-08	Probable Fault	[55/184]	29	
		Possible Fault	[54/296]	42	Noted as either shear or joint on log, polished to smooth
XVIII	RKS-46-08	Probable Fault	[43/100]	14	
		Probable Fault	[66/018]	44	
		Fault	[42/155]	53	3 other shear/joints noted between 53 and 54.5 feet
		Fault	[48/325]	59	
		Possible Flow Boundary	[53/079]	65	
	RKS-47-08	Fault	[45/280]	25	
		Possible Fault	[45/245]	32	
		Healed Fault	[39/116], [45/168]	67	

GEOLOGIC SUMMARY: Design Sector VIII Snowshed

Station Interval: 1352+50 to 1363+50

Plans and Sections: Refer to Figures 3 and 12

Overburden Thickness: Encountered approximately 40 to 60 feet from stationing 1352+50 to 1355+50 in the existing cut. Between 1355+50 and 1363+50 overburden thickness above bedrock ranges from 5 to 10 feet in the existing cut with 10 feet and 9 feet of overburden observed in boreholes RKS-08-07 and MRE-05-08 respectively. Overburden is a mix of snow avalanche transported rock debris, colluvial and glacial till deposits.

Rock Type: Meta-welded lapilli dacite tuff

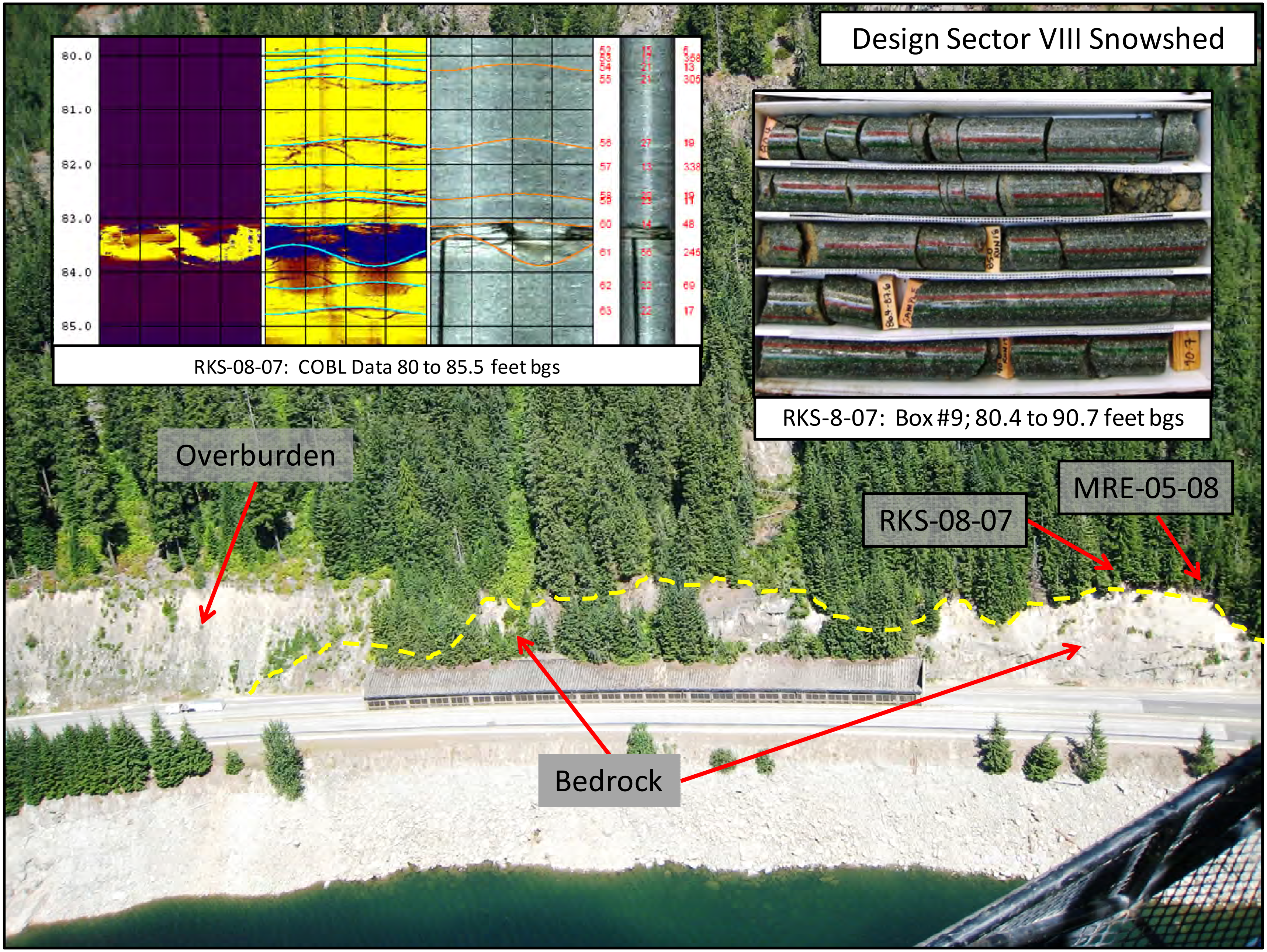
Rock Strength and Weathering: Strong to very strong rock (R4 – R5) when fresh (I). Bedrock was typically fresh (I) with slight weathering (II) in upper 3 to 5 feet of bedrock.

Instrumentation: Dual-stage VWP's were installed in RKS-08-07.

Discontinuities: 4 joint sets

Other Comments:

1. Additional boreholes drilled by URS Corporation for the snowshed design on the east side of the structure. Borehole numbers were SSD-02-07 and SSD-03-07 and several other boreholes for this design were drilled in the ditch north and south of the existing structure (SSD-01-07, SSD-04-07, and SSD-05-07).
2. A flow boundary was present in outcrop at the border between Sector # Snowshed and IX. No measured orientation was available, however the apparent orientation in outcrop is ~45/280.
3. This sector has a high probability of flow boundary intersection during cut slope excavation.
4. Several snow avalanche chutes upslope from the proposed cut slope are present through this sector.
5. A perennial stream is located at the boundary between Sector VIII Snowshed and Sector IX. Runoff has been observed during high rain fall and rain on snow events.


Design Sector VIII Snowshed
RKS-08-07: COBL Data 80 to 85.5 feet bgs
RKS-8-07: Box #9; 80.4 to 90.7 feet bgs
Overburden
MRE-05-08
RKS-08-07
Bedrock

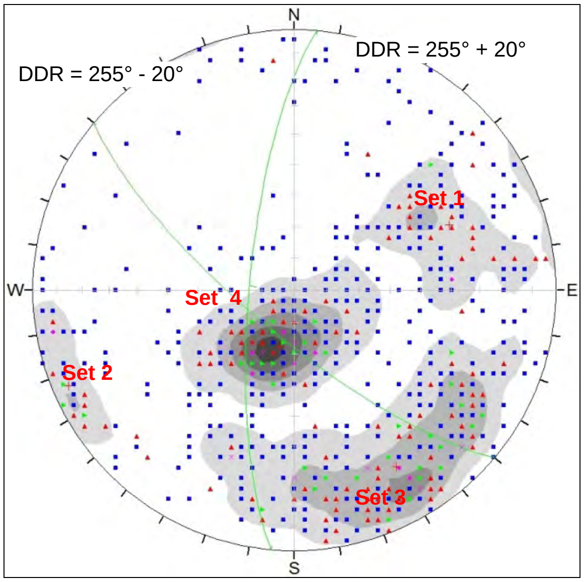
ROCK SLOPE ENGINEERING SUMMARY: Design Sector VIII Snowshed

Station Interval: 1352+50 to 1366+00

Proposed Cut Slopes: 40 ft increasing to 90 ft at S portal (combined soil and rock)

Slope Dip Direction Range (DDR): 240° to 255° (at westbound and eastbound sector limits)

Overall Slope Stability: Slopes will be comprised of Strong to Very Strong dacite tuff. Overall stability will be controlled by structural fabric.



Extensive data set consists of 777 joint measurements and 1 fault measurements. Sirovision and structural mapping reported 605 and 85 measurements respectively, with balance derived from COBL data from boreholes RKS-08. Joint sets:

Set 1: 54°/247°-well-defined, out-of-slope dip
Set 2: 83°/067°-moderately-defined, steep in-slope dip
Set 3: 67°/330°- well-defined, orthogonal to slope
Set 4: 18°/014°-very well-defined, shallow in-slope dip

Set 1 could cause localized or overall planar instability. Combinations of Sets 1 and 3 could form wedges with lines of intersection plunging to the WSW at ± 50°. Rock quality in borehole RKS-8-07 features widely spaced fractures (0 to 1 per foot) in a strong rock mass with several closely fractured zones.

Rockfall Potential: Overburden, upper weathered dacite tuff and joint-defined blocks will be primary rockfall sources. Existing slope performance indicates elevated rockfall incidence is probable. Rockfall control requirements will be based on required protection for south (eastbound) portal structure.

Overburden Stability: Except for portal areas snowshed will be backfilled so overburden stability not an issue. At portal areas loose to medium dense overburden of variable thickness will ravel and erode.

Design Issues: Except for portal areas, rock cuts will be temporary to gain access for footing construction. Reinforcement for temporary cuts to be determined during construction. Rock cuts at south portal to have overburden at 1.5H:1V and rock at ¼H:1V. Local overburden removal is a possible option. South portal rock cuts require extensive pattern reinforcement, possible localized shotcrete application and slope drainage to minimize planar failure potential and to protect portal structure.

Constructability: Moderately inclined slopes above cuts will be favorable for equipment access to start excavation. Limited thickness of overburden may require rock excavation for pioneer road along top-of-cut.

Additional Comments: Monitor geologic conditions during construction.

GEOLOGIC SUMMARY: Design Sector IX

Station Interval: 1363+50 to 1366+00

Plans and Sections: Refer to Figures 4, 13 and 14

Overburden Thickness: Encountered approximately 3 feet in RKS-09-07 and 4 feet in RKS-10-07, 2 to 5 feet observed in existing rock cut. Overburden was primarily colluvial deposits.

Rock Type: Meta-welded lapilli dacite tuff

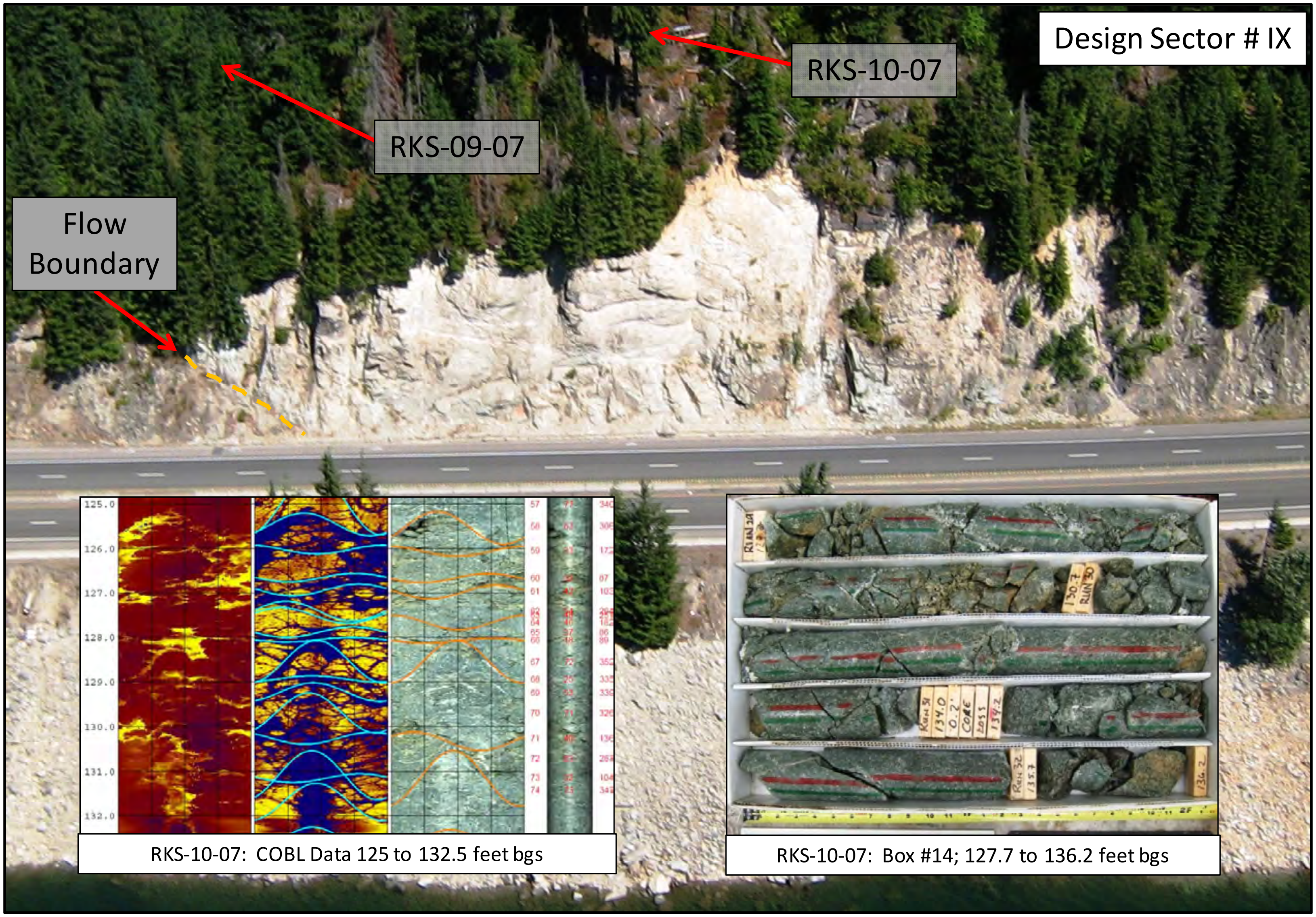
Rock Strength and Weathering: Very weak to moderately weak rock (R1 - R2) where slightly weathered (II) near overburden contact (upper 15 to 20 feet in boreholes), in existing cut slope and along significant discontinuities. Moderately weak to moderately strong rock (R2 – R3) encountered where fresh (I).

Instrumentation: Single-stage VWP's installed in RKS-09-07 and RKS-10-07.

Discontinuities: 4 joint sets and 1 sub-set

Other Comments:

1. The existing outcrop was more susceptible to erosion than adjacent sectors. Talus accumulation in the existing ditch was greater with tabular shaped blocks typically ranging from 6 to 18 inches in length.
2. Closely spaced, well-defined vertical structure was observed along the northern half of the sector.
3. Very poor quality rock (RQD = 0 – 15%) was observed in RKS-10-08 between 125 and 130 feet bgs. Rock was moderately weathered with 0.5 feet of core loss recorded.
4. A flow boundary was present in outcrop at the border between Sector VIII Snowshed and IX. No measured orientation from the outcrop was available; however the apparent orientation is ~45/170. RKS-09-07 encountered a possible intersection of this boundary at 69 feet bgs with orientations measured from the COBL data of 27/187 and 42/211 (Figure 14).
5. This sector has a high probability of flow boundary intersection during cut slope excavation.
6. A perennial stream is located at the boundary between Sector IX and Sector VIII Snowshed. Runoff has been observed during high rain fall and rain on snow events.



RKS-10-07: COBL Data 125 to 132.5 feet bgs

RKS-10-07: Box #14; 127.7 to 136.2 feet bgs

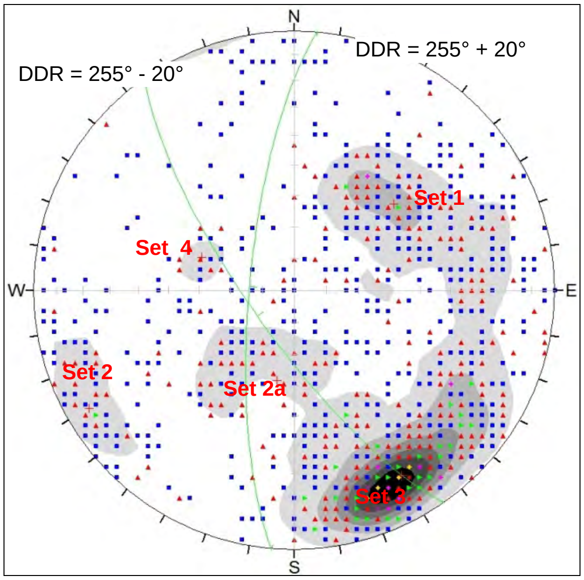
ROCK SLOPE ENGINEERING SUMMARY: Design Sector IX

Station Interval: 1363+50 to 1366+00

Proposed Cut Slopes: 90 to 135 feet (combined soil and rock)

Slope Dip Direction Range (DDR): 255° (uniform tangent)

Overall Slope Stability: Slopes will be comprised of Moderately Weak to Moderately Strong dacite tuff. Overall stability will be controlled by structural fabric.



Extensive data set consists of 1192 joint measurements and 3 fault measurements. Sirovision and structural mapping reported 911 and 60 measurements respectively, with balance derived from COBL data from boreholes RKS-09-07 and RKS-10-07. Joint sets:

Set 1: 42°/229°-well-defined, out-of-slope dip
Set 2: 80°/060°-moderately-defined, steep in-slope dip
Set 2a: 29°/011°- poorly-defined, shallow in-slope dip
Set 3: 71°/330°-very well-defined, normal to slope
Set 4: 31°/110°-poorly-defined, oblique to slope

Set 1 could cause localized or overall planar instability. Combinations of Sets 1 and 3 could form wedges with lines of intersection plunging to the WSW at ± 40°. Rock quality in borehole RKS-10-07 features widely spaced fractures (0 to 1 per foot) in a moderately weak rock mass with a closely fractured zone near the base of the proposed cut.

Rockfall Potential: Overburden, upper weathered dacite tuff and joint-defined blocks will be primary rockfall sources. Existing slope performance indicates elevated rockfall incidence is probable. Based on sector rockfall history and site specific rockfall analyses, ditch catchment per DM M22-01 will have to be supplemented by slope drape for rockfall control for slope heights proposed.

Overburden Stability: Loose to medium dense overburden up to 5 feet thick will ravel and erode.

Design Issues: Design to assume compound slope with overburden at 1.5H:1V and rock at ¼H:1V. Local overburden removal is a possible option. Rock cuts require extensive pattern reinforcement, localized shotcrete application and slope drainage to minimize planar failure potential and possibly to treat flow boundary intersections. Portal structure requires protection from rockfall.

Constructability: Moderately inclined slopes above cuts will be favorable for equipment access to start excavation. Limited thickness of overburden may require rock excavation for pioneer road along top-of-cut.

Additional Comments: Monitor geologic conditions during construction.

GEOLOGIC SUMMARY: Design Sector X

Station Interval: 1366+00 to 1375+50

Plans and Sections: Refer to Figures 4, 5, 15 and 16

Overburden Thickness: Encountered approximately 2 feet (RKS-11-07) to 7 feet (MRE-08-08) of overburden in the boreholes excluding RKS-12-07 and RKS-18-07 which were collared on bedrock. 2 to 5 feet of overburden was observed in existing rock cut which thickened to 10 to 25 feet between stationing 1374+00 to 1375+50. Overburden was primarily colluvial deposits.

Rock Type: Meta-welded lapilli dacite tuff

Rock Strength and Weathering: Moderately strong to strong rock (R3 – R4) where slightly weathered (II) near overburden contact (upper 3 to 10 feet in boreholes), in existing cut slope and along significant discontinuities. Moderately strong to very strong rock (R3 – R5) encountered where fresh (I).

Instrumentation: Single-stage VWP's installed in RKS-12-07 and RKS-18-07. TDR cable installed in RKS-18-07 and RKS-38-08.

Discontinuities: 4 joint sets

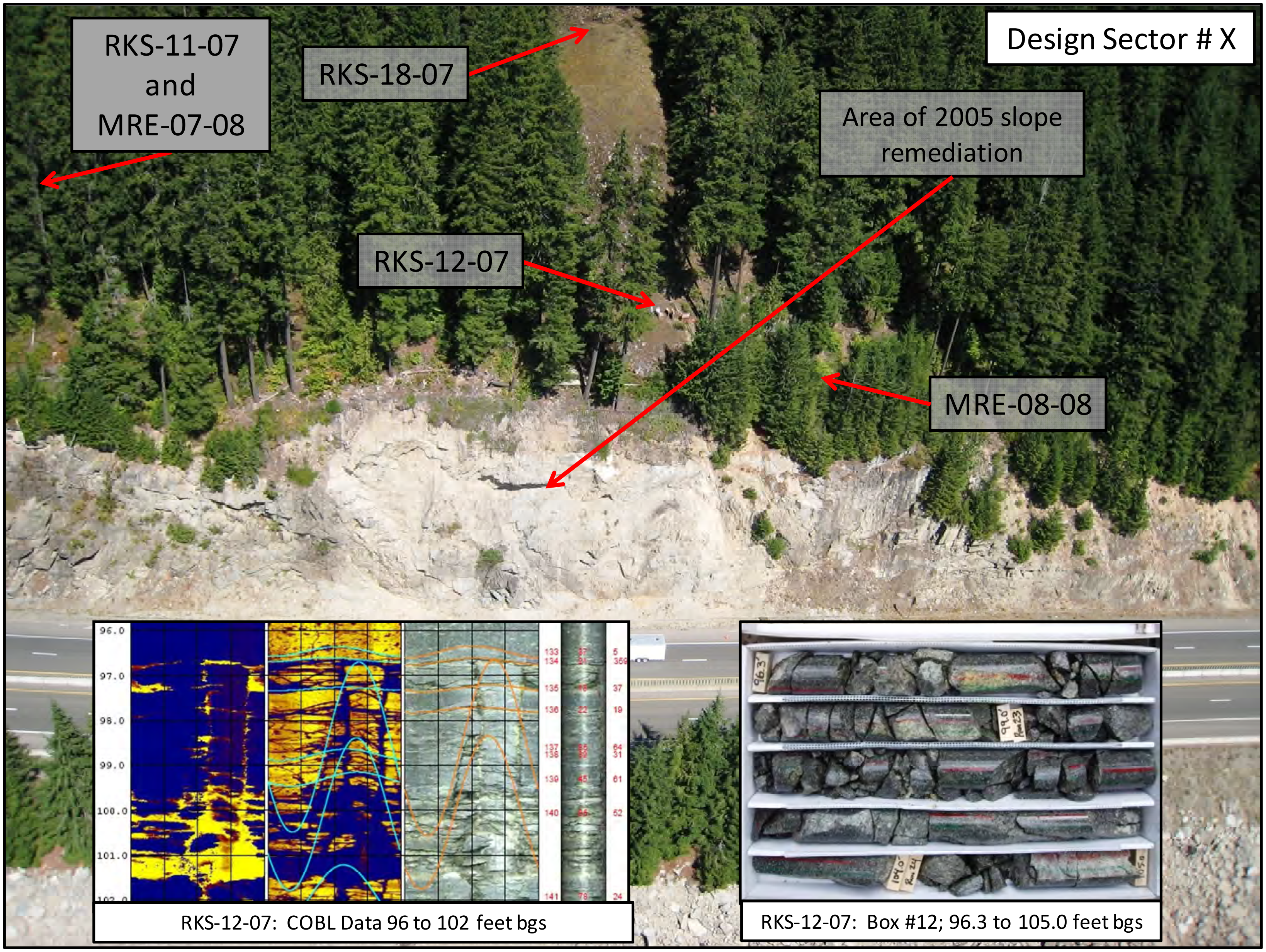
Other Comments:

1. Emergency stabilization was completed in 2005 near Station 1372+00. Stabilization efforts recorded several wide (3 to 12 inches) 'clay filled' discontinuities observed during the rock bolt installation. Seasonal discharge is observed from drain holes installed during the stabilization efforts.
2. Bedrock outcrops are frequent above the top of existing cut from WB station 1369+00 south to the end of the sector. Several of these rock outcrops have a bulbous shaped nose dipping towards the highway. This could be the result of glacial activity or represent terminal and lateral flow margins. These outcrops contribute to intermittent benched and over-steepened terrain above the existing cut. The sector is strongly joint controlled with occasional 100+ foot persistent discontinuities dipping out of slope as noted in the existing slope face during the geologic and structural mapping and above the existing cut at station 1371+50.
3. There is a 200+ foot persistent discontinuity [42/151] observed in the bedrock outcrop which likely represents a flow boundary. Survey data of the discontinuity and borehole locations were used by Haneberg Geosciences to calculate the intersections of the feature in boreholes RKS-12-07 and RKS-18-07 (Appendix A). The discontinuity appears to daylight in the slope above the existing cut at the base of an outcrop. Field observations at the base of this outcrop support this interpretation. Figure 16 provides a cross-sectional view of the feature and borehole intersections. The table below provides a summary of the analysis with the highlighted values representing the most probable borehole intersection.

Borehole	Collar Elevation (feet MSL)	Calculated Discontinuity Intersection (feet MSL)	Significant Discontinuity Intersection Elevation (feet MSL)	
RKS-12-07	2648	2630	2630	2621
		COBL Orientations:	41/192	32/283
RKS-18-07	2746	2701	2702	2698
		COBL Orientations:	38/182	50/198

4. This sector has a high probability of flow boundary intersection during cut slope excavation.

Photo looking east at the persistent discontinuity



RKS-12-07: COBL Data 96 to 102 feet bgs

RKS-12-07: Box #12; 96.3 to 105.0 feet bgs

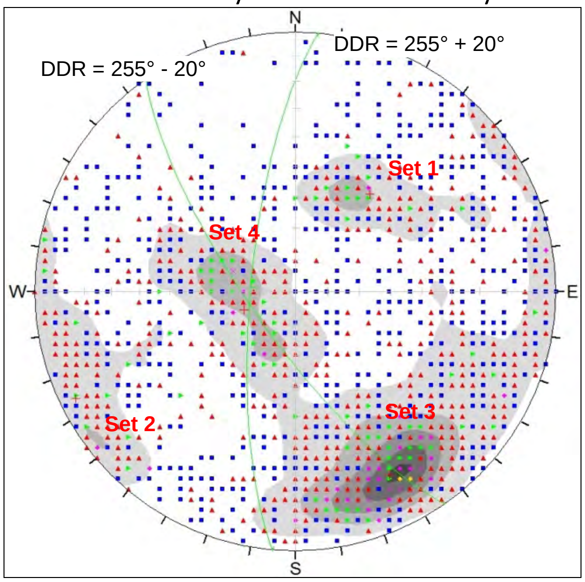
ROCK SLOPE ENGINEERING SUMMARY: Design Sector X

Station Interval: 1366+00 to 1375+50

Proposed Cut Slopes: 105 to 140 feet decreasing to east (combined soil and rock)

Slope Dip Direction Range (DDR): 255° (uniform tangent alignment)

Overall Slope Stability: Slopes will be comprised of Moderately Strong to Very Strong dacite tuff. Overall stability will be controlled by structural fabric.



Extensive data set consists of 2115 joint measurements and no fault measurements. Sirovision and structural mapping reported 1273 and 251 measurements respectively, with balance derived from COBL data from boreholes RKS-11-07, RKS-12-07 and RKS-18-07 and RKS-38-08. Joint sets:

Set 1: 39°/217°- well-defined, out-of-slope dip
Set 2: 83°/064°-moderately-defined, steep in-slope dip
Set 3: 68°/333° -very well-defined, normal to slope
Set 4: 17°/071°- moderately-defined, oblique to slope

Set 1 could cause localized or overall planar instability. Combinations of Sets 1 and 3 could form wedges with lines of intersection plunging to the WSW at 30 to 35°. Set 3 dips steeply into the slopes and will form block release surfaces through toppling. Rock quality in boreholes typically exhibits 1 to 2 fractures per foot with local zones of closely spaced fractures (>5/ft).

Rockfall Potential: Overburden, upper weathered dacite tuff and joint-defined blocks will be primary rockfall sources. Existing slope performance indicates elevated rockfall incidence is probable. Based on sector rockfall history and site specific rockfall analyses, ditch catchment per DM M22-01 will have to be supplemented by slope drape for rockfall control for slope heights proposed.

Overburden Stability: Loose to medium dense overburden up to 8 feet thick will ravel and erode.

Design Issues: Design to assume compound slope with overburden at 1.5H:1V and rock at ¼H:1V. Local overburden removal is a possible option. Rock cuts require extensive pattern reinforcement, localized shotcrete application and slope drainage to minimize planar failure potential and possibly to treat flow boundary intersections. Outcrop above top-of-cut at 1372+40 WB will require reinforcement.

Constructability: Limited thickness of overburden and irregular topography will require rock excavation for pioneer road along top-of-cut.

Additional Comments: Monitor geologic conditions during construction.

GEOLOGIC SUMMARY: Design Sector XI

Station Interval: 1375+50 to 1382+50

Plans and Sections: None referenced.

Overburden Thickness: 0 to greater than 20 feet observed in existing outcrops. The overburden is a mix of colluvial and glacial till deposits.

Rock Outcrop Type: Meta-welded lapilli dacite tuff

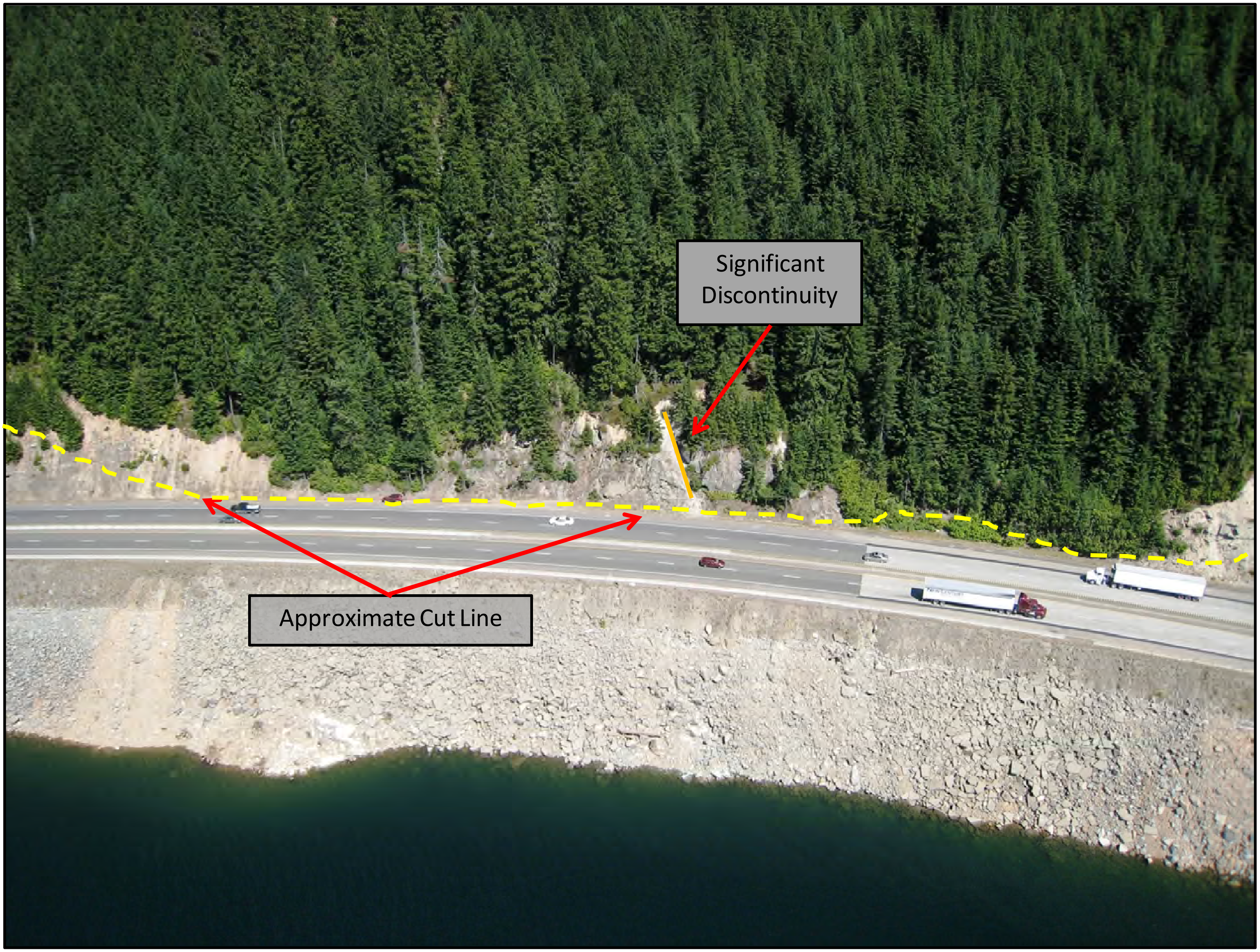
Rock Strength and Weathering: Moderately strong rock (R3) and slightly weathered (II) in outcrop.

Instrumentation: None

Discontinuities: 3 joint sets

Other Comments:

1. Rock outcrops are intermittent throughout sector.
2. At WB station 1379+50 is a bedrock outcrop with a persistent (greater than 75 feet) discontinuity dipping obliquely out of the slope (refer to photo below).
3. Approximately 300 feet of rock outcrop observed along the central portion of the sector.


Significant
Discontinuity
Approximate Cut Line

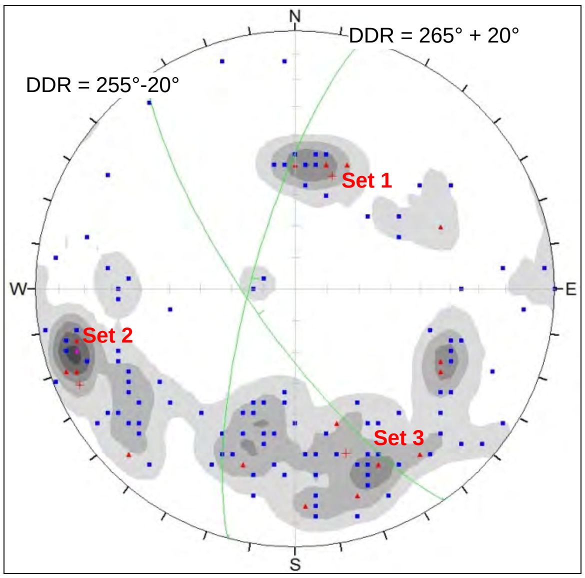
ROCK SLOPE ENGINEERING SUMMARY: Design Sector XI

(Note: Design Sector XI is an at-grade interval with minimal cuts and fills. Existing rock slopes will require stabilization.)

Station Interval: 1375+50 to 1382+50 **Proposed Cut Slopes:** nominal

Slope Dip Direction Range (DDR): 255° to 265° (at westbound and eastbound sector limits)

Overall Slope Stability: Existing rock slopes are comprised of Moderately Strong dacite tuff. Overall stability is controlled by structural fabric and not by rock mass strength.



Limited data set consists of 144 joint measurements and no fault measurements. 86 data points derived from Sirovision mapping and 58 from structural mapping. Joint sets:

Set 1: 38°/198° °-well-defined, out-of-slope dip
Set 2: 80°/066°-well-defined, steep in-slope dip
Set 3: 56°/343° -dispersed, normal to slope

Set 1 could cause localized or overall planar instability. Combinations of Sets 1 and 3 could form wedges with lines of intersection plunging to the WSW . Set 3 dips steeply into the slopes and will form block release surfaces through toppling.

Rockfall Potential: Low probability of rockfall from weathered dacite tuff and overburden. For existing slopes, proposed ditch catchment per DM M22-01 should be adequate for rockfall control.

Overburden Stability: Loose overburden will not be influenced by any new cuts.

Design Issues: Preliminary design issue relates to stabilization requirements to minimize large-scale block failures that could overwhelm the ditch catchment. Existing rock cuts will require localized reinforcement to minimize planar and wedge failure potential.

Constructability: No constructability issues pertaining to new cuts. Existing cuts will be accessed for stabilization using cranes operating from the current grade.

Additional Comments: None.

GEOLOGIC SUMMARY: Design Sector XII

Station Interval: 1382+50 to 1396+50

Plans and Sections: Refer to Figures 6, 7, 8, 17, 18, 19, 20, and 21

Overburden Thickness: Encountered approximately 3 to 7 feet of overburden in the boreholes and 3 to 10 feet observed in existing rock cut. The overburden comprises colluvial deposits from 1382+50 to 1395+50. From 1395+50 anthropogenic (man-made) fill with colluvium is predominate in the re-graded area through the end of the sector with overburden thickness increasing to 10 to 15 feet.

Rock Outcrop Type: Meta lapilli dacite tuff with variable welding observed

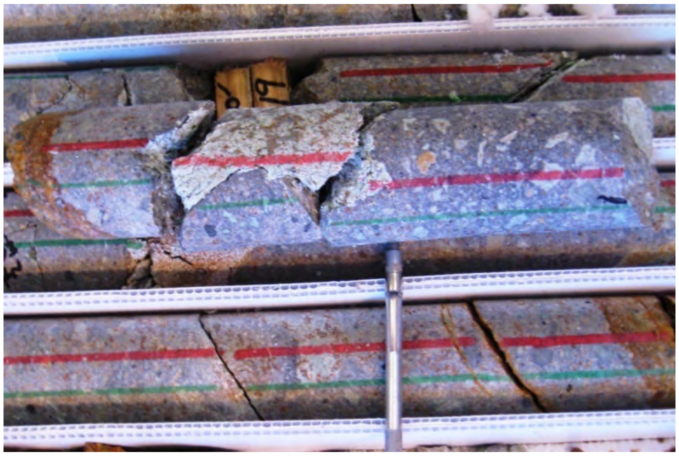
Rock Strength and Weathering: Extremely weak to moderately weak rock (R0 to R2) for slightly to highly weathered rock (II – IV), moderately strong to strong rock (R3 – R4) for fresh (I) bedrock. Weathering was highly variable between and within the various boreholes. Moderately to highly weathered rock was encountered along significant discontinuities (faults and joints) and within the upper 15 to 25 feet of the bedrock. In general, RKS-13-07, RKS-15-07 and RKS-41-08 encountered predominantly slightly to highly weathered throughout the borehole with zones of fresh rock. The remaining boreholes encountered fresh rock with slightly to moderately weathered rock proximal to discontinuities and in upper portions of bedrock.

Instrumentation: Dual stage VWP's were installed in H-110-06, RKS-13-07, RKS-15-07 and RKS-40-08. A single-stage VWP was installed in RKS-19-07. TDR Cable was installed in H-110-06, RKS-40-08 and RKS-41-08.

Discontinuities: 4 joint sets and 1 sub-set, 1 fault set

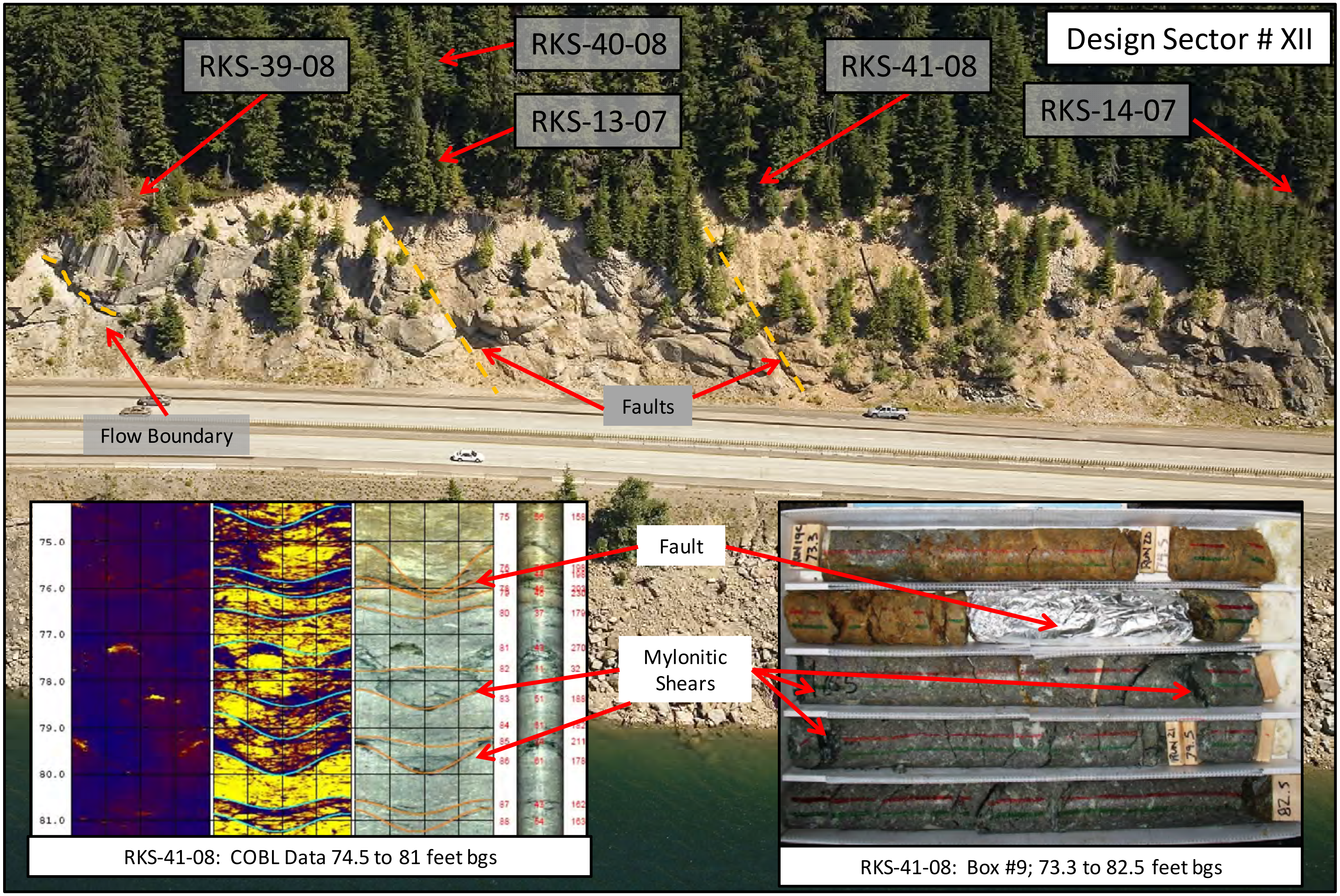
Other Comments:

1. Several high angle faults dipping to the south [60/225; 52/266, 50/195] were observed in the existing rock cut through this sector. All boreholes drilled in this sector intersected possible faults. Orientations were generally 40 to 70 degrees, dipping to the southwest (refer to Table 10 and Figure 49).
2. A flow boundary was observed in outcrop on the northern end of the sector (WB station 1383+75) and was intersected by RKS-39-08. Several possible flow boundaries were noted in other boreholes. When available, orientations of the flow boundaries were consistently dipping to the southwest with variable dips (refer to Table 10 and Figure 49).
3. Engineering Domains 1 through 3 were observed in this sector. Moderately weathered Domain 3 comprised most of the existing cut slope with Domain 1 observed adjacent to the flow boundary (WB station 1383+75 to 1384+75) and near the center of the sector (WB station 1389+00 to 1390+50).
4. Frequent rockfall (less than 6 inches to 3 feet) accumulated at the toe of the slope, below outcrops of Domain 3 to form a continuous talus wedge that is periodically removed or graded by WSDOT Maintenance personnel.



5. Domain 2 rock observed in boreholes drilled in 2007 and reviewed in 2008 showed spotty iron staining of fresh rock and swelling clays present in altered pumice fragments (refer to margin photo from RKS-19-07), damaging the existing core. This alteration occurred within one year. The presence of these clays and high susceptibility to weathering further support that Domain 2 rock will degrade in cut faces over a life-of-highway timeframe.
6. Water seepage or flow was noted along the existing cut for all boreholes drilled within this sector except RKS-14-07 and RKS-40-08. Actual water flow was observed below RKS-17-07 at the face of the cut and was contained to one location while the other boreholes seeped water at several locations. All seepage or flow was associated with a significant discontinuity (i.e.: fault or flow boundary).
7. Crest failure was observed along the top of existing cut whenever Domain 3 rock was present. Most of the failures appeared to be less than 25 cubic yards in size with two or three locations estimated at approximately 50 cubic yards of material.
8. Between WB station 1382+50 and 1388+50 poor rock quality was observed both in outcrop and in borehole. Several faults and at least one flow boundary are present within this interval. 9 to 16 foot thick zones of poor rock quality (due to faulting or flow boundaries) were observed in RKS-13-07, RKS-39-08 and RKS-41-08 that included up to 6 feet of cumulative core loss in RKS-39-08 between 72 and 94 feet bgs. Upslope borehole RKS-40-08 encountered better rock quality but significant discontinuities at depth increase concerns regarding global stability of the rock slope. Figure 48 provides an example of discontinuity intersections in borehole RKS-39-08.
9. Overburden thickness above RKS-19-07 exceeds 10 feet due to fill material placed during the historic re-grading activities of Slide Curve.

1


Design Sector # XII
RKS-39-08
RKS-40-08
RKS-13-07
RKS-41-08
RKS-14-07
Faults
Flow Boundary
Fault
Mylonitic Shears
RKS-41-08: COBL Data 74.5 to 81 feet bgs
RKS-41-08: Box #9; 73.3 to 82.5 feet bgs

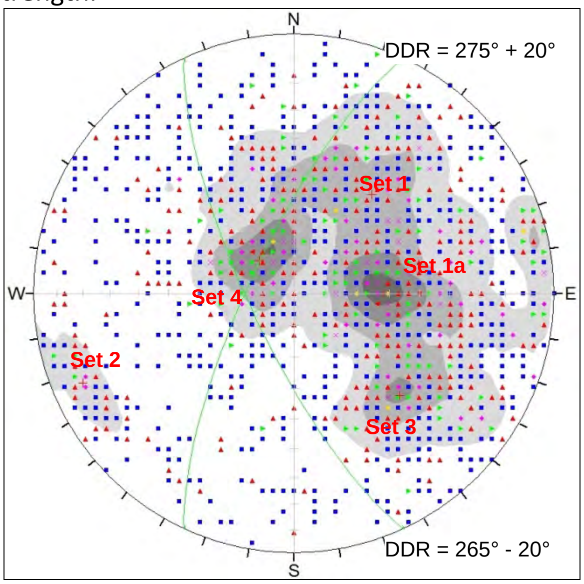
ROCK SLOPE ENGINEERING SUMMARY: Design Sector XII

Station Interval: 1382+50 to 1396+50

Proposed Cut Slopes: Variable 60 to 125 feet (combined soil and rock)

Slope Dip Direction Range (DDR): 265° to 275° (at westbound and eastbound sector limits)

Overall Slope Stability: Slopes will be comprised of variable strength rock dependent on weathering intensity. Overall stability will be controlled by both structural fabric and rock strength.



Extensive data set consists of 1707 joint measurements, 20 fault measurements and 3 bedding measurements. Sirovision and structural mapping reported 290 and 392 measurements respectively, with balance derived from COBL data from boreholes H-110-06, RKS-13-07 through RKS-17-07, RKS-19-07 and RKS-39-08 through RKS-41-08.

Joint sets:

Set 1: 40°/218°-well-defined, out-of-slope dip
Set 1a: 28°/270°- very well-defined, out-of-slope dip
Set 2: 77°/067°-moderately-defined, steep in-slope dip
Set 3: 47°/314°-well-defined, oblique to slope
Set 4: 15°/134°-well-defined, oblique to slope

Sets 1 and 1a could cause localized or overall planar instability. Combinations of Sets 1 and 3 could form wedges with lines of intersection plunging to the WSW at 30 to 35°. Set 3 dips will form block release surfaces through toppling. Rock quality in boreholes typically exhibits 1 to 3 fractures /ft.

Rockfall Potential: Overburden, upper weathered dacite tuff and joint-defined blocks will be primary rockfall sources. Existing slope performance indicates elevated rockfall incidence is probable. Based on sector rockfall history and site specific rockfall analyses, ditch catchment per DM M22-01 will have to be supplemented by slope drape for rockfall control for slope heights proposed.

Overburden Stability: Loose overburden up to 10 to 15 feet thick toward east will ravel and erode until vegetated.

Design Issues: Preliminary design to assume compound slope with overburden at 1.5H:1V and rock at ¼H:1V. Overburden removal is a possible option. Rock cuts will require extensive reinforcement (pattern layout in places) to minimize planar and toppling failure potential. Extensive application of shotcrete anticipated along fault zones and highly weathered areas exposed in the new cut faces.

Constructability: Steeply inclined slopes above cuts will be unfavorable for equipment access to start excavation. Limited thickness of overburden may require rock excavation for pioneer road construction along top-of-cut.

Additional Comments: Monitor geologic conditions during construction.

GEOLOGIC SUMMARY: Design Sector XVII

Station Interval: 1442+50 to 1468+00

Plans and Sections: Refer to Figures 9, 10, 22 and 23

Overburden Thickness: Overburden thickness ranged from 12 feet (RKS-42-08) to 28 feet (RKS-45-08) in the boreholes. No rock outcrops were observed in the existing highway cut slope. Overburden comprised glacial till with occasional boulders up to 4 feet in diameter observed on the surface.

Rock Type: Various rock types were encountered including sandstone (RKS-42-08), meta-volcanics and tuffs (RKS-43-08 and RKS-45-08) and meta-basalt (RKS-44-08).

Rock Strength and Weathering: Rock strength was generally moderately weak to moderately strong rock (R2 – R3) for all rock types when fresh (I). Slightly weathered (II) rock was observed in the upper 2 to 6 feet of bedrock and along significant discontinuities with rock strength typically moderately weak rock (R2).

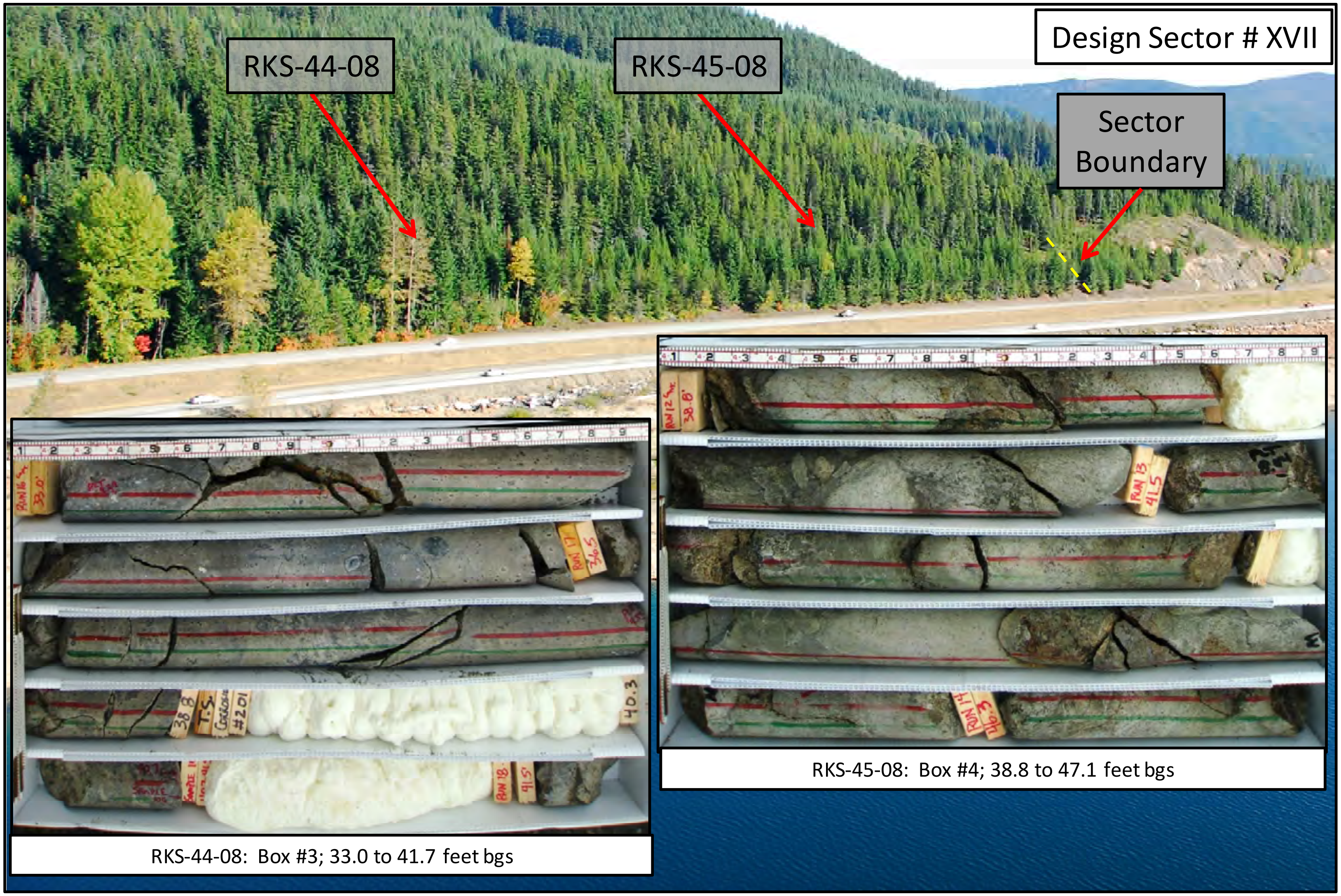
Instrumentation: Dual-stage VWP's were installed in RKS-44-08.

Discontinuities: 4 joint sets

Other Comments:

1. Existence of bedrock was not confirmed in the existing cuts along this sector. Possible bedrock was observed at approximately WB station 1447+00 to 1448+00.
2. Several possible glacial erratics were observed on the ground surface up to 4 feet in diameter (refer to margin photograph). Clast sizes observed in the boreholes did not exceed 1 foot in diameter.
3. A topographic low and drainage is present between 1456+00 and 1460+00 where cut heights are limited. The area appears to have been previously logged as evidenced by the uneven terrain and frequency of overgrown nurse logs proximal to the stream channel.

4. Rock types were various with all four (4) boreholes in this sector encountering a different geologic unit.
5. Borehole RKS-44-08 encountered bedrock with strong hydrothermal alteration and significant sulfide mineralization (pyrite) with concentrations up to 5 percent.
6. Overburden thickness will decrease into Design Sector XVIII.



RKS-44-08: Box #3; 33.0 to 41.7 feet bgs

RKS-45-08: Box #4; 38.8 to 47.1 feet bgs

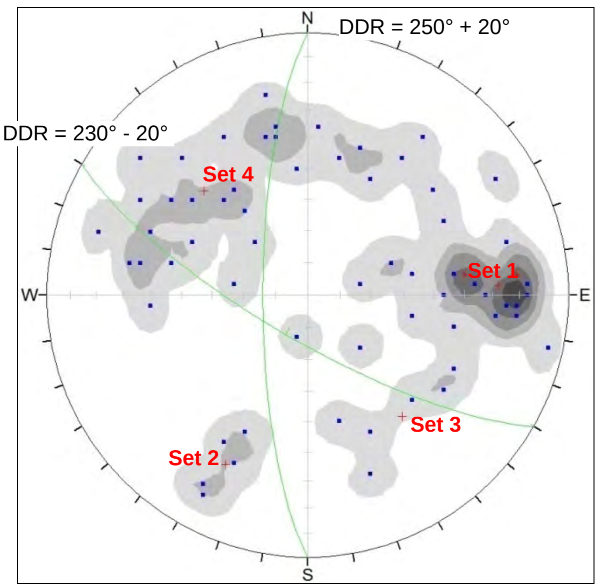
ROCK SLOPE ENGINEERING SUMMARY: Design Sector XVII

Station Interval: 1442+50 to 1468+00

Proposed Cut Slopes: 10 to 70 feet (combined soil and rock)

Slope Dip Direction Range (DDR): 230° to 250° (at westbound and eastbound sector limits)

Overall Slope Stability: Slopes will be comprised of Moderately Weak to Moderately Strong volcanic and sedimentary rock types. Overall stability will be controlled primarily structural fabric and locally by rock mass strength.



Data set consists of 66 joint measurements and 2 fault measurements all derived from COBL data from boreholes RKS-42-08 through RKS-45-08. Joint sets:

Set 1: 62°/267° - well-defined, out-of-slope dip
Set 2: 61°/026° - poorly-defined, oblique to slope
Set 3: 49°/322° - well-defined, oblique to slope
Set 4: 46°/135° - well-defined, oblique to slope

Set 1 develops planar slabs that will affect localized or overall stability.

Rock quality in boreholes feature widely spaced fractures (0 to 2 per ft) in a moderately weak / moderately strong rock mass.

Rockfall Potential: Upper weathered meta-basalt, meta-tuff, sandstone and overburden will be primary rockfall sources. For slope heights proposed, ditch catchment per DM M22-01 should be adequate for rockfall control.

Overburden Stability: Medium dense to dense overburden up to 30 feet thick will ravel and erode until vegetation is established.

Design Issues: Design to assume compound slope with overburden at 1.5H:1V or flatter, rock at ½H:1V.

Constructability: Moderately inclined slopes above cuts will be favorable for equipment access to start excavation. Variable thickness of overburden should be accommodated with variable width bench on top of bedrock or by means of temporary steep cut on pioneer road followed by final top-of-cut staking.

Additional Comments: Monitor geologic conditions during construction.

GEOLOGIC SUMMARY: Design Sector XVIII

Station Interval: 1468+00 to 1480+00

Plans and Sections: Refer to Figures 11 and 24

Overburden Thickness: Overburden thickness observed in boreholes ranged from 3 feet (RKS-46-08) to 13 feet (RKS-47-08). Overburden thickness in the existing highway cut slope ranged from a few feet to more than 25 feet with no observed bedrock outcrop between station 1468+00 to 1469+50 and 1473+25 and 1474+50. Overburden comprised of glacial till.

Rock Type: Meta-andesite or meta-basalt and a silicic volcanic rock were encountered in RKS-46-08 and RKS-47-08 respectively, and in the existing outcrop proximal to the borehole locations.

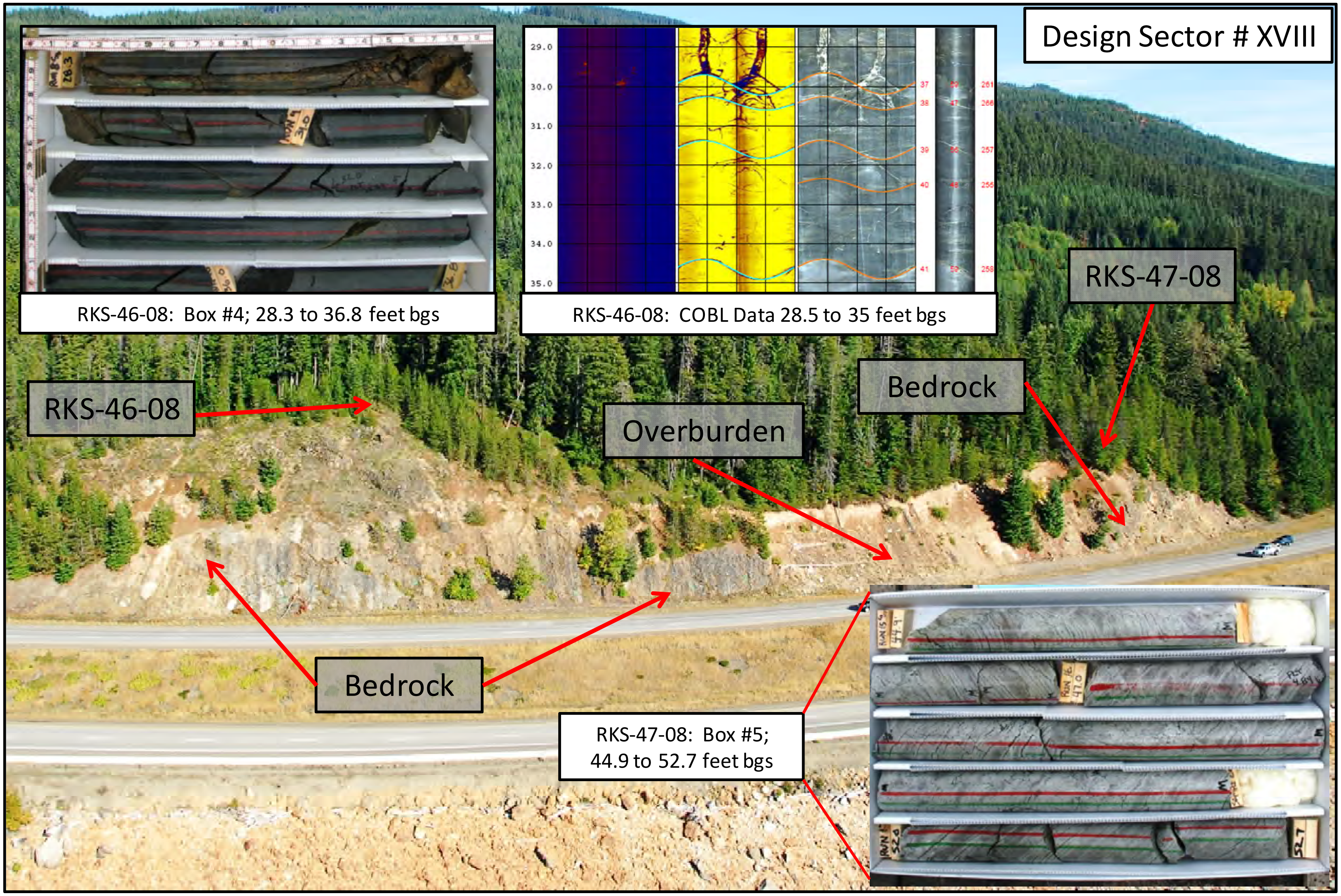
Rock Strength and Weathering: Rock strength was generally strong to very strong rock (R4 – R5) for the fresh (I) meta-basalts and moderately strong (R3) when slightly weathered (II). The silicic volcanics were moderately strong to strong (R3 – R4) when fresh (I) and moderately weak where slightly weathered (II).

Instrumentation: Dual-stage VWP's were installed in RKS-46-08.

Discontinuities: 4 joint sets

Other Comments:

1. The northern end of this sector is a transition zone as bedrock elevation rises between boreholes RKS-45-08 (~ 2560) and RKS-46-08 (~ 2615 MSL) between sectors XVII and XVIII. The boreholes are separated by approximately 475 feet. There is no bedrock outcrop in the existing cut until approximately 1469+50.
2. No observed bedrock in outcrop between 1473+25 and 1474+50 with existing slope comprised of glacial till. Overburden thickness decreases (with bedrock exposure increasing) to the north and south of this interval with generally thicker overburden deposits to the south.
3. Existing cut slopes in the meta-basalts and meta-andesites are in good condition. The exposed face was fresh to slightly weathered with limited potential for rock fall.
4. Existing cut slope in the silicic volcanic (approximately WB station 1474+50 to 1477+00) is slightly to moderately weathered and does not appear to perform well in outcrop with apparent preferential erosion along foliation and rock fall development block size of up to 2 feet (refer to margin photograph).
5. The silicic rocks were foliated and slightly weaker parallel to the foliation [~55/ 260].


Design Sector # XVIII
RKS-46-08: Box #4; 28.3 to 36.8 feet bgs
RKS-46-08: COBL Data 28.5 to 35 feet bgs
RKS-47-08
RKS-46-08
Bedrock
Overburden
Bedrock
RKS-47-08: Box #5;
44.9 to 52.7 feet bgs

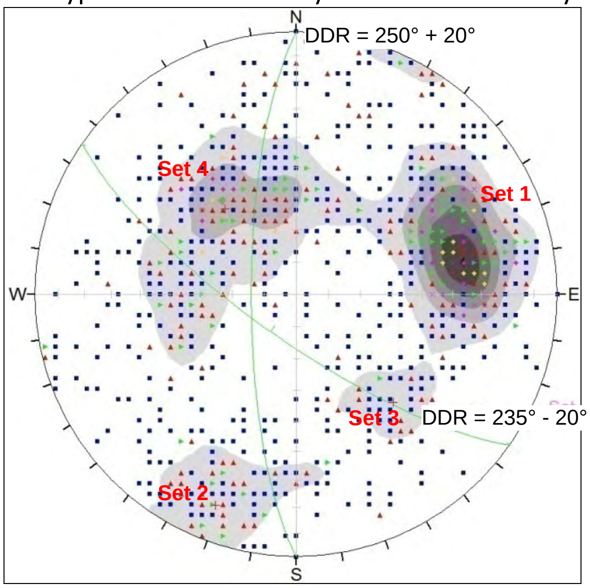
ROCK SLOPE ENGINEERING SUMMARY: Design Sector XVIII

Station Interval: 1468+00 to 1480+00

Proposed Cut Slopes: 25 to 80 ft (combined soil and rock)

Slope Dip Direction Range (DDR): 250° to 235° (at westbound and eastbound sector limits)

Overall Slope Stability: Slopes will be comprised of Moderately Strong to Very Strong volcanic rock types. Overall stability will be controlled by structural fabric.



Extensive data set consists of 1230 joint measurements and 8 fault measurements. Data sources: Sirovision 959 measurements, structural mapping 44 measurements, COBL 227 measurements from boreholes RKS-46-08 and RKS-47-08. Joint sets:

Set 1: 55°/255° - very well-defined, out-of-slope dip
Set 2: 75°/021° - moderately defined, steep in-slope dip
Set 3: 46°/318° - moderately defined, oblique to slope
Set 4: 35°/140°- well-defined, oblique to slope

Set 1 could define localized or overall planar instability. Sets 1and 2 could form wedges with intersections plunging to NW at ±40°.

Rock quality shows 0 to 2 fractures per foot with occasional closely fractured intervals.

Rockfall Potential: Upper weathered basalt and andesite, overburden and joint-defined blocks will be primary rockfall sources. For slope heights proposed, ditch catchment per DM M22-01 should be adequate for proposed slope heights (See Section 8.4).

Overburden Stability: Medium dense to dense overburden generally less than 30 feet thick will ravel and erode until vegetation is established.

Design Issues: (see Section 8.3) Design to assume compound slope with overburden at 1.5H:1V or flatter, rock at ½H:1V.

Constructability: Moderately inclined slopes above cuts will be favorable for equipment access to start excavation. Variable thickness of overburden should be accommodated with variable width bench on top of bedrock or by means of temporary steep cut on pioneer road followed by final top-of-cut staking.

Additional Comments: Monitor geologic conditions during construction.

8.0 ROCK SLOPE STABILITY ANALYSES

8.1 ROCK STRENGTH

Slope design is concerned with two distinct categories of rock strength. In situations where the potential slope instability is in the form of blocks defined by discontinuities[1], then the rock strength of interest is the shear strength of the fault, joint or bedding plane along which movement may occur. Discontinuity strength is typically determined in the laboratory by means of direct shear tests on representative samples of the feature. The results of shear tests are usually interpreted through a linear relationship between the effective normal stress acting across the discontinuity and the shear stress necessary to initiate movement. This is referred to as the Mohr-Coulomb strength criterion and is fully expressed by the friction angle, **φ**, (slope of the shear stress – normal stress line) and the cohesion, **c** (intercept on Y axis when normal stress is zero). In more advanced analyses where the effects of discontinuity roughness are included, a curvilinear relationship between normal stress and shear stress is utilized. The tangent to a curvilinear envelope defines instantaneous values for **φ** and **c** at a specified effective normal stress.

The second major category of rock strength applies to non-structurally controlled instability where the failure surface propagates through the rock mass, partially along discontinuities of limited persistence and partially through intact rock. The shear strength of a rock mass with this mode of failure is almost impossible to measure through laboratory testing because of the scale effect; it is not possible to test a volume of rock sufficiently large so as to be representative of the rock mass. Of the two alternatives to this dilemma, the most favorable is the back analysis of failures in slopes comprised of the same material. This is analogous to a very large-scale direct shear test. The second approach, and the one that rock mechanics researchers have been striving to perfect, is an empirical approach wherein the intact rock and the structural fabric are independently characterized through observation and index testing. From these data an empirical relationship can be derived for the expected engineering properties of the rock mass, namely deformability and shear strength. One such approach to this relationship is referred to as the Hoek-Brown Strength Criterion after the namesakes (Hoek and Brown (1981), Hoek et al., (2002)). The method defines a curvilinear shear strength envelope that can be directly input to software to analyze slope stability.

8.1.1 Discontinuity Shear Strength

For Phase 1C rock slope design the shear strength that is also of interest is that applicable to discontinuities, specifically the parameters friction angle, **φ**, and cohesion, **c**. As summarized in Table 7, direct shear tests were performed on a variety of natural and artificial surfaces to determine these values. The average friction angle results can be summarized as follows:

---

[1] Discontinuity is a general term for a plane of weakness in a rock mass. Genetic types include faults, joints, contacts, bedding planes, cleavage, schistosity, etc.

- 7 tests on saw cut surfaces: φ = 28 to 33° (this is referred to as φ_b)
- 1 test on natural joints: φ = 30 to 32° (this is referred to as $\varphi_b + i$, where i = roughness angle)
- 11 tests clay-filled joints: φ = 14 to 15°

The range of values represents initial and final test cycle reported by the laboratory (See Appendix H).

The difference between the friction values for saw cut and natural joints relates to the degree of roughness characteristic of the feature. Typically smooth surfaces are selected for small-scale direct shear tests and the natural roughness component tends to be under represented. Detailed structural mapping recorded the roughness characteristics so that the increment to the total friction angle could be estimated (Appendix B).

Table 8 summarizes the Atterberg Limit testing performed to characterize the clays recovered from flow boundaries and from fault gouge. The results of the seven tests indicate a mean plasticity index of 14 with a range from 6 to 31. The latter value was an outlier as all other reported plasticity index (PI) values were less than 20. Using empirical relationships between PI and residual friction angles proposed by Mesri and Shahien (2003), the shear strength of these features can be estimated at φ = 25° for the average PI and 15° to 17° for the highest PI value (shear zone from RKS-35-08 at 51 ft BGS).

Based on the foregoing, engineering judgment and shear strength evaluations for other portions of the I-90 Snoqualmie East project, design values were selected for Phase 1C as follows:

Natural joints: φ_b = 33°, i = 5°, total friction $\varphi_b + i$ = 38°, cohesion **c** = 0
Clay-filled discontinuities: φ = 25°, cohesion **c** = 0

8.1.2 Rock Mass Shear Strength

Under the Hoek-Brown criterion, the rock mass behavior is a function of the ***intact*** rock properties (as developed through laboratory testing) and the nature and orientation of the discontinuities (joints, bedding planes, etc.) that intersect the rock mass. For the purposes of this project the approach developed by Hoek et al (2002) has been followed. This method incorporates the Rock Mass Rating (RMR)[2] value as proposed by Bieniawski (1989) and the intact rock strength value to calculate rock mass shear strength.

For Design Sectors where rock mass shear strength was required the stratigraphy was idealized into rock type units for which representative values for Rock Quality Designation (RQD), Fracture Frequency per foot (F/ft) and intact rock strength were calculated. From these values and the nature of the discontinuity surfaces recorded in the structural core logging, the RMR

[2] For the I-90 project the RMR system was adopted rather than the more recent Geological Strength Index (GSI) system. The GSI and RMR values are nominally interchangeable for the range of rock qualities on this project.

value for each idealized engineering unit was calculated. Rocscience program RocLab, Version 1.031, was used to generate the curvilinear shear strength envelopes for each rock type.

8.2 GROUNDWATER CONDITIONS FOR SLOPE DESIGN

As detailed in Section 5.6.1, the exploration program for the Phase 1C alignment included the installation of VWP instruments to capture the variation in groundwater pressures within the existing rock slopes. Typically two instruments were installed in a single borehole to measure the head variability with depth below the ground surface. If a simple hydrostatic groundwater regime was present, the piezometers in a single borehole would measure equivalent total heads, that is, pressure head plus elevation head. However, low permeability intervals such as unfractured bedrock, clay-rich faults or flow boundaries can lead to groundwater "compartmentalization" reflected as differing total heads for piezometers within a single borehole.

The VWP instruments were monitored by WSDOT using a remote automatic data acquisition system (ADAS) that recorded water level readings every two hours. With the exception of brief intervals lost due to equipment malfunction or wire damage, this data set provides a near continuous record of groundwater fluctuation within the existing rock slopes. Available piezometric records for the Phase 1C project date from November 2007 to present for Design Sector IX, X and XII and are shown in Figures 30 through 33. The salient features of these records are:

- Groundwater levels exhibit a strong seasonal periodicity.
- The rise in head between the summer and winter ranges from 20 to 60 feet.
- The increase in head from seasonal low to seasonal high is more rapid (±1½ months) compared to the decrease from seasonal high to seasonal low (± 4 months).
- The variability of seasonal high values between October and May reflects transient storm and runoff events.
- Deeper piezometers measure lower heads than do shallow piezometers indicative of "perched" water tables (Figure 32).

From a rock slope design perspective, the important conclusion from the VWP instrumentation data is that discontinuities will be regularly subject to transient elevated pressures; a phenomena consistent with rockfall experience and drilling for emergency stabilization in the Snoqualmie Pass vicinity (for example, MP 50 and MP 58 in 2005).

8.3 SECTOR-SPECIFIC ROCK SLOPE DESIGN ISSUES

Rock slope design employed the following steps:

- Kinematic analysis to compare preferred discontinuity orientations to the orientations and inclinations of proposed cut slopes.
- From kinematic analysis determine if structural control for overall slope stability is present.

- Design for overall cut slope stability for feasible failure mechanisms to achieve margins of safety in accordance with WSDOT (2005).
- Evaluate rockfall potential and rockfall control methodology. If ditch catchment area is inadequate perform analyses to design mechanical enhancement.

For those sectors where slope stabilization measures are recommended, such measures have been categorized as either “Prescribed” or “Provisional”. For the former a design layout and quantity estimate is provided while for the latter only the quantity estimate is provided.

Stabilization for most of the Design Sectors includes the installation of “crest dowels”. It is important that these dowels be installed prior to drilling and blasting the first lift of rock. The location of the dowels relative to the rock crest is dependent on the overburden template at that site. In general the dowels should be three to five feet (measured horizontally) behind the rock crest as defined by the commencement of controlled blast hole drilling.

8.3.1 Design Sector VIII (Snowshed)

The detailed kinematic analysis of the structural geology for Design Sector VIII is shown in Figure 34. Potential planar failure along Set 1 with mean orientation of 54°/247° is indicated. Similarly, potential wedges formed by the intersection of Sets 1 and 3 are present. The plunge and trend of the wedge intersection is estimated at 52° / 271°, directly out of the slopes. Set 1 is only moderately defined thereby limiting the potential for large-scale planar or wedges blocks.

As shown in Figure 12, rock cuts in Design Sector XIII are required to provide access for the snowshed footing excavation and construction. These will be temporary cuts as the final design specifies that the space between the wall and the cut be backfilled to the elevation of the snowshed roof. It is therefore appropriate that the contract plans designate these cuts as temporary construction cuts and stipulate that the contractor be responsible to select cut slope inclinations and to provide necessary rock reinforcement and rockfall control. WSDOT should monitor the geologic conditions to ensure that no potential rock instability is present that could affect the long term integrity of the snowshed. If such is determined to be the case, the cost of any additional long term stabilization measures should be borne by WSDOT.

The east portal will be vulnerable to slope instability and rockfall impact from the immediately adjacent rock cut. The portal design includes a walkway overpass structure with two towers to provide enclosures for the stairways (Figures 35 and 36). The inboard portal tower will be particularly vulnerable to rockfall impacts due to its location (Figure 36). Enhanced slope stabilization and rockfall control measures are required to protect this tower. These requirements straddle the Design Sector VIII and IX boundary and are discussed in Section 8.3.2 following.

8.3.2 Design Sector IX

Figure 37 shows the kinematic stability analysis for the tangent highway alignment for Design Sector IX. This analysis indicates that the primary mode of potential instability is planar failure along Set 1 with Set 2 acting as a release surface. Although Set 2 is also unfavorably oriented for block topping, the joints are not sufficiently persistent or closely spaced to cause overall slope instability. Localized rockfall by block toppling should be anticipated. Figure 37 also identifies possible wedge failures defined by Sets 1 and 3 and with an intersection orientation of 39°/256°. Due to the steep inclination of Set 3 such wedges will be highly asymmetrical. As discussed in Section 7.6, numerous flow boundaries were identified in Design Sector IX. These features will have variable inclinations but will be generally out-of-slope.

Rock quality in the sector is generally good with occasional discrete closely fractured zones (Figures 13 and 14). An extensive zone of poor quality rock was observed just below ditch grade in borehole RKS-10-07 (Figure 14).

Based on the structural analysis, the inference of additional flow boundaries and the natural topographic conditions, it is confirmed that the rock slope template should consist of ¼ H:1V (76°) cuts. Controlled blasting in accordance with WSDOT (2008) should be employed.

Slope stabilization for Design Sector IX should consist of the following:

Prescribed: For the portal area from Station 1663+25 to 1664+25 six rows of untensioned pattern dowels should be installed in accordance with Figures 38 and 39. For the remainder of the sector, two rows of crest dowels should be installed with the uppermost installed at or above top-of-rock PRIOR to first excavation lift (see Figure 40). All dowels should consist of 100-kip (ultimate) bar, 25 feet long, spaced at 12 ½ feet (nominal), and inclined at 15° (nominal). For the entire Design Sector IX this will require 70 dowels with a cumulative length of 1750 feet. Two rows of 50-foot long drains, each with a spacing of 50 feet, are recommended in accordance with Figures 38, 39 and 40 (cumulative length = 800 feet). Rockfall mitigation measures are discussed in Section 8.4.

Provisional: An allowance in the quantities should be provided for the stabilization of three flow boundaries or poor quality rock zones as prescribed by the generic design shown in Figure 41. This could require up to 1200 feet of 120-kip working load rock bolts and 100 CY of shotcrete.

8.3.3 Design Sector X

The kinematic analysis for Design Sector X shown in Figure 43 identifies the potential for a wedge geometry defined by Sets 1 and 3 (32°/258° intersection) and for planar failure along Set 1 (39°/217°). Set 2 will form release surfaces or tension cracks facilitating the wedge and planar failure mechanisms. This structural fabric is similar to Design Sector IX. The persistence of Set 1 averages less than 10 feet and Set 3 between 10 and 20 feet and thus these features will influence face stability (rockfall) but will not control overall slope stability. As discussed in Section 7.3, numerous flow boundaries with associated clay infilling were identified in Design Sectors IX and X (see also Figure 44). These features form trough-like geometries and will have variable inclinations that are generally out-of-slope. These features could be large enough to encompass a majority of the cut slope height.

Rock quality is generally good with low fracture frequencies and high intact strength. Multiple poor quality discrete zones are present (See Figures 15 and 16).

Based on the structural analysis, the inference of additional flow boundaries and the natural topographic conditions, the confirmed rock slope template should consist of ¼ H:1V (76°) cuts. Controlled blasting in accordance with WSDOT (2008) should be employed.

As illustrated in Figure 45, slope stabilization should consist of the following:

Prescribed: Two rows of crest dowels should be provided with the uppermost row located at or above the top-of-rock and installed PRIOR to the first excavation lift (see Figures 42 and 45). All dowels should consist of 100-kip (ultimate) bar, 25 feet long, spaced at 12 ½ feet (nominal), and inclined at 15° (nominal). For the entire Design Sector X this will require 185 dowels with a cumulative length of 4625 feet. Two rows of 50-foot long drains, each with a spacing of 50 feet, are recommended in accordance with Figure 45 (cumulative length = 2300 feet). Rockfall mitigation measures are discussed in Section 8.4.

Provisional: An allowance in the quantities should be provided for the stabilization of six flow boundaries or poor quality rock zones as prescribed by the generic design shown in Figure 41 and the example shown in Figure 42. This could require up to 2400 feet of 120-kip working load rock bolts and 200 CY of shotcrete.

8.3.4 Design Sector XI

Design Sector XI was designated as a separate sector because it does not require cut slopes of any significant dimension. The only issue from a slope perspective relates to the stability of existing outcrops that will be proximal to the I-90 alignment. Due to the highway geometry no exploration drilling was performed. Structural data was obtained from mapping for which the kinematic analysis is shown in Figure 46. The analysis identifies a potential wedge geometry formed by a combination of Sets 1 and 3.

Figure 47 shows the slope stabilization measures recommended for the outcrop in the vicinity of Station 1379+50. These measures include scaling, tensioned rock bolts, untensioned dowels and shotcrete. The prescribed and provisional quantity estimates are shown on the figure.

8.3.5 Design Sector XII

Design Sector XII extends from WB Station 1382+50 to 1396+50; the latter location proximal to the western boundary of the 1957 slope failure at Slide Curve (Wyllie & Norrish, 2009a). To avoid new cuts at Slide Curve, a median wall and embankment have been designed by WSDOT to carry the westbound lanes. The western terminus of this embankment is approximately Station 1883+00 with the grade ascending to the east.

Previous feasibility studies have characterized Design Sector XII as particularly acute in terms of potential cut slope instability (Wyllie & Norrish, 2008a). For this reason, a comprehensive stability assessment was undertaken within the current design study. The issue at hand is whether the proposed cut slopes within the sector will be susceptible to global (overall) slope instability in a manner similar to the 1957 failure. In order to reach an opinion, multiple aspects were considered:

1957 Forensic Studies: Coombs (1958) in reference to the slide area states: "*In the immediate vicinity of the slide and for 75 yards to the north the joints contain a thick clay-like material that greatly reduces friction and makes the steeply cut faces unstable. The rock generally is decomposed and rotten for this distance of 75 yards north of the present slide. The steep face above the present cut is overloaded and in danger of sliding in a manner similar to that of the October 4, 1957, slide.*". He further states "*The avoidance of the unstable rock face to the north is equally as important as the slide area insofar as danger is concerned.*"

The Coombs rock description is inferred to correspond to the poor quality Domain 3 rock identified at Slide Curve and in Sector XII (as discussed in Section 7.2).

Clay Mineralogy: Observation of pumice fragments in core one year after recovery revealed rapid disintegration ascribed to swelling clays (Sections 7.5.6 and 7.6). This characteristic can lead to progressive strength loss when exposed in slope cuts.

Existing Slope Behavior: The existing 50-year old slopes show evidence of crest failures, perhaps as large as 50 cy, and large accumulations of recently-generated talus present at the base of the slopes. During exploration, drilling fluid was often observed seeping or flowing from the slope face. These attributes are consistent with a slope susceptible to rapid degradation relative to the design life of the highway.

Rock Quality in Boreholes: Figures 17 through 21 show the highly variable rock quality encountered in boreholes near the proposed cut line. For much of Design Sector XII, Domain 3 rock will comprise the upper half of the new cut slopes and Domain 2 the lower half. The higher quality Domain 1 rock will only be locally present in the proposed cuts. As a point of comparison, borehole H-110-06 on Figure 21 shows the rock quality contrast between Domain 1 rock and the Domain 2 and 3 rock encountered down slope. To further illustrate the rock quality issue, a collage of features developed from down hole COBL logging and from structural core logging are shown in Figure 48. These features include discrete, closely fractured zones, discontinuities with aperture, faults and mylonitic shears, many of which were encountered in boreholes throughout Design Sector XII.

Structural Geology: Figure 49 shows the kinematic analysis for joint discontinuities for the sector. The dominant potential modes of instability are plane failures defined by Set 1 for southwest dipping slopes, plane failures defined by Set 1a for west-dipping slopes and defined by Set 3 for northwest-dipping slopes. Potential wedge failures are defined by the intersection of Sets 1 and 3. The planar features and the wedge intersection are inclined at angles between 30º and 50º (Figure 49). Set 2 is oriented so as to create release surfaces or tension cracks thereby facilitating wedge or planar mechanisms.

Figure 50 shows the comparable kinematic analysis for faults. A strong concentration of southwest dipping faults (average orientation 49º/227º) is indicated. These persistent, low shear strength features will daylight proposed cut slopes in Design Sector XII.

The conclusion from the above five considerations is that structurally-controlled planar failures are kinematically viable and highly probable for Design Sector XII. The next step was to determine the degree of stability for this potential failure mode by constructing a geotechnical model for a representative cross section. In this case the section at Station 1385+25 WB was selected because of the information provided by a pair of boreholes (Figure 51). Features of the model include the existing slope topography, rock quality distribution (Domains 1 to 3) interpreted from boreholes RKS-13-07 and RKS-40-08 and an inferred phreatic surface. Rock mass shear strengths were assigned for Domains 1 through 3 using the approach described in Section 8.1.2. In the case of Domains 1 and 2 the rock mass shear strength was simplified to a linear envelope with the **φ** and **c** values indicated. In both cases the rock mass strength precludes failure at the low shear stresses failure developed for the comparatively low cut slopes. For Domain 2, the kinematic analysis described above demonstrated a concentrated population of southwest-dipping joints and faults with inclinations between 30° and 50°. This characteristic was incorporated in the model by assigning an anisotropic shear strength to the Domain 2 rock such that those portions of potential failure surfaces within this 30° to 50° dip range would exhibit a “joint” shear strength. The friction angle for these joint surfaces equaled that determined by laboratory testing (Section 8.1.1), namely **φ** =38°. A back analysis was performed to estimate a reasonable cohesion value. As shown in Figure 51 the critical non-circular surfaces in the back analysis reported FS values of 1.15 and 1.02 under static and pseudo static conditions respectively, assuming a cohesion value of 800 psf for the joint surfaces in Domain 2. A seismic coefficient of 0.07g was assumed for the 2001 Nisqually earthquake as presented in Wyllie & Norrish, 2007a. The derived cohesion value represents the minimum value that must be acting to provide the observed slope stability and therefore is a conservative value to use for forward analyses.

The forward analysis shown in Figure 52 indicates that the proposed ¼ H:1V (76°) cut slope will have a FS value of about unity in the absence of reinforcement under static conditions and a value much less than unity at a pseudo static coefficient of 0.175g (project design parameter). Figure 53 shows the required reinforcement pattern to increase the margin of stability to WSDOT minimum criteria. It consists six rows of passive dowels with an equivalent design load of 225 kips arranged on a 12-foot vertical by 15-foot horizontal pattern. To provide sacrificial steel to offset potential corrosion, #20 Grade 75, 491 kip ultimate bars should be used.

To maintain rock mass strength and joint cohesion behind the cut slope face, controlled blasting should be employed in accordance with WSDOT (2008).

As illustrated in Figures 54, slope stabilization should consist of the following:

Prescribed: Pattern dowels should be located as generally shown. Variation to suit lift locations is allowable provided the uppermost row is installed at top-of-rock PRIOR to the first excavation lift. The vertical spacing of 12 feet is intended to match each half lift for blasting. One row will be installed at the base of each lift and the subsequent row from a muck pile (or crane) at the next half lift following drilling and shooting. Crest dowels should be installed between the two pattern-doweled areas. All dowels should consist of 491-kip (ultimate) bar, variable lengths from 30 to 60 feet, and be inclined at 15° (nominal). For estimating purposes the conceptual pattern layout will require a total of 251 dowels with a cumulative dowel footage of 13,550 feet. To prevent ongoing degradation of the upper weathered rock portions of the cut slope, the face should be covered with a minimum 3-inch thick layer of fiber-reinforced shotcrete extending from the top-of-cut down to nominal elevation 2590 feet. The estimated shotcrete volume is 350 cy. Two rows of 50-foot long drains spaced at 50 feet are recommended (cumulative length = 2550 feet).

Provisional: To provide for spot reinforcement in the vicinity of 1389+00 WB between the pattern-doweled areas, 1000 feet of low capacity dowels and 1000 feet of high capacity rock bolts should be allotted for variable rock conditions. An additional 50 cy of shotcrete should be included for specific poor quality zones encountered during construction. Careful monitoring of the structural geology encountered during excavation in accordance with Appendix I is recommended to identify face areas requiring provisional treatment.

8.3.6 Design Sector XVII

The kinematic analysis for Design Sector XVII shown in Figure 55 identifies the potential for a wedge geometry defined by Sets 1 and 2 (43°/327° intersection) and for planar failure along Set 1 (62°/267°). Due to a lack of surface exposure the persistence of Set 1 joints is not known. As discussed in Section 7.3, multiple rock types and thick glacial overburden were encountered in the Sector XVII boreholes.

Rock quality is generally good with low fracture frequencies and low to moderate intact strength.

Based on the structural analysis, the natural topographic conditions and the proposed low cut heights, the recommended slope template consists of 1.5H:1V overburden cuts and ½ H:1V (63°) rock cuts. Controlled blasting in accordance with WSDOT (2008) should be employed. The reason for the flatter rock cuts is to align the face parallel to the Set 1 joints. Due to the gentle topography above the cuts, flatter rock slope inclinations could also be considered.

The thick and variable overburden will complicate the establishment of the top-of-cut staking line. This can be refined during construction when the pioneer road is built thereby offering the opportunity for probe holes or other explorations. Alternatively, the cut slope could be staked at 1.5H:1V as if the entire cut will be in overburden and when the cut is brought down to top-of-rock a step-out bench left to the rock cut line. The latter procedure results in a degree of over excavation.

Prescribed: No prescribed stabilization is recommended for the low cut heights.

Provisional: To provide for local spot reinforcement during construction, 500 feet of bar is recommended and to be applied as low capacity (100 kip ultimate) dowels.

8.3.7 Design Sector XVIII

The kinematic analysis for Design Sector XVIII shown in Figure 56 identifies the potential for a wedge geometry defined by Sets 1 and 2 (42°/305° intersection) and for planar failure along Set 1 (55°/255°). As discussed in Section 7.3, multiple rock types and thick glacial overburden were encountered in the Sector XVIII boreholes.

Rock quality is generally good with low fracture frequencies and moderate to high intact strength.

Based on the structural analysis, the natural topographic conditions and the proposed low cut heights, the recommended slope template consists of 1.5H:1V overburden cuts and ½ H:1V (63°) rock cuts. Controlled blasting in accordance with WSDOT (2008) should be employed. The reason for the flatter rock cuts is to align the face parallel to the Set 1 joints. Due to the gentle topography above the cuts, flatter rock slope inclinations could be also considered.

The locally thick and variable overburden will complicate the establishment of the top-of-cut staking line. This can be refined during construction when the pioneer road is built thereby offering the opportunity for probe holes or other explorations. Alternatively, the cut slope could be staked at 1.5H:1V as if the entire cut will be in overburden and when the cut is brought down to top-of-rock a step-out bench left to the rock cut line. The latter procedure results in a degree of over excavation.

Prescribed: No prescribed stabilization is recommended for the low cut heights.

Provisional: To provide for local spot reinforcement during construction, 500 feet of bar is recommended and to be applied as low capacity (100 kip ultimate) dowels.

8.4 ROCKFALL CONSIDERATIONS FOR DESIGN SECTOR IX AND X

The following approach to rockfall is adapted from the Phase 1B report (Wyllie & Norrish, 2009b.)

As presented in Sections 8.3.2 and 8.3.3, Design Sectors IX and X will require rock cuts greater than 100 feet high, reaching a maximum of about 140 feet (see also Figure 57). To control rockfall beneath slopes of these heights, Design Manual M22-01 (WSDOT, 2006) requires a catchment area width of 29 feet. However, the guidance document also makes provision that site specific rockfall modeling can take precedence over the prescriptive generic designs therein. For the Design Sector IX and X cuts, the site specific design approach was adopted because the slope heights significantly exceed customary slope heights addressed in the manual and because the experience base for the use of catchment width for rockfall control at such slope heights is limited.

Computer programs that simulate rockfalls are inherently sensitive to the input parameters used by the algorithms to calculate the trajectories. These parameters include coefficients of normal and tangential restitution, friction angle, slope height and angle, slope uniformity and roughness, rock fragment size and shape, rockfall source area, and others. Consequently, when using such programs it is highly advisable to verify the predicted results against measured results at similar sites. The Rocscience program Rockfall, Version 4.048 was used as the simulation software. Parameters were assigned in accordance with the manufacturer's recommendations, and modified to provide results that reasonably matched comparative data.

Figure 58 shows the predicted trajectories for three slope height cases: a) 144 feet b) 96 feet and c) 72 feet. The slope profile for the computer model was matched to the WSDOT design template with the exception that the offsets between the 24-foot lifts were inclined at 1H:1V to more closely match blasting and excavation effects. The reliability of the model is based on two comparisons:

- ODOT (2001) reports a 99 percent probability that the first impact location beneath an 80-foot high ¼ H:1V slope will be 22 feet or less. The model shows all rocks impacting within 20 feet for a 72-foot high slope (Figure 58C), which is reasonably consistent given the slope height difference.
- Norrish and Findley (2005) performed a rock rolling test at the Rock Island Cut in volcanic rock. The tests reported the following percentages of rocks airborne at specific measurement distances from the toe of the slope: 20 percent at 25 feet, 8 percent at 30 feet and 0 percent at 35 feet. The model (Figure 58B) slightly under estimates these measurements; however the measurement point for the latter was on a horizontal bench approximately 6 feet lower than the model. When corrected for this elevation difference, the respective results were 21 percent at 25 feet, 1 percent at 30 feet and 0 percent at 35 feet.

Having reasonably verified the model, Figure 58A shows the trajectory predictions for a 140-foot high cut slope as proposed for Design Sectors IX and X. It is noted that approximately 50 percent of the rockfalls are airborne at a point 30 feet from the toe and a few percent are still airborne at 35 feet from the toe. It is our opinion that these values are somewhat low (un-conservative) because experience has shown that in reality a greater percentage of the falling rocks will strike the lower lift offsets and be directed horizontally across the catchment area. Based on this experience, our understanding that shoulder barriers will not be used, and the simulation analyses, it is recommended that the upper portions of rock cuts for Design Sectors IX and X be draped with cable net or equivalent down to an elevation corresponding to 72 feet above the slope toe. The effect of this design modification on the predicted catchment performance is shown in Figure 59. Figures 39 and 42 shows the required drape areas and quantity estimates for these Design Sectors IX (Portal area), IX, and X, respectively. These are conceptual designs subject to refinement by WSDOT during the preparation of PS&E.

8.5 OVERBURDEN TEMPLATES

Overburden deposits vary in thickness and origin throughout the various design sectors of the Phase IC project. Data on overburden thickness was collected from field observations of the existing rock cuts, field surveys of bedrock elevation and borehole results. Based on the available data and the geologic understanding of the project site, suggested overburden thickness values are provided in Table 11 for input to WSDOT highway design models thereby facilitating quantity takeoffs. Typically the suggested design value is at or slightly thicker than the observed condition. The exception to this is for intervals where no bedrock was observed and local geologic observations suggest the presence of bedrock.

Recent glacial activity and localized geologic conditions result in bedrock surfaces that undulate over short distances. Therefore, variability in overburden thickness should be expected. This could include narrow (<10 feet) swales in the bedrock that are exposed during construction. Thickness values provided should be verified during construction activities prior to final top-of-cut staking, especially through areas where significant changes in bedrock elevation have been recorded, for example between Design Sectors XVII and XVIII.

Loose colluvial deposits are most common for Design Sectors VIII (Snowshed) through XII. In addition to the colluvium, loose snow avalanche-transported debris was observed in Sector VIII and anthropogenic (man-made) fill was observed at the east end of Sector XII. These deposits are typically a silty sand or sandy silt with gravels (SM/ML). Occasional (<10%) angular boulders are present, typically between 1 and 2 feet in diameter. Alpine till was observed throughout Sectors XVII and XVIII as well as the western portion of Sector VIII. The alpine till is typically medium dense grading to dense with depth and comprised of silty sands with gravels (ML) and occasional (<10%) boulder sized clasts, ranging from 1 to 3 feet in diameter.

Overburden cut slopes should follow a hierarchal design approach in order of decreasing preference is as follows:

1. 1.5H:1V with a 5-foot offset at top-of-rock

2. 1.5H:1V with no offset

3. 1.3H:1V with no offset to a maximum OB thickness of 20 feet

4. 1H:1V with no offset and OB thickness less than 10 feet

5. Site specific design with or without retention.

In all cases slope rounding at the overburden crest should be implemented in accordance with WSDOT (2008) M41-10 Section 2-03.3 (5).

Table 11 – Overburden Template Design

Design Sector	WB Stationing		Overburden Thickness (feet)		Recommended Overburden Design Thickness (feet)	Type of Deposit	Relative Density	Notes
	Start	End	Existing Slope	Boreholes				
Snowshed	1352+50	1355+50	40 - 60	None	Variable (See Notes)	Colluvium, avalanche deposits, alpine till	Loose; Med. Dense to Dense	Overburden thickness decreases up station; alpine till deposits likely overlie bedrock and are med. dense to dense; proposed cuts limited by snowshed construction; thickness should be confirmed during construction
	1355+50	1363+50	5 - 10	9 - 10	10	Colluvium, avalanche deposits	Loose	Variable thickness along interval based on rock outcrops and snow avalanche chutes; proposed cuts limited by snowshed construction
IX	1363+50	1366+00	2 - 5	3 - 4	5	Colluvium	Loose	Possible increase in thickness near north end of sector
X	1366+00	1373+00	2 - 5	0 - 7	8	Colluvium	Loose	Variable thickness and several locations with bedrock outcrops along proposed cut line
	1373+00	1375+50	10 - 25	7	See Notes	Colluvium	Loose to Med. Dense	Increasing thickness up station as proposed cut slope decreases; possible alpine till in deposit near base (medium dense to dense)
XI	1375+50	1382+50	2 - 5	No Data	5	Colluvium	Loose	Only 300 feet of rock outcrop in sector, no proposed cut slopes
XII	1382+50	1395+00	3 - 10	3 - 7	10	Colluvium	Loose	Near surface bedrock is typically moderately to highly weathered; may cause localized increase in depth of overburden excavation
	1395+00	1396+50	10 - 15	11	15	Colluvium, man-made fill	Loose	Colluvium underlying fill prism from re-grading activities associated with historic Slide Curve excavation activities
XVII	1442+50	1459+00	10 - 25	12 - 20	15 - 25	Alpine till	Med. Dense to Dense	Increasing overburden thickness moving up section; interval ends on south side of stream drainage; boulders up to 3 feet in diameter; thickness should be confirmed during construction
	1459+00	1468+00	25 - 35	27 - 28	30	Alpine till	Med. Dense to Dense	Decreasing overburden thickness at southern end of interval with Sector XVIII; gradational change in overburden elevation between RKS-45-08 (~2560 feet MSL) and RKS-46-08 (~2615 feet MSL); boulders up to 3 feet in diameter; thickness should be confirmed during construction
XVIII	1468+00	1470+00	5 - 20	3	5 - 15	Alpine till	Med. Dense to Dense	Overburden thickness decreases from northern end of interval to approximately 1371+00 where it is only 3 feet thick; thickness on north end should be confirmed during construction
	1470+00	1473+25	3 - 7	3	8	Alpine till	Med. Dense to Dense	Slight increase in overburden thickness as bedrock elevation decreases to the south
	1473+25	1474+50	>25 feet	No Data	30	Alpine till	Med. Dense to Dense	Assume 30 foot thickness to allow for bedrock present back into slope; thickness should be confirmed during construction
	1474+50	1480+00	10 - 15	13	15	Alpine till	Med. Dense to Dense	Rock outcrops (and proposed rock cuts) end at 1477+25 with thin soil cuts proposed to 1480+00

9.0 DESIGN RECOMMENDATIONS

9.1 INTRODUCTION

The general approach to slope stabilization was to minimize the types of reinforcement materials. For example for the reinforcement, three bar sizes are recommended and will be common to both tensioned rock bolts and untensioned dowels. Further, the design lengths have been specified such that cut lengths derived from mill stock will be usable on the project, either singly or coupled, to minimize wastage. This approach provides maximum flexibility for the contractor and will be the most economical for WSDOT.

9.2 MATERIAL SPECIFICATIONS

9.2.1 Rock Dowels

To the maximum extent feasible, rock dowels have been specified in preference to tensioned rock bolts or cable anchors. The reasons for this approach include technical adequacy as well as construction expediency. The proposed cut slopes for Phase 1C and particularly for Design Sectors IX, X and XII, will require extensive reinforcement. This will disrupt the normal productivity cycle of rock excavation, namely; drill, shoot, muck, and haul. By selecting dowels instead of tensioned members, the installation time will be minimized thereby reducing the impact on the rock excavation cycle. This will ultimately provide cost savings to WSDOT.

The dowel designs are based on the use of Grade 75 ductile steel. Do not substitute higher strength, stiffer steel. Grade 60 steel could be considered if proposed by the contractor.

High Capacity Rock Dowels (Designated **"Dowel H")**

#20 Nominal 2.5" diameter Grade 75 bar, continuously threaded

Minimum ultimate tensile strength:	491 kips
Minimum yield tensile strength:	368 kips
Minimum hole diameter:	5 inches
Grout:	Per WSDOT Special Provisions for rock bolts.
Corrosion Protection:	Not required
Centralizers:	10-foot spacing, minimum 3 per bar
Bearing Plates:	10 in x 10 in x 1 in
Lengths:	Deliver to the site in mill lengths of 50 feet, cut to suit design
Design Load:	Torque nut to 1000 ft-lb nominal
Testing:	Per WSDOT Special Provisions

Low Capacity Rock Dowels (Designated **"Dowel L")**

Minimum ultimate tensile strength:	100 kips (Typically #9, Grade 75 steel bar)
Minimum Yield Strength:	75 kips
Minimum Hole Diameter:	2 to 2.5 times bar diameter
Grout:	Per WSDOT Special Provisions for rock bolts.
Corrosion Protection:	Epoxy coated
Centralizers:	10-foot spacing, minimum 3 per bar
Bearing Plates:	One half of quantity at 8 in x 8 in x ¾ in One half of quantity at 10 in x 10 in x ¾ in
Lengths:	Deliver to the site in mill lengths of 50 feet, cut to suit design
Design Load:	Torque nut to 500 ft-lb nominal
Testing:	Per WSDOT Special Provisions

9.2.2 Rock Bolts

Rock bolts are tensioned steel members that are grouted in drill holes and intended to place the rock mass between the distal or anchor zone and the collar into compression. Typically this is done across a plane of weakness (e.g. joint or fault) to improve its shear strength. For the Phase 1C project sites, the following recommendations are made with respect to rock bolts:

High Capacity Rock Bolts (Designated **"Bolt H")**

#14 Nominal 2.25" diameter Grade 75 bar, continuously threaded

Minimum ultimate tensile strength:	225 kips
Minimum yield tensile strength:	169 kips
Minimum hole diameter:	4 ½ inches
Grout:	Per WSDOT Special Provisions for rock bolts.
Corrosion Protection:	Epoxy-coated
Centralizers:	10-foot spacing, minimum 2 per bar in bond zone
Bearing Plates:	10 in x 10 in x 1 in
Lengths:	Deliver to the site in mill lengths of 50 feet, cut to suit design
Design Load:	120 kips
Testing:	Per WSDOT Special Provisions

Low Capacity Rock Bolts (Designated **"Bolt L")**

Minimum ultimate tensile strength:	100 kips (Typically #9, Grade 75 steel bar)
Minimum Yield Strength:	75 kips
Minimum Hole Diameter:	2 to 2.5 times bar diameter
Grout:	Per WSDOT Special Provisions for rock bolts.
Corrosion Protection:	Epoxy coated

Bond Zone:	Minimum 5 feet
Centralizers:	10-foot spacing, minimum 3 per bar
Bearing Plates:	One half of quantity at 8 in x 8 in x ¾ in One half of quantity at 10 in x 10 in x ¾ in
Lengths:	Deliver to the site in mill lengths of 50 feet, cut to suit design
Design Load:	50 kips
Testing:	Per WSDOT Special Provisions

9.2.3 Drain Holes

Drain holes will be drilled in the lower slope to prevent buildup of groundwater pressure. Lengths of individual drains may vary, but will typically be 50 feet. The holes should be 3-inch diameter and completed with 2-inch diameter machine-slotted PVC pipe. Centralizers for the pipe should not be used. The outermost five feet of each drain should be solid with the collars grouted in place to avoid pullouts and vandalism.

The locations shown in the plans for the lower row above the ditch line have some flexibility as to collar location. Depending on water conditions encountered during excavation, the site engineer may require additional drain holes on the slope face.

Materials for drains should be in accordance with WSDOT Special Provisions.

9.2.4 Slope Drape

Complementing the ditch catchment, slope drape will be the secondary rockfall control method for the upper portions of the new cut slopes in Design Sectors IX and X. The target minimum lower limit for the slope drape is 72 feet above the ditch line.

Due to the strong blocky character of the rock in these Design Sectors, the slope drape should be capable of arresting blocks up to four feet in size. Suitable drape designs include cable net, ring net or TECCO® mesh. The primary net should be overlain with chainlink or double-twisted hexagonal mesh to retain small rocks that would pass through the primary net grid.

WSDOT has developed Special Provisions that are appropriate for this item. The quantity estimates shown on Figures 39, 40 and 44 are approximations based on surface area. Typically for steep slopes blasted with controlled blasting, slope roughness will add about 10 to percent to such surface area calculations for the slope drape quantity. It is recommended that a preliminary design be developed by WSDOT during the PS&E preparation. This preliminary design should include guidance on mesh options, anchor capacity and spacing, anchor locations, seaming and aesthetic constraints. Based on architectural guidelines the mesh will be coated to color match the rock face. It is anticipated that a single slope drape design will be appropriate for all the required rock cut locations in Phase 1C.

After the cut is brought down to grade for the locations specified, the contractor should verify the preliminary design by performing an accurate assessment using panel and wire rope layouts and an allowance for slope surface roughness before the materials are procured. Favorable conditions for anchors should be sought along the slope crest in Design Sectors IX and X (moderately strong, fresh to slightly weathered, shallow bedrock).

9.2.5 Shotcrete

The purpose of the shotcrete is to isolate degradable rock zones from ongoing weathering. The use of shotcrete is recommended as a prescribed stabilization methodology for Design Sector XI and XII and elsewhere as a provisional or contingency item for the rock cuts in Design Sectors IX and X. Actual rock conditions will dictate whether shotcrete is required for the latter two sectors.

Steel fiber reinforced shotcrete is recommended in accordance with WSDOT Special Provisions. Shotcrete thickness is expected to vary but should be typically about four inches. Anchor bars (“L-bars”) should be incorporated into the shotcrete to enhance surface adhesion. Requirements for shotcrete color can be added to meet architectural guidelines at the discretion of the Region and will involve either dye or stain.

9.3 SCALING REQUIREMENTS

Scaling of new cuts that are excavated using controlled blasting is an incidental item to the rock excavation per WSDOT Standard Specifications (2008). This will apply to the new cut slopes throughout Phase 1C.

Scaling of the bedrock outcrops above the alignment in Design Sectors XI will be outside the slope stakes and will therefore qualify for payment under force account (Section 2-03.3(2) of 2008 Standard Specifications). Accordingly, WSDOT will control the effort required to achieve adequate mitigation of the rockfall hazard. The stabilization requirement for Design Sector XI includes duration estimates for scaling assuming a 3-man crew.

The intent of the scaling is to remove marginally-stable, large rock blocks that could compromise the safety of the highway or of other design elements. However, worker safety for construction activity lower on the slope face is the over-riding issue with regard to the extent of scaling. Intensive hand scaling is recommended to meet this objective and should include conventional scaling bars, air pillows, hydraulic jacks and trim blasting. Scaling includes removal of trees from the slope face and to a point approximately 15 feet behind the final slope crest. Tree butts and root balls can be left in place.

9.4 PRELIMINARY SCHEDULE OF QUANTITIES

Table 12 provides estimated quantities based on the project elements described in Section 8. The schedule is developed by Design Sector and by type of stabilization measure. The quantities are limited to slope stabilization items only and preclude other activities normally associated with rock excavation (drilling, blasting, loading and hauling). The stabilization items are designated as either primary or secondary. The former applies to priority items required to provide for overall slope stability or rockfall control. The latter refers to less important items generally associated with localized stability of blocks on or adjacent to cut slopes.

The quantity estimate does not include site geotechnical engineering, instrumentation and construction monitoring. It is anticipated that quantities will be refined during preparation of PS&E.

Table 12
Preliminary Schedule of Quantities

	Primary Stabilization Measures						**Secondary Stabilization Measures**			
	Dowel H Quantity (ft)	Dowel L Quantity (ft)	Bolt H Quantity (ft)	Bolt L Quantity (ft)	Scaling Quantity (crew hr)	Drains Quantity (ft)	Drape Quantity (sf)	Fence Quantity (ft)	Shotcrete Quantity (cy)	Excavation Quantity (cy)
Design Sector XIII (snowshed) **1352+50 to 1363+50**										
Design Sector IX **1363+50 to 1366+00**		1750	1200			800	22500		100	
Design Sector X **1366+00 to 1375+50**		4625	2400			2300	77500		200	
Design Sector XI **1375+50 to 1382+50**		80		200	50				5	
Design Sector XII **1382+50 to 1396+50**	13550	1000		1000		2550			400	
Design Sector XVII **1443+00 to 1466+25**		500								
Design Sector XVIII **1466+25 to 1473+00**		500								
Totals by Item	13,550	8,455	3,600	1,200	50	5,650	100,000	0	705	0

Incidental Items:			
Design Sector		Quantity	Unit
All	Grout loss allowance	500	cy

Type	Ultimate Capacity	Min Yield	Working Load	
Dowel H	491	368	Variable	kips
Dowel L	100	75	50	kips
Bolt H	225	169	120	kips
Bolt L	100	75	50	kips

9.5 DEVELOPMENT OF PS&E DOCUMENTS

PS&E should be prepared in accordance with WSDOT Standard Specifications (2008), modified to include special provisions typically used by WSDOT for slope stabilization work. It is strongly recommended that Wyllie & Norrish participate with WSDOT in the preparation of PS&E to ensure that the geotechnical recommendations herein are accurately captured in the bidding documents.

10.0 CONSTRUCTION ISSUES

10.1 PHILOSOPHY

Rock slope excavation will require a coordinated set of activities between the contractor and WSDOT to provide for stabilization as the cuts are incrementally brought down. These cyclic activities can be generalized as drill, shoot, excavate, evaluate, stabilize, monitor and redesign (if necessary). Consequently, WSDOT should anticipate a slower rate of rock excavation than is customary, coupled with higher unit rates for rock excavation than is customary.

10.2 OVERBURDEN THICKNESS

Variable thicknesses of overburden will be encountered throughout the Phase 1C alignment. This will make staking of the top-of-cut for compound slopes (soil over rock) difficult to achieve. Two options are available:

- Assume conservative overburden thicknesses and stake accordingly. If rock is encountered at higher elevation than anticipated leave a variable width bench on the top of rock. This approach minimizes the possibility of having to re-access the top-of-cut but does result in extra excavation volume.
- Use the pioneer road along the top-of-cut as an exploration opportunity, supplemented with test pits or probe holes, and adjust the stake line accordingly. This approach requires greater field engineering for optimization but minimizes excavation volume.

Given that the project has an excavation surplus, the latter approach is probably preferable.

On a related topic, the overburden templates in this report often show a nominal bench on the top of rock. If the second option above is adopted, this bench can be minimized or eliminated and slope rounding implemented in accordance with WSDOT (2008).

10.3 ACCESS

Overburden soils should facilitate the construction of a pioneer road to provide access for the contractor to initiate the rock cuts. Local areas of shallow bedrock may require ripping or blasting to achieve a reasonable width.

10.4 BLASTING

Blasting will be required for the cut slopes of Design Sectors XIII, IX, X, XII, XVII and XVIII XVI and should be in accordance with WSDOT Standard Specifications Section 2-03.3(2) (WSDOT, 2008). In addition, WSDOT should include a special provision stipulating that a blasting consultant be required to develop the blasting plans on behalf of the contractor. During construction, blasting plans should be confirmed through the performance of limited-scale test blasts. Lift heights must be limited to 24 feet to be compatible with the installation of dowel reinforcement at 12-foot vertical intervals.

10.5 GROUT LOSS

The experience with migration of drilling fluid to the existing cut slopes during the exploration drilling underscores the potential for significant grout loss associated with the installation of dowels and rock bolts. It is recommended that the bid item for both dowels and bolts is by the foot to include a grout volume up to 200 percent of the open-hole volume calculated from the hole diameter and length. For grout usage in each hole above this calculated volume, the Contractor should be reimbursed <u>at cost</u> for material and labor but WSDOT should retain the decision as to what volume of grout will be pumped over-and-above the 200% threshold. This approach is consistent with WSDOT Special Provisions for Rock Bolts. The grout loss issue has the highest potential in Design Sector XII.

10.6 SEQUENCING OF ROCK EXCAVATION AND STABILIZATION SUPPORT

For new rock cuts it is essential that reinforcement is installed on each lift before the next lift is blasted or excavated. The recommended lift height is 24 feet and the stipulated vertical reinforcement interval is 12 feet. This means that after a lift is blasted the muck should be partially retained to create a temporary work berm for the intermediate row of dowels. The muck can be removed 48 hours after the cement grout in the boreholes has cured. Blasting should not be permitted within 50 feet of an installed dowel or rock bolt until the grout has achieved 60 percent of the 7-day strength, equivalent to 2400 psi.

Although it may be stating the obvious, all new or existing rock slopes should be stabilized from the top down. Construction expediencies often interfere with this truism and such digressions must not be allowed to occur.

10.7 GEOTECHNICAL MONITORING PLAN

An inherent attribute of complex rock slope stabilization projects is that specific conditions encountered during construction may differ from those observed during the site investigation. This is the result of geologic variability as well as the unpredictable success of scaling and rock excavation activities. The mitigation designs herein include a reasonable percentage of contingency items that can be deployed to overcome such unanticipated conditions. To assist the

Region with these construction design decisions, it is recommended that experienced professional rock slope engineering personnel be periodically and regularly involved with the work.

Wyllie & Norrish (2008b) presented the recommendations of Mr. Erik Mikkelsen, P.E. concerning geotechnical performance monitoring during construction. In accordance with those recommendations and with the needs of the project, the following instrumentation plan is recommended:

Design Sector VIII:
Instrumentation not required, rely on visual observation.

Design Sectors IX and X:
Install a minimum of six equally spaced prisms through the two sectors, suitably protected from fly-rock, on the cut slope face near elevation 2675 feet. Monitor using conventional surveying on a weekly basis while active project work is underway.

Extend five dowels of the upper row to a depth of 75 feet and instrument these dowels with vibrating wire or weldable resistance gages. The dowels should correspond to the to the highest cut slope locations as shown in Figure 42. Integrate with WSDOT's automatic data acquisition system (ADAS).

Monitor TDR cable installed in borehole RKS-38-08 at Station 1366+98, 84L and in RKS-18-07 at Station 1372+12, 170L. Integrate with WSDOT's automatic data acquisition system.

Design Sector XI:
Instrumentation not required, rely on visual observation.

Design Sectors XII:
Install a minimum of eight equally spaced prisms, suitably protected from fly-rock, on the cut slope face above elevation 2625 feet or near the top-of-rock if the cut does not reach that elevation. Monitor using conventional surveying on a weekly basis while active project work is underway.

Extend five dowels of the upper row to a depth of 75 feet and instrument these dowels with vibrating wire or weldable resistance gages. The dowels should correspond to the to the highest cut slope locations as shown in Figures 54. Integrate with WSDOT's automatic data acquisition system (ADAS).

Monitor TDR cable installed in boreholes H-110-06 (1395+84, 204L), RKS-40-08 (1385+23, 128L) and RKS-41-08 (1386+73, 96L) using the ADAS.

Design Sectors XVII and XVIII:
Instrumentation not required, rely on visual observation.

11.0 REFERENCES

Badger, T.C., 2002, Fracturing Within Anticlines and its Kinematic Control on Slope Stability: Engineering & Environmental Geoscience, Vol. VII, No. 1, February

Brown, E. T. (ed.), 1981, Rock Characterization Testing & Monitoring, ISRM Suggested Methods: International Society of Rock Mechanics, Pergamon Press, Oxford, 211p.

Burk Geoconsult, 2008, Interim Technical Memorandum-Materials Resource Evaluation, 2007 Geotechnical Analysis and Reporting, I-90 Snoqualmie Pass East; prepared for Washington State Department of Transportation, Yakima, Washington dated May, 2008; 72pp.

Chadbourne, D.G. and L.J. Moses, 1994, Geotechnical Reconnaissance Report Hyak to Ellensburg – SR 90 M.P. 55.17 to M.P. 110.00: Washington State Department of Transportation, February 23, 52p. + maps + discontinuity data

Compton, R.R., 1972, Geology in the Field: John Wiley & Sons, New York, 398p

Coombs, H 1957, Letter to R. H. Pyle, District Engineer, Washington State Department of Highways dated October 29, 1957: University of Washington, Seattle, Department of Geology, 2p.

Coombs, H 1958, Letter to R. H. Pyle, District Engineer, Washington State Department of Highways dated May 19, 1958: University of Washington, Seattle, Department of Geology, 2p.

Ferrero, A.M., 1995. The Shear Strength of Reinforced Rock Joints. International Journal of Rock Mechanics and Mining Sciences, Vol 32, No 6, pp. 595-605.

FHWA, 1998. Rock Slopes Reference Manual. Report No. FHWA-HA-99-007. National Highway Institute U.S. Department of Transportation Federal Highway Administration, Washington, D.C.

Findley, D.P. and N.I. Norrish, 2005a, Memorandum on I-90 Keechelus Lake Slide Curve Rock Slope Stability Evaluation and Design Parameters for Conceptual Alternative 4 Rock Cuts: Letter to Steve Lowell, WSDOT from Golder Associates Inc and Wyllie &

Norrish Rock Engineers, September 29, 24pp.

Findley, D.P. and N.I. Norrish, 2005b, Summary Geotechnical Report Alternatives 1, 2, 3 and 4, I-90 Keechelus Lake (MP 57.5 to MP 59.4) Kittitas County, Washington: prepared for Washington State Department of Transportation, October 31 by Golder Associates Inc., 20pp. + Figures+ Appendices.

Fisher R. V. and Schmincke H. U., 1984. Pyroclastic Rocks. Springer-Verlag Berlin Heidelberg, New York, 472 pp.

Fiske, R.S., 1963, Subaqueous pyroclastic flows in the Ohanapecosh Formation, Washington: Geological Society of America Bulletin, v. 74, no. 4, p. 391-406

Frizzell, V.A., Jr., R.W. Tabor, D.B. Booth, K.M. Ort, and R.B. Waitt, Jr., 1984, Preliminary geologic map of the Snoqualmie Pass 1:100,000 quadrangle Washington, U.S. Geological Survey Open File Report 84-693, 42p.

Hallet, B., 2006, Why Do Freezing Rocks Break? Science 17 November 2006: Vol. 314. no. 5802, pp. 1092 – 1093

Hammond, P.E., 1963, Structure and stratigraphy of the Keechelus Volcanic Group and associated Tertiary rocks in the west-central Cascades Range: Seattle, University of Washington, Ph.D. dissertation, 264pp.

Haneberg, W. C., N. I. Norrish, D. P. Findley, 2006, Digital Outcrop Characterization for 3-D Structural Mapping and Rock Slope Design Along Interstate 90 Near Snoqualmie Pass, Washington: Proceedings, 57th Annual Highway Geology Symposium, Breckenridge, CO, September 27-29, 2006.

Hoek, E. and Brown, E.T.,1981. *Underground Excavations in Rock*, Institution of Mining and Metallurgy, London, 527pp.

Hoek, E., 1983, Strength of Jointed Rock Masses: Geotechnique, v.33, No.3, pp.187-223.

Hoek, E., Carranza-Torres, C., and Corkum, B. (2002). Hoek-Brown Failure Criterion – 2002 Edition. Proceedings 5th North American Rock Mechanics Symposium, Toronto, July 2002, 267-273.

ISRM Suggested Methods. Suggested Method for Determining Point Load Strength, 1985, 8 pages.

Mesri, G., and Shahien, A. M. (2003). Residual shear strength mobilized in first-time slope failures. Journal of Geotechnical and Geoenvironmental Engineering. Vol. 129 No. 1 pp. 12-31

Norrish, N.I. and Findley, D.P. (2005). Rockfall Modeling and Animation, Rock Island Cut, WA, Presentation to 31st Northwest Geotechnical Workshop, Jackson, WY, September 1, 2005.

Oregon Department of Transportation (2001). Rockfall Catchment Area Design Guide, ODOT Research Group Report SPR-3 (032), Salem, OR, 77 pp with appendices.

PTI, 1996, Recommendations for Prestressed Rock and Soil Anchors, 3rd Ed. Post Tensioning Institute, Phoenix, Arizona.

Ritchie, A.M., 1957, Geological Report on Keechelus Lake Rockslide Snoqualmie Pass Highway: Washington State Highway Commission, Department of Highways, December, 7pp.

Ritchie, A.M., 1960, Letter to Richard H. Mackey, Claims Representative, United Pacific Insurance Company dated February 9, 1960: Washington State Highway Commission, Department of Highways

Rocscience, 2000, SWEDGE Version 4.0. Rocscience Inc., Toronto, Ontario.

Sheridan, M.F., 1979, Emplacement of Pyroclastic Flows: A Review: Geological Society of America Special Paper 180, pp.125-136

Spang, K. and Egger, P. (1990) Action of fully-grouted bolts in jointed rock for fractured ground. Journal Rock Mechanics and Rock Engineering, 23, 201-229.

Tabor, R.W., V.A. Frizzell, Jr., D.B. Booth, and R.B. Waitt, 2000, Geologic Map of the Snoqualmie Pass 30x60 minute Quadrangle, Washington: US Geological Survey Geologic Investigations Series I-2538.

URS, 2007. 2006 Conceptual Geotechnical Report I-90 Snoqualmie Pass East Hyak to Keechelus Dam Washington, Volume 1 of 5 – *Geotechnical Narrative*, February 12, 2007.

Walder, J. S., B. Hallet, 1985, A Theoretical Model of the Fracture of Rock during Freezing: *Geol. Soc. Am. Bull.* 96, 336

WSDOT, 2006. Design Manual M22-01. November, 2006.

WSDOT, 2005. Geotechnical Design Manual M46-03. September, 2005.

Washington State Department of Transportation (WSDOT), 2008, Standard Specifications for Road, Bridge, and Municipal Construction, 1021 pp.

Wyllie, D.C. and Mah, C.W., 2004. Rock Slope Engineering, 4th Edition, Spon Press, New York, 431 pp.

Wyllie, D. C., and Norrish, N. I., 1996, Rock Slopes Stability Analysis *in* Landslides: Investigation and Mitigation. Transportation Research Board, National Research Council, Special Publication 247, Washington D.C., Academy National Press.

Wyllie & Norrish Rock Engineers Inc., 2007a. FINAL REPORT: Rock Cut Feasibility Investigation – Slide Curve I-90 MP 59. February 12, 2007.

Wyllie & Norrish Rock Engineers Inc., 2007b. FINAL REPORT: Rock Cut Feasibility Investigation - Jenkins' Knob (MP 57.5). February 12, 2007.

Wyllie & Norrish Rock Engineers Inc., 2007c. MEMORANDUM: I-90 Snoqualmie Pass East Hyak to Keechelus Dam – 2007 Geotechnical Program for Rock Slopes, April 20, 2007.

Wyllie & Norrish Rock Engineers Inc., 2008a. TECHNICAL MEMORANDUM: Slope Stability Analyses for New Alignment. July, 2008.

Wyllie & Norrish Rock Engineers Inc., 2008b. DRAFT TECHNICAL MEMORANDUM: Instrumentation Recommendations and Monitoring. July, 2008.

Wyllie & Norrish Rock Engineers Inc., 2009a. FINAL REPORT: Slide Curve Rock Slope Design. January, 2009.

Wyllie & Norrish Rock Engineers Inc., 2009b. Phase 1B – Rock Slope Engineering Draft Report. January, 2009.