I-90 SNOQUALMIE PASS EAST PROJECT

Final Report – Slide Curve Rock Slope Design

2007 Geotechnical Analysis & Reporting

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Washington State Department of Transportation

U. S. Department of Transportation
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Acknowledgements

I-90 Snoqualmie Pass East

Agreement No. 9764

Task Order CA

Final Report

Slide Curve Rock Slope Design

2007 Geotechnical Analysis & Reporting

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ACRONYMS

ADAS	Automatic data acquisition system
ASTM	American Society for Testing and Materials
bgs	Below Ground Surface
CTM	California Department of Transportation (Caltrans) California Test Method
DM	WSDOT Design Manual
I-90	Interstate 90
ISRM	International Society of Rock Mechanics
MP	Milepost
MSE	Mechanically stabilized earth
MSL	Mean sea level
NAF	New Alignment Feasibility
ODOT	Oregon Department of Transportation
pcf	pounds per cubic foot
PS&E	Plans, Specifications and Estimates
psf	Pounds per square foot
psi	Pounds per square inch
SCD	Slide Curve Design
TDR	Time Domain Reflectometer
UCS	Unconfined compressive strength
URS	URS Corporation
VWP	Vibrating Wire Piezometer
WBS	Work Breakdown Structure
Wilder	Wilder Construction
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 the area known as "Slide Curve". 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. The historical performance of slopes was used to "calibrate" new designs wherever possible.

The primary results of the study were to confirm the WSDOT design change for the area which had the effect of raising the westbound grade up to 50 feet above the current grade. This has provided dual benefits of reducing the size and extent of required cut slopes while buttressing existing marginally stable slopes. The cost of the raised grade has to a large extent been offset by reduced stabilization costs.

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.

1.0 INTRODUCTION

In order to increase the safety and capacity of Interstate 90 (I-90) through the Snoqualmie Pass area, the Washington State Department of Transportation (WSDOT) is undertaking a series of reconstruction projects to provide additional travel lanes and to improve the geometric parameters of the existing lanes. The western-most project is referred to as Hyak to Keechelus Dam and extends from MP 55.1 to MP

59.9. To assist other planning and design disciplines, Wyllie & Norrish Rock Engineers Inc. (Wyllie & Norrish) was subcontracted through URS Corporation (URS) to provide conceptual-level, leading to design-level, engineering recommendations pertaining to the new and existing rock slopes for this project.

This design report addresses a portion of the project alignment referred to as "Slide Curve" so named for the rock slope failure that occurred in 1957 during the original construction of I-90 (Figure 1). The design report follows an earlier feasibility report (Wyllie & Norrish, 2007a). For ease of reference "Slide Curve Design" is designated by the acronym "SCD". The SCD report is part of a series of reports by Wyllie & Norrish that document various elements of the 2007 field program. Companion reports include the "New Alignment Feasibility" or NAF report (Wyllie & Norrish, 2008a) and a memorandum entitled "Instrumentation Recommendations and Monitoring" (Wyllie & Norrish, 2008b).

The general scope of the SCD rock slopes project included planning and execution of the site characterization studies, testing, analyses and reporting. Strickler's Geological Consulting LLC, Burk GeoConsult LLC and Haneburg Geoscience collaborated with Wyllie & Norrish and contributed significantly to the rock slope design studies herein.

2.0 SCOPE-OF-SERVICES

The work was performed under URS Master Services Agreement No. 131491-UB, Work Order No. 176221-US. The WSDOT prime contract No. was Y-9764 and the Task Order Number was CA. Under this work order the Work Breakdown Structure (WBS) specified a total of four activities. This SCD report fulfills the requirement under URS cost code 33758623.50202 titled "2007 Rock Slope Geotechnical Reports". Note that the original task order was modified to defer reporting of the Jenkins' Knob confirmation studies until after the 2008 field program.

As stated in the task order, the broad objective of this WBS package was to "develop recommendations for cut slopes between Sta. 1395+00 and 1411+00 in addition to stabilization of existing unstable slopes with specific reference to the "scarp area" located upslope of Sta. 1398+00." Project activities to support this objective were to include:

1. Compile all available structural geologic data.
2. For each design sector, to perform kinematic analyses to determine potential controlling modes of failure.
3. Develop slope inclination or rock reinforcement to achieve required stability under static conditions.

4. Prepare slope design templates by station interval.
5. Design rockfall catchment widths and/or mechanical methods for rockfall control.
6. Develop preliminary layouts for rock reinforcement including shotcrete, dowels, rock bolts and drains. Provide guidance on material specifications and a preliminary schedule of quantities.
7. Provide recommendations pertinent to construction.
8. Prepare draft and final reports.

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

3.0 PROJECT BACKGROUND

As mentioned above, Slide Curve was the subject of an extensive feasibility-level study undertaken in 2006 (Wyllie & Norrish, 2007a). An important detail for readers of the feasibility report is that the station reference was changed from Eastbound for the 2006 study to Westbound for the current design study. To avoid ambiguity, this report uses the convention xxxx+yy WB. Based on the observed uniformity of geologic conditions, the project limits for the SCD study were amended to 1396+50 to 1411+ 50 WB, differing slightly from the contractual definition of the project limits described in Section 2.0.

In recognition of the comprehensive nature of the aforementioned feasibility study and to avoid unnecessary repetition, portions of the narrative in this report represent synopses of the previous report. The reader is encouraged to read the two reports in tandem to acquire complete treatment of technical issues. However, the quantitative data on which the design analyses and recommendations are based represents a composite database from the 2006 and 2007 Wyllie & Norrish studies, as well as from previous investigators.

The feasibility report (Wyllie & Norrish, 2007a) affirmed the viability of high rock cuts as a methodology to provide a wider alignment around Slide Curve. Key findings in this regard were as follows:

1. Due to the height of the proposed rock cuts (up 130 feet), extensive mechanical reinforcement was anticipated to provide minimum WSDOT stability margins for new slope construction.
2. The 2006 field program identified for the first time the presence of mid slope fissures on the regraded slope proximal to the location of the 1957 failure. The scope and schedule for the feasibility study did not permit the implications of these features to be determined and thus it was recommended that the follow-on design study investigate their nature and genesis. During the

execution of the 2007 field program the fissures were identified as potential fatal flaws to the preferred alignment at that time. This led WSDOT to develop a split-grade alignment with the westbound lane being elevated relative to the eastbound by as much as 50 feet.

The combination of these two findings is that the alignment and profile for this design study differ significantly from those for the feasibility study. The effect of the raised grade for the westbound lanes has been to not only buttress unstable existing slopes but also to reduce the heights required for new rock cuts; an important collateral benefit to reduce stabilization requirements. This evolution of the highway alignment must also be borne in mind when reading the feasibility and design reports for Slide Curve.

A final point of clarification with respect to the feasibility report concerns the definition of “Design Sectors”. The feasibility report adopted Design Sectors 1 through 6. For this design study the designations have been changed in order to be integrated with overall project alignment rock cut nomenclature. Within the project limits for Slide Curve the Design Sectors progress from XIII to XVI in the eastbound direction. The locations of these Design Sectors on an oblique aerial photograph and topographic plan are shown on Figures 2 and 3, respectively. Figures 4 through 6 present plan and profile views of each Design Sector, while Figures 7 through 17 depict representative cross sections for the sectors. Extensive reference to these figures is made throughout this report for the presentation of geological and geotechnical information.

4.0 PROJECT SETTING

Topography, climate, and regional geology of the Snoqualmie Pass area are all factors that affect assessment of the feasibility of the highway realignment. This section presents a brief overview of these factors and sets the stage for more detailed assessment of geologic, engineering, and stabilization issues in following sections of this report. Wyllie & Norrish (2007a) provides additional information on the project setting.

4.1 Topography and Climate

Snoqualmie Pass is located along I-90 in the Cascade Mountain Physiographic Province of Washington State. The Interstate is located on the northeast side of a steep, open parabolic-shaped (U-shaped) valley. The proposed highway realignment area is bordered to the west by Keechelus Lake and by Keechelus Ridge on the east. From approximately a 2,550-foot elevation above mean sea level at the highway, Keechelus Ridge rises to over 4,000 feet. Further north along this Ridge, mountain peaks including Mount Margaret reach elevations in excess of 5,500 feet (Figure 1).

Precipitation is highly variable from west to east across the Cascade Mountain Range. Major storms often enter Washington from the southwest and there is a strong orographic effect along the west side of the Cascades. Based on 30+ years of precipitation data from a monitoring station at Snoqualmie Pass, the average annual precipitation was approximately 105 inches. The Snoqualmie Pass station annual snowfall ranged from 172 to 800 inches with an average of approximately 460 inches of snowfall. Variations in temperature are common throughout the winter with rapid snowmelt and rain-on-snow events that generate large amounts of runoff occurring frequently at the elevation and location of the highway realignment.

4.2 Regional Geology

Glacial processes dominate the surficial geology of the area around Keechelus Lake 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 to the west. The valley that now contains Keechelus Lake is one of these glaciated valleys. Glacial scouring removed many of the previously existing surficial deposits and scoured the rock outcrops visible today. Keechelus Lake was dammed in part by glacial deposits and in part by an earthfill structure completed in 1917.

The Snoqualmie Pass area bedrock geology comprises an older basement of accreted terranes overlain by younger Tertiary rocks of the Cascade volcanic arc. Some of the earliest rocks related to the Cascade Arc include the Keechelus Lake member of the Ohanapecosh Formation. These are the rocks that underlie much of the realignment area north of MP 59. Further south older Naches Formation comprise sedimentary and rocks that pre-date the formation of the Cascade arc. Igneous dike rocks, likely related to magmas associated with the Cascade Arc volcanoes, are present along the highway realignment area and attest to the intrusive history of the area. Hydrothermal fluids associated with these intrusive bodies were potentially significant in the alteration of the earlier Tertiary volcanic rocks, such as the Ohanapecosh and Naches Formations. Most of the volcanic rocks in this area contained volcanic glass which led to rapid alteration and locally significant loss of rock strength when exposed to weathering and hydrothermal alteration.

Understanding the complex volcanic and tectonic history of the highway realignment area is central to rock mass characterization and the rock slope engineering based on that characterization. Volcanic flow boundaries and associated interbeds represent discontinuities with potential significance to rock slope stability. Variability of volcanic rocks both within flows and between flows has led to differences in weathering and consequently rock strength that have implications for rock slope stability. Similarly, persistent joints and faults, probably associated with a major non-active strike-slip fault in the nearby Kachess drainage, affect rock mass stability. Additional information on the geologic aspects of rock mass characterization is provided in Wyllie & Norrish (2007a).

5.0 FIELD PROGRAM

The 2007 SCD geotechnical field program included surface mapping, borehole structural core logging, borehole televiewer imaging, borehole instrumentation and sample section. These field activities were targeted to supplement the existing database for final design analyses leading to cut slope recommendations between stations 1396+50 and 1411+50. A specific aspect of the 2007 field program was to further investigate three fissures identified by Wyllie & Norrish in 2006. These fissures were proximal to the inferred head scarp from the 1957 rock slide and required additional investigation to determine their genesis and possible significance to the proposed new I-90 alignment.

The Wyllie & Norrish scope-of-services was submitted in a memorandum entitled *I-90 Snoqualmie Pass East Hyak to Keechelus Dam – 2007 Geotechnical Program for Rock Slopes* dated April 20, 2007 (W&N, 2007c). Based on the findings of the fissure excavation, further subsurface investigation of the area was proposed under Wyllie & Norrish Memorandum *I-90 Snoqualmie Pass East Hyak to Keechelus Dam - Supplemental Scarp ("Fissure") Investigation*, dated September 23, 2007 (W&N 2007d). The detailed objectives of the 2007 SCD field investigation were:

Fissure Investigation and Instrumentation

1. Excavation of three fissure locations for visual observation to determine the genetic origin of the fissures and the lateral/vertical extent of the fissures;
2. Completion of structural mapping and remote video of exposed rock outcrops and openings exposed from excavation activities to characterize the structural fabric and extent of the fissures;
3. Execution of subsurface drilling program to assess subsurface geologic and structural conditions of the rock mass and evaluate subsurface discontinuity frequency and condition through detailed rock core logging and completion of downhole optical and acoustical televiewer surveys;
4. Installation of vibrating wire piezometers (VWP) for long term monitoring (via telemetry) of groundwater levels;

5. Installation of slope displacement monitoring equipment including inclinometer casing and in-place inclinometers for long-term monitoring (via telemetry) of possible slope movement.

Rock Slope Design Investigation and Instrumentation

6. Assessment of subsurface geologic and structural conditions for rock mass characterization;
7. Evaluation of subsurface discontinuity frequency and character through detailed rock core logging and completion of downhole optical and acoustical televiewer surveys;
8. Installation of vibrating wire piezometers (VWP) for long term monitoring (via telemetry) of groundwater levels.

5.1 Fissure Excavation

The locations of the three fissures identified during the 2006 Slide Curve Feasibility Investigation are shown on Figure 18. After initial discovery, two hypotheses for the origin of the fissures were proposed:

> '*Hypothesis 1 is that the re-grading did not completely remove the 1957 landslide to the slide plane and these fissures are evidence of reactivation along this plane. Hypothesis 2 is that these fissures represent settlement and creep of uncontrolled fill materials that were cast across the slope during unloading of the slope in 1957 and further re-grading in 1970.*'

It was recommended that an excavator be employed to exhume the fissures as part of the 2007 field investigation. The term "fissure" was selected because it lacks a genetic denotation and the origin of these features was unknown when first discovered. As described in Section 7.0, two of the three fissures are considered bedrock landslide head scarps and hence the nomenclature was changed to reflect the genetic origin of these features. The following summarizes the 2006 and 2007 nomenclature for the field investigation and reporting efforts:

2006	2007
Fissure #1	Fissure #1
Fissure #2	Fissure #2 and/or Scarp #1
Fissure #3	Fissure #3 and/or Scarp #2

URS contracted Wilder Construction (Wilder), based out of Bellingham Washington, to complete the excavation and backfill of each fissure. Following access and excavation permitting by WSDOT, the contractor mobilized to the project site in early August, 2007. Initially Wilder reopened access to the staging area just south of the Slide Curve regraded slope. On August 14, WSDOT, URS, Wilder and Wyllie & Norrish personnel were onsite to assess the capability and safety of accessing the fissure locations using an excavator. Due to the presence of bedrock, limited outboard stability along the proposed access road and the risk of dislodged rock falling against the temporary rock fall control fence protecting the westbound lane of I-90, it was determined that the excavator was not able to mobilize to the fissure locations to complete the necessary excavation work. Consequently, hand scaling and hand excavation methods were substituted at each of the three locations.

The hand excavation labor commenced at Fissure #2 on August 15, 2007. Scaling bars, splitters, and other hand tools were used to excavate the loose rock and fencing was used to store the spoils adjacent to the excavations for subsequent backfilling. Typically one large excavation was advanced at each fissure with supplemental 'potholing' to identify the lateral extent of the fissure. Wyllie & Norrish personnel completed daily inspection of the excavation and provided direction regarding additional work. URS provided full-time oversight of the contractor's activities.

The order of fissure investigation was Fissure #2, Fissure #3 and Fissure #1.

At the completion of the excavation of each fissure the following tasks were completed:

1. *Detailed structural mapping of the exposed bedrock*

   Structural mapping included joint set mapping at each location with correlative mapping of joints across the fissure.

2. *Assessment of Displacement*

   Various points along each side of the opening judged to be previously connected were marked and subsequently surveyed. The intent of this task was to assess the magnitude and direction of rock displacement.

3. *Surveying*

   This included the location of the main excavation and the associated 'potholes' for each fissure, the cross-fissure displacement locations (see above) and three cross section survey lines from the westbound centerline to approximately 250 feet past the upper fissure.

4. *Video imaging of the openings exposed by the excavation*

   A downhole video camera with VHS recording capabilities was attached to the end of a graduated rod. The rod was lowered or maneuvered (as possible) through the various openings to view subsurface conditions and obtain information regarding persistence and aperture of the fissure structure.

Figures 18 and 19 provide the locations of each excavation. A detailed field report of the investigation is provided in Appendix A.

Subsequent to the excavation and mapping activities, detailed drilling investigation was deemed necessary to characterize the displaced rock mass for the proposed I-90 alignment and to determine if the rock mass posed a hazard to the existing I-90 alignment. Following approval of the Wyllie & Norrish scope-of-services by URS and WSDOT, a 12-hole drilling program with associated installation of downhole instrumentation was undertaken. To uniquely identify the holes associated with the fissure investigation, the designation SI-xx-07 was adopted.

5.2 Rock Slope Drilling Program

The objectives of the rock slope drilling program were to investigate and characterize the engineering and geologic subsurface conditions along the proposed highway realignment. This work included a detailed rock mass characterization and the orientation and nature of discontinuities for structural analysis. Wyllie & Norrish developed the proposed location and depth of the boreholes and the type of instrumentation based on: 1) the results of the 2006 field program, 2) existing data available prior to the 2006 field program, 3) proposed realignment and 4) surface geologic mapping activities (W&N, 2007c). The proposed subsurface investigation was reviewed and approved by WSDOT and URS. Modifications to the field program were made based on the results of the ongoing surface and subsurface investigations and were approved by WSDOT and URS, including the addition of twelve boreholes associated with the scarp and fissure investigations (Section 5.1). Twenty-five boreholes were advanced during the 2007 field program totaling 2358.7 feet of drilling. This total was comprised of 1152.7 feet for the rock slope boreholes (designated RKS-xx-07) and 1206.0 feet for the fissure investigation boreholes (designated SI-xx-07). Table 1 provides a summary of the borehole locations from the 2007 and 2006 field programs while Figure 3 shows the borehole locations.

Table 1. Borehole Summary

Borehole #	Dates Drilled	Total Depth (feet bgs)	Northing	Easting	Elevation (feet MSL)	Station (WB)	Offset (feet)
2006 Slide Curve Feasibility Investigation							
H-101-06	9/12 to 9/14/2006	100.0	1062859.39	1754543.14	2,610.85	1398+11	120'L
H-102-06	9/13 to 9/20/2006	200.0	1062885.81	1754804.44	2,800.28	1397+64	380'L
H-103-06	9/7 to 9/11/2006	150.0	1062502.14	1754718.25	2,717.36	1402+44	252'L
H-104-06	9/10 to 9/12/2006	205.5	1062630.17	1754917.84	2,847.59	1401+40	476'L
H-105-06	8/29 to 8/31/2006	135.0	1061849.33	1754935.41	2,652.61	1410+17	94'L
H-106-06	8/25 to 8/28/2006	200.0	1062024.33	1755044.47	2,774.58	1409+65	296'L
H-107-06	9/21 to 9/23/2006	124.0	1062838.20	1754695.85	2,714.70	1398+31	273'L
H-108-06	9/6 to 9/8/2006	159.7	1062681.32	1754761.72	2,752.00	1400+34	330'L
H-109-06	8/15 to 8/23/2006	235.0	1062104.35	1754898.37	2,779.30	1407+73	237'L
H-110-06	9/26 to 9/28/2006	149.3	1063053.54	1754648.93	2,697.62	1395+84	204'L
2007 Slide Curve Design Investigation							
RKS-19-07	9/19 to 9/20/2007	105.1	1063055.73	1754522.696	2,612.07	1396+03	79'L
RKS-20-07	9/22 to 9/23/2007	76.1	1062949.64	1754503.64	2,580.84	1397+17	75'L
RKS-21-07	10/11 to 10/16/2007	70.0	1062783.19	1754497.32	2,579.59	1398+92	74'L
RKS-22-07	10/17 to 10/17/2007	60.5	1062674.24	1754509.849	2,583.71	1400+08	79'L
RKS-23-07	10/18 to 10/19/2007	65.4	1062585.33	1754517.491	2,583.96	1401+02	74'L
RKS-24-07	11/1 to 11/2/2007	65.1	1062493.96	1754536.313	2,580.93	1402+01	74'L
RKS-25-07	11/7 to 11/8/2007	64.9	1062405.78	1754578.036	2,589.17	1403+03	89'L
RKS-26-07	11/3 to 11/5/2007	59.7	1062230.09	1754639.328	2,574.08	1405+00	81'L
RKS-27-07	10/31 to 11/2/2007	75.1	1062142.02	1754678.801	2,585.63	1406+02	74'L
RKS-28-07	10/23 to 10/25/2007	110.0	1062073.86	1754769.593	2,630.99	1407+16	114'L
RKS-29-07	10/20 to 10/22/2007	125.1	1062008.54	1754803.964	2,638.38	1407+95	104'L
RKS-30-07	9/22 to 9/25/2007	135.9	1061918.98	1754853.478	2,645.92	1409+02	84'L
RKS-31-07	9/10 to 9/19/2007	139.8	1061840.10	1754968.698	2,648.34	1410+50	110'L
SI-01-07	9/24 to 9/26/2007	99.6	1062840.90	1754669.88	2,699.90	1398+28	247'L
SI-02-07	9/26 to 10/3/2007	100.1	1062806.25	1754680.48	2,702.77	1398+70	258'L
SI-03-07	10/3 to 10/5/2007	100.3	1062881.96	1754669.25	2,703.38	1397+79	245'L
SI-04-07	10/6 to 10/10/2007	99.8	1062838.58	1754735.04	2,741.47	1398+30	312'L
SI-05-07	10/5 to 10/6/2007	99.8	1062816.02	1754792.34	2,775.53	1398+59	370'L
SI-06-07	10/8 to 10/10/2007	100.0	1062568.87	1754748.12	2,728.41	1401+73	297'L
SI-07-07	10/11 to 10/17/2007	100.2	1062840.29	1754608.98	2,656.52	1398+30	186'L
SI-08-07	10/18 to 10/20/2007	100.1	1062775.03	1754615.84	2,655.08	1399+06	192'L
SI-09-07	10/22 to 10/23/2007	100.1	1062709.95	1754625.68	2,658.34	1399+82	198'L
SI-10-07	10/24 to 10/31/2007	106.0	1062646.46	1754638.01	2,664.25	1400+55	203'L
SI-11-07	11/3 to 11/5/2007	100.3	1062575.56	1754639.04	2,659.45	1401+38	192'L
SI-12-07	11/6 to 11/7/2007	99.75	1062924.46	1754626.98	2,674.02	1397+32	200'L

Notes:

Survey data based on on-ground survey by White Shield, Inc in 2006 and 2007
North and Easting are based on the project datum
bgs – below ground surface MSL – mean sea level

The 2007 subsurface investigation was completed in one phase of work from September 10, 2007 to November 9, 2007. The sequence of drilling was based on upslope rock fall safety considerations to avoid one rig working beneath the other on a steep slope, priority of completing the SI boreholes, and efficiency of helicopter supported drill moves. WSDOT subcontracted CRUX Subsurface, Inc. (CRUX) based in Spokane, Washington, to complete the drilling program using skid-mounted Burley 5500-1 and Burley 5500-2 drill rigs. Drilling began on September 10, 2007 with the Burley 5500-1 and on September 19, 2007 with the Burley 5500-2. Both drill rigs had mobilized to the site in July 2007 to commence the New Alignment Feasibility Investigation before initiating the SCD drilling program. All boreholes were accessed by helicopter with the rig, tooling and instrument installation or backfill supplies flown to each location.

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 or chevron advancer. Rock core drilling was initiated when bedrock was encountered, typically between 0 and 5 feet below ground surface (bgs). Six boreholes (RKS-19, 20, 25, 26, 27 and SI-10) encountered bedrock at between 9 and 19 feet bgs. Rock core drilling and sampling was completed using HQ (2.4-inch core diameter) triple-tube wire line coring methods.

Rock core was described in accordance with the Geotechnical Design Manual (M 46-03), Chapter 4 (WSDOT, 2005). Discontinuity data was further described in accordance with a modified ISRM Suggested Methods (Brown, 1981) procedure. Point-load strength testing was completed at the drill site on the rock core using the ISRM Suggested Method for Determining Point Load Strength (ISRM, 1985). Point load test data from the field testing are provided in Appendix I. 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 WSDOT Hyak Maintenance Facility for temporary storage before transportation to Tumwater, Washington for permanent storage. Boring logs are provided in Appendix B and photographs of the rock core are provided in Appendix C.

All boreholes drilled under SCD were surveyed using downhole optical and acoustical televiewer methods. CRUX Optical Borehole Logging (COBL), a division of CRUX, provided all necessary equipment and staff to complete the downhole surveys and process the data. Wyllie & Norrish personnel reviewed and commented on the draft data sets before COBL issued their final report. The televiewer logs and summary tables of the orientation data are provided in Appendix D.

To further synthesize the borehole data, core from past and present Slide Curve investigations was reviewed by N. Norrish, R. Burk and B. Strickler at the WSDOT State Materials Laboratory in Tumwater, WA on December 1 and 2, 2007. This was accomplished by laying out the core boxes in sequential order, by station, to examine geologic consistency between boreholes and the rationale for definition of Design Sectors. This thorough review compared boreholes logs and televiewer logs to the actual core and provided the preliminary basis for engineering approaches to cut slope design.

5.3 Instrumentation Program

Vibrating wire piezometers (VWP) and slope inclinometer casing were installed in selected boreholes during the 2007 field program. Up to four in-place inclinometers were installed into the slope inclinometer casing. Time-domain reflectometer (TDR) cable was installed during the 2006 field program in select boreholes. A brief summary of the piezometer and slope monitoring instrumentation is provided in the sections 5.3.1 and 5.3.2. Instrumentation installed during both field programs are summarized in Table 2 and 3. The instrument installation procedures, calibration sheets and summary data plots are attached in Appendix E.

5.3.1 Piezometer Instrumentation

VWPs allow for measurement of piezometric pressure from a borehole without the standard screened well construction. The instruments are grouted into the borehole with a variable sand pack and a cable extending from the instrument to the top of the borehole. Changes in pore water pressure within the rock formation are transmitted through the sand and/or grout column and are reflected as a change in the frequency (hertz) of the vibrating wire read at the ground surface. The instrument is calibrated to correlate frequency with pressure which can be used to calculate groundwater level. A total of eight VWPs were installed in six boreholes in 2007 and fourteen VWPs were installed in eight boreholes in 2006. The location, serial number and depth information are summarized in Table 2.

Advantages of the VWP technology for this project included:

1. Ease of installation of multiple VWPs in one borehole.
2. Traditional well development is not required.
3. Discrimination of potential hydrogeologic units within a single borehole.
4. Increased frequency of data collection.
5. 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. One VWP was installed at or near the bottom of each boring with a second VWP instrument (if installed) located above a potential hydrogeologic boundary.

Table 2. Vibrating Wire Piezometer Installation Summary

Borehole #	Ground Surface Elevation (feet MSL)	Borehole Total Depth (feet bgs)	Borehole Total Depth (feet MSL)	Vibrating Wire Piezometer Instrument Installation Data (VWP)				
				VWP Serial Number	Instrument Depth (feet bgs)	Instrument Elevation (feet MSL)	Sand Pack Depth (feet bgs)	Sand Pack Elevation (feet MSL)
2006 Slide Curve Feasibility Investigation								
H-101-06	2610.85	100	2510.85	87455	42.2	2568.6	41.5 - 43.0	2569.4 - 2567.9
				87195	97.2	2513.6	90.0 - 100.0	2520.9 - 2510.9
H-102-06	2800.28	200	2600.28	87451	74	2726.3	73.2 - 74.7	2727.1 - 2725.6
				87179	197.2	2603.1	160.0 - 200.0	2640.3 - 2600.3
H-103-06	2717.36	150	2567.36	87187	147.2	2570.2	120.0 - 150.0	2597.4 - 2567.4
H-105-06	2652.61	135	2517.61	87454	132.2	2520.4	115.0 - 135.0	2537.6 - 2517.6
H-106-06	2774.58	200	2574.58	87453	65	2709.6	64.2 - 65.7	2710.4 - 2708.9
				87178	197.2	2577.4	150.0 - 200.0	2624.6 - 2574.58
H-107-06	2714.7	124	2590.7	87186	68.2	2646.5	66.0 - 73.5	2648.7 - 2641.2
				87188	121.2	2593.5	110.0 - 124.0	2604.7 - 2590.7
H-108-06	2752	159.7	2592.3	87180	95.5	2656.5	94.8 - 96.3	2657.2 - 2655.7
				87175	157	2595.0	130.0 - 159.7	2622.0 - 2592.3
H-110-06	2697.62	149.2	2548.42	87189	72.2	2625.4	71.5 - 73.0	2626.1 - 2624.6
				87177	147.2	2550.4	129.0 - 149.2	2568.6 - 2548.4
2007 Slide Curve Design Investigation								
RKS-19-07	2612.07	105.1	2506.97	92270	102.3	2509.8	90.0 - 105.1	2522.1 - 2507.0
RKS-23-07	2583.96	65.4	2518.56	92089	62.2	2521.8	55.0 - 65.4	2529.0 - 2518.6
RKS-27-07	2585.63	75.1	2510.53	92946	72.3	2513.3	60.0 - 75.1	2525.6 - 2510.5
SI-03-07	2703.38	100.3	2603.1	92060	31.2	2672.2	30.5 - 32.0	2672.9 - 2671.4
				92083	97.2	2606.2	90.0 - 100.3	2613.4 - 2603.1
SI-05-07	2775.53	99.8	2675.7	92082	97.2	2678.3	88.0 - 99.8	2687.5 - 2675.7
SI-07-07	2656.52	100.2	2556.3	92274	40.7	2615.8	40.0 - 41.5	2616.5 - 2615.0
				92081	97.2	2559.3	88.0 - 100.2	2568.5 - 2556.3

Notes:

Survey data based on ground survey by White Shield, Inc in 2006 and 2007
Northing and Easting are based on the project datum
bgs – below ground surface MSL – mean sea level

5.3.2 Slope Monitoring Instrumentation

Slope inclinometer casing allows for accurate subsurface measurement of ground movement that may be a precursor to slope instability. Measurements from in-place inclinometers installed in the casing allow for determination of the displacement vector (direction and magnitude) for quantitative analysis of slope movement. 2.75-inch diameter inclinometer casing was installed to the total depth of five boreholes. The A+ direction of the grooved casing was oriented in the direction of suspected slope movement and grouted in place. “Little Dipper” in-place inclinometers were installed by WSDOT into the casing at select intervals to allow for near-continuous monitoring of slope displacement. The “Little Dippers” measure deflection using an electrolytic tilt transducer and provide an output in voltage that varies in direct proportion to rotation of the instrument and that can be transmitted to the surface. The elevation of each instrument was selected by Wyllie & Norrish Rock Engineers personnel using the rock core logs, observations of the core, review of the core photographs and review of the raw COBL data provided by the optical and acoustical televiewer surveys. A total of 20 in-place inclinometers were installed in five boreholes (four per borehole) which are summarized in Table 3.

TDR cable was installed during the 2006 field program in boreholes judged to be within or adjacent to the historic scarp of the 1957 rock slide. Each cable was installed to the total depth of the borehole. TDR cable is used as a semi-quantitative methodology for monitoring slope stability in that the depth of shear movement is reported but not the magnitude or direction. Data collection consists of attaching a TDR cable tester to the 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 increase correlates to the magnitude of movement on the ground surface. TDR cable was installed in 4 boreholes which are summarized in Table 3.

Table 3. Displacement Monitoring Instrumentation Summary

Borehole #	Ground Surface Elevation (feet MSL)	Borehole Total Depth (feet bgs)	Borehole Total Depth (feet MSL)	Time Domain Reflectometer Instrument Installation Data (TDR Cable)		Slope Inclinometer Instrument Installation Data			
				Cable Depth (feet bgs)	Cable Elevation (feet MSL)	Casing Depth Interval (feet bgs)	Instrument Serial #	Approximate A+ Direction (degrees)	In-Place Inclinometer Instrument Depth (feet bgs)
2006 Slide Curve Feasibility Investigation									
H-101-06	2610.85	100	2510.85	0.0 - 98.0	2610.9 - 2512.9	Instrumentation Not Installed			
H-107-06	2714.7	124	2590.7	0.0 - 122.0	2714.7 - 2592.7	Instrumentation Not Installed			
H-108-06	2752	159.7	2592.3	0.0 - 158.0	2752.0 - 2594.0	Instrumentation Not Installed			
H-110-06	2697.62	149.2	2548.42	0.0 - 148.0	2697.6 - 2549.6	Instrumentation Not Installed			
2007 Slide Curve Design Investigation									
SI-01-07	2703.19	99.6	2603.6	TDR Cable Not Installed		0.0 - 99.6	3515	288	49
							3503		40
							3520		30
							3501		25
SI-02-07	2702.77	100.1	2602.7	TDR Cable Not Installed		0.0 - 100.1	3517	295	67
							3510		51
							3493		43
							3502		28
SI-04-07	2741.47	99.8	2641.7	TDR Cable Not Installed		0.0 - 99.8	3508	274	67
							3513		58
							3518		53
							3519		33
SI-08-07	2655.08	100.1	2555.0	TDR Cable Not Installed		0.0 - 100.1	3523	270	60
							3506		34
							3494		26
							3497		18
SI-09-07	2658.34	100.1	2558.3	TDR Cable Not Installed		0.0 - 100.05	3511	274	69
							3505		45
							3504		31
							3414		22

Notes:

Survey data based on ground survey by White Shield, Inc in 2006 and 2007
North and Easting are based on the project datum
bgs – below ground surface MSL – mean sea level
In-place inclinometer depths were measured from the top of the inclinometer casing
A+ direction measured using declination of 17 degrees East

6.0 LABORATORY PROGRAM

Laboratory testing of rock core samples was used for:

1. Refinement of the field classification of rock strength.
2. Development of a ratio between unconfined compressive strength and point load index for extrapolation of field point load data.
3. Assessment of friction angles of clay-filled discontinuities.
4. Refinement of the field classification of rock type, mineral assemblages, and geologic origin.
5. Evaluation of joint infilling and intact rock for corrosion potential.

6.1 Rock Strength and Corrosivity 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 for diametral and axial point load tests (ASTM D5731), unconfined compressive strength testing (ASTM D2938) and direct shear testing (ISRM Suggested Methods). Selected material from the rock strength testing program was submitted to Sunland Analytical of Rancho Cordova, California for analysis by pH and Minimum Resistivity (CTM Test #643), Sulfate (CTM Test #417) and Chloride (CTM Test #422). The rock strength and corrosion testing reports are provided in Appendix F.

6.2 Petrographic Analysis

Three bulk rock samples were collected from outcrop in 2007 as part of the Material Source Evaluation between stations 1408+50 and 1411+50 and were submitted for petrographic analyses (Burk GeoConsult, 2008). 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 and Dr. Robert Burk (Burk GeoConsult, LLC) contributed to the petrographic work, and Dr. Ray Ingersoll Professor of Geology at University of California at Los Angeles reviewed that work. The Petrographic Report is provided as Appendix G.

7.0 GEOTECHNICAL DATABASE

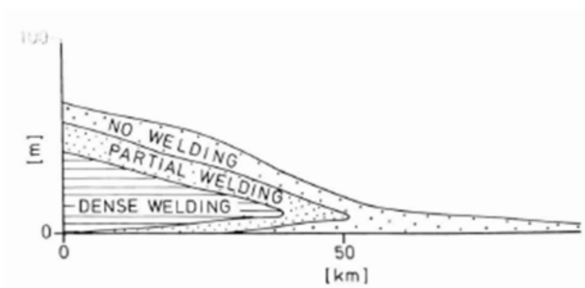
7.1 Previous studies

A more detailed summary of previous work was provided in the Rock Cut Feasibility Investigation – Slide Curve I-90 MP 59 (W&N, 2007a). The previous work considered for this geotechnical database is provided below.

1. Ph.D. dissertation by Paul Hammond (1963).
2. Geologic mapping by Frizzell and others (1984) and Tabor and others (2000).
3. Forensic analyses of the 1957 rockslide at Slide Curve (Ritchie, 1957; 1960; Coombs, 1957; 1958).
4. Select slope characterization between MP 55.17 and MP 110.00 by Chadbourne and Moses (1994).
5. Discontinuity assessment in relation to the anticline present on Keechelus Ridge was completed by Badger (2000).
6. Ongoing site visits and site characterization by WSDOT, specifically work by Steve Lowell in 1998 and Doug Anderson in 1999.
7. Stability analysis and conceptual design parameters for Slide Curve was completed by Golder Associates and Wyllie &Norrish (Findley and Norrish, 2005).
8. A detailed Feasibility Investigation was completed in 2006 (W&N, 2007a).
9. Regional Geologic Mapping (URS, 2007)

7.2 Project Geology

As summarized in Section 4.2, the regional geology of the Slide Curve area comprises pyroclastic flows from the Keechelus Lake member of the Ohanapecosh Formation. In general, pyroclastic flows are extremely heterogeneous in their chemical and depositional characteristics. Frequency of flows, 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 glassy (vitric) fragments) that affect the deposit. An idealized representation of welding within a relatively homogeneous and simple cooling unit is shown from Fisher, 1984. The variability of the original rock composition (protolith), degree of welding and in-situ porosity and permeability



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

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

The complex geologic history of the bedrock in the vicinity of Slide Curve limits the confidence in subsurface extrapolation between boreholes and limited surface outcrops. Because of this variability, Engineering Domains 1 through 3 were developed to characterize the observed geologic boundaries within the rock mass. These bedrock domains are not bound by linear features or by defined flow units. A comprehensive table of the engineering domains was previously developed by Wyllie & Norrish (2007a). The titles and descriptions for each engineering domain were an integral part of the 2007 field investigation, analysis and reporting. A summary of each engineering 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 rock mass of Domain #2 after significant weathering (moderately or greater) occurs.

7.3 2007 Field Results

The field investigative efforts of 2007 focused on two primary objectives.

1. Identify the genesis of the mid-slope fissures and characterize the associated engineering and geologic conditions.
2. Complete a design level site characterization to support the rock cut design.

Additionally, regional geologic mapping by URS (2007) showed the entire regraded area as a Quaternary landslide and that assessment was examined along with other previous work in the area.

7.3.1 Mid-Slope Fissure Excavation

Figure 3 shows a plan view of the Slide Curve area including the fissure locations. Figure 18 provides an oblique view of the areas of investigation and Figure 19 provides a detailed view of the excavation activities for Fissure #2 and #3. Hand excavation tools were utilized to remove small debris up to boulder-sized rocks from the primary excavation for each fissure (Figure 20). The location and sequence of excavation was directed by onsite Wyllie &Norrish personnel. A detailed field report is provided in Appendix A.

Fissure #1

Fissure #1 was exposed with one excavation approximately 20 feet long and up to 3.5 feet deep. It was located across the center of the former access road used during the 1970 slope regrading activities. Bedrock was exposed on the upslope side overlain by 1 to 3 feet of overburden (mixed fill and colluvium) which thickened to the north. A downslope block of rock was observed with 0.8 feet of separation at the top and 0.5 feet of separation near the bottom of the block. At the bottom of the detached block, bedrock was encountered as inferred by the presence of a continuous joint (aperture of less than 1 cm). A remnant blast hole was present on the southern end of the downslope block. The orientation of the discontinuity along which the separation occurred was measured as 72°/071° (Dip/ Dip Direction). This orientation was similar to the regional joint set observed throughout the Slide Curve area and specifically upslope from the excavation where orientations of 87°/ 086°, 82°/ 096° and 86°/ 074° were measured.

Fissure #2

Fissure #2 was exposed with eight excavations spanning more than 200 feet laterally across the slope as shown on Figure 19. The general strike of the fissure was measured at 350° to 355° with local variations. The maximum observed depth was greater than 18 feet; however this was the limit of the exploratory method and does not necessarily represent the maximum depth. Evidence of a backscarp was observed in several of the excavated areas with measured orientations of 70° to 78°/ 076° to 079° (Dip/ Dip Direction). Measured separation between the two bedrock blocks at Station #2-400 and #2-600 ranged from 2.0 to 2.3 feet. Depressions noted in the fill to the north (between Stations #2-100 and #2-300) could be interpreted either as post-1970 (i.e. post regrading) bedrock movement or as settlement of fill into preexisting voids.

In order to quantify direction and magnitude of displacement across this feature, seven specific locations at Station #2-400 were identified as being contiguous prior to movement. The locations were selected based on careful review of the upslope and downslope bedrock faces to discriminate signature areas of intersecting joints or irregular surfaces. Each location was marked, labeled and measured using a Brunton ™ Transit and steel tape measure. The locations were subsequently surveyed by Whiteshield for a more accurate location. Using the survey data the mean orientation of the 7 displacement vectors was calculated to be 19°/258° (plunge / trend) as shown on Figure 22. As a point of comparison, Golder Associates (Findley and Norrish, 2005) estimated the direction of movement on the 1957 slide at 257°.

A downhole video camera was utilized to review and film specific sections of the fissure beyond the limits of the excavation. The camera was lowered into the void spaces and manipulated from the surface to complete a limited visual assessment of the condition and extent of the feature. The maximum measured depth at Fissure #2 was approximately 18 feet.

Fissure #3

The lateral and vertical extent of Fissure #3 was explored with seven excavations spanning more than 125 feet laterally across the slope as shown on Figure 19. The general strike was measured at 005° – 010 ° with local variations. The maximum measured depth was greater than 27 feet; however this represented the limit of the exploratory method. Evidence of a backscarp was observed in two of the excavated areas with orientations of 86°/ 087° and 70°/ 275° (dip/dip direction). Measured separation at Station #3-100 between the two bedrock blocks was approximately 1.7 feet (Figure 23). Displacement points were not identified for surveying due to the limited access at the opening. Station #3-100 represents the northern most limit of the fissure as identified by excavation. Based on the video camera survey, the void space extends at least another 32 feet northward; however it was not exposed due to thick overburden/bedrock overlying the area. The southern termination appears to merge with Fissure #2 based on the observed strike between Station #3-100 and #3-200. The excavations advanced between the two fissures did not encounter any openings or voids; however this may be due to thicker overburden.

A downhole video camera was utilized to review and film specific sections of the fissure void beyond the excavation. The camera was lowered into these void spaces and manipulated from the surface to complete a limited visual assessment of the condition and extent of the fissure. The maximum measured depth was approximately 27 feet.

Presentation of Findings and Recommendations

Wyllie & Norrish presented the findings of the fissure excavation to WSDOT and URS personnel at Hyak, Washington on September 21, 2007. A summary of the findings and recommendations made at that meeting are as follows:

Key Findings:

1. Fissure #1 represents a shallow surface fill failure associated with the former access road and probably developed during the regrading activities of Slide Curve in 1970.

2. Fissure #2 is a bedrock failure scarp with approximately 4 feet of displacement of the downslope block (cumulative measured displacement across Fissure #2 and Fissure #3).
3. Fissure #3 is a bedrock failure scarp with between 1.5 and 2.0 feet of displacement of the downslope block.
4. The location of these two bedrock scarps is proximal to the inferred location of the 1957 rock failure headscarp.
5. The vertical and lateral extent of the two scarps is greater than 18 feet deep and 200 feet long for the lower scarp and more than 27 feet deep and 125 feet long for the upper scarp.
6. Based on a more detailed assessment of the geology of the regraded area than would be possible in a regional geologic investigation (URS, 2007) no evidence was found for other slope instability in the regraded area of the slope.

Recommendations:

1. Complete a more detailed surface and subsurface investigation to characterize the rock mass.
2. Install slope movement monitoring equipment to monitor the current stability of the slope.
3. Modify the proposed alignment to reduce or remove the proposed cuts at the base of the slope.

A plan for the additional scarp investigation was developed by Wyllie & Norrish (2007d), approved by WSDOT and URS, and subsequently executed in the fall of 2007.

7.3.2 Subsurface Exploration

Twenty-five boreholes were advanced for the subsurface investigation. Thirteen boreholes (1,152 feet) were advanced along the proposed cut line and 12 boreholes (1,206 feet) were advanced upslope as part of the Scarp “Fissure” investigation. This complemented 10 boreholes (1,658 feet) drilled in 2006 as part of the Slide Curve Feasibility Investigation (W&N, 2007a). The borehole locations are shown on Figure 3 and summary table of locations is provided as Table 1.

The findings of the subsurface drilling investigation are summarized by borehole in Tables 4 and 5. Table 4 provides a summary of total core recovery, rock quality designation, rock strength, weathering, fracture frequency and distribution of domains by percent for all boreholes. Table 5 provides a summary of key observations by borehole.

Table 4 Borehole Rock Quality Summary

Borehole	Total Depth (feet)	Borehole Elevation (MSL)	Depth to Bedrock (feet)	Total Bedrock Footage Drilled	Total Core Recovery (TCR)	Rock Quality Designation (RQD)					Rock Strength [1]			Weathering [1]		Fracture Frequency/Foot			Domains Observed				
						0 - 25%	25 - 50%	50 - 75%	75 - 90%	90%+	R0 - R1	R2 - R3	R4 - R5	I - II	III - VI	0 - 1	2 - 3	3+	OVB	D1	D2	D3	Core Loss - Broken Zones
2006 Slide Curve Feasibility Investigation																							
H-101-06	100.0	2610.85	0.0	100.0	99.5%	0.0%	0.0%	15.0%	15.0%	70.0%	19.5%	80.5%	0.0%	76.6%	23.4%	83.0%	10.0%	7.0%	0.0%	0.0%	77.9%	20.0%	2.1%
H-102-06	200.0	2800.28	1.1	198.9	99.8%	0.0%	1.8%	7.9%	7.5%	82.8%	0.0%	0.0%	100.0%	100.0%	0.0%	87.4%	10.6%	2.0%	0.6%	99.5%	0.0%	0.0%	0.0%
H-103-06	150.0	2717.36	1.4	148.6	99.5%	0.0%	0.0%	0.0%	9.8%	90.2%	0.7%	0.0%	99.3%	99.6%	0.4%	91.3%	8.1%	0.7%	1.2%	98.8%	0.0%	0.0%	0.0%
H-104-06	205.5	2847.59	0.0	205.0	99.2%	0.0%	2.4%	7.3%	5.0%	85.3%	1.0%	4.9%	94.1%	92.4%	7.6%	87.8%	10.7%	1.5%	0.0%	75.7%	12.8%	11.4%	0.0%
H-105-06	135.0	2652.61	1.6	133.4	98.8%	0.0%	3.6%	7.9%	26.5%	62.1%	0.0%	0.0%	100.0%	100.0%	0.0%	77.5%	19.5%	3.0%	2.4%	97.6%	0.0%	0.0%	0.0%
H-106-06	200.0	2774.58	1.7	198.3	100.0%	3.0%	0.0%	3.0%	5.5%	88.4%	0.5%	8.5%	91.0%	99.5%	0.5%	91.4%	5.0%	3.5%	0.9%	47.7%	51.5%	0.0%	0.0%
H-107-06	124.0	2714.70	0.3	123.7	98.7%	0.0%	2.4%	28.2%	27.8%	41.6%	5.7%	8.8%	85.4%	96.3%	3.7%	60.4%	32.3%	7.3%	0.2%	85.3%	0.0%	8.3%	6.1%
H-108-06	159.7	2752.00	2.8	156.1	99.5%	2.6%	6.4%	13.8%	28.2%	49.0%	4.5%	8.1%	87.4%	93.3%	6.7%	62.2%	32.0%	5.8%	1.8%	85.7%	4.4%	4.5%	3.6%
H-109-06	235.0	2779.25	4.0	231.0	100.0%	0.0%	0.0%	2.2%	8.7%	89.2%	0.0%	11.6%	88.4%	99.8%	0.2%	92.2%	6.5%	1.3%	1.8%	83.7%	14.5%	0.0%	0.0%
H-110-06	149.3	2697.62	5.7	143.6	99.8%	0.0%	1.4%	8.8%	17.4%	72.4%	1.0%	2.1%	96.9%	98.2%	1.8%	86.1%	11.1%	2.8%	3.8%	79.2%	15.5%	0.0%	1.4%
2007 Slide Curve Design Investigation																							
RKS-19-07	105.1	2612.07	10.9	94.2	99.7%	0.0%	4.6%	10.6%	15.7%	69.1%	5.2%	94.8%	0.0%	87.6%	12.4%	72.4%	22.3%	5.3%	10.5%	0.0%	78.1%	11.4%	0.0%
RKS-20-07	76.1	2580.84	8.8	67.3	99.1%	0.0%	0.0%	2.1%	7.4%	90.5%	2.1%	97.9%	0.0%	97.9%	2.1%	92.6%	7.4%	0.0%	11.6%	0.0%	88.4%	0.0%	0.0%
RKS-21-07	70.0	2579.59	1.7	68.3	99.6%	0.0%	7.3%	0.0%	19.9%	72.8%	6.6%	78.8%	14.6%	93.4%	6.6%	91.2%	7.3%	1.5%	2.4%	0.0%	90.3%	1.7%	5.6%
RKS-22-07	60.5	2583.71	1.0	59.5	100.3%	7.6%	8.1%	0.0%	25.2%	59.2%	13.3%	36.8%	49.9%	86.7%	13.3%	69.7%	28.6%	1.7%	1.7%	0.0%	85.1%	13.2%	0.0%
RKS-23-07	65.4	2583.96	1.0	64.4	96.7%	0.0%	7.8%	30.0%	27.3%	34.9%	0.8%	19.1%	80.1%	99.2%	0.8%	61.2%	26.4%	12.4%	1.5%	55.4%	40.2%	0.0%	2.9%
RKS-24-07	65.1	2580.93	5.2	59.9	99.7%	0.0%	0.0%	0.0%	31.7%	68.3%	0.0%	2.0%	98.0%	100.0%	0.0%	86.6%	11.7%	1.7%	7.7%	92.3%	0.0%	0.0%	0.0%
RKS-25-07	64.9	2589.17	13.7	51.2	99.5%	0.0%	0.0%	0.0%	39.1%	60.9%	0.8%	27.0%	72.3%	98.0%	2.0%	82.4%	17.6%	0.0%	21.1%	78.9%	0.0%	0.0%	0.0%
RKS-26-07	59.7	2574.08	17.3	42.4	100.1%	0.0%	11.6%	38.2%	11.6%	38.7%	0.0%	12.3%	87.7%	100.0%	0.0%	67.7%	13.4%	18.9%	29.0%	71.0%	0.0%	0.0%	0.0%
RKS-27-07	75.1	2585.63	19.3	55.8	100.0%	0.0%	0.0%	17.9%	26.9%	55.2%	0.0%	12.2%	87.8%	100.0%	0.0%	69.5%	19.7%	10.8%	25.7%	56.1%	18.2%	0.0%	0.0%
RKS-28-07	110.0	2630.99	2.6	107.4	100.2%	3.7%	9.3%	14.0%	12.5%	60.5%	32.5%	27.1%	40.4%	63.0%	37.0%	75.4%	17.7%	6.9%	2.4%	22.4%	42.6%	32.6%	0.0%
RKS-29-07	125.1	2638.38	0.4	124.7	99.7%	4.0%	4.0%	8.0%	15.8%	68.2%	0.0%	36.8%	63.2%	97.1%	2.9%	64.7%	26.5%	8.8%	0.3%	78.6%	14.5%	6.6%	0.0%
RKS-30-07	135.9	2645.92	2.0	133.4	100.0%	0.0%	2.5%	0.7%	7.5%	89.3%	0.0%	9.2%	90.8%	96.3%	3.7%	88.0%	12.0%	0.0%	1.5%	98.5%	0.0%	0.0%	0.0%
RKS-31-07	139.8	2648.34	1.5	135.3	99.0%	0.0%	0.0%	0.8%	17.2%	82.0%	0.0%	0.0%	100.0%	100.0%	0.0%	90.4%	9.6%	0.0%	3.6%	96.4%	0.0%	0.0%	0.0%
SI-01-07	99.6	2703.19	1.0	98.6	95.8%	0.0%	15.2%	21.9%	12.7%	50.2%	11.9%	8.5%	79.6%	86.6%	13.4%	65.5%	29.4%	5.1%	1.0%	78.9%	0.0%	12.1%	7.9%
SI-02-07	100.1	2702.77	0.7	99.4	99.1%	0.0%	10.8%	29.3%	35.2%	24.7%	2.1%	29.8%	68.1%	92.4%	7.6%	52.7%	39.2%	8.0%	0.7%	44.9%	26.5%	25.5%	2.5%
SI-03-07	100.3	2703.38	2.2	97.9	98.8%	0.0%	0.0%	11.6%	25.5%	62.8%	0.0%	3.5%	96.5%	100.0%	0.0%	67.0%	29.9%	3.1%	2.4%	93.8%	0.0%	0.0%	3.8%
SI-04-07	99.8	2741.47	3.0	96.8	99.0%	2.8%	0.0%	17.1%	12.2%	67.8%	0.0%	16.1%	83.9%	92.3%	7.7%	77.3%	20.7%	2.1%	3.0%	88.0%	0.0%	6.3%	2.7%
SI-05-07	99.8	2775.53	1.5	98.3	99.2%	6.8%	5.1%	5.1%	5.1%	77.9%	4.1%	20.3%	75.6%	96.7%	3.3%	75.6%	24.4%	0.0%	5.0%	40.5%	41.0%	12.6%	0.9%
SI-06-07	100.0	2728.41	1.2	98.8	99.9%	0.0%	1.6%	5.1%	15.2%	78.1%	0.0%	4.0%	96.0%	100.0%	0.0%	90.9%	9.1%	0.0%	1.2%	98.8%	0.0%	0.0%	0.0%
SI-07-07	100.2	2656.52	1.5	98.7	98.3%	5.1%	2.5%	15.4%	36.5%	40.5%	0.0%	10.2%	89.8%	92.4%	7.6%	51.4%	42.6%	6.1%	1.5%	82.3%	5.8%	7.5%	2.9%
SI-08-07	100.1	2655.08	2.7	97.4	93.7%	3.9%	10.6%	20.8%	22.1%	42.6%	9.4%	29.7%	60.9%	85.5%	14.5%	59.2%	26.4%	14.4%	2.7%	59.0%	9.8%	16.2%	12.2%
SI-09-07	100.1	2658.34	2.0	98.1	94.4%	12.7%	14.2%	26.1%	29.8%	17.2%	20.9%	21.9%	57.2%	74.2%	25.8%	56.1%	24.5%	19.4%	2.0%	48.4%	7.0%	16.0%	26.5%
SI-10-07	106.0	2664.25	8.9	97.1	93.0%	5.1%	2.6%	16.4%	33.8%	42.1%	0.9%	11.1%	88.0%	96.4%	3.6%	70.1%	27.8%	2.1%	8.4%	70.0%	12.7%	4.3%	4.5%
SI-11-07	100.3	2659.45	1.5	98.8	98.8%	0.0%	4.9%	0.0%	13.1%	82.1%	0.0%	0.0%	100.0%	100.0%	0.0%	85.8%	14.2%	0.0%	1.5%	98.5%	0.0%	0.0%	0.0%
SI-12-07	99.75	2674.02	1.0	98.8	98.4%	0.0%	3.5%	16.2%	12.4%	67.9%	2.7%	2.4%	94.8%	98.4%	1.6%	69.6%	25.3%	5.1%	1.0%	91.3%	0.0%	2.9%	4.8%

Notes;
[1] = Rock strength and Weathering values based on Washington State Department of Transportation. 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

Table 5 Key Observations by Borehole

Borehole	Total Depth (feet)	Select Key Observations
2006 Slide Curve Feasibility Investigation		
H-101-06	100.0	Upper 21 feet of borehole is D3 rock, likely part of remnant displaced rock mass from 1957 failure
H-102-06	200.0	~ 80 feet upslope (northeast) of Fissure #3, all D1 rock, no evidence of displacement
H-103-06	150.0	Bedrock is D1, strong structural control, good rock quality
H-104-06	205.5	Bedrock is D1 with D2 and D3 rock between 165 feet bgs and the bottom of the boring
H-105-06	135.0	Bedrock is D1, strong structural control, good rock quality
H-106-06	200.0	Bedrock is D1 with interbeds of D2 at 48 to 71 feet bgs and 100 to 180 feet bgs.
H-107-06	124.0	D3 rock with heavy broken zones between 36 and 48 feet bgs, likely remnant failure plane from 1957, D1 bedrock for remainder of borehole
H-108-06	159.7	Bedrock is D1 with D2 and D3 rock from 87 to 104 feet bgs, ~ 3 foot broken zone in D1 rock at 40 to 43 feet bgs
H-109-06	235.0	Bedrock is D1 with D2 rock from 123 to 157 feet bgs
H-110-06	149.3	Bedrock is D1 to 126 feet then D2, strong structural control, good rock quality
2007 Slide Curve Design Investigation		
RKS-19-07	105.1	Thick overburden prism extends upslope (~11 feet thick in borehole), D3 in upper 12 feet of bedrock then D2
RKS-20-07	76.1	~ 9 feet of overburden with D2 bedrock, no evidene of displaced rock mass from 1957
RKS-21-07	70.0	D3 bedrock encountered between 30 and 35 feet bgs, possible zone of failure from 1957
RKS-22-07	60.5	D3 bedrock in upper 9 feet of borehole, then D2; no evidence of failure plane in D2 rock
RKS-23-07	65.4	D1 to 39 feet, then D2 for remainder of borehole; D1 has broken zones 0.2 to 1.1 feet thick near the surface, possibly damage from excessive blasting during construction and re-grading
RKS-24-07	65.1	Bedrock is D1, strong structural control, good rock quality
RKS-25-07	64.9	Thick overburden prism (14 feet in borehole), bedrock is D1, strong structural control, good rock quality
RKS-26-07	59.7	Thick overburden prism (17 feet in borehole), bedrock is D1, strong structural control, good rock quality
RKS-27-07	75.1	Thick overburden prism (19 feet in borehole), bedrock is D2 to 33 feet then D1 for remainder of borehole
RKS-28-07	110.0	Interbedded D2 and D3 rock to 85 feet, D1 bedrock for remainder of borehole
RKS-29-07	125.1	D2 and D3 bedrock from 84 to 110 feet bgs, D1 bedrock for remainder of borehole
RKS-30-07	135.9	Bedrock is D1, strong structural control, good rock quality
RKS-31-07	139.8	Bedrock is D1, strong structural control, good rock quality
SI-01-07	99.6	D3 bedrock from 25 to 43 feet bgs, likely zone of failure from 1957, D1 bedrock for remainder of borehole
SI-02-07	100.1	D3 with zones of D2 bedrock from 26 to 81 feet bgs, likely zone of failure from 1957 between 41 and 44 feet bgs, D1 bedrock for remainder of borehole
SI-03-07	100.3	D1 bedrock with three 0.3 to 1.9 feet thick zones of core loss or heavily broken rock between 14.6 and 33.6 feet bgs
SI-04-07	99.8	D3 bedrock from 51 to 60 feet bgs, likely zone of failure from 1957, D1 bedrock for remainder borehole
SI-05-07	99.8	~ 60 feet upslope (east) of Fissure #3, D3 with zones of D2 bedrock from 26 to 81 feet bgs, D1 bedrock for remainder of borehole
SI-06-07	100.0	Bedrock is D1, strong structural control, good rock quality
SI-07-07	100.2	Broken zone in D1 bedrock from 16 to 19 feet bgs, D2 to D3 bedrock from 1.5 to 14 feet bgs, D1 bedrock for remainder of borehole
SI-08-07	100.1	D2 to D3 bedrock from 17 to 55 feet bgs, moderately to highly weathered zone of D3 at 17 to 29 feet, D1 bedrock for remainder of borehole
SI-09-07	100.1	D2 to D3 bedrock from surface to 53 feet bgs, highly weathered zone of D3 from 20 to 45 feet, D1 bedrock for remainder of borehole
SI-10-07	106.0	D1 bedrock from surface to 106 feet, upper 20 feet has three zones of wide aperature joints (possibly due to blasting), D2 bedrock between 71 and 98 feet bgs with D3 bedrock between 85 and 88 feet bgs.
SI-11-07	100.3	Bedrock is D1, strong structural control, good rock quality
SI-12-07	99.75	Bedrock is D1. Heavily broken rock mass in upper 13 feet. Moderately weathered discontinuity at 89 feet bgs.

The subsurface exploration results from Station 1395+00 to 1402+00 WB were incorporated into a 3-dimensional model by Haneberg Geosciences. The model was generated using the software Wolfram Mathematica and included engineering domains by borehole, topography and the proposed I-90 alignment. Screen shots of the model were utilized in the Design Sector summaries. An electronic copy on compact disc of the model with free viewer software is attached as Appendix H.

7.4 Laboratory Results

7.4.1 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 and submitted to the testing laboratory for analysis. The results are summarized on Table 6 and the laboratory reports are provided in Appendix F.

Table 6 – 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)	Shear Intercept (psi)	Friction Angle (degrees)
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	NT	NT	NT	NT	0.6[i]	19.9[i]
								3.3[f]	15.9[f]
	30.4-31.4	102	Light bluish white friable clasts in light gray, I-II, MWLT	156.4	--	259	5,959	NT	NT
	55.6-56.3	103A	Light bluish white friable clasts in light gray, I, MWLT	--	405	342	--	NT	NT
	56.3-57.0	103B		160.9	--	--	9,148	0.2[i]	27.6[i]
								1.7[f]	31.5[f]
RKS-20-07	64.8-65.4	102A	Light gray, I, MWLT	160.4	--	--	8,358	NT	NT
	65.4-66.1	102B		--	230	282	--	NT	NT
RKS-22-07	11.7-12.5	101A	Light to medium gray, I-II, MWLT	166.0	--	--	9,327	NA[1]	28.1[i]
								0.8[f]	33.3[f]
	12.5-13.4	101B		--	495	645	--	NT	NT
RKS-23-07	10.3-11.0	101A	Medium blue gray, I, MWLT	163.2	--	--	23,917	NT	NT
	11.0-11.7	101B		--	1,097	996	--	NT	NT
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	NT	NT	NT	NT	3.7[i]	14.5[i]
								NA[1]	NA[1]
RKS-26-07	28.2-28.8	101A	Medium greenish to medium bluish gray, I, MWLT	--	852 [I]	974	--	NT	NT
	32.1-32.8	101B		163.9	--	--	16,267	NT	NT
RKS-27-07	28.5-29.2	102A	Light to medium gray to medium greenish gray, I, MWLT	164.6	--	--	18,597	NT	NT
	29.2-29.9	102B		--	1,061	1,197	--	NT	NT
RKS-28-07	65.4-66.0	102A	Light gray, I, MWLT	164.0	--	--	10,535	NT	NT
	66.0-66.6	102B		--	1,286	468	--	NT	NT
	33.0-33.8	201	Medium brown, II-III, MWLT	158.3	--	128 [W,I]	3,588	NT	NT
	27.8-28.5	202	Medium brown, II-III, MWLT	156.2	--	85	1,970	NT	NT
RKS-29-07	18.8-19.5	102A	Medium greenish gray to medium gray, I, MWLT	159.4	--	--	14,605	NA[1]	26.7[i]
								0.6[f]	31.1[f]
	20.6-21.2	102B		--	384	602	--	NT	NT
RKS-30-07	55.1-55.8	103A	Medium blue gray, I, MWLT	161.7	--	--	43,320	0.1[i]	33.4[i]
								0.8[f]	39.2[f]
	55.8-56.4	103B		--	1,134	1,280	--	NT	NT

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)	Shear Intercept (psi)	Friction Angle (degrees)
RKS-31-07	84.8-85.6	103A	Medium blue gray, I, MWLT	--	1,403	1,580	--	NT	NT
	83.5-84.3	103B		162.4	--	--	38,505	NT	NT
SI-1-07	32.0-32.7	101	Medium brown, II-III, MWLT	155.1	--	--	4,680	NT	NT
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	NT	NT	NT	NT	1.1[i]	15.7[i]
								1.7[f]	11.1[f]
	53.4-54.0	103A	Medium brown, II, MWLT	160.5	--	203 [I,W]	2,901	NT	NT
	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	NT	NT	NT	NT	6.6[i]	16.7[i]
								1.2[f]	13.0[f]
SI-4-07	23.6-24.5	101A	Medium blue gray, I, MWLT	162.9	--	--	23,469	NT	NT
	24.5-25.3	101B			993	1,531	--	NT	NT
SI-5-07	33.9-34.8	102A	Light greenish gray, I, MWLT	165.0	--	--	29,523	NT	NT
	34.8-35.7	102B			1,043	1,170	--	NT	NT
	56.9-57.6	103	Medium brown, II MWLT	158.6	--	95 [W]	3,782	NT	NT
SI-7-07	35.6-36.3	101A	Medium blue gray, I-II, MWLT		774	993 [I]	--	NT	NT
	36.9-37.4	101B		163.5	--	--	26,444	NT	NT
SI-9-07	42.4-43.1	201	Light to medium brown, II-III, MWLT	158.9	--	72 [W]	3,348	NT	NT
SI-10-07	24.7-25.4	101A	Medium blue gray, I, MWLT	163.5	--	--	21,770	NT	NT
	26.8-27.6	101B		--	934	1,464	--	NT	NT
	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	NT	NT	NT	NT	3.2[i]	16.9[i]
								1.9[f]	15.7[f]

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
[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 (I) to residual soil (VI)
NA = Not applicable, the shear intercept is negative
NT = Not tested
bgs = below ground surface
pcf = pounds per cubic foot
psi = pounds per square inch

7.4.2 Corrosion Testing

Select samples of bedrock and discontinuity infillings were collected and submitted for analysis. The results are summarized on Table 7 and the Laboratory reports are provided in Appendix F.

Table 7 – 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	6.0	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	315.0
	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	7.90	0.86	11.2	252.0
SI-05-07	66.8-67.3	104	II, R2, MWLT (Clay filled joint-disturbed)	7.70	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 light bluish white clay and rock fragments in fractured blue gray, I, MWLT	8.10	0.70	27.0	271.9
SI-12-07	59.9-60.9	102	Clay-filled Disc, 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
Non-aggressive | Moderately Aggressive | Highly Aggressive

7.4.3 Petrographic Analysis

Limited petrographic analysis was completed in 2007. Hand samples collected from outcrop were analyzed as part of the Material Resource Evaluation (Burk GeoConsult, 2008). Three samples were collected in the vicinity of Slide Curve between WB station 1408+50 and 1411+50 (Sector XVI). The Petrographic Report is provided in Appendix G. The analysis confirmed the rock type in this sector as a meta-welded lapilli tuff of dacitic composition. Of particular importance was the high percentage (25 to 40%) of devitrified glass observed in the matrix of the rock. Because of its clay content and its presence throughout the rock fabric, devitrified glass can increase the rate of weathering of volcanic rocks. The presence of devitrified glass is significant to aggregate use as well as weathering rate of rock slopes when exposed to the weathering environment.

7.5 Salient Characteristics of Design Sectors

Twelve Design Sectors were previously delineated and described between station 1305+00 and 1396+50 WB in the *Technical Memorandum – Slope Stability Analyses for New Alignment* (Wyllie & Norrish, 2008). This section incorporates results from the 2006 and 2007 field investigation to provide concise geologic and engineering summaries for the continuation of these Design Sectors around Slide Curve. Figure 24 provides an oblique image of Slide Curve that shows the boundaries of the four Design Sectors (XIII to XVI) under consideration and the rock domains present at the surface in each sector. Table 8 provides a summary of the various sectors by station and includes the available borehole and surface information collected within each sector.

Table 8 – Design Sector Summary

Design Sector	Stationing		2006 and 2007 Borehole Investigation	Petrographic Analysis		Other Available Data	
	Start	End		Depth (feet bgs)	Collected from rock cut	COBL Logs	Sirovision
XIII	1396+50	1400+50	RKS-20-07	No	None	Yes	No
			RKS-21-07	No		Yes	
			RKS-22-07	No		Yes	
			SI-01-07	No		Yes	
			SI-02-07	No		Yes	
			SI-03-07	No		Yes	
			SI-04-07	No		Yes	
			SI-05-07	No		Yes	
			SI-07-07	No		Yes	
			SI-08-07	No		Yes	
			SI-09-07	No		Yes	
			SI-10-07	No		Yes	
			SI-12-07	No		Yes	
			H-101-06	34.0 [1]		Yes	
			H-102-06	No		Yes	
			H-107-06	No		Yes	
			H-108-06	No		Yes	
XIV	1400+50	1406+00	RKS-23-07	No	None	Yes	No
			RKS-24-07	No		Yes	
			RKS-25-07	No		Yes	
			RKS-26-07	No		Yes	
			SI-06-07	No		Yes	
			SI-11-07	No		Yes	
			H-103-06	No		Yes	
			H-104-06	175.0 [1]		Yes	
			H-17-98	No		No	
XV	1406+00	1408+50	RKS-27-07	No	None	Yes	No
			RKS-28-07	No		Yes	
			RKS-29-07	No		Yes	
			H-109-06	No		Yes	
XVI	1408+50	1411+50	RKS-30-07	No	MRE-SI6A, MRE-SI6B, MRE-SI6C	Yes	001, 002, 003, 004
			RKS-31-07	No		Yes	
			H-105-06	131.9 [1]		Yes	
			H-106-06	56.9 [1]		Yes	

Notes: [1] Petrographic analysis of borehole samples included by reference (Wyllie & Norrish, 2007a).

GEOLOGIC SUMMARY: Design Sector XIII

Station Interval: 1396+50 to 1400+50 WB

Geologic Data Presentation: Figures 4, and 7 to 10

Rationale: The sector boundaries were defined by the inferred and observed limits of the 1957 rockslide and by the results of the surface and subsurface investigation of the two mid-slope scarps.

Proposed Cut Slopes: There are no proposed cut slopes in this section. An MSE Wall will be constructed to raise the proposed westbound grade of the alignment through this Sector.

Overburden Thickness: Depth of overburden ranges from 1 to 5 feet. Overburden is comprised a mix of colluvial and fill material associated with the 1970's regrading of Slide Curve.

Rock Type: All three rock domains were present in this Sector. Geologic cross sections (referenced above) and screen shots of the 3-D model show the distribution of the domains (Figures 25 and 26).

Rock Strength: Rock strength averages for the three domains were:

Domain 1 = Strong to Very Strong Rock (R4 – R5)

Domain 2 = Moderately Strong to Strong Rock (R3 – R4)

Domain 3 = Extremely Weak to Weak (R0 – R2)

Weathering: Weathering is generally defined by domain type. Domains 1 and 2 are typically fresh (I) with slight weathering (II) along some discontinuities. Domain 3 is generally moderately to highly weathered (III – IV).

Discontinuities: Four prominent joint sets.

Key Findings:

1. The two mid-slope fissures identified in 2006 were concluded to be bedrock scarps, likely associated with the 1957 rockslide.
2. The inferred direction of movement associated with the scarps from the 2007 investigation was 258 degrees.
3. Additional movement post 1970 regrading activities cannot be categorically precluded. No quantitative or qualitative data collected to date has verified or discounted this movement.
4. The current depth below ground surface of the shear surface is inferred to be generally between 35 and 45 feet.
5. Geologic extrapolation of the Domain boundaries is provided on the Geologic Cross sections referenced above and on Figure 24. The following observations are made:
    i. Domain 1 is present at the surface above the Domain 3 outcrops exposed along the toe of the slope. This domain comprises the majority of the subsurface geology in the sector.
    ii. Domain 2 is present at depth (typically 15 to 20 feet bgs) adjacent to the alignment and as discrete lenses in several boreholes upslope.
    iii. Domain 3 is present at surface outcrop continuously along the sector and extending upslope between 150 and 200 feet. Discrete lenses of Domain 3 were observed in many of the upslope boreholes.

    iv. There is an apparent dip to the south of Domain 2/3 rock and it is observed at depth in boreholes within Sector XIV.

6. Vibrating wire piezometers (single and dual stage), time domain reflectometer cable and slope inclinometer casing with in-place inclinometers were installed within this Sector.
    i. Vibrating Wire Piezometers were installed into seven boreholes within this sector. The shallower piezometers are reactive to precipitation events. Holes with multiple piezometers show decreasing heads with depth signifying perched water table(s) above a regional groundwater table.
    ii. Time Domain Reflectometer Cable was installed in four boreholes, two outside the inferred slide boundary and two inside the boundary. To date no evidence of deflection has been observed in any of the four instrumented boreholes.
    iii. Slope inclinometer casing with four in-place inclinometers per casing were installed in five boreholes within the inferred slide boundary. No evidence of deflection has been observed.
7. Intermittent surface springs were observed at approximately 1397+30 WB; 175 feet left (upslope). The locations are shown on Figure 3 and in the margin photograph. The springs are highly responsive to precipitation events with discharges at zero during dry periods and greater than 25 gallons per minute during heavy rainfall.

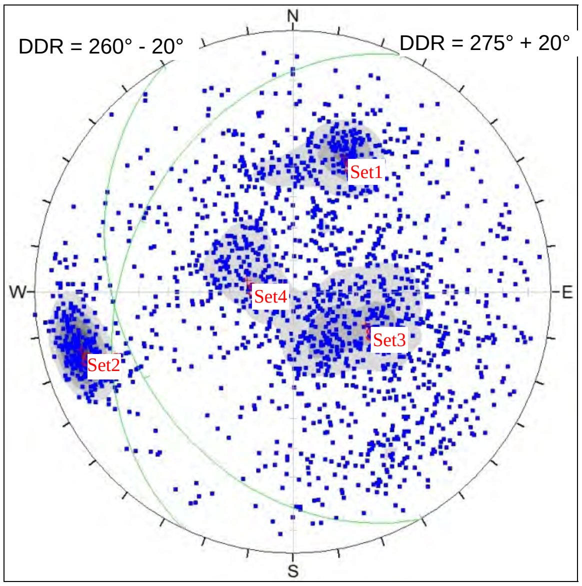
Spring Discharge after heavy rainfall.

ROCK SLOPE ENGINEERING SUMMARY: Design Sector XIII

Station Interval: 1396+50 to 1400+50 WB **Proposed Cut Slopes:** None – embankment only

Slope Dip Direction Range (DDR): 275° to 260° (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 (Domains 1,2) and rock mass strength (Domain 3).



Extensive data set consists of 1778 discontinuity measurements. Structural mapping reported 55 measurements with balance of 1723 derived from COBL data from RKS, SI series boreholes as well as H-101-06, H102-06, H103-06 and H108-06.

Joint sets: Set 1: 46°/201° Set 2: 74°/075°
Set 3: 25°/294° Set 4: 15°/104°

The regraded slope in Design Sector 1 has an inclination of approximately 34° (1.5H:1V). The diffuse Set 3 discontinuities dip slightly less than this inclination and could form basal planes for slabs for west dipping slopes.

Rock quality and fracture frequency in boreholes is highly variable dependent on weathering.

Rockfall Potential: The regraded slope is planned to have an extensive series of snow retention fences installed. These fences combined with the limited slope inclination will prevent rockfalls from impacting the highway alignment. Rockfall from the crest of the regraded slope could potentially damage the upper snow retention fences.

Overburden Stability: Not applicable.

Design Issues: Preliminary design issue is the stability of the existing displaced rock mass identified below the scarps (fissures). See Section 8.4 for analyses and detailed design.

Constructability: Reinforcement of existing slope may experience significant grout loss in displaced rock mass. See Section 10 for specific construction recommendations.

Additional Comments: None

GEOLOGIC SUMMARY: Design Sector XIV

Station Interval: 1400+50 to 1406+00 WB

Geologic Data Presentation: Figures 5 and 11 to 13

Rationale: Boundaries were defined to the north by the southern edge of the 1957 failure, the upslope bedrock outcrop, the extent of the proposed raised grade alignment and subsurface observations.

Proposed Cut Slopes: There are no proposed cut slopes in this section. An MSE Wall will be constructed to raise the existing grade of the alignment through this sector.

Overburden Thickness: Depth of overburden ranges from 1 to 5 feet between 1400+40 and 1402+50 WB and becomes progressively thicker to the south towards RKS-25 through RKS-27 with depths ranging between 13.7 and 19.3 feet bgs respectively. SI-10-07 encountered approximately 9 feet of overburden. The overburden is a mix of colluvial and fill material to 1402+50. South of 1402+50 the overburden is mixed colluvial, glacial till deposits, rock fall deposits and man-made (anthropogenic) fill. The fill was placed for the former access road between the quarry and the regraded area

Rock Type: Meta-welded lapilli dacite tuff. Domain 1 rock was encountered for the majority of the Sector. Domain 2 and 3 rock was observed at depth in H-104-06, SI-10-07 and RKS-23-07.

Rock Strength: Strong Rock to Very Strong Rock (R4 – R5) for fresh (I) Domain 1 and 2 bedrock.

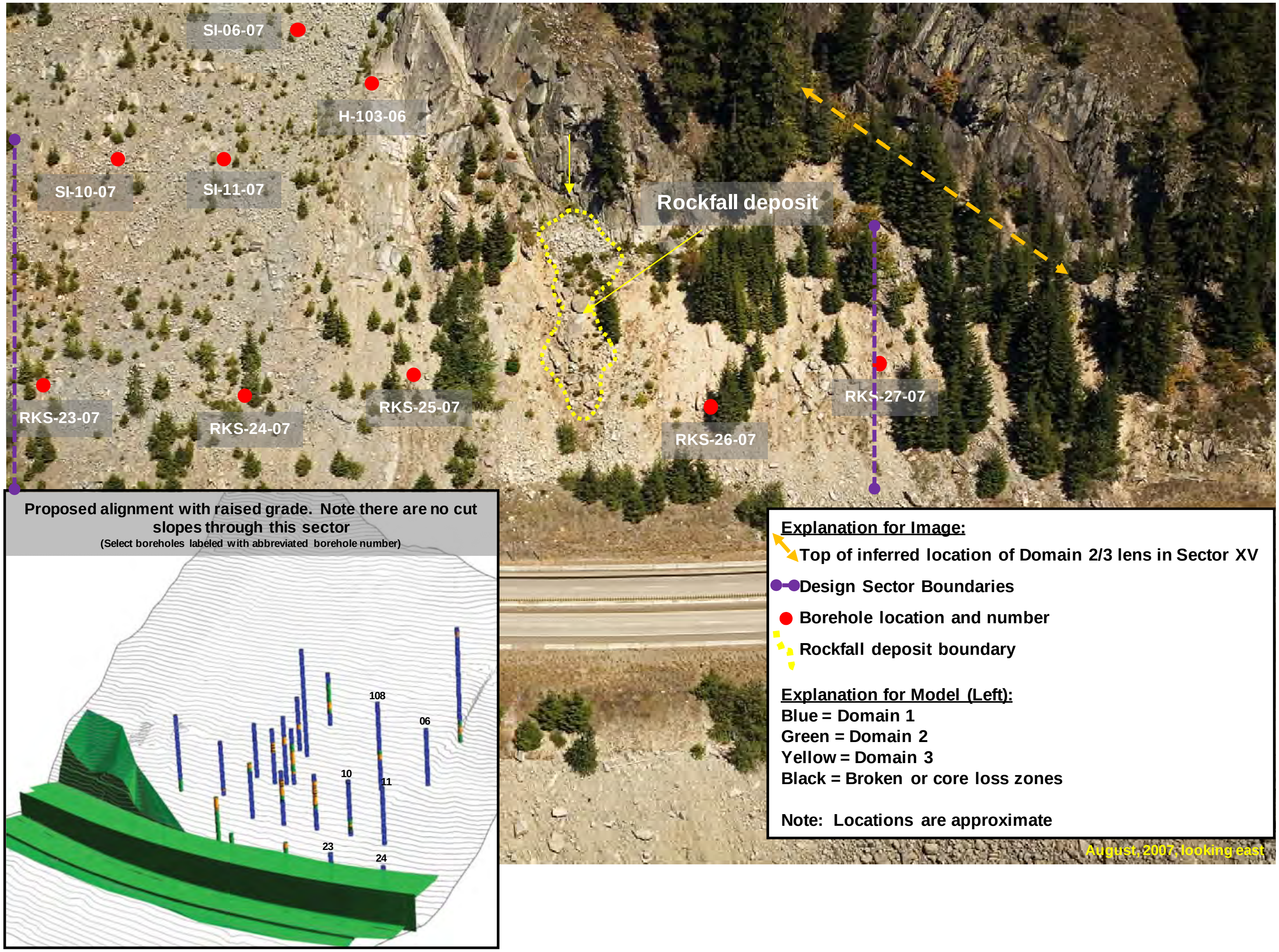
Weathering: Fresh (I) with Slight weathering (II) in upper 3 to 5 feet of bedrock. Slight weathering (II) observed along the discontinuities of Domain 2 bedrock.

Discontinuities: Four well-defined joint sets consistent with adjacent Design Sectors.

Key Findings:

1. Rock quality within this sector is good with generally fresh, strong to very strong Domain 1 rock encountered in all boreholes.
2. A lens of Domain 2 rock was encountered at depth in the boreholes along the northern edge of the sector (SI-10-07, RKS-23-07). The upper contact between Domain 1 and Domain 2 in these boreholes has an apparent dip to the southwest.
3. Rockfalls (single rock blocks and up to ~25 cy multiple block events) have occurred from the bedrock outcrop located above the existing highway (see illustration below) .
4. Fissure #1 is limited in extent and appears to have been caused by blasting and road construction during the regrading activities completed in the 1970's.
5. Evidence of shallow or deep failure surfaces were not observed in SI-06-07 nor in any of the boreholes advanced lower on the slope.
6. Single stage vibrating wire piezometers were installed in H-103-06, RKS-23-07 and RKS-27-07. Groundwater measurement data from H-103-06 (at 147.2 feet bgs) ranged from 49.83 (11/7/06) feet to 84.2 (11/10/07) feet bgs and is responsive to precipitation with elevation values changing 5 to 20 feet each month (refer to Appendix E for summary graphs). Similarly, data from RKS-23-07 ranged from 50 to 55 feet bgs and RKS-27-07 from58 to 63 feet bgs.

Design Sector XIV

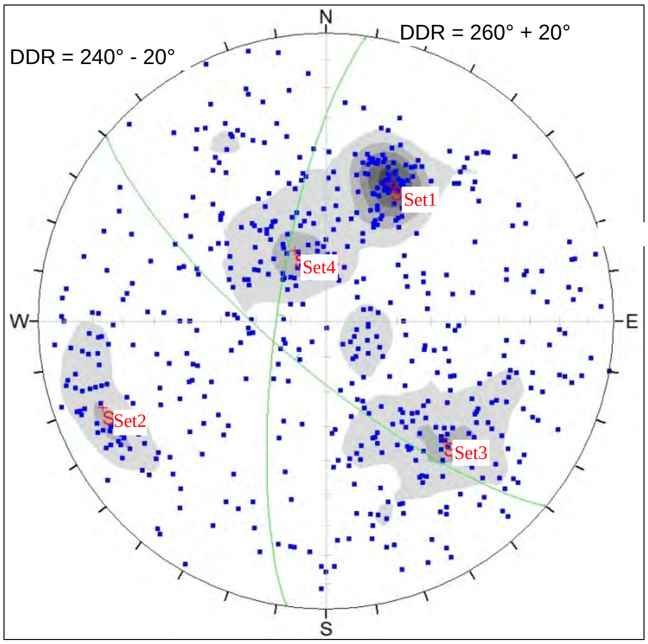


ROCK SLOPE ENGINEERING SUMMARY: Design Sector XIV

Station Interval: 1400+50 to 1406+00 **Proposed Cut Slopes:** None – embankment only

Slope Dip Direction Range (DDR): 260° to 240° (at westbound and eastbound sector limits)

Overall Slope Stability: Slopes will be comprised of strong to very strong rock. Overall stability will be controlled by structural fabric.



Extensive data set consists of 589 discontinuity measurements. Structural mapping reported 55 measurements with balance of 534 derived from COBL data from RKS-23-07 to RKS-27-07, SI-06-07 and SI-11-7 series boreholes as well as H-103-06 and H-104-06.

Joint sets: Set 1: 44°/205° Set 2: 72°/069°
Set 3: 48°/315° Set 4: 22°/156°

Sets 1 and 3 can develop asymmetrical wedges. Slopes are favorably oriented to preclude planar failure along dominant joints.

Rock strength and fracture frequency in boreholes is consistent and indicative of higher quality rock mass.

Rockfall Potential: The existing slopes above the alignment have been a source of rockfalls in the past. The new alignment should include stabilization measures to mitigate this hazard.

Overburden Stability: Embankment only – not applicable.

Design Issues: Primary design issue is the mitigation of rockfall. Overall rock slope stability is not an issue.

Constructability: Stabilization of existing outcrops will require access from scaling ropes. Reinforcement requirements should utilize portable, hand-held equipment.

Additional Comments: None

GEOLOGIC SUMMARY: Design Sector XV

Station Interval: 1406+00 to 1408+50 WB

Geologic Data Presentation: Figures 6 and 14 to 15

Rationale: The Sector boundaries were defined by the upslope bedrock outcrop, the extent of the proposed raised grade alignment, the start of proposed cut slopes, and subsurface observations.

Proposed Cut Slopes: An MSE Wall will be constructed to raise the existing grade of the alignment through this Sector. The cut slopes begin at 1407+50 WB and gradually increase in height to approximately 45 feet at 1408+50 WB.

Overburden Thickness: Approximately 20 feet thick at RKS-27-07 and becomes progressively shallower until RKS-29-07 where it is less than 1 foot thick. Overburden is a mix of colluvial and glacial till deposits with man-made (anthropogenic) fill placed for the former access road between the quarry and the regraded area.

Rock Type: Domain 1 and 2 were observed in RKS-27-07. RKS-28-07 and RKS-29-07 have all three rock domains with approximately one-third of RKS-28 being comprised of Domain 3. RKS-29-07 has less than 7% Domain 3.

Rock Strength: Rock strength averages for the three domains are:

Domain 1 = Strong to Very Strong Rock (R4 – R5)

Domain 2 = Moderately Strong to Strong Rock (R3 – R4)

Domain 3 = Extremely Weak to Weak (R0 – R2)

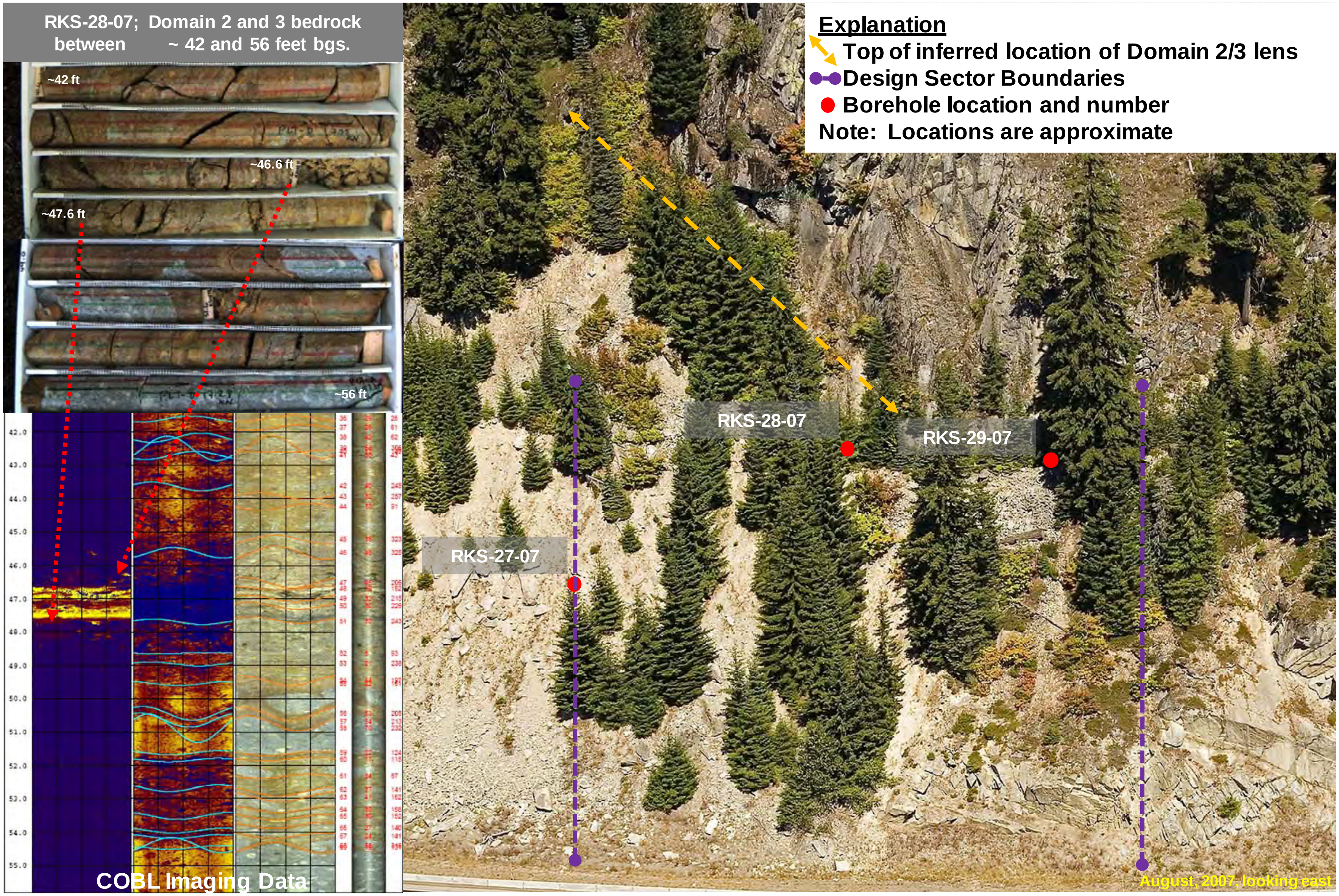
Weathering: Weathering was generally defined by domain type. Domains 1 and 2 were typically fresh (I) with slight weathering (II) along some discontinuities. Domain 3 is generally moderately to highly weathered (III – IV).

Discontinuities: Two prominent joint sets and two subdued joint sets. Orientations consistent with adjacent Design Sectors.

Key Findings:

1. Minor rockfall (block size ranging from < 1 ft^3 to 2.5 ft^3) has occurred from the bedrock outcrop located above the existing highway.
2. Variable thickness of overburden is present along the proposed cutline with steep soil slopes terminating in bedrock outcrops above the alignment.
3. A lens of Domain 2 and 3 bedrock was observed in the subsurface with an apparent dip out of slope of approximately 30 to 35 degrees and was intersected in all boreholes within this sector. This lens of rock was preferentially eroded during previous glacial episodes which formed the “sawtooth” topography observed in Sectors XIV, XV and XVI (Figure 24).
4. The start of the proposed cut at 1407+50 will be in mixed Domain 2 and 3 bedrock, transitioning to Domain 1 bedrock by RKS-29-07.
5. A single stage vibrating wire piezometer was installed in RKS-27-07. Available data from this instrument indicated groundwater levels ranging from 58 to 63 feet bgs but was limited to only two winter months (11/07 to 1/08).

Design Sector XV

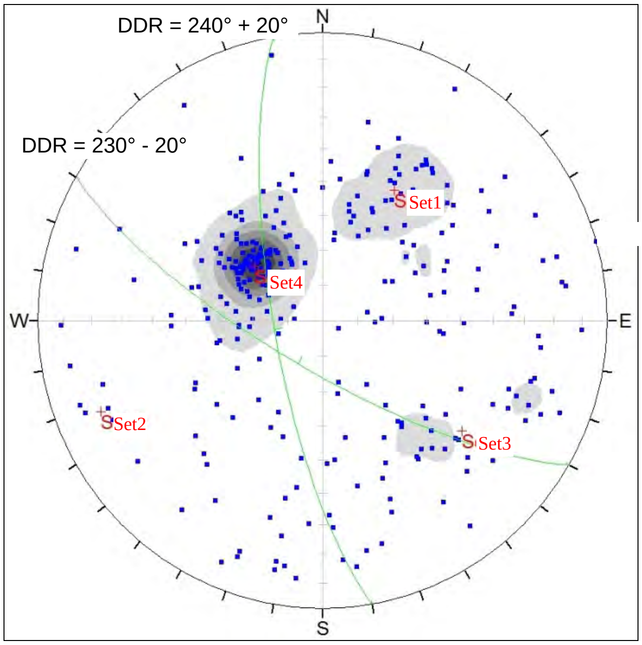


ROCK SLOPE ENGINEERING SUMMARY: Design Sector XV

Station Interval: 1406+00 to 1408+50WB **Proposed Cut Slopes:** 0 to 45 ft increasing to east from 1407+50 WB.

Slope Dip Direction Range (DDR): 240° to 230° (at westbound and eastbound sector limits)

Overall Slope Stability: Slopes will be comprised of variable strength rock dependent on location. Overall stability will be controlled by both structural fabric (Domains 1,2) and rock strength (Domain 3) in the reentrant.



Extensive data set consists of 301 discontinuity measurements. Structural mapping reported 5 measurements with balance of 296 derived from COBL data from RKS-28-07 and RKS-29-07 boreholes as well as H-109-06.

Joint sets: Set 1: 43°/209° Set 2: 73°/068°
Set 3: 52°/308° Set 4: 25°/128°

Set 1 is oriented to create kinematically viable planar failures for southeasterly dipping slopes. Sets 2 through 4 have limited potential to create instability.

Rock quality and fracture frequency in boreholes is reflective of poor quality rock mass in the northern portion of the Design Sector corresponding to the slope reentrant. Elsewhere rock quality is good.

Rockfall Potential: The existing slopes above the alignment have been a source of rockfalls in the past. The new alignment should include stabilization measures to mitigate this hazard.

Overburden Stability: Variable thickness talus and fill above new cuts at eastbound end should be removed.

Design Issues: Primary design issue is the mitigation of rockfall. Cut slopes at the southern end of the Design Sector will require reinforcement in keeping with the adjacent Design Sector XVI.

Constructability: Stabilization of existing outcrops will require access from scaling ropes. Reinforcement requirements should utilize portable, hand-held equipment. A remnant road will facilitate access to the top-of-cut and to the outcrops above the alignment that require stabilization.

Additional Comments: Domain 3 rock could be encountered at the northern limits of the cut slope. Depending on cut height and rock quality, additional stabilization in the form of shotcrete could be required. Actual conditions to be evaluated during construction.

GEOLOGIC SUMMARY: Design Sector XVI

Station Interval: 1408+50 to 1411+50 WB

Geologic Data Presentation: Figures 6 and 16 to 17.

Rationale: The Sector boundaries were defined by the upslope bedrock exposure, the proposed cut slopes, the condition of the existing cut slopes, and subsurface observations.

Proposed Cut Slopes: 45 to 80 feet between 1408+50 and 1410+50 and dropping to zero by 1411+50. An MSE Wall will be constructed to raise the existing grade of the alignment through this Sector.

Overburden Thickness: Ranges from 1 to 2 feet thick along top of cut. Overburden was a mix of colluvial and glacial till deposits with man-made (anthropogenic) fill placed for the former access road between the quarry and the regraded area.

Rock Type: Meta-welded lapilli dacite tuff. Domain 1 rock encountered along proposed cut line. Domain 2 rock was observed at depth in H-106-06 drilled approximately 250 feet upslope.

Rock Strength: Strong Rock to Very Strong Rock (R4 – R5) for Fresh (I) bedrock

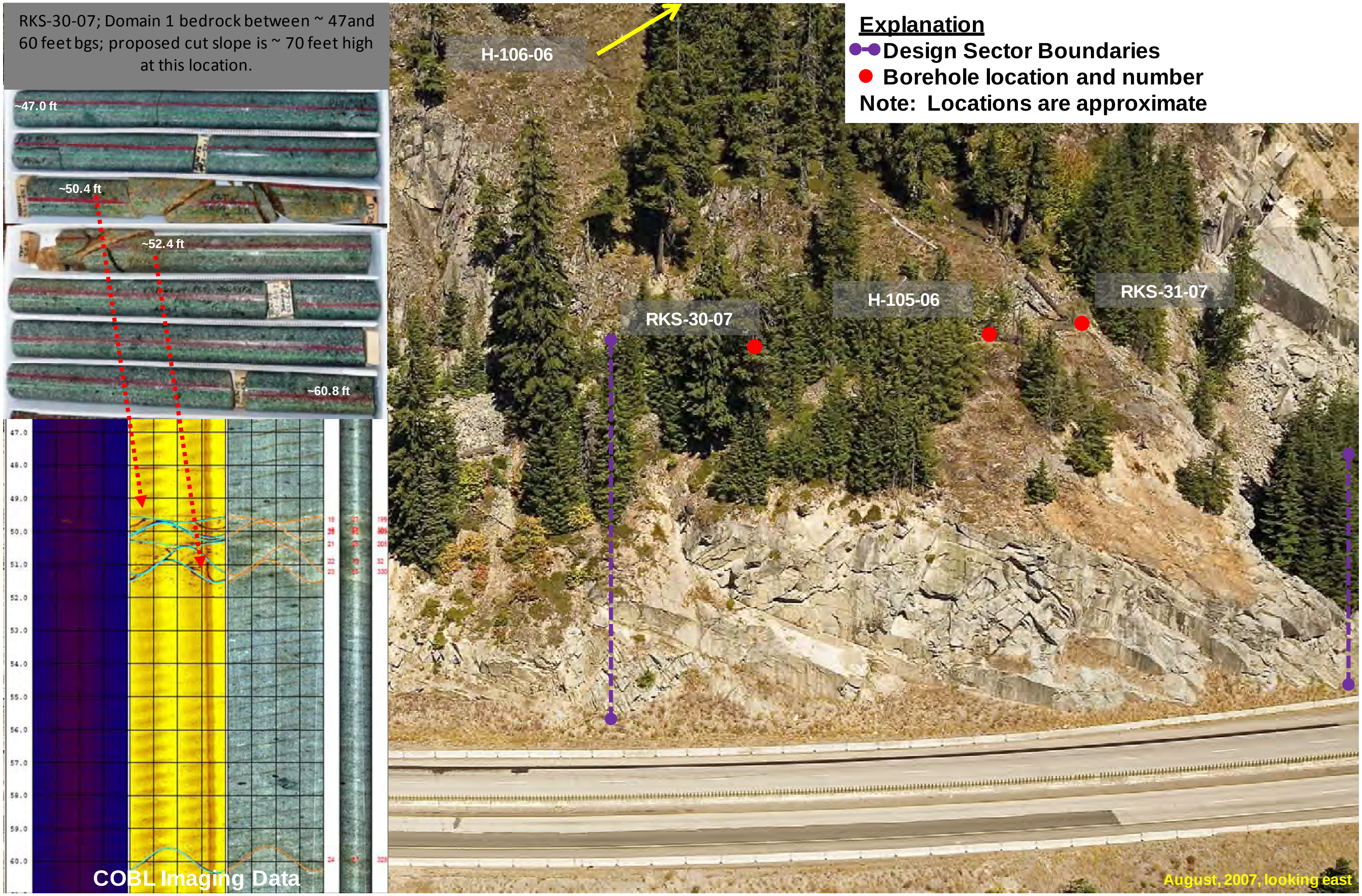
Weathering: Fresh (I) with slight weathering (II) in upper 3 to 5 feet of bedrock.

Discontinuities: Three prominent joint sets and one subdued set. Orientations consistent with adjacent Design Sectors.

Key Findings:

1. Rock quality within this sector was good with generally fresh, strong to very strong Domain 1 rock encountered in all boreholes.
2. Stability of existing rock mass is controlled by persistent and well defined joint sets.
3. Extensive rock bolting of existing cut slope was previously completed by WSDOT.
4. Upslope terrain consists of 25 – 35 degree massive bedrock slope dipping to the southwest. No significant source of rock fall was observed.
5. Proposed cut line traverses former access road between the quarry and the regraded area.
6. Shallow vibrating wire piezometers in H-105-06 and H-106-06 show strong seasonal fluctuation and appear to respond quickly to precipitation events.

Design Sector XVI

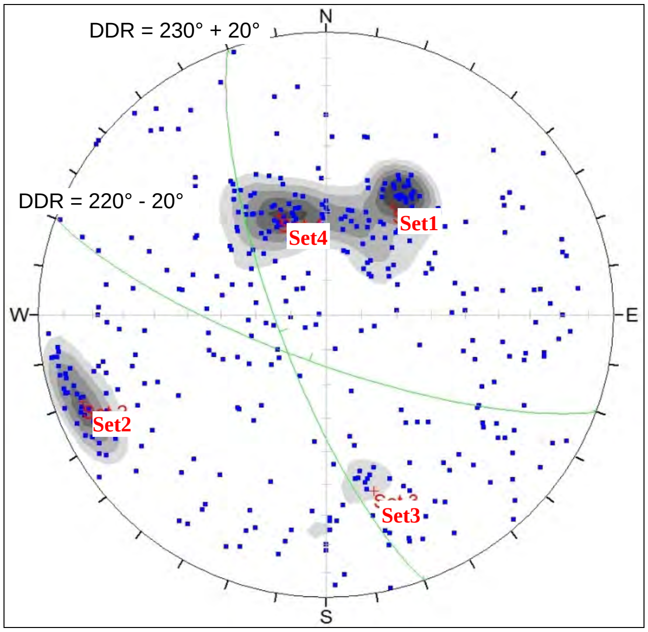


ROCK SLOPE ENGINEERING SUMMARY: Design Sector XVI

Station Interval: 1408+50 to 1411+50 WB **Proposed Cut Slopes:** Up to 80 feet

Slope Dip Direction Range (DDR): 230° to 220° (at westbound and eastbound sector limits)

Overall Slope Stability: Slopes will be comprised of strong to very strong rock. Overall stability will be controlled by structural fabric.



Extensive data set consists of 386 discontinuity measurements. Structural and Sirovision mapping reported 77 and 33 measurements, respectively, with balance of 276 derived from COBL data from boreholes RKS-30-07 and RKS-31-07 as well as H-105-06 and H-106-06.

Joint sets: Set 1: 37°/211° Set 2: 79°/070°
Set 3: 54°/345° Set 4: 32°/155°

Set 1 is oriented so as to facilitate planar failure for southerly dipping slopes. Set 2 may act as a release surface for planar slabs.

Rock quality and fracture frequency are consistent with good quality rock mass.

Rockfall Potential: Rockfall potential from above the cut line is limited. The persistent and blocky rock mass will lead to the potential for large rock falls from cut slopes in the absence of stabilization.

Overburden Stability: Not an issue.

Design Issues: Primary design issue is the stability of cut slopes with persistent planar Set 1 joints that daylight in the face. Extensive cot slope reinforcement will be required (see detailed design recommendations in Section 8. 7.

Constructability: Access to cut slope appears to be feasible from old construction road or by crane from the existing I-90 alignment.

Additional Comments: Historical photographs exist that show planar rock slope failures in the vicinity of Design Sector XVI (see Section 8.7.3.2).

8.0 ROCK SLOPE STABILITY ANALYSES

8.1 Shear Strengths for Slope Design

Section 8.4 of Wyllie & Norrish (2007a) presented the rationale for the selection of rock mass shear strengths and for discontinuity shear strengths for the feasibility-level study of the Slide Curve vicinity. As previously discussed, the design concept for the new alignment has changed significantly from that considered for the feasibility report. The incorporation of the median wall to avoid rock cuts beneath the scarp area has resulted in reduced cut slope heights on the approach grades from the eastbound and westbound directions. This is highly advantageous from a rock cut stability viewpoint, as the stabilization force required to achieve a specified margin of stability is proportional to the square of the cut slope height. Thus, the assignment of shear strength values for the design of the currently favored alignment and elevated westbound grade for I-90 is less critical than that previously considered.

8.2 Groundwater Conditions for Slope Design

WSDOT has employed its remote data acquisition capabilities to develop detailed time histories for the VWP groundwater instruments installed in select geotechnical boreholes in the existing rock slopes throughout the I-90 project. This technology enables very frequent polling of the instruments, typically every two hours, to record electronic frequencies that can be converted to groundwater depth. The overall system has been operational since the fall of 2006 with periodic interruption due to telemetry instrument or wire damage. This data set was used extensively in the rock slope stability analyses and is included as Appendix E. Detailed discussions of groundwater assumptions by Design Sector are included in the appropriate sections below.

8.3 Reinforcement Methodology

Dowel reinforcement refers to the use of untensioned, fully grouted steel members to stabilize a rock mass. The use of dowels in lieu of tensioned rock bolts is attractive for a number of reasons:

1. The requirement to tension the member is eliminated thereby simplifying the installation procedure. It is not necessary to set up on the hole twice (installation and grouting on first cycle, tensioning on second cycle), thereby speeding up the process.
2. Reduced cost for dowels vs. rock bolts.
3. The dowel member is fully grouted in the borehole and its capacity is mobilized only to the extent that the rock mass deforms. Such rock mass deformation tensions the dowel and mobilizes its strength at the location required to resist the movement. In contrast, a tensioned member requires the design of a free-stressing length situated on the collar side of the potential plane of movement.

Thus, the location of the controlling geologic structure must be known with some certainty for proper design.

4. A tensioned rock bolt relies indefinitely on the bearing capacity of the surface rock under the bearing plate to maintain tension. Weathering, blasting or other agents can degrade the integrity of surface rock leading to loss of tension in the rock bolt. Such circumstance can cause the stress in the bolt to be shed to adjacent rock bolts leading to the potential for progressive failure should the bolts become over-stressed. Untensioned dowels are not reliant on the surface conditions because they are bonded to the entire length of the drill hole.

For soil slopes the use of pattern dowel reinforcement, namely soil nailing, has gained technical acceptance as a cost-effective means of slope support. In contrast, and in spite of the advantages enumerated above, the widespread use of untensioned dowels for rock slope stabilization has been limited. The reason for this dichotomy is that the mechanics of the steel dowel – rock discontinuity deformation behavior has been poorly understood. As an example, early practitioners who installed dowels only credited the dowel with its shear strength, typically 50 to 60 percent of the tensile strength. Others would design the dowel reinforcement so that strains remained in the elastic range and maximum bending moments were not exceeded.

The seminal technical paper on the behavior of dowel reinforcement in rock was presented by Spang and Egger (1990). Using physical models and finite element analyses they demonstrated that the materials at the dowel – grout – rock interface reach yield at low shear displacement and that the shear resistance of the doweled joint depends on the post-yield properties of the plasticized materials (Figure 27). From the laboratory and analytical methods, Spang and Egger developed an empirical relationship to predict the beneficial effect of installing a dowel across a discontinuity surface. Note that because this relationship is empirical the calculations must be performed in metric units.

$$R_b = \sigma_{t(s)}(1.55 + 0.011\sigma_{ci}^{1.07}\sin^2(\alpha+i)) \text{ x } \sigma_{ci}^{-0.14}(0.85+0.45\tan\varphi) \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots(1)$$

R_b = *Ultimate shear resistance of joint due to dowel (kN)*

$\sigma_{t(s)}$ = *Ultimate tensile strength of steel bar (kN)*

σ_{ci} = *Ultimate compressive strength of rock / grout (kN)*

α = *Dowel inclination to joint (0 = perpendicular)*

i = *Joint roughness (≡ dilation angle)*

φ = *Friction angle*

The important aspect of this formulation is that the shear resistance provided by the dowel is not only a function of the tensile strength and inclination of the bar, but also of the physical characteristics of the joint surface and the strength of the rock/grout. The action of the dowel is enhanced for discontinuities with greater friction angles and surface roughness because the dowel acts to resist dilation. The consequence of this behavior is that the resistance to movement provided by a dowel can actually exceed its tensile strength, depending upon the dowel inclination. As shown in Figure 27 for inclinations greater than about 10° (measured from the perpendicular to the joint) this strength premium can reach 25 to 35 percent.

The computational methodology for the shear strength of jointed rock was further advanced by Ferrero (1995). Again using laboratory tests on reinforced joints combined with finite element modeling, Ferrero developed analytical models to calculate reinforced joint strength. Important findings from this work were:

1. Yielding steel provides greater incremental improvement to reinforced joint strength than does stiffer, stronger steel. For example, the maximum shear resistance for the most ductile steel tested showed a maximum strength equal to 130 percent of the tensile strength of the bar alone, whereas with the most brittle steels the equivalent value was 115 percent.
2. The beneficial effect of dowels is proportionally greater in weaker rocks. In stronger rock materials, deformation leads to higher stresses in the steel and less overall resistance.
3. Pre-tensioning of the steel bars <u>does not influence the maximum resistance</u> of the system but does affect the stress-strain behavior of the reinforced joint (makes it stiffer).
4. The analytical model developed by Ferrero confirmed the Spang and Egger empirical model for strong rock (UCS > 7250 psi) where the strains are less and the failure is due to a combination of axial and shear force acting at the bar-joint intersection.
5. At rock strengths less than the 7250 psi threshold, Ferrero's work predicted greater strains leading to formation of plastic hinges and a pure tensile stress in the bar at failure. Consequently, Ferrero's methodology results in <u>higher shear resistance</u> than predicted by Spang and Egger.

Based on the above, Wyllie & Norrish propose using untensioned steel dowels as the primary reinforcement type and incorporating the following design methodology:

1. Determine minimum ultimate strength from manufacturer's literature for appropriately sized bar.
2. Calculate ultimate shear resistance force per dowel according to Spang and Egger equation (1).
3. Discount maximum design load to 60% of ultimate.
4. Include the design load as a passive force as follows:

$$FS = \frac{\text{Shear Strength + Dowel Resistance } (R_b)}{\text{Shear Force}} \quad \text{......(2)}$$

The dowel resistance R_b acts on the unstable block parallel to the plane of potential movement and in the direction opposite to movement.

5. Determine horizontal and vertical spacing of dowels to provide appropriate FS.
6. Select minimum hole diameter as twice the bar diameter.
7. Determine allowable bond stress appropriate for rock mass strength.
8. Calculate required bond length beneath potential plane of movement to match design load for dowel.
9. Position bond zone to provide safety margin beneath deepest potential movement surface.
10. Calculate and specify total dowel length.
11. Verify that distal ends of dowels are staggered on adjacent rows to avoid potential movement on a parallel geologic feature just beyond the dowels.

8.4 Design Sector XIII

8.4.1 Background

Design Sector XIII was defined to correspond to the area encompassed by the 1957 rock slope failure and by the presence of mid-slope scarps identified in the 2006 surface mapping program. The 2007 subsurface investigations positively identified the genesis of these features as being slide related, quite possibly as a sympathetic basal slab that did not evacuate with the 1957 failure mass. Earlier versions of the highway realignment required cut slopes that would have “daylighted” or exposed this displaced rock slab. Immediately upon confirmation of the origin of these features, Wyllie & Norrish made a technical presentation to WSDOT South Central Region (September 2007). Soon thereafter WSDOT changed the alignment and grade such that the westbound lanes are situated on an elevated grade some 50 feet above the current I-90 grade through this sector. The effect of this realignment is to not only avoid rock cuts through design Sector XIII but also to provide a buttress to the potentially unstable material. The geotechnical issue at hand for Design Sector XIII is the stability of the regraded slope above the elevated alignment.

8.4.2 Groundwater Conditions

Figure 28 summarizes the available piezometric data for monitoring instruments located along a cross section at station 1398+20 WB. The data record was developed by WSDOT through remote acquisition and storage of frequent instrument readings (typically every two hours). Such reading frequency enables the very short term transient groundwater responses to storm and runoff events to be documented. Figure 28 shows that in general the shallow piezometers (those with “s” suffixes) tend to exhibit more short term fluctuation while the deeper piezometers (“d” suffixes) exhibit a more muted history. Both the shallow and deep piezometers reflect seasonal low water levels in the summer and seasonal high levels in the spring. For engineering analysis purposes “transient” and “high seasonal” water tables were designated.

Figure 29 shows the distribution of measured water levels in cross section. The interesting feature was the inferred presence of an upper and lower water table. The deeper piezometers report lower water levels while the shallow piezometers report higher water levels. Coupled with the transient response, this leads to the interpretation that the upper level represented a perched water table resident in the closely fractured and dilated rock that has displaced while the deeper level represents the true water table reflective of the regional groundwater regime. The ephemeral springs located downslope of borehole SI-12-07 (Figure 29) support this interpretation.

8.4.3 Structural Geology

Kinematic analyses reported in Wyllie & Norrish (2007a) assumed that the structural geology was consistent for the entire Slide Curve interval. Following the 2007 field program the greater quantity of data allowed the structural geology to be discriminated on a Design Sector basis. Figure 30 shows a detailed kinematic analysis for Design Sector XIII based on some 1778 discontinuity measurements compiled from COBL, Sirovison and conventional surface mapping sources. Four discontinuity sets are inferred with Sets 1 and 2 being the most concentrated and most consistent with adjacent sectors at Slide Curve. Sets 3 and 4 are somewhat shallower than elsewhere and of particular interest, Set 3, with a mean orientation of 25°/294°, forms kinematically viable plane failure surfaces for the slope dip directions in Design Sector XIII. However, it should be pointed out the abundance of features with the Set 3 orientation could be simply a sampling artifact of the vertical boreholes.

As discussed in Section 7.3.1 the cumulative rock slope movement in Design Sector XIII is approximately 4 feet with a minimum depth of movement of at least 30 feet as measured at the scarps. With these dimensions in mind, a detailed examination of each borehole along a section at Station 1398+20 WB was carried out to correlate possible planes of movement (shear surfaces) between boreholes. For each borehole possible shear surfaces were interpreted from rock quality histograms, core photographs and televiewer logs and rated as high, medium or low probability. Figures 31 to 36 summarize the ratings for all boreholes located on the cross section in sequence from upslope to downslope. The possible shear surfaces were correlated between boreholes leading to the interpretation of a shallow surface of high probability and a deep surface of lower probability (Figure 37). The shallow surface is constrained to Domain 2 and 3 rock with a steep tension crack at the headscarp connecting with the observed scarps through the Domain 1 rock. The thickness of the displaced rock mass varies from 35 feet to 15 feet, decreasing in the downslope direction. The deep shear surface is equally situated within Domain 1 rock and Domain 2/3 rock. The interpretation for this case is that the shallow discontinuities of Set 3 may have contributed to brittle failure through the strong Domain 1 rock. For the deep surface the thickness of the displaced rock mass varies from 60 feet to 25 feet, decreasing in the downslope direction.

8.4.4 Back Analysis

Back analyses of existing slopes that have experienced movement are useful when considering means of improving stability. Such analyses require knowledge of the slope geometry, the location of the plane or planes of movement, the acting groundwater conditions at the time of movement and whether external forces such as seismic or blasting may have been operative. If all this information is available, or reasonably estimated, the back analysis will yield the average acting shear strength on the plane of movement. The shear strength can then be portioned into frictional and cohesive components using laboratory testing and engineering judgment. Typically laboratory testing can constrain the friction angle to a narrow range thereby permitting the back analysis to yield the cohesion.

For the Sector XIII regraded slope the extensive surface and subsurface investigations provided detailed information concerning the parameters required for a back analysis. The greatest uncertainty concerns the state of stability for the current slope; that is whether it is at a FS value near unity (marginal stability) or whether it is at a somewhat greater degree of stability. The arguments for both interpretations are as follows:

Marginal Stability:

1. Scarps developed during or after the 1957 failure and additional movement has occurred post-1957. The depressions observed above the linear voids would be interpreted as evidence of movement subsequent to the 1970 regrading (Figure 21).
2. A rock slope in a state of marginal stability generally exhibits telltale signs of distress such as slope raveling, vegetation ‘pull-aparts’, hairline tension cracks, etc. The observed scarps may be indicative of this condition.

Greater Stability:

1. Scarps developed during the 1957 event as the basal slab was dragged a short distance (nominally four feet) before coming to rest after which no further movement occurred. Instability of the slab was entirely dependent on the instability of the larger overriding rock mass. Under this scenario,

the depressions observed at the surface represent erosion and sloughing of surficial regraded material into preexisting tension cracks in the bedrock.

2. The regraded slope is at an inclination of 1.5H:1V (34°) which is very shallow for an unstable rock slope, unless it contains persistent, low shear strength, adversely oriented discontinuities.
3. The existing slope does not visually appear to be in a state of marginal stability.

While it is the opinion of Wyllie & Norrish that the latter condition is more probable, the evidence is not incontrovertible and hence the conservative approach is to assume marginal stability for the existing slopes and to provide engineering means to improve stability to a level compatible with the adjacent freeway. In this regard, even though a retaining structure will be proximal, the appropriate minimum FS for slope design is assumed to be 1.25. This is consistent with Section 9.2.3.1 of the Geotechnical Design Manual (WSDOT, 2005).

The approach was to assume shear strength parameters (based on testing and engineering judgment) for various discontinuity and rock mass types and to verify through back analysis that these yielded FS values for the existing slopes at or near unity. The shear strength assumptions are listed in Table 9 below:

Table 9. Design Sector XIII Shear Strength Values

Application	φ (deg)	C (psf)
Rock Mass Domain 1	50	15,000
Rock Mass Domain 2	45	10,000
Rock Mass Domain 3	Hoek Brown: RMR = 35, UCS = 6000 psi, D=0.9	
Shear Surface Domain 1	35	0
Shear Surface Domain 2	32	0
Shear Surface Domain 3	28	0

Incorporating the high seasonal perched water table, these shear strength values yield back analysis FS values of 0.98 and 0.94 for the shallow and deep surfaces, respectively (Figure 37). It might be argued that the extreme transient perched water levels should be used in place of the high seasonal levels. Such an assumption is not conservative for a back analysis as it requires higher shear strength values to resist the groundwater pressures while still reporting FS values near unity. Furthermore, it could be argued that not all parts of the slope would feel the extreme transient water levels simultaneously and hence the approach is invalid.

From the Design Sector XIII back analyses it was concluded that the shear strengths in Table 9 were reasonable for forward analyses of the existing slopes.

It is noted that removal of the displaced rock mass, whether defined by the shallow or deep shear surface, was discounted on the basis of poor quality Domain 3 rock identified in borehole SI-05-07 situated above the scarps (see Figure 8). Removal of the rock mass below the scarps may have created an equally unstable slope above them.

8.4.5 Forward Analysis

Forward analyses for the shallow and deep shear surfaces are summarized in Figures 38 and 39. For the reinforced embankment, the stability analyses used material properties $\varphi = 40°$, $c = 0$, total unit weight = 140 pcf assuming a high quality processed backfill with crushed rock. These values are consistent with Section 5.8.5 of the Geotechnical Design Manual WSDOT (2005).

In keeping with the philosophy described in Section 8.3, Grade 75 #20 bars were selected as the reinforcement steel. These bars have a nominal diameter of 2½ inches and are available in 50-foot lengths. Couplers are 4-in diameter x 9 ½ inches long. The published minimum ultimate strength for this bar is 491 kips.

The allowable or design strength for these bars was determined as follows:

- Published ultimate tensile strength: 491 kips
- Provide 5mm radius allowance for sacrificial steel: Reduce ultimate to 349 kips
  (After Wyllie and Mah, 2004 assuming 100-year service life and intermediately aggressive ground)
- Allowable at 60% of ultimate : 209 kips
- Premium to allowable per Spang and Egger(1990):
  - Deep surface upper row: 269 kips (29% premium)
  - Deep surface lower row: 232 kips (11% premium)
  - Shallow surface upper row: 269 kips (29% premium)
  - Shallow surface lower row: 206 kips (0% premium)

For the shallow and deep failure surfaces three cases were analyzed: 1) embankment only with high transient water table, 2) embankment with two rows of dowels and high transient perched water table, and 3) embankment, dowels and hypothetical drawdown of perched water table. For the geologically more probable shallow surface the stability results are summarized in Figure 38 and below:

Shallow Failure Surface: Case	FS	Relative Improvement
Existing regraded slope - static	0.98	Base case
Embankment only, transient WT - static	1.22	22%
Embankment & dowels, transient WT - static	1.28	29%
Embankment & drawdown - static	1.45	46%
Embankment & dowels & drawdown - static	1.50	51%
Embankment & dowels & drawdown - pseudo static 0.175g	1.07	

For the geologically less probable deep surface the stability results are summarized in Figure 39 and below:

Deep Failure Surface: Case	FS	Relative Improvement
Existing regraded slope - static	0.94	Base case
Embankment only, transient WT - static	1.19	20%
Embankment & dowels, transient WT - static	1.22	23%
Embankment & drawdown - static	1.30	32%
Embankment & dowels & drawdown - static	1.33	35%
Embankment & dowels & drawdown - pseudo static 0.175g	0.91	

The noteworthy aspects of these analyses are the stability improvement margins provided by the buttressing effect of the embankment and by the hypothetical drawdown relative to the high transient perched water table. Conversely the stability improvement provided by the two rows dowels is modest for both the shallow and deep failure surfaces. Note that the failure surfaces in the model were constrained to exit at or near of the toe of the slope in keeping with the location of the 1957 failure. In other words, it was assumed that a new failure surface would not be feasible beneath the proposed embankment because the undisturbed Domain 3 rock has greater shear strength than the embankment at the prevailing low normal stresses.

Based on the analyses, for the segment of Design Sector XIII between Stations 1297+00 and 1300+00 WB, the following stabilization observations and recommendations are provided:

1. The decision to raise the westbound grade around Slide Curve to buttress the unstable rock slopes was absolutely the better alternative rather than to have created new cut slopes that would have destabilized the slopes.
2. The degree of stability provided by the buttress alone does not meet minimum WSDOT requirements for cut slope stability, assuming that the existing regraded slope is at marginal stability.
3. Although the two rows of dowels provide only modest stability improvement (3 to 6%), it is recommended that they be installed because:
    - i. The dowels will protect the integrity of the embankment by forcing upslope instability, should it occur, to toe-out above the embankment.
    - ii. The dowels will improve the global stability of the embankment in the downslope direction. This will be confirmed in a subsequent report by URS as a component of the wall analyses and design.
    - iii. The dowels will be economical to install from the raised grade of the new alignment.

4. Due to the sensitivity of the regraded slope to groundwater conditions, measures should be implemented to prevent water infiltration to the scarps and to stimulate drainage at the toe. The former should be implemented by filling the scarp voids with pea gravel followed by an impermeable bentonite cap (two-foot nominal thickness) at the surface. To stimulate drainage at the toe, 75-foot long drain holes should be installed at 20-foot spacing. Drain holes are typically 3-inch diameter and completed with 2-inch machine-slotted PVC pipe. The outermost 10-feet should be solid pipe grouted in place to prevent vandalism.

It is pointed out that the regraded slope could potentially fail above the dowel reinforcement under adverse groundwater conditions. For the reasons enumerated previously this is considered a very low probability event supported by the fact that the slope has existed in its regraded condition since 1970 without observed distress. Accordingly, the engineering recommendation herein is to provide economical stabilization improvement in the toe area near the embankment and not to pursue higher cost reinforcement options in the dilated rock mass higher up the slope. It is fully anticipated that long term monitoring will corroborate this approach.

Figure 40 illustrates a possible design detail and construction sequence for the interface between the proposed embankment and the existing slope. The important aspects are to create a stable vertical face on which to collar the drain holes and to provide positive drainage from the drains to a collector system that minimizes infiltration to the existing slope beneath the embankment.

8.4.6 Rockfall

Rockfall control will not be an issue for Design Sector XIII. This assertion is based on the following:

1. During the 2007 scarp investigation, a rock rolling field test was performed on the regraded slope. A suite of large, equi-dimensional boulders were rolled from the remnants of the old construction road that traverse the upper portion of the slope. The boulders that reached the 10-foot high temporary rockfall control fence at the I-90 grade were those that were started at locally steeper sections of the regraded slope with the assistance of hand scalers to provide initial energy. Those boulders that reached the toe of the slope were rolling and bouncing upon impact with the fence but none cleared the fence.
2. It is understood that the regraded slope at Slide Curve will have up to ten evenly-spaced snow retention fences installed to mitigate avalanches.

Though not designed for rockfall control, these fences will serve to prevent the build-up of kinetic energy in the unlikely event that any loose boulders are naturally dislodged. Therefore, no specific rockfall control measures are warranted for Design Sector XIII.

8.5 Design Sector XIV

8.5.1 Issues

Design Sector XIV extends from Station 1400+50 to 1406+00. While there are no proposed cut slopes in this sector, a steep blocky “sawtooth” outcrop is present above the alignment (Refer to Figure 24). Rockfalls up to 25 cy have previously originated from this outcrop (see Section 7.5). Without stabilization this outcrop will represent an unacceptable hazard to the proposed westbound I-90 alignment.

8.5.2 Stabilization Design

It is recommended that a combination of rock removal, rock reinforcement and protection be implemented to control rock falls from the upslope outcrop. The rock is strong and blocky and will be favorable for anchoring rock bolts or dowels. Figure 41 summarizes the stabilization requirements, estimated quantities and stabilization sequence for Design Sector XIV. The general approach should be to remove loose blocks and coniferous trees from the slope, followed by installation of reinforcement. Rock removal is anticipated to include trim blasting and hand scaling using bars, air pillows and hydraulic jacks. The majority of the rock removal work will be performed using rope access to the face. Scaling should also remove loose blocks greater than three feet in diameter that are less than half buried in the talus slope beneath the rock outcrops (Figure 41). Excessive digging to remove these blocks should be avoided.

To facilitate drilling for rock reinforcement, small diameter cement-grouted bars are recommended. Dowels and rock bolts will use the same materials with the selection contingent on rock quality at the face. Dowel and bolt holes could be drilled from an air-powered “spider” with personnel suspended from ropes as shown in the margin photograph. For worker safety, reinforcement should be sequenced from the top down.

Of necessity, stabilization requirements must be refined during construction when the results of scaling and detailed geotechnical evaluations are known. The estimated quantities in Figure 41 include a contingency for site engineering. Further variance of these quantities should be anticipated for the preparation of bidding documents.

8.5.3 Rockfall

In addition to the rock removal and rock reinforcement activities described above, it is recommended that a rockfall barrier (fence) be installed on the slope beneath the upslope outcrop. This is described more fully in Section 8.8.

8.6 Design Sector XV

8.6.1 Issues

Design Sector XV extends from Station 1406+00 to 1408+50 WB. The sector is primarily on an embankment with a new cut slope commencing at Station 1407+50 WB and extending to eastbound limit. As with Design Sector XIV, the primary slope issue relates to the presence of a steep natural outcrop upslope of the alignment (Figure 24). This outcrop will pose a rockfall hazard to the proposed elevated westbound grade. A secondary issue is the variable thickness of talus and fill that will be present above the new cut slope that may encounter Domain 3 quality rock (refer to Figure 15).

8.6.2 Stabilization Design

Figures 42 and 43 summarize the stabilization recommendations for Design Sector XV. The same requirements and considerations for scaling, dowel and rock bolt reinforcement, tree removal and drain holes apply as for the previous sector for the existing upslope outcrop. The new cut slope should be stabilized as for the Design Sector XVI design with proviso that shotcrete may be required in addition to the dowels if Domain 3 rock quality is encountered. The variable thickness talus and debris slope above the cut line should be removed down to bedrock (see Note 4, Figure 53).

8.6.3 Rockfall

In addition to the rock removal and rock reinforcement activities described above, it is recommended that a rockfall barrier or fence be installed on the slope beneath the upslope outcrop. This is described more fully in Section 8.8.

8.7 Design Sector XVI

8.7.1 Introduction

Sector XVI was delineated as a design sector based on the requirement to excavate rock cuts ranging up to 80 feet in height. Figures 16 and 17 shows the locations of boreholes RKS-30-07, H-105-06 and H-106-06 relative to the cut slope and median wall proposed for this portion of Slide Curve. Notable from these boreholes is the predominance of Domain 1 and 2 quality rock with the latter found in the lower portion of H-106-06. Very few “significant discontinuities” as defined by clay infilling and/or aperture were identified in the televiewer information, in spite of the presence of highly persistent joints readily observed in outcrop. Although not confirmable, it is prudent engineering practice to assume that a member of Joint Set 1 will daylight the proposed new cut at the toe and that the joint is continuous upslope to the limit of the topography. While this conservative assumption may not be reality, it is a virtual certainty that as the new cut slope is brought down during excavation, members of Joint Set 1 will be "daylighted", and will be potentially unstable. The design issue, therefore, is to determine the type, capacity and location of mechanical reinforcement that, if progressively installed, will provide both short and long term slope stability to meet WSDOT Standards.

8.7.2 Structural Geology and Kinematic Analysis

Figure 44 represents a detailed kinematic or stereonet analysis for Design Sector XVI. The discontinuity poles plotted on Figure 44 were derived from conventional surface mapping, Sirovision mapping and televiewer sources and are therefore minimally affected by sampling bias. The discontinuities form concentrations that are highlighted by contouring. Of the four concentrations (or sets) identified in the stereonet analysis, Sets 1 and 4 are of particular importance to cut slope stability within Design Sector XVI. Set 1 represents the persistent, planar and smooth joints readily observable in outcrop. Based on the structural analysis, the mean orientation of Set 1 within the sector is 37°/211°. The range of slope dip directions for the sector ranges from 220° to 230°. Plane failure is generally considered kinematically feasible if the dip direction of the potential plane of movement is within 20° of the dip direction of the slope (Wyllie and Norrish, 1996). Thus, by adding and subtracting 20° to the dip direction range for the slope, the viability of planar failure can be evaluated. In the present case this range is delineated by the yellow shaded area on the stereonet. As shown, the red great circle representing the mean orientation of Set 1 passes through the shaded area and exceeds the nominal friction angle of 35° signifying that planar failure is kinematically feasible for cut slopes in design Sector XVI.

The significance of Set 2 is that it dips steeply into the slopes and strikes sub parallel to the slopes. With its relative orientation to Set 1 and to the slopes, Set 2 forms release surfaces for potential failure slabs defined by Set 1. This combination increases the potential for planar failures if the slope cuts are steeper than the dip of Set 1.

Having confirmed that the structural geology is amenable to planar failure, the second phase of the stability analysis is to compute the degree of stability base on shear strengths, block size, structural orientations and groundwater pressures. For this application, the design dip of Set 1 was conservatively assumed to be 40°, slightly greater than the mean inclination derived from the structural analysis.

8.7.3 Shear Strengths for Slope Design

The approach for this final design report was to update the feasibility-level database with strength testing performed in connection with the current study. As appropriate, laboratory data was verified with back analyses of current or historic slopes to verify the assignment of shear strength design values for forward analyses.

8.7.3.1 Previous Work: Joint Set 1

Table 10 extracted from Wyllie & Norrish (2007b) summarizes the available data and selected design value for the shear strength for Joint Set 1:

Table 10. Design Sector XVI Shear Strength Values

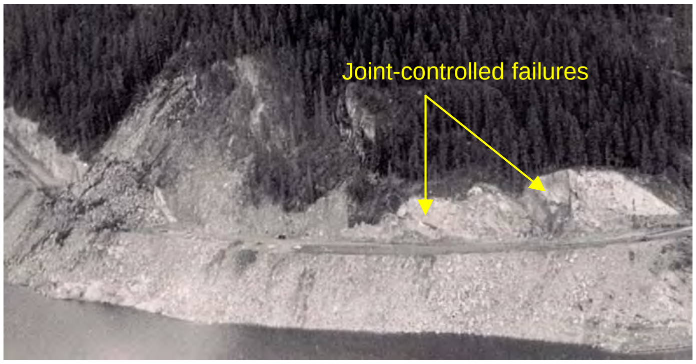
Source	Type of Test / Measurement	Friction Angle (degrees)	Cohesion (psi)	Roughness (degrees)	Comments
WSDOT - Hole No H-17-98	13 multi-stage tests on natural joints	35	0	± 2	Test value incorporates nominal second order roughness
Wyllie & Norrish (1996)	Dacite tuff typical values	27 to 34		0	Φ is function of grain size and mineralogy
Findley and Norrish (2005b)	Field roughness measurements			2	Primary roughness with chord length ± 10 inches
Findley and Norrish (2005b)	Field JRC measurements			5	Corrected JRC for slope scale per Wyllie and Mah (2004)
Findley and Norrish (2005b)	Mapping observation	<40			Blocks resting on joints dipping at 40° have failed Establishes upper bound friction angle with roughness.
Geotest Unlimited (2006)	Ground surface artificial joint	30.6 initial 39.4 final	4.5 initial 6.0 final	0 0	Cohesion is probably artifact of test interpretation, c=0

DESIGN VALUES (means / ranges)
Basic friction angle, Φ = 33° ± 2°
Primary roughness, *i* = 2° ±2°
Total friction, Φ+ *i* = 35° SD = 2°
Cohesion, c = 0 (for displaced blocks)

8.7.3.2 Additional Information

As summarized in Table 6, four additional direct shear tests were performed by GeoTest Unlimited on artificially created “saw-cut” joint surfaces. The samples for these tests were composed of meta-welded lapilli tuff ranging in strength (WSDOT scale) from R2/R3 to R5 and corresponding to Rock Domains 1 or 2. The average initial and final friction angles for the four tests averaged 29° and 34° for the initial and final peaks (see Appendix F for explanation of interpretation). It is the author’s opinion that the initial peak values are more representative and conservative and for these tests the range of interpreted friction angles was 27° to 33°. (see Table 6 and Appendix F). Direct shear testing of artificial joints with zero roughness report values which are primarily dependent on mineralogy and grain size of the rock. Such testing establishes the <u>**absolute minimum friction**</u> value for natural joints of the same material and without strength-modifying infillings.

The **maximum friction** value which includes roughness is best exemplified in circumstances where natural joints are disturbed by blasting so that the cohesion component of the shear strength is destroyed. In such case the stability of the joint surface is directly related to its inclination and frictional strength, including whatever roughness component is available.



The photograph to the left shows that during construction in 1957, slope failures were experienced along Joint Set 1 surfaces in the vicinity of current Station 1412+00 WB, coinciding with the abandoned quarry site. This historic evidence and observations in the quarry lead to the conclusion that the maximum friction angle, including roughness, is on the order of 40°.

The conclusion from the above is that a reasonable allocation of friction shear strength for Joint Set 1 is as follows:

Total friction angle ($\varphi + \boldsymbol{i}$) : Mean = 35° Standard Deviation = 2°

Assuming a normal distribution this means there is a 95 percent probability that the actual total friction angle is between 31° and 39° (mean ± two standard deviations), consistent with the testing results and observational evidence cited above.

8.7.3.3 Back Analysis for Cohesion

The availability and permanence of cohesion along a natural joint surface is more difficult to ascertain. Sampling and testing constraints preclude the testing of a portion of the joint surface that is representative of an in situ feature with a persistence of several hundred feet. Cohesion is the result of secondary infillings such as quartz or calcite, or the presence of intact rock bridges between segments of a discontinuous joint. Such features are difficult to sample and preserve and may not be uniformly distributed across the area of the joint. Two approaches to this enigma are used in rock engineering practice. The first, and most conservative, is to simply assume that the cohesion is zero. In some situations, particularly such as for the 1957 era blasting, the cohesion will be destroyed by the construction practices and the zero cohesion assumption is warranted. The second approach is to back analyze an existing slope from which the cohesion can be calculated assuming the frictional strength is known. The accuracy of such back analyses is dependent on the accuracy of the input variables; slope topography, joint surface orientation, groundwater pressures, external forces etc. In spite of these limitations, back analyses are useful as sensitivity simulations that can lead to a reasonable bracketing of the acting cohesion along a joint surface. Such a back analysis was performed for Section 1410+20, the type section for Design Sector XVI, and is presented in Section 8.7.4.

8.7.4 Back Analysis

An idealized model of the current topography for a back analysis at Station 1410+20 is shown in Figure 45. The slope is assumed to be comprised of Domain 1 and 2 rock that is much too strong to fail except along discontinuities. For purposes of the back analysis, a single Joint Set 1 surface inclined at 40° is assumed to be situated at the toe of the existing cut. Although groundwater has been measured in piezometers installed in H-105-06 and H-106-06, for the purposes of back analysis it is conservative to ignore the groundwater pressures. The shear strength along the joint surface was assumed to be:

Friction angle, φ : mean = 35°, standard deviation = 2°

Cohesion, c: mean = 800 psf, standard deviation = 400 psf

Using Rocscience software Slide Version 5.036, a simplified probabilistic analysis of the model was performed wherein 1000 combinations of the normally distributed random variables, φ and c, were analyzed for the Factor of Safety against planar sliding. In keeping with normal practice, the variables were assumed to be negatively correlated with a coefficient of -0.5. This means lower friction angles are associated with higher cohesion values and vice versa. The scatter plot in Figure 46 upper, shows the demarcation between combinations of φ and c that generate FS values greater or less than unity. It is noted that as the joint cohesion approaches zero, the slope is unstable even at the maximum friction value of 40°, consistent with historical evidence and site observations as described above. Figure 47 lower shows the corresponding distribution for the Factor of Safety for the existing slope and indicates less than a 10 percent probability of failure (FS<1.0) and a most probable FS of 1.18 for the friction angle and cohesion distributions assumed.

As presented in Wyllie & Norrish (2007b) the site has experienced two significant ground motions since construction in 1957. The peak ground acceleration for the Slide curve site from the February 28, 2001 Nisqually earthquake was approximately 0.07g. Performing a back analysis assuming the pseudo static coefficient is equivalent to the peak ground acceleration yields the results shown in Figure 47. In this case the probability of failure rises to 34% and the most probable FS reduces to 1.05.

Because the existing slope is apparently stable and without precursor deformation characteristics indicative of marginal stability, and because the slope withstood the 2001 earthquake, it could be argued that the assumed cohesion distribution is too low. However, given the importance of the I-90 corridor and the uncertainties associated with back analyses, the cohesion values selected for design have been further discounted to a truncated normal distribution. Thus for forward analyses for slope design the following were adopted for Set 1 joints:

Friction angle, φ : Mean= 35° Standard Deviation = 2°
Cohesion, c : Mean = 400 psf Standard Deviation = 400psf
Correlation coefficient = -0.5

These values correspond to a 95 percent probability that the actual total friction angle is between 31° and 39° and that the actual cohesion is between 0 and 1200 psf.

8.7.5 Forward Analysis

For the purposes of forward analysis the cross section at Station 1410+20 was selected as the representative section. The proposed template consists of a cut slope up to 80 feet in vertical height inclined at nominally ¼H:1V (76°) with the westbound highway grade half in cut and half on an embankment (Figure 48). The analysis assumed the presence of a single joint from Joint Set 1 inclined at 40° and situated at the exact toe of the proposed cut. While it is not possible to determine with precision if this location will be encountered, it is known with certainty that members of Joint Set 1 will be encountered as the cut is brought down. The conservative approach, therefore, is to assume the location at the toe and to ensure that other members higher in the cut are stabilized as the cut is incrementally advanced.

Water levels were inferred from seasonal highs measured in piezometers installed in boreholes H-105-06 and H-106-06. Water levels are not anticipated to influence the assumed failure plane, particularly in view of the horizontal drains to be incorporated as integral to the cut slope design.

The model indicates that in the absence of reinforcement the cut slope would have a FS value of about 1.0 and a probability of failure of 40% for the strength and groundwater assumptions discussed above. The selected method of reinforcement to increase the stability margin to meet WSDOT standards was passive (untensioned) steels bars fully grouted in inclined holes. The selection of bar diameter is a tradeoff between number of dowel holes, hole diameter, weight of bar for ease of installation and required drill size compatible with hole diameter and access limitations. Further, as recommended by Ferrero (1995), yielding steel provides greater incremental support than stiff steel. Weighing these factors and referring to manufacturer's specifications, a generic bar consisting of #20 Grade 75 steel was selected. This bar is available in 50-foot lengths, has a nominal diameter of 2 ½ inches, and is rated at 491 kips minimum ultimate strength.

For design purposes the strength of this generic bar was modified as follows:

1) A strength discount based on an allowance of 10mm diameter reduction as sacrificial steel due to corrosion. This was based on the method proposed by Wyllie and Mah (2004) assuming a 100 year service life and moderately aggressive conditions (see Table 7). Furthermore, the analysis assumed the protective encapsulation of the grout is compromised by cracking to allow the corrosion to occur.
2) A strength premium based on the approach by Spang and Egger (1990) and Ferrero (1995).
3) An allowable design load (working load) based on 60% of the ultimate strength as adjusted by 1) and 2) above.

These procedures resulted in an allowable design load rating of **253 kips for Grade 75 #20 bar.**

The dowel pattern for these bars was varied by adjusting the horizontal and vertical spacing. The preferred pattern consists of a 15-foot horizontal spacing and a 12-foot vertical spacing. For this configuration the models predicts a deterministic FS = 1.25 and a most probable FS = 1.29 (Figure 48). The sensitivity of the FS to the distributions assumed for the friction angle and cohesion are shown in Figure 49. It is noted that even if the cohesion of the Joint Set 1 failure surface is zero the predicted FS is about 1.15, and that the calculated probability of failure for the reinforced slope is 0%.

It should be emphasized that the above results apply to the residual steel strength after a 100-year service life. The comparable FS values at the date of completion prior to any corrosion loss are 1.59 and 1.62 (deterministic / most probable). Given the protective capacity of the grout and the fact that not all bars would be compromised simultaneously even if grout cracking and corrosion did occur, there is even greater confidence that the actual FS for the proposed slope will meet WSDOT minimums if the reinforcement is installed in accordance with the proposed design.

The recommended dowel reinforcement layout for the Design Sector XVI cut slope is shown in Figure 50. Note that the cut does enter the adjacent Design Sector XV at the northern (westbound) extremity. The important aspects of this design are as follows:

1. Bar lengths have been varied by row to account for the Joint Set 1 and slope face inclinations and to provide for staggered locations for the distal ends.

2. Bar lengths have been assigned so that cutting and coupling will minimize wastage. This bar should be delivered to the site in 50-foot lengths and assembled to suit the design shown.
3. It is ABSOLUTELY MANDATORY that Row 1 dowels be installed before the rock excavation is initiated. This will present an access challenge to the contractor and will require either a very substantial crane or development of temporary drill platforms if access is from the top.
4. Subsequent rows should be installed each time that 12 feet of the face is exposed. For maximum 24-foot lift heights, this means that the top row within a lift will be drilled and installed from a temporary shot rock berm.
5. The dowels on adjacent rows should have centers offset by half the horizontal spacing.

8.7.6 Rockfall Control

The highway template proposed by WSDOT for Design Sector XVI includes a catchment width of 24 feet inclined at 6H:1V. This design is in accordance with WSDOT design Manual M22-01 Figure 640-7a.

The potential for rockfall from the proposed Design Sector XVI rock cut should be influenced by the following factors:

1. The final slope face will be comprised of strong to very strong (R4-R5) meta-welded lapilli dacite tuff. This rock will be resistant to weathering within the design life of the project.
2. The upper 3 to 5 feet of rock that has been exposed since deglaciation exhibits slight weathering.
3. Joint spacing is generally greater than 1 to 2 feet and the prominent joint sets are oriented such that the cut face will have a blocky character.

Given the alpine climatic environment and the geologic characteristics of the rock mass, it should be anticipated that rockfalls will be a low but tangible hazard to I-90. This hazard is exacerbated by the fact that the westbound alignment will be situated on a curve above a 45-foot high retaining wall and by the fact that any rock fragments that do fall will be strong and will therefore remain intact.

In consideration of the proposed catchment design, the following pertinent narrative (along with Figure 25) was excerpted from Wyllie & Norrish (2007b):

"At issue is the required catchment width which in turn is a function of the slope face angle, slope height, slope face uniformity, catchment geometry and desired performance in terms of rockfall retention. The latter parameter is usually expressed as the percentage of individual rock blocks retained in the catchment area assuming a trajectory that originates from the crest of the slope cut, i.e. from the worst location on the slope due to energy potential. Current WSDOT guidance for roadway sections in rock cuts is provided by Figure 640-7a of Design Manual M 22-01 (WSDOT, 2006b). Therein, a three-stage design approach dependent on the severity of the rockfall conditions is described. Another source of

*design guidance for catchment width is based upon research by the Oregon Department of Transportation (Pierson, et al, 2001) in which thousands of rocks were rolled from the crests of slopes with various inclinations and for heights between 40 and 80 feet. While there is no consistent practice for the selection of the rockfall retention parameter for the catchment, most agencies target a value in the range of 90 to 95 percent. Figure 51 (*reassigned number for this report*) compares the Stage 1 WSDOT design practice for catchment width beneath 0.25H:1V (76º) cut slope with that from the ODOT research project. ……..”*

Figure 51 shows that for an 80-foot slope height the ODOT research approach requires a greater catchment width than provided by WSDOT DM M 22-01(approximately 30 feet and 24 feet respectively), primarily to contain roll out of dislodged rocks after they impact the ditch. Given the potential consequences of rockfalls to the elevated roadway at this site, it is recommended that a site specific design variance be implemented to increase the rockfall retention performance. Possible approaches include:

1) Increase the catchment width to 30 feet by moving the cut slope into the slope. This approach is not recommended because the required width increase only applies to the central and highest portion of the cut slope.

2) Adopt a Stage 2 Alternate design with a concrete barrier as per DM M22-0. This alternative would also require roadway widening to accommodate the barrier width, may impact sight distance, and would have to be compatible with anticipated snow plowing operations.

3) Drape the upper portion of the cut slope such that the effective cut slope height is 50 feet. This approach targets the critical rockfall source zone in terms of height above the roadway (greatest potential energy) and least favorable rock quality due to weathering. As shown in Figure 52, the estimate face treatment area is approximately 7000 sf including allowance for wrap-over at the slope crest. If adopted, the conceptual design in Figure 52 should be refined by WSDOT in the preparation of PS&E.

An acceptable alternative to the rockfall control strategy would be to reevaluate the cut slope after it is excavated and based on the uniformity and quality of the blasted rock face, determine whether the adoption of either the Stage 2 Alternate design or the slope drape is warranted. At such time the selected approach could be implemented by the contractor under force account.

8.8 Rockfall Control from Existing Slopes – Design Sectors XIV to XVI

As discussed in Sections 8.5 and 8.6, the proposed elevated westbound alignment for Design Sectors XIV and XV, and the a lesser extent, Design Sector XVI, will be subject to rockfalls originating from the existing slopes above the grade. Although slope stabilization recommendations have been proposed to reduce the size and severity of these rockfalls, as a further mitigation it is recommended that a continuous rockfall control barrier (fence) be installed from Station 1402+50 WB to Station 1410+50 WB. This fence should take advantage of an abandoned construction road that corresponds to nominal elevation 2630 feet (Figure 53). Because the slope topography is highly variable in this interval, the design was based on engineering experience rather than by rockfall simulations. The recommended height is 10 feet with a minimum design impact energy of 150 ft-tons. WSDOT has considerable experience with these fences from several manufacturers.

In Design Sector XIV the fence will traverse a talus slope at elevation 2630 feet that will present challenges for the installation of posts. Much of the fence alignment in Design Sector XV will be along the old road that remains after the slope below is cleared of loose rock. In this case the road should be rehabilitated and the fence situated on the outboard side (Figure 54) or alternatively into bedrock. Although less critical, it is recommended the fence be extended through Design Sector XVI above the crest of the proposed cut. This will serve the dual purpose of intercepting rockfall from above as well as precluding public access to the top of the cut slope.

9.0 DESIGN RECOMMENDATIONS

9.1 Introduction

The general approach to slope stabilization was to minimize the types of materials. For example for the reinforcement, two bar sizes are recommended; a high capacity 491 kip ultimate bar and a low capacity 100 kip ultimate bar. In the latter case both tensioned rock bolts and untensioned dowels will use the same bar stock. 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 at Slide Curve 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

#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

Minimum ultimate tensile strength:	100 kips (Typically a #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

9.2.3 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 Slide Curve project sites, the following recommendations are made with respect to rock bolts:

Low Capacity Rock Bolts

Minimum ultimate tensile strength:	100 kips (Typically a #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:	60 kips
Testing:	Per WSDOT Special Provisions

9.2.4 Drain Holes

Drain holes will be drilled in the lower slope to prevent buildup of groundwater pressure. Lengths of individual drains will vary, but will be less than 75 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. Conversely, drain holes shown on the slope face have a specific objective relative to geologic features and should be located as close as possible to the locations shown.

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

9.2.5 Rockfall Control Fence

Based on its considerable experience with rockfall control barriers (fences) WSDOT has developed special provisions that are appropriate for the Slide Curve project. The fence should have a minimum height of 10 feet and have a minimum design rating for impact energy of 150 ft-tons. Highly variable ground conditions should be anticipated for the posts. These could range from strong intact bedrock to talus boulders. It is recommended that the fence alignment be field staked prior to bidding so that the contractors can assess the probable ground conditions. It is not necessary that the fence be continuous. If it is necessary to avoid difficult terrain, the fence segments can be offset with an appropriate overlap depending on the magnitude of the offset.

9.2.6 Slope Drape

Complementing the ditch catchment, slope drape will be a possible secondary rockfall control method for the upper portions of the new cut slopes in Design Sector XVI. The target minimum lower limit for the slope drape is 50 feet above the ditch line.

Due to the strong blocky character of the Domain 1 rock in this Design Sector, 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.

WSDOT has developed Special Provisions that are appropriate for this item. The quantity estimate shown on Figure 52 is an 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.

If slope drape is selected after the cut is brought down to grade, 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 encountered along the slope crest (strong, fresh bedrock).

9.2.7 Shotcrete

The purpose of the shotcrete is to isolate degradable rock zones from ongoing weathering. Shotcrete has not been specified in the slope designs for stabilization. Its inclusion as a bid item is on a contingency basis for the rock cut in Design Sector XV near Station 1408+00 WB where cuts in Domain 3 rock may be necessary. Actual rock conditions will dictate whether shotcrete is required.

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 should be added 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 in Design Sectors XV and XVI.

Scaling of the bedrock outcrops above the alignment in Design Sectors XIV and XV 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 designs for Design Sector XIV and XV include 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 11 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. Noteworthy is the fact that the total estimated stabilization scope is less than the comparable estimate provided in the Slide Curve feasibility report (Wyllie & Norrish, 2007a). This reduction is primarily due to the design change that elevated the grade of the westbound lanes. This eliminated hundreds of feet of cut slope while reducing the height, and hence stabilization intensity, of the remaining cut slopes.

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 item for excavation is an allowance to rehabilitate the access road above the alignment in design Sectors XIV through XVI.

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 11. Preliminary Schedule of Quantities

	Primary Stabilization Measures					Secondary Stabilization Measures			
	Dowel H Quantity (ft)	Dowel L Quantity (ft)	Bolt Quantity (ft)	Scaling Quantity (hr)	Drains Quantity (ft)	Drape Quantity (sf)	Fence Quantity (ft)	Shotcrete Quantity (cy)	Excavation Quantity (cy)
Design Sector XIII **1396+50 to 1400+50**	2625		0	0	1,200			40	200
Design Sector XIV **1400+50 to 1406+00**		240	1,200	400	160		350	10	800
Design Sector XV **1406+00 to 1408+50**		360	720	240	120		250	30	1400
Design Sector XVI **1408+50 to 1411+50**	5000	500	500	40	1,000	7,000	200	10	200
Totals by Item	7,625	1,100	2,420	680	2,480	7,000	800	90	2,600

Incidental Items:

Design Sector		Quantity	Unit
XIII	Grout loss allowance	25	cy
	Scarp void backfill	200	cy
XIV			
XV			
XVI	Grout loss allowance	10	cy

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

The following statement was excerpted from Wyllie & Norrish (2007a):

> *"Notwithstanding the above assertions, the contemplation of rock cuts greater than 100 feet in height in a rock mass with frequent potential planes of movement dipping at more than 40° at a site that has experienced one or more major rock slides is a considerable undertaking. The Slide Curve MP 59 site contains all these elements and therefore the proposed project demands utmost respect. This respect must manifest itself as uncompromised care and attention to detail in the design, construction and performance monitoring of the proposed rock cuts."*

Since that statement was written the ongoing geotechnical investigations at Slide Curve have resulted in a major design change that has elevated the westbound grade by approximately 50 feet thereby eliminating cut slopes in Design Sectors XIII and XIV and reducing maximum cut slope heights in Design Sectors XV and XVI to 80 feet. However, the theme expressed above is still valid and the rock excavation should not be considered routine. 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 Access

A remnant road that was used to access the top-of-slope for the original construction was field located in Design Sectors XV and XVI; and is shown on Figure 42. This road is partially overgrown with immature trees and at several locations has been buried by rockfall debris to a depth of several feet. It is probable that the road can be rehabilitated and possibly extended to the west into Design Sector XIV. This will facilitate movement of materials and equipment to an advantageous position on the upper part of the slope for the initiation of cut slopes and for the execution of stabilization activities. The road may also be usable to move excavated rock to the abandoned quarry site at the eastbound limit of Design Sector XVI for temporary storage and processing. The access road rehabilitation will include some tree removal and side casting and/or loading and hauling of rock debris. It is strongly recommended that WSDOT gain the necessary clearances in advance to enable the contractor to use this road and quarry site for access and egress. Assuming that the road and quarry are made available, the contract documents should stipulate site restoration work for the sites upon completion of the project.

10.3 Blasting

Blasting will be required for the cut slopes of Design Sectors XV and 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.

In addition, the recommendations of Mr. Gerald B. Dilley of Superior Blasting should be considered by the blasting consultant in the preparation and evaluation of blasting plans (see Wyllie & Norrish, 2007a):

1. *"The blast holes should be no larger than 3.5 inches. The 3.5 inch production holes should be fired at only one hole per 17ms delay."*
2. *"The blasting contractor should have experience in extremely close-in blasting, such as shooting very near operating hydroelectric sites, industrial sites with green concrete or any other type of blasting that requires a skill above and beyond those used in general blasting operations."*
3. *"The drillers should have experience on projects that require the accurate logging of anomalies found in the rock while drilling a blast hole."*
4. *"The blaster in charge and the loading crew should have extensive experience loading explosives into blast holes that require the decking of charges through logged anomalies in a blast hole."*
5. *"Electronic detonators should be considered for the initiation system."*
6. *"Instrumentation should be used to monitor vibration and rock movement."*

10.4 Grout Loss

Due to the potential for significant grout loss in the high capacity dowels, it is recommended that the bid item for these dowels 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 high capacity dowel hole above this calculated volume, the Contractor should be reimbursed at cost 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 XIII and a much lower potential in Design Sector XVI.

10.5 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 96 hours of cure time for the grout has been achieved.

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.

10.6 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 XIII:

1. Install in-place inclinometers in one borehole located at Station 1398+50 WB, 80feet left, to a depth of 60 feet. Integrate with WSDOT’s automatic data acquisition system (ADAS).
2. Instrument three dowels of the upper row with vibrating wire or weldable resistance gages. The dowels should correspond to the quarter points and mid-point of the dowel interval (1397+75WB, 1398+50WB and 1399+25WB). Integrate with WSDOT’s automatic data acquisition system (ADAS).

Design Sector XIV:

3. Install a minimum of 4 equally spaced prisms, suitably protected from fly-rock, on the existing outcrop cut slope face above elevation 2650 feet. Monitor using conventional surveying on a weekly basis while active project work is underway.

Design Sector XV:

4. Install a minimum of 4 equally spaced prisms, suitably protected from fly-rock, on the existing outcrop cut slope face above elevation 2650 feet. Monitor using conventional surveying on a weekly basis while active project work is underway.

Design Sector XVI:

5. Instrument three dowels of the upper row with vibrating wire or weldable resistance gages at locations shown on Figure 50. Integrate with WSDOT's automatic data acquisition system (ADAS).
6. Install a minimum of 6 equally spaced prisms, suitably protected from fly-rock, on the cut slope face above elevation 2600 feet. Monitor using conventional surveying on a daily basis during rock excavation and weekly thereafter until project completion.

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